



Upper Turtle River Watershed Dam #9, Larimore Dam

*Appendix D-1: Existing Conditions
Assessment Report (DRAFT)*

**Grand Forks County Water Resource
District
USDA - NRCS**

February 19, 2026

Table of Contents

0	EXECUTIVE SUMMARY	1
1	DAM SUMMARY DATA	3
2	FIELD SURVEY	4
2.1	DOWNSTREAM CHANNEL SURVEY	4
2.1.1	METHODS	4
2.2	LARIMORE DAM TOPOGRAPHIC SURVEY	4
2.2.1	METHODS	5
2.2.2	CHANGES IN ELEVATION	5
2.3	NORMAL POOL BATHYMETRIC AND SEDIMENTATION SURVEY	5
2.3.1	METHODS	5
2.3.2	RESULTS	6
2.4	STRUCTURE FOUNDATION ELEVATIONS	6
2.4.1	METHODS	6
3	REVIEW AND INSPECTION OF STRUCTURE	6
3.1	PRINCIPAL SPILLWAY CONCRETE INLET STRUCTURE	7
3.2	PRINCIPAL SPILLWAY CONDUIT AND OUTLET	7
3.3	AUXILIARY SPILLWAY	10
4	DAM BREACH ANALYSIS AND HAZARD CLASSIFICATION	11
4.1	BREACH DISCHARGE CRITERIA	11
4.2	BREACH RESULTS	11
4.3	HAZARD CLASSIFICATION	17
5	HYDROLOGIC AND HYDRAULIC MODELING	18
5.1	HYDROLOGY MODEL	18
5.1.1	2013 USACE HEC-HMS MODEL	18
5.1.2	2015 SITES MODEL	18
5.1.3	2023 NRCS HEC-HMS MODEL	19
5.1.4	SUBBASIN BOUNDARIES	19

5.1.5	SUBBASIN PARAMETERS	19
5.1.6	RESERVOIR PARAMETERS	20
5.2	HYDRAULIC (HEC-RAS) MODEL	21
5.2.1	NRCS UTR10 DOWNSTREAM HYDRAULIC MODEL	21
5.2.2	UPDATED DOWNSTREAM HYDRAULIC MODEL	21
5.2.3	PROBABLE MAXIMUM PRECIPITATION	22
5.3	SPILLWAY DESIGN HYDROGRAPHS	22
5.3.1	PRINCIPAL SPILLWAY INFLOW HYDROGRAPH	23
5.3.2	AUXILIARY SPILLWAY STABILITY DESIGN HYDROGRAPH	24
5.3.3	FREEBOARD INFLOW HYDROGRAPH	25
6	GEOLOGIC INVESTIGATION	26
7	SITES ANALYSIS	27
7.1	SITES MODEL INPUTS	27
7.2	PRINCIPAL SPILLWAY RESULTS	28
7.3	AUXILIARY SPILLWAY RESULTS	28
7.4	SUMMARY OF IDENTIFIED DEFICIENCIES	30
7.4.1	PRINCIPAL SPILLWAY	30
7.4.2	AUXILIARY SPILLWAY	30
7.4.3	EMBANKMENT	30
8	REFERENCES	31

Tables

Table 1: Larimore Dam Summary Data (based on as-built drawings).....	3
Table 2: Larimore Dam Structural Deficiency Summary	8
Table 3: Residential Structures Potentially Impacted by a Breach of Larimore Dam	12
Table 4: Road Overtopping Data During a Breach of Larimore Dam	16
Table 5: Storms used in the 2015 SITES Assessment	Error! Bookmark not defined.
Table 6: TR 210-60 Minimum Precipitation Data for High Hazard Dams	22
Table 7: Principal Spillway Inflow Hydrograph Data	24
Table 8: Auxiliary Spillway Inflow Hydrograph Data and Results	25
Table 9: Freeboard Inflow Hydrograph Data	26

Figures

- Figure 1. Downstream Channel Survey
- Figure 2. Larimore Dam Topographic Survey
- Figure 3. Larimore Dam Bathymetric Survey
- Figure 4. Larimore Dam Elevation-Flood Storage Relationship
- Figure 5. Larimore Dam Breach Outflow Hydrograph
- Figure 6. Larimore Dam Breach Inundation – All
- Figure 7. Larimore Dam Breach Inundation – 1
- Figure 8. Larimore Dam Breach Inundation – 2
- Figure 9. Larimore Dam Breach Inundation – 3
- Figure 10. Larimore Dam Breach Inundation – 4
- Figure 11. Depth-Velocity Flood Danger Level Relationship for Houses Built on Foundations
- Figure 12. Depth-Velocity Flood Danger Level Relationship for Passenger Vehicles
- Figure 13. Subbasin 24-hour curve numbers
- Figure 14. Subbasin time of concentration
- Figure 15. Subbasin storage coefficients
- Figure 16. Reservoir Storage – Outflow Comparison
- Figure 17. HEC-RAS Model Schematic
- Figure 18. Principal Spillway Mass Curves
- Figure 19. Principal Spillway Inflow Hydrographs
- Figure 20. Auxiliary Spillway Hydrographs and Routing
- Figure 21. Freeboard Hydrograph and Routing
- Figure 22. Larimore Dam Auxiliary Spillway Breach Profile

Attachments

- Attachment 1. As Built Drawings
- Attachment 2. Site Inspection Photos/Checklist
- Attachment 3. TR 210-60 Peak Breach Discharge Calculations.
- Attachment 4. SITES Output
- Attachment 5. Principle Spillway Conduit – Corrosion Detection Testing Report
- Attachment 6. Geotech Report Addendum and 2001 EWP Spillway Repair Plans

0 EXECUTIVE SUMMARY

The data and analysis described in this report were developed and collected to complete a Watershed Plan for Upper Turtle River Dam. Field survey, site inspections, hydraulics and hydrologic analysis, and geotechnical analysis were completed to better understand the existing condition of Larimore Dam.

A site inspection and field survey were completed by HDR Engineering Inc. and Pribula Engineering Inc. Survey data collected for the project was meant to better understand the geometry and layout of the existing embankment and the site inspection was to better assess the condition of the infrastructure. Survey data for the channel downstream of Larimore Dam, the dam embankment, spillways, and bathymetric survey data for the pool upstream of the dam was collected. The survey data was used to determine the dam's storage capacity, the embankment's topography, and to develop accurate cross-sectional data for the South Branch Turtle River and Turtle River channels downstream. Information on critical elevations, storage capacities, and surface areas for Larimore Dam are displayed in **Table 1**.

The most recent site review and inspection of Larimore Dam was conducted by HDR Engineering Inc. (HDR) staff in November of 2022. The inspection showed that the principal spillway concrete inlet structure was in good condition. The conduit had minor deficiencies and it was recommended to replace or repair the steel plates at the joints. The condition of the dam embankment and earthen auxiliary spillway were also determined to be adequate with only minor issues such as tree growth within the dam abutments, gopher holes in the auxiliary spillway and an erosion area on the left side slope of the auxiliary spillway. Additionally, a conduit inspection was performed by the NRCS in November 2021 to look at the corrosion condition of the reinforcing steel in the box culvert. Results indicated a moderate rate of corrosion with a potential lifespan of 6-20 years remaining for the conduit. As a result, the length of a rehab plan for any alternative that did not remove or decommission the existing conduit would be no more than 20 years.

A geologic investigation was also conducted as part of this analysis. The investigation indicated that the current drain fill does not meet criteria for seepage control. The investigation also concluded that the slope stability associated with the embankment at Larimore Dam is adequate for the rapid drawdown, normal pool conditions, and surcharge conditions as required in *Technical Release 210-60: Earth Dams and Reservoirs* (NRCS, 2019). Subsurface geologic data was used to determine the integrity of the auxiliary spillway for hydraulic events on Larimore Dam.

As part of the existing conditions assessment, a two-dimensional dam breach model, a combination one-dimensional and two-dimensional downstream impacts hydraulic model, and hydrologic model of the Turtle River Basin were created or modified. The dam breach analysis was conducted to for alternative analysis for this study based on guidance from Technical Release 210-60: Earth Dams and Reservoirs (NRCS, 2019). The hydrologic analysis, completed in HEC-HMS, was used to develop inflow events to determine if the structure was appropriately sized for its hazard classification. Subsequent hydraulic modeling completed using the NRCS's

SITES program helped determine if the inflow events from the hydrologic model could meet the NRCS's and State of North Dakota's standards for high hazard dams.

The principal spillway analysis yielded that the spillway is undersized to draw the reservoir down in an appropriate timeframe. The auxiliary spillway's integrity was determined to be inadequate when simulating the freeboard hydrograph. The freeboard design hydrograph analysis indicates that the top of dam is at an adequate elevation.

1 DAM SUMMARY DATA

Table 1: Larimore Dam Summary Data (based on survey and as-built drawings)

Dam Name	Larimore Dam
Stream	Turtle River
Year Completed	1979
Design Life	50 Years
Cost	\$357,761
Purpose	Flood Prevention/Recreation
Original Hazard Classification	Significant
Revised Hazard Classification	High
Drainage Area	65.5 square miles
Dam Height	66 Feet
Dam Type	Earthen
Dam Crest Length	1,040 Feet
Storage Capacity	
Permanent Pool	637 Acre-Feet
Principal Spillway – Stage 1	637 Acre-Feet
Principal Spillway – Stage 2	2,410 Acre-Feet
Auxiliary Spillway	4,493 Acre-Feet
Maximum Pool	9,643 Acre-Feet
Principal Spillway	
Type	Concrete
Riser Height	2.5 Feet
Conduit Size	6' X 6' Box
Stages	2
Auxiliary Spillway	
Type	Earthen
Width	300 Feet
Normal Pool Surface Area	66 Acres
Normal Pool Elevation	1067.2 ft (NGVD29) 1068.4 ft (NAVD88)
Second Stage Principal Spillway Elevation	1082.0 ft (NGVD29) 1083.2 ft (NAVD88)
Auxiliary Spillway Elevation (Adjusted from as-builts based on recent survey)	1089.8 ft (NGVD29) 1091.0ft (NAVD88)
Top of Dam Elevation	1099.6 ft (NGVD29) 1100.8 ft (NAVD88)

2 FIELD SURVEY

Survey data obtained for this report was collected at various times by the HDR team, including Pribula Engineering, and the United States Geological Survey (USGS). The data collected specifically for this report includes surveyed cross sections for the downstream channel (HDR), a topographic survey of the dam embankment and auxiliary spillway (Pribula Engineering), a bathymetric survey in the reservoir upstream of the dam (USGS), and the collection of structure elevations throughout the breach zone downstream of Larimore Dam (HDR). Each of the survey processes is described in more detail in the following sub-sections.

2.1 DOWNSTREAM CHANNEL SURVEY

The channel survey conducted in the South Branch Turtle River was completed in the summer of 2020. The extent of the survey spanned from the outlet of Larimore Dam to approximately 700 feet downstream of the outlet. The survey consisted of river channel cross sections along the South Branch Turtle River.

Additionally, channel survey data was collected for the Turtle River downstream of Larimore Dam in October 2020. The survey spanned from the South Branch Turtle River confluence and extended to the Grand Forks Air Force Base (21st Avenue NE). All downstream channel survey data is shown in **Figure 1**.

2.1.1 METHODS

The South Branch Turtle River channel cross sections used a spacing of approximately 200 feet. Four cross sections were collected along this reach. The Turtle River channel cross sections do not utilize a consistent spacing but have 15 cross sections within this reach. Data also includes the hydraulic structures within this reach.

Three permanent survey benchmarks were set for the 2020 downstream channel survey using GPS. The benchmarks used are shown in **Figure 1**. The cross sections and profile of the downstream river channel were surveyed using real-time kinematic (RTK) global positioning system (GPS) equipment. Cross section data is supplemented with LiDAR collected from 2019-2021 (NDDWR, 2019-2021). The horizontal datum used throughout the downstream channel survey collect was NAD83, North Dakota State Plane, North. The vertical datum used was NAVD 1988.

2.2 LARIMORE DAM TOPOGRAPHIC SURVEY

The topographic survey of Larimore Dam was conducted in the summer of 2020. Some of the elements of the dam that were surveyed include the dam centerline, auxiliary spillway, downstream berm, upstream toe and downstream toe. **Figure 2** shows the topographic data collected at Larimore Dam. Supporting information on topographic survey data can be made

available upon request from the North Dakota National Resources Conservation Service (NRCS).

2.2.1 METHODS

A maximum spacing of 50 feet was used to collect cross sections across the dam. The maximum distance between profile shots for the top of dam and emergency spillway centerline was also 50 linear feet. Three permanent survey benchmark control points were set for the topographic survey using GPS. The horizontal datum used throughout the topographic survey collect was NAD83, North Dakota State Plane, North. The vertical datum used was NAVD 1988.

2.2.2 CHANGES IN ELEVATION

The topographic survey was compared to key elevations shown on the as-built drawings presented in **Attachment 1**. Elevations in the as-built drawings reference the NGVD 1929 vertical datum. Elevations at the dam site can be converted from NGVD 1929 to NAVD 1988 by adding 1.2 feet based on the Vertical Datum Conversion Program (VERTCON) available through the National Geodetic Survey (NGS, 2023).

The elevation of the auxiliary spillway crest was approximately 0.5 feet higher than the as-built elevations and the elevation of the top of the dam was approximately equal to the as built drawings on the right abutment.

2.3 NORMAL POOL BATHYMETRIC AND SEDIMENTATION SURVEY

The normal pool bathymetric and sediment surveys were done simultaneously in August of 2020. A survey of the pool upstream of Larimore Dam was completed. The elevation of the pool upstream of Larimore Dam was 1069.33 feet (NAVD 1988) at the time of the survey and is documented in Appendix D-7.

2.3.1 METHODS

Bathymetry data for the pool upstream of Larimore Dam was collected using Global Navigation Satellite System (GNSS) surveying equipment in static and real-time kinematic (RTK) mode to establish a local reference mark and check-in/check-out point, to collect water surface elevations, and to collect topographic points along established data collection transects.

Topographic points were collected on land, in water too shallow for boating, and in areas of the lake where thick aquatic vegetation prohibited use of an echosounder and transducer. In areas of the lake deep enough for boating and where aquatic vegetation was not present, depth data was collected using an echosounder with a dual frequency (200/24 kilohertz with a 4/20 degree beam) transducer. Data were collected on a set of transects roughly perpendicular to the axis of the waterbody, plus on a set of oblique transects that intersected the regular transects. GNSS RTK was used to navigate the survey vessel on data collection transects. The bathymetry data is shown on **Figure 3**.

2.3.2 RESULTS

A cumulative sediment volume in the reservoir upstream of Larimore Dam was estimated based on the data that was collected in August of 2020. The estimated volume of sediment that accumulated in the reservoirs since the construction of the dam in 1979 is 186 acre-feet. This equates to an approximate retention of 4.5 acre-foot per year during the first 41 years of reservoir life (1980-2020). The distribution of the sediment covers the length of both pools at varying depths.

A Supplemental Watershed Work Plan for the Upper Turtle River Watershed (Grand Forks and Nelson Counties, 1969) indicates that the predicted sediment retainage was 5.8 acre-feet per year and the total sediment storage available for Larimore Dam was estimated to be 292 acre-feet.

A stage-storage relationship for Larimore Dam was developed based on the bathymetric survey data collected in August of 2020 and LiDAR data (NDDWR, 2019-2021). **Figure 4** shows the computed elevation-flood storage relationship for current Rehab conditions, as well as the design and intermediate evaluation elevation-storage curves for the dam. The accumulated sediment volume and more precise LiDAR makes a notable difference in curves from past modeling efforts.

2.4 STRUCTURE FOUNDATION ELEVATIONS

Foundation elevations of structures downstream of Larimore Dam were collected during multiple survey efforts to verify the hydraulic depth associated with the structures during breach conditions and other synthetic events simulated. In total, there were approximately 121 structures surveyed along the South Branch Turtle River and Turtle River downstream of Larimore Dam.

2.4.1 METHODS

The permanent survey benchmarks that were set for the downstream channel survey discussed in Section 2.1.1 were also used to collect elevations at the structures. Finished floor elevations of structures identified as being within the designated breach zone or being impacted by a 100-year rainfall event were surveyed using real-time kinematic (RTK) global positioning system (GPS) equipment. The horizontal datum used throughout the downstream channel survey collect was NAD83 North Dakota State Plane, North. The vertical datum used was NAVD 1988.

3 REVIEW AND INSPECTION OF STRUCTURE

Site inspections for Larimore Dam were conducted by HDR Engineering, Inc. in November of 2022. The objective of the inspections was to assess the condition of all elements of the dam including the embankment, slope protection, concrete inlet structure, principal spillway conduit, auxiliary spillway, and all related miscellaneous elements. A dam inspection checklist was used

to record findings and the 2022 Larimore Dam inspection checklist is presented in **Attachment 2**. The condition of the dam embankment during the 2022 inspection was overall fair. The upstream slope had a small tree near the left abutment and trees in the right abutment. The riprap along the upstream slope had minor woody vegetation growing within. The crest had some minor rutting from wheel tracks. The downstream slope had minor woody vegetation along it and trees in both abutments. The downstream berm had severe rutting from wheel tracks. It also had wetland vegetation on it but the ground was dry at the time of inspection. The auxiliary spillway had a large area with severe rutting from ATV activity. There were large areas on the left side slope of the auxiliary spillway that have collapsed and eroded. The auxiliary spillway also had patches of woody vegetation growing throughout and gopher holes present. The District intends to complete vegetation removal on the dam and fill/stabilize/grass seed ruts and cut slope on the ASW in the summer of 2024.

Photographs were also taken during all inspection visits. Photographs of the dam site as well as photographs of any noted deficiencies during the site inspections are presented in **Attachment 2**. Previous inspection reports were reviewed to verify existing conditions and to evaluate deteriorating conditions. The reported conditions are based on observations of field conditions at the time of inspection, and it is important to note that the condition of the various elements of the dam depend on numerous and constantly changing internal and external factors.

3.1 PRINCIPAL SPILLWAY CONCRETE INLET STRUCTURE

All visible sections of the principal spillway concrete inlet structure exterior were inspected for potential damage or movement. Additionally, the interior of the principal spillway conduit was inspected during the site visit. The inspection collected images of the interior of the concrete conduit. It was noted that the principal spillway inlet appears to be in good overall condition. The steel plates around the joints in the interior of the conduit are corroding and there were multiple steel plates that were separated from the joints or missing entirely. These areas are noted and can be seen in the photographs in **Attachment 2**.

The reservoir level at the time of the 2022 inspection was above the first stage inlet orifice of the principal spillway structure. There were no signs of significant debris/trash accumulation around the weir openings and all trash racks appeared to be in good condition. Overall, all visible portions of the concrete inlet structure appear to be in good condition and appear to be structurally sound. No external evidence of any settlement or movement of the structure was observed.

3.2 PRINCIPAL SPILLWAY CONDUIT AND OUTLET

Larimore Dam is nearing the end of its design life of 50 years. Due to the age of the dam, the NRCS conducted an inspection of the principal spillway conduit on November 2, 2021 and October 25, 2024; results are summarized in Table 2. The 2021 testing report is provided as **Attachment 5**, which includes details regarding detection system, estimating service life, and

locations on plan drawings. The purpose of the inspections were to complete Non-Destructive Corrosion Detection Testing of the reinforced steel within the concrete of the principal spillway conduit. The testing was performed at two locations in 2021 and the remaining in 2024. There is one conduit with “severe” classification, two conduits with “high” classification, which leads to estimated service life of less than 2 years and 2-6 years, respectively. The remaining conduits have “passive/low” or “moderate” corrosion rate classifications. The “high” and “severe” classifications indicate significant corrosion with very limited remaining service life.

Table 2: Larimore Dam Principal Spillway Corrosion Detection Results Summary

Year Tested	Test Location	Max. Corrosion Rate (um/year)	Corrosion Rate Classification	Estimated Service Life
2024	Conduit 3 - Left Wall	54.90	High	2 to 6 Years
2024	Conduit 4 - Left Wall	9.03	Passive/Low	No Corrosion
2024	Conduit 5 - Left Wall	18.00	Moderate	6 to 20 Years
2024	Conduit 6 - Left Wall	16.34	Moderate	6 to 20 Years
2024	Conduit 6 - Right Wall	10.90	Moderate	6 to 20 Years
2024	Conduit 7 - Left Wall	41.29	High	2 to 6 Years
2024	Conduit 7 - Right Wall	101.89	Severe	Less than 2 Years
2021	Conduit 8 - Left Wall	15.65	Moderate	6-20 Years
2024	Conduit 8 - Left Wall	29.92	Moderate	6 to 20 Years
2024	Conduit 8 - Right Wall	12.78	Moderate	6 to 20 Years
2024	Conduit 9 - Left Wall	10.57	Moderate	6 to 20 Years
2024	Conduit 9 - Right Wall	15.49	Moderate	6 to 20 Years
2024	Conduit 10 - Left Wall	9.07	Passive/Low	No Corrosion
2024	Conduit 10 - Right Wall	12.23	Moderate	6 to 20 Years
2024	Conduit 11 - Left Wall	19.98	Moderate	6 to 20 Years
2024	Conduit 11 - Right Wall	29.13	Moderate	6 to 20 Years
2024	Conduit 12 - Left Wall	31.72	High	2 to 6 Years
2024	Conduit 12 - Right Wall	12.62	Moderate	6 to 20 Years
2024	Conduit 13 - Left Wall	8.83	Passive/Low	No Corrosion
2024	Conduit 13 - Right Wall	19.12	Moderate	6 to 20 Years
2024	Conduit 14 - Left Wall	8.43	Passive/Low	No Corrosion
2024	Conduit 14 - Right Wall	25.19	Moderate	6 to 20 Years
2024	Conduit 15 - Left Wall	8.79	Passive/Low	No Corrosion
2024	Conduit 16 - Left Wall	12.39	Moderate	6 to 20 Years
2024	Conduit 16 - Right Wall	9.86	Passive/Low	No Corrosion
2024	Conduit 17 - Left Wall	16.01	Moderate	6 to 20 Years

2024	Conduit 17 - Right Wall	15.05	Moderate	6 to 20 Years
2021	Conduit 19 - Ceiling	8.17	Passive/Low	No Corrosion
2024	Conduit 19 - Right Wall	6.64	Passive/Low	No Corrosion

An inspection of the interior of the conduit was completed on November 2nd, 2022 by HDR Engineering Inc. At the joints, steel plates were missing at five separate locations. Steel joint corrosion was observed at most joints along the conduit. Additionally, water was observed to be dripping from the top of joints 14 and 15 (also known as joints 16 and 17 in the as-built drawings). Photographs of the interior of the principal spillway conduit are provided in **Attachment 2**. It was also noted during the inspection that the right foundation drain was missing the animal guard. The District intends on installing an animal guard on the right foundation drain in the summer of 2024.

A new animal guard on the outlet of the right foundation drain should be added. The principal spillway conduit appears to be adequate to last through the expected dam rehabilitation design life of 50 years. All structural elements should continue to be monitored on a regular basis. Observations from the inspection associated with the principal spillway concrete inlet structure, conduit, and outlet are provided in Table 3.

Table 3: Larimore Dam Structural Deficiency Summary

Deficiency Location	Description
Concrete Inlet Structure	Appears to be in good condition
Principal Spillway Conduit: As-Built Joint #3	Right side spalling and cracking at wingwall intersection, crack on left side and loose joint filler.
Principal Spillway Conduit: As-Built Joint #7	Steel plate on left side is missing
Principal Spillway Conduit: As-Built Joint #8	Steel plate on left side is missing.
Principal Spillway Conduit: As-Built Joint #9	Steel plate on left side is missing.

Principal Spillway Conduit: As-Built Joint #10	• Plate on left side is separated near the bottom
Principal Spillway Conduit: As-Built Joint #16	• Water dripping from top of joint.
Principal Spillway Conduit: As-Built Joint #17	• Water dripping from top of joint.
Principal Spillway Conduit: As-Built Joint #19	• Steel plate on left side is missing.
Principal Spillway Conduit: As-Built Joint #22	• Steel plate on left side is missing.
Foundation Drains	• Right foundation drain animal guard is missing.

Note that inspection was done starting at the outlet, moving upstream.

The upstream slope was in good overall condition. Heavy vegetation was present in the riprap. Trees were present in the right upstream abutment. The crest of the dam had minor rutting from wheel tracks. The downstream slope had severe wheel rutting on the berm. It also had trees in both downstream abutments.

The auxiliary spillway had a large area with rutting from ATVs and a large area where soil has collapsed and eroded at the top of the left side slope. Numerous gopher holes were present throughout the spillway.

The deficiencies along the dam embankment and auxiliary spillway that were recorded during the dam inspection are summarized in an Inspection Checklist, which is provided in **Attachment 2. Attachment 2** also contains photographs of the embankment and auxiliary spillway of Larimore Dam.

3.3 AUXILIARY SPILLWAY

The auxiliary spillway experienced erosion in June 2000, following a significant rainfall with maximum point precipitation measured to be 20.0 inches. An Emergency Watershed Protection (EWP) program project was completed to regrade the erosion experienced during the event and remove trees along chute, which was completed in 2001. Approximately 150 feet of head cut advance is shown on the plan set, which plan and profile views are included in **Attachment 6**. With the information related to flood event and measured erosion the geotechnical parameters were re-evaluated and summarized in Addendum to Larimore Rehab Geotechnical Engineering Report, also included in **Attachment 6**.

4 DAM BREACH ANALYSIS AND HAZARD CLASSIFICATION

Based on NRCS and State of North Dakota criteria, the design of a dam's spillways and height is significantly affected by its hazard classification. Guidelines for determining a dam's hazard classification and subsequent design are established in Technical Release 210-60 (NRCS, 2019).

4.1 BREACH DISCHARGE CRITERIA

Equations found in Chapter 1 of *Technical Release 210-60 Earth Dams and Reservoirs* (NRCS, 2019) were used to develop breach discharge criteria for the Larimore Dam. The NRCS criteria states that a dam breach must take place at either the reservoir stage at the probable maximum flood (PMF) or top of Dam. Part I of Technical Release 210-60 contains equations that were used to compute the peak breach discharge for Larimore Dam. The peak breach discharge calculated for the dam was approximately 110,000 cubic feet per second. Peak breach discharge calculations and data are provided in **Attachment 3**.

The HEC-RAS modeling program (U.S. Army Corps of Engineers, 2023) version 6.4.1 was used to develop the downstream water surface profiles for the dam breach. Model and elements extents used in the model are shown in **7** (details on the development of the HEC-RAS model are explained in Section 5.2.1). The dam breach simulation was developed using tools within the HEC-RAS framework. A dam breach hydrograph was developed and used as a boundary condition in the hydraulic model with a peak flow of 110,000 cubic feet per second. The outflow hydrograph from the breach simulation is shown in **Figure 5**.

4.2 BREACH RESULTS

The inundation area produced from the simulated breach based on *TR 210-60* criteria is shown through the breach zone on **Figure 6**. More detailed views of the inundation mapping along with structures affected and roads overtopped throughout the breach zone are shown on **Figure 7** through **Figure 10**. The extent of the breach zone is established at the point where the breach scenario water surface profile converges with the 100-year synthetic rainfall event water surface profile.

Table 4 summarizes all residential structures that were potentially impacted by the dam breach. These structures are labeled in **Figure 7** through **Figure 10**. Structures were identified using hydraulic modeling extents and aerial photography. Data such as finished floor elevation of each structure, maximum inundation depth of the structure, maximum velocity of flow at the structure location, and the amount of time it would take for the breach discharge to reach the structure are shown in Table 4. An estimated 88 non-residential structures would be impacted in addition to

the residential structures impacted by the breach. The additional structures consist mostly of agricultural storage facilities, grain bins, and garage space.

There are various instances of roads being overtopped during the breach scenario. For this analysis, only roads with an average annual daily traffic (AADT) value greater than 400 are considered. This AADT value was chosen based on other similar studies. Smaller roads, such as township roads, are less likely to have vehicles on them during a breach. There were 27 roads in the breach zone with an AADT value greater than 400. **Figure 7** through **Figure 10** show the road overtopping locations. Information about the 27 overtopping locations is provided in Table 5. Table 5 provides data on maximum inundation depth where the road overtops, maximum velocity over the road, and the amount of time it would take for the breach discharge to overtop the road.

