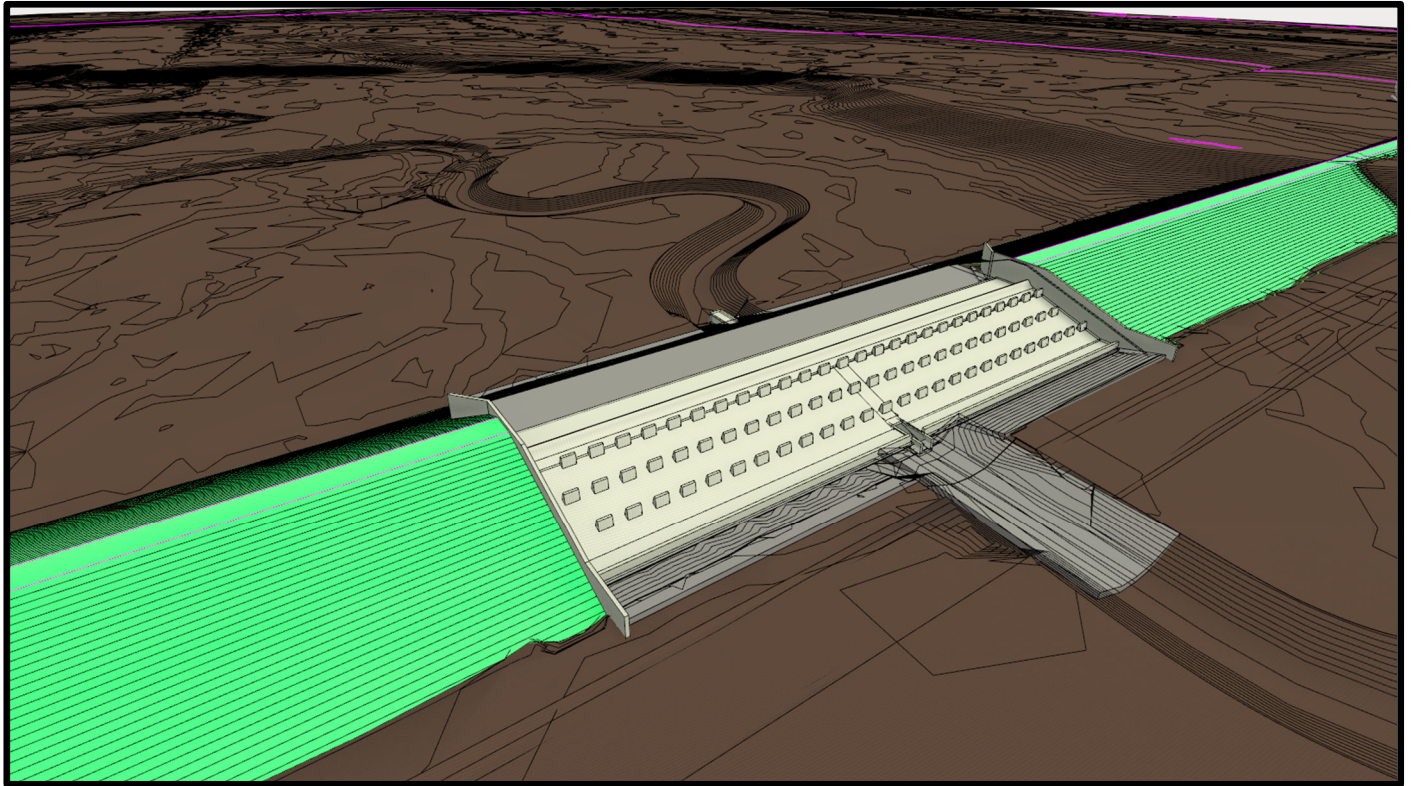




Upper Turtle River Dam 10 Grand Forks County Water Resource District Concept Design Report



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1 Project Summary

1.1 NRCS Practice Standard and Job Class

CPS 402: Dam	Job Class VII (high hazard, DA > 50 sqmi)
CPS 582: Open Channel	Job Class III (Q= 256 cfs, v=2.3 fps)
CPS 560: Access Road	Job Class V (L > 3 miles)

1.2 Project Location

The project is located on the North Branch Turtle River, within Elm Grove Township of Grand Forks County, in Section 3, Township 152, Range 55. The Latitude is 48.011 and Longitude -97.676. It is located within Hydrologic Unit 0902030701, downstream of Upper Turtle River Dams #1, #2, #4, and #5, constructed through the SCS Watershed Program in 1974-1980.

1.3 Project Data Summary

Table 1: Project Dam Summary

General data		
Purpose	Flood Control	
Hazard Classification	High Hazard	
Drainage Area	158.7 square miles	
Dam Height (Average)	27 feet	
Maximum Dam Height	35 feet	
Embankment Length	3,000 feet	
Embankment Top Width	14 feet	
Embankment Upstream Slope	3H:1V	
Embankment Downstream Slope	2.5H:1V	
Critical Elevations (NAVD88)		
Principal Spillway Invert	1102.0 feet	
Auxiliary Spillway Crest	1127.0 feet	
Top of Dam	1136.0 feet	
Storage Capacities	Volume	Surface Area
Principal Spillway Invert (Dry Dam)	0 acre-feet	0 acres
Auxiliary Spillway Crest	7,000 acre-feet	849 acres
Top of Dam	20,300 acre-feet	2,771 acres
Spillway Features	Dimensions	Area
Principal Spillway Opening	6' x 6'	36 ft²
Principal Spillway Extent	160 feet	



Auxiliary Spillway Length	360 Feet
Auxiliary Spillway Crest Type	Ogee
Auxiliary Spillway Chute Type	Baffle Rows

UTR Dam 10 Elevation Summary	Elevation (NAVD88, ft)	Storage Volume (acre-feet)	Surface Area (acres)
Principal Spillway Invert (Dry Dam)	1102	0	0
2-year	1113.9	486	180
5-year	1117.3	1,389	343
10-year	1120.2	2,594	487
25-year	1125.5	5,809	750
Auxiliary Spillway Crest	1127	7,000	849
50-year	1127.8	7,757	902
100-year	1128.7	8,635	1,005
500-year	1130.2	10,279	1,229
Stability Design Hydrograph	1130.2	10,262	1,229
Freeboard Hydrograph	1134.8	17,674	2,026
Top of Dam	1136	20,300	2,771

2 Survey

2.1 LiDAR Survey

High-resolution LiDAR data was utilized for significant portions of hydraulic modeling efforts, including floodplain elevations, valley extents, and channel dimensions outside dam extents. This area was part of the James River Basin QL2 Phase 8, Block C collect (ND DWR, 2023). The flights were completed between 5/30/2019 and 4/22/2021, and represent bare-earth elevations. The raster digital elevation model horizontal resolution is 1.0 meters, and vertical accuracy is 10 cm or better.

2.2 GPS Surveys

RTK GPS surveys were completed in the fall of 2020 for the UTR 10 Dam footprint area. Field survey data from within the channel was merged with the LiDAR to generate a final surface and quantities.

3 Site Characteristics

The site characteristics are based on criteria provided by Technical Release 210-60: Earth Dams and Reservoirs (USDA NRCS, 2019a), hereafter TR-60, and North Dakota Dam Safety Standards (ND DWR, 2024).

3.1 Embankment

The top of dam is elevation 1136; approximate height of the dam is 28 feet above floodplain level. The basis for this is to maximize flood pool storage, while minimizing embankment length. Along the top of



terrace, beyond elevation contour 1136 on west abutment, is very flat and expansive. The top width is 14 feet. See section 7.0 for more details on embankment structure.

3.2 Principal Spillway

The reservoir is a dry dam; therefore no permanent pool will exist. The principal spillway is a 6 foot by 6 foot box culvert placed at grade with river channel. The basis for dimensions are to maximize downstream flood reductions, while meeting TR-60 Principal Spillway Design Hydrographs criteria; more details about this analysis can be found in Section 5.1. The principal spillway outlets to same stilling basin as auxiliary spillway.

3.3 Auxiliary Spillway

The auxiliary spillway is a structural chute spillway with ogee crest and chute baffles. The ogee crest is used to maximize overflow efficiency, which reduces entire spillway width. There are three rows of chute baffles, which eliminate the need for extensive stilling basin to dissipate energy. The auxiliary spillway width is 360 feet. The basis for dimensions are to meet TR-60 Auxiliary Spillway and Freeboard Design Hydrographs criteria and North Dakota Dam Safety criteria; more details about this analysis can be found in Sections 5.2 and 5.3.

3.4 Wetlands

There are 1.69 acres of RIVERINE wetlands that are expected to be permanently lost, see **Figure 1**. The wetlands will be filled and hydrology cutoff as well. The new channel connecting delineated wetlands to dam principal spillway are considered created RIVERINE wetlands totaling 2.28 acres, see **Figure 1**. The created channel and wetlands meet adjacent river planform parameters as well as channel dimensions from regional curves. Final design channel features need to meet or exceed functions lost from the 1.69-acre area. The wetlands created being greater than wetlands lost negates the need for wetland mitigation.

4 Hazard Classification

4.1 Breach Criteria and Results

The peak discharge criteria for the dam breach were developed using equations found in Chapter 1 of TR-60. The failure or breach of the dam is to be evaluated with the water surface elevation of the reservoir at the dam crest or the peak reservoir stage resulting from the probable maximum flood (PMF). The peak breach discharge calculated for the proposed conditions at UTR 10 Dam is approximately 31,000 cubic feet per second (cfs). Peak breach discharge calculations and data are provided in **Figure 2**. The resulting breach outflow hydrograph for UTR 10 Dam is shown on **Figure 3**.

The downstream water surface profiles for the dam breach were developed using the hydraulic model described in the *Existing Conditions Assessment Report (Appendix D-1)*. The breach dimensions were selected to generate the aforementioned peak discharge. The inundation depth produced from the simulated breach extending from the dam to USGS gauge at Hwy 2 is shown in **Figure 4 and Figure 5**, including labeled structures affected and roads overtopped throughout the breach zone. **Table 1 and Table 2** provides data on maximum inundation depth, maximum velocity of flow, and the amount of time it would take for the breach discharge to reach the structure and roads.



Table 2: Residential Structures Impacted by a Breach of UTR10 Dam

Structure ID	Depth (feet)	Velocity (ft/sec)	Arrival Time (Hours)	Danger Zone	D*V (ft ² /s)
S1	6.5	1.3	2	High	8.5
S2	4	1.9	5	Judgement	7.6
S3	1.7	5.5	6	Low	9.4
S4	8.8	1.7	6	High	15.0
S5	1.1	1.7	7	Low	1.9
S6	0.9	1.7	7	Low	1.5
S7	1.6	2	7	Low	3.2
S8	4.7	1.9	9	Judgement	8.9
S9	5.5	1.1	9	Judgement	6.1
S10	9.5	1	9	High	9.5
S11	9.5	0.8	9	High	7.6
S12	4	0.4	10	Judgement	1.6
S13	10	0.1	9	High	1.0

Table 3: Road Overtopping Data for ND Highway 32 During a Breach of UTR10 Dam

Road ID	Common Name	ADT (vehicles)	Max Depth (feet)	Average Velocity (ft/sec)	Arrival Time (Hours)	Danger Zone
R1	23rd Ave	270	8	1.6	2	High
R2	Hwy 18	775	2.5	5	3	High
R3	CR 11 (21st Ave)	40	4.2	3.1	4	High
R4	36th St	N/A	5.2	4	5	High
R5	20th Ave	N/A	4.4	4.8	5	High
R6	Hwy 2	4860	0.6	0	6	Low
R7	CR 4A (35th St)	185	3.3	6.3	6	High
R8	Local (17th Ave)	N/A	5.9	5.3	8	High
R9	32nd St	100	10.6	3.9	9	High
R10	Hwy 2	6150	3.9	4.9	10	High

The structures and roadways listed in **Table 1** and **Table 2** were analyzed further to determine if there is the potential for loss of life during a breach of the magnitude described. Depth and velocity flood danger level relationships established in *Downstream Hazard Classification Guidelines* (U.S. Bureau of Reclamation, 1988) were used to determine which structures and roads have a high danger potential during a breach at UTR 10 Dam.

The chart from *Downstream Hazard Classification Guidelines* that shows the depth-velocity flood danger level relationship for homes built on foundations is shown on **Figure 6**. The structures corresponding to **Table 1** are also plotted on **Figure 6**. Structures plotted in the red are categorized as having a high



danger level, indicating that loss of life is likely. Structures in the yellow fall into what is called the judgement zone where some level of engineering judgement should be used to determine if the structure has a high or low danger potential. Structures plotted in the green area have a low danger level, and loss of life is not likely. **Figure 5** shows that there are six structures in the high danger (red) zone. Therefore, a total of six out of the thirteen total residential structures would have a high danger potential for loss of life if UTR 10 Dam were to breach with the magnitude required in *TR-60*. The six structures that have a high danger potential are shown as red triangles in **Figure 4** and **Figure 5**. The remaining structures that are in the judgment or low danger potential category are shown as yellow and green triangles in those same figures, respectively.

The hazard potential for habitable structures was also reviewed based on guidance in the National Engineering Manual (NRCS, 2017), which indicates that products of four or greater that result from depth (in feet) and velocity (in feet per second) combinations could result in loss of life. Eight of the thirteen structures identified have depth and velocity products greater than four. Even though results vary slightly, the results and methods used to identify habitable structures within the breach zone that may experience loss of life during a breach were verified by criteria in the National Engineering Manual.

Another chart in *Downstream Hazard Classification Guidelines* (DOI USBR, 1988) shows the depth-velocity flood danger level relationship for passenger vehicles. That chart can be seen on **Figure 7**, which shows ten road overtopping locations along the breach wave (listed in **Table 2**) are plotted on **Figure 7** as well. **Figure 7** shows that nine overtopping locations fall in the high danger category, where loss of life is likely. One roadway is in low danger category, where loss of life due to flooding over the road is not likely.

4.2 Hazard Classification

Title 210, National Engineering Manual, Part 520 Subpart C “Dams” (NRCS, 2017) describes the hazard potential resulting from failure of dams. According to this guidance, a high hazard potential is “Dams where failure may cause loss of life or serious damage to homes, industrial or commercial buildings, important public utilities, main highways, or railroads.” A similar definition is outlined in Article 89-08 of the North Dakota Century Code (ND SWC, 2015) where a high hazard dam is defined as, “A dam located upstream of developed or urban areas where failure may cause serious damage to homes, industrial and commercial buildings, and major public utilities. There is potential for the loss of more than a few lives if the dam fails.”

Based on the data presented in Section 4.1, UTR 10 Dam will be classified as a high hazard dam with the proposed dimension and features.

5 Hydrology Design

Requirements for minimum hydrologic criteria associated with the proposed alternative for UTR 10 Dam were determined based on guidance provided in *TR-60*. Based on results previously presented in Section 4, UTR 10 Dam is classified as a high hazard dam with the proposed dam in place. The minimum precipitation criteria outlined in *TR-60* for high hazard dams is shown in **Table 3**, and each of the design hydrographs is described in more detail in the following sub-sections. The calibrated hydrologic and hydraulic models used to develop results in **Appendix D-1** were also used for the analysis completed for the proposed condition.



Table 4: TR-60 Minimum Precipitation Data for High Hazard Dams

Design Event Hydrograph	Hydrologic Criteria ^[1]	Depth (inches)
Principal Spillway Hydrograph (PSH)	P_{100}	4.65 ^[2] 7.87 ^[3]
Auxiliary Spillway Hydrograph (ASH)	$P_{100} + 0.26 (PMP - P_{100})$ ^[4]	8.72 ^[4]
Freeboard Hydrograph (FBH)	PMP	18.6 ^[5]

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

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

[3] Rainfall depth based on NOAA Atlas 14, 10-day duration (8.15 inches), with areal adjustment factor of 0.966

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

[5] PMP from statewide PMP study for North Dakota, 48-hr aerially adjusted

5.1 Principal Spillway Design

Based on TR-60, the principal spillway of a high hazard dam must pass, at a minimum, the 100-year return period storm without activating the auxiliary spillway and drawdown 85% of the reservoir in 10-days. However, the structural auxiliary spillway acts as dual-purpose principal spillway, and these requirements are not required. However, the remaining volume must be set as initial storage for ASH and FBH runs. Guidance dictates this storm shall have a duration of not less than 10-days. Two methods to determine runoff volume for the design of the principal spillway are presented in *National Engineering Handbook, Part 630 Hydrology, Chapter 21 Design Hydrographs* (USDA NRCS, 2019b), hereafter Chapter 21, including runoff volume maps and runoff curve number procedure. After consideration of various principal spillway sizes, a 6' x 6' box culvert at natural stream grade is selected as it meets TR-60 design criteria and maximizes downstream benefits.

5.1.1 Runoff Volume Maps

Runoff volume maps presented in *Chapter 21* were generated for areas where runoff from snowmelt can potentially produce greater runoff volumes than rainfall events. The 100-year 10-day runoff volume for Turtle River Basin is 4.94 inches according to Figure 21-2. No areal reduction is applied to this runoff volume. Curve Numbers in the HMS hydrologic model are set to 99, initial abstraction set to zero, and impervious areas set to 100%, causing entire 4.65 inches to runoff as excess volume. Guidance from *NEH Part 630 Chapter 21* was used to develop the principal spillway mass curve, or runoff distribution curve, for the 10-day event. 15-minute time increments were used to develop the distribution. Using equation 21-2 from *Chapter 21*, the total runoff at any given time during the event can be calculated. These 15-minute values can then be arranged in either a decreasing order (Curve A), an increasing order (Curve B), or a critical stacking order (Curve C). The principal spillway mass curves are shown on **Figure 9**. The runoff volume of 4.65 inches was simulated through the hydrologic and hydraulic models using the principal spillway mass curves A, B, and C.

Quick Return Flow (QRF) is the rate of discharge that persists beyond the flood period of the principal spillway hydrograph. Based on guidance from *Chapter 21 (Figure 21-4)*, the QRF for UTR 10 Dam is approximately 1.6 cubic feet per second, per square mile. The drainage area to the dam is 158.7 square miles, which results in a QRF of 253.9 cubic feet per second (cfs). There is no adjusting for channel



losses because the climatic index is 1.38. The critical principal spillway mass curve was determined to be Curve B. The calculated QRF was added to the trailing limb of the critical principal spillway discharge hydrograph in RAS and is shown on **Figure 9**. All three mass curve hydrographs and resulting reservoir elevations are plotted in **Figure 10**, and summaries included in **Table 4**.

5.1.2 Runoff Curve Number Procedure

The runoff curve number method presented in *Chapter 21* uses climatic data and watershed characteristics to convert rainfall data into runoff volumes. The 100-year, 10-day rainfall event was simulated using depths published in *NOAA Atlas 14* (NOAA, 2017). For Turtle River watershed the 100-year, 10-day rainfall depth is 8.15 inches; the area adjustment factor from Table 21-3 for watershed greater than 100 square miles is 0.966 resulting in 7.87 inches applied to each subbasin of the HMS hydrology model. The simulation used a nested distribution with HMS Frequency Storm meteorologic model, which applied each smaller duration in the next larger duration; this is more intense and conservative than using Chapter 21, Figure 21-3. For the intermediate values of nested values, the 1-day adjustment factor (0.932) was used for 1-day and less durations, and 10-day adjustment factor (0.966) was used for 2-day through 10-day precipitation values. The Frequency Storm 75% intensity position was adopted as it produced the most critical results compared to other positions available (25, 33, 50, 66%); this intensity position still nests all smaller durations precipitation depths, but positions them at 75% of the full (10-day) duration.

The 10-day Curve Number with an average moisture condition (AMC II) was used; 10-day Curve Number was calculated using Chapter 21, Table 21-2. Similar to runoff volume calculations, there is no adjustment for channel losses. The hydrograph was evaluated in RAS model with proposed dam features. The stage and discharge hydrograph from this event are shown on **Figure 11**, and summary included in **Table 4**. The Figure also includes the critical event, mass Curve B, from the runoff volume maps procedure discussed in Section 5.1.1 for comparison; the resulting peak reservoir level for runoff curve number is less critical than runoff volume mass curve B.

5.1.3 Principal Spillway Adequacy

Guidance from TR-60 states that since the auxiliary spillway is structural, it serves dual purpose as principal spillway. Therefore, the peak reservoir elevation does not need to be lower than auxiliary spillway invert (1127). However, PSH's were analyzed to verify 10-day drawdown for initial reservoir level for ASH and FBH runs. The PSH results are summarized in **Error! Reference source not found..** Runoff Volume Curve A is the critical event for drawdown; the storage remaining after 10-day drawdown is 63% of the peak volume, which correlates to elevation 1124.5. Sensitivity was completed in increasing the 6'x6' principal spillway culvert, however has very low decrease in auxiliary spillway length. The ASH and FBH runs are required to start at elevation 1124.5, which correlates to 5,100 acre-feet of volume.

Table 5: Principal Spillway Hydrograph results

PSH	Peak			10-day		
	Inflow (cfs)	Reservoir Elevation	Volume (ac-ft)	Reservoir Elevation	Volume (ac-ft)	Storage Remaining
Volume A	3,408	1128.27	8,150	1124.5*	5,100	63%
Volume B	4,894	1128.98*	8,800	1117.6	1,500	17%



Volume C	4,694	1128.8	8,700	1120.4	2,700	31%
100yr						
10Day	4,106	1128.17	8,050	1115.3	800	10%

*Critical

The 6' x 6' box culvert principal spillway meets the 10-day drawdown requirement. However, the auxiliary spillway is activated during critical PSH. Since the auxiliary spillway is structural, it is dual purpose principal spillway, and all PSH TR-60 requirements are met.

5.2 Auxiliary Spillway Design

The critical considerations for auxiliary spillway design are Auxiliary Spillway Hydrograph (ASH), Freeboard Hydrograph (FBH), and energy dissipation criteria. A vegetated spillway was determined infeasible due to high erodibility index of sandy loam soils along abutments; over 800 feet of length would be necessary to meet ASH and FBH stability and integrity requirements. This extensive length through irrigated cropland was not economical, nor meet sponsor and landowner conditions. After consideration of various auxiliary spillway lengths and types, a 360-foot long ogee crest with chute baffles is selected as it meets TR-60 design criteria and minimizes construction costs. The alternative to the 360-foot long ogee crest with baffles was a RCC (Roller Compacted Concrete) stepped spillway. Although the two alternatives would be hydraulically comparable, the width of the RCC spillway would be about 1.5 times the length of the ogee crest with baffles spillway and the stilling basin was substantially longer resulting in a large increase in concrete quantities when compared to the ogee crest with baffles.

5.2.1 Auxiliary Spillway Hydrograph

The capacity of the auxiliary spillway should be adequate to pass the auxiliary spillway hydrograph in combination with either wave height, or frost conditions, without overtopping the dam embankment based on requirements in *TR-60*. Stability design analysis is not applicable due to auxiliary structural spillway. The ASH starting reservoir level is 1124.5, from PSH critical results. The adopted ASH duration is 24-hours. The Time of Concentration (T_c) for UTR Dam 10 is ~48 hours, therefore this is used for FBH. The ASH is short-duration check, for smaller basins typically 6-hours, or 25% of FBH 24-hour duration. However, since this is a larger basin with $T_c > 24$ -hours, the ASH duration is conservatively chosen as 50% of FBH duration. The ASH total rainfall is 8.72 inches, which is calculated from arially adjusted 24-hour PMP (17.9 inches) and NOAA Atlas 14 P_{100} (5.5 inches) using equation in Table 3. Incremental durations similarly use NOAA Atlas 14 values and nested using Frequency Storm Meteorological model. The critical 6-hour rainfall ratio is ~72%, which is more than Figure 21-9 (~57%) and 5-point ratio (~64%). The rainfall hyetograph and distribution are summarized in **Figure 13**.

The aforementioned precipitation is applied to 24-hour Curve Number basin from Existing Conditions Report. The HMS model resulting inflow hydrograph peak flow is 9,203 cfs. The hydrograph was evaluated in RAS model with proposed dam features resulting in reservoir stage is 1130.16; the full hydrographs are plotted in **Figure 13**. Therefore, the standard freeboard is 5.84 feet. Wave height calculated for this reservoir level are summarized in Section 5.2.3, and adequacy in section 5.2.4.



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

5.2.2 Freeboard Hydrograph

The capacity of the auxiliary spillway should be adequate to pass the freeboard hydrograph without overtopping the dam embankment based on requirements in *TR-60*. However, the North Dakota Dam Safety Standards (ND DWR, 2024) requires minimum residual freeboard of 1 foot or wind-generated setup and runup from a 10% exceedance wind. The state standard is more critical than NRCS federal standard.

The FBH starting reservoir level is 1124.5, from PSH critical results. The adopted FBH duration is 48-hours based on Time of Concentration (T_c) for UTR Dam 10 is ~48 hours. The FBH total rainfall is 18.6 inches for the entire watershed, which is reduced from peak value of 19.99 inches calculated using the ND PMP tool (Applied Weather Associates, 2021). Frequency Storm Meteorological model incremental durations use nested values from lower duration tool results. The critical 6-hour rainfall ratio is ~68%, which is more than Figure 21-9 (~57%) and 5-point ratio (~64%). The rainfall hyetograph and distribution are summarized in **Figure 14**.

The aforementioned precipitation is conservatively applied to 24-hour Curve Number basin from Existing Conditions Report. Interpolating a 48-hour CN was determined to be not reasonable. The HMS model resulting inflow hydrograph peak flow is 32,971 cfs. The hydrograph was evaluated in RAS model with proposed dam features resulting in reservoir stage is 1134.83; the full hydrographs are plotted in **Figure 15**. The maximum flow through the dam 30,600 cfs, which 850 cfs is through principal spillway and 29,750 cfs is over auxiliary spillway, both outlet in the stilling basin. Therefore, the residual freeboard is 1.17 feet. Wave height calculated for this reservoir level are summarized in Section 5.2.3, and adequacy in section 5.2.4.

5.2.3 Wind Induced Wave Action

The ND DWR residual freeboard is based on 10% hourly exceedance probability wind, or a typical wind that would occur during an average 10-hour period (ND DWR, 2024). The 10% exceedance wind speed is 21 mph (NOAA, 2024). The effective fetch length at reservoir level ~1130 is 5,041 feet using 14 radial lines at 6 degree intervals (see **Figure 16**). Based on equation 3 (USDA, NRCS, 2014), the significant wave height is 0.66 feet.

The ND DWR standard freeboard is based on 100 mph wind-generated setup and runup (ND DWR, 2024). The NRCS federal requirement is based on 50-year recurrence, which based on Figure 4 for the UTR Dam 10 site is ~80 mph (USDA, NRCS, 2014). Therefore, the ND state standard is critical. The effective fetch length at reservoir level ~1135 is 2,485 feet using 15 radial lines at 6 degree intervals (see **Figure 17**). Based on equation 3 (USDA, NRCS, 2014), the significant wave height is 1.84 feet.

5.2.4 Auxiliary Spillway Adequacy

The ASH criteria in *TR-60* and state standards are met as standard freeboard is 5.84 feet, which meets the minimum requirement of 3 feet and 1.84 feet for wind-generated setup and runup. The FBH criteria are met as residual freeboard is 1.17 feet, which meets the minimum requirement of 0.0 feet for federal NRCS (*TR-60*) and 1 foot minimum and 0.66 feet for wind-generated setup and runup (ND DWR, 2024).



The 360-foot ogee crest structural auxiliary spillway meets TR-60 and North Dakota Dam Safety Standards.

5.3 Energy Dissipation

The rating curve downstream of UTR Dam 10 is developed from the hydraulic model described in the *Existing Conditions Assessment Report* (**Appendix D-1**) and plotted in **see Figure 18**. For consideration of structural concrete chute or Roller-Compacted Concrete (RCC), the stilling basin dimensions are set so the sequent depth, at the end of hydraulic jump or d_2 , matches downstream rating curve. If these don't match downed jump or sweepout, as described in *Design of Small Dams* (DOI USBR, 1987), will occur resulting in jump beyond the stilling basin that is likely to cause high velocity and erosion in the downstream channel. In order to match the stilling basin hydraulic jump sequent depth to rating curve the invert needs to be ~1108.5; for structural concrete chute the length needed is 1,000 feet, and for RCC the length needed is 600 feet. Both lengths are well beyond the required length (360-feet) for spillway adequacy, which substantially increases cost of the stilling basin as well as entire auxiliary spillway. Therefore, chute baffle blocks are an economical feature to dissipate energy of supercritical flow over the auxiliary spillway, and included below ogee crest along 360-foot long auxiliary spillway chute.

Auxiliary spillway design unit discharge (q) is 82.6 cfs/ft, which is higher than ideal conditions (69 cfs/ft) and extents of design curves. However, baffle spillways do operate effectively up to 150 cfs/ft and design equations and curves are allowed to be extrapolated per *Design of Small Dams* (DOI USBR, 1987). Baffle design equations from *Design of Small Dams* are used to set dimensions; these assumptions and calculations are summarized in **Figure 19**.

Since the baffles dissipate flow energy along the entire chute, limiting supercritical flow from forming, a hydraulic jump is not expected or very small. Baffled chute spillway studies (Nugroho, 2019) showed that adequately designed baffled chutes dissipate ~80% of energy, and flow velocities at toe are reduced significantly and similar to downstream conditions. Therefore, a stilling basin is not required and there are no design considerations due to lack of supercritical flow. However, the chute still needs to terminate and tie into floodplain elevation (1107 – 1110) with row of baffles below outlet channel grade and principal spillway tie into bankfull channel dimensions. Therefore, a flat 12-foot long stilling basin at elevation 1104 is included with end sill to direct lower flows back to the main channel. Riprap extends downstream from the stilling basin to tie into floodplain elevations. The natural channel invert of 1101 is maintained at the end of the stilling basin.

6 Geotechnical Design

6.1 Geotechnical Site Summary

6.1.1. Soil Stratigraphy

The soil stratigraphy at the proposed dam site can be summarized as lacustrine sediments (silt) from glacial Lake Agassiz, overlaid by delta sediments (silty sand) from glacial runoff. The delta sediment is delineated and confined by the Pembina escarpment on the west and the Edinburg moraine on the east.



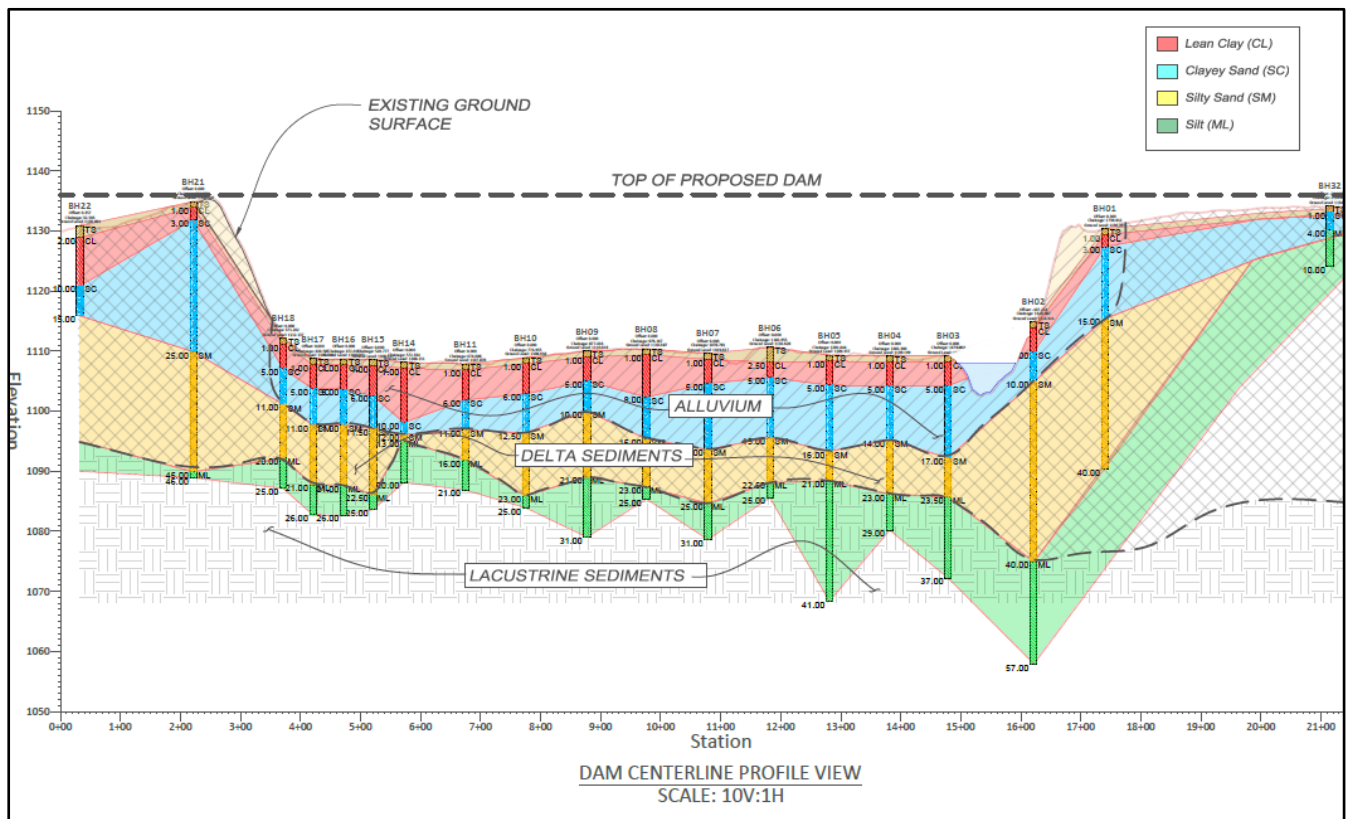
The delta sediments, in turn, are overlayed by an alluvium layer (lean clay and clayey sand). The alluvium layer is characterized mostly as clayey sand overlayed by a top layer of lean clay, which appears to be acting as a confining layer. The alluvium appears to be isolated to the flood plain of the river and the delta sediment layer mantles atop the abutments of the proposed dam as well as the proposed auxiliary spillway location.

A geological investigation of the proposed dam site was conducted starting in the winter of 1981 through the spring of 1982. A total of 39 soil borings were drilled, and subsurface evaluation of the soil borings was conducted during the investigation. Soil boring logs, findings and results of laboratory testing were summarized in soil mechanics and geological investigation reports. 22 borings were drilled along and near the proposed centerline of the dam location, 12 borings throughout the proposed location of the auxiliary spillway and 5 additional borings were utilized for assessment of possible borrow sources.

Along the centerline of the proposed dam, the boring logs indicate a confining layer of lean clay averaging about 5 to 8 feet thick, overlaying a layer comprised of silty sand, lean clay, and clayey sand material. This combined lean clay confining layer and mix of silty sand, lean clay and clayey sand is characterized as the alluvium material which appears to be isolated to the flood plain of the river. Below this alluvium layer the soil boring logs indicate a generally silty sand layer of soil, which is characterized as the delta sediments layer. The geological investigation notes indicate that the transition from the alluvium material to the delta sediment material was difficult to determine in the field, since the material makeup was similar between the two materials. Below the delta sediments layer the soil boring logs indicate a silt layer, which is characterized as lacustrine sediments. The geological investigation notes indicate that the transition from the delta sediments to the lacustrine sediments was easily determined, based upon an apparent and abrupt change in stratigraphy from a generally silty sand material to a silt material.



Figure 1: Soil Profile Along Proposed Dam Embankment Centerline



The existing river channel meanders throughout the flood plain but at the location of the proposed dam location the main river channel hugs the right proposed dam abutment. It continues along the right side of the floodplain for a way downstream of the proposed dam location. The main river channel has embedded itself into the alluvium of the wide flood plain and is accompanied by several shallow abandoned channels that are also spread throughout. The soil profile along the proposed dam centerline resulting from the soil boring investigation, indicates that a buried eroded channel may be present in the underlying lacustrine silts. This may be characteristic of an old channel which has been buried by subsequent layering deposits of the delta sediments and alluvium.

