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Evaluating design criteria for high hazard dams in a changing climate

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Thesis submitted
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at West Virginia University

in partial fulfillment of the requirements for the degree of

Master of Science in
Civil and Environmental Engineering

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ABSTRACT

Evaluating design criteria for high hazard dams in a changing climate

Aaron R. Sutton

With changes in climate, there is the potential for future flooding events to vary in frequency and magnitude. These changes may stress the 432 high hazard dames in West Virginia. The 100-year flowrate is an important design criterion for emergency spillways of high hazard dams. Emergency spillways are designed to be reached only by 100-year flow and above. This work quantified how changes in the 100-year flowrate may affect emergency spillway activation. Peakflow data from the Central Appalachian Ecoregion in WV, taken from 24 USGS gages, were used to analyze changes in the 100-year flowrate.

Flow frequency analysis revealed that for unregulated gages, 100-year flow consistently increased, but for regulated gages, 100-year flow consistently decreased. Reservoir routing was completed at a high hazard dam in Greenbrier County under potential future flow scenarios altering peak inflow (-7%, +6%, +12%, +20%, and +30%). The spillway of the dam was predicted to be reached by approximately a 12% increase in 100-year flow, which was matched and exceeded by historical increases in 100-year flow from unregulated gages of up to 19%. These results suggest that emergency spillway designs need to consider potential changes in 100-year flow.

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Table of Contents

Table of Figures

Table of Tables

1 Introduction

1.1 Background

West Virginia (WV) has experienced an increased frequency of major rainstorms and floods partly due to climate change, and further increases are predicted due to past temperature increases in the state of 0.5°F to 1°F over the last century (USEPA 2016). The increased rainfall results in increased flow through WV streams, putting increased stress on the 432 state high hazard dams (from National Inventory of Dams), dams that can cause loss of life if they fail (USACE 2018b). More than 60% of the dams in WV were built before 1970, so many of these dams are nearing the end of their design life of 50 years (NRCS 2016b). To ensure effective functioning of these dams, they will soon require rehabilitation.

Recently, there have been flood-related disaster declarations in WV almost every year (USEPA 2016). The emergency spillway of a high hazard dam is intended to operate only once each 100 years, known as 100-year storm. With the intense rainfall experienced in WV, the 100-year storm and 100-year flow have risen, resulting in increased flow toward dams (Milly et al. 2002). Additionally, urban development and other land use changes have increased the area of impervious surfaces in WV, resulting in decreased infiltration and increased runoff into streams (Ferrari et al. 2009). Surface coal mining is one of these significant land use changes, increasing runoff during extreme events (Evans et al. 2015). The increased flow in streams means that flood risks are higher than the past.

1.2 Objectives

The objective of this research was to determine if dam design criteria for high hazard dam emergency spillways remain adequate, despite changing climate with increased extreme precipitation events. The first task was to determine how the 100-year flood has changed over time in the Central Appalachian Ecoregion (CAE) within WV. A number of scenarios were generated with the 100-year flow data in WV in the CAE. The second task was to determine how the changes in 100-year flowrate may impact dam design criteria for an existing dam. Specifically, Howard Creek Dam, an earth/rockfill dam in Greenbrier County, was examined to find how changes in the 100-year flood may impact flow in the emergency spillway. The dam is in a rural watershed with few changes in land use over the study period, so climate changes were the focus of research.

2 Literature Review

2.1 Hydrological design criteria for dam design

2.1.1 100-year flood

A 100-year flood is a flood that has a 1% annual exceedance probability (Holmes 2017). The measurement of streamflow data over at least 30 years gives the average over that time period, and the 100-year flood is calculated from these data. The 100-year portion is known as the average recurrence interval of such a flood, so there would be a 1-in-100 chance that a flood with equal or greater discharge would occur in any given year. The 100-year flood is a crucial consideration for dam design because it is used to design emergency spillways in high hazard dams (WVDEP 2009). The reservoir pool elevation that a 100-year flood is projected to reach is the elevation at which the crest of an emergency spillway of a high hazard dam is placed. Urban development and dams can have a strong impact on the occurrence of floods, so flow data before a large-scale human change can become obsolete after development changes are made (USGS 2018).

2.1.2 Probable maximum precipitation and probable maximum flood

Another important metric for designing a dam is the probable maximum precipitation (PMP). PMP is the most precipitation that could feasibly occur in a specific time period in a specific location and time of year without considering long-term changes in climate (Tetzlaff and Zimmer 2013). Some high-hazard dams need to be designed to handle these PMP storms to prevent catastrophic failure and minimize extreme flooding events. From PMP and local watershed qualities such as soil moisture and upstream regulation, the probable maximum flood (PMF) can be calculated. PMF is the largest flood that could feasibly occur in a specific area and is often considered in dam design (LaRocque 2013).

2.1.3 Dam regulations

High hazard dams are the dams which have the largest consequences of failure. Hazard designation is based on the failure of a dam during worst case scenarios, such as floodflow conditions. The risk of dam failure is calculated from probability of failure and consequence of failure (FEMA 2014). Notably, it is possible for hazard designation of a dam to change due to downstream development. In West Virginia, more than 60% of the dams were built prior to 1970. West Virginia has 285 high hazard dams as of 2017. Now, these dams are nearly at the end of their design life of 50 years, and they may require rehabilitation (NRCS 2016b). If the frequency and intensity of storms are increasing, current spillway designs may not be sufficient for flood management.

The following are the hazard classifications for dams in West Virginia (WVDEP 2009):

1. Class 1: For class 1 (high hazard) dams, failure may cause loss of human life, or major damage to infrastructure. These are designed to the 6-hour PMP and no less than 70% PMP. Their spillways should operate no more than once in 100 years for a 6-hour rainfall event.
2. Class 2: For class 2 (significant hazard) dams, failure may cause minor damage to infrastructure, but loss of human life is unlikely. These are designed to 50% of the 6-hour PMP, and no less than 25% PMP. Their spillways should operate no more than once in 50 years for a 6-hour rainfall event.
3. Class 3: For class 3 (low hazard) dams, failure would cause little damage to adjacent property. The main loss would be to the dam itself. These are designed to 25% of the 6-hour PMP, and no less than the 6-hour 100-year storm. Their spillways should operate no more than once in 25 years for a 6-hour rainfall event.
4. Class 4: For class 4 (negligible hazard) dams, failure causes almost no harm. Class 4 dams are often associated with other, larger dams. These are designed to the 6-hour 100-year storm. As with class 3 dams, their spillways should operate no more than once in 25 years for a 6-hour rainfall event. For dams of any hazard designation that are designed to overtop, they should be designed not to overtop more than once in 100 years for a 6-hour rainfall event (WVDEP 2009).

Notably, the Federal Emergency Management Agency has since merged class 3 and class 4 dams, resulting in only three classifications at the national level, but WV has not updated its dam safety rule since 2009 (FEMA 2014).

2.2 Extreme events and flooding in Central Appalachia

In the Northeastern U.S., rising trends have been found for extreme precipitation and streamflow events. In the 2000s, the frequency of extreme precipitation events was higher than any previous decade on record. However, the 1970s and 2000s had similar peaks in the frequency of extreme streamflow events. The most significant trends are observed when looking only at the changes between the warm seasons of each year. From 1980 to the 2000s, the frequency of warm season extreme precipitation events increased 30-40%, and the frequency of warm season extreme streamflow events doubled (Frei et al. 2015).

The number of "2-day, 1-in-5-year storms" from 2001 to 2012 is nearly double the average from 1901-2012, indicating a strong increase in extreme precipitation events in the 2000s (Walsh et al. 2014). The intensity of extreme precipitation events is increasing as well. In the heaviest 1% of daily rainfall events from 1958 to 2012 in the Northeast which contains WV, the amount of precipitation has increased by 71% (Walsh et al. 2014).

In a study by Milly et al. (2002), climate change, defined by a projected quadrupling of atmospheric CO_2, is predicted to increase the risk of 100-year floods in several large (>200,000 km^2) river basins around the world. One of these basins is the Ohio River Basin which encompasses most of West Virginia. This basin is predicted to have a 2.3% chance each year of what is currently only a 1% AEP flood.

One study predicts that climate change will increase the frequency of extreme precipitation events in the Northeast, and the average yearly flood damages in the Northeast are expected to rise by $750 million by 2100 as a result. The southern Appalachians and Ohio River Valley are expected to experience 2 to 5 times more 100-year floods by 2100. The model accounts for the projected changes in frequency and adjusts the 100-year flood over time (Wobus et al. 2017).

Another study uses General Circulation Models (GCMs) for large-scale climate modeling and prediction (temperature and precipitation) and scales them down with statistical methods. These GCMs were used to identify climate extremes in the northeastern U.S. under different emissions scenarios from 2050-2099, comparing them to results from 1950-1999. As emissions increased, the number of warm days in the region increased dramatically. In WV (Ohio River Valley), the following results were found for the change in the number of extreme warm days per year ("number of days with maximum temperature higher than 90th percentile of daily maximum temperature") across three emissions scenarios:

- Low emissions: 30-40 more warm days
- Moderate emissions: 50-70 more warm days

- High emissions: 80+ more warm days.

Because of the increase in number of extreme warm days, an increase in precipitation intensity is also projected in the Ohio River Valley. Across the northeastern U.S., the variability from year to year of extreme climate events is predicted to increase, making preparation for these events much more difficult (Ning et al. 2015).

In the Appalachian Region, surface coal mining has impacted the hydrology of those areas through changes in topography, soil structure, and vegetation. The peakflows during extreme precipitation events at mined or recently reclaimed watersheds have increased (Evans et al. 2015). The mechanisms for these mining influences are similar to the hydrological impacts of urbanizing an area, resulting in low infiltration and high runoff (Ferrari et al. 2009). However, there is a lack of understanding of the effects of mining on discharge during different intensities of precipitation events because it is likely that the varying intensities activate different flow paths (Murphy et al. 2014). Messinger (2003) recorded the per-unit-area peakflows from two adjacent sites in West Virginia, one mined and one unmined, and found that high intensity storms with rainfall greater than 2.5 cm/hr had higher peakflows at the mined site as found above. However, storms of lower intensity had higher peakflows at the unmined site. These differences are likely due to soil saturation differences between the two sites.

3 Methods

For the first task, 100-year flow frequency analysis was completed for 24 USGS gages using HEC-SSP (USACE 2018a). Eighteen scenarios were generated for peakflow change over time. For the second task, reservoir routing was used to find the water surface elevations reached at Howard Creek Dam under the scenarios from the first task.

3.1 100-year flow frequency analysis

For this task, peak annual flowrate data were compiled from 24 existing United States Geological Survey (USGS) stream monitoring stations (USGS 2018). From these flowrate data, the 100-year flood was calculated for these existing gage stations. The criteria for selecting these stations was as follows: 1) station reports peak annual flowrate; 2) station has a long period of record (at least 30 years) without gaps; and 3) station is located in WV and the Central Appalachian Ecoregion. The selections were accomplished using Excel and ArcMap. Then ArcMap was used to determine whether each selected gage was located in a regulated or unregulated channel (Figure 1). A regulated channel was defined as a channel with a dam anywhere on the length of the channel; an unregulated channel was defined as a channel without any dams. Ecoregions are expected to be relatively uniform in climate. The Central Appalachian Ecoregion is known for its mountains and environmental diversity, and the ecoregion was chosen because of its associations with surface coal mining and high variations in annual precipitation from year to year (USEPA 2018).

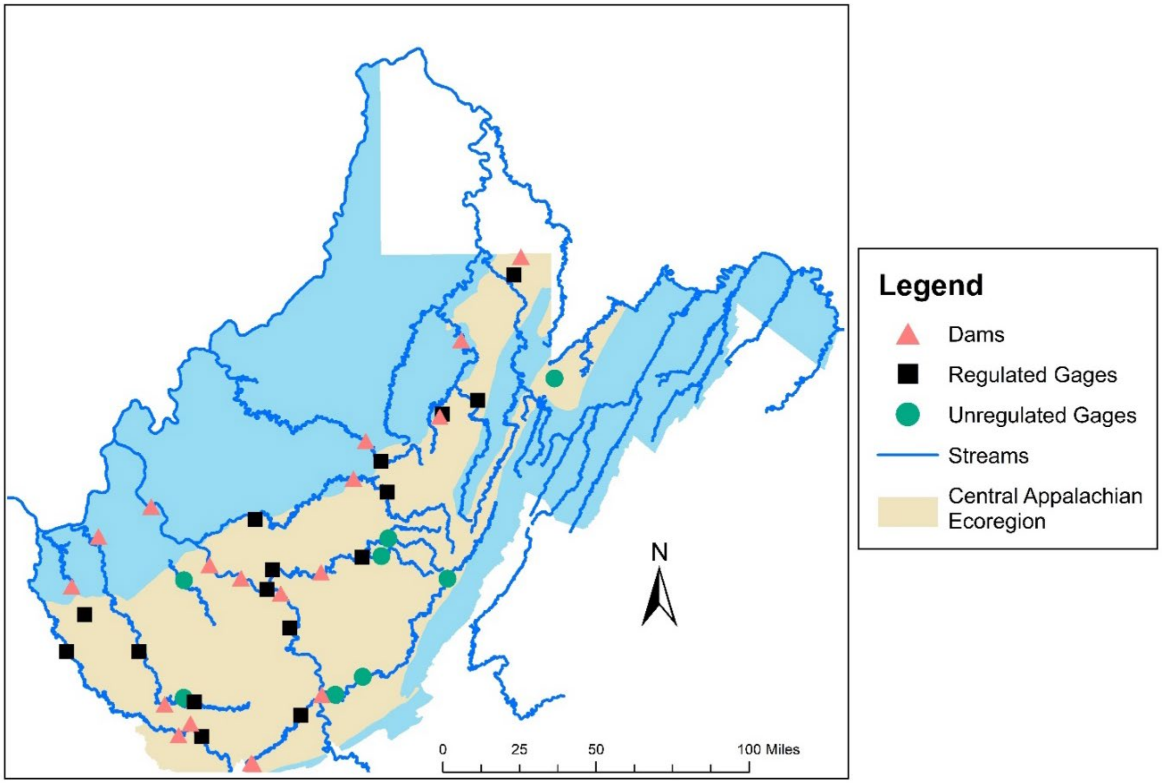

Figure 1: Map of selected USGS gages with streams and relevant dams

This selection process resulted in a total of 24 gages with varying periods of record. All periods of record extend to the present (Table 1) (Figure 2). Sixteen of these gages were regulated due to dams on the stream (Table 2), and eight were unregulated. The drainage areas of the gages ranged from approximately 14 to 8,371 mi^2. One gage station, Greenbrier River at Durbin, was added only to a second analysis (Table 3) which used the four unregulated gages in the Greenbrier watershed because Howard Creek Dam is located in the Greenbrier watershed.

