US Army Corps of Engineers®
St. Paul District

Appendix E: Hydraulics & Hydrology

Kinnickinnic River Continuing Authorities Program Section 206 Feasibility Report

May 2025

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Appendix E

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1 Project Summary

For the feasibility study, seven alternatives were analyzed. Alternative 1 is no change from the existing conditions. Alternative 2 is to remove only Junction Falls Dam and rehabilitate Lake George and the upstream area, leaving Powell Falls Dam and the drained Lake Louise area as is. Alternative 3 is to remove only Powell Falls Dam and rehabilitate the drained Lake Louise area, downstream of Junction Falls Dam. Junction Falls Dam would remain in place. Alternative 4 is to remove both dams and rehabilitate both lake areas. The hydraulic modelling includes runs for the existing conditions, alternative 2, and alternative 3. Due to time constraints, the cost and benefits for alternative 4 were assumed to be the combined results of alternative 2 and alternative 3. This is reasonable due to the waterfall separating the two areas. Alternative 5 is alternative 2 with the addition of Spring Ponds restoration, alternative 6 is alternative 3 with the addition of spring ponds restoration, and alternative 7 is alternative 4 with the addition of spring ponds restoration.

2 Watershed Description

2.1 General Information

The Kinnickinnic River watershed (Figure 1) lies in southwestern Wisconsin in the St. Croix River basin and drains approximately 172 square miles across Pierce and St. Croix Counties. The watershed encompasses the entirety of the USGS's Hydrologic Unit Code (HUC)-10 watershed 0703000511 (Reference 1). The watershed is located 30 miles east of the Minneapolis-St. Paul Metro area, just east of the Minnesota-Wisconsin border. The watershed is dominated by agriculture (57%), grassland (22%), and forest (17%), with approximately 2% of the watershed consisting of wetlands and lakes (Reference 2). The Kinnickinnic River begins its journey as the culmination of flows from several intermittent, spring-fed streams, approximately 16 miles northeast of River Falls, WI. The Kinnickinnic then flows 26 miles southwest, through the center of River Falls, discharging as the last major tributary to the St. Croix River at Kinnickinnic State Park, approximately halfway between Prescott, WI and Hudson, WI. The average slope of the Kinnickinnic is approximately 10 feet/mile with middle portions of the river being flatter. Elevations in the watershed vary from 1,205 feet NAVD 88, in the upper portions of the watershed, to 680 feet NAVD 88 at its confluence with the St. Croix.

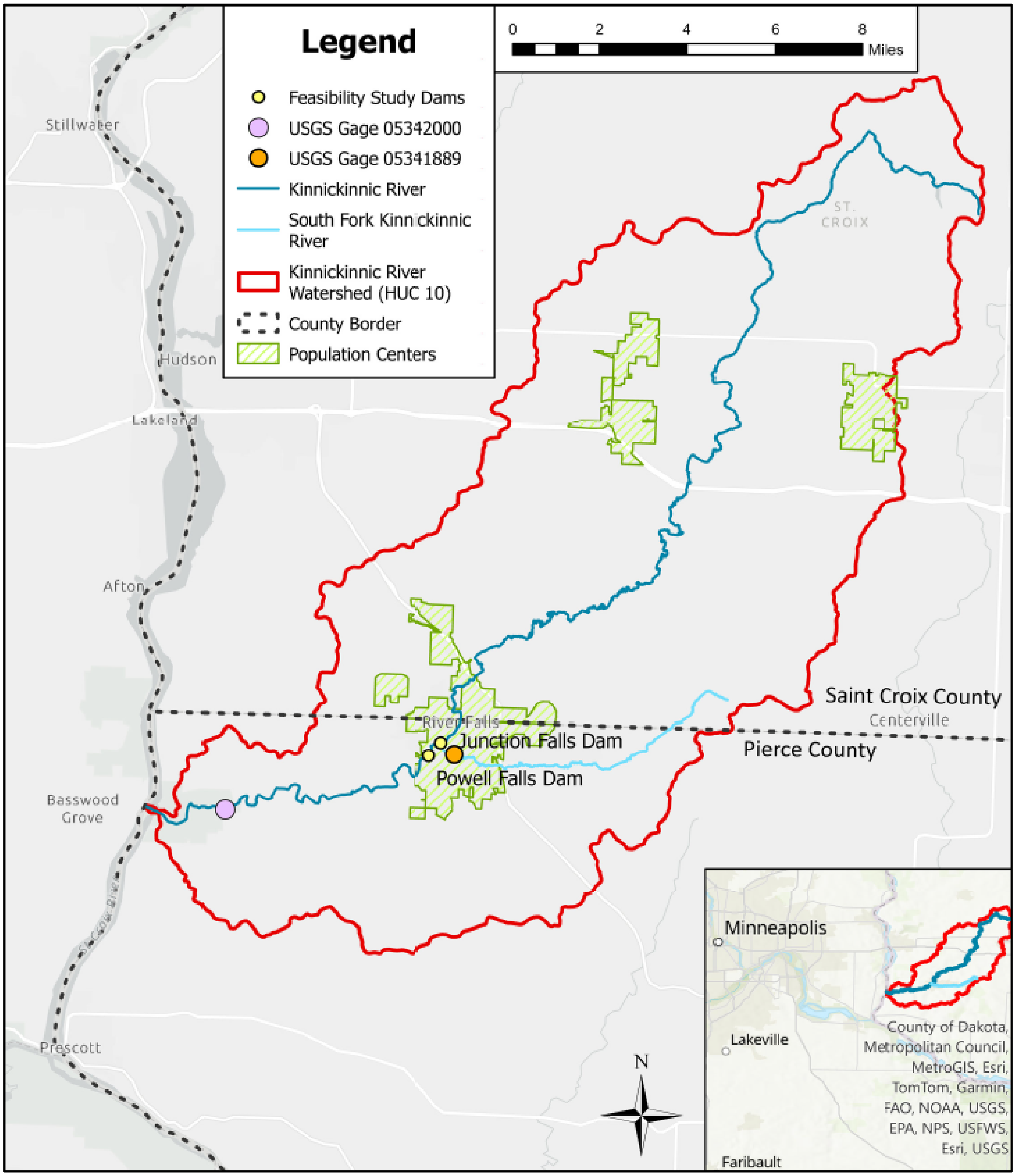

Figure 1. Kinnickinnic River watershed

2.2 Geodesy

The North American Vertical Datum of 1988 (NAVD 88) was used as the vertical datum throughout the study. The North American Datum of 1983 (NAD 83) was used throughout the study as the horizontal datum, with NAD 1983 High Accuracy Referenced Network (HARN) Wisconsin Transverse Mercator (TM) used as the projection for mapping and calculating areas.

2.3 Geomorphological Setting

The Kinnickinnic River watershed is located in a unique ecoregion for the state of Wisconsin. The watershed is in the small portion of Wisconsin which is designated as the Environmental Protection Agency's Level IV ecoregion 47g. Ecoregion 47g is the Prairie Pothole Region, or the Lower St. Croix and Vermillion Valleys of the Western Corn Belt Plains (Reference 3). This ecoregion is characterized by,

> "Smooth to undulating topography, productive prairie soils, and loess- and till-capped dolomite bedrock. The potential natural vegetation is predominantly tall grass prairie with a gradual transition eastward to more mixed hardwoods, distinguishing 47g from the greater concentration of mixed hardwoods of both 51a to the north and 51b to the east, and the mixed prairie and oak savanna of 52b to the south" (Reference 3)

The geology of the Kinnickinnic River basin consists of loess and glacial till deposited as moraines during the Quaternary Period. The soil overlies Ordovician bedrock, with the depth to bedrock ranging from 0 to 50 feet. The glacial till consists of unstratified clay, silt, sand, gravel, and boulders. The uppermost bedrock units include Galena Dolomite, Decorah Shale, and Platteville Limestone ranging from 0 to 115 feet thick. Below these units lies St. Peter Sandstone, ranging in thickness from 0 to 200 feet (Reference 44).

2.4 Climatological Setting

Most of the Kinnickinnic River basin is located within the warm summer, humid continental Köppen climate type, characterized by warm summers with ample rainfall, and cold to frigid winters with moderate snowfall, (Reference 5). Average monthly temperatures in River Falls, WI vary from a minimum of 13.2 degrees Fahrenheit (°F) in January to a maximum of 69.9 °F in July. Average monthly precipitation ranges from a minimum of 0.85 inches in February to a maximum of 5.03 inches in June, (Reference 6). Figure 2 shows a climatograph depicting typical monthly temperatures and average cumulative precipitation depths for River Falls. Precipitation that falls during the months of November through March is typically snow. However, snow has the potential to begin accumulating as early as October and fall as late as April. The closest snow recording station to River Falls is located in Baldwin, WI, (USC00470486), where average annual snowfall is 42.2 inches, (Reference 7).

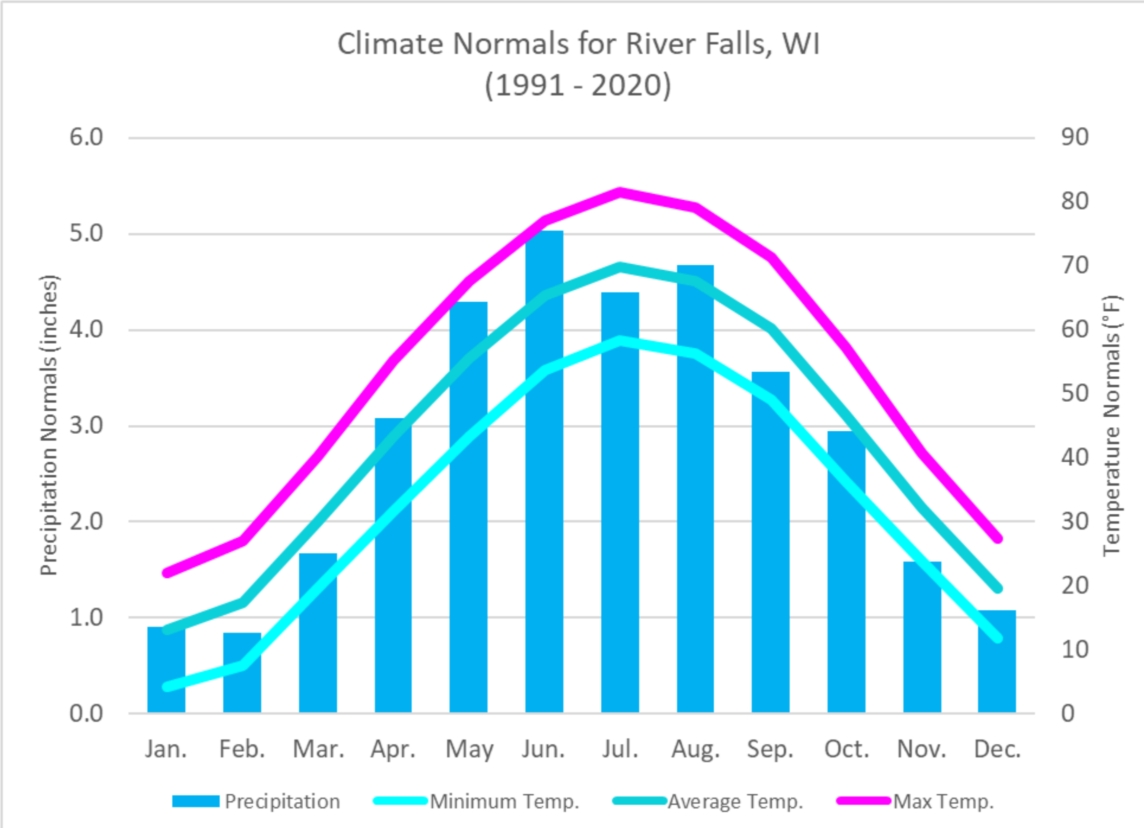

Figure 2. Climate Normals for River Falls, WI (Reference 7)

2.5 Flooding

Potential drivers of flooding on the Kinnickinnic River are intense rainfall brought by thunderstorms during the summer months, rapid melting of snowpack during the spring, and rain on snow during the spring (Reference 8).

Peak annual flows measured at the Kinnickinnic River near River Falls, WI gage (USGS 05342000) are shown in Table 1. The maximum flow recorded at that gage was 6,450 cfs on June 29, 2020. This flood was a result of 6.5+ inches of rain across the watershed. There were also floods in 1894, 1934, and 1965 which are not captured by the USGS gage's record. Flow estimates for these floods were made in a study at the University of Wisconsin – River Falls and are shown in Table 2 (Reference 12).

Floods at this gage are typically of short duration, with peak flows occurring over the course of one to three days. It is not uncommon to have multiple flood events per year.

The USACE's 2021 River Falls Hydroelectric Project Hydraulic and Hydrologic Analysis through the Planning Assistance to States (PAS) program, goes into further detail about the history and driving factors of floods on the Kinnickinnic River. (Reference 8).

Table 1. Annual peak streamflow measurements for all water years for Kinnickinnic River near River Falls (USGS 05342000) (Reference 10)

Water Year	Date	Gage Height (feet)	Streamflow (cfs)
1917	27-Mar-1917	5.67	1,970
1918	5-Jun-1918	6.6	3,080
1919	12-Mar-1919	7	3,560
1920	15-Mar-1920	7.98	4,760
1921	14-Jun-1921	5.35	410
2002	21-Aug-2002	12.53	774
2003	25-Jun-2003	15.05	2,130
2004	2-Mar-2004	11.31	516
2005	6-Mar-2005	12.84	1,120
2006	30-Mar-2006	12.06	842
2007	13-Mar-2007	15.21	2,490
2008	14-Mar-2008	10.92	360
2009	8-Aug-2009	13.69	1,560
2010	11-Aug-2010	17.98	4,340
2011	22-Jun-2011	12.56	997
2012	20-Jun-2012	12.23	718
2013	26-Jun-2013	13.37	1,380
2014	1-Jun-2014	15.68	2,180
2015	6-Jul-2015	17.84	3,100
2016	5-Jul-2016	12.69	909
2017	18-May-2017	13.32	1,230
2018	24-Aug-2018	12.24	745
2019	17-Apr-2019	18.02	3,160
2020	29-Jun-2020	21	6,450
2021	12-Oct-2020	10.24	344
2022	12-May-2022	12.43	886
2023	1-Apr-2023	9.7	280

Table 2. Historic flood estimates for the Kinnickinnic River at River Falls, WI (Reference 12)

Date	1976 Report AEP Estimates (%)	Discharge Estimate at County Road MM (cfs)
15-May-1894	<0.5	8,900 – 9,900
5-Apr-1934	1	8,600
1-Jun-1965	2	7200

2.6 Hydrologic Model Objectives

The purpose of the hydrologic model is to use watershed characteristics and historic data to create a representation of the Kinnickinnic River Basin. The model generated flows for 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, and 0.2% flow events for the basin. These flows were used as inputs to the HEC-RAS model. An HEC-HMS model was developed to represent the flow inputs for each subbasin and reach. The HEC-RAS model used the flows to determine with and without project conditions in the project area.

The hydrologic and hydraulic analyses will inform the project's study of the river corridor. The project's goal is to study the impacts of the removal Junction Dam and Powell Dam, which create Lake Louise and Lake George, respectively. Alternatives for this study are removal of both dams, removal of Junction Falls Dam, removal of Powell Falls Dam, and without project conditions, meaning both dams remain. The HEC-RAS included all alternatives and flow events. The HEC-RAS outputs inform the selection of the alternative. The restoration of the Spring Ponds area was not included in the hydrologic and hydraulic modeling work.

3 Hydrologic Data

3.1 Previous USACE Hydrologic Efforts

The USACE's River Falls Hydroelectric PAS Project in 2021 included updating flow-frequency curves on the Kinnickinnic River (Reference 8).

The study also attempted to extend the period of record using Bulletin 17C record extension methods. The analysis was conducted in HEC-SSP and correlates a stream gage that has a short period of record or gap in data with a hydrologically similar gage that is longer. The study's analysis included more than thirteen stream gages nearby the Kinnickinnic River with sufficient overlapping period of records. The results showed that none of those gages correlated strongly enough with the Kinnickinnic River near River Falls, WI to justify a record extension.

The flow frequency curve for the Kinnickinnic River near River Falls, WI Gage USGS gage, 04342000 using was updated using Bulletin 17C guidelines. Bulletin 17C accounts for systematic streamflow data, in this case spanning 1917 – 1921, 2002 – 2020 and historical flood record data. By incorporating historic flood data, a frequency curve was generated which included perception thresholds dating back to 1854. That flow frequency curve was then transposed to areas of interest on the Kinnickinnic River.

Flow frequencies were also estimated using USGS Regression Equations for Wisconsin, which had been updated in 2020. These equations relate basin characteristics to flood frequency. The results of this analysis produced flow frequency curves with a high amount of uncertainty.

In addition to the flow frequency work, basin delineations were done in ArcGIS pro and a soil and land use analysis was to determine hydraulic conductivity of saturated soils in the basin.

The hydrologic work done in this study builds off the work done in the 2021 study. The 2021 study was used to establish existing conditions for the HEC-HMS model, provided valuable background information on the characteristics of the Kinnickinnic watershed, and was used to verify flows generated by the model.

3.2 Streamflow

There is one active streamflow gage in the Kinnickinnic River basin. The active gage used to calibrate the hydrologic model is the Kinnickinnic River near River Falls, WI USGS Gage (USGS 05342000). Table 3 describes the history of this gage. There is also an inactive stream gage on the South Fork Kinnickinnic River at River Falls USGS Gage (USGS 05341900) with peak annual streamflow reported for years 1959 – 2022. This record was used to assist in calibration of the subbasin encompassing the South Fork of the Kinnickinnic River. Stream gages in the Kinnickinnic River Basin are displayed in Figure 1.

Table 3: Available Data for the Kinnickinnic River Near River Falls, WI UGSS Gage (USGS 05342000)

Gage Data Type	Dates Available
15-minute instantaneous flow and stage	04 Dec1998 – present
Annual peak flow and stage	1917 – 1921 2002 – present
Daily average flow	01 Oct 1916 – 29 Sept 1921 01 Oct 1998 – 29 Sept 1999 01 Jul 2002 – present

All source gage data was downloaded in Universal Time Coordinated (UTC) to match gridded precipitation which is in the same time zone. The gridded precipitation uses Greenwich Mean Time (GMT) which is the same as UTC.

3.3 Dams/Reservoirs

The City of River Falls Municipal Utilities (RFMU) currently owns and operates two hydroelectric dams along the Kinnickinnic River in River Falls, WI. Known as Junction Falls and Powell Falls, these hydroelectric dams are licensed by the Federal Energy Regulatory Commission (FERC) under the River Falls Hydroelectric Project (License 10489). Both Junction Falls and Powell Falls are included in the National Inventory of Dams (NID) under NID IDs WI00021 and WI 00079, respectively (Reference 9). Junction Falls and Powell Falls Dams discussed in further detail in the "Study Area" section of the Main Report. Both are shown on the Figure 1.

No other dams are currently listed on the Kinnickinnic River in NID (Reference 9).

4 Meteorologic Data

4.1 Precipitation

Hourly, gridded precipitation data covering the study area is accessible via the Corps Water Management System (CWMS) internal website. The source data for the Kinnickinnic River hydrologic model is from the National Weather Service (NWS) North Central River Forecast Center (NCRFC). Gridded precipitation for the project area is continuous from 2008 to the present and reported in GMT.

There are a number of hourly, point precipitation gages in and around the Kinnickinnic River watershed. However, there is variation in the temporal distribution and type of meteorologic data between gages. Only daily precipitation gages are available, and their period of record is limited. Those gages were used to evaluate the accuracy of the gridded precipitation data. Where available, temperature and precipitation data were used to analyze the antecedent conditions that may affect model calibration. Seven gages in and near the Kinnickinnic watershed were used. These gages are listed in Table 4 and shown in Figure 3.

Table 4. Precipitation gages in and around the Kinnickinnic River Basin; used to verify gridded precipitation in HEC-HMS model

Map Number	Name	Network	Gage ID	Daily Record
1	RIVER FALLS, WI US	NOAA	GHCND:USC00477226	1918-2023
2	RIVER FALLS 0.4 E, WI US	NOAA	GNCND:US1WIPC0003	2009-2017
3	RIVER FALLS 1.4 S, WI US	NOAA	GHCND:US1WIPC0010	2013-2023
4	RIVER FALLS 1.2 SSW	NOAA	US1WIPC0005	2010-2023
5	HAMMOND 0.4 NE, WI US	NOAA	GHCND:US1WISC0009	2012-2022
6	STILLWATER 1.4 NE, MN USA	NOAA	US1MNWG0020	2011-2021
7	AFTON 1.6 E, MN US	NOAA	GHCND:US1MNWG0047	2016-2023

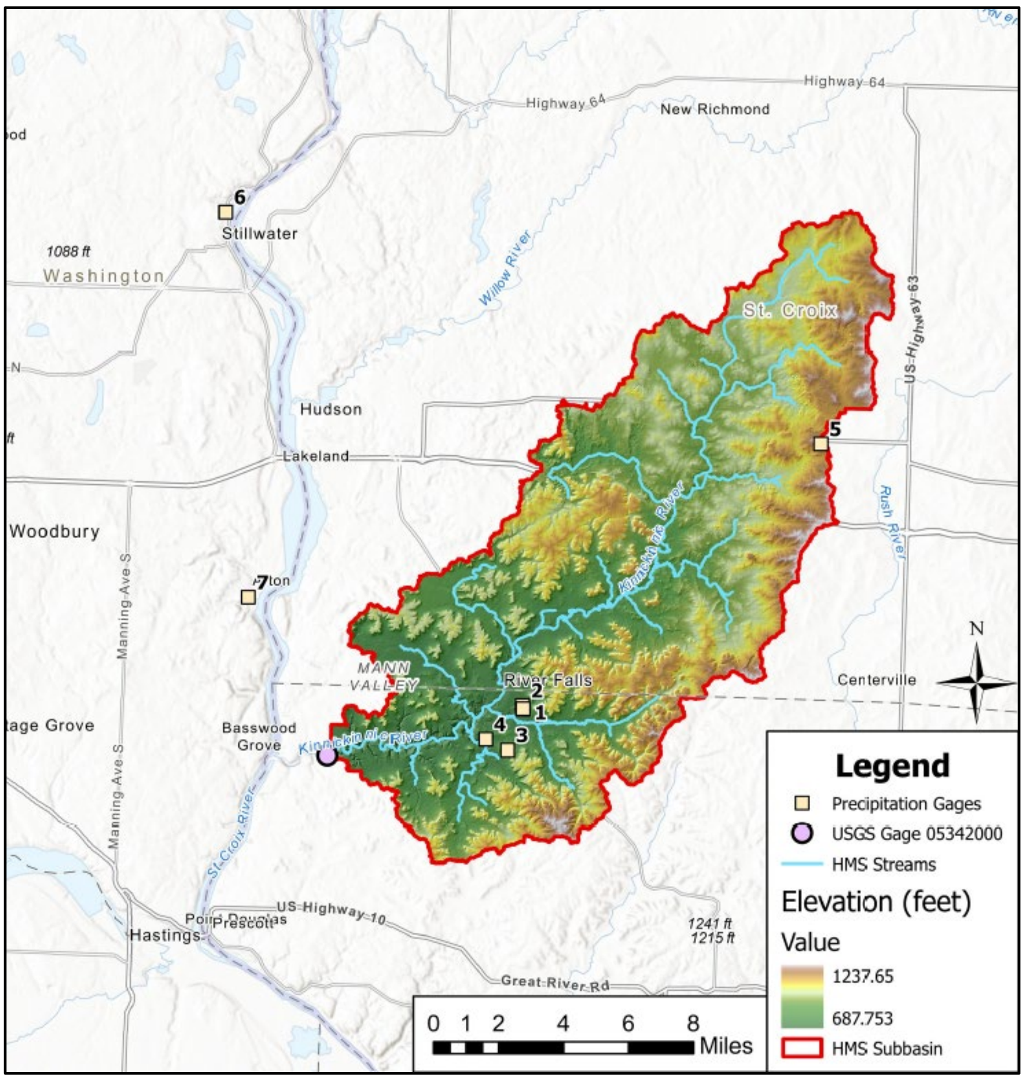

Figure 3.Precipitation gages in and near Kinnickinnic River Basin

Gage precipitation amounts and their comparisons to the gridded precipitation used to calibrate the HEC-HMS model are show in Table 5 – Table 10.

Table 5. Cumulative precipitation for the June 2020 peak flow event for gages in and near the Kinnickinnic River Basin

June 2020 Event				
Map Number	**Name**	**Daily Record** *=missing some data	**Gage Event Precipitation (in)**	**Closest HMS Subbasin**
1	RIVER FALLS, WI US	1918-2023	7.65	S-SouthFork
2	RIVER FALLS 0.4 E, WI US	2009-2017	-	S-SouthFork
3	RIVER FALLS 1.4 S, WI US	2013-2023	7.34	S-LowerKinni
4	RIVER FALLS 1.2 SSW	2010-2023	6.97	S-LowerKinni
5	HAMMOND 0.4 NE, WI US	2012-2022	8.20	S-UpperKinni
6	STILLWATER 1.4 NE, MN USA	2011-2021*	4.14	S-UpperKinni
7	AFTON 1.6 E, MN US	2016-2023	6.01	S-MiddleKinni
		Average	6.72	

Table 6. Precipitation totals used to model streamflow for June 2020 event in HEC-HMS

June 2020 Event		
HMS Subbasin	**Event Precipitation (in)**	**Percent Difference**
S-UpperKinni	7.43	20%
S-SouthFork	6.49	-15%
S-MiddleKinni	7.5	25%
S-LowerKinni	6.88	-4%
Average	7.08	5%

Table 7. Cumulative precipitation for the August 2010 peak flow event for gages in and near the Kinnickinnic River Basin

August 2010 Event				
Map Number	**Name**	**Daily Record**	**Gage Event Precipitation (in)**	**Closest HMS Subbasin**
1	RIVER FALLS, WI US	1918-2023	5.41	S-SouthFork
2	RIVER FALLS 0.4 E, WI US	2009-2017	-	S-SouthFork
3	RIVER FALLS 1.4 S, WI US	2013-2023	-	S-LowerKinni
4	RIVER FALLS 1.2 SSW	2010-2023	4.37	S-LowerKinni
5	HAMMOND 0.4 NE, WI US	2012-2022	-	S-UpperKinni
6	STILLWATER 1.4 NE, MN USA	2011-2021	-	S-UpperKinni
7	AFTON 1.6 E, MN US	2016-2023	-	S-MiddleKinni
		Average	4.89	

Table 8. Precipitation totals used to model streamflow for August 2010 event in HEC-HMS

August 2010 Event		
HMS Subbasin	**Event Precipitation (in)**	**Percent Difference**
S-UpperKinni	4.25	-
S-SouthFork	5.09	-6%
S-MiddleKinni	4.33	-
S-LowerKinni	4.68	7%
Average	4.59	-6%

Table 9. Cumulative precipitation for the May 2017 peak flow event for gages in and near the Kinnickinnic River Basin

May 2017 Event				
Map Number	**Name**	**Daily Record** *=missing some data	**Gage Event Precipitation (in)**	**Closest HMS Subbasin**
1	RIVER FALLS, WI US	1918-2023	5.25	S-SouthFork
2	RIVER FALLS 0.4 E, WI US	2009-2017	4.45	S-SouthFork
3	RIVER FALLS 1.4 S, WI US	2013-2023*	3.22	S-LowerKinni
4	RIVER FALLS 1.2 SSW	2010-2023*	4.4	S-LowerKinni
5	HAMMOND 0.4 NE, WI US	2012-2022	5.7	S-UpperKinni
6	STILLWATER 1.4 NE, MN USA	2011-2021	4.5	S-UpperKinni
7	AFTON 1.6 E, MN US	2016-2023*	4.18	S-MiddleKinni
		Average	4.53	

Table 10. Precipitation totals used to model streamflow for May 2017 event in HEC-HMS

May 2017 Event		
HMS Subbasin	**Event Precipitation (in)**	**Percent Difference**
S-UpperKinni	4.60	-10%
S-SouthFork	4.22	-13%
S-MiddleKinni	4.91	17%
S-LowerKinni	4.74	24%
Average	4.62	2%

5 HEC-HMS Watershed Delineation

The digital elevation model (DEM) of the Kinnickinnic watershed was prepared using ArcGIS Pro 3.0.2 and the GIS tools built into HEC-HMS 4.10 were used to create the geospatially-linked stream network, subbasins, and grid cell file necessary to run the HEC-HMS model.

5.1 DEM Processing

HEC-HMS uses a DEM of the watershed to develop the model elements and files. The DEM for the Kinnickinnic watershed was created by clipping a DEM file sourced from Wisconsin LiDAR which was supplied by USACE's GIS section. The source DEM had a cell size of 2 ft x 2 ft. The DEM was resampled using ArcGIS Pro's Resample tool to generate a file with a cell size of 10 m x 10 m in order to reduce geoprocessing time in HEC-HMS (Reference 16).

The DEM was added to the HEC-HMS model and the DEM's projection, NAD 1983 HARN Wisconsin TM (US Feet), was selected as the project projection. Within HEC-HMS, the GIS tools were used to create flow direction and flow accumulation rasters, identify the stream network, and delineate subbasins. The delineated subbasins and streams were verified using the National Hydrography Dataset's (Reference 1) HUC10 and HUC12 delineations.

5.2 Subbasin Delineation

There are four subbasins in the Kinnickinnic HEC-HMS model. Subbasins were delineated to capture contributions to the Kinnickinnic upstream of the HEC-RAS model boundary, River Falls, contributions to the Kinnickinnic South Fork River, and the area downstream of the South Fork's confluence but upstream of the USGS Gage 05342000. The final subbasin delineation differentiates these elements and provides model outputs at important gaged locations.

5.3 Naming

Each subbasin and reach in the Kinnickinnic HEC-HMS model follows a naming convention using abbreviations of element types followed by watershed descriptors. Subbasin names consist of the identifier "S-" followed by a description of the element. Junction names consist of the identifier "J-" followed by a descriptor of an upstream element. Reach names consist of the identifier "R-" followed by descriptors of the reach's start and end points.

5.4 Subbasin Definition

The subbasins and reaches adopted for the Kinnickinnic model are listed in Table 11 along with their associated drainage areas and lengths. Figure 4 shows a map of delineatedsubbasins and streams.