6.1.2. Seepage Control Measures

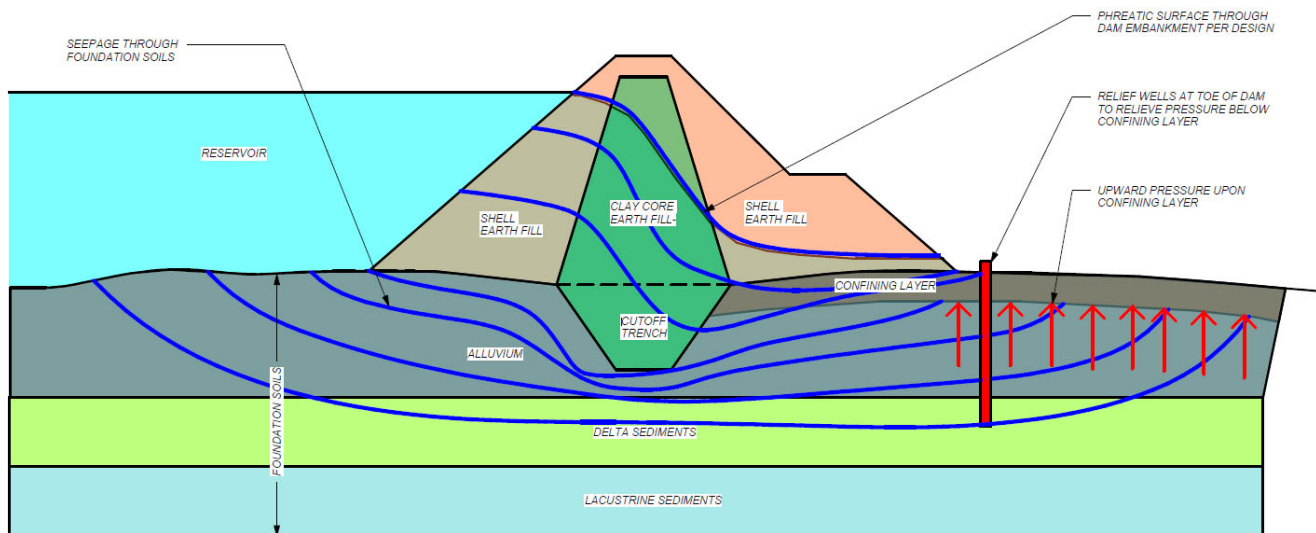
The presence of high ground water within the flood plain of the Upper Turtle River, creates challenges for design and construction of an earth embankment dam structure. At typical means of combating seepage through and beneath a dam embankment structure is to incorporate a cutoff trench to create a less permeable zone within the dam foundation which would be resistant to high permeable flow of seepage beneath the dam. This also pushes seepage paths deeper into the foundation materials, creating a longer path for the seepage and reducing seepage gradients horizontally through the foundation soil as well as reducing vertical seepage gradients exiting the ground surface downstream of the dam. Generally, for a cutoff trench to be effective, the trench must completely cutoff the foundation soil zones that exhibit high permeability and stop at either a low permeable soil layer or known bedrock. The cutoff trench backfill is typically a low-permeable clay material, which effectively resists seepage flows through the foundation soils. In some situations, a complete cutoff trench is not



economical if high permeable foundation zones are deep. In that case, a partial cutoff trench may be constructed, but even the smallest reduction in cutoff percentage of the low permeable soil zones results in a significantly less outcome of seepage reduction. For example, an 80 percent partial cutoff trench, would result in only about a 55 percent reduction in seepage, compared to a 100 percent seepage reduction for a complete cutoff trench. In conclusion, about half the seepage reduction potential is lost with an 80 percent partial cutoff trench.

Construction of a cutoff trench at UTR10 would involve significant challenges given the extent of high permeable soils (alluvium and delta sediments). These soils extend about 20 to 30 feet below the ground surface, meaning that a complete cutoff trench would have to extend to that depth in order to be effective. A partial cutoff trench could be constructed, but the challenges of dewatering the excavations would still remain with very little benefit to seepage reduction.

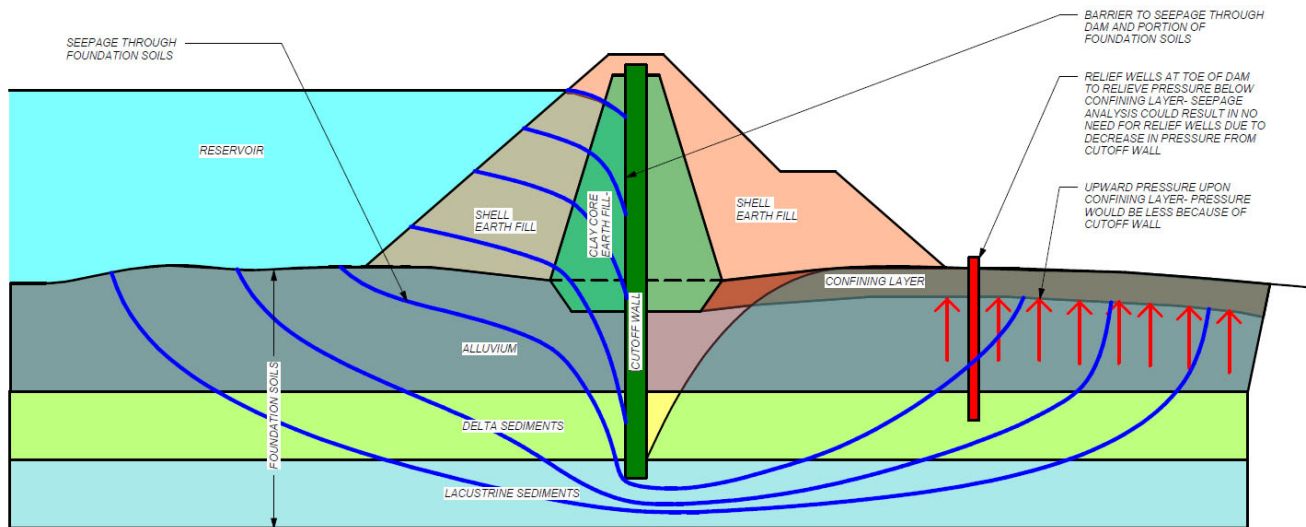
Figure 2: Typical Dam Embankment with Cutoff Trench



An alternative to a cutoff trench is a cutoff wall. A cutoff wall is constructed from various materials and are installed using a variety of methods, but the general goal of a cutoff wall is to create a seepage barrier element within the dam, that would not just slow seepage but would create a barrier to seepage. Cutoff walls can be designed to cut off just the foundation soils or designed to create a cutoff barrier through the entire height of the dam and foundation. Cutoff walls are commonly used where partial cutoff trenches are installed, where ground water or other factors create challenges in constructing a complete cutoff trench. The cutoff wall extends deeper below the cutoff trench and continues the seepage resistance deeper into the foundation.

The great benefit of a cutoff wall is not only that it creates a barrier to seepage, but also a cutoff wall can be installed within foundations where saturated soils or ground water makes the construction of a cutoff trench impossible. The downside of a cutoff wall is that the construction is costly, but the reduction to seepage is a large benefit. In this specific situation with the proposed dam location, the construction of a cutoff wall can be conducted without the need for extensive dewatering measures.

Figure 3: Typical Embankment with Cutoff Wall



Delta sediments are present upon the proposed abutments each side of the proposed dam location. These delta sediments may be highly permeable and if so, will relay seepage easily downstream from the reservoir. Robust abutment groin drains and other drainage systems may be used to combat seepage through the groin, but with the initial shallow ground water, an increase in head pressure from the reservoir will promote increased conveyance through these soils. In order to better combat seepage through the groin and interface of the embankment fill with the native abutment material, the cutoff wall will be extended past the abutment each end of the dam to maintain the seepage barrier past the abutments.

6.1.3. Downstream Hydrostatic Pressure at Toe of Dam

With the presence of shallow ground water, any increase in head pressure of the ground water, such as from a dam reservoir, would raise the local ground water level downstream of the dam and create vertical pressure gradients which would promote the ground water to exit onto the downstream flood plain through the alluvium layer. The alluvium is capped by a clay top which ranges from 3 to 9 feet thick. This clay cap would act as a non-permeable layer, maintaining a level of resistance to the increasing ground water's vertical pressures. It may already be acting in that way, maintaining the ground water level slightly below the surface. The clay cap can be referred to as a confining layer, which resists the vertical water pressures below it, by means of its soil weight and by material mechanical means such as soil shear and cohesive strength. At some point though, the resistance of the confining layer may be overcome if the vertical pressure is large enough. Failures of the confining layer can result in many types of failures, such as sand boils, confinement layer heave, slides, springs, etc. The presence of a cutoff wall can decrease vertical pressure gradients drastically, but with the ground water level being as shallow as it is, any small increase would need to be assessed with care in design.

7 Structural Design

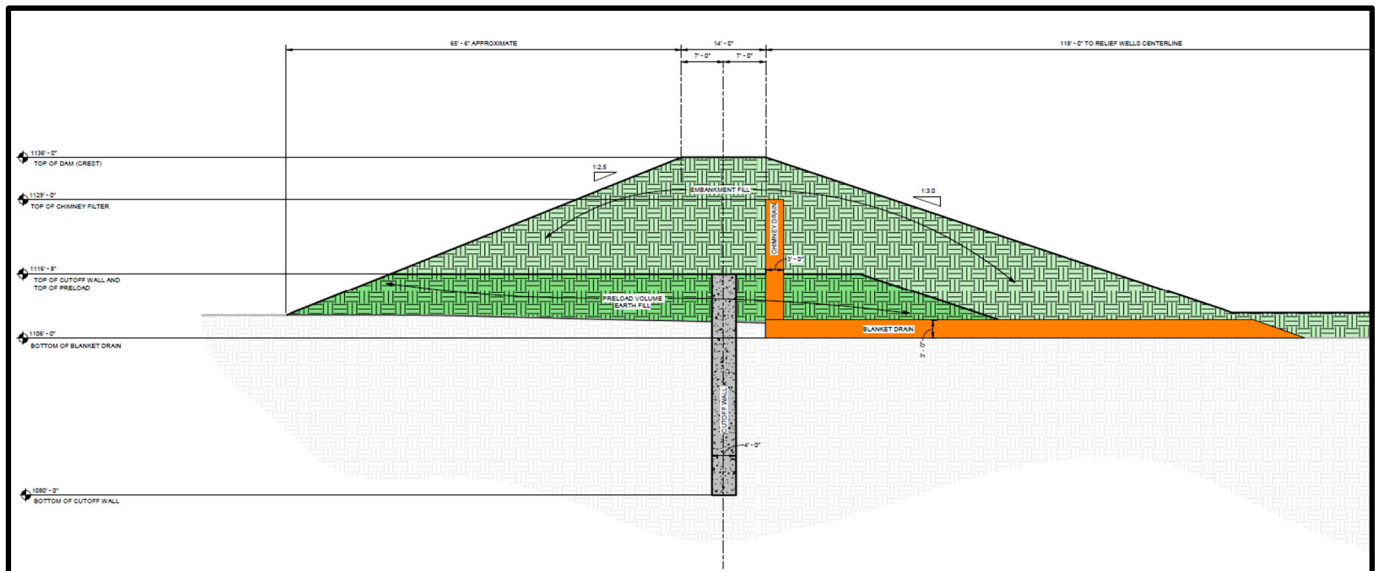
7.1 Embankment Design



7.1.1. Embankment Geometry

The main embankment structure is a homogenous embankment with a maximum height of about 28'-0" (from existing floodplain to top of proposed dam), having a top of dam elevation of 1136.00 and 14'-0" dam crest width. The side slopes of the dam embankment are 2.5:1 slope on the upstream dam face and a 3:1 on the downstream face. There is no dam core zone nor is there a cutoff trench implemented. Instead a cutoff wall is to be constructed to impede seepage through the foundation soil. The cutoff wall has a bottom elevation of 1080.00 and a top elevation of 1116.67, resulting in a total height of 36'-8". The goal of the cutoff wall is to penetrate through the entirety of the high permeability soil and to reach the less permeable lacustrine sediments.

Figure 4: Embankment Geometry



7.1.2. Embankment and Foundation Soil Materials

The embankment profile and underlying foundation material is comprised of (6)- main soil material types. The soil material types are the Embankment Fill and Drain Material contained within the constructed embankment as well as foundation soil material beneath the embankment which is comprised of the Clay Cap at the surface, layers of Clayey Sand and Silty Sand beneath the cap with a deep layer of Silt. See the location of these materials in the below figure. The material properties used for the engineering design/assessment of the embankment slope stability and seepage potential is presented as well in the below figure.

The embankment fill is representative of a typical lean clay – silt material (CL-ML), which is representative of tested soil of proposed borrow areas, indicated in the soil mechanics report from 1982. The unit weight, cohesion and friction angle values are taken directly from the recommendations for embankment material soil properties made in the 1982 soil mechanics report. The only property modified for the design/assessment is the coefficient of permeability. A previous recommended value of 8.2×10^{-8} was modified to 1.8×10^{-7} to present a slightly more conservative seepage rate to the analysis. The new value is still within the expected range of permeability for the type of soil.



The material properties used for design/assessment of the underlying foundation soil material was taken directly from the 1982 soil mechanics report. The CL, SC and SM materials were recommended for a $k = 5.8 \times 10^{-5}$ ft/sec based upon a 5 ft/day value used in the 1982 report. The ML material was recommended for a $k = 1.2 \times 10^{-6}$ ft/sec based upon a 0.1 ft/day value used in the 1982 report. The only modification to these original values is the CL material acting as the clay cap. The higher permeability of the clay cap was determined to be less conservative when looking at the possibility of heave due to a confinement layer at the surface. The clay cap was given a more restrictive value of $k = 8.2 \times 10^{-6}$ ft/sec.

Soil mechanical properties for the foundation material is given the same unit weight, cohesion and friction angle based on the triaxial shear tests conducted on the underlying foundation materials. The most conservative was used for use in this design/assessment and was given to every foundation material. The resulting foundation material acts like a homogeneous foundation regarding slope stability and like a multiple layered foundation when regarding seepage potential.

Figure 5: Embankment Soil Material Properties

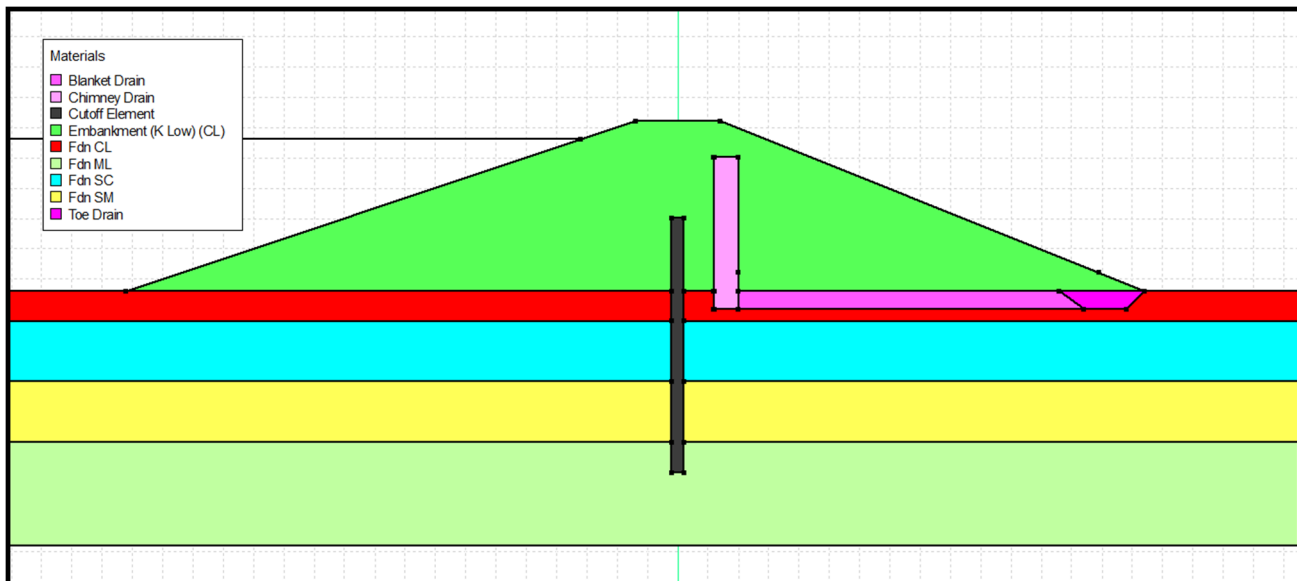


Table 6: Embankment Material Properties

Embankment Material Properties						
Material	Location	Thickness	γ (unit weight)	C (cohesion)	Φ (friction angle)	K (permeability coefficient)
Embankment Fill	Embankment	28'-0" ~	90 pcf	350 psf	25.5°	1.8×10^{-7} ft/sec
Drain Fill	Embankment	4'-0" max.	115 pcf	0 psf	30°	0.00082 ft/sec
Foundation CL (Clay Cap)	Foundation	5'-0"	77.4 pcf	200 psf	25°	8.2×10^{-6} ft/sec
Foundation SC	Foundation	10'-0"	77.4 pcf	200 psf	25°	5.8×10^{-5} ft/sec
Foundation SM	Foundation	10'-0"	77.4 pcf	200 psf	25°	5.8×10^{-5} ft/sec
Foundation ML	Foundation	∞	77.4 pcf	200 psf	25°	1.2×10^{-6} ft/sec



7.1.3. Embankment Slope Stability

Slope stability engineering design and assessment of the proposed embankment structure atop the underlying foundation material, is conducted in accordance with NRCS guidance and recommendations detailed in Technical Release (TR 210-60) for Earth Dams and Reservoirs. Four design conditions are required by TR-60 to be assessed for a given dam embankment structure. For each condition TR-60 requires a minimum factor of safety against slope stability failure to be met. Slope stability was assessed using GeoStudio Software.

Condition #1 is a check of the Construction Stability of the dam embankment, which is the ability of the embankment material to remain stable when subjected to residual moisture within the compacted material. This is mainly to ensure that during the construction process, when material is wetted beyond optimum it is able to remain stable.

Condition #2 is a check of the Rapid Drawdown Stability of the dam embankment, which is the ability of the embankment material to remain stable when the reservoir is drawdown in a quick manner such that the embankment material is still saturated.

Condition #3 is a check of the Steady-State Seepage Stability of the dam embankment at normal reservoir pool elevation.

Condition #4 is a check of the Floor Surge Stability of the dam embankment at freeboard hydrograph level of the reservoir.

For this engineering design/assessment of the slope stability three of the four conditions have been compared to the results. Since the dam will act as a dry dam, condition #3 is not applicable. Along with these three conditions a fourth analysis was conducted, which implemented a dynamic analysis of the rise and fall of the reservoir pool during a flood surge event. The most critical slope stability failure surface was determined for each time increment of the transient analysis.

Table 7: Slope Stability Results

Slope Stability Results			
Design Condition	Definition	Required Factor of Safety	Design Factor of Safety
Steady State Condition #1	Embankment Construction Stability	F.S. $\geq$ 1.4	F.S. = 2.11
Steady State Condition #2	Embankment Rapid Drawdown Stability	F.S. $\geq$ 1.2	F.S. = 2.10
Steady State Condition #4	Embankment Floor Surge Stability	F.S. $\geq$ 1.4	F.S. = 1.94
Dynamic Condition	Transient Reservoir Rise and Fall Stability	F.S. $\geq$ 1.4	F.S. = 2.07



Figure 6: Steady-State Stability - Condition #1

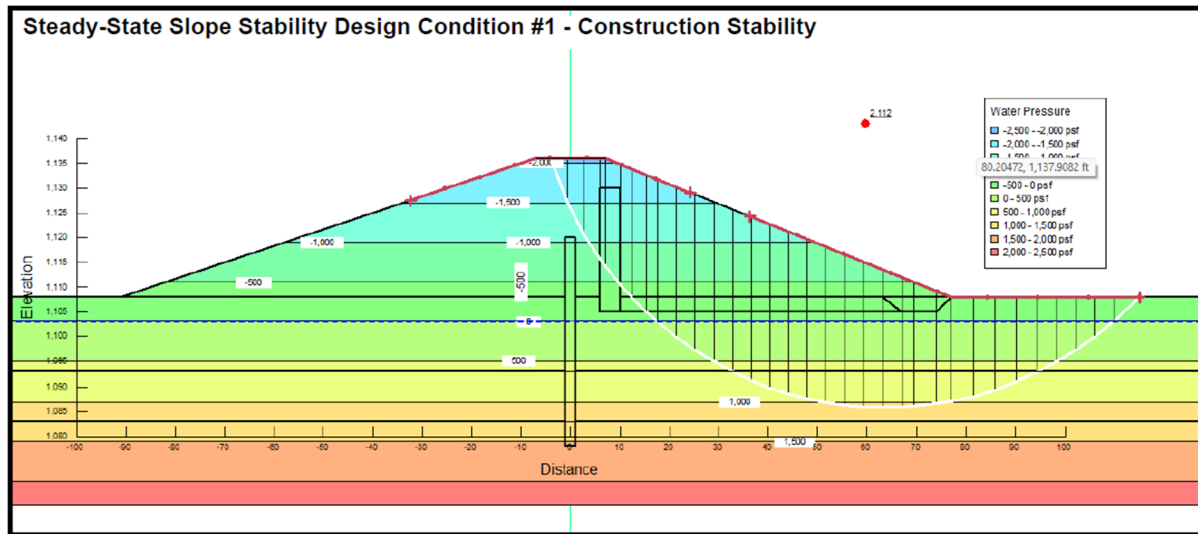


Figure 7: Steady-State Stability - Condition #2

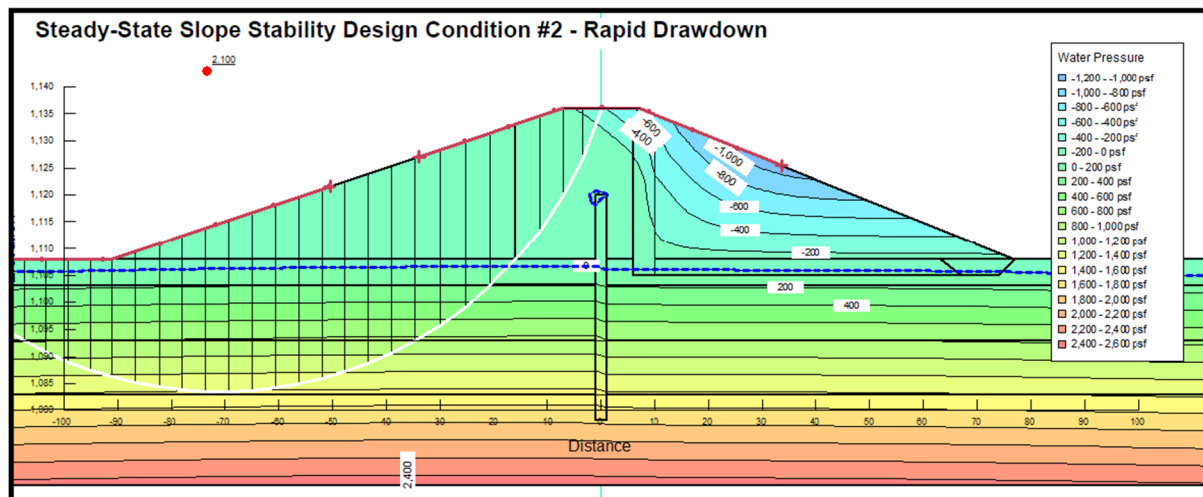




Figure 8: Steady-State Stability - Condition #4

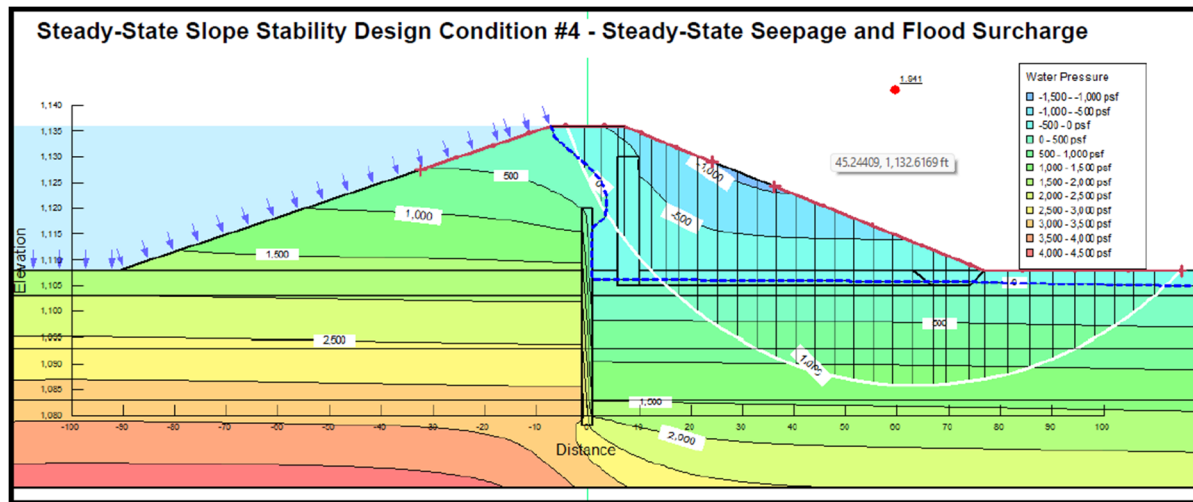
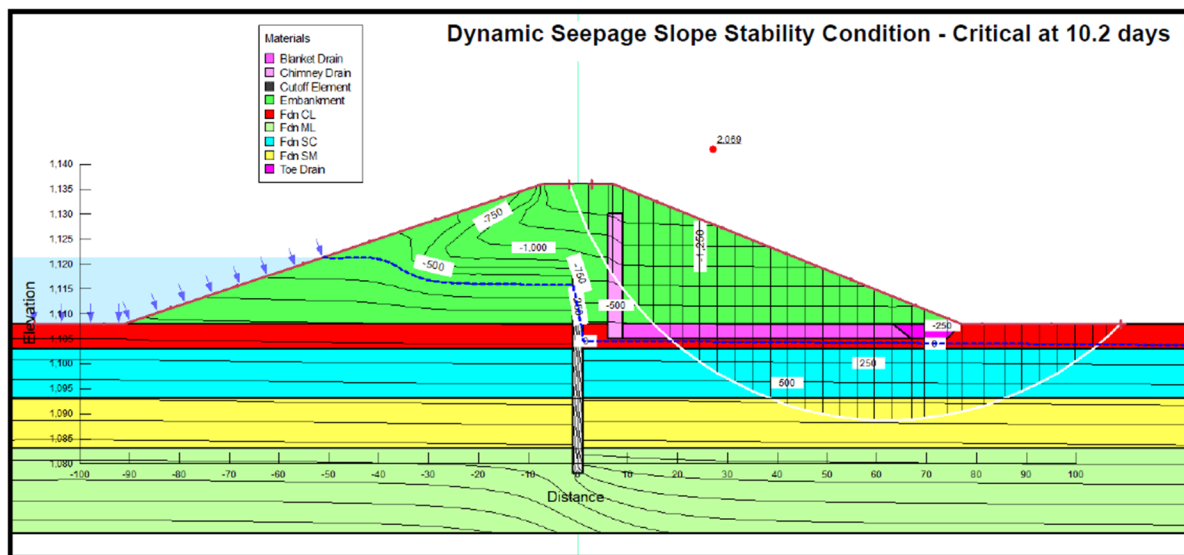


Figure 9: Dynamic Seepage Stability Condition



7.1.4. Embankment and Foundation Seepage

Seepage analysis of the proposed embankment and underlying foundation soil material was conducted for three alternatives. The three alternatives involved were: the inclusion of a cutoff wall just within the foundation soils, a cutoff wall within both the foundation soils and embankment and lastly an exclusion of a cutoff wall. The cutoff wall proved to be crucial in controlling the increased seepage gradients below the proposed embankment through the highly permeable foundation soils when the reservoir pool rises during a flood event. It was determined that a cutoff wall extending down to the lower ML soil layer and extending up into the embankment would be the most prudent alternative for this seepage control application.

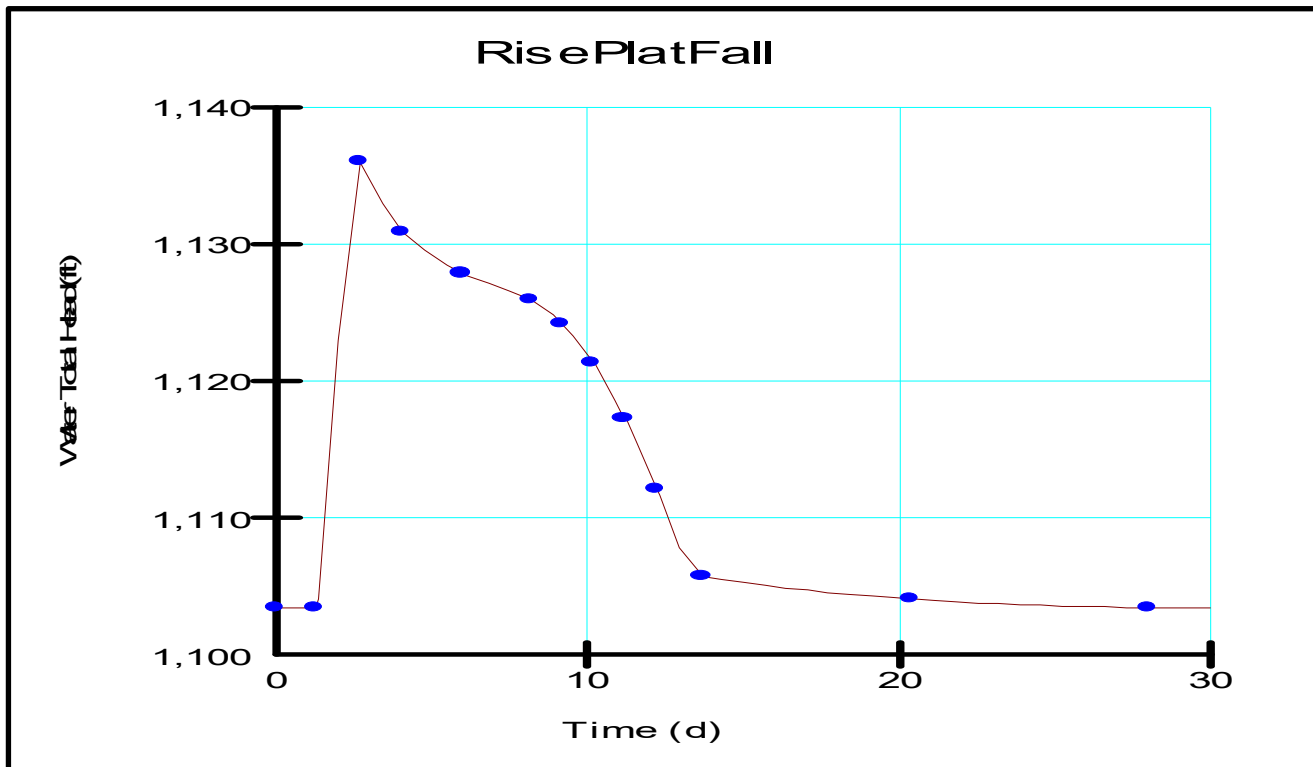


Seepage was assessed using GeoStudio Software and was conducted for both a steady-state condition and a dynamic (transient) condition. A steady-state condition is the condition when the reservoir pool elevation is sustained long enough to develop a steady-state seepage condition through the embankment. For clay embankments this is highly conservative since although a significant rise in pool elevation will begin to push seepage through the dam a flood typically does not last long enough to create a true steady-state seepage condition. This conservative approach is required since critical spillway elements responsible for lowering the pool may become obstructed which would prolong the fall of the flood pool. A dynamic condition was also assessed since the rise and fall of the proposed pool is expected to happen rather quick and a steady-state seepage condition would most likely not occur. For this reason the dynamic seepage condition was used in the analysis.

The dynamic seepage model analyzes seepage rates at several different time steps, which re-creates the change in seepage rate as the reservoir rises and falls during a flood event. The time steps are separated into 12 steps from 0 days to 28 days, which represents the expected time frame that the reservoir will rise and fall based upon the hydraulic analysis conducted for this design. See below for the water surface elevation at each time step used in the analysis.

Days	0	1.25	2.7	4.1	6.0	8.2	9.2	10.2	11.2	12.2	13.7	20.4	28
Elevation	1103.5	1103.5	1136.0	1130.8	1127.8	1125.9	1124.1	1121.3	1117.3	1112.1	1105.8	1104.0	1103.5

Figure 10: Water Surface Elevation vs. Time for Dynamic Seepage Analysis



The results of the seepage analysis conclude seepage quantities for four elements of the embankment structure. The four elements are: seepage through the embankment downstream face, seepage upward into the floor plain from hydrostatic pressure buildup, seepage through foundation filter material and seepage through foundation soils. The analysis utilizes a 2-D analysis which assesses a 1'-0" section of the embankment. The values are then multiplied by 1,400 feet, which is the width of the flood plain at



the proposed embankment site. The final values are presented below and are representative of total estimated seepage values through each particular element for the entire floor plain. The presence of the cutoff wall is also compared with the results when no cutoff wall is present. The results are presented in ft^3 per 28 days which is the time period of the modeled flood event.