Table 4: Residential Structures Potentially Impacted by a Breach of Larimore Dam

Structure ID	Finished Floor Elevation (ft, NAVD88)	Depth (ft)	Velocity (ft/s)	Arrival Time ^[1] (hours)
S1	1030.5	8.7	6.5	0.6
S2	1029.3	9.7	3.7	0.6
S3	1029.6	9.4	3.3	0.6
S4	1034.1	5.9	6.4	0.6
S5	1035.7	3.4	3.0	0.6
S6	1034.3	4.9	1.0	0.6
S7	1033.3	5.8	2.8	0.6
S8	1022.6	6.7	3.1	0.8
S9	994.5	6.5	6.4	1.7
S10	992.3	6.4	6.8	1.7
S11	991.3	5.7	5.8	1.7
S12	992.1	5.4	5.9	1.7
S13	989.4	9.7	4.6	1.7
S14	989.3	9.5	4.5	1.7
S15	989.3	9.1	4.4	1.7
S16	989.4	8.8	4.2	1.7
S17	989.1	11.0	4.6	1.7
S18	988.4	11.7	4.4	1.7
S19	991.9	9.0	3.4	1.7
S20	990.5	9.8	3.3	1.7
S21	990.5	9.4	3.7	1.7

Grand Forks County WRD | Upper Turtle River Dam #9 (Larimore Dam)
DAM BREACH ANALYSIS AND HAZARD CLASSIFICATION

S22	990.8	9.2	3.8	1.7
S23	990.3	9.2	3.9	1.7
S24	991.0	9.4	4.9	1.7
S25	990.9	9.4	5.1	1.7
S26	991.6	8.6	6.0	1.7
S27	991.9	8.1	4.6	1.7
S28	992.8	7.1	2.9	1.7
S29	991.9	9.0	5.1	1.7
S30	975.5	13.1	4.6	2.2
S31	975.1	13.4	4.5	2.2
S32	977.8	1.3	4.9	2.1
S33	978.1	1.0	4.1	2.5
S34	974.7	3.7	7.6	2.5
S35	972.7	7.2	4.4	2.5
S36	971.1	6.8	4.1	2.5
S37	959.8	4.6	6.6	2.8
S38	961.5	5.9	7.1	2.7
S39	960.9	6.8	6.8	2.7
S40	926.2	3.6	2.6	4.3
S41	922.2	2.6	2.5	4.7
S42	922.2	2.6	2.5	4.7
S43	918.0	1.5	6.4	4.7
S44	919.9	2.2	6.6	4.7
S45	916.3	0.2	0.8	4.9
S46	915.9	0.4	0.9	4.9
S47	911.7	4.9	2.8	4.9
S48	914.4	3.2	3.1	4.9
S49	861.2	0.4	0.5	10.6
S50	863.2	0.9	0.6	10.6
S51	862.8	0.9	2.4	8.8
S52	863.2	0.6	2.4	8.8
S53	863.8	1.2	1.3	8.9
S54	863.6	0.8	2.4	9.0
*S55	1040.5	1.0	0.9	0.8
*S56	1031.0	10.2	7.9	0.6

Grand Forks County WRD | Upper Turtle River Dam #9 (Larimore Dam)
DAM BREACH ANALYSIS AND HAZARD CLASSICIATION

*S57	1024.7	8.6	8.8	0.7
*S58	1033.7	2.0	4.9	0.7
*S59	1000.5	0.5	4.2	1.7
*S60	992.7	9.0	4.1	1.7
*S61	991.1	7.1	5.3	1.7
*S62	980.5	10.0	2.2	2.2
S63	976.3	13.1	1.5	2.2
S64	975.4	13.2	4.4	2.2
*S65	978.9	1.9	5.8	2.1
*S66	977.7	3.3	8.0	2.4
*S67	977.8	1.9	7.4	2.5
*S68	978.1	1.7	7.5	2.5
*S69	976.4	3.2	7.5	2.5
*S70	958.8	9.2	7.2	2.7
S71	964.9	2.1	1.5	2.7
S72	963.8	1.5	2.0	2.8
*S73	964.0	0.9	2.8	2.8
S74	958.1	2.6	1.0	3.5
*S75	918.2	0.4	0.9	4.9
*S76	918.3	0.2	0.8	4.9
*S77	917.9	0.5	0.9	4.9
*S78	918.1	0.3	0.9	4.9
S79	909.1	4.7	2.2	5.1
S80	913.1	0.5	2.9	5.0
*S81	910.7	2.8	2.8	5.0
*S82	888.5	0.6	2.0	6.9
*S83	888.9	0.4	2.4	6.9
*S84	879.4	0.6	1.2	7.9
*S85	886.2	1.8	2.6	6.9
*S86	887.3	0.9	1.8	6.9
*S87	873.6	0.3	0.6	8.4
*S88	873.3	0.7	1.0	8.4
*S89	859.2	0.7	0.4	14.3
*S90	863.2	0.8	0.9	11.2
*S91	862.0	0.9	0.4	10.7

Grand Forks County WRD | Upper Turtle River Dam #9 (Larimore Dam)
DAM BREACH ANALYSIS AND HAZARD CLASSIFICATION

*S92	862.7	0.3	1.6	11.5
*S93	861.3	0.9	0.4	11.4
*S94	862.2	0.1	0.6	10.5
*S95	862.5	0.2	0.7	10.7
*S96	860.9	0.5	1.5	11.0
*S97	861.0	0.1	1.1	11.0
*S98	861.1	0.8	2.0	11.0
*S99	861.9	0.2	1.8	11.0
*S100	862.3	0.5	1.1	10.7
*S101	862.7	0.2	1.5	10.7
*S102	862.1	0.8	0.5	10.7
*S103	862.8	0.1	0.5	10.7
*S104	862.3	0.5	0.7	10.7
*S105	862.1	0.5	1.9	10.7
*S106	861.0	0.0	0.1	10.9
*S107	858.7	2.3	1.4	10.9
*S108	859.3	1.7	1.1	10.9
*S109	861.4	0.7	1.2	10.6
*S110	861.0	0.8	1.1	10.5
*S111	860.8	1.1	1.5	10.4
*S112	860.7	0.9	2.0	10.4
*S113	853.6	1.5	0.8	13.5
*S114	854.7	0.5	1.1	13.5
*S115	858.6	1.4	1.3	10.9
*S116	859.6	0.6	1.3	10.9
*S117	858.5	0.5	1.1	12.1
*S118	859.1	0.8	0.4	14.2
*S119	857.5	0.3	0.1	13.7
*S120	857.0	0.7	0.9	13.7
*S121	856.5	1.2	0.9	13.7

* Surveyed finished floor elevation was unable to be obtained. Floor elevation estimated from available LiDAR data

[1] Breach arrival time is relative to the initiation of the dam breach

Table 5: Road Overtopping Data During a Breach of Larimore Dam

Road ID	Depth (ft)	Velocity (ft/s)	Arrival Time (hours)
R1	10	8.6	0.6
R2	8.4	8.8	0.7
R3	6.6	10.6	2.2
R4	7.2	8.2	2.0
R5	6	5.4	2.7
R6	8.4	7.6	2.7
R7	9.3	3.6	2.9
R8	7	7.3	4.0
R9	8.3	6.6	4.2
R10	9.1	8.5	4.8
R11	6.3	5.2	5.6
R12	3.1	4.1	5.4
R13	9.1	6.6	6.6
R14	6.8	2.8	6.9
R15	2	2.8	8.6
R16	2.7	3	7.9
R17	2.9	4.1	10.5
R18	1.4	5.4	7.9
R19	1	6	10.6
R20	0.8	1.7	13.9
R21	4.5	2.5	12.3
R22	4.6	1.5	15.5
R23	4.2	2.3	20.3
R24	2.3	1.1	20.6
R25	1.7	2.6	10.6
R26	0.9	1	12.2
R27	0.5	0.9	10.3

[1] See Figures 7 through 10 for road overtopping locations

[2] Breach arrival time is relative to the initiation of the dam breach

The structures and roadways listed in Table 4 and Table 5 were analyzed further to determine if there is the potential for loss of life during a breach of the magnitude described. Depth and velocity flood danger level relationships established in *Downstream Hazard Classification Guidelines* (U.S. Bureau of Reclamation, 1988) were used to determine which structures and roads have a high danger potential during a breach at Larimore Dam.

The chart from *Downstream Hazard Classification Guidelines* that shows the depth-velocity-flood danger level relationship for homes built on foundations is shown on **Figure 11**. **Figure 11** also plots the structures corresponding to Table 4. Structures plotted in the red fall in the category of high danger level, indicating that loss of life is likely. Structures in the yellow fall into what is called the judgement zone, where some level of engineering judgement should be used to determine if the structure has a high or low danger potential. Structures plotted in the green area have a low danger level, depicting low potential for loss of life. **Figure 11** depicts 15 structures in the red (high danger) zone and five structures in the yellow (judgment) zone. Therefore, a total of 15 out of the 121 total structures have a high danger potential for loss of life if Larimore Dam were to breach with the magnitude described in Section 4.1.

There are 15 structures that have a high danger potential and five structures in the judgement zone, while the remaining 101 structures have a low danger potential.

The hazard potential for habitable structures was also reviewed based on guidance in the National Engineering Manual (NRCS, 2017), which indicates that products of four or greater that result from depth (in feet) and velocity (in feet per second) combinations could result in loss of life. The 15 structures identified as high danger potential all have depth and velocity products greater than four and structures in the low danger potential category have depth and velocity combinations that result in a product of less than four, with exception to structure 33, with a product of 4.1. Therefore, the methods used to identify habitable structures within the breach zone that may experience loss of life during a breach were verified by criteria in the National Engineering Manual.

Another chart in *Downstream Hazard Classification Guidelines* (U.S. Bureau of Reclamation, 1988) shows the depth-velocity-flood danger level relationship for passenger vehicles, shown on **Figure 12**. **Figure 12** depicts 27 road overtopping locations (listed in Table 5 and shown on **Figure 7** through **Figure 10**). **Figure 12** shows that 18 overtopping locations fall in the high danger category and loss of life due to flooding over these roads is likely.

4.3 HAZARD CLASSIFICATION

The hazard potential resulting from failure of dams is described in *Title 210, National Engineering Manual, Part 520 Subpart C "Dams"* (NRCS, 2017). According to this guidance, high hazard potential is "Dams where failure may cause loss of life or serious damage to homes, industrial or commercial buildings, important public utilities, main highways, or railroads."

Article 89-08 of the North Dakota Century Code (ND SWC, 2015) outlines a similar definition where a high hazard dam is defined as “A dam located upstream of developed or urban areas where failure may cause serious damage to homes, industrial and commercial buildings, and major public utilities. There is potential for the loss of more than a few lives if the dam fails.” Based on the data presented in Section 4.2, the high hazard designation for Larimore Dam was confirmed.

5 HYDROLOGIC AND HYDRAULIC MODELING

5.1 HYDROLOGY MODEL

Several hydrologic modeling efforts have been completed or are currently underway for other projects in the Turtle River Watershed. This section summarizes the current and ongoing hydrologic efforts in the watershed.

5.1.1 2013 USACE HEC-HMS MODEL

In 2013, the United States Army Corps of Engineers (USACE) developed a Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) model for the Turtle River Watershed in version 3.5. The model along with many others was created to support flood control preliminary engineering for the Red River of the North. The results of the model are documented in the *Red River of the North Hydrologic Modeling – Phase 2* (USACE, 2013).

The Turtle River Watershed modeling completed by USACE in 2013 was calibrated to the June 2000 event, June 2002 event, and the June 2009 event. While modeling goals were not met at all locations of the larger study, the study aimed to match observed peaks within 10-percent, volume within 10 percent, and match the observed timing by 0.5 days. All subbasins contributing to calibrated stream gages were adjusted proportionally by scaling the initial values up or down by the same ratio.

5.1.2 2015 SITES MODEL

A hydrologic and hydraulic analysis of the Larimore Dam watershed was completed using NRCS’s Site Analysis Computer Program (SITES) model, version 2005.1.8. The goal of the modeling was to determine if Larimore Dam met NRCS and State of North Dakota dam safety criteria. The model was run using physical data collected at the dam and standard parametrization, independent of events that had occurred in the watershed’s historic record.

SITES evaluates dam safety using the NRCS’s Technical Reference 60 (TR-60). TR-60 evaluates dam safety using specific storms to determine the capacity of the principal spillway, auxiliary spillway, and dam crest. The 2015 SITES critical assumptions included uncalibrated hydrology model, original storage-elevation curve, and only the downstream first stage inlet opening present. SITES does not have the ability to input two different principal spillway first stage opening sizes and invert elevations. The SITES evaluation found that the principal

spillway, auxiliary spillway, and dam crest were undersized. The SITES modeled estimated that the auxiliary spillway needed to be at elevation 1095.4 feet (NAVD88) versus the existing 1090.5 feet (NAVD88) and that the current dam crest would be overtopped by 7.3 feet.

5.1.3 2023 NRCS HEC-HMS MODEL

The NRCS adapted the 2013 USACE HEC-HMS model to complete a Watershed Plan/Environmental Assessment for Upper Turtle River Dam #10 under the Watershed Flood Prevention Operations (WFPO) Program. The curve numbers are associated with 24-hour duration.

The HEC-HMS model was updated from version 3.5 to version 4.9. The NRCS calibrated the hydrologic model to the 2022 rain on snow event in addition to the June 2000 flood event. The addition of the 2022 frozen ground calibration was done to support calibration of a HEC-RAS model of the same event. Synthetic events were evaluated and compared a Bulletin 17C analysis completed at the United States Geological Survey (USGS) Turtle River near Arvilla stream gage, gage number 5082625.

Modifications to the model were made beyond what was originally developed by the NRCS, including the removal of a split flow junction that removes water from Upper Turtle River Dam #9. The split flow junction is along Whisky Creek between UTR Dam #6 and start of Channel H. The split flow was developed with 2D hydraulic model of existing topographic conditions, and represented a realistic overland breakout from South Branch Turtle River watershed to North Branch Turtle River watershed. However, this breakout/split flow was removed because it removes water from the South Branch watershed, which could be maintained with future berm or unexpected backwater condition. Since this is a dam safety study, this level of conservatism seemed appropriate.

5.1.4 SUBBASIN BOUNDARIES

Subbasins were developed as part of the analysis for Upper Turtle River Dam #10 by the NRCS. Subbasin boundaries were reviewed and adopted for use in evaluating Upper Turtle River Dam #9. The NRCS delineation focused on matching the HEC-RAS model geometry, such as hydraulic routing, storage locations, road crossing and size limitations for design.

5.1.5 SUBBASIN PARAMETERS

Subbasin parameters were originally developed as part of the *Turtle River Watershed Plan Appendix D-1: Existing Conditions Hydrology and Hydraulics Report* (NRCS, 2023) effort mentioned previously. Additional details on the various input parameters used in the hydrologic model are provided in the following sections.

5.1.5.1 CURVE NUMBERS

Curve numbers utilized as part of the UTR 9 evaluation were developed by NRCS and adopted from UTR10 study. The initial curve numbers developed by the NRCS were developed for the

2000 summer rainstorm using methods described in Part 650 NEH Chapter 2: Estimating Runoff Volume and Peak Discharge (NRCS, 2021). The curve numbers developed for this event were adopted as 24-hour curve numbers and are illustrated in **Figure 13**.

5.1.5.2 TIME OF CONCENTRATION

The NRCS calculated the time of concentration to the summer 2000 precipitation event with the watershed lag method (NRCS, 2010). Equation 15-4b was used for seven subbasins in the upper Turtle River watershed, considering flow length, watershed land slope, and runoff curve number (variable). These calculated T_c were plotted against drainage area (DA) size, resulting in equation $T_c = -0.0149DA^2 + 0.9581DA + 3.1173$. Furthermore, five other watersheds were used for validation, one of which fit on the line, two above and two below. The remaining 97 subbasins were applied time of concentrations based on equation and drainage basin size. T_c for each subbasin is illustrated in **Figure 14**. Total T_c for Upper Turtle River Dam #9 is approximately 36 hours.

5.1.5.3 CLARK'S STORAGE COEFFICIENT

The NRCS used Clark's Storage Coefficient (R) as the main calibration component to the USGS gage at Arvilla, gage number 05082625. R was determined from T_c and R/T_c ratio. The R/T_c ratio is based on amount of non-contributing area, landscape slopes and drainage. The NRCS evaluated the non-contributing area based on a shapefile provided by IWI (IWI, 2022). Additionally, R was calibrated based on water surface elevations at the dams above Larimore Dam. R is illustrated in **Figure 15** for each subbasin.

5.1.6 RESERVOIR PARAMETERS

Upper Turtle River Dam #6, #7, and #8 all exist upstream of Upper Turtle River Dam #9 (Larimore Dam). Storage-discharge and elevation-storage curves were utilized in the HEC-HMS model for Upper Turtle River Dam #6 and #8. Upper Turtle River Dam #7 utilized storage-discharge curves in the HEC-HMS model. During large events, such as the PMP, all dams upstream of Larimore Dam are close to inflow being equal to outflow. All dams were assumed to be sustained, following their discharge curves, during large events.

There are two substantial changes to Larimore Dam parameters from past modeling efforts, including storage curve and outflow rating curve. The storage volume curve from Appendix D-7 Accumulated Sediment, Table 5 is adopted in all hydrology and hydraulic models; which accounts for the estimated 186 ac-ft of sediment accumulation from 2020 bathymetry survey as well as LiDAR calculations above the water surface. There is a shift in elevation up to ~2 feet for same storage volume in updated curve from past curves, see **Figure 4**. The updated storage-outflow curve accounts for adding second first-stage orifice, higher auxiliary spillway elevation, and auxiliary spillway adjustable weir coefficients. Historic models did not include the upstream first stage orifice (4.5-foot width by 0.75-foot height) due to SITES model limit of only one first stage orifice. The downstream (4.5-foot width by 2.5-foot height) first stage orifice was the only one included in model as it was the larger opening. The combined first and second

stage orifice curve is adopted from updated SITES model where both openings were modeled individually and combined. The recent survey and LiDAR find the auxiliary spillway is 0.5 feet higher than as-built level. The auxiliary spillway flows are calculated with adjustable weir coefficients based on TR-39 (USDA, 1968). Due to the resistance of grass lined very long level section, the weir coefficient can be lower with small depth and raise to typical values with higher depths. TR-39 guide curves indicate for 1-foot of depth the weir coefficient (WC) is 0.8, 3-feet of depth WC=1.1, 5-feet of depth WC=1.7, and 10-feet depth WC=2.2. The 2015 Assessment assumed straight line from invert to top of dam, which matches at some points of the graph, but different at others; see **Figure 16**.

5.2 HYDRAULIC (HEC-RAS) MODEL

Flooding concerns in the Red River Valley prompted substantial study of the region. The United States Army Corps of Engineers (USACE) completed a Hydraulic Engineering Center's River Analysis System (HEC-RAS) model of the Turtle River as part of the Red River of the North Hydrologic Modeling – Phase 2 effort (USACE, 2013). This model has been modified several times for purposes within the basin.

Channel cross sections, 1-dimensional storage areas, and 2-dimensional storage areas in the HEC-RAS model schematic are shown on **Figure 17**.

5.2.1 NRCS UTR10 DOWNSTREAM HYDRAULIC MODEL

The NRCS modified the USACE Phase 2 HEC-RAS model for the purpose of developing a Watershed Plan (NRCS, 2023) for Upper Turtle River Dam #10. Modifications to the HEC-RAS model are documented in *Tongue River Watershed Plan Appendix D-1: Existing Conditions Hydrology and Hydraulics Report* (NRCS, 2023). Some of the modifications include bringing the model forward to HEC-RAS version 6.2, extension of the model, bathymetric changes, two-dimensional areas, Manning's Roughness Modifications, and weir coefficient modifications. The NRCS calibrated the HEC-RAS model to the 2022 April rain on snow event.

5.2.2 UPDATED DOWNSTREAM HYDRAULIC MODEL

Minor modifications to the HEC-RAS model were made including reducing the computational time step, reducing the two-dimensional grid cell size from 1100x1100 feet to 200x100 feet, and modification of lateral structure weir coefficients in the two-dimensional region. These modifications were made to increase detail in downstream regions and reduce instability on the downstream section of the hydraulic model. Calibration accuracy was maintained while updating these parameters from the NRCS's downstream model.

An unsteady run was completed for low level outlet for consideration of emergency drawdown, which is a 24" diameter pipe with critical invert elevation of 1045.7. The North Dakota State Dam Safety requirement to release 5-feet of reservoir volume within 5 days is easily met by the existing low level outlet pipe which can drawdown the top 12 feet in 5 days.

5.2.3 PROBABLE MAXIMUM PRECIPITATION

The probable maximum precipitation (PMP) event for Larimore Dam was obtained based on a recent study for updated PMP depths in the state of North Dakota. Guidance for using the state's PMP GIS tool was obtained from the North Dakota Department of Water Resources website. The general PMP was evaluated for the 6-hour, 24-hour, 48-hour, and 72-hour durations using temporal distributions provided by the tool. Local storm distributions were deemed ineffective due to the size of the basin. A Cool Season Storm was not critical compared to the General Storm. The storm distribution center over Larimore Dam's watershed was also utilized as part of this study.

5.3 SPILLWAY DESIGN HYDROGRAPHS

Designation of a dam's hazard classification helps determine what events a dam must be able to pass to meet the State of North Dakota and the NRCS's safety standards. Technical Release 210-60: Earth Dams and Reservoirs (NRCS, 2019) provides guidance on what storms must be utilized for design of the structure and how to develop each of the design events. Based on results previously presented in Section 4, Larimore Dam is classified as a high hazard dam. **Table 6** provides the controlling precipitation criteria for high hazard dams based on the analysis completed to Technical Release 210-60 standards. Details regarding the creation of each design hydrograph are detailed in the following subsections.

Table 6: TR 210-60 Minimum Precipitation Data for High Hazard Dams

Design Event Hydrograph	Hydrologic Criteria ^[1]	Duration	Depth (inches)
Principle Spillway Hydrograph (PSH)	P_{100}	10-day	4.69 ^[2]
			8.07 ^[3]
Auxiliary Spillway Hydrograph (ASH)	$P_{100} + 0.26 (PMP - P_{100})$ ^[4]	6-hour	6.35
		12-hour	7.48
		1-day	9.37
		2-day	10.22
Freeboard Hydrograph (FBH)	PMP ^[5]	6-hour	12.96
		12-hour	15.27
		1-day	20.45
		2-day	21.23

[1] P_{100} represents precipitation for the 100-yr return period. PMP is the probably maximum precipitation

[2] Runoff depth based on NEH Part 630 Chapter 21 Figure 21-2, 10-day duration

[3] Rainfall depth based on NOAA Atlas 14, 10-day duration (8.31 inches), with areal adjustment factor of 0.971

[4] P_{100} from NOAA Atlas 14 published rainfall depth for corresponding duration (24-hour), aerially adjusted

[5] PMP from statewide PMP study for North Dakota, value aerially adjusted at centroid

5.3.1 PRINCIPAL SPILLWAY INFLOW HYDROGRAPH

Technical Release 210-60 provides the criteria for events that the principal spillway of a high hazard structure must pass. The principal spillway must pass a critical design event without activating the auxiliary spillway. The critical event for a dam is the design event that provides the most conservative answer. Based on Technical Release 210-60 events to analyze to determine the critical event include the 100-year 10-day hydrologic event and principal spillway mass curves outlined in the National Engineering Handbook Part 630, Chapter 21. The resulting depth for the 10-day duration rainfall event considered for this analysis is shown in **Table 6** along with the runoff depths used to simulate the runoff volume maps procedure described previously.

The 100-year 10-day rainfall depth was obtained from *NOAA Atlas 14* (NOAA, 2022). The rainfall depth was then analyzed using the hydrologic model developed and described in section 5.1.3. No areal reduction is applied to this runoff volume. The rainfall 10-day Curve Number with an average moisture condition (AMC II) was used; 10-day Curve Number was calculated using Chapter 21, Table 21-2. Similar to runoff volume calculations, there is no adjustment for channel losses. The Frequency Storm 75% intensity position was adopted as it produced the most critical results compared to other positions available (25, 33, 50, 66%); this intensity position still nests all smaller durations precipitation depths, but positions them at 75% of the full (10-day) duration.

Runoff volume Curve Numbers in the HMS hydrologic model are set to 99, initial abstraction set to zero, and impervious areas set to 100%, causing entire 4.69 inches to runoff as excess volume. The principal spillway mass curves outlined in the National Engineering Handbook Part 630, Chapter 21, were developed using 15-minute time increments for this study. Curve A was arranged in decreasing order, Curve B was arranged in increasing order, and Curve C was arranged in center oriented. The mass curve are documented in **Figure 18**.

Quick Return Flow (QRF) is the rate of discharge that persists beyond the flood period of the principal spillway hydrograph. Based on guidance from *Chapter 21 (Figure 21-4)*, the QRF for UTR 10 Dam is approximately 1.6 cubic feet per second, per square mile. The drainage area to the dam is 65.5 square miles, which results in a QRF of 104.8 cubic feet per second (cfs). There is no adjusting for channel losses because the climatic index is 1.38. The critical principal spillway mass curve was determined to be Curve A, which require the auxiliary spillway invert to be 1091.6. Since the 15% drawdown requirement was not met for Curve A, the additional volume remaining was added to peak volume to determine auxiliary spillway invert minimum to be 1091.6. The reservoir elevation after 10-days (1081.1) shall be used for starting level for auxiliary and freeboard hydrographs. The calculated QRF was added to the trailing limb of the critical principal spillway discharge hydrograph in SITES and RAS. All three mass curve and Curve Number (CN) with 10-Day 100-year rainfall hydrographs and resulting reservoir elevations are plotted in **Figure 19**, and summaries included in **Table 7**.

All events were used to determine whether the principal spillway meets the State of North Dakota and NRCS design criteria for high hazard dams. The hydrographs for each event were

applied to the SITES, and validated with RAS, model developed as part of this study to determine if this criterion was met and which event was critical.

Table 7: Principal Spillway Inflow Hydrograph Data

PSH	Peak			10-day Drawdown			AS Invert
	Inflow (cfs)	Reservoir Elevation	Volume (ac-ft)	Reservoir Elevation	Volume Remaining (ac-ft)	Storage Remaining	
Volume A	1,268	1087.7	3,429	1081.1 [!]	1,342	39%	1091.6*
Volume B	1,945	1090.6	4,349	1072.9	356	8%	1090.6
Volume C	1,775	1089.6	4,008	1075.1	560	14%	1089.6
100yr 10Day	1,441	1087.2	3,290	1072.9	356	11%	1087.2
*Critical (Peak + Remaining Volume)							
! 10-DD is SDH & FBH starting WSEL							

5.3.2 AUXILIARY SPILLWAY STABILITY DESIGN HYDROGRAPH

The capacity of the auxiliary spillway should be adequate to pass the auxiliary spillway hydrograph in combination with either wave height, or frost conditions, without overtopping the dam embankment based on requirements in *TR-60*. Stability of grass lined spillways also need to meet threshold vegetation and soil stress criteria. Stability design analysis is analyzed for short duration storm. The ASH starting reservoir level is 1081.1, from PSH critical results. The adopted ASH duration is 12-hours. The Time of Concentration (T_c) for UTR Dam 9 is ~36 hours, therefore 48-hour duration is analyzed for FBH. The ASH is short-duration check, for smaller basins typically 6-hours, or 25% of FBH 24-hour duration. However, since this is a larger basin with $T_c > 24$ -hours, the ASH duration is similarly chosen as ~25% of FBH duration, which is 12-hour duration.

Frequency Storm Meteorological model incremental durations use nested values from lower duration tool results. The critical 6-hour rainfall ratio is ~63%, which is more than Figure 21-9 (~57%) and similar to 5-point ratio (~63%). However, the shorter durations, i.e. 1, 2, and 3 hour values are more critical than NEH and 5-point. The Frequency Storm 75% intensity position was adopted as it produced the most critical results compared to other positions available (25, 33, 50, 66%); this intensity position still nests all smaller durations precipitation depths, but positions them at 75% of the full (12-hour) duration. The storms were simulated in the HEC-HMS model to develop inflow hydrographs at Larimore Dam. The inflow hydrographs and routing at Larimore Dam relevant to the 12-hour auxiliary spillway hydrograph are shown on **Figure 20** and peak inflows are provided in **Table 8**.

Table 8: Auxiliary Spillway Inflow Hydrograph Data and Results

ASH	Peak				
	Inflow (cfs)	Reservoir Elevation	Volume (ac-ft)	AS Depth (ft)	Freeboard (ft)
6-hour	1,898	1088.1	3,663	-2.4	12.7
12-hour	2,479	1090.9	4,455	0.4	9.9
24-hour	3,457	1093.2	5,583	2.7	7.6
48-hour	4,183	1093.7	5,812	3.2	7.1

5.3.3 FREEBOARD INFLOW HYDROGRAPH

The minimum design event for the freeboard hydrograph (FBH) associated with a high hazard dam is defined by *TR 210-60* as the probable maximum precipitation event described in Section 5.4. The duration of the FBH was developed based on guidance from *TR 210-60*. *NEH, Part 630, Chapter 21* states that a storm duration equal to or greater than the time of concentration shall be analyzed. The time of concentration for the watershed upstream of Larimore Dam is ~36 hours. For this analysis, durations beyond the time of concentration, documented in 5.1.6.2, for the watershed upstream of the dam were analyzed, and the most critical duration was evaluated for spillway adequacy.

Two durations were analyzed using North Dakota statewide PMP depths. The durations analyzed were the 24- and 48-hour PMP events. The rainfall distribution used for each of those events was obtained from the North Dakota statewide PMP tool, developed by the North Dakota Department of Water Resources. The aforementioned precipitation is conservatively applied to 24-hour Curve Number basin from Existing Conditions Report. Interpolating a 48-hour CN was determined to be not reasonable. The default of centering the PMP event over Larimore Dam's watershed was assumed. The spatial distribution selected by the PMP tool was based on the Parkman, Saskatchewan storm in August of 1985, as it was the most critical. The general synthetic temporal distribution from the statewide PMP tool was utilized for the study. Additionally, a five-point type-II SCS hyetograph of the maximum 24-hour PMP was evaluated and compared to the critical stacking. The five-point SCS hyetograph was less conservative than the critical stacking distribution, so the critical stacked distribution was used throughout the study. The maximum PMP precipitation depth and peak inflow to Larimore Dam is shown in **Table 9**. The inflow hydrographs to Larimore dam are illustrated in **Figure 21**. The inflow hydrographs to the dam were input into the SITES and RAS programs to determine if Larimore Dam has appropriate freeboard; the results of these two programs were very similar, however RAS is slightly more conservative and therefore those results documented.

The ND DWR Dam Safety Standards (NDDWR, 2024) require auxiliary spillway to safely pass IDF without accounting for capacity of principal spillway. Therefore, the same PMP assumptions as above are used, however without capacity of principle spillway (w/o PS) with results included in **Table 9**.

Table 9: Freeboard Inflow Hydrograph Data

FBH	Peak				
	Inflow (cfs)	Reservoir Elevation	Volume (ac-ft)	Residual Freeboard (ft)	Outflow (cfs)
24-hour	15,506	1098.5	8,116	2.3	14,369
48-hour	15,575	1098.6	8,178	2.2	14,708
48-hour w/o PS	15,575	1099.1	8,495	1.7	14,918

6 GEOLOGIC INVESTIGATION

HDR Engineering Inc. completed a geologic investigation in 2021 at Larimore Dam, and the subsequent geotechnical engineering report was completed in February 2022. The purpose of the geologic investigation was to develop subsurface profiles, and evaluate the dam's foundation and embankment soils, seepage and slope stability, and auxiliary spillway integrity. The Geotechnical Engineering Report is provided in **Appendix D-2**.

Throughout 2020 and 2021, four soil borings were completed on the embankment, and three soil borings were completed within the auxiliary spillway to gather in situ soil data, and collect soil samples for lab testing. Pertinent tests completed include hydraulic conductivity, consolidation, triaxial compression, crumb test (dispersion) Atterberg limits, sieve analysis, in situ density and soil moisture content. A map showing the location of each soil boring location along with the laboratory test results are presented in **Appendix D-2**.

