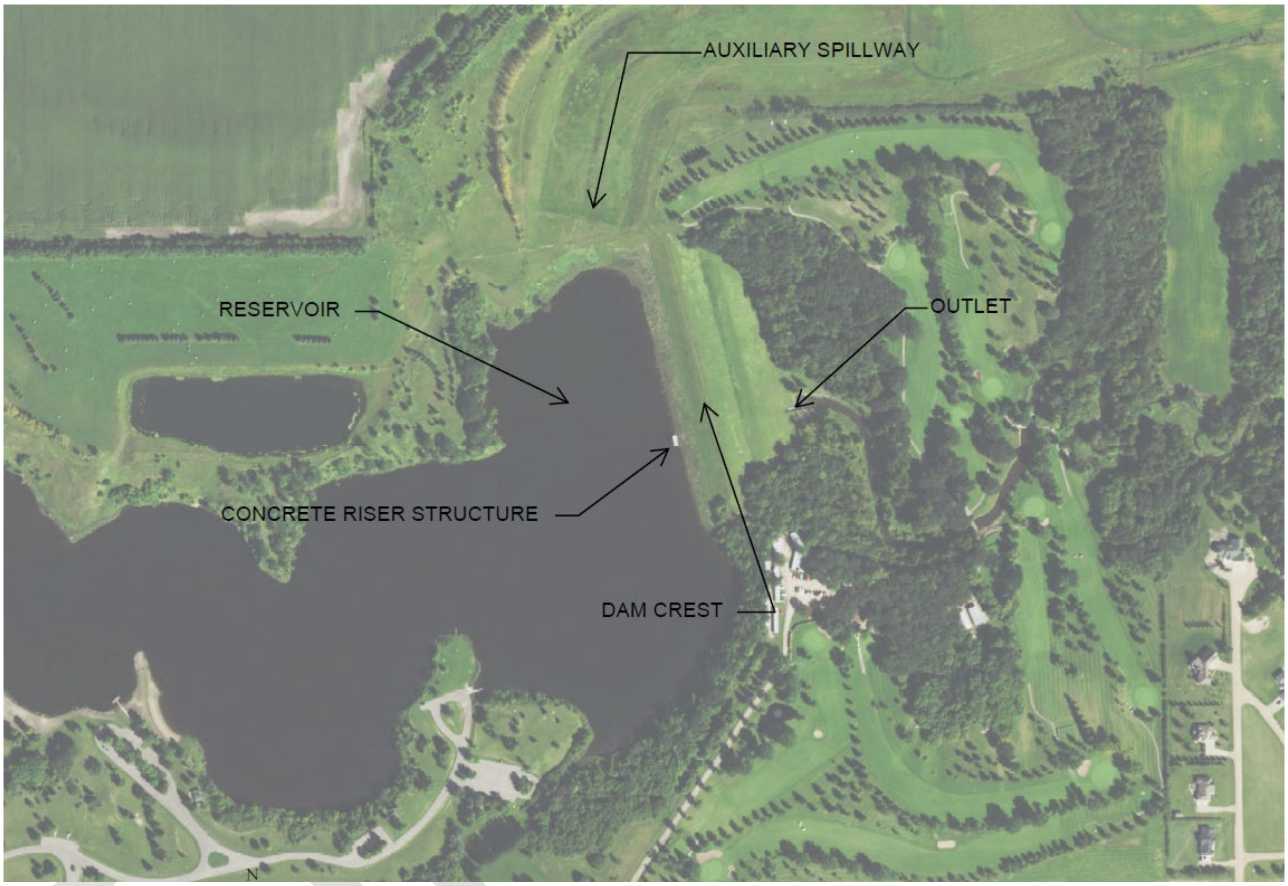
UPPER TURTLE RIVER DAM NO.9 (LARIMORE DAM)

Appendix D-3 Alternatives Evaluation Report



Prepared for: **Grand Forks County Water Resources District**
151 South 4th Street
Grand Forks, North Dakota 58201

Prepared by:

Natural Resources Conservation Service
North Dakota Engineering Staff
220 E Rosser Ave, Box 1458
Bismarck, ND 58502-1458

United States Department of Agriculture

February 20 , 2026

TABLE OF CONTENTS

TABLES

FIGURES

ACRONYMS

ACB	Articulated Concrete Block
ASCE	American Society of Civil Engineers
FBH	Freeboard Hydrograph
FEMA	Federal Emergency Management Agency
IDF	Inflow Design Flood (For high hazard dams, synonymous with FBH)
GFCWRD	Grand Forks County Water Resource District
NASS	National Agricultural Statistics Service
NDDWR	North Dakota Department of Water Resources
NDSWC	North Dakota State Water Commission
NRCS	Natural Resources Conservation Service
PMP	Probable Maximum Precipitation
RCC	Roller Compacted Concrete
RRRA	Red River Retention Authority
SAF	St. Anthony Falls
USFWS	U.S. Fish and Wildlife Services
USACE	U.S. Army Corps of Engineers

1 BACKGROUND

This report documents the development of structural alternatives addressing deficiencies documented in **Appendix D-1** (Existing Conditions Assessment Report) to develop a Watershed Plan for the Upper Turtle River Watershed, Upper Turtle River Dam No. 9 (also known as Larimore Dam). Larimore Dam is located in Section 32 of Hegton Township (T152N, R54W), Grand Forks County, ND, and has a drainage area of approximately 65.5 square miles. Project maps showing the location of Larimore Dam are provided in **Appendix B** of the Watershed Plan – Environmental Assessment. The dam was built with joint purposes, to provide flood control and recreation opportunities along the South Branch Turtle River.

1.1 PREVIOUS EVALUATIONS

An existing conditions assessment was completed for Larimore Dam during the planning process. The assessment verified that Larimore Dam is a high hazard structure. During the existing conditions assessment, Larimore Dam was evaluated using technical standards and current practices within the industry for a high hazard structure. Three primary categories were evaluated including: geotechnical, structural, and hydrologic and hydraulic. The Existing Conditions Assessment Report located in **Appendix D-1** contains documentation for the evaluation of the dam. A summary of the existing condition assessment is also provided in **Table D-3-1**.

***Table D-3-1**: Existing Conditions Assessment Summary Table*

Criteria	Larimore Dam
Foundation Drain Fill	Does not meet state-of-the-practice criteria for seepage control/conveyance
Downstream Slope – Normal Pool	Factor of Safety Met (Required = 1.5, Calculated = 2.5)
Downstream Slope – Flood Surcharge Pool	Factor of Safety Met (Required = 1.4, Calculated = 1.7)
Rapid Drawdown	Factor of Safety Met (Required = 1.2, Calculated = 2.3)
Riser Tower / Principal Spillway Inlet	Good overall condition
Principal Spillway Conduit and Outlet	Corrosion with maximum estimated life of 20 years
Embankment and Auxiliary Spillway	Lightly used, some scattered vegetation, rodent activity; erosion along auxiliary spillway slopes
Principal Spillway Capacity	Not Sufficient
Principal Spillway Drawdown	Not Sufficient
Auxiliary Spillway Capacity	Sufficient
Auxiliary Spillway Stability	Stable
Auxiliary Spillway Integrity	Breach

1.2 PURPOSE

The purpose of this report is to identify all practical alternatives that rehabilitate the dam to meet NRCS and NDDWR (formerly known as the NDSWC) dam safety standards for a high hazard structure. NRCS design and safety standards are provided in *Earth Dams and Reservoirs* (NRCS, 2019). The NDDWR standards are provided in Policy Reg_05.2024 (NDDWR, 2024). Once a range of practical structural alternatives were identified, comparative concept level cost estimates were prepared for each alternative. The cost estimates were used to assist in the selection of the alternative(s), however, cost alone was not the only factor in determining the structural alternative to be carried forward for detailed analysis. Other considerations include social, environmental, and logistical consequences. After the range of structural alternatives for a high hazard structure was narrowed, non-structural alternatives were identified along with a structural alternative to rehabilitate the dam to a lower hazard classification. The non-structural alternatives and the structural alternative to rehabilitate the dam to a lower hazard classification were then compared to the structural alternative.

1.3 PUBLIC / INTERAGENCY MEETING

The Public and Interagency meeting was held on July 19th, 2021 in Larimore, North Dakota. The purpose of the meeting was to review the deficiencies for Larimore dam and identify the categories of practical structural rehabilitation alternatives that should be considered. The categories of alternatives that were discussed included principal spillway modifications, widening the auxiliary spillway, various spillway armoring options, and foundation filter modifications. Participants were invited to a tour following the meeting. Feedback from the meeting was used to further-develop the alternatives. The main purpose of the meeting was to focus on the structural alternative, therefore, the no-action, structural rehabilitation to a lower hazard classification, and non-structural alternatives were not discussed in significant detail.

2 ALTERNATIVE IDENTIFICATION AND DEVELOPMENT

2.1 RANGE OF ALTERNATIVES

The range of alternatives that are considered for dam rehabilitation are categorized as:

- Future Without Federal Investment
- No Action
- Non-Structural or Decommissioning
- Structural
- Lower Hazard Classification

The Future Without Federal Investment (FWOFI) and no action alternatives are to be carried forward for additional analysis. The FWOFI is discussed within the main body of the Watershed Plan – Environmental Assessment [The FWOFI is discussed in detail in Section 5.4]. The no action alternative, the methods used to develop the no action breach scenario, and the resulting damage from the no action alternative is described in Section 5.5 of this report. The other non-structural alternative evaluated for Larimore Dam is a full decommissioning of the dam, discussed in Section 5.3.

2.2 STRUCTURAL ALTERNATIVES

The structural alternatives that were considered consist of various modifications to the principal spillway, auxiliary spillway, and embankment to bring Larimore Dam into compliance with dam safety standards for a high hazard structure for the rehabilitation design life. Modifications considered for Larimore Dam are discussed in the following sections. Modifications to the principal spillway are only to address design life expectancy as principal spillway capacity requirements are all met. The auxiliary spillway consists of modifications to the exit channel, which addresses integrity design of the vegetated earth spillway, i.e. limiting headcut development causing breach of the level crest during passage of FBH/IDF. Modifications to the embankment are only to the drain fill filter in order to meet NRCS's filter design procedure, as all seepage and slope stability analyses meet minimum NRCS factor of safety recommendations (see **Appendix D-2** – Geotechnical Engineering Report).

2.2.1 PRINCIPAL SPILLWAY

As noted in **Appendix D-1** (Existing Conditions Assessment Report), the current principal spillway does not meet PSH criteria and is approaching the end of its original design life of 50 years (2027). The results of the corrosion analysis undertaken in November of 2021 of conduit reinforcing steel show that service life estimates for a moderate corrosion rate range from 6 to 20 years, and thus the conduit would be a candidate for replacement to accommodate a 100-year extension of design life.

If the GFCWRD elects not to replace the principal spillway, then they must take on the responsibility of replacing the principal spillway within the conduit's design life. At a minimum, for alternatives that do not pursue replacement of the principal spillway, the upstream first stage orifice needs to be larger, i.e. same size as downstream opening, and a filter diaphragm would need to be constructed around the existing conduit in conjunction with the replacement of the foundation drain. The steel plates would need to be replaced and the conduit joints would need to be sealed as well. In order to properly convey any seepage, the filter diaphragm would need to feed into the proposed two-stage filter system outlined in Section 2.2.3. For purposes of developing the conceptual cost comparison, it was assumed with each alternative the principal spillway conduit would be replaced with 6.5-foot diameter round culvert with boring and jacking method with first stage upstream opening the same as downstream (4.5' x 2.5').

2.2.2 AUXILIARY SPILLWAY EXIT CHANNEL

Four practical auxiliary spillway armoring or lining options were identified for the auxiliary spillway exit channels, including earthen grass-lined spillways, articulated concrete block (ACB) armored spillways, roller compacted concrete (RCC) structural spillways, and reinforced concrete chute structural spillways. While earthen grass-lined spillways are generally more economical lining options, they are not always feasible based on the landscape and sub-surface soil conditions. Structural lining treatments typically provide the highest level of erosion resistance; however, they may not be the most economical solution to meet dam safety standards.

For the alternatives analysis, each spillway armoring or lining option was conceptually designed to meet hydrologic criteria defined by *TR 210-60* (NRCS, 2019) using appropriate NRCS guidance material. The RCC structural spillways were designed using guidance material developed by ASCE and the Portland Cement Association. Design guidance material used for the structural alternatives is listed in the **References** section of this report. From the Existing Conditions Report (**Appendix D-1**), the FBH peak flow through the existing auxiliary spillway is 14,854 cfs, which used 1-D HEC-RAS routing calculations. SITES and HEC-RAS 2-D auxiliary spillway runs have slightly different results due to attenuation within spillway and routing calculations; all input assumptions are the same but peak flow results are as low as 13,550 cfs down the chute. For design purposes, the conservative 14,854 cfs was used for sizing spillway and features, but for determination of differences between 2-D and 1-D results due to curve the correlating values were used.

The Larimore Dam auxiliary spillway has two steep sections that cause supercritical flow dynamics followed by hydraulic jumps. The existing auxiliary spillway profile and FHB maximum Water Surface Elevation (WSE) are documented in **Figure D-3-1**. The second/downstream drop has larger descent (~35 feet) and steeper (~15.2%) than the first/upstream drop, plus curves ~15 degrees between the transition from second level section to descent. Due to this 15-degree curve in the existing auxiliary spillway, flow dynamics are better represented with a 2-Dimensional (2-D) hydraulic model rather than typical 1-Dimensional (1-D) model. Centrifugal forces and momentum cause super-elevation and differential velocity and shear stresses from 1-D cross section mean values. However hydraulic variations are sensitive due to curve geometry, channel width, and flow rates. Therefore, a 2-D HEC-RAS model of auxiliary spillway assumed variable grid cell with average size of 310 square feet to increase the detail of the 15-degree curve. The roughness was set to 0.035, based on 2015 Assessment where tall grass was present. The 2-D model results indicate the existing bend occurs right as the flow begins to go supercritical, which forces more flow on the right side of the spillway. The 2-D maximum shear stresses is 22.7 lb/sq ft and velocity is 30.7 ft/s; FBH maximum velocity model results are included in **Figure D-3-1**. Normal depth 1-D hydraulic calculations find the maximum shear stresses is 17.1 lb/sq ft and velocity is 24.3 ft/s, see **Figure D-3-2**. The 2-D HEC-RAS results to account for the 15-degree bend show the maximum velocity is 26.1% and shear stress is 30.8% greater than 1-D hydraulic computation, which has no bend. Therefore, the 2-D results were used to analyze the effectiveness of armored ACB on the auxiliary spillway where this bend is maintained. The largest ACB blocks typically used in the region (individual block surface area of 1.34 square feet and 134 lb weight) do not meet required NEH Part 628, Chapter 54 required factor of safety of 2.0, see **Figure D-3-2**. Therefore, decreasing slope and removing bend are adjustments to increase factor of safety discussed in Section 3; widening is not included because any extra width would remove 15-degree curve, which in itself allows factor of safety to be met.

2.2.3 EMBANKMENT

The downstream embankment for Larimore Dam does was found to satisfy all slope stability criteria for the flood surcharge pool under steady state conditions. However, the foundation drain fill material does not meet the standard of practice for drains.

The proposed lean clay fill for the original dam construction was assumed not to pipe and to take a considerable amount of time to saturate (beyond the dam's design life), therefore a foundation drainage system was constructed on the downstream side of the dam to relieve seepage pressures in the dam's foundation caused by water bypassing the cutoff. The foundation drain fill was designed to be compatible with the alluvium soils at the proposed foundation elevation. The alluvium at the proposed foundation elevation was found to consist of silty sand, silt and lean clay material. Upon reviewing the original boring data and the most recent data obtained from the geologic investigation, foundation soils appear to predominantly consist of lean clay soils rather than the assumed alluvium materials. This lean clay at the base of the dam was assumed not to pipe, and therefore the foundation drain fill gradation was designed to be compatible with the silty sand encountered in the base alluvium soils during the original geotechnical exploration, rather than the lean clay soils encountered. Furthermore, initial crumb tests performed for the Geotechnical Engineering Report (**Appendix D-2**) show that embankment and foundation soils are potentially dispersive and therefore have the ability to pipe, contrary to the assumption made during original dam construction.

The filter compatibility analysis completed by HDR indicates the constructed filter is not compatible with the material it is meant to filter at the foundation of the dam, and does not account for dispersive embankment fill or foundation samples. Additional sample collection and testing will be necessary to confirm the uniformity of the foundation alluvium soils, and the dispersive properties of the embankment and foundation soils. Once the additional exploration and testing is complete, the foundation drain can be designed to conform to NRCS NEH Chapter 26 Gradation Design of Sand and Gravel Filters. The locations of the drains are shown in **Attachment 1** of the Existing Conditions Report (**Appendix D-1**). Notes from the as-built drawings indicate the drains consist of perforated and non-perforated asbestos-cement piping, and the fill consists of granular material with 25 percent to 50 percent retained on the No. 4 sieve and 100 percent passing the 2-inch sieve.

For the purposes of this report, all structural alternatives assume the foundation and embankment soils are dispersive, and therefore a two-stage chimney and blanket filter system will be constructed to properly convey any seepage in the dam. In the event soils are found to be non-dispersive, in accordance with standard practice, a filter diaphragm will still be constructed around the principal spillway conduit. The estimated cost difference between a chimney filter or filter diaphragm is minimal. The phreatic surface developed by HDR in the Geotechnical Engineering Report (**Appendix D-2**) was used to estimate the size of the filter systems. Calculating the concept level costs for the downstream embankment modifications as it relates to filter design will not influence the selection of the alternative(s) carried forward for further review. Therefore, costs for modifications to the downstream embankment related to filter criteria concerns are included in the conceptual level comparative cost estimate for each alternative identically.

2.3 UNIT PRICES

Part of this analysis includes developing concept level cost estimates for various alternatives. Unit prices were based on local knowledge of prior projects in the region, dam rehabilitation projects constructed through NRCS planning around the United States, and supplier-provided costs for ACBs. The unit prices used for the different construction elements considered for the concept level costs are provided in **Table D-3-2**.

***Table D-3-2**: Unit Prices Used for Various Work Items*

Item	Unit	Unit Price	Assumptions
Stripping and Topsoiling	CY	$5.00	Includes stripping and topsoiling of material.
Earthwork - Excavation	CY	$6.00	Includes excavation of material to be re-used as fill.
Earthwork - Fill	CY	$5.00	Includes placement, compaction, final grading and seeding.
Excavation Waste	CY	$10.00	Includes excavation, haul, spoiling, and stripping and topsoiling of material.

Item	Unit	Unit Price	Assumptions
Imported General Fill	CY	$9.00	Includes hauling, placement, compaction, final grading and seeding.
Imported Embankment Fill	CY	$14.00	Includes hauling, placement, compaction, final grading and seeding.
Structural Concrete	CY	$1,000.00	Includes Concrete, reinforcing steel, form work, and subgrade preparation
RCC	CY	$350.00	Includes batch plant, materials, mixing, conveying, placing and curing
ACBs (EPEC SD 475 OCT)	SF	$11.50	Includes ACB and all geosynthetics
ACBs (EPEC System 900 OCT)	SF	$15.00	Includes ACB and all geosynthetics
ACB - Stone Drainage Layer	CY	$20.00	Drainage Layer required under ACB
Clearing and Grubbing	AC	$9,000.00	Includes all incidentals associated with clearing and grubbing.
Land Acquisition - Not Farmed	AC	$500.00	Full purchase required for construction. No flowage easements
Land Acquisition - Farmed	AC	$1,000.00	Full purchase required for construction. No flowage easements

As discussed in Section 2.2, conceptual cost estimates in the following section assume the principal spillway will be replaced via an open cut method, and the foundation drain system will be replaced with a two-stage chimney and blanket drain system. Furthermore, all of the alternatives that place the auxiliary spillway over the embankment of the dam assume the principal spillway replacement, foundation drain replacement, and auxiliary spillway construction will take place concurrently.

The estimated cost for replacing the principal spillway and foundation drain system with a chimney and blanket drain system is $6,998,700. If the GFCWRD elects not to replace the principal spillway at this time, each of the alternatives' costs can be reduced by $4,703,600. This figure assumes the minimum construction items listed in Section 2.2.1 will be performed in lieu of the complete replacement of the conduit. However, the GFCWRD would then bear the full responsibility of replacing the principal spillway conduit within its design life.

3 STRUCTURAL ALTERNATIVES IDENTIFICATION

The practical structural alternatives that were identified for Larimore Dam are shown in **Table D-3-3** and are described in more detail in the following sections. The structural alternatives provided in **Table D-3-3** include modifications to the auxiliary spillway, principal spillway, and embankment.

***Table D-3-3**: Larimore Dam – Structural Alternatives*

Larimore Dam Structural Alternatives							
Hydraulic Control		Auxiliary Spillway Exit Channel		Other Considerations		Option	Cost
Broad Crested	→	Earthen	→	Widen Aux. Spillway to 1250 ft	→	1	$67,088,800
	→	ACB	→	Keep bend, 11% slope	→	2	$13,024,600
			→	Straighten bend, keep 15% slope	→	3	$14,447,900
	→	RCC	→	200 ft width in Aux. Spillway	→	4	$14,394,200
			→	200 ft width over dam embankment	→	5	$14,259,800
	→	Reinforced Concrete	→	200 ft width in Aux. Spillway, no baffle blocks	→	6	$13,638,900
			→	200 ft width in Aux. Spillway, baffle blocks	→	7	$13,215,300
			→	200 ft width over dam embankment, no baffle blocks	→	8	$14,476,400
			→	200 ft width over dam embankment, baffle blocks	→	9	$13,197,500

3.1 ALTERNATIVE 1 – EARTHEN SPILLWAY 1,250' WIDE

This alternative includes a broad crested control section with an earthen auxiliary spillway. The control section and spillway would need to be widened from the current geometry of 300 feet to 1,250 feet to pass the freeboard hydrograph without a complete breach of the spillway, assuming no flow through principal spillway. The need for the 1,250-foot-wide control section is largely driven by erodibility of subsoils. The glacial till soils encountered near the surface in the auxiliary spillway have relatively high erodibility, as they have been deposited relatively recently in geologic terms and have not been consolidated by overburden. Widening the spillway control section to 1,250 feet and eliminating the bend at the transition from level crest to drop will reduce unit flow rates in exit channel and prevent a complete breach of Larimore Dam. The SITES model results for this scenario are documented in **Figure D-3-2**. A conceptual drawing of the alternative is shown on **Figure D-3-3**. This alternative would also require modifications to existing land uses in the area, which would result in several acres of agricultural land being taken out of production.

3.2 ALTERNATIVE 2 – ARMORED ACB, KEEP BEND, 11% SLOPE

This alternative includes a broad crested hydraulic control section with an Articulated Concrete Block (ACB) lined auxiliary spillway. The control section and spillway maintain the existing 300-foot width located north of the existing embankment. The individual block size requirements include surface area of 1.34 square feet and 134 lb weight; all other block assumptions are documented in **Figure D-3-4**. The critical hydraulic computations from Section 2.2.2 assume this alternative maintains the existing alignment, including the 15-degree curve. The profile is modified to remove first drop and only have a single slope. In order to meet NEH Part 628, Chapter 54 required factor of safety of 2.0, the critical slope is 11.0%; ACB computations are documented in **Figure D-3-4.** The specific energy at the hydraulic jump is 15.6 feet. The termination basin requires tapered ACB revetment with 3-dimensionally stabilized stone due to specific energy greater than 8.03 feet, but less than 17.2 feet. The minimum required length of ACB in termination basin is 46.4 feet. A conceptual drawing of the alternative is shown on **Figure D-3-5**.

3.3 ALTERNATIVE 3 – ARMORED ACB, KEEP 15% SLOPE, NO CURVE

This alternative includes a broad crested hydraulic control section with an ACB-lined auxiliary spillway. The control section and spillway maintain the existing 300-foot width located north of the existing embankment, with the alignment modified to remove the existing 15-degree curve. Alignments straightening the auxiliary spillway to the south and north were both evaluated. The costs for both alignments were comparable, but ultimately the alignment extending the auxiliary spillway to the north was selected because it required less land acquisition and did not affect the golf course located to the south of the auxiliary spillway while maintaining stable side slopes.

The individual block size requirements include surface area of 1.34 square feet and 134 lb weight; all other block assumptions are documented in **Figure D-3-5**. The critical hydraulic computation from 1-D normal depths are assumed as this alternative maintains the similar profile of 15.2 percent slope. Both drops in the existing auxiliary spillway are maintained. With assumed block size NEH Part 628, Chapter 54 required factor of safety of 2.0 is met; ACB computation are documented in **Figure D-3-6.** The specific energy of the first drop hydraulic jump is 6.3 feet, while that of the second drop is 11.8 feet. The first drop does not require a termination stilling basin, or further stabilization features as the specific energy is below 8.03 (Thornton & Nadeau). However, the second drop requires tapered ACB revetment with 3-dimensionally stabilized stone due to specific energy greater than 8.03 feet, but less than 17.2 feet. The minimum required length of ACB in termination basin is 42.2 feet. A conceptual drawing of the alternative is shown on **Figure D-3-7.**

3.4 ALTERNATIVE 4 – RCC CHUTE, IN CURRENT AUXILARY SPILLWAY

This alternative includes a broad crested hydraulic control section with a RCC chute placed within the existing auxiliary spillway alignment. The control section maintains the existing 300-foot width and is tapered to the 200-foot spillway width via concrete wingwalls just upstream of the RCC chute. The required spillway width is ~195 feet, which is rounded up to nearest 10-foot increment. The resulting reservoir water surface elevation is 1099.6 feet, which provides State of North Dakota required 1.0 foot of residual freeboard. RCC design calculations followed procedures and equations outlined in “Simplistic Design Methods for Moderate-Sloped Stepped Chutes” (Hunt et all, 2014). All RCC computations are summarized in **Figure D-3-8**; inputs are highlighted in yellow, recommended wall heights are in blue. The potential energy dissipation due to steps is expected to be 67%.

The Froude Number arriving at the stilling basin is 4.4, which is considered an oscillating jump, or in the transition stage because a true hydraulic jump has not fully developed. There is potential for sweepout within an oscillating jump. The Type IV (Alternative Low Froude) stilling basin applicable Froude number range is 2.5-4.5 and has a large factor of safety against sweepout (USBR, 1984). Other basin types do not meet both the applicable Froude number and unit discharge ranges, and St. Anthony Falls spillways are similar to Type III but have lower factor of safety, and therefore were eliminated from design consideration. In order to match river tailwater elevation from HEC-RAS modeling with conjugate depth elevation, the stilling basin invert is required to be at 1028 feet; this is lower than the floodplain and will need to be considered for excavations and de-watering. All Type IV (Alternative Low Froude) stilling basin computations are summarized in **Figure D-3-9.** A conceptual drawing of the alternative is shown on **Figure D-3-10**.

3.5 ALTERNATIVE 5 –RCC CHUTE OVER TOP OF DAM

This alternative includes a broad crested hydraulic control section with a RCC chute spillway placed over the top of the existing embankment, to the north of the principal spillway and south of the existing auxiliary spillway. The control section and spillway are 200 feet wide, and the current auxiliary spillway is closed off with a new dam section. The required spillway width is ~195 feet, which is rounded up to nearest 10-foot increment. This alternative follows very similar design and calculations to Alternative 4 (Section 3.4), with the exception of the floodplain elevation at termination basin. All RCC computations are summarized in **Figure D-3-11**; inputs are highlighted in yellow, recommended wall heights are in blue. The potential energy dissipation due to steps is expected to be 64%.

The Froude Number arriving at the stilling basin is 4.4, which is considered an oscillating jump, or in the transition stage because a true hydraulic jump has not fully developed. There is potential for sweepout within an oscillating jump. The Type IV (Alternative Low Froude) stilling basin applicable Froude number range is 2.5-4.5 and has a large factor of safety against sweepout (USBR, 1984). Other basin types do not meet both applicable Froude number and unit discharge ranges, and St. Anthony Falls spillways are similar to Type III but have a lower factor of safety, and were therefore eliminated from design consideration. In order to match river tailwater elevation from HEC-RAS modeling with conjugate depth elevation, the stilling basin invert is required to be at 1033 feet; this is lower than the floodplain and will need to be considered for excavations and de-watering. All Type IV (Alternative Low Froude) stilling basin computations are summarized in **Figure D-3-12.** A conceptual drawing of the alternative is shown on **Figure D-3-13**.

3.6

3.7 ALTERNATIVE 6 – REINFORCED CONCRETE CHUTE, IN CURRENT AUXILARY SPILLWAY WITH TYPE II STILLING BASIN

This alternative includes a broad crested hydraulic control section with a reinforced concrete chute within the existing auxiliary spillway alignment, located to the north of the dam embankment. The control section retains the existing 300-foot width, and is tapered to 200 feet via concrete wingwalls just upstream of the entrance of the concrete chute. The required spillway width is ~195 feet, which is rounded up to nearest 10-foot increment.

The Froude Number arriving at the stilling basin is 11.8, which is considered to develop a strong jump with the potential for strong surface waves downstream from the jump. The Type II stilling basin applicable Froude number is >4.5, and unit discharge is < 500 cfs/ft. Other basin types do not meet both applicable Froude number and unit discharge ranges and were therefore eliminated from design considerations, and St. Anthony Falls spillways are similar to Type III but have lower factor of safety; baffle blocks are not recommended for incoming velocities > 60 cfs/ft due to large impact forces and potential for cavitation (USBR, 1984). In order to match the river tailwater elevation from HEC-RAS modeling with conjugate depth elevation, the stilling basin invert is required to be at 1027 feet; this is lower than the floodplain and will need to be considered for excavations and de-watering. All Type II stilling basin computations are summarized in **Figure D-3-14.** A conceptual drawing of the alternative is shown on **Figure D-3-15**.