Figure 11: Seepage through dam Face Illustration

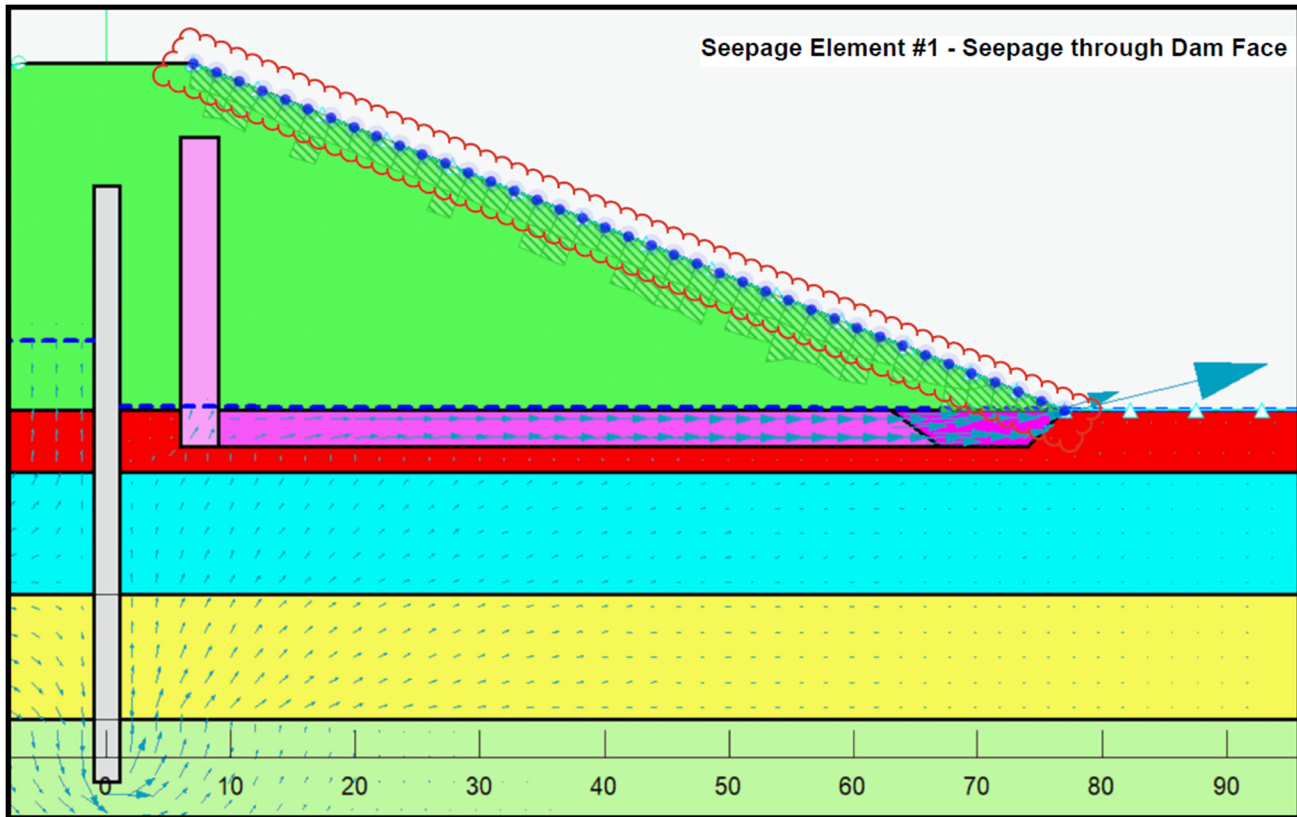




Figure 12: Seepage up into Flood Plain Illustration

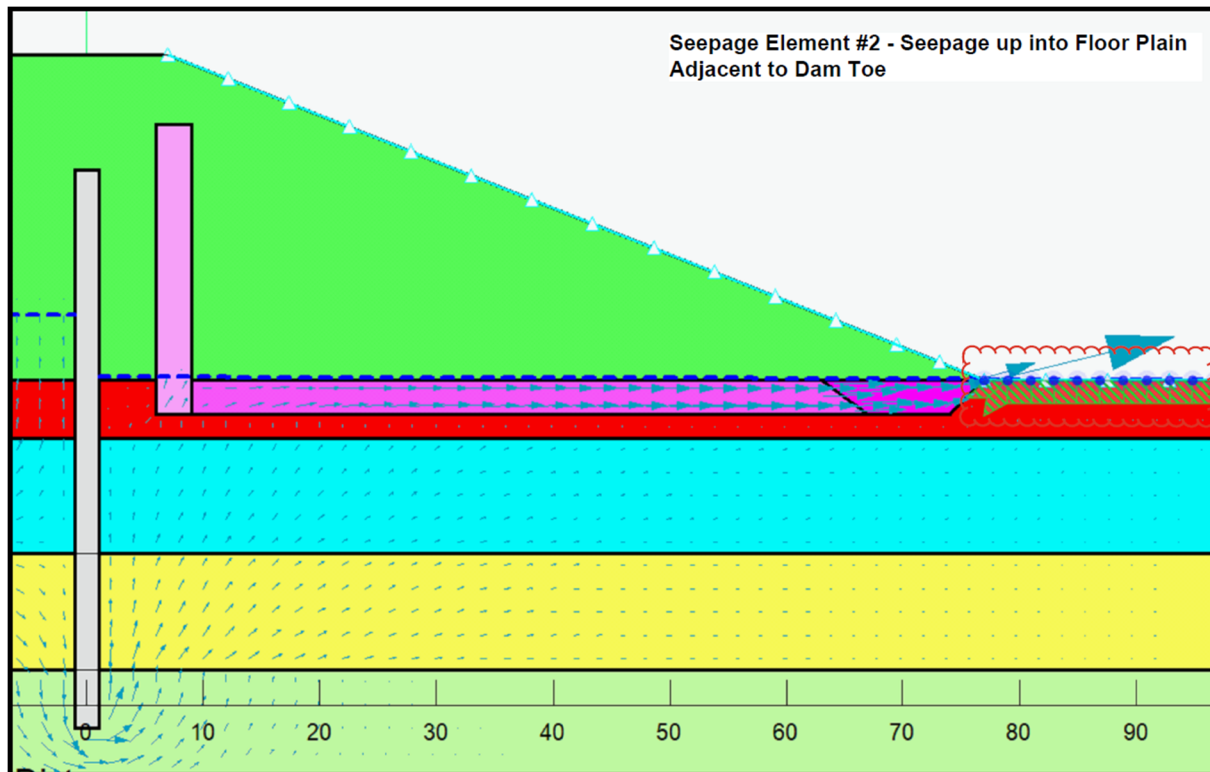


Figure 13: Seepage through Filter Illustration

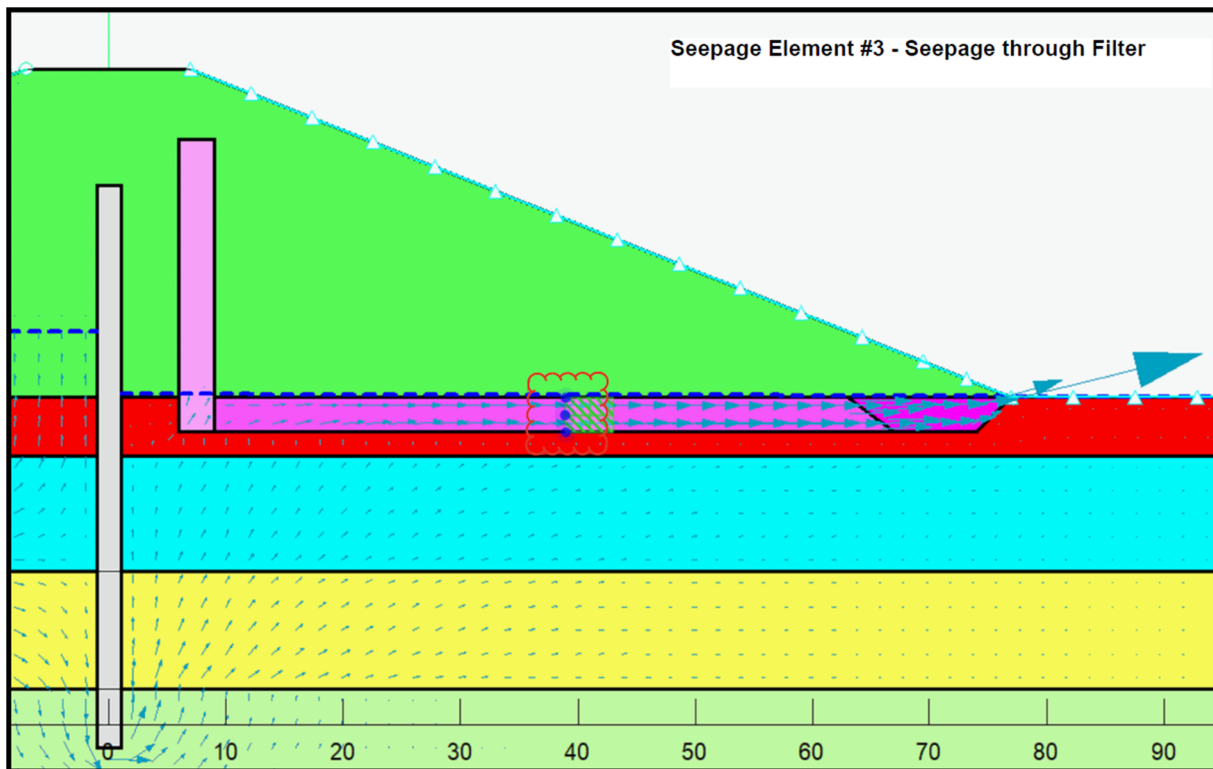




Figure 14: Seepage through Foundation Soils Illustration

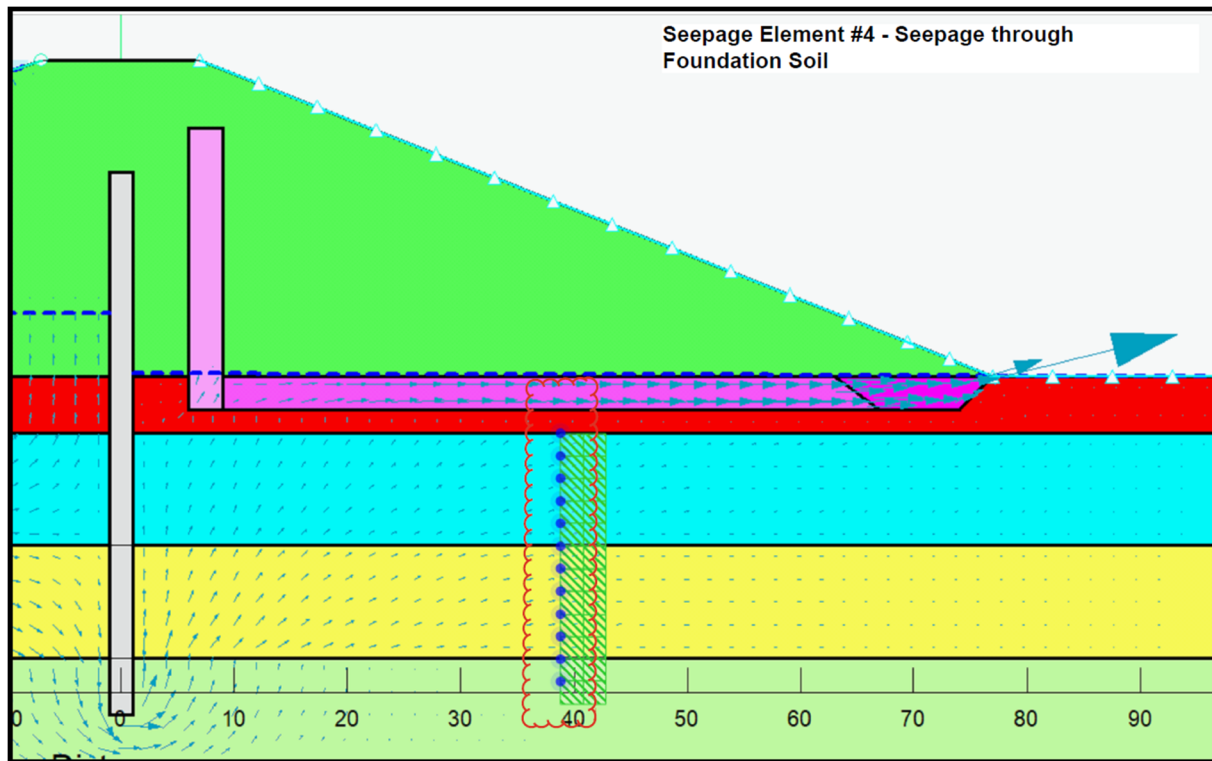


Table 8: Embankment Seepage Analysis Results

Dynamic Seepage Results (ft ³ / 28 days)				
Model	Seepage Element #1 Dam Face	Seepage Element #2 Flood Plain	Seepage Element #3 Filter	Seepage Element #4 Foundation
Cutoff Wall	0	3,567	6,897	4,606
No Cutoff Wall	13	38,892	107,773	51,247

The results of the seepage analysis show that a significant decrease in seepage is achieved when a cutoff wall is implemented into the model. The presence of a cutoff wall creates an impervious barrier to the flow of seepage beneath the dam as well as through the dam embankment. Although there is significant decrease to seepage, there is still enough seepage present due to the highly permeable foundation soils, that warrants the need for a well-managed filter system and relay of capture seepage away and downstream from the dam.

7.2 Structural Design

7.2.1. Relief Wells for Release of Confinement Pressure at the Dam Toe



The seepage analysis results shows that there are vertical seepage gradients near the toe of the dam that pose concern for hydrostatic uplift of the clay cap material at the surface of the existing ground. The estimated uplift load at the toe of the dam acting upon the underside of the clay cap is about 437 pcf and the conservative dead weight of the clay cap based upon a minimum thickness of 5'-0" is 550 pcf. The factor of safety for dead weight to overcome the uplift pressures is 1.26 and the recommended factor of safety for uplift conditions is 2.0. It is safely determined that the clay cap is not able to safely resist the expected uplift pressures and would have high risk in heaving during reservoir rise.

A solution to combat high vertical seepage gradients and the risk to heave is to relieve the confinement pressure by means of relief wells. Relief wells are vertical wells drilled into the foundation near the downstream toe of the dam embankment and they offer a path for the vertical water pressure to exit through. The exited water is either allowed to surface drain in a controlled channel away and downstream from the dam or is pumped out and drained downstream of the dam. Relief wells have been implemented in this preliminary design.

7.2.2. Structural Spillway Design

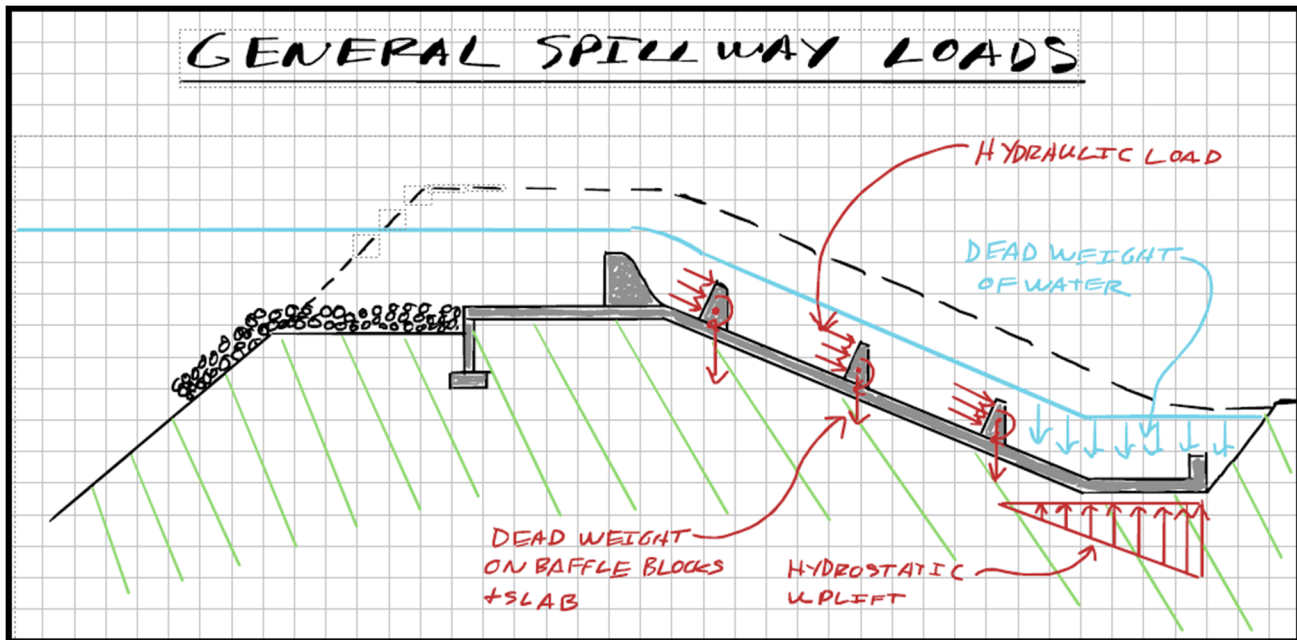
The structural concrete auxiliary spillway is comprised of a structural concrete spillway slab, three rows of concrete baffle blocks, a level concrete entrance section preceded by riprap armoring, a concrete ogee weir with a crest elevation of 1127.00 and a flat concrete stilling basin with a concrete sill at the end. The slope of the spillway is 3:1 matching the downstream slope face of the embankment. The overall width of the spillway (inside-to-inside dimension between retaining walls) is 360'-0".

The structural concrete spillway slab is comprised of 16" thick concrete supported upon a 2'-0" thick granular filter bedding. The entrance slab is 12" thick and supported the ogee weir construction. At the upstream end of the entrance slab a foundation wall sill is placed. Riprap armoring is placed upstream of this foundation wall sill and extended down the upstream face to prevent erosion during rapid drawdown conditions.

The baffle blocks are arranged in three rows spaced about 18'-0" apart along the slope of the spillway and spaced at 6'-9" apart along each row. The baffle blocks have dimensions of 6'-9" long, 4'-6" tall, 3'-3" wide at base and 1'-0" wide at top. There are a total of 78 baffle blocks are designed to be upon the spillway slope surface.

The design of the spillway slab incorporates several loading forces: uplift loads from hydrostatic pressure beneath the dam; gravity loads from the weight of the concrete slab, concrete baffle blocks and the weight of water; and the hydraulic loads enacted upon the spillway from the flow of water over the spillway.

Figure 15: Illustration of Spillway Loading



RISA Foundation software is used to apply loads upon the spillway slab from the hydraulic loading enacted upon the baffle blocks (See Section 7.2.3 Baffle Block Design for summation of loads from hydraulic forces), hydrostatic pressure at the base of the spillway and the dead weight of the slab. RISA calculates both bending stress and shear stress in the concrete produced by the overturning loads and gravity loads of the baffle blocks. Bearing pressure upon the subgrade soil is also calculated to ensure that appropriate bearing pressures are maintain as well as if there are any instances where the slab is not heavy enough to resist uplift pressures.

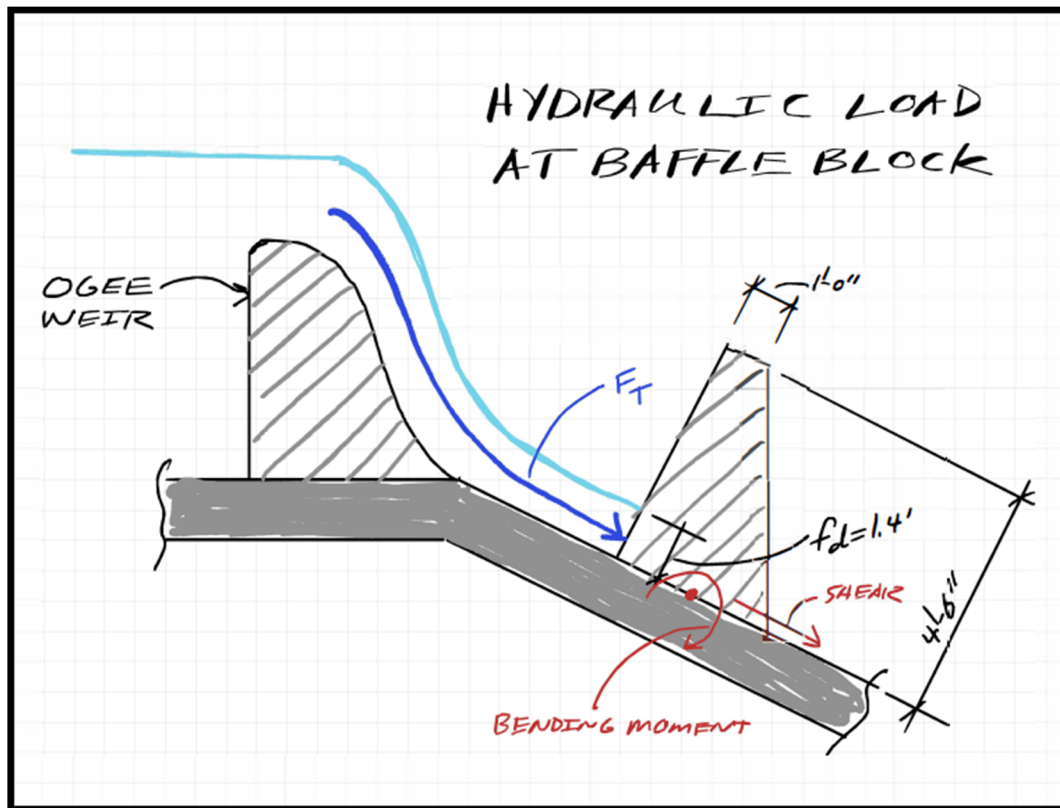
The results of the RISA model show that a maximum bending moment of 26.34 k-in is produced by the baffle blocks. A 16" thick slab reinforced with #6 bars spaced at 12" o.c. each way and at top and bottom of slab produces a maximum moment capacity of 225.3 k-in which is far greater than the applied bending moment. The 16" slab thickness is control by the need to have additional dead load for resisting uplift pressures and for having enough thickness for development length of rebar connecting the baffle block reinforcement to the slab.

7.2.3. Baffle Block Design

Baffle blocks are proposed to be constructed upon the spillway sloped slab arranged in three rows spaced 18'-0" apart down the spillway (Parallel to spillway flow) with each individual block in a row space at 6'-9" apart (perpendicular to the spillway flow). A total of 78 baffle blocks will be utilize to impeded and slow spillway flow enough to decrease hydraulic shear stress at the still basin. During spillway activation the baffle blocks are subjected significant forces enacted by the flow of water and impact of that water upon the surface of the baffle block.



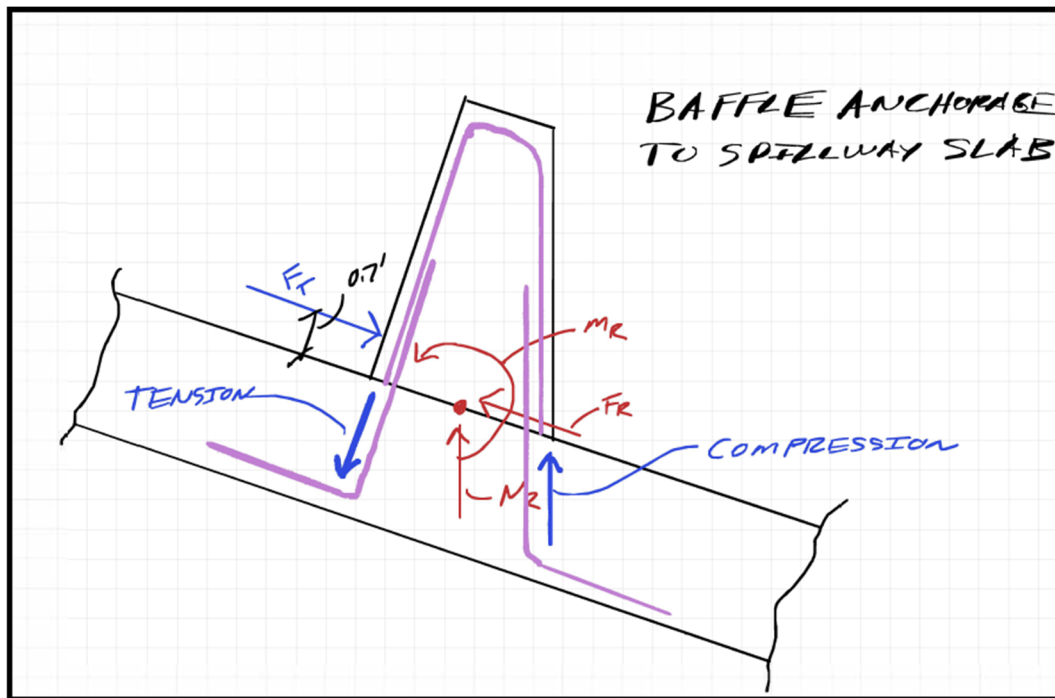
Figure 16: Illustration of Hydraulic Loading of Baffle Blocks



Through hydraulic analysis the maximum depth of flow of water at the interface with the first baffle block row is about 1.4 feet. This depth of water is used to calculate the total force on each baffle block. The force F_T is a function of velocity, area of water contact and unit weight of water. See appendices for calculations.

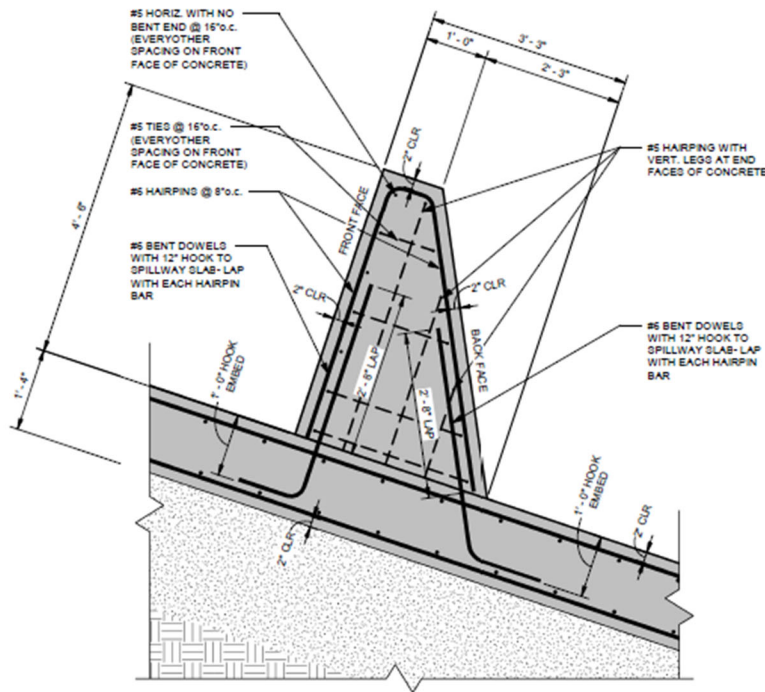


Figure 17: Illustration of Anchor Loads from Baffle Blocks to Structural Spillway Slab



The resulting total force (F_T) acting at each baffle block due to hydraulic loading of the flowing water is 98,550 lbs. This force equates to an overturning moment on each baffle block of 69,000 lb-ft. The anchorage of each baffle block to the spillway slab is required to have a tension capacity of 25,100 lbs. The design anchorage reinforcement is #6 bent dowel rebar @ 8" o.c. along the length of the block. The capacity of a single #6 bent dowel in tension with 10" embed into slab and 32" lap into baffle block is 19,800 lbs. Based upon 10 dowels per baffle block the total tension capacity of the rebar anchorage is 198,000 lbs, which is far greater than required tension load of 25,100 lbs. See appendices for calculations.

Figure 18: Reinforcement of Baffle Blocks



7.2.4. Spillway Retaining Wall Design

Structural concrete cantilevered retaining walls will be used to retain soil on each end of the structural auxiliary spillway. The retaining walls will be supported at their foundation with spread footings, which will be stepped with the sloped top of the wall, following the sloped spillway slab. The retaining walls each side of the spillway are 153'-0" in length, starting at the entrance section of the spillway with a wing wall configuration and ending at the still basin near the end of the riprap armoring.

The loading of the retaining wall includes an application of 66 pcf of equivalent fluid pressure lateral load based upon a soil unit weight of 120 pcf and an active pressure coefficient of 0.55. A surcharge of 100 psf at the top of grade at the retained side of the wall is applied as well for allowance for snow accumulation as well as possible vehicle loading.

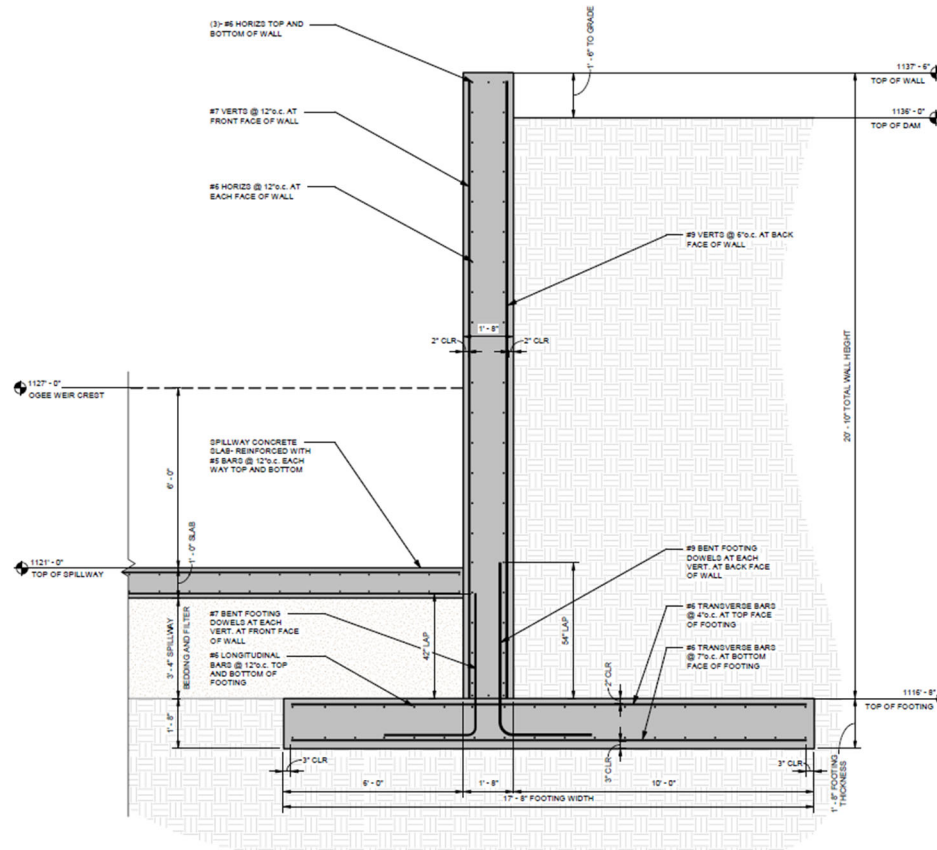
The concrete retaining wall is 20 inches thick and spanning 21'-0" at its maximum height, measured from top of footing elevation to the top of the wall. 2'-0" of grade free board will be present at the top of the wall, resulting in a maximum retained soil depth at the heel of the wall of 19'-0". At the toe of the wall the minimum soil depth over the footing toe will be about 4'-4". The concrete spread footing will be 20" thick and will have a 6'-0" toe and a 10'-0" heel resulting in a total footing width of 17'-8".

The retaining wall stem is reinforced with #9 vertical bars @ 6" o.c. on the tension face and #7 vertical bars @ 12" o.c. on the compression face. Longitudinally the wall is reinforced with #6 horizontal bars @ 12" o.c. each face placed on the inside face of the vertical bars. Vertical bent bars will be placed at each wall vertical matching bar size and developed into footing with 3" clearance to bottom of footing.



The retaining wall footing is reinforced with #6 transverse bars @ 4" o.c. at top face of footing and #6 transverse bars @ 7" o.c. at bottom face of footing. The longitudinal bars will be #6 @ 12" o.c. each top and bottom of footing.

Figure 19: Reinforcement of Retaining Wall



7.3 Phased Construction Considerations

7.3.1. Preloading of Embankment Foundation

Embankment foundation preloading utilizes the dead weight of placed soil to consolidate the existing subsurface soil intended for foundation support of the proposed dam embankment. The geotechnical investigation conducted in 1982 showed that there are concerns with settlement upon the introducing of a surcharge soil load above the native soils. In order to combat this concern, a preload will be used to consolidate the foundation soils. A preload is generally placed with low to moderate compaction (just enough compaction to keep it in place and well compacted enough for equipment to work upon it.) and placed to an elevation at which appropriate earth load would enact a large part of the expected consolidation. Expected consolidation would be determined through soil consolidation analysis.

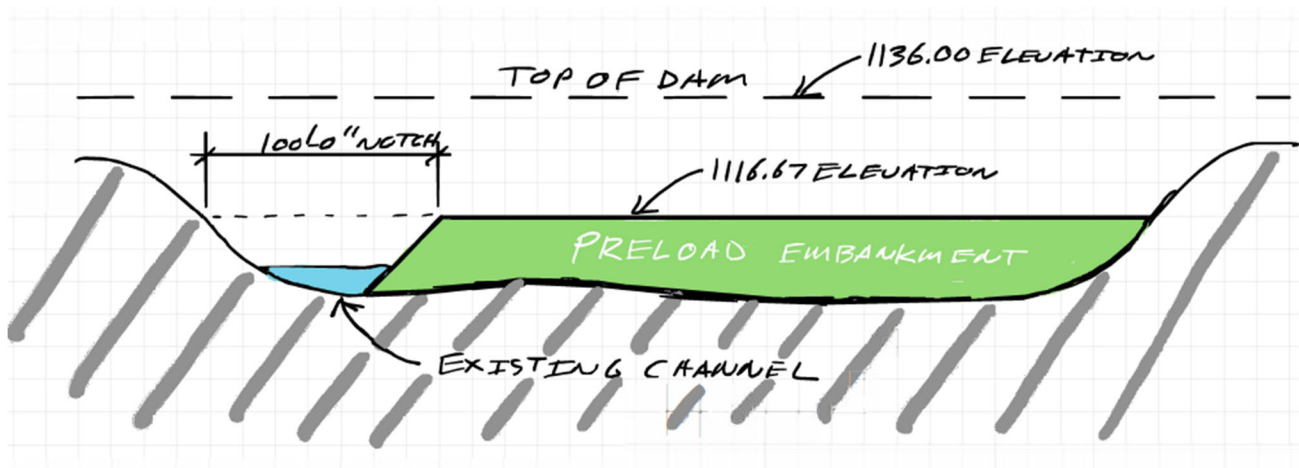
The preload earthen embankment would be placed and for about a one year time frame (or as estimated through analysis) settlement would be measured and recorded to determine what percentage of the total expected consolidation of the foundation was achieved. A goal percentage would be determined. After the preload and expected consolidation is achieved, then the preload earthen material is removed and recompacted with the appropriate compaction as indicated by the design for



the final earthen embankment. The final earthen embankment would be constructed with the understanding that additional settlement will occur, but since the preload was conducted the amount of settlement would be significantly less than if the entire embankment was constructed at one time.

The preload embankment would be constructed to an elevation of 1116.67 which will result in an embankment height of about 9'-0". Very little surface preparation will be conducted prior to preloading fill placement, as the earth placed in the preloading phase will be taken out and recompacted during the final construction of the embankment. No other construction would be implemented which would include the filling in of the existing channel, excavation of new channel, excavation for structural spillway and construction of on-grade conduit.

Figure 20: Illustration of Notch in Preload Embankment



7.3.2. Passing of 100-year floor event during Preloading Phase

Constructing the preload embankment will create a 9'-0" high temporary impedance to large flows through the flood plain. To allow for continued conveyance of the river, the existing channel will remain during the preloading phase and the river will be allowed to pass as it typically does through the preload embankment by simply not constructing the preload embankment across the existing channel near the right proposed abutment. A 100'-0" length of the preload embankment would be left out which is wide enough to allow the 100-year flood event to pass without over topping the preload embankment as well as passing with allowable flow shear stresses upon the embankment and existing abutment. The existing embankment and existing abutment would have to be lined with TRM (turf reinforcement matting) to reinforce the exposed grade against high shear stresses during high flood events.

7.3.3. Installation of Cutoff Wall after Preloading Phase

After the preloading phase the preload embankment would be removed and the material would be recompacted back up to the 1116.67 elevation. At this elevation the cutoff wall would be installed with either a long reach excavator or a hydromill equipment which is a cutting and mixing equipment which extends down into the embankment and foundation soils. At the desired level bentonite and soil are allowed to mix together along with the ground water to create the soil/bentonite slurry cutoff wall. In conditions where ground water is not present water is added, but in this case the high ground water will aid in the creation of the cutoff wall.



Figure 21: Photo of Long-Reach Excavator Installation of Cutoff Wall



Figure 22: Photo of HydroMill Installation of Cutoff Wall





8 Synthetic Events and Site Performance

To analyze the performance of UTR Dam 10, synthetic rainfall events were simulated and routed through the hydraulic model with dam features added. Synthetic rainfall assumptions and events are defined in the *Existing Conditions H&H Report*. The same assumptions are adopted for this analysis because flow recurrence matches strongly to rainfall recurrence, including critical 4-day duration, antecedent moisture conditions, nested precipitation, and no aerial reduction factor. The events include 2-year through 500-year return periods based on NOAA Atlas 14 rainfall depths with a 4-day duration. Runoff Curve Numbers were adjusted from a 24-hour Curve Number to a 4-day Curve Number based on guidance from *NEH, Part 630, Chapter 21* and were set to average antecedent moisture condition (AMC II). The rainfall distribution used for the synthetic events was developed using a “nesting” technique using the HMS Frequency Storm meteorologic model; intensity duration is 5 minutes, intensity position is 75%, and no area reduction factor applied.

8.1 Dam and Reservoir Site Performance

Floodwaters are stored in the UTR Dam 10 reservoir for the 2-year through 500-year events. The average rainfall depth, peak pool elevation, storage volume at peak elevation, pool area at peak elevation, and peak discharge through the dam for each event are shown in Error! Reference source not found..

Table 9: UTR Dam 10 Synthetic Rainfall Event Results

Event	NOAA Atlas 14 4-Day Rainfall Depth (inches)	Peak Inflow (cfs)	Peak Flood Pool Elevation (feet)	Storage Volume (ac-ft)	Pool Area (acres)	Peak Principle Spillway Discharge (cfs)	Peak Auxiliary Spillway Discharge (cfs)	Peak flow change due to dam (cfs)
2-year	2.93	744	1113.9	486	180	498	0	-33%
5-year	3.65	1312	1117.3	1389	343	616	0	-53%
10-year	4.31	1914	1120.2	2594	487	699	0	-63%
25-year	5.34	2988	1125.5	5809	750	825	0	-72%
50-year	6.21	3974	1127.8	7757	902	876	796	-58%
100-year	7.15	5106	1128.7	8635	1005	873	2360	-37%
500-year	9.66	8493	1130.2	10279	1229	874	6326	-15%

Roads within the pool area are raised to accommodate local residents travel, plus one additional road is planned to replace 40th Street NE that will be inundated during flood events. Road raises minimum elevation is 1129, which is above the 100-year peak pool elevation. This project cost was considered appropriate by Ralph Smith, Chief of Watershed Programs, under the flood damage reduction purpose.

