

Trowbridge Dam Removal and River Stabilization Design Summary, Part 2

OU5 Area 4 Time-Critical Removal Action
Allied Paper/Portage Creek/
Kalamazoo River Superfund Site





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Prepared for:
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

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- Attachment A. Area 4 TCRA Hydrology, Hydraulics, and Sediment Transport
Modeling Technical Memorandum
- Attachment B. BSTEM Calibration and Application Technical Memorandum

Abbreviations and Acronyms

1-D	one-dimensional
2-D	two-dimensional
BSTEM	Bank Stability and Toe Erosion Model
CD	Consent Decree
cfs	cubic feet per second
CSM	conceptual site model
CY	cubic yards
DCP	dynamic cone penetrometer
EGLE	Michigan Department of Environment, Great Lakes, and Energy
El.	elevation
EPA	United States Environmental Protection Agency
FSP	Field Sampling Plan
H&H	hydrologic and hydraulic
HEC-RAS	Hydrologic Engineering Center's River Analysis System
LDB	left descending bank
LLC	Kalamazoo River Areas 2, 3 and 4 Remediation LLC
MDNR	Michigan Department of Natural Resources
OU5	Operable Unit 5
PCB	polychlorinated biphenyl
PDI	pre-design investigation
QAPP	Quality Assurance Project Plan
RDB	right descending bank
RM	river mile
SOP	standard operating procedure
SRI	supplemental remedial investigation
SSP	steel sheet piling
STAAD	3D Structural Analysis and Design program
TCRA	time-critical removal action
USGS	United States Geological Survey
WCS	water control structure
WSE	water surface elevation

1. Introduction

GEI Consultants of Michigan, P.C. is pleased to provide this *Trowbridge Dam Removal and River Stabilization Design Summary* for Area 4 of Operable Unit 5 (OU5) of the Allied Paper/Portage Creek/Kalamazoo River Superfund Site (Site; Figs. 1 and 2). This work was performed by GEI on behalf of the Kalamazoo River Areas 2, 3 and 4 Remediation LLC (LLC), which NCR Corporation (now NCR Voyix Corporation) formed to meet certain requirements of the Consent Decree (CD) between NCR, the United States Environmental Protection Agency (EPA), and the state of Michigan.

The CD was lodged in December 2019 and entered (approved) by the United States District Court for the Western District of Michigan on December 2, 2020. The CD requires NCR to conduct response activities in Areas 2, 3, and 4 of OU5 of the Allied Paper/Portage Creek/Kalamazoo River Superfund Site. Work activities mandated by the CD to address polychlorinated biphenyls (PCBs) in soil and sediment in OU5 include performance of a time-critical removal action (TCRA) in Area 4 and performance of remedial design and remedial action in Areas 2 and 3.

As described in the TCRA Memorandum dated April 1, 2020 (EPA, 2020), the LLC is implementing a TCRA to address elevated levels of total PCBs in bank soils and channel sediment in a 2.4-mile stretch of the Kalamazoo River. The stretch spans from river mile (RM) 47.25 to the Trowbridge Dam, which serves as the downstream Area 4 boundary at RM 44.9. In addition to PCB removal, the TCRA requires removal of the Trowbridge Dam, stabilization of impacted banks, and ensuring a stable channel exists post-removal. As detailed in the EPA letter dated August 4, 2023 (EPA, 2023) and GEI's Removal Work Plan Part 1, Draft 4 (GEI, 2024b), a subset scope of work to the overall TCRA (referred to herein as Part 1 TCRA) was previously defined. The remaining scope of work described in the TCRA Memorandum necessary to complete the TCRA (referred to herein as Part 2 TCRA) is described in GEI's Removal Work Plan Part 2 (GEI, 2024c).

1.1 Purpose

The purpose of this document is to present a summary of the following information and analyses that have been used to inform the overall design as it is outlined in the Removal Work Plan Part 2.

- The data collection and modeling tools used to inform the river stabilization design (Section 3).

- The dam removal steps (Section 4).
- The river channel alignment design, characteristics, and associated analyses (Section 5).
- The bank stabilization design and associated analyses (Section 6).

The design work completed as part of this dam removal and river stabilization was conducted in accordance with the Area 4 Pre-Design Investigation Field Sampling Plan (FSP; GEI 2020), the Multi-Area Quality Assurance Project Plan, Revision 2 (QAPP; GEI, 2022a), the task-specific addendum to the QAPP (Appendix A of the Area 4 Pre-Design Investigation [PDI] FSP [GEI, 2020]), applicable standard operating procedures (SOPs; Appendix B of the Area 4 PDI FSP [GEI, 2020]), the PDI FSP Addendum (GEI, 2021a), the OU5 Area 4 PDI Addendum (GEI, 2021b), the program-wide Health and Safety Plan (GEI, 2024a), and in response to agency comments, unless otherwise indicated in this report. This design summary has been amended since the 2022 submission based on discussions with EPA on June 5, 2024 to include targeted channel widening and targeted riffle crest lowering in Subareas C and D. The revised scope is reflected in the design and work plans.

2. Site Background

2.1 TCRA Overview

A site description and conceptual site model are presented in the Removal Work Plan Part 2 (GEI, 2024c) and the PDI Data Summary Report (GEI, 2022b).

The TCRA for OU5 Area 4 focuses on riverbank soil removal within Subareas C, D, and E as well as instream sediment removal in Subareas E, F, and G. The TCRA also requires the removal of Trowbridge Dam (downstream end of Subarea E), which includes the remnant powerhouse, spillway, and left embankment.

The project approach includes stabilization of the channel within Subareas C, D, E, and G and stabilization of tributaries to the main channel as appropriate. The existing river alignment will largely be maintained, except for a deviation through lower Subarea G, as is explained in Sections 3 and 5. Results of sediment poling and coring identified this alignment as the historical thalweg prior to construction of the Trowbridge Dam. A pilot channel will be constructed to achieve this configuration and facilitate the initial phase of bank remediation and channel construction.

Following water level lowering via the water control structure and ultimate removal of the dam, river waterlevels in areas E, F, and G are expected to be lower, resulting in portions of subareas F and G that are routinely inundated under current conditions to drain and revert to upland floodplain. These areas will no longer be included in the main river channel but instead will consist of upland floodplain.

2.2 Trowbridge Dam Overview

The Trowbridge Dam was built in 1898 and operated from 1902 to 1967 to generate hydroelectric power. The dam is currently managed by Michigan Department of Natural Resources (MDNR) for the people of the State of Michigan. The powerhouse associated with the dam was demolished and the gates at the dam were opened to lower the impoundment level in the early 1970s. In 1986, the remainder of the dam's superstructure and spillway gates were removed, and its concrete piers and abutment walls were lowered. A timeline of the dam history is provided as Fig. 3.

Because the Trowbridge Dam was listed as a high-hazard dam (MDNR, 2005), steel sheet piling (SSP) and grouted riprap were installed to address the potential for overtopping at flows exceeding the 5-year recurrence interval event (AECOM, 2019b). These repairs stabilized the left embankment and right abutment slopes and provided auxiliary spill capacity over the powerhouse section (Fig. 4). These

measures improved safety and reduced the risk of a potential dam breach. The sheet pile system was designed to allow for phased excavation and removal of the existing structure while reducing seepage and limiting impacts of overtopping up to and including the 200-year recurrence interval flows (AECOM, 2019b).

The improvements were detailed in the design report and drawings prepared by AECOM (AECOM, 2017, 2019a, 2019b). The improvements (identified as Phase 1A improvements) did not change the hydraulic capacity of the dam to pass normal flows of 1,130 cubic feet per second (cfs; Amec Foster Wheeler, 2018). In its condition at the time of preparation of the AECOM reports (2017; 2019a; 2019b), the dam created a hydraulic head of approximately 11 feet, a minimum freeboard of 3 feet, and an impoundment surface area of approximately 59 acres at the normal pool elevation of 657 feet (AECOM, 2017).

The dam, as it exists today, consists of an approximate 170-foot-wide left embankment, 85-foot-wide concrete spillway, and a 140-foot-wide former powerhouse structure that serves as the auxiliary spillway, for a total width of approximately 395 feet (Fig. 4).

The Conceptual Site Model (CSM) for Area 4 is presented in the Supplemental Remedial Investigation (SRI) Report (Amec Foster Wheeler, 2018). The CSM was refined based on pre-design data collection as presented in the PDI report (GEI, 2022b).

3. Data Assessment and Modeling Tools

This section describes the types of data collected to support the models developed to evaluate the river conditions under current and post-dam-removal conditions. Section 5 discusses how these models have been used to develop parameters that guided the design.

The dam removal plan and river stabilization design were informed by the collection and analysis of hydrology, geomorphology, sediment, and soils data collected in Area 4. For reference and evaluation of floodplain connectivity between TCRA and remedial efforts, some reference data were collected in upstream reaches of the Kalamazoo River. Post dam-out bathymetry was collected in Area 3 in 2020 by Affiliated Researchers to evaluate the change in bed elevations and to calibrate the models of Area 3 hydraulics and bed lowering following dam removal.

The demonstration that these modeling tools projected the change in bed elevation following dam removal in Area 3 provides confidence these same models applied with Area 4 data can project a reasonable estimate of conditions following the Trowbridge Dam removal. This section summarizes data collection and the calibration of the hydrologic and hydraulic (H&H), sediment transport, and bank stability models. More detail on data and the calibration supporting model development are in Attachment A and Attachment B. The data files will also be submitted to EPA separately.

3.1 Field and Data Assessments

The field and data assessments to support dam removal and river stabilization design included a compilation of historical data sets and collection of field data between 2020 and 2022. These data informed the PCB-impacted sediment dredging/excavation plan, river hydrology, hydraulics, sediment transport, fluvial geomorphology evaluations as well as the bank stability design. Field data collected and analysis conducted included:

- Sampling, analysis, and evaluation of total PCBs in river sediment and bank soil completed in various phases of investigation.
 - Sampling of in-stream sediment in Subareas E, F, and G and bank soil/toe-of-bank sediment in Subareas C, D, and E (2020 PDI Phase 1).

- Step-out sampling at select locations to define the extent of material with PCBs ≥ 50 mg/kg requiring handling and disposal per Toxic Substances Control Act regulations (2020 PDI Phase 1).
- Additional sampling of in-stream sediment to alluvium in Subarea E and bank soil in Subareas C, D, and E where required for vertical delineation of total PCBs (2021 PDI Phase 2).
- Installation and monitoring of four stream gages with continuous-recording transducers (Fig. 5) to complement USGS gage data to enhance understanding of river hydrology in the main channel and tributaries.
- Collection via site surveys of river dimensions, patterns, profiles, and sediment sizes to calculate entrenchment ratios within specific stable reaches, including reference data from areas upstream of Area 4.
- Collection of tributary field data, including topography and bathymetry in Osgood Drain and Schnable Brook at their confluences with the Kalamazoo River.
- Analysis of design flow conditions for Osgood Drain, Schnable Brook, and other contributing drainage areas.
- Collection of geomorphology data, including depth to pre-dam alluvial layer, alluvial layer samples for visual observation and grain-size analysis, and riffle particle size distributions for exposed riffles upstream of the impoundment influence (GEI, 2022b).
- Collection of bank soils data including soil type with depth and soil strength properties using dynamic cone penetrometer (DCP) testing (GEI, 2022b).
- Geostatistical analysis of PCB data and depths to develop both the depth to topline and the depth to clean line for three separate remedial action limits: 1, 5, and 50 mg/kg as described in Appendix E of the PDI report (GEI, 2022b).
- Development of a continuous alluvial surface across the study area incorporating different datasets. The comprehensive dataset used in the model included historical poling data from USGS and Michigan Department of Environment, Great Lakes, and Energy (EGLE) (Rheaume et al., 2002; Peabody, 2021) and more than 600 poling points collected by GEI (GEI, 2022b). Fig. 6 presents poling locations collected by GEI.

- Collection, analysis, and evaluation of sediment within a proposed pilot channel to evaluate options for onsite placement of material that may be removed from the pilot channel dredge.
- Collection of additional sediment in Subareas C, D, and E to characterize potentially mobile post-dredge sediment that will remain following completion of the Area 4 TCRA.
- Installation of a boring on the left descending bank (LDB) between Trowbridge Dam and the 26th Street bridge to collect geotechnical information to help inform a slope stability analysis of the proposed post-remediation condition.

PCB data is presented and discussed in the Area 4 PDI Phase 1 and Phase 2 Data Summary Report (GEI, 2022b) and the Area 4 PDI Phase 3 Data Summary Report (GEI, 2023).

3.2 Model Use and Calibration

The following discussion is a summary of the H&H, sediment transport, and Bank Stability and Toe Erosion Model (BSTEM) model calibrations. The complete calibration datasets and analyses for the applied models are included in Attachments A and B.

The H&H and sediment transport models were calibrated using historical data and field data collected as described in Section 3.1. Using Area 3 as a calibration baseline, the data generated as part of the Area 3 effort including the SRI data, design data, and pre-dam-out bathymetry were incorporated into the the 1-dimensional (1-D) H&H and sediment transport models. The results of this modeling effort indicated the model accurately forecasted the post-dam removal bathymetry in Area 3. This calibration demonstrates that the use of this model can provide reasonable projections of bed lowering following dam removal in Area 4 of the Kalamazoo River.

Area 4 was modeled with both 1-D H&H and sediment transport models. For future proposed conditions, the TCRA subareas (C, D, E, F, G) were also modeled with a 2-dimensional (2-D) H&H model to estimate the velocities and shear stresses in the main channel at a finer spatial scale than the 1-D model.

3.2.1 Bankfull Flow Estimate

Regional curve studies by USGS in 2009 and Stantec in 2015 evaluated watersheds with drainage areas of 500 square miles or less (Rachol and Boley-Morse, 2009;

Stantec, 2015). The drainage area at Trowbridge Dam is approximately three times larger, totaling a drainage area of 1,532 square miles. Because the information needed was not directly available from USGS, additional desktop and field investigation was completed to define the estimated flow at the bankfull stage.

The estimate of bankfull discharge at Trowbridge Dam was developed using data collected from upstream gages in Marshall, Battle Creek, and Comstock as well as downstream gages in Fennville and New Richmond. Each gage discharge was plotted to generate an equation for the bankfull discharge flow. This equation was used to calculate the bankfull flow at Trowbridge Dam with a known drainage area of 1,532 square miles (Fig. 7).

Before starting field work, GEI obtained USGS rating curves and tables for each of the active gages as well as current site conditions for each location. Field work confirmed the current water level on the staff plate if present or on the USGS website. Bankfull indicators were identified following a visual survey upstream and downstream of each gage. Where bankfull features were identified, the depth from the current water surface to the bankfull feature was measured. This depth was used to relate the stage and discharge of the current water surface to the bankfull stage and discharge using the rating curve and tables supplied by the USGS.

Using the field-determined bankfull stage and relating it to a discharge from the USGS curve, a flow return interval was calculated based on the gage record, with a predicted return interval between 1 and 2 years. This process is further described by Stantec (2015). Return intervals calculated for the four active gages that were visited were 1.4, 1.5, 1.6, and 2.6 for the Battle Creek, Comstock, New Richmond, and Marshall gages, respectively.

The four discharge estimates for bankfull flow, along with other local watershed 1.5-year return flow estimates were plotted on a logarithmic-to-logarithmic scale relating the drainage area (x-axis) in square miles to the estimated discharge (y-axis) in cubic feet per second.

Using a power function to generate a trendline for the regional curve resulted in an estimated bankfull discharge of 3,662 cfs for a drainage area of 1,531 square miles. This estimate was decreased to 3,630 cfs when modeling the bankfull discharge given that the values included from Table E-2.1 of the SRI were for the 1.5-year return interval and were not field verified. Bankfull flow, average flow, and flood flows used for hydraulic modeling are provided in Table 1.

3.2.2 River Hydraulics and Sediment Transport Calibration

The results of the 1-D H&H model were calibrated to Area 4 field-measured water surface elevation (WSE) data collected in 2020 and 2021. Example calibration profile plots for steady-flow conditions are shown in Fig. 8. Dynamic (i.e., unsteady-flow) 1-D model-predicted water depths at the Trowbridge Dam were compared to the measured transducer depths at the dam during a flow event between March 26, 2021 to April 5, 2021. The predicted WSEs over the event matched the measurements from the transducer at the dam as shown in Figs. 9 and 10.

During a high-flow event in June 2020, headwater and tailwater elevations were recorded. The model was adjusted after comparing measured elevation data with modeled flow rates. The 2-D dam-out hydraulic model was checked against the 1-D dam-out hydraulic model, and the results were comparable. An example comparison of the water-surface profile plot for the 1-D and 2-D models is shown in Fig. 11.

The 1-D sediment transport model was calibrated to repeat bathymetric datasets in both Areas 3 and 4. Calibration of the Area 3 model required setting vertical bed erosion limits at the riffle crest elevations after dam removal. For the rest of the Area 3 river model, the potential vertical erosion limit was set below the current bed elevations. Modeled volumes in Area 3 were within 1% of the total net difference in bathymetric datasets between the 2016 and 2020 (Table 2). The model-estimated thalweg elevation was usually within 1 foot of the measured elevation. These results suggest the assumption that the riffle crests essentially “control” bed elevations throughout the reach is reasonable. See Attachment A for more details.

Applying the potential vertical erosion limit used in Area 3 with the alluvial layer as reference, the Area 4 sediment transport model was calibrated to the observed and calculated changes in volume, profile, and cross sections from the 2013 to 2020 Area 4 bathymetry. The model calibration results were within 17% of the calculated volume change (Table 2).

3.2.3 Bank Stability and Toe Erosion Model Calibration

Bank stability in Area 4 was evaluated using the BSTEM model. Bank stability is calculated based on soil properties including weight, cohesion, hydrostatic confining forces, and pore water pressure. Toe scour is calculated based on critical shear stress and erodibility of the bank toe material.

Mechanical soil properties determined during the PDI were used in this effort. GEI collected strength parameters from 16 in situ dynamic cone penetrometer (DCP) tests in Subareas C, D, and lower E banks as described in the PDI summary report

(GEI, 2022b). The Area 4 BSTEM model was calibrated to data from the BBL 2003 Erosion Pin Report and Bank Survey (BBL, 2003), USGS geotechnical data from Area 3 (Rachol et al, 2005), and other reference values for similar soils. See Attachment B for more details.

4. Trowbridge Dam Removal Design

The dam removal design encompasses both installation and operation of a water control structure (WCS) and removal of the remaining Trowbridge Dam structures above finished grade. The WCS will be installed to manage Area 4 impoundment water levels and staged removal of the Trowbridge Dam. A steady-state hydraulic model was used to model the headwater and tailwater elevations during each of the dam removal steps (Fig. 12). The WCS will be installed during Part 1, as described in the Part 1 Removal Work Plan Draft 4 (GEI, 2024b). The WCS activation and removal of the remaining Trowbridge Dam structures will be completed during Part 2, and are described below. The Part 1 major steps are included below for reference. The proposed sequence of dam removal includes the following major steps (separated into Parts 1 and 2):

Part 1 – WCS Installation

- Step 1 – Upstream sediment removal.
- Step 2 – WCS construction.
- Step 3 – Downstream stilling basin and apron construction.
- Step 4 – WCS hydraulic loading test.

Part 2 – WCS Activation

- Step 5 – Existing Steel Sheet Piling (SSP) removal and WCS activation.
- Step 6 – Lower reservoir to El. 650 feet.
- Step 7 – Lower spillway.
- Step 8 – Lower reservoir to El. ~646 feet.
- Step 9 – Riffle construction at former dam location.
- Step 10 – Final dam removal.

These steps are identified on the design drawings and steps 1 through 4 are detailed in the Part 1 Removal Work Plan, including supporting hydraulic and geotechnical calculations. Steps 5 through 10 are detailed below.

4.1 Step 5 – Existing SSP Removal and WCS Activation

To activate the WCS, a significant portion of the 2019 SSP wall immediately upstream of the seven bays will be removed. This will be completed following confirmation that an adequate hydraulic seal is achieved between the new WCS and the left and right sides of the structure. This would be done by flooding the space between the WCS and 2019 SSP and documenting steady state performance.

During Step 4, the flow through the WCS is assumed to be a straight drop over the stoplogs to a designed basin bottom at El. 641.0 feet. Results from the average daily, 2, 10, 100, and 200-year-event model runs were used to calculate minimum basin lengths and impact block dimensions and location to dissipate the energy within the outflow prior to it exiting the WCS.

4.2 Step 6 – Lower Reservoir to El. 650 Feet

This step assumes that upstream PCB dredging is complete. Once the WCS is loaded and seepage is well controlled, stoplogs can be removed to begin lowering the reservoir. The rate of stoplog removal will depend on upstream river conditions at the discretion of the GEI engineer of record and in consultation with EPA and MDNR. It is anticipated that the reservoir can be lowered to El. ~650 feet under normal flow conditions within several months. During this phase after water levels are below 654.5 feet, the WCS will become the primary spillway for the dam and the existing spillway will become an auxiliary spillway to be used during a flood event.

Once stoplog removal begins in Step 5, the tailwater elevation increases in relation to the stoplog elevation until the stoplogs are submerged by the tailwater and the spillway no longer experiences free overfall, except during smaller flow events (average daily flow during construction step 5; 2-year and average daily flows during construction step 6). This change in flow is analyzed further in the Part 1 Removal Work Plan (GEI, 2024b).

4.3 Step 7 – Lower Spillway

To minimize the risk of large floods being routed through the WCS, this step lowers the existing spillway structure. The top portion of the existing concrete spillway will be removed to from El. ~655 to El. ~649 feet, leaving the majority of the spillway concrete intact. The area upstream of the spillway will be excavated to El. ~647 feet and backfilled with grouted riprap. To facilitate demolition and construction, an SSP wall will be installed across the upstream side of the spillway. This SSP will become integral with the 2019 SSP walls, as there are jokers installed to accept the new SSP wall sheets. Following demolition and construction, the upstream SSP wall would be

cut down to El. ~650 feet and made available as an auxiliary overflow spillway. It is anticipated this step may take 0.5–1 month to complete.

Flows through the WCS continue to be submerged by tailwater elevations during higher flow events, as shown in GEI, 2024b. The partially demolished spillway will also become submerged during flow events greater than the 2-year return interval.

4.4 Step 8 – Lower Reservoir to El. ~646 Feet

Following Step 6, stoplogs will continue to be removed down to the bottom of the water control structure to an elevation of 642.0 feet. The modeled water surface under this end condition matches the tailwater elevation, which allows for the reconnection of the impoundment with the flowing river downstream. During this step, sediment transport through the WCS is anticipated. The rate of stoplog removal will depend on the upstream river conditions and sediment transport rate, and will be at the discretion of GEI engineer of record, in consultation with EPA, MDNR, and EGLE.

4.5 Steps 9 and 10 – Initial Riffle Construction and Final Dam Removal

Once all the stoplogs have been removed, final dam removal activities would begin. This step first includes removal of the remainder of the existing spillway and the left embankment downstream of the SSP walls and construction of the riffle structure. Once the riffle structure is in place, the SSP wall can be removed, allowing flow through the newly constructed riffle. The WCS and remaining original powerhouse structures can then be removed. It is anticipated that the stilling basin slab or other remaining powerhouse structures below El. 641 feet will remain in place and become buried under constructed riffle and floodplain fill materials. This step would also include removal of the grouted riprap slope and finished grading activities on the right abutment.

4.6 Construction Considerations

Trowbridge Dam removal will require access to and use of both the LDB and right descending bank (RDB) adjacent to Trowbridge Dam. These areas are primarily MDNR-managed land except for a privately owned parcel on the RDB. If use of this parcel is necessary, GEI will coordinate with EPA to obtain “consent to access” forms from the property owner(s) prior to any activities on the property. Before construction, the staging areas and access roads will be cleared, grubbed, and prepared in accordance with the procedures, applicable regulations, and best management practices discussed in GEI’s Removal Work Plan Part 1 (GEI, 2024b)

and Part 2 (GEI, 2024c). Additionally, security measures will be installed at each staging area as discussed in the Removal Work Plan.

The LDB staging area (referred to herein as the Trowbridge Dam staging area) will be constructed for use throughout the entirety of the Area 4 TCRA and will serve as the main staging area. However, for the WCS construction, it is anticipated the RDB staging area will serve as the primary construction and laydown area. For Trowbridge Dam removal, both the RDB and Trowbridge Dam staging areas will be used as needed to access the Trowbridge Dam structures.

5. Channel Design

This section discusses the results of modeling for conditions that will exist after removal of the dam. The hydraulic, sediment transport, and bank stability models were used to project future conditions and generate the parameters necessary to design a stable channel with appropriate bank restoration treatments. The model also assessed the effectiveness of a sequenced drawdown to facilitate the drying and consolidation of banks with the lowering of the riverbed to its approximate, former location while avoiding uncontrolled bank erosion associated with a rapid impoundment drawdown.

The design considered:

- The objectives of the TCRA Action Memorandum (EPA, 2020), including removal of PCB-containing sediments and bank soils, bank restoration grading, removal of the Trowbridge Dam, and construction of a stable channel.
- The location of the former thalweg through the lower portion of Subarea G (see Section 5.1).
- The footprint of the dredge prism necessary to remove ~333,000 cubic yards (CY) of in-stream sediment with PCBs ≥ 1 mg/kg.
- The extent and location of the ~41,000 CY of bank soil ≥ 5 mg/kg PCB to be removed.
- A dredged pilot channel (80-foot bottom width, average 4-foot depth, with 3:1 side slopes; approximately 150,000 CY) from approximately RM 45.35 to RM 47 (see Section 5.2).
- The final projected channel attributes as discussed in this section, which includes the incorporation of a dredged pilot channel from approximately RM 45.35 to RM 47, a constructed riffle at the removed Trowbridge Dam location, and 11 coarsened riffle crests from approximately RM 45 to RM 47, and restored banks throughout the channel.
- Targeted channel widening (approximately 10,000 CY) and targeted riffle crest lowering in Subareas C and D based on discussions with EPA on June 5, 2024 and HEC-RAS hydraulic modeling files provided by EPA (EPA, 2024).

5.1 Historical Thalweg

Evaluation of the alluvial base (described in the PDI summary report) revealed the location of the pre-dam thalweg in Subarea E as it meanders under the lower part of Subarea G rather than along the current channel alignment. This finding appears to be consistent with pre-dam, hand-drawn maps, but differs from the SRI conceptual site model, as discussed in further detail in the PDI summary report (GEI, 2022b). Based on multiple lines of evidence, this revised alignment has been incorporated into the channel design.

A pilot channel will be constructed to achieve this configuration and facilitate the initial phase of bank remediation and channel construction.

5.2 Pilot Channel

To facilitate the initial phase of channel construction, a pilot channel will be constructed using hydraulic dredge methods, and will extend from at or near the completed environmental dredge prism footprint to approximately RM 47.0. The pilot channel cross-section design consists of an 80-foot bottom width, with 3:1 side slopes, and an average depth of about 4 feet. The pilot channel offers many benefits to the overall preparation of the channel including: removing a relatively large volume of potentially mobilized sediment from the system, providing a defined river flow path away from the exposed banks during the drawdown sequence, and initially establishing the thalweg elevation at or near the project dam-out equilibrium (Fig. 13).

The channel design assumes that bank excavation and restoration work will be conducted along the banks of the future channel to maintain bank stability and to control erosion. See Section 6 of this report for detailed design discussion regarding bank restoration.

5.3 Riffle Installation / Coarsening

The channel design includes constructing an engineered riffle within the vicinity of the removed Trowbridge Dam and constructing multiple coarsened riffles throughout the channel (Fig. 14). The riffles were included in the design to enhance hydraulic grade control and reduce the quantity of potentially mobilized bed sediment. The riffles will also help dissipate energy throughout the TCRA area rather than allowing high velocities or shear stress to concentrate in any one location. The design locations of the proposed coarsened riffles generally align with riffle features identified in the poling data (Fig. 15). The proposed riffles are located in straight sections of the river between bends and where the thalweg crosses over from one

bend to the next. The proposed coarsening is sufficient so the riffles will be stable up to at least 100-year flow conditions. The riffles will be constructed on coarse subgrade. During construction, the field engineering team will evaluate riffle subgrades in conjunction with EPA oversight and improve as necessary (e.g., by undercutting erodible material) until coarse subgrade is achieved. Similarly, the transition from each riffle to downstream pool will be evaluated during construction, and improved as necessary by placing additional stone downstream of the riffle as needed for stability.

5.4 Future-Conditions Modeling

Modeling of future conditions in Area 4 started with the calibrated 1-D H&H and sediment transport model. The cross sections were then edited to reflect removal of the proposed dredge volume, the bank excavation, and construction of the pilot channel. Some additional smoothing of the dredge surfaces (in the cross-section data) was performed to account for constructability.