3.8 ALTERNATIVE 7 – REINFORCED CONCRETE CHUTE, IN CURRENT AUXILARY SPILLWAY, BAFFLE BLOCKS

This alternative includes a broad crested hydraulic control section with a reinforced concrete chute with baffle blocks within the existing auxiliary spillway alignment. The control section retains the existing 300-foot width, and is tapered to 200 feet via concrete wingwalls just upstream of the entrance of the concrete chute. The required spillway width is 195 feet, which is rounded up to nearest 10-foot increment. To maintain free flowing inlet weir dynamics and coefficient, the broad crested weir shall have a small drop to flat apron and then baffle block chute to start. Baffle blocks are placed in alternating rows so that flow that passes through an opening in one row will hit the next row of baffles. All computations relating to this alternative are summarized in **Figure D-3-16.** A conceptual drawing of the alternative is shown on **Figure D-3-17**. Riprap should be placed at the downstream portion of the chute to prevent erosion (USBR, 1984).

3.9 ALTERNATIVE 8 – REINFORCED CONCRETE CHUTE, OVER TOP OF DAM WITH TYPE II STILLING BASIN

This alternative includes a broad crested hydraulic control section with a reinforced concrete chute spillway placed over the top of the existing embankment, to the north of the principal spillway and south of the existing auxiliary spillway. The control section and spillway are 200 feet wide, and the current auxiliary spillway is closed off with a new dam section. The required spillway width is 195 feet, which is rounded up to nearest 10-foot increment.

The Froude Number arriving at the stilling basin is 11.8, which is considered to develop a strong jump with potential for strong surface waves downstream from the jump. The Type II stilling basin applicable Froude number is >4.5, and unit discharge < 500 cfs/ft. Other basin types do not mee both applicable Froude number and unit discharge ranges, and St. Anthony Falls spillways are similar to Type III but have lower factor of safety; baffle blocks are not recommended for incoming velocities > 60 cfs/ft due to large impact forces and potential for cavitation (USBR, 1984). In order to match river tailwater elevation from HEC-RAS modeling with conjugate depth elevation, the stilling basin invert is required to be at 1027; this is lower than the floodplain and will need to be considered for excavations and de-watering. All Type II stilling basin computations are summarized in **Figure D-3-18.** A conceptual drawing of the alternative is shown on **Figure D-3-19**.

3.10 ALTERNATIVE 9 – REINFORCED CONCRETE CHUTE, OVER TOP OF DAM, BAFFLE BLOCKS

This alternative includes a broad crested hydraulic control section with a reinforced concrete chute spillway placed over the top of the existing embankment, to the north of the principal spillway and south of the existing auxiliary spillway. The control section and spillway are 200 feet wide, and the current auxiliary spillway is closed off with a new dam section. The required spillway width is ~195, which is rounded up to nearest 10-foot increment. To maintain free flowing inlet weir dynamics and coefficient, the broad crested weir shall have small drop to flat apron and then baffle block chute to start. Baffle blocks are placed in alternating rows so that flow that passes through an opening in one row will hit the next row of baffles. All computations concerning this alternative are summarized in **Figure D-3-20.** A conceptual drawing of the alternative is shown on **Figure D-3-21**. Riprap should be placed at the downstream portion of the chute to prevent erosion (USBR, 1984).

3.11 STRUCTURAL ALTERNATIVES IDENTIFICATION SUMMARY

At the sponsor meeting held on February 5th, 2025, the sponsor recommended Alternative 2 (Section 3.2) to be carried forward for detailed analysis. The summary of alternatives, including considerations, numbering, and costs are documented in Table D-3-3, which is at the beginning of section 3. There are three alternatives that are very close in price, including alternatives 2 (ACB), 7 (concrete chute in auxiliary spillway), and 9 (concrete chute overtop embankment). Since the costs were very similar, discussion and preference to Alternative 2 (ACB) focused on it being the lowest cost and strong performance of ACB chutes in North Dakota. Strong performance relates to flexibility of mats during operation and non-operations, i.e. annual freeze thaw and piping. Secondary benefits discussed included keeping auxiliary spillway flows entering North Branch Turtle River, quicker turnaround with design and construction, and issues with roller compacted concrete and reinforced concrete in North Dakota.

4 ADDITIONAL CONSIDERATIONS

4.1 PRINCIPAL SPILLWAY

For the conceptual cost comparative analysis completed, all alternatives assumed the principal spillway conduit would be removed using boring and jacking method. A new riser tower will be constructed to adapt to the new conduit shape and dimensions.

As discussed in Sections 2.2.1 and 2.3, replacing the principal spillway conduit would be necessary to extend the design life of the dam and was therefore assumed for development of each of the alternatives. If the GFCWRD had not elected to replace the conduit with this project, they must take the responsibility of replacing the conduit within the estimated design life. This is estimated to be 6 to 20 years from the time of the last corrosion testing conducted in 2021. If the conduit is not replaced, at a minimum a filter diaphragm must be constructed and tied to the proposed two-stage filter system to properly convey seepage, the steel plates must be replaced and the joint sealed.

4.2 EMBANKMENT

Further analysis of the embankment and foundation soils is required to confirm the dispersivity of the embankment and foundation. However, based on initial soil test results, the soils encountered in the embankment and foundation are potentially dispersive and therefore have the capability to pipe, which was counter to the assumption made at the time the dam was constructed. Furthermore, the analysis performed by HDR showed that the designed filter material is not compatible with the embankment soils encountered during the field investigation. Therefore, for all of the conceptual costs prepared for each alternative, it was assumed that the existing foundation drain would be replaced with a drain system consisting of a two-stage chimney drain and blanket drain. A chimney, blanket and toe drain filter system is recommended in FEMA's *Filters for Embankment Dams (2011)* for embankment and/or foundation soils composed of dispersive soils (Table 2-1).

4.3 ENVIRONMENTAL IMPACTS

Fen wetlands are recognized to be an aquatic resource that cannot be reproduced (Environmental Protection Agency, 2008). Fen wetlands are groundwater driven with pH above 5.5, meet soil histic epipedon which is considered muck or peat, and growth exceeds decomposition and decomposition progresses very slowly. Therefore, fens are in rare groundwater locations and typically take hundreds of years to form required soils; and are essentially irreplaceable (US Department of Interior, 1999). Five Fen wetland exists immediately upstream of the normal reservoir pool.

Since none of the alternatives analyzed alter the existing invert elevations for the principal spillway, auxiliary spillway or embankment, and conduit change only affects floods larger than 50-year recurrence frequency, there will be no practical impact expected on the fen wetlands from existing conditions. The 50-year frequency, 4-day duration flood event was analyzed for existing 6x6-foot box and proposed 6.5-foot diameter spillway conduit of three lowest fen wetlands, including W55, W60, and W64. The results are identical for peak water surface elevation and duration above wetland boundary, see Table D-3-4 for details. The peak reservoir level of 1085.64 is above second stage inlet but below where inlet control changes to conduit controlling flow rates. Therefore, 50-year and smaller floods have no changed hydrology to the fen wetlands. Larger floods, i.e. 100-year, have slightly higher water levels and longer durations than existing conditions, but these very infrequent flood events and minor changes to depth and duration are not expected to change the composition of the fen wetlands.

Table D-3-4: Larimore Pool Fen Analysis for 50-year Flood Event

<table>
<tr><th rowspan="2">Wetland ID</th><th rowspan="2">~Invert Elevation</th><th rowspan="2">~Boundary Elevation (Low End)</th><th colspan="2">Peak WSEL</th><th colspan="2">Duration above boundary</th></tr>
<tr><th>Existing-(6'x6')</th><th>Prop -6.5' Diameter</th><th>Existing-(6'x6')</th><th>Prop -6.5' Diameter</th></tr>
<tr><td>W60</td><td>1072.5</td><td>1074.5</td><td rowspan="3">1085.64</td><td rowspan="3">1085.64</td><td>7d 4 hrs</td><td>7d 4 hrs</td></tr>
<tr><td>W64</td><td>1078.2</td><td>1080</td><td>4d 21.5 hrs</td><td>4d 21.5 hrs</td></tr>
<tr><td>W55</td><td>1083</td><td>1084</td><td>2d 11 hrs</td><td>2d 11 hrs</td></tr>
</table>

5 ALTERNATIVES CARRIED FORWARD FOR DETAILED ANALYSIS

The structural alternatives outlined in Section 3 were presented to the project sponsor on February 5, 2025 for additional discussion. Discussions and decisions on whether to carry alternatives forward or eliminate them during the February 5th meeting are described in the following Sections 5.1 through 5.5. Other alternatives described in this section include the structural rehabilitation to a lower hazard classification and the non-structural alternatives that were considered.

5.1 ALTERNATIVE 2 – ARMORED ACB, KEEP BEND, 11% SLOPE

During the sponsor meeting held on February 5, 2025, the members present at the meeting decided that Alternative 2 was selected as the recommended structural alternative to be carried forward for discussion within the supplemental watershed plan. Alternative 2 was the least costly structural alternative analyzed. Members of the Grand Forks County Water Resource District indicated that the lowest cost alternative was the most appealing from their perspective.

5.2 CHANGE TO SIGNIFICANT HAZARD CLASSIFICATION

Larimore Dam is currently a high hazard dam because there are 15 habitable structures and 18 overtopped roadways downstream of the dam that would have a high susceptibility to loss of life if the dam were to breach using TR-60 requirements. See the *Existing Conditions Assessment Report* for Larimore Dam in **Appendix D-1** for additional information on hazard classification and downstream risk associated with the existing structure. The change from high to significant hazard lowers the inflow peak and volume so that all design criteria are met. **Table D-3-1** summarizes the existing conditions assessment, which indicated that Auxiliary Spillway Integrity criteria is not met and a breach of the dam is expected. The significant hazard criteria 48-hr FBH rainfall is 12.30 inches, which results in a peak inflow of 6,435 cfs and a peak reservoir level elevation of 1094.96 feet. Auxiliary spillway integrity is maintained from the breach, as 357 feet of the level section remains after routing the FBH through the spillway, see **Figure D-3-22**.

A change from high to significant hazard classification of Larimore Dam would involve property buy-outs for the parcels that contain the habitable structures, and constructing larger culverts or bridges for roads that are overtopped. Many of the parcels that contain habitable structures are farm sites with other agricultural buildings on the property. Several of the habitable structures would have flood depths that exceed 8 feet during a dam breach scenario. There is likely negative social fallout from floodproofing these structures by means of ring levees or raising the structures for a dam breach of such rare frequency likelihood, as described in Section 5.10.1. There are also likely environmental concerns (i.e. Endangered Species Act) with ring levees due to abundant trees adjacent to the structures. Therefore, floodproofing was considered impractical and was not considered for this alternative; rather property buy-outs were used. The cost that was developed for purchasing the property with the habitable structures was based on the replacement value for residential structures presented in Exhibit D5.1 in **Appendix D-5**.

The cost to buy out the properties with habitable structures is assumed to be $7,524,000. The assumptions for this calculation include 15 structures, roughly 2,000 sq. ft per structure with a replacement value of $167.20 per sq ft., and a contents value of 50% of the structure value. The total cost to raise overtopped roadways is assumed to be $72,000,000. The assumptions for this calculation include 18 roads and bridges that need to be raised 3 to 10 feet or replaced with structures with increased capacity to match the breach overtopping flows. There is significant variability as flow rates immediately downstream of the dam due to a potential breach are extremely high (~88,400 cfs), but reduce downstream due to attenuation and storage. Designing a bridge opening for these extremely high flow rates immediately downstream of the dam would likely be very expensive, but bridges further downstream would be much less. Therefore, a conservative estimate for all 18 bridges used is $4 million/bridge. The hazard class change would allow reduction to the dam height by roughly 5 feet, which only reduces breach peak flow rate from high hazard calculation of 110,000 to 88,400 cfs. The total cost to change hazard class to significant hazard is approximately $79.5 million, which is considerably more than structural alternatives discussed in Section 3. For this reason, rehabilitating Larimore Dam to a lower hazard classification was removed from further consideration.

5.3 DECOMMISSIONING

An alternative involving the full decommissioning of Larimore Dam in combination with non-structural measures to provide some replacement for the current flood prevention benefits provided by the dam was considered. As summarized in **Appendix D-5**, Table 16, the incremental crop damage contributions at the flood events are variable. However, nonstructural measures to protect cropland would not be reasonable beyond the 25-year due to expansive extents, tying into high ground, and increased flood damages downstream. **Figures D-3-23** through **D-3-30** show the proposed plan and profile of the decommissioning with non-structural measures alternative, details of the channel cross section and grade stabilization of the alternative, and the location of the downstream protection elements considered for the alternative. The overall cost of this alternative was calculated to be approximately $24.5 million. Ultimately, this alternative was eliminated from further evaluation due to the low benefit- cost ratio, as well as negative environmental and social consequences associated with it. The following sections describe the various elements of the decommissioning with non-structural measures alternative.

5.3.1 RIVER ALIGNMENT AND CORRIDOR

The channel alignment for the river corridor through the Larimore Dam reservoir was developed using historic graphics and imagery of the South Branch Turtle River before the construction of Larimore Dam. The channel upstream of Larimore Dam follows the historic channel, which was traced from relic Digital Raster Graphics (DRG), also called USGS topographic maps. The historic centerline was validated with imagery available in Grand Forks County taken between 1957 and 1962 (International Water Institute Map Portal). The proposed channel alignment throughout the reservoir is shown in **Figure D-3-23** and **Figure D-3-24**.

The channel dimensions used for the proposed river corridor were determined based on the Rosgen stream classification design parameters using regional curves. Regional curves have been developed for many areas of the country, with some based on certain rivers, while others for regions. Those plotted in NEH 654.11 and 654.TS3C include the San Francisco Bay region in CA, the Eastern U.S., Upper Green River, WY, and Upper Salmon River, ID. There are also many others developed by practitioners for other regions, including nearby states Minnesota and Montana. The variations are predominately due to annual average precipitation and landscape characteristics. The curves correlate drainage area to bankfull area, width, depth, and flow. Regional bankfull curves for use in North Dakota have been developed based on USGS gauge station and corresponding field measurements at five sites. The North Dakota curves, along with nearby and similar rainfall region curves, are plotted in **Figure D-3-25**, including best fit line equations for North Dakota curves. There is a strong agreement between all of the curves presented in this figure, which validates the North Dakota curve for use in this planning design. The most important result from these curves for the South Branch River is the bankfull area, which is 60 square feet. From the corresponding curves, bankfull width is rounded to 30 feet and the average bankfull depth is 2.0 feet.

Appendix D-8 (Stream Classification and Riparian Area Assessment & Addendum) documents five channel cross sections taken of the South Branch Turtle River, and one along Turtle River. Four of the cross sections – three riffles and one pool – are upstream of the reservoir and, one riffle is downstream of the reservoir along South Branch Turtle River and one along Turtle River. The entrenchment ratio at these cross sections ranges from ~3.4 to 7.2 (75 feet – 150 feet/~30 feet), which classifies the stream type as “C” or “E”. There are some reaches where the floodplain width is even larger, notably upstream of the reservoir, where historic channels have extended the floodplain width. Channel cross sections determined the Width/Depth ratio ranges from 7.9 to 16.1; values below 12 are located upstream of the reservoir, while values above 12 are located downstream of the dam. Therefore, the stream classification is “E” upstream of the reservoir and “C” downstream of dam. The “E” cross sections average bankfull width is 22 feet, average depth is 2.6 feet, and the average D_{max}/D_{ave} ratio is 1.2. These values vary slightly from regional curve values, as the measured depth is greater, and width less; however measured values (width and depth) are given more weight in design parameters because regional curves are best fit lines.

The proposed channel is assumed to follow form from upstream of the reservoir, rather than downstream. The upstream reach will need to seamlessly transition to the decommissioned reach. The downstream reach will still have a grade control structure at dam. Therefore, the upstream decommissioned channel will be stream type “E”, with bankfull area of 60 square feet, width of 22 feet, and average depth of 2.6 feet. The floodplain width used for the proposed channel is approximately 150 feet, which was measured using LiDAR in floodplains along stream upstream and downstream of Larimore Dam. Therefore, the riffle maximum depth for typical cross section is 3.1 feet. The profile of the proposed channel was based on LiDAR data collected adjacent to the dam and outside of the reservoir footprint. The proposed typical section of the restored channel is shown on **Figure D-3-26**.

The bankfull floodplain elevation immediately downstream of dam is 1038.5 feet, therefore the channel invert should be 1035.5 feet. The existing downstream channel slope is ~0.4%, which remains for the design downstream of the dam. According to **Appendix D-7** (Accumulated Sediment) Figure 5, the sediment thickness near the dam and throughout the majority of reservoir ranges from 0 – 1.5 feet; however there is increased thickness in the delta in the western portion of the reservoir. To maintain accumulated sediment in place, the floodplain and channel upstream of dam are assumed to be conservatively 1.5 feet higher than downstream grade line. Therefore, sheetpile cutoff walls and grade stabilization are required to connect these separate grade lines; see Section 5.3.2 for details. The grade line throughout reservoir will follow the existing downstream grade line (0.4%), which will meet the South Branch Turtle River at the head of the normal reservoir pool of 1068.4 feet. The ~4,500 foot reach of South Branch Turtle River just above flood pool (1068.5 feet to 1071.5 feet) has much lower slope grade line (0.045%) than that below dam, which could be result of deposition, natural grade change, or a combination of both. Either way, consideration should be given to grade change; which is why three cross vanes are included in the design. One cross vane shall be at normal pool (1068.4 feet), with another ~500 feet upstream and another ~500 feet downstream of normal pool cross vane. The cross vane at normal pool shall include sheetpile for additional support.

The channel downstream of the sheetpile cutoff walls and grade stabilization is <10% of the entire alignments; in order to tie into downstream “C” channel, this section should also be a “C” channel. If decommissioning becomes the preferred alternative this 700’ downstream section should be fleshed out further, in order to account for over-widened outlet, armoring, golf course considerations, and geomorphologically stable channel considerations.

After channel restoration construction has been completed, the floodplain will be seeded with vegetation native to the project area. The minimum required floodplain shall be 105 feet, which aligns with the lower range of observed stable entrenchment ratios upstream and downstream of the dam; this entrenchment ratio is 3.5 ft/ft. Materials excavated from the sediment that has accumulated in the reservoir are to be hauled off site to ensure that chemical runoff from the sediment is not introduced into the river system. All channel dimensions that were determined are based on a riffle section of the river. Pools within E type river channels are approximately 1.6 times larger in cross sectional area than riffle sections based on E type channel data obtained for several rivers in North Dakota, as well as the cross sections documented in Appendix D-8 (Stream Classification and Riparian Area Assessment & Addendum). To account for river pools, the total excavation through the proposed channel corridor that was computed for the decommissioning alternative was multiplied by 1.3, assuming similar length pools and riffles.

5.3.2 GRADE STABILIZATION

A rock riprap section would be placed where the existing embankment is located to provide controlled passage of flood flows through the embankment. A riffle section helps improve fish passage and reduces sediment migration downstream. Sheet pile will be installed at a depth of 5 feet at the beginning and end of the proposed riprap section (riffle) to ensure that the riprap ramp is not undercut in any way. The grade of the proposed riffle is 5.5% with a total elevation drop of approximately 17.5 feet. The riffle footprint is 120 feet wide, approximately 300 feet long, and approximately 36 inches deep. The width of the proposed riffle was selected to keep the 100-year flood event (Peak flow = 1,737 cfs) at or below the current bankfull maximum channel depth of 3.0 feet. Details of the grade stabilization section are shown in **Figure D-3-26**.

5.3.3 DOWNSTREAM FLOOD PROTECTION

The South Branch Turtle River reach downstream of Larimore Dam is ~3,300 feet in length, and the floodplain is ~1,600 feet; at this point the North Branch Turtle River and South Branch combine to form the Turtle River. The North Branch Turtle River watershed is 180.1 square miles, whereas the South Branch Turtle River watershed is 65.5 square miles. Therefore, the North Branch Turtle River has considerably greater drainage that contributes to a significant portion of hydrologic inputs, including flow rates, to the Turtle River and downstream portion of Larimore Dam. Much of the South Branch Turtle River floodplain downstream of the dam is shared with the North Branch Turtle River and Turtle River. Furthermore, the dam auxiliary spillway terminates in North Branch Turtle River floodplain, rather than South Branch Turtle River. In summary, the downstream benefits to Larimore Dam are affected by storage in the dam, but also to a large degree by the North Branch Turtle River watershed.

Frequency flows are adopted from the design hydrographs used in the Watershed Plan/Environmental Assessment for the Upper Turtle River Dam #10 under the Watershed Flood Prevention Operations (WFPO) Program, which is referenced in **Appendix D-1 (Existing Conditions Assessment Report)**. Notable existing conditions are the peak flow rates for the 1% ACE (100-year flood); from the North Branch Turtle River the peak flow rate is 6,090 cfs, while South Branch Turtle River is 715 cfs, and the combined is ~6,600 cfs. As noted in Section 5.7.2, the peak inflow to Larimore Dam is 1,737 cfs, which is the same flow that would occur if the dam were decommissioned. Therefore, Larimore Dam regulation reduces the peak flow by ~1,000 cfs. Accordingly, the decommissioning alternative evaluates inundation increases due to this 1,000 cfs, increase, resulting in a peak flow at the confluence of ~7,600 cfs compared to the existing flow of 6,600 cfs. The hydraulic model results indicate expanded inundation extents, as reflected in **Figure D-3-27.** This figure identifies the 100-year flood existing conditions maximum depth grid with blue histogram coloring and the decommissioned dam maximum depth grid with red histogram coloring. The decommissioned dam histogram varies based on topography, but the typical additional width is approximately 50 additional feet from the existing histogram.

There are 18 downstream structures listed in **Table D-3-5** that would be inundated by at least 0.1 feet with the decommissioning of Larimore Dam, and nine levees that would need to be put in place to protect the structures. The location of the levees downstream of Larimore Dam are shown in **Figure D-3-28**. Levees would be put around the structures to protect them up to a 100-year flood event with Larimore Dam removed. Alignment considerations include proximity to structures, the river, trees, and resident/farm operations. The total length of levees is approximately 4,800 feet. The levees are designed to have a 10-foot top width and 4:1 side slopes down to existing ground, and tie into high ground at beginning and termination. The levees would be seeded after construction is completed. Some sites require pipes with flap gates to allow for one directional flow through the levee embankment. Excavation, embankment placement, culverts, topsoiling and seeding quantities were developed for each levee.

Table D-3-5*: Infrastructure Affected by Decommissioning of Larimore Dam*

Decommissioning 100-yr Flood Hydraulics						
Structure		Finished Floor	Max WSEL (ft)		Max Depth (ft)	
ID	Type	WSEL (ft, NAVD88)	Existing Conditions	w/o Larimore Dam	Existing Conditions	w/o Larimore Dam
*S56	Shed	1031	1030.96	1031.1	0	0.1
S20	Garage	990.5	990.4	991.8	0	1.3
S21	Grain Bin	990.5	991.1	993.3	0.6	2.8
S22	Grain Bin	990.8	991.2	993.3	0.4	2.5
S23	Grain Bin	990.3	991.3	993.3	1	3
S24	Grain Bin	991	991.4	993.3	0.4	2.3
S25	Grain Bin	990.9	991.4	993.3	0.5	2.4
S26	Grain Bin	991.6	991.9	993.3	0.3	1.7
S29	Shed	991.9	992.2	993.3	0.3	1.4
*S60	Grain Bin	992.7	991.5	993.3	0	0.6
S30	Garage	975.5	975.9	976.6	0.4	1.1
S31	Shed	975.1	976.3	976.8	1.2	1.7
S63	Shed	976.3	976	976.7	0	0.4
S36	Garage	971.1	970.8	972	0	0.9
*S70	Cabin	958.8	959.3	959.7	0.5	0.9
S47	Garage	911.7	913.6	913.9	1.9	2.2
S79	Garage	909.1	909.6	910	0.5	0.9
*S93	Garage	861.3	862.4	862.5	1.1	1.2

Setback levees were also considered for the protection of downstream agricultural land, however were determined to be impractical. The additional inundation is typically 50 feet or less, and a levee would take a high portion of this additional width out of production. The existing conditions inundation and flow paths breakout of the channel and floodplain around Grand Forks Air Force Base, causing extensive breakout flows to the north. Levees are not practical to limit breakout flows because this would result in unnaturally higher flows to remain in the Turtle River floodplain, which would cause increased flood depth, inundation, velocities, and likely flooding, erosion and deposition, notably to the Cities of Mekinock and Manvel.

An analysis of the first 18 bridges downstream of Larimore Dam was completed in order to evaluate existing and decommissioned conditions in terms of meeting state design standards. North Dakota Century Code

Chapter 24-3 states bridge construction shall meet the stream crossing standards with sufficient capacity to permit the water flow freely and unimpeded under the bridge Stream crossing design flood frequencies are documented in North Dakota Century Code Chapter 89-14-01, which range from 15-year for nonmajor roads, 25-year for major collectors, and 50-year for arterials. For this analysis, all the roads within Turtle River basin are considered Couty-Rural-Major Collectors with the exception of Hwy 2 (State Highway System-Rural-Principal-Other). The correlating stream crossing frequency is used to evaluate and determine adequacy, which is considered unimpeded flow under the low bridge chord. A summary of the bridge analysis is included in **Table D-3-6.** An analysis of further bridges downstream was not considered necessary due to breakout flows reducing main channel flow to very similar values downstream of 21st Ave NE for existing conditions and decommissioning. There are three bridges (28th St., 21st Ave., and 23rd Ave) that meet design standards for existing conditions, however would not with Larimore Dam decommissioned. The remaining currently meet and continue to meet standards with Larimore Dam decommissioned, or do not currently meet and would continue not to meet standards with dam decommissioning. Therefore, these three bridges that change adequacy would need to be re-built with greater hydraulic capacity. The same assumption of $4 million/bridge from Section 5.2 is assumed, which for three bridges would result in an approximate cost of $12 million to address bridge adequacy related to decommissioning Larimore Dam. The locations of the three bridge replacements are shown in **Figure D-3-28.**

***Table D-3-6**: Bridges Affected by Decommissioning of Larimore Dam*

Decommissioning Bridge Hydraulic Design Standards									
Crossing			Frequency	Existing Conditions			Decommissioned		
Roadway	Type*	Low Chord Elevation	(year)	Flow (cfs)	WSEL	Adequate	Flow (cfs)	WSEL	Adequate
35th St NE	C, R, MC	1026.7	25	3,523	1027.6	No	4,407	1028.3	No
Local (17th Ave)	C, R, MC	990.1	25	3,730	988.4	Yes	4,570	989.0	Yes
32nd St NE	C, R, MC	974.0	25	3,767	972.0	Yes	4,606	973.1	Yes
Hwy 2	SHS, R, PA, O	978.0	50	4,997	972.4	Yes	6,111	973.4	Yes
30th St NE	C, R, MC	926.0	25	3,612	931.3	No	4,146	931.8	No
20th Ave NE	C, R, MC	921.3	25	3,986	924.7	No	4,391	925.4	No
29th St NE	C, R, MC	909.3	25	3,180	912.1	No	3,584	912.9	No
28th St NE	C, R, MC	903.7	25	3,916	903.1	Yes	4,430	904.5	No
21st Ave NE	C, R, MC	894.4	25	4,500	893.9	Yes	5,081	895.5	No
27th St NE	C, R, MC	882.8	25	2,712	887.8	No	2,949	888.2	No
23rd Ave NE (CR11)	C, R, MC	878.2	25	3,915	877.7	Yes	4,139	878.5	No
26th St NE	C, R, MC	867.5	25	1,366	869.1	No	1,394	869.2	No
24th Ave NE	C, R, MC	865.6	25	1,694	866.9	No	1,718	867.0	No
25th St NE (CR 3)	C, R, MC	858.1	25	2,352	858.3	No	2,374	858.5	No
24th Ave NE (West)	C, R, MC	849.5	25	1,754	853.3	No	1,799	853.4	No
24th Ave NE (East)	C, R, MC	847.8	25	2,003	852.4	No	2,085	852.5	No
24th St NE	C, R, MC	847.3	25	2,185	850.8	No	2,250	851.0	No
24th Ave NE	C, R, MC	844.3	25	2,772	846.8	No	2,897	847.1	No
* C-County, SHS-State Highway System, R-Rural, MC-Major Collector, PA-Partial Arterial, O-Other									

5.3.4 DECOMMISSIONING PROBABLE COST

Unit costs for recent construction projects were utilized to develop unit prices for the various items needed to complete the earthwork necessary to re-establish the Upper Turtle River through the former Larimore Dam reservoir, and to construct the grade stabilization through the former dam embankment. Similarly, unit costs to construct the protection levees were applied to the engineer's opinion of probable cost. The estimated costs for the bridge replacements and home buyouts are detailed in Sections 5.2 and 5.3.3. The locations for the structures impacted, and levees and bridge replacements are shown in **Attachments D-3-22** and **D-3-28.** The costs for all elements to construct the federal decommissioning alternative were compiled and are provided in **Table D-3-7**.