8.2 Dam and Reservoir Performance Downstream of Site

Five reporting locations were selected to evaluate modeling results based on existing conditions and with UTR Dam 10 in place; the sites include major highways and cities, which are shown in **Figure 20**. Peak flows for each location and flood recurrence and are summarized in



. Flooding along the Turtle River, upstream of Hwy 2, is restricted to the confined valley walls, which range from 500 to 1,500 feet wide. However, below Hwy 2 the river enters lacustrine lakebed and floodplain extends for miles. Flood flows breakout into this extensive floodplain, which is why 1-D modeling extends are used from bank to bank, but floodplain is modeled with 2-D grid. Flows in the lower Turtle River are identified as Main channel and including breakouts to show more details as to flood flow reductions. Hydrographs for the frequency flood events are documented in **Figures 21 – 25**. Inundation depth plots are included for the 5, 25, and 100-year frequency rainfall events; the existing condition (red) and proposed UTR Dam 10 inundation (blue) are shown on **Figures 26 – 31**. The red represents lands no longer flooding with the project for that event; blue represents lands that are flooded for both conditions. In addition to extents, all structures (i.e. municipal, residences, ag out, residential out/garages, and bins) affected by the existing conditions inundation are shown in each figure, with total number within each view included in the legend. The total structures affected within the watershed are summation of upper and lower figures, i.e. 21 for 5-year, 74 for 25-year, and 205 for 100-year.

The figures show the maximum inundation extent that occurs during the event; therefore the full benefit of the project is not apparent on the inundation figures. Flood damages, especially damages to agricultural lands, are caused both by the extent of the inundation and, almost equally as important, the duration of inundation. The total inundated acres and cropland inundated acres for the analyzed events based on duration are shown in Error! Reference source not found. **Report**. Typical crops within the Turtle River Watershed include wheat, soybeans, corn, dry beans, potatoes, and sugar beets. Flood inundation durations greater than four days generally represents the maximum anticipated damages, or total loss, for the crop types in the study area. Error! Reference source not found. **Report** shows total inundated acres for durations less than 4-days. To provide benefit to agricultural lands, flood durations between 0 and 4 days should be reduced. Zero to 4 days represents the time between no inundation and total crop loss inundation.

*Table 10: Peak Flow Changes Downstream of UTR Dam 10*

Location	Event	Existing Conditions	With UTR Dam 10	% Change
Hwy 18	2-year	872	613	-30%
	5-year	1,546	817	-47%
	10-year	2,274	1,003	-56%
	25-year	3,580	1,309	-63%
(5 river miles below dam)	50-year	4,808	1,848	-62%
	100-year	5,983	3,571	-40%
	500-year	10,403	8,118	-22%
Hwy 2 - West Crossing	2-year	870	613	-30%
	5-year	1,542	817	-47%
	10-year	2,265	1,002	-56%
	25-year	3,546	1,306	-63%
(11 river miles below dam)	50-year	4,673	1,844	-61%
	100-year	5,935	3,548	-40%
	500-year	10,567	8,115	-23%
Hwy 2 - East Crossing	2-year	964	707	-27%
	5-year	1,675	954	-43%
	10-year	2,444	1,184	-52%
	25-year	3,800	1,573	-59%
(23 river miles below dam)	50-year	5,021	2,401	-52%
	100-year	6,333	4,203	-34%
	500-year	10,918	9,689	-11%
Mekinock - Channel Only (24th Ave NE)	2-year	1,070	931	-13%
	5-year	1,520	1,301	-14%
	10-year	1,694	1,550	-8%
	25-year	1,851	1,762	-5%
(42 river miles below dam)	50-year	1,931	1,898	-2%
	100-year	2,026	2,023	0%
	500-year	2,319	2,319	0%
Mekinock - Including Breakouts	2-year	1,220	989	-19%
	5-year	2,188	1,623	-26%
	10-year	3,109	2,293	-26%
	25-year	4,628	3,418	-26%
(42 river miles below dam)	50-year	5,975	4,453	-25%
	100-year	7,528	5,571	-26%
	500-year	13,943	10,248	-27%
Manvel - Channel Only (CR 33)	2-year	1,216	1,188	-2%
	5-year	1,740	1,653	-5%
	10-year	2,354	2,197	-7%
	25-year	3,248	3,065	-6%
(72 river miles below dam)	50-year	3,456	3,434	-1%
	100-year	3,592	3,578	0%
	500-year	3,241	3,245	0%
Manvel - Including Breakouts	2-year	1,216	1,188	-2%
	5-year	1,756	1,671	-5%
	10-year	2,421	2,245	-7%
	25-year	4,060	3,684	-9%
(72 river miles below dam)	50-year	6,242	5,067	-19%
	100-year	7,865	6,292	-20%
	500-year	17,239	13,445	-22%



8.3 Temporary (During Construction)

During construction the majority of the embankment will be preloaded to consolidate foundation soils. An approximately 100-foot-wide notch at the current river channel will not be preloaded to maintain an area for river flows. The preloading top elevation is set at 1116.0. Frequency flow hydraulic results are summarized in table 11; which the 100-year and smaller floods are not expected to overtop the preloaded section. Erosive forces at higher floods become significant, therefore protection measures along the preloaded slope includes turf reinforcement mats.

Table 11: Frequency Flow Hydraulic related to Preloading Embankment

Event	Flow	Channel	WSEL	Depth	Top Width	Channel Velocity	Overbank Velocity	EG Slope
	(cfs)	(feet)	(feet)	(feet)	(feet)	(fps)	(fps)	(ft/ft)
2-yr	744	1102.43	1108.6	6.17	73.87	3.58	1.62	0.0017
5-yr	1312	1102.43	1109.79	7.36	79.59	4.67	2.4	0.0020
10-yr	1914	1102.43	1110.41	7.98	81.29	6.18	3.25	0.0031
25-yr	2988	1102.43	1111.27	8.84	82.5	9.06	4.82	0.0063
50-yr	3974	1102.43	1112.25	9.82	81.64	12.6	6.65	0.0128
100-yr	5106	1102.43	1113.35	10.92	84.61	13.68	7.42	0.0123

9 Implementation Cost Estimate

The cost estimates below were developed on the basis of recent past construction projects in ND, supplier quotes, and experience of NRCS engineers.

Table 12: Construction Costs

CONSTRUCTION ITEM	UNIT	QUANTITY	UNIT COST	SUB-TOTAL
CLEARING AND GRUBBING	ACRE	15.4	\$3,200.00	\$49,280
RIP-RAP	CY	4,175.0	\$110.00	\$459,250
EARTH FILL	CY	137,600.0	\$11.00	\$1,513,600
EARTH CUT	CY	45,700.0	\$6.00	\$274,200
DRAIN MATERIAL FILL	CY	24,905.0	\$75.00	\$1,867,875
DRAIN PIPE AND BEDDING	LF	5,000.0	\$15.00	\$75,000
CONCRETE WALLS	CY	503.0	\$1,300.00	\$653,900
CONCRETE FOOTINGS	CY	438.0	\$900.00	\$394,200
CONCRETE CONDUIT (ON-GRADE SPILLWAY)	CY	280.0	\$1,600.00	\$448,000
STRUCTURAL SPILLWAY SLAB	CY	1,496.0	\$1,100.00	\$1,645,600
CONCRETE BAFFLE STRUCTURES	CY	187.0	\$1,300.00	\$243,100
Ogee WEIR STRUCTURE	CY	405.0	\$1,600.00	\$648,000
SOIL/BENTONITE SLUURY CUTOFF WALL	CY	8,313.0	\$215.00	\$1,787,295



RELIEF WELL STRUCTURES	EACH	65.0	\$10,233.80	\$665,197
EROSION CONTROL AND RE-VEGETATION	ACRES	16.3	\$7,500.00	\$122,250
DE-WATERING	LUMP SUM	1.0	\$150,000.00	\$150,000
TRAFFIC CONTROL	LUMP SUM	1.0	\$30,000.00	\$30,000
ROAD IMPROVEMENTS	LS	1.0	\$795,385.00	\$795,385
MOBILIZATION	LS	1.0	\$650,000.00	\$650,000
<i>Total Construction Costs</i>				\$ 12,472,132

Estimates completed for land rights were made on the basis of the conservatively, on the basis of the ND Department of Trustlands 2024 County Rents and Prices. Flowage easements were conservatively estimated at 100% of the estimated land value, given general opposition to the project in the area upstream of the dam.

Table 13: Land Rights, Relocation Costs

	Acreage	Unit Cost	Cost
<i>Fee Title Land Purchase (land under dam, spillways)</i>			
Irrigated Cropland	3	\$ 5,531/ac	\$ 10,593
Pastureland, Natural Lands	7	\$1,182/ac	\$ 8,274
Subtotal	10		\$ 18,867
<i>Flowage Easements (upstream of dam, to top of dam elevation)</i>			
Irrigated Cropland	372	\$ 5,531/ac	\$2,057,532
Non-irrigated Cropland	671	\$3,531/ac	\$2,369,301
Pastureland, Natural Lands	1,608	\$1,182/ac	\$3,116,304
Subtotal	2,651		\$7,543,137
<i>Construction Access Easements (2 years)</i>			
Irrigated Cropland	42	\$182.40/ac	\$7,661
Pastureland, Natural Lands	75	\$22.50/ac	\$1,687
Subtotal	117		\$9,348
House Relocation			\$300,000
<i>Total Land Rights, Relocation</i>			\$7,871,352

Assumptions for engineering costs assume NRCS would complete the majority of final design work, with the exception of hiring out geologic drilling work and preparation of legal land surveys. Construction engineering would be completed with a mix of NRCS staff and hired consulting firm staff.

Table 14: Summary, Estimated Implementation Costs

Item	Estimate
Final Engineering Design	\$ 550,000
Land Rights, Relocation	\$ 7,871,352
Sponsor Admin, Legal	\$ 310,000



Construction Engineering	\$ 750,000
Construction	\$ 12,472,132
<i>Total</i>	\$ 21,953,484

Annual Operation and Maintenance Costs for the dam are estimated to be \$5,000/year.

10 Preliminary Design Team

Jonathan Petersen, P.E. ND Hydraulic Engineer

Tom Schanandore, P.E. ND Design Engineer

Preliminary Design Reviewers:

Christi Fisher, P.E. ND State Conservation Engineer

Karl Visser, P.E. NDCSMC Hydraulic Engineer

Carmen Berry, P.E., NDCSMC Geotechnical Engineer

James Smith, P.E., NDCSMC Design Engineer

Jacob Robison, P.E., Design Engineer

Hong Wong, P.E., NDCSMC Hydraulics Engineer

Ted Huscher, P.E., Geologist

John Markov, P.E., Geologist

The NRCS National Design, Construction, and Soil Mechanics Center (NDCSMC) completed a preliminary design review. The review included concept level hydraulic and geotechnical evaluation of the site, alternative analysis, and preliminary design. The hydraulic review focused on roller compacted concrete and chute baffle computations, as well as principal spillway outletting to auxiliary spillway basin. The hydraulic reviewers agreed with validity of design references and dimension assumptions based on most efficient design at this planning level to see if project will meet positive benefit to cost ratio. However, if the project goes past this planning level a scale model or 3-D modeling should be completed to analyze flow dynamics of ogee with chute baffles, and combined principal and auxiliary spillways.

The geotechnical review focused on geologic investigation, relief system, foundation preparation, cutoff trench, seepage gradients, filter system, and homogeneous embankment. The designers and reviewers concluded that additional geologic investigation would be necessary with any future final design work past this current planning stage. A more robust comparative analysis between steady-state and transient models was implemented to determine more precise net uplift pressure at the dam. The result of the analysis is addition of relief wells to the design. Foundation preparation discussion



focused on slightly pre-consolidated sediments that will require pre-loading of the foundation as part of the design. Due to high permeability of foundation soils, a soil-cement-bentonite slurry trench is added to design as a cutoff wall. There was considerable discussion regarding cutoff wall since this is a dry dam. Due to the dam having no permanent pool, a filter system is still required, but zoned embankment is not required. Frost heave and freeze/thaw cyclic loading will need to be considered in future design work.

11 References

Applied Weather Associates (2021). Probable Maximum Precipitation Study for North Dakota Final Report, June 2021.

NOAA (2024). Wind speed accessed from www.climate.gov

Nugroho, J., Soekarno, I., and Soeharno, A.W.H., 2019. Experimental Study of Energy Dissipation at Baffled Chute Spillway. Jurnal Teknik Sipil, ISSN 0853-2982. 01 April 2019

ND DWR (2023). ND LiDAR Dissemination MapService, accessed from <https://lidar.dwr.nd.gov/>

ND DWR (2024). North Dakota Dam Safety Standards Policy Reg_05.2024. 01/01/2024.

USACE. (2020). HEC-HMS Hydrologic Modeling System Version 4.4.1. Davis, California, USA.

USACE. (2019). HEC-RAS River Analysis System Version 5.0.7. Davis, California, USA.

USACE. (2013). Red River of the North Hydraulic Modeling – Phase 2.

USDA, NRCS (2019a). Technical Release 210-60 – Earth Dams and Reservoirs, March 2019.

USDA, NRCS (2019b). National Engineering Handbook Part 630 Hydrology, Chapter 21 Design Hydrographs, May 2019

USDA, NRCS (2019c). National Engineering Handbook Part 630 Hydrology, Chapter 4 Storm Rainfall Depth and Distribution, Aug 2019

USDA, NRCS (2017). National Engineering Manual Part 520 – Soil and Water Resource Development, Subpart C – Dams.

USDA, NRCS (2014). Technical Release 210-56 – A Guide for Design and Layout of Vegetated Wave Protection for Earthen Embankments and Shorelines.

USDA, NRCS. (2008). National Engineering Handbook Part 650 Surveying

USDA, NRCS. (2004). National Engineering Handbook Part 630 Hydrology

DOI, USBR. (1988). ACER Technical Memorandum 11 – Downstream Hazard Classification Guideline. Engineering and Research. Denver, CO: U.S. Department of Interior.

DOI, USBR. (1987). Design of Small Dams, Third Edition

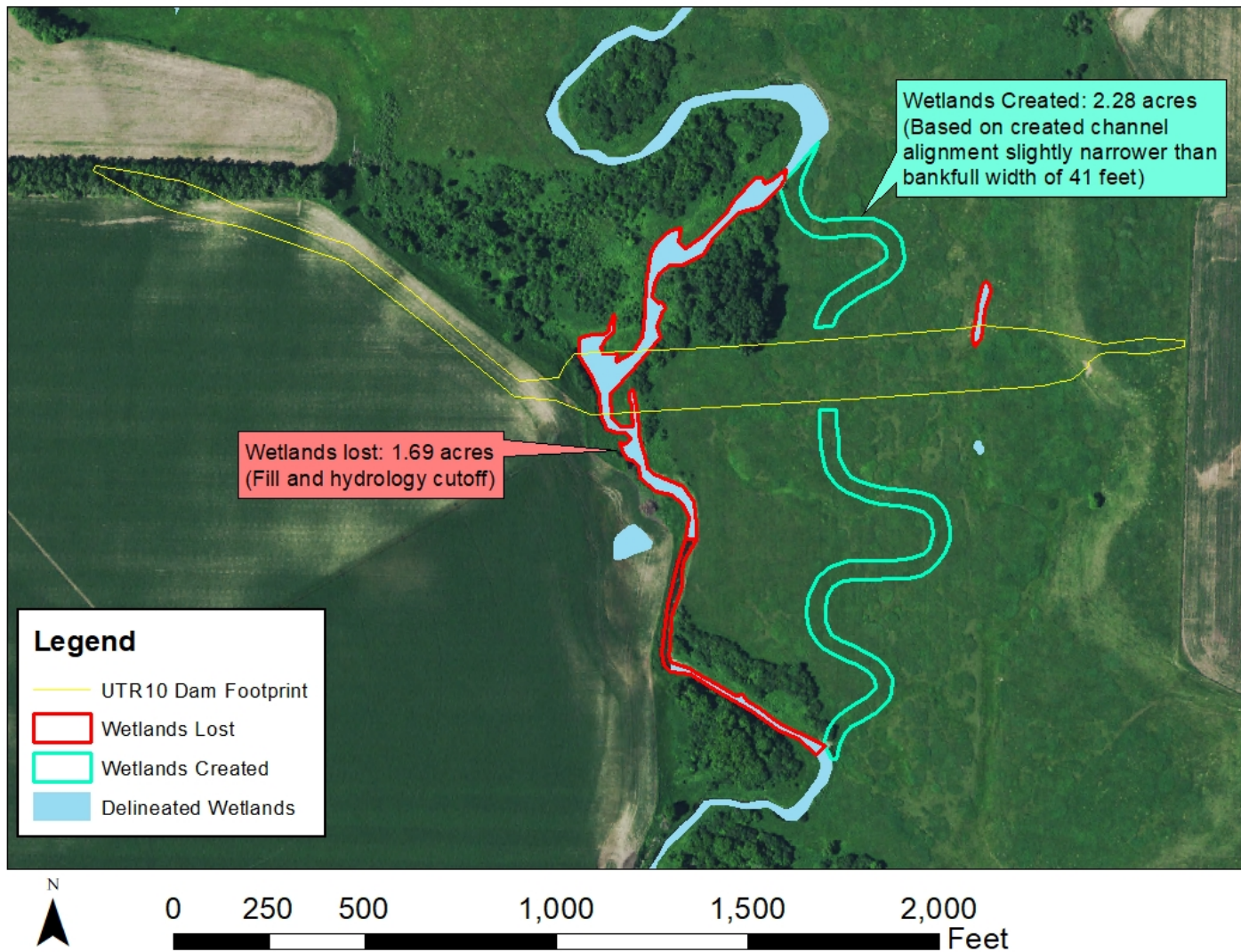


Figure 1: Wetlands Lost and Wetlands Created



TR 210-60 Breach Equations			
Basic Information			
Watershed:	North Branch Turtle River	Date:	6/12/2024
State:	North Dakota	Prepared By:	Jonathan Petersen
County:	Grand Forks	Checked By:	Brian Mager
Dam Information			
Name	Upper Turtle River Dam #10		
Elevations (NAVD88, ft)			
Top of Dam:	1136.0		
WSEL at Breach:	1136.0		
Valley Floor (Avg):	1108.0		
Misc.			
Valley Width (L):	1,300 feet		
Full Volume (V_s):	20,300 acre-ft		
Cross Section Area (A):	2,550 sq. ft		
Breach Discharge Computations			
Depth of Water (H_w):	28.0 feet		
Therefore, use equations in #2 ($H_w < 103$ ft)			
$B_r (V_s H_w / A) =$	222.9 acres		
Theoretical Width (T)=	501.6 feet	(L>T, therefor use L)	
Q_{max} (Standard) =	1,626,846 cfs		
Q_{max} (Upper Limit) =	30,914 cfs		
Q_{max} (Lower Limit) =	13,275 cfs		
Q_{max} (Adopted):	30,914 cfs		

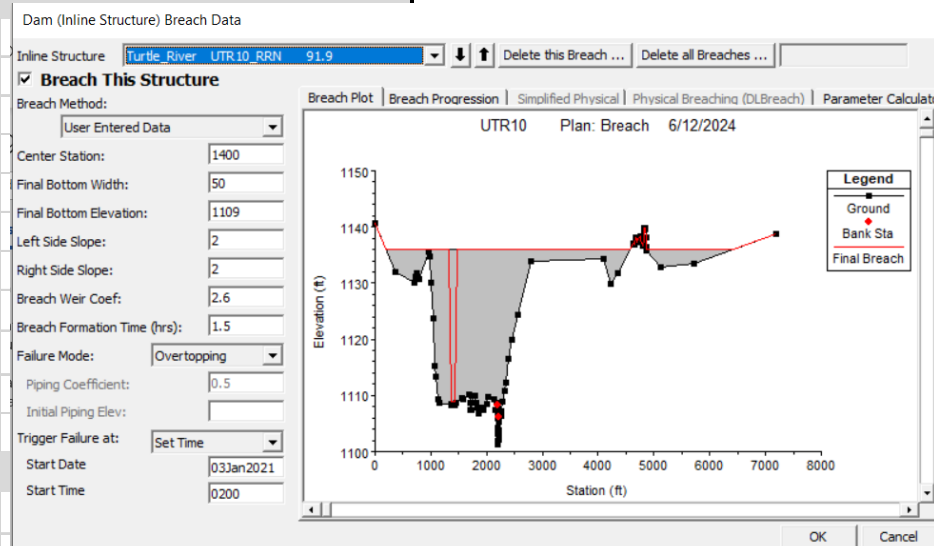


Figure 2: Peak Breach Discharge calculations and data



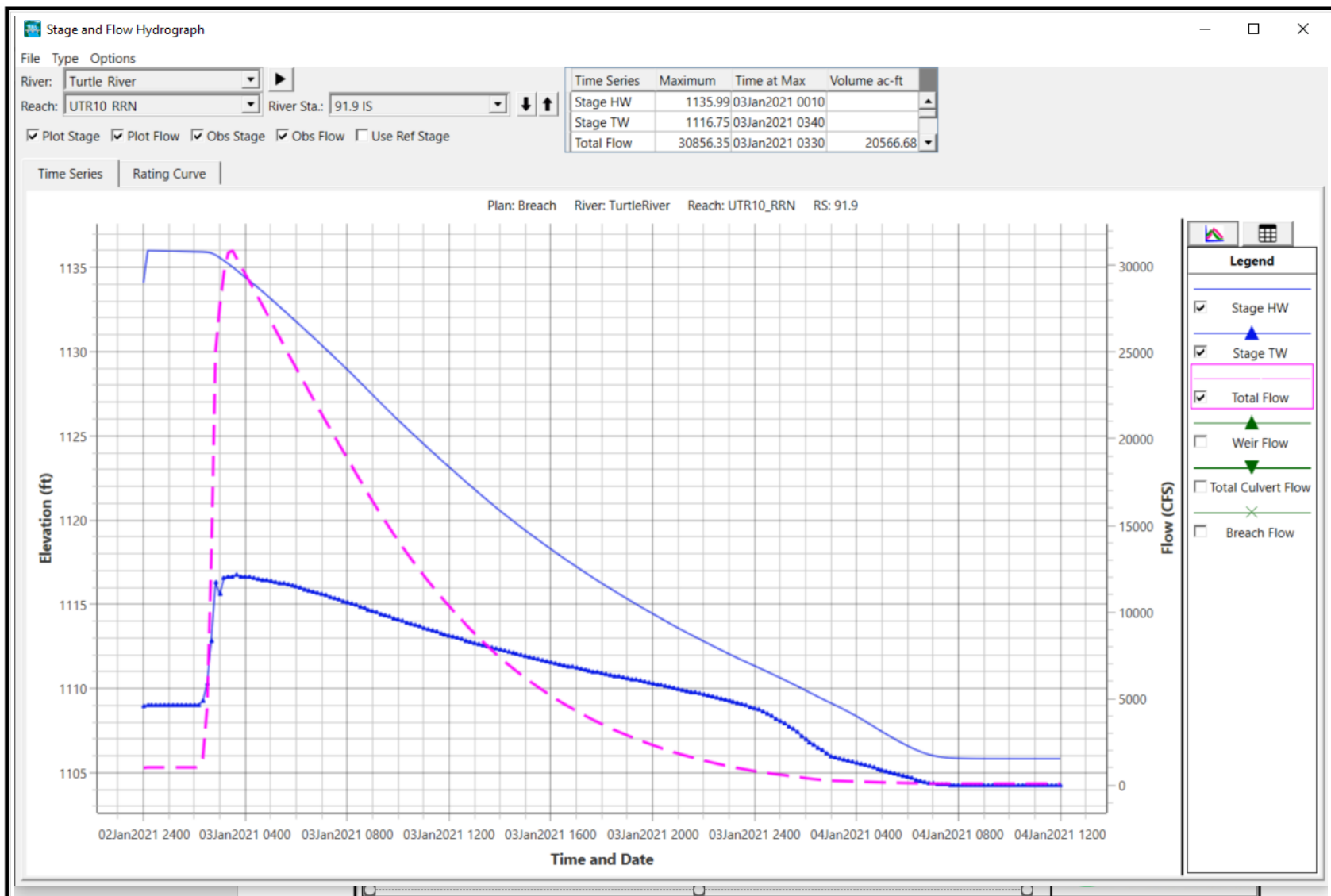


Figure 3: Breach Discharge hydrograph

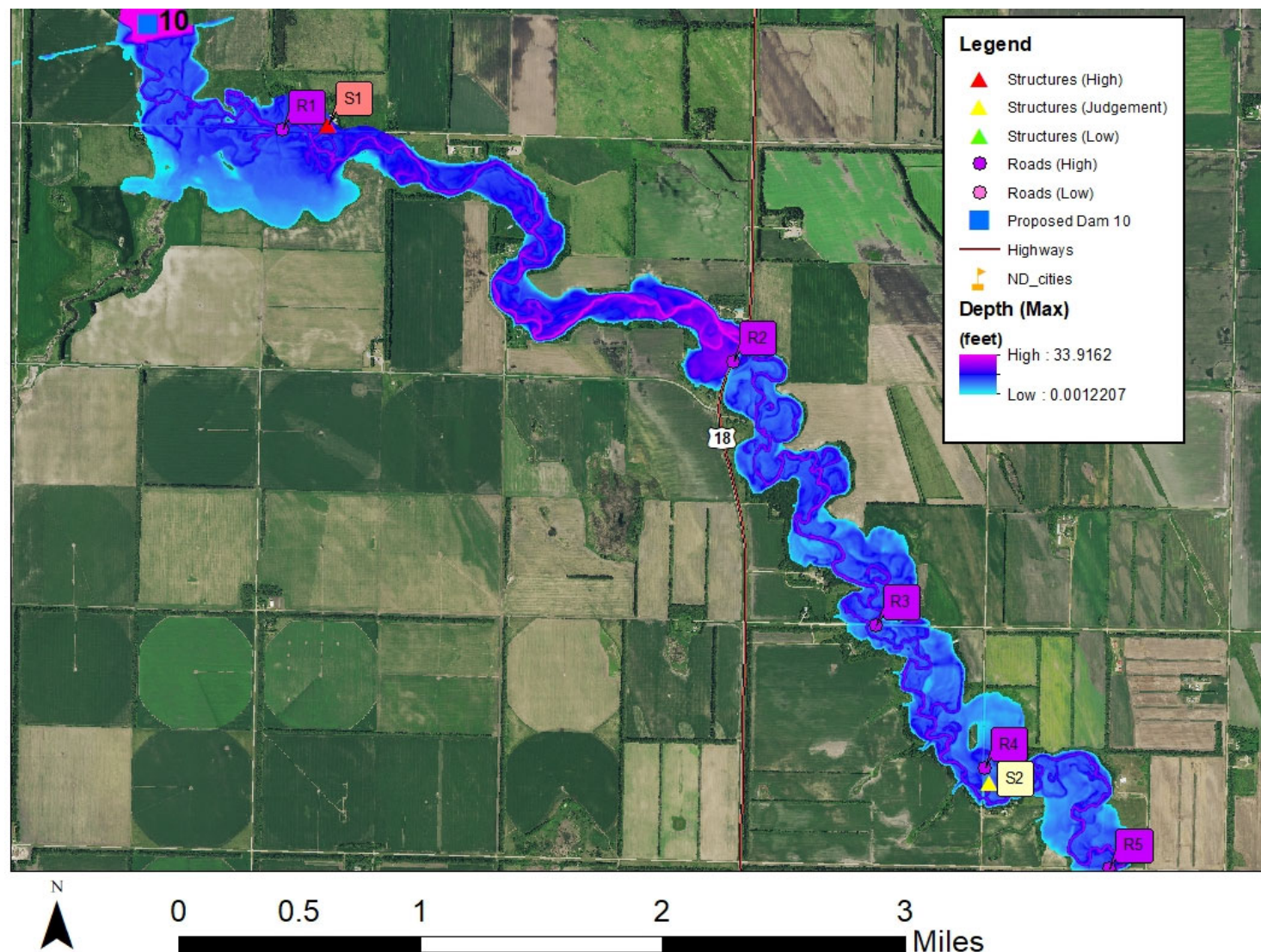


Figure 4: Breach Inundation Maximum Depths, including structures and roads (Upper Reach)

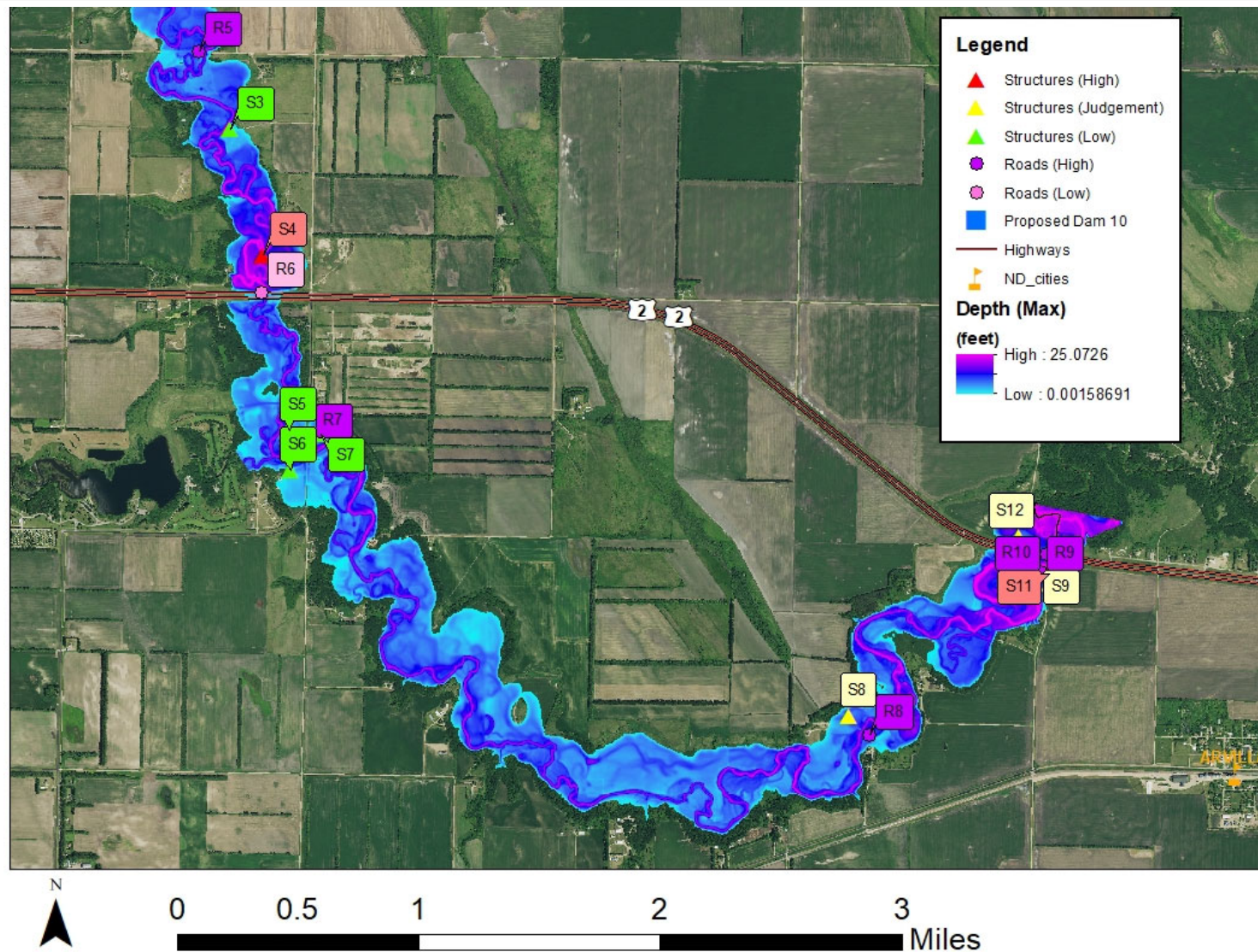


Figure 5: Breach Inundation Maximum Depths, including structures and roads (Lower Reach)