Geotechnical analyses were performed with the data obtained from the geotechnical exploration and laboratory test results. The analyses completed included SITES analyses, drain fill compatibility analysis, seepage and slope stability analyses. The results of the SITES analyses are presented in **Section 7**.

The compatibility analysis completed for the foundation drain fill indicates that the existing fill material does not meet state-of-the-practice gradation criteria for seepage control/conveyance.

Gradational analyses show that the existing drain fill is too coarse to provide adequate filtration and does not account for the potential variability and dispersiveness of the embankment fill.

Seepage analyses through the dam embankment were performed to estimate the phreatic surface in the embankment during normal pool and flood surcharge pool levels for the purpose of evaluating slope stability. Results from the shear strength tests were used to accurately predict slope stability of the embankment. The results of the seepage and slope stability analyses indicate that the dam meets current Technical Release 210-60 requirements if the current upstream and downstream slopes are maintained.

7 SITES ANALYSIS

The NRCS's SITES modeling platform was utilized to determine if the existing dam structure was designed to handle inflow events based on its high hazard designation. Physical information regarding the reservoir, principal spillway, and auxiliary spillway were entered utilized in the model. Additionally, geotechnical information was entered to help identify the auxiliary spillway's integrity/stability. Results from the SITES analysis would determine whether the existing structure meets State of North Dakota and NRCS standards or if modifications to the structure are warranted to bring it into compliance.

7.1 SITES MODEL INPUTS

Physical information regarding the reservoir, principal spillway, auxiliary spillway, and geotechnical information are required to analyze a dam in SITES. Inflow hydrographs to the dam were developed as part of this study and are described in earlier sections of this report and can be found in **Figure 19**, **Figure 20**, and **Figure 21**.

Reservoir information utilized by SITES includes the elevation-storage relationship and the permanent pool elevation. The elevation-flood storage relationship for Larimore Dam is illustrated in **Figure 4**. The permanent pool elevation utilized for the analysis was elevation 1068.9 feet, which is the pool elevation of the crest of the low stage principal spillway orifice.

The principal spillway for Larimore Dam is a two-stage concrete riser tower. The riser tower first stage consists of 2.5-foot by 4.5-foot rectangular orifice opening at elevation 1068.9-feet (NAVD88) on downstream side and 0.75-foot by 4.5-foot rectangular orifice opening at elevation 1068.4 on upstream side. The second stage includes a 24-foot overflow weir at elevation 1083.2-feet (NAVD88). Elevations of the orifice and overflow weir elevation of the riser tower were collected during the survey of the structure. The size of the orifice and overflow weir were determined from as-built drawings and verified by survey data. Information on the conduit going from the riser tower through the embankment and to the outlet of the structure was also entered. The length of the conduit was determined from as-built drawings and verified by aerial imagery. The conduit size was also determined from as-built drawings and was verified by survey in the

field. The elevation of the conduit is based on survey data. The plot of reservoir volume vs. outflow is summarized in **Figure 16**; there is a noticeable increase in flow from first stage orifice as previous models only included the 2.5-foot by 4.5-foot opening.

The auxiliary spillway for Larimore Dam is an earthen/grass lined channel that runs along the north side of the embankment. Survey and as built data of the auxiliary spillway was used to develop the inputs of the spillway for use in SITES. The spillway consists of a 300-foot bottom width, 3 to 1 channel side slopes, and has an exit slope of approximately 15% downstream of the crest.

Geotechnical information included in the SITES model was used to help determine whether the auxiliary spillway would erode, completing the NRCS's standard Integrity Analysis. A plasticity index of 14, dry density of 110 lbs/cu ft, head cut index of 0.16, percent clay of 25.4, and a representative diameter of 0.009 were entered into the SITES model to make up the material properties of the auxiliary spillway. The head cut index value of 0.16 was adopted based on the Addendum to Larimore Dam Rehab Geotechnical Engineering Report (**Attachment 6**). The addendum documents that the Geotechnical Engineering Report original calculation of head cut index was 0.035. However, using calibration data from 2000 flood event that resulted in ~150 feet of head cut advance resulted in adoption of $K_h = 0.16$. The 48-hr FBH breach profile of the auxiliary spillway is shown in **Figure 24**.

7.2 PRINCIPAL SPILLWAY RESULTS

The design criteria for a high hazard dam requires the critical event to be passed without activation of the auxiliary spillway. The results of the SITES analysis yielded Curve A and B from **Table 7**. *Technical Reference 210-60* requires the principal spillway capacity should empty at least 85 percent of the principal spillway hydrograph routed through the retarding pool in 10 days or less. If more than 15 percent of the retarding storage volume remains after 10 days, the elevation of the crest of the auxiliary spillway should be raised by adding the volume remaining after 10 days to the initial retarding storage volume to determine the raised auxiliary spillway crest elevation. Based on this, the auxiliary spillway crest (1091.0) in existing conditions is 0.6 feet lower than required by critical Volume A analysis. **Therefore, the dam current features do not meet PSH requirements.**

7.3 AUXILIARY SPILLWAY RESULTS

The design criteria for auxiliary spillways of earthen dams should be analyzed for discharge capacity, stability, and integrity. The freeboard hydrograph, which is the probable maximum flood (PMF), is used to analyze the capacity and integrity of the dam. The PMF is determined by routing the PMP through a hydrologic model, in this case HEC-HMS. The events used to size a high hazard dam's auxiliary spillway are presented in **Table 6**. *Technical Reference 210-60* requires high hazard dams to pass the PMF without overtopping the dam. Durations of the PMF included in the study include 24-hours through 48-hours using general storm PMP depths

obtained from the statewide PMP study for North Dakota. Results for the existing top of dam elevation and required top of dam elevation with PMP depths from all scenarios described are presented in **Table 9**. The 48-hour PMP event is the controlling duration based on the generated water surface elevation. The dam would not need to be raised under any duration, as the existing top of dam is 1100.8 feet (NAVD88), and the controlling event yields a maximum water surface elevation of 1098.6 feet (NAVD88), which results in 2.2 feet of residual freeboard. The ND DWR residual freeboard is based on 10% hourly exceedance probability wind, or a typical wind that would occur during an average 10-hour period (ND DWR, 2024). The 10% exceedance wind speed is 21 mph (WeatherSpark, 2024). The effective fetch length at reservoir level 1099.1 is 1,652 feet using 14 radial lines at 6-degree intervals (see **Figure 25**). Based on equations 3, 6, 8, 9, and 10 (NRCS, 2014), the significant wave height is 0.39 feet, resulting in a computed residual freeboard of 0.94 feet which is less than the 1.0 foot minimum freeboard required.

Technical Reference 210-60 (NRCS, 2019) recommends a short-duration storm should be used to check the stability of vegetated auxiliary spillways. The stability of the auxiliary spillway for Larimore Dam was evaluated using the stability design hydrograph that utilized the 12-hour general ASH event. The auxiliary spillway stability design hydrograph, which was described in earlier sections, must be passed without eroding the auxiliary spillway, as per Technical Reference 210-60. NRCS staff developed a spreadsheet using the criteria provided in Technical Reference 210-60 that was used to develop allowable shear stresses for this study. The peak reservoir level for stability design is 1090.9, which is 0.1 feet below the invert of the auxiliary spillway. Therefore, the modeled peak auxiliary spillway exit velocity is 0 ft/sec, soil shear stress is 0.0 psf, and vegetal stress is 0.0 psf. The soil allowable stress is 0.14 psf and vegetal allowable stress 4.2 psf. Overall, the auxiliary spillway for Larimore Dam is considered stable because the stresses produced during the stability design hydrograph are less than the allowable stresses.

The ND DWR standard freeboard is based on 100 mph wind-generated setup and runup (ND DWR, 2024). The NRCS federal requirement is based on 50-year recurrence, which based on Figure 4 for the UTR Dam 10 site is ~80 mph (USDA, NRCS, 2014). Therefore, the ND state standard is critical. The effective fetch length at reservoir level 1090.9 feet using 14 radial lines at 6 degree intervals (see **Figure 26**) is 1,622 feet. Based on equations 3, 6, 8, 9, and 10 (NRCS, 2014), the significant wave height is 1.85 feet, resulting in a normal freeboard of 4.2 feet which is greater than the 3.00 foot minimum freeboard required.

Technical Release 210-60 requires Larimore Dam to pass the PMF without breaching the control section of the auxiliary spillway. The SITES auxiliary spillway analysis shows that the auxiliary spillway will completely breach during passage of the freeboard hydrograph. **Figure 22** shows the eroded portion of the spillway during the freeboard hydrograph corresponding to the

48-hour PMP event. The results of the Integrity Analysis for the 48-hour PMP event are provided in **Attachment 4**.

7.4 SUMMARY OF IDENTIFIED DEFICIENCIES

7.4.1 PRINCIPAL SPILLWAY

Larimore Dam's principal spillway does not meet criteria outlined in Technical Release 210-60. The SITES analysis indicates the auxiliary 0.6 feet too low when routing the principal spillway hydrographs.

Larimore Dam is approaching the end of its original design life of 50 years (2027), a duration prescribed by its design class. Since it was installed, it has been reclassified by the State of North Dakota and the NRCS to a higher design class, and thus rehabilitation of the structure will necessitate it extending its design life.

A corrosion analysis of the principal spillway conduit reinforcing steel was conducted by NRCS staff in November 2021. Connectionless Electrical Pulse Response Analysis (CERPA) technology was utilized to determine corrosion potential of steel rebar within 6 inches of the testing surface (inside of the conduit). Two separate locations were analyzed, with one showing a passive or low corrosion rate, and the other resulting in a moderate corrosion rate. Remaining service life estimates for a moderate corrosion rate range from 6 to 20 years, and thus the conduit would be a candidate for replacement to accommodate a 100-year extension of design life.

7.4.2 AUXILIARY SPILLWAY

The auxiliary spillway capacity is sufficient to pass the freeboard hydrograph for Larimore Dam. However, the auxiliary spillway integrity is not sufficient to pass the freeboard hydrograph without breaching. Similarly, the ND DWR Dam Safety Standards (NDDWR, 2024) for auxiliary spillway channel to withstand anticipated flow velocity up to the design event is not met with the current principal spillway operating. The ND DWR Dam Safety Standards further limit anticipated flow velocity check to not allow principal spillway flows, which would increase auxiliary spillway flows. Since the auxiliary spillway does not pass even with principal spillway capacity included, without principal spillway flows was not checked at this time, but will need to be in evaluation of vegetated alternatives in Appendix D-3.

7.4.3 EMBANKMENT

The drain filter compatibility analysis completed by HDR indicates the constructed filter is not compatible with the material it is meant to filter at the foundation of the dam, and does not account for dispersive embankment fill (see **Appendix D-2**). The drain fill at Larimore Dam was designed for a coarser-grained material than what was encountered in the dam foundation during the geotechnical exploration. Additionally, the lean clay soils encountered for use as the embankment fill were assumed not to pipe in the original design, however laboratory test results

show the embankment soils are potentially dispersive. Additional sample collection and testing will be necessary to confirm the dispersive properties of the embankment and foundation soils, and to confirm the physical properties and uniformity of the embankment fill and foundation soils to support the design of a drain fill that conforms to NRCS NEH Chapter 26 Gradation Design of Sand and Gravel Filters.

The NRCS TR-60 (NRCS, 2019) and ND DWR Dam Safety Standards (NDDWR, 2024) for minimum crest width are not met. The existing width is 20 feet. The minimum NRCS requirement is 20.2 feet $((66+35)/5)$ and NDDWR minimum is 23.2 feet (10 feet + 20% (66 feet)).

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9 FIGURES

- Figure 1. Downstream Channel Survey
- Figure 2. Larimore Dam Topographic Survey
- Figure 3. Larimore Dam Bathymetric Survey
- Figure 4. Larimore Dam Elevation-Flood Storage Relationship
- Figure 5. Larimore Dam Breach Outflow Hydrograph
- Figure 6. Larimore Dam Breach Inundation – All
- Figure 7. Larimore Dam Breach Inundation – 1
- Figure 8. Larimore Dam Breach Inundation – 2
- Figure 9. Larimore Dam Breach Inundation – 3
- Figure 10. Larimore Dam Breach Inundation – 4
- Figure 11. Depth-Velocity Flood Danger Level Relationship for Houses Built on Foundations
- Figure 12. Depth-Velocity Flood Danger Level Relationship for Passenger Vehicles
- Figure 13. Subbasin 24-hour curve numbers
- Figure 14. Subbasin time of concentration
- Figure 15. Subbasin storage coefficients
- Figure 16. Reservoir Storage – Outflow Comparison
- Figure 17. HEC-RAS Model Schematic
- Figure 18. Principal Spillway Mass Curves
- Figure 19. Principal Spillway Inflow Hydrographs
- Figure 20. Auxiliary Spillway Hydrographs and Routing
- Figure 21. Freeboard Hydrograph and Routing
- Figure 22. Larimore Dam Auxiliary Spillway Breach Profile