***Table D-3-7**: Decommissioning with Non-Structural Measures: Engineer's Opinion of Probable Construction Cost*

No.	Item	Unit	Quantity	Unit Price	Total Price
1	Mobilization	LS	1	$800,000.00	$800,000.00
2	Clearing and Grubbing	AC	10	$9,000.00	$90,000.00
3	Stripping and Topsoiling	CY	5,000	$5.00	$25,000.00
4	Erosion Control and Revegetation	AC	42	$1,200.00	$50,400.00
5	Embankment Excavation	CY	125,000	$25.00	$3,125,000.00
6	Excavation	CY	280,000	$6.00	$1,680,000.00
7	Excavation Material Haul	CY	180,000	$10.00	$1,800,000.00
8	Fill material	CY	98,000	$5.00	$490,000.00
9	Sheetpile Cutoff	SF	2,000	$20.00	$40,000.00
10	Rip Rap (NDDOT Grade II 28")	CY	100	$100.00	$10,000.00
11	Levees - Land Acquisition	AC	2	$1,000.00	$2,000.00
12	Levees - Stripping and Topsoiling	CY	6500	$5.00	$32,500.00
13	Levees - Embankment Fill	CY	60,000	$14.00	$840,000.00
14	Levees - Seeding	AC	3	$1,200.00	$3,600.00
15	Levees - Culverts with Flap Gates	EA	8	$1,500.00	$12,000.00
16	Riser Tower - Removal of Existing Structure	LS	1	$30,000.00	$30,000.00
17	Removal of Existing Principal Spillway	LF	304	$100.00	$30,400.00
18	Bridge Replacements	EA	3	$4,000,000.00	$12,000,000.00
19	Construction Dewatering	LS	1	$100,000.00	$100,000.00
Construction Cost					**$21,160,900.00**

5.4 FUTURE WITHOUT FEDERAL INVESTMENT (FWOFI)

The NRCS National Watershed Program Manual (Part 501, Subpart B, 501.12) (NRCS, 2014), dictates that the FWOFI alternative must be presented in the watershed plan. FWOFI would involve a breach of the existing dam and removal of the outlet works associated with the dam; this work is similar to that described in Section 5.3 (Decommissioning), without stabilizing the river corridor throughout the reservoir or providing downstream flood protection. Riprap and sheet piling would be used to minimize sediment transport downstream. Implementation costs would be substantially less than decommissioning alternative, but economic damages during flooding substantially higher as a result. Ultimately, this alternative was eliminated from further evaluation due to the low benefit-cost ratio, as well as negative environmental and social consequences associated with it. Removal of the dam without additional downstream flood protection would create additional risk to public safety and property downstream and is counter to the objectives of the project. A detailed breakdown of cost and benefit-cost calculations are provided in **Table D-3-8.**

***Table D-3-8**: Decommissioning without Non-Structural Measures: Engineer's Opinion of Probable Construction Cost and Benefits*

No.	Item	Unit	Quantity	Unit Price	Total Price
1	Mobilization	LS	1	$200,000.00	$200,000.00
2	Clearing and Grubbing	AC	10	$9,000.00	$90,000.00
3	Stripping and Topsoiling	CY	5,000	$5.00	$25,000.00
4	Erosion Control and Revegetation	AC	42	$1,200.00	$50,400.00
5	Embankment Excavation	CY	170,000	$25.00	$4,250,000.00
6	Embankment Fill	CY	500	$14.00	$7,000.00
7	Sheetpile Cutoff	SF	1,200	$20.00	$24,000.00
8	Rip Rap (NDDOT Grade II 28")	CY	700	$100.00	$70,000.00
9	Riser Tower - Removal of Existing Structure	LS	1	$30,000.00	$30,000.00
10	Removal of Existing Principal Spillway	LF	304	$100.00	$30,400.00
11	Construction Dewatering	LS	1	$100,000.00	$100,000.00
Construction Cost					**$4,876,800.00**

5.5 NO-ACTION ALTERNATIVE

The no action alternative represents a scenario where the existing dam remains in place with no measures taken to address the dam safety inadequacies associated with the dam. In this scenario, the dam would remain in place until it fails, and would not be subsequently rebuilt. This alternative is required to ensure that all reasonable alternatives are considered based on guidance in Title 390 – National Watershed Program Manual, Part 500 Watershed Program Management, Amended 2024 (NRCS, 2024). Additional information on the analysis completed for the no-action alternative is available in the following sections. Discussion on the environmental impacts and economic implications of the no action alternative are described in more detail in the Watershed Plan – Environmental Assessment.

5.5.1 NO ACTION BREACH EVENT

The analysis described in the *Existing Conditions Assessment Report* (**Appendix D-1**) shows that Larimore Dam would breach during the probable maximum precipitation event (PMP). To assess the no action alternative, an additional analysis was completed to determine the highest frequency rainfall event that would cause a breach of the dam. Various rainfall depths were applied to the hydrologic model and then the resulting hydrograph was routed through the dam using the NRCS's Water Resources Site Analysis Computer Program (USDA and Kansas State University, 2014), which is commonly referred to as the SITES program. The auxiliary spillway of Larimore Dam breached when a rainfall depth of 14.5 inches was applied to the upstream watershed. Conversely, the dam did not breach when 14.0 inches of rain was applied. This was the minimum 24-hour rainfall depth applied to the models that resulted in a breach of the auxiliary spillway of Larimore Dam. The NRCS's "Dam Failure Probability Estimation Tool" was then utilized to attain the return interval associated with the 14.25 inches of rain. The resultant return interval for the breach event was 1,844-year (this is a rainfall event that has a 0.05% chance of occurrence). The 1,844-year rainfall event and breach of Larimore Dam were then routed through the hydraulic model.

5.5.2 BREACH HYDROGRAPH

The breach hydrograph at Larimore Dam resulting from the 1,844-year rainfall event was developed using methods similar to what is described in Section 4.1 of the *Existing Conditions Assessment Report* (**Appendix D-1**), which involved using equations in Chapter 1 of *Technical Release 210-60 Earth Dams and Reservoirs* (NRCS, 2019). The peak water surface elevation that occurs during the 1,844-year event is approximately 1095.7 and the resultant peak breach discharge is 96,100 cubic feet per second.

The downstream water surface profiles for the dam breach were developed using the HEC-RAS modeling program (U.S. Army Corps of Engineers, 2024). The same HEC-RAS model described in the *Existing Conditions Assessment Report* was used to route flows for the no action breach event. Tools within the HEC-RAS framework were utilized to develop the dam breach simulation. To meet the peak breach discharge, the elevation of the reservoir was set to the breach elevation (1095.7), and the breach formation time within the hydraulic model was altered to yield a peak flow of 96,100 cubic feet per second. The progression of the breach occurred at a sine wave rate. The breach formation characteristics such as breach width, side slopes, and temporal characteristics were based on the Froehlich Equations (Froehlich, 2008). The breach HEC-RAS model consisted of 14.5 inches of rainfall applied to the UTR9 watershed, but did not include a coincidental peak on the mostly unregulated upper portion of the Upper Turtle Watershed. To account for this additional flow the 1844-yr Breach was combined with the 1844-yr and 500-yr preferred alternative impacts to account for the unregulated upper portion of the Upper Turtle Watershed.

5.5.3 BREACH RESULTS

5.5.3.1 POPULATION AT RISK

The inundation produced from the simulated breach based on *TR 210-60* criteria is shown through the breach zone on **Figure D-3-29**. All 32 residential structures that were potentially impacted by the dam breach are summarized in **Table D-3-9,** with 27 roads summarized in **Table D-3-10**; "High Danger Zone" structures are labeled in **Figure D-3-29** as well. The structures shown in **Table D-3-9** are the only potential residential structures impacted by the no action breach event within the breach zone; structures which people reside regularly within Turtle River State Park are also included. The flood depth and velocity for these structures was also evaluated to determine if the potential for loss of life exists for the no action breach event. The chart from *Downstream Hazard Classification Guidelines* that shows the depth-velocity flood danger level relationship for homes built on foundations is shown on **Figure D-3-29**.

Structures plotted in the red fall in the category of high danger level, indicating that loss of life is likely. Structures in the yellow fall into what is called the judgement zone and were assumed to have risk for loss of life for this analysis. Structures plotted in the green area have a low danger level, and loss of life is not likely. **Figure D-3-29** shows that there are 14 structures in the high danger (red) zone and two structures in the judgment (yellow) zone. There are a total of 14 structures where lives are at risk from the No Action, therefore, and 3 lives per structure were assumed base on NRCS Population at Risk computation procedures, resulting in 42 lives at risk. There are a total of 16 local roads and 1 Federal Highway (Hwy 2) where lives are at risk from the No Action, assuming 2 lives per local road and 8 lives per Federal Highway based on NRCS Population at Risk computation procedures, resulting in 40 lives at risk. Therefore, 82 lives are at risk from selection of the No Action Alternative. Repair costs due to inundation damages to all 40 residential structures (as well as other categories of structures) and 55.4 miles of roads are described in the *Economics Evaluation* in **Appendix D-5**.

Table D-3-9: Residential Structures Impacted by a Breach of Larimore Dam During the 1,844-year Rain Event

House/Lodge				
	Label	Depth (ft)	Velocity (ft/s)	Arrival Time (hours)
House/Lodge	S1	10.9	4.6	0.5
House/Lodge	S4	7.5	6.1	0.4
House/Lodge	S5	5.7	2.6	0.5
House/Lodge	S12	6.7	6.1	1.6
House/Lodge	S19	9.9	3.5	1.5
House/Lodge	S30	12.7	3.5	1.9
House/Lodge	S32	1.6	2.2	2.6
House/Lodge	S33	1.3	1.4	2.6
House/Lodge	S35	6.0	4.9	2.1
House/Lodge	S37	6.3	6.5	2.5
House/Lodge	S38	7.2	6.8	2.4
House/Lodge	S50	4.1	0.9	9.3
House/Lodge	*S58	11.3	5.2	0.7
House/Lodge	*S59	10.0	4.4	1.8
House/Lodge	*S62	9.2	6.2	1.9
House/Lodge	S63	7.3	4.0	1.9
House/Lodge	S64	7.8	6.0	1.9
House/Lodge	*S70	0.6	7.0	2.3
House/Lodge	S71	2.4	1.4	2.5
House/Lodge	S74	1.8	0.9	2.9
House/Lodge	*S84	0.9	2.0	6.4
House/Lodge	*S85	1.0	0.7	7.8
House/Lodge	*S88	1.1	0.9	9.4
House/Lodge	*S89	0.7	0.3	9.8
House/Lodge	*S97	0.4	1.8	9.7
House/Lodge	*S98	0.6	2.4	9.3
House/Lodge	*S100	0.6	1.4	9.4
House/Lodge	*S102	0.3	0.7	9.3
House/Lodge	*S106	0.4	0.8	9.3
House/Lodge	*S114	1.4	1.4	11.0
House/Lodge	*S117	1.9	1.3	10.6
House/Lodge	*S119	1.0	0.3	11.1

***Table D-3-10**: Roads Impacted by a Breach of Larimore Dam During the 1,844-year Rain Event*

Road Overtopping				
Name	Road ID	Depth (ft)	Velocity (ft/s)	Arrival Time (hours)
35th St	R1	11.1	9.3	0.4
18th Ave	R2	8.4	9	0.6
32nd St	R3	6.6	9.1	1.9
Hwy 2	R4	7.2	5.4	2.0
Park Ave	R5	6.9	9.7	2.5
Wood St	R6	7.8	7.9	2.5
Forest St	R7	9.8	4.1	2.9
30th St	R8	6.3	8.6	3.4
20th Ave	R9	7.8	6.6	3.6
29th St	R10	8.3	7.7	4.2
21st Ave	R11	6.3	5.6	5.5
28t St	R12	3	4.1	5.0
27th St	R13	9	5	5.9
22nd Ave	R14	7.1	3.4	6.2
26th St	R15	0.6	3	8.5
27th St	R16	3.3	3	7.4
24th Ave	R17	1	3.5	9.5
23rd Ave	R18	1.4	5.4	9.2
25th St	R19	1.4	4	10.9
25th Ave	R20	1	1.2	13.0
24th Ave	R21	4.5	2.5	11.5
24th St	R22	3.2	2	12.0
23rd St	R23	0.2	2.5	12.5
27th Ave	R24	2.3	1.1	13.0
26th St	R25	2	2.4	9.0
Nye St	R27	0.3	1.7	9.7

5.5.3.2 OTHER DAMAGES ASSOCIATED WITH THE NO ACTION BREACH

The landscape at the dam site and downstream of the dam would also be negatively impacted by a breach of Larimore Dam. The amount of sediment eroded from the auxiliary spillway of the dam during the no action breach event was estimated to be 552,500 cubic yards. Breach dimensions were developed based on the Froehlich Equations (Froehlich, 2008), however adjusted to meet TR-60 peak breach flow. The result is bottom width of 200 feet. Froehlich equations also assume a 1:1 (horizontal:vertical) side slope for the headcut area. The assumed footprint that the breach would create within the existing auxiliary spillway is shown on **Figure D-3-30**. The sediment volume eroded during the breach was computed assuming that the breach would stop after it reaches the upstream valley floor. The headcut would continue to advance upstream through accumulated sediments that have deposited in the reservoir of Larimore Dam as time goes on. Eventually, the headcut would progress to the middle of the reservoir. The amount of sediment that erodes from the continued headcut up to the middle of the reservoir was estimated to be ~198,000 cubic yards based on approximate headcut path and dimensions. All 560,000 cubic yards of sediment eroded from the spillway, and any additional material that is eroded from the deposited sediment in the reservoir, would be deposited somewhere downstream of the dam. Some sediment deposits would continue on through the channel and be deposited within the Turtle and Red River, but most of the deposits would likely settle out in the floodplain. Sediment deposited in the floodplain would have negative impacts to the adjacent grass and forest land, as well as the agricultural land.

There is also the potential for erosion and scouring in the floodplain during the breach event as the breach flood wave travels downstream. Concepts from Chapter 51 – Earth Spillway Erosion Model of Part 628 in the National Engineering Handbook (NRCS, 2014) were used to develop an estimate for erosion in the downstream floodplain that would occur as a result of the no action breach event. Equation 51-2 from Chapter 51 was used to develop an erosion rate of sediment in the floodplain. The erosion rate is a function of allowable shear stress, the shear stress on the surface, and a detachment rate coefficient. Allowable (or critical) shear stress for the floodplain was determined using concepts from Chapter 8 – Threshold Channel Design of part 654 in the National Engineering Handbook (NRCS, 2007). Allowable shear stress is a function of soil type, plasticity index, and void ratio of the soil. The surface horizon is assumed from Web Soil Survey (WSS) (NRCS, 2019), which returned values for plasticity index and soil type. The void ratio is not available in WSS, therefore the value of 0.9 is assumed for all soils based on typical values for lacustrine deposits. Equation 51-8 is used to calculate detachment rate coefficient (k_d), which used WSS surface horizon parameters for dry bulk density and percent clay. ArcGIS tools were used to create grids for allowable shear stress and detachment rate coefficient in the floodplain of the Turtle River watershed. The shear stress on the surface of the floodplain is estimated based on output from the hydraulic model that was used for the existing conditions analysis. The hydraulic model is described in more detail in the *Existing Conditions Assessment Report* in **Appendix D-1**.

The analysis was simplified by assuming that only agricultural land would be affected by erosion because of limited vegetal cover. Agricultural land use areas were identified using grids available from the National Agricultural Statistics Service (NASS). With gridded data for the detachment rate coefficient, the allowable shear stress throughout the floodplain, and the shear stress on the surface resulting from the no action breach event, the erosion rate was calculated for the no action breach event on agricultural land. Shear stress grids are developed at two-hour time increments for the first 8 hours and 4-hour increment from 8 to 24 hours from start of breach from the hydraulic model output. The erosion rate was computed for each of the eight time increments, and then summed to get the total erosion expected across the landscape. The resultant cumulative erosion throughout the floodplain is shown in **Figure D-3-30**. Areas shown as blue on the map experience inundation but will not be susceptible to erosion based on the allowable shear stress values calculated or because there is assumed to be vegetal cover that would prevent the erosion. Many portions of the Turtle River riparian area does not experience erosion because the floodplain is forested, contains hydrophytic vegetation, or grazed for cattle with perennial grasses in that area. The highest erosion rates are at locations immediately downstream of the dam and where the floodplain slopes away from the river, the terrain is steeper, and the land use is agricultural.

The expected erosion volume at incremental depths was determined to assess the damage caused by erosion. The deeper the incision on the landscape, the lower the value for agricultural production. **Table D-3-11** shows the volume of erosion for each half-foot increment up to a depth of 5 feet. The majority of the erosion that occurs in agricultural production areas as a result of the breach is less than one foot (72%). Approximately 95% of the erosion volume is expected to be at depths less than three feet. The total, cumulative volume of material that is eroded from the floodplain is estimated to be 523,000 cubic yards based on the results shown in **Figure D-3-30** and in **Table D-3-11**.

The soil material eroded from the floodplain and from the auxiliary spillway of Larimore Dam would then be transported downstream and deposited somewhere either in the floodplain or in one of the downstream river channels. A high-level analysis was completed to determine an estimated quantity of how much of the eroded material would be deposited on agricultural land. The analysis involved comparing the shear velocity of the no-action breach event to the settling (terminal) velocity of the sediment that was eroded from the landscape. The shear velocity was determined for various timesteps on the trailing limb of the hydrograph for the no action event. Shear velocity was computed using the relationship provided in equation 8-19 in *Chapter 8 – Threshold Channel Design* within Part 654 – Stream Restoration Design of the National Engineering Handbook (NRCS, 2007). The settling velocity of the soil particles eroded from the landscape was determined using Stokes' Law. The settling velocity of soil particles using Stokes' Law requires information on soil density and particle size. Soil density information was obtained from the USGS Web Soil Survey. The particle size of the soils was estimated based on the dominant soil types present in the downstream floodplain area that is impacted by breach inundation. The dominant soil types in the downstream floodplain are silt loams, clay loams, and silty clay loams based on a review of the soil types using the Web Soil Survey. Furthermore, an investigation of the soils in Grand Forks County by Bluemle (1973) found that till samples had an average of 6 percent gravel, 30 percent sand, 40 percent silt, and 24 percent clay. Therefore, the median particle size for the downstream floodplain was assumed to be a silt material, which typically range in size from 0.05 millimeters to 0.002 millimeters. The average particle diameter was assumed to be 0.026 millimeters.

***Table D-3-11**: Incremental Erosion Depth and Volume Resulting from No-Action Breach Event*

Erosion Depth Range	Cubic Yards of Incremental Volume (% of Total)	Cubic Yards of Cumulative Volume (% of Total)
0.0 - 0.5	292,219	292,219
	55.9%	55.9%
0.5 - 1.0	86,680	378,898
	16.6%	72.4%
1.0 - 1.5	52,582	431,481
	10.0%	82.5%
1.5 - 2.0	29,764	461,244

	5.7%	88.2%
2.0 - 2.5	21,235	482,479
	4.1%	92.2%
2.5 - 3.0	14,660	497,140
	2.8%	95.0%
3.0 - 3.5	9,734	506,874
	1.9%	96.9%
3.5 - 4.0	6,551	513,425
	1.3%	98.1%
4.0 - 4.5	3,455	516,879
	0.7%	98.8%
4.5 - 5.0	3,571	520,450
	0.7%	99.5%
5	2,769	523,220
	0.5%	100.0%

Comparison of the settling and shear velocities showed that approximately 69% of the eroded soil material would be deposited on agricultural land. A map of deposition areas resulting from No Action breach is summarized in Figure D-3-32, including red coloring for agricultural land and yellow for non-agricultural areas. An assumption was made that sediment deposits on agricultural land would be cleaned up or removed. All deposits in the river channel and floodplain that is non-agricultural were assumed to be left in place, resulting in impacts to habitat from the sediment deposition. The soil material eroded from the auxiliary spillway (552,500 cubic yards) along with the erosion that was calculated for the floodplain (523,200 cubic yards) would leave a total amount of material suspended in the system of approximately 1,075,700 cubic yards. With 69% of the eroded material being deposited on agricultural land, a total volume of 747,400 cubic yards would need to be removed from the floodplain initially after breach. In following years, the headcut erosion through reservoir would erode another ~198,000 cubic yards of soil; the total erosion amount from initial breach, floodplain erosion, and subsequent years headcut erosion is 1.3 million cubic yards of soil.

Stream channels and wetlands downstream of the dam within D-AA are susceptible to erosion and deposition due to breach. Some stream banks and wetlands would experience erosion from the high erosive currents during peak, others by deposition during receding limb of breach hydrograph, and some both erosion and subsequent deposition. The National Wetland Inventory (NWI) was used to compute potential wetlands effects due to vast size of D-AA. The NWI indicates there are ~7,129 wetlands, accounting for 15,486 acres, which could be affected to varying degrees ranging from minimal disturbance to complete removal. However, NWI maps Turtle River stream channel, which is an "Other Waters" according to 1987 Corps Manual. Therefore, the Turtle River channel length in miles is considered for erosion. Allowable velocity for erosion was assumed to be 5 ft/s based on grass mixture cover, <5% slope, with erosion resistant soils, as listed in Table 8-6 in *Chapter 8 – Threshold Channel Design* within Part 654 – Stream Restoration Design of the National Engineering Handbook (NRCS, 2007). Maximum velocities from hydraulic model breach event are compared to allowable velocity to estimate potential erosion of stream channels and wetlands. Allowable velocities are exceeded, resulting in erosion potential, of 15.9 miles of streams and 5.1 acres of wetlands.

Sediment deposition is determined spatially based on typical sediment particles (silt), hydraulic model shear stresses, and Stokes's Law for settling velocity. Approximately 1,323 wetlands, accounting for 999.2 acres, would fill with sediment ranging in depths from 0 to 2 feet of fill. Additionally, rivers and creeks categorized as "other waters" would fill with varying degrees of sediment ranging in depth from 0 to 4 feet of deposition. **Table D-3-12** shows the estimated significant area (1,004.3 acres) and wetland functional losses from both scour and sediment deposition in the downstream analysis area. Depressional wetland class functional values are less than total area due to reductions resulting from deviation from pristine conditions of average current conditions. The remaining 14,482 acres of wetland may have affects from breach, but are expected to be much less severe than 1,004 acres that are predicted to erode or deposit significant amounts of sediment based on breach hydraulic modeling.

Table D-3-12*: Breach Wetland Functional Losses in D-AA*

Area (acres)	1,004.3
Static	664
Dynamic	586
Cycling	642
Removal	537
Retention	565
Plants	697
Structure	610
Habitat	259

5.5.4 NO-ACTION BREACH PROBABLE COST

Costs to repair and clean up the watershed downstream of Larimore Dam as a result of the no-action breach event were estimated. Estimated quantities for repair and clean up work are based on the erosion and deposition analyses completed and described in Section 5.10.3.2. The estimated cost for the various activities that would be required for the repair and clean up work are provided in **Table D-3-13** below.

Table D-3-13*: Estimated Costs Associated with No-Action Breach Event*

Item	Unit	Quantity	Unit Price	Total Price
Rework of Soil (Erosion Areas <1.0 feet)	AC	6,560	$ 85.00	$ 558,000.00
Embankment Haul and Placement (Erosion Areas >1.0 feet)	CY	144,300	$ 15.00	$ 2,165,000.00
Topsoil Preparation (Erosion Areas >1.0 feet)	CY	144,300	$ 15.00	$ 2,165,000.00
Clean Up of Soil Deposition on Cropland	CY	747,400	$ 10.00	$ 7,474,000.00
Rework of Soil in Deposition Zones on Cropland	AC	3,280	$ 85.00	$ 279,000.00

Item	Unit	Quantity	Unit Price	Total Price
Construction Total			**$**	**12,641,000.00**

Areas with less than one foot of erosion depth were assumed to only require tilling practices to recover the agricultural land. Three passes were assumed for reworking the agricultural land affected by erosion depths less than one foot. There were approximately 6,560 acres of agricultural land that experienced erosion depths of less than one foot. Embankment was assumed to be hauled onto site in areas where erosion depths exceeded one foot. Topsoil for areas experiencing erosion depths greater than one foot would also need to be imported. The assumed depth of topsoil to be hauled to areas with excessive erosion was one foot. Clean up is also assumed to be necessary for sediment deposition areas. All of the deposited soil would need to be removed and hauled off site. Tilling practices to recover ag land in the depositional areas were also assumed. Overall, the total clean up and repair cost associated with the no-action breach event is $12,641,000, amongst other ecological consequences. The cost for cleanup associated with the no-action breach event (1,844-year flood) is not included for other flood events due to the extreme nature of the breach.

6 SUMMARY – ALTERNATIVES CARRIED FORWARD

6.1 STRUCTURAL REHABILITATION

There are nine structural alternatives identified and analyzed, which are described in Section 3. There is one earthen, two armored, and six structural auxiliary spillways. The armored and structural spillways have similar expected cost amounts, see Table D-3-3. The only structural alternative to be recommended to be carried forward for detailed review is Alternative 2 described in Section 5.1. Alternative 2 involves the use of articulated concrete block (ACB) within the auxiliary spillway, replacing the existing principal spillway with a new riser tower and conduit, two-stage chimney drain and blanket drain, and widening top of dam. This alternative was evaluated in greater detail during the following phases of the planning effort. Refer to **Appendix D-4: Concept Design Report** for additional information on Alternative 2.

6.2 NO-ACTION

The no-action alternative described in Section 5.5 was carried forward for detailed review as required by current NRCS policy. While carried forward for detailed review, the alternative was not considered viable due to the significant impacts associated with an uncontrolled breach, most notably placing 42 lives at risk. The uncontrolled breach evaluated as part of the no-action alternative also has significant immediate and long term environmental and economic impacts associated with the erosion and deposition of 1,189,900 cubic yards earthen material.

6.3 FUTURE WITHOUT FEDERAL INVESTMENT (FWOFI)

The FWOFI alternative carried forward for detailed analysis is the minimal decommissioning described in Section 5.4. Due to the impacts associated with the no-action alternative the local sponsor or the state dam safety authority (ND Department of Water Resources) would not allow the no-action alternative to occur. Rather, a least-cost alternative to mitigate the downstream flood risk would be more likely. Regarding Larimore Dam, the minimally decommissioning the embankment to mitigate downstream risk with the least associated costs. While this alternative would meet the purpose reducing downstream risk associated with an uncontrolled breach, it would also increase flooding during the 2-year through 500-year flood events.

7 REFERENCES

AASHTO. (2018). *A Policy on Geometric Design of Highways and Streets* (6th ed.). Washington, DC: AASHTO.

FEMA. (2011). *Filters for Embankment Dams, Best Practices for Design and Construction.* Retrieved from https://www.fema.gov/sites/default/files/2020-08/filters_embankment_dams_update.pdf

Froehlich, D. C. (2008). Embankment Dam Breach Parameters and Their Uncertainties. *ASCE, Journal of Hydraulic Engineering, Vol. 134, No. 12*, 1708-1721.

Hunt, S.L, Kadavy, K.C., Hanson, G.J. (2014). Simplistic Design Methods for Moderate-Sloped Stepped Chutes. ASCE, Journal of Hydraulic Engineering, Vol. 140, Issue 12, December 2014.

North Dakota Department of Water Resources. (n.d.). Aerial Imagery Map Service. Bismarck, ND, USA. Retrieved from https://aerial.swc.nd.gov/swcwms?

North Dakota Legislature. (2015, January). Article 89-14 - Public Highway Stream Crossings. Bismarck, ND.

NDDWR. (2024). North Dakota Dam Safety Standards, Policy Reg_05.2024

NRCS. (2007, August). National Engineering Handbook Part 654 Stream Restoration Design; Chapter 8: Threshold Channel Design.

NRCS. (2014, February). National Engineering Handbook Part 628 Dams; Chapter 51: Earth Spillway Erosion Model.

NRCS. (2019). *Technical Release 210-60: Earth Dams and Reservoirs.* NRCS.

NRCS. (2019). TR 210-60. *Earth Dams and Reservoirs*.

NRCS. (2019, 7 31). Web Soil Survey. Retrieved from https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

NRCS. (2022, December). Part 303 - Clarification and Instructions for the No-Action Alternative in Supplemental Watershed Rehabilitation Plans. *Title 390 - National Instruction*. USDA.

Portland Cement Association. (2002). *Design Manual for RCC Spillways and Overtopping Protection.* Skokoie, IL: Portland Cement Association.

U.S. Army Corps of Engineers. (2024, February). Hydrologic Engineering Center River Analysis System (HEC-RAS) Computer Program, Version 6.5. Retrieved from http://www.hec.usace.army.mil/

U.S. Department of the Interior. (1999). Regional Policy on the Protection of Fens, As Amended, Fish and Wildlife Service, Mountain-Prairie Region, January 20, 1999.

U.S. Environmental Protection Agency (2008). Federal Register, Compensatory Mitigation for Losses of Aquatic Resources; Final Rule, 40 CFR Part 230, April 10, 2008

USDA and Kansas State University. (2014). SITES Integrated Development Environment (Version 2005.1.8).

