



**US Army Corps
of Engineers®**

St. Paul District

Hydrology and Hydraulics Appendix D

Fargo Moorhead Metropolitan Area
Flood Risk Management Project

Supplemental Environmental
Assessment Document

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Hydrology and Hydraulics

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- Attachment 2. Southern Embankment Wind/Wave Analysis
- Attachment 3. Example Detailed Operation Plan

Hydrology and Hydraulics

1 OVERVIEW OF CHANGES SINCE 2013 SUPPLEMENTAL ENVIRONMENTAL ASSESSMENT

The Fargo-Moorhead Metropolitan Area is a flood prone area located in Cass County, North Dakota and Clay County, Minnesota. The Final Feasibility Report and Environmental Impact Statement (FEIS) for the Fargo-Moorhead Metropolitan Area Flood Risk Management Project (Project) was completed in July 2011 by the U.S. Army Corps of Engineers (USACE), and a Supplemental Environmental Assessment was completed in September 2013 to address proposed modifications to the Project (2013 SEA). Current river and floodplain conditions will be referred to as “Existing” conditions. With-project conditions have changed as the Project has evolved. The proposed with-project plan, developed based on recommendations by the Governors’ Task Force and informed by additional analysis by the Technical Advisory Group, will be referred to as “Plan B”. This report on the hydrology and hydraulics of Plan B will be an appendix to USACE’s 2018 Supplemental Environmental Assessment (2018 SEA). Additional information on the background of the Project and the history of the Plan B modifications is located in section 2.1 of the 2018 SEA.

As illustrated in Figure 1, Plan B includes physical modifications to the Southern Embankment alignment, including the location of the Wild Rice River and Red River Structures and the addition of culverts where the Southern Embankment crosses Wolverton Creek. Plan B also involves using Period of Record (POR) hydrology vs. Expert Opinion Elicitation (EOE) hydrology and operating with a 37-foot stage through town instead of the previously proposed 35-foot stage through town. While the Project purposes are the same as they were prior to the Task Force meetings, the Plan B modifications make it difficult to perform an apples-to-apples comparison between pre-Governors’ Task Force hydraulic model results (Phase 8.1) and the Plan B hydraulic model results (Phase 9.0). Comparisons to previous hydraulic model results should be made with care.

It should be noted that all elevations in this document are referenced to the North American Vertical Datum 1988 (NAVD 88), unless otherwise specified. Additionally, all dam embankment elevations referenced in this document refer to post-settlement elevations, unless otherwise specified.

2 HYDROLOGY

Inflow hydrology was originally developed during the Project’s FEIS. Initially, POR hydrology was developed; but as the study progressed, the hydrology was revised to focus on a shorter period of record developed by an EOE panel. The EOE hydrology produced peak flow and balanced hydrographs that vary over time; however, the design effort focused on assuring the Project would meet operational goals for the highest peak flow and volume conditions identified via the EOE panel. This hydrology is known as the Wet Cycle Hydrology, which will be referred to as the EOE/WET hydrology. The inflow hydrographs (main

stem balanced hydrographs and local inflow hydrographs) required for the subsequent unsteady flow modeling efforts were developed for the EOE/WET condition.

The Governors' Task Force recommended that the Project should use the POR hydrology instead of the EOE/WET hydrology, which presented challenges to the study effort. First, while POR hydrology is being used to report Project impacts in this report, USACE is still required to assess impacts based on EOE/WET hydrology. However, in acknowledgement of the Governors' Task Force recommendation and because a comparison of EOE results to POR results indicates that water surface elevations are similar, results based on the POR hydrology will be presented in this document. Second, as mentioned in the preceding paragraph, the detailed hydrology required for the unsteady flow modeling had been developed for the EOE/WET condition, not the POR. The non-Federal sponsor's consultant, the Houston-Moore Group (HMG), developed the inflow hydrographs needed for the POR unsteady flow modeling efforts. HMG's efforts are documented in the attached Technical Memorandum, POR Hydrology Development (Attachment 1).

It is important to note that both the EOE/WET and POR hydrology developed up to this point do not include flood events after 2009. After Project completion, USACE will perform a levee system evaluation of the Project in support of FEMA's accreditation effort, as specified in EC 1110-2-6067, *USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation*. USACE's levee system evaluation will rely upon the best available data, hydrologic methods, and most up-to-date guidance available at the time of the evaluation. As stated in the Advisory Committee on Water Information Bulletin 17C (2018), "As more years of record become available at each location, the determination of flood potential may change. Thus, an estimate may be outdated a few years after it is made." As a result of changing guidance (i.e.: Bulletin 17C) and advancements in the application of more advanced statistical techniques (i.e.: stochastic hydrology), it is very likely that the hydrology will need to be updated at the time of Project completion. A cursory investigation of updating the hydrology through 2017 indicates that peak flows could increase approximately 10 percent.

3 SOUTHERN EMBANKMENT FEATURES

Since the dam is located south of the Fargo-Moorhead Metropolitan Area, the dam has been referred to as the Southern Embankment in many Project documents. The Southern Embankment consists of the earthen embankment, three gated structures (the Diversion Inlet Structure (DIS), the Wild Rice River Structure (WRRS), and the Red River Structure (RRS)), and one ungated structure (the Wolverton Creek Crossing). Figure 2 shows the alignment of the Southern Embankment and the location of the four structures. Except for the location of the DIS, the alignment and structure locations have changed from what was presented in the 2013 SEA. The western-most portion of the Southern Embankment is referred to as the Western Tieback. A portion of the Western Tieback is designed to be overtopped in the event of a significant gate failure at one of the gated structures. The eastern-most portion of the Southern Embankment is referred to as the Eastern Tieback and ensures that breakout flows under Plan B conditions do not exceed breakout flows under Existing conditions during the Probable Maximum Flood (PMF) event.

It is noted that detailed design of the Southern Embankment has not yet been conducted and that all values reported in this document are subject to change during the detailed design phase. In particular, a primary design criterion for the Southern Embankment is the maximum pool elevation. The maximum pool elevation along the Western Tieback will be limited to 924.0 feet (if necessary the gated structures and/or the operation plan will be modified to keep the maximum pool elevation at or below 924.0 feet). However, the detailed design phase of this Project will attempt to achieve a maximum pool of 923.5 feet, which appears achievable based on preliminary modeling results.

3.1 Wind-Wave Analysis

An updated wind-wave analysis was performed due to the modification of the alignment of the Southern Embankment, particularly the shift of the alignment between the DIS and the WRRS, which increases the longest fetch length. The updated analysis involves a coupled two-dimensional modeling approach for capturing both wind setup and wave growth across areas inundated upstream of the Southern Embankment. Two wind speeds of 40 mph and 50 mph were considered, which roughly bound the 50% Annual Chance Exceedance (ACE) and 1% ACE wind speeds, adjusted to a 1-hour fetch-limited duration for over-water winds. These winds were considered for wind directions ranging from southeast to northwest (SE, SSE, S, SSW, SW, WSW, W, WNW, and NW) in order to capture the influence of wind from any hazardous direction across the pool. A maximum pool elevation of 924.0 feet was assumed for the analysis based on an early estimate of the PMF pool elevation. The pool's sloping water surface is also incorporated so that wind fetch lengths are accurately captured in the model. The results of this modeling indicate that 5 feet of freeboard is sufficient to keep wave overtopping associated with 40 mph and 50 mph winds well below allowable rates of wave overtopping flow. The 50 mph wind model runs indicate a few areas along the northern-most portion of the embankment have 2% wave runup values exceeding 5 feet of freeboard; however, the estimated coincident probability of this event occurring is approximately 1.97×10^{-5} ACE (or 1 in 50,800). In the event that this rare combination of pool and wind occurs, wave overtopping is still not expected to lead to damage on the downstream slope of the embankment and initiation of dam failure is extremely remote. For additional information on the wind-wave analysis, please see *Southern Embankment Wind-Wave Analysis* (Attachment 2).

3.2 Crest Profile

The maximum pool elevation profile plus the required freeboard, which is controlled by the results of the wind-wave analysis, determine the required crest elevation of the dam, except in the tieback reaches where overtopping will be accounted for in the design. The design elevations of the Southern Embankment crest profile are provided in Figure 2 (non-tieback reaches), Figure 3 (Western Tieback), and Figure 4 (Eastern Tieback). Between the DIS and the RRS, the dam crest will have an elevation of 929.0 feet. The gated structures and a short reach of the embankment on either side of these structures will have a crest elevation of 931.0 feet. While the wind-wave analysis indicates that a crest elevation of 929.0 feet is sufficient, the portion of the Southern Embankment between the RRS and the Eastern Tieback will transition from a crest elevation of 929.0 feet just east of the RRS to a crest elevation of 931.0 feet at its southern end just prior to transitioning to the Eastern Tieback. This transition is necessary to maintain five (5) feet of freeboard above the sloping maximum pool elevation, which is produced by the PMF event.

A deviation from USACE criteria to have less than (5) feet of freeboard and keep the crest elevation at 929.0 feet may be investigated, but at this time it cannot be assumed that a deviation from criteria would be granted by USACE Headquarters. The crest profile of the tieback reaches will be described in detail in the following tieback-specific sections.

3.3 Western Tieback

The Western Tieback centerline will extend from the DIS in a southwesterly direction towards the Sheyenne River, as shown in Figure 3. The previous location of the Western Tieback assumed that a portion of Cass County Road 17 would serve as a dam embankment, but this is no longer the case. The proposed alignment of the Western Tieback does not follow an existing road. However, as proposed with the previous alignment, a portion of the Western Tieback will be constructed at the maximum pool elevation. The portion of the Western Tieback constructed at the maximum design pool elevation would only be overtopped in the event of a significant gate failure leading to a rise in the pool water surface elevation above the maximum design pool.

Specifically, the Western Tieback crest profile will transition from 931.0 feet just southwest of the DIS down to the maximum pool elevation, which will be no greater than 924.0 feet. The crest will remain at the maximum pool elevation for approximately 3,800 feet in a southwesterly direction until a natural ridge is intersected. At the natural ridge the crest will rise to an elevation of 929.0 feet to again provide at least 5 feet of freeboard. The crest will remain at an elevation of 929.0 feet in a southwesterly direction until natural ground having an elevation of 929.0 feet is reached. This occurs approximately 1,200 feet west of County Road 36 (168th Avenue SE).

3.4 Eastern Tieback and Wolverton Creek Crossing

The Eastern Tieback embankment begins where the crest elevation drops below elevation 931.0 feet just west of Highway 75. The Eastern Tieback embankment elevation and the Wolverton Creek Crossing culvert sizes will be selected to ensure Comstock is not adversely impacted by the Project up through the PMF event and that stage impacts are less than 0.5 foot upstream of the county line road. The embankment will be designed to be overtopped, and the design of the culverts will minimize adverse effects to connectivity. The Eastern Tieback centerline, shown in Figure 4, will be located approximately 500 feet north of the Wilkin/Clay County line.

The preliminary design indicates the embankment will transition from elevation 931.0 feet to an elevation of 925.9 feet just east of Highway 75. This transition will not require a raise of the Highway 75 roadway profile. From just east of Highway 75 to just east of the BNSF Railway railroad embankment, the Eastern Tieback will be built to an elevation of 925.9 feet. As with Highway 75, the railroad embankment elevation will not need to be altered as part of this plan. After a short transition from elevation 925.9 feet to elevation 924.3 feet just east of the railroad embankment, the embankment will be built to an elevation of 924.3 feet until it ties into natural high ground east of Wolverton Creek. Because the alignment will cross Wolverton Creek, culverts will be installed at the crossing location. The preliminary design indicates that three 10-ft by 10-ft box culverts will be required.

The Eastern Tieback embankment elevations were selected to allow flow to overtop during the PMF event. However, flows passing over the embankment and through the culverts during the PMF event will not be greater than the flows passing through this location under Existing conditions, ensuring that PMF water surface elevations in Comstock do not increase.

3.5 Red River Structure

The updated Southern Embankment alignment changes the location of the RRS. The new location of the RRS is located on the North Dakota side of the Red River approximately two-thirds of a mile (straight-line distance) south of the previous location shown in the 2013 SEA. While the location has changed, the design of the RRS is expected to be very similar to what had been proposed previously. The structure is expected to consist of three tainter gates, each having a width of 50 feet and a sill elevation of 873.0 feet.

3.6 Wild Rice River Structure

The updated Southern Embankment alignment changes the location of the WRRS. The new location of the WRRS is approximately nine-tenths of a mile (straight-line distance) southwest of the previous location shown in the USACE 2013 Supplemental Environmental Assessment. While the location has changed, the design of the WRRS is expected to be very similar to what had been proposed previously. The structure is expected to consist of two tainter gates, each having a width of 40 feet and a sill elevation of 886.6 feet.

3.7 Diversion Inlet Structure

The location of the DIS does not change with Plan B, but the plan for conveying large flows to the DIS during extreme flood events does change. Prior to Plan B, conveyance of large flows was to be accomplished by making the borrow ditch along the Southern Embankment larger than what was necessary simply for local drainage. This oversized borrow ditch was referred to as the connecting channel. Plan B changes the location of the Southern Embankment and its associated borrow ditch such that it no longer makes sense to convey large flows to the DIS via an oversized borrow ditch. Instead an arc of excavation upstream of the DIS is expected to be the most efficient means of providing the necessary conveyance of large flows.

3.8 Local Drainage

The Southern Embankment severs existing drainage paths and the embankment itself will produce runoff. Plan view maps showing the general drainage strategy along the embankment are shown in Figure 5 (ND side) and Figure 6 (MN side). Preliminary profile figures of the drainage ditches are shown in Figure 7 through Figure 14. The drainage ditches also serve as borrow ditches for the Southern Embankment. The locations of culverts and grade control structures are shown on both the plan view maps and profile figures. Ditch slope and cross section configurations will be similar to those typically used by local water resource districts and watershed districts except that the size of the pool-side ditch may be larger than what is necessary for local drainage in order to provide material for the embankment. The longitudinal slope of ditches will generally be 0.05 percent, although slightly flatter and steeper slopes may be used to satisfy cut/fill needs. The bottom and side slopes of ditches will be conducive to future maintenance. In general, vegetation will suffice for erosion control, although riprap is expected to be installed at culverts,

grade control structures, and other locations where erosion potential is increased. Local drainage will be further refined during detailed design.

As shown in Figure 9, water will pond upstream of the Southern Embankment along Drain 27. This is a result of the combined goals of keeping the ditch invert above the 50% ACE flood elevation of the Wild Rice River at the ditch outlet, maintaining a minimum ditch slope of 0.04 percent, and producing wetland mitigation acres. Additional wetland mitigation acres will be investigated near the intersection of Drain 27 and the Southern Embankment and near the intersection of Drain 51 and the Southern Embankment.

4 PROJECT OPERATION AND IMPACTS

4.1 Operation Plan

4.1.1 Revisions to Operation Plan

Modifications to the Southern Embankment alignment and the decision to use a 37-foot stage through town through the 1% ACE event and a stage of 40 feet through town up through the 0.2% ACE event have resulted in modifications to the operation plan. Results presented in the 2018 SEA are based on an approximated operations plan, which continues to be addressed and refined. Therefore, it is likely that slightly different impacts than those reported in this document will occur in future modeling after the full operation plan is implemented. The Project operation plan described below is the intended full operation plan that will be incorporated into the hydraulic models prior to making any final determinations of pool levels and extents, real estate requirements, cultural impacts, gated structure design, and Western Tieback and Eastern Tieback design.

4.1.2 Use of Enloe and Abercrombie Gages

During times of normal river flow (i.e., no flooding) and for all flood events where the stage through town would not exceed 37.0 feet (produced during a flow of 21,000 cfs): 1) the RRS and WRRS will remain completely open, and 2) the gates at the DIS will be essentially closed (local drainage ditch runoff will be allowed through the structure prior to the event). To determine if the stage would exceed 37.0 feet (flow of 21,000 cfs), the sum of the flows at USGS Gage 0505152130 (Red River of the North at Enloe, ND) and USGS Gage 05053000 (Wild Rice River near Abercrombie, ND) will be evaluated. An analysis of historical floods indicates close to a 1:1 relationship between the sum of the Enloe gage and Abercrombie gage flows and the future flow at USGS Gage 05054000 (Red River at Fargo, ND), indicating that the combined flow at these two gages is a good predictor for the flows at the Fargo gage. Therefore, operation of the RRS and WRRS will only begin when the combined flow at Enloe/Abercrombie reaches 21,000 cfs. It is noted that a flow of 21,000 cfs is approximately a 5% ACE event (20-year flood) at the Fargo gage, meaning that only flow events less frequent than the 5% ACE will require operations.

On the rising limb of a typical flood hydrograph, when the total flow at the Enloe/Abercrombie gages is 21,000 cfs, the total flow passing through town at the same point in time will likely be between 10,000 cfs and 15,000 cfs, depending on how quickly the flows at Enloe/Abercrombie are rising and how much flow is contributed by Wolverton Creek. Historically, the travel time for peak flows from Enloe/Abercrombie

to Fargo is approximately two days. Beginning gate operations before the flows into the benefitted area exceed 21,000 cfs is necessary to store water during the rising limb of the hydrograph in order to minimize downstream stage impacts.

4.1.3 Beginning of Gate Operation

Gate operations begin by partially closing all gates at both the RRS and WRRS to restrict flow entering the benefitted area. Flows into the benefitted area are gradually reduced during this initial time period to meet the downstream stage impacts, resulting in storage of water upstream of the dam. Flows into the benefitted area will not be reduced by more than 2,000 cfs per day to ensure the rate of stage fall in the benefitted area does not exceed the natural fall rate, as a quick stage fall may impact bank stability.

Gate flow releases are based on an algorithm that considers the flows of six watercourses (Red River, Wild Rice River, Sheyenne River, Maple River, Rush River, and Wolverton Creek) and the physical storage characteristics of the areas inundated upstream of the Southern Embankment. This algorithm considers the flows on each of the six watercourses and operational limits in order to determine the appropriate flow releases through the three gated structures necessary to meet the stage impact requirements of the Project. The portion of the algorithm that accounts for the flows and timing of the six watercourses is based on a power law function in the form of $Q = aV^b$, where Q is the gate flow release, V is the storage volume, and a and b are user-defined coefficients. The user-defined coefficients are determined by simulating numerous synthetic and historic simulations and fitting the operated hydrographs to the existing condition hydrographs at a location downstream of the diversion outlet.

A general outline of the operational targets based on Enloe/Abercrombie flows is summarized in the bullet points below. A detailed operation plan for Plan B will be generated in the future. An example detailed Project operation plan, created for the pre-Governors' Task Force plan, is provided in Attachment 3 and shows the general organization and level of detail that will be incorporated into the detailed operation plan for Plan B. In general, pool elevations are referenced to the gage that will be added upstream of the RRS. However a gage along the Western Tieback will be used for extreme flood events where the maximum pool elevation might be achieved.

4.1.4 Operation Details

Enloe/Abercrombie flow > 21,000 cfs but ≤ 39,000 cfs

- Follow the gate flow release algorithm
 - Fargo gage: Expected stage of 37.0 feet
 - Approximately 21,000 cfs total flow into benefitted area (RRS + WRRS + Wolverton) produces an expected stage of 37.0 feet at the Fargo gage
 - Due to concerns with FEMA accreditation after construction is complete, 39,000 cfs was selected as an upper end estimate of the 1% ACE flow to ensure a stage of 37.0 feet is not exceeded for events up through the 1% ACE event. A preliminary investigation into the hydrology indicated that an increase of approximately 10% in the 1% ACE flow could occur as a result of incorporating the large 2010 and 2011 floods into the analysis that occurred after the original hydrology was developed. A

final hydrologic investigation will be conducted to determine the 1% ACE flow after the Project is constructed in support of the FEMA accreditation process. Methodology for determining the 1% ACE flow will be determined based on the best hydrologic methods and USACE guidance effective at that time.

- DIS: Maximum flow of 20,000 cfs. DIS gates will be operated to limit flow increase to 2,000 cfs per hour until a sufficient flow depth in the diversion channel is realized. This will minimize the potential for erosion in the diversion channel.
- Pool Elevation: Maximum water surface elevation of 921.0 feet
- Enloe/Abercrombie flow > 39,000 cfs but ≤ 66,000 cfs
 - Follow the gate flow release algorithm
 - Fargo gage: Expected stage between 37.0 feet and 40.0 feet
 - 37.0 feet is the target benefitted-area stage for 39,000 cfs Enloe/Abercrombie flow and 40.0 feet is the target benefitted-area stage for 66,000 cfs Enloe/Abercrombie flow; linearly interpolate for Enloe/Abercrombie flow between 39,000 cfs and 66,000 cfs
 - Approximately 21,000 cfs total flow into benefitted area (RRS + WRRS + Wolverton) produces an expected stage of 37.0 feet at the Fargo gage
 - Approximately 27,000 cfs total flow into benefitted area (RRS + WRRS + Wolverton) produces an expected stage of 40.0 feet at the Fargo gage
 - Pool Elevation: Maximum water surface elevation of 922.5 feet
 - DIS: Maximum flow between 20,000 cfs and 25,000 cfs. DIS gates will be operated to limit flow increase to 2,000 cfs per hour until a sufficient flow depth in the diversion channel is realized. This will minimize the potential for channel erosion.
 - 20,000 cfs is the expected maximum for 39,000 cfs Enloe/Abercrombie flow and 25,000 cfs is the expected maximum for 66,000 cfs Enloe/Abercrombie flow; linearly interpolate for Enloe/Abercrombie flow between 39,000 cfs and 66,000 cfs
 - An unusually high-volume flood could require a DIS flow greater than 25,000 cfs to maintain a stage of 40.0 feet at the Fargo gage and prevent the pool from rising above 922.5 feet
- Enloe/Abercrombie flow > 66,000 cfs
 - Gate flow release algorithm no longer applies
 - Fargo gage: Maximum stage of 40.0 feet until pool reaches the maximum pool elevation
 - DIS: Maximum flow of 25,000 cfs until pool exceeds 922.5 feet; increase opening of gates until gates are fully open to keep pool level at or below the maximum pool elevation and Fargo gage ≤ 40.0 feet as long as possible
 - Using the 0.2% ACE balanced hydrograph and the PMF hydrograph as guides for estimating hydrographs for intermediate events, an event with a peak flow of between 90,000 cfs and 100,000 cfs with a 15-day volume of approximately 1,600,000 acre-feet would generate the maximum pool elevation without requiring a stage of > 40.0 feet at the Fargo gage
 - RRS and WRRS gates will be opened as needed to maintain the maximum pool elevation;

the Fargo gage is allowed to exceed 40.0 feet

- The PMF hydrograph, having a peak flow of 204,000 cfs and a 15-day volume of approximately 3,876,000 acre-feet, would generate the maximum pool elevation

4.1.5 Maximum Pool Elevation

An evacuation order will be issued for the Fargo-Moorhead urban area as the pool approaches the maximum pool elevation. To prevent the pool elevation from exceeding the maximum pool elevation, the RRS and WRRS gates would be opened to maintain the maximum pool elevation and stages would rise above 40.0 feet at the Fargo gage resulting in flooding of the Fargo-Moorhead urban area. There is sufficient flow capacity at the gated structures and Eastern Tieback to maintain the maximum pool level up through the PMF event. Figure 15 shows the maximum water surface elevations within the pool during the PMF event. As shown in Figure 15, the pool is not a flat pool; generally, water surface elevations increase going east from the Western Tieback and going south from the dam. This is due in part to the topography and slope of the basin, as well as the numerous roads that are elevated above natural ground. For example, the PMF pool increases approximately 2.6 feet in elevation between the dam embankment near the RRS and the Cass/Richland County line along the Red River.

4.1.6 Pool Drawdown

After the flood peak has passed and the pool begins to be drawn down, RRS and WRRS gate opening changes will be limited to ensure the rate of stage fall is in line with the natural rate of stage fall, which reduces the potential for bank instability and fish stranding. The operation plan limits the reduction in pool stage to no more than 2 feet per day, which is the historically-observed rate at USGS gage 05051522 – Red River of the North at Hickson, ND.

4.2 Floodplain Difference

Floodplain difference figures were generated to compare Existing to Plan B conditions. Comparisons were made for these conditions assuming two scenarios: “with emergency measures,” which assumes that emergency measures are fully in place, as required by the MN DNR and “without emergency measures,” which assumes that emergency measures are not in place, as required by USACE policy. It is noted that the “without emergency measures” scenario only accounts for levees that are certified. It does not account for levees that have been constructed but are not certified. All figures contained in this document representing the “with emergency measures” condition equate to a levee crest elevation equivalent to a water surface profile of 44 feet measured at USGS Gage 05054000, Red River at Fargo.

Figure 16 through Figure 25 illustrate the Existing and Plan B condition floodplains for each of the 10%, 5%, 2%, 1%, and 0.2% ACE events under both the “with emergency measures” and “without emergency measures” scenarios using POR hydrology. It should be noted that only structures located upstream of the Southern Embankment are displayed. In each of these figures, the dark blue areas represent regions that will be inundated under both the Existing and Plan B conditions. The light blue areas characterize the regions that are dry under Existing conditions, but will now be within the floodplain as a result of the Project. Finally, the green-hatched areas describe regions that are within the floodplain under Existing conditions, but will remain dry as a result of the Project, for that particular event. It should be noted that

each of these boundaries does not quantify the depth of inundation, rather only the footprint of inundation itself.

Similar to Figure 16 through Figure 25, Figure 26 through Figure 35 describe the floodplain and benefitted areas for each of the 10%, 5%, 2%, 1%, and 0.2% ACE events, but also annotates stages and how flow is moving throughout the system. Like Figure 16 through Figure 25, each of the boundaries illustrated in Figure 26 through Figure 35 does not quantify the depth of inundation, but rather only the footprint of inundation itself.

The additional flooding seen immediately upstream of the Southern Embankment for the 10% and 5% ACE events (Figures 16, 17, 21, 22, 26, 27, 31, and 32), events where the gates are not operated, will be investigated as the design effort progresses. Ditch sizing, culverts, berms, and flap gates will be investigated to reduce this additional flooding as much as possible.

The reduction in flooding within the benefitted area for the 10% and 5% ACE events (see Figures 16, 17, 21, 22, 26, 27, 31, and 32) is due to the presence of the Southern Embankment and the diversion channel, which alter the path of floodplain flow, even for events where the gates are not operated.

4.3 Stage Impacts

Stage impacts in Canada have been a concern for the Project. In both the FEIS and the 2013 SEA, the hydraulic modeling effort demonstrated that the Project could be operated to produce no stage impacts at the border with Canada for the 10%, 2%, and 1% ACE events and minimal stage impacts at the border with Canada for the 0.2% ACE event when comparing with-project conditions to Existing conditions, assuming no emergency measures for either with-project or Existing conditions.

Previous iterations of the hydraulic model extended downstream to the border with Canada, but the additional model size and run time made this impractical to sustain. However, this modeling indicated that stage impact results at the border with Canada are no worse than they are at Drayton, North Dakota. Therefore the stage impacts reported at Drayton represent the stage impacts at the border with Canada. While stage impacts are reported to the traditional 0.01 foot typical of steady flow hydraulic modeling, the model results are, at best, considered accurate to the nearest 0.1 foot (i.e., 0.04 foot impacts should be interpreted as 0.0 foot impacts).

Stage impact tables comparing the Existing condition water surface elevations to the Plan B condition water surface elevations have been generated at 17 different locations throughout the basin for the 10%, 5%, 2%, 1%, and 0.2% ACE events. The results of this analysis are included in Table 1 to Table 10. Stage impact results for the 5% ACE event were not reported in the FEIS or 2013 SEA. Previous with-project alternatives included storing water upstream of the Southern Embankment for the 5% ACE event, and therefore Project operation would have been adjusted to reduce stage impacts at Drayton to 0.0 foot. However, Plan B does not store water upstream of the Southern Embankment for the 5% ACE event. The minor stage impact is a result of the change in Project operation.

Figure 36 through Figure 40 represent the maximum total depth difference between the Existing and Plan B conditions water surface elevations for each of the 10%, 5%, 2%, 1%, and 0.2% ACE events using POR

hydrology. Only structures located upstream of the Southern Embankment are displayed. Because releases through the gated structures are the same for both the “with emergency measures” and “without emergency measures” scenarios, the Plan B water surface elevations are the same under both scenarios upstream of the dam. Therefore, only one set of figures was provided to represent both “with” and “without” emergency measures. Differences in maximum water surface elevation depth have been grouped into 6 categories:

- Clear: < 0.1 feet
- Blue: 0.1 feet – 0.5 feet
- Green: 0.5 feet – 1 foot
- Yellow: 1 foot – 2 feet
- Orange: 2 feet – 5 feet
- Red: > 5 feet

Table 11 shows the number of additional structures and acres impacted upstream of the dam compared to Existing conditions. The impacts are specified individually for Cass County, Richland County, Clay County, and Wilkin County. The table shows that the impacts are greater in Cass County and Clay County relative to Richland County and Wilkin County. The additional acreage impacted in Richland County and Wilkin County during the 1% ACE event using POR hydrology is less than 1 square mile (640 acres) in each county. A check of EOE/WET hydrology compared to POR hydrology for the 1% ACE event indicates the total number of additional residential and non-residential structures would be within 10 percent of the values reported.

Figure 41 shows the Plan B total depth upstream of the Southern Embankment during the 1% ACE event. This informs the treatment of existing structures, as discussed in section 5 of the 2018 SEA.

5 ASSUMPTIONS

The following is a list of assumptions that have been made for the 2018 SEA and will be further addressed during detailed design.

- Approximate operation plan does not provide substantially different water surface elevations compared to final proposed operation plan
- Maximum pool water surface elevation will be determined by a gage along the Western Tieback
- Five (5) feet of freeboard is required along the entire embankment
- Eastern Tieback needs to extend across Wolverton Creek
- Western Tieback will not convey flow under max pool conditions
- DIS, WRRS, and RRS gates have sufficient capacity to maintain the maximum pool elevation during the PMF event
- Drainage of areas inundated will be assessed during detailed design (likely decreasing impacts occurring during the more frequent events)

TABLES

Table 1: 10% ACE Water Surface Elevation and Impact Table, No Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	797.41	797.40	0.01
Oslo, MN	810.96	810.96	0.00
Grand Forks, ND	824.53	824.51	0.02
Thompson, ND	835.69	835.68	0.01
Climax, MN	843.44	843.42	0.02
Nielsville, MN	847.86	847.84	0.02
Shelly, MN	854.39	854.37	0.02
Halstad, MN	862.76	862.73	0.03
Hendrum, MN	866.65	866.64	0.01
Perley, MN	872.53	872.58	-0.05
Georgetown, MN	878.38	878.46	-0.08
Fargo/Moorhead (Fargo Gage)	895.57	895.78	-0.21
Western Tieback	-	-	-
Upstream of Dam, Wild Rice River	912.03	912.13	-0.10
Upstream of Dam, Red River	907.84	907.78	0.06
County Line @ Red River	910.80	910.76	0.04
County Line @ Wild Rice River	916.61	916.66	-0.05

Table 2: 10% ACE Water Surface Elevation and Impact Table with Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	797.41	797.41	0.00
Oslo, MN	810.97	810.96	0.01
Grand Forks, ND	824.54	824.52	0.02
Thompson, ND	835.71	835.69	0.02
Climax, MN	843.46	843.45	0.01
Nielsville, MN	847.88	847.87	0.01
Shelly, MN	854.42	854.40	0.02
Halstad, MN	862.78	862.76	0.02
Hendrum, MN	866.67	866.66	0.01
Perley, MN	872.55	872.60	-0.05
Georgetown, MN	878.41	878.48	-0.07
Fargo/Moorhead (Fargo Gage)	895.61	895.82	-0.21
Western Tieback	-	-	-
Upstream of Dam, Wild Rice River	912.03	912.13	-0.10
Upstream of Dam, Red River	907.85	907.79	0.06
County Line @ Red River	910.80	910.76	0.04
County Line @ Wild Rice River	916.61	916.66	-0.05

Table 3: 5% ACE Water Surface Elevation and Impact Table, No Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	799.37	799.31	0.06
Oslo, MN	812.33	812.28	0.05
Grand Forks, ND	827.97	827.80	0.17
Thompson, ND	840.61	840.30	0.31
Climax, MN	849.30	848.91	0.39
Nielsville, MN	853.57	853.18	0.39
Shelly, MN	859.47	859.16	0.31
Halstad, MN	866.39	866.10	0.29
Hendrum, MN	869.86	869.49	0.37
Perley, MN	875.42	875.28	0.14
Georgetown, MN	880.72	880.59	0.13
Fargo/Moorhead (Fargo Gage)	899.20	899.43	-0.23
Western Tieback	-	-	-
Upstream of Dam, Wild Rice River	915.24	915.05	0.19
Upstream of Dam, Red River	910.90	910.86	0.04
County Line @ Red River	913.65	913.62	0.03
County Line @ Wild Rice River	920.34	920.35	-0.01

Table 4: 5% ACE Water Surface Elevation and Impact Table with Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	799.38	799.31	0.07
Oslo, MN	812.33	812.29	0.04
Grand Forks, ND	827.97	827.81	0.16
Thompson, ND	840.63	840.31	0.32
Climax, MN	849.32	848.93	0.39
Nielsville, MN	853.59	853.20	0.39
Shelly, MN	859.48	859.17	0.31
Halstad, MN	866.40	866.12	0.28
Hendrum, MN	869.88	869.51	0.37
Perley, MN	875.44	875.30	0.14
Georgetown, MN	880.74	880.61	0.13
Fargo/Moorhead (Fargo Gage)	899.28	899.53	-0.25
Western Tieback	-	-	-
Upstream of Dam, Wild Rice River	915.24	915.05	0.19
Upstream of Dam, Red River	910.91	910.87	0.04
County Line @ Red River	913.66	913.63	0.03
County Line @ Wild Rice River	920.34	920.35	-0.01

Table 5: 2% ACE Water Surface Elevation and Impact Table, No Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	801.19	801.18	0.01
Oslo, MN	813.08	813.06	0.02
Grand Forks, ND	830.81	830.68	0.13
Thompson, ND	844.12	844.15	-0.03
Climax, MN	853.23	853.39	-0.16
Nielsville, MN	857.28	857.51	-0.23
Shelly, MN	862.46	862.70	-0.24
Halstad, MN	867.27	867.46	-0.19
Hendrum, MN	871.05	871.45	-0.40
Perley, MN	876.67	876.78	-0.11
Georgetown, MN	881.61	881.47	0.14
Fargo/Moorhead (Fargo Gage)	899.61	902.00	-2.39
Western Tieback	919.74	913.48	6.26
Upstream of Dam, Wild Rice River	919.76	915.99	3.77
Upstream of Dam, Red River	919.72	912.90	6.82
County Line @ Red River	920.47	916.22	4.25
County Line @ Wild Rice River	921.81	921.77	0.04

Table 6: 2% ACE Water Surface Elevation and Impact Table with Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	801.19	801.19	0.00
Oslo, MN	813.08	813.06	0.02
Grand Forks, ND	830.81	830.69	0.12
Thompson, ND	844.13	844.21	-0.08
Climax, MN	853.25	853.46	-0.21
Nielsen, MN	857.30	857.58	-0.28
Shelly, MN	862.47	862.76	-0.29
Halstad, MN	867.27	867.49	-0.22
Hendrum, MN	871.06	871.50	-0.44
Perley, MN	876.68	876.80	-0.12
Georgetown, MN	881.61	881.49	0.12
Fargo/Moorhead (Fargo Gage)	899.70	902.42	-2.72
Western Tieback	919.74	913.48	6.26
Upstream of Dam, Wild Rice River	919.76	915.99	3.77
Upstream of Dam, Red River	919.72	912.92	6.80
County Line @ Red River	920.47	916.22	4.25
County Line @ Wild Rice River	921.81	921.77	0.04

Table 7: 1% ACE Water Surface Elevation and Impact Table, No Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	802.36	802.33	0.03
Oslo, MN	813.53	813.52	0.01
Grand Forks, ND	832.78	832.71	0.07
Thompson, ND	846.74	846.80	-0.06
Climax, MN	856.80	856.93	-0.13
Nielsville, MN	860.84	860.97	-0.13
Shelly, MN	864.92	865.03	-0.11
Halstad, MN	868.14	868.25	-0.11
Hendrum, MN	872.41	872.61	-0.20
Perley, MN	877.21	877.30	-0.09
Georgetown, MN	881.93	881.82	0.11
Fargo/Moorhead (Fargo Gage)	899.65	903.28	-3.63
Western Tieback	920.85	913.86	6.99
Upstream of Dam, Wild Rice River	920.92	916.18	4.74
Upstream of Dam, Red River	921.02	914.10	6.92
County Line @ Red River	921.92	918.27	3.65
County Line @ Wild Rice River	922.43	922.27	0.16

Table 8: 1% ACE Water Surface Elevation and Impact Table with Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	802.37	802.33	0.04
Oslo, MN	813.53	813.52	0.01
Grand Forks, ND	832.79	832.72	0.07
Thompson, ND	846.76	846.78	-0.02
Climax, MN	856.83	856.88	-0.05
Nielsville, MN	860.86	860.92	-0.06
Shelly, MN	864.94	864.99	-0.05
Halstad, MN	868.15	868.22	-0.07
Hendrum, MN	872.42	872.57	-0.15
Perley, MN	877.22	877.26	-0.04
Georgetown, MN	881.93	881.79	0.14
Fargo/Moorhead (Fargo Gage)	899.73	904.16	-4.43
Western Tieback	920.85	913.87	6.98
Upstream of Dam, Wild Rice River	920.92	916.18	4.74
Upstream of Dam, Red River	921.02	914.12	6.90
County Line @ Red River	921.92	918.28	3.64
County Line @ Wild Rice River	922.43	922.27	0.16

Table 9: 0.2% ACE Water Surface Elevation and Impact Table, No Emergency Measures (POR Hydrology)

Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	804.03	803.95	0.08
Oslo, MN	814.33	814.28	0.05
Grand Forks, ND	836.97	836.72	0.25
Thompson, ND	849.97	849.95	0.02
Climax, MN	862.55	862.57	-0.02
Nielsville, MN	866.41	866.47	-0.06
Shelly, MN	868.34	868.40	-0.06
Halstad, MN	870.47	870.61	-0.14
Hendrum, MN	874.54	874.73	-0.19
Perley, MN	878.08	878.24	-0.16
Georgetown, MN	882.74	882.75	-0.01
Fargo/Moorhead (Fargo Gage)	902.10	905.48	-3.38
Western Tieback	922.42	914.48	7.94
Upstream of Dam, Wild Rice River	922.54	916.66	5.88
Upstream of Dam, Red River	922.73	915.71	7.02
County Line @ Red River	923.83	922.34	1.49
County Line @ Wild Rice River	923.96	923.86	0.10

Table 10: 0.2% ACE Water Surface Elevation and Impact Table with Emergency Measures (POR Hydrology)

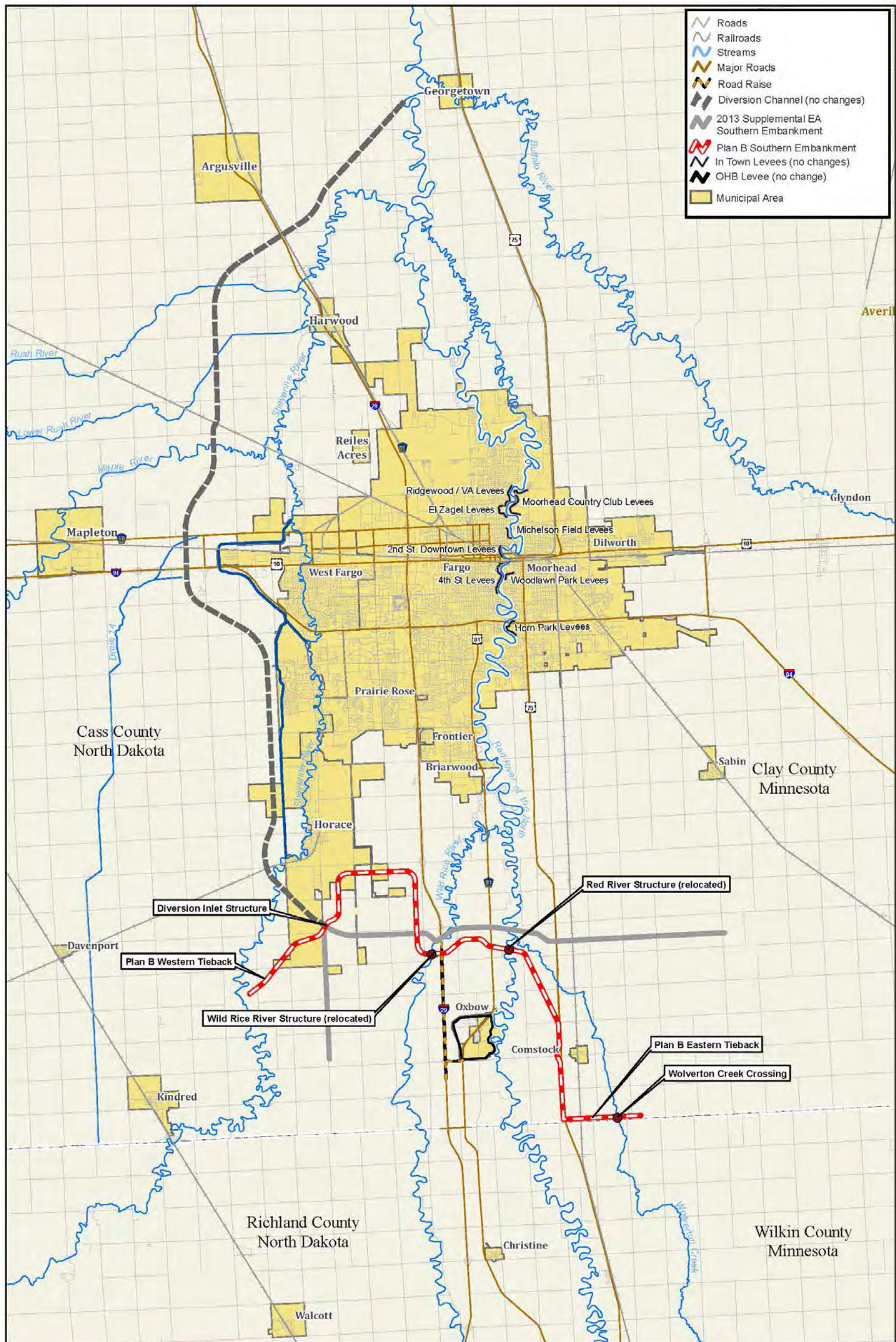
Location Name	With Project	Existing Conditions	Difference (ft)
Drayton, ND (30 mi. to Canada)	804.04	803.87	0.17
Oslo, MN	814.33	814.21	0.12
Grand Forks, ND	836.98	836.40	0.58
Thompson, ND	849.98	849.73	0.25
Climax, MN	862.56	862.06	0.50
Nielsville, MN	866.43	865.93	0.50
Shelly, MN	868.35	868.00	0.35
Halstad, MN	870.48	870.27	0.21
Hendrum, MN	874.56	874.36	0.20
Perley, MN	878.09	878.08	0.01
Georgetown, MN	882.75	882.58	0.17
Fargo/Moorhead (Fargo Gage)	902.70	907.32	-4.62
Western Tieback	922.42	914.49	7.93
Upstream of Dam, Wild Rice River	922.54	916.66	5.88
Upstream of Dam, Red River	922.73	915.74	6.99
County Line @ Red River	923.83	922.34	1.49
County Line @ Wild Rice River	923.96	923.86	0.10

Table 11: Pool Structure and Acre Impact above Existing Conditions Summary Table (POR Hydrology)

County	Location	10% ACE	5% ACE	2% ACE	1% ACE	0.2% ACE
Cass County	Additional Acres	462	624	6,576	7,294	5,233
Cass County	Number of Additional Residential Structures	0	0	37	42	31
Cass County	Number of Additional Non-Residential Structures	0	8	165	173	135
Richland County	Additional Acres	0	0	321	616	268
Richland County	Number of Additional Residential Structures	0	0	0	3	1
Richland County	Number of Additional Non-Residential Structures	0	0	5	18	7
Clay County	Additional Acres	0	0	2,828	3,090	1,529
Clay County	Number of Additional Residential Structures	0	0	8	9	7
Clay County	Number of Additional Non-Residential Structures	0	0	62	90	52
Clay County	Additional Acres	0	0	88	372	5
Clay County	Number of Additional Residential Structures	0	0	0	1	2
Clay County	Number of Additional Non-Residential Structures	0	0	8	10	3

FIGURES

Figure 1: Project Change Map



Project Change Map - June 2018
Fargo Moorhead Metro Area Flood Risk Management

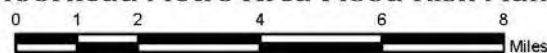


Figure 2. Southern Embankment Preliminary Design Schematic

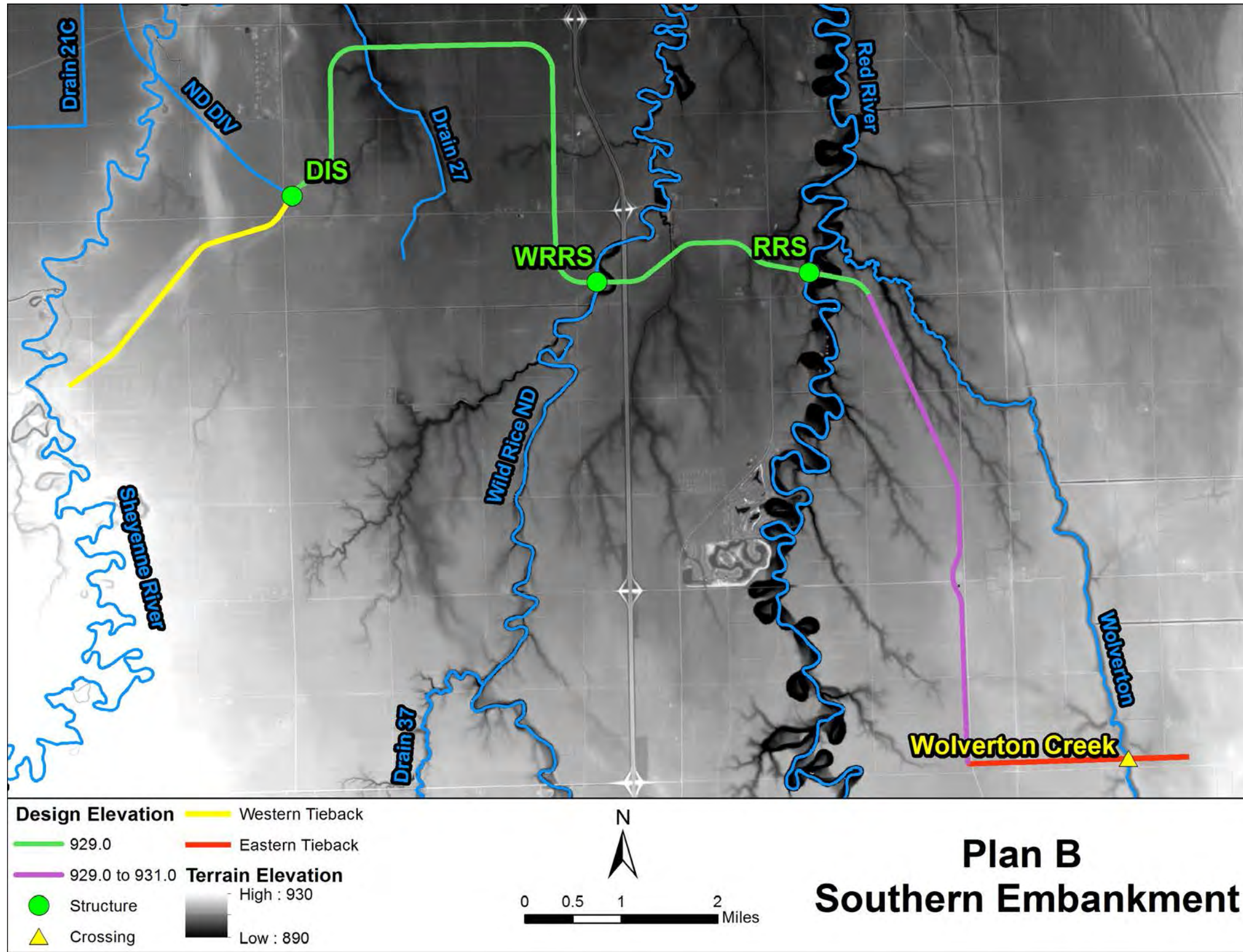


Figure 3. Western Tieback Preliminary Design Schematic

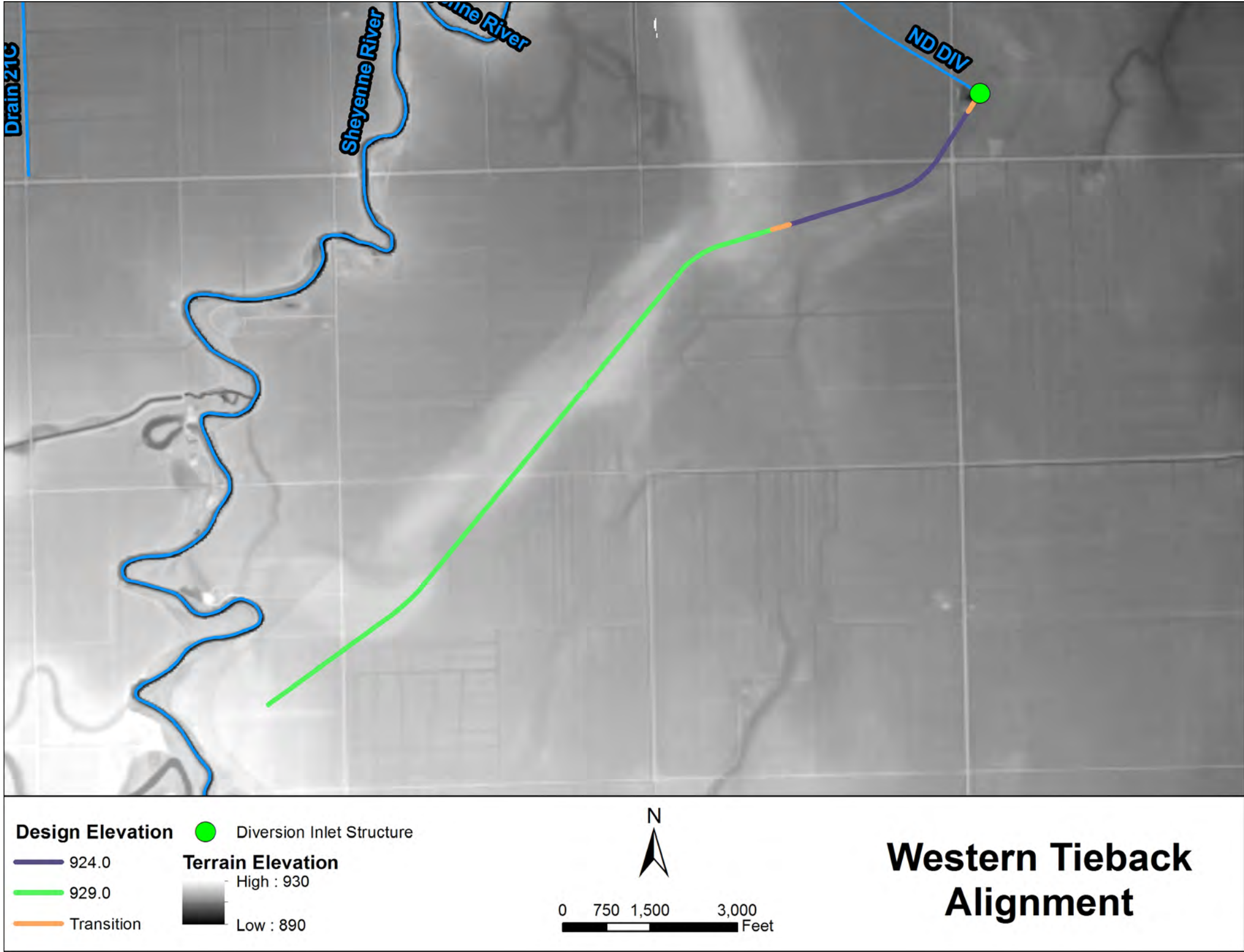


Figure 4. Eastern Tieback Preliminary Design Schematic

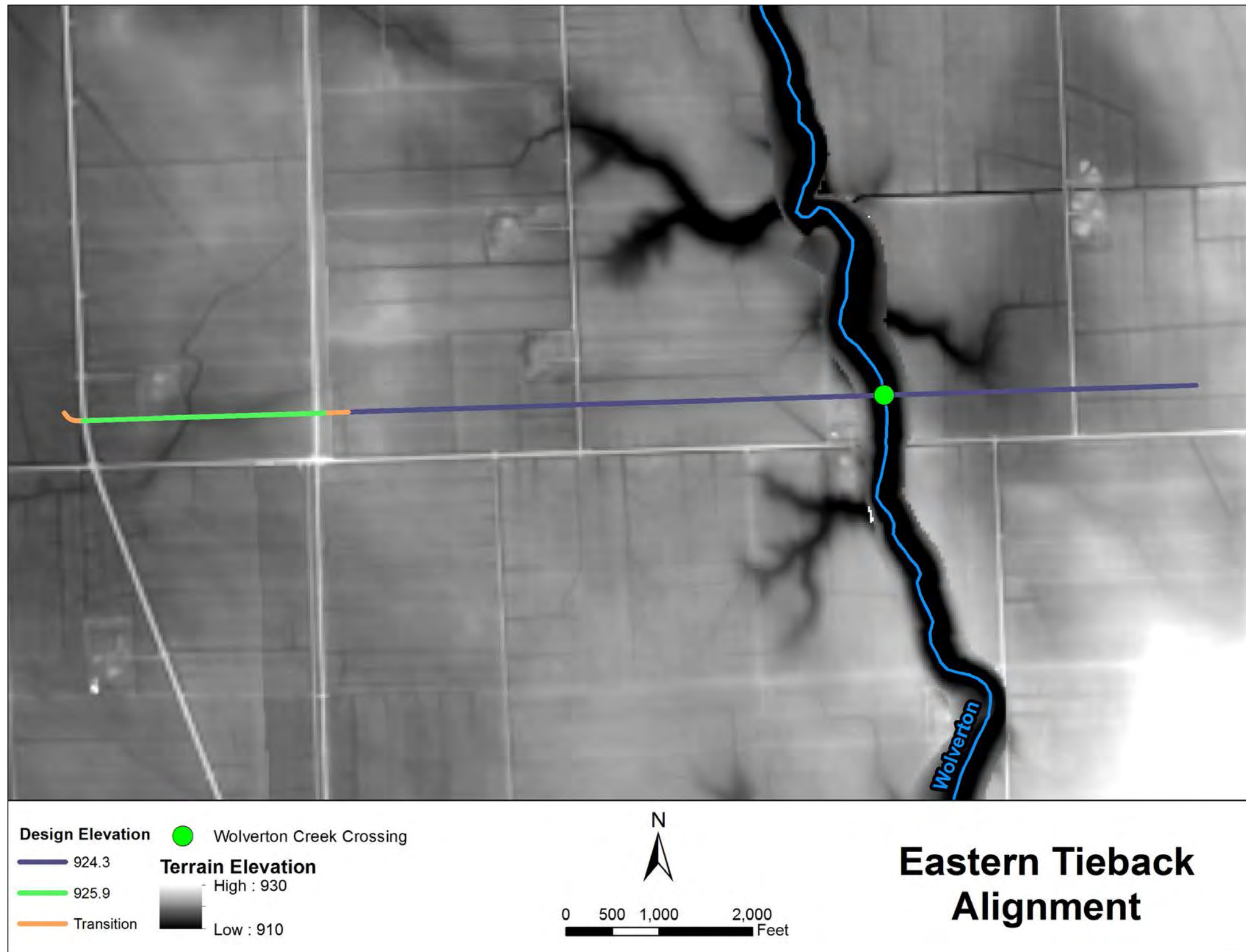


Figure 5. Local Drainage along Southern Embankment in North Dakota

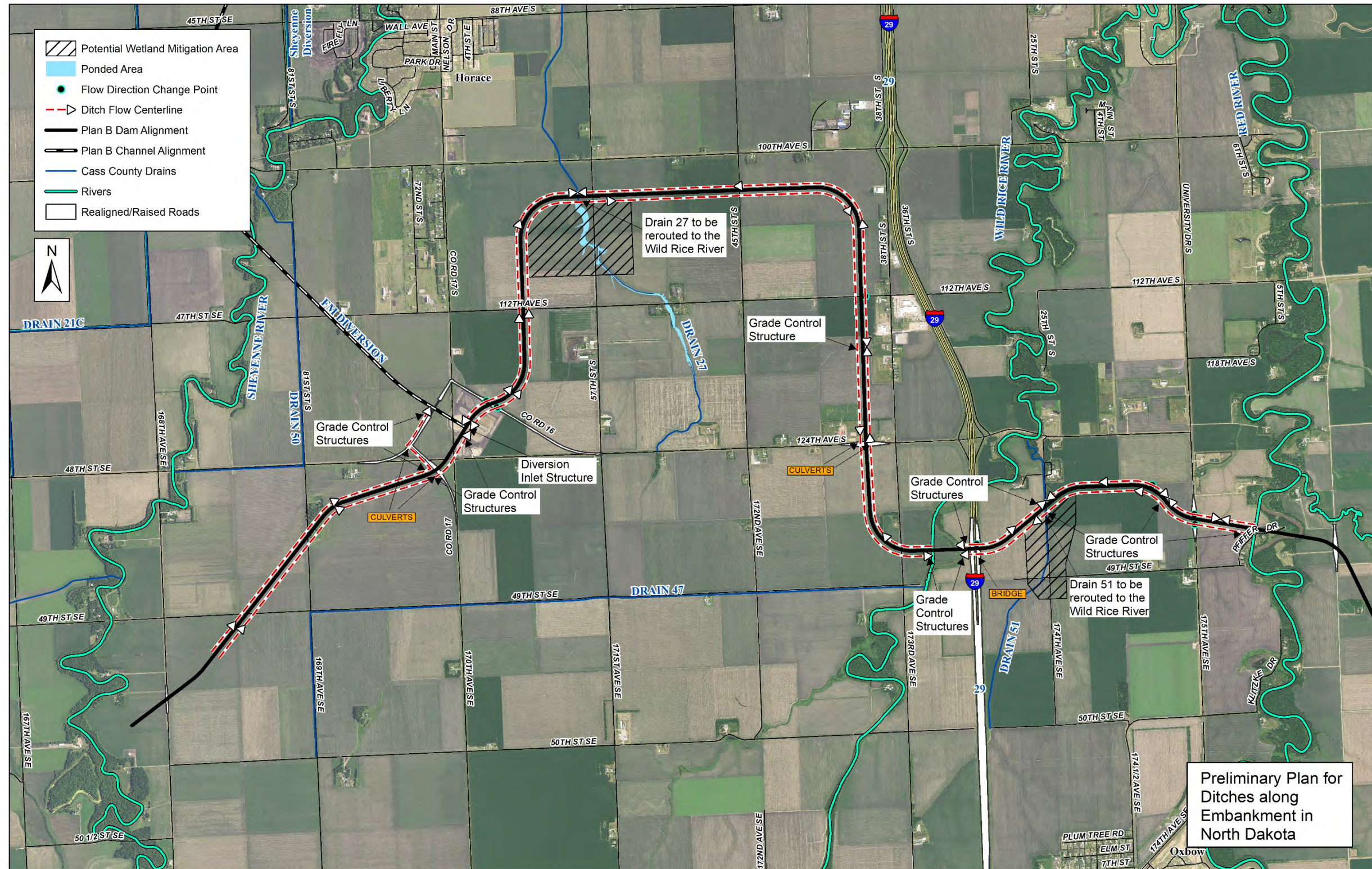


Figure 6. Local Drainage along Southern Embankment in Minnesota

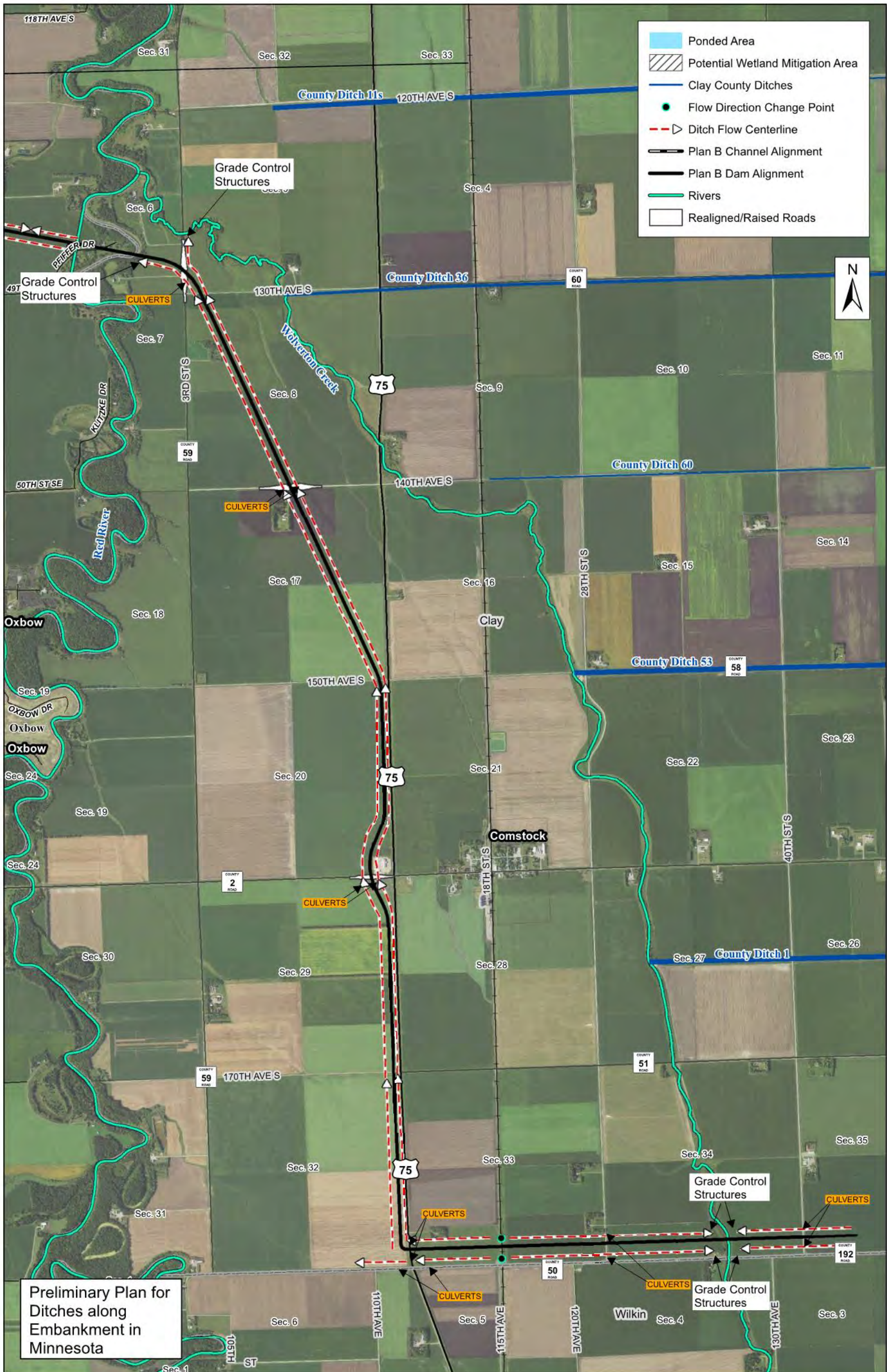


Figure 7. Pool-Side Ditch Profile in North Dakota, Western Tieback

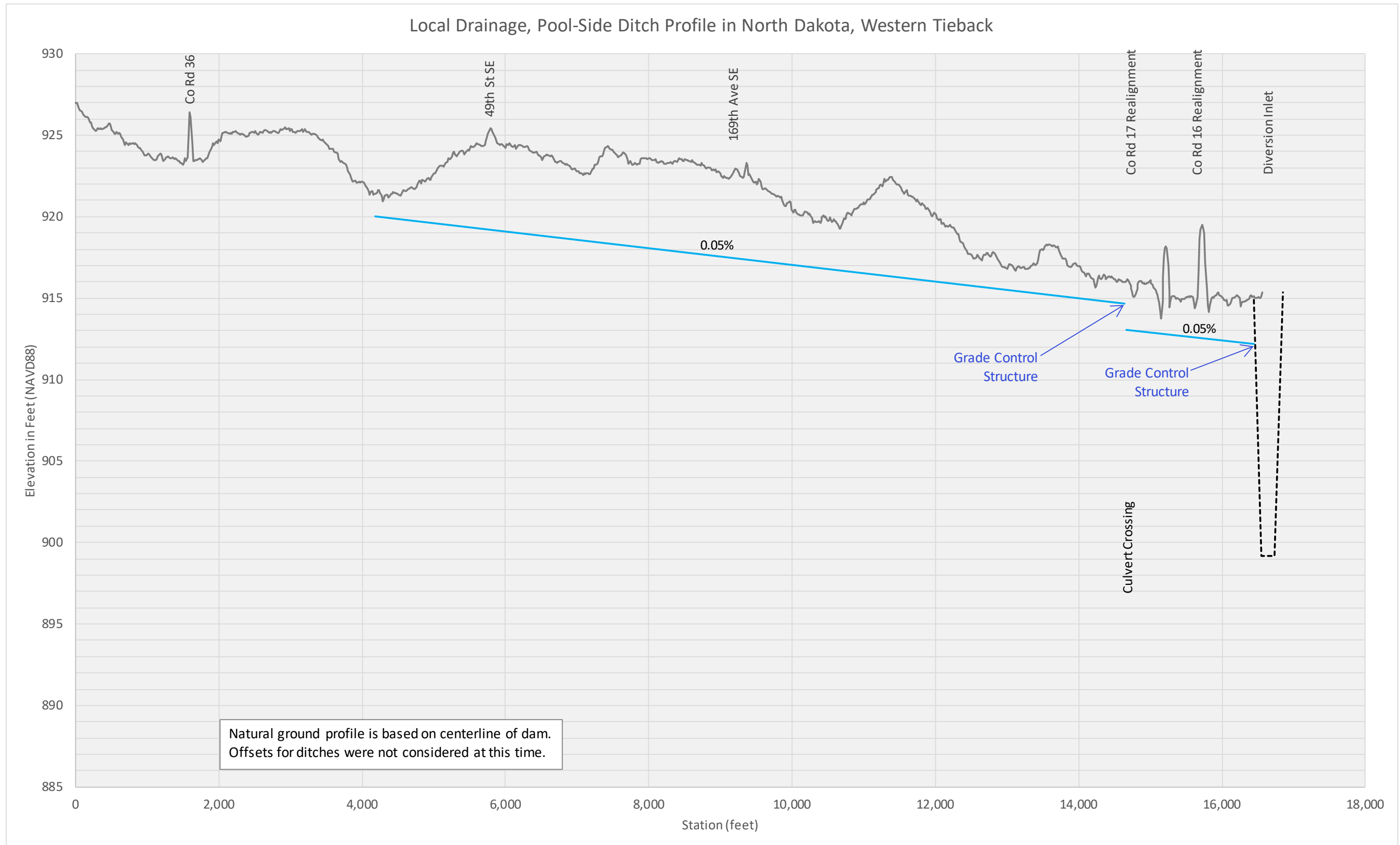


Figure 8. Benefitted-Side Ditch Profile in North Dakota, Western Tieback

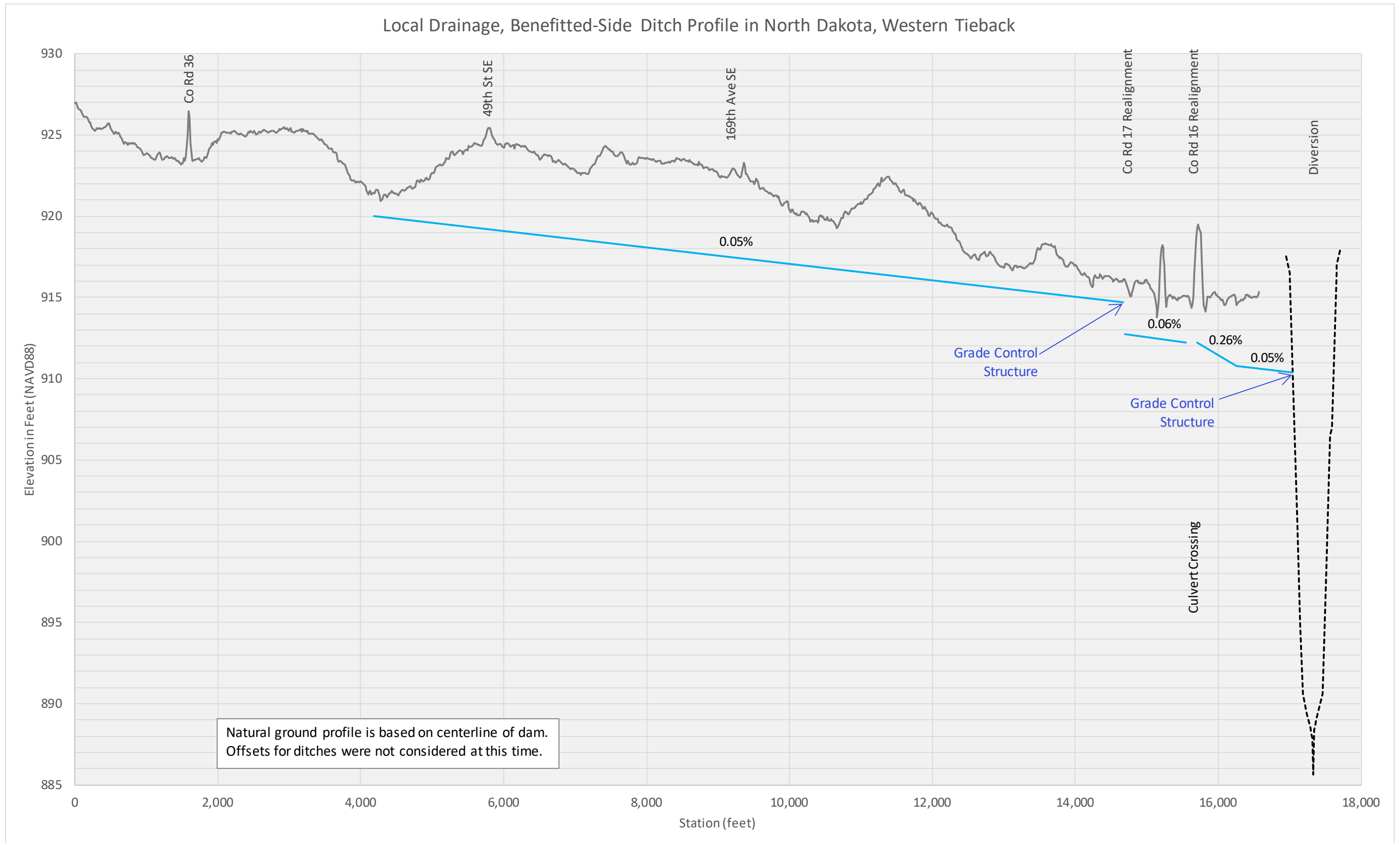


Figure 9. Pool-Side Ditch Profile in North Dakota, Diversion Inlet to Wild Rice River Structure