Table 11: Naming Conventions for Kinnickinnic River HMS Model

Subbasin Name	**Subbasin Area (mi^2)**
S-UpperKinni	98.70
S-MiddleKinni	12.77
S-SouthFork	19.25
S-LowerKinni	30.97
SUM	161.69
Reach Name	**Reach Length (mi)**
R-UpperKinni_to_SouthFork	4.31
R-SouthFork_to_Gage	8.39

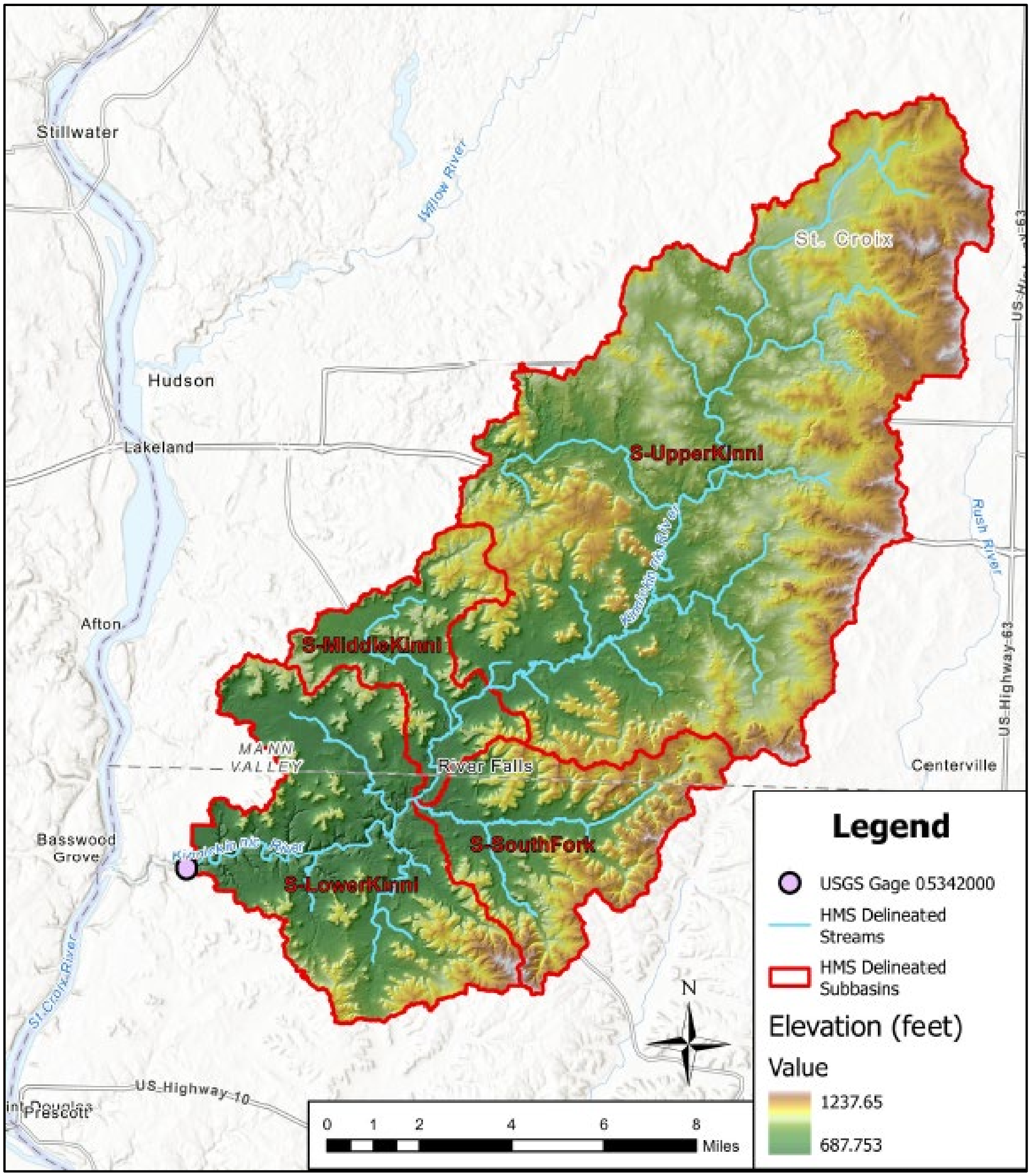

Figure 4: Subbasin Delineation for the Kinnickinnic River HMS Model

6 HEC-HMS Model Construction

The purpose of building the HEC-HMS model was to compare the hydrologic impacts of no action and dam removal alternatives. The model was constructed using HEC-HMS 4.10 (Reference 11). The HEC-HMS model used observed streamflow data from the United States Geological Survey (USGS).

6.1 Methods

The model uses the Initial and constant loss method, the Clark Unit Hydrograph transform method, the Linear Reservoir baseflow method, and the Muskingum-Cunge routing method.

6.1.1 Canopy: Simple

Adding a canopy method to a hydrologic model accounts for the precipitation interception in plants. A canopy method should be selected in multiple-event and/or continuous simulation applications. The Kinnickinnic hydrologic model's purpose is to simulate one and two-day annual peak events. A canopy method was not selected since the presence of a canopy method and/or evapotranspiration methods is generally not a critical consideration in single-event simulations.

6.1.2 Rainfall-Runoff Transform: Clark Unit Hydrograph Parameterization

The Clark Unit Hydrograph method uses two processes: translation and attenuation. Translation describes the movement of excess runoff to the watershed outlet in response to gravity. It is modeled using a time area curved and is influenced by the parameter time of concentration. Attenuation is the reduction of discharge magnitude due to frictional forces and storage that resist flow. It is modeled using a linear reservoir and is controlled by a storage coefficient. A translation hydrograph is routed through a linear reservoir to create a resultant synthetic unit hydrograph.

Time of concentration values were calculated for each subbasin using Equation 1 found in HEC guidance (Reference 17) where T_c is time of concentration, L is the longest flow path, L_c is the centroidal flow path in miles, and $Slope_{10\text{-}85}$ is the average slope of the flow path represented by 10 to 85 percent of the longest flow path:

$$T_c = 2.2 * \left(\frac{L * L_c}{\sqrt{Slope_{10-85}}}\right)$$

Equation 1. Time of concentration, T_C

Once the time of concentration values were calculated, the storage values (R) were calculated using the following Equation 2. For the initial value the target ratio was 0.80. This value was selected based on other HEC-HMS models developed by USACE in and around the St. Croix basin.

$$\frac{R}{T_c + R}$$

Equation 2. Ratio used to calculate storage value, R

Initial T_c and R values are presented in Table 12.

Table 12: Initial Clark Unit Hydrograph Parameters for the Kinnickinnic River Model

Subbasin	Initial Tc	Initial R	Initial R/(T_c+R)
S-UpperKinni	35.37	141	0.799
S-MiddleKinni	17.43	70	0.801
S-SouthFork	15.73	63	0.800
S-LowerKinni	18.72	75	0.800

6.1.3 Loss: Deficit and Constant

The Deficit and constant loss method is appropriate for the Kinnickinnic model because it is event-based and does not require the simulation of the effects of evaporation or evapotranspiration between storm events.

The Deficit and Constant loss method is a simple representation of the soil layer. This method specifies the amount of incoming precipitation that will be infiltrated or stored in the watershed before surface runoff begins (Reference 18). Runoff begins once the initial deficit storage is used up and if the precipitation rate exceeds the infiltration rate. The constant loss rate determines the rate of infiltration that will occur after the initial loss is satisfied (Reference 18). With the exception of impervious areas, if the constant loss rate exceeds the precipitation rate, no runoff occurs.

The percentage of each subbasin that is impervious area is also a parameter. In this method, no loss calculations are carried out on the impervious area; all precipitation on that portion of the subbasin is converted to direct runoff (Reference 18).

6.1.3.1 Initial Loss

The initial loss defines the volume of water that is required to fill the soil layer at the start of the simulation (Reference 18). Initial loss was determined by looking at antecedent conditions for calibration events. There was minimal precipitation and no snowmelt in the days preceding the storms that generated flow for the calibration events. The calibration events did not occur during dry years. A value of 1 in/hr was initially estimated for initial loss.

6.1.3.2 Maximum Storage

The maximum storage indicates the soil's maximum capacity to hold water. The soil is saturated when the soil layer is at the maximum storage capacity, and it is not saturated when the layer contains less than the maximum storage capacity (Reference 18). In this case the maximum storage capacity was estimated to be 2.0 in and tested for sensitivity. During early iterations of calibration it was found that the calibrated flows were not sensitive to this value.

6.1.3.3 Constant Loss Rate

The constant loss rate defines the rate at which precipitation will be infiltrated into the soil layer after the initial deficit has been satisfied in addition to the rate at which percolation occurs once the soil layer is saturated (Reference 18). To estimate initial constant loss rates for each subbasin, the gridded Soil Survey Geographic Database (gSSURGO) was accessed and processed in ArcGIS Pro (Reference 19). The Soil Hydrologic Group loss rate value was used to calculate hydraulic conductivity for each subbasin using an area-weighted average (Reference Infiltration Rate Calculations Based on gSSURGO Data Spatially Averaged in ArcGIS Pro, Table 13).

Table 13: Infiltration Rate Calculations Based on gSSURGO Data Spatially Averaged in ArcGIS Pro

	Infiltration Rate (in/hr)		
Subbasin	**Low Estimate**	**Middle Estimate**	**High Estimate**
S-UpperKinni	1.03	1.93	2.82
S-MiddleKinni	1.08	2.39	3.71
S-SouthFork	1.51	2.25	2.98
S-LowerKinni	1.55	2.84	4.12

Because these infiltration rates are high compared to soils in nearby watersheds, more sources were accessed to verify the SSURGO calculations. Wisconsin Department of Natural Resources

(WI DNR) Groundwater Contamination Susceptibility Model (GCSM) soil characteristics data to determine area-weighted hydraulic conductivities for each subbasin. Percent of land cover type for each subbasin was calculated using the GCSM Data. Each land cover type was assigned a hydraulic conductivity (Reference 21). An area-weighted average hydraulic conductivity was calculated for each subbasin and used as initial the initial constant loss rate value. Those values are along with the weights used for the calculation are presented in Table 14.

Table 14: Infiltration Rate Calculations Based on WI DNR GCSM Soil Characteristic Data

Subbasin	Soil Type Breakdown	Infiltration Rate (in/hr)
S-UpperKinni	14% Sand and Gravel, 62% Loam, 24% Clay	0.695
S-MiddleKinni	51% Sand and Gravel, 29% Loam, 21% Clay	2.362
S-SouthFork	18% Sand and Gravel, 82% Clay	0.843
S-LowerKinni	50% Sand and Gravel, 50% Clay	2.328

These results were cross-referenced with a 2021 USACE Hydraulic and Hydrologic analysis in the watershed (Reference 8). This study used average saturated hydraulic conductivity values between 2 and 2.5 in/hr.

In field infiltration testing was also completed in Lake Louise as a part of the drainage basin design for this project. Results of that testing (Table 54) indicate high infiltration rates in that area, which is in the S-MiddleKinni sub-watershed. All of the 14 tests conducted resulted in infiltration rates higher than 1.5 in/hr.

Based on consistent information from soil data, past reports, and calibration, the Kinnickinnic River watershed has higher infiltration rates than what is typical for the region.

6.1.3.4 Impervious Percentage

The percent impervious value in HEC-HMS represents the percentage of the subbasin which is directly connected to impervious area. No loss calculations are carried out on the impervious area. All precipitation on the impervious portion of a subbasin becomes excess precipitation and is subject to surface storage and direct runoff. Impervious area for each subbasin in the Kinnickinnic HMS model was computed using the 2019 National Land Cover Database (NLCD) Percent Developed dataset (Reference 28). This database assigns a range of imperviousness to each developed land type. Those values are listed in Table 15. Values generated by multiplying land type by its respective impervious percentage were summed for each subbasin. Resulting percent impervious initial values are in Table 16.

Table 15: Percentage of Impervious Area for Each Land Type in the Kinnickinnic River Watershed, According to the NLCD

Land Type	Percent Impervious
Open Water	100%
Developed Open Space	10-20%
Developed Low Intensity	29-49%
Developed Medium Intensity	50-79%
Developed High Intensity	100%

Table 16: Percentage of Impervious Area for Each Subbasin in the Kinnickinnic River HMS Model

Subbasin	Percent Impervious
S-UpperKinni	0%
S-MiddleKinni	2.25%
S-SouthFork	1.71%
S-LowerKinni	0.88%

The percent impervious parameter was not sensitive within the range of reasonable values. For this reason percent impervious was not changed during calibration.

6.1.4 Baseflow: Linear Reservoir

The linear reservoir technique was used in this study and is the only baseflow method that conserves mass within the subbasin. It uses a linear reservoir to model the recession of baseflow after a storm event. Infiltration or percolation computed by the loss method is connected as inflow to the linear reservoirs. Up to three reservoirs can be used in this method. Prior to calibration, one linear reservoir was selected for the model.

Initial runs of the model resulted in far too much volume and peaks three times higher than the observed value. Not all of this volume overage was accounted for by loss rates and transform parameters. More layers were incorporated to represent flow that occurs in interflow, slow moving groundwater flow, or lost to deep aquifers. The baseflow recession timing and volume was also calibrated using linear reservoir groundwater (GW) fractions and GW coefficients. All initial baseflow parameters are presented in Table 17 and Table 18.

6.1.4.1 Initial Discharge per Square Mile

The initial discharge per square mile represents the amount of baseflow before the storm event occurs and the model begins producing runoff (Reference 18). This value was calculated averaging the flow amounts preceding some of the top flood events in the period of record. That value of 110 cfs was divided by the subbasin area (161.7 mi^2) for a resulting initial discharge value of 0.680 cfs/mi^2. This value was applied to GW layer 2 and is shown with other baseflow parameters in Table 18.

6.1.4.2 Groundwater Fraction

The groundwater fraction (GW 1, 2, or 3) determines how much of the baseflow will go to each layer of the reservoir. GW 1 represents the fastest moving baseflow layer, GW 2 represents a layer with slightly slower moving water, often called interflow, and GW 3 represents the water that takes the longest to return to the channel flow. This model only has two layers in the linear reservoir. If the GW fractions add up to one, all precipitation is accounted for in streamflow. To represent groundwater, spring, or aquifer recharge during a storm, the sum of the GW fraction layers will be less than one. This is the case for the Kinnickinnic River HMS model, as it has many soils which are fast draining and dolomite bedrock which may channel the water to such springs. This value is shown with other baseflow parameters in Table 17 and Table 18.

The groundwater fraction was altered in calibration after transform and loss values with the goal of getting an accurate shape of the hydrograph’s rising and falling limbs, reaching an accurate baseflow before and after the event hydrograph, and account for volume losses in the subbasins.

6.1.4.3 Groundwater Coefficient

The groundwater storage coefficient is the time constant for each groundwater layer. It gives a sense of the response time for a component of subsurface flow within a subbasin (Reference

18). Initial values for the fastest moving layer, were selected based on estimated storage coefficient (R) values for each subbasin. The first GW coefficient was determined by multiplying the R value by three. The second GW coefficient was determined by multiplying the GW 1 coefficient by three. These values are shown with other baseflow parameters in Table 17 and Table 18.

The groundwater coefficient was calibrated to achieve accurate shape and timing on the rising and falling limbs of interflow and baseflow components in all subbasins.

6.1.4.4 Number of Reservoirs

The number of steps can be used to subdivide the routing through a reservoir and is related to the amount of attenuation during the routing. Attenuation of the baseflow increases as the number of steps increases.

Table 17: Initial Baseflow Parameters for Groundwater (GW) Layer 1

Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (HR)	GW 1 Reservoirs
S-UpperKinni	0	0.4	423	1
S-MiddleKinni	0	0.4	210	1
S-SouthFork	0	0.4	189	1
S-LowerKinni	0	0.4	225	1

Table 18: Initial Baseflow Parameters for Groundwater (GW) Layer 2

Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (HR)	GW 2 Reservoirs
S-UpperKinni	0.68	0.4	1,269	1
S-MiddleKinni	0.68	0.4	630	1
S-SouthFork	0.68	0.4	567	1
S-LowerKinni	0.68	0.4	675	1

6.1.5 Routing: Muskingum-Cunge

The routing method dictates the equations that route flow through each reach. The Muskingum-Cunge method was selected for the Kinnickinnic HMS model. The method uses a combination of the continuity equation and a simplified form of the momentum equation (Reference 18). The parameters that are required for this method within HEC-HMS include the initial condition, the reach length, the friction slope, manning's n roughness coefficient, a space-time interval method and value, an index method and value, and a cross-section shape and parameters/dimensions.

The Inflow = Outflow initial condition method was selected. The method assumes that the initial outflow is the same is the initial inflow to the reach from upstream elements. Reach length and slope for each reach was calculated using the Reach Characteristics tool within HEC-HMS. Those values are located in Table 11.

6.1.5.1 Index Method and Value

The flow index method was selected. This combines a reference flow and cross-section properties to infer a celerity. Experience has shown that a reference flow (or celerity) based upon average values of the hydrograph is, in general, the most suitable choice (Reference 18). The index flow for the Kinnickinnic River HMS model was taken by finding the midway flow between base flow and peak flow. For the R-UpperKinni_to_SouthFork reach this value was

4075 cfs and for the R-SouthFork_to_Gage reach this value was 6475 cfs. These values did not change during calibration.

6.1.5.2 Cross-section shape and parameters

An eight-point cross section shape was selected because it can represent the main channel plus left and right overbank areas. These cross sections were selected by examining the terrain in the HEC-RAS model and picking out a representative cross section for each reach. These cross sections did not change during calibration.

6.1.5.3 Manning's n Value

Manning's n-values were adopted from the HEC-RAS model. More information on this can be found in Sections 8.1.2 and 8.1.3.

6.2 Control Specifications

The control specifications dictate the start and end dates of a simulation and the computational time interval used in the simulation (Reference 18). The control specifications for each calibration and verification event were set up to capture the precipitation causing the runoff response as well as the complete runoff hydrograph. The model simulations began several days prior to the start of a rainfall event and continued past the hydrograph peak until the discharge appeared to reach baseflow conditions. A 15-minute computation step was used. Table 19 summarizes the control specifications used for the calibration and validation events.

Table 19: Control Specifications for the Kinnickinnic River HMS Model

Purpose	Event	Simulation Start Date	Simulation End Date
Calibration	June 2020	27 June 2020	04 July 2020
Calibration	August 2010	10 August 2010	15 August 2010
Calibration	May 2017	15 May 2017	24 May 2017
Validation	July 2015	05 July 2015	12 June 2015

6.3 Calibrations and Validations

Once the HEC-HMS model was constructed, it was calibrated to three peak annual events to model the rainfall-runoff response of the basin. Events were selected to capture peak annual flows caused by rainfall-driven events at the Kinnickinnic River near River Falls, WI gage (USGS 05342000) during its period of record. Two of the calibration events were the largest rain-driven events in the period of record and of the events had an approximately 50% annual exceedance probability (AEP). All events had 15-minute gage data available and were the main calibration events. Snowmelt-driven events were not selected because rainfall events accurately represent runoff timing during flooding and snowmelt modeling was not included in the scope of this project.

Table 20 lists each calibration event along with its peak flow and rank among all flood events taking place between 1917-1921 and 2002-2022 at the Kinnickinnic River near River Falls, WI gage. Annual exceedance probabilities for each event were determined from the flow frequency analysis conducted by USACE in 2021 (Reference 8).

Table 20: Events Used to Calibrate and Validate the Kinnickinnic River HMS Model

Calibration Event	Purpose	Peak Flow at USGS Gage (cfs)	Rank at USGS Gage	Approximate Exceedance Probability
June 2020	Calibration	6450	1	0.05 (5%)
August 2010	Calibration	4340	3	0.2 (20%)
May 2017	Calibration	1230	14	0.6 (60%)
July 2015	Validation	3100	6	0.3 (30%)

6.3.1 Calibration Parameters and Approach

The parameters used to calibrate the Kinnickinnic River HMS model included subbasin parameters for initial loss, constant loss rate, Clark Unit Hydrograph time of concentration (Tc) and storage coefficient (R), the number of baseflow reservoirs and their parameters (GW Initial, GW Fraction, and GW Coefficient), and the routing parameters. The parameters discussed in the Methods section were used as initial values in each calibration event model. Table 21 describes a summary of each calibration parameter and the corresponding calibration approach.

Table 21: Summary of Calibration Approach for Each Parameter in the Kinnickinnic River HMS Model

Parameter	Calibration Approach
Initial Loss	Initial loss was adjusted independently for each calibration event and therefore is not consistent across all three calibration events. This parameter is highly dependent upon antecedent moisture conditions in the watershed and varies greatly from event to event.
Constant Loss Rate	Subbasin-average saturated hydraulic conductivity values were estimated from WI-DNR GCSM soil data as an initial constant loss rate estimate. The goal was to obtain reasonable loss rates across the basin that reflected soil types and maintained the spatial distribution of the constant loss values present in the WI-DNR GCSM soil data.
Time of Concentration (Tc)	Time of concentration values were estimated by comparing the time of peak flow at the Kinnickinnic River Near River Falls, WI USGS gage to the time of peak precipitation generated by HEC-HMS for each subbasin.
Storage Coefficient (R)	Initial R/(Tc+R) values were selected using HEC guidance to estimate the storage coefficient. The goal was to maintain consistent basin-averaged R/(Tc+R) values for each subbasin.
Number of Layers	Number of layers for baseflow were determined based on the presence of short-duration baseflow (Layer 1), interflow (Layer 2), and long-duration baseflow (Layer 3) during calibration events.
GW Initial	GW Initial varied minimally between calibration events because initial baseflow values were similar prior to each event.
GW Fraction	GW Fraction parameters for each layer were modified to adjust the baseflow volume and timing. For this model, much of the infiltrated water flows to replenish spring-fed reservoirs which did not contribute to flow at the computation point. This was accounted for in the model by having GW fractions for each layer sum less than 1.
GW Coefficient	The GW coefficient was calibrated based on the amount of time water spent in each reservoir. The goal was to have the GW coefficients vary between layers to represent baseflow residence time for Layers 1, 2, and 3. These values were then further adjusted to represent different baseflow residence times for each subbasin.
Number of Reservoirs	Number of reservoirs was initially one for each basin. Then, this number was increased as necessary to represent longer duration of baseflow.
Routing	Muskingum routing was utilized throughout the model. The goal was to obtain adequate timing of flows through each reach.

Because of limited streamflow data within the basin and at the outlets of subbasins, all subbasins were calibrated to match the Kinnickinnic River near River Falls, WI gage (USGS 5342000). Because peak streamflow values (without timing) were available for all events at the Kinnickinnic South Fork gage (USGS 5341900), adjustments were made to the S-SouthFork subbasin to match the peak volume when needed.

6.3.2 Calibration Targets

The goal for calibration was to match the timing of the observed flood peak, the shape of the observed hydrograph, the magnitude of the observed flood peak, and the volume of discharge during the simulation period for two calibration events. Nash-Sutcliffe Efficiency Index Coefficients were computed for each calibration event to assess how well the simulated results replicated the observed data. Nash-Sutcliffe Coefficients can range from negative infinity to one. A coefficient of one indicates the model being applied is able to replicate observed data exactly. Typically, a Nash-Sutcliffe Coefficient of 0.7 or greater indicates the model adequately represents the observed data. Because it is the only streamflow gage used in this study, results are compared to observed data at the Kinnickinnic River near River Falls, WI gage.

6.3.3 June 2020 Calibration Event

The June 2020 calibration event simulation began on 21 June 2020 and ended on 05 July 2020. 2020 began with under average monthly precipitation for January through April. There were rain events in May and June, causing smaller spikes in streamflow. The most significant rainfall event causing a spike in streamflow prior to the Calibration event cause an increase in streamflow to 475 cfs on 27 May. A consistent baseflow around 150 cfs was recorded for late May through the start of the calibration event. On gages in the area reported between 6.5 and 9 inches of rain. This led to the Kinnickinnic River near River Falls USGS gage to reach its highest peak in the period of record (1916-1921, 2002-present) with a flow of 6450 cfs.

Figure 5 shows how modeled discharge compares to observed data at the Kinnickinnic for the June 2020 event. The "June 2020 Calibration" discharge is the model output using the final calibration parameters for the 2018 event. Observed data was recorded on a 15-minute timestep. Table 22 shows how the model results compare to observed data at the Kinnickinnic River near River Falls, WI gage in terms of calibration metrics.

Table 23 shows the parameters used for calibration of the June 2020 event.

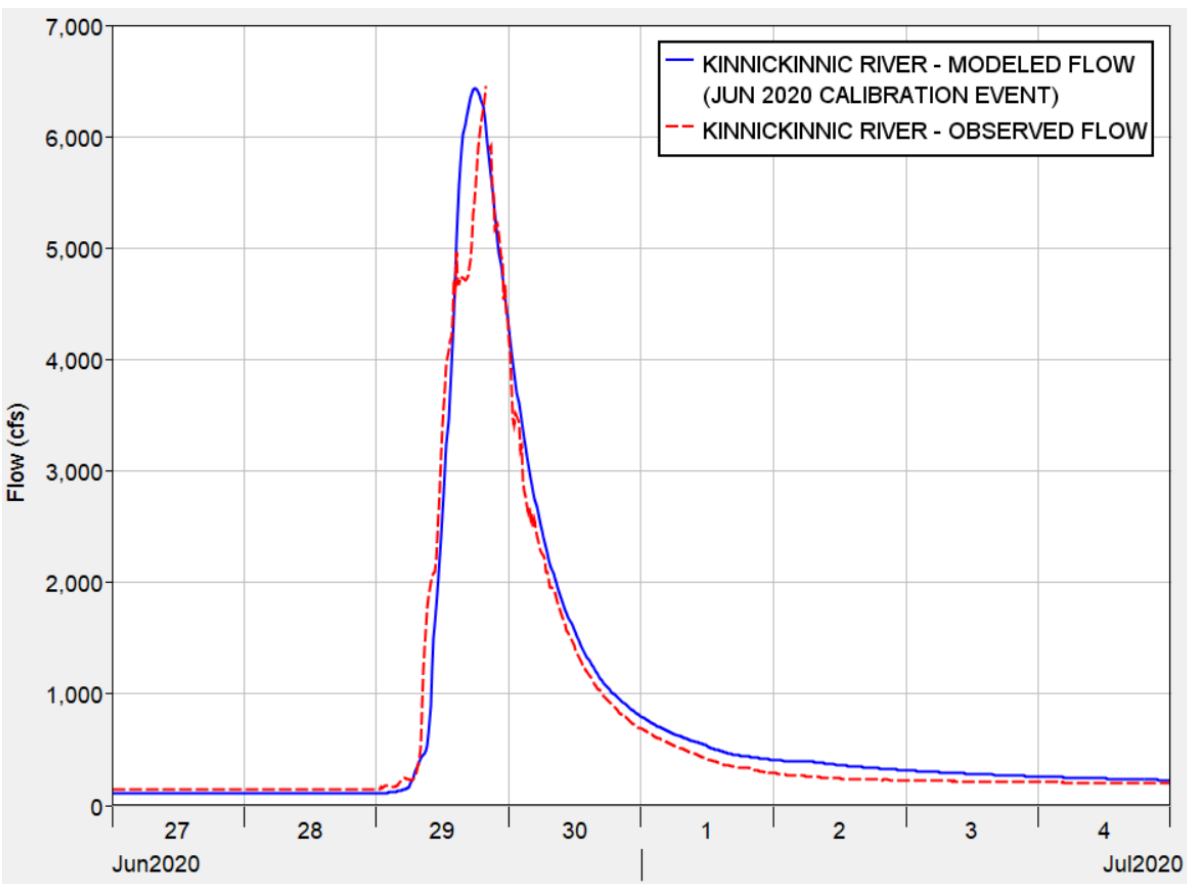

Figure 5: Calibration Results for Jun 2020 Event - Kinnickinnic River near River Falls, WI

Table 22: Comparison of Modeled and Observed Calibration Metrics for June 2020 Event - Kinnickinnic River near River Falls, WI

	Calibrated Model	Observed Data	Difference
Peak Discharge (cfs)	6430	6450	-0.31%
Volume (in)	1.48	1.36	+8.8%
Timing of Peak (date, time)	29 Jun 2020 17:45	29 Jun 2020 19:45	-2:00 hrs
Nash-Sutcliffe Coefficient	0.966		

Table 23: Calibration Parameters for June 2020 Event

Transform Parameters				
Subbasin	Tc (hr)	R	R/(T_c+R)	
S-UpperKinni	8.82	8.59	0.49	
S-MiddleKinni	4.34	3.83	0.47	
S-SouthFork	3.91	4.27	0.52	
S-LowerKinni	4.64	5.15	0.53	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	1.5	2	1.108	0
S-MiddleKinni	1.5	2	1.892	2.25
S-SouthFork	1.5	2	1.344	1.71
S-LowerKinni	1.5	2	1.862	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.1	48.0	1
S-MiddleKinni	0	0.1	22.5	1
S-SouthFork	0	0.1	21.0	1
S-LowerKinni	0	0.1	25.5	1
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.68	0.08	128	3
S-MiddleKinni	0.68	0.08	56	3
S-SouthFork	0.68	0.08	60	3
S-LowerKinni	0.68	0.08	68	3

6.3.4 August 2010 Calibration Event

The August 2010 Calibration event simulation began on 10 August 2010 and ended on 17 August 2010.

Figure 6 shows how modeled discharge compares to observed data at the Kinnickinnic for the June 2020 event. The “August 2010 Calibration” discharge is the model output using the final calibration parameters for the 2018 event. Observed data was recorded on a 15-minute timestep. Table 24 shows how the model results compare to observed data at Black Earth in terms of calibration metrics. Table 25 shows the parameters used for the calibration of the August 2010 event.

The calibration captures the timing and quantity of the peak discharge well. Because of the secondary peak in streamflow, it was challenging to match volume and timing of recession of the peak. This caused a Nash-Sutcliffe value lower than the target value of 0.8.