Table 1: USGS Gages used in larger analysis

USGS Site Number	Station Name	Abbrev.	Years of record	Year range	Drainage area (mi^2)	Regulated
03066000	Blackwater River at Davis	BRD	96	1922-2017	85.9	N
03182500	Greenbrier River at Buckeye	GRB	88	1930-2017	540	N
03183500	Greenbrier River at Alderson	GRA	122	1896-2017	1364	N
03184000	Greenbrier River at Hilldale	GRH	82	1936-2017	1619	N
03186500	Williams River at Dyer	WRD	89	1930-2017	128	N
03187500	Cranberry River Near Richwood	CRR	34	1984-2017	80.4	N
03198500	Big Coal River at Ashford	BCA	87	1931-2017	391	N
03202750	Clear Fork at Clear Fork	CFC	43	1975-2017	126	N
03051000	Tygart Valley River at Belington	TVR	110	1908-2017	406	Y
03052500	Sand Run Near Buckhannon	SRB	71	1947-2017	14.3	Y
03070500	Big Sandy Creek at Rockville	BSC	96	1922-2017	200	Y
03151400	Little Kanawha River nr Wildcat	LKR	32	1986-2017	112	Y
03179000	Bluestone River Near Pipestem	BRP	67	1951-2017	395	Y
03185400	New River at Thurmond	NRT	37	1981-2017	6687	Y
03189100	Gauley River Near Craigsville	GRC	32	1986-2017	529	Y
03192000	Gauley River Above Belva	GRV	89	1929-2017	1317	Y
03193000	Kanawha River at Kanawha Falls	KRK	140	1878-2017	8371	Y
03194700	Elk River Below Webster Springs	ERW	32	1986-2017	266	Y
03197000	Elk River at Queen Shoals	ERQ	89	1929-2017	1145	Y
03202400	Guyandotte River near Baileysville	GRN	49	1969-2017	306	Y
03203600	Guyandotte River at Logan	GRL	57	1961-2017	833	Y
03206600	East Fork Twelvepole Creek Near Dunlow	EFT	53	1965-2017	37.9	Y
03212750	Tug Fork downstrm of Elkhorn Creek at Welch	TFE	32	1986-2017	174	Y
03214500	Tug Fork at Kermit	TFK	88	1930-2017	1280	Y

Note: Y = Yes, N = No

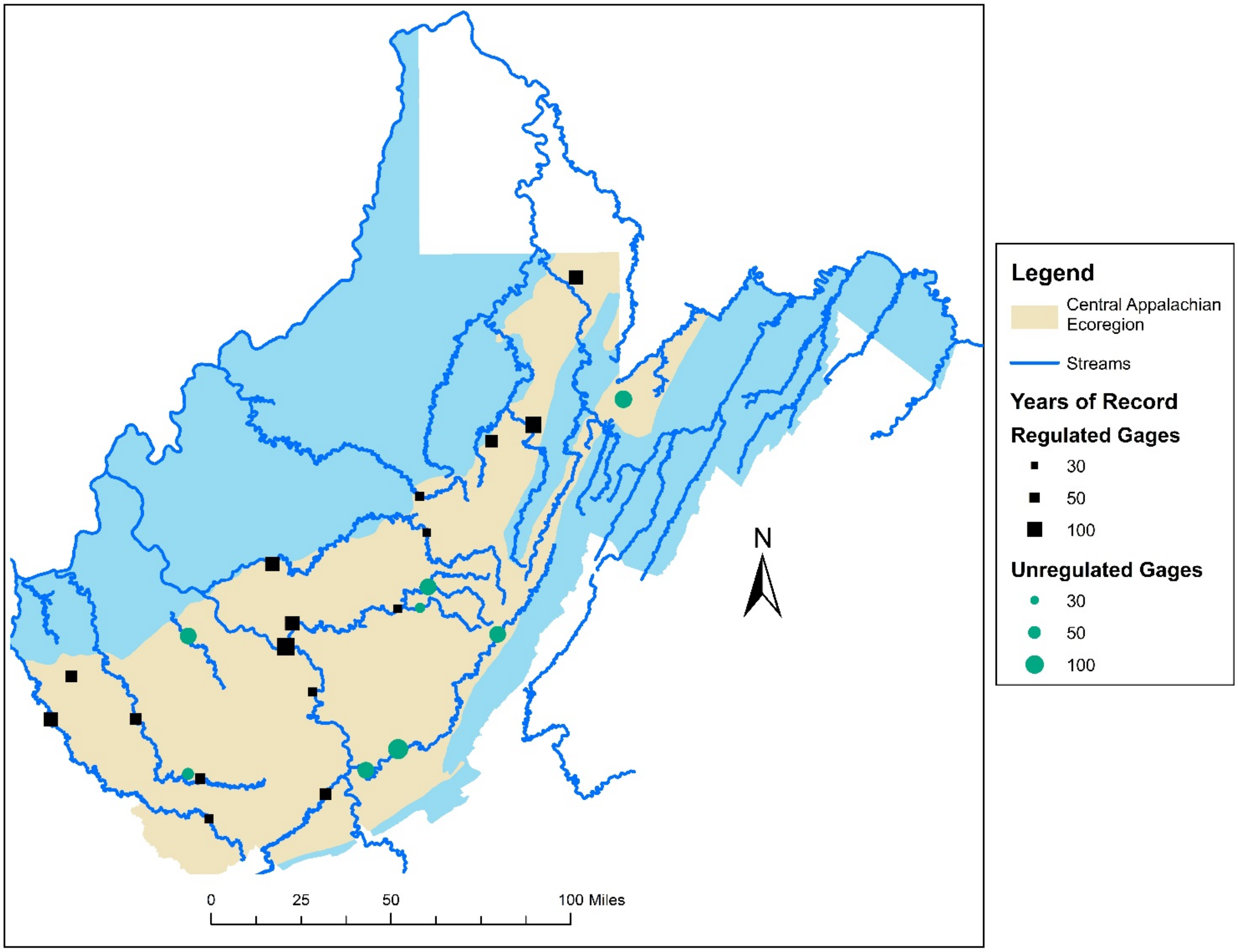

Figure 2: Map of gages with years of record indicated by size of symbology

Table 2: High hazard (H) and significant hazard (S) dam classification on the streams of the gages

Dam Name	River	Year completed	Hazard
Camp Roy Weler Lake	Big Sandy Ck	1965	S
Jimmy Lewis Dam	Bluestone River	1965	H
New Bramwell Dam	Bluestone Rv	1970	H
Old Bramwell Dam	Bluestone Rv	1929	H
East Lynn Dam	East Fk Twelvepole Creek	1971	H
Sutton Dam	Elk River	1960	H
Summersville Dam	Gauley River	1965	H
Hatfield Farm Lake	Guyandotte River	1955	H
R D Bailey Dam	Guyandotte River	1976	H
London L & D	Kanawha River	1934	S
Marmet L & D	Kanawha River	1934	S
Winfield L & D	Kanawha River	1937	S
Burnsville Lake Dam	Little Kanawha River	1976	H
Bluestone Dam	New River	1947	H
Hawks Nest	New River	1936	H
Hall's Farm Pond	Sand Run	1959	H
Twin Branch Dam No.1	Tug Fork	1920	H
Wilmore Dam	Tug Fork	1950	S
Tygart Dam	Tygart River	1938	H

Note: S = Significant, H = High

Table 3: USGS gages used in Greenbrier River analysis

USGS Site Number	Station Name	Abbrev.	Years of record	Year range	Drainage area (mi^2)	Regulated
03180500	Greenbrier River at Durbin	GRD	74	1944-2017	133	N
03182500	Greenbrier River at Buckeye	GRB	88	1930-2017	540	N
03183500	Greenbrier River at Alderson	GRA	122	1896-2017	1364	N
03184000	Greenbrier River at Hilldale	GRH	82	1936-2017	1619	N

Note: N = No

After selecting the gages, relative frequency histograms were graphed from the raw peakflow data to determine the general behavior of the data for each gage with an example graphed below

(Figure 3). To determine the number of class intervals for each histogram, the following formula was used:

$$k = 5\, log_{10}\, n \tag{1}$$

with k equal to the number of class intervals and n equal to the number of data values (Bedient et al. 2013). Complete data are available in Appendix A.

The histogram of each gage is expected to be positively skewed, and all gages have apparent positive skew with the exceptions of the New River at Thurmond and Big Coal River at Ashford gages. The New River at Thurmond gage had peakflow values that were marked as "affected by regulation or diversion" on nearly every year of data (USGS 2018). The Big Coal River at Ashford gage appears to have peakflows with a bimodal distribution which could be due to the proximity of the gage to the Kanawha River, which has many dams, or due to changes in the Big Coal River watershed.

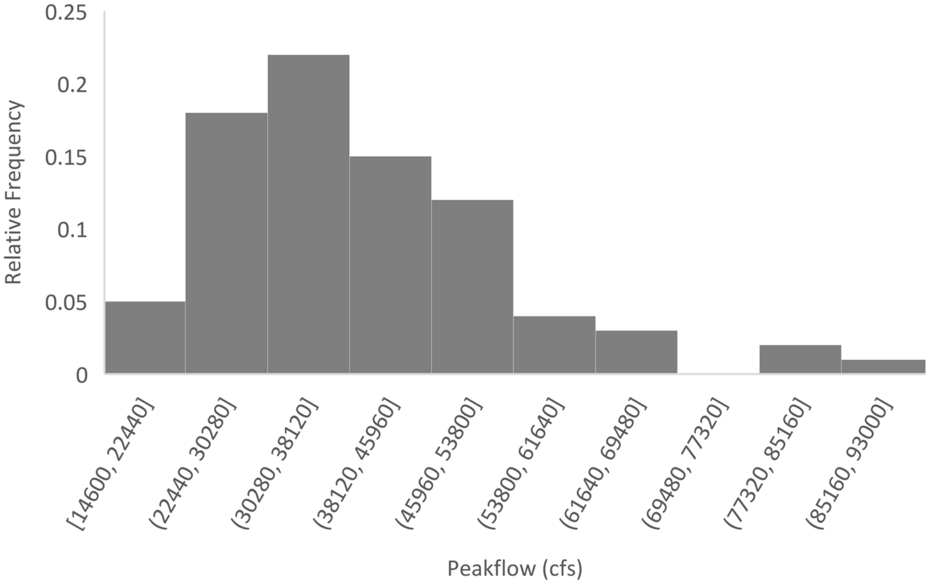

Figure 3: Example relative frequency histogram for Greenbrier River at Hilldale gage

Frequency analysis was completed for all selected stream gages to determine the 100-year flowrate for each stream over its entire period of record. In addition, the 100-year flowrates for each of the following periods of record were calculated: the most recent 20 years of data, the most recent 30 years of data, the most recent 40 years of data, the most recent 50 years of data, the most recent 60 years of data, the most recent 70 years of data, and the most recent 80 years of data.

Hydrologic Engineering Center Statistical Software Package (HEC-SSP), developed by the Army Corps of Engineers, was used to perform these analyses using the flow frequency analysis option which fits the peakflow data from a gage to the log Pearson type III distribution with the Expected Moments Algorithm (EMA) to determine the flow frequency curve (USACE 2018a). Log Pearson type III distribution was used because this distribution is bounded on the left, positively skewed, transformed by logarithms, and implemented in HEC-SSP, allowing the flow magnitudes to be easily computed for a 100-year return period (Bedient et al. 2013). The distribution does generally match the peakflow data from the gages with positive skew. From the results of this analysis, the 1% AEP (100-year flow) was determined. When analyses with different periods of record were compared, they yielded the change in frequency of extreme events over time. For each gage, percent change between analyses of different periods of record was calculated because it gives each gage equal weight in the analysis (Figure 4). The equation used is as follows:

$$P_C = \frac{Q_{new} - Q_{old}}{Q_{old}} \times 100 \tag{2}$$

with P_C = percent change, Q_{new} = new flowrate, and Q_{old} = old flowrate.

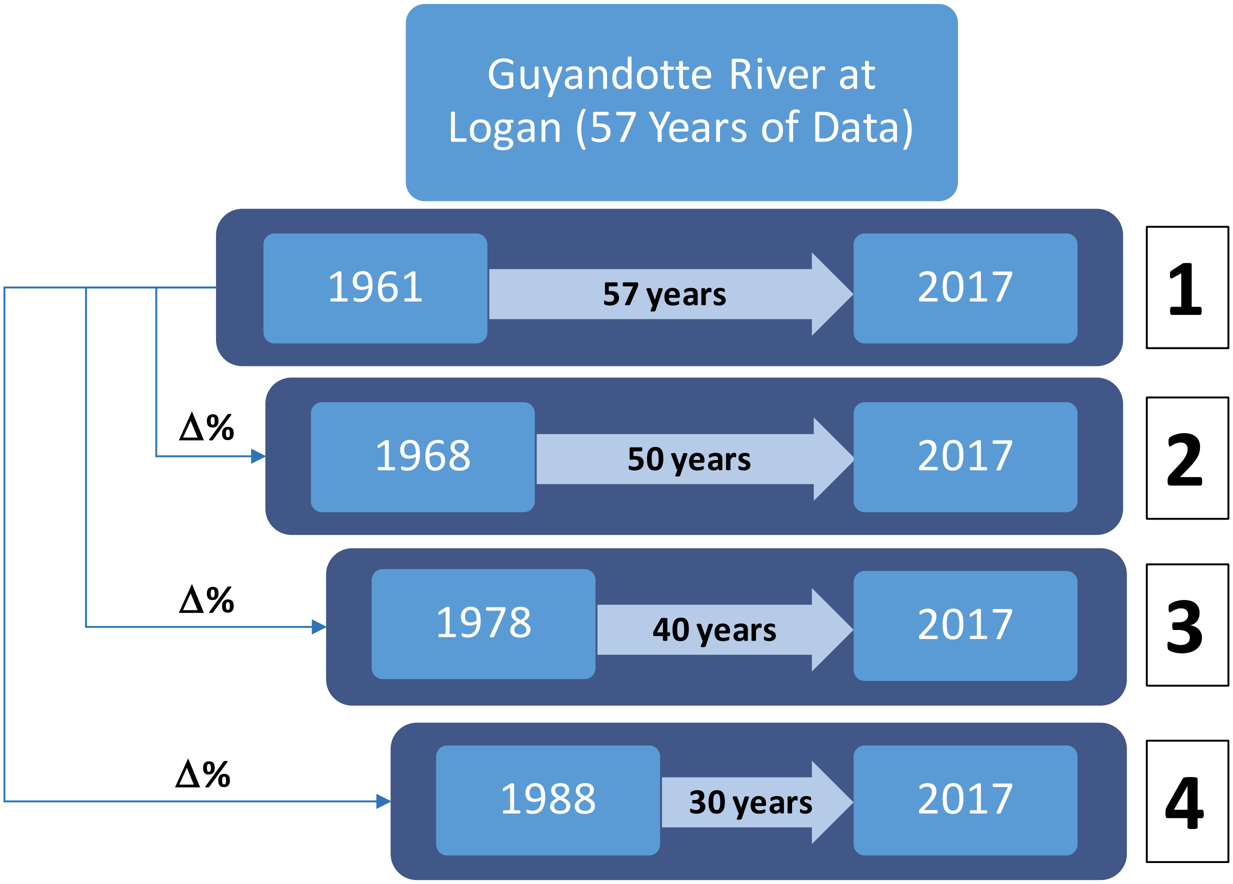

Figure 4: Periods of record used in each HEC-SSP analysis (1 to 4) with 1 using all available years of record and the method of comparison (percent change) for an example gage (Guyandotte River at Logan)

3.2 Reservoir routing

For the second task, changes in 100-year flow design values were evaluated at Howard Creek Dam in White Sulphur Springs, WV. Howard Creek Dam is located in Dry Creek in Greenbrier County upstream of White Sulphur Springs. The reservoir of the dam is Lake Tuckahoe. The Dry Creek watershed has a drainage area of 13.5 mi^2. The land use in the area near the dam is mostly dominated by 9000 acres of forest with around 640 acres of farmland and pastures as well (NRCS 1992b). From the previous analysis, 20 scenarios were developed and the simulated flows were routed through the dam using the storage-indication method (Bedient et al. 2013). The elevations were then compared to elevations of Howard Creek Dam spillways and dam crest (Figure 5) to determine whether the water surface reached these points in the scenarios. The dam elevations and properties were acquired from the design report (NRCS 1992a)

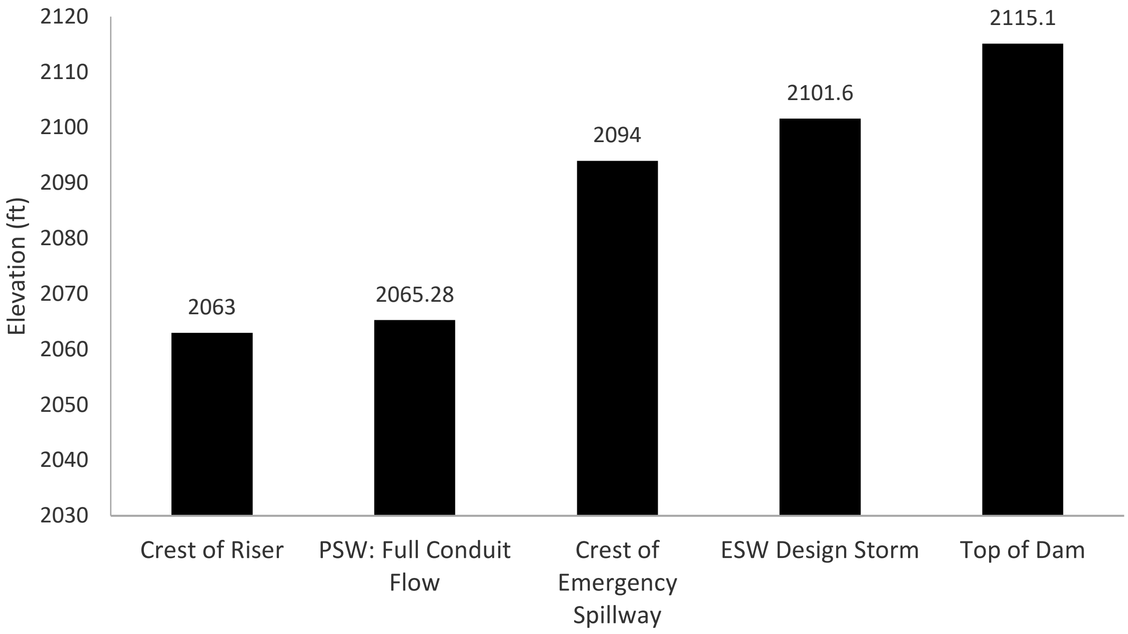

Figure 5: Important elevations of Howard Creek Dam

3.2.1 Reservoir outflow

Outflows were composed of three outlet structures: principal spillway, emergency spillway, and dam crest. The relationships used to model outflow are described in the following paragraphs. The normal principal spillway flow was modeled as a sharp-crested weir and calculated by the following equation (Bedient et al. 2013):

$$Q_{weir} = C_e \frac{2}{3} B \sqrt{2g} (h - h_0)^{1.5} \tag{3}$$

where Q is spillway discharge; C_e is a weir discharge coefficient; g is acceleration due to gravity; B is the length of the spillway perpendicular to flow; h is the input elevation; and h_0 is the elevation of the crest of the spillway. The emergency spillway flow and dam overflow were modeled as broad-crested weirs and calculated by the following equation (Bedient et al. 2013):

$$Q_{weir} = C_e \frac{2}{3} L \sqrt{2g} (h - h_0)^{1.5} \quad (4)$$

where Q is spillway discharge; C_e is a weir discharge coefficient; g is acceleration due to gravity; L is the length of the spillway parallel to flow; h is the input elevation; and h_0 is the elevation of the crest of the spillway. Discharge coefficients and other features were informed by the hydrology report of the dam (NRCS 1992b).