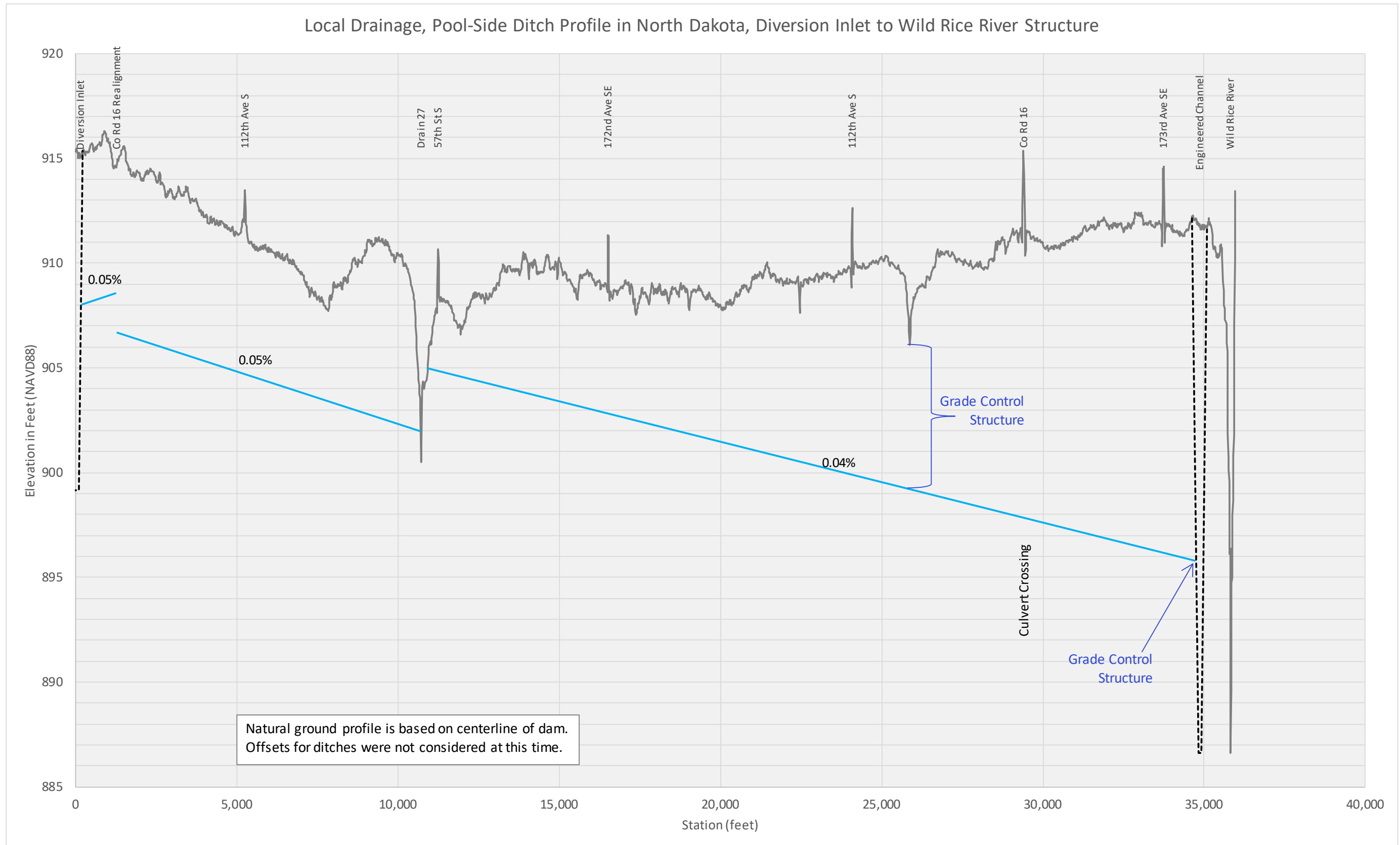


Figure 10. Benefitted-Side Ditch Profile in North Dakota, Diversion Inlet to Wild Rice River Structure

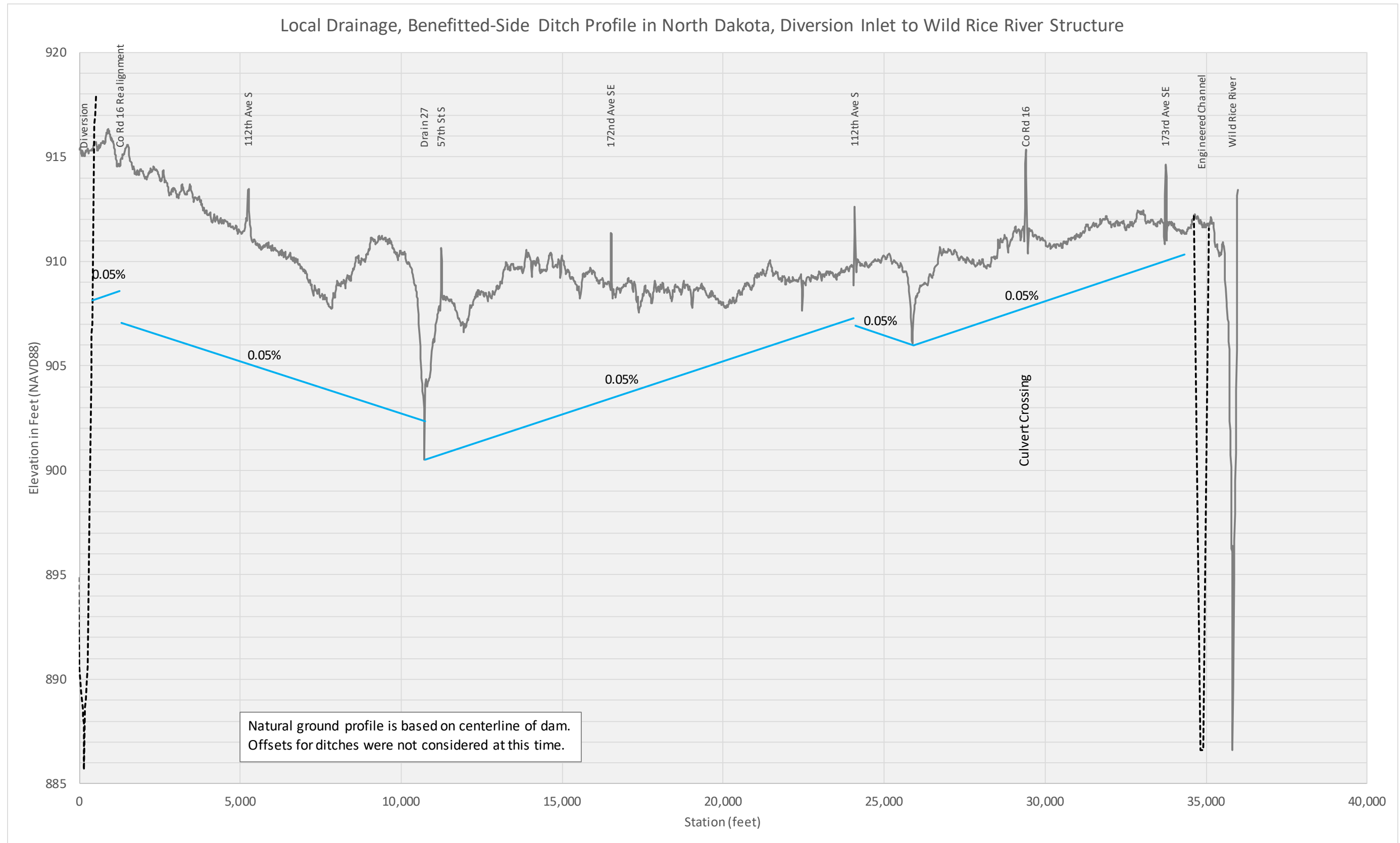


Figure 12. Benefitted-Side Ditch Profile in North Dakota, Wild Rice River Structure to Red River Structure

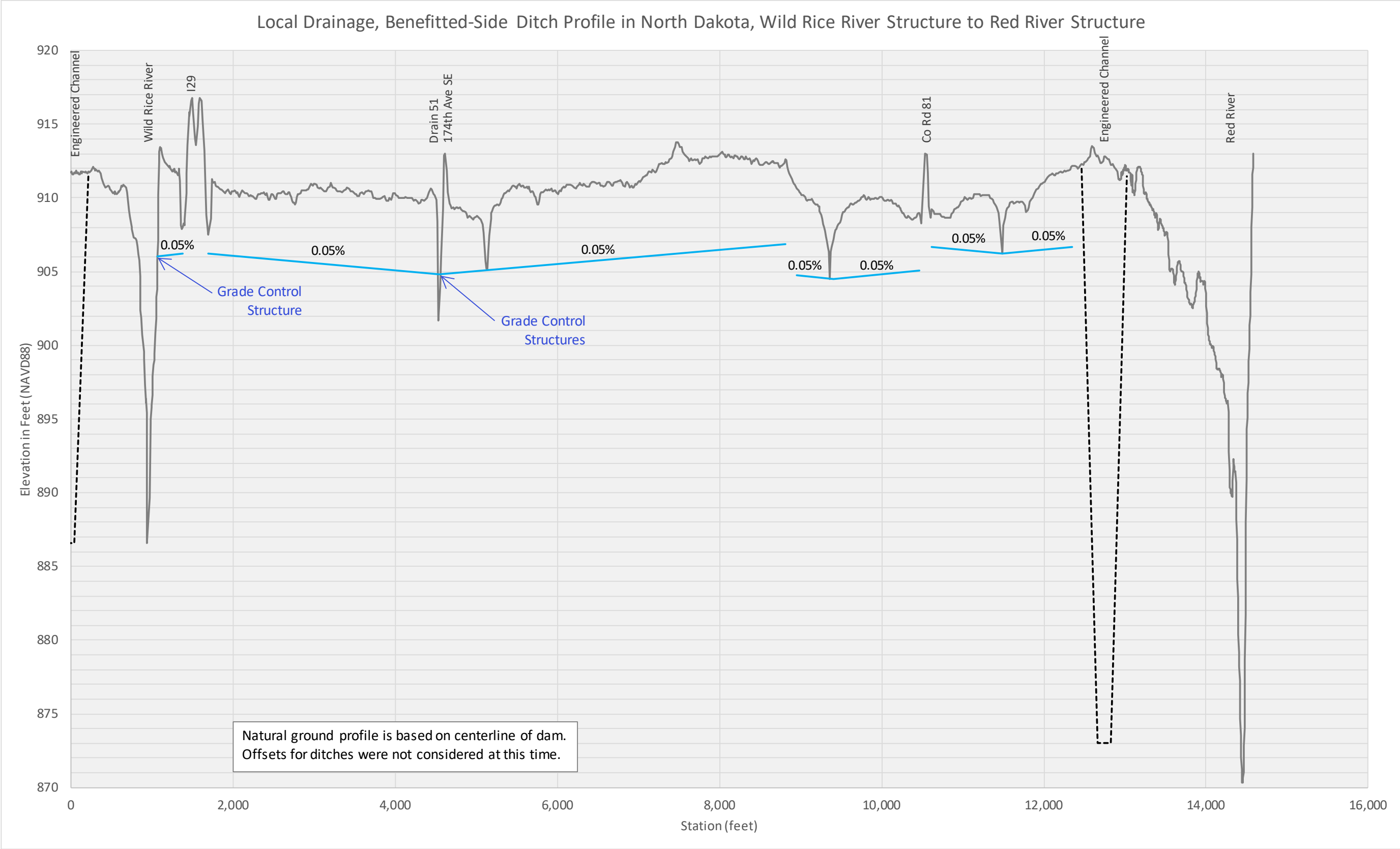


Figure 13. Pool-Side Ditch Profile in Minnesota

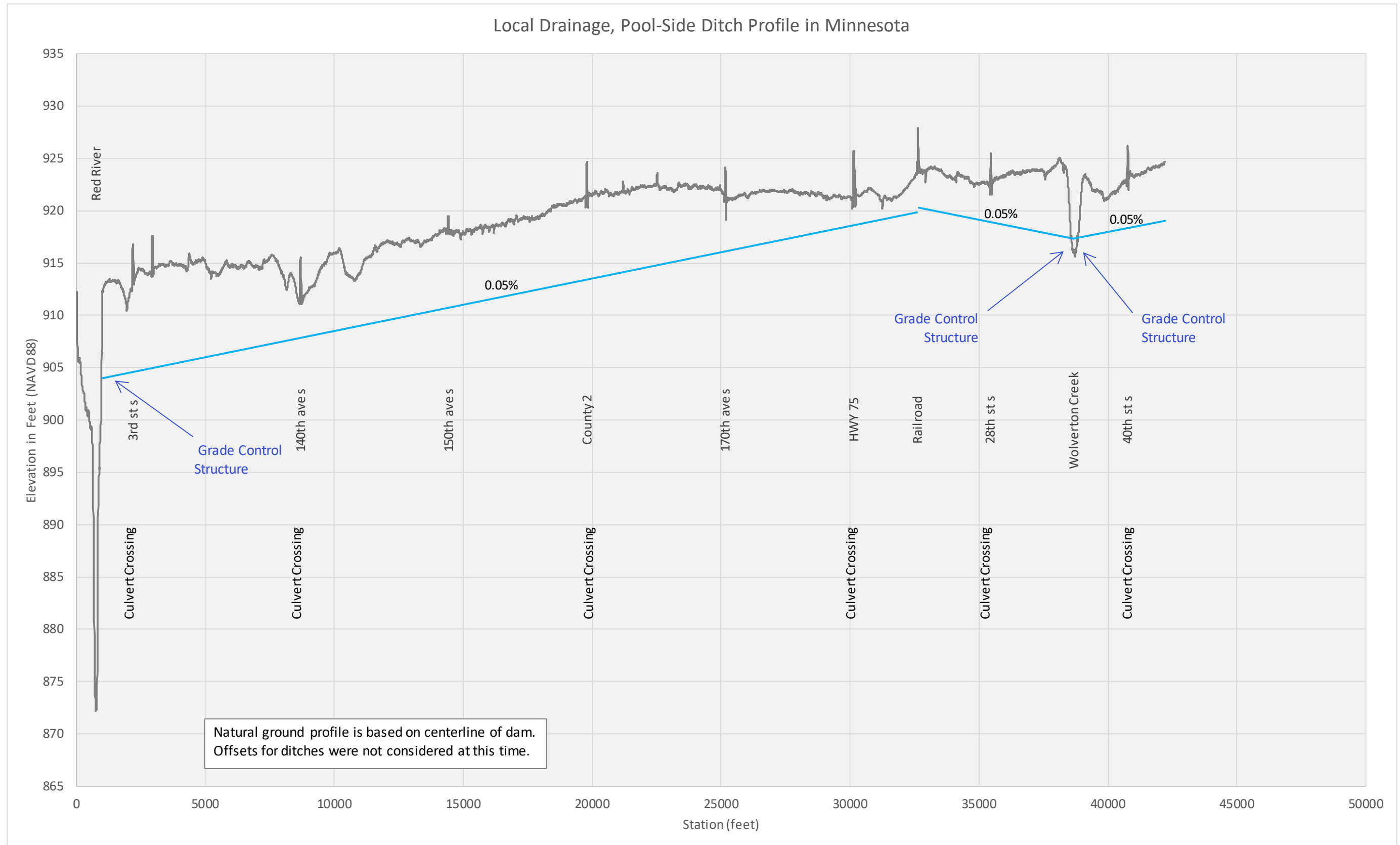


Figure 14. Benefitted-Side Ditch Profile in Minnesota

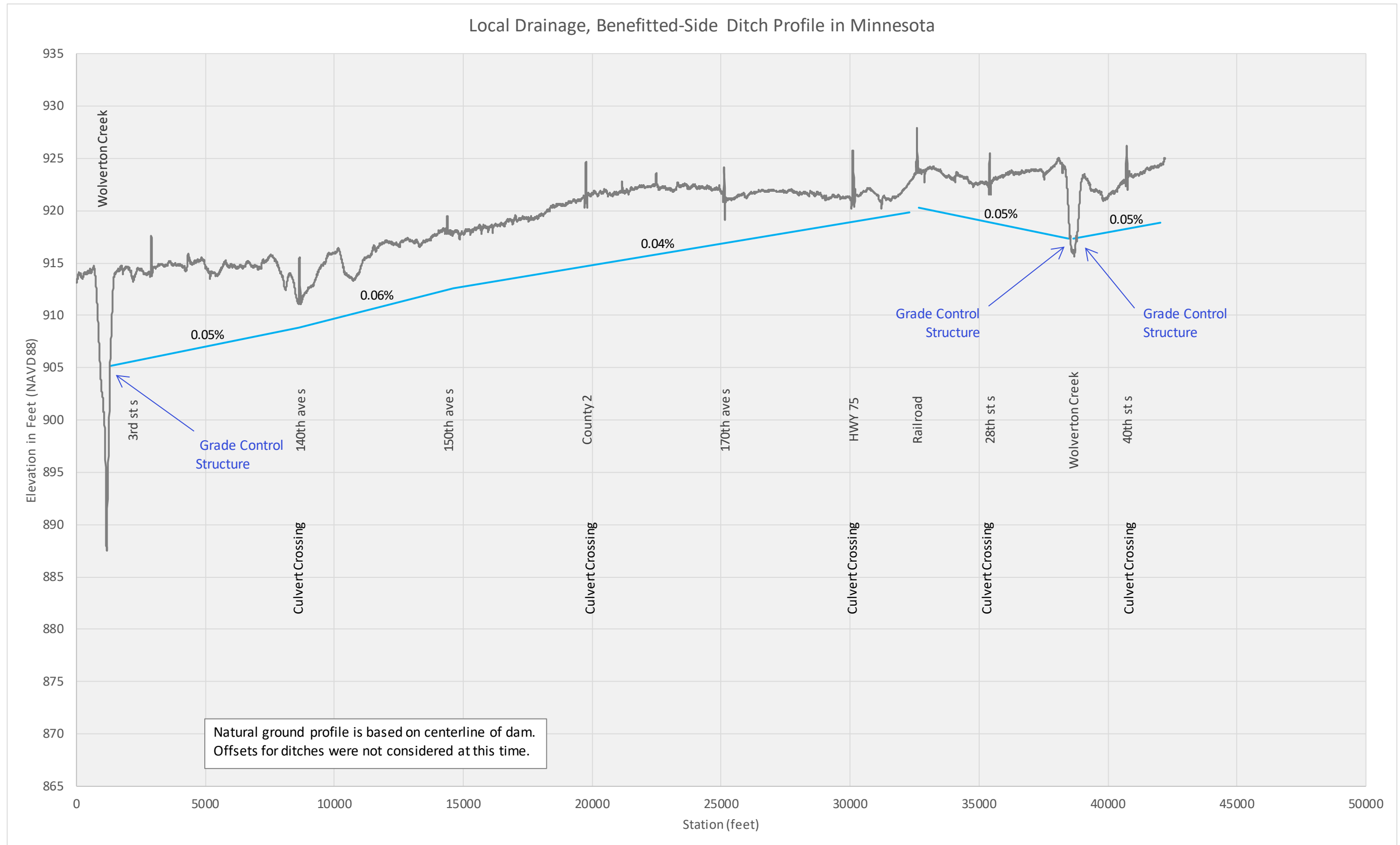


Figure 15. PMF Pool, Water Surface Elevations

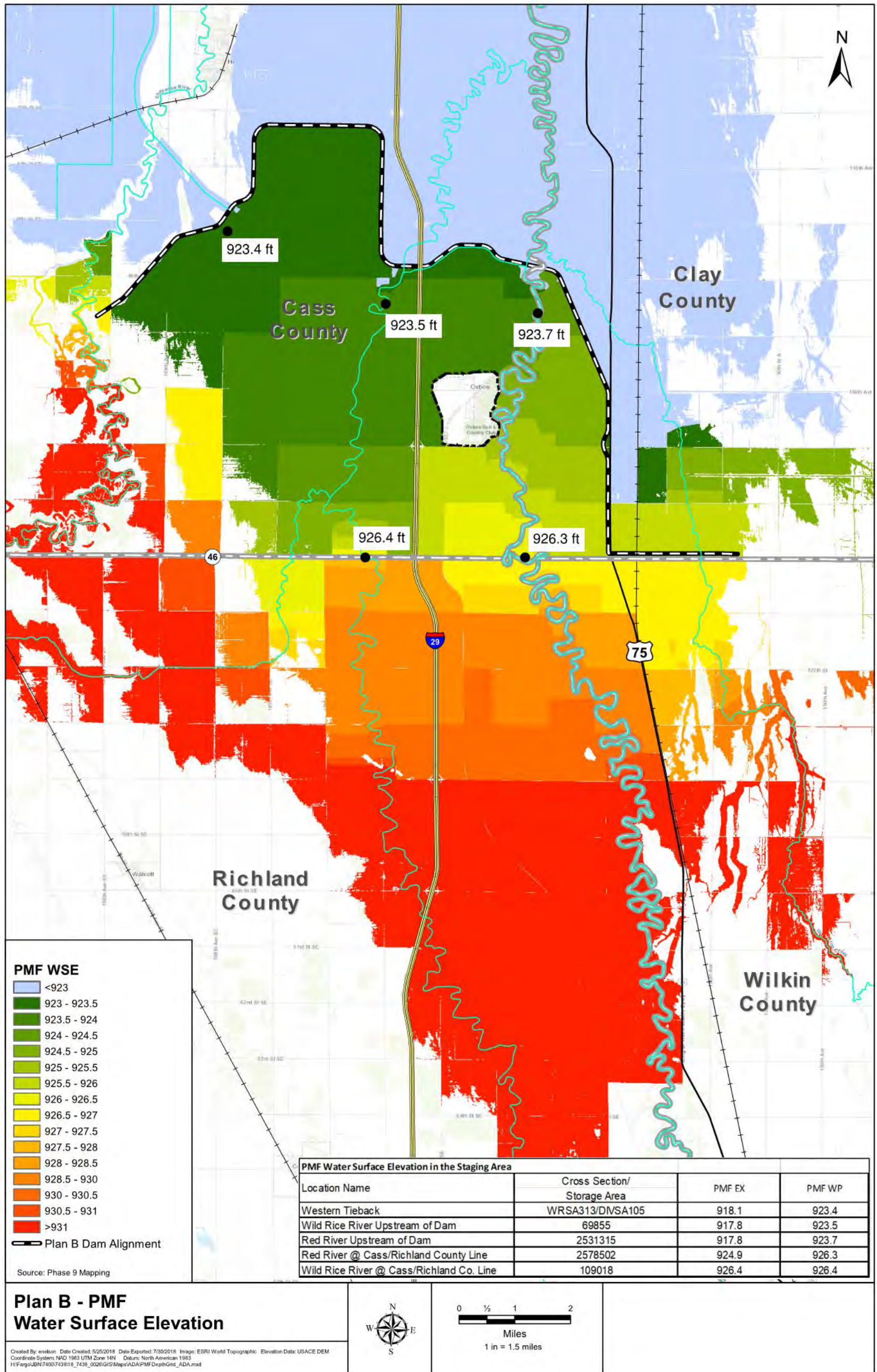


Figure 16: Floodplain, Existing and Plan B, Entire Metro Area (with Emergency Measures), 10% ACE Event

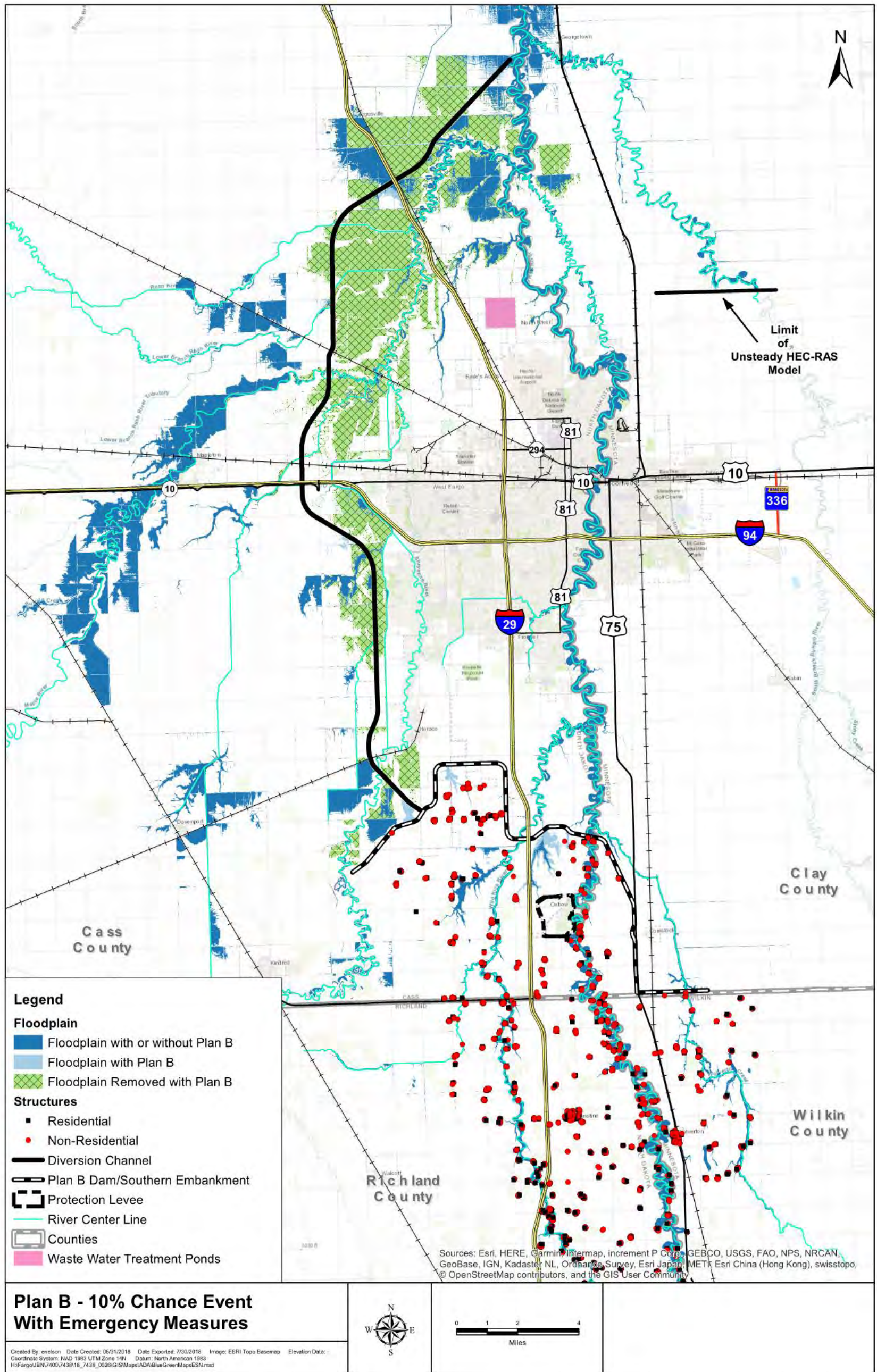


Figure 17: Floodplain, Existing and Plan B, Entire Metro Area (with Emergency Measures), 5% ACE Event

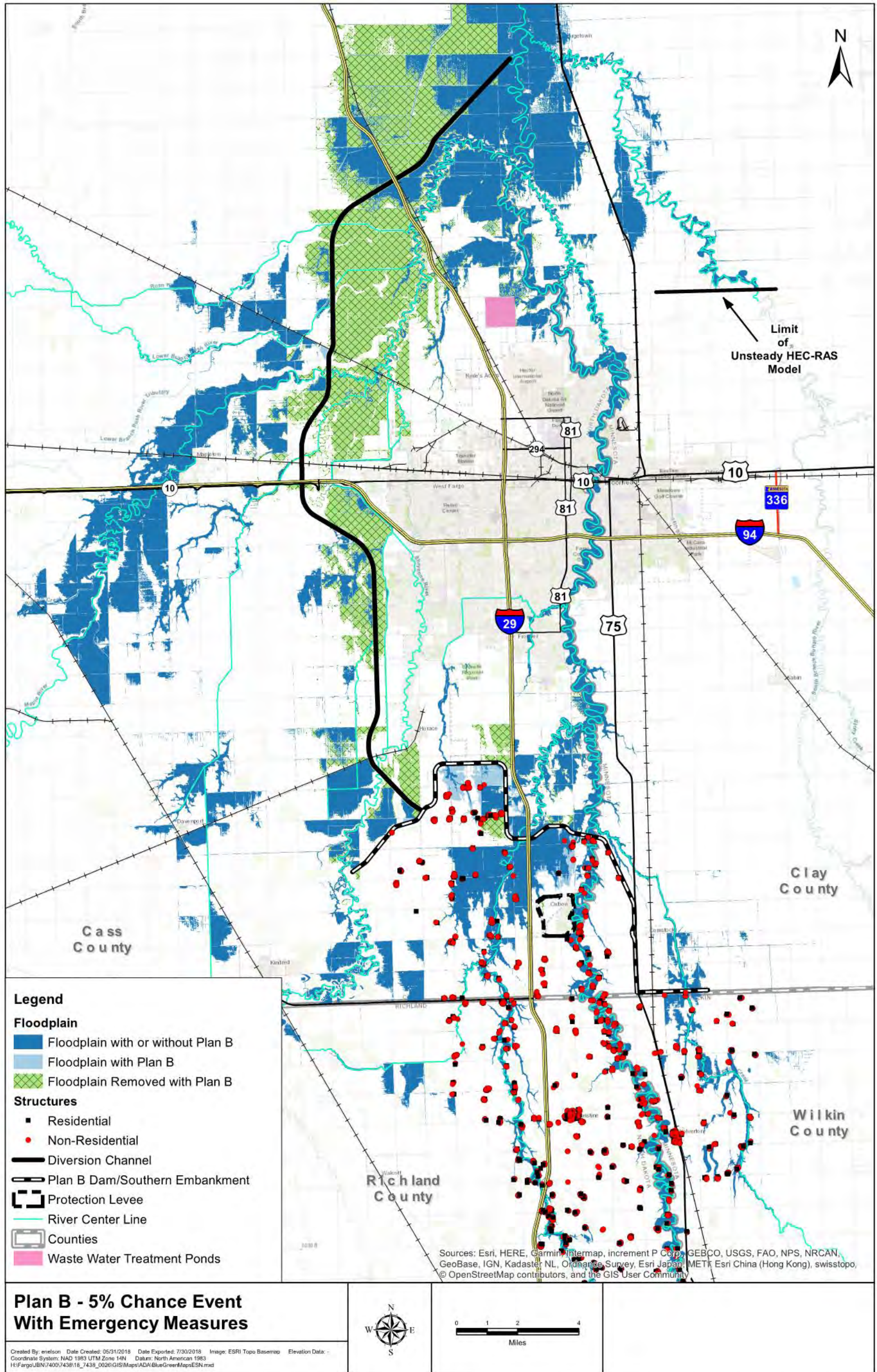


Figure 18: Floodplain, Existing and Plan B, Entire Metro Area (with Emergency Measures), 2% ACE Event

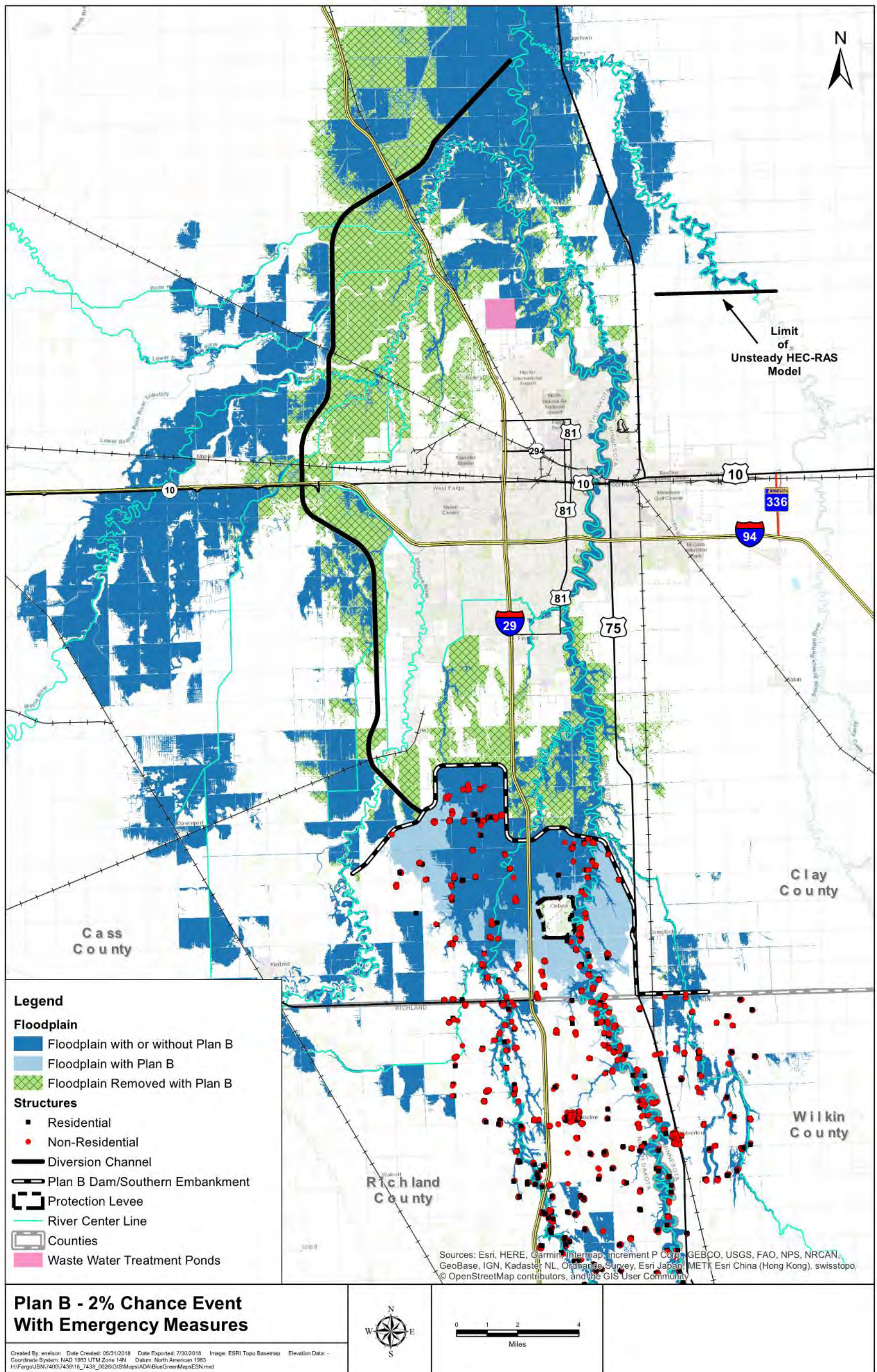


Figure 19: Floodplain, Existing and Plan B, Entire Metro Area (with Emergency Measures), 1% ACE Event

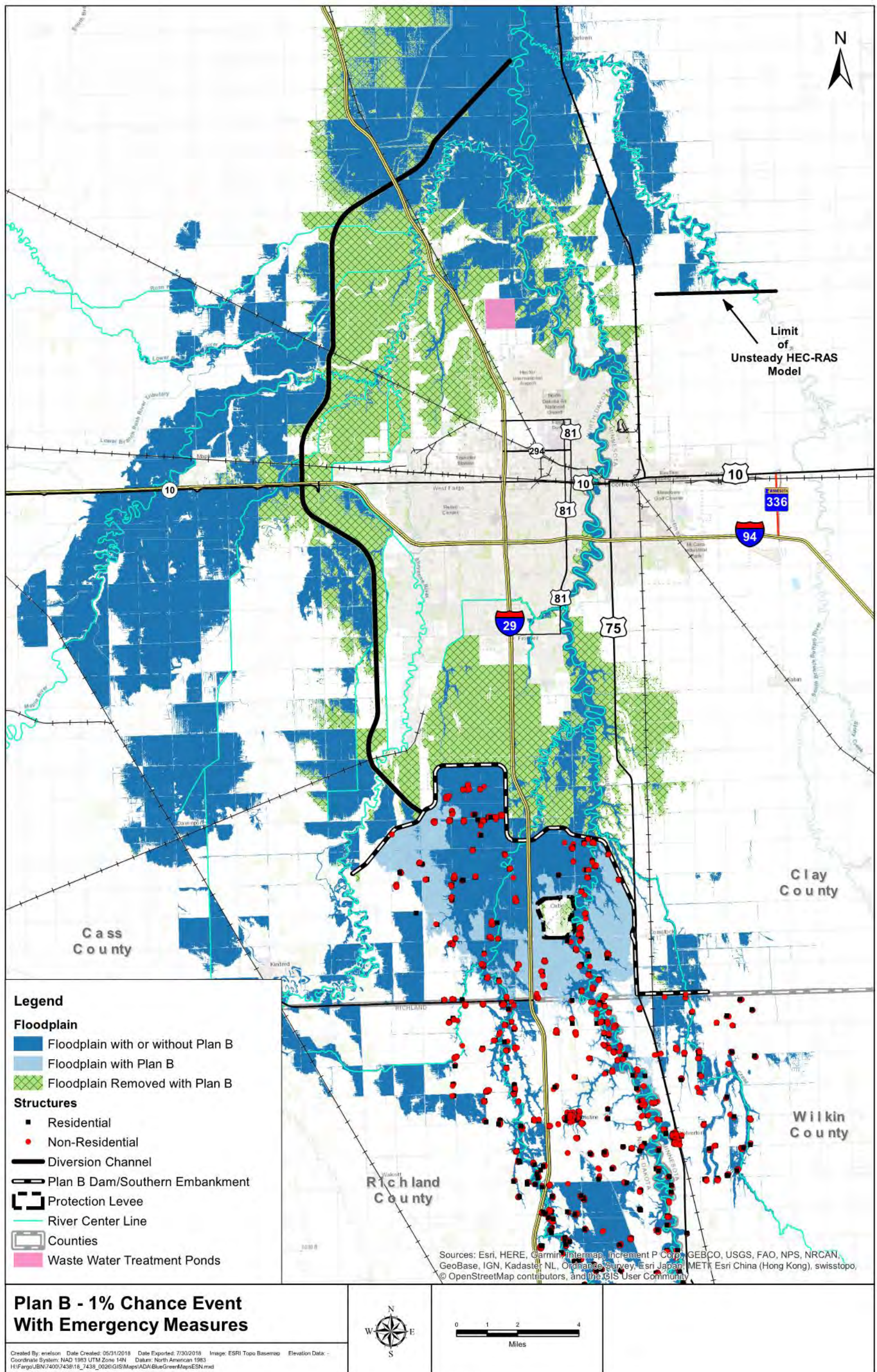


Figure 20: Floodplain, Existing and Plan B, Entire Metro Area (with Emergency Measures), 0.2% ACE Event

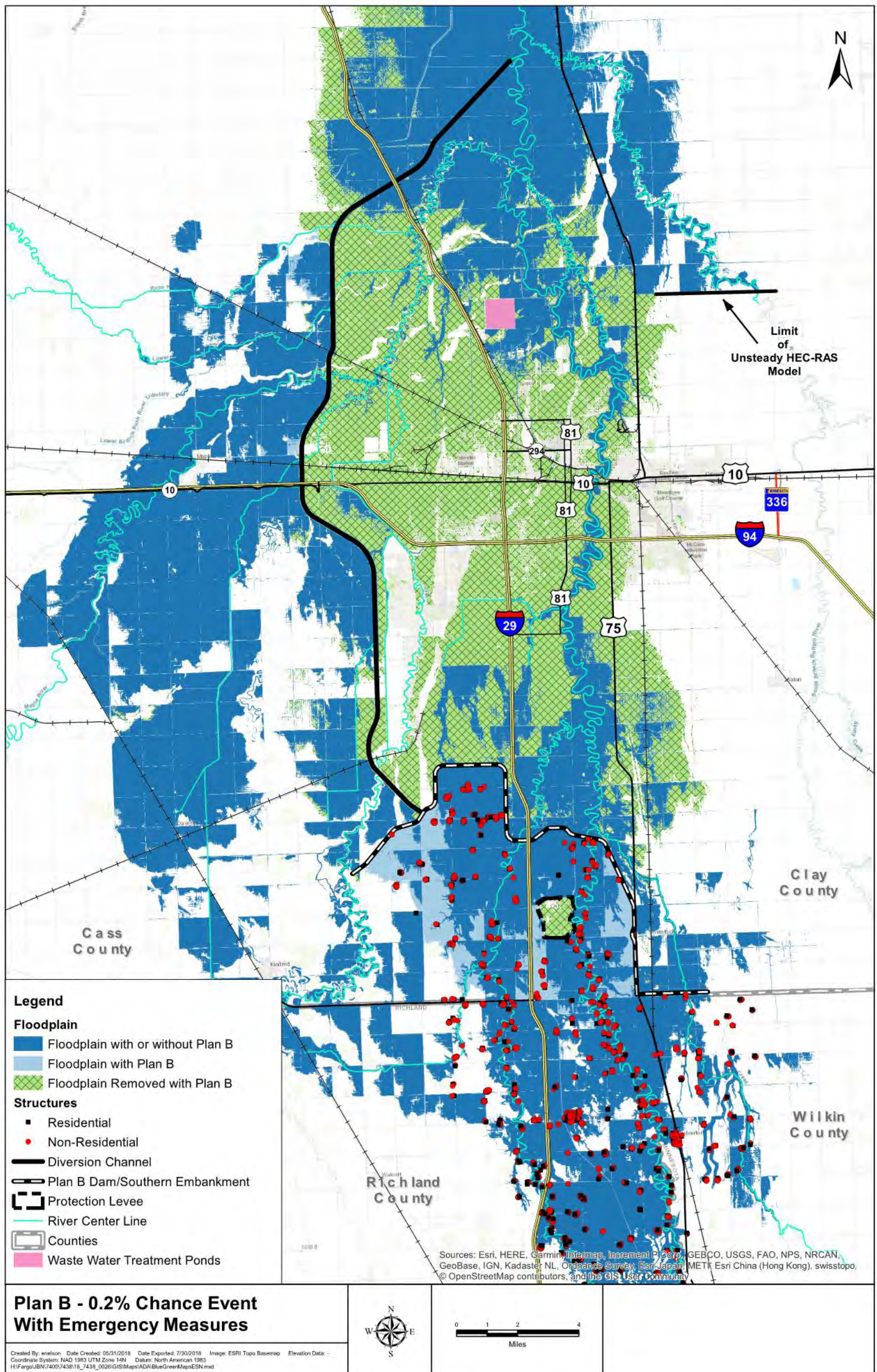


Figure 21: Floodplain, Existing and Plan B, Entire Metro Area (without Emergency Measures), 10% ACE Event

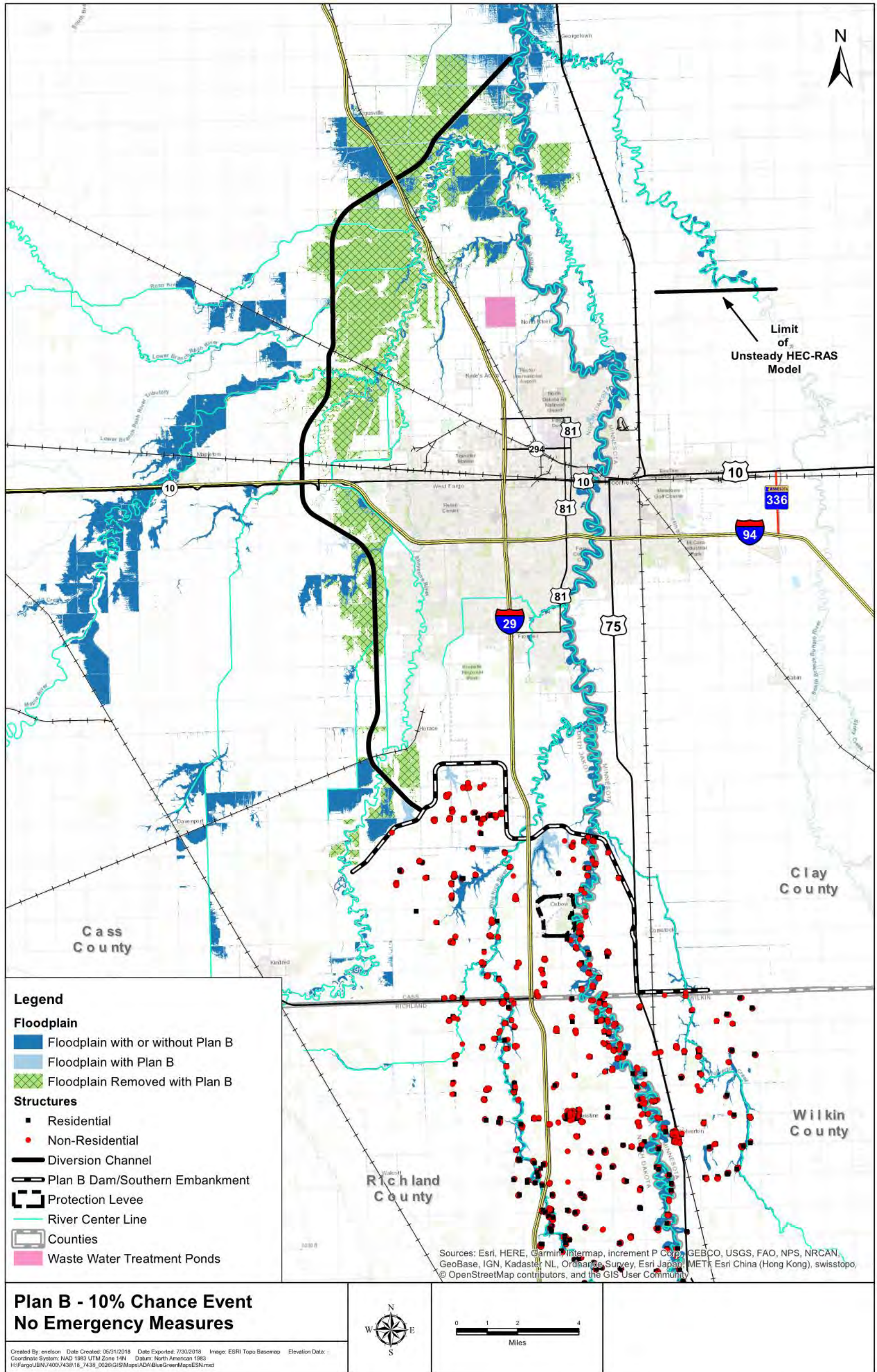


Figure 22: Floodplain, Existing and Plan B, Entire Metro Area (without Emergency Measures), 5% ACE Event

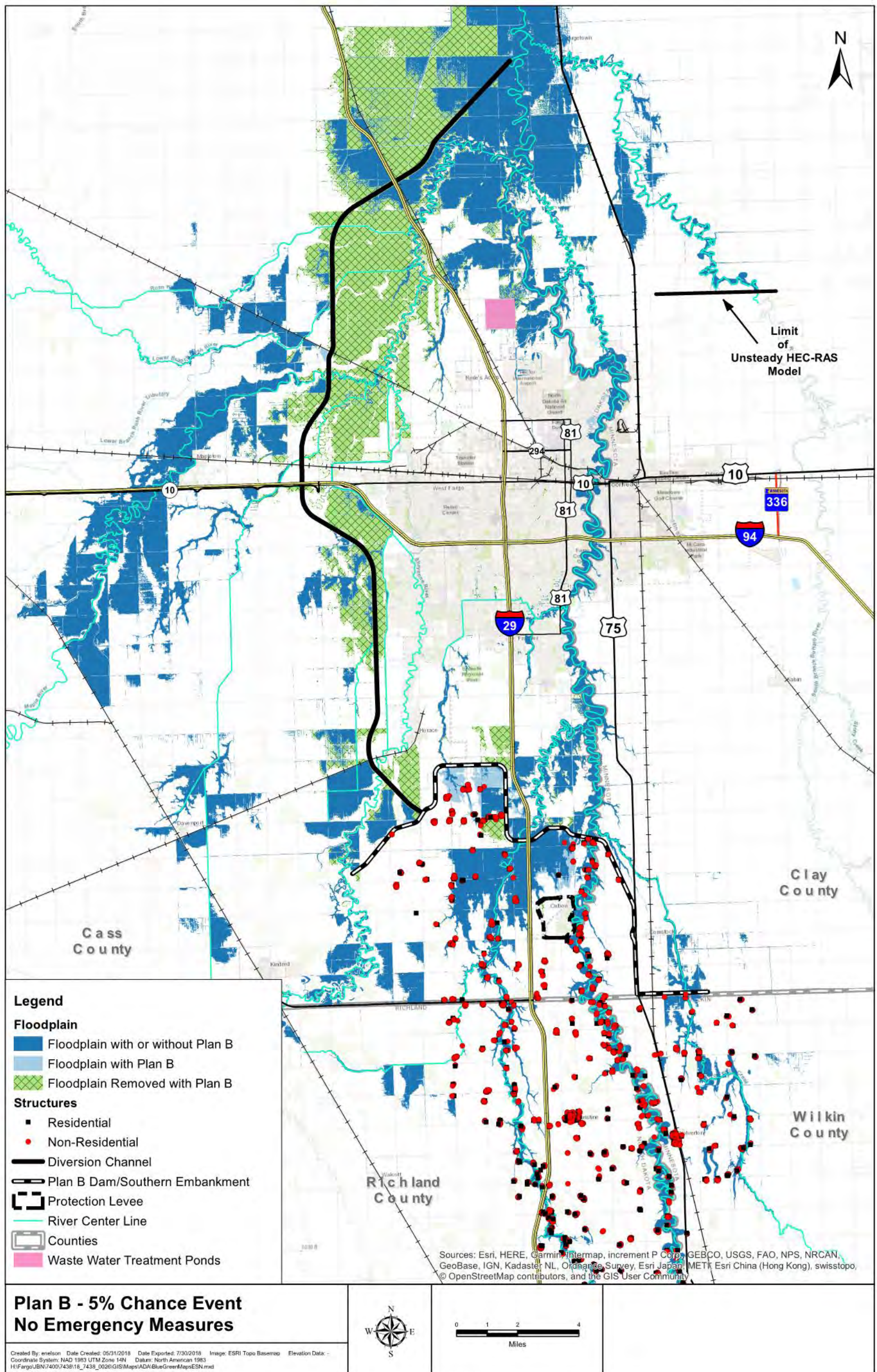


Figure 23: Floodplain, Existing and Plan B, Entire Metro Area (without Emergency Measures), 2% ACE Event

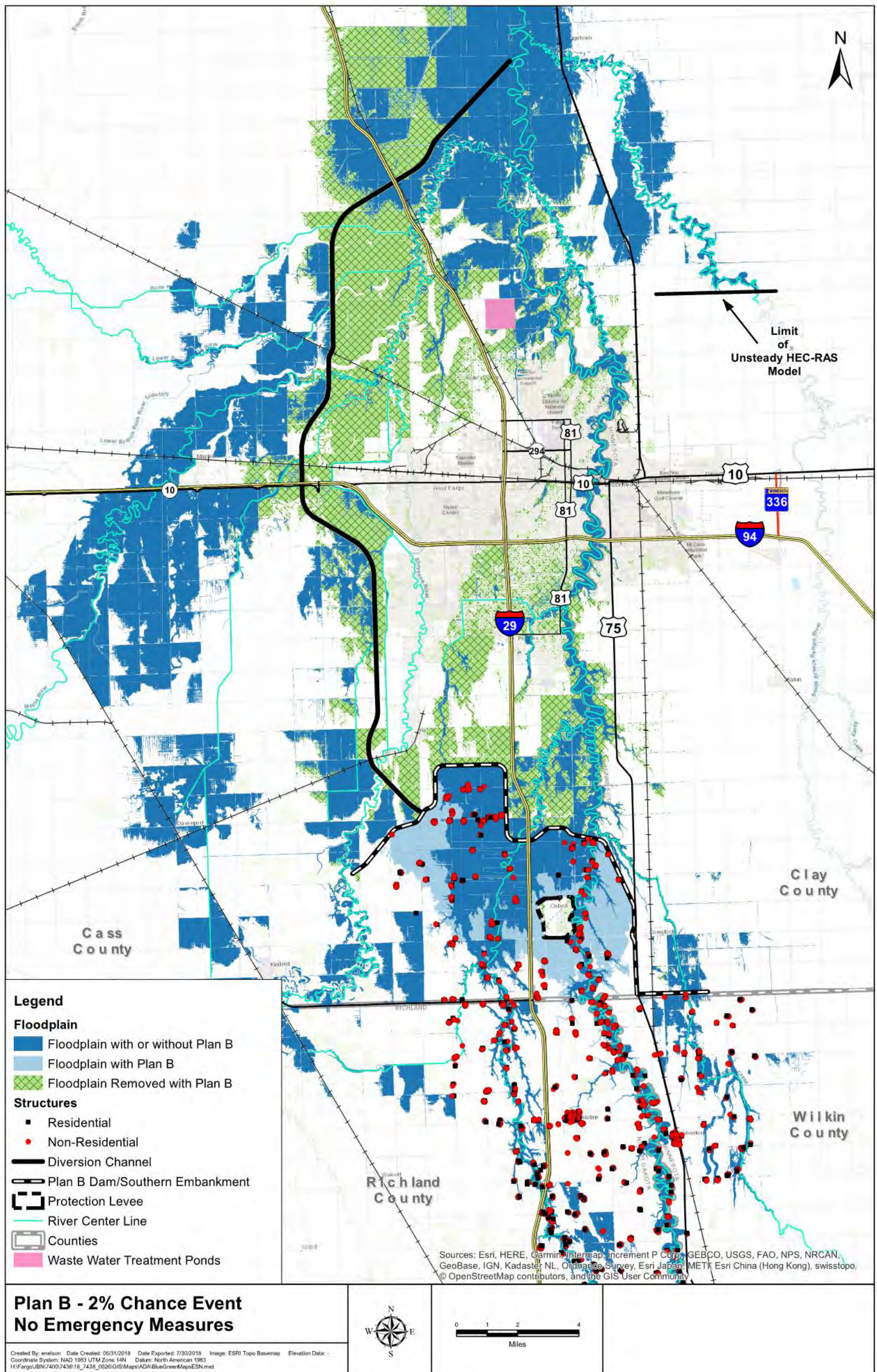


Figure 24: Floodplain, Existing and Plan B, Entire Metro Area (without Emergency Measures), 1% ACE Event

