UPPER TURTLE RIVER DAM NO.9 (LARIMORE DAM)

Appendix D-4 Concept Design Report

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February 20, 2026

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ACRONYMS

ACB	Articulated Concrete Block
ASH	Auxiliary Spillway Hydrograph
FBH	Freeboard Hydrograph
KSU	Kansas State University
NDDWR	North Dakota Department of Water Resources
NRCS	Natural Resources Conservation Service
PMP	Probable Maximum Precipitation
PSH	Principal Spillway Hydrographs
SAF	St. Anthony Falls
USACE	U.S. Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Services
USDA	U.S. Department of Agriculture

1 BACKGROUND

The purpose of this report is to document the work completed related to design for the preferred structural alternative associated with Larimore Dam. Larimore Dam is located in Section 32 of Hegton Township (T152N, R54W), Grand Forks County, ND, and has a drainage area of 65.5 square miles. The dam was built with joint purposes, to provide flood control and recreation opportunities along the South Branch Turtle River and Turtle River.

1.1 RELEVENT ANALYSIS COMPLETED PRIOR TO THIS REPORT

For information related to the existing conditions analysis for Larimore Dam, refer to **Appendix D-1** and **D-2** of the Watershed Plan. **Appendix D-1** (Existing Conditions Assessment Report) details the current hazard classification and deficiencies associated with Larimore Dam. Information on the geotechnical characteristics of the existing structure and the proposed alternatives are available in **Appendix D-2** (Geotechnical Engineering Report). Information related to the development of alternatives as well as how the range of alternatives was narrowed is available in **Appendix D-3** (Alternatives Evaluation Report).

1.2 VERTICAL DATUM

All elevations referenced within this report refer to the NAVD 1988 vertical datum, feet.

2 PROPOSED SITE CHARACTERISTCS

2.1 EMBANKMENT

The auxiliary spillway capacity is sufficient to pass the freeboard hydrograph with sufficient freeboard for Larimore Dam. Therefore, the top of dam elevation does not need to change. In order to meet the NDDWR Dam Safety Standards (NDDWR, 2024) for minimum crest width; the existing width of 20 feet is increased to 24.0 feet (10 feet + 20% (66 feet)).

Appendix D-2 (Geotechnical Engineering Report) and FEMA (FEMA, 2011) recommends a two-stage chimney and blanket drain system for embankment or foundation soils comprised of dispersive soils with existing embankment fill properties. A two-stage chimney and blanket drain replaces the existing foundation drain due to the presence of potentially dispersive soils, and compatibility analysis results documented in **Appendix D-2.** The two-stage chimney drain would convey any seepage within the dam to the horizontal blanket drain and collector toe drain pipe to protect against erosion and piping in the dam. The filter and drain significantly reduce seepage path development due to potential defects, dispersion of clay participles into solution, or high phreatic surface levels. The first stage consists of a sand chimney filter placed on the downstream slope of the dam; the goal is to place it as close to the center of the embankment as possible without compromising the stability of the embankment. Therefore, the stability berm is excavated and sand filter installed. The chimney filter serves to provide drainage within the embankment to permit the passage of water and prevent the movement of the embankment soil. The second stage consists of a gravel chimney drain, which is downstream of the sand filter, and serves to collect the water and convey it safely to the horizontal blanket drain and ultimately the toe drain and collector pipe at the toe of the dam.

2.2 PRINCIPAL SPILLWAY

The principal spillway is being replaced due to the remaining service life estimate range from 6-20 years based on corrosion rate testing, which would not accommodate a 100-year extension of the design life. Larimore Dam's principal spillway (6' x 6' box) meets capacity criteria outlined in Technical Release 210-60 (NRCS, 2019 March). The SITES (USDA and KSU, 2014) analysis indicates the auxiliary is not reached by routing the Principal Spillway Hydrographs (PSH). Corrosion analysis and hydraulic routing results are documented in **Appendix D-1**. Due to bore and jack preferred construction method, a round conduit is required. Therefore, the box culvert is replaced with similar but slightly smaller round conduit (6.5' diameter), which meets PSH criteria, while choosing a standard round number diameter. The riser tower will be replaced the existing tower with identical dimensions, with upstream first stage orifice height increased to 2.5 feet. Additional information on the proposed principal spillway conduit, riser tower, and energy dissipation method is provided in the following sub-sections.

2.2.1 PRINCIPAL SPILLWAY CONDUIT

The proposed conduit through the embankment is to be installed via bore and jack construction methods. The bore and jack method is the preferred method over an open cut method for several reasons. Using an open cut method would be more costly due to the excavation of embankment material and the placement of that material at an acceptable compaction level after the conduit has been placed. Differential settlement of the embankment is possible with the open cut conduit installation method, which could potentially cause discontinuities along the embankment.

The proposed principal spillway conduit is to be placed with similar alignment to the existing conduit and will be approximately 240 feet north of the existing conduit alignment. The NRCS construction specification for Boring and Jacking (NRCS, 2015) will be used for the conduit installation. The proposed conduit will be a 78" reinforced concrete pipe (RCP). A special design for the conduit may be required to achieve higher concrete compressive strength to counter the extreme jacking pressures anticipated for the long boring distance. The existing 6' x 6' box conduit will be grouted with microfiber concreate and abandoned, so that it will not cause seepage or stability issues in the future.

The slope of the conduit through the embankment was determined based on dam geometry and inlet/outlet elevations, which is slightly lower slope compared to existing conditions (0.026 feet/feet). The proposed slope of the 78" conduit will be 0.025 feet/feet. The conduit was simulated in the SITES program and the resultant reservoir levels are documented in sections 4 and 5. Approximately 300 feet of the 78" conduit will be installed with a bore and jack construction methods through the center portion of the embankment. The additional 150 feet of conduit at connection to riser tower and under new downstream stability berm will be placed under the proposed embankment fill using open trench methods. Preliminary dimensions of the principal spillway conduit are shown on sheet 9 in the preliminary plan set shown in **Attachment D-4-1.**

2.2.2 PRINCIPAL SPILLWAY RISER TOWER

A new principal spillway riser tower is proposed for the preferred alternative at Larimore Dam. The orifice openings for the existing principal spillway riser tower do not pass PSH without activating the auxiliary spillway. However, increasing size of the upstream first stage opening to 2.5-feet high would adequately meet the PSH requirements. Even so, the existing riser tower would not be salvaged for use with the preferred alternative due to the new shape of principal spillway conduit and limited design life of similarly aged concrete and rebar in conduit. A corrosion analysis has not been completed within riser due to accessibility, however the conduit is of similar age and construction methods. Therefore, a similar 6 to 20 years of service life remaining is also assumed for the riser tower. Retrofitting the existing riser tower with the 78" conduit has the potential to cause issues with the structural stability of the existing riser tower. There were also minor issues associated with the existing riser tower structure as closing the low-level outlet was very challenging when it was opened in 2024. The configuration for the proposed two-way covered riser aligns with existing two-way covered riser.

Dimensions of the proposed two-way covered top riser were determined based on guidance provided in *Hydraulics of Two-Way Covered Risers* (Alling, 1965). The NRCS SITES program was used to verify that the proposed dimensions would pass the PSH without activating the auxiliary spillway. The two orifice openings on the proposed riser tower will be 4.5 feet wide by 2.5 feet tall. The total weir length associated with the second stage of the riser tower is proposed to be 24 feet. Preliminary dimensions of the proposed two-way covered riser tower are shown on sheet 11 in the preliminary plan set shown in **Attachment D-4-1**. Wall and slab thicknesses shown for the riser tower were determined based on guidance provided in *Building Code Requirements for Structural Concrete* (ACI 318-19) and were verified to pass NRCS requirements in *Technical Release 210-30 – Structural Design of Standard Covered Risers* (Alling, 1965). Calculations used to develop wall and slab thicknesses are shown in **Attachment D-4-2 and D-4-3**.

2.2.3 LOW-LEVEL DRAWDOWN AND SEDIMENT STORAGE

A low-level drawdown conduit is shown in the preliminary plan set provided in **Attachment D-4-1**, sheet 9 of 11. The elevation of the low-level drawdown conduit is ~1045, which is flat or very low slope from reservoir and base of dam to riser tower. The new drawdown pipe is ~5 feet higher than as-built low-level drawdown conduit, which is 1040.2 feet. The minimum elevation in reservoir at the time of the construction of Larimore Dam was approximately 1025 feet (based on bathymetric pre-dam quad maps). The depth of sediment at the base of the dam is no greater than 4 feet based on results documented in **Appendix D-7**, including addendum**.** Operation of the low-level outlet in 2024 had some mechanical issues with opening and closing, but when the gate was open, flow was present. Sedimentation around the inlet did not appear to be causing limitations to operating the low-level drawdown. Therefore, the depth from current sediment pool to the proposed low-level drawdown invert was only reduced from ~15 feet to 10 feet since construction of the dam. Within **Appendix D-7** there was a recommendation that the drawdown elevation should no lower than 1050.0, which is based on volumes. This would limit future complete drawdowns and seems to neglect that sedimentation depths near the dam are generally less than 4 feet, whereas much greater depths occur in the upper portion of the reservoir at the delta.

If deposition continues to occur in a similar manner over the next 100 years after rehabilitation of Larimore Dam, the as-built elevation of the low-level drawdown will be adequate to ensure that there is enough sediment storage below the conduit. Based on the current farming practices in the watershed upstream of Larimore Dam, along with sheet, rill, and streambank erosion estimates upstream of the dam, the estimated sediment accumulation rate in the reservoir following rehabilitation is 4.5 acre-feet per year. The project life of the rehabilitation is 100 years. An additional 5 years was used for sediment prediction because bathymetric survey data was completed in 2020 which was the beginning of the planning effort. The timeframe used for sediment accumulation analysis is 105 years. Therefore, the total sediment accumulation expected in the reservoir at the end of the project life is approximately 659 acre-feet.

Throughout the 100-year design life of the dam, sediment will continue to deposit upstream of the reservoir but will not reach the proposed low-level drawdown elevation. Of the 659 acre-feet of sediment accumulation, 593 acre-feet (90% of total sediment) is expected to be submerged sediment and 66 acre-feet (10% of total sediment) is expected to be aerated sediment.

The low-level drawdown conduit is similar to existing conditions, which includes 24" diameter RCP with invert elevation at riser tower of 1045.7.

2.2.4 PRINCIPAL SPILLWAY ENERGY DISSIPATION

The existing St. Anthony Falls (SAF) stilling basin will be replaced with a USACE stilling basin (USACE, 1980). The USACE stilling basin was chosen because it was developed for circular culverts, length is reduced based on baffle blocks, parabolic apron reduces uplift pressures, and this design is common and has performed well across the country. Sheet 10 of the preliminary plan set provided in **Attachment D-4-1** shows the location of the proposed principal spillway centerline, which the dissipation basin will be located at the termination of this line. Typical USACE stilling basin layouts and details are shown in in **Figure D-4-1** and **Figure D-4-2**. Calculations used to develop the dimensions of the stilling basin are provided in **Figure D-4-1**, the calculations are minimum required values which are rounded up to nearest foot increment**.** The design criteria include identical existing pipe slope and outlet invert and apron elevations, with maximum FBH flow rate of 1,331 cfs. The total outlet length is 96 feet, which is summation of fillet length, drop, and basin length. The sidewall flare ratio is chosen to be 9.5, which is greater than minimum require of 6.0, in order to match sequent depth with downstream tailwater level during FBH. However, at lower flow rates the sequent depth is less than downstream tailwater due to auxiliary spillway flows diverted to North Branch Turtle River, which induces drowned jump conditions, see **Figure D-4-2**. Therefore, an inverted V is used for raised center-line trajectory along the parabolic chute; this reduces the risk and likelihood of eddies forming during drowned jump conditions. Baffle blocks are also good for breaking up drowned jumps because the incoming jet plunges to the bottom of the basin where the blocks can keep the jet from carrying along the entire length. An end sill is the terminal feature of the stilling basin. Two rows of baffles are used to reduce basin length and scour downstream of the stilling basin. However, riprap below the stilling basin is still required due to remaining high turbulence, inverse velocity gradients, and large surface waves; riprap gradation requirement is D_{50} = 7 inches.

2.2.5 CONSTRUCTION DIVERSION

The diversion during construction of new principal spillway includes cofferdam and 90-inch diameter High-Density Polythylene (HDPE) or Thermoplastic pipe connecting reservoir to existing conduit. The cofferdam top elevation shall be set to 1060.5, and will protect the new riser tower demolition as well as existing riser tower construction. The HDPE pipe shall connect the lower portion of the reservoir, through cofferdam, and connected to existing conduit.

The construction diversion design event is a 10-year flood event, which peak reservoir inflow is 662 cfs. The design event standard of practice for other agencies rehabilitating dams varies; US Army Corps of Engineers (USACE) minimum frequency is 10-year event, while US Bureau of Reclamation frequency is based on construction period (USBR, 2013), which for this project expected timeline results in design event of ~2-year flood. The diversion is expected to be operational from June-December (6 months), where critical construction period for floods is June-September (4 months). Therefore, the USACE design standard is adopted as it is more conservative and reduces risk to construction progress and efficiencies if a flood event occurs. Furthermore, since construction is not expected to start until June, the high-risk flood months of April and May are eliminated and further reduce flood risk below standard annual frequency values.

The 10-year flood inflow hydrograph is routed through the reservoir below cofferdam crest using HEC-RAS (USACE, 2024); the HDPE or Thermoplastic inside diameter must be 7.37 feet to maintain peak water level at design level (1059.3). A pressure rating of 100 psi is recommended to reduce risk of damage during installation; based on inside diameter and pressure rating requirements, the standard outside diameter is 8.2 feet (2,500 mm). The 6'x6' box culvert has slightly greater capacity than HDPE pipe, therefore does not limit cofferdam design. However, the 90" will need to be connected to existing conduit with watertight concrete poured block connection. The cofferdam crest elevation is based on freeboard of 1.2 feet, which is based on significant wave height calculated from TR-56 (NRCS, 2014 April). The typical cofferdam height is ~13 feet based on pre-dam contour in this area of 1046, plus sedimentation documented in **Appendix D-7**. The maximum cofferdam height is ~20 feet based on existing low-level inlet and minimum pre-dam contour in this area of 1040, plus sedimentation. The riser tower will need to be removed immediately and expeditiously after cofferdam is completed and interior is dewatered with pumps because the HDPE needs to directly connect to existing conduit.

2.3 AUXILIARY SPILLWAY

Improvements to the auxiliary spillway for Larimore Dam are required to allow the spillway to pass the freeboard hydrograph (FBH) without breaching. Parameters determined during the geologic exploration of soils in the auxiliary spillway and application of Probable Maximum Precipitation (PMP) showed that the dam would breach under the existing condition. The preferred alternative involves lining the existing auxiliary spillway with articulated concrete block (ACB) to avoid breaching the spillway during passage of the FBH. Different hardening options for the spillway were analyzed and the ACBs were chosen because they are the most cost effective, as described in Appendix D-3 (Alternatives Evaluation Report). The proposed plan and profile view of the auxiliary spillway is provided on sheets 5 and 6 in the preliminary plan set located in **Attachment D-4-1**. Additional information on the preliminary design of the auxiliary spillway is provided in the following sub-sections.

A detailed two-dimensional HEC-RAS hydraulic model (USACE, 2024) was developed to determine more accuracy of flow dynamics within level section and down the chute. The model geometry, including proposed smoothed terrain and features are shown in **Figure D-4-3**, along with calculated peak flow shear stresses**.** Flows for the FBH were routed through the detailed two-dimensional model for design purposes. A roughness value of 0.035 was assumed for the entire auxiliary spillway as approach section typically has tall grass, see appendix D-3, and chute has high turbulence and gravel present.

2.3.1 APPROACH SECTION

The approach section of the auxiliary spillway will remain the same as the existing conditions auxiliary spillway approach section for the first ~1,000 feet (Station 0 to 10+00), with same invert elevation. The approach section is 300 feet wide, at elevation 1090.5, and is vegetated. The level section was designed and constructed to be elevation 1090.5. However, settlement, erosion, and deposition has made for a not completely level section, which elevations vary slightly from this 1090.5 level. As noted in **Appendix D-1**, the critical elevation of approach section is 1091.0. There are two existing drops downstream of the level section; the first drop is ~10 feet while the second is ~45 feet in height with slope of 15.2%. The project profile reduces the typical slope to 11%; in order to balance the cut volume the level section is extended, which eliminates the first 10-foot drop. Therefore, the new level section is extended from ~1,000-feet long to ~1,600 feet long. The approach section turns ~90-degrees within the level section from stations 5+00 to 8+50. Since there is 750-feet of straight level section following curve before the drop, centrifugal forces from curve have redistributed and equal normal depth reattained.

At the end of the level section armoring starts 50 feet upstream of the drop off to start the chute based on observed shear stresses meeting allowable shear stress at this location. Allowable shear stress is calculated to be 3.3 psf, based on SCS retardance Class D cover and condition. Beginning of the ACBs will include a turndown and backfilled tie into level section to secure the edge, as described in *National Engineering Handbook, Part 628 – Dams, Chapter 54 – Articulated Concrete Block Armored Spillways* (Fripp & Visser, 2019).

2.3.2 CHUTE SECTION

The proposed spillway will have a broad crest that has a 300-foot bottom width and 3:1 (H:V) side slopes. At the auxiliary spillway control section, the crest and side slopes of the spillway would be lined with ACB to just above water surface elevations. Therefore, the auxiliary spillway control section will be protected from any potential erosion at the spillway crest elevation up to the reservoir level, near the top of dam elevation.

The chute downstream of the crest will also have a 300-foot bottom width and 3:1 (H:V) side slopes. There is another ~15-degree curve the chute passes in the middle of the alignment. The channel would have a typical slope of 0.11 feet/feet (11%), however due to 15-degree curve the inside bend has increased slope with maximum value of 0.15 feet/feet (15%) for short local length. The outside of the bend has similarly reduced slope from typical. The size of ACB required was determined using guidance provided in the *National Engineering Handbook, Part 628 – Dams, Chapter 54 – Articulated Concrete Block Armored Spillways* (Fripp & Visser, 2019). Calculations used to ensure the adequacy of the ACB are provided in **Figure D-4-3**. The peak flow values for shear stress (18.7 psf) and velocity (25.6 fps) are found along the max slope (15%) along inside bend, which are shown in **Figure D-4-3.** The ACB sizes were provided by supplier ACF environmental; the individual block size requirements include surface area of 1.34 square feet and 134 lb weight. The resultant Factor of Safety is 2.02, which is above required value of 2.0. According to Chapter 54, Figure 39 (Fripp & Visser, 2019) the base under ACB from bottom up shall include in-place fine grained subsurface, sand, geotextile, gravel, and geogrid. Final design should consider series of cutoff walls (geotextile imbedded though the filter layer into the subgrade) installed across the slope to allow water to pond and slowly drain rather than flow under the ACB system.

The vertical height to which the ACB would line the spillway would transition peak reservoir level near the crest of the spillway to a minimum vertical height of three feet along the steep portion of the channel (see sheet 7 in the preliminary plan set provided in **Attachment D-4-1** for a cross section view of the auxiliary spillway channel). The vertical height of ACB on the side slope of the auxiliary spillway was determined based on review of the maximum depth, which is 2.5 feet; therefore, rounding up to three feet height provides additional factor of safety. Edge extents of ACB where flow is parallel should include termination turndown trench with compacted earthfill to tie into side slopes.

2.3.3 OUTLET

The proposed outlet of chute is continuation of ACB 50 feet beyond end of 11% chute slope. The specific energy at the hydraulic jump is 12.68 feet. The termination basin requires tapered ACB revetment with 3-dimensionally stabilized stone drainage layer due to specific energy greater than 8.03 feet, but less than 17.2 feet (Thornton & Nadeau, 2019). The requirement for 3-dimensionally stabilized stone drainage layer is validated in Chapter 54 (Fripp & Visser, 2019). A 3-dimensionally stabilized stone drainage layer is typically called geocells or cellular confinement systems. Geocells increase strength and stiffness of the infill in order to increase weight bearing capacity and stability. Above geocells remains flat geogrid that is used along remainder of the chute; geogrid openings are called apertures, that allow aggregate to strike though and provide confinement and interlock. This interlocking base layer provides reinforcement as well as filtration. The minimum required length of ACB in termination basin is 31.3 feet; beyond this length modeled shear stresses are also below allowable shear stress of 3.3 psf. Termination basin calculations are summarized in **Figure D-4-3.** Ending of the ACBs will include a turndown and backfilled tie into level section to secure the edge, similar to beginning of ACBs. Final design should consider permeability of compacted earthfill material versus concrete cutoff wall with weep holes to provide suitable outlet for filter and drainage layer.

3 HAZARD CLASS

The process to determine the current hazard classification for Larimore Dam is provided in the *Existing Conditions Assessment Report* for Larimore Dam located in **Appendix D-1**. Since no breach estimation parameters have changed, i.e. height of dam, or volume of reservoir, the exact same process can be referenced to determine the hazard classification for the proposed structural alternative for Larimore Dam.

There are 15 structures, of which 11 are habitable structures, that have a high danger potential and loss of life is likely due to breach. There and five structures in the judgement zone, while the remaining 101 structures have a low danger potential. There are 18 road overtopping locations fall in the high danger category and loss of life due to flooding over these roads is likely. There is potential for the loss of more than a few lives if the dam fails.” Based on the data presented in Appendix D-1, the high hazard designation for Larimore Dam remains.

4 DAM SAFETY REQUIREMENTS

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 March) 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 3, Larimore Dam is classified as a high hazard dam. **Table D-4-1** 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 D-4-1: 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

4.1 PRINCIPAL SPILLWAY DESIGN

Technical Release 210-60 provides the criteria for events that the principal spillway of a high hazard structure must pass, which is called the Principal Spillway Hydrograph (PSH). 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 (NRCS, 2019 May).

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 **Appendix D-1**. 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 D-4-4**.

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 Larimore 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. Model runs were initiated at elevation 1070.0, which are 1.1

and 1.5 feet above the first stage low level inlets. This starting elevation, above the low level inlets, allowed for low flow (27 cfs) in downstream channel for model stability at initial time step.

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. The critical principal spillway mass curve was determined to be Curve B, which auxiliary spillway invert requirement is 1089.8. 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 1089.4. The reservoir elevation after 10-days (1076.7) 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 D-4-5**, and summaries included in **Table D-4-2**.

Table D-4-2: Principal Spillway Inflow Hydrograph Data

	Peak			10-day Drawdown			
PSH	Inflow (cfs)	Reservoir Elevation	Volume (ac-ft)	Reservoir Elevation	Volume Remaining (ac-ft)	Storage Remaining	AS Invert
Volume A	1,268	1087	3,236	1076.7*	735	23%	1089.4
Volume B	1,945	1089.8	4,074	1071.4	236	6%	1089.8*
Volume C	1,775	1088.8	3,754	1071.8	256	7%	1088.8
100yr 10Day	1,441	1086.5	3,105	1071.3	228	7%	1086.5
*Critical (Peak is AS invert, 10-DD is SDH & FBH starting WSEL)							

4.2 AUXILIARY SPILLWAY STABILITY

The capacity of the auxiliary spillway should be adequate to pass the Auxiliary Spillway Hydrograph (ASH) 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 1076.7, from PSH critical results. The adopted ASH duration is 12-hours. The Time of Concentration (T_c) for Larimore Dam 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 Tc>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 D-4-6**; the peak inflow is 2,479 cfs, and peak reservoir level is 1089.9.

The NDDWR standard freeboard is based on 100 mph wind-generated setup and runup (NDDWR, 2024). The NRCS federal requirement is based on 50-year recurrence, which for Larimore Dam site is ~80 mph (NRCS, 2014 April). Therefore, the ND state standard is critical. The effective fetch length at conservative reservoir level 1093.4 feet using 14 radial lines at 6-degree intervals (see **Appendix D-1**). Based on equation 3 (NRCS, 2014 April), the significant wave height is 1.85 feet, resulting in a required normal freeboard of 4.2 feet which is greater than the 3.0 foot minimum freeboard required. Therefore, the auxiliary spillway resulting freeboard of 10.9 feet is greater than required freeboard of 4.2 feet.

Frost heave in excess of the 4-5 feet is not practical, which means that the top of embankment elevation is adequate to pass the auxiliary spillway hydrograph without overtopping due to frost conditions.

4.3 AUXILIARY SPILLWAY CAPACITY

As documented in **Appendix D-1**, the 48-hour Freeboard Hydrograph (FBH) without principal spillway capacity accounted for provides 1.7 feet of residual freeboard. The auxiliary spillway armored with ACB does not change this analysis or results. The FBH and routing is documented in **Figure D-4-7**. In summary, the peak inflow is 15,575 cfs, peak outflow is 14,854 cfs, peak reservoir elevation is 1099.1 and peak reservoir volume is 8,495 ac-ft. The residual freeboard is 1.7 feet, which meets the required value of 0.0 feet by NRCS and 1.0 feet by NDDWR. Therefore, the auxiliary spillway capacity is sufficient to pass the freeboard hydrograph for Larimore Dam regarding NRCS TR-60 (NRCS, 2019) and NDDWR Dam Safety Standards (NDDWR, 2024).

4.4 LOW-LEVEL OUTLET

NDDWR Dam Safety Standards (NDDWR, 2024) require low level outlet with minimum diameter of 12 inches, that can release the top 5 feet of the reservoir volume within five days and evacuate the entire reservoir within one to two months. The existing and new low-level outlet is 24-inch diameter reinforced concrete pipe. An unsteady analysis determined within the first five days the reservoir could be lowered by 12 feet and completely evacuated within two weeks.