The flow through the dam was broken down into equations (Table 4) describing different stages of flow (Figure 6). The C_e=0.58 for equation 4 was taken from the dam hydrology report and applied to equations 6 and 7 as well due to lack of weir discharge coefficient data on these sections (NRCS 1992b). The lengths of the spillways parallel to flow were L=19.33 ft, L=190 ft, and L=451.3 ft for normal principal spillway flow, emergency spillway flow, and dam crest flow respectively. The dam crest length was acquired from an average of four ArcMap measurements of the full length of the dam at different points along the dam. The following are descriptions of the equations used for flow through the dam:

- From the elevation of the crest of the riser (2063 ft) to the elevation of full conduit flow (2065.28 ft), a weir equation was used for normal principal spillway flow (NRCS 1992b) resulting in equation 4 in Table 4.
- From the elevation of full conduit flow through the principal spillway (2065.28 ft) to the elevation of the crest of the emergency spillway (2094 ft), a quadratic equation fit to the rating table in the hydrology report (NRCS 1992b) was used for full conduit flow through the principal spillway resulting in equation 5 in Table 4.
- From the elevation of the crest of the emergency spillway (2094 ft) to the elevation of the dam crest (2115.1 ft), the full conduit flow through the principal spillway and a weir equation for emergency spillway flow were added together resulting in equation 6 in Table 4.

- Above the dam crest (2115.1 ft), full conduit flow through the principal spillway and flow through the emergency spillway and a weir equation for flow over the dam crest were added together resulting in equation 7 in Table 4.

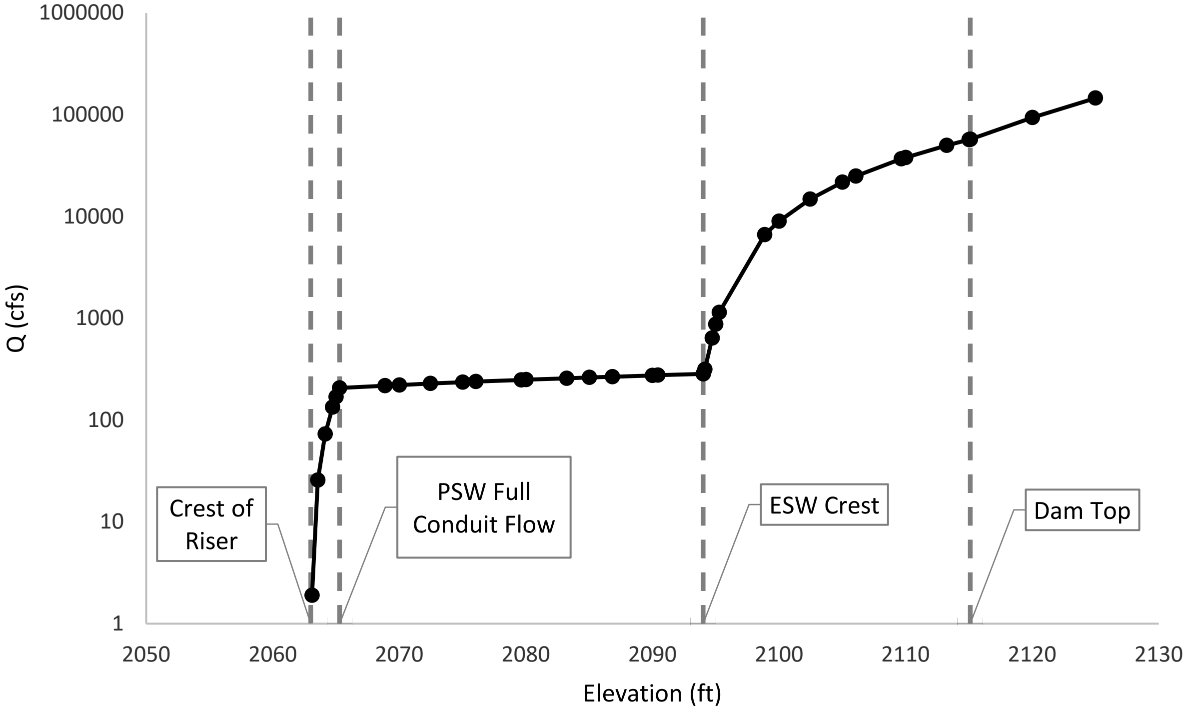

Figure 6: Outflow of the dam vs. elevation using equations 1 through 4

Table 4: Equations used to find discharge as function of elevation

Stage	**Elevation (ft)**	**Flowrate Equation (cfs)**	
Crest of Riser	2063-2065.28	$Q = 59.92(h - 2063)^{1.5}$	(5)
Full Conduit Flow through PSW	2065.29-2094	$Q = -0.014809(h - 2063)^2 + 3.1784(h - 2063) + 199.45$	(6)
Crest of ESW	2094.01-2115.1	$Q = 589.6(h - 2094)^{1.5} - 0.014809(h - 2063)^2 + 3.1784(h - 2063) + 199.45$	(7)
Dam Crest	>2115.1	$Q = 1400(h - 2115.1)^{1.5} + 589.6(h - 2094)^{1.5} - 0.014809(h - 2063)^2 + 3.1784(h - 2063) + 199.45$	(8)

3.2.2 Reservoir storage

Storage-indication curves were calculated using the known cumulative volumes of the reservoir at specific elevations and the discharges from the above equations at those given elevations (Table 5) (NRCS 1992b). The curves had discharge on the y-axis and $Q + 2S/\Delta t$ in the x-axis with S = storage (ft^3) and Δt = time interval (hrs) of the inflow hydrographs. Ponded condition was assumed, meaning that the water surface elevation of the permanent pool was assumed to be the same as the elevation of the crest of the riser.

Table 5: Discharge and storage at specified elevations used for storage-indication curve

Elevation (ft)	Head (H) (ft)	Discharge (Q) (cfs)	Storage (S) (ft^3)
2063	0	0	0
2065	2	169.5	3568430
2070	7	221.0	13837685
2075	12	235.5	26115052
2080	17	249.2	40692384
2085	22	262.2	57873730
2090	27	274.5	77339358
2095	32	997.6	98519505
2100	37	10755.1	121377581
2105	42	26268.0	145965857
2110	47	45858.5	172323974
2115	52	68804.9	201069175

3.2.3 Inflow hydrographs

Inflow hydrographs were calculated for three conditions: i) ESW crest inflow, ii) ESW design storm inflow, and iii) dam overtop inflow. Peak flowrates and time to peak were determined from the hydrology report (NRCS 1992b). Time of fall was calculated assuming a synthetic triangular hydrograph, using the following equation (Bedient et al. 2013):

$$T_F = 1.67 * T_R \tag{8}$$

where T_F = time of fall and T_R = time of rise. The given peak inflow to reach the crest of the emergency spillway was 5618 cfs at 3.8 hours which was the 100-year flow from the design report (Figure 7). The given peak inflow for the ESW design storm was 20387 cfs at 3.8 hours which was calculated from 100-year flow and PMP in the design report (Figure 8). The given peak inflow for reaching the dam crest was 67923 cfs at 3.7 hours which was the PMP from the design report (Figure 9) (NRCS 1992a). The inflow was routed from the reservoir through the dam using the inflow hydrograph and the storage-indication curve iteratively to calculate the outflow through the dam.

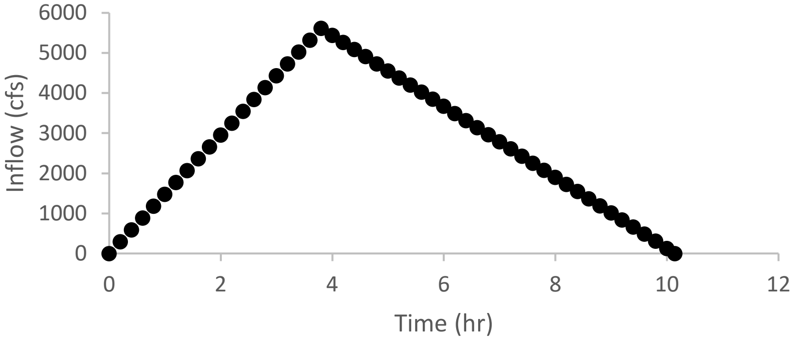

Figure 7: Crest of ESW inflow hydrograph with peak of 5618 cfs at 3.8 hrs, Δt=0.2 hr

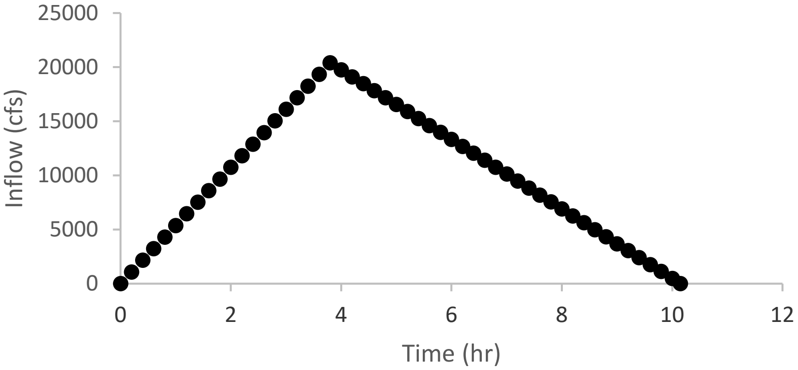

Figure 8: ESW design storm inflow hydrograph with peak of 20387 cfs at 3.8 hrs, Δt=0.2 hr

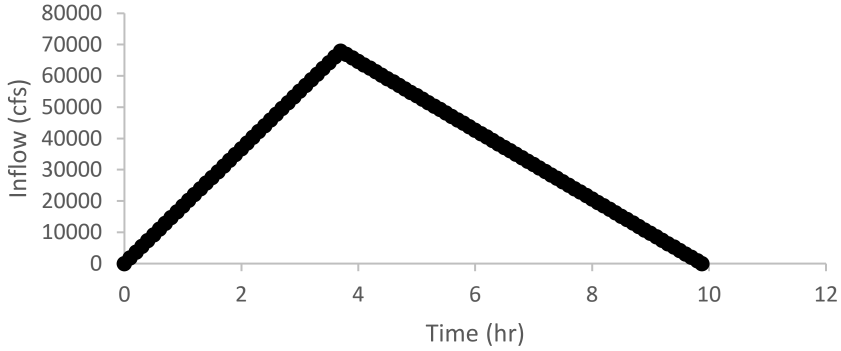

Figure 9: Dam overtopping inflow hydrograph with peak of 67923 cfs at 3.7 hrs, Δt=0.1 hr

3.3 Simulated inflow hydrographs

Several scenarios were developed that simulated changes in peak inflow over time based on the results of the 100-year flow analysis portion of research. The scenarios included the following changes to peak inflow: -7%, +6%, +12%, +20%, +30%. These changes were applied mainly to the crest of ESW inflow because the scenarios were based on 100-year flow. The justification for each scenario will be explained in the results and discussion section.

After applying the above scenarios to the crest of ESW, the scenarios were applied to ESW design storm and dam overtopping as well. However, the changes were only based on 100-year flow, so the scenarios may not fully explain changes to ESW design storm and dam overtopping since they include PMP.

4 Results

4.1 100-year flowrate

The results of the 100-year flood analysis revealed varying patterns for regulated and unregulated gages and depending on period of record. Period of record had a larger effect on 100-year flow for unregulated gages than for regulated gages as shown by the smaller magnitude in percent change for regulated gages (Figures 10 and 11). This makes intuitive sense because the regulated gages are on streams with dams that regulate flow to keep it from changing too drastically over time. In fact, the constant negative percent change in 100-year flow shows that over time 100-year flow for regulated gages has decreased over time despite climate change (Figure 10).

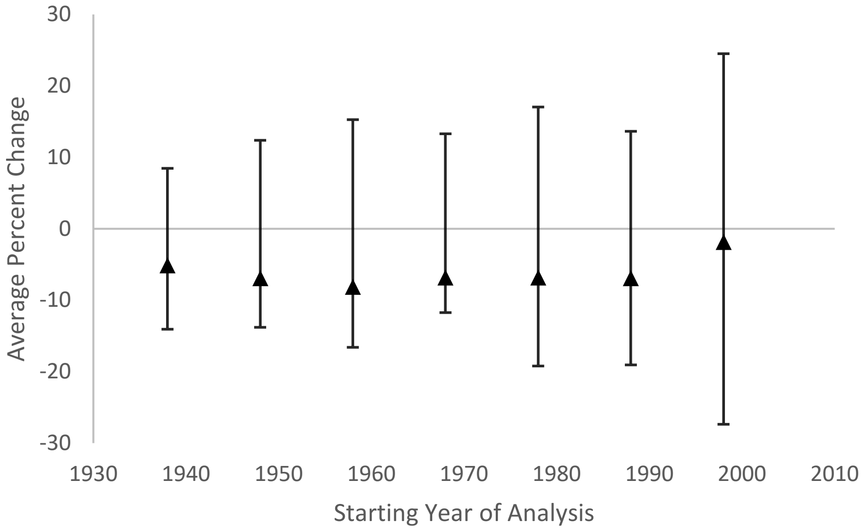

Figure 10: Average percent change in 100-year flow for regulated gages from (1) analysis using all years of record, to (2) analysis with given starting year, with standard deviation error bars

One possible explanation for this effect is the building of dams in the middle of peakflow period of record of the dam, lowering the peakflows for the gages in that stream. An example of this effect is the 100-year flow graph for each analysis on the Kanawha River at Kanawha Falls gage (Figure 11). Dams were built on the Kanawha River in 1901, 1934 (two), and 1937, and these are the years at which there are drastic decreases in 100-year flow. Once these large percent decreases in 100-year flow due to dam construction or simply dam regulation are averaged across all the regulated gages, they could cause the average percent change to be negative.

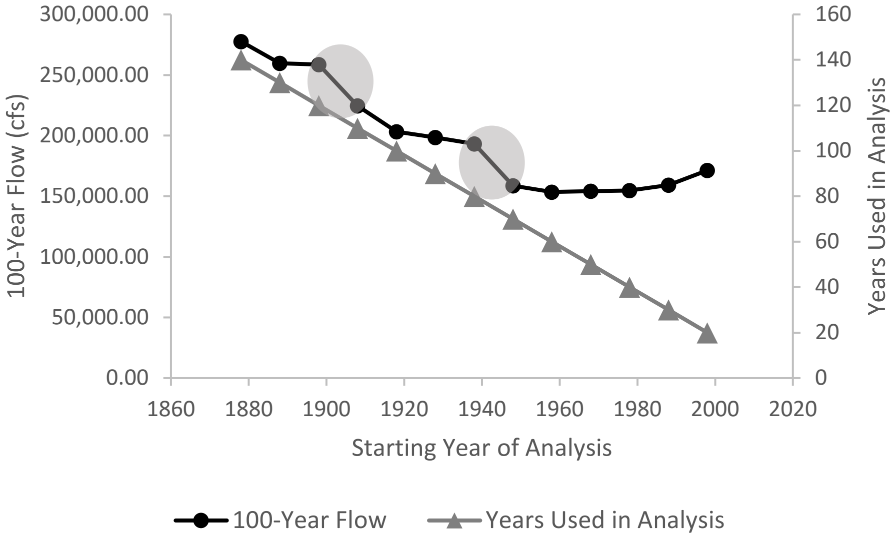

Figure 11: 100-year flow for each analysis on Kanawha River at Kanawha Falls gage with large decreases marked with gray ovals

Though regulated gage 100-year flow has decreased over time, the 100-year flow for unregulated gages has increased as expressed by the consistently positive average percent changes for unregulated gages (Figure 12). Also, the unregulated gages have approximately 47% lower standard deviation than the regulated gages on average.