FIGURES

DRAFT

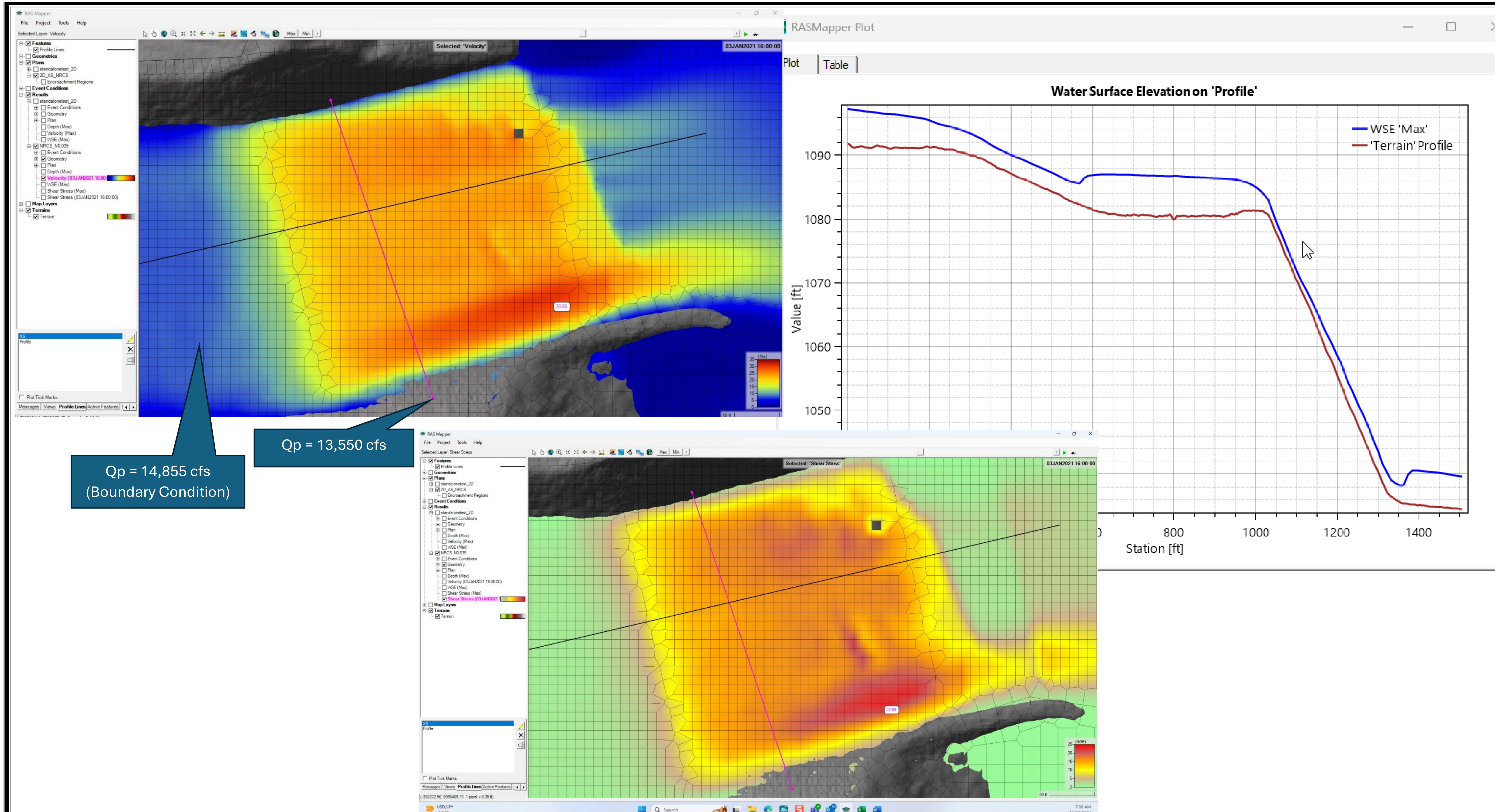


Figure D-3-1: Existing Conditions Auxiliary Spillway Modeling - Profile FBH Maximum WSE, Velocity, and Shear Stresses from HEC-RAS 2D Modeling

NRCS

Solve For: Water Surface (normal depth)

Slope: 0.152 (ft/ft)

WSE: 101.83 (ft)

Channel Manning n: 0.035

Flow: 13550 (ft3/s)

100 40000

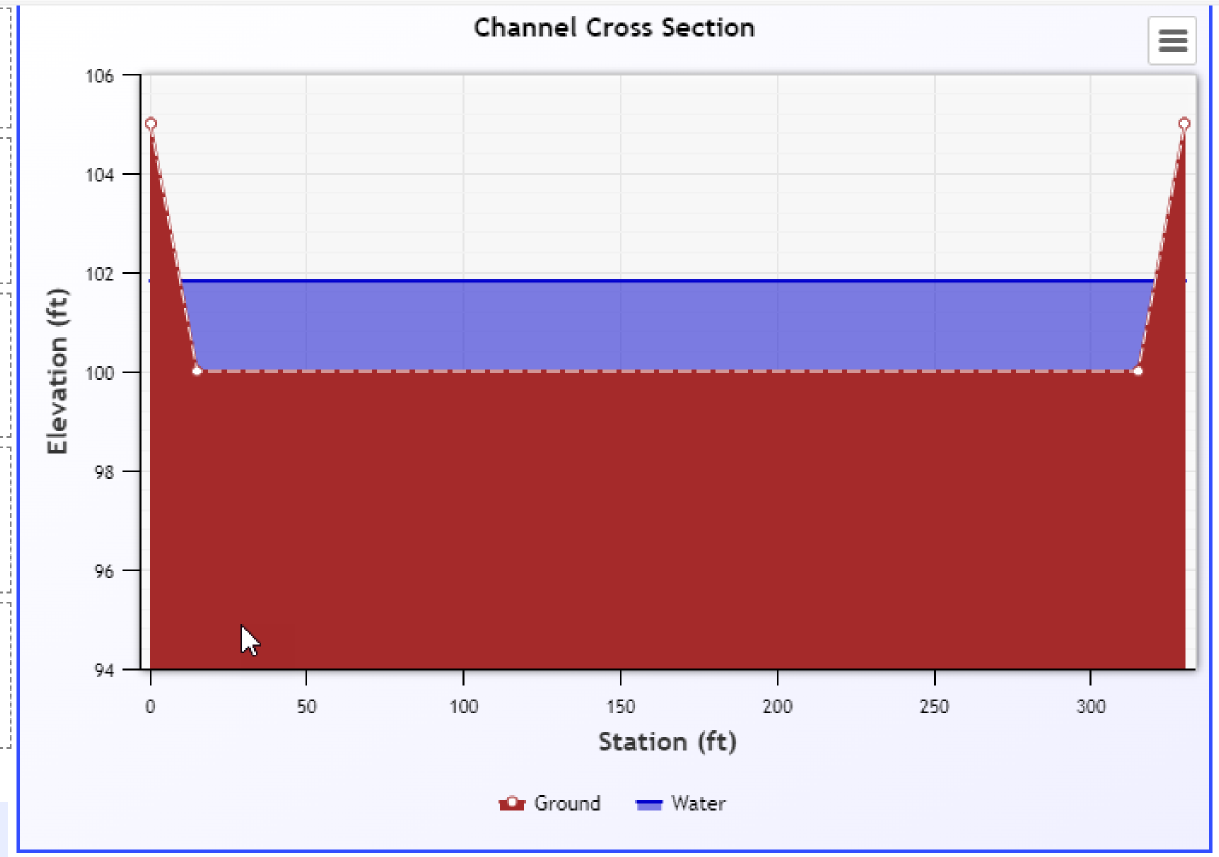
Reset Cross Section

Flow Area:	556.6 (ft2)
Wetted Perimeter:	310.6 (ft)
Max Depth:	1.83 (ft)
Average Velocity:	24.34 (ft/s)
Top Width:	310 (ft)
Iterations:	640
Froude Number:	3.21

Load CSV XS Data Below (station,elevation)

Update Plot

0.00,105.00
15.00,100.00
315.00,100.00
330.00,105.00



Select HECRAS Geometry: Choose File No file chosen

Load HEC-RAS Data

(Shear Stress) τ = 62.4 * depth * slope

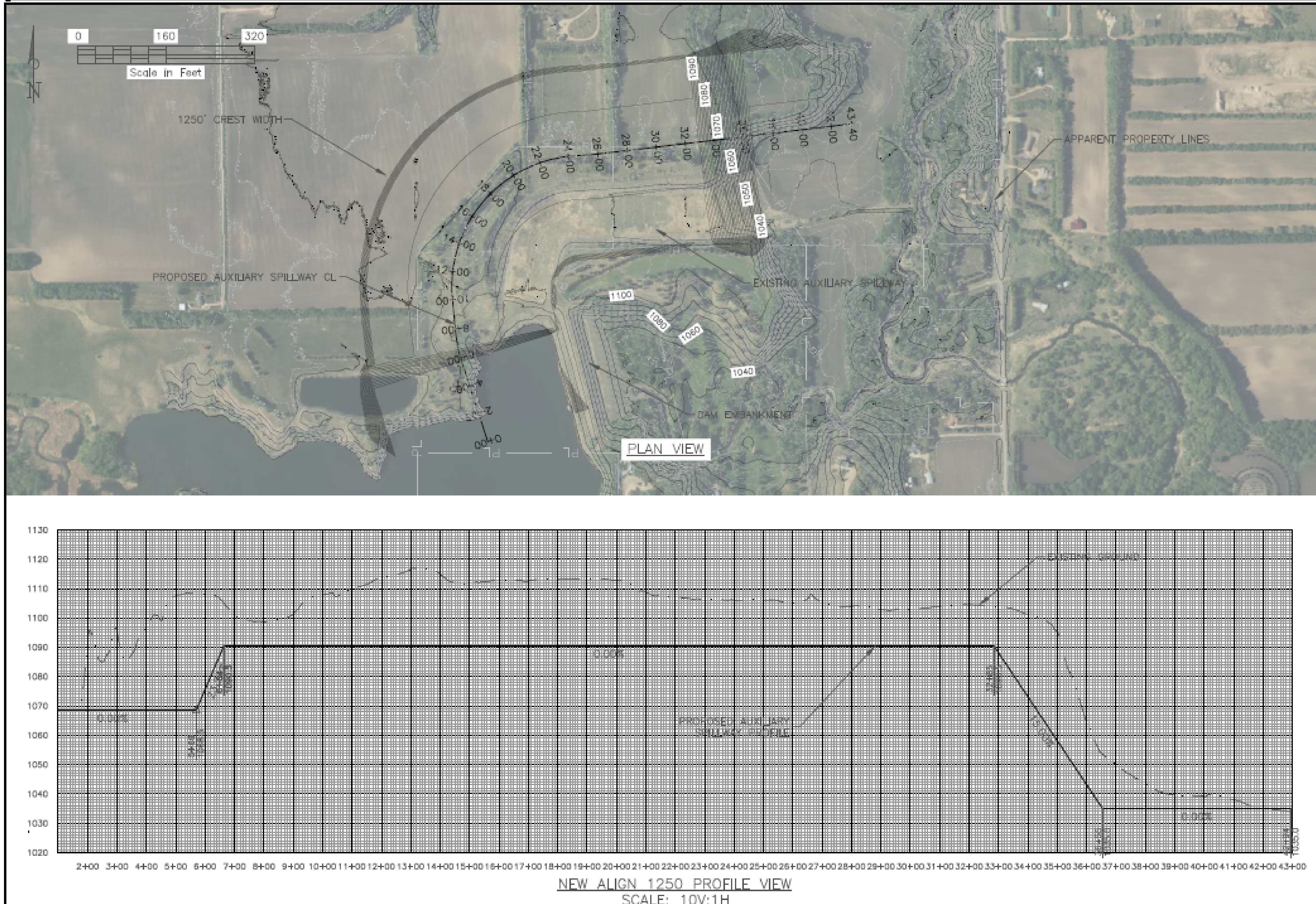
(2D, w/ curve) v = 26.1% above (1D, wo/ curve) v; (30.7/24.34)

(2D, w/ curve) τ = 30.8% above (1D, wo/ curve) τ; (22.7/17.4)

Constants			
y	weight of water	62.4	lb/ft^3
p	mass density of water	1.94	$slugs/ft^3$
Site conditions			
	Bed Slope	0.152	ft/ft or %
	Side Slope Horizontal	3	ft (H:V)
τ	shear stress	22.70	lb/ft^2
V	flow velocity	30.70	ft/sec
ACB Inputs			
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 force	0.354	ft
l3	moment arm for lift force submerged wei	0.725	ft
b	block width normal to flow	1.23	ft
ΔZ	heigh of block protrusion	0	ft
Wb	weight of block	134	lbs
SC	specific gravity of block	2.2	
Equations (NEH Part 628 Dams, Ch. 54 ACB)			
W_{SY}	Submerged weight of block	73.09	lbs
O0	Bed slope angle	0.15	radians
O1	Side Slope Angle	0.32	radians
O2	Angle between bed slope and side slope	0.32	radians
FD	Drag force	30.42	lbs
FL	Lift Force	6.13	lbs
F_D', F_L'	Additional drag force by protrusion	0.00	lbs
WSx	Submerged unit weight of block (Parallel)	10.98	lbs
Wsy	Submerged unit weight of block (Normal)	68.63	lbs
FS	**Factor of Safety**	**1.67**	
Termination			
d	depth	2.19	ft
τ	shear stress	20.77	
Es	Specific Energy	16.82	ft

Figure D-3-2: Existing Conditions Auxiliary Spillway Calculations - Hydraulics from 1D Equation and ACB Design Calculations

NRCS



SITES Integrated Development Environ

File View Windows Help

	g14_1250noPS
Site Identification	9
Watershed Runoff Curve Number	N/A
Total Watershed Drainage Area (Sq.Miles)	65.50
Watershed Time of Concentration (Hours)	N/A
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	N/A
FBH or Storm Rainfall Duration (Hours)	N/A
SDH Inflow Peak (CFS)	2478.9
FBH or Storm Inflow Peak (CFS)	15574.0
Initial Reservoir Elevation (Feet)	1076.80
Maximum WS SDH (Feet)	1092.60
Maximum WS FBH or Storm (Feet)	1094.62
Storage at Max. WS FBH or Storm (Acre-Ft)	6190.2
Top Dam (Feet)	1094.62
Storage, Top Dam (Acre-Ft)	6160.0
Emb. Yardage (CY)	N/A
PSH Drawdown (Days)	N/A
378 Drawdown (Days)	N/A
PS Crest (Feet)	1068.90
PS Number of Conduits	1
PS Conduit Diameter (Inches)	N/A
PS Conduit Height (Feet)	0.10
PS Conduit Width (Feet)	0.10
PS Conduit Area (Sq. Feet)	0.01
Storage, PS Crest (Acre-Ft)	672.
PS Discharge at AS Crest (CFS)	N/A
PS Discharge for SDH (CFS)	N/A
PS Discharge FBH or Storm (CFS)	N/A
AS Crest (Feet)	1090.50
Storage, AS Crest (Acre-Ft)	4434.2
AS Width (Feet)	1250.
AS Exit Slope (%)	14.31
AS Ret. Curve Index	5.60
AS Veg. Cover Factor	0.85
AS Maintenance Code	2
AS Max. Head SDH (Feet)	2.10
AS Peak Discharge SDH/Storm (CFS)	2452.5
AS Exit Velocity SDH or Storm (Ft/S)	5.03
AS Stress SDH or Storm (Lb./Sq.Ft.)	0.239
Hp FBH or Storm (Feet)	4.12
AS Peak Discharge FBH/Storm (CFS)	15529.
AS Integ. Dist. FBH or Storm (Feet)	10.
Qe/B FBH or Storm (Acre-Ft/Ft)	43.9
Uncontrolled Drainage Area (Sq.Miles)	65.50
Number of Errors	0
Number of Warnings	0

Figure D-3-3: Alternative 1: Earthen Spillway Conceptual Plan and Profile – 1,250 feet Wide SITES Model Results, Conceptual Design Plan & Profile

Constants				Notes
γ	weight of water	62.4	lb/ft^3	
ρ	mass density of water	1.94	$slugs/ft^3$	
Site conditions				
	Bed Slope	0.11	ft/ft or %	Critical slope is 0.11 ft/ft
	Side Slope Horizontal	3	ft (H:V)	
τ	shear stress	19.0	lb/ft^2	From 1D Calc with %↑
V	flow velocity	28.9	ft/sec	from 2D at bottom rigth side of AS
ACB Inputs				
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	0.11	radians	
Θ1	Side Slope Angle	0.32	radians	
Θ2	Angle between bed slope and side sl	0.32	radians	
FD	Drag force	25.50	lbs	628-54.19
FL	Lift Force	5.45	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	7.99	lbs	628-54.20
Wsy	Submerged unit weight of block (Norn	68.97	lbs	628-54.20
FS	**Factor of Safety**	**2.01**		628-54.23
Termination				
d_1	depth	2.57	ft	1D Calc w/ %↑
τ	shear stress	17.62		628-54.19
Es	Specific Energy	15.58	ft	628-54.26
d_2	depth	8.93	ft	USBR Engr Mono 25
L	Length	46.4	ft	USBR EM25 Fig 7

Solve For: Water Surface (normal depth)

Slope: 0.11 (ft/ft)

WSE: 102.12 (ft)

Channel Manning n: 0.035

Flow: 14855 (ft3/s)

100 40000

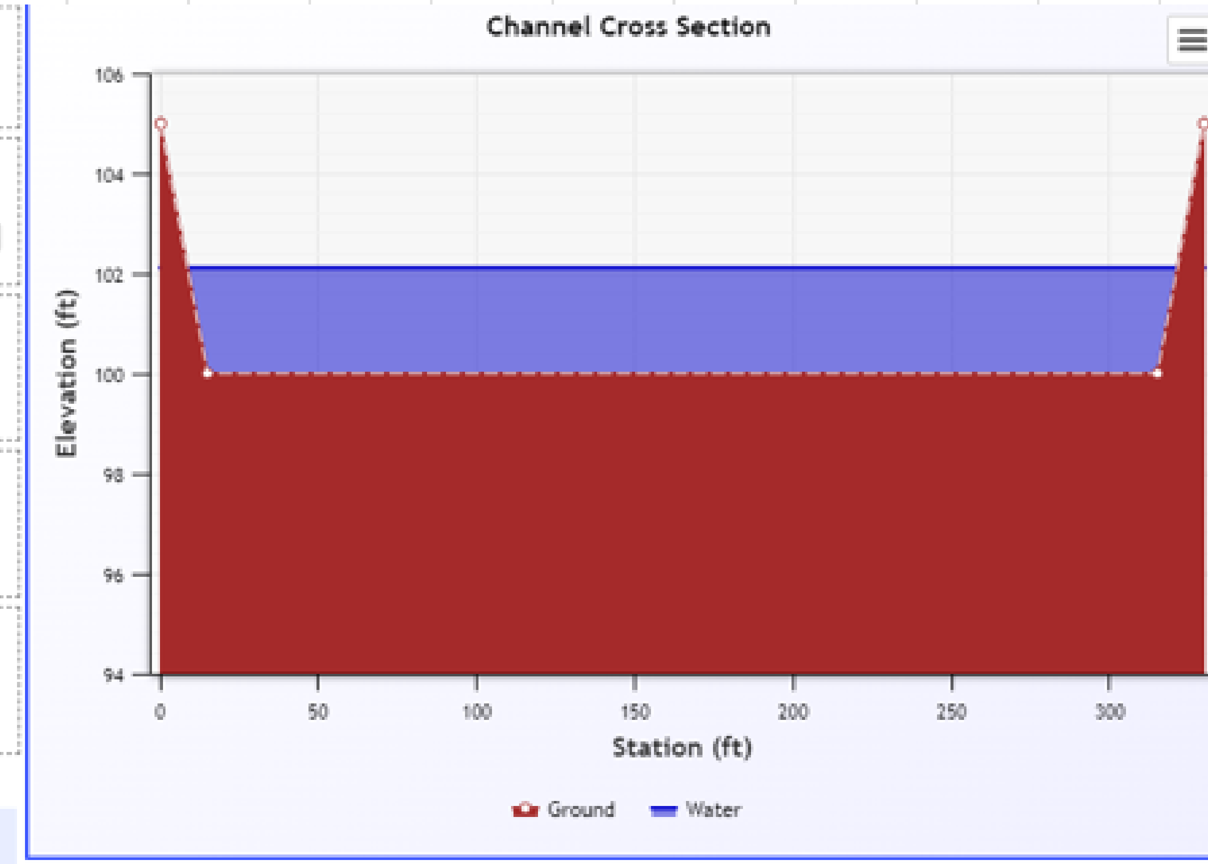
Reset Cross Section

Flow Area:	647.2 (ft2)
Wetted Perimeter:	312.7 (ft)
Max Depth:	2.12 (ft)
Average Velocity:	22.95 (ft/s)
Top Width:	312 (ft)
Iterations:	44
Froude Number:	2.82

Load CSV XS Data Below (station,elevation)

Update Plot

0.00,105.00
15.00,100.00
315.00,100.00
330.00,105.00



Selec

Tapered ACB Cross Section

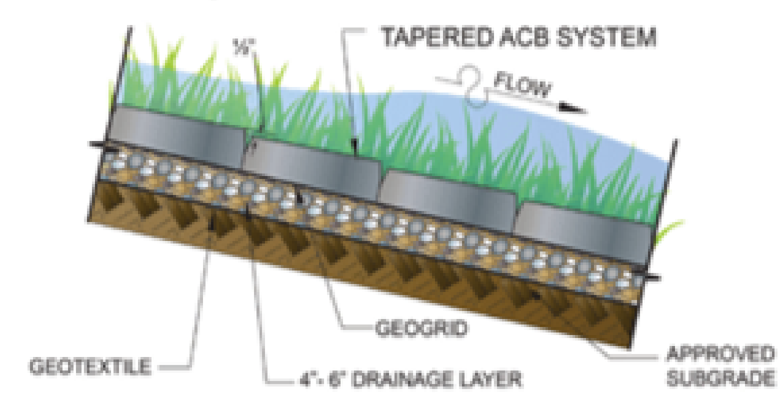
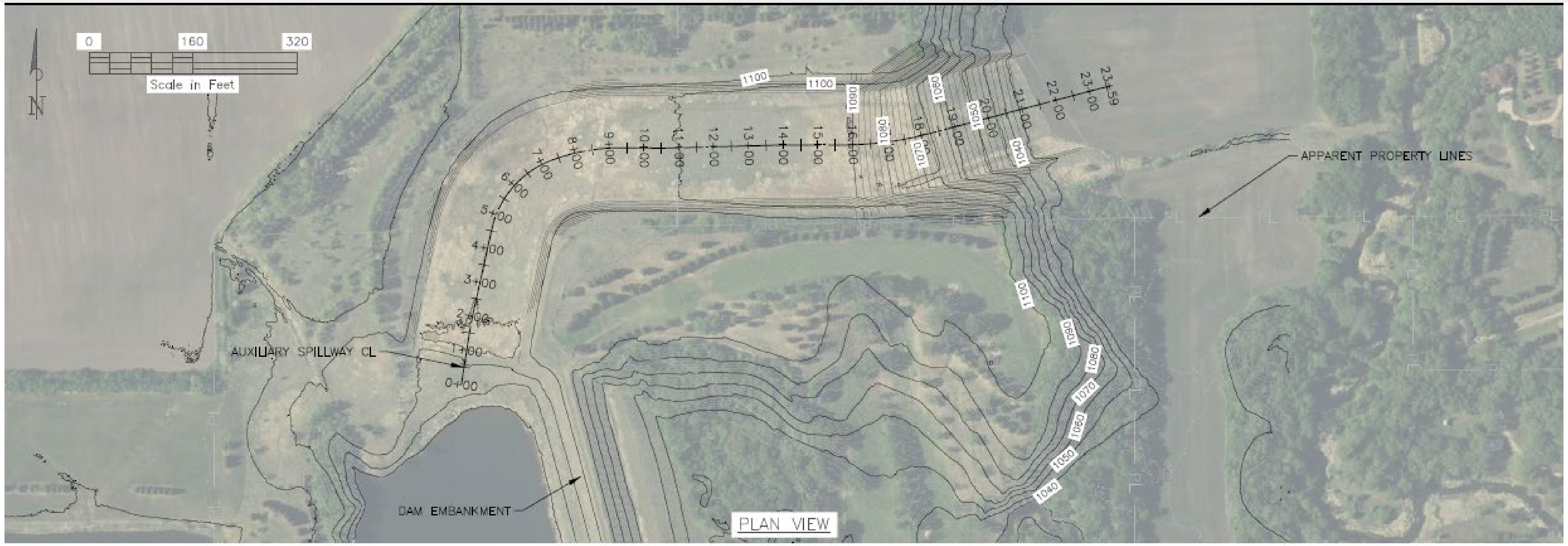


Figure 1. Typical ACB cross sections.

Figure D-3-4: Alternative 2: Armored ACB Computations - Keep Bend, 11% Slope



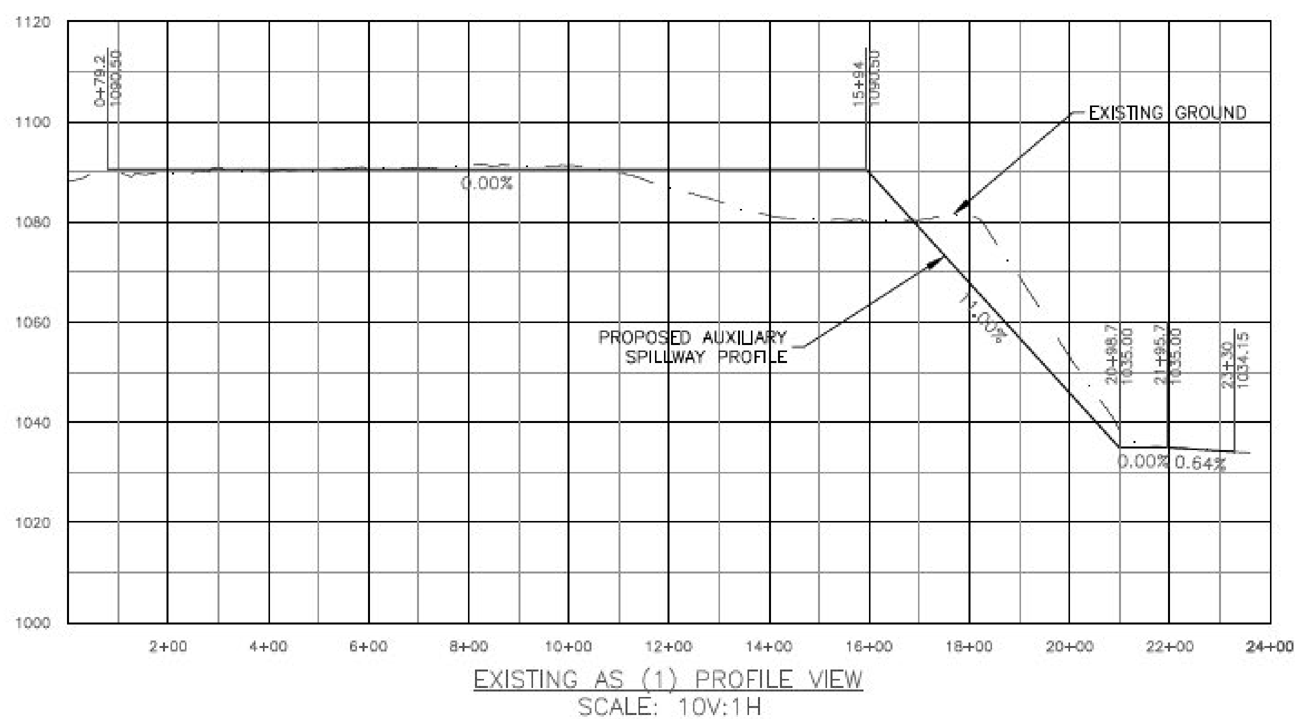


Figure D-3-5: Alternative 2: Armored ACB Conceptual Design Plan & Profile - Keep Bend, 11% Slope

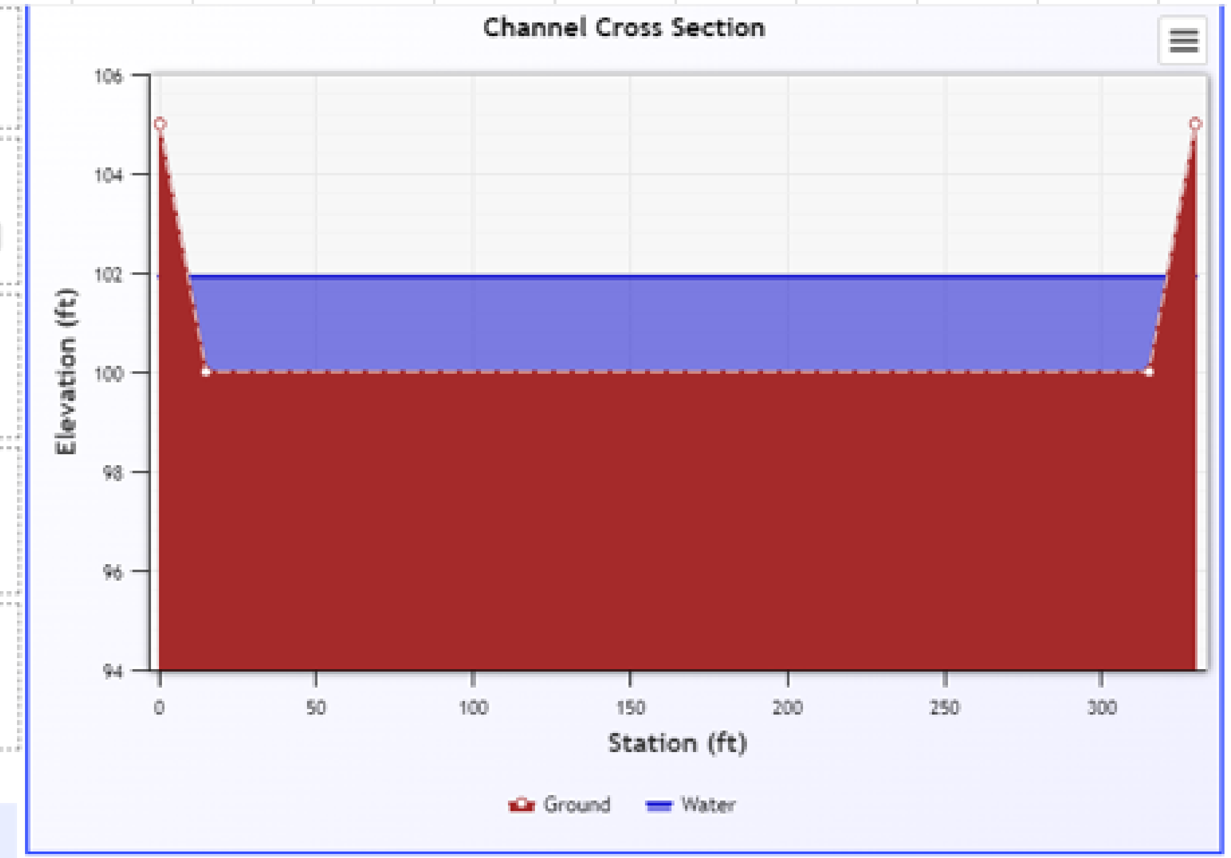
Constants				
γ	weight of water	62.4	lb/ft^3	
ρ	mass density of water	1.94	$slugs/ft^3$	
Site conditions				
	Bed Slope	0.152	ft/ft or %	From AS_Profile
	Side Slope Horizontal	3	ft (H:V)	
τ	shear stress	18.2	lb/ft^2	1D calcs (Normal Depth)
V	flow velocity	25.28	ft/sec	
				Max Q from 1-D RAS, 2D RAS and SITES are
ACB Inputs				
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	24.40	lbs	628-54.19
FL	Lift Force	4.15	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	10.98	lbs	628-54.20
Wsy	Submerged unit weight of block (Norr	68.63	lbs	628-54.20
FS	**Factor of Safety**	**2.06**		628-54.23
Termination				
d_1	depth	1.91	ft	
τ	shear stress	18.12		628-54.19
Es	Specific Energy	11.83	ft	628-54.26
d_2	depth	7.82	ft	USBR Engr Mono 2
L	Length	42.2	ft	USBR EM25 Fig 7

Solve For: Water Surface (normal depth)
Slope: 0.152 (ft/ft)
WSE: 101.92 (ft)
Channel Manning n: 0.035
Flow: 14855 (ft3/s)
100 40000
Reset Cross Section

Flow Area:	587.5 (ft2)
Wetted Perimeter:	312.6 (ft)
Max Depth:	1.92 (ft)
Average Velocity:	25.28 (ft/s)
Top Width:	312 (ft)
Iterations:	4
Froude Number:	3.26

Load CSV XS Data Below (station,elevation)
Update Plot

0.00,105.00
15.00,100.00
315.00,100.00
330.00,105.00



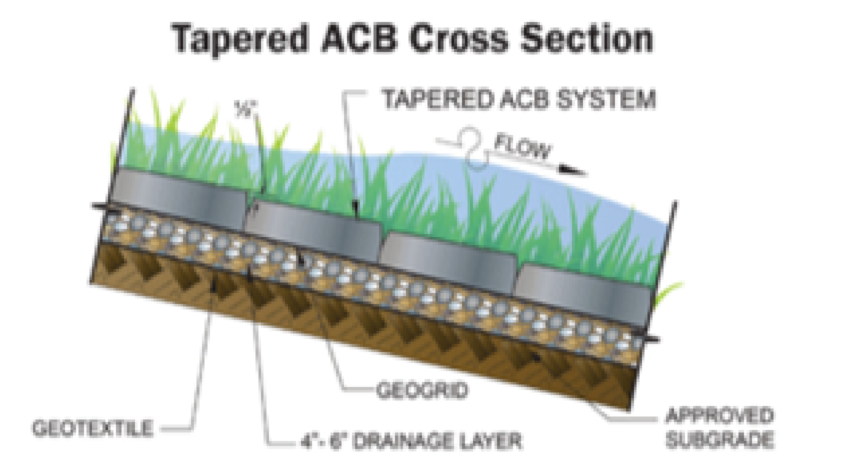


Figure 1. Typical ACB cross sections.