4.5 EMBANKMENT

The NDDWR Dam Safety Standards (NDDWR, 2024) for minimum crest width is 23.2 feet (10 feet + 20% (66 feet)). Therefore, the top width of 20 feet will be increased to 24.0 feet.

Preliminary geotechnical investigation and analysis indicates that the embankment soils potentially contain dispersive clay soils. This is a chemical property of the clay particles, that result in deterioration of the clays when they come into contact with water, which can lead to internal erosion of the embankment. While further investigation and more detailed analyses are required to confirm the soils contain dispersive clays, a two-stage chimney and blanket filter system are recommended in FEMA's *Best Practices for Design and Construction, Filters for Embankment Dams* for dispersive soils (FEMA, 2011). Therefore, the existing foundation drain will be grouted in place, and the two-stage filter system will be constructed as close to the center of the embankment as practical so as not to jeopardize the stability of the dam (see sheet 10 in the preliminary plan set shown in **Attachment D-4-1.)**. The gradation control points for the sand filter are designed to conform to NEH Part 633, Chapter 26 Gradation Design of Sand and Gravel Filter (NRCS, 2017), see **Figure D-4-8**. There are three lower and four upper control points calculated using corrected average base soil, with assumption that it is dispersive. ASTM C33 Sand envelope falls between control points, which makes it a standard product commonly available. The gravel drain gradation can meet any of the ASTM D448 blends (FEMA, 2011), also see **Figure D-4-8**. The potential seepage is collected via toe drain pipe and outlets at the proposed stilling basin.

The drain and filter widths are 3-feet each, based on seepage modeling required capacity for the site conditions and minimum width based on guidance from Montana Dam Safety guidance (MT DNRC, 2010). Cover above the drain at top of chimney and above toe drain is set at 5-feet based on stability modeling. Vibrating wire piezometers installed in 2021 were read to establish the phreatic surface in the dam, developed by HDR in the Geotechnical Engineering Report (**Appendix D-2**). These vibrating wire piezometers should be read throughout final design to confirm the phreatic surface and ensure the proposed filter system is placed appropriately to intercept any potential seepage.

Final geotechnical investigation and analysis will be required to confirm the soils contain dispersive clays, including pinholes and double hydrometer analyses. Sufficient borings, recommend 4 or 5, along embankment should be completed to determine if two zones are distinguishable or one zone is homogenous. Vibrating wire piezometer data from the installation in 2021 should be assessed to confirm the phreatic surface in the dam to ensure the drain system intercepts the phreatic surface. Additionally, final geotechnical design should ensure the stability berm over the chimney and blanket drain are sufficient against slope failure for the steady state seepage and rapid drawdown conditions, and that there is sufficient soil cover at the toe of the dam to overcome anticipated hydraulic pressures.

5 SYNTHETIC EVENTS AND SITE PERFORMANCE

The calibration of the hydrologic and hydraulic models is discussed *Turtle River Watershed Plan Appendix D-1: Existing Conditions Hydrology and Hydraulics Report* (NRCS, 2023) and summarized in **Appendix D-1**. The Turtle River Watershed Plan focused on Dam Site 10, also called Upper Turtle River Dam 10, or UTR 10. During planning, the project was found not to meet necessary benefit to cost relationship and further planning was halted. Therefore, the Existing Conditions Hydrology and Hydraulics Report, and Economics Report were basis for much of the existing conditions modeling of Larimore Dam modeling.

5.1 HYDROLOGY

Synthetic rainfall events were simulated for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year recurrence intervals in the Turtle River Watershed, including South Branch Turtle River. The same synthetic rainfall events were simulated within Forest River Watershed Drain #12 catchments; this is the area of the Forest River Watershed that is affected by Turtle River breakout flows that occur near Mekinock as shown in **Figure B-1**. Rainfall is assumed across the entire Turtle River and Forest River Drain #12 watersheds because results represents the largest potential benefited area and the HEC-HMS model was calibrated with those inputs.

Rainfall depths were obtained from NOAA Atlas 14, Volume 8, Version 2. Previous planning efforts to complete a Watershed Plan through the Watershed Operations Program included an evaluation of 24-

hour, 4-day, and 10-day duration rainfall events. That analysis showed that the 4-day duration rainfall events were the most critical for the Turtle River Watershed (i.e., the 4-day duration rainfall would cause more damage than the 24-hour or 10-day rainfall events). The average rainfall and runoff depths for subbasins culminating at Larimore dam in the South Branch Turtle River Watershed are shown in **Table D-4-3**.

Runoff depths were computed using the SCS Curve Number method within a HEC-HMS hydrologic model. That 24-hour curve number grid was used for this analysis. 24-hour curve numbers were adjusted to reflect the four-day duration used for this analysis by interpolating between 24- hour curve numbers and 10-day curve numbers. The conversion was completed using Table 21-2 in Chapter 21 of Part 630 within the National Engineering Handbook (NRCS, 2019 May). Subbasin flows were developed using a Clark Unit Hydrograph transform, which involves the use of the time of concentration and a storage coefficient. Reach routing parameters were developed with Muskingum-Cunge or Lag routing methods. Those parameters were calibrated by simulating historic rainfall events.

Table D-4-3: South Branch Turtle River Watershed 4-day Rainfall and Runoff Depths

Event	NOAA Atlas 14 4-Day Rainfall Depth (inches)	4-Day Runoff Depth (inches)
2-year	2.93	0.43
5-year	3.65	0.68
10-year	4.31	0.94
25-year	5.34	1
50-year	6.21	1.95
100-year	7.15	2.58
500-year	9.66	4.44

5.2 HYDRAULICS AND RESULTING INUNDATION

Flows developed within the HEC-HMS hydrologic model were routed through a HEC-RAS version 6.5 hydraulic model. The performance of Larimore Dam during the various recurrence intervals is shown in **Table D-4-4**. **Figure D-4-9** shows the inundation at the reservoir upstream of Larimore Dam during each of the recurrence intervals analyzed. The elevation of the auxiliary spillway for Larimore Dam will be at 1091.0 feet, which matches the existing auxiliary spillway elevation. For the rainfall events simulated, only the 500- year rainfall event would cause the auxiliary spillway to be activated. Similarly, the elevation of the second stage of the principal spillway will stay the same as the existing second stage of the spillway, which is at 1083.2 feet. The second stage of the principal spillway would only be activated during the 100- and 500-year events. Discharge downstream of Larimore Dam is significantly reduced with the proposed alternative in place, in comparison without dam. The peak inflow value listed in **Table D-4-4** would be the discharge at the dam location if there were no dam in place. The percent reduction to the flow is also listed in **Table D-4-4** to quantify the decreased discharge because of the dam.

Table D-4-4: Larimore Dam Site Performance During Synthetic Rainfall Events

Event	NOAA Atlas 14 4-Day Rainfall Depth (inches)	Peak Inflow (cfs)	Peak Flood Pool Elevation (feet)	Storage Volume (ac-ft)	Pool Area (acres)	Peak Principle Spillway Discharge (cfs)	Peak Auxiliary Spillway Discharge (cfs)	Peak flow change due to dam (cfs)
2-year	2.93	276	1073.5	1,046	104	201	0	-27%
5-year	3.65	463	1076.2	1,314	131	273	0	-41%
10-year	4.31	662	1079	1,661	163	331	0	-50%
25-year	5.34	1017	1083.4	2,399	227	417	0	-59%
50-year	6.21	1350	1085.6	2,881	264	728	0	-46%
100-year	7.15	1737	1087.3	3,317	296	1073	0	-38%
500-year	9.66	2860	1092.7	5,154	416	1340	901	-22%

To assess the impact of the proposed alternative, two scenarios were simulated in HEC-RAS; one with the preferred alternative in place, and one with the dam decommissioned and removed from the system. The resulting inundation for the four-day synthetic rainfall events was obtained using the RASMapper application. Inundation grids were extracted for the simulated events. The total inundation in the Turtle River Watershed for the scenario without Larimore Dam, and with the proposed alternative in place, are provided in **Table D-4-5**. Inundation extents for the various synthetic events simulated are provided in **Appendix C**. Benefits include reduced inundation, depth, and durations. Benefits reduce the further away from Larimore dam in downstream direction due to increasing watershed catchment, runoff, and timing. The benefited area is expansive, as shown in **Figure B-1**. However, lower in the Turtle River and Forest River Drain #12 watersheds the decrease in inundation, depth, and duration are very small and don't apply to all frequency events.

6 ESTIMATED PROJECT COST

The engineer's estimated project cost is shown in **Table D-4-5**; the rehabilitation project investment is $14,099,890. Quantities were based on preliminary design of the proposed alternative. Unit prices were estimated based on previous projects completed in the region, estimates from suppliers, or other NRCS dam rehabilitation projects. Unit prices are estimated in 2024 dollars. Costs will likely change due to market conditions, fluctuations in material costs, inflation, and other factors at the time of bidding and construction for the project. Costs for ACB material and installation have increased significantly over the past 12 months and there is uncertainty as to whether the material and installation cost will stabilize prior to construction of the dam rehabilitation.

A preliminary plan set was completed for the proposed alternative and is available in **Attachment D-4-1**.

Table D-4-5: Preliminary Engineer's Opinion of Probable Cost

Categories		No.	Item	Unit	Quantity	Unit Price [1]	Total Price		
Non-Construction	Engineering	1	Final Design	LS	1		$ 608,000	$1,554,000	$ 1,672,359
		2	Construction				$ 946,000		
	Non-Engineering	3	Permitting & Easements				$ 11,359	$ 118,359	
		4	Project Administration				$ 50,000		
		5	Wetland Mitigation	acres	0.95	$ 60,000	$ 57,000		
Construction	Preparations / Surfaces	6	Mobilization	LS	1	600000	$ 600,000	$ 724,809	$12,427,531
		7	Stripping and Topsoiling	CY	19,628	$ 5	$ 98,140		
		8	Clearing and Grubbing	AC	2	$ 9,000	$ 13,812		
		9	Erosion Control and Revegetation	AC	11	$ 1,200	$ 12,857		
	Temporary	10	Cofferdam Earthfill	CY	60,000	$ 5	$ 300,000	$1,029,328	
		11	Sheetpile cofferdam	SF	2,000	$ 50	$ 100,000		
		12	90" Thermoplastic pipe	LF	300	$ 1,032	$ 309,657		
		13	Concrete connection to existing conduit	CY	10	$ 300	$ 3,000		
		14	Dewatering	LS	1	$ 100,000	$ 100,000		
		15	Haul Road	LS	1	$ 216,671	$ 216,671		
	Principal Spillway Replacement	16	Removal of Riser Tower	LS	1	$ 25,000	$ 25,000	$1,472,575	
		17	Riser Tower - Structural Concrete	CY	75	$ 1,000	$ 75,000		
		18	Riser Tower Misc (Gate, Thimble, etc.)	LS	1	$ 30,075	$ 30,075		
		19	Removal & Fill of existing stilling basin	LS	1	$ 29,500	$ 29,500		
		20	Excavation - outlet channel	CY	2,500	$ 6	$ 15,000		
		21	Concrete for stilling basin cradle & wingwalls	CY	55	$ 1,000	$ 55,000		
		22	Rip rap - stilling basin	CY	300	$ 100	$ 30,000		
		23	Concrete for existing conduit plugging	CY	510	$ 300	$ 153,000		
		24	Microfiber add in for concrete:	CY	500	$ 140	$ 70,000		
		25	Boring and Jacking operations	LS	1	$ 605,000	$ 605,000		
		26	78" Diameter Prestressed Concrete Pipe	LF	450	$ 800	$ 360,000		
		27	Crane & Concrete Pump Truck + Operators	DAY	5	$ 5,000	$ 25,000		
	Drain Replacement	28	Grout existing drain & new toe drain pipe	LS	1	$ 10,400	$ 10,400	$1,673,231	
		29	Excavation - chimney, blanket and toe drains	CY	46,311	$ 25	$1,157,775		
		30	Sand (Stage 1 Filter)	CY	3,500	$ 20	$ 70,000		
		31	Gravel (Stage 2 Drain)	CY	3,500	$ 25	$ 87,500		
		32	Embankment Fill	CY	12,889	$ 14	$ 180,446		
		33	Excavation waste	CY	33,422	$ 5	$ 167,110		
	Auxiliary Spillway	34	Excavation	CY	76,604	$ 6	$ 459,624	$7,527,588	
		35	Fill	CY	67,197	$ 5	$ 335,985		
		36	Excavation Waste	CY	9,407	$ 5	$ 47,035		
		37	Drainfill gravel	CY	3,835	$ 25	$ 95,876		
		38	Drainage sand	CY	3,835	$ 20	$ 76,701		
		39	Geogrid	SF	207,300	$ 1	$ 207,300		
		40	Geotextile fabric	SY	23,033	$ 2	$ 46,067		
		41	Geocell for Stilling Basin	EA	100	$ 400	$ 40,000		
		42	ACBs (EPEC System 900 OCT)	SF	207,300	$ 30	$6,219,000		

1. 2024 Dollars

7 SUMMARY

The proposed design elements of the structural alternative for Larimore Dam are described in this report and a preliminary construction cost estimate for the alternative is provided. NRCS and North Dakota State guidelines and requirements were followed to develop the dimensions and elevations of the proposed structural alternative. The alternative includes constructing a new riser tower, installing a proposed 78" diameter conduit via bore and jack construction methods, modifying the slope of the auxiliary spillway channel, and lining the auxiliary spillway with ACB. Following completion of the Watershed Plan for the Turtle River Watershed Dam #9, final design of the structural alternative will be completed. The final design of the proposed structural alternative may indicate that minor modifications to the plan are required before construction takes place.

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Assumptions:			**Checks:**		Notes:
Y (Crest:1033, Invert:1022)	11	feet			Basin elev from d2 requirement, AS maintained.
Q (pmf)	1,331	cfs			Routing reservoir PMF through PS and AS in RAS
Initial Pipe Hydaulic Calcs:					
Slope	0.016	ft/ft			Existing
Angle	0.02	rad			Conversion
Angle	0.92	deg			Conversion
Pipe Diameter	6.5	feet			Proposed
Pipe Area	33.2	sq. feet			Geometry
Vsm	40.11	ft/s			Assume full pipe flow, as done in Appendix F (Verify in final design)
F (Froude #)	2.77				Appendix F, F-1
USACE Basin Layout Dimensions & Calculatins					
Lf (fillets length)	9.8	feet			Plate C-41
ΔL (Flare ratio)	9.5	feet			eq. 5-2, adjust to 9.5 from min 6 for less abrupt angle & match sequent depth with TW better
Lt (tangent length)	1.7	feet			Example
Assume x	40.2	feet			2nd order polynomial, so better to guess and check.
y check	(11.0)	feet	yes		eq. 5-3, Validates x assumption
Basin Width	16.7	feet			Appendix F
Y_p (exit portal loss)	4.0	feet			Plate C-3
E (culvert exit)	29.0	feet			Appendix F
d_1 (Start jump depth)	1.57	feet			Appendix F
E_1 (jump start)	29.2	feet	yes		Appendix F
V_1 (jump start)	50.90	ft/s			Appendix F
F_1 (Froude #)	7.16				Appendix F
d_2 (sequent depth)	15.13	feet			eq. 5-4
Apron Elevation	1023.20				Assume same as existing
d_2 (elevation)	1038.33				No adjustment of d_2, most conservative. Also training wall elevation
RAS TW	1038.3	yes	At Max Q	none	If "no" adjust ΔL, or see USBR 1987, page 397
Summary					
Basin Width	16.7	feet			From above
Basin Length	45.4	feet			3.0 * d_2, because baffles included.
Total Outlet Length	95.1	feet			Lf + x + Basin Length
Baffle Dimensions					
1st Row Baffle location	26.5	feet			Plate C-41, 1.75*d_2
2nd Row Baffle location	34.0	feet			Plate C-41, 2.25*d_2
Baffle Height	1.6	feet			Plate C-41
Baffle Top Length	0.8	feet			Plate C-41
Baffle Width & Spacing	1.6	feet			Plate C-41
1st Row Baffle Count	5				Plate C-41
2nd Row Baffle Count	4				Plate C-41
End Sill Height	0.8	feet			Plate C-41

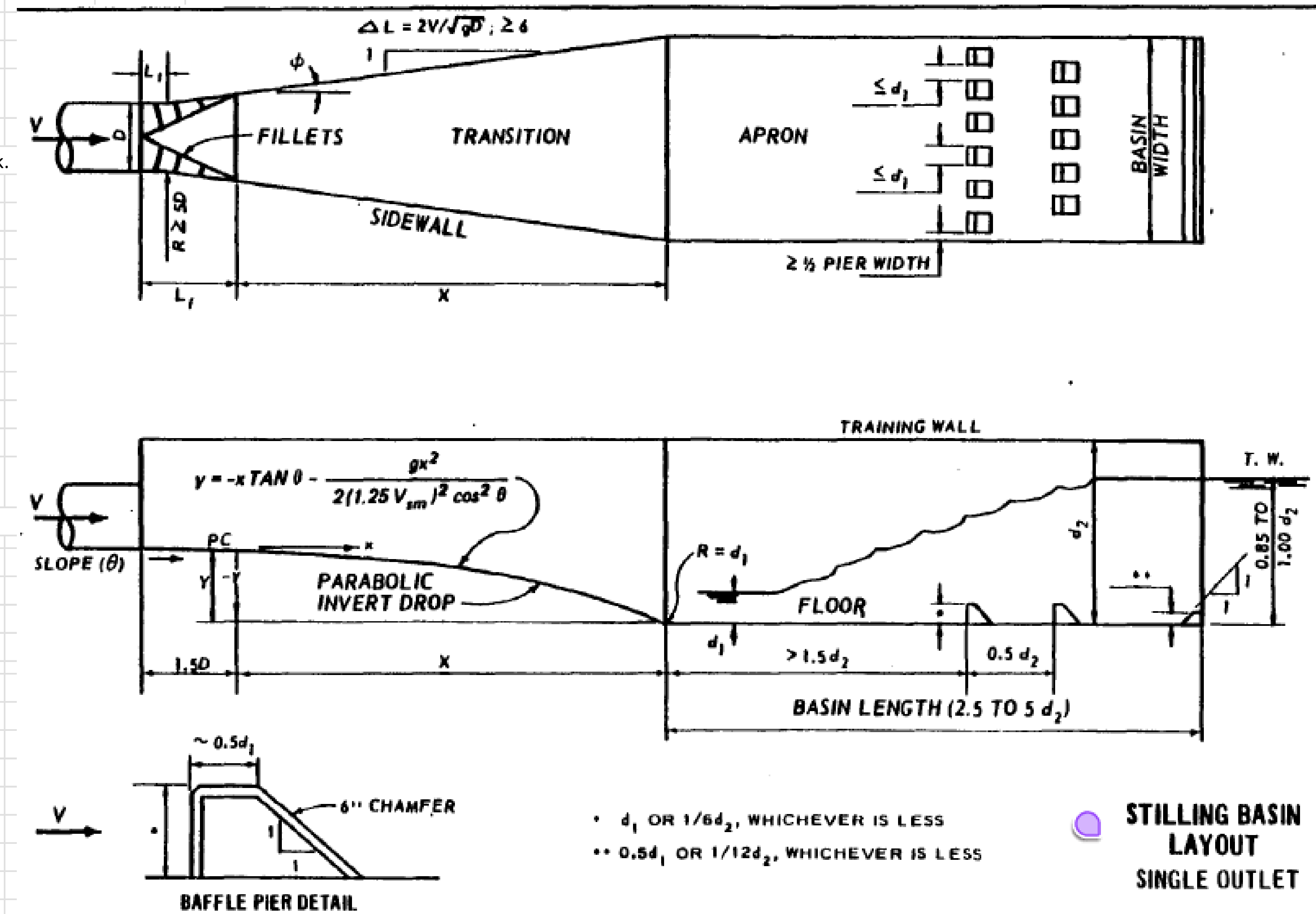
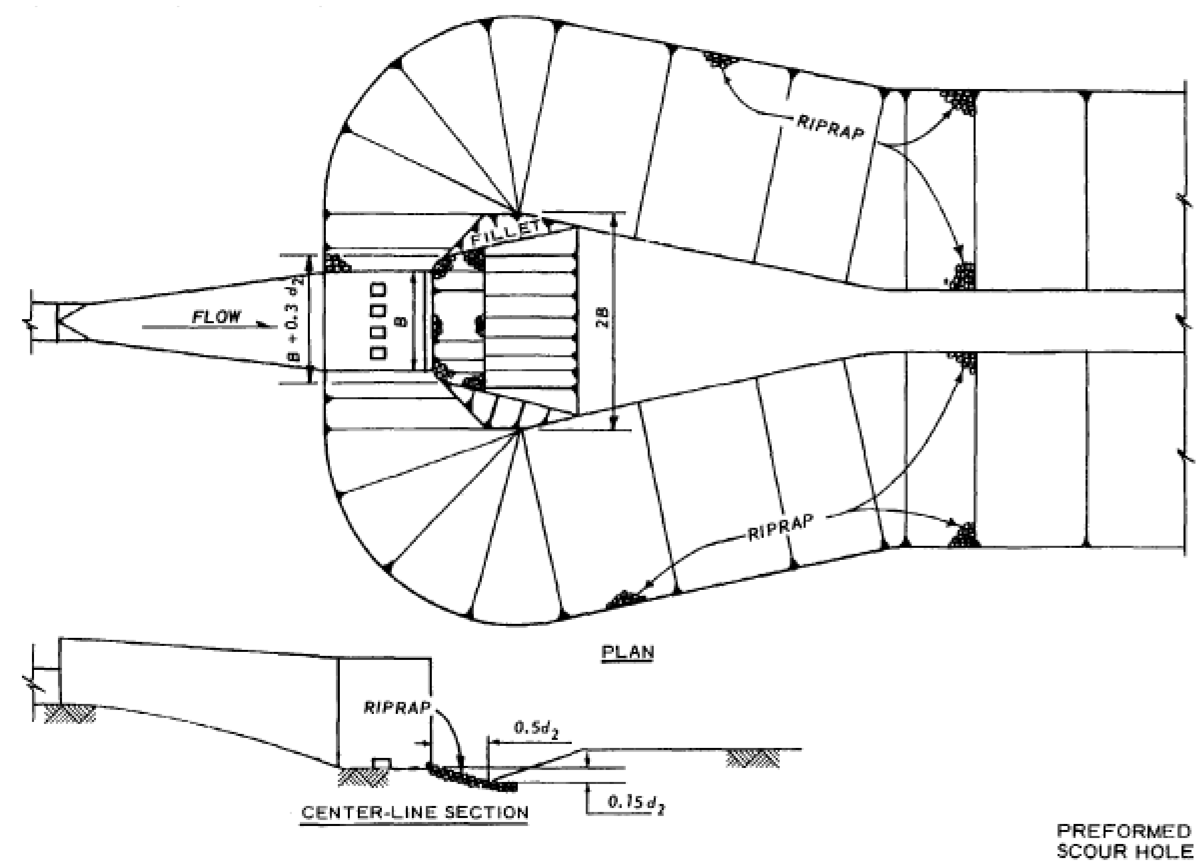
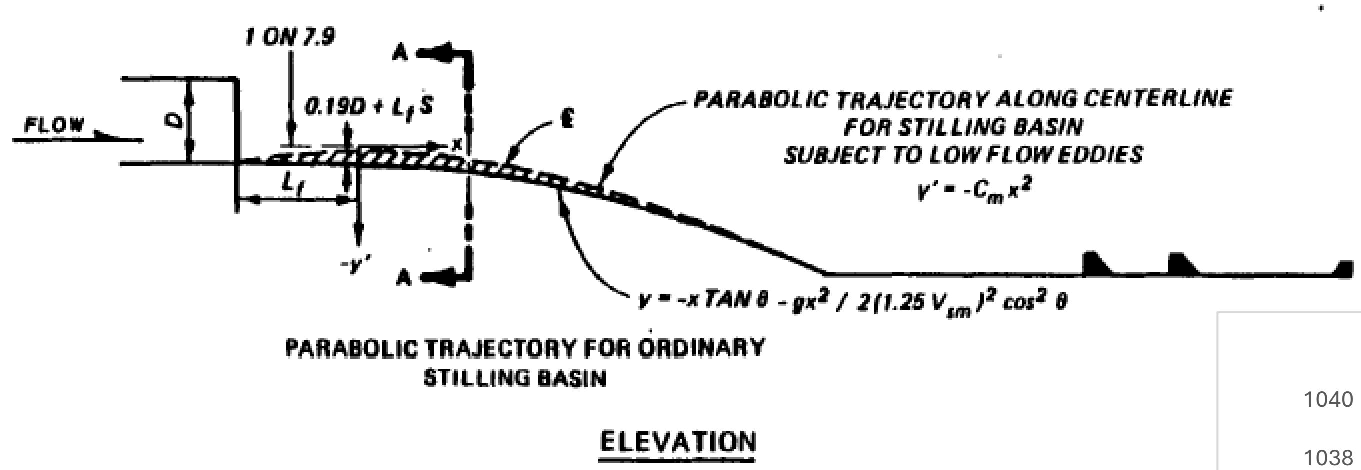
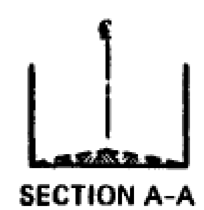