The model cross sections were then revised at the dam to reflect partial or complete dam removal/drawdown. In the sediment transport models, the horizontal limits of the moveable bed were set at the toe of the bank treatments, as bank treatments are designed to withstand lateral migration of the channel. The coarsened riffles were set as the vertical limit of bed erosion at those locations. Bank treatments (further discussed in Section 6), are designed to be stable during 100-year dam-out modeled shear stresses and velocities. The revised alignment through upper Subarea E and lower Subarea G provides an opportunity for wider floodplain grading in those areas.

Separate hydrologic and hydraulic models for Schnable Brook and Osgood Drain were developed to design modifications to stabilize the tributaries as they connect to the mainstem after the dam is removed. The modifications include grading the adjacent tributary corridors as well as installing grade control features. Estimated drainage areas and flows for 12 other ephemeral tributaries were used to facilitate proper grading to maintain positive flow through the floodplain.

The 2-D hydrodynamic model was also used to estimate the velocities and shear stresses in the main channel at a finer scale than the 1-D model to refine the estimated future river conditions and inform stability designs.

5.5 Channel Alignment

The channel alignment was designed by:

- Incorporating the proposed environmental dredging extents, bank excavations, and the pilot channel configuration into the future-conditions models.
- Defining the alluvial surface as the vertical erosion limit.
- Representing the WCS as an inline structure with gates to represent stoplog elevations.
- Running various dam-removal scenarios to evaluate sediment transport and bed- and water-surface lowering.

The 1-D hydraulic model was run multiple times to evaluate locations and elevations for riffle crests to distribute grade control throughout the approximately 2-mile length of river from Trowbridge Dam to the upstream end of the TCRA boundary. The existing bed profile and the 1-D projected dynamic equilibrium bed profile of the channel following 7 years of sediment transport after dredging, bank excavation, dam removal, and riffle coarsening are shown in Fig. 15. The 7-year timeframe was used because post-dam-removal modeling showed that after 5–7 years of average, annual flow conditions, the bed reaches a dynamic equilibrium (i.e., no longer appreciably degrades or aggrades). The overall slope of the dam-out equilibrium channel is 0.0007 feet/feet based on the 1-D model bed projections. The projected channel was then converted from the 1-D model cross sections into a 2-D model surface for a better visual and hydraulic representation of the entire channel alignment. The anticipated alignment, including the proposed riffle locations, is shown in Fig. 14.

5.6 Targeted Channel Widening and Riffle Crest Refinement

Based on recommendations given to GEI by EPA on June 5, 2024 (EPA, 2024), select modifications were made to the channel design in Subareas C and D. These modifications included approximately 30 feet of channel widening on the right bank between river mile 46.07 and 46.39, approximately 30 feet of channel widening on the left bank from river mile 46.07 to 46.26, and approximately 40 feet of channel widening between river miles 47 and 47.14. These modifications resulted in bankfull channel widths between 185 and 225 feet in these locations, which is comparable to channel widths upstream and downstream of these targeted locations. GEI refined the channel widening recommendations by smoothing the bank edges so there were no abrupt changes to the river cross section from upstream to downstream, and by adding constructable bank side slopes (i.e., 2H:1V) at the widened locations. The modifications also included minor adjustments to three riffle crest elevations. Riffle crest elevations at river miles 47.07, 46.84, and 46.16 were lowered between 0.3

feet and 0.5 feet. These changes resulted in average channel bankfull velocities between 3.7 and 4.2 feet per second at riffles in Subareas C and D.

5.7 Channel Characteristics

The channel alignment and dimensions are summarized in terms of bankfull parameters (i.e., 1½ year flow event) at riffle cross sections in Table 3. This includes channel width, depth, cross-sectional area at bankfull stage, and the entrenchment ratio. The entrenchment ratio is defined as the ratio of width of flow at a depth two times the maximum bankfull riffle depth (i.e., the flood-prone width) to bankfull width (Fig. 16).

Figs. 17 through 23 illustrate predicted water depths for average daily, bankfull, 10-year, 100-year, 150-year, 200-year, and 500-year flows. These figures show that after dam removal, Subareas F and G will not be inundated for 30 consecutive days and can be considered floodplain.

A plan view, cross section at the Trowbridge Dam constructed riffle crest, and a profile through the constructed riffle are shown in Figs. 24, 25, and 26. Because the riffle at the dam acts as a critical grade-control point for the river, the dimensions are specified.

Figs. 27 and 28 show the 100-year return interval event velocities and shear stresses. Figs. 29 and 30 show the bankfull velocities and shear stresses.

6. Bank Stabilization

River conditions will adjust over time as the impoundment is lowered and the river adapts. Several major principles have been considered during the design of bank stabilization:

- Quantified and modeled conditions (i.e., projected bed elevations, water levels, velocities, and shear stresses) that influence the performance of bank treatments.
- Bank height and geometry.
- Timing of drawdown of the impoundment to promote vegetative bank treatments.
- Incorporation of native species typically found along the banks and floodplains of the Kalamazoo River.
- Toe and bank stabilization measures designed to withstand anticipated forces.
- Incorporation of temporary erosion control materials specifically used for ecological restoration, constructed of 100% biodegradable material including coir and/or straw.

Bank stability results are reported as factors of safety, where values <1 indicate potential bank failure, values between 1 and 1.3 indicate marginal stability, and values >1.3 indicate stability (Table 4). Modeling results of current conditions indicate that in Subareas C and D the factor of safety is >1.3 at most cross sections, except near RM 46. In lower Subarea E (in the narrow valley), the factor of safety is >1.3 at most cross sections except for the left descending bank (LDB) near RM 45, which has a slope steeper than 1:1.

Based on the BSTEM results, the banks at highest risk of failure are located along the main channel in areas Subarea F and G islands (Fig. 31). Steel sheeting will be considered as a means of providing temporary bank stability and protection while the exposed soils dry out and strengthen.

An additional factor affecting bank stability in Area 4 is the presence of high groundwater within the banks. A gradual drawdown period not to exceed 6 inches per day can reduce this risk by allowing time for the groundwater levels to equilibrate

as the river water level is lowered. Details of the impoundment drawdown can be found in the sequence of work drawings.

Based on the aforementioned, varying treatment techniques have been developed for the riverbanks, depending on modeled post-dam conditions in respective areas along the river. Treatments were selected based on a combination of publications and field experience, including research that has been conducted by the U.S. Army Corps of Engineers on the stability thresholds for various stream restoration techniques (Fischenich, 2001). These treatments (see Part 2 design drawings) include:

- Vegetated bank stabilization without toe protection and without bioengineered soil lifts.
- Bioengineered soil lifts with stone toe protection.
- Bioengineered soil lifts with toewood.

These techniques have been developed based on dynamic equilibrium conditions during a 100-year flood event. A plan view showing the anticipated bank treatments is shown in Fig. 32.

The techniques are described individually below, but they can be combined based on conditions. For example, toewood may be combined with bioengineered lifts on taller banks to create stability from the river toe to the top of the bank. Specific conditions in the field may require modifications to these techniques.

6.1 Bank Sloping and Revegetation

The Kalamazoo River is a relatively low-energy system with comparatively low shear stresses and velocities along the riverbanks, even during 100-year flood conditions. Modeling has projected that some locations along the Area 4 reach will have maximum shear stresses less than 0.5 pounds per square foot and velocities less than 4 feet per second at the 100-year flood event. Vegetation alone is typically adequate to stabilize riverbanks under these conditions. In such locations, following any required removal action, clean fill may be used to rebuild the riverbank to a 3:1 or 2:1 slope. Areas with naturally stable slopes may simply be revegetated to provide stability.

Following resloping, a custom native seed mix will be installed and covered by a layer of 100% biodegradable erosion-control fabric, which is proposed to be NAG C-125BN or an approved equivalent. This erosion-control fabric can be expected to last 2–5 years before biodegrading, which will provide time for the native vegetation

to establish. Following the installation of erosion blankets, native shrubs will be installed via bare root, live stake, or shrub-plug stock, as appropriate.

6.2 Bioengineered Lifts with Rock Toe

In the highest energy areas of the river (typically where near bank 100-year flow shear stresses exceed approximately 1 pounds per square foot or 100-year velocities exceed 5 feet per second), in locations where bank geometry warrants, in places that may have higher levels of human traffic, or in places where PCBs are presumed to remain in the adjacent floodplain, a rock toe will be used to stabilize the river toe and/or riverbanks. The rock toe will be keyed into alluvium at the banks to protect against scour and potential undercutting at the bank.

In locations with higher banks, steeper banks, or in areas where river forces are higher than those where bank sloping is sufficient, bioengineered lifts will be used to reconstruct the riverbanks above the rock toe. Bioengineered lifts use prefabricated material constructed of 100% biodegradable substances to build the bank in soil-wrapped tiers, or lifts, which will hold the soil in place while vegetation establishes. It is anticipated lifts will be constructed from Rolanka Bio-D block 12-300 or approved equivalent. Each sequential layer is seeded with native vegetation and planted with native shrubs via bare root, live stakes, or shrub-plug stock. As the erosion control fabric biodegrades over time, the native vegetation will establish and naturally stabilize the riverbank.

6.3 Toewood

Toewood incorporates natural logs, roots, and brush to stabilize the toe of the river while creating habitat for fish, macroinvertebrates, and herpetofauna. Toewood can resist river forces that exceed those that living vegetation alone cannot withstand. It is anticipated that toewood will be installed in outside bends of the river and/or areas that are projected to have higher velocities and shear stresses. The toewood will generally be placed underwater (considering the dynamic equilibrium dam-out condition) to provide longevity while also slowing river flows in near-bank areas. The toewood would be installed from the alluvial surface to approximately 1-foot above the dam-out average daily water surface. As needed, bioengineered lifts or erosion blankets will be used to stabilize and revegetate soil that is placed above the toewood.

6.4 Restoration Sequencing

Coordination of the construction sequencing and the impoundment drawdown are important to the successful installation of bioengineered bank treatments. This

approach allows soils to dewater and subsequently become more stable. Additionally, this approach allows native vegetation to establish contemporaneously with lowering of the water level.

Figures depicting a work sequence plan and a general work sequence description are provided separately as part of the design documents.

7. Summary

Key attributes of the Dam Removal and River Stabilization Design Summary, including additional design elements incorporated since the original design submittal are as follows:

- The design of the channel following dam removal is a combination of defining the dredge and excavation limits; forecasting river lowering; and then specifying the bank and toe erosion controls necessary to maintain stability.
- To address shear stress and velocities along the bank, the channel design includes proposed riffles and bank and toe stabilization measures to withstand flow events and corresponding erosive forces up to the 100-year design interval event.
- Based on the revised channel alignment and associated bank grading within Subareas E and G, the design includes floodplain grading at or near the bankfull discharge elevation throughout this reach.
- At the request of EPA, riffles and channel widening were incorporated into the revised design to decrease the initially projected velocities. Additionally, the riffles enhance hydraulic grade control and reduce the quantity of potentially mobilized bed sediment.
- The design includes an 80-foot wide, on average, 4-foot deep pilot channel over most of the length of the impoundment. The pilot channel offers many benefits to the overall preparation of the channel including: removing a relatively large mass of potentially mobilized sediment from the system, providing a defined flow path away from the non-excavated banks during the drawdown sequence, and initially establishing the thalweg elevation at or near the project dam-out equilibrium.
- Tributary grade controls and associated channel grading have been incorporated into the restoration designs for Osgood Drain and Schnabel Brook. These features provide channel stability and appropriate connectivity to the main channel.
- The drawdown will be accomplished by operation of a water control structure in the approximate location of the former powerhouse and gradual lowering of the water levels in a controlled manner over a scheduled drawdown period.

The dam structure itself will be removed in phases to also help control drawdown during construction.

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Tables

1. Flood Discharge Values
2. Predicted and Observed Bed Volume Changes in Areas 3 and 4
3. Bankfull and Entrenchment Dimensions of Future Alignment
4. Representative BSTEM Factors of Safety (pre-design with no bank treatments applied)

Table 1. Flood Discharge Values

Return Period	Discharge Amec (2018) (cfs)¹	Discharge AECOM (2019b) (cfs)¹	Discharge GEI 2022² (cfs)¹
Baseflow	na	600	600
Normal flow	1,130	na	1,130
50% (2 year)	4,900	na	4,900
10% (10 year)	7,800	7,100	7,800
2% (50 year)	na	11,000	11,000
1% (100 year)	11,800	12,000	11,800
0.7% (150 year)	na	na	13,100
0.5% (200 year)	na	14,000	14,000
0.2% (500 year)	na	17,000	17,000

Notes:

1. cfs = cubic feet per second.

2. GEI 2022 values indicate the discharge values used in GEI hydraulic modeling.

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Table 2. Predicted and Observed Bed Volume Changes in Areas 3 and 4

Area	Years	Observed Volume Change (yd ³)	Simulated Volume Change (yd ³)	Difference (%)
Area 3	2016-2020	108,870	108,140	-1%
Area 4	2013-2020	53,320	44,070	-17%

Notes:

1. Observed volume changes calculated from repeated bathymetry data

yd³ = cubic yards

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Table 3. Bankfull and Entrenchment Dimensions of Future Alignment ¹

River Mile	Flow Area (ft ²)	Top Width (ft)	Maximum Depth (ft)	Entrenchment Ratio ²	Feature ³
47.07	1028	233	5.2	2.8	Riffle
46.84	882	183	5.7	6.2	Riffle
46.36	891	195	5.1	1.9	Riffle
46.16	955	222	5.8	4.4	Riffle
46.00	795	198	4.9	1.4	Riffle
45.77	1003	226	5.6	1.5	Riffle
45.59	1258	274	5.9	2.1	Riffle
45.48	1038	224	5.9	2.8	Riffle
45.32	995	277	6.2	1.8	Riffle
45.16	1028	275	5.9	1.2	Riffle
45.08	912	266	5.5	1.2	Riffle
44.96	921	238	5.2	1.4	Riffle

Notes:

1. All bankfull dimensions calculated at the 1.5-year return interval (bankfull) flow from the future conditions 1-D HEC-RAS model at all model cross sections.
 2. The entrenchment ratio is the width of flow at two times the maximum bankfull riffle depth (called the flood-prone width) divided by the bankfull width.
 3. Feature refers to the likely morphological feature present in the main channel.
- ft² = square feet; ft = feet

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Table 4. Representative BSTEM Factors of Safety (Pre-design with no bank treatments applied)

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
C	47.14	Left	0.9:1	35 to 38	1.5 to 2.5	1.5 to 3
C	47.14	Right	3.4:1	35 to 38	5.3 to 1.00E+08	5.4 to 1.00E+08
C	47.07	Left	3.2:1	35 to 40	2.6	2.4 to 3.1
C	47.07	Right	3.8:1	34 to 39	2.8	2.8
C	47.00	Left	3.6:1	35 to 40	2.7	2.5 to 2.9
C	47.00	Right	3.4:1	34 to 39	2.8	1.00E+08
C	46.92	Left	4.3:1	35 to 38	1.00E+08	1.00E+08
C	46.92	Right	3.4:1	35 to 38	2 to 14	2.7 to 25.5
C	46.84	Left	3.4:1	35 to 38	2.6	2.5 to 2.9
C	46.84	Right	2.6:1	35 to 38	3.6 to 4	3.5 to 4.4
C	46.8	Left	2.9:1	34 to 39	2.0	2.0
C	46.8	Right	4.8:1	33 to 39	0.6 to 3.3	0.6 to 2.8
C	46.67	Left	3:1	34 to 39	2.2	2.2
C	46.67	Right	2.6:1	33 to 39	5.0 to 1.00E+08	5.0 to 1.00E+08
D	46.46	Left	5.9:1	32 to 37	3.9	3.8 to 4.0
D	46.46	Right	3.7:1	39 to 40	3.5 to 5.5	4.0 to 6.4
D	46.36	Left	3:1	35 to 38	1.00E+08	1.00E+08
D	46.36	Right	2.9:1	35 to 38	2.2	2.1 to 2.2
D	46.26	Left	1.1:1	36 to 39	1.00E+08	1.00E+08
D	46.26	Right	3.1:1	32 to 38	2.0	2.0
D	46.16	Left	2.9:1	36 to 39	2.3	2.3
D	46.16	Right	3.1:1	32 to 38	0.3 to 2.7	2.1
D	46.07	Left	4.3:1	35 to 38	1.1 to 3.6	3.0 to 3.6
D	46.07	Right	3.3:1	35 to 38	2.3	2.3
D	46.00	Left	2.9:1	35 to 38	0.5 to 2.0	2.0
D	46.00	Right	3.2:1	34 to 40	0.5 to 2.6	2.2
D	45.89	Left	3.7:1	35 to 38	5.5 to 10	5 to 16.5
Upper E / F / G	45.89	Right	2.9:1	20	1.0 to 1.1	1.1
D	45.83	Left	2.6:1	35 to 38	1.8 to 1.0E+08	1.8 to 6
Upper E / F / G	45.83	Right	5:1	20	1.9 to 3.2	4 to 25
D	45.77	Left	1.8:1	35 to 38	5.5 to 1.00E+08	5.5 to 8
Upper E / F / G	45.77	Right	4:1	20	2 to 2.7	2 to 3.7
D	45.72	Left	2.3:1	35 to 38	9 to 15	6.5 to 19
Upper E / F / G	45.72	Right	3.3:1	20	0 to 1.00E+08	2.8 to 3

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
Upper E / F / G	45.68	Left	2:1	35 to 38	2.9 to 1.00E+08	2.9 to 15
Upper E / F / G	45.68	Right	4.2:1	20	0 to 1.00E+08	0 to 2.1
Upper E / F / G	45.64	Left	25:1	35 to 38	1.00E+08	1.00E+08
Upper E / F / G	45.64	Right	3.1:1	20	0 to 1.00E+08	1.2 to 3.5
Upper E / F / G	45.59	Left	25:1	20	1.00E+08	1.00E+08
Upper E / F / G	45.59	Right	3.8:1	20	0 to 1.00E+08	3.4 to 4.2
Upper E / F / G	45.56	Left	7.7:1	20	0.5 to 1.00E+08	0.5 to 1.0E+08
Upper E / F / G	45.56	Right	4.0:1	20	0 to 1.00E+08	1.8 to 5.3
Upper E / F / G	45.51	Left	3.6:1	20	0 to 1.00E+08	0 to 4
Upper E / F / G	45.51	Right	3:1	20	0 to 1.00E+08	1.8 to 4.1
Upper E / F / G	45.48	Left	5.3:1	20	0 to 2.8	1.5 to 2.6
Upper E / F / G	45.48	Right	2.4:1	20	0 to 1.00E+08	1.3 to 2
Upper E / F / G	45.44	Left	5.9:1	20	1.00E+08	1.00E+08
Upper E / F / G	45.44	Right	7.1:1	20	3.9 to 1.00E+08	4.9 to 1.00E+08
Upper E / F / G	45.41	Left	7.1:1	20	3.3	2.8
Upper E / F / G	45.41	Right	4.8:1	33 to 41	6.9 to 8.3	7 to 9.4
Upper E / F / G	45.37	Left	9.1:1	20	7 to 1.00E+08	7 to 1.00E+08
Upper E / F / G	45.37	Right	4:1	33 to 41	1.00E+08	1.00E+08
Upper E / F / G	45.35	Left	4.3:1	20	1.5 to 2.9	2.8
Upper E / F / G	45.35	Right	14.3:1	33 to 41	0.7 to 35	27 to 41
Upper E / F / G	45.32	Left	3.7:1	20	0 to 1.00E+08	6.5 to 11.5
Upper E / F / G	45.32	Right	4.3:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.27	Left	1.2:1	33 to 41	2.1 to 90	2.3
Lower E	45.27	Right	0.8:1	33 to 41	5.5 to 10.2	7.6 to 10.2
Lower E	45.24	Left	1.3:1	33 to 41	1.7 to 2.5	1.8 to 2.2
Lower E	45.24	Right	3.2:1	33 to 41	9 to 15	12 to 17.5
Lower E	45.2	Left	7.7:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.2	Right	5.9:1	33 to 41	0.96 to 24	18.5 to 24
Lower E	45.16	Left	1.3:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.16	Right	5.9:1	34 to 39	1.00E+08	1.00E+08
Lower E	45.13	Left	2.9:1	33 to 41	3.7 to 5.4	4.4 to 5.3
Lower E	45.13	Right	0.1:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.1	Left	1.6:1	33 to 41	2 to 10.6	2.2 to 2.7
Lower E	45.1	Right	1.6:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.08	Left	1.8:1	33 to 41	1.4 to 9.1	1.4 to 1.6
Lower E	45.08	Right	4.0:1	33 to 41	9.9 to 1.00E+08	14.5 to 18.5

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
Lower E	45.05	Left	1.2:1	33 to 41	1.3 to 6.5	1.5 to 1.6
Lower E	45.05	Right	3.4:1	32 to 36	15.6 to 1.00E+08	21 to 27
Lower E	45.02	Left	2:1	33 to 41	1.5 to 9.8	1.6
Lower E	45.02	Right	3.2:1	33 to 41	3.5 to 4.9	4.2 to 4.9
Lower E	45.00	Left	1.2:1	33 to 41	1 to 9.7	1.1 to 1.5
Lower E	45.00	Right	3.3:1	33 to 41	7.8 to 24.4	10.8 to 14.2

Notes:

1. Bank Slope measured at beginning of simulation.
2. Factor of Safety presented as range of values during the 7-year 1-D HEC-RAS simulation. Factor of Safety < 1 is unstable; 1-1.3 is marginally stable; >1.3 is stable.
3. At Factor of Safety values < 1 the model will simulate a bank failure. The Factor of Safety is often higher after a bank failure due to the resulting shallower bank slope.

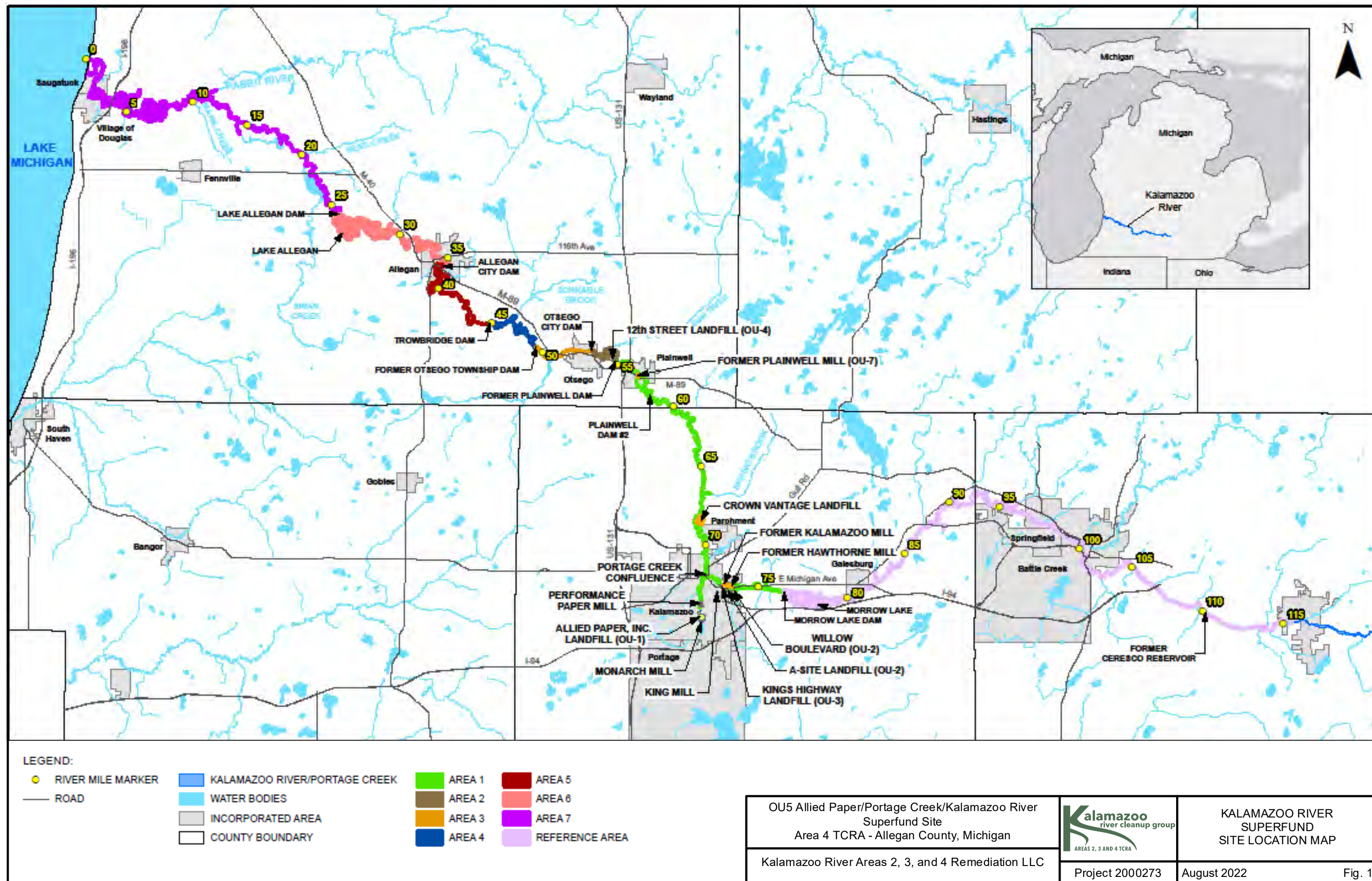
BSTEM = Bank Stability and Toe Erosion Model; H = horizontal; V = vertical; ft = feet.

