Appendix A
Engineering

FY: 2024
Project Title: Beattyville, KY FRM Project
Project No.: 498982

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1 INTRODUCTION

1.1 BACKGROUND

In late February and early March of 2021, a series of heavy rainstorms collectively caused major flooding in the Kentucky River basin as well as other parts of central and eastern Kentucky (NWS). The City of Beattyville (Beattyville), Kentucky, experienced one of the worst and most damaging floods in its history. The flood crested at approximately 666.5 Ft. NAVD88. Though there were no fatalities, most if not all businesses in downtown Beattyville had five feet of water and were closed for some time after the event (WKYT).

1.2 SCOPE OF STUDY

The Beattyville General Investigation was launched around January 2023. The objective is to analyze all available flood risk reduction measures and provide a recommendation for the optimal measure for Beattyville.

2 GENERAL

2.1 REVIEWER ORIENTATION AND CLARIFICATION

Agency Technical Review (ATR) comments received in September 2024 requested clarifying verbiage for the reader and future reviewers. The Hydrology and Hydraulics (H&H) engineering appendix in the Design Documentation Report (DDR) is for the most part a carryover from the feasibility report because the Recommended Plan does not include structural measures as of September 2024. The floodwall design was hydraulically modeled, and economic analysis was applied. Information regarding the omitted floodwall structural measure has been retained in the report as the structural measure may be revisited per decisionmaker mandate prior to project closeout (see Section 4.9).

The Recommended Plan involves nonstructural measures (See Section 9.1 for nonstructural measures) following guidance for applying floodproofing, property buyouts, and other similar strategies. The H&H modeling for future conditions is identical to the existing conditions for the Recommended Plan. The Planning team has pursued an economic analysis of nonstructural measures based on targeted water surface elevations. The nonstructural alternative was not specifically modeled hydraulically but relies on the existing conditions hydraulic modeling results for further economic computations.

The report has been appended to clarify other areas. The exposure of nonstructural measures to overbank stream velocities is addressed in Section 4.9.1. Information pertaining to Regulatory requirements is in Section 4.10.7.

2.2 RECOMMENDED PLAN

The Recommended Plan least cost alternative proved to be a nonstructural plan consisting of acquisition (12 structures), floodproofing (10 structures dry floodproofed and 30 wet floodproofed) and raising in place (1 structure) paired with providing the City of Beattyville with a Flood Warning

Emergency Evacuation Plan (FWEEP). The FWEEP will be further developed and provided in detail at the ADM milestone. See the first volume of this report for further breakdown of how this plan was selected.

2.2.1 Nonstructural Plan

Objectives prioritized for the Nonstructural plan included life safety, risk of structures and community cohesion. See Section 9.1 Nonstructural of this report for more information.

2.2.2 Flood Warning Emergency Evacuation Plan (FWEEP)

Flood warning and preparedness planning (referred to as Increment 1 in the Feasibility Report) will be further investigated in this study. See Section 9.2 FWEEP for further information to be included and explored.

2.3 Terminology

2.3.1 Base Flood Elevation (BFE)

FEMA established the BFE for most of Beattyville as the elevation 669.1 ft NAVD88. However, as the model moves upstream, this elevation increases. For the purposes of this study, a BFE of 669.2 ft NAVD88 was selected for all of Beattyville for consistency. Based on hydrological analysis, the USACE H&H modeling determined the 1% annual flood exceedance for Beattyville as 672.08 ft NAVD88, which was rounded for simplicity and the purposes of this report to match the BFE plus 3 ft elevation of 672.2 ft NAVD88. In summary, for the purposes of this study, FEMA's BFE for the study area was assumed to be 669.2 ft NAVD88, and the BFE plus 3 feet was assumed to be 672.2 ft NAVD88. See Section 3.6.5, 4.1.4, and 4.9.3 for more information on the various elevations used for this study.

3 Hydrologic Hazards

3.1 Introduction

This chapter requires development of several data sets to assist in best estimates of hydrologic loading to be used in the evaluation of potential Flood Risk Management (FRM) Projects. FRM's include potential new levee systems which require hydrologic loading to help establish initial levee elevation estimates. Hydrologic loading was reviewed for the Beattyville, Kentucky as well as the three contributing forks, the North Fork Kentucky River (North Fork), Middle Fork Kentucky River (Middle Fork), and South Fork Kentucky River (South Fork). Flow-frequency and stage-frequency curves were developed for the Kentucky River at Beattyville and for each of the three Kentucky River forks. Hydrologic loadings were reviewed in the North, Middle, and South Fork because additional reservoirs were initially investigated in the upper forks. The hydrologic loadings are documented for the upper forks, but additional reservoirs are not anticipated at the time of report completion.

Elevation data provided in this report on stream conditions and structural aspects are in North American Vertical Datum of 1988 (NAVD88) unless otherwise stated. The conversion from NVGD29 to NAVD88 used was -0.54 ft for the vicinity of the project area based on https://www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html.

3.2 BACKGROUND DATA

Beattyville is the county seat of Lee County and lies at the confluence of the North Fork and South Fork. The North Fork and South Fork are both tributaries of the Kentucky River and the Middle Fork is a tributary of the North Fork.

Hydrologic loading data used in this evaluation was primarily taken from the United States Geological Survey (USGS) gages summarized in

Table 1. USGS gages at Lock 6 and Lock 8 are shown in

Table 1 as they are referenced in this chapter but no systematic data from either gage was used in the development of the final flow-frequency and stage-frequency curves.

Table 1: USGS Reference Gages

Gage No.	Gage ID	Location	Gage Elevation (ft)	Drainage Area (sq. mi.)
03280000	North Fork Kentucky River	Jackson, KY	697.70	1,101
03281000	Middle Fork Kentucky River	Tallega, KY	641.55	537
03281500	South Fork Kentucky River	Booneville, KY	641.91	722
03282000	Kentucky River Lock 14	Heidelberg, KY	625.39	2,657
03284000	Kentucky River Lock 10	Winchester, KY	556.10	3,955
03284500	Kentucky River Lock 8	Camp Nelson, KY	503.25	4,414
03287000	Kentucky River Lock 6	Salvisa, KY	487.02	5,102
03287500	Kentucky River Lock 4	Frankfort, KY	461.58	5,411

Data from Kentucky River Lock (Lock) 14 USGS gage 03282000 at Heidelberg, KY (Heidelberg) was the primary gage used for the development of flow- and stage-frequency curves due to the proximity of the gage to Beattyville, which is approximately 3.5 miles west of Beattyville. Data from Lock 10 USGS gage 03284000 at Winchester, KY (Winchester) and Lock 4 USGS gage 03287500 at Frankfort, KY (Frankfort) was used in various capacities as transposed and perception threshold (PT) data depending on correlation between USGS sites. Data transposition and the PT are discussed in further detail later in the report. The North Fork at Jackson, KY (Jackson), Middle Fork at Tallega, KY (Tallega), and South Fork at Booneville, KY (Booneville) each have a USGS gage and were also individually reviewed in coordination with Lock 14 as part of this study. See Section 4.4 for additional details on the data used.

Flows in the headwater area of the Kentucky River Basin are controlled by two primary flow regulation structures: (1) Carr Creek Dam located in the North Fork Basin and (2) Buckhorn Dam located in the Middle Fork Basin. The South Fork currently has no flow regulation. It should be noted that Carr Creek Dam is on a tributary of North Fork and regulates a drainage area of 58 square miles. The North Fork USGS gage 03280000 at Jackson has a contributing drainage area of 1,101 square miles. Therefore, Carr Creek Dam has a minor impact on inflow at the North Fork USGS gage. Conversely, Buckhorn Dam regulates a drainage area of approximately 408 square miles, which compared to the Middle Fork USGS gage 03281000 at Tallega drainage area of 537 square miles has a much larger impact on inflows at the Middle Fork USGS gage.

Points of regulation downstream of Lock 14 include Lock 10 at Winchester, Lock 8 at Camp Nelson, Lock 6 at Salvisa, and Lock 4 at Frankfort. The Kentucky River ends at its confluence with the Ohio River at Carrollton, KY. The Dix Dam (also noted as Herrington Lake), placed into operation in 1926, is located on the Dix River approximately 2.2 miles upstream of its confluence with the Kentucky River and has a drainage area of 439 square miles. The Dix River confluence with the Kentucky River is located upstream of the Lock 6 USGS gage 03287000 at Salvisa and downstream of the Lock 8 USGS gage 03284500 at Camp Nelson.

Figure 3-1 shows the location of Beattyville, USGS gages, and other pertinent project features referenced in this chapter.

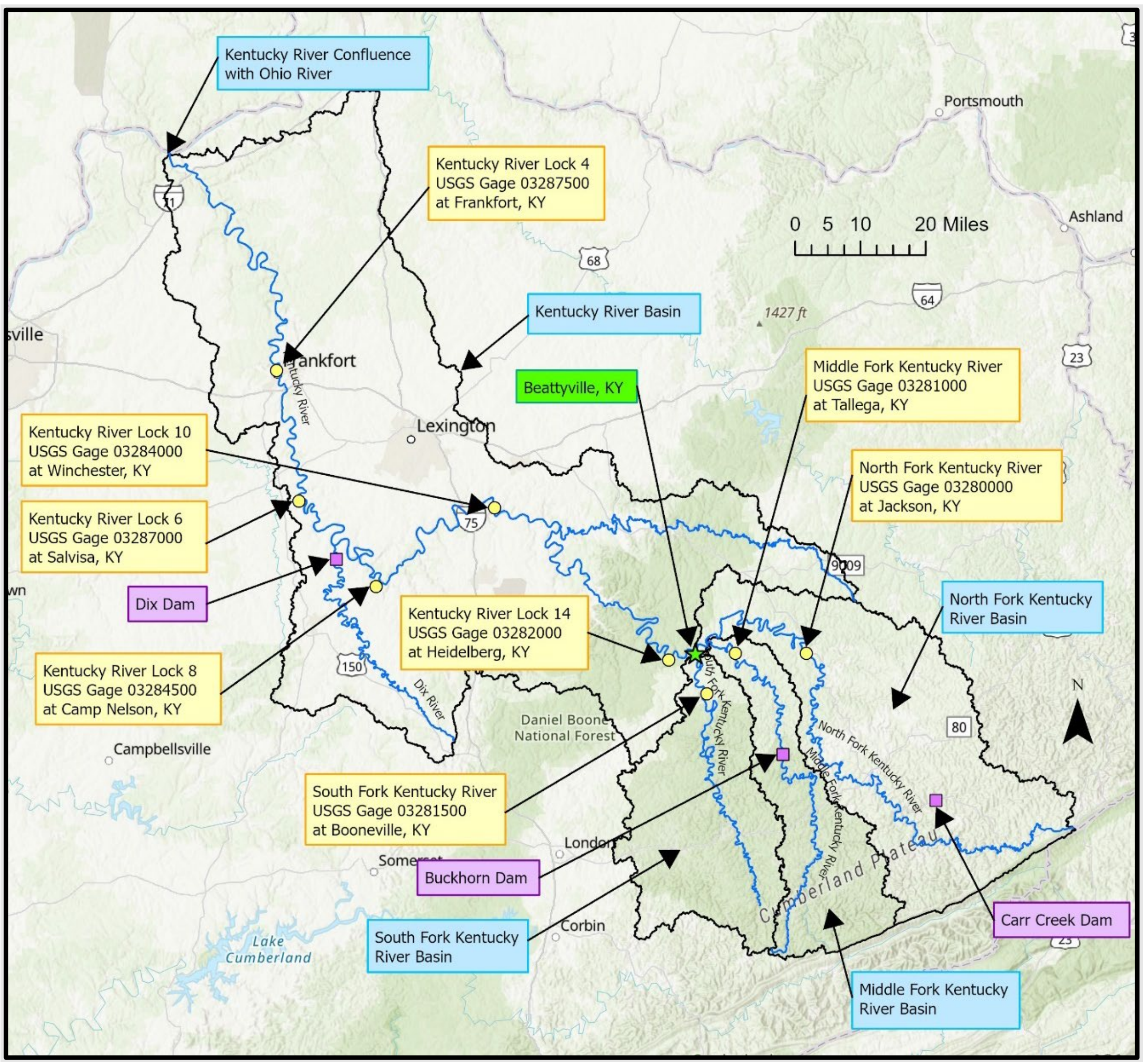

Figure 3.1: Project Vicinity Map

Prior to the establishment of the above mentioned USGS gages, there was generally no reliable data for floods occurring at Lock 14 or in the North, Middle, or South Fork. Consistent USGS gage data collection began in 1921 for Lock 14 and in 1939 for North, Middle, and South Fork.

3.3 HYDROLOGY

3.3.1 Kentucky River Basin Hydrology

Information regarding the climate and hydrology of the North, Middle, and South Fork basins was primarily taken from the Buckhorn and Carr Creek WCMs. The North, Middle, and South Fork basins are designated as humid subtropical climates according to the Koppen Climate Classification, which is characterized by hot, humid summers and cool to mild winters. The area is subject to frequency temperature changes, high humidity, and periods of intense precipitation. The basins lie in the Eastern Kentucky Coal Field physiographic region, which is dominated by forested hills and highly dissected by V-shaped valleys. The Kentucky River basin extends northwest through the Bluegrass physiographic region until its confluence with the Ohio River.

Storms of 2.0 to 3.0 inches per 24-hour period are not uncommon and total annual rainfall generally varies between approximately 44 to 49 inches. Records indicate precipitation is fairly well distributed through the year with approximately 56 percent of total annual precipitation occurring between March and August with July being the month of great precipitation contribution. Minimum months of rainfall varied but usually occur in October.

Storms that produce the most intense rainfall are typically formed in the southwestern United States or in the Gulf of Mexico and move northeasterly toward the north Atlantic coast. Cyclonic storms are the most frequent cause of excess runoff in these basins. Storms of this type generally occur mid-winter to early spring when conditions are conducive to high runoff, and many have produced severe flooding in the North, South, and Middle Fork basins. Convective storms which produce rainfall of high intensity generally occur during the summer months and seldom cause important flooding since they are usually have smaller area coverage and transpiration losses and infiltration rates are high. Topography of the basins are such that orographic rainfall does not occur.

3.3.2 Major Floods

Flooding in the basins has been experienced at some point in every month of the year. Summer and fall season floods generally have less coverage than those occurring during the winter and spring. The principal cause of floods in the basins is excessive rainfall. Snow melt can aggravate flood conditions but is not a direct cause of flooding. The headwater areas of the Kentucky River often have characteristic steep, rugged terrain. This terrain along with the periods of intense precipitation make for conditions of extremely rapid runoff and high susceptibility to flash flooding. Table 2 summarizes the three largest flood events recorded by USGS gages referenced.

Table 2: Major Flood Events

Date	*Peak Inflow (cubic feet per second [cfs])*	*Elevation (ft)*	*Gage Height (ft)*

USGS Gage 03280000 – North Fork			
July 29, 2022*	54,400	739.70	42.00
May 8, 1984*	53,500	739.67	41.97
January 30, 1957	53,500	738.11	40.41
USGS Gage 03281000 – Middle Fork			
January 30, 1957	52,700	684.88	43.33
February 1939	37,300	682.05	40.50
February 2, 1951	35,300	681.62	40.07
USGS Gage 03281500 – South Fork			
January 30, 1957	66,100	685.31	43.40
February 28, 1962	54,700	682.65	40.74
May 8, 1984	51,600	683.03	41.12
USGS Gage 03282000 – Kentucky River Lock 14			
February 04, 1939	120,000	660.99	35.60
January 30, 1957	116,000	660.39	35.00
March 24, 1929	113,000	659.79	34.40
USGS Gage 03284000 – Kentucky River Lock 10			
December 10, 1978*	101,000	596.25	40.15
February 05, 1939	92,400	590.90	34.80
March 01, 1962*	91,500	592.17	36.07
USGS Gage 03287500 – Kentucky River Lock 4			
December 09, 1978*	118,000	510.05	48.47
January 25, 1937	115,000	509.04	47.46
February 16, 1989*	105,000	505.75	44.17

**Events occurred after regulation of the respective USGS gage*

The February 1939, January 1957, and March 2021 flood events are discussed further below. The 1939 and 1957 events are discussed in further detail because they are the two floods of record at Lock 14 and were large events that also impacted the three upper forks. The March 2021 event is discussed in further detail because it was the most significant flooding the Beattyville area had seen in 50 years and was the event that prompted this study.

3.3.3 Storm and Flood of February 1939

The storm and flood of February 1939 is the flood of record at Lock 14. There is limited data available regarding the February 1939 flood. Precipitation data could not be found in eastern Kentucky dating back to the event and the flood appears to be overshadowed by the July 4-5 floods of 1939, which caused catastrophic flash flooding across much of eastern Kentucky. It is possible the February 1939 floods were more localized to the Kentucky River basin and did not cause the as much overall damage or flooding as the July 1939 flood event. The storm and flood of February 1939 resulted in a flow of 120,000 cubic feet per second (cfs) and stage elevation of 660.99 feet according to the Lock 14 USGS gage 03282000 at Heidelberg. Both floods are shown below in Figure 3-2.

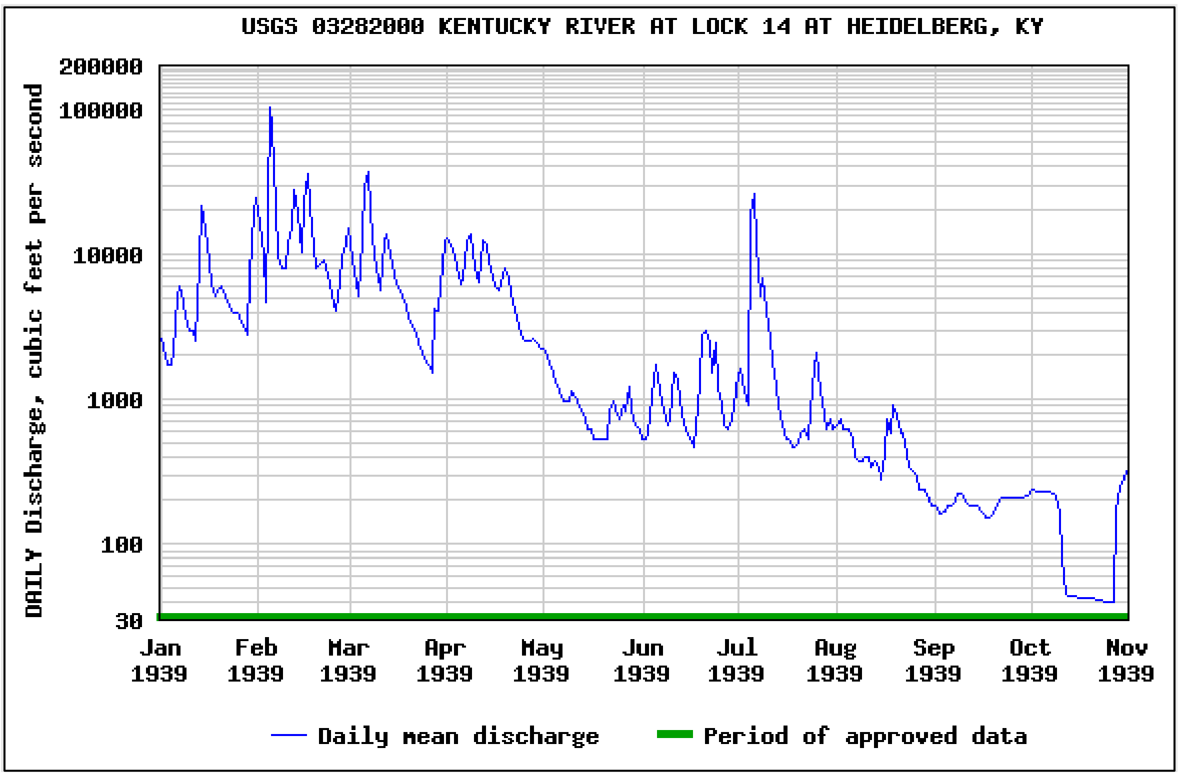

Figure 3.2: Daily Discharge for the Kentucky River at Lock 14 for 1939

3.3.4 Storm and Flood of January 1957

Heavy rains over an extensive area on January 27 to February 2, 1957, caused extreme flooding in southeastern Kentucky and adjacent areas in West Virginia, Virginia, and Tennessee. Total rainfall for the storm period ranged from 6 to 9 inches over most of the reported area and was estimated to be around 12 inches at the eastern end of the Virginia-Kentucky State line. The principal basins affected by the storm were those of the Big Sandy, Kentucky, Cumberland, and Tennessee Rivers. Maximum discharge of record occurred in many streams. The 1957 flood was the second largest flood of record at Lock 14, the flood of record for the Middle Fork and South Fork, and the third largest flood of record for the North Fork according to the respective USGS gages. Refer to Table 2 above for the recorded flow values and stages at each of the gage locations.

3.3.5 Storm and Flood of March 2021

Several rounds of heavy rain moved across eastern Kentucky from late Friday, February 26, 2021, through early Monday, March 1, 2021. The first round of heavy rain came through late Friday, February 26 and in the morning of Saturday, February 27. The second round of heavy rain occurred late Saturday night as a warm front lifted northward across eastern Kentucky. The heavy rain stalled over portions of eastern Kentucky, leading to flash flooding in the early morning hours. The third and final round of heavy rain came late Sunday afternoon through Sunday night. Due to the antecedent moisture conditions from the first two rounds of rainfall, the third round caused widespread flash flooding across much of eastern Kentucky and ultimately led to larger river and creek flooding.

The combination of all the heavy rainfall led to significant flooding across a good portion of central and east Kentucky, including the project area. For some areas, this was the most significant flooding in the last 50 years or more. 72-hour rainfall totals at Heidelberg were approximately 6.5 inches according to the National Weather Service.

3.4 Overall Systematic, Regional, and Historical Data

3.4.1 Source Data Overview

As previously noted, the USGS gages in
Table 1 were the primary data sources used in this analysis. Annual peak inflow data from each of the gages was obtained and used to develop the hydrologic loading curves. Table 3 summarizes the various annual peak inflow data available at each of the USGS gages in
Table 1.

Figure 3-3 show a visual representation of the available annual peak inflow data at Lock 4, Lock 10, Lock 14, North Fork, Middle Fork, and South Fork USGS gages. Table 4 summarizes the drainage area ratios between the USGS gages in

Table 1.

Table 3: USGS Annual Peak Inflow Source Data Summary

Gage No.	***Gage ID***	***Historic Data***	***Unregulated Data***	***Regulated Data***
03281500	South Fork	N/A	1926 – 1931, 1937, 1939 – Present	N/A
03281000	Middle Fork	N/A	1929, 1931 – 1932, 1935, 1937, 1939 - 1960	1961 – Present
03280000	North Fork	N/A	1917 – 1921, 1927 – 1931, 1935 – 1975	1976 – Present
03282000	Lock 14	N/A	1921 - 1960	1961 – Present

03284000	Lock 10	N/A	1908 – 1960	1961– 2010, 2012 – Present
03287500	Lock 4	1817, 1847, 1854, 1880, 1883	1895 – 1926	1927 – Present

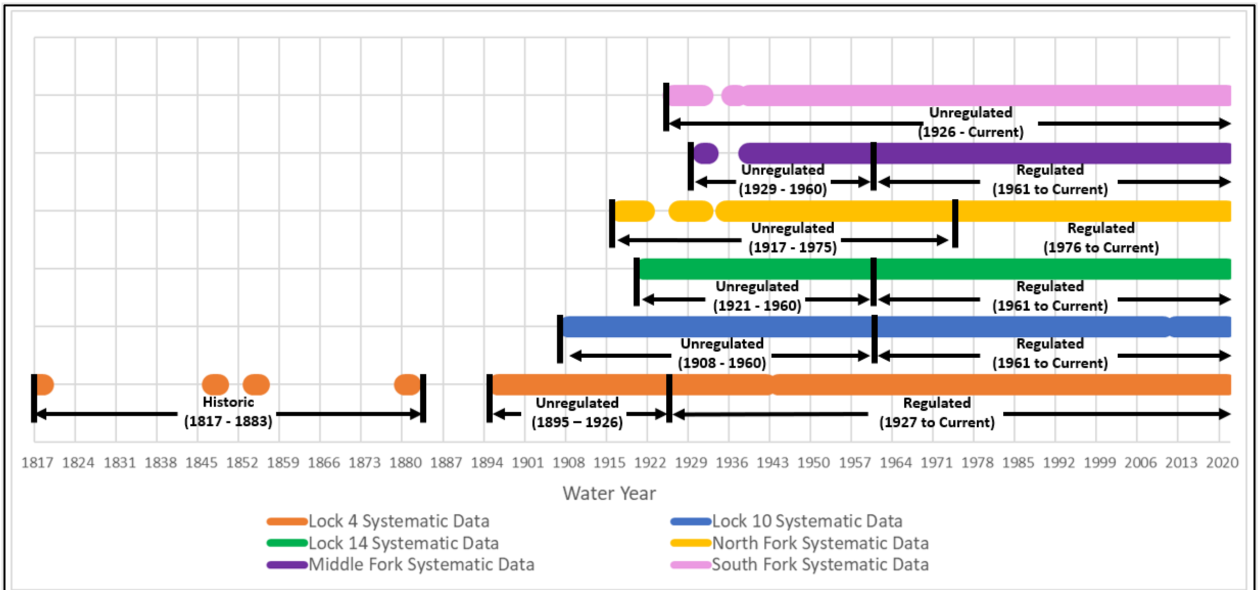

Figure 3.3: USGS Peak Inflow Source Data Summary Visualization

Table 4: USGS Source Data Drainage Area Ratio Summary

To / From	North Fork (1,101 sq mi)	Middle Fork (537 sq mi)	South Fork (722 sq mi)	Lock 14 (2,657 sq mi)	Lock 10 (3,955 sq mi)	Lock 4 (5,411 sq mi)
North Fork (1,101 sq mi)	1.00	0.49	0.66	2.41	3.59	4.91
Middle Fork (537 sq mi)	2.05	1.00	1.34	4.95	7.36	10.08
South Fork (722 sq mi)	1.52	0.74	1.00	3.68	5.48	7.49
Lock 14 (2,657 sq mi)	0.41	0.20	0.27	1.00	1.49	2.04
Lock 10 (3,955 sq mi)	0.28	0.14	0.18	0.67	1.00	1.37

Lock 4 (5,411 sq mi)	0.20	0.10	0.13	0.49	0.73	1.00

3.4.2 Systematic Data Record Extensions

Several different methods of systematic data record extension were reviewed and used as part of this study for Beattyville and the associated headwater drainage areas. These methods are summarized below and the priority that was placed on their use.

1. The Maintenance of Variance Extension (MOVE.3) function as described in *Guidelines for Determining Flood Flow Frequency, Bulletin 17C*, 2019 (B17C). This is the recommended method to extend records based on B17C and was therefore the highest priority for record extension if applicable between two gages.
2. Concurrent annual peak flows between two gages were compared and best fit trendlines created. R squared values associated with the trendline were reviewed to determine if there was sufficient correlation for record extension. Generally, if the trendline yielded an r squared value of 0.8 or more, correlation was considered sufficient. This method was reviewed if drainage areas between the two gages were similar, and the MOVE.3 function was not applicable.
3. Concurrent annual peak flow between two gages were compared on a log-log basis and best fit linear trendline created. R squared values associated with the trendline were reviewed to determine if there was sufficient correlation for record extension. Generally, if the trendline yielded an r squared value of 0.8 or more, correlation was considered sufficient. This method was reviewed if drainage areas between the two gages were similar, and the MOVE.3 function was not applicable. The use of direct flow comparisons or log-log comparisons was determined based on whichever method yielded a higher r squared value of the trendline created.
4. If the above three options did not yield sufficient correlation, scaling factors were reviewed for flows from concurrent peak annual events between locations. This type of analysis was used to review perception thresholds since the r squared value and drainage area ratios between compared locations did not yield sufficient correlation to use as direct systematic data extension.

3.4.3 Lock 14 Rating Curve

Figure 3-4 shows the Lock 14 USGS gage 03282000 at Heidelberg developed rating curve using recorded systematic flow and stage data from the gage over its period of record with a linear trendline, overlay of the rating curve from the Buckhorn Lake WCM, and overlay from the current published USGS rating curve. The developed rating curve was used based on the high r squared value of 0.9903 when correlating the flow and stage data and a visual review of the data in Figure 3-4. The current USGS published rating curve typically yields lower flows at the same stage value compared to the developed curve and the previously established rating curve in the Buckhorn WCM. It is unknown when or why the current USGS published rating curve was updated to its current state. The developed rating curve based on the linear trendline was used as discussed in Section 4.5.4 as part of the spreadsheet tool provided by the LRL Water Management Section. Observed and modeled flood stages were converted to flow values utilizing the developed rating curve linear trend line shown in this figure.

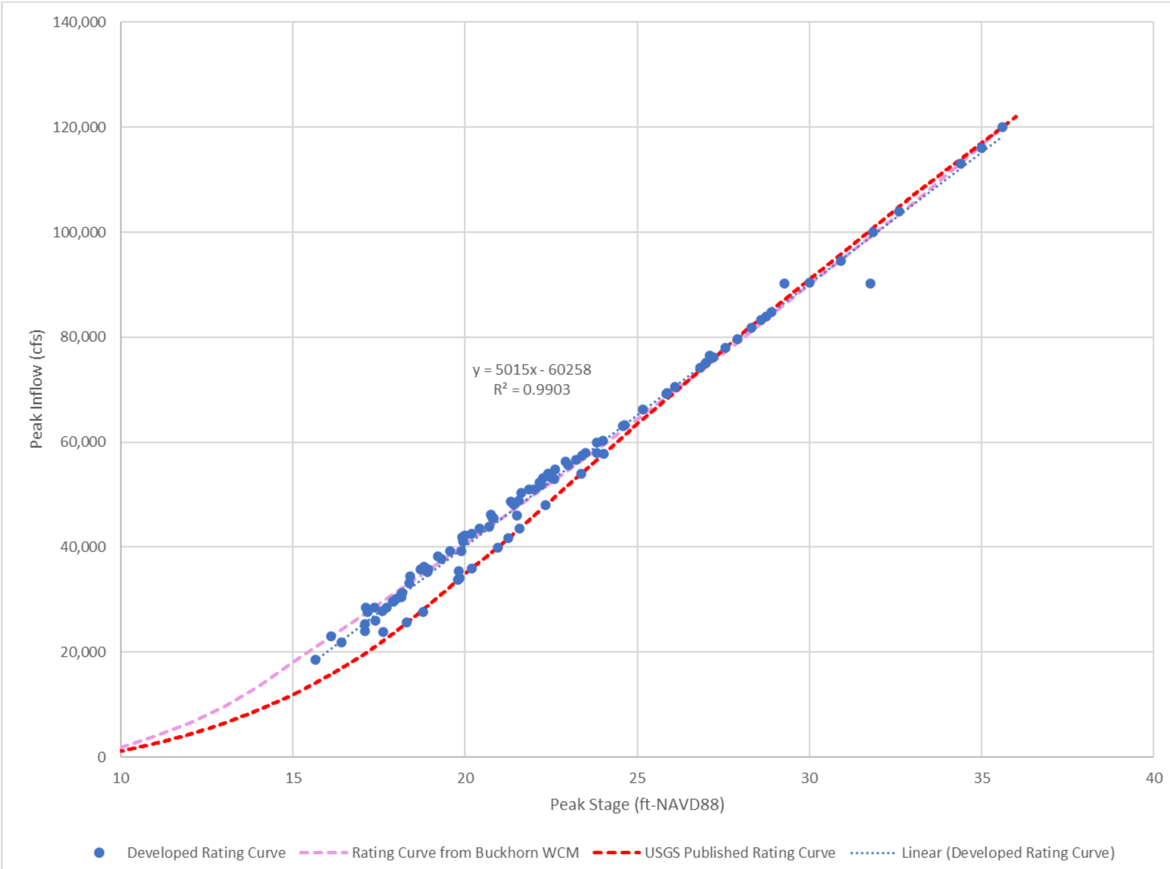

Figure 3.4: Lock 14 Rating Curves – Buckhorn WCM and USGS Published Rating Curves

As part of the hydraulic modeling effort for Beattyville, the Project Delivery Team (PDT) coordinated with the USGS on the potential of a sloped rating curve for the Kentucky River upstream of Lock 11. Backwater effects along the Kentucky River have influenced stages at Lock 14, and subsequently at Beattyville. Figure 3-5 shows the sloped rating cure that was observed for the 2021 event along with field measurements performed by the USGS. Discussions with USGS staff indicated USGS trusted their peak inflow data and sloped rating curve but did not have the additional data or availability yet to review the data further to determine how and when the sloped rating curve occurs. The Hydrologic Engineering Center's River Analysis System (HEC-RAS) model created for this study used the 2021 event for calibration purposes, which partially reflected the sloped rating curve as the final rating curve created a hysteresis loop. A composite rating curve was created from the hysteresis loop and used for the final stage frequency curve analysis at Beattyville, see Section 4.6.4 for additional details. Since the composite rating curve partially incorporates the sloped rating curve, it is slightly more conservative when considering calculated stage values based on flows.

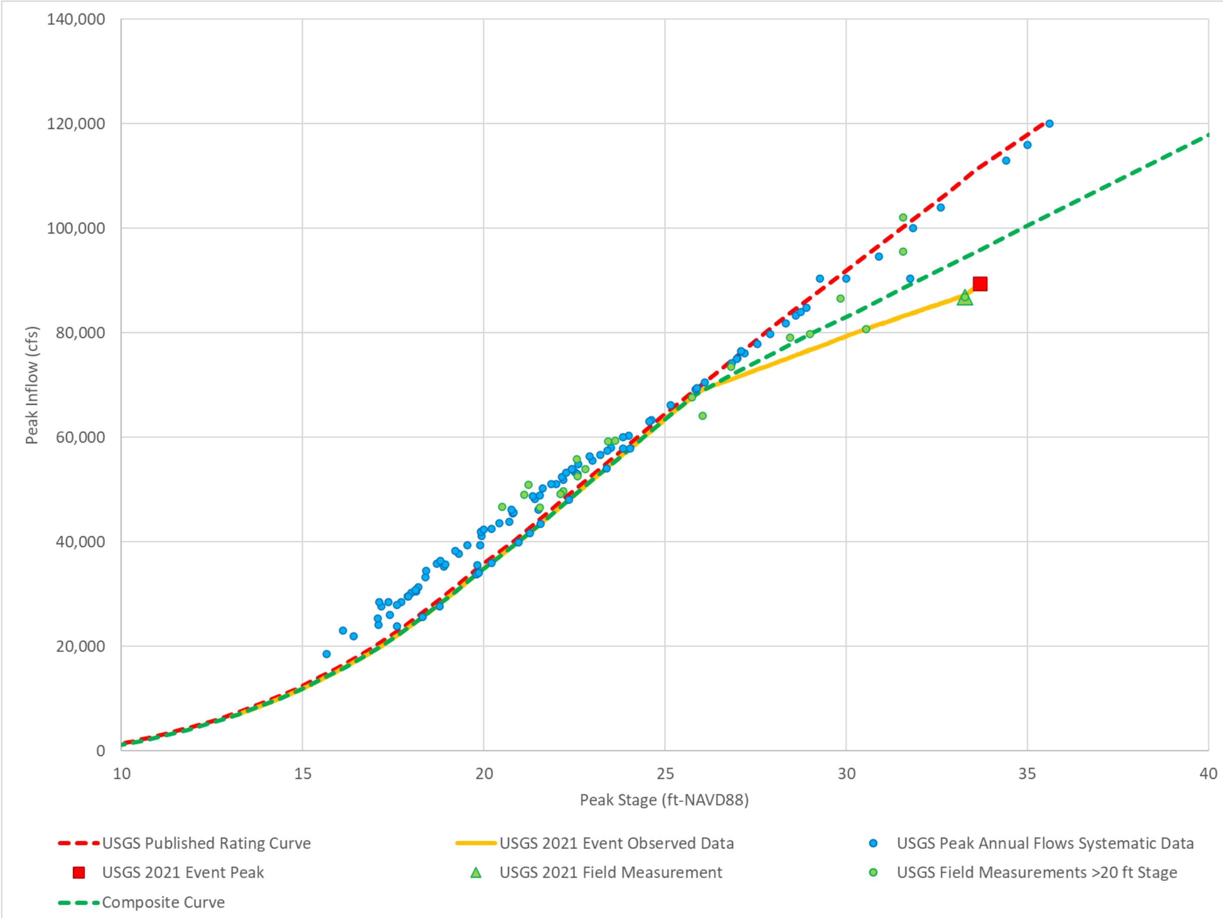

Figure 3.5: Lock 14 Sloped Rating Curve Data

3.5 Lock 14 at Heidelberg

3.5.1 Period of Record Extension Lock 10 to Lock 14

Unregulated annual peak flow data was used to calculate the instantaneous unregulated peak inflow frequency curve. USGS gages at Lock 14 (drainage area of 2,657 square miles) and Lock 10 (drainage area of 3,955 square miles) were compiled to create the composite period of record (POR). Instantaneous peak inflow is available at Lock 14 from 1921 to present and available at Lock 10 from 1908 to present. The Lock 14 and Lock 10 gages are listed as unregulated through 1960 and regulated from 1961 to present after the construction and operation of Buckhorn Lake. The unregulated POR at Lock 14 was extended between 1908 to 1920 by transposing Lock 10 data to Lock 14 data.

To transpose Lock 10 USGS Gage 03284000 at Winchester data to extend the Lock 14 POR, the MOVE.3 function as described in B17C was used. The transposition is based on observed correlation between the base-10 logarithms of the annual maximum series available at two sites. Several primary conditions should be met before use of the MOVE.3 function, which include:

- A minimum of 10 concurrent years of record between the two sites
- A correlation coefficient $\rho > 0.80$

- Slope of the linear regression model $b > 0$

The Lock 10 USGS gage 3284000 at Winchester has unregulated annual peak flow data from 1908 to 1960 and the Lock 14 USGS gage 03282000 at Heidelberg has unregulated annual peak flow data from 1921 to 1960. This yields 40 years of concurrent similar observed data between the two sites and 13 potential years of record extension. Calculation of the correlation coefficient for the two data sets from MOVE.3 yields a value of $\rho = 0.92$ and the slope yielded a value of $b = 1.21$. Therefore, the above conditions were met, and the MOVE.3 function was used for record extension. The MOVE.3 function also generates the estimated extended record length for the most recent non-overlapping years, which in this instance was an extension of 8 years. The drainage area ratio between the Lock 10 USGS gage 03284000 at Winchester and the Lock 14 USGS gage 03282000 at Heidelberg is 1.49, which is a reasonable drainage area ratio for systematic data transposition. Therefore, the Lock 14 USGS gage 03282000 at Heidelberg annual peak flow data was extended 8 years between 1913 to 1920 by transposing Lock 10 USGS gage 3284000 at Winchester annual peak flow data via the MOVE.3 function.

Figure 3-6 shows the results of the MOVE.3 function with 1 to 1 ratio line, concurrent period data, extend period data, and linear best fit line for the concurrent period data on a log-log scale. Table 5 shows the record extension values at the Lock 14 USGS gage 03282000 at Heidelberg via the MOVE.3 function.

It should be noted that the Lock 14 transposed annual peak flows shown in Table 5 can exceed the corresponding annual peak flows at Lock 4 which has a larger drainage area. It is believed that flows attenuate as they move down the Kentucky River where the floodplain is substantially wider at Lock 4. This can result in longer periods of elevated flows at Lock 4 but also mitigates the peak flow. The steep, rugged terrain contributing to Lock 14 results in the opposite conditions where the flashy nature of the floods result in high peak values but shorter durations. These conditions cause higher peak flows at Lock 14 compared to Lock 4 but the duration and total volume of flow at Lock 4 is still considerably higher compared to Lock 14.

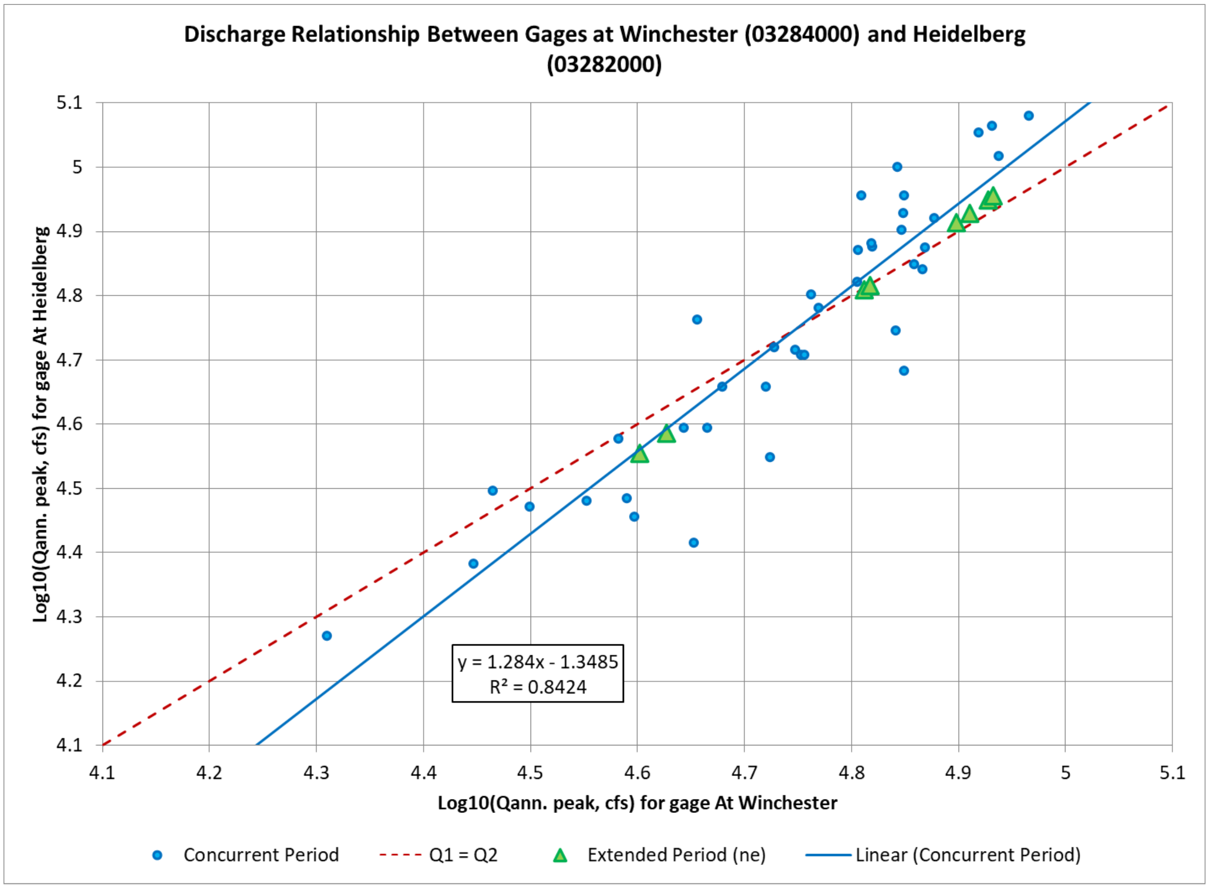

Figure 3.6: Lock 14 and Lock 10 Inflow Correlation with MOVE.3 Extension (Log-Log Scale)

Table 5: Lock 14 MOVE.3 Function Record Extension

Year	***Lock 10 Flow (cfs)***	***Lock 14 Transposed MOVE.3 Flow (cfs)***
1913	84,600	88,900
1914	40,000	35,900
1915	42,400	38,600
1916	85,600	90,100
1917	64,800	64,400
1918	81,300	84,700
1919	65,600	65,300
1920	79,100	81,900

3.5.2 Period of Record Extension Lock 4 to Lock 14

USGS gage data at Lock 4 was also used to supplement the data set with historic flows. Lock 4 USGS gage 03827500 at Frankfort has annual flood and gage data dating back to 1895 and has historic flood gage heights from 1817, 1847, 1854, 1880, and 1883. Therefore, Lock 4 data was reviewed for correlations to see if the POR for Lock 14 could be extended using transposed data.

Lock 4 has a drainage area of 5,292 square miles and Dix Dam has a drainage area of 439 square miles, which is approximately 8% of the drainage area for Lock 4. Due to the small percentage of regulation impact, flows between Lock 4 and Lock 14 were reviewed neglecting the impacts of regulation of the Dix Dam to see if a correlation could be identified. Concurrent annual peak flow data from 1921 through 1960 from Lock 4 USGS gage 03284500 at Frankfort was plotted against Lock 14 USGS gage 03282000 at Heidelberg and various trendlines were fit to the data based on unregulated conditions for Lock 14 to see if a correlation could be made. The same data was also plotted and compared on a log-log scale with a linear trendlines fit to the data. Figure 3-7 and Figure 3-8 show the Lock 4 to Lock 14 direct flow and log-log flow comparisons, respectively. Direct flow comparisons show the drainage area ratio, 1 to 1 ratio line, and exponential trendline. Log-log comparisons show a linear trendline.

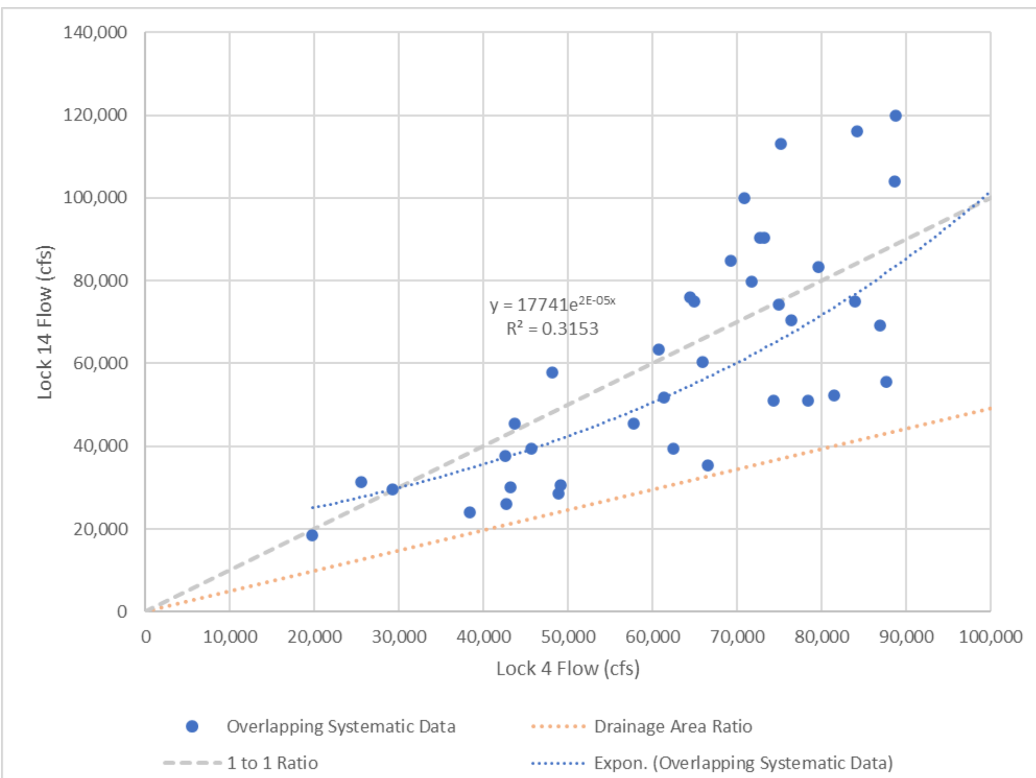

Figure 3.7: Lock 4 and Lock 14 Annual Peak Flow Data Comparison

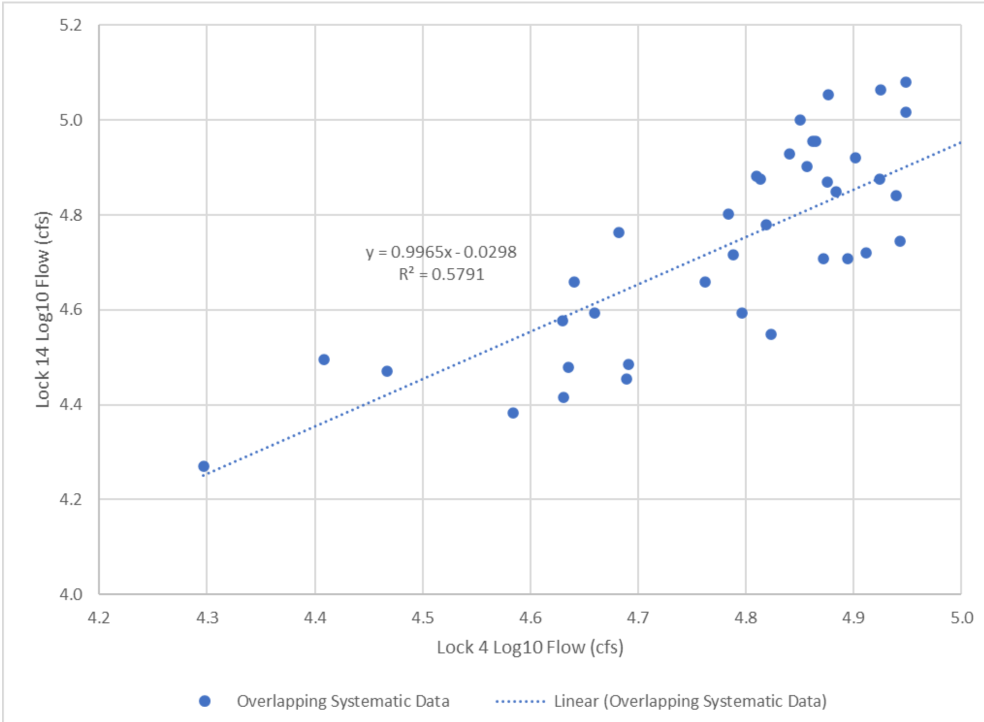

Figure 3.8: Lock 4 and Lock 14 Annual Peak Flow Data Comparison (Log-Log)

All r squared values were below 0.6, therefore correlation was not considered high enough for including this data as systematic data in the composite period of record. Although r squared values resulted in a value where systematic record extension was not considered feasible, events between Lock 4 and Lock 14 for annual peak inflows between 1921 and 1960 were reviewed to identify if peak flow values occurred from the same precipitation event at each location. Table 6 summarizes the events and corresponding annual unregulated peak inflows. Events shaded in green indicate peak flow values that occurred from the same precipitation event.

Table 6: Lock 4and Lock 14 Corresponding Event Comparison

	Lock 14		***Lock 4***	
Water Year	***Date***	***Peak Flow (cfs)***	***Date***	***Peak Flow (cfs)***
1921	4/17/1921	29,600	4/19/1921	29300
1922	2/21/1922	75,100	2/23/1922	65,000
1923	2/4/1923	84,800	2/6/1923	69,300
1924	1/4/1924	55,600	1/4/1924	87700
1925	2/16/1925	51,000	12/9/1924	74400

1926	1/22/1926	51,900	1/24/1926	61400
1927	12/23/1926	90,400	12/26/1926	72,700
1928	6/29/1928	35,300	7/1/1928	66500
1929	3/24/1929	113,000	3/27/1929	75,200
1930	11/18/1929	24,100	2/5/1930	38400
1931	4/23/1931	45,500	4/25/1931	43700
1932	1/31/1932	79,700	2/2/1932	71,800
1933	2/21/1933	45,500	2/23/1933	57800
1934	3/4/1934	76,100	3/6/1934	64,500
1935	3/13/1935	83,300	3/13/1935	79,700
1936	4/7/1936	60,300	4/10/1936	65900
1937	1/26/1937	48,200	1/25/1937	115000
1938	7/15/1938	26,000	7/16/1938	42700
1939	2/4/1939	120,000	2/7/1939	88,800
1940	4/21/1940	39,300	4/23/1940	45700
1941	7/6/1941	31,300	7/7/1941	25600
1942	7/10/1942	30,200	8/24/1942	43200
1943	3/20/1943	66,200	-	-
1944	4/12/1944	52,400	12/31/1943	81500
1945	3/6/1945	51,000	3/7/1945	78400
1946	1/9/1946	90,300	1/12/1946	73,200
1947	6/29/1947	57,900	7/1/1947	48100
1948	2/15/1948	104,000	2/17/1948	88,700
1949	3/19/1949	39,300	2/16/1949	62500
1950	2/2/1950	69,200	2/2/1950	87000
1951	2/2/1951	100,000	2/5/1951	70,900
1952	3/24/1952	70,500	3/26/1952	76,400

1953	5/7/1953	30,500	3/4/1953	49100
1954	1/17/1954	18,600	3/5/1954	19800
1955	3/1/1955	75,000	3/6/1955	84,000
1956	2/19/1956	74,200	2/19/1956	75,000
1957	1/30/1957	116,000	2/3/1957	84,200
1958	5/8/1958	63,300	5/11/1958	60800
1959	1/23/1959	37,700	1/22/1959	42600
1960	3/18/1960	28,500	6/24/1960	48900

As shown in Table 6, the same precipitation event caused the annual peak inflow at Lock 4 and Lock 14 for 28 years out of the total 40 years of overlapping unregulated data. Therefore, annual peak inflows from Lock 14 to Lock 4 are typically caused by the same precipitation event. In years where the annual peak inflows do not align between Locks 4 and Lock 14 (12 of 40 years), it is believed this may be due to storm orientation and placement. For these events, the storms may be centered below Lock 14 within the Kentucky River Basin, resulting in higher peak flows at Lock 4 that do not correlate back to Lock 14.

Numerous overlapping annual peak inflow data points in Table 6 show that Lock 14 flows can exceed the corresponding flows at Lock 4, which has a larger drainage area. As previously noted, this is believed to be a result flow attenuation as flow moves downstream and the flashy nature of the headwater basins. Due to uncertainty in the Lock 4 to Lock 14 transposition, it was determined that the Lock 4 data should not be utilized as systematic or flow interval data as part of the inflow frequency analysis. However, since peak flows at Lock 14 generally coincide with the same precipitation event at Lock 4, it was used to develop perception threshold data as discussed further in Section 4.6.1.

3.5.3 Period of Record Extension Lock 6 and Lock 8 to Lock 14

Both Lock 6 USGS gage 03287000 at Salvisa and Lock 8 USGS gage 03284500 at Camp Nelson were reviewed to potentially extend the POR for Lock 14 USGS gage 03282000 at Heidelberg. Lock 8 USGS gage has annual peak flow data dating back to 1911. A MOVE.3 function review indicated annual peak flow data from Lock 8 could only extend the POR by 4 years, which is less than Lock 10. Therefore, Lock 8 data was not used to further supplement the Lock 14 POR.

A MOVE.3 function review was also performed for Lock 6 correlations were not high enough to warrant record extension. Similar analysis was performed on Lock 6 gage data as described for Lock 4 in Section 4.5.2, but correlations were found to be substantially lower. Therefore, Lock 6 data was not used to further supplement the Lock 14 POR.

3.5.4 Period of Record Extension at Lock 14 From 1960 to the Present

Three data sources were reviewed to establish the regulated-unregulated flow relationship at Lock 14. These sources include:

- Spreadsheet model provided by LRL Water Management
- The Buckhorn Lake WCM
- Modified HEC Hydrologic Modeling System (HMS) analysis of the Kentucky River Basin

The spreadsheet model provided by LRL Water Management includes estimated unregulated stage heights and stage height reductions of numerous regulation structures in Kentucky and Indiana for major storm events from 2015 to 2022, including reductions at Lock 14 based on the regulation of Buckhorn Lake. Stage heights at Lock 14 from the spreadsheet model were converted to estimated flow values using a rating curve developed based on Lock 14 USGS gage 03282000 at Heidelberg, which is included previously in Figure 3-4. As noted in Section 4.4.2, this rating curve does not account for the uncertainty in the sloped rating curve that has been developed based on field measurements performed by the USGS.

Table 7 summarizes the spreadsheet model storm events, estimated unregulated stage height, estimated regulated stage height, and estimated flows based on the Lock 14 rating curve. The unregulated and regulated flow estimates in this analysis are plotted in Figure 3-11 at the end of this section.

Table 7: Spreadsheet Model Lock 14 Impacts from Buckhorn Lake Regulation

Date	***Unregulated Stage (ft)***	***Regulation Stage (ft)***	***Regulation Reduction (ft)***	***Unregulated Flow (cfs)***	***Regulated Flow (cfs)***
2/23/2015	22.4	20.3	2.1	52,100	41,500
3/6/2015	26.2	23.3	2.9	71,100	56,600
4/4/2015	21.7	20.4	1.3	48,600	42,000
4/15/2015	22.2	20.7	1.5	51,100	43,600
7/16/2015	24.0	21.5	2.5	60,100	47,600
12/26/2015	20.0	18.2	1.8	40,000	31,000
2/4/2016	20.4	19.0	1.4	42,000	35,000
2/17/2016	23.1	20.9	2.2	55,600	44,600
5/2/2016	22.4	20.9	1.5	52,100	44,600
7/16/2015	24.0	21.5	2.5	60,100	47,600
4/24/2017	21.3	19.8	1.5	46,600	39,000

2/12/2018	24.4	22.3	2.1	62,100	51,500
2/18/2018	20.0	19.3	0.7	40,000	36,500
3/26/2018	20.6	19.8	0.8	43,100	39,100
12/16/2018	20.4	19.9	0.5	42,000	39,500
2/21/2019	22.8	21.5	1.3	54,100	47,600
2/24/2019	25.5	23.9	1.6	67,600	59,600
12/18/2019	22.1	21.5	0.7	50,600	47,300
12/31/2019	20.5	20.2	0.3	42,500	41,000
2/7/2020	23.9	21.6	2.4	59,600	47,800
2/14/2020	22.0	20.8	1.2	50,100	43,900
3/22/2020	20.8	20.1	0.8	44,100	40,300
3/2/2021	36.0	33.6	2.4	120,300	108,200
3/29/2021	22.9	21.3	1.6	54,600	46,600
7/29/2022	27.1	26.2	0.9	75,600	71,100

Table 5 from the Buckhorn Lake WCM shows a summary of flood frequency data, including a relationship between unregulated and regulated flows for the 2-, 5-, 10-, 25-, 50-, and 100-year events for Lock 14. Figure 3-9 shows Table 5 from the Buckhorn WCM.

TABLE 4-5

SUMMARY OF FLOOD FREQUENCY DATA

USGS Station Number	Stream Name and Location	Type of Flow	Peak Discharge in cfs 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	Record No. of Years	Period	Max. Known Discharge Date	cfs
03280600	Middle Fork Kentucky River nr Hyden	N *	14,800	23,200	31,000	42,600	53,600	64,800	28	1957-84	1957	60,000
03281000	Middle Fork Kentucky River	N *	15,400	23,700	35,000	49,500	61,000	70,000	14	1939-53	1957	52,700
	at Tellega	M **	4,660	5,900	7,040	9,260	11,100	13,400				
03282000	Kentucky River @ Lock 14	N *	63,000	82,500	96,700	114,000	127,000	139,000	59	1921-79	1939	120,000
	@ Heidelberg	M ***	53,400	69,700	81,300	96,200	118,000	119,300				
		M ***	53,400	69,000	80,200	95,000	116,600	118,300				

* Natural
** Modified by Buckhorn Lake
*** Modified by Buckhorn lake + Carr Fork Lake

Figure 3.9: Buckhorn Lake WCM Table 5

The unregulated values for Lock 14 and the regulated values noted as 'Modified by Buckhorn Lake + Carr Fork Lake" were plotted in Figure 3-11. It should be noted that two rows in the

TABLE 4-5

SUMMARY OF FLOOD FREQUENCY DATA

USGS Station Number	Stream Name and Location	Type of Flow	Peak Discharge in cfs 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	Record No. of Years	Period	Max. Known Discharge Date	cfs
03280600	Middle Fork Kentucky River nr Hyden	N *	14,800	23,200	31,000	42,600	53,600	64,800	28	1957-84	1957	60,000
03281000	Middle Fork Kentucky River at Tellega	N *	15,400	23,700	35,000	49,500	61,000	70,000	14	1939-53	1957	52,700
		M **	4,660	5,900	7,040	9,260	11,100	13,400				
03282000	Kentucky River @ Lock 14	N *	63,000	82,500	96,700	114,000	127,000	139,000	59	1921-79	1939	120,000
	@ Heidelberg	M ***	53,400	69,700	81,300	96,200	118,000	119,300				
		M ***	53,400	69,000	80,200	95,000	116,600	118,300				

* Natural
** Modified by Buckhorn Lake
*** Modified by Buckhorn lake + Carr Fork Lake

Figure 3-9 are designated with "***" for Lock 14 flows. The row second from the bottom in

TABLE 4-5

SUMMARY OF FLOOD FREQUENCY DATA

USGS Station Number	Stream Name and Location	Type of Flow	Peak Discharge in cfs 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	Record No. of Years	Period	Max. Known Discharge Date	cfs
03280600	Middle Fork Kentucky River nr Hyden	N *	14,800	23,200	31,000	42,600	53,600	64,800	28	1957-84	1957	60,000
03281000	Middle Fork Kentucky River at Tellega	N *	15,400	23,700	35,000	49,500	61,000	70,000	14	1939-53	1957	52,700
		M **	4,660	5,900	7,040	9,260	11,100	13,400				
03282000	Kentucky River @ Lock 14	N *	63,000	82,500	96,700	114,000	127,000	139,000	59	1921-79	1939	120,000
	@ Heidelberg	M ***	53,400	69,700	81,300	96,200	118,000	119,300				
		M ***	53,400	69,000	80,200	95,000	116,600	118,300				

* Natural
** Modified by Buckhorn Lake
*** Modified by Buckhorn lake + Carr Fork Lake

Figure 3-9 is assumed to be a typo and should be designated as "**", which based on the footnote means values are "Modified by Buckhorn Lake".

Additionally, the modified Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) model used a calibrated HEC-HMS CWMS model of the entire Kentucky River basin created in 2016 by the USACE Louisville District as a basis. No further calibration or validation was performed on the model as part of this analysis. The following modifications were made to the model for the purposes of this assessment:

- All nodes and reaches downstream of Lock 14 removed.
- All nodes and reaches upstream of Buckhorn Lake removed.

- All nodes and reaches in North Fork upstream of the North Fork and Middle Fork confluence removed.
- All nodes in the South Fork removed.

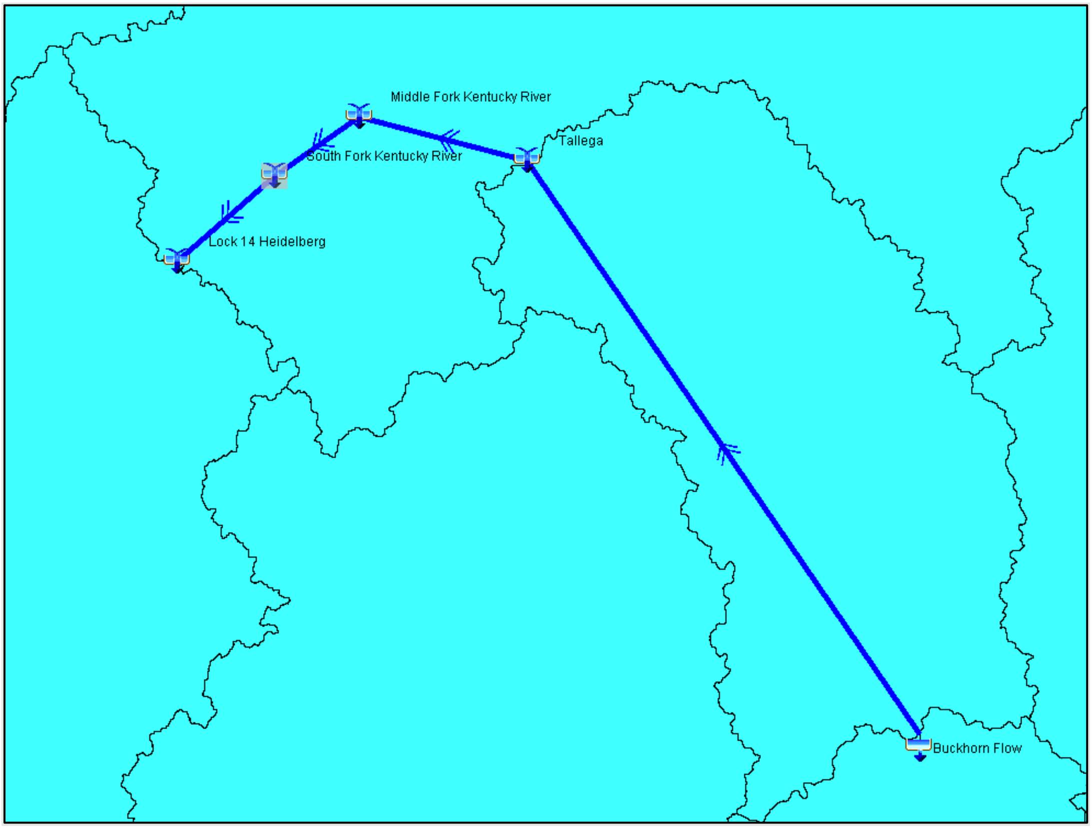

Figure 3.10: HEC-HMS Modified Model for Lock 14 and Buckhorn Lake

These edits created a model representing only the nodes and reaches between Buckhorn Lake and Lock 14. The North Fork and Carr Creek Lake were removed from the model to simplify the modeling due to the fact that Carr Creek provides minimal regulation compared to Buckhorn Lake. Operational data from Buckhorn Lake was obtained from the Louisville District Water Management section, which included the following data between November 24, 1982, through May 2022 for Buckhorn Lake in 6-hour time increments. Operational data included:

- Buckhorn Lake stage height (ft)
- Buckhorn Lake outflow (cfs)
- Buckhorn Lake inflow (cfs)

A new 6-hour increment time series was created by subtracting the Buckhorn Lake outflow data from the inflow data during storm events. This time series represents the amount of flow stored, or regulated, by Buckhorn Lake during storm events.