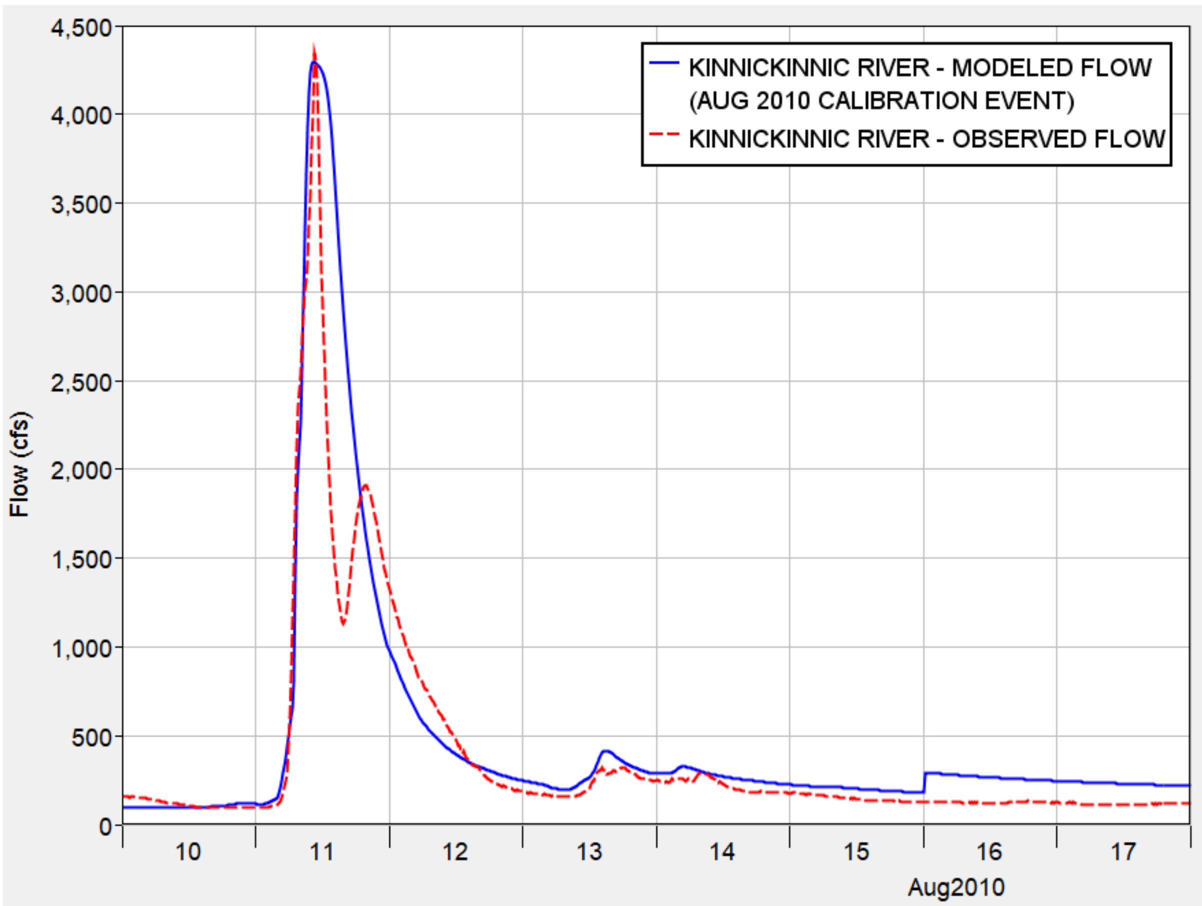

Figure 6: Calibration Results for Aug 2010 Event - Kinnickinnic River Near River Falls, WI

Table 24: Comparison of Modeled and Observed Calibration Metrics for Aug 2010 Event - Kinnickinnic River near River Falls, WI

	Calibrated Model	Observed Data	Difference
Peak Discharge (cfs)	4298	4340	-0.97%
Volume (in)	0.76	0.65	+16.92%
Timing of Peak (date, time)	11 Aug 2010 10:15	11 Aug 2010 10:30	-0:15 hrs
Nash-Sutcliffe Coefficient	+0.705		

Table 25: Calibration Parameters for August 2010 Event

Transform Parameters				
Subbasin	Tc (hr)	R	R/(T_c+R)	
S-UpperKinni	6.00	7.75	0.56	
S-MiddleKinni	2.95	3.46	0.54	
S-SouthFork	2.66	3.86	0.59	
S-LowerKinni	3.16	4.85	0.61	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	0	2	1.118	0
S-MiddleKinni	0	2	1.909	2.25
S-SouthFork	0	2	1.355	1.71
S-LowerKinni	0	2	1.878	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.05	48	1
S-MiddleKinni	0	0.05	22.5	1
S-SouthFork	0	0.05	25.5	1
S-LowerKinni	0	0.05	21	1
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.6	0.15	128	3
S-MiddleKinni	0.6	0.15	60	3
S-SouthFork	0.6	0.15	56	3
S-LowerKinni	0.6	0.15	68	3

6.3.5 May 2017 Calibration Event

The May 2017 calibration event simulation began on 11 May 2017 and ended on 25 May 2017. The May calibration event is at approximately 50% AEP and was selected to get calibration metrics to cover a wide range of events. 2017 began with below average precipitation in January through March, but well above average precipitation in April and early May causing some spikes in streamflow. Baseflow levels of 140 cfs were maintained for the two weeks leading up to the peak. Precipitation began on 16 May with 0.5 – 1.0 inches of rain throughout the basin, continued on 17 May with 1 – 1.5 inches of rain, and continued on 18 May with 1.5 – 3 inches of rain. This event caused a double peak event. The first peak was the largest with a maximum flow value of 1,230 at the Kinnickinnic River near River Falls, WI USGS gage.

Figure 7 shows how the modeled discharge compares to observed data at the Kinnickinnic River for the May 2017 event. The “May 2017 Calibration” discharge is the model output using the final calibration parameters for the 2017 event. Observed data was recorded on a 15-minute timestep. Table 26 shows how the model results compare to observed data at the Kinnickinnic River in terms of calibration metrics. Table 27 shows the parameters used for the calibration of the May 2017 event.

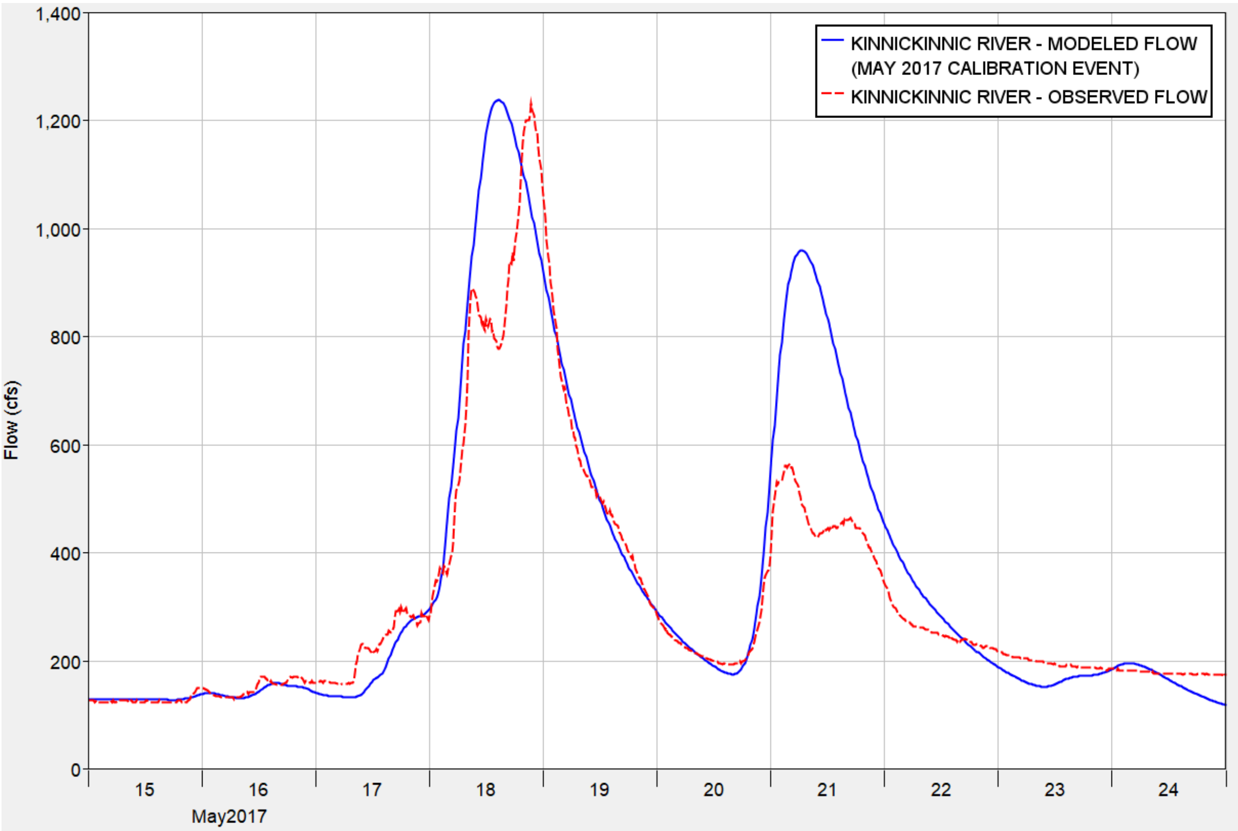

Figure 7: Calibration Results for May 2017 Event - Kinnickinnic River near River Falls, WI

Table 26: Comparison of Modeled and Observed Calibration Metrics for May 2017 Event - Kinnickinnic River near River Falls, WI

	Calibrated Model	Observed Data	Difference
Peak Discharge (cfs)	1239	1230	+0.7%
Volume (in)	0.81	0.72	+13.9%
Timing of Peak (date, time)	18 May 2017 14:00	18 May 2017 21:30	-7:30 hrs
Nash-Sutcliffe Coefficient	0.688		

Table 27: Calibration Parameters for May 2017 Event

Transform Parameters				
Subbasin	Tc (hr)	R	$R/(T_c+R)$	
S-UpperKinni	8.82	8.59	0.49	
S-MiddleKinni	4.34	3.83	0.47	
S-SouthFork	3.91	4.27	0.52	
S-LowerKinni	4.64	5.15	0.53	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	1.75	2	1.11	0
S-MiddleKinni	1.75	2	1.89	2.25
S-SouthFork	1.75	2	1.34	1.71
S-LowerKinni	1.75	2	1.86	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.19	12	2
S-MiddleKinni	0	0.19	4	3
S-SouthFork	0	0.19	4	3
S-LowerKinni	0	0.19	5	3
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.8	0.05	12	3
S-MiddleKinni	0.8	0.05	4	3
S-SouthFork	0.8	0.05	4	3
S-LowerKinni	0.8	0.05	5	3

6.3.5.1 Notes on May 2017 Calibration

Multiple parameter sets were found that calibrated well to the May 2017 event. The initial calibration results for May 2017 are found in Table 29 and the calibration results are shown in Figure 8 and Table 28.

The infiltration rates generated for the June 2020 and August 2010 events during calibration were good fits for each event and similar between events, varying by less than 2%. During the first round of calibration of the August 2017 event, a very good fit was developed with infiltration rates lower than the June 2020 and August 2010 events by 58% in the S-MiddleKinni and S-LowerKinni subbasins and by 79% in the S-UpperKinni and S-SouthFork subbasins.

Because of this variance in constant loss rate from the June 2020 and August 2010 events, further calibration for the May 2017 event was pursued. In this process a set of calibration parameters were found which had constant loss rates much more similar to the other two calibration events. Those parameters and calibration results are presented in Table 26 and Table 27 and Figure 7. Although the modeled results do not replicate the quantity of the secondary peak well, the model does capture the timing and quantity of the initial, larger, peak and the timing of the secondary peak. One explanation for why the quantity of the secondary peak of the May 2017 was higher than the observed data is that there was not enough time between peaks for the model to reach initial abstraction. These were the parameters that were used to select the final parameter set to be used for the synthetic events.

Still, further discussion on why a very good calibration was generated for the May 2017 event using notably lower calibration values than the other two calibration events was necessary to justify the final parameter set, especially because final infiltration rates are higher than those in many other watersheds in MVP's area of responsibility. The work done to verify infiltration rate ranges in the Kinnickinnic River watershed in "Loss: Deficit and Constant" establishes confidence in the higher than typical infiltration rates for this basin. That work included using multiple databases of infiltration rate data and verifying that work against previous studies done in region. It can also be noted that HEC has commented on the temporal variability of transform parameters and has established that they can vary based on the type of event (Reference 13).

The adapted parameters for the May 2017 event were considered sufficient because of the scope of the project. The product of the HMS model was used to provide flows to the HEC-RAS hydraulic model for the basin, reaches, and subbasins. The model provides sufficient calibration for the low-frequency, high-flow events in the basin. This meets project goals because it is most important that these types of events perform well for the HEC-RAS model.

However, if the project scope is expanded and requires more in-depth study of peak events during high-frequency, low-flow years, re-examining infiltration rates in the basin for these types of events could be justified.

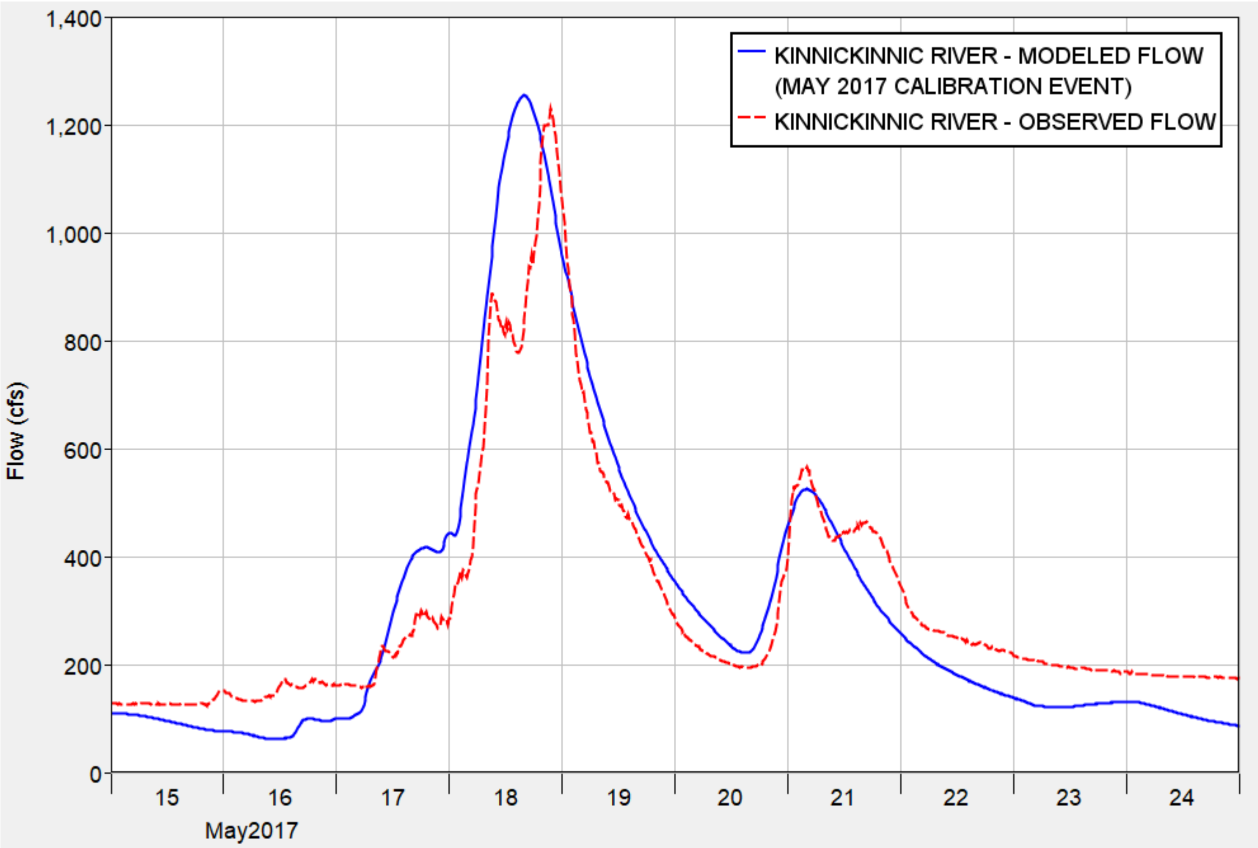

Figure 8: First Round Calibration Results for May 2017 Event - Kinnickinnic River near River Falls, WI

Table 28: Comparison of Modeled and Observed Calibration Metrics for First Round of Calibration for the May 2017 Event - Kinnickinnic River near River Falls, WI

	Calibrated Model	Observed Data	Difference
Peak Discharge (cfs)	1255	1230	+2.03%
Volume (in)	0.72	0.72	0.00%
Timing of Peak (date, time)	18 May 2017 16:00	18 May 2017 21:30	-5:30 hrs
Nash-Sutcliffe Coefficient	0.799		

Table 29: First Round Calibration Parameters for May 2017 Event

Transform Parameters				
Subbasin	Tc (hr)	R	R/(T_c+R)	
S-UpperKinni	15.42	21.15	0.58	
S-MiddleKinni	7.57	10.5	0.58	
S-SouthFork	6.85	9.45	0.58	
S-LowerKinni	6.10	11.25	0.65	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	1	2	0.234	0
S-MiddleKinni	1	2	0.801	2.25
S-SouthFork	1	2	0.285	1.71
S-LowerKinni	1	2	0.787	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.05	7.5	2
S-MiddleKinni	0	0.05	5	2
S-SouthFork	0	0.05	5	2
S-LowerKinni	0	0.05	2.5	2
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.68	0.05	24	2
S-MiddleKinni	0.68	0.05	16	2
S-SouthFork	0.68	0.05	16	2
S-LowerKinni	0.68	0.05	8	2
Baseflow Parameters - Layer 3				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0	0.05	72	3
S-MiddleKinni	0	0.05	48	3
S-SouthFork	0	0.05	48	3
S-LowerKinni	0	0.05	24	3

6.3.6 Representative Calibration Parameters

To finish calibration, refinements were made to find one set of parameters to represent the August 2020, June 2010, and May 2017 events. This was done by looking at the parameters established while calibrating individual events, finding values that were representative across all events, running the model, and adjusting until calibration metrics were satisfied for all events. The representative set of parameters are in Table 30.

Table 30: Representative Parameters for the Kinnickinnic River HMS Model Based on Calibration Events

Transform Parameters				
Subbasin	Tc (hr)	R	R/(T_c+R)	
S-UpperKinni	8.82	8.59	0.49	
S-MiddleKinni	4.34	3.83	0.47	
S-SouthFork	3.91	4.27	0.52	
S-LowerKinni	4.64	5.15	0.53	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	0-1.75	2	1.11	0.00
S-MiddleKinni	0-1.75	2	1.90	2.25
S-SouthFork	0-1.75	2	1.35	1.71
S-LowerKinni	0-1.75	2	1.87	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.07-0.19	12	2
S-MiddleKinni	0	0.07-0.19	4	3
S-SouthFork	0	0.07-0.19	4	3
S-LowerKinni	0	0.07-0.19	5	3
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.8	0.05	69	3
S-MiddleKinni	0.8	0.05	34	3
S-SouthFork	0.8	0.05	31	3
S-LowerKinni	0.8	0.05	41	3

6.3.7 July 2015 Validation Event

The July 2015 validation event simulation began on 05 July 2015 and ended on 13 July 2015. 2015 began with under average monthly precipitation for February through May. The highest spring/winter flow was 187 cfs, which is not significantly above baseflow. These relatively dry conditions continued until a 2-inch rainfall event occurred on June 22. Streamflow increased to 243 cfs after this event, and returned to a baseflow of 111 cfs prior to the calibration event on 06 July. On July 5, approximately 5.3 inches of rain fell. This was a top-five event for the NCEI gage, which has a period of record from 1918 to 2023, but is missing some data. The peak flow at the Kinnickinnic River near River Falls, WI USGS gage was 3,100 cfs.

Figure 9 shows how the modeled discharge compares to observed data at the Kinnickinnic River for the July 2015 event. The “July 2015 Validation” discharge is the model output using the representative parameters generated after calibration of individual events. Observed data was recorded on a 15-minute timestep. Table 31 shows how the model results compare to observed data at the Kinnickinnic River in terms of calibration metrics. Table 32 shows the parameters used for the calibration of the July 2015 event.

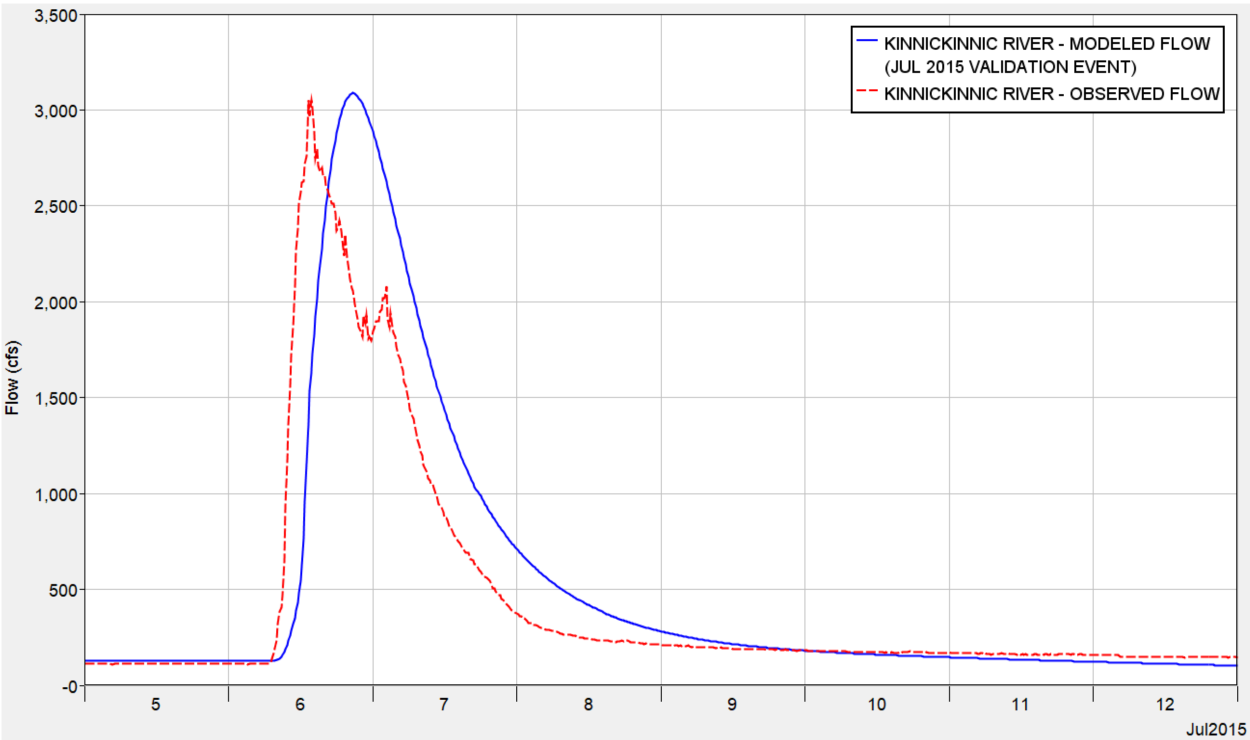

Figure 9: Calibration Results for July 2015 Validation Event - Kinnickinnic River near River Falls, WI

Table 31: Comparison of Modeled and Observed Validation Metrics for July 2015 Event - Kinnickinnic River near River Falls, WI

	Calibrated Model	Observed Data	Difference
Peak Discharge (cfs)	3088	3050	+1.2%
Volume (in)	0.95	0.82	+15.9%
Timing of Peak (date, time)	06 Jul 2015 20:45	06 Jul 2015 13:15	-7:30 hrs
Nash-Sutcliffe Coefficient	0.650		

Table 32: Parameters for July 2015 Validation Event

Transform Parameters				
Subbasin	Tc (hr)	R	R/(T_c+R)	
S-UpperKinni	8.82	8.59	0.49	
S-MiddleKinni	4.34	3.83	0.47	
S-SouthFork	3.91	4.27	0.52	
S-LowerKinni	4.64	5.15	0.53	
Loss Parameters				
Subbasin	Initial Deficit (in)	Maximum storage (in)	Constant Rate (in/hr)	Percent Impervious (%)
S-UpperKinni	1.15	2	1.11	0.00
S-MiddleKinni	1.15	2	1.90	2.25
S-SouthFork	1.15	2	1.35	1.71
S-LowerKinni	1.15	2	1.87	0.88
Baseflow Parameters - Layer 1				
Subbasin	GW 1 Initial (cfs/mi^2)	GW 1 Fraction	GW 1 Coefficient (hr)	GW 1 Reservoirs
S-UpperKinni	0	0.19	12	2
S-MiddleKinni	0	0.19	4	3
S-SouthFork	0	0.19	4	3
S-LowerKinni	0	0.19	5	3
Baseflow Parameters - Layer 2				
Subbasin	GW 2 Initial (cfs/mi^2)	GW 2 Fraction	GW 2 Coefficient (hr)	GW 2 Reservoirs
S-UpperKinni	0.8	0.05	69	3
S-MiddleKinni	0.8	0.05	34	3
S-SouthFork	0.8	0.05	31	3
S-LowerKinni	0.8	0.05	41	3

6.3.8 Calibration Comparison Between Events

Table 33, Table 34, and Table 35 are presented to show the variance in parameters during calibration.

Table 33. Loss Parameters for Kinnickinnic River HMS Model

Initial Deficit (in)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	1.5	0	1.75	1.15	0-1.75
S-MiddleKinni	1.5	0	1.75	1.15	0-1.75
S-SouthFork	1.5	0	1.75	1.15	0-1.75
S-LowerKinni	1.5	0	1.75	1.15	0-1.75
Maximum Storage (in)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	2	2	2	2	2
S-MiddleKinni	2	2	2	2	2
S-SouthFork	2	2	2	2	2
S-LowerKinni	2	2	2	2	2
Constant Rate (in/hr)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	1.11	1.12	1.11	1.11	1.11
S-MiddleKinni	1.89	1.91	1.89	1.89	1.90
S-SouthFork	1.34	1.36	1.34	1.34	1.35
S-LowerKinni	1.86	1.88	1.86	1.86	1.87
Percent Impervious (%)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	0.00	0.00	0.00	0.00	0.00
S-MiddleKinni	2.25	2.25	2.25	2.25	2.25
S-SouthFork	1.71	1.71	1.71	1.71	1.71
S-LowerKinni	0.88	0.88	0.88	0.88	0.88

Table 34: Transform Parameters for Kinnickinnic River HMS Model

Tc (hr)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	8.82	6.00	8.82	8.82	8.82
S-MiddleKinni	4.34	2.95	4.34	4.34	4.34
S-SouthFork	3.91	2.66	3.91	3.91	3.91
S-LowerKinni	4.64	3.16	4.64	4.64	4.64
R					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	8.59	7.75	8.59	8.59	8.59
S-MiddleKinni	3.83	3.46	3.83	3.83	3.83
S-SouthFork	4.27	3.86	4.27	4.27	4.27
S-LowerKinni	5.15	4.85	5.15	5.15	5.15

R/(Tc+R)					
Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
S-UpperKinni	0.49	0.56	0.49	0.49	0.49
S-MiddleKinni	0.47	0.54	0.47	0.47	0.47
S-SouthFork	0.52	0.59	0.52	0.52	0.52
S-LowerKinni	0.53	0.61	0.53	0.53	0.53

Table 35: Baseflow Parameters for Kinnickinnic River HMS Model

	GW Initial (cfs/mi2)					
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 1	S-UpperKinni	0	0	0	0	0
	S-MiddleKinni	0	0	0	0	0
	S-SouthFork	0	0	0	0	0
	S-LowerKinni	0	0	0	0	0
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 2	S-UpperKinni	0.68	0.6	0.8	0.8	0.8
	S-MiddleKinni	0.68	0.6	0.8	0.8	0.8
	S-SouthFork	0.68	0.6	0.8	0.8	0.8
	S-LowerKinni	0.68	0.6	0.8	0.8	0.8
	GW Fraction					
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 1	S-UpperKinni	0.1	0.05	0.19	0.19	.07-0.19
	S-MiddleKinni	0.1	0.05	0.19	0.19	.07-0.19
	S-SouthFork	0.1	0.05	0.19	0.19	.07-0.19
	S-LowerKinni	0.1	0.05	0.19	0.19	.07-0.19
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 2	S-UpperKinni	0.08	0.05	0.05	0.05	0.05
	S-MiddleKinni	0.08	0.05	0.05	0.05	0.05
	S-SouthFork	0.08	0.05	0.05	0.05	0.05
	S-LowerKinni	0.08	0.05	0.05	0.05	0.05
	GW Coefficient (hr)					
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 1	S-UpperKinni	48	48	12	12	12
	S-MiddleKinni	22.5	22.5	4	4	4
	S-SouthFork	21	21	4	4	4
	S-LowerKinni	25.5	25.5	5	5	5
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 2	S-UpperKinni	128	71	69	69	69
	S-MiddleKinni	60	35	34	34	31

	S-SouthFork	56	31	31	31	34
	S-LowerKinni	68	37	41	37	41
	GW Reservoirs					
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 1	S-UpperKinni	1	1	2	2	2
	S-MiddleKinni	1	1	3	3	3
	S-SouthFork	1	1	3	3	3
	S-LowerKinni	1	1	3	3	3
	Subbasin	Jun 2020	Aug 2010	May 2017	Jun 2015	Generalized
Layer 2	S-UpperKinni	3	3	3	3	3
	S-MiddleKinni	3	3	3	3	3
	S-SouthFork	3	3	3	3	3
	S-LowerKinni	3	3	3	3	3

6.4 Factors Influencing Calibration and Validation Success

6.4.1 Limited Gage Data and Period of Record

One limitation of the HEC-HMS model is the lack of and relatively short duration of gage data on the Kinnickinnic River and in the basin. There is one gage with continuous flow records to the present day. That is the Kinnickinnic River near River Falls USGS gage (05342000) which has a record of 1998-1999, 2002-present of continuous data and additional years 1917-1921 of daily average and annual peak flow record. This gage does not capture the three peak flood events known in the basin (Reference 12).

The result is only one event to calibrate to which has flow greater than the 10% AEP flow. The lack of high flow events for calibration and validation results in a higher uncertainty in calibration parameters.

A potential improvement to the HEC-HMS model would be to do further calibration and validation using estimated flows for historic flood events, estimated by a professor at the University of Wisconsin-River Falls (Reference 12). This would allow for higher confidence in results for less-frequent, higher-flow events.

6.4.2 Temporal Variation in Transform and Initial Loss Parameter

Another limitation of the HEC-HMS model is the potential temporal variability. When modeling the precipitation-runoff process in a hydrologic model, precipitation or melted snow that is not infiltrated is subjected to a transformation process in order to estimate a runoff hydrograph. The most commonly utilized transform method within the USACE is the unit hydrograph theory, which has been widely used as a means to predict the timing and magnitude of runoff since its inception in 1932. This theory implies this implies that a watershed will linearly respond to any changes excess precipitation (Reference 14).