Figure 1. Downstream Channel Survey

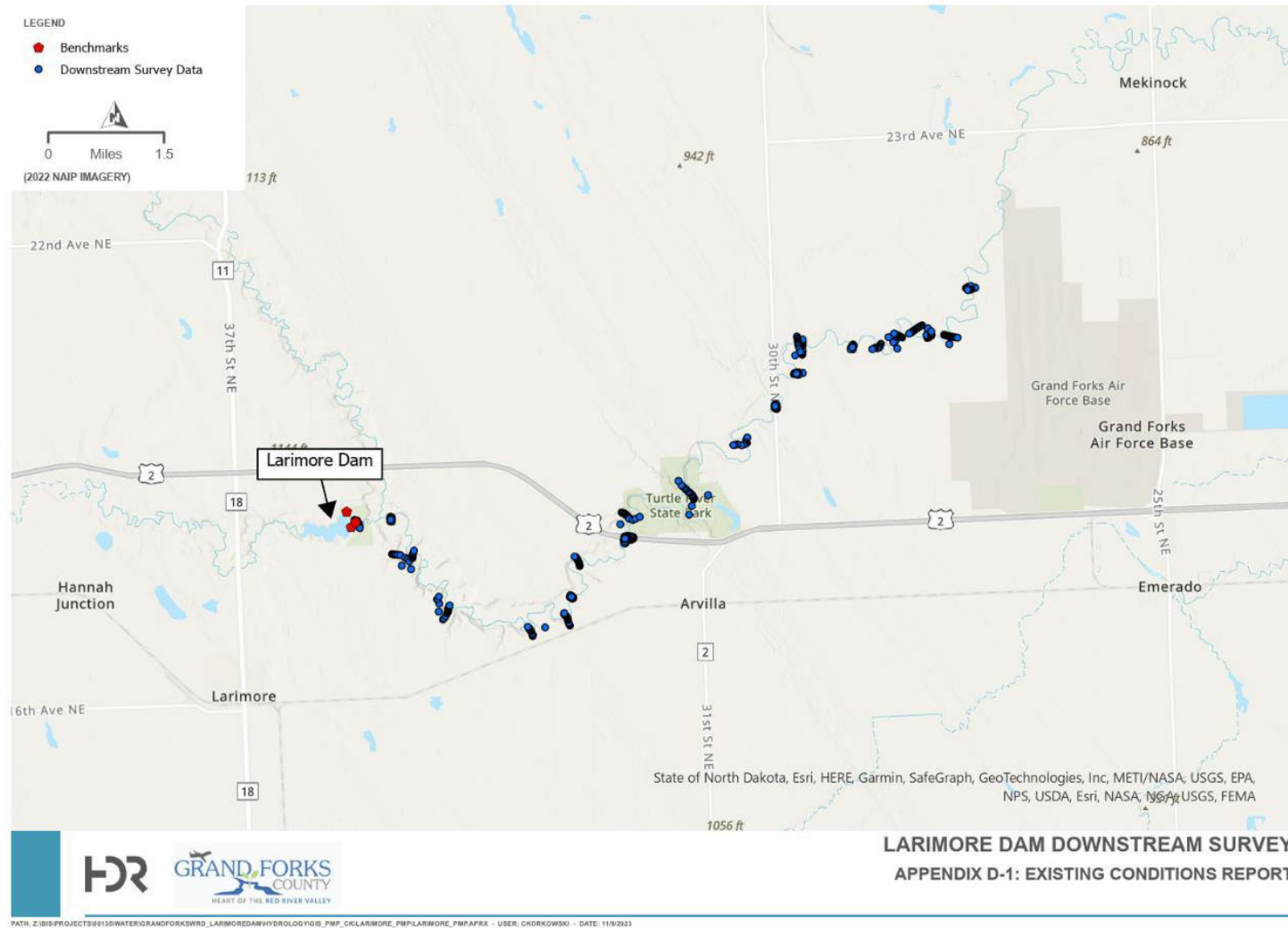


Figure 2. Larimore Dam Topographic Survey



Figure 3. Larimore Dam Bathymetric Survey

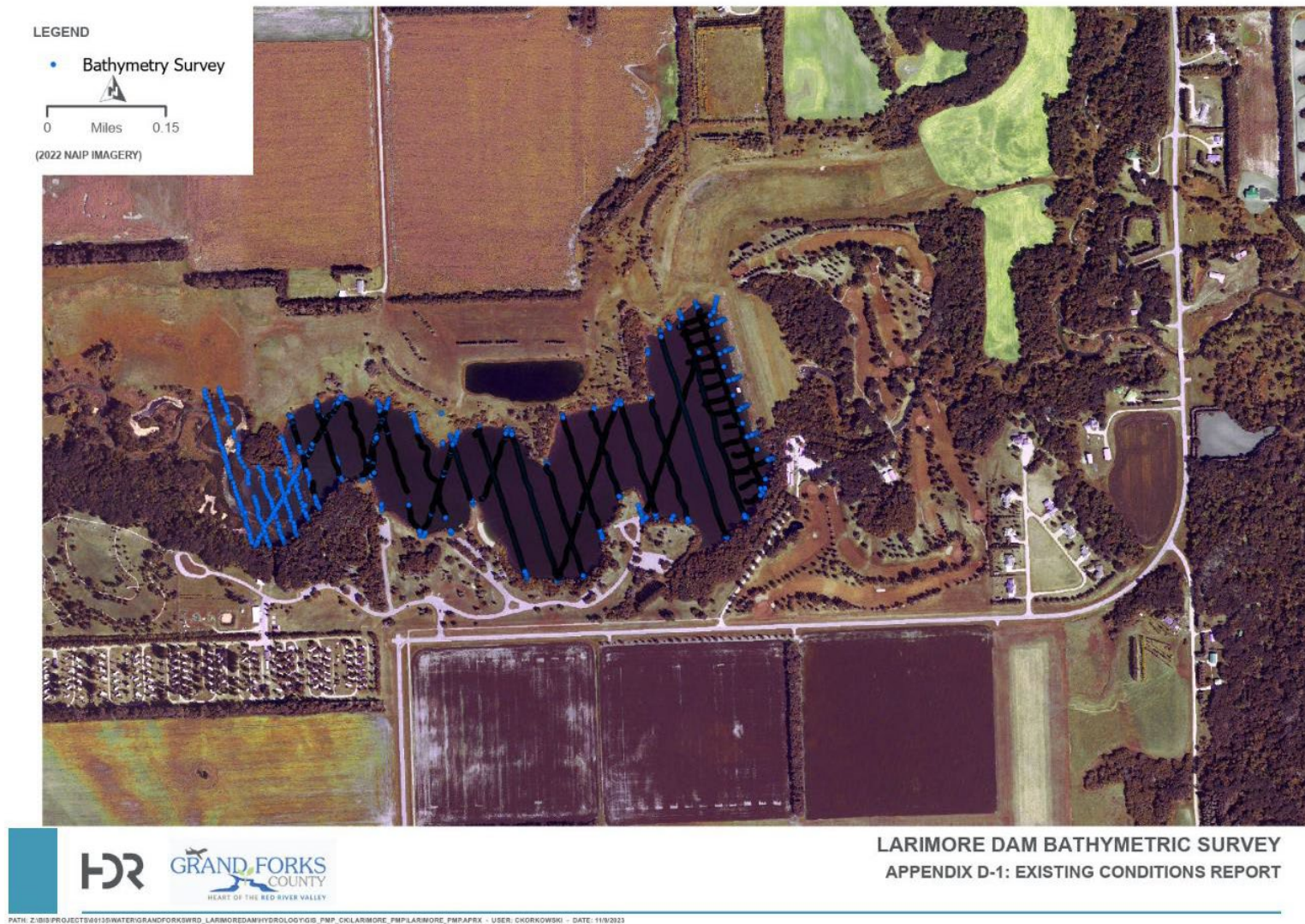


Figure 4. Larimore Dam Elevation-Flood Storage Relationship

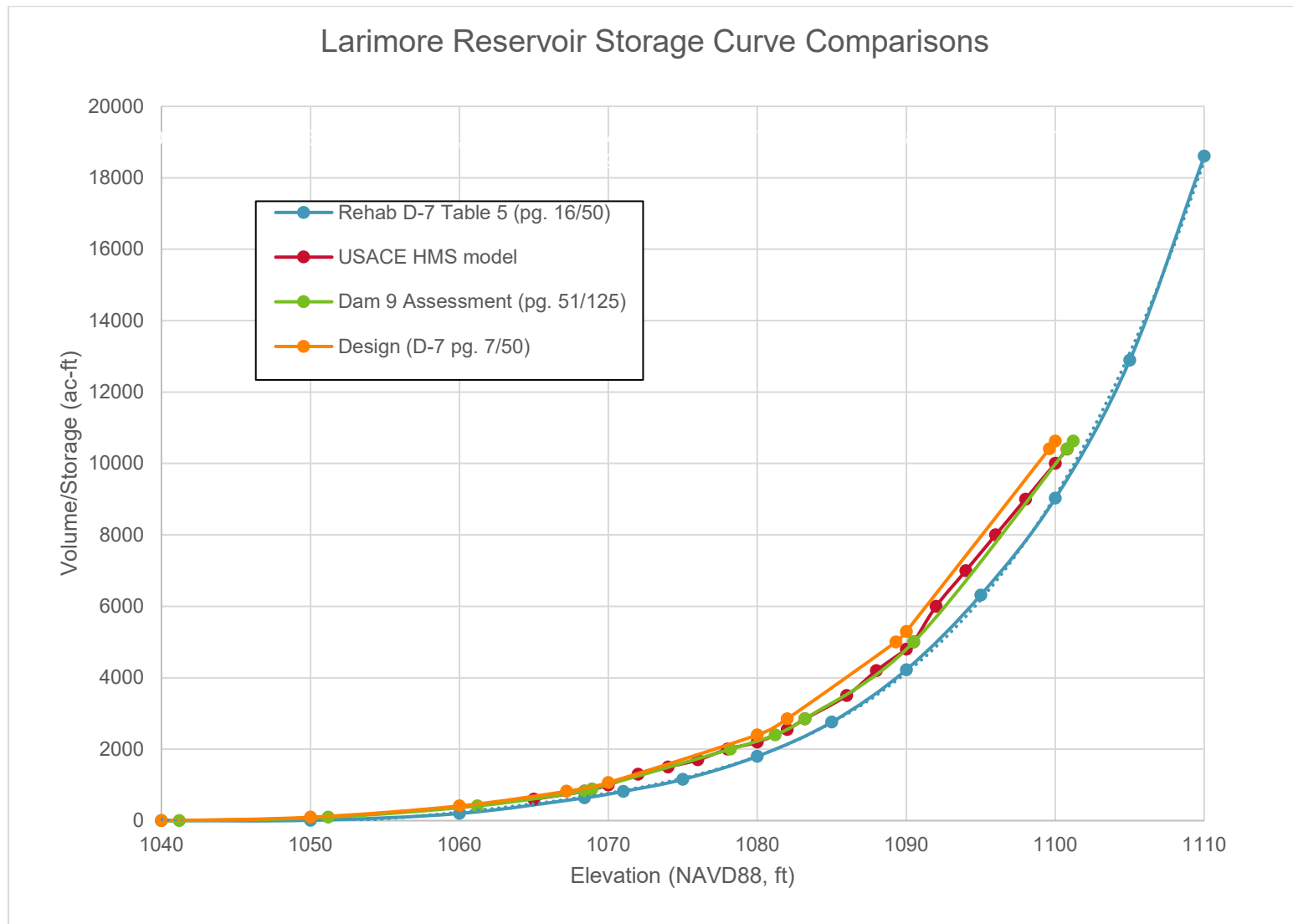


Figure 5. Larimore Dam Breach Outflow Hydrograph

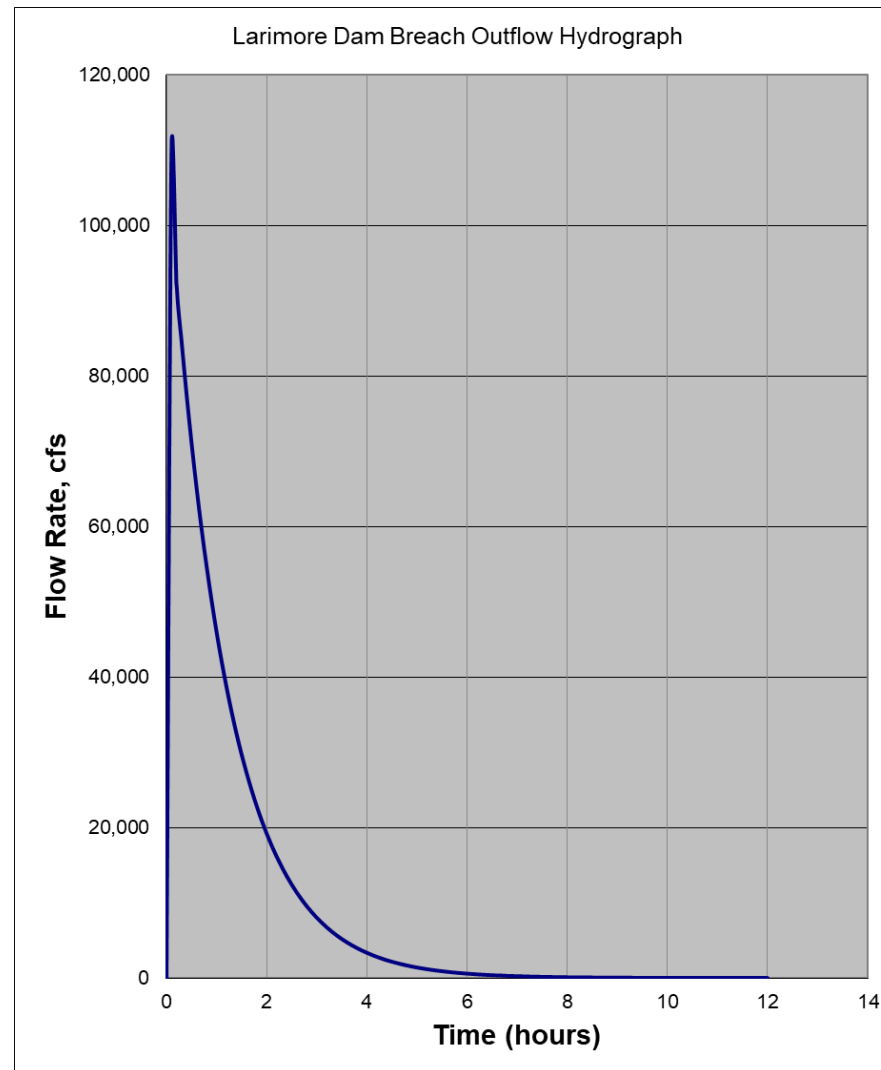


Figure 6. Larimore Dam Breach Inundation – All

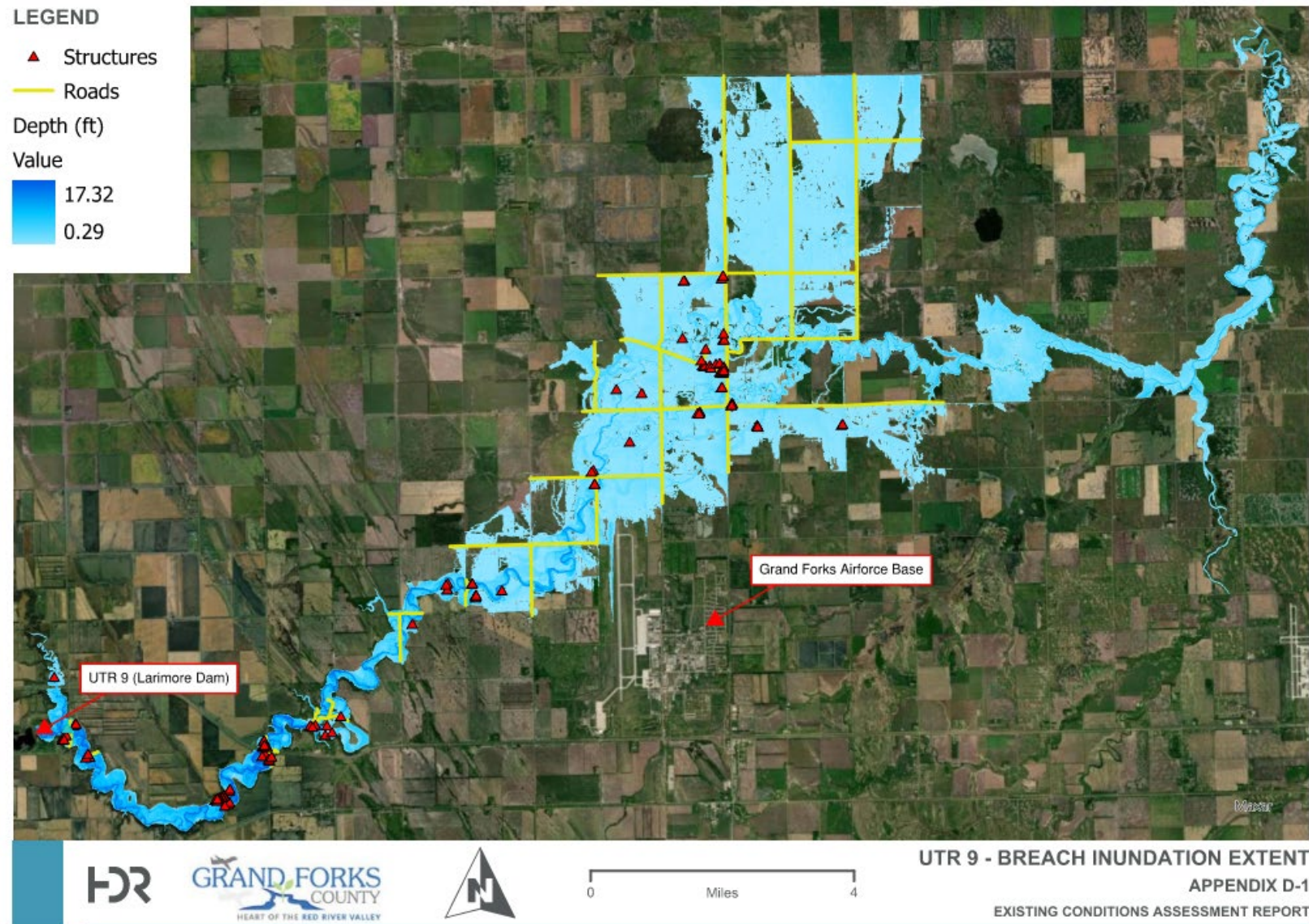


Figure 7. Larimore Dam Breach Inundation – 1

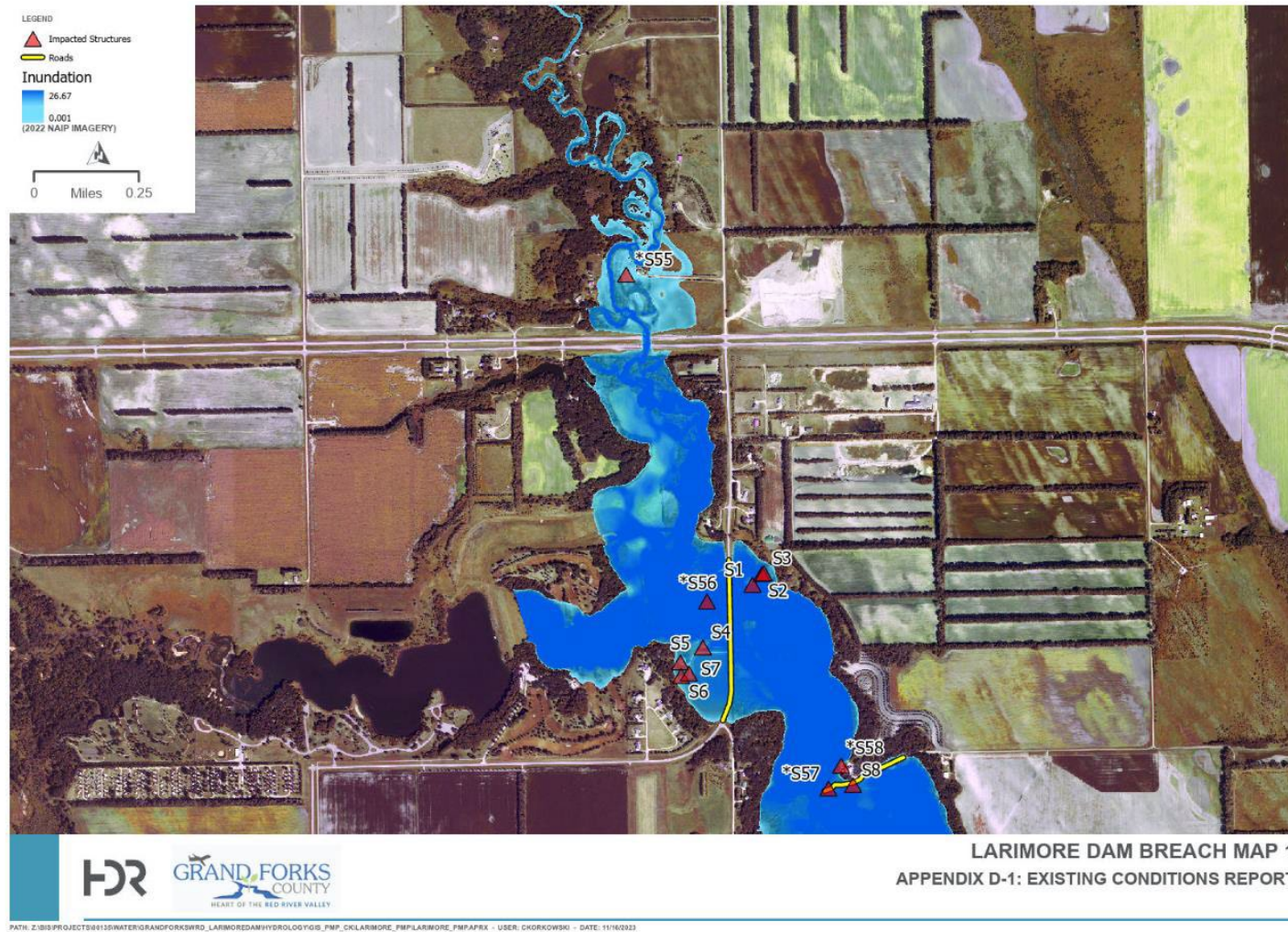


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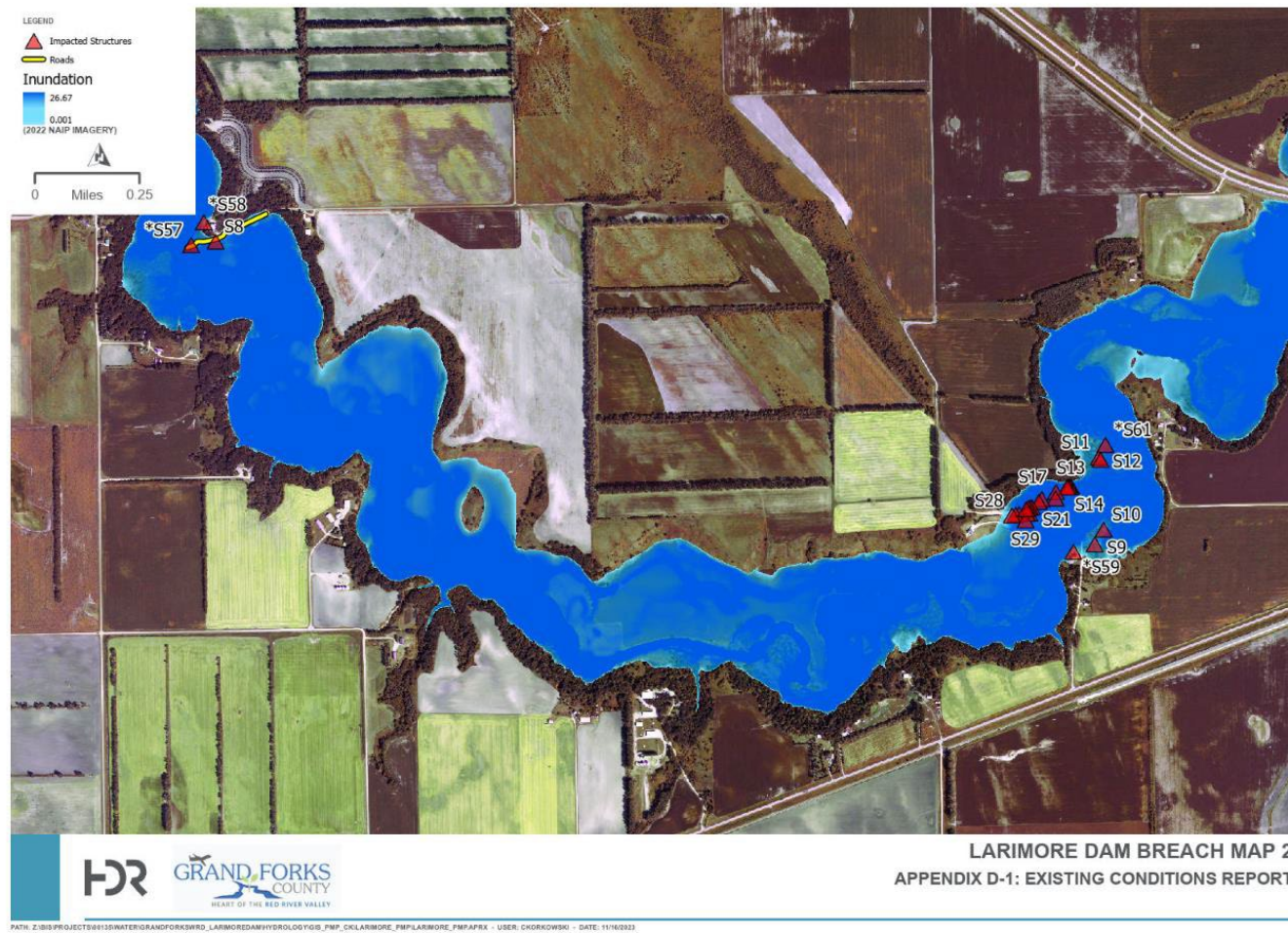


Figure 9. Larimore Dam Breach Inundation – 3

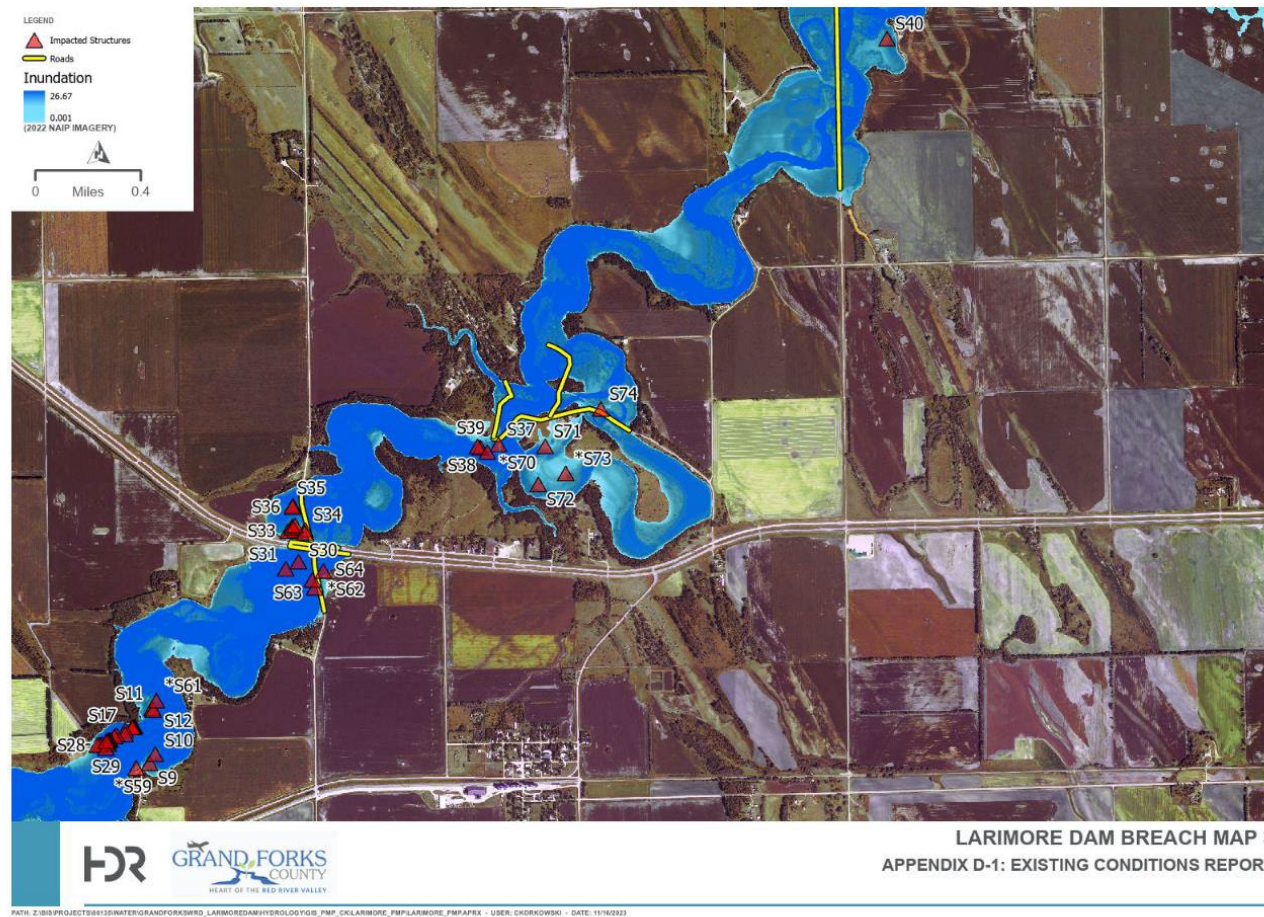


Figure 10. Larimore Dam Breach Inundation – 4

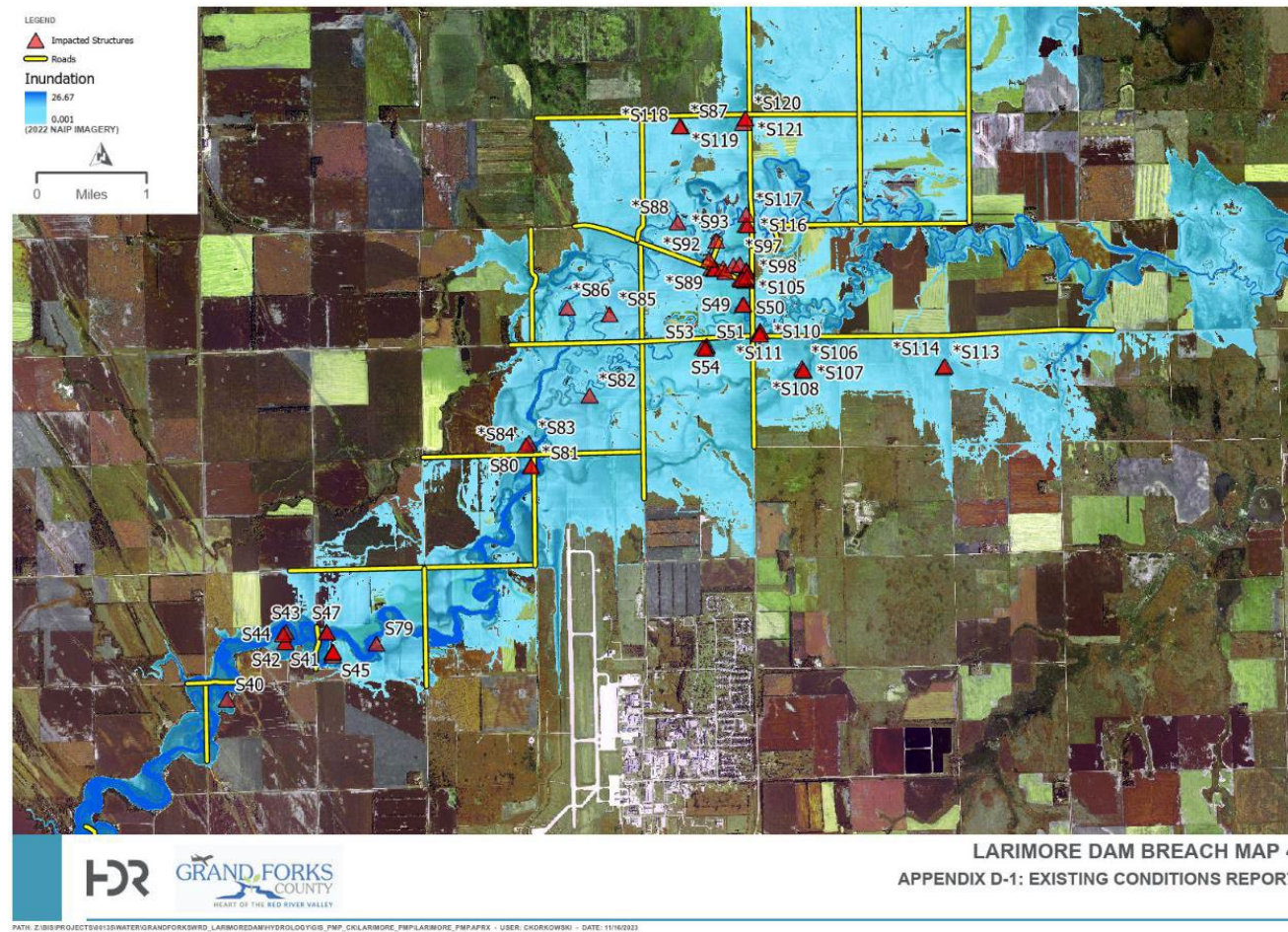


Figure 11. Depth-Velocity Flood Danger Level Relationship for Houses Built on Foundations