The flow stored time series was then routed through the modified model as a specific discharge gage at the Buckhorn Lake location. This analysis created an estimated 6-hour time series of the reduction in flows in Middle Fork at the Tallega gage and at Lock 14 by the regulation of Buckhorn Lake.

Detailed USGS data was then downloaded at Lock 14 and Tallega. Detailed data included discharge values in hourly increments for Lock 14 USGS gage 03282000 at Heidelberg from October 1987 to February 2004 and for Middle Fork USGS gage 03281000 at Tallega from October 1987 to May 2004, after which both gage locations had 15-minute increment discharge data available through the present. The discharge data from the Lock 14 and Tallega gages were converted to 6-hour time increments to align with the operational data for Buckhorn Lake. Output results at Lock 14 and Tallega from the modified HEC-HMS simulation using the Buckhorn Lake flow stored time series were added to corresponding converted 6-hour time increment peak discharges from the USGS gage to estimate flow in the unregulated condition. Annual peak flow events from 1988 to 2014 and corresponding major flow events between 2015 and 2021 in the spreadsheet model were analyzed in this way to develop the unregulated to regulated flow relationship. Table 8 summarizes the results at Lock 14 of these events for the modified HEC-HMS model analysis.

Table 8: HEC-HMS Lock 14 Regulation Analysis Results

Date	*Lock 14 Regulated Flow (cfs)*	*HEC-HMS Flow Stored (cfs)*	*Estimated Unregulated Flow (cfs)*
1/20/1988	22,667	389	23,056
6/16/1989	55,317	707	56,023
10/18/1989	52,233	21,018	73,251
2/20/1991	41,667	9,351	51,018
12/4/1991	53,633	10,082	63,716
3/5/1993	34,250	3,025	37,275
3/29/1994	54,417	9,116	63,533
5/19/1995	41,667	676	42,342
5/29/1996	42,300	202	42,502
3/4/1997	35,800	4,531	40,331
4/20/1998	53,900	16,029	69,929
1/10/1999	38,300	636	38,936
4/5/2000	28,233	7,453	35,686
2/17/2001	35,883	3,887	39,770
3/19/2002	45,717	19,381	65,098

2/17/2003	76,500	5,784	82,284
2/7/2004	59,867	3,528	63,394
5/1/2005	45,708	8,969	54,677
1/24/2006	27,450	6,310	33,760
4/16/2007	35,154	13,628	48,782
4/5/2008	23,608	1,379	24,987
5/9/2009	52,454	3,497	55,951
1/25/2010	40,450	10,154	50,604
4/17/2011	35,500	9,860	45,360
1/21/2012	25,117	1,918	27,035
1/17/2013	35,458	11,619	47,078
12/7/2013	33,546	4,496	38,042
2/23/2015	34,621	10,357	44,978
3/6/2015	51,542	19,695	71,237
4/4/2015	35,608	700	36,308
4/15/2015	37,958	4,399	42,357
7/16/2015	41,054	12,863	53,917
12/26/2015	24,567	3,785	28,352
2/4/2016	28,483	4,012	32,495
2/17/2016	39,496	13,106	52,602
5/2/2016	37,867	5,358	43,224
7/16/2015	41,054	12,863	53,917
4/24/2017	33,063	12,443	45,506
2/12/2018	47,617	24,112	71,728
2/18/2018	30,254	7,378	37,632
3/26/2018	33,258	10,667	43,926
12/16/2018	33,604	4,685	38,289

2/21/2019	42,808	10,695	53,504
2/24/2019	57,425	13,452	70,877
12/18/2019	41,325	6,969	48,294
12/31/2019	34,642	5,710	40,352
2/7/2020	43,167	24,491	67,658
2/14/2020	39,046	11,599	50,644
3/22/2020	33,904	8,143	42,047
3/2/2021	88,400	24,667	113,067
3/29/2021	41,492	20,228	61,719

The three data sources were plotted overlapped to select the regulated-unregulated flow relationships for Lock 14. Figure 3-11 shows the regulated-unregulated flow relationships from the three data sources with linear trendlines of each. Trendlines for each source had good correlation and plotted similarly against one another. The spreadsheet model and Buckhorn Lake WCM data trendlines were very similar with the modified HEC-HMS analysis plotting only slightly below. The Buckhorn Lake WCM relationship was used as the final regulated-unregulated flow relationship for Lock 14. This was the original estimated relationship when Buckhorn Lake was constructed, incorporates the regulation of North Fork based on the construction of Carr Creek Lake, and the other two data sources plotted similarly against it, indicating it is still a valid relationship for current conditions. A 10% upper and lower uncertainty bound is also shown on Figure 3-11 for reference.

Utilizing this relationship, the unregulated POR at Lock 14 was extended between 1961 to 2022 by transposing regulated flows with an established regulated-unregulated flow relationship for Lock 14.

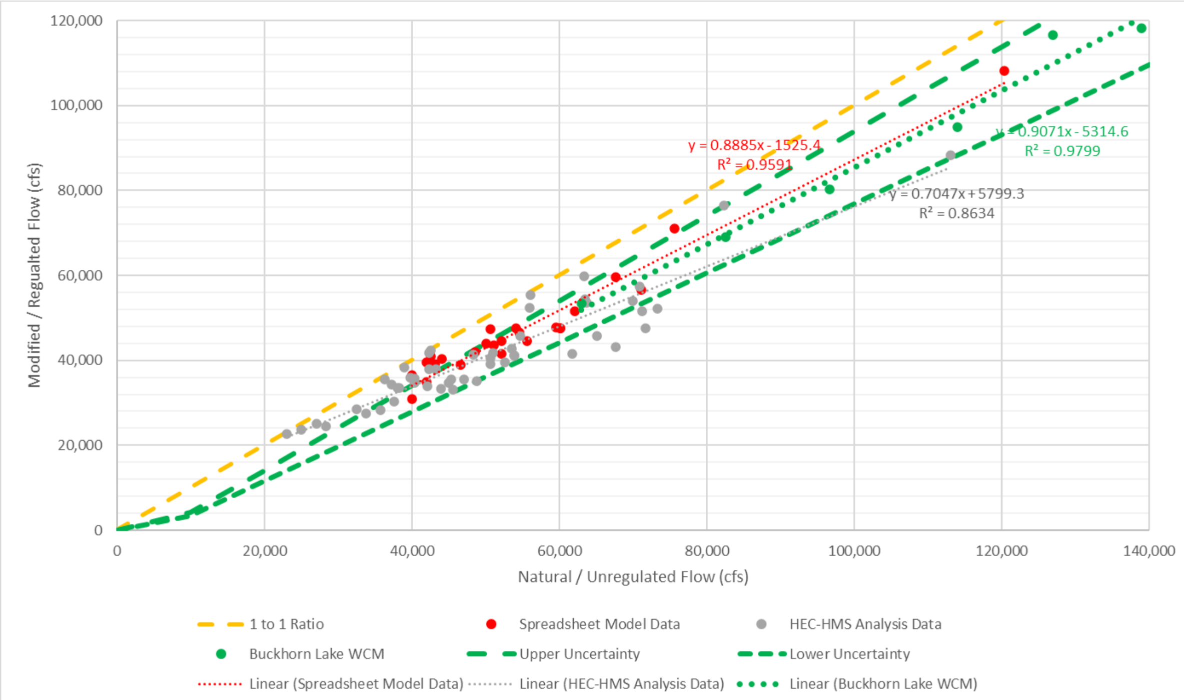

Figure 3.11: Lock 14 Regulated-Unregulated Relationship Review

3.6 HYDROLOGIC HAZARD CURVES – LOCK 14 AT HEIDELBERG

3.6.1 Perception Thresholds

Historic flood data dating back as far as 1817 at Lock 4 was also used to supplement the POR using PTs. In lieu of including direct systematic data transpositions from the Lock 4 USGS gage 03287500 at Frankfort, PTs were developed. Two PTs for Lock 14 were established, one based on the historic data between 1817 and 1894, and one for the annual peak inflow data at the Lock 4 USGS gage between 1895 and 1912 available at the Lock 4 USGS gage 03287500 at Frankfort.

For the historic data PT, a rating curve for Lock 4 was used to approximate the historic flood peak flows from 1817, 1847, 1854, 1880, and 1883 which only included gage heights. Figure 3-12 shows the Lock 4 USGS rating curve developed best on USGS data with linear best fit line and the current published USGS rating curve. The best fit line was used in this analysis to calculate Lock 4 flows as the current published USGS rating curve appears to be slightly overestimating flows based on the gage data. Those values are displayed below in Table 9.

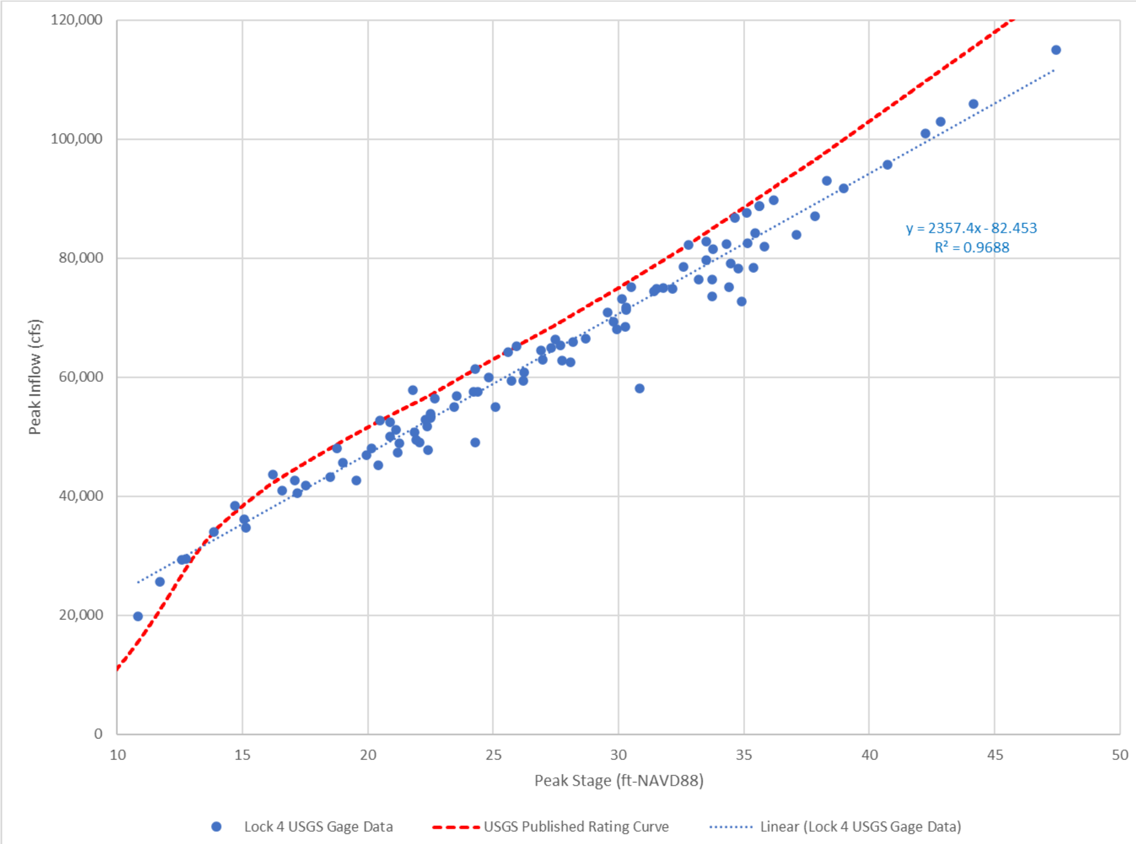

Figure 3.12: Lock 4 Rating Curve

Table 9: Peak Flows Calculated from Lock 4 Historic Stages

Water Year	***Lock 4 Peak Flow (cfs)***
1817	88,300
1847	87,100
1854	86,000
1880	87,100
1883	98,900

Next, scaling factors for unregulated annual peak flows for corresponding events from Lock 4 to 14 were developed. Table 10 shows a summary of the calculated scaling factors from corresponding annual peak inflow events between 1921 and 1960 that caused annual peak flows at Lock 14 USGS gage 03282000 at Heidelberg and Lock 4 USGS gage 03287500 at Frankfort.

Table 10: Lock 14 and Lock 4 Inflow Scaling Factors

Date	***Lock 14 Peak Flow (cfs)***	***Lock 4 Peak Flow (cfs)***	***Scaling Factor***
4/17/1921	29,600	29,300	1.01
2/21/1922	75,100	65,000	1.16
2/4/1923	84,800	69,300	1.22
1/4/1924	55,600	87,700	0.63
1/22/1926	51,900	61,400	0.85
12/23/1926	90,400	72,700	1.24
6/29/1928	35,300	66,500	0.53
3/24/1929	113,000	75,200	1.50
4/23/1931	45,500	43,700	1.04
1/31/1932	79,700	71,800	1.11
2/21/1933	45,500	57,800	0.79
3/4/1934	76,100	64,500	1.18
3/13/1935	83,300	79,700	1.05
4/7/1936	60,300	65,900	0.92
7/15/1938	26,000	42,700	0.61
2/4/1939	120,000	88,800	1.35
4/21/1940	39,300	45,700	0.86
7/6/1941	31,300	25,600	1.22
3/6/1945	51,000	78,400	0.65
1/9/1946	90,300	73,200	1.23
2/15/1948	104,000	88,700	1.17
2/2/1950	69,200	87,000	0.80
2/2/1951	100,000	70,900	1.41

3/24/1952	70,500	76,400	0.92
3/1/1955	75,000	84,000	0.89
2/19/1956	74,200	75,000	0.99
1/30/1957	116,000	84,200	1.38
5/8/1958	63,300	60,800	1.04
1/23/1959	37,700	42,600	0.88
		Mean	1.02
		Minimum	0.53
		Maximum	1.50

Based on Table 10, maximum, mean, and minimum scale factors were calculated to be 1.5, 1.02, and 0.50, respectively. Table 11 summarizes the Lock 4 USGS gage 03287500 at Frankfort historic data converted to Lock 14 based on the minimum, maximum and average scale factors.

Table 11: Lock 4 Historic Scaled Data for Lock 14

Water Year	***Lock 4 Peak Flow (cfs)***	***Lock 14 Flow, Minimum Scale Factor (cfs)***	***Lock 14 Flow, Mean Scale Factor (cfs)***	***Lock 14 Flow, Maximum Scale Factor (cfs)***
1817	88,300	46,800	90,100	132,500
1847	87,100	46,200	88,800	130,700
1854	86,000	45,600	87,700	129,000
1880	87,100	46,200	88,800	130,700
1883	98,900	52,400	100,900	148,400

Based on the maximum scale factor flows in Table 10, a rounded PT of 150,000 cfs was selected based on historic data from Lock 4 USGS gage 03287500 at Frankfort between 1817 and 1894.

For the PT for annual peak flow data from 1895 to 1912, a similar process to the historic data was used. Table 12 summarizes the Lock 4 USGS data 03287500 at Frankfort from 1895 to 1912 converted to Lock 14 data based on the maximum and average scale factors.

Table 12: Lock 4 1895 to 1912 Scaled Data for Lock 14

Water Year	***Lock 4 Peak Flow (cfs)***	***Lock 14 Flow, Minimum Scale Factor (cfs)***	***Lock 14 Flow, Mean Scale Factor (cfs)***	***Lock 14 Flow, Maximum Scale Factor (cfs)***
1895	41,600	22,000	42,400	62,400
1896	47,800	25,300	48,800	71,700
1897	76,900	40,800	78,400	115,400
1898	59,800	31,700	61,000	89,700
1899	76,400	40,500	77,900	114,600
1900	29,600	15,700	30,200	44,400
1901	57,600	30,500	58,800	86,400
1902	68,000	36,000	69,400	102,000
1903	70,800	37,500	72,200	106,200
1904	33,400	17,700	34,100	50,100
1905	50,200	26,600	51,200	75,300
1906	50,800	26,900	51,800	76,200
1907	63,400	33,600	64,700	95,100
1908	71,300	37,800	72,700	107,000
1909	82,400	43,700	84,000	123,600
1910	52,900	28,000	54,000	79,400
1911	52,400	27,800	53,400	78,600
1912	64,200	34,000	65,500	96,300

Based on the maximum scale factor flows in Table 12, a rounded PT of 125,000 cfs was selected based on the annual peak unregulated flow data from Lock 4 USGS gage 03287500 at Frankfort between 1895 and 1912.

Data Summary

To summarize, the following information was used to develop the final unregulated POR data used for Lock 14.

- PTs were used for on historic data from 1817 to 1894 and annual peak inflow data from 1895 to 1912 based on Lock 4 USGS gage 0327500 at Frankfort data using scaling factors established between consistent events at Lock 4 and Lock 14.
- Unregulated Lock 10 USGS gage 03284000 at Winchester peak inflow data between 1913 to 1920 was transposed to Lock 14 based on the MOVE.3 function.
- Unregulated flows from Lock 14 USGS gage 03282000 at Heidelberg from 1921 to 1960 were directly used.
- The Buckhorn Lake WCM unregulated to regulated flow relationship was used to transpose Lock 14 USGS gage 03282000 at Heidelberg regulated flows to unregulated flows between 1961 and 2022.

Figure 3-13 shows a visual of the final composite POR used in the assessment.

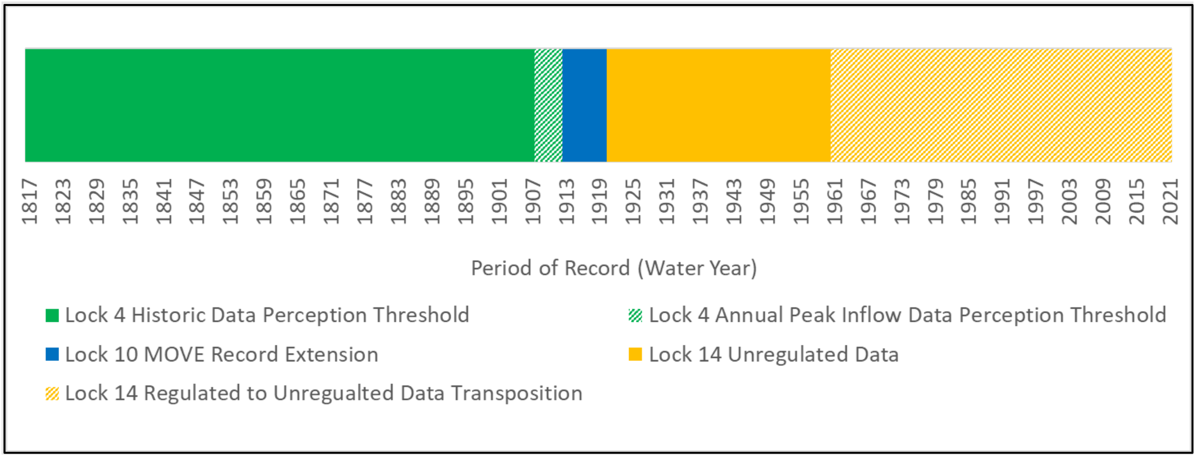

Figure 3.13: Lock 14 Final POR Visualization

3.6.2 Unregulated Flow-Frequency Curve

The composite unregulated annual peak flow POR was used to calculate the peak unregulated flow frequency curve using a B17C analysis in the Hydrologic Engineering Center's (HEC) Statistical Software Package (SSP). The final POR included systematic and transposed data from 1817 to 2022 as previously discussed. Figure 3-14 shows the HEC-SSP expected moment algorithm (EMA) data for the unregulated POR.

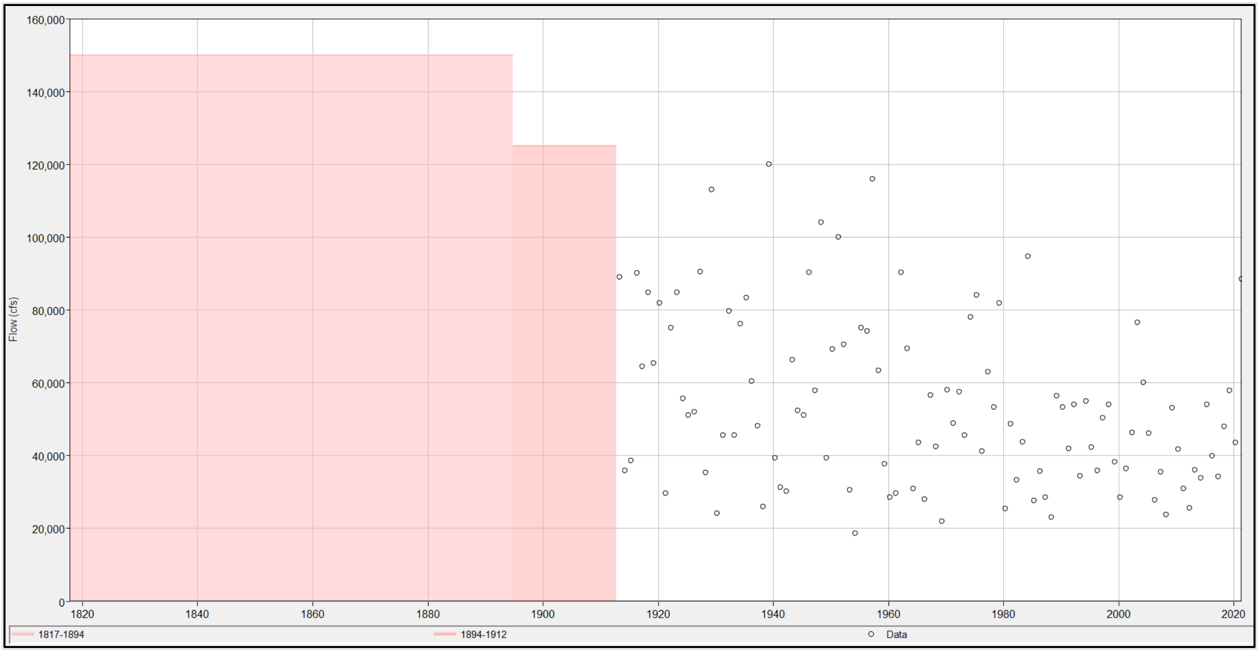

Figure 3.14: Lock 14 Unregulated POR HEC-SSP EMA Data

Regional Skew Information

Regional skew information was available from an existing study published by the USGS in cooperation with the Kentucky Transportation Cabinet. The study is titled "*Estimating the Magnitude of Peak Flows for Streams in Kentucky for Selected Recurrence Intervals*" and was published in 2003. The regional skew values for peak flows for the state of Kentucky is 0.011 with a standard error and mean square error of 0.52 and 0.27, respectively. The regional skew was applied to the B17C analysis to develop a weighted skew, which was the final adopted skew for all flow frequency analyses.

A sensitivity analysis was performed comparing the impacts of using the station skew to the weighted skew by performing a B17C analysis with each skew for Lock 14. Figure 3-15 shows the results of the comparison in HEC-SSP. As shown in Figure 3-15, the analysis is not sensitive to the skew used and the difference in results between the two is negligible. As noted above, the weighted skew was the final adopted skew used for all subsequent flow frequency analyses.

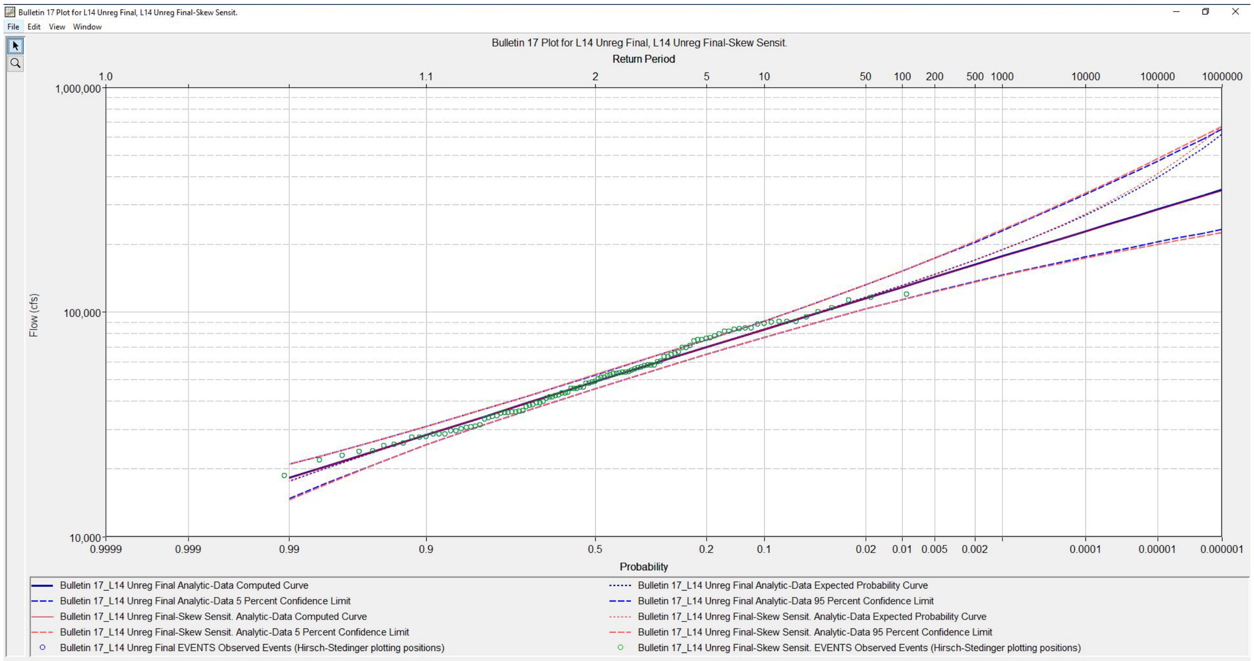

Figure 3.15: Lock 14 Unregulated Station Skew and Weighted Skew Sensitivity

Potentially Influential Low Outliers

The Multiple Grubbs-Beck Test was performed on the data to determine if the POR contained any potentially influential low flood (PILF) values. PILF's are reviewed to confirm that small observed inflows do not have an inappropriately large impact on the inflow frequency analysis. The test identified no PILF values.

Unregulated Flow Frequency Sensitivities

To determine how sensitive the analysis is to the inclusion of additional data sources, multiple sensitivities were reviewed by plotting overlapping data from the various data sets. Flow-frequency curves for the following data sets were developed using a B17C analysis in HEC-SSP:

- Lock 14 Unregulated Only – includes unmodified unregulated flow data from Lock 14 USGS gage 03282000 at Heidelberg from 1921 to 1960
- Lock 10 and Lock 14 Combined Unregulated Only – includes transposed unregulated Lock 10 USGS gage 03284000 at Winchester to Lock 14 USGS gage 03282000 at Heidelberg data via the MOVE.3 function from 1913 to 1920 and unmodified unregulated flow data from Lock 14 from 1921 to 1960.
- Lock 14 Transposed Unregulated – includes unmodified regulated flow data from Lock 14 USGS gage 03282000 at Heidelberg from 1921 to 1960 and transposed regulated to unregulated Lock 14 data from 1961 to 2022.
- Lock 10 Combined and Lock 14 Transposed Unregulated – includes transposed unregulated Lock 10 USGS gage 03284000 at Winchester to Lock 14 USGS gage 03282000 at Heidelberg data from 1913 to 1920 via the MOVE.3 function, unmodified

regulated flow data from Lock 14 from 1921 to 1960 and transposed regulated to unregulated Lock 14 data from 1961 to 2022.

- Final POR – includes the final period of record as previously discussed with PTs.

Figure 3-16 show a plot of the curves for the above data sets along with the 1939 unregulated flood of record at 120,000 cfs and the final POR systematic data points.

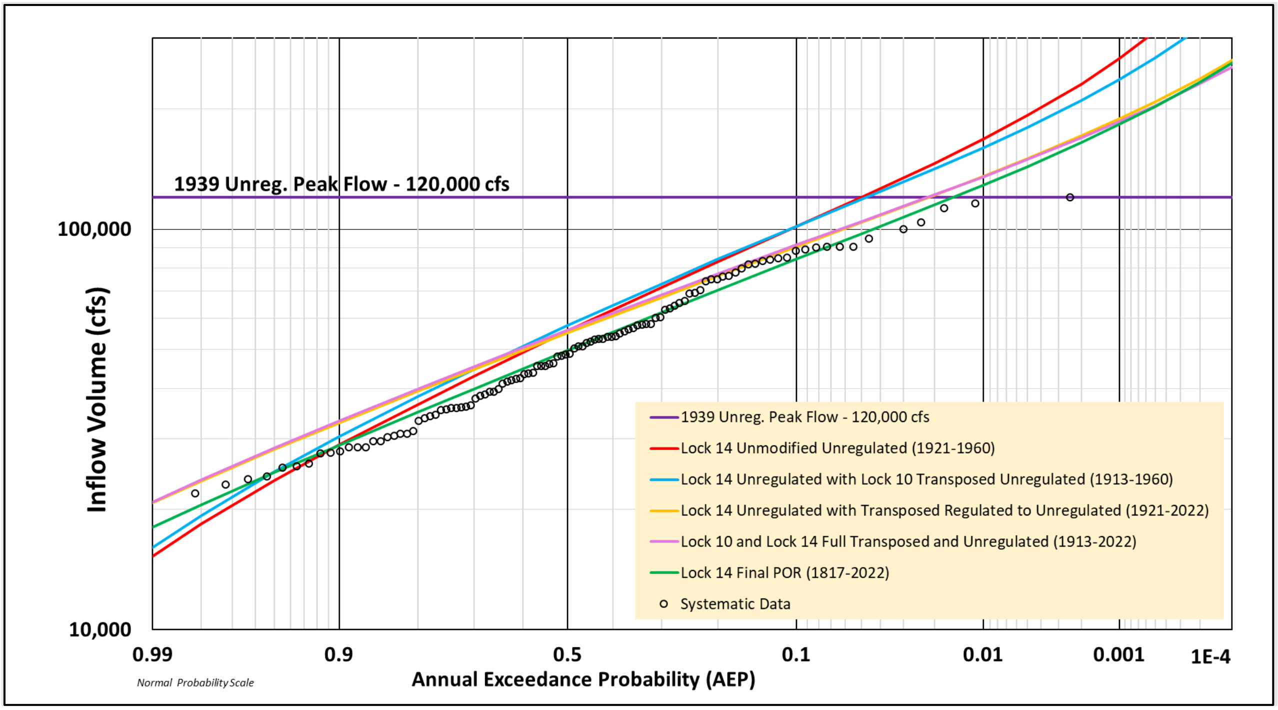

Figure 3.16: Lock 14 Unregulated Inflow Frequency Sensitivities

Table 13 summarizes the mean, standard deviation, and skew of the sensitivities plotted in Figure 3-16, which were similar between the sensitivities reviewed. All plot more frequent for the 1939 flood value compared to the POR final curve.

Table 13: Lock 14 Statistical Summary of Bulletin 17C Sensitivities

Data Set Description	***Mean (of log) (μ)***	***Std. Dev. (of log) (σ)***	***Skew (of log) (γ)***
Lock 14 Unmodified Unregulated (1921-1960)	4.740	0.208	-0.192
Lock 14 Unregulated with Lock 10 Transposed Unregulated (1913-1960)	4.752	0.201	-0.269
Lock 14 Unregulated with Transposed Regulated to Unregulated (1921-2022)	4.739	0.170	-0.098
Lock 10 and Lock 14 Full Transposed and Unregulated (1913-2022)	4.744	0.169	-0.138

Lock 14 Final POR (1817-2022)	4.685	0.179	-0.022

Figure 3-17 shows the final unregulated flow-frequency curve for Lock 14 using the B17C analysis in HEC-SSP.

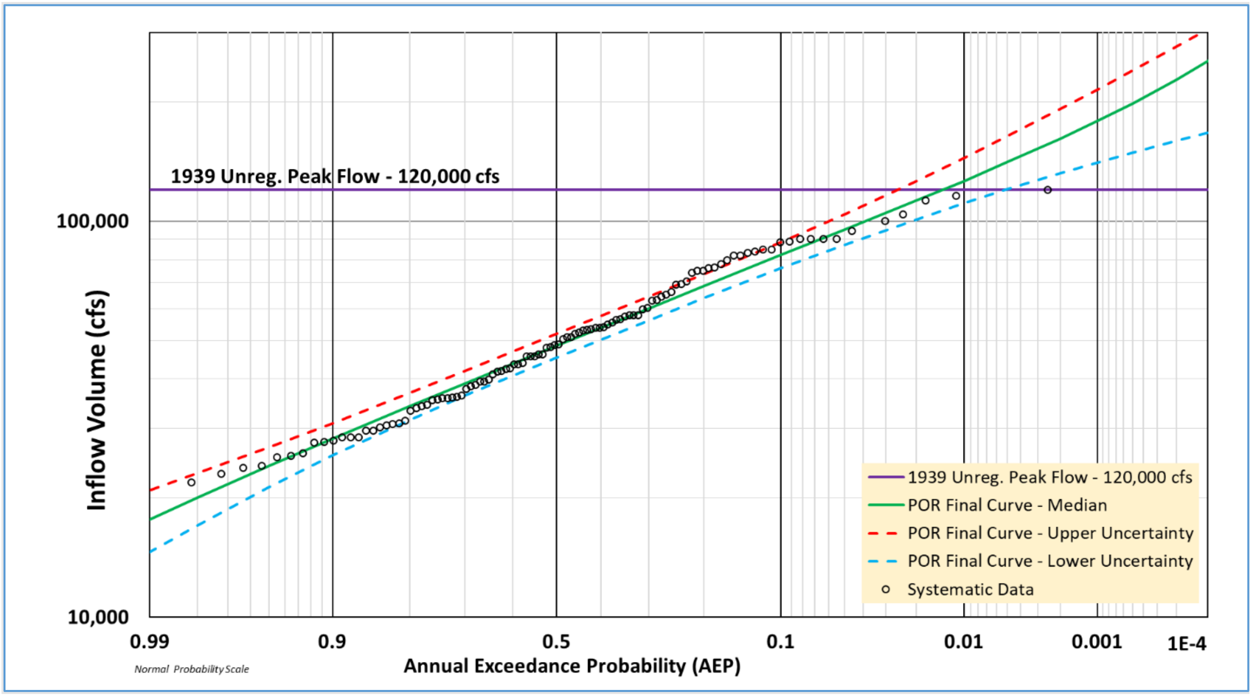

Figure 3.17: Lock 14 Unregulated Flow Frequency Curve

3.6.3 Regulated Flow-Frequency Curve

The previously established regulated-unregulated relationship shown in Figure 3-11 was used to develop the regulated flow frequency curve from the final unregulated flow frequency curve. A sensitivity analysis was performed transposing the unregulated flow frequency curve to a regulated flow frequency curve by directly transposing the data using the best estimate and by also using the upper and lower uncertainty bounds. The comparison of the two transpositions is shown in Figure 3-18.

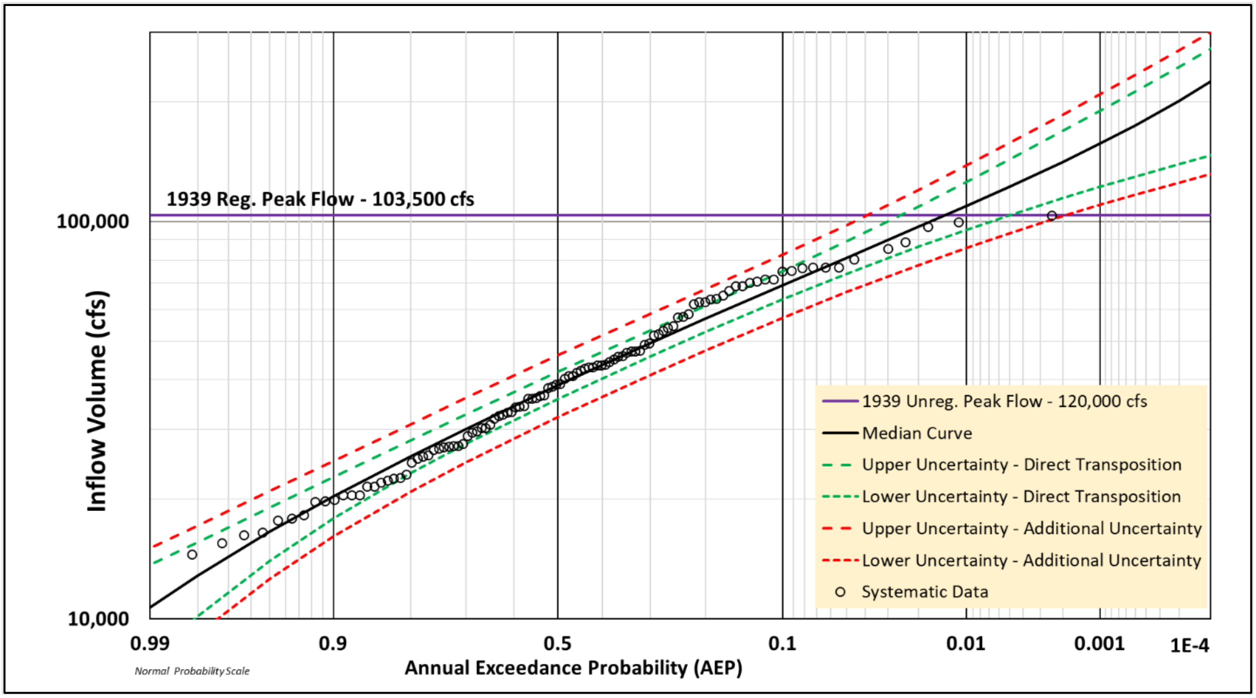

Figure 3.18: Lock 14 Transposed Regulated Flow Frequency Comparison

To incorporate the uncertainty associated with the regulated-unregulated transposition, the transposition using the upper and lower uncertainty bounds was selected.

Figure 3-19 shows the final regulated flow frequency curve for Beattyville.

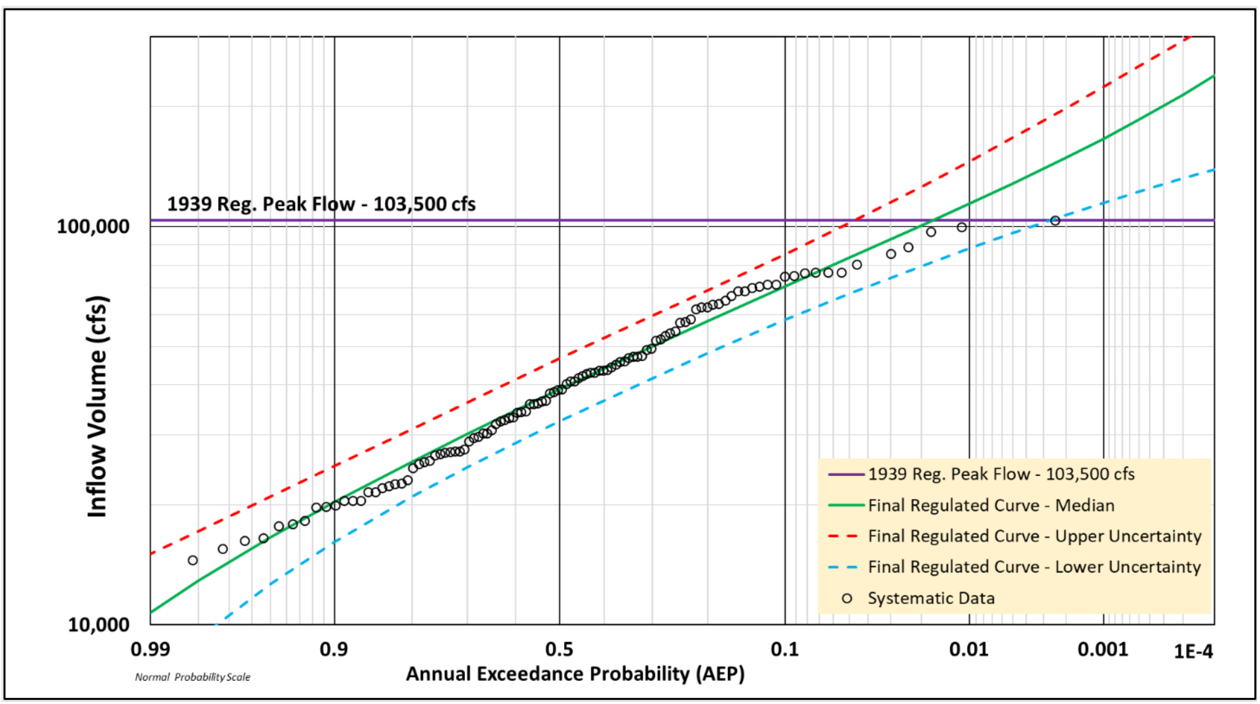

Figure 3.19: Lock 14 Final Regulated Flow-Frequency Curve

The final POR unregulated and regulated flow-frequency curves are plotted in Figure 3-20. The unregulated and regulated systematic data points are shown for reference.

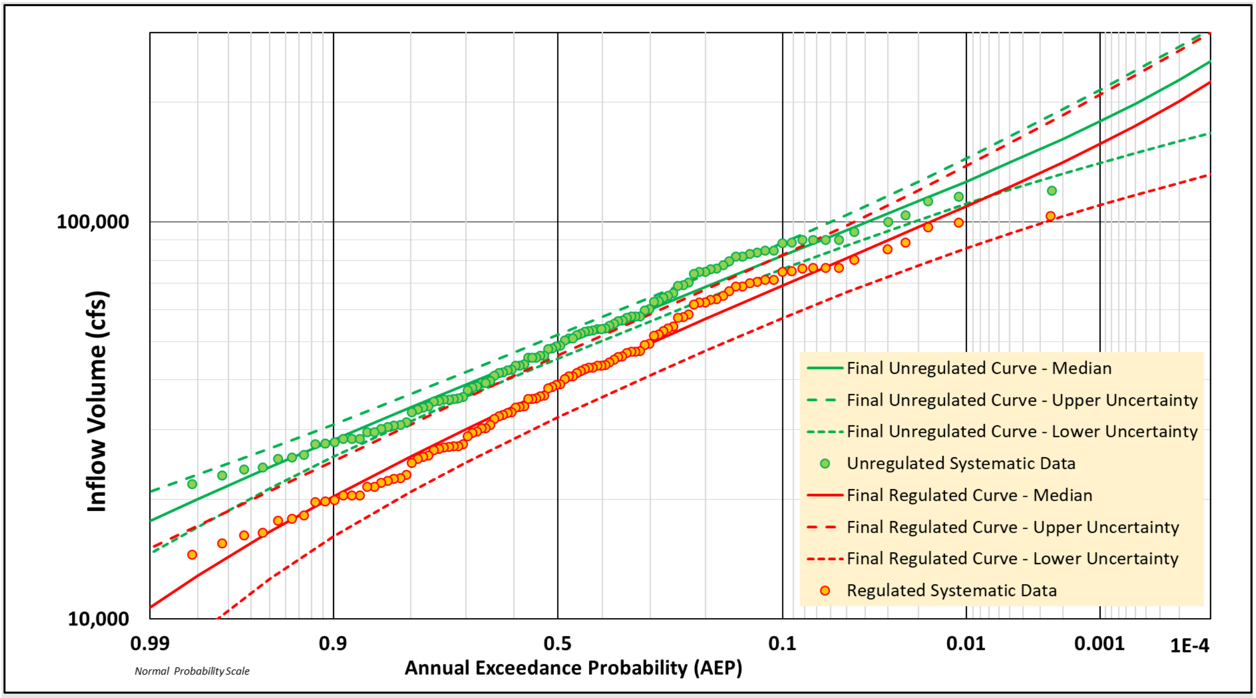

Figure 3.20: Lock 14 Regulated and Unregulated Flow Frequency Curve Comparison

3.6.4 Regulated Stage-Frequency Curve

To develop the final stage-frequency curve for Beattyville, a composite rating curve was developed based on the 500-year AEP event at Cross Section 257.5170 of the HEC-RAS model developed for the planning study. Cross Section 257.5170 is on the Kentucky River at Beattyville immediately downstream of the confluence of the North Fork and South Fork. Figure 3-21 shows the location of Cross Section 257.5170 from the HEC-RAS model. Figure 3-22 shows the rating curve at Cross Section 257.5170 for the 500-year AEP event with 15% upper and lower uncertainty bounds and the composite curve used to create the stage-frequency curve at Beattyville.

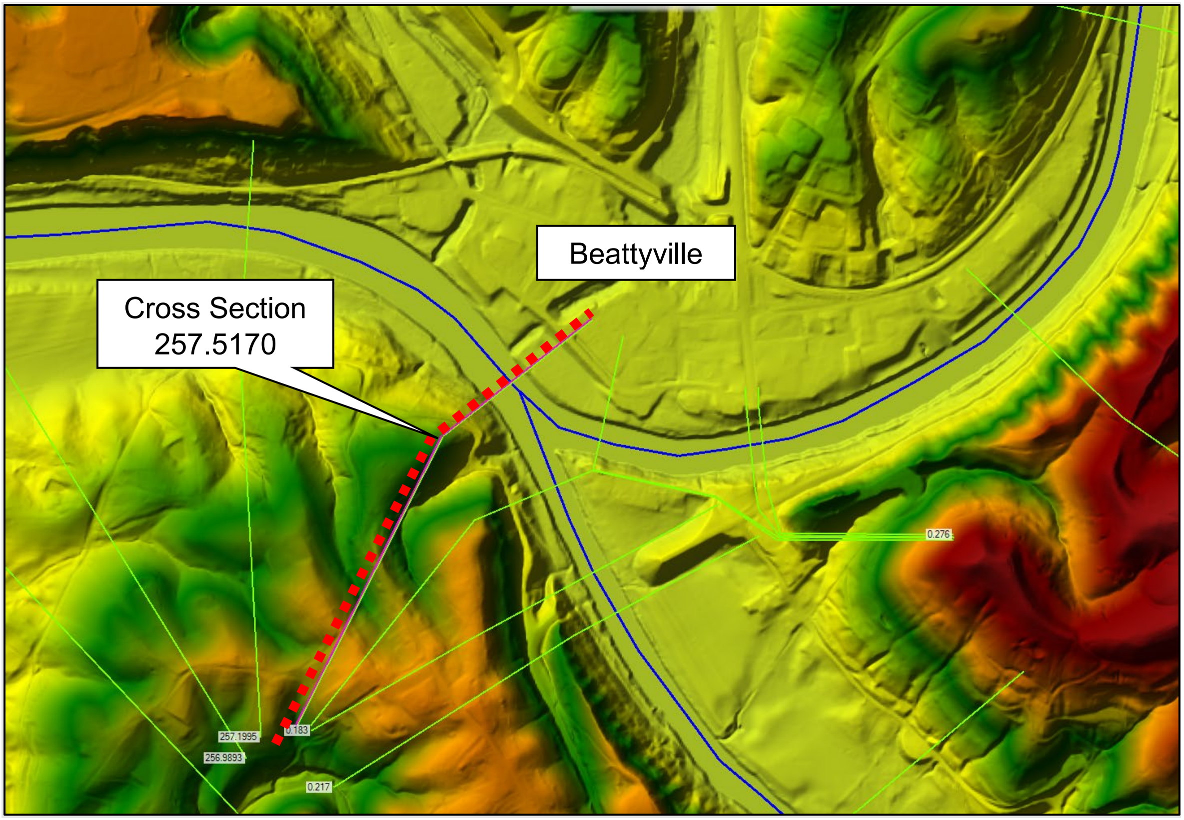

Figure 3.21: Beattyville HEC-RAS Cross Section 257.5170

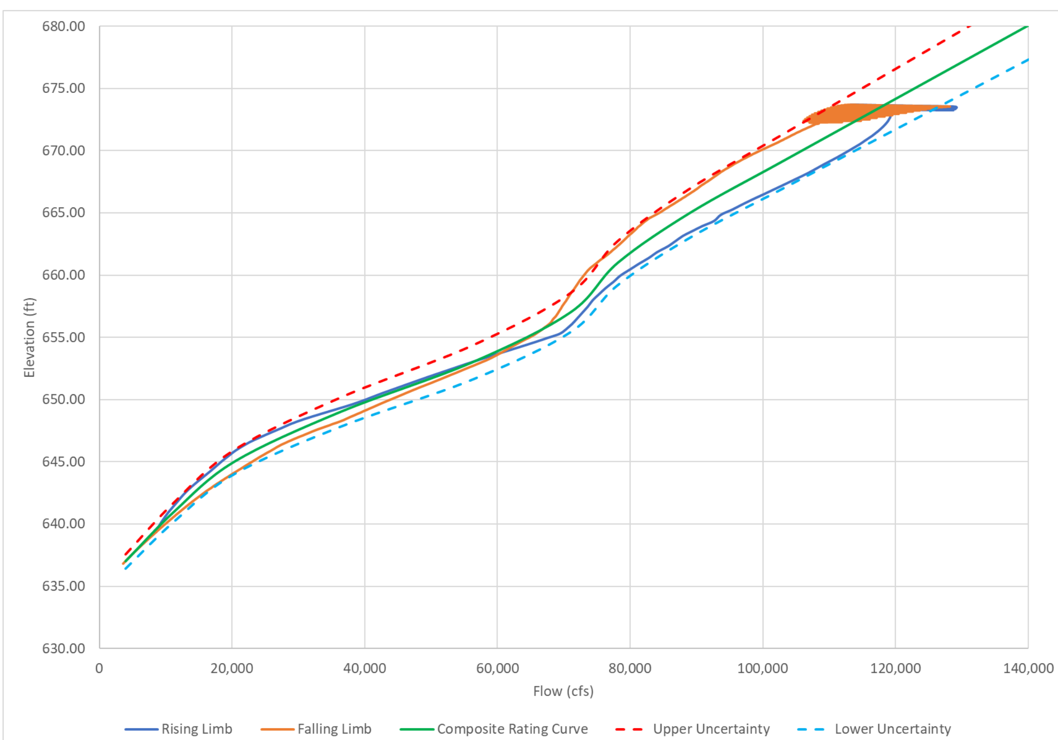

Figure 3.22: HEC-RAS Model Cross Section 257.5170 Rating Curve

As shown in Figure 3-22, the rating curve from the HEC-RAS model creates a hysteresis loop. There is not enough information available with the annual peak flow data to determine if peak inflows occurred during the rising or falling limbs. Therefore, a composite curve was created and used for the development of the stage-frequency curve at Beattyville. For the purposes of creating the composite curve, the noise at the top of the hysteresis loop was simplified.

Annual peak inflows at Cross Section 257.5170 at Beattyville used for the development of the stage frequency curve were adjusted based on a drainage area ratio between the Lock 14 USGS gage 03282000 at Heidelberg drainage area and the Cross Section 257.5170 drainage area. Cross Section 257.5170 is immediately downstream of the confluence of North and South Fork, therefore the total drainage area of the three headwater basins of 2,631 square miles was used for the Beattyville drainage area. The listed drainage area of Lock 14 USGS gage 03282000 at Heidelberg drainage area is 2,657 square miles, therefore a drainage area ratio of 0.99 was used.

To select the degree of uncertainty to incorporate into the final stage frequency curve, a sensitivity analysis was performed. A series of data set conversions for the unregulated to regulated transposition and the transposition from regulated flow to stage were plotted. The data sets compare various stage frequency curves using direct transpositions and using the upper/lower uncertainty bounds shown in Figure 3-11 for the unregulated/regulated relationship and Figure 3-22 for the stage transposition. Four different comparisons were made, which are described below and shown in Figure 3-23.

1. Data set 1 shows the upper/lower stage frequency curve based on a direct transposition of the unregulated/regulated relationship and direct transposition of stage using the composite rating curve.

2. Data set 2 shows the upper/lower stage frequency curve based on using the upper/lower uncertainty bounds shown in Figure 3-11 for the unregulated/regulated relationship and direct transposition of stage using the composite rating curve.

3. Data set 3 shows the upper/lower stage frequency curve based on using the upper/lower uncertainty bounds shown in Figure 3-11 for the unregulated/regulated relationship and the upper/lower uncertainty bounds shown in Figure 3-22 for the stage transposition.

4. Data set 4 shows the upper/lower stage frequency curve based on a direct transposition of the unregulated/regulated relationship and the upper/lower uncertainty bounds shown in Figure 3-22 for the stage transposition.

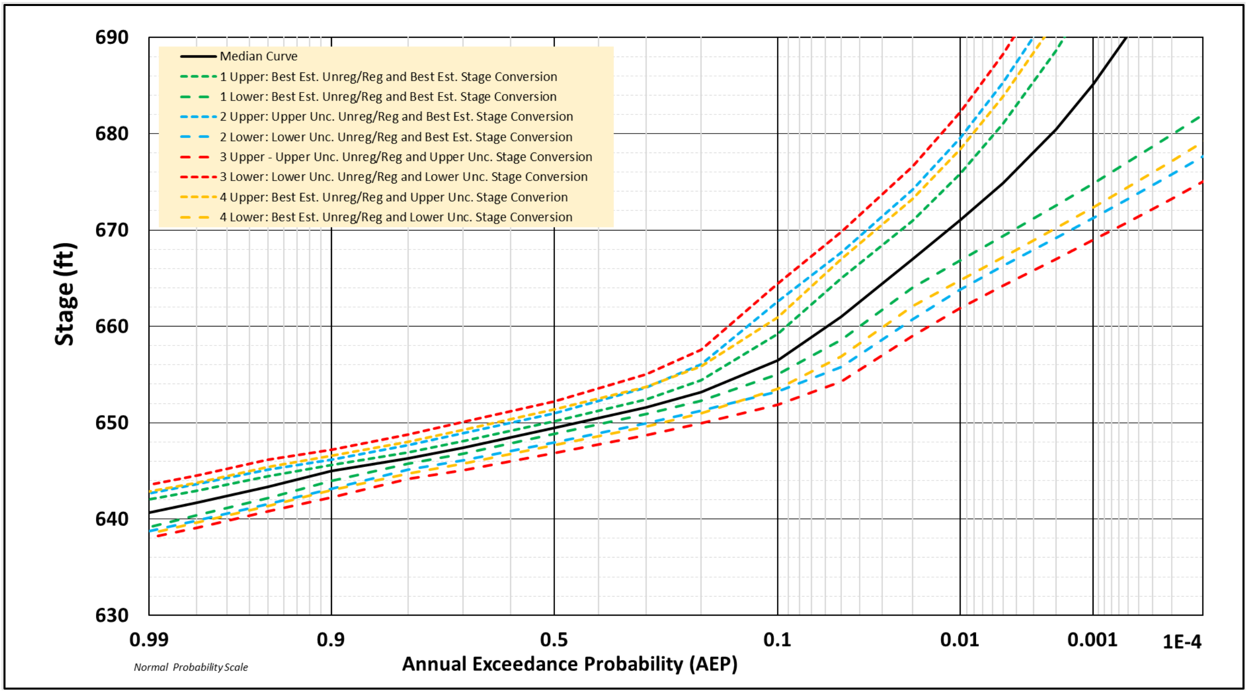

Figure 3.23: Beattyville Stage Frequency Sensitivity Analysis

Data set 2 was selected for the final stage frequency curve to incorporate some uncertainty into the analysis. Data set 1 does not incorporate any additional uncertainty and Data set 4 was considered to incorporate too much uncertainty. Data set 2 and 3 yielded similar results, but Data set 2 was selected based on engineering judgement.

Figure 3-24 shows the final regulated stage frequency curve for Beattyville.

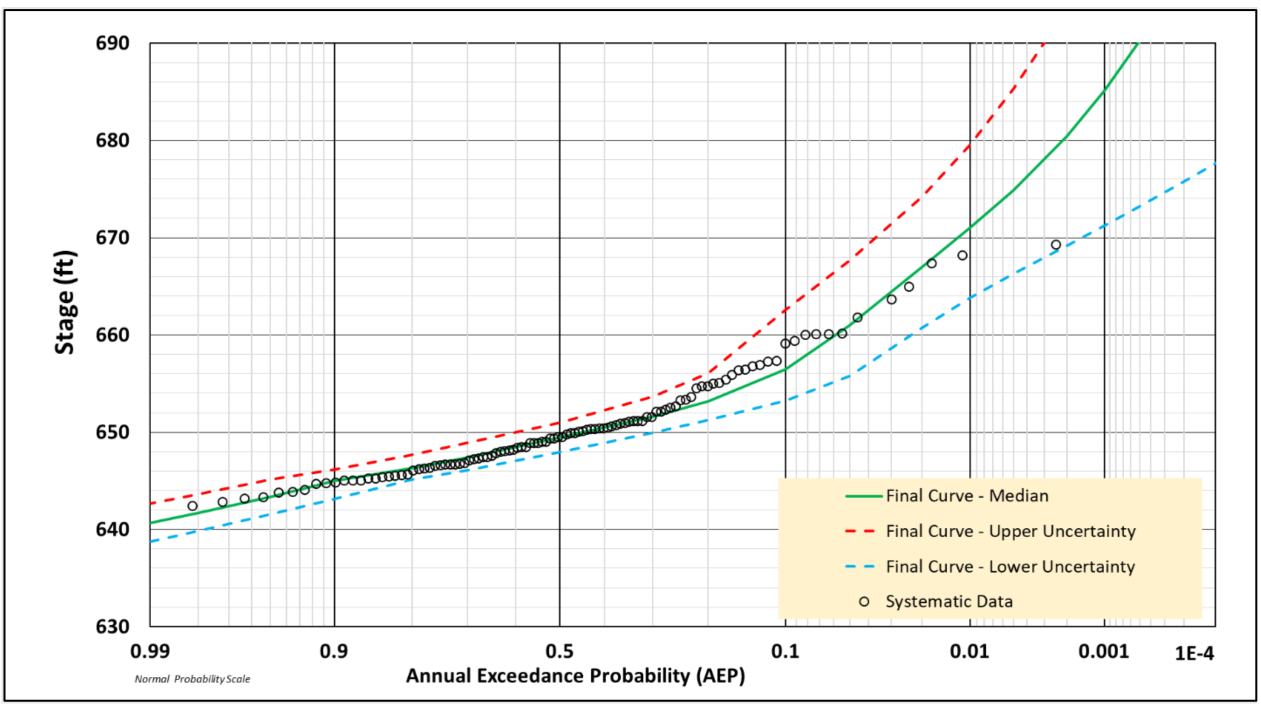

Figure 3.24: Final Beattyville Regulated Stage Frequency Curve

3.6.5 Hydrologic Hazard Curve Summary Tables

Table 14 and Table 15 summarize the final regulated flow and stage frequency curve for Lock 14 USGS gage 03282000 at Heidelberg at various AEPs. The published USGS rating curve shown in Figure 3-4 was used to develop the stage frequency curve for Lock 14.

Table 14: Lock 14 Final Regulated Flow Frequency Curve Summary Table

AEP	*Return Period (Years)*	*95% Confidence Interval (cfs)*	*5% Confidence Interval (cfs)*	*Expected [Mean] Curve (cfs)*	*Computed [Median] Curve (cfs)*
0.002	500	198,293	106,880	148,998	141,811
0.01	100	145,696	88,107	114,086	111,842
0.04	25	105,933	69,718	86,338	85,723
0.1	10	85,225	58,249	70,846	70,604
0.2	5	69,158	48,056	57,975	57,864
0.5	2	46,568	32,416	39,069	39,054
0.99	1	15,043	7,265	11,218	10,712

Table 15: Lock 14 Final Regulated Stage Frequency Curve Summary Table

AEP	*Return Period (Years)*	*95% Confidence Interval (ft)*	*5% Confidence Interval (ft)*	*Expected [Mean] Curve (ft)*	*Computed [Median] Curve (ft)*
0.002	500	676.55	658.37	666.75	665.32
0.01	100	666.10	654.84	659.81	659.36
0.04	25	658.19	651.51	654.51	654.39
0.1	10	654.30	649.50	651.71	651.67
0.2	5	651.41	647.74	649.46	649.44
0.5	2	647.49	644.95	646.18	646.17
0.99	1	641.29	638.72	640.16	639.99

Table 16 and Table 17 summarize the final regulated flow and stage frequency curve for Beattyville at various AEPs.

Table 16: Beattyville Final Regulated Flow Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	196,353	105,834	147,540	140,423
0.01	100	144,270	87,245	112,970	110,748
0.04	25	104,896	69,036	85,493	84,884
0.1	10	84,391	57,679	70,153	69,913
0.2	5	68,481	47,586	57,408	57,298
0.5	2	46,112	32,099	38,686	38,672
0.99	1	14,896	7,194	11,108	10,607

Table 17: Beattyville Final Regulated Stage Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (ft)***	***5% Confidence Interval (ft)***	***Expected [Mean] Curve (ft)***	***Computed [Median] Curve (ft)***
0.002	500	696.66	669.98	682.27	680.17
0.01	100	681.31	664.34	672.08	671.43
0.04	25	669.70	656.46	663.69	663.47
0.1	10	663.29	653.38	656.77	656.70
0.2	5	656.31	651.26	653.31	653.28
0.5	2	650.96	647.94	649.47	649.47
0.99	1	642.59	638.77	640.90	640.62

3.6.6 Levee Assurance and Accreditation

The planning study requires the development and review of alternatives to reduce flood risks and flood impacts to Beattyville. This includes levee system alternatives that could potentially be accredited to protect the Special Flood Hazard Areas (SFHA) from being inundated with reasonable assurance by the 1-percent-annual-chance flood, or the 100-year flood event. Reasonable assurance may vary but typically includes 65 to 85 percent assurance protection from the 100-year flood event or a minimum freeboard of 3 feet above the 100-year 50 percent (median) assurance protection flood event.

Figure 3-25 shows the 50 percent (median) stage frequency curve at Beattyville with the 85 percent and 65 percent assurance upper credible limits. 85 and 65 percent assurance credible limits for Beattyville were developed using the same process discussed Section 3.6.4 except the 85 and 65 upper confidence limits were used for the HEC-SSP analyses.

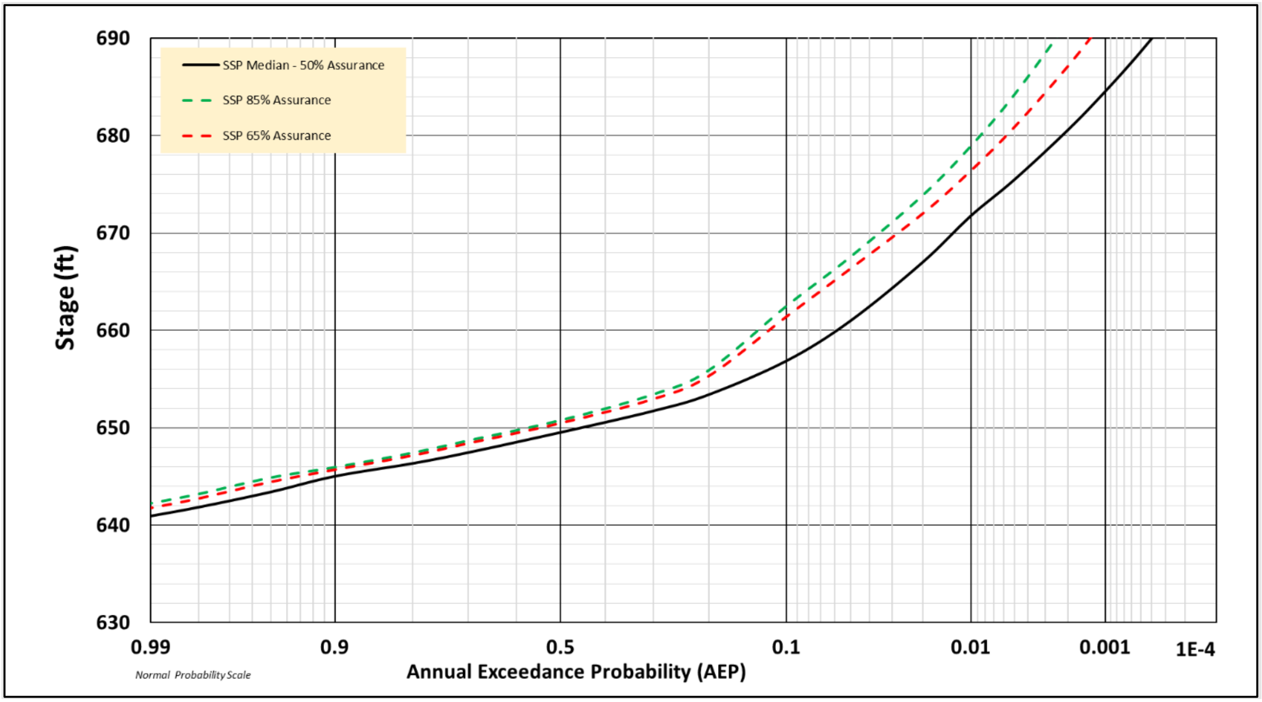

Figure 3.25: Beattyville Assurance Stage Frequency Plot

Table 18 summarizes the estimated flow and BFE values based on 85 percent assurance, 65 percent assurance, and 50 percent assurance plus 3 ft of freeboard.