Figure D-3-6: Alternative 3: Armored ACB Computations - Keep 15% Slope, No Curve

NRCS

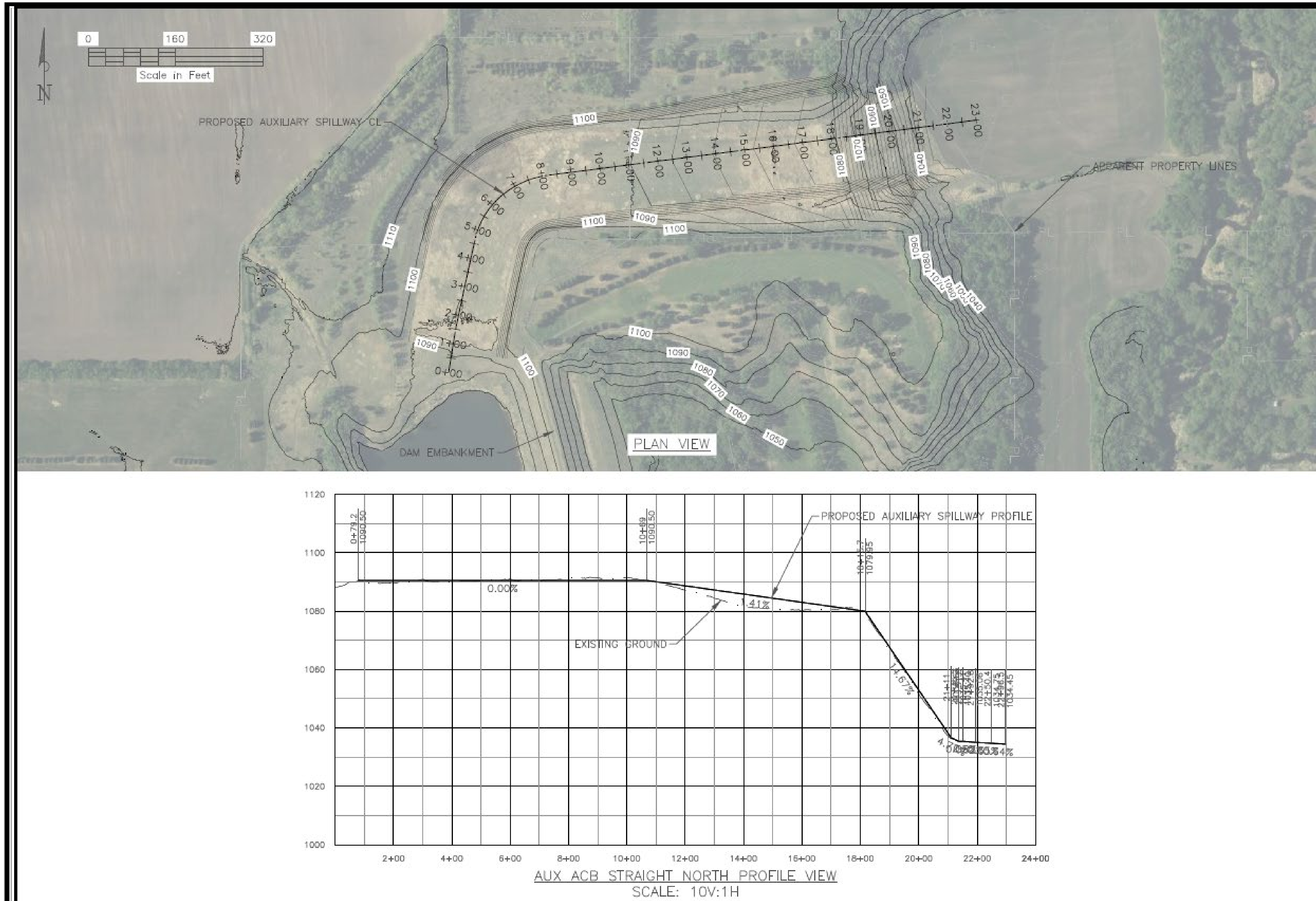
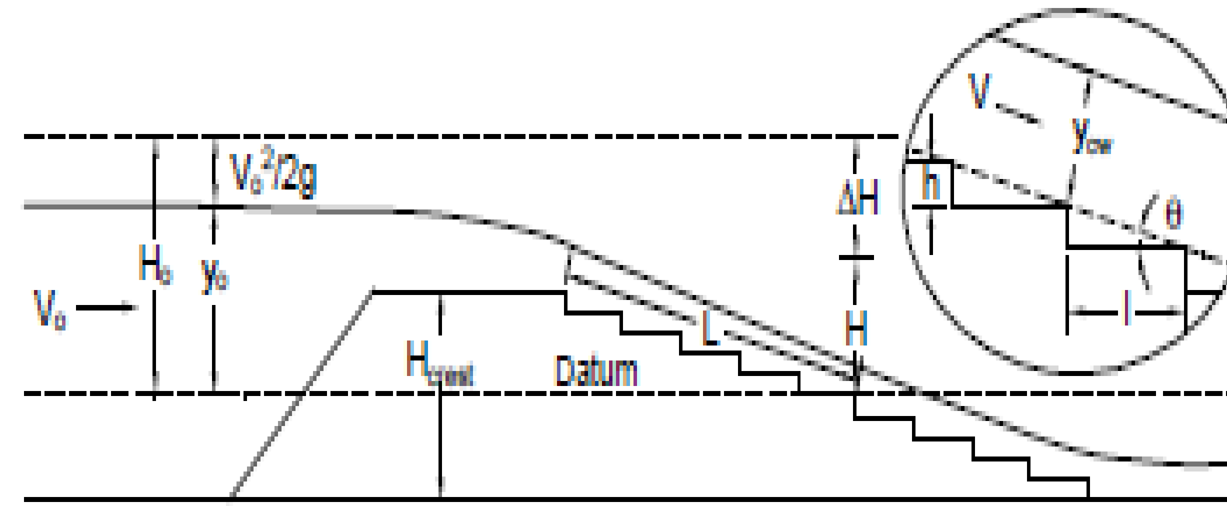


Figure D-3-7: Alternative 3: Armored ACB Conceptual Design Plan & Profile - Keep 15% Slope, No Curve

NRCS

Assumptions:					Notes:
Horizontal aspect of Slope:	3	H:1V			Typcial are 2.5, 3, and 4, which appear to be based on dam
θ	0.32	rad			
θ	18.44	deg			
B (width)	200	feet			Based on Q (pmf), TOD, weir C=2.6. From RAS Results
Stilling Basin (SB) Invert Elev	1028				Floodplain Elev 1028, Rqrd for stilling basin design
Hcrest (AS: 1090.5, stream invert:1035)	62.5	feet			Basin elev from stream geometry, AS from PSH elev.
Q (pmf)	14,855	cfs			Routing reservoir PMF through AS only in RAS, 14,855 cfs.
h (step)	1	feet			Typical are 1 or 2 feet.
Initial Hydaulic Calcs:					
q (unit discharge)	74.3	cfs/ft			Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	ft			Std hydraulic calc (Critical depth)
L (Chute Length)	197.6	ft			Based on geometry
Skimming Flow Conditions:					
h/dc (flow depth ratio)	0.18	Check ↓			Ratio
1. Ohtsu et al. (h/dc)<(7/6)(tanθ)^1/6	0.97	yes			HL355 Eq. 1 (Ohtsu et al. (2001, 2004))
2. Chanson (h/dc)<1/(1.2-0.325tanθ)	0.92	yes			HL355 Eq. 2 (Chanson (2002))
Surface Inception Point:			Applicable Range		Notes:
ks (step surface roughness)	0.95	feet			HL355 Eq. 3 Explanation
F* = q/[g sin θ (h cos θ)3]0.5	25.2	unitless		Check ↓	HL355 Eq. 3 Explanation; Froude surface roughness
Li (Hunt & Kadavy, 2013)	87.0	feet	0.1<F*<28	yes	HL355 Eq. 4 & 5: Use Hunt and Kadavy since newest research and broad-crested. If
Li (Hunt & Kadavy, 2013)	87.9	feet	28<F*		Ogee use Chanson (1994), if broad crested and F*<10 consider Meireles & Matos
Li (Adopt based on applicable range)	87.0	feet			
L/Li (Normalized Length)	2.27	unitless			Ratio If <1 there is no aerated flow, i.e. incetion point is below stilling basin which limits energy dissipation of steps
Flow Depth:				Check ↓	
ycw/dc	0.37	unitless	L/Li >=1.0	yes	HL355 Eq. 17; ycw/dc (clear water flow to critical flow depth ratio)
ycw/dc	0.31	unitless	L/Li <1.0	no	HL355 Eq. 18
ycw/dc (Adopt based on applicable range)	0.37	unitless			
ycw (clear water flow depth)	2.08	feet			
Hw (Recommended training wall height, above inception point)	4.16				Above inception point. USBR Design of Small Dams (1987), Eq. 22 page 385.
Cmean (USDA-ARS Emperical)	0.26		1.0<L/Li <2.0		Depth-averaged or mean air concentration
y90 (Aerated flow depth)	2.81	feet			HL355 Eq. 10
FoS (Recommended range 1.2 - 1.5)	1.5				HL355 Eq. 20 Explanation & Ohtsu el al (2004) & Boes (2000)
Hw (Alt. 1 recommended training wall ht)	4.21	feet			HL355 Recommendation considering FoS, Below inception point.
Hw (Alt. 2 recommended training wall ht)	6.07	feet			USBR Design of Small Dams (1987), Eq. 22 page 385.
Hw (Recommended training wall height, below inception point)	6.07	feet			Take greater of Alt. 1 & 2.
Energy:					
α (Energy Coef)	1.079		L=Li > 1.0, 0.10 ≤ h=dc ≤ 1.1, ;		HL355 Eq. 27
α (Simplified Energy Coef)	1.099		L=Li > 1.0, 0.10 ≤ h=dc ≤ 1.1, ;		HL355 Eq. 28
Ho (Total Energy at crest)	70.83	feet			HL355 Eq. 23
y (normal depth, pseudo-bottom)	2.08	feet			This is assumed the same as ycw for energy as bulked depth (with air) is not approp
V (mean velocity)	35.7	ft/s			Std hydraulic calc (Q=v*A)
H (Bottom/residual energy)	23.36	feet			Std hydraulic calc (Total Energy = static head + velocity head)
ΔH/Ho (Potnl energy dissipated w/ steps)	67%				



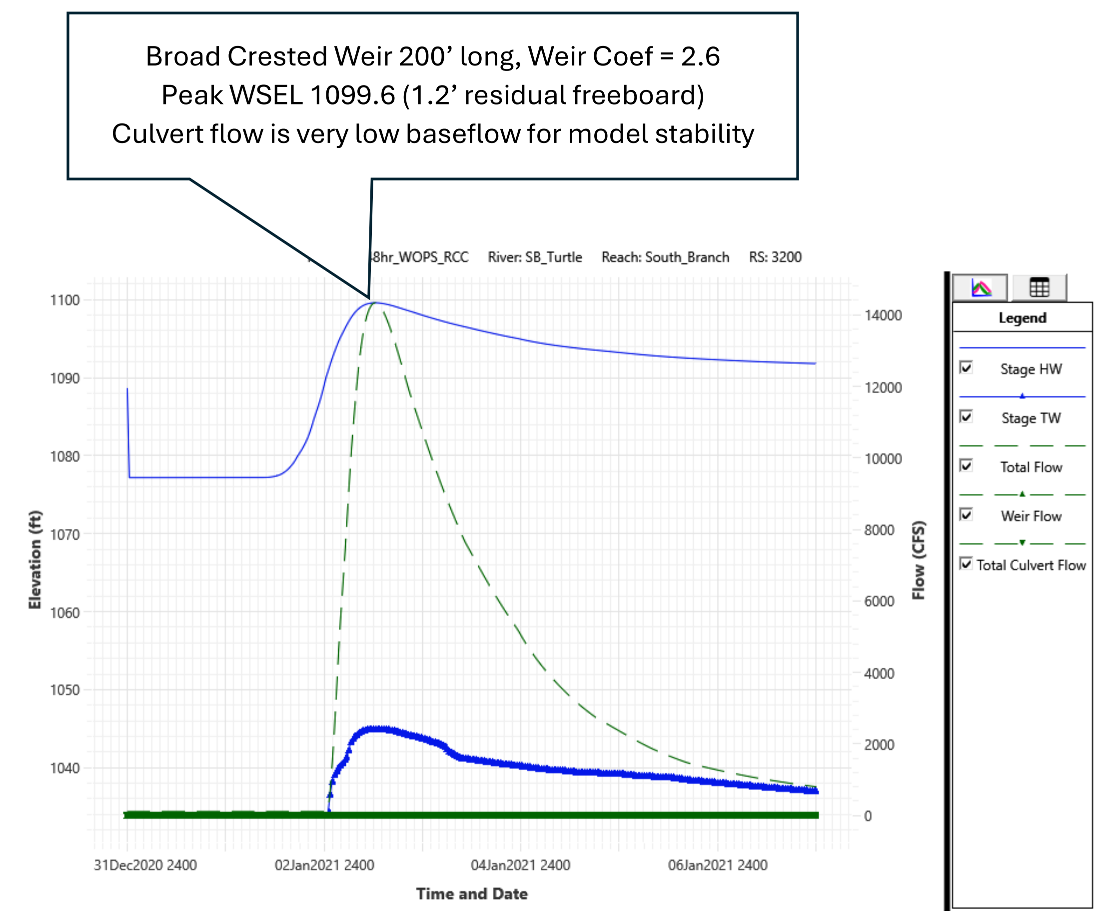


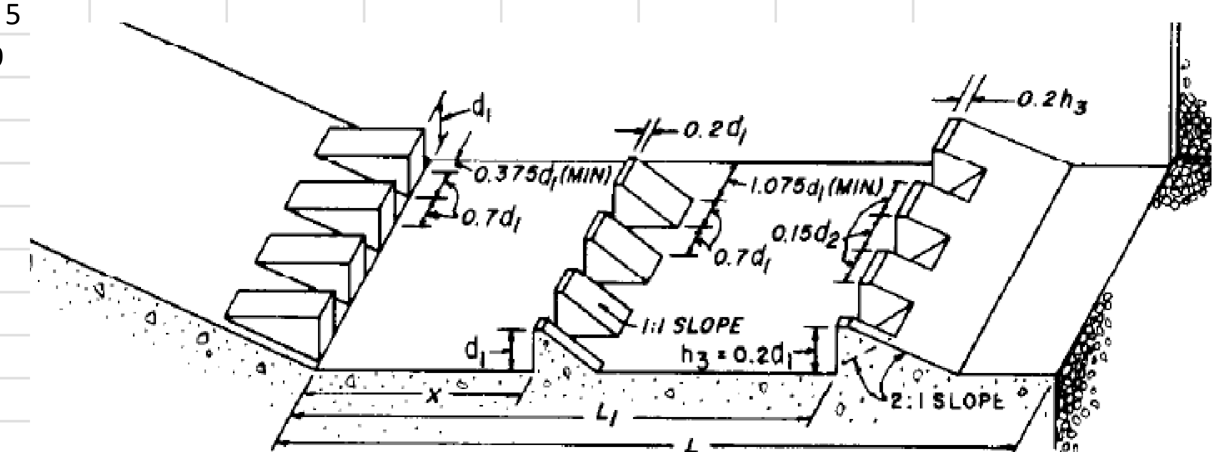
Figure D-3-8: Alternative 4: Structural RCC Computations - in Current Auxiliary Spillway

NRCS

Stilling Basin:					From HL360 & USBR (Frizell) HL-2012-02, recommend type III or IV.
q (unit discharge)	74.3	cfs/ft			
F (Froude #)	4.37				HL355 Eq. 30; Froude # is ratio of inertial and gravitation forces, governs the formation of free surface vorticies.
F (Alt. Froude #)	4.37				HL355 Eq. 31
d1 (Incoming depth to stilling basin)	2.08	feet			same as y and ycw; can also be called initial depth
d2 (Depth at end of basin)	11.8	feet			From USBR Engr Mono 25 8th printing (Peterka, 1984), Fig. 5; also be called conjugate or sequent depth
d2 elevation	1039.84	Check ↓	**Result**	**Recommendation:**	
RAS TW	1040	yes	At Max Q Matches	none	USBR 1987, page 397. If check is "no" see guidance how to correlate TW RC with basin RC. Tailwater depth should be greater than conjugate depth.
Type IV (Alt Low Froude) could be applied as they fit site conditions best. See table below for criteria of various basins.					

Alternative Low Froude Basin feature Dimensions			
L1 (Basin length to end sill)	26.1	feet	From USBR 1987 Fig. 9-40; values in column B are for F=4.36
L2 (Total basin length)	36.7	feet	
x (basin start to baffle)	11.3	feet	
h4 (End sill height)	0.4	feet	
Chute blocks & baffle height	2.08	feet	
Stilling basin freeboard	3.52	feet	USBR 1987, page 398 Eq. 25
Riprap below Stilling Basin:			HL360 (Use Ishbash or USBR for Type III)
Avg. channel velocity	6.27		Hydraulic Calcs (Q=V*A)
D50 (Ishbash)	2	inches	NEH Part 654 TS14C- Fig. 5
D50 (USBR)	0.54	feet	NEH Part 654 TS14C Eq. 9
D50 (USBR)	6.43	inches	

	F (Froude #)	q (unit Q)	Note (FEMA, 2010)
Hydraulic Conditions	4.37	74.3	See above
Basin Type			
Type I	< 1.7 - 2.5		Page 49
Type II	>4.5	< 500	Page 49
Type III	>4.5	<60	Page 50
Type IV (Alternative Low Froude)	2.5-4.5		Page 53,56
SAF	1.7 - 17		Low FOS



(A) DIMENSIONS FOR ALTERNATIVE LOW FROUDE NUMBER BASIN

Figure D-3-9: Alternative 4: Structural RCC Termination Basin Computations - in Current Auxiliary Spillway

NRCS

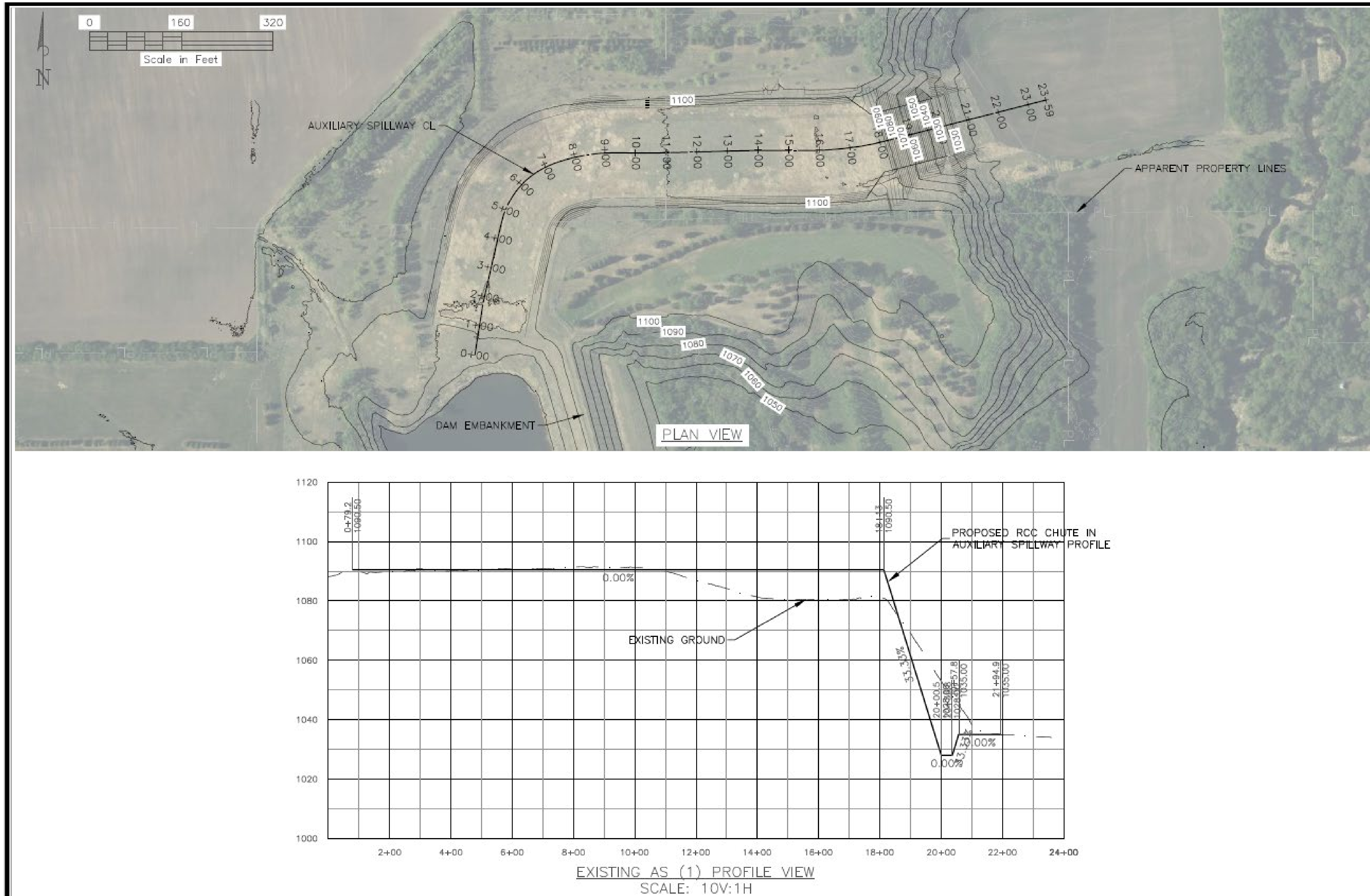
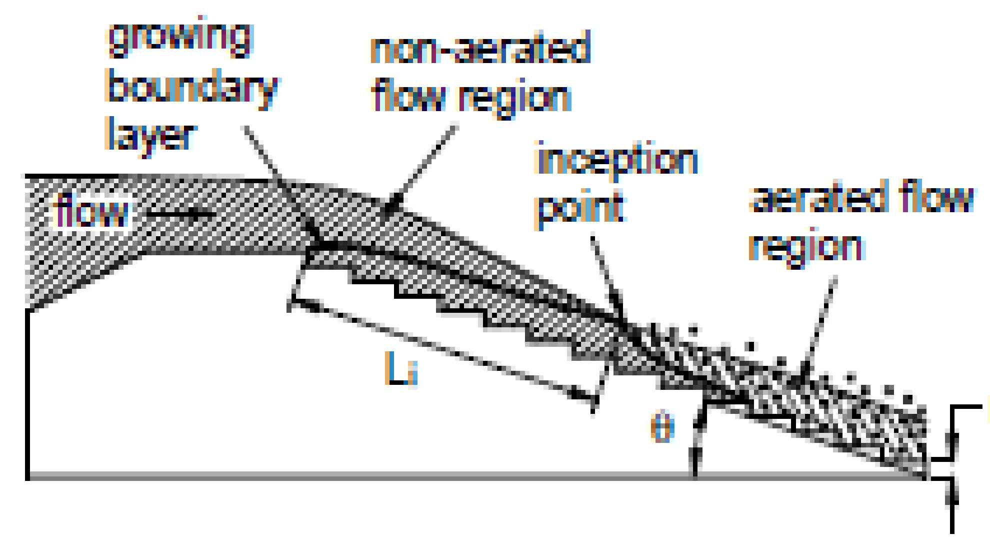


Figure D-3-10: Alternative 4: Structural RCC Plan and Profile - in Current Auxiliary Spillway

NRCS

Assumptions:					Notes:
Horizontal aspect of Slope:	3	H:1V			Typcial are 2.5, 3, and 4, which appear to be based on dam geometries.
θ	0.32	rad			
θ	18.44	deg			
B (width)	200	feet			Based on Q (pmf), TOD, weir type. From SITES Results
Stilling Basin (SB) Invert Elev	1033				Floodplain Elev 1040, at base of dam
Hcrest (AS: 1090.5, stream invert:1040)	57.5	feet			Basin elev from stream geometry, AS from PSH elev.
Q (pmf)	14,855	cfs			Routing reservoir PMF through PS and AS in RAS, 14,631 cfs.
h (step)	1	feet			Typical are 1 or 2 feet.
Initial Hydaulic Calcs:					
q (unit discharge)	74.3	cfs/ft			Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	ft			Std hydraulic calc (Critical depth)
L (Chute Length)	181.8	ft			Based on geometry
Skimming Flow Conditions:					
h/dc (flow depth ratio)	0.18	Check ↓			Ratio
1. Ohtsu et al. (h/dc)<(7/6)(tanθ)^1/6	0.97	yes			HL355 Eq. 1 (Ohtsu et al. (2001, 2004))
2. Chanson (h/dc)<1/(1.2-0.325tanθ)	0.92	yes			HL355 Eq. 2 (Chanson (2002))
Surface Inception Point:			Applicable Range		Notes:
ks (step surface roughness)	0.95	feet			HL355 Eq. 3 Explanation
F* = q/[g sin θ (h cos θ)3]0.5	25.2	unitless		Check ↓	HL355 Eq. 3 Explanation; Froude surface roughness
Li (Hunt & Kadavy, 2013)	87.0	feet	0.1<F*<28	yes	HL355 Eq. 4 & 5: Use Hunt and Kadavy since newest research and broad-crested. If
Li (Hunt & Kadavy, 2013)	87.9	feet	28<F*		Ogee use Chanson (1994), if broad crested and F*<10 consider Meireles & Matos
Li (Adopt based on applicable range)	87.0	feet			
L/Li (Normalized Length)	2.09	unitless			Ratio If <1 there is no aerated flow, i.e. incetion point is below stilling basin which limits energy dissipation of steps
Flow Depth:				Check ↓	
ycw/dc	0.37	unitless	L/Li >=1.0	yes	HL355 Eq. 17; ycw/dc (clear water flow to critical flow depth ratio)
ycw/dc	0.32	unitless	L/Li <1.0	no	HL355 Eq. 18
ycw/dc (Adopt based on applicable range)	0.37	unitless			
ycw (clear water flow depth)	2.08	feet			
Hw (Recommended training wall height, above inception point)	4.16				Above inception point. USBR Design of Small Dams (1987), Eq. 22 page 385.
Cmean (USDA-ARS Emperical)	0.25		1.0<L/Li <2.0		Depth-averaged or mean air concentration
y90 (Aerated flow depth)	2.78	feet			HL355 Eq. 10
FoS (Recommended range 1.2 - 1.5)	1.5				HL355 Eq. 20 Explanation & Ohtsu el al (2004) & Boes (2000)
Hw (Alt. 1 recommended training wall height)	4.16	feet			HL355 Recommendation considering FoS, Below inception point.
Hw (Alt. 2 recommended training wall height)	6.03	feet			USBR Design of Small Dams (1987), Eq. 22 page 385.
Hw (Recommended training wall height, below inception point)	6.03	feet			Take greater of Alt. 1 & 2.
Energy:					
α (Energy Coef)	1.080		L=Li > 1.0, 0.10 ≤ h=dc ≤ 1.1, ar		HL355 Eq. 27
α (Simplified Energy Coef)	1.099		L=Li > 1.0, 0.10 ≤ h=dc ≤ 1.1, ar		HL355 Eq. 28
Ho (Total Energy at crest)	65.83	feet			HL355 Eq. 23
y (normal depth, pseudo-bottom)	2.08	feet			This is assumed the same as ycw for energy as bulked depth (with air) is not appropriate.
V (mean velocity)	35.7	ft/s			Std hydraulic calc (Q=v*A)
H (Bottom/residual energy)	23.39	feet			Std hydraulic calc (Total Energy = static head + velocity head)
ΔH/Ho (Potential energy dissipated w/ steps)	64%				