However, due to differences in areal distributions of rainfall and the hydraulic reactions between large and small precipitation events, the corresponding unit hydrographs have not been found to be equal, as implied by unit hydrograph theory (Reference 14).

This phenomena may be applicable to the differences in calibrations between the June 2020 and May 2017 events. The difference in basin parameters to achieve calibrations between these events may be an indication that the basin response to precipitation varies depending on the scale of the event. For the purposes of this study, parameters were selected which better fit the higher flow calibration events. This results in a model that should predict the flows in and around the 5% AEP event most accurately.

In Spring 2023, the Hydrologic Engineering Center (HEC) incorporated a transform method which allowed for variance in the transform parameters applied to the Clark Unit-Hydrograph method. (Reference 14). It was not in the scope of this study to incorporate this new modeling method, but applying the Variable Clark Method to the Kinnickinnic Watershed is an opportunity for future study.

Finally, the model results were sensitive to changes in the initial loss parameter. Initial loss was estimated by examining streamflow, precipitation, and temperature in the weeks and months leading up to calibration and validation events. Based on that data, an assumption was made whether the soil conditions were relatively dry or wet and an initial loss parameter was estimated.

7 HEC-RAS Input Generation

Synthetic precipitation events with annual exceedance probabilities of 50%, 10%, 4%, 2%, 1%, 0.5% and 0.2% were run through the calibrated HEC-HMS model to generate hydrographs for each event at significant locations in the Kinnickinnic River Basin HEC-RAS model.

7.1 HEC-HMS Basin Model

The calibrated and validated basin model, discussed in Section 6.3 was used for all exceedance probabilities. The parameters for that basin model are presented in Table 30.

7.2 Meteorological Model: Atlas 14 Precipitation Data Selection

For this study, the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% synthetic precipitation events were used to generate the same-chance exceedance frequency hydrographs in HEC-HMS.

7.2.1 Atlas 14 Synthetic Precipitation Database

A synthetic precipitation event consists of a cumulative precipitation volume and a temporal distribution. Precipitation frequency estimates for all exceedance probabilities were obtained from the NOAA Precipitation Frequency Data Server (PFDS) (National Weather Service, 2023). The PFDS is a web portal that displays NOAA Atlas 14 data in various forms including point precipitation estimates in tabular and gridded formats, as well as probabilistic temporal distributions in csv format. The site was used to download ascii grids of spatially interpolated precipitation frequency estimates for all exceedance probabilities at the RIVER FALLS (47-7226) station (Reference 15)

7.2.2 Storm Pattern

The June 2020 event pattern was selected as the storm pattern for the meteorological models for synthetic storm simulations in HEC-HMS. This was because the June 2020 event replicates the typical response of the Kinnickinnic River during a large rainfall-driven flow event. Figure 10 shows hydrograph shapes for the three events used to calibrate the HEC-HMS model.

The May 2017 event has a different shape from the June 2020 and August 2010 events. It should be noted that the storm pattern used for synthetic precipitation event modeling was selected to represent higher AEP events in the watershed. These events look more similar to the June 2020 and August 2010 events, which peak flows correspond to approximately 5% and 20% AEPs. The May 2017 event has a peak flow which is approximately a 60% AEP event. For this reason the June 2020 storm pattern was deemed more applicable for this analysis. More discussion of the May 2017 event and its calibration in HEC-HMS can be found in Section 6.3.5.

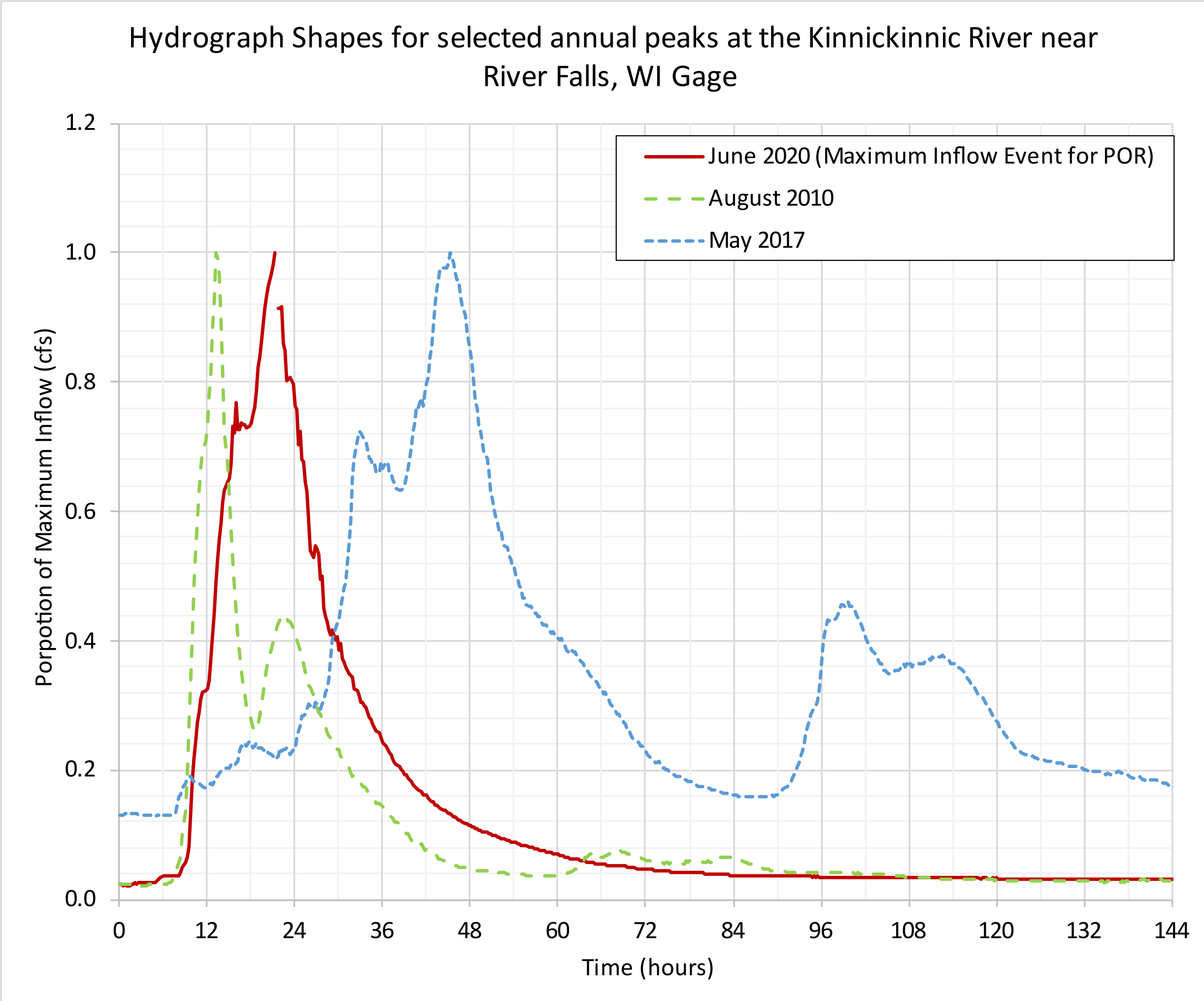

Figure 10. Flow hydrographs Kinnickinnic River near River Falls, WI USGS gage for the events used to calibrate the HEC-HMS model

7.2.3 Storm Duration

Time of concentration (T_C) was considered when determining synthetic storm duration. T_C values varied between 2 and 9 hours throughout the watershed. However, T_C values calculated using basin characteristics varied between 15 and 36 hours. For this reason, storm durations of 12 and 24-hours were tested. Ultimately, the 12-hour synthetic precipitation events produced peak flow values and hydrograph shapes which more closely matched historic data, particularly the June 2020 event.

7.3 HEC-HMS Model Results for HEC-RAS Locations of Interest

Peak flow values at the Kinnickinnic River near River Falls, WI USGS gage are shown in Table 36. Also in that table are comparative values to the Bulletin 17C analysis done as a part of the 2021 Kinnickinnic River Study (Reference 8) and to the FEMA Flood Insurance Study (FIS) values (Reference 23). HEC-RAS results with the modeled HEC-HMS flows are compared to the FEMA FIS flows in Table 37 through Table 39.

Table 36. Peak flows at the Kinnickinnic River near River Falls, WI gage (USGS 05342000) (Reference 8, 23)

Annual Exceedance Probability	HEC-HMS Modeled Peak Flow (cfs)	2021 Study Bulletin 17C Peak Flow (cfs)	FEMA FIS Flows* (cfs)
0.5 (50%)	1687	1700	NA
0.2 (20%)	3444	3440	NA
0.1 (10%)	4896	4910	8900
0.04 (4%)	6869	6550	NA
0.02 (2%)	9140	9000	14600
0.01 (1%)	11719	11070	16900
0.005 (0.5%)	15774	13350	NA
0.002 (0.2%)	21960	16690	22500

* FEMA FIS flows are pulled from downstream of Powell Falls Dam at the confluence of Rocky Branch. The FIS model does not extend to the gage.

Table 37. Peak flows at the Kinnickinnic River near River Falls, WI downstream of Powell Falls Dam (Reference 23).

Annual Exceedance Probability	HEC-RAS Modeled Peak Flow (cfs)	FEMA FIS Flows (cfs)
0.1 (10%)	3970	6800
0.02 (2%)	8050	11000
0.01 (1%)	10380	12800
0.002 (0.2%)	17898	16900

Table 38. Peak flows at the Kinnickinnic River near River Falls, WI upstream of Junction Falls Dam (Reference 23).

Annual Exceedance Probability	HEC-RAS Modeled Peak Flow (cfs)	FEMA FIS Flows (cfs)
0.1 (10%)	3272	3350
0.02 (2%)	7125	7050
0.01 (1%)	9318	8700
0.002 (0.2%)	16038	13000

Table 39. Peak flows at the Kinnickinnic River near River Falls, WI Upstream of State Hwy 35 (Reference 23).

Annual Exceedance Probability	HEC-RAS Modeled Peak Flow (cfs)	FEMA FIS Flows (cfs)
0.1 (10%)	2825	3050
0.02 (2%)	6763	6450
0.01 (1%)	8935	8000
0.002 (0.2%)	15944	11900

8 HEC-RAS Model Construction

8.1 Existing Conditions

As part of their 2017 Kinni Corridor study, the engineering consulting firm Short, Elliot, and Hendrickson (SEH) conducted a hydraulic analysis of the Kinnickinnic River with hypothetical, post-removal conditions in conjunction with the dam removal feasibility studies. SEH updated the model's terrain data and added stream cross-sections from downstream of Powell Falls dam through just upstream of Lake George. These updates were based on the sediment survey conducted by Inter-Fluve as part of their 2016 study (Reference 24), LiDAR data, and as-built drawings of the dams (Reference 25, 26). When conducting the analysis, however, SEH assumed that both Powell Falls Dam and Junction Falls Dam would be removed; this is reflected in their Hydrologic Engineering Center – River Analysis System (HEC-RAS) model and their technical memo on the study, (Reference 27).

8.1.1 Terrain

An updated terrain was developed for this study from available DEMs based on 2019 LiDAR data. The DEMs were obtained for St. Croix and Pierce Counties with a 1m resolution (Reference 2828). The vertical datum is NAVD88.The horizontal projection of the terrain is NAD 1983 HARN Wisconsin TM US Ft. Survey cross-sections were collected in the Spring of 2023 through the project area, from the Maple Street bridge to about 450 ft downstream of Powell Falls Dam. A civil surface created from the collected data was incorporated into the terrain. The model and terrain is shown in Figure 11.

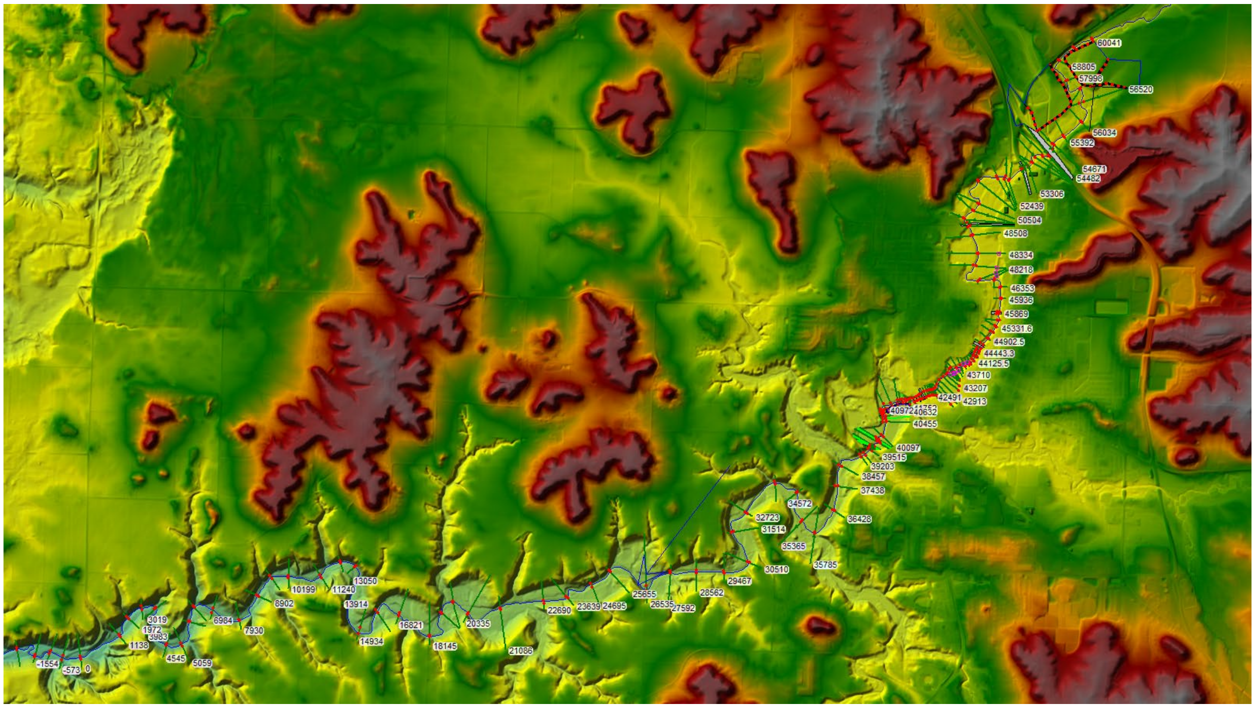

Figure 11: HEC-RAS Model and DEM

8.1.2 Model Development

Several updates were made to the SEH model for this study. The cross sections were georeferenced, based on the reach lengths in the SEH model. Upstream of the Maple Street bridge, the model cross section geometry is based on the SEH model for the channel and the DEM for the overbanks. New survey data was obtained between the Maple Street bridge through roughly 450 ft downstream of Powell Falls Dam. This survey data included the drained Lake Louise channel. The model was extended downstream to just beyond the USGS gage (see Figure 13). For this downstream portion of the model geometry, the cross sections were based solely on the DEM, as obtaining survey data throughout this reach was beyond the scope of this study. Throughout the model, cross sections were extended to capture the updated 0.2% AEP inundation area. This included the addition of some storage areas in the overbanks. Model ineffective areas for floodplain obstructions were also updated.

Manning's n values vary based on terrain conditions in the area. The channel is generally rocky with patches of vegetation and occasional debris and uses a manning's n value of 0.04. The channel banks are generally densely vegetated with trees and debris and use a manning's n value of 0.09. A summary of conditions and the assumed manning's n values are included in Table 40 (Reference 29).

Table 40: Manning's n Values

Condition	Manning's n Value
Channel- r clean, winding, some pools and shoals, occasional weeds and stones	0.04
Banks – medium to dense brush and trees	0.09
Pavement and urban features	0.016-0.02
Maintained grassy areas	0.03

Marsh- sluggish, weedy reaches and pools, scattered brush	0.05-0.07

All bridges were updated based on bridge plans obtained from the City of River Falls or those available online through the Wisconsin Department of Transportation (DOT) (Reference 34, 35, 36, 37, 38). Ineffective flow areas were added, as required. At the bridges, the upstream contraction used a 1:1 ratio. The downstream expansion used a 2:1 ratio.

The dams were updated according to the plans (Reference 25,26). Junction Falls dam is currently used for hydropower. Powell Falls Dam has not been in operation since it was damaged during the storm in 2020. The gate at Powell Falls Dam has remained fully open since the lake was drained in 2021. Both dams have been considered run-of-the-river dams, with the outflow assumed to be equal to the inflow. Because of this, the model does not include gate operations. All flow is assumed to run over the dam at Junction Falls Dam, and the gate at Powell Falls Dam was modelled as open for the duration of the simulation.

The model used a downstream boundary condition of normal depth, located roughly 2600 ft downstream of the USGS gage. The upstream boundary condition was a flow hydrograph, and several lateral inflow hydrographs were included throughout the reach. The HEC-HMS model included four subbasins, as shown in Figure 4. The subbasin flows are shown in Table 41 The subbasins were divided up further using flow multipliers in the HEC-RAS model flow file. The input cross-sections are identified in Figure 12, and the locations and associated multipliers are shown in Table 42. The input cross-sections and multipliers were estimated based on topography and known stormwater outfall locations. The largest portion of flow comes from the upstream basin and the South Fork Kinnickinnic tributary, which had more easily identifiable input locations. The input locations through town were more approximate, and there weren't any flow measurements to calibrate to, at this time. A recommendation will be included to collect data for the design phase. The flow inputs downstream of Powell Falls Dam were also approximate but also do not impact the project area.

Table 41: HEC-HMS Subbasin Flows

HMS Subbasin	Low Flow (cfs)	50% AEP (cfs)	20% AEP (cfs)	10% AEP (cfs)	4% AEP (cfs)	2% AEP (cfs)	1% AEP (cfs)	0.2% AEP (cfs)
UpperKinni	64.1	782	1686	2825	4720	6763	8961	11790
MiddleKinni	8.3	227	416	475	455	651	1005	1587
USGS 05341900 (South Fork Kinnickinnic)	12.5	333	617	758	1280	1905	2746	3823
Lowerkinni	20.1	423	791	898	844	1189	1897	3064

Table 42: HEC-HMS Subbasin Linking

HMS Subbasin	Model Cross Section	Flow Multiplier
UpperKinni	60041	1
	54671	0.8

MiddleKinni	50504	0.1
	43207	0.1
USGS 05341900 (South Fork Kinnickinnic)	41704	1
Lowerkinni	38457	0.6
	35785	0.2
	21086	0.2

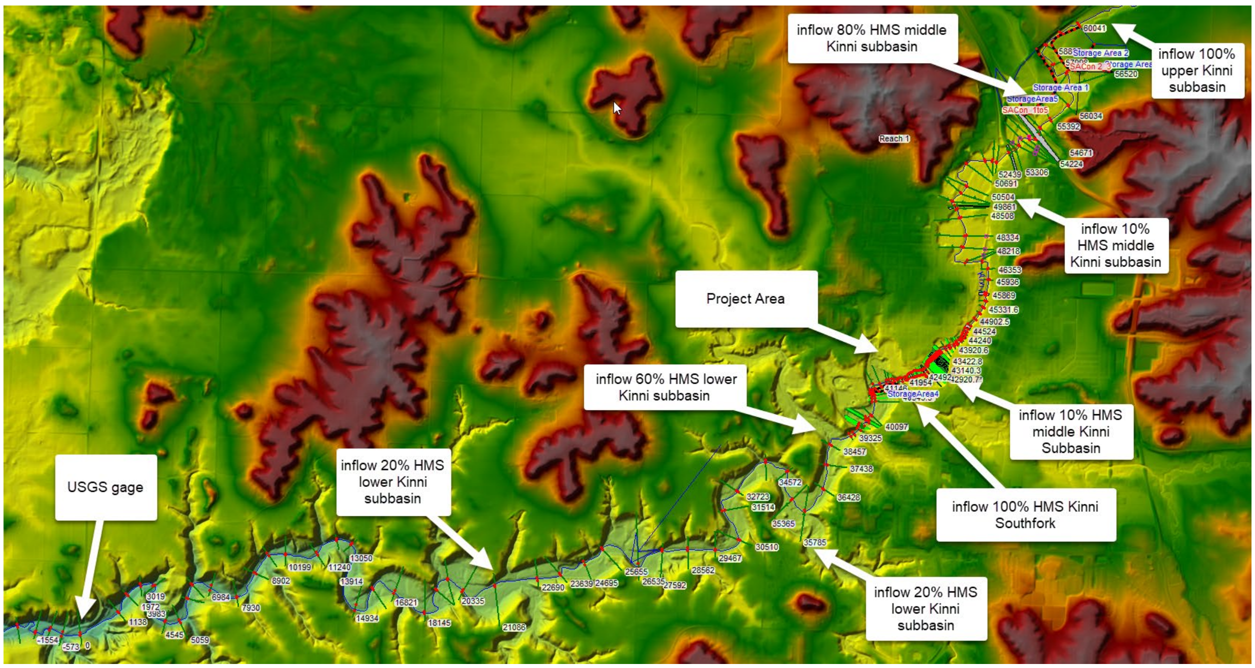

Figure 12: HEC-HMS Flow Input Locations

The hydrology used for the modelling for fish habitat was based on a low flow duration analysis at the USGS gage and scaled for the subbasins, based on area. 105 cfs was used as the flow for the total basin, as that corresponds to the flow in February roughly 50% of the time (Reference 39).

8.1.3 Model Calibration

The hydraulic model is lacking calibration data at this stage of the study. The only active gage on the reach is USGS gage 05342000 (Reference 39), as shown in Figure 13. The model was extended downstream to this gage based on the DEM. Being relatively distant downstream from the project area and downstream of the Powell Falls Dam, calibration to this gage does not significantly inform the modeling in the project area. The dams produce the controlling tailwater levels for each of the project areas. Information on flows and water surface elevations within the project area were not able to be collected within the timeframe of this study but are recommended for the design phase.

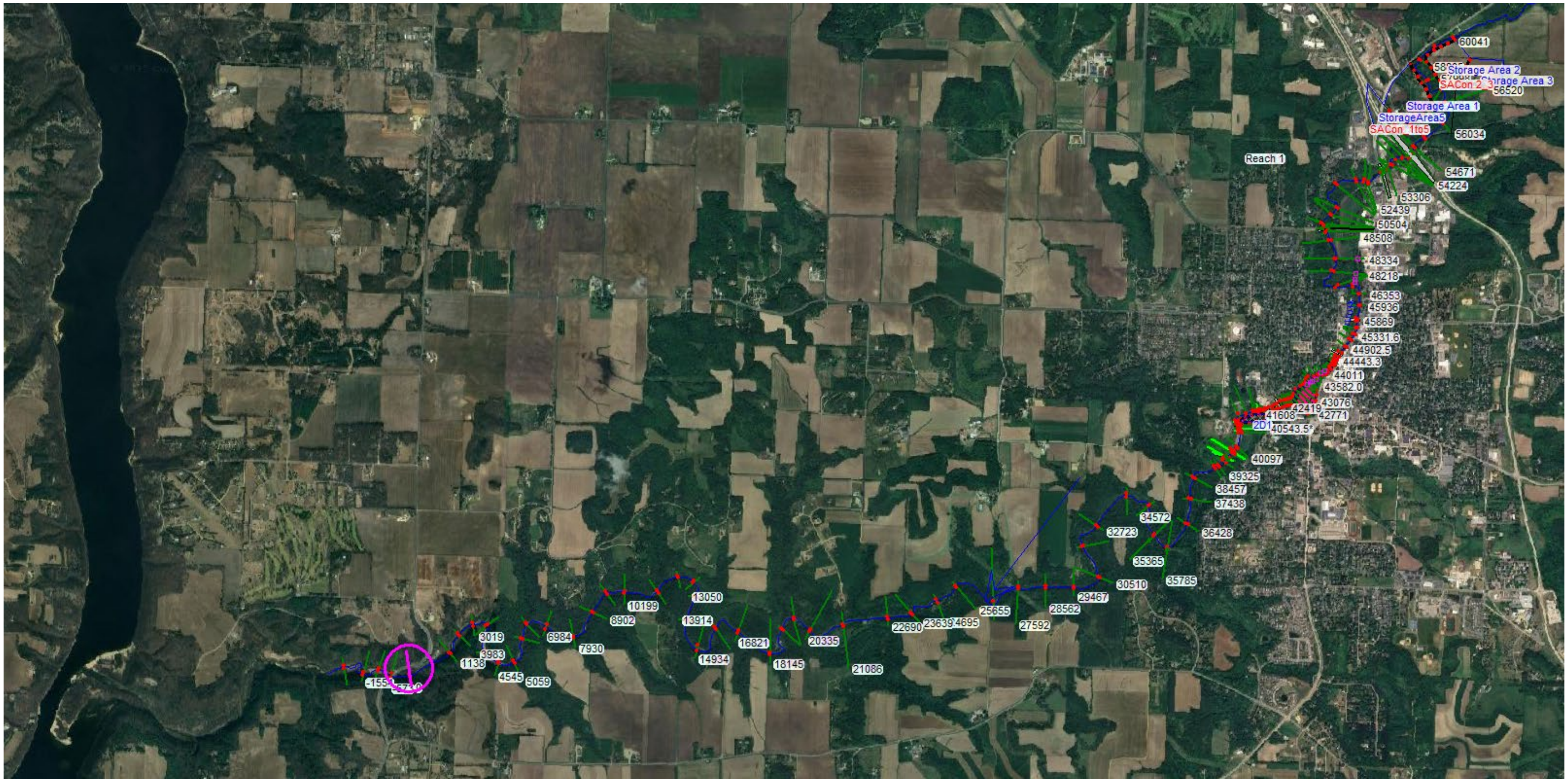

Figure 13: USGS Gage Location

A validation run was done using the 2020 storm event flows to get some indicator of model performance. Figure 14 shows the results of the validation run with comparison to the observed stage and flow at the USGS gage. The model stage and flow are matching adequately at this location.

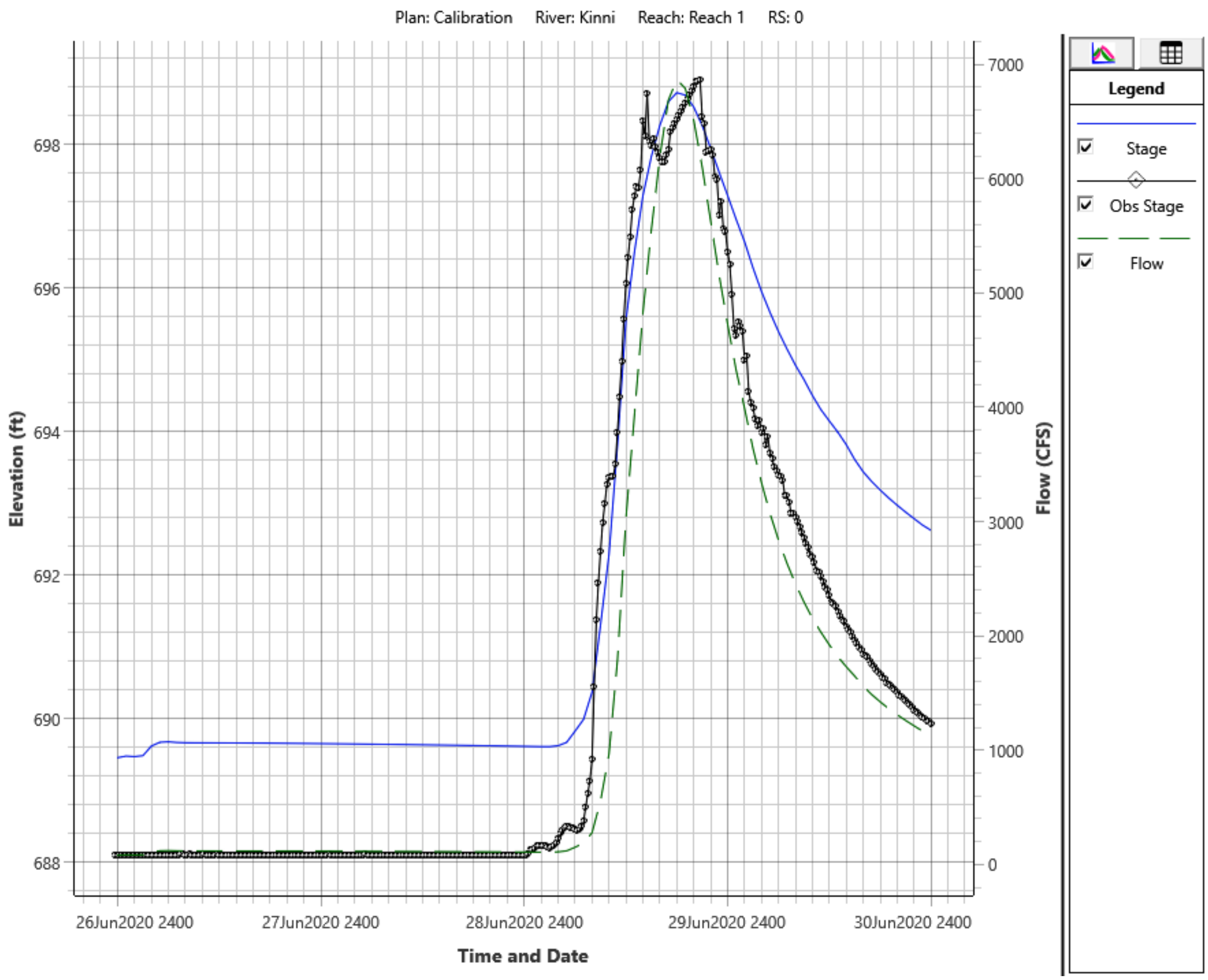

Figure 14: HEC-RAS Model Calibration Plot

Because there was inadequate data for calibration, a couple of runs were done on the sensitivity of the manning's n values for the 50% AEP and 1% AEP events. For the 50% AEP, the differences in velocities between the high manning's n, low manning's n, and existing conditions runs were mostly within a 0-2 ft/s range. Within the project area, they were within 1 ft/s. A few cross-sections outside of the project area had velocity differences up to 4 ft/s. Water surface elevations were generally within 0.6 ft between the three runs. A few cross-sections, outside of the project area, were seeing higher differences, up to 4.2 ft, but the high manning's n values were causing negative flow results. Velocity differences were similar for the 1% AEP. Water surface elevations for the 1% AEP were within 1.6 ft.