Table 18: Beattyville Levee Assurance Summary

Data Set Description	***Flow – 0.01 AEP Event (cfs)***	***BFE – 0.01 AEP Event (ft)***
Beattyville 85 Percent Assurance	134,754	678.50
Beattyville 65 Percent Assurance	126,232	675.99
Beattyville 50 Percent Assurance	110,748	674.43*

*Includes 3 feet of freeboard

3.7 Coincident Loading and Hydrologic Loading Analysis upstream of Beattyville

The main hydrologic loading analysis utilized for Beattyville is the loading at Kentucky River Lock 14 outlined above in Section 3.6. This analysis accounts for observed flooding downstream of the confluence of the North, Middle and South Forks of the Kentucky River. The PDT considered the potential for looking at coincident loading for the 3 forks of the Kentucky River, however as noted below, determined it was not appropriate based on correlation of the forks to the main stem of the Kentucky River as well as the lack of available data within the system.

Lock 14 is approximately 6 miles downstream of Beattyville and has a similar drainage area to Beattyville (2,657 square miles compared to 2,631 square miles). Flooding at Lock 14, and subsequently Beattyville, has historically been caused by floods that occurred on all 3 forks of the Kentucky River. Peak annual unregulated inflow data for Lock 14 was compared to peak annual unregulated inflow data for the 3 forks for the period or record that was available for each gage between 1921 and 1960. There is a total of 25 years where all gages have peak floods recorded that are summarized below in Figure 3-26 . Sorted by peak flow observed at Lock 14, 17 of the 25 years the observed peak flood on each fork aligned with the peak flow at Lock 14. Based on this data, the PDT believes that for damaging floods to occur at Beattyville, there needs to be significant contribution from all three forks of the Kentucky River.

	Lock 14		North Fork		Middle Fork		South Fork	
WY	Date	Peak Flow	Date	Peak Flow	Date	Peak Flow	Date	Peak Flow
1939	2/4/1939	120,000	2/4/1939	46,800	2/4/1939	37,300	Feb 1939	48,100
1957	1/30/1957	116,000	1/30/1957	53,500	1/30/1957	52,700	1/30/1957	66,100
1929	3/24/1929	113,000	3/24/1929	46,000	3/24/1929	24,400	3/23/1929	37,900
1948	2/15/1948	104,000	2/14/1948	44,000	2/14/1948	20,700	2/14/1948	40,500
1951	2/2/1951	100,000	2/2/1951	45,400	2/2/1951	35,300	2/2/1951	41,000
1946	1/9/1946	90,300	1/8/1946	41,800	1/8/1946	25,700	1/8/1946	42,100
1955	3/1/1955	75,000	2/28/1955	37,500	3/1/1955	14,600	2/22/1955	26,700
1956	2/19/1956	74,200	2/19/1956	34,900	2/19/1956	19,400	2/18/1956	39,600
1952	3/24/1952	70,500	3/23/1952	32,600	3/24/1952	14,300	3/23/1952	31,900
1950	2/2/1950	69,200	2/3/1950	35,800	2/3/1950	15,200	2/2/1950	21,300
1943	3/20/1943	66,200	3/20/1943	26,500	12/31/1942	14,200	3/20/1943	23,900
1958	5/8/1958	63,300	5/7/1958	32,200	5/8/1958	15,400	5/8/1958	23,800
1947	6/29/1947	57,900	8/6/1947	22,000	6/29/1947	17,600	6/29/1947	50,700
1944	4/12/1944	52,400	3/1/1944	21,500	2/19/1944	14,600	4/12/1944	22,300
1945	3/6/1945	51,000	3/6/1945	24,900	2/18/1945	9,610	3/6/1945	19,300
1937	1/26/1937	48,200	1/25/1937	23,700	1/25/1937	8,770	1/25/1937	16,100
1931	4/23/1931	45,500	4/22/1931	21,400	4/23/1931	9,820	4/23/1931	9,540
1940	4/21/1940	39,300	4/20/1940	18,000	4/21/1940	7,910	4/20/1940	13,800
1949	3/19/1949	39,300	3/19/1949	17,900	3/19/1949	11,500	3/19/1949	16,800
1959	1/23/1959	37,700	1/22/1959	18,300	1/23/1959	8,990	1/22/1959	18,900
1941	7/6/1941	31,300	7/5/1941	7,820	7/6/1941	8,750	7/6/1941	16,600
1953	5/7/1953	30,500	5/20/1953	16,900	5/8/1953	8,440	5/7/1953	12,600
1942	7/10/1942	30,200	7/9/1942	27,300	8/10/1942	7,390	8/9/1942	12,400
1960	3/18/1960	28,500	3/17/1960	11,000	3/18/1960	6,100	6/24/1960	24,600
1954	1/17/1954	18,600	1/17/1954	9,050	1/17/1954	5,480	1/16/1954	8,230

Figure 3.26 Summary of Unregulated Peak Floods on All Three Forks of the Kentucky River

Additionally, the three forks showed a strong degree of correlation with Lock 14 as can be seen below in Figure 3-27 to Figure 3-29. Typically, coincident frequency analyses are performed on streams that are considered independent variables. USGS peak annual streamflow data indicates that the three forks are generally caused by the same rainfall events.

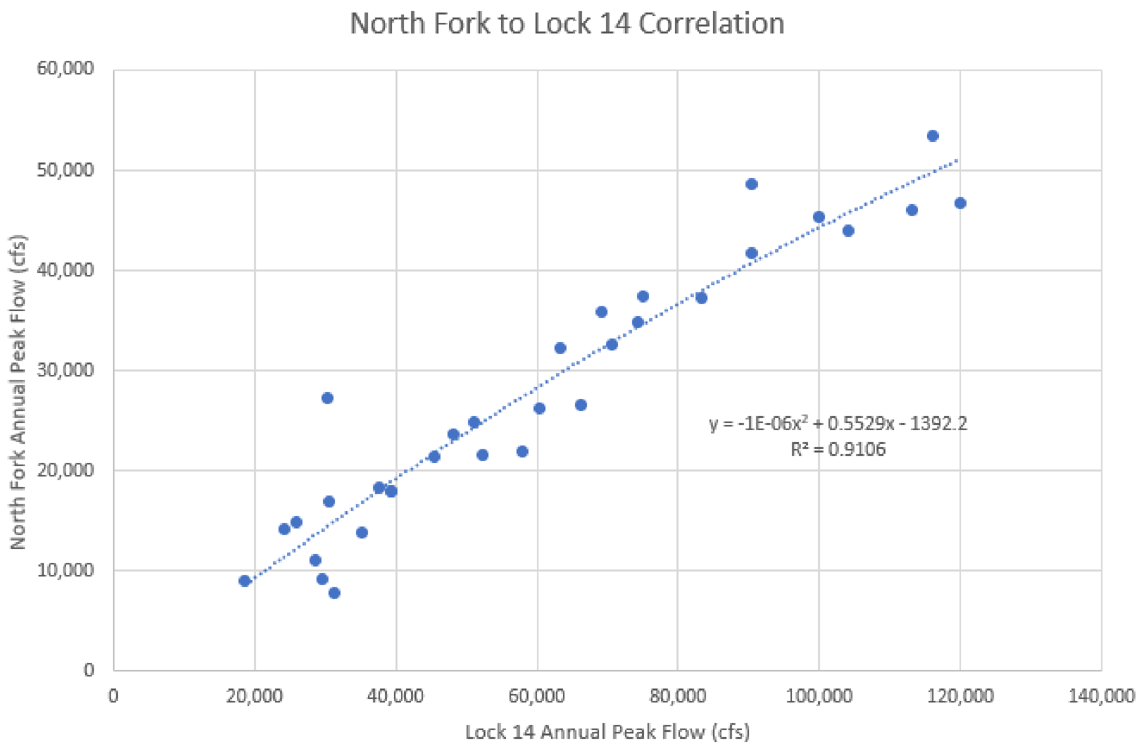

Figure 3.27 North Fork Correlation to Lock 14 for Annual Peak Flows

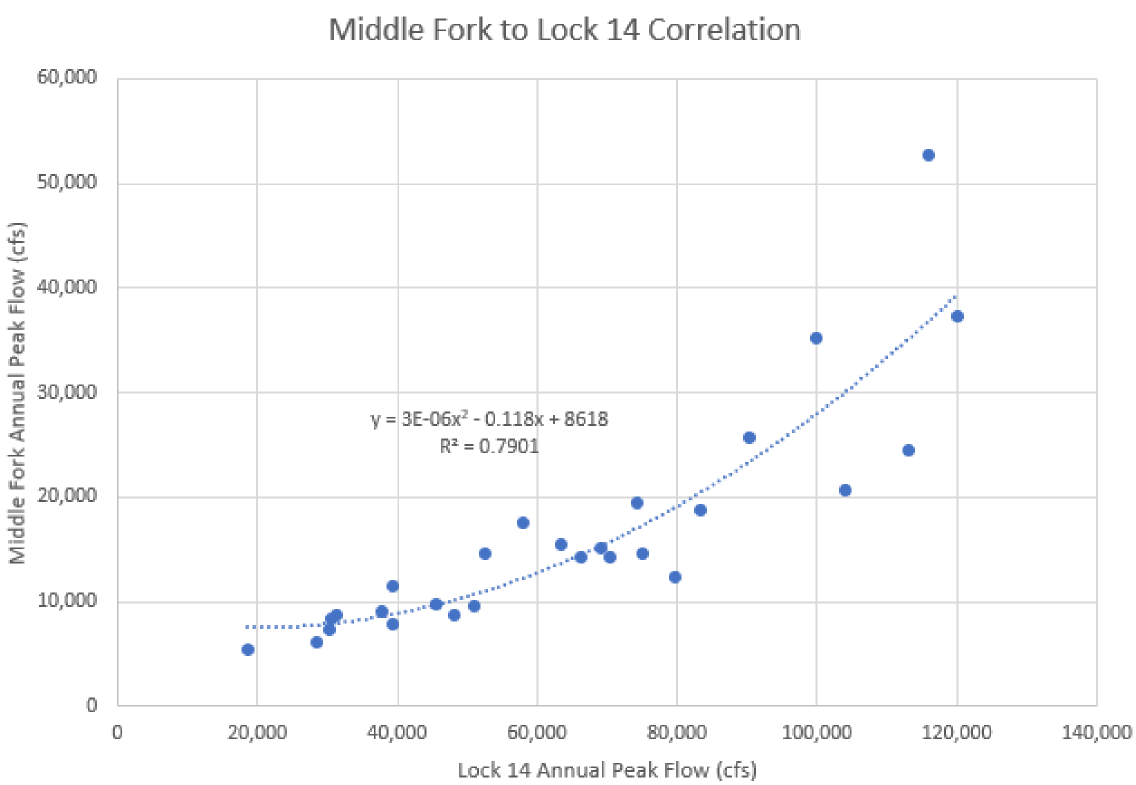

Figure 3.28 Middle Fork Correlation to Lock 14 for Annual Peak Flows

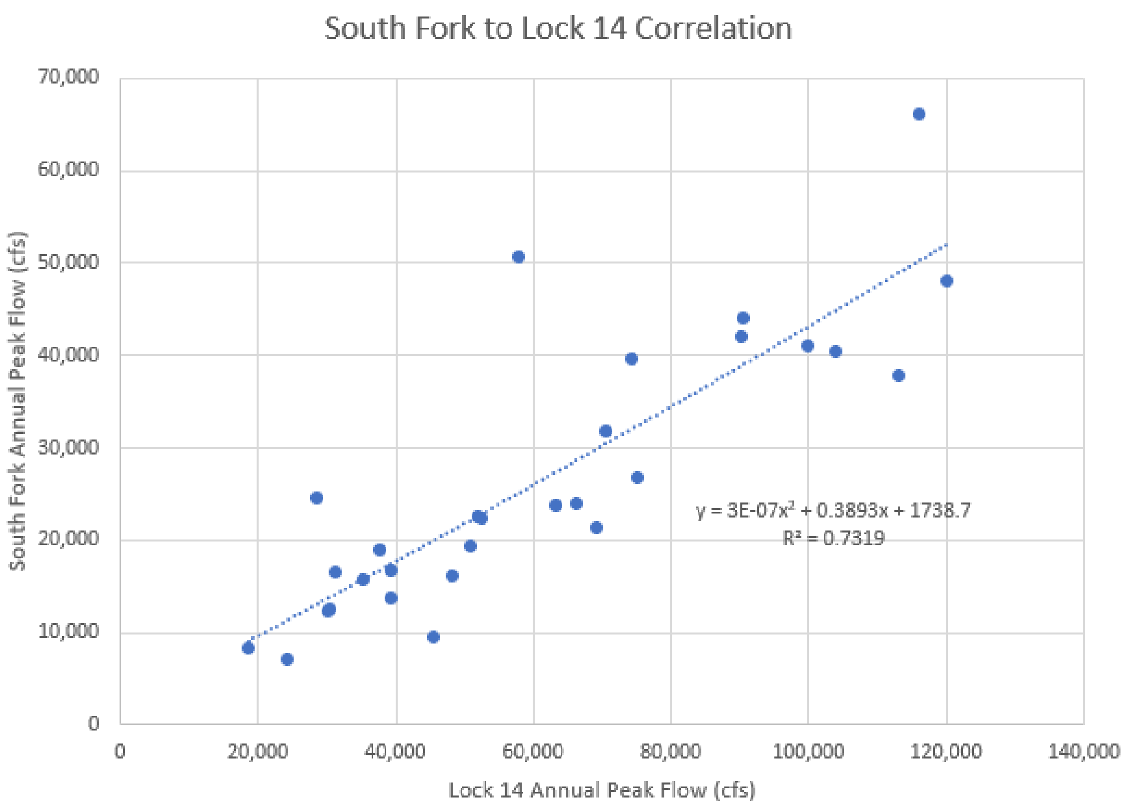

Figure 3.29 South Fork Correlation to Lock 14 for Annual Peak Flows

The PDT also believes that adequate data was not available to properly perform a coincident loading analysis for the 3 forks of the Kentucky River without significant modeling and assumptions. For a simplified application, the dominant variable would need to be the Kentucky River downstream of the confluence with the North and Middle forks. However, there is no gage data available at this location. To perform a coincident loading analysis, period of record modeling would have to be performed combining flows from the North and Middle forks to develop flows at this location, which would then need to be converted to stages so that exceedance stages could be developed. This could also be performed utilizing precipitation data which should be available dating back to 1915.

Lastly, there is significant uncertainty in the rating curve at Lock 14, and subsequently at Beattyville, as noted in Figure 3-5. This uncertainty may be due to the timing of when floods occur on the three forks of the Kentucky River and is included in the final stage frequency curve provided for Beattyville.

Analyses of the North, Middle and South Forks of the Kentucky River is included below. These analyses were completed in case potential flood protection measures were identified on the forks that could have an effect on Beattyville as well as other communities within the region.

Future studies should consider performing a coincident frequency analysis to potentially reduce these sources of uncertainty and to potentially gain a better understanding of the hydraulics related to the Kentucky river, specifically at Beattyville.

3.8 Hydrologic Hazard Curves – North Fork

3.8.1 Overall Systematic, Regional and Historical Data

Systematic Data Overview

Refer to Section 4.2 and 4.4 for general background and summary information on the USGS gages used as part of this analysis. Unregulated peak flow data from several sources was used to calculate the instantaneous peak inflow frequency curve for North Fork. The North Fork USGS gage 03280000 at Jackson was the primary gage used which includes unregulated data in 1905 to 1907, 1917 to 1921, 1927 to 1931, and 1935 to 1975. Regulation in the North Fork began in 1976 with the operation of Carr Creek Lake and regulated peak flow data is available from 1976 to present.

Record Extension – Lock 14 to North Fork

The MOVE.3 function was reviewed to transpose Lock 14 USGS gage 03282000 at Heidelberg to North Fork USGS gage 03280000 at Jackson systematic data. The linear regression slope yielded a negative value of b = -0.466, indicating extension of the data set with the MOVE.3 function is not possible.

In lieu of using the MOVE.3 function to transpose Lock 14 flow data to North Fork, the concurrent annual peak unregulated data from 1921 to 1960 was plotted and a distribution was fit to the data, which is shown in Figure 3-30. A drainage area ratio line, a 1:1 ratio line, and a polynomial trendline were added to the figure for reference. Figure 3-31 shows the same systematic flow data plotted on a log-log basis.

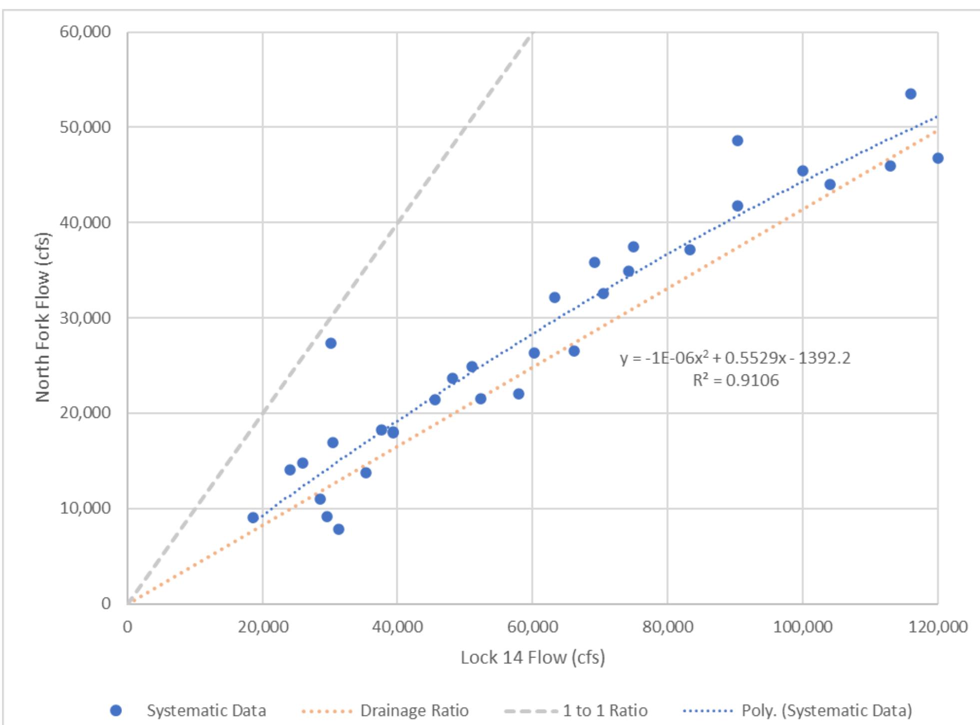

Figure 3.30: North Fork and Lock 14 Unregulated Annual Peak Flow Data Comparison

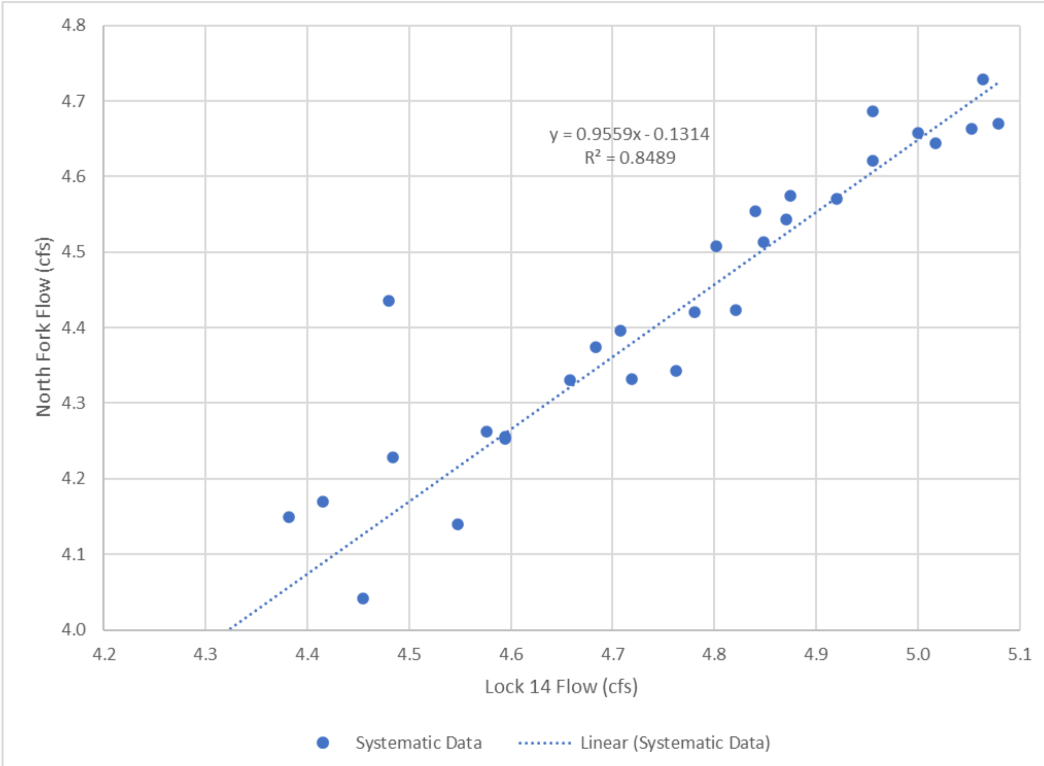

Figure 3.31: North Fork and Lock 14 Unregulated Log-Log Annual Peak Flow Data Comparison

The polynomial trendline of the direct flow data comparisons resulted in an r squared value of 0.9106 which indicates a good correlation. A linear trendline fit to the log-log distribution of the data yielded an r squared value of 0.8489. Since the polynomial trendline of the direct comparisons resulted in a higher r squared value, it was used to transpose Lock 14 USGS gage 03282000 at Heidelberg data to North Fork data for the years 1922 to 1926 and 1932 to 1934.

Record Extension – Lock 4 and Lock 6 to North Fork

Locks 4 (drainage area of 5,411 square miles) USGS gage 03287500 at Frankfort and Lock 6 USGS gage 03287000 at Salvisa, KY (drainage area of 5,102 square miles) flow data were reviewed for potential transposition to North Fork (drainage area of 1,101 square miles) because they have annual flood and gage data dating back to 1895. Lock 4 also has historic gage heights from 1817, 1847, 1854, 1880, and 1883.

Overlapping flow data from Lock 4 and 6 was plotted against North Fork and polynomial trendlines were fit to the data. The same data was also plotted and compared on a log-log scale with linear trendlines fit to the data. Table 19 summarizes the r squared values of the direct and log-log scale data comparisons.

Table 19: Lock 4 and Lock 6 Annual Peak Flow Data Comparison to North Fork

Location Comparisons	***Direct Data Comparison R Squared***	***Log-Log Scale Data Comparison R Squared***
Lock 6 to North Fork	0.0459	0.0261
Lock 4 to North Fork	0.5369	0.6379

All r squared values were below 0.70, indicating there was poor correlation. In addition, drainage area ratios for the North Fork USGS gage 03280000 at Jackson compared to the Lock 4 USGS gage 03287500 (drainage area ratio of 4.91) at Frankfort and Lock 6 USGS gage 03287000 at Salvisa (drainage area ratio of 4.63) exceeds a factor of 2. Therefore, gages at Lock 4 and Lock 6 were not used for record extension at North Fork.

Unregulated-Regulated Transposition

The unregulated POR at North Fork USGS gage 03280000 at Jackson was extended between 1976 to 2022 by transposing flows with an established regulated-unregulated relationship for North Fork. Two data sources were compared to establish the regulated-unregulated relationship. These sources include:

- Spreadsheet model provided by the LRL Water Management section.
- The Carr Creek Lake WCM

The spreadsheet model also includes stage reductions in North Fork USGS gage 03280000 at Jackson based on the regulation of Carr Creek Lake for various events between 2015 to 2022. Stage heights at Jackson from the spreadsheet model were converted to estimated flow values using a rating curve developed based on USGS flow data at the North Fork gage. Figure 3-32 shows the North Fork at Jackson rating curve developed from USGS with systematic data and10 percent upper and lower uncertainty bounds. The USGS published rating curve was used for the purposes of this analysis.

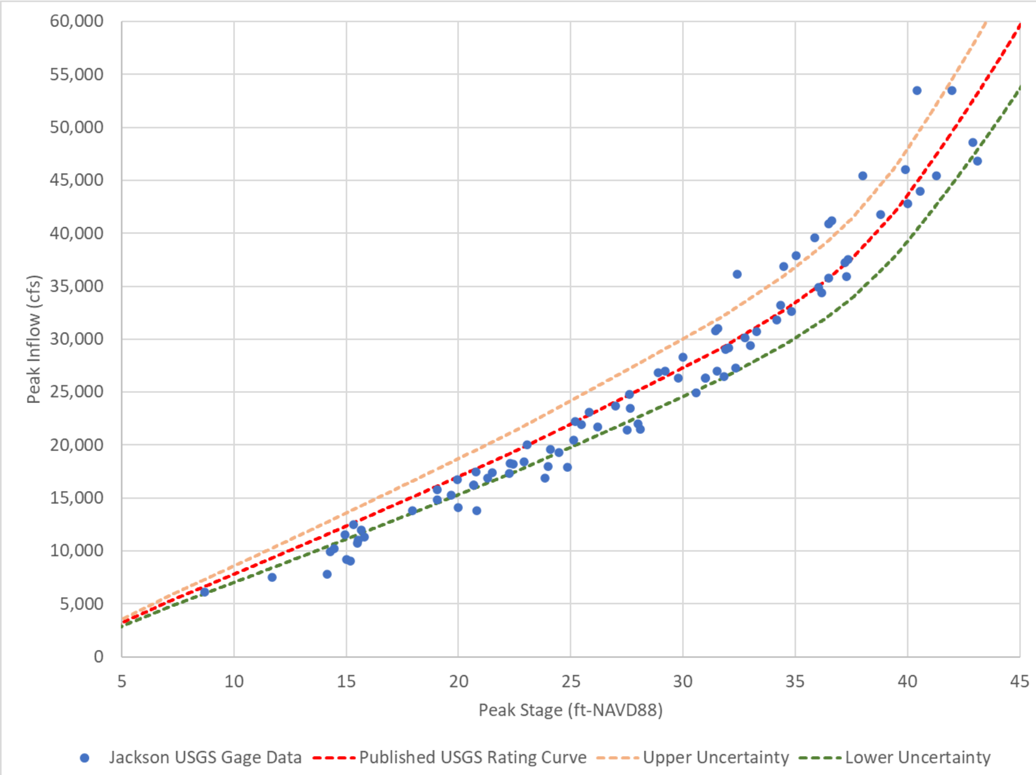

Figure 3.32: North Fork at Jackson USGS Rating Curve with Uncertainty

Table 20 summarizes the events and stage height results from the spreadsheet model for Carr Creek Lake regulation impacts on the North Fork USGS gage 03280000 at Jackson with estimated flows based on the Jackson rating curve.

Table 20: Spreadsheet Model for North Fork Impacts from Carr Creek Lake Regulation

Date	***Unregulated Stage (ft)***	***Regulated Stage (ft)***	***Regulated Reduction (ft)***	***Unregulated Flow (cfs)***	***Regulated Flow (cfs)***
3/5/2015	32.4	29.7	2.7	30,000	27,000
7/15/2015	27.2	24.7	2.5	24,300	21,700
5/1/2016	24.7	22.0	2.7	21,700	18,900
2/12/2018	32.5	29.0	3.5	30,200	26,200
2/21/2019	26.0	23.5	2.5	23,000	20,400
2/25/2019	31.4	28.1	3.3	28,800	25,300
2/8/2020	27.5	25.0	2.5	24,600	22,000
3/2/2021	43.0	39.0	4.0	52,800	41,000

3/29/2021	26.7	24.2	2.5	23,800	21,100
½/2022	26.8	24.4	2.4	23,900	21,400
7/29/2022	46.4	43.5	2.9	64,700	54,500

Plate 16 from the Carr Creek Lake WCM shows a natural and modified flow frequency curve for North Fork, which is shown in Figure 3-33. Plate 16 data was plotted and shown on Figure 3-34 to develop the regulated-unregulated flow relationship.

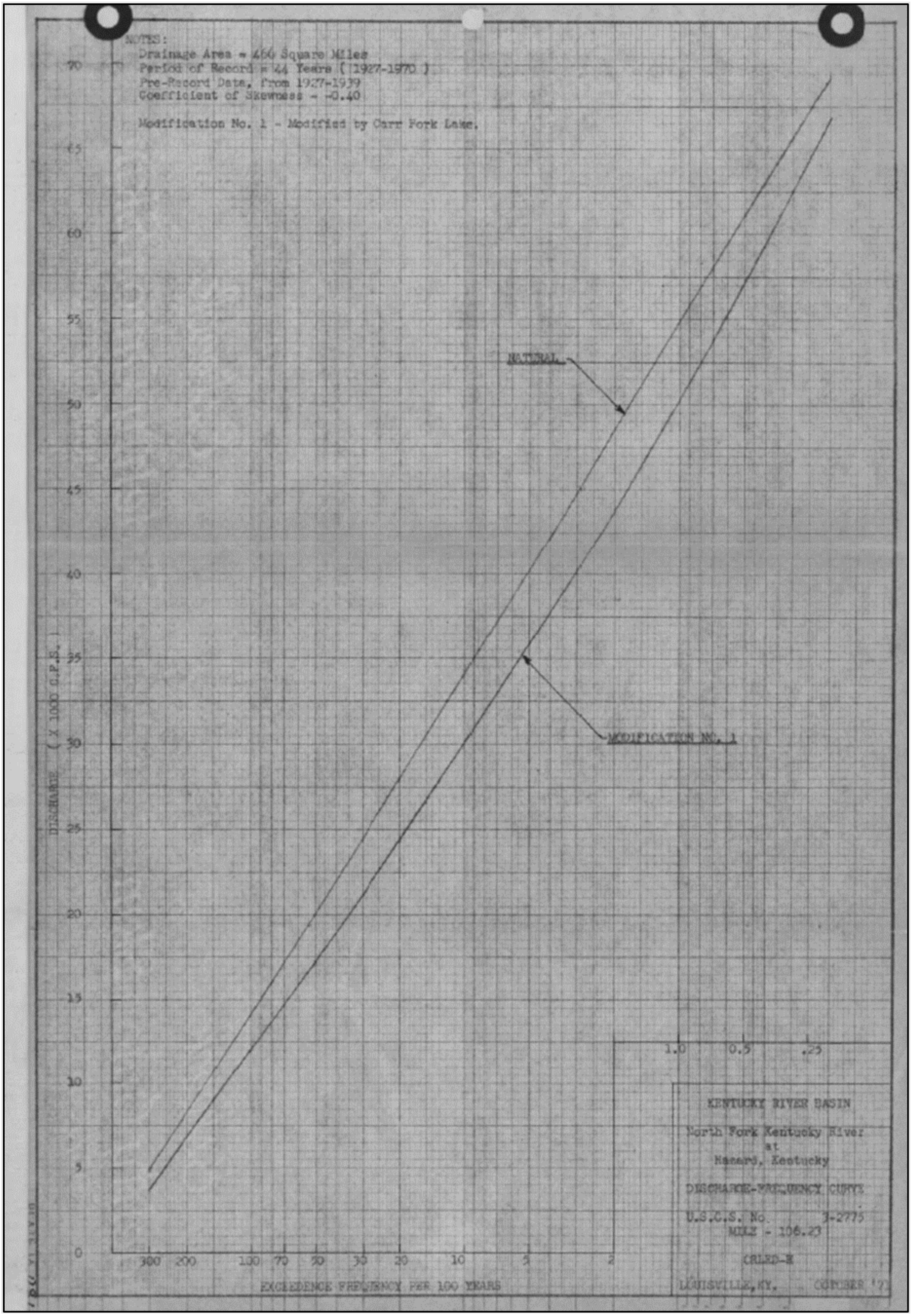

Figure 3.33: Carr Creek Lake WCM Plate 16

The two data sources were plotted overlapped to select the unregulated to regulated flow relationships at North Fork USGS gage 03280000 at Jackson. Figure 3-34 shows the overlapping plot from the two data sources with linear trendlines of each and 5 percent upper and lower uncertainty for the Carr Creek Lake WCM relationship, which was the selected unregulated-regulated relationship for this analysis. Five percent upper uncertainty and 15 percent lower uncertainty were selected for this analysis. A 5 percent upper uncertainty was selected because any higher percent uncertainty results in calculated upper uncertainty values exceeding the 1 to 1 flow ratio at higher flows, which is not possible. Each data source is discussed in further detail below.

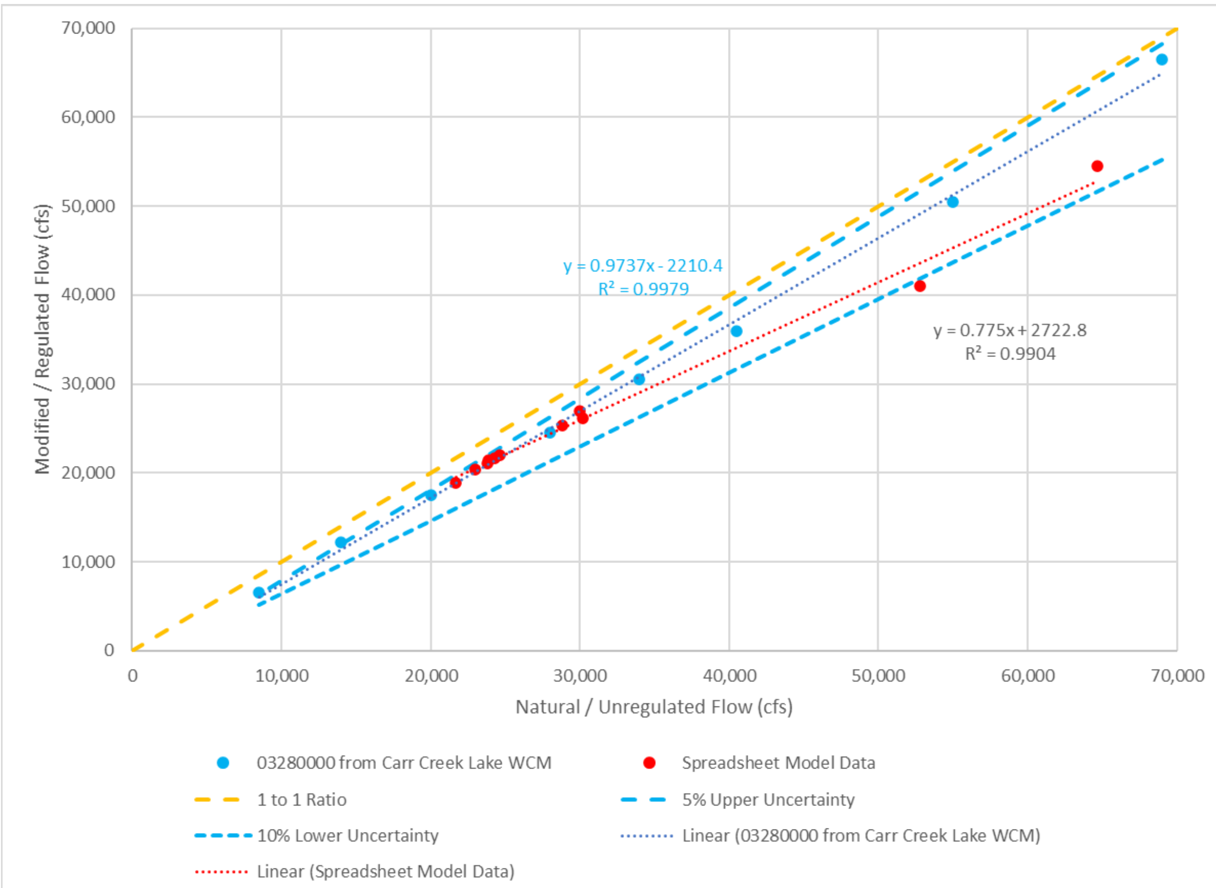

Figure 3.34: North Fork Regulated-Unregulated Relationship

As seen in Figure 3-34, trendlines for each source had good correlation. The Carr Creek Lake WCM relationship was used as the final relationship for North Fork. This was the original estimated relationship when Carr Creek Lake was constructed, and the spreadsheet model plots similarly against it. This indicates it is still a valid relationship for current conditions. Therefore, this relationship was used to convert the regulated flows from 1976 to the present to unregulated values and utilized as part of the unregulated inflow frequency analysis.

Perception Thresholds

As previously noted, annual peak flow data at Lock 4 USGS gage 03287500 and Lock 6 USGS gage 03287000 at Salvisa were reviewed for record extension at the North Fork USGS gage 03280000 at Jackson. A comparison of the flow data indicated poor correlations and drainage area ratios that exceed a factor of 3. Therefore, the data at the Lock 4 and Lock 6 USGS gages was not used. Missing annual peak inflow data in the POR between 1908 and 1916 was assigned a PT of 53,500 cfs, which is the unregulated flood of record.

Data Summary

To summarize, the following information was used to develop the final unregulated POR data used for Lock 14.

- A PT of 53,500 cfs was used between 1908 and 1916 to supplement data gaps in the unregulated POR. The PT was based on the flood of record.
- Unregulated Lock 14 USGS gage 03287000 at Heidelberg flow data between 1922 to 1926 and 1932 to 1934 was transposed to North Fork USGS gage 03280000 at Jackson based on good correlations of the annual peak flows.
- Unregulated flows from North Fork USGS gage 03280000 at Jackson from 1905 to 1907, 1917 to 1921, 1927 to 1931, and 1935 to 1975 were directly used.
- The Carr Creek Lake WCM unregulated to regulated flow relationship was used to transpose North Fork USGS gage 03280000 at Jackson regulated flows to unregulated flows between 1975 to 2016.

Figure 3-35 shows a visual of the final composite North Fork POR used in the assessment.

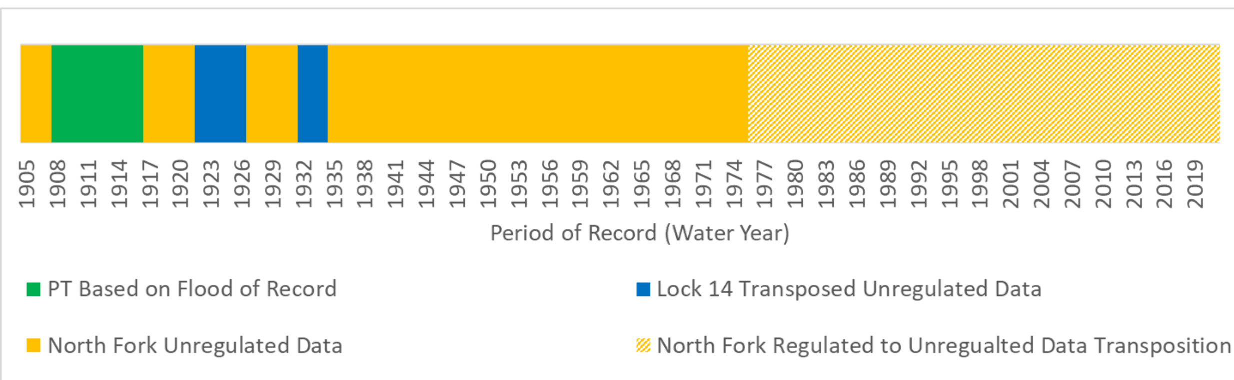

Figure 3.35: North Fork Final POR Visualization

3.8.2 Unregulated Flow-Frequency Curve

The composite unregulated annual peak flow POR was used to calculate the peak flow frequency curve using a B17C analysis in HEC-SSP. The final POR included systematic and transposed data from 1905 to 2022 as previously discussed. Figure 3-36 shows the HEC-SSP EMA Data for the unregulated POR for North Fork.

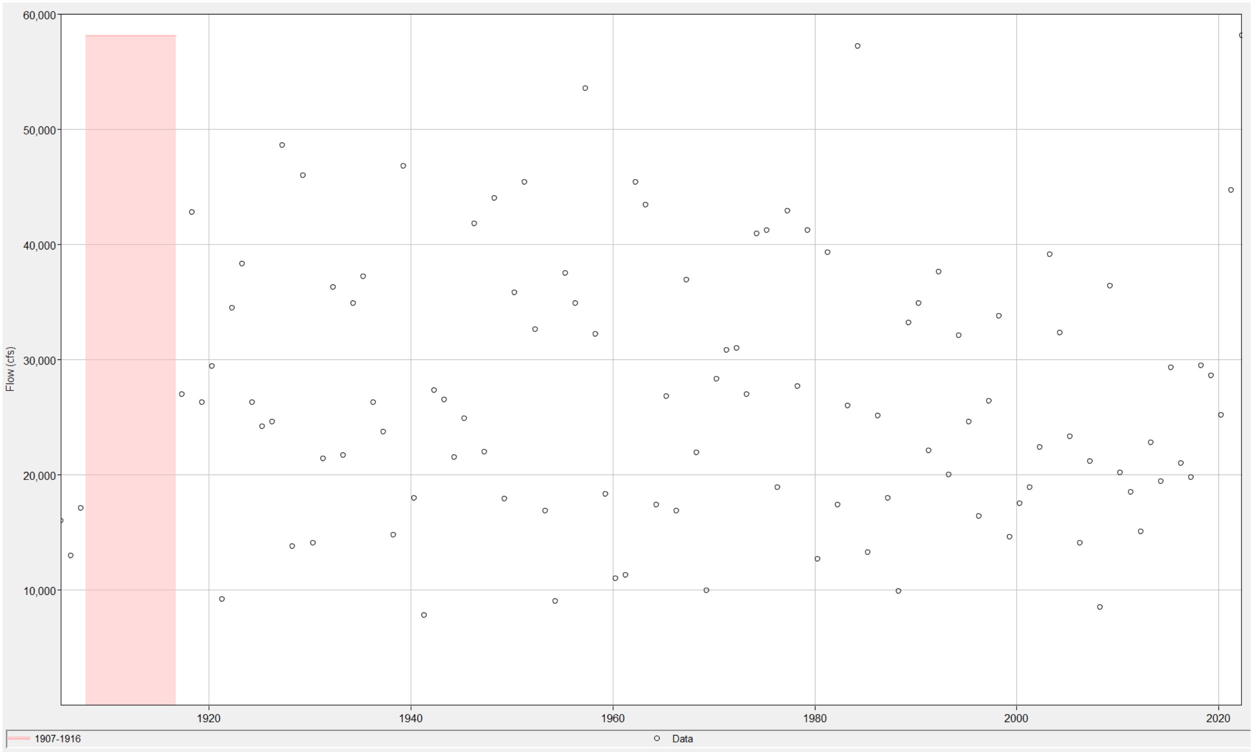

Figure 3.36: North Fork Unregulated POR HEC-SSP EMA Data

Regional Skew Information

Regional skew information was available from an existing study published by the USGS in cooperation with the Kentucky Transportation Cabinet. The study is titled "*Estimating the Magnitude of Peak Flows for Streams in Kentucky for Selected Recurrence Intervals*". The regional skew values for peak flows for the state of Kentucky is 0.011 with a standard error and mean square error of 0.52 and 0.27, respectively. The regional skew was applied to the B17C analysis, and the weighted skew was used as the final adopted skew.

Potentially Influential Low Outliers

The Multiple Grubbs-Beck Test was performed on the data to determine if the POR contained any PILF values. PILF's are reviewed to confirm that small observed inflows do not have an inappropriately large impact on the inflow frequency analysis. The test identified no PILF values.

Unregulated Flow Frequency Sensitivities

To determine how sensitive the analysis is to the inclusion of additional data sources, multiple sensitivities were modeled with various data sets. Flow-frequency curves for the following data sets were developed using a B17C analysis in HEC-SSP:

- North Fork Unregulated Only – includes unmodified unregulated flow data from North Fork from 1905 to 1907, 1917 to 1921, 1927 to 1931, and 1935 to 1975.
- North Fork Transposed Unregulated – includes unmodified unregulated flow data from North Fork from 1905 to 1907, 1917 to 1921, and 1927 to 1975 and transposed regulated to unregulated flow data from 1976 to 2022.

- Final POR – includes the final period of record as previously discussed.

Figure 3-37 show a plot of the curves for the above data sets.

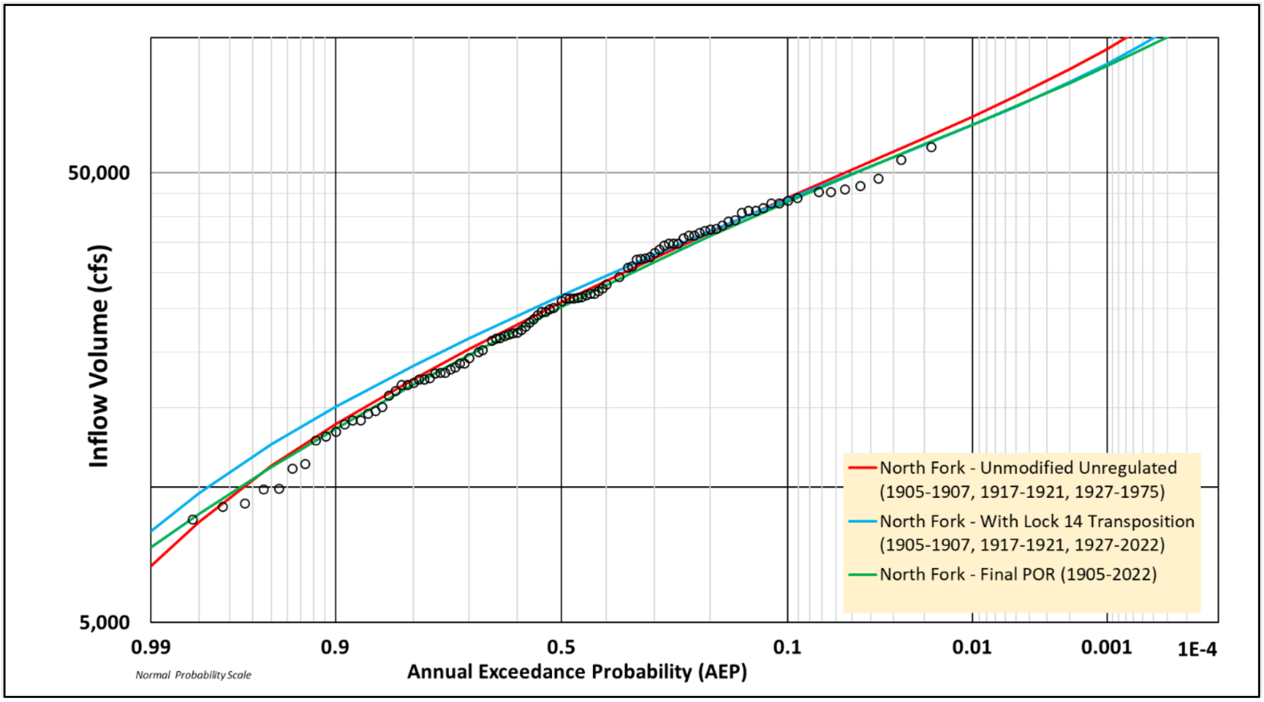

Figure 3.37: North Fork Unregulated Inflow Frequency Sensitivities

Table 21summarizes the mean, standard deviation, and skew of the sensitivities plotted in Figure 3-37, which were similar between the sensitivities reviewed.

Table 21: North Fork at Jackson Statistical Summary of Bulletin 17C Sensitivities

Data Set Description	***Mean (of log) (μ)***	***Std. Dev. (of log) (σ)***	***Skew (of log) (γ)***
North Fork Unmodified Unregulated (1905-1907, 1917-1921, 1927-1975)	4.408	0.186	-0.237
North Fork with Unregulated Lock 14 Transpositions (1905-1907, 1917-1921, 1927-2022)	4.419	0.178	-0.397
North Fork Final POR (1905-2022)	4.391	0.196	-0.337

Figure 3-38 shows the final unregulated flow-frequency curve for North Fork using the B17C analysis in HEC-SSP.

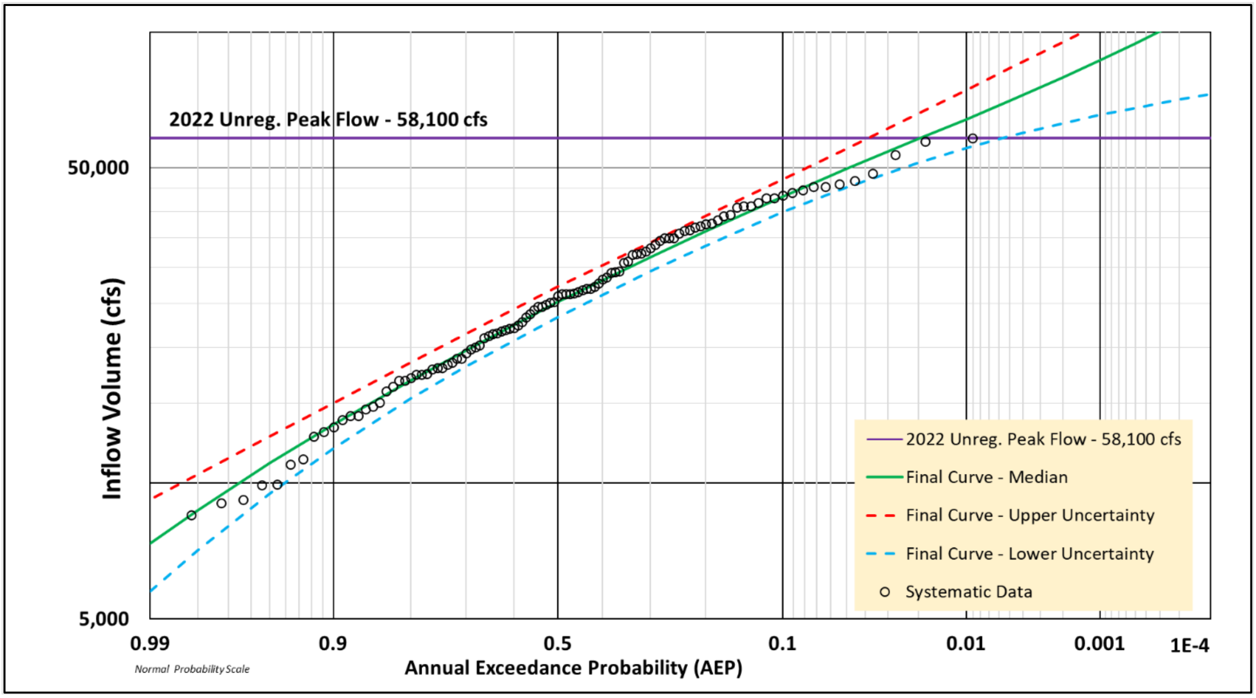

Figure 3.38: North Fork Final Unregulated Flow Frequency Curve

3.8.3 Regulated Flow-Frequency Curve

The previously established regulated-unregulated relationship shown in Figure 3-11 was used to develop the regulated flow frequency curve from the final unregulated flow frequency curve. A sensitivity analysis was performed transposing the unregulated flow frequency curve to a regulated flow frequency curve by directly transposing the data and by using the upper and lower uncertainty bounds. The comparison of the two transpositions is shown in Figure 3-39.

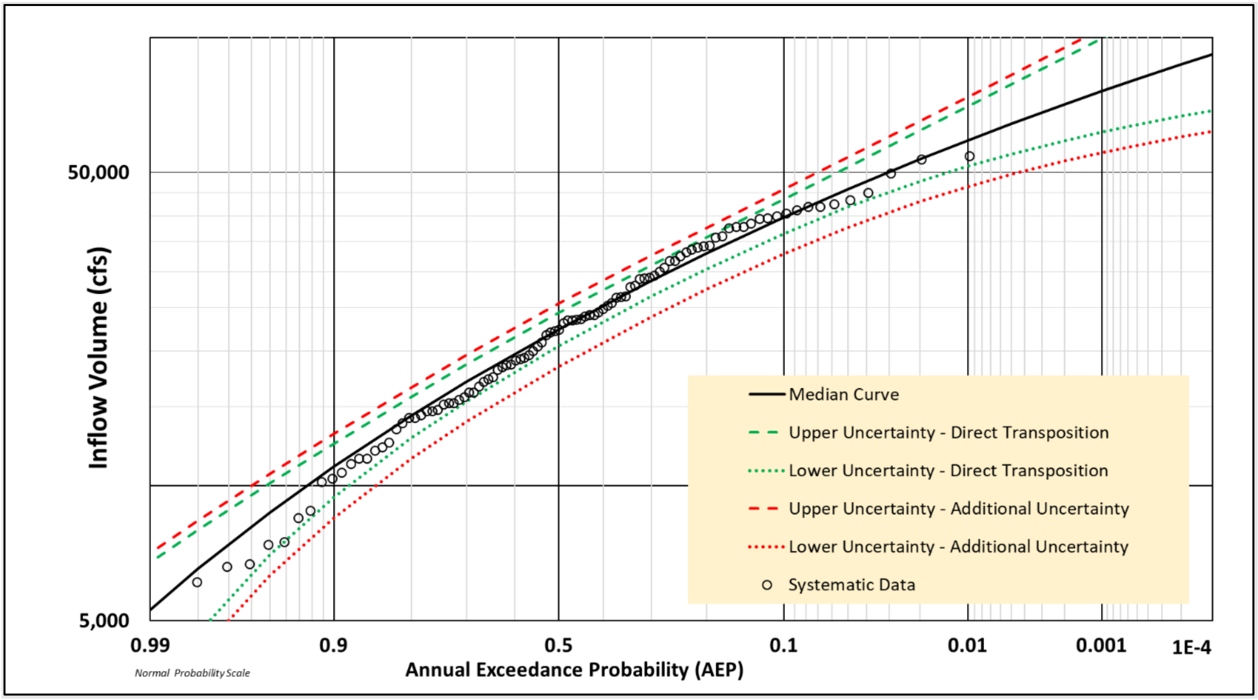

Figure 3.39: North Fork Transposed Regulated Flow Frequency Comparison

To incorporate the uncertainty associated with the regulated-unregulated transposition, the transposition using the upper and lower uncertainty bounds was selected. Figure 3-40 shows the final regulated flow frequency curve for North Fork.

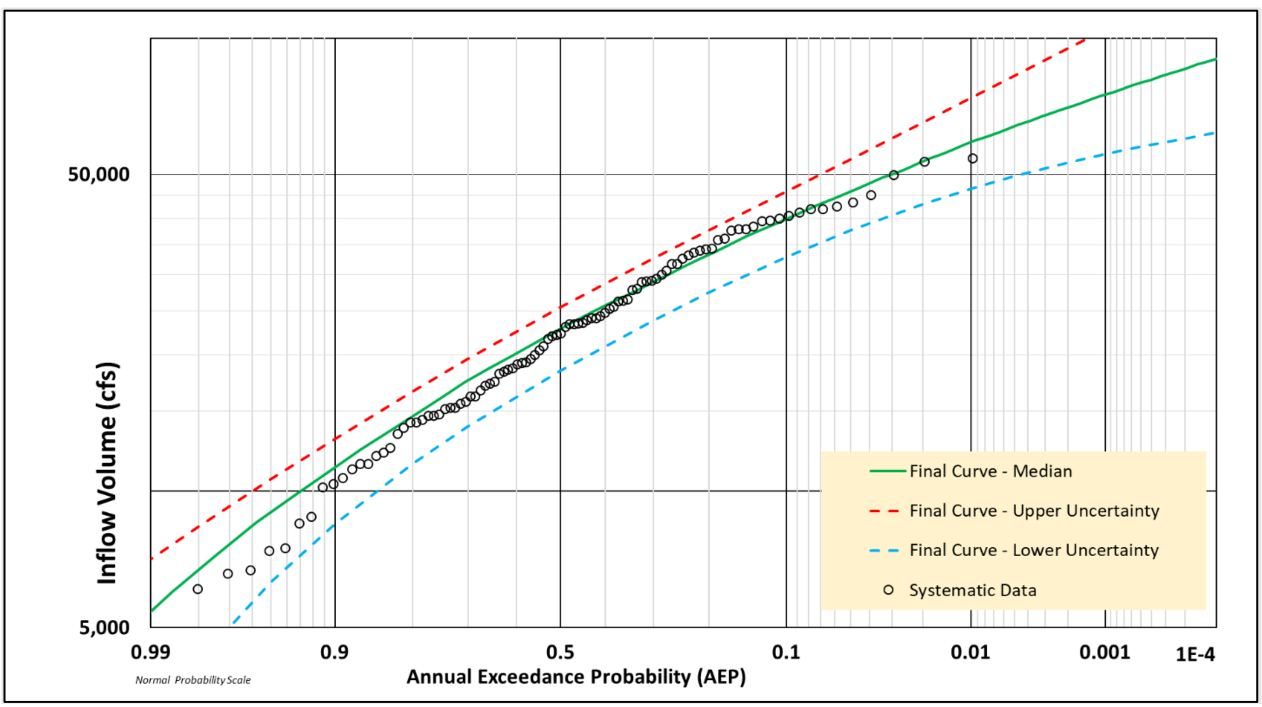

Figure 3.40: North Fork Final Regulated Flow Frequency Curve

The final unregulated and regulated flow-frequency curves are plotted in Figure 3-41. The unregulated and regulated systematic data points are shown for reference.

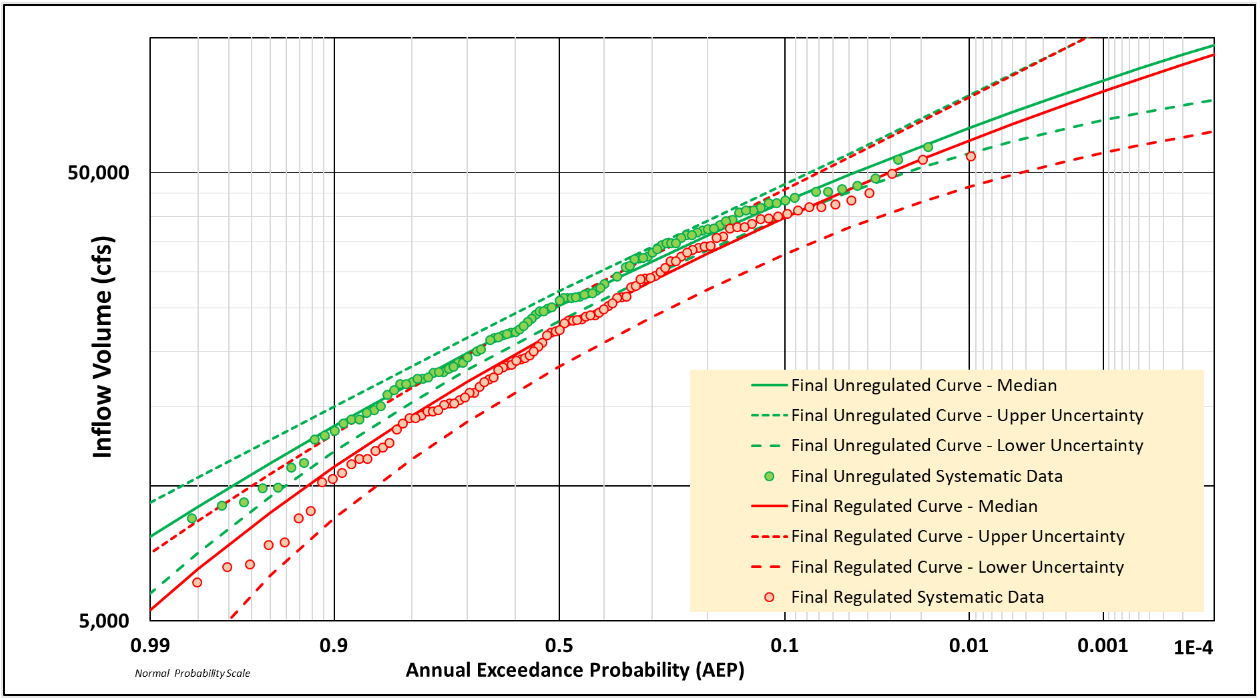

Figure 3.41: North Fork Regulated and Unregulated Flow Frequency Curve Comparison

3.8.4 Regulated Stage-Frequency Curves

The stage frequency curve for North Fork was developed using the same process discussed in Section 4.6.4 using the additional uncertainty in the regulated-unregulated relationship and a direct transposition using the published North Fork USGS rating curve shown in Figure 3-32. Figure 3-42 shows the final regulated stage frequency curve for the North Fork.

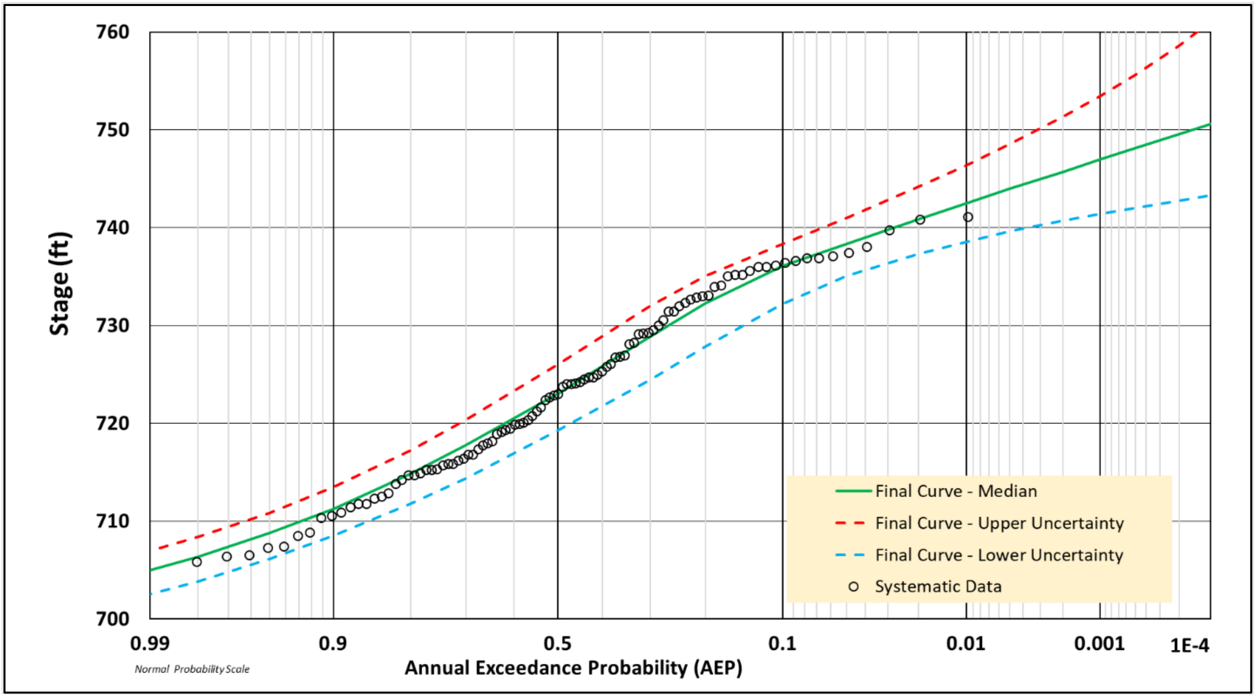

Figure 3.42: North Fork at Jackson Final Stage Frequency Curve

3.8.5 Hydrologic Hazard Curve Summary Tables

Table 22 and Table 23 summarize the final regulated flow and stage frequency curves for North Fork at Jackson for various AEPs, respectively.