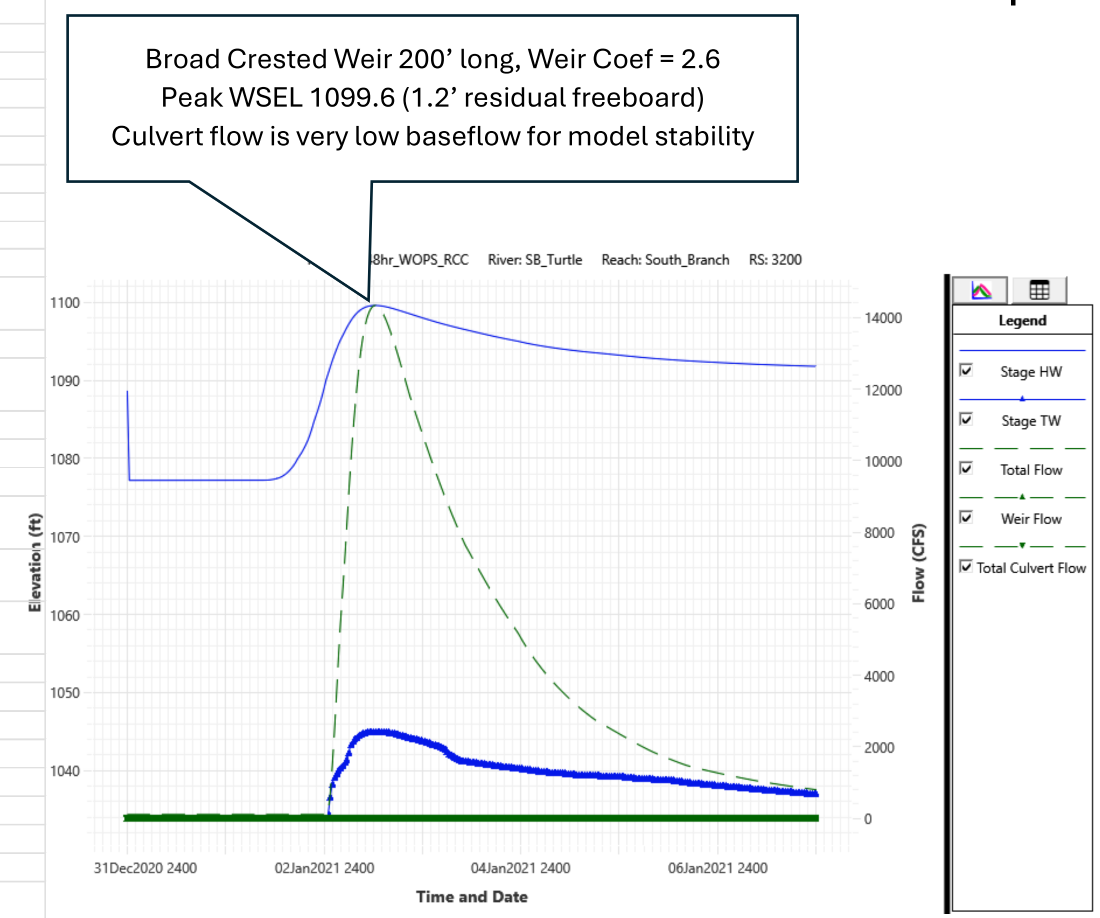


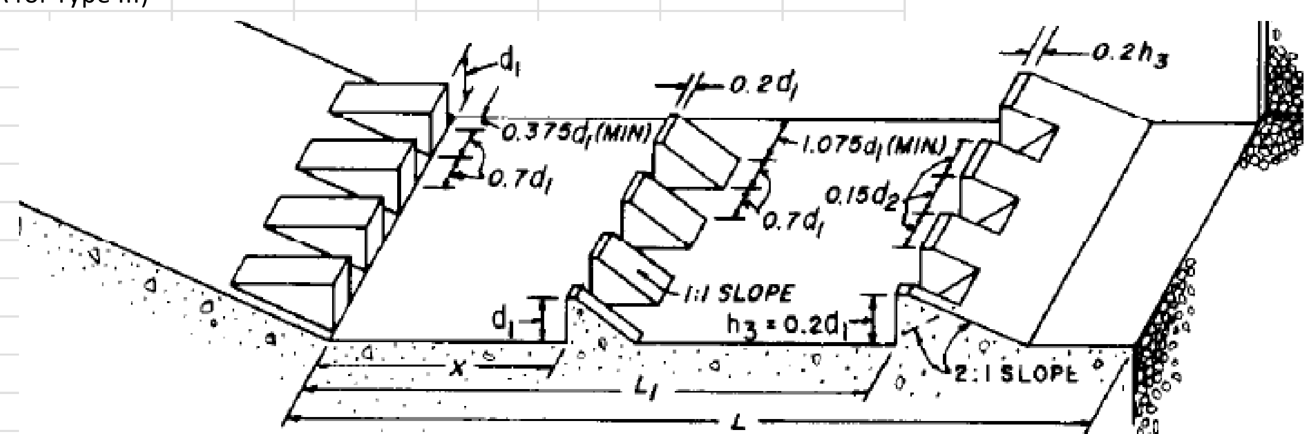
Figure D-3-11: Alternative 5: Structural RCC Computations - Overtop Dam

Stilling Basin:					From HL360 & USBR (Frizell) HL-2012-02, recommend type III or IV.
q (unit discharge)	74.3	cfs/ft			
F (Froude #)	4.37				HL355 Eq. 30; Froude # is ratio of inertial and gravitation forces, it governs the formation of free surface vorticies.
F (Alt. Froude #)	4.37				HL355 Eq. 31
d1 (Incoming depth to stilling basin)	2.08	feet			same as y and ycw; can also be called conjugate depth
d2 (Depth at end of basin)	11.8	feet			From USBR Engr Mono 25 8th printing (Peterka, 1984), Fig. 5; can also be called sequent depth
d2 elevation	1044.84	Check ↓	Result	Recommendation:	
RAS TW	1045.1	yes	At Max Q Matches	none	USBR 1987, page 397. If check is "no" see guidance how to correlate TW RC with basin RC. Consider Type III as chute blocks, baffles, and end sill will assist with sweepout.
Type IV Length	68.7	feet	Requirs chute blocks & end sill		From USBR Design of Small Dams (1987), Fig. 9-39. Values in Column B for F=4.36, which document suggests type IV because 2.5<F<4.5.
Type III Length	36.7	feet	Requirs chute blocks, dentated sill, and baffle piers		From USBR 1987 Fig. 9-40. USBR Frizell states Type III ok for F>4.

Type IV (Alt Low Froude) could be applied as they fit site conditions best. See table below for criteria of various basins.

Type III Stilling Basin feature Dimensions			
L1 (Basin length to end sill)	26.1	feet	From USBR 1987 Fig.
L2 (Total basin length)	36.7	feet	9-40; values in
x (basin start to baffle)	11.3	feet	column B are for
h4 (End sill height)	0.4	feet	F=4.36
Chute blocks & baffle height	2.08	feet	
Stilling basin freeboard	3.52	feet	USBR 1987, page 398 Eq. 25
Riprap below Stilling Basin:			HL360 (Use Ishbash or USBR for Type III)
Avg. channel velocity	6.27		Hydraulic Calcs (Q=V*A)
D50 (Ishbash)	2	inches	NEH Part 654 TS14C- Fig. 5
D50 (USBR)	0.54	feet	NEH Part 654 TS14C Eq. 9
D50 (USBR)	6.43	inches	

	F (Froude #)	q (unit Q)	Note (FEMA, 2010)
Hydraulic Conditions	4.37	74.3	See above
Basin Type			
Type I	< 1.7 - 2.5		Page 49
Type II	>4.5	< 500	Page 49
Type III	>4.5	<60	Page 50
Type IV (Alternative Low Froude)	2.5-4.5		Page 53,56
SAF	1.7 - 17		Low FOS



(A) DIMENSIONS FOR ALTERNATIVE LOW FROUDE NUMBER BASIN

Figure D-3-12: Alternative 5: Structural RCC Spillway Termination Basin Computations - Overtop Dam

NRCS

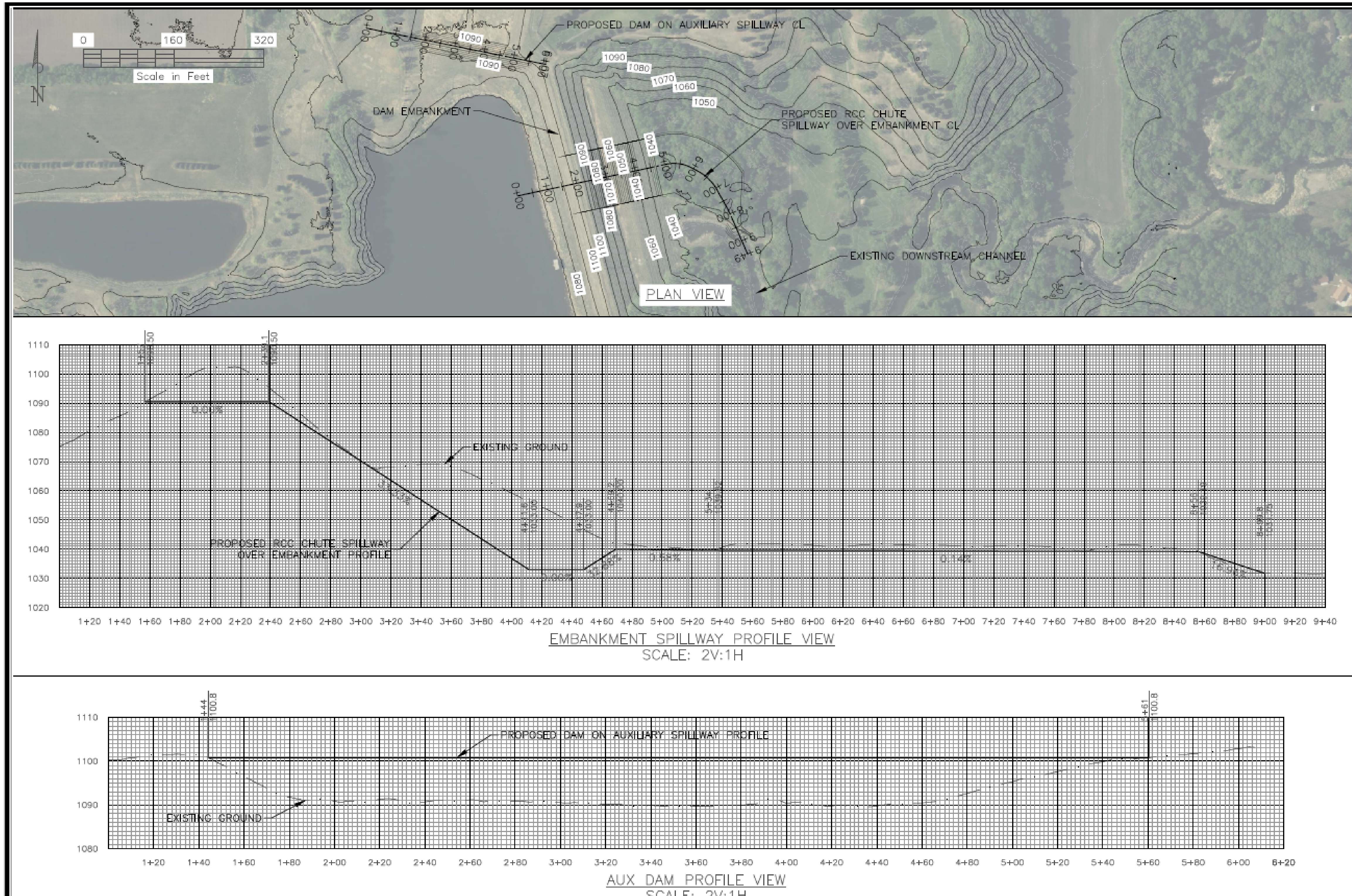
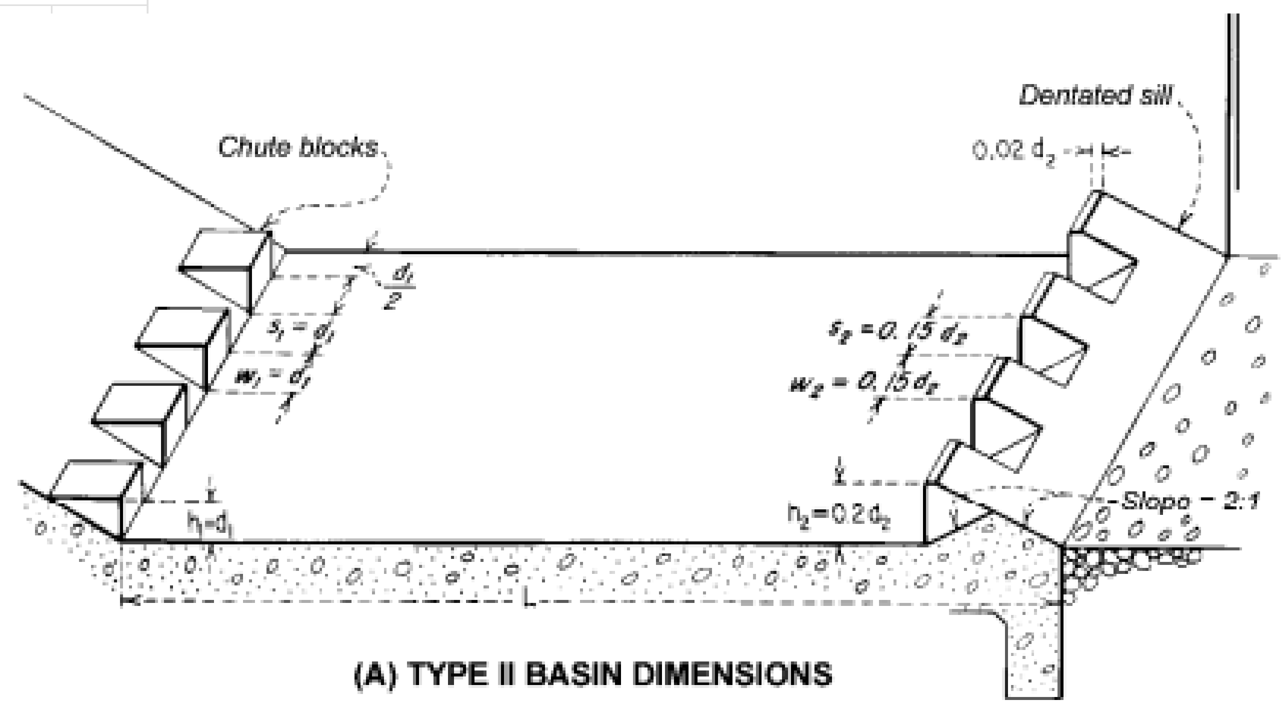


Figure D-3-13: Alternative 5: Structural RCC Dam Plan and Profile – Overtop Dam

NRCS

Assumptions:					Notes:
Horizontal aspect of Slope:	3	H:1V			Typcial are 2.5, 3, and 4, which appear to be based on dam geometries.
θ	0.32	rad			
θ	18.44	deg			
B (width)	200	feet			Based on Q (pmf), TOD, weir type. From SITES Results
Hcrest (AS: 1090.5, Invert:1022)	68.5	feet			Basin elev from d2 requirement, AS maintained.
Q (pmf)	14,855	cfs			Routing reservoir PMF through PS and AS in RAS
Initial Hydaulic Calcs:					
q (unit discharge)	74.3	cfs/ft			Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	feet			Std hydraulic calc (Critical depth)
L (Chute Length)	216.6	feet			Based on geometry
yn (Supercritical)	1.06	feet			Change until cell B15 matches B7. Go until Check in D15 is "yes".
Velocity	68.8	ft/s	Check ↓		Mannings Equation
Flow Check	14,584	cfs	yes		
Freeboard	3.8	feet			USBR Design of Small Dams (1987), Eq. 22 page 385.
Wall Height Reqrd	4.8	feet			
Stilling Basin (SB):					From HL360 & USBR (Frizell) HL-2012-02, recommend type III or IV.
q (unit discharge)	74.3	cfs/ft			PS flows, does this need to be added? Start with yes.
F (Froude #)	11.78				USBR EM25, 1984 Sec 1, Eq. 1 (Page 6); Froude # is ratio of inertial and gravitation forces, it governs the formation of free surface vorticies.
d1 (Incoming depth to SB)	1.06	feet			same as y and ycw; can also be called conjugate depth
d2 (Depth at end of basin)	17.1	feet			From USBR Engr Mono 25 8th printing (Peterka, 1984), Fig. 5; can also be called sequent depth
d2 elevation	1039.13	Check ↓	Result	Recommendation:	
RAS TW	1040	no	drowned-	move SB invert up	USBR 1987, page 397. If check is "no" see guidance how to correlate TW RC with basin RC. See below to apply 5% FOS against sweepout.
Type II fits USBR criteria					
Type II Basin feature Dimensions					
L (Total basin length)	73.7	feet			From USBR 1987 Fig. 9-42; values in column B are for F=11.78
d2 (+5%)	18.0				From USBR 1987 Fig. 9-42; values in column B are for F=11.78
Chute Block Width & Spacing	1.1	feet			
Chute & Floor Block Height	1.1	feet			
Dentated Sill Width & Spacing	2.6	feet			
Dentated Sill Width & Spacing	3.4	feet			
Riprap below Stilling Basin:					HL360 (Use Ishbash or USBR for Type III)
Avg. channel velocity	4.13				Hydraulic Calcs (Q=V*A)
D50 (Ishbash)	6	inches			NEH Part 654 TS14C- Fig. 5
D50 (USBR)	0.23	feet			NEH Part 654 TS14C Eq. 9
D50 (USBR)	2.71	inches			
	F (Froude #)	q (unit Q)	Note (FEMA, 2010)		
Hydraulic Conditions	11.78	74.3	See above		
__Basin Type__					
Type I	< 1.7 - 2.5		Page 49		
Type II	>4.5	< 500	Page 49		
Type III	>4.5	<60	Page 50		
Type IV (Alternative Low Froude)	2.5-4.5		Page 53,56		
SAF	1.7 - 17		Low FOS		



(A) TYPE II BASIN DIMENSIONS

Figure D-3-14: Alternative 6: Structural Chute Computations - Current Auxiliary Spillway with Type II Stilling Basin

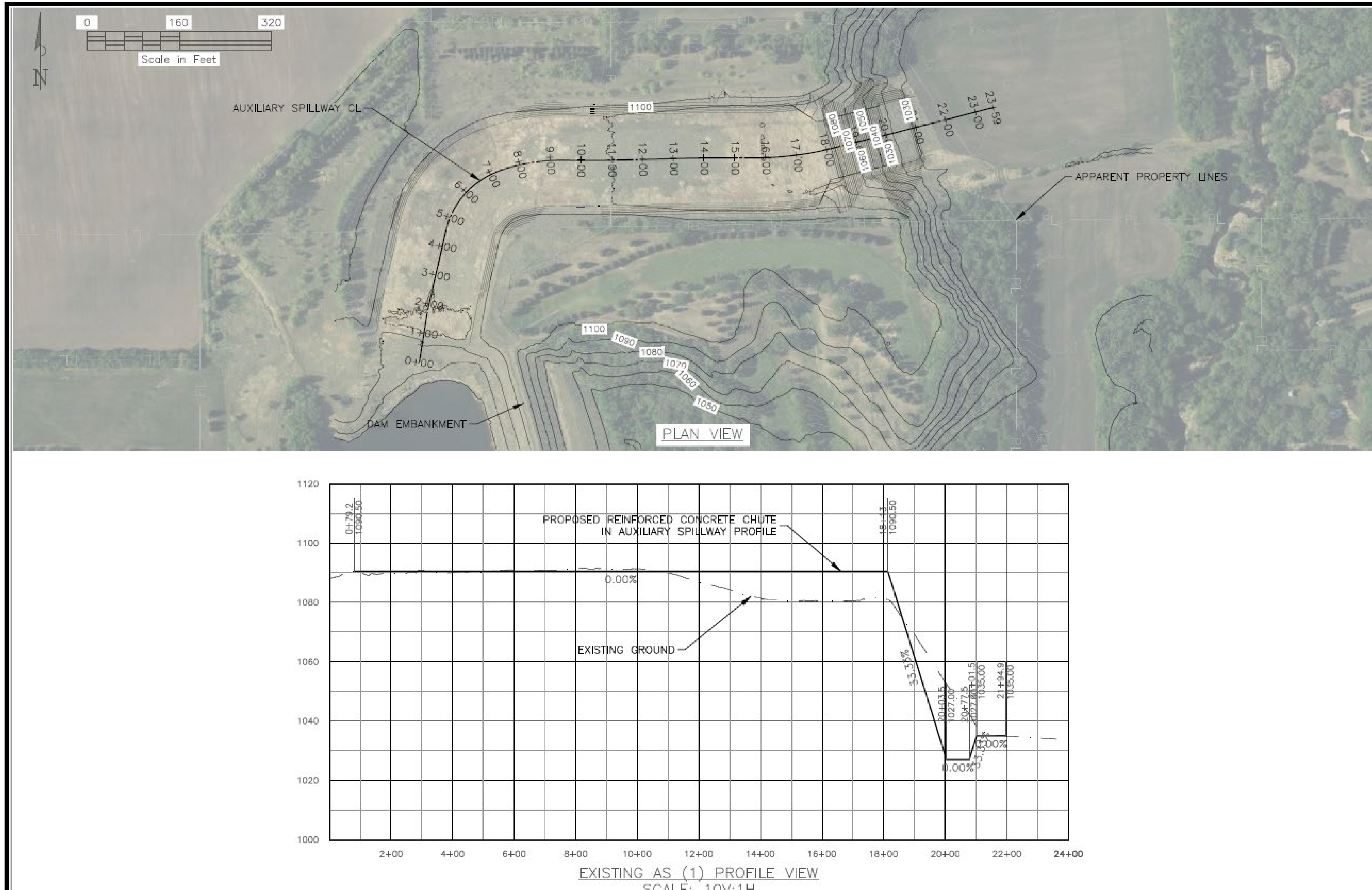


Figure D-3-15: Alternative 6: Structural Chute Plan and Profile - Current Auxiliary Spillway

Assumptions:				**Notes:**
Horizontal aspect of Slope:	3	H:1V		Typcial are 2:1 or flatter (USBR 1987, page 358)
θ	0.32	rad		
θ	18.44	deg		
B (width)	200	feet		Spillway capacity and 1' residual freeboard to TOD
Hcrest (AS: 1090.5, Invert:1035)	55.5	feet		Basin elev from stream geometry, AS from PSH elev.
AS Q (pmf)	14,855	cfs		Routing reservoir PMF through PS and AS in RAS
Initial Hydaulic Calcs:				
q (unit discharge)	74.3	cfs/ft		Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	feet		Std hydraulic calc (Critical depth)
L (Chute Length)	175.5	feet		Based on geometry
yn (Supercritical)	1.06	feet		Change until cell B15 matches B7. Go until Check in D15 is "ye
Velocity	68.8	ft/s	Check ↓	Mannings Equation
Flow Check	14,584	cfs	yes	Within 2% of AS Q (pmf)
Baffle Design Calcs:				
H (baffle pier height)	4.4	feet		USBR 1987, page 360. Asume 0.8*Dc
W & S_h (width and horizontal	6.7	feet		USBR 1987, page 360. Asume 1.5*H
S_{LS} (Sloped Longitudinal spacing)	13.3	feet		USBR 1987, page 360. Asume H*slope is max
S_{LH} (Horizontal Longitudinal spacing)	12.6	feet		Geometry
R (Rows)	13			USBR 1987, page 360. L / S
D_T (Depth, top)	0.9	feet		USBR 1987, page 361. Asume 0.2*H
D_B (Depth, Bottom)	3.1	feet		USBR 1987, page 361. Asume H/2 + 0.2*H
Wall Height	13.3	feet		USBR 1987, page 361 & 362. Asume 3*H
Calculations by:	Jon Petersen, NRCS			
Checked by:				
Date				

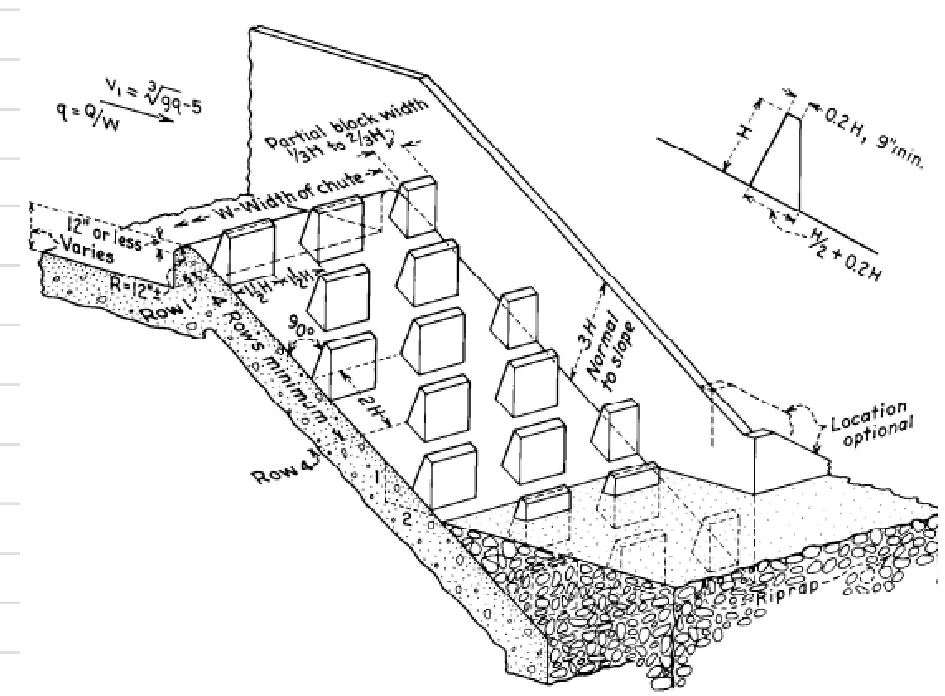


Figure 9-18.—Basic proportions of a baffled chute spillway. 288-D-2807.