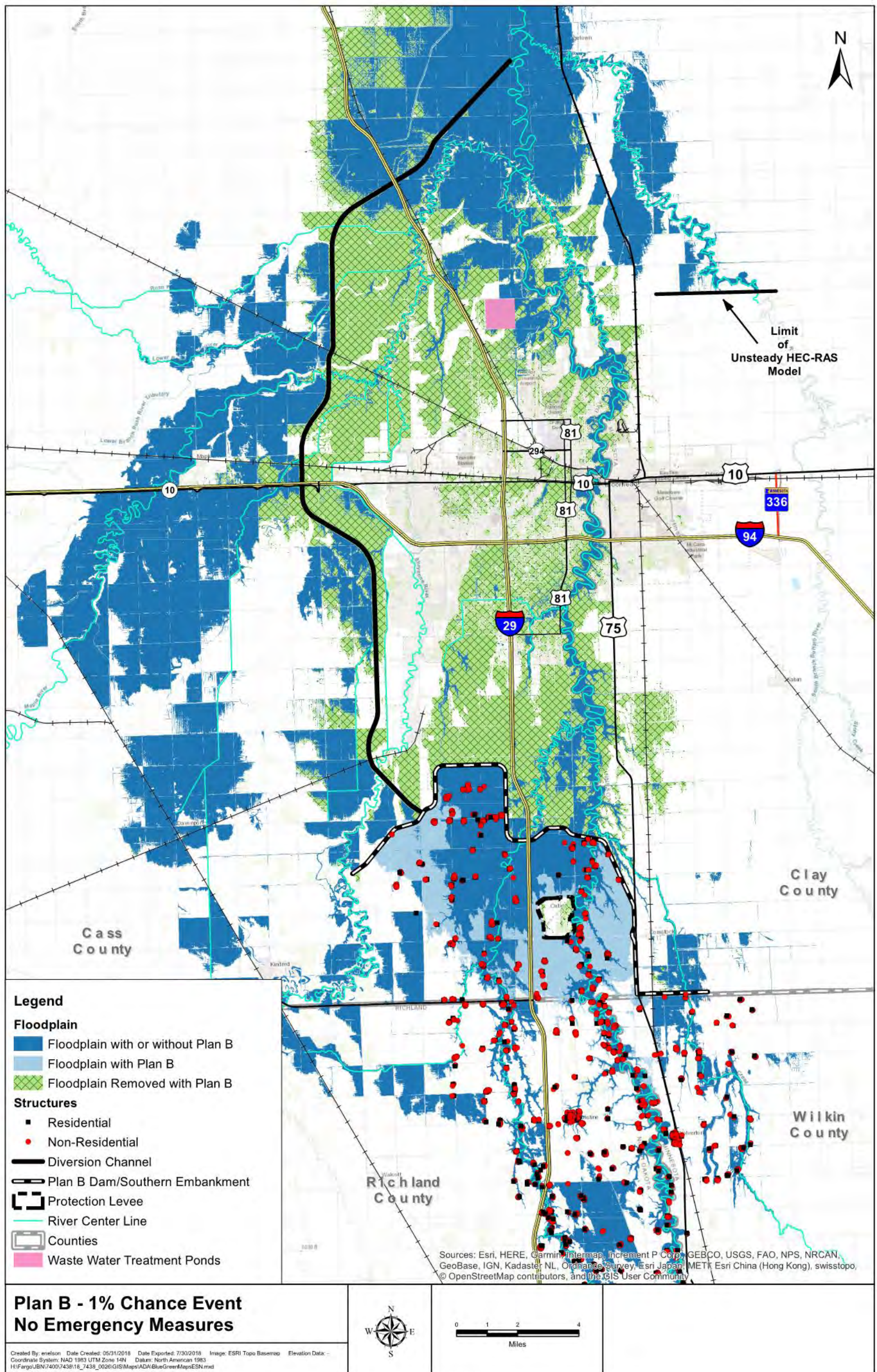


Figure 25: Floodplain, Existing and Plan B, Entire Metro Area (without Emergency Measures), 0.2% ACE Event

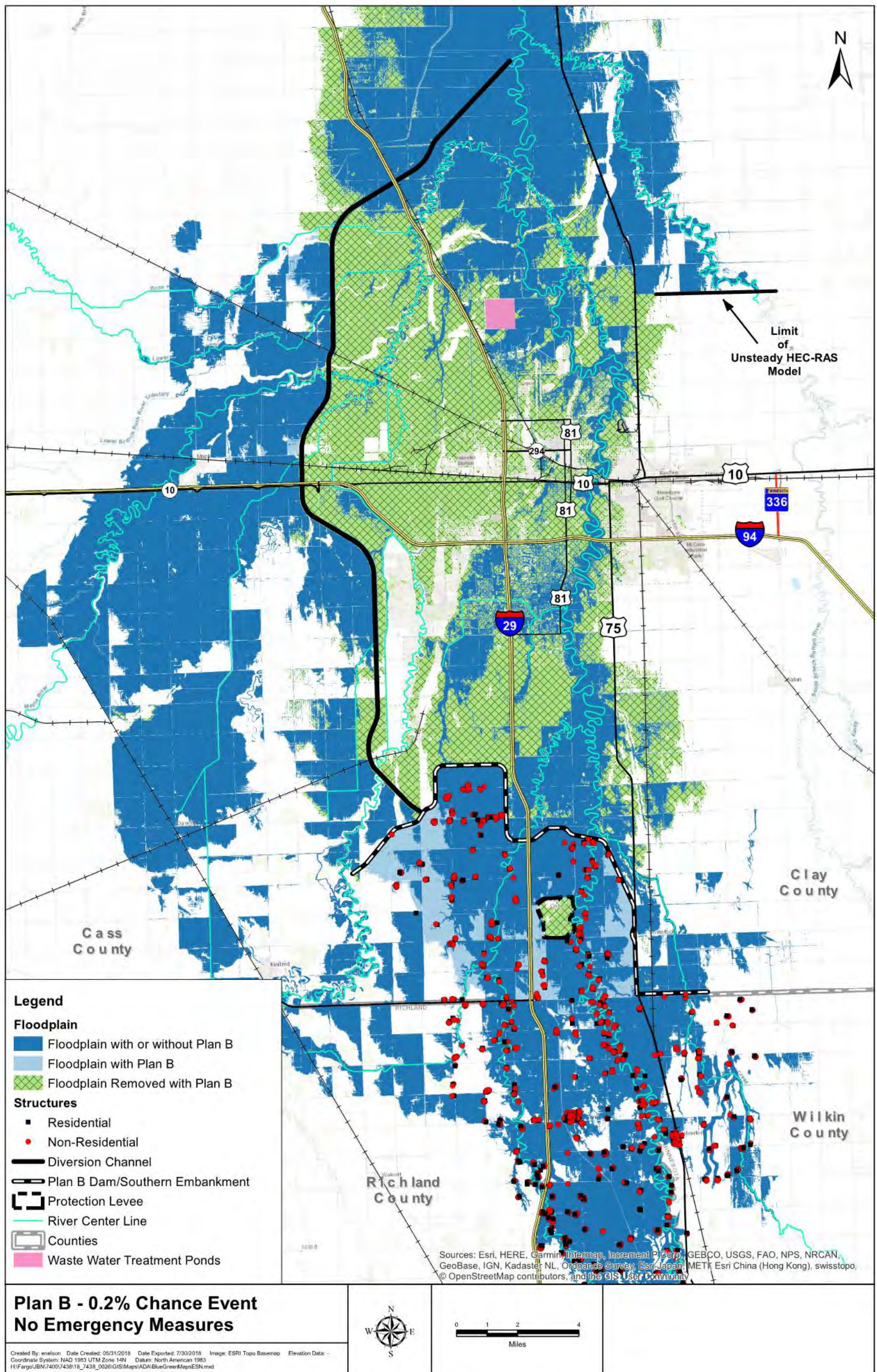


Figure 26: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (with Emergency Measures), 10% ACE Event

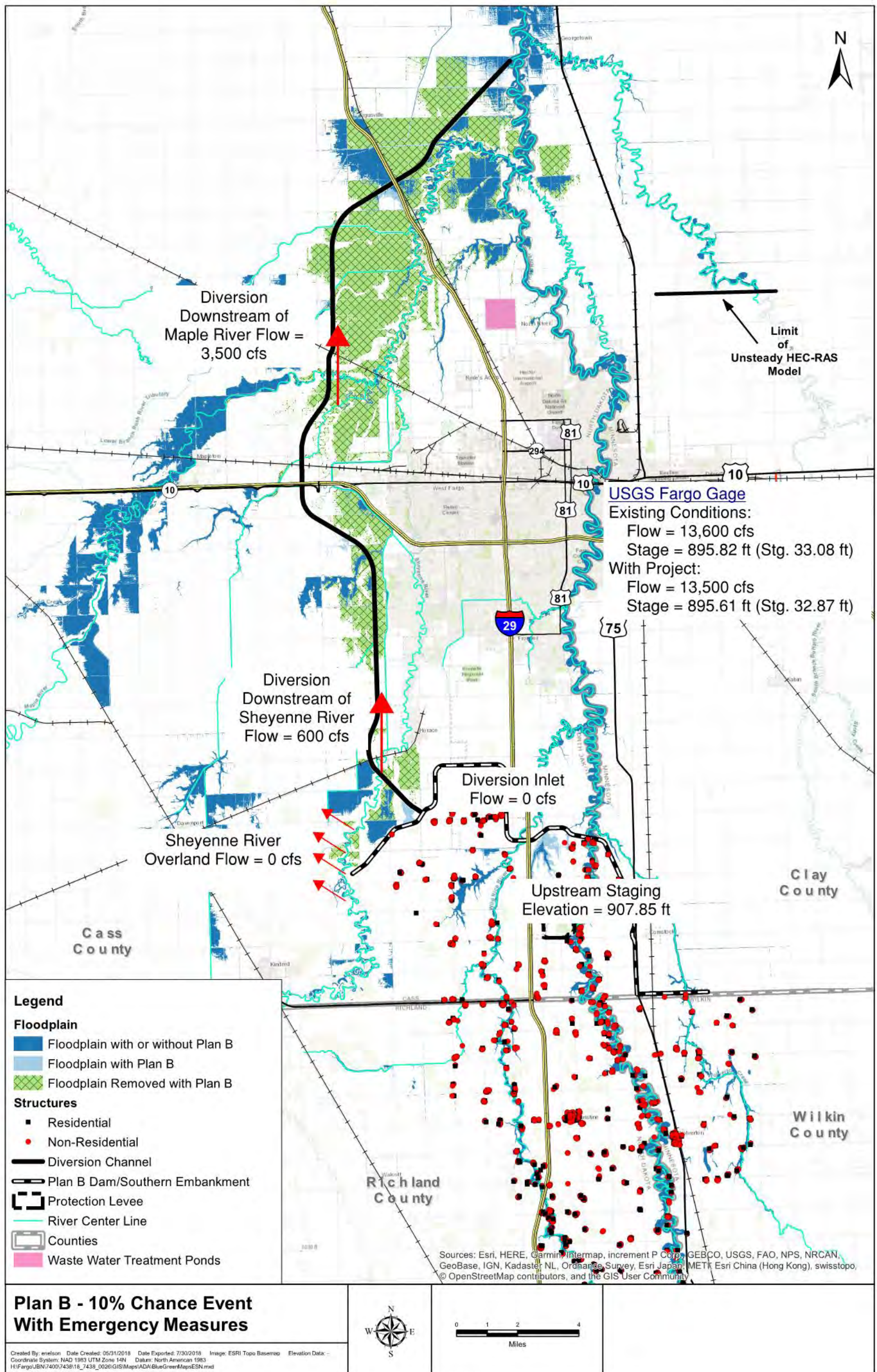


Figure 27: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (with Emergency Measures), 5% ACE Event

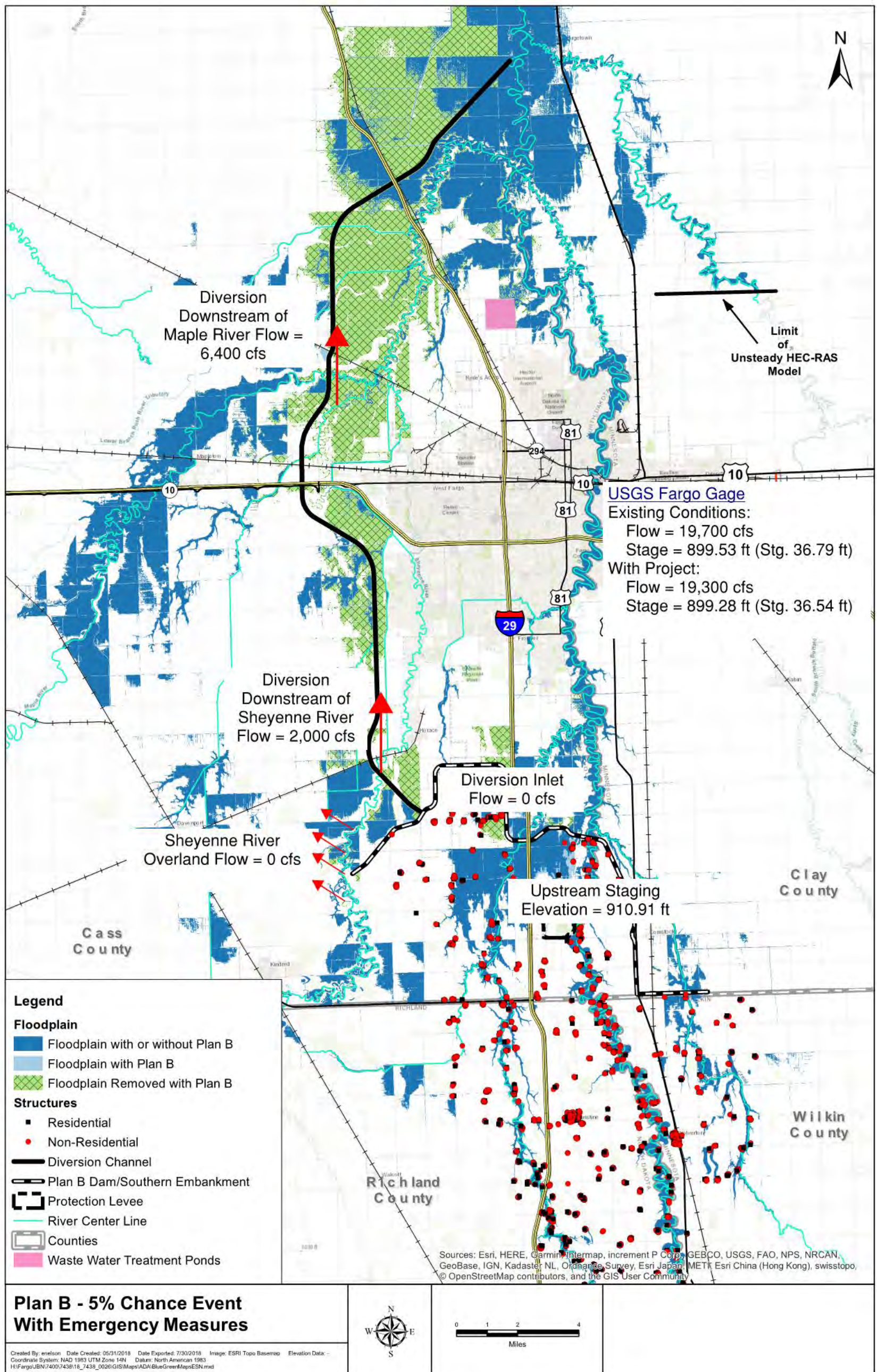


Figure 28: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (with Emergency Measures), 2% ACE Event

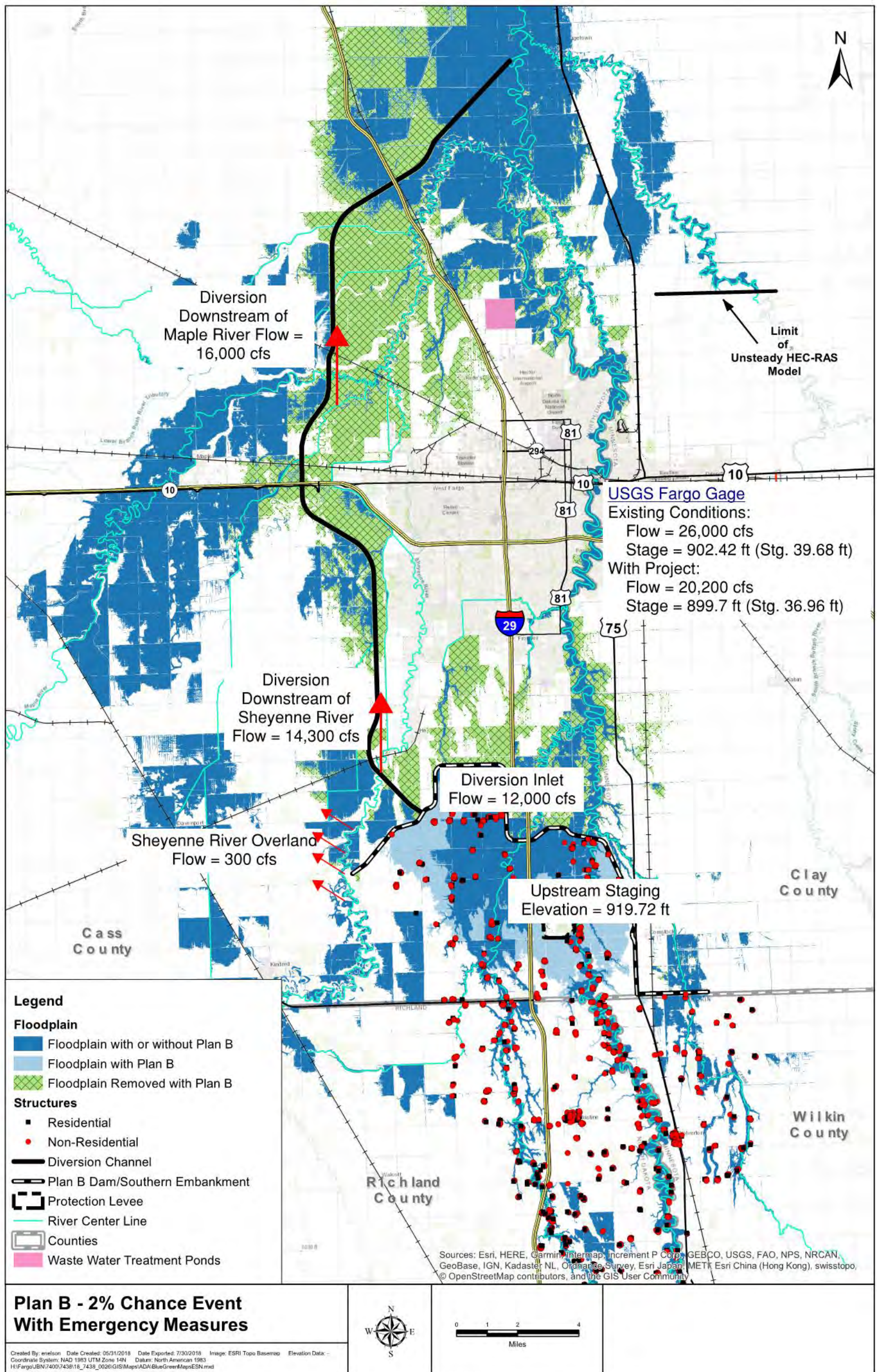


Figure 29: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (with Emergency Measures), 1% ACE Event

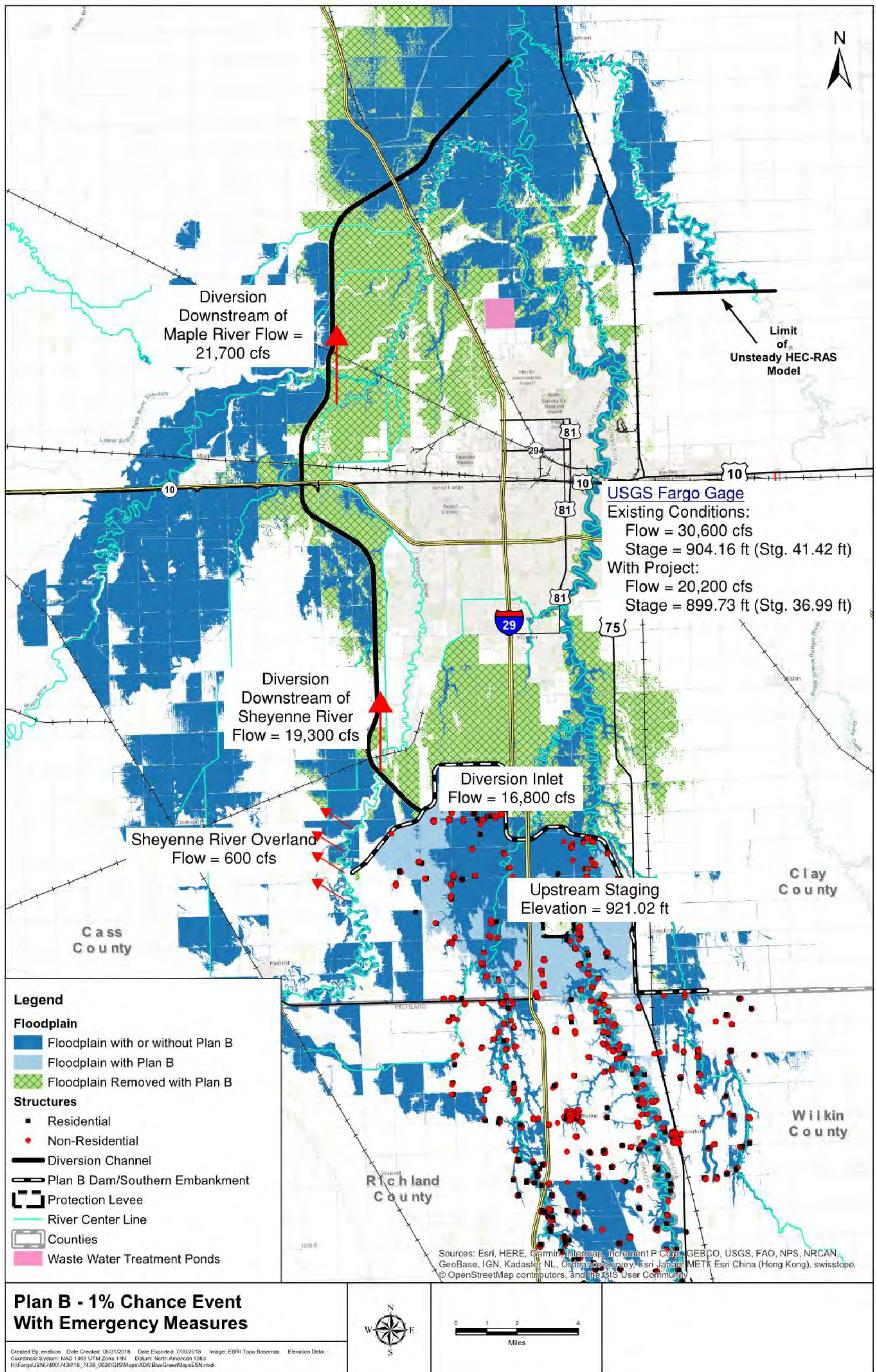


Figure 30: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (with Emergency Measures), 0.2% ACE Event

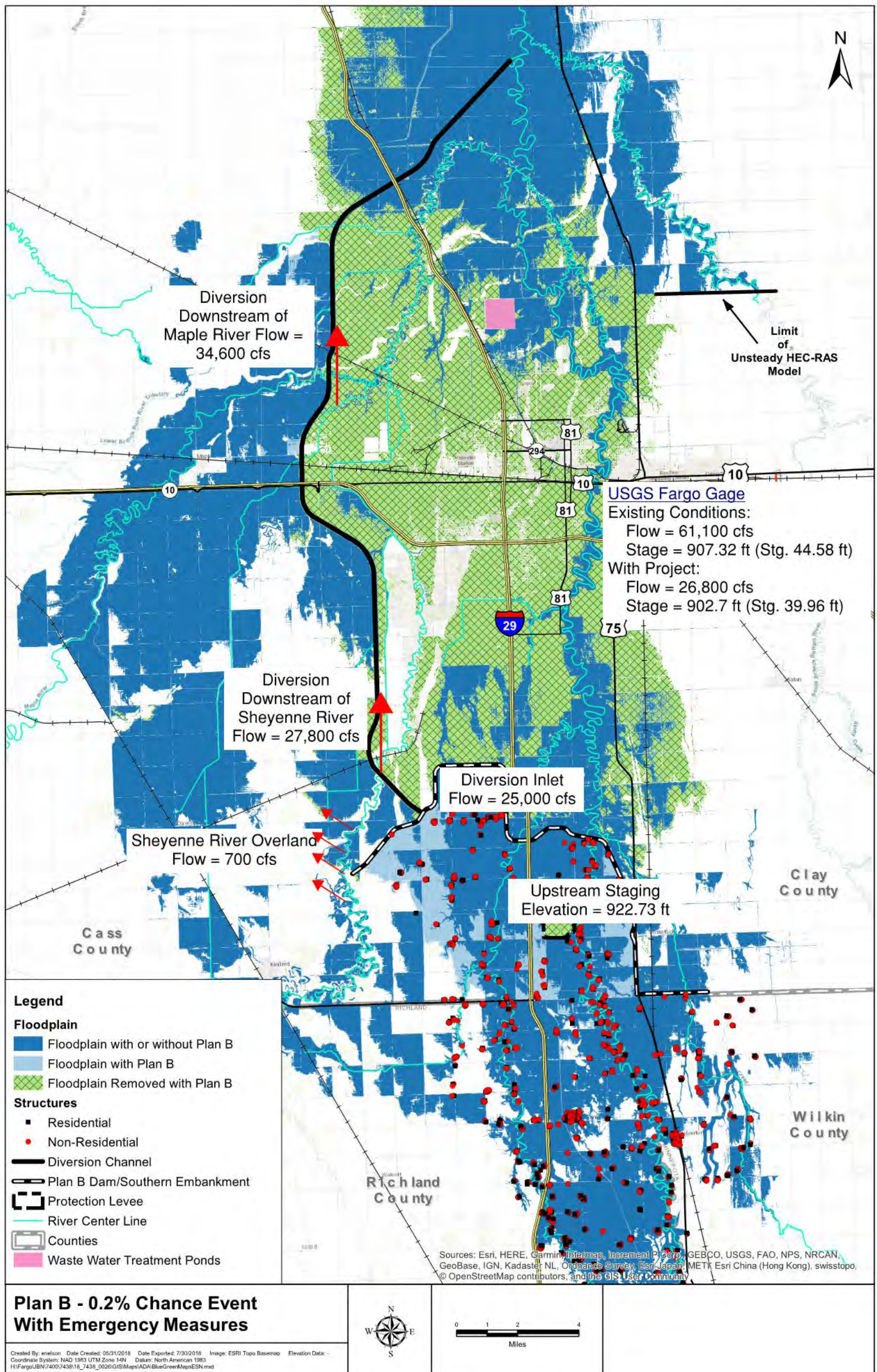


Figure 31: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (without Emergency Measures), 10% ACE Event

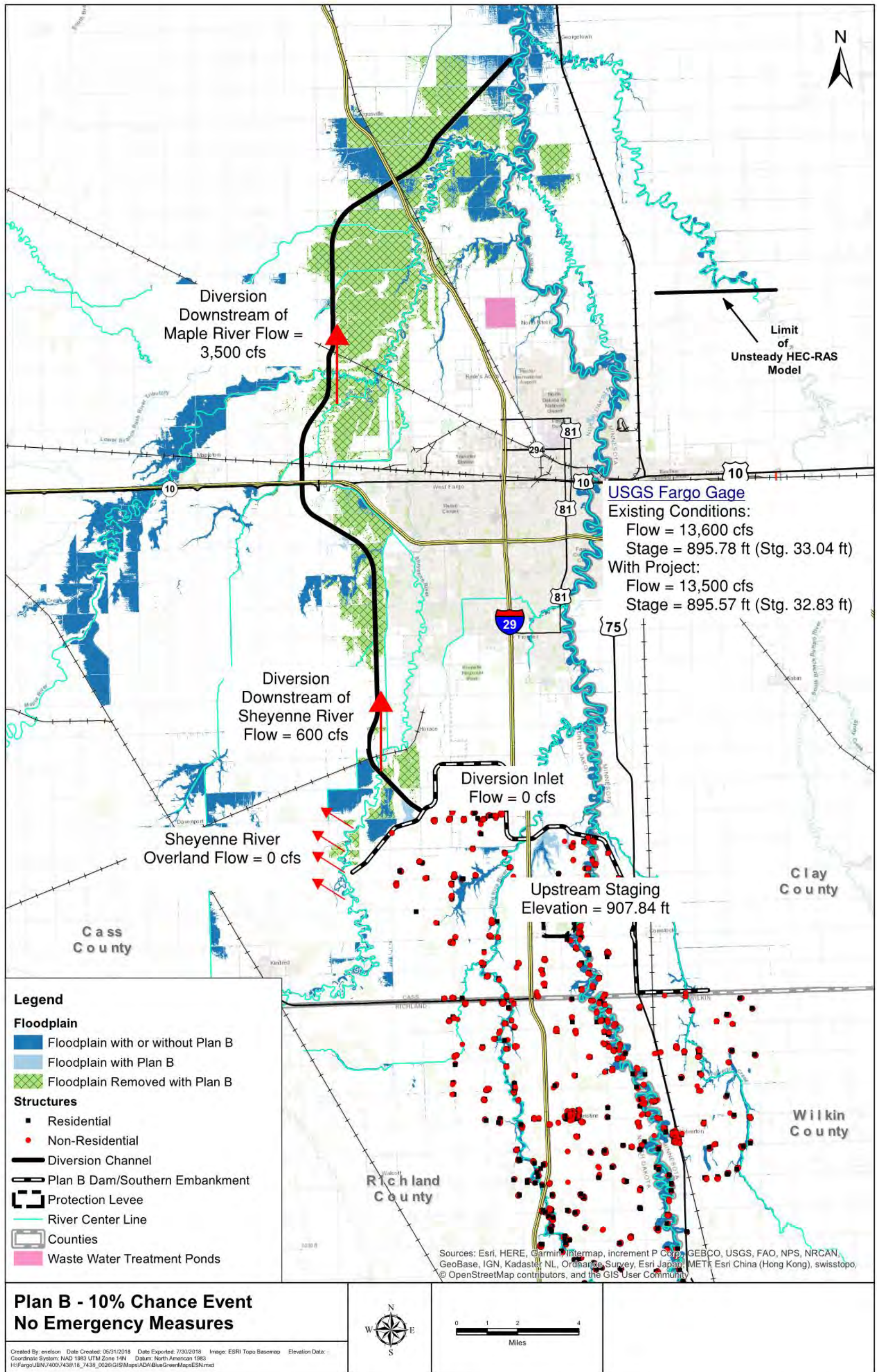


Figure 32: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (without Emergency Measures), 5% ACE Event

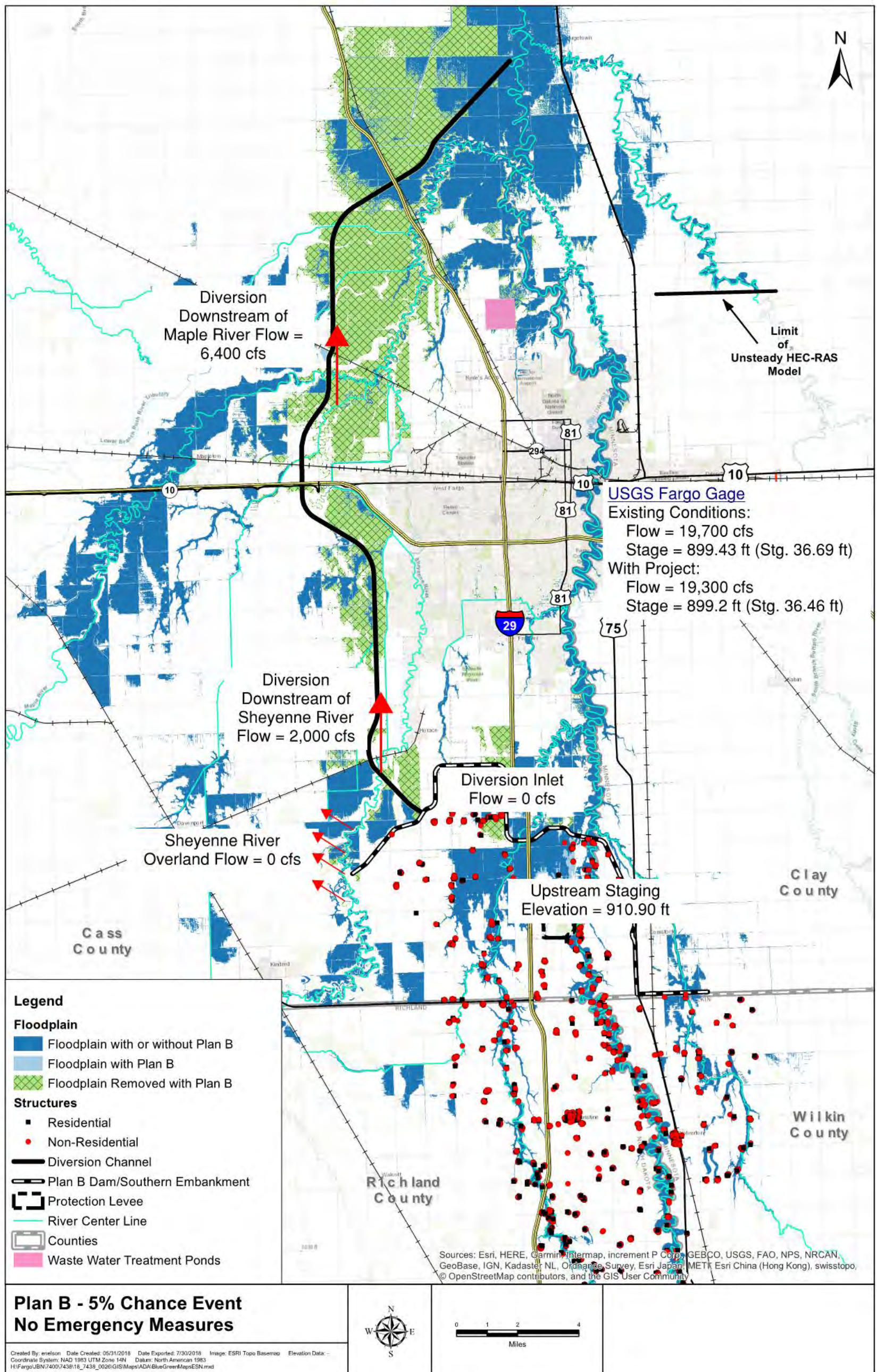


Figure 33: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (without Emergency Measures), 2% ACE Event

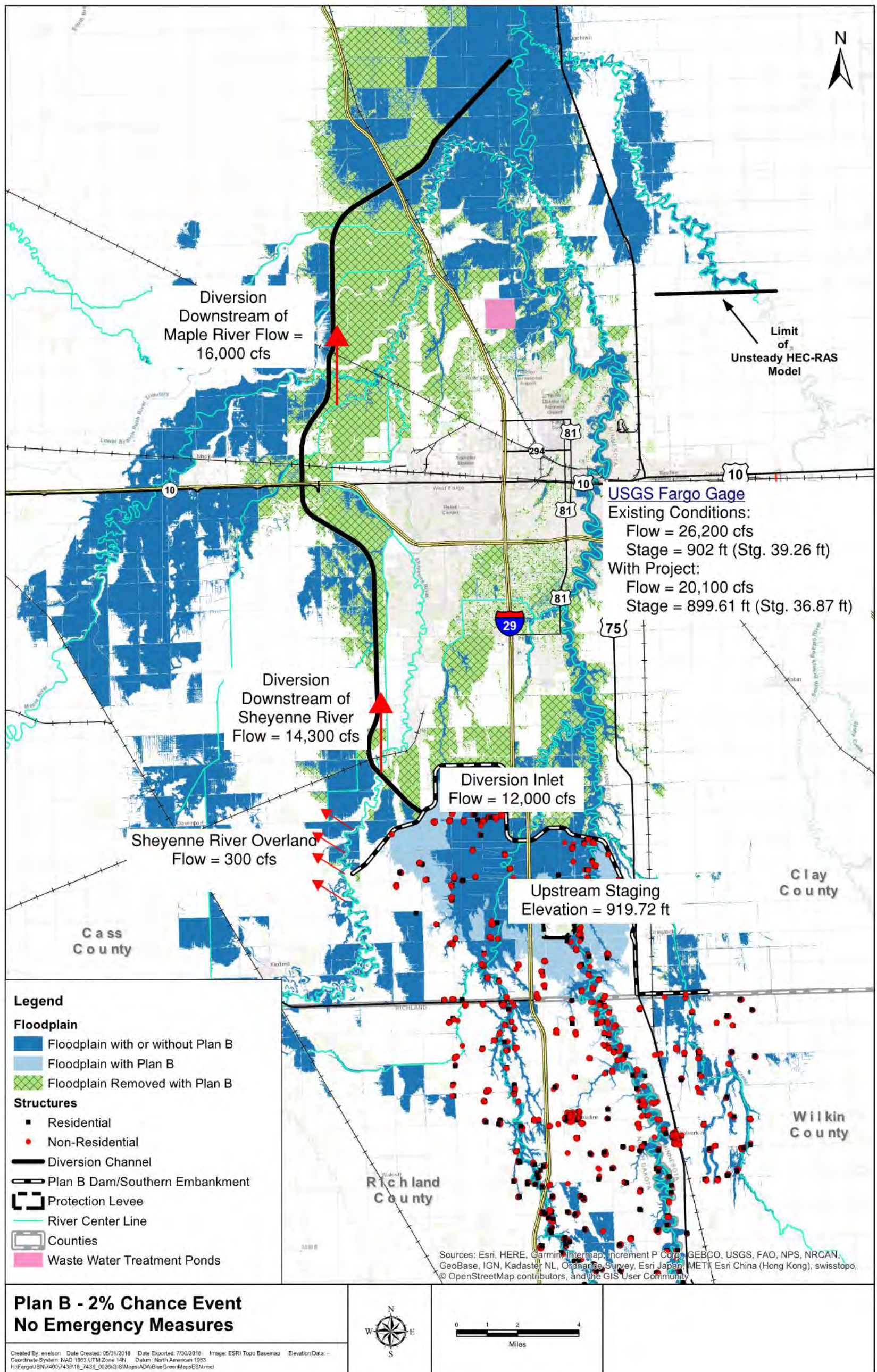


Figure 34: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (without Emergency Measures), 1% ACE Event

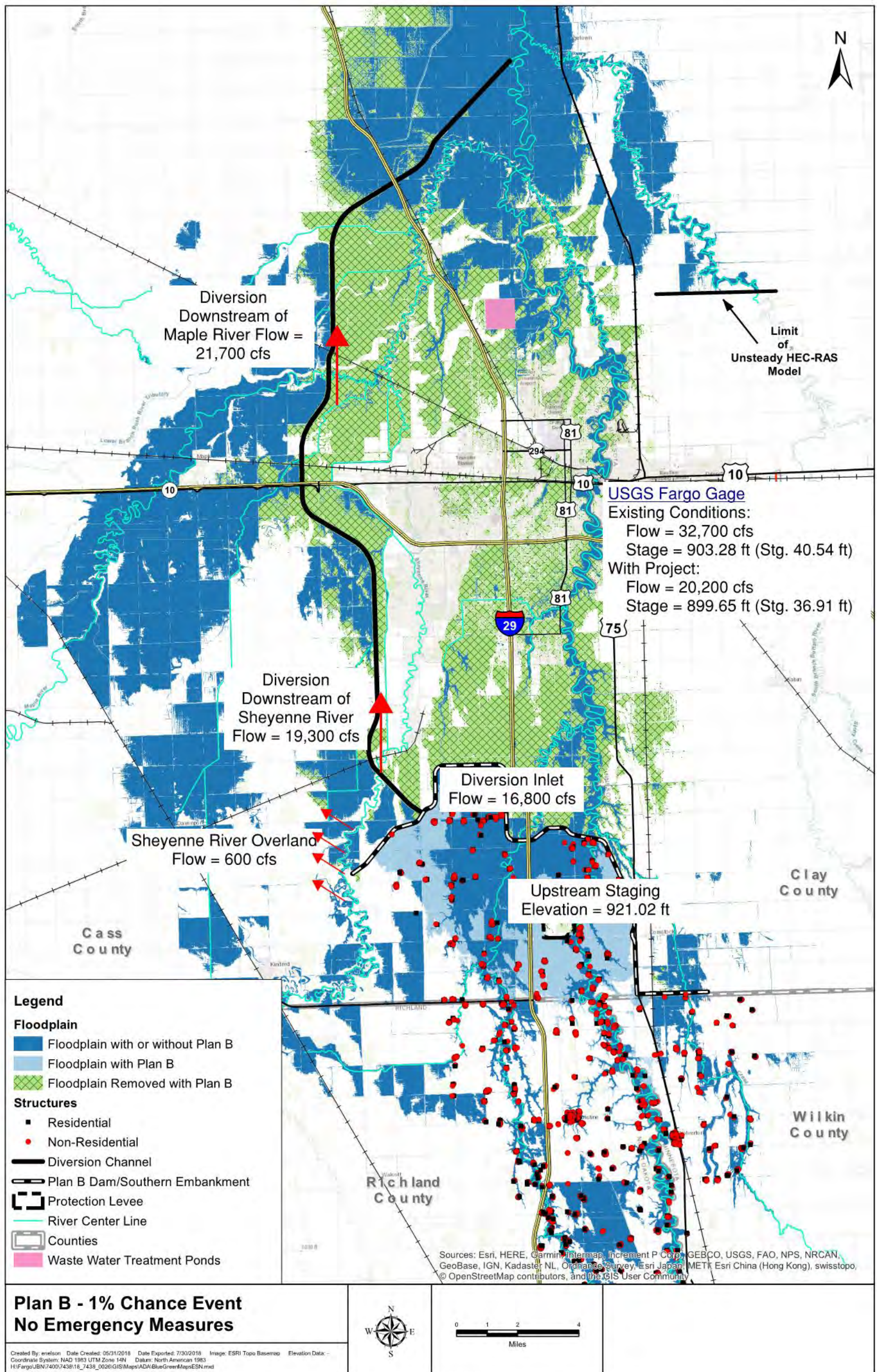


Figure 35: Floodplain/Flow/Stage, Existing and Plan B, Entire Metro Area (without Emergency Measures), 0.2% ACE Event

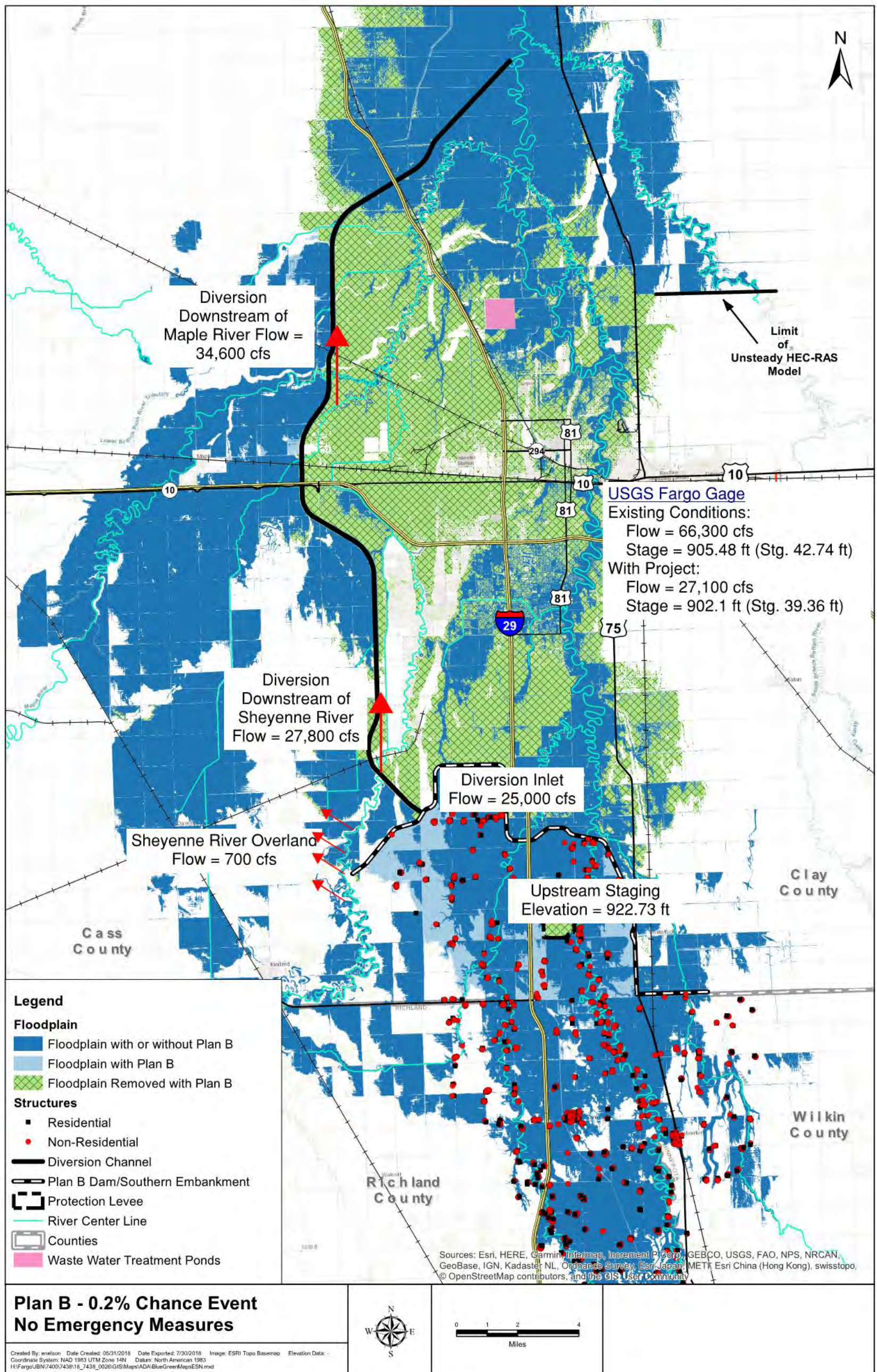


Figure 36: Depth Difference Pool, 10% ACE Event

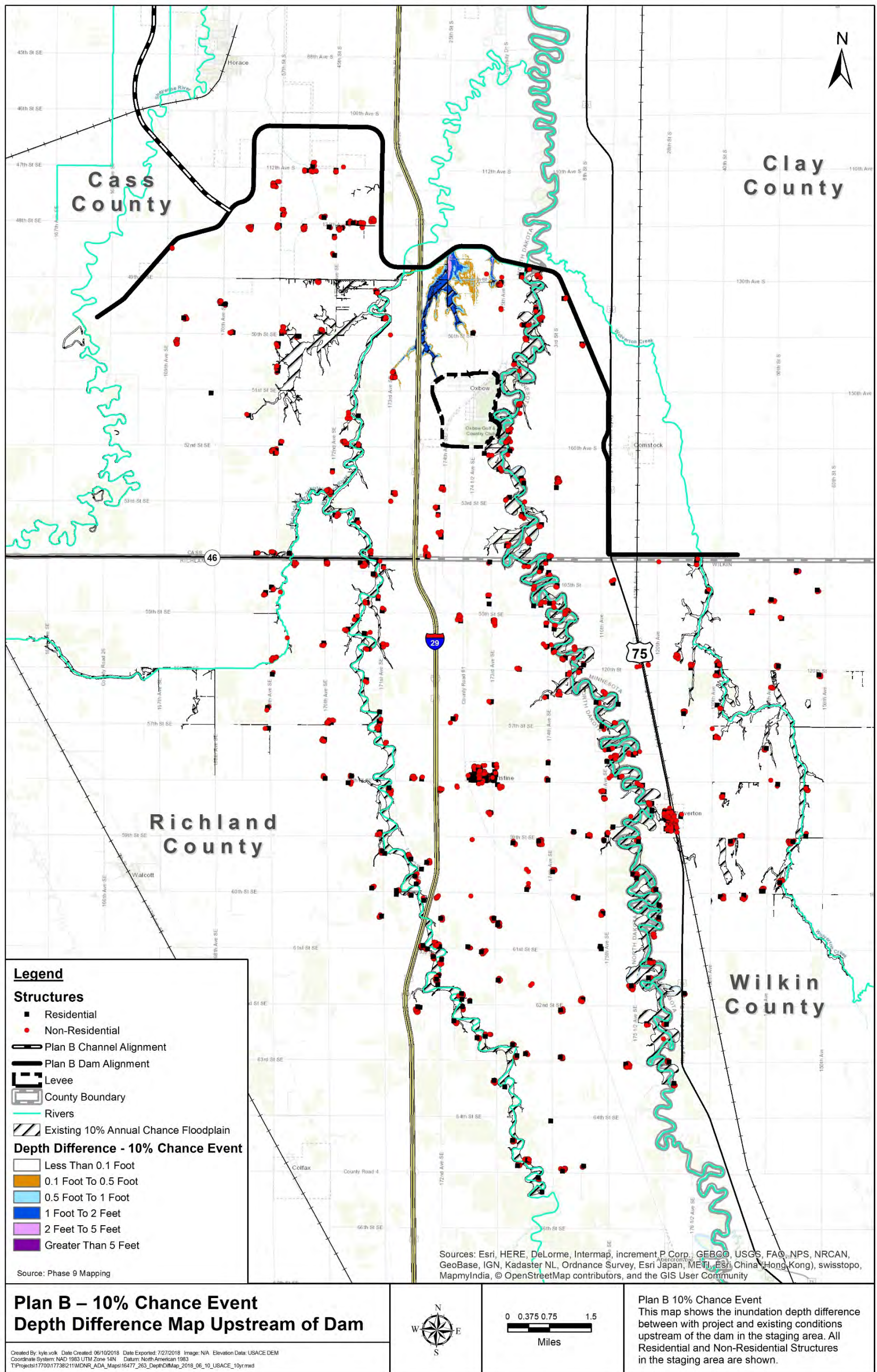


Figure 37: Depth Difference Pool, 5% ACE Event

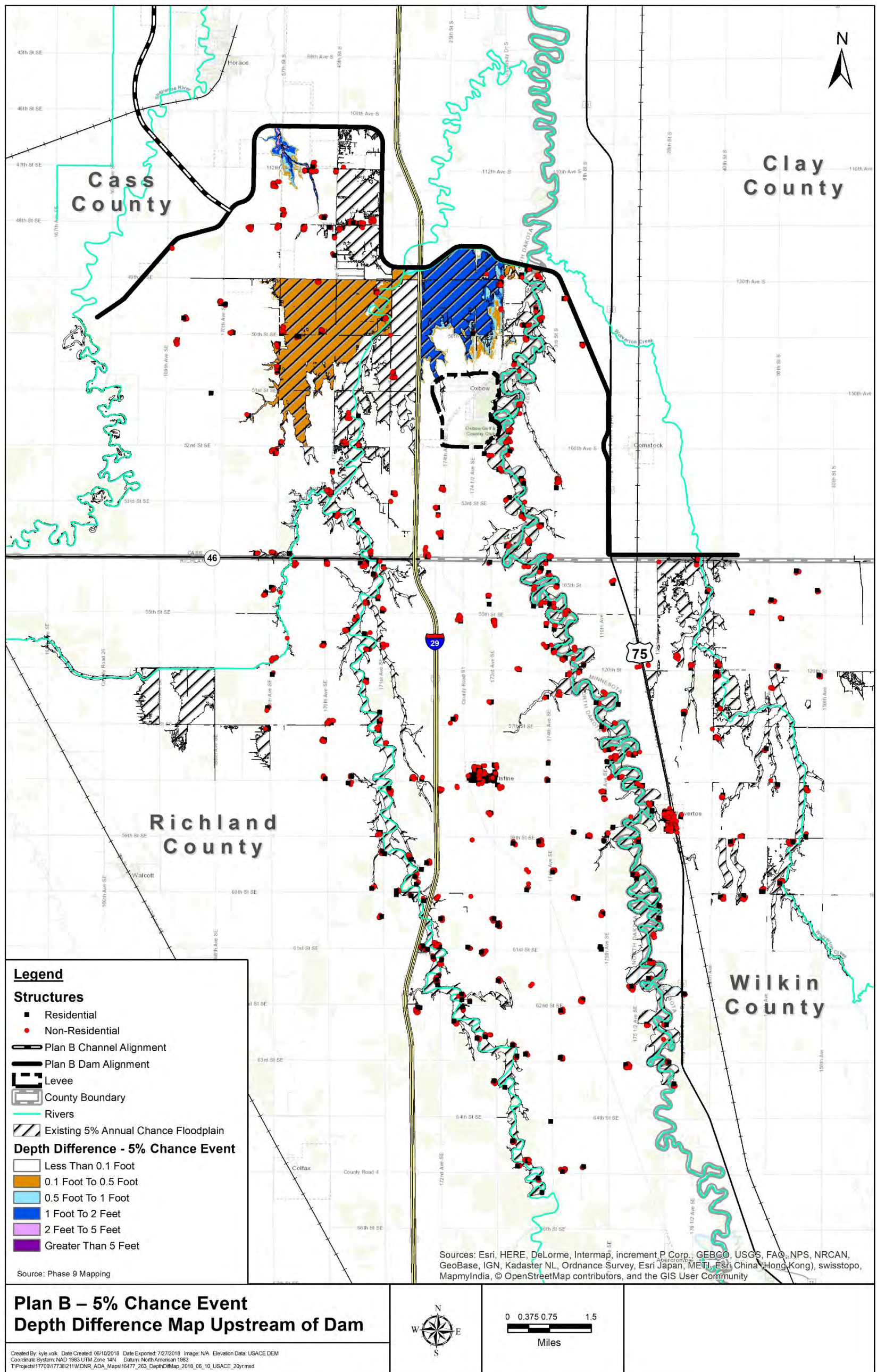


Figure 38: Depth Difference Pool, 2% ACE Event

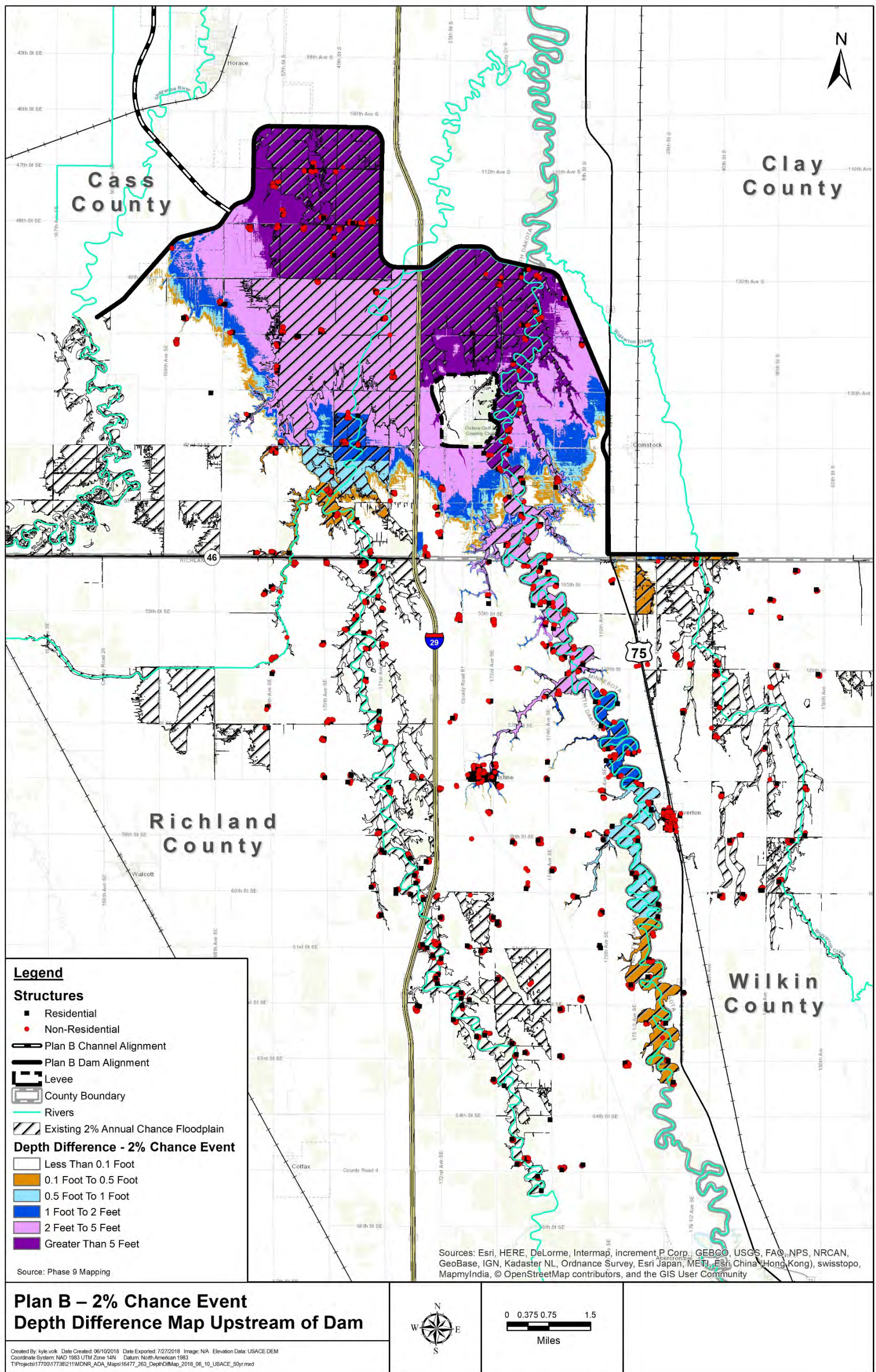
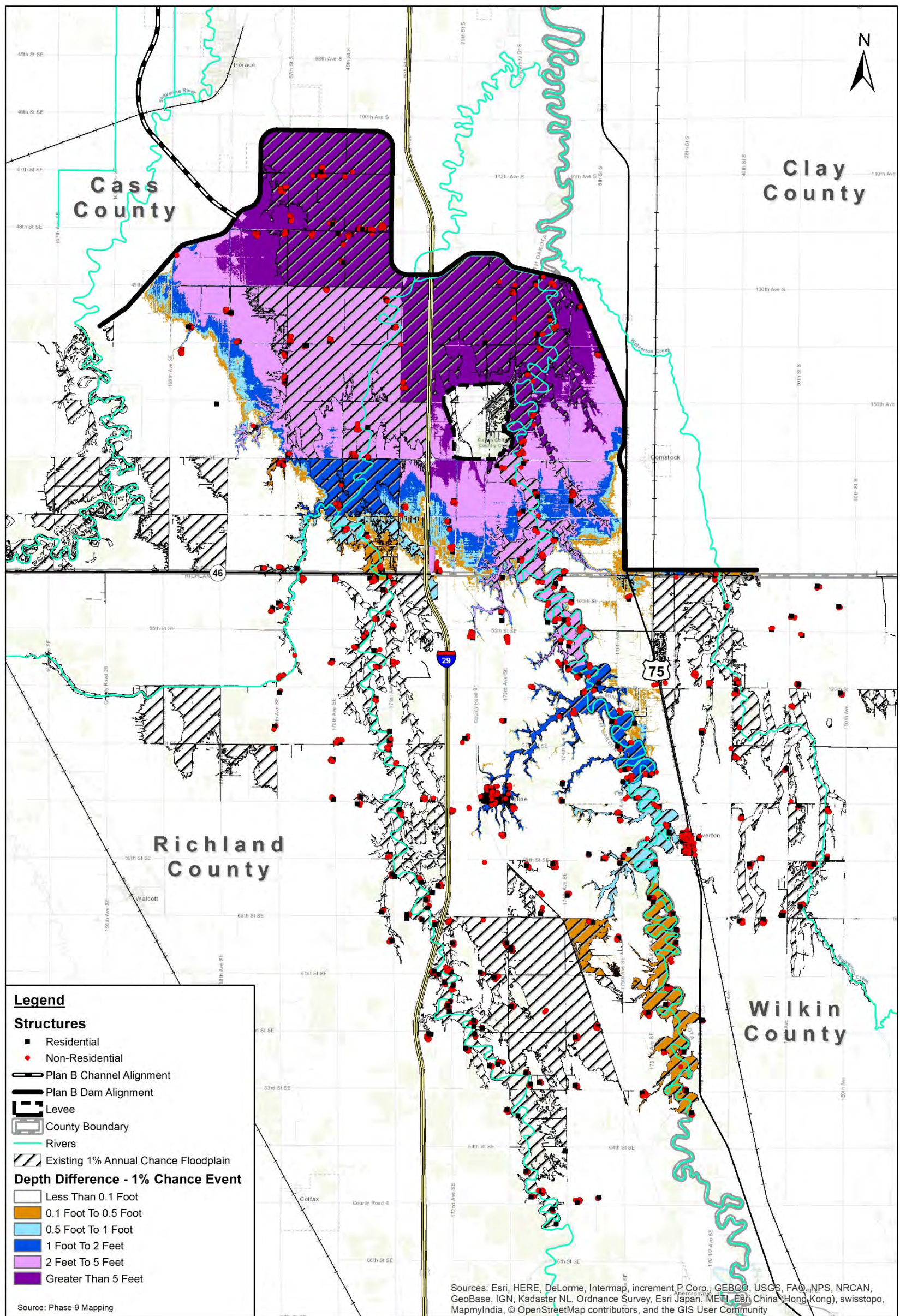
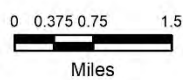


Figure 39: Depth Difference Pool, 1% ACE Event



**Plan B – 1% Chance Event
Depth Difference Map Upstream of Dam**



Created By: kyle.volk Date Created: 06/10/2018 Date Exported: 7/27/2018 Image: N/A Elevation Data: USACE DEM
Coordinate System: NAD 1983 UTM Zone 14N Datum: North American 1983
T:\Projects\1770017736211\MDNR_ADA_Maps\16477_263_DepthDifMap_2018_06_10_USACE_100yr.mxd

Figure 40: Depth Difference Pool, 0.2% ACE Event

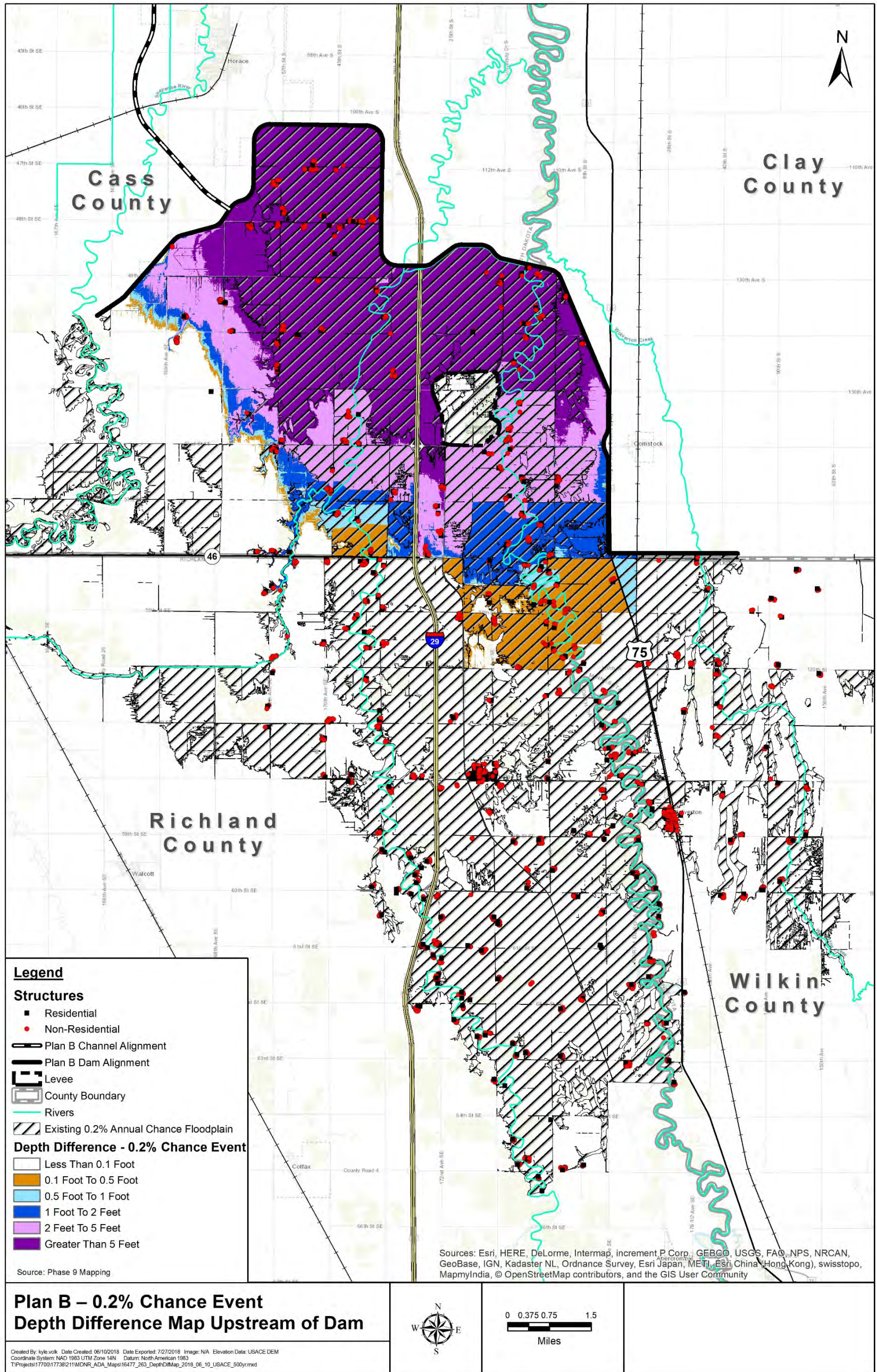
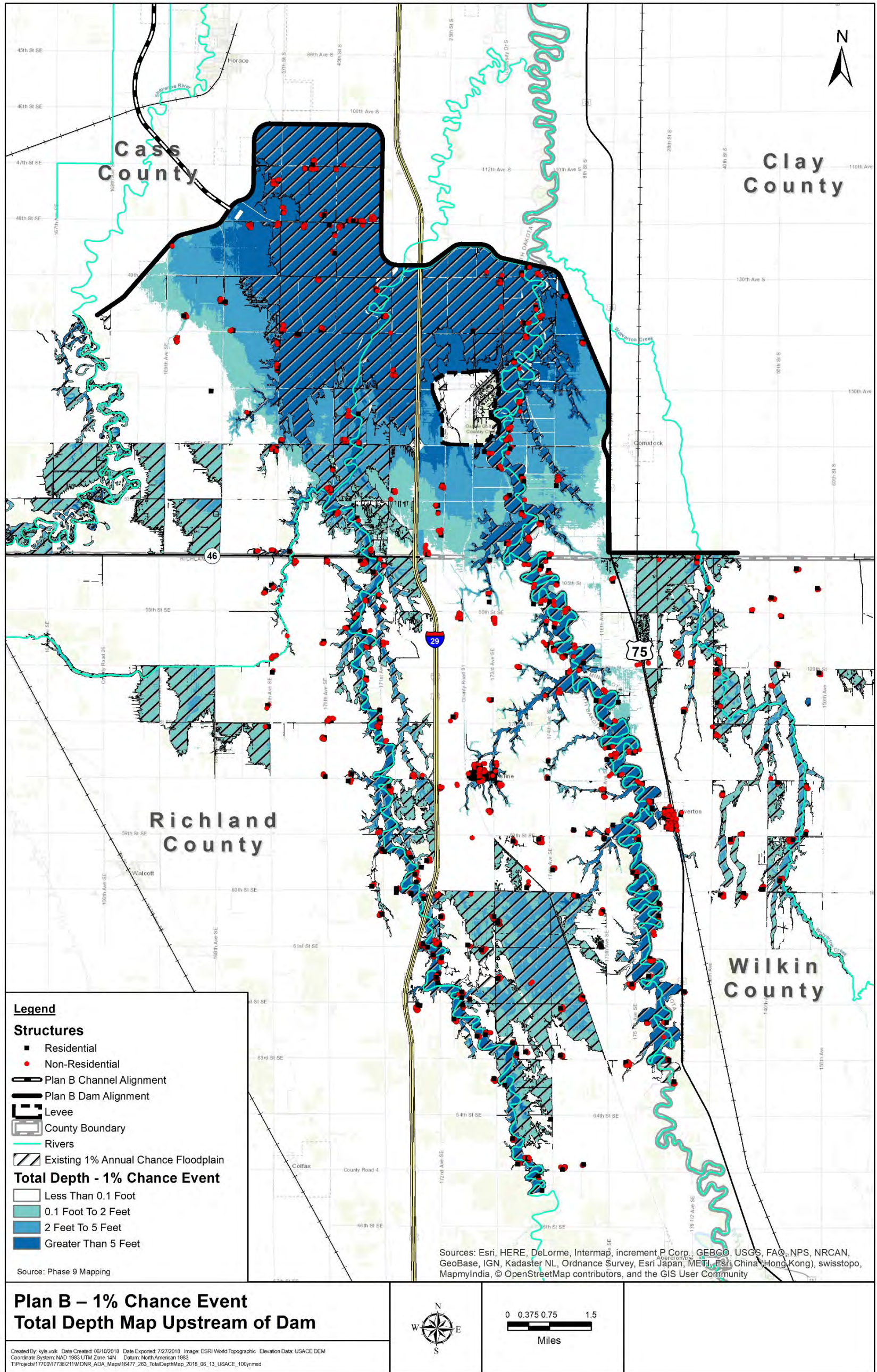


Figure 41: Total Depth, 1% ACE Event



Attachment 1

Attachment 1 – POR Hydrology Development

To: Supplemental Environmental Assessment Document
From: Greg Thompson, PE, CFM; Jun Yang, PhD, PE
Subject: Appendix D Hydrology and Hydraulics – Attachment 1 POR Hydrology Development
Date: July 24, 2018
Project: Fargo-Moorhead Metropolitan Area Flood Risk Management Project EA Document

1. INTRODUCTION

This Technical Memorandum (TM) was written to document the development of the Period of Record (POR) Hydrology for Plan B of the Fargo-Moorhead Metropolitan Area Flood Risk Management Project (Project).

