Clark Fork River — Smurfit-Stone Site

Hydraulic Modeling, Flood Mapping and Breach Analysis Report

Aerial Photo of Clark Fork River near upstream end of Smurfit-Stone Mill site during 2018 flood.

Prepared for CLARK FORK COALITION

In Partnership with AMERICAN RIVERS

RDG RIVER DESIGN GROUP

July 2024

Clark Fork River - Smurfit-Stone Site Hydraulic Modeling, Flood Mapping and Breach Analysis

Prepared for:
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In Partnership with:
American Rivers

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Table of Acronyms

Acronym	Meaning
AEP	Annual Exceedance Probability
cfs	cubic feet per second
CFR	Clark Fork River
CMZ	Channel Migration Zone
DA	drainage area
EPA	Unites States Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
HEC	Hydrologic Engineering Center
MOVE.3	maintenance of variance type I
NAVD88	North American Vertical Datum of 1988
POR	period of record
Q100	denotes 100-year recurrence interval
RAS	River Analysis System
sq. mi.	square miles
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WY	Water Year (Oct. 1 – Sept. 30)

Executive Summary

This report presents the results of a hydraulic modeling study undertaken on the Clark Fork River to improve the understanding of potential effects from berm failures at the former Smurfit-Stone Frenchtown Mill Site located near Frenchtown, Montana. The Clark Fork Coalition, in association with American Rivers, retained River Design Group (RDG) to develop a reach-scale hydraulic model for use in evaluating potential failure mechanisms for the berms separating the Clark Fork River from contaminated floodplain areas at the Site and the resulting consequences in terms of flooding and mobilization of potentially contaminated sediment. The study improves the understanding of river flows and hydraulics associated with historical and potential future conditions resulting from climate change scenarios.

Hydraulic modeling and mapping was completed for three flood scenarios. Beyond the risk of overtopping and inundation, the report also analyzed and modeled potential berm failure risks of different breach scenarios under modest, moderate and extreme high flow conditions. The three berm breach scenarios modeled were:

1. Piping failure at historical 1% AEP (100-year) flow of 66,000 cfs.
2. Mass wasting failure at future 1% AEP (100-year) flow of ~100,000 cfs.
3. Overtopping failure at future 0.2% AEP (500-year) flow of ~130,000 cfs.

Modeling results show that inundation through the Site is constrained by the existing berms for the historical 1% Annual Exceedance Probability (AEP) (100-year) flow of 66,000 cfs. Results for the future 1% AEP (100-year) flow of ~100,000 cfs show limited overtopping of the berms near the upstream end of the Site. The future 0.2% AEP (500-year) flow of ~130,000 cfs results in extensive overtopping of the berms and inundation of the Site. Modeling results are presented as a series of videos showing water velocity for the three scenarios:

- **Piping failure: https://youtu.be/tmd_WWIucRg**
  This video shows a piping berm failure at the Smurfit-Stone mill site modeled at 66,000 cfs. This flow is based on historical flow data at the USGS stream gauge below Missoula. Piping in this model occurs at the location of an existing outfall pipe. Piping occurs when water seeps under and creates boils or seeps through the berm where weak points exist, such along existing pipes and where rodents have dug holes into the berm.
- **Mass wasting failure: https://youtu.be/lZU9RnB88ws**
  This video shows a mass wasting breach at the Smurfit-Stone mill site modeled at about 100,000 cfs. This flow is based on future climate predictions, which forecast larger, more frequent floods. During this type of failure, water impinging on the berm erodes away soil and ultimately causes failure of the berm resulting in flood water entering Holding Ponds 1 and 2 with potential to wash contaminants downstream as water exits the breach.

- **Overtopping failure: https://youtu.be/GuAcFnerdTM**
  This video shows floodwaters overtopping the berm at the Smurfit-Stone mill site modeled at 130,000 cfs. This flow is based on future climate predictions, which forecast larger, more frequent floods. During overtopping, water flows over the top of the berm at high velocity at lower points along the berm and erodes the back side of the berm, ultimately causing it to breach. Floodwater inundates most of the former wastewater ponds in the floodplain (#s 1, 1a, 2, 7, 9, 10, 11, 12, 13 and 13a) and one of the landfills (#6). Water flows swiftly across the site, potentially sweeping toxins downstream.

The analysis shows that there is potential for berm failure under all three scenarios. There are also feasible berm failure scenarios outside the scope of this study including, but not limited to, multiple concurrent breaches, sequential breaches (ex. piping followed by mass wasting), and secondary failures of the inner berms that surround the waste dumps. Failure of the berms modeled in this study could lead to dispersal of contaminated soil from the Site with resulting deposition throughout the floodplain downstream of the Site. Maps were developed showing flood inundation extents for historic and future flows and the three berm breach scenarios.

1 Introduction

This report presents the results of a hydraulic modeling study undertaken on the Clark Fork River (CFR) to improve the understanding of potential effects of berm failures at the former Smurfit-Stone Frenchtown Mill Site (Site) shown in Figure 1-1. The Clark Fork Coalition (CFC), in association with American Rivers, retained River Design Group, Inc. (RDG) to develop a reach-scale hydraulic model for use in evaluating the risk associated with failure of the berms separating the Clark Fork River from contaminated floodplain areas. The study improves the understanding of river flows and hydraulics associated with historical and potential future conditions.

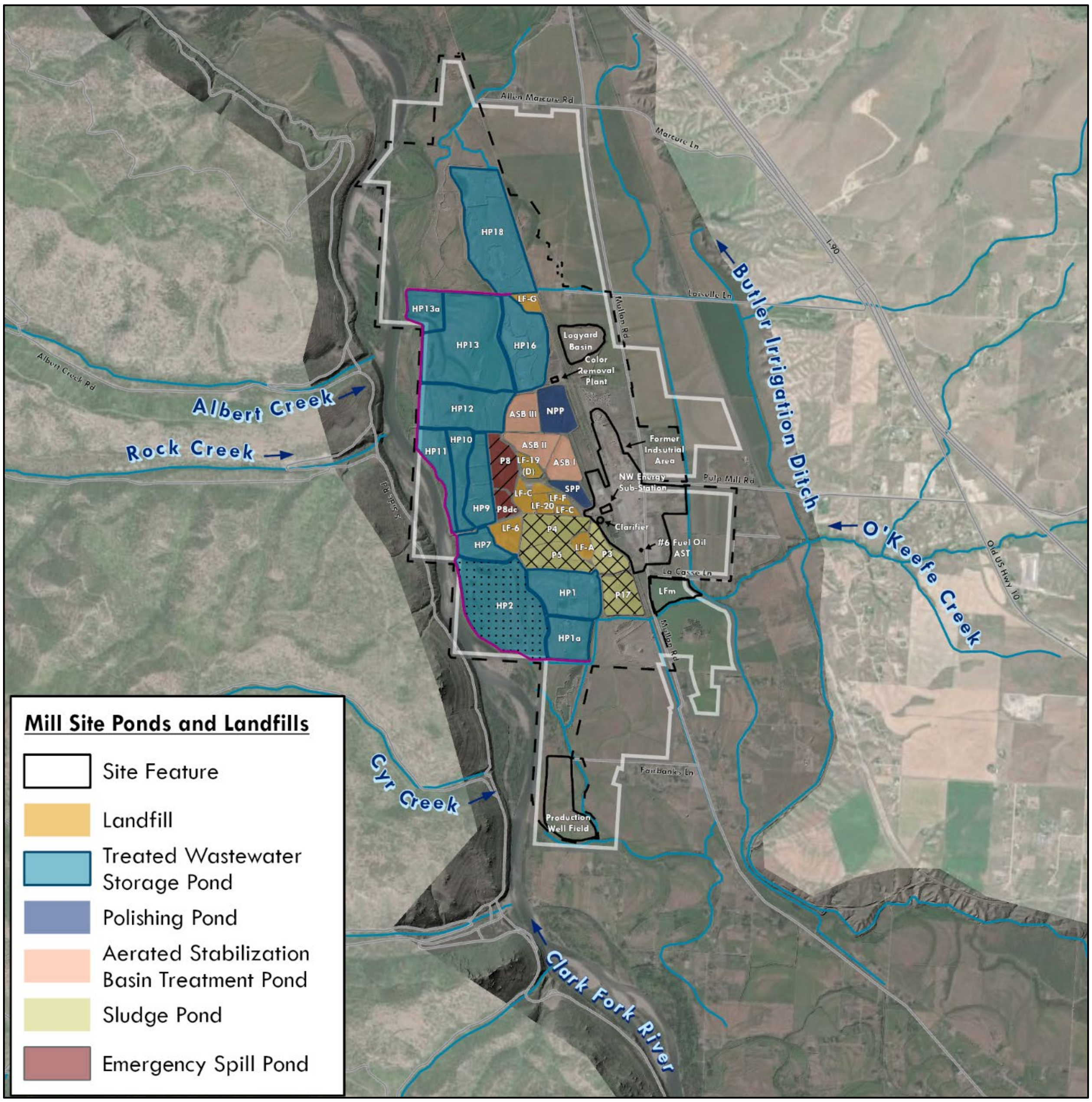

Figure 1-1. Smurfit-Stone Mill Site vicinity map.

1.1 Project Background

The Site is a former industrial pulp and paper manufacturing facility (Mill) that is being evaluated for cleanup and redevelopment using guidance from the Comprehensive Environmental Response, Compensation, and Liability Act administered by the Environmental Protection Agency (EPA). A significant portion of the Site is situated in the floodplain of the CFR. The Site is separated from the CFR by a contiguous series of berms of various heights that extend for a total distance of approximately 4.4 miles (Figure 1-1). The berms have altered river function and there is concern that the berm does not provide adequate protection against a potential breach that could lead to environmental damage.

The berms were built for the purpose of creating holding ponds to retain wastewater that had been previously discharged directly to the river year-round without treatment. The holding ponds facilitated infiltration to groundwater, which dampened the impact of discharges to the river by delaying discharge to the river and reducing Biological Oxygen Demand and total suspended solids. The Mill's wastewater permit was eventually conditioned to prohibit direct discharges to the river during periods of low flow in the summertime, and the ponds allowed waste to be stored until most of the discharge could be released during spring runoff. Under these permit conditions, the limiting factor in the Mill's permit was downstream color, which was limited to a five standard color unit increase (Skidmore pers comm. 2024).

This report describes the development of the hydraulic model that was used to evaluate potential mechanisms for berm failure. The model was used to simulate potential berm failure mechanisms and potential consequences that could result from berm failure. The terms berm, dike and levee are generally interchangeable in the context of this report and are defined as an embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year.

This report is not a comprehensive stability analysis, nor does it provide recommendations for collection or development of the full suite of data required for a comprehensive berm stability analysis. Stability analysis methods are described in EM 1110-2-1913 (USACE 2000). This report does not replace the need to seek judgement of a qualified engineer for completion of a stability analysis.

1.2 Project Scope

The scope of work for this project includes analysis of historical and future flows for the Clark Fork River, development of a hydraulic model to map flood scenarios and analysis of the potential for dispersal of contaminants resulting from a breach of the berm(s) containing contaminated materials at the Site. The scope of work did not include assessment of potential for an avulsion to occur in response to channel changing events upstream or across the river from the site.

1.3 Project Objectives.

The objectives of the project were to:

- Map various flood scenarios at the Site under both existing conditions and future conditions as influenced by climate change;
- Identify vulnerable areas of the CFR berm at the Site and assess inundation risks from berm failure;
- Analyze and simulate the release of contaminants from a breach into the floodwaters and estimate dispersion of flow over time through the reach downstream of the Site.

1.4 Document Organization

This document is organized into the following sections and appendices.

- ***Section 1 - Introduction*** provides project background information and describes the purpose and scope of the study;
- ***Section 2 - Hydrologic Analysis*** summarizes the information and methods used for hydrologic analysis of past and future conditions;
- ***Section 3 - Hydraulic Modeling*** summarizes methods used for hydraulic modeling and maps developed from the hydraulic model output showing depth and velocity of flow;
- ***Section 4 - Berm Failure Analysis*** summarizes berm failure analysis methods and results***;***
- ***Section 5 - References*** includes citations for literature and studies referenced***;***
- ***Appendix A*** includes supporting information for the hydrologic analysis;
- ***Appendix B*** includes the inundation and velocity maps; and,
- ***Appendix C*** includes the berm failure scenario data and maps.

1.5 Previous Studies and Available Data

Several studies have been completed that evaluate various aspects of flooding and potential for failure of the berms at the Site. Information relevant to this modeling effort can be found in the Flood Insurance Study (FIS), geomorphic investigations, and topographic data referenced below.

1.5.1 Berm Data

A series of berm monitoring and assessment reports is available from the EPA website (EPA 2024). Reports relevant to this analysis include a Geotechnical Evaluation Report (NewFields 2018a) that describes sampling and analysis of berm materials and evaluates stability of the CFR berm during high water conditions. The stability assessment revealed that the majority of the soil samples taken from the berms have good drainage characteristics and are classified as sandy or silty sands. Some samples also contained gravel and silt, which may affect the stability and engineering properties of the soil. Potential deficiencies in the berm were identified, including animal burrowing, vegetation growth, and encroachments. The stability assessment also highlighted the presence of fine-grained foundation soils, such as sandy silt and lean clay, which may contribute to instability in certain areas. The allowance language from the report reads as follows: "This report has been prepared exclusively for the PRPs. No third party, other than NewFields, shall be entitled to rely on any information, conclusions, opinions, or other information contained herein without the express written consent of the PRPs. Any third party that relies upon any information, conclusions, opinions, or other information contained herein without the express written consent of the PRPs understands and acknowledges that NewFields is not liable for any claim arising out of such use." As such, no information from the NewFields report was used in the analysis of the berms presented in this report.

In addition to the Geotechnical Evaluation Report, a Berm Assessment and Reinforcement Report (NewFields 2018b) describes work conducted to reinforce the CFR berm at the location of sand boils that were identified during high water in 2018. A Surveillance and Contingency Plan (NewFields 2019) identifies monitoring activities to be undertaken during CFR high flow events and identifies actions that would be taken when issues are identified.

1.5.2 Flood Insurance Study

Portions of the Site are located within the FEMA 100-year floodplain. Montana Department of Natural Resources and Conservation (DNRC) worked with FEMA to update Flood Insurance Rate Maps for Missoula County which became effective on October 5, 2023. Inundation mapping completed for the FIS assumes that the non-certified levees (berms) constrain flow between the berms and the left bank resulting in conservatively high flood elevations. The Flood Insurance Rate Maps use the conservatively high flood elevations to map inundation across the floodplain as though the non-certified levees (berms) do not prevent flow through the floodplain. As such, inundation areas shown on the Flood Insurance Rate Maps should not be assumed to represent actual flooding conditions, but rather potential inundation depth relative to modeled water surface elevations in the CFR channel.

1.5.3 Geomorphic Assessments

Missoula County funded a Channel Migration Zone (CMZ) and Mapping Study for the CFR (Boyd and Thatcher 2021). That study includes information about the geomorphic context of the Smurfit reach. As part of the CMZ study, historical information about channel locations and conditions of the existing flood protection facilities was summarized.

1.5.4 Topographic Data

Available topographic data for the CFR berms includes LiDAR (Quantum 2019). These data provide a reasonable basis for numerical modeling of overland flows, for example as a result of a berm breach, and for measurement and characterization of the berm (height, top width, side slope, etc.). All elevation data used in this study references the North American Vertical Datum of 1988 (NAVD88).