Figure D-3-16: Alternative 7: Structural Chute Computations - Current Auxiliary Spillway with Baffle Blocks

NRCS

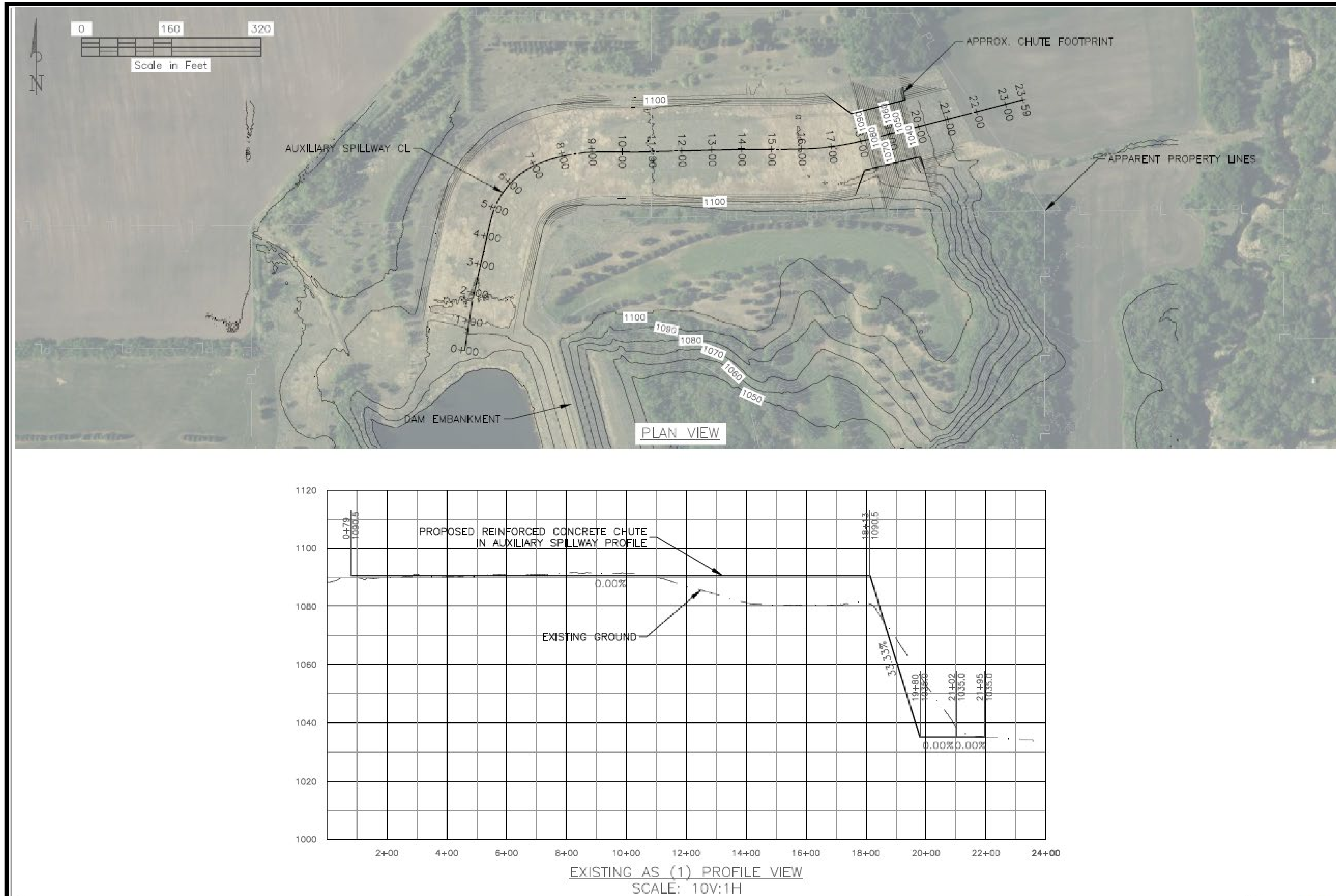
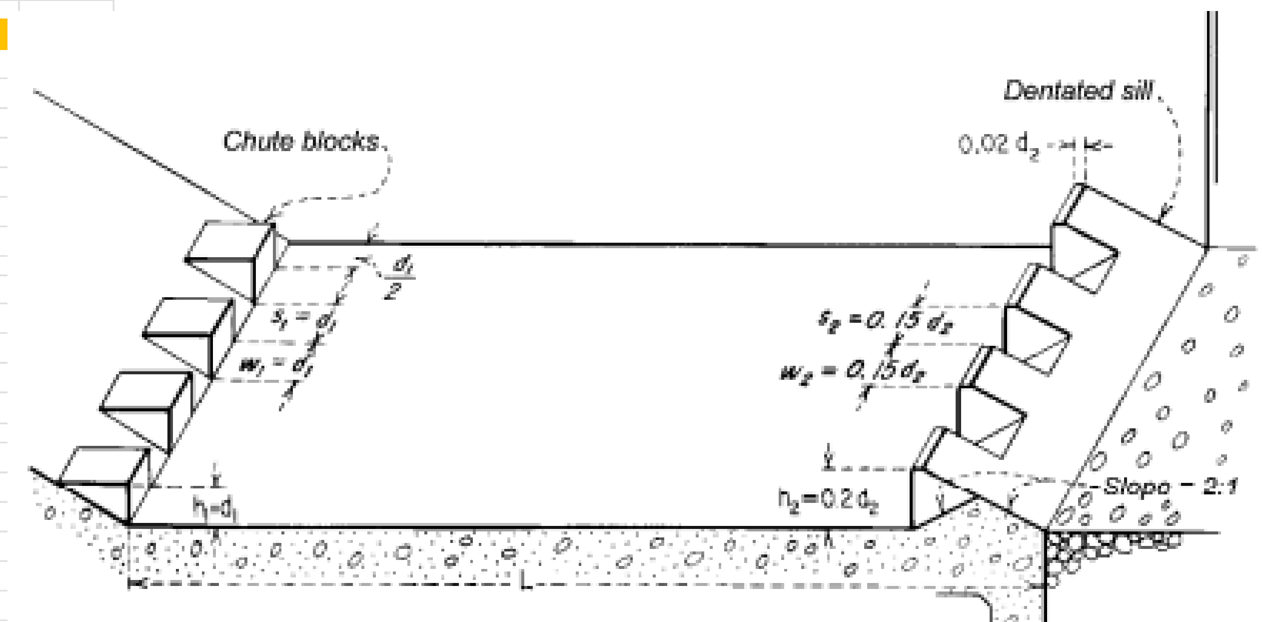


Figure D-3-17: Alternative 7: Structural Chute Plan and Profile - Current Auxiliary Spillway with Baffle Blocks

Assumptions:					Notes:
Horizontal aspect of Slope:	3	H:1V			Typcial are 2.5, 3, and 4, which appear to be based on dam geometries.
θ	0.32	rad			
θ	18.44	deg			
B (width)	200	feet			Based on Q (pmf), TOD, weir type. From SITES Results
Hcrest (AS: 1090.5, Invert:1027)	63.5	feet			Basin elev from d2 requirement, AS maintained.
Q (pmf)	14,855	cfs			Routing reservoir PMF through PS and AS in RAS
Initial Hydaulic Calcs:					
q (unit discharge)	74.3	cfs/ft			Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	feet			Std hydraulic calc (Critical depth)
L (Chute Length)	200.8	feet			Based on geometry
yn (Supercritical)	1.06	feet			Change until cell B15 matches B7. Go until Check in D15 is "yes".
Velocity	68.8	ft/s	Check ↓		Mannings Equation
Flow Check	14,584	cfs	yes		
Freeboard	3.8	feet			USBR Design of Small Dams (1987), Eq. 22 page 385.
Wall Height Reqrd	4.8	feet			
Stilling Basin (SB):					From HL360 & USBR (Frizell) HL-2012-02, recommend type III or IV.
q (unit discharge)	74.3	cfs/ft			PS flows, does this need to be added? Start with yes.
F (Froude #)	11.78				USBR EM25, 1984 Sec 1, Eq. 1 (Page 6); Froude # is ratio of inertial and gravitation forces, it governs the formation of free surface vorticies.
d1 (Incoming depth to SB)	1.06	feet			same as y and ycw; can also be called conjugate depth
d2 (Depth at end of basin)	17.1	feet			From USBR Engr Mono 25 8th printing (Peterka, 1984), Fig. 5; can also be called sequent depth
d2 elevation	1044.13	Check ↓	Result	Recommendation:	
RAS TW	1045.1	no	drowned-	move SB invert up	USBR 1987, page 397. If check is "no" see guidance how to correlate TW RC with basin RC. See below to apply 5% FOS against sweepout.
Type II fits USBR criteria					
Type II Basin feature Dimensions					
L (Total basin length)	73.7	feet			From USBR 1987 Fig. 9-42; values in column B are for F=11.78
d2 (+5%)	18.0				From USBR 1987 Fig. 9-42; values in column B are for F=11.78
Chute Block Width & Spacing	1.1	feet			
Chute & Floor Block Height	1.1	feet			
Dentated Sill Width & Spacing	2.6	feet			
Dentated Sill Width & Spacing	3.4	feet			
Riprap below Stilling Basin:					HL360 (Use Ishbash or USBR for Type III)
Avg. channel velocity	3.22				Hydraulic Calcs (Q=V*A)
D50 (Ishbash)	6	inches			NEH Part 654 TS14C- Fig. 5
D50 (USBR)	0.14	feet			NEH Part 654 TS14C Eq. 9
D50 (USBR)	1.62	inches			
	F (Froude #)	q (unit Q)	Note (FEMA, 2010)		
Hydraulic Conditions	11.78	74.3	See above		
Basin Type					
Type I	< 1.7 - 2.5		Page 49		
Type II	>4.5	< 500	Page 49		
Type III	>4.5	<60	Page 50		
Type IV (Alternative Low Froude)	2.5-4.5		Page 53,56		
SAF	1.7 - 17	<60	Low FOS		



(A) TYPE II BASIN DIMENSIONS

Figure D-3-18: Alternative 8: Structural Chute Computations - Overtop Dam with Type II Stilling Basin

NRCS

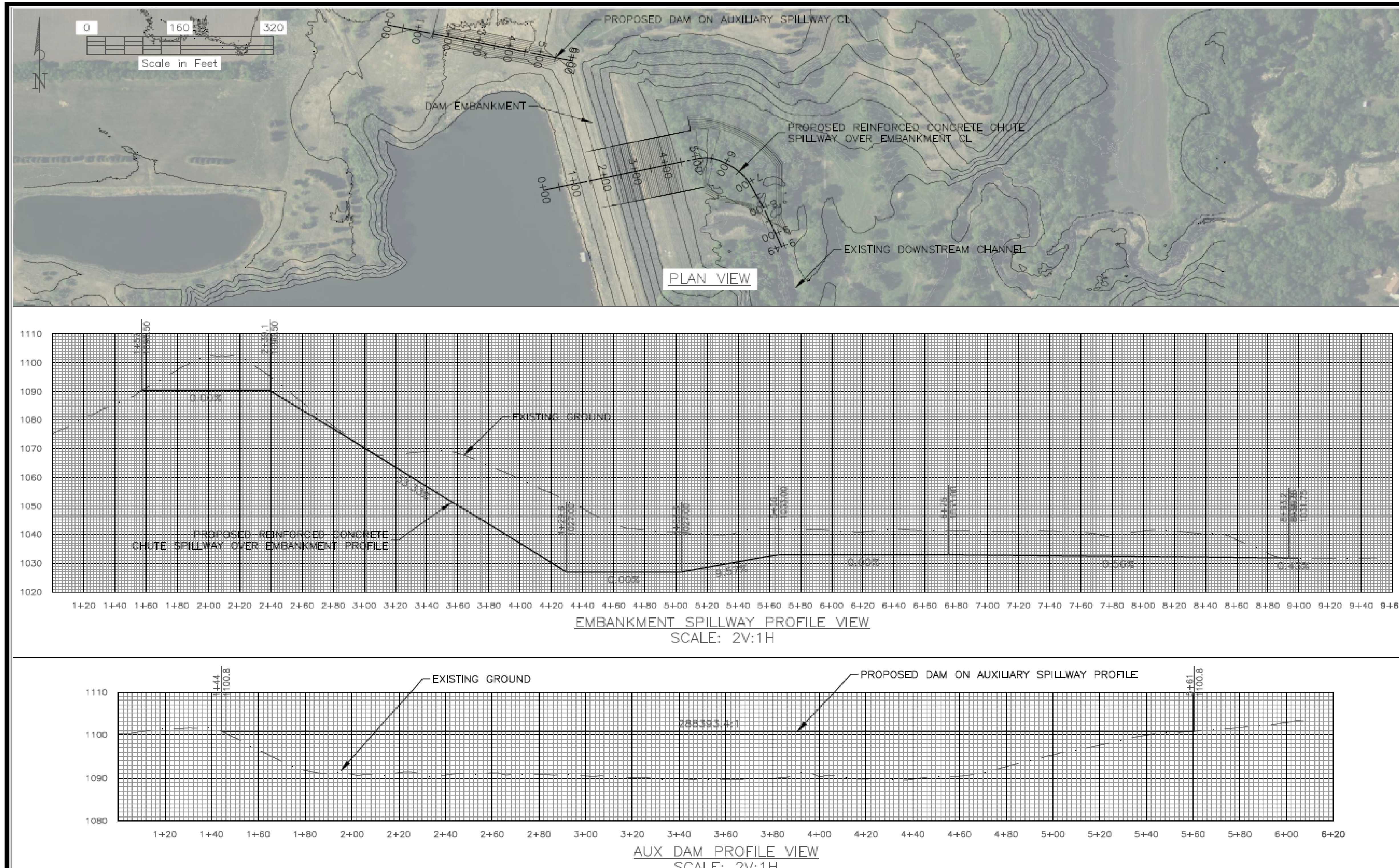


Figure D-3-19: Alternative 8: Structural Chute Plan and Profile - Over Top Dam with Type II Stilling Basin

Assumptions:				**Notes:**
Horizontal aspect of Slope:	3	H:1V		Typcial are 2:1 or flatter (USBR 1987, page 358)
θ	0.32	rad		
θ	18.44	deg		
B (width)	200	feet		Spillway capacity and 1' residual freeboard to TOD
Hcrest (AS: 1090.5, Invert:1040)	50.5	feet		Basin elev from stream geometry, AS from PSH elev.
AS Q (pmf)	14,855	cfs		Routing reservoir PMF through PS and AS in RAS
Initial Hydaulic Calcs:				
q (unit discharge)	74.3	cfs/ft		Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	5.6	feet		Std hydraulic calc (Critical depth)
L (Chute Length)	159.7	feet		Based on geometry
yn (Supercritical)	1.06	feet		Change until cell B15 matches B7. Go until Check in D15 is "ye
Velocity	68.8	ft/s	Check ↓	Mannings Equation
Flow Check	14,584	cfs	yes	Within 2% of AS Q (pmf)
Baffle Design Calcs:				
H (baffle pier height)	4.4	feet		USBR 1987, page 360. Asume 0.8*Dc
W & S_h (width and horizontal	6.7	feet		USBR 1987, page 360. Asume 1.5*H
S_{LS} (Sloped Longitudinal spacing)	13.3	feet		USBR 1987, page 360. Asume H*slope is max
S_{LH} (Horizontal Longitudinal spacing)	12.6	feet		Geometry
R (Rows)	12			USBR 1987, page 360. L / S
D_T (Depth, top)	0.9	feet		USBR 1987, page 361. Asume 0.2*H
D_B (Depth, Bottom)	3.1	feet		USBR 1987, page 361. Asume H/2 + 0.2*H
Wall Height	13.3	feet		USBR 1987, page 361 & 362. Asume 3*H
Calculations by:	Jon Petersen, NRCS			
Checked by:				
Date				

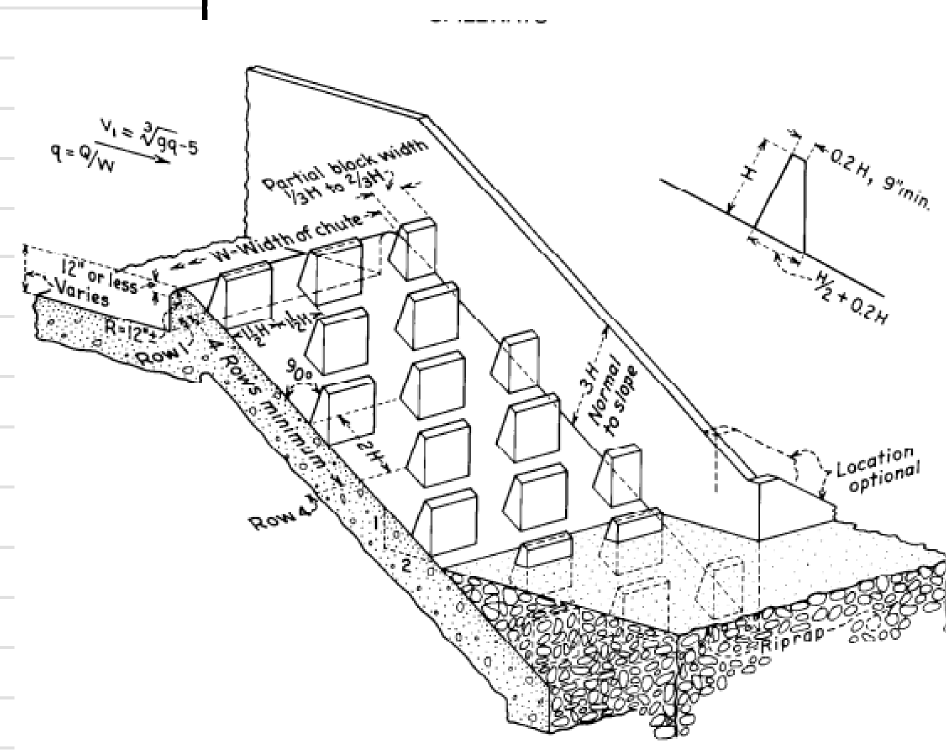


Figure 9-18.—Basic proportions of a baffled chute spillway. 288-D-2807.

Figure D-3-20: Alternative 9: Structural Chute Computations - Overtop Dam with Baffle Blocks

NRCS

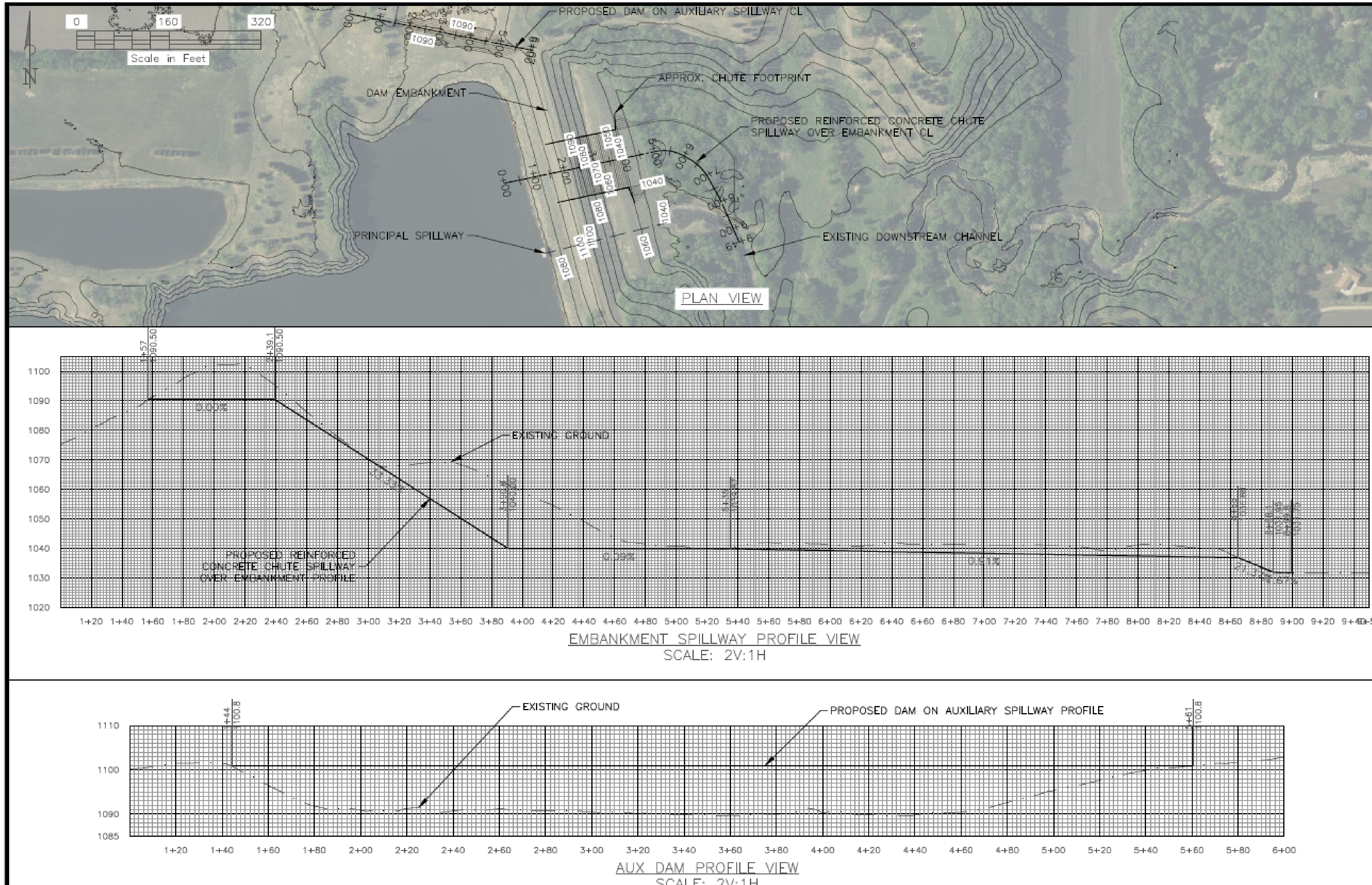


Figure D-3-21: Alternative 9: Structural Chute Plan and Profile - Over Top Dam with Baffle Blocks

NRCS

SITES Integrated Development Environ	UTR9_FBH_SigHzrd
File View Windows Help	9
	N/A
Site Identification	65.50
Watershed Runoff Curve Number	N/A
Total Watershed Drainage Area (Sq.Miles)	N/A
Watershed Time of Concentration (Hours)	N/A
SDH Rainfall Total (Inches)	N/A
SDH Rainfall Duration (Hours)	N/A
FBH or Storm Rainfall Total (Inches)	1714.3
FBH or Storm Rainfall Duration (Hours)	6435.4
SDH Inflow Peak (CFS)	1076.80
FBH or Storm Inflow Peak (CFS)	1086.53
Initial Reservoir Elevation (Feet)	1094.96
Maximum WS SDH (Feet)	6358.8
Maximum WS FBH or Storm (Feet)	1094.96
Storage at Max. WS FBH or Storm (Acre-Ft)	6299.0
Top Dam (Feet)	N/A
Storage, Top Dam (Acre-Ft)	N/A
Emb. Yardage (CY)	N/A
PSH Drawdown (Days)	1068.90
378 Drawdown (Days)	1
PS Crest (Feet)	N/A
PS Number of Conduits	6.00
PS Conduit Diameter (Inches)	6.00
PS Conduit Height (Feet)	36.00
PS Conduit Width (Feet)	672.
PS Conduit Area (Sq. Feet)	1316.2
Storage, PS Crest (Acre-Ft)	891.4
PS Discharge at AS Crest (CFS)	1371.2
PS Discharge for SDH (CFS)	1090.50
PS Discharge FBH or Storm (CFS)	4434.2
AS Crest (Feet)	
Storage, AS Crest (Acre-Ft)	300.
AS Width (Feet)	3.00
AS Exit Slope (%)	5.60
AS Ret. Curve Index	0.85
AS Veg. Cover Factor	2
AS Maintenance Code	0.00
AS Max. Head SDH (Feet)	N/A
AS Peak Discharge SDH/Storm (CFS)	N/A
AS Exit Velocity SDH or Storm (Ft/S)	N/A
AS Stress SDH or Storm (Lb./Sq.Ft.)	4.46
Hp FBH or Storm (Feet)	4380.
AS Peak Discharge FBH/Storm (CFS)	357.
AS Integ. Dist. FBH or Storm (Feet)	30.8
Oe/B FBH or Storm (Acre-Ft/Ft)	65.50
Uncontrolled Drainage Area (Sq.Miles)	0
Number of Errors	0
Number of Warnings	

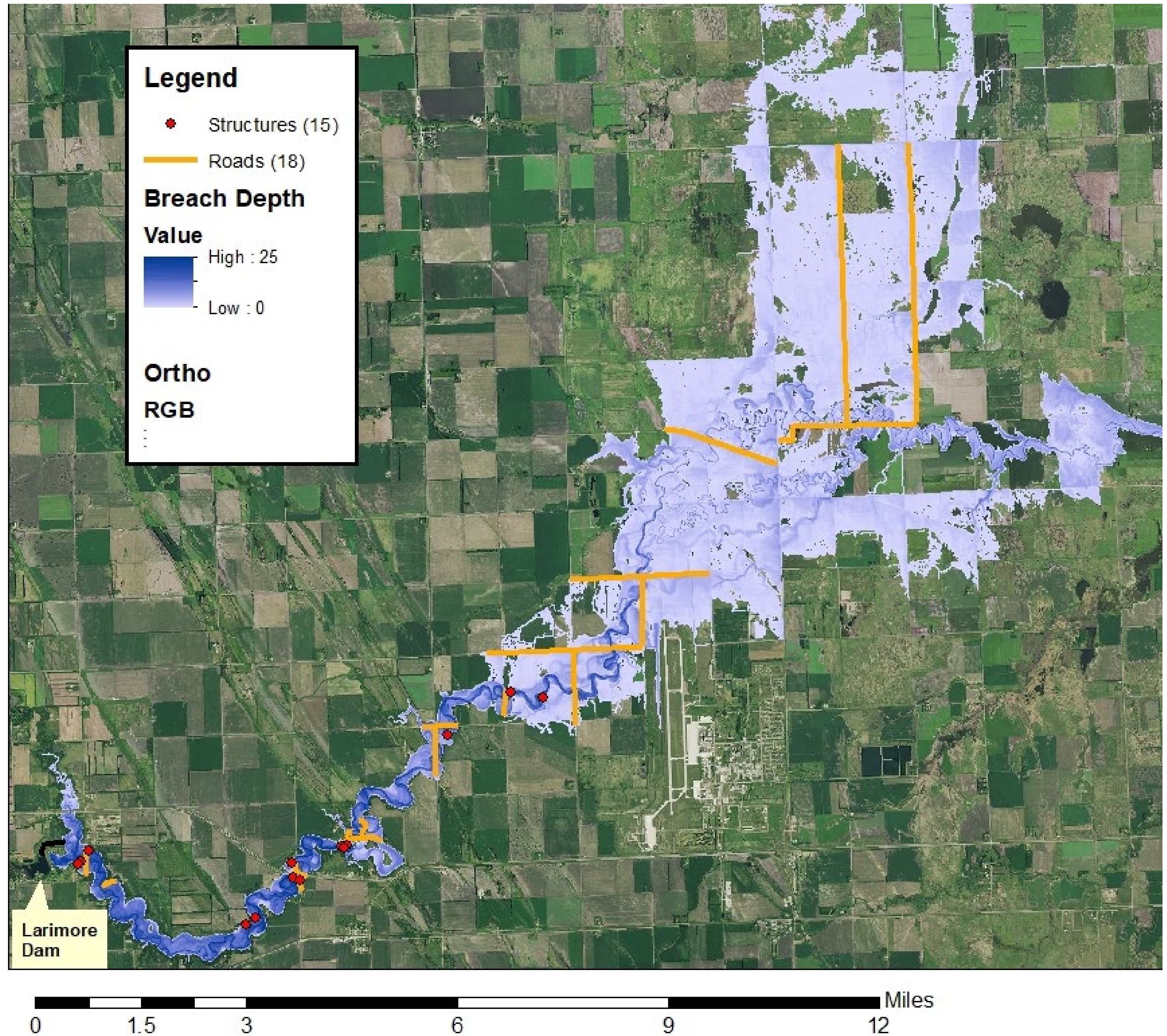


Figure D-3-22: Change to Significant Hazard Classification – Structures and Roads/Bridges Impact

NRCS

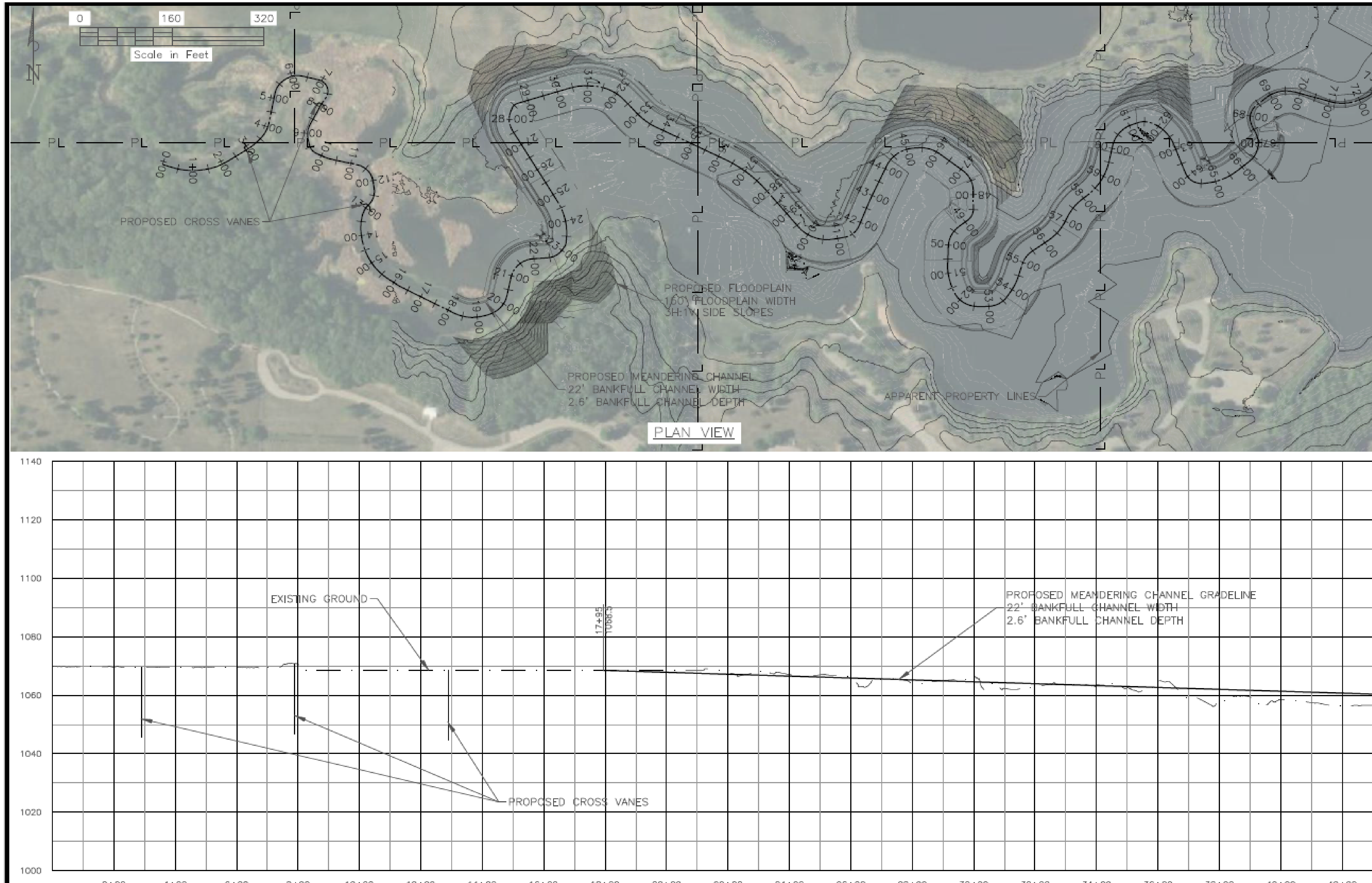


Figure D-3-23: Non-Structural Decommissioning Plan and Profile (Upper)