Inflow hydrology for the Project was originally developed by the US Army Corps of Engineers (USACE) during the Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study and Environmental Impact Statement, Phase 4, 2011 (FEIS). Within that study, the peak discharge and volume-duration hydrology had been developed over a period of approximately two years as new information became available and as the project changed. Initially, POR hydrology was developed for modeling within the immediate Fargo/Moorhead Metro area, but, as documented in a series of FEIS appendices, the hydrology was revised to focus on a shorter period of record developed by an Expert Opinion Elicitation (EOE) panel. The EOE hydrology produced peak flow and balanced hydrographs that varied over time. Project design focused on assuring the project would perform for the highest peak flow and volume conditions identified via the EOE panel. This hydrology has since been referred to as the Wet Cycle Hydrology, and in this TM, it will be referred to as EOE/WET. Shortly after the EOE/WET hydrology was developed, the model was extended downstream to the Canadian Border to adequately simulate downstream impacts. Then, to offset downstream impacts, the Southern Embankment (Dam) and Upstream Staging Area were incorporated into the project, which required the modeling and subsequent inflow hydrology to be extended further upstream. Each project change resulted in a change to the hydrology. The most recent change occurred in 2017/2018 during development of Plan B where the Governors' Task Force decided that the project should use the POR hydrology instead of the EOE/WET hydrology.

Since EOE hydrology had been chosen as the path forward, the POR hydrology developed by USACE during the FEIS was not completed to support the current modeling efforts. Therefore, as documented in this TM, additional hydrology was created by Houston-Moore Group (HMG) for Plan B within a modeling effort referred to as Phase 9. The tables in this TM describe a progression of how the EOE/WET and POR hydrology components were created during and after the FEIS, and how the POR hydrology was created for Phase 9. Modeling for Phase 9 included 10-, 5-, 2-, 1-, and 0.2-percent Annual Chance Events (ACE), also commonly referred to as 10-year, 20-year, 50-year, 100-year, and 500-year flood events, respectively. The 4-percent chance event (25-year) was also included in Phase 9 because it is commonly used in flood insurance study evaluations, and the 0.5-percent chance event (200-year) was included in the Phase 9 analysis because it provides an intermediate reference point between the 100-year and 500-year events.

2. BACKGROUND/HYDROLOGY TERMINOLOGY

Peak Discharges - Annual instantaneous peak discharges were created for each streamflow gage along the Red River including gages at Enloe ND, Hickson ND, Fargo ND, Halstad MN, Thompson ND, Grand Forks ND, Oslo MN, and Drayton ND. The initial peak discharges were created by USACE, and the Plan B POR discharges were developed by HMG as described below.

Balanced Hydrographs – Volume-Duration-Frequency analyses were conducted by USACE for the EOE/WET hydrology at the gage locations along the Red River. For the Plan B modeling effort, HMG developed hydrographs that closely resemble the USACE derived balanced hydrographs using known information from the previous USACE EOE/WET hydrology analysis.

3. FEASIBILITY STUDY/ENVIRONMENTAL IMPACT STATEMENT HYDROLOGY

a) Initial Inflow Hydrology – FEIS Appendix A-2

The discharges displayed in

Table 1 originated from FEIS Appendix A-2, Table 24 (POR), and the discharges displayed in Table 2 originated from FEIS Appendix A-2, Table 25 (EOE/WET). At the time, only these locations were needed for model simulation because the focus of the modeling was near the Fargo/Moorhead Metro area. However, the remaining blank cells in

Table 1 show the locations and flood events that are required for the Plan B analysis. This TM documents the development of the remaining POR discharges and modifications made to some of the discharges presented in Table 1. Notice that peak discharges for Enloe, Thompson, Oslo, and Drayton were not initially developed in the FEIS. Additionally, the 4-percent chance event discharges had not been developed.

Table 1: Red River Peak Discharges, POR Hydrology from FEIS, Appendix A-2, Table 24

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton							
Oslo							
Grand Forks	50,500	67,300		91,700	112,000	134,000	165,000
Thompson							
Halstad	29,800	39,900		54,600	66,900	80,200	99,200
Fargo	13,865	19,831		26,000	33,000	43,500	66,000
Hickson	8,400	12,000		19,000	23,100	28,300	35,000
Enloe							

Table 2: Red River Peak Discharges, EOE/WET Hydrology from FEIS Appendix A-2, Table 25

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton							
Oslo							
Grand Forks	56,354	70,956		91,026	106,838	123,201	145,675
Thompson							
Halstad	34,871	45,014		59,306	70,798	82,872	99,713
Fargo	17,000	22,000		29,300	34,700	46,200	61,700
Hickson	10,500	14,800		21,000	25,000	28,500	32,000
Enloe							

b) Inflow Hydrology – FEIS Appendix A-4b

The EOE/WET peak discharges displayed in Table 3 originated from FEIS Appendix A-4b, Table 24. This hydrology effort included developing additional EOE/WET hydrology peak discharges for streamflow gages at downstream locations, such as Thompson, ND, Oslo, MN and Drayton, ND. At this point in time the project was to focus on EOE/WET hydrology, therefore no POR discharges were recorded for the new locations. Also, the peak discharges for Hickson were revised from what was presented in Appendix A-2. Enloe was not considered at this point in time because the staging area was not a project component and the model did not need to be extended upstream to Enloe.

Table 3: Red River EOE/WET Peak Discharges - Inflow Hydrology - FEIS Appendix A-4b, Table 24

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton	62,847	79,061		101,292	118,757	136,789	161,486
Oslo	58,970	74,459		95,773	112,569	129,950	153,811
Grand Forks	56,354	70,956		91,026	106,838	123,201	145,675
Thompson	42,899	55,519		72,898	86,765	101,001	121,080
Halstad	34,871	45,014		59,306	70,798	82,872	99,713
Fargo	17,000	22,000		29,300	34,700	46,200	61,700
Hickson	10,500	14,000		19,000	22,000	28,500	37,000
Enloe							

c) Hickson Gage Inflow Hydrology Revision – USACE Report, January 2015

EOE/WET peak discharges for the Hickson Gage were revised in January 2015 to better reflect breakout characteristics from the Wild Rice River to the Red River near Abercrombie, North Dakota. As displayed in Table 4, revisions from this analysis were made to the EOE/WET hydrology, as documented in "The Use of Synthetic Floods for Defining the Regulated Flow-Frequency & Volume Duration Frequency Curves for the Red River at Hickson, North Dakota" (January 2015), Table 12.



Following the January 2015 report, USACE also provided peak discharges for the Enloe Gage for the WET hydrology, which is approximately 30 river miles upstream of the Hickson Gage. The Enloe Gage data was provided for use as the inflow at the upstream end of the Red River, which was needed after incorporating the Dam and Upstream Staging Area. Note that Enloe hydrology was not provided for the 4- or 5-percent chance events because they were not in the scope of the analysis at that point in time.

Table 4: Hydrology Updates – EOE/WET Hydrology, Hickson and Enloe – USACE, January 2015

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton	62,847	79,061		101,292	118,757	136,789	161,486
Oslo	58,970	74,459		95,773	112,569	129,950	153,811
Grand Forks	56,354	70,956		91,026	106,838	123,201	145,675
Thompson	42,899	55,519		72,898	86,765	101,001	121,080
Halstad	34,871	45,014		59,306	70,798	82,872	99,713
Fargo	17,000	22,000		29,300	34,700	46,200	61,700
Hickson	9,600	13,200		19,000	23,500	28,500	36,000
Enloe	10,031			20,053	24,164	29,512	35,303

d) 4-Percent Chance Event Peak Discharges – USACE, May 2015

In May 2015, peak discharges for the 4-percent chance event were provided for gages at Hickson, Fargo, and Halstad. Table 5 presents the additional EOE/WET peak discharges.

Table 5: EOE/WET Hydrology, 4-Percent Chance Event Discharge Updates - USACE, May 2015

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton	62,847	79,061		101,292	118,757	136,789	161,486
Oslo	58,970	74,459		95,773	112,569	129,950	153,811
Grand Forks	56,354	70,956		91,026	106,838	123,201	145,675
Thompson	42,899	55,519		72,898	86,765	101,001	121,080
Halstad	34,871	45,014	48,348	59,306	70,798	82,872	99,713
Fargo	17,000	22,000	23,900	29,300	34,700	46,200	61,700
Hickson	9,600	13,200	14,450	19,000	23,500	28,500	36,000
Enloe	10,031			20,053	24,164	29,512	35,303

e) Fargo Gage Hydrograph Volume Revisions – USACE, July 2015

Prior to July 2015, the shapes of the balanced hydrographs were relatively narrow compared to observed historical hydrographs of similar magnitude. In 2013, the Red River Basin Commission (RRBC) conducted a basin-wide hydrology modeling effort where a HEC-HMS hydrology model was developed for each watershed upstream of the Red River Gage at Halstad, MN. Hydrographs from this modeling effort were compared to the balanced hydrographs developed for the project. Hydrographs from the RRBC modeling effort were similar in shape to historical events, but displayed more volume than the balanced hydrographs at the time. Therefore, the balanced hydrograph procedure was reevaluated, and modifications were made to the Hickson and Fargo

Gage balanced hydrographs. Revisions reflected in this effort only changed the volume-duration relationships, not the peak discharges. Due to the scope of the project at this point in time, higher volume hydrographs were only created for the 10-, 2-, 1-, 0.5-, and 0.2-percent chance events using EOE/WET hydrology.

4. PLAN B – PERIOD OF RECORD HYDROLOGY

This section documents the POR hydrology development to be used with Plan B. As presented in Table 1, POR hydrology was previously developed for Hickson, Fargo, Halstad, and Grand Forks. However, due to several changes following the FEIS, and an incomplete POR data set, HMG was tasked with developing the POR hydrology using relationships from available POR records as well as previously developed EOE/WET hydrology.

a) Hickson and Enloe POR Peak Discharges and Hydrographs

Annual instantaneous peak discharges for Hickson were first presented in the FEIS Appendix A-2, using both EOE/WET and POR hydrology. Now, since POR hydrology is required for Plan B, and the EOE discharges have been revised since the FEIS, the old POR/EOE relationship from the FEIS will be used to create updated POR hydrology for Hickson. Note flows for the 4-percent chance event POR and EOE/WET at Hickson (Table 6) were derived using flow frequency curves in the FEIS Appendix A-2 (Figures 34 and 35). As presented in Table 6, a unique ratio for each design event has been established to apply to the updated EOE/WET discharges from Table 5 for producing updated POR discharges as shown in Table 7 for Hickson and Table 8 for Enloe. After evaluating peak discharge and volume proportions between Enloe, Abercrombie, and Fargo, and after reviewing the discharge-frequency curves for Hickson and Enloe, the peak discharges will be further refined, with final numbers presented in Table 20.

Table 6: Discharge Relationship between EOE/WET and POR Hydrology at Hickson, ND (Source: FEIS, January 2011)

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Hickson POR (Table 1)	8,400	12,000	13,000	19,000	23,100	28,300	35,000
Hickson EOE/WET (Table 2)	10,500	14,800	15,700	21,000	25,000	28,500	32,000
Ratio POR to EOE/WET	0.80	0.81	0.83	0.90	0.92	0.99	1.09

Table 7: New POR Discharges for Hickson Gage (Not Final)

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Hickson EOE/WET (Table 5)	9,600	13,200	14,450	19,000	23,500	28,500	36,000
Ratio, POR to EOE/WET (Table 6)	0.80	0.81	0.83	0.90	0.92	0.99	1.09
Hickson POR (2018)	7,700	10,700	12,000	17,200	21,700	28,300	39,400

Table 8: New POR Discharges for Enloe Gage (Not Final)

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Enloe EOE/WET (Table 5)	10,031			20,053	24,164	29,512	35,303
Ratio, POR to EOE/WET (Table 6)	0.80	0.81	0.83	0.90	0.92	0.99	1.09
Enloe POR (2018)	8,000			18,100	22,300	29,300	38,600

b) Peak Discharges for 4-Percent and 5-Percent Chance Events

Reservoirs in the upper portions of the Red River Basin and breakout flows between upstream watersheds produce complex discharge-frequency relationships upstream of Fargo. Because of this, it is understood that standard Log-Pearson Type III plotting procedures should not be used at Enloe, Hickson, or Fargo, but it can be used for locations downstream of Fargo because of the extended distance downstream of reservoirs. While recognizing this, yet also observing smooth relationships on such plots, 4-percent chance (25-year) and 5-percent chance (20-year) event peak discharges were created from larger and smaller events. Exhibits 1 through 8 display Log-Pearson Type III plots for the POR and EOE/WET hydrology at each of the streamflow gages along the Red River. From these plots, the peak discharges have been estimated for the 4-percent and 5-percent chance events for the EOE/WET hydrology (Table 9) and POR hydrology (Table 10).

Table 9: EOE/WET 0.4-Percent Chance Event Peak Discharge Development Using Discharge-Frequency Relationships

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton	62,847	79,061	84,500	101,292	118,757	136,789	161,486
Oslo	58,970	74,459	79,500	95,773	112,569	129,950	153,811
Grand Forks	56,354	70,956	75,000	91,026	106,838	123,201	145,675
Thompson	42,899	55,519	59,400	72,898	86,765	101,001	121,080
Halstad	34,871	45,014	48,348	59,306	70,798	82,872	99,713
Fargo	17,000	22,000	23,900	29,300	34,700	46,200	61,700
Hickson	9,600	13,200	14,450	19,000	23,500	28,500	36,000
Enloe	10,031	14,500	15,500	20,053	24,164	29,512	35,303

Table 10: POR 0.4-Percent Chance Event Peak Discharge Development Using Discharge-Frequency Relationships

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton							
Oslo							
Grand Forks	50,500	67,300	72,000	91,700	112,000	134,000	165,000
Thompson							
Halstad	29,800	39,900	43,200	54,600	66,900	80,200	99,200
Fargo	13,865	19,831	21,400	26,000	33,000	43,500	66,000
Hickson	7,700	10,700	12,000	17,200	21,700	28,300	39,400
Enloe	8,000	11,800	13,000	18,100	22,300	29,300	38,600

c) POR Hydrology Peak Discharge Development for Thompson, Oslo, and Drayton

As previously described, EOE/WET peak discharges are available at all reporting locations. From FEIS Appendix A-2, a POR to EOE/WET ratio was created for Grand Forks to be used in generating POR hydrology for Oslo and Drayton. This is shown in Table 11. Using the Grand Forks ratios, the POR peak discharges that were created for Oslo and Drayton are shown in Table 12 and Table 13, respectively.

Table 11: POR to EOE/WET Annual Instantaneous Peak Discharge Ratios for Grand Forks, North Dakota

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Grand Forks POR	50,500	67,300	72,000	91,700	112,000	134,000	165,000
Grand Forks EOE/WET	56,354	70,956	75,000	91,026	106,838	123,201	145,675
Ratio POR to EOE/WET	0.90	0.95	0.96	1.01	1.05	1.09	1.13

Table 12: Oslo, Minnesota POR Peak Discharges Created from Grand Forks POR to EOE/WET Ratios

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Oslo EOE/WET (Table 9)	58,970	74,459	79,500	95,773	112,569	129,950	153,811
Ratio POR to EOE/WET (Table 11)	0.90	0.95	0.96	1.01	1.05	1.09	1.13
Oslo POR	52,800	70,600	76,300	96,500	118,000	141,300	174,200

Table 13: Drayton, North Dakota POR Peak Discharges Created from Grand Forks POR to EOE/WET Ratios

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton EOE/WET (Table 9)	62,847	79,061	84,500	101,292	118,757	136,789	161,486
Ratio POR to EOE/WET (Table 11)	0.90	0.95	0.96	1.01	1.05	1.09	1.13
Drayton POR	56,300	75,000	81,100	102,000	124,500	148,800	182,900

The POR peak discharges for Thompson, North Dakota were created in a similar manner as the discharges for Oslo and Drayton, except for the ratios for Thompson were created using an average ratio from Fargo, Halstad, and Grand Forks. The Halstad POR to EOE/WET ratios are presented in Table 14, and the Fargo POR to EOE/WET ratios are presented in Table 15. Combining the ratios from Grand Forks, Halstad, and Fargo, Table 16 presents the average POR to EOE/WET ratio to be used for developing the Thompson discharges. Table 17 presents the POR discharges for Thompson, North Dakota.

Table 14: POR to EOE/WET Annual Instantaneous Peak Discharge Ratios for Halstad, Minnesota

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Halstad POR (Table 10)	29,800	39,900	43,200	54,600	66,900	80,200	99,200
Halstad EOE/WET (Table 9)	34,871	45,014	48,348	59,306	70,798	82,872	99,713
Ratio POR to EOE/WET	0.85	0.89	0.89	0.92	0.94	0.97	0.99



Table 15: POR to EOE/WET Peak Discharge Ratios for Fargo, North Dakota

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Fargo POR (Table 10)	13,865	19,831	21,400	26,000	33,000	43,500	66,000
Fargo EOE/WET (Table 9)	17,000	22,000	23,900	29,300	34,700	46,200	61,700
Ratio POR to EOE/WET	0.82	0.90	0.90	0.89	0.95	0.94	1.07

Table 16: Average POR to EOE/WET Ratios Used to Develop POR Peak Discharges for Thompson, North Dakota

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Grand Forks Ratio (Table 11)	0.90	0.95	0.96	1.01	1.05	1.09	1.13
Halstad Ratio (Table 14)	0.85	0.89	0.89	0.92	0.94	0.97	0.99
Fargo Ratio (Table 15)	0.82	0.90	0.90	0.89	0.95	0.94	1.07
Average Ratio	0.86	0.91	0.92	0.94	0.98	1.00	1.07

Table 17: Thompson, North Dakota Peak Discharges Using Average POR to EOE/WET Ratios from Grand Forks, Halstad, Fargo.

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Thompson EOE/WET (Table 9)	42,899	55,519	59,400	72,898	86,765	101,001	121,080
Ratio POR to EOE/WET (Table 16)	0.86	0.91	0.92	0.94	0.98	1.00	1.07
Thompson POR	36,700	50,600	54,400	68,400	85,200	100,900	129,000

d) Modifications Made to Enloe/Hickson Discharges

At the upstream end of the project, the Enloe and Hickson Gages are relatively close in proximity to each other. Also, there are no significant tributary inflows between the gages, so the calculated peak inflows are very similar to each other. Depending on the volume of the event and the magnitude of the local inflows, the peak discharges from Enloe to Hickson are either reduced due to attenuation, remain the same, or are increased due to local inflows. To verify if the newly developed POR hydrology for Hickson and Enloe seemed reasonable, the general trends from Enloe to Hickson and Enloe to Fargo were reviewed. The Enloe to Fargo relationship provides a comparison between the Red River and the Wild Rice River flow contributions. The initial iteration of Hickson and Enloe discharge development produced Enloe to Fargo relationships that were not consistent across various events (shown in Table 18). The ratios ranged from 0.58 to 0.70. Therefore, the peak discharges at Enloe and Hickson were adjusted as shown in the Table 19 and Table 20. The specific changes are noted as follows:

- 10% ACE – Enloe was increased from a calculated 8,000 cfs to 9,000 cfs, which produces an Enloe to Fargo ratio of 0.65. The difference between Enloe and Hickson from January 2015 was approximately 400 cfs. The assumption here is that the difference is approximately 600 cfs.
- 5% ACE – Enloe remained as-is, but the Hickson discharge appeared to be too low so it was increased to 11,400 cfs to produce a 400 cfs difference (Enloe to Hickson), similar to the January 2015 10% differences.

- 4% ACE – no changes. The Enloe to Fargo ratio is 0.61.
- 2% ACE – The preliminary results from the 2% ACE showed the highest ratio for Enloe to Fargo (0.70), so this was the largest change.
- 1% ACE – Enloe to Fargo was originally calculated to be 0.68, which is on the high side. The peak discharge was decreased from 22,300 to 21,000 cfs, which reduced the ratio to 0.64.
- 0.5% ACE – This calculated ratio was on the high side, but it wasn't used for the HEC-RAS analysis, so it wasn't adjusted.
- 0.2% ACE – This calculated ratio appeared very low, so it was increased from 38,600 to 40,000 cfs, which increased the Enloe to Fargo ratio from 0.58 to 0.61.

Table 20 presents the final POR discharges to be used in Plan B modeling.

Table 18: Discharges Prior to Final Calibration (Not final)

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Fargo (Table 10)	13,865	19,831	21,400	26,000	33,000	43,500	66,000
Hickson (Table 10)	7,700	10,700	12,000	17,200	21,700	28,300	39,400
Enloe (Table 10)	8,000	11,800	13,000	18,100	22,300	29,300	38,600
Ratio, Enloe/Fargo	0.58	0.60	0.61	0.70	0.68	0.67	0.58

Table 19: Calibrated POR Discharges at Hickson and Enloe Gages

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Fargo	13,865	19,831	21,400	26,000	33,000	43,500	66,000
Hickson	8,400	11,400	12,000	16,000	21,300	28,300	38,700
Enloe	9,000	11,800	13,000	16,000	21,000	29,300	40,000
Ratio, Enloe/Fargo	0.65	0.60	0.61	0.62	0.64	0.67	0.61

Table 20: Final POR Peak Discharges

Return Period (year)	10	20	25	50	100	200	500
% Annual Chance Event	10	5	4	2	1	0.5	0.2
Drayton (Table 13)	56,300	75,000	81,100	102,000	124,500	148,800	182,900
Oslo (Table 12)	52,800	70,600	76,300	96,500	118,000	141,300	174,200
Grand Forks (Table 10)	50,500	67,300	72,000	91,700	112,000	134,000	165,000
Thompson (Table 17)	36,700	50,600	54,400	68,400	85,200	100,900	129,000
Halstad (Table 10)	29,800	39,900	43,200	54,600	66,900	80,200	99,200
Fargo (Table 10)	13,865	19,831	21,400	26,000	33,000	43,500	66,000
Hickson (Table 19)	8,400	11,400	12,000	16,000	21,300	28,300	38,700
Enloe (Table 19)	9,000	11,800	13,000	16,000	21,000	29,300	40,000

5. PLAN B – PERIOD OF RECORD BALANCED HYDROGRAPHS

USACE developed the balanced hydrographs for the EOE/WET hydrology throughout various stages of the FEIS and as necessary during the project. HMG developed hydrographs closely resembling balanced



hydrographs using HEC-DSSVue and multipliers applied to each EOE/WET hydrograph ordinate. The EOE/WET balanced hydrographs and POR analysis hydrographs are shown in Exhibits 9 through 16 for all streamflow gages along the Red River.



Exhibits



Exhibit 1 - Discharge Frequency Curve at Enloe Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

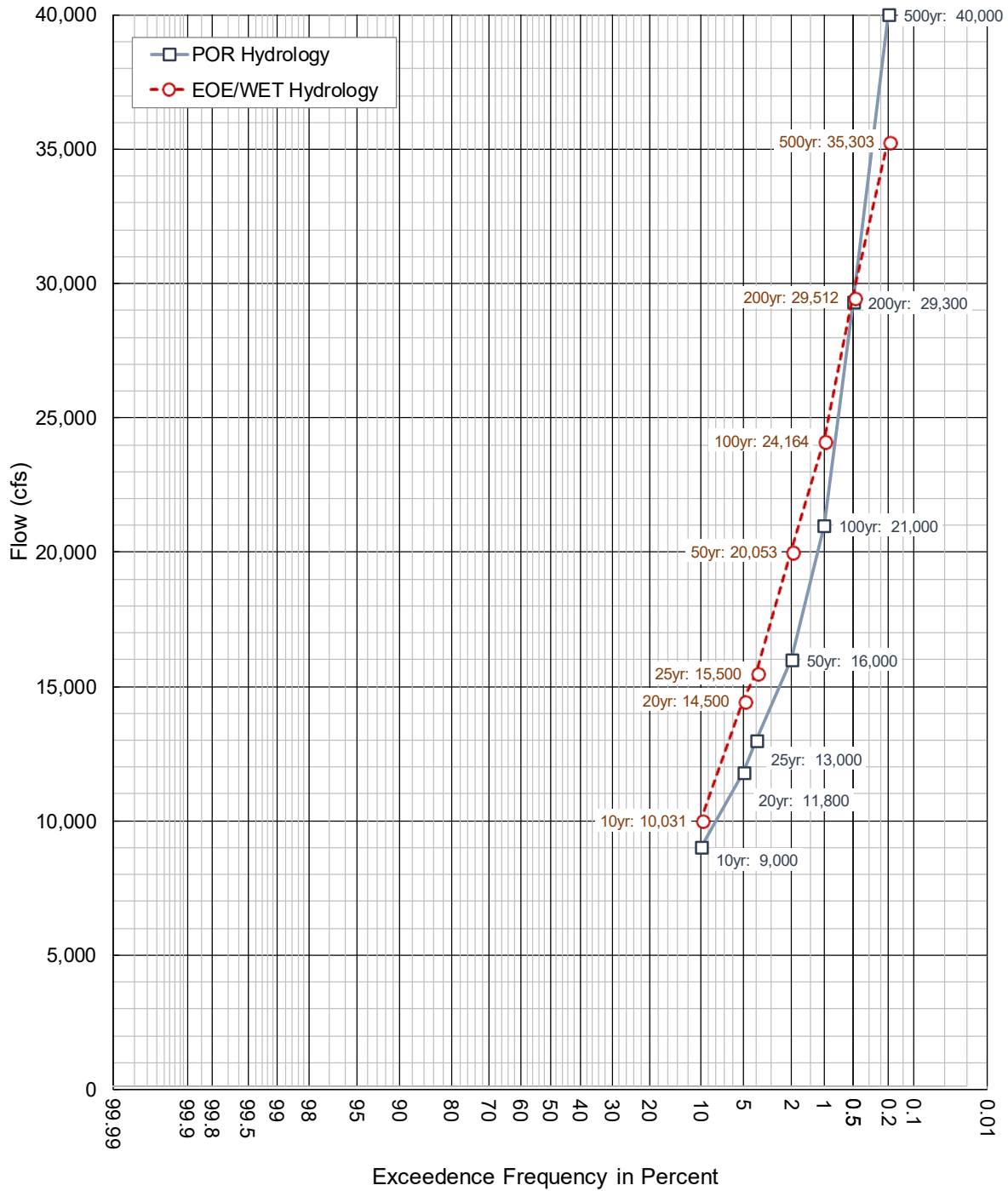


Exhibit 2 - Discharge Frequency Curve at Hickson Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

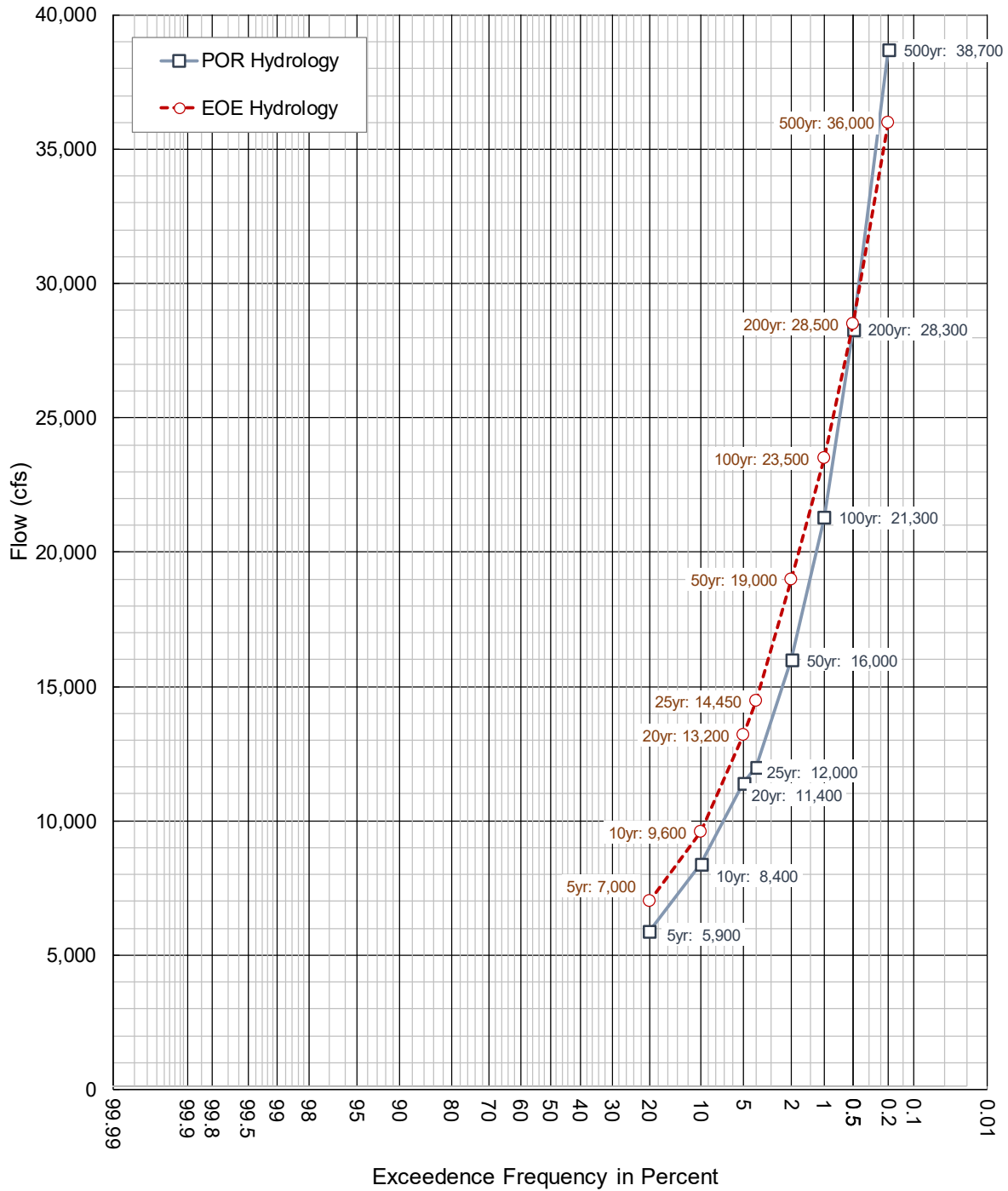


Exhibit 3 - Discharge Frequency Curve at Fargo Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

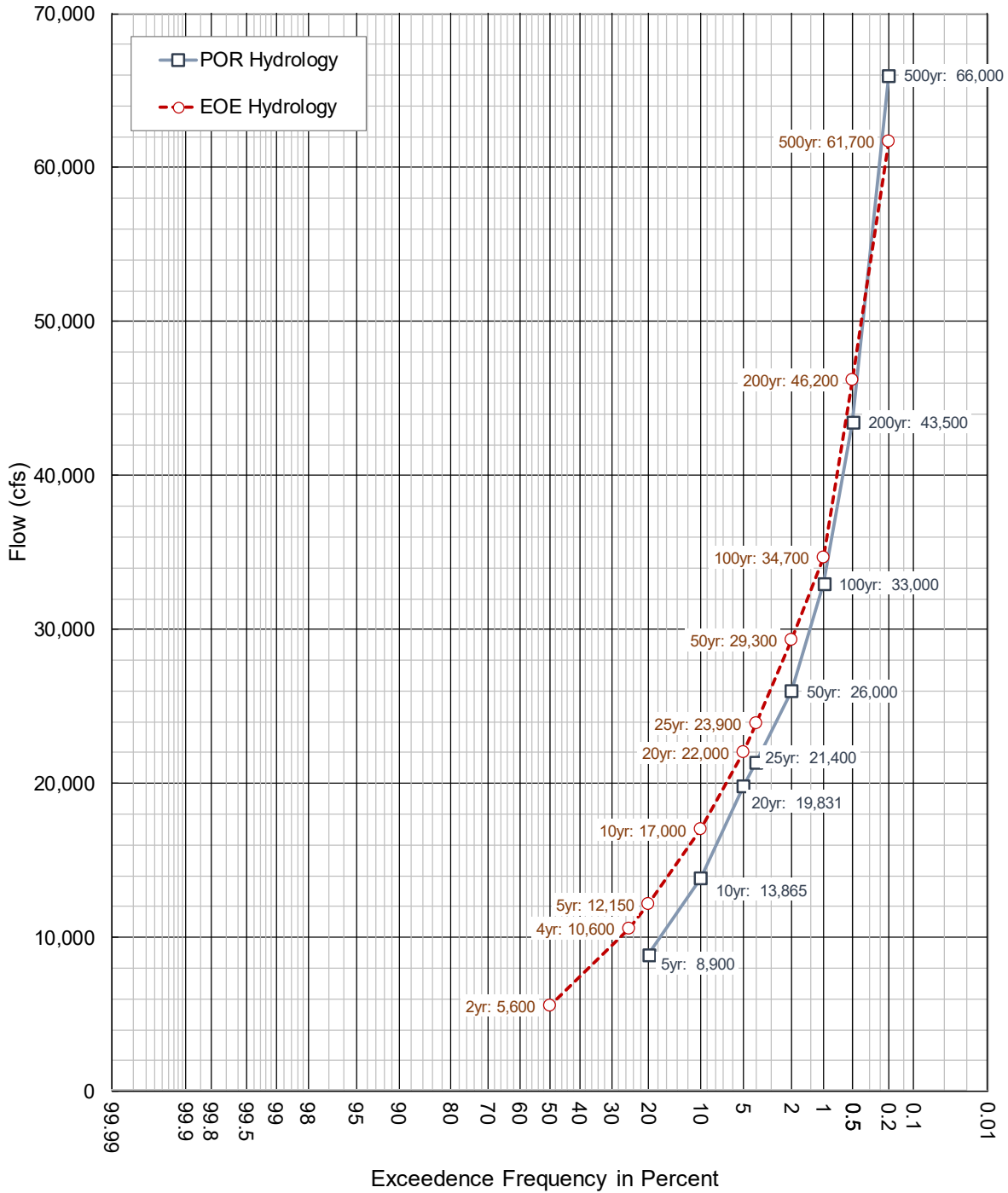


Exhibit 4 - Discharge Frequency Curve at Halstad Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

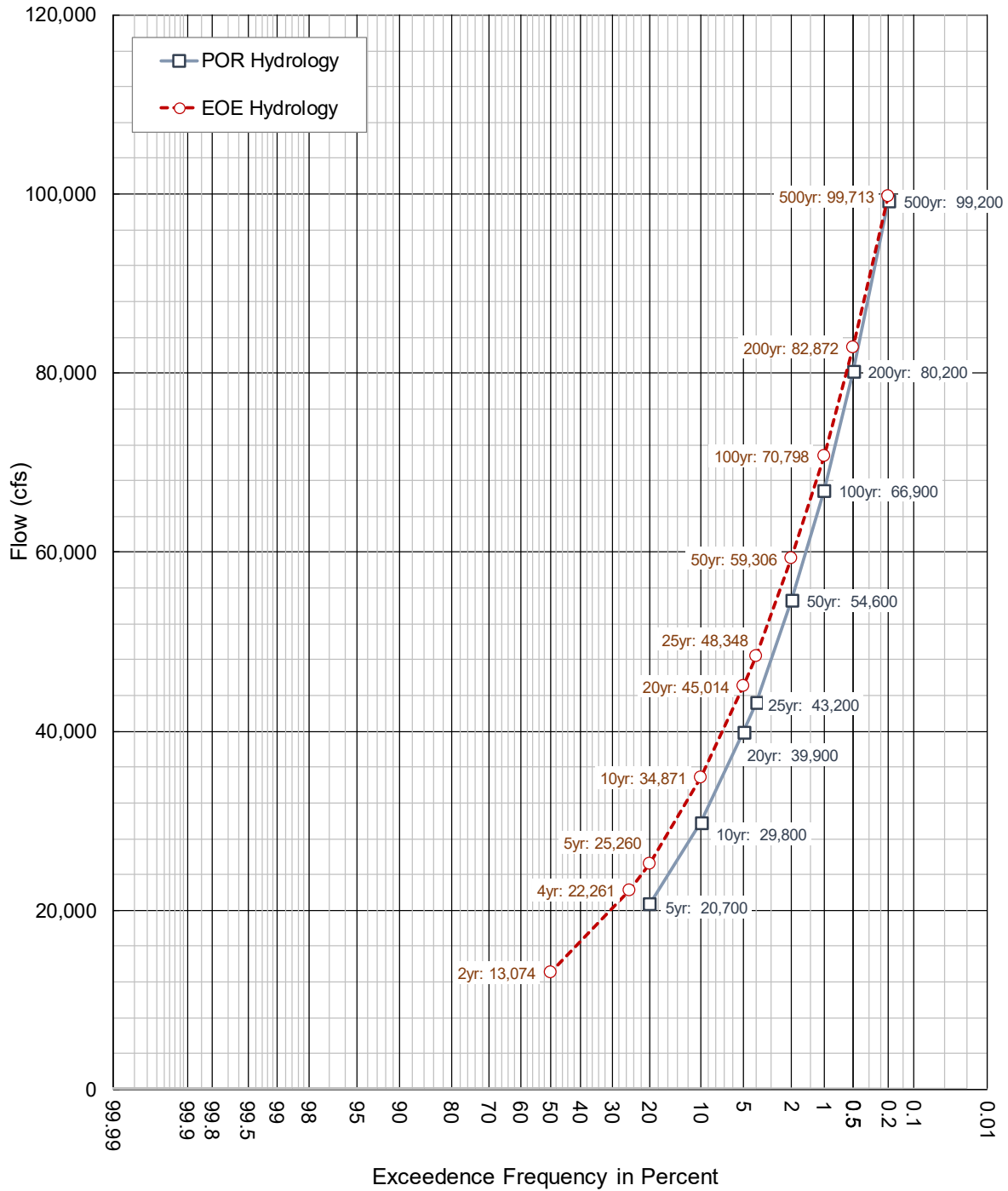


Exhibit 5 - Discharge Frequency Curve at Thompson Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

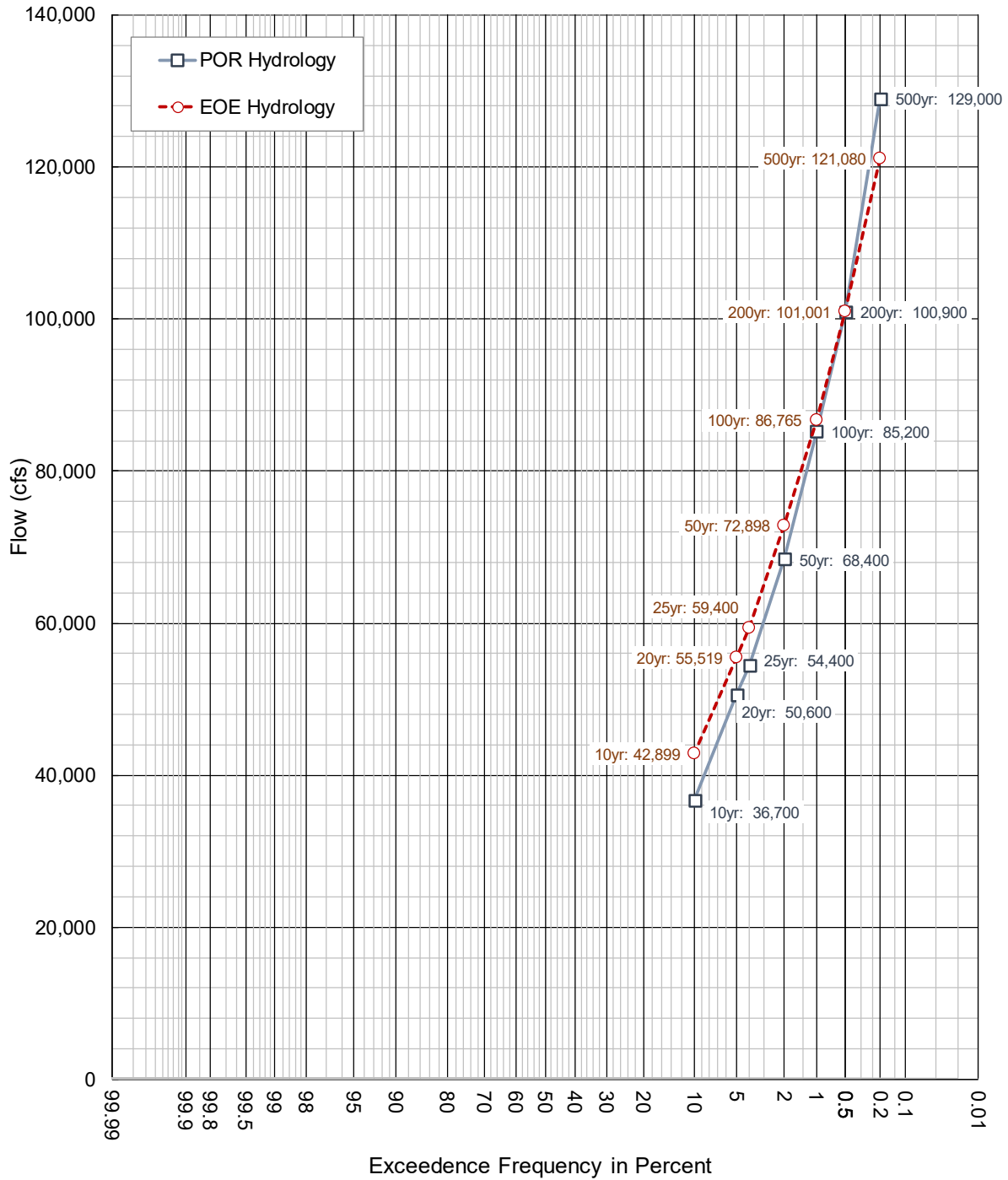


Exhibit 6 - Discharge Frequency Curve at Grand Forks Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

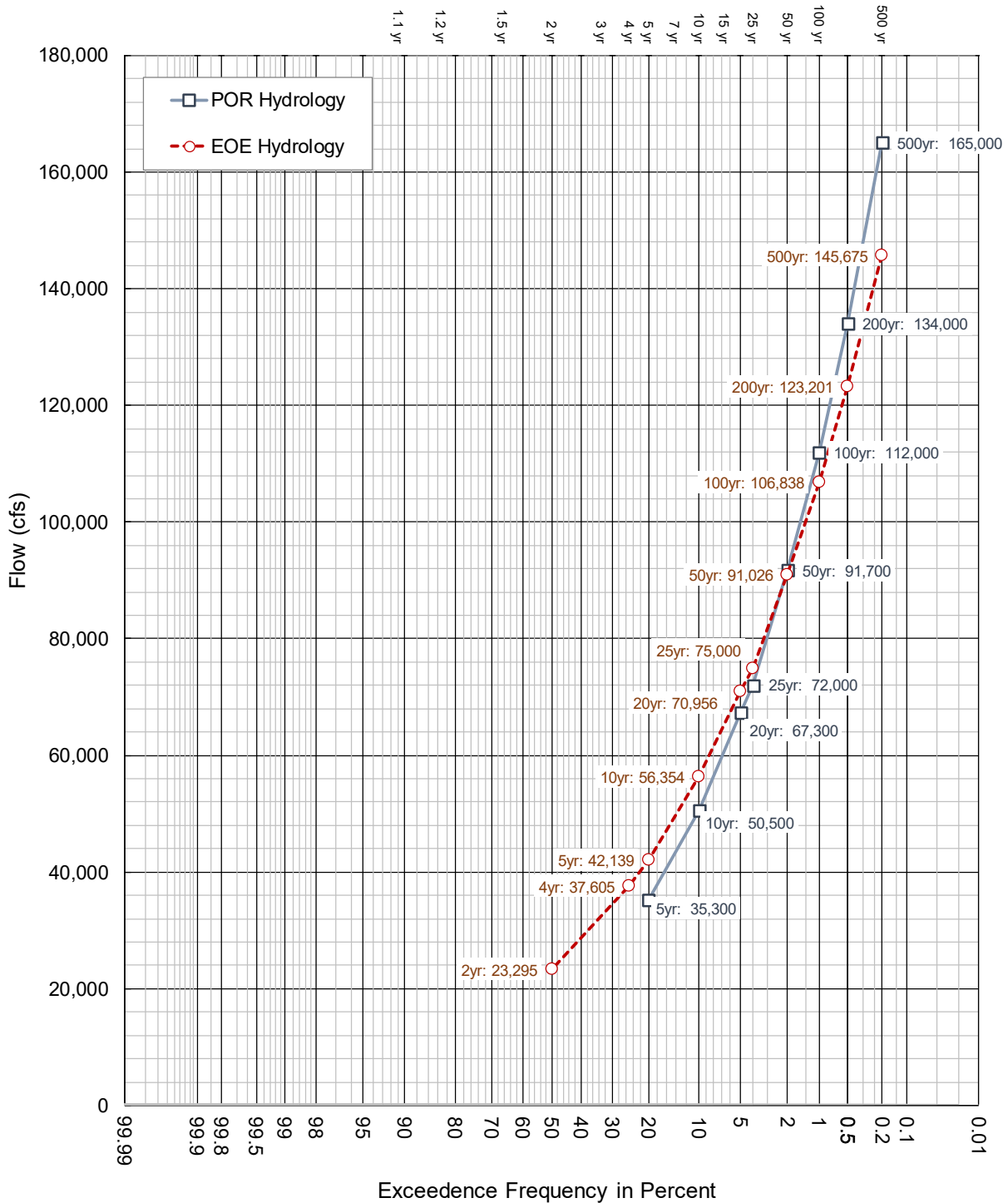


Exhibit 7 - Discharge Frequency Curve at Oslo Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

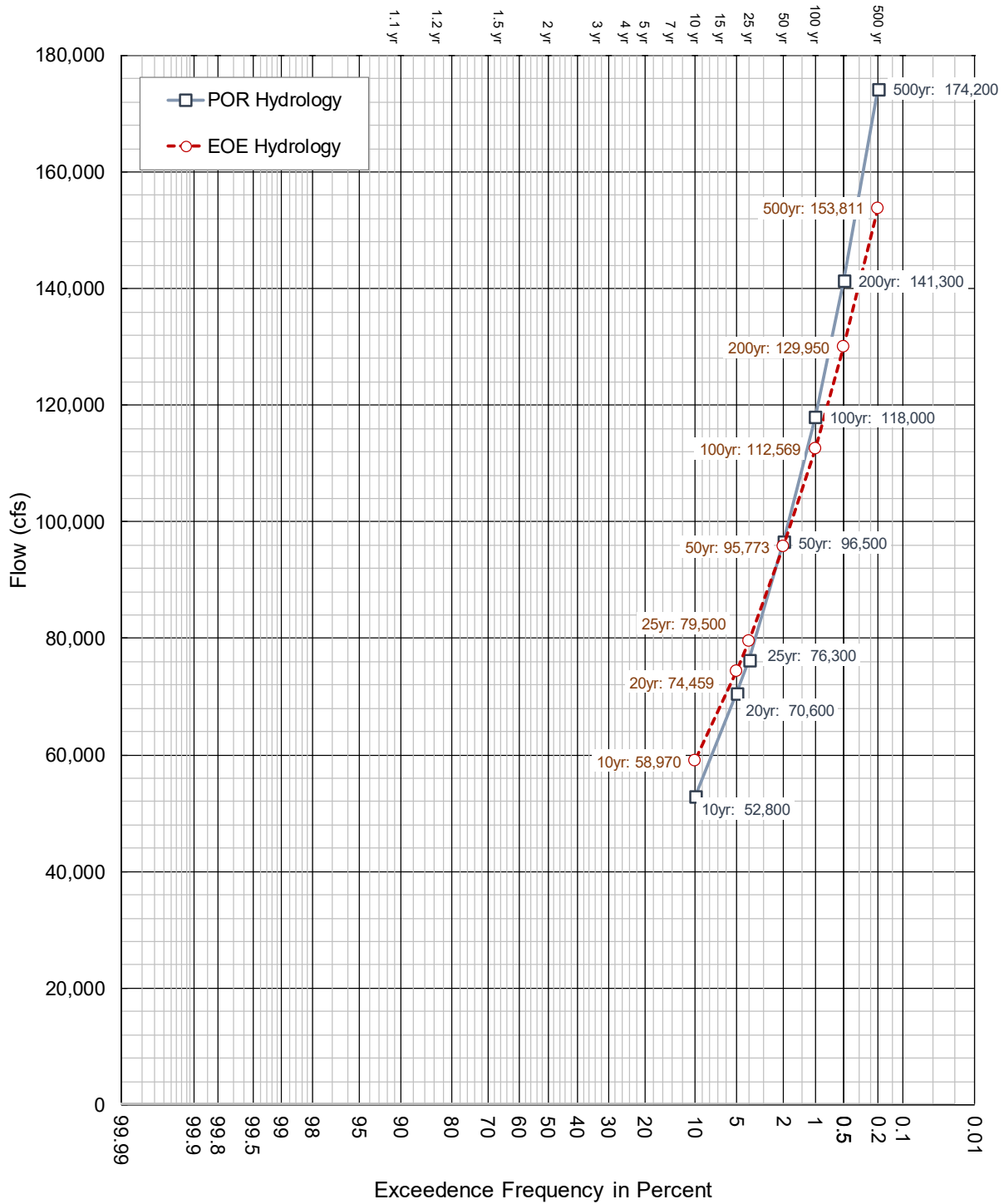


Exhibit 8 - Discharge Frequency Curve at Drayton Gage

Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

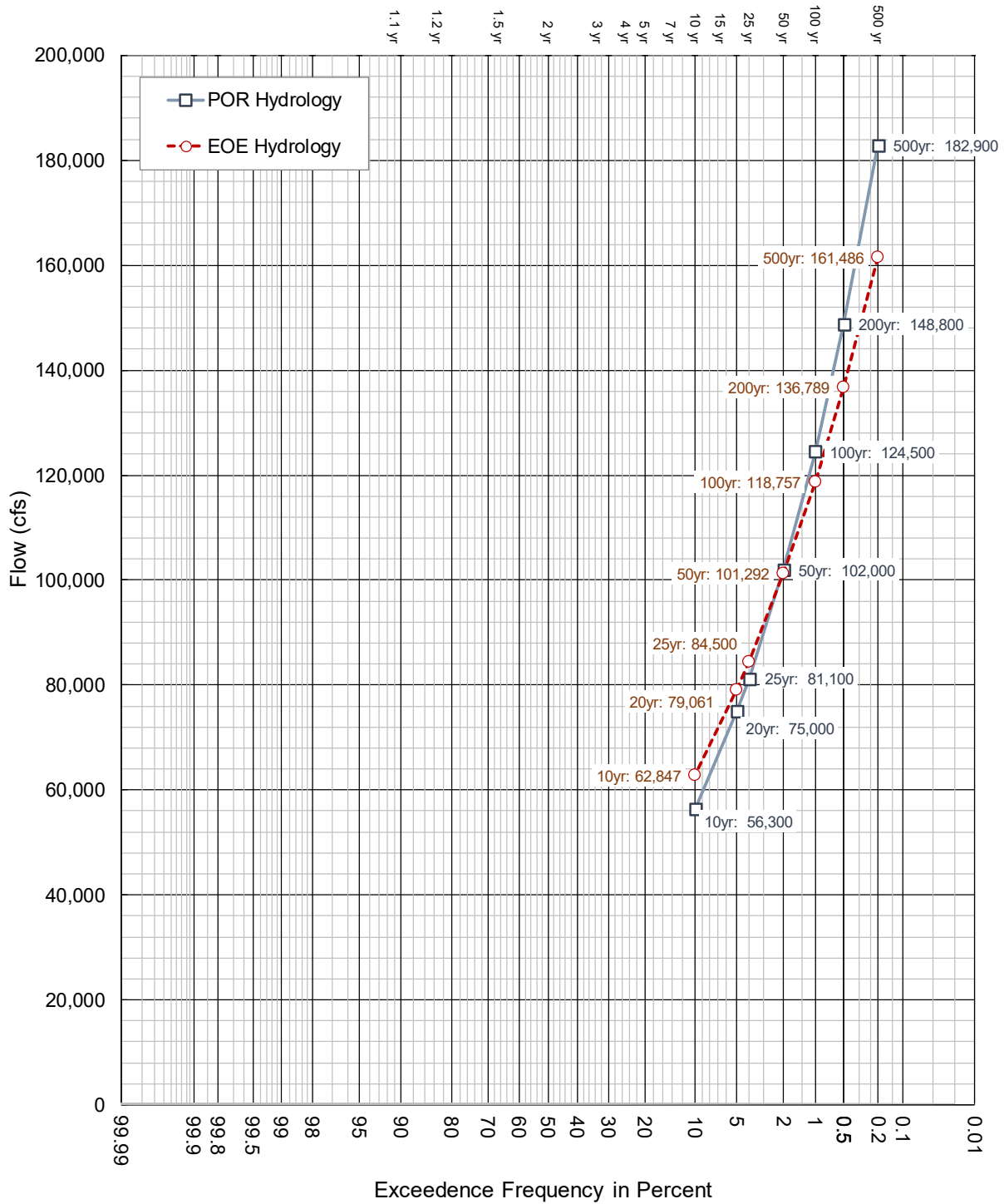


Exhibit 9 - Balanced Hydrograph Comparison at Enloe Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

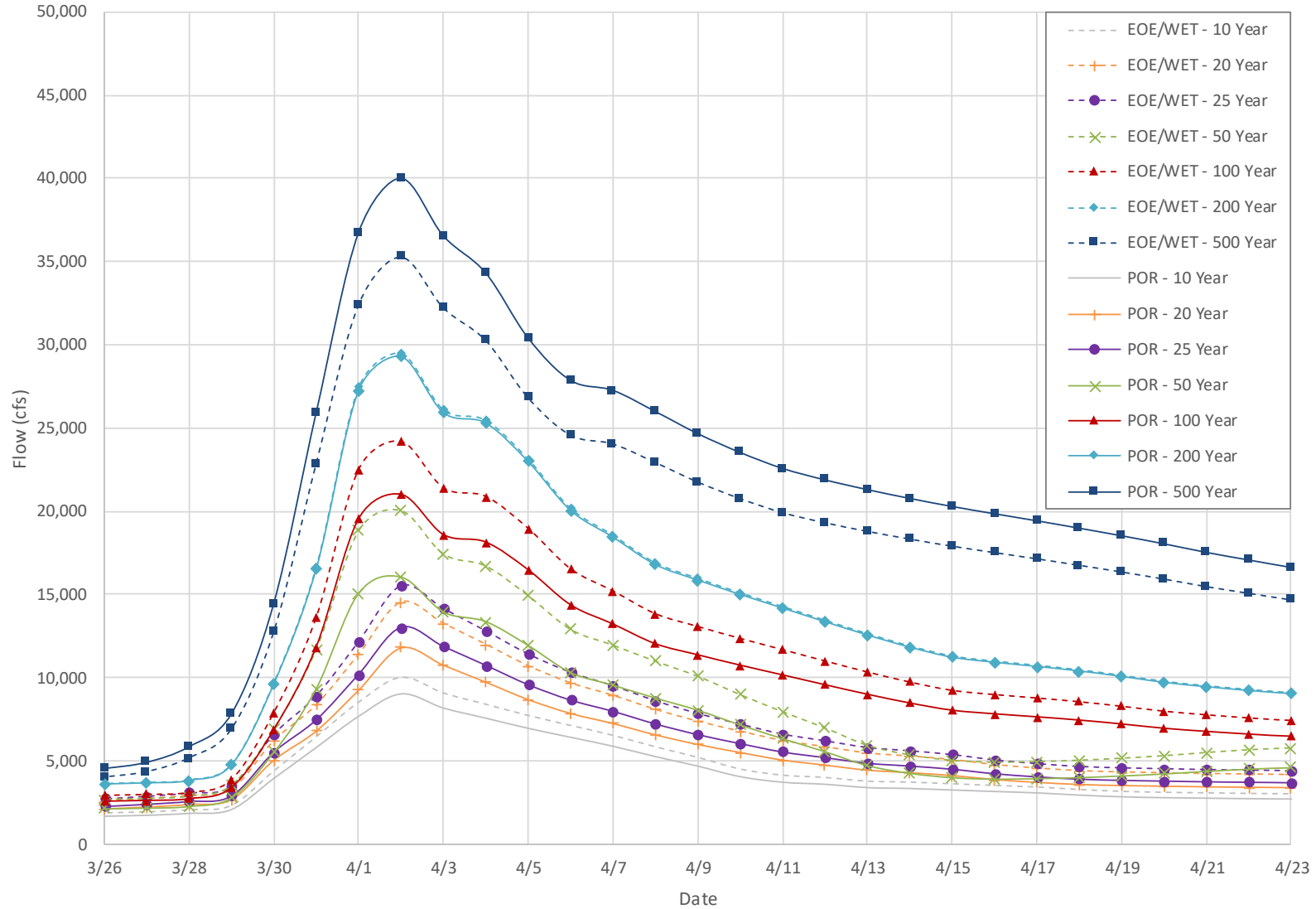


Exhibit 10 - Balanced Hydrograph Comparison at Hickson Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