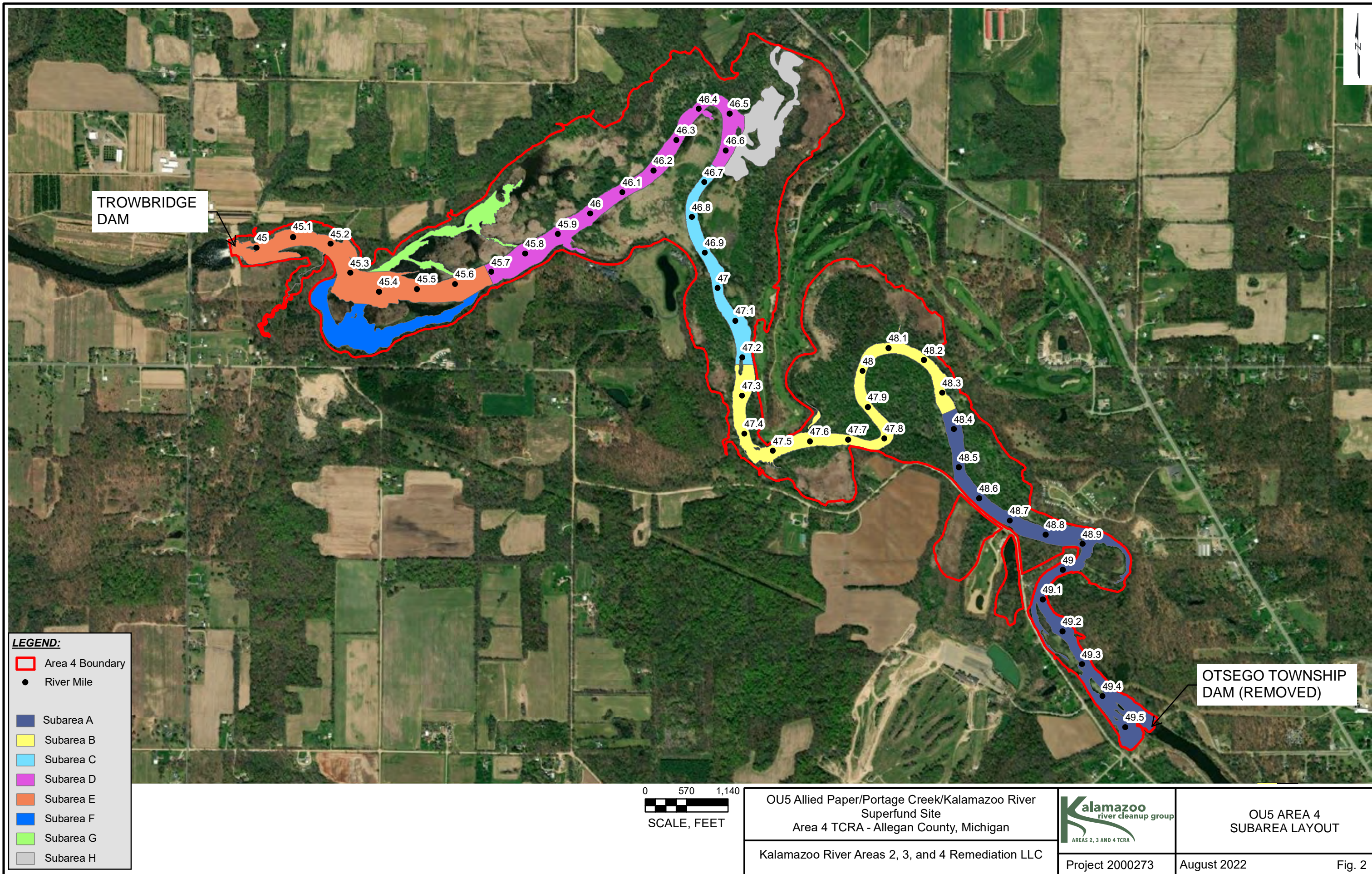
Created by: EG 9/17/2021

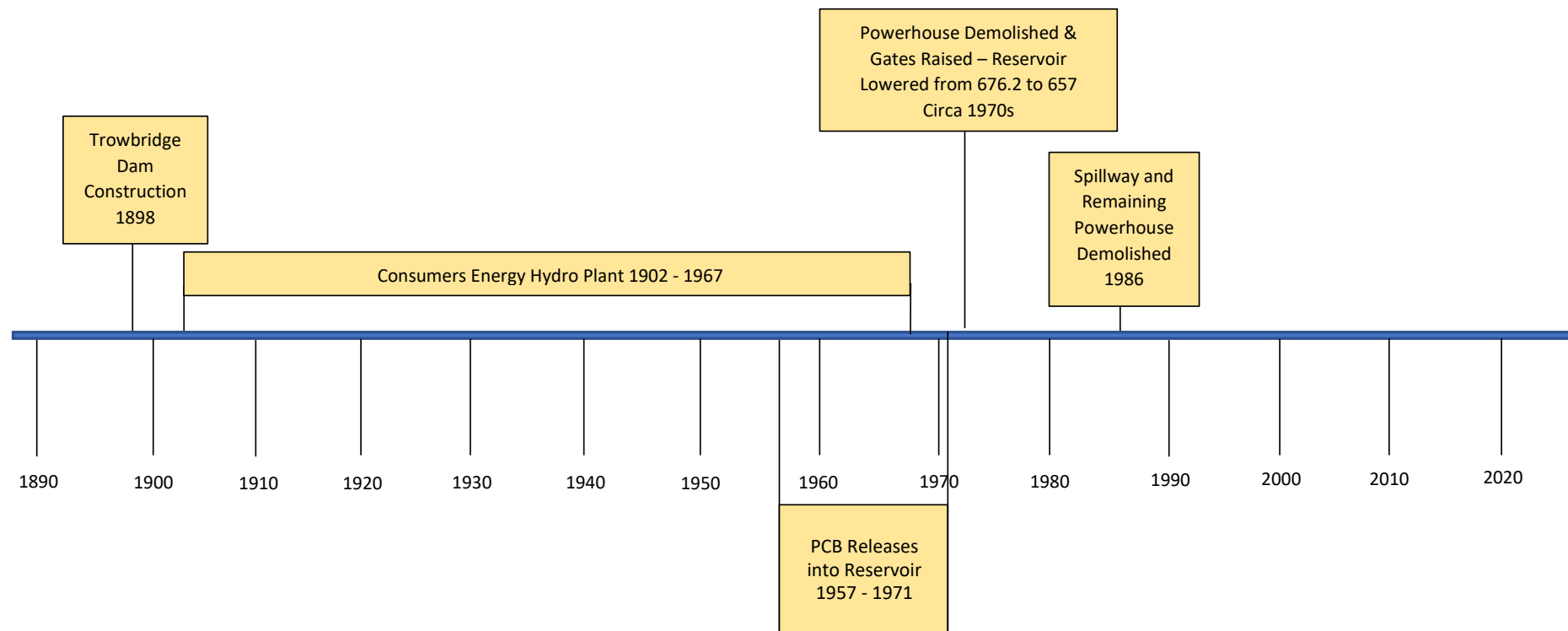
Checked by: LAP 9/24/2021

Figures

1. Kalamazoo River Superfund Site Location Map
2. OU5 Area 4 Subarea Layout
3. Trowbridge Dam Timeline
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5. Transducer Locations
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20. Area 4 Plan View of Water Depth for 100-Year Flow in Modeled Alignment
21. Area 4 Plan View of Water Depth for 150-Year Flow in Modeled Alignment
22. Area 4 Plan View of Water Depth for 200-Year Flow in Modeled Alignment
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25. Proposed Riffle at Trowbridge Dam - Cross-section at Crest
26. Proposed Riffle at Trowbridge Dam - Profile
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28. Area 4 Plan View of 2-D Shear Stress 100-Year Flow in Modeled Alignment
29. Area 4 Plan View of 2-D Velocities to Bankfull Flow in Modeled Alignment
30. Area 4 Plan View of 2-D Shear Stress for Bankfull Flow in Modeled Alignment
31. Example Cross-Section Showing Sensitivity Analysis for One Bank
32. River Bank Treatment Plan








Prepared by MC/Date: 12/1/2020
 Checked by LN/Date: 8/3/2022

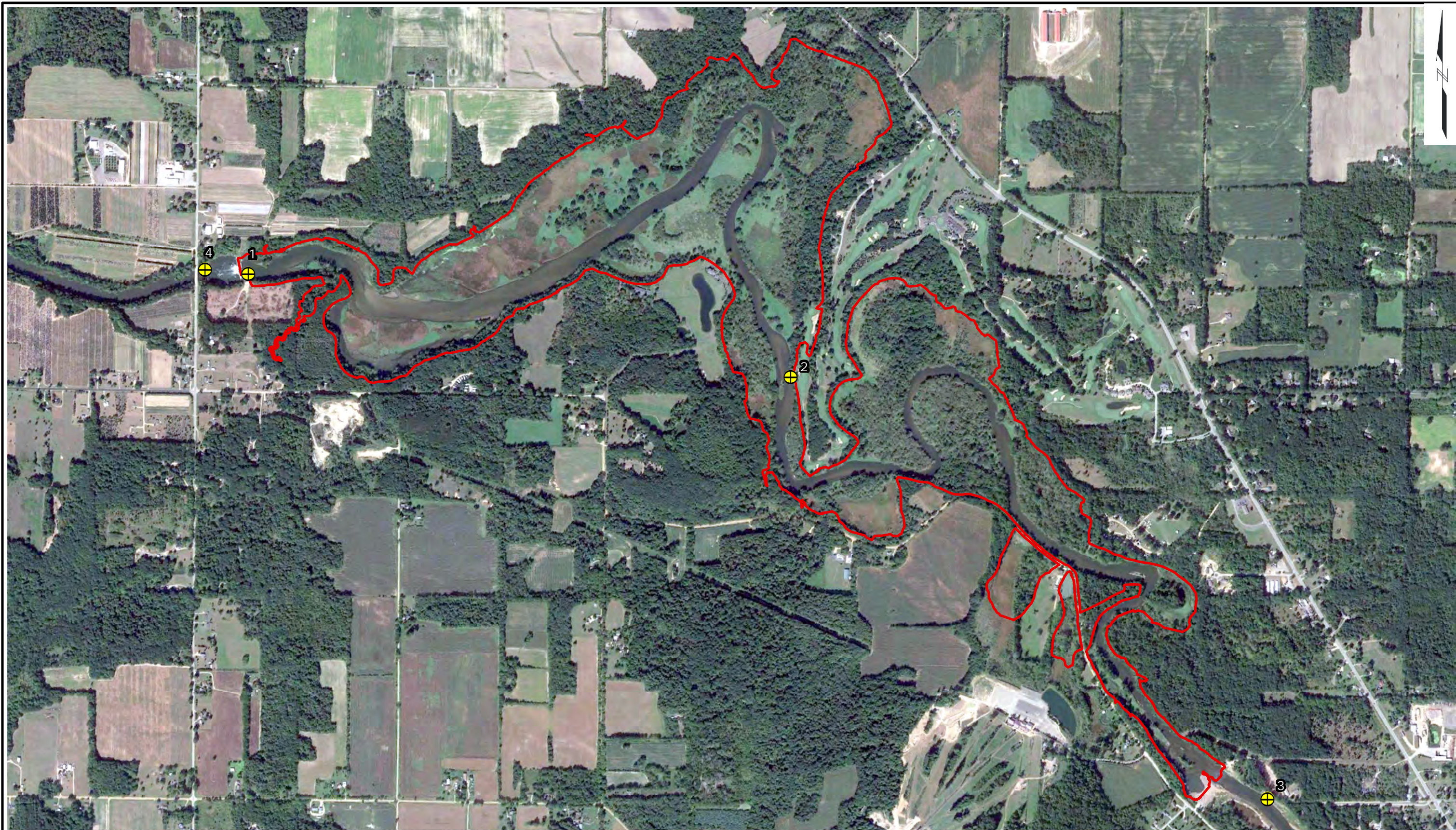
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA – Allegan County, Michigan		TROWBRIDGE DAM TIMELINE	
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	August 2022	Fig. 3





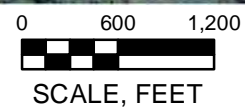
NOTES:
1. AERIAL IMAGERY COLLECTED BY GEI CONSULTANTS OF MICHGAN, P.C. IN APRIL 2020 AND MAY NOT REPRESENT CURRENT SITE CONDITIONS.
2. LEFT AND RIGHT DETERMINATIONS ARE MADE LOOKING IN THE DOWNSTREAM FLOW DIRECTION.

Prepared by SP/Date: 11/1/2021
Checked by EG/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan Kalamazoo River Areas 2, 3, and 4 Remediation LLC	 AREAS 2, 3 AND 4 TCRA	TROWBRIDGE DAM COMPONENTS	Project 2000273	August 2022	Fig. 4



-  Transducer
-  Area 4 Study Boundary



Prepared by AG/Date: 5/27/2020
Checked by LN/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

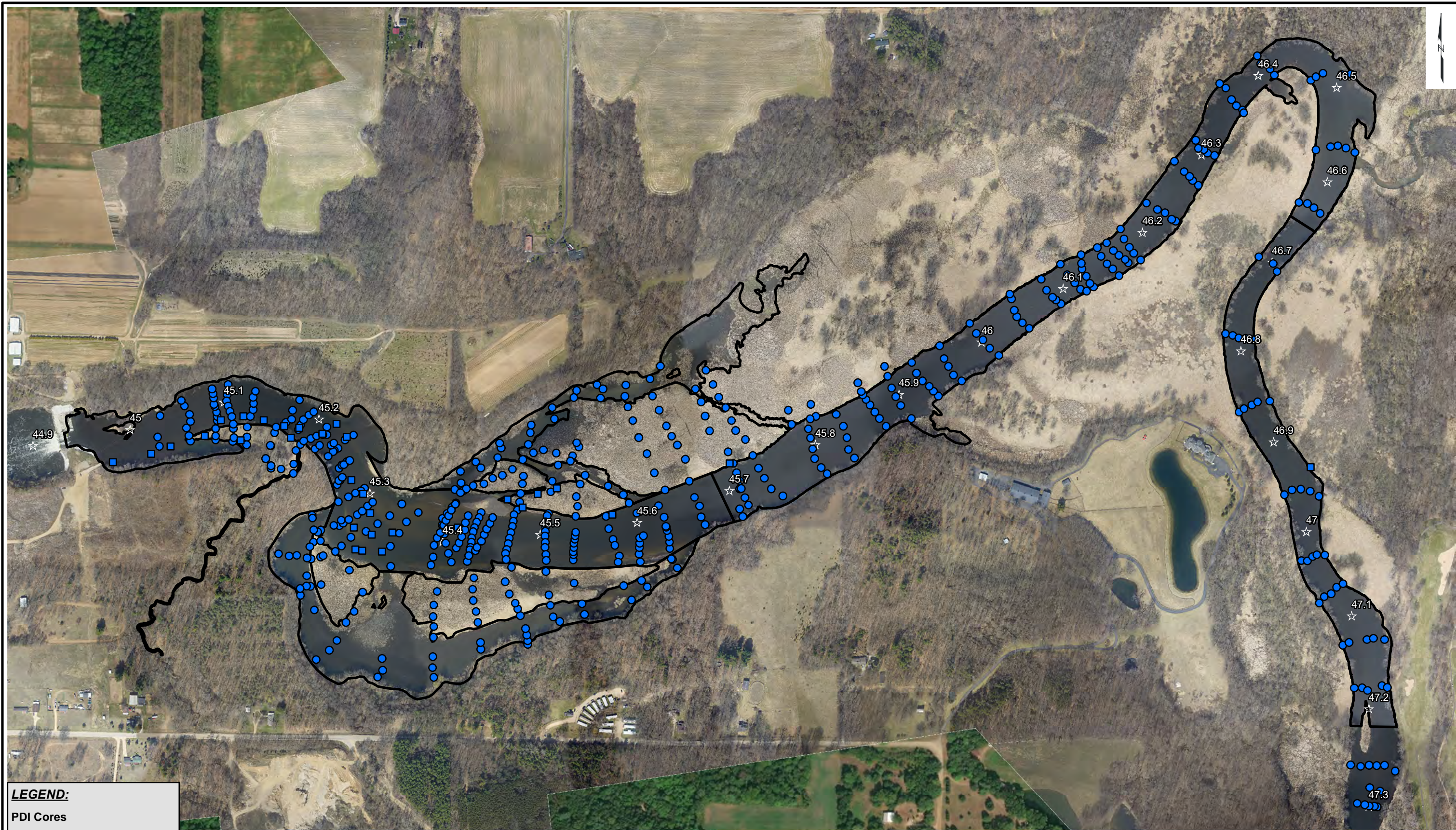


TRANSDUCER
LOCATIONS

Project 2000273

August 2022

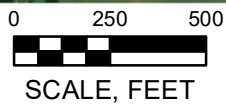
Fig. 5



LEGEND:

PDI Cores

- PDI Phase 2 poling at core location
- Poling location
- 1/10th mile marker
- Subarea boundary



Prepared by AG/Date: 08/30/2021
Checked by LN/Date: 08/3/2022

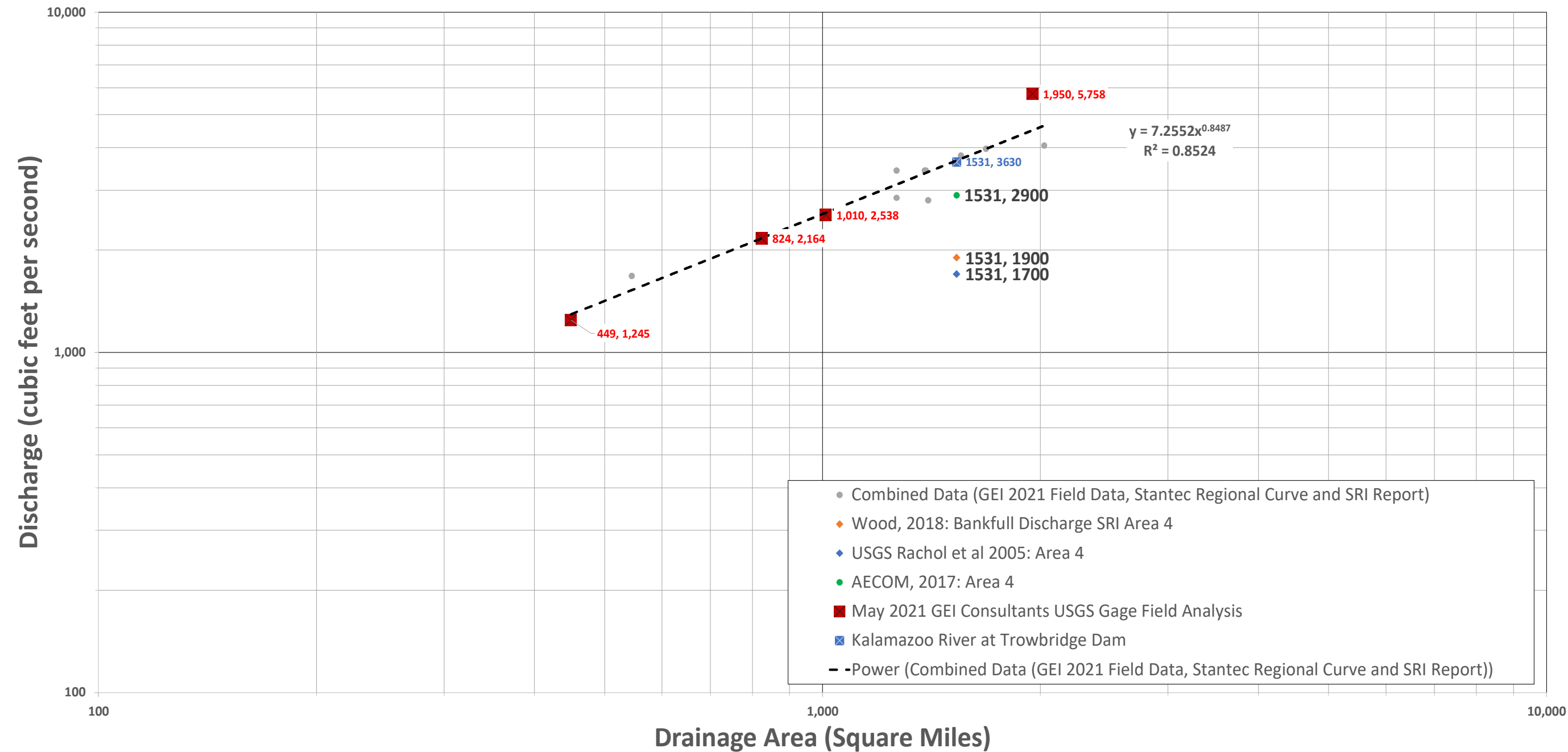
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

AREAS 2, 3 AND 4 TCRA

Project 2000273

AREA 4 PDI POLING LOCATIONS SUBAREAS C, D, E, F, & G
August 2022

Kalamazoo River Regional Curve - Bankfull Discharge



NOTES:
1. REGIONAL DISCHARGE CURVE BASED ON DESKTOP AND FIELD ANALYSIS OF THE KALAMAZOO RIVER AT DIFFERENT LOCATIONS
2. POWER FUNCTION BASED ON BOTH DESKTOP AND FIELD ANALYSIS USING LOG-LOG SCALE AND LINEAR FITTING OF DATA

Prepared by SP/Date: 11/1/2021
Checked by EG/Date: 8/4/2022


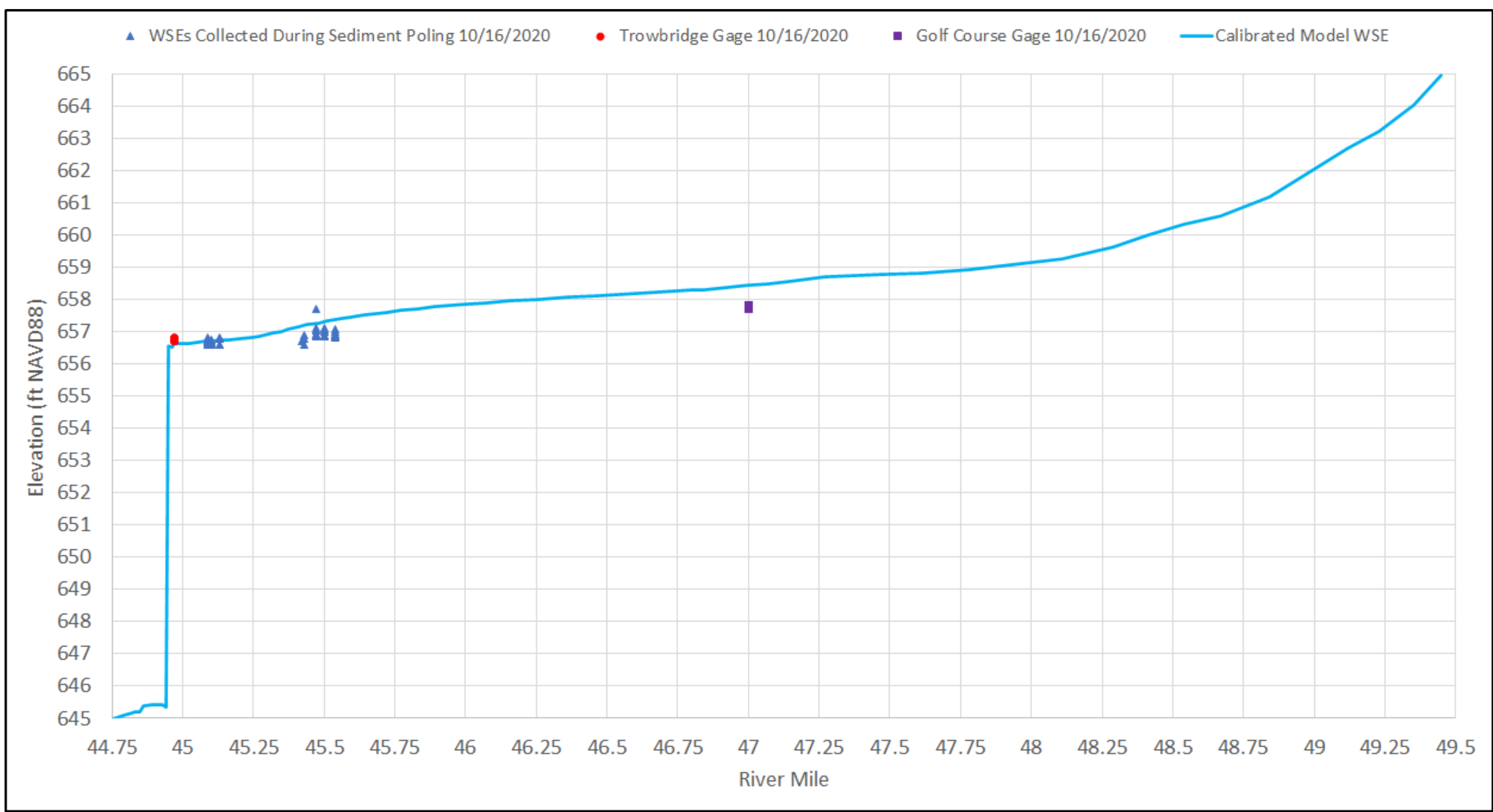
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		REGIONAL CURVE BANKFULL DISCHARGE ESTIMATION	
		Project 2000273	August 2022

Fig. 7



Prepared by LN/Date: 11/1/2021
Checked by EG/Date: 8/4/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

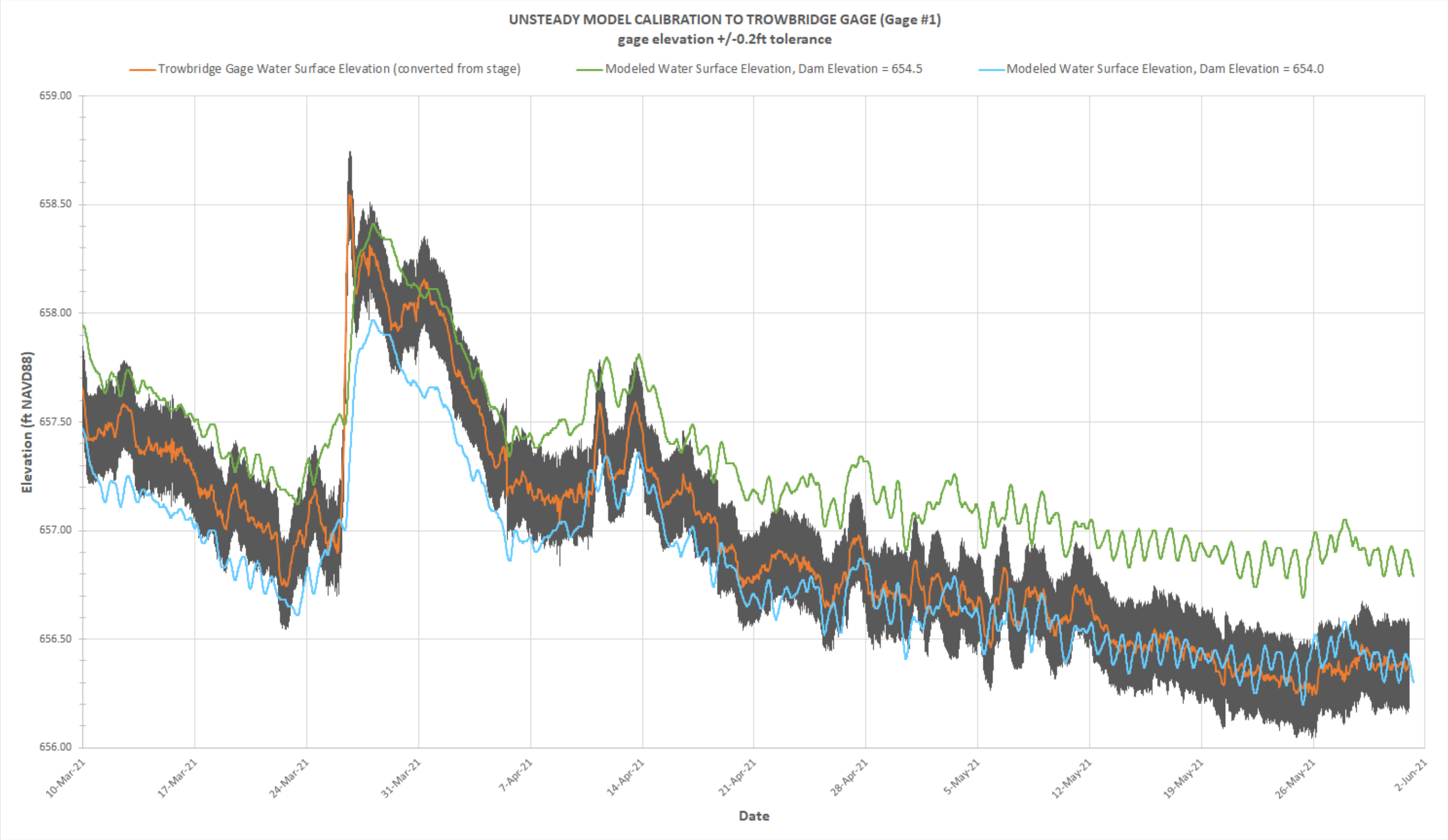


1-D MODEL CALIBRATION
PROFILE PLOT

Project 200273

August 2022

Fig. 8



Prepared by LN/Date: 11/1/2021
Checked by EG/Date: 8/4/2022


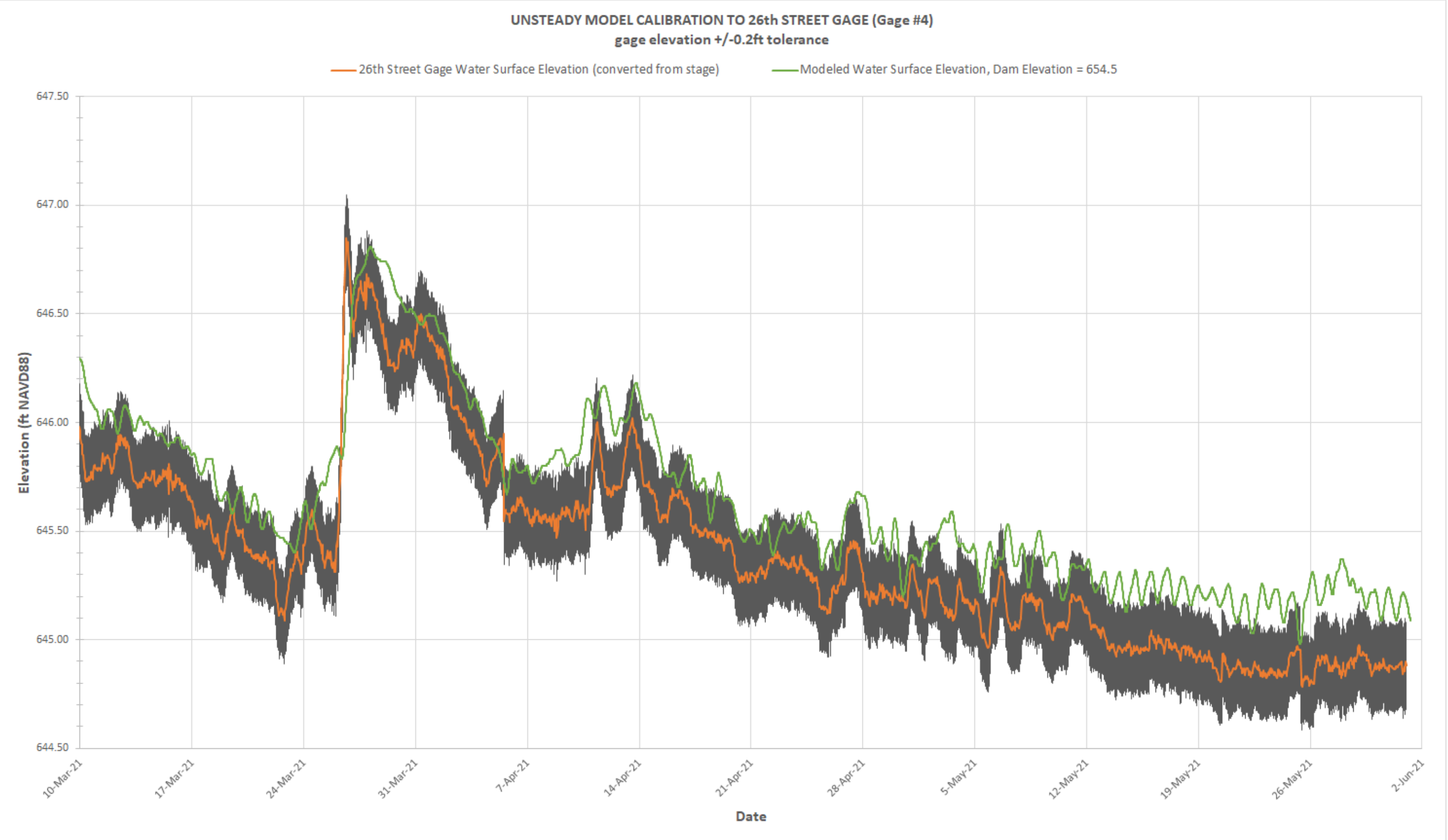
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	UNSTEADY MODEL CALIBRATION TO TROWBRIDGE GAGE	
		Project 2000273	August 2022