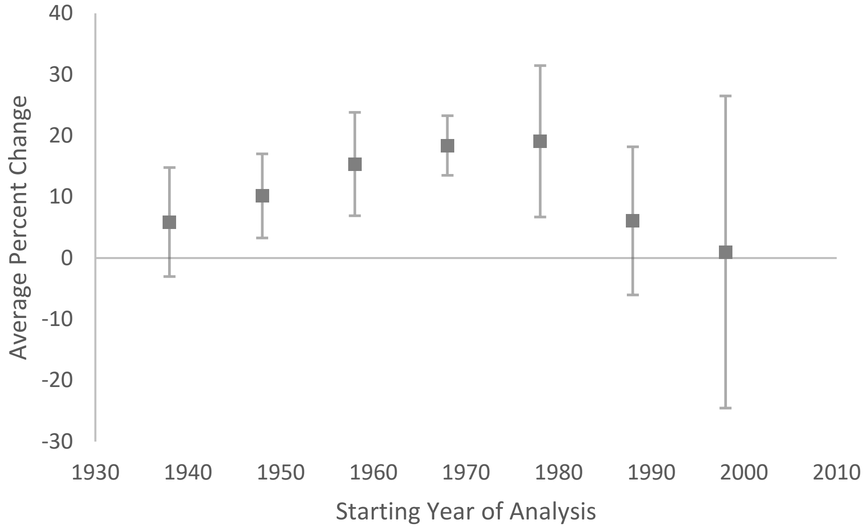

Figure 12: Average percent change in 100-year flow for unregulated gages from (1) analysis using all years of record, to (2) analysis with given starting year, with standard deviation error bars

These increases in 100-year flow are likely explained by climate change, increase in impervious surfaces from urbanization, long term cyclic climate trends, and surface coal mining among other effects (Evans et al. 2015) (Ferrari et al. 2009). As an example, the Greenbrier River at Alderson gage has increased in 100-year flow with each successive analysis until the last couple analyses (Figure 13).

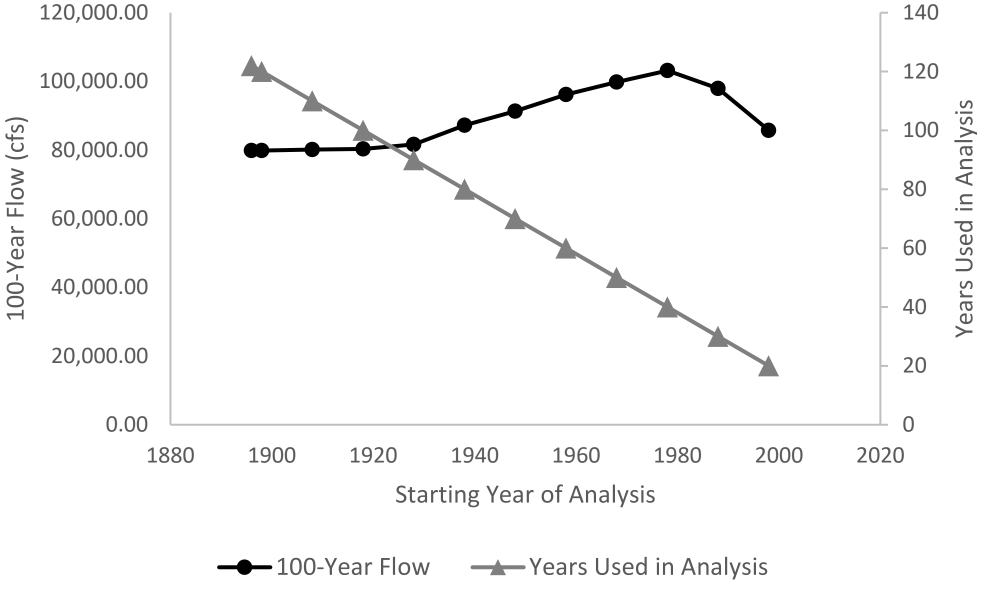

Figure 13: 100-year flow for each analysis on Greenbrier River at Alderson gage

Because of the location of Howard Creek Dam in the Greenbrier watershed, another analysis focused only on the gages in the Greenbrier watershed (Figure 14). This analysis follows the general shape of the unregulated gage analysis but with fewer periods of record and with a higher peak percent change at approximately 30%. Also, the analysis starting in 1998 drops to negative percent change, but this analysis only includes 20 years of data.

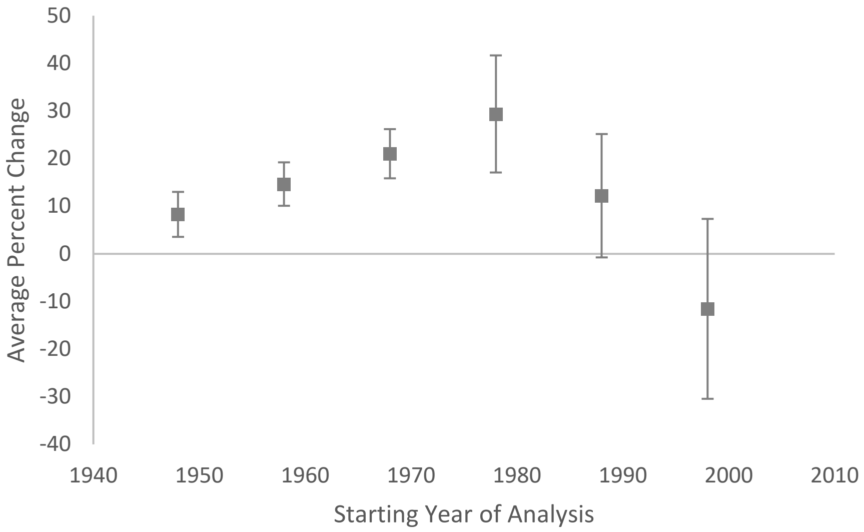

Figure 14: Average percent change in 100-year flow for gages in the Greenbrier watershed from (1) analysis using all years of record, to (2) analysis with given starting year, with standard deviation error bars

100-year flow was also compared for different gages based on drainage area. When 100-year flows from all years of data were plotted vs. drainage area and linearly fitted, the unregulated slope (47.8 cfs/mi^2) was higher than regulated slope (23.5 cfs/mi^2) (Figure 15). However, when the two large drainage areas (6687 mi^2 and 8371 mi^2) associated with regulated gages were excluded, the regulated slope (62.7 cfs/mi^2) rose above unregulated slope (47.8 cfs/mi^2) (Figure 16). This suggests that the two large drainage areas had a substantial impact on the shape of the trendline of the regulated gages, increasing the slope by a factor of approximately 2.7.

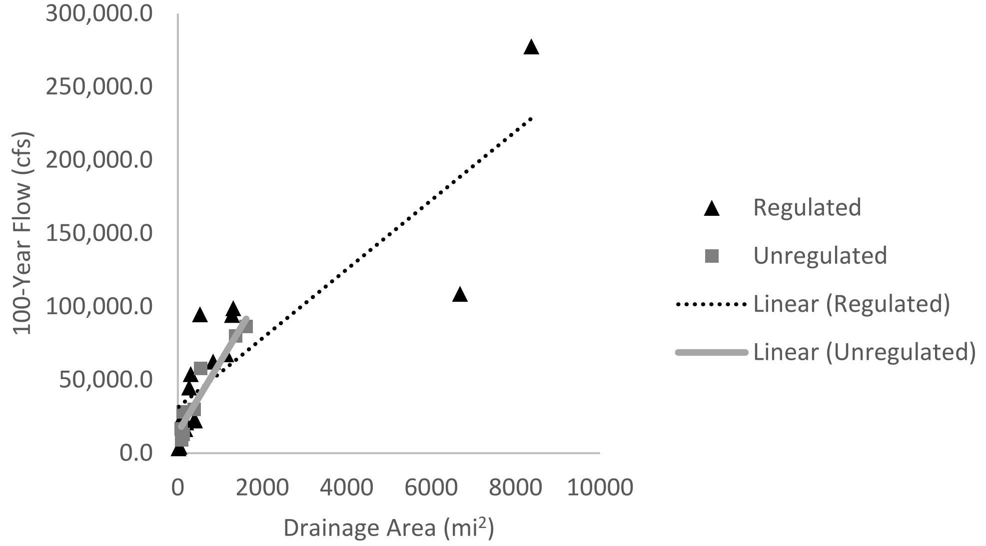

Figure 15: 100-year flow for all years of record by drainage area for regulated and unregulated gages

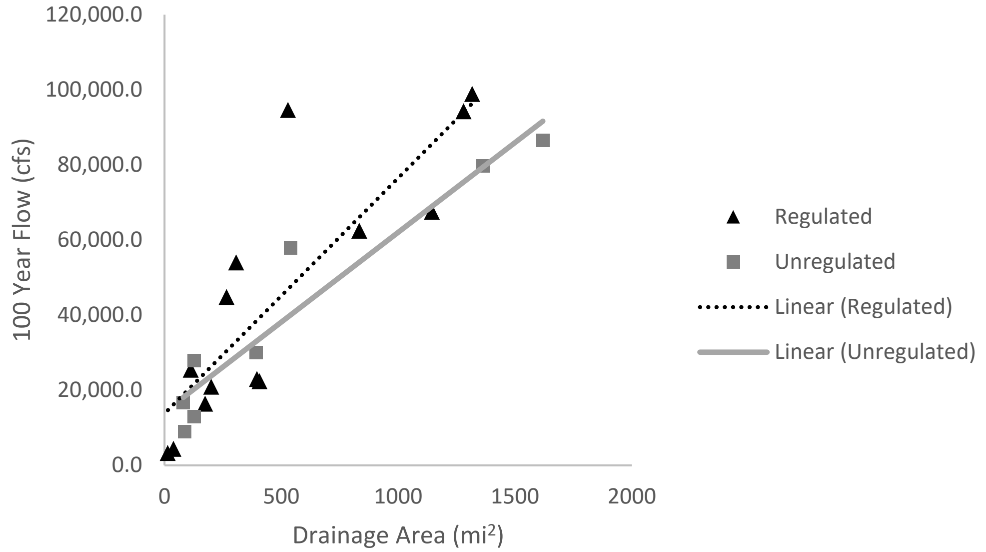

Figure 16: 100-year flow for all years of record by drainage area for regulated and unregulated gages with two large drainage areas removed

From the HEC-SSP analysis, 5% and 95% confidence limits are generated. These confidence limits form a confidence interval that gives the probable range of values of the analysis. The mean of the magnitudes of the confidence intervals for all the regulated and unregulated gages reveals that the confidence intervals tend to shrink as more years of data are used, which is expected since larger periods of record have more data points, reducing the impacts of short-term variations and outliers (Figure 17) (Chapra and Canale 2015). Also, for the 20-year analyses on the regulated gages, the standard deviation is higher than the mean by a factor of 1.15, suggesting that these 20-year data are distributed too widely to maintain explanatory power when combined.

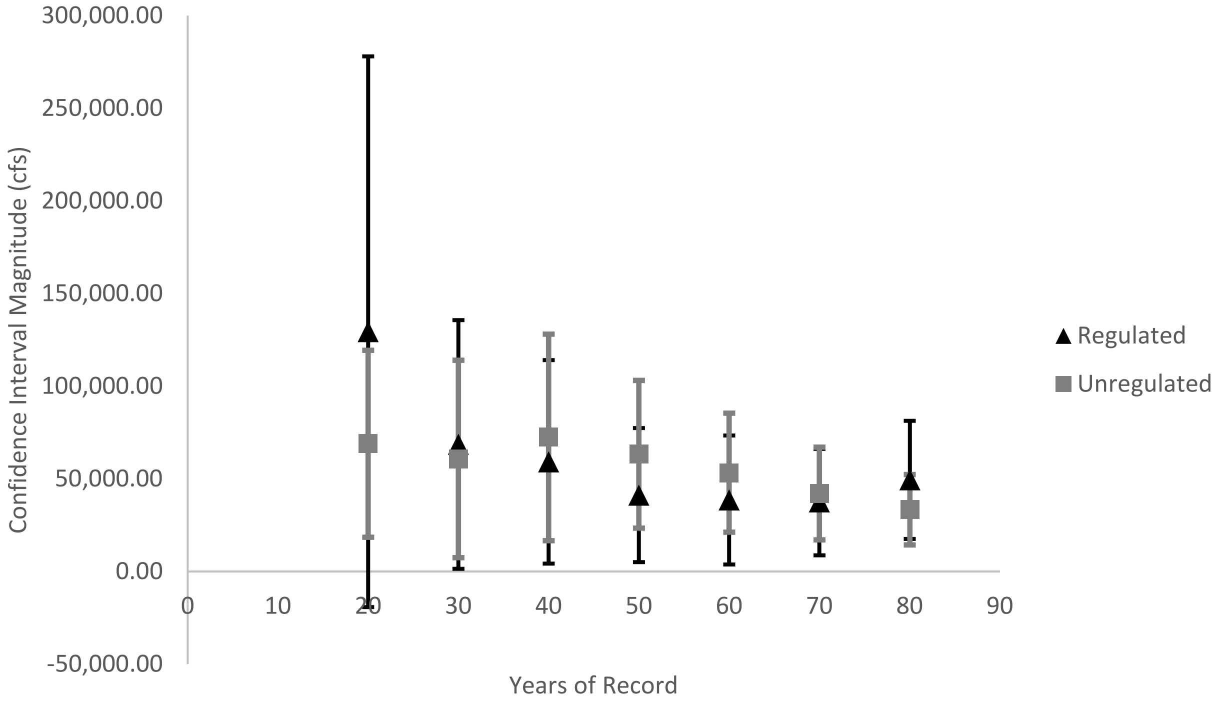

Figure 17: Mean confidence interval magnitude of 100-year flow by years of record for all regulated and unregulated gages with standard deviation error bars

From a geographical perspective, the higher 100-year flows were focused in the central and southeastern portions of the ecoregion due primarily to the streams with larger (>1000 mi^2) drainage areas (Figure 18). The lower 100-year flows were focused particularly in the northeastern area where the smaller (<1000 mi^2) drainage areas were located.

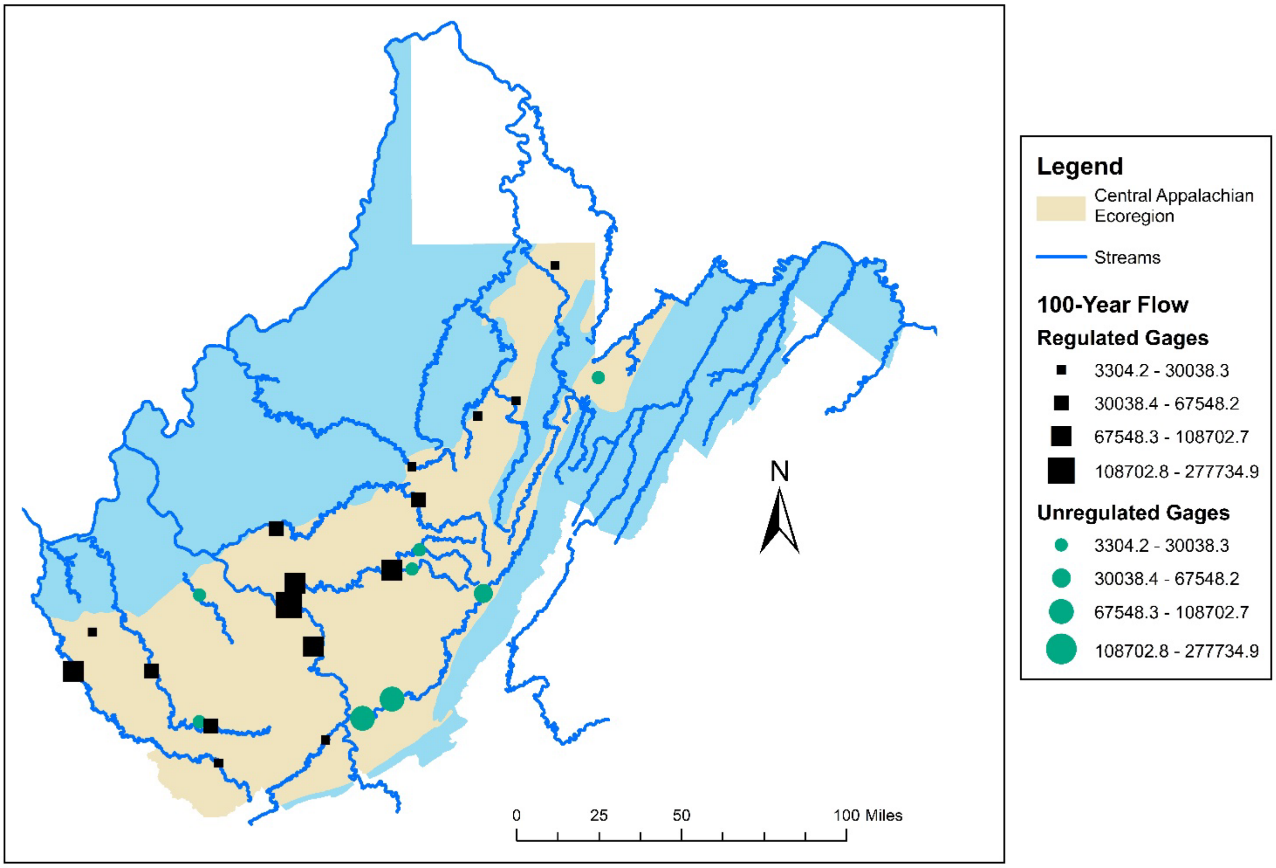

Figure 18: Map of regulated and unregulated gage stations with magnitude of 100-year flow indicated by size of symbology

When percent change in 100-year flow was mapped, the westernmost 80 miles of the ecoregion experienced the most substantial decreases in 100-year flow over the years, while the largest increases were seen in the southern and northern portions of the ecoregion (Figure 19).