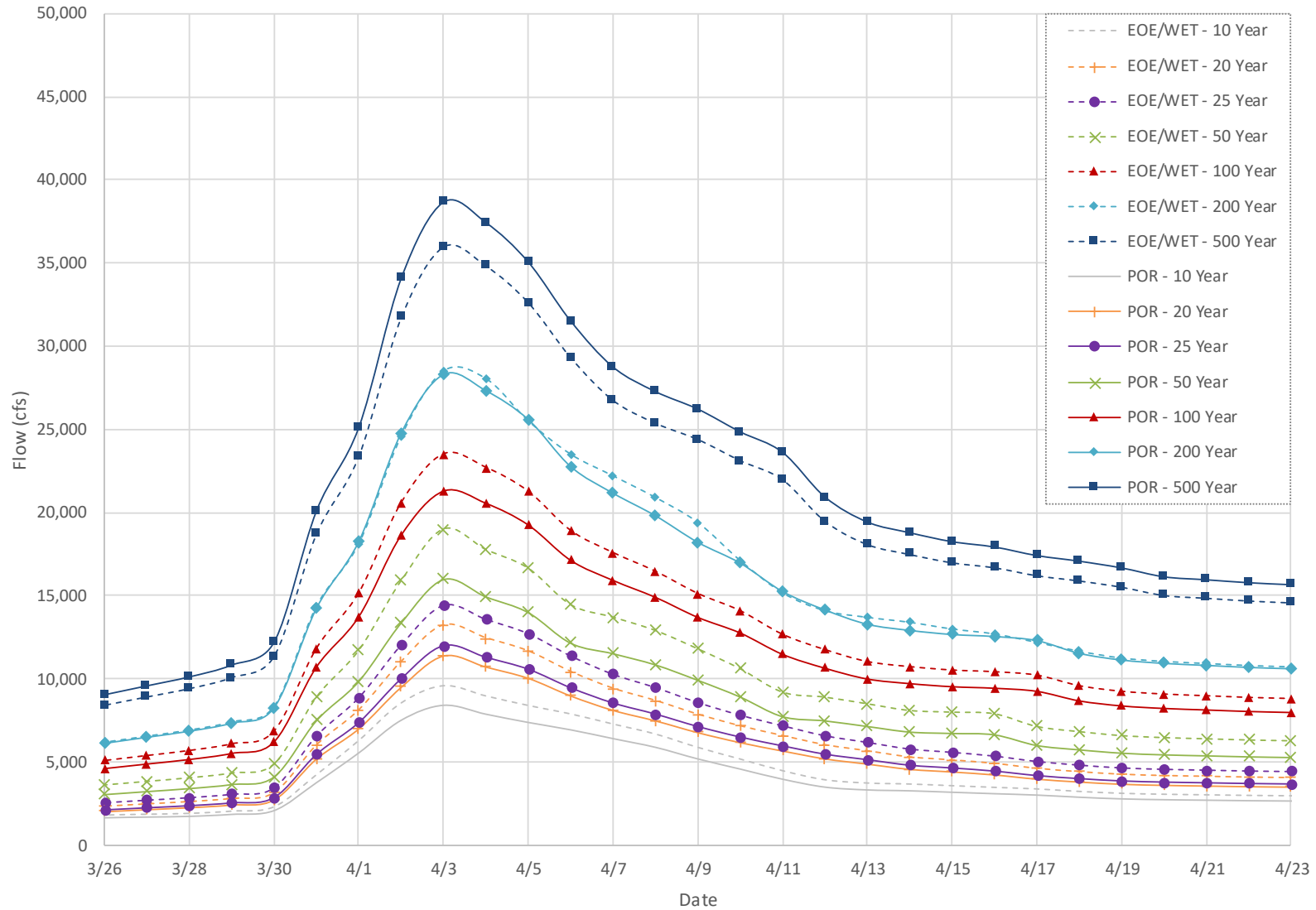


Exhibit 11 - Balanced Hydrograph Comparison at Fargo Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

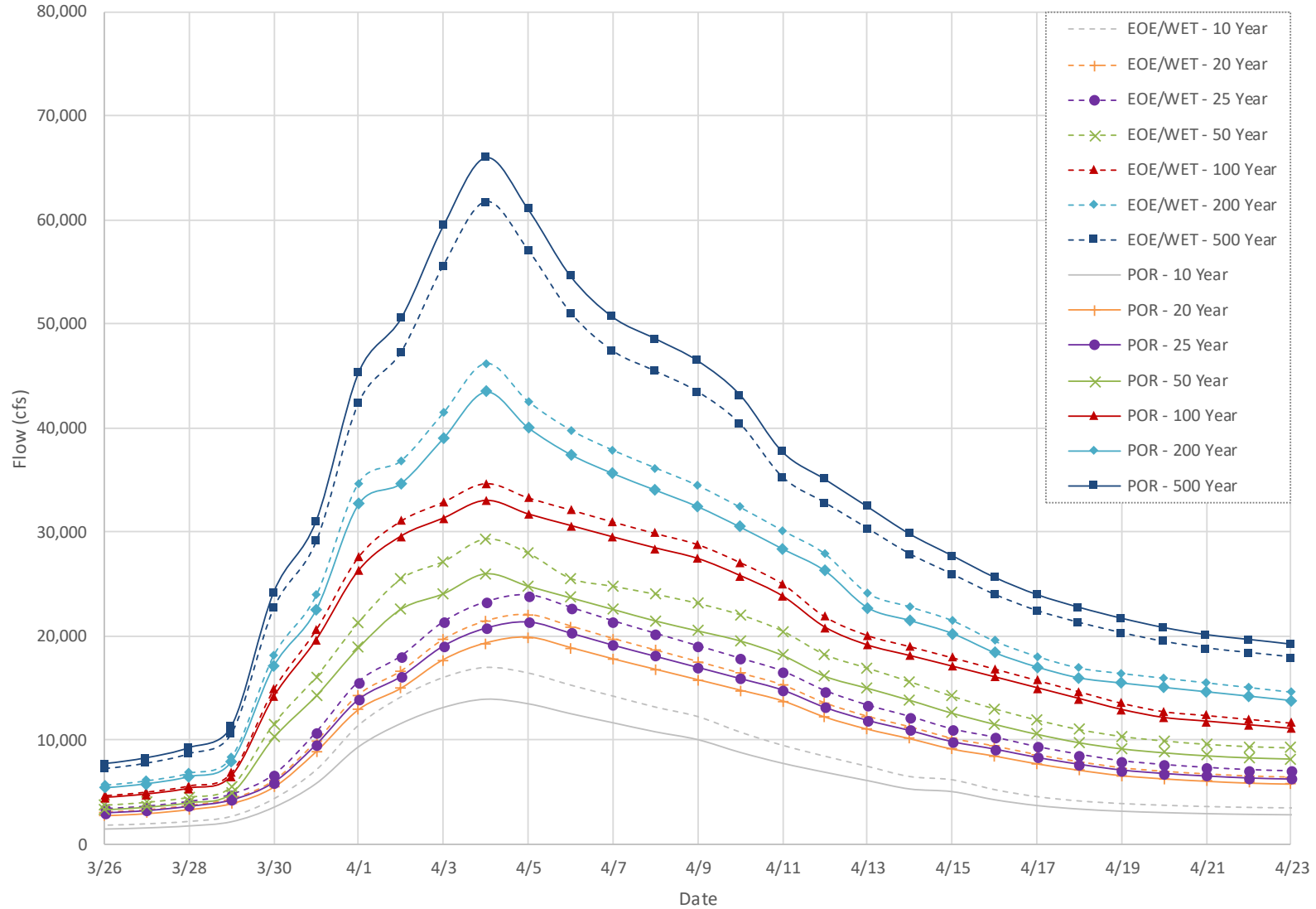


Exhibit 12 - Balanced Hydrograph Comparison at Halstad Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

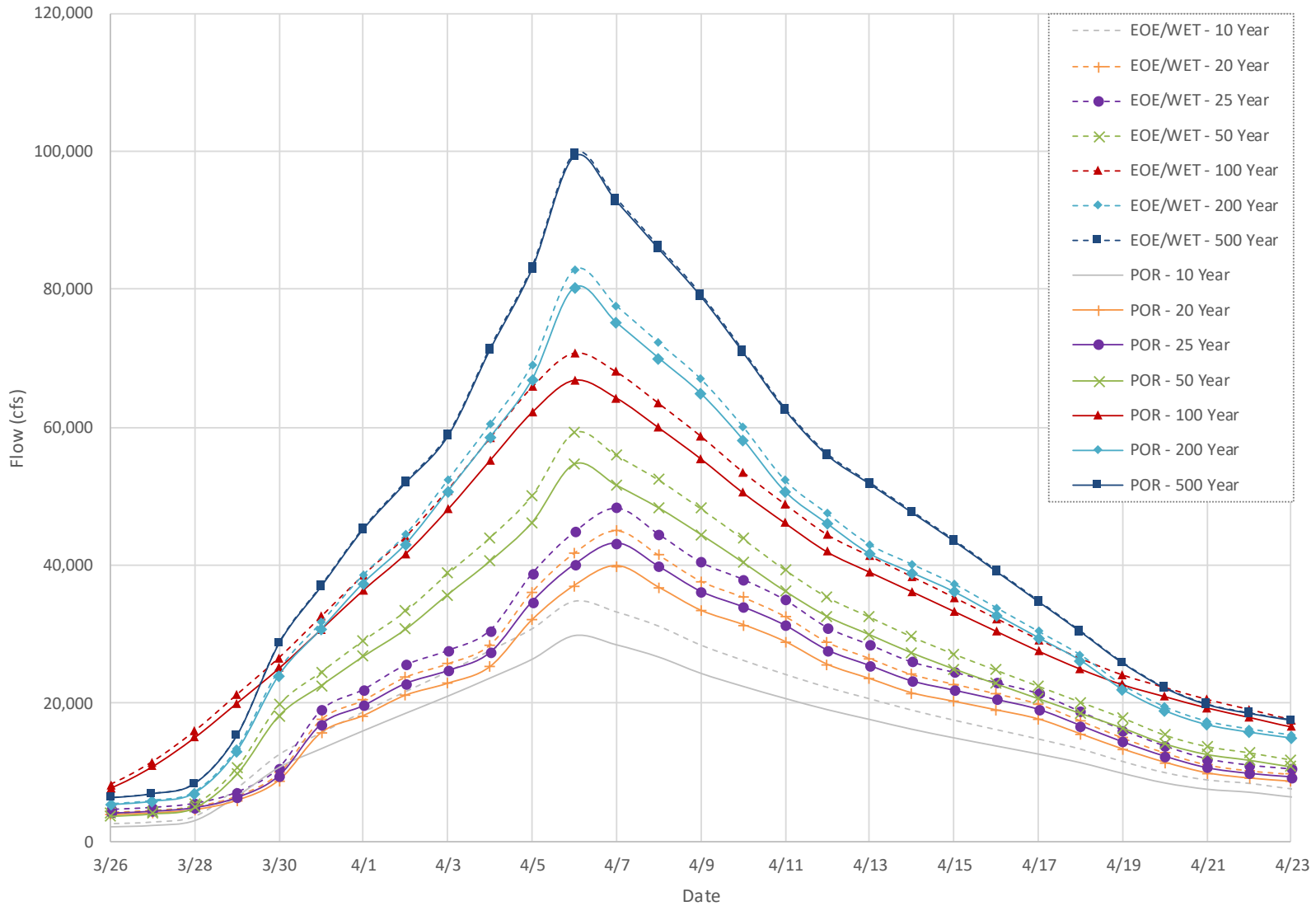


Exhibit 13 - Balanced Hydrograph Comparison at Thompson Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

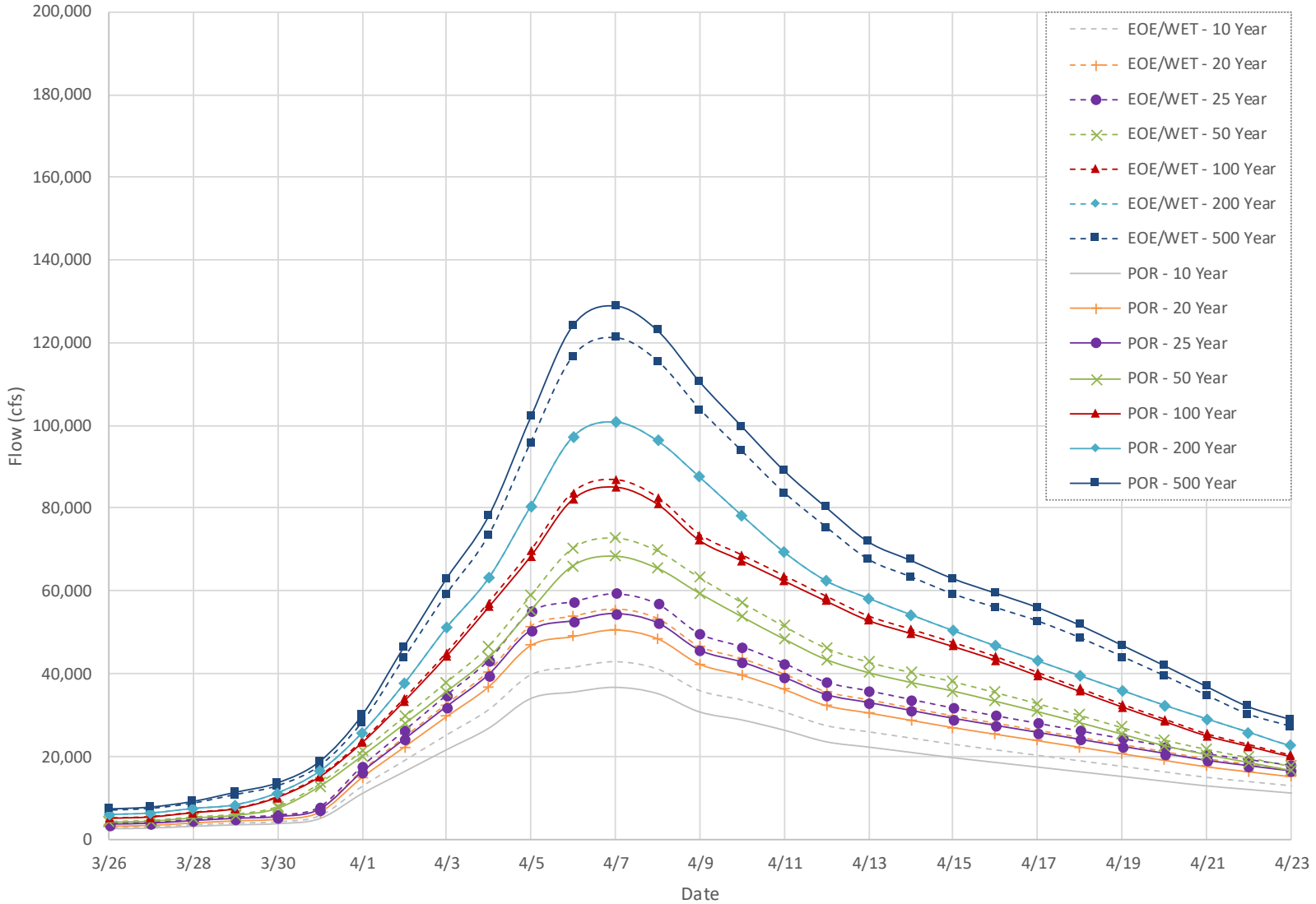


Exhibit 14 - Balanced Hydrograph Comparison at Grand Forks Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

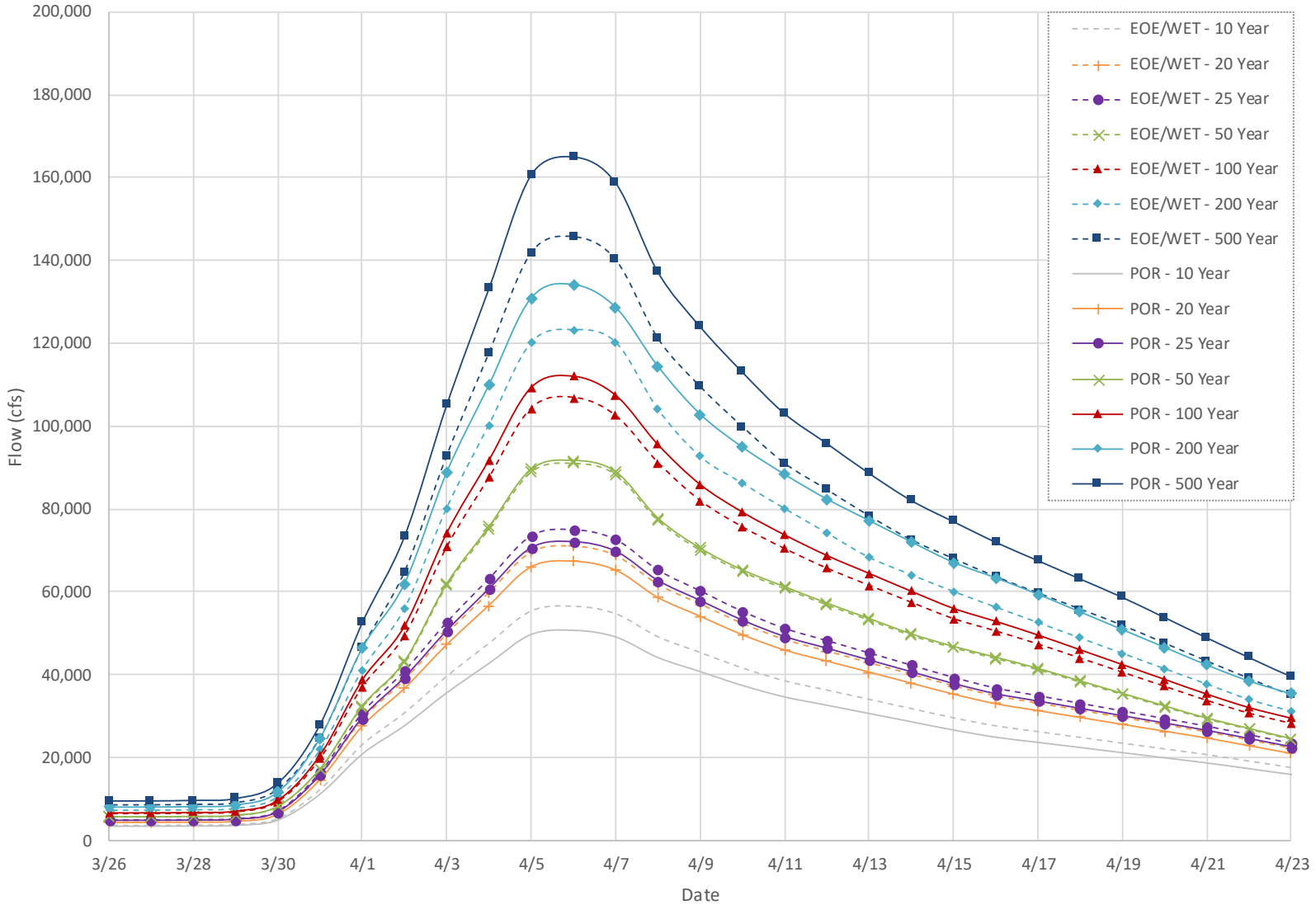


Exhibit 15 - Balanced Hydrograph Comparison at Oslo Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology

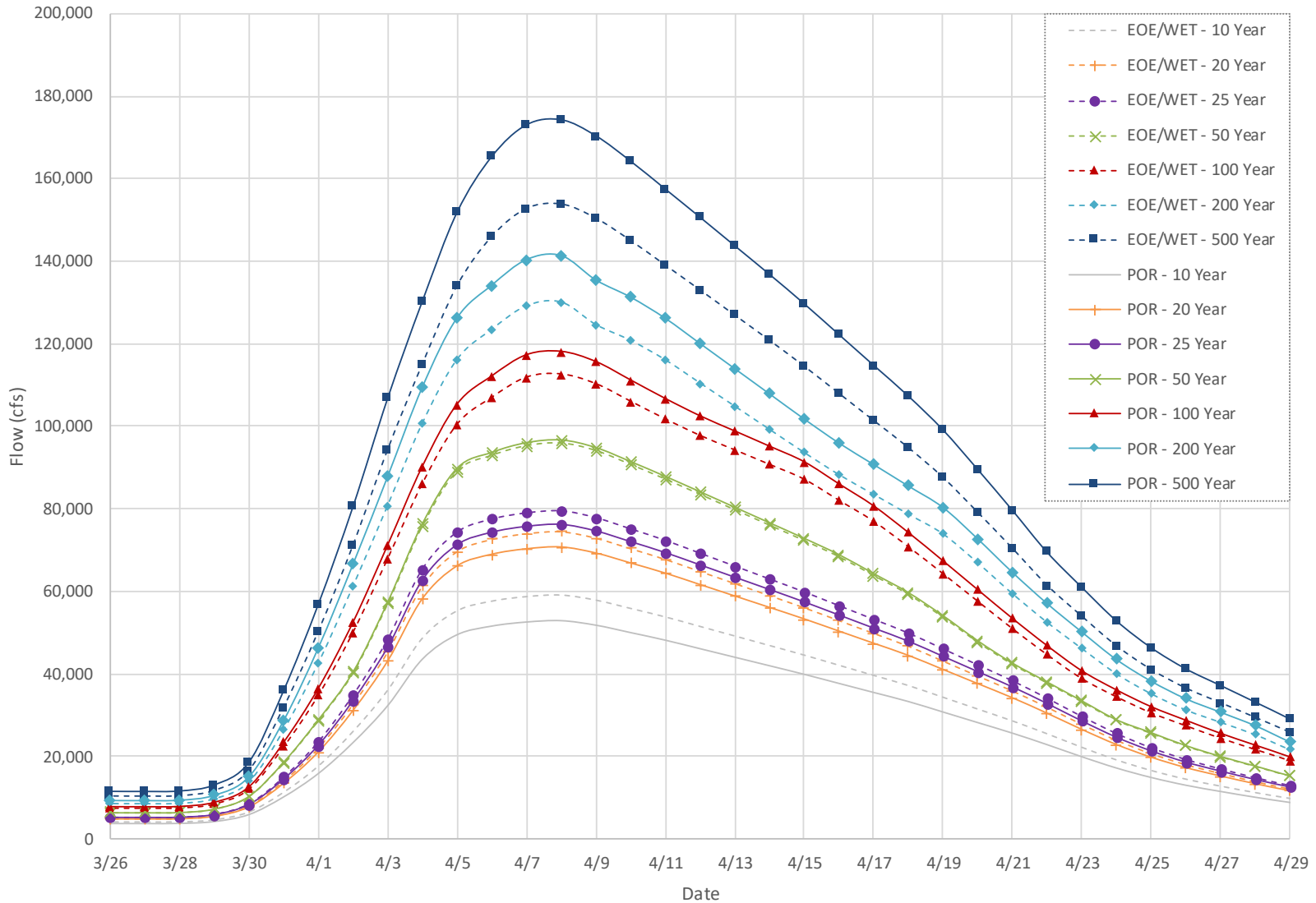
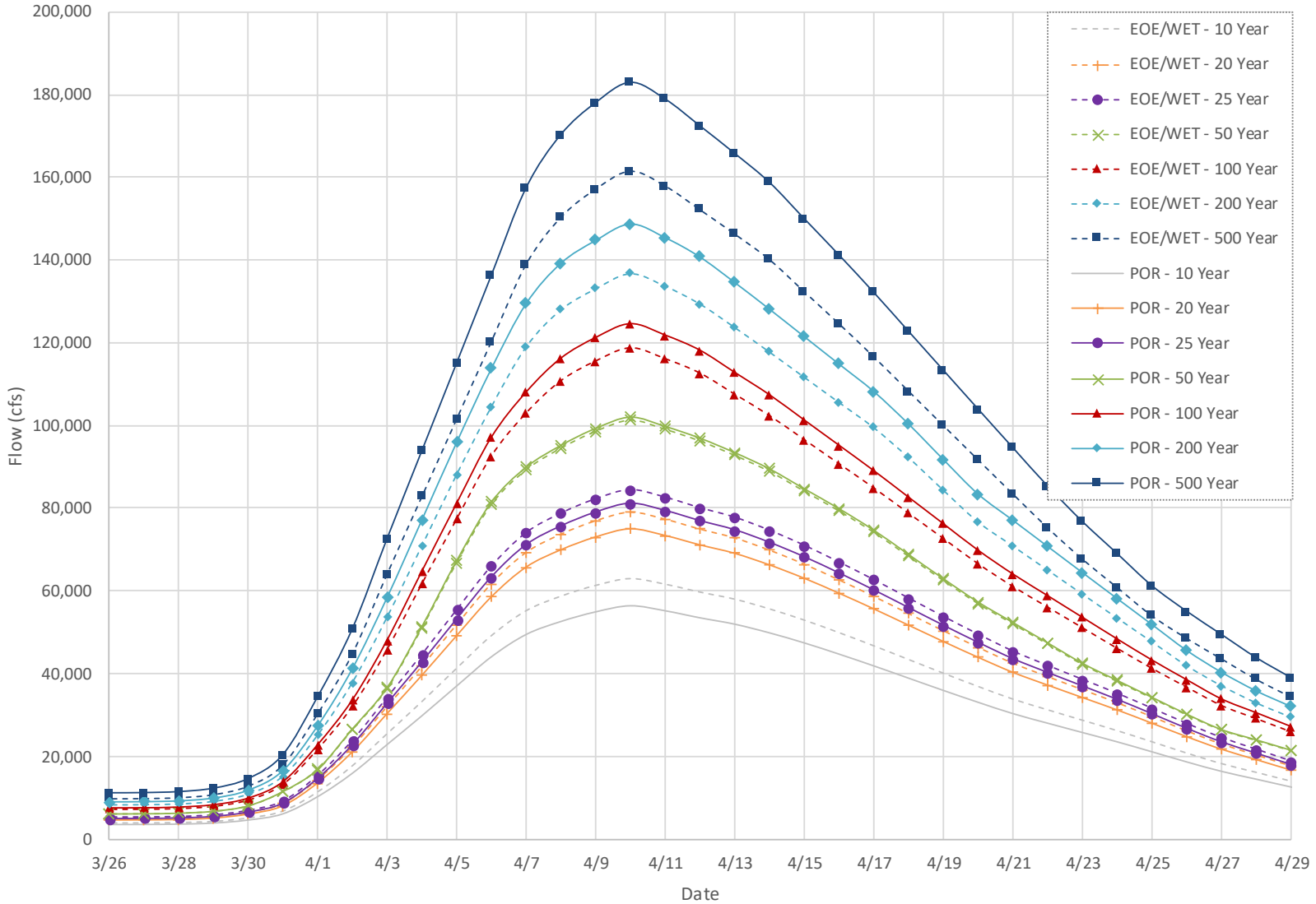


Exhibit 16 - Balanced Hydrograph Comparison at Drayton Gage
 Period of Record (POR) Hydrology vs Expert Opinion Elicitation / Wet Cycle (EOE/WET) Hydrology



Attachment 2



**US Army Corps
of Engineers®**
St. Paul District

Attachment 2: Southern Embankment Wind-Wave Analysis

Fargo Moorhead Metropolitan Area
Flood Risk Management Project

08 August 2018

1. OVERVIEW

The revised alignment of the Southern Embankment for the Fargo-Moorhead Metropolitan Flood Risk Management Project (FMM Project) requires a reevaluation of the wind-wave analysis to ensure that the top of dam elevation is sufficient to “assure that failure of the dam will not result from wind set-up, wave action, uncertainties in analytical procedures, and uncertainties in project function in combination with the most critical pool elevation” as stated in ER 1110-8-2 (FR), *Inflow Design Floods for Dams and Reservoirs* (USACE, 1991). These uncertainties include the uncertainty in the shape, duration, and peak value of the inflow hydrograph, uncertainties in operation of the gated structures, uncertainty in climate change, wind and precipitation estimates, and model uncertainty. The most critical pool elevation for the design of this project is at elevation 924.0 feet (all elevations in this document are in NAVD88), which corresponds to the maximum pool that will be maintained at the structures before flood-fighting downstream is abandoned and the Red and Wild Rice River structures are opened to maintain dam safety; flooding the Fargo-Moorhead area in the process.

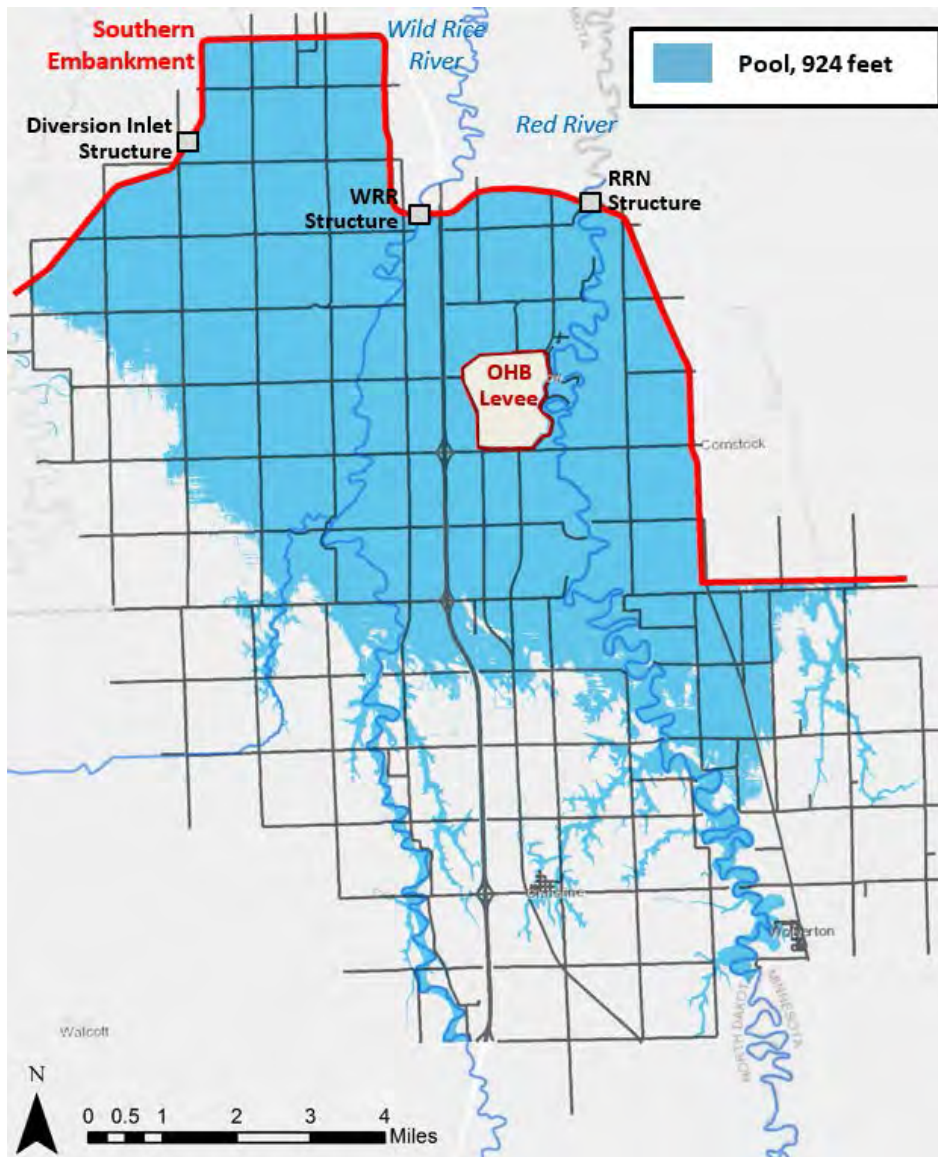
While a simple one-dimensional fetch length calculation can be used to determine wave and wind setup estimates at a structure, it may not fully capture the complexities of a pool like the FMM staging area. The new alignment of the Southern Embankment is such that adverse winds can approach from a wide range of directions. Additionally, the shallow depths of the pool and presence of raised roadways that intersect the pool create disconnected wind fetches and areas of energy dissipation (i.e. waves breaking on roadways) that are difficult to capture in a simplified 1D approach. For these reasons, a coupled 2D modeling approach is considered for this effort using a 2D hydrodynamic model (Adaptive Hydraulics Model, or AdH) to model wind setup and a 2D wave growth and transformation model (Steady State Spectral Wave, or STWAVE) to capture the wave height, wave period, and wave direction along the entire structure length. These wave characteristic outputs can then be used to calculate wave runup and wave overtopping values. This calculated required height, in combination with the vertical rise in water surface calculated in the wind setup values, can be used to ensure the top of dam elevation is designed to a sufficient height above the maximum pool level.

Traditionally for dam freeboard assessment, the required height above the Still-Water Level (SWL) is calculated as the wind setup plus the 2-percent wave runup value. This means that for a certain set of wind conditions, only 2% of all wave runup values would exceed this elevation. According to the Dutch guidance for wave runup and overtopping calculation (EurOtop Manual, 2007) “the choice for 2% has been made long ago and was probably arbitrary” and “it can be concluded that the choice for the 2% value was made in 1936, but the reason why is not clear as the design report itself could not be retrieved.” The current state of the practice focuses more on the calculation of wave overtopping rates rather than wave runup alone because the rate at which waves and water flow over the structure can be compared with rates that are thought to be hazardous to the integrity of the structure. The calculation of wave runup does not give any indication if the waves that do exceed the crest could initiate a failure of an embankment structure. Corps’ guidance for the design of dams (ER 1110-8-2) also states that “zero over-wash is not always required under infrequent high pool conditions, but it is required that the over-wash will not be of such a magnitude and duration as to threaten the safety of the dam.” Therefore, while this wind-wave analysis will consider the calculation of the required top of dam elevation based on both limiting wave overtopping rates and on containing the wave runup elevation, the primary focus of

this effort will be to ensure the top of dam limits wave overtopping to acceptable levels for the embankment that is being designed.

An overview map of the Southern Embankment and a level pool of 924.0 feet is shown in Figure 1. The Southern Embankment (shown in red) is generally aligned east-west, with three gated control structures (Red River of the North Structure, Wild Rice River Structure, and the Diversion Inlet Structure) located along the alignment. The Diversion Inlet Structure passes flows downstream around the Fargo-Moorhead area through a 30-mile long diversion channel. The Oxbow-Hickson-Bakke (OHB) ring levee provides flood risk management for three local communities with the FMM project in place.

Figure 1: Overview Map for Wind-Wave Analysis



2. METHODS

2.1. Wind Analysis

Previous analysis for fetch-limited duration winds of around 1 hour in duration (adjusted for over-water conditions) give wind frequency estimates as shown in Table 1. These values were developed as part of the design for OHB ring levee (USACE, 2015) and represent all-seasonal, all-directional winds.

Table 1: Fetch-limited, adjusted wind speed frequencies

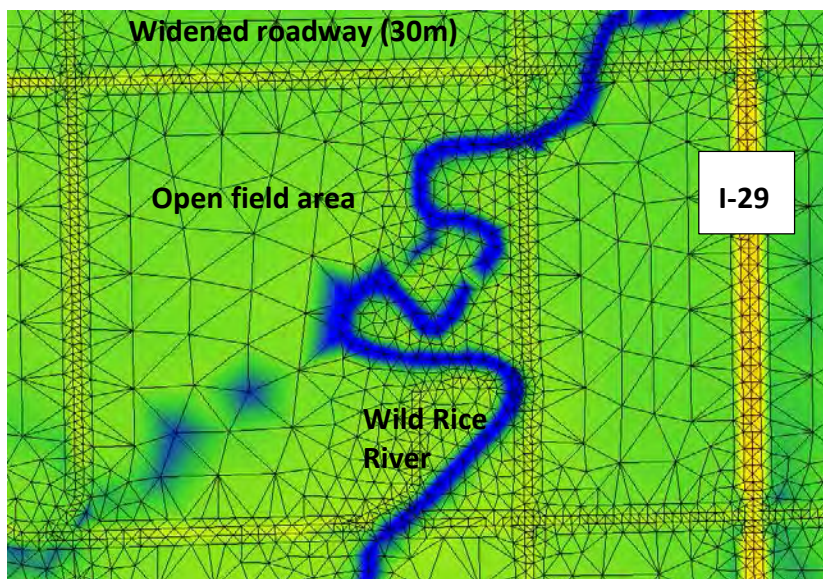
Annual Chance Exceedance (ACE)	Return Period (years)	Wind Speed (mph)
1%	100	48.8
2%	50	47.4
10%	10	44.5
50%	2	40.9

The wind speeds for frequencies between the 1% and 50% ACE are bracketed by wind speeds of 40 mph and 50 mph. Therefore, the wind setup and wave calculations are made for those two wind speeds, allowing for interpolation between to roughly approximate the various frequency wind speeds.

2.2. Hydrodynamic Model

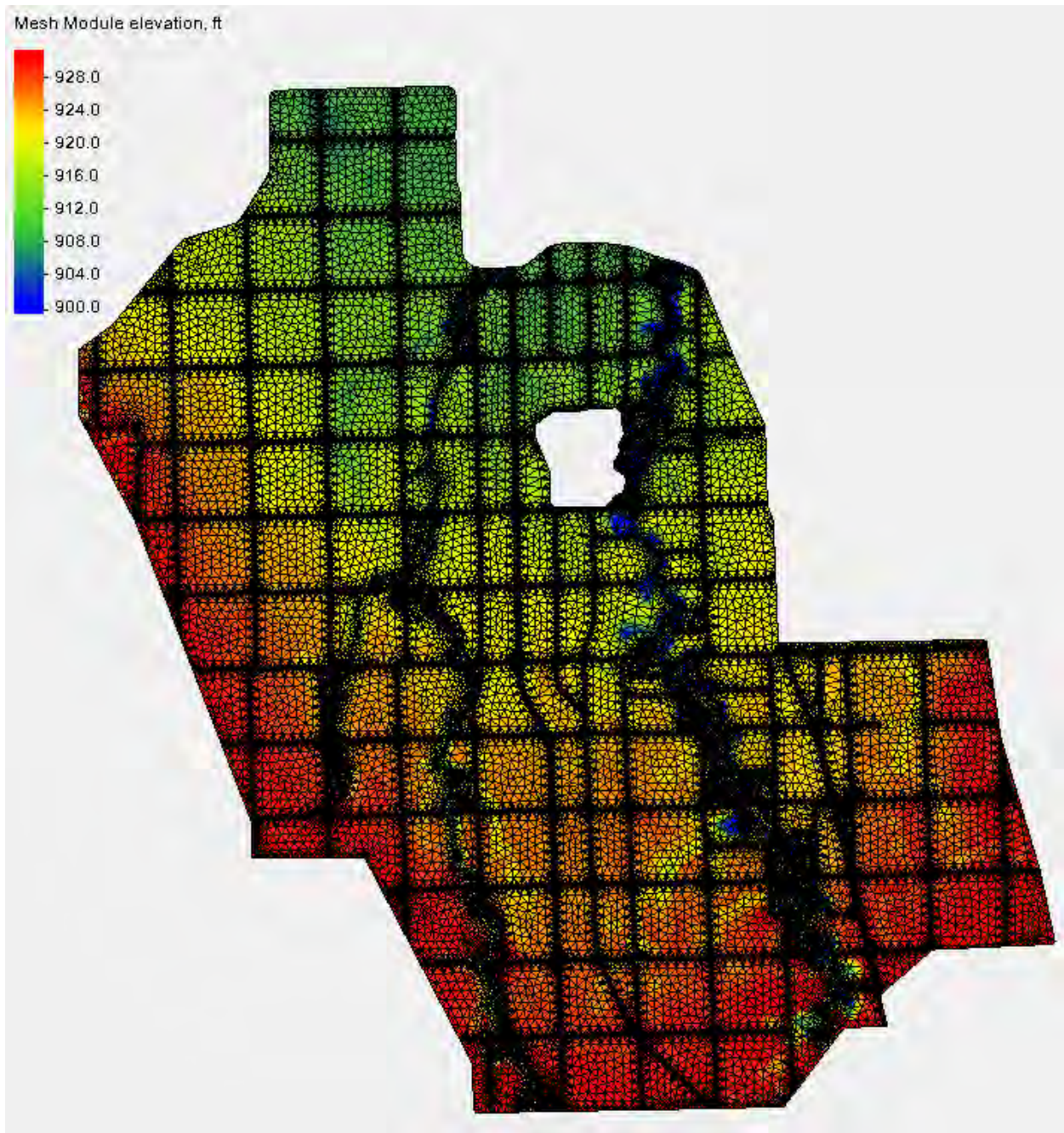
The hydrodynamic model used for calculating wind setup is AdH. The irregular mesh model was developed in metric units with spacing of 30 meters along roadways, 60 meters along riverine features, up to 250 meters in open field areas. Figure 2 shows an example of the AdH element spacing. The roadways are artificially widened to capture a broad crested structure rather than a modeled triangular structure that could allow flow to “leak” through artificial low points. Culverts are not included in this model but are not conveying much total volume compared to the extreme flood magnitudes considered for this effort. Additionally, Interstate 29 which runs north to south through the project area has been raised to the 0.2% pool elevation of 922.5 feet.

Figure 2: Typical irregular mesh element spacing



The AdH mesh extends to the 929 feet elevation contour for sufficient coverage. The mesh and its extents are shown in Figure 3.

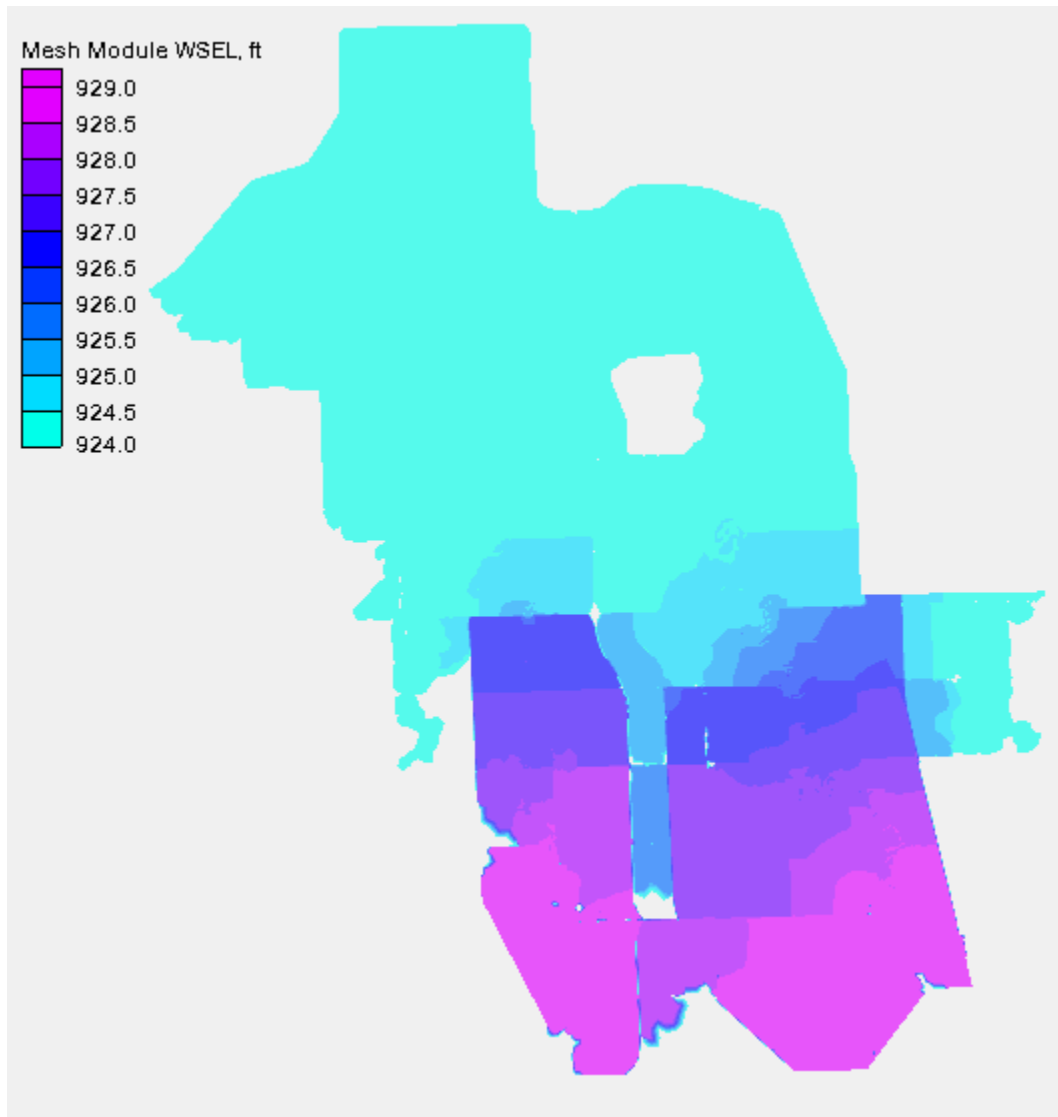
Figure 3: AdH Mesh with elevation contours



The hydrologic event modeled for this effort is the maximum pool level maintained before the gated structures are fully opened to maintain dam safety. This event, which is associated with an inflow of less than half the PMF inflow, results in a water surface elevation of 924.0 feet at the embankment and structures and has a 1/2000 annual chance exceedance probability. The large inflow associated with a maximum pool event causes a sloping water surface to be a more accurate assumption than a flat pool

of 924.0 feet. To capture this sloping pool, an inflow of 102,000 cfs is modeled at the upstream end to reflect the appropriate hydrologic conditions. The results of this sloping pool are shown in Figure 4.

Figure 4: Water surface elevations accounting for sloping pool with 102,000 cfs inflow



The sloping pool allows for more accurate wind fetches and average depths across the wind fetch to produce more realistic wind setup and wave height values. The downstream boundary is controlled at each of the hydraulic structures (Diversion Inlet, Wild Rice River Structure, and Red River Structure) by holding an elevation of 924.0 feet. This boundary condition limits wind setup from growing immediately at the structure, however, due to the large capacity of the gated structures this assumption is viewed as realistic because the operator would not allow the pool to go higher than the maximum pool simply because it may be a wind-induced rise.

2.3. Wave Growth & Transformation Model

STWAVE model is a regular gridded model, meaning that the x and y length of each cell are constant. This model was setup with a 30m by 30m cell size. The depth value for each cell was based on the sloping water surface hydraulic profile generated in AdH. Each STWAVE simulation for the various wind directions and wind speeds assumed the same depth values, so additional depth due to wind setup was not included in the wave calculation. This was a simplifying assumption that allowed for a drastically reduced modeling effort with a slight decrease in model accuracy. Since wind-set-up is not included in the simulation, the depths passed to STWAVE will be smaller than in reality, which could result in artificially lower wave heights, if the waves are depth-limited. An overview map of the depth contours (up to 10 feet of depth) for the gridded STWAVE model is shown in Figure 5. A zoomed in area of the gridded model, highlighting the 30 meter cell size, is shown in Figure 6.

Figure 5: Overview of depth data for the STWAVE model

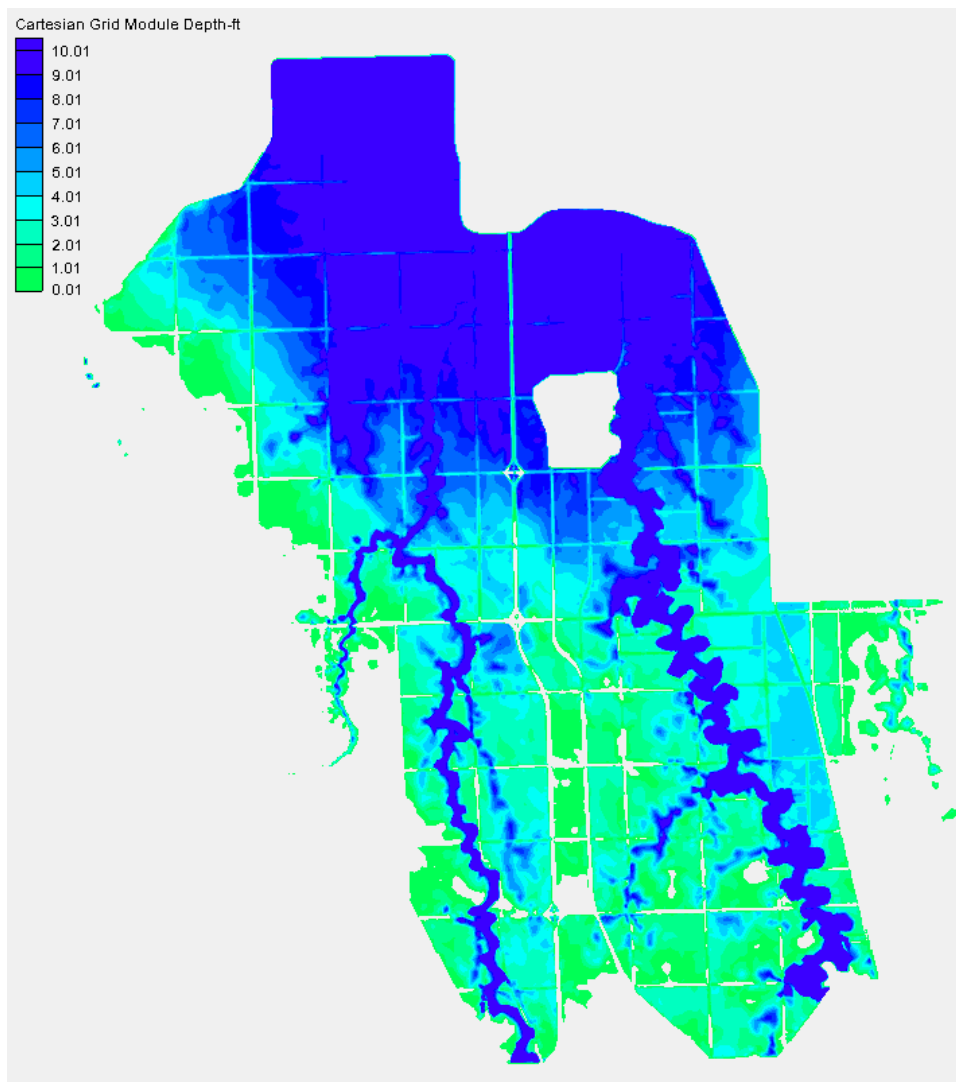
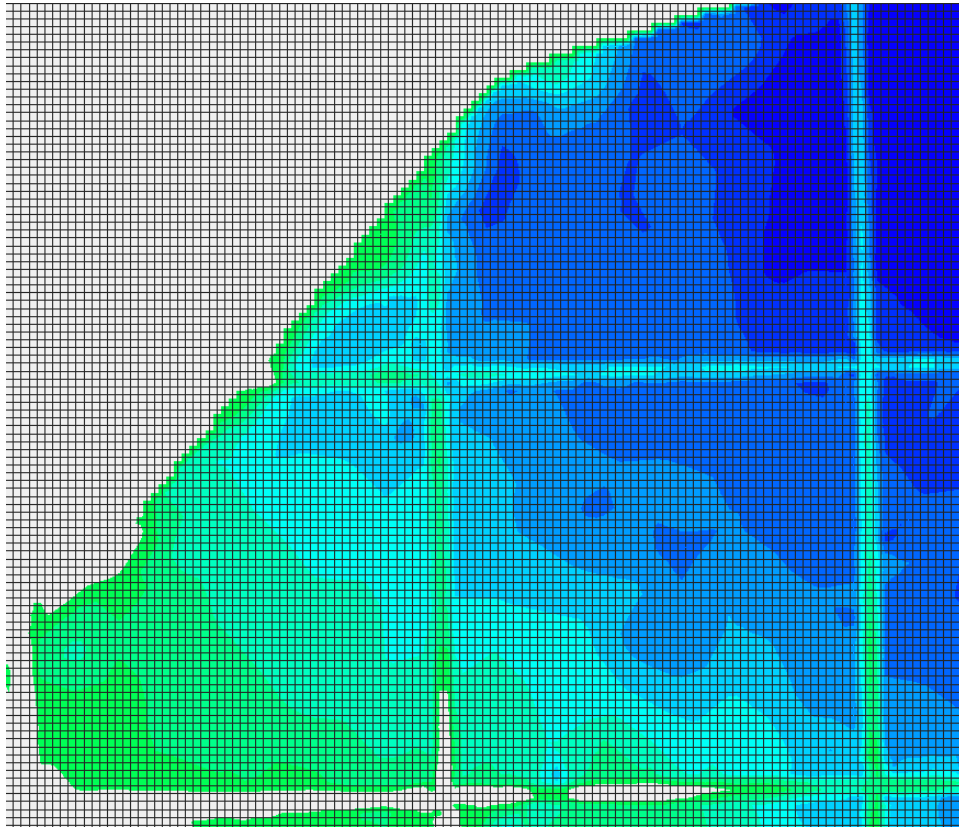


Figure 6: Selected area of the 2D regular grid STWAVE model showing the fidelity of 30 meter cells



3. RESULTS

3.1. Wave Height, Period, and Direction Results

See Figures 13 - 17 at the end of this report for the wave height and wave direction results for the 9 modeled wind directions and 2 modeled wind speeds. See supporting file "FMM_WindWave_Results_20180405.xlsx" for the full wave characteristic results.

3.2. Wind Setup Results

See Figures 18 - 22 at the end of this report for the wind setup results for the 9 modeled wind directions and 2 modeled wind speeds. See supporting file "FMM_WindWave_Results_20180405.xlsx" for the full wind setup results.

3.3. Wave Overtopping and Wave Runup Results

The required top of dam elevation was calculated in two ways. The primary method used in this analysis was to determine the required height to limit wave overtopping rates to acceptable levels. The following equation (van der Meer and Janssen) from the Coastal Engineering Manual was selected:

Wave overtopping equation:

Table VI-5-11
Overtopping Formula by van der Meer and Janssen (1995)

Straight and bermed impermeable slopes including influence of surface roughness, shallow foreshore, oblique, and short-crested waves, Figures VI-5-14a and VI-5-14b.

$$\xi_{op} < 2$$

$$\frac{q}{\sqrt{g H_s^3}} \sqrt{\frac{s_{op}}{\tan \alpha}} = 0.06 \exp \left(-5.2 \frac{R_c}{H_s} \frac{\sqrt{s_{op}}}{\tan \alpha} \frac{1}{\gamma_r \gamma_b \gamma_h \gamma_\beta} \right) \quad (\text{VI-5-24})$$

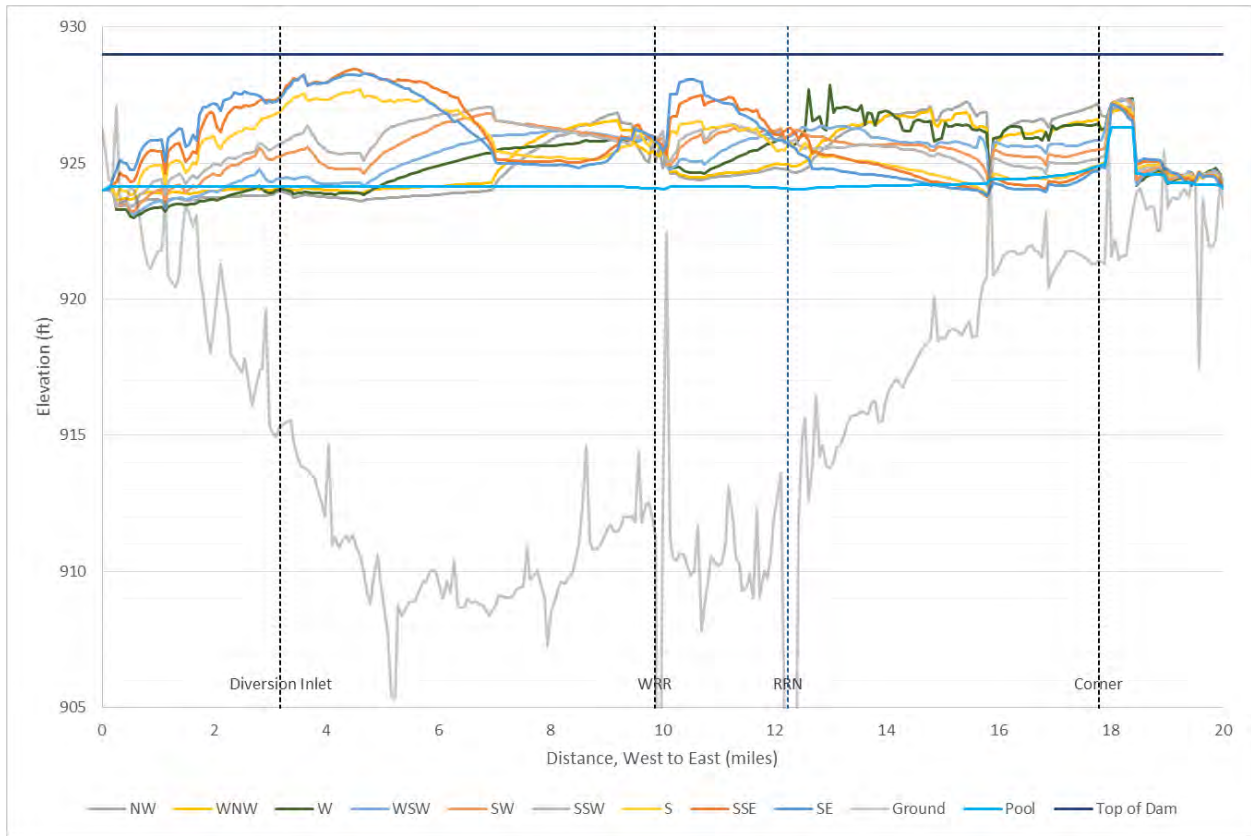
$$\text{application range: } 0.3 < \frac{R_c}{H_s} \frac{\sqrt{s_{op}}}{\tan \alpha} \frac{1}{\gamma_r \gamma_b \gamma_h \gamma_\beta} < 2$$

Uncertainty: Standard deviation of factor 5.2 is $\sigma = 0.55$ (See Figure VI-5-15).

The wave heights, periods, and embankment slope are such that the breaker parameter (ξ_{op}) for all conditions is below 2, so this equation is appropriate. An allowable overtopping rate (q) of 0.01 cfs/ft was selected to use to solve for the required height (R_c). Reduction factors (γ) were not considered (i.e. set to 1) with the exception of γ_β , which was estimated from the angle between the wave direction angle and the orientation of the embankment. The slope of the embankment ($\tan \alpha$) was set to 1:4.

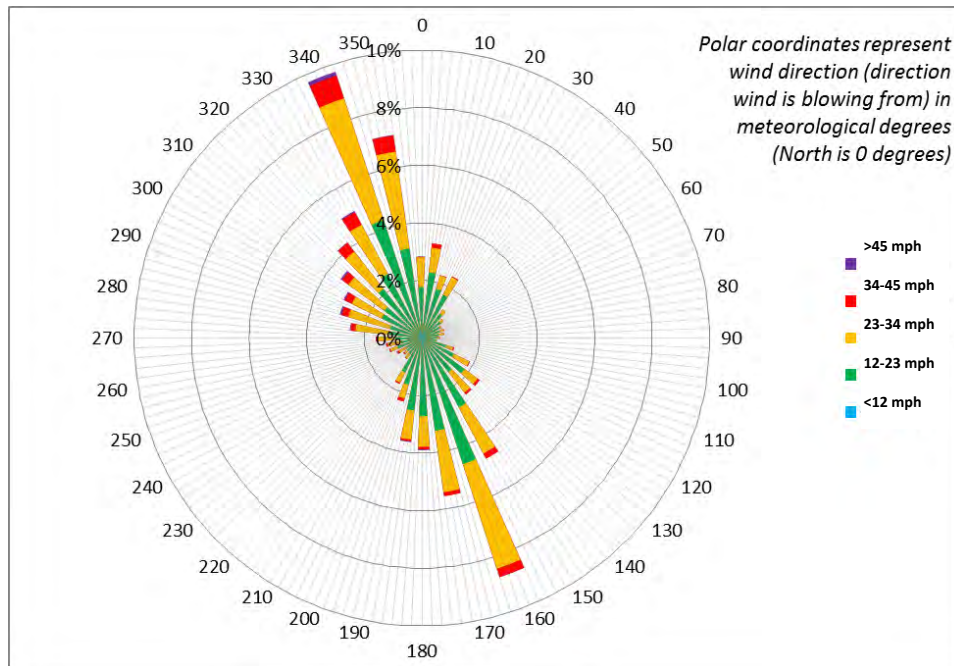
Using the wave overtopping equation and the 50 mph wave and wind setup results, the required design elevation does not exceed 929 feet at any location; however minimum freeboard requirements may result in higher top of dam elevations. Figure 7 shows the summary for all wind directions along the Southern Embankment alignment, from west to east.

Figure 7: Summary of Required Height to limit Wave Overtopping for 50 mph winds (elev., ft NAVD88)



The two most concerning wind directions are the SE and NW directions for various reasons. Firstly, as shown in the wind rose in Figure 8, the most dominant wind directions for the fastest 2-minute winds in the Fargo-Moorhead area are from these two directions.

Figure 8: Wind Rose for 2-minute daily wind data in Fargo, ND



Secondly, the SE and NW directions are the most concerning based on the wind-wave modeling results. The SE direction has the longest wind fetch lengths of any of the directions modeled due to the generally longer north-south inundation than east-west inundation and the additional length of embankment that extends north of the three structures. The required freeboard is highest at this north extension area, however, it does not exceed 5 feet for 50 mph wind conditions as shown in Figure 9.

The NW wind direction is critical in that the sloping water surface elevation of the pool leaves portions of the most upstream extent of the Southern Embankment with less than 5 feet of freeboard and NW winds create the longest fetch length for the areas with higher water surface elevations. However, using the wave overtopping criteria, the 50 mph winds from the NW do not require the top of dam to exceed elevation 929.0 feet, as shown in Figure 10.