Figure D-4-1: Principal Spillway Energy Dissipation Layout and Dimension Calculations

Trajectory			
C_m (Trajectory Coef)	0.0068		eq. 5-3a
Original Trajectory	$y=-0.016x-0.0064x^2$		Plate F-5
Centerline Trajectory	$y=-0.0068x^2$		Plate F-5
Max Centerline Height	1.24	feet	Plate C-41A
Riprap below Stilling Basin:			HL360 (Use Ishbash or USBR)
Avg. channel velocity	6.53		Max Hydraulic Calcs (Q=V*A) or RAS
D50 (Ishbash)	7	inches	NEH Part 654 TS14C- Fig. 5
D50 (USBR)	0.58	feet	NEH Part 654 TS14C Eq. 9
D50 (USBR)	6.99	inches	Conversion







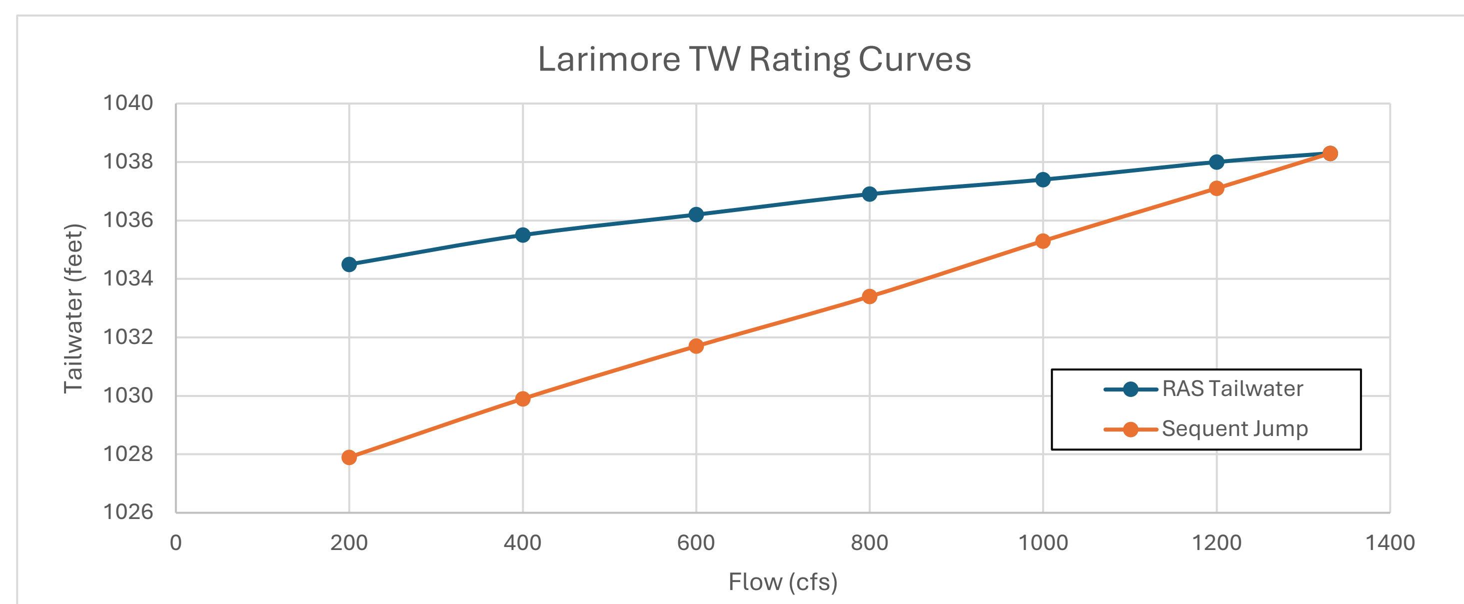
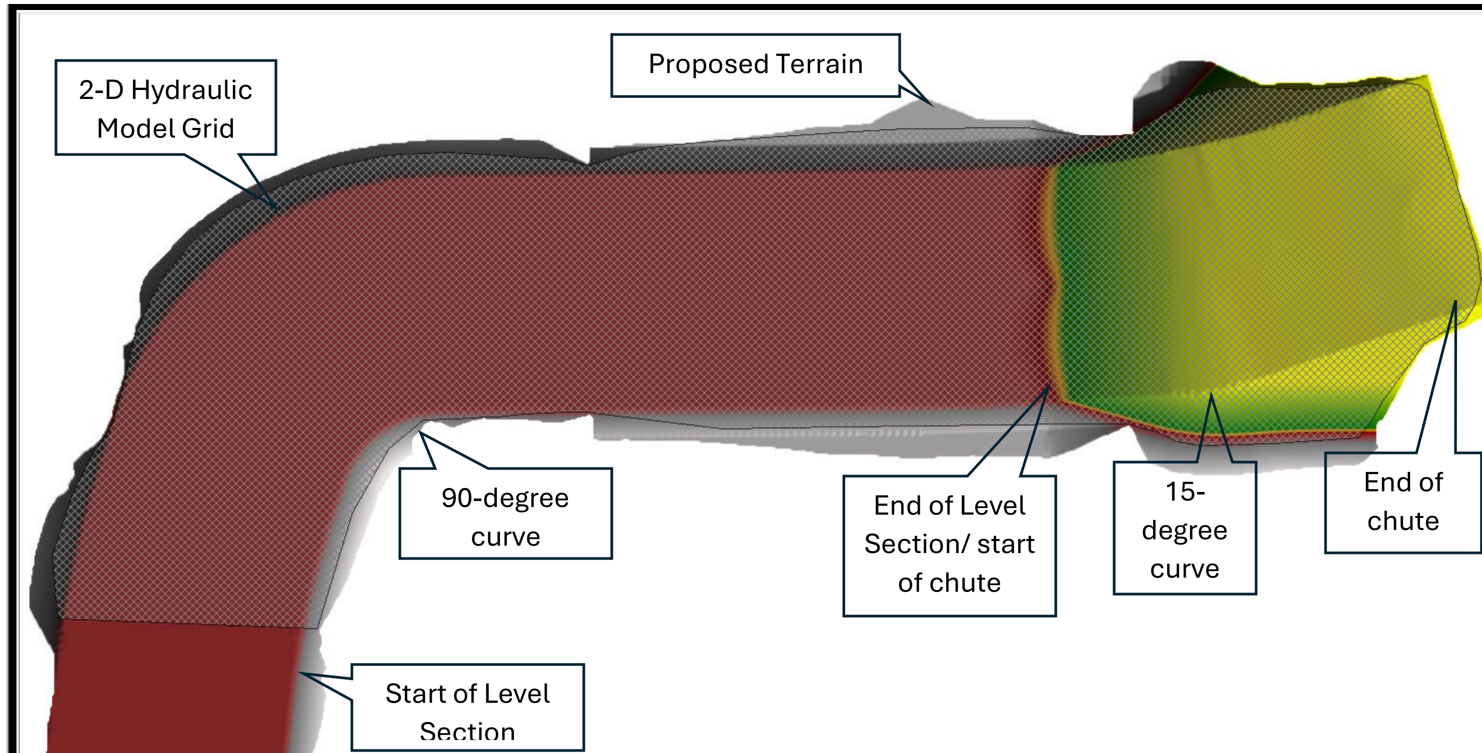
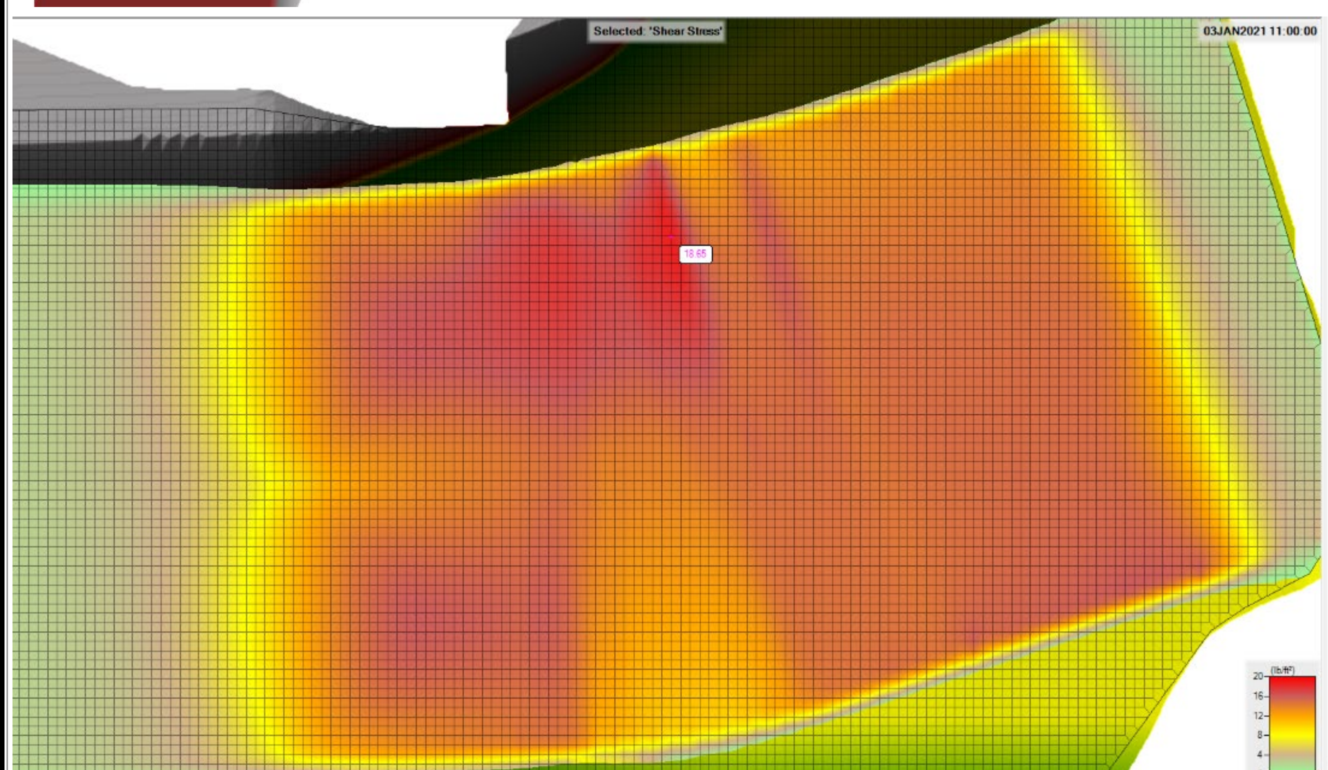


Figure D-4-2: Principal Spillway Energy Dissipation Trajectory and Riprap Calculations

NRCS





				Notes
Constants				
y	weight of water	62.4	lb/ft^3	
p	mass density of water	1.94	slugs/ft^3	
Site conditions				Typical slope is 0.11%, but small (35')
	Bed Slope	0.15	ft/ft or %	section along inside max is 0.15%
	Side Slope Horizontal	3	ft (H:V)	
τ	shear stress	18.7	lb/ft^2	From 2D Calcs, along inside bend at
V	flow velocity	25.6	ft/sec	max slope of 15%
ACB Inputs (Supplier: ACF environmental)				
cl	lift coefficient	0.005		
Aa	block surface area	1.34	ft^2	
l1	block height	0.708	ft	
l2	moment arm for submerged weight fo	0.354	ft	
l3	moment arm for lift force submerged v	0.725	ft	
b	block width normal to flow	1.23	ft	
ΔZ	heigh of block protrusion	0	ft	Assume tapered block
Wb	weight of block	134	lbs	
SC	specific gravity of block	2.2		
Equations (NEH Part 628 Dams, Ch. 54 ACB)				Page
W_{sy}	Submerged weight of block	73.09	lbs	628-54.20
Θ0	Bed slope angle	0.15	radians	
Θ1	Side Slope Angle	0.32	radians	
Θ2	Angle between bed slope and side sl	0.32	radians	
FD	Drag force	25.06	lbs	628-54.19
FL	Lift Force	4.26	lbs	628-54.19
F_D', F_L'	Additional drag force by protrusion	0.00	lbs	628-54.19
WSx	Submerged unit weight of block (Para	10.84	lbs	628-54.20
Wsy	Submerged unit weight of block (Norr	68.65	lbs	628-54.20
FS	**Factor of Safety**	**2.02**		628-54.23
Termination				
d_1	depth	2.50	ft	From RAS Mappr, lower outside
τ	shear stress	17.16		628-54.19
Es	Specific Energy	12.68	ft	628-54.26
d_2	depth	8.69	ft	USBR Engr Mono 25
L	Length	31.3	ft	USBR EM25 Fig 7