2 Hydrologic Analysis

This section summarizes data and methods used to assess hydrology for the Site. Hydrologic data for the historic condition was developed using analysis of nearby streamflow gage data and gage data extension techniques. Future flows were estimated using climate modeling data developed by the University of Washington and Oregon State University (Chegwidden et al 2017). Resulting peak flows and hydrographs were used in the hydraulic modeling described in Section 3.

2.1 Watershed and Tributaries

The headwaters of the CFR originate in Silver Bow Creek and Anaconda Creek. The CFR joins the Bitterroot River just downstream of Missoula (Figure 2-1).

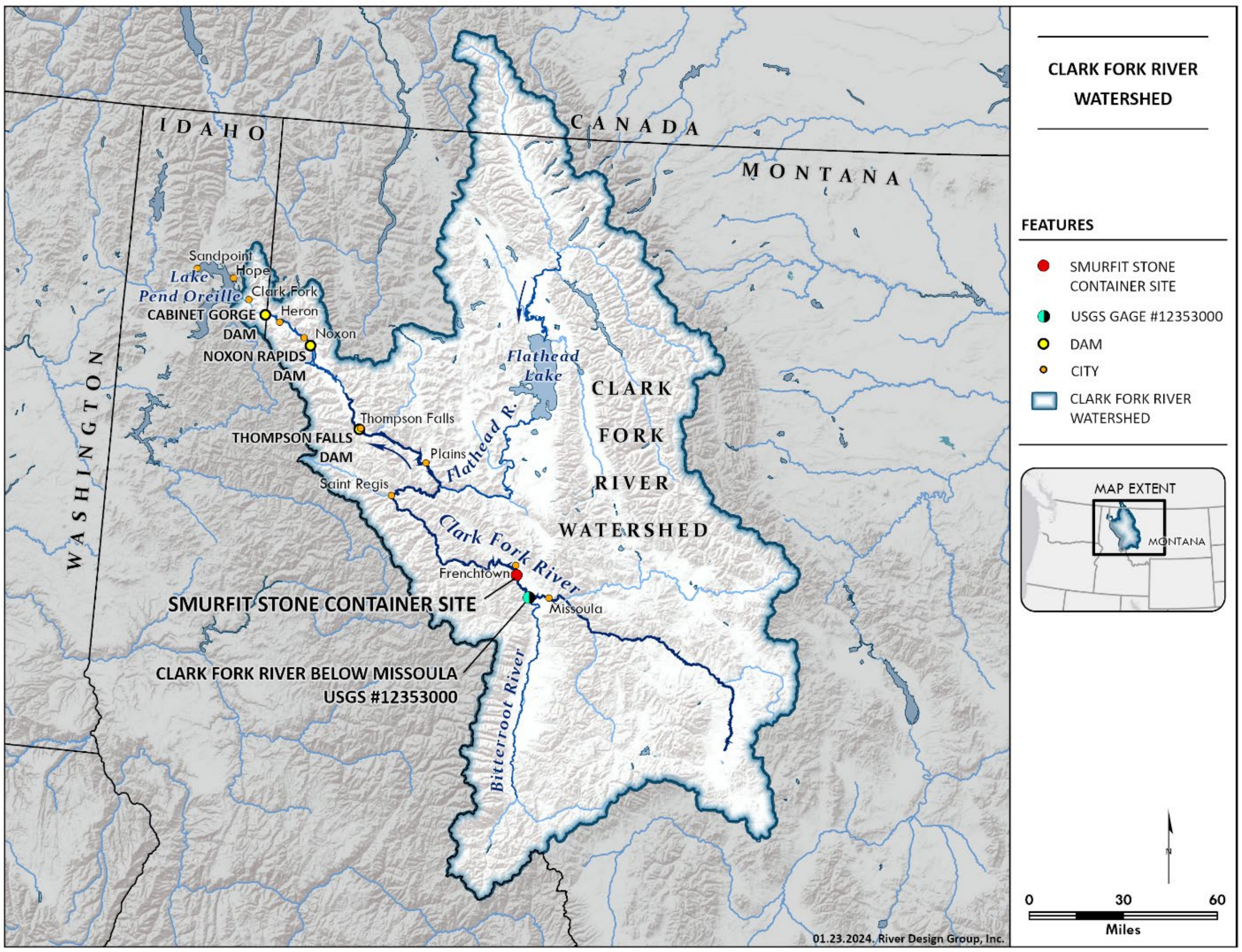

Figure 2-1. Study Reach watershed and gage map.

2.2 Hydrologic Data

The USGS has maintained a streamflow gaging station on the CFR downstream of Missoula (USGS 12353000) since 1929. This gage is located approximately one mile upstream of the Site and is the primary source of data used for the historical flood frequency analysis. Hydrographs of mean daily flows are presented in Figures 2-2 and 2-3.

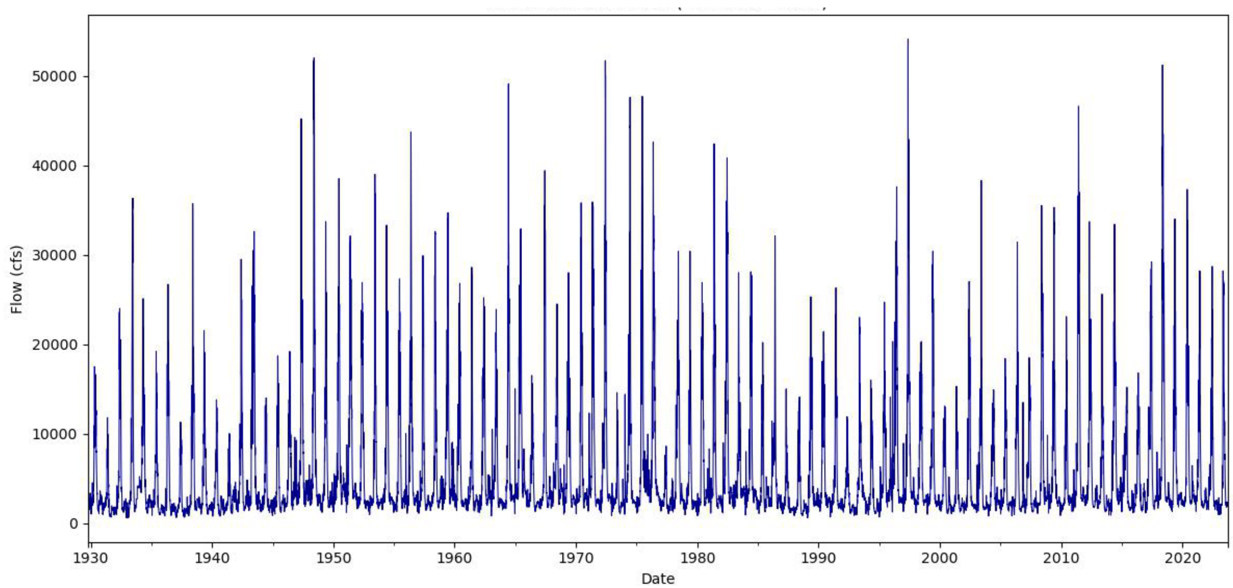

Figure 2-2. Hydrograph of CFR mean daily flows from 1927 through 2023.

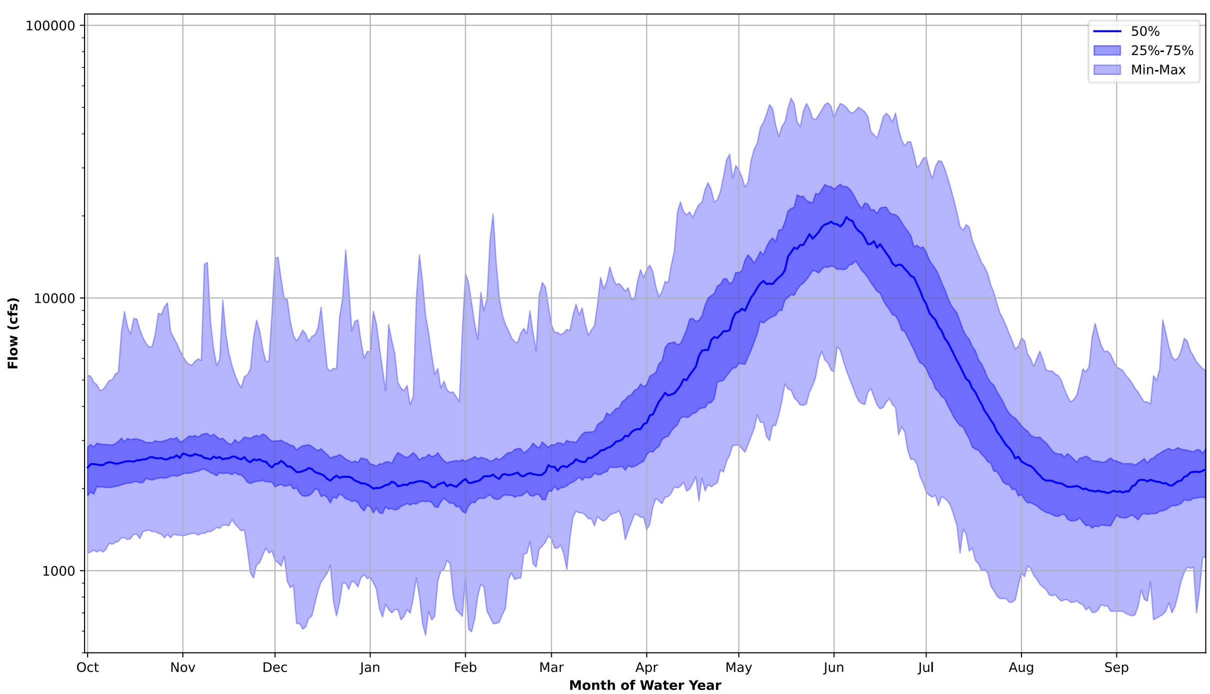

Figure 2-3. Summary hydrograph of CFR mean daily flows from 1927 through 2023.

Annual peak flows have historically occurred in May, June, and July indicating a snowmelt dominated runoff pattern. The 96-year period of record includes several notable flood events. In May 1997, the peak flow reached 55,100 cubic feet per second (cfs), which is roughly equivalent to the 4% Annual Exceedance Probability (AEP) (25-year) event. The second highest flood on record occurred in 1948 with a flow of 52,800 cfs, followed closely by floods in 1972 and 2018 with flows of 52,200 cfs.

2.2.1 Historical Flood Frequency

Historical peak flow estimates were developed for the recent Flood Insurance Study (FEMA 2021). Flood frequency statistics were computed using Bulletin 17c methods (Sando et al. 2018). The peak flow record was extended using the MOVE.3 record extension technique (USGS 2018). Peak flow quantiles were transposed to the location of the Smurfit-Stone Mill Site based on drainage area (Allied 2023). Results of the flood frequency analysis in the vicinity of the Site are summarized in Table 2-1. The Rock Creek tributary (Node #7001) is located adjacent to the project site. Peak flows from the FIS are compared with future peak flow estimates in Table 2-2.

Table 2-1. Peak flow estimates for CFR below Missoula (USGS 12353000).

USGS # / Node	Flooding Source and Location	Peak Flow (cfs)				
		10% AEP (10-yr)	4% AEP (25-yr)	2% AEP (50-yr)	1% AEP (100-yr)	0.2% AEP (500-yr)
123530002	CFR below Missoula, MT	47,200	54,900	60,200	65,000	75,200
8001	Deep Creek	47,500	55,200	60,600	65,400	75,600
7001	Rock Creek	48,000	55,800	61,200	66,000	76,300

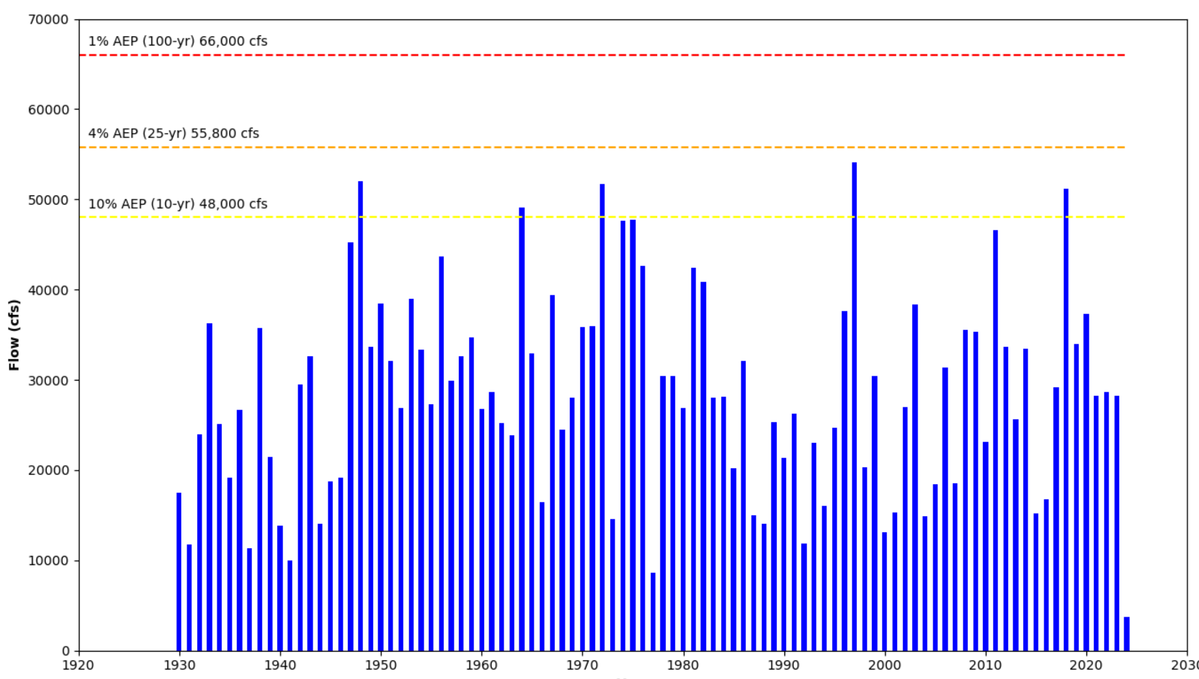

Figure 2-4. Summary hydrograph of CFR mean daily flows from 1927 through 2023.

2.2.2 Climate Change Data

Climate change is expected to affect temperature and precipitation in the Pacific Northwest and change the region's hydrology. A comprehensive study was completed by researchers at the University of Washington and Oregon State University to model the impacts of climate change on future streamflow in the Columbia River Basin (Chegwidden et al 2017). The study provides projected streamflow information for the 21st century for 396 locations throughout the Columbia River Basin, including the location of the CFR below Missoula gage (USGS 12353000). The effort began with the selection of appropriate approaches to estimating future climate scenarios. The research team reviewed available climate models and identified an ensemble of hydrologic and climate change simulations appropriate for the Pacific Northwest spanning the years 1950 to 2099. This included the selection of relevant Representative Concentration Pathways (RCPs) to represent various greenhouse gas emission trajectories. RCPs describe potential 21st century greenhouse gas emissions based on different levels of air pollutant emissions and land use changes.

Translation of climate data into runoff and streamflow was a stepwise process. First, global climate models were employed to simulate the future meteorological data based on the selected RCPs. Second, downscaling methods were utilized to translate coarse-scale meteorological data to finer scales suitable for hydrological models. Third, hydrological models were then used to simulate runoff from snowmelt and rainfall. Fourth, runoff was routed downstream using a routing model to understand how water moves through the stream network. Each of these components plays a crucial role in understanding and projecting future hydrological conditions under different climate scenarios.