Figure 9: Required top of dam height for SE 50 mph wind conditions based on wave overtopping

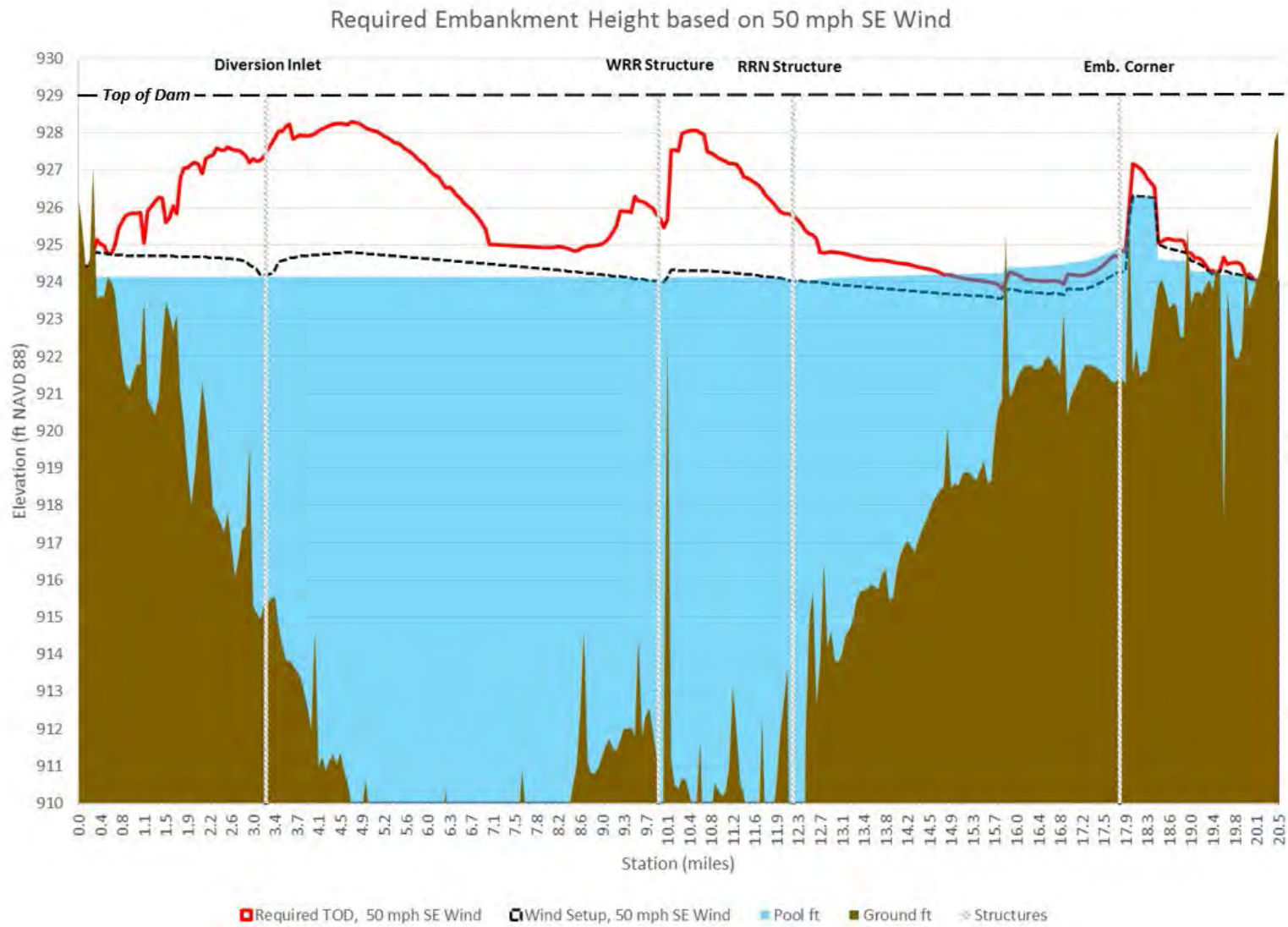
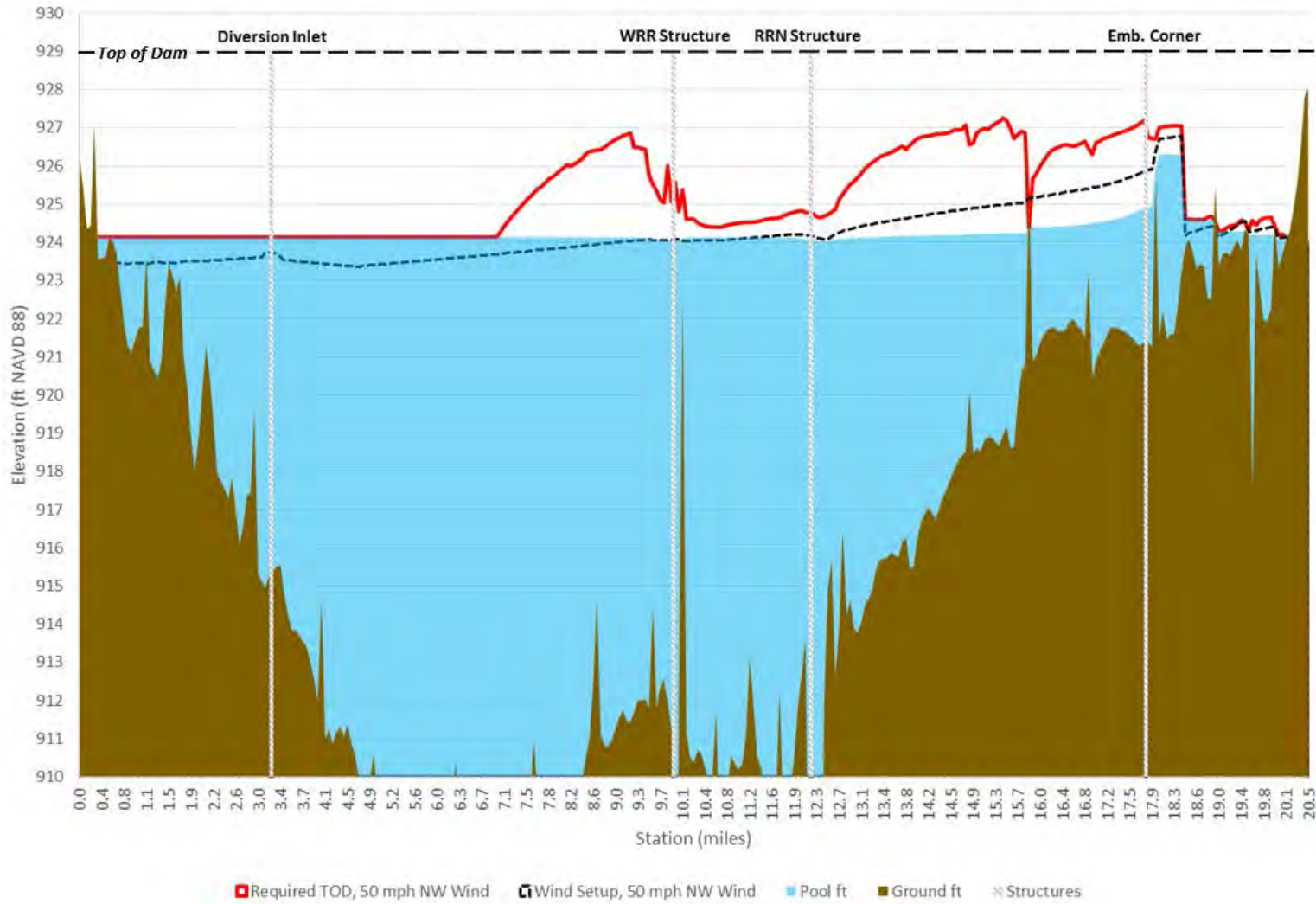


Figure 10: Required top of dam height for NW 50 mph wind conditions based on wave overtopping

Required Embankment Height based on 50 mph NW Wind



A more traditional approach to determining the required top of dam elevation above the pool level is to consider the 2% wave runup value. As discussed in the overview section of this analysis, the 2% runup is “arbitrary” and some amount of wave overtopping is allowable. The wave runup approach was considered, however, as a comparison to the wave overtopping calculation, using the following equation:

Wave runup equation:

$$\frac{R_{mi\%}}{H_s} = (A\bar{\zeta} + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \quad (\text{VI-5-3})$$

where

$R_{mi\%}$ = runup level exceeded by i percent of the incident waves

$\bar{\zeta}$ = surf-similarity parameter, $\bar{\zeta}_{om}$ or $\bar{\zeta}_{op}$

A, C = coefficients dependent on $\bar{\zeta}$ and i but related to the reference case of a smooth, straight impermeable slope, long-crested head-on waves and Rayleigh-distributed wave heights

γ_r = reduction factor for influence of surface roughness ($\gamma_r = 1$ for smooth slopes)

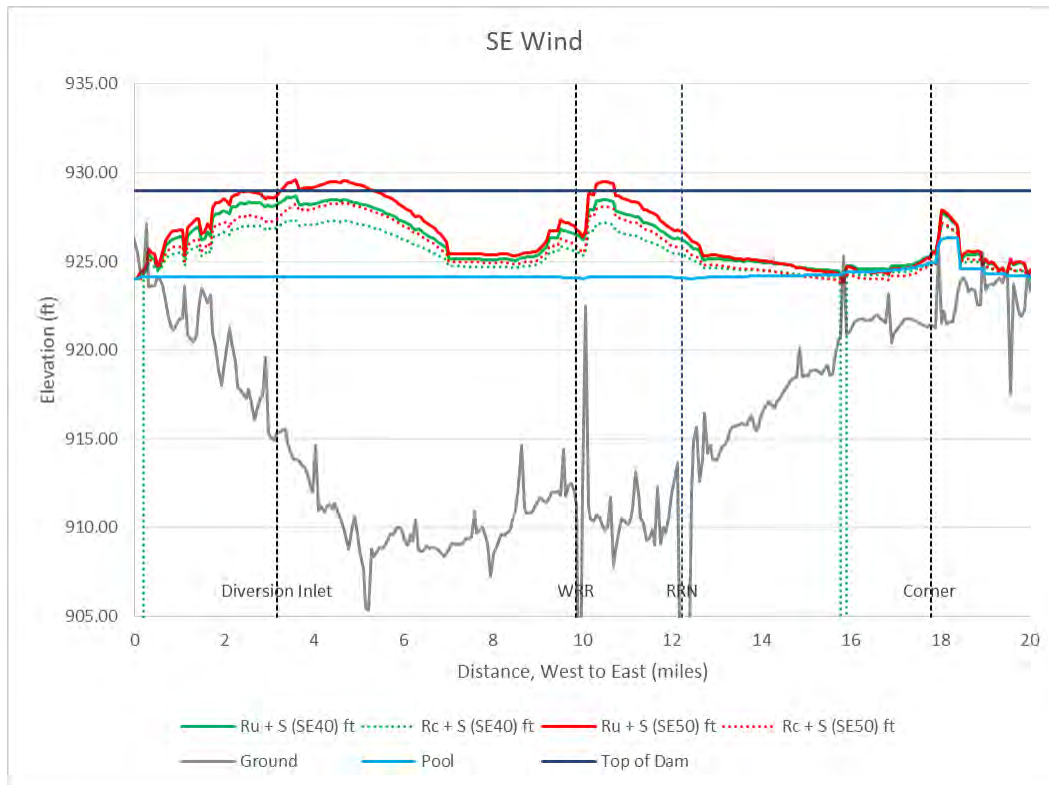
γ_b = reduction factor for influence of a berm ($\gamma_b = 1$ for non-bermed profiles)

γ_h = reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution ($\gamma_h = 1$ for Rayleigh distributed waves)

γ_β = factor for influence of angle of incidence β of the waves ($\gamma_\beta = 1$ for head-on long-crested waves, i.e., $\beta = 0^\circ$). The influence of directional spreading in short-crested waves is included in γ_β as well

Similar to the wave overtopping approach, the only reduction factor included was for the angle of incidence of the waves. Coefficients A & C are based on values for the $i = 2\%$. Using this approach, some areas in the northern most portion of the embankment require that the top of dam height above the pool to exceed 5 feet. However, this is only the case for southeasterly wind directions of 50 mph or greater. The 40 mph SE winds do not result in wave runup plus wind setup values greater than 5 feet and. As mentioned previously, allowable wave overtopping rates do not result in a requirement of greater than 5 feet for 40 or 50 mph southeasterly winds. Figure 11 shows the results for SE winds of both 40 and 50 mph, using both approaches to estimating the required top of dam elevation.

Figure 11: Required top of dam height for SE winds, for 40 & 50 mph, using both the allowable wave overtopping approach ($R_c + S$) and the more conservative wave runup approach ($R_u + S$)



These areas that are exceeded by the traditional wave runup calculation, the northern most extension and the area between the Wild Rice and Red River structures, should be the highest area of concern for considering alternative measures to mitigate the risk of floodside erosion from waves, however, the entire Southern Embankment alignment should have reasonable assurance that “failure of the dam will not result from wind set-up” or “wave action” with a top elevation of 929.0 feet and a maximum pool of 924.0 feet.

See supporting file “FMM_WindWave_Results_20180405.xlsm” for the required top of dam elevation estimates based on the allowable wave overtopping approach and the 2% wave runup approach.

3.4. Wind Frequency and Coincident Probability

The maximum pool of 924.0 feet is estimated to represent an event with a frequency of $5E-4$ (1/2000) ACE. The estimated duration that the pool would be within 3 feet of the maximum level for that event is 4 days. The chance of a wind event occurring resulting in severe wave runup heights or wave overtopping rates is estimated to be remote, as shown in the following simplified probability calculation for an upper bound estimate:

Probability	Value	Description
p(40 mph wind)	~0.5	annual chance exceedance that 40 mph wind occurs
p(924 ft pool)	0.0005	annual chance exceedance that 924.0 foot pool occurs
Correlation	1	upper bound correlation between wind and pool
p(40 mph 4 days)	$(1 - (1 - 0.5)^4) = 0.9375$	chance that at least one day has winds exceeding 0.5 ACE
Joint Probability	0.00047	$p(924 \text{ ft pool}) * p(40 \text{ mph} 4 \text{ days}) * \text{correlation}$
Return Period	2,133	Expected recurrence interval, in years

This is considered an upper bound estimate to the probability of the 0.5 ACE wind occurring coincident with the 1/2000 ACE pool because the correlation value is set to the maximum value of 1. This assumes that the wind frequency estimate is precisely calculated for the four day period of concern and for the wind fetch direction of concern. In reality, the wind frequency estimate reflects all-season and all-directional wind speeds. The joint probability of this wind & pool event would likely be reduced further by either refining the correlation value or refining the wind speed frequency by seasonality or direction. Ultimately, however, this upper bound estimate is sufficient as it is meant to demonstrate the likelihood of the 40 mph wind event where wave runup and wave overtopping are not at concerning levels. The next calculation shows the same approach for a more infrequent wind of 50 mph:

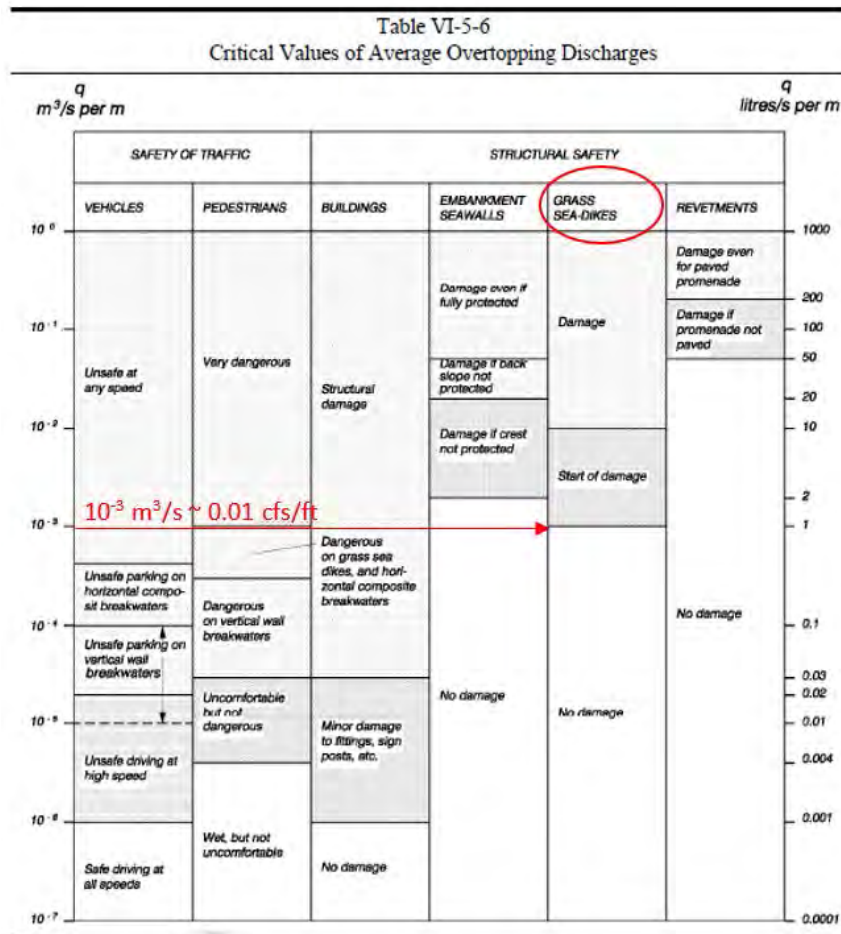
Probability	Value	Description
p(50 mph wind)	~0.01	annual chance exceedance that 50 mph wind occurs
p(924 ft pool)	0.0005	annual chance exceedance that 924.0 foot pool occurs
Correlation	1	upper bound correlation between wind and pool
p(50 mph 4 days)	$(1 - (1 - 0.01)^4) = 0.0394$	chance that at least one day has winds exceeding 0.01 ACE
Joint Probability	1.97E-05	$p(924 \text{ ft pool}) * p(50 \text{ mph} 4 \text{ days}) * \text{correlation}$
Return Period	50,756	Expected recurrence interval, in years

The 50 mph wind speed does not result in concerning wave overtopping rates but does highlight some portions of the embankment that exceed the 5 feet required for a traditional wave runup analysis. This coincident probability calculation is meant to demonstrate the remote probability that wave conditions result in wave runup values higher than the crest but wave overtopping rates below tolerable levels. Assuming this more infrequent wind condition results in a joint probability less frequent than 1/50,000 ACE. This is still considered an upper bound because the correlation is assumed to be 1; it is simply a matter of the low likelihood that a 4 day period would result in an exposure to a 0.01 ACE wind.

4. CONCLUSIONS

A top of dam height that limits wave overtopping to 0.01 cfs/ft would, according to guidance from the Coastal Engineering Manual limit the backside flows to the “Start of Damage” for ‘Grass Sea-Dikes’. Due to the non-erosive nature of the cohesive clay embankment material and the anticipated well-maintained vegetation for the project, significant erosion would not be associated with overtopping rates of 0.01 cfs/ft. Figure 12 shows the critical overtopping rate estimates associated with various levels of damage for different types of structures.

Figure 12: Critical Values of Average Overtopping Discharges from the Coastal Engineering Manual.



Due to the remote probability (1/50,000 ACE) of a 50 mph 1-hour sustained wind occurring coincidentally with the 1/2000 ACE pool level and the fact that this remote event still results in wave overtopping rates below critical levels where damage may be expected to start, a top of dam elevation of 929.0 feet is sufficient; however minimum freeboard requirements may result in higher top of dam elevations

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Figure 13: Wave heights in feet for winds from the southeast and south-southeast

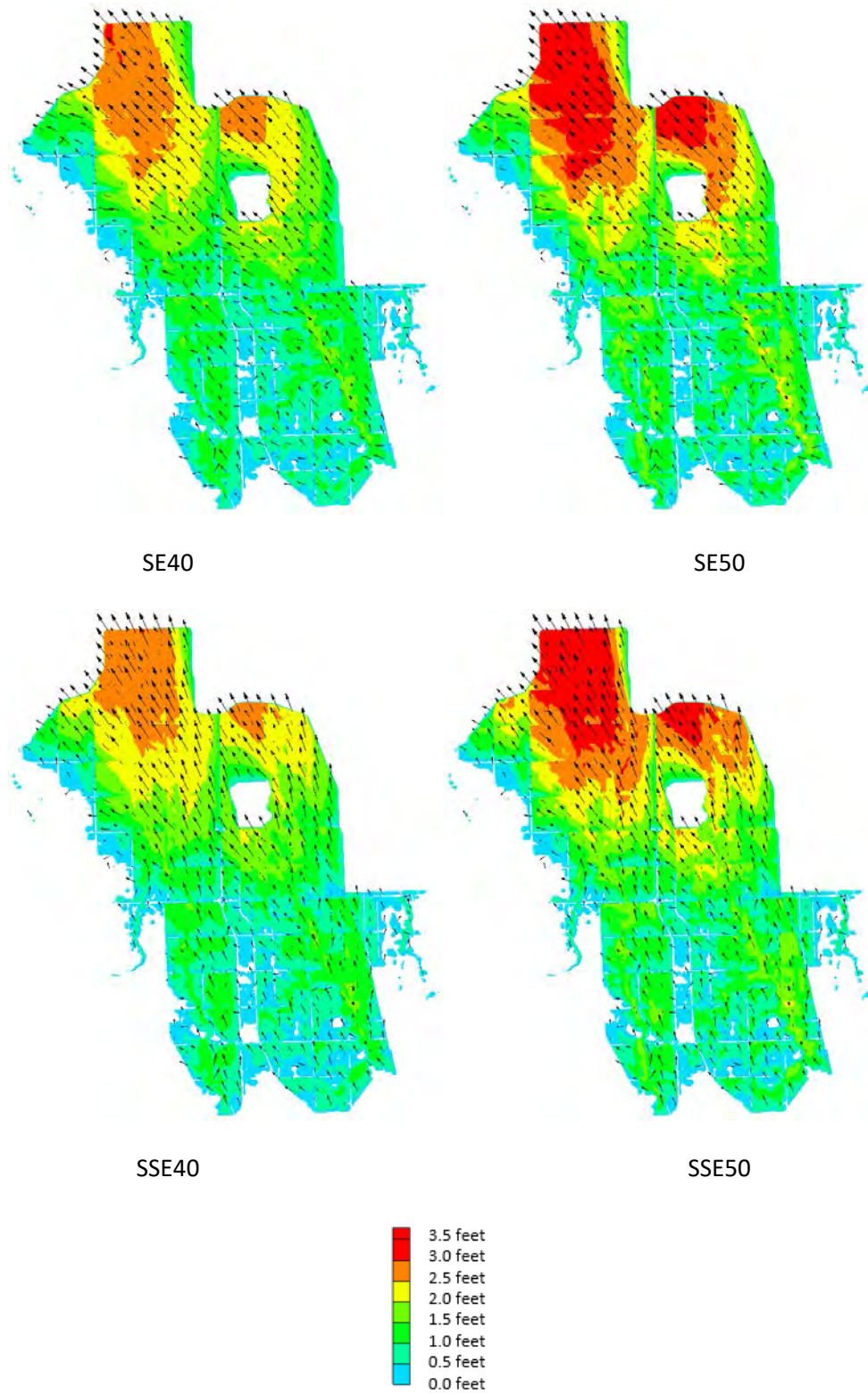
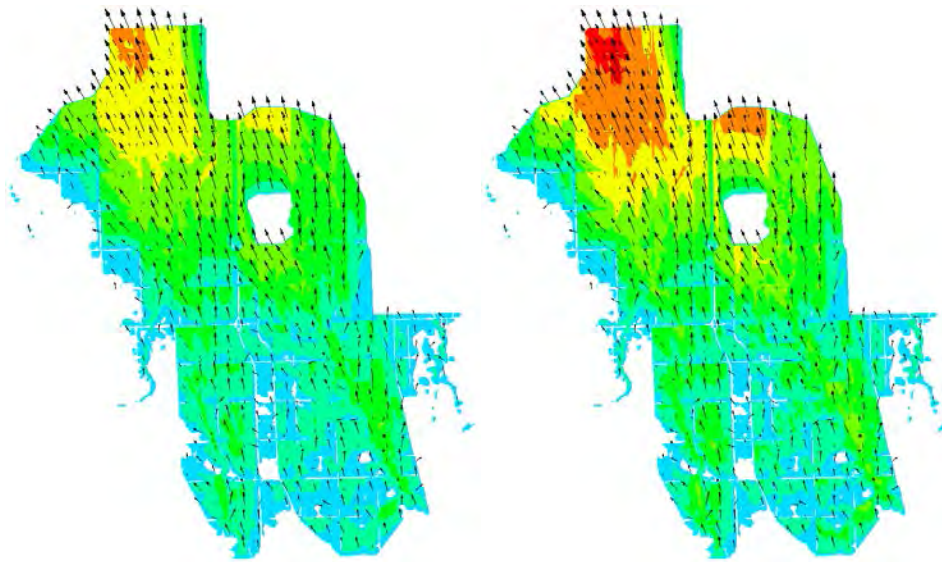
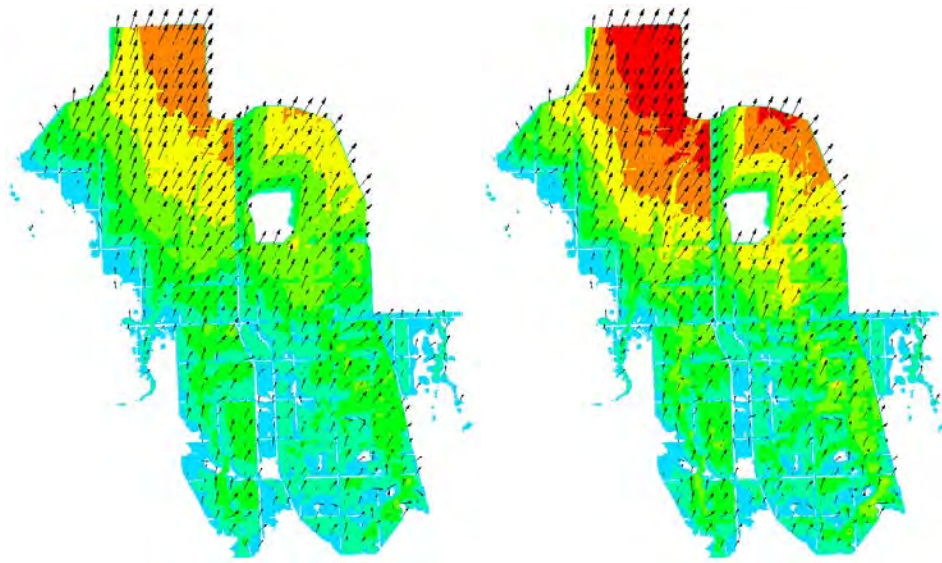


Figure 14: Wave heights in feet for winds from the south and south-southwest



S40

S50



SSW40

SSW50

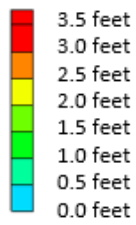


Figure 15: Wave heights in feet for winds from the southwest and west-southwest

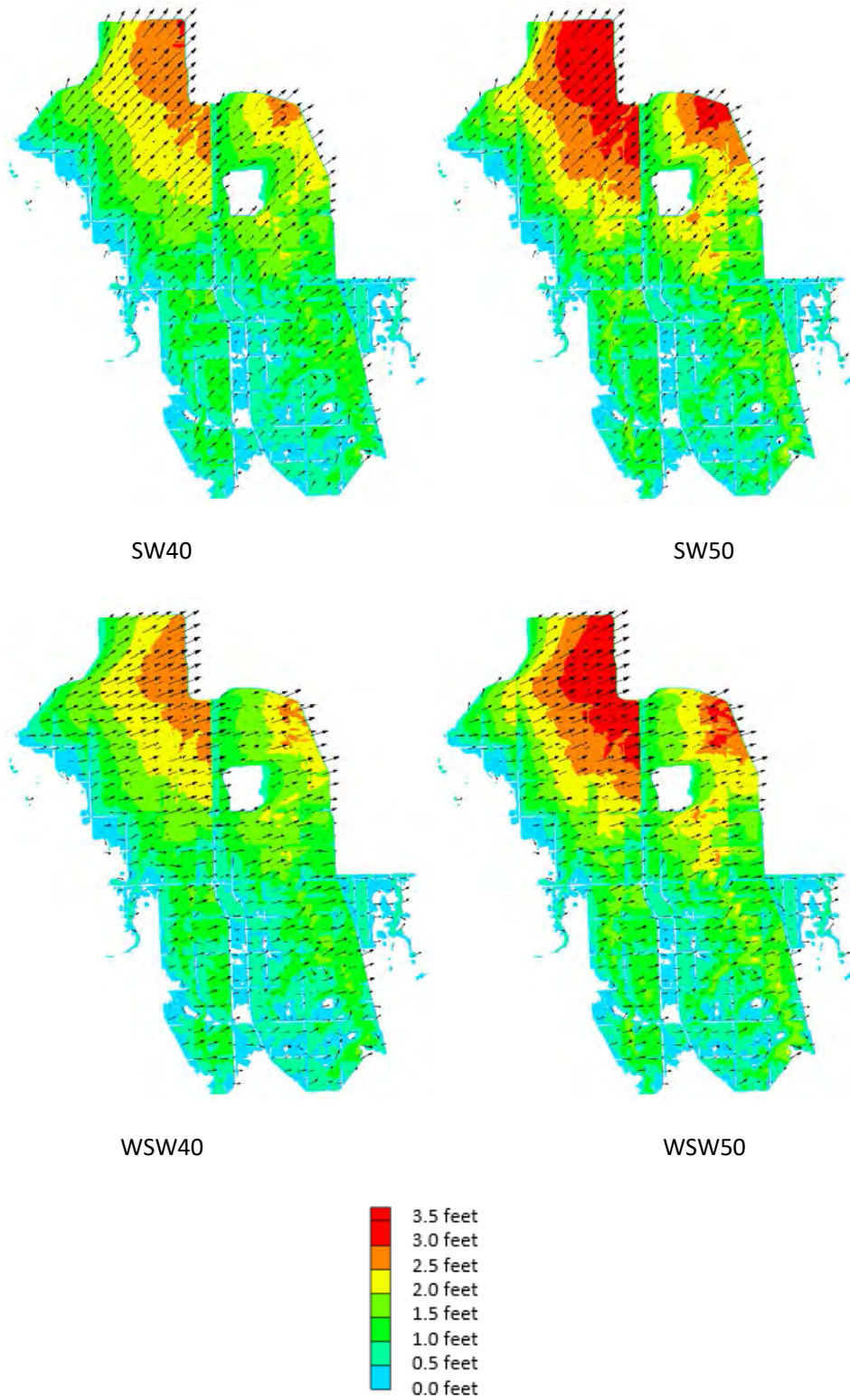


Figure 16: Wave heights in feet for winds from the west and west-northwest

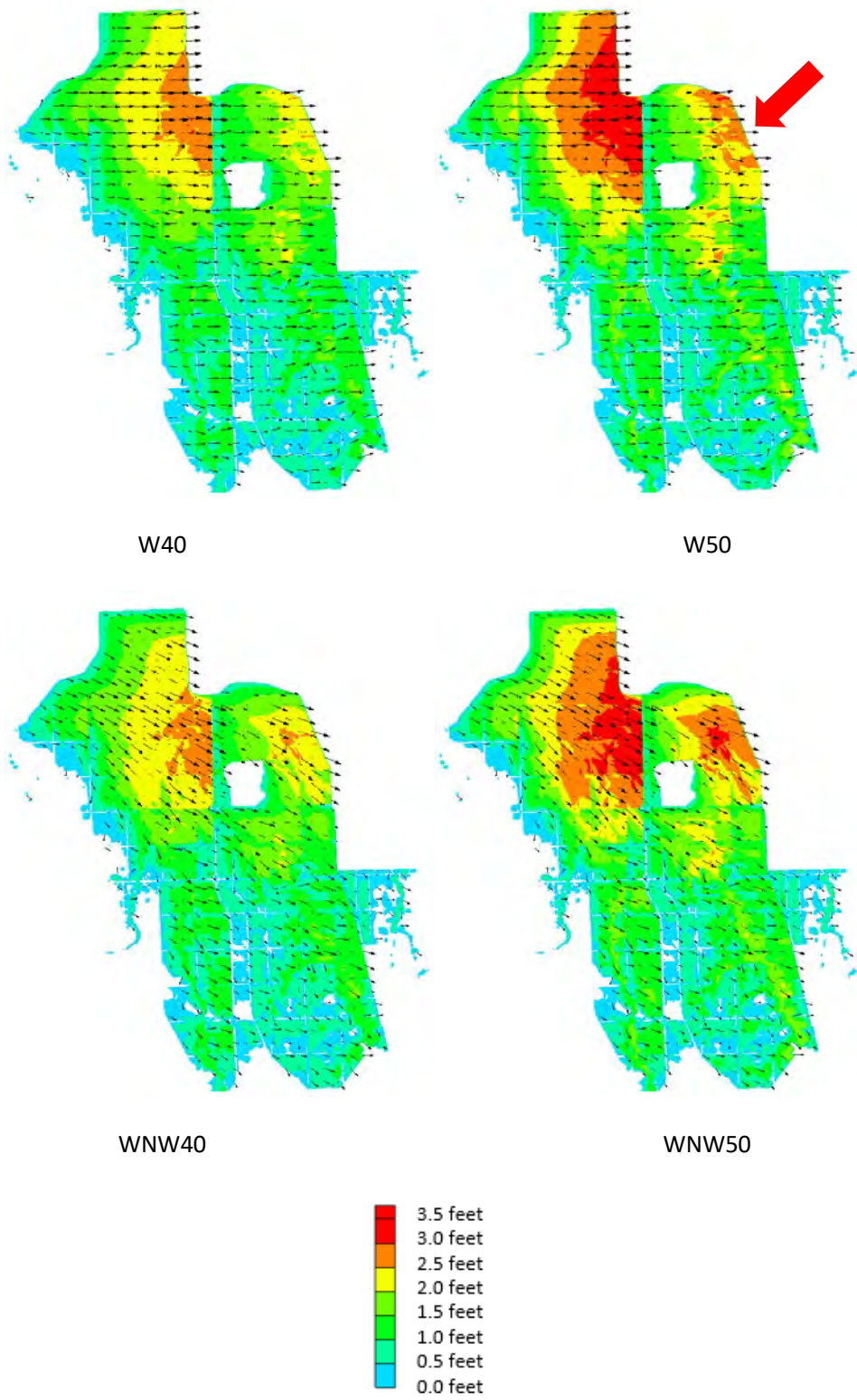
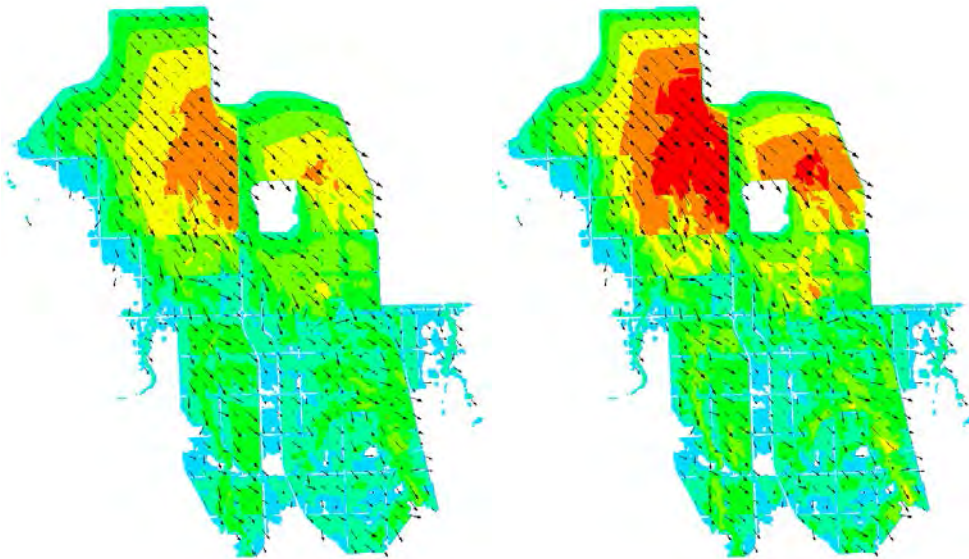


Figure 17: Wave heights in feet for winds from the northwest



NW40

NW50

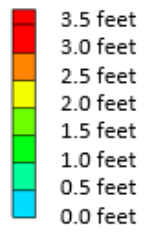
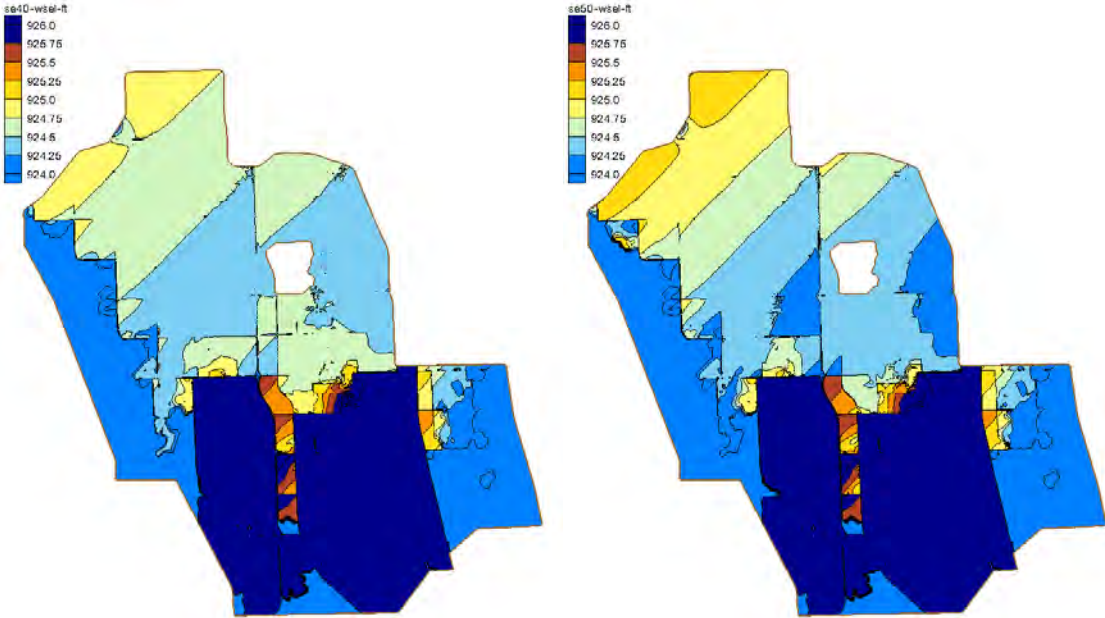
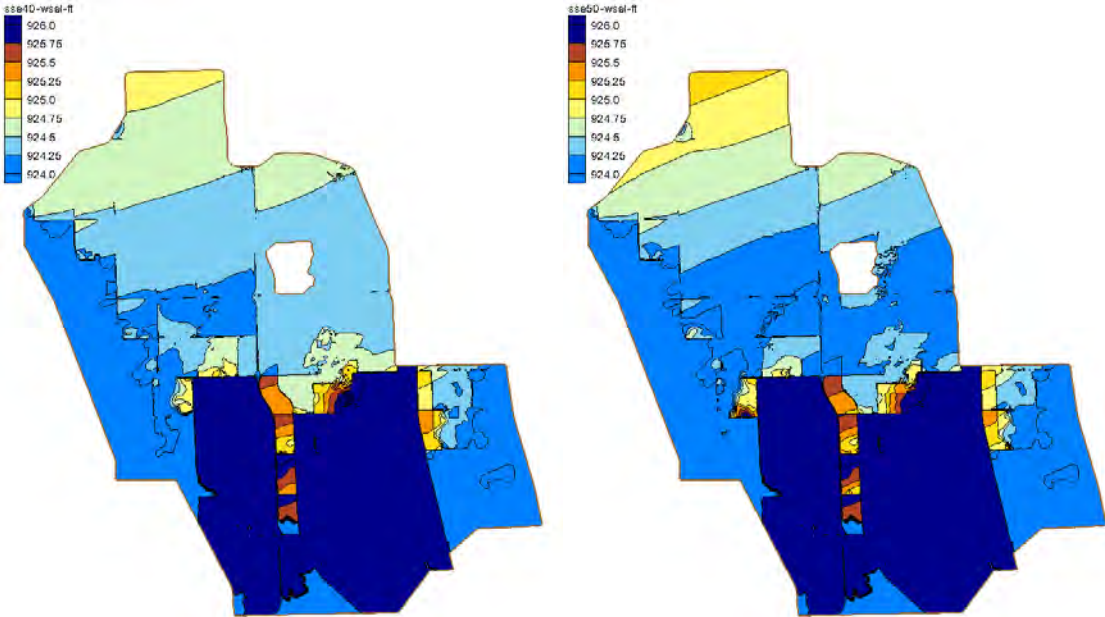


Figure 18: Water surface elevation with wind setup for winds from the southeast and south-southeast



SE40

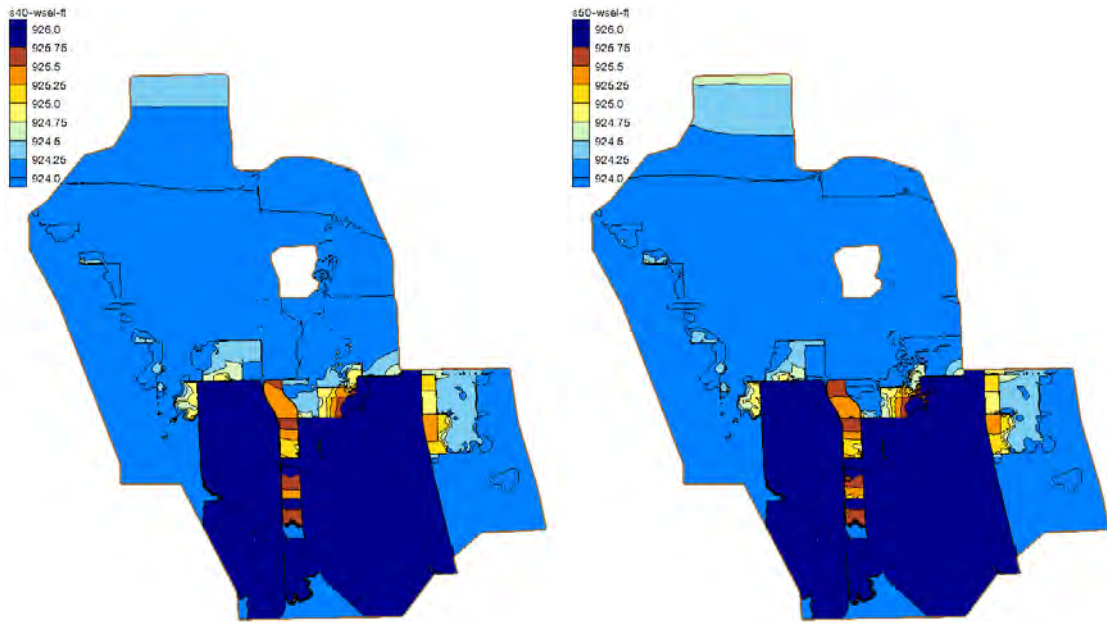
SE50



SSE40

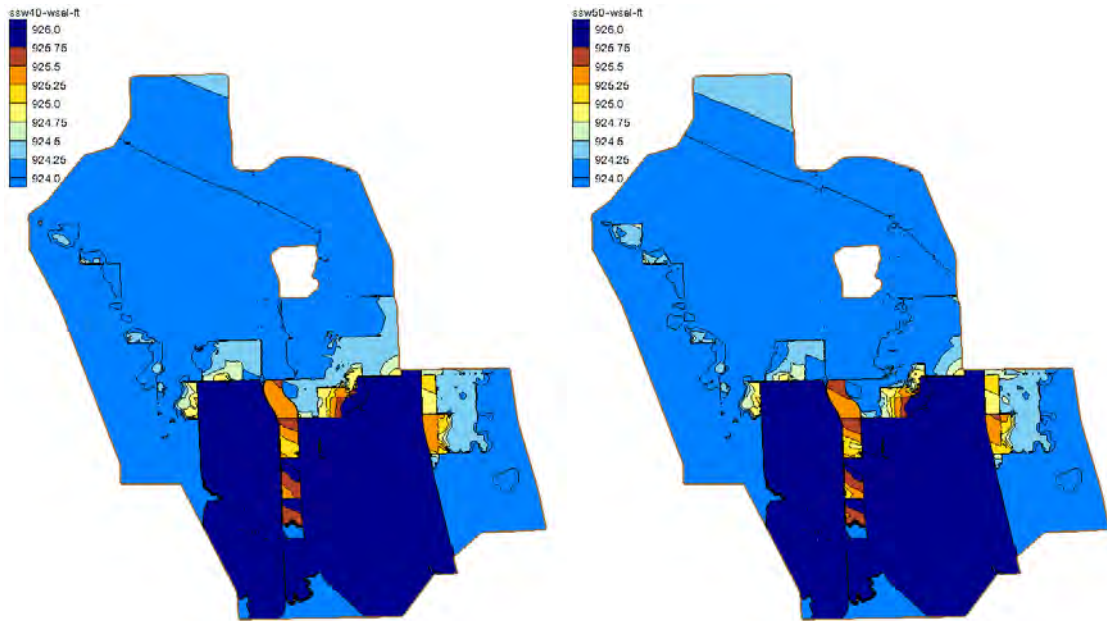
SSE50

Figure 19: Water surface elevation with wind setup for winds from the south and south-southwest



S40

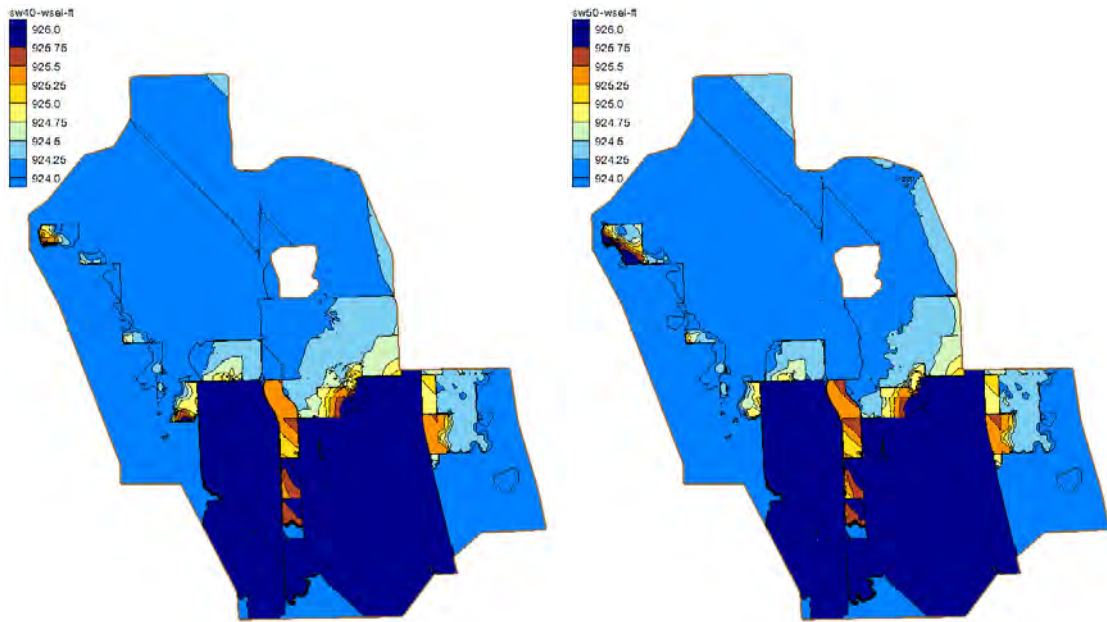
S50



SSW40

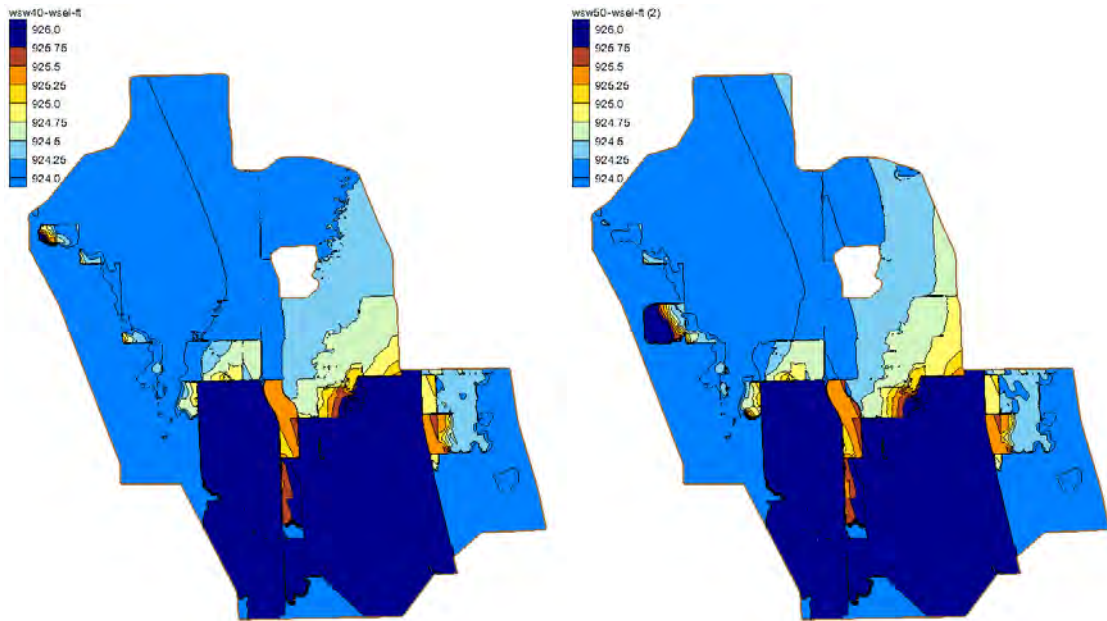
SSW50

Figure 20: Water surface elevation with wind setup for winds from the southwest and west-southwest



SW40

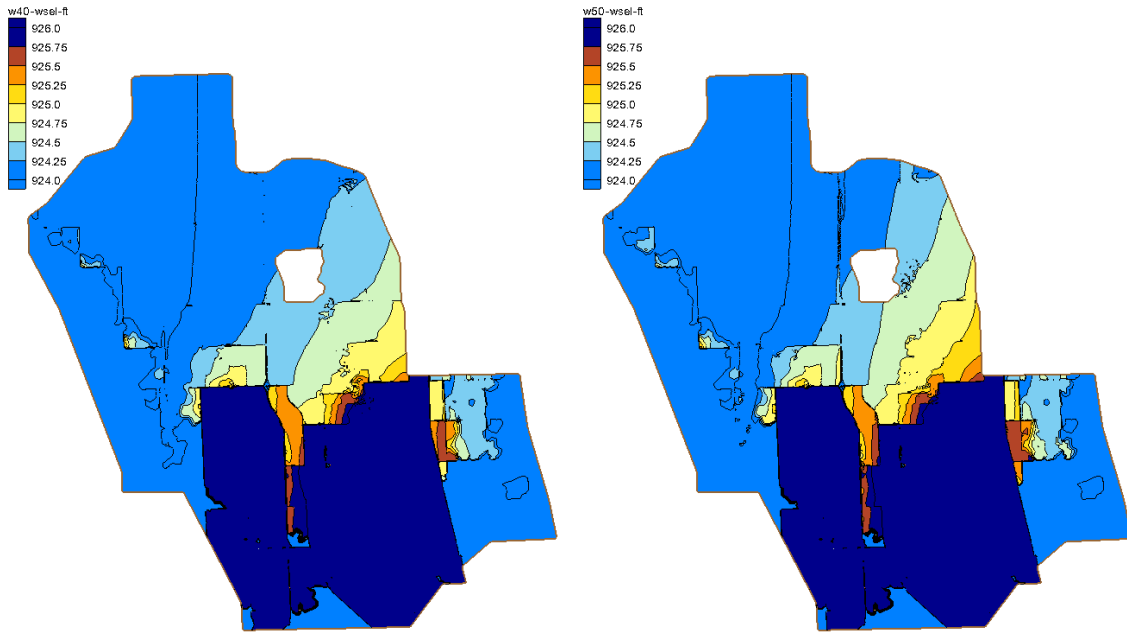
SW50



WSW40

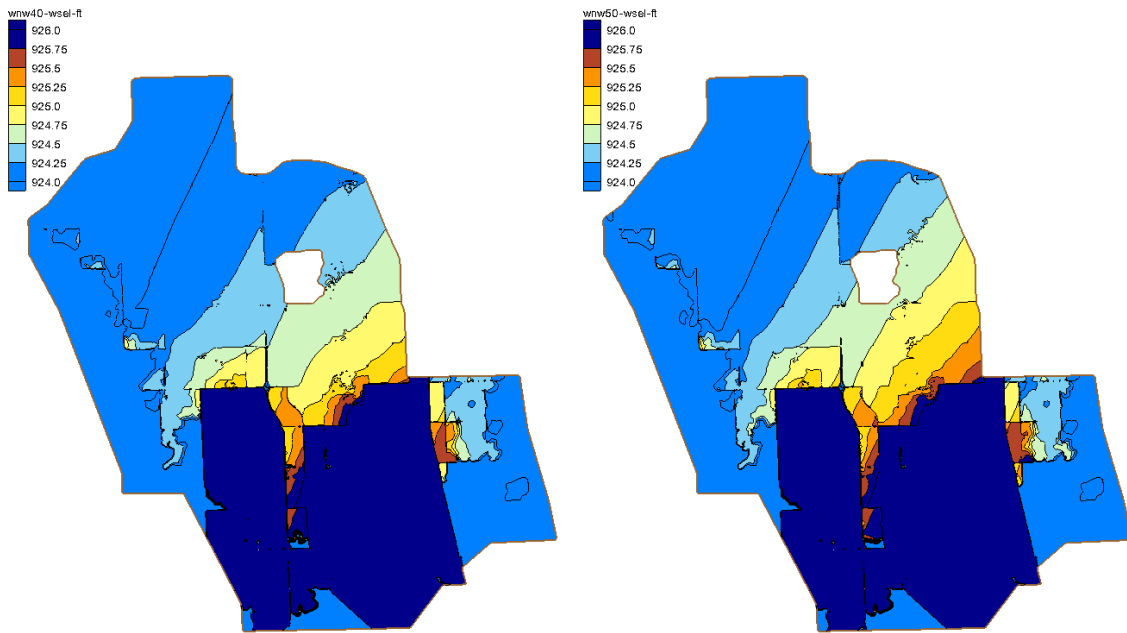
WSW50

Figure 21: Water surface elevation with wind setup for winds from the west and west-northwest



W40

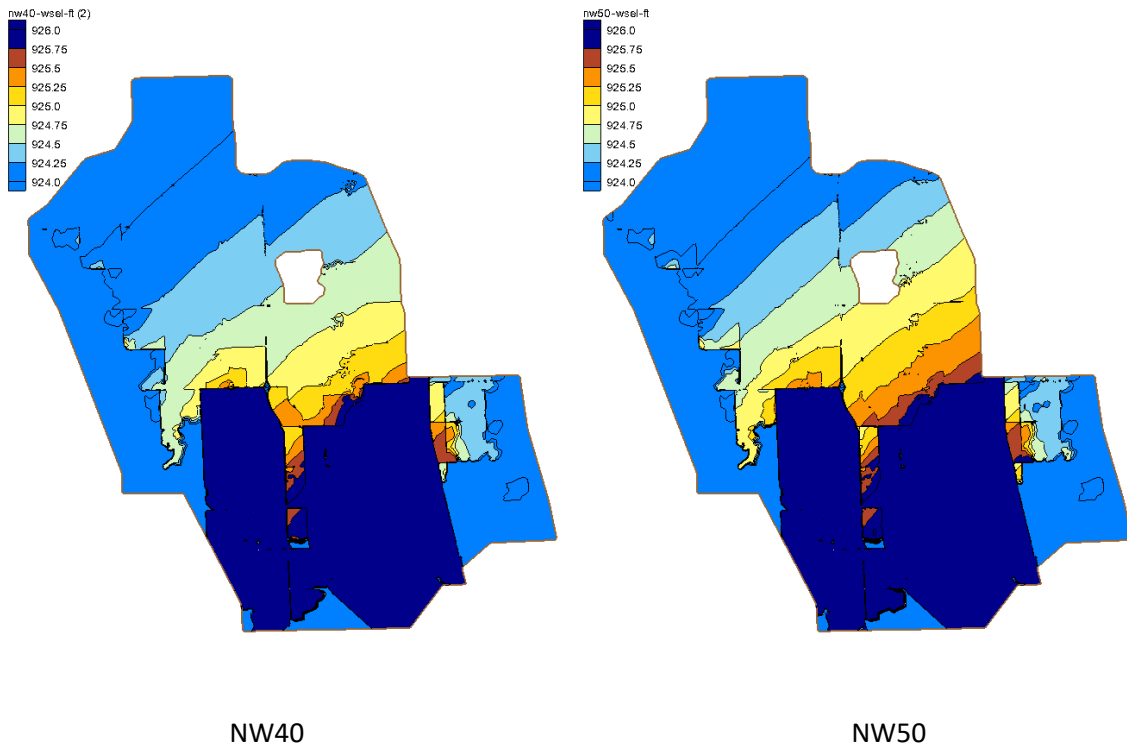
W50



WNW40

WNW50

Figure 22: Water surface elevation with wind setup for winds from the northwest



Attachment 3

Fargo Moorhead Red River Diversion Operation Plan

1. Background

1.1. The Project

- The Project includes a 30 mile long diversion channel that diverts floodwaters around the Fargo-Moorhead Metro Area, which includes: Fargo, North Dakota; Moorhead, Minnesota; and surrounding communities in Cass County, North Dakota and Clay County, Minnesota.
- Since the diversion channel provides additional hydraulic conveyance around the communities, a temporary retention area is needed to mitigate the increases in discharge caused by the diversion channel. The proposed retention area, referred to as the Staging Area, is located at the upstream end of the diversion.
- The area within the diversion that will benefit from the flood mitigation is referred to as the Flood Damage Reduction Area (FDRA).

1.2. Operation Plan Concepts

- The diversion channel diverts flood waters from the Red River of the North (RRN) and five tributaries; Wild Rice River (WRR), Sheyenne River, Maple River, Rush River, and Lower Rush River.
- Without the Staging Area, the diversion channel would increase downstream water surface elevations (as compared to existing no-project conditions).
- The Red River and Wild Rice River are directly regulated by gated control structures that will be constructed on both rivers.
- The Rush and Lower Rush Rivers will intersect and flow directly into the diversion channel.
- Flows from the Sheyenne and Maple Rivers are divided at the Aqueducts where low flows pass over the diversion and into the FDRA through a concrete channel, and flows in excess of this capacity pass over a weir and into the diversion channel. Meanwhile, the diversion flows pass under the tributaries through box culverts.
- Any conveyance that differs from existing conditions will have impacts on the water surface elevation of the Red River, downstream of the project. Since much of the tributary flows pass directly into the diversion channel, the Staging Area must be used to offset the impacts.
- The dynamics of the Staging Area are controlled by the storage volume within the Staging Area, the magnitudes and temporal variations of the upstream inflows and the gate operating rules.
- Gated control structures are located on the Red River (Red River Control Structure - RRCS), on the Wild Rice River (Wild Rice River Control Structure - WRRCS), and at the upstream end of the diversion channel (Diversion Inlet Control Structure - DICS).
- The flood mitigation approach utilized in this study is unique in that a targeted downstream flow condition is determined by the magnitudes and temporal variations of the flows from the six rivers and the gate operating rules. However, regulation can only occur on hydrology delivered by the Red River and Wild Rice River because the other tributary flows cannot be regulated.

1.3. Power law relationships in hydrology

Recent studies have shown that the storage-discharge relationship can be characterized by a general power law function, $Q=aV^b$ (Wittenberg, 1994; Wittenberg and Sivapalan, 1999; Harman and Sivapalan, 2009). The power law relationship application has also been proven through field evidence and physical experiments (Mein et al., 1974, Wittenberg, 1994; Chapman, 1999).

Storage-discharge relationships have been widely used in many hydrology fields:

- Flood estimation (Mein et al., 1974; Georgakakos et al., 1982; Zhang et al., 2000; Rahman and Goonetilleke, 2001; Rezaei-Sadr et al., 2012)
- Rainfall-runoff simulation (Horton, 1938; Dooge, 1973; Nash, 1958; Wang and Gupta, 1981; Hughes and Murrell, 1986; Basha, 2000; Nourani et al., 2009)
- Groundwater flow (Vogel and Kroll, 1992)
- Base flow recession (Cheng, 2008; Aksoy and Wittenberg, 2011; Wang, 2011; Gan and Luo, 2013), and
- Sewer water flow (Gernaey et al., 2011).

1.4. Storage-discharge relationships for optimal flood release

- The power law relationship was introduced in this study to calculate the optimal discharge releases from the Staging Area.
- The calculated releases were derived from the available flow to be sent to the downstream reaches from the Staging Area.
- The coefficients of the power law relationships were determined by testing the equations with numerous synthetic and historic simulations and fitting the operated hydrographs to the existing condition hydrographs at a location downstream of the diversion outlet.
- The power law relationships that were developed for the operations plan utilize the physical storage characteristics within the staging area as inflows arrive from the WRR and RRN. The relationships also take into account the mitigation requirements from the WRR, RRN, and all tributary inflows to the diversion channel.

The normal operation plan is expected to apply 99.8% of the time. However, 0.2% of the time, a National Weather Service (NWS) flood forecast will be required to determine if extreme event operation shall take place. Flowchart A shows the general guidelines of project operation for varying flood magnitudes:

- Normal Operation (99.8% of the time, 500-year flood event and smaller)
- Inflow Design Flood (IDF) Operation (greater than 500-year flood event through IDF)
- Probable Maximum Flood (PMF) Operation (greater than IDF).

The operation rules provide iterative step-by-step procedures to operate the gates after reading various gages for flow and stage from the rising limb through the falling limb of the flood hydrograph. The procedures are well defined and should not require subjective human decision making during operation. However, it is expected that calculations will be made periodically throughout operation to verify adequacy of the automated operation. The operation will differ if a flood prediction exceeds a 500-year event, at which point the project objective transitions from mitigating downstream impacts to protecting

the project infrastructure. Currently, this transition is incorporated into the HEC-RAS rules using an index function based on upstream flows.

For a flood that is less than or equal to a 500-year event, the project operation follows the detailed operation rules as demonstrated in Flowchart B and described in Section 2. For a flood that is 500-year event through the IDF or greater than the IDF, the control structure gates are either partially operated or not operated as defined in Sections 3 and 4 and Flowchart A.

2. Operation Plan for 500-Year Flood Event and Smaller

The real-time gate operation for a flood less than or equal to a 500-year event is generally guided through six Operation Segments followed in sequence as described below and in Flowcharts B and C:

- Operation Segment 1 - No Operation
- Operation Segment 2 - Gate Preparation
- Operation Segment 3 - Hold
- Operation Segment 4 - Rising
- Operation Segment 5 - Transition
- Operation Segment 6 – Falling

Each of the Operation Segments is specially designed based on the flood characteristics to:

- (1) hold and store water, and mitigate the downstream impacts on the rising limb of the hydrograph,
- (2) efficiently release water from the Staging Area to minimize the effects on the Staging Area and downstream water level changes.

Throughout the operation process, the corresponding allowable flow, and gate openings for the WRRCS, RRCS, and DICS are calculated for each operation iteration.

2.1 Operation Segment 1 – No Operation

On the rising limb of the hydrograph, when the combined flow at Enloe, Red River and Abercrombie, Wild Rice River is less than a 10 year flood, the project will not operate.

2.1.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q_{ABER}^t
- 0505152130, Red River at Enloe, Q_{ENLOE}^t

Where t = Number of the project operation iteration since operation began (e.g., 1, 2, 3. etc.)

2.1.2 Sum current flows at Abercrombie and Enloe, Q_{AE}^t

$$Q_{ABER}^t + Q_{ENLOE}^t = Q_{AE}^t$$

2.1.3 Compare the combined flow at Abercrombie and Enloe, Q_{AE}^t with 17,000 cfs.

- If the combined flow at Abercrombie and Enloe, Q_{AE}^t , is less than 17,000 cfs, continue with Operation Procedure 2.1.4.
- If the combined flow at Abercrombie and Enloe, Q_{AE}^t , becomes greater than 17,000 cfs, move to “Operation Segment 2 - Gate Preparation”.

2.1.4 If the combined flow at Abercrombie and Enloe, Q_{AE}^t , is less than 17,000 cfs, WRRCS and RRCS gates are completely open, and DICS gates are closed.

2.1.5 Continue the iteration and repeat the procedures in “Operation Segment 1 - No Operation”.

2.2 Operation Segment 2 – Gate Preparation

Operation Segment 2 is a single-iteration process to set the initial gate openings for WRRCS and RRCS and calculate the initial flow to hold through town.

2.2.1 Record water surface elevations in the Staging Area

- Wild Rice River, $Z_{STAGING_AREA_WRR}^t$
- Red River, $Z_{STAGING_AREA_RR}^t$

The Staging Area is large enough that the water surface elevation may vary from one side to another. Therefore, the water surface elevations are measured at both the Wild Rice River (XS64862, DR37 to DIV) and Red River (XS2531315, Breck to Wolv) and the difference in the elevations is used to account for the proportion of water arriving from the two upstream sources.

2.2.2 Compute the flow through WRRCS and RRCS corresponding to the staging area water surface elevations using Tables 1 and 2

- Record the flow at WRRCS, Q_{WRRCS_HOLD}
 $Q_{WRRCS_HOLD} = \text{Table 1} (Z_{STAGING_AREA_WRR}^t)$ when $Q_{AE}^t = 17,000 \text{ cfs}$
- Record the flow at RRCS, Q_{RRCS_HOLD}
 $Q_{RRCS_HOLD} = \text{Table 2} (Z_{STAGING_AREA_RR}^t)$ when $Q_{AE}^t = 17,000 \text{ cfs}$

2.2.3 Compute the total flow to hold through town, Q_{HOLD}^{t0} , which will be used in “Operation Segment 3 - Hold”

$$Q_{HOLD}^{t0} = Q_{WRRCS_HOLD}^{t0} + Q_{RRCS_HOLD}^{t0} \quad \text{when } Q_{AE}^t = 17,000 \text{ cfs}$$

2.2.4 Prepare the WRRCS and RRCS gates by setting their gate openings using the following equations:

- Gate opening at Wild Rice River Control Structure
 $H_{WRRCS}^t = Z_{STAGING_AREA_WRRCS}^t - 889.55_{GATE_INVERT}$
- Gate opening at Red River Control Structure
 $H_{RRCS}^t = Z_{STAGING_AREA_RRCS}^t - 874.00_{GATE_INVERT}$

2.2.5 Continue to “Operation Segment 3 - Hold”.

2.3 Operation Segment 3 - Hold

In Operation Segment 3, the gate openings for WRRCS and RRCS are controlled to maintain a specified flow through town. At this point, the Staging Area water surface elevation will begin to rise.

2.3.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q_{ABER}^t
- 0505152130, Red River at Enloe, Q_{ENLOE}^t
- xxxxxxxx, Maple River at Durbin, $Q_{MAPLE}^{t-36/\Delta t}$ (flow at gage 36 hours ago)
- 05058980, Sheyenne River at Gol Road, $Q_{SHEY}^{t-36/\Delta t}$ (flow at gage 36 hours ago)
- 05060500, Rush River at Amenia, $Q_{RUSH}^{t-36/\Delta t}$ (flow at gage 36 hours ago)

Where Δt = Number of hours per operation iteration, and $-36/\Delta t$ = the number of iterations looking back to obtain the flow at tributary gages, i.e., flow at 36 hours ago, and $t-36/\Delta t$ = the number of operation iteration since operation began to obtain the flow at tributary gages.