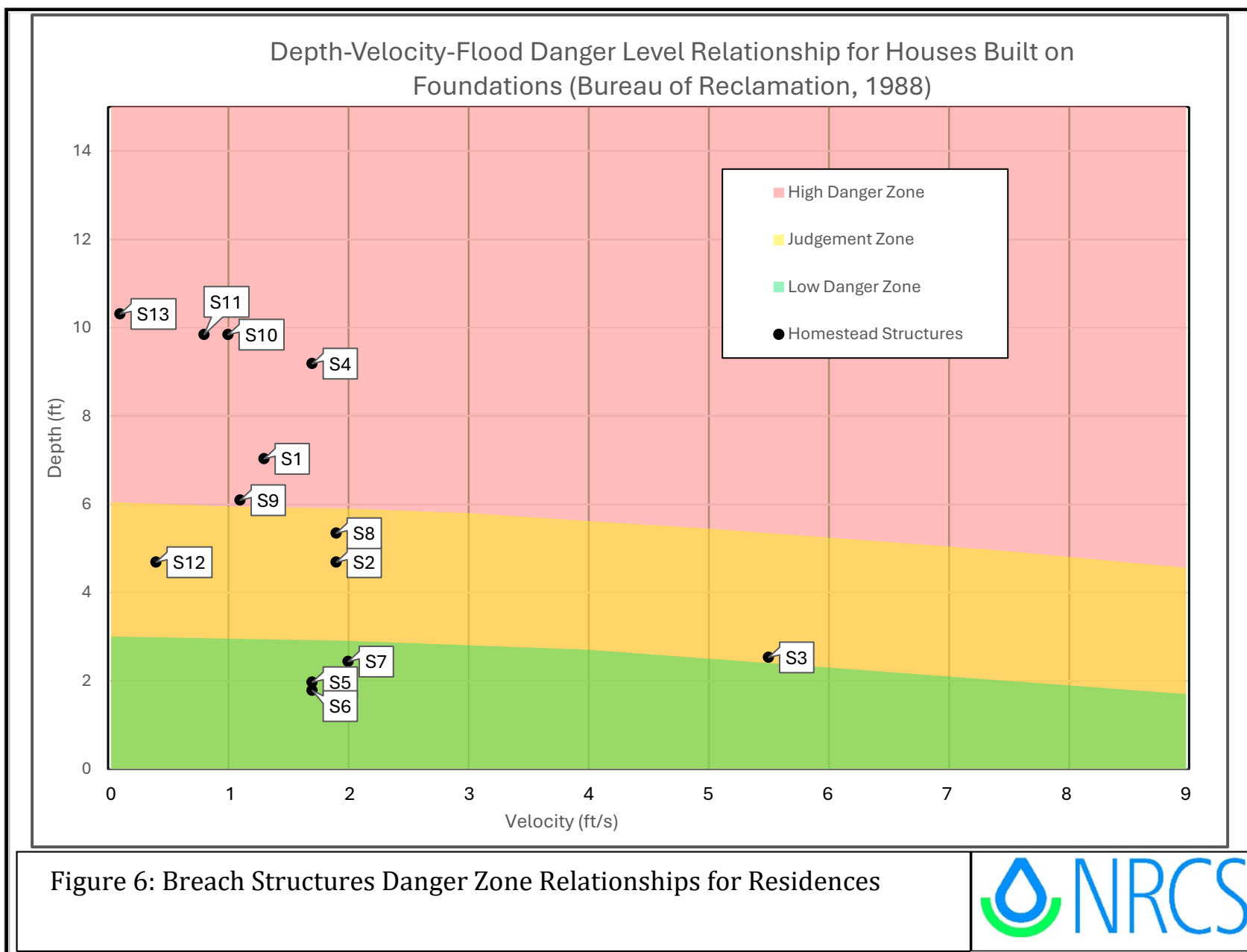


Figure 6: Breach Structures Danger Zone Relationships for Residences

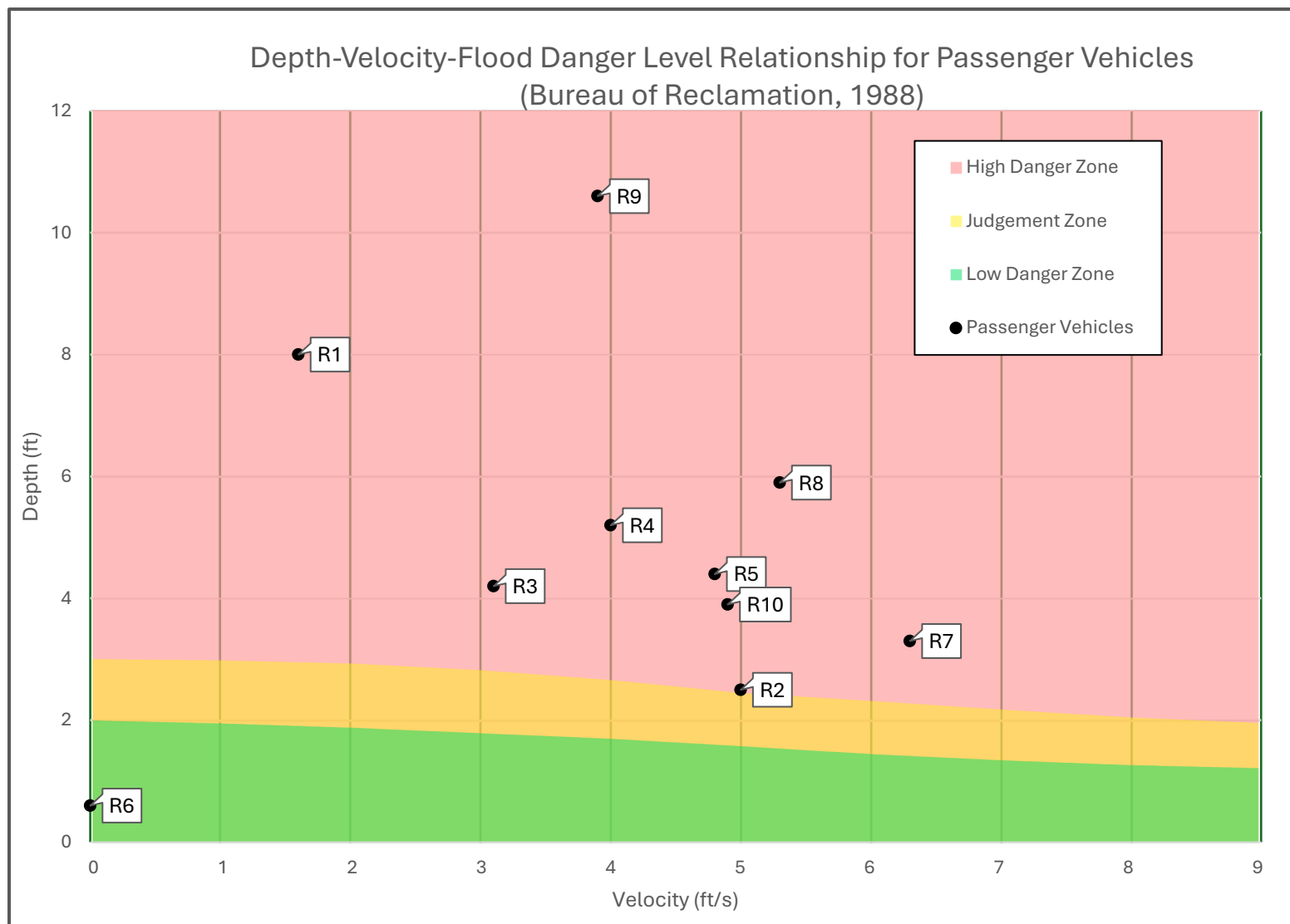
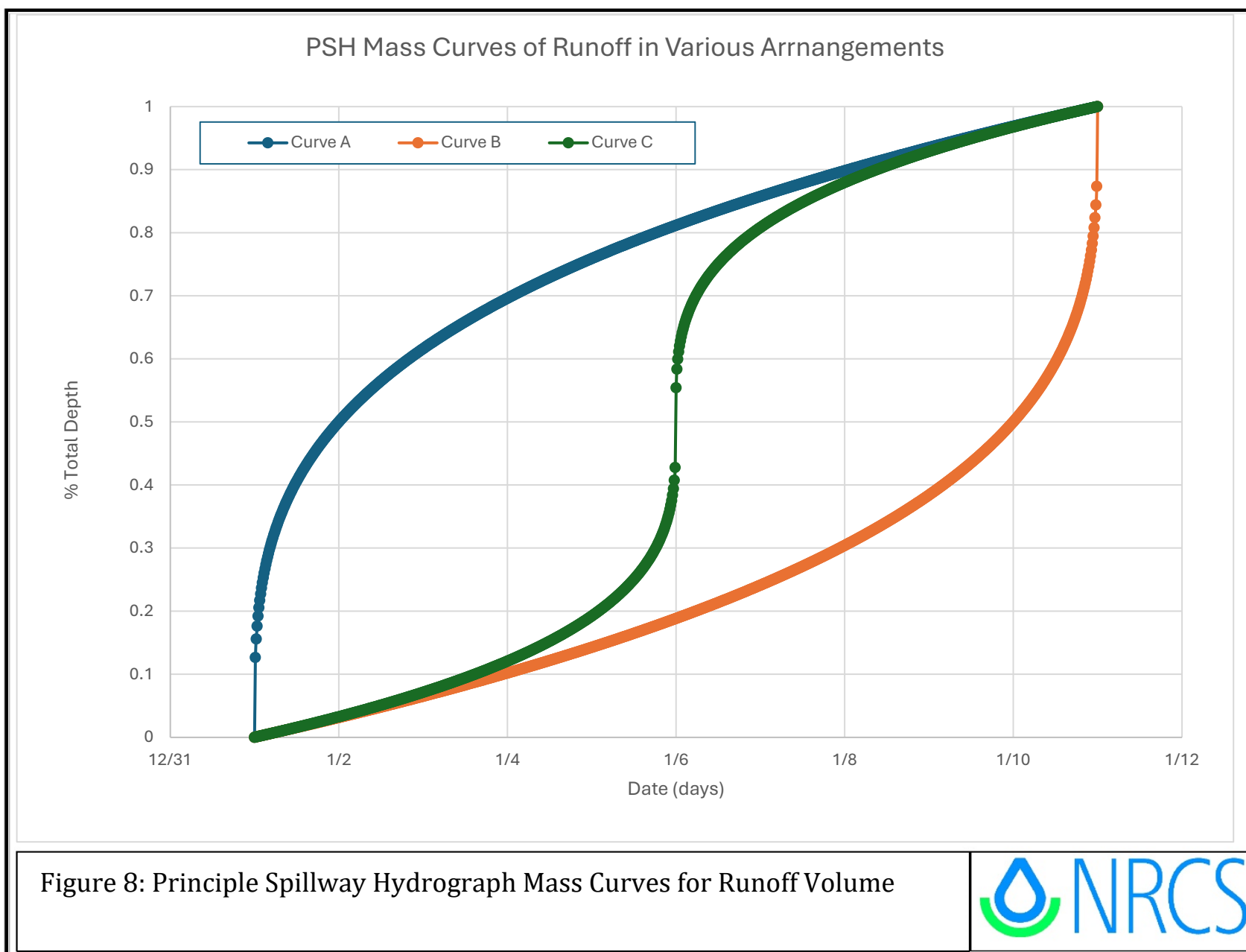


Figure 7: Breach Vehicle Danger Zone Relationships for Roads



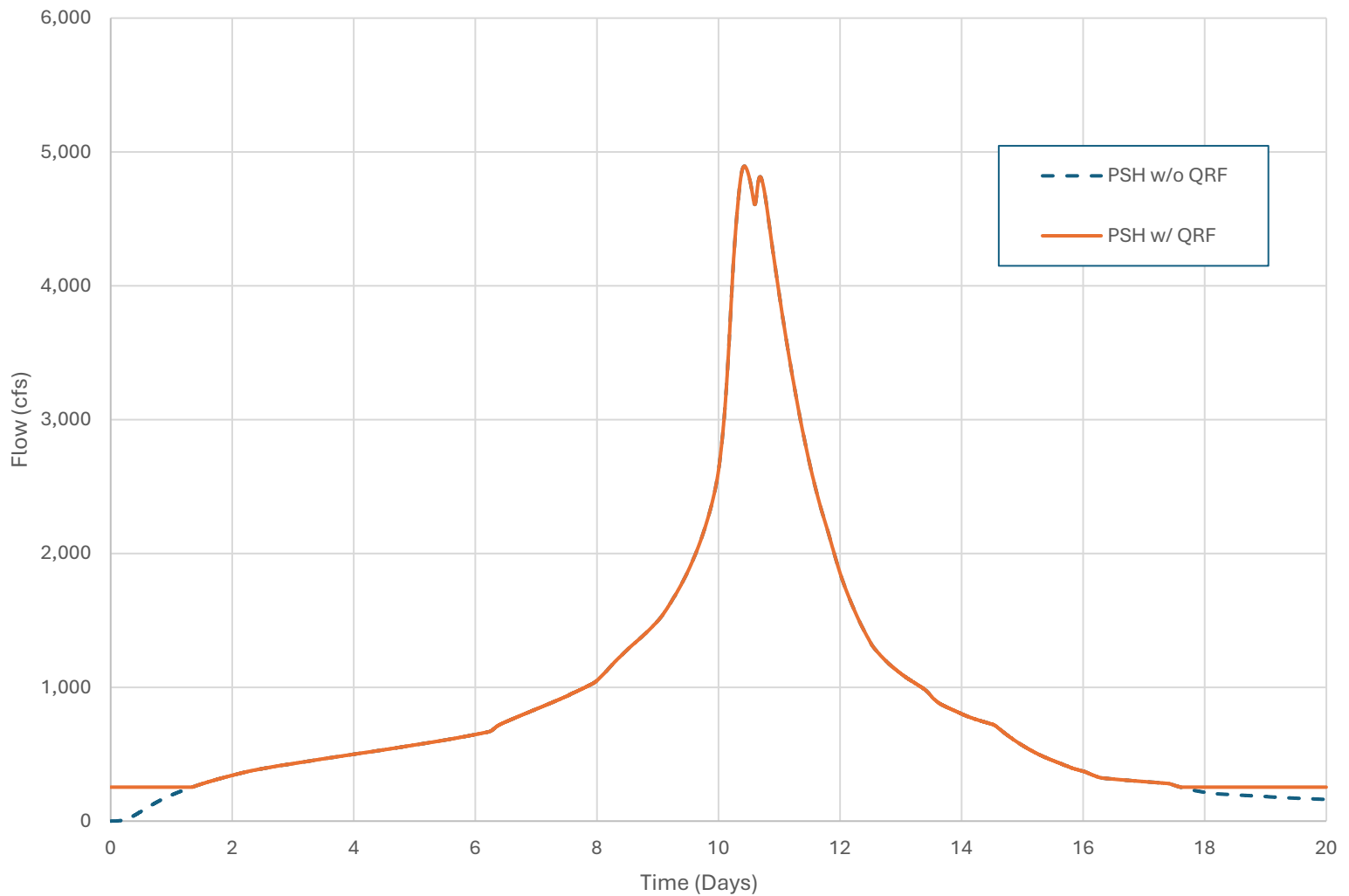


Figure 9: Principle Spillway Hydrograph Mass Curve UTR 10 Dam Inflow



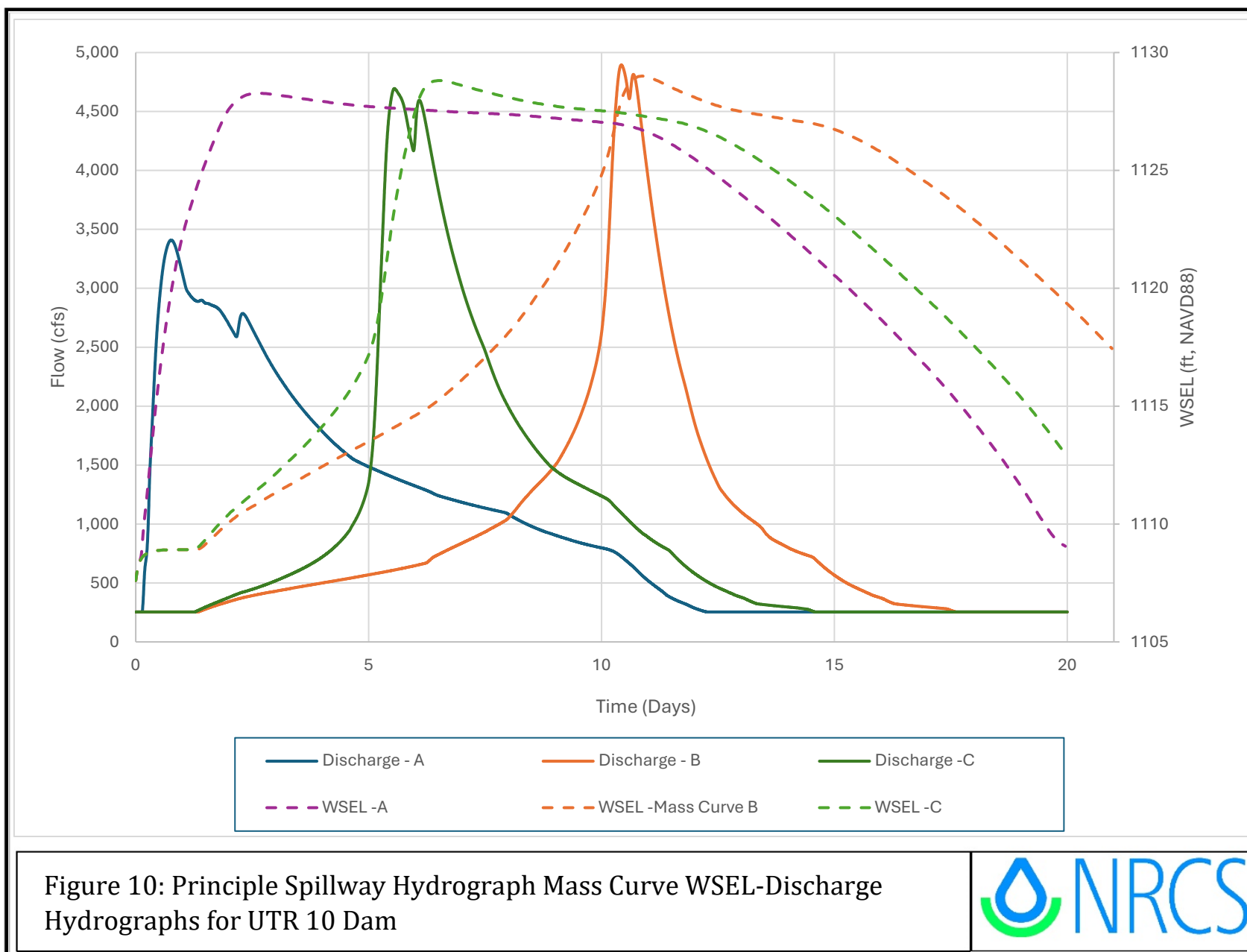
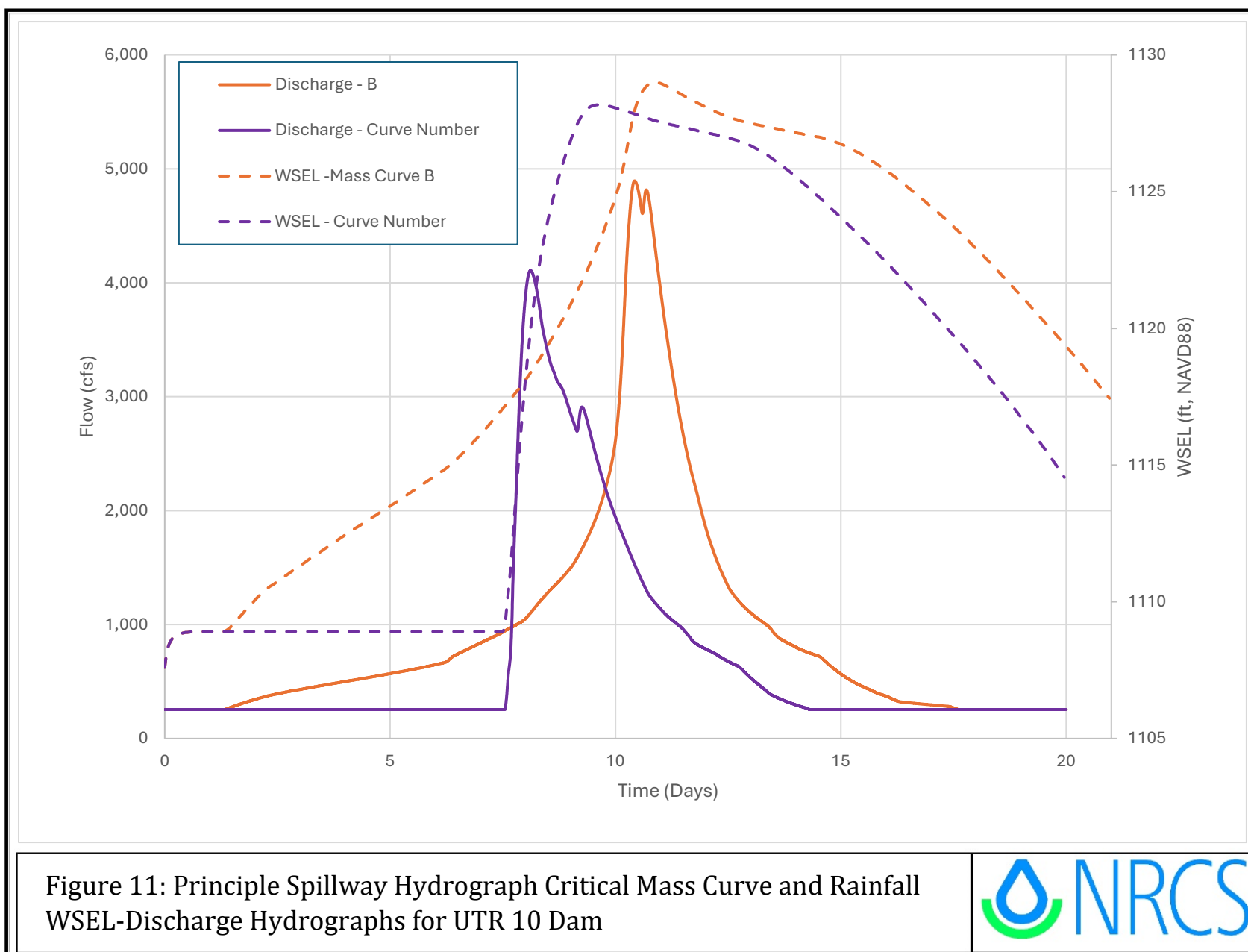


Figure 10: Principle Spillway Hydrograph Mass Curve WSEL-Discharge Hydrographs for UTR 10 Dam





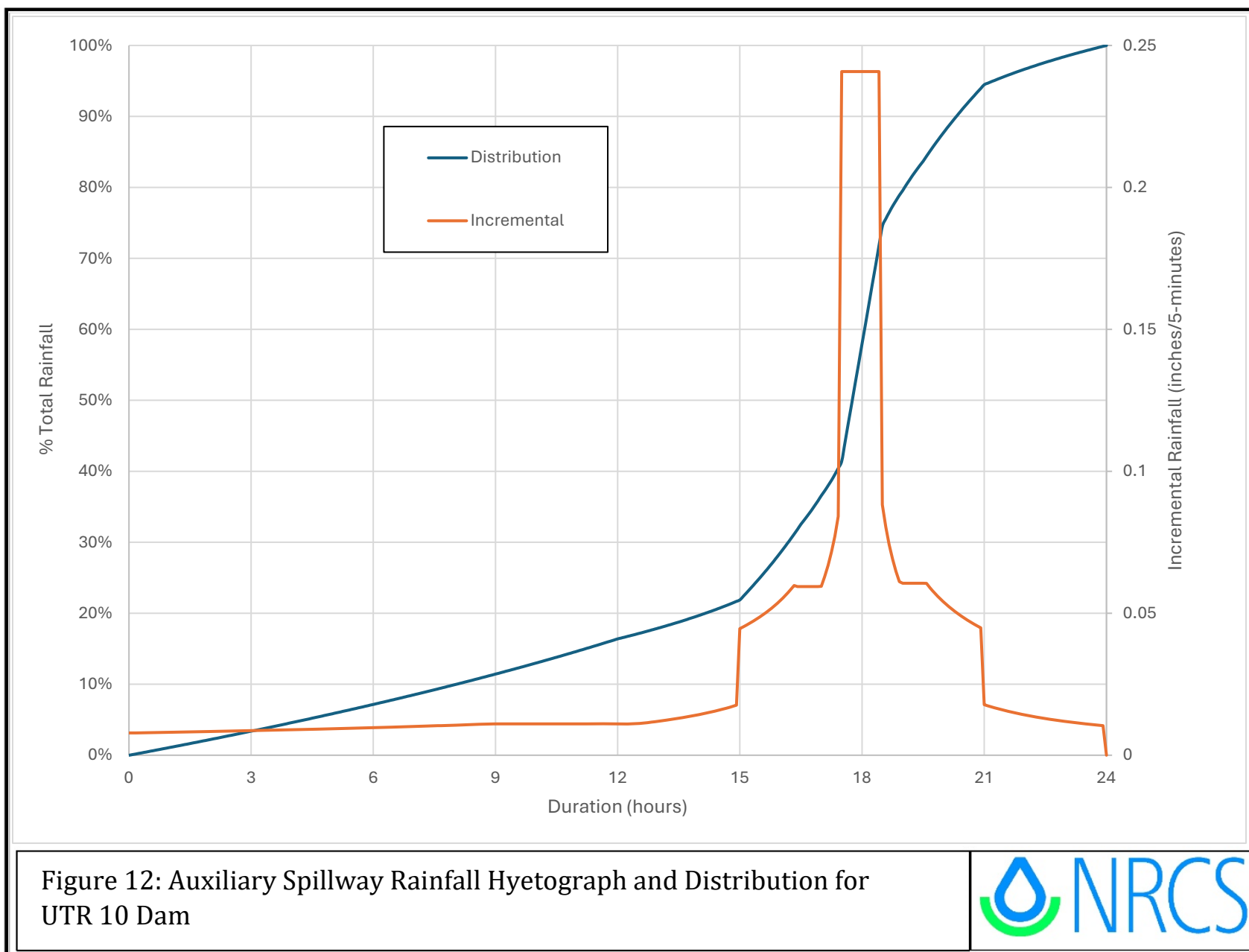


Figure 12: Auxiliary Spillway Rainfall Hyetograph and Distribution for UTR 10 Dam



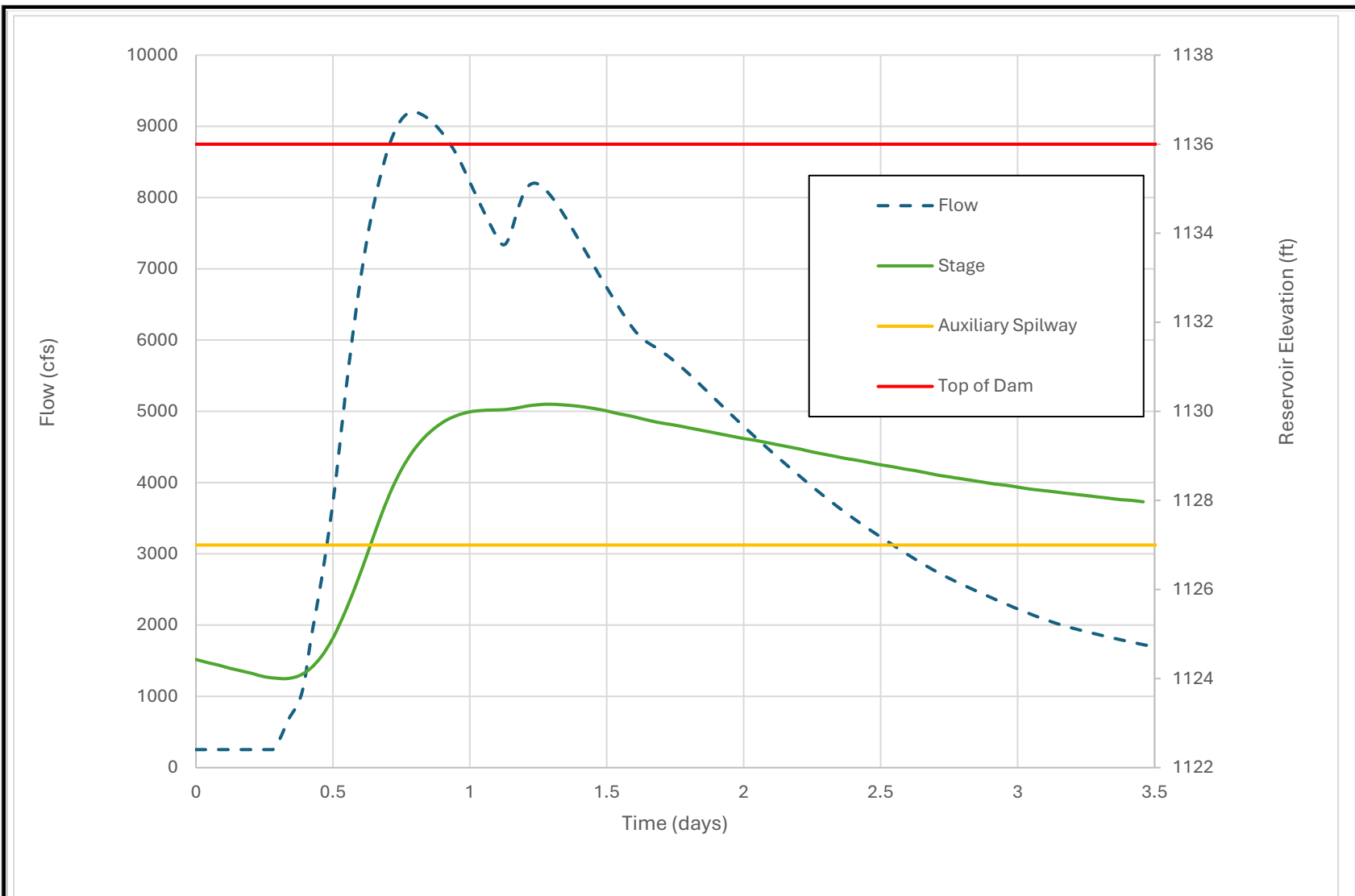
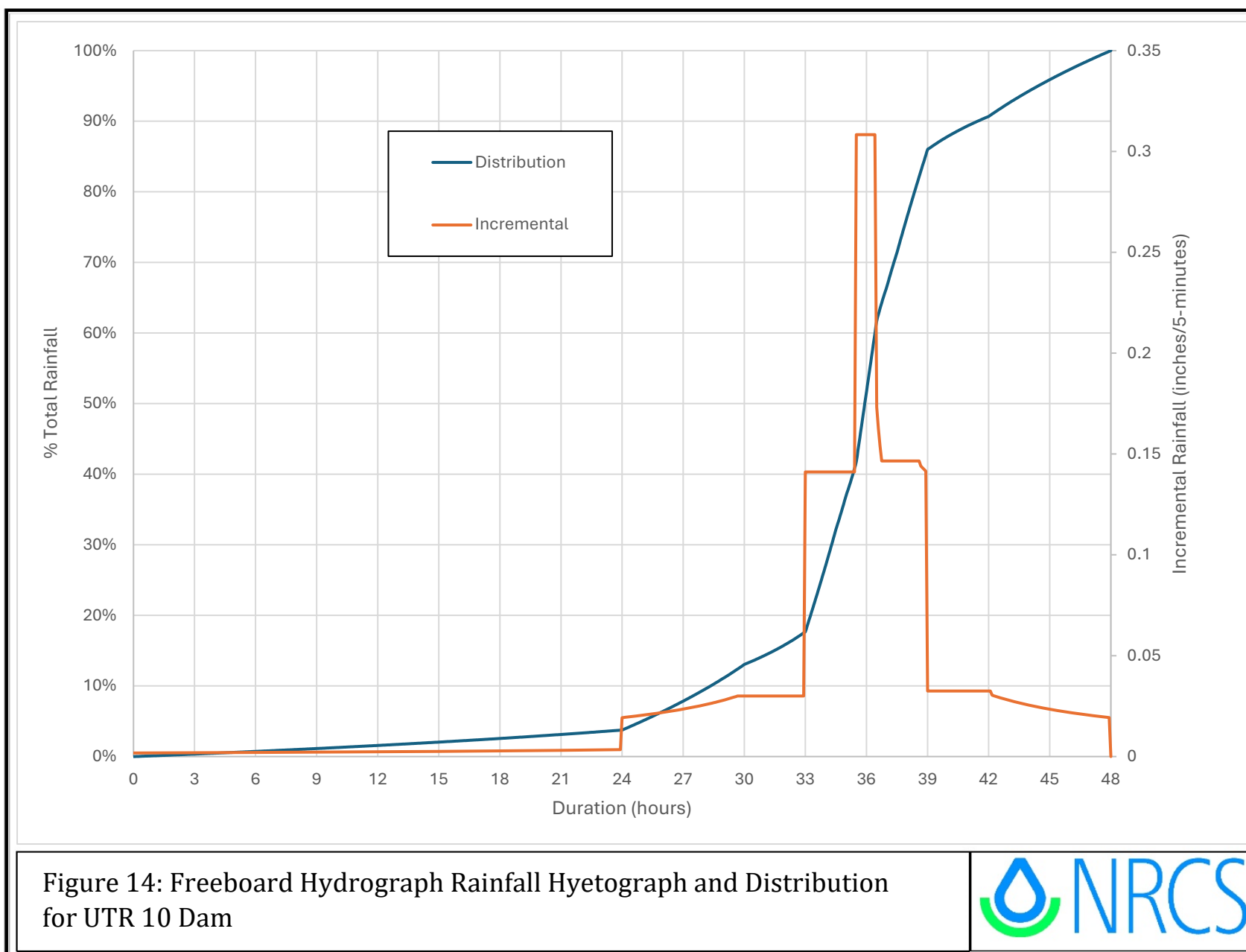