8.2 With-Project Conditions

There are two geometries for the with-project conditions. One is for Alternative 2 (Figure 15), removal of Junction Falls Dam. The model geometry layout is shown in Figure 16. The other is for Alternative 3 (Figure 17), removal of Powell Falls Dam. The model geometry layout is shown in Figure 18. The Alternative 4, TSP (Figure 19), model was made by combining the with-project sections of both of those models. The reaches are separated hydraulically by Junction Falls Dam in the existing conditions and by the waterfall that would replace it in the proposed conditions.

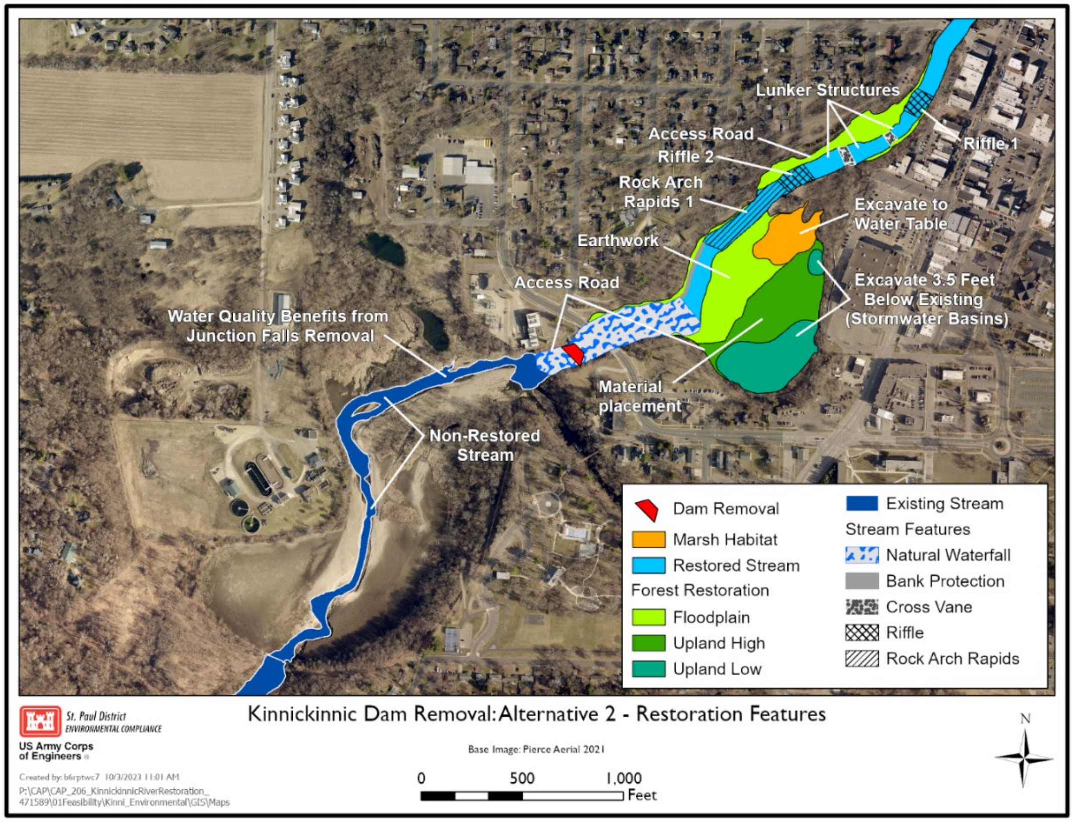

Figure 15: Alternative 2 Plan

Figure 16: Alternative 2 Model Geometry

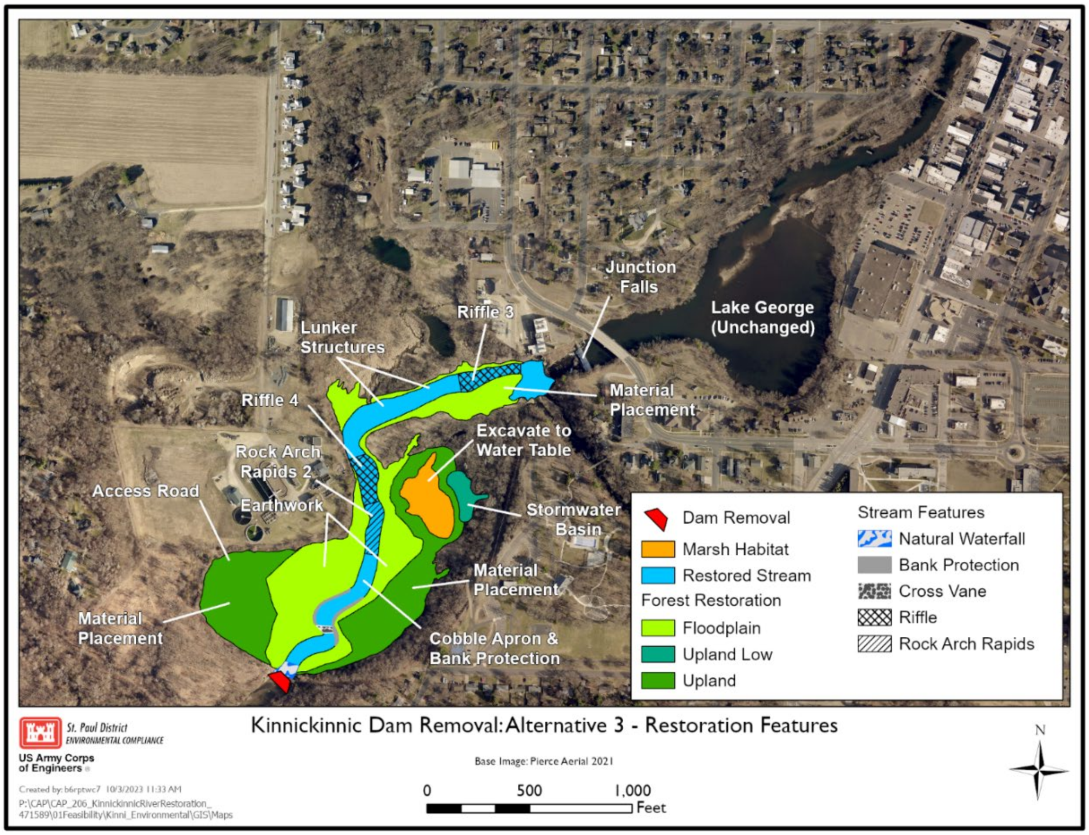

Figure 17: Alternative 3 Plan

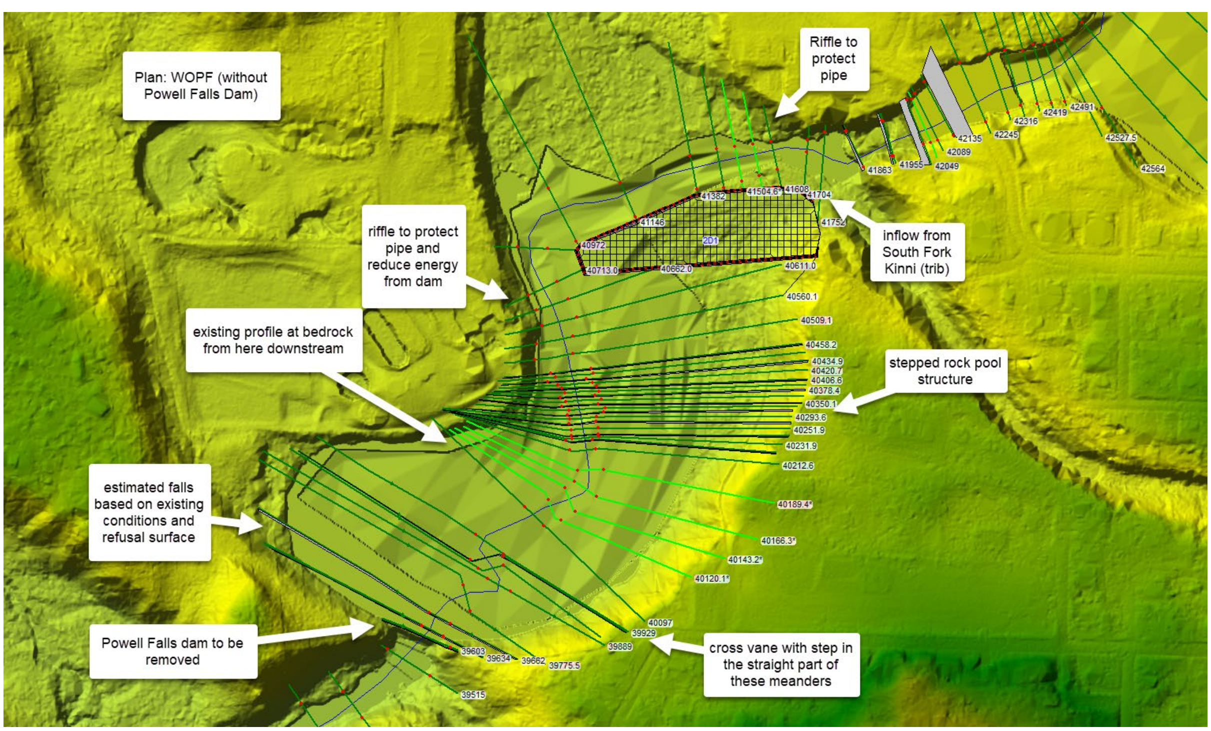

Figure 18: Alternative 3 Model Geometry

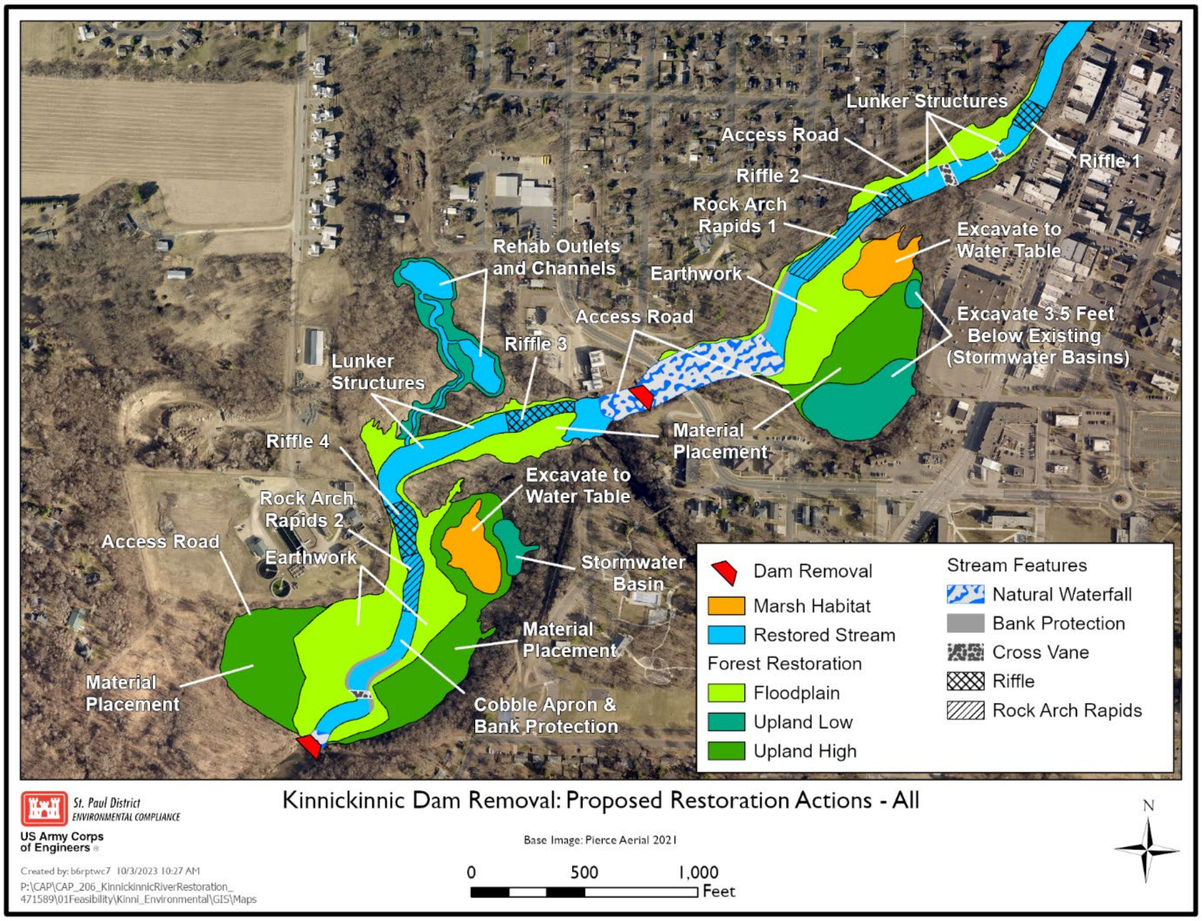

Figure 19: Tentatively Selected Plan

8.2.1 Objectives

8.2.1.1 Protection of existing Infrastructure

The first priority for design consideration was the protection of existing infrastructure. The river runs through town, and there are adjacent homes and public recreation areas. There are bridges upstream of Junction Falls Dam that could be impacted by head cutting after removal of the dams without the addition of in-channel rock features to prevent this. There are also two utility pipes that cross the river in the Lake Louise area; the downstream of which is currently abandoned and acting as a grade-control structure under current

8.2.1.2 Geomorphic Stability

Another key priority, related to the protection of infrastructure, was developing a design that would be geomorphologically stable. A typical stable cross-section width in the non-impacted parts of the river was estimated to be around 60 ft. While some cross-sections in the with-project designs are wider, this target width was kept in mind when developing the cross-sections and incorporating that into existing conditions, where applicable. The existing channel alignment was maintained. An overbank floodplain bench was designed such that flow would exceed channel banks in a 50% AEP event but would be contained to the main channel during normal flows. The floodplain bench will include plantings suitable for that frequency of inundation.

The design targeted stability of the features and sediment up to the 1% AEP event. Rock features and bank protection were sized accordingly. The 2016 Sediment Assessment conducted by Interfluve (Reference 24), included a probe investigation of the refusal surface below the existing lake beds. This refusal surface was used to estimate the future channel profile and inform the need for erosion protection.

8.2.1.3 Trout Habitat

A component of the benefits calculation consisted of trout habitat benefits. The benefit objectives include the following:

- Average velocity over spawning areas of 1.3-2.3 fps
- 50-70% of the project reach consisting of pools
- If feasible, a “Class A” rating for pools, which consists of greater than or equal to 30% of the area composed of first-class pools. Otherwise, a “class B” rating, which has greater than or equal to 10% but less than 30% first-class pools, and greater than or equal to 50% second-class pools.
  - A first-class pool is described as “large and deep. Pool depth and size are sufficient to provide a low velocity resting area for adult trout. More than 30% of the pool bottom is obscured due to depth, surface turbulence, or the presence of structures such as logs, debris piles, boulders, or overhanging banks and vegetation. Or, the greatest pool depth is greater than or equal to 1.5m in streams less than or equal to 5m wide or greater than or equal to 2m deep in streams greater than 5 m wide.”
  - A second-class pool is described as “moderate size and depth. Pool depth and size are sufficient to provide a low velocity resting area for adult trout. From 5% to 30% of the bottom is obscured due to surface turbulence, depth, or the presence of structures. Typical second-class pools are large eddies behind boulders and low velocity, moderately deep areas beneath overhanging banks and vegetation.
- Floodplains are to be planted with forest to meet shade requirements and maintain lower water temperatures.
- There are substrate objectives that could be impacted by shear or velocity values that should be looked at more in the design phase, in combination with sediment modelling.
  - 0-10% fines (<3mm) in riffle-run areas during average summer flows
    - Shear > 0.04 psf in riffle-runs
  - 0-5% fines (<3mm) in spawning areas during average summer flows
    - Shear > 0.04 psf in riffles
  - At least 10% of substrate (10-40 cm)
    - At least 10% area with Shear max 1.73-4.66+
  - At least 5% of substrate with gravel 0.3-1 cm and 7-10 cm
    - 5% area with shear max 0.04-0.15 and 1.19-1.73
  - At least 5% area with gravel 1-7 cm
    - 5% area with Shear max 0.15-1.19

8.2.1.4 Stormwater Detention Basins

Some of the lake space is to be used as marsh habitat to increase habitat area. Detention basins are to be used to collect stormwater from storm sewer outlets. These features were not included in the HEC-RAS model. Design of the detention basins is covered in Section 10.

8.2.2 Removal of Junction Falls Dam

Underneath Junction Falls Dam, there is a natural waterfall, as indicated by historical records and photos (Figure 20). Upon removal of the dam, the natural waterfalls will be exposed and the upstream gorge cleared.

Figure 20: Historical Photo prior to Junction Falls Dam (Reference 40)

The Winter St bridge is in the gorge upstream of the dam. According to the plans, the central pier is embedded in the bedrock (Reference 30). The feasibility design includes estimates for concrete quantities to armor the pier up to the 0.2% AEP event. Further investigation into the elevation of the bedrock and the necessity of protections should be done in the design phase.

There is a pedestrian bridge upstream of Lake George (Junction Falls dam area) and two roadway bridges upstream of there (Maple Street and Division Street) that are within range for potential impacts from removal of the dam. Based on the plans and estimated refusal surface, it is unclear if the pier at the pedestrian bridge and upstream bridges are embedded in bedrock adequately enough to safely maintain the bridges if the head cutting from the dam removal were to reach that far upstream. Further investigation of the bridge foundations and the potential for head-cutting is recommended for the design phase – the rock features included in the TSP are anticipated to prevent head-cutting upstream of Lake George. Additionally, there is infrastructure along the banks that could be impacted, if the channel were to cut several feet, as indicated by the estimated refusal surface. In lieu of this uncertainty, the feasibility design recommends maintaining roughly the existing grade and channel upstream of Lake George. At the upstream end of Lake George, a proposed rock arch rapids would bring the channel from the existing grade down to the refusal surface and natural waterfalls.

With-project model elevations for the waterfalls are based on the estimated refusal surface, except for the channel stretch between the Winter Street Bridge and the dam, which was not included in the refusal surface. The model elevations for this stretch are estimated from the structure plans and old photographs. An inline structure was added upstream of the gorge, at a relatively larger drop seen in the estimated refusal surface, that roughly aligns with documentation of an old mill, Foster's Mill, at the location (see Figure 21).

Figure 21: Historical Photo and Location of Fosters Mill (Reference 40)

8.2.3 Removal of Powell Falls Dam

In the Lake Louise area, the lake had already been drained through the gate in the dam, and the river meandered accordingly through the lakebed. Although, some of the banks are still actively eroding. It was generally assumed that the main channel bottom was the refusal surface. The proposed channel alignment followed this existing alignment.

There are two exposed pipes that run across the channel. The most downstream of these, running to the wastewater treatment plant, is currently acting as a small weir within the channel. The design placed the rock arch rapids downstream of this pipe to protect it and provide fish passage. The riffle (riffle 4) just upstream of the rapids extends over the pipe. The other pipe is protected by the other riffle (riffle 3).

9 Project In-Channel Feature Design

Project feature placement and design was a result of following the basic channel design objectives (channel width and floodplain capacity), discussed in Section 8.2.1.2, the basic feature design parameters, discussed in this section, and achieving model velocity results compatible with the project objectives, discussed in Section 8.2.1.3, while also maintaining a manageable velocity for the 1% AEP (reasonable rock sizing). Features were located and shaped within the model to meet all of these objectives. Key parameters include channel/feature slopes, placement, and elevation inverts. Many model geometry variations were run in an attempt to meet the combined objectives.

Feature dimensions were estimated from the model to be used for the typicals in the plan set. Many of the features still need refinement and are only roughly incorporated in the 1D model. In the design phase, simplification of feature geometries and consolidation of gradations should be considered, as much as practicable, to simply construction.

9.1.1 Proposed In-stream Features

Features were proposed and placed to meet the design objectives. In conjunction with the removal of Junction Falls Dam, the proposed design includes one rock arch rapids feature, two riffles, and two cross-vanes, with pools and lunker structures in between. In conjunction with the removal of Powell Falls Dam, the proposed design includes one rock arch rapids feature, a cobble apron, two riffles, and a cross-vane with a step, with pools and lunkers structures in between.

9.1.1.1 Rock Arch Rapids

Rock Arch Rapids consist of a rock ramp with boulder weirs in a step configuration. They are designed to act as a grade control structure while also providing fish passage through a sloped section. Boulders are strategically placed to dissipate energy and direct flow towards the center of the channel. The stepped weirs create pools for fish passage. Conceptual feature layouts, used as a reference, are shown in Figure 22 and Figure 23 (Reference 41). However, the actual design applied to this project may not follow these conceptual plans exactly.

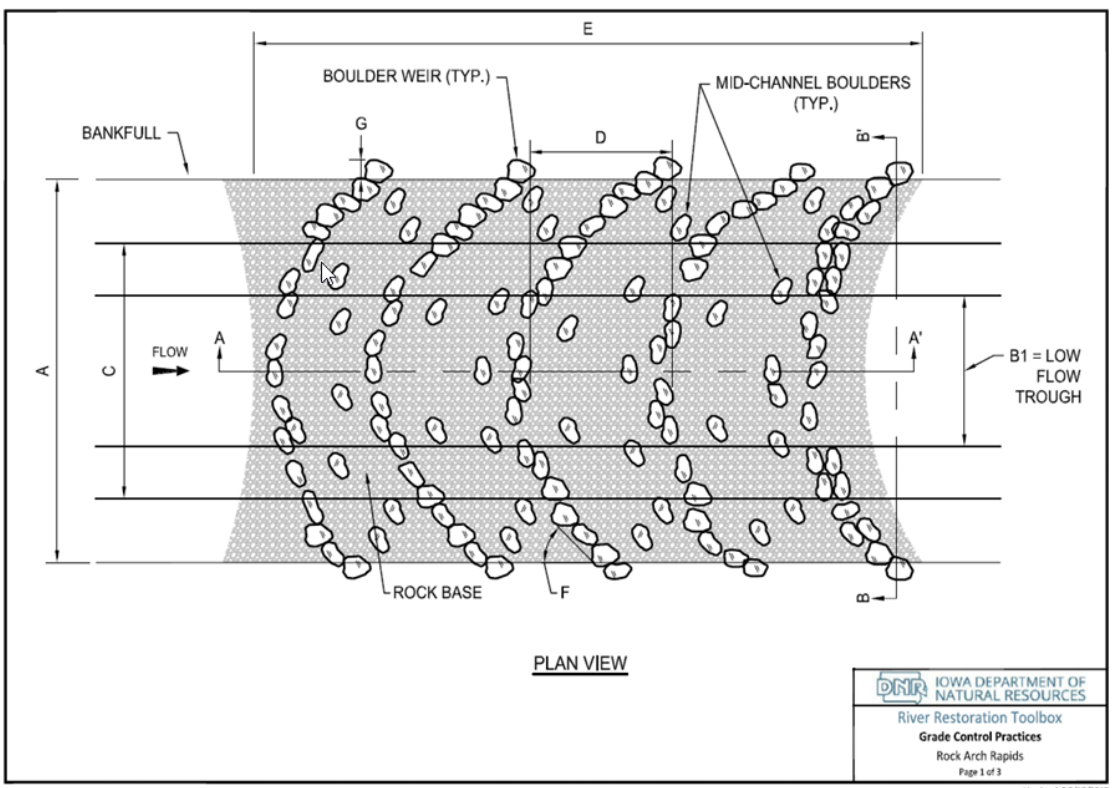

Figure 22: Rock Arch Rapids Concept Plan View (Refence 41)

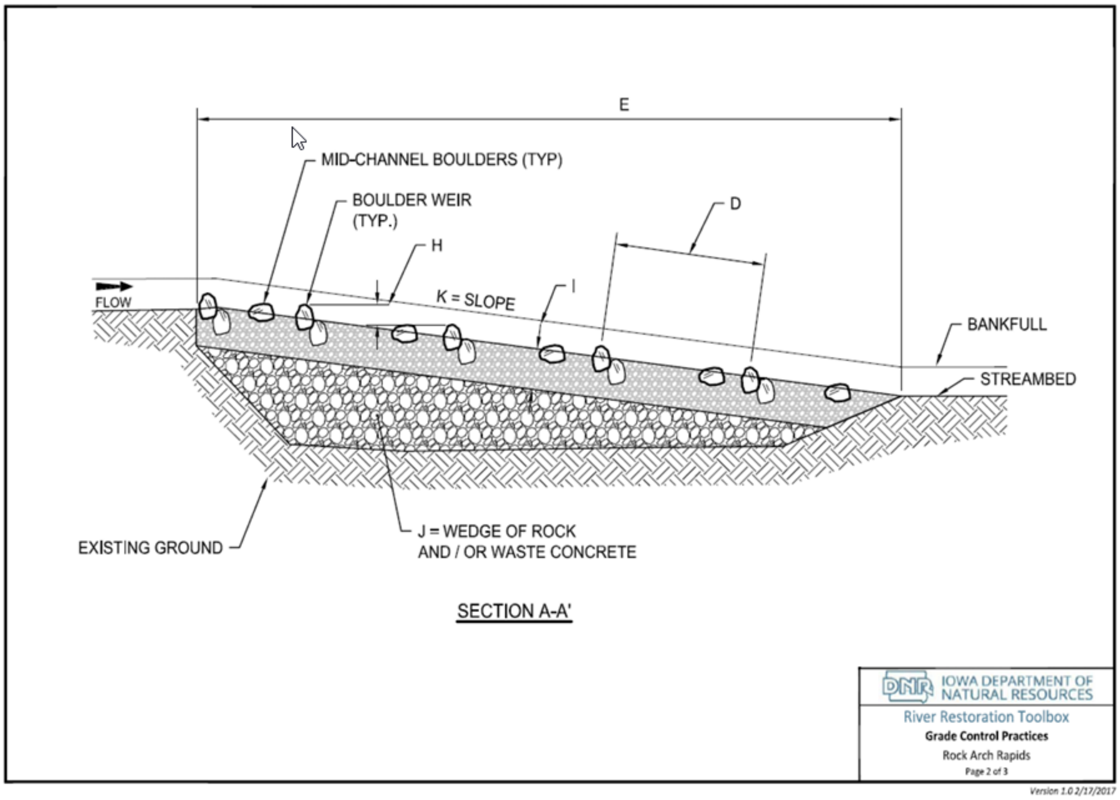

Figure 23: Rock Arch Rapids Concept Profile View (Reference 41)

For this project, the rock arch rapids include 0.5 ft steps at approximately a 2% slope. The rock arch rapids were modelled as a series of steps, using an inline structure with a 0.5 ft drop at each step. Table 43 shows key elevation inverts used in the modeling. The manning's n value through the feature is set to 0.05. Rock was sized for the maximum velocity throughout the rapids for the 1% AEP event. The velocities for the low flow event range from 2.0-4.3 ft/s. The details of the boulder spacing and low flow path will need to be finalized in the design phase and updated in the modelling. Strategic boulder placement could provide a greater range of velocities. A rough design based on the current model is included in the plan set.

Table 43: Rock Arch Rapids Key Elevation Inverts

Feature	Upstream Invert (ft NAVD88)	Downstream Invert (ft NAVD88)	Number of Steps
Rock Arch Rapids 1	861.0	856.5	12
Rock Arch Rapids 2	816.86	812.36	9

9.1.1.2 Cross-Vane

Cross-vanes consist of a rock weir that spans the channel in a u-shape. They are used to establish grade control and direct flow towards the center of the channel, creating a pool. A step is included in one of the cross-vanes. This step breaks up the elevation drop of the structure and improves fish passage. The edges of the structure are keyed into the bank, and a filter fabric may be used to prevent scour. Conceptual feature layouts, used as a reference, are shown in Figure 24 and Figure 25 (Reference 41). The reference figures show a cross-vane with a step. However, the actual design applied to this project may not follow these conceptual plans exactly.