Fig. 9



Note: Modeled water surface elevation for downstream gage location is not sensitive to dam elevation

Prepared by LN/Date: 11/1/2021
Checked by EG/Date: 8/4/2022


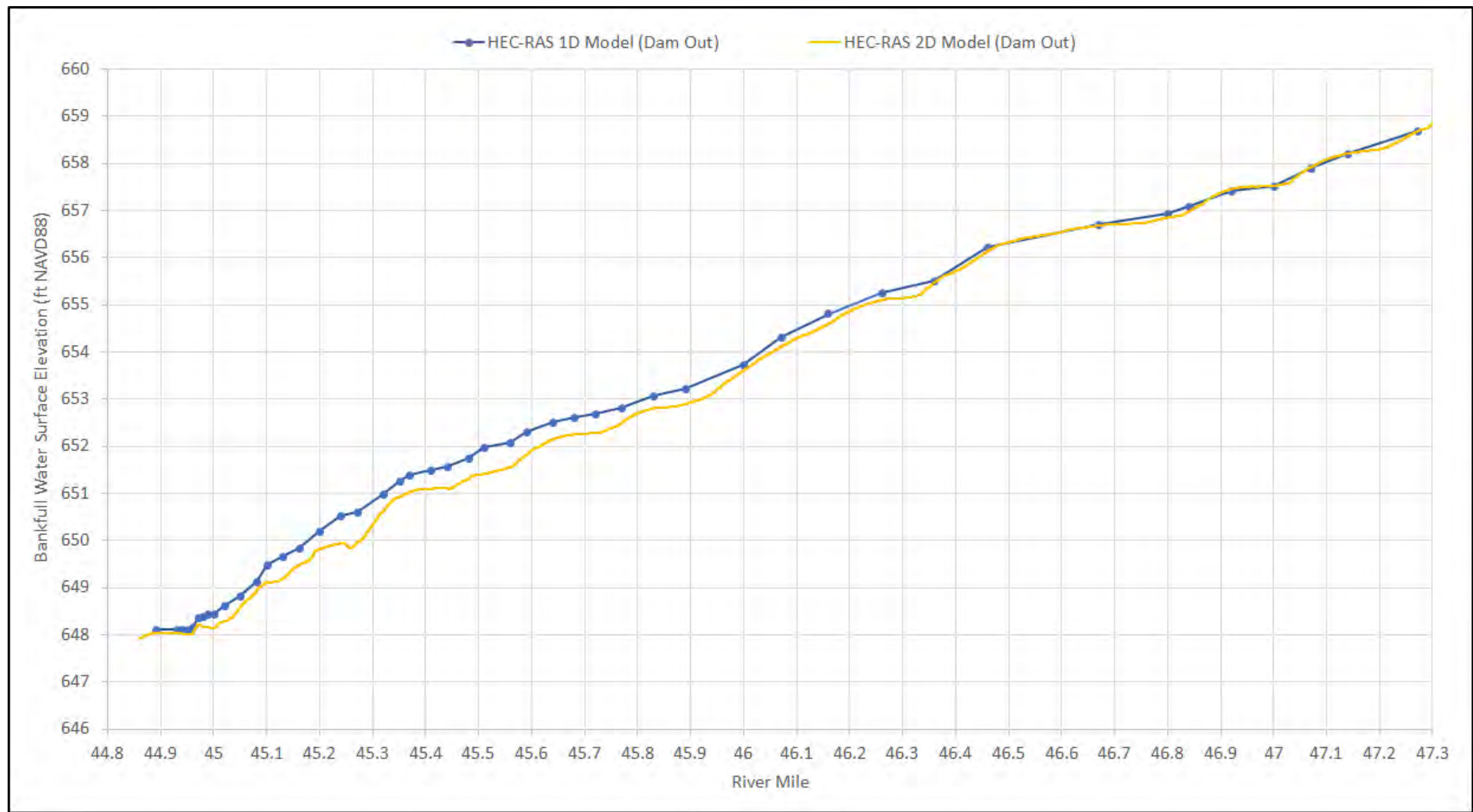
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	UNSTEADY MODEL CALIBRATION TO 26TH STREET GAGE	
		Project 2000273	August 2022

Fig. 10



NOTES:
 1. FLOW= 3,630 CFS

Prepared by EG/Date: 9/18/2024
 Checked by LN/Date: 9/19/2024


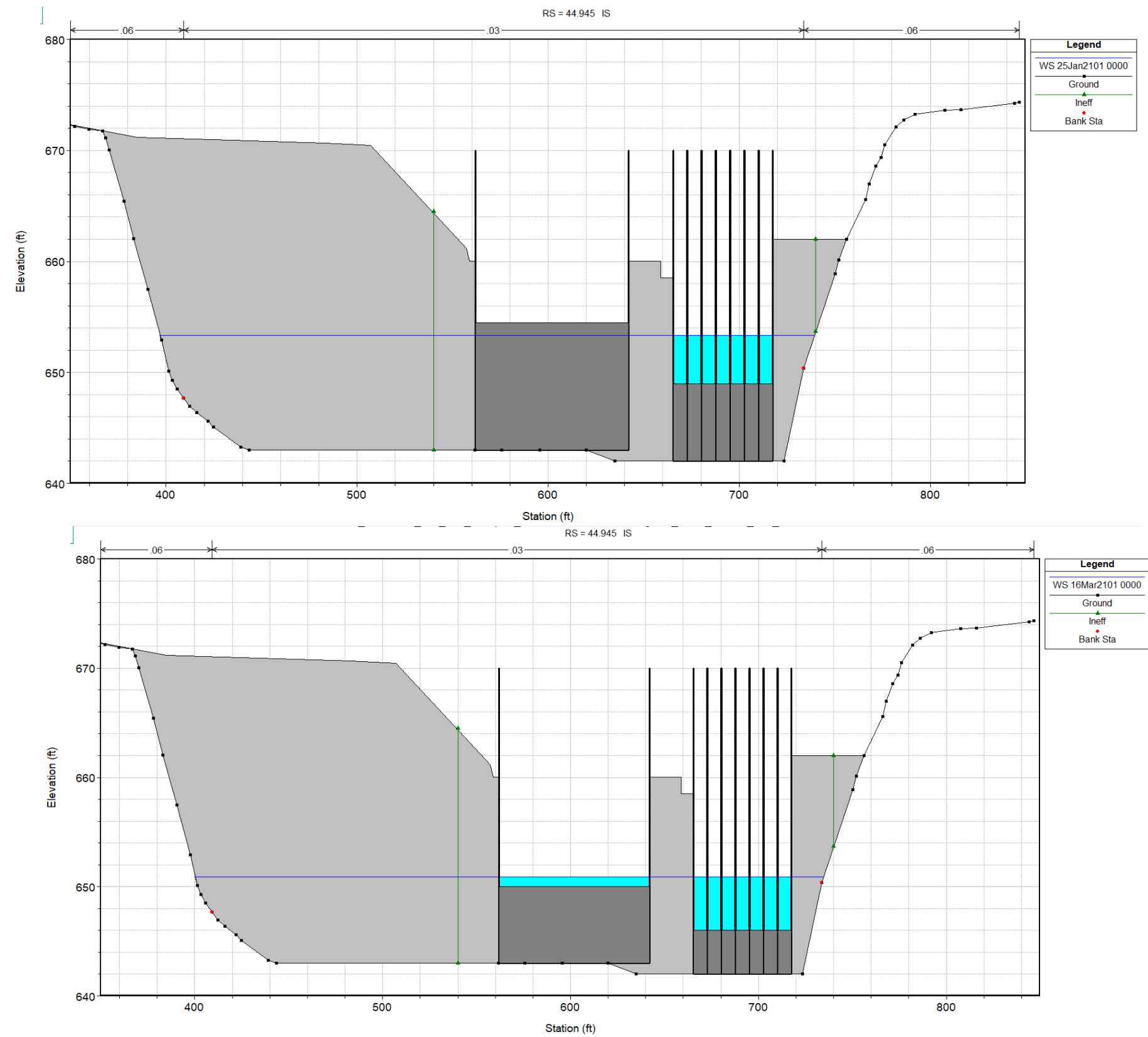
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		2-D COMPARISON PROFILE PLOT	
		Project 20002732	October 2024

Fig. 11



Prepared by SP/Date: 11/1/2021
Checked by EG/Date: 8/4/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

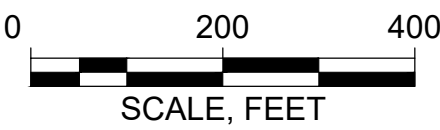
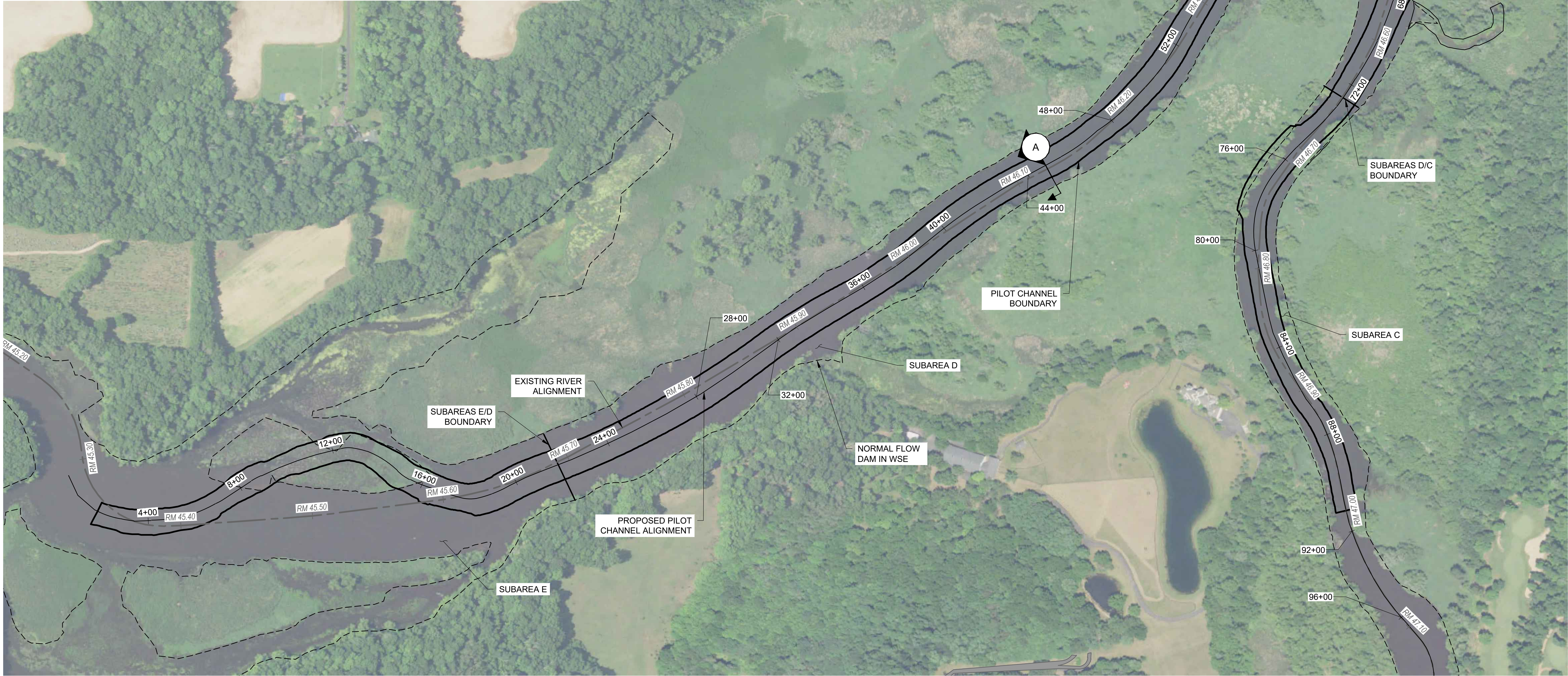
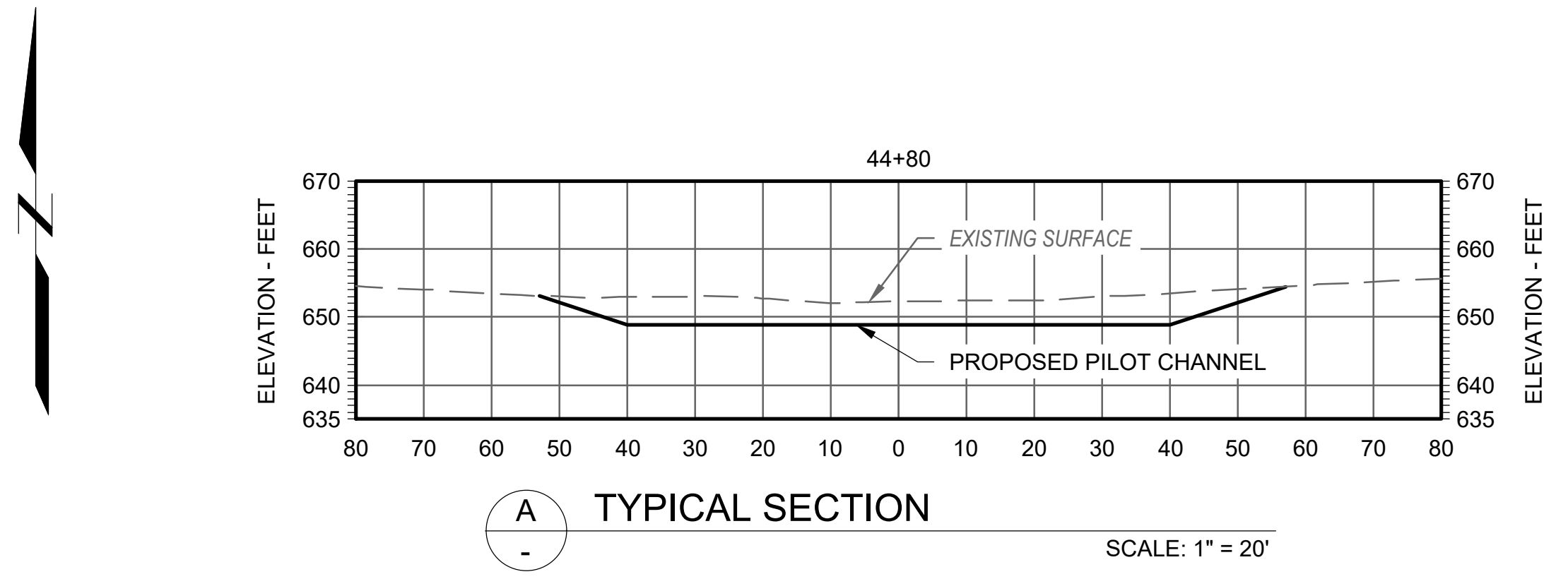





CROSS-SECTION OF
WATER CONTROL
STRUCTURE IN 1D MODEL

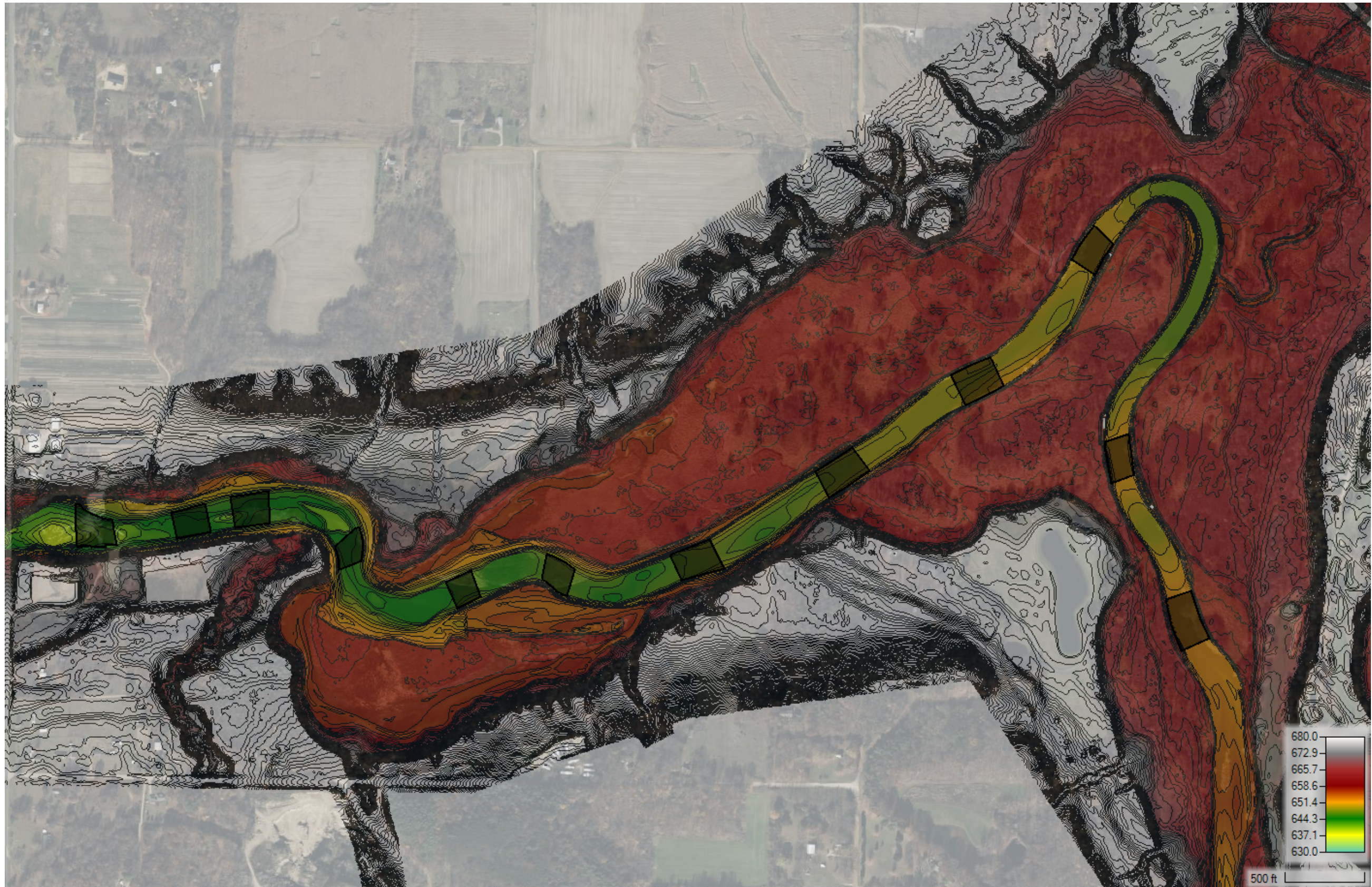
Project 2000273

August 2022

Fig. 12



<p>Attention:</p>  <p>If this scale bar does not measure 1" then drawing is not original scale.</p>		<p>Prepared by GS/Date: 6/8/2022 Checked by EG/Date: 7/8/2022</p>	 <p>GEI Consultants GEI CONSULTANTS OF MICHIGAN, P.C. 3065 AKERS MILL ROAD, SUITE 235 ATLANTA, GEORGIA 30339</p>	<p>August 2022</p>	<p>Kalamazoo River Areas 2, 3, and 4 Remediation LLC</p>  <p>AREAS 2, 3 AND 4 TCRA GEI Project 2000273</p>	<p>OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan</p>	<p>Fig. 13</p>
						<p>AREA 4 PLAN VIEW OF PILOT CHANNEL</p>	



PROPOSED, COARSENED RIFFLES

Prepared by EG/Date: 10/4/2024
 Checked by LN/Date: 10/4/2024

OU5 Allied Paper/Portage Creek/Kalamazoo River
 Superfund Site
 Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

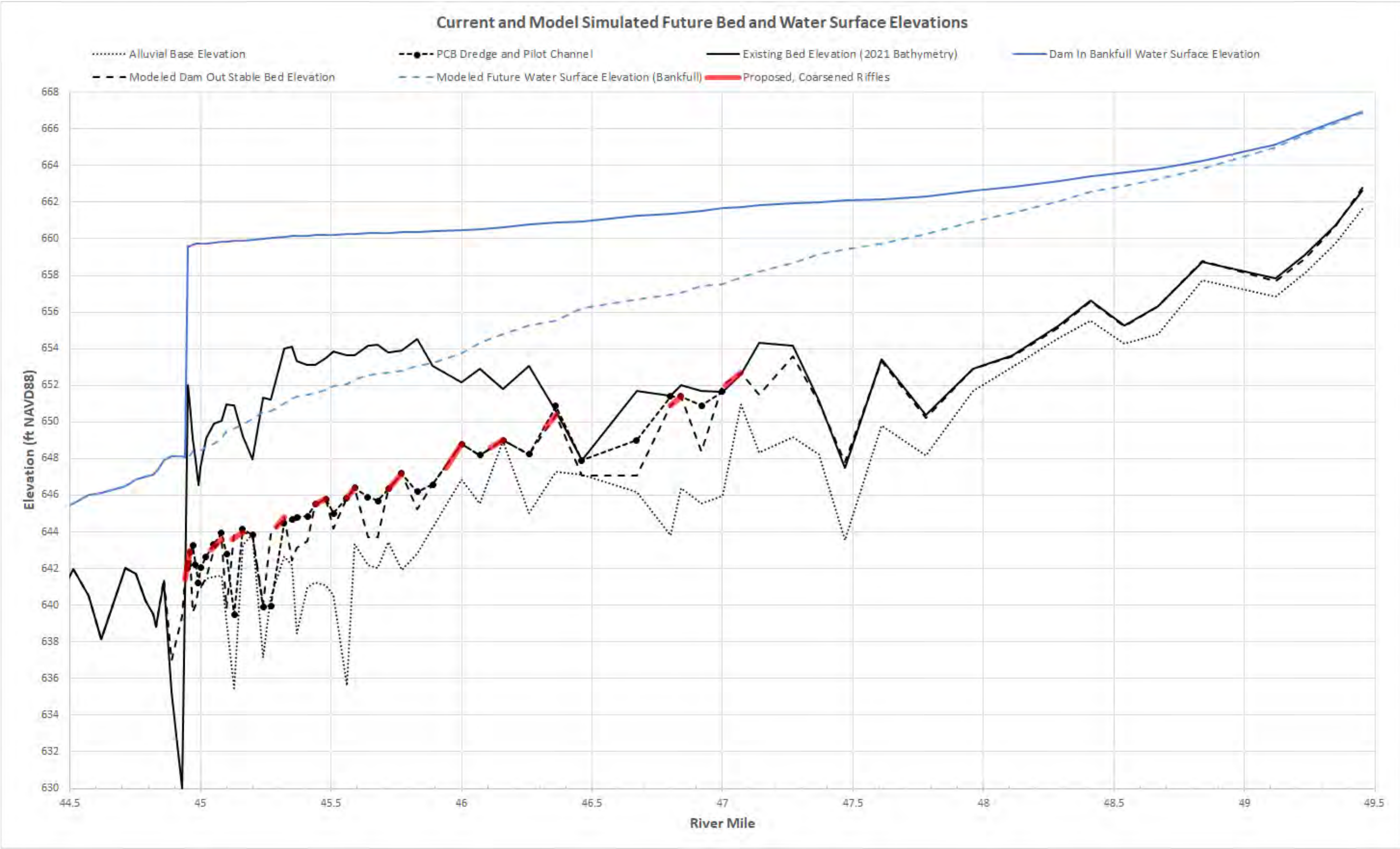


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AREA 4 PLAN VIEW OF
 ANTICIPATED CHANNEL
 ALIGNMENT AND PROPOSED,
 COARSENED RIFFLES

October 2024

Fig.14



NOTE: DAM OUT CONDITION MODELED USING REPRESENTATIVE 7-YEAR HYDROGRAPH

Prepared by EG/Date: 9/18/2024
Checked by LN/Date: 9/19/2024


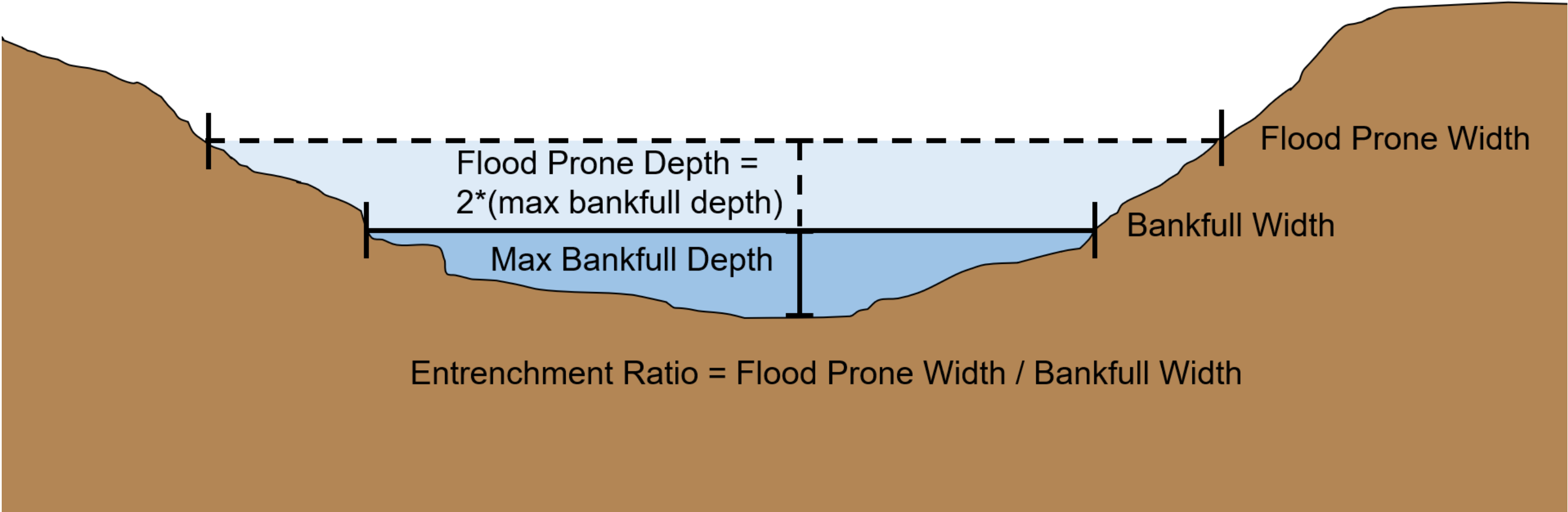

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	Existing and Simulated Bed Profile After Dam Removal
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	October 2024

Fig.15



Prepared by SM/Date: 11/1/2021
Checked by LN/Date: 8/4/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		ENTRENCHMENT RATIO	
		Project 2000273	August 2022

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

Fig. 16



LEGEND:

Depth Average Daily Flow
(ft)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 7
- 7 - 8+

0 230 460
SCALE, FEET

Prepared by LN/Date: 09/09/2024
Checked by EG/Date: 09/16/2024

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



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AREA 4 PLAN VIEW OF WATER
DEPTH FOR AVERAGE DAILY
FLOW IN MODELED ALIGNMENT

October 2024

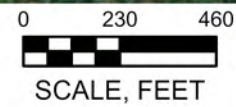
Fig. 17



LEGEND:

Depth Bankfull Flow (ft)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8+



Prepared by LN/Date: 09/10/2024
Checked by EG/Date: 09/16/2024

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

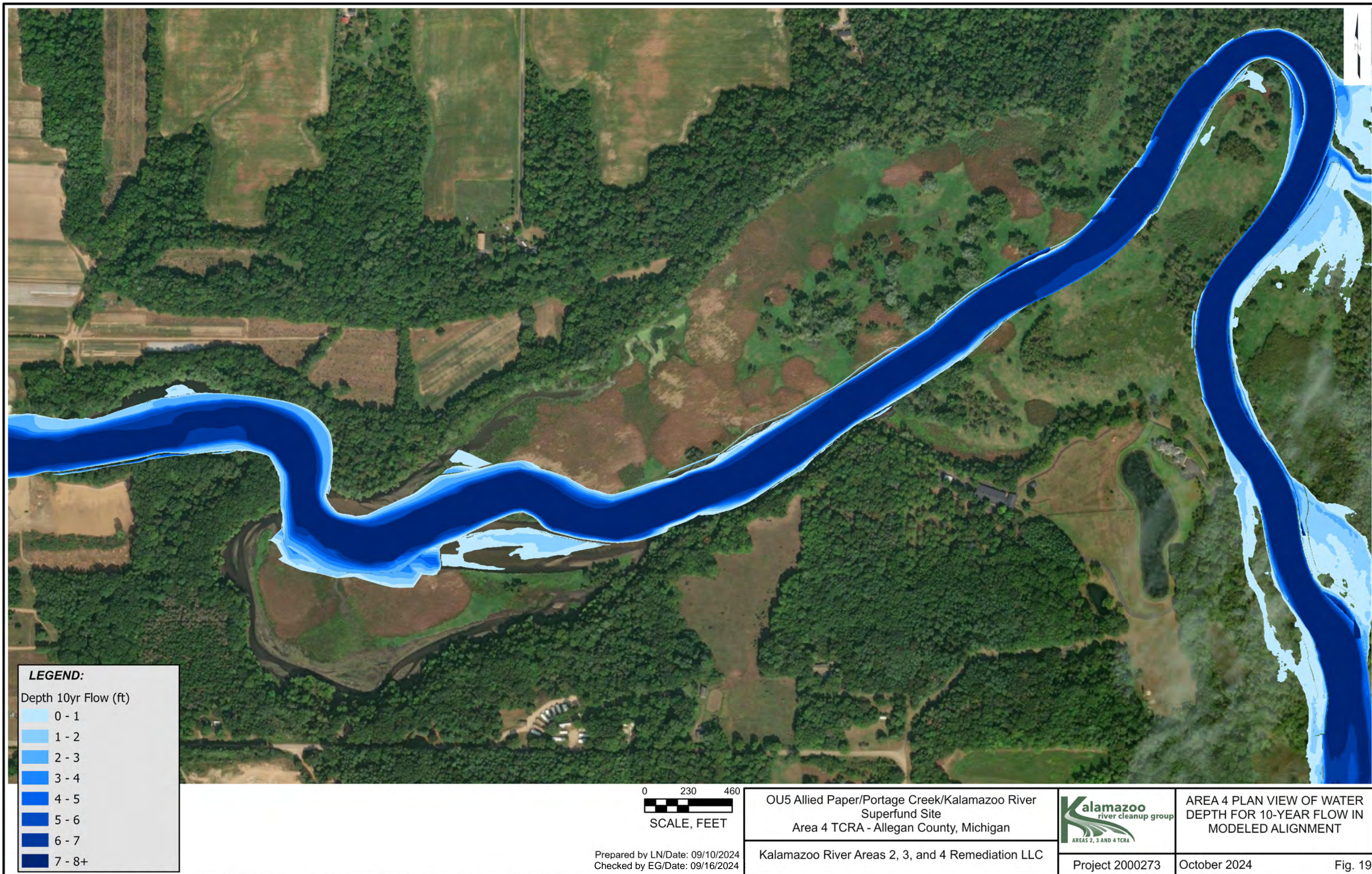


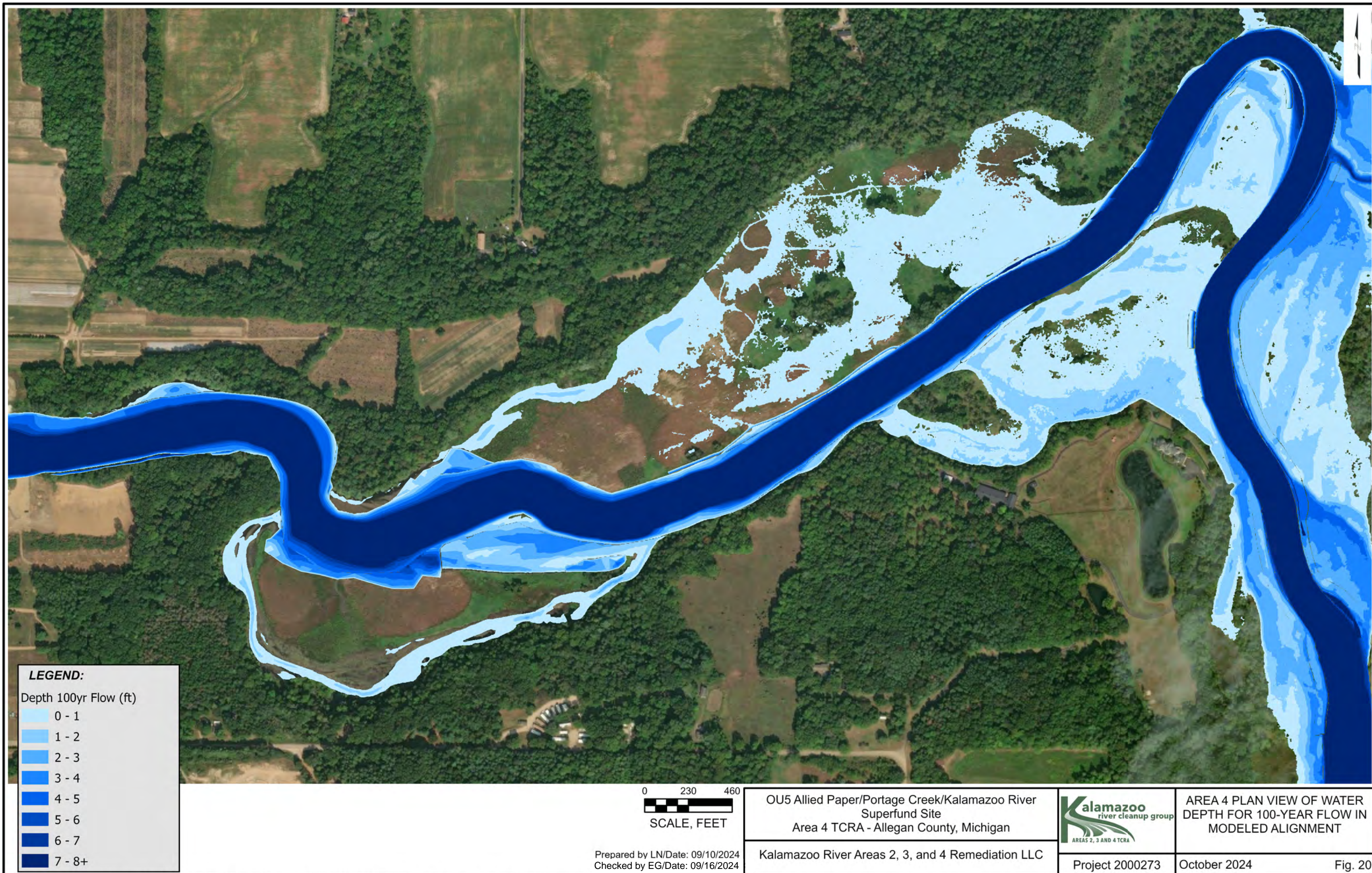
AREA 4 PLAN VIEW OF WATER
DEPTH FOR BANKFULL FLOW IN
MODELED ALIGNMENT

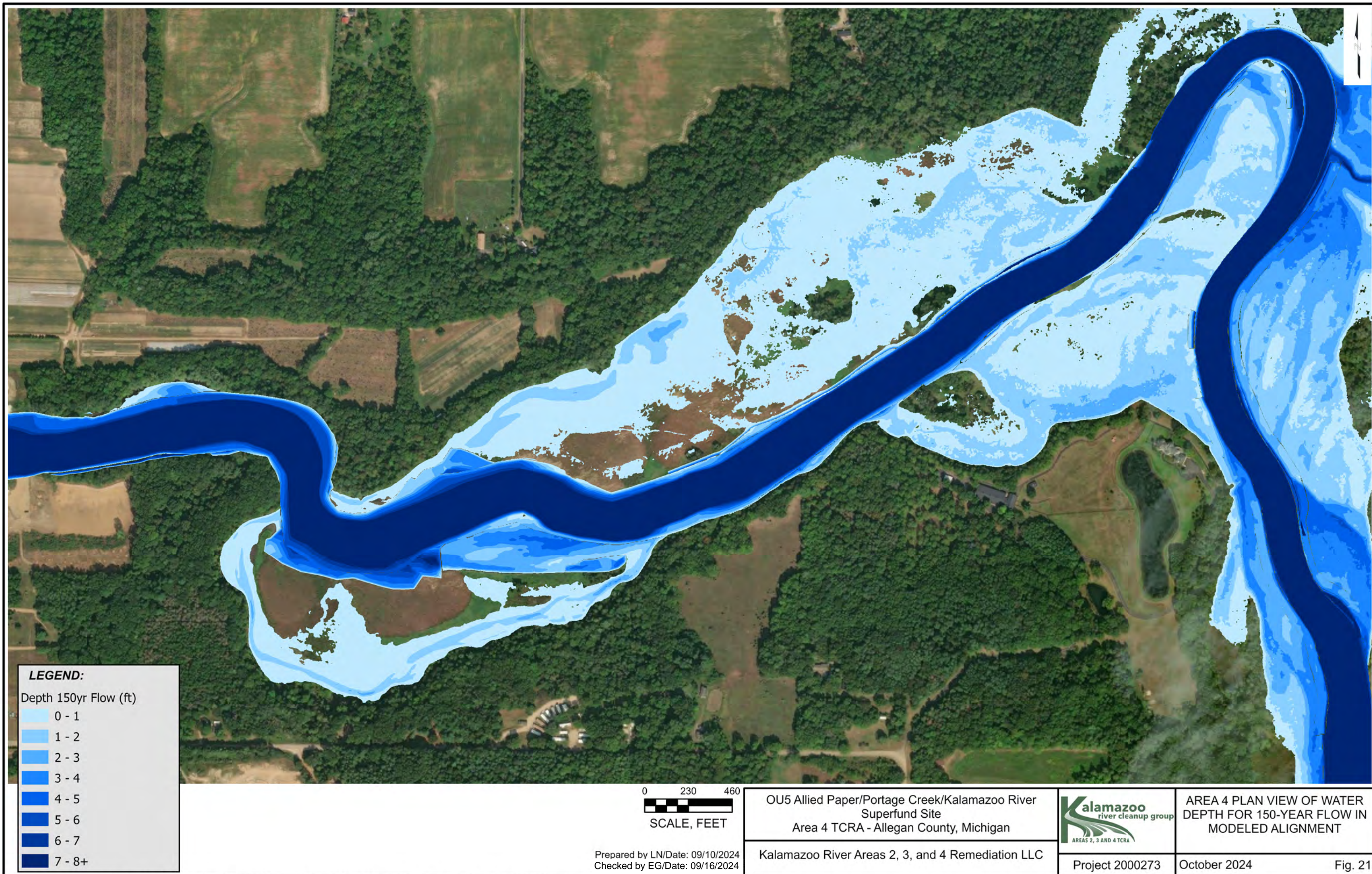
Project 2000273

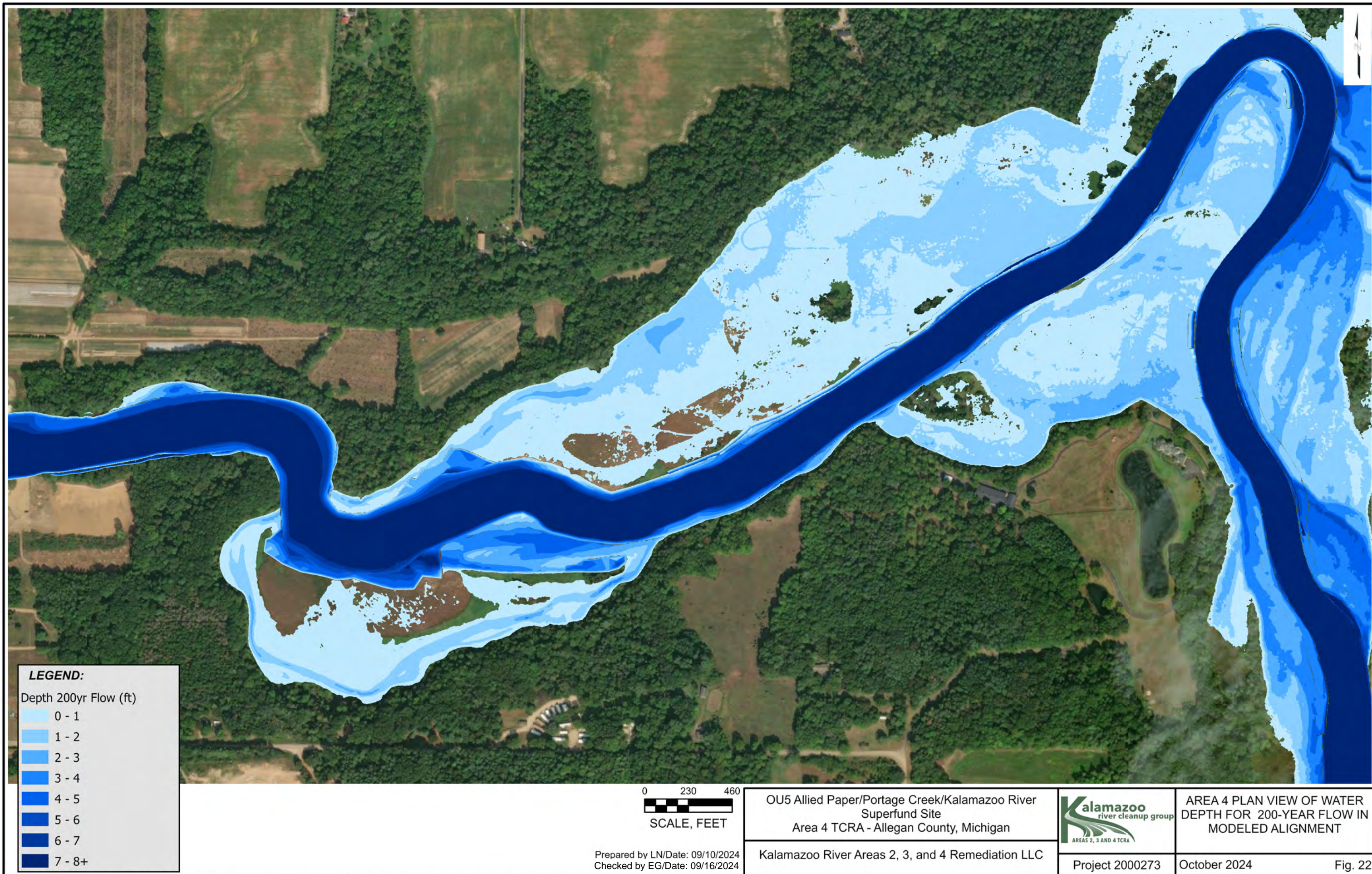
October 2024

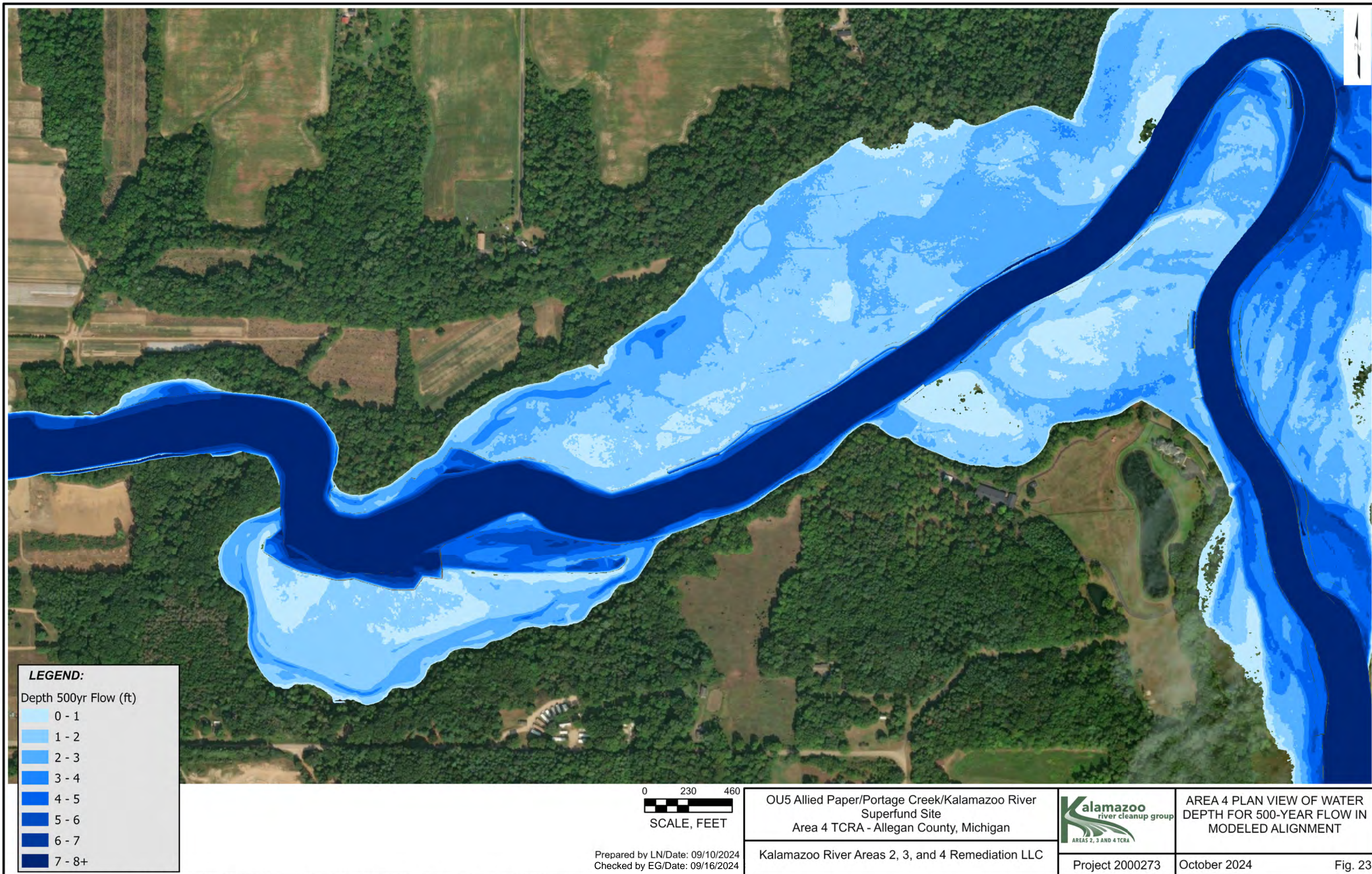
Fig. 18

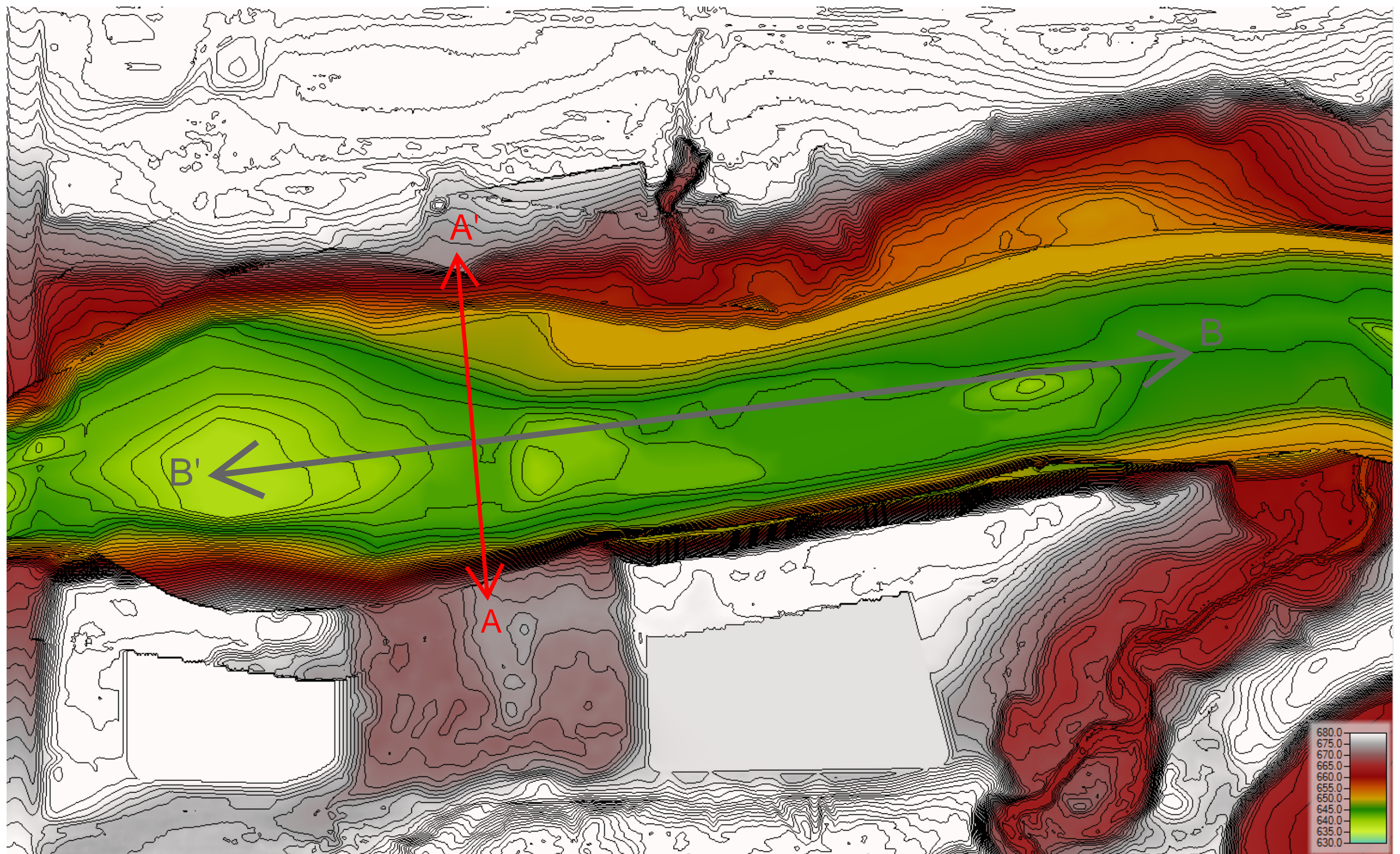












A \longleftrightarrow A' RIFFLE CREST CROSS SECTION LOCATION (FIG. 25)

B \longleftrightarrow B' RIFFLE PROFILE LOCATION (FIG. 26)

Prepared by EG/Date: 6/23/2022
Checked by LN/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

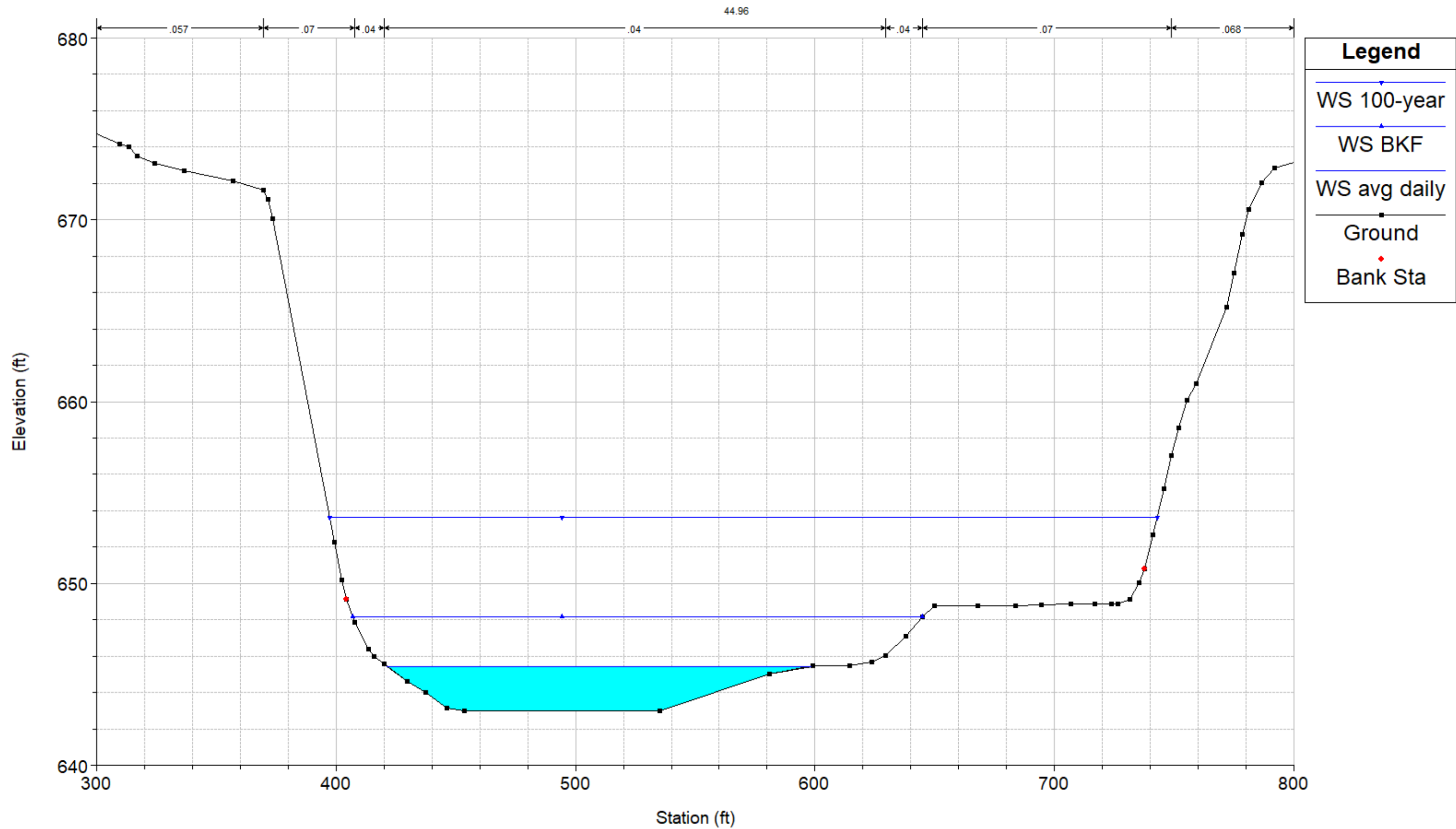


PROPOSED RIFFLE AT
TROWBRIDGE DAM
PLAN VIEW

Project 2000273

August 2022

Fig.24



NOTE: CROSS SECTION LOCATION IS SHOWN ON FIGURE 24 (A-A')

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Checked by LN/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan
Kalamazoo River Areas 2, 3, and 4 Remediation LLC