Figure 13: Auxiliary Spillway Hydrograph and Stage for UTR 10 Dam





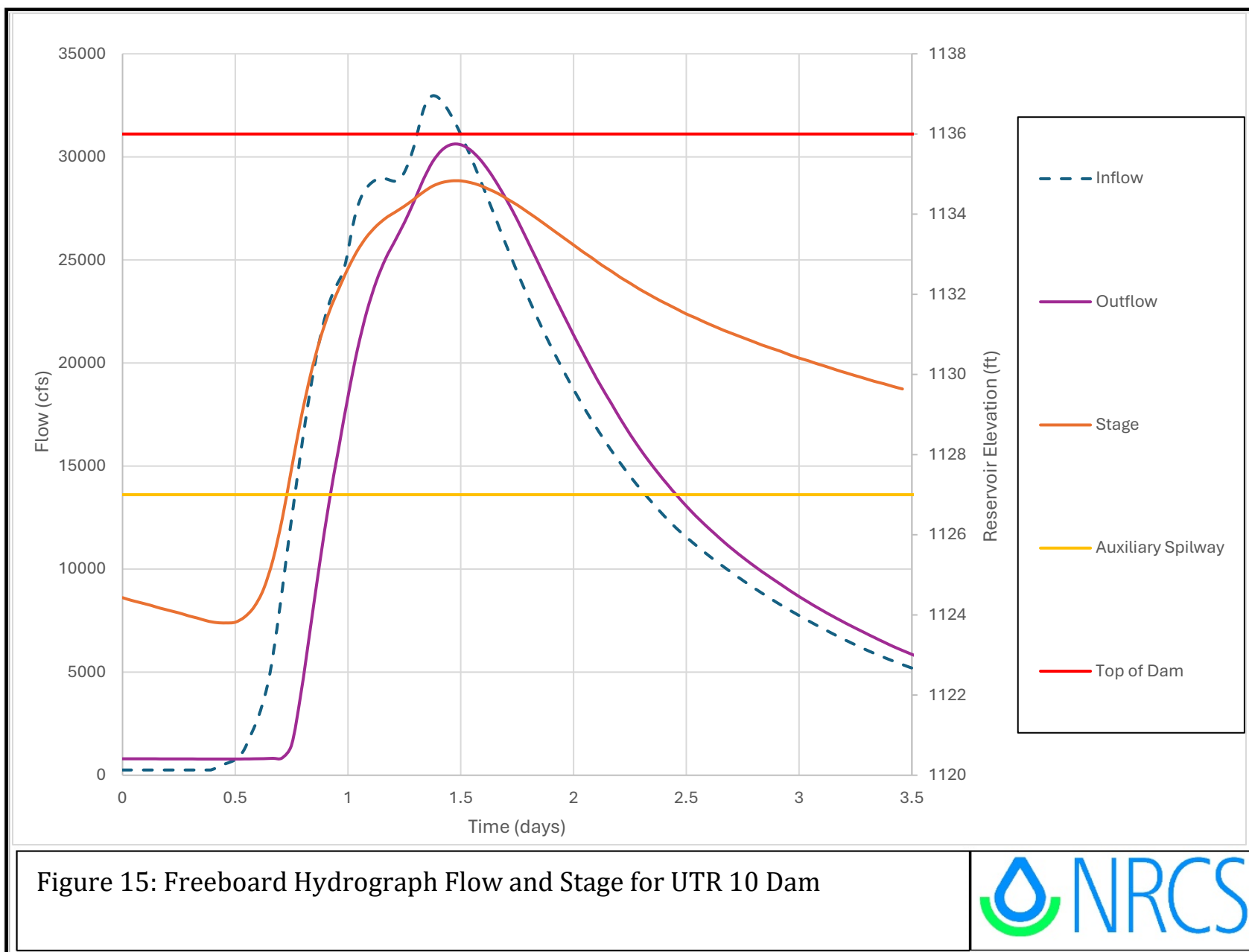


Figure 15: Freeboard Hydrograph Flow and Stage for UTR 10 Dam

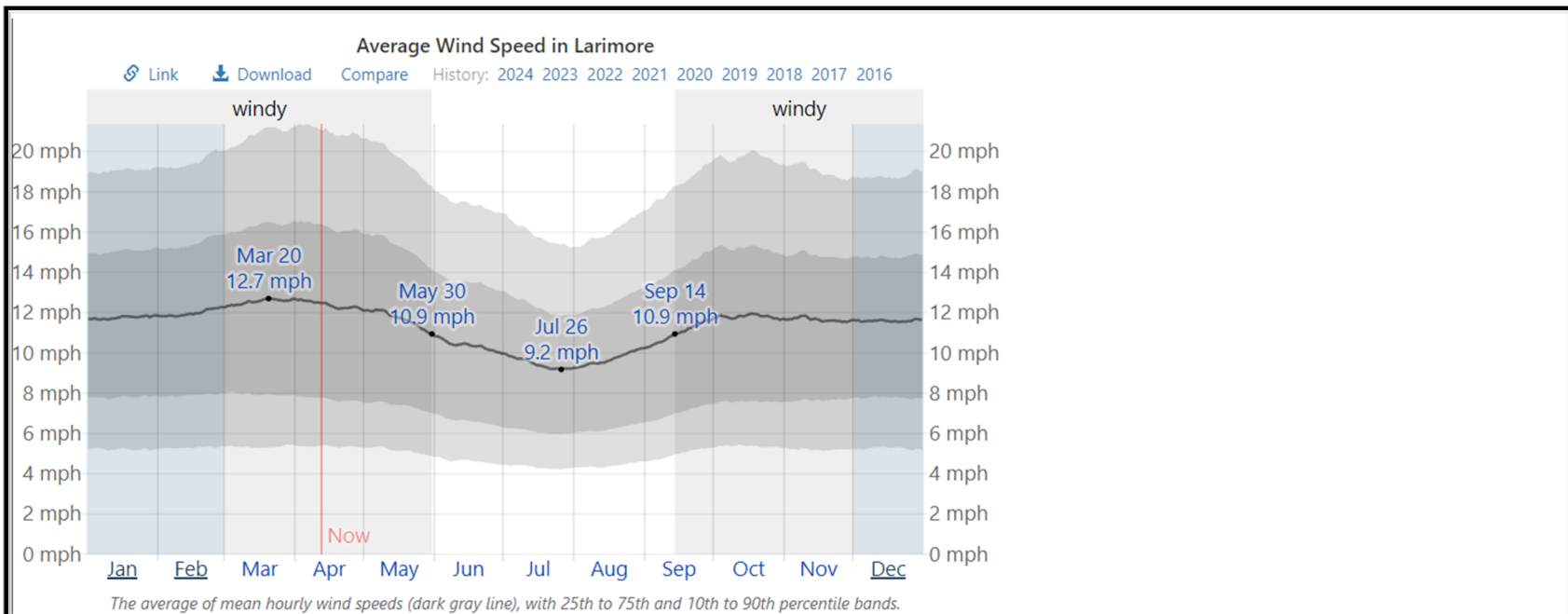


Figure 16: Residual Freeboard Wind and Wave Height Parameters for UTR 10 Dam



Figure 17: Standard Freeboard Wind and Wave Height Parameters for UTR 10 Dam



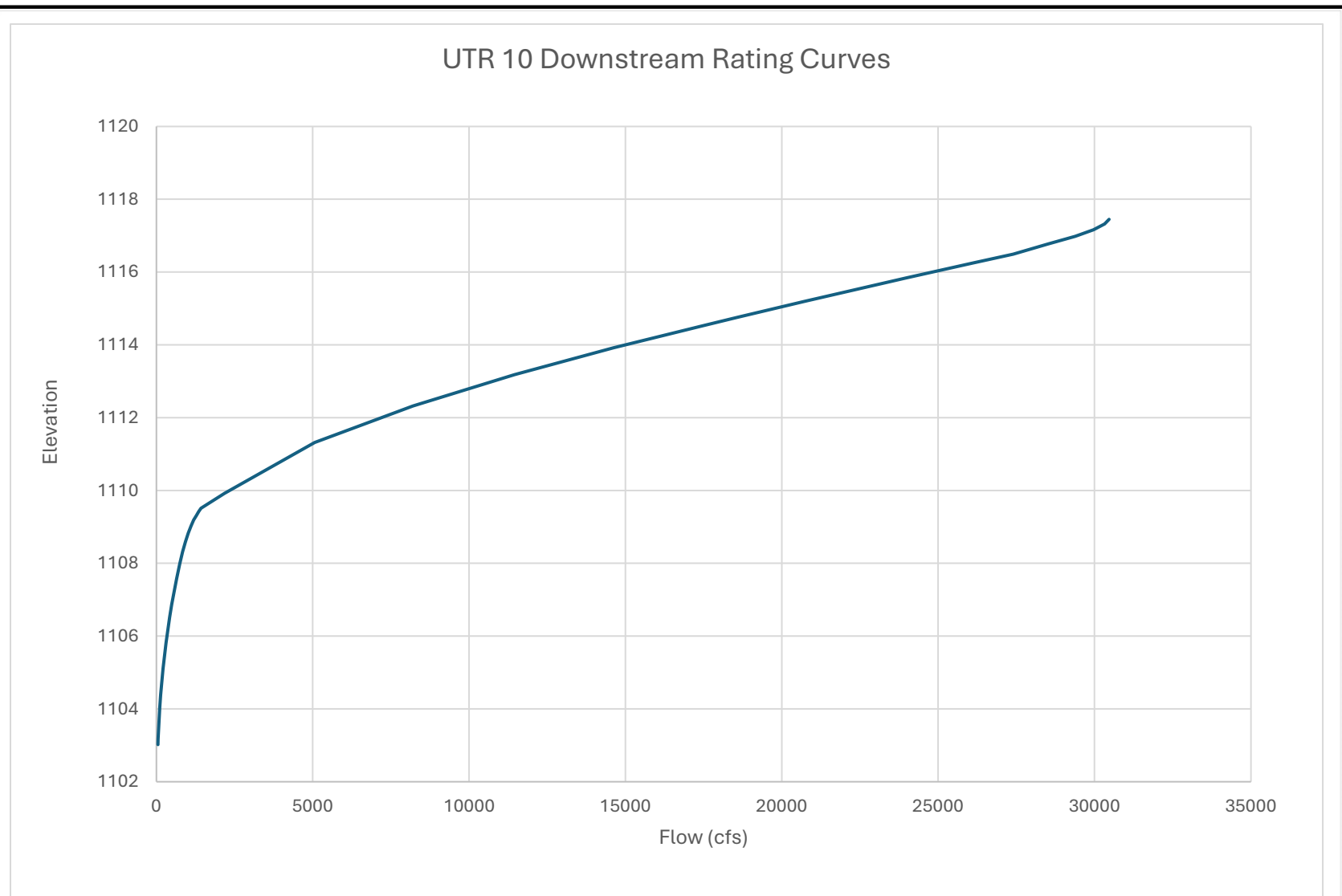


Figure 18: UTR 10 Dam Downstream Rating Curve



Assumptions:			Notes:
Horizontal aspect of Slope:	3	H:1V	Typical are 2.5 - 4.
θ	0.32	rad	
θ	18.44	deg	
B (width)	360	feet	Spillway capacity and 1' residual freeboard to TOD
Hcrest (AS: 1127, floodplain~1104)	26	feet	Basin elev from stream geometry, AS from PSH elev.
AS Q (pmf)	29,750	cfs	Routing reservoir PMF through PS and AS in RAS
Initial Hydraulic Calcs:			
q (unit discharge)	82.6	cfs/ft	Std hydraulic calc (unit discharge is Q/w)
dc (critical depth)	6.0	feet	Std hydraulic calc (Critical depth)
L (Chute Length)	82.2	feet	Based on geometry
yn (Supercritical)	1.14	feet	Change until cell B15 matches B7. Go until Check in D15 is "yes".
Velocity	72.2	ft/s	Check ↓ Mannings Equation
Flow Check	29,636	cfs	yes Within 2% of AS Q (pmf)
Baffle Design Calcs:			
H (baffle pier height)	4.8	feet	USBR 1987, page 360. Assume $0.8 \cdot D_c$
W & S_h (width and horizontal spacing)	7.2	feet	USBR 1987, page 360. Assume $1.5 \cdot H$
S_{LS} (Sloped Longitudinal spacing)	14.3	feet	USBR 1987, page 360. Assume $H \cdot \text{slope}$ is max
S_{LH} (Horizontal Longitudinal spacing)	13.6	feet	Geometry
R (Rows)	5		USBR 1987, page 360. L minus ogee length/ S_L , Min 4 rows
D_T (Depth, top)	1.0	feet	USBR 1987, page 361. Assume $0.2 \cdot H$
D_B (Depth, Bottom)	3.3	feet	USBR 1987, page 361. Assume $H/2 + 0.2 \cdot H$
Wall Height	14.3	feet	USBR 1987, page 361 & 362. Assume $3 \cdot H$
Calculations by:	Jon Petersen, NRCS		
Checked by:	Tom Schanandore, NRCS		
Date	7/9/2024		

Figure 19: UTR 10 Dam Auxiliary Spillway Baffle Design Calculations



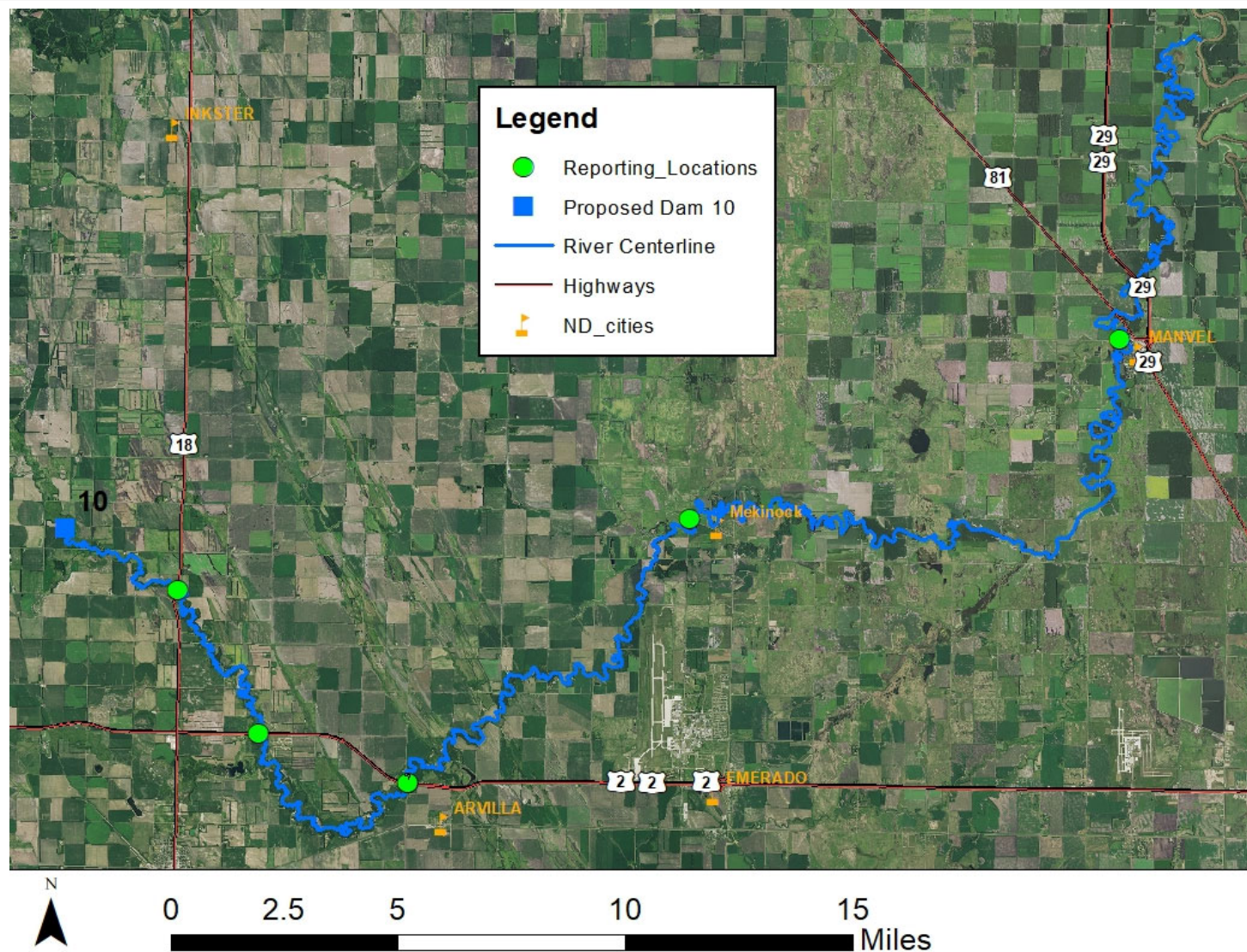
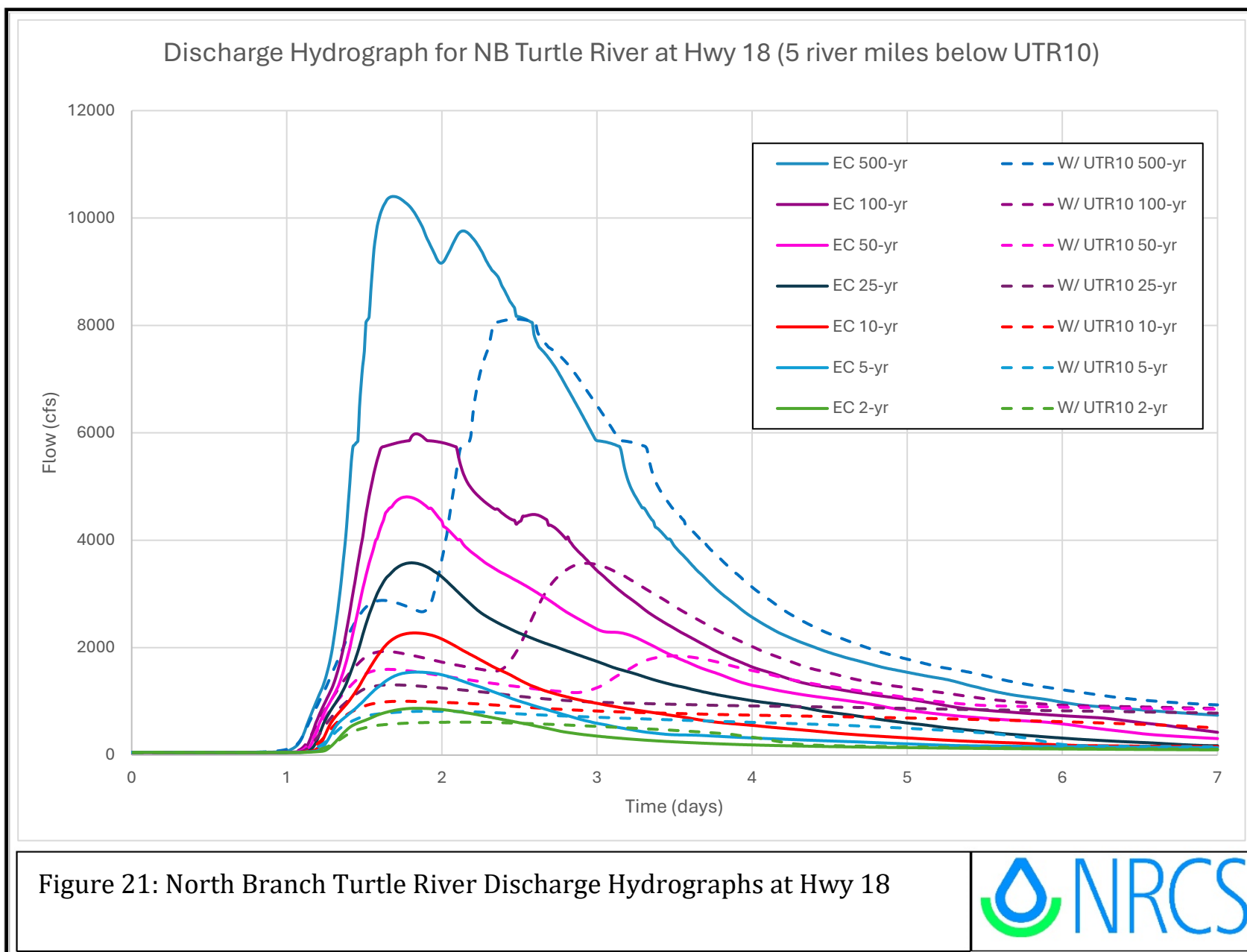
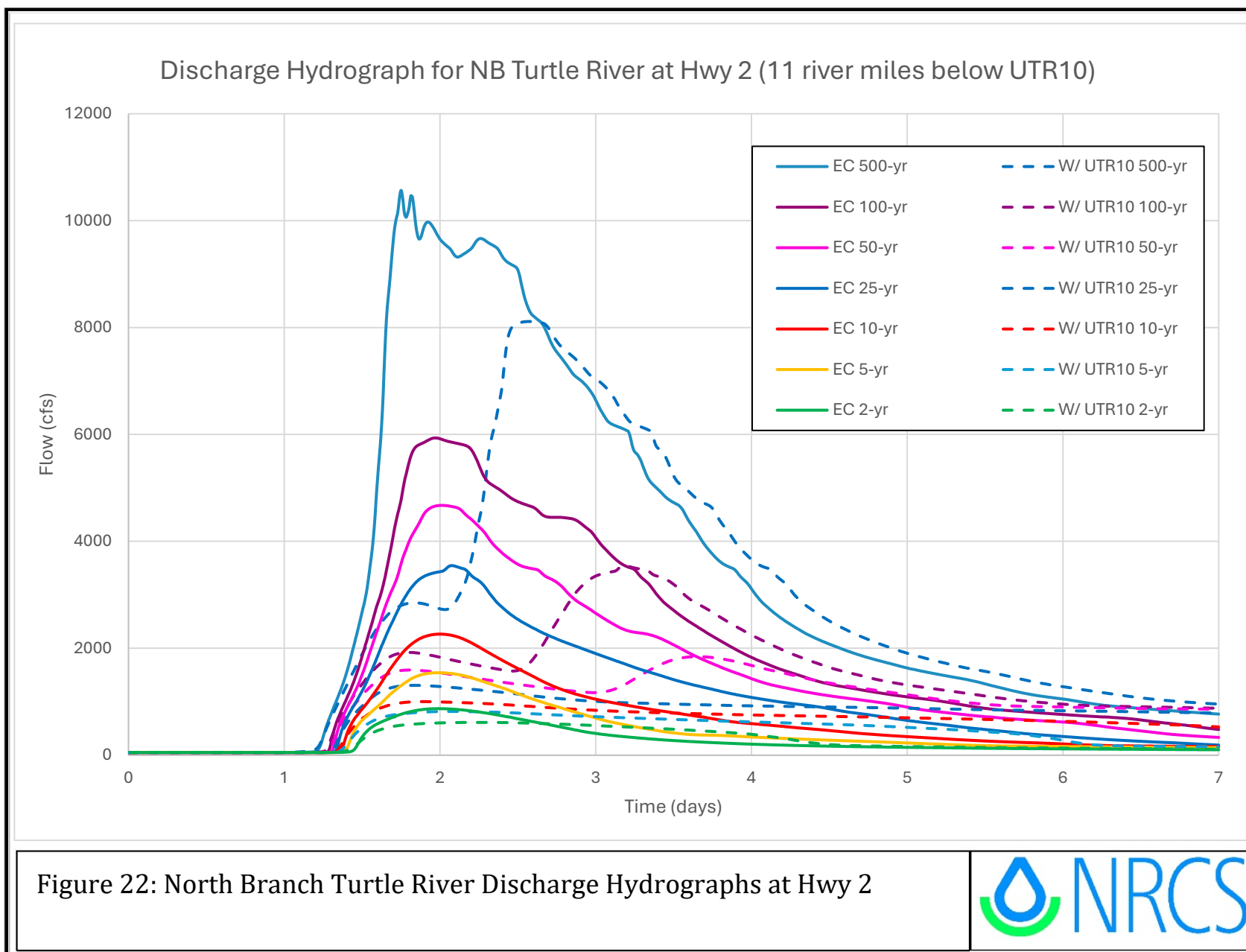
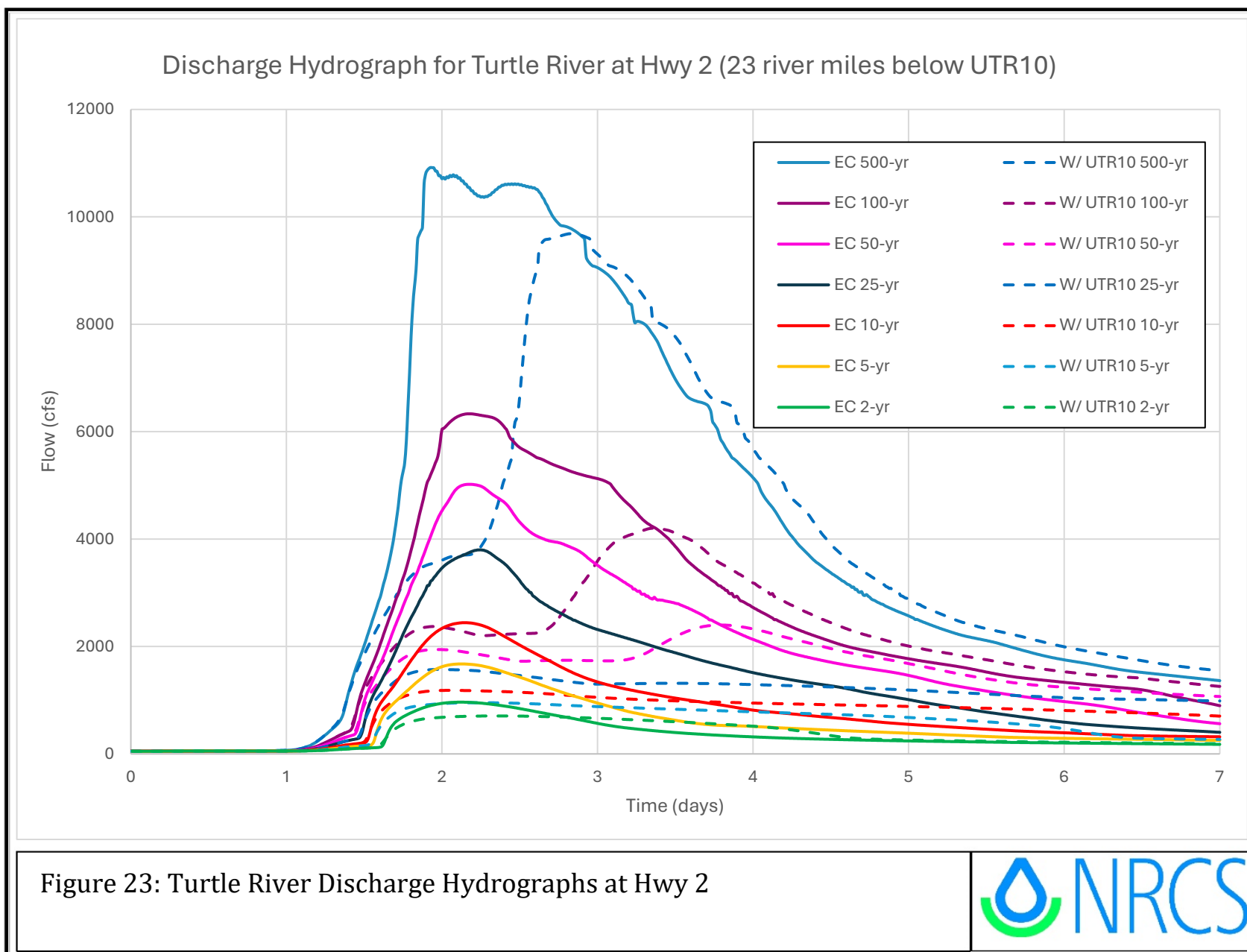


Figure 20: UTR Dam 10 Performance Locations for Synthetic Events







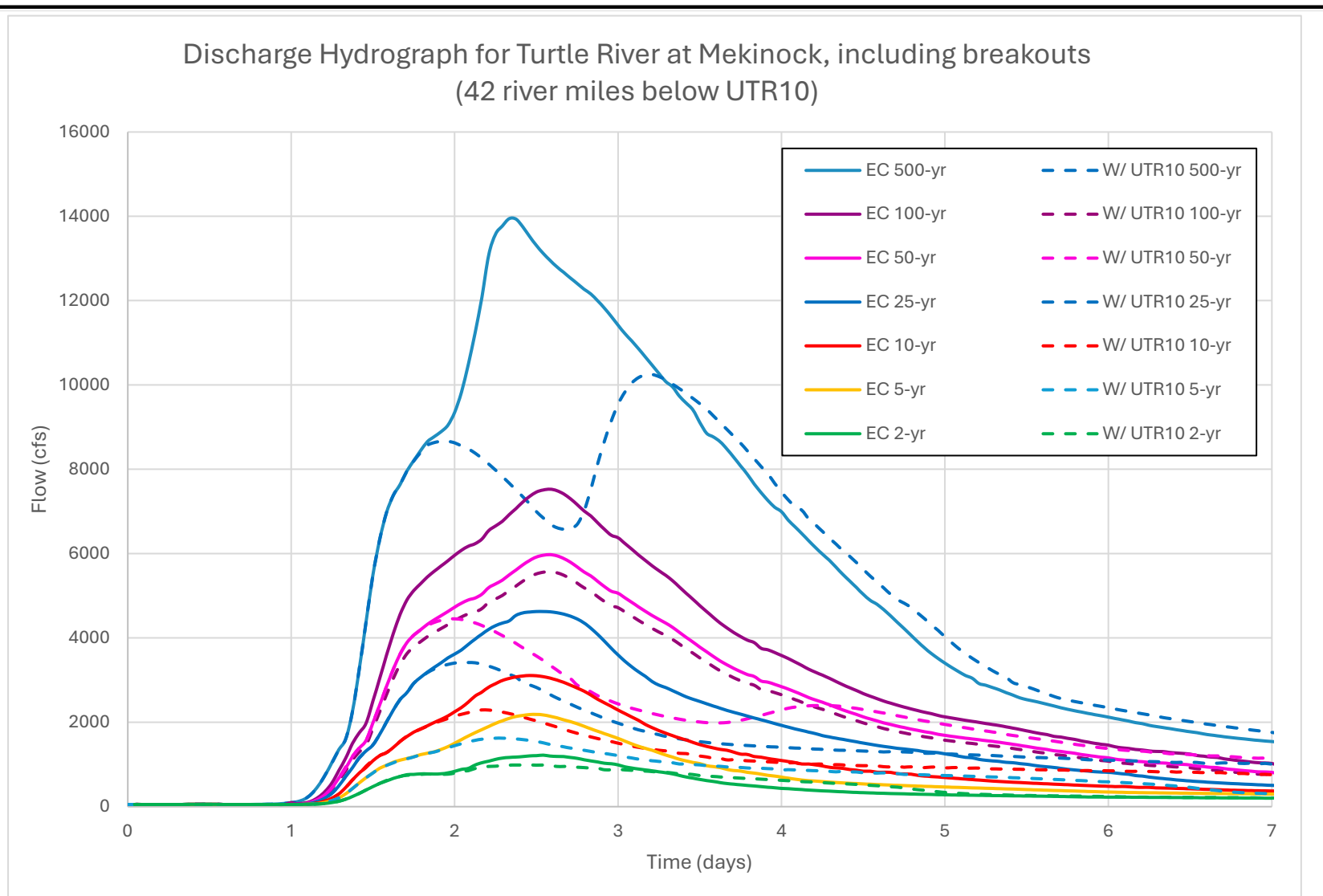


Figure 24: Turtle River Discharge Hydrographs at Mekinock



Discharge Hydrograph for Turtle River at Manvel, including breakouts
(72 river miles below UTR10)

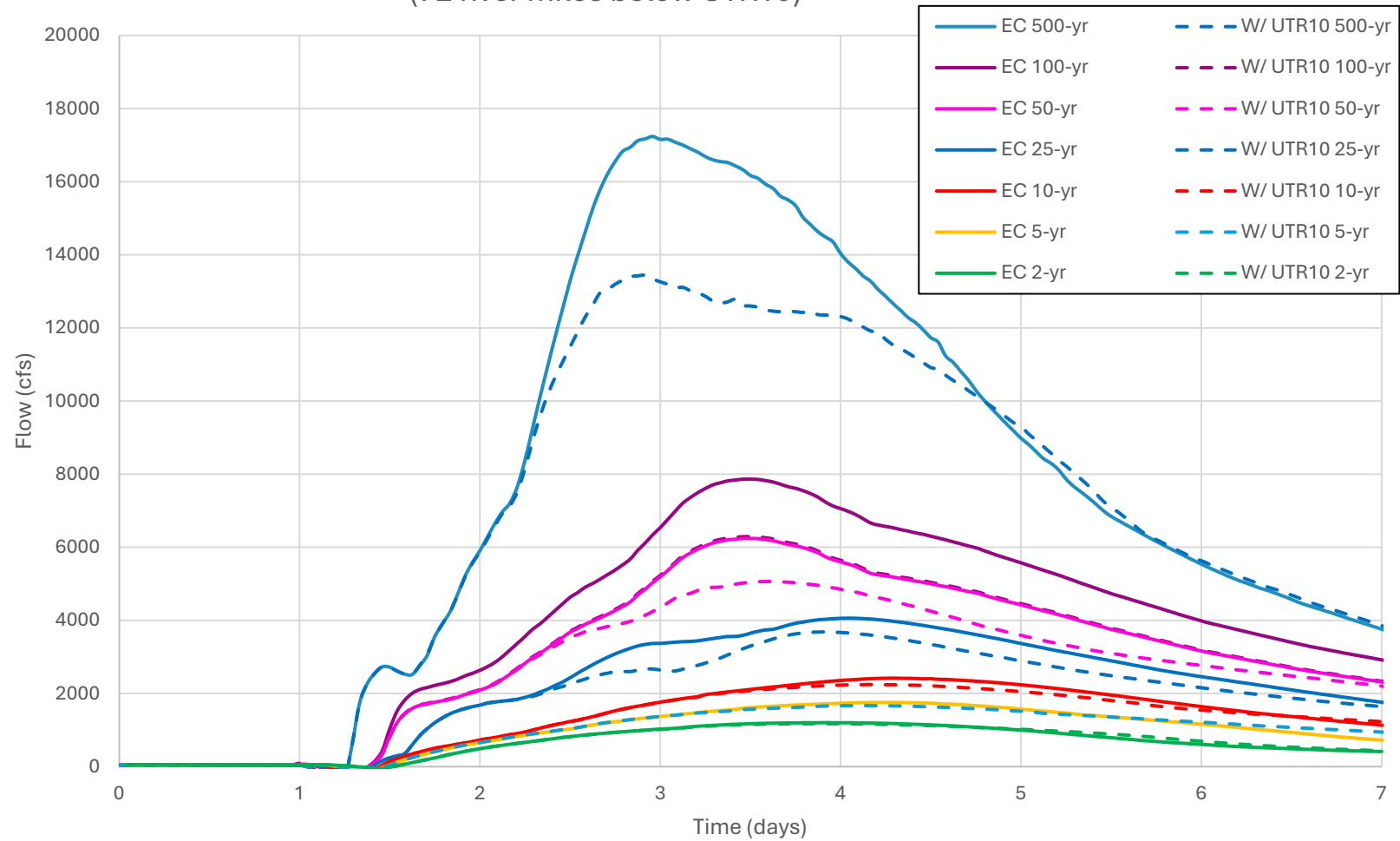


Figure 25: Turtle River Discharge Hydrographs at Manvel



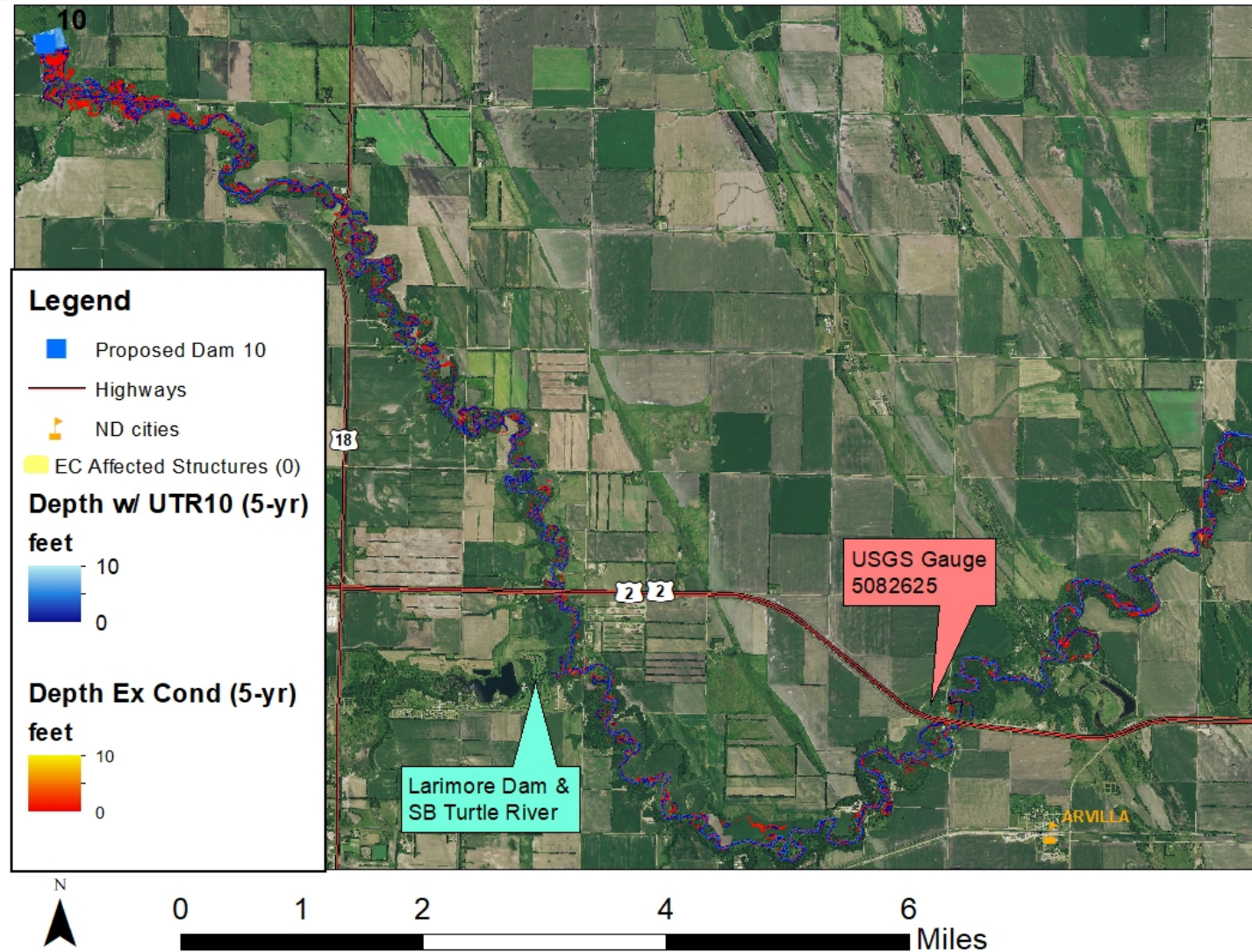


Figure 26: North Branch Turtle River & Turtle River 5-year, 4-day Rainfall Inundation Depths

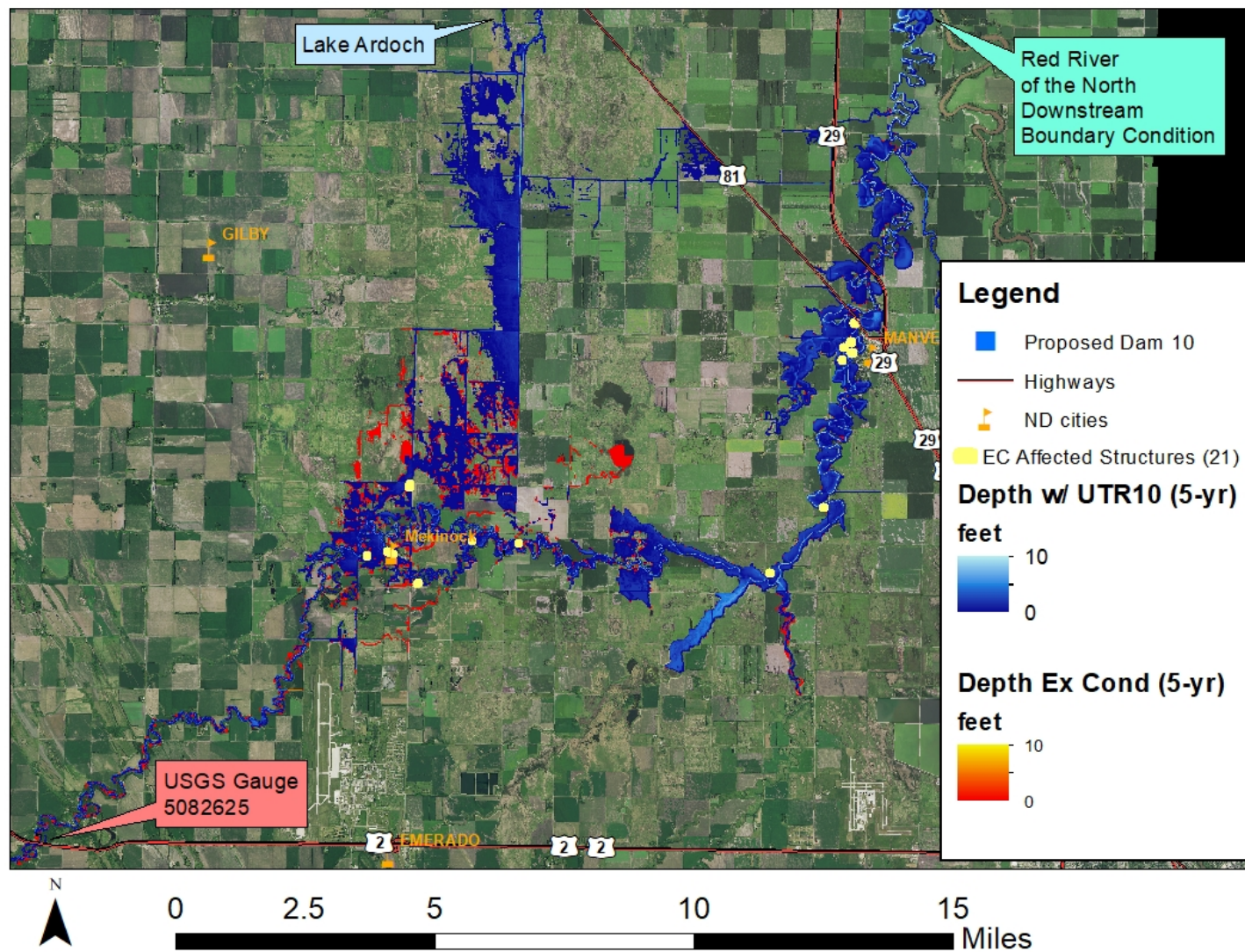


Figure 27: Turtle River 5-year, 4-day Rainfall Inundation Depths

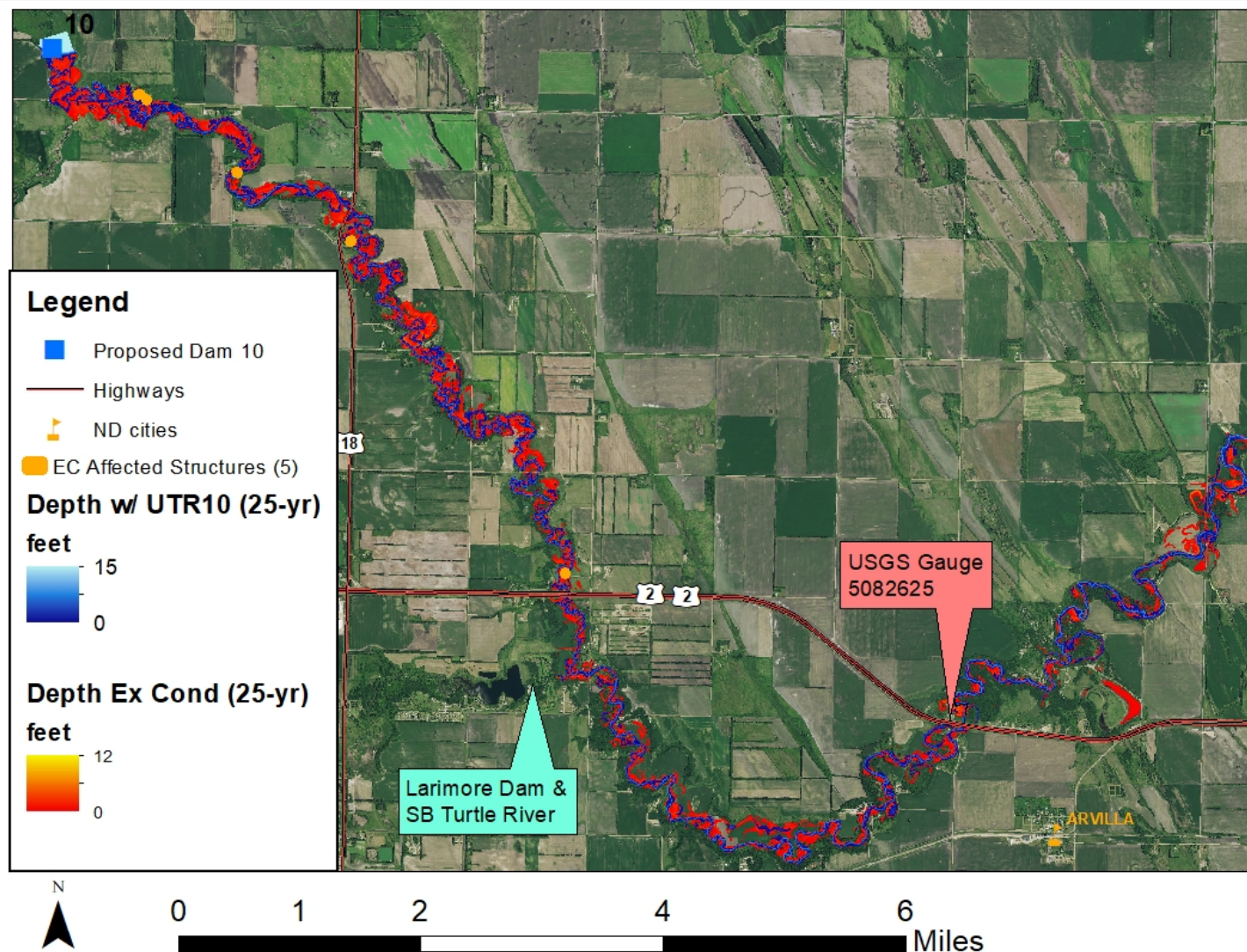


Figure 28: North Branch Turtle River & Turtle River 25-year, 4-day Rainfall Inundation Depths