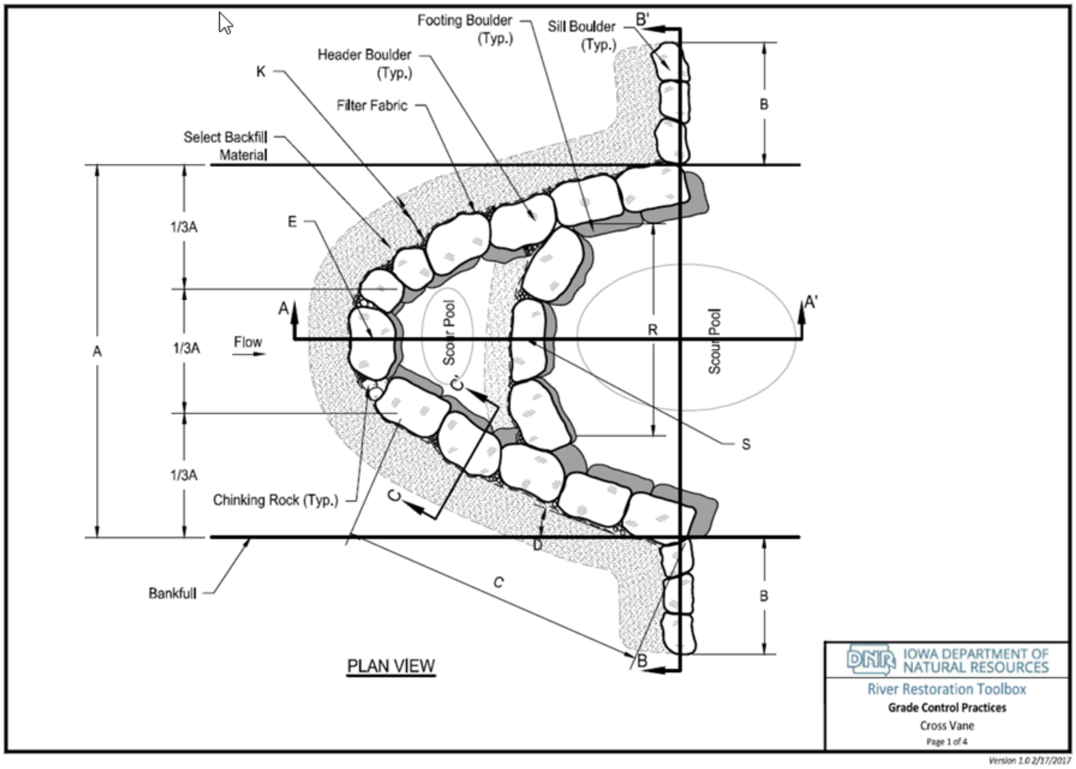

Figure 24: Cross-Vane with Step Concept Plan View (Reference 41)

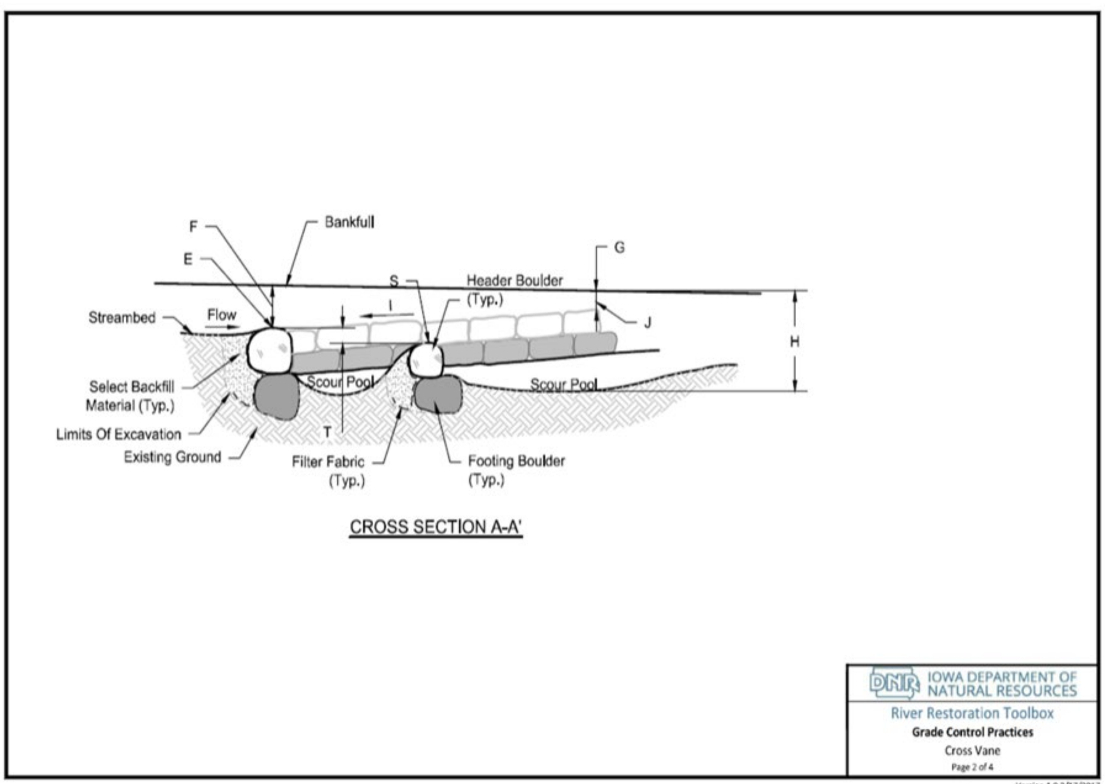

Figure 25: Cross-vane with Step Concept Profile View (Reference 41)

The cross-vanes above Junction Falls were modelled with a series of five cross-sections to capture the boulders in the channel. The first cross-section includes the center of the upstream weir, followed by three cross-sections with a widening notch for the scour within the vane, with one final cross-section at the target channel elevation. The structure drops the channel elevation 0.5 ft. Table 44 shows key elevation inverts used in the modeling. An assessment of sediment movement should be considered in the design phase, as these elevations are estimated. The model uses a manning's n of 0.05 for these features. These structures were designed for the target velocities (1.3-2.3 ft/s) at low flow. Rock was sized for the maximum velocity for the 1% AEP event. A rough design based on the current model is included in the plan set.

Table 44: Cross-Vane Key Elevation Inverts

Feature	Upstream Invert (ft NAVD88)	Downstream Invert (ft NAVD88)
Cross-vane 1	862.8	862.3
Cross-vane 2	862.05	861.56

The cross-vane above Powell Falls is a cross-vane with a step. There is already a drop-off at this location in the existing conditions. The area has already been exposed to high velocities and the channel carved as sediment washed out when Lake Louise was drained. The channel here is already expected to have reached the refusal surface. Additionally, as part of the proposed conditions, the area on the right bank is intended to be washed out down to the refusal surface, to provide more cross-sectional area for the 1% AEP. The objective of the cross-vane with the step is to make the section passable to fish. Due to model stability issues and time and budget constraints, this one was not modeled in as much detail as the others. It is represented by three cross-sections. The first includes the center of the upstream boulder weir to capture the invert. The second cross-section captures the invert of the step with raised sides to represent the outside arms of the vane. The third is used to capture the downstream invert. The structure drops the channel elevation 1.9 ft total, 1.1 ft to the step and another 0.8 ft to the bottom. This is larger than typically recommended for a fish passage feature. However due to the tailwater, the water surface profile does not drop more than 0.5 ft at each interval, in low flow conditions. Revisions to the design, such as multiple structures or a small rock arch rapids feature, should be considered in the design phase, if the head differential is deemed too large. Table 45 shows key elevation inverts used in the modeling. The model uses a manning's n of 0.05 for this feature. The structure was designed for the target velocities at low flow. Rock was sized for the maximum velocity for the 1% AEP event. Further design work is needed to confirm the exact layout and fit into the proposed channel and how the structure will tie-in. A rough design based on the current model is included in the plan set.

Table 45: Cross-Vane with Step Key Elevation Inverts

Feature	Upstream Invert (ft NAVD88)	Step Invert (ft NAVD88)	Downstream Invert (ft NAVD88)
Cross-vane with step	810.91	809.80	809.00

9.1.1.3 Rock Riffle

Rock Riffles are used in the design upstream of the rock arch rapids and as grade control for structure protection. They also provide spawning habitat for trout (Reference 41). These structures were designed for the target velocities (1.3-2.3 ft/s) at low flow and as a result, have very mild slopes. They were modelled with a series of cross-sections to capture, at a minimum, the upstream and downstream cross-sections and associated channel inverts. Table 46 shows key elevation inverts used in the modeling. Rock was sized for the maximum velocity throughout the rapids for the 1% AEP event. Conceptual feature layouts, used as a reference, are shown in Figure 26. However, the actual design applied to this project may not follow these conceptual plans exactly. A rough design based on the current model is included in the feasibility plan set.

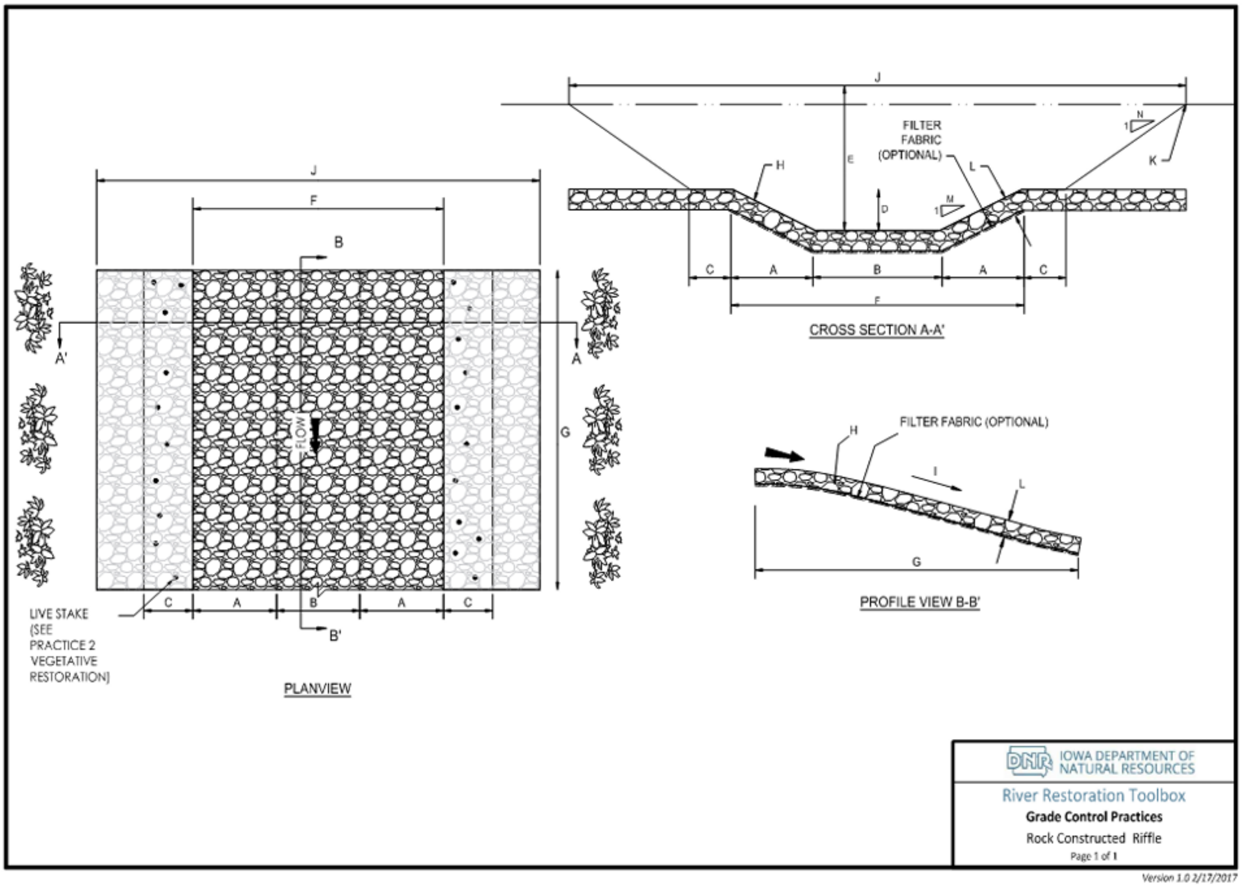

Figure 26: Riffle Concept View (Reference 41)

Table 46: Riffle Key Elevation Inverts

Feature	Upstream Invert (ft NAVD88)	Slope
Riffle 1	863.15	1.5%
Riffle 2	862.5	0.15%
Riffle 3	817.77	0.2%
Riffle 4	817.45	0.8%

9.1.1.4 Cobble Apron

The cobble apron is rounded rock placed downstream of the rock arch rapids in conjunction with the removal of Powell Falls Dam. It runs for roughly 200 ft. The feature is to protect the riverbed from erosion and from undermining the rapids. Velocities coming out of the rapids during larger events are high enough to potentially erode the sand bed.

9.1.1.5 LUNKERS Structures

LUNKERS (Little Underwater Neighborhood Keepers Encompassing Rheotactic Salmoids) are structures installed along banks to provide cover for fish habitat. They are essentially protected slots along the bank. They were originally developed for trout. They also provide bank stabilization. They should be built at an elevation low enough to be accessible during low flows. They can be prone to sedimentation and should be placed strategically (Reference 42). As part of this project, they are proposed to improve the habitat benefits. They can be built with lumber and boulders, but for this project, the intention is to build them and shape the openings solely with rock, for longevity. The design involves shaping the openings with boulders along the bank edge, with a layer of riprap embedded into the bank behind it. Figure 27 shows the conceptual design for the feature. For this feasibility effort, due to time and cost constraints, the lunkers structure design was not assessed in depth, and the quantity is a very rough estimate of what may fit along the streambanks. Further assessment should be done on the design and strategic placement of these to maximize benefits and avoid sedimentation.

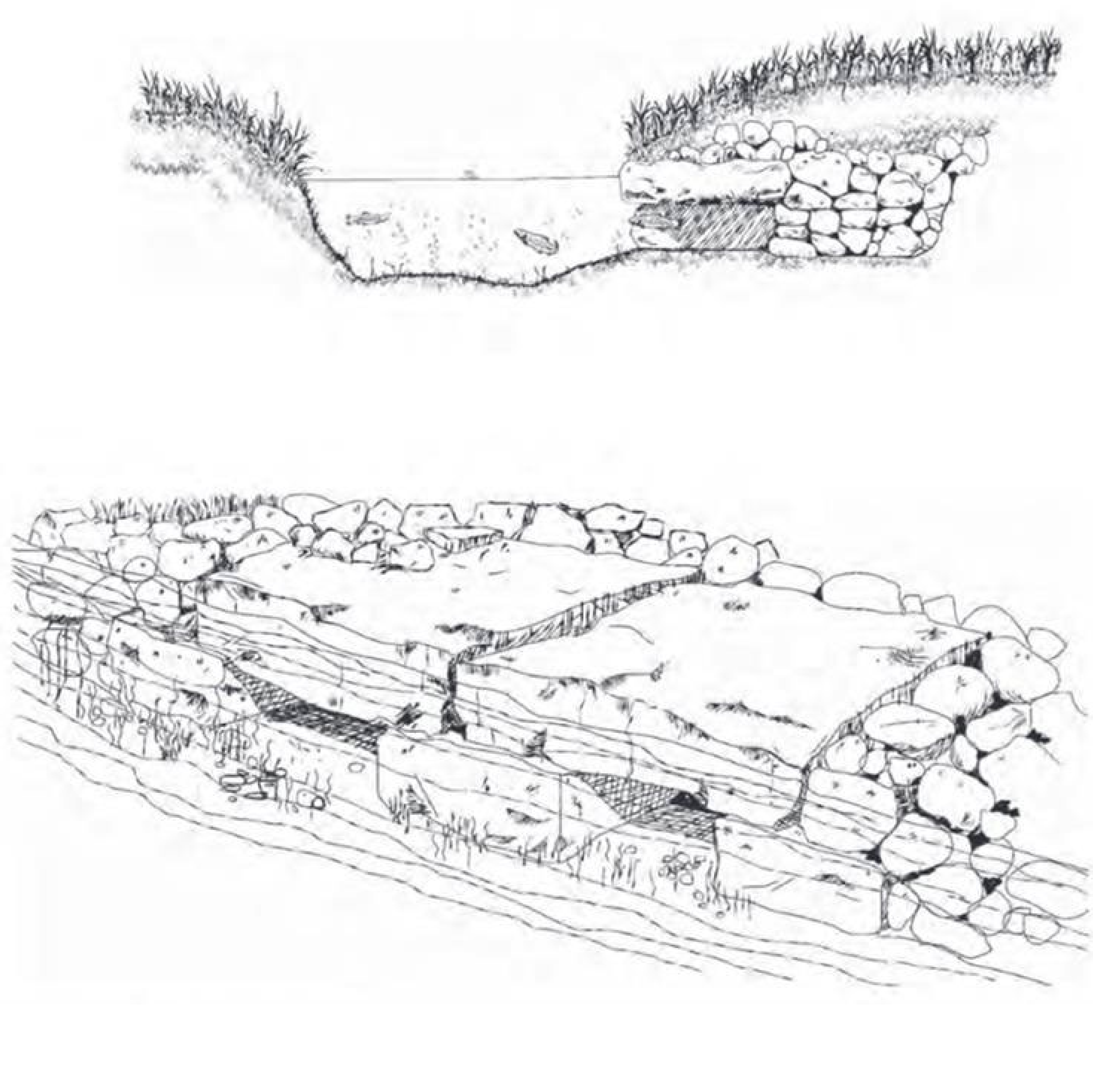

Figure 27: Lunkers Structure Concept (Reference 43)

9.1.1.6 Summary of In-stream Feature Selection and Placement

The design associated with the removal of Junction Falls Dam includes two riffles, two cross-vanes, one rock arch rapids, and several lunkers structures. The model profile plot is shown in Figure 28 for reference. Due to the concern for head-cutting and erosion through town and at the upstream pedestrian bridge, the design intent is to hold the existing grade just downstream of the pedestrian bridge. Riffle 1 was placed here as a fish habitat feature, but also to protect the bridge. A rock arch rapids would be needed to maintain fish passage and bring the channel down to the elevation refusal surface at the waterfalls. However, the area (channel width) between the bridge and the existing lake is relatively constrained. Placing the rock arch rapids within the constrained area would cause high velocities during larger events, such as the 1% AEP, that would require a larger rock size. Instead, the rock arch rapids was designed in the lake area, where there was enough space to allow for a larger floodplain and reduce velocities for the 1% AEP. Riffle 2, just upstream of the rock arch rapids, generally matches with the existing cross-section inverts there. There does appear to be a natural rock riffle there in existing conditions. Between Riffle 1 and Riffle 2 are two cross-vanes. The placement of these is flexible. The intention is to provide sufficient space between the riffles and each of these features for pools. The velocities within these features (targets for low flows and maximums for 1% AEP) were fairly sensitive to the placement and elevation inverts. Further assessment should be done on the erosion and pool formation details surrounding these. The exact placement of the lunkers structures also needs further assessment.

The design associated with the removal of Powell Falls Dam includes two riffles, a rock arch rapids, a cross-vane with a step, and several lunkers structures. The model profile plot is shown in Figure 29 for reference. Riffle 3 is located where the channel narrows from the pool downstream of the falls. There is a pipe here that is to be protected. Downstream the existing grade drops abruptly at a pipe-crossing to the wastewater treatment plant. Riffle 4 was placed over this pipe for protection, but a rock arch rapids is required to maintain fish passage and bring the channel down to grade. Similar to the problems in the Lake George area, the rock arch rapids needed to be placed further downstream in order to gain more floodplain for the 1% AEP to reduce velocities. There are also trees on the banks at the pipe-crossing (Riffle 4). The Riffle 4 and rock arch rapids cross-section widths are constrained. The marsh habitat and infiltration basin location on the left needs to be separated by high ground from the floodplain bench that floods during a ~50% AEP. The bench width through the rock arch rapids is currently design to extend about as far as it can without impeding on this area. The cross-vane with a step was placed where it is due to the drop in the existing channel that is already believed to have eroded to bedrock. More consideration for the design of this feature, the modeling, and how it will be keyed into the bank should be considered in the design phase.

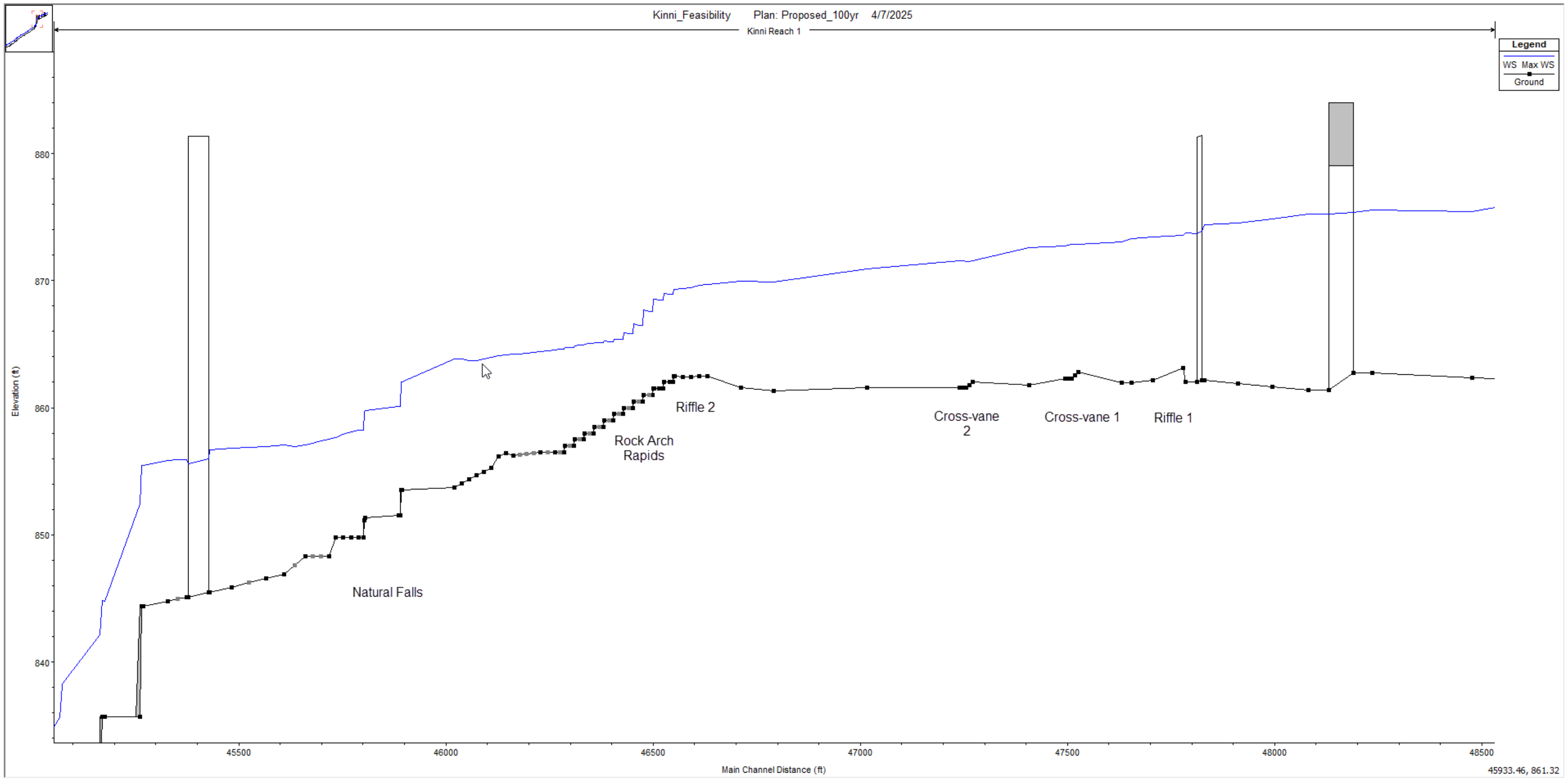

Figure 28: Removal of Junction Falls Dam Profile Plot

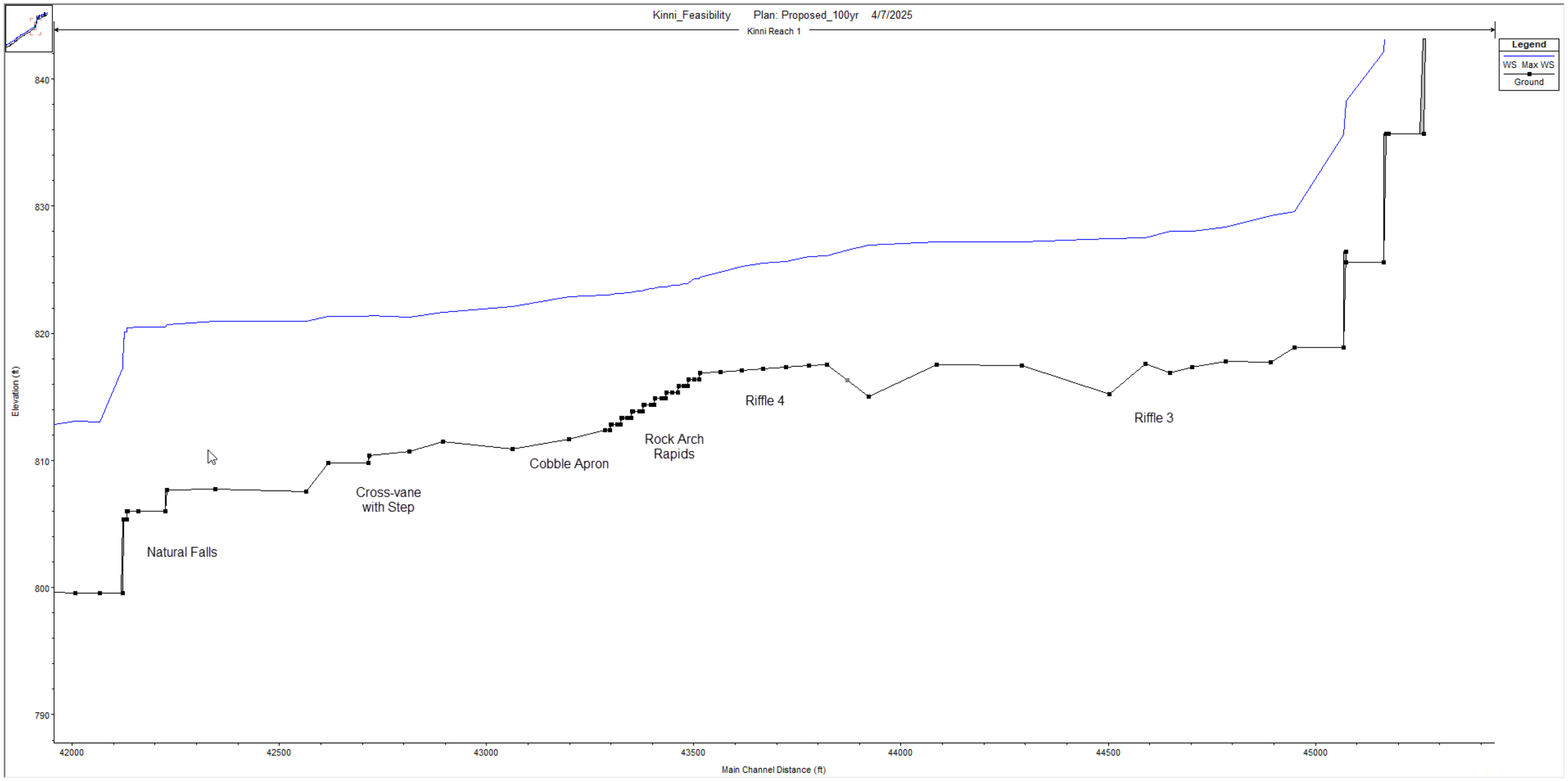

Figure 29: Removal of Powell Falls Dam Profile Plot

9.2 In-stream Feature Rock Sizing

Rock for the riffles, bank protection, and lunker structures was sized using the hydraulic design tool in HEC-RAS, aligning with guidance in EM 1110-2-1601 (Reference 44). Table 47 shows a summary of the model results at these features and selected rock sizing. Refer to the screenshots in Attachment E-1 to see the assumptions used for each cross-section. Base rock for the cross-vanes and rock arch rapids was sized based on peak model velocities, using the Isbash equation, assuming high turbulent flow (Reference 45). Table 48 shows a summary of the model results at these features and selected rock sizing.

Several gradations of riprap and bedding were included in the proposed design. After the design is refined, the PDT should consider consolidation. Rounded rock was used in fish habitat features, where possible. Angular rock was used for bank protection and where 1% AEP velocities were large enough to require angular R270. The rock for the features needed to be small enough that larger boulders could be utilized to shape the features. Boulders were assumed to be roughly 3.5 ft x 3 ft x 1.5 ft, but this will depend on refinement of the feature design and local availability. Additional bedding was included in the quantities for chinking within the larger rock.

In general, a factor of safety of 1.2 was the target, considering 1% AEP velocities. Most features meet or exceed this. Riffle 4 only has a factor of safety of 1.0, but this is under the assumption that the riffle is part of a curve, and the critical section is on the outer bank. It is recommended to further assess that curve in the design phase. It may be able to be straightened out more. Alternatively, the riprap size could be increased. The cross-vane in Lake Louise has a factor of safety of 1.0 and the rock arch rapids has a factor of safety of 1.1, due to the high peak velocity at some part of the structure. However, it would be impractical to use rock above gradation R270 and would require the boulders to be significantly larger. The lower factor of safety was tolerated for practicability, but also because the project is a habitat project with relatively lower consequence of failure. The velocity may also be reduced with more detailed modeling and shaping of the features in the design phase. The lunkers structure rock upstream of Junction Falls has a factor of safety of 1.1, but it is maximized at R270 and is being used for fish habitat with lower consequence of failure. Further assessment on the placement of the structures is to be done in design phase, so strategic placement may avoid this critical velocity.

Table 47: Rock Sizing Summary for Riffles, Bank Protection, and Lunker Structures.

Removal of Junction Falls Dam					
Feature	**Critical Model Cross-section**	**Critical 1% AEP Avg Velocity (ft/s)**	**Factor of Safety (1% AEP**	**Rock Type**	**Selected Rock Gradation**
Riffle 1	44240	8.9	1.7	Rounded	R45
Riffle 2	43361.9	8.3	1.7	Rounded	R45
Bank Protection (right bank downstream of rock arch rapids)	42887.4	7.8	1.6	Angular	R20
Lunker Structures	43585.3	9.7	1.1	Angular	R270
Removal of Powell Falls Dam					

Feature	Critical Model Cross-section	Critical 1% AEP Avg Velocity (ft/s)	Factor of Safety (1% AEP	Rock Type	Selected Rock Gradation
Riffle 3	41608	11.8	1.3	Rounded	R140
Riffle 4	40662	8.4	1.3	Rounded	R45
Cobble Apron	40154.8	6.1	3.0	Rounded	R20**
Bank Protection (cobble apron)	40154.8	6.1	3.0	Angular	R20
Bank Protection (left bank, downstream curve of cobble apron)	40154.8	6.1	1.9	Angular	R20
Bank Protection (left bank, through meander)	39902.3	4.1	1.5	Angular	R45
Lunker Structures	41146	6.9	1.2	Angular	R140

Table 48: Rock Sizing Summary for Cross Vanes and Rock Arch Rapids.

Removal of Junction Falls Dam					
Feature	**Critical Model Cross-section**	**Critical 1% AEP Avg Velocity (ft/s)**	**Factor of Safety (1% AEP**	**Rock Type**	**Selected Rock Gradation**
Cross Vane 1	43993.9	8.7	1.2	Angular	R270
Cross Vane 2	43749.8	10.7	1.0	Angular	R270
Rock Arch Rapids	43260.9	9.2	1.2	Angular	R270
Removal of Powell Falls Dam					
Feature	**Critical Model Cross-section**	**Critical 1% AEP Avg Velocity (ft/s)**	**Factor of Safety (1% AEP**	**Rock Type**	**Selected Rock Gradation**
Cross Vane with Step	39915.6	6.1	1.2	Rounded	R80
Rock Arch Rapids	40265.4	9.5	1.1	Rounded	R270

9.3 Quantity Calculations for In-channel Rock Features

Due to time and cost constraints, the rock features were not modeled in the Civil surface, and quantities were roughly estimated. Dimensions were based on what is in the HEC-RAS model. These dimensions were provided to Civil for inclusion of a typical in the plan set during PED. These quantities will all need to be refined in the design phase as the geometries of these features are refined and modeled by Civil. Table 49 and Table 50 show a summary of the calculations.

Riffle quantities were calculated based on a sheet of riprap and bedding multiplied by the thickness of the layer. The length is length of the sheet in the direction of flow, based on cross-section distances in the HEC-RAS model. The width is based on the channel bottom width, plus the distance extending up each side slope of the cross-section. The side slope distance was determined based on the main channel section in the model, where the 1% AEP velocities are high.

Cross vane riprap and bedding quantities were calculated based on a sheet, similar to the riffles. The riprap and bedding were estimated to extend from the start of the feature to the end of the feature and extend up the side slopes of the main channel portion of the cross-section. The geotextile quantity was estimated from the same dimensions. Boulder quantities were calculated under the assumption that 3.5 ft boulders would be used to cover the channel width and tie into the slopes. In actuality, they may vary in size.