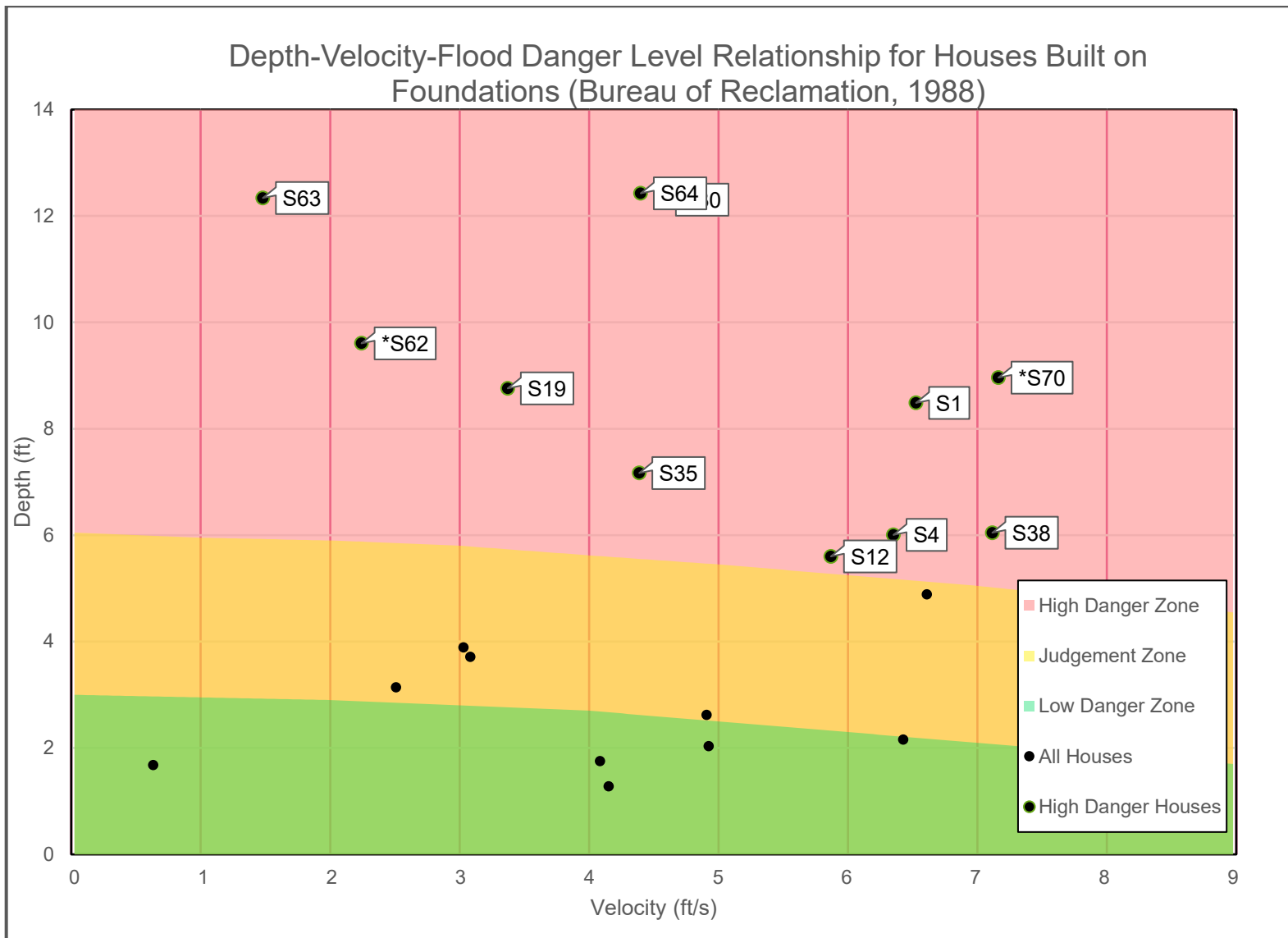


Figure 12. Depth-Velocity Flood Danger Level Relationship for Passenger Vehicles

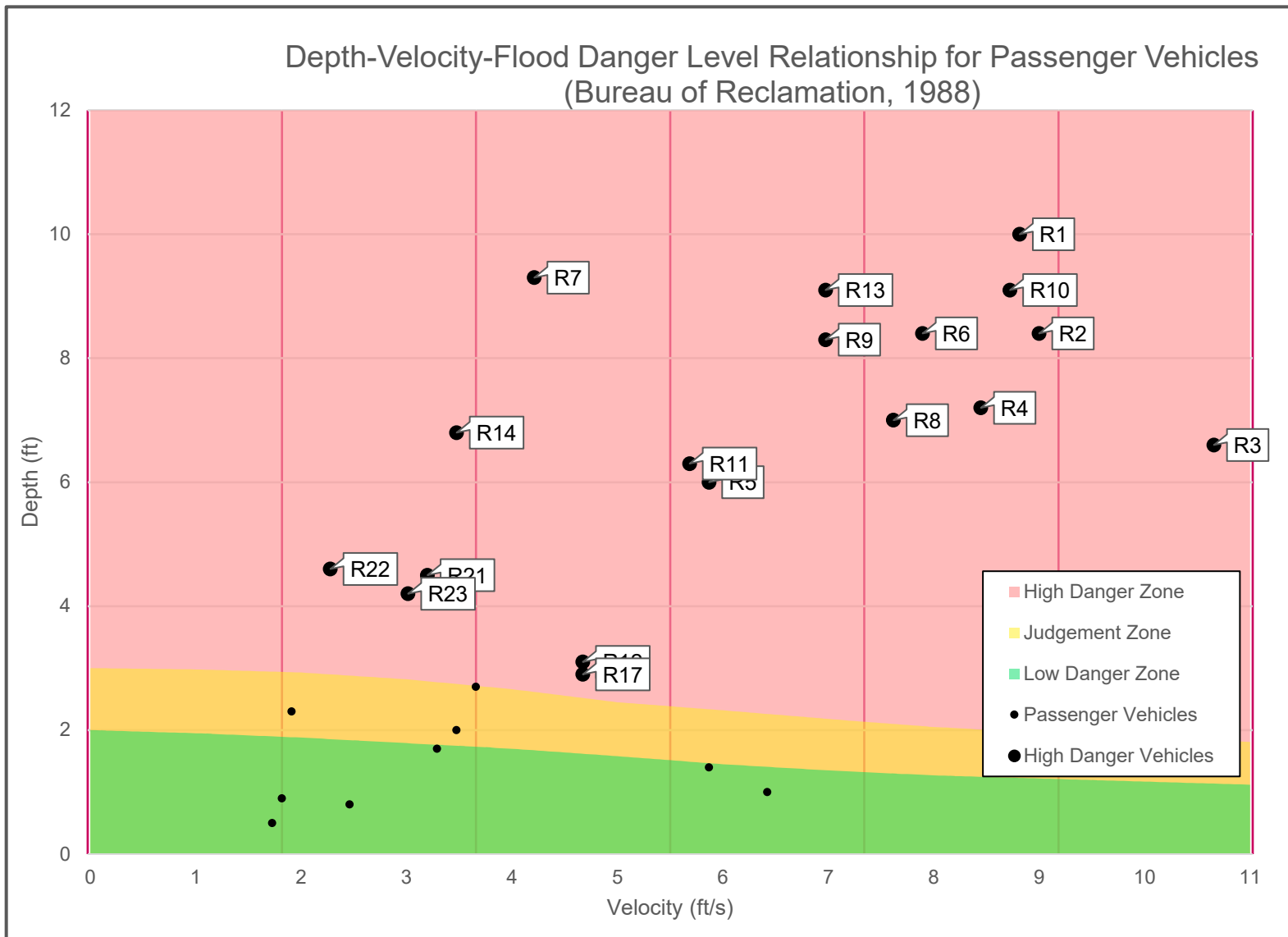


Figure 13. Subbasin 24-hour curve numbers

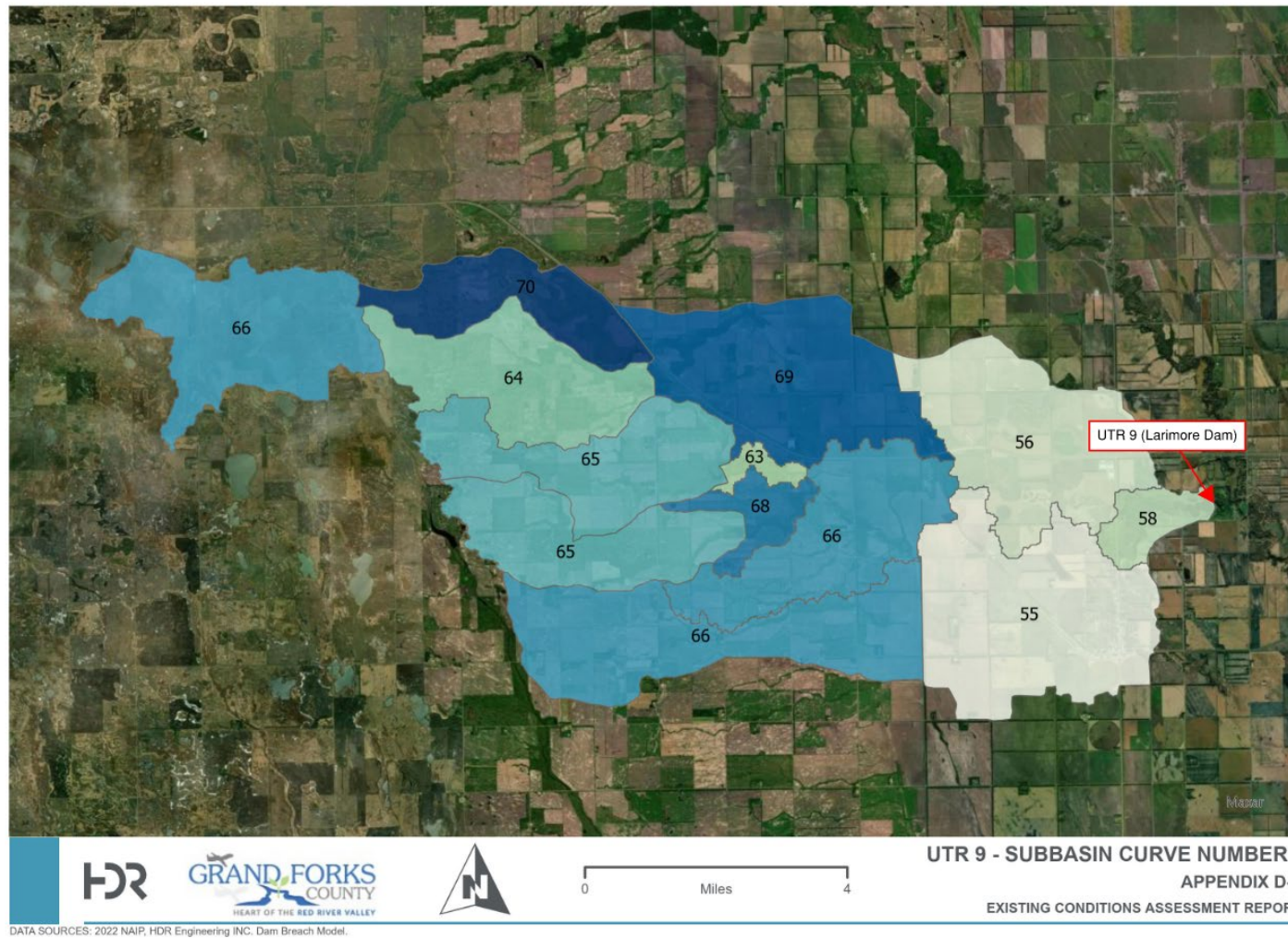


Figure 14. Subbasin time of concentration

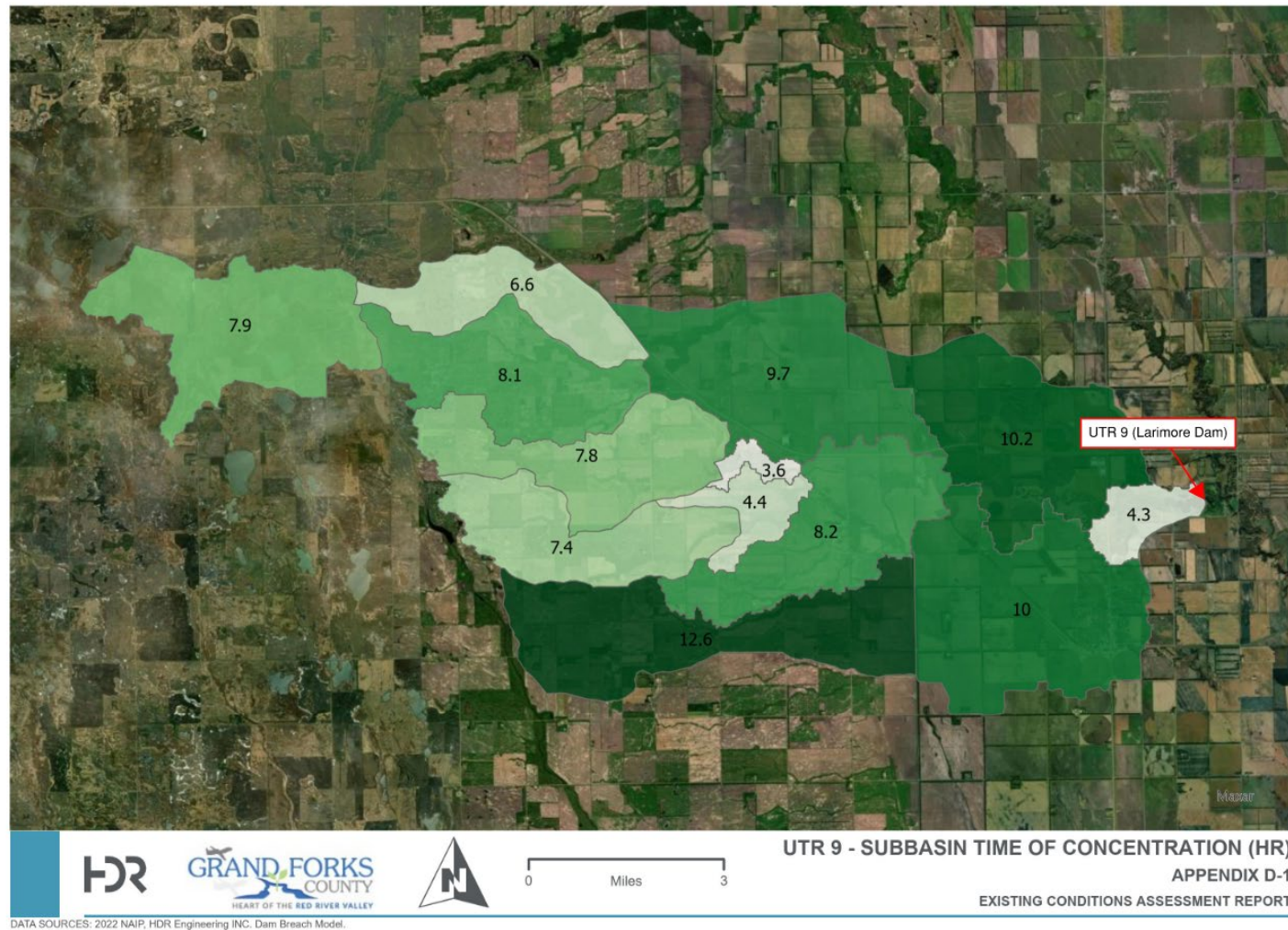


Figure 15. Subbasin storage coefficients

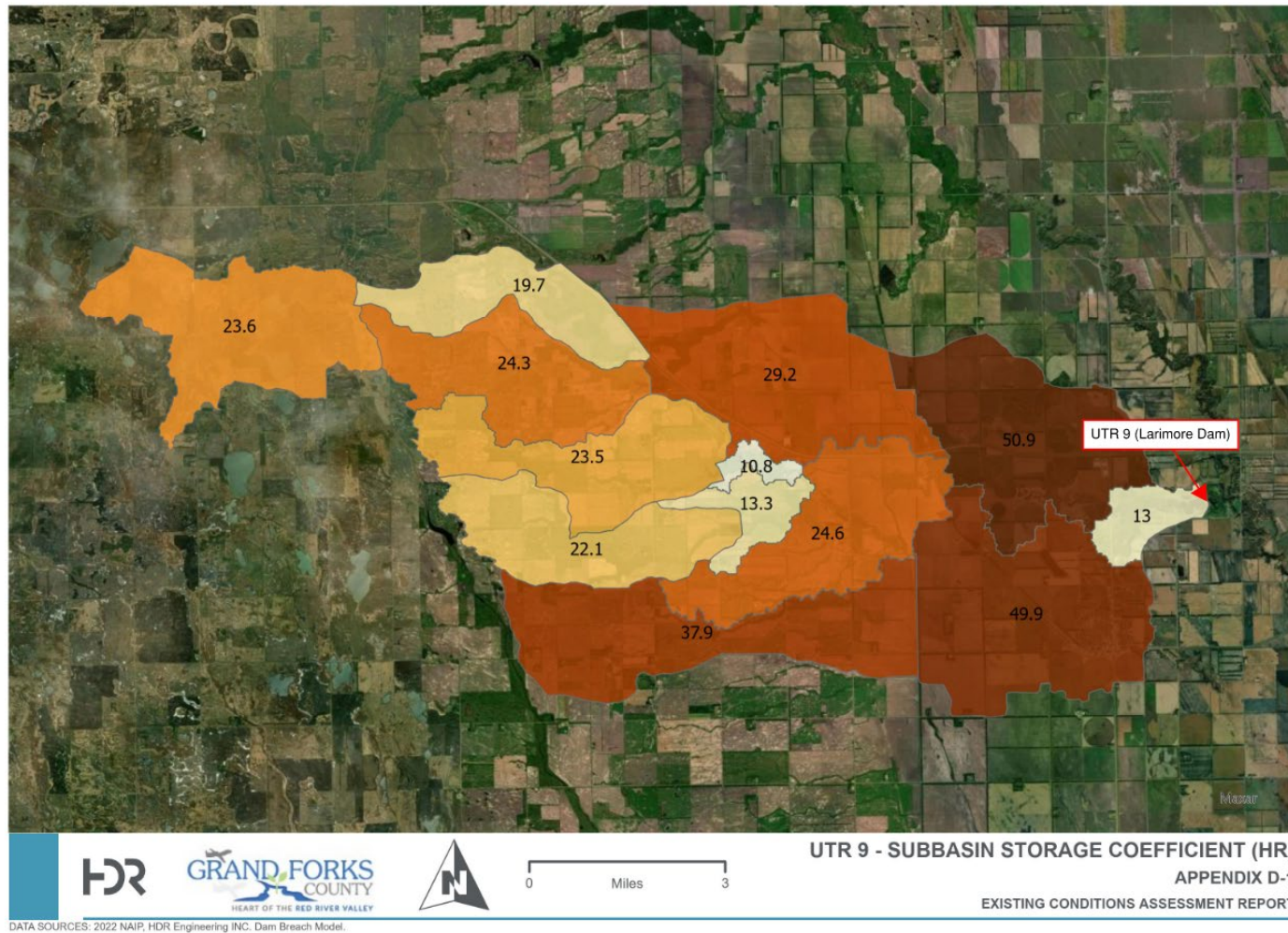


Figure 16. Reservoir Storage – Outflow Comparison

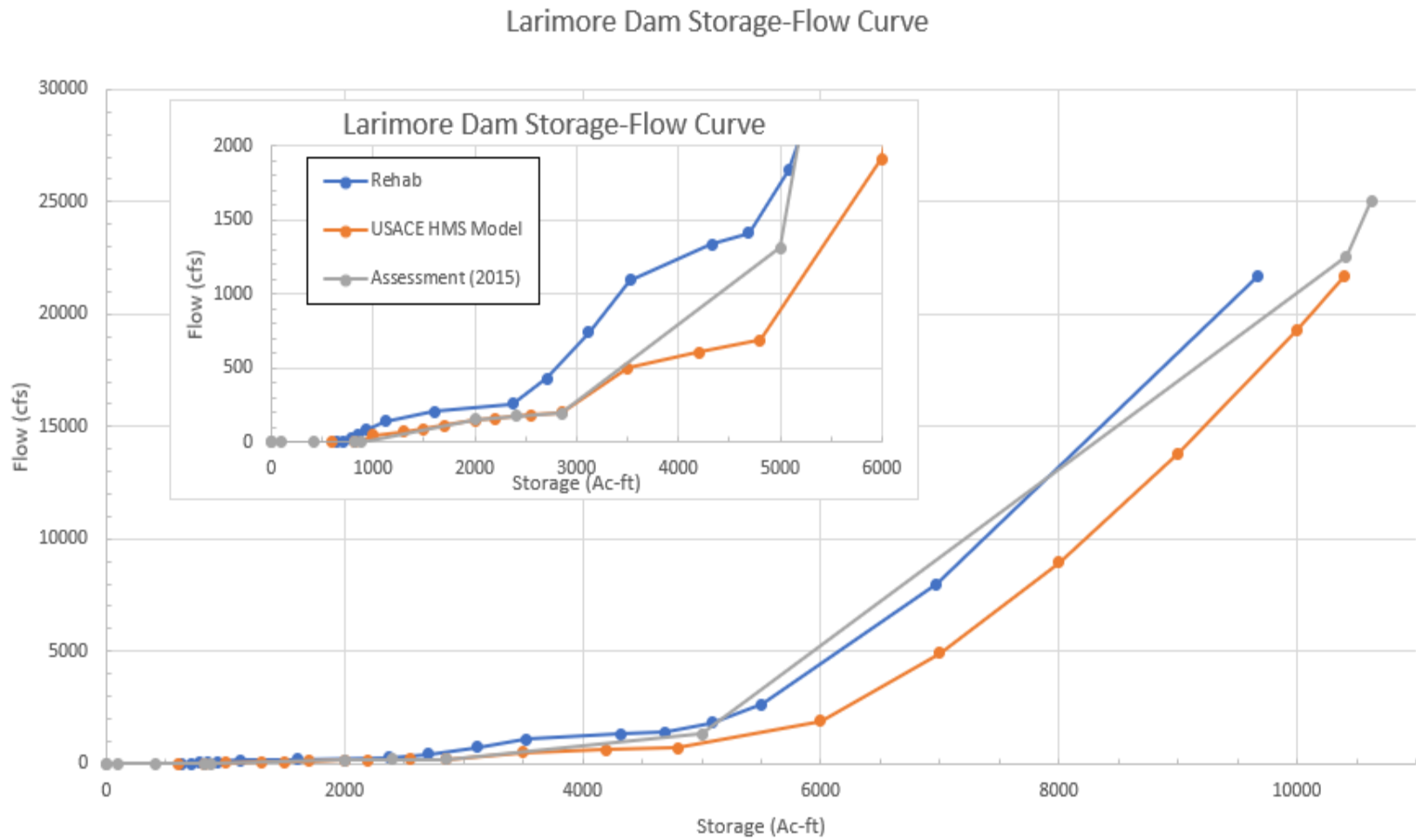


Figure 17. HEC-RAS Model Schematic

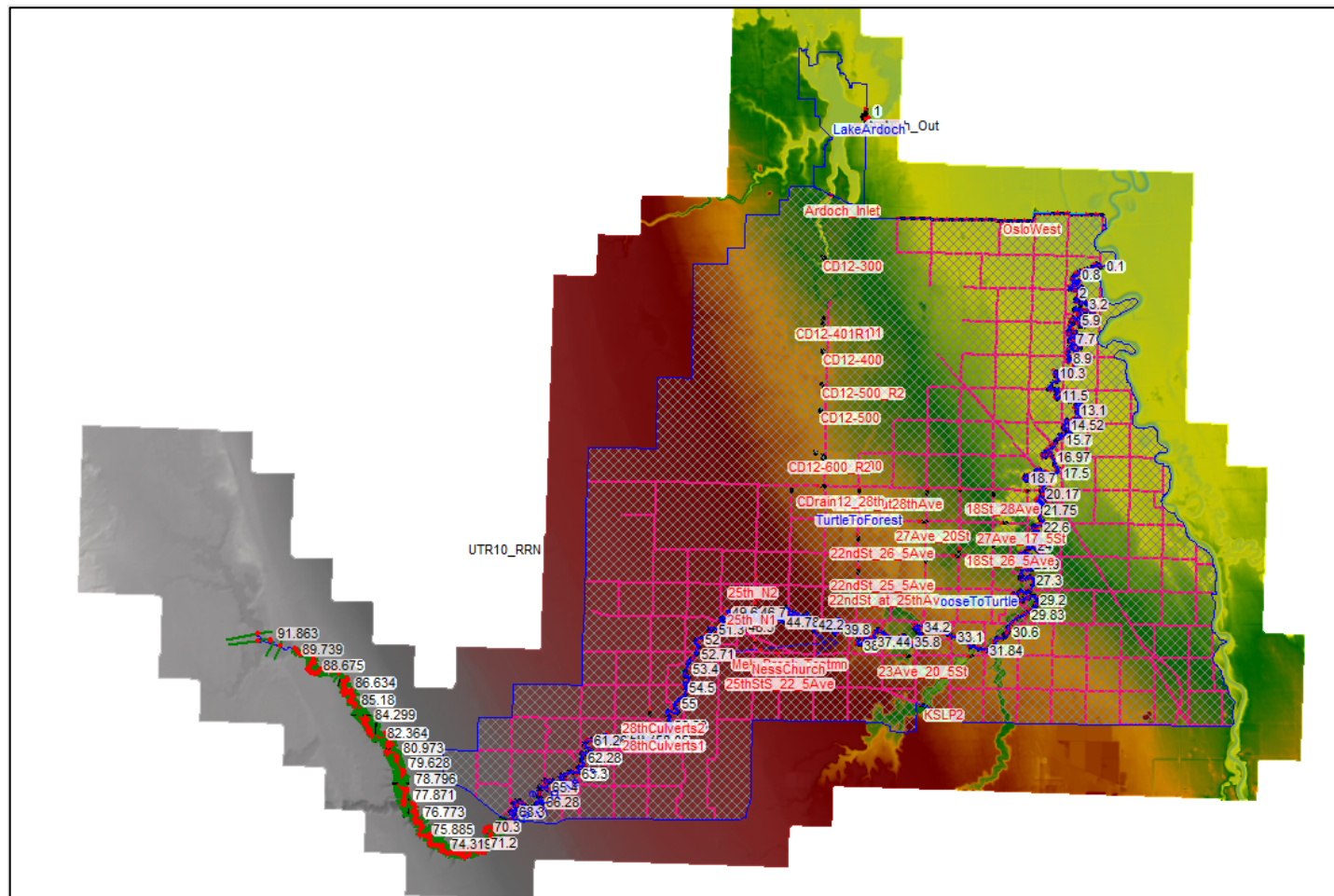


Figure 18. Principal Spillway Mass Curves

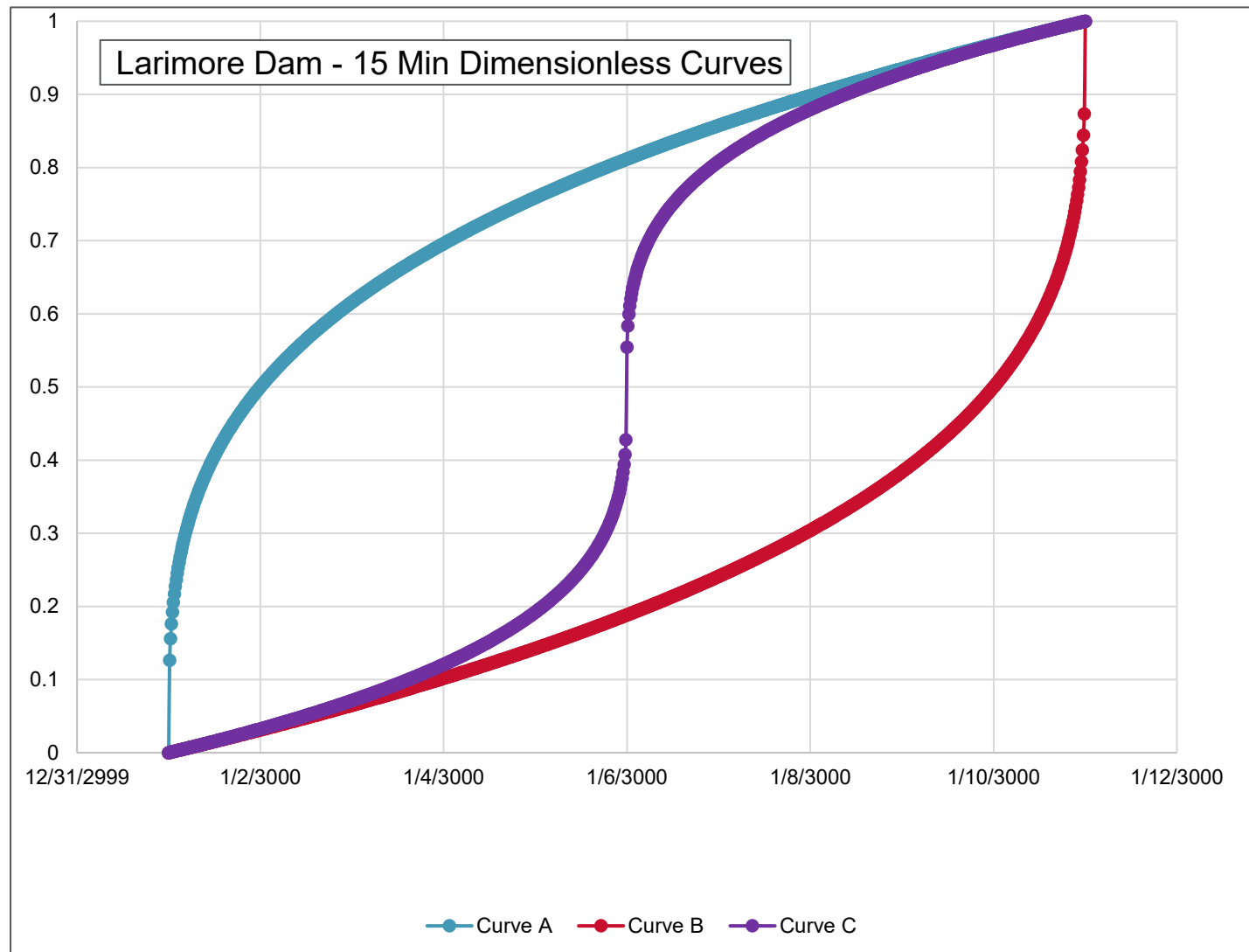


Figure 19. Principal Spillway Inflow Hydrographs

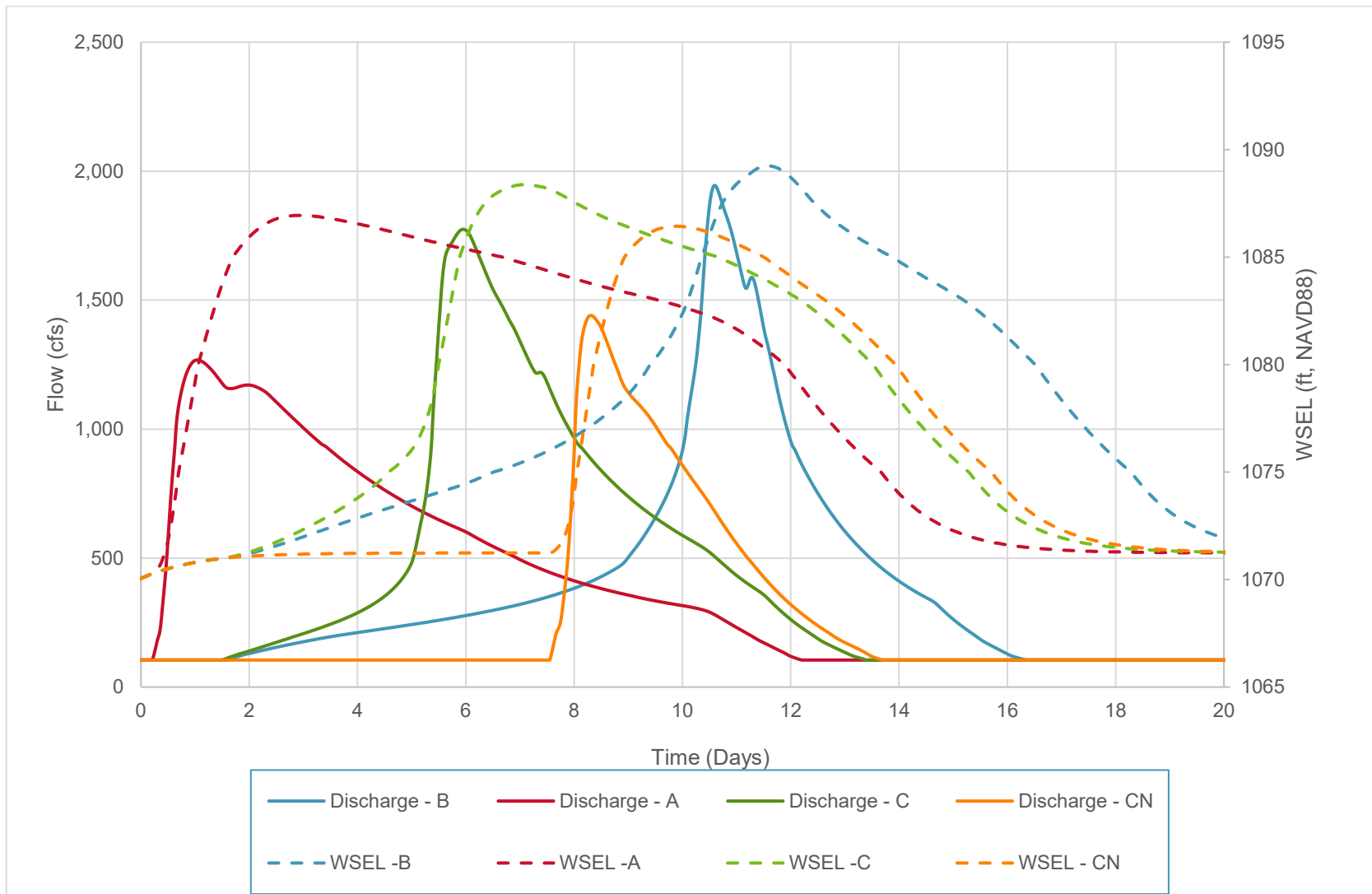


Figure 20. Auxiliary Spillway Hydrographs and Routing

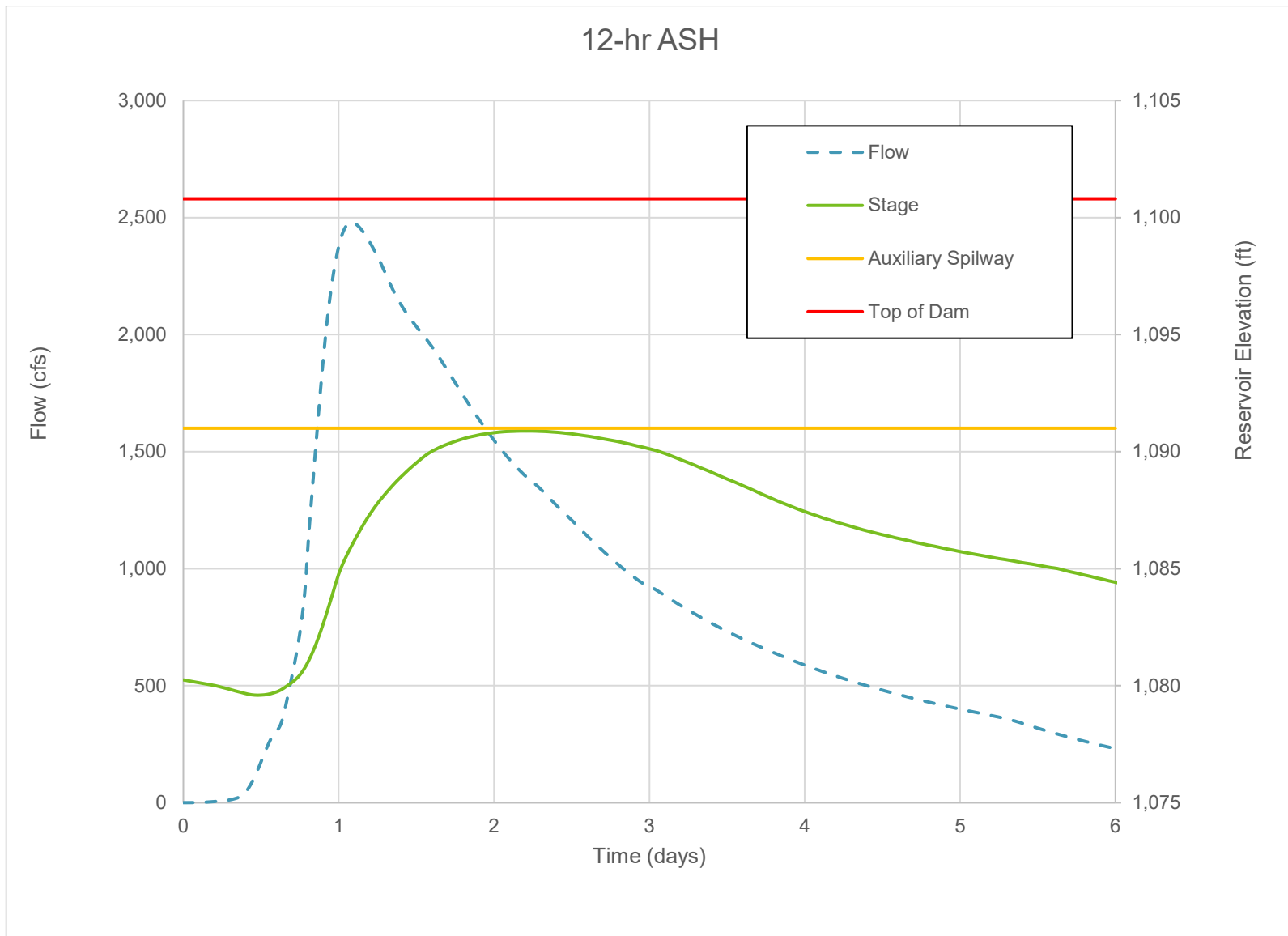


Figure 21. Freeboard Hydrograph and Routing

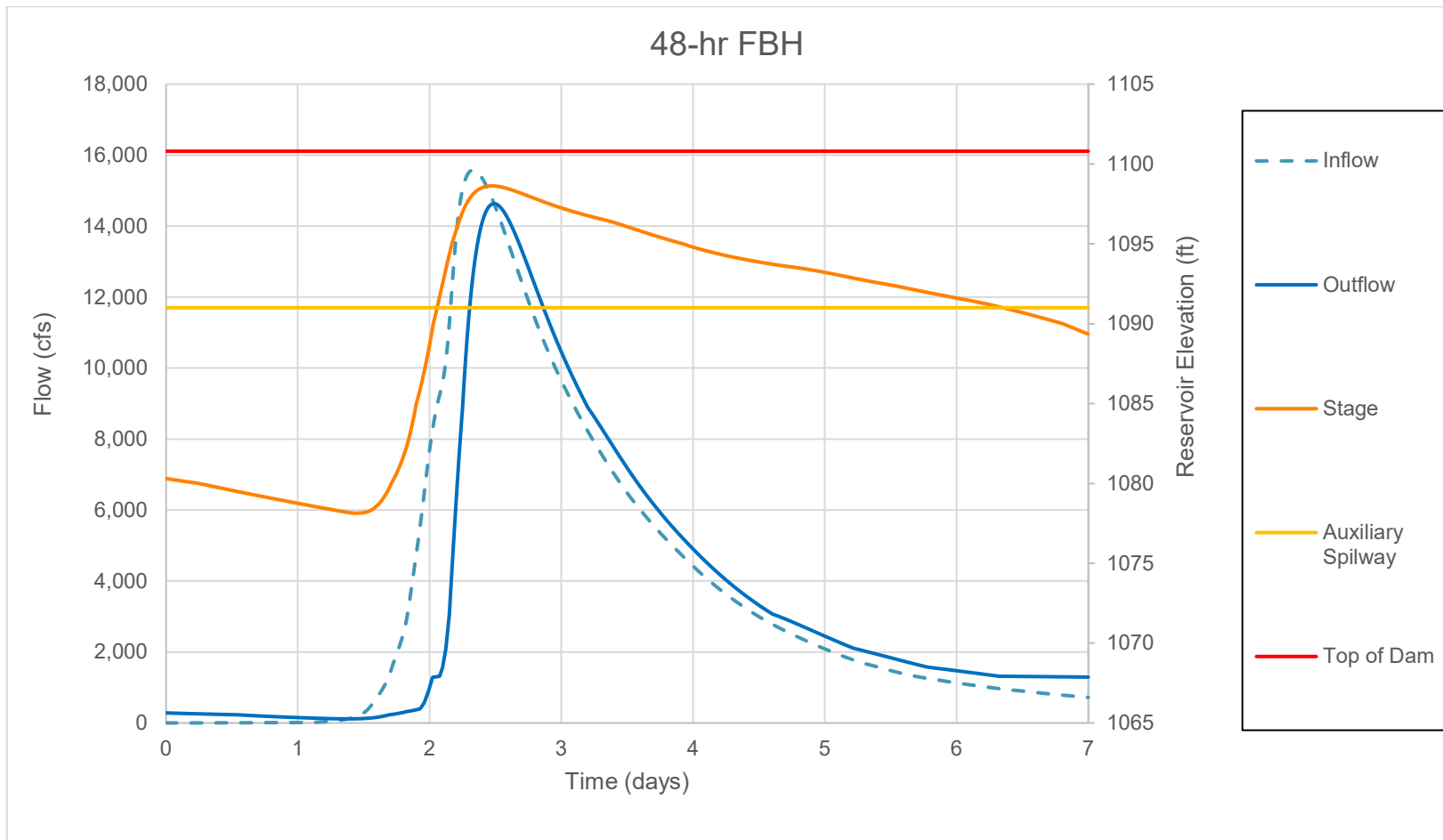


Figure 22. Larimore Dam Auxiliary Spillway Breach Profile

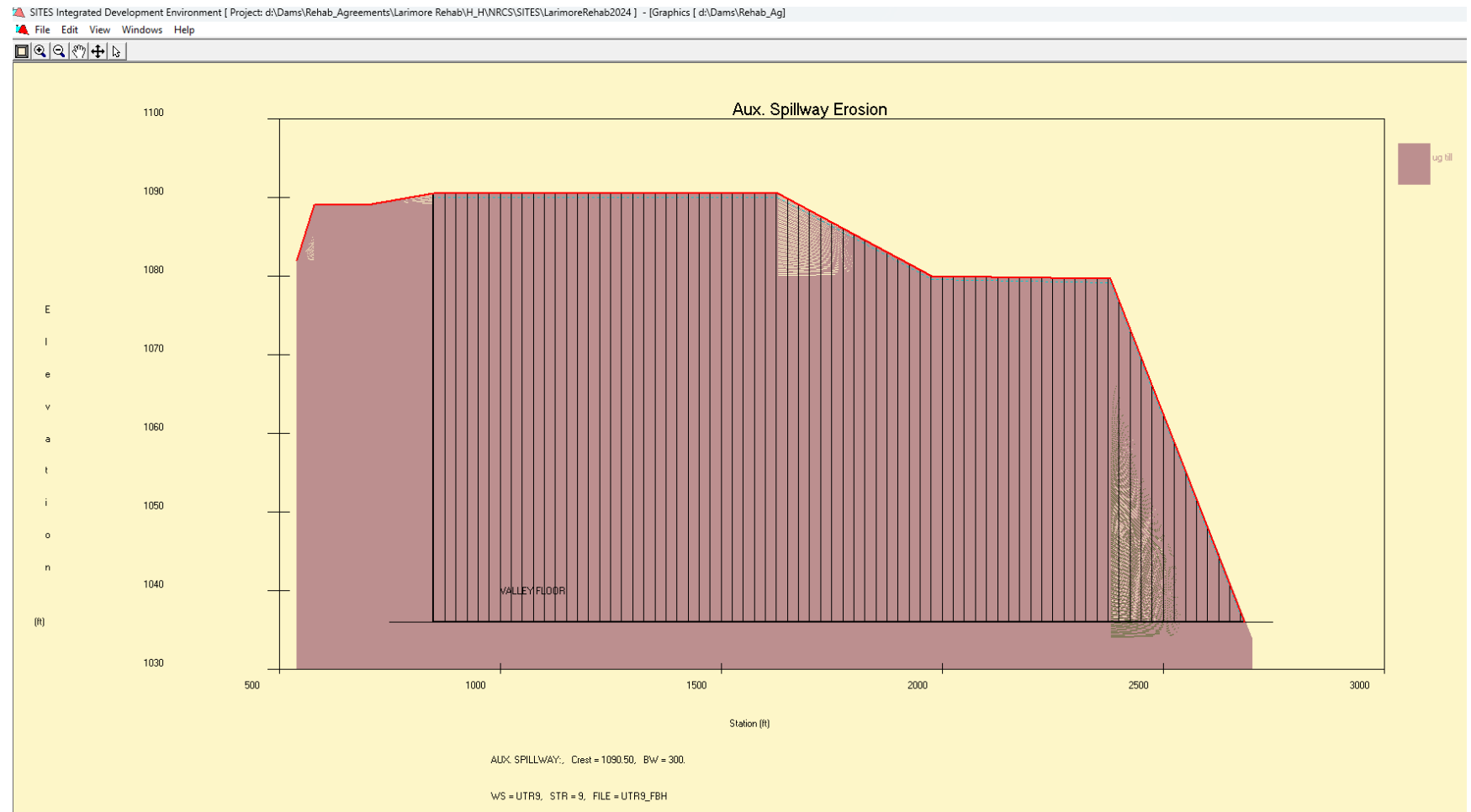


Figure 25. Wave Runup and Fetch Lengths –Residual Freeboard Assessment

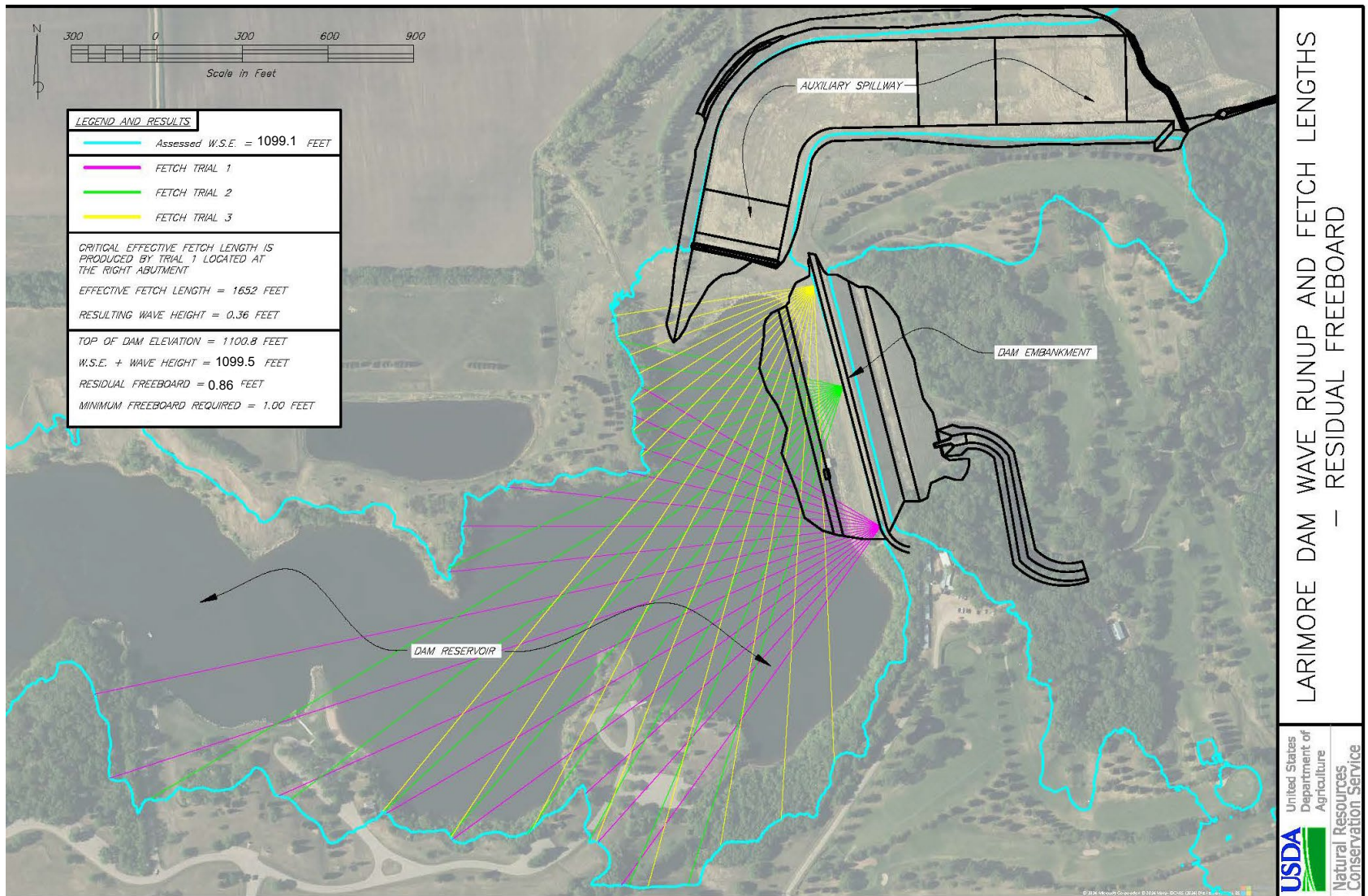
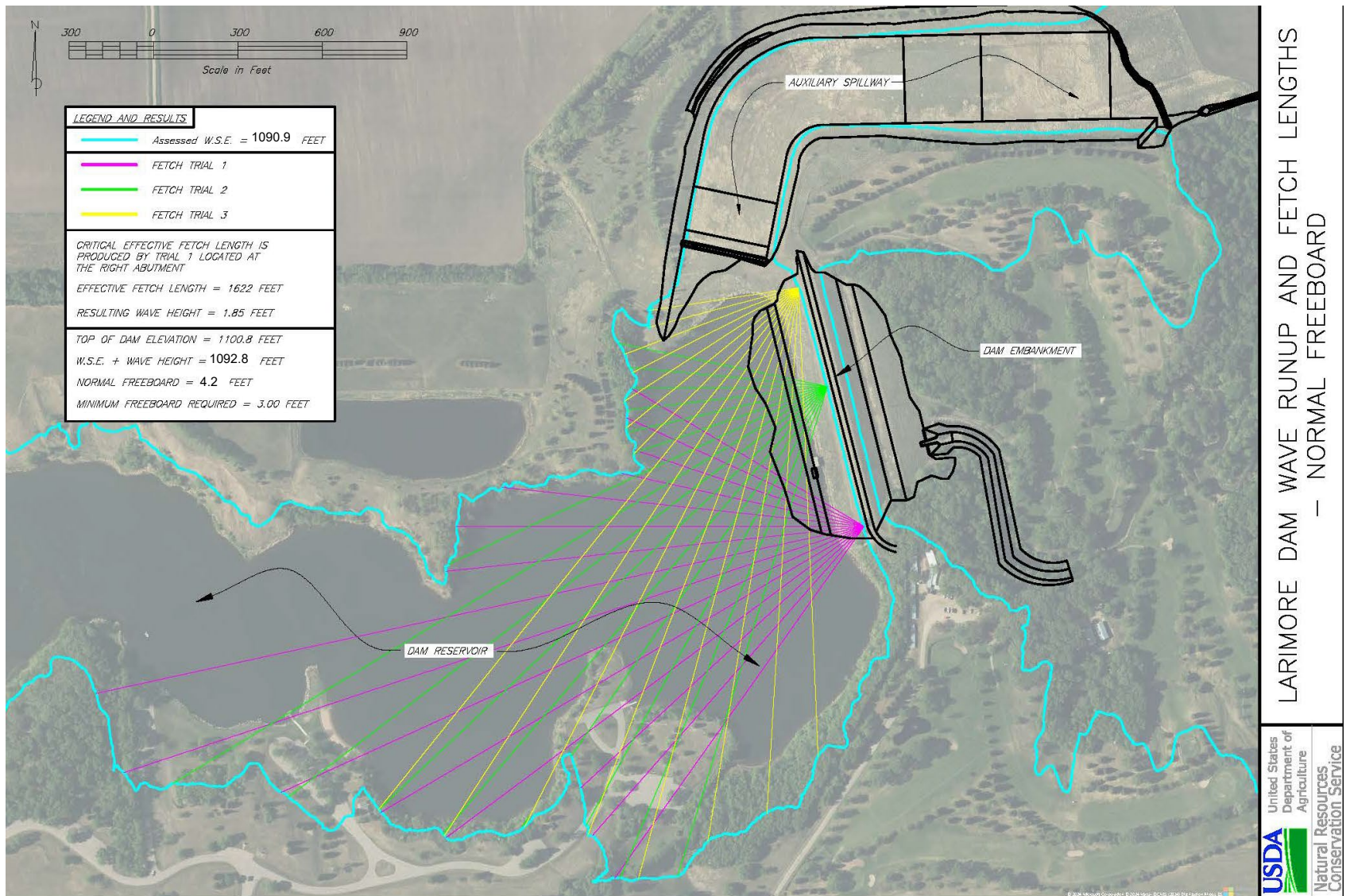
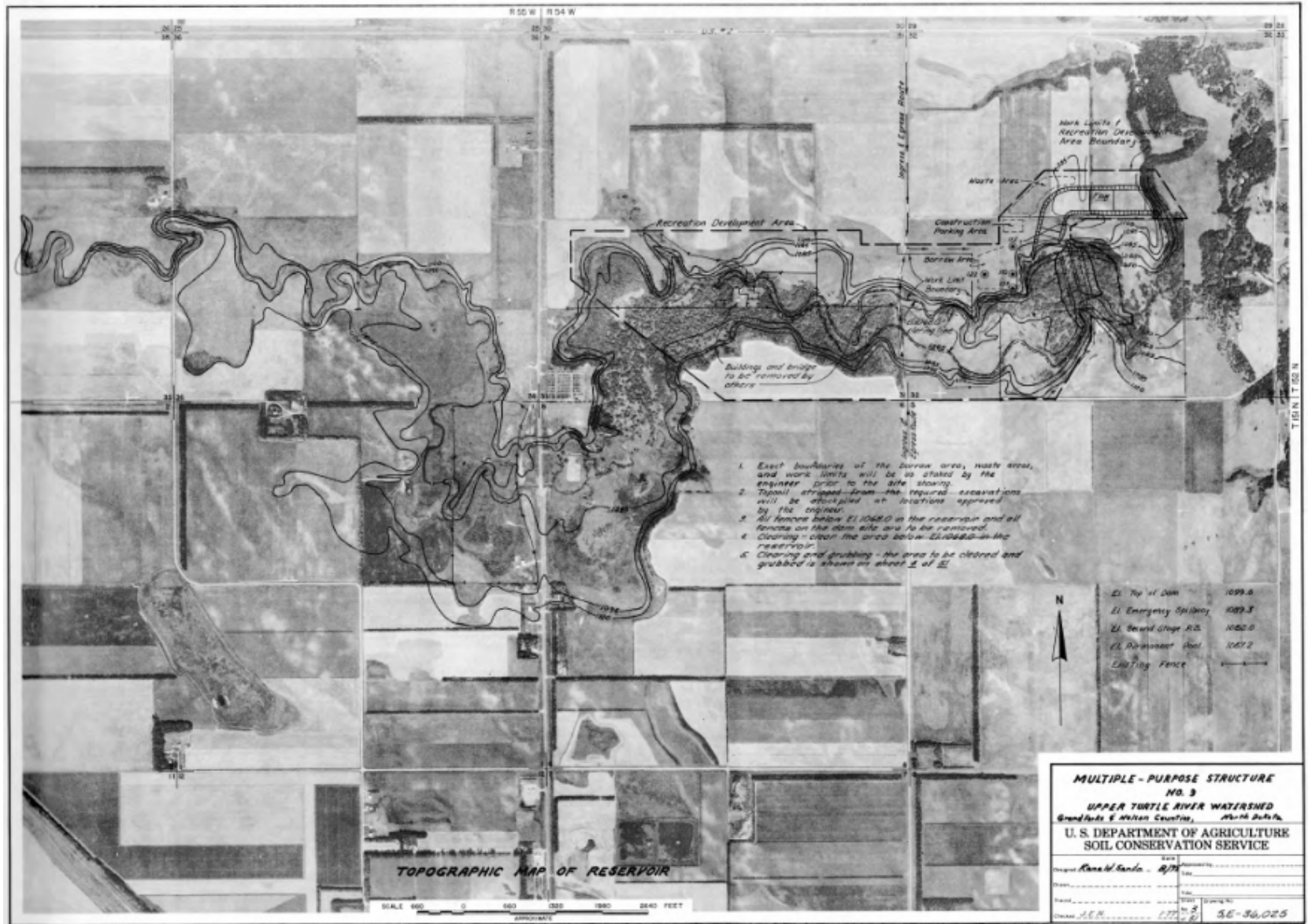


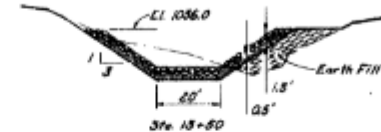
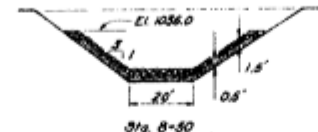
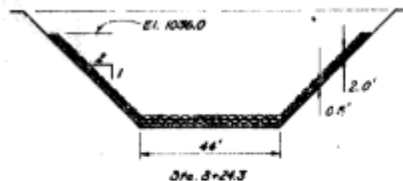
Figure 26. Wave Runup and Fetch Lengths –Normal Freeboard Assessment



10 ATTACHMENTS

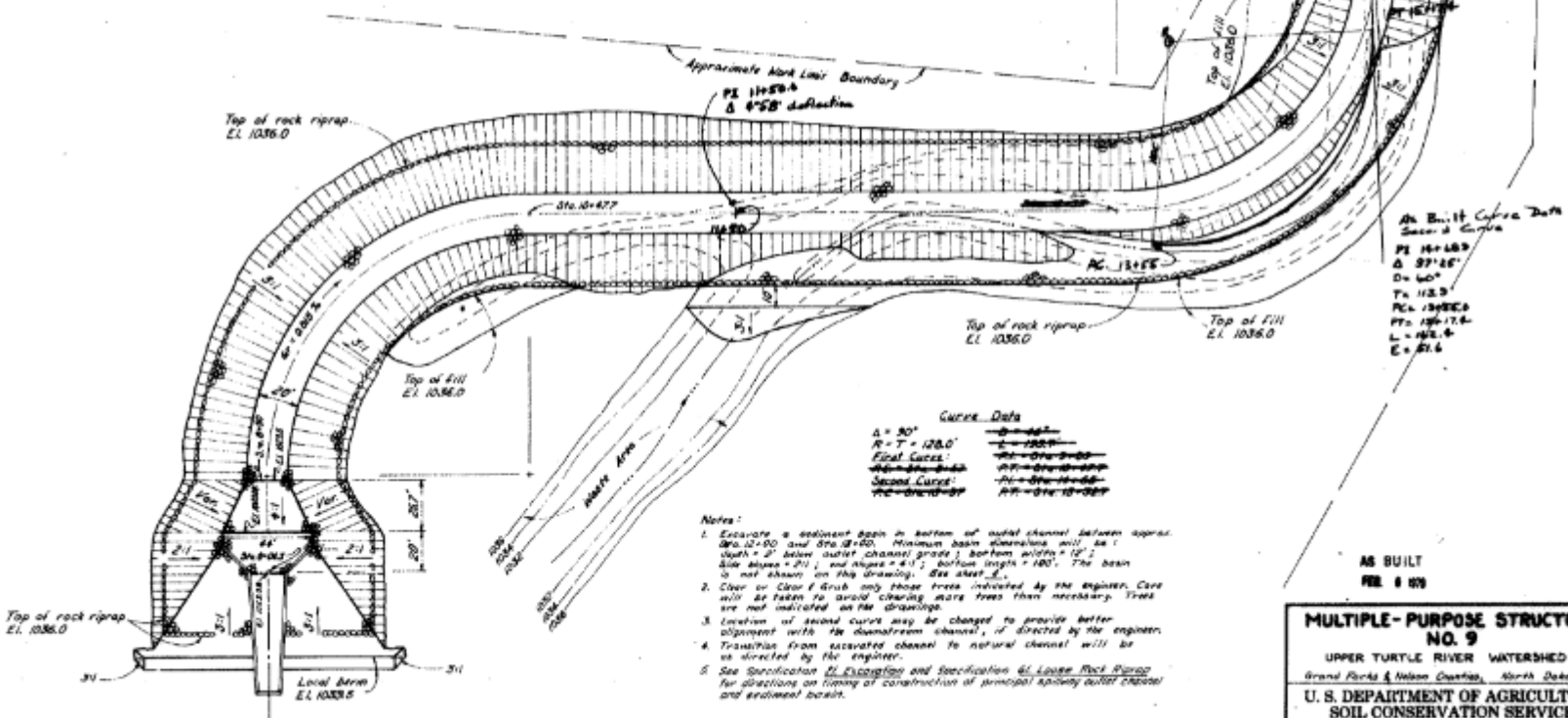
10.1 Attachment 1. As-Built Drawings



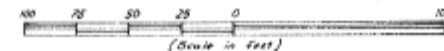


1. Thickness of rock riprap and bedding is measured vertically.
2. Upstream from outlet channel Sta. 8+50, rock riprap will be 2.0' thick plus 0.5' bedding.
3. Between Sta. 8+50 and Sta. 13+50, rock riprap will be 1.5' thick plus 0.5' bedding.

TYPICAL SECTIONS OF PRINCIPAL SPILLWAY OUTLET CHANNEL
(not to scale)



PLAN VIEW OF PRINCIPAL SPILLWAY OUTLET CHANNEL



- Notes:
1. Excavate a settlement basin in bottom of outlet channel between approx. Sta. 12+00 and Sta. 12+50. Minimum basin dimensions will be: Length = 2' below outlet channel grade; bottom width = 12'; Side slopes = 2:1; and depth = 4'; bottom length = 100'. The basin is not shown on this drawing. See sheet 5.
 2. Clear or Cut & Fill only those trees indicated by the engineer. Care will be taken to avoid clearing more trees than necessary. Trees are not indicated on the drawings.
 3. Location of second curve may be changed to provide better alignment with the downstream channel, if directed by the engineer.