Future climate projections from two RCPs were selected for analysis in the Columba River Climate Change study: RCP 4.5, representing mid-range greenhouse gas concentrations, and RCP 8.5, representing very high concentrations. Output from ten global climate models were used to investigate the uncertainty within hydrologic projections. Output from the global climate models was downscaled from a relatively coarse spatial scale of ~150 km to a finer spatial resolution of ~6 km for hydrologic modeling using the Multivariate Adaptive Constructed Analogs (MACA) downscaling method. Four different hydrologic models were employed, including three implementations of the Variable Infiltration Capacity (VIC) model and one implementation of the Precipitation Runoff Modeling System (PRMS-P1) model.

A total of 160 datasets representing streamflow resulting from two future climate scenarios were developed for the CFR below Missoula. A summary hydrograph for one of the datasets that exemplifies potential future flows is presented in Figure 2-4.

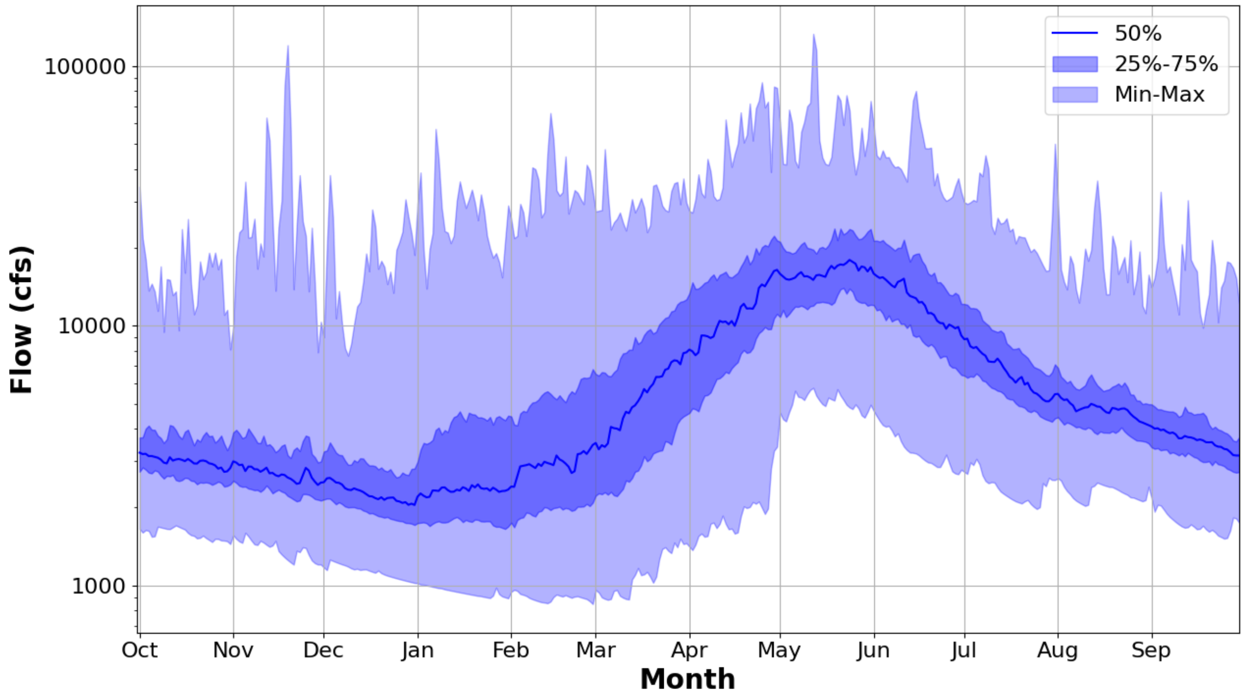

Figure 2-5. Summary hydrograph that exemplifies potential future flows under RCP 8.5.

Temperature increases will result in more rainfall in winter, less water stored as snow, and earlier melt of the thinner snowpack. For some rivers, peak flows may no longer occur in spring, but may occur in fall and winter instead. Warmer summers may increase drought conditions, especially if less spring and summer runoff is available from mountain snowpack.

2.2.3 Future Flood Frequency

In the warmer future climate projected by climate scientists, we can expect a shift in the causes of high flow events, with fewer occurrences due to snowmelt and more due to precipitation. Additionally, high flow events triggered by precipitation are more responsive to increased precipitation levels compared to those driven by snowmelt. This double impact of more frequent and intense precipitation-driven high flow events increases both the probability and magnitude of flooding compared to either change in isolation.

Future peak flow quantiles were estimated using modeled future flow data (Chegwidden 2017). Maximum daily flows for each water year were extracted from 160 datasets representing two different future climate scenarios for the period from WY 1950 - 2099. Bulletin 17c flood frequency analyses were completed for each of the datasets. Median future flows for each climate forcing scenario are compared with historical flows in Table 2-1 and Figure 2-5. Unsteady flow hydrographs scaled from actual events that were used for unsteady flow modeling are shown in Figure 2-6. Peak flow quantiles were grouped by climate forcing scenario and the mean and median values were computed for each group (Figures B-1 and B-1 in Appendix B).

Histograms showing the distribution of 1% AEP (100-year) flows are shown in Figures B-3 and B-4 in Appendix B.

Table 2-2. Comparison of historic and future flood flows for the CFR.

Return Interval (years)	Annual Exceedance Probability	Historical Flow From FEMA FIS at Rock Creek (Allied 2023)	Future Flows - Median by Emission Scenario Mid-range (RCP 4.5)	High (RCP 8.5)
500	0.2%	76,300	118,000	**130,000**
200	0.5%	--	103,000	111,000
100	1%	**66,000**	92,300	**98,000**
50	2%	61,200	81,900	86,000
25	4%	55,800	71,900	75,000
10	10%	48,000	59,100	61,000
5	20%	--	49,500	50,000
2	50%	--	35,700	36,000
1.5	67%	--	30,300	30,000

Bold values indicate flows used in hydraulic modeling.

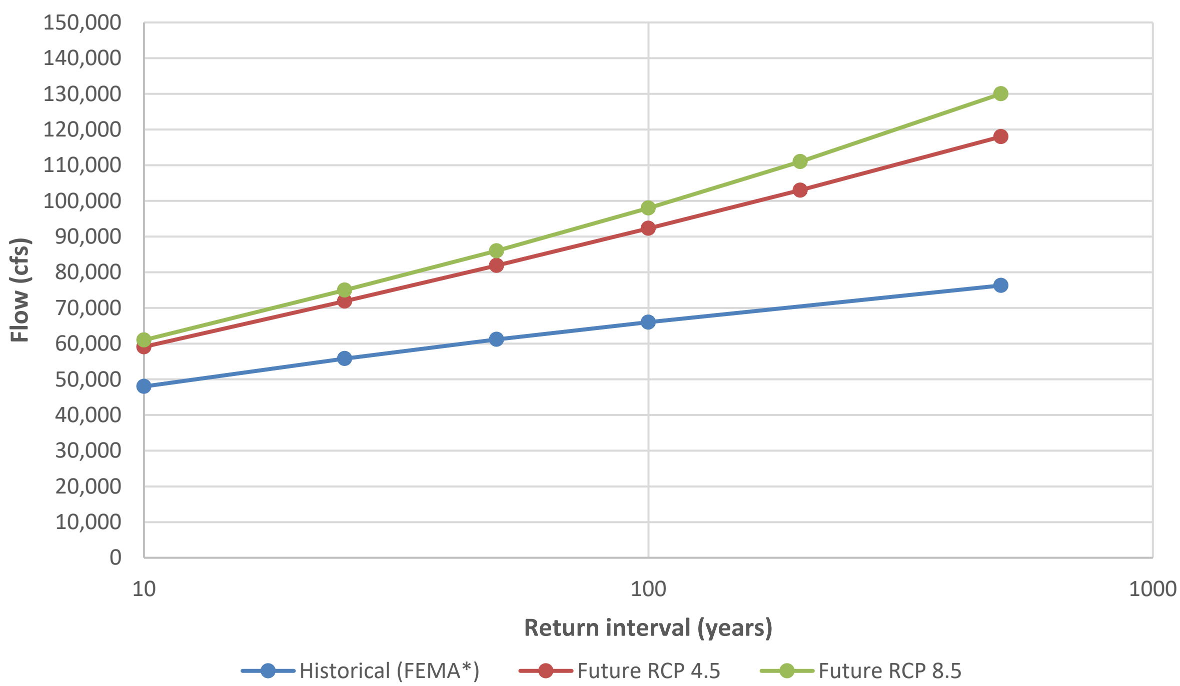

Figure 2-6. Comparison of flood quantiles for historical and future flows.

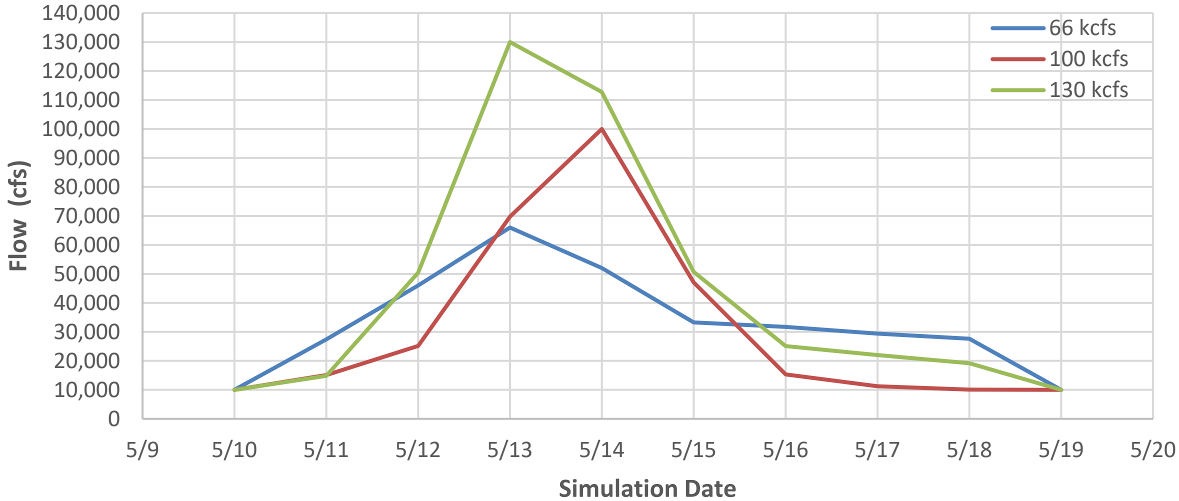

Figure 2-7. Synthetic flow hydrographs for historical and future flows.

2.3 Hydrologic Data Uncertainty

Estimates of peak flow quantiles contain some level of error due to several factors. Measured peak flow values reported by USGS typically have an uncertainty range of 5% to 10%. Uncertainties in modeled future flows include uncertainty inherent in the choice of climate forcing scenario, global circulation model, downscaling method, and hydrologic routing model. Uncertainty estimates for Bulletin 17c flood frequency analyses are reported in terms of confidence limits for historical flood quantiles in Appendix B with differences between upper and lower confidence limits for the historical 100-year flow of +17% and -10% respectively. Differences between instantaneous peak flows and mean daily flows estimated using historical flow data are relatively small at about 2% (Figure 2-7).

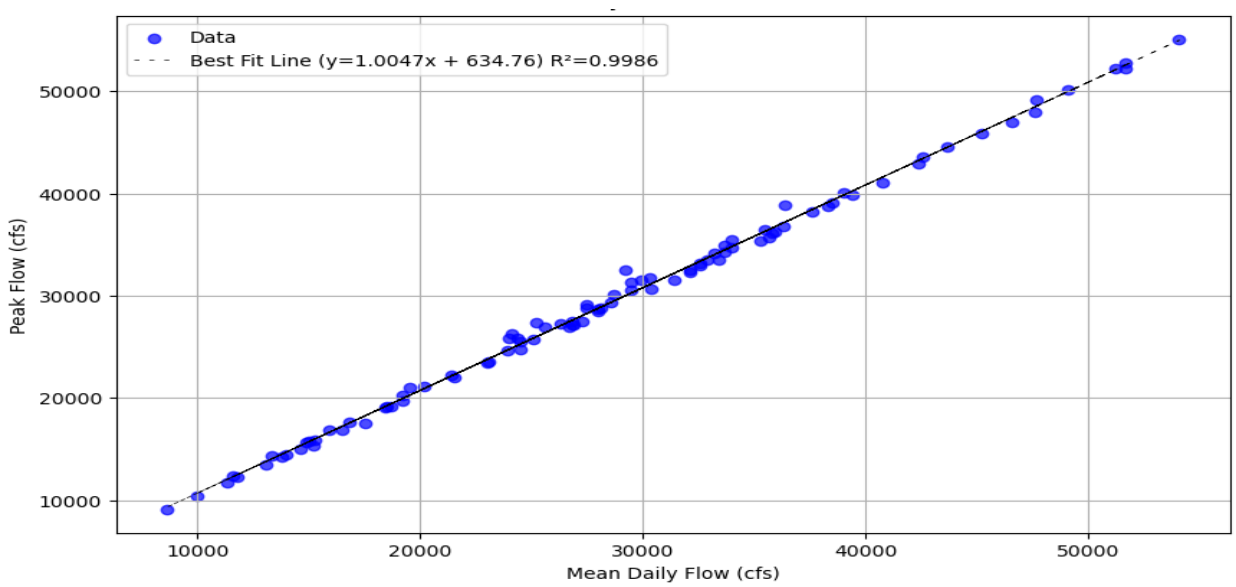

Figure 2-8. Comparison of daily flows with peak flows.

3 Hydraulic Modeling

Hydraulic models were used to evaluate potential for overtopping of the berms separating the Site from the CFR. Model outputs were used to map depth, velocity, and inundation area for historical and future flows. The models were also used to evaluate three berm breaching scenarios.

The hydraulic modeling was conducted using the HEC-RAS modeling software (USACE 2023). Both 1-dimensional (1D) steady-flow and 2-dimensional (2D) unsteady-flow models were developed. The models were adapted from models developed for the recently completed Flood Insurance Study (Allied 2023). The hydraulic models and terrain surfaces developed for the flood study were used as a starting point for development of the models used for this study.

The hydraulic model of Study Reach was developed using HEC-RAS v6.1.4 (USACE 2023). The model begins near the confluence with the Bitterroot River and extends downstream to Alberton Gorge. The model schematic is included in Appendix C.

3.1 Topographic Data

Geometric data for the existing channel and floodplain were sampled from the terrain data developed for the FIS (Allied 2023). Lidar data was used for mapping the model outputs (Quantum 2019).

3.2 Model Geometry

A total of 350 cross sections were used to represent the geometry of the existing channel and floodplain in the 1D model. The cross sections were oriented to remain perpendicular to the expected flow lines for both small (1- to 50-year) and large magnitude (100- to 500-year) flood events, sometimes requiring multiple horizontal inflection points. The cross sections extend orthogonally across the floodplain to capture the maximum potential inundation for the estimated 500-year flood elevations.

3.3 Boundary Conditions

The downstream boundary for the model was approximated using a normal depth slope of 0.0006 ft/ft. The downstream boundary was selected approximately 4 miles downstream of the study reach to avoid the potential for influencing study results.

3.4 Roughness Parametrization

Roughness coefficients were estimated based on land cover (Allied 2023). A single channel roughness value was used to enable calibration to observed data. Selected roughness values are within the range of values for natural streams reported in Arcement and Schneider (1989).

3.5 Model Validation

Output from the FEMA FIS model was compared with gage data to ensure that the model is reasonably accurate. Differences in water surface elevations between the model and the gage range were generally within 0.5 feet (Allied 2023).

3.6 Model Results

Water surface elevations were compared with berm elevations to determine potential for overtopping during the three flood events modeled (Figure 3-1). Results for the future 1% AEP (100-year) flow of ~100,000 cfs show limited overtopping of the berms near the upstream end of the Site. The future 0.2% AEP (500-year) flow of ~130,000 cfs results in extensive overtopping of the berms and inundation of the Site.