2.3.2 Record water surface elevations in the Staging Area

- Wild Rice River, $Z_{STAGING_AREA_WRR}^t$
- Red River, $Z_{STAGING_AREA_RR}^t$

2.3.3 Sum current flows at Abercrombie and Enloe, Q_{AE}^t

$$Q_{ABER}^t + Q_{ENLOE}^t = Q_{AE}^t$$

2.3.4 Calculate the flow proportion ratios for WRRCS and RRCS, R_{WRRCS}^t and R_{RRCS}^t

Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z_{max}^t . If the Staging Area maximum water surface elevation is lower than 914, the ratio between the WRRCS and RRCS is calculated as the percentage of flow contribution from Wild Rice River and Red River to the Staging Area. If the maximum water surface elevation is higher than 914, the proportion ratios are calculated based on a stage difference-driven approach.

$$Z_{max}^t = \max(Z_{STAGING_AREA_RR}^t, Z_{STAGING_AREA_WRR}^t)$$

$$R_{RRCS}^t = \begin{cases} \frac{Q_{ENLOE}^t / Q_{AE}^t}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}{0.2} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \\ \frac{0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

$$R_{WRRCS}^t = \begin{cases} \frac{Q_{ABER}^t / Q_{AE}^t}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \\ \frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

2.3.5 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q_{AE_MAX}^t = \text{Max}(Q_{AE}^t)$$

2.3.6 Compute flow reduction for tributary rivers

- Sheyenne River

$$Q_{SHEY_REDUC}^t = \begin{cases} 0 & \text{when } Q_{SHEY}^{t-36/\Delta t} \leq 2,000 \text{ cfs} \\ Q_{SHEY}^{t-36/\Delta t} - 2,000 & \text{when } 2,000 \leq Q_{SHEY}^{t-36/\Delta t} \leq 4,600 \text{ cfs} \\ 0.5 \times (Q_{SHEY}^{t-36/\Delta t} - 4,600) + 2,600 & \text{when } Q_{SHEY}^{t-36/\Delta t} > 4,600 \text{ cfs} \end{cases}$$

- Maple River

$$Q_{MAPLE_REDUC}^t = \begin{cases} 0 & \text{when } Q_{MAPLE}^{t-36/\Delta t} \leq 3,000 \text{ cfs} \\ 0.8 \times (Q_{MAPLE}^{t-36/\Delta t} - 3,000) & \text{when } Q_{MAPLE}^{t-36/\Delta t} > 3,000 \text{ cfs} \end{cases}$$

- Rush River

$$Q_{RUSH_REDUC}^t = Q_{RUSH}^{t-36/\Delta t}$$

2.3.7 Compute total flow reduction for tributary rivers

$$Q_{REDUCTION}^t = Q_{SHEY_REDUC}^t + Q_{MAPLE_REDUC}^t + Q_{RUSH_REDUC}^t$$

2.3.8 Record maximum flow reduction up to time t

$$Q_{REDUCTION_MAX}^t = \text{Max}(Q_{REDUCTION}^t)$$

2.3.9 Compute the parameter *a* using Table 3, which is a function of the maximum combined Abercrombie and Enloe flow and the maximum flow reduction.

$$a = \text{Table 3}(Q_{AE_MAX}^t, Q_{REDUCTION_MAX}^t)$$

2.3.10 Compute the volume of water in the Staging Area using Table 4

$$V_{STAGING_AREA}^t = \text{Table 4}(Z_{STAGING_AREA_RR}^t)$$

2.3.11 Compute the designed flow for the rising limb of the hydrograph, Q_{RISING}

$$Q_{RISING}^t = a(V_{STAGING_AREA_RR}^t)^{1.3} - Q_{REDUCTION}^t$$

where the unit for volume of water in the Staging Area, $V_{STAGING_AREA}^t$, is ac-ft and the unit for $Q_{REDUCTION}^t$ and Q_{RISING}^t is cfs.

2.3.12 Calculate Q_{HOLD}^t , $Q_{WRRCS_HOLD}^t$, and $Q_{RRCS_HOLD}^t$. In the first iteration of segment 3, Q_{HOLD}^t , $Q_{WRRCS_HOLD}^t$, and $Q_{RRCS_HOLD}^t$ values shall be used as recorded from “Operation Segment 2 - Gate Preparation”, and the Q_{HOLD}^t in the subsequent operation iterations is then calculated using the following equation. A gradual decreasing Q_{HOLD}^t is applied. A decreasing rate, 2,000cfs/day, was determined based on the historical daily maximum flow decreasing rate at Hickson and Fargo gages in consideration of bank stability.

$$Q_{HOLD}^t = \begin{cases} Q_{HOLD}^{t0} & \text{First iteration} \\ Q_{HOLD}^{t-1} - (2,000/24)\Delta t & \text{Subsequent iterations} \end{cases}$$

where Δt = number of hours per operation iteration.

- Allowable flow to pass the WRRCS (Q_{WRRCS}^t)

$$Q_{WRRCS}^t = R_{WRRCS}^t \times Q_{HOLD}^t$$

- Allowable flow to pass the RRCS (Q_{RRCS}^t)

$$Q_{RRCS}^t = R_{RRCS}^t \times Q_{HOLD}^t$$

2.3.13 Compare Q_{RISING} with Q_{HOLD}^t .

- If Q_{RISING} is less than Q_{HOLD}^t , continue with the procedures in “Operation Segment 3 - Hold”. Use the flow calculated from procedure 2.3.12 as the allowable flow to pass WRRCS and RRCS, with the DICS remains closed. Continue with the Operation Procedure 2.3.14.
- If Q_{RISING} is greater than Q_{HOLD}^t , begin “Operation Segment 4 - Rising”.

2.3.14 Determine the gate opening height for WRRCS and RRCS using the calculated flow from the procedure 2.3.12 and the Staging Area water surface elevations using Tables 5 & 6.

- Gate opening at Wild Rice River Control Structure

$$H_{WRRCS}^t = \text{Table 5}(Q_{WRRCS}^t, Z_{STAGING_AREA_WRR}^t)$$

- Gate opening at Red River Control Structure

$$H_{RRCS}^t = \text{Table 6}(Q_{RRCS}^t, Z_{STAGING_AREA_RR}^t)$$

2.3.15 Continue to the next iteration and repeat procedures for the gate operation in “Operation Segment 3 - Hold”.

2.4 Operation Segment 4 – Rising

In Operation Segment 4, the gates at the WRRCS, RRCS, and DICS are gradually opened to let more water pass through town and into the diversion. A real-time flow through the WRRCS, RRCS, and DICS is calculated using a storage-discharge relationship based on gate opening, and the available flow to release is determined based on the volume of water in the Staging Area, the magnitude of the flood and the tributary river flows that need to be mitigated.

In Operation Segment 4, the priority to send flow through the WRRCS and RRCS versus the DICS is given to the WRRCS and RRCS until the WRRCS and RRCS flow has reached the maximum designed flow through town. The remaining flow is then sent through the DICS. The flow split between WRRCS and RRCS is calculated using the water surface elevations from Wild Rice River and Red River. Gate openings for WRRCS, RRCS, and DICS are then calculated separately based on the corresponding designed flow and water surface elevation in the Staging Area.

2.4.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q'_{ABER}
- 0505152130, Red River at Enloe, Q'_{ENLOE}

2.4.2 Record the water surface elevations in the Staging Area

- Wild Rice River, $Z'_{STAGING_AREA_WRR}$
- Red River, $Z'_{STAGING_AREA_RR}$

2.4.3 Sum current flows at Abercrombie and Enloe, Q'_{AE}

$$Q'_{ABER} + Q'_{ENLOE} = Q'_{AE}$$

2.4.4 Calculate the flow proportion ratios for WRRCS and RRCS, R'_{WRRCS} and R'_{RRCS}

Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z'_{max} . The ratios between WRRCS and RRCS are calculated based on the following equations:

$$Z'_{max} = \max(Z'_{STAGING_AREA_RR}, Z'_{STAGING_AREA_WRR})$$

$$R'_{RRCS} = \begin{cases} \frac{Q'_{ENLOE} / Q'_{AE}}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{0.2}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.4} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \\ \frac{0.2}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & \end{cases}$$

$$R'_{WRRCS} = \begin{cases} \frac{Q'_{ABER} / Q'_{AE}}{0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.4}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \\ \frac{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.2}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & \end{cases}$$

2.4.5 Check Q'_{AE} hydrograph

- If Q'_{AE} curve has crested, continue with procedure 2.4.6.

- If Q'_{AE} curve has not crested, skip procedures 2.4.6-2.4.13 and continue with procedure 2.4.14.

2.4.6 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q'_{AE_MAX} = \text{Max} (Q'_{AE})$$

2.4.7 Compute designed flow through town using Table 8

$$Q'_{TOWN} = \text{Table 8}(Q'_{AE_MAX})$$

2.4.8 Compare Q'_{AE_MAX} with 22,000 cfs, which is the upper limit of what is considered a small flood.

- If Q'_{AE_MAX} value is greater than 22,000 cfs, skip procedures 2.4.9-2.4.13 and continue with procedure 2.4.14.
- If Q'_{AE_MAX} value is less than 22,000 cfs (i.e., small flood), continue with the procedure 2.4.9.

2.4.9 Check if this flood is a 1997-type long duration flood. 1997 flood had a relative small peak (i.e., $Q'_{AE_MAX} = 20,000$ cfs) and a long duration (17 days with the flow greater than 17,000 cfs). This type of floods should be operated normally due to high flow volume.

- If Z'_{MAX} (obtained from procedure 2.4.4) is greater than 915.5 feet, this is a 1997-type long duration flood. Skip procedures 2.4.9-2.4.13 and continue with procedure 2.4.14.
- If Z'_{MAX} is less than 915.5 feet, (i.e., small flood), continue with the procedure 2.4.10 for small flood operation.

2.4.10 Compute the Q'_{SF} for small flood event

$$Q'_{SF} = \begin{cases} 1.01^{\Delta t} \times Q'^{-1}_{HOLD} & \text{First iteration} \\ 1.01^{\Delta t} \times Q'^{-1}_{SF} & \text{Subsequent iterations} \\ Q'_{TOWN} & \text{when } Q'^{-1}_{SF} \geq Q'_{TOWN} \end{cases}$$

where Δt = number of hours per operation iteration.

2.4.11 Compute the allowable flow to pass through WRRCS and RRCS, Q'_{WRRCS} and Q'_{RRCS} , respectively, for small flood events. The DICS remains closed.

- Allowable flow to pass the WRRCS (Q'_{WRRCS})

$$Q'_{WRRCS} = R'_{WRRCS} \times Q'_{SF}$$

- Allowable flow to pass the RRCS (Q'_{RRCS})

$$Q'_{RRCS} = R'_{RRCS} \times Q'_{SF}$$

2.4.12 Determine the gate openings for WRRCS and RRCS using the computed allowable flow and the Staging Area water surface elevations using Tables 5 & 6.

- Gate opening at Wild Rice River Control Structure

$$H'_{WRRCS} = \text{Table 5}(Q'_{WRRCS}, Z'_{STAGING_AREA_WRR})$$

- Gate opening at Red River Control Structure

$$H'_{RRCS} = \text{Table 6}(Q'_{RRCS}, Z'_{STAGING_AREA_RR})$$

2.4.13 Continue and repeat the procedures 2.4.1-2.4.13 for small flood events until the computed gate opening + gate invert > Staging Area WSE.

2.4.14 Determine if the water surface elevation in the Staging Area has crested, which would trigger the operation transitioning to the “Operation Segment 5 - Transition”. Compare the current water surface elevation in the Staging Area, $Z'_{STAGING_AREA_RR}$, with the water surface elevation from previous operation, $Z'^{-1}_{STAGING_AREA_RR}$.

- If the $0.9999 \times Z'^{-1}_{STAGING_AREA_RR}$ is greater than $Z'^{-1}_{STAGING_AREA_RR}$ (i.e., not crested), continue with the procedures in “Operation Segment 4 - Rising”. Continue with the Operation Procedure 2.4.15.

- If the $0.9999 \times Z^t_{STAGING_AREA_RR}$ is less than $Z^{t-1}_{STAGING_AREA_RR}$ (i.e., crested), begin “Operation Segment 5 - Transition”.

2.4.15 Record flows from USGS gages:

- xxxxxxxx, Maple River at Durbin, $Q^{t-36/\Delta t}_{MAPLE}$
- 05058980, Sheyenne River at Gol Road, $Q^{t-36/\Delta t}_{SHEY}$
- 05060500, Rush River at Amenia, $Q^{t-36/\Delta t}_{RUSH}$

Record flows from the Operation Procedure 2.4.1

- 05053000, Wild Rice River at Abercrombie, Q^t_{ABER}
- 0505152130, Red River at Enloe, Q^t_{ENLOE}

2.4.16 Obtain the combined flows at Abercrombie and Enloe, Q^t_{AE} from procedure 2.4.3

2.4.17 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q^t_{AE_MAX} = \text{Max}(Q^t_{AE})$$

2.4.18 Compute flow reduction for tributary rivers

- Sheyenne River

$$Q^t_{SHEY_REDUC} = \begin{cases} 0 & \text{when } Q^{t-36/\Delta t}_{SHEY} \leq 2,000 \text{ cfs} \\ Q^{t-36/\Delta t}_{SHEY} - 2,000 & \text{when } 2,000 \leq Q^{t-36/\Delta t}_{SHEY} \leq 4,600 \text{ cfs} \\ 0.5 \times (Q^{t-36/\Delta t}_{SHEY} - 4,600) + 2,600 & \text{when } Q^{t-36/\Delta t}_{SHEY} > 4,600 \text{ cfs} \end{cases}$$

- Maple River

$$Q^t_{MAPLE_REDUC} = \begin{cases} 0 & \text{when } Q^{t-36/\Delta t}_{MAPLE} \leq 3,000 \text{ cfs} \\ 0.8 \times (Q^{t-36/\Delta t}_{MAPLE} - 3,000) & \text{when } Q^{t-36/\Delta t}_{MAPLE} > 3,000 \text{ cfs} \end{cases}$$

- Rush River

$$Q^t_{RUSH_REDUC} = Q^{t-36/\Delta t}_{RUSH}$$

2.4.19 Compute total flow reduction

$$Q^t_{REDUCTION} = Q^t_{SHEY_REDUC} + Q^t_{MAPLE_REDUC} + Q^t_{RUSH_REDUC}$$

2.4.20 Record maximum flow reduction up to time t

$$Q^t_{REDUCTION_MAX} = \text{Max}(Q^t_{REDUCTION})$$

2.4.21 Compute the parameter a using Table 3

$$a = \text{Table 3}(Q^t_{AE_MAX}, Q^t_{REDUCTION_MAX})$$

2.4.22 Compute the volume of water in the Staging Area using Table 4

$$V^t_{STAGING_AREA} = \text{Table 4}(Z^t_{STAGING_AREA_RR})$$

2.4.23 Compute Q^t_{RISING}

$$Q^t_{RISING} = a(V^t_{STAGING_AREA_RR})^{1.3} - Q^t_{REDUCTION}$$

2.4.24 Compute the designed flow through town using Table 8

$$Q^t_{TOWN} = \text{Table 8}(Q^t_{AE_MAX})$$

2.4.25 Compare Q^t_{TOWN} with Q^t_{RISING} .

- If Q^t_{TOWN} is greater than Q^t_{RISING} , use Q^t_{RISING} as the allowable flow. Continue with Operation Procedure 2.4.26.
- If Q^t_{RISING} is greater than Q^t_{TOWN} , use Q^t_{TOWN} as the allowable flow. Continue with Operation Procedure 2.4.27.

2.4.26 If Q^t_{TOWN} is greater than Q^t_{RISING} , compute the allowable flow to pass through WRRCS and RRCS, Q^t_{WRRCS} and Q^t_{RRCS} . The DICS remains closed. Then skip to Operation Procedure 2.4.29.

- Allowable flow to pass the WRRCS (Q^t_{WRRCS})

$$Q^t_{WRRCS} = R^t_{WRRCS} \times Q^t_{RISING} \quad \text{when } Q^t_{RISING} \leq Q^t_{TOWN}$$
- Allowable flow to pass the RRCS (Q^t_{RRCS})

$$Q^t_{RRCS} = R^t_{RRCS} \times Q^t_{RISING} \quad \text{when } Q^t_{RISING} \leq Q^t_{TOWN}$$
- Allowable flow to pass the DICS (Q^t_{DICS})

$$Q^t_{DICS} = 0 \quad \text{when } Q^t_{RISING} \leq Q^t_{TOWN}$$

2.4.27 Record the time as the calculated Q^t_{RISING} exceeds Q^t_{TOWN} , T^0_{TOWN} .

2.4.28 If Q^t_{RISING} is greater than Q^t_{TOWN} , compute the allowable flow to pass through WRRCS, RRCS, and DICS, Q^t_{WRRCS} , Q^t_{RRCS} , and Q^t_{DICS} , respectively.

- Allowable flow to pass the WRRCS (Q^t_{WRRCS})

$$Q^t_{WRRCS} = R^t_{WRRCS} \times Q^t_{TOWN} \quad \text{when } Q^t_{RISING} > Q^t_{TOWN}$$
- Allowable flow to pass the RRCS (Q^t_{RRCS})

$$Q^t_{RRCS} = R^t_{RRCS} \times Q^t_{TOWN} \quad \text{when } Q^t_{RISING} > Q^t_{TOWN}$$
- Allowable flow to pass the DICS (Q^t_{DICS})

$$Q^t_{DICS} = \begin{cases} Q^t_{RISING} - Q^t_{TOWN} & \text{when } Q^t_{RISING} < Q^t_{TOWN} + Q^t_{DI_MAX} \\ Q^t_{DI_MAX} & \text{when } Q^t_{RISING} - Q^t_{TOWN} \geq Q^t_{DI_MAX} \end{cases}$$

Note: $Q^t_{DI_MAX} = 20,000$ cfs for flood events less than 200-year i.e., $Q^t_{AE_MAX} = 48,000$ cfs, and $Q^t_{DI_MAX} = 25,000$ cfs for the flood equal to the 500-year event i.e., $Q^t_{AE_MAX} = 60,000$ cfs. For any flood events that are larger than the 200-year and less than the 500-year event, $Q^t_{DI_MAX}$ can be calculated using linear interpolation method.

2.4.29 Determine the gate openings for WRRCS, RRCS, and DICS using the computed allowable flow and the Staging Area water surface elevation using Tables 5-7.

- Gate opening at Wild Rice River Control Structure

$$H^t_{WRRCS} = \text{Table 5}(Q^t_{WRRCS}, Z^t_{STAGING_AREA_WRR})$$
- Gate opening at Red River Control Structure

$$H^t_{RRCS} = \text{Table 6}(Q^t_{RRCS}, Z^t_{STAGING_AREA_RR})$$
- Gate opening at Diversion Inlet Control Structure

$$H^t_{DICS} = \text{Table 7}(Q^t_{DICS}, Z^t_{STAGING_AREA_WRR})$$

2.4.30 Continue to the next operation iteration (i.e., Operation Procedure 2.4.1) and repeat procedures for the gate operation in “Operation Segment 4 - Rising” until the operation procedure transitions to the “Operation Segment 5 – Transition” (i.e., 2.4.14).

2.5 Operation Segment 5 – Transition

Operation Segment 5 serves as a transition period between Rising and Falling Segments to calculate the available flow to release through WRRCS, RRCS, and DICS.

2.5.1 Record the water surface elevations in the Staging Area:

- Wild Rice River, $Z^t_{STAGING_AREA_WRR}$

- Red River, $Z'_{STAGING_AREA_RR}$

2.5.2 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q'_{ABER}
- 0505152130, Red River at Enloe, Q'_{ENLOE}
- xxxxxxxx, Maple River at Durbin, $Q'^{t-36/\Delta t}_{MAPLE}$
- 05058980, Sheyenne River at Gol Road, $Q'^{t-36/\Delta t}_{SHEY}$
- 05060500, Rush River at Amenia, $Q'^{t-36/\Delta t}_{RUSH}$

2.5.3 Sum current flows at Abercrombie and Enloe, Q'_{AE}

$$Q'_{ABER} + Q'_{ENLOE} = Q'_{AE}$$

2.5.4 Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z'_{max} . Calculate the flow proportion ratios for WRRCS and RRCS, R'_{WRRCS} and R'_{RRCS}

$$Z'_{max} = \max(Z'_{STAGING_AREA_RR}, Z'_{STAGING_AREA_WRR})$$

$$R'_{RRCS} = \begin{cases} \frac{Q'_{ENLOE} / Q'_{AE}}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{0.2}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \end{cases}$$

$$R'_{WRRCS} = \begin{cases} \frac{Q'_{ABER} / Q'_{AE}}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.4} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{0.2}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \end{cases}$$

2.5.5 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q'_{AE_MAX} = \text{Max}(Q'_{AE})$$

2.5.6 Compute flow reduction for tributary rivers

- Sheyenne River

$$Q'_{SHEY_REDUC} = \begin{cases} 0 & \text{when } Q'^{t-36/\Delta t}_{SHEY} \leq 2,000 \text{ cfs} \\ Q'^{t-36/\Delta t}_{SHEY} - 2,000 & \text{when } 2,000 \leq Q'^{t-36/\Delta t}_{SHEY} \leq 4,600 \text{ cfs} \\ 0.5 \times (Q'^{t-36/\Delta t}_{SHEY} - 4,600) + 2,600 & \text{when } Q'^{t-36/\Delta t}_{SHEY} > 4,600 \text{ cfs} \end{cases}$$

- Maple River

$$Q'_{MAPLE_REDUC} = \begin{cases} 0 & \text{when } Q'^{t-36/\Delta t}_{MAPLE} \leq 3,000 \text{ cfs} \\ 0.8 \times (Q'^{t-36/\Delta t}_{MAPLE} - 3,000) & \text{when } Q'^{t-36/\Delta t}_{MAPLE} > 3,000 \text{ cfs} \end{cases}$$

- Rush River

$$Q'_{RUSH_REDUC} = Q'^{t-36/\Delta t}_{RUSH}$$

2.5.7 Compute total flow reduction

$$Q'_{REDUCTION} = Q'_{SHEY_REDUC} + Q'_{MAPLE_REDUC} + Q'_{RUSH_REDUC}$$

2.5.8 Record maximum flow reduction up to time t

$$Q'_{REDUCTION_MAX} = \text{Max}(Q'_{REDUCTION})$$

2.5.9 Compute parameter b using Table 9. Similar to parameter a , b is a function of the maximum combined Abercrombie and Enloe flow and the maximum flow reduction.

$$b = \text{Table 9}(Q'_{AE_MAX}, Q'_{REDUCTION_MAX})$$

2.5.10 Compute the volume of water in the Staging Area based on the water surface elevation in the Staging Area using Table 4

$$V'_{STAGING_AREA} = \text{Table 4}(Z'_{STAGING_AREA_RR})$$

2.5.11 Compute $Q_{TRANSITION}$. It should be noted that in the first iteration of $Q_{TRANSITION}$ computation (Operation Segment 5 - Transition), Q_{RISING} is used.

$$Q'_{TRANSITION} = \begin{cases} b^{\Delta} \times Q'^{t-1}_{RISING} - Q'_{REDUCTION} & \text{First iteration} \\ b^{\Delta} \times Q'^{t-1}_{TRANSITION} - Q'_{REDUCTION} & \text{Subsequent iterations} \end{cases}$$

2.5.12 Compute parameter c based on the maximum combined Abercrombie and Enloe flow using Table 10

$$c = \text{Table 10}(Q'_{AE_MAX})$$

2.5.13 Compute $Q_{FALLING}$ using parameter c , the volume of water in the Staging Area, and tributary flow reduction.

$$Q'_{FALLING} = c \times (V'_{STAGING_AREA} + 344,000)^{1.35} - Q'_{REDUCTION}$$

2.5.14 Compare $Q_{TRANSITION}$ and $Q_{FALLING}$.

- If the $Q_{FALLING}$ is greater than $Q_{TRANSITION}$, continue with procedure 2.5.16.
- If $Q_{TRANSITION}$ is greater than $Q_{FALLING}$, conduct procedure 2.5.15 and then begin “Operation Segment 6 - Falling”.

2.5.15 Record the Q^{t-1}_{DICS} from the previous operation iteration, which will be used in the allowable flow calculation for DICS in “Operation Segment 6 – Falling”.

2.5.16 Compute the designed flow through town using Table 8

$$Q'_{TOWN} = \text{Table 8}(Q'_{AE_MAX})$$

2.5.17 Compute the designed time duration to allow Q'_{TOWN} , DT'_{QTOWN} , based on the maximum combined Abercrombie and Enloe flow using Table 11

$$DT'_{QTOWN} = \text{Table 11}(Q'_{AE_MAX})$$

2.5.18 Compute the actual duration, $\Delta T'_{QTOWN}$, that the allowable flow, Q'_{TOWN} , has been maintained. T^0_{QTOWN} is obtained from Operation Procedure 2.4.27.

$$\Delta T'_{QTOWN} = T'_{CURRENT} - T^0_{QTOWN}$$

2.5.19 Compare the $\Delta T'_{QTOWN}$ with DT'_{QTOWN} .

- If DT'_{QTOWN} is greater than $\Delta T'_{QTOWN}$, Q'_{TOWN} is used to compute the allowable flow to pass WRRCS and RRCS. Continue with the Operation Procedure 2.5.20.
- If $\Delta T'_{QTOWN}$ is greater than DT'_{QTOWN} , compute the designed receding flow to pass through town. Continue with the Operation Procedure 2.5.21.

2.5.20 If DT'_{QTOWN} is greater than $\Delta T'_{QTOWN}$, Q'_{TOWN} is used to compute the allowable flow to pass WRRCS and RRCS, Q'_{WRRCS} and Q'_{RRCS} and compute the DICS flow. Then skip to Operation Procedure 2.5.23.

- Allowable flow to pass the WRRCS (Q^t_{WRRCS})

$$Q^t_{WRRCS} = R^t_{WRRCS} \times Q^t_{TOWN}$$

- Allowable flow to pass the RRCS (Q^t_{RRCS})

$$Q^t_{RRCS} = R^t_{RRCS} \times Q^t_{TOWN}$$

- Allowable flow to pass the DICS (Q^t_{DICS})

$$Q^t_{DICS} = \begin{cases} Q^t_{TRANSITION} - Q^t_{TOWN} & \text{when } Q^t_{TRANSITION} < Q^t_{TOWN} + Q^t_{DI_MAX} \\ Q^t_{DI_MAX} & \text{when } Q^t_{TRANSITION} - Q^t_{TOWN} \geq Q^t_{DI_MAX} \end{cases}$$

2.5.21 If $\Delta T^t_{Q_{TOWN}}$ is greater than $DT^t_{Q_{TOWN}}$, compute the designed receding flow to pass through town,

$$Q^t_{TOWN-RECEDING}$$

$$Q^t_{TOWN-RECEDING} = \begin{cases} 0.994^{\Delta t} \times Q^{t-1}_{TOWN} & \text{First iteration} \\ \begin{cases} 0.994^{\Delta t} \times Q^{t-1}_{TOWN-RECEDING} & Q^{t-1}_{TOWN-RECEDING} > 8,000 \text{ cfs} \\ 8,000 \text{ cfs} & Q^{t-1}_{TOWN-RECEDING} < 8,000 \text{ cfs} \end{cases} & \text{Subsequent iterations} \end{cases}$$

where Δt = number of hours per operation iteration.

2.5.22 Use the computed $Q^t_{TOWN-RECEDING}$ as the allowable flow to pass through the towns

- Allowable flow to pass the WRRCS (Q^t_{WRRCS})

$$Q^t_{WRRCS} = R^t_{WRRCS} \times Q^t_{Q_{TOWN-RECEDING}}$$

- Allowable flow to pass the RRCS (Q^t_{RRCS})

$$Q^t_{RRCS} = R^t_{RRCS} \times Q^t_{Q_{TOWN-RECEDING}}$$

- Allowable flow to pass the DICS (Q^t_{DICS})

$$Q^t_{DICS} = \begin{cases} Q^t_{TRANSITION} - Q^t_{Q_{TOWN-RECEDING}} & \text{when } Q^t_{TRANSITION} < Q^t_{TOWN} + Q^t_{DI_MAX} \\ Q^t_{DI_MAX} & \text{when } Q^t_{TRANSITION} - Q^t_{TOWN} \geq Q^t_{DI_MAX} \end{cases}$$

2.5.23 Determine the gate openings for WRRCS, RRCS, and DICS using the computed allowable flow and the Staging Area water surface elevation using Tables 5-7.

- Gate opening at Wild Rice River Control Structure

$$H^t_{WRRCS} = \text{Table 5}(Q^t_{WRRCS}, Z^t_{STAGING_AREA_WRR})$$

- Gate opening at Red River Control Structure

$$H^t_{RRCS} = \text{Table 6}(Q^t_{RRCS}, Z^t_{STAGING_AREA_RR})$$

- Gate opening at Diversion Inlet Control Structure

$$H^t_{DICS} = \text{Table 7}(Q^t_{DICS}, Z^t_{STAGING_AREA_WRR})$$

2.5.24 Continue to the next operation iteration and repeat procedures for the gate operation in “Operation Segment 5 - Transition”.

2.6 Operation Segment 6 – Falling

Operation Segment 6 is designed to efficiently release water from the Staging Area as fast as possible without adversely affecting adjacent land. The drawdown has been designed to match historical receding limb hydrographs.

2.6.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q'_{ABER}
- 0505152130, Red River at Enloe, Q'_{ENLOE}

2.6.2 Record the water surface elevations in the Staging Area:

- Wild Rice River, $Z'_{STAGING_AREA_WRR}$
- Red River, $Z'_{STAGING_AREA_RR}$

2.6.3 Sum current flows at Abercrombie and Enloe, Q'_{AE}

$$Q'_{ABER} + Q'_{ENLOE} = Q'_{AE}$$

2.6.4 Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z'_{max} . Calculate the flow proportion ratios for WRRCS and RRCS, R'_{WRRCS} and R'_{RRCS}

$$Z'_{max} = \max(Z'_{STAGING_AREA_RR}, Z'_{STAGING_AREA_WRR})$$

$$R'_{RRCS} = \begin{cases} \frac{Q'_{ENLOE} / Q'_{AE}}{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.2}{0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \\ \frac{0.2}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & \end{cases}$$

$$R'_{WRRCS} = \begin{cases} \frac{Q'_{ABER} / Q'_{AE}}{0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} \leq 914 \\ \frac{Z'_{STAGING_AREA_RR} - Z'_{STAGING_AREA_WRR} + 0.4}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.2} & Z'_{STAGING_AREA_RR} > Z'_{STAGING_AREA_WRR} & Z'_{max} > 914 \\ \frac{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4}{Z'_{STAGING_AREA_WRR} - Z'_{STAGING_AREA_RR} + 0.4} & Z'_{STAGING_AREA_RR} < Z'_{STAGING_AREA_WRR} & \end{cases}$$

2.6.5 Compute the actual duration, $\Delta T'_{QTOWN}$, that the allowable flow, Q'_{TOWN} , has been maintained.

T^0_{QTOWN} is obtained from Operation Procedure 2.4.27.

$$\Delta T'_{QTOWN} = T'_{CURRENT} - T^0_{QTOWN}$$

2.6.6 Compare the $\Delta T'_{QTOWN}$ with DT'_{QTOWN} . DT'_{QTOWN} is obtained from Operation Procedure 2.5.17.

- If DT'_{QTOWN} is greater than $\Delta T'_{QTOWN}$, continue with procedure 2.6.7.
- If DT'_{QTOWN} is less than $\Delta T'_{QTOWN}$, continue with procedure 2.6.8.

2.6.7 Use Q'_{TOWN} to compute the allowable flow to pass WRRCS and RRCS, Q'_{WRRCS} and Q'_{RRCS} .

Q'_{TOWN} is obtained from Operation Procedure 2.5.16. Then skip to Operation Procedure 2.6.10.

- Allowable flow to pass the WRRCS (Q'_{WRRCS})

$$Q'_{WRRCS} = R'_{WRRCS} \times Q'_{TOWN}$$

- Allowable flow to pass the RRCS (Q'_{RRCS})

$$Q'_{RRCS} = R'_{RRCS} \times Q'_{TOWN}$$

2.6.8 If DT'_{QTOWN} is less than $\Delta T'_{QTOWN}$, compute the designed receding flow to pass through town,

$$Q'_{TOWN-RECEDING}$$

$$Q_{TOWN-RECEDING}^t = \begin{cases} 0.994^{\Delta t} \times Q_{TOWN}^{t-1} & \text{First iteration} \\ \begin{cases} 0.994^{\Delta t} \times Q_{TOWN-RECEDING}^{t-1} & Q_{TOWN-RECEDING}^{t-1} > 8,000 \text{ cfs} \\ 8,000 \text{ cfs} & Q_{TOWN-RECEDING}^{t-1} < 8,000 \text{ cfs} \end{cases} & \text{Subsequent iterations} \end{cases}$$

2.6.9 Use computed $Q_{TOWN-RECEDING}^t$ as the allowable flow to pass through the town

- Allowable flow to pass the WRRCS (Q_{WRRCS}^t)

$$Q_{WRRCS}^t = R_{WRRCS}^t \times Q_{TOWN-RECEDING}^t$$

- Allowable flow to pass the RRCS (Q_{RRCS}^t)

$$Q_{RRCS}^t = R_{RRCS}^t \times Q_{TOWN-RECEDING}^t$$

2.6.10 Calculate the allowable flow to pass the DICS, Q_{DICS}^t

The calculation of the allowable flow to pass through the DICS is based on the following three steps using Equations A-C. Each of these steps might take more than one operation iterations based on the magnitude of diversion flow and water surface elevation in the Staging Area. It should be noted that Equation 2 may or may not be necessary depending on the magnitude of the flood and the water surface elevation in the Staging Area.

- Maintain a constant increasing rate through DICS until the maximum design flow is reached, $Q_{DI_MAX}^t$, if possible (Equation A). If the water surface elevation in the Staging Area drops below the top of the gates and the calculated diversion flow, Q_{DICS}^t , using equation A does not reach the maximum design flow, then skip Equation B and continue to Equation C.
- Maintain the maximum DICS flow, $Q_{DI_MAX}^t$, as long as possible (Equation B), and
- If DICS flow cannot maintain maximum design flow, open gates (Equation C).

$$Q_{DICS}^t = \begin{cases} 1.005^{\Delta t} \times Q_{DICS}^{t-1} & Q_{DICS}^{t-1} < Q_{DI_MAX}^t \text{ \& SA WSE is above the top of gates (A)} \\ Q_{DI_MAX}^t & Q_{DICS}^{t-1} = Q_{DI_MAX}^t \text{ \& SA WSE is above the top of gates (B)} \\ \text{Table 12 } (Z_{STAGING_AREA_WRR}^t) & Q_{DICS}^{t-1} < Q_{DI_MAX}^t \text{ \& SA WSE is below the top of gates (C)} \end{cases}$$

where Δt = number of hours per operation iteration.

2.6.11 Determine the gate openings for WRRCS, RRCS, and DICS using the computed allowable flow and the Staging Area water surface elevation using Tables 5-7.

- Gate opening at Wild Rice River Control Structure

$$H_{WRRCS}^t = \text{Table 5} (Q_{WRRCS}^t, Z_{STAGING_AREA_WRR}^t)$$

- Gate opening at Red River Control Structure

$$H_{RRCS}^t = \text{Table 6} (Q_{RRCS}^t, Z_{STAGING_AREA_RR}^t)$$

- Gate opening at Diversion Inlet Control Structure

$$H_{DICS}^t = \text{Table 7} (Q_{DICS}^t, Z_{STAGING_AREA_WRR}^t)$$

2.6.12 Continue the iteration and repeat the procedures in “Operation Segment 6 - Falling” until the sum of the computed gate opening and gate invert is greater than the Staging Area water surface elevation for the DICS, WRRCS, and RRCS. Then the gates on these structures can be returned to their normal (pre-flood) positions. The time at which the three control structure gates are returned to their normal position will be dependent on the receding limb of the current flood.

3. Operation Plan for Floods Greater than 500-year and Less than or Equal to the IDF

A National Weather Service (NWS) flood forecast will be required to determine if preparation for extreme event operation shall take place. This is expected to occur less than 0.2% of the time. Within the extreme event analysis, the objectives shift from minimizing downstream impacts (normal operation) to protecting the project infrastructure from failure. The operation procedures and guidelines for the floods greater than a 500-year through IDF have been summarized as follows:

- Provide no restriction through any of the control structures until the maximum stage through the FDRA reaches 40 feet, at which point the RRCS and WRRCS will be operated to maintain 40 feet (or less) through the FDRA.
- The DICS gates are to be wide open throughout the duration of the event.
- The RRCS and WRRCS will be operated as necessary to maintain a maximum water surface elevation in the Staging Area of 925 feet.

The WRRCS and RRCS gates are operated based on the following rules:

3.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q_{ABER}^t
- 0505152130, Red River at Enloe,

3.2 Sum current flows at Abercrombie and Enloe, Q_{AE}^t

$$Q_{ABER}^t + Q_{ENLOE}^t = Q_{AE}^t$$

3.3 Record the water surface elevations in the Staging Area

- Wild Rice River, $Z_{STAGING_AREA_WRR}^t$
- Red River, $Z_{STAGING_AREA_RR}^t$

3.4 Calculate the flow proportion ratios for WRRCS and RRCS, R_{WRRCS}^t and R_{RRCS}^t

Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z_{max}^t .

$$Z_{max}^t = \max(Z_{STAGING_AREA_RR}^t, Z_{STAGING_AREA_WRR}^t)$$

The ratios between WRRCS and RRCS are calculated based on the following equations:

$$R_{RRCS}^t = \begin{cases} \frac{Q_{ENLOE}^t / Q_{AE}^t}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

$$R_{WRRCS}^t = \begin{cases} \frac{Q_{ABER}^t / Q_{AE}^t}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

3.5 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q_{AE_MAX}^t = \text{Max}(Q_{AE}^t)$$

3.6 Compute the designed flow through town using Table 8

$$Q_{TOWN}^t = Table\ 8(Q_{AE_MAX}^t)$$

3.7 Compute the allowable flow to pass through WRRCS and RRCS, Q_{WRRCS}^t and Q_{RRCS}^t .

- Allowable flow to pass the WRRCS (Q_{WRRCS}^t)

$$Q_{WRRCS}^t = R_{WRRCS}^t \times Q_{TOWN}^t$$

- Allowable flow to pass the RRCS (Q_{RRCS}^t)

$$Q_{RRCS}^t = R_{RRCS}^t \times Q_{TOWN}^t$$

3.8 Determine the gate openings for WRRCS, RRCS, and DICS using the computed allowable flow and the Staging Area water surface elevation (Tables 5-7).

- Gate opening at Wild Rice River Control Structure

$$H_{WRRCS}^t = Table\ 5(Q_{WRRCS}^t, Z_{STAGING_AREA_WRR}^t)$$

- Gate opening at Red River Control Structure

$$H_{RRCS}^t = Table\ 6(Q_{RRCS}^t, Z_{STAGING_AREA_RR}^t)$$

- Gate opening at Diversion Inlet Control Structure

$$H_{DICS}^t = 26\ feet$$

3.9 Continue the iteration and repeat the procedures in the IDF operation flow chart (Flow chart A) until the sum of the computed gate opening and gate invert is greater than the Staging Area water surface elevation for the DICS, WRRCS, and RRCS. Then the gates on these structures can be returned to their normal (pre-flood) positions.

4. Operation Plan for Flood Greater than the IDF

If the NWS forecast indicates a possibility of a flood greater than an IDF, the diversion operation is to be based on the following guidelines:

- The cities send out an evacuation preparation order when the NWS forecast shows a chance of flood greater than the IDF.
- Provide no restriction through any of the control structures until the maximum stage through the Flood Damage Reduction Area (FDRA, area through town) reaches 40 feet, at which point the RRCS and WRRCS will be operated to maintain 40 feet (or less) through the FDRA.
- The DICS gates are to be wide open throughout the duration of the event.
- The cities send out an evacuation order when the water surface elevation in the Staging Area reaches a critical elevation (i.e., an elevation between 918 to 922 feet depending on the flood magnitudes).
- The RRCS and WRRCS will gradually release more flow to maintain a relatively constant acceptable water surface elevation in the Staging Area (i.e., an elevation of 924.5 feet).

This approach is designed to:

- Provide sufficient time for the evacuation
- Maximize the use of the Staging Area storage
- Reduce the impact through the Flood Damage Reduction Area by only releasing what is necessary based on the magnitude of the flood.

The Diversion Inlet Control Structure Gates are wide open throughout the duration of this flood event to divert as much flow as possible, and the WRRCS and RRCS gates are operated based on the following rules:

4.1 Record flows from USGS gages:

- 05053000, Wild Rice River at Abercrombie, Q_{ABER}^t
- 0505152130, Red River at Enloe,

4.2 Sum current flows at Abercrombie and Enloe, Q_{AE}^t

$$Q_{ABER}^t + Q_{ENLOE}^t = Q_{AE}^t$$

4.3 Record the water surface elevations in the Staging Area at current iteration:

- Wild Rice River, $Z_{STAGING_AREA_WRR}^t$
- Red River, $Z_{STAGING_AREA_RR}^t$

Record the water surface elevations at Red River in the Staging Area from previous operation iterations $t-1$ and $t-2$:

- Red River, $Z_{STAGING_AREA_RR}^{t-1}$ and $Z_{STAGING_AREA_RR}^{t-2}$

4.4 Calculate the flow proportion ratios for WRRCS and RRCS, R_{WRRCS}^t and R_{RRCS}^t

Record the maximum water surface elevation in the Staging Area at Wild Rice River and Red River up to time t , Z_{max}^t .

$$Z_{max}^t = \max(Z_{STAGING_AREA_RR}^t, Z_{STAGING_AREA_WRR}^t)$$

The ratios between WRRCS and RRCS are calculated based on the following equations:

$$R_{RRCS}^t = \begin{cases} \frac{Q_{ENLOE}^t / Q_{AE}^t}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

$$R_{WRRCS}^t = \begin{cases} \frac{Q_{ABER}^t / Q_{AE}^t}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t \leq 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.2}{Z_{STAGING_AREA_RR}^t - Z_{STAGING_AREA_WRR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t > Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \\ \frac{0.2}{\frac{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.2}{Z_{STAGING_AREA_WRR}^t - Z_{STAGING_AREA_RR}^t + 0.4}} & Z_{STAGING_AREA_RR}^t < Z_{STAGING_AREA_WRR}^t & Z_{max}^t > 914 \end{cases}$$

4.5 Determine if the maximum water surface elevation, Z_{max}^t , in the Staging Area has reached the critical elevation (e.g., 924 feet)

- if the maximum water surface elevation, Z_{max}^t , in the Staging Area is less than the critical elevation (e.g., 924 feet), continue with the procedure 4.6.
- if the maximum water surface elevation, Z_{max}^t , in the Staging Area is equal to or greater than the critical elevation (e.g., 924 feet), continue with the procedure 4.10.

4.6 Record the maximum combined Abercrombie and Enloe flow up to time t

$$Q_{AE_MAX}^t = \text{Max}(Q_{AE}^t)$$

4.7 Compute the designed flow through town using Table 8

$$Q_{TOWN}^t = \text{Table 8}(Q_{AE_MAX}^t)$$

4.8 Compute the allowable flow to pass through WRRCS and RRCS, Q'_{WRRCS} and Q'_{RRCS} .

- Allowable flow to pass the WRRCS (Q'_{WRRCS})

$$Q'_{WRRCS} = R'_{WRRCS} \times Q'_{TOWN}$$

- Allowable flow to pass the RRCS (Q'_{RRCS})

$$Q'_{RRCS} = R'_{RRCS} \times Q'_{TOWN}$$

4.9 Determine the gate openings for WRRCS, RRCS, and DICS using the computed allowable flow and the Staging Area water surface elevation (Tables 5-7).

- Gate opening at Wild Rice River Control Structure

$$H'_{WRRCS} = \text{Table 5}(Q'_{WRRCS}, Z'_{STAGING_AREA_WRR})$$

- Gate opening at Red River Control Structure

$$H'_{RRCS} = \text{Table 6}(Q'_{RRCS}, Z'_{STAGING_AREA_RR})$$

- Gate opening at Diversion Inlet Control Structure

$$H'_{DICS} = 26 \text{ feet}$$

Continue the iteration and repeat the operation procedures 4.1-4.9 in PMF operation flow chart (Flow chart A) until the maximum water surface elevation in the Staging Area is equal to or greater than the critical elevation (e.g., 924 feet).

4.10 Record the Diversion Inlet Control Structure flow from previous operation iteration, Q^{t-1}_{DICS}

4.11 Compute the volume of water in the Staging Area for the current operation iteration t and previous operation iterations $t-1$ and $t-2$ (Table 4):

$$V'_{STAGING_AREA} = \text{Table 4}(Z'_{STAGING_AREA_RR})$$

$$V^{t-1}_{STAGING_AREA} = \text{Table 4}(Z^{t-1}_{STAGING_AREA_RR})$$

$$V^{t-2}_{STAGING_AREA} = \text{Table 4}(Z^{t-2}_{STAGING_AREA_RR})$$

4.12 Compute average volume of water in the Staging Area, $\overline{V'_{STAGING_AREA}}$, from the recent three operation iterations t , $t-1$ and $t-2$:

$$\overline{V'_{STAGING_AREA}} = (V'_{STAGING_AREA} + V^{t-1}_{STAGING_AREA} + V^{t-2}_{STAGING_AREA}) / 3$$

This equation is applied to estimate average volume of water in the Staging Area by smoothing the errors caused by the variations in the water surface elevation that might occur during the extreme large flood events.

4.13 Compute the available flow to pass through the town and DICS gates

$$Q'_{PMF} = \frac{(V'_{STAGING_AREA} - 308496) \times 43560}{3600 \times \Delta t}$$

where 308,496 is the volume of water in the Staging Area in ac-ft corresponding to a water surface elevation of 924 feet. The equation is derived to calculate the volume of water that is above 924 feet in the Staging Area, which will be released through DICS, WRRCS, and RRCS gates.

4.14 Compute the allowable flow to pass through WRRCS and RRCS, Q'_{WRRCS} and Q'_{RRCS} . Since the water surface elevation in the Staging Area is constantly high during the PMF operation (i.e., above 924 feet), it is reasonable to assume that the flow passing through the DICS gates at current iteration remains the same as the previous iteration.

- Allowable flow to pass the WRRCS (Q'_{WRRCS})

$$Q'_{WRRCS} = R'_{WRRCS} \times (Q'_{PMF} - Q^{t-1}_{DICS})$$

- Allowable flow to pass the RRCS (Q_{RRCS}^t)

$$Q_{RRCS}^t = R_{RRCS}^t \times (Q_{PMF}^t - Q_{DICS}^{t-1})$$

4.15 Determine the gate openings for WRRCS and RRCS using the computed allowable flow and the Staging Area water surface elevation (Tables 5-7).

- Gate opening at Wild Rice River Control Structure

$$H_{WRRCS}^t = \text{Table 5}(Q_{WRRCS}^t, Z_{STAGING_AREA_WRR}^t)$$

- Gate opening at Red River Control Structure

$$H_{RRCS}^t = \text{Table 6}(Q_{RRCS}^t, Z_{STAGING_AREA_RR}^t)$$

- Gate opening at Diversion Inlet Control Structure

$$H_{DICS}^t = 26 \text{ feet}$$

4.16 Continue the iteration and repeat the procedures in the PMF operation flow chart (Flow chart A) until the sum of the computed gate opening and gate invert is greater than the Staging Area water surface elevation for the DICS, WRRCS, and RRCS. Then the gates on these structures can be returned to their normal (pre-flood) positions. The sensitivity analysis shows that 2-hour diversion operation iteration (i.e., $\Delta t = 2$ hour) generates more reasonable operation results for the PMF operation.

5. Verification of Diversion Operation to Maintain a Level Pool in the Staging Area

To verify if the gate operation is maintaining a relatively level pool from one side of the staging area to the other (between the Wild Rice River and the Red River), a sensitivity analysis was conducted with varied inflow hydrology to the project. First, the historical records at Enloe on the Red River, and Abercrombie on the Wild Rice River were examined. The peak flow ratios between Abercrombie and Enloe were mostly near 45:55, and the most extreme ratio was 36:64. Several model runs were created that had varying inflow ratios between Abercrombie and Enloe. The ratios were 20:80, 30:70, 50:50, 60:40, 70:30 and were all evaluated for the 50-, 100-, and 500-year flood events.

The simulation results showed that water surface differential between the Wild Rice River and Red River in the Staging Area for most of the scenarios ranged from 0.1 to 0.5 feet, and the maximum water surface differential between the Wild Rice River and Red River of the North was 0.9 feet with the flow ratio between Abercrombie and Enloe of 20:80. These results indicate that the approach to calculate the allowable flow through DICS, WRRCS, and RRCS using the current operation is able to maintain a relatively level pool in the Staging Area during a flood operation.

6. Abbreviations and Notations

a: Coefficient for the Nonlinear Reservoir Model (Operation Segment 4 - Rising)

b: Coefficient for the Nonlinear Reservoir Model (Operation Segment 5 - Transition)

c: Coefficient for the Nonlinear Reservoir Model (Operation Segment 5 - Transition)

DT_{QTOWN}^t : Designed time duration to allow maximum flow through the town, Q_{TOWN}^t (hr)

H_{DICS}^t : Gate opening at the Diversion Inlet Control Structure

H_{RRCS}^t : Gate opening at the Red River Control Structure
 H_{WRRCS}^t : Gate opening at the Wild Rice River Control Structure
 Q_{ABER}^t : Flow at Abercrombie, Wild Rice River
 Q_{AE}^t : The combined flow at the Abercrombie, Wild Rice River, and Enloe, Red River
 $Q_{AE_MAX}^t$: The maximum of the combined flow at Abercrombie and Enloe
 $Q_{DI_MAX}^t$: Designed maximum flow through Diversion Inlet Control Structure (Note: $Q_{DI_MAX}^t = 20,000$ cfs for flood events less than 200-year i.e., $Q_{AE_MAX}^t = 48,000$ cfs, and $Q_{DI_MAX}^t = 25,000$ cfs for the flood equal to the 500-year event i.e., $Q_{AE_MAX}^t = 60,000$ cfs. For any flood events that are larger than the 200-year and less than the 500-year event, $Q_{DI_MAX}^t$ can be calculated using linear interpolation method.)
 Q_{ENLOE}^t : Flow at Enloe, Red River
 $Q_{FALLING}^t$: Calculated allowable flow for the falling limb of the hydrograph for Operation segment 6
 Q_{HOLD}^t : Calculated allowable flow to be hold through the RRCS and WRRCS for Operation segment 3
 Q_{HOLD}^{t0} : Calculated maximum flow to be hold through the RRCS and WRRCS for Operation segment 2
 $Q_{MAPLE}^{t-36/\Delta t}$: Flow at Durbin, Maple River, 36 hours ago
 $Q_{MAPLE_REDUC}^t$: Calculated flow reduction for the Maple River
 Q_{PMF}^t : Calculated allowable flow to pass through the town and DICS gates for the PMF operation
 $Q_{REDUCTION}^t$: Calculated total flow reduction for the Maple River, Rush River, and Sheyenne River
 $Q_{REDUCTION_MAX}^t$: Calculated maximum total flow reduction for the Maple River, Rush River, and Sheyenne River
 Q_{RISING}^t : Calculated designed flow for the rising limb of the hydrograph
 Q_{RRCS}^t : Calculated allowable flow through Red River Control Structure
 $Q_{RRCS_HOLD}^{t0}$: Calculated maximum flow to be hold through the RRCS for Operation segment 3
 $Q_{RUSH}^{t-36/\Delta t}$: Flow at Amenia, Rush River, 36 hours ago
 $Q_{RUSH_REDUC}^t$: Calculated flow reduction for the Rush River
 Q_{SF}^t : Calculated allowable flow for small flood operation
 $Q_{SHEY}^{t-36/\Delta t}$: Flow at Gol Road, Sheyenne River, 36 hours ago
 $Q_{SHEY_REDUC}^t$: Calculated flow reduction for the Sheyenne River
 Q_{TOWN}^t : The maximum allowable flow to be sent through the town
 $Q_{TOWN-RECEDING}^t$: The allowable receding flow to be sent through the town
 $Q_{TRANSITION}^t$: Calculated allowable flow for the transition limb of the hydrograph for Operation segment 5
 Q_{WRRCS}^t : Calculated allowable flow though Wild Rice River Control Structure
 $Q_{WRRCS_HOLD}^{t0}$: Calculated maximum flow to be hold through the WRRCS for Operation segment 3

R_{RRCS}^t : Splitting ratio for flow releasing through the Red River Control Structure

R_{WRRCS}^t : Splitting ratio for flow releasing through the Wild Rice River Control Structure

t : The number of project operation iteration since operation began (e.g., 1, 2, 3,...)

$T_{CURRENT}^t$: HEC-RAS model simulation time

T_{TOWN}^0 : The HEC-RAS model simulation time as the calculated Q_{RISING}^t exceeds Q_{TOWN}^t

$V_{STAGING_AREA}^t$: The volume of water stored in the Staging Area at current operation iteration t

$\overline{V_{STAGING_AREA}^t}$: The average volume of water stored in the Staging Area from the recent three operation iterations t , $t-1$, and $t-2$

$V_{STAGING_AREA}^{t-1}$: The volume of water stored in the Staging Area at previous operation iteration $t-1$

$V_{STAGING_AREA}^{t-2}$: The volume of water stored in the Staging Area at operation iteration $t-2$

$Z_{STAGING_AREA_RR}^t$: Current water surface elevation in Staging Area at the Red River, Breck to Wolv, XS 2531315

$Z_{STAGING_AREA_RR}^{t-1}$: Water surface elevation in Staging Area at the Red River, Breck to Wolv, XS 2531315 from previous operation iteration $t-1$

$Z_{STAGING_AREA_RR}^{t-2}$: Water surface elevation in Staging Area at the Red River, Breck to Wolv, XS 2531315 from the operation iteration $t-2$

$Z_{STAGING_AREA_WRR}^t$: Current water surface elevation in Staging Area at the Wild Rice River, DR37 to DIV, XS 64862

Z_{max}^t : Maximum water surface elevation in the Staging Area at Wild Rice River, DR37 to DIV, XS 64862 and Red River Breck to Wolv, XS 2531315 up to time t

Δt = Number of hours per operation iteration

ΔT_{TOWN} : Actual duration that the allowable flow, Q_{TOWN}^t , has been maintained (hr).