 4. Transition from excavated channel to natural channel will be as directed by the engineer.
 5. See Specification for Excavation and Specification for Loose Rock Riprap for directions on timing of construction of principal spillway outlet channel and settlement basin.

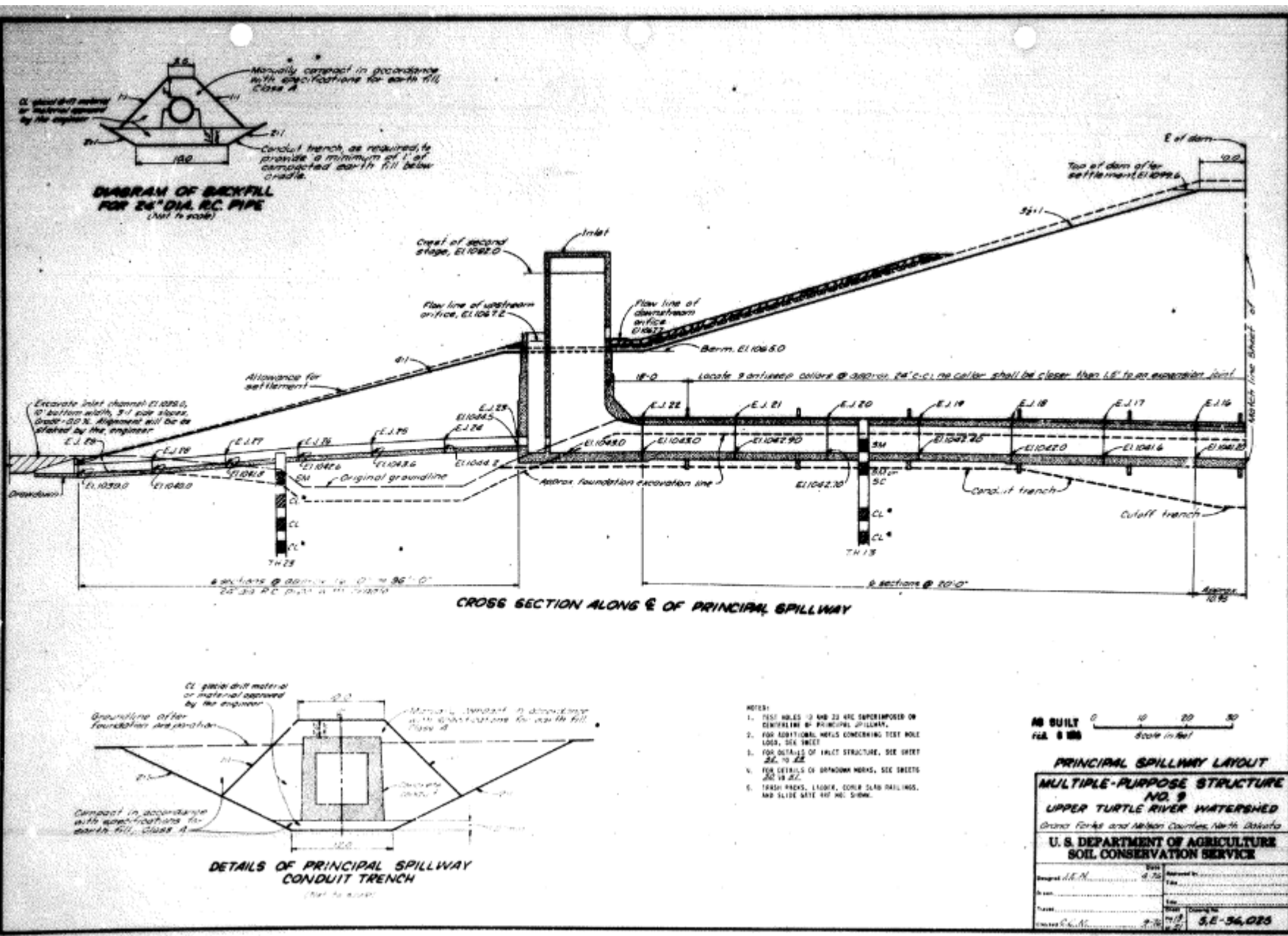
Curve Data

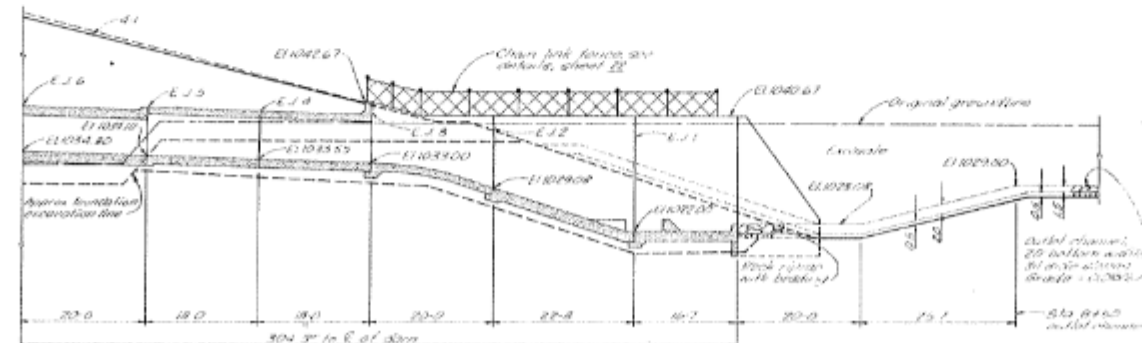
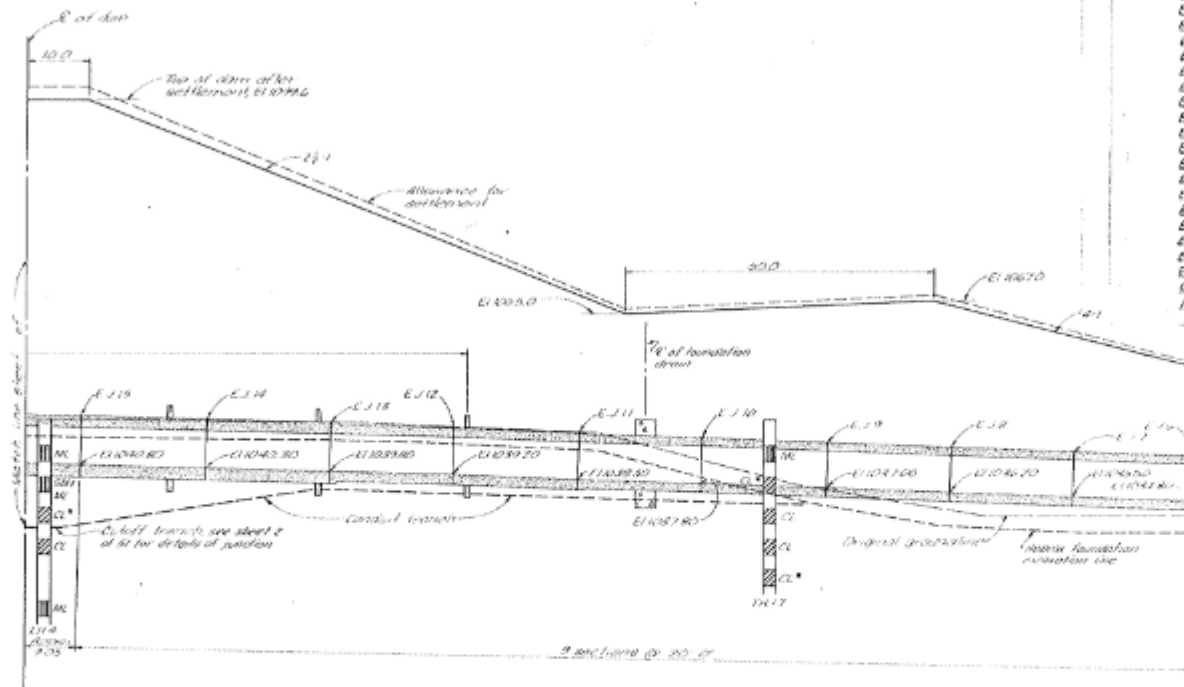
$\Delta = 90^\circ$	$R = 120.0'$
$R = 120.0'$	$L = 120.0'$
First Curve:	$PI = 11+50.0$
Second Curve:	$PI = 13+50.0$

AS BUILT
FILE # 100

**MULTIPLE-PURPOSE STRUCTURE
NO. 9**
UPPER TURTLE RIVER WATERSHED
Grand Forks & Nelson Counties, North Dakota
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed by	JEN	Approved by	JEN
Drawn by	JEN	Checked by	JEN
Project	5E-36,025	Sheet	1 of 2





CROSS SECTION ALONG C OF PRINCIPAL SPILLWAY

Station	Left	Right	Center	Right	Station
E.1040.67	100.00	100.00	100.00	100.00	100.00
E.1041.00	100.00	100.00	100.00	100.00	100.00
E.1041.30	100.00	100.00	100.00	100.00	100.00
E.1041.60	100.00	100.00	100.00	100.00	100.00
E.1041.90	100.00	100.00	100.00	100.00	100.00
E.1042.20	100.00	100.00	100.00	100.00	100.00
E.1042.50	100.00	100.00	100.00	100.00	100.00
E.1042.80	100.00	100.00	100.00	100.00	100.00
E.1043.10	100.00	100.00	100.00	100.00	100.00
E.1043.40	100.00	100.00	100.00	100.00	100.00
E.1043.70	100.00	100.00	100.00	100.00	100.00
E.1044.00	100.00	100.00	100.00	100.00	100.00
E.1044.30	100.00	100.00	100.00	100.00	100.00
E.1044.60	100.00	100.00	100.00	100.00	100.00
E.1044.90	100.00	100.00	100.00	100.00	100.00
E.1045.20	100.00	100.00	100.00	100.00	100.00
E.1045.50	100.00	100.00	100.00	100.00	100.00
E.1045.80	100.00	100.00	100.00	100.00	100.00
E.1046.10	100.00	100.00	100.00	100.00	100.00
E.1046.40	100.00	100.00	100.00	100.00	100.00
E.1046.70	100.00	100.00	100.00	100.00	100.00
E.1047.00	100.00	100.00	100.00	100.00	100.00
E.1047.30	100.00	100.00	100.00	100.00	100.00
E.1047.60	100.00	100.00	100.00	100.00	100.00
E.1047.90	100.00	100.00	100.00	100.00	100.00
E.1048.20	100.00	100.00	100.00	100.00	100.00
E.1048.50	100.00	100.00	100.00	100.00	100.00
E.1048.80	100.00	100.00	100.00	100.00	100.00
E.1049.10	100.00	100.00	100.00	100.00	100.00
E.1049.40	100.00	100.00	100.00	100.00	100.00
E.1049.70	100.00	100.00	100.00	100.00	100.00
E.1050.00	100.00	100.00	100.00	100.00	100.00

- NOTES:
1. THIS DRAWING IS SUBMITTED IN CONNECTION WITH THE PROPOSED SPILLWAY.
 2. FOR ADDITIONAL NOTES CONCERNING THIS DRAWING, SEE SHEET 10.
 3. E.L. - ELEVATION 2000.
 4. FOR DETAILS OF PRINCIPAL SPILLWAY SEE SHEET 10.
 5. FOR DETAILS OF PRINCIPAL SPILLWAY SEE SHEET 10.