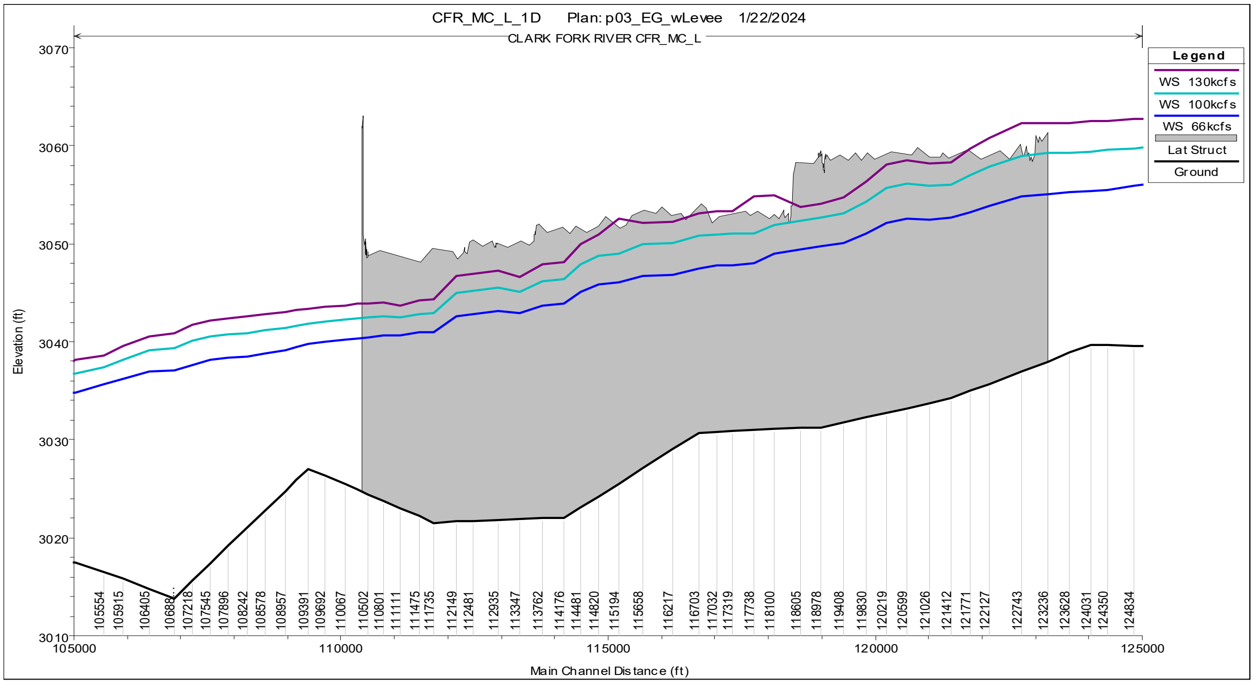

Figure 3-1. Modeled water surface elevations and berm profile.

3.7 Inundation and Velocity Mapping

Hydraulic model output was used to map potential depth of inundation and velocity for the historical 1% AEP (100-year) flow of 66,000 cfs, the future 1% AEP (100-year) flow of ~100,000 cfs, and the future 0.2% AEP (500-year) flow of ~130,000 cfs. The inundation area through the Site is constrained by the berms for the historical 1% AEP (100-year) flow of 66,000 cfs. Results for the future 1% AEP (100-year) flow of ~100,000 cfs show limited overtopping of the berms near the upstream end of the Site. The future 0.2% AEP (500-year) flow of ~130,000 cfs leads to extensive overtopping of the berms and inundation of the Site. Depth and velocity maps are included in Appendix B.

3.8 Model Limitations

The Study Reach model is a reach-scale model that was developed primarily as a tool to aid in the determination of floodplain inundation extents. There may be deviations between the reported water surface elevations and actual water surface elevations between cross sections due to features (e.g., riffles and pools) located in the area between cross sections that are not represented in the model geometry.

Additional model limitations include simplification of channel roughness. Channel roughness was treated as a uniform value as overbank flows were of primary importance to this effort. More detailed parameterization of channel roughness may be warranted to ensure adequate model performance for evaluation of flows other than the 1% AEP (100-year) stage.

4 Berm Failure Analysis

The three primary causes of berm failures are overtopping, piping, and mass wasting. Overtopping is the most common cause of failures (Chaudhry 2022). Many potential failure scenarios are possible depending on flow and velocity conditions. This report analyzed three failure scenarios under modest, moderate and extreme high flow conditions. The focus of this effort was to develop plausible breach scenarios and model those scenarios to predict flow through the breaches resulting from berm failures and the map the resulting flows through the Site. It should be noted that piping and mass-wasting failure modes could also occur at different peak flows. This study did not identify 'threshold' flows at which such failures become likely.

4.1 Berm Inspections and Pertinent Observations

Berm inspection records can provide useful information related to potential failure mechanisms and risks. General observations based on the Missoula Conservation District permit requests between 1976 and 2018 are summarized in **Table 4-1**. Regular visual inspections at low water conditions as well as during floods have been conducted since 2010.

Table 4-1. Missoula Conservation District summary of 310 permits requested for berm repair.

Date	Repair requested
5/17/1976	300 LF Pond 2 (west side), 300 LF Pond 11, 50 LF outfall from Pond 2, 200 LF Pond 2 south, 1000 LF along well field
10/1/1976	30 in outfall pipe
10/1/1978	200 LF Pond 2 south, 1200 LF Pond 2 west, 1600 LF pond 11 west, 50 LF Area D near Pond 13A, 200 LF NW corner 13A
10/1/1985	500 LF Pond 2 (same area as 1976 and 1978 permits)
4/1/1990	100 LF riprap plus reinforcement of 500 LF on both sides of failure (location unspecified)
9/1/1991	2 rock barbs at Pond 2
… No records noted for the period from 1991 to 2018	
5/24/2018	EPA directed Potentially Responsible Parties to place ~2,100 cy of fill material on top of several hydraulic boils observed within HP13

4.2 Berm Failure Scenarios

The berm failure process is complex, involving interaction between the water flow, sediment transport and geomorphologic changes. Estimation of breach parameters for non-cohesive earthen berms and the modeling of potential breaches caused by piping, mass wasting and overtopping are discussed in the following sections. The three berm failure scenarios evaluated are listed in Table 4-2.

Table 4-2. Berm failure scenarios.

Scenario	Failure mechanism	Flood event	Flow (cfs)
1	Piping	Historical 1%	66,000
2	Mass wasting	Future 1%	100,000
3	Overtopping	Future 0.2%	130,000

4.3 Berm Characterization (Physical Properties)

Documentation or design information from the construction of the berms is not readily available. Channel surveys and topographic data (LiDAR) available for the study reach were used to characterize the geometric properties of the existing berms (height, top width, side slopes). The composition and quality of construction of the berms is generally unknown and thus this information cannot be used to predict specific locations particularly susceptible to breaching. The PRPs recently completed a geotechnical stability evaluation of the CFR Berm and a visual reconnaissance survey. The study found under high water conditions, similar to those seen in 2018, that there is a potential at some locations on the berm for under-seepage to occur, and that under-seepage has the potential to increase the instability of the berm via erosion of materials below the berm (NewFields 2018).

At the time of construction, techniques used to construct these types of berms generally consisted of dredging material from the river channel or adjacent floodplain, placing this material into berms on the banks with minimal compaction and then facing the riverward side of the berms with rock riprap, sometimes without any toe rock. Since then, portions of these berms may have been raised and widened but the berms have not been comprehensively rebuilt to today's engineering standards. As such the potential for breaches due to seepage, boils, sloughing, or overtopping is considerably higher than for engineered levees built to current standards.

4.4 Berm Failure Modeling

Modeling of the breach process is based on the numerical solution of the two-dimensional shallow water equations simultaneously with the parameterized breach development. Breaches with time-variant trapezoidal cross sections were modeled. Three different cases were simulated by varying the breach width, depth and submergence of the breach crest.

As part of this study, the HEC-RAS hydraulic model was updated to include the geometry of the berm separating the CFR from the Site using the LiDAR and channel bathymetry data in the FEMA terrain. The model was used to identify locations along the berm that are subject to high velocities on the top and backside of the berm during overtopping. These locations would be at significantly greater risk of an overtopping Berm failure than locations that do not overtop or experience only minimal overtopping flows and velocities.

The 2D model geometry was refined to improve performance for breach modeling. The model domain was reduced to the area within about one mile of the project site. The water surface elevations at the downstream end of the Site were compared to ensure that the breach model hydraulics are similar to those of the full model.

Breach parameters for each of the three scenarios were estimated using empirical equations. Breach parameters estimated include breach width and lateral erosion rate. Breach development time was calculated based on these parameters. Breach parameters used for each scenario are summarized in Table C-1 in Appendix C.

Breach width was estimated using equations developed by Zomorodi (2020) that define the relationship between breach width and levee/berm height for non-cohesive levees/berms based on data from actual breaches. According to Zomorodi, the following equations are representative of older non-engineered riverine levees and berms.

> *The following equation (Zomordi, Eq. 10) gives a reasonable conservative design value for breach width for non-cohesive levees with heights from 2.0 to 6.5 m:*
>
> $Wb = 3.5\,(Hl+1.5) * 2.0$, *where* Wb *is breach width and* Hl *is levee (or berm) height.*
>
> *The upper limit for breach width was calculated using the following equation (Zomordi, Eq. 11):*
>
> $Wb = 3.5\,(Hl+2) * 2.32$, *where* Wb *is breach width and* Hl *is levee (or berm) height.*
>
> *The lower limit for breach width was calculated using the following equation (Zomordi, Eq. 12):*
>
> $Wb = 1.5\,(Hl+0.5) * 1.9$, *where* Wb *is breach width and* Hl *is levee (or berm) height.*
>
> *The following equation (Zomordi, Eq. 16) gives a reasonable conservative average breach lateral erosion rate for non-cohesive levees in m/hr for levee heights from 2.0 to 4.0 m:*
>
> $LLa = 15.0 + 10.0 * Hl$, *where* LLa *is lateral erosion rate and* Hl *is levee (or berm) height.*
>
> *The upper limit of the lateral erosion rate was calculated using the following equation (Zomordi, Eq. 17):*
>
> $LLa = 54.7 + 23.4 * Hl$, *where* LLa *is lateral erosion rate and* Hl *is levee (or berm) height.*
>
> *The upper limit of the lateral erosion rate was calculated using the following equation (Zomordi, Eq. 18):*
>
> $LLa = 12 + 2.7 * Hl$, *where* LLa *is lateral erosion rate and* Hl *is levee (or berm) height.*

Breach weir and piping flow coefficients were selected within the range reported HEC-RAS Hydraulic Reference Manual Table 13-2 (USACE 2023). Overflow/weir coefficients for earthen sand and gravel dams ranged from 2.6 to 3.0 and piping coefficients ranged from 0.5 to 0.6.

4.5 Berm Failure Modeling Results

Berm failure modeling indicates that there is potential for potential for berm failure under all three scenarios modeled. Results for Scenario 1, the piping failure at the historical 1% AEP (100-year) flow of 66,000 cfs, show that a piping failure could lead to partial inundation of Holding Pond 2 as shown in Figure 4-1. Flows through the breach approach 200 cfs with maximum velocities through the breach of 5 ft/sec (Figure 4-2). The piping failure model output is captured as a video clip (https://youtu.be/tmd_WWIucRg).

Results for Scenario 2, the mass wasting failure at the future 1% AEP (100-year) flow of ~100,000 cfs, show that a mass wasting failure could lead to complete inundation of Holding Pond 2 as shown in Figure 4-3. Water also flows across the interior berm into Holding Pond 1 in this scenario (Figure 4-4). Flows through the breach exceed 10,000 cfs with maximum velocities through the breach of 5 ft/sec (Figure 4-5). The mass wasting failure model output is captured as a video clip (https://youtu.be/lZU9RnB88ws).

Results for Scenario 3, the overtopping failure at the future 0.2% AEP (500-year) flow of ~130,000 cfs, show that an overtopping failure could lead to complete inundation of Holding Ponds 1, 2, 7, 9, 10, 11, 12, 13, 13a, as well as potential for failure of interior berms as shown in Figure 4-6. Flows through the breach approach 200 cfs with maximum velocities through the breach of 5 ft/sec (Figure 4-7). The overtopping failure model output is captured as a video clip (https://youtu.be/GuAcFnerdTM).

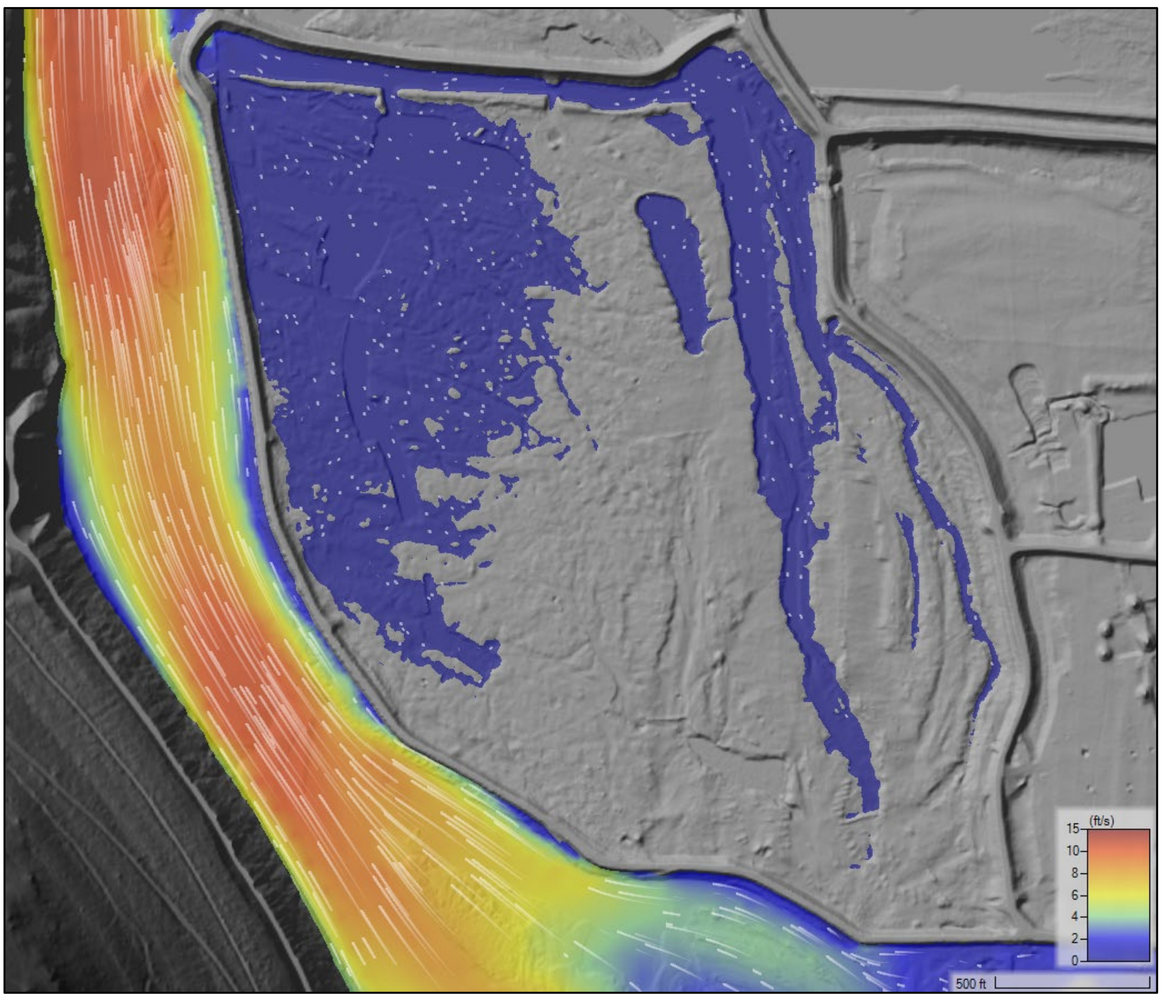

Figure 4-1. Plot of modeled velocities and particle traces for the Scenario 1 piping failure near the north end of Holding Pond 2.