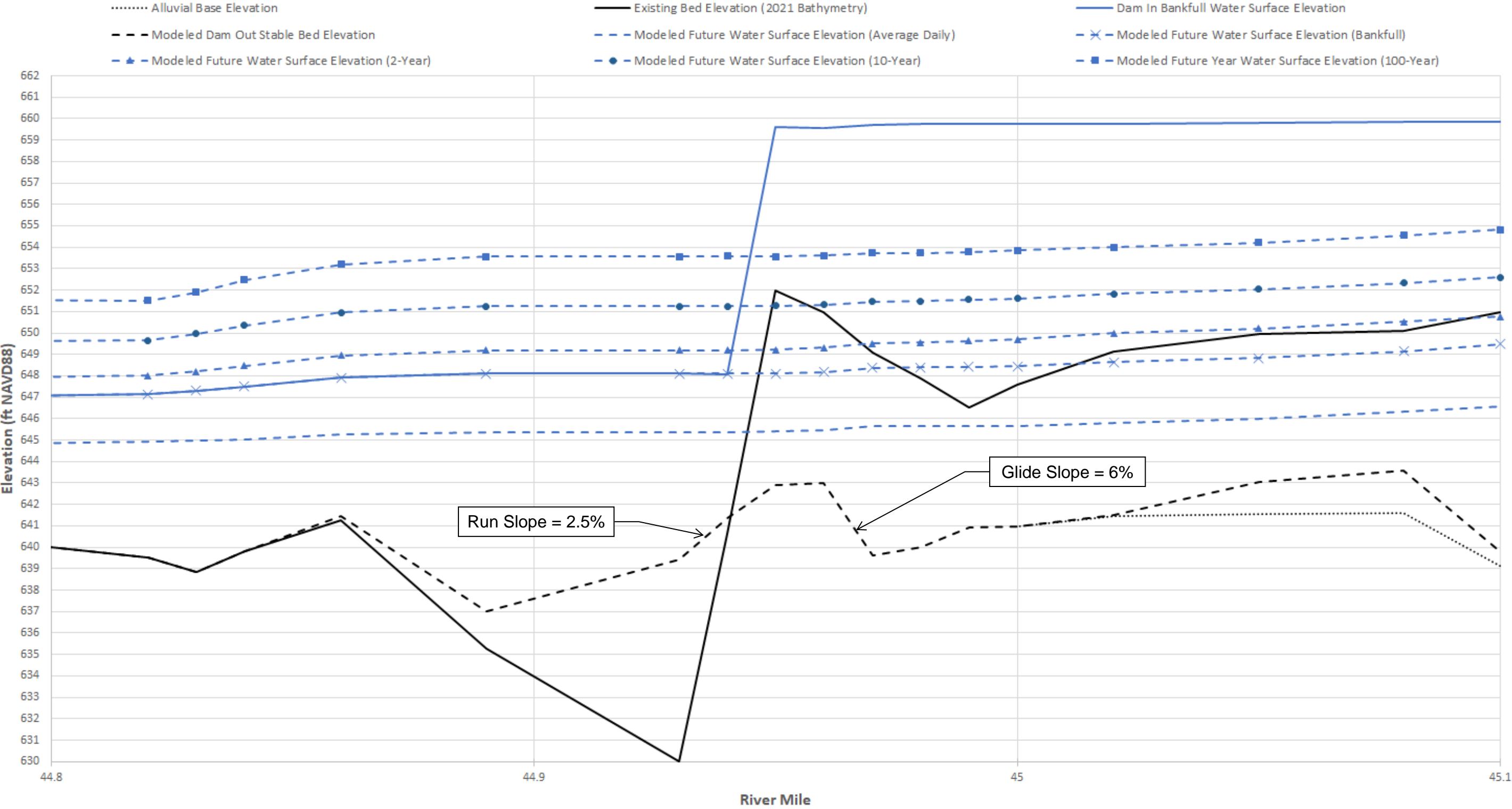
PROPOSED RIFFLE AT
TROWBRIDGE DAM
CROSS-SECTION AT CREST

Project 2000273

August 2022


Fig. 25

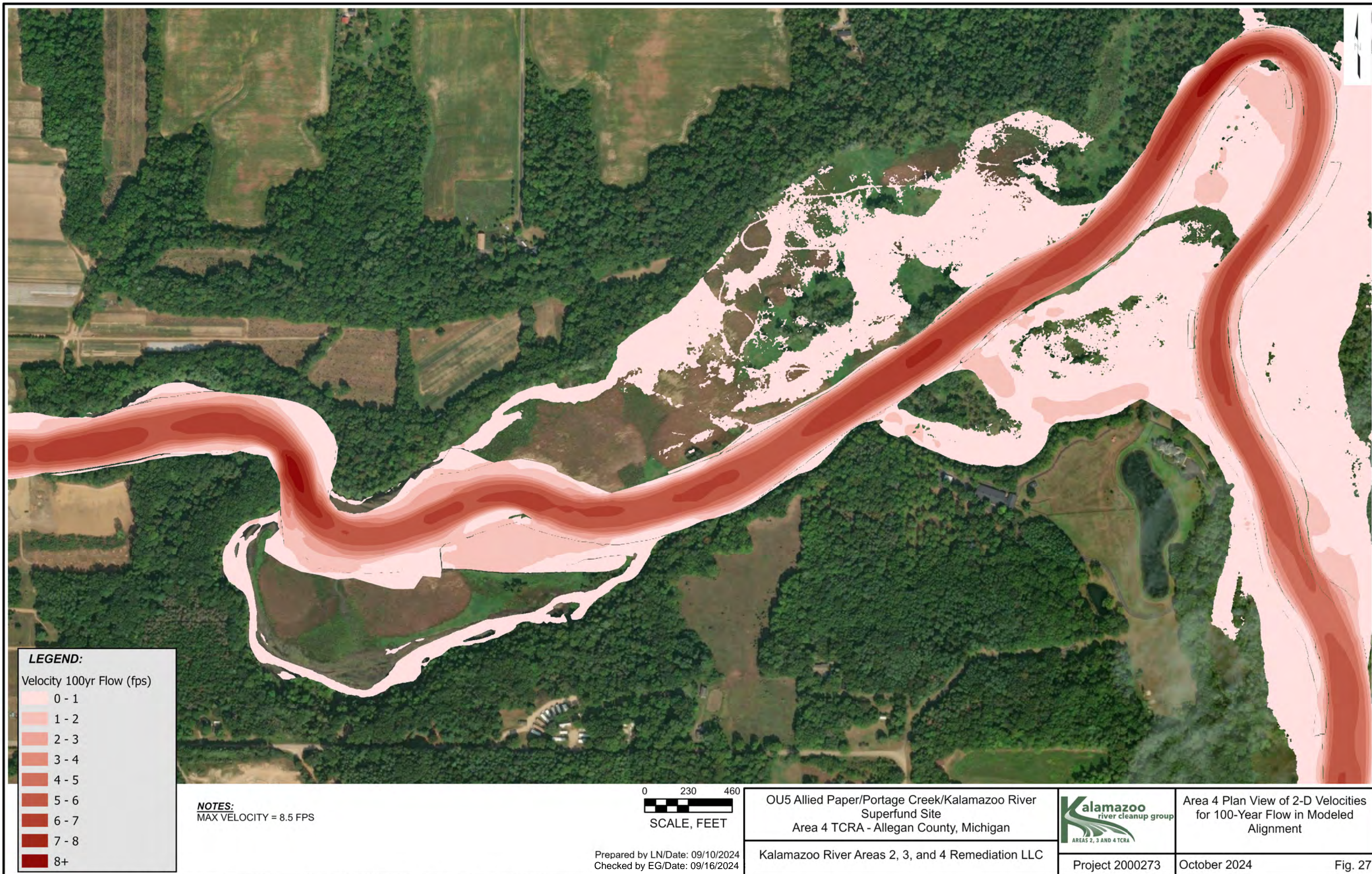
Current and Model Simulated Future Bed and Water Surface Elevations

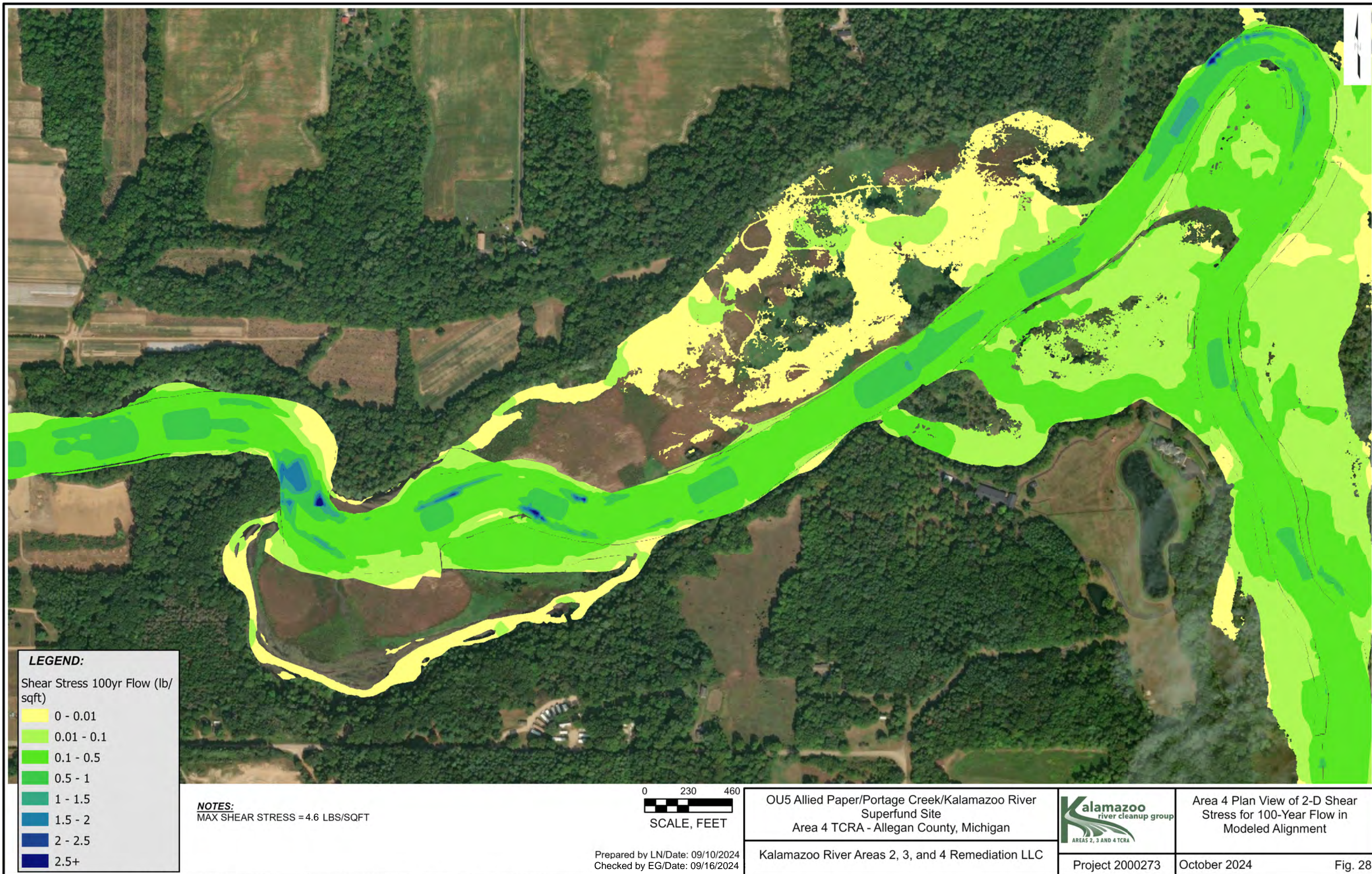


NOTE: PROFILE LOCATION IS SHOWN ON FIGURE 24 (B-B')

Prepared by EG/Date: 6/23/2022
Checked by LN/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	PROPOSED RIFFLE AT TROWBRIDGE DAM PROFILE
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	August 2022







LEGEND:

Velocity Bankfull Flow (fps)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8
8+

NOTES:
MAX VELOCITY = 5.8 FPS



Prepared by LN/Date: 09/16/2024
Checked by EG/Date: 09/17/2024

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



Area 4 Plan View of 2-D Velocities
for Bankfull Flow in Modeled
Alignment

Project 2000273

October 2024

Fig. 29



LEGEND:

Shear Stress Bankfull Flow
(lb/sqft)

0 - 0.01
0.01 - 0.1
0.1 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 2.5
2.5+

NOTES:
MAX SHEAR STRESS = 1.9 LBS/SQFT



Prepared by LN/Date: 09/16/2024
Checked by EG/Date: 09/17/2024

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

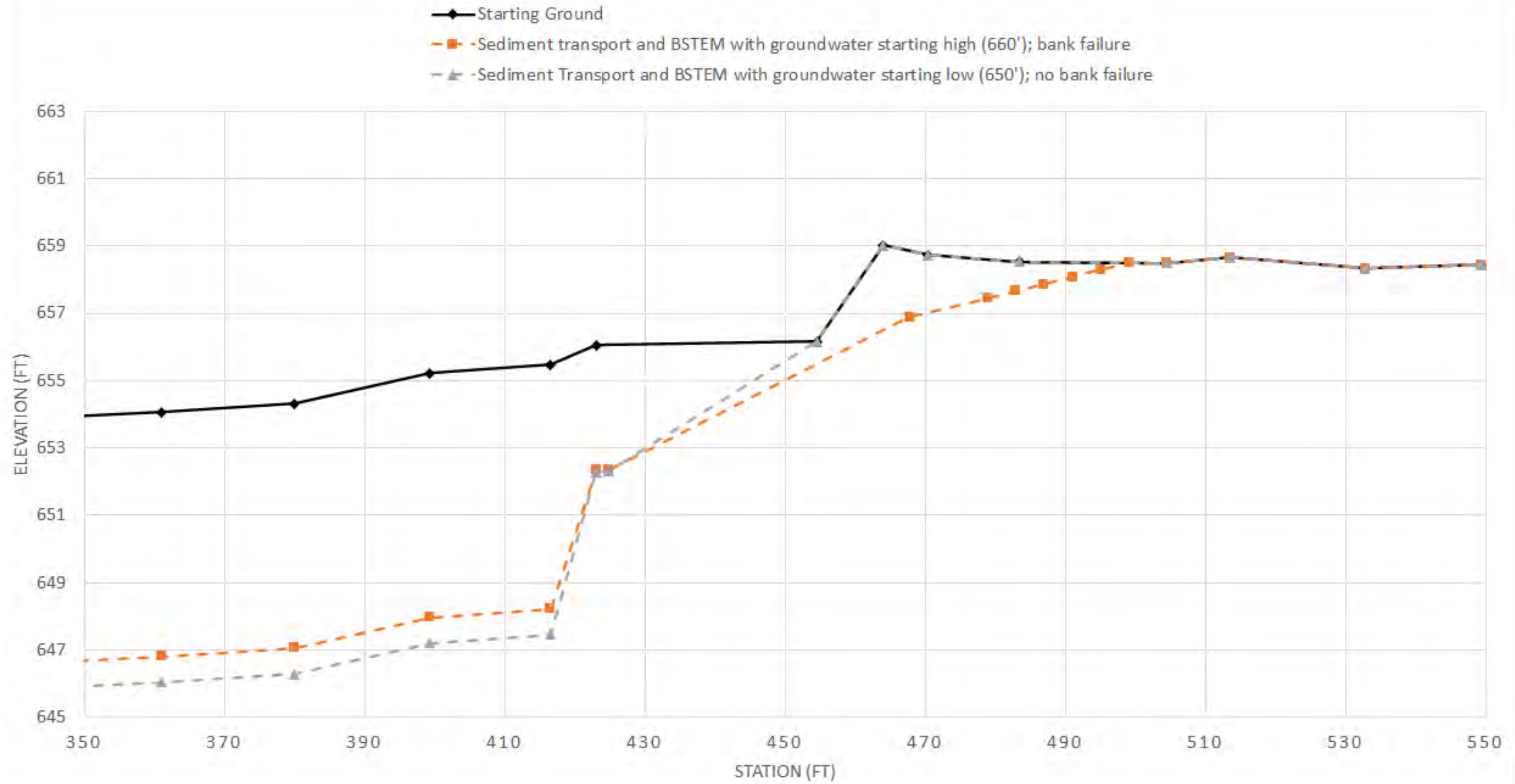
Kalamazoo River Areas 2, 3, and 4 Remediation LLC



Project 2000273

Area 4 Plan View of 2-D Shear
Stress for Bankfull Flow in Modeled
Alignment

October 2024 Fig. 30



Prepared by EG/Date: 11/1/2021
 Checked by LN/Date: 7/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
 Superfund Site
 Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



Project 2000273

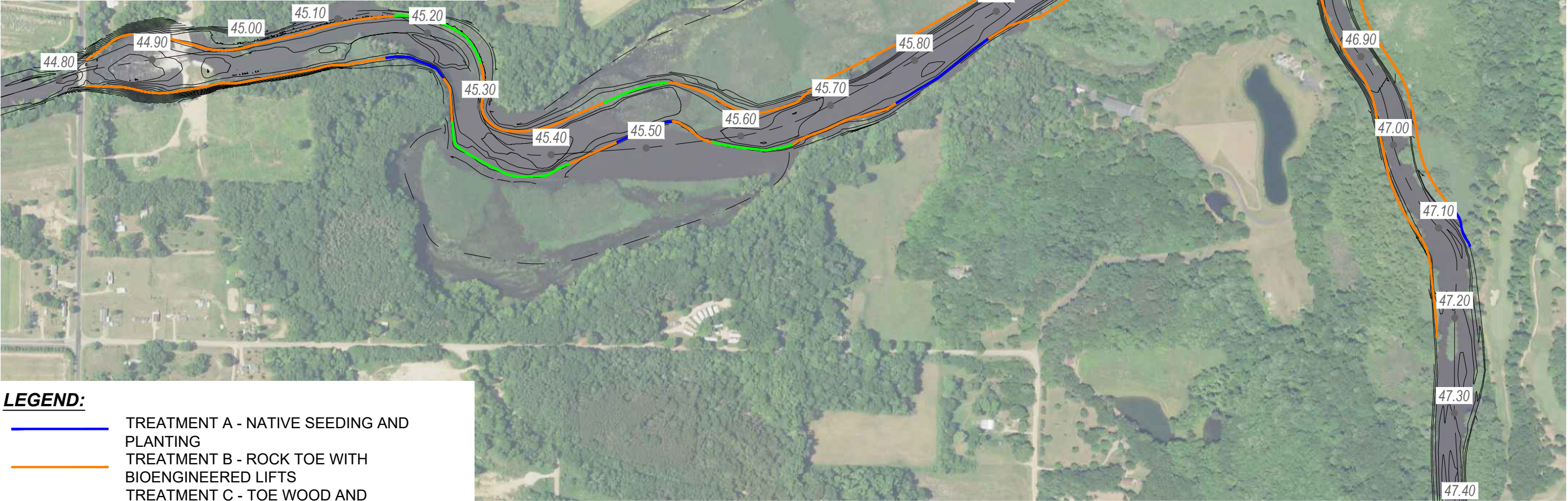
EXAMPLE CROSS-SECTION
 SHOWING SENSITIVITY
 ANALYSIS FOR ONE BANK

August 2022

Fig. 31

LDB RM station	Treatment	Dam Out avg. Reach Bankfull WSE
44.83-45.16	B	648.4
45.16-45.25	A	649.6
45.25-45.31	B	650.4
45.31-45.42	C	651.0
45.42-45.47	B	651.2
45.47-45.53	A	651.4
45.53-45.57	B	651.4
45.57-45.65	C	652.2
45.65-45.76	B	652.3
45.76-45.88	A	652.9
45.88-46.72	B	654.7
46.72-46.76	A	656.7
46.76-47.21	B	657.6

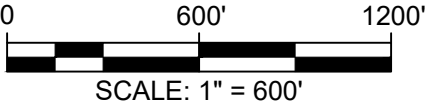
RDB RM station	Treatment	Dam Out avg. Reach Bankfull WSE
44.83-45.16	B	648.4
45.16-45.26	C	649.6
45.26-45.46	B	650.5
45.46-45.53	C	651.4
45.53-46.36	B	653.6
46.36-46.55	C	656.9
46.55-47.10	B	657.4
47.10-47.13	A	658.3



LEGEND:

- TREATMENT A - NATIVE SEEDING AND PLANTING
- TREATMENT B - ROCK TOE WITH BIOENGINEERED LIFTS
- TREATMENT C - TOE WOOD AND BIOENGINEERED LIFTS

Created by GS 9/25/24
 Checked by EG 9/25/24



OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		RIVER BANK TREATMENT PLAN	
		Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273 September 2024 Fig. 32

Attachment A

Area 4 TCRA Hydrology, Hydraulics, and Sediment Transport Modeling Technical Memorandum

Memo

To: Brian Kelly, EPA
From: Roger Hathaway, PE, GEI
CC: Amber Ahles, PMP, GEI
Date: 10/23/2024
Re: Area 4 TCRA Hydrology, Hydraulics, and Sediment Transport Modeling Technical Memo
Kalamazoo River Superfund Site, Operable Unit 5 Area 4 TCRA

Modeling Approach

General Approach

The goals for hydrologic/hydraulic/sediment transport modeling in Area 4, the reach that includes Trowbridge Dam were: 1) to understand the existing and future trends and patterns of sediment movement and hydraulics in Area 4, and 2) to inform Trowbridge Dam removal and channel restoration design options and scenarios. To understand the impacts of Trowbridge Dam removal in Area 4, observed changes and modeled forecasts for the completed Time Critical Removal Action (TCRA) work in Area 3 were compared and analyzed. Area 3 encompassed the upstream reach that included the former Otsego Township Dam prior to its removal. This analysis was used to estimate the incoming sediment load from Area 3 to Area 4 and inform current conditions and potential dam-out conditions in Area 4. The Area 3 modeling evaluation demonstrates that the models are robust tools for forecasting the influence of dredging and dam removal on bed- and water-level lowering in Area 4.

Results of the Area 3 TCRA provide both a relevant model of potential dam-out conditions for Area 4 and a sediment-loading curve from Area 3 into Area 4 for use in sediment transport calibration. A physical model obtained from a dam removal in the same river and adjacent reach is rare and having a well-documented removal process, such as that conducted in Area 3, is unique and of use to the work in Area 4. Simulating Area 3 both for dam-in and dam-out conditions allowed for the development of reasonably constrained boundary conditions for the Area 4 sediment transport model calibration.

Model calibration provided the framework to demonstrate that the models were appropriate for their intended use. Current conditions were modeled in Area 4 and calibrated to data collected by GEI as well as data collected by others during previous assessments. As new and more detailed data became available, such as Area 4 bathymetry collected in 2020 and in 2021, the current-conditions model was refined and recalibrated.

The calibrated one-dimensional (1-D) current-conditions model was used as a basis to develop the dam-out model (i.e., future-conditions model) to demonstrate how PCB removal, dam removal, and channel restoration design alternatives would influence river hydraulics and sediment transport. Future- and current-condition models underwent a comprehensive sensitivity analysis as they were developed and refined to understand the influence of input parameters on model results. Sensitivity analyses were included as part of model calibration

and during the development of design scenarios. Design scenarios were evaluated using the current-conditions model as a base. Model parameters and geometry were modified to reflect the conceptual designs for PCB dredging, pilot channel construction, targeted channel widening, dam removal, and subsequent constructed grade control riffles, temporary installation of a water control structure (WCS), and for bank and channel stabilization design. Several iterations were evaluated in 1-D as the design evolved. Two-dimensional (2-D) hydrodynamic modeling was used to evaluate the hydraulics with finer spatial resolution in two dimensions. Following discussions with EPA on June 5, 2024, 2-D modeling was used to evaluate targeted channel widening and targeted riffle crest lowering in Subareas C and D (EPA, 2024). Model plans are listed in Table 1.

North American Vertical Datum of 1988 (NAVD88) is the reference datum for this memo unless otherwise noted. To convert from National Geodetic Vertical Datum of 1929, subtract 0.44 feet (NOAA NCAT).

Hydraulics and Hydrology

GEI developed and calibrated a robust 1-D hydraulic model to confirm that model inputs including upstream flows, boundary conditions, model computation routines, and channel and overbank characteristics were appropriate for the intended model use of evaluating and informing design options. The models were developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 5.0.7, (the current version of the software at the time this evaluation commenced).

Developing the 1-D hydraulic models to flow and sediment transport conditions – and calibrating the models to conditions in Area 3 for pre- and post-dam removal – provided key information about the influence of input parameters on model results and helped develop and refine the Area 4 models. Sensitivity analysis allowed for “testing” the influence of parameter inputs and associated datasets for both static and dynamic (i.e., change over time and space) model inputs. Hydraulic models for Areas 3 and 4 were developed using robust datasets for input parameters, including channel and floodplain terrain and bathymetry (i.e., model geometry), stream flow, sediment particle size distributions, historical alluvial surface stratigraphy, and Manning’s n-values (i.e., model roughness coefficients). Models previously developed for the supplemental remedial investigations (SRIs) in Areas 3 and 4 were also analyzed and the relevant portions incorporated in this work (Amec, 2016; Amec, 2018; Wood, 2019).

Sediment Transport

GEI performed 1-D sediment transport modeling on the Kalamazoo River in Areas 3 and 4 HEC-RAS version 5.0.7, (the current version of the software at the time this evaluation commenced). This modeling was performed to understand areas of sediment erosion, deposition, and accretion along the reach, as well as the influence of dredge design and dam removal on the transport of sediment in these reaches. Sediment transport modeling is most efficient in 1-D because the software program can run multi-year, continuous simulations with short run times. The 1-D modeling provided the capability to run simulations over multiple years until sediment transport dynamic equilibrium was reached and stable bed and water surface elevations (WSEs) were achieved.

Sediment transport modeling is standard practice for understanding the movement of sediment along river reaches. Due to the inherent uncertainty of predicting future flows and representing

the complex nature of bed sediment and other river characteristics, sediment transport modeling is best used for evaluating trends and patterns rather than for predicting a specific outcome at a specific location in time. For this project, part of the design is based on the predicted pre-dam channel characteristics (i.e., overall channel slope) in the river, intended design modifications (i.e., channel geometry and bank and bed stabilization techniques), and the predicted bed differentiation that may be realized as the current water levels are lowered.

Sediment transport depends on the sediment's physical characteristics (e.g., size, weight, density, etc.) and is driven by hydraulic factors such as flows, velocities, and applied shear stresses, which are a function of flow events, the watershed, channel bed slope, and river channel geometry. Possible uncertainties associated with sediment transport modeling include but are not limited to, incoming sediment loads from upstream sources (total mass, timing of delivery and particle-size distributions), variation in bed sediment grain-size distribution, erodible limits, bed sorting and erosion calculations, and localized hydraulics. These uncertainties are due partly to the potential for wide variability in sediment and hydraulic characteristics within a reach and partly due to the general lack of readily available data, such as upstream sediment loading. Even with good-quality field data, the full heterogeneity of the incoming load, the channel and effected overbank regions are difficult to fully simulate in any model development project.

Calibrating a sediment transport model is important for reducing uncertainty and for demonstrating that a given model is a reasonable characterization of observed conditions. Sediment transport models for Areas 3 and 4 were calibrated by simulating a time period that was bracketed with channel bathymetry or cross-section data to compare the model's simulated (i.e., forecasted) channel changes and net sediment movement with the observed (i.e., measured) changes. Calibration of the sediment transport models in Areas 3 and 4 included robust sensitivity analyses to understand the influence of parameter inputs on model results.

The Bank Stability and Toe Erosion Model (BSTEM), a module within HEC-RAS, was applied to the Area 3 sediment transport model to help with the Area 4 calibration. In particular, it was used to simulate the pilot channel widening that occurred after it was created in Area 3.

Current and Historical Conditions

Area 3: Current-Conditions Hydraulic Models

The Area 3 current-conditions model was developed to represent observed conditions in 2020, 2 years after the Area 3 TCRA was completed. The current-conditions model was calibrated to the suite of Area 3 dam-in and dam-out hydraulic, sediment transport, and BSTEM models (covered in Appendix B) based on observed conditions. This model was used to inform the calibration needs for modeling hydraulics and sediment transport in Area 4. Area 3 was modeled exclusively using HEC-RAS 1-D by GEI.

Area 3: Bathymetry, Topography, and Channel Geometry

The Area 3 model extends from the M-89 Bridge crossing to the former Otsego Township Dam location about 1.7 river miles downstream. Model cross sections were numbered in river miles (RMs) representing miles upstream of Lake Michigan. To represent the channel and upland elevations, bathymetry data collected in 2020 (Affiliated Researchers, 2020) and Light Detection and Ranging (LiDAR) data collected in 2015 (NOAA, 2015) were combined and used to define model river and floodplain cross sections in HEC-RAS. This combined model terrain retained the left embankment at the former Otsego Township Dam location that was present in 2015 and

was modified further using as-built drawings to represent the terrain as it was constructed in 2018 (Wood, 2019). Cross-section locations for the Area 3 models are shown in Fig. 1.

Due to unsafe conditions, as-built elevations at the constructed riffle at the former Otsego Township Dam location were not collected during the 2020 bathymetric survey. The Area 3 TCRA reports the riffle crest at 666 feet NGVD29 (665.6 feet NAVD88) (Wood, 2019). Through field observations by GEI, this elevation was estimated to be the elevation at the top of the installed rocks and boulders in the constructed riffle rather than the bottom invert of the channel. GEI field personnel located the bottom of the channel at the riffle crest to be approximately 663 feet NAVD88. This value was used along with a 2% downstream slope as reported in the Area 3 TCRA to define the bottom of the riffle in the model (Wood, 2019).

United States Geological Survey (USGS) computed Manning's n-values for the channel in Area 3 ranging from 0.02 to 0.04 (Syed and Bennet, 2005). The SRI used these same values for dam-in conditions and used a constant channel value of 0.04 for dam-out conditions (Amec, 2016). Manning's n-values were set to 0.03 in the main channel and 0.12 for floodplain regions in the GEI current-conditions model. Manning's n-values for the floodplain regions were determined using USGS guidance (USGS, 1989). The constructed riffle at the former Otsego Township Dam was the only exception, with a channel Manning's n-value of 0.04 to represent the additional roughness due to the large boulders at the riffle.

Area 3: Boundary Conditions

The upstream boundary condition for the Area 3 current-conditions model was set to critical depth in the steady-state simulations. Incoming flows for specified flow events were taken from the SRI (Amec, 2016). The downstream boundary condition was set to normal depth with slope = 0.001 feet/feet based on the 2020 bathymetry downstream of the former Otsego Township Dam and slope gradients developed by Michigan Department of Natural Resources (MDNR) (DNR, 2005).