AS BUILT 0 10 20 30
 FEB 6 1910
 SCALE IN FEET

PRINCIPAL SPILLWAY LAYOUT
MULTIPLE-PURPOSE STRUCTURE
NO. 9
UPPER TURTLE RIVER WATERSHED
 Grand Forks and Red River Counties, N. D.
 U. S. DEPARTMENT OF AGRICULTURE
 SOIL CONSERVATION SERVICE

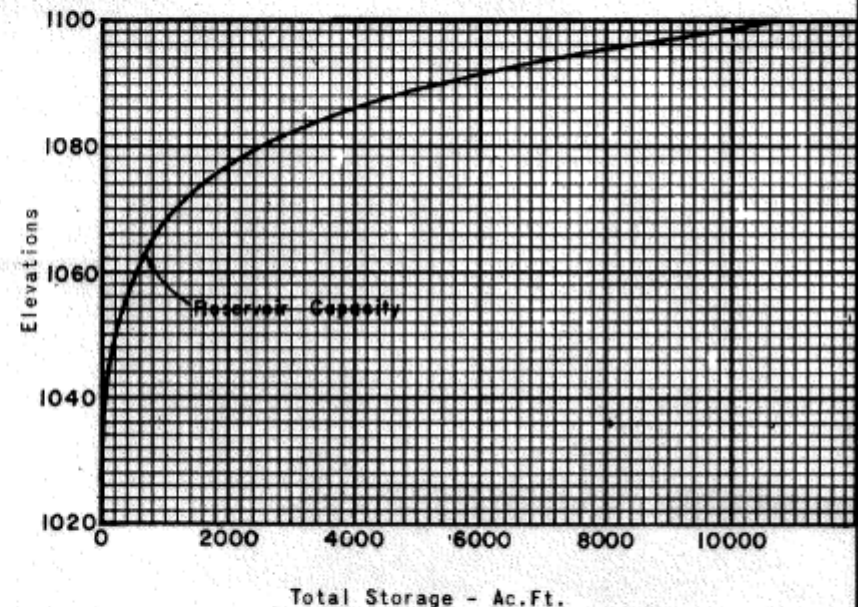
Drawn by: A. N. 4.15
 Check: A. N. 4.15
 Date: 10/10/10
 Sheet: 10 of 10
 Project: 5, E-36, 025

STRUCTURE DATA

Class of Structure b
 Drainage Area (total) 41.350 Ac. 64.6 Sq.Mi.
 (uncontrolled) 27.350 Ac. 42.7 Sq.Mi.
 Time of Concentration 13.3 Hours
 Soil Cover Complex Number 77 For A.M.C. II
 Sediment Capacity Available 292 Ac.Ft. below Elev. _____
 Total Sediment Capacity Available 292 Ac.Ft.
 Capacity Equivalents (Vol.) 0.13 In.
 Retarding Capacity Provided 4,180 Ac.Ft.
 2027 A.F. between 1st & 2nd stage
 Capacity Equivalents (Vol.) 1.84 In.
 Water Supply Provided 527 Ac.Ft. - Identify Uses
 (Recreation)
 Principal Spillway:
 Maximum Capacity (low stage) 240 c.f.s.
 Maximum Capacity (high stage) 1350 c.f.s.
 10 Day Drawdown Elev. 1075.6
 Emergency Spillway:
 Percent Chance Use 2 Storm Duration 6 Hr.
 Type Vegetated "n" Value Used 0.04
 Emergency Spillway Hydrograph for Class b Structures
 Rainfall 6.2 in.
 Runoff 3.7 in.
 Peak Inflow 9725 c.f.s.
 Maximum Discharge - Emergency Spillway 4783 c.f.s.
 Maximum Water Surface Elev. 1094.6
 Velocity of Flow (V) 7.95 f.p.s.
 Supplementary Data and Special Design Features:
 Duration of flow (ESH) - 30.75 hrs.

Freeboard Hydrograph for Class b Structures

Rainfall 11.44 in.
 Runoff 8.5 in.
 Peak Inflow 24,047 c.f.s.
 Maximum Discharge - Emergency Spillway 18,786 c.f.s.
 Maximum Water Surface Elev. 1099.6



MULTIPLE-PURPOSE STRUCTURE			
NO. 9			
UPPER TURTLE RIVER WATERSHED			
Grand Forks and Nelson Counties, North Dakota			
U. S. DEPARTMENT OF AGRICULTURE			
SOIL CONSERVATION SERVICE			
Designed <u>RDS</u>	Date <u>10-71</u>	Reviewed by _____	Title _____
Drawn _____			
Traced _____			
Checked <u>JEN</u>	<u>3-76</u>		<u>SE-36,025</u>

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021

10.2 Attachment 2: Site Inspection Photos/Checklist

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021

INSPECTION PHOTOS:



Photo 1: Principal spillway inlet, looking southwest (note water level, above crest of weir).

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 2: Principal spillway inlet, looking northwest.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 3: Gap in chain link fence of principal spillway outlet.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 4: Principal spillway outlet, looking east.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 5: Principal spillway outlet, looking northwest.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 6: Left side of principal spillway outlet.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 7: Right side of principal spillway outlet.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 8: Right foundation drain outlet, missing animal guard.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 9: Left foundation drain outlet with animal guard.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 10: Upstream slope, looking south.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 11: Debris on upstream slope.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 12: Woody vegetation on upstream slope.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 13: Downstream slope with trees in right abutment, looking north.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 14: Downstream slope, looking south.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 15: Wetland vegetation on downstream berm, looking south.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 16: Rutting on downstream berm.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 17: Crest of dam, looking north.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 18: Crest of dam, looking south.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 19: Auxiliary spillway, looking north.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 20: Auxiliary spillway, looking south towards inlet.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 21: Auxiliary spillway, looking southeast.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 22: Exit section of auxiliary spillway, looking east.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 23: Structure in inlet section of auxiliary spillway.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 24: Rutting from ATVs, auxiliary spillway.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 25: Woody vegetation, auxiliary spillway.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 26: Gopher mounds, auxiliary spillway.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 27: Erosion/slough in left side slope of auxiliary spillway.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 28: Cracking, Joint #1.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 29: Joint filler, Joint #2.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 30: Joint filler, Joint #3.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 31: Steel plate corrosion, Joint #4.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 32: Missing steel plate, Joint #5.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 33: Missing steel plate, Joint #6.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 34: Separated steel plate, Joint #7.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 35: Separated steel plate, Joint #8.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 36: Steel plate corrosion, Joint #9.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 37: Steel plate corrosion, Joint #10.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 38: Steel plate corrosion, Joint #11.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 39: Steel plate corrosion, Joint #12.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 40: Steel plate corrosion, Joint #13.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 41: Steel plate corrosion and wet on top, Joint #14.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 42: Steel plate corrosion, Joint #15.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 43: Steel plate corrosion, Joint #16.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 44: Steel plate corrosion and missing plate, Joint #17.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 45: Steel plate corrosion, Joint #18.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 46: Steel plate corrosion, Joint #19.

Upper Turtle River Dam No. 9 (Larimore Dam) Site Inspection
September 1st, 2020 and April 2nd, 2021



Photo 47: Missing steel plate, Joint #20.

INSPECTION CHECKLIST:

**NORTH DAKOTA DEPARTMENT OF WATER RESOURCES
INSPECTION CHECKLIST FOR EMBANKMENT DAMS**

Name of Dam: Upper Turtle River Flood Retention Dam 9

Date Inspected: 11/2/2022 Weather: 49°, clear skies, breezy

Hazard Classification: High Pool Elevation: Refer to Photo 1, approximate elevation 1067.4' (NGVD29), using downstream orifice as reference

Inspected By: Karli Frohling, PE and Ken Nysether, PE – HDR Engineering

Accompanied By: N/A

UPSTREAM SLOPE	Status	Comments
Erosion	No	
Wave erosion / scarp at waterline	No	
Riprap inadequate	No	Heavy vegetation and some woody vegetation in riprap
Grass cover inadequate	No	
Trees / bushes	Yes	Small tree near left abutment, trees in right abutment
Animal burrows	No	
Cracks	No	
Settlement / depressions	No	
Sinkholes	No	
Slides / bulges	No	
Additional Comments:	Debris and driftwood on slope	
CREST	Status	Comments
Erosion	No	
Trees / bushes	No	
Animal burrows	No	
Visible settlement / low areas	No	
Sinkholes	No	
Misalignment	No	
Cracks	No	
Grass cover inadequate	No	
Ruts or puddles	Yes	Minor rutting from wheel tracks
Additional Comments:		

DOWNSTREAM SLOPE		Status	Comments
Erosion		Yes	Severe wheel rutting on berm
Grass cover inadequate		No	
Trees / bushes		Yes	Minor woody vegetation, trees in both abutments
Animal burrows		No	
Livestock damage		No	
Cracks		No	
Settlement / depressions		No	
Sinkholes		No	
Slides / bulges		No	
Seepage		No	
Boils visible		No	
Toe drains flowing	Left gpm (est.)	No	No flow, clear of debris, animal guard intact
	Right gpm (est.)	No	No flow, clear of debris, missing animal guard
Abutment drain flow	gpm (est.)	N/A	
Relief wells flowing	gpm (est.)	N/A	
Additional Comments:		Wetland vegetation on berm – dry during inspection, wildlife trails on embankment	
PRINCIPAL SPILLWAY Not Applicable ☐		Status	Comments
A. Inlet			
Inlet clogged or obstructed		No	
Trash rack damaged or corroded		No	
Concrete cracking		No	
Concrete spalling		No	
Concrete reinforcement exposed		No	
Metal corroding / rusting		N/A	
Separation / displacement of joints		N/A	
Gates damaged		N/A	
Gates leaking		N/A	
Additional Comments:		Inlet appears to be in good condition	

B. Conduit	Not Applicable ☐	Status	Comments
Visible Damage		Yes	
Visible seepage		Yes	Water dripping at as-built joints 16 and 17
Interior Inspected		Yes	
Additional Comments	Joints at outlet are in good condition and are sealed		
C. Concrete Chute Spillway	Not Applicable ☒	Status	Comments
Concrete cracking			
Concrete spalling			
Concrete reinforcement exposed			
Joints show separation			
Joints displacement			
Joints show loss of joint material			
Joint seepage			
Drains / weep holes flowing			
Additional Comments:			
D. Outlet / Stilling Basin		Status	Comments
Erosion		No	
Riprap inadequate		No	
Outlet channel obstructed		No	
Energy dissipators deteriorated		No	
Seepage		No	
Additional Comments:	Some vegetation in riprap, fence around principal spillway completely rusted and has holes on each side		
LOW LEVEL DRAWDOWN	Not Applicable ☐	Status	Comments
Valve/Stem damage		N/A	Unknown
Valve Leaking		N/A	Unknown
Stoplogs damaged		N/A	
Stoplogs leaking		N/A	
Low-level operated		No	
Additional Comments:			

AUXILIARY SPILLWAY	Not Applicable ☐	Status	Comments
Grass cover inadequate		No	
Erosion		Yes	Large area with rutting from ATVs (20' x 40'), large areas where soil has collapsed/eroded at top of left side slope
Slides on spillway slopes		No	
Obstructions		No	
Additional Comments:	ATV tracks present, patches of woody vegetation, gopher holes on left side, post in exit slope, break in grade in exit slope (covered in wetland vegetation), cross drainage channel running transverse across spillway with cattails growing around it in exit slope		
INSTRUMENTATION	Not Applicable ☒	Comments	
Type of Instrumentation:			
Piezometers ☐ Other ☐			
Was instrumentation read? Yes ☐ No ☐			
ADDITIONAL COMMENTS			
Upper Turtle River Flood Risk Retention Dam 9 has an Emergency Action Plan.			

Attachment 1

Dam Name Upper Turtle River Flood Retention Dam 9

Principal Spillway Conduit Joint Inspection Summary

Date Inspected	Reservoir Elevation	Joint Number	Top Joint Gap	Bottom Joint Gap	Comments
11/2/2022		1			Right side spalling and cracking at wingwall intersection, crack on left side and loose seal
		As-built #3			
11/2/2022		2			Seal in good condition
		As-built #4			
11/2/2022		3			Start of straight section, seal in good condition
		As-built #5			
11/2/2022		4			Steel joint corrosion
		As-built #6			
11/2/2022		5			Steel joint corrosion, plate on left side is missing
		As-built #7			
11/2/2022		6			Steel joint corrosion, plate on left side is missing
		As-built #8			
11/2/2022		7			Steel joint corrosion, plate on right side is separated and plate on left side is missing, located 2-3' downstream
		As-built #9			
11/2/2022		8			Steel joint corrosion, plate on left side is separated near the bottom
		As-built #10			
11/2/2022		9			Steel joint corrosion
		As-built #11			
11/2/2022		10			Steel joint corrosion
		As-built #12			
11/2/2022		11			Steel joint corrosion
		As-built #13			
11/2/2022		12			Steel joint corrosion
		As-built #14			
11/2/2022		13			Steel joint corrosion
		As-built #15			
11/2/2022		14			Steel joint corrosion, water dripping from top
		As-built #16			
11/2/2022		15			Steel joint corrosion, water dripping from top
		As-built #17			

Date Inspected	Reservoir Elevation	Joint Number	Top Joint Gap	Bottom Joint Gap	Comments
11/2/2022		16			Steel joint corrosion
		As-built #18			
11/2/2022		17			Steel joint corrosion, plate on left side is missing
		As-built #19			
11/2/2022		18			Steel joint corrosion
		As-built #20			
11/2/2022		19			Steel joint corrosion
		As-built #21			
11/2/2022		20			Steel joint corrosion, plate on left side missing, concrete spalling
		As-built #22			
Notes: The interior of the conduit was inspected by HDR starting from the outlet, moving upstream. Did not measure joint gaps due to them generally being covered by steel plates.					

10.3 Attachment 2. TR 210-60 Peak Breach Discharge Calculations.

Watershed Name: Upper Turtle River					
Site No.: No.9					
Input Data					
Elevations					
Top of Dam, TD	1099.6	Ft msl	Top Width	20.0	Ft
Breach Hydrograph	1099.6	Ft msl	Upstream Slope Above Berm	3.5	:1
Wave Berm, WB	1065.0	Ft msl	Upstream Slope Below Berm	4.0	:1
Stability Berm	1065.0	Ft msl	Downstream Slope Above Berm	2.5	:1
Average Valley Floor	1044.0	Ft msl	Downstream Slope Below Berm	4.0	:1
Length of Dam at Breach Elev.	1050	Ft	Wave Berm Width	30.0	Ft
Volume of Breach, Vs	10410	Ac Ft	Stability Berm Width	50.0	Ft
Breach Discharge Computations					
Volume of Breach (Vs)				10,410	Ac Ft
Height Of Breach (Hw)				55.6	Ft
Cross-Section Area at Breach (A)				12,066	Ft ²
1. For depth of Water at the dam at the time of failaure where $Hw \geq 103$ ft					
$Q_{max} = 65 Hw^{1.85}$				110,000	cfs Eqn 1
2. For depth of Water at the dam at the time of failure where $Hw < 103$ ft					
$Q_{max} = 1,100 Br^{1.35}$				204,500	cfs Eqn 2
where					
$Br = (Vs * Hw)/A$				48	acres Eqn 3
but not less than					
$Q_{max} = 3.2 Hw^{5/2}$				73,800	cfs Eqn 4
nor more than Eqn 1					
3. When the width of the valley, L is less than,					
$T = 65 Hw^{0.35} / 0.416$				640	Ft Eqn 5
then replace Eqn 1 with					
$Q_{max} = 0.416 L Hw^{1.5}$				181,100	cfs Eqn 6
Breach Qmax for Hazard Classification =				110,000	cfs ←

10.4 Attachment 3. SITES Output

 SITES Integrated Development Environment [Project: d:\Dams\Rehab_Agreements\Larimore Rehab\H_H\NRCS\SITES\LarimoreRef

 File View Windows Help

	PSH_MassCurveB	PSH_MassCurveB1	PSH_MassCurveA	UTR9_2000	UTR9_FBH
Site Identification	D9	D9	D9	9	9
Watershed Runoff Curve Number	N/A	N/A	N/A	N/A	N/A
Total Watershed Drainage Area (Sq.Miles)	65.50	65.50	65.50	65.50	65.50
Watershed Time of Concentration (Hours)	N/A	N/A	N/A	N/A	N/A
SDH Rainfall Total (Inches)	N/A	N/A	N/A	N/A	N/A
SDH Rainfall Duration (Hours)	N/A	N/A	N/A	N/A	N/A
FBH or Storm Rainfall Total (Inches)	N/A	N/A	N/A	N/A	N/A
FBH or Storm Rainfall Duration (Hours)	N/A	N/A	N/A	N/A	N/A
SDH Inflow Peak (CFS)	N/A	N/A	N/A	6888.3	2478.9
FBH or Storm Inflow Peak (CFS)	N/A	N/A	N/A	6888.3	15574.0
Initial Reservoir Elevation (Feet)	1080.51	1072.86	1081.11	1076.80	1081.10
Maximum WS SDH (Feet)	N/A	N/A	N/A	1094.96	1090.89
Maximum WS FBH or Storm (Feet)	N/A	N/A	N/A	1094.96	1098.59
Storage at Max. WS FBH or Storm (Acre-Ft)	N/A	N/A	N/A	7382.8	8318.7
Top Dam (Feet)	N/A	N/A	N/A	1094.96	1098.59
Storage, Top Dam (Acre-Ft)	N/A	N/A	N/A	7379.0	8264.0
Emb. Yardage (CY)	N/A	N/A	N/A	N/A	N/A
PSH Drawdown (Days)	18.82	9.59	13.38	N/A	N/A
378 Drawdown (Days)	N/A	N/A	N/A	N/A	N/A
PS Crest (Feet)	1068.70	1068.70	1068.70	1068.90	1068.70
PS Number of Conduits	1	1	1	1	1
PS Conduit Diameter (Inches)	N/A	N/A	N/A	N/A	N/A
PS Conduit Height (Feet)	6.00	6.00	6.00	6.00	6.00
PS Conduit Width (Feet)	6.00	6.00	6.00	6.00	6.00
PS Conduit Area (Sq. Feet)	36.00	36.00	36.00	36.00	36.00
Storage, PS Crest (Acre-Ft)	658.	658.	658.	883.	658.
PS Discharge at AS Crest (CFS)	1300.7	1266.4	1278.1	1316.2	1271.5
PS Discharge for SDH (CFS)	N/A	N/A	N/A	1371.3	1270.1
PS Discharge FBH or Storm (CFS)	N/A	N/A	N/A	1371.3	1360.0
AS Crest (Feet)	1093.51	1090.63	1091.61	1090.50	1091.00
Storage, AS Crest (Acre-Ft)	5692.0	4490.2	4900.1	5003.0	4643.4
AS Width (Feet)	N/A	N/A	N/A	300.	300.
AS Exit Slope (%)	N/A	N/A	N/A	14.31	3.14
AS Ret. Curve Index	N/A	N/A	N/A	5.60	5.60
AS Veg. Cover Factor	N/A	N/A	N/A	0.85	0.85
AS Maintenance Code	N/A	N/A	N/A	2	2
AS Max. Head SDH (Feet)	N/A	N/A	N/A	4.46	0.00
AS Peak Discharge SDH/Storm (CFS)	N/A	N/A	N/A	4602.7	N/A
AS Exit Velocity SDH or Storm (Ft/S)	N/A	N/A	N/A	16.08	N/A
AS Stress SDH or Storm (Lb./Sq.Ft.)	N/A	N/A	N/A	1.836	N/A
Hp FBH or Storm (Feet)	N/A	N/A	N/A	4.46	7.59
AS Peak Discharge FBH/Storm (CFS)	N/A	N/A	N/A	4603.	13416.
AS Integ. Dist. FBH or Storm (Feet)	N/A	N/A	N/A	63.	BREACH
Oe/B FBH or Storm (Acre-Ft/Ft)	N/A	N/A	N/A	42.2	117.5
Uncontrolled Drainage Area (Sq.Miles)	65.50	65.50	65.50	65.50	65.50
Number of Errors	0	0	0	0	0
Number of Warnings	0	0	0	0	0

10.5 Attachment 5. Principle Spillway Conduit – Corrosion Detection Testing Report



Prepared for Larimore Dam Rehabilitation Project

Larimore Dam Principal Spillway Conduit - Corrosion Detection Testing Report (Draft)



Prepared by:



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United States
Department of
Agriculture

March 10, 2022

1: SUMMARY AND PURPOSE

On November 2nd, 2021, an inspection of the principal spillway conduit was conducted, by NRCS ND State Engineering staff (Nick Reynolds and Tom Schanandore), at the Larimore Dam site. The purpose of the inspection was to conduct NDT (Non-Destructive Testing) Corrosion Detection Testing of the reinforcing steel within the concrete of the conduit. This testing was conducted with the aid of an iCOR handheld device developed by Giatec Scientific Inc. This iCOR device uses CEPRA (Connectionless Electrical Pulse Response Analysis) technology (developed and patented by Giatec Scientific Inc.) to determine the corrosion potential of any given rebar that is relatively superficial to the surface of the concrete (within 6 inches). The iCOR device, when contacted with the concrete, creates an electrical circuit using the concrete and the integral rebar as the mechanisms for this flow of electrical current. The result of this testing gives an estimated rate of expected corrosion or “corrosion rate” for that rebar at that particular location.



Picture by Giatec Scientific Inc.

The goal of this testing was to collect a representative sampling of the corrosion potential of the principal spillway conduit concrete reinforcing steel at various locations throughout the conduit, for the overall purpose of evaluating the further service-life of the existing concrete. Corrosion potential is another way of stating the level of susceptibility of a particular reinforcing steel element to future corrosion as well as the time-frame within which this future corrosion damage would occur. The results collected, which will be detailed further along in this report, include two measurements: the first being the “Concrete Resistivity” and the second being the “Corrosion Rate” of the rebar. With these two measurements, an assessment can be made of the integrity of the concrete and the level of corrosion potential a particular reinforcing steel rebar is exhibiting.

In-order to conduct the corrosion detection testing, the conduit was accessed through the outlet works at the downstream toe of the dam embankment. The testing procedure involved a two-part process. The first step was to locate the rebar, to be tested, at each test location. A profoscope was used to locate rebar, determine size of rebar and to determine the depth of the rebar within the concrete (rebar cover). A grid was drawn on the surface of the concrete to mark the vertical and horizontal rebar locations along with the depth of the rebar measured. The second step was to conduct the corrosion detection test, using the iCOR device. The surface of the concrete was prepped by wetting the surface to

help the iCOR device create the electrical circuit through the concrete. Input values of rebar size, spacing and depth are entered into the device controller. The device is then placed at each vertical and horizontal rebar intersection and a corrosion detection measurement is taken for both the vertical and horizontal rebar at that location. Step 1 was conducted first at each test location, while progressing through the conduit starting at the outlet. The surface of the concrete was wetted after marking each location and allowed to penetrate the surface of the concrete while preparations were made to conduct step 2. Step 2 was then conducted repeated the progression through the conduit starting from the outlet. The iCOR device is linked to a tablet monitor controller, which stores and displays the measurement data.

2: iCOR NDT (NON-DESTRUCTIVE TESTING) CORROSION DETECTION SYSTEM

The iCOR NDT Corrosion Detection System, as discussed previously, uses CEPRA technology to create an electrical circuit through the concrete. Once this circuit is successfully created, the device then can measure the “concrete resistivity” of the concrete and the “corrosion rate” of the rebar of interest. See below for a schematic of this created circuit.

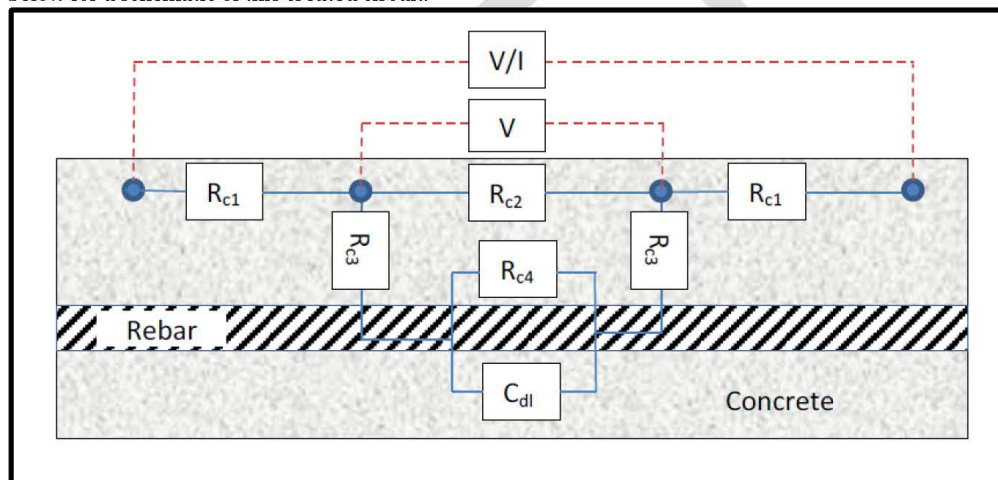


Figure 1: Schematic of Electrical Circuit Induced within Reinforced Concrete (Giatec Scientific Inc.)

The circuit in the schematic shown above is used to measure the corrosion potential via the inducing of electrical current through the circuit and recording the voltage response of the system. By prepping the concrete through wetting the surface, the water that seeps into the concrete helps to encourage conductivity within the concrete aiding in the creation of the electrical circuit. A circuit is created between the nodes of the device (shown by the four blue dots) through the concrete surface between them as well as through the depth of the concrete to the particular rebar being tested. The rebar in turn completes the circuit by means of its higher natural electrical conductivity. The recording of the voltage response then is assessed by means of the resistivity of the system. The electrical resistivity is broken down into three main resistivities: R_{c1} and R_{c2} which denote the electrical resistivity of the concrete between the nodes of the device; R_{c3} which denotes the electrical resistivity of the concrete between the nodes and rebar; and R_{c4} which denotes the electrical resistivity or charge transfer resistance of the steel rebar. Through this electrical resistivity and the rebar size, the “corrosion rate” (i_{cor}) can be

attained. The “concrete resistivity” is also attained through the direct recording of the concrete resistivity.

The two measured types of data from the corrosion detection test are “concrete resistivity” and “corrosion rate”. Concrete resistivity is measured in ohms-meter (ohms-m) and corrosion rate is measured in micro-meter per year ($\mu\text{m}/\text{year}$) loss of steel diameter. These two data measurements can then aid in assessing the concrete corrosion potential.

Corrosion Rate ($\mu\text{m}/\text{year}$)

The corrosion rate of the particular rebar being tested is presented in the form of the loss of bar diameter per year. The particular unit of measurement is micro-meter per year ($\mu\text{m}/\text{year}$), which is a rather small unit of material loss per year, but when the corrosion rate is found to be significant this material loss per year quickly can create problems for a reinforced concrete structure. The iCOR Giatec system along with other research on the subject of corrosion rate and loss of rebar material through corrosion, comprise a method of understanding what is considered a significant corrosion rate versus a non-significant corrosion rate. The corrosion rate table, given by Giatec for use with the iCOR device, is presented below. The table indicates four levels of corrosion rate significance. The passive/low classification indicates that the particular rebar being tested has a relatively low corrosion rate or has passive corrosion which means that the corrosion process has most likely not initiated within the rebar. When a particular rebar shows low corrosion rate (indicated by a green color on the iCOR device), it is most likely that the rebar is not in a state of active corrosion. The next three classifications (Moderate, High and Severe), indicate active corrosion in the rebar, but are separated into levels of significance. There has been much research on what signifies a significant corrosion rate and of course the higher the corrosion rate the more significant the damage that can be expected. In order for an objective assessment of the further serviceability of existing concrete structures to be accomplished, a translation from corrosion rate to a remaining service-life of the concrete must be conducted. See section 3 below for this conversion.

	Corrosion Rate		
Color Code	($\mu\text{A}/\text{cm}^2$)	($\mu\text{m}/\text{year}$)	Classification
Green	< 1	< 10	Passive/Low
Yellow	1 – 3	10 - 30	Moderate
Orange	3- 10	30 - 100	High
Red	> 10	> 100	Severe

Table 1: Corrosion Rate Table (Giatec Scientific Inc.)

Concrete Resistivity (ohms-m)

The concrete resistivity table, given by Giatec for use with the iCOR device, is presented below. The concrete resistivity helps to give an assessment of the durability and integrity of the concrete. Although, this resistivity of concrete is affected by many factors including moisture content; density; original water-cement ratio; presence and quantity of rebar; type, size, and quantity of aggregate; cracks and defects; etc. With all these variable factors it is difficult to make an assessment of specific structural material properties (like concrete compressive strength), so the use of the data gathered from the concrete resistivity measurement, should be considered an auxiliary or supporting measurement which can aid in further understanding of the concrete durability and corrosion potential. The corrosion rate table above indicates that the higher the corrosion rate the greater the potential for corrosion and damage of the concrete. The concrete resistivity is the inverse of that, where the lower the value of concrete resistivity the greater the potential for corrosion and damage of the concrete. The very high classification (indicated by a green color), has the greatest range of values, ranging from 200 ohms-m to 100,000 ohms-m. This signifies the greater variability of the concrete resistivity due to the large number of variables associated with the measurement. One of the main factors of this high range is the presence of water within the concrete pores. The presence of water in the pores helps to decrease the concrete resistivity and also aids in creating the electrical circuit through the concrete and to the rebar. Hence, the preparation work of wetting the concrete prior to conducting the iCOR test. In general, as long as the concrete resistivity is found to be in the very high classification the durability of the concrete is adequate. There is also a general correlation between the corrosion rate and concrete resistivity, which follows the trend of the higher the corrosion rate the lower the concrete resistivity. This correlation is location specific since the concrete material may be different from location to location, but the trend remains as a sign of decreasing concrete durability and possible damage through corrosion.

Color Code	Resistivity (Ω .m)	Classification
Green	>200	Very High
Yellow	100-200	High
Orange	50-100	Moderate
Red	<50	Low

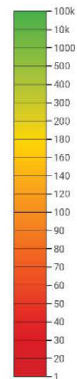


Table 2: Concrete Resistivity Table (Giatec Scientific Inc.)