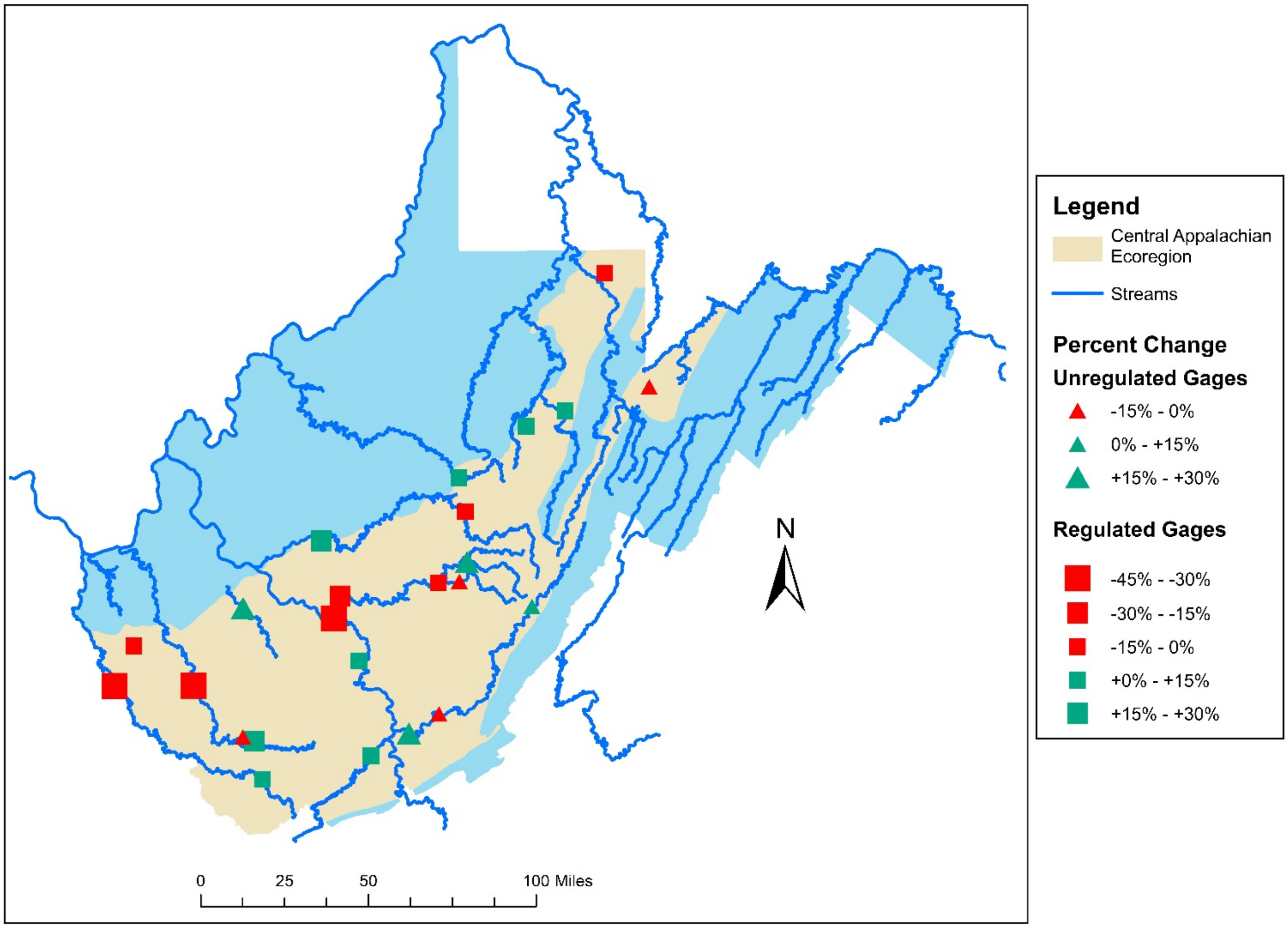

Figure 19: Map of regulated and unregulated gage stations with percent change in 100-year flow from all-year analysis to 30-year analysis indicated by color and size of symbology

4.2 Reservoir routing

The results from the reservoir routing analysis yielded peak outflows for each type of scenario: ESW crest, ESW design storm, and dam overtopping. The peak outflow was identified and applied to the corresponding flow equation to calculate the peak elevation of the scenario. For the more complicated equations (Equations 6 and 7), the Newton-Raphson method (Chapra and Canale 2015) was used with R coding to find an approximate solution for elevation given flow, and these solutions were checked by applying the newfound elevation to the equation to check that it matched the original outflow (R Core Team 2017) (Gilbert and Varadhan 2016) (Schlegel 2017). To be sure of the accuracy of the outflow, the flow volumes under the inflow curve and outflow curve were calculated and compared, as they should be equal. The inflow volume was calculated with a simple triangle equation, and the outflow volume was calculated with the trapezoidal rule.

The scenarios that will be used in the second task were drawn from the 100-year flow data results. The following are justifications for each scenario:

1. (-7%): Change from all years of data included to 30 years of data included for all regulated gages
2. (+6%): Change from all years of data included to 30 years of data included for all unregulated gages
3. (+12%): Change from all years of data included to 30 years of data included for only gages in the Greenbrier watershed
4. (+20%): Change from all years of data included to 40 years of data included for all unregulated gages
5. (+30%): Change from all years of data included to 40 years of data included for only gages in the Greenbrier watershed

For each relevant gage, these scenarios were matched with the new corresponding frequency storm using the frequency plots from HEC-SSP (USACE 2018a). Then the range of frequency storms was found for each scenario (Table 6). For example, since the -7% scenario was based on regulated gage results, the new frequency storm range was found using all the frequency plots of the regulated gages.

Table 6: Frequency storms corresponding to percent change scenarios

Scenario	-7%	+6%	+12%	+20%	+30%
Frequency Storm (yr)	35 to 82	125 to 229	127 to 256	204 to >500	177 to >500

4.2.1 ESW crest

The ESW crest was reached at around an 11.68% increase in peak inflow from the peak inflow given in the design report (Figure 20). The ESW crest is designed to 100-year flow, so the peak inflow from the design report matches the 100-year flow at the time of construction.

With the analysis of this research, the peak inflow to reach the ESW crest is calculated to be around 11.68% higher than in the design report.

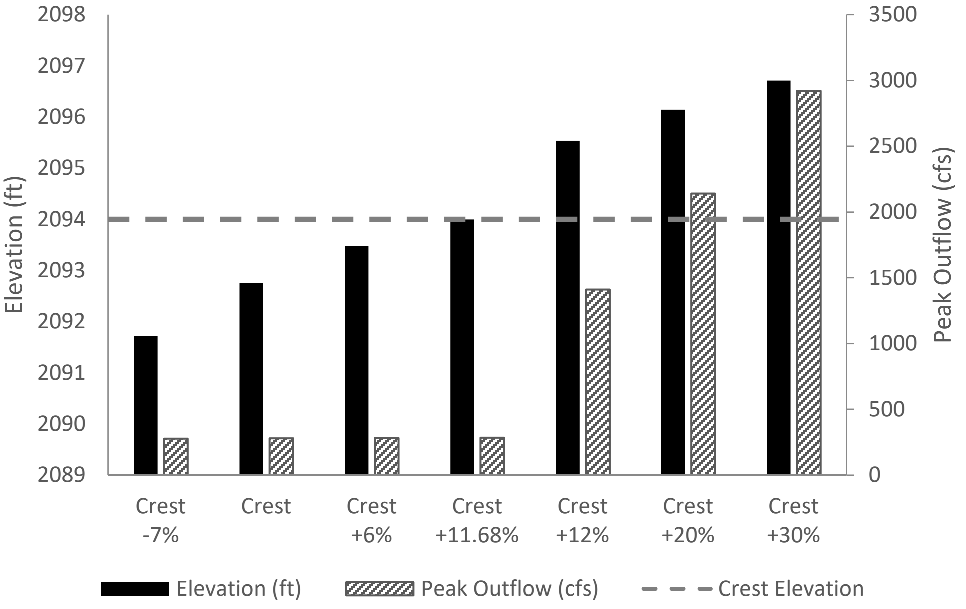

Figure 20: Peak outflow and peak elevation for crest of ESW scenarios with oval indicating where peak outflow reaches crest of ESW

4.2.2 ESW design storm

As expected, the ESW design storm scenarios are all over the crest of the ESW, but below the dam crest (Figure 21). However, the scenarios are based on increases in 100-year flow, and the ESW design storm is calculated using both 100-year flow and PMP.

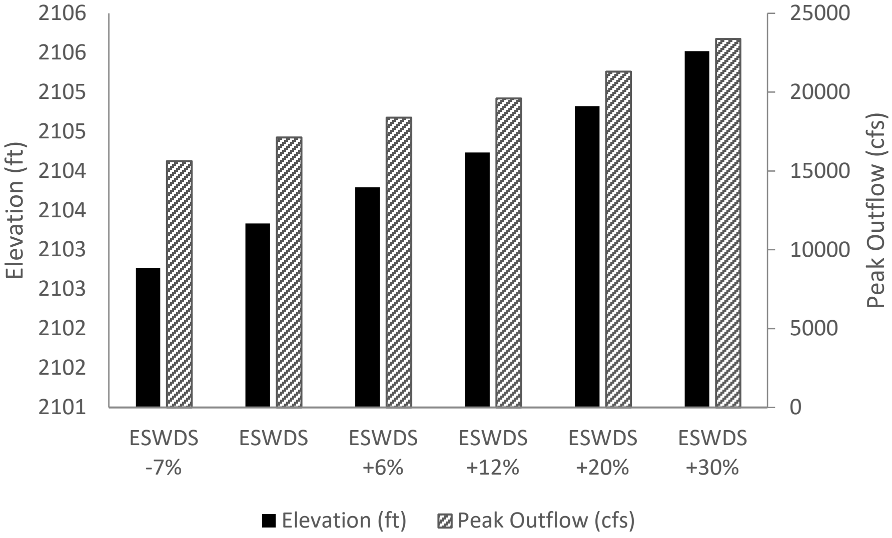

Figure 21: Peak outflow and peak elevation for ESW design storm scenarios

4.2.3 Dam overtopping

The dam overtopping was actually found to occur at around 9.579% smaller peak inflow than the design report predicted (Figure 22). Importantly, however, the scenarios are based on increases in 100-year flow, and dam overtopping design values are calculated with PMP.

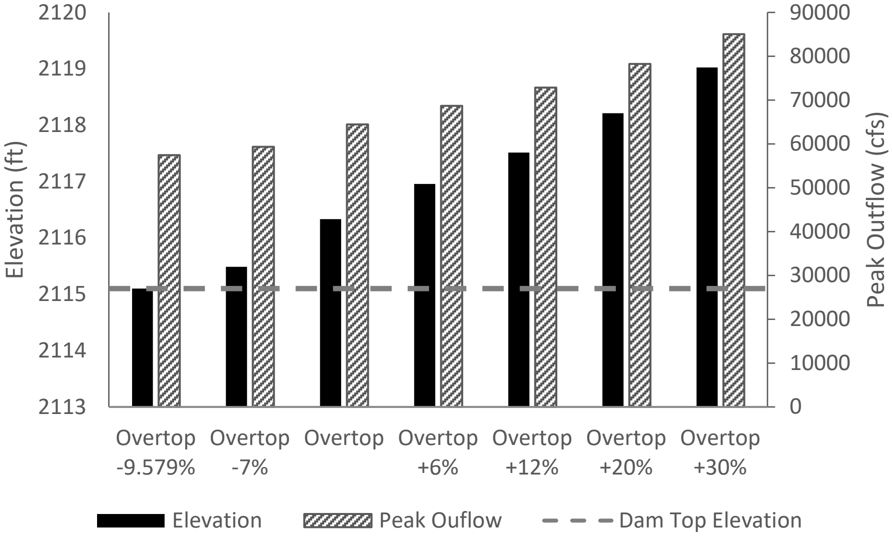

Figure 22: Peak outflow and peak elevation for dam overtopping scenarios with oval indicating where peak outflow reaches dam crest

4.2.4 Routing Results

The comprehensive table of the scenario results gives the following for each scenario: peak inflow, peak outflow, equation used for outflow, maximum head, maximum elevation, inflow area, outflow area, and notes where water reached a notable elevation (Table 6).

Table 7: Outcomes of reservoir routing analysis

Scenario	**Peak Inflow (cfs)**	**Peak Outflow (cfs)**	**Equation**	**Max Head (ft)**	**Max Elevation (ft)**	**Inflow Volume (cfs-hr)**	**Outflow Volume (cfs-hr)**
	AT 3.8 HRS						
Crest -7%	5225	279	5	28.72	2091.72	26506.4	26508.3
Crest	5618	281	5	29.76	2092.76	28500.1	28503.6
Crest +6%	5955	283	5	30.48	2093.48	30209.7	30213.8
Crest +11.68%	6274	284	5	31.00	**2094.00**	31828.5	31832.3
Crest +12%	6292	1411	6	32.54	2095.54	31920.1	31924.0
Crest +20%	6742	2141	6	33.14	2096.14	34202.2	34204.3
Crest +30%	7303	2921	6	33.71	2096.71	37048.1	37054.7
	AT 3.8 HRS						
ESW Design Storm	20387	17118	6	40.33	2103.33	103423.3	103435.8
ESWDS -7%	18960	15616	6	39.77	2102.77	96184.1	96195.3
ESWDS +6%	21610	18373	6	40.79	2103.79	109627.5	109642.0
ESWDS +12%	22833	19604	6	41.23	2104.23	115831.8	115848.1
ESWDS +20%	24464	21298	6	41.82	2104.82	124105.9	124123.0
ESWDS +30%	26503	23361	6	42.52	2105.52	134449.7	134466.6
	AT 3.7 HRS						
Dam Overtop -9.579%	61417	57470	6	52.10	**2115.10**	303368.8	303377.0
Dam Overtop -7%	63168	59371	7	52.48	2115.48	312020.3	312028.7
Dam Overtop	67923	64482	7	53.33	2116.33	335505.7	335514.8
Dam Overtop +6%	71998	68709	7	53.96	2116.96	355636.0	355645.6
Dam Overtop +12%	76074	72818	7	54.52	2117.52	375766.3	375776.5
Dam Overtop +20%	81508	78233	7	55.21	2118.21	402606.8	402617.7
Dam Overtop +30%	88300	85033	7	56.03	2119.03	436157.4	436169.2

5 Discussion

It is important to note that the analyses performed at all stages of this research used only past data, and no predictive models for the future. The scenarios for change in peak inflows to the dam were based on 100-year flow results, so they might not apply directly to ESW design storm and dam crest. The ESW design storm is partially based on PMP and dam overtopping is based solely on PMP. Any of the 432 high hazard dams in West Virginia could be affected by the changes in 100-year flow over time, but it will likely depend on the specifications of a particular dam and the watershed characteristics (USACE 2018b). Some dams have changed to high hazard from a lower hazard category because of developments downstream of the dam, and these developments could have also affected the peakflow levels from the gages in those streams

The reason both unregulated and regulated streams were analyzed was that urban development and dams can have a strong impact on the occurrence of floods, and flow data after a large-scale human change may be completely different than flow data before a change (USGS 2018). Regulated streams have dams to control the flowrates through the streams and lower peakflows for flood control (Bedient et al. 2013). The results of this research support the expected effects of stream regulation because regulated streams have experienced a decreased 100-year flow over time. Conversely, unregulated streams have experienced an increase in 100-year flow over time, likely due to climate change effects.

Only changes in 100-year storm magnitude were considered, but there is evidence of increases in frequency of 100-year storms as well. The increase in frequency of these storms in the region was not considered. In the 2000s, the frequency of extreme precipitation events was higher than any previous decade on record (Frei et al. 2015). Also, the southern Appalachians and Ohio River Valley are expected to experience 2-5 times more 100-year floods by 2100 (Wobus et al. 2017).

The dam overtopping scenarios, calculated from the weir equation over the crest of the dam, predict that the dam crest elevation would be reached by about a -9.6% change from the given flow from the design report. This would indicate that the dam crest elevation would be reached with a much lower flow than that predicted by the design report. However, the methods used in the design report were from PMP, and the methods used in the scenarios in this research were based on a weir equation from average dam length. Possible problems with the research methods include errors in measurement of dam length or an equation that oversimplifies the physics of dam overtopping. This research also did not consider the heights of flood waves or freeboard, only the rise in elevation of the reservoir. Recently, an assessment report of the dam described the crest of the dam as a “non-level crest that varies from elevation 2115.9 to 2119.7 ft” rather than the design elevation of 2115.1 ft due to “less settlement than predicted” (NRCS 2016a).

Differences in regulation strategies between dams of different organizations such as USGS, NRCS, or private ownership could affect how flow through those dams is regulated, which would affect the results from the regulated gages in the analysis of this research. Also, the order in which dam reservoirs are drained would also influence the peakflows through the gages in those streams. These variations between dams are influenced by both the management strategies of different dams and the state of repair of those dams. The flows through dams that are determined to be fragile may be controlled to reduce the stresses on those dams, instead placing stress on more resilient dams.