Table 22: North Fork at Jackson Final Regulated Flow Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	94,802	53,038	74,963	71,017
0.01	100	73,681	46,475	60,191	58,988
0.04	25	55,875	38,572	47,505	47,169
0.1	10	45,831	32,921	39,854	39,735
0.2	5	37,631	27,447	33,031	33,019
0.5	2	25,520	18,479	22,305	22,361
0.99	1	7,081	3,052	4,945	5,292

Table 23: North Fork at Jackson Final Regulated Stage Frequency Curve Summary Table

AEP	*Return Period (Years)*	*95% Confidence Interval (ft)*	*5% Confidence Interval (ft)*	*Expected [Mean] Curve (ft)*	*Computed [Median] Curve (ft)*
0.002	500	751.30	740.70	746.72	745.71
0.01	100	746.39	738.54	742.81	742.46
0.04	25	741.53	735.45	738.87	738.75
0.1	10	738.30	732.21	736.12	736.08
0.2	5	735.05	727.82	732.27	732.26
0.5	2	726.03	719.24	722.99	723.04
0.99	1	706.97	702.50	704.63	705.01

3.9 Hydrologic Hazard Curves – Middle Fork

3.9.1 Overall Systematic, Regional and Historical Data

Systematic Data Overview

Refer to Section 4.2 and 4.4 for general background and summary information on the USGS gages used as part of this analysis. Unregulated peak flow data from several sources was used to calculate the instantaneous peak flow frequency curve for Middle Fork. The Middle Fork USGS gage 03281000 at Tallega was the primary gage used which includes unregulated data in 1929, 1931, 1932, 1935, 1937, and 1939 to 1960. South Fork USGS gage 03281500 at Booneville was transposed to supplement unregulated data in 1926 and North Fork USGS gage 03280000 at Jackson was transposed to supplement unregulated data in 1927 to 1928, 1930, 1936, and 1938.

Regulated data was reviewed separately based on not being able to establish a consistent regulated-unregulated flow relationship. This is discussed in further detail in Section 4.8.3.

Record Extension – North Fork to Middle Fork

The overlapping unregulated data from 1917 to 1960 for North Fork USGS gage 03280000 at Jackson and Middle Fork USGS gage 03281000 at Tallega were plotted and a distribution was fit to the data, which is shown in Figure 3-43. A drainage area ratio line, a 1:1 ratio line, and a polynomial trendline were added to Figure 3-43 for reference. Figure 3-44 shows the same information in Figure 3-43 on a log-log basis.

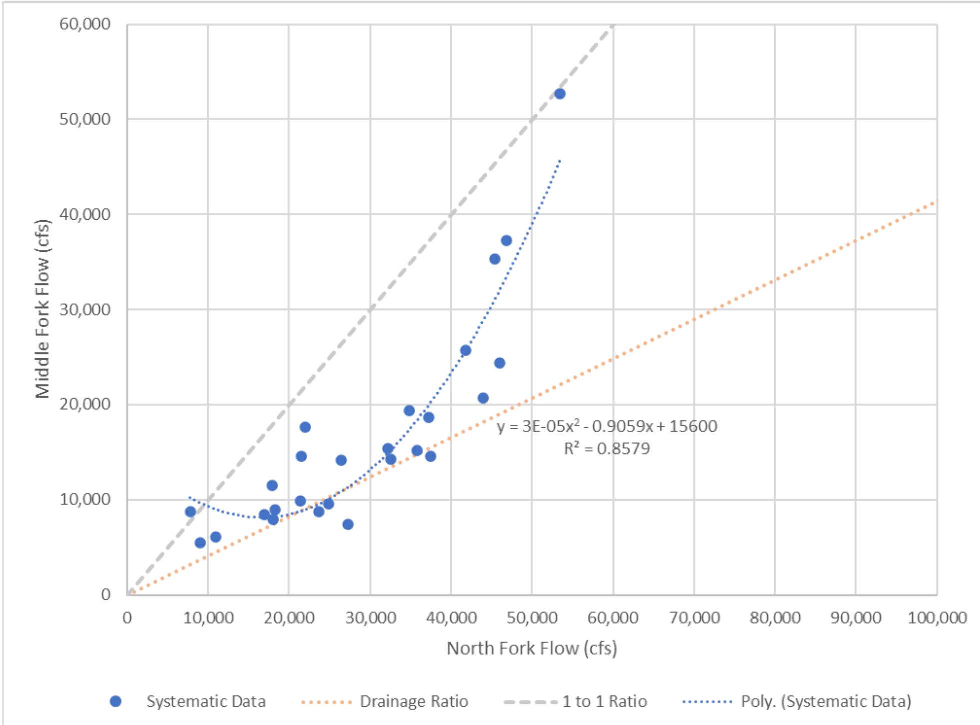

Figure 3.43: North Fork Regulated and Middle Fork Unregulated Data Comparison

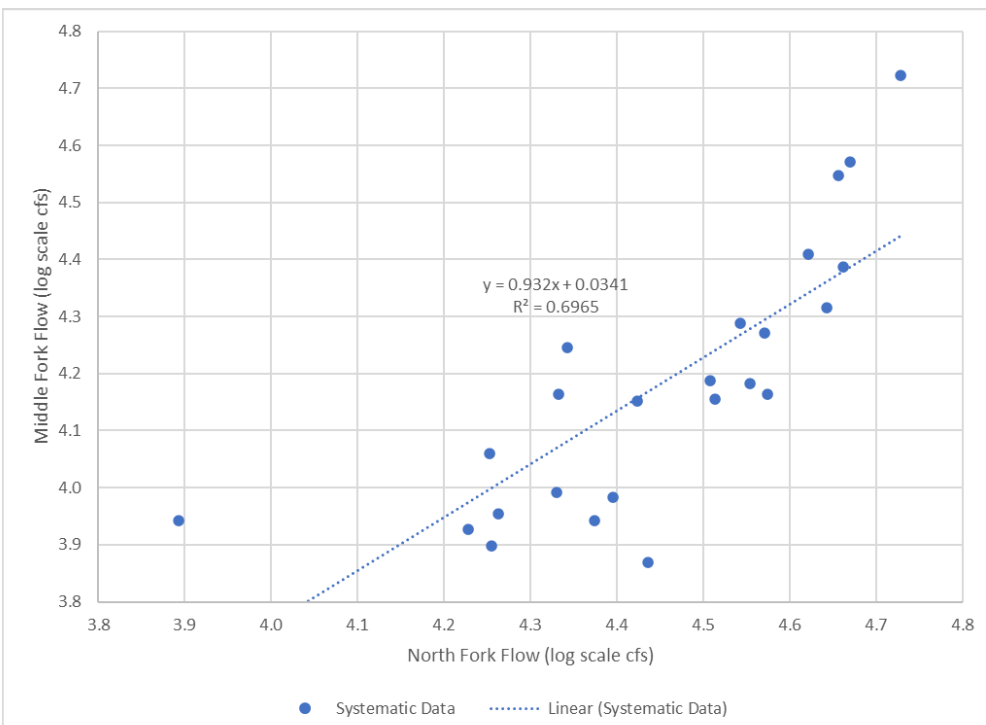

Figure 3.44: North Fork Regulated and Middle Fork Unregulated Data Comparison (Log-Log Basis)

As shown in Figure 3-43, the polynomial trendline resulted in an r squared value of 0.8579 which indicates a good correlation. The linear trendline fit to a log-log distribution of the above data yielded an r squared value of 0.6965. The drainage area ratio between the two gages is 2.05, which is a reasonable ratio for data transposition. Since the polynomial trendline resulted in a higher r squared value, it was used to transpose North Fork USGS gage 03280000 at Jackson to Middle Fork USGS gage 03281000 at Tallega.

Record Extension – South Fork to Middle Fork

The overlapping unregulated data from 1921 to 1960 for South Fork and Middle Fork gages were plotted and a distribution was fit to the data, which is shown in Figure 3-45. A drainage area ratio line, a 1:1 ratio line, and a polynomial trendline were added to Figure 3-45 or reference. Figure 3-46 shows the same information in Figure 3-45 on a log-log basis.

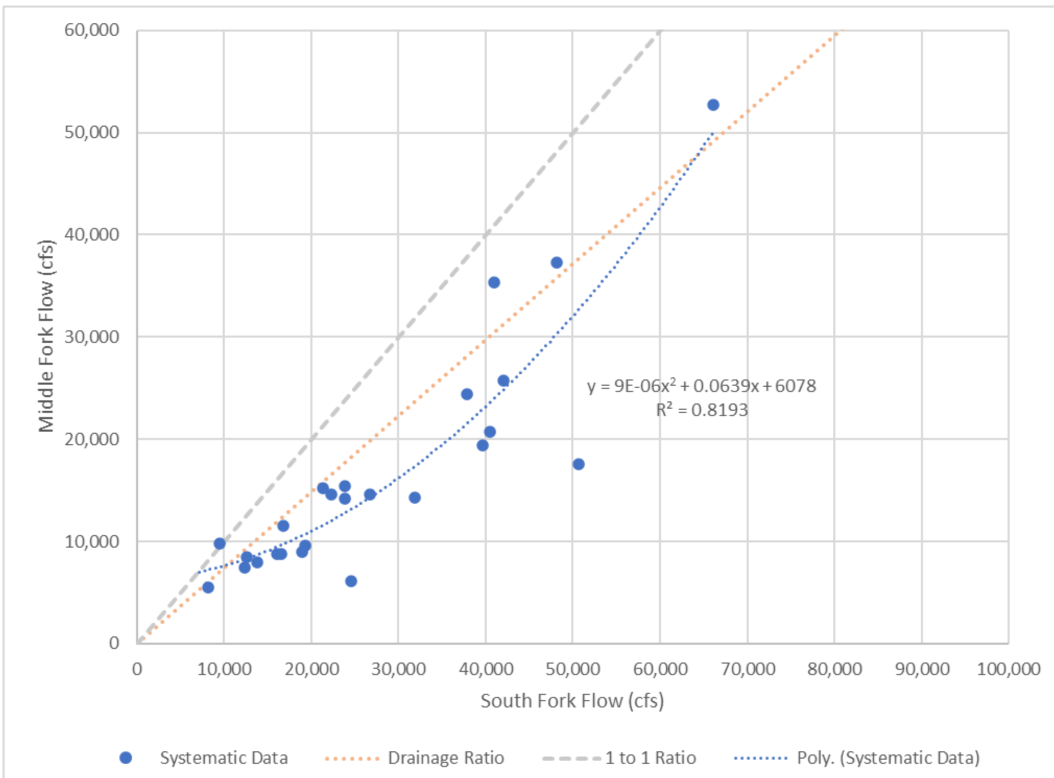

Figure 3.45: South Fork Regulated and Middle Fork Unregulated Data Comparison

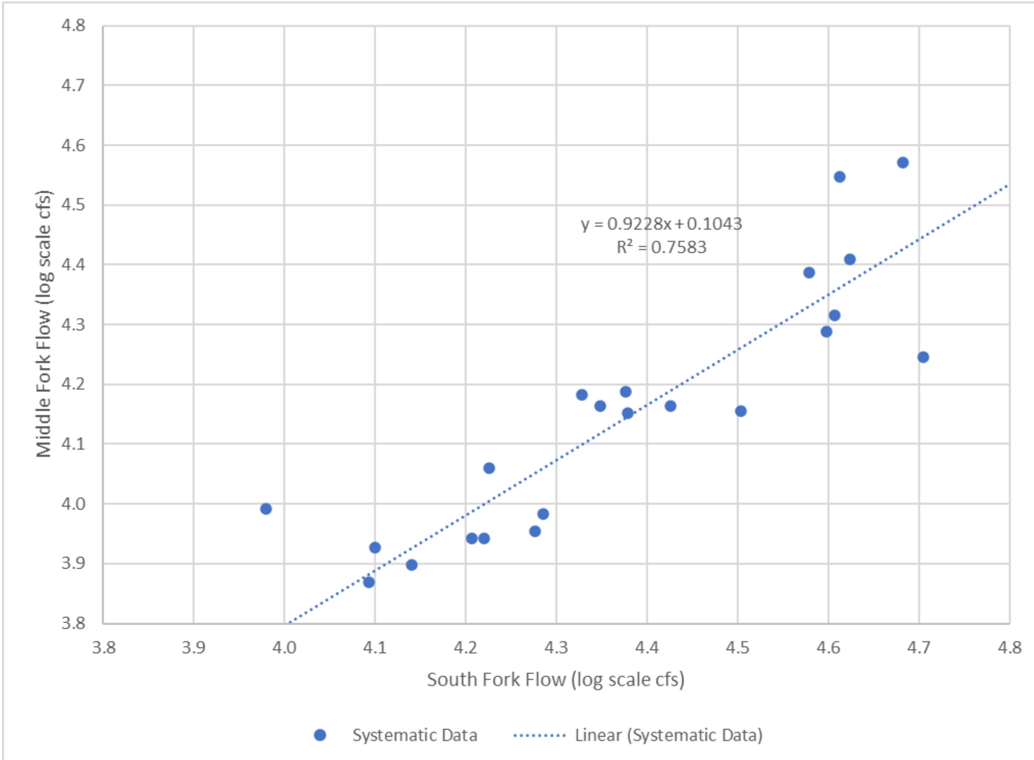

Figure 3.46: South Fork Regulated and Middle Fork Unregulated Data Comparison (Log-Log Basis)

As shown in Figure 3-45, the polynomial trendline resulted in an r squared value of 0.8193 which indicates a good correlation. The linear trendline fit to a log-log distribution of the above data yielded an r squared value of 0.7583. The drainage area ratio between the two gages is 1.34, which is a reasonable ratio for data transposition. Since the polynomial trendline resulted in a higher r squared value, it was used to transpose South Fork USGS gage 03281500 at Booneville to Middle Fork USGS gage 03281000 at Tallega.

Perception Thresholds

Missing annual peak inflow data in the POR between 1922 and 1925 was assigned a PT of 52,700 cfs, which is the unregulated flood of record at the Middle Fork USGS gage 03281000 at Tallega.

Data Summary

To summarize, the following information was used to develop the final unregulated POR data used for Middle Fork:

- A PT of 52,700 cfs was used between 1922 and 1925 to supplement data gaps in the unregulated POR. The PT was based on the flood of record.
- Unregulated North Fork flow data between 1917 to 1921, 1927 to 1928, 1930, 1936, and 1938 was transposed to Middle Fork based on good correlations of the annual peak flows.
- Unregulated South Fork flow data from 1926 was transposed to Middle Fork based on good correlations of the annual peak flows.

- Available unregulated Middle Fork flow data was used from 1929 to 1960.

Figure 3-47 shows a visual of the final composite unregulated POR for Middle Fork used in the assessment.

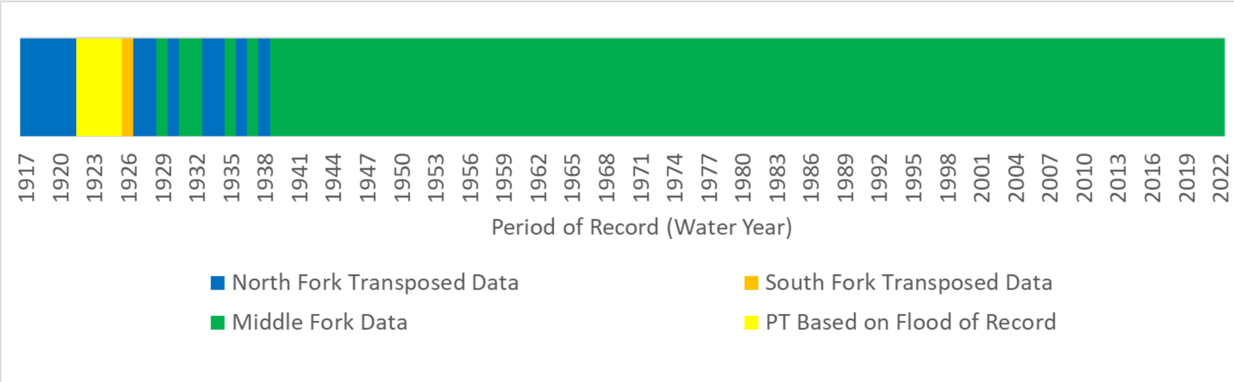

Figure 3.47: Middle Fork Final Unregulated POR Visualization

3.9.2 Unregulated Flow-Frequency Curve

The composite unregulated peak flow POR was used to calculate the peak flow frequency curve using a B17C analysis in HEC-SSP. Figure 3-48 shows the HEC-SSP EMA Data for the unregulated POR.

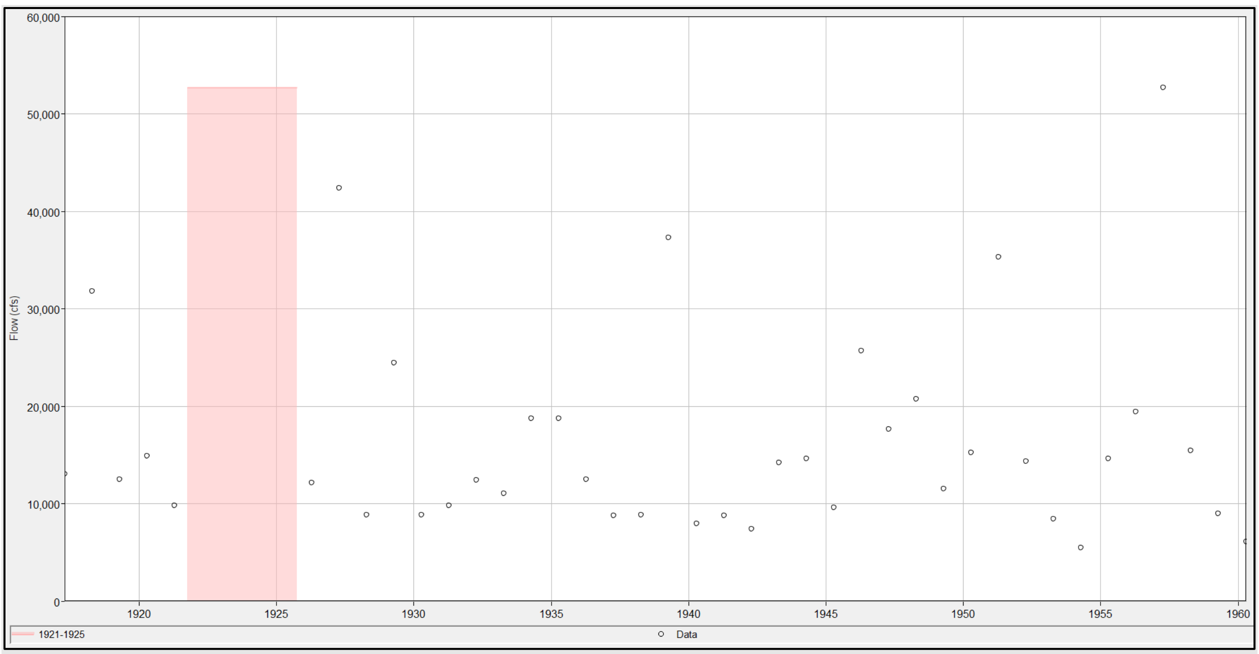

Figure 3.48: Middle Fork Final Unregulated POR HEC-SSP EMA Data

Regional Skew Information

Regional skew information was available from an existing study published by the USGS in cooperation with the Kentucky Transportation Cabinet. The study is titled “Estimating the Magnitude of Peak Flows for Streams in Kentucky for Selected Recurrence Intervals”. The regional skew values for peak flows for the state of Kentucky is 0.011 with a standard error and mean square error of 0.52 and 0.27, respectively. The regional skew was applied to the B17C analysis, and the weighted skew was used as the final adopted skew.

Potentially Influential Low Outliers

The Multiple Grubbs-Beck Test was performed on the data to determine if the POR contained any PILF values. PILF's are reviewed to confirm that small observed inflows do not have an inappropriately large impact on the inflow frequency analysis. The test identified no PILF values.

Unregulated Flow Frequency Sensitivities

To determine how sensitive the analysis is to the inclusion of additional data sources, flow-frequency curves for the following data sets were developed and compared using a B17C analysis in HEC-SSP:

- Middle Fork unmodified unregulated – includes unmodified unregulated flow data from Middle Fork between 1929 to 1960
- Middle Fork unregulated POR – includes the final period of record as previously discussed.

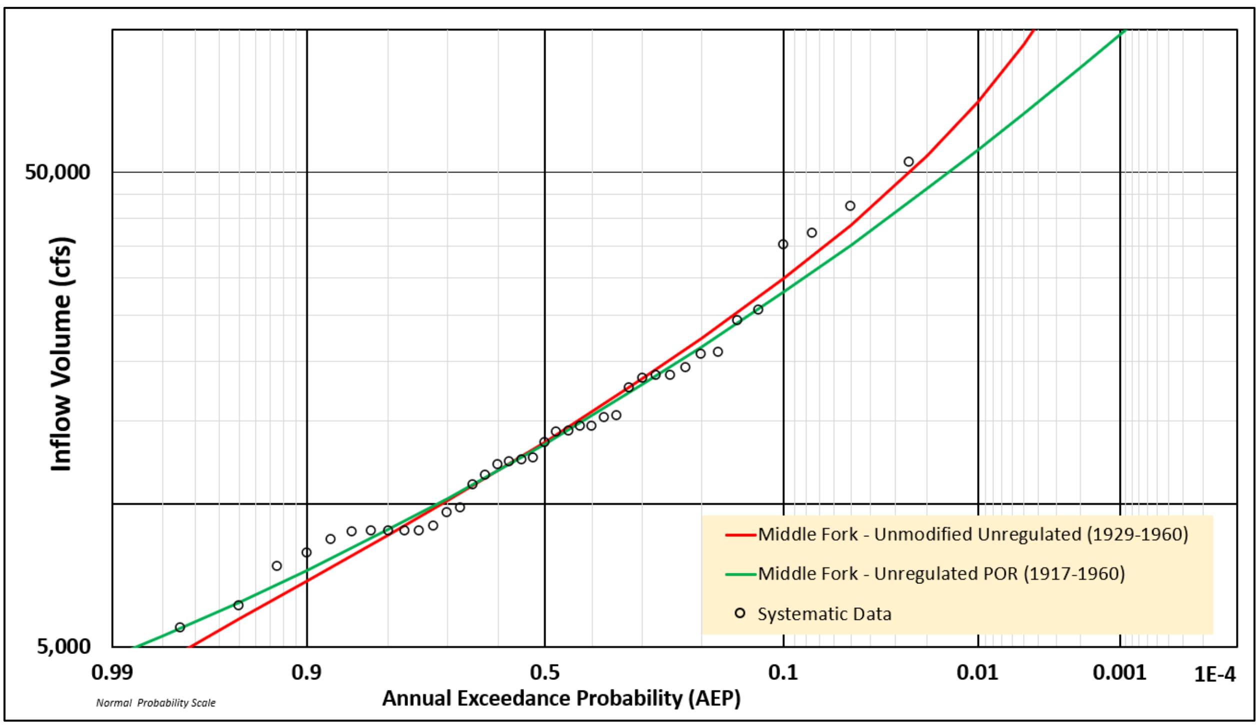

Figure 3-49 show a plot of the curves for the above data sets along with the POR systematic data.

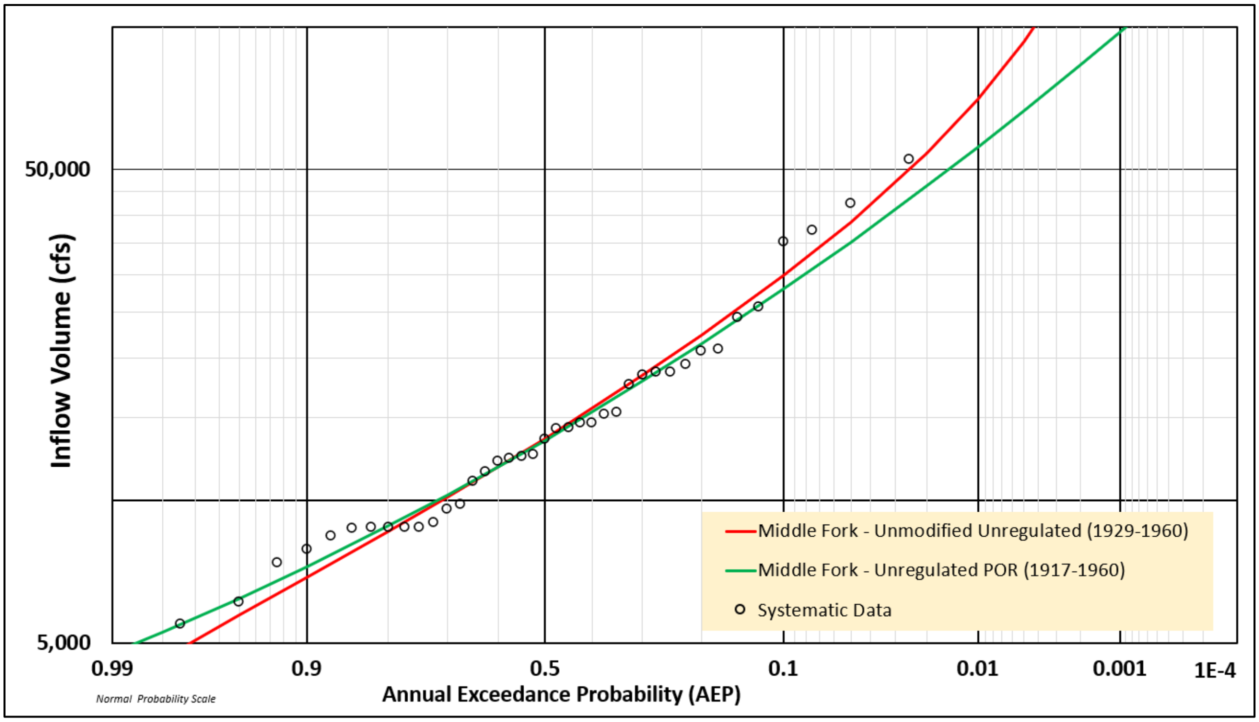

Figure 3.49: Middle Fork Unregulated Inflow Frequency Sensitivities

Table 24 summarizes the mean, standard deviation, and skew of the sensitivities plotted in

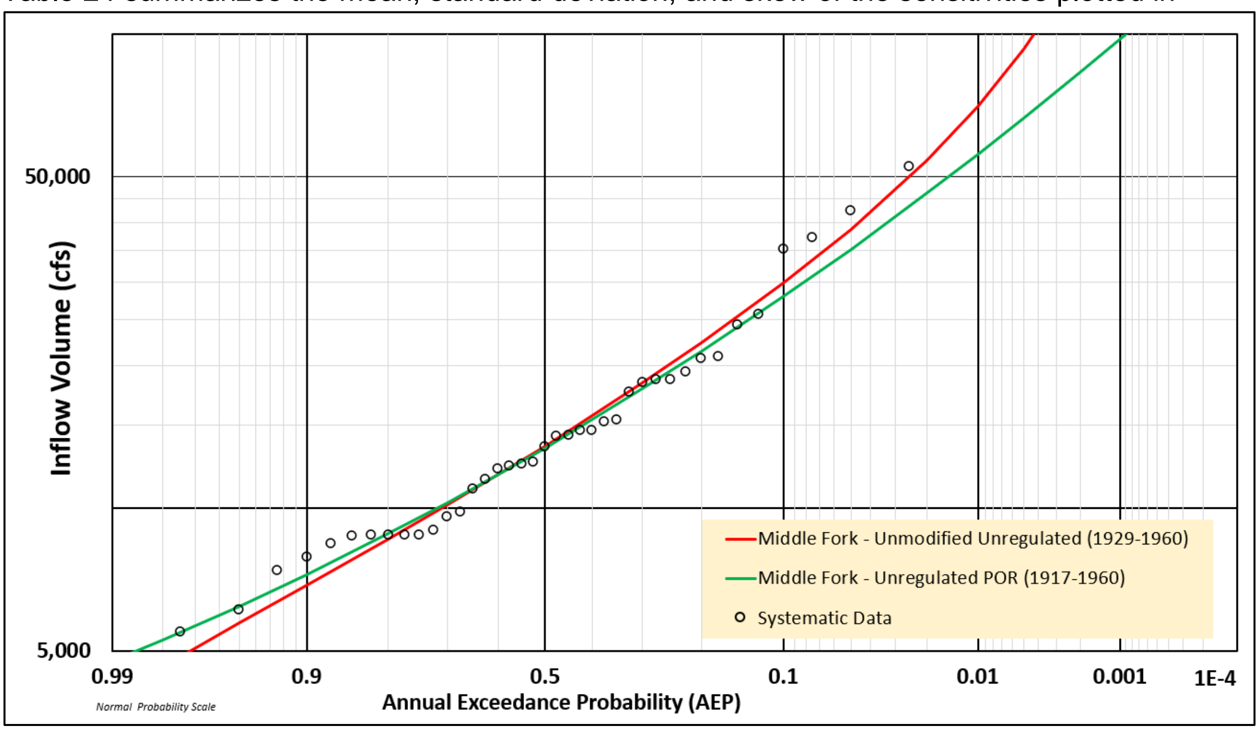

Figure 3-49, which were similar between the sensitivities reviewed.

Table 24: Middle Fork at Tallega Statistical Summary of Bulletin 17C Sensitivities

Data Set Description	***Mean (of log) (μ)***	***Std. Dev. (of log) (σ)***	***Skew (of log) (γ)***
Middle Fork Unmodified Unregulated (1929-1960)	4.145	0.241	0.273
Middle Fork Final POR (1917-1960)	4.144	0.230	0.408

Figure 3-50 shows the final unregulated flow-frequency curve for Middle Fork using the B17C analysis in HEC-SSP.

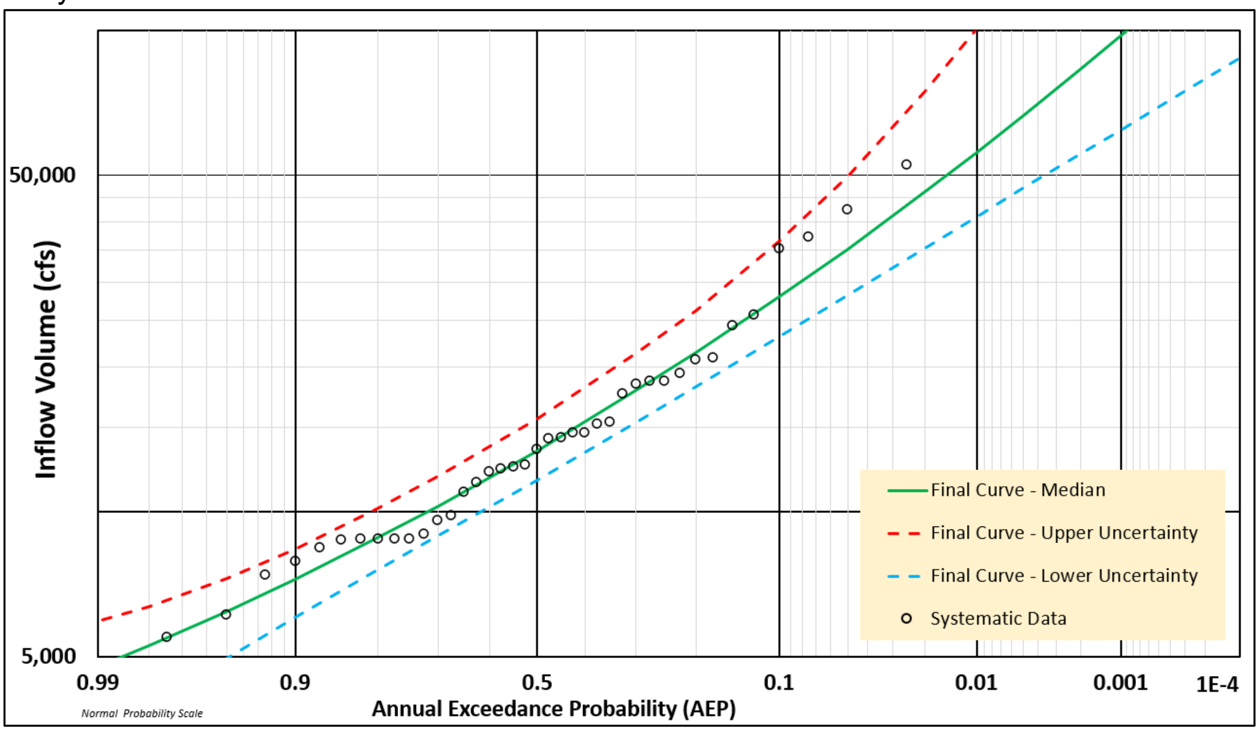

Figure 3.50: Middle Fork Final Unregulated Flow Frequency Curve

3.9.3 Regulated Flow-Frequency Curve

A regulated-unregulated relationship for Middle Fork with good correlation could not be established as part of this study. Therefore, regulated systematic data from the Middle Fork USGS gage 0328100 at Tallega from 1961 to 2022 was used directly to develop the regulated flow frequency curve. Figure 3-51 shows the HEC-SSP EMA Data for the regulated POR.

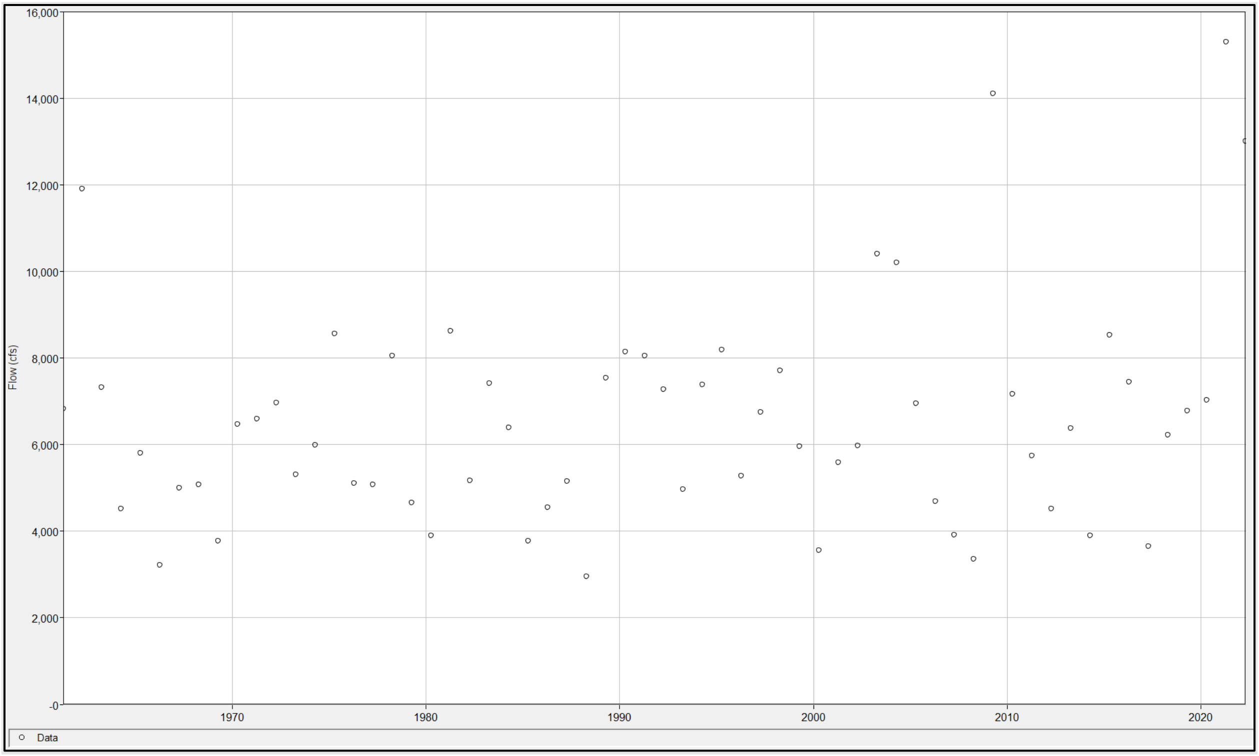

Figure 3.51: Middle Fork Regulated POR HEC-SSP EMA Data

Regional Skew Information

Regional skew information was available from an existing study published by the USGS in cooperation with the Kentucky Transportation Cabinet. The study is titled "Estimating the Magnitude of Peak Flows for Streams in Kentucky for Selected Recurrence Intervals". The regional skew values for peak flows for the state of Kentucky is 0.011 with a standard error and mean square error of 0.52 and 0.27, respectively. The regional skew was applied to the B17C analysis, and the weighted skew was used as the final adopted skew.

Potentially Influential Low Outliers

The Multiple Grubbs-Beck Test was performed on the data to determine if the POR contained any PILF values. PILF's are reviewed to confirm that small observed inflows do not have an inappropriately large impact on the inflow frequency analysis. The test identified no PILF values.

Regulated Flow Frequency Sensitivities

The final POR unregulated and regulated flow frequency curves are plotted in

Figure 3-52.

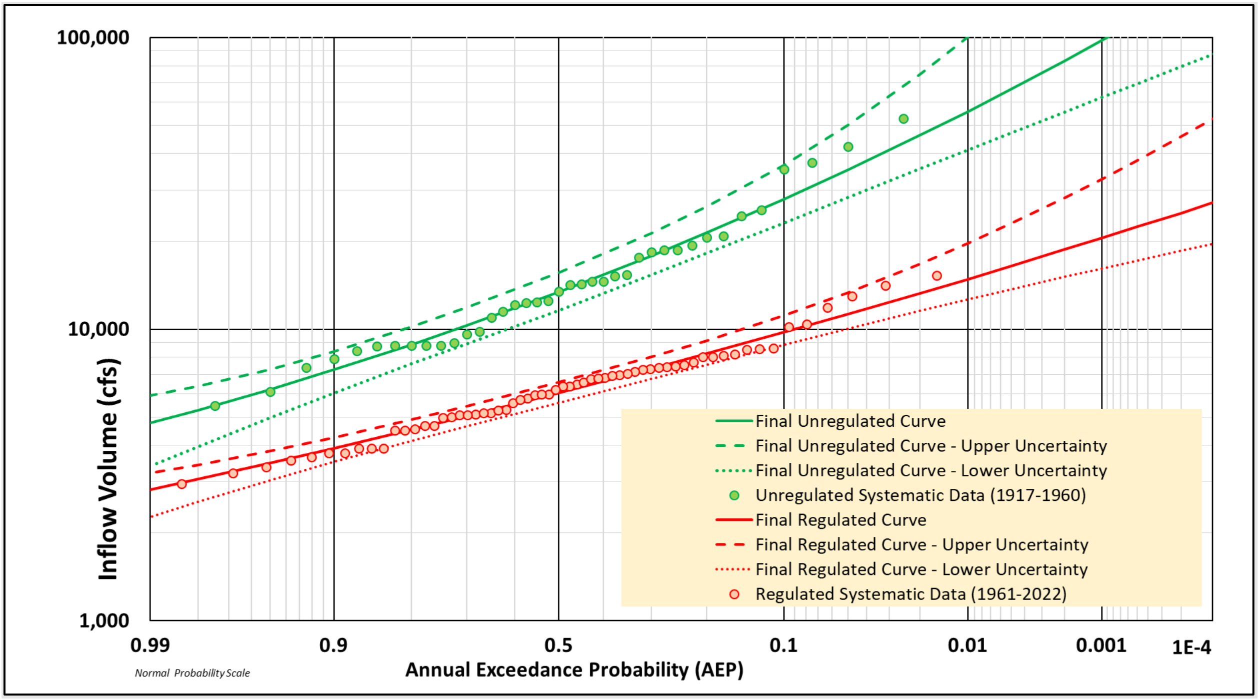

Figure 3.52: Middle Fork Regulated and Unregulated Flow Frequency Curve Comparison

It should be noted that systematic regulated flow data is typically not used to generate regulated flow frequency curves. Systematic regulated data when plotted in a flow frequency analysis can create ‘steps’, where flows are consistent over a series of AEPs and then steps up for another series of AEPs. This is a result of the operation of regulated infrastructure controlling outflow conditions and is not as well suited for Log-Pearson Type III (LP3) distributions as unregulated data. LP3 curves fit through systematic regulated data can result in a highly positive skew that cause the regulated flow frequency curve to cross and exceed the unregulated flow frequency curve for less frequent events. Graphical peak flow frequencies are often used for regulated data in these instances but as shown in

Figure 3-52, this is not the case with the Middle Fork systematic regulated data and the LP3 distribution fits the data reasonably well. Because a regulated-unregulated relationship could not be established and the two frequency curves do not cross, the LP3 distribution was considered acceptable for this study to use the systematic regulated data for the regulated flow frequency curve.

Figure 3-53 shows the final POR regulated flow frequency curve with systematic data and 90 percent upper and lower credible interval.

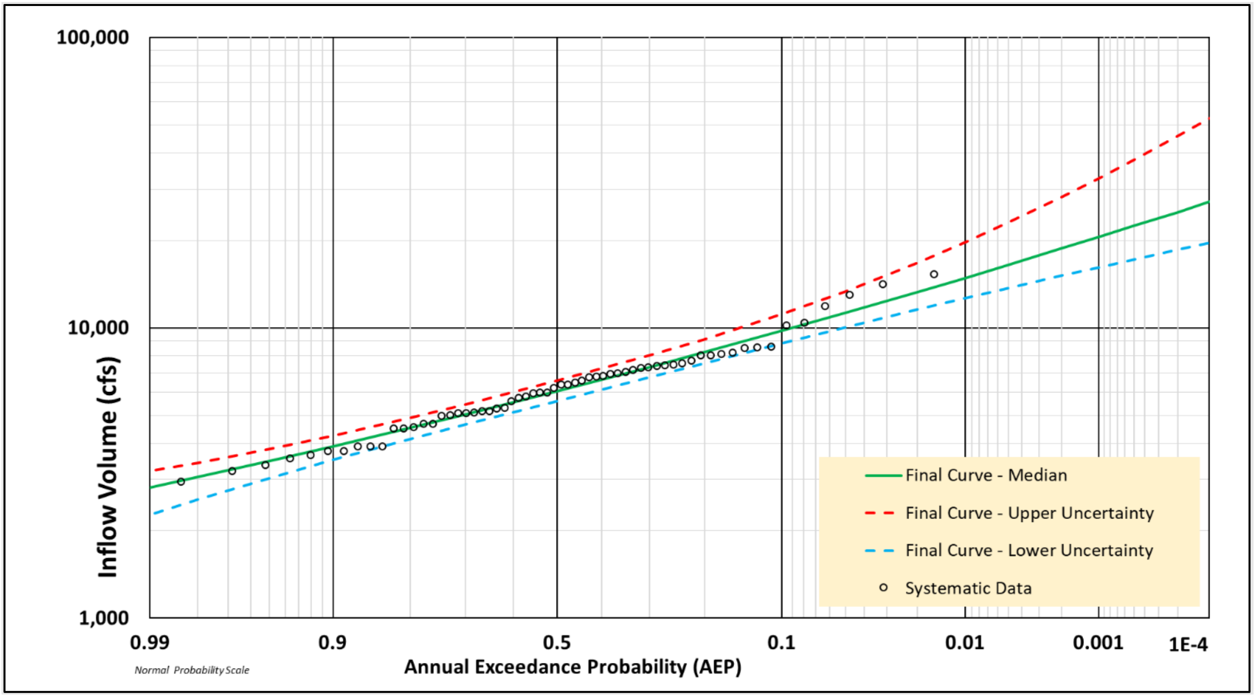

Figure 3.53: Middle Fork Final Regulated Flow Frequency Curve

Regulated-Unregulated Relationship

As previously noted, a regulated-unregulated relationship was not established for Middle Fork. Numerous attempts were made to develop a flow relationship, but values were deemed too variable and had poor data correlation. The following data sources were reviewed to establish a regulated-unregulated flow relationship at Middle Fork:

- Spreadsheet model provided by the LRL Water Management Section
- The Buckhorn Lake WCM
- Modified HEC-HMS analysis of the Kentucky River Basin at 1-hour time step from 2008 to 2022 based on available 1-hour data at Buckhorn Lake.
- Modified HEC-HMS analysis of the Kentucky River Basin at 6-hour time step from 1988 to 2022 based on available 6-hour data at Buckhorn Lake.

Figure 3-54 shows the data from the above sources, associated trendlines, and 1 to 1 ratio for the Middle Fork.

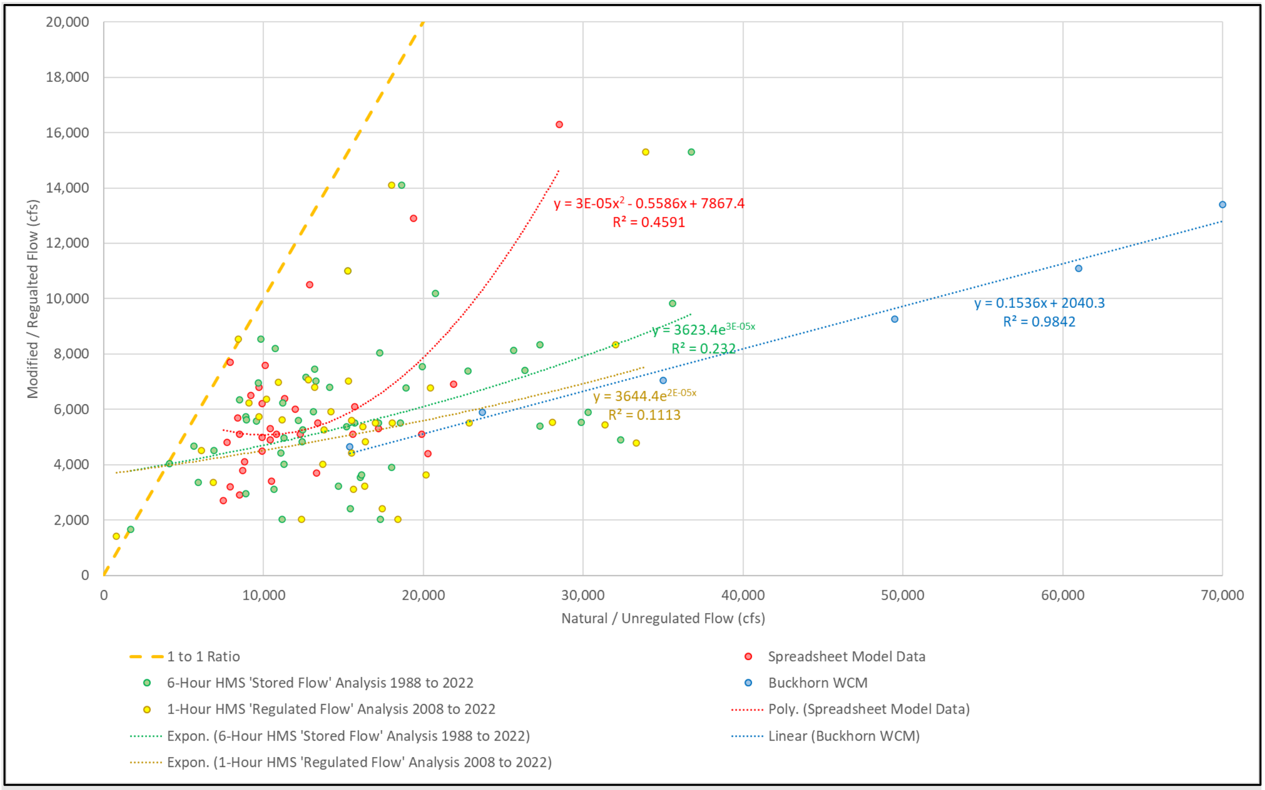

Figure 3.54: Middle Fork Regulated-Unregulated Relationship Comparisons

The HMS analyses used a similar process as previously described for Lock 14. Different time steps were used based on the availability of data at the referenced time step. As shown in Figure 3-54, the various analyses produced a wide range of results that did not have good data correlations.

It is unknown exactly why attempts to create a regulated-unregulated relationship for Middle Fork were unsuccessful. One possibility is the combination of the high amount of regulation Buckhorn Lake provides compared to the basin area, the orientation of storm events, and the generally flashy conditions of the upper Kentucky River Basins. If a storm event is centered at/or upstream of Buckhorn Lake, the Middle Fork USGS gage will show a significant reduction in natural flow based on the amount of regulation Buckhorn Lake provides. Conversely, if a storm event is centered downstream of Buckhorn Lake, the flashy conditions of the basin could result in near natural flow conditions at the Middle Fork USGS gage because Buckhorn Lake is not able to provide any regulation. This could result in a wide range of regulated flow conditions. Additional analyses on the effects of precipitation on the Middle Fork basin could be performed to refine the regulated-unregulated relationship but it was not deemed necessary at the time of this planning study.

3.9.4 Regulated Stage-Frequency Curve

Figure 3-55 shows the published Middle Fork USGS gage 003282000 at Tallega rating curve. Systematic data at the Middle Fork USGS gage 003282000 at Tallega rating curve as well as 10 percent upper and lower uncertainties are also shown for reference.

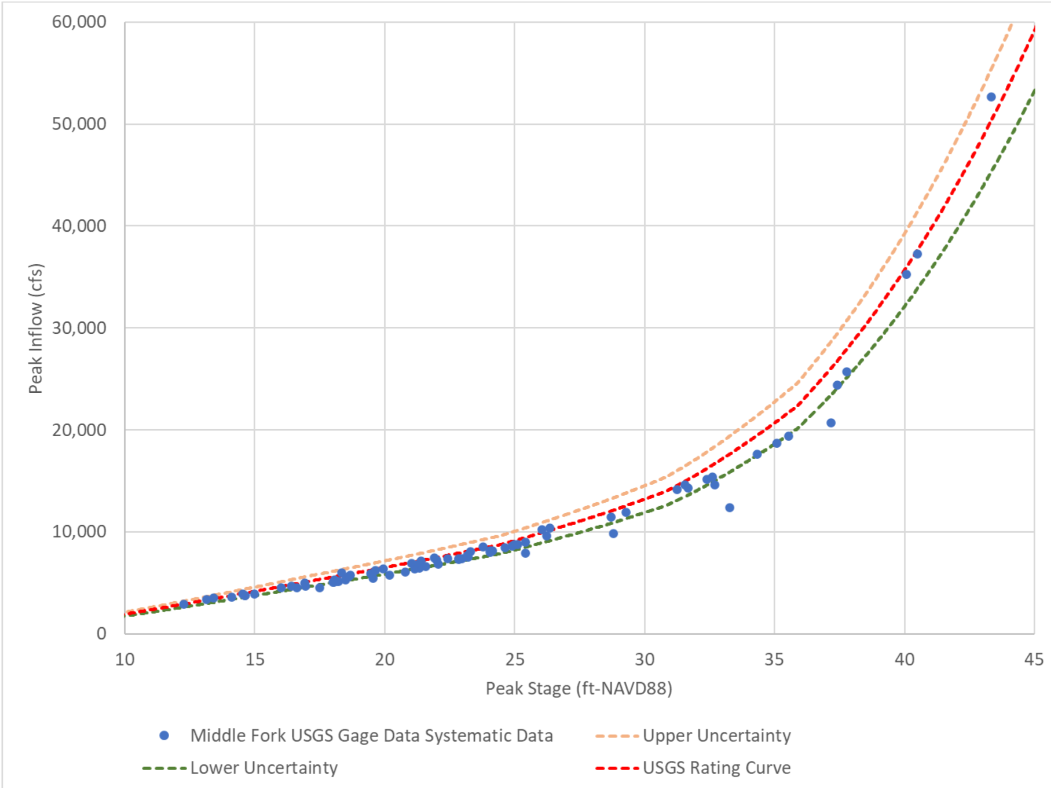

Figure 3.55: Middle Fork at Tallega USGS Published Rating Curve

The stage frequency curve for Middle Fork was developed by directly transposing the regulated flow frequency curve in Figure 3-53 with the rating curve shown in Figure 3-55.

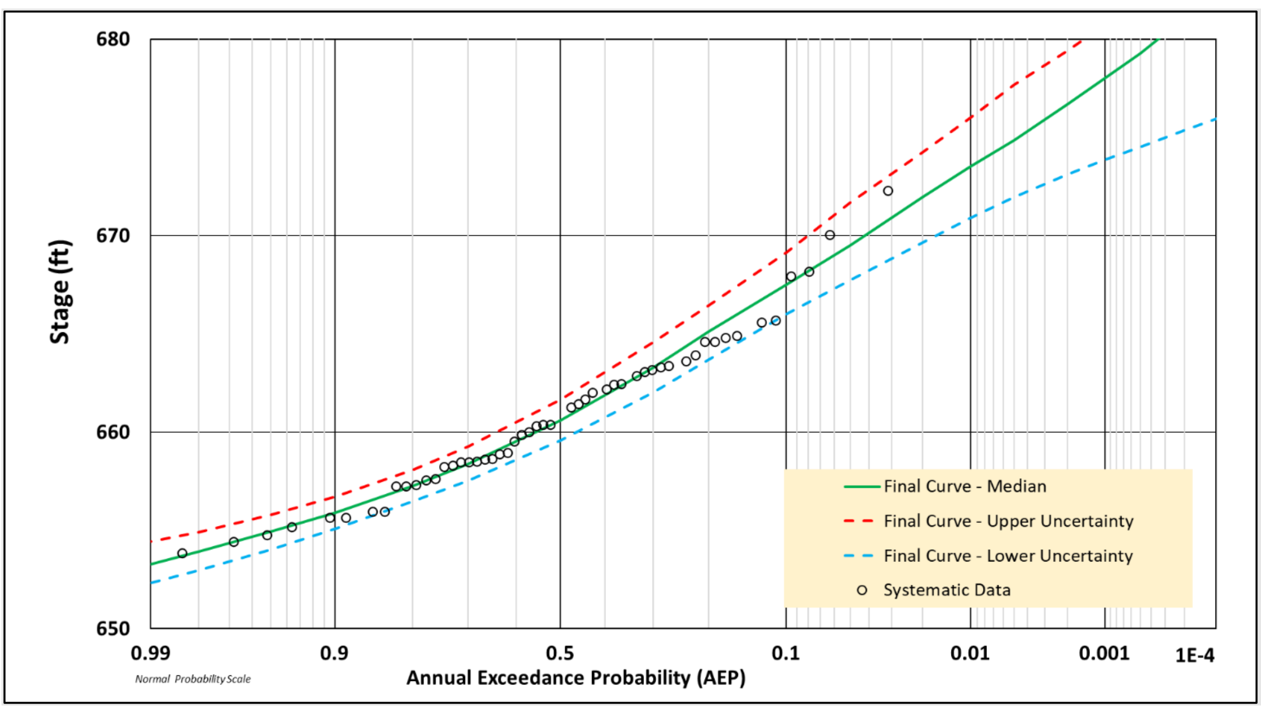

Figure 3.56: Middle Fork Final Regulated Stage-Frequency Curve

3.9.5 Hydrologic Hazard Curve Summary Tables

Table 26: Middle Fork at Tallega Final Regulated Stage Frequency Curve Summary Table and

Table 26 summarize the final regulated flow and stage frequency curves for Middle Fork at Tallega for various AEPs, respectively.

Table 25: Middle Fork at Tallega Final Regulated Flow Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	28,250	15,110	21,049	18,748
0.01	100	19,730	12,637	15,591	14,856
0.04	25	13,932	10,289	11,817	11,601
0.1	10	11,138	8,844	9,854	9,766
0.2	5	9,137	7,564	8,294	8,252
0.5	2	6,568	5,595	6,057	6,067
0.99	1	3,207	2,272	2,813	2,698

Table 26: Middle Fork at Tallega Final Regulated Stage Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (ft)***	***5% Confidence Interval (ft)***	***Expected [Mean] Curve (ft)***	***Computed [Median] Curve (ft)***
0.002	500	676.68	675.46	679.42	673.12
0.01	100	673.52	672.91	675.99	670.88
0.04	25	669.93	669.68	672.10	668.05
0.1	10	667.49	667.36	669.13	666.01
0.2	5	665.10	665.02	666.44	663.67
0.5	2	660.56	660.58	661.65	659.58
0.99	1	653.55	653.29	654.43	652.31

3.10 HYDROLOGIC HAZARD CURVES – SOUTH FORK

3.10.1 Overall Systematic, Regional and Historical Data

Systematic Data Overview

Refer to Section 4.2 and 4.4 for general background and summary information on the USGS gages used as part of this analysis. South Fork USGS gage 03281500 at Booneville was the primary gage used for the development of stage and flow frequency curves. The South Fork USGS gage 03281500 includes annual peak streamflow data from 1926 to 1931, 1937 and 1939 to the present. The POR was also supplemented with data from the Middle Fork USGS gage 03281000 at Tallega for annual peak flows in 1932 and 1935 (the Middle Fork USGS gage 03281000 also does not include data from 1933, 1934, 1936 or 1938). There is no regulation in the South Fork and there are no current plans to include regulation, therefore only the unregulated peak flow frequency curve was created as part of this assessment.

Record Extension

Inflow correlations were reviewed to supplement South Fork data by plotting overlapping flow data between South Fork and Middle and North Forks. Inflow correlations were also reviewed between South Fork for and Locks 14, 10, 8, 6, and 4 but all the Locks have a drainage area ratio in excess of 3.5 compared to the South Fork USGS gage and were therefore considered not suitable for data transposition. The drainage area ratio between Middle Fork to South Fork and North Fork to South Fork are approximately 0.74 and 1.52, respectively, which indicates it is suitable for data transposition.

Direct overlapping data and log-log scale data was reviewed. Table 27 summarizes the r squared values of the direct and log-log scale data comparisons.

Table 27: South Fork Annual Peak Inflow Data Comparisons

Location Comparisons	***Direct Data Comparison Polynomial Trendline R Squared***	***Log-Log Scale Data Comparison R Squared***
North Fork to South Fork	0.7314	0.6447
Middle Fork to South Fork	0.8193	0.6965

As shown in Table 27, South Fork USGS gage 03281500 at Booneville had a better correlation with Middle Fork USGS gage 03281000 at Tallega. Therefore, the Middle Fork USGS gage 03281000 at Tallega was used for record extension. Figure 3-57 shows the direct flow comparison with drainage area ratio, 1 to 1 ratio, and polynomial trendline between South Fork and Middle Fork USGS gages.

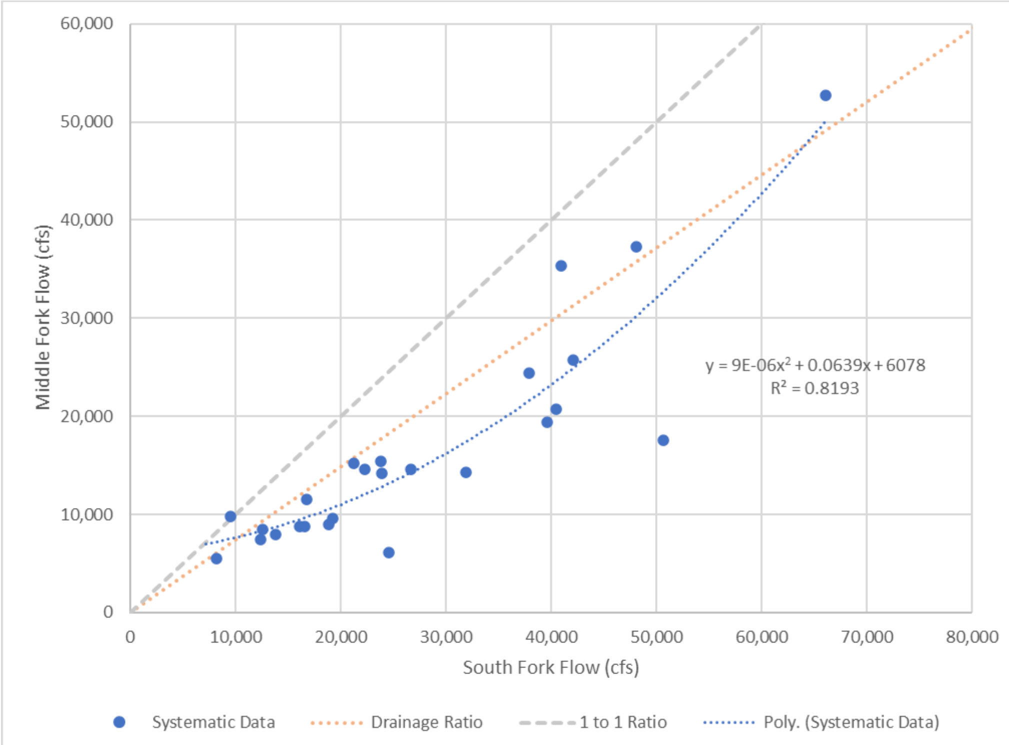

Figure 3.57: South Fork and Middle Fork Flow Data Comparison

Perception Thresholds

Missing annual peak inflow data in the POR between 1933 to 1934, 1936, and 1938 was assigned a PT of 66,100 cfs, which is the unregulated flood of record for the South Fork.

Data Summary

To summarize, the following information was used to develop the final unregulated POR data used for South Fork.

- A PT of 66,100 cfs was used between 1933 to 1934, 1936, and 1938 to supplement data gaps in the unregulated POR. The PT was based on the flood of record.
- Transposed unregulated data from Middle Fork to South Fork was used for 1932 and 1935.
- Unregulated flows from South Fork from 1926 to 1931, 1937, and 1939 to present were directly used.

Figure 3-58 shows a visual of the final composite South Fork POR used in the assessment.

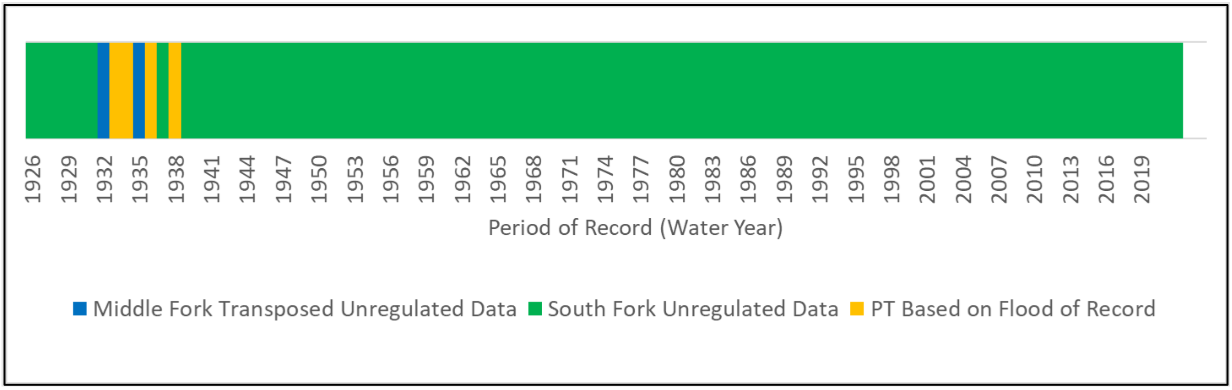

Figure 3.58: South Fork Final POR Visualization

3.10.2 Unregulated Flow-Frequency Curve

The composite unregulated peak flow POR was used to calculate the peak flow frequency curve using a B17C analysis in HEC-SSP. The final POR included systematic and transposed data from 1908 to 2022 as previously discussed. Figure 3-59 shows the HEC-SSP EMA Data for the unregulated POR for South Fork.

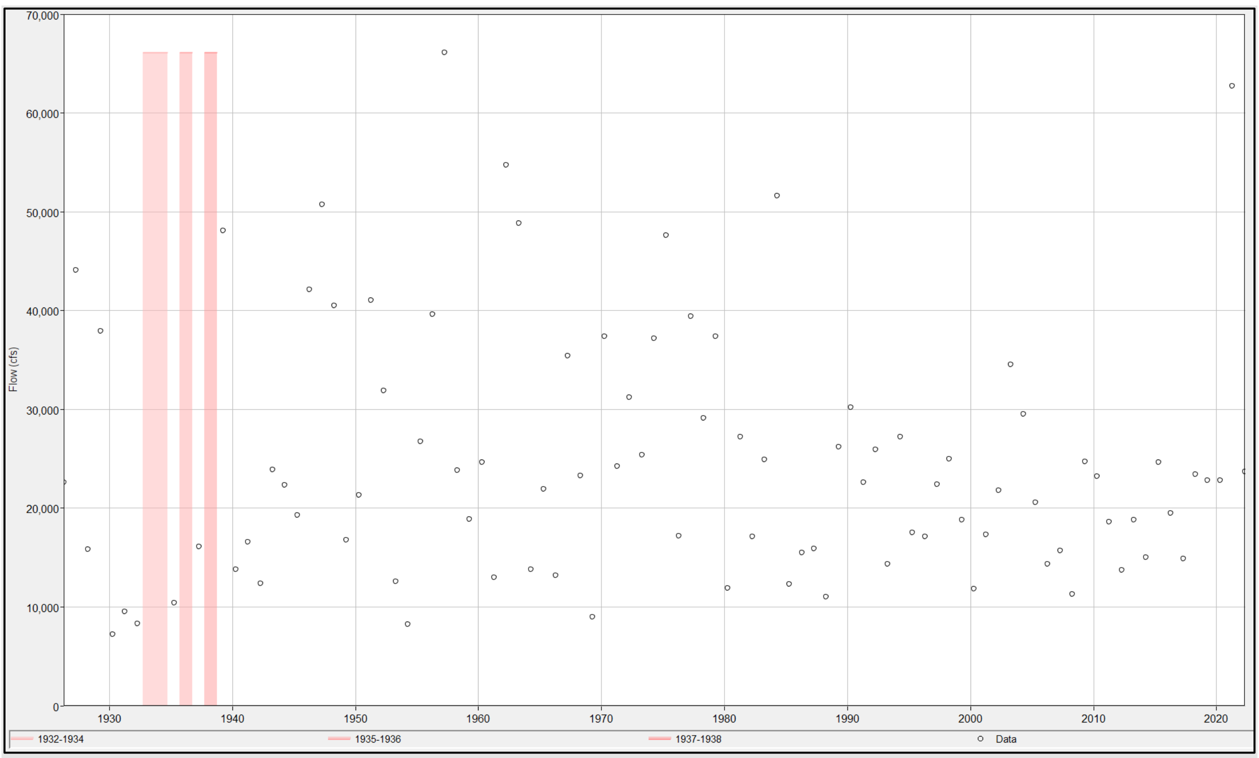

Figure 3.59: South Fork Unregulated POR HEC-SSP EMA Data

Regional Skew Information

Regional skew information was available from an existing study published by the USGS in cooperation with the Kentucky Transportation Cabinet. The study is titled “Estimating the Magnitude of Peak Flows for Streams in Kentucky for Selected Recurrence Intervals”. The regional skew values for peak flows for the state of Kentucky is 0.011 with a standard error and mean square error of 0.52 and 0.27, respectively. The regional skew was applied to the B17C analysis, and the weighted skew was used as the final adopted skew.