3: ESTIMATION OF REMAINING SERVICE-LIFE OF EXISTING CONCRETE

There are several factors that influence the structural integrity of reinforced concrete. In regard to corrosion of reinforcing steel rebar, there are three critical factors. These factors are: 1 - area of reinforcing steel, 2 - the bond strength of the concrete to the reinforcing steel rebar and 3 - the compressive strength of the concrete itself. All three of these factors are influenced negatively by the corrosion of reinforcing steel. As reinforcing steel corrodes the area of steel decreases. This area of steel is used in the design of reinforced concrete, where a particular amount of steel is needed to resist tensile or shear forces within the concrete. As this steel decreases in area, through corrosion, the intended design and load carrying capacity of the reinforced concrete also decreases. Along this same line, as the surface of the reinforced steel corrodes the bond between the concrete and the steel rebar lessens. For reinforced concrete applications, where reinforcing steel is designed to resist high tensile loads within the concrete, this loss of bond strength is of critical concern. The last factor is the decrease in compressive strength. While one face of a concrete member is subjected to tensile loads, the opposite face is subjected to compressive loads. Corrosion of reinforcing steel, when significant, can cause breakout and cracking of concrete. This would decrease the compressive strength of the concrete and where high compressive loads are present, this could create compressive failure.

The corrosion rate, measured by the iCOR device, can be used to assess the amount of steel left from the original steel area or make estimation on how much material loss (bar diameter loss) can be expected over a period of time. This loss in reinforcing steel area can be directly utilized to assess the decrease in structural load-carrying capacity of the concrete, based upon the area of steel. This assessment being based upon area of steel alone, although theoretically correct, does not consider when the bond strength between the rebar and concrete decreases to a point when the reinforcing steel is no longer effectively bonded to the concrete. Since bond between the steel rebar and the concrete is fundamental and crucial for the structural design and function of the reinforced concrete, this ends up being the controlling mechanism of structural concern.

Mechanisms of Rebar/Concrete Bond Loss

There are three major components that attribute to the bond between concrete and reinforcing steel rebar. These components are chemical adhesion bond, friction bond, and mechanical bond. The chemical adhesion bond between the concrete and rebar are enacted during the pouring and curing process; the friction bond is created by the confinement forces that the concrete enacts upon the reinforcing rebar; and the mechanical bond is the interlocking of concrete cement and aggregate between the ribs upon the rebar. The chemical adhesion bond is generally removed after the smallest of loading, leaving only the frictional and mechanical bonding components. These remaining two bonding components would then have to be overcome by means of mechanical force. This mechanical force is what is designed for when reinforcing steel is embedded into concrete and as long there is adequate embedment of the steel within the concrete the rebar would have enough bond strength to resist loads up to the yielding point of the steel. The one detrimental factor to this design, would be the addition of corrosion into the environment. The corrosion of the reinforcing steel significantly reduces the resistance of these frictional and mechanical bonding components, resulting in the loss of bond strength to a point where the bar would fail in slipping instead of in yielding of the steel.

Critical Bond Loss due to Rebar Corrosion

Current research (ACI Materials Journal, Title no. 97-M28, Bond Behavior of Corroded Reinforcement Bars), concludes that a 2% of mass loss in a #6 rebar is the critical point at which bond loss between the concrete and the rebar can be considered a significant loss of bond strength and slipping of the rebar begins to control as the most critical failure mechanism. The 2% mass loss of a #6 bar (diameter = 0.75") corresponds to a loss of bar diameter of 0.00752 inches or 191 μm . The 191 μm is the total amount of rebar diameter loss that would be acceptable before structural integrity of the reinforced concrete member would be considered to be inadequate due to the loss of sufficient bond strength. Of course, this total diameter loss does not happen overnight. The corrosion rate, once the corrosion process is activated, would control the time over which the loss of diameter would occur. For example, a rebar corroding at a corrosion rate of 10 $\mu\text{m}/\text{year}$, if remained constant, would take a time of about 19 years for the 191 μm of diameter loss to be reached. The conclusion here, would be that the particular rebar would have a remaining service life of 19 years before it would be considered structural inadequate.

The experiment, that details these findings of the 2% critical mass loss, was conducted for the ACI Materials Journal Article and is referenced above. In this experiment, only #6 bars were considered. When considering differing bars sizes, a 2% mass loss does not correlate to a diameter loss of 191 μm . Since the iCOR device records corrosion rate in diameter loss per year, the 191 μm of critical diameter loss is the datum at which all bar sizes are considered and not the 2% mass loss. Furthermore, the loss of bar diameter can be directly related to the loss of bond strength since it is the surficial area of the bar, when corroded, that losses bond with the concrete.

The results shown in this report use the 191 μm as the corrosion threshold; the point at which the reinforcing steel would be considered structurally inadequate. Based upon the documented research and experimental data, this threshold holds true for #6 bar, but there is a lack of research and experimentation that would support the assertion that the 191 μm threshold is true for differing bar sizes. Because of this lack of research and experimental data, engineering judgement and assumptions are conservatively made in regard to using the 191 μm threshold and may be subject to minor changes as research is developed for this specific technology.

Estimating Service-Life

Using the 191 μm threshold, an estimation of the remaining service-life of existing reinforced concrete can be conducted. Simply dividing the 191 μm threshold by the corrosion rate, measured by the iCOR device, will result in the estimated service-life. See figure 2 below for the resulting service-life based upon corrosion rate measured. Note the service-life is given in a range that corresponds with the iCOR corrosion rate classifications presented earlier.

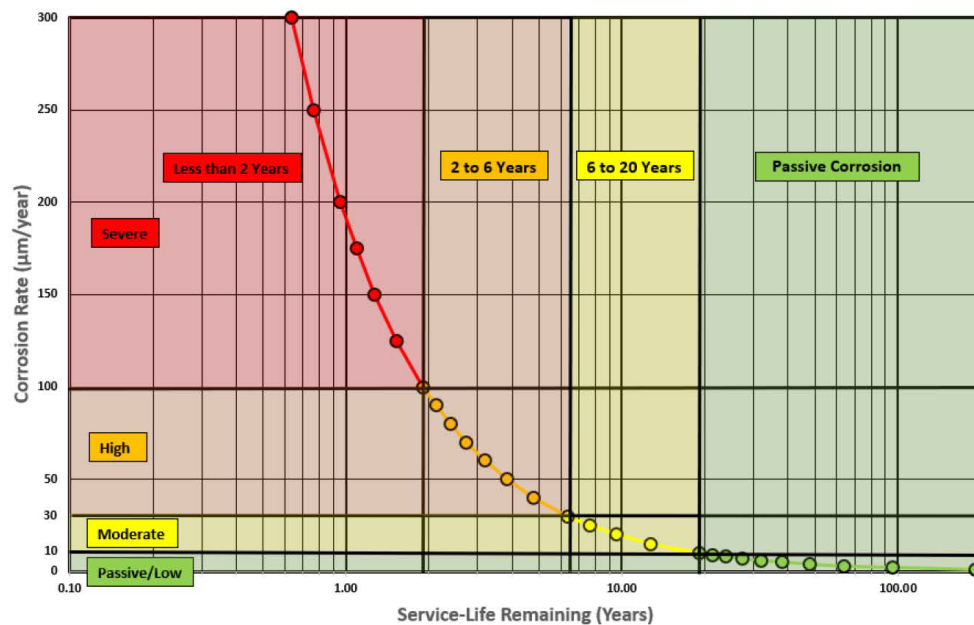


Figure 2: Service-Life Estimate based upon Measured Corrosion Rate

A “passive to low” corrosion potential classification is given to the range of corrosion rate from 0 to 10 µm; a “moderate” classification is given to the range of corrosion rate from 10 to 30 µm; a “high” classification is given to the range of corrosion rate from 30 to 100 µm; and a “severe” classification is given to corrosion rates higher the 100µm. The respective estimated service-life ranges are given as less than 2 years for “severe”; 2 to 6 years for “high”; 6 to 20 years for “moderate”; and passive corrosion given for “passive to low”. The “passive to low” classification is not given an estimated service-life time since the corrosion rate is low enough to be considered passive. At these low levels, the corrosion rate is either very small or the corrosion process has not yet initiated within the steel. Corrosion rates measured within the “passive to low” classification may vary throughout the range, but generally when a “passive to low” classification is measured, the concrete reinforcing can objectively be considered structurally sound, although engineering judgement must be used for tested locations where the data is consistently at the upper range of the “passive to low” (7-10 µm). This may be evidence of the initiation of the corrosion process. Corrosion rates are still low, but these locations must be consistently monitored. “Passive to low” classification will be considered as “No Corrosion” present.

Interpreting Measurement Data

Careful interpretation of measurement data must be considered when using data to estimate service-life of reinforced concrete structures. The service-life estimations presented above, are based upon the whole diameter of the particular rebar being tested. Determination of exact service-life is difficult for existing reinforced concrete structures, since the history of corrosion rate has not been recorded over

their lifespan. Although this is true, there is a lot that can still be taken from the iCOR corrosion rate measurement results. The classifications can also be considered as levels of certainty of corrosion. The higher the classification the more certainty there is, that corrosion is present. Furthermore, as the total amount of corrosion increases the rate of corrosion increases. This is due to the damage to the concrete caused by the corrosion process, which promotes faster and more severe corrosion. For example: just because the measured corrosion rate is 40 $\mu\text{m}/\text{year}$, which translates to 4.8 years of remaining service-life, does not mean that an exact 4.8 years remains before the concrete is structurally inadequate. It is very possible that the corrosion rate may increase. This means that consistent inspection and testing must be conducted to record the change in corrosion rate over time. Ultimately, a plan of corrosion detection should be implemented if a corrosion history is to be determine for the structure. The corrosion rate measured over any given time period can be used to assess the actual loss of rebar diameter based upon the changing corrosion rate.

4: CORROSION DETECTION RESULTS

Two locations along the principal spillway conduit, were tested using the iCOR corrosion detection device. One test location was conducted in the first conduit in from the riser tower inlet and the other was conducted upon the left wall of conduit 8. See attachment 1 for these locations. The results at each location are shown below in table 3. See also Attachment 2 which shows the results overlayed upon the conduit locations.

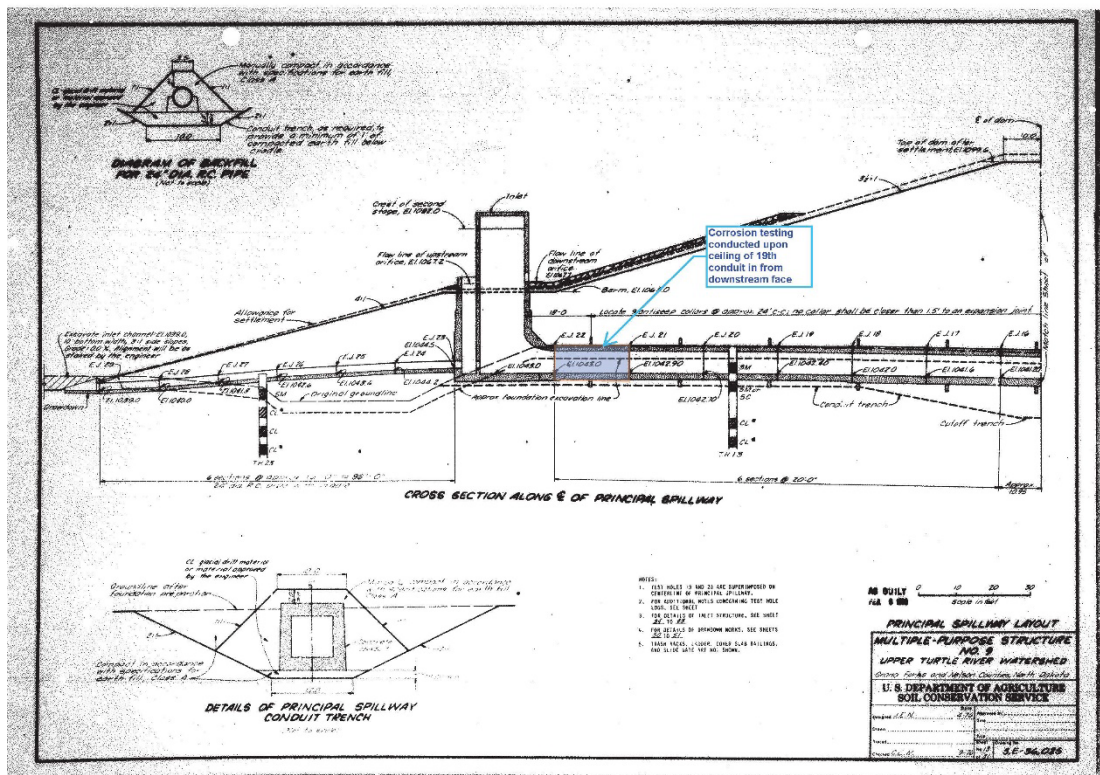
Test Location	Max. Corrosion Rate ($\mu\text{m}/\text{year}$)	Min. Concrete Resistivity ($\Omega\text{-m}$)	Corrosion Rate Classification	Estimated Service-Life
Conduit 8 – Left Wall	15.65	813	Moderate	6-20 years
Conduit 19 - Ceiling	8.17	913	Passive/Low	No Corrosion

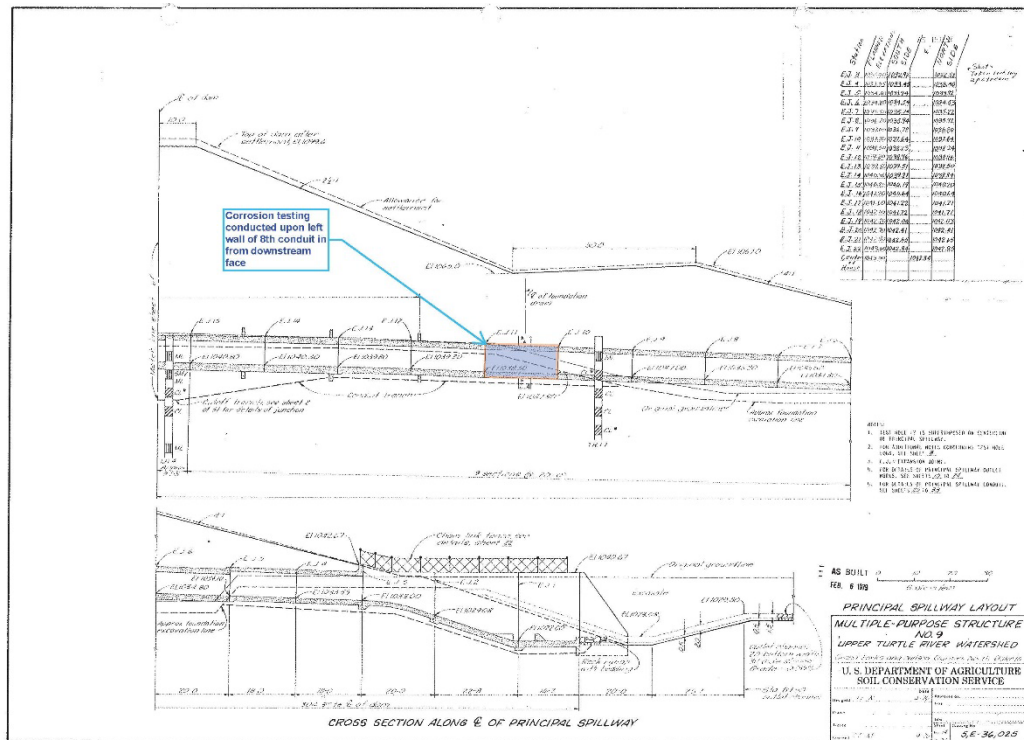
Table 3: Corrosion Detection Results

A passive/low classification was found at conduit 19 and a moderate classification was found at conduit 8 location. Although the corrosion rate classification was found to be low at conduit 19, the location of the maximum corrosion rate (8.17 $\mu\text{m}/\text{year}$) occurred at the joint edge nearest the riser tower. At this location the joint showed significant signs of corrosion occurring at the joint dowels resulting in concrete damage and missing concrete from the joint. This location should be subjected to further testing.

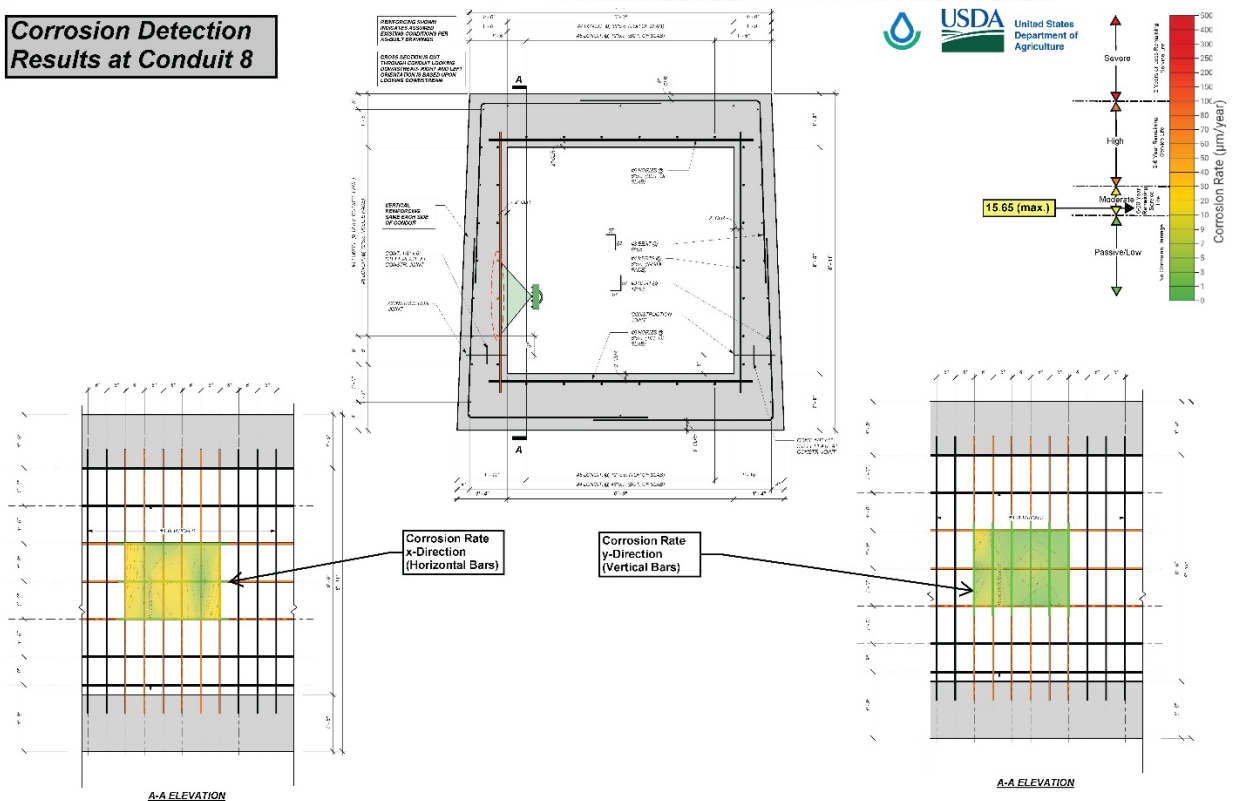
The location that recorded a moderate classification may be signs of early on-set initiation of the corrosion process. The amount of testing conducted for this report is only representative of 0.12 % of the total conduit interior surface area. In addition, tests were only conducted in 2 out of the 19 conduit sections and upon selective locations within these particular conduits. There is much more of the principal spillway structure that would need to be tested to achieve an appropriate representative sampling of the structure as a whole. The data collected to date, cannot be considered representative of the entire structure. Although, the data still is beneficial in understanding the corrosion potential.

It is recommended that further corrosion detection testing be conducted throughout the conduit structure. This report is considered preliminary in nature and shall be updated with future test results. A strategy for future testing would include the testing of the walls, floors and ceiling of each conduit section, as well as the conduit concrete near joints. Testing of inlet and outlet structures (Riser Tower, SAF stilling basin, etc.) may also be conducted.

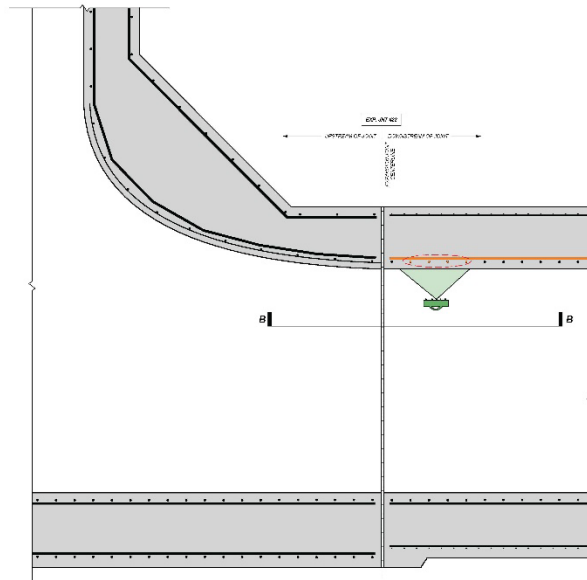




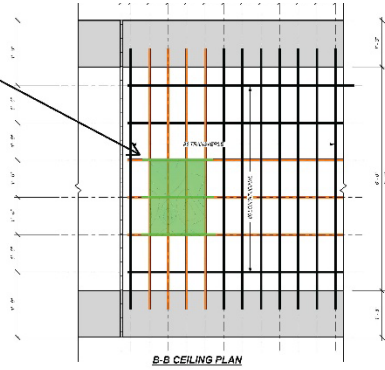
Corrosion Detection Results at Conduit 8



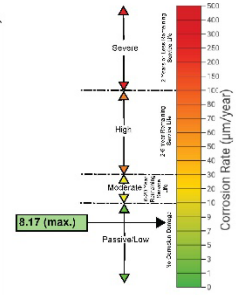
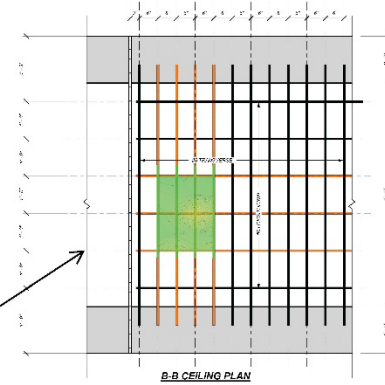
Corrosion Detection Results at Conduit 19



Corrosion Rate
x-Direction
(Horizontal Bars)



Corrosion Rate
y-Direction
(Vertical Bars)



10.6 Attachment 6. Geotech Report Addendum #1 and 2001 EWP Auxiliary Spillway Repair Plans

August 13, 2024 Addendum to Larimore Dam Rehab Geotechnical Engineering Report

Geotechnical Engineering Report, prepared February 28, 2022 by HDR Engineering, Inc., evaluated the geotechnical properties of the Upper Turtle River Dam #9 (Larimore Dam), including the head cut erodibility index (Kh) calculated value of 0.035 for auxiliary spillway SITES analysis. Table 1 is screen capture from HDR Engineering, Inc. 2022. HDR Engineering, Inc. (2022) Report states that the SITES analysis will be completed at later date. However, the values presented in HDR (2022) are based on laboratory test results from all of the borings advanced in 2020 and 2021 rather than just the borings completed for the auxiliary spillway. As such, the percent clay used is higher than the soils encountered in the auxiliary spillway. The three samples sieved have a percent clay of 28.7, 23.7, and 23.7 and percent fines of 67, 57, and 47. The Liquid Limit (LL) is 33, 32, and 34, the Plastic Limit (PL) is 19, 18, and 19, and the Plasticity Index (PI) is 14, 14, 15. Using the LL and PL for the three samples, all the samples classify as a CLAY (CL). Therefore, the values in this Table are not accurate to the soils in the auxiliary spillway.

Table 1. Auxiliary Spillway Soil Parameters from HDR Engineering, Inc. (2022)

Layer	Dry Unit Weight, γ_d (pcf)	% Clay	K_h	D_{75} (mm)	PI
Upper Glacial Till	104.13	29.48	0.035	0.17	18.12

The auxiliary spillway material properties were adjusted based on the laboratory test results on the soil obtained from the 2020 borings, boring numbers 201, 202 and 203. These results, as summarized above result in an average PI of 14, and average percent clay of 25.4 for the gray Sandy Lean Clays encountered in the auxiliary spillway. The dry density of this strata are reported as 110 pounds per cubic foot (pcf) at a depth of 16 feet below grade, 112 pcf 21 feet below grade, and 112 31 feet below grade. A dry density of 110 pcf was used as to best reflect the density of the soils closest to the surface. The representative particle diameter, D_{75} in millimeters, is reported for the auxiliary spillway borings as 0.16 mm, 0.25 mm, and 0.30 mm resulting in an average D_{75} of 0.24 mm or approximately 0.009 inches.

During existing conditions and alternatives development by NRCS North Dakota, it was determined that the June 2000 flood should be used for calibration of the SITES model, including auxiliary spillway erosion. During the June 2000 flood event the auxiliary spillway eroded about 150 feet from the EWP drawings, and Figure 1 show the location of the erosional event within the auxiliary spillway. Figure 1 is aerial photography from Google Earth of the auxiliary prior to June 2000 flood. The aerial photo dated

September 16, 1997, shows the auxiliary had trees on a hill slope. The June 2000 flood eroded at the location of trees. In 2001, an Emergency Watershed Protection (EWP) project was completed to remove the trees within the auxiliary spillway and re-grade erosion that occurred during the flood event. Figure 2 shows aerial photography from Google Earth after the EWP regrading was completed. The date of the aerial photography is June 27, 2003.



Figure 1. Google earth aerial photography dated September 16, 1997. The Pale yellow polygon represents the location of auxiliary spillway erosion and the EWP tree removal and grading repair.

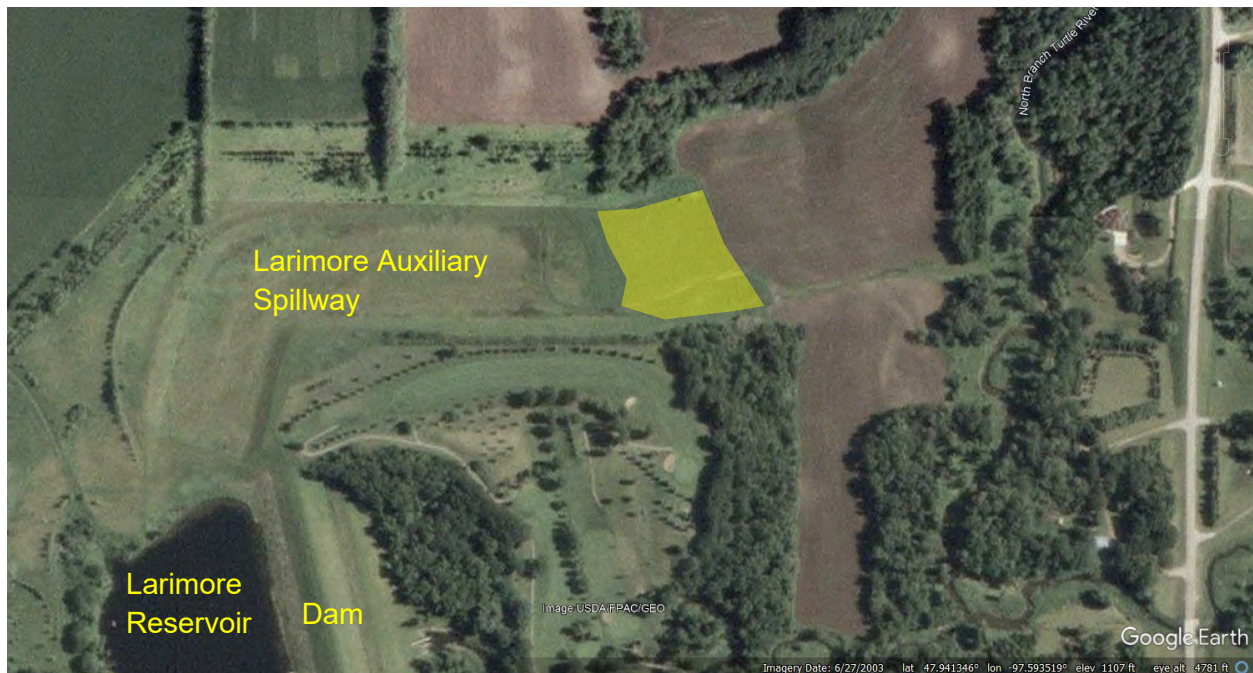


Figure 2. Google earth aerial photography dated June 27, 2003. The Pale yellow polygon represents the location of auxiliary spillway erosion and the EWP tree removal and grading repair.

The maximum point precipitation for the June 2000 event in the watershed measured 20.0 inches. The June event was used for H&H modeling calibration, and peak reservoir level results of 4.5 feet of depth over auxiliary spillway level section matched local testimonials that peak depth was between 4 and 5 feet. The head cut advance shown on EWP design drawings was ~150 feet. Adjustment of the head cut erodibility index to match ~150 feet of head cut advance, resulted in a value of 1.3. With originally assumed head cut erodibility index of 0.035, the 2000 flood event SITES model result would breach the auxiliary spillway; therefore, the head cut erodibility index will need adjusted from the HDR Engineering, Inc. (2022) Report.

A head cut erodibility index value of 1.3 aligns with soft or jointed rock, typically between the competence of shale and sandstone. However, the findings of boring logs along the auxiliary spillway are not rock but considered glacial till with classification of “CL” and description of “gray and dark gray medium stiff to stiff sandy lean clay, trace gravels, and occasional cobbles and boulders”. NRCS examples with calculated head cut erodibility value for “stiff CL” is 0.16 or 0.17.

Kh = 0.16



DESCRIPTION: CL, stiff, glacial till
SITE: Misteguay 4, Michigan
COMMENT: Eroding material on headcut face

Kh = 0.17



DESCRIPTION: CL, stiff
SITE: Rush Creek 12R, Oklahoma
COMMENT: Eroding material in headcut face

With the adoption of head cut erodibility index of 0.16 based on geology here considered glacial till, the 2000 flood auxiliary spillway erosion is greater than ~150' noted in EWP plans; however, the 2000 flood did not fully breach the auxiliary spillway. Furthermore, the 0.16 value aligns better with NRCS guidance and still considers calibration results. Therefore, the head cut erodibility index to be used for existing conditions and alternative development is 0.16. Head cut erodibility index for in-situ soils has high sensitivity and uncertainty. The following table, Table 2, summarizes the revised soil parameters as discussed and used for SITES analyses.

Soil Classification	Dry Density (pcf)	% Clay	Plasticity Index	D ₇₅ (inches)	Kh
Sandy Lean Clay	110	25.4	14	0.009	0.16

Table 2. Revised Auxiliary Spillway Soil Parameters based on 2020 Borings 201, 202 and 203

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