To verify Area 3 model inputs and to calibrate the Area 3 current-conditions model, GEI performed flow measurements in Area 3 and in Pine Creek by taking depth-appropriate (i.e., six-tenths or two-point) velocity readings using a top-setting wading rod at set intervals across the channel in accordance with USGS stream gaging techniques for current-meter measurements by wading (Buchanan and Somers, 1969). These velocity measurements were used to calculate the observed flow at the time of measurement. Observed flows were used to validate model boundary conditions and velocity outputs. On June 9, July 30, and August 25, 2020, flow measurements in Pine Creek were 87.5 cubic feet per second (cfs), 54.6 cfs, and 41.7 cfs, respectively. On July 30, 2020, a flow of 1,086 cfs was measured in the Kalamazoo River just upstream of Pine Creek. These flow measurements were performed once on each day of observation as spot checks.

These observed flows were compared with estimated discharges from USGS gages and based on drainage-area ratios (DARs) for Pine Creek and the Area 3 M-89 Bridge crossing. The DARs used the drainage area of the USGS Comstock gage and the drainage area of Pine Creek and Area 3 as determined by Michigan Department of Environment, Great Lakes, and Energy (EGLE), for flood-flow analysis. The DAR for Pine Creek is 0.089 and the DARs for Area 3 are 1.37 at M-89 Bridge crossing and 1.47 at the former Otsego Township Dam. USGS recommends using DAR only when the ratio of the ungaged area is within 0.5 to 1.5 of the gaged area (Ries et al., 2008). The DAR for Pine Creek (0.089) is outside of the USGS recommended ratio range but was used due to a lack of continuous gage data required for unsteady simulations. Use of the Pine Creek DAR was verified by comparing the measured

discharges with DAR estimated discharges for the same time of day. The DAR-estimated discharges for Pine Creek were 15–19% higher than the measured discharge. The discharge measured for the Kalamazoo River was 9% higher than the DAR-estimated value. The flows used for steady-flow simulations are listed in Table 2.

To verify the current-conditions Area 3 hydraulic model, average measured velocity was compared to the model-simulated average velocity at the nearest cross sections. The average velocity of the measured discharge in the Kalamazoo River for the July 30, 2020, observation was 2.5 feet per second (fps). The model-estimated average velocity for the cross sections closest to the discharge measurement ranged from 2.0 fps to 2.4 fps.

Area 3: Hydraulic Model Calibration

The Area 3 current-conditions model was calibrated by comparing model-predicted and observed WSEs collected by Affiliated Researchers during their bathymetric survey on April 28, 2020. A steady flow value of 2,000 cfs upstream of Pine Creek for April 28, 2020, was developed using the flow recorded at the USGS Comstock stream gage on the same day and adjusting for drainage area. The DAR from the Comstock gage location to the M-89 Bridge crossing is approximately 1.37. A lateral flow for Pine Creek of 130 cfs was developed and entered in the model using the same method, with a DAR of 0.089.

Modeled WSEs were generally within ± 0.5 feet of the observed and recorded WSEs for Area 3, as shown in Fig. 2. A tolerance of ± 0.5 feet was considered acceptable due to some imprecision in the upstream flow estimate and some gaps in bathymetry data, such as the apparent thalweg elevations at the constructed riffle located at the former Otsego Dam location, the “narrows” section from RM 50.10 to RM 50.15, and the exposed till area located at RM 50.25. All these areas were not fully captured in the bathymetric survey completed in 2020 due to unsafe working conditions caused by high velocities. Data for these areas were approximated in the model geometry using neighboring cross section data.

Area 3: Historical-Conditions Hydraulics and Sediment Transport Model

The selection of the simulation period for the Area 3 sediment transport calibration was partly driven by the need to create an upstream sediment-load boundary condition for Area 4. Because the Area 4 bathymetric datasets used to calibrate the Area 4 sediment transport model were collected in 2013 and 2020, the model simulation period in Area 3 was also set to bracket those periods. However, the dam-in bathymetric dataset in Area 3 was collected in 2016. It was assumed that the former Otsego Township Dam impoundment was in dynamic sediment equilibrium and the 2016 bathymetric data were not significantly different than the bathymetry in 2013 in terms of representing the total volume of impounded sediment due to the former Otsego Township Dam acting as grade control. The Otsego Township Dam had been in place for more than a century before its complete removal in 2018 (Amec, 2016). Considering this length of time and the relative size of Otsego Township Dam and its impoundment, significant loss of reservoir capacity had likely occurred by 2013 due to sedimentation behind the dam structure (Kondolf et al., 2014). The Otsego City Dam, located approximately 3.4 miles upstream of the Otsego Township Dam, had been in place for approximately 60 years prior to the construction of the Otsego Township Dam (Amec, 2016). The presence of Otsego City Dam caused a reduction in the sediment supply in the Area 3 reach, which also contributed to sediment volumes in the Otsego Township Dam impoundment remaining relatively controlled.

Area 3: Dam-In and Post-Water Control Geometries

Two historical-conditions model geometries were developed to calibrate sediment transport in Area 3. The dam-in geometry used bathymetry data collected in 2016 and represents conditions from 2013 through 2018 when the Otsego Township Dam and WCS were in place. The dam-out geometry represents the conditions after May 2018 when the dam and the WCS were removed. These two geometries were used together to simulate sediment transport over a continuous period from 2013 to 2020 in Area 3 and to develop an upstream sediment rating curve for Area 4. The Area 3 model used quasi-unsteady flows with sediment loads across this period. The dam was represented in the model as an inline structure with a spillway elevation of 671.45 feet NAVD29, as reported in the Area 3 SRI report (Amec, 2016).

The Area 3 post-WCS geometry represented the conditions in May 2018 after the dam and WCS were removed, the riffle at the former dam location was constructed, and a pilot channel was dredged. The restoration actions were represented in this geometry by adding a 60-foot-wide pilot channel with invert elevation of 655 feet to the model cross sections from RM 50.0 to RM 49.7. The inline WCS structure was removed, and the riffle crest set to 663 feet with a 2% downstream slope, consistent with the current-conditions model. It was assumed that the riffle has not changed configuration since construction and the riffle substrate has been stable.

Area 3: Boundary Conditions and Hydrology

Daily mean flows from the USGS Comstock gage from May 2013 to May 2020 were used to develop the inflows at the upstream boundary of the sediment transport model. The flows from the Comstock gage were prorated using a DAR of 1.37 for the upstream boundary near the M-89 Bridge crossing and a DAR of 0.089 for the Pine Creek tributary. This hydrograph, shown in Fig. 3, contained several larger flow events including two 2-year annual recurrence interval events and one 10-year or greater annual recurrence interval event before the WCS was removed, and three 2-year annual recurrence interval events after the WCS was removed and the constructed riffle installed. Quasi-unsteady models allow the user to specify smaller computation increments at high flows (when more sediment is generally transported if supply allows) and larger computation increments at low flows. A sensitivity analysis was performed on the computation increment for this hydrograph by reducing the computational increments and observing the resulting modeled bed change and sediment results with all other variables held constant. The computational increments were reduced until no major changes in cross-section geometry or longitudinal cumulative volume change were observed between sensitivity analysis model runs. Through this process, computation increment was determined to be a low-sensitivity parameter. Computation increments for the Area 3 sediment transport model are provided in Table 3.

Flow data in HEC-RAS quasi-unsteady simulations require water temperature data to compute water density. Water temperature was derived from mean daily air temperatures recorded at the Kalamazoo Battle Creek International Airport (Morrill et al., 2005; Stefan and Preud'homme, 1993). A sensitivity analysis of water temperatures was also performed using a constant water temperature in comparison to the fluctuating water temperature derived from the air temperature readings. This sensitivity analysis indicated that varying water temperatures did not significantly influence the results as indicated by minimal changes (less than 0.1 feet) in water surface elevations when all other variables were held constant.

Area 3: Bed Gradation and Erodible Limits

Initial conditions for bed gradations and their locations for the Area 3 models were taken directly from the Area 3 SRI report except for the constructed riffle, which had a gradation based on field estimates and observed coarse material. Observations of substrate type in other locations in Area 3 were also considered for the post-WCS geometry. Photos of exposed glacial till and surface substrate taken in 2020 are provided in Figs. 4a and 4b. Bed gradations are shown in Table 4.

The 1-D sediment transport model simulated potential channel erosion and deposition within user-defined horizontal and vertical limits in each cross section. The horizontal limits for erosion were set as the toe of each bank and horizontal deposition was allowed to extend onto the floodplain. The vertical limits to erosion were informed by sediment depths ranging from 2 feet upstream of Pine Creek to 8 feet near the dam collected by USGS in the Otsego Township Dam impoundment in 2000 (Rheaume et al., 2002). A conservative erodible limit of 10 feet was used at most cross sections. The exceptions to this were at the glacial till near RM 50.2–50.3 and at the former dam location where the channel has large cobbles and boulders at the surface. At RM 50.2–50.3 vertical erosion was limited to 0.5 feet; at the constructed riffle vertical erosion was not allowed based on the observation that the riffle was constructed to withstand erosion for the flow events analyzed. The vertical erodible limits for the Area 3 model are shown in Table 5.

Erodible limits went through a series of sensitivity analyses and were determined to be a sensitive model parameter, with bed profiles and longitudinal cumulative bed change volumes varying significantly after a change in erodible limits at one cross section. Bed profiles showing an example of the sensitivity associated with the erodible limits analysis are shown in Fig. 5. This analysis illuminated the importance of representing maximum erosion potential accurately and detailed as possible. The presence of a non-erodible layer, particularly at riffles, acts as grade control for riverbed upstream. When limits were not constrained at the glacial till shown in Fig. 5, overall erosion within Area 3 was much greater than when erosion at the glacial till was depth-limited. This illustrates the importance of setting erosion limits or specifying a non-erodible bed particle size that functions as grade control in sediment transport models. Grade controls can include constructed riffles, naturally occurring riffles, and inline structures such as dams and weirs.

Area 3: Sediment-Rating Curve

The incoming sediment load for a reach of interest is typically set using a rating curve that relates the sediment load in tons to flow. This curve is divided into distinct particle-size classes that compose the total load. The mass and size distributions of these sediment classes can be specified to change as flow changes. Generally, as flow increases, the capacity for water to carry more sediment and larger grain sizes also increases. This capacity is captured in rating curves by increasing the total mass of sediment and size of sediment as the flow increases. As flow fluctuates through the quasi-unsteady simulation, the total incoming sediment load fluctuates – increasing and decreasing with flow.

The sediment load rating curve for Area 3 was based on the same power function as the SRI TCRA sediment modeling, which was based on suspended-sediment data collected at the 26th Street Bridge by USGS at gage 04107850 (Amec, 2016). Sensitivity analyses on the sediment-rating curve included doubling and halving the incoming load. The SRI curve was also extended to include higher flows. The model was not very sensitive to incoming sediment load, as indicated by a doubled sediment loading curve resulting in 80% of cross sections experiencing less than 0.5 feet of additional erosion or deposition. A maximum of 1.1 feet of additional

deposition at model cross section 50.903 and a maximum of 0.75 feet of additional erosion at model cross section 49.666 were observed with a doubled sediment loading curve. The most important variable controlling the sediment balance in this reach was the abrupt lowering of grade control at the dam. The modeling indicates that the reach was net erosional following dam removal and grade control was a local phenomenon. Therefore, sediment load had less of an impact on the evolution of the bed profile during the period of dam removal. The sediment rating curve is shown in Fig. 6.

Sediment Transport Functions

Sediment models are typically sensitive to the selected transport function, sorting method, and fall-velocity equations selected for the simulation. To capture the range of possible results, various combinations of these functions were tested. The sediment transport functions developed by Yang and Ackers-White were selected for their applicability to sand-bed systems and for covering the range of slopes and velocities typical in the Kalamazoo River (Yang, 1973; Ackers and White, 1973).

The Copeland bed-sorting method, developed for sand-bed streams, was the bed-sorting method selected for the Area 3 sediment transport model. This method represents the bed as a vertical three-layer system: a surface layer that armors the bed and limits erosion of deeper layers, a subsurface layer, and an inactive layer (Copeland and Thomas, 1992). The surface layer is available for erosion or deposition during each computational time step, and material is transferred between layers depending on whether the cross section is depositional or erosional.

The fall-velocity equation determines how fast a particle will settle out of the water column based on the particle weight and water velocity. Several fall-velocity equations were tested, and the results for longitudinal cumulative volume varied a maximum of 7% between the different fall-velocity equations, therefore indicating that the selection of this equation does not have a significant impact on the model results. The Rubey fall-velocity equation was used for the Area 3 model and all Area 4 1-D models (Rubey, 1933).

Area 3: Sediment-Transport Model Calibration and Results

Calibration of the Area 3 sediment transport model relied on comparing two bathymetry sets: one collected in 2016 by EGLE and one collected in 2020 by Affiliated Researchers (EGLE, 2016). The goal was to compare the results of sediment-transport simulations to observed changes in profile, cross sections, and overall volumes between 2016 and 2020. The net change in volume between the bathymetry sets was calculated by subtracting the volumes defined by the two bathymetric surfaces (as raster files).

The BSTEM routine was used to simulate widening of the pilot channel following dam removal. Pilot-channel widening as simulated with the BSTEM routine was limited to a short period of time so the channel did not widen beyond the widths defined in the 2020 bathymetry. It was found that without allowing for some channel widening, bed erosion in the pilot channel was higher than indicated by the 2020 data. Allowing for some bed widening reduced total erosion in the pilot channel and produced a bed profile that was much closer to the observed bed profile.

The final simulation results were within +15% to -1% of observed volume change between 2016 and 2020. The most sensitive parameters were flow, initial geometry, bed gradation, maximum erosion depth, movable bed limits, and sediment-transport function. The profile-plot comparison showed close agreement between simulated bed elevation and the measured 2020 bathymetry,

particularly upstream of RM 50.25. The final simulated bed profiles are shown in Fig. 7 and the volume comparison is shown in Table 6.

The observed and simulated volumes of sediment change between 2016 and 2020 in Table 6 are approximately half of the total impounded sediment volume estimated by USGS (Rheaume et al., 2002). GEI estimates of total impounded sediment mobilized were higher than those estimated by Wood during the TCRA (~70,000 cubic yards) (Wood, 2019). There are at least two potential reasons for this difference. First, the Area 3 TCRA sediment transport model used flows occurring between 2001 and 2005, whereas GEI models simulated the flows between 2018 and 2020 after the WCS was removed. Second, the TCRA/SRI model used the design elevation for the constructed riffle (665.6 feet). GEI's field verification indicates the channel bottom is at approximately 663 feet. The TCRA model also had extents along the reach from the M-89 Bridge crossing through Areas 3 and 4 to Trowbridge Dam with erosion limited to a maximum of 5 feet. The GEI model in Area 3 applied shallow bed-erosion maximum limits at the exposed, coarse riffle-crest at the constructed riffle and at the exposed till and coarse material just upstream and in "the narrows." At other locations the GEI model set maximum erosion limits to 10 feet below the initial-conditions bed elevation. The riffle-crest elevation and maximum erodible limits were highly sensitive parameters to estimating the total sediment mobilized and post-WCS bed profiles.

Area 4: Current-Conditions Models and Calibration

Area 4: Bathymetry, Topography, and Channel Geometry

The Area 4 current-conditions model represents the conditions with Trowbridge Dam in place in 2021. Area 4 subareas are shown on Figs. 8a and 8b. This model is based on bathymetry data collected in Spring 2021 by Seaworks Group (Seaworks, 2021). Bathymetry data was also collected in 2020 by Affiliated Researchers. The 2021 dataset exhibited high resolution due to utilizing a boat drone that could access shallow waters, allowing the surveyors to collect data at higher resolution at the banks and to the water line. Along with bathymetry data, Seaworks conducted a LiDAR topographic survey of the ground surface surrounding the channel in Area 4 in 2021. This combination of LiDAR and boat-drone bathymetry produced a more consistent match between the bathymetric data and the land-based topography than the bathymetry data collected by Affiliated Researchers in 2020. The extents of the Seaworks LiDAR survey were not sufficient to model the complete lateral extents of the terrain through and beyond the valley walls or downstream of the 26th Street Bridge, so LiDAR data from the NOAA 2015 survey were used to supplement the Seaworks LiDAR (NOAA, 2015). Bathymetry data just downstream of Trowbridge Dam in the scour pool were supplemented with bathymetry data from EGLE collected in 2019 (EGLE, 2019).

Trowbridge Dam elevation data were not captured in the 2020 bathymetry, 2021 bathymetry, or LiDAR datasets due to unsafe access conditions and where water surface elevations passing through the existing spillway structure interfered with collection of structure elevations. To develop that terrain, details of the dam structure were obtained from the AECOM drawings associated with the stabilization work done in 2019 and "stitched" into the 2021 bathymetry and LiDAR collected by Seaworks (Seaworks, 2021; AECOM, 2019). For 1-D current-conditions modeling, the dam was represented as an inline structure with a constant elevation of 654.5.

New bathymetry data were not available downstream of the 26th Street Bridge. To fully capture hydraulic behavior around Trowbridge Dam and for 26th Street Bridge scour simulations, the model boundary was extended approximately 1 mile past Trowbridge Dam to RM 44.1. Cross sections downstream of RM 44.70 were interpolated using bed slopes determined by MDNR,

aerial imagery, and LiDAR data (DNR, 2005; NOAA, 2015). Cross-section locations for Area 4 are shown in Fig. 9.

USGS calculated the channel Manning's n-values for Area 4 at between 0.02 and 0.05 (Syed et al., 2005). The SRI used a channel Manning's n-value of 0.02 and floodplain Manning's n-values ranging from 0.02 to 0.12. GEI used a channel Manning's n-value of 0.03 and used the USGS National Landcover Dataset to develop floodplain Manning's n-values by land-cover type (Arcement and Schneider, 1989; NLCD, 2019).

Area 4: Boundary Conditions and Hydrology

The upstream boundary condition for the steady flows modeled in the Area 4 current-conditions model was set to critical depth. Critical depth is the depth of maximum discharge where the specific energy is held constant. The downstream boundary condition was set to normal depth with a slope of 0.001 feet/feet. The downstream condition was based on bed slopes and gradients for the Kalamazoo River developed by MDNR (DNR, 2005).

For steady-flow simulations, several flows associated with different annual recurrence interval events were used, including low flow, average daily, bankfull, 2-year, 10-year, 100-year, 150-year, 200-year, and 500-year. The bankfull flow of 3,630 cfs was developed from analysis of the USGS gages upstream and downstream of Trowbridge Dam and a regional curve analysis done in 2015 by Stantec (Stantec, 2015). The regional curve and GEI-estimated flows (based on GEI estimation of bankfull indicators at the referenced USGS gaging stations) are shown in Fig. 10. The 200-year and 500-year flows were developed by MDNR (EGLE, 2009). The 150-year flow was determined by interpolating between the 100-year and 200-year flows. Other flows were based on the SRI (Amec, 2018). These flows were assumed to be the flow going over Trowbridge Dam for these return intervals. Table 7 shows the steady flows used in Area 4 models.

For the 1-D version of the model, a lateral inflow at Schnable Brook was input to represent the tributary flows for Area 4. The flow value for this lateral inflow was developed using a drainage area ratio (DAR). To derive the drainage area of interest for the DAR, the drainage area of the Kalamazoo River at the upstream end of Area 4 near the constructed riffle at the former Otsego Township Dam of 1,488 square miles was subtracted from the drainage area of the downstream boundary of Area 4 at Trowbridge Dam of 1,531 square miles. The difference was 43 square miles, which results in a DAR of 0.028 for Schnable Brook to the Kalamazoo River at Trowbridge Dam.

Osgood Drain is the only other named tributary within Area 4. Due to its small size and drainage area of approximately 2.5 square miles (~0.16% of total drainage area at the dam), it was not included in the larger Area 4 1-D or 2-D models for current or future conditions. Grade control structures for both Schnable Brook and Osgood Drain were designed to provide stable channels and prevent head-cutting in the upstream direction from water level and bathymetry changes at the confluence of each tributary with the Kalamazoo River. These grade control structures were assessed by GEI as part of this study with separate 1-D hydraulic models. The models' downstream boundary conditions were set at the water levels estimated for the following scenarios: dam-out, equilibrium, stable channel for average daily, and bankfull flows. The model details and grade control design of the Schnable Brook and Osgood Drain are discussed later in this memo.

Area 4: Hydraulic Model Calibration

The Area 4 current-conditions model was calibrated to WSEs collected during the pre-design investigation (PDI) and continuous water-level data from pressure transducers installed in Area 4 in 2020. Steady-flow simulations were used to calibrate to WSEs over the river profile bracketed by transducers and collected during the poling events that were part of the PDI. Flows for these calibration points were developed using flows recorded at the Comstock and New Richmond USGS gages and adjusted for drainage area and lag time. An unsteady flow simulation was used to calibrate continuous WSE data from installed transducers in Area 4. This calibration was focused on the high flow event that occurred in March 2021.

To calibrate the current-conditions model, some geometry parameters were adjusted so the model results matched the observed calibration data. These parameters were largely centered around the characteristics of Trowbridge Dam, including the representation of the dam's spillway elevation and weir discharge coefficient. On as-built drawings from the work done on the dam in 2019, the spillway elevation is listed as 655 feet +/- NAVD88. A survey of the dam done by Wood in 2014 (included in the SRI) noted sill elevations of 653.69 and 652.23 feet NGVD29 (653.29 and 651.83 feet NAVD88); the SRI specified these elevations were difficult to acquire based on high water and unsafe conditions (Amec, 2019). The spillway has an irregular shape with two concrete pier structures and crest elevations that likely vary across the sill. For modeling purposes, the spillway crest was assumed to be one elevation across. A series of sensitivity analyses determined an appropriate value for the sill elevation and weir coefficient. Sill elevation was a higher-sensitivity parameter than the weir coefficient. The weir coefficient for Trowbridge Dam was set as 3.3. The sill elevation with the greatest agreement was 654.5 feet. Unsteady- and steady-flow calibration results are shown in Figs. 11a, 11b, and 12.

In both the steady-flow and unsteady-flow calibrations, the most sensitive parameters were flow and Manning's n-value. Inconsistencies were noted in discharge and peak flow timing and were resolved by using flow derived from both New Richmond and Comstock USGS gages adjusted for "lag time" (i.e., the time in between flow peaks observed at each USGS gage location and flow peaks observed at Trowbridge Dam). Variations in Manning's n-value were largely localized and determined to be less sensitive than the overall flow values. These variations were most sensitive at grade control features and areas upstream of the TCRA boundary where coarser bed material was observed. These variations were resolved by using a constant channel Manning's n-value of 0.03.

Area 4: Bed Gradation and Erodible Limits

Generally, dams and impoundments can limit sediment supply by either limiting total mass or size of material passed over the dam. In Areas 3 and 4, the predominant sediment type observed was fine-to-medium sand, however, there were locations in Area 3 and Subareas A and B of Area 4 where coarser material was observed. This indicates there is coarse material already in the system, which may have been transported into the reach via incoming load or was already there before some or all of the dams in these reaches were constructed. The coarse material from upstream likely settled out when it reached the head of backwater area from the Trowbridge Dam near the golf course (approximately river mile 47.25). As water levels were lowered and sediment mobilized during Area 3 TCRA activities, the bed was observed to coarsen. We attribute these findings to two possible causes: 1) coarser sediment entered Area 3 from upstream and settled out and/or 2) pre-dam material was exposed during the Otsego Township Dam removal. The coarser material observed in Subareas A and B in Area 4 could be a combination of pre-dam alluvium and coarser sediment mobilized from Area 3 following dam removal. The particle size distributions (PSDs) of this coarser material are very similar to the

PSDs that USGS found in their pre-dam removal investigations in Areas 3 and 4 and was identified by USGS as alluvial material (Rheume et al., 2002).

GEI staff conducted three pebble counts in Subareas A and B in Area 4 in Spring 2021 using the technique described by Harrelson (Harrelson et al., 1994). Locations and particle-size distributions of the pebble counts are shown in Figs. 13a–13c. These particle-size distributions had D50 (i.e., median particle diameter) results ranging between 11 mm to 21 mm, which is classified within the fine-to-coarse gravel range. This is consistent with the USGS assessment of the pre-dam alluvium, which also reported D50s in the fine-to-coarse gravel range between approximately 7 mm and 20 mm (Rheume et al., 2002).

Bed gradations for the impoundment (Subareas C, D, and E) in the Area 4 models were obtained from the sediment and soil cores collected by GEI for the PDI. Gradation analyses were performed on select samples, with those samples and their locations used to set the bed gradations for Subareas C, D, and E in the Area 4 sediment transport models. Bed gradations for Subareas A and B were defined by the pebble count data collected by GEI (Figs. 13a–13c). Bed gradations used in the Area 4 sediment transport model are listed in Table 8.

Horizontal erodible limits were set to the existing toe of each bank in each cross section, and sediment was allowed to deposit onto the overbank regions. The last cross section of the model was set as a pass-through node, with no erosion or deposition to allow sediment to move out of the system, based on HEC-RAS sediment-transport model guidance (USACE, 2016). Extensive poling data were used to develop a vertical limit to erosion within the impoundment behind Trowbridge Dam. This poling identified the coarse pre-dam alluvial layer, assumed to be stable for pre-dam conditions. The details of these data are in the Area 4 PDI Report (GEI, 2022). The extent of poling data near the dam was limited due to difficulties in collection and unsafe conditions, so a conservative sediment depth of 15 feet was assumed. USGS estimated the Trowbridge impoundment to be up to 16.5 feet thick in certain areas in 2000 (Rheume et al., 2002), and GEI located several areas with more than 15 feet of sediment, including the G1 Island in Subarea G (see Fig. 8b).

Area 4: Sediment-Transport Model Calibration and Results

GEI assumed in the current-conditions model that sediment mobilization would be limited due to the Trowbridge Dam acting as a significant grade control. Observations of the impoundment made during the PDI noted that the bed was primarily sand and highly mobile within the impoundment, with shallow sandbars and ripples aggrading and degrading on a short-term basis. Some of these phenomena can be observed in aerial photos. Because of this dynamic equilibrium state, comparing profiles and cross sections for alignment as calibration parameters is not as sensitive or appropriate as comparing the overall volume change as measured by repeat bathymetric datasets of impounded sediment.

The calibration of the Area 4 current-conditions sediment transport model was very similar to the calibration of the Area 3 sediment transport model. The sediment outflow file from the Area 3 model was used as the upstream sediment-boundary condition in Area 4. These data were imported to the Area 4 model using a data storage system (DSS) file. The upstream and lateral flows were based on the drainage-area-adjusted flow from the USGS Comstock gage for May 2013 to May 2020 with a lateral flow included at Schnable Brook. This model was also run with the reservoir sediment transport option in HEC-RAS that deposits more material in the deepest part of the cross section than the standard transport options.

For Area 4, two bathymetry sets were used to compare the observed and predicted changes in profile and overall volume. The first bathymetry set was collected in 2013 by EGLE and the second was collected in 2020 by Affiliated Researchers (EGLE, 2013; Affiliated Researchers, 2020). The two sets were compared to compute an overall volume change of approximately 53,000 cubic yards of deposition. This estimate accounted for the differences in extents of the two bathymetry sets. The model-predicted sediment volume change was within 10% of the calculated volume change between the 2013 and 2020 bathymetry datasets depending on the sediment-transport equation used. Results of the sediment calibration are shown in profile in Fig. 14 and with volume comparisons in Table 6.

The Area 4 sediment transport model was run with the same two equations used to calibrate the Area 3 sediment transport model – Yang and Ackers-White – since these two equations are most applicable to sand-bed systems (Yang, 1973; Ackers and White, 1973). A third equation, Laursen-Copeland (Laursen 1958, and Copeland 1993), which is also applicable to sand bed systems, was tested as described in (GEI, 2023) and found to produce similar results as the other two equations. The same sorting method and fall-velocity equation were used (Copeland and Thomas 1992; Rubey, 1933).

As with the Area 3 model, the Area 4 sediment transport model underwent sensitivity analyses as part of the calibration process. The most sensitive parameters were maximum erodible depth, flow (specifically the inclusion of lateral flow at Schnable Brook), and cross-section geometry. Maximum erodible depth was highly sensitive. When the model was run with vertical erosion limits greater than the limits defined by the alluvial surface, sediment was mobilized from the impoundment partially filling the scour hole below the dam, which had not been observed to occur. Channel and bank migration were not accounted for in the current-conditions model since the current conditions represent a stable impoundment with no historical evidence of channel evolution or migration across the 2013 to 2020 calibration period.