Potentially Influential Low Outliers

The Multiple Grubbs-Beck Test was performed on the data to determine if the POR contained any PILF values. PILF's are reviewed to confirm that small observed inflows do not have an inappropriately large impact on the inflow frequency analysis. The test identified no PILF values.

Unregulated Flow Frequency Sensitivities

To determine how sensitive the analysis is to the inclusion of additional data sources, sensitivities were modeled with various data sets. Flow-frequency curves for the following data sets were developed using a B17C analysis in HEC-SSP:

- South Fork Unmodified Unregulated – includes unmodified unregulated flow data from 1926 to 1931, 1937, and 1939 to present.
- Final POR – includes the final period of record as previously discussed with interval data and PTs.

Figure 3-60 show a plot of the curves for the above data sets using a B17C analysis in HEC-SSP.

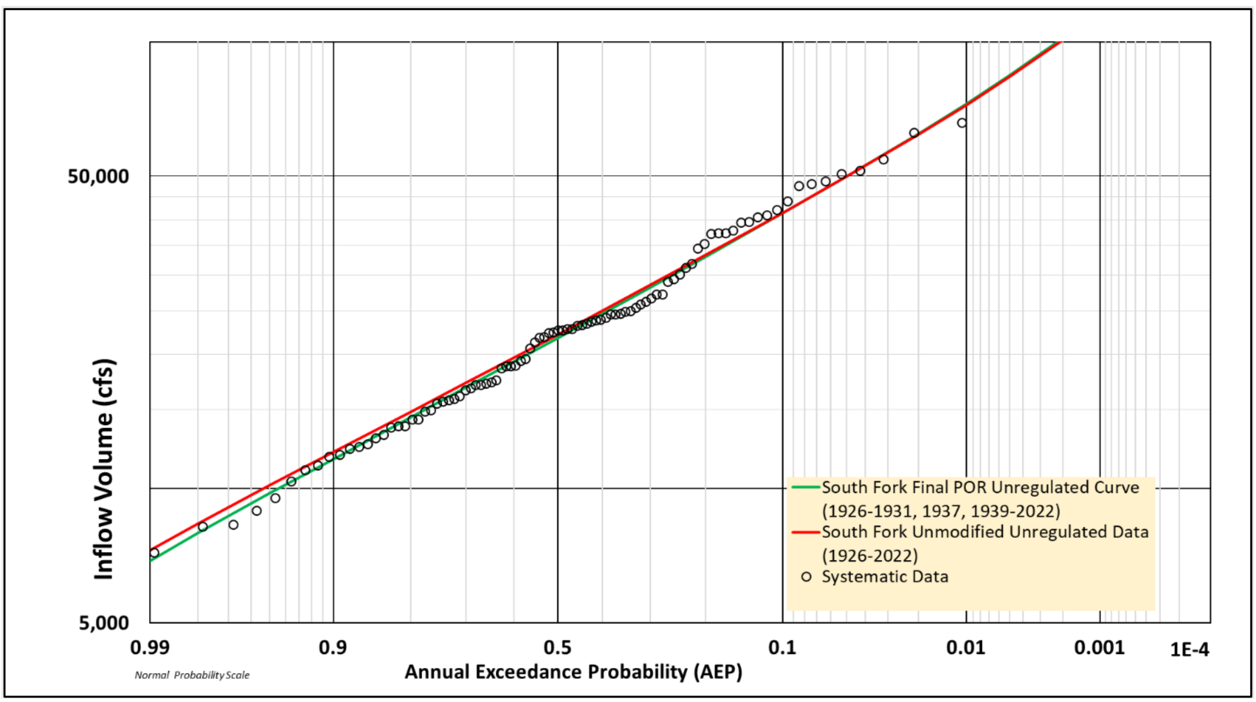

Figure 3.60: South Fork Unregulated Inflow Frequency Sensitivities

Table 28 summarizes the mean, standard deviation, and skew of the sensitivities plotted in Figure 3-60. Both sensitivities are similar in terms of mean, standard deviation, and skew.

Table 28: South Fork at Booneville Statistical Summary of Bulletin 17C Sensitivities

Data Set Description	***Mean (of log) (μ)***	***Std. Dev. (of log) (σ)***	***Skew (of log) (γ)***
South Fork Unmodified Unregulated (1926-1931, 1937, 1939-2022)	4.348	0.207	0.084
South Fork Final POR (1926-2022)	4.340	0.212	0.066

Figure 3-61 shows the unregulated POR final curve with systematic and B17C analysis in HEC-SSP.

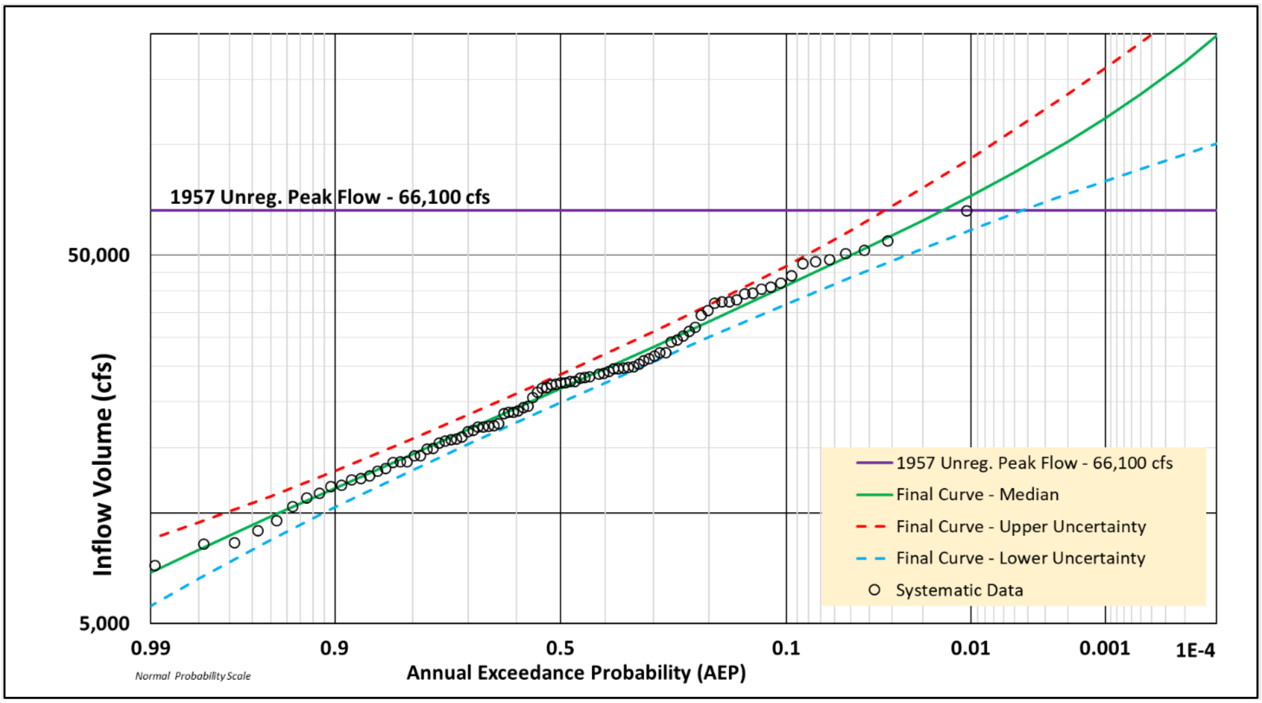

Figure 3.61: South Fork Final Unregulated Flow Frequency Curve

3.10.3 Regulated Flow-Frequency Curve

South Fork flows are currently not regulated and as of the time of this analysis, there were no plans to incorporate regulation within the South Fork basin. Therefore, regulated flow-frequency curves for South Fork were not developed.

3.10.4 Stage-Frequency Curves

Figure 3-62 shows the published South Fork USGS gage 003281500 at Booneville rating curve. Systematic data at the South Fork USGS gage 003281500 at Booneville rating curve as well as a best fit exponential trendline through the systematic data and 10 percent upper and lower uncertainties for the best fit trendline are also shown for reference.

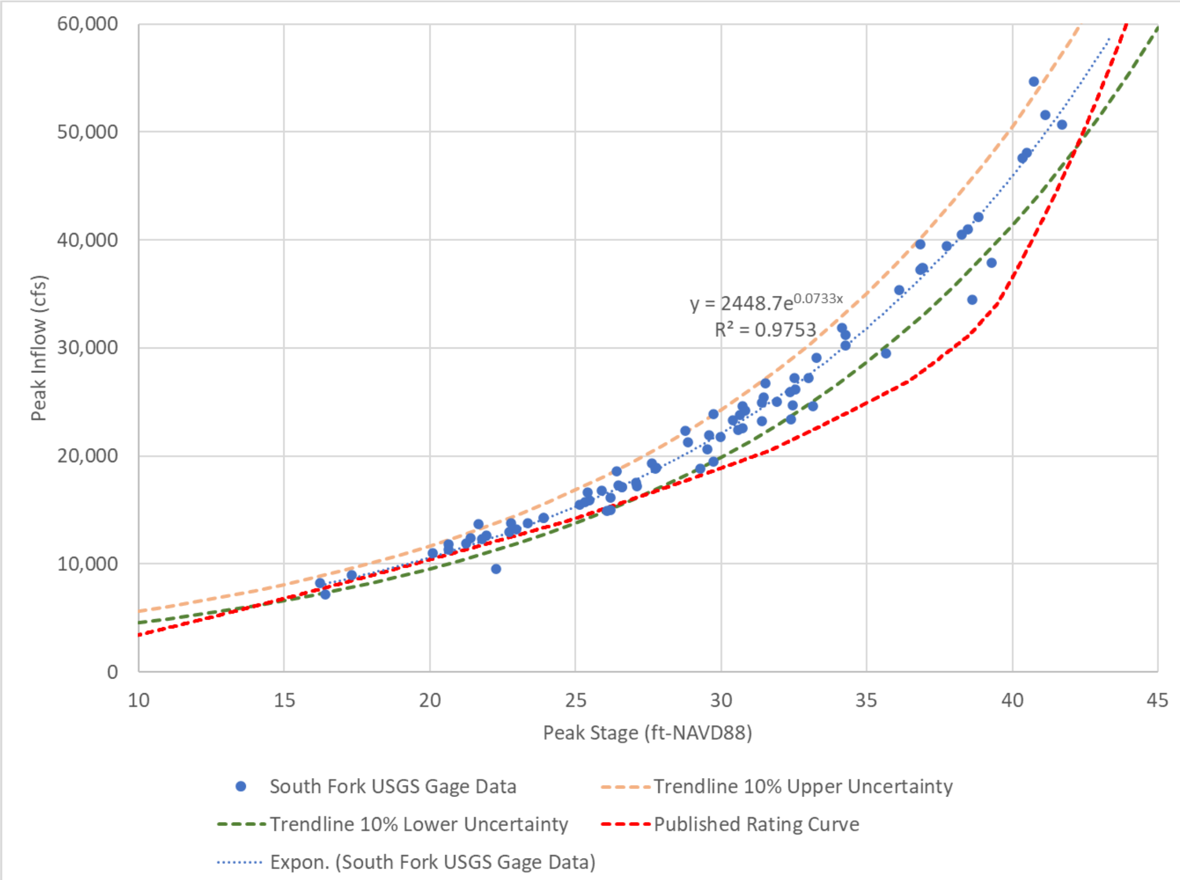

Figure 3.62: South Fork at Booneville USGS Published Rating Curve

As shown in Figure 3-62, the published USGS rating curve does not match the systematic data as well as the exponential trendline fit through the systematic data. It is not certain why the published USGS curve appears shifted from the systematic data. The best fit trendline was used to develop the South Fork stage frequency curve, which is shown in Figure 3-63.

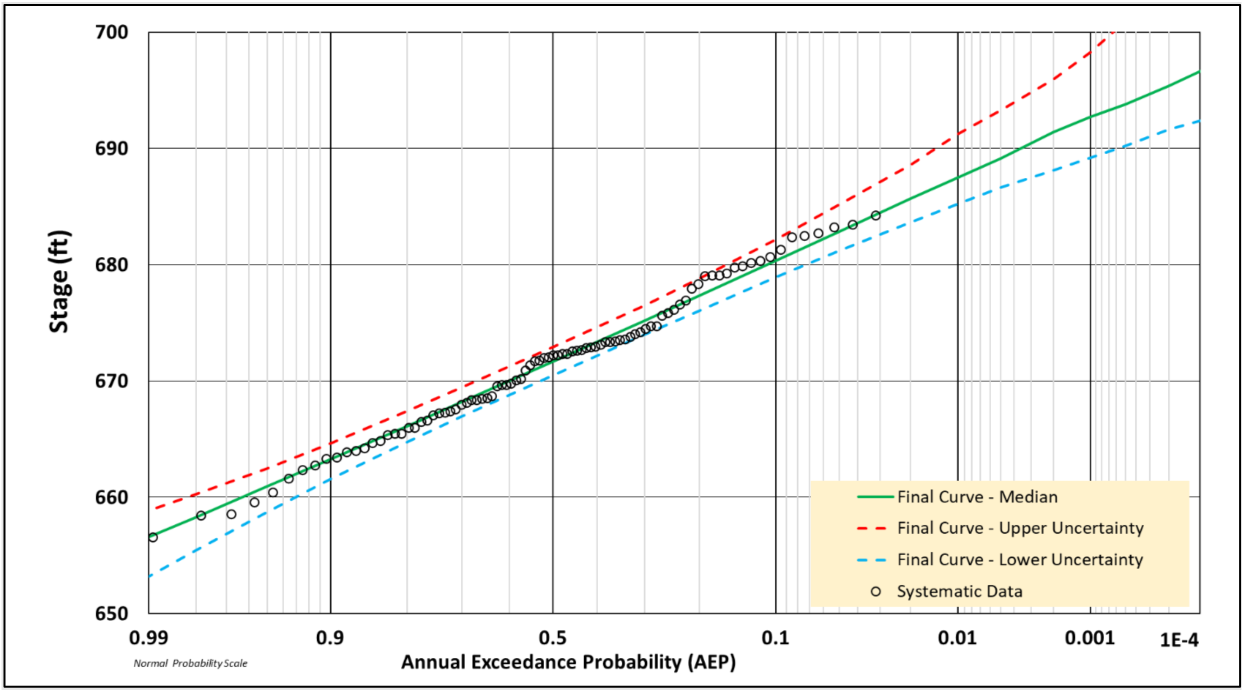

Figure 3.63: South Fork Final Unregulated Stage-Frequency Curve

3.10.5 Hydrologic Hazard Curve Summary Tables

Table 29 and Table 30 summarize the final unregulated flow and stage frequency curves for South Fork at Booneville for various AEPs, respectively.

Table 29: South Fork at Booneville Final Unregulated Flow Frequency Curve Summary Table

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	137,285	73,299	101,973	92,877
0.01	100	91,801	58,604	72,572	69,817
0.04	25	61,320	45,016	51,975	51,207
0.1	10	46,835	36,890	41,338	41,046
0.2	5	36,602	29,957	33,063	32,937
0.5	2	23,782	19,889	21,756	21,746
0.99	1	8,471	5,589	6,883	7,181

Table 30: South Fork at Boonville Final Unregulated Stage Frequency Curve Summary Table

AEP	*Return Period (Years)*	*95% Confidence Interval (ft)*	*5% Confidence Interval (ft)*	*Expected [Mean] Curve (ft)*	*Computed [Median] Curve (ft)*
0.002	500	695.94	688.10	692.52	691.44
0.01	100	691.26	685.22	687.98	687.51
0.04	25	685.73	681.59	683.54	683.34
0.1	10	682.16	678.91	680.46	680.36
0.2	5	678.80	676.07	677.41	677.36
0.5	2	672.92	670.48	671.70	671.70
0.99	1	658.84	653.16	656.01	656.58

3.11 UNCERTAINTY

This section discusses potential points of uncertainty within the data or previously discussed analyses beyond standard inaccuracies in gage data. As part of the SMART planning guidance, the risk associated with the assumptions made in this report, and subsequently the uncertainty outlined below is also noted.

Lock 14 Rating Curve

As previously noted, the USGS published data at Lock 14 is based on a non-sloped rating curve. Discussions with USGS and review of their field measurements captured during the March 2021 event indicate that the Lock 14 rating curve reacts as a sloped rating curve during certain sustained high flow and downstream conditions that create backwater effects. The HEC-RAS model was calibrated and validated based on the March 2021 event, which yielded a rating curve at Beattyville that created a hysteresis loop in the model. USGS noted there is insufficient observed data to determine how and when Lock 14 reacts to flow based on a sloped rating curve. Therefore, a composite rating curve created from the model rating curve with additional uncertainty was used for the final transform of the flow frequency curve to stage frequency curve at Beattyville. Not knowing which events react based on the sloped and non-sloped rating curves introduces additional uncertainty to the analyses.

If deemed necessary as the study continues, additional sensitivities could be evaluated comparing the impacts of using the various rating curves to develop the final stage frequency curve at Beattyville. This would help quantify the level of uncertainty and sensitivity between the various rating curves.

Beattyville HEC-RAS Rating Curve

The final Beattyville stage frequency curve was created using a rating curve developed in HEC-RAS. There is no gage or significant field data at cross section 257.5170 where the rating curve was developed to calibrate to or determine the level of uncertainty in the HEC-RAS rating curve. As noted above, the HEC-RAS rating curve created a hysteresis loop, which was simplified to a composite rating curve for the purposes of this report.

Middle Fork

As previously noted, an unregulated-regulated relationship was not established for Middle Fork USGS gage 03281000 at Tallega. Several methods and data sources were used in an attempt to establish a relationship, but none were deemed accurate or consistent enough to use. Therefore, the regulated flow data set was input directly into HEC-SSP to develop the regulated flow frequency curve and fit to a Log-Pearson type III (LP3) distribution. This results in some uncertainty because regulated data is typically not fit to an LP3 distribution for the creation of flow frequency curves.

Coincident Loading

Uncertainty related to timing on the North, Middle and South Forks of the Kentucky River and how that coincident loading affects stages at Beattyville is documented in Section 4.7. This study accounts for flooding on all 3 forks by performing the Bulletin 17C Study at Lock 14 which is located downstream of Beattyville.

SMART Planning Risk

The risk associated with assumptions made as part of this analysis affect the final stage frequency results utilized to determine the 0.01 AEP flood event. The stage frequency curve accounts for these sources of uncertainty, which subsequently affects the level of assurance calculations. The PDT believes that adequate analysis has been performed to quantify the uncertainty, and therefore the risk, associated with the development of the stage frequency curve at Beattyville. Future studies should consider if additional analysis is warranted to further reduce uncertainty, and therefore further reduce planning risk, associated with the hydrologic analysis. The key sources of uncertainty that affect the final stage frequency curve are summarized in the following list.

- The final POR was extended to include data from Lock 4 and Lock 10. Extending the period of record can reduce the uncertainty in the Bulletin 17C analysis. However, there is also uncertainty in the relationship between Lock 14 and the downstream Locks used to extend the POR.
- The final regulated inflow frequency curve utilizes a regulated-unregulated relationship to convert the unregulated inflow frequency curve to a regulated flow frequency curve. Uncertainty in this relationship was incorporated as part of the final analysis but could be further investigated in future studies to try to reduce the uncertainty in that relationship.
- The rating curve at Beattyville was utilized to convert the regulated inflow frequency curve to a regulated stage frequency curve. This is the largest source of uncertainty and is also accounted for in the final results. Additional modeling could be performed to better understand the rating curve at both Lock 14 and Beattyville.

4 HYDROLOGY

4.1 PROJECT DESCRIPTION

4.1.1 Project Site

Beattyville is considered a city and is the county seat of Lee County, Kentucky. Beattyville is situated at the confluence of the North Fork and South Fork Kentucky Rivers which forms the Kentucky River mainstem. The community is on the North bank of the confluence on flat terrain that rises quickly with the steep topography away from the river. About 6 miles downstream is Lock 14 on the Kentucky River at Heidelberg, Kentucky. See Figure 4-1.

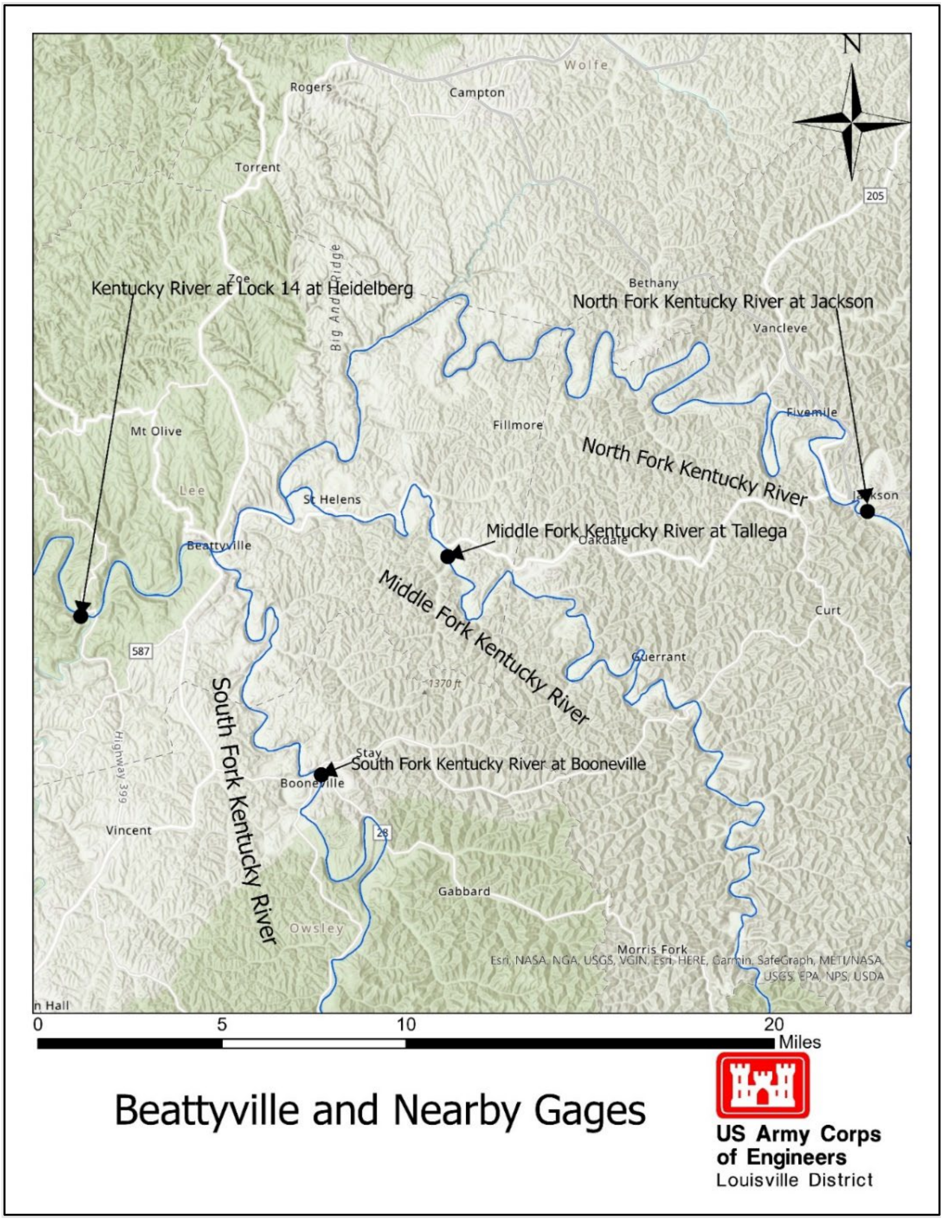

Figure 4.1: Beattyville and Surrounding Area

The climate is typical of the Commonwealth of Kentucky characterized by hot, humid summers and mild to cool winters. The steep terrain is prone to flash flooding because intense storms can produce high runoff rates, particularly in the winter to early spring when vegetation is dormant, and soils have higher moisture content.

4.1.2 Primary Flooding Source

Two local streams pass through downtown Beattyville. Crystal Creek flows south on the east side of downtown (drainage area of 5.2 mi^2) and Silver Creek flows south on the west side (drainage area of 3.3 mi^2), both terminating at the Kentucky River. Mirey Creek is third stream that flows enters the Kentucky River west of downtown Beattyville (drainage area of 0.4 mi^2). See Figure 4-2 for interior drainage watersheds.

Existing information does not indicate principal flooding is caused by any of the Crystal and Silver Creeks (the Creeks). The contributing drainage areas are relatively small, and the overall basins slopes are steep. The streams are likely very flashy, but not such as to contribute to damaging floods. The time of concentrations for the watersheds are small relative to the entire uncontrolled Kentucky River watershed at Beattyville (2,164 mi^2). Coincident flooding of the Creeks and the Kentucky River is not likely. The significance of coincidence is discussed later in Section 4.9.7 during the evaluation of a floodwall alternative.

The primary flooding is due to the Kentucky River backwater, as stated in previous reports such as the Flood Insurance Study (FIS). The FIS indicates that although flashy with high stream velocities, the Creeks do not flood Beattyville when the Kentucky River is at normal stage and scour is not an issue. There is no evidence to suggest the Creeks are a flooding concern in downtown Beattyville. Currently, flooding from the Creeks is not being investigated further as a flooding source and the study focus remains on the Kentucky River.

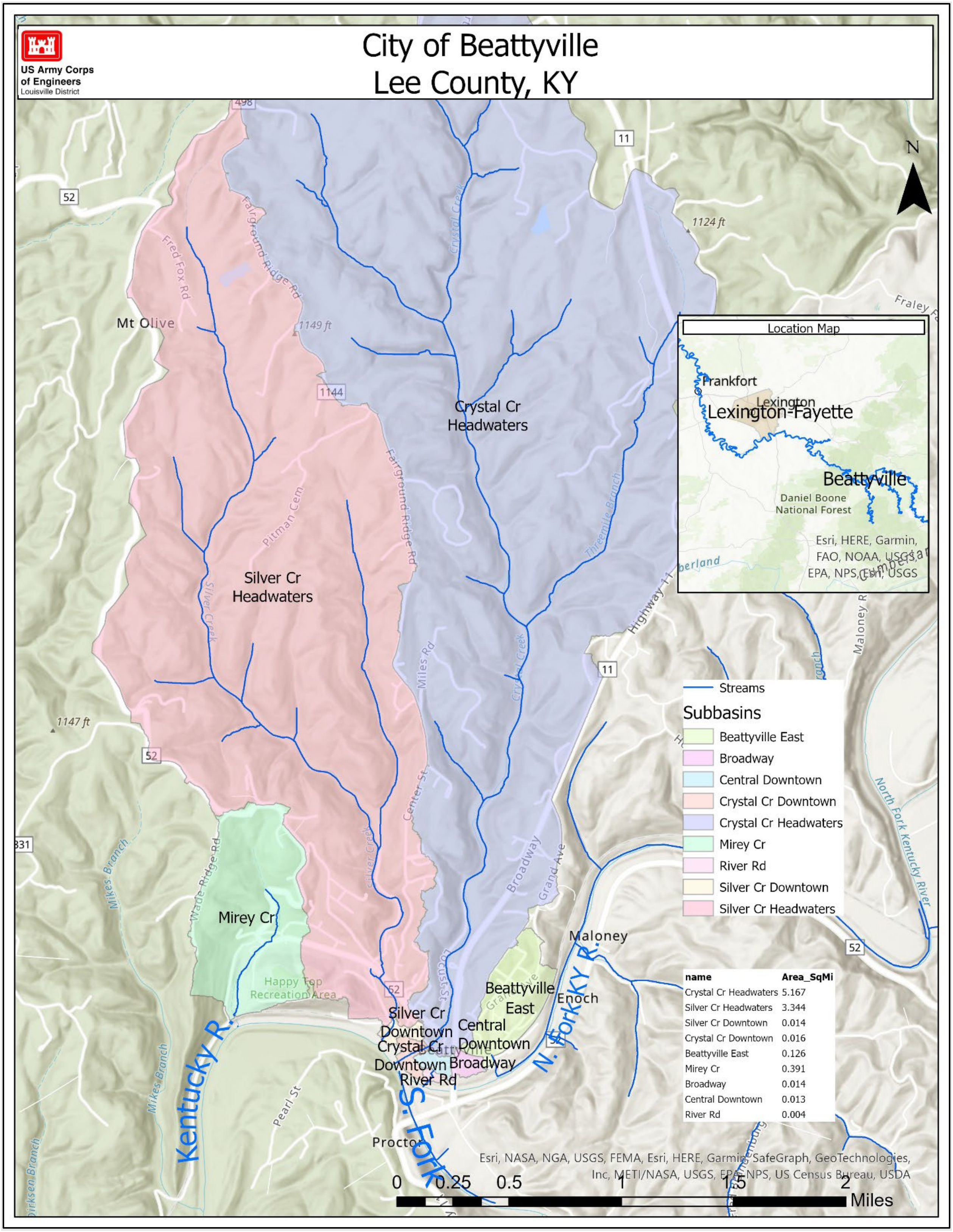

Figure 4.2: Beattyville Interior Watersheds Map

4.1.3 Flood Insurance Study

The current Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Lee County contains peak discharges that have been in effect since 1976. The preliminary FIS Number 211219CV000B is dated 27-October-2022, where the Beattyville Community Number is 210136 and the unincorporated areas of Lee County is Community Number 210135. See the Summary of Discharges from the preliminary FIS in Figure 4-3.

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Crystal Creek	Mile 0.31 (above mouth)	5.1	2,500	*	4,100	5,200	7,400
Kentucky River	Heidelberg Gage	2,657	79,500	*	105,000	115,000	138,800
Mirey Creek	Mile 0.26 (above mouth)	0.3	520	*	850	1,060	1,500
Mirey Creek	Kentucky Highway 52	0.1	285	*	460	580	800
North Fork Kentucky River	140 feet downstream of Broadway	1,883	60,000	*	79,300	86,800	105,000
Silver Creek	Mile 0.14 (above mouth)	3.3	1,940	*	3,200	4,000	5,800

*Not calculated for this Flood Risk Project

Figure 4.3: Summary of Discharges from the Preliminary FIS

4.1.4 Regulatory Floodway Established

A Regulatory Floodway has been established for Beattyville (shown in blue and red hatch marks in Figure 4-4). The Regulatory Floodway marks the calculated location where farther riverside encroachment into the floodplain increases the water surface of the base flood elevation (BFE) greater than the regulated limit of 1.0 feet. The local regulator is the Kentucky Division of Water (KYDoW). The effective BFE for Beattyville is 669.1 Ft. NAVD88 see section 2.3.1 identified BFE.

Figure 4.4: Current Flood Hazard Mapping of Beattyville Established by Regulators

4.1.5 Watershed Regulation

The Kentucky River is formed at Beattyville where the North and South Forks merge. The South Fork Kentucky River flows generally north from southern headwaters encompassing a 748-mi^2 unregulated drainage area. The North Fork Kentucky River near the mouth accounts for 1883 mi^2, which captures also includes the Middle Fork Kentucky River. The Middle Fork enters the North Fork about 2.6 miles upstream of Beattyville. The Middle Fork is regulated by Buckhorn Lake Dam (operated by USACE LRL beginning in 1961) regulating 73% of the Middle Fork watershed. The North Fork is regulated by Carr Creek Lake Dam (also operated by USACE LRL beginning in 1976) controlling 58 mi^2 of the watershed. The total drainage area at Beattyville is approximately 2,630 mi^2 with 2,164 mi^2 unregulated. The total percentage of upstream drainage area regulated by the two dams is 18% at Beattyville. See Table 31 below.

Table 31: Natural and Regulated Drainage Areas of the Kentucky River

		Drainage Area Regulated (Mi^2)		% Drainage Area Controlled by Year	
		408	58		
Location	Natural conditions	Buckhorn (1961)	Carr Creek (1976)	1961	1976
	Drainage Area NOT Regulated				
Heidelberg Gage	2657	2249	2191	15%	18%
Beattyville @ Fork Confluence	2630	2222	2164	16%	18%
North Fork Mouth	1883	1475	1417	22%	25%
South Fork Mouth	748	748	748	0%	0%
Middle Fork Mouth	559	151	151	73%	73%
North Fork U/S of Middle Fork	1320	1320	1262	0%	4%

Carr Creek Dam mostly benefits Hazard, Kentucky, and has little impact on the flood stage at Beattyville. Also, note the March 2021 event that initiated this study was marked by high discharges from the unregulated South Fork and a near 1% AEP event at the Tallega gage on the Middle Fork despite the storm largely missing Buckhorn Lake itself. For all intents and purposes, the Buckhorn Dam is the only significant regulation structure upstream of Beattyville. See Figure 4-5.

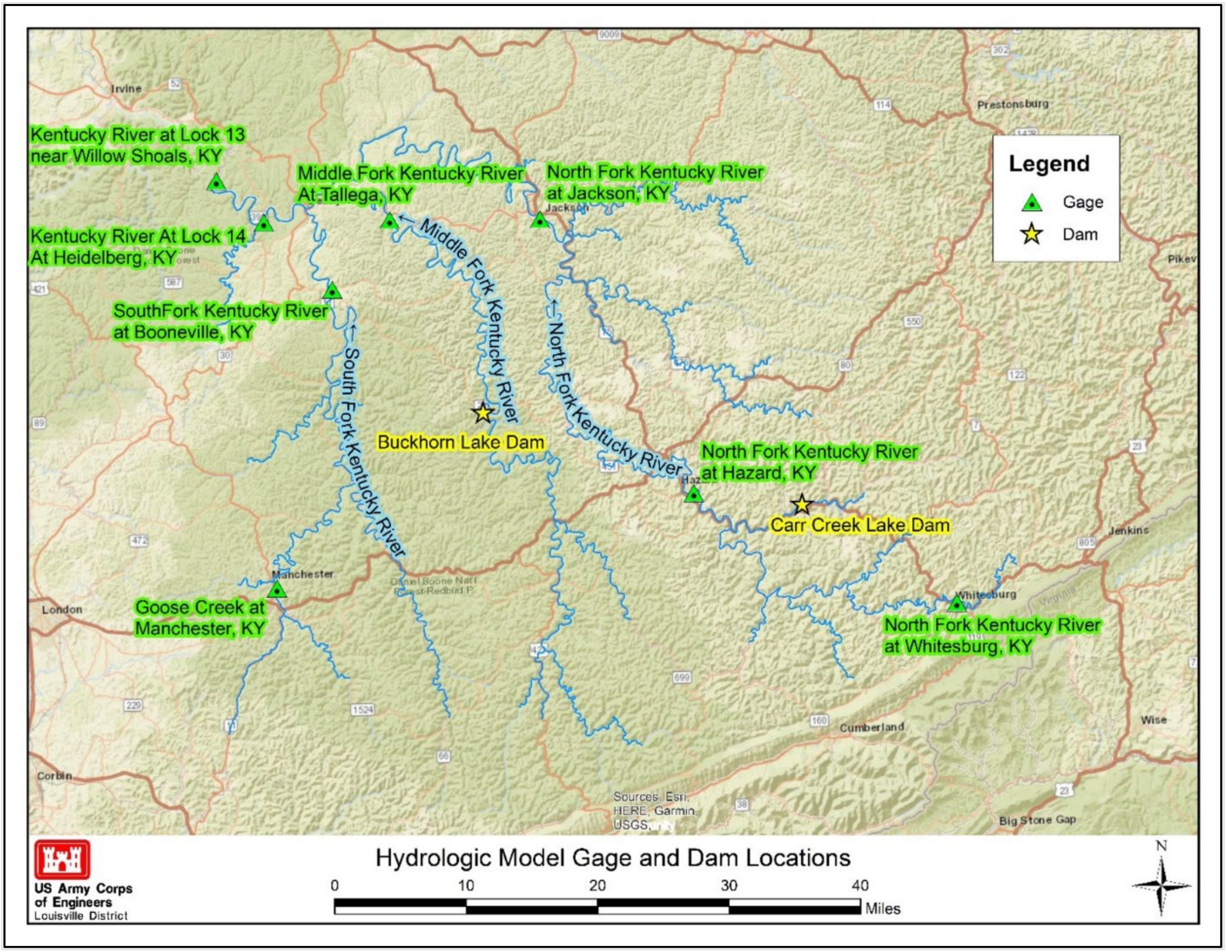

Figure 4.5: Location of Upper Kentucky River Basin Stream Gages and Reservoirs

4.1.5.1 USACE Reservoir Operations

Buckhorn and Car Creek Dams are USACE projects authorized for flood control. Water Management (WM) oversees the flood control strategy written in the Water Control Manuals (WCM) of the respective projects. In general, when heavy precipitation arrives, the Dams decrease water releases to a prescribed outflow until downstream flooding (if any) recedes. The WCM contains the Schedule of Regulation which is the primary instruction for reservoir operations. The Schedule of Regulation prescribes the reservoir releases given the reservoir pool elevation and the gage readings at downstream control points. The Dams share the same downstream control point that impacts Beattyville, which is Lock 14 at Heidelberg, KY.

The Schedules of Regulation cite control stages at Lock 14(shown in Figure 4-6 and Figure 4-7), which corresponds roughly to just below the National Weather Service ‘action’ and flood stages for the stream gage. The reservoirs are operated to this control point as to not worsen flooding according to the Schedule.

TABLE 7-2

BUCKHORN LAKE
SCHEDULE OF FLOOD CONTROL AND LOW FLOW REGULATION

Schedule	Controlling Stages (feet): Kentucky River Upper Gage Lock 14	Kentucky River Upper Gage Lock 4	Ohio River Cincinnati	Range in Pool Elevation (feet)	Time of Year	Regulation
(All stages must exist for application of Schedules A and B)						
A	Below 15 or Below 25 F	Below 10 or Below 30 F	Below 40 or Below 60 F	Minimum Pool (757)	1 December - 31 March	- Release inflow to maintain pool, provided release indicated in Maximum Release Table is not exceeded.
B	Same as Schedule A			Minimum Pool (757) to Flood Pool (840)	1 December - 31 March 1 April - 30 November	- Release at a rate indicated in Maximum Release Table. — Maintain pool as near Rule Curve as possible while meeting flood control and minimum flood requirements.
(Any one condition to exist for application of Schedule C or D)						
C	Above 15 R* or Above 25	Above 10 R* or Above 30	Above 40 R* or Above 60	Maximum Pool (757) to Spillway Crest (820)	1 December - 31 March 1 April - 30 November	- Release at a constant rate of 200 c.f.s. - Release at a constant rate of 200 c.f.s. when pool is above elevation prescribed by Rule Curve. When pool is at or below Rule Curve elevation release at rate of 40 cfs.
D	Same as Schedule C			Spillway Crest (820) to Flood Pool (840)	At Any Time	- Release in accordance with computed inflows using bypasses or conduit gates supplemented by spillway gates as necessary to effect prescribed outflow. If pool peaks and release rate has not exceeded 200 c.f.s. maintain this outflow until conditions permit regulation in accrodance with Schedule B. When release rate has been increased above 200 c.f.s. to peak the pool at elevation 840, maintain all gate settings attained (conduit and spillway) until pool recedes to spillway crest (820). At this time, reduce outflow gradually toward 3,500 c.f.s without causing rising pool conditions. When release is at 3,500 c.f.s., resume normal regulation as dictated by appropriate schedule.
E	Control Stations No Longer Considered			Above Flood Pool (840)	At Any Time	- Conduit and spillway are completely open and will remain open, passing flow uncontrolled until pool recedes to spillway crest (820), at which time gates will be closed and conduit gates regulated as in Schedule D.

*These stages applicable only when flood damage is anticipated; otherwise, continue regulation in accordance with schedule B. (Stages indicating anticipated flood damages - Lock 14, 20 feet - Lock 4, 20 feet - Cincinnati, 52 feet.)

MAXIMUM RELEASE

Tallega Stage (feet)	Release (c.f.s.)
7.5 or less	3,500
11.0 or less	2,500
15.0 or less	1,500
16.5 or less	800
Not Considered	200
Below 16.0	3,500

October 1995

Figure 4.6: Buckhorn Lake Schedule of Regulation

TABLE 6

CARR FORK LAKE
SCHEDULE FOR FLOOD CONTROL
WATER QUALITY CONTROL AND SEASONAL REGULATION

CONTROLLING FACTORS				
Schedule	Upper Gage Lock 14 Stage (feet)	Range in Pool Elevation (feet)	Period	Regulation
A	Below 15 or Below 25♯	1017*-1009 ∅	1 Dec - 31 Mar	Maintain water quality pool as near as possible provided release indicated in maximum release table is not exceeded and firm flows at Hazard are never less than 10 cfs in December and 5 cfs in January, February, and March, and outflow never less than 5 cfs.
B	Same as Schedule A	1017-1055**	1 Dec - 31 Mar	Release at rate indicated in Maximum Release table.
		1009-1055	1 Apr - 30 Nov	Regulate reservoir to maintain pool as near guide curve as possible while meeting flood control requirements with outflow never less than 5 cfs and firm flows at Hazard are never less than 22 cfs in April, May, and June; 25 cfs in July, August and September; 10 cfs in October and November.
C	Above 15♯ or Above 25	1009-1017	1 Jan - 31 Dec	Regulate reservoir to meet water quality requirements.
		1017-1055	1 Jan - 31 Dec	Release at constant rate of 100 cfs.
D	Control no longer considered at or above 1055		1 Jan - 31 Dec	Open conduit gates at rate of 2/10 per hour or until outflow equals inflow. Gates will remain at setting reached until pool recedes to spillway crest.

ALLOWABLE RELEASE TABLE

Natural Flow at Hazard (cfs)	Allowable Release (cfs)
Below 5000	2000
5000 - 8000	1600
8000 - 11,000	1200
11,000 - 14,000	800
14,000 - 17,000	400
Above 17,000	100

* Water quality pool
∅ Minimum pool
** Flood pool (spillway crest)
♯ These stages applicable only when flood damage is anticipated; otherwise, continue regulation in accordance with Schedule B.

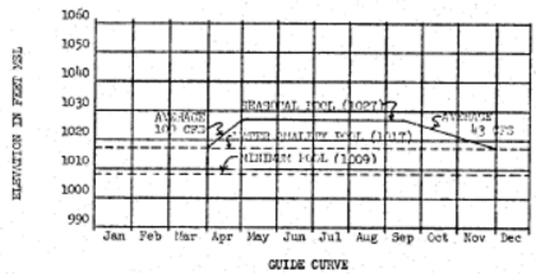

Figure 4.7: Carr Creek Lake Schedule of Regulation

4.1.5.2 Reservoir Re-Operation

Both Carr Creek and Buckhorn Dams utilize Heidelberg (Lock 14) as a downstream control point located about 6 miles downstream of Beattyville. WCMs specify the control point stage at 15 feet and 25 feet (falling/rising) to regulate releases. Heidelberg stage 25 feet gives an approximate discharge of 65,000 cfs which equates to about a stage 5 feet below the damage stage at Beattyville. Considering the long travel times and large unregulated areas between the reservoirs and Beattyville there is currently no need to re-examine the operation of the reservoirs.

4.1.6 Kentucky River Navigation Dams

The Kentucky River was dammed with a series of 14 low-head dams in the early 1900's for the purpose of navigation. The dams were constructed by USACE and have since been removed from the project portfolio. The Kentucky River Authority owns and operates Locks 5 through 14. Beattyville lies within the upper pool of Lock 14 at Heidelberg, KY. Under normal conditions, the Kentucky River water surface elevation at Beattyville is approximately the same as at Lock 14 (~634 Ft. NAVD88).

4.2 Flow Frequency Analysis

The Risk Cadre support team was engaged early in the study to perform a comprehensive flow frequency analysis (FFA) using the methods employed in risk analysis of flood risk management projects. Such analysis was necessary as the flow frequency analysis results will likely be applied to additional upcoming studies in the watershed.

Current guidance requires a Bulletin 17C flow frequency analysis, which recommends the analysis be performed on unregulated flow data. Regulation in the Kentucky River watershed complicates the analysis. The Risk support team utilizes multiple data sources to develop an unregulated-to-regulated relationship where an unregulated flow record is assembled for analysis with Bulletin 17C methodology. The Risk support team provides more accounting for uncertainties in data, flow to stage conversions, etc. than typical flood risk management study efforts. The results are then converted to regulated flows for use in the study. See the Risk Report on hydrology for more information.

Frequency flows at Beattyville were calculated by drainage area ratio using the Heidelberg flow frequency values. The Beattyville flow frequencies are included in this report for reference, but the modeling only targets the Heidelberg gage peak discharges.

Table 32 through Table 36 present the regulated frequency flows for the USGS gages at Heidelberg, Jackson, Tallega, and Booneville, KY.

Table 32: Regulated Frequency Flows on the Kentucky River at Heidelberg (Lock 14)

AEP	Return Period (Years)	95% Confidence Interval (cfs)	5% Confidence Interval (cfs)	Expected [Mean] Curve (cfs)	Computed [Median] Curve (cfs)
0.002	500	198,293	106,880	148,998	141,811
0.01	100	145,696	88,107	114,086	111,842
0.02	50	125,888	78,781	100,568	99,337
0.04	25	105,933	69,718	86,338	85,723
0.1	10	85,225	58,249	70,846	70,604
0.2	5	69,158	48,056	57,975	57,864
0.5	2	46,568	32,416	39,069	39,054
0.99	1	15,043	7,265	11,218	10,712

Table 33: Regulated Frequency Flows on the Kentucky River at Beattyville

AEP	*Return Period (Years)*	*95% Confidence Interval (cfs)*	*5% Confidence Interval (cfs)*	*Expected [Mean] Curve (cfs)*	*Computed [Median] Curve (cfs)*
0.002	500	196,353	105,834	147,540	140,423
0.01	100	144,270	87,245	112,970	110,748
0.02	50	124,656	78,010	99,584	98,365
0.04	25	104,896	69,036	85,493	84,884
0.1	10	84,391	57,679	70,153	69,913
0.2	5	68,481	47,586	57,408	57,298
0.5	2	46,112	32,099	38,686	38,672
0.99	1	14,896	7,194	11,108	10,607

Table 34: Regulated Frequency Flows on the North Fork at Jackson, KY

AEP	*Return Period (Years)*	*95% Confidence Interval (cfs)*	*5% Confidence Interval (cfs)*	*Expected [Mean] Curve (cfs)*	*Computed [Median] Curve (cfs)*
0.002	500	94,802	53,038	74,963	71,017
0.01	100	73,681	46,475	60,191	58,988
0.02	50	65,082	43,025	54,174	53,515
0.04	25	55,875	38,572	47,505	47,169
0.1	10	45,831	32,921	39,854	39,735
0.2	5	37,631	27,447	33,031	33,019
0.5	2	25,520	18,479	22,305	22,361
0.99	1	7,081	3,052	4,945	5,292

Table 35: Regulated Frequency Flows on the Middle Fork at Tallega, KY

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	28,250	15,110	21,049	18,748
0.01	100	19,730	12,637	15,591	14,856
0.02	50	16,761	11,540	13,700	13,281
0.04	25	13,932	10,289	11,817	11,601
0.1	10	11,138	8,844	9,854	9,766
0.2	5	9,137	7,564	8,294	8,252
0.5	2	6,568	5,595	6,057	6,067
0.99	1	3,207	2,272	2,813	2,698

Table 36: Regulated Frequency Flows on the South Fork at Booneville, KY

AEP	***Return Period (Years)***	***95% Confidence Interval (cfs)***	***5% Confidence Interval (cfs)***	***Expected [Mean] Curve (cfs)***	***Computed [Median] Curve (cfs)***
0.002	500	137,285	73,299	101,973	92,877
0.01	100	91,801	58,604	72,572	69,817
0.02	50	76,126	52,196	6,253	60,711
0.04	25	61,320	45,016	51,975	51,207
0.1	10	46,835	36,890	41,338	41,046
0.2	5	36,602	29,957	33,063	32,937
0.5	2	23,782	19,889	21,756	21,746
0.99	1	8,471	5,589	6,883	7,181

4.3 THE MARCH 2021 FLOODING OF BEATTYVILLE

4.3.1 The March 2021 Event

The storm event of March 2021 flooded Beattyville and was described by city officials as the worst seen in a generation in the downtown business area. City officials recall the river stage climbing so quickly that flow appeared to be rushing through the downtown from multiple directions at times. The event spurred investigative action from U.S. Congressional Representative Hal Rogers to evaluate flooding issues and reduction measures.

The flood began in late February of 2021 and extended into early March. The USGS gage at Heidelberg recorded a steep hydrograph rising limb with a measured peak discharge of 89,100 cfs and a peak water surface elevation of 659.1 Ft. NAVD88. See the event flow hydrograph in Figure 4-8. Surveys of highwater marks (HWMs) in Beattyville indicate an approximate peak water surface elevation (WSE) of 666.5 Ft. NAVD88.

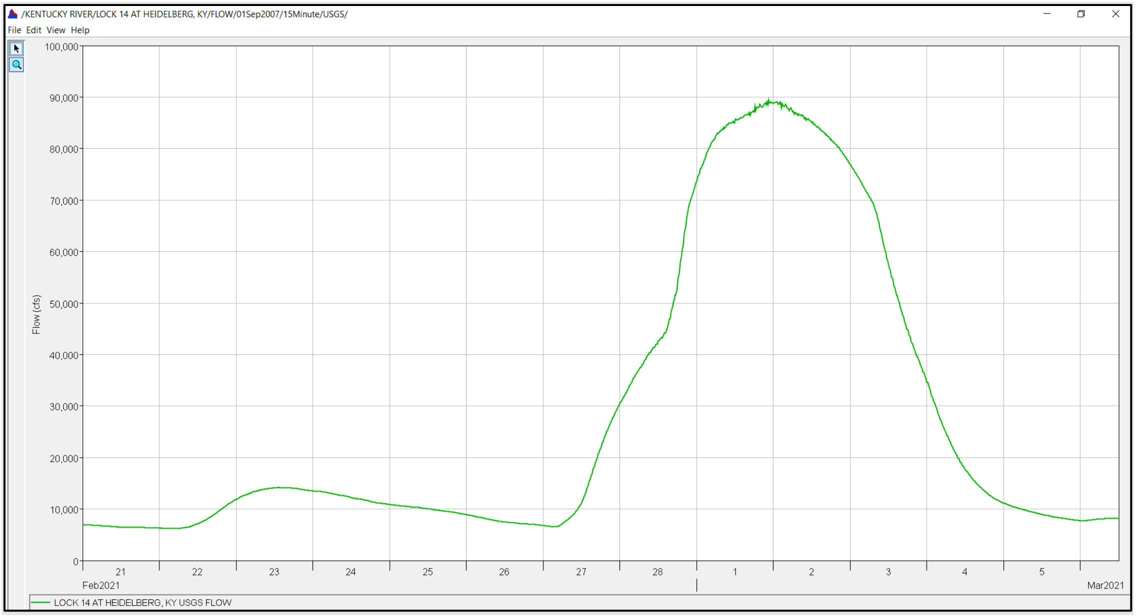

Figure 4.8: March 2021 Observed Flood Hydrograph at Heidelberg, KY

4.3.2 Discharge-Stage Discrepancy Identified

The March 2021 flood event inundated Beattyville to an approximate elevation of 666.5 Ft. NAVD88 (surveyed HWMs): however, the HEC-RAS model indicated the peak stage to be about 5 feet lower for the flow recorded downstream at Heidelberg, KY. The HEC-RAS model was calibrated to the USGS gage rating curves during HEC-CWMS model development and has held up well over years of use by Water Management. However, the March 2021 flood event recorded flow did not produce the recorded stage at the gage.

4.3.3 The USGS on the March 2021 Event

Discussions with the USGS Louisville Office gave more insight into the discharge-stage issues with the March 2021 high-flow event. USGS field personnel took flow measurements during the March 2021 event and stated the published data is accurate.

4.3.3.1 Slope Rating Curve on the Kentucky River

The USGS is confident in the published rating curve at Heidelberg (Lock 14) under normal circumstances. However, the gage at Lock 14 experiences a significant shift in the rating curve during high flows because of backwater. In general, most of the gages on the upper end of the Kentucky River main channel seem to begin showing deviation from the normal stage-discharge rating in the 40,000-50,000 cfs range.

The source of backwater on the Kentucky River is primarily the flow obstruction of the series of fourteen low-head dams themselves, but there are other factors. The lack of bed slope gradient, inflow tributaries, and overbank return flow are all likely sources that contribute to the backwater affect. At approximately 60,000 cfs, the rating curve transitions to a sloped rating. This forces a rating shift up then back down along the pro-rated curve.

The pro-rated curve refers a rating curve in development that will over time be used to finalize an updated rating curve for publishing. The pro-rated curve was applied to the March 2021 event using flow measurements taken at Lock 14 during the flood event. The pro-rated curve was used by the USGS in publishing the gage data for the event. That is, the observed March 2021 event curve (the published observed stage and flow hydrographs plotted together stage vs flow) captures the published rating curve and pro-rated curve together (see Figure 4-9). If the observed hydrograph data points (data from the USGS website) were converted to stage using only the published rating curve (the black plotted line), the pro-rated portion departing from the curve would not be captured (the red plotted line), and the converted higher flows would instead follow the black line yielding an incorrect stage. The is the case for the stage-flow discrepancy in Section 4.3.2 and explains why the HEC-RAS model failed to capture both the flow and stage at Lock 14 as well as the Beattyville highwater mark stage (recall the HEC-RAS model was originally calibrated to the published rating curve).

The USGS applied the pro-rated curve to the March 2021 event so the recorded data would be accurate for publishing. The sloped ratings have been typically developed using the upstream and downstream hydraulic structures (the low-head locks and dams) along the Kentucky River. A sloped rating is difficult to establish at Lock 14 because it is the most upstream low-head dam. The closest published slope-rated gage on the Kentucky River is at Lock 11 (which is being used as the downstream boundary condition in the HEC-RAS model update). For the March 2021 event, the USGS applied the measured flow/stage at Lock 14 and estimated the sloping water surface from Lock 11 through Locks 12 and 13 to produce the pro-rated curve at Lock 14.

As further validation, the USGS estimated the March 2021 event stage at Beattyville at 666.0 Ft. NAVD88 (within 0.5 feet of the surveyed HWMs) by extending the estimated slope from Lock 11 through Lock 14 to Beattyville.

Figure 4-9 shows rating curves at Lock 14. The black plotted line represents the published USGS rating curve while the red line shows the observed rating curve for the March 2021 event. The observed rating curve depicts the shift used by the USGS to capture the transition to the pro-rated slope rating for the March 2021 event. The transition appears abrupt, but this is understood to be a representation of a single event. Any future developed slope rating curve would likely be a best fit line for multiple events where the transition would be more gradual or completely blended into a new rating curve entirely, like the established slope rating curve at Lock 11 (shown later in Figure 4-22).

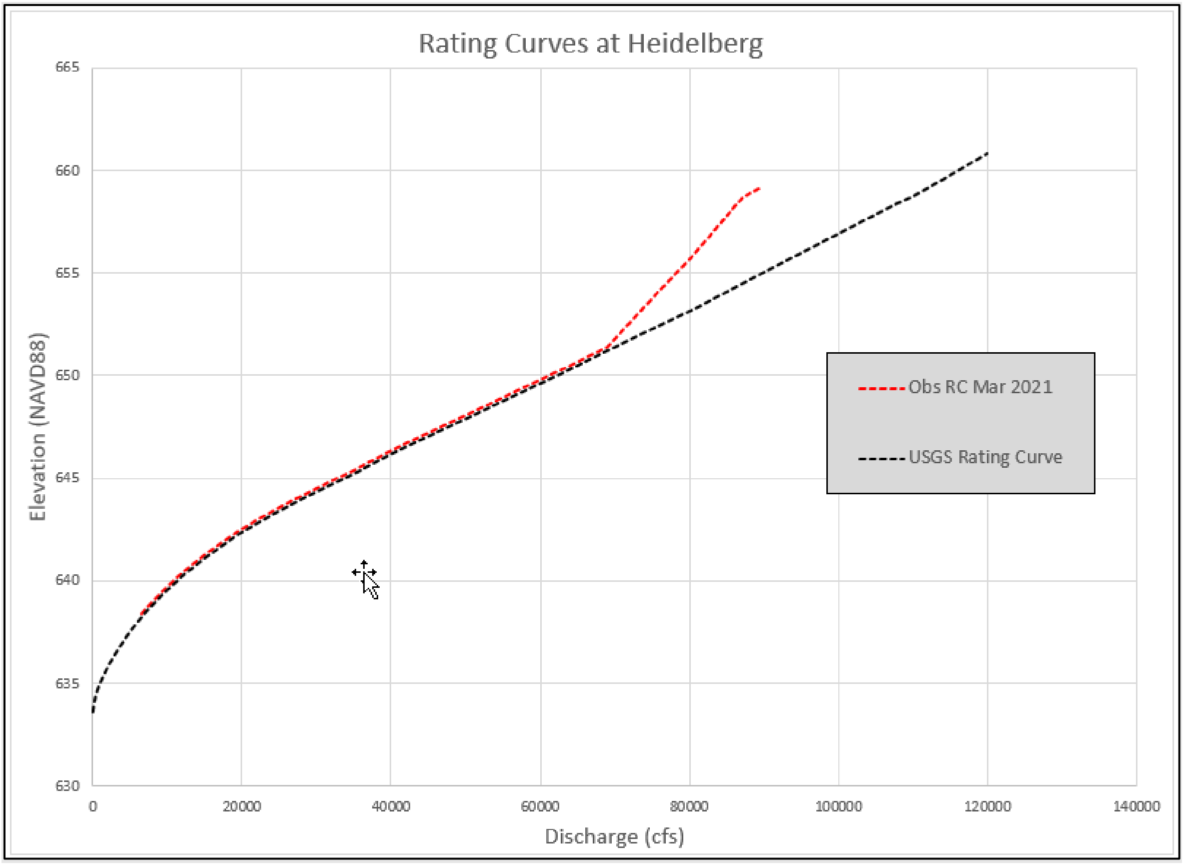

Figure 4.9: Lock 14 USGS Published Rating Curve and the Slope-Rated Curve for March 2021

4.4 Modeling Considerations and Version History

The Kentucky River Basin HEC-CWMS model was developed in 2016. The modeling component software (HEC-RAS, HEC-HMS, etc.) versions have been updated with the HEC-CWMS versions over time. However, software developers added Geographic Information System (GIS) features to HEC-HMS in 2021. While the current HEC-HMS version used with the HEC-CWMS model is version 4.10, it was originally developed with an HEC-HMS version that did not have GIS capabilities. That is, the ArcGIS was used outside HEC-HMS.

4.4.1 The HEC-CWMS Hydrologic Model

The HEC-HMS component of the HEC-CWMS model was developed to work with HEC-CWMS for daily Water Management use. HEC-CWMS is an application that links the modeling components which inherently complicates the modeling setup and data requirements. HEC-CWMS users (Water Management) prefer simplicity in the modeling for reliability reasons. Reasonably accurate results in most situations are sought for making daily reservoir release decisions.

For HEC-CWMS reliable functionality, the HEC-HMS and HEC-ResSim components require the use of the same routing routines for identical routing reaches within the models. The HEC-CWMS components use Muskingum routing routines for simplicity. However, natural parameter-based routing routines are more typical of detailed flood risk management projects. The HEC-HMS model for this project uses Muskingum-Cunge where channel slope, generalized shape, and roughness contribute to the attenuation of peak flows during reach routing.

4.4.2 HEC-ResSim Usage Excluded from Study

HEC-HMS is required to transform precipitation into runoff to assess local flows upstream and downstream of the reservoir because HEC-ResSim cannot. Positioning multiple storm events for assessing optimal storm position, or even creating a randomness for statistical analysis is not possible in HEC-ResSim. Such an analysis requires an HEC-WAT model development, which is outside the current scope and likely unnecessary. While HEC-ResSim boasts capabilities of making reservoir release decisions based on rule settings, those same rules would capture reservoir operations at minimum flow during large storm events. Minimum flow can be set in HEC-HMS for simulations of the storm events to adequately capture the reservoir discharges.

HEC-HMS and HEC-ResSim employ the same reach routing routines, and because of this, both fall short of detail required for this study. While the software programs serve adequately for daily Water Management reservoir operation decisions with the HEC-CWMS application, both fail to capture the backwater affects in the system due to the downstream low-head locks and dams.

HEC-ResSim can incorporate rating curve that gives a stage for a given flow/downstream pool combination. However, such a rating curve would need to be developed through HEC-RAS simulations thus extending the analysis already completed by the HEC-HMS/RAS combination. The use of modified Puls could be employed in the HEC-HMS or HEC-ResSim models, but again require the use of HEC-RAS to develop the proper stage-discharge curves, also additional calibration would be required. When HEC-HMS is used in conjunction with HEC-RAS, a solid case for the HEC-HMS/HEC-RAS combination is made versus HEC-ResSim.

4.4.3 Watershed Re-Delineation

The delineation process was repeated to further delineate (or break up) watersheds and recalculate basin parameters for the study. Prior to the Alternative Milestone Meeting (AMM), alternative dam sites were investigated within the watershed which required the full capabilities of the current HEC-HMS version. The HEC-HMS component of the HEC-CWMS model was updated by re-delineating the watershed using the same subbasin outlets. The subbasins were then further broken down to provide inflows at areas of interest for this study (such as the flows at Beattyville from the North and South Forks separately). Further delineation of smaller subbasins for added detail means few subbasins remain the same as the original HEC-CWMS model component making direct parameter comparison cumbersome. For anticipated future studies, the subbasin above the Whitesburg gage was added. Routing reach routines were changed to Muskingum-Cunge, as stated in Section 4.4.1. This became the new base HEC-HMS model from which frequency storms were modeled and the alternative dam sites were investigated.

4.4.3.1 Dam Sites Investigated

The alternative dam sites basin models are not included in the existing conditions HEC-HMS model. Dam sites were investigated on the North Fork at St. Helens, Jackson, Hazard, and on Troublesome Creek; all sites were ineffective at preventing the 1% AEP flood at Beattyville.

The previously authorized reservoir project at Booneville on the South Fork was modeled and was shown to prevent flooding at Beattyville (project classified inactive in 1976). Regulating the large South Fork drainage area approximately 7.5 miles upstream of Beattyville effectively prevents the development of the sloped water surface at Heidelberg for the 1% AEP frequency flow where the published rating curve shifts upward along the pro-rated curve as discussed in Section 4.3.3.1. The Booneville Dam was hydrologically modeled in HEC-HMS and then discharges routed in the HEC-RAS model as part of the investigation.

The PDT screened out reservoir alternatives prior to the AMM. The dam site modeling is not included as part of this report.

4.4.4 Overall Modeling Approach

The updated HEC-HMS model was calibrated to the March 2021 flood event (discussed in Section 4.6.1). No reliable high peak flow validation event was available with both stream gage records and precipitation. Hydrologic modeling efforts proceeded to frequency storm simulations using Atlas 14 annual maximum series precipitation data.

The calibration and frequency storm events were applied to the HEC-RAS hydraulic model. The inflow hydrographs were scaled to reach the calibration and frequency flow targets at Lock 14. It should be noted that while the March 2021 flood was used for calibration event, calibration was focused on the Lock 14 rating curve with provisional rating (pro-rated) and the highwater marks surveyed at Beattyville (discussed in Section 4.3.3.1). There is more confidence in rating curve up to the peak discharge of the March 2021 event, after which uncertainty in stage-discharge relationship increases.

For this General Investigation, the HEC-HMS model supplies inflow hydrographs from the Forks for the area of interest (Beattyville). It is reasonable to simply the modeling simulations by targeting the FFA peak discharges at Lock 14 because the analysis is based on streamflow records at that gage, regardless upon which Forks contributed to the annual peak discharges in the historical record.