Termination Basin: 3D stabilized stone drainage layer details

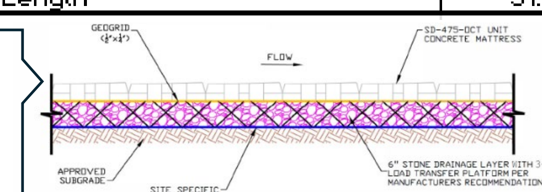


Figure D-4-3: Auxiliary Spillway Modeling and ACB Computations

NRCS

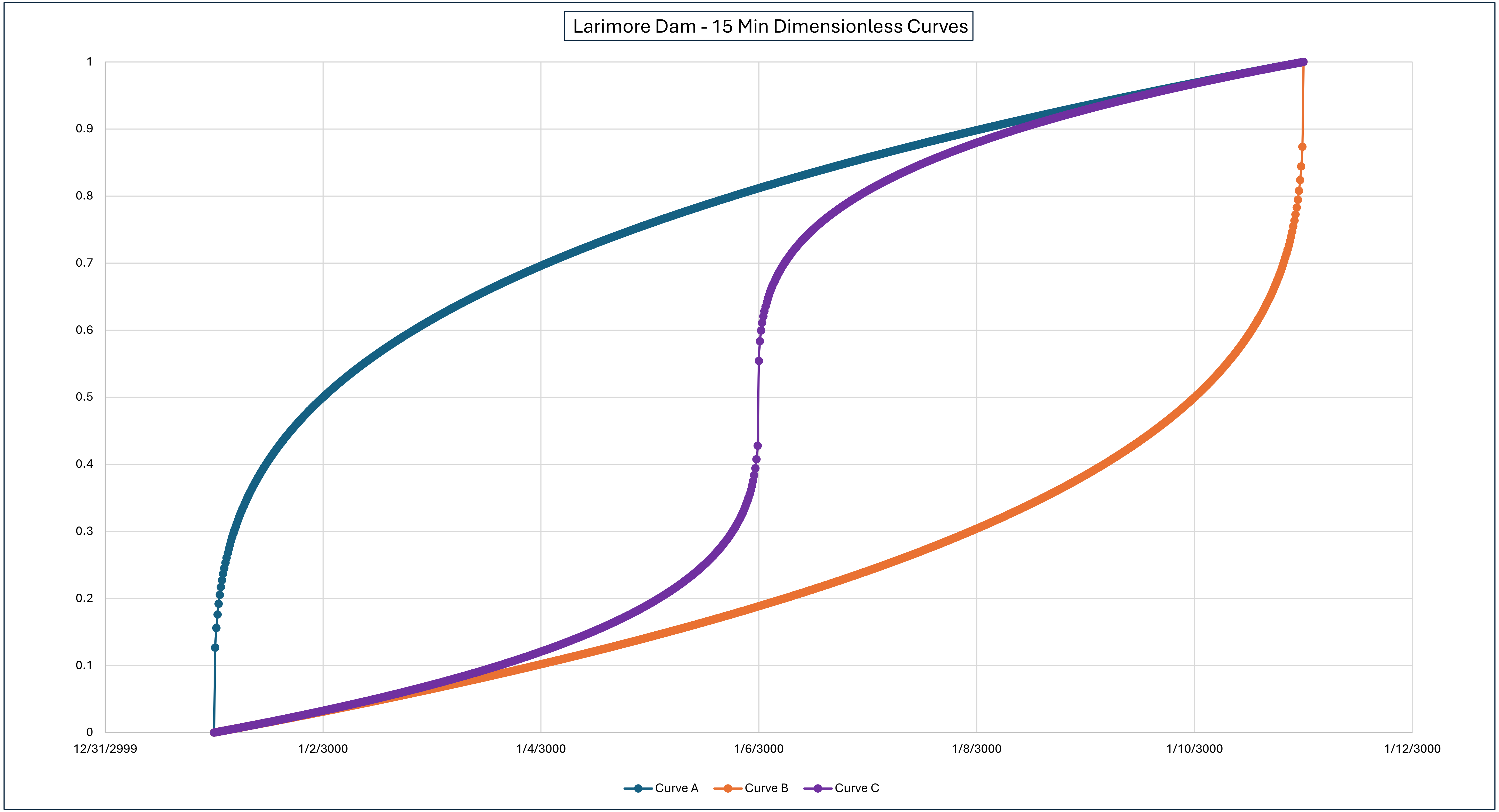


Figure D-4-4: Principal Spillway Mass Curves

NRCS

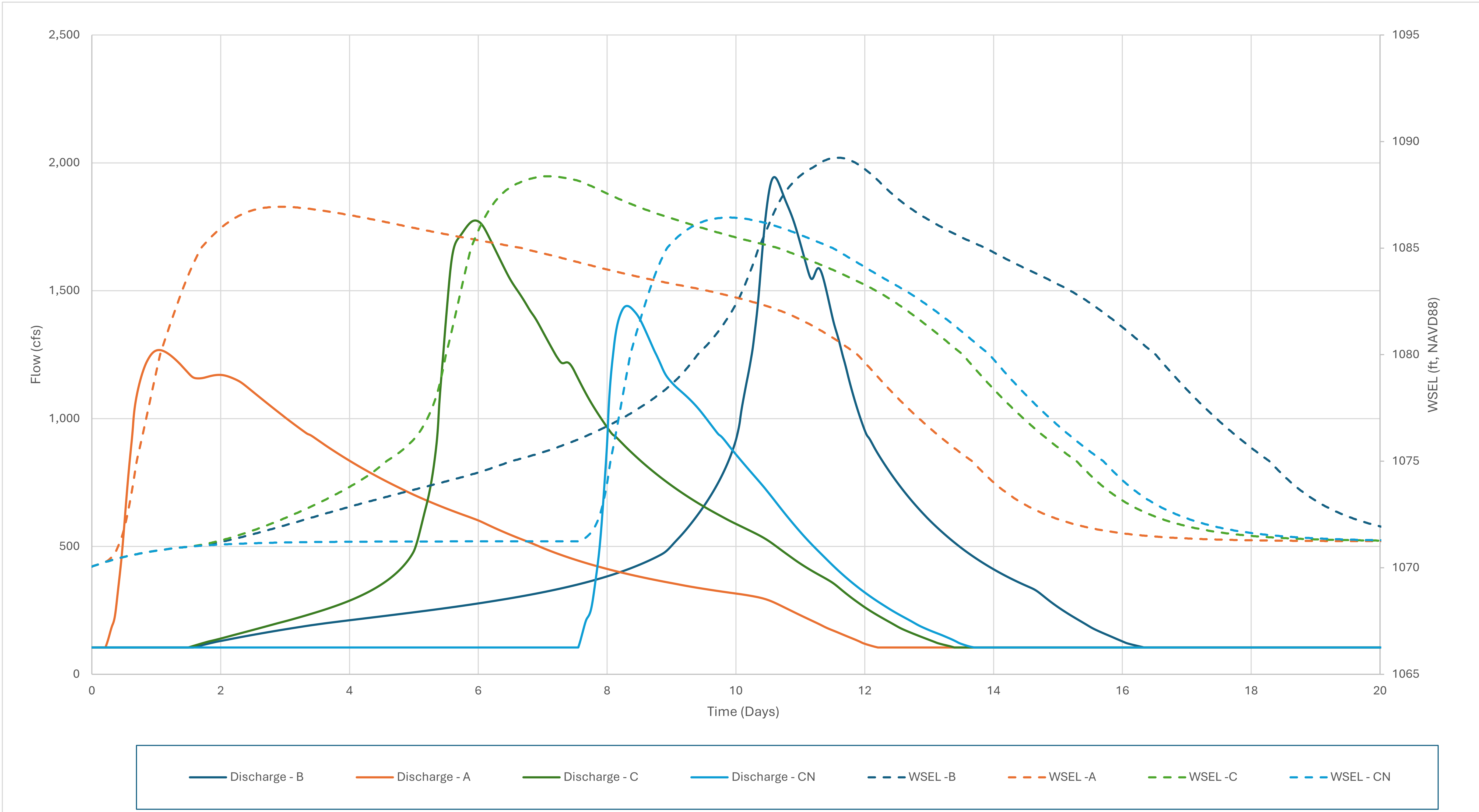


Figure D-4-5: Principal Spillway Inflow Hydrographs

NRCS

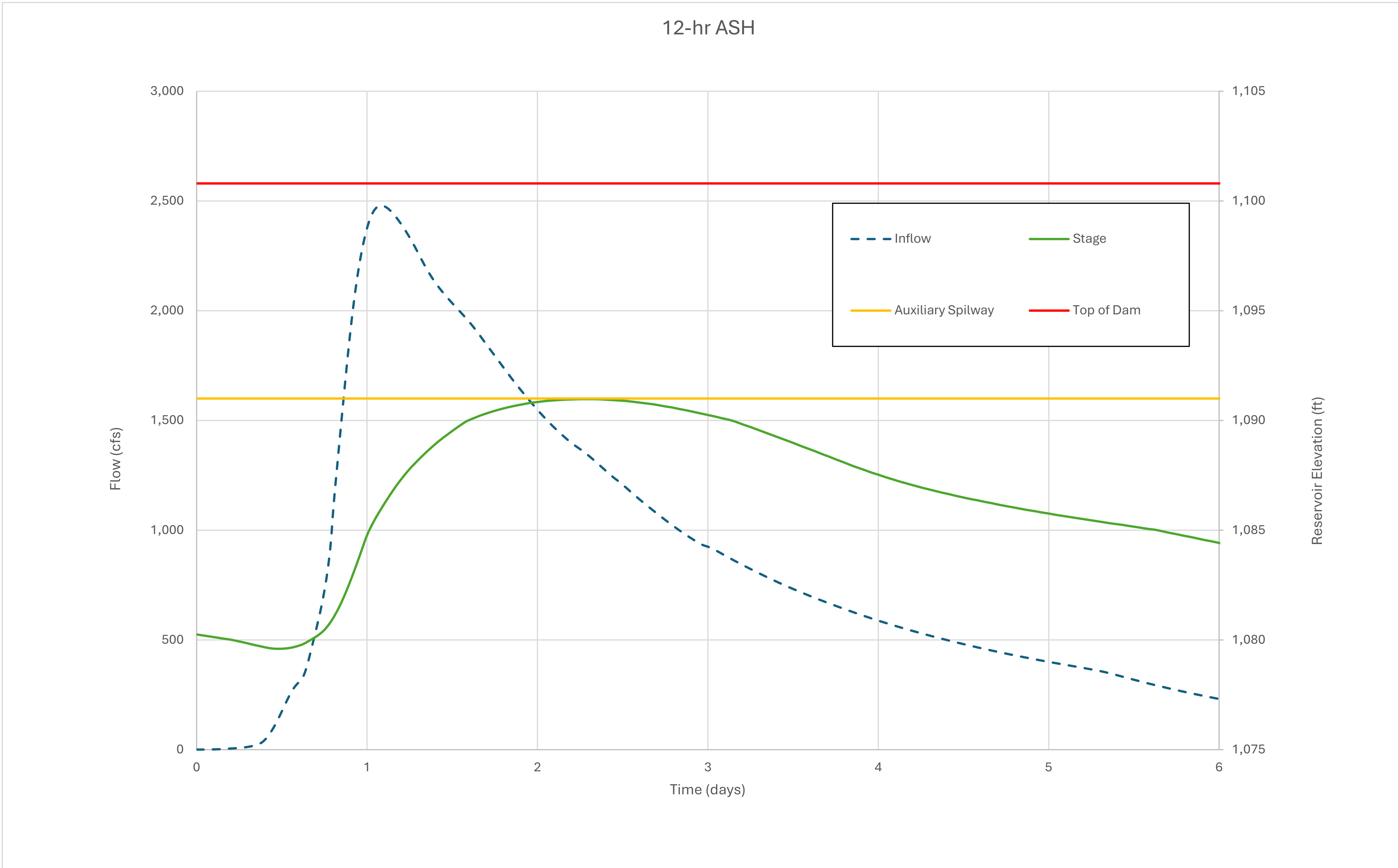


Figure D-4-6: Auxiliary Spillway Hydrograph and Routing

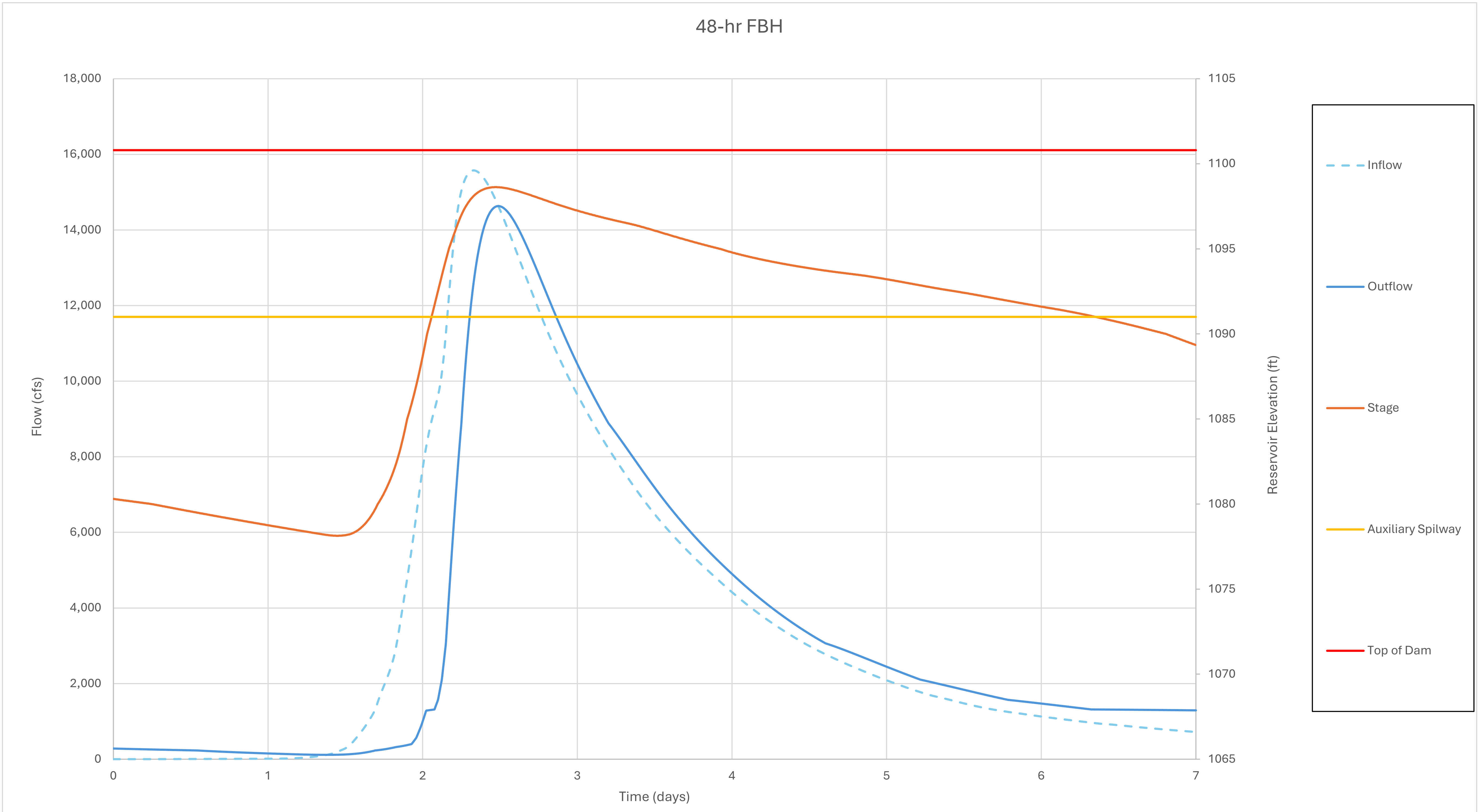
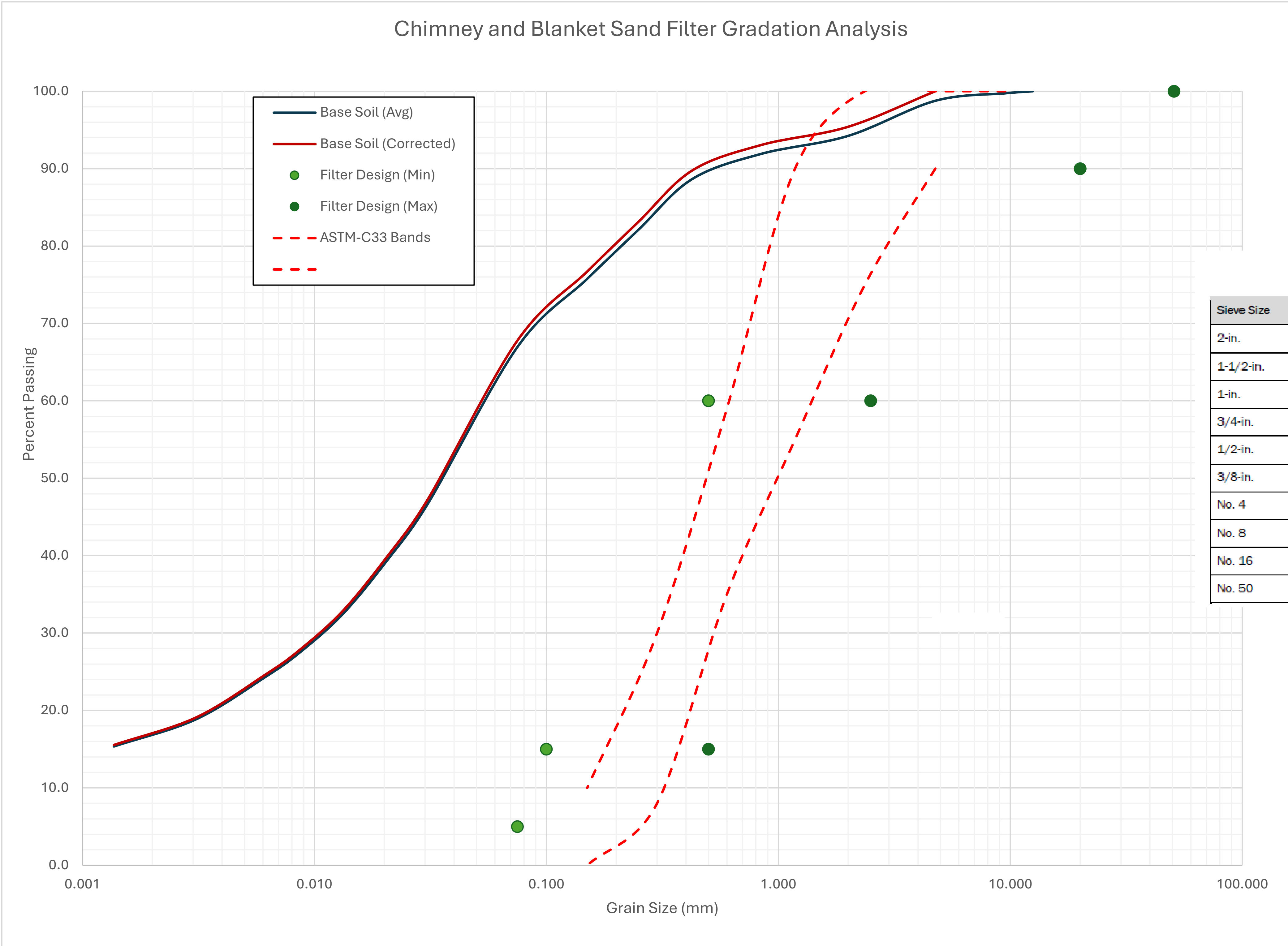


Figure D-4-7: Freeboard Hydrograph and Routing



Chimney and Blanket Gravel Drain Potential Gradations with C33 Sand Filter (FEMA, 2011)

Table 6-4. Gradation for ASTM D448 drain materials (percent passing by weight).

Sieve Size	Blend 5791	No. 8	No. 89
2-in.	–	–	–
1-1/2-in.	100	–	–
1-in.	90-100	–	–
3/4-in.	75-85	–	–
1/2-in.	–	100	100
3/8-in.	45-60	85-100	90-100
No. 4	20-35	10-30	20-55
No. 8	5-15	0-10	5-30
No. 16	0-5	0-5	0-10
No. 50	–	–	0-5

Figure D-4-8: Two-Stage Chimney and Blanket Sand Filter and Gravel Drain Gradations

NRCS

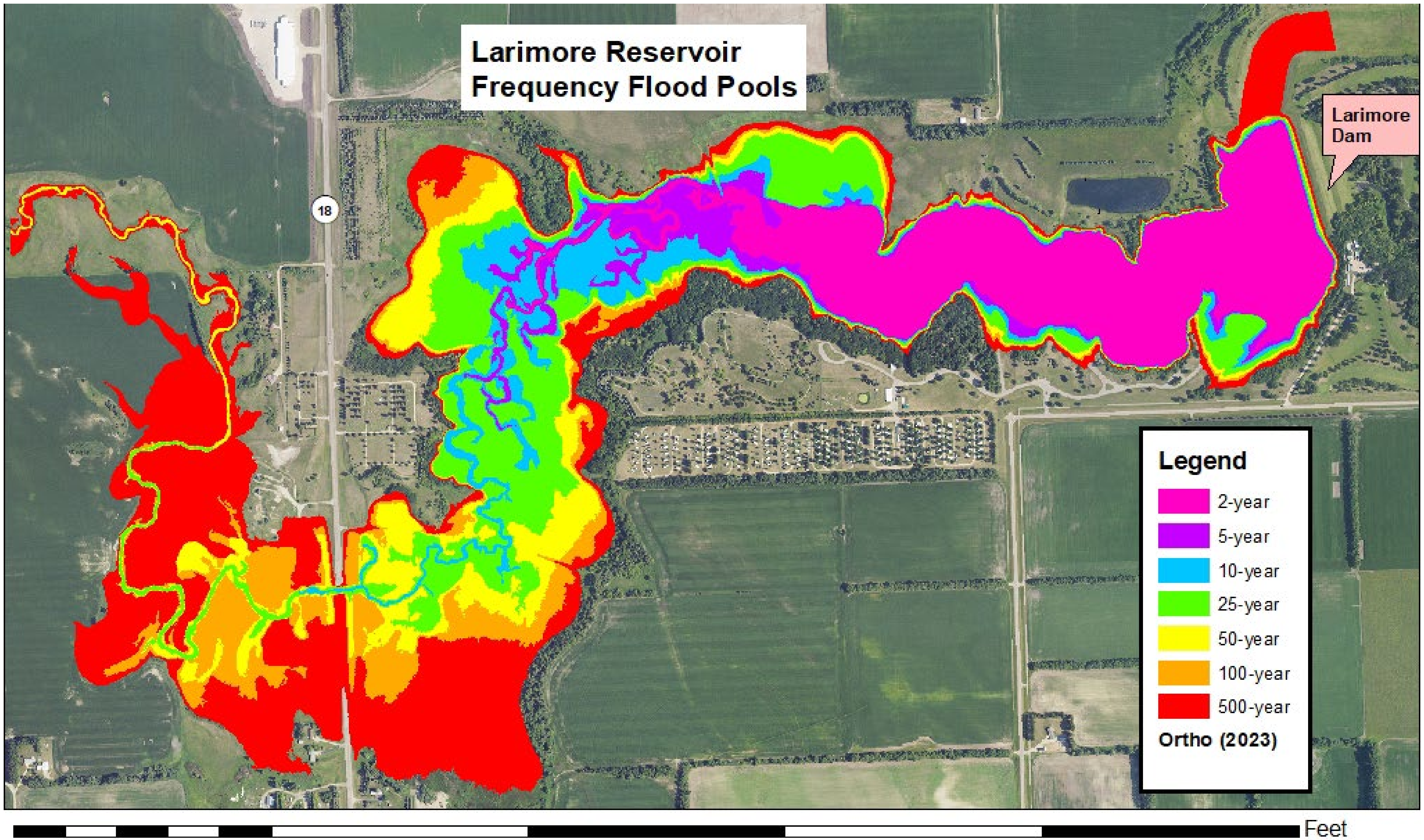


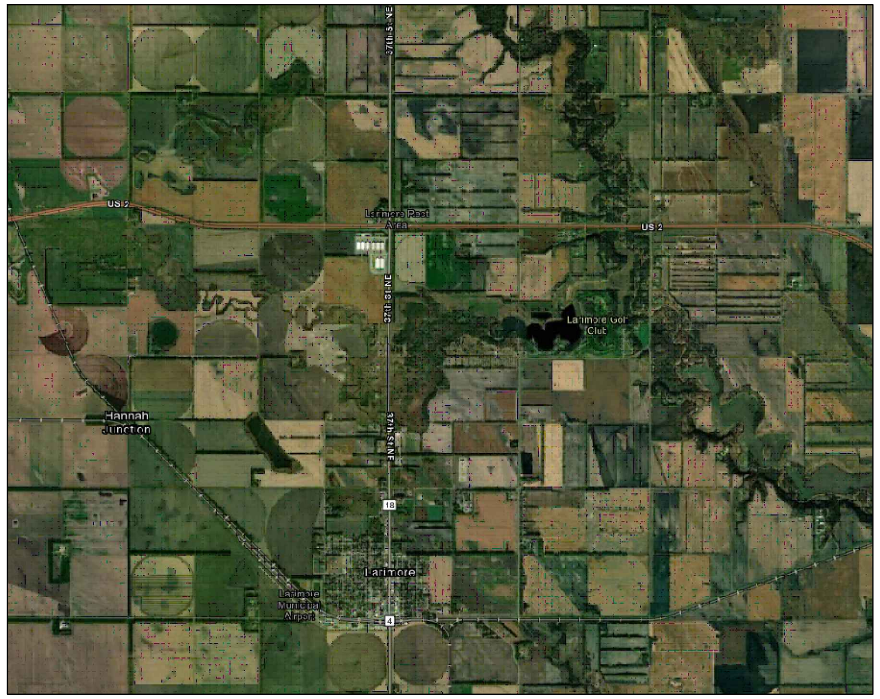
Figure D-4-9: Frequency Flood Larimore Reservoir Inundation Extents

NRCS

ATTACHMENT D-4-1:
PRELIMINARY PLAN SET

30 % DESIGN PLANS FOR THE CONSTRUCTION OF UPPER TURTLE RIVER DAM NO. 9 (LARIMORE DAM) REHABILITATION FOR THE GRAND FORKS COUNTY WATER RESOURCE DISTRICT IN GRAND FORKS COUNTY, NORTH DAKOTA

PREPARED BY
U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE



PROJECT LOCATION MAP
(NOT TO SCALE)

Sec 31 & 32 T 152N R 54W

ESTIMATED PROJECT QUANTITIES		
ITEM	UNIT	EST QTY
Stripping and Topsoiling	CY	19,628
Clearing and Grubbing	AC	1.5
Erosion Control and Revegetation	AC	10.7
Earth Fill	CY	83,948
Earth Cut	CY	131,577
Gravel Surfacing	CY	4,556
Drainage Sand	CY	7,335
Drainfill Gravel	CY	7,335
Rip Rap	CY	300
Grout	CY	81
Removal of Riser Tower and Stilling Basin	LS	1
Reinforced Concrete to plug Existing Conduit	CY	510
Structural Concrete (Riser Tower and Stilling Basin)	CY	130
78" Prestressed Concrete Pipe (Bore and Jack)	LF	450
Low Level 24" RCP	LF	96
Drain Pipe 6" Slotted HDPE	LF	500
Geogrid	SF	207,300
Geotextile Fabric	SY	23,033
Geocell Mats for 3D Stabilization	EA	100
ACBs (EPEC System 900 OCT)	SF	207,300
Cofferdam Earthfill	CY	60,000
Cofferdam 7.5' diameter HDPE pipe	LF	300

JOB APPROVAL AUTHORITY		
PRACTICE NAME	CODE	JOB CLASS
DAM	402	VII
ACCESS ROAD	560	V

GENERAL NOTES:

1. ND Century Code 49-23 and NRCS policy requires that the installer contact the North Dakota One Call Center at 1-800-795-0555 at least 48 hours prior to any excavation work. The Contractor is responsible for the notification of all utility companies affected by the project.

2. The Owner is responsible for acquiring all necessary approvals, permits, and easements required for installation of the project. The Owner and Contractor are responsible for compliance with all ordinances and laws pertaining to installation of the project.

3. Survey work for the project was completed with a Survey Grade GPS with a local site calibration mode with the following settings: UTM NAD83 (Conus MOL), Zone 14N, Geoid Model 18, International Foot for vertical and horizontal datums. Coordinates shown throughout the drawings are in reference to UTM NAD83 Zone 14N. Any property lines, easements, or right-of-ways depicted on the drawings are approximate.

4. North Dakota specifications for all materials and construction work apply, these specifications are part of this plan. Changes in the drawings or specifications must be authorized by the owner and the NRCS representative with the proper authority.

5. All stationing and distances shown on the drawings are measured horizontal distance. All pipeline elevations shown throughout the drawings indicate invert elevations. All quantities are listed as in-place yardage.

6. The project will be staked in the field by NRCS immediately prior to construction. Survey control point markers will be preserved during construction to the extent practical.

7.The Contractor will be responsible for final erosion control measures, as required by regulatory permits for the project and construction specifications.

INDEX OF DRAWINGS

SHT #	TITLE
1	COVERSHEET
2	CONSTRUCTION LAYOUT
3	HAUL ROADS
4	DEWATERING
5	ACB PLAN
6	ACB PROFILE
7	ACB TYPICAL CROSS SECTIONS
8	ACB DETAILS
9	PROPOSED PRINCIPAL SPILLWAY PROFILE
10	PROPOSED CHIMNEY AND BLANKET DRAIN TYPICAL CROSS SECTIONS
11	RISER TOWER DETAILS

NRCS Practice Certification Statement

I certify that these practices have been installed in accordance with these construction drawings and specifications. I approve all modifications made during construction as meeting NRCS Practice Standards, and I have documented those on the as-built drawings and supporting design computations. I have the required Construction Job Approval Authority for all practices, and hereby certify that the installed quantities I have listed are correct based on field measurements.

Construction Approval Signature Date

Designed Kate Staley Date 3/13/25
Drawn Kate Staley 3/13/25
Checked -
Approved-

30% DESIGN UPPER TURTLE RIVER DAM NO. 9 REHAB
COVER SHEET
OWNER: GRAND FORKS COUNTY WRD
COUNTY: GRAND FORKS

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name cover_sheet.dwg
Date 3/13/2025
Sheet 1 of 11

Designed	Kate Staley	3/12/25
Drawn	Kate Staley	3/12/25
Checked		
Approved		

Date

UPPER TURTLE RIVER DAM NO. 9 REHAB
CONSTRUCTION LAYOUT

OWNER: GRAND FORKS COUNTY WRD

COUNTY: GRAND FORKS

USDA United States Department of Agriculture

Natural Resources Conservation Service

File Name Construction_easements_recover.

Date 3/14/2025

Sheet 2 of 11

UPPER TURTLE RIVER DAM NO. 9 REHAB
HAUL ROAD LAYOUT

OWNER: GRAND FORKS COUNTY WRD

COUNTY: GRAND FORKS

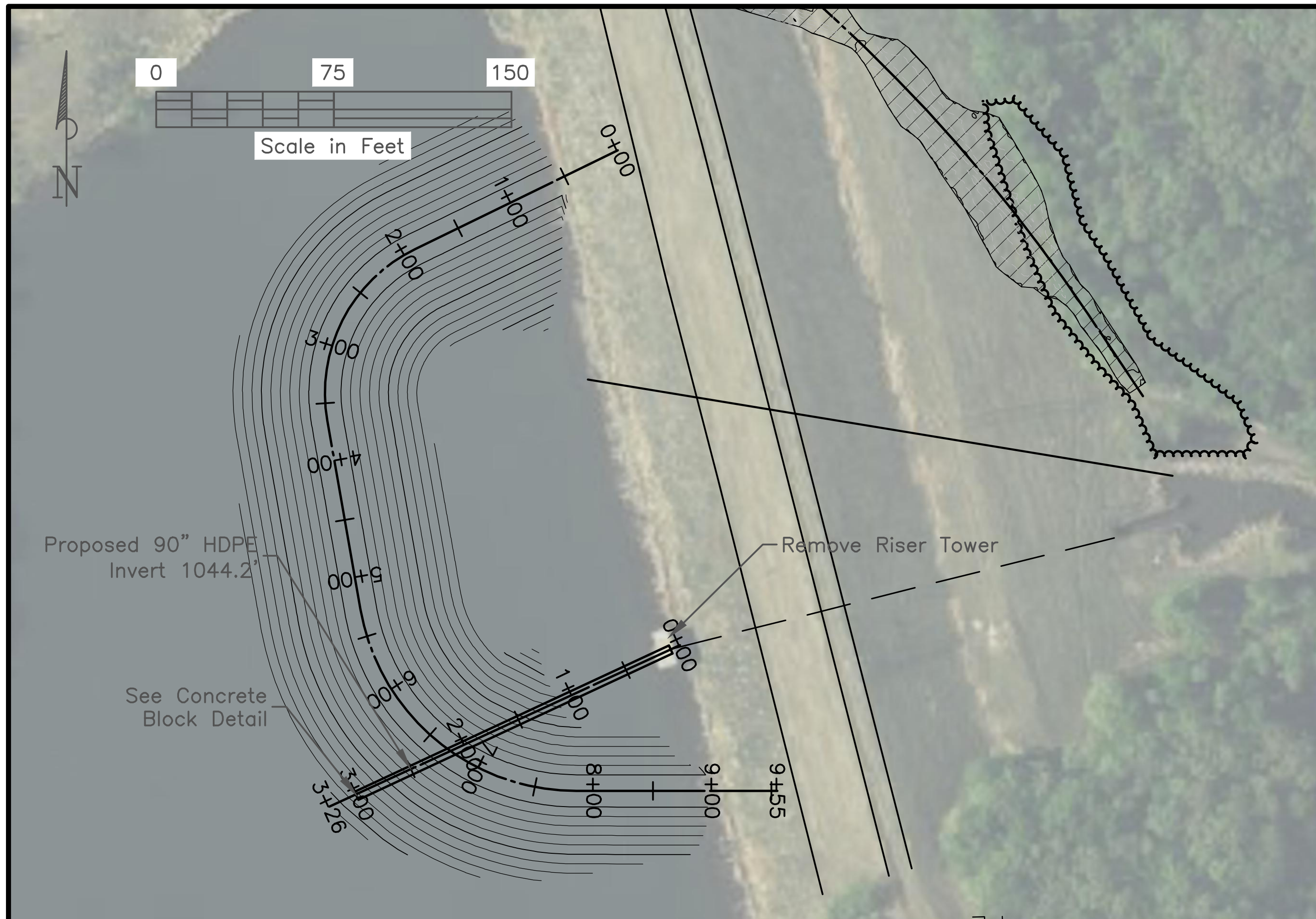
		Date
Designed	Kate Staley	3/12/25
Drawn	Kate Staley	3/12/25
Checked		
Approved		

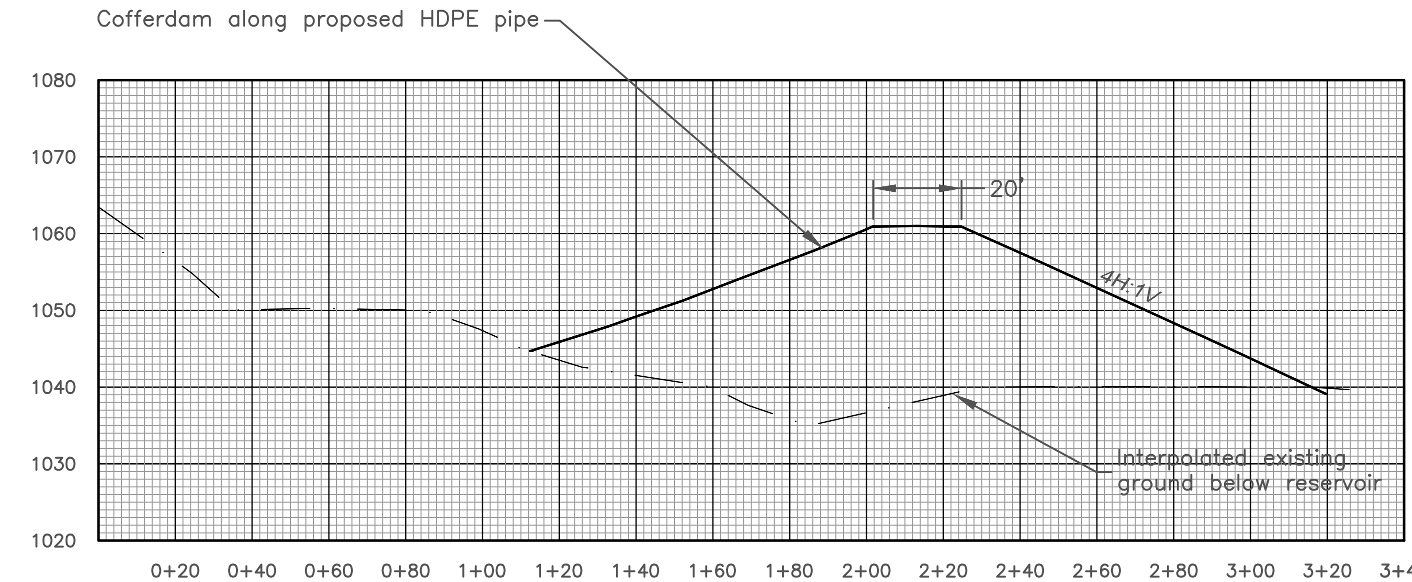
USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name