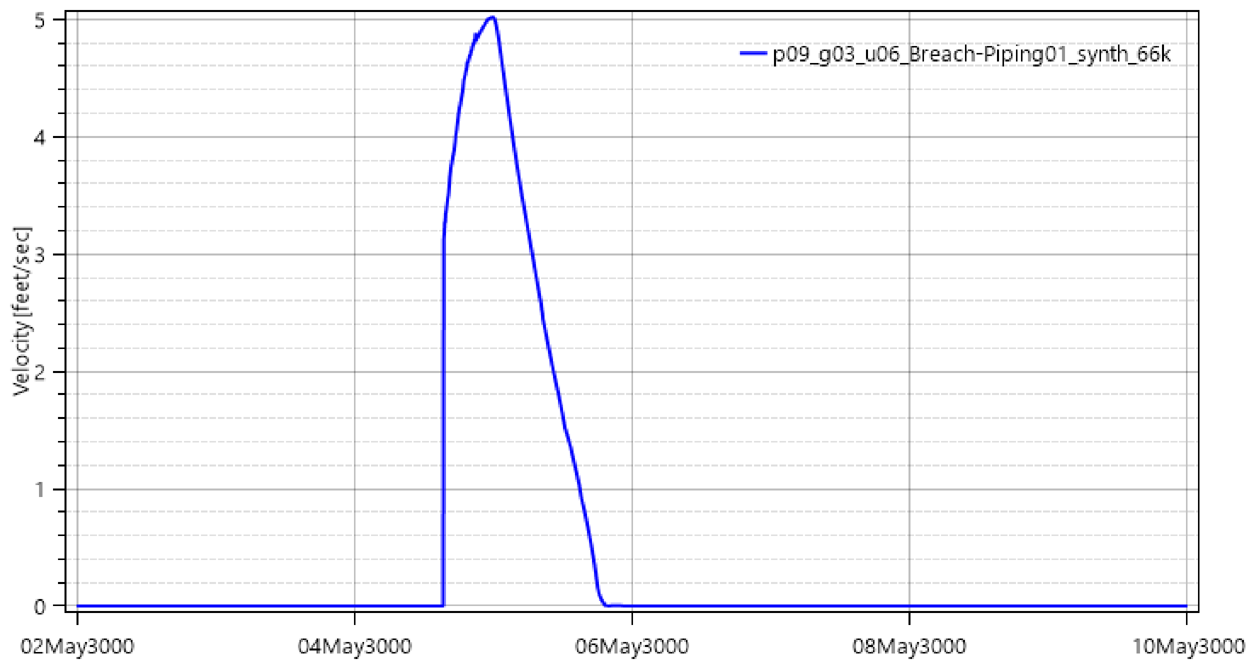

Figure 4-2. Plot of modeled velocities inside of berm at failure point for the Scenario 1 piping failure near the north end of Holding Pond 2.

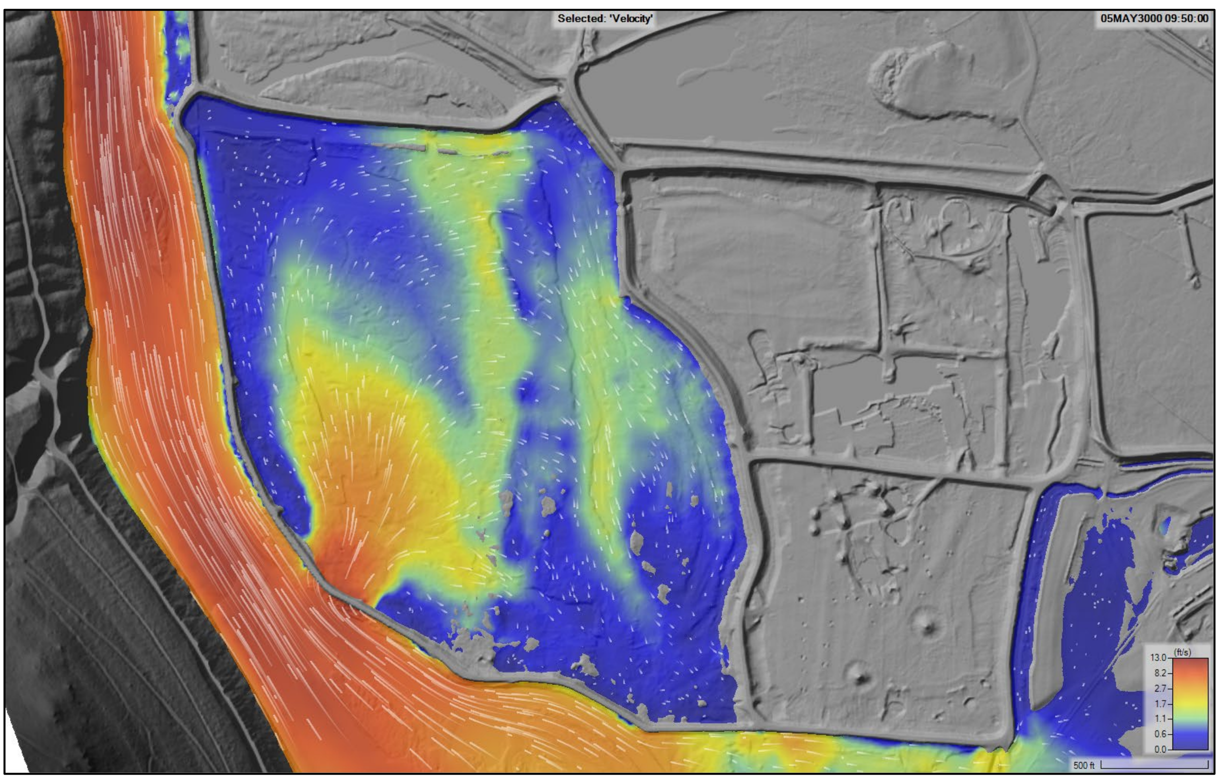

Figure 4-3. Plot of modeled velocities and particle traces at the peak of the Scenario 2 mass-wasting failure near the southeast end of Holding Pond 2.

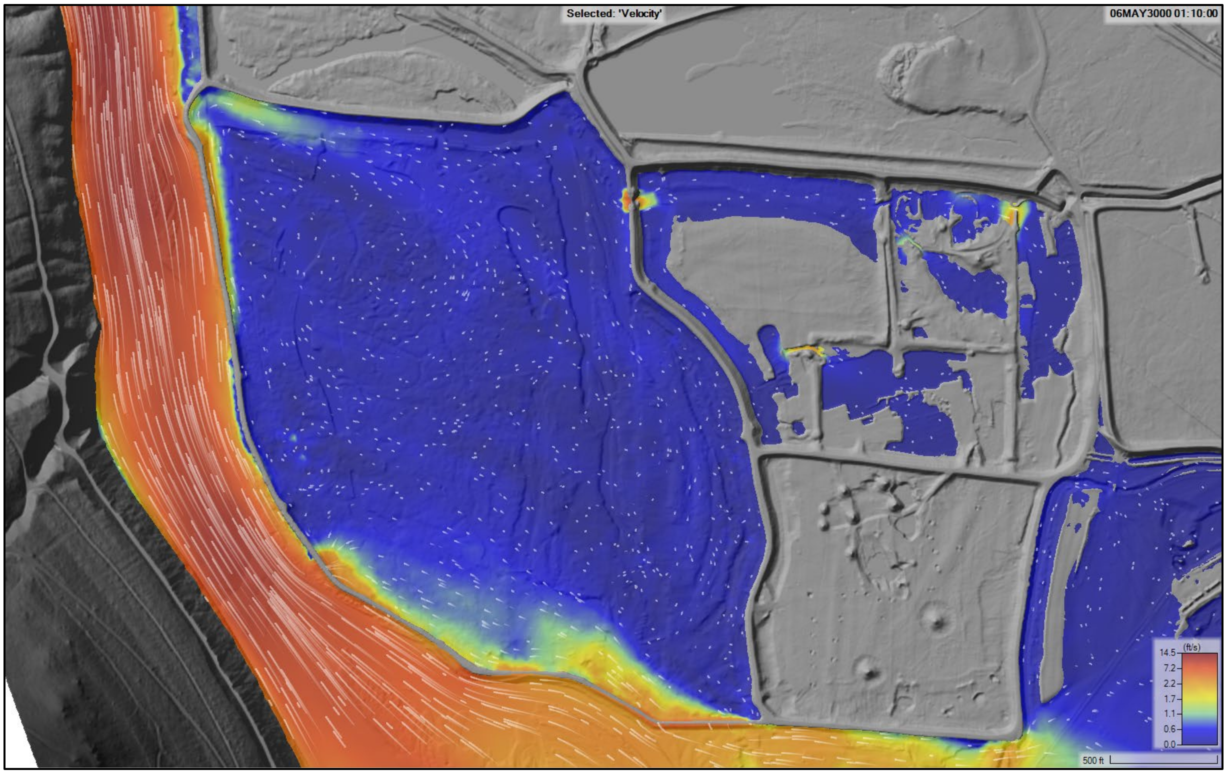

Figure 4-4. Plot of modeled velocities and particle traces for the Scenario 2 mass-wasting failure showing flow across the interior berm into Holding Pond 1.

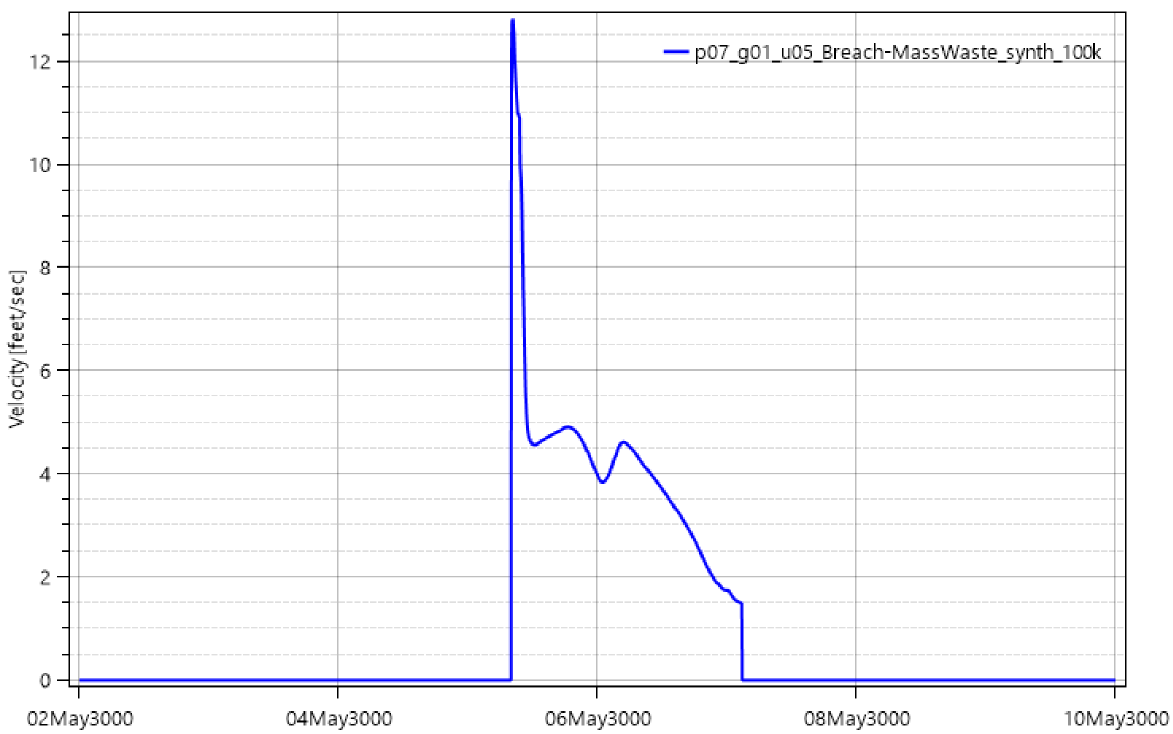

Figure 4-5. Plot of modeled velocity through the breach in the Scenario 2 mass-wasting failure.

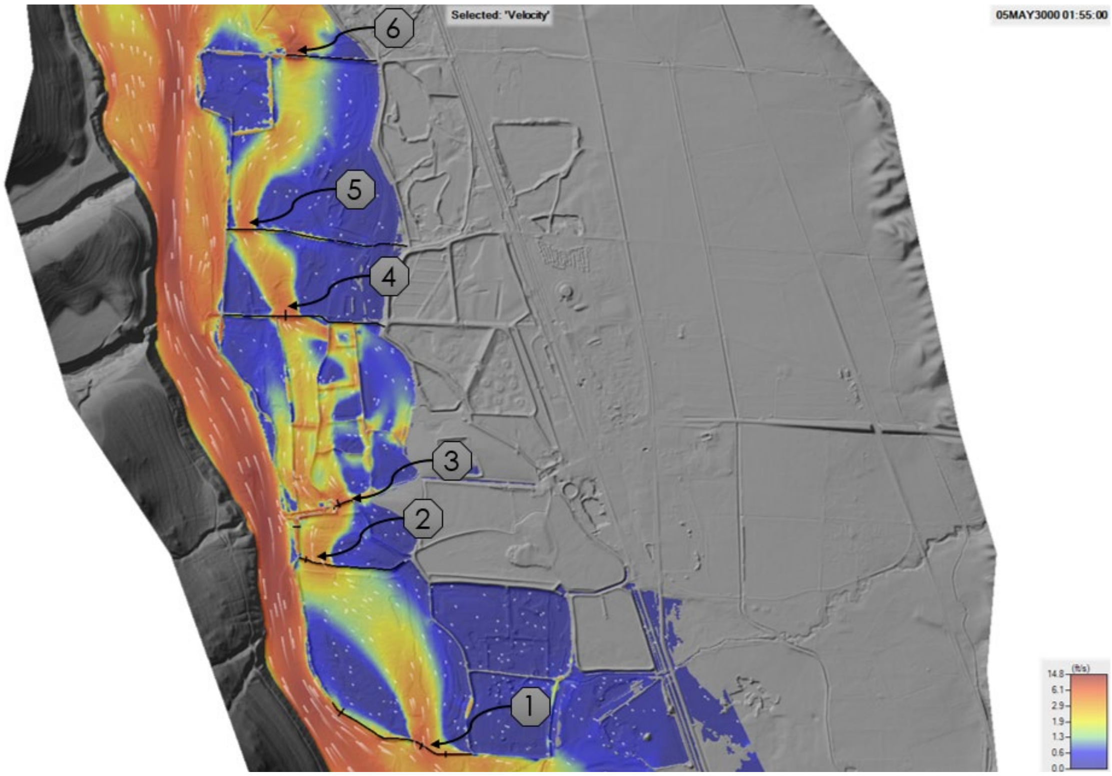

Figure 4-6. Plot of modeled velocities and particle traces at the peak of the Scenario 3 overtopping failure.