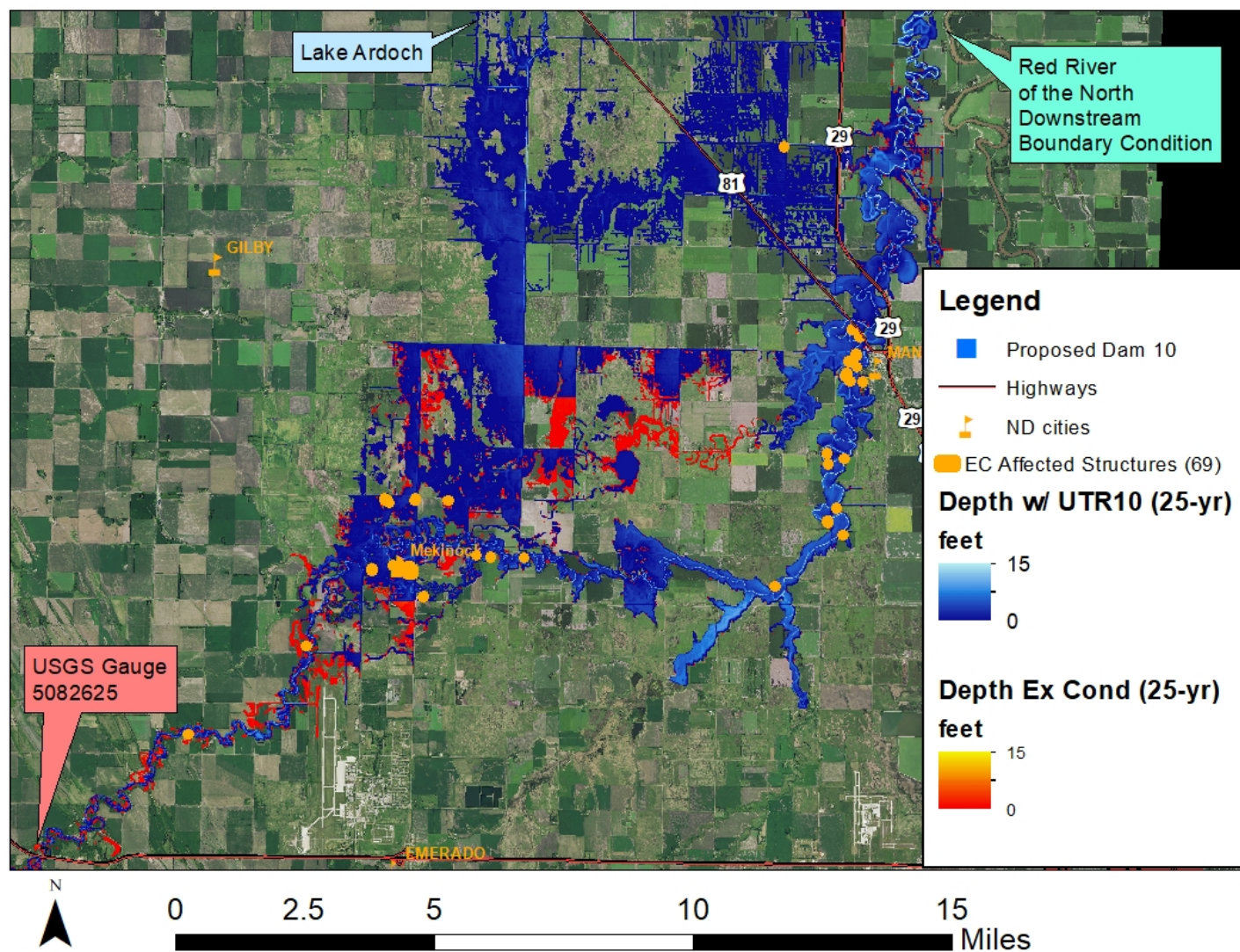


Figure 29: Turtle River 25-year, 4-day Rainfall Inundation Depths

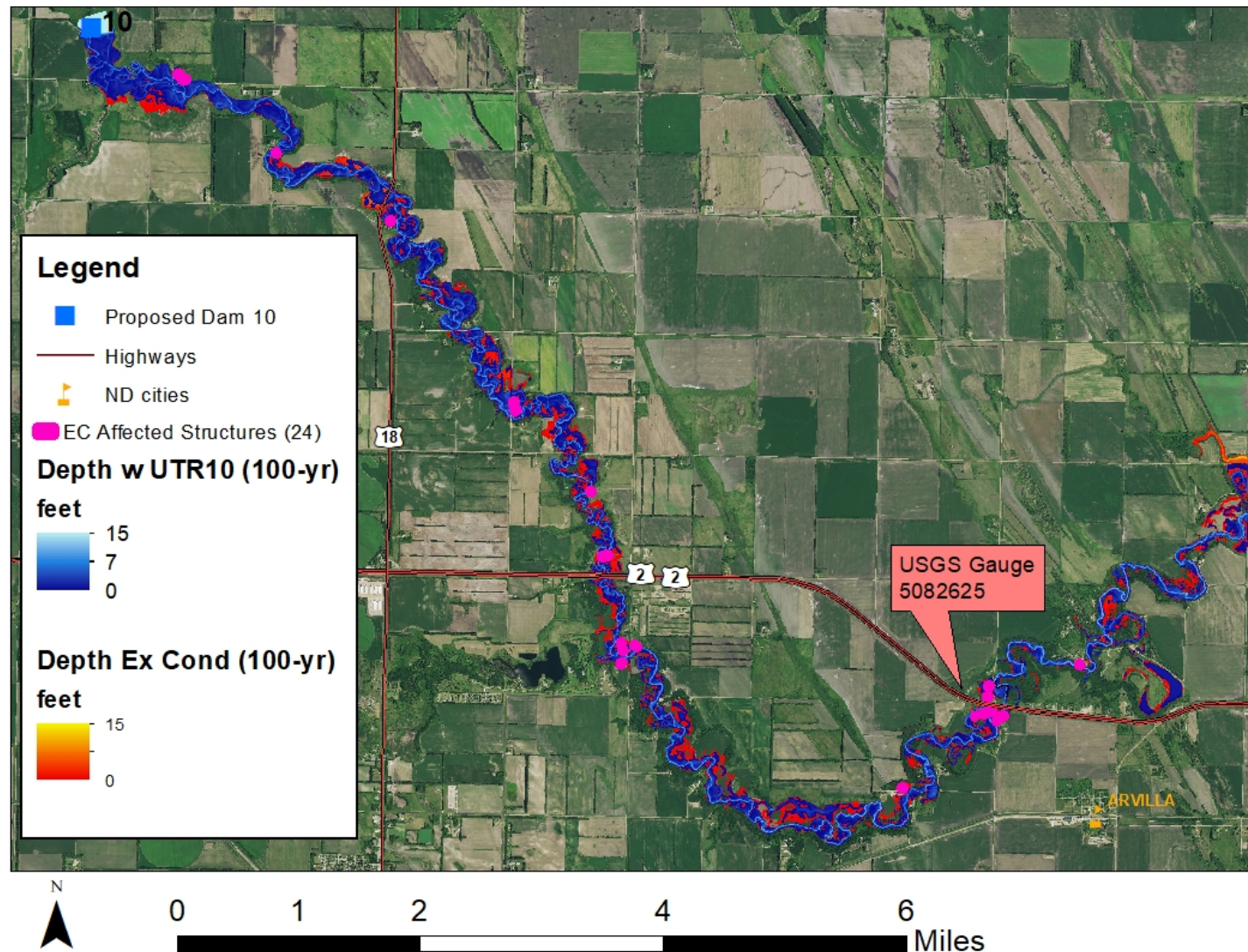
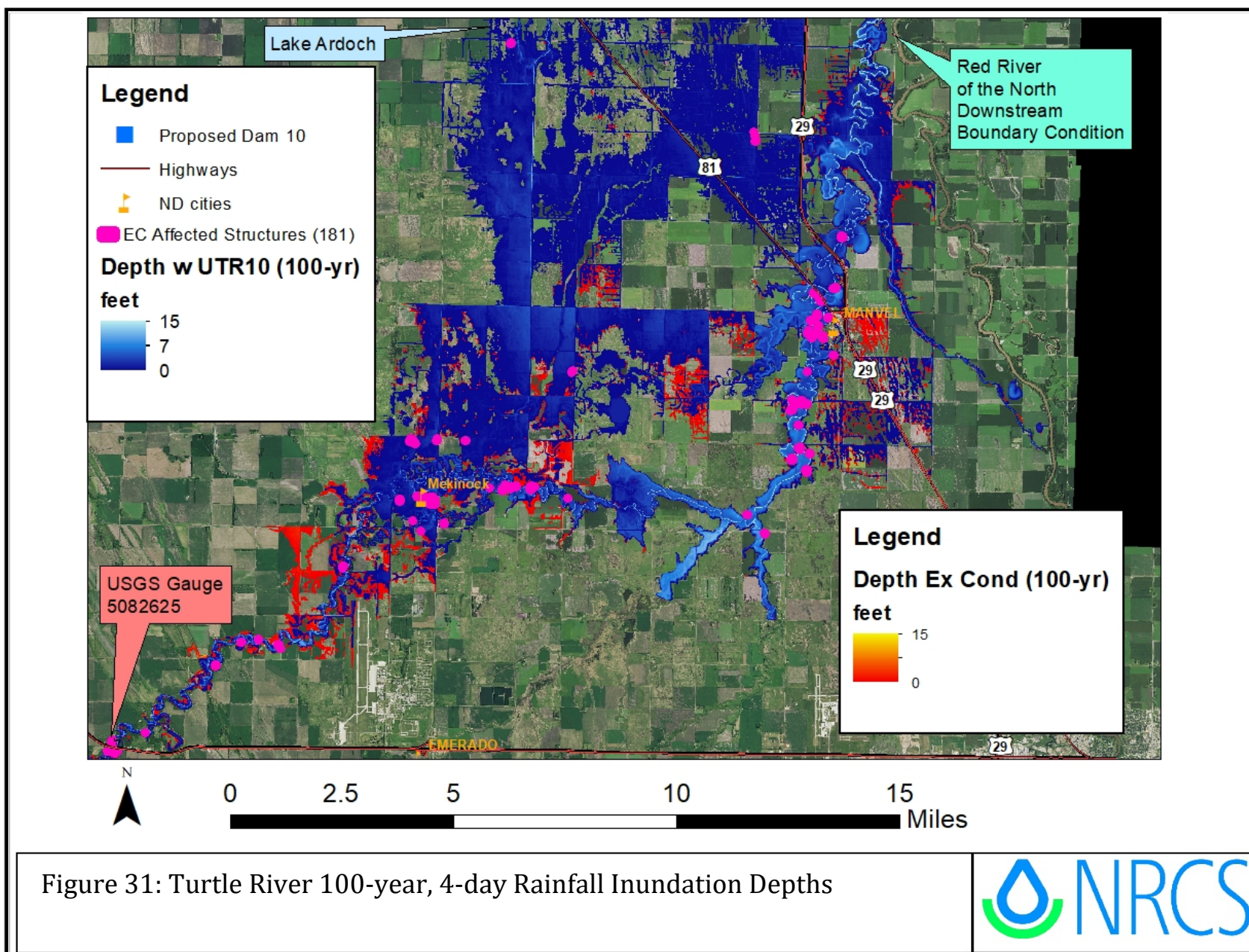
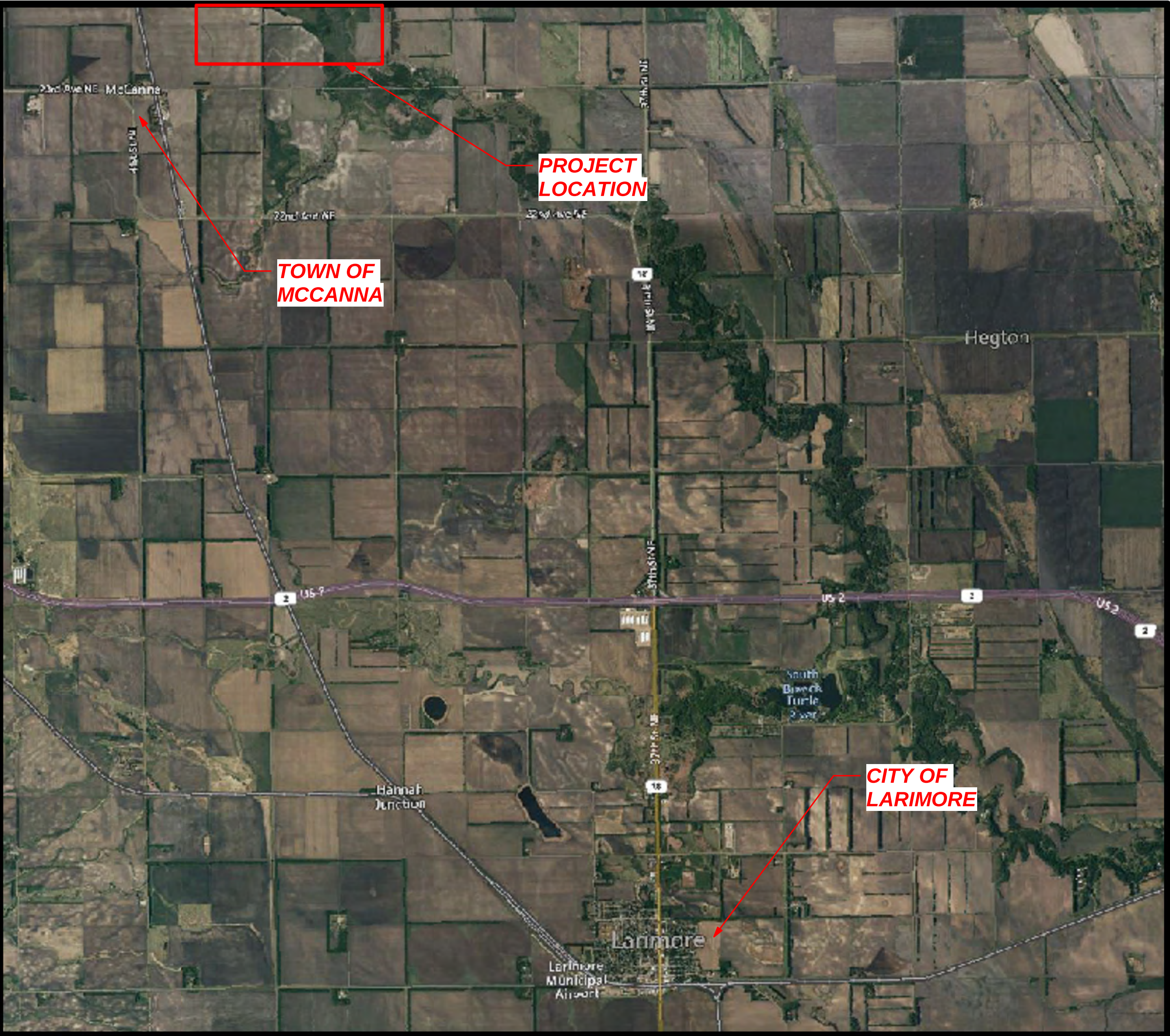


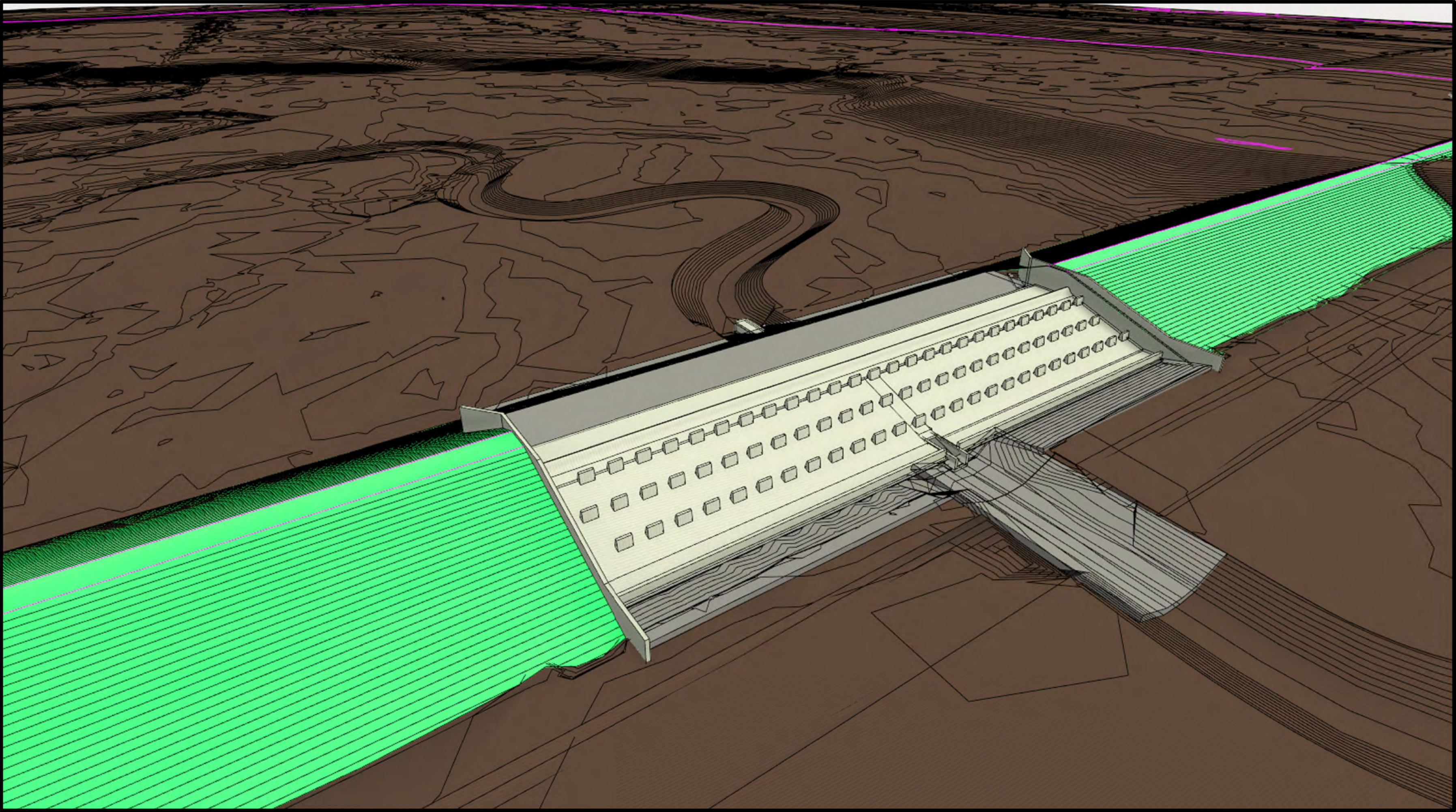
Figure 30: North Branch Turtle River & Turtle River 100-year, 4-day Rainfall Inundation Depths





PROJECT LOCATION MAP
(NOT TO SCALE)

Sec 3 and 4 T152N R55W



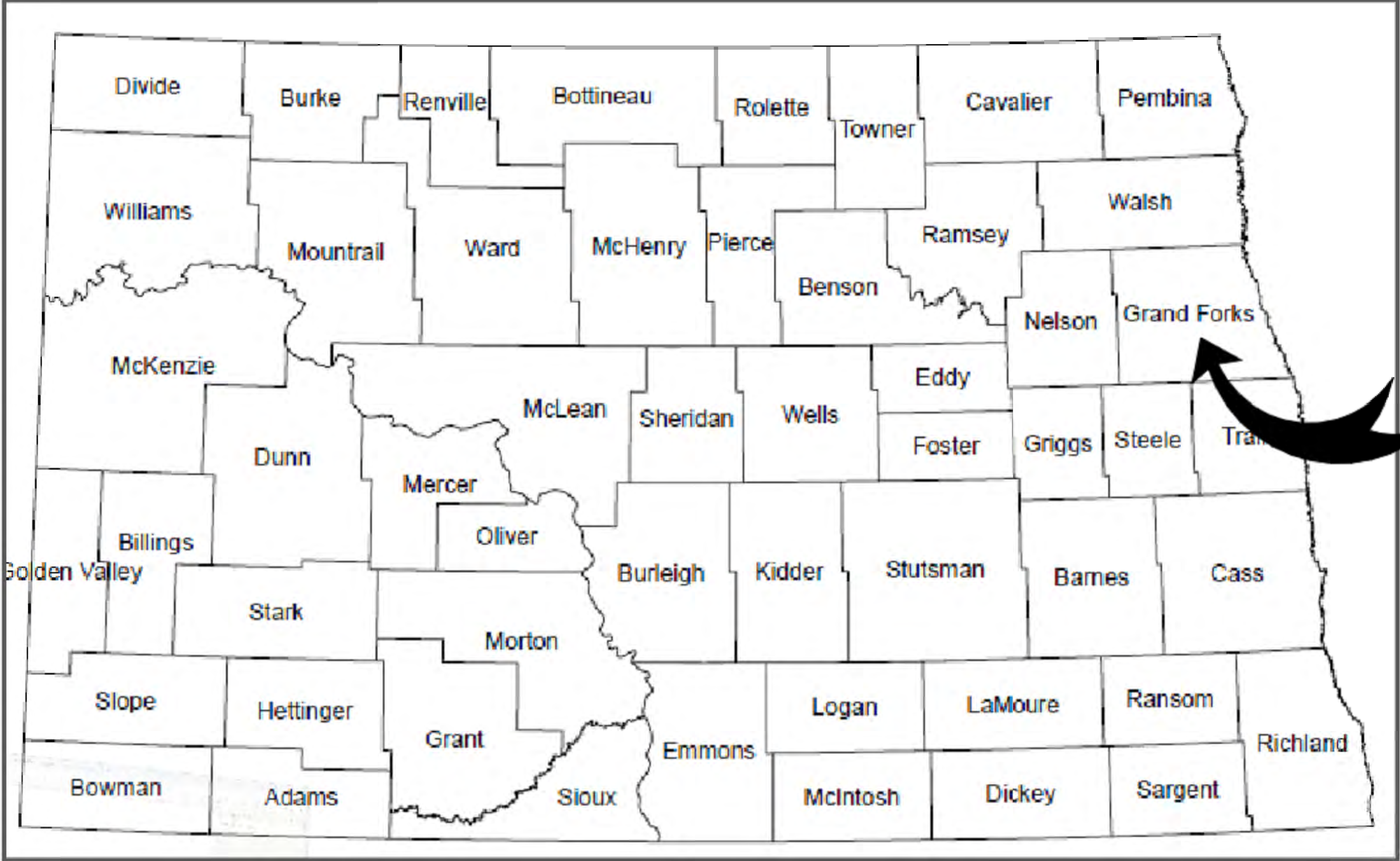
CONCEPT DESIGN DRAWINGS FOR UPPER TURTLE DAM #10
PREPARED FOR GRAND FORKS COUNTY WATER
RESOURCE DISTRICT IN GRAND FORKS COUNTY, ND

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

ESTIMATED PROJECT QUANTITIES		
CONSTRUCTION ITEM	UNIT	QUANTITY
CLEARING AND GRUBBING	ACRE	15.4
RIP-RAP	CY	4,175
EARTH FILL	CY	137,600
EARTH CUT	CY	45,700
DRAIN MATERIAL FILL	CY	24,905
DRAIN PIPE AND BEDDING	LF	5,000
CONCRETE WALLS	CY	503
CONCRETE FOOTINGS	CY	438
CONCRETE CONDUIT (ON-GRADE SPILLWAY)	CY	280
STRUCTURAL SPILLWAY SLAB	CY	1,496
CONCRETE BAFFLE STRUCTURES	CY	187
OGEE WEIR STRUCTURE	CY	405
SOIL/BENTONITE SLURRY CUTOFF WALL	CY	8,313
RELIEF WELL STRUCTURES	ACRE	65
EROSION CONTROL AND RE-VEGETATION	ACRE	16.3

JOB APPROVAL AUTHORITY		
PRACTICE NAME	CODE	JOB CLASS
DAM	402	VII
ACCESS ROAD	560	V
OPEN CHANNEL	582	III

INDEX OF DRAWING SHEETS	
SHEET #	SHEET TITLE
SHT0	COVER SHEET
SHT1	DAM SITE OVERALL PLAN
SHT2	EXISTING SOIL BORING SITE PLAN
SHT3	EMBANKMENT FOUNDATION SOIL PROFILE
SHT4	ROAD IMPROVEMENTS SITE MAP
SHT5	ROAD PROFILES AND CROSS SECTION
SHT6	DAM STRUCTURE AREA A PLAN
SHT7	DAM STRUCTURE AREA B PLAN
SHT8	DAM STRUCTURE AREA C PLAN
SHT9	DAM STRUCTURE AREA D PLAN
SHT10	STRUCTURAL SPILLWAY PLAN
SHT11	STRUCTURAL SECTIONS
SHT12	STRUCTURAL SECTIONS



COVER SHEET

UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS

OWNER: Grand Forks County Water Resource District

COUNTY: Grand Forks



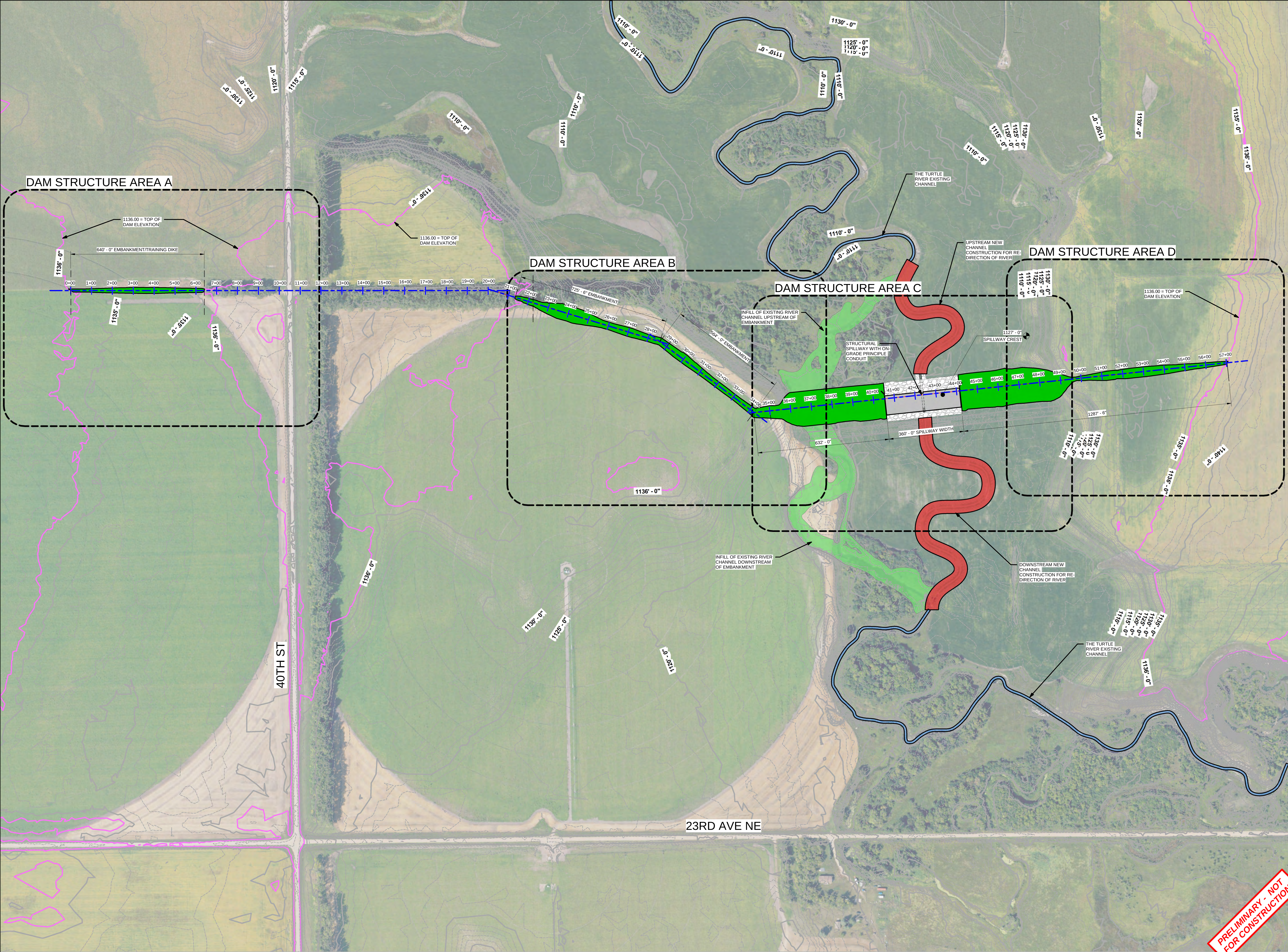
PRELIMINARY - NOT
FOR CONSTRUCTION

Designed: Tom Schandore

Drawn: Tom Schandore

Checked: Kate Staley

Approved: Christi Fisher



PRELIMINARY - NOT
FOR CONSTRUCTION

Designed:	Tom Schandore
Drawn:	Tom Schandore
Checked:	Kate Staley
Approved:	Approver

DAM SITE OVERALL PLAN
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



Date	01/30/2025
	SHT1



Designed:	Designer
Drawn:	Author
Checked:	Checker
Approved:	Approver

EXISTING SOIL BORING SITE PLAN
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS

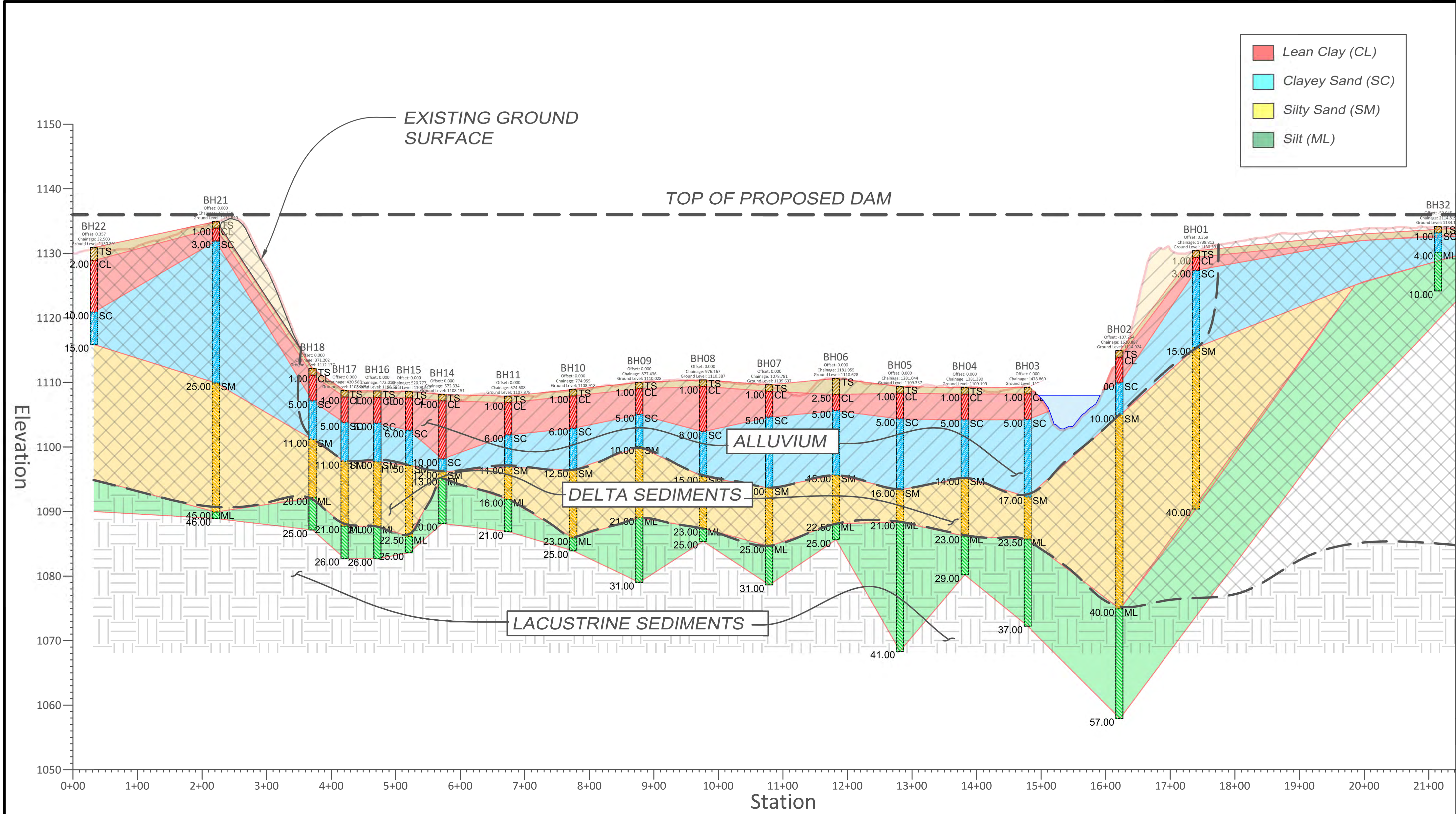
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



Natural Resources Conservation Service

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SHT2	

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DAM CENTERLINE PROFILE VIEW
SCALE: 10V:1H