Date 3/13/2025

Sheet 3 of 11





HDPE PROFILE VIEW
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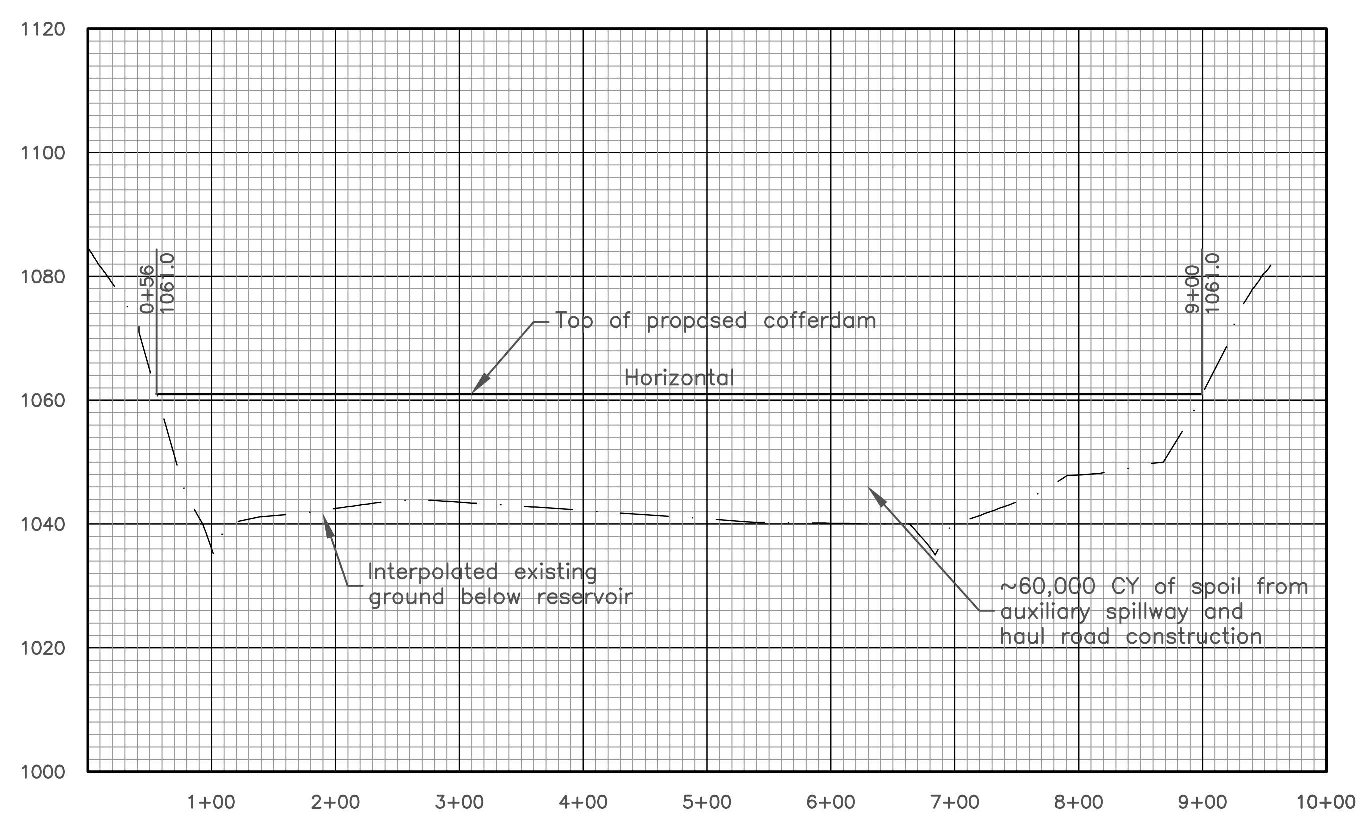
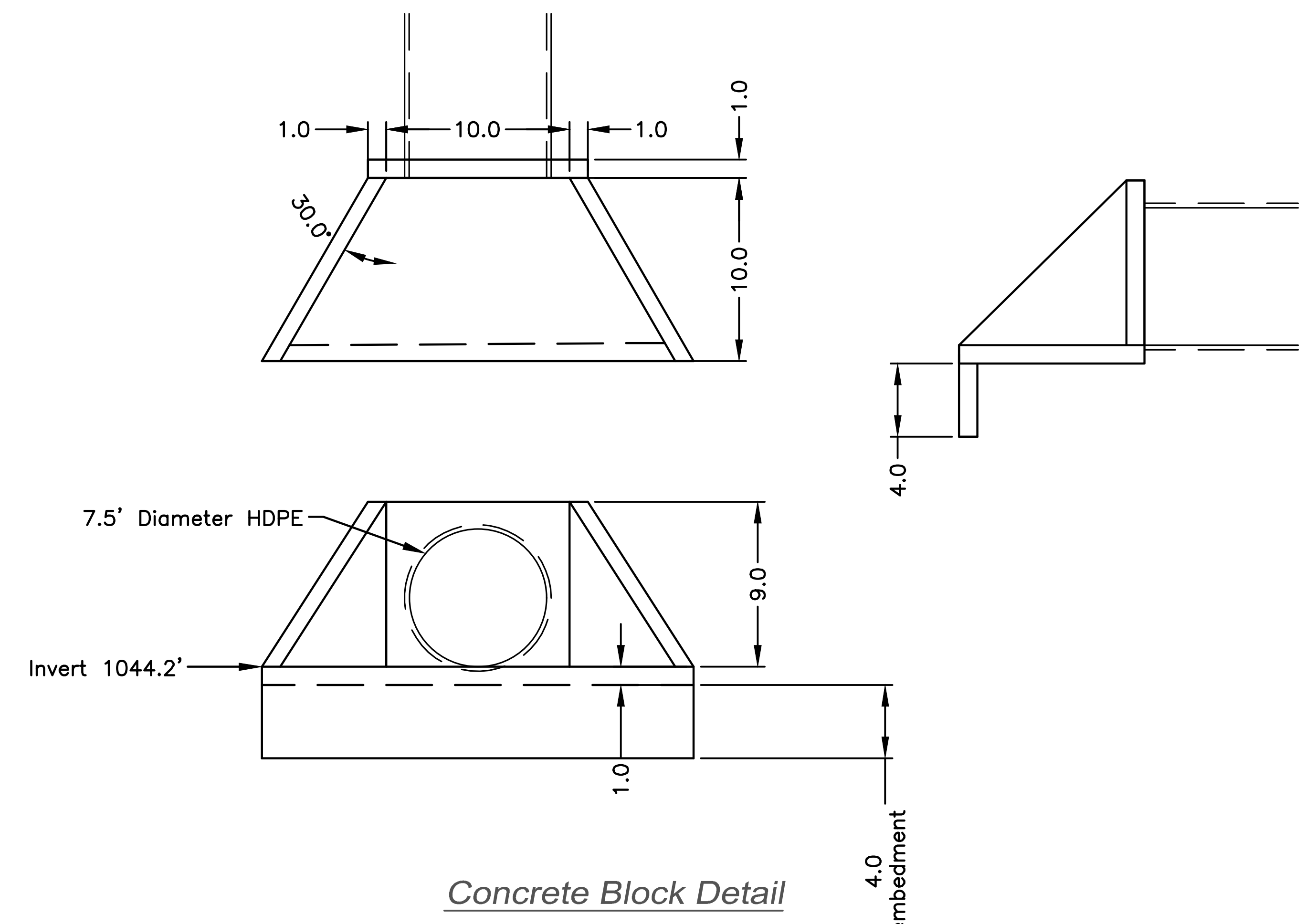


COFFERDAM PROFILE VIEW
SCALE: 5V:1H



Concrete Block Detail

UPPER TURTLE RIVER DAM NO. 9 REHAB
CONSTRUCTION LAYOUT

OWNER: *GRAND FORKS COUNTY WRD*
COUNTY: *GRAND FORKS*

		Date
Designed	Kate Staley	3/12/25
Drawn	Kate Staley	3/12/25
Checked:		
Approved:		

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name

Date 3/13/2025

Sheet 4 of 11

Designed	Kate Staley	Date 3/6/25
Drawn	Kate Staley	3/6/25
Checked		
Approved		

AUXILIARY SPILLWAY PLAN VIEW
UPPER TURTLE RIVER DAM NO. 9 REHAB

OWNER: GRAND FORKS COUNTY WRD

COUNTY: GRAND FORKS

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name ACB_earthwork_smooth.dwg

Date 3/11/2025

Sheet 5 of 11

Notes

1) Begin ACBs Sta 15+25.

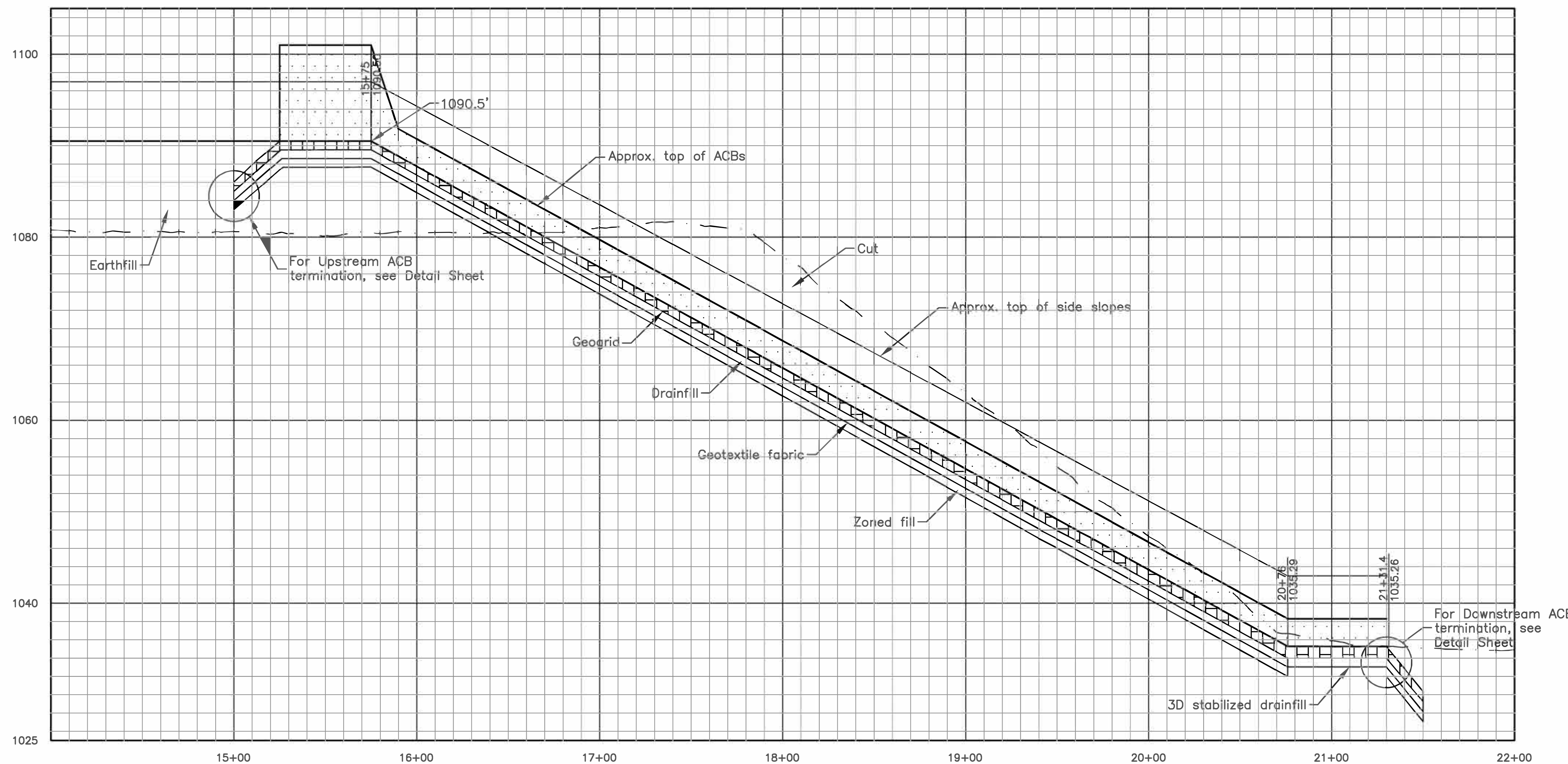
2) Along level control section ACBs match top of dam, 1100.8'.

3) 3' minimum height ACBs along side slopes.

4) Begin 3D stabilized drainfill Sta 20+76.

5) End ACBs Sta 21+31.

6) For terminations and drainfill details, see Details Sheet.



AUXILIARY SPILLWAY CL PROFILE VIEW
SCALE: 5V:1H

Designed	Kate Staley	3/6/25
Drawn	Kate Staley	3/6/25
Checked		
Approved		

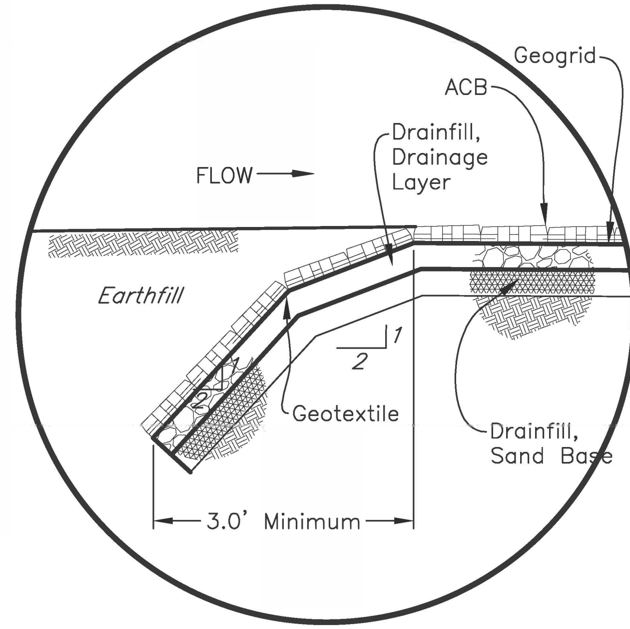
AUXILIARY SPILLWAY PROFILE
UPPER TURTLE RIVER DAM NO. 9 REHAB

OWNER: GRAND FORKS COUNTY WRD
COUNTY: GRAND FORKS

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name
...CB_earthwork_smooth.dwg

Date
3/11/2025



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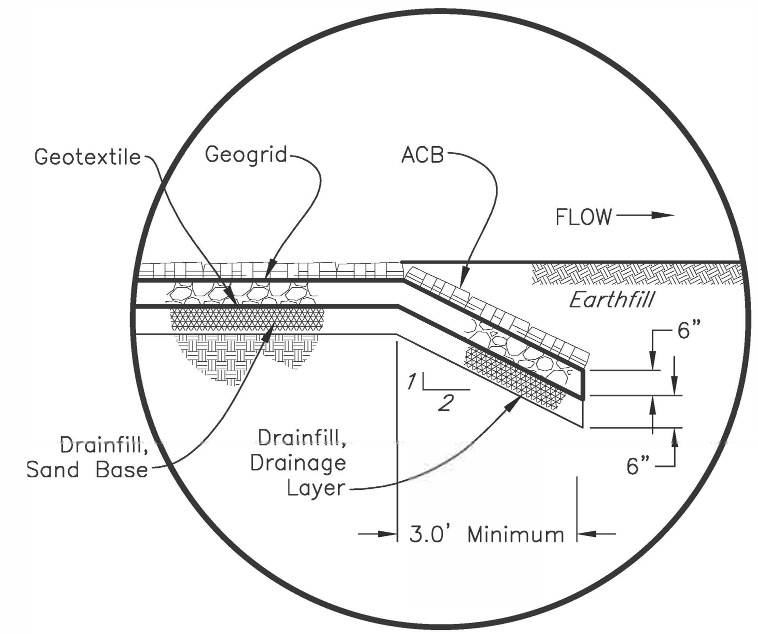

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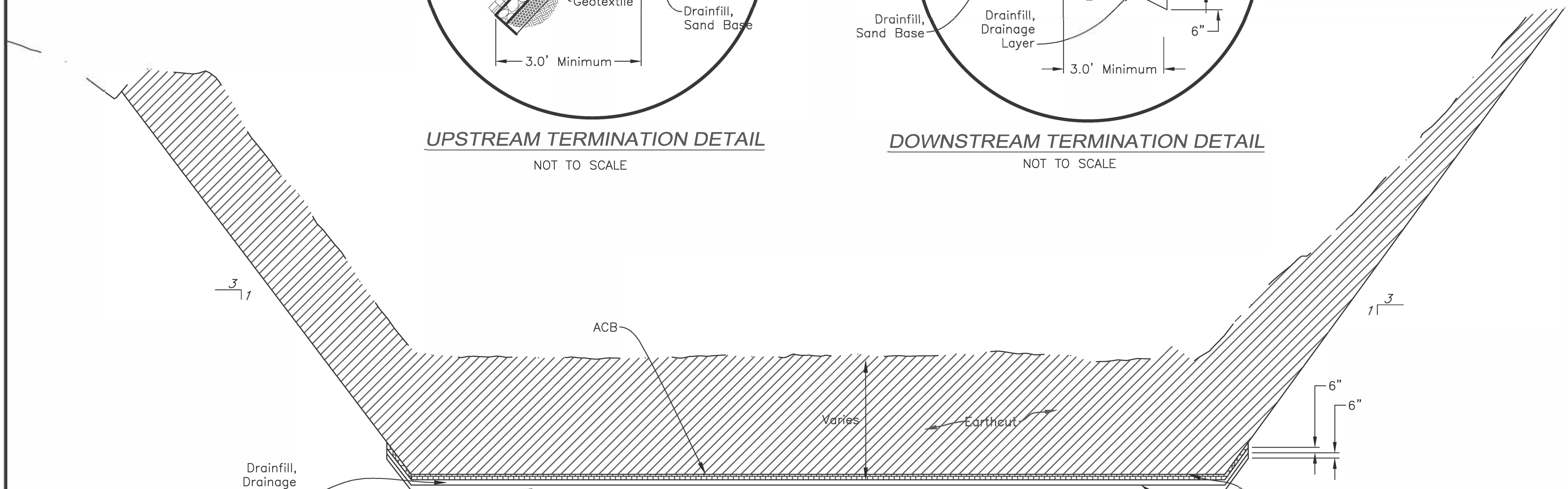

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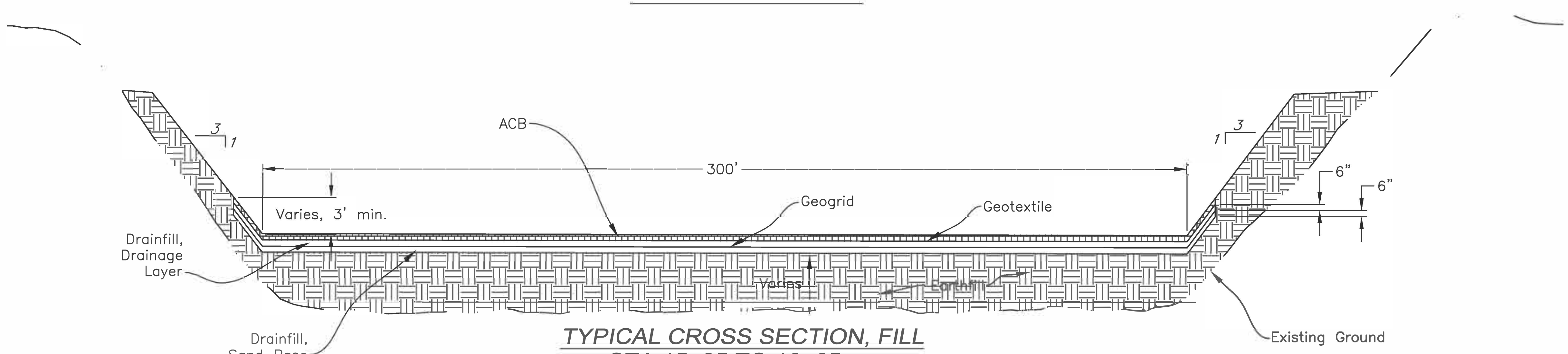
STA 16+65 TO 21+31



TYPICAL CROSS SECTION, FILL
STA 15+25 TO 16+65

Designed	Kate Staley	3/6/25
Drawn	Kate Staley	3/6/25
Checked		
Approved		

ACB TYPICAL CROSS SECTIONS
UPPER TURTLE RIVER DAM NO. 9 REHAB

OWNER: *GRAND FORKS COUNTY WRD*

COUNTY: *GRAND FORKS*

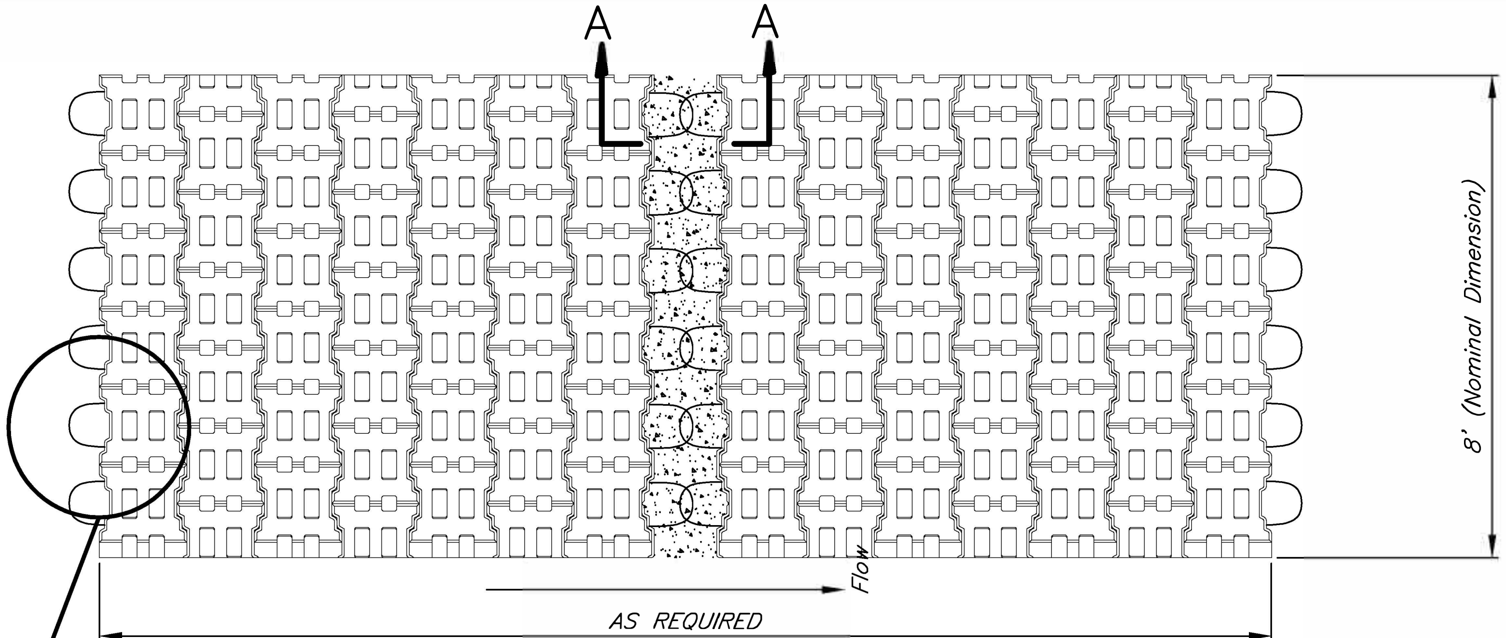
USDA United States Department of Agriculture

Natural Resources Conservation Service

File Name
ACB_earthwork_smooth.dwg

Date
3/13/2025

Sheet 7 of 11



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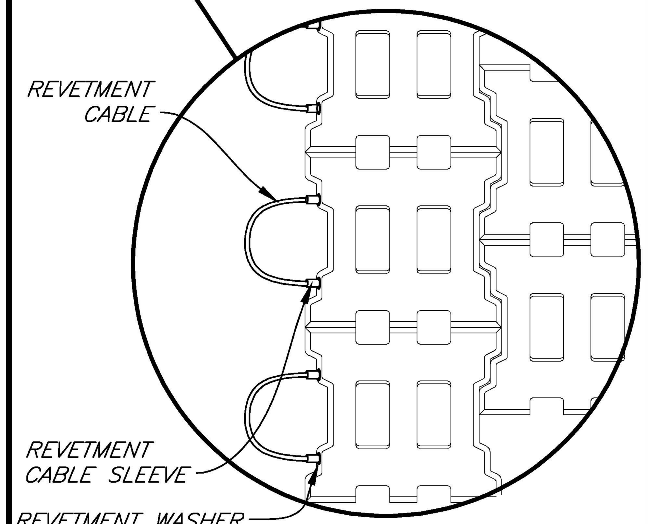