6 Summary and Conclusions

The goal of this research was to determine if dam design criteria for high hazard dam emergency spillways remain adequate despite changing climate with increased extreme precipitation events. To measure climate change effects, the 100-year flow was calculated for 24 USGS gages in the CAE in WV using HEC-SSP, and several scenarios were generated with this 100-year flow data to be used to analyze flow through the emergency spillway of Howard Creek Dam. The central finding of the research were the following:

- The 100-year flow for regulated gages has decreased over time, indicating that installation of dams and subsequent regulation of flow through the dams has lowered the peakflows of those gages.
- The 100-year flow for unregulated gages has increased over time, indicating that without dams on the stream, climate change has caused higher peakflows due to higher frequencies and magnitudes of extreme precipitation events.
- The emergency spillway of Howard Creek Dam was predicted to be reached by an 11.68% increase in 100-year flow from the value given in the design report of the dam. This 11.68% increase is exceeded by the three of the generated scenarios: the 12% increase, the 20% increase, and the 30% increase. The 12% and 30% scenarios were generated from Greenbrier watershed gage data, and the 20% scenarios was generated with CAE unregulated gage data. As a result, an 11.68% increase in 100-year flow does seem feasible within the next several decades, so emergency spillway designs should factor in potential increases in 100-year flow.

Future work could include a predictive model including how 100-year flow is expected to change in the coming years rather than just analysis of past data. Also, to fully describe the behavior for the emergency spillway design storm and dam overtopping scenarios, calculations involving PMP should be included. A more specific analysis could be performed on which dams in the region would be affected by the changes in 100-year flow. To achieve a more sophisticated model, research could account for increase in storm frequency, flow equations accounting for specific dam profiles, dam operation differences, dams from other organizations, and routing flood waves toward dams.

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Appendix A – Relative frequency histograms

Regulated

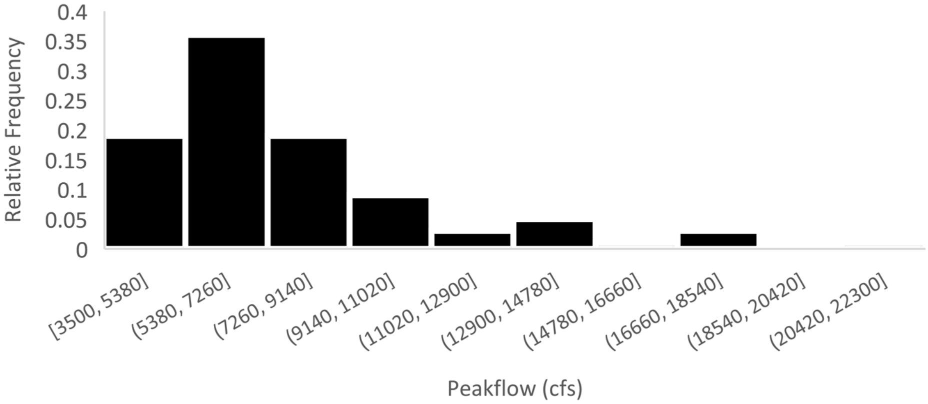

Figure 23: Relative frequency histogram for Big Sandy Creek at Rockville gage

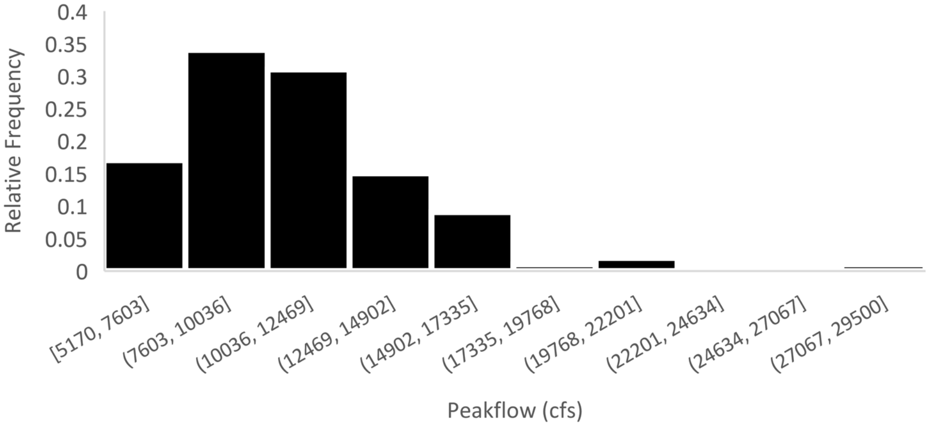

Figure 24: Relative frequency histogram for Tygart Valley River at Belington gage

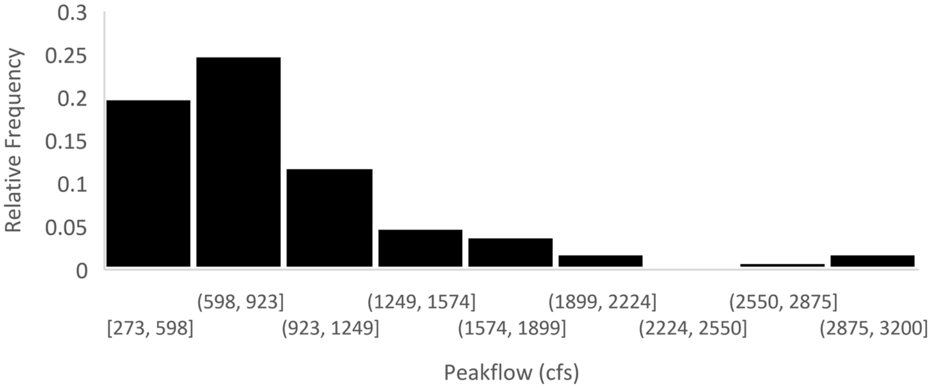

Figure 25: Relative frequency histogram for Sand Run near Buckhannon gage

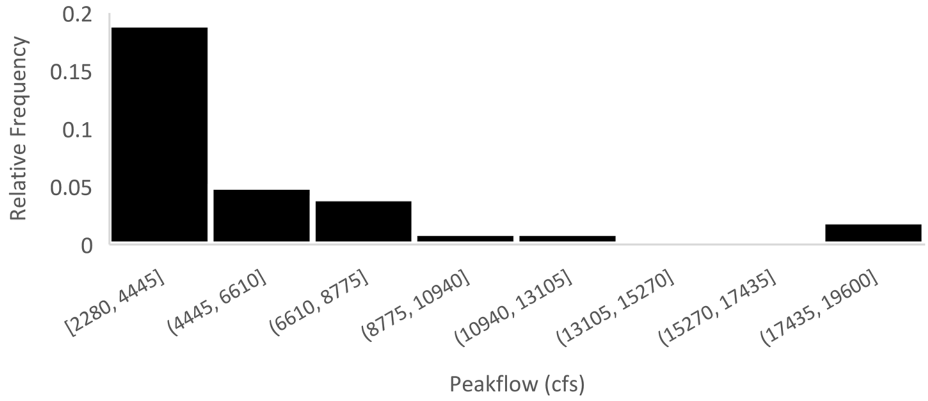

Figure 26: Relative frequency histogram for Little Kanawha River near Wildcat gage

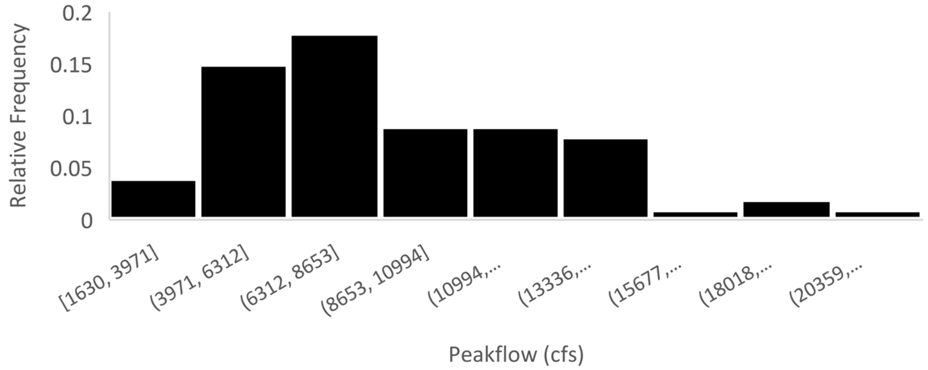

Figure 27: Relative frequency histogram for Bluestone River near Pipestem gage

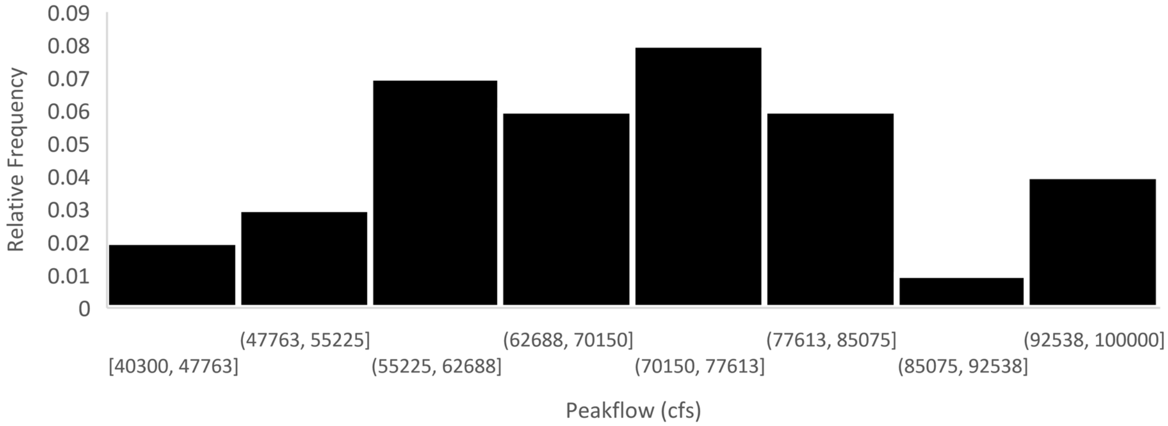

Figure 28: Relative frequency histogram for New River at Thurmond gage

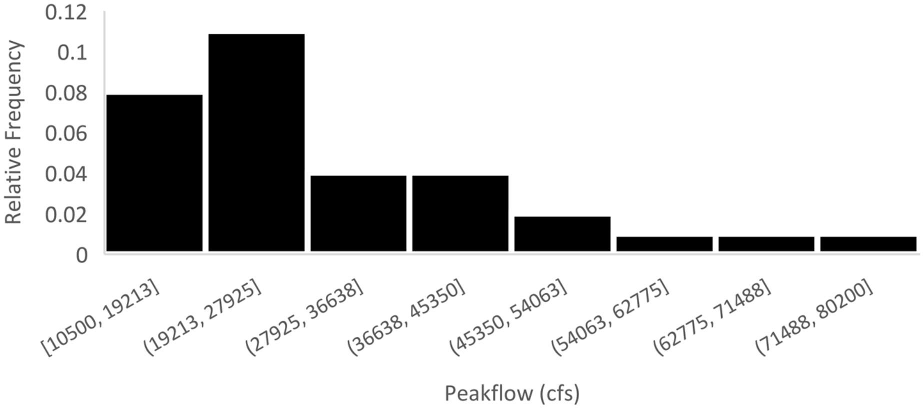

Figure 29: Relative frequency histogram for Gauley River near Craigsville gage

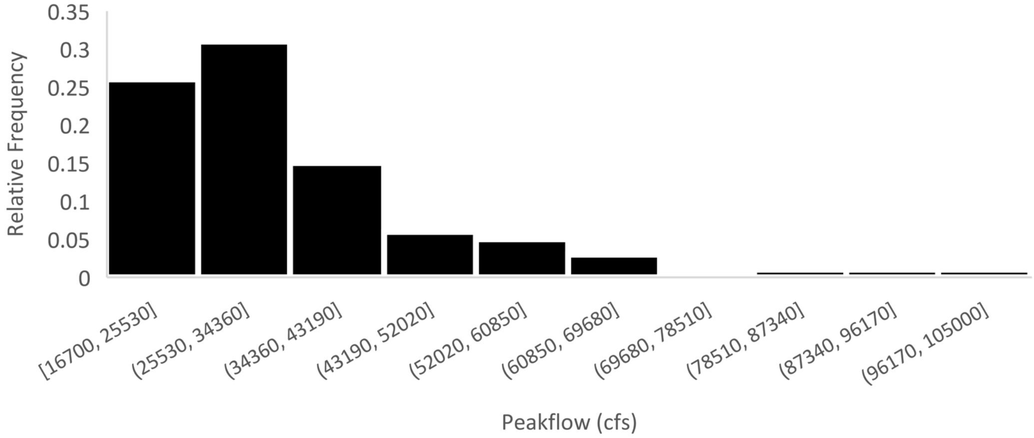

Figure 30: Relative frequency histogram for Gauley River above Belva gage

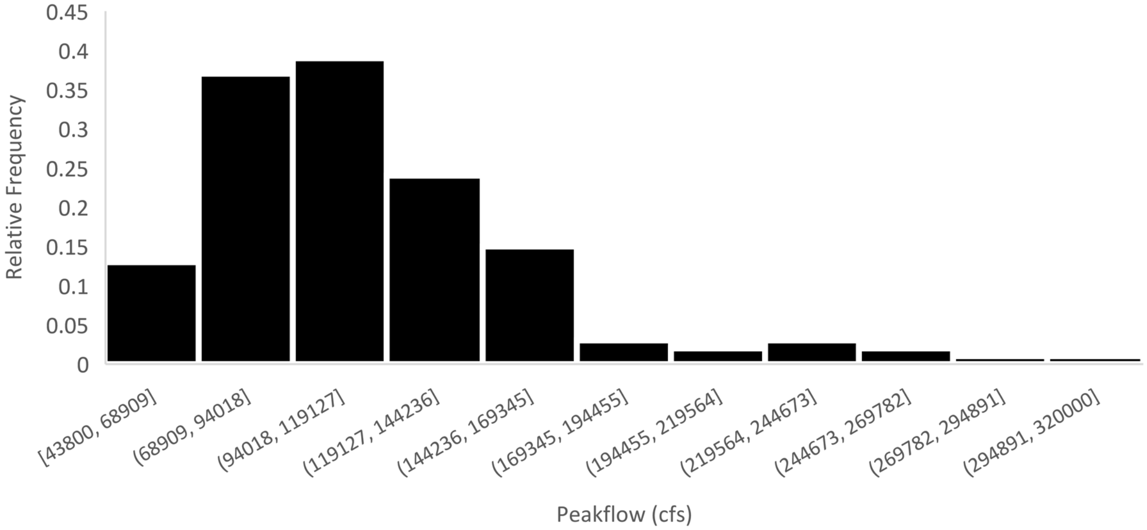

Figure 31: Relative frequency histogram for Kanawha River at Kanawha Falls gage

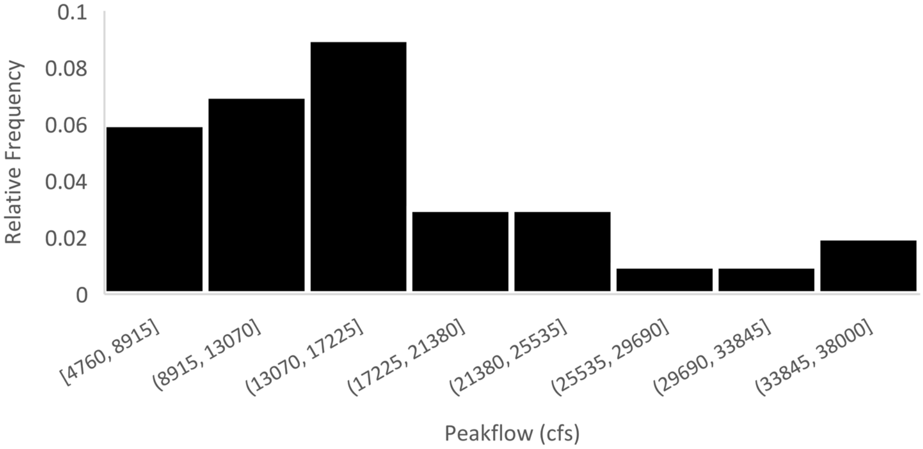

Figure 32: Relative frequency histogram for Elk River below Webster Springs gage

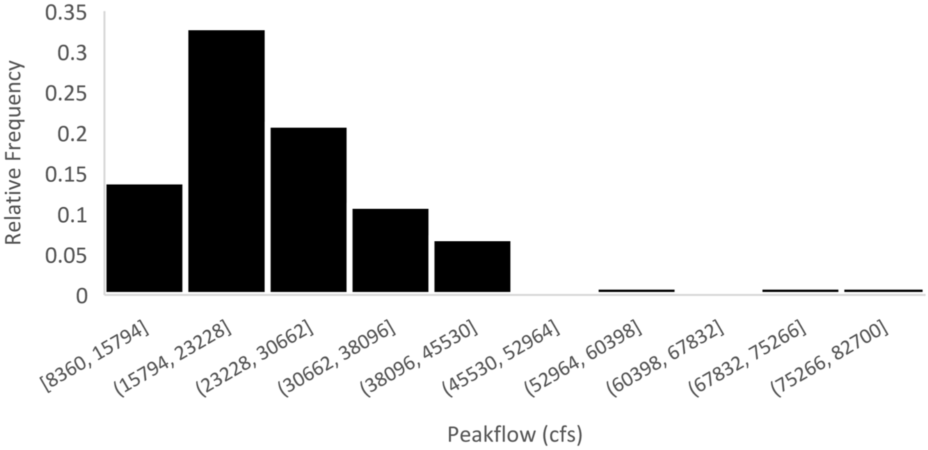

Figure 33: Relative frequency histogram for Elk River at Queen Shoals gage

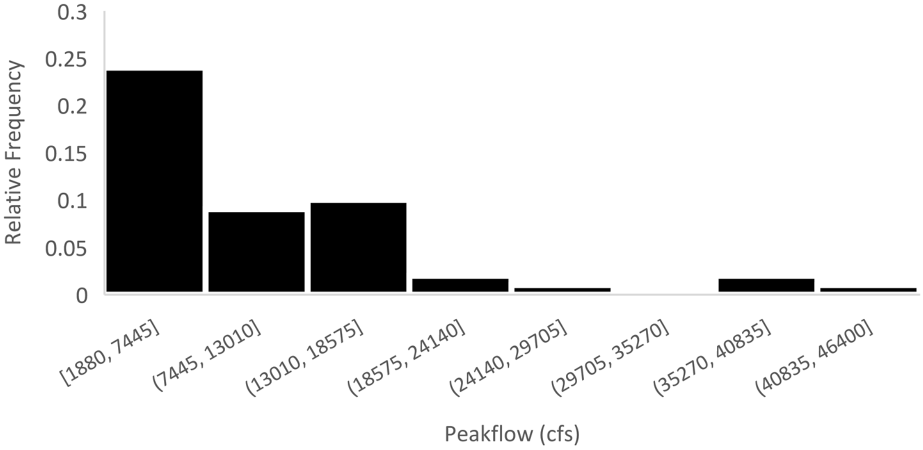

Figure 34: Relative frequency histogram for Guyandotte River near Baileysville gage