Rock Arch Rapids riprap and bedding quantities were calculated assuming a rectangular shaped step, multiplied by the thickness of the layer and by the number of steps in the rock arch rapids. Similar to the riffle and cross-vane, the width is based on the channel bottom width, plus the distance extending up each side slope of the cross-section. Boulder quantities were calculated under the assumption that 3.5 ft boulders would be used to cover the channel width at each step. In actuality, they may vary in size. There will be gaps and more strategic placement.

Lunkers structures riprap quantities were calculated with the assumption that the structures would be 4 ft tall by 8 ft in length. The riprap would cover that area at the thickness required for the riprap type. Boulder quantities estimated that each structure would require 8 boulder (3 ft x 3.5 ft x 1.5 ft). The quantities are based on the assumption that there would be 33 lunkers structures upstream of Junction Falls and 42 structures in the area downstream. These were very roughly estimated based on sample structure designs and the number of structures that would fit within a reach length. Refinement on the design and placement will be needed in the design phase.

Table 49: Cost Calculation Summary for Removal of Junction Falls Dam

Removal of Junction Falls Dam (Alternative 2)							
Feature	**Dimensions**	**Calculated Area (sq. ft)**	**Layer Thickness (in)**	**Rock Quantity (cu. yd)**	**Bedding Gradation**	**Bedding Thickness (in)**	**Bedding Quantity (cu. yd)**
Riffle 1	78'L x 85'W	6630	16	327	B2	15*	307
Cross Vane 1	40' L x 120' W	4800	45	667	B3	18*	267
Cross Vane 2	40' L x 120' W	4800	45	667	B3	18*	267
Boulders for Cross Vanes	a row (weir) ~130'L x2 20+44*2 +10 on each side	910	18	51			

Geotextile Fabric (non-woven) for cross vanes	40' L x 120' W	9600					
Riffle 2	92'L x 150' W	13800	16	681	B2	15*	639
Rock Arch Rapids	(25' L x 110' W) per step x 12 steps	33000	45	4583	B3	18*	1833
Boulders for Rock Arch Rapids	a row 110'W x 12 steps	9240	18	513			
Bank Protection downstream of rapids up to falls (right bank)	375' L x 40' W	15000	12	556	B1	6	278
Riprap for Lunker Structures	8'L x 4' H x 33 lunker structures	1056	30	98			
Boulders for Lunker Structures	8 per lunker x33	2772	18	154			

*Extra 6 inches of bedding added for chinking

Table 50: Cost Calculation Summary for Removal of Powell Falls Dam

Removal of Powell Falls Dam (Alternative 3)							
Feature	**Dimensions**	**Calculated Area (sq. ft)**	**Layer Thickness (in)**	**Rock Quantity (cu. yd)**	**Bedding Gradation**	**Bedding Layer thickness (in)**	**Bedding Quantity (cu. yd)**
Riffle 1	169.4' L x 120' W	20400	24	1511	B2	15*	944
Riffle 2	255' L x 110' W	28050	16	1385	B2	15*	1299
Rock Arch Rapids	(30' L x 120' W) per step x 9 steps	32400	45	4500	B3	18*	1800
Boulders for rock arch rapids	a row 120'W x 9 steps	7560	18	420			

Cobble Apron	200'L x80'W	16000	12	593	B1	12*	593
Bank Protection at Cobble Apron (right bank)	200'L x11'W	2200	12	81	B1	6	41
Bank Protection at Cobble Apron (downstream curve - left bank)	220'Lx35'W	7700	12	285	B1	6	143
Bank Protection through Meander (left bank)	290'Lx40'W	11600	16	573	B2	9	322
Cross Vane with Step	80' L x 180' W	14400	30	1333	B2	15*	667
Boulders for Cross Vane	a row (weir) ~260'L 33.33+86.66 *2+206 +66.5 step + 10 for side	990.5	18	55			
Geotextile Fabric (non-woven) for Cross Vane	80' L x 180' W	14400					
Lunker Structures	8'L x 4' H per structure x 42 structures	1344	24	100			
Boulders for Lunker Structures	8 per structure x 42	3528	18	196			

*Extra 6 inches of bedding added for chinking

9.4 Flood Stage Impacts

The current FEMA 1% AEP flood stages are based on an HEC-2 model with both dams fully operational. The flows used are significantly lower than the updated flows used for this study (Table 36). The FEMA cross-sections were incorporated into the model. Comparisons between the published FEMA 1% AEP flood elevations, the existing conditions (corrected effective), and proposed project conditions are shown in Table 51.

Flood stage and duration impacts need to be considered as the project moves into the design phase. The Wisconsin DNR (WIDNR) has a no-rise requirement that would be exceeded by even a 0.01 ft increase in stage. The team should discuss the type of modeling (steady or unsteady) and stage impact and floodplain storage requirements with the WIDNR. The project will also require a Conditional Letter of Map Revision (CLOMR)/Letter of Map Revision (LOMR) to be processed through FEMA, because of updates to the hydrology and changes within the floodway. Early coordination is critical during design to ensure agreement with hydrology and approach for tie-ins.

Comparing the existing (corrected effective) to proposed model runs, the cross-sections downstream of Powell Falls are seeing 0.01-0.02 ft of stage increases for the 1% AEP event and more frequent events. Additional survey data will be collected, and model refinements will be made during the design phase. The team may also modify stream or hydraulic design features. Design and model refinement is anticipated to ensure that the project does not cause a rise in water levels for the 1% AEP.

Further assessment is needed on the impacts of sediment removal/movement. The intent is to keep the excavated sediment on site and place it elsewhere in the lakebed, but this has not been modelled. Stage and velocity impacts of this should be evaluated in the design phase. There are areas in the floodplain that are not within the inundation extents of the 1% AEP. If the sediment fits within these areas, placement of sediment there is not expected to cause stage impacts. Alternatively, the team may consider hauling excess sediment off-site or adding attenuation areas for flood storage. There is also the potential for sediment to be washed downstream during the removal of the dam. A sediment analysis would indicate where the sediment might land downstream. This should also be considered when assessing stage impacts. However, the team intends to minimize downstream sediment mobilization as much as possible.

Table 51. Cross-section 1% AEP Stage Comparison

Location		FEMA Effective (ft. NAVD88)	Corrected Effective (ft. NAVD88)	Proposed (ft. NAVD88)
55109C0363E	G	901.1	901.3	901.3
	F	899.5	899.5	899.5
55109C0501E	E	891.1	890.9	890.9
55109C0482E	D	890.2	890.3	890.3
	C	888.8	889.0	889.0
	B	884.4	883.1	883.1
55109C0501E	A	880.6	879.7	879.6
55093C0107E	E	873.0	874.3	873.1
	D	872.4	873.3	869.9
	Junction Falls Dam Location	867.0	872.8	855.5
	Powell Falls Dam Location	822.0	830.1	819.6
	C	815.7	813.0	813.0
	B	808.9	806.8	806.8

	A	807.4	804.8	804.8

10 Detention Basins Design

Detention basins are vegetated depressions that are designed to capture stormwater runoff and reduce the intensity of peak flows from existing stormwater outfalls. Four existing stormwater outfalls from the City's storm sewer system currently discharge directly into Lake George and Lake Louise, as shown in Figure 30. Upon removal of Junction Falls and Powell Falls dams, these outfalls will remain in place. To prevent unmitigated erosion to the newly constructed features within the project area, detention basins are recommended for inclusion to capture the discharge from these existing outfalls. The primary function of the detention basins is to capture flow from the outfalls, completely reducing the outflow velocities, and eliminating erosive forces. A secondary benefit will be to create ephemeral wetland habitat within the proposed riparian habitat. The City of River Falls requested that these basins be designed to the 10-year 24-hour event for pipes 24" and smaller and designed to the 25-year 24-hour event for pipes larger than 24". This request will be verified in the design phase.

10.1 Drainage Area Delineation

Four drainage areas were delineated to each of the four outfalls (013, 019, 023, and 061), shown in Figure 30. Drainage areas showing CNs based on land use and soil classification.Figure 30. The land use types and soil classifications of the drainage areas are also shown in Figure 30. Land use data was obtained from the 2020 National Land Cover Database (NLCD) data (Reference 28). Soil classification data was obtained from the Web Soil Survey (WSS) produced by the US Department of Agriculture Natural Resources Conservation Service (USDA NRCS) (Reference 19).

The drainage areas discharge flow into either Lake George or Lake Louise. The drainage areas were delineated using the digital elevation model (DEM) of the study area and the storm sewer network data provided by the City of River Falls. The drainage areas include runoff that is captured by the storm sewer system. These drainage areas are used in the volume calculations and culvert sizing.

10.2 Curve Number Determination

A curve number (CN) dataset was created for the study area, using the land use and soils classification data, and the information in Reference 47. A composite curve number was determined for each drainage area using area-weighted average of the curve numbers within the drainage area.

The weighted CN for each drainage area is shown in Table 52.

Table 52. Delineated Drainage Area and Curve Number (CN)

Outfall #	Pipe Size	Drainage Area [acres]	CN_w
061	24"	26.4	57
019	24"	2.5	82
013	15"	5.6	66
023	48"	21.2	88

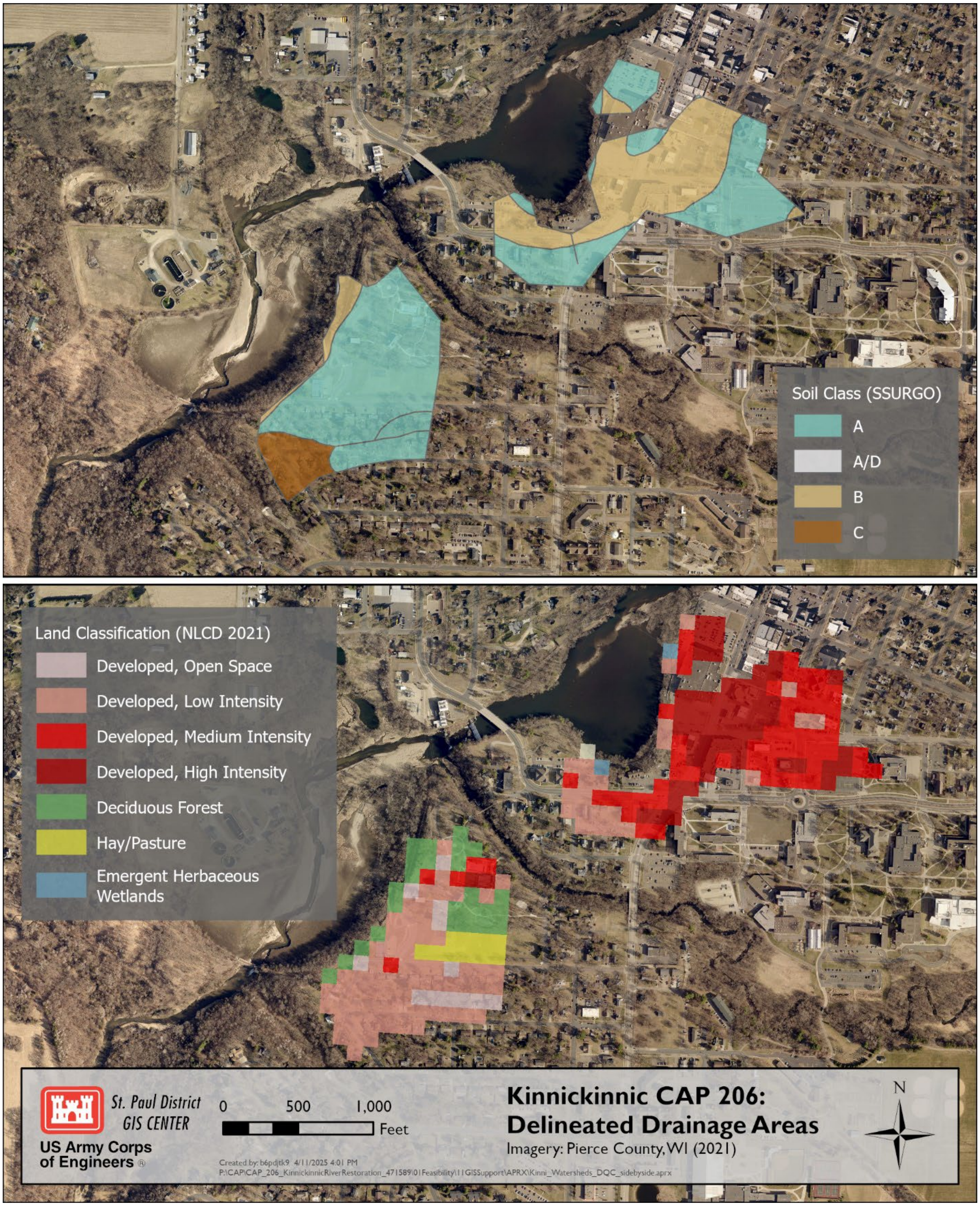

Figure 30. Drainage areas showing CNs based on land use and soil classification.

10.3 Volume of Runoff

The SCS Curve Number Loss Model was used to estimate the total excess volume of runoff for each drainage area (Reference 49). The composite CN for each drainage basin was used in Equation 3 to determine the maximum retention, S.

$$S = \frac{(1000 - 10CN)}{CN}\ (foot - pound\ system); where\ CN = curve\ number$$

Equation 3. Maximum Retention, S

The rainfall event depth, P, was determined using the Altas 14 Precipitation Depths for a 24-hour rain event at River Falls, WI. In accordance with the City's request, the 10-year 24-hour storm event was selected for pipes less than or equal to 24" and the 25-year 24-hour storm event was selected for pipes larger than 24. Calculations were done for each outfall and the value of P reflected the pipe size of each outfall. The difference in storm event based on pipe size is to accommodate for the pipe capacity. S and P can be plugged into Equation 4 to determine the excess precipitation, P_e.

$$P_e = \frac{(P - 0.2 * S)^2}{(P + 0.8S)};\ \ where\ P = accumulated\ rainfall\ depth\ at\ time, t$$

Equation 4. Excess Precipitation, P_e

P_e was multiplied by the acreage of the drainage area, A, to determine the total volume of flow that the basin will be able to accommodate, V_{wq}.

$$V_{wq} = P_e * A;$$

$$where\ A = total\ acreage\ of\ the\ watershed, P_e = accumulated\ rainfall\ depth, inches$$

Equation 5. Volume of Flow, V_{wq}

The resulting volume for each basin is included in Table 53.

Table 53. Volume of Flow (V_{wq}) Quantities for each Detention Basin

Outfall #	Pipe Size	Design Storm Event	S	P [in]	Pe [in]	V_{wq} [in-acre]
061	24"	10-year	7.54	4.2	0.71	18.7
019	24"	10-year	2.20	4.2	2.37	6.0
013	15"	10-year	5.15	4.2	1.21	6.7
023	48"	25-year	1.36	5.28	3.94	83.5

10.4 Field Infiltration Testing

Field infiltration testing was done in Lake Louise, using a double ring infiltrometer, to determine a design infiltration rate and compare field values against values from literature. The test results are shown in Table 54. Figure 31 shows the test locations.

At each test site, three tests were done to compare values. Prior to any recorded tests, water was allowed to filter through the soil to saturate it. Even with this prior saturation, the infiltration rates were often more consistent for the final two tests that were done at each site.

While the test samples were taken in Lake Louise, it is assumed that these soils are also reflective of the soils that are in Lake George. A Sediment Assessment Study done by Inter-Fluve, Inc (Reference 24) assessed the sediment quantity and quality in Lakes George and Louise. From the findings in the report, the soils in each lake are expected to be similar.

Figure 31. Location of Infiltration Test Sites

Table 54. Infiltration Rates Calculated from Field Collection

Infiltration Rates, I							
Test #	**Location**	**Site #**	**Time**	**Starting Depth**	**Ending Depth**	**Elapsed Time**	**Infiltration Rate [in/hr]**
1	44.8531571, -92.6372123	1	9:20a	2-5/8”	0”	5 min 20.59 sec	29.48
2	44.8531976, -92.6371663	2	9:40a	2-3/16”	0	7 min 30.58 sec	17.48
3			10:00a	2-1/4”	0”	9 min 30.73 sec	14.19
4			10:20a	2-3/8”	0”	9 min 53.13 sec	14.42
5	44.8531341, -92.6366252	3	10:40a	3-1/4”	0”	7 min 10.01 sec	27.21
6			11:00a	2-1/4”	1-1/8”	15 min	4.50
7				2-1/4”	1-1/4”	15 min	4.00
8	44.8527132, -92.6364613	4	12:45p	1-5/8”	0”	7 min 13.80 sec	13.49
9			1:15p	1-5/8”	0”	10 min 42.60 sec	9.10
10			1:25p	1-5/8”	0”	11 min 59.21 sec	8.13
11				1-1/2”	0”	14 min 45.89 sec	6.10
12	44.8519747, -92.6374771	5	2:10p	2-3/4”	2”	15 min	3.00
13			2:25pm	2”	1-5/8”	15 min	1.50
14				2-5/8”	2-1/8”	15 min	2.00

The field infiltration rates collected vary greatly between test sites. One reason for this is likely due to the variability of the sediment in Lake Louise. Testing was done after Lake Louise had been drawn down, and the majority of testing was done on the upstream end of the lake. According to the Inter-Fluve study, much of the upstream end of the lake was made up of medium to coarse sands, which would explain the high infiltration rates from Sites 1-4. Site 5 was located in the more middle to main lakebed where soils are more representative of the soils deposited in Lake Louise and Lake George. The spread in infiltration rates across individual test sites is likely due to the soil not being fully saturated for the first test. The second two tests are more consistent for Sites 1-3 and 5.

Test Site #5 had the lowest infiltration rate, whereas Test Sites #1 and #2 had the highest. Test Site #5's infiltration rates most closely matched expected literature values, which ranged from 1.63in/hr to 0.06in/hr shown in Table 55 (Reference 46).

Table 55. Literature Infiltration Rates Based on Soil Classification (Reference 46).

Hydrologic Soil Group	Infiltration Rate [in/hr]	Soil Textures	Corresponding Unified Soil Classification
A	Field infiltration testing highly recommended.	gravel sandy gravel	GW - Well-graded gravels, fine to coarse gravel GP - Poorly graded gravel
	1.63	silty gravels gravelly sands sand	GM - Silty gravel SW - Well-graded sand, fine to coarse sand
	0.8	sand loamy sand sandy loam	SP - Poorly graded sand
B	0.45	silty sands	SM - Silty sand
	0.3	loam, silt loam	MH - Elastic silt
C	0.2	Sandy clay loam, silts	ML - Silt
D	0.06	clay loam silty clay loam sandy clay silty clay clay	GC - Clayey gravel SC - Clayey sand CL - Lean clay OL - Organic silt CH - Fat clay OH - Organic clay, organic silt

Table 56 compares literature rates to field collected rates. When using field infiltration rates, a safety factor of 2 is recommended (Reference 48). The soil in this area falls into hydrologic soil group A, which reduces the range of accepted literature values to 1.63-0.8 in/hr. According to the sediment size distributions in the Inter-Fluve report, it is anticipated that poorly graded sands better represent the lakebed soils in Lake George and Lake Louise. This supports the use of 0.8 in/hr according to literature. Due to the wide range of field collected infiltration rates, the literature rate of 0.8 in/hr was used to reduce uncertainty. This value is most conservative and aligns with the results of Inter-Fluve's study. Further soil testing throughout the basin is recommended during the design phase. For feasibility, an infiltration rate of 0.8 in/hr is assumed representative of the soils in the area.

Table 56. Comparison of Field vs. Literature Infiltration Rates

Hydrologic Soil Group	Literature Infiltration Rate [in/hr]	Range of Field Infiltration Rate		Infiltration Rate used in Design
		Collected [in/hr]	Safety Factor Applied [in/hr]	
A	1.63-0.8	29.5-1.5	14.75-0.75	0.8

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10.5 Detention Basin Design

Using the runoff volumes calculated in Section 10.3, the surface area of the detention basins, (A_s) was calculated using Equation 6. The ponding depth, D_0, of the basin was assumed to be 1.5 feet of ponding depth (Reference 48). As determined in Section 10.3, the infiltration rate of 0.8 in/hr was used in these calculations. This literature rate is more conservative than the field rates and removes human error and inconsistencies. Duration of the storm, t, was assumed to be 24 hours. This duration was based on the 24-storm event, as well as the discharges from the detention basins entering a trout stream (Reference 48).

$$A_s = \frac{V_{wq}}{(D_o + (I_R * t))}$$

Equation 6. Detention Basin Surface Area, A_s

$$where\ I_R = infiltration\ rate\ of\ soils\ \left(\frac{ft}{day}\right)$$

$$t = duration\ of\ the\ storm\ event, D_0 = ponding\ depth$$

The area of each basin is then used to estimate the earthwork quantities for the basin. A 3d trapezoidal prism was assumed to estimate the excavation volume. The surface area was assumed to be the midpoint of the prism, so that the prism could be simplified down to a rectangular prism. Equation 7 was used to calculate the excavation volume of the basin, V_{cb}. This V_{cb} corresponds to the earthwork quantities for each outfall. The depth of the basin was assumed to be the ponding depth, D_0, plus 1-foot of additional freeboard.

$$V_{cb} = A_s * d; where\ d\ is\ the\ depth\ of\ the\ catch\ basin$$

Equation 7. Volume of Detention Basin, V_{cb}

The resulting basin surface area and earthwork quantities for each outfall are summarized in Table 57. The basins are assumed to have 4:1 side slopes.

Table 57. Detention Basin Size and Quantities

Outfall #	Design Storm Event	Pipe Size	Basin Surface Area [ft^2]	Earthwork Quantities [yd^3]
061	10-year	24”	21,840	2,220
019	10-year	24”	7,028	720
013	10-year	15”	78,58	800
023	25-year	48”	97,758	9,960

10.6 Hydraulic Design

10.6.1 Peak Discharge

The City of River Falls requested that basins that are fed by 24” or smaller outlets would be designed for the 10-year storm event, whereas basins fed to by outlets larger than 24” would be designed for the 25-year storm event.

The USDA NRCS TR-55 worksheet was used to calculate the peak discharge for each outfall. This method assumes overland flow only and does not account for pipe conveyance. The peak discharge was calculated based on the corresponding storm events, along with the CNs calculated in Section 10.2. The longest flow paths and time of concentration were determined for each drainage area. Using the DEM and contours of the area, two potential flow paths were drawn for each drainage area. The time of concentration was calculated for each flow path. The longest time of concentration for each drainage area was used to calculate the peak flow. The peak flow was determined for multiple storm events. Table 58 displays peak discharge for each outfall. See Attachment XX for the TR-55 worksheets.

Table 58. Peak Discharge for Detention Basin Outfalls

Outfall #	Peak Design Storm Event	Pipe Size	Peak Discharge [cfs]

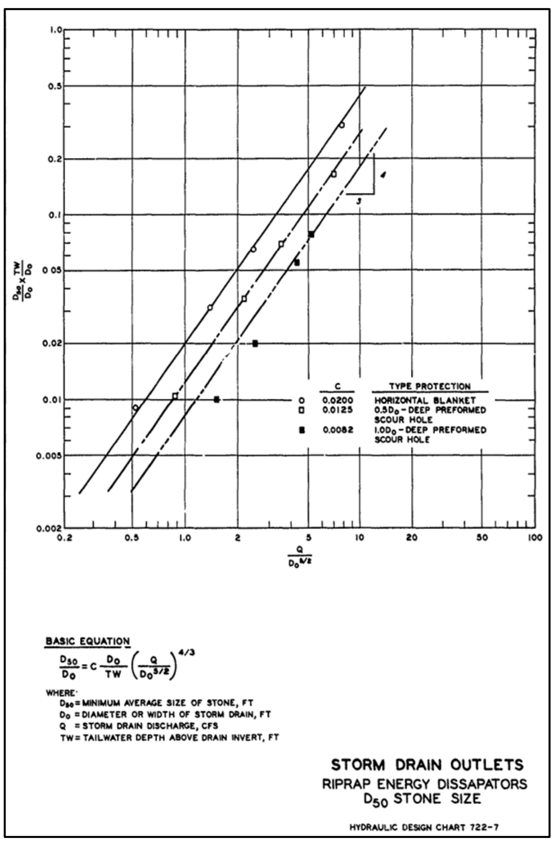
061	10-year	24”	12.3
019	10-year	24”	8.7
013	10-year	15”	6.6
023	25-year	48”	88.5

10.6.2 Riprap Sizing

To prevent erosion downstream of the outlets, riprap would be placed below the outfalls. A riprapped preformed scour hole will be placed at the outlet of all four outfalls to ensure energy is appropriately dissipated. A riprap chute will be placed below Outfalls 061, 019, and 013 to facilitate flow into the detention basins and prevent erosion along the slopes. Outfall 023 will not require a riprap chute as the outfall discharges directly into the basin.

Using the calculated peak discharge rates, riprap sizing was determined using the drop structure spreadsheet for Outfalls 061, 019, 013. The drop structure spreadsheet follows guidance provided in *Design of Rock Chutes* by Robins, Rice and Kadavy (Reference 50). The minimum flow was the 5-year storm event, and the maximum flow was the 10-year storm event. In the drop structure spreadsheet, the riprap layer thicknesses for the drop structures and culvert outlet were sized for high turbulence flow, in accordance with HDC 712-1.

The riprapped preformed scour holes were designed using Hydraulic Design Charts 722-7 and 722-6. The Basic Equation in Figure 32 was used to size the riprap and Figure 33 (Hydraulic Design Chart 722-6) was used to determine the riprap quantity. Type of protection assumed to be one-half the diameter of the storm drain. A safety factor of 1.2 was applied to the result of the basic equation. A bedding thickness of 6 inches for high turbulent flow was used.



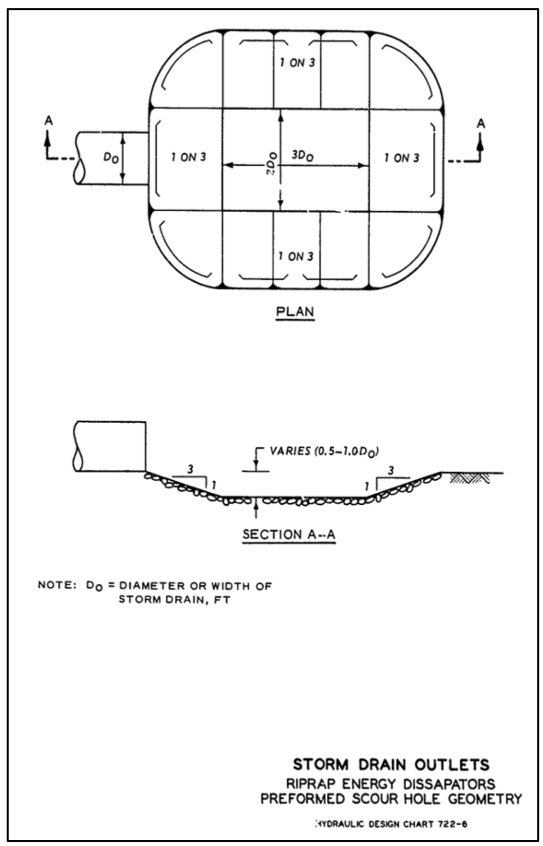


Figure 32. Hydraulic Design Chart 722-7 (Reference 45)

Figure 33. Hydraulic Design Chart 722-6 (Reference 45)

The calculated riprap sizes and quantities for each outfall is summarized in Table 59.

Table 59. Quantities of Riprap for Detention Basin Outfalls.

Outfall #	Rock sizing	Angular Rock[1] [yd^3]	Geotextile [yd^2]	B1 Bedding[2] [yd^3]
061	R20	52	135	36
019	R20	90	238	43
013	R20	59	157	29
023	R30	33	57	9

[1] Note that the angular rock will accommodate high turbulent flow and be 18in thick for R20 and 21in thick for R30.
[2] Note that the B1 bedding will be 6in thick.

11 Opportunities for Further Study

11.1 Planning Engineering and Design (PED) Recommendations

During the PED phase, the following recommendations are included:

- Refine the HEC-RAS model, consider transferring it from a 1-D to a 2-D model to better incorporate stream features. Channel alignment and stream feature designs will need be refined during PED.
- Perform a scour analysis to verify that the foundations of the Winter Street bridge piers will not be impacted by conditions going from that of a lake to a stream with flowing water. This would be followed by a visual inspection and monitoring of foundation conditions during dewatering and dam removal during construction. Rehabilitation of the foundation interface with dam removal and implementation of scour mitigation may be recommended if deemed necessary and is currently included in the cost estimate.
- Assess the flow rates from the Spring Ponds and refine the designs for the rehabilitated outlets of both ponds.
- Evaluate the use of a meandering infiltration swale in place of detention basins during PED. This is a unique area that infiltration swales may be an option due to the constricted, consistent outflows from the sewer network outfalls.

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13 Attachments

Attachment E-1: Hydraulic Design Tool Output for Rock Sizing

Attachment E-2: Map of Delineated Drainage Basins

Attachment E-3: Detention Basin Peak Discharge TR-55 Worksheets

Attachment E-4: Detention Basin Riprap Calcs – Drop Structure Spreadsheets

Attachment E-5: Outfall Performed Scour Hole Calculations

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Appendix F: Cost Engineering

Kinnickinnic River Continuing Authorities Program Section 206 Feasibility Report with Integrated Environmental Assessment

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Appendix F

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1 Introduction

This appendix contains a feasibility study level 3 cost estimate and Total Project Cost Summary prepared for the Kinnickinnic River CAP 206 Feasibility Study Tentatively Selective Plan (TSP). The project area includes the Powell Falls and Junction Falls Dams that impound the Kinnickinnic River. These dams create Lake Louise and Lake George, respectively. The area is located in the City of River Falls, WI. USACE along with non-federal partners: City of River Falls and Wisconsin Department of Natural Resources (WNDR) developed this estimate based on selection of a TSP.

The estimate covers a conceptual level of design detail, and all costs are based on quantities developed from the project implementation report and design layout as reviewed.

This estimate includes planning, engineering and design (PED); construction and construction management (CM) costs for the TSP to allow for final design and construction to proceed subsequent to document approval.