Two approaches were considered for frequency storm modeling. Approach 1 involved adjusting the HEC-HMS loss parameters targeting the computed FFA 1% AEP peak discharges at the four gages nearest Beattyville (at Jackson, Tallega, Booneville, and Lock 14). The HEC-HMS results were then applied to the HEC-RAS model. The inflow hydrographs above each gage were scaled in HEC-RAS targeting the 1% AEP peak discharges.

Approach 2 was to simplify efforts by applying uniform loss parameters across the entire watershed in HEC-HMS, then scaling the inflows in HEC-RAS uniformly only targeting the 1% AEP peak discharge at Lock 14 gage. This approach simplifies the task of iterating HEC-RAS simulations to ascertain the correct inflow hydrograph scaling factor, where the first approach iterates simulations for each gage individually.

The two approaches were compared for sensitivities near the study area. The difference in stage between the two approaches did not exceed +/-0.25 feet, which is within the margin of error of the hydraulic modeling software (see Table 37).

Table 37: Stage Sensitivity in Modeling Approaches for the 1% AEP Simulation

Stream	River Station	Approach 1 Stage (Ft.)	Approach 2 Stage (Ft.)	Difference Stage (Ft.)
North Fork Kentucky River	0.101	672.42	672.29	-0.13
North Fork Kentucky River	0.262	672.65	672.55	-0.10
South Fork Kentucky River	0.105	672.42	672.29	-0.13
South Fork Kentucky River	0.183	672.58	672.37	-0.21
Kentucky River	257.5170	663.90	663.96	0.06
Kentucky River	251.6741	672.42	672.29	-0.13

Approach 2, scaling the uniform loss inflows was chosen for simplicity. The effort in additional simulation iterations is not warranted given such small potential improvements in results.

More discussion of the HEC-HMS uniform loss parameters can be found in Section 4.7.1.1.

4.5 DATA COLLECTION

4.5.1 Vertical Datum

The vertical datum is referenced in this report are in the North American Vertical Datum of 1988 (NAVD88) in units of feet (Ft).

USACE dams still reference their native project datum using the National Geodetic Vertical Datum of 1929 (NGVD29). Consult the National Geodetic Survey for site-specific conversions to NAVD88. For the Beattyville study area, use the following formula to convert to NAVD88:

Ft. NAVD88 = Ft. NVGD29 + (-0.54 feet)

4.5.2 GIS Terrain and Layers

Terrain data for the model is a 10-foot resolution DEM resampled KY from Above statewide 5-foot DEM dataset.

4.5.3 Existing Data and Supporting Analysis

4.5.3.1 Stream Gage Data

The stream gage data analyzed for this study was 15-minute and annual peak discharge data obtained from the USGS streamflow gages shown in Table 38.

Table 38: USGS Stream Gages Used

Gage Name	USGS Gage No.	NWS Gage Identification	Drainage Area (SQMI)
South Fork Kentucky River at Booneville, KY	03281500	BOOK2	722
Goose Creek at Manchester, KY	03281100	MCHK2	163
Middle Fork Kentucky River at Tallega, KY	03281000	TLLK2	537
North Fork Kentucky River at Jackson, KY	03280000	JKNK2	1,101
North Fork Kentucky River at Hazard, KY	03277500	HAZK2	466
North Fork Kentucky River at Whitesburg, KY	03277300	WHTK2	66
Kentucky River at Lock 13 Near Willow Shoals, KY	03282060	WLWK2	2,784
Kentucky River at Lock 14 at Heidelberg, KY	03282000	HLDK2	2,657

4.5.3.2 Dam Discharge Data

Discharges for Carr Creek Dam and Buckhorn Dam were estimated using two different approaches:

4.5.3.2.1 Calibration Event

Discharge flows from the USACE dams are maintained by Water Management (WM). Outflows are calculated based on the gate rating curves using the water surface elevation (WSE) at the lake and the recorded gate opening heights. The outflow records reside in the WM database and were used to estimate flows at Carr Creek Dam and Buckhorn Dam for the 2021 calibration event.

4.5.3.2.2 Frequency Storm Modeling

For the frequency storm modeling, minimum outflows from the dams were used for downstream flooding simulations, per the Water Control Manual Schedules of Regulation. See Figure 4-6 and Figure 4-7 for dam Schedules of Regulation.

4.5.3.3 Precipitation

Gridded precipitation data was used for calibration events of the model due to its availability and to provide better temporal and spatial accuracy. Gridded radar precipitation is in 1-hour increments and was provided from the Hydrologic Engineering Center's Corps Water Management System (HEC-CWMS) database, which is Stage 3 data ingested from the National Weather Service (NWS).

Atlas 14 gridded precipitation-frequency data based on the annual maximum series (AMS) were used for this study. Atlas 14 is an official peer-reviewed record of precipitation frequency estimates produced by the National Weather Service (NWS) Office of Water Prediction, part of Hydrologic Modeling Efforts.

The Water Management team utilizes the Hydrologic Engineering Center's (HEC) Corps Water Management Software (CWMS) in daily reservoir operations. The HEC-CWMS model contains three main components, all part of the HEC software suite: Hydrologic Modeling Software (HEC-HMS) for watershed modeling, River Analysis System (HEC-RAS) for hydraulic modeling, and Reservoir System Simulator (HEC-ResSim) for reservoir simulations.

The Kentucky River Basin HEC-CWMS model was developed for the Kentucky River basin in 2016, with the report completed in 2018.

The respective components of HEC-HMS and HEC-RAS were used as the base for the Beattyville General Investigation hydrologic and hydraulic models.

The hydrologic component of the HEC-CWMS model is used by Water Management team is in earlier versions of HEC-HMS. The study effort prior to the Alternative Milestone Meeting (AMM) called for exploring various potential locations for reservoirs. The HEC-HMS version was upgraded to v4.10 for the dam site exploration and general ease of compatibility for the remainder of the study. The upgrade required the re-delineation of the watershed using the subbasins already established in the original HEC-HMS model.

4.5.4 General Hydrologic Methodology for Basin Analysis

The purpose of this section is to describe the efforts performed to estimate the physical hydrologic parameters for the Kentucky River watershed. The proceeding sub-sections outlines efforts performed to delineate the watershed with subbasins and determine hydrologic elements within the watershed. HEC-HMS, version 4.10 was used during this effort.

The 8 gages from Table 38, Buckhorn Dam, and Carr Creek Dam are shown in Figure 4-10.

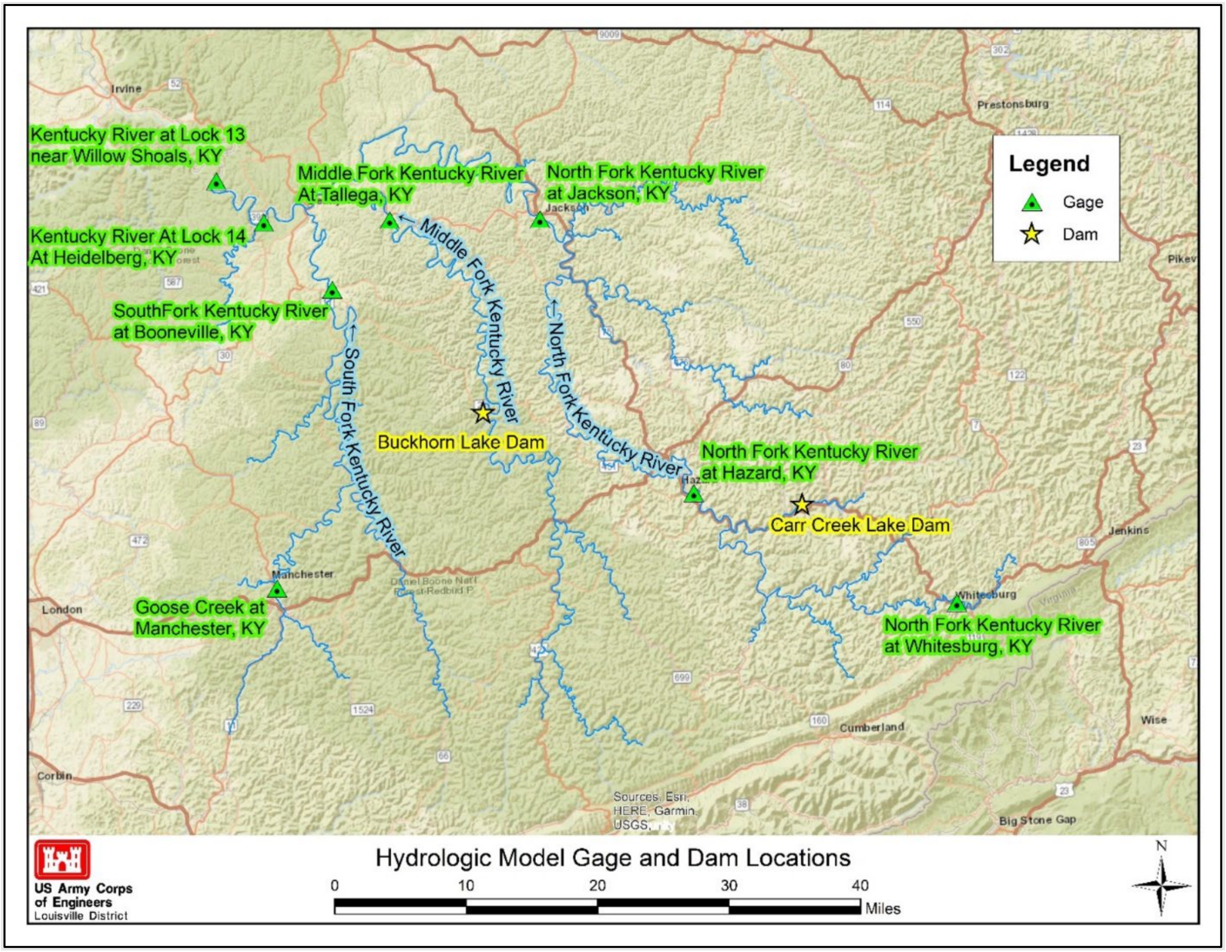

Figure 4.10: Hydrologic Model Gage and Dam Locations

4.5.5 General HEC-HMS Model Development

An HEC-HMS model was created using three main components: A basin model, a meteorologic model, and control specifications. The parameters used for the basin model is shown in Table 39 and Table 40.

Table 39: Summary of Parameters of Hydrologic Calibration Modeling

Modeling Method	Parameter	Description
Deficit and Constant Loss	Initial Deficit	The initial deficit defines the volume of water that is to be required to fill the soil layer at the start of the simulation.
	Constant Rate	The constant rate defines the rate at which precipitation will be infiltrated into the soil layer after the initial deficit has been satisfied in addition to the rate at which percolation occurs once the soil layer is saturated. Typically, this parameter is equated with the saturated hydraulic conductivity of the soil.
	Maximum Deficit	Maximum Deficit specifies the total amount of water the soil layer can hold.
	Percent Impervious Area	Impervious area directly connected to the channel network (no losses are computed).
ModClark Transform Method	Time of Concentration	Defines the maximum travel time within a subbasin.
	Storage Coefficient	The storage coefficient is used in the linear reservoir for each grid cell.
Recession Baseflow	Initial Discharge	Initial baseflow discharge
	Recession Constant	The recession constant describes the rate at which baseflow recedes between storm events. It is defined as the ratio of baseflow at the current time, to the baseflow one day earlier.
	Ratio to Peak	The baseflow is reset when the current flow divided by the peak flow falls to a specified ratio value.
Muskingum-Cunge Routing	Length	Total length of the reach element, computed within HEC-HMS.
	Slope	Average slope for the reach.
	Manning's "n"	Manning's "n" roughness coefficients were selected based on previous hydraulic modeling or judgment using aerial photography.
	Shape	The trapezoidal cross-section was selected as the shape of the reach, based on cross sections of the previous hydraulic modeling and/or survey data.

Table 40: Summary of Parameters for Hydrologic Frequency Flow Modeling in HEC-HMS

Modeling Method	**Parameter**	**Description**
Frequency Storm	Storm Duration	The storm duration determines how long the precipitation will last.
	Intensity Duration	The intensity duration specifies the shortest time period of the storm.
	Intensity Position	The intensity position determines where in the storm the period of peak intensity will occur, thus how the depth is distributed during a storm.
	Area Reduction	The area reduction determines the area-reduction curves for reducing point precipitation, precipitation at a gage, to precipitation over a storm area.

4.5.6 Delineation

Using GIS capabilities within HEC-HMS and the terrain from Section 4.5.2, the relatively larger subbasins along North Fork, Middle Fork, and South Fork were further delineated to reduce their drainage area.

This is to provide additional lateral inflow locations for the hydraulic model. The delineated subbasins are shown in Table 41 and Figure 4-11.

The CWMS HEC-HMS model was trimmed by approximately 2,950 square miles downstream so that the most downstream portion is at Kentucky River at Lock 13 near Willow Shoals, KY. Some HEC-CWMS subbasins were broken down into smaller subbasins where additional inflow information was needed for the Beattyville study.

Table 41: Subbasins Calibrated for the HEC-HMS Model

Subbasin	Drainage Area (SQMI)	Subbasin	Drainage Area (SQMI)
BOOK2	236.4	LOST	42.3
BUCK2	157.3	MCHK2	163.3
CFLK2	60.5	ODAK2	167.1
HAZK2	338.6	ODAK2E	155.1
HAZK2E	66.5	QUICK	162.1
HLDK2C	16.8	QUICK_SF	40.3
HLDK2E	218.4	TLLK2	120.2
HLDK2W	57.3	TRBLE	24.9
HYDK2	203.0	TRBLW	1.9
JKNK2	12.9	WLWK2	111.2
JKNK2E	176.8	WLWK2N	15.8
JKNK2S	173.6	WTNK2	62.0
		Total Area	2784.4

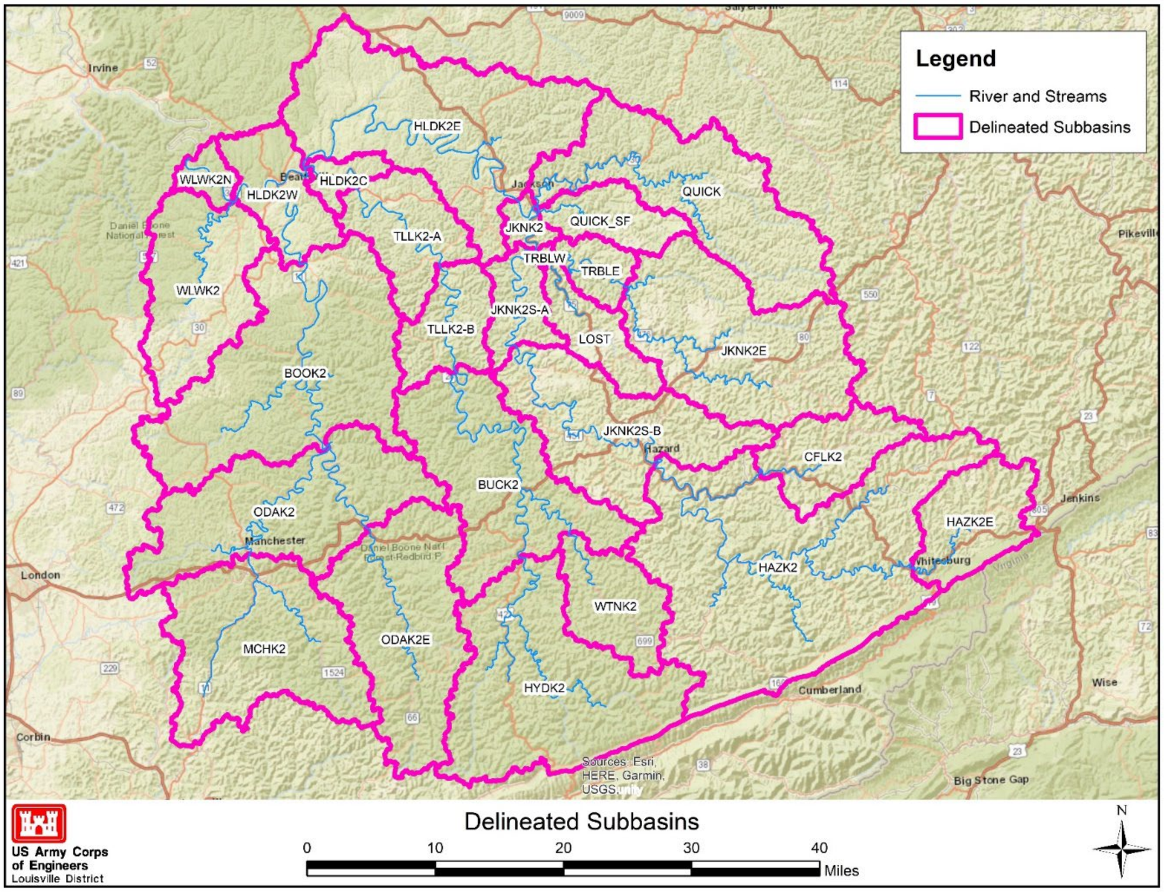

Figure 4.11: Subbasin Delineation

4.5.7 Estimated Loss Parameters

Loss rate parameters were computed using the Deficit and Constant Loss method. Initial estimates for subbasin losses were selected from the original HEC-CWMS subbasins and adjusted during calibration.

4.5.8 Clark Unit Hydrograph Parameters

Initial time of concentration (Tc) estimates for the new delineated subbasins were calculated from the Soil Conservation Service (SCS), now referred to as the Natural Resource Conservation Service (NRCS), curve number equation adopted from the NRCS National Engineering Handbook, Part 630 Hydrology, Chapter 15 Time of Concentration. The equations are shown in Equation 1 and Equation 2.

Equation 1: Estimated Time of Concentration

$$\boldsymbol{T_c = (\frac{L^{0.8} * (S+1)^{0.7}}{1,140Y^{0.5}})^{0.3}}$$

where,

$T_c = time\ of\ concentration (hours)$

$L = Longest\ flow\ path\ (ft)$

$S = maximum\ potential\ retention\ (in)$

$Y = average\ watershed\ land\ slope\ (\%)$

Equation 2: Maximum Potential Retention

$$\boldsymbol{S = \frac{1000}{CN} - 10}$$

where,
$S = maximum\ potential\ retention\ (in)$
$CN = SCS\ Curve\ Number$

A curve number raster was assembled using soil and land use data according to procedures from the HEC product online manuals. The USDA publishes soil data with the NRCS Soil Survey Geographic (SSURGO), from which the 2017 Kentucky dataset was used. The National Land Cover Dataset (2019) and Percent Impervious Surface (2019) raster files were obtained from the Multi-Resolution Land Characteristics (MRLC) Consortium. The average values were computed in HEC-HMS for each subbasin using percent impervious area and curve number input raster files. The curve number raster was then used to calculate the Time of Concentration with Equations 1 and 2.

The Modified Clark (ModClark) unit hydrograph storage coefficient (R) is a calibration parameter that affects the shape of the runoff hydrograph and is derived using observed rainfall runoff data. The ratio of Clark's unit hydrograph parameters, as shown in Equation 3 tends to be a regional value (USACE, EM 1110-2-1417). The ratio value from the HEC-CWMS model was carried over to the newly delineated subbasins and the storage coefficient was updated respectively.

Equation 3: Clark Unit Hydrograph Ratio

$$\boldsymbol{\frac{R}{T_c + R} = Ratio}$$

4.5.9 Estimated Baseflow Parameters

In the HEC-HMS model, an approximation of the baseflow contribution to the total flow hydrograph is made using the recession method.

The recession baseflow method is designed to approximate the typical behavior observed in watersheds when channel flow recedes exponentially after a rain event. This method is appropriate for this modeling effort because it is primarily intended for use in event-based simulation. Three parameters were defined for baseflow recession computations in the HEC-HMS model: Initial discharge, recession constant, and a ratio to peak value. The initial discharge comes from the observed inflow. The recession constant and ratio to peak value carried over from the HEC-CWMS model.

4.5.10 Estimated Muskingum-Cunge Parameters

The HEC-CWMS model utilizes Muskingum routing methods for computational stability. When HEC-CWMS is in daily use, the HEC-ResSim and HEC-HMS routing routines must be the same for corresponding stream reaches and so Muskingum is favored for stability. For the Beattyville effort, the HEC-HMS routing routines were updated to Muskingum-Cunge to favor more natural parameters such as length, slope, and channel roughness. The Muskingum-Cunge routing method is based on the combination of the conservation of mass and the diffusion representation of the conservation of momentum.

The length and slope of the reach was computed through basin characteristics in HEC-HMS initially. The slopes were updated using the channel bottom survey from the Kentucky River Basin Profile for the Flood of January-February 1957 plans, dated 1967.

The Manning's n value was determined from either judgement using aerial imagery or the value within the current HEC-CWMS HEC-RAS model component.

An Index celerity of 5 ft/s was chosen for the index method.

The channel was defined using a trapezoidal channel. The data for the channel was obtained from representative cross-sections from the HEC-CWMS HEC-RAS model component. Where such data was not available, values were selected based on engineering judgement of the available data.

4.5.11 Time Zone Considerations

The gridded precipitation datasets are provided in Coordinated Universal Time (UTC) and the observed inflows are provided in local time, Eastern Standard Time (EST). The gridded precipitation was adjusted depending on the time of year relating to daylight savings time so that the simulated and observed data would be presented in the same time zone.

4.6 Basin Model Calibration

The purpose of this section is to describe the efforts for calibration and validation of the basin.

4.6.1 Selection of a Calibration Storm Event

The March 2021 storm event (shown in Table 42) was chosen for calibration based on the recent available precipitation gridded data, stream flow records, and availability of highwater marks. Note that the HEC-CWMS model was updated in 2016 and calibrated to the and April 2011, August 2011, and July 2015 events. In 2019, the HEC-HMS model component was re-calibrated using continuous calibration methods. The 2019 HEC-CWMS calibrations were leveraged to the validity of this model as a starting point for this study. The continuous calibration is preferred by Water Management due to suitability for most daily simulations.

4.6.1.1 Observed Inflows

The observed discharges from the USGS gages in Table 38 were used for this study. The observed discharges were compared against the discharges of the subbasin hydrologic elements in the model.

4.6.1.2 Observed Outflows

The observed outflows recorded for Carr Creek Dam and Buckhorn Dam, as mentioned in Section 4.5.3.2, were used for the respective Dams. Note that inflows upstream of the respective reservoirs were routed to their own sinks and not routed downstream into the project area. A source element was used as the dam outlet to route the observed discharge data into the project area.

Table 42: Period of Record Used for Calibration

Event Date	Observed Peak Discharge at Tallega, KY	Observed Peak Discharge at Jackson, KY	Observed Peak Discharge at Booneville, KY	Control Specification	
	(CFS)	(CFS)	(CFS)	Start	End
Feb-March 2021	16,900	41,300	62,700	25FEB2021 00:00	10MAR2021 12:00

4.6.2 Calibration Techniques

Calibration was conducted by first optimizing transform, loss rates, and baseflow parameters against the observed inflows. The calibration for the model is based on single event modeling with emphasis on peak discharge as opposed to continuous simulation. The objectives of single-event modeling are better suited in determining the peak flow rate and timing, flow volume, and recession curve shape (Moriasi, 2007).

The gages listed in Table 38 were used for calibration. The model was calibrated until the model had a performance of “Satisfactory” or better, referencing Table 43 which shows the performance rating from Moriasi, 2007.

Table 43: Recommended Performance Ratings (Source: Moriasi, 2007)

Performance Rating	RSR	NSE	PBIAS for Streamflow (%)
Very Good	0.00 ≤ RSR ≤ 0.50	0.75 < NSE ≤ 1.00	PBIAS < ±10
Good	0.50 < RSR ≤ 0.60	0.65 < NSE ≤ 0.75	±10 ≤ PBIAS < ±15
Satisfactory	0.60 < RSR ≤ 0.70	0.50 < NSE ≤ 0.65	±15 ≤ PBIAS < ±25
Unsatisfactory	0.70 < RSR	NSE ≤ 0.5	±25 ≤ PBIAS

where,

$RSR = Root\ Mean\ Square\ of\ Observations\ Standard\ Deviation\ Ratio$

$NSE = Nash - Sutcliffe\ Efficiency$

$PBIAS = Percent\ Bias$

4.6.3 Calibration Results

The March 2021 storm event exhibited much higher flows in the watershed than what was assumed in the continuously calibrated HEC-CWMS modeling. The most significant change to watershed parameters were with the ModClark transform parameters. The calculated time of concentration discussed in Section 4.5.8 was reasonably close to the HEC-CWMS values (given the watershed drainage areas); however, no direct comparison could easily be made since the updated model further reduced the size of most subbasins. The final calibrated transform parameters used both the initial calculated and the HEC-CWMS values as range guides.

There was no validation storm event simulated for the calibration due to scarcity of data on such representative and infrequent events. The 2016 HEC-CWMS calibration of the HEC-HMS component attempted to validate with multiple events, few of which were satisfactory. The model has since been re-calibrated using the continuous calibration approach, which at best, inform Water Management decisions. Event validation is a moving target in general, but more so in this area where the time of concentration and storage coefficients can change significantly depend on season and antecedent conditions. The steep terrain of the Upper Kentucky River basin is a good candidate for variable time of concentration values touted in future HEC-HMS versions.

The final calibration results are summarized in Table 44 and the modeled calibrated graphs are shown in Figure 4-12 to Figure 4-19. Missing observed data for Lock 13 (Willow Shoals) is likely impacting performance ratings for that element. Note that the calibration for Middle Fork Kentucky River at Tallega, KY (Figure 4-15) did not meet the satisfactory performance ratings from Table 43; this is because Muskingum-Cunge routing in HEC-HMS cannot compute backwater. It is assumed that Middle Fork experienced backwater from the North Fork because the subbasin (TLLK2) fell well short of the observed peak discharge with zero constant losses applied, and therefore the performance ratings were ignored.

Negative values for Percent Bias performance rating indicates the observed discharge volume exceeds the calculated hydrograph discharge volume in the model. However, the rating takes in to account the full period of simulation where variations in flow before and after the peak impacts the rating. The peak discharge, timing, and general shape indicate an otherwise good calibration. Note that HEC-HMS did not report volume for some locations and is denoted by 'n/a.'

Table 44: Summary of Calibration Results

	Unit	Whitesburg	Hazard	Jackson	Tallega	Manchester	Booneville	Lock 14 at Heidelberg	Lock 13 at Willow Shoals
Computed Peak	**CFS**	3,640	18,440	41,348	12,353	14,225	60,065	110,298	115,182
Observed Peak	**CFS**	3,680	18,400	41,300	16,900	13,100	62,700	89,400	88,900
Computed Volume	**IN**	3.80	n/a	n/a	n/a	5.05	6.19	n/a	n/a
Observed Volume	**IN**	4.57	n/a	n/a	n/a	6.44	7.06	n/a	n/a
Performance Ratings									
RMSE Std Dev:		0.3	0.4	0.3	1.1	0.3	0.2	0.4	0.3
Percent Bias:	%	-17.92	-27.46	-25.82	-68.79	-21.46	-12.29	-21.61	-25.17
Nash-Sutcliffe		0.915	0.855	0.910	0.177	0.922	0.943	0.851	0.893

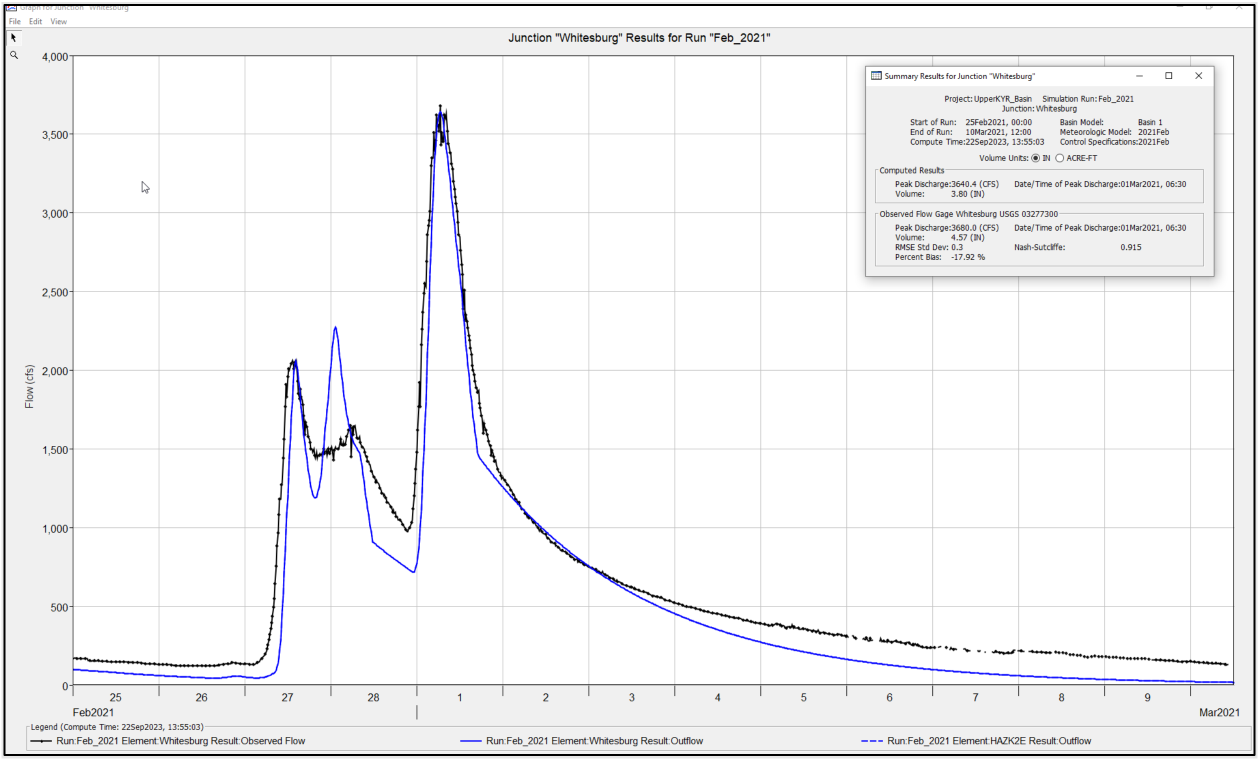

Figure 4.12: North Fork Kentucky River at Whitesburg, KY Calibration Results

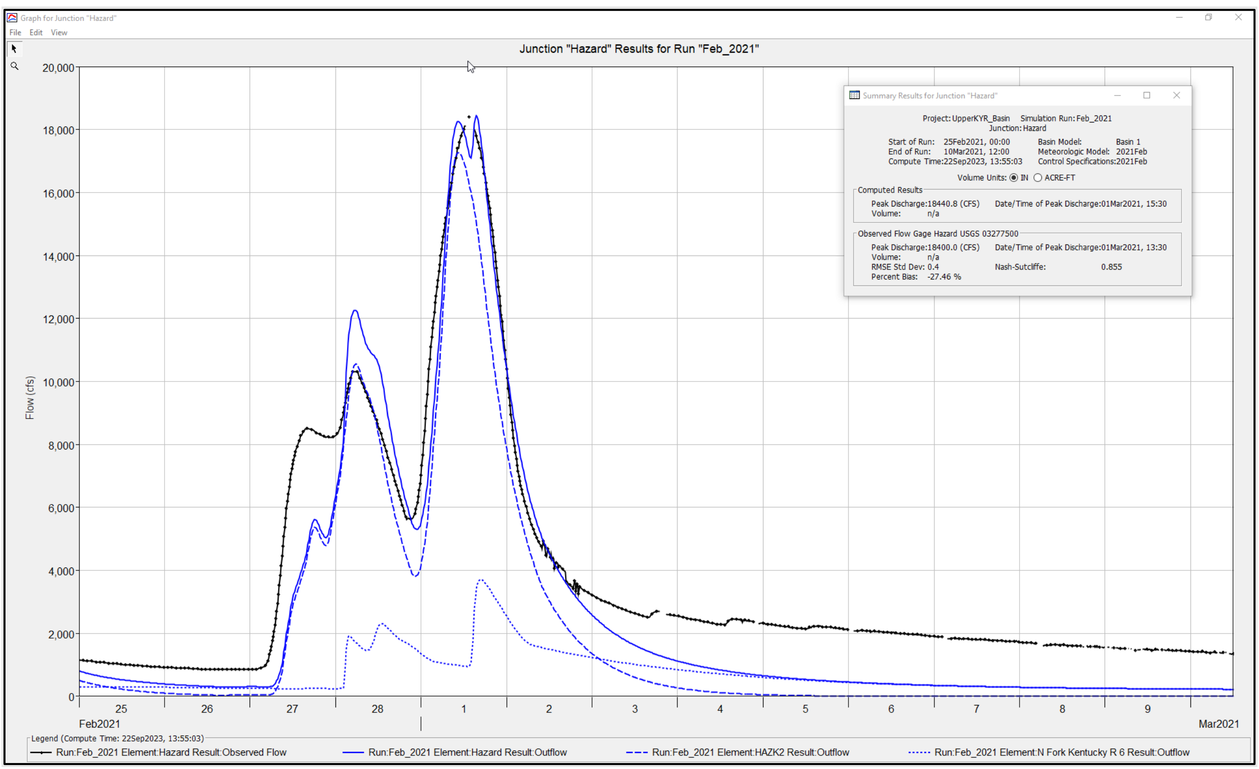

Figure 4.13: North Fork Kentucky River at Hazard, KY Calibration Results

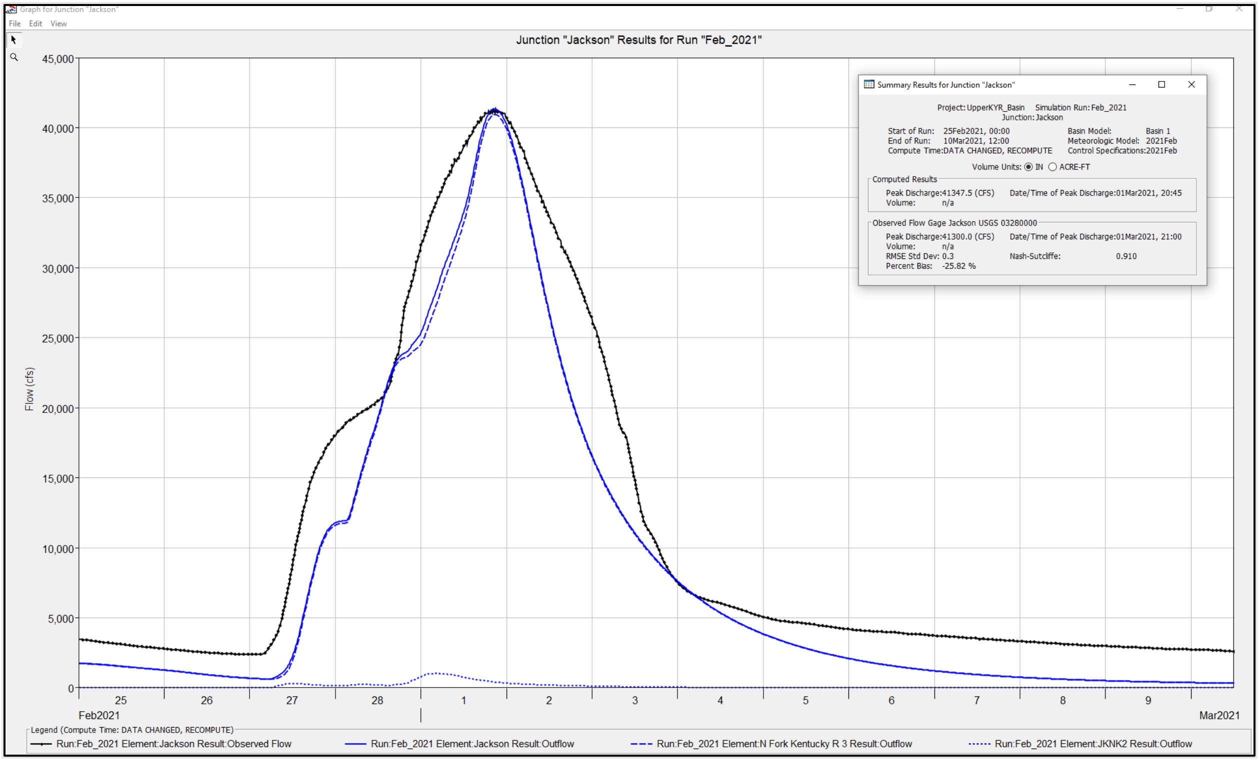

Figure 4.14: North Fork Kentucky River at Jackson, KY Calibration Results

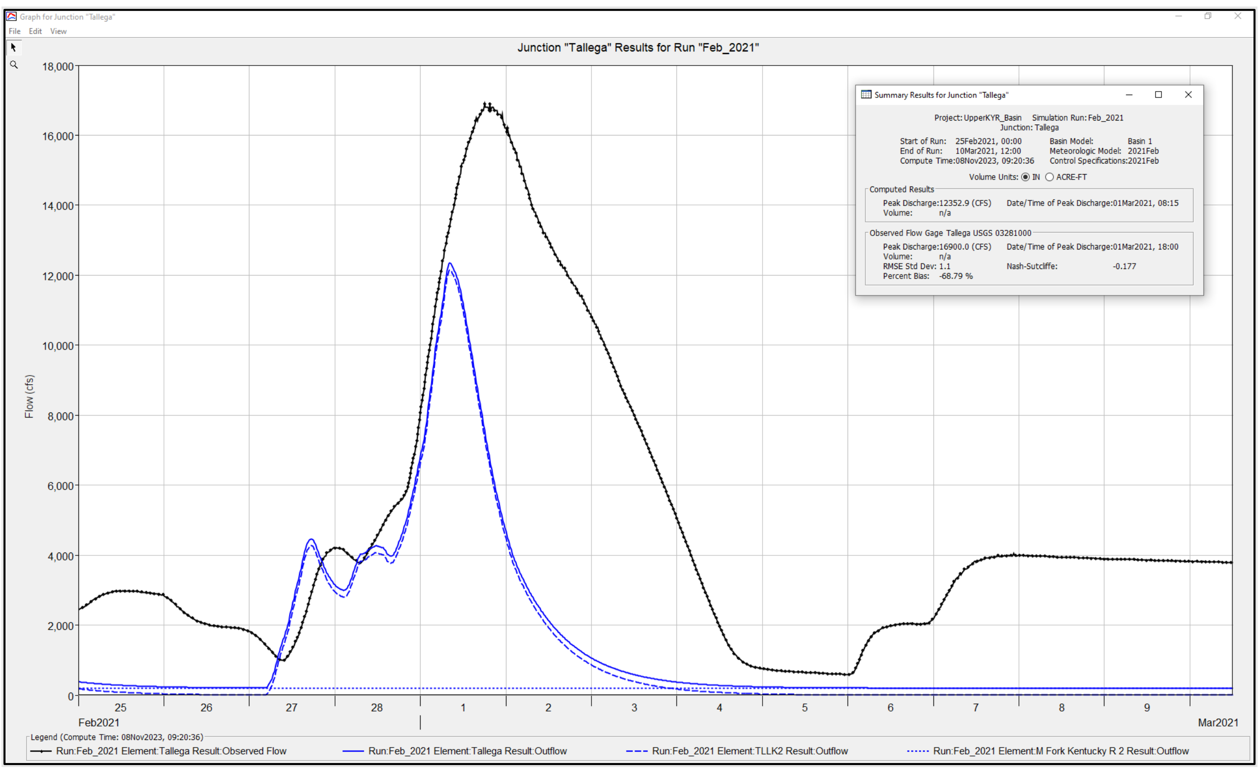

Figure 4.15: Middle Fork Kentucky River at Tallega, KY Calibration Results

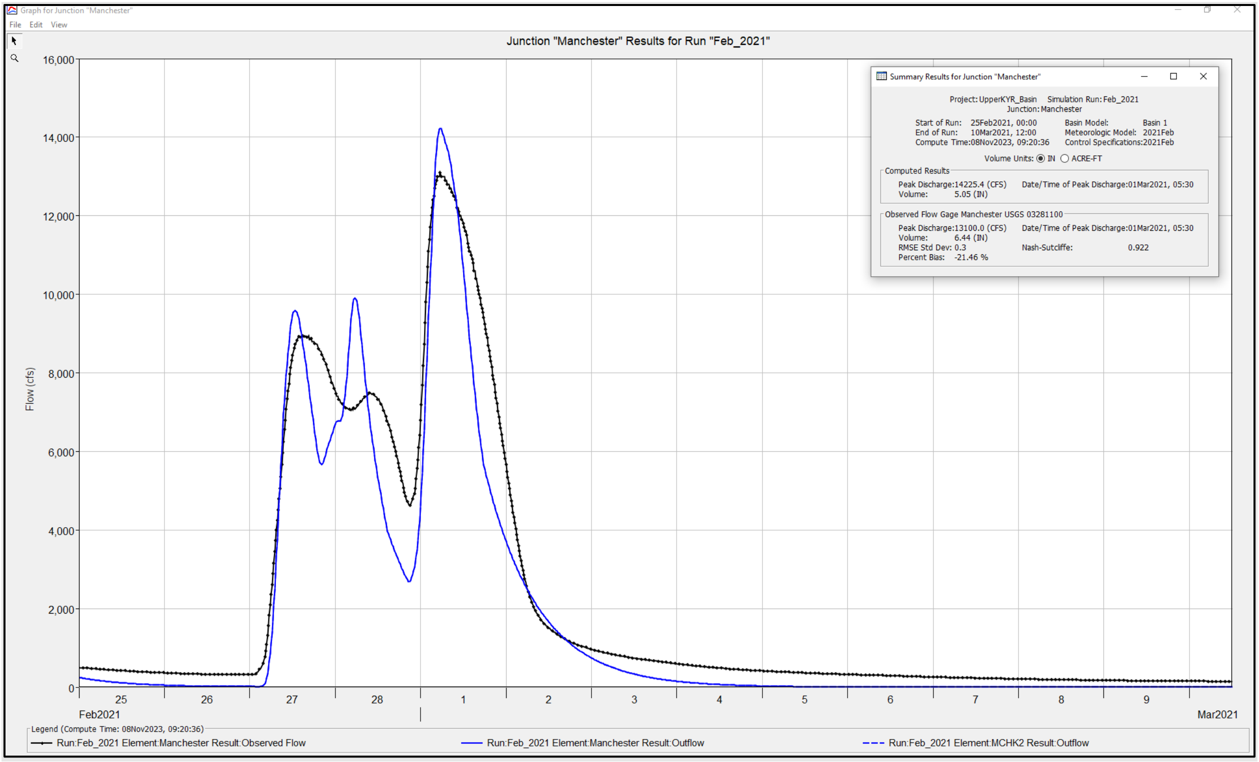

Figure 4.16: Goose Creek at Manchester, KY Calibration Results

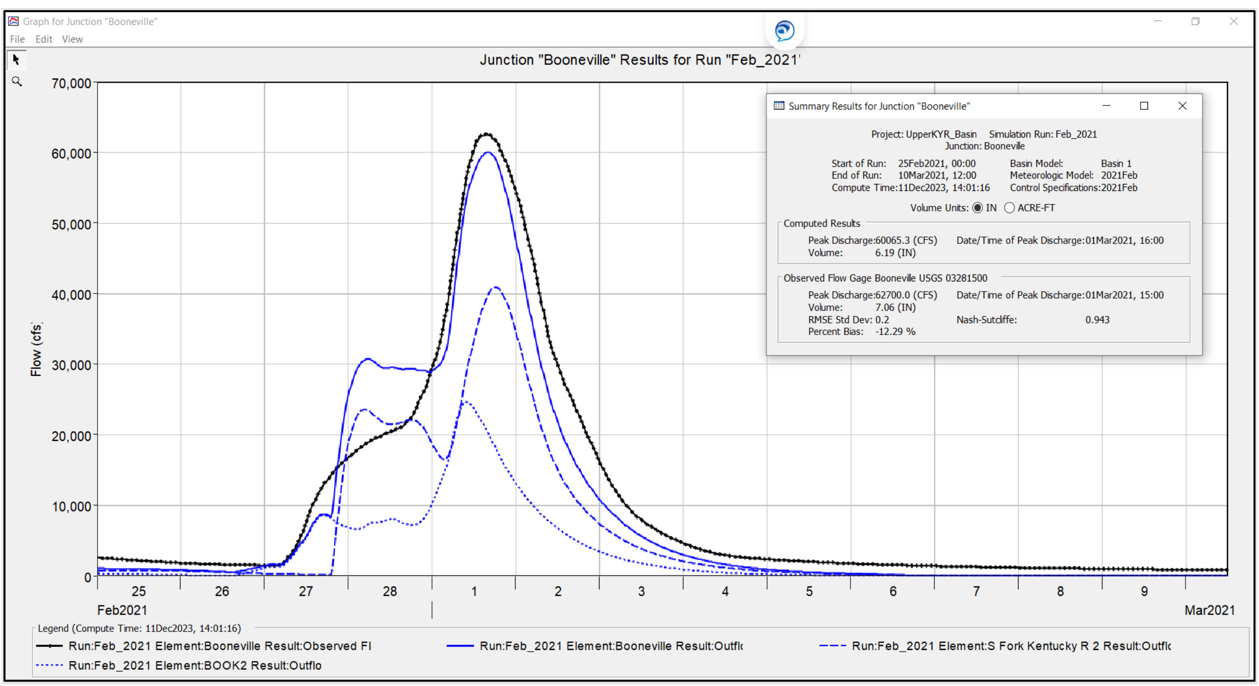

Figure 4.17: South Fork Kentucky River at Booneville, KY Calibration Results

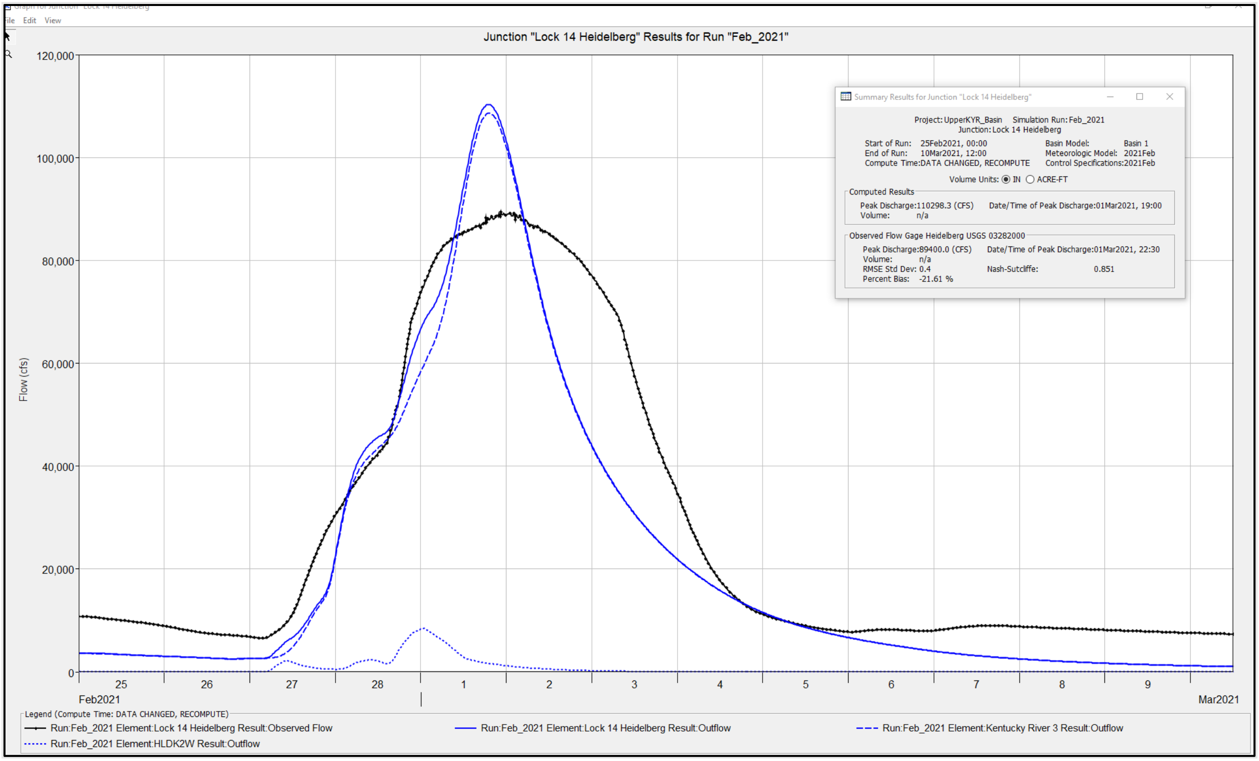

Figure 4.18: Kentucky River at Lock 14 at Heidelberg Calibration Result

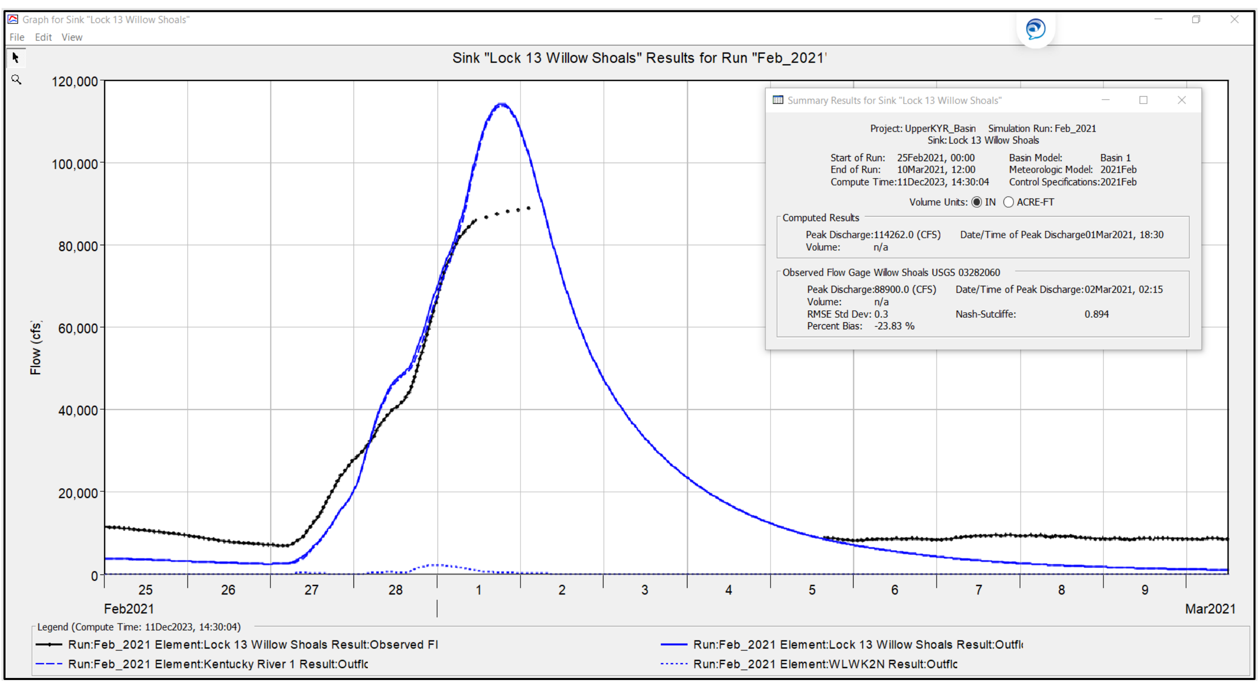

Figure 4.19: Kentucky River at Lock 13 Near Willow Shoals, KY Calibration Results

4.7 Basin Frequency Modeling

The purpose of this section is to describe the efforts performed to develop the 1% Annual Exceedance Probability (AEP) for the watershed. Note that the 1% AEP is analogous to the 100-year Average Recurrence Interval (ARI), which both refer to the probability that a given rainfall volume over a given duration will be exceeded in any one year. The computed 1% AEP flows will be used in phase 2 of this study for the hydraulic modeling.

The primary purpose of the HEC-HMS model was to investigate potential reservoir sites and provide an appropriate inflow hydrograph for frequency storm modeling. The PDT screened out reservoirs as an option for the study prior to the Alternatives Milestone Meeting (AMM). The calibrated hydrologic model was then used for developing the likely inflow hydrograph at Beattyville.

4.7.1 Storm Simulations

Initially, the 1% AEP storm event was modeled adjusting loss parameters beginning with upstream subbasins. The calibrated HEC-HMS model transformation, routing, and base flow parameters were retained for frequency storm modeling. Loss parameters were adjusted to target the frequency flow peak discharges in Section 4.2 at the Jackson, Tallega, and Booneville stream gages (where the flow frequencies were determined on the Forks).

Loss parameters for the 1% AEP storm were found to be reasonable across the basin. However, adjusting loss parameters per the same method for higher frequency storm varied widely. Some loss parameters were unreasonably high for subbasins between the lower Fork gages (Jackson, Tallega, and Booneville gages) and the downstream Heidelberg gage.

Furthermore, when HEC-HMS basin flows were applied to the HEC-RAS model, the differences in reach routing routines between the software made simulating the entire watershed for the full range of frequency flows difficult as inflows hydrographs would still need to be scaled in an iterative process to target the peak flows at Heidelberg (Lock 14).

4.7.1.1 Supplying an Inflow Hydrograph

For the Beattyville General Investigation study, the inflow hydrograph approaching the study area was the primary concern. It was more reasonable to simulate in HEC-HMS with uniform losses across the entire watershed because the computed flow frequency analysis peak discharges at the Heidelberg gage are based on the recorded peak flows regardless upon which upstream Fork the precipitation occurred (no discernable storm pattern could be determined between the Forks). While the upstream gage flow frequencies were calculated, those at Heidelberg are the primary concern for Beattyville. Relegating the losses to uniform values essentially makes the flows from each Fork a function of drainage area. Appropriate Atlas 14 storm (annual maximum series) precipitations and areal reduction factors were used in simulations for the designated frequency event.

4.7.1.1.1 Applying Concepts from Similar Terrain

The constant loss values for the initial 1% AEP simulations ranged from near zero to about 0.1 inches per hour upstream of Beattyville, and about 0.3 inches per hour downstream of Beattyville. As the inflow from upstream was the primary concern, experience from the Johnson County study was leveraged to assign typical loss values.

The low constant loss rates in the 1% AEP simulations are consistent with loss rates used for the Johnson County study (the nearest watershed East of the North Fork Kentucky River, just beyond the mountain ridge). The water control manual for Paintsville Lake Dam indicated an initial deficit of 0.3 inches and constant loss rates of 0.02 inches per hour were representative values for the terrain (Paint Creek watershed). Like the Paint Creek watershed, the Upper Kentucky River basin terrain is similar across the watersheds, for the most part. Low constant loss rates are typical for such terrain in winter and spring months when peak flows are highest.

As an additional validation, the HEC-HMS component of the HEC-CWMS model was re-calibrated in 2019. The final constant loss rates values using continuous calibration found that 0.02 inches per hour is typical for the watershed in winter/spring applications.

4.7.1.1.2 Applying Precipitation

Each frequency storm event was modeled in HEC-HMS with respective Atlas 14 precipitation data using uniform losses with soil infiltration values typical to the region when seasonal runoff is greatest (discussed above). Simulations were completed for the 0.2%, 1%, 2%, 4%, 10%, 20%, and 99% AEP events.

4.7.1.2 Applying Results to the Hydraulic Model

The HEC-HMS output files were placed in the HEC-RAS home folder then appropriately linked to the HEC-RAS unsteady flow files for simulations for ease of model transport.

The hydrographs were then scaled accordingly in the HEC-RAS model flow file until the desired flow at Heidelberg was achieved.

The inflow hydrographs for the frequency simulations are scaled uniformly in HEC-RAS across the contributing subbasins, with the respective frequency flow at Heidelberg as the target.

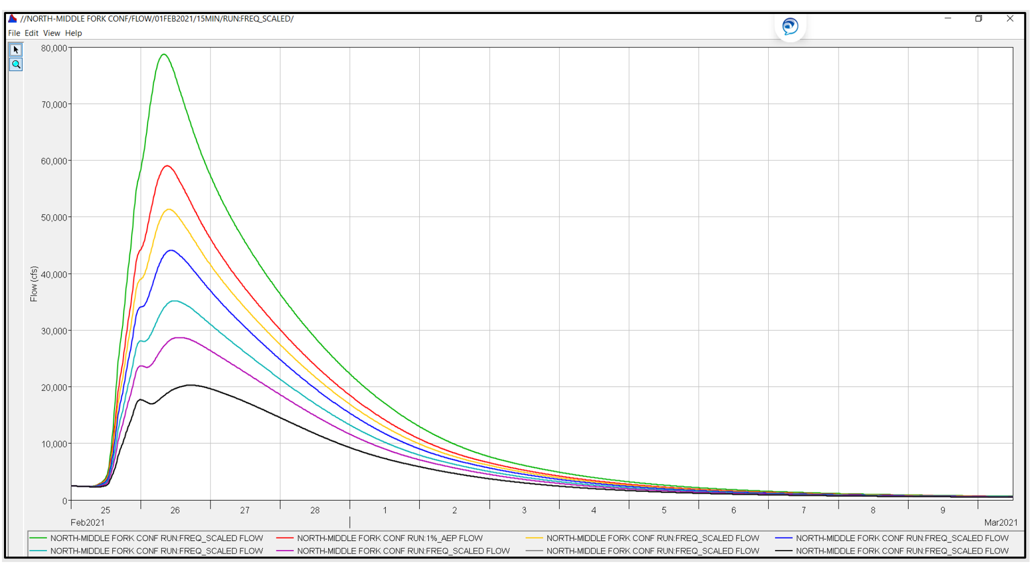

Figure 4.20: Inflow Hydrographs at the North-Middle Fork Confluence

4.8 HYDRAULIC MODEL DEVELOPMENT

4.8.1 General Hydraulic Methodology

The purpose of this section is to describe the existing conditions modeling effort. A one-dimensional (1D) unsteady state hydraulic model for Beattyville was developed using HEC-RAS version 6.3. The development began with the existing CWMS model for the Kentucky River Basin, which was updated for this project with new terrain, cross sections, and a two-dimensional (2D) flow area at Beattyville.

4.8.2 Hydraulic Model Setup

4.8.2.1 Terrain Data

The terrain was created by the aforementioned DEM. No bathymetry was available for the HEC-CWMS model upstream of Lock 11 on the Kentucky River at Irvine, KY. Existing bed profiles and aerial photography were used to estimate channel depth and width.

Bathymetry upstream of Lock 11 was adjusted based on a 1957 USACE study channel bottom (USACE Flood of January-February 1957). The channel bottom slopes of the main stem Kentucky River and the 3 Forks appeared to be the most consistent approaching the study area free of steep drops and inclines, which were present when applying separate data sources. The adjustment was made by approximating the channel bottom at various river mile stations to capture change in slope. The cross-sectional channel bottoms were adjusted using the Channel Design Modification Tool with a trapezoidal channel. The channel bathymetry was initially assumed using the estimated depth from Low Bank to the Thalweg from the 1957 study and estimated top width from satellite imagery and an assumed side slope of 2:1. The side slope and bottom width were then adjusted throughout the Kentucky River basin upstream of Lock 11 to better approximate the bathymetry using engineering judgement. The new channel was burned into the terrain and the cross sections recut to the new terrain.

4.8.2.2 Cross Section Data

The overbanks portions of the existing cross sections were recut based on the updated terrain. New cross sections were added in and around Beattyville to model the bridges more appropriately and to better capture flow and inundation in the area of interest (AOI).

Manning's “n” values were approximated with reference to the CWMS model and the 2019 National Land Cover Database (NLCD) land use layer. When the NLCD was used to approximate the Manning's “n” value, the Mapping, Modeling, and Consequence (MMC) recommendations from Figure 4-21 was referenced.

NLCD ID	Land Cover Description	Manning's "n" Value Range
11	Open Water	0.025–0.05
21	Developed, Open Space	0.03–0.05
22	Developed, Low Intensity	0.06–0.12
23	Developed, Medium Intensity	0.08–0.16
24	Developed, High Intensity	0.12–0.20
31	Barren Land	0.023–0.030
41	Deciduous Forest	0.10–0.20
42	Evergreen Forest	0.08–0.16
43	Mixed Forest	0.08–0.20
52	Shrub/Scrub	0.07–0.16
71	Grassland/Herbaceous	0.025–0.05
81	Pasture/Hay	0.025–0.05
82	Cultivated Crops	0.02–0.05
90	Woody Wetlands	0.045–0.15
95	Emergent Herbaceous Wetlands	0.05–0.085

Figure 4.21: MMC Recommended Manning's Roughness n-Value Ranges for Land Cover

4.8.2.3 Bridges and Culverts
A total of two bridges (both KY-11) were added to the model based on As-Builts from the Kentucky Transportation Cabinet (KYTC).

4.8.2.4 Stream Reaches
The Kentucky River reaches from Lock 11 up to Beattyville and the three Forks were included in the model. A new reach from the confluence of Carr Creek and North Fork Kentucky River to Whitesburg, KY was added from the Carr Creek SQRA hydraulic modeling effort.

4.8.2.5 Boundary Conditions
The HEC-CWMS model was trimmed to only include the basins upstream of Lock 11. The upstream boundaries were inflow hydrographs with lateral inflow hydrographs at the gage locations for Booneville, Tallega, and Jackson, KY. Hydrographs of the 2021 event were used as were the frequency storm inflow hydrographs from the Hydrologic Study.

For the downstream boundary, the USGS published rating curve was used (see Figure 4-22). This was done as part of the effort to calibrate the model to the 2021 event. See Section 4.8.3.2 for further discussion.

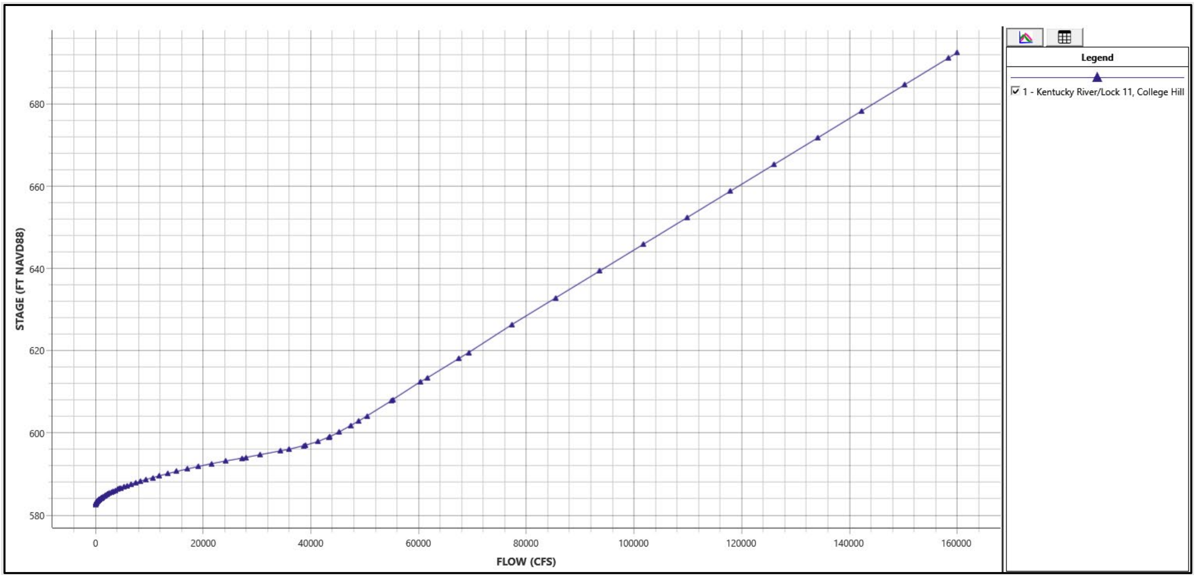

Figure 4.22: Published USGS Rating Curve at Lock 11 on the Kentucky River

4.8.2.6 Storage Areas

Storage areas from the HEC-CWMS model were used except for at Beattyville where the storage area was converted to a 2D flow area. This was done to provide better inundation modeling and mapping at Beattyville.

4.8.2.7 2D Flow Area

A 2D flow area was used at Beattyville to provide a better understanding of the inundation occurring as floodwaters rise on the Kentucky River.

Note that the 2D area is connected to lateral structure LS 275.5 on the Kentucky River reach which extends from just upstream of Crystal Creek to just downstream of Mirey Creek. The lateral structure was placed slightly behind intermediate high ground (a railroad embankment) at Mirey Creek, so the weir ground elevations appear low. This allows backwater to flow into the 2D area freely without the need to model the culvert/bridge opening under the railroad at Mirey Creek. Since no damageable structures are present in this area and no useable culvert/bridge data exists, the arrangement allows the inundation to properly map without affecting the study area results.