NRCS

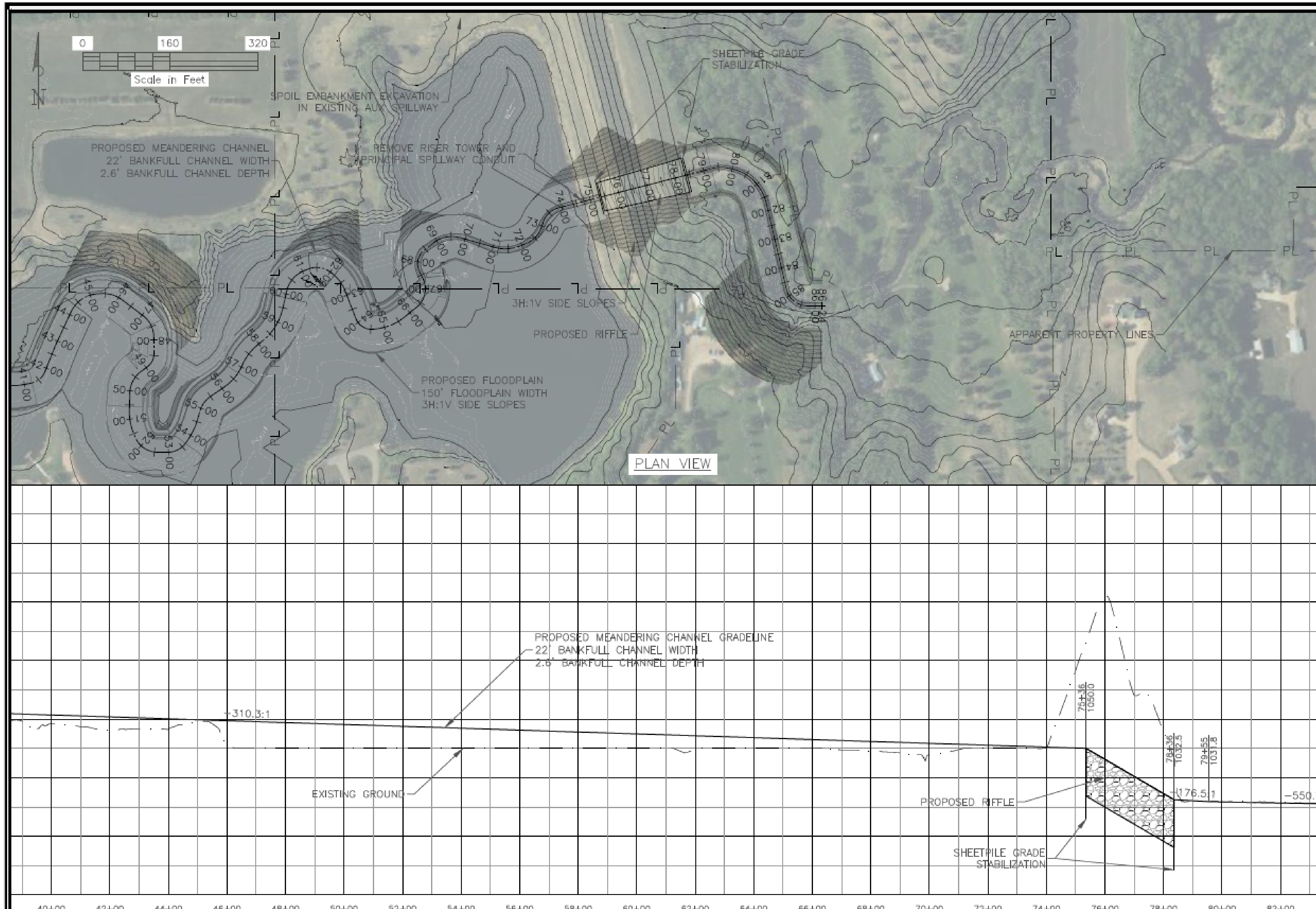


Figure D-3-24: Non-Structural Decommissioning Plan and Profile (Lower)

NRCS

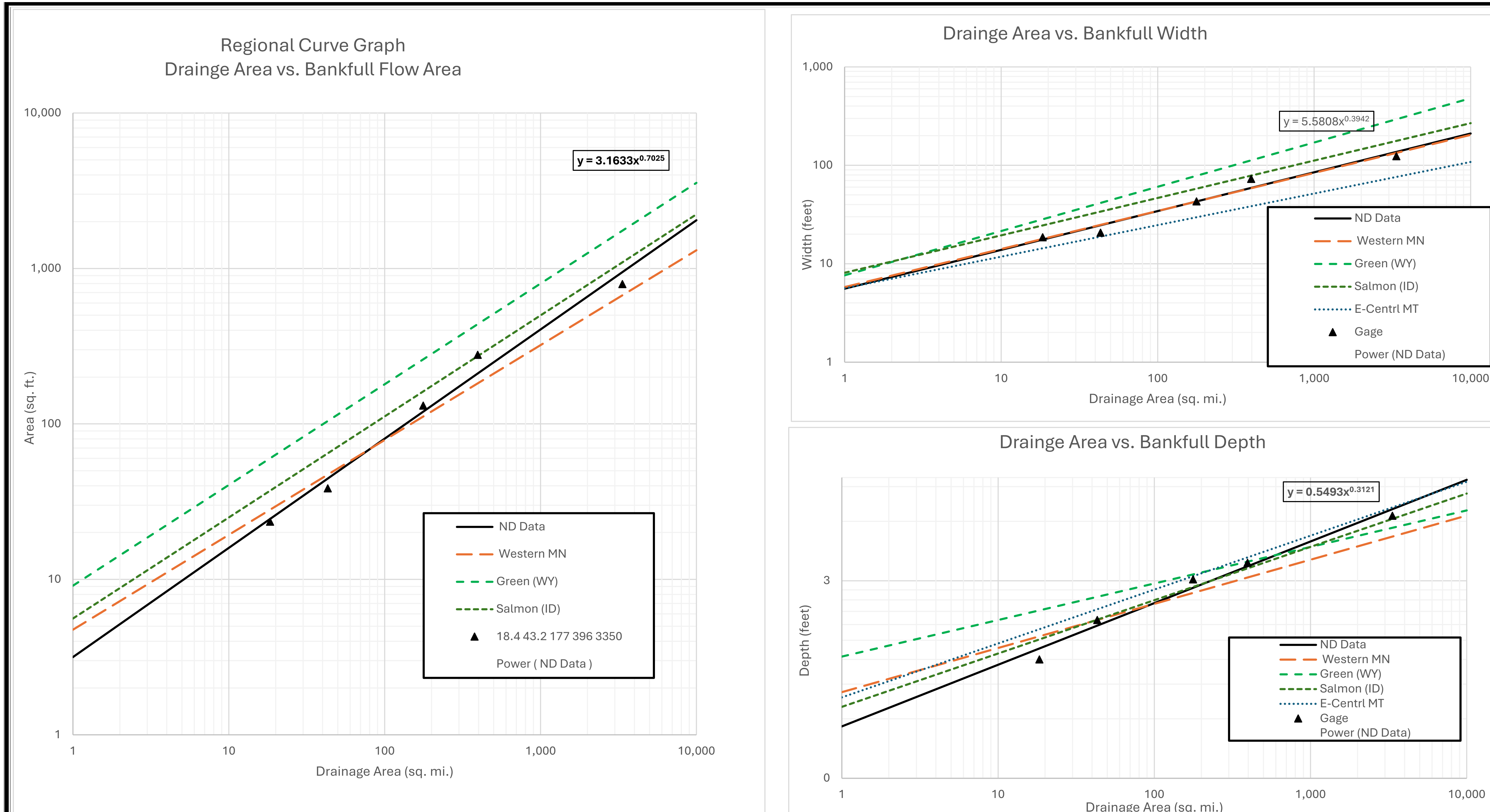


Figure D-3-25: Non-Structural Decommissioning Regional Curves

NRCS

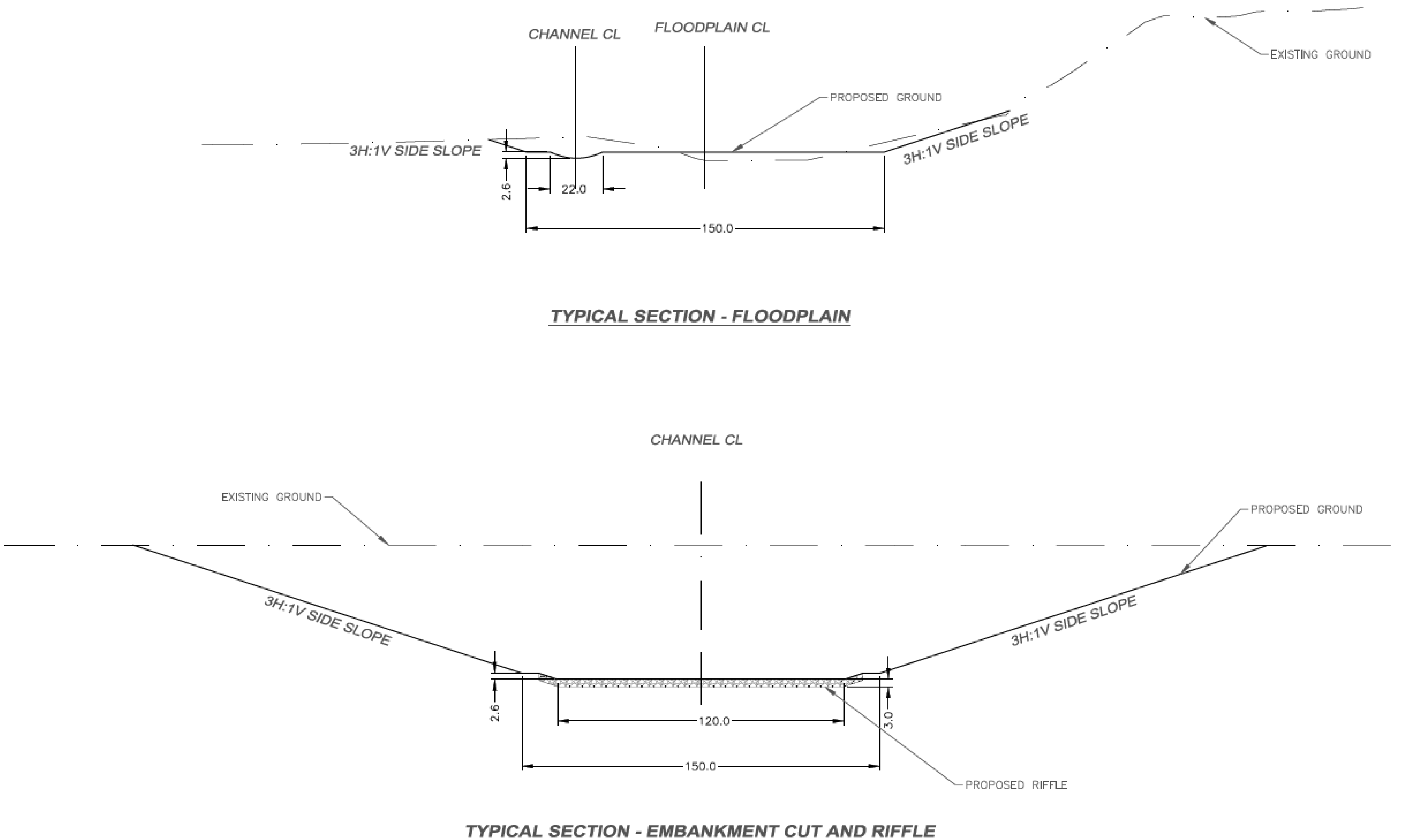


Figure D-3-26: Non-Structural Decommissioning Typical Cross Sections

NRCS

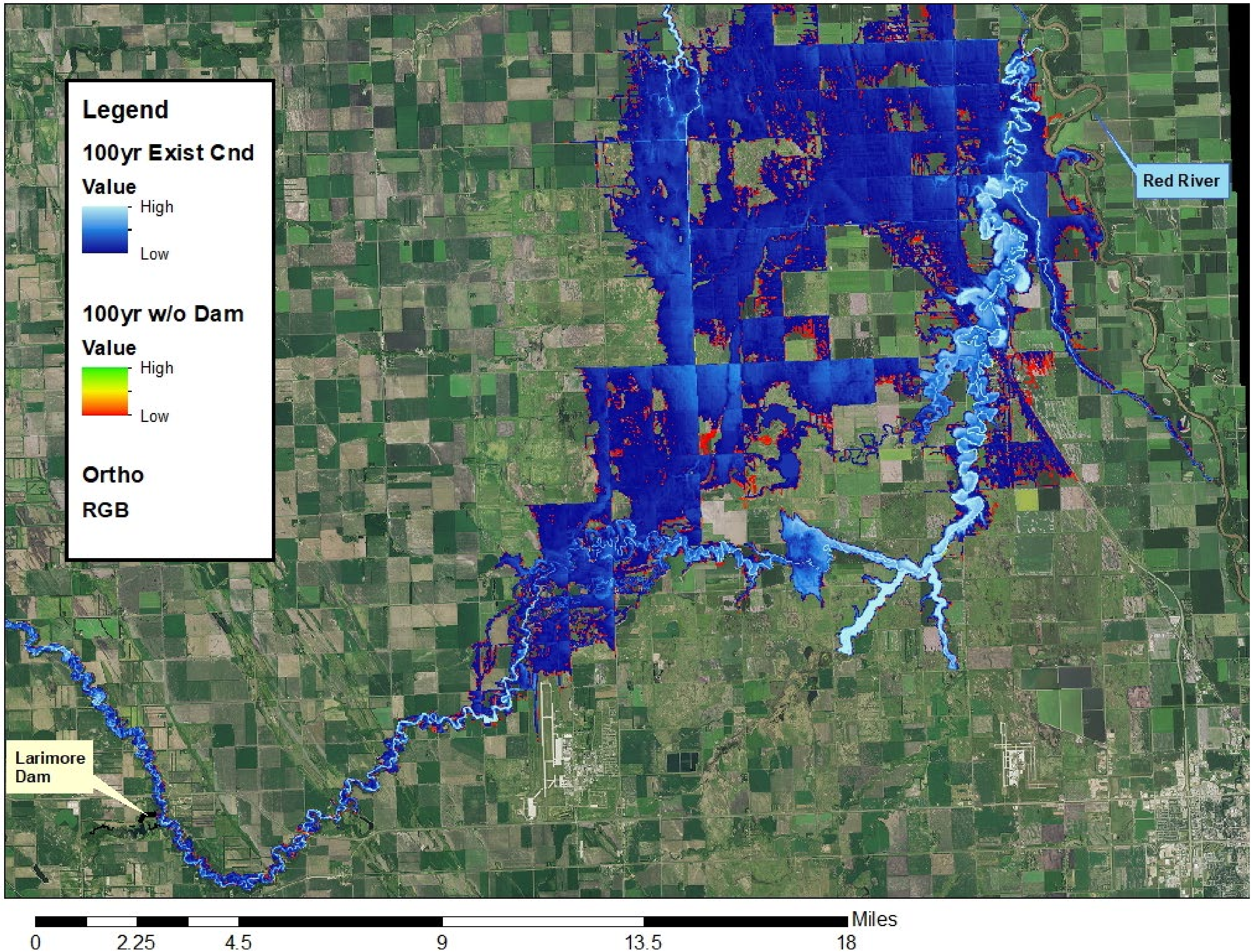


Figure D-3-27: Non-Structural Decommissioning Downstream 100-yr Flood Max Depth Change

Figure D-3-28: Non-Structural Decommissioning Downstream Infrastructure Protection Levee Alignments and Bridge Replacements

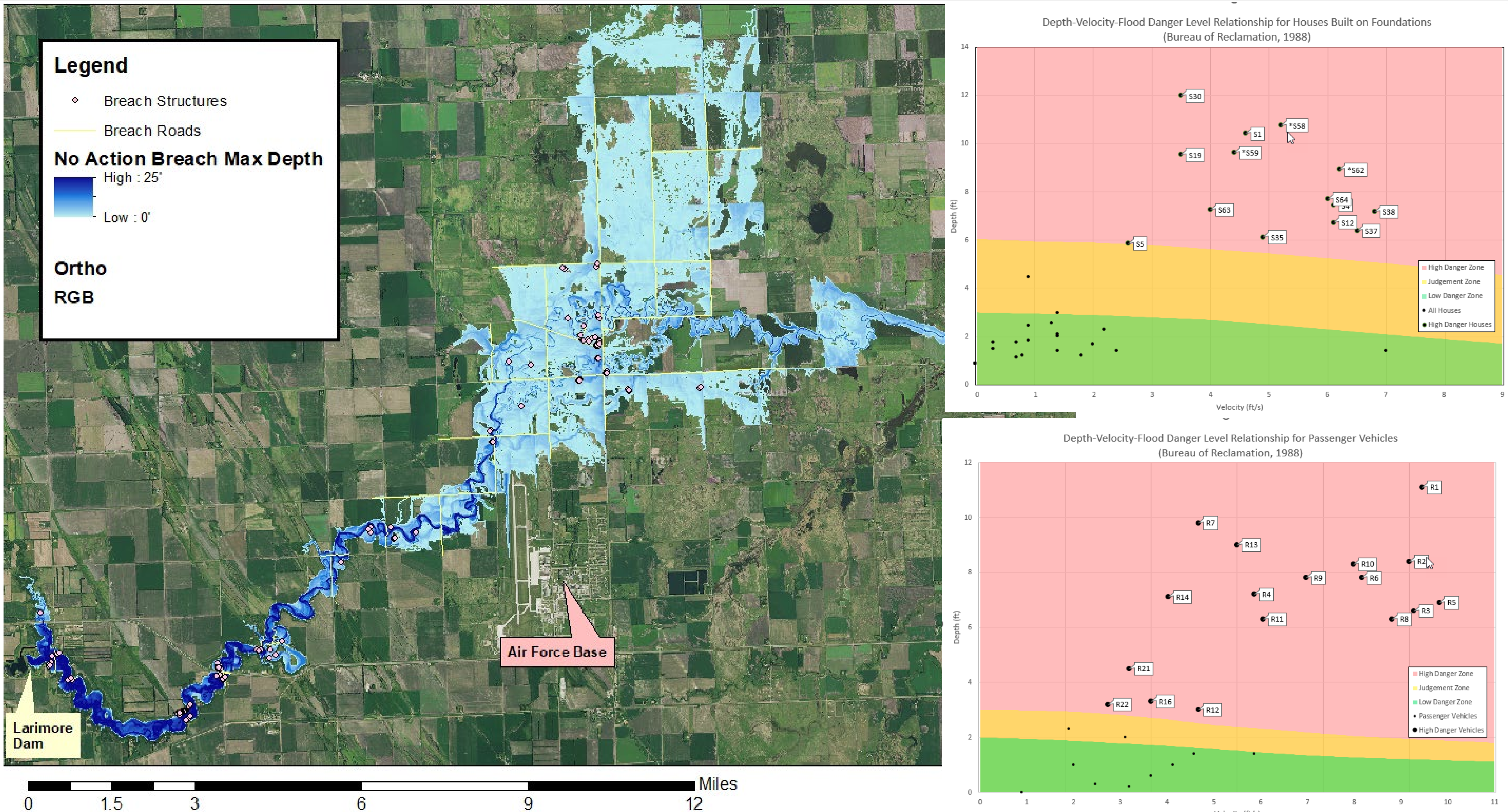


Figure D-3-29: No Action Dam Breach Event – Inhabitable Structures and Roads Impact

NRCS

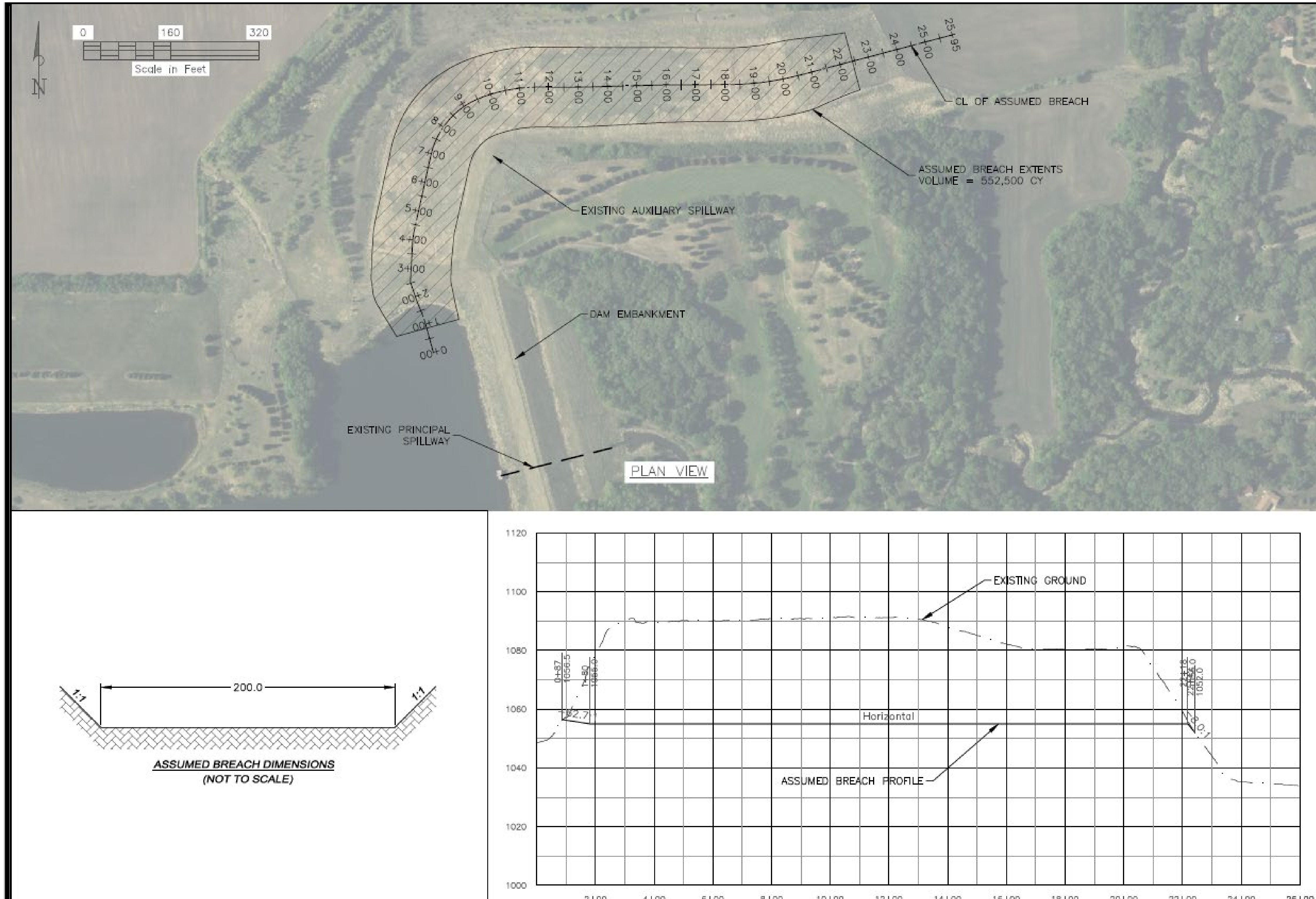


Figure D-3-30: No Action Dam Breach Event – Headcut Breach footprint and Cross Section

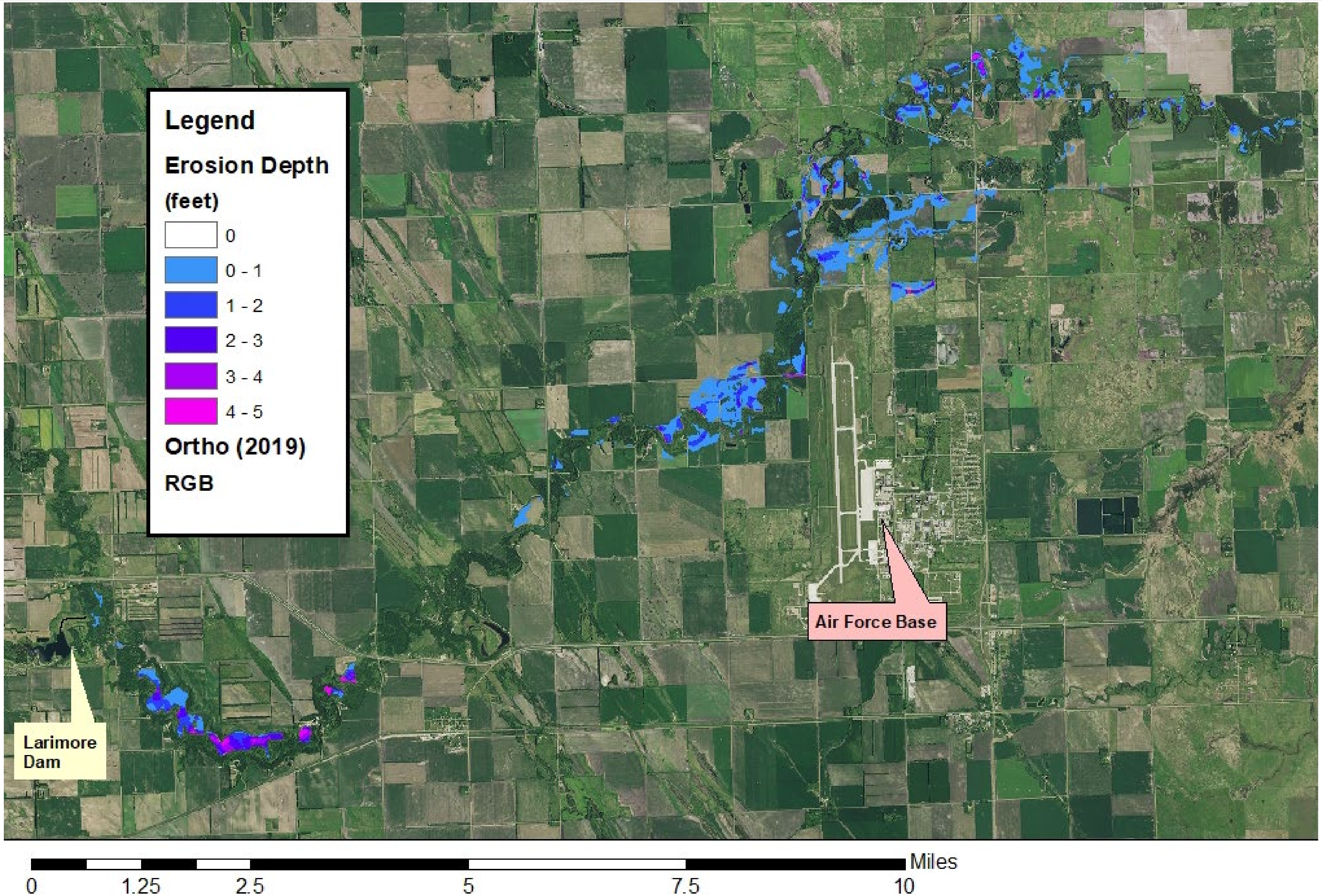


Figure D-3-31: No Action Dam Breach Event – Floodplain Erosion Depth

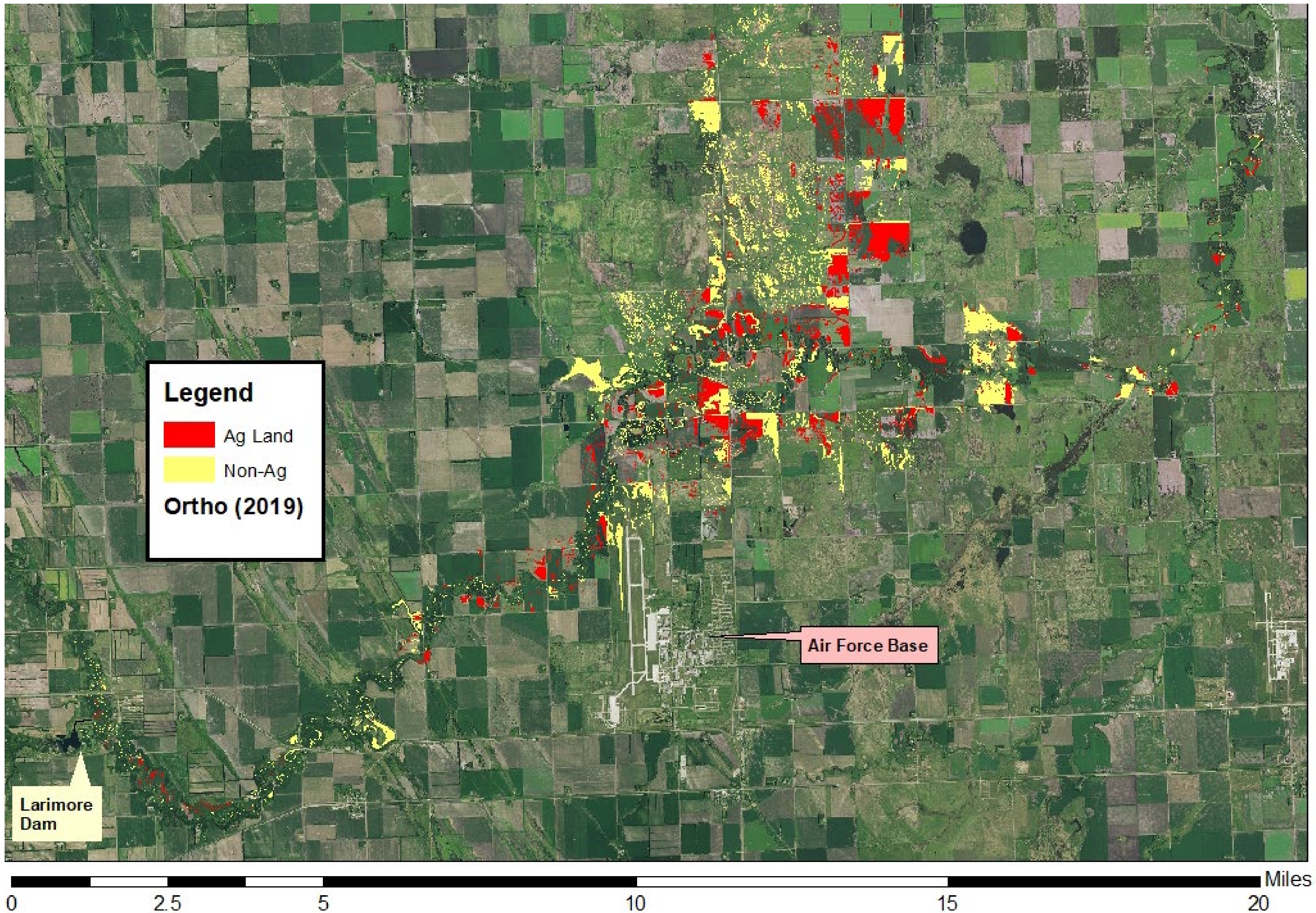


Figure D-3-32: No Action Dam Breach Event – Floodplain Deposition Areas