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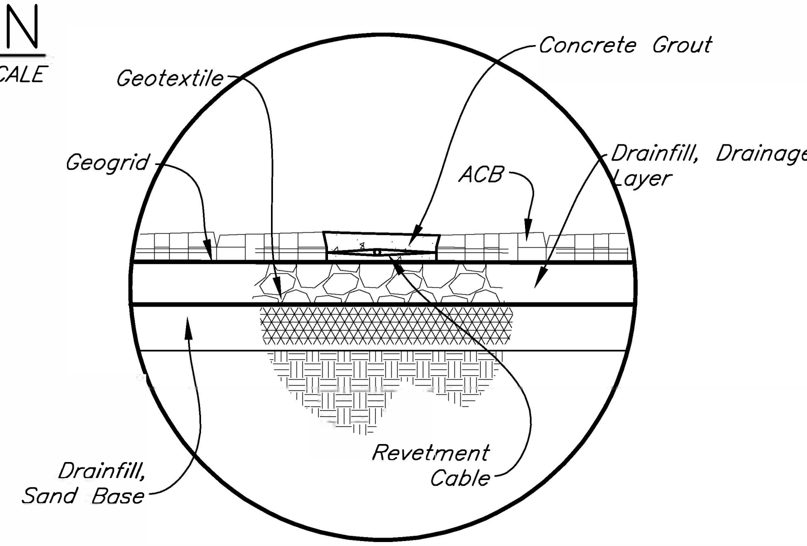
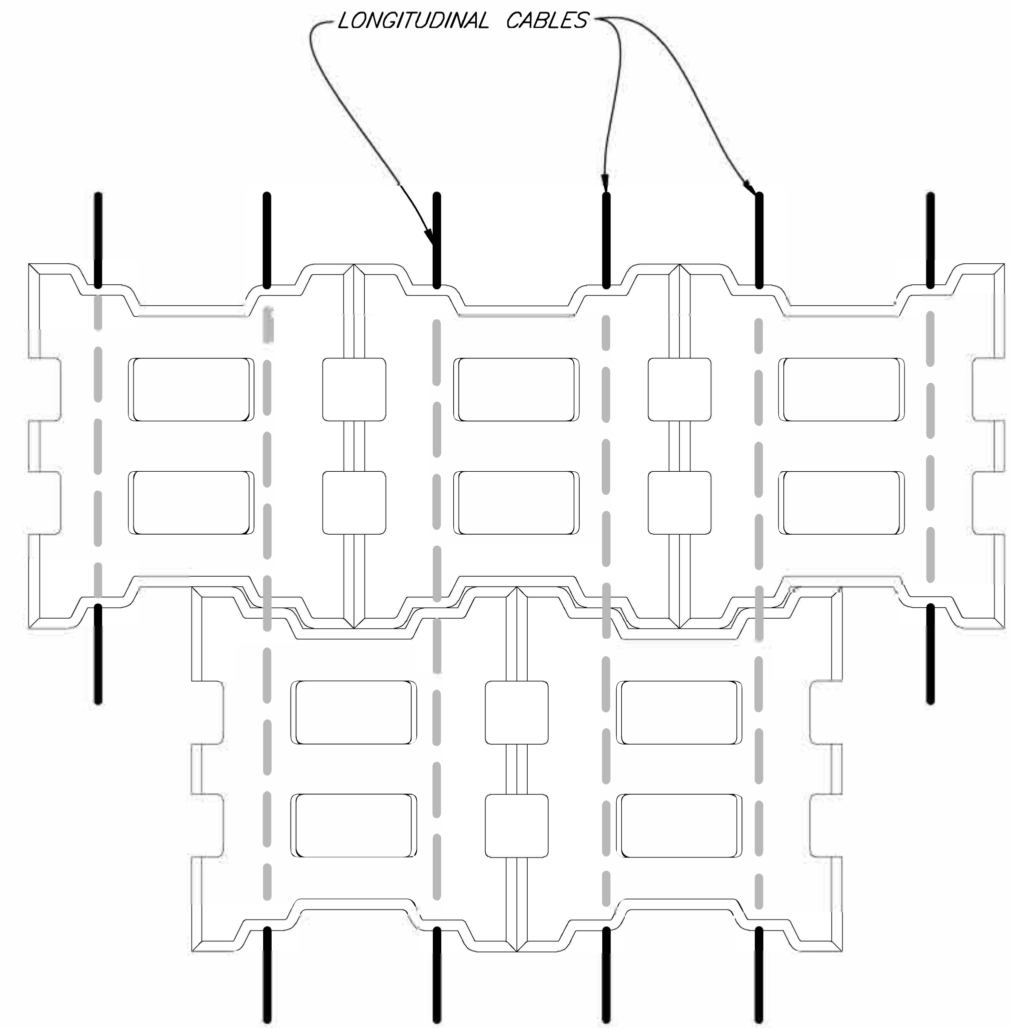


SECTION A–A



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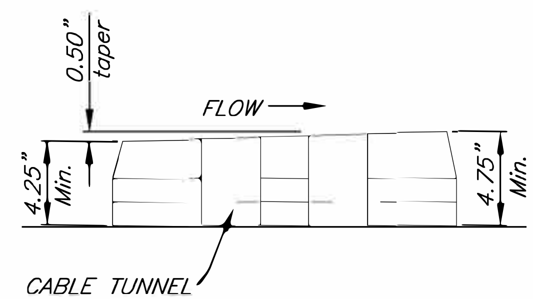

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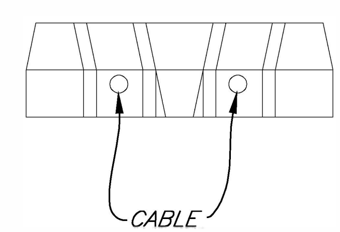


END VIEW
NOT TO SCALE

Designed	Kate Staley	3/6/25
Drawn	Kate Staley	3/6/25
Checked :		
Approved:		

Date

ARTICULATED CONCRETE BLOCK DETAILS
UPPER TURTLE RIVER DAM NO. 9 REHAB

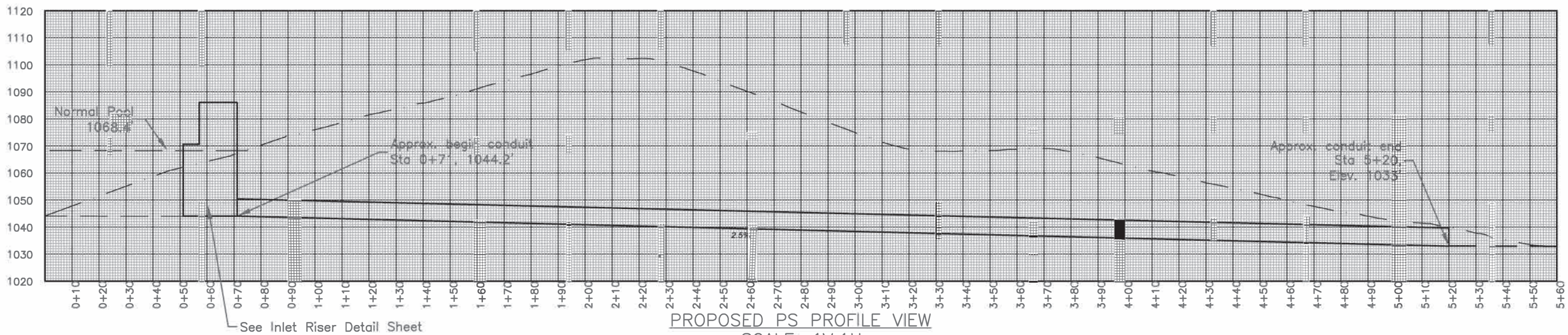
OWNER: GRAND FORKS COUNTY WRD

COUNTY: GRAND FORKS

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name
Articulated Concrete Block Details.dwg

Date
3/13/2025



PROPOSED PS PROFILE VIEW
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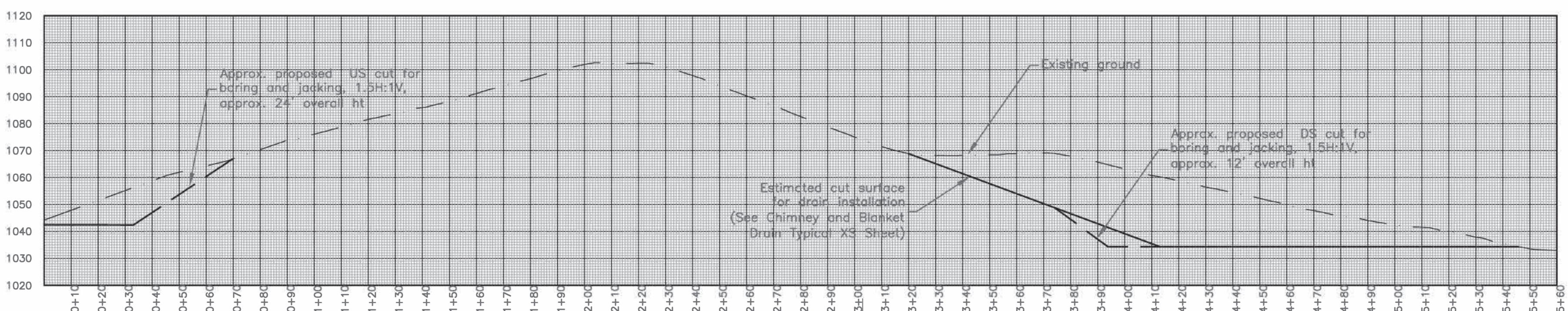
PROPOSED PRINCIPAL SPILLWAY PROFILE



PROPOSED PS PROFILE VIEW
SCALE: 1V:1H

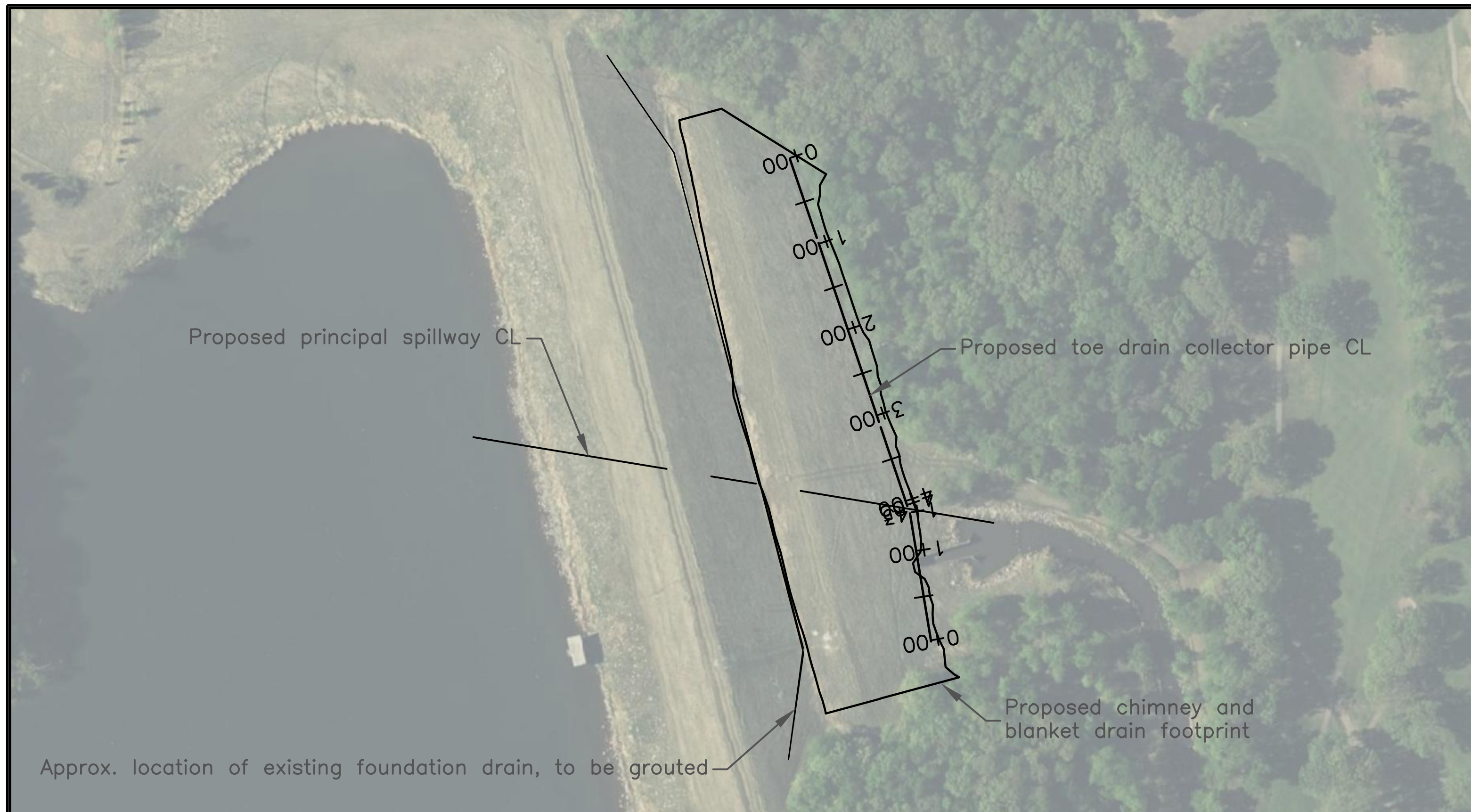
PROPOSED CUTS FOR DRAIN INSTALLATION AND ADDITIONAL CUT FOR BORING AND JACKING

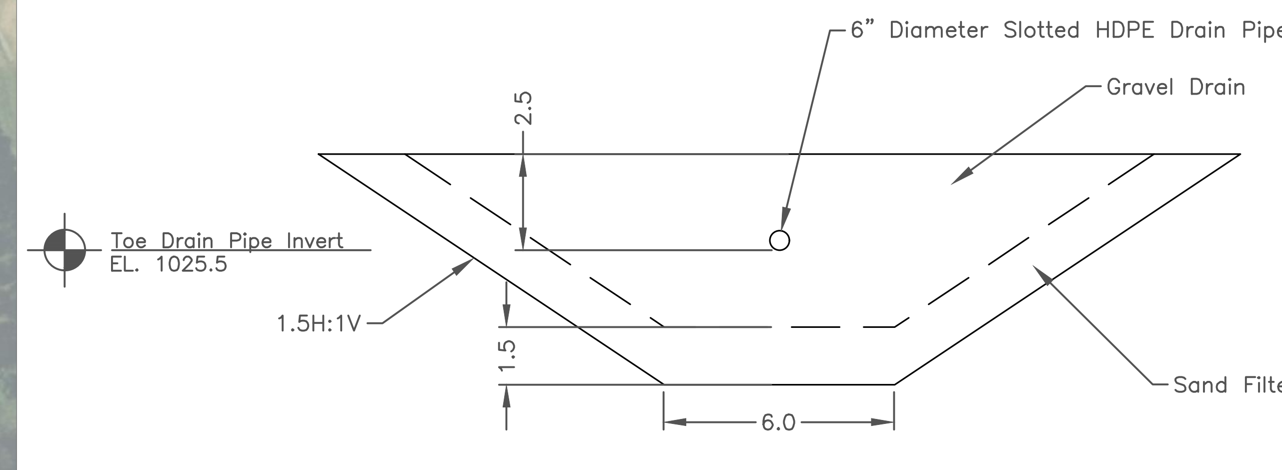
Designed	Kate Staley	Date 3/6/25
Drawn	Kate Staley	3/6/25
Checked		
Approved		

PROPOSED PRINCIPAL SPILLWAY PROFILE
UPPER TURTLE RIVER DAM NO. 9 REHAB
OWNER: GRAND FORKS COUNTY WRD
COUNTY: GRAND FORKS

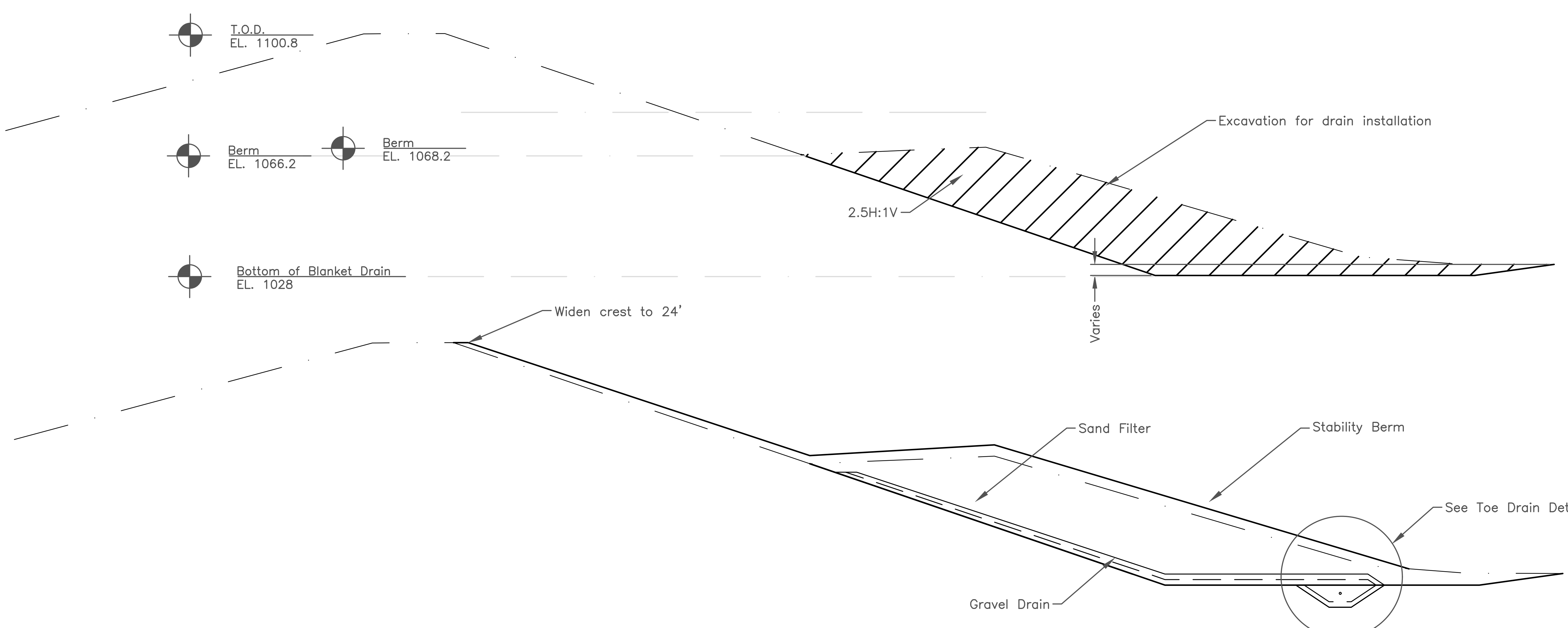
USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name bore_and_jack.dwg
Date 3/13/2025
Sheet 9 of 11





TOE DRAIN DETAIL



CHIMNEY AND BLANKET DRAIN TYPICAL CROSS SECTIONS
UPPER TURTLE RIVER DAM NO. 9 REHAB

OWNER: *GRAND FORKS COUNTY WRD*

COUNTY: *GRAND FORKS*

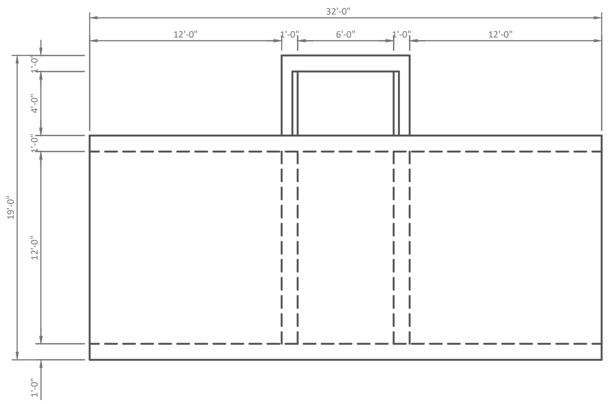
		Date
Designed	Kate Staley	3/6/25
Drawn	Kate Staley	3/6/25
Checked		
Approved		

United States Department of Agriculture
Natural Resources Conservation Service

File Name
chimneyblanket_earthwork.dwg

Date
3/14/2025

Sheet 10 of 11



TOP PLAN
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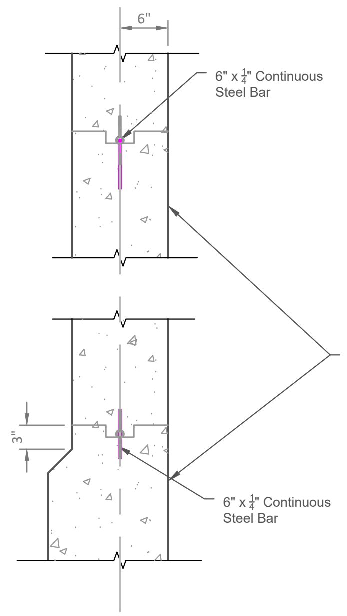
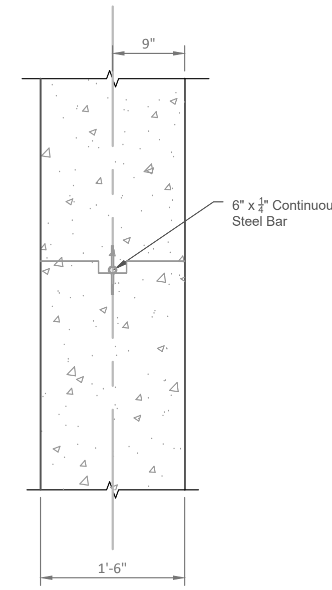
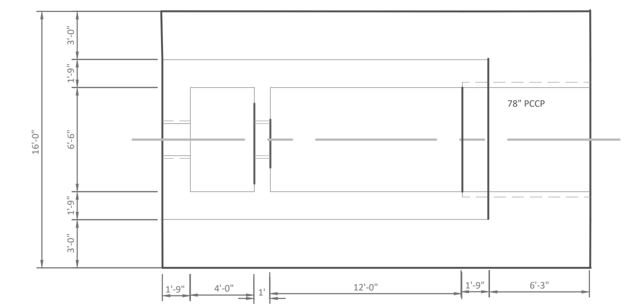


Riser & Cover Slab Walls



Counterforts



SECTION B-B
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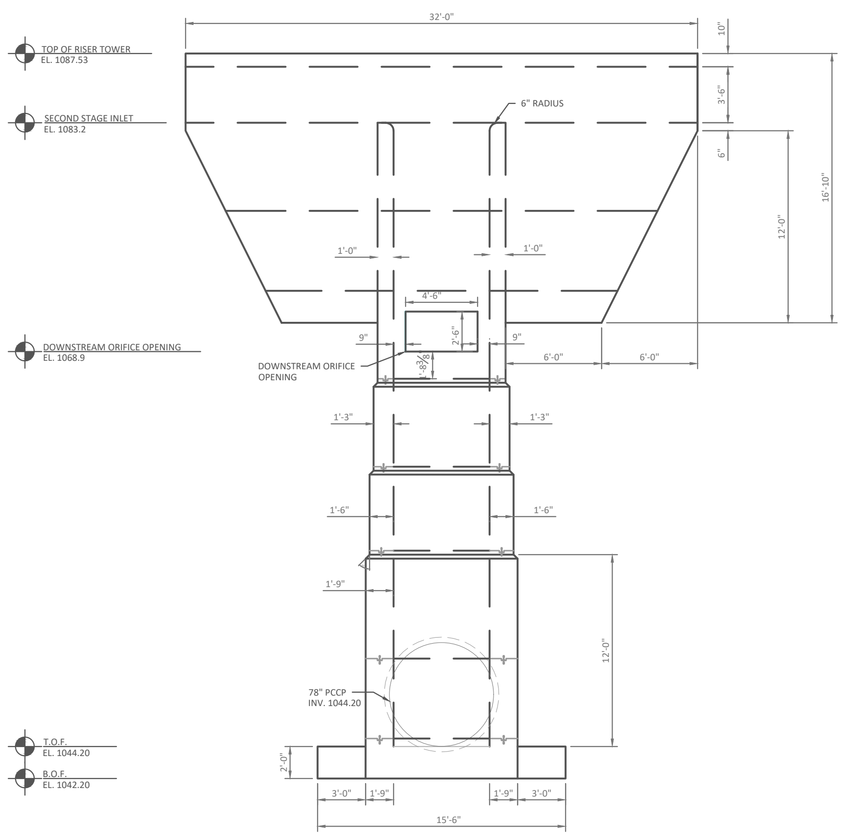
1 PLATE CONSTRUCTION JOINT DETAIL



DOWNSTREAM ELEVATION
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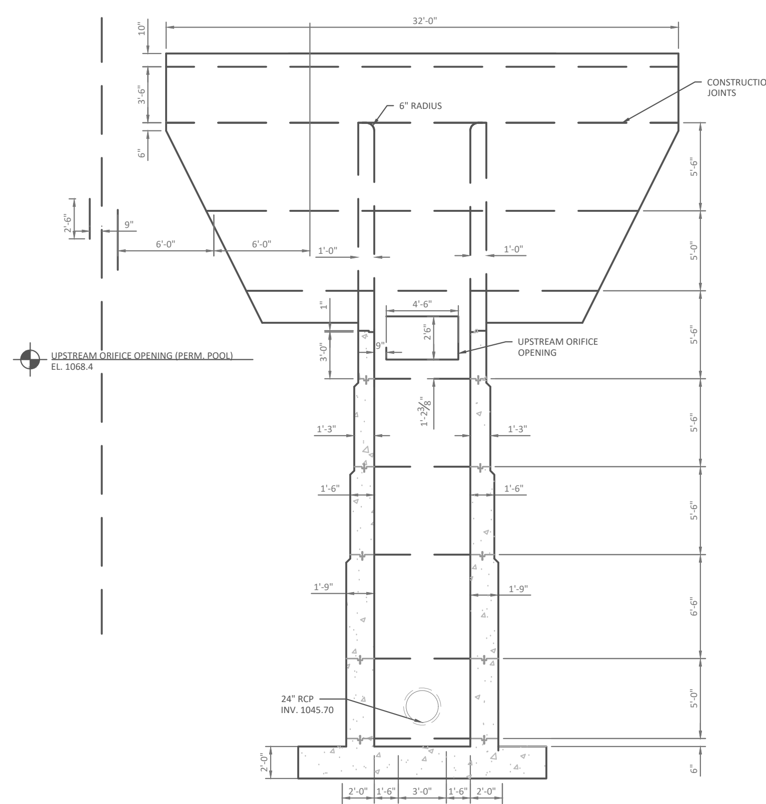