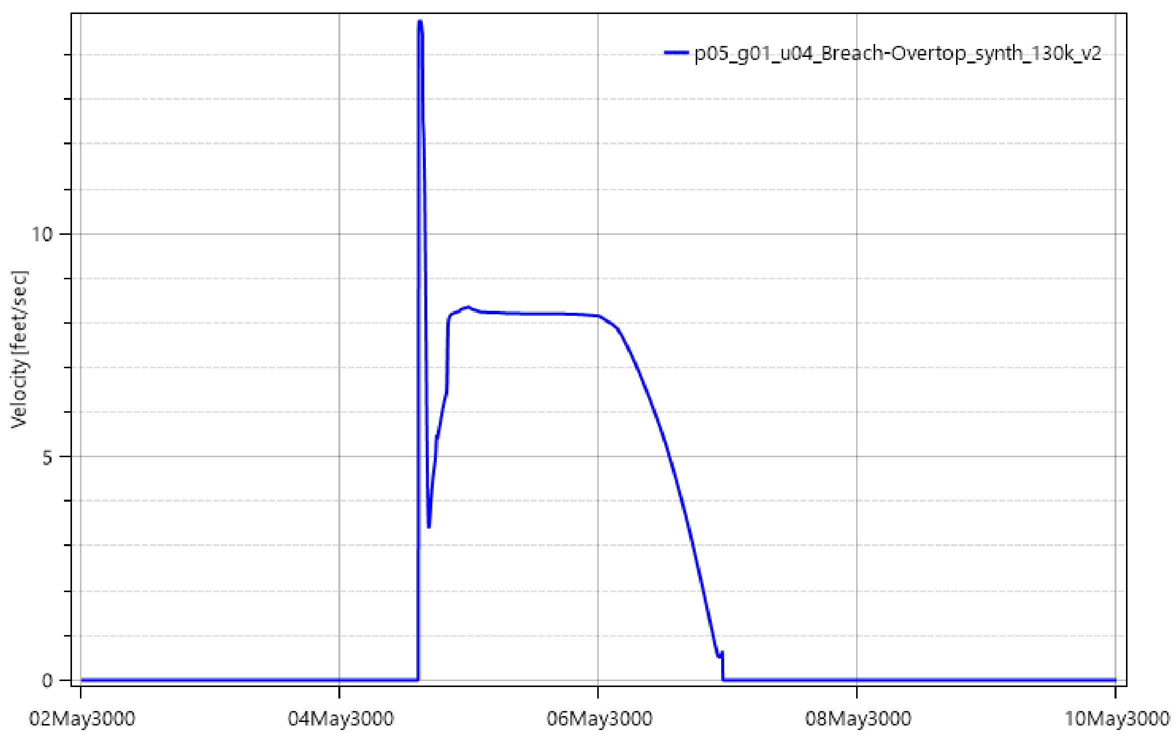

Figure 4-7. Plot of modeled velocity through the breach in the Scenario 3 overtopping failure.

Resulting flows through the berms into and out of the Site for Scenario 3, overtopping failure, are summarized in **Table 4-3**.

Table 4-3. Scenario 3 – overtopping failure modeling results.

Breach Location	Peak Flow (cfs)	Volume (ac-ft)
1	22,531	57,130
2	22,896	56,090
3	15,921	45,831
4	18,447	49,337
5	20,112	49,171
6	14,504	41,095

4.6 Berm Failure Modeling Limitations and Recommendations for Future Analysis

Berm failure modeling results are sensitive to estimated breach parameters including breach width and breach development time as noted in the preceding section. Other parameters that may influence results include peak flow magnitude, hydrograph shape, weir and piping flow coefficients, and antecedent soil moisture conditions, which may influence soil erodibility.

The extent of inundation caused by a breach in a levee or berm is highly influenced by the widening rate of the breach. An approach for calculating the widening rate function based on the soil erodibility and embankment height is presented in ERDC/GSL TR-22-8 (USACE 2022). The most accurate approach for using this method is to calculate site-specific widening rate curves based on estimates of local soil erodibility, which would require soil samples from berm to characterize soil erodibility. Alternatively, the default curve for non-cohesive soil presented in TR-22-8 would provide a suitable starting point for initial breach widening rate for use in the Simplified Breach Analysis Method implemented in HEC-RAS.

Comprehensive sensitivity analyses were not completed for this analysis. Multi-parameter sensitivity analyses should be completed as part of a rigorous engineering evaluation of the risk of berm failure. The sensitivity analysis should include determination of worst-case scenarios including combination of maximum breach width with minimum breach development time to estimate the maximum flow through the site. Minimum breach width should be paired with maximum breach development time to estimate the minimum flow through the site.

Potential for avulsions to occur in response to changes in channel upstream or across the river from the site were not evaluated in this study. Avulsions could occur in response to events such as a landslide originating along the road on the south side of the river or a blowout of a stream such as Cyr Creek. While this type of event was not modeled, an avulsion could result in higher velocities in the areas adjacent to the waste dumps. These waste dumps are only protected by comparatively smaller gravel berms, and waste has been buried below flood elevation and even below peak seasonal groundwater elevation. An avulsion event could have greater impacts on the dumps than the mass wasting event described in Scenario 2.

4.7 Data Gaps

Based on review of available studies and reports, the following data gaps have been identified:

- Berm composition and construction data
- Geotechnical (structural stability) assessment
- Groundwater and seepage assessment
- Breach modeling sensitivity analyses
- Breach flood progression mapping

4.8 Contaminant Mapping

Flow paths related to berm failure scenarios were considered in estimating the fate of contaminated materials that would be transported out of the Site under the berm failure scenarios. Contaminants are mixed with fine-grained soil materials in the holding ponds and waste dumps. The fine-grained materials in the unregulated waste dumps present the greatest threats. This is particularly true for the sludge ponds, which contain not only sludge but also fly ash from the Mill's boilers and other industrial process wastes. The initial EPA site assessment indicated that the berms containing the sludge ponds had already been intentionally breached to allow draining of the sludge ponds after mill closure. This amplifies the concern over potential fine grained material erosion from the site. Some of the waste types identified in the initial EPA site assessment (EPA 2013) that have been detected include, but are not limited to:

- 2,3,7,8-Tetrachlorodibenzo-p-dioxin (2,3,7,8-TCDD);
- 1,2,3,7,8-Pentachlorodibenzo-pdioxin (1,2,3,7,8-PeCDD);
- 1,2,3,6,7,8-Hexachlorodibenzo-p-dioxin (1,2,3,6,7,8-HxCDD);
- 1,2,3,4,6,7,8Heptachlorodibenzo-p-dioxin (1,2,3,4,6,7,8-HpCDD);
- Total Tetrachlorodibenzo-p-dioxin (Total TCDD);
- 2,3,7,8-tetrachlorodibenzofuran (2,3,7,8-TCDF);
- 2,3,4,7,8-Pentachlorodibenzofuran (2,3,4,7,8-PeCDF); and,
- metals including arsenic, cadmium, lead, and manganese.

It is assumed that contaminated materials released from the Site due to failure of the berm(s) would result in widespread dispersal downstream of the Site. Given the fine-grained nature of materials in the holding ponds and waste dumps, it is likely that contaminants would be well mixed in the water leaving the site. The contaminants would likely be dispersed across the floodplain downstream of the Site and deposited in low-velocity areas such as Frenchtown Slough. Contaminants that entered the main flow of the river would likely be transported further downstream, through Alberton Gorge and possibly as far as Thompson Falls Reservoir or Lake Pend Oreille. A map showing potential deposition areas is included in Appendix C.

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APPENDIX A – HYDROLOGIC ANALYSIS DATA

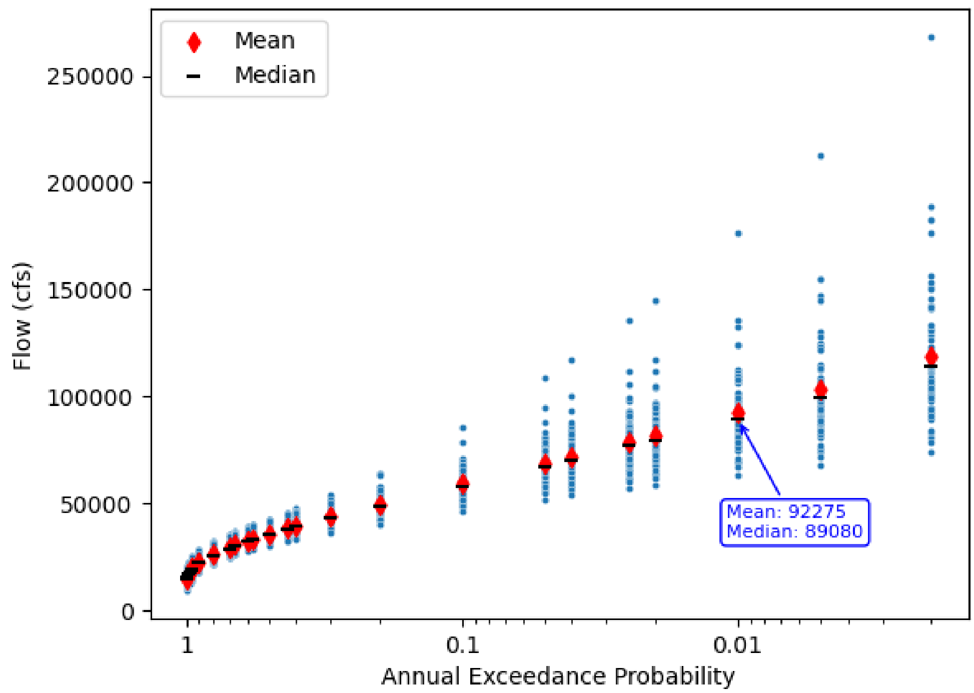

Figure A-1. Distribution of flood frequency quantiles for the CFR downstream of Missoula developed using modeled future flow data assuming a mid-range greenhouse gas scenario (RCP 4.5).

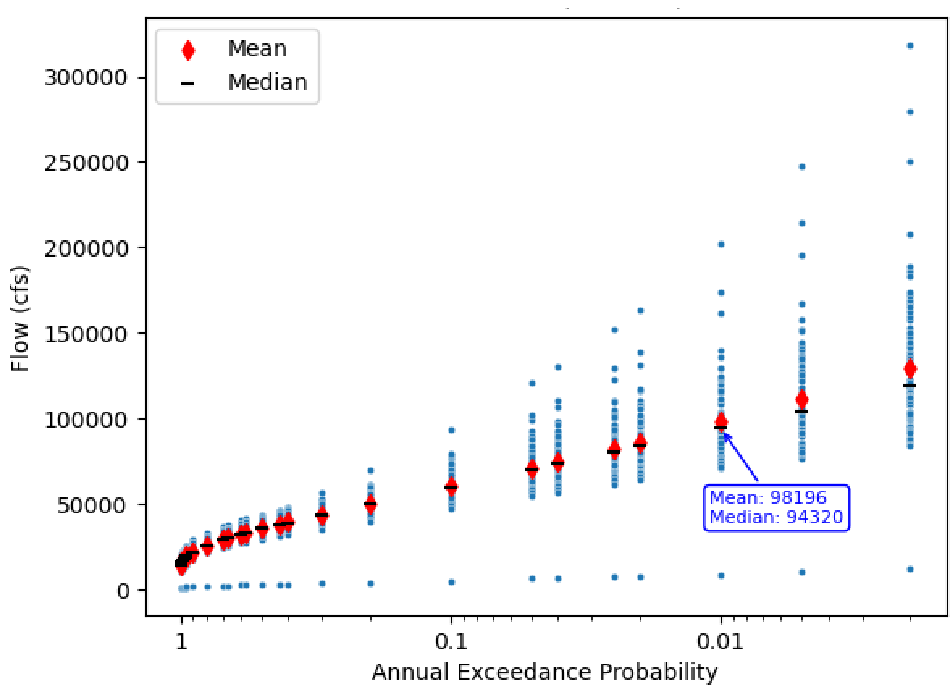

Figure A-2. Distribution of flood frequency quantiles for the CFR downstream of Missoula developed using modeled future flow data assuming a very high greenhouse gas scenario (RCP 8.5).

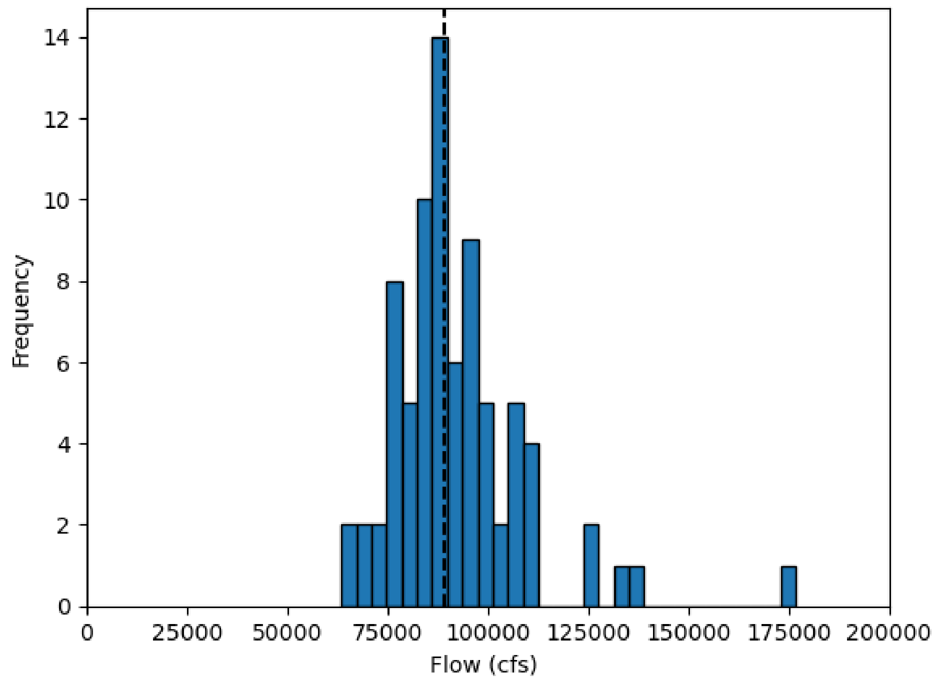

Figure A-3. Histogram of 1% annual chance (100-year) flows for the CFR downstream of Missoula developed using modeled future flow data assuming a mid-range greenhouse gas scenario (RCP 4.5).

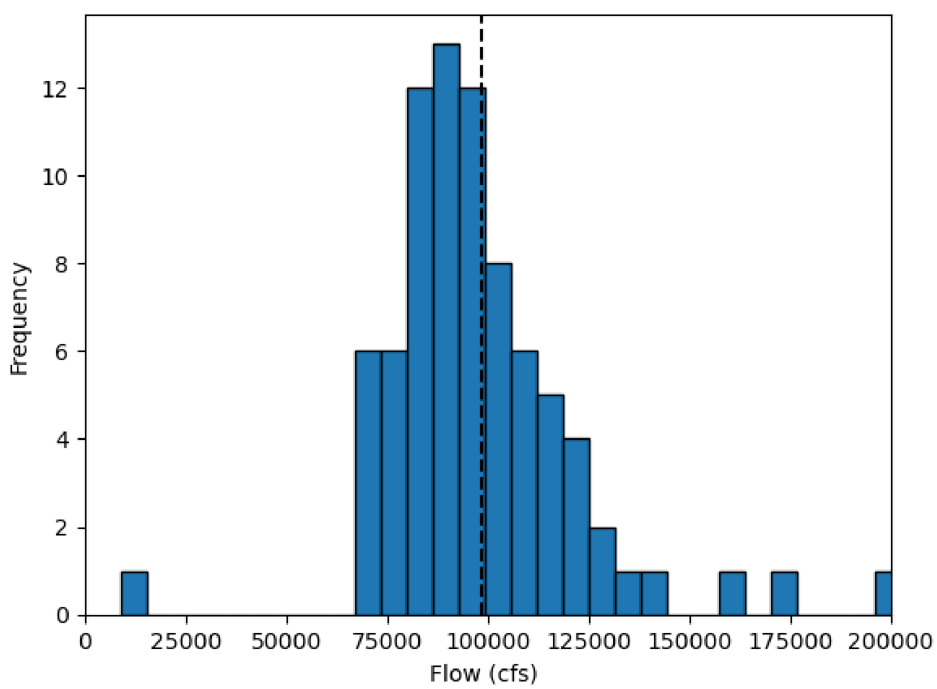

Figure A-4. Histogram of 1% annual chance (100-year) flows for the CFR downstream of Missoula developed using modeled future flow data assuming a very high greenhouse gas scenario (RCP 8.5).