Guidance for the preparation of the estimate and attachments was obtained from Engineer Regulations (ER)1110-2-1150, Engineering and Design for Civil Work Projects; ER 1110-1-1300, Cost Engineering Policy and General Requirements; ER 1110-2-1302, Civil Works Cost Engineering; and ER 1105-2-103, Policy for Conducting Civil Works Planning Studies ; Engineer Technical Letters (ETL) 1110-2-573, Construction Cost Estimating Guide for Civil Works; EM 1110-2-1304, Civil Works Construction Cost Index System; Engineering Circular (EC) 1105-2-410, Review of Decision Documents, and the Cost DX website at Walla Walla District Cost Engineering Mission

2 Background

The objective of the project is to rehabilitate and enhance the project area. The TSP consists of miscellaneous removals and the following improvements: dam removal, channel and upland work, in-channel rock features, habitat restoration, bridge protection and ecosystem restoration.

A complete description of plan formulation measures and alternatives can be found in the main report of the Section 206 Continuing Authorities Program (CAP) Feasibility Report. This cost appendix presents only the cost for the TSP, optimized in the second phase of the feasibility study by parametric comparison of system performance level. The performance levels studied, and parametric costs are shown in Section 4: Evaluation and Comparison of Final Array of Alternatives of the Feasibility Report.

3 Tentatively Selected Plan (TSP)

The TSP documents the features of Alternative 7 found in Section 5 of the main report. Additional details on design assumptions for the TSP are included in technical appendices.

4 USACE Civil Works Work Breakdown Structure (CWWBS)

This section provides elements of the Civil Works Work Breakdown Structure (CWWBS). Feasibility study costs in excess of the first $100,000 are cost shared 50 percent federal and 50 percent non-federal. Design and construction costs are 65 percent federal and 35 percent non-federal. The sponsor is responsible for the operation and maintenance of the project after its completion.

4.1 CWWBS (01) Lands and Damages

The lands and damages account includes the costs for the lands and administrative costs necessary for the construction of the TSP. Costs included the acquisition 52.5 acres of property, permanent and temporary construction easements, and fee title lease agreements. A separate real estate contingency was supplied by Real Estate in the amount of 25 percent for this feasibility effort.

4.2 CWWBS (06) Fish and Wildlife Facilities

4.2.1 Dam Removal

The TSP includes the demolition and removal of the existing dam structures at Powell Falls and Junction Falls. For demolition, the following features were accounted for: mobilization, haul and access roads, flow and sediment management, demolition, and erosion control. Complete dam removal would involve demolishing and excavating the entire width of the dam up to the embankment walls and restoring the river to its natural, free-flowing state. Complete dam removal would include the removal of all physical components of the dam, including the spillway, gates, and any other infrastructure that impedes water flow. By dismantling these barriers, a more natural river flow is restored and promotes the recovery of aquatic and terrestrial habitats both within the riverbed and surrounding area. Dam removal also facilitates the natural transport of sediment, helping rebuild downstream habitats that have been sediment starved since the dams were built. The reestablished river reduces stagnant water zones, increasing dissolved oxygen levels and decreasing water temperatures, both of which are critical for coldwater aquatic life.

4.2.2 Channel and Upland Work

Channel and upland work includes the excavation, placement, and hauling of sediment from the existing lake beds. Additional stormwater outlet erosion control is included to maintain the upland work and prevent washouts. Hauling off-site will be minimized by reusing the excavated material to build the upland features.

4.2.3 In-Channel Rock Features

In-channel rock features for this project include riffles, cross vanes, arch rapids, bank protection, cobble apron, lunker structure rock, and boulders. These features will help enhance and stabilize the restored channel and provide habitat for wildlife.

4.2.4 Habitat Restoration

Restoration includes habitat restoration and working limits. Habitat restoration includes forest and wetland plantings to restore the area to a more natural ecosystem. Working limits include staging areas, disturbed areas, and access roads.

4.2.5 Bridge Protection

Bridge protection is necessary to prevent scour along the bridge piers. Concrete, steel reinforcement, and rock will be used to reinforce the bridge and protect it from scour.

4.2.6 Spring Pond Restoration

Restoration includes stream restoration and minimal forest restoration.

4.3 CWWBS (30) Planning, Engineering, and Design

The work covered under this account includes the project management, planning, engineering, and design costs spent to date as well as the remaining estimated costs that will be associated with the engineering and design for this project.

4.4 CWWBS (31) Construction Management

The work covered under this account includes the expected costs for contract supervision, contract and construction administration, technical management activities, district office supervision, and administration costs.

5 Methodology

5.1 General

This appendix summarizes the cost estimate prepared for the TSP. The estimate includes planning, engineering, design, construction, and construction management costs. The estimate was developed after the review of preliminary project schematics, project data, project requirements, and attending regular PDT meetings.

This Fully Funded Estimate (FFE) has been prepared according to March 2025 price levels. The costs are considered to be fair and reasonable to a well-equipped and capable contractor and include overhead and profit. The preparation of this estimate was created in accordance with Engineer Regulations (ER)1110-2-1150, Engineering and Design for Civil Work Projects; ER 1110-1-1300, Cost Engineering Policy and General Requirements; ER 1110-2-1302, Civil Works Cost Engineering; and ER 1105-2-100, Planning Guidance Notebook - Appendix E; Engineer Technical Letters (ETL) 1110-2-573, Construction Cost Estimating Guide for Civil Works; EM 1110-2-1304, Civil Works Construction Cost Index System; Engineering Circular (EC) 1105-2-410, Review of Decision Documents, and the Cost DX website at Walla Walla District Cost Engineering Mission.

The estimate is organized in accordance with the Civil Works Work Breakdown Structure (CW - WBS). The estimate was developed using Micro Computer Aided Cost Estimate System MII v4.4 cost estimating software. Applicable crews and equipment were applied in the estimate to correspond with the work being performed. Material prices were developed using the MII Cost Book 2024, RSMeans references, and abstract data from similar projects. The midpoint of construction is anticipated to be the 4th quarter of 2027, which was used to determine the FFE.

5.2 Acquisition Strategy

This Project is assumed to be an unrestricted competitive bid, although the possibility of a restricted Small Business type contract was discussed and accounted for during the Abbreviated Risk Analysis (ARA) process.

5.3 Price level

The feasibility report cost estimate is based on FY25, fiscal year prices, unless noted otherwise. This level 3 estimate was prepared using version 4.4 of the MII Cost Estimating Program, see Attachment 1. Project costs were developed using MII English Cost Book 2024, and the MII 2024 EP Region 4 Equipment Manual. Estimated costs are considered fair and reasonable for a prudent and capable contractor and include overhead, subcontractor profit, and bond.

5.4 Unit Prices

Unit costs were developed using internal estimates of similar projects, contractor conversations, material quotes from suppliers, recent bid abstracts, published construction cost index resources and the 2024 English Cost book for MII cost engineering software.

5.5 Quantity Takeoffs

Approximate dimensions, areas and volumes were determined using hand computation, digital drawings, scaling, and comparison to similar order-of-magnitude installations. These dimensions were used to generate quantity tabulations in spreadsheet and hand computation formats. Most major dimensions are the result of preliminary engineering analysis. Minor approximations were necessary to account for costly items. In most cases, major dimensions used to calculate dimensional variation associated with differing flow scenarios at each structure location and preliminary structural analysis. See Attachment 2 for the current quantities as developed above and the quantity assurance and quality certification. Please see the following appendices for detailed discussion on the calculations behind the quantity estimates:

- Dam Removal Quantities:
    - Haul and Access Roads
        - Appendix D Geotechnical Engineering
        - Appendix H Civil Engineering
    - Flow Management
        - Appendix D Geotechnical Engineering
    - Demolition
        - Appendix I Structural Engineering
        - Appendix N Mechanical & Electrical Engineering
- Channel and Upland Earthwork
    - Appendix E Hydraulics & Hydrology
    - Appendix H Civil Engineering
- In-Channel Rock and Bedding
    - Appendix E Hydraulics & Hydrology
- Habitat and Site Restoration
    - Main Report
- Spring Ponds Restoration
    - Appendix G Geotechnical Engineering

5.6 Labor Rates

Based on 2025 Davis-Bacon Wage Rates General Decision WI20250015, Heavy construction projects (Excluding Tunnel, Sewer, and Water Lines), for Pierce County, WI.

5.7 Mark-Ups

5.7.1 Overtime

Overtime was based on a 5-day, 8-hour workweek with multipliers of 1.5 for Monday through Saturday.

5.7.2 Contractor Mark-Ups

Contractor mark-ups were based on mark-ups used on District projects of similar size and scope if not specified as separately calculated.

5.7.2.1 Mobilization and Demobilization

Mobilization and Demobilization are assumed to be 5 percent of direct construction costs.

5.7.2.2 Job Office Overhead (JOOH)

JOOH are those indirect cost which occur specifically and only as a result of a particular project and hence are charged directly to the project. Cost contributors include, support vehicles, contractor's superintendent, small tools, site maintenance, and clean up.

JOOH was applied as a running percentage at 10% based on industry data from RSMeans Heavy Construction.

JOOH was applied as a running percentage at 10% for the subcontractors as it is assumed they have reduced burden for QA/QC, safety, signage, and site maintenance.

5.7.2.3 Home Office Overhead (HOOH)

HOOH are those expenses incurred by the contractor in the overall operation of the business which are not associated with a particular project. A certain percentage of these expenses are charged to each project. HOOH includes such items as office rental or ownership costs, utilities, office equipment, office staff, and insurance. The range of home office overhead can be quite broad and depends largely on the contractor's annual volume of work and the type of work that is generally performed by the contractor.

HOOH was applied as a running percentage at 8.6% based on calculations from data collected over time from contractors in the project area.

5.7.2.4 Profit

Profit is defined as a return on investment and provides the contractor with an incentive to perform the work as efficiently as possible. For the TSP estimate, profit was developed using the weighted guideline method, which considers the contractor's degree of risk, the relative difficulty of work, the monetary size of the job, the period of performance, the contractor's investment, assistance by the Government, and the amount of subcontracting. Profit for the prime contractor was calculated at 9.04%

Profit for subcontractor's is applied at a running percentage of 9.04%.

5.7.2.5 Bond

Bond contract costs are for performance and payment security bonds that protect the government from a non-performing contractor and subcontractors and suppliers from non-payment. The bond markup is applied only to the prime contractor's own work and the prime's subcontractor's work at 1.5%. This is an assumption based of industry standard data.

5.8 Taxes

Sales tax of 5.5% was applied to the material costs.

5.9 Productivity

Normal productivity (100%) was applied. User crews were created using the estimator's judgment. Crew selections and production rates were assisted by information from CAT Book, 42nd Edition.

6 Construction Methodology

Generally, a balance must be struck to provide reasonable access for construction while minimizing the environmental disturbances associated with dredging and construction. The sections below discuss how access is envisioned for the specific project sub-areas. Any significant changes to these plans would be evaluated on a case-by-case basis for approval and may require additional environmental review.

More details on construction access, dam dewatering and dam removal can be found on the main report in Section 5.4: Design and Construction Considerations.

7 Project Schedule

The length of the schedule was determined to allow the contractor to construct during low water conditions and/or winter construction 2027/2028. The project duration is assumed to be 2-year to complete the construction.

8 Abbreviated Risk Analysis

An Abbreviated Risk Analysis (ARA) process for Total Project Costs under $40 million has been established by Walla Walla Cost Engineering Mandatory Center of Expertise (MCX). The purpose of this risk analysis is to determine a contingency cost for each project feature including, planning, engineering, design, and construction management. Based on the results of the analysis, the Cost Engineering Mandatory Center of Expertise for Civil Works (MCX located in Walla Walla District) recommends a cost contingency of approximately 29 percent of the base project cost at an 80 percent confidence level of successful execution. For full ARA report see Attachment 4. Contingencies used are intended to identify an estimated construction cost amount that is not likely to be exceeded, given the current project scope. The contingency selected for this project is not a means of adding costs to the project for possible schedule slippage or future cost growth, or to cover items that are not specifically being considered in the current scope. Contingencies were chosen to account for uncertainties in quantities, uncertainties in unit pricing, and pure unknowns. Contingencies were not included in quantity computations.

9 Total Project Cost Summary (TPCS)

A total project cost summary (TPCS) was developed for the estimated construction costs, see Attachment 5. The TPCS was developed using the current Cost DX Excel spreadsheet which incorporates the cost for all feature accounts developed in the recommended plan estimate at the current FY price level and escalated to the midpoint of design and midpoint of construction. The non-Federal cost share includes feasibility costs, lands, easements, rights-of-way, relocations (LERRDs) and related administrative costs. The current estimate assumes fee title acquisition of the project footprint. Fee title is appropriate where features are constructed (e.g., a site not already within State or county right-of-way). Maximum use of easements will improve project acceptability to local interests (the non-federal local sponsor, city, county, and landowners) because it will reduce costs, retain the tax base, and mitigate major impacts on individuals.

9.1 Project First Cost

The project first cost as found on the TPCS:

Table 1: TPCS Project First Cost

Description	Project First Costs
01 Lands and Damages	$328,125
06 Fish & Wildlife	$17,584,005
30 Planning, Engineering, Design	$2,607,708
31 Construction Management	$1,396,170
Project First Cost	$21,916,007

9.2 Total Project Cost (Fully Funded)

The total project cost (Fully Funded) as found on the TPCS:

Table 2: TPCS Total Project Cost (Fully Funded)

Description	Project Cost (Fully Funded)
01 Lands and Damages	$341,366
06 Fish & Wildlife	$19,385,253
30 Planning, Engineering, Design	$2,846,673
31 Construction Management	$1,565,565
TOTAL Project Cost (Fully Funded)	$24,138,857

10 Attachments

ATTACHMENT 1	MII SUMMARY REPORT
ATTACHMENT 2	QUANTITIES
ATTACHMENT 3	P6 PROJECT SCHEDULE
ATTACHMENT 4	ABBREVIATED RISK ANALYSIS (ARA)
ATTACHMENT 5	TOTAL PROJECT COST SUMMARY (TPCS)

ATTACHMENT 1

MII SUMMARY

ATTACHMENT 2

QUANTITIES

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P6 PROJECT SCHEDULE

ATTACHMENT 4

ABBREVIATED RISK ANALYSIS (ARA)

ATTACHMENT 5

TOTAL PROJECT COST SUMMARY (TPCS)

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Appendix G: Real Estate Plan

Kinnickinnic River Continuing Authorities Program Section 206 Feasibility Report and Integrated Environmental Assessment

May 2025

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1. STATEMENT OF PURPOSE

The Real Estate Plan (REP) is tentative in nature; it is for planning purposes only and both the final real property acquisition lines and the real estate cost estimates provided are subject to change even after approval of the Kinnickinnic River Ecosystem Restoration Project under the Continuing Authorities Program (CAP), Section 206, Aquatic Ecosystem Restoration Project Feasibility Report with Integrated Environmental Assessment (FR/EA).

The REP supports and complements the Main Report and is intended to be used for long term planning purposes; and, as a Real Estate Decision Document for the purpose of meeting the pre-acquisition criteria in ER 405-1-11, paragraphs 3-14.

2. PROJECT AUTHORIZATION

Planning, design and implementation of this project is authorized by Section 206 of the Water Resources Development Act (WRDA) 1996 (P.L.104-3030), as amended. Under this authority USACE may carry out aquatic ecosystem restoration projects that will improve the quality of the environment, are in the public interest, and are cost effective.

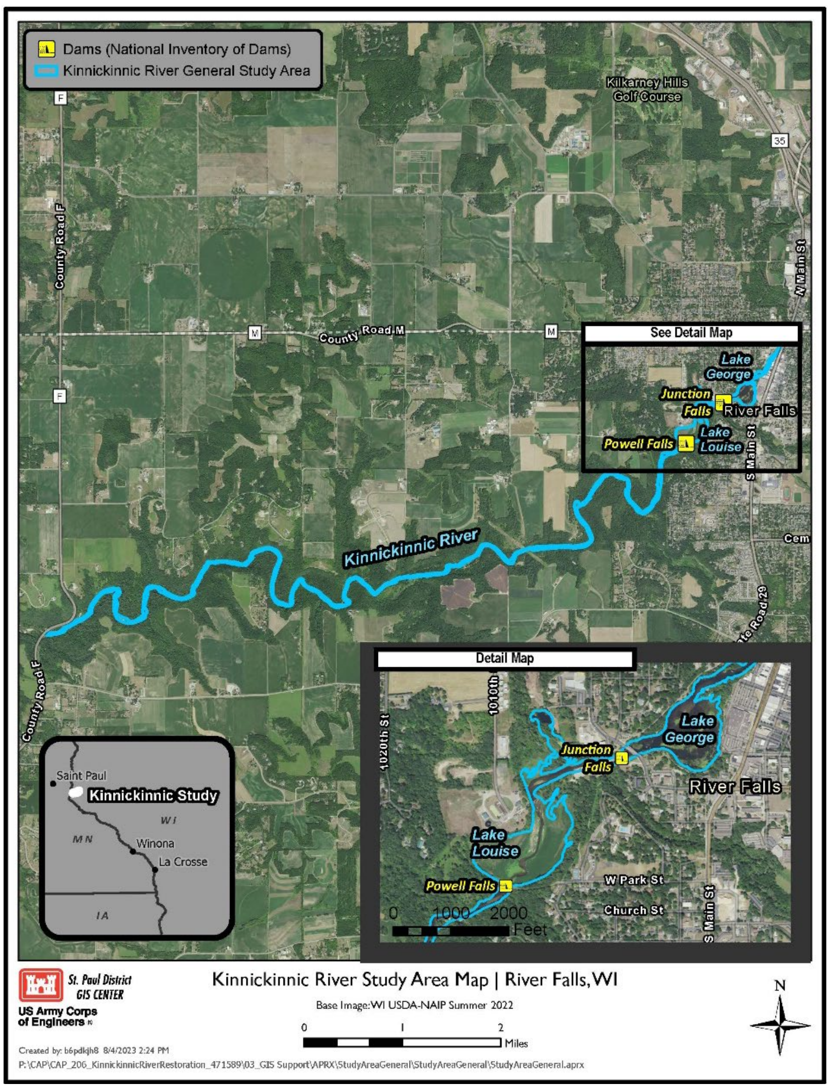

Figure 1: Study Area Map

3. PROJECT LOCATION AND DESCRIPTION

The project takes place on the Kinnickinnic River that flows through the community of River Falls in Pierce County, Wisconsin, as shown in Figure 1 above. The two hydropower dams, Junction Falls and Powell Falls, were built along the river and have since altered its hydrogeomorphic regime.

The removal of the Junction Falls Dam, and Powell Falls Dam present a unique opportunity to restore the river to its natural setting. The two impoundments have altered the hydrology of the Kinnickinnic River and its two lakes (Lake George and Lake Louise) shown in Figure 1, resulting in increased sedimentation, higher water temperatures, and a lack of aquatic diversity, particularly to local brown and brook trout species. The U.S. Army Corps of Engineers (USACE) - St. Paul District has assessed the problems and considered potential solutions with their associated ecosystem restoration benefits.

The objectives of the project are to:

1. Restore natural hydrothermal/hydrogeomorphic dynamics within the stream to support native cold-water species.
2. Increase riffle and pool geomorphic sequence to increase the use and availability of cold-water habitat.

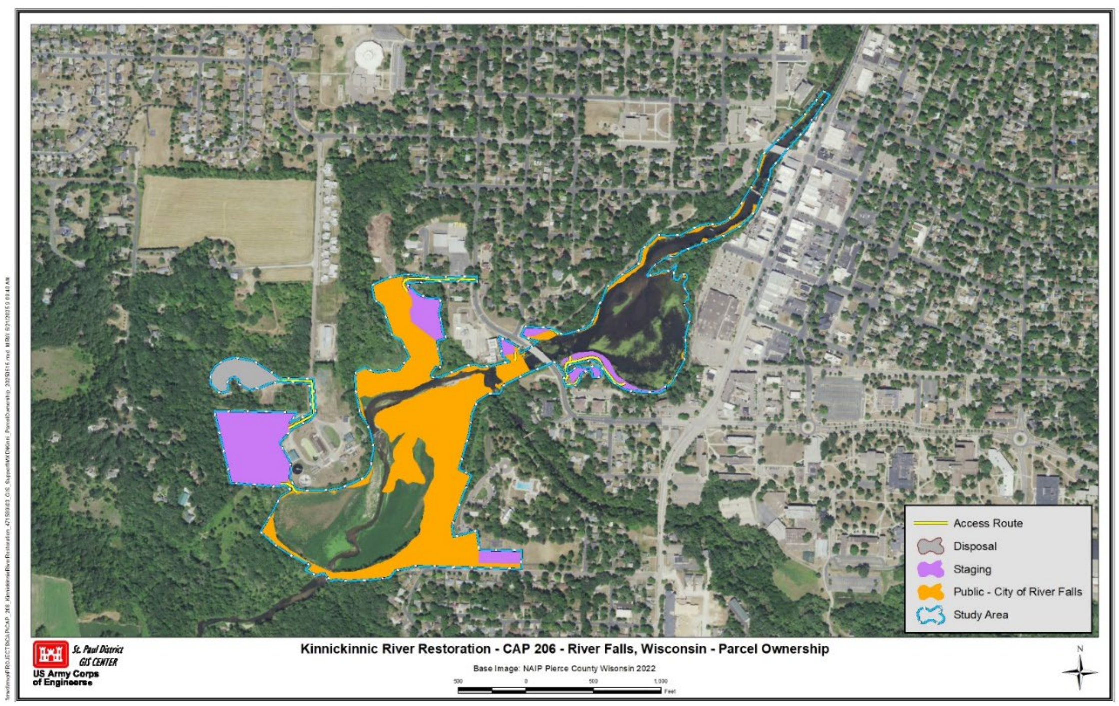

Figure 2: Project Features and Parcel Ownership

3.1 The Tentatively Selected Plan

Under the tentatively selected plan (TSP) for this project, Alternative 7, Powell Falls Dam and Junction Falls Dam would be removed, and stream and riparian restoration would be efforts would be undertaken. The removal process would help promote the rehabilitation of the river’s ecosystem. The TSP will also include improvements to the natural waterfalls, riffles, rock arch rapids, cross vanes, various bank protection efforts, and habitat restoration. A full description of Alternative 7 is included in the Main Report.

All work that is being proposed is within the City of River Falls, Wisconsin. The City of River Falls, (Non-Federal Sponsor or Project Sponsor), would be responsible for all provision and acquisition of the lands, easements, rights of way, disposal sites, and performance of any relocations required for the project.

The lands, easements, and rights of way (LER), that are required to be provided for the proposed project (Alternative 7), include staging areas, ingress/egress routes, borrow and disposal sites are described below.

Ecosystem Restoration Project Area

Current estimates for the project footprint are 42.87 acres. Fee is the standard estate for ecosystem restoration and will be required.

Staging Areas

The staging areas that have been proposed, that are not already owned by the NFS would require a Temporary Work Area Easement (Standard Estate #15). Current estimates for staging areas total 5.71 acres, with staging areas on lands owned by the NFS equaling 4.43 acres and staging areas on private land equaling 1.28 acres. Maps of the staging areas can be found in Figure 2 above.

Ingress/Egress

Any proposed access roads to and from the project that are not on lands owned by the NFS's would require Temporary Work Area Easements (Standard Estate #15). Current estimates for access roads total 0.06 acres.

Disposal Sites

Disposal of some excavated material would occur as reuse from the construction of project features. Any material not reutilized in the project features would be placed at a disposal site owned by the city. Should any additional disposal sites be deemed necessary for the project, then a Temporary Work Area Easement would be required if the NFS is not the land owner. The current estimate for the disposal site equals 1.89 acres.

Borrow Sites

Borrow sites are not anticipated for Alternative 7.

Table 1: Necessary Estate per Project Feature with Acreage

Feature	**Ownership**	**Minimum Estate Required for Project**	**Approximated Acres**
Project Area	City of River Falls	Fee	42.87
Staging Areas	City of River Falls	Temporary Work Area Easement	4.43

	Private Owners	Temporary Work Area Easement	1.28
Ingress/Egress Possible Access Routes	City of River Falls /Private Owners	Temporary Work Area Easement	0.06
Disposal Site	City of River Falls	Temporary Work Area Easement	1.89

4. NON-FEDERALLY OWNED LAND

Under the recommended plan the entire project footprint is non-federally owned. The NFS owns approximately 42.87 acres of the project. The NFS owns the majority the lands required for the staging areas, access road areas, and the proposed disposal site. Other areas where temporary interests are required are owned by private landowners, approximately 1.28 acres.

5. ESTATES REQUIRED

There are no proposed non-standard estates needed for the TSP. The required standard estate for ecosystem restoration is Fee simple. The minimum required standard estate for temporary requirements including staging and disposal is the Temporary Work Area Easement. The estate is described below:

<u>Temporary Work Area Easement</u>

A temporary easement and right-of-way in, on, over and across (the land described in Schedule A) (Tracts Nos. ____,____,and____), for a period not to exceed ________, beginning with date possession of the land is granted to the United States or the non-Federal sponsor, for use by the United States or the non-Federal sponsor, its representatives, agents, and contractors as a (borrow area) (work area), including the right to (borrow and/or deposit fill, spoil and waste material thereon) move, store and remove equipment and supplies, and erect and remove temporary structures on the land and to perform and other work necessary and incident to the construction of the Project, together with the right to trim, cut fell and remove therefrom all trees, underbrush, obstructions, and any other vegetation, structures, or obstacles within the limits of the right-of-way; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

6. EXISTING FEDERAL OR OVERLAPPING PROJECTS

There are no overlapping Federal Civil Works projects. The dams subject to removal are not federally owned or operated.

7. FEDERALLY OWNED LANDS OR OTHER INTERESTS

There are no federally owned lands or interests within the boundaries of the TSP.

8. NAVIGATIONAL SERVITUDE

The use of navigational servitude is not required or proposed for this project, nor does it apply to the Kinnickinnic River.

9. INDUCED FLOODING

There is no induced flooding anticipated under the current TSP. For additional information, refer to Appendix E: Hydraulics & Hydrology.

10. SUMMARY OF REAL ESTATE COSTS

The Baseline Cost Estimate for Real Estate (BCERE) establishes the estimated financial costs (for both the Government and Sponsor) that are attributed to the TSP's real estate requirements. The TSP's total estimated real estate cost is $385,000. The table below provides a summary of the BCERE. This is subject to change and will be reassessed if determined necessary during the development of plans and specifications.

Table 2: Real Estate Base Cost Estimate

Real Estate Base Cost Estimate			
Description	**# Tracts**	**Cost per Tract**	**Total Cost**
01 LANDS AND DAMAGES			
TWAE (Access Area, Disposal Site, Staging Areas 7.67 acres)	7	7,500.00	52,500.00
Project Footprint (42.86 mostly submerged lands)	42	5,000.00	210,000.00
Contingency 25%		3,125.00	65,625.00
02 RELOCATION COSTS			
Unknown at this time			
30 RE PED COSTS			
NFS Administrative Expenses	1	20,000.00	20,000.00
USACE Administrative Expenses	1	25,500.00	25,500.00
Contingency 25%		11,375.00	11,375.00
TOTAL ESTIMATED PROJECT LERRD COSTS			**$385,000.00**

11. PUBLIC LAW 91-646 RELOCATIONS

There are no residential, business, or farm relocations to be undertaken within the proposed project area.

12. MINERAL ACTIVITY/TIMBER RIGHTS

There are no known mineral recovery activities currently ongoing or anticipated, nor are there any oil/gas wells present on the project LERRD, or in the immediate vicinity, that could/would impact the construction, operation, or maintenance of the project. No acquisition of any mineral interest(s) from surface owners or rights outstanding in third parties will be required.

Within the project area, no timber rights held by third parties exist.

13. ASSESSMENT OF NON-FEDERAL SPONSOR ACQUISITION

The City of River Falls (NFS) is a municipality with a Real Estate office that performs acquisitions routinely as part of their mission; they possess the capability to perform any acquisitions necessary for this project. Please see the enclosed NFS Capability Assessment.

14. ZONING ORDINANCE REQUIREMENTS

There are no zoning considerations or restrictions proposed for this project in lieu of acquisition.

15. ACQUISITION SCHEDULE

The project features to be constructed are on lands owned by the Project Sponsor, therefore no acquisition schedule has been prepared for the REP. Should it be determined at a later date that lands do need to be acquired for a temporary work area easement, the REP will be updated to reflect the acquisition needs, and a schedule will be provided.

16. UTILITY OR FACILITY RELOCATIONS, ALTERATIONS, OR REPLACEMENT

Appendix I, Structural Engineering, identifies that the Winter Street Bridge may be affected by scour subsequent to the removal of Junction Falls Dam. An alteration of the bridge footing is preliminarily identified as a potential facility relocation. The need for the alteration will be further evaluated in design.

ANY CONCLUSION OR CATEGORIZATION CONTAINED IN THIS REAL ESTATE PLAN, OR ELSEWHERE IN THIS REPORT, THAT AN ITEM IS A UTILITY OR FACILITY RELOCATION TO BE PERFORMED BY THE SPONSOR AS PART OF ITS LERRD RESPONSIBILITIES IS PRELIMINARY ONLY. THE GOVERNMENT WILL MAKE A FINAL DETERMINATION OF THE RELOCATIONS NECESSARY FOR THE CONSTRUCTION, OPERATION, OR MAINTENANCE OF THE PROJECT AFTER FURTHER ANALYSIS AND COMPLETION AND APPROVAL OF AN ATTORNEY'S OPINION OF COMPENSABILITY FOR EACH OF THE IMPACTED UTILITIES AND FACILITIES.

17. HAZARDOUS, TOXIC, RADIOACTIVE WASTE (HTRW) CONCERNS

A Phase I Environmental Site Assessment was completed for the site. No HTRW concerns for implementation are anticipated at this time.

18. LANDOWNER ATTITUDE / PUBLIC CONCERN

The non-Federal Sponsor, state, and local government authorities have expressed support for the project. The project is not expected to create dispute among adjacent landowners or the general public. There is much excitement within the local community for the construction of the project which will produce benefits for the river and public uses compatible with the restoration.

19. NOTIFICATION TO PROJECT SPONSOR

Prior to conclusion of the study phase, the NFS will be made aware of the risks associated with acquiring real estate in advance of executing the Project Partnership Agreement (PPA).

20. OTHER RELEVANT REAL ESTATE ISSUES

The public utilizes the project area to engage in recreational pursuits such as fishing, boating, bird watching, and other recreational activities which will be temporarily disrupted during construction. The project may have ancillary benefits to restoration-compatible recreation after construction.

21. RISK ANALYSIS

There are no known or anticipated real estate related risks associated with the Project.

______________________________	____________________
Morgan Peterson, Realty Specialist (Preparer)	Date
______________________________	____________________
Brittney Haupert, Chief, Planning & Acquisition Branch (Reviewer)	Date
______________________________	____________________
Kevin Sommerland, Chief, Real Estate Division (Approver)	Date