SECTION A-A
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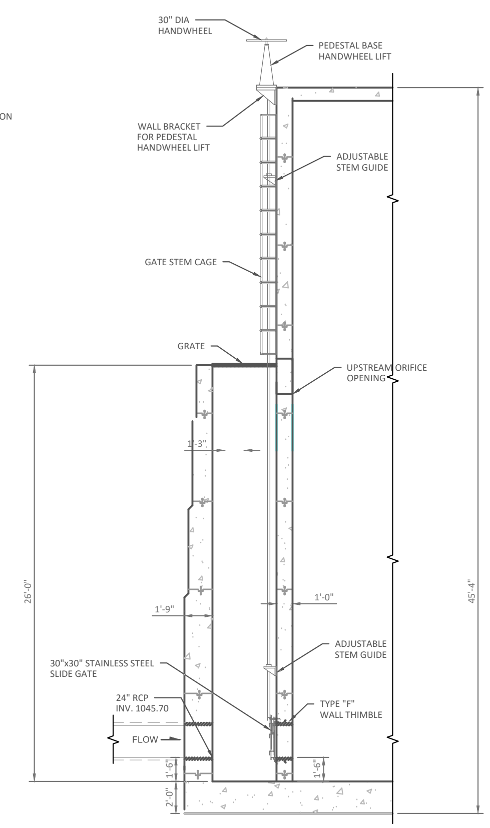

DETAILS OF SLIDE GATE
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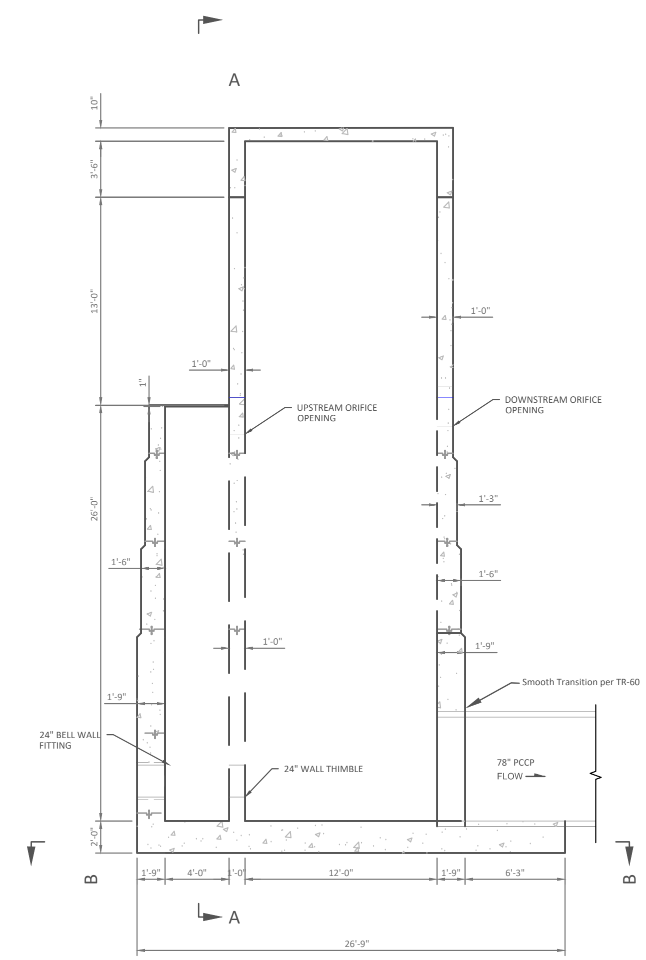


SIDEWALL ELEVATION
NOT TO SCALE

Designed	Kate Staley	Date 3/11/25
Drawn	Kate Staley	3/11/25
Checked	#	#
Approved	#	#

RISER TOWER DETAILS
UPPER TURTLE RIVER DAM NO. 9 REHAB
OWNER: GRAND FORKS COUNTY WRD
COUNTY: GRAND FORKS

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name riser_tower.dwg
Date 4/8/2025
Sheet 1 of 1

ATTACHMENT D-4-2:
PRINCIPAL SPILLWAY RISER TOWER CALCULATIONS

Upper Turtle River Dam No. 9 (Larimore Dam)

Attachment D-4-# Principal Spillway Riser Tower Calculations

References

- National Engineering Manual Part 536 - Structural Engineering
- Technical Release 210-30, Structural Design of Standard Covered Risers
- ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings

By:

- Jonathan Petersen
- April 8, 2025
- 30% Preliminary Design

Riser tower Data

$D := 78\ in$	D: pipe conduit diameter,
$N_{ih} := 40\ ft$	N_{ih}: vertical distance from pipe invert at riser to crest of covered inlet
$N_{is} := 23\ ft$	N_{is}: vertical distance from pipe invert at riser to soil surface/embankment
$N_{sh} := 17\ ft$	N_{sh}: vertical distance from soil surface to crest of covered riser inlet

Riser located in the embankment [TR-210-30.1-6]
Location of riser wall construction joints from top of footing:
6 ft, 12 ft, 18 ft, 24 ft, 30 ft

Minimum thickness of cover slab is 8 in [TR-210-30.1-2]
Minimum thickness of riser walls is 10 in [TR-210-30.1-2]
Wall thickness increments shall not exceed 3 in [TR-210-30.1-2]
For saturated foundation, allowable maximum excess pressure is 2,000 psf [TR-210-30.1-2]
Foundation is below normal pool, assumed saturated

Material properties:

$F_c := 5000\ psi$
$w_c := 150\ pcf$

Cover Slab and Cover Slab Walls
Use standard design [TR-210-30.1-2]:

- Cover slab thickness = 10 in
- Riser wall and cover slab wall thickness = 12 in
- Top slab live loading = 100 psf

Riser Wall Loading
The design of horizontal and vertical sections of riser walls must consider both lateral soil

pressure and water pressure loadings. Lateral soil pressures shall be assumed uniformly distributed around the riser for concrete design only. [TR-210-30.1-2]

Different loading conditions will be analyzed for stability analysis in accordance with TR-210-30.2-30.

The following embankment fill soil properties were obtained from the Geotechnical Engineering Report prepared by HDR, dated February 22, 2022:

Boring TH-32, 19.5 - 21.5' depth, Lean Clay with Sand

$\phi' := 24.3\ °$

$\gamma_m := 125\ pcf$

$\gamma_w := 62.4\ pcf$

$\gamma_d := \gamma_m - \gamma_w$

$$K_a := \frac{1-\sin(\phi')}{1+\sin(\phi')} = 0.417$$

$$K_o := 1-\sin(\phi') = 0.588$$

$$K_p := \frac{1+\sin(\phi')}{1-\sin(\phi')} = 2.399$$

Tests on risers of standard proportions show that the pressure difference may be taken as $(\Delta p/\text{w})/h_{vr}$ = 6.0 from the crest of the covered inlet of the riser to a distance equal to 1.5D below the crest, and the pressure distance is $(\Delta p/\text{w})/h_{vr}$ = 3.0 below distance 1.5D below the crest, where h_{vr} is the velocity head in the riser. [TR-210-30.1-3]

For V_b (max) = 30 fps:

$\Delta p/\text{w}$ = 5.76 ft
$\Delta p/\text{w}$ = 2.88 ft

Volume and weight for subsequent computations:

Cover slab:

$$slab := 32\ ft \cdot 12\ ft \cdot \frac{10}{12}\ ft = 320\ ft^3$$

Cover walls:

$$walls := 2 \cdot ((3.5\ ft \cdot 32\ ft) + (6\ ft \cdot 12\ ft)) \cdot 1\ ft = 368\ ft^3$$

Riser walls:

$$t_{12} := 16\ ft \cdot ((8\ ft \cdot 14\ ft) - (6\ ft \cdot 12\ ft)) = 640\ ft^3$$

$$t_{15} := 5.5\ ft \cdot ((8.5\ ft \cdot 14.5\ ft) - (6\ ft \cdot 12\ ft)) = 281.875\ ft^3$$

$$t_{18} := 5.5\ ft \cdot ((9\ ft \cdot 15\ ft) - (6\ ft \cdot 12\ ft)) = 346.5\ ft^3$$

$t_{21} := 11.5\ ft \cdot ((9.5\ ft \cdot 15.5\ ft) - (6\ ft \cdot 12\ ft)) = 865.4\ ft^3$

$gate_{wall} := 26\ ft \cdot (8\ ft + 3\ ft + 3\ ft) \cdot 1\ ft = 364\ ft^3$

Openings:

$Low_{inlet} := 2 \cdot 2.5\ ft \cdot 4.5\ ft \cdot 1\ ft = 22.5\ ft^3$

$Pipe_{outlet} := \frac{\pi}{4} \cdot (6.5\ ft)^2 \cdot 1.75\ ft = 58.1\ ft^3$

$Pipe_{inlets} := 2 \cdot \frac{\pi}{4} \cdot (2\ ft)^2 \cdot 1.75\ ft = 11\ ft^3$

Volume of riser of above footing:

$V_{riser} := slab + walls + t_{12} + t_{15} + t_{18} + t_{21} + gate_{wall} - (Low_{inlet} + Pipe_{outlet} + Pipe_{inlets}) = 3094\ ft^3$

Weight of riser of above footing:

$W_{riser} := V_{riser} \cdot 150\ pcf = 464\ kip$

Weighted Wall Width:

$$B := \frac{(12\ ft \cdot 9.5\ ft)}{38.5\ ft} + \frac{(5.5\ ft \cdot 9\ ft)}{38.5\ ft} + \frac{(5.5\ ft \cdot 8.5\ ft)}{38.5\ ft} + \frac{(14\ ft \cdot 8\ ft)}{38.5\ ft} = 8.4\ ft$$

Stability Analysis [TR-210-30]

Following the guidelines in NRCS Technical Release No. 30 - Structural Design of Standard Covered Risers the conditions that shall be considered for stability analysis can be found below.

Riser in the Reservoir Area [TR-210-30.2-31]

The following conditions should be investigated:

1. No sediment, wind on sidewall, moist soil condition.
2. No sediment, no wind, water surface to design sediment surface.
3. No sediment, wind on sidewall, water surface to design sediment surface.
4. No sediment, no wind, water surface to crest of covered inlet.
5. Sediment to design sediment surface, no wind, water surface to design sediment surface.
6. Sediment to design sediment surface, no wind, water surface to crest of covered inlet.
7. Sediment to design sediment surface, no wind, water surface to bottom of cover slab (riser primed)
8. The flotation criteria.

Riser in the Embankment [TR-210-30.2-32]

The following conditions should be investigated:

1. Embankment present, moist soil condition
2. Embankment present, water surface to embankment surface
3. Embankment present, water surface to crest of covered inlet
4. Embankment present, water surface to bottom of cover slab (riser primed)
5. No embankment placed, moist soil condition
6. The flotation criteria

Because the proposed riser tower structure will not be fully constructed within the reservoir or the embankment, it meets both criteria when checking stability. Many of the required load cases do not apply and can be omitted from analysis. In coordination with the NRCS the required load cases for the proposed riser tower were simplified into the conditions below.

Riser (Combined)

The following conditions should be investigated:

1. Embankment present, wind on sidewall, moist soil condition
2. Embankment present, wind on sidewall, water surface to embankment surface
3. Embankment present, no wind, water surface to crest of covered inlet
4. Embankment present, no wind, water surface to bottom of cover slab (riser primed)
5. No embankment placed, wind on sidewall, moist soil condition
6. The flotation criteria

Stability Analysis continued.. [TR-210-30]

Embankment Load on Riser

Assume the difference between upstream and downstream lateral earth pressures acting on the end wall of the structure:

Lateral earth pressures for moist conditions as k_{wm} = 50 pcf

Lateral earth pressures for saturated conditions as k_{wb} = 30 pcf

Unit soil weights for moist or saturated conditions as w_m = w_s = 140 pcf. Neglect friction which may act on the side-walls. [TR-210-30.1-5]

Wind Load

Risers located in the reservoir area shall be designed for wind acting over the entire sidewall using 50 pounds per square foot of pressure. [TR-210-30.1-6]

Risk Category 1, Exposure C [ASCE 7-16 26.10.2]

Wind Speed: $V := 105\ mph$

Wind Directionality Factor: $K_d := 0.9$

Velocity Pressure Coefficient: $K_z := 0.98$

Topographic Factor: $K_{zt} := 1.0$

Ground Elevation Factor: $K_e := 1.0$

Velocity Pressure: $q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 \cdot \left(\frac{1}{mph^2}\right) \cdot \left(\frac{lb}{ft^2}\right) = 24.9\ \frac{lb}{ft^2}$

50 *psf*>q_z

Assume, $W_{LL} := 50\ psf$

Wind Load on End Wall

During the construction phase when no embankment is present, the most conservative wind load will be acting with the negative moment on the footing caused by the dead load of the riser tower. The wind projection is the vertical distance between the top of footing and the top of the riser tower.

$$A_{wall.x} := (B \cdot N_{ih}) + (3 \cdot 12\ ft \cdot 6\ ft) + (0.5\ ft \cdot 24\ ft) + (4.43\ ft \cdot 32\ ft) = 704.6\ ft^2$$

$$cg_{wall.x} := \frac{\left(\left(B \cdot N_{ih} \cdot \frac{N_{ih}}{2}\right) + \left(A_{wall.x} - (B \cdot N_{ih})\right) \cdot 38\ ft\right)}{A_{wall.x}} = 29.4\ ft$$

$$M_{wind.x} := -\left(W_{LL} \cdot A_{wall.x} \cdot cg_{wall.x}\right) = -1037\ ft \cdot kip$$

Wind Load on End Wall (Embankment)

For stability analysis, the most conservative moment acting on the footing for both moist and saturated soil conditions is the combined reaction caused by wind load and the embankment loading on the structure. The wind projection is the vertical distance between the surface of the backfill and the top of the riser tower.

$$A_{wall.emb.x} := \left(N_{sh} \cdot 6\ ft\right) + \left(3 \cdot 12\ ft \cdot 6\ ft\right) + \left(4.43\ ft \cdot 32\ ft\right) = 459.8\ ft^2$$

$$cg_{wall.emb.x} := \frac{\left(\left(B \cdot N_{sh} \cdot \frac{N_{ih}}{2}\right) + \left(A_{wall.emb.x} - \left(B \cdot N_{sh}\right)\right) \cdot 38\ ft\right)}{A_{wall.emb.x}} = 32.4\ ft$$

$$M_{wind.emb.x} := \left(W_{LL} \cdot A_{wall.emb.x} \cdot cg_{wall.emb.x}\right) = 745\ ft \cdot kip$$

Wind Load on Side Wall

During the construction phase when no embankment is present, wind load acting on the side of the structure must be checked for overturning. The wind projection is the vertical distance between the top of footing and the top of the riser tower.

$$A_{wall.y} := \left(20.5\ ft \cdot 12\ ft\right) + \left(20\ ft \cdot 5.5\ ft\right) + \left(19.5\ ft \cdot 5.5\ ft\right) + \left(19\ ft \cdot 3\ ft\right) + \left(14.75\ ft \cdot 17.3\ ft\right) = 775.4\ ft^2$$

$$cg_{wall.y} := \frac{\left(\left(20\ ft \cdot 26\ ft \cdot \frac{26}{2}\ ft\right) + \left(14.75\ ft \cdot 17.3\ ft \cdot 34.7\ ft\right)\right)}{A_{wall.y}} = 20.1\ ft$$

$$M_{wind.y} := \left(W_{LL} \cdot A_{wall.y} \cdot cg_{wall.y}\right) = 781\ ft \cdot kip$$

Wind Load on Side Wall (Embankment)

The structure will be backfilled uniformly around the sides of the structure. Without the added moment from embankment, the condition where the wind load is projected on the end wall of the structure will govern. By inspection, this load case would not control, therefore, was omitted from analysis.

$$A_{wall.emb.y} := 14.75\ ft \cdot N_{sh} = 250.8\ ft^2$$

$$cg_{wall.emb.y} := cg_{wall.emb.x} = 32.4\ ft$$

$$M_{wind.emb.y} := \left(W_{LL} \cdot A_{wall.emb.y} \cdot cg_{wall.emb.y}\right) = 407\ ft \cdot kip$$

Volume outside riser walls but inside projected area 15.25'x6' the maximum section:
Between footing and earth surface,

$V_{1.75} := 12\ ft \cdot ((20.5\ ft \cdot 10\ ft) - (20.5\ ft \cdot 10\ ft)) = 0\ ft^3$

$V_{1.5} := 5.5\ ft \cdot ((20.5\ ft \cdot 10\ ft) - (20\ ft \cdot 9.5\ ft)) = 82.5\ ft^3$

$V_{1.25} := 5.5\ ft \cdot ((20.5\ ft \cdot 10\ ft) - (19.5\ ft \cdot 9\ ft)) = 162.25\ ft^3$

$V_1 := V_{1.75} + V_{1.5} + V_{1.25} = 244.75\ ft^3$

Between earth surface and crest of inlet, (to be conservative, neglect slab walls)

$V_2 := 17\ ft \cdot ((20.5\ ft \cdot 10\ ft) - (14.75\ ft \cdot 8\ ft)) = 1479\ ft^3$

Displacement volume V_d of riser between footing & crest of covered inlet (CI), slab walls (SW)

$V_{CI} := 1\ ft \cdot ((2 \cdot 3 \cdot 6\ ft \cdot 12\ ft) + (2 \cdot 0.5\ ft \cdot 12\ ft)) = 444\ ft^3$

$V_{SW1} := 17\ ft \cdot (8\ ft \cdot 14\ ft) = 1904\ ft^3$

$V_{SW2} := 5.5\ ft \cdot (8.5\ ft \cdot 19.5\ ft) = 912\ ft^3$

$V_{SW3} := 5.5\ ft \cdot (9\ ft \cdot 20\ ft) = 990\ ft^3$

$V_{SW4} := 12\ ft \cdot (9.5\ ft \cdot 20.5\ ft) = 2337\ ft^3$

$V_d := V_{CI} + V_{SW1} + V_{SW2} + V_{SW3} + V_{SW4} = 6587\ ft^3$

Footing (f),

$A_f := 15.5\ ft \cdot 26.75\ ft = 415\ ft^2$ $\qquad$ $footing_{thk} := 2\ ft$

$V_f := A_f \cdot footing_{thk} = 829\ ft^3$

$W_f := V_f \cdot w_c = 124\ kip$

Various working volumes,

$V_{B1} := N_{is} \cdot (26\ ft \cdot 3\ ft) \cdot 2 = 3588\ ft^3$ $\qquad$ $V_{B2} := N_{is} \cdot (10\ ft \cdot 6.25\ ft) = 1437.5\ ft^3$

$V'_{B1} := N_{sh} \cdot (20.5\ ft \cdot 3\ ft) = 1046\ ft^3$ $\qquad$ $V'_{B2} := N_{sh} \cdot (10\ ft \cdot 6.25\ ft) = 1062.5\ ft^3$

(1) Embankment present, wind on sidewall, moist soil conditions:

Allowable avg. pressure: $P_{avg.all.1} := 140\ pcf \cdot \left(N_{is} + footing_{thk}\right) + 2\ ksf = 5.5\ ksf$

Allowable max pressure: $P_{max.all.1} := 140\ pcf \cdot \left(N_{is} + footing_{thk}\right) + 4\ ksf = 7.5\ ksf$

Embankment Moment: [TR-210-30}

The moment may be computed as indicated by Figure 2-17. Thus:

$$M = \frac{1}{2} F h_s = 0.0125\ B h_s^3 \quad \text{ft kips}$$

where

B = width of endwall, ft (for convenience, use some constant "weighted" width)

h_s = as previously defined, ft

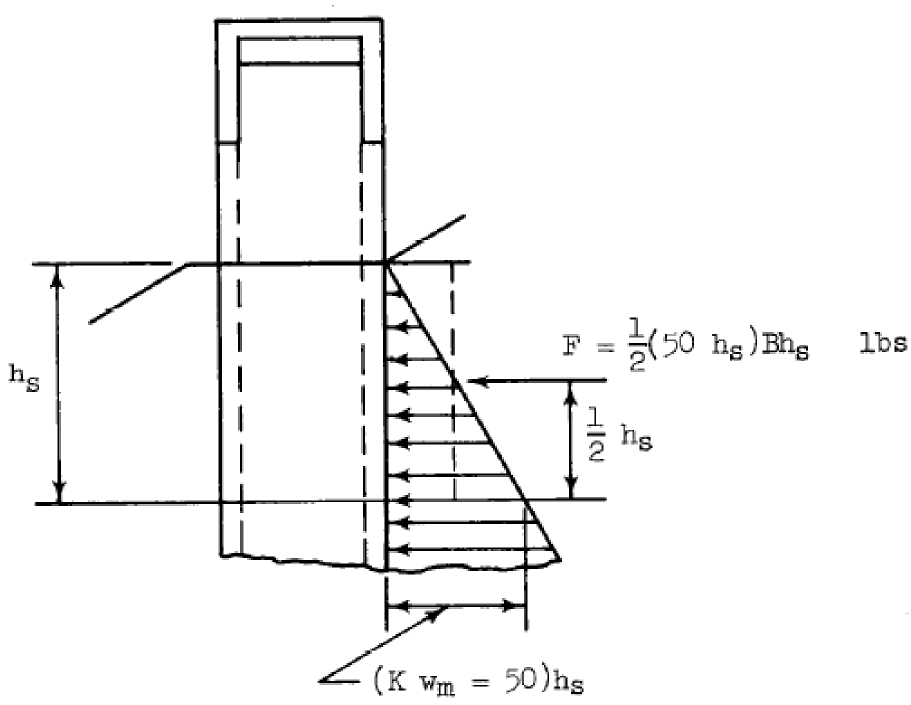
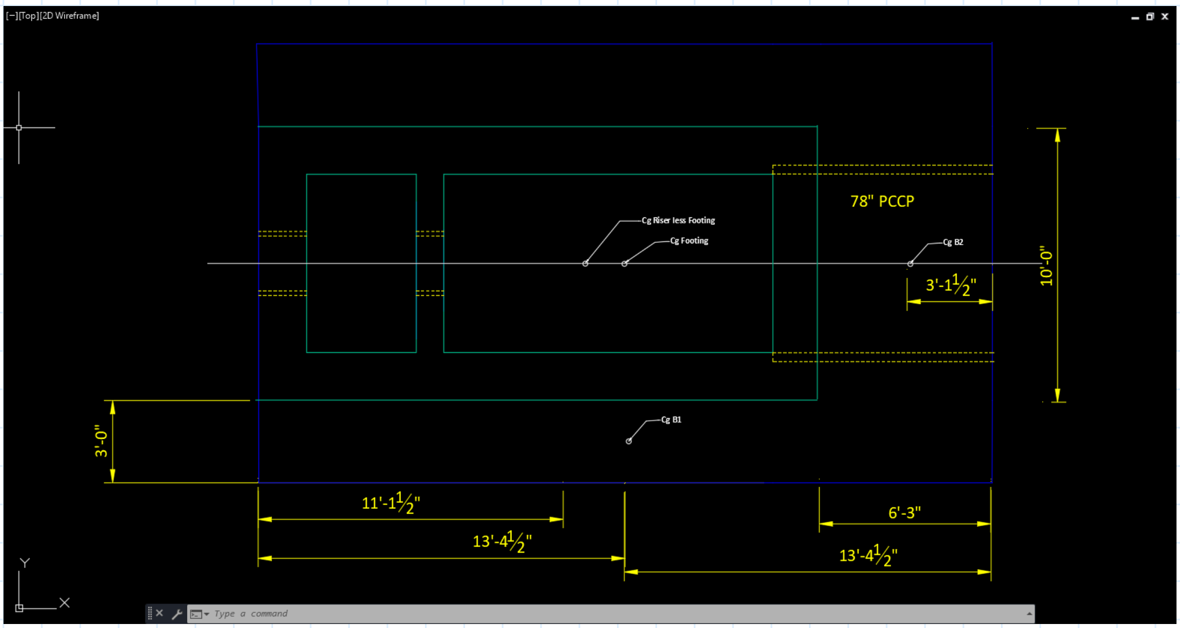


Figure 2-17. Assumed embankment loading.

$$M_{emb} := \left(\frac{1}{2} \cdot \left(\frac{1}{2} \cdot 50\ pcf\right)\right) \cdot B \cdot \left(N_{is} + footing_{thk}\right)^3 = 1635\ ft \cdot kip$$

Cg Riser less Footing			
Name	Vol (ft^3)	Dist (ft)	Vol*Dist (ft^4)
Upstream Wall	401.15	0.9	351.0
Inner Wall	425	6.3	2656.3
Top	122.9	12.8	1567.2
Downstream wall	566.2	19.6	11110.7
Inner Riser	328	6.3	2050.0
Outer Riser	328	19.6	6437.0
sum	**2171**		**24172**
Cg (from Upstrm edge)		11.1	



$W_{riser} = 464\ kip$

$M_{riser} := W_{riser} \cdot 2.3\ ft = 1067\ ft \cdot kip$

$W_f = 124\ kip$

$M_f := W_f \cdot 0\ ft = 0\ ft \cdot kip$

$W_{V1} := V_1 \cdot 140\ pcf = 34\ kip$

$M_{V1} := W_{V1} \cdot 2.3\ ft = 79\ ft \cdot kip$

$W_{VB1} := V_{B1} \cdot 140\ pcf = 502\ kip$

$M_{VB1} := W_{VB1} \cdot 0\ ft = 0\ ft \cdot kip$

$W_{VB2} := V_{B2} \cdot 140\ pcf = 201\ kip$

$M_{VB2} := W_{VB2} \cdot -10.33\ ft = -2079\ ft \cdot kip$

$W_{total.1} := W_{riser} + W_f + W_{V1} + W_{VB1} + W_{VB2} = 1326\ kip$

$M_{total.1} := M_{riser} + M_f + M_{V1} + M_{VB1} + M_{VB2} = -933\ ft \cdot kip$