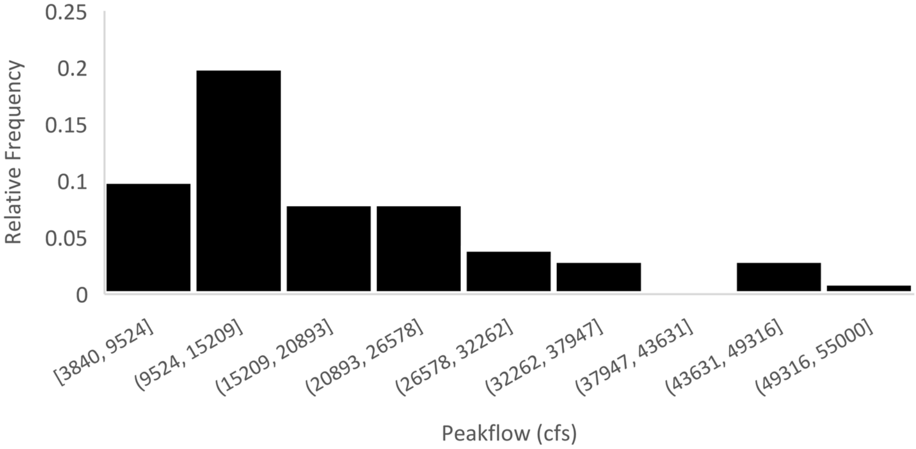

Figure 35: Relative frequency histogram for Guyandotte River at Logan gage

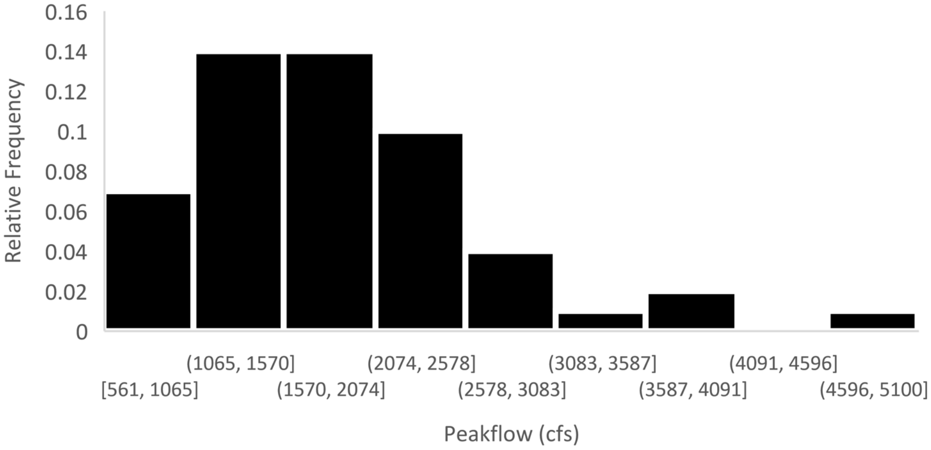

Figure 36: Relative frequency histogram for East Fork Twelvepole Creek nr Dunlow gage

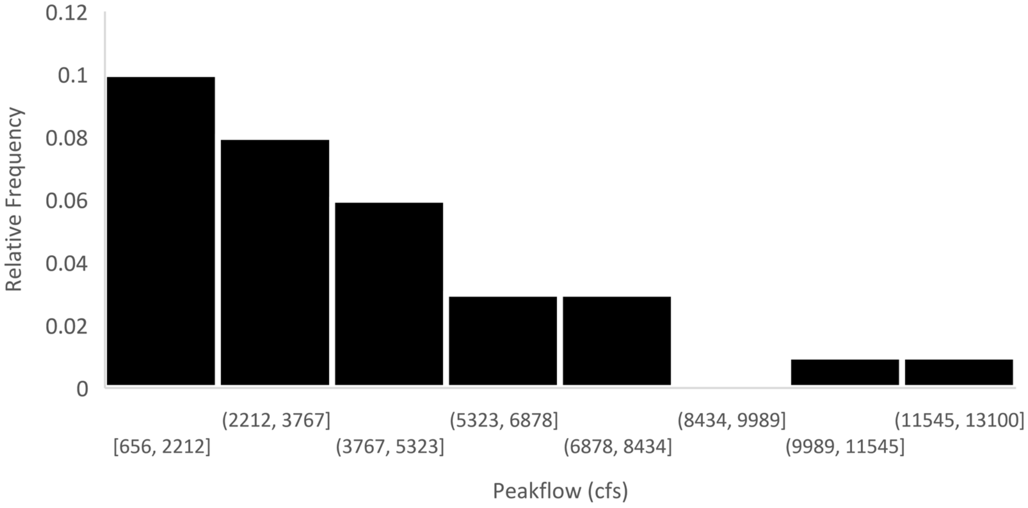

Figure 37: Relative frequency histogram for Tug Fork downstream of Elkhorn Creek at Welch gage