4.8.3 Calibration Efforts

In general, the channel roughness from the HEC-CWMS model was preserved because the model was calibrated to the USGS rating curves for each gaged reach. Flow roughness factors were used in calibration, which has held up well for years used by Water Management. The calibration was considered fit for use for nearly all stream reaches. The reach of the Kentucky River from approximate River Mile (RM) 242.8 upstream to Beattyville required calibration to adjust for the slope affects discussed in Section 4.3.3.1.

4.8.3.1 Existing Data

The USGS gage at Heidelberg and surveyed HWMs were used for the calibration effort. The observed rating curve for the March 2021 event was used for calibration due to the slope effect present in the Kentucky River, see Section 4.3.3.1 for further discussion. The observed flow and stage at Heidelberg were also used as additional references.

4.8.3.2 Lock 11 Gage Rating Curve as the Downstream Boundary Condition

The HEC-RAS model was truncated at Lock 11 (RM 203.8) on the Kentucky River to reduce computation times. The published USGS rating curve at Lock 11 was assigned as the downstream boundary condition (recall this is the most downstream gage with an established published slope rating).

4.8.3.3 Flow Roughness Factors

The reach of the Kentucky River from approximate RM 242.8 upstream (past Lock 14) to Beattyville was calibrated using flow roughness factors, which scale the Manning's 'n' roughness coefficients for the desired flows. The process is highly iterative because as flow roughness factors are increased, the stage increases but the flow decreases and vice versa.

The flow roughness factors were adjusted starting with the lowest flows working to the highest. After the flow roughness factors were altered, the simulation was computed and then the flow was adjusted by scaling entering flows in the reach within the unsteady flow file. Results were checked against the observed and published rating curves, then repeated. The iterative process continued for the full range of flows.

The Kentucky River system exhibits looped rating curves. Looped rating curves have a hysteresis effect that simply implies the conditions that occur before a given flow impact the associated stage, where a higher stage can be produced with less than the peak discharge. The hysteresis effect is present at the Lock 14 gage (Heidelberg). Note the purple loop plot in Figure 4-23.

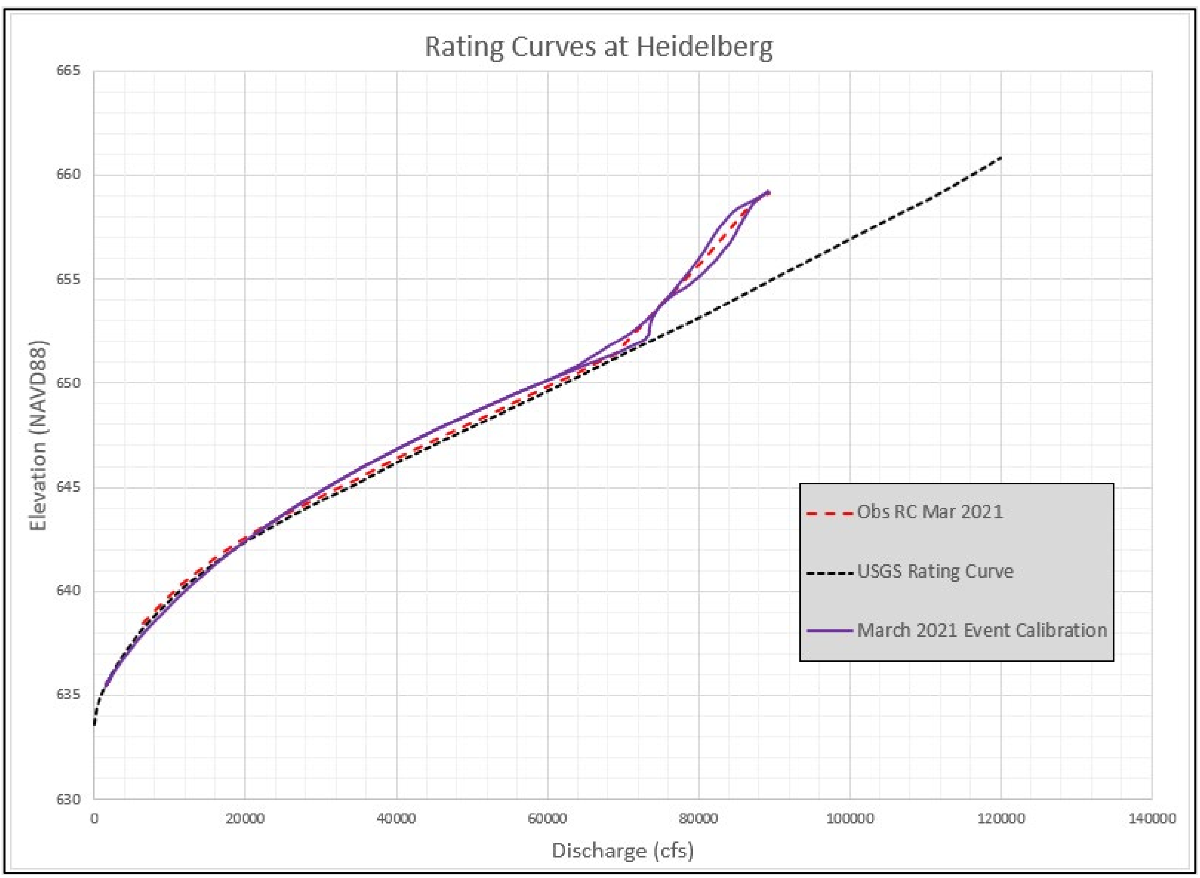

Figure 4.23: Calibration of March 2021 Event to the Observed Rating Curve

4.8.3.4 Calibration

Figure 4-23 represents the calibration effort for the March 2021 event. The inflow hydrographs were input from the HEC-CWMS forecast flow file (CWMS - RFC- Zero) of the event for local flows, and the observed gage data on the three river Forks (North Fork at Jackson, Middle Fork at Tallega, and South Fork at Booneville). The inflow hydrographs supplied by the observed gage data was scaled minor amounts to achieve the flows at Heidelberg during calibration.

The calibration targeted peak flows along the entire March 2021 event observed rating curve. That is, calibration began by scaling the event to lower peak flows on the observed rating curve, calibrating the flow roughness factor, then continuing up the observed rating curve to the next peak flow target. The objective is to utilize the flow roughness factors that track well with the computed hydrograph, with the peak stage close to the observed rating curve and the peak flow marking the end of the rising limb and beginning of the falling limb of the hydrograph. The higher the target peak flow is on the curve, the more pronounced the hysteresis effect which is visible in Figure 4-23 for the March 2021 event.

The March 2021 event was the first recorded event to utilize a pro-rated slope rating curve, therefore, higher peak flows were assumed to extend the sloped rating curve parallel to the observed, similar to the published slope rating curve at Lock 11 (see Figure 4-22). The assumption was necessary to calibrate flows beyond the observed event up to the calculated 0.2% AEP flow while maintaining continuity in the model simulations at higher flows.

The flow roughness factors are stored in the geometry file. However, additional flow roughness factors are stored in plan files for peak flows greater than 80,000 cfs at Heidelberg. This arrangement was necessary because the falling hydrograph limb at higher peak flows should track along the observed rating curve. When the higher peak flow roughness factors are stored in the geometry file, this causes the hydrographs of lower peak flow events to track further off the observed rating curve. Therefore, the solution was to store the higher peak flow roughness factors in the plan file for those peak flow simulations.

4.8.4 Calibration Results

Calibration effort resulted in a reasonable reproduction of the March 2021 hydrograph (see Figure 4-24). Additionally, the simulated rating curve plotted reasonably well at the peak stage and flow (presented above in Figure 4-23).

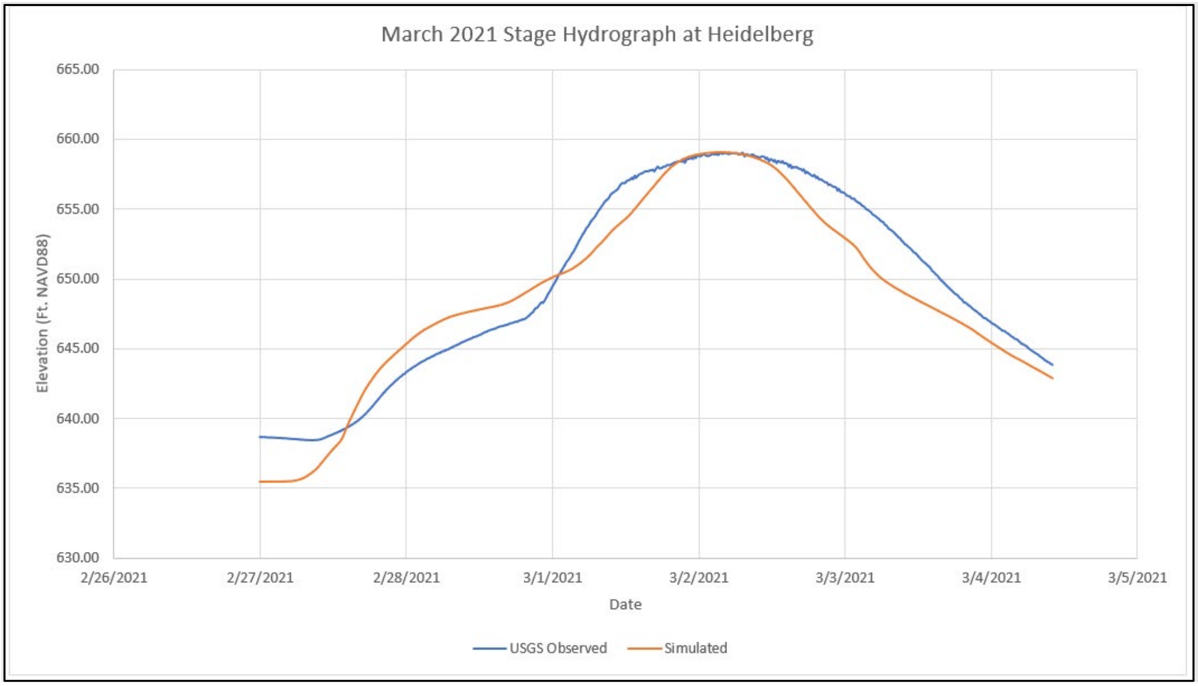

Figure 4.24: Observed and Simulated Stage Hydrograph

The March 2021 event is the only flood event with available highwater marks at Beattyville and a stream gage record at Heidelberg. It must be recognized that the observed rating curve developed by the USGS for the event record leverages measurements from a single event and slope ratings from downstream gages. The HEC-RAS model was calibrated to the same event, which implies there is likely higher uncertainty in the stage-discharge curve at Beattyville for events larger than the March 2021 flood.

4.8.5 General Approach to Modeling Frequency Storms

Each frequency storm was simulated in HEC-RAS using the respective HEC-HMS results output. The HEC-HMS output files were input into the HEC-RAS flow files. Upstream boundary condition flow hydrographs and subbasin outflows were linked in the unsteady flow files. Flows from Buckhorn Dam and Carr Creek Dam were set to their respective minimum flows for storm events.

HEC-RAS simulations were executed in an iterative process of uniformly scaling the upstream boundary condition flow hydrographs and subbasin outflows until the target flow was reached at Heidelberg, then the flow roughness factors were adjusted (recall Section 4.8.3.3), and then the process was repeated. In a sense, simulations for events greater than the March 2021 event were calibrated to the rating curve (see Figure 4-22) in the same process.

Each simulated event was checked against the observed rating curve for reasonableness. Figure 4-25 shows selected simulations plotted with the observed rating curve. Note the hysteresis effect.

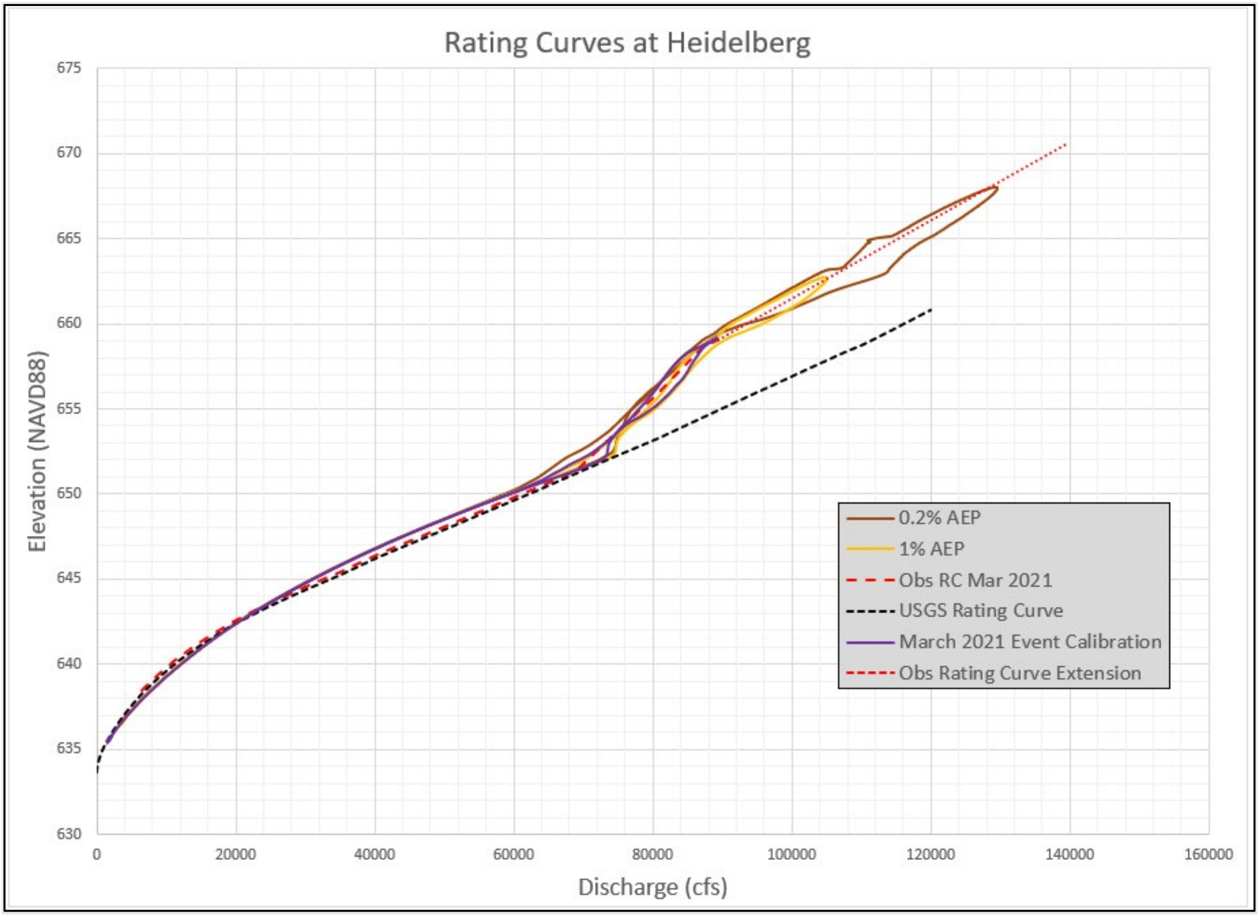

Figure 4.25: Rating Curves of Various Simulations

4.8.6 Conclusions for Existing Conditions

The existing conditions modeling is adequate for the study area. Future studies of upstream areas on the Forks should extend the HEC-RAS model scaling inflows to the Jackson, Tallega, and Booneville gages, then revisit the scaled inflows at the Heidelberg gage as it establishes the downstream condition for the Forks.

Note the lateral structures near the confluence in combination with the inflows from the Forks and the inflows from subbasin outlets all converge in this area. It causes peak discharges to vary quite a bit from the cross sections nearest the confluence to XS 256.9893 as flows computationally balance. Hydrograph plots tend to show iterations across the lateral structure as the difference in head attempt to equalize each computational time step. Attempts were made to resolve or lessen this affect by changing the weir overflow computation method. The normal 2D equation method produced the better results verses using the weir equation option. Adding time slices to the 2D flow area mostly resolved the issue.

4.8.6.1 Uncertainties and Risk Assumptions

Uncertainty exists in both hydrology and hydraulics, the terrain data, the channel bathymetry, and survey data.

4.8.6.1.1 Hydrologic Uncertainties

Hydrologic uncertainties in the flow frequency analysis are addressed in the Risk Report. The flow frequency analysis utilized Bulletin 17C methods intended to capture uncertainties in the stream flow record.

Hydrologic uncertainties also exist in rainfall patterns. No discernable pattern was found in the annual peak flow records indicating the percent of time each of the three Forks contributes to flooding at Beattyville. Uncertainty in storm event precipitation timing and patterns exists and may not be quantifiable without a more thorough analysis.

4.8.6.1.1.1 Frequency Storm Events and Fork Coincidence

The modeled frequency storm events are theoretical, and the drainage area above Beattyville exceeds the size of typical storm areas. The modeling utilizes watershed drainage areas as the primary distributor of flows converging into the Kentucky River, with the peak discharges targeting calculated values from the flow frequency analysis (also theoretical). The flow hydrographs from contributing subbasins are scaled to the calculated values associated with the frequency peak flows at Heidelberg (downstream of Beattyville). While North and South Fork peak hydrograph timing is within about a day at Beattyville for modeled storms, coincidence may be irrelevant as the resulting peak discharge would still be scaled to the frequency flow target at Heidelberg.

True historical coincidence is not only unknown, but the flow would have been recorded at the Heidelberg gage regardless. The risk of frequency storms not capturing increases in water surface elevations due to coincident peaks from the Forks appears low. However, if a floodwall alternative is considered part of the Recommended Plan, then some level of coincident frequency analysis may be warranted. The complexity of such analysis may result in an overly conservative water surface elevation value (in the simplest analysis) or an order of magnitude larger data requirement for a continuous record analysis. The most straightforward path to managing coincidence risk would incorporate additional uncertainty in the stage-discharge curve at Beattyville. See the Risk Report for such considerations.

4.8.6.1.2 Hydraulic Uncertainties

4.8.6.1.2.1 Channel Bathymetry

Recent channel bathymetry was not available for the Forks or the Kentucky River. Bathymetric data in the HEC-RAS model uses the most complete and best available albeit from the data is over 50 years old and acquisition methods were not well-documented. With residents complaining of sedimentation, the bathymetric data remains a source of uncertainty.

Channel bathymetry is based on historical data but is assumed to be adequate. Drop elevations taken from the Broadway Street (KY-52) bridge indicate the bathymetry used is within 1 to 2 feet of the channel data used. In-channel conveyance is hindered by the Kentucky River low-head dams, which contribute significantly to backwater effects. Early simulations assumed channel dredging 10 feet deeper showed the excavation provided an insignificant amount of in-channel storage lowering the water surface less than one foot, thus the backwater effects remain dominant. The risk to proceeding without current channel data appears low.

4.8.6.1.2.2 The Observed Rating Curve and Calibration Event

Uncertainty increases for higher stream discharges. The calibration event is the only event with both measured flow and stage at Heidelberg and highwater marks (also a source of uncertainty) at Beattyville within the period of regulation. The observed rating curve, to which higher flows are calibrated, is a construct of the USGS best estimate as explained in Section 4.3.3.1. Uncertainties exist throughout entire rating curve, with higher uncertainty at flows exceeding the calibration event peak flow. Additional information can be found in the Risk Report addressing uncertainties in the flow to stage conversion at Heidelberg and Beattyville.

The observed rating curve is assumed to be conservative for higher flows; however, there is risk to underestimating or overestimating the stage for infrequent flood events (such as the 1% AEP flood) upon which project alternatives are evaluated.

4.8.6.2 Economic Analysis

Uncertainty in terrain and bathymetry has been conveyed for economic analysis. Engineering Manual 1110-02-1619 (EM 1619) Table 5-2 presents the minimum standard deviation of error in stage for economic analysis using HEC-FDA. The recommended minimum standard deviation of error is 0.9 for this study. The value provided is assumed adequate given the study data; however, this is the minimum value, and the true error is unknown.

4.8.6.3 Flood Control Reservoirs

Buckhorn and Carr Creek Dam are assumed to operate as prescribed in the Water Control Manuals. The Water Management and Lake Operations teams are well-seasoned, and the risk of mis-operation of the dams is assumed low.

4.8.6.4 The Creeks

Flooding from Crystal and Silver Creeks are not a driving factor in flooding Beattyville. Section 4.1.2 describes the Creeks and reasons for focusing on Kentucky River backwater flooding. The Creeks contribute a small drainage area and coincident flooding with the Kentucky River is unlikely. However, coincident timing of peak hydrographs is possible. Section 4.9.7 details an unlikely coincident flooding scenario where both creeks are peak discharging with Kentucky River backwater at peak stage. Uncertainty is present with the ungagged Creeks and streamflow data non-existent. However, the risk of significant increases in water surface elevation due to coincidence is low.

4.9 Conceptual Alternatives

This section describes efforts modeling the various improvement alternatives and analyzing the hydraulic impacts of proposed improvements at Beattyville.

Proposed conditions were modeled for the 0.2%, 1%, 2%, 4%, 10%, 20%, 50% and 99% AEP flow frequencies for economic analysis against the proposed condition.

As part of the Planning Process, the Future Without Project (FWOP) conditions must be considered. For the hydraulic engineering discipline, the FWOP and the existing conditions analyses are the same for this study. The hydraulic data for existing conditions will be used for the FWOP conditions during economic analysis. No further hydraulic analysis is anticipated for FWOP conditions.

4.9.1 Non-Structural and Structural Alternatives Considered

Both non-structural and structural alternatives were considered. Hydraulic data outputs for existing conditions were furnished to the PDT for analyzing non-structural solutions.

The PDT chose four floodwall heights for feasibility-level analysis. Economic analysis will determine the floodwall height with most net economic benefits for consideration as the Recommended Plan. While the hydraulic analysis of proposed floodwall heights was performed on the forementioned annual exceedance probabilities, this report primarily deals with the 1% AEP given the FEMA emphasis on the flood reoccurrence. All data necessary for economic analysis was furnished to the PDT. Analyzed floodwall heights are addressed below in Section 4.9.2. However, the TSP selection resulted in a non-structural measure. Nonstructural measures design and criteria are addressed in sections 9.1.

Non-Structural Alternative: Wet and Dry Floodproofing

Nonstructural measures will likely be subjected to low stream velocities exiting the channel during flood events and the risk of damaging hydraulic forces is considered low. Average in-channel stream velocities can approach 7.5 ft/s but decrease moving from the channel centerline to the overbanks. Crystal and Silver Creek stages rise with the Kentucky River stage, and the overbanks inundate with relatively low velocities in the vicinity of proposed nonstructural measures. The peak velocities in the Beattyville downtown are not expected to exceed 2 ft/s. The hydraulic modeling includes a 2D flow area representing the right overbanks and has demonstrated low velocities around existing structures. See Figure 89 2689.

The height of dry proofing mitigation is generally limited to 3 feet. (see Figure 9-3: Dry Floodproofing Depiction). This limit is based on the structure's ability to resist the forces of floodwater. Since the elevation of dry floodproofing protection is lower than the BFE for most structures, wet floodproofing will also be required. Wet floodproofing is a combination of mitigation measures taken to reduce damage to finishes, utilities, and equipment while allowing water to enter the structure. Structures zoned as Commercial or Residential are eligible. All materials below the flood elevation must be water resilient. To ensure resilience, interior finishes such as gypsum sheetrock wall board and carpet will be replaced. Examples of other measures include raising equipment and adding flood vents to the exterior (See Figure 9-4: Wet Floodproofing Depiction).

Wet and dry proofing will not be expected to remove properties out of the 1 percent AEP zone. Flood insurance requirements are anticipated to be unaffected by this project. Table 1 in Appendix G summarizes the expected level of protections by implementing the non-structural solutions.

There are multiple flow velocity limits, but the single best for a response is probably Section 6.2.1 of ASCE 7-22: “5 ft/s maximum flood velocity adjacent to building for use of Dry Floodproofing.” Also, based on parametric estimates, the anticipated velocities do not appear to significantly impact design, and the 3 ft of protection appears to be feasible even including hydrodynamic effects. Since any determination beyond quotation of ASCE relies on detailed, building-specific calculations, we cannot ethically include a specific statement in this feasibility study. This determination must be part of the design effort, and it is dangerous to include any statement regarding sufficiency of specific details in this kind of document.

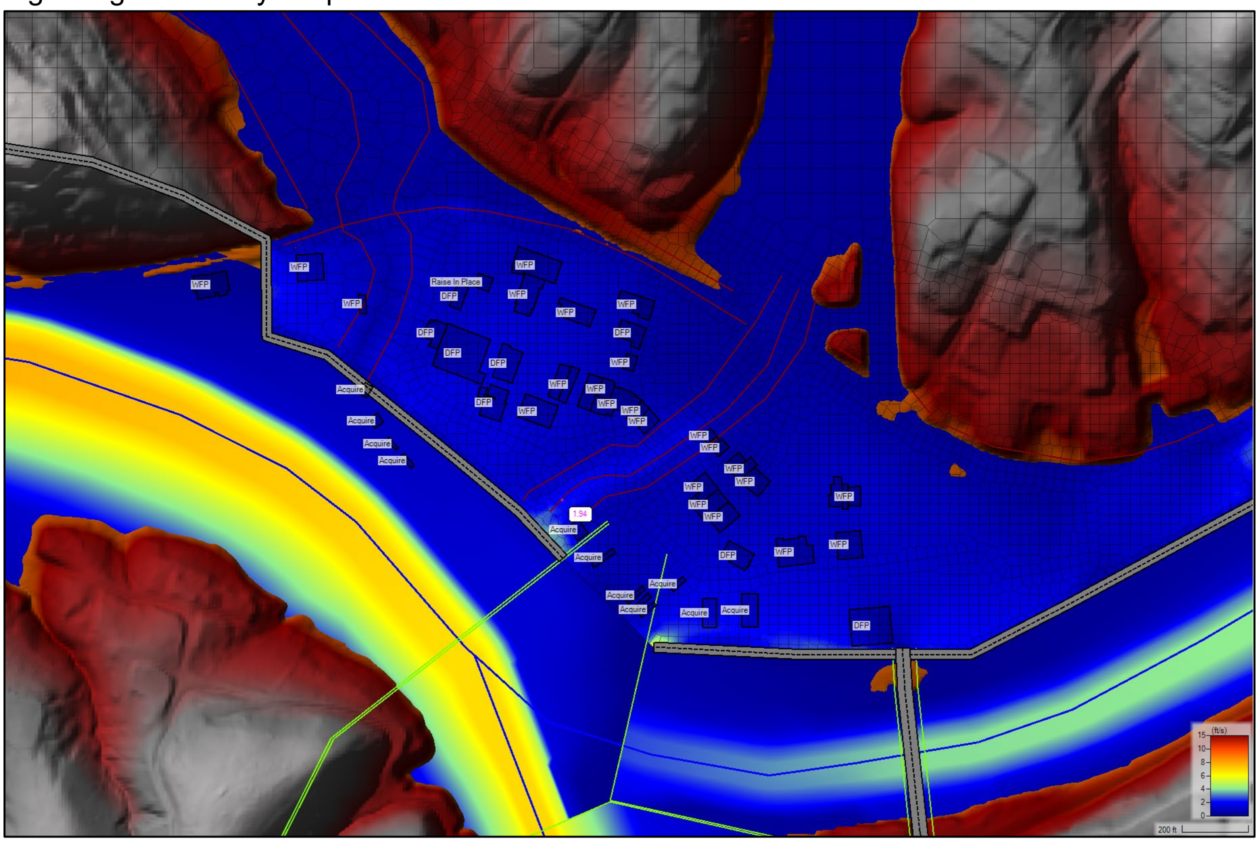

Figure 89 4.26: Anticipated Velocities of Overbank Flooding

4.9.2 Structural Alternative: Floodwalls

The PDT geotechnical and civil design members provided recommended floodwall alignments for hydraulic analysis. Analysis prior to the AMM indicated the lack of storage in Crystal and Silver Creeks requiring high-capacity pump stations if the alignments crossed the streams.

The alignments were chosen to avoid the Kentucky River Regulatory Floodway (see Section 4.1.4). Note that in Figure 4-4, the Floodway includes the historical alignment of Crystal Creek which was established by the effective Kentucky River hydraulic analysis. The Floodway likely included the creek to discourage development, but the stream alignment has since been altered through downtown Beattyville.

There are two recommended floodwall alignments: one surrounding the central downtown between Crystal and Silver Creeks, and the other between Crystal Creek and Broadway Street. The alignments are visible in Figure 4-27.

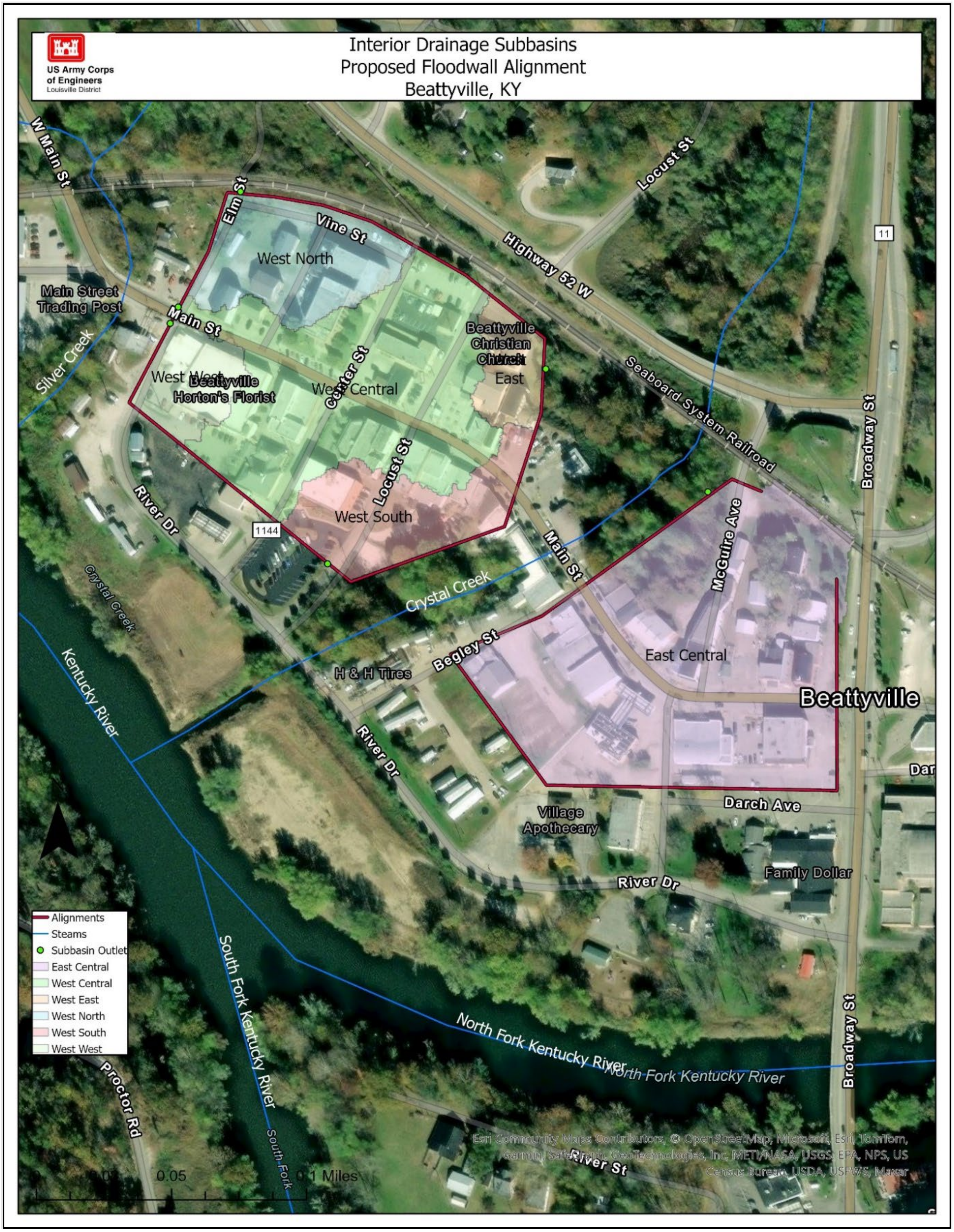

Figure 4.27: Floodwall Alignments and Interior Drainage Subbasins

4.9.3 Floodwall System Modeling Approach

The PDT requested hydraulic analysis of four potential floodwall heights, all with same horizontal alignment. Appropriate data from the hydraulic modeling results was supplied for economic analysis.

The floodwall elevations analyzed were 663.0 Ft. NAVD88, 666.5 Ft. NAVD88 (the reoccurrence of the March 2021 flood event), 669.2 Ft. NAVD88 (the effective base flood elevation), and 672.2 Ft. NAVD88 (the effective base flood elevation plus 3 feet of freeboard).

The HEC-RAS existing conditions model was adapted to these alternative floodwall heights by adding 2D flow area connections to the 2D flow area representing Beattyville. Each analyzed floodwall height has an associated HEC-RAS geometry file. Since the horizontal floodwall alignment is same for each height alternative, the floodwall geometry files are duplicates with different heights assigned to the 2D flow area connections within the associated geometric file.

HEC-RAS plans were created for each floodwall height geometry and the eight (8) frequency flows. The same flow files representing the existing conditions frequency flows were applied to each floodwall height geometry. Simulations were completed and results checked for reasonableness.

Flow data tables and mapping output files were furnished for economic analysis (HEC-FDA) and HEC-LifeSim modeling.

4.9.4 Impacts on River Stage

The maximum rise in river stage for the 1% AEP flood due to the presence of floodwalls would occur with the maximum floodwall height analyzed. The rise in river stage for the BFE plus 3 ft. with a floodwall elevation of 672.2 Ft. NAVD88 is less than 0.1 feet, well below the maximum 1-foot surcharge regulatory limit.

4.9.5 Duration above Damage Stage and Rate of Rise

Hydraulic modeling simulations of the 1% AEP frequency flood indicate the damage stage at Beattyville of 657 Ft. NAVD88 is exceeded for approximately 2.5 days (see Figure 4-28).

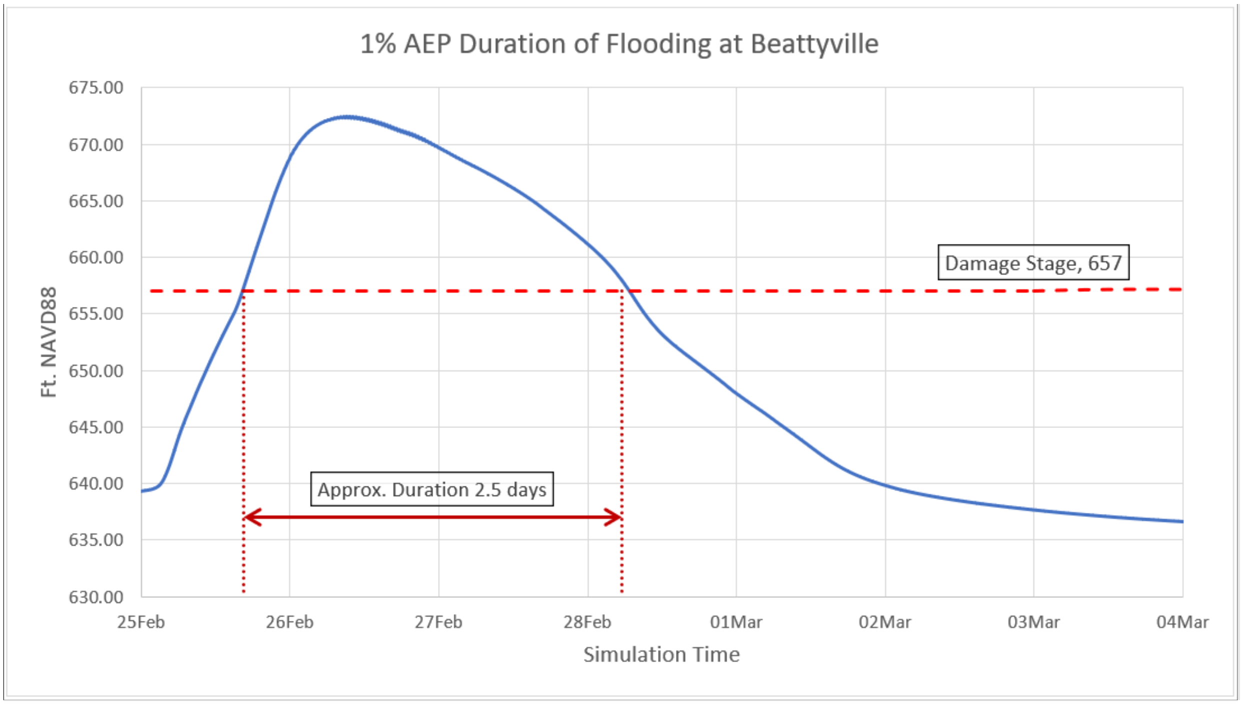

Figure 4.28: Simulated 1% AEP Duration of Flooding at Beattyville

Once at damage stage, the rate of rise for the 1% AEP frequency flood at Beattyville can be up to 1.7 feet per hour, approximately 130% faster than at the Heidelberg gage. This occurs as the gage discharge-stage rating shifts upward in the manner discussed in Section 4.3.3.

4.9.6 Overtopping Sections and Superiority

The overtopping sections were calculated using the HEC-RAS model. The overtopping sections were simulated in HEC-RAS with 1D storage areas to achieve initial estimates of overtopping section length and superiority increments height requirements. The estimates were then applied to the HEC-RAS geometry with 2D flow areas for detailed simulation.

Engineering and Construction Bulletin 2019-8 (ECB 2019-8) addresses the managed overtopping of levee systems. The intent of overtopping and superiority sections is to force an overtopping event into prescribed portion of the floodwall thus providing a controlled manner of leveed area inundation. Initial simulations indicate when the overtopping section is set to 673 Ft. NAVD88 (slightly higher than the 1% AEP stage), the required superiority increment is about 6.75 feet. This arrangement utilizes an overtopping length of about 1970 feet for the downstream floodwall alignment. The 0.2% AEP flood rises to 6 feet above the overtopping section inundating the leveed area in less than 1 hour, which may be unacceptable for life-safety parameters.

Designated overtopping sections may not be feasible with the floodwall alignment and targeted flood defense elevations for large flood events. The alignment may require the full length between high ground tie-ins be considered the overtopping section and reinforced accordingly.

4.9.7 Coincident Flooding of the Creeks

It is not impossible for coincident flooding to occur with Crystal/Silver Creeks and the Kentucky River; however, such coincidence in is highly unlikely given the relative difference in drainage areas.

Crystal and Silver Creek (the Creeks) do not cause flooding for storm events with a normal Kentucky River stage. Crystal Creek (5.2 mi^2) and Silver Creek (3.3 mi^2) have times of concentrations between 0.75 to 1.5 hours. The Kentucky River (and the Forks) above Beattyville encompasses 2,164 mi^2 of uncontrolled drainage area with times of concentration of 22-24 hours to Beattyville. Peak discharging of the Creeks would likely have already occurred prior to the Kentucky River flood wave arriving at Beattyville.

Completing a coincident frequency analysis poses several challenges. A lack of data makes establishing a correlation (an indicator of independence/dependence) difficult at best as the small watersheds are not gaged and a period of record is not available for analysis. It would then be reasonable to assume the watersheds are hydrologically independent given such differences in drainage area. Independence of variables would be the starting assumption in a coincident frequency analysis (CFA).

Regardless, the 1% AEP Kentucky River stage with 1% AEP Crystal and Silver Creek flows give a stage increase of 0.4 feet at Beattyville. This can be taken as a worst case with both Creeks reaching peak discharge when the Kentucky River flood wave arrives. The total chance 1% AEP using total probability will be less than a 0.4-foot increase in stage, which is not significant with HEC-RAS modeling.

If a floodwall alternative advances as the Recommended Plan, then a full CFA would be warranted.

4.9.8 Interior Drainage

A floodwall/levee system alternative requires an interior drainage analysis. Engineering Manual 1110-2-1413 (EM 1413) addresses hydrologic design and analysis of interior areas.

4.9.8.1 Minimum Facilities

For feasibility-level design, interior drainage is assessed for minimum facilities which determines the components required for the drainage system to operate as it did prior to project installation. Common components of interior drainage systems may include gravity outlets, pump stations, detention storage, diversions, and pressure conduits.

When assessing minimum facilities, it is reasonable to use the design storm of the existing storm sewer system to determine the additional system components required. The analysis can be completed using the design storm for the sewer network in questions; however, if the design storm is unknown, then another appropriate design storm may be used. For this analysis, the interior drainage assumes a 100-year, 24-hour storm for comparing existing and proposed conditions. The indication of adequacy is the proposed condition ponding level is no higher than the existing condition ponding level.

4.9.8.2 Interior Drainage Modeling

The interior drainage was modeled using HEC-HMS. The interior subbasins were delineating using a high-resolution digital elevation model (from KY Above statewide 5-foot DEM dataset). Information from archived Kentucky Transportation Cabinet (KYTC) project plans, Google Earth imagery, and field visits was used to assemble a rudimentary drainage network, which was then used in the subbasin delineation and identification of gravity outlet locations at the line of defense. Some basic sewer depths below grade were assumed and storage-elevation curves were assembled. The data was entered in HEC-HMS to represent each subbasin as a reservoir with outlet facilities.

Precipitation transformation was modeled using SCS unit hydrograph method. The interior areas are urban with connected impervious areas. Losses were estimated using SCS curve number methodology.

The 100-year, 24-hour storm was used for simulation and gravity outlets were sized to keep the ponding below damage elevations. A normal Kentucky River tail water elevation of 635.0 Ft. NAVD88 was used in simulations.

4.9.8.2.1 Gravity Drains

The initial interior drainage analysis indicates gravity outlets will be the primary means to convey interior runoff through the line of defense, each outlet equipped with a flap gate for backflow prevention. The existing storm sewer system will be utilized where possible matching or upsizing the existing storm sewer connections at the line of defense. The proposed floodwall alignment is situated such that the gravity pipes will likely drain to Crystal or Silver Creek. See the proposed gravity outlet locations in Figure 4-27. There is a potential need for basic collector storm sewer drains in some interior areas, but the need appears to be minimal.

4.9.8.2.2 Pumping Stations

The interior areas encompass relatively small drainage areas. Maximum precipitation runoff can be assumed for the nearly impervious leveed area without creating a need for pumping. The gravity outlets are shown to be adequate. If the floodwall/levee alignments are altered to cross Crystal or Silver Creek, then pump stations will likely be required. The current floodwall alignments, as analyzed, will not require pump stations.

4.9.8.2.3 Interior Detention Storage

In Figure 4-27, the interior area topography within the alignment east of Crystal Creek exhibits a low area in the northwest portion near the gravity outlet. This would be the likely location of designated detention storage if required. Based on available data, storm sewers may not outlet in this area. Collector sewers may be needed to move interior runoff efficiently to the outlet. The gravity outlet through the line of defense may require some detention storage. Field surveys are necessary to determine the need.

4.9.8.2.4 Diversions

For the current floodwall alignment, interior drainage areas are relatively small, and streams will not pass through the line of defense. There is no anticipated need for diversions.

4.9.8.2.5 Pressure Conduits

Prior to the AMM, a pressure conduit was considered, and analysis showed it was feasible for Crystal Creek. This analysis applies to an unfavorable alignment which crossed Crystal Creek. The pressure conduit was screened out prior to the AMM mainly for environmental and maintenance concerns. Pressure conduits are no longer being considered.

4.9.8.3 Minimum Facility Recommendations

For the proposed floodwall alignments, minimum facilities will likely include gravity outlets to move interior runoff through the line of defense. The modeled gravity outlets are 30 inches in diameter or less, thus avoiding the requirement for secondary means of closure. The eastern alignment may require minor detention storage, if any. Collector sewers are more likely to be needed in the eastern alignment. Field surveys of terrain and the locations and conditions of the existing storm sewer system will be required for more detailed analysis.

Precipitation runoff will enter the storm sewers incurring head losses and attenuating hydrograph peaks. However, the gravity outlets were modeled in HEC-HMS as outlet culverts exiting reservoirs, which assumes the runoff goes directly to the outlet. This is reasonable for assessing whether the proposed exiting gravity outlets through the line of defense hinder the existing storm sewer system with the floodwall system in place. The outlet will receive the hydrograph unattenuated by the storm sewer system and can be considered conservative for comparison with existing conditions.

Table 45 exhibits the anticipated gravity outlets required to move interior runoff through the line of defense. The calculated gravity outlets assume concrete pipes with Manning's roughness coefficients of 0.012, but other materials with similar roughness may be allowable pending Levee Safety review. Flap gates were assumed for all outlets.

Table 45: Anticipated Gravity Outlets Required

Interior Subbasin	Length (Ft)	Diameter (Ft)	Inlet Elevation (Ft)	Outlet Elevation (Ft)	Slope (Ft/Ft)
West N	193	1.0	645.0	639.2	0.030
West S	198	1.0	656.7	650.8	0.030
West E	200	1.0	649.6	643.6	0.030
West W	91	1.0	657.0	654.3	0.030
West Central	123	2.0	657.0	653.3	0.030
East Central	91	2.0	640.5	637.8	0.030
*Floodwall/levee penetrations with diameters greater than 2.5 feet require second means of closure.					

There is no need for pump stations, diversions, and pressure conduits currently. If the alignments are modified such that the line of defense crosses Crystal or Silver Creek, then such system components may be required.

4.9.9 Results Discussion

The hydraulic analysis is complete for feasibility-level design. By avoiding the Regulatory Floodway, the proposed floodwall alignments minimized water surface elevation increases due to presence of the project and avoid the FEMA 'zero rise' in base flood elevation mandate. The alignments do not cross the Creeks thus avoiding the need for costly pumping stations. Precipitation runoff within the small interior areas would be passed through the line of defense with gravity outlets.

Challenges with the proposed floodwall alignments remain, despite the above advantages. Superiority in overtopping may not be feasible with the alignments. Additional erosion armoring may be required to reduce risk to floodwall foundations. Economic modeling with HEC-LifeSim may indicate increased life safety risk with egress. NFIP eligibility appears very unlikely.

Additional hydraulic analysis will be required if a floodwall alternative is carried forward as the Recommended Plan. Continue below for anticipated analysis and considerations in detailed design.

4.10 Future Analysis and Considerations

4.10.1 Detailed Floodwall Analysis

Floodwall alternatives require a more in-depth analysis if carried forward as part of the Recommended Plan. It is not uncommon for efforts to be iterative in nature between engineering disciplines as a solution is sought.

4.10.1.1 Overtopping and Superiority Sections Detailed

Floodwall overtopping sections are typically flanked by a raised superiority increment designed to force larger flood events to overtop the floodwall in the designated locations. The larger flood events targeted are the next two larger frequency flows greater than for what the floodwall is designed. The overtopping sections are intended to allow the gradual filling of the leveed area as not to cause panic, serve as warning of inundation, and to allow the hydrostatic pressures on the floodwalls/levees to equalize thus lowering the risk of catastrophic failure.

When the flood mitigation system is overtopped and the leveed area inundated without catastrophic failure, the system design level will have been considered exceeded and has performed as intended.

Additional analysis will be required to establish effective overtopping sections. Overtopping and superiority section issues were previously discussed regarding the maximum floodwall height evaluated prior to the Recommended Plan. Carrying the floodwall option forward will require a collaborative effort with the PDT to determine if designated overtopping sections are desirable with the selected floodwall elevation, or if the entire floodwall length must be designed for overtopping. Superiority sections are not always feasible and further analysis must be completed to make a determination.

4.10.1.2 Plunging Force

Plunging force refers to the force exerted on the landside ground and the potential erosion of the floodwall/levee foundation. Plunging force is primarily driven by the approach velocities of overtopping flows, the water surface height above the overtopping section, and the fall from that elevation to the landside ground surface. It is a force per unit of floodwall length. The plunging force dictates the amount of surface armoring required to mitigate the eroding of floodwall foundations and reduce the risk of the subsequent collapse.

Plunging force analysis will be required for floodwall alternatives carried forward as the Recommended Plan. Geotechnical and structural engineers use the analysis to determine appropriate foundation armoring and erosion protection.

4.10.2 Interior Drainage

Cursory interior drainage has been completed to address minimum facilities. The analysis has indicated the relativity small, leveed areas (and thus small drainage areas) will likely drain well-ahead of the Kentucky River flood wave arrival. The basic interior drainage facilities consist of gravity outlets and have shown to be adequate given assumptions regarding existing storm sewer networks.

Advancing the floodwall alternative as the Recommended Plan will require additional interior drainage analysis after survey data is acquired, including a complete Coincident Frequency Analysis (CFA) of gravity outlets. Collector sewers will likely be required to facilitate proper drainage within the leveed area. The process is iterative with the Civil Design engineers to select the optimal network configuration and outlet sizes. Field surveys of terrain and the locations and conditions of the existing storm sewer system will be required for more detailed analysis.

4.10.2.1 Outlet protection

Gravity outlets designed to evacuate interior runoff from the interior areas require erosion protection. Erosion is typically mitigated with riprap protection at the outlets. The US Federal Highway Administration (US FHWA) design manuals are used to size minimum riprap gradations.

The gravity outlet protection measures would be exposed to riparian forces nearly perpendicular to the outlet centerlines. The FIS report states that Crystal and Silver Creeks can generate high stream velocities in flash flood conditions. However, the report also indicates scour has not been an issue. Additionally, the initial gravity pipe configurations supporting the interior drainage analysis empty to Crystal and Silver Creeks located in the Kentucky River backwater influence where velocities are typically low. High stream velocities are not a concern for washing out riprap protection at the gravity outlets.

4.10.3 The Creek Hydraulic Models

Crystal and Silver Creeks have not been identified as the primary flooding source. Further analysis of the creeks has not been warranted. Survey data was obtained for the bridges along the creeks but have not been incorporated in HEC-RAS models for the individual streams. Creek channel bathymetry through downtown Beattyville was included in the main existing conditions HEC-RAS model terrain file.

Crystal and Silver Creeks will need to be modeled in HEC-RAS if the PDT requires hydraulic analysis. The effective steady flow HEC-RAS models obtained from KYDoW terminate just upstream of the KY-52 bridge crossings. The creek water surface elevations though downtown are controlled by the Kentucky River backwater. Updating and extending the Creek models would likely require no more than 2 weeks effort per creek.

4.10.4 Flood Inundation Mapping

The modeling applied to this study thus far could be used to develop flood inundation mapping (FIM). The resulting maps would carry forward with uncertainties already discussed with existing conditions. The level of processing effort would be about a 5-week effort. If Crystal and Silver Creeks are to be included as flooding sources, the effort nearly doubles. Such mapping may be useful to emergency management and planning officials.

4.10.5 Flood Warning Systems

A flood warning system for Beattyville would likely only provide riverine flood warning for the Kentucky River stage. The primary gage for detection would be the USGS gage at Heidelberg for which the National Weather Service (NWS) Ohio River Forecast Center (OHFC) issues stage forecasts. Warning time would be up to 24 hours and relies solely on the forecast for the event. Real-time warning from the Heidelberg gage may not provide sufficient warning time.

Beattyville floods when all three Forks provide significant flow contributions. For the March 2021 flood, the peak flows at the Fork gages could have provided some indication of the coming flood at Beattyville. However, configuring such a warning system based on the upstream gages would require significant effort to establish patterns and thresholds. The best estimate of potential warning time based on such a system is 6 to 12 hours. The required level of effort and likelihood of producing a usable product is unknown.

4.10.6 FEMA Flood Hazard Mapping Considerations

FEMA submittal processes can be lengthy, and many standards must be met. Typically, the effective models FEMA has on file must be used for mapping updates. Additionally, FEMA mandates the models be updated with the latest bridge and terrain data. If the model is not executable (such as the model on file being in paper form), then whole model replacement is an option. However, additional requirements apply such as water surface profiles at boundary tie-ins must be within +/-0.5 feet. Continue below for other hurdles to this approach.

4.10.6.1 The Effective Hydraulic Model

The effective model was reconstructed from HEC-2 files found in LRL digital archives. The model was verified to be the effective model by comparing the cross-section points with the scanned paper format PDF obtained from FEMA.

The effective FEMA hydraulic model includes the Kentucky River from Heidelberg through the confluence at Beattyville to approximately 1 mile upstream on the North Fork. The Kentucky River and the North Fork are combined in a single stream reach. The South Fork is represented by a flow change location. The model only contains 6 cross sections, and no bridges were included. The encroachment stations were not included in the model.

The model was converted to HEC-RAS and the cross sections were georeferenced using the effective lettered cross sections and river mileage from USGS quad sheets (long used in-house as 'official' river mileage). HEC-2 models converted to HEC-RAS do not produce the same results but were comparable to the results in the FEMA PDF file.

The simplicity of the steady flow model allowed for quick encroachment analyses. An encroachment analysis was completed on the original (now georeferenced) cross section geometry, then a second on the cross-section geometry re-cut from the current terrain model. The two encroachment analyses were compared and confirms the effective Floodway extents are reasonably correct along the Kentucky River and the North Fork. When the initial Floodway was submitted (prior to becoming effective), some ethics-guided leeway is permitted when drawing Floodway boundaries between the model cross sections. This is likely the reason for the effective Floodway not reflecting the current channel alignment of Crystal Creek. The old channel alignment is still intact on FEMA flood hazard maps (see Figure 4-4).

4.10.6.2 Updating the Effective FEMA Hydrologic and Hydraulic Analyses

The effective discharges and the updated flow frequency discharges (Section 4.2) are comparable. A submitted hydrologic analysis (updated peak discharges) must change the base flood effective peak discharges by at least 5% for FEMA to consider updating the effective peak discharges published in the current FIS. The updated flow frequency analysis does not meet that criterion for the base flood (1% AEP/100-yr flood). See Table 46 below.

Table 46: Comparison of Flow Frequency Analysis and FIS Effective Discharges at Heidelberg

AEP	Return Period (Years)	Flow Frequency Analysis (2023) (cfs)	Effective Discharges (1978) (cfs)	Percent Difference
0.002	500	141,811	140,000	1.3%
0.01	100	111,842	116,000	-3.6%
0.02	50	99,337	106,000	-6.3%
0.1	10	70,604	80,200	-12.0%

The flow to stage conversion differs noticeably for the 1% AEP and 0.2% AEP flows between the effective (converted HEC-2 model) and the updated (HEC-RAS unsteady flow model for this study). The difference can be explained by the slope effect that develops when flows at Heidelberg exceed 70,000 cfs (recall Section 4.3.3.1). The updated HEC-RAS model uses the pro-rated rating curve to capture the slope effect for these higher flows, where the effective model does not. The effective model (converted from HEC-2) utilizes a known water surface elevation as the downstream boundary condition. The known water surface elevation is dictated by the adjacent Kentucky River downstream effective HEC-RAS model.

The unsteady flow HEC-RAS model produces a 1% AEP stage approximately 1.9 feet higher than the effective model. However, leveraging the unsteady HEC-RAS model to update the effective FEMA hydraulic analysis is not recommended as this would result in a higher base flood elevation, even with the lower 1% AEP discharges. Also, use of the model would not satisfy the continuity in water surface requirements with the next downstream Kentucky River model. Updating the effective hydraulic modeling would not likely yield impactful results for Beattyville and would require a significant level of effort through FEMA acceptance.

4.10.6.3 Regulatory Floodway Revision

The effective Regulatory Floodway boundaries are likely to change very little with a Floodway Revision. The converted HEC-2 model encroachment analysis indicates the effective Floodway is reasonable for the main flooding source. An update would likely only impact the Floodway alignment of the Crystal Creek, which may not yield many benefits. A Floodway Revision is not currently recommended.

4.10.7 Regulatory Implications for Working in the Floodway

The Recommended Plan nonstructural measures (as of September 2024) are addressed outside the engineering appendices. However, the proposed approach involves removing existing structures from the established Regulatory Floodway. Other plans may include ecosystem restoration or the establishment of parks or other floodable facilities within the Floodway. Any work within the Floodway requires the demonstration of ‘no rise’ in the 1% AEP water surface profile between existing and proposed conditions using the effective hydraulic model, which in this case, is not in an executable format. As discussed in above sections, the HEC-2 model located in USACE archives has been converted to HEC-RAS but additional work remains before it can be considered an effective duplicate model. The proposed plan will likely reduce flow obstructions and overbank roughness coefficients such that a ‘no rise’ will be achieved.

4.10.7.1 No Impacts on Flood Insurance Rates

The ‘no rise’ demonstration will not change the Flood Insurance Rate Maps (FIRMs) of which only two panels encompass the Floodway (Panels 21189C0075C and 21129C0129E); thus, flood insurance requirements will not change for nonstructural measures below the FEMA elevation requirement. A Letter of Map Revision (LOMR) or Amendment (LOMA) submittal would be required to adjust the flood insurance requirement for structures.

5 CONCEPT ALTERNATIVES NOT SELECTED

5.1 FLOODWALLS

The flood wall alignments shown in Figure 5-1 were selected with a priority to protect as many structures as possible while also avoiding the floodway, utilities and minimizing demolition impacts of existing structures. Further explanation on the screening process of the floodwall alignments can be found in Section 3.7 of the Integrated Feasibility Report and Environmental Assessment.

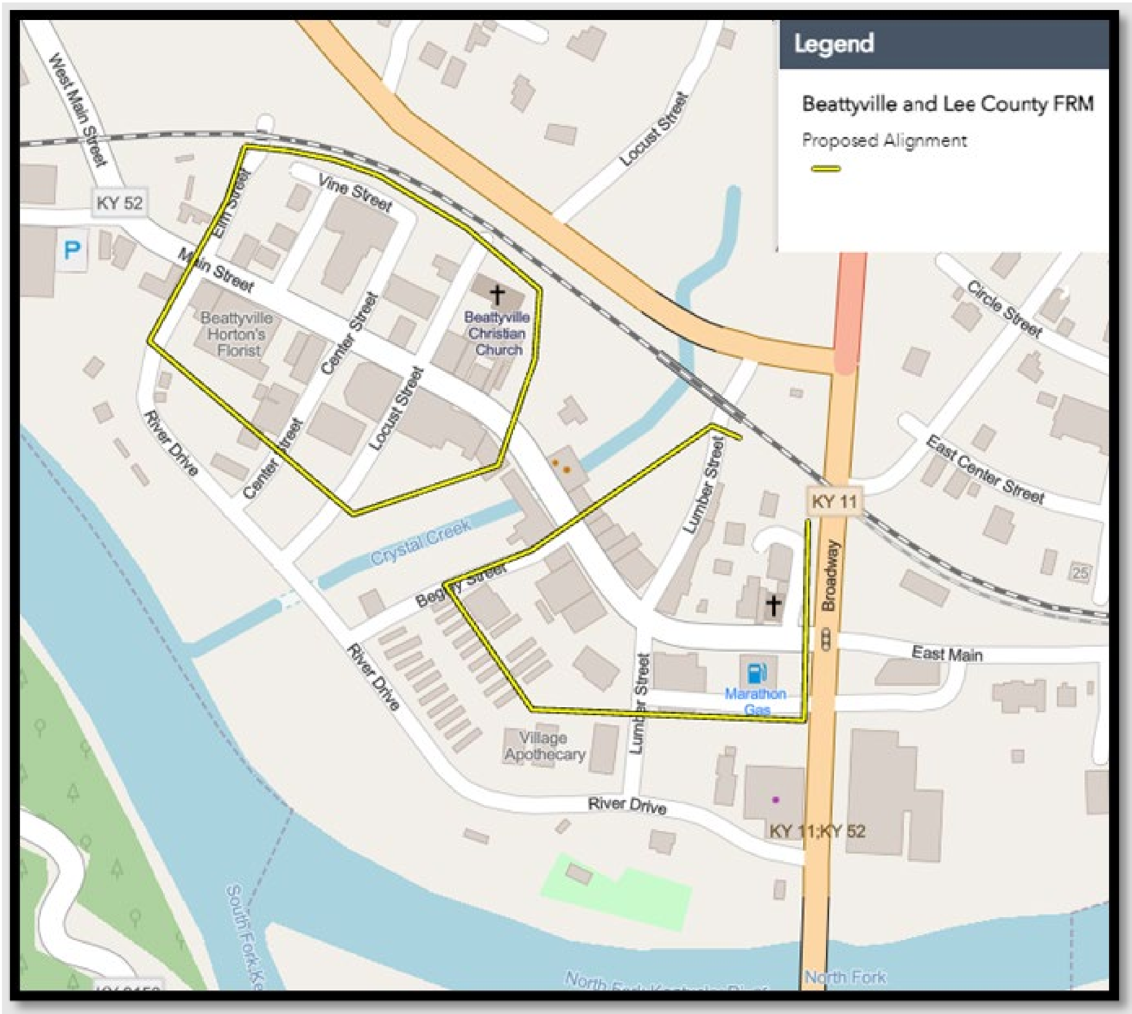

Figure 5.1: Floodwall Alignments

6 Survey, Mapping and other Geospatial Data Requirements

6.1 Overview

Primary survey control consists of 3 semi-permanent control points set and adjusted utilizing least-squares adjustment of the primary network to achieve Second Class, First Order Horizontal accuracy (1:50,000). The 3 control points were levelled through to vertically tie them together utilizing differential levelling with a digital level with precision of 0.001 ft, achieving First Order, Class 1 Vertical Accuracy. Original elevations for the 3 primary control points were derived utilizing the National Geodetic Survey Online Position User Service (OPUS) to obtain NAD83 horizontal position and NAVD1988 elevations, then levels and adjustments were made as mentioned above. Horizontal/Vertical Positions to obtain Finished Floor Elevations and Lowest Adjacent Grade were measured utilizing a 1” Robotic Total Station from secondary control, achieving a horizontal and vertical accuracy of 0.01 ft. These secondary networks were tied back into the primary network with a horizontal accuracy of Second Class, Second Order (1:25,000) and vertical of First Order Class 1. See Table 47 below.

Table 47: Control Point Table

Control Point Table				
Point ID	**Northing***	**Easting***	**Elevation***	**Description**
BFRM1	2098493.13	2231085.84	659.99	#5 Rebar with USACE Cap
BFRM2	2097597.68	2232680.05	663.40	#5 Rebar with USACE Cap
BFRM3	2098244.62	2234404.86	664.28	#5 Rebar with USACE Cap

** US Survey Feet*

6.1.1 Datum

- Horizontal Datum: NAD1983 (2011) SPC KY South (Zone 1602)
- Vertical Datum: NAVD1988
- Date of Survey: 03 – 28 April 2023

6.2 Field Work Data Collection

Field work investigations consisted of determining the type of pavement surrounding the structure, the type of pavement (i.e., concrete, or asphalt), foundation type, construction material (vinyl siding, brick, masonry, or CMU), the number of openings at grade and the number of openings above grade. Using these metrics along with zoning data and the finished floor elevation (FFE), we were able to develop a plan of action on the best possible non-structural measure for each individual structure in our study area.

6.3 GIS Data Collection, Management and Distribution

Existing conditions geospatial data was gathered from various federal, state, and local sources. New geospatial data was developed using ESRI ArcGIS software, ArcGIS Pro. All geospatial data was compiled in a centralized ESRI geodatabase for data consistency and organization. Project geospatial data was shared among the PDT using ArcGIS Portal (see Figure 6-1), enabling collaboration, data-driven decision-making, and efficient workflows.

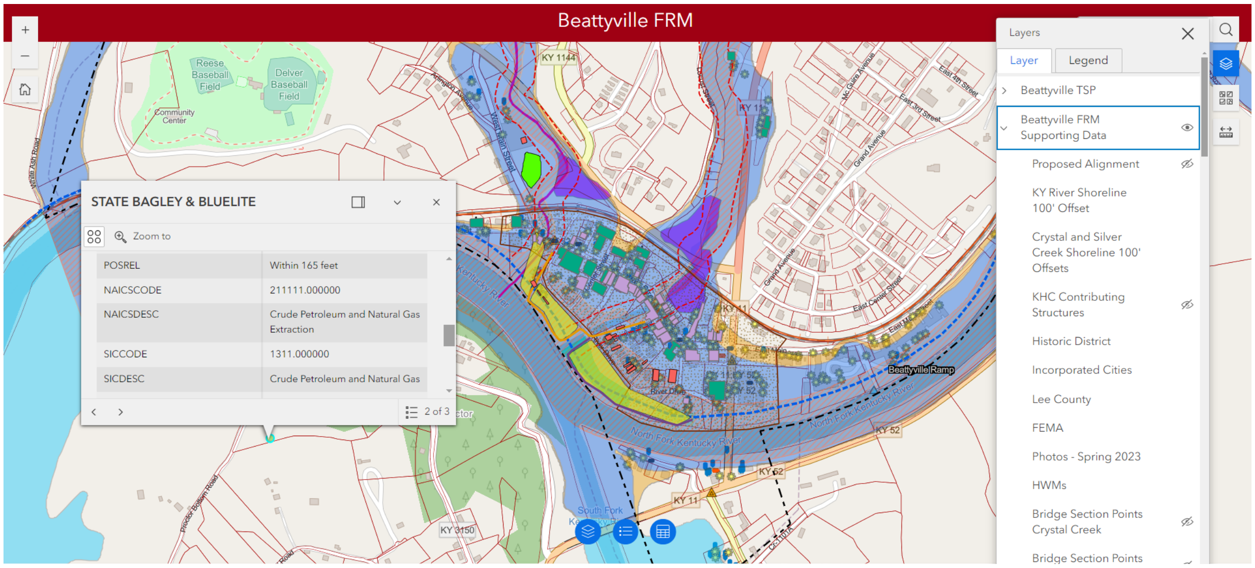

Figure 6-1: Beattyville FRM Web Mapping Application on ArcGIS Portal

7 GEOTECHNICAL

7.1 LOCAL GEOLOGY

The Kentucky Geologic map service, published by the Kentucky Geological Survey – University of Kentucky, indicates that most of the project is underlain by alluvium (Figure 7-1). The information on the geology at the project site is based on the descriptions provided on the Beattyville geologic quadrangle.