Moment about centerline of footing:

$$M_{CL1} := M_{emb} + M_{wind.emb.x} + M_{total.1} = 1448\ ft \cdot kip$$

$$P_{max.1} := \left(\frac{W_{total.1}}{A_f}\right) \cdot \left(1 + \left(\frac{(6 \cdot M_{CL1})}{26.75\ ft \cdot W_{total.1}}\right)\right) = 3.982\ ksf$$

$$test.P_{max.1} := \text{if}\left(P_{max.1} < P_{max.all.1}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{avg.1} := \frac{W_{total.1}}{A_f} = 3.2\ ksf$$

$$test.P_{avg.1} := \text{if}\left(P_{avg.1} < P_{avg.all.1}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{min.1} := \left(\frac{W_{total.1}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL1})}{26.75\ ft \cdot W_{total.1}}\right)\right) = 2.4\ ksf$$

$$test.P_{min.1} := \text{if}\left(P_{min.1} > 0, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

(2) Embankment present, wind on sidewall, water surface to embankment:

Allowable avg. pressure: $P_{avg.all.2} := 140\ pcf \cdot \left(N_{is} + footing_{thk}\right) + 1\ ksf = 4.5\ ksf$

Allowable maximum pressure: $P_{max.all.2} := 140\ pcf \cdot \left(N_{is} + footing_{thk}\right) + 2\ ksf = 5.5\ ksf$

$$M_2 := \left(\frac{(30\ pcf)}{50\ pcf}\right) \cdot \left(M_{emb}\right) = 980.9\ ft \cdot kip$$

Moment about the centerline of footing

$$M_{CL2} := M_2 + M_{wind.emb.x} + M_{total.1} = 793.7\ ft \cdot kip$$

$$P_{max.2} := \left(\frac{W_{total.1}}{A_f}\right) \cdot \left(1 + \left(\frac{(6 \cdot M_{CL2})}{26.75\ ft \cdot W_{total.1}}\right)\right) = 3.6\ ksf$$

$$test.P_{max.2} := \text{if}\left(P_{max.2} < P_{max.all.2}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{avg.2} := \frac{W_{total.1}}{A_f} = 3.2\ ksf$$

$$test.P_{avg.2} := \text{if}\left(P_{avg.2} < P_{avg.all.2}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{min.2} := \left(\frac{W_{total.1}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL2})}{26.75\ ft \cdot W_{total.1}}\right)\right) = 2.8\ ksf$$

$$test.P_{min.2} := \text{if}\left(P_{min.2} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{uplift.2} := 62.4\ pcf \cdot \left(N_{is} + footing_{thk}\right) = 1.56\ ksf$$

$$P_{net.2} := P_{min.2} - P_{uplift.2} = 1.21\ ksf$$

$$test.P_{net.2} := \text{if}\left(P_{net.2} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

(3) Embankment present, water surfce to crest of inlet:

Allowable avg. pressure: $P_{avg.all.3} := 3.91\ ksf + \left(N_{sh} \cdot 62.4\ pcf\right) = 4.97\ ksf$

Allowable maximum pressure: $P_{max.all.3} := 4.91\ ksf + \left(N_{sh} \cdot 62.4\ pcf\right) = 5.97\ ksf$

Previous:

$W_{total.1} = 1326\ kip$ $\qquad M_{total.1} = -932.6\ ft \cdot kip$

$W_{V2} := V_2 \cdot 62.4\ pcf = 92.3\ kip$ $\qquad M_{V2} := W_{V2} \cdot 2.3\ ft = 212.3\ ft \cdot kip$

$W_{V'B2} := V'_{B2} \cdot 62.4\ pcf = 66.3\ kip$ $\qquad M_{V'B2} := W_{V'B2} \cdot -10.33\ ft = -684.9\ ft \cdot kip$

$$W_{total.3} := W_{total.1} + W_{V2} + W_{V'B2} = 1484.9\ kip$$

$$M_{total.3} := M_{total.1} + M_{V2} + M_{V'B2} = -1405.2\ ft \cdot kip$$

Moment about centerline of footing:

$$M_{CL3} := M_2 + M_{total.3} = -424.3\ ft \cdot kip$$

$$P_{max.3} := \left(\frac{W_{total.3}}{A_f}\right) \cdot \left(1 + \left(\frac{(6 \cdot M_{CL3})}{26.75\ ft \cdot W_{total.3}}\right)\right) = 3.352\ ksf$$

$$test.P_{max.3} := \text{if}\left(P_{max.3} < P_{max.all.3}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{avg.3} := \frac{W_{total.3}}{A_f} = 3.6\ ksf$$

$$test.P_{avg.3} := \text{if}\left(P_{avg.3} < P_{avg.all.3}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{min.3} := \left(\frac{W_{total.3}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL3})}{26.75\ ft \cdot W_{total.3}}\right)\right) = 3.8\ ksf$$

$$test.P_{min.3} := \text{if}\left(P_{min.3} > 0, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

$$P_{uplift.3} := 62.4\ pcf \cdot \left(N_{sh} + N_{is} + footing_{thk}\right) = 2.6\ ksf$$

$$P_{net.3} := P_{min.3} - P_{uplift.3} = 1.19\ ksf$$

$$test.P_{net.3} := \text{if}\left(P_{net.3} > 0, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$$

(4) Embankment present, water surface to bottom of cover slab (riser primed):

Allowable avg. pressure: $P_{avg.all.4} := 3.91\ ksf + \left(\left(N_{sh} + 3.5\ ft\right) \cdot 62.4\ pcf\right) = 5.19\ ksf$

Allowable maximum pressure: $P_{max.all.4} := 4.91\ ksf + \left(\left(N_{sh} + 3.5\ ft\right) \cdot 62.4\ pcf\right) = 6.19\ ksf$

Previous: $W_{total.3} = 1484.9\ kip$ $M_{total.3} = -1405.2\ ft \cdot kip$

$$W_{water.riser} := \left(\left(39\ ft \cdot 12\ ft \cdot 6\ ft\right) + \left(26\ ft \cdot 4\ ft \cdot 6\ ft\right)\right) \cdot 62.4\ pcf = 214.2\ kip$$

$$M_{water.riser} := W_{water.riser} \cdot 2.3\ ft = 492.6\ ft \cdot kip$$

$$W_{water.crest} := \left(A_f \cdot 3.5\ ft\right) \cdot 62.4\ pcf = 90.6\ kip$$

$$M_{water.crest} := W_{water.crest} \cdot 0\ ft = 0\ ft \cdot kip$$

$$W_{displaced.water} := \left(-20.25\ ft \cdot 3.5\ ft \cdot 1\ ft\right) \cdot 2 \cdot 62.4\ pcf = -8.8\ kip$$

$$M_{displaced.water} := W_{displaced.water} \cdot 2.3\ ft = -20.3\ ft \cdot kip$$

$$W_{total.4} := W_{total.3} + W_{water.riser} + W_{water.crest} + W_{displaced.water} = 1780.8\ kip$$

$$M_{total.4} := M_{total.3} + M_{water.riser} + M_{water.crest} + M_{displaced.water} = -933.006\ ft \cdot kip$$

Moment about centerline of footing

$$M_{CL4} := M_2 + M_{total.4} = 47.9\ ft \cdot kip$$

$$P_{max.4} := \left(\frac{W_{total.4}}{A_f}\right) \cdot \left(1 + \frac{(6 \cdot -M_{CL4})}{26.75\ ft \cdot W_{total.4}}\right) = 4.3\ ksf$$

$$test.P_{max.4} := \text{if}\left(P_{max.4} < P_{max.all.4}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{avg.4} := \frac{W_{total.4}}{A_f} = 4.3\ ksf$$

$$test.P_{avg.4} := \text{if}\left(P_{avg.4} < P_{avg.all.4}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{min.4} := \left(\frac{W_{total.4}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL4})}{26.75\ ft \cdot W_{total.4}}\right)\right) = 4.3\ ksf$$

$$test.P_{min.4} := \text{if}\left(P_{min.4} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{uplift.4} := 62.4\ pcf \cdot \left(3.5\ ft + N_{ih} + footing_{thk}\right) = 2.8\ ksf$$

$$P_{net.4} := P_{min.4} - P_{uplift.4} = 1.43\ ksf$$

$$test.P_{net.4} := \text{if}\left(P_{net.4} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

(5) No embankment placed, wind on sidewall, moist soil condition

Allowable avg. pressure: $P_{avg.all.5} := 0 + 2\ ksf = 2\ ksf$

Allowable maximum pressure: $P_{max.all.5} := 0 + 4\ ksf = 4\ ksf$

$W_{riser} = 464.1\ kip$ $\quad M_{riser} = 1067.5\ ft \cdot kip$

$W_f = 124.4\ kip$ $\quad M_{wind.x} = -1037.3\ ft \cdot kip$

$W_{total.5} := W_{riser} + W_f = 588.5\ kip$ $\quad M_{wind.y} = 780.7\ ft \cdot kip$

Moment about centerline of footing

$$M_{CL5.x} := M_{riser} + M_{wind.x} = 30.1\ ft \cdot kip$$

$$P_{max.5.x} := \left(\frac{W_{total.5}}{A_f}\right) \cdot \left(1 + \frac{(6 \cdot -M_{CL5.x})}{26.75\ ft \cdot W_{total.5}}\right) = 1.4\ ksf$$

$$test.P_{max.5} := \text{if}\left(P_{max.5.x} < P_{max.all.5}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{avg.5.x} := \frac{W_{total.5}}{A_f} = 1.4\ ksf$$

$$test.P_{avg.5} := \text{if}\left(P_{avg.5.x} < P_{avg.all.5}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{min.5.x} := \left(\frac{W_{total.5}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL5.x})}{26.75\ ft \cdot W_{total.5}}\right)\right) = 1.4\ ksf$$

$$test.P_{min.5.x} := \text{if}\left(P_{min.5.x} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$M_{CL5.y} := M_{wind.y} = 780.7\ ft \cdot kip$$

$$P_{max.5.y} := \left(\frac{W_{total.5}}{A_f}\right) \cdot \left(1 + \frac{(6 \cdot -M_{CL5.y})}{26.75\ ft \cdot W_{total.5}}\right) = 0.997\ ksf$$

$$test.P_{max.5.y} := \text{if}\left(P_{max.5.y} < P_{max.all.5}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{avg.5.y} := \frac{W_{total.5}}{A_f} = 1.4\ ksf$$

$$test.P_{avg.5.y} := \text{if}\left(P_{avg.5.y} < P_{avg.all.5}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$P_{min.5.y} := \left(\frac{W_{total.5}}{A_f}\right) \cdot \left(\left(1 - \frac{(6 \cdot M_{CL5.y})}{26.75\ ft \cdot W_{total.5}}\right)\right) = 0.997\ ksf$$

$$test.P_{min.5.y} := \text{if}\left(P_{min.5.y} > 0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

(6) Floatation

Flotation Criteria [TR-210-30.1-6]

When the riser is located in the reservoir area, the ratio of the weight of the riser to the weight of the volume of displaced water by the riser shall not be less than 1.5. Low stage inlet(s), if any, shall be assumed plugged for this computation.

When the riser is located in the embankment, same as 1, but add to the weight of the riser, the buoyant weight of the submerged fill over the riser footing projections. Take the buoyant weight as w_b = 50 pcf.

Because the riser tower fits both criteria of being in the reservoir and in the embankment the buoyant weight of submerged fill over footing projections will not count unless needed.

$$Res. := \frac{W_{riser}}{\left(V_d + V_f\right) \cdot 62.4\ pcf} = 1.003$$

$$Emb. := \frac{W_{riser} + \left(\left(V_1 + V_{B1} + V_{B2}\right) \cdot 50\ pcf\right)}{\left(V_d + V_f\right) \cdot 62.4\ pcf} = 1.572$$

$$F.S. := \text{if}\left(Res. > 1.5, Res., Emb.\right) = 1.572$$

$$\text{if}\left(F.S. > 1.5, \text{“OK”}, \text{“Fail”}\right) = \text{“OK”}$$

ATTACHMENT D-4-3:
PRINCIPAL SPILLWAY RISER TOWER REINFORCEMENT CALCULATIONS

LARIMORE

* RISER TOWER WALL FLEXURE CALCS

* LOADING

γ_m = 125 PCF (MOIST UNIT WEIGHT)

γ_w = 62.4 PCF

FORCE PRESSURE

WIND = $F = q_h G C_s A_s \Rightarrow P = q_h G C_s$ = 47.6 PSF

→ FROM ASCE 7-29.3.1
FOR FREESTANDING WALLS

$G = 0.85$

$q_h = 0.00256 K_z K_d V^2$ = 40 PSF

V = 123 MPH (RISK CAT IV)

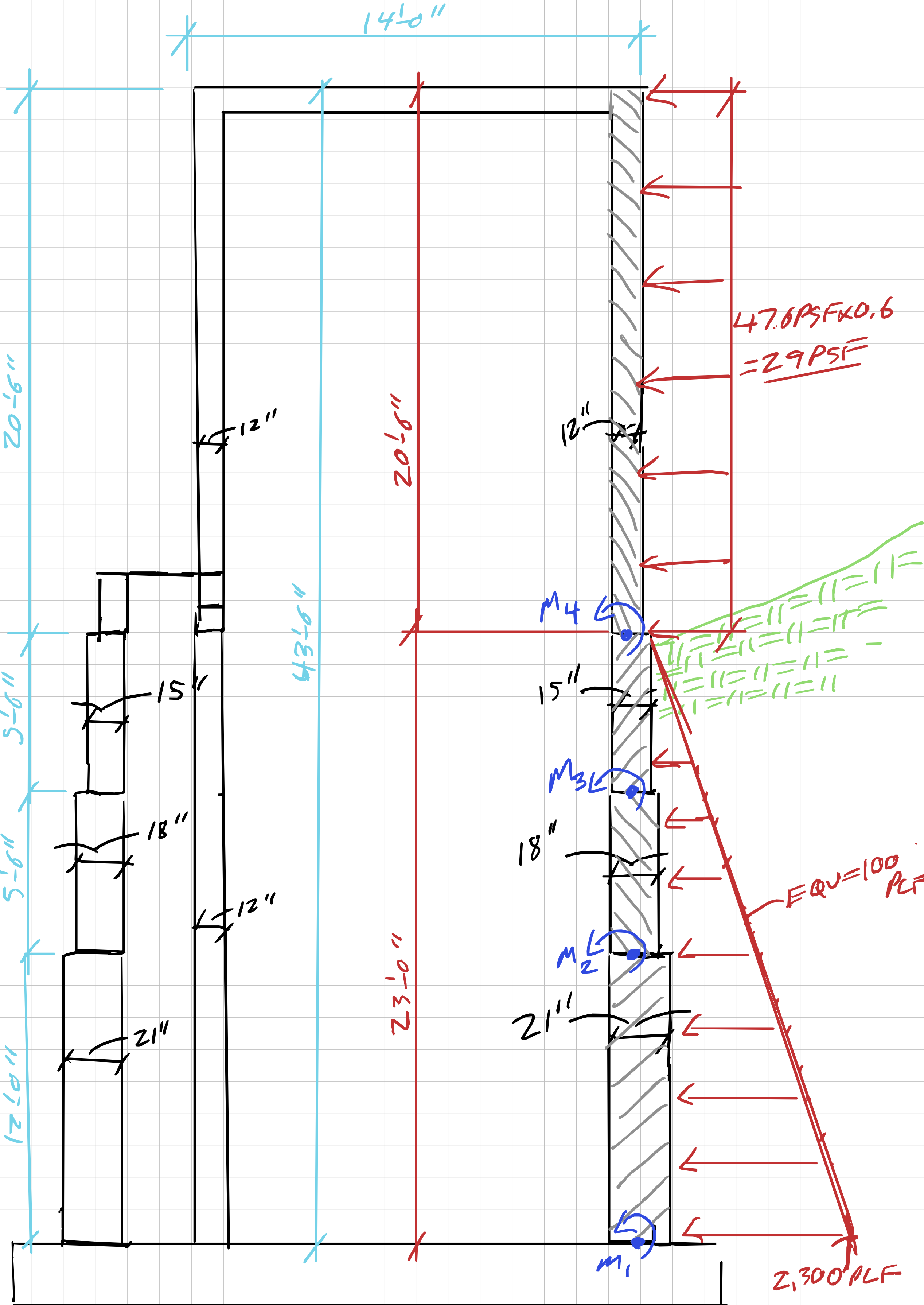
K_z = 1.22 (EXP D, 40 ft)

K_d = 0.85 (TABLE 26.6-1)

C_s = 1.40 (FIG 29.3-1, B/s = 2.0, s/h = 1.0)

A_s = NOT USED

By: Tom Schanandore, Feb. 8, 2025



14'-0"
20'-6"
5'-6"
5'-6"
12'-0"
12"
15"
18"
12"
21"
43'-6"
20'-6"
23'-0"
47.6PSF×0.6
=29PSF
M4
M3
M2
M1
EQU=100 PCF
2,300 PLF

MOMENT ASSESSMENT

$LC: 1.\cancel{2}D + 1.0W + 1.6H$

W = WIND
H = EARTH

$M_{MAX} = M_W + M_H = 5{,}585.2\ k\text{-}in/ft$

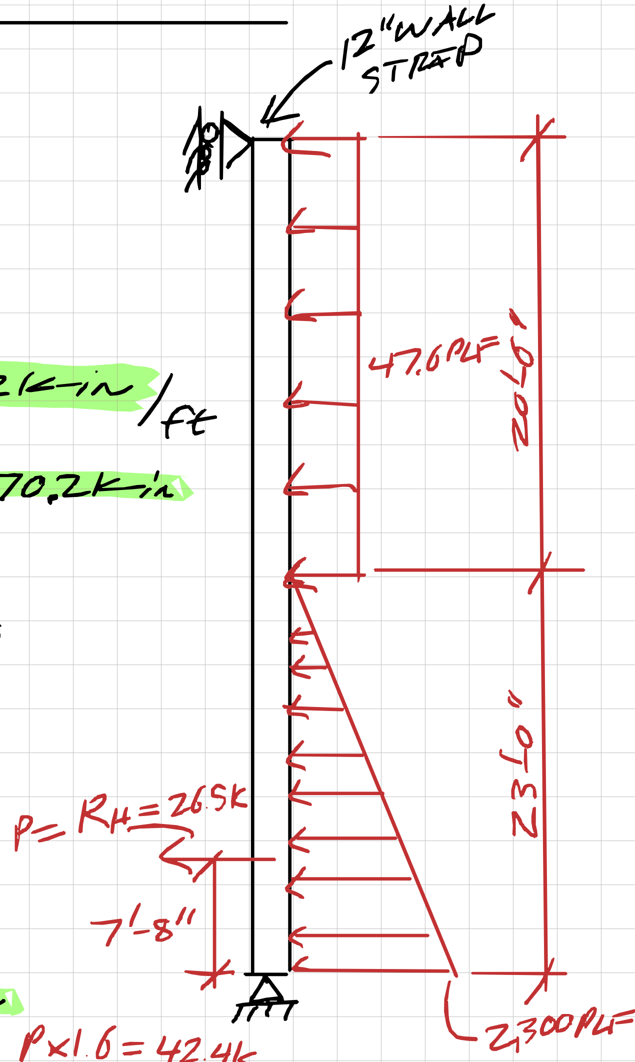
$$M_W = \frac{R_1^2}{2w} = 5{,}846\ lbs\text{-}ft \Rightarrow 70.2\ k\text{-}in$$

$$R_1 = \frac{wa}{2L}(2L-a) = 746\ lbs$$

a = 20'-6"
L = 43'-6"
wa = 975.8 lbs

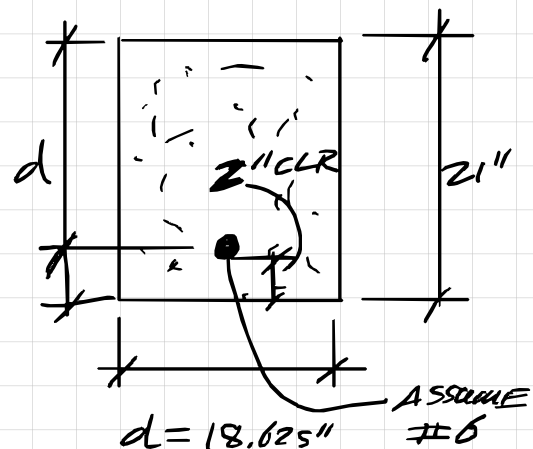
$$M_H = \frac{Pab}{L} = 459{,}577\ lb\text{-}ft \Rightarrow 5{,}515\ k\text{-}in$$

a = 20'-6
b = 23'-0"
L = 43'-6"



* ASSESS FLEXURE REINFORCEMENT @ 7'-8" (MAX MOMENT

$M_u = 5{,}585.2\ k\text{-}in$



$\bar{K} = \dfrac{M_u}{\phi b d^2} = 1.49$

$\phi = 0.9$

$$\rho = \frac{f_y - \sqrt{f_y^2 - \left[4\left(0.59\frac{f_y^2}{f'_c}\right)\bar{K}\right]}}{2\left(0.59\frac{f_y^2}{f'_c}\right)}$$

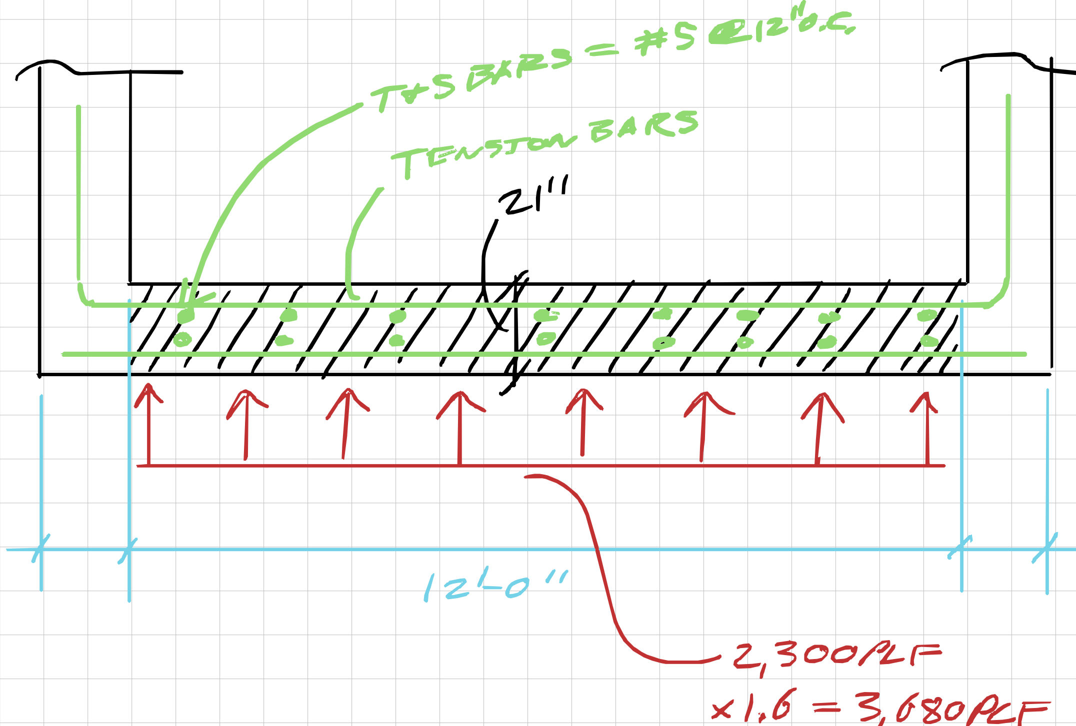
$f_y = 60{,}000\ ksi$

$f'_c = 5{,}000\ ksi$

$$\rho = \frac{60 - \sqrt{3600 - 1{,}699.2\bar{K}}}{849.6} = 0.0322$$

$A_{st} = \rho b d = 0.0322(12)(18.625) = 7.1967\ in^2/ft$

* HORIZONTAL WALL BAR SIZING (21" WALL)



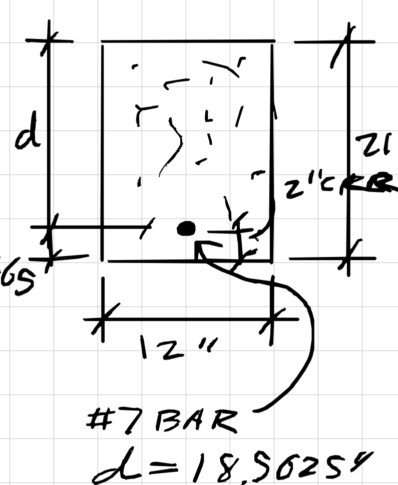
$$M_{max} = \frac{wL^2}{8} = 66{,}240\,lb\text{-}ft \Rightarrow 794.9\,k\text{-}in$$

SIMPLE BEAM MOMENT

$$\bar{K} = \frac{M_u}{0.9bd^2} = 0.2136$$

$$\rho = \frac{60 - \sqrt{3600 - 1{,}699.2\bar{K}}}{849.6} = 0.00365$$

FOR. $f_y = 60{,}000\,ksi$
$f'_c = 5{,}000\,ksi$



$$A_{st} = \rho bd = 0.813\,in^2/ft$$

$\Rightarrow$ #7 @ 8" o.c.