APPENDIX B – HYDRAULIC MODEL OUTPUT

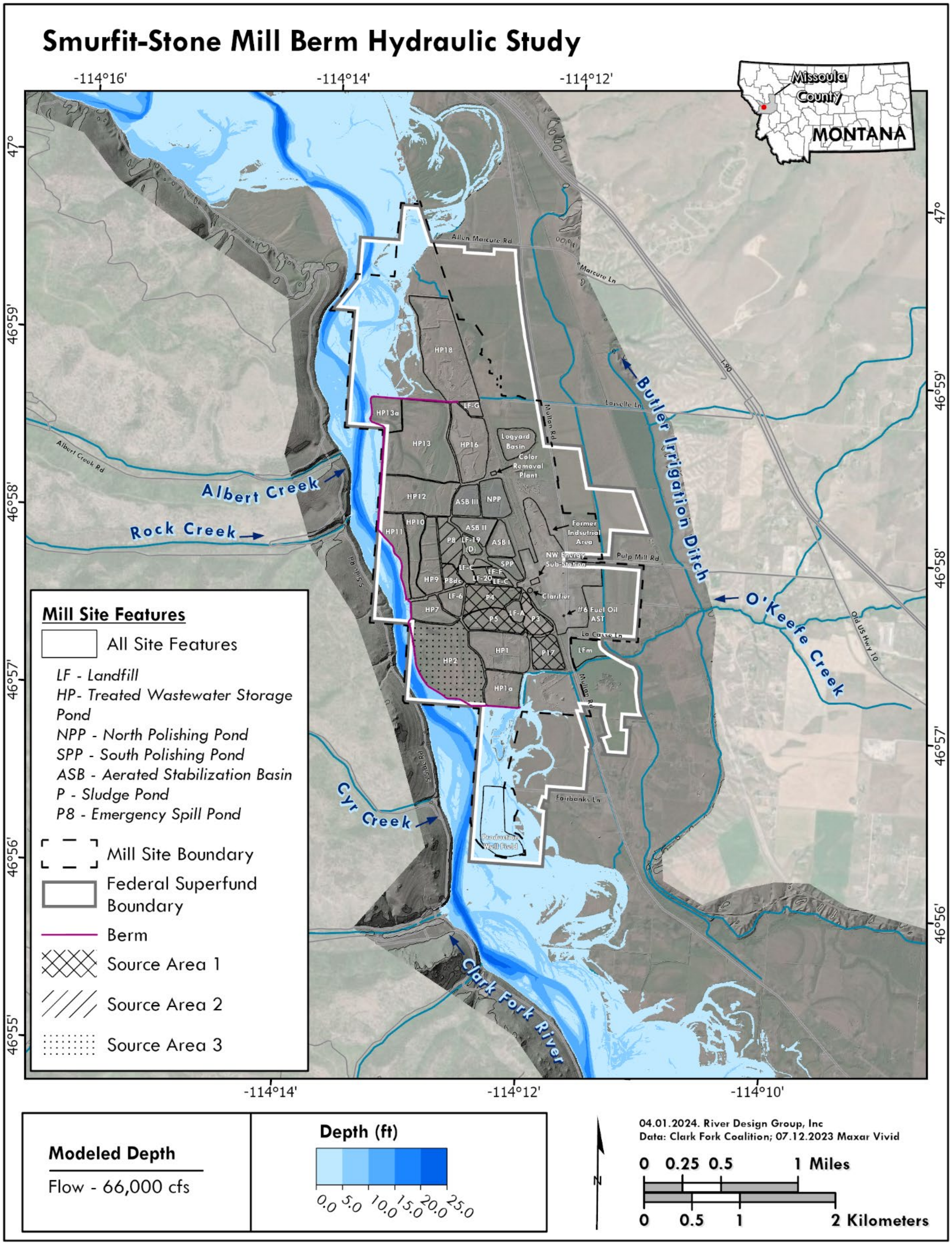

Figure B-1. Depth map for historical 100-year flood event of 66,000 cfs.

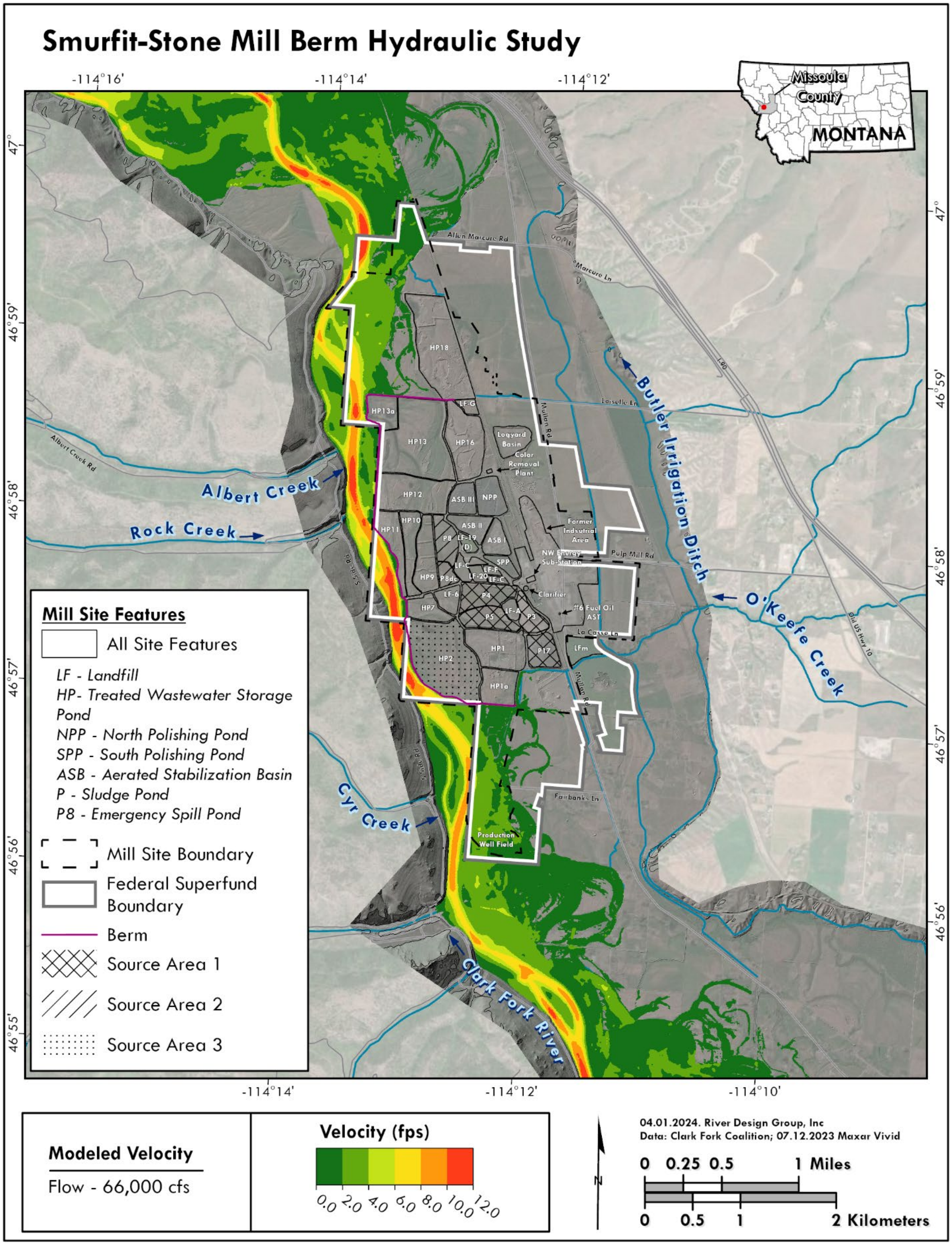

Figure B-2. Velocity map for historical 100-year flood event of 66,000 cfs.

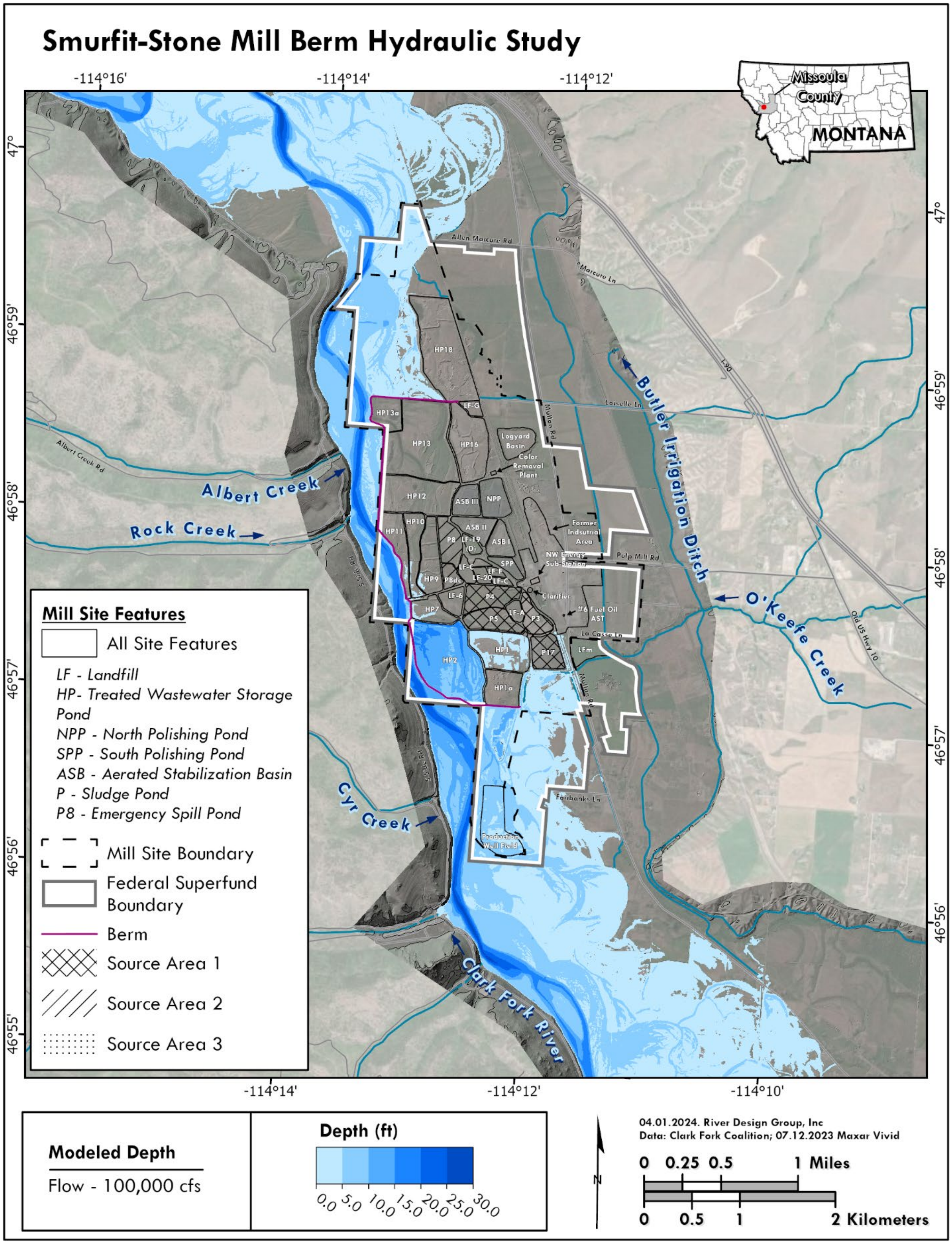

Figure B-3. Depth map for future 100-year flood event of ~100,000 cfs.

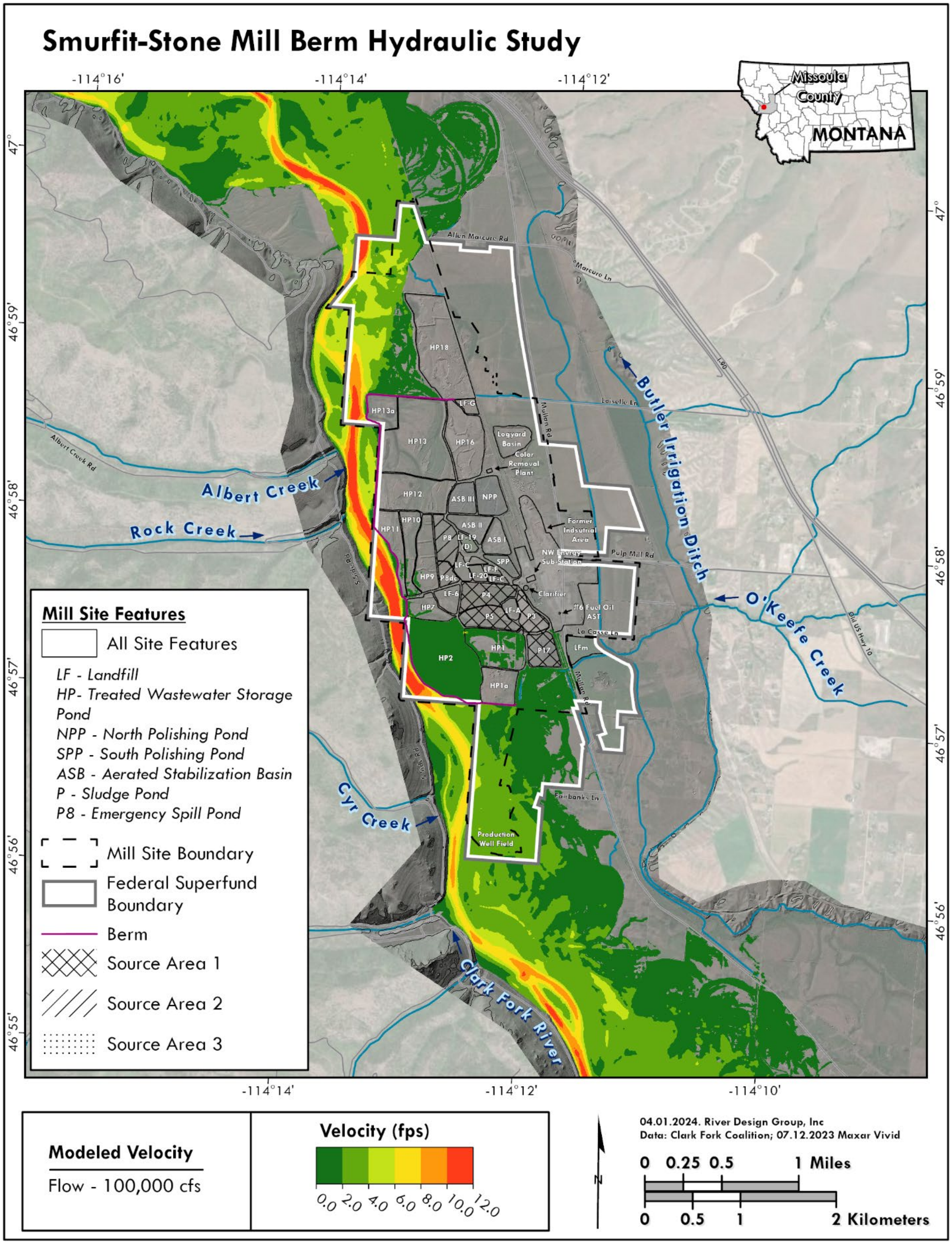

Figure B-4. Velocity map for future 100-year flood event of ~100,000 cfs.