The alluvium on-site consists of silt, clay, sand, and gravel, all intergrading and intertonguing chiefly on the flood plain of the Kentucky River and its major tributaries (like the North Fork). Alluvium predominantly consists of yellowish brown clayey and sandy silt that contains lenses of light gray, very fine to fine quartz sand that commonly weathers grayish yellow. Pebbles, cobbles, and boulders of sandstone are locally common, especially in the northern one-third of the quadrangle. The formation includes older alluvium on terraces, and in abandoned meanders. Contacts are generally indistinct and are approximately located. The unit commonly grades into colluvium (not mapped) along valley walls. A veneer of alluvium may cover some terraces and parts of valleys shown as bedrock (small outcrops of bedrock surrounded by alluvium not mapped).

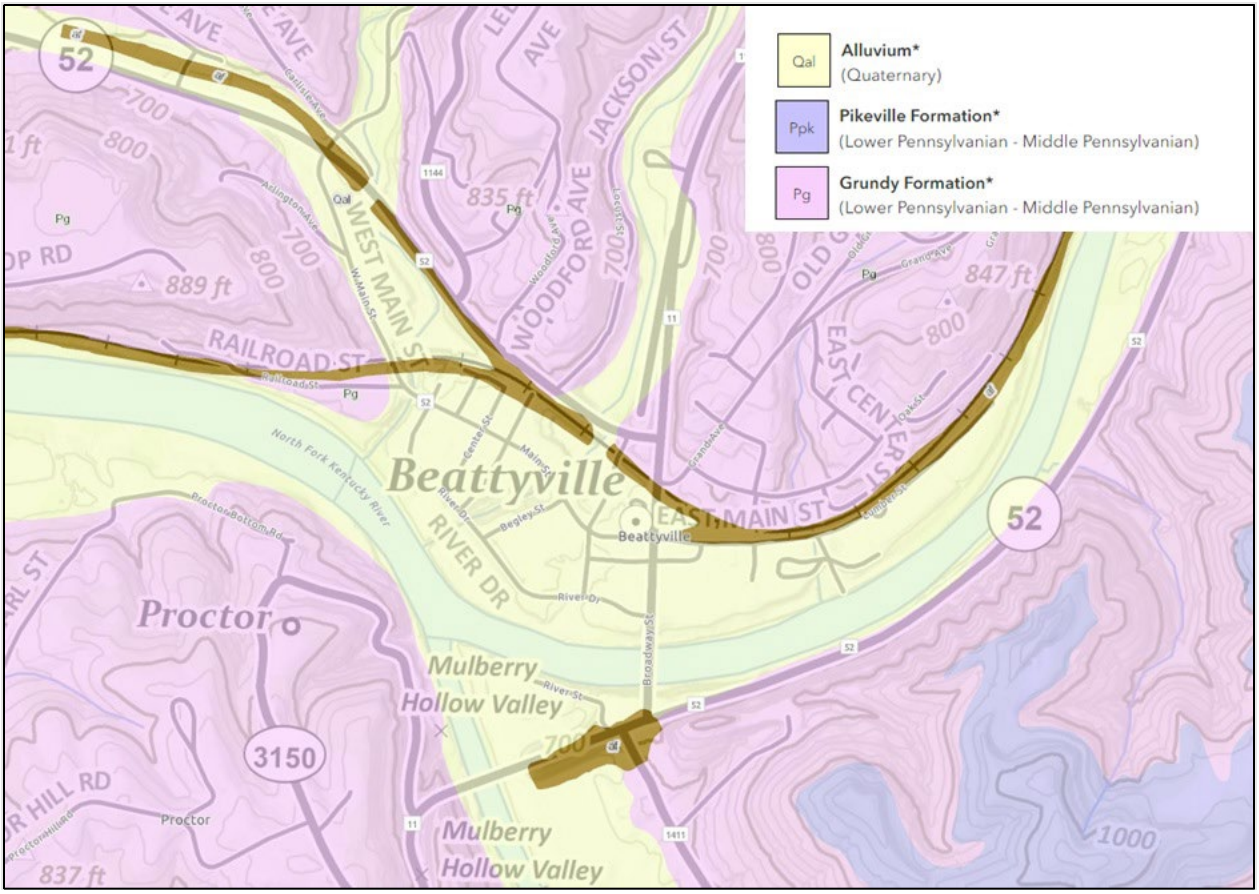

Figure 7.1: Local Geology of Beattyville (Source - UKY KGS website)

The Grundy Formation of the Breathitt Group is expected below the alluvium. The Grundy Formation consists of siltstone (30 to 50 percent), shale (30 to 50 percent), sandstone (10 to 30 percent), coal, and underclay.

The project site is in an area of Kentucky where the mining of coal can impact development. Surface mines have resulted in mountain top removal, as well as the filling of valleys, while the underground mining of coal has resulted in mines that have caused surface subsidence or otherwise impacted construction. The Gray Hawk coal zone contains two coal beds about 20 to 30 feet apart. The upper bed is traceable into the Gray Hawk coal bed of the Heidelberg and Sturgeon quadrangles. It is persistent and generally more than 12 inches thick. The lower bed is sporadic and is generally less than 12 inches thick. The upper bed of the Gray Hawk coal zone is probably the same as Lee No. 3 coal bed, the Barren Fork coal bed, and Travelers Rest coal bed. The Beattyville coal zone contains two coal beds about 10 to 25 feet apart. The upper coal bed is much more persistent and generally thicker than the lower. This upper bed is probably the same as Tattlers coal bed (Weir and Mumma, 1973), Lee No. 2 coal bed (Miller, 1910), and Hudson coal bed (Lyons, 1963). The Contrary coal bed on the Right Fork of Contrary Creek is projected from exposures in the adjoining Heidelberg quadrangle (Black, 1977). The strata below the Corbin Sandstone Member of the Lee Formation were named Beattyville Shale Member of Lee

Formation by Miller (1917); but as noted by Weir (1973), these strata are not lithologically separable from Breathitt Formation.

Artificial fill (mapped in brown in Figure 7-1) is mapped along the rail alignments and generally follows the interface of the Grundy Formation and the alluvium in the Beattyville area.

7.2 Geotechnical Considerations/Hazards

The geotechnical considerations/hazards listed below were identified as the most likely to increase remediation costs and impact non-structural measure approaches, design, and cost.

7.2.1 Existing Fill

As indicated on the geologic maps, fill was placed for the railroad; however, to provide more area for development, fill also is often placed in roadways and in areas that are lower in elevation that are more prone to flooding. This fill may consist of materials dredged from the nearby river(s). Other sources of fill material include construction debris, waste materials generated by nearby manufacturing, and/or soils generated from nearby earthwork projects (i.e., rock cuts from highway projects, benching hillsides). In areas where coal mines are present, mine spoils are often a source of readily available fill material depending on the proximity of the mine to the town. Existing fill often is not placed in a controlled manner, resulting in elevated risks and increase design and construction costs.

Existing fill tends to be poor quality and irregular in content and quality. Often existing fill contains debris that can be difficult to penetrate. Depending on the depth of the existing fill, existing fill could be undercut and either replaced with controlled fill or foundations extended to suitable soils. If the fill extends too deep and undercutting is not feasible, deep foundation systems may be required. The presence of the existing fill must be considered and could significantly impact the design and implementation of the non-structural measures and other planned improvements. In addition, the presence of the existing fill may create a preferential seepage path that may impact slabs-on-grade in structures to be dry floodproofed.

It was reported by Beattyville officials that fill has been placed in areas between River Drive and the Kentucky River. The fill reportedly consisted of excess materials that were deposited during flood events (clays, silts, and sands). As the deposits were removed during flood clean-up, the materials were placed between River Drive and the Kentucky River as a disposal site. Additionally, fill was placed in the southern portion of Beattyville, generally north of River Drive, when Crystal Creek was re-routed. Based on available USGS topographic maps and aerial photography, Crystal Creek was re-routed to the south between 1961 and 1995. The former alignment of Crystal Creek flowed from the northeast to the southwest until it passed under Main Street. Crystal Creek then flowed generally west through the current location of several parking lots north of River Drive until it emptied into the Kentucky River at the same location as Silver Creek. The new alignment for Crystal Creek continued to the southwest after it passed under Main Street, passing under River Drive, and emptying into the Kentucky River approximately 625 feet south of the former exit with Silver Creek. Figure 7-2 and Figure 7-3 show both the general area of mass fill placement between River Drive and the Kentucky River and the former/current Crystal Creek Alignment.

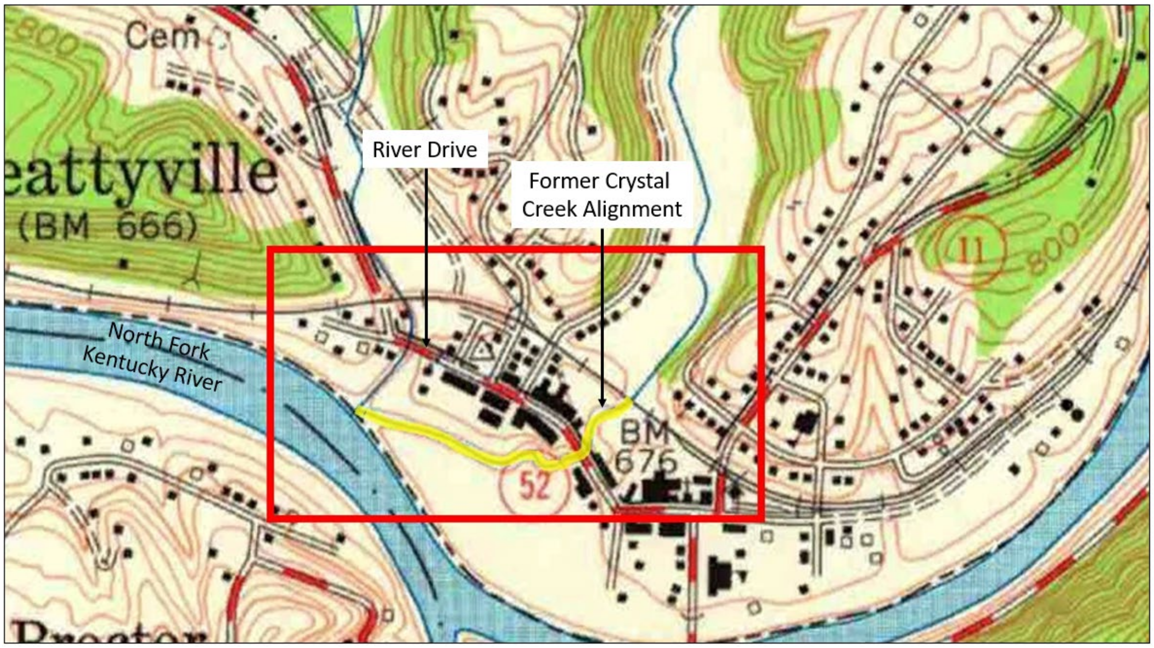

Figure 7.2: Former Crystal Creek Alignment (USGS Beattyville Topographic Map, 1961)

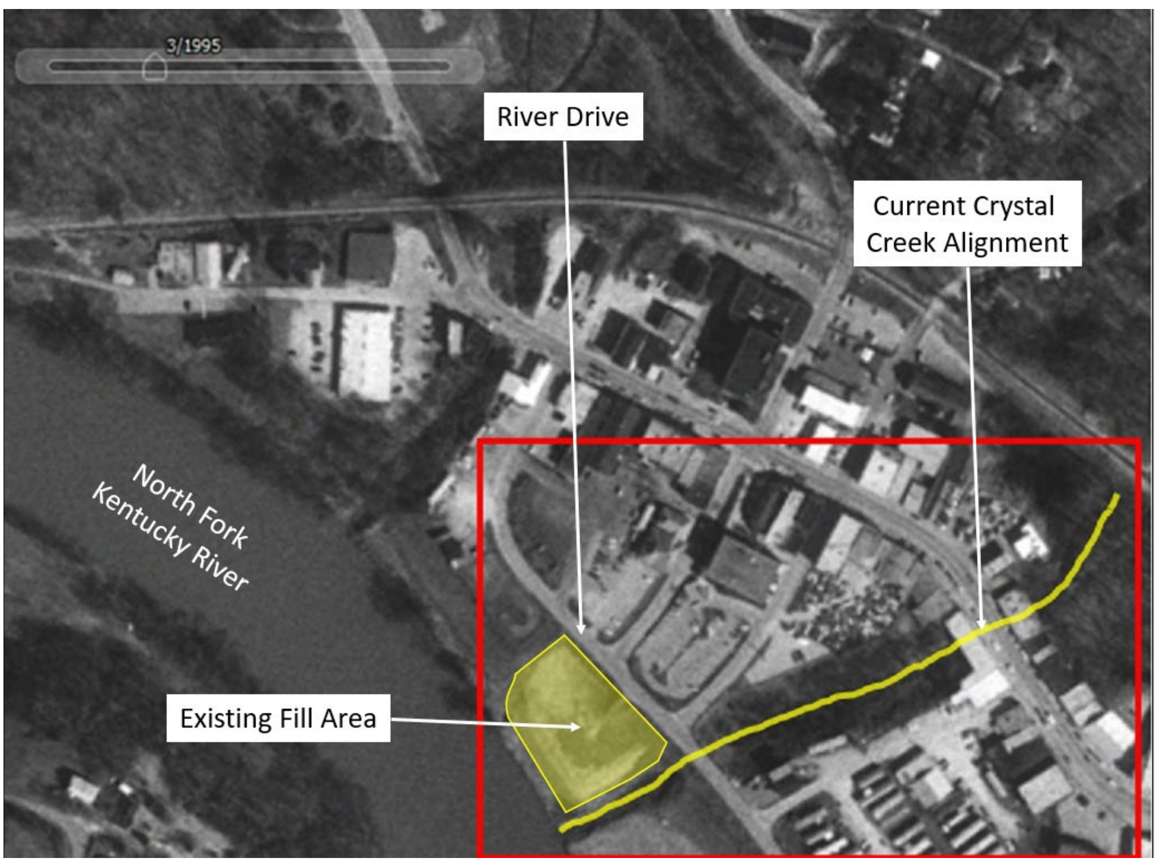

Figure 7.3: Current Crystal Creek Alignment and Existing Fill Area (Google Earth Aerial Photograph, 1995)

Available Sanborn fire insurance maps indicated that a sawmill and lumber yard and coal company (Day Lumber and Coal Company) were present to the east of Broadway and south of East Main Street until at least 1921. In the 1914 Sanborn map, there was a waste conveyor leading to a sawdust and refuse burner on the eastern portion of the property. In the 1908 map, this area was labeled as "Hell Dump" for sawdust and refuse. On the map, the area was an irregular shaped area that was approximately 110 feet by 60 feet. The location and size of the dump/burn area should be considered approximate. Often materials from former dump sites are still present, and the extents may be much larger than was mapped. The presence and location of these fill materials should be considered for any planned improvements or alterations in that area.

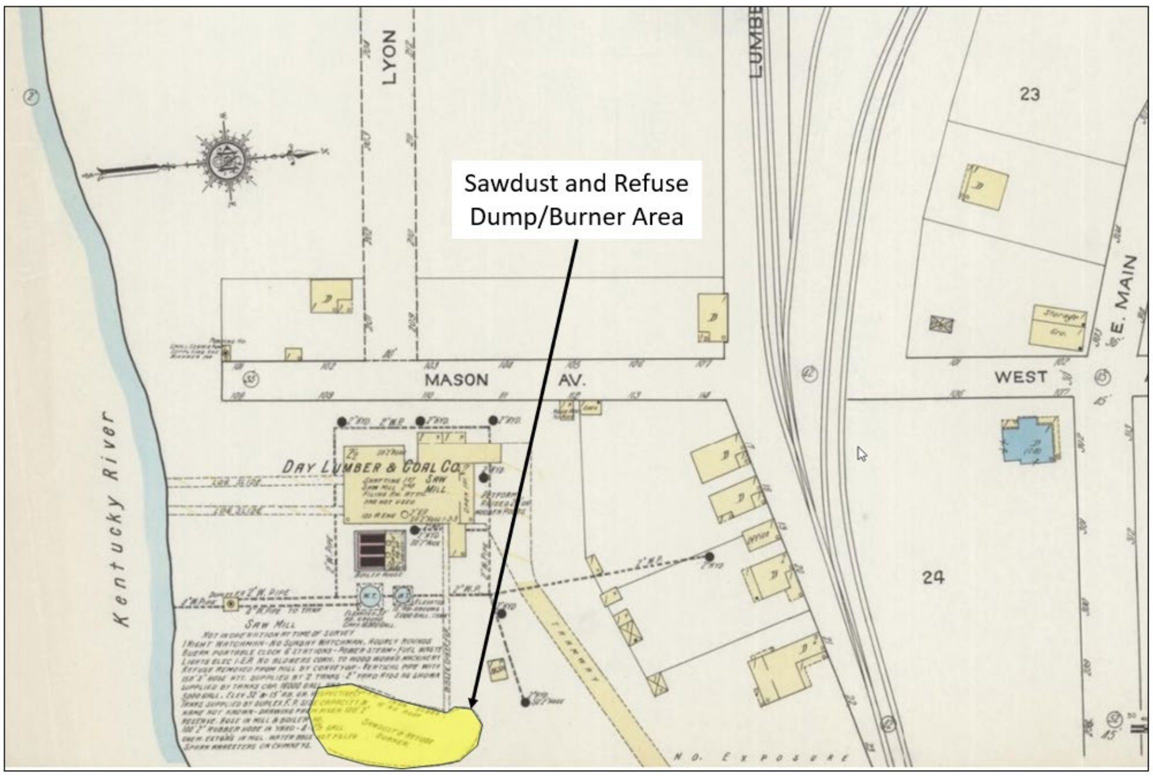

Figure 7.4: Approximate Location of Sawdust and Refuse Burner (Sanborn Map, 1914)

7.2.2 Soft Soils

As indicated on the geologic maps, alluvium is present across most of the project area. Soft soils are typically associated with alluvial deposition and can create a concern with new foundations or additional loading to existing foundations. Isolated undercut and replacement of soft soils and/or deep foundation systems may be required to address soft alluvial soils.

7.3 Future Exploration(s)

The Recommended Plan will consist of an array of floodproofing measures throughout the City of Beattyville. The measures planned consist of dry floodproofing, wet floodproofing, and raising one existing structure in place. Buildings that are not structurally suitable for floodproofing and/or buildings within the floodway will be acquired and removed.

The bearing conditions and soils, foundation systems, and conditions under the floor slab must be evaluated for each building where dry floodproofing is planned. The exploration for each building will likely consist of a series of test pits and borings around the perimeter of the building with some form of limited exploration likely consisting of hand auger borings and/or test pits within or adjacent to the building. It is critical to understand the subsurface conditions (underlying soil types and soil strength) and the building construction (foundation size and bearing depth) to evaluate if dry floodproofing is feasible for the building and, if dry floodproofing is possible, to properly design a dry floodproofing system that will be effective for each structure. The exploration should be sensitive to identifying soils or materials that are present that would be susceptible to allowing seepage under the building foundations or any other conditions that would need to be addressed during design and construction of the selected dry floodproofing system (such as existing fill or soft soil).

8 Environmental Engineering

An Environmental Assessment (EA) is being developed to identify the likely environmental effects of a proposed project and its reasonable alternatives. Environmental effects analyzed include ecological, aesthetic, historic, cultural, economic, social, and human health. Impacts to the environment were considered throughout the feasibility planning phase and reduced to the extent practicable for all alternatives. The EA will outline compliance with all applicable environmental laws and regulations for implementation of the Recommended Plan.

9 Civil Design

9.1 Nonstructural

Data collection for the Recommended Plan was gathered through field investigations consisting of survey and fieldwork investigations. Details into the data collection regarding survey can be found in Section 6.2 Field Work Data Collection of this report.

Nonstructural measures are designed to reduce or avoid flood damages without directly controlling riverine waters; they are not designed to reduce or eliminate flood heights. Stockpiles of sandbags and other O&M supplies are recommended. See local U.S. Army Corps of Engineers sandbagging guidance.

USACE divided the structures within Beattyville into two categories: Historic and Essential. An example of a Historic structure may be an old church. An example of an Essential structure may be a grocery store.

Within the Historic designation, there are two structures to be dry floodproofed and twenty-two structures are to be wet floodproofed and one structure to be raised in place. See Section 9.1.1 for more information on dry floodproofing structures. See Section 9.1.2 for more information on wet floodproofing structures. See Section 9.1.39.1.3 for more information on raising a structure in place. See Figure 9-1: Historic Structures for an overview of the above breakdown.

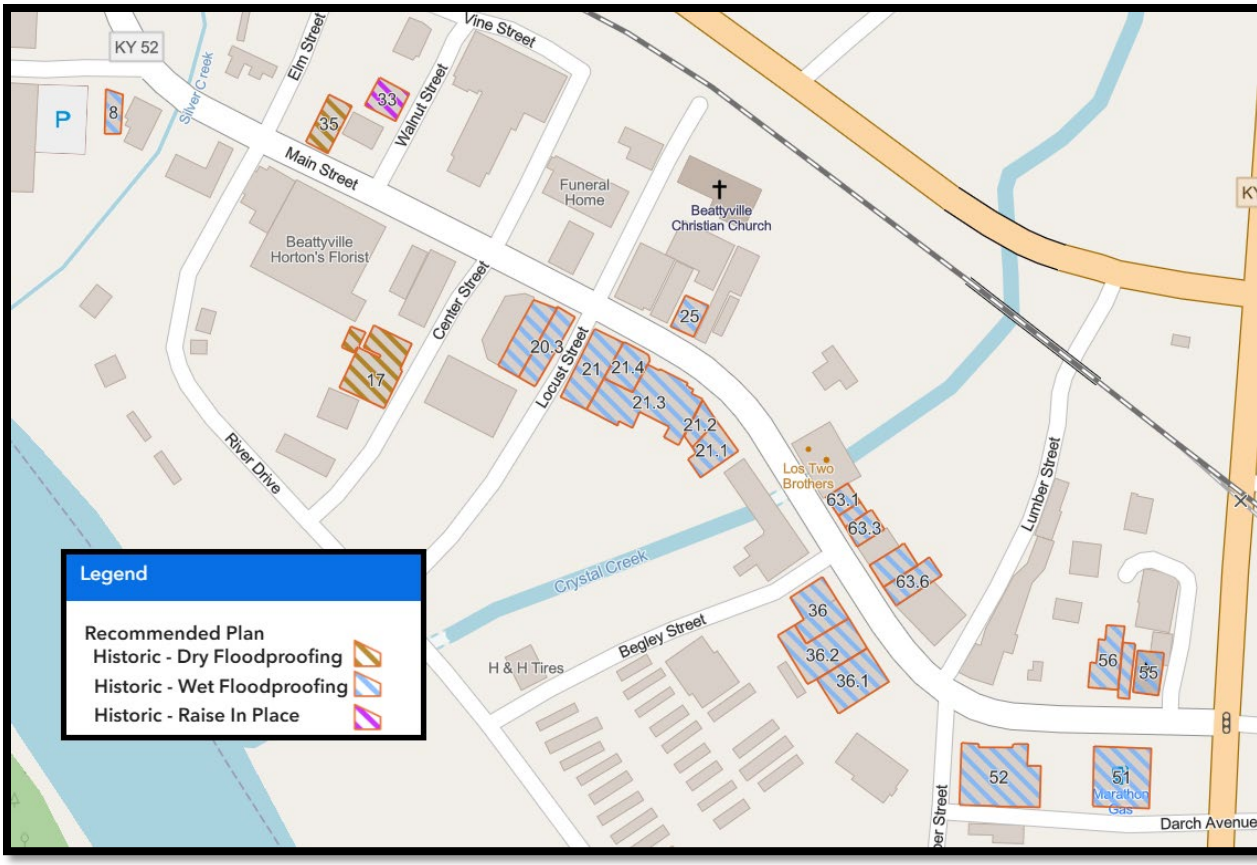

Figure 9-1: Historic Structures

Within the Essential designation, there are eight structures to be dry floodproofed. Eight Structures are to be wet floodproofed. (Note: three of the eight wet floodproofing Essential structures also fall under the Historic category. For purposes of this study, Essential designation takes precedence over Historic designation). See Section 9.1.1 for more information on dry floodproofing structures. See Section 9.1.2 for more information on wet floodproofing structures. See Figure 9-2: Essential Structures for an overview of the above breakdown.

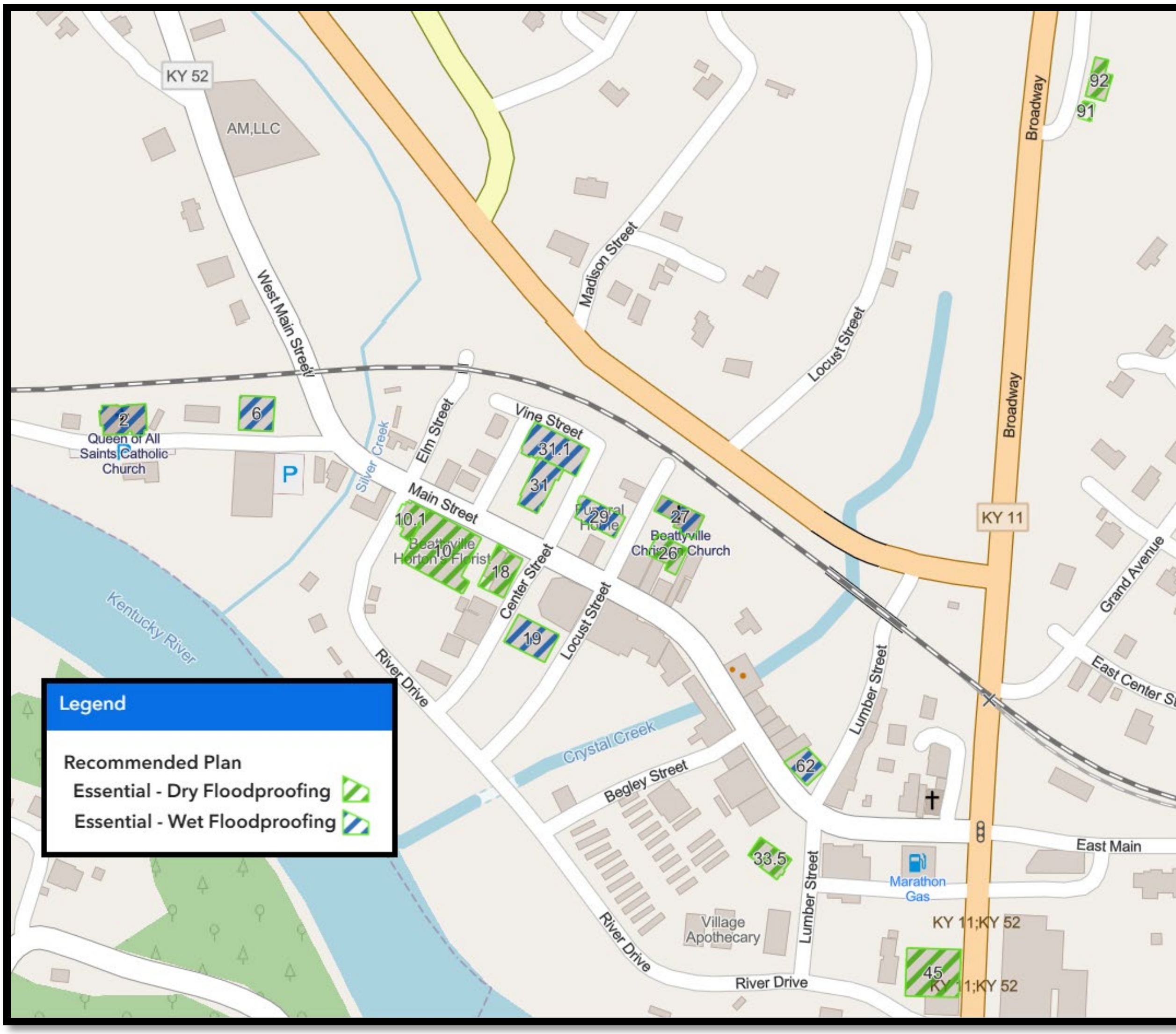

Figure 9-2: Essential Structures

While each eligible structure will be evaluated for the most cost-effective nonstructural measure, the federal government reserves the right to determine which measure shall be implemented for each structure.

After the Agency Decision Milestone, we will develop diagrams for each structure category to show what type of work is covered and what type of work isn't covered. The diagrams will be made for each structure type. Types may be Historic structures, wet floodproofing structures, dry floodproofing structures that are stick build, dry floodproofing structures that are CMU, and raising in place residential structures.

9.1.1 Dry Floodproofing

Dry floodproofing is a combination of mitigation measures designed to prevent water from entering a structure up to a certain height. Structures zoned as Commercial are eligible while residential structures are not. Examples of dry floodproofing include providing temporary water-tight covers for door openings, replacing windows with flood-rated assemblies, and constructing a veneer wall around the perimeter. The height of this kind of mitigation is generally limited to 3 feet. (see Figure 9-3: Dry Floodproofing Depiction). This limit is based on the structure's ability to resist the forces of floodwater. Regardless of the height of protection, foundation and floor drains with sump pumps will be required to alleviate floor jacking or undermining of the slab and foundations. Since the elevation of dry floodproofing protection is lower than the BFE for most structures, wet floodproofing will also be required.

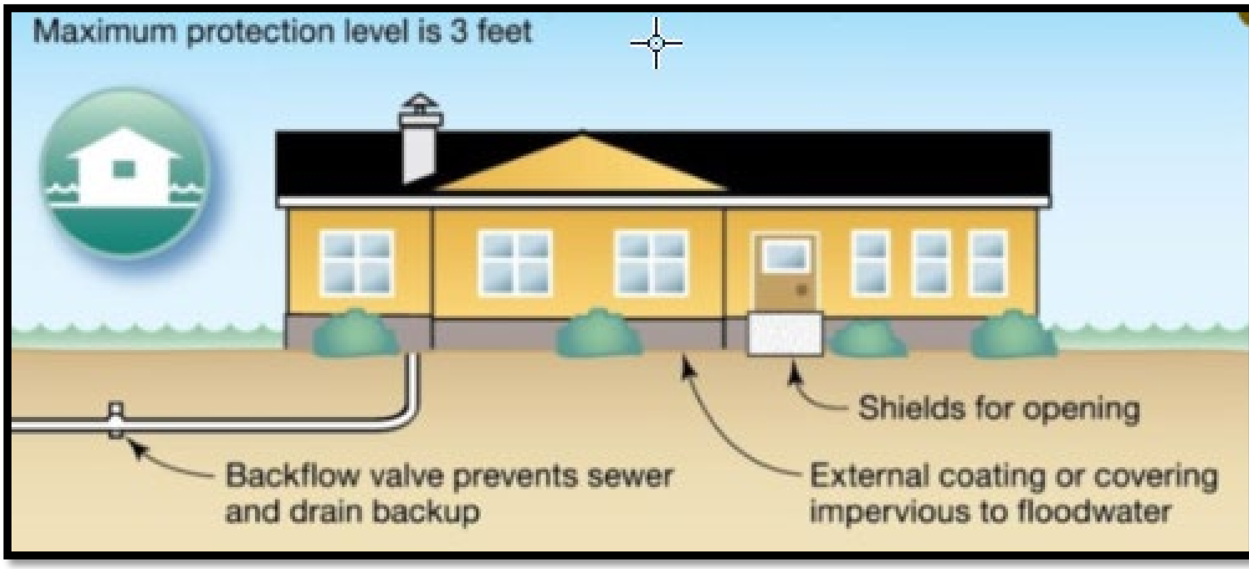

Figure 9-3: Dry Floodproofing Depiction (FEMA 551, 2007)

9.1.2 Wet Floodproofing

Wet floodproofing is a combination of mitigation measures taken to reduce damage to finishes, utilities, and equipment while allowing water to enter the structure. Structures zoned as Commercial or Residential are eligible. All materials below the flood elevation must be water resilient. To ensure resilience, interior finishes such as gypsum sheetrock wall board and carpet will be replaced. Examples of other measures include raising equipment and adding flood vents to the exterior (See Figure 9-4: Wet Floodproofing Depiction (FEMA 551, 2007)

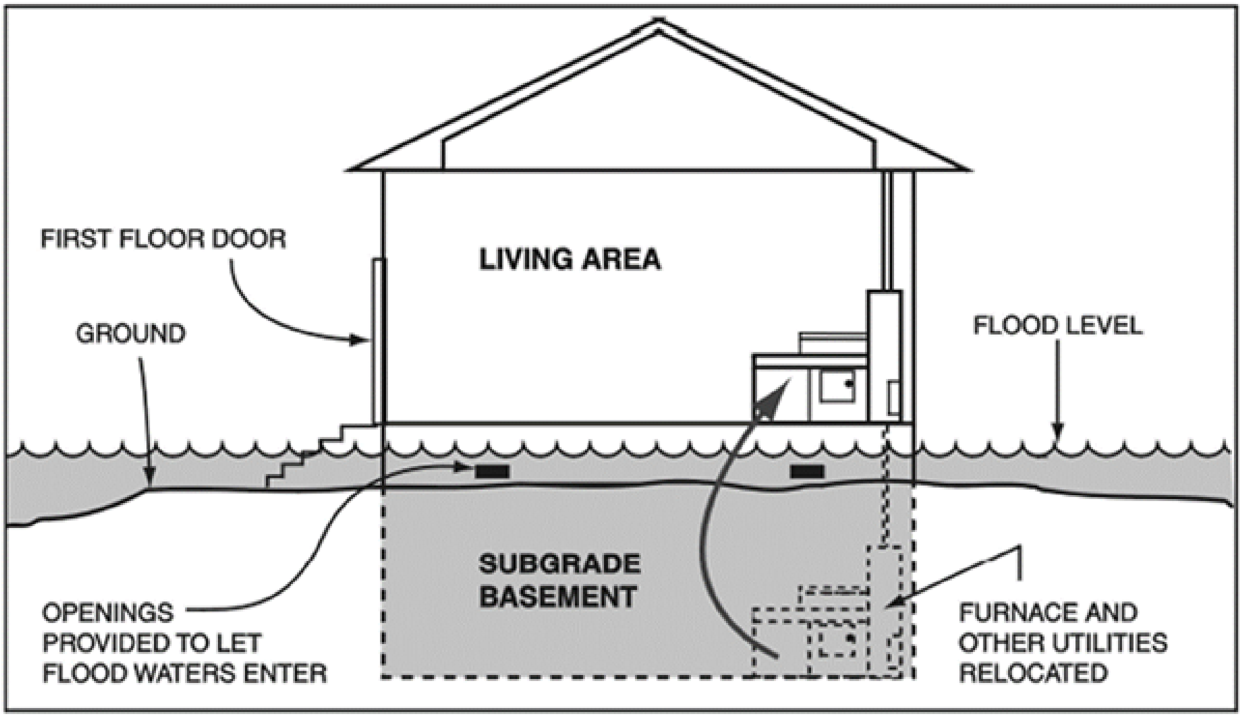

Figure 9-4: Wet Floodproofing Depiction (FEMA 551, 2007)

9.1.3 Raising Structure in Place

Raising a structure in place is accomplished by permanently lifting an existing structure to the point that the lowest occupied floor will not flood. Only structures zoned as Residential are eligible to be raised. Structures are to be raised targeting the Base Flood Elevation (BFE). The entire structure will be lifted and placed on a new foundation. Parking, building access, and storage will be the only occupancies permissible below the BFE. All utilities and mechanical equipment, including HVAC and water heaters, will also be raised. Flood vents will be installed for any enclosed spaces below the BFE. Enclosed spaces below the BFE will be Wet Floodproofed and must be kept free of items that could be damaged during a flood.

9.2 FWEEP

Flood Warning and Emergency Evacuation Plan (FWEEP) relies upon stream gages, rain gages, and hydrologic computer modeling to determine the impacts of flooding for areas of potential flood risk. A flood warning system, when properly installed and calibrated, can identify the amount of time available for residents to implement emergency measures to protect valuables or to evacuate the area during serious flood events. Local officials are encouraged to develop and maintain a flood emergency action plan (EAP) that identifies hazards, risks, and vulnerabilities, and encourages the development of local flood risk mitigation. The EAP should include the community's response to flooding, location of evacuation centers, evacuation routes, and flood recovery processes.

For purposes of this study, USACE will support the community when they prepare their floodplain management plan. USACE will provide the City of Beattyville with inundation mapping and a warning speaker or intercom system in the downtown area.

9.3 STRUCTURE INVENTORY ASSESSMENT

On January 13 and 14, four engineers from the Beattyville PDT—specializing in cost, civil, structural, and geotechnical disciplines—conducted a site visit to Beattyville to gather field data. The visit aimed to further verify the assumptions made during previous project milestones. The team collected both exterior and interior data (when accessible) on 25 structures: 6, 10, 10.1, 17, 19, 21, 21.1, 21.2, 21.3, 21.4, 25, 26.1, 27, 31, 31.1, 35, 36, 36.2, 55, 62, 63.1, 63.2, 63.3, 63.5, 63.6.

During the site visit, the team gathered exterior data including the total number of exterior doors and their respective combined width, as well as the total number and width of exterior windows. Additionally, the team assessed the perimeter conditions surrounding the structure, such as asphalt, concrete, grass, or landscaping and estimated the percentage of applicable coverings around the structure. The respective percentage coverage of these materials around the structure was documented to better understand the surrounding conditions and accessibility.

Interior data collection focused on functional details. The team recorded the number of interior doors and their total combined width, as well as the number of interior rooms and the total length of interior walls. Flooring types were categorized and calculated as a percentage of the total flooring area. Additional data included the number of electrical panels, HVAC equipment, water heaters, bathrooms, and kitchens (if present), providing a comprehensive understanding of the building's interior infrastructure and layout. Backflow preventors and utility modifications will fall under the property owners' portion of responsibility.

It was determined that many of the structures observed during the site visits would be able to have a phased construction approach.

For additional details on this site visit, please refer to Attachment A, which includes estimated floor plan drawings created during the data collection process, as well as an inventory sheet documenting our observations.

9.4 CONSTRUCTION LAYDOWN

Construction laydown is expected for this project. Potential laydown area is at the corner of Begley Street and River Drive. Construction laydown will ultimately be coordinated with the City of Beattyville during the Design and Implementation phase of this project to limit the disturbance to the town.

9.5 RECREATION AND ECOSYSTEM RESTORATION

Recreational additions have been incorporated into the plan. The recreation plan includes a multi-use shelter, 39 parking spaces, a tennis court, a half-court basketball area, a playground, pickleball courts, 21,000 square feet of green space, and over a half mile of walking trails. Ecosystem restoration including native grasses, shrubbery and trees will be incorporated. See Figure 9-5: Recreational and Ecosystem Restoration Area

for the proposed general location of these items. See Figure 9-6 ROW Plan for information regarding the ROW for the Recreation and Ecosystem Restoration.

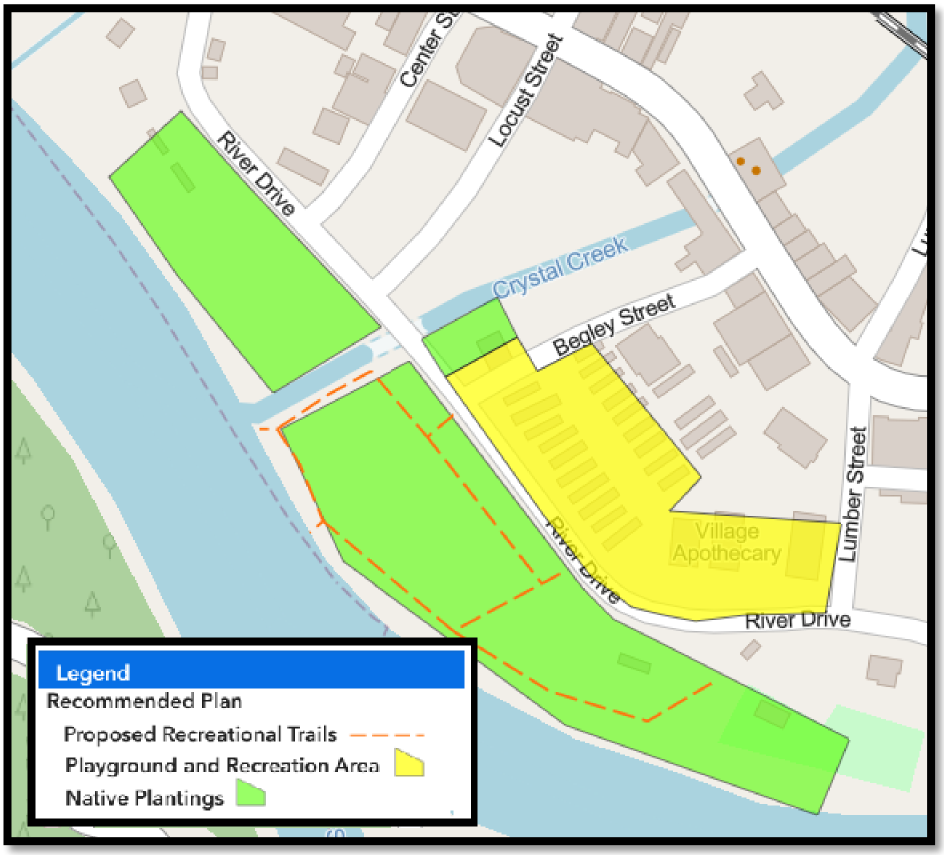

Figure 9-5: Recreational and Ecosystem Restoration Area

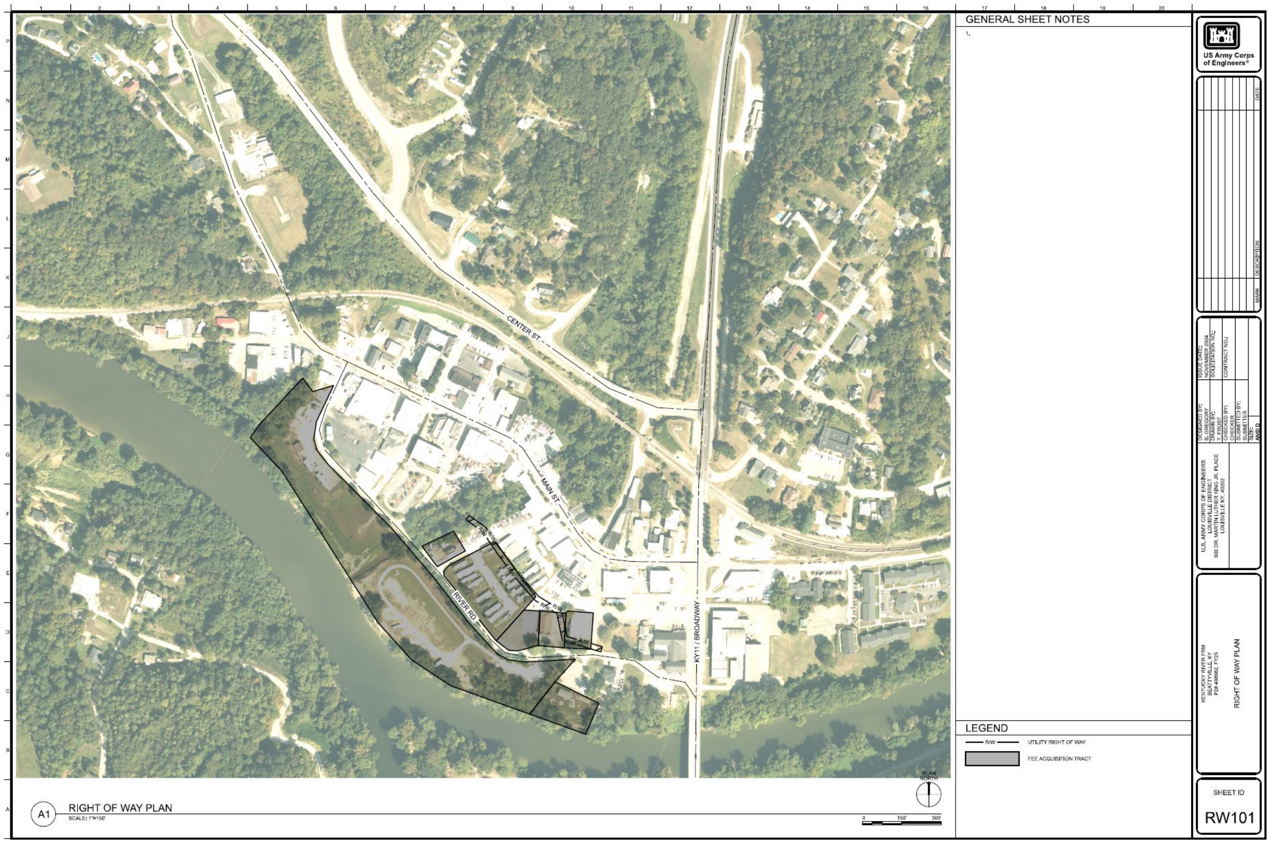

Figure 9-6: ROW Plan

10 HAZARDOUS AND TOXIC MATERIALS

A search of applicable federal databases revealed the following hazardous, toxic, radioactive waste (HTRW) resources within their individual search radii around the project area; no National Priority List (NPL) sites, no Proposed NPL sites, No Delisted NPL sites, no comprehensive environmental response compensation, and liability information system (CERCLIS) sites, no CERCLIS no further remedial action planned (NFRAP) sites, no corrective action sites, no resource conservation and recovery act (RCRA) treatment, storage, and disposal facilities (TSDF), two RCRA very small quantity generators, one used oil program hazardous waste generator, 7 RCRA hazardous waste facilities, and no federal institutional controls.

A search of applicable state databased and open records requests (ORR) revealed the following HTRW resources within their individual search radii around the project area; five Kentucky Department of Environmental Quality (KDEP) Superfund sites, no KDEP Institutional Controls, No KDEP Environmental Covenant Sites, no KDEP Voluntary Remediation Program sites, one KDEP brownfield site, and 48 underground storage tanks sites.

According to the Kentucky Department of Environmental Quality (KDEP) underground storage tank (UST) statewide report, the following resources are in proximity to the project area: 3 active

UST's, 24 storage tanks that were removed, ten active leaking underground storage tanks (LUST), 3 closed LUSTs, 5 duplicate LUSTs, and 3 LUSTs that were removed.

There is a high likelihood of lead-based paint and asbestos within the structures found in proximity to the project area. A Phase I Environmental Site Assessment will need to be performed for any property that will be acquired for project purposes to determine the potential presence of HTRWs. Appropriate measures will be taken to properly handle and dispose of HTRWs encountered.

11 Environmental Objectives and Requirements

An EA is being developed with the feasibility report to determine compliance with all applicable environmental laws and regulations.

12 Operations and Maintenance

Structures acquired by the Lee County, along with recreational facilities and ecosystem features, are to be maintained in perpetuity.

Occupants in structures that have been floodproofed will be responsible for maintaining the integrity of the floodproofing and securing all enclosures when a flood event is forecast.

13 Cost Estimates

13.1 Cost Determination

14 Using the metrics described in Section 13.1 References

14.1 Data

Digital Elevation Model:

- KY From Above: KY Aerial Photography & Elevation Data Program (KYAPED), 2017. https://kyfromabove.ky.gov/
- Land Use Data:
- NLCD 2019 Land Cover (CONUS). Accessed February 2023.
- https://www.mrlc.gov/data/nlcd-2019-land-cover-conus

14.2 Software

- Environmental Systems Research Institute Inc (ESRI), ArcGIS Pro Software: http://www.esri.com/

- HEC-HMS, Version 4.10 Software: https://www.hec.usace.army.mil
- HEC-HMS, Version 4.11 Software: https://www.hec.usace.army.mil
- HEC-RAS, Version 6.3.1 Software: https://www.hec.usace.army.mil
- HEC-RAS, Version 6.4.1 Software: https://www.hec.usace.army.mil
- HEC-SSP, Version 2.2 Software: https://www.hec.usace.army.mil

14.3 Reports, Manuals, and Bulletins

- E.C.B. 2019-8. Engineering and Construction Bulletin. “Managed Overtopping of Levee Systems.” Department of the Army, (24 April 2019) US Army Corps of Engineers, Washington, DC.
- E.M. 1110-2-1619. Engineering Manual. Engineering and Design: “Risk-Based Analysis for Flood Damage Reduction Studies.” Department of the Army, (1 August 1996), US Army Corps of Engineers, Washington, DC.
- E.M. 1110-2-1413. Engineering Manual. Engineering and Design: “Hydrologic Analysis of Interior Areas.” Department of the Army, (24 August 2018), US Army Corps of Engineers, Washington, DC.
- E.R. 1110-2-1150. Engineering Regulation. Engineering and Design: “Engineering and Design for Civil Works Projects.” Department of the Army, (31 August 1999), US Army Corps of Engineers, Washington, DC.
- Federal Emergency Management Agency, Flood Insurance Study: City of Beattyville, Lee County, Kentucky, and Incorporated Areas, (27 October 2022). Report No. 21129CV000B. Washington, D.C.
- Federal Emergency Management Agency, *Guidance for Flood Risk Analysis and Mapping: Levees,* (February 2018). Guidance Document 95. Washington, D.C.
- Federal Insurance Administration, Flood Insurance Study: City of Beattyville, Lee County, Kentucky, (January 1978). U.S. Department of Housing and Urban Development. Washington, D.C.
- FEMA P-312 3rd edition. Homeowner’s Guide to Retrofitting, Six Ways to Protect Your Home From Flooding. (dated June 2014)
- FEMA Technical Bulletin 3, Requirements for the Design and Certification of Dry Floodproofed Non-Residential and Mixed-Use Buildings, Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program. (dated January 2021)
- FEMA 551, “Selecting Appropriate Mitigation Measures for Flood-prone Structures” (dated March 2007)
- HEC-HMS, Hydrologic Modeling System, Technical Reference Manual. Hydrologic Engineering Center, (March 2000), U.S. Army Corps of Engineers, Davis, CA.
- HEC-HMS, Hydrologic Modeling System, User’s Manual. Hydrologic Engineering Center, (February 2016), US Army Corps of Engineers, Davis, CA.
- HEC-RAS, River Analysis System, Hydraulic Reference Manual. Hydrologic Engineering Center, (February 2016), U.S. Army Corps of Engineers, Davis, CA.
- HEC-RAS, River Analysis System, User’s Manual. Hydrologic Engineering Center, (February 2016), U.S. Army Corps of Engineers, Davis, CA.

- HEC-SSP, Statistical Software Package, User's Manual. Hydrologic Engineering Center, (June 2019), U.S. Army Corps of Engineers, Davis, CA.
- Paintsville Lake Dam Water Control Manual. Project Manual for Water Control Management: Paintsville Lake – Paint Creek Basin. U.S. Army Corps of Engineers, (February 2002) Huntington District, Huntington, WV.
- Technical Manual for Levees: Modeling, Mapping, and Consequences. Technical Manual. Modeling, Mapping, and Consequences Production Center, (January 2017). U.S. Army Corps of Engineers, Washington, D.C.
- USACE HEC "Physical and Economic Feasibility of Nonstructural Flood Plain Management Measures" (dated 1973),
- USACE National Flood Proofing Committee EP 1165-2-314 "Flood Proofing Regulations" (dated 1995)
- USACE National Flood Proofing Committee "Non-Structural Flood Damage Reduction Within the Corps of Engineers: What Districts are Doing" (dated 2001)
- USGS Water Supply-Paper 1651-A. "Floods of January-February 1957 in Southeastern Kentucky and Adjacent Areas: Floods of 1957. USGS Water Supply-Paper 1651-A." Department of the Interior (1964). Washington, D.C.

Cost Determination of this Appendix, a cost estimate was developed providing quantities throughout the study area. See Appendix C for a detailed description of the cost estimate for this Study.

15 CLIMATE ASSESSMENT

See Appendix F for the Climate Assessment.

16 SCHEDULE FOR DESIGN AND CONSTRUCTION

16.1 ELIGIBILITY

The USACE and Non-Federal Sponsor will require additional structure-specific analysis during design to determine final eligibility, which could result in structures being removed and/or additional structures being added prior to implementation. Design of specific floodproofing on a structure-to-structure basis will rely on a third-party contractor.

16.1.1 Type 1 (Acquisition)

All structures in the project area in the floodway and structures that have been deemed structurally unsound, regarding their ability to be floodproofed (12 structures), are to be acquired.

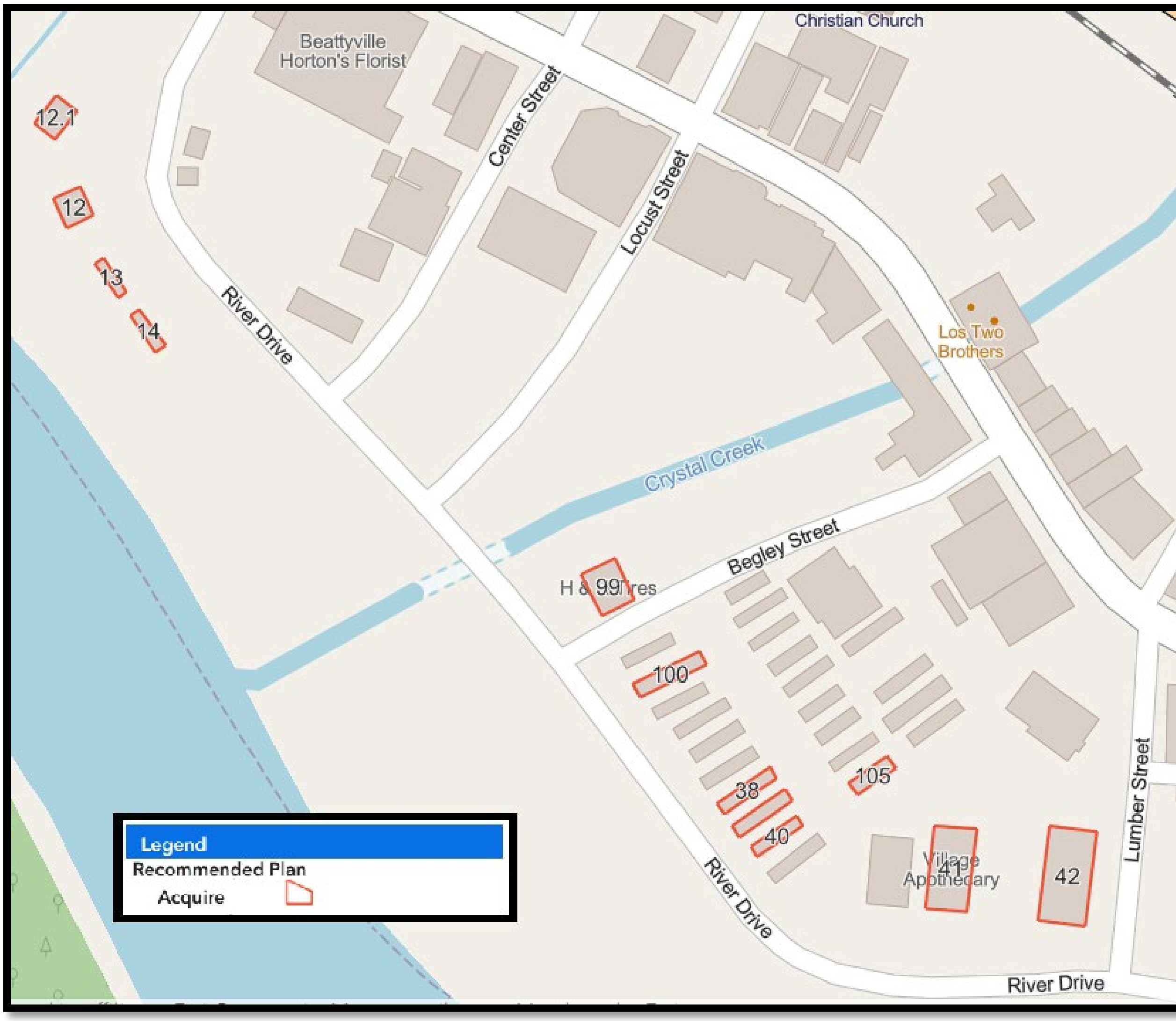

Figure 16.1: Type 1 Structures

16.1.2 Type 2 (Raise In-Place)

All structures in the project area zoned as “residential” are to be raised in place (1 structure).

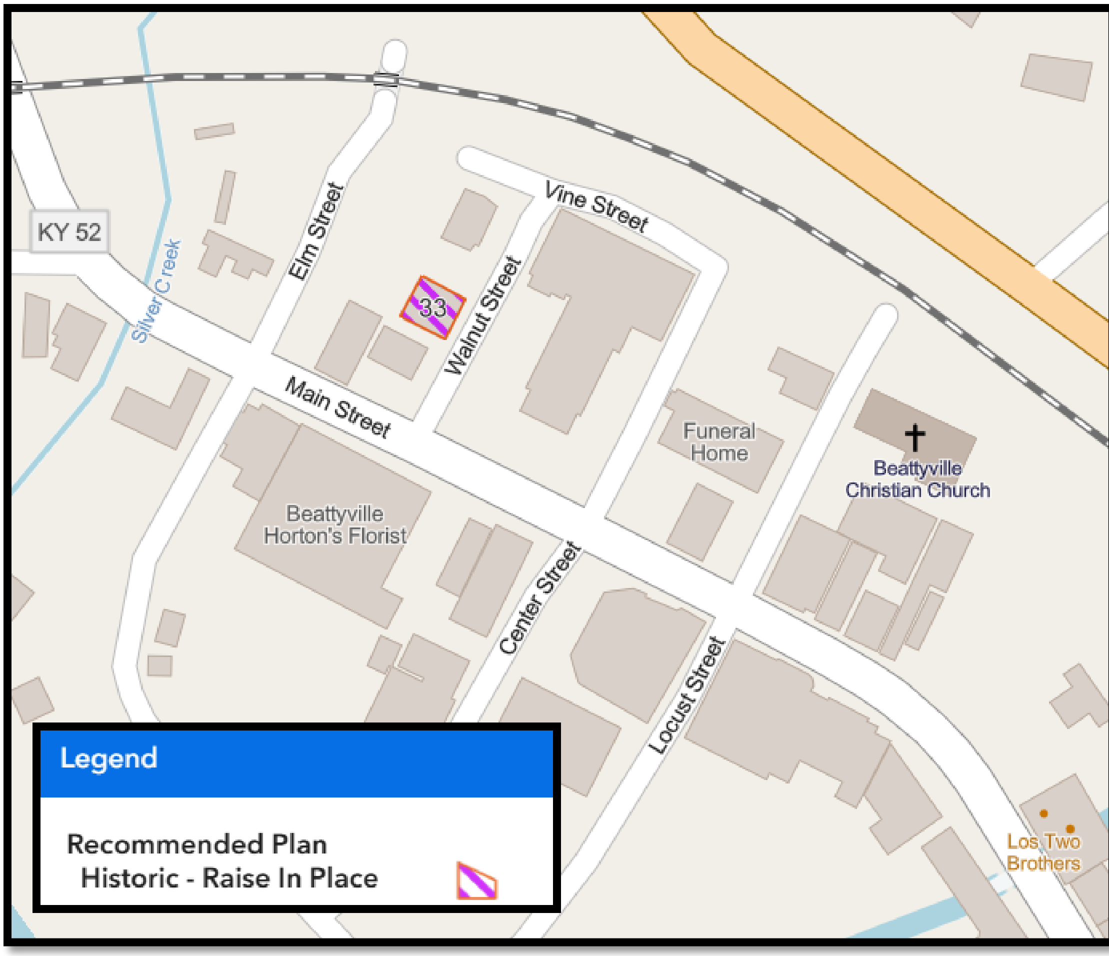

Figure 16.2: Type 2 Structures

16.1.3 Type 3 (Floodproofing)

All remaining structures in project area zoned “commercial” are to be dry floodproofed (10 structures) or wet floodproofed (30 structures).

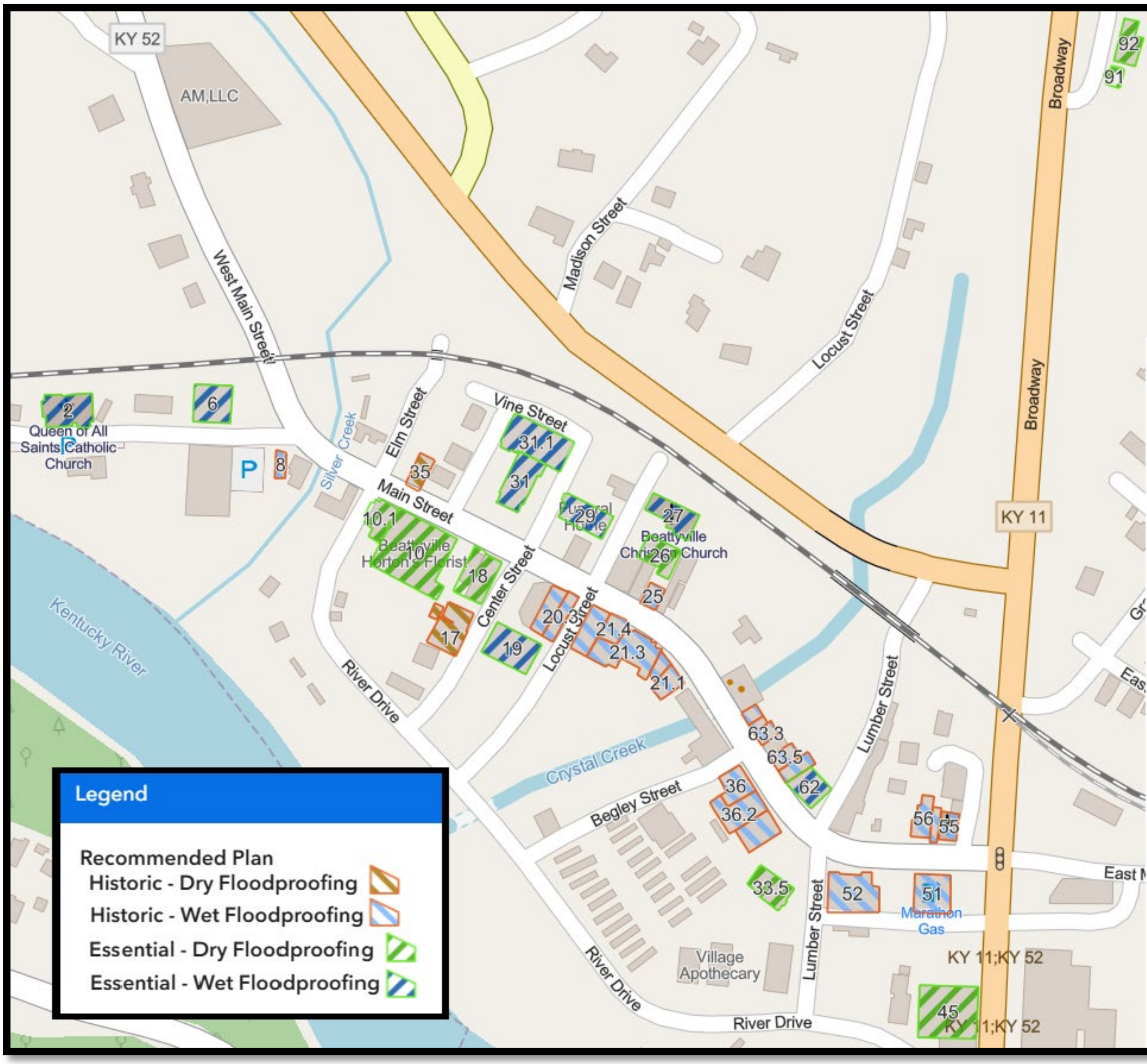

Figure 16.3: Type 3 Structures

17 Design Criteria

- ASCE 24-14
- ASCE 7-22
- FEMA FIRM 21129C0129E (dated 14-AUG-2024)