DAM CENTERLINE SOIL PROFILE VIEW



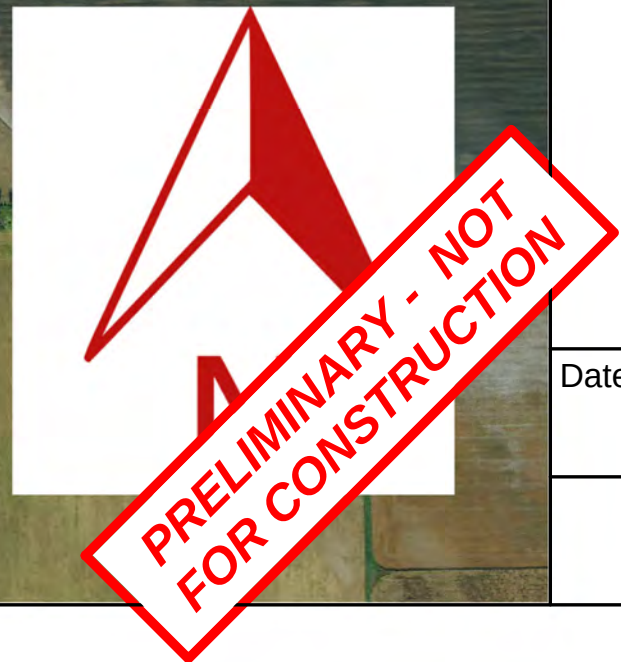
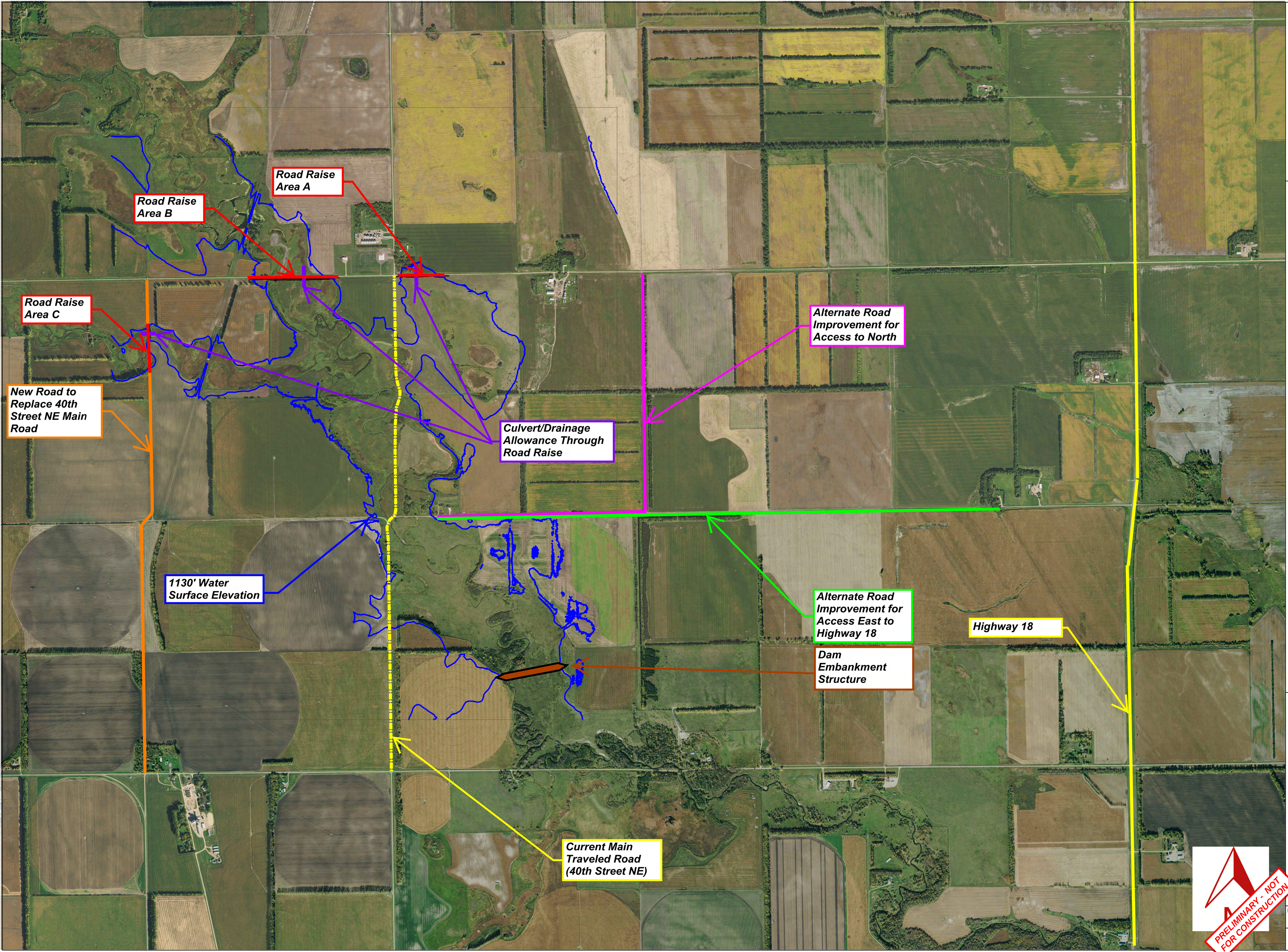
Designed: Designer
Drawn: Author
Checked: Checker
Approved: Approver

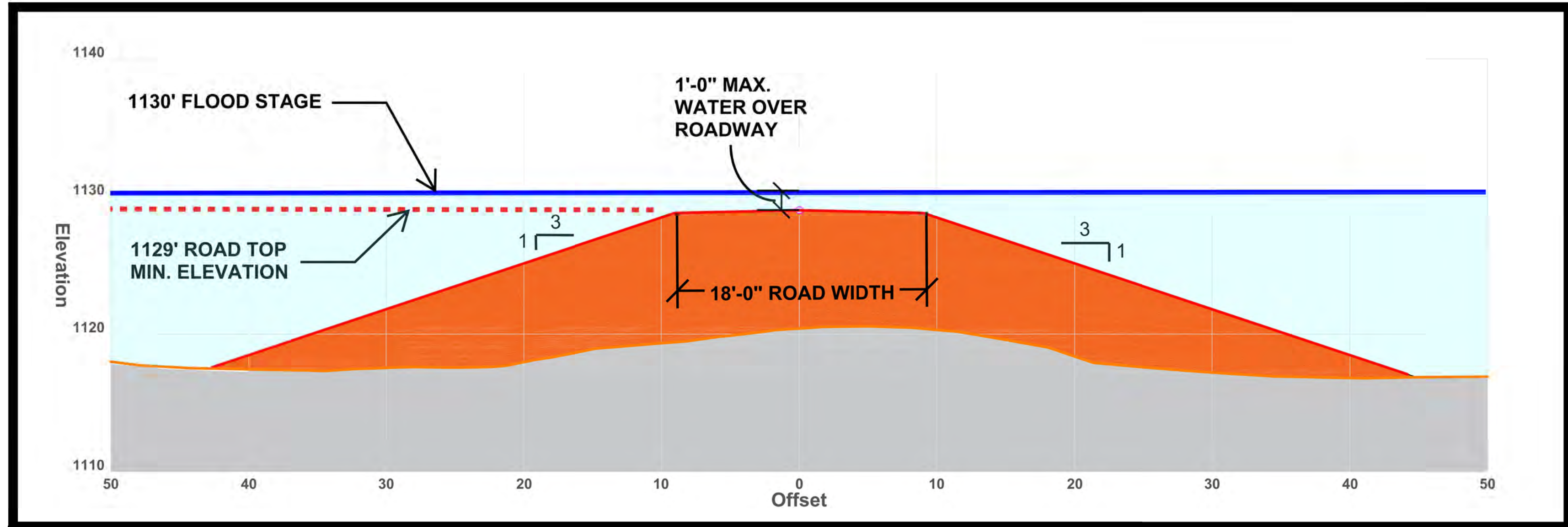
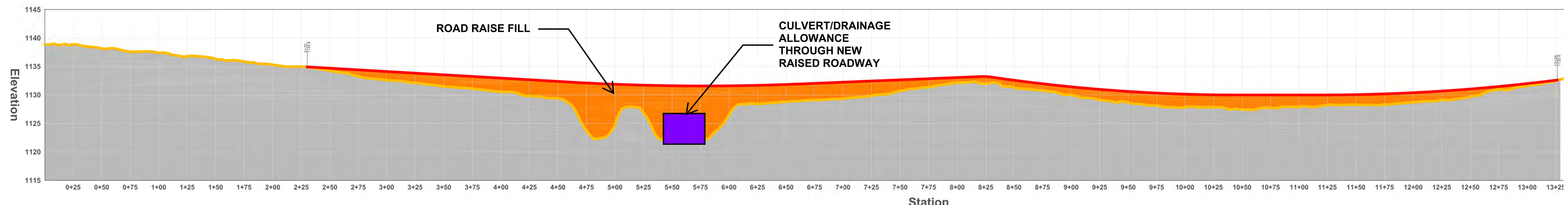
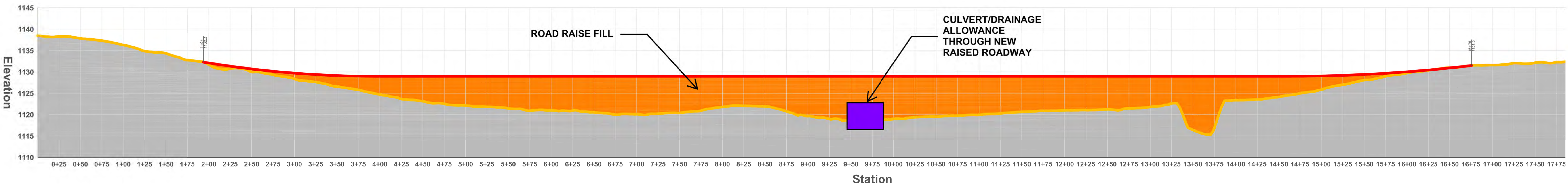
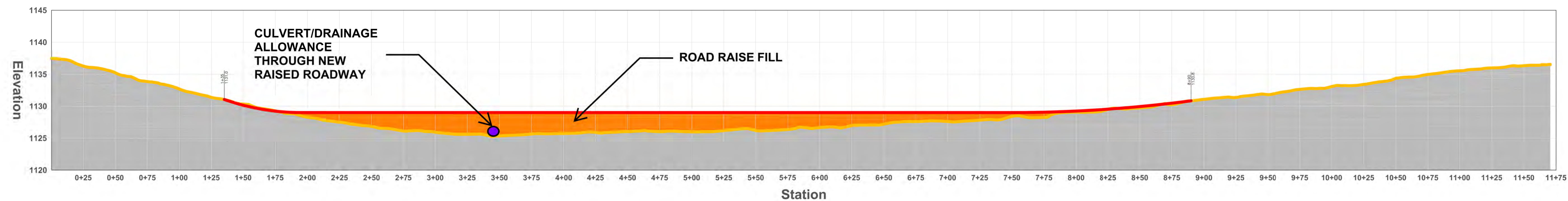
EMBAKMENT FOUNDATION SOIL PROFILE
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



Date	01/31/25
SHT3	

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Typical Road Cross Section

PRELIMINARY - NOT FOR CONSTRUCTION

Designed:	Designer
Drawn:	Author
Checked:	Checker
Approved:	Approver

ROAD PROFILES AND CROSS SECTION
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS

OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



Date	01/31/25
	SHT5



Designed:	Tom Schanandore
Drawn:	Tom Schanandore
Checked:	Kate Staley
Approved:	Christi Fisher

DAM STRUCTURE AREA A PLAN
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS



Natural Resources Conservation Service

Date: 01/30/2025

SHT6

OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



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FOR CONSTRUCTION

Designed:	Tom Schanandre
Drawn:	Tom Schanandre
Checked:	Kate Staley
Approved:	Christi Fisher

DAM STRUCTURE AREA B PLAN
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



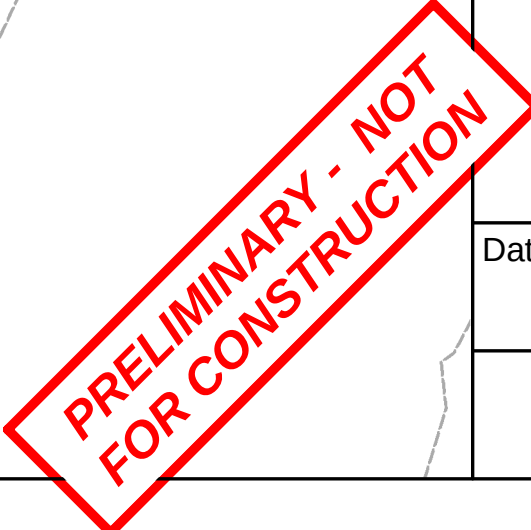


PRELIMINARY - NOT
FOR CONSTRUCTION

Designed:	Tom Schandore
Drawn:	Tom Schandore
Checked:	Kate Staley
Approved:	Christi Fisher

DAM STRUCTURE AREA D PLAN
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



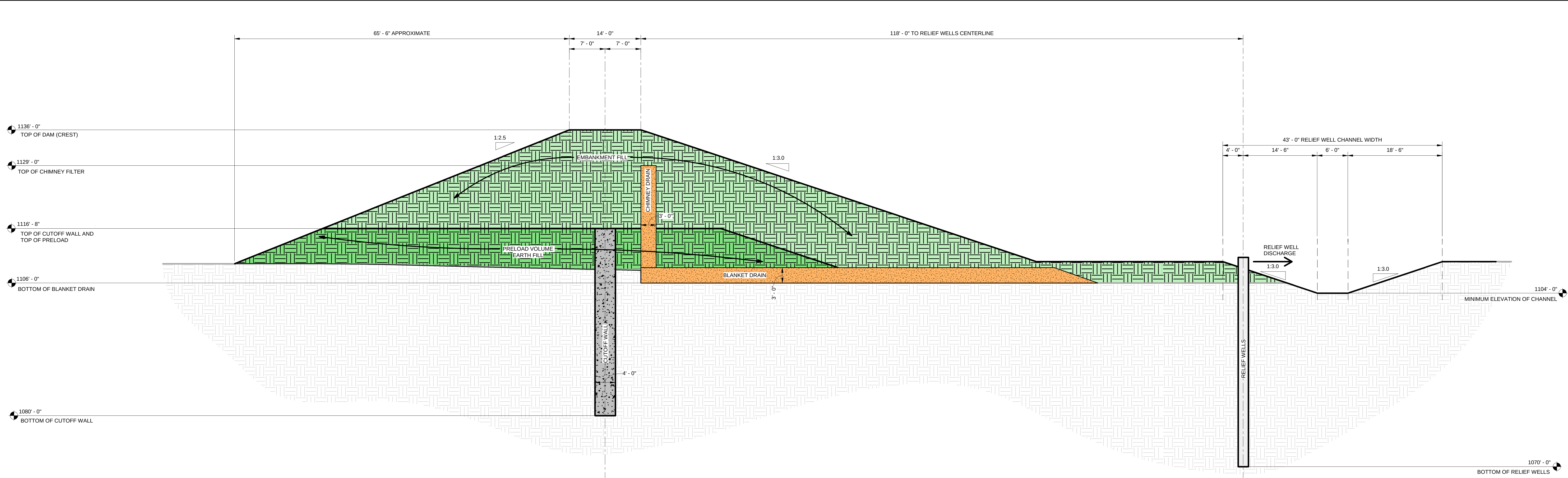


NRC
Natural Resources Conservation Service

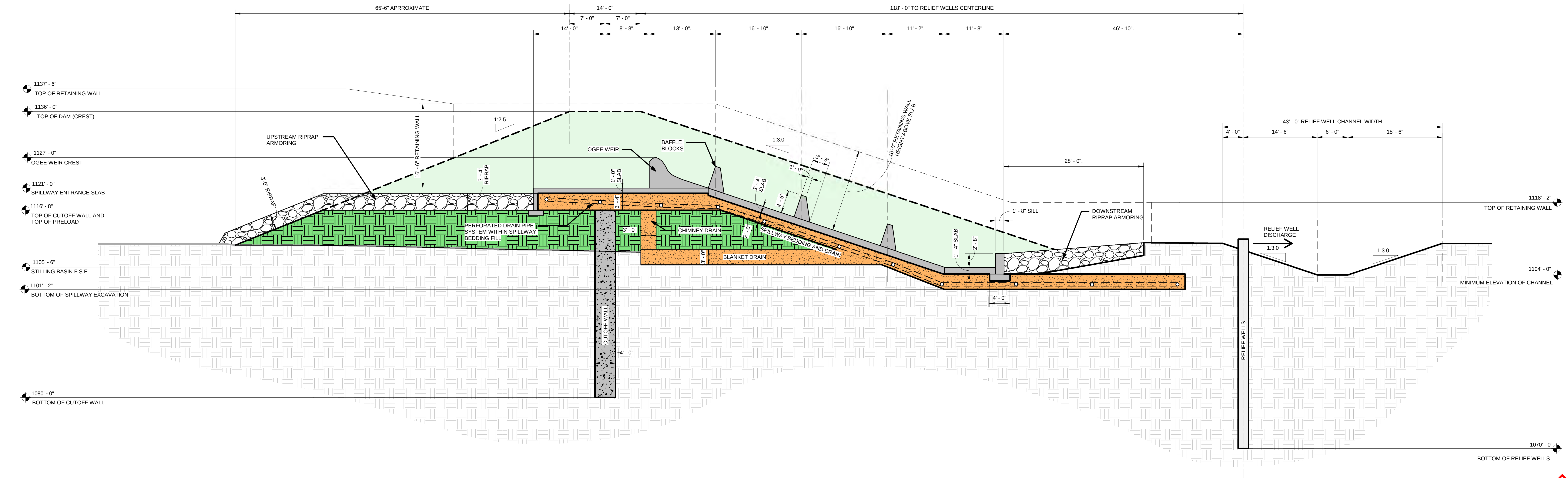
OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks

Designed:	Tom Schanadore
Drawn:	Tom Schanadore
Checked:	Kate Staley
Approved:	Christi Fisher

01/30/2025
SHT10



1 TYPICAL EMBANKMENT SECTION
1/8" = 1'-0"



2 STRUCTURAL SPILLWAY SECTION
1/8" = 1'-0"

STRUCTURAL SECTIONS
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS

OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks

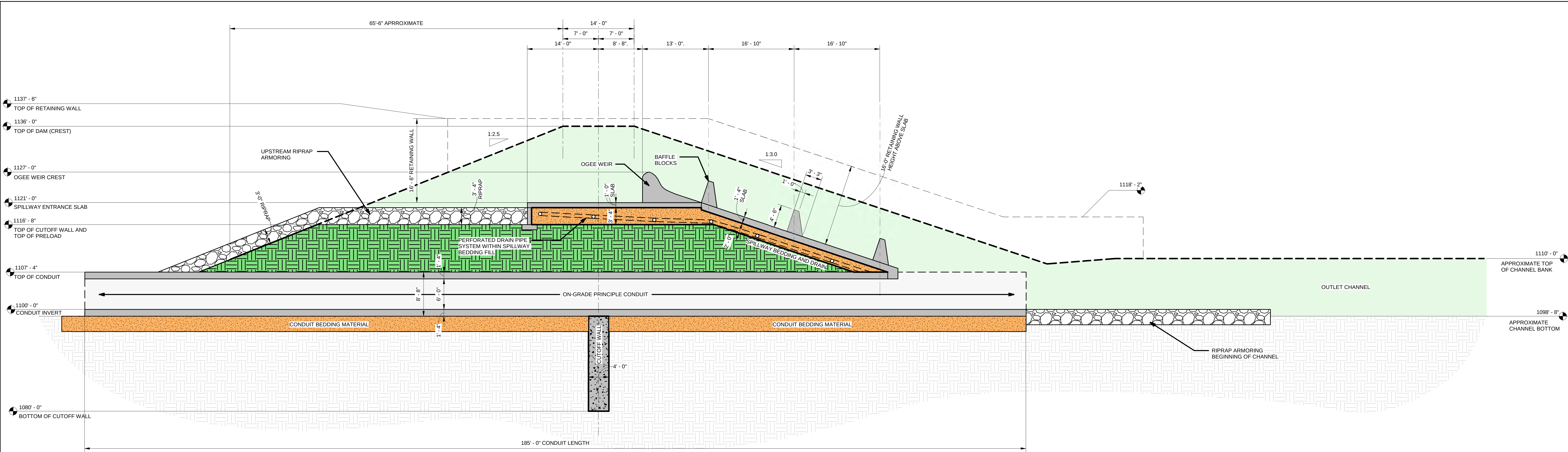


Date
01/30/2025

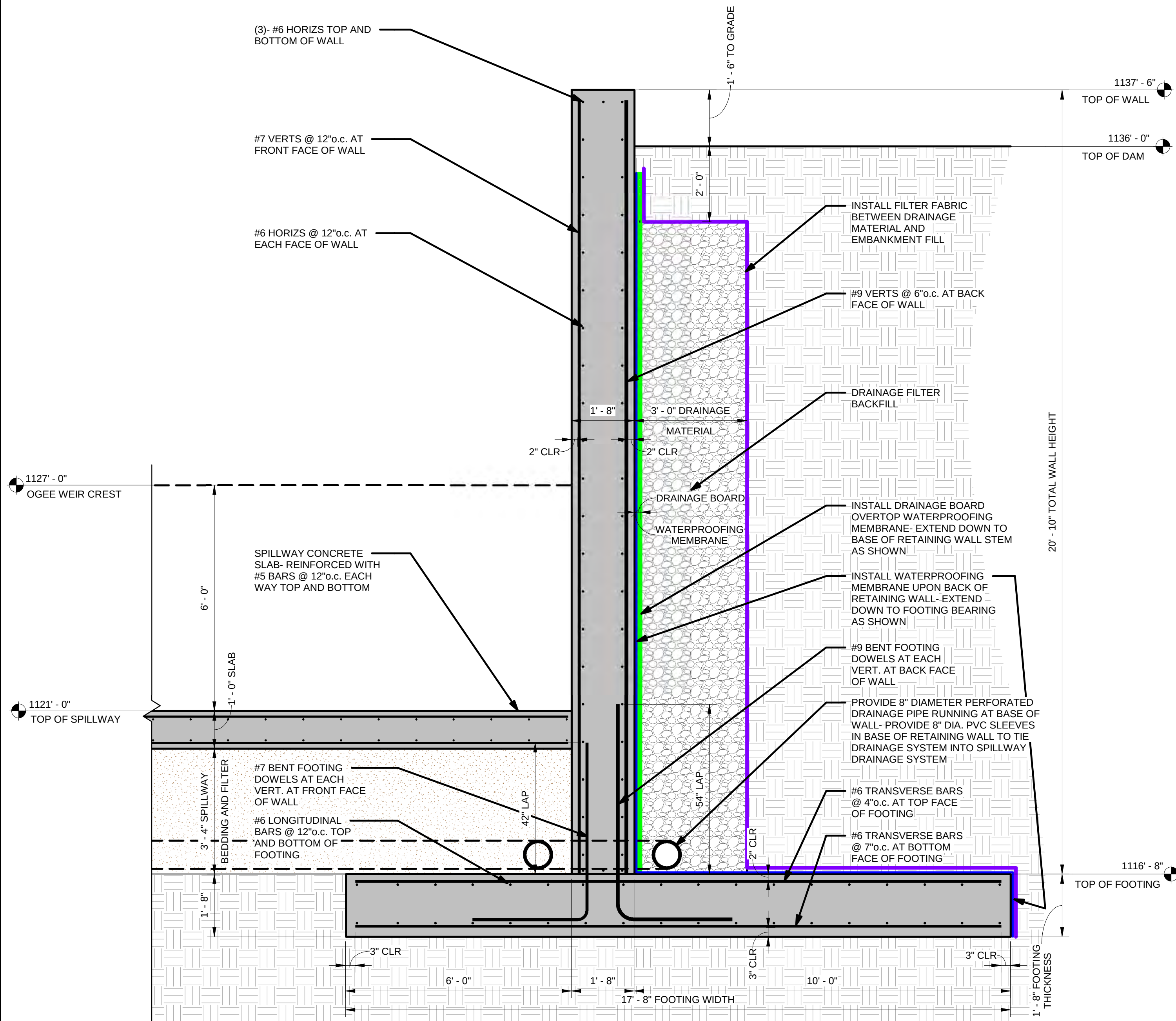
SHT11

Designed: Tom Schandore
Drawn: Tom Schandore
Checked: Kate Staley
Approved: Christi Fisher

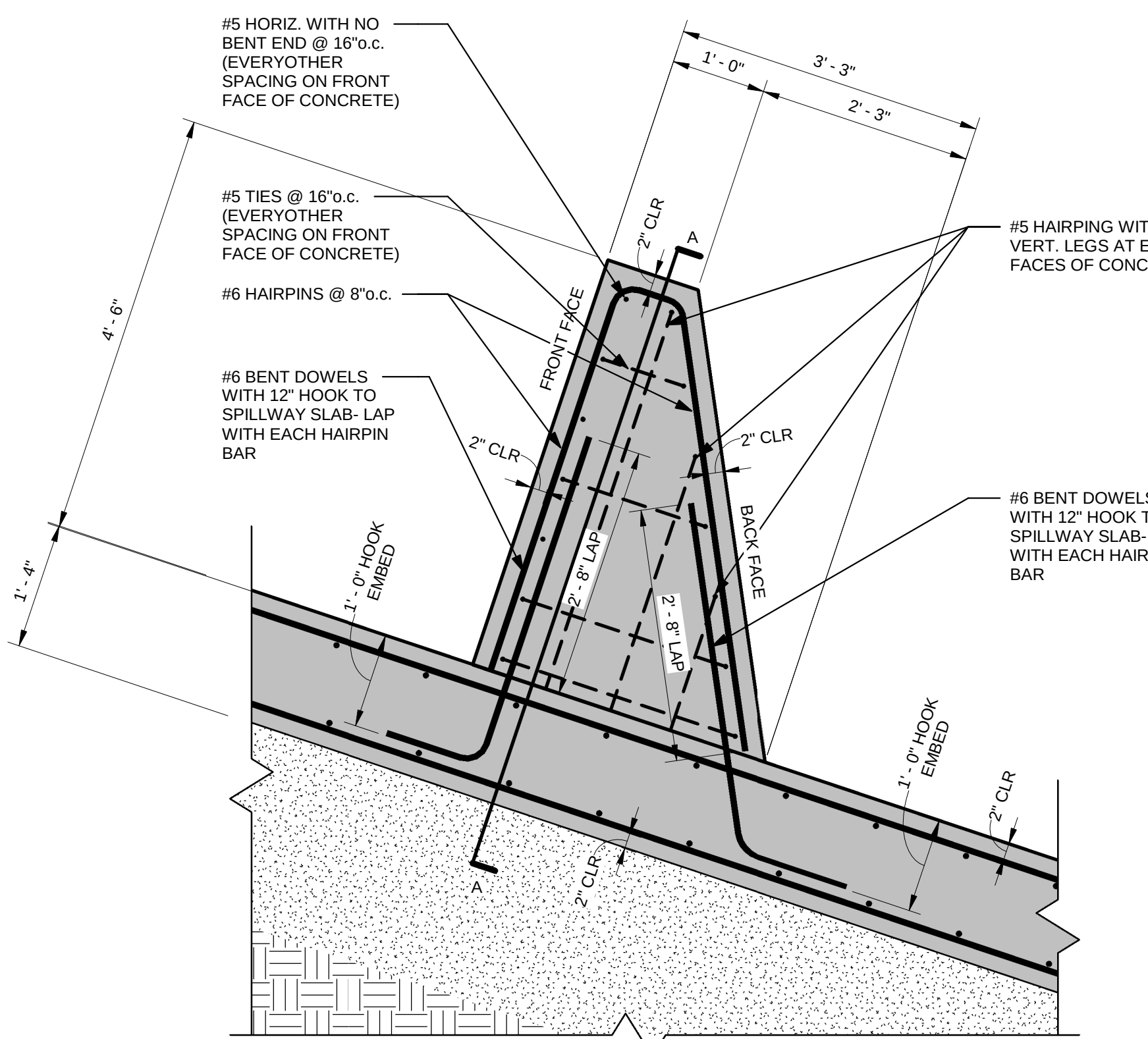
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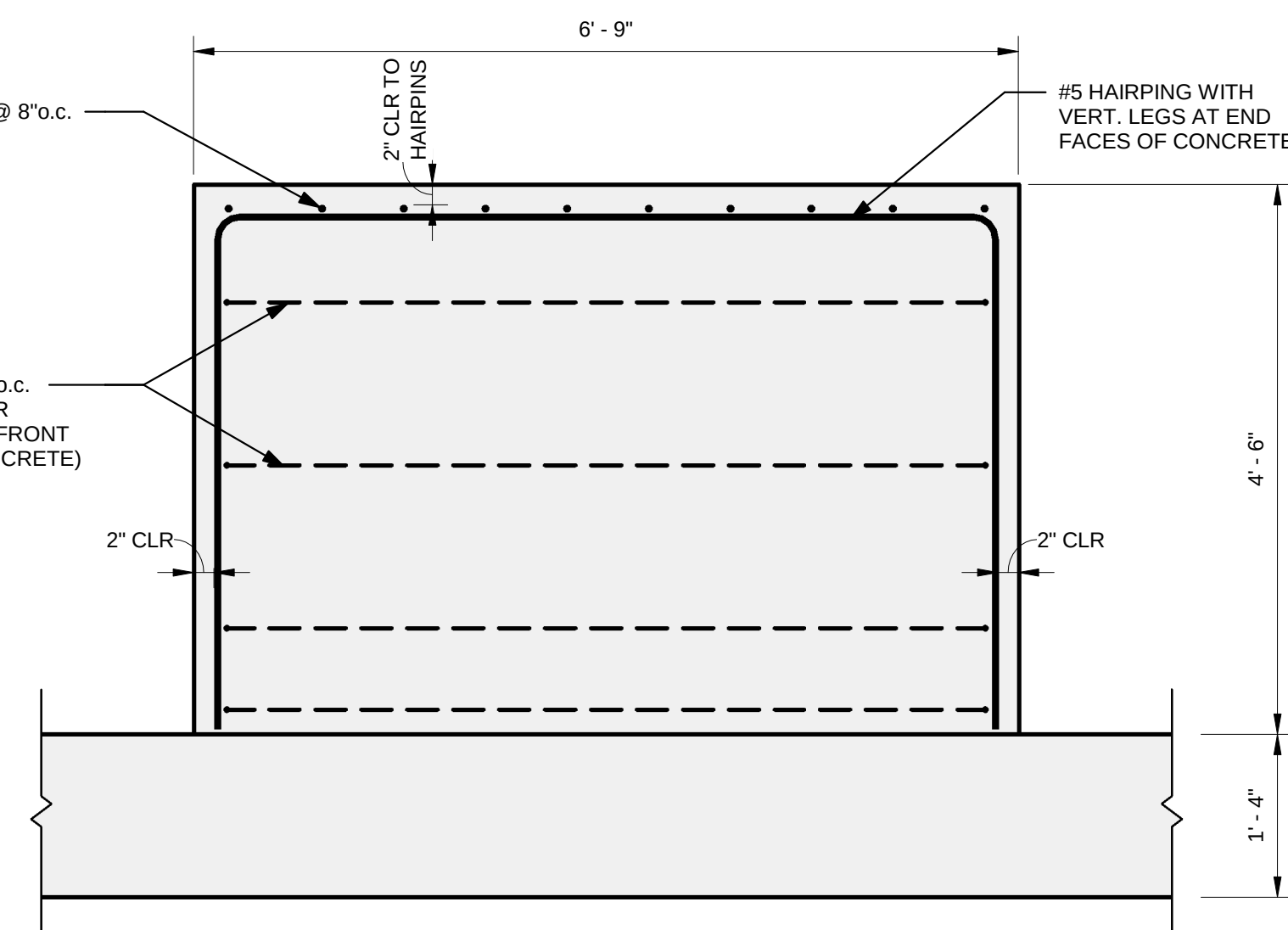
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1/8" = 1'-0"



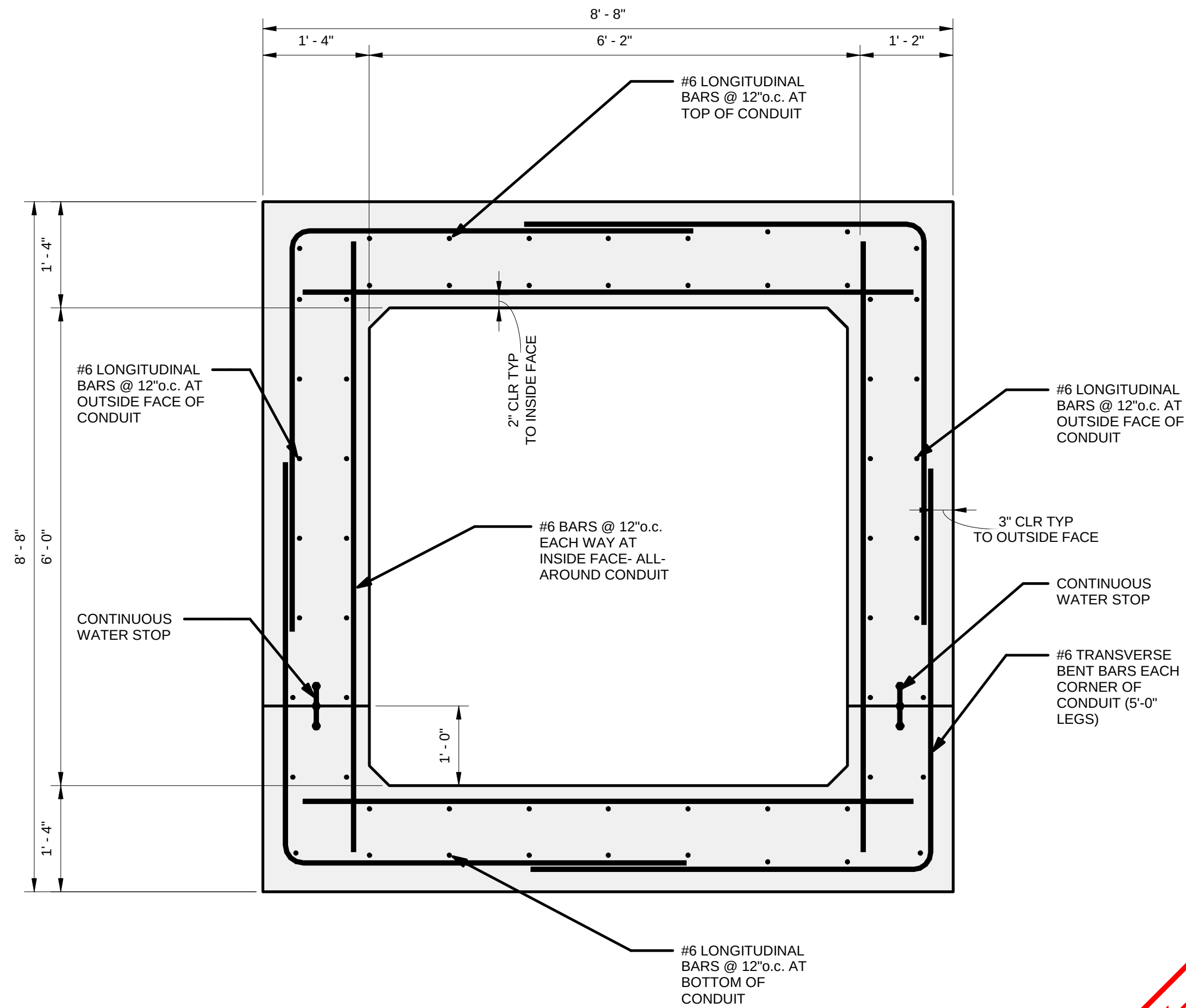
2 RETAINING WALL SECTION
1/2" = 1'-0"



3 BAFFLE BLOCK REINFORCING SECTION
3/4" = 1'-0"



SECTION A-A



5 CONDUIT REINFORCING SECTION
3/4" = 1'-0"

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STRUCTURAL SECTIONS
UPPER TURTLE RIVER DAM #10 - CONCEPT DESIGN DRAWINGS

OWNER: Grand Forks County Water Resource District
COUNTY: Grand Forks



Designed: Tom Schanndore
Drawn: Tom Schanndore
Checked: Kate Staley
Approved: Christi Fisher