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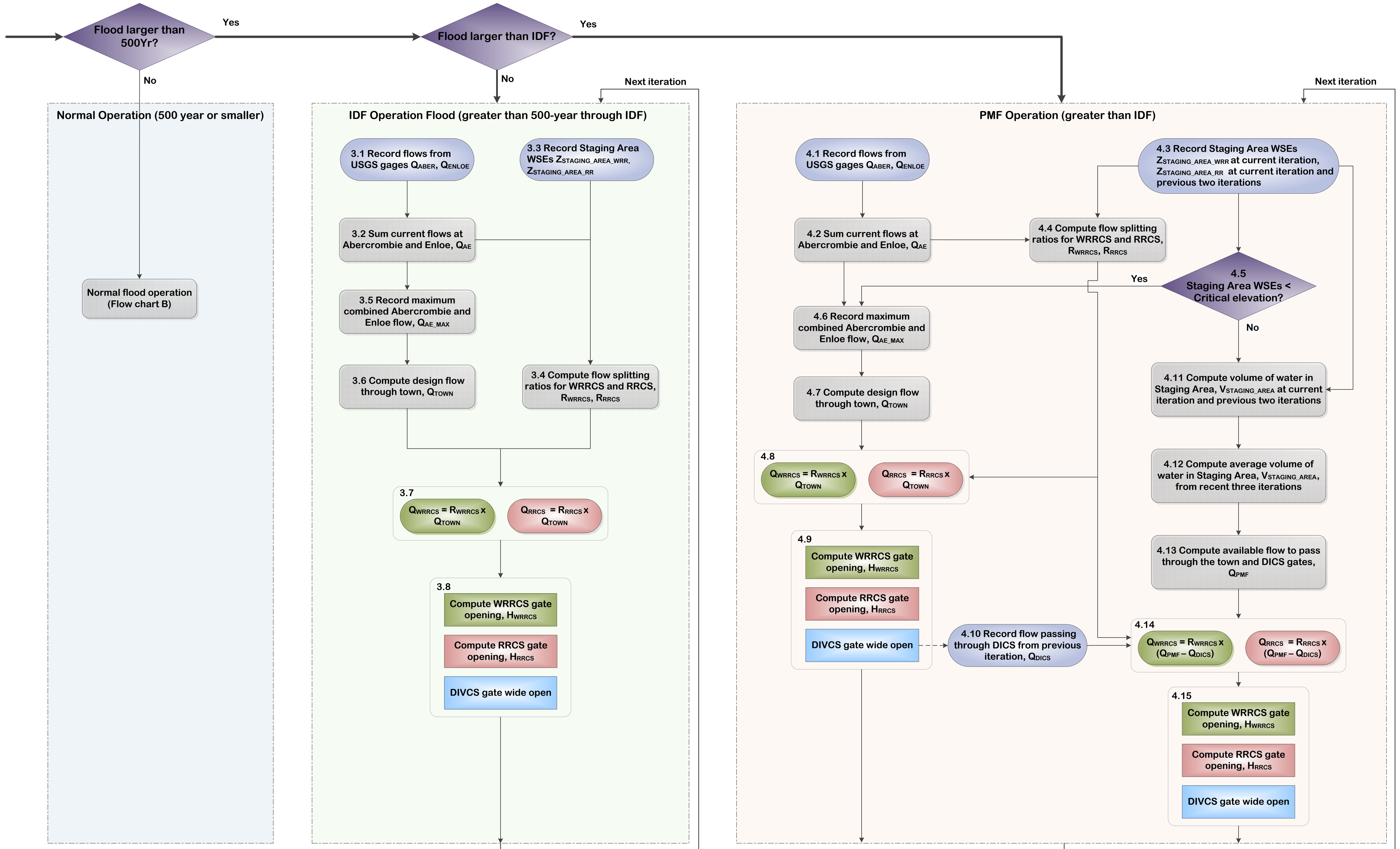
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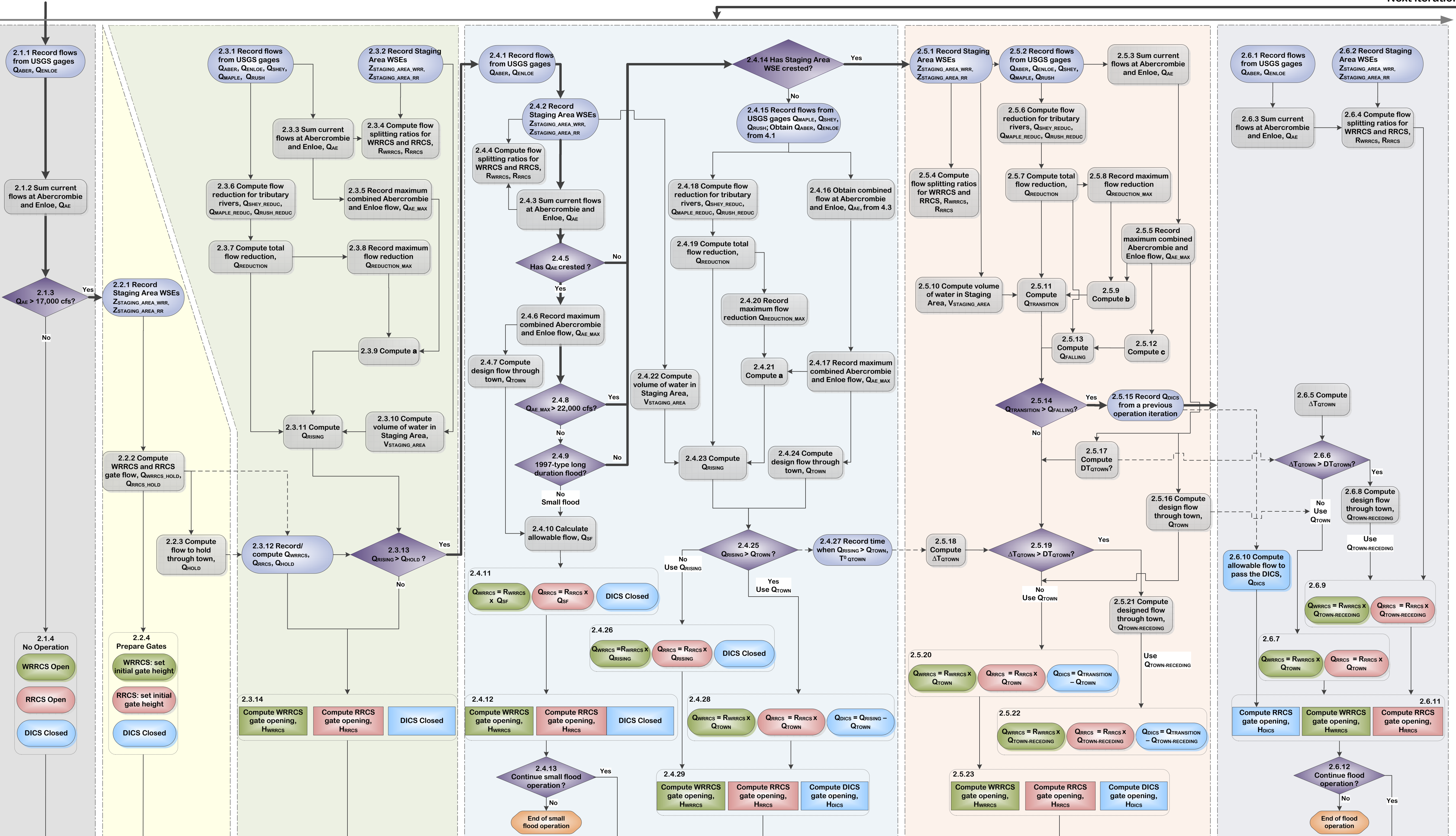
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Fargo-Moorhead Diversion Project Operation Plan (Flowchart A)



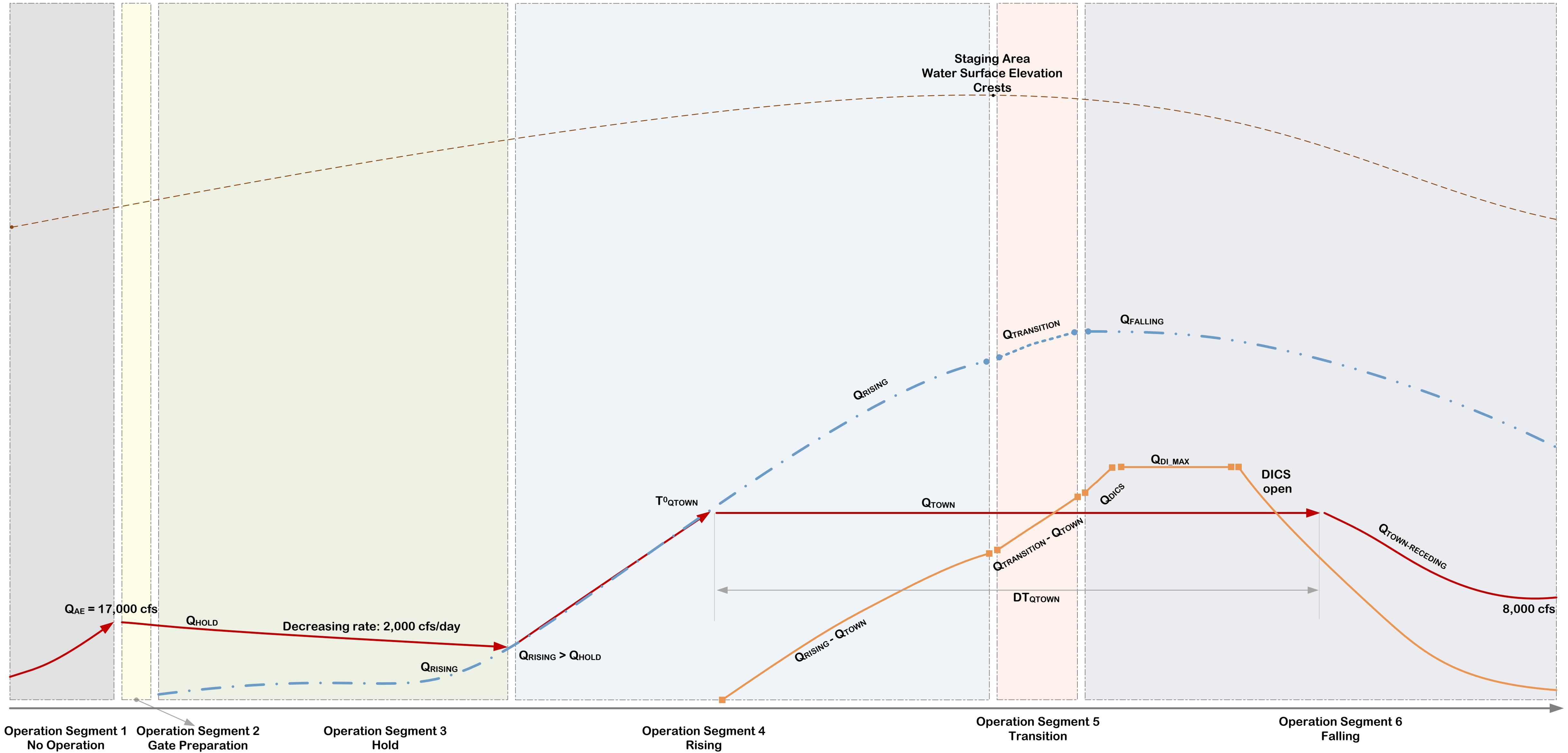
Fargo-Moorhead Diversion Project Operation Plan (Flowchart B)

Next iteration



Operation Segment 1 No Operation Operation Segment 2 Gate Preparation Operation Segment 3 Hold Operation Segment 4 Rising Operation Segment 5 Transition Operation Segment 6 Falling

Fargo-Moorhead Diversion Project Operation Plan (Flowchart C)



Operation Segment 1
No Operation
Operation Segment 2
Gate Preparation
Operation Segment 3
Hold
Operation Segment 4
Rising
Operation Segment 5
Transition
Operation Segment 6
Falling

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Staging Area Elevation at Wild Rice Rive, Z _{STAGING-AREA-WRR} (ft)	Flow at WRRCS, Q _{WRRCS} (cfs)	Staging Area Elevation at Wild Rice Rive, Z _{STAGING-AREA-WRR} (ft)	Flow at WRRCS, Q _{WRRCS} (cfs)
900.4	1,403	913.4	9,105
900.6	1,455	913.6	9,234
900.8	1,506	913.8	9,387
901.0	1,557	914.0	9,589
901.2	1,609	914.2	9,869
901.4	1,660	914.4	10,114
901.6	1,712	914.6	10,346
901.8	1,763	914.8	10,583
902.0	1,815	915.0	10,898
902.2	1,866	915.2	11,189
902.4	1,918	915.4	11,671
902.6	1,969	915.6	12,393
902.8	2,021	915.8	13,288
903.0	2,072	916.0	14,344
903.2	2,124	916.2	15,562
903.4	2,175	916.4	16,957
903.6	2,239	916.6	18,541
903.8	2,312	916.8	20,129
904.0	2,386	917.0	21,729
904.2	2,459	917.2	23,458
904.4	2,532	917.4	25,322
904.6	2,605	917.6	27,171
904.8	2,678	917.8	29,109
905.0	2,753	918.0	31,112
905.2	2,835	918.2	32,938
905.4	2,916	918.4	34,708
905.6	2,997	918.6	36,499
905.8	3,081	918.8	37,947
906.0	3,186	919.0	39,320
906.2	3,291	919.2	40,359
906.4	3,395	919.4	40,361
906.6	3,511	919.6	40,363
906.8	3,640	919.8	40,365
907.0	3,770	920.0	40,367
907.2	3,900	920.2	40,369
907.4	4,029	920.4	40,371
907.6	4,206	920.6	40,373
907.8	4,392	920.8	40,375
908.0	4,579	921.0	40,377
908.2	4,766	921.2	40,379
908.4	4,953	921.4	40,381
908.6	5,139	921.6	40,383
908.8	5,326	921.8	40,385
909.0	5,513	922.0	40,387
909.2	5,700	922.2	40,389
909.4	5,886	922.4	40,391
909.6	6,073	922.6	40,393
909.8	6,235	922.8	40,395
910.0	6,396	923.0	40,397
910.2	6,558	923.2	40,399
910.4	6,719	923.4	40,401
910.6	6,880	923.6	40,403
910.8	7,042	923.8	40,405
911.0	7,203	924.0	40,407
911.2	7,364	924.2	40,409
911.4	7,526	924.4	40,411
911.6	7,687	924.6	40,413
911.8	7,831	924.8	40,415
912.0	7,948	925.0	40,417
912.2	8,065	925.2	40,419
912.4	8,181	925.4	40,421
912.6	8,336	925.6	40,423
912.8	8,511	925.8	40,425
913.0	8,699	926.0	40,427
913.2	8,927		

Table 2 Red River Control Structure (RRCS) Flow-Stage Relationship (Gate Open)

Staging Area Elevation at Red River, $Z_{\text{STAGING-AREA-RR}}$ (ft)	Flow at RRCS, Q_{RRCS} (cfs)	Staging Area Elevation at Red River, $Z_{\text{STAGING-AREA-RR}}$ (ft)	Flow at RRCS, Q_{RRCS} (cfs)
891.2	1,268	905.8	8,438
891.4	1,313	906.0	8,607
891.6	1,383	906.2	8,786
891.8	1,454	906.4	8,936
892.0	1,527	906.6	9,114
892.2	1,596	906.8	9,287
892.4	1,662	907.0	9,448
892.6	1,728	907.2	9,659
892.8	1,792	907.4	9,864
893.0	1,856	907.6	10,067
893.2	1,919	907.8	10,263
893.4	1,983	908.0	10,462
893.6	2,047	908.2	10,663
893.8	2,113	908.4	10,866
894.0	2,179	908.6	11,082
894.2	2,246	908.8	11,303
894.4	2,312	909.0	11,525
894.6	2,377	909.2	11,755
894.8	2,443	909.4	11,995
895.0	2,510	909.6	12,245
895.2	2,578	909.8	12,508
895.4	2,648	910.0	12,847
895.6	2,719	910.2	13,274
895.8	2,788	910.4	13,782
896.0	2,858	910.6	14,271
896.2	2,927	910.8	14,709
896.4	3,005	911.0	15,127
896.6	3,085	911.2	15,521
896.8	3,163	911.4	15,892
897.0	3,242	911.6	16,234
897.2	3,322	911.8	16,607
897.4	3,403	912.0	16,978
897.6	3,487	912.2	17,352
897.8	3,579	912.4	17,714
898.0	3,671	912.6	18,095
898.2	3,761	912.8	18,490
898.4	3,853	913.0	18,906
898.6	3,944	913.2	19,357
898.8	4,039	913.4	19,773
899.0	4,136	913.6	20,327
899.2	4,231	913.8	20,964
899.4	4,320	914.0	21,826
899.6	4,397	914.2	23,157
899.8	4,472	914.4	24,441
900.0	4,545	914.6	25,873
900.2	4,612	914.8	26,972
900.4	4,675	915.0	28,167
900.6	4,775	915.2	29,436
900.8	4,894	915.4	30,698
901.0	5,017	915.6	32,016
901.2	5,141	915.8	33,496
901.4	5,273	916.0	35,146
901.6	5,403	916.2	37,001
901.8	5,532	916.4	39,016
902.0	5,660	916.6	40,711
902.2	5,784	916.8	40,731
902.4	5,908	917.0	40,751
902.6	6,038	917.2	40,771
902.8	6,168	917.4	40,791
903.0	6,304	917.6	40,811
903.2	6,445	917.8	40,831
903.4	6,594	918.0	40,851
903.6	6,743	918.2	40,871
903.8	6,876	918.4	40,891
904.0	7,015	918.6	40,911
904.2	7,162	918.8	40,931
904.4	7,319	919.0	40,951
904.6	7,474	919.2	40,971
904.8	7,633	919.4	40,991
905.0	7,793	919.6	41,011
905.2	7,950	919.8	41,031
905.4	8,107	920.0	41,051
905.6	8,274	920.2	41,071

Table 3 Coefficient α [-] for the Nonlinear Reservoir Model (Rising Segment)

Maximum flow from combined Enloe and Abercrombie Q^t_{AE-MAX} (cfs)	Maximum flow reduction, $Q^t_{REDUCTION-MAX}$ (cfs)																	
	0	500	1,000	1,500	2,000	2,500	3,000	3,500	4,000	5,000	6,000	7,000	8,000	9,000	10,000	15,000	20,000	50,000
20,000	0.0043	0.0043	0.0044	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0048	0.0048	0.0049	0.0054	0.0060	0.0065
21,000	0.0042	0.0043	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0048	0.0048	0.0053	0.0060	0.0064
22,000	0.0042	0.0043	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0048	0.0053	0.0059	0.0064
23,000	0.0042	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0047	0.0048	0.0053	0.0059	0.0064
24,000	0.0042	0.0043	0.0043	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0052	0.0059	0.0063
25,000	0.0041	0.0042	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0052	0.0059	0.0063
26,000	0.0041	0.0042	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0052	0.0058	0.0062
27,000	0.0041	0.0042	0.0042	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0046	0.0051	0.0057	0.0062
28,000	0.0040	0.0041	0.0042	0.0042	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0045	0.0046	0.0046	0.0051	0.0057	0.0061
29,000	0.0040	0.0041	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0050	0.0056	0.0061
30,000	0.0040	0.0040	0.0041	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0050	0.0056	0.0060
31,000	0.0039	0.0040	0.0041	0.0041	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0050	0.0056	0.0060
32,000	0.0039	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0049	0.0055	0.0059
33,000	0.0039	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0043	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0049	0.0055	0.0059
34,000	0.0038	0.0039	0.0040	0.0040	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0048	0.0054	0.0059
35,000	0.0038	0.0039	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0048	0.0054	0.0058
36,000	0.0038	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0048	0.0054	0.0058
37,000	0.0038	0.0038	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0043	0.0043	0.0047	0.0053	0.0057
38,000	0.0037	0.0038	0.0039	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0043	0.0047	0.0053	0.0057
39,000	0.0037	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0047	0.0052	0.0056
40,000	0.0037	0.0038	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0041	0.0042	0.0042	0.0046	0.0052	0.0056
41,000	0.0036	0.0037	0.0038	0.0038	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0041	0.0046	0.0051	0.0055
42,000	0.0036	0.0036	0.0037	0.0037	0.0038	0.0038	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0040	0.0041	0.0045	0.0050	0.0054
43,000	0.0035	0.0036	0.0036	0.0037	0.0037	0.0037	0.0038	0.0038	0.0038	0.0039	0.0039	0.0039	0.0039	0.0040	0.0040	0.0044	0.0049	0.0053
44,000	0.0036	0.0037	0.0038	0.0038	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0045	0.0051	0.0055
45,000	0.0037	0.0038	0.0039	0.0039	0.0039	0.0040	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0042	0.0047	0.0053	0.0057
46,000	0.0038	0.0039	0.0040	0.0040	0.0041	0.0041	0.0041	0.0042	0.0042	0.0042	0.0043	0.0043	0.0043	0.0043	0.0044	0.0048	0.0054	0.0058
47,000	0.0039	0.0040	0.0041	0.0041	0.0042	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0044	0.0045	0.0045	0.0050	0.0056	0.0060
48,000	0.0040	0.0041	0.0042	0.0043	0.0043	0.0043	0.0044	0.0044	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046	0.0046	0.0051	0.0057	0.0062
49,000	0.0042	0.0043	0.0044	0.0044	0.0045	0.0045	0.0046	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0048	0.0048	0.0053	0.0060	0.0064
50,000	0.0044	0.0045	0.0046	0.0046	0.0047	0.0047	0.0047	0.0048	0.0048	0.0048	0.0049	0.0049	0.0049	0.0050	0.0050	0.0055	0.0062	0.0067
51,000	0.0046	0.0047	0.0048	0.0048	0.0048	0.0049	0.0049	0.0050	0.0050	0.0050	0.0051	0.0051	0.0051	0.0052	0.0052	0.0057	0.0064	0.0069
52,000	0.0047	0.0048	0.0049	0.0050	0.0050	0.0051	0.0051	0.0051	0.0052	0.0052	0.0053	0.0053	0.0053	0.0054	0.0054	0.0060	0.0067	0.0072
53,000	0.0049	0.0050	0.0051	0.0051	0.0052	0.0052	0.0053	0.0053	0.0054	0.0054	0.0054	0.0055	0.0055	0.0055	0.0056	0.0062	0.0069	0.0074
54,000	0.0051	0.0052	0.0053	0.0053	0.0054	0.0054	0.0055	0.0055	0.0056	0.0056	0.0056	0.0057	0.0057	0.0057	0.0058	0.0064	0.0072	0.0077
55,000	0.0052	0.0053	0.0055	0.0055	0.0056	0.0056	0.0056	0.0057	0.0057	0.0058	0.0058	0.0059	0.0059	0.0059	0.0060	0.0066	0.0074	0.0080
56,000	0.0054	0.0055	0.0056	0.0057	0.0057	0.0058	0.0058	0.0059	0.0059	0.0060	0.0060	0.0060	0.0061	0.0061	0.0062	0.0068	0.0076	0.0082
57,000	0.0056	0.0057	0.0058	0.0059	0.0059	0.0060	0.0060	0.0061	0.0061	0.0062	0.0062	0.0062	0.0063	0.0063	0.0064	0.0070	0.0079	0.0085
58,000	0.0057	0.0059	0.0060	0.0060	0.0061	0.0061	0.0062	0.0062	0.0063	0.0063	0.0064	0.0064	0.0065	0.0065	0.0065	0.0072	0.0081	0.0087
59,000	0.0059	0.0060	0.0062	0.0062	0.0063	0.0063	0.0064	0.0064	0.0065	0.0065	0.0066	0.0066	0.0067	0.0067	0.0067	0.0074	0.0083	0.0090
60,000	0.0061	0.0062	0.0063	0.0064	0.0064	0.0065	0.0066	0.0066	0.0067	0.0067	0.0068	0.0068	0.0068	0.0069	0.0069	0.0077	0.0086	0.0092
65,000	0.0065	0.0067	0.0068	0.0069	0.0069	0.0070	0.0071	0.0071	0.0072	0.0072	0.0073	0.0073	0.0074	0.0074	0.0075	0.0082	0.0092	0.0099
70,000	0.0070	0.0071	0.0073	0.0074	0.0074	0.0075	0.0075	0.0076	0.0077	0.0077	0.0078	0.0078	0.0079	0.0079	0.0080	0.0088	0.0099	0.0106

Table 4 Stage-Volume Relationship of Staging Area

Water Surface Elevation at Red River, Z _{STAGING-AREA-RR} (ft)	Volume of water in Staging Area (Ac-ft)	Water Surface Elevation at Red River, Z _{STAGING-AREA-RR} (ft)	Volume of water in Staging Area (Ac-ft)
887	0	911	27,290
887.5	1	911.5	30,016
888	6	912	33,179
888.5	20	912.5	36,858
889	42	913	41,029
889.5	72	913.5	45,722
890	111	914	50,956
890.5	157	914.5	56,740
891	210	915	63,119
891.5	271	915.5	70,137
892	343	916	77,810
892.5	432	916.5	86,202
893	534	917	95,277
893.5	651	917.5	105,009
894	784	918	115,437
894.5	932	918.5	126,567
895	1,099	919	138,433
895.5	1,286	919.5	151,018
896	1,498	920	164,371
896.5	1,736	920.5	178,557
897	1,999	921	193,566
897.5	2,286	921.5	209,553
898	2,596	922	226,687
898.5	2,930	922.5	245,031
899	3,287	923	264,738
899.5	3,669	923.5	285,908
900	4,077	924	308,496
900.5	4,517	924.5	332,538
901	4,996	925	358,020
901.5	5,513	925.5	385,005
902	6,070	926	413,489
902.5	6,665	926.5	443,349
903	7,303	927	474,428
903.5	7,986	927.5	506,660
904	8,715	928	540,128
904.5	9,492	928.5	574,978
905	10,319	929	611,194
905.5	11,199	929.5	649,011
906	12,135	930	688,497
906.5	13,130	930.5	729,538
907	14,194	931	772,007
907.5	15,337	931.5	815,674
908	16,576	932	860,418
908.5	17,929	932.5	906,131
909	19,411	933	952,781
909.5	21,043	933.5	1,000,341
910	22,864	934	1,048,739
910.5	24,928	934.5	1,097,939
		935	1,147,955

Table 5 Wild Rice River Control Structure (WRRCS) Gate Opening (ft) - Part B

Flow through WRRCS (cfs)	Staging Area Elevation at Wild Rice River, Z _{STAGING-AREA-WRR} (ft)																																							
	907.8	908.0	908.2	908.4	908.6	908.8	909.0	909.2	909.4	909.6	909.8	910.0	910.2	910.4	910.6	910.8	911.0	911.2	911.4	911.6	911.8	912.0	912.2	912.4	912.6	912.8	913.0	913.2	913.4	913.6	913.8	914.0	914.2	914.4	914.6	914.8	915.0			
0	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
100	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
200	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
300	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
400	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
500	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
600	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
700	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
800	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	
900	0.43	0.42	0.42	0.42	0.42	0.41	0.41	0.41	0.41	0.40	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.39	0.39	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	
1,000	0.47	0.47	0.47	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.45	0.44	0.44	0.44	0.44	0.44	0.43	0.43	0.43	0.43	0.43	0.42	0.42	0.42	0.42	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	
1,100	0.52	0.52	0.51	0.51	0.51	0.51	0.51	0.50	0.50	0.50	0.49	0.49	0.49	0.48	0.48	0.48	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	0.46	
1,200	0.57	0.56	0.56	0.56	0.56	0.55	0.55	0.55	0.55	0.54	0.54	0.54	0.53	0.53	0.53	0.52	0.52	0.52	0.52	0.52	0.51	0.51	0.51	0.51	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	
1,300	0.63	0.62	0.61	0.61	0.61	0.60	0.60	0.60	0.60	0.59	0.59	0.59	0.58	0.58	0.58	0.57	0.57	0.57	0.56	0.56	0.56	0.55	0.55	0.55	0.55	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	
1,400	0.70	0.69	0.68	0.67	0.66	0.65	0.65	0.64	0.63	0.63	0.63	0.62	0.62	0.62	0.61	0.61	0.61	0.60	0.60	0.60	0.60	0.59	0.59	0.59	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	
1,500	0.78	0.76	0.75	0.73	0.72	0.71	0.70	0.69	0.68	0.67	0.67	0.66	0.66	0.65	0.65	0.65	0.64	0.64	0.64	0.63	0.63	0.63	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	
1,600	0.90	0.87	0.83	0.80	0.78	0.76	0.75	0.74	0.73	0.72	0.72	0.71	0.71	0.71	0.70	0.70	0.69	0.69	0.68	0.68	0.68	0.67	0.67	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	
1,700	1.04	1.00	0.95	0.91	0.88	0.84	0.81	0.79	0.78	0.77	0.76	0.76	0.75	0.75	0.74	0.74	0.73	0.73	0.73	0.72	0.72	0.71	0.71	0.71	0.70	0.70	0.70	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.69	
1,800	1.20	1.14	1.09	1.04	0.99	0.95	0.92	0.88	0.85	0.83	0.82	0.81	0.80	0.79	0.79	0.78	0.78	0.77	0.77	0.77	0.76	0.76	0.75	0.75	0.75	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	
1,900	1.37	1.30	1.23	1.18	1.12	1.08	1.03	0.99	0.95	0.92	0.90	0.88	0.87	0.86	0.85	0.84	0.83	0.83	0.82	0.81	0.81	0.80	0.80	0.79	0.79	0.79	0.78	0.78	0.78	0.77	0.77	0.77	0.77	0.77	0.77	0.77	0.77	0.77	0.77	
2,000	1.56	1.47	1.39	1.33	1.26	1.21	1.16	1.11	1.06	1.02	0.98	0.96	0.94	0.92	0.90	0.89	0.88	0.88	0.87	0.86	0.86	0.85	0.84	0.84	0.84	0.83	0.83	0.82	0.82	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	
2,100	1.77	1.67	1.57	1.49	1.41	1.35	1.29	1.23	1.18	1.13	1.09	1.05	1.01	0.98	0.96	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.89	0.88	0.88	0.87	0.87	0.86	0.86	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	
2,200	2.01	1.88	1.77	1.67	1.58	1.51	1.43	1.37	1.31	1.25	1.20	1.15	1.11	1.07	1.03	1.01	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.92	0.92	0.91	0.91	0.90	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	
2,300	2.26	2.13	1.99	1.87	1.77	1.68	1.59	1.51	1.44	1.38	1.32	1.27	1.22	1.17	1.13	1.10	1.07	1.04	1.02	1.01	0.99	0.98	0.97	0.97	0.96	0.96	0.95	0.95	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	
2,400	2.48	2.38	2.24	2.10	1.97	1.86	1.76	1.67	1.59	1.52	1.45	1.39	1.34	1.28	1.23	1.18	1.15	1.12	1.09	1.07	1.05	1.03	1.02	1.01	1.00	1.00	0.99	0.99	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
2,500	2.69	2.61	2.50	2.36	2.20	2.07	1.95	1.85	1.76	1.67	1.59	1.53	1.46	1.40	1.34	1.29	1.24	1.19	1.16	1.13	1.11	1.09	1.07	1.06	1.05	1.04	1.04	1.03	1.03	1.02	1.02	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	
2,600	2.87	2.79	2.70	2.59	2.46	2.30	2.16	2.04	1.93	1.84	1.75	1.67	1.59	1.52	1.46	1.41	1.34	1.29	1.24	1.20	1.17	1.15	1.13	1.11	1.10	1.09	1.08	1.07	1.07	1.06	1.06	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	
2,700	3.07	2.97	2.88	2.78	2.67	2.54	2.40	2.25	2.13	2.02	1.92	1.82	1.74	1.65	1.58	1.51	1.45	1.39	1.34	1.29	1.25	1.23	1.21	1.19	1.18	1.16	1.15	1.14	1.13	1.11	1.11	1.10	1.10	1.10	1.10	1.10	1.10	1.10		
2,800	3.29	3.18	3.08	2.98	2.88	2.77	2.64	2.49	2.34	2.21	2.10	1.98	1.89	1.79	1.71	1.64	1.57	1.51	1.45	1.39	1.34	1.30	1.26	1.23	1.20	1.18	1.17	1.16	1.15	1.14	1.14	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	
2,900	3.52	3.39	3.28	3.17	3.08	2.99	2.88	2.76	2.62	2.50	2.43	2.29	2.16	2.05	1.95	1.86	1.77	1.70	1.62	1.56	1.50	1.44	1.39	1.35	1.32	1.28	1.25	1.22	1.20	1.19	1.18	1.18	1.17	1.17	1.17	1.17	1.17	1.17	1.17	
3,000	3.76	3.62	3.49	3.37	3.27	3.17	3.08	3.00	2.85	2.67	2.50	2.35	2.22	2.11	2.01	1.91	1.83	1.75	1.68	1.61	1.55	1.49	1.45	1.41	1.37	1.34	1.31	1.28	1.26	1.24	1.23	1.22	1.21	1.21	1.21	1.21	1.21	1.21	1.21	
3,100	4.03	3.86	3.72	3.59	3.47	3.36	3.26	3.15	3.04	2.90	2.74	2.56	2.41	2.29	2.17	2.07	1.97	1.88	1.80	1.73	1.66	1.60	1.55	1.51	1.47	1.43	1.39	1.36	1.33	1.31	1.29	1.28	1.28	1.28	1.28	1.28	1.28	1.28	1.28	
3,200	4.32	4.13	3.96	3.81	3.68	3.56	3.44	3.32	3.20	3.08	2.92	2.80	2.62	2.48	2.34	2.																								

Table 5 Wild Rice River Control Structure (WRRCS) Gate Opening (ft) - Part D

Flow through WRRCS (cfs)	Staging Area Elevation at Wild Rice River, Z _{STAGING-AREA-WRR} (ft)													
	922.6	922.8	923.0	923.2	923.4	923.6	923.8	924.0	924.2	924.4	924.6	924.8	925.0	
0	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
100	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
200	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
300	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
400	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
500	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
600	0.21	0.21	0.21	0.21	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
700	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.23	0.23	0.23	0.23
800	0.28	0.28	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
900	0.31	0.31	0.31	0.31	0.31	0.31	0.31	0.30	0.30	0.30	0.30	0.30	0.30	0.30
1,000	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.33
1,100	0.38	0.38	0.38	0.38	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37
1,200	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.40	0.40	0.40	0.40	0.40	0.40	0.40
1,300	0.45	0.45	0.45	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.44	0.43	0.43	0.43
1,400	0.48	0.48	0.48	0.48	0.48	0.48	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47
1,500	0.52	0.52	0.51	0.51	0.51	0.51	0.51	0.50	0.50	0.50	0.50	0.50	0.50	0.50
1,600	0.55	0.55	0.55	0.55	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.53	0.53	0.53
1,700	0.59	0.58	0.58	0.58	0.58	0.58	0.58	0.57	0.57	0.57	0.57	0.57	0.57	0.56
1,800	0.62	0.62	0.62	0.62	0.61	0.61	0.61	0.61	0.61	0.61	0.60	0.60	0.60	0.60
1,900	0.66	0.65	0.65	0.65	0.65	0.65	0.64	0.64	0.64	0.64	0.64	0.63	0.63	0.63
2,000	0.69	0.69	0.69	0.68	0.68	0.68	0.68	0.68	0.67	0.67	0.67	0.67	0.67	0.66
2,100	0.72	0.72	0.72	0.72	0.72	0.71	0.71	0.71	0.71	0.70	0.70	0.70	0.70	0.70
2,200	0.76	0.76	0.75	0.75	0.75	0.75	0.74	0.74	0.74	0.74	0.74	0.73	0.73	0.73
2,300	0.79	0.79	0.79	0.79	0.78	0.78	0.78	0.78	0.77	0.77	0.77	0.77	0.77	0.76
2,400	0.83	0.83	0.82	0.82	0.82	0.81	0.81	0.81	0.81	0.80	0.80	0.80	0.80	0.80
2,500	0.86	0.86	0.86	0.85	0.85	0.85	0.85	0.84	0.84	0.84	0.84	0.83	0.83	0.83
2,600	0.90	0.89	0.89	0.89	0.89	0.88	0.88	0.88	0.87	0.87	0.87	0.87	0.87	0.86
2,700	0.93	0.93	0.93	0.92	0.92	0.92	0.91	0.91	0.91	0.91	0.90	0.90	0.90	0.90
2,800	0.97	0.96	0.96	0.96	0.95	0.95	0.95	0.94	0.94	0.94	0.94	0.93	0.93	0.93
2,900	1.00	1.00	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.97	0.97	0.97	0.96
3,000	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.01	1.01	1.01	1.01	1.00	1.00	1.00
3,100	1.07	1.07	1.06	1.06	1.06	1.05	1.05	1.05	1.04	1.04	1.04	1.04	1.03	1.03
3,200	1.10	1.10	1.10	1.09	1.09	1.09	1.08	1.08	1.08	1.07	1.07	1.07	1.07	1.06
3,300	1.14	1.13	1.13	1.13	1.12	1.12	1.12	1.11	1.11	1.11	1.10	1.10	1.10	1.10
3,400	1.17	1.17	1.17	1.16	1.16	1.15	1.15	1.15	1.14	1.14	1.14	1.13	1.13	1.13
3,500	1.21	1.20	1.20	1.20	1.19	1.19	1.18	1.18	1.18	1.17	1.17	1.17	1.17	1.16
3,600	1.24	1.24	1.23	1.23	1.23	1.22	1.22	1.21	1.21	1.21	1.20	1.20	1.20	1.20
3,700	1.28	1.27	1.27	1.26	1.26	1.26	1.25	1.25	1.24	1.24	1.24	1.23	1.23	1.23
3,800	1.31	1.31	1.30	1.30	1.29	1.29	1.29	1.28	1.28	1.27	1.27	1.27	1.26	1.26
3,900	1.35	1.34	1.34	1.33	1.33	1.32	1.32	1.31	1.31	1.30	1.30	1.30	1.30	1.30
4,000	1.38	1.38	1.37	1.37	1.36	1.36	1.35	1.35	1.34	1.34	1.34	1.33	1.33	1.33
4,100	1.41	1.41	1.41	1.40	1.40	1.39	1.39	1.38	1.38	1.37	1.37	1.37	1.36	1.36
4,200	1.45	1.44	1.44	1.43	1.43	1.43	1.42	1.42	1.41	1.41	1.40	1.40	1.40	1.40
4,300	1.48	1.48	1.47	1.47	1.46	1.46	1.45	1.45	1.44	1.44	1.44	1.43	1.43	1.43
4,400	1.52	1.51	1.51	1.50	1.50	1.49	1.49	1.48	1.48	1.48	1.47	1.47	1.46	1.46
4,500	1.55	1.55	1.54	1.54	1.53	1.53	1.52	1.52	1.51	1.51	1.50	1.50	1.50	1.50
4,600	1.59	1.58	1.58	1.57	1.57	1.56	1.56	1.55	1.55	1.54	1.54	1.53	1.53	1.53
4,700	1.62	1.62	1.61	1.61	1.60	1.60	1.59	1.59	1.58	1.58	1.57	1.57	1.56	1.56
4,800	1.66	1.65	1.64	1.64	1.63	1.63	1.62	1.62	1.61	1.61	1.60	1.60	1.59	1.59
4,900	1.69	1.68	1.68	1.67	1.67	1.66	1.66	1.65	1.64	1.64	1.63	1.63	1.63	1.63
5,000	1.72	1.72	1.71	1.71	1.70	1.70	1.69	1.68	1.68	1.67	1.67	1.66	1.66	1.66
5,100	1.79	1.79	1.78	1.78	1.77	1.77	1.76	1.75	1.74	1.74	1.73	1.73	1.73	1.73
5,200	1.86	1.86	1.85	1.84	1.84	1.83	1.83	1.82	1.81	1.80	1.80	1.80	1.79	1.79
5,300	1.93	1.92	1.92	1.91	1.91	1.90	1.89	1.89	1.88	1.88	1.87	1.87	1.86	1.86
5,400	2.00	1.99	1.99	1.98	1.97	1.97	1.96	1.96	1.95	1.94	1.94	1.93	1.93	1.93
5,500	2.07	2.06	2.06	2.05	2.04	2.04	2.03	2.02	2.02	2.01	2.01	2.00	2.00	1.99
5,600	2.14	2.13	2.12	2.12	2.11	2.10	2.10	2.09	2.08	2.08	2.07	2.07	2.06	2.06
5,700	2.21	2.20	2.19	2.19	2.18	2.17	2.16	2.16	2.15	2.15	2.14	2.13	2.13	2.13
5,800	2.28	2.27	2.26	2.25	2.25	2.24	2.23	2.22	2.22	2.21	2.21	2.20	2.19	2.19
5,900	2.34	2.34	2.33	2.32	2.31	2.31	2.30	2.29	2.29	2.28	2.27	2.27	2.26	2.26
6,000	2.41	2.41	2.40	2.39	2.38	2.37	2.37	2.36	2.35	2.35	2.34	2.33	2.32	2.32
6,100	2.48	2.47	2.47	2.46	2.45	2.44	2.44	2.43	2.42	2.42	2.41	2.40	2.39	2.39
6,200	2.55	2.54	2.53	2.53	2.52	2.51	2.50	2.49	2.48	2.48	2.47	2.46	2.46	2.46
6,300	2.63	2.62	2.60	2.59	2.59	2.58	2.57	2.56	2.55	2.55	2.54	2.53	2.52	2.52
6,400	2.76	2.72	2.69	2.67	2.65	2.65	2.64	2.63	2.62	2.61	2.61	2.60	2.59	2.59
6,500	2.90	2.84	2.78	2.75	2.72	2.71	2.71	2.70	2.69	2.68	2.67	2.66	2.66	2.66
6,600	3.04	2.98	2.91	2.85	2.79	2.78	2.77	2.76	2.75	2.74	2.73	2.72	2.72	2.72
6,700	3.18	3.12	3.05	2.98	2.91	2.87	2.85	2.83	2.82	2.81	2.81	2.80	2.79	2.79
6,800	3.33	3.26	3.18	3.11	3.04	2.98	2.94	2.91	2.89	2.88	2.87	2.86	2.85	2.85
6,900	3.48	3.41	3.33	3.25	3.18	3.11	3.04	2.98	2.96	2.95	2.94	2.93	2.92	2.92
7,000	3.64	3.57	3.49	3.40	3.33	3.25	3.18	3.11	3.05	3.02	3.01	3.00	2.99	2.99
7,100	3.82	3.74	3.66	3.57	3.48	3.40	3.32	3.25	3.18	3.13	3.10	3.07	3.05	3.05
7,200	4.01	3.92	3.83	3.73	3.65	3.56	3.48	3.39	3.32	3.25	3.19	3.15	3.12	3.12
7,300	4.19	4.11	4.00	3.91	3.81	3.72	3.63	3.55	3.46	3.39	3.32	3.27	3.19	3.19
7,400	4.38	4.29	4.19	4.08	3.98	3.88	3.79	3.70	3.61	3.53	3.45	3.38	3.31	3.31
7,500	4.58	4.48	4.37	4.26	4.15	4.05	3.95	3.85	3.76	3.68	3.59	3.52	3.44	3.44
7,600	4.78	4.67	4.55	4.43	4.32	4.21	4.11	4.00	3.91	3.82	3.74	3.65	3.57	3.57
7,700	4.98	4.86	4.74	4.62	4.50	4.38	4.27	4.17	4.07	3.97	3.88	3.79	3.71	3.71
7,800	5.17	5.06	4.93	4.79	4.67	4.55	4.43	4.32	4.22	4.12	4.02	3.93	3.84	3.84
7,900	5.37	5.25	5.10	4.96	4.83	4.70	4.58	4.47	4.36	4.25	4.15	4.06	3.97	3.97
8,000	5.56	5.42	5.27	5.13	4.99	4.86	4.73	4.61	4.50	4.39	4.28	4.18	4.09	4.09
8,100	5.74	5.59	5.44	5.29	5.14	5.01	4.87	4.75	4.63	4.52	4.41	4.31	4.21	4.21
8,200	5.91	5.76	5.60	5.44	5.29	5.15	5.01	4.88	4.76	4.64	4.53	4.42	4.32	4.32
8,300	6.08	5.92	5.75	5.58	5.43	5.28	5.14	5.01	4.88	4.76	4.65	4.54	4.43	4.43
8,400	6.24	6.07	5.89	5.72	5.57	5.41	5.27	5.13	5.00	4.88	4.76	4.65	4.54	4.54
8,500	6.40	6.22	6.04	5.86	5.70	5.54	5.39	5.25	5.12	4.99	4.87			

Table 7 Diversion Inlet Control Structure (DICS) Gate Opening (ft) - Part E

Flow through DICS (cfs)	Staging Area Elevation at Wild Rice River, Z _{STAGING-AREA-WRR} (ft)												
	923.8	923.9	924	924.1	924.2	924.3	924.4	924.5	924.6	924.7	924.8	924.9	925
0	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
100	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
200	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
300	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
400	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
500	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.20	0.20
600	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
700	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29
800	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
900	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37	0.37
1,000	0.42	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41
1,100	0.46	0.46	0.46	0.46	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
1,200	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.49	0.49	0.49	0.49	0.49	0.49
1,300	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.53	0.53	0.53	0.53	0.53
1,400	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.57	0.57	0.57	0.57
1,500	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.61	0.61
1,600	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.65
1,700	0.71	0.71	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
1,800	0.75	0.75	0.75	0.75	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74	0.74
1,900	0.79	0.79	0.79	0.79	0.79	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78
2,000	0.83	0.83	0.83	0.83	0.83	0.83	0.82	0.82	0.82	0.82	0.82	0.82	0.82
2,100	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.86	0.86	0.86	0.86	0.86	0.86
2,200	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.90	0.90	0.90	0.90	0.90
2,300	0.96	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.94	0.94	0.94	0.94
2,400	1.00	1.00	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.98
2,500	1.04	1.04	1.04	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.02	1.02
2,600	1.08	1.08	1.08	1.08	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.06	1.06
2,700	1.12	1.12	1.12	1.12	1.12	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.10
2,800	1.16	1.16	1.16	1.16	1.16	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15
2,900	1.21	1.20	1.20	1.20	1.20	1.20	1.20	1.19	1.19	1.19	1.19	1.19	1.19
3,000	1.25	1.25	1.24	1.24	1.24	1.24	1.24	1.24	1.23	1.23	1.23	1.23	1.23
3,100	1.29	1.29	1.28	1.28	1.28	1.28	1.28	1.28	1.27	1.27	1.27	1.27	1.27
3,200	1.33	1.33	1.33	1.32	1.32	1.32	1.32	1.32	1.31	1.31	1.31	1.31	1.31
3,300	1.37	1.37	1.37	1.37	1.36	1.36	1.36	1.36	1.36	1.35	1.35	1.35	1.35
3,400	1.41	1.41	1.41	1.41	1.41	1.40	1.40	1.40	1.40	1.39	1.39	1.39	1.39
3,500	1.45	1.45	1.45	1.45	1.45	1.44	1.44	1.44	1.44	1.44	1.44	1.43	1.43
3,600	1.50	1.49	1.49	1.49	1.49	1.49	1.48	1.48	1.48	1.48	1.48	1.47	1.47
3,700	1.54	1.54	1.53	1.53	1.53	1.53	1.53	1.52	1.52	1.52	1.52	1.52	1.51
3,800	1.58	1.58	1.58	1.57	1.57	1.57	1.57	1.56	1.56	1.56	1.56	1.56	1.55
3,900	1.62	1.62	1.62	1.61	1.61	1.61	1.61	1.61	1.60	1.60	1.60	1.60	1.60
4,000	1.66	1.66	1.66	1.66	1.65	1.65	1.65	1.65	1.64	1.64	1.64	1.64	1.64
4,100	1.70	1.70	1.70	1.70	1.69	1.69	1.69	1.69	1.69	1.68	1.68	1.68	1.68
4,200	1.75	1.74	1.74	1.74	1.74	1.73	1.73	1.73	1.73	1.72	1.72	1.72	1.72
4,300	1.79	1.78	1.78	1.78	1.78	1.78	1.77	1.77	1.77	1.77	1.76	1.76	1.76
4,400	1.83	1.83	1.82	1.82	1.82	1.82	1.81	1.81	1.81	1.81	1.80	1.80	1.80
4,500	1.87	1.87	1.87	1.86	1.86	1.86	1.86	1.85	1.85	1.85	1.85	1.84	1.84
4,600	1.91	1.91	1.91	1.90	1.90	1.90	1.90	1.89	1.89	1.89	1.89	1.88	1.88
4,700	1.95	1.95	1.95	1.95	1.94	1.94	1.94	1.94	1.93	1.93	1.93	1.93	1.92
4,800	2.00	1.99	1.99	1.99	1.98	1.98	1.98	1.98	1.97	1.97	1.97	1.97	1.96
4,900	2.04	2.03	2.03	2.03	2.03	2.02	2.02	2.02	2.02	2.01	2.01	2.01	2.00
5,000	2.08	2.08	2.07	2.07	2.07	2.06	2.06	2.06	2.06	2.05	2.05	2.05	2.05
5,200	2.16	2.16	2.16	2.15	2.15	2.15	2.14	2.14	2.14	2.14	2.13	2.13	2.13
5,400	2.24	2.24	2.24	2.24	2.23	2.23	2.23	2.22	2.22	2.22	2.21	2.21	2.21
5,600	2.33	2.32	2.32	2.32	2.32	2.31	2.31	2.31	2.30	2.30	2.29	2.29	2.29
5,800	2.41	2.41	2.40	2.40	2.40	2.39	2.39	2.39	2.39	2.38	2.38	2.38	2.37
6,000	2.49	2.49	2.49	2.48	2.48	2.48	2.47	2.47	2.47	2.46	2.46	2.46	2.45
6,200	2.58	2.57	2.57	2.57	2.56	2.56	2.56	2.55	2.55	2.55	2.54	2.54	2.54
6,400	2.66	2.66	2.65	2.65	2.65	2.64	2.64	2.63	2.63	2.63	2.62	2.62	2.62
6,600	2.74	2.74	2.74	2.73	2.73	2.73	2.72	2.72	2.71	2.71	2.70	2.70	2.70
6,800	2.83	2.82	2.82	2.82	2.81	2.81	2.80	2.80	2.80	2.79	2.79	2.79	2.78
7,000	2.91	2.91	2.90	2.90	2.89	2.89	2.89	2.88	2.88	2.88	2.87	2.87	2.86
7,200	2.99	2.99	2.99	2.98	2.98	2.97	2.97	2.96	2.96	2.95	2.95	2.95	2.95
7,400	3.08	3.07	3.07	3.06	3.06	3.06	3.05	3.05	3.04	3.04	3.03	3.03	3.03
7,600	3.16	3.16	3.15	3.15	3.14	3.14	3.13	3.13	3.13	3.12	3.12	3.11	3.11
7,800	3.24	3.24	3.23	3.23	3.23	3.22	3.22	3.21	3.21	3.20	3.20	3.20	3.19
8,000	3.33	3.32	3.32	3.31	3.31	3.30	3.30	3.29	3.29	3.29	3.28	3.28	3.27
8,200	3.41	3.40	3.40	3.40	3.39	3.39	3.38	3.38	3.37	3.37	3.36	3.36	3.36
8,400	3.49	3.49	3.48	3.48	3.47	3.47	3.46	3.46	3.46	3.45	3.45	3.44	3.44
8,600	3.58	3.57	3.57	3.56	3.56	3.55	3.55	3.54	3.54	3.53	3.53	3.52	3.52
8,800	3.66	3.65	3.65	3.64	3.64	3.63	3.63	3.62	3.62	3.61	3.61	3.61	3.60
9,000	3.74	3.74	3.73	3.73	3.72	3.72	3.71	3.71	3.70	3.70	3.69	3.69	3.68
9,200	3.83	3.82	3.81	3.81	3.80	3.80	3.79	3.79	3.78	3.78	3.77	3.77	3.76
9,400	3.91	3.90	3.90	3.89	3.89	3.88	3.88	3.87	3.87	3.86	3.86	3.85	3.85
9,600	3.99	3.99	3.98	3.98	3.97	3.96	3.96	3.95	3.95	3.94	3.94	3.93	3.93
9,800	4.08	4.07	4.06	4.06	4.05	4.05	4.04	4.03	4.03	4.02	4.02	4.01	4.01
10,000	4.16	4.15	4.15	4.14	4.14	4.13	4.12	4.12	4.11	4.11	4.10	4.10	4.09
10,200	4.24	4.24	4.23	4.22	4.22	4.21	4.21	4.20	4.20	4.19	4.18	4.18	4.17
10,400	4.32	4.32	4.31	4.31	4.30	4.30	4.29	4.28	4.28	4.27	4.27	4.26	4.26
10,600	4.41	4.40	4.40	4.39	4.38	4.38	4.37	4.37	4.36	4.35	4.35	4.34	4.34
10,800	4.49	4.49	4.48	4.47	4.47	4.46	4.45	4.45	4.44	4.44	4.43	4.43	4.42
11,000	4.57	4.57	4.56	4.56	4.55	4.54	4.54	4.53	4.53	4.52	4.51	4.51	4.50
11,200	4.66	4.65	4.64	4.64	4.63	4.63	4.62	4.61	4.61	4.60	4.60	4.59	4.58
11,400	4.74	4.73	4.73	4.72	4.72	4.71	4.70	4.70	4.69	4.68	4.68	4.67	4.67
11,600	4.82	4.82	4.81	4.80	4.80	4.79	4.78	4.78	4.77	4.77	4.76	4.75	4.75
11,800	4.91	4.90	4.89	4.89	4.88	4.88	4.87	4.87	4.86	4.85	4.84	4.84	4.83
12,000	4.99	4.98	4.98	4.97	4.96	4.96	4.95	4.94	4.94	4.93	4.92	4.92	4.91
12,200	5.07	5.07	5.06	5.05	5.05	5.04	5.03	5.03	5.02	5.01	5.01	5.00	4.99
12,400	5.16	5.15	5.14	5.14	5.13	5.12	5.12	5.11	5.10	5.09	5.09	5.08	5.07
12,600	5.24	5.23	5.23	5.22	5.21	5.20	5.20	5.19	5.18	5.18	5.17	5.16	5.16
12,800	5.32	5.32	5.31	5.30	5.29	5.29	5.28	5.27	5.27	5.26	5.25	5.25	5.24
13,000	5.41	5.40	5.39	5.38	5.38	5.37	5.36	5.36	5.35	5.34	5.33	5.33	5.32
13,200	5.49	5.48	5.48	5.47	5.46	5.45							

Table 8 The Relationship between Allowable Flow through the Town and the Maximum Flow from Combined Enloe and Abercrombie

Flood Event	Maximum flow from combined Enloe and Abercrombie, Q_{AE-MAX} (cfs)	Allowable flow through the town, Q_{TOWN} (cfs)
0 to 100-year	0	18,000
	38,000	18,000
100-year to 500-year	39,000	18,000
	40,000	18,255
	41,000	18,892
	42,000	19,529
	43,000	20,167
	44,000	20,804
	45,000	21,441
	46,000	22,078
	47,000	22,716
	48,000	23,353
	49,000	23,990
	50,000	24,627
60,000	31,000	
500-year to Inflow Design Flood	102,000	31,000
Greater than Inflow Design Flood	102,001	200,000

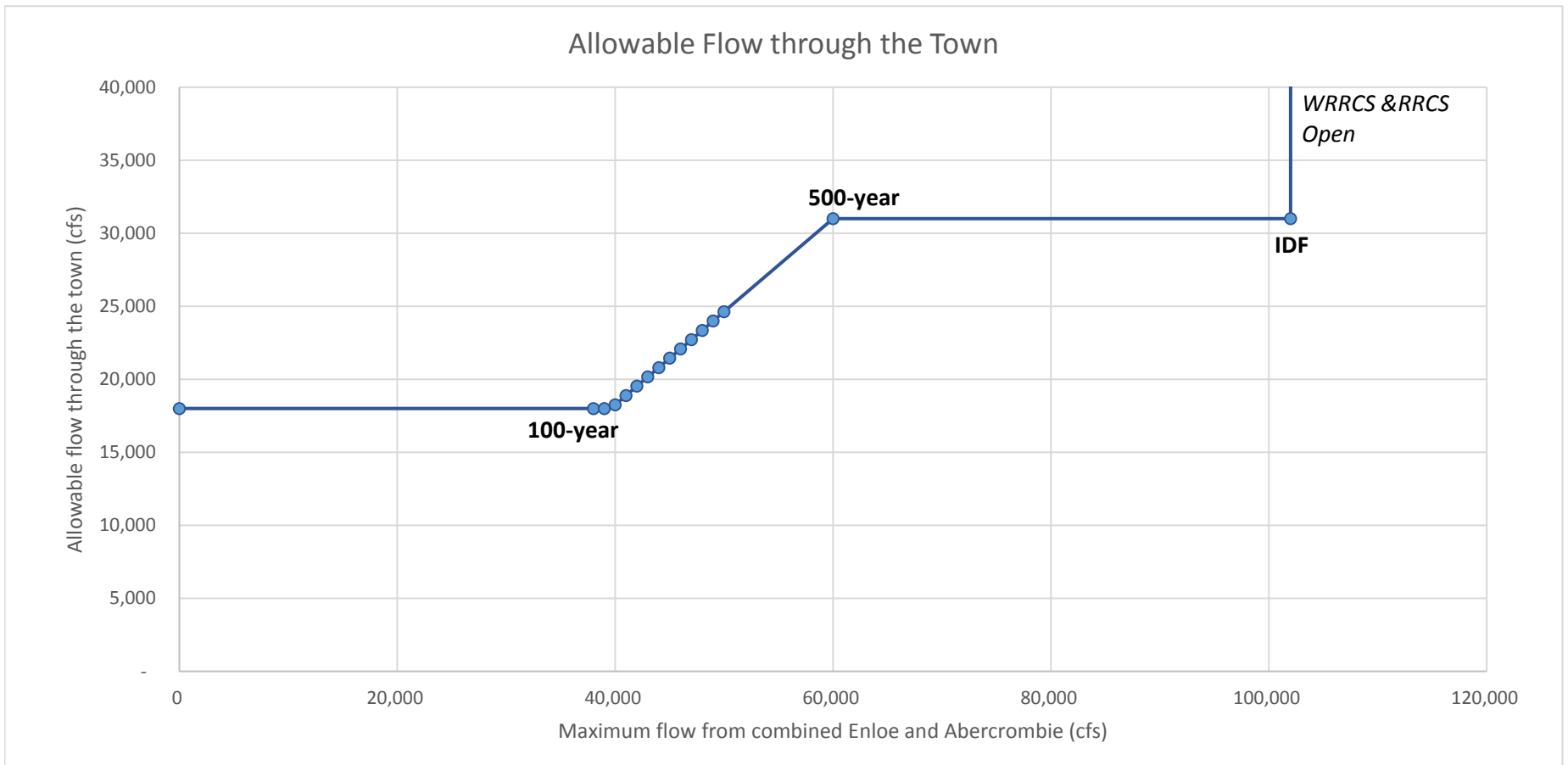


Table 9 Coefficient b [-] for the Nonlinear Reservoir Model (Transition segment)

Maximum flow from combined Enloe and Abercrombie Q^t_{AE-MAX} (cfs)	Maximum flow reduction, $Q^t_{REDUCTION-MAX}$ (cfs)																	
	0	500	1,000	1,500	2,000	2,500	3,000	3,500	4,000	5,000	6,000	7,000	8,000	9,000	10,000	15,000	20,000	50,000
20,000	1.0117	1.0119	1.0120	1.0124	1.0127	1.0130	1.0133	1.0136	1.0140	1.0144	1.0148	1.0152	1.0157	1.0161	1.0165	1.0172	1.0184	1.0204
21,000	1.0116	1.0118	1.0119	1.0122	1.0126	1.0129	1.0132	1.0135	1.0138	1.0142	1.0147	1.0151	1.0155	1.0159	1.0164	1.0170	1.0183	1.0202
22,000	1.0115	1.0117	1.0118	1.0121	1.0124	1.0127	1.0131	1.0134	1.0137	1.0141	1.0145	1.0149	1.0154	1.0158	1.0162	1.0168	1.0181	1.0200
23,000	1.0114	1.0115	1.0117	1.0120	1.0123	1.0126	1.0129	1.0132	1.0136	1.0140	1.0144	1.0148	1.0152	1.0156	1.0160	1.0167	1.0179	1.0198
24,000	1.0113	1.0114	1.0116	1.0119	1.0122	1.0125	1.0128	1.0131	1.0134	1.0138	1.0142	1.0146	1.0151	1.0155	1.0159	1.0165	1.0177	1.0196
25,000	1.0112	1.0113	1.0115	1.0118	1.0121	1.0124	1.0127	1.0130	1.0133	1.0137	1.0141	1.0145	1.0149	1.0153	1.0157	1.0163	1.0175	1.0194
26,000	1.0111	1.0112	1.0113	1.0116	1.0119	1.0122	1.0125	1.0129	1.0132	1.0136	1.0140	1.0144	1.0148	1.0152	1.0156	1.0162	1.0174	1.0192
27,000	1.0109	1.0111	1.0112	1.0115	1.0118	1.0121	1.0124	1.0127	1.0130	1.0134	1.0138	1.0142	1.0146	1.0150	1.0154	1.0160	1.0172	1.0190
28,000	1.0108	1.0110	1.0111	1.0114	1.0117	1.0120	1.0123	1.0126	1.0129	1.0133	1.0137	1.0141	1.0145	1.0148	1.0152	1.0158	1.0170	1.0188
29,000	1.0107	1.0109	1.0110	1.0113	1.0116	1.0119	1.0122	1.0125	1.0127	1.0131	1.0135	1.0139	1.0143	1.0147	1.0151	1.0157	1.0168	1.0186
30,000	1.0106	1.0107	1.0109	1.0112	1.0115	1.0118	1.0120	1.0123	1.0126	1.0130	1.0134	1.0138	1.0142	1.0145	1.0149	1.0155	1.0167	1.0184
31,000	1.0104	1.0106	1.0107	1.0110	1.0113	1.0116	1.0118	1.0121	1.0124	1.0128	1.0132	1.0136	1.0139	1.0143	1.0147	1.0153	1.0164	1.0181
32,000	1.0103	1.0104	1.0105	1.0108	1.0111	1.0114	1.0117	1.0119	1.0122	1.0126	1.0130	1.0133	1.0137	1.0141	1.0145	1.0150	1.0161	1.0178
33,000	1.0101	1.0102	1.0104	1.0106	1.0109	1.0112	1.0115	1.0117	1.0120	1.0124	1.0127	1.0131	1.0135	1.0138	1.0142	1.0148	1.0159	1.0175
34,000	1.0099	1.0101	1.0102	1.0105	1.0107	1.0110	1.0113	1.0115	1.0118	1.0122	1.0125	1.0129	1.0133	1.0136	1.0140	1.0145	1.0156	1.0172
35,000	1.0098	1.0099	1.0100	1.0103	1.0106	1.0108	1.0111	1.0113	1.0116	1.0120	1.0123	1.0127	1.0130	1.0134	1.0137	1.0143	1.0153	1.0169
36,000	1.0096	1.0097	1.0099	1.0101	1.0104	1.0106	1.0109	1.0112	1.0114	1.0118	1.0121	1.0125	1.0128	1.0132	1.0135	1.0140	1.0151	1.0166
37,000	1.0094	1.0096	1.0097	1.0099	1.0102	1.0104	1.0107	1.0110	1.0112	1.0116	1.0119	1.0122	1.0126	1.0129	1.0133	1.0138	1.0148	1.0164
38,000	1.0093	1.0094	1.0095	1.0098	1.0100	1.0103	1.0105	1.0108	1.0110	1.0114	1.0117	1.0120	1.0124	1.0127	1.0130	1.0135	1.0145	1.0161
39,000	1.0091	1.0092	1.0093	1.0096	1.0098	1.0101	1.0103	1.0106	1.0108	1.0111	1.0115	1.0118	1.0121	1.0125	1.0128	1.0133	1.0143	1.0158
40,000	1.0089	1.0090	1.0092	1.0094	1.0096	1.0099	1.0101	1.0104	1.0106	1.0109	1.0113	1.0116	1.0119	1.0122	1.0126	1.0130	1.0140	1.0155
41,000	1.0087	1.0089	1.0090	1.0092	1.0094	1.0097	1.0099	1.0102	1.0104	1.0107	1.0110	1.0113	1.0117	1.0120	1.0123	1.0128	1.0137	1.0151
42,000	1.0086	1.0087	1.0088	1.0090	1.0092	1.0095	1.0097	1.0099	1.0102	1.0105	1.0108	1.0111	1.0114	1.0117	1.0120	1.0125	1.0134	1.0148
43,000	1.0084	1.0085	1.0086	1.0088	1.0090	1.0093	1.0095	1.0097	1.0099	1.0103	1.0106	1.0109	1.0112	1.0115	1.0118	1.0122	1.0131	1.0145
44,000	1.0092	1.0093	1.0094	1.0097	1.0099	1.0102	1.0104	1.0107	1.0109	1.0113	1.0116	1.0120	1.0123	1.0126	1.0130	1.0135	1.0145	1.0160
45,000	1.0100	1.0102	1.0103	1.0106	1.0109	1.0111	1.0114	1.0117	1.0120	1.0123	1.0127	1.0130	1.0134	1.0138	1.0141	1.0147	1.0158	1.0174
46,000	1.0109	1.0110	1.0112	1.0115	1.0118	1.0121	1.0124	1.0127	1.0130	1.0133	1.0137	1.0141	1.0145	1.0149	1.0153	1.0159	1.0171	1.0189
47,000	1.0117	1.0119	1.0120	1.0124	1.0127	1.0130	1.0133	1.0136	1.0140	1.0144	1.0148	1.0152	1.0157	1.0161	1.0165	1.0172	1.0184	1.0204
48,000	1.0126	1.0127	1.0129	1.0132	1.0136	1.0139	1.0143	1.0146	1.0150	1.0154	1.0159	1.0163	1.0168	1.0172	1.0177	1.0184	1.0198	1.0218
49,000	1.0145	1.0147	1.0149	1.0153	1.0157	1.0161	1.0165	1.0169	1.0172	1.0178	1.0183	1.0188	1.0194	1.0199	1.0204	1.0212	1.0228	1.0252
50,000	1.0164	1.0166	1.0168	1.0173	1.0177	1.0182	1.0186	1.0191	1.0195	1.0201	1.0207	1.0213	1.0219	1.0225	1.0231	1.0240	1.0258	1.0286
51,000	1.0183	1.0186	1.0188	1.0193	1.0198	1.0203	1.0208	1.0213	1.0218	1.0225	1.0232	1.0238	1.0245	1.0252	1.0259	1.0269	1.0289	1.0319
52,000	1.0202	1.0205	1.0208	1.0213	1.0219	1.0224	1.0230	1.0236	1.0241	1.0249	1.0256	1.0263	1.0271	1.0278	1.0286	1.0297	1.0319	1.0353
53,000	1.0222	1.0225	1.0228	1.0234	1.0240	1.0246	1.0252	1.0258	1.0264	1.0272	1.0280	1.0288	1.0297	1.0305	1.0313	1.0325	1.0350	1.0387
54,000	1.0241	1.0244	1.0247	1.0254	1.0260	1.0267	1.0274	1.0280	1.0287	1.0296	1.0305	1.0314	1.0322	1.0331	1.0340	1.0353	1.0380	1.0421
55,000	1.0260	1.0263	1.0267	1.0274	1.0281	1.0288	1.0296	1.0303	1.0310	1.0319	1.0329	1.0339	1.0348	1.0358	1.0367	1.0382	1.0411	1.0454
56,000	1.0279	1.0283	1.0287	1.0294	1.0302	1.0310	1.0317	1.0325	1.0333	1.0343	1.0353	1.0364	1.0374	1.0384	1.0395	1.0410	1.0441	1.0488
57,000	1.0298	1.0302	1.0306	1.0315	1.0323	1.0331	1.0339	1.0347	1.0356	1.0367	1.0378	1.0389	1.0400	1.0411	1.0422	1.0438	1.0472	1.0522
58,000	1.0318	1.0322	1.0326	1.0335	1.0344	1.0352	1.0361	1.0370	1.0378	1.0390	1.0402	1.0414	1.0425	1.0437	1.0449	1.0467	1.0502	1.0555
59,000	1.0337	1.0341	1.0346	1.0355	1.0364	1.0374	1.0383	1.0392	1.0401	1.0414	1.0426	1.0439	1.0451	1.0464	1.0476	1.0495	1.0532	1.0589
60,000	1.0356	1.0361	1.0365	1.0375	1.0385	1.0395	1.0405	1.0414	1.0424	1.0437	1.0451	1.0464	1.0477	1.0490	1.0503	1.0523	1.0563	1.0623
65,000	1.0427	1.0432	1.0438	1.0450	1.0462	1.0474	1.0485	1.0497	1.0509	1.0525	1.0541	1.0557	1.0573	1.0588	1.0604	1.0628	1.0676	1.0749
70,000	1.0498	1.0504	1.0511	1.0525	1.0539	1.0552	1.0566	1.0580	1.0594	1.0613	1.0631	1.0650	1.0668	1.0687	1.0705	1.0734	1.0790	1.0875

Table 11 Designed Time Duration to Allow Maximum Flow through the Town

Maximum flow from combined Enloe and Abercrombie, Q_{AE-MAX} , (cfs)	Time Duration to Allow Maximum Flow through the Town, DT_{QTOWN}^t , (hr)
20,000	144
24,000	168
26,000	192
28,000	240
33,000	276
37,000	288
40,000	306
46,000	324
56,000	384
100,000	400

Table 12 Diversion Inlet Control Structure (DICS) Flow-Stage Relationship (Gate Open)

Staging Area Elevation at Wild Rice Rive, $Z_{\text{STAGING-AREA-WRR}}$ (ft)	Flow at DICS, Q_{DIVCS} (cfs)	Staging Area Elevation at Wild Rice Rive, $Z_{\text{STAGING-AREA-WRR}}$ (ft)	Flow at DICS, Q_{DIVCS} (cfs)
910.0	1,096	916.3	19,134
910.1	1,324	916.4	19,754
910.2	1,551	916.5	20,369
910.3	1,779	916.6	20,879
910.4	2,007	916.7	21,309
910.5	2,234	916.8	21,625
910.6	2,462	916.9	22,067
910.7	2,647	917.0	22,411
910.8	2,763	917.1	22,740
910.9	2,880	917.2	23,074
911.0	2,996	917.3	23,396
911.1	3,112	917.4	23,709
911.2	3,194	917.5	24,022
911.3	3,274	917.6	24,334
911.4	3,353	917.7	24,610
911.5	3,425	917.8	24,869
911.6	3,497	917.9	25,139
911.7	3,570	918.0	25,382
911.8	3,650	918.1	25,615
911.9	3,729	918.2	25,849
912.0	3,809	918.3	26,065
912.1	3,891	918.4	26,239
912.2	3,995	918.5	26,422
912.3	4,099	918.6	26,607
912.4	4,203	918.7	26,795
912.5	4,307	918.8	26,982
912.6	4,411	918.9	27,165
912.7	4,516	919.0	27,353
912.8	4,620	919.1	27,537
912.9	4,724	919.2	27,721
913.0	4,828	919.3	27,903
913.1	5,158	919.4	28,085
913.2	5,496	919.5	28,263
913.3	5,834	919.6	28,441
913.4	6,172	919.7	28,617
913.5	6,510	919.8	28,794
913.6	6,848	919.9	28,972
913.7	7,279	920.0	29,167
913.8	7,769	920.1	29,358
913.9	8,260	920.2	29,570
914.0	8,748	920.3	29,844
914.1	9,236	920.4	30,154
914.2	9,700	920.5	30,485
914.3	10,088	920.6	30,836
914.4	10,407	920.7	31,201
914.5	10,808	920.8	31,580
914.6	11,244	920.9	31,971
914.7	11,616	921.0	32,369
914.8	12,016	921.1	32,776
914.9	12,420	921.2	33,193
915.0	12,817	921.3	33,616
915.1	13,210	921.4	34,049
915.2	13,613	921.5	34,489
915.3	14,024	921.6	34,927
915.4	14,422	921.7	35,362
915.5	14,756	921.8	35,723
915.6	15,194	921.9	36,062
915.7	15,611	922.0	36,404
915.8	16,026	922.1	36,747
915.9	16,543	922.2	37,095
916.0	17,128	922.3	37,443
916.1	17,625	922.4	37,794
916.2	18,476	922.5	38,147