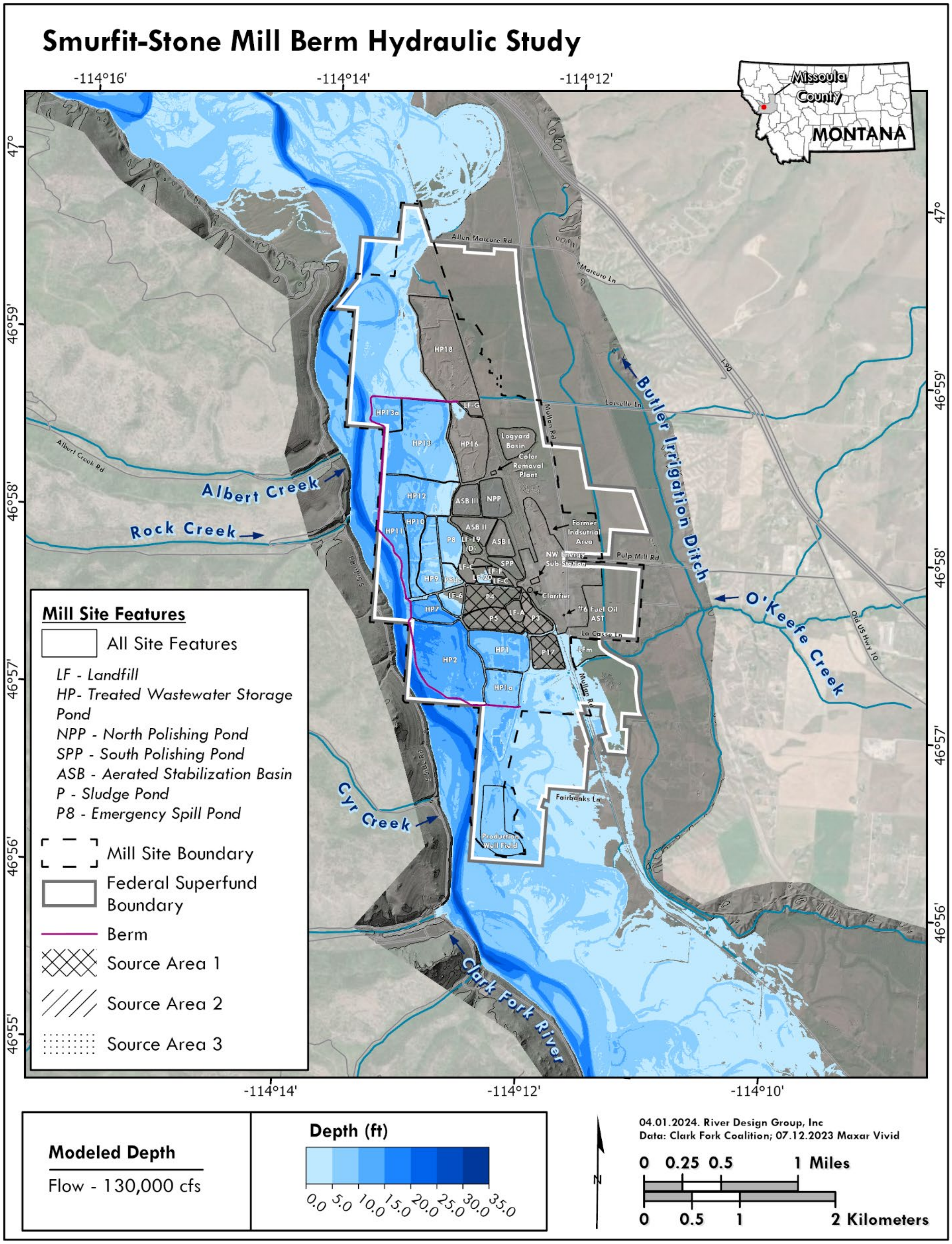

Figure B-5. Depth map for future 500-year flood event of ~130,000 cfs.

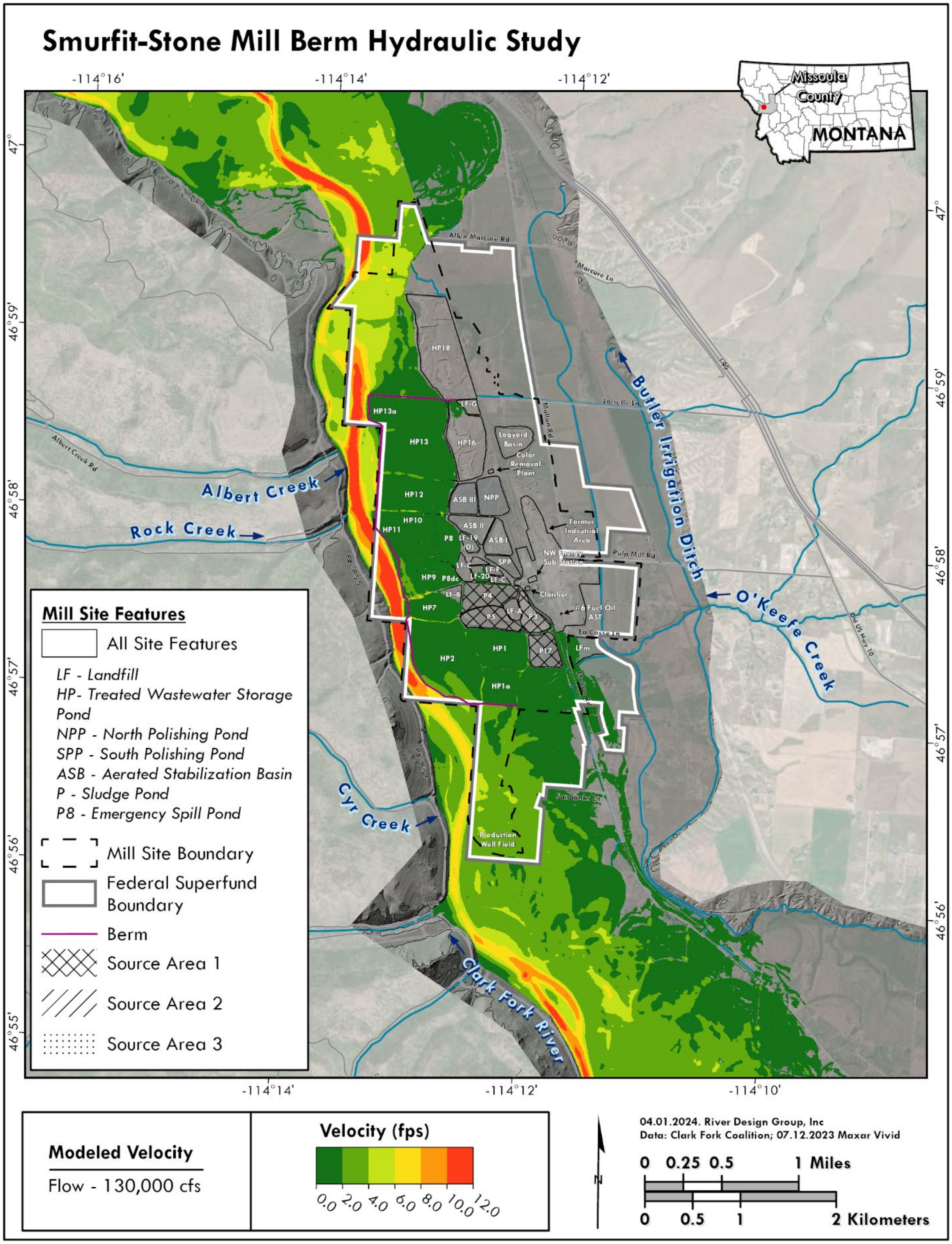

Figure B-6. Velocity map for future 500-year flood event of ~130,000 cfs.

APPENDIX C – BERM FAILURE ANALYSIS DATA

Table C-1. Breach parameters used for berm failure analysis.

Scenario #	**Scenario 1**	**Scenario 2**	**Scenario 3**					
Breach #	**1**	**1**	**1**	**2**	**3**	**4**	**5**	**6**
SA Connection name	SA2D Conn 1	SA2D Conn 1	SA2D Conn 1	SA2D Conn 2	SA2D Conn 3	SA2D Conn 4	SA2D Conn 5	SA2D Conn 6
Breach Center Station	150	200	800	250	120	925	300	130
Berm Top Elevation	3058.1	3059.6	3058.6	3058.9	3057.5	3053.1	3049.9	3049.4
Failure WSEL	**3043.6**	**3060.1**	**3059.1**	**3059.4**	**3058**	**3053.6**	**3050.4**	**3049.9**
Downstream Toe Elev.	3042.1	3047.5	3050.2	3042.2	3045.4	3039.4	3038.7	3040.2
Final Bottom Elev.	**3042.6**	**3048**	**3050.7**	**3042.7**	**3045.9**	**3039.9**	**3039.2**	**3040.7**
Berm Height (ft)	16	12.1	8.4	16.7	12.1	13.7	11.2	9.2
Hl_m = Hl_ft / 3.28	4.9	3.7	2.6	5.1	3.7	4.2	3.4	2.8
Breach Width (m)								
Wb_m = 3.5 * (Hl_m + 1.5) ** 2.0	142	94	58	152	94	113	85	65
Wb_max_m = 3.5 * (Hl_m + 2) ** 2.32	307	198	118	329	198	239	176	134
Wb_min_m = 1.5 * (Hl_m + 0.5) ** 1.9	37	23	13	39	23	28	20	15
Breach Width (ft)								
Wb_ft = Wb_m * 3.28	**467**	**309**	**189**	**499**	**309**	**370**	**277**	**213**
Wb_max_ft = Wb_max_m * 3.28	**1007**	**648**	**388**	**1081**	**648**	**784**	**578**	**438**
Wb_min_ft = Wb_min_m * 3.28	**120**	**75**	**41**	**129**	**75**	**92**	**66**	**48**
Lateral Erosion Rate (m/hr)								
LE_m_hr = 15.0 + 10.0 * Hl_m	63.8	51.9	40.6	65.9	51.9	56.8	49.1	43.0
LE_max_m_hr = 54.7 + 23.4 * Hl_m	168.8	141.0	114.6	173.8	141.0	152.4	134.6	120.3
LE_min_m_hr = 12 + 2.7 * Hl_m	25.2	22.0	18.9	25.7	22.0	23.3	21.2	19.6
Lateral Erosion Rate (ft/hr)								
LE_ft_hr = LE_m_hr * 3.28	209	170	133	216	170	186	161	141
LE_max_ft_hr = LE_max_m_hr * 3.28	554	463	376	570	463	500	441	395
LE_min_ft_hr = LE_min_m_hr * 3.28	83	72	62	84	72	76	70	64
Breach Development Time (hr)								
Design	**2.2**	**1.8**	**1.4**	**2.3**	**1.8**	**2.0**	**1.7**	**1.5**
Max	**12.2**	**9.0**	**6.3**	**12.8**	**9.0**	**10.3**	**8.3**	**6.8**
Min	**0.2**	**0.2**	**0.1**	**0.2**	**0.2**	**0.2**	**0.1**	**0.1**

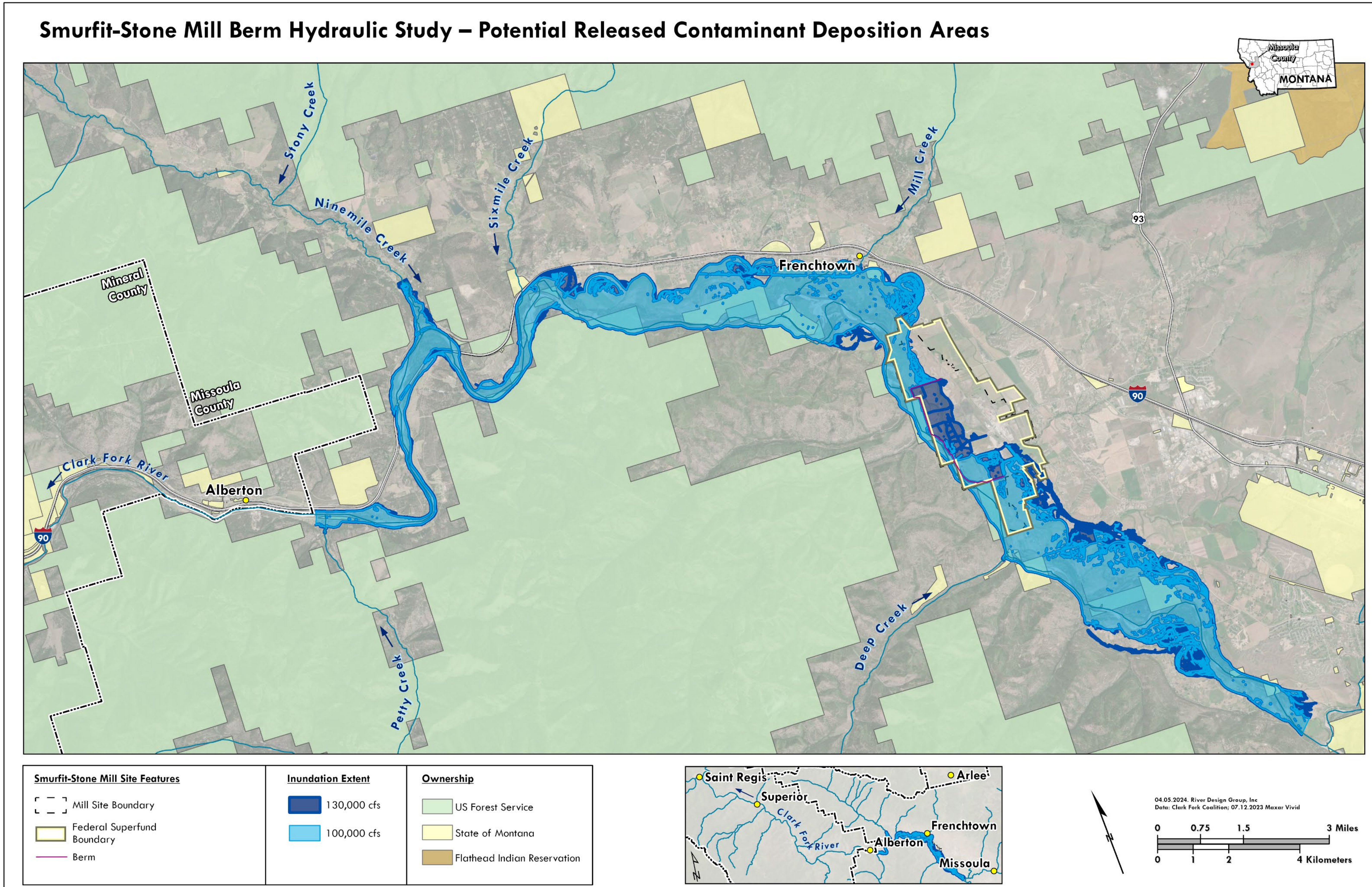

Figure C-1. Potential released contaminant deposition area map.

RDG

RIVER DESIGN GROUP