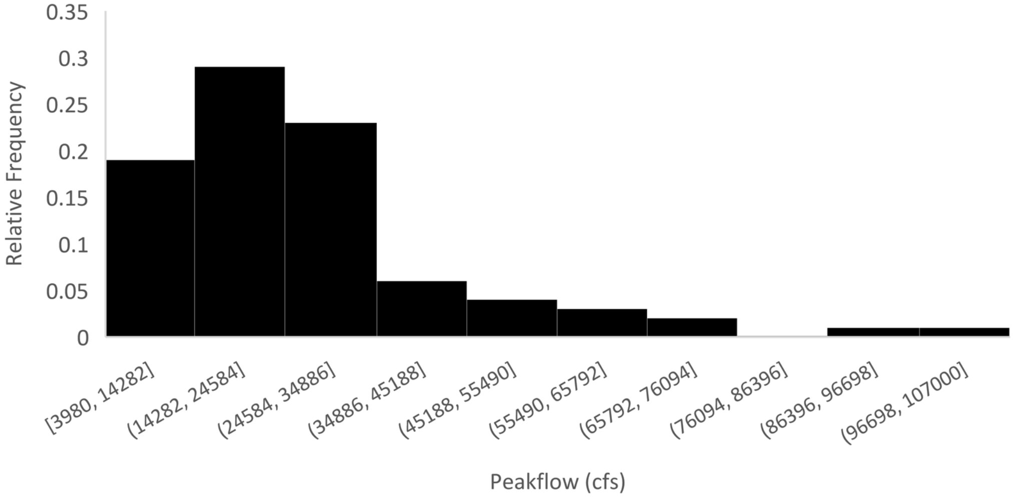

Figure 38: Relative frequency histogram for Tug Fork at Kermit gage

Unregulated

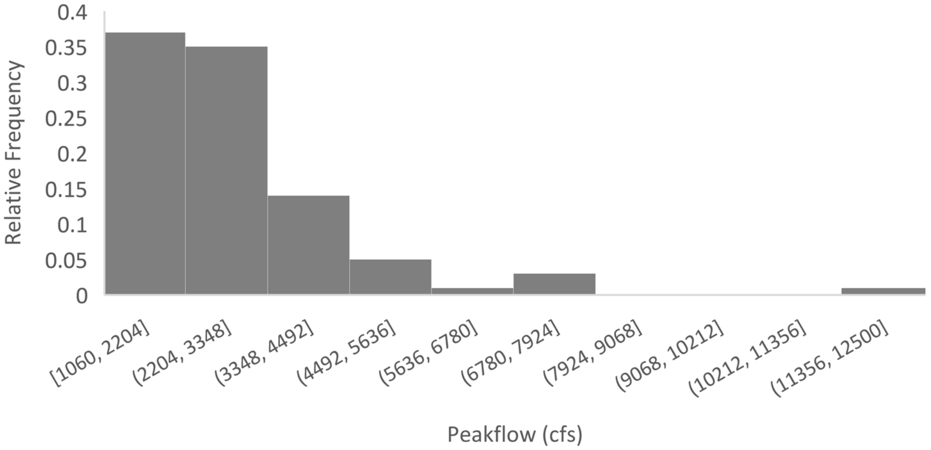

Figure 39: Relative frequency histogram for Blackwater River at Davis gage

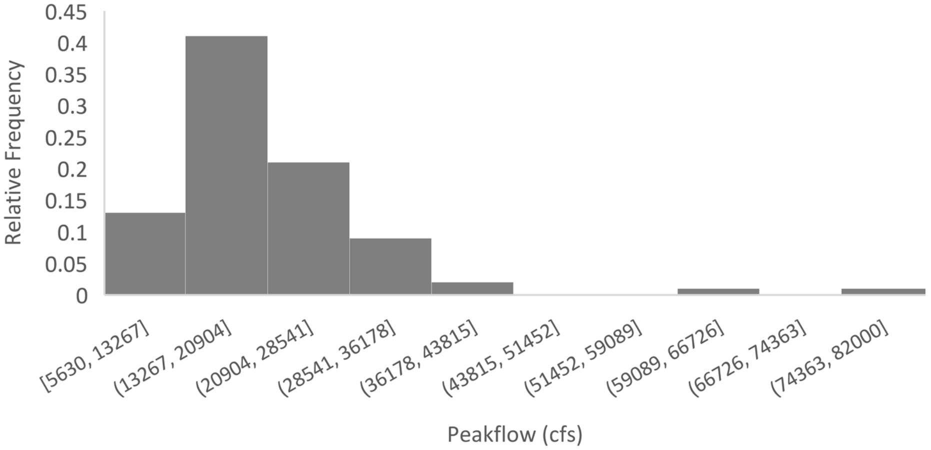

Figure 40: Relative frequency histogram for Greenbrier River at Buckeye gage

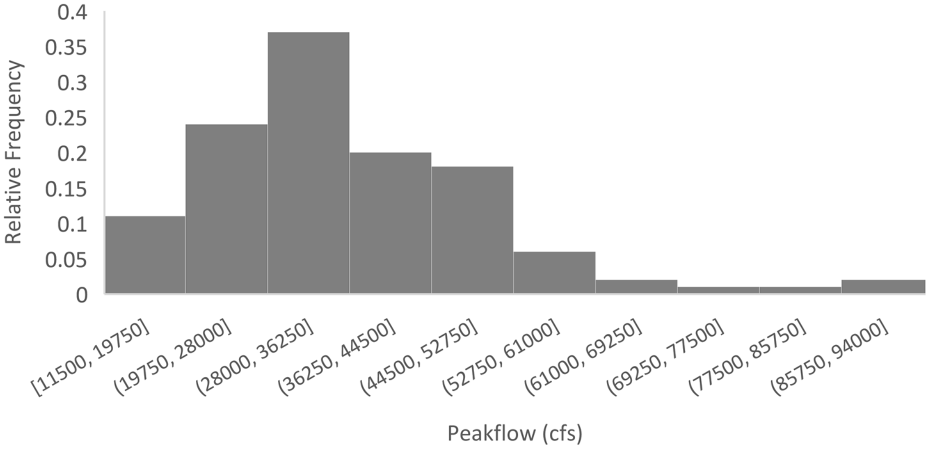

Figure 41: Relative frequency histogram for Greenbrier River at Alderson gage

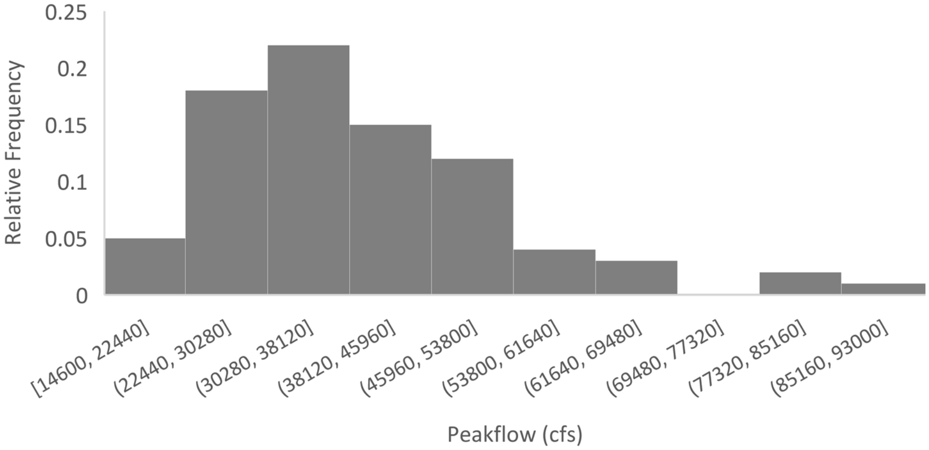

Figure 42: Relative frequency histogram for Greenbrier River at Hilldale gage

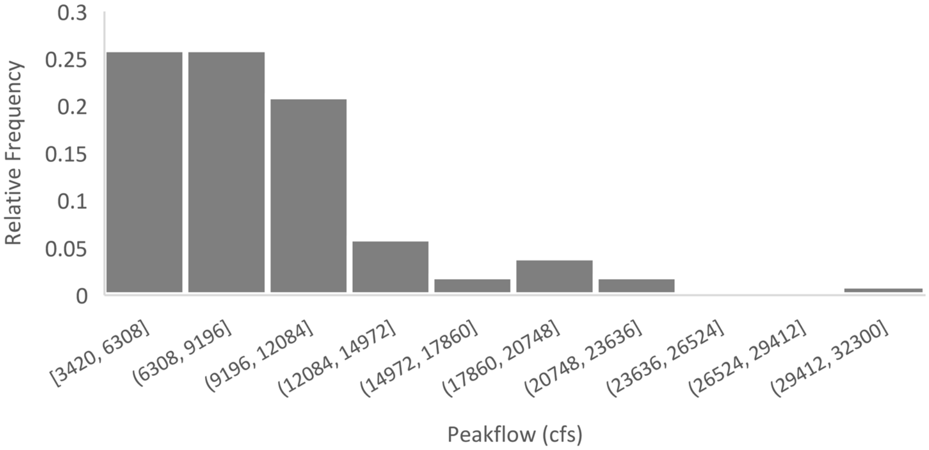

Figure 43: Relative frequency histogram for Williams River at Dyer gage

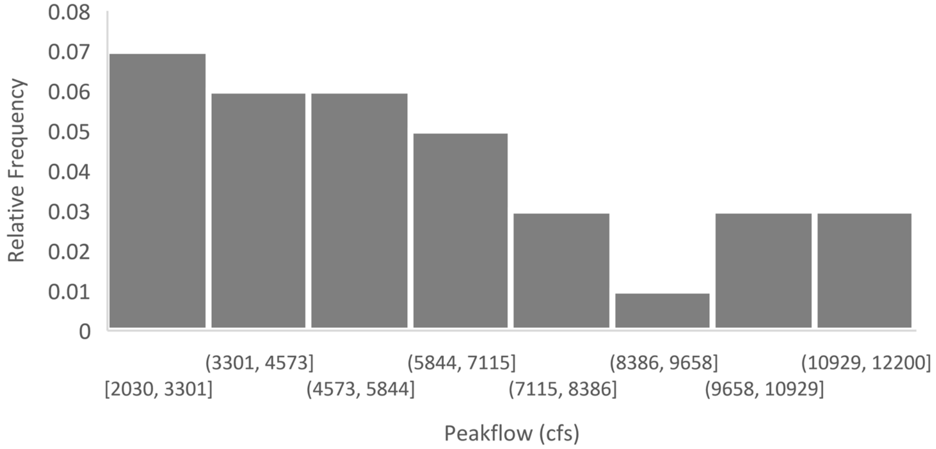

Figure 44: Relative frequency histogram for Cranberry River near Richwood gage

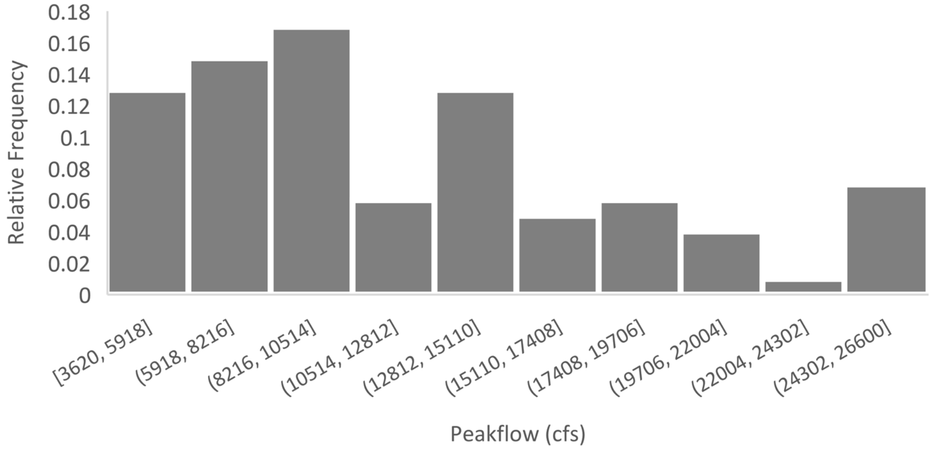

Figure 45: Relative frequency histogram for Big Coal River at Ashford gage

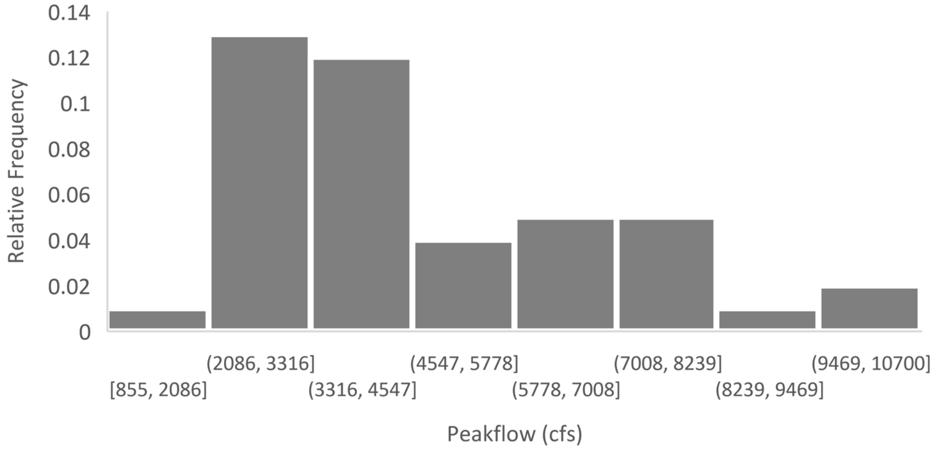

Figure 46: Relative frequency histogram for Clear Fork at Clear Fork gage

Appendix B: 100-year flow data from HEC-SSP results

Table 8: All results for all gages from HEC-SSP frequency analyses including gage name, USGS gage number, and total years of record along with 100-year flow, variance, and confidence limits for each period of record

BIG SANDY CREEK AT ROCKVILLE, WV				
3070500				
96 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1922-2017	20,976.30	0.00487	32,469.10	17,252.60
1928-2017	21,373.20	0.00522	33,830.10	17,473.80
1938-2017	21,815.60	0.00573	35,607.60	17,687.50
1948-2017	22,385.30	0.00659	38,445.10	17,908.40
1958-2017	19,413.50	0.00578	32,440.10	15,761.60
1968-2017	19,239.80	0.00705	35,477.60	15,360.70
1978-2017	17,947.90	0.00744	34,419.70	14,268.40
1988-2017	19,267.20	0.01108	44,394.50	14,664.60
1998-2017	17,812.60	0.01108	41,367.00	13,601.90

TYGART VALLEY RIVER AT BELINGTON, WV				
3051000				
110 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1908-2017	22,431.70	0.00152	27,490.70	19,910.40
1918-2017	22,717.10	0.0018	28,445.10	19,995.40
1928-2017	22,963.60	0.00203	29,455.80	20,087.70
1938-2017	23,476.40	0.00243	31,107.80	20,319.30
1948-2017	24,201.10	0.00314	34,130.30	20,645.80
1958-2017	25,749.20	0.00501	42,543.90	21,277.70
1968-2017	26,588.70	0.00615	47,330.50	21,569.50
1978-2017	28,205.30	0.00931	56,640.90	21,779.50
1988-2017	23,440.10	0.0057	41,509.60	19,143.70
1998-2017	18,578.60	0.00344	27,560.90	15,860.90

SAND RUN NEAR BUCKHANNON, WV				
3052500				
71 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1947-2017	3,304.20	0.00805	5,663.90	2,554.70
1948-2017	3,330.20	0.00853	5,767.60	2,555.40
1958-2017	3,498.70	0.01173	7,207.30	2,596.40
1968-2017	3,658.30	0.01222	7,701.70	2,697.50
1978-2017	4,110.60	0.01576	9,755.50	2,915.90
1988-2017	3,765.40	0.01393	8,234.70	2,714.10
1998-2017	3,317.70	0.02154	10,707.10	2,257.20

BLACKWATER RIVER AT DAVIS, WV				
3066000				
96 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1922-2017	9,081.70	0.00631	14,781.50	7,255.50
1928-2017	8,868.30	0.0064	14,640.70	7,087.80
1938-2017	9,258.00	0.00747	16,244.00	7,289.00
1948-2017	9,598.60	0.00912	18,129.70	7,390.50
1958-2017	9,575.30	0.01095	19,484.10	7,202.00
1968-2017	10,267.00	0.01709	23,639.60	7,159.50
1978-2017	11,566.60	0.02438	32,178.60	7,553.70
1988-2017	8,545.90	0.01543	22,732.40	6,178.10
1998-2017	5,484.10	0.00597	9,070.80	4,422.50

LITTLE KANAWHA RIVER NR WILDCAT, WV				
3151400				
32 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1986-2017	25,483.30	0.03455	95,925.30	15,542.10
1988-2017	25,561.00	0.03874	102,276.40	15,131.40
1998-2017	20,213.70	0.0428	81,073.90	11,604.60

BLUESTONE RIVER NEAR PIPESTEM, WV				
3179000				
67 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1951-2017	23,055.30	0.00327	31,170.40	19,387.70
1958-2017	22,379.40	0.00319	30,226.60	18,875.30
1968-2017	22,738.70	0.0036	31,494.20	18,962.90
1978-2017	23,676.60	0.00506	35,678.50	19,222.10
1988-2017	26,458.90	0.00878	50,835.80	20,417.10
1998-2017	28,780.00	0.01336	56,737.10	20,601.30

GREENBRIER RIVER AT BUCKEYE, WV				
3182500				
88 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1930-2017	57,956.90	0.00438	84,764.00	47,819.80
1938-2017	60,129.60	0.00624	99,975.50	48,289.80
1948-2017	63,397.00	0.0079	115,400.60	49,744.00
1958-2017	67,298.90	0.01011	133,891.60	51,223.60
1968-2017	70,103.00	0.01144	152,970.50	52,682.70
1978-2017	74,611.90	0.01466	187,433.10	54,183.60
1988-2017	54,462.30	0.00691	95,673.50	43,307.60
1998-2017	41,361.00	0.00443	64,733.50	34,320.60

GREENBRIER RIVER AT ALDERSON, WV				
3183500				
122 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1896-2017	79,865.50	0.0013	94,778.60	71,187.50
1898-2017	79,856.90	0.00133	94,935.10	71,077.30
1908-2017	80,123.80	0.00155	96,773.80	70,746.70
1918-2017	80,344.10	0.00198	99,108.60	70,402.50
1928-2017	81,613.30	0.00242	107,500.90	70,653.20
1938-2017	87,183.50	0.00408	130,674.10	72,949.50
1948-2017	91,290.00	0.00505	147,654.70	75,172.60
1958-2017	96,209.20	0.00653	169,110.10	77,343.70
1968-2017	99,789.60	0.00801	190,518.80	78,537.10
1978-2017	103,144.90	0.0115	224,193.50	77,391.60
1988-2017	97,900.00	0.01222	233,763.80	73,272.10
1998-2017	85,774.30	0.0112	200,047.60	65,335.50

GREENBRIER RIVER AT HILLDALE, WV				
3184000				
82 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1936-2017	86,556.00	0.00219	111,483.40	75,260.10
1938-2017	86,543.40	0.00232	112,725.60	74,961.70
1948-2017	91,893.70	0.00337	130,662.60	77,892.20
1958-2017	95,957.70	0.00445	148,572.40	79,759.30
1968-2017	98,456.50	0.0054	162,165.80	80,503.00
1978-2017	99,338.30	0.00725	188,587.80	79,193.40
1988-2017	97,054.20	0.00825	199,566.40	76,574.50
1998-2017	88,265.10	0.00841	182,826.30	69,357.60

NEW RIVER AT THURMOND, WV				
3185400				
37 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1981-2017	108,702.70	0.00108	128,806.10	98,299.90
1988-2017	110,318.40	0.00133	133,927.40	98,750.60
1998-2017	115,253.50	0.00238	153,297.50	100,061.60

WILLIAMS RIVER AT DYER, WV				
3186500				
88 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1930-2017	28,013.10	0.00573	44,130.20	22,570.40
1938-2017	27,489.70	0.00578	43,507.20	22,136.70
1948-2017	28,990.00	0.0071	49,451.50	22,898.20
1958-2017	30,898.60	0.00885	57,294.00	23,811.70
1968-2017	32,429.80	0.00988	63,475.10	24,665.50
1978-2017	34,746.20	0.01149	72,716.90	25,916.60
1988-2017	32,978.50	0.01352	79,161.50	24,180.50
1998-2017	36,352.70	0.02671	131,478.70	23,919.60

CRANBERRY RIVER NEAR RICHWOOD, WV				
3187500				
34 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1984-2017	16,787.90	0.00833	28,854.70	13,026.50
1988-2017	15,628.40	0.00868	26,758.30	12,155.30
1998-2017	15,420.90	0.01073	32,921.10	11,658.80

GAULEY RIVER NEAR CRAIGSVILLE, WV				
3189100				
32 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1986-2017	94,684.90	0.01235	205,434.20	69,847.40
1988-2017	88,902.50	0.0118	190,712.20	66,086.40
1998-2017	98,159.50	0.02069	310,203.50	67,598.60

GAULEY RIVER ABOVE BELVA, WV				
3192000				
89 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1929-2017	98,932.40	0.00595	162,909.30	79,898.80
1938-2017	90,829.40	0.00585	150,660.70	73,595.30
1948-2017	77,907.60	0.00536	128,697.30	63,837.60
1958-2017	65,089.70	0.00536	106,448.10	53,283.40
1968-2017	68,173.20	0.00969	122,044.20	51,559.50
1978-2017	73,401.10	0.01188	137,371.80	53,706.10
1988-2017	80,192.60	0.01636	174,119.60	56,110.60
1998-2017	96,643.50	0.02776	289,682.00	61,752.00

KANAWHA RIVER AT KANAWHA FALLS, WV				
3193000				
140 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1878-2017	277,734.90	0.00215	355,599.00	241,691.40
1888-2017	259,760.40	0.00186	325,768.20	228,004.00
1898-2017	258,678.60	0.00221	334,074.60	224,930.30
1908-2017	224,703.80	0.00185	284,990.80	197,921.30
1918-2017	203,201.10	0.00153	251,950.20	180,939.00
1928-2017	198,602.70	0.00162	248,536.70	176,426.00
1938-2017	193,150.70	0.00196	252,641.80	170,273.40
1948-2017	158,779.60	0.00089	185,566.60	145,379.40
1958-2017	153,540.40	0.00098	179,262.40	140,603.00
1968-2017	154,137.70	0.0011	185,248.50	140,124.50
1978-2017	154,777.90	0.00134	192,991.40	139,408.30
1988-2017	159,244.60	0.00185	209,443.40	141,150.70
1998-2017	171,467.90	0.00396	282,892.30	145,171.50

ELK RIVER BELOW WEBSTER SPRINGS, WV				
3194700				
32 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1986-2017	44,838.70	0.00817	75,596.10	34,498.30
1988-2017	38,866.90	0.0065	59,305.20	30,394.20
1998-2017	33,658.50	0.00637	52,727.70	26,554.20

ELK RIVER AT QUEEN SHOALS, WV				
3197000				
89 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1929-2017	67,548.20	0.00345	92,893.10	56,739.00
1938-2017	63,766.90	0.00364	88,826.90	53,261.40
1948-2017	61,277.10	0.00391	86,990.00	50,961.80
1958-2017	63,093.40	0.00592	101,335.80	50,689.00
1968-2017	65,652.00	0.00772	115,457.40	51,305.80
1978-2017	72,837.20	0.01315	170,058.90	53,597.20
1988-2017	78,580.90	0.01805	228,457.70	55,387.80
1998-2017	92,748.70	0.04046	431,372.60	55,002.30

BIG COAL RIVER AT ASHFORD, WV				
3198500				
87 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1931-2017	30,038.30	0.00367	51,245.90	25,965.40
1938-2017	36,751.40	0.00478	52,428.50	29,876.20
1948-2017	36,618.10	0.00534	53,910.10	29,464.50
1958-2017	38,748.20	0.00673	61,403.40	30,443.70
1968-2017	36,614.50	0.00693	59,138.60	28,736.10
1978-2017	34,496.30	0.00795	58,912.50	26,716.90
1988-2017	34,606.40	0.01101	67,780.90	25,790.00
1998-2017	39,363.00	0.02223	128,802.80	26,536.80

GUYANDOTTE RIVER NEAR BAILEYSVILLE, WV				
3202400				
49 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1969-2017	54,058.90	0.01721	118,256.60	37,088.20
1978-2017	54,324.40	0.02455	156,497.20	35,326.30
1988-2017	64,010.80	0.03797	284,336.10	38,110.10
1998-2017	80,977.70	0.05842	557,203.10	42,941.90

CLEAR FORK AT CLEAR FORK, WV				
3202750				
43 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1975-2017	12,938.00	0.00808	24,846.10	10,146.50
1978-2017	12,251.90	0.0076	23,086.40	9,676.30
1988-2017	12,912.90	0.01257	31,612.80	9,667.90
1998-2017	14,761.60	0.02112	47,234.30	10,161.80

GUYANDOTTE RIVER AT LOGAN, WV				
3203600				
57 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1961-2017	62,491.20	0.00765	101,912.50	48,295.80
1968-2017	55,356.60	0.00805	91,122.40	42,758.60
1978-2017	39,589.90	0.00581	59,723.20	31,531.60
1988-2017	34,925.00	0.00807	61,945.40	27,688.10
1998-2017	37,251.70	0.01257	86,763.40	27,580.00

EAST FORK TWELVEPOLE CREEK NEAR DUNLOW, WV				
3206600				
53 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1965-2017	4,459.50	0.0034	5,908.40	3,708.70
1968-2017	4,428.90	0.00343	5,867.50	3,677.90
1978-2017	4,624.10	0.00468	6,487.00	3,734.80
1988-2017	4,090.80	0.00334	5,745.20	3,446.90
1998-2017	4,520.60	0.00916	7,747.60	3,402.10

TUG FORK DOWNSTREAM OF ELKHORN CREEK AT WELCH, WV				
3212750				
32 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1986-2017	16,510.30	0.0187	36,041.80	11,393.20
1988-2017	16,649.70	0.01958	37,566.60	11,402.10
1998-2017	21,112.90	0.03105	75,375.70	13,098.30

TUG FORK AT KERMIT, WV				
3214500				
88 Years				
Frequency Curves (1%)			Confidence Limits	
Years	Flow (CFS)	Variance	0.05	0.95
1930-2017	94,328.50	0.00563	138,303.40	75,143.40
1938-2017	98,743.10	0.00653	150,347.50	77,401.10
1948-2017	103,316.40	0.00765	163,615.20	79,716.80
1958-2017	108,736.80	0.01026	193,213.40	80,831.80
1968-2017	101,676.20	0.01178	192,952.50	74,350.30
1978-2017	85,522.20	0.01657	227,575.10	60,780.60
1988-2017	56,408.60	0.00975	108,091.90	43,373.40
1998-2017	73,420.70	0.02521	261,478.50	48,624.60