Tributary Analysis: Schnable Brook Hydraulic Model

To assess possible head-cutting impacts within Schnable Brook and Osgood Drain, a 1-D hydraulic model was developed for each tributary. Bathymetry and LiDAR data were collected in Schnable Brook by Seaworks Group in 2021 and were used to define model cross sections. Bed material in Schnable Brook was observed to consist of sand with large woody debris located in and adjacent to the channel. To represent these conditions, Manning's n-values of 0.03 and 0.08 were used in the channel and in the overbanks, respectively.

With Trowbridge Dam in place, Schnable Brook experienced significant back-water effects at its confluence with the Kalamazoo River. Because of this condition, the current-conditions downstream boundary condition for Schnable Brook was set using WSEs at the mouth of Schnable Brook from the 1-D hydraulic current-conditions model for Area 4. The upstream boundary condition for the Schnable Brook model was set as critical depth.

Tributary Analysis: Osgood Drain Hydraulic Model

The 1-D hydraulic model for Osgood Drain was developed using channel cross section survey data collected by GEI in Spring of 2021 and supplemented with LiDAR data from Seaworks Group also collected in Spring of 2021. The bed substrate varied in composition from cohesive soils (clay) to granular sand and gravel. The banks were observed to contain coarser material ranging in size from sand to cobbles with some large woody debris. Baseline values of 0.03 and 0.08 were used for channel and overbank Manning's n-values based on these observations. The upstream boundary condition was set to critical depth. The downstream boundary condition

was set to known WSE based on modeled water surfaces at the confluence of the Kalamazoo River and Osgood Drain.

Flow estimates were developed using WSE, velocity estimates, and cross-sectional areas collected during the survey and ranged from 5 to 15 cfs. Additionally, data from the National Hydrography Dataset from EPA estimated mean annual flow in Osgood Drain to be 3.7 cfs (NHDPlus, 2019). Using this data, a calibration flow was set to 8 cfs and channel Manning's n-values were adjusted ± 0.05 from the baseline 0.03 value to align with observed water surfaces from the cross-section survey. The downstream boundary condition was set to the observed WSE in the Kalamazoo River on the day of survey.

Future Conditions

Process and Approach

Future conditions in Area 4 were evaluated using a combination of 1) hydraulics and sediment transport models in both one- and two-dimensions and 2) the proposed elements of the dredge design, including the proposed pilot channel and bank excavation and stabilization design. Sediment transport simulations for different scenarios were used to develop the stable bed elevation after removal of the Trowbridge Dam. The stable bed elevations generated from the sediment model were used in a hydraulic model to simulate the flow interval events specified in Table 7.

One-dimensional modeling was primarily used to evaluate and develop design options. 1-D sediment transport modeling was used iteratively with a 1-D hydraulics-only simulation to assess and forecast trends in sediment mobilization and possible changes in channel morphology as a result of proposed design options. After sediment transport runs were performed, the resulting channel geometry was exported to a hydraulics-only model to assess future conditions such as shear stress, average velocity, and water surface elevations at specific return interval flows. The results of the hydraulics-only simulations were used to inform and refine design parameters, such as grade control locations and elevations, bank stability treatments, and channel shape. Necessary design refinements based on the hydraulics-only results were made in the sediment transport simulation and rerun to evaluate the impacts of design changes on sediment mobilization. This iterative process continued until a design with stable future conditions was achieved.

Two-dimensional modeling was implemented to fine tune design parameters after the 1-D modeling was complete. Cross sections from the 1-D model were used to develop a complete terrain for stable dam-out conditions. Refinements to the 2-D model were made directly to the terrain. On June 5, 2024, EPA recommended to GEI targeted channel widening and targeted riffle crest lowering design refinements in Subareas C and D (EPA, 2024). Changes were made to the 1-D and 2-D models to reflect these recommendations.

Reference reach data collected and analyzed by AECOM (2017) and GEI were used to evaluate stable channel dimensions in the Kalamazoo River (Table 9). The Trowbridge reach just upstream of the dam passes through a narrow valley, and the Rosgen stream type changes from a C-type channel upstream to a Bc-type channel through this narrow valley (Rosgen, 1996). Therefore, GEI used the AECOM analysis to define stable channel dimensions for the C-type channel upstream and collected its own data for stable dimensions for the Bc-type channel.

A desktop evaluation was performed by searching the Kalamazoo River watershed with special attention paid to narrow, confined alluvial river corridors with stable channel morphology and minimal anthropogenic impacts, such as dams, channel straightening, etc. There were a few reaches in the watershed near Cooper and Ceresco that exhibited similar narrow-valley characteristics. A reference reach survey was performed near Battle Creek over three distinct sub-reaches, including a reach from 11 Mile Road downstream, a reach located approximately 3,500–5,000 feet downstream from 11 Mile Road in the middle of the reference reach, and a third reach upstream from Interstate 94 and Historic Bridges Park.

One-Dimensional Model Inputs

The Area 4 future-conditions model used the Area 4 current-conditions model as a base for all the design model iterations. The major changes from the current-conditions to the future-conditions model centered around the following:

- A new channel alignment in Subareas E and G, guided by estimates of the historical thalweg from the poling data.
- PCB dredging limits.
- Bank cuts and bank restoration.
- Installation of an 80-foot-wide pilot channel with an average depth of 4 feet, see Fig. 15.
- A water control structure (WCS) for drawdown modeling.
- Changes in the channel and bank geometry near the dam along with a constructed riffle in that location with bed material coarsened to meet 100-year flow conditions.
- Eleven riffles in subareas C, D, and E with bed material coarsened to meet 100-year flow conditions.

Bed gradations, flows, transport functions, and vertical erodible limits were all kept consistent with the current-conditions model. The upstream sediment load for the proposed-conditions model was the original rating curve developed in the SRI from the 26th Street suspended-sediment data collected by USGS, which was also used for the Area 3 model. A sensitivity analysis was performed on this sediment load using the same variations as used to calibrate the Area 3 sediment transport model. Similar to Area 3, the model was not particularly sensitive to the incoming sediment load.

Channel Alignment

The new channel alignment in Subareas E and G was derived from the poling and sediment core data collected during the PDI. This data indicated a historical thalweg in Subarea G (under the G1 Island) that deviated from the current thalweg alignment in Subarea E. This historical thalweg was likely present before dam construction and likely filled with sediment after the construction of the dam in 1898. This section of the historical thalweg contains PCB-impacted material that will be removed. This removal and the location of the historical thalweg were assumed to alter the channel alignment from Subarea E to the G1 Island in Subarea G. The future-conditions channel alignment with the thalweg through G1 is shown in Fig. 16.

PCB Dredge, Bank Cuts, and Pilot Channel

Removal of PCB-impacted sediment was assumed to take place before impounded sediment would be mobilized from Area 4. This was represented in the model in two ways: 1) Inclusion of the PCB dredge surface geometry and 2) Inclusion of environmental bank cut and bank

restoration grading geometry. Removal of “Beaver Island” and sediment near that island at approximately RM 45.0 was also represented in the model to facilitate construction and the passage of water and sediment through the WCS. The sediment transport model was developed to capture the trends and patterns of impounded sediment mobilization, toe scour, and potential bank-failure locations.

Bank treatments were represented with horizontal variation in Manning’s n-values to capture the varied roughness of bank treatments and channel material. Toe wood was represented with $n = 0.12$, with roots expected to have similar roughness as tree branches (Chow, 1959). Toe stone with median particle size ranging from small to large cobble was represented with n-values ranging from 0.035 to 0.045 (Arcement and Schneider, 1989). Planted vegetation was represented with n-values of 0.07 for live stakes and robust plantings (Chow, 1959).

Following PCB removal, a pilot channel is proposed to be dredged to remove some of the sediment that is expected to mobilize following dam removal and to focus bed lowering toward the middle of the current and proposed channel alignments (Fig. 15). The pilot channel dredge was represented in the model geometry as an 80-foot-wide bottom channel, an average of 4 feet deep, with 3:1 side slopes from RM 45.35 to RM 47.00. The pilot channel invert was set at or close to the final design grade for the dam-out channel.

The 80-foot-wide pilot channel is expected to naturally widen as the impoundment draws down and the river finds its equilibrium. This widening was simulated during sediment transport runs with the Simplified Channel Evolution Model in HEC-RAS. At each of the pilot-channel cross sections, when the river had capacity to erode, the channel first widened to the vicinity of the proposed toe of bank treatments before eroding vertically. At non-pilot-channel cross sections, when the river had capacity to erode, the mass was removed in an even layer from the wetted portion of the cross section, which is also referred to as the “veneer method.” Because actual channel evolution is complex and difficult to predict, the pilot channel cross sections were also simulated using the veneer method as a sensitivity analysis. Although the final cross section shape varied, the results were within 10% of the same volume of sediment mobilized using either method.

Water Control Structure

During construction, water levels are proposed to be managed with the WCS. Reservoir drawdown through the WCS was modeled with a series of 1-D sediment transport simulations to evaluate construction sequencing and timing. For this set of simulations, the WCS was modeled as an inline structure with open air gates to represent the stoplog elevations, and a weir coefficient of 3.2. Drawdown was simulated to occur with removal of WCS stoplogs to elevation 646 feet, followed by removal of the existing spillway to elevation 650 feet, followed by removal of stoplogs to the bottom of the WCS at elevation 642 feet.

Constructed and Coarsened Riffle Geometry and Characteristics

Following dredging and drawdown, the WCS and the remainder of Trowbridge Dam and its left embankment will be removed and replaced with a designed riffle. Sensitivity analyses for the constructed riffle dimensions, crest elevation, and location were performed. The most sensitive parameters associated with the constructed riffle were the crest elevation, riffle and run shape, and run slopes. The final approach (glide) slope was -6%, and the final run slope was 2.5%. This run slope was achieved by raising the elevation of the scour hole below Trowbridge Dam to 637 feet. It was assumed the scour hole would partially fill with mobilized sediment once the impoundment water level dropped and the dam was removed. Additional fill may be required to

meet the elevation needed to achieve a run slope of 2.5%. Cross section, profile, and plan views of the constructed riffle are shown in Figs. 17, 18, and 19.

The shape of the crest and run included a bankfull bench on the right descending bank. The riffle dimensions approximate the dimensions developed from the reference reach survey. Tables 9 and 10 contain reference reach dimensions for riffle features. Table 11 contains cross sectional widths, depths, and areas and entrenchment ratios from the 1-D model of future conditions.

The Manning's n-values for the constructed riffle were set to 0.04 in the main channel starting at the crest cross section and extended through the run to represent added roughness from the larger material used to construct the riffle (Arcement and Schneider, 1989). Manning's n-values in the overbank regions and bankfull bench at the constructed riffle were set to 0.07 to account for live stakes and robust plantings along the bench and banks (Chow, 1959).

Eleven other riffles in Subareas C, D, and E are proposed to be coarsened during construction. Riffle-coarsening locations and elevations were selected based on 1-D hydraulic model design iterations, the approximate riffle crests identified from the poling and alluvial layer, and appropriate placement in relation to generalized characteristics of pool-riffle morphology (i.e., in the crossovers between bends). The goal of sizing and placing the riffles within the accepted morphological context was to evenly distribute energy dissipation throughout Area 4. This is consistent with fluvial geomorphology theory that one of the prime morphological organizing principles of rivers is to minimize the variance of energy dissipation along their length (Leopold, 1994). Manning's n-values for the coarsened riffles were set to range from 0.035 to 0.045 to represent the roughness of median grain sizes between small to large cobble (Arcement and Schneider, 1989). Details of the riffle particle sizing were evaluated in 2-D as discussed below.

Two-dimensional Model Inputs

Two-dimensional modeling was implemented to evaluate dam-out shear stresses and velocities in more detail than the 1-D results, which report cross-section averages. Two-dimensional modeling requires the use of a grid (or mesh surface) that contains computation points rather than cross-section information. The mesh used in the 2-D models was set to a cell spacing of 20 feet by 20 feet in the main channel and overbanks and a cell spacing of 50 feet by 50 feet in the floodplain. The boundary-condition lines were placed at the upstream end of the model extent at approximately RM 47.2, Schnable Brook below the M-89 Bridge crossing, and the downstream end of the model extent near RM 44.8. The downstream boundary was set as a stage-hydrograph rating curve based on the 1-D hydraulic model output at that location as presented in Table 12. Manning's n-values were adjusted in the 2-D models to represent coarsened riffles and bank treatments: toe wood, toe stone, and plantings consistent with the 1-D model. Manning's n-value regions for the HEC-RAS model are shown in Fig. 20.

Once the 1-D hydraulic model of future conditions was developed, further edits were made to the proposed conditions within the 2-D model terrain. These edits were minor compared to the original iterations of the 1-D model and consisted of small adjustments in channel dimensions and smoothing the main channel to remove abrupt changes in elevation and sinuosity. To reduce computation times and model complexity, the 2-D model extents were shortened to approximately RM 47.2 near the golf course and RM 44.8 just downstream of the 26th Street Bridge. The 2-D model was run with the full momentum equation set and a 1-second computational time step. Resulting Courant numbers were generally less than 1, indicating an

acceptable combination of cell size and computational time step. The 2-D model results were compared to WSEs developed in the 1-D future-conditions model, shown in Fig. 21.

Targeted Channel Widening and Riffle Crest Refinement

Following recommendations from EPA (EPA, 2024), select modifications were made to the channel design in subareas C and D using HEC-RAS version 6.5. These modifications included approximately 30 feet of channel widening on the right bank between river mile 46.07 and 46.39, approximately 30 feet of channel widening on the left bank from river mile 46.07 to 46.26, and approximately 40 feet of channel widening between river miles 47 and 47.14. These changes resulted in bankfull channel widths between 185 and 225 feet in these locations, which is comparable to channel widths upstream and downstream of these targeted locations. GEI refined the channel widening provided by EPA by smoothing the bank edges so there were no abrupt changes to the river cross section from upstream to downstream, and by adding constructable bank side slopes (i.e., 2H:1V) at the widened locations. The modifications also included minor adjustments to three riffle crest elevations. Riffle crest elevations were lowered between 0.3 feet and 0.5 feet at river miles 47.07, 46.84, and 46.16. These changes resulted in average channel bankfull velocities between 3.7 and 4.2 ft/s at riffles in subareas C and D.

Sensitivity Analyses

The Area 4 future-conditions sediment transport and hydraulic models were most sensitive to erodible limits determined by alluvial data and the geometry and elevations of the constructed and coarsened riffles.

The limits to vertical erosion were set at absolute elevations based on the poling data and alluvial-base surface generated from the poling data collected by USGS, EGLE, and GEI (GEI, 2022). The location of this alluvial surface in the vertical direction was determined to be very sensitive. The model run with unbounded erosion produced unrealistic deposition and erosion patterns with artificial high points in the profile composed of fine material. With the addition of the alluvial surface data and limits to erosion, the model produced much more realistic profiles with the alluvial material being exposed in some locations and retaining material in others, specifically within the pools or immediately upstream of riffles. This riffle-pool behavior is consistent with typical fluvial geomorphology patterns in sand and gravel bed rivers (Leopold, 1994).

The constructed riffle acted as a major control point in future-condition scenarios. Changing the elevation or overall configuration of the riffle caused significant differences in sediment mobilized and differences of several feet in WSEs.

Modeled WSEs and velocities were sensitive to the addition of coarsened riffles and their elevations. The elevations of coarsened riffles were modeled iteratively to dissipate energy evenly as much as possible throughout Area 4.

Riffle design gradations were based on evaluating the 100-year shear stresses and velocities in 2-D and determining the D50 particle size that would be stable under each of those conditions. 2-D results were used because they were more conservative than the 1-D cross-section average velocities and shear stresses. Stable particles based on shear-stress criteria were calculated with critical shear thresholds (Berenbrock and Tranmer 2008). Stable particles based on velocity criteria were calculated with the Maynard method (NRCS, 2007). The most conservative D50 at each cross section was used to inform design (Table 13).

Final Future-Conditions Results

One-dimensional Model

One-dimensional model results are presented by cross section in Tables 11 and 13. Table 11 contains bankfull parameters including width, depth, area, and entrenchment ratio. Table 13 contains the smallest stable particle based on the maximum cross-sectional shear stress and velocity from the 2-D results. Water-surface profiles for average daily, bankfull, and the 2-year, 10-year, and 100-year annual recurrence intervals are shown in Fig. 22. Figure 22 also shows the current-conditions and future-conditions bed profile and alluvial surface profile. Cross-sectional average velocities along the profile for the average daily, bankfull, 2-year, 10-year, and 100-year annual recurrence intervals are shown in Fig. 23.

Two-dimensional Model

Plan views of the 2-D future-conditions model for velocities, shear stresses, and depths for the average daily, bankfull, and 100-year annual recurrence interval events are shown in in Figs. 24a–24c, 25a–25c, and 26a–26c.

Tributary Future Conditions

Schnable Brook

Grade control in Schnable Brook was designed to minimize uncontrolled head cutting and increased erosive forces as a result of changed conditions at its confluence with the Kalamazoo River. Two grade control structures (riffles) and targeted bed grading within the channel and along the banks were designed to prevent significant sediment mobilization and increased erosive potential within the tributary. The upstream grade control is located at Station 667 and the downstream grade control is located at Station 233.

Schnable Brook experienced backwater effects depending on the water level in the main channel of the Kalamazoo River for proposed conditions as well as existing conditions. Due to these effects, the downstream boundary was set as the water surface elevation of the 2-D Kalamazoo future-conditions model for several return interval flow events.

Water surface profiles, velocity profiles, and locations of grade control structures for Schnable Brook existing and proposed conditions are shown in Figs. 27a-27c.

Osgood Drain

Grade control in Osgood Drain was also designed to minimize uncontrolled head cutting and increased erosive forces as a result of changed conditions at its confluence with the Kalamazoo River. Due to the much larger estimated elevation drop at the mouth of Osgood Drain from current to proposed conditions than in Schnable Brook, grade controls will include a series of riffles and pools, and step-pools in high gradient areas.

Osgood Drain proposed-conditions downstream boundary was set to the 2-D modeled water surface elevation for the Kalamazoo future conditions model. Water surface profiles, velocity profiles, and locations of grade control structures for Osgood Drain existing and proposed conditions are shown in Figs. 28a-28c.

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Table 1. Model Files Directory

Model Name ¹		Model Type		Plan Name	Flows
1	Area 3 Current Conditions	1D	Hydraulics	Area 3 Current Conditions Calibration	Calibration flow
		1D	Hydraulics	Area 3 Current Conditions Hydraulics	Return interval flows
2	Area 3 Dam In	1D	Sediment Transport	Area 3 Dam in/WCS in sediment transport	2013-2018 hydrograph
3	Area 3 Post WCS	1D	Sediment Transport	Area 3 Post WCS sediment transport	2018-2020 hydrograph
4	Area 4 Calibration and Sediment Transport	1D	Hydraulics	Area 4 Dam In Hydraulic Calibration	Calibration flow
		1D	Hydraulics	Area 4 Dam In Hydraulics	Return interval flows
		1D	Sediment Transport	Area 4 Sediment Transport Calibration	2013 - 2020 hydrograph
		1D	Sediment Transport	Area 4 Sediment Transport post dam removal	7-year hydrograph
5	Area 4 Dam Out Equilibrium ²	1D	Hydraulics	Area 4 Dam Out Equilibrium 1D Hydraulics	Return interval flows
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D Avg Daily	Average Daily Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D Bankfull	Bankfull Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D 10-Year	10-Year Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D 100-Year	100-Year Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D 150-Year	150-Year Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D 200-Year	200-Year Flow
		2D	Hydraulics	Area 4 Dam Out Equilibrium 2D 500-Year	500-Year Flow
6	Area 4 BSTEM Calibration	1D	Sediment Transport & BSTEM	Area 4 BSTEM Calibration	2000-2002 hydrograph
7	Area 4 BSTEM Dam Out	1D	Sediment Transport & BSTEM	Area 4 BSTEM High groundwater	7-year hydrograph
		1D	Sediment Transport & BSTEM	Area 4 BSTEM Low groundwater	7-year hydrograph
8	Area 4 Tributary Analysis	1D	Hydraulics	Schnable Existing Conditions	Return interval flows
		1D	Hydraulics	Schnable Proposed Conditions	Return interval flows
		1D	Hydraulics	Osgood Existing Conditions	Return interval flows
		1D	Hydraulics	Osgood Proposed Conditions	Return interval flows

Notes:

1. Please see accompanying hydraulic model data. All models were initially developed using HEC-RAS v.5.0.7.
2. Area 4 Dam Out Equilibrium Model updated to HEC-RAS v6.5 in 2024.

Created by: EG 10/4/2024

Checked by: LN 10/4/2024

Table 2. Area 3 Flood Discharge Values

Return Period	M-89 (cfs) ¹	Pine Creek Tributary (cfs) ¹	Former Otsego Township Dam (cfs) ¹
Average Daily	1000	100	1100
2-Year	4300	400	4700
10-Year	7000	600	7600
100-Year	10400	1000	11400

Notes:

1. cfs = cubic feet per second

Created by: EG 9/17/2021

Checked by: LAP 9/23/2021

Table 3. Sediment Computation Increments

Flow (CFS) (low)	Flow (CFS) (high)	Computation Increment (hours)
0	900	12
900	2000	6
2000	3000	2
3000	4000	1
4000	5000	1
5000	6000	0.5
6000	7000	0.2
7000	8000	0.1
8000	10000	0.05

1. Sediment transport computation increments used for HEC-RAS quasi unsteady flow.

Created by: EG 9/17/2021

Checked by: LAP 9/23/2021

Table 4. Area 3 Sediment Gradations

Material	Diameter (mm)	River Mile 51.123 to 50.354¹ (% Finer)	River Mile 50.257 to 49.666¹ (% Finer)	Riffle at Former Dam² (% Finer)
Silt	0.0625	--	2	--
Very Fine Sand	0.125	--	5	--
Fine Sand	0.25	--	10	--
Medium Sand	0.5	4	21	--
Coarse Sand	1	27	50	--
Very Coarse Sand	2	65	74	--
Very Fine Gravel	4	72	85	--
Fine Gravel	8	87	93	--
Medium Gravel	16	98	97	0.5
Coarse Gravel	32	100	99	1
Very Coarse Gravel	64	--	100	2
Small Cobbles	128	--	--	10
Large Cobbles	256	--	--	20
Small Boulders	512	--	--	50
Medium Boulders	2048	--	--	100

Notes:

1. Gradations from: Amec Foster Wheeler (2016). "Supplemental Remedial Investigation Report, Area 3, OU-5 Allied Paper, Inc./Portage Creek/Kalamazoo River Superfund Site." Prepared for Georgia Pacific LLC. April 2016.

2. Gradation based on field estimates.

Created by: EG 9/17/2021

Checked by: LAP 9/23/2021

Table 5. Area 3 Erodible Limits

River Mile	Erodible Limit (feet)	Note
51.123	10	
51.096	10	
51.048	10	
51.009	10	
50.98	10	
50.946	10	
50.903	10	
50.865	10	
50.831	10	
50.801	10	
50.771	10	
50.734	10	
50.709	10	
50.683	10	
50.615	10	
50.571	10	
50.526	10	
50.454	10	
50.421	10	
50.354	0.5	Glacial Till ²
50.257	0.5	Glacial Till ²
50.215	10	
50.168	10	
50.115	10	
50.059	10	
50.046	10	
50.015	10	
49.922	10	
49.842	10	
49.779	10	
49.666	10	
49.603	0	Riffle at Former Dam
49.562	0	Riffle at Former Dam

Notes:

1. Erodible depth limits set in HEC-RAS for 1-D sediment transport simulations
2. See Figure 4a

Created by: EG 9/17/2021

Checked by: LAP 9/23/2021

Table 6. Predicted and Observed Bed Volume Changes in Areas 3 and 4

Area	Years	Observed Volume Change (CY)	Simulated Volume Change (CY)	Difference (%)
Area 3	2016-2020	108,870	108,140	-1%
Area 4	2013-2020	53,320	44,070	-17%

Notes:

1. Observed volume changes calculated from repeated bathymetry data, rounded to the nearest 10 cubic yards (CY).

Created by: EG 9/17/2021

Checked by: LAP 9/23/2021

Table 7. Flood Discharge Values

Return Period	Discharge Amec (2018) (cfs)¹	Discharge AECOM (2019b) (cfs)¹	Discharge GEI 2021² (cfs)¹
Baseflow	na	600	600
Normal flow	1,130	na	1,130
50% (2-year)	4,900	na	4,900
10% (10-year)	7,800	7,100	7,800
2% (50-year)	na	11,000	11,000
1% (100-year)	11,800	12,000	11,800
0.7% (150 year)	na	na	13,100
0.5% (200 year)	na	14,000	14,000
0.2% (500 year)	na	17,000	17,000

Notes:

1. cfs = cubic feet per second.
2. GEI 2021 values indicate the discharge values used in GEI hydraulic modeling.
3. Based on data from Amec (2018) and AECOM (2019b).

Created by: EG 9/17/2021
Checked by: LAP 9/23/2021

Table 8. Area 4 Sediment Gradations¹

Material	Diameter (mm)	4S-ED03-1 (60-72) (% Finer)	4S-EB09-2 (0-12) (% Finer)	4S-EB09-2 (48-54) (% Finer)	4S-EB13-2 (0-12) (% Finer)	4S-EB13-2 (72-84) (% Finer)	4S-ED15-2 (0-12) (% Finer)	4S-ED15-2 (72-84) (% Finer)	4S-EF16-1 (0-28) (% Finer)	4S-EF16-1 (28-69) (% Finer)	4S-EH19-1 (% Finer)	4S-EH18-1 (0-72) (% Finer)	4S-EH18-1 (0-12) (% Finer)	4S-EH18-1 (36-48) (% Finer)	AVG Surf GRAD ² (% Finer)	Pebble Count 49.45 (% Finer)	Pebble Count 48.84 (% Finer)	Pebble Count 47.27 (% Finer)	AVG Upstream Grad
River Mile		44.98	45.1	45.1	45.17	45.17	45.25	45.25	45.3	45.3	45.35	45.34	45.34	45.34		49.45	48.84	47.27	
Silt	0.0625															3.0	3.9	4.0	3.6
Very Fine Sand	0.125	84.3	0.5	34.3	0.9	48.9	1.8	63.8	0.2	35.8	0.6	1.6	0.4	1.9	1.2	3.0	3.9	4.0	3.6
Fine Sand	0.25	86.3	1.9	62.7	3.9	53.1	2.3	72.4	3.3	44.9	1.8	19.1	1.2	5.4	4.7	5.0	4.9	4.0	4.6
Medium Sand	0.5	89	28.6	96	32.3	58.8	11.6	80.7	46.7	58.3	36.8	85.6	39.9	37.8	40.3	5.0	4.9	4.0	4.6
Coarse Sand	1	93.2	64.3	97.9	66.1	68.2	53.2	88	84.6	72.9	73	94.9	76.7	66.7	72.7	7.9	7.8	8.9	8.2
Very Coarse Sand	2	100	83.2	98.4	82.9	84.4	66.6	94.9	94.6	86.5	87.6	97.4	88.2	78.9	85.4	7.9	7.8	9.9	8.5
Very Fine Gravel	4		95.7	98.8	95.6	100	92.9	100	98.7	99.4	96.1	98.8	95.7	90.1	96.3	8.9	9.7	18.8	12.5
Fine Gravel	8		98.7	99.3	99		95		99.8	100	99.1	99.6	98.7	96.4	98.5	17.8	20.4	39.6	25.9
Medium Gravel	16		100	100	100		99.5		100		100	100	99.7	99.3	99.8	39.6	39.8	57.4	45.6
Coarse Gravel	32						100						100	100	100	70.3	69.9	76.2	72.1
Gravel	64															92.1	87.4	97.0	92.2
Small Cobble	128															96.0	99.0	99	98
Large Cobble	256															99	100	100	99.7
Small Boulder	512															100			100

Notes:

1. Gradations from: GEI (2021b). "Pre-Design Investigation, Phase 1 and Phase 2, Data Summary Report, OU5 Area 4 Time-Critical Removal Action Allied Paper/Portage Creek/Kalamazoo River Superfund Site." September 2021.

2. Average of all impoundment sediment gradations.

3. Average of all subarea A and B pebble counts.

Created by: EG 6/14/2022

Checked by: LN 7/21/2022

Table 9. Reference Reach Dimensions
Kalamazoo River Downstream of 11 Mile Road to Interstate 94

Design Variables	Upstream of Historic Bridges Park			Middle of Reach			Downstream 11 Mile Road		
Downstream Limit (Lat/Long)	42.288477, -85.112124			42.280856, -85.102511			42.276664, -85.086545		
Upstream Limit (Lat/Long)	42.283342, -85.10572			42.279553, -85.099808			42.275154, -85.077002		
River Mile	107			108			109		
Total Length (ft)	2800			2090			2950		
Stream Type	B5c			B5c			B4c		
Valley Type	Confined Alluvial			Confined Alluvial			Confined Alluvial		
Drainage Area (sq. miles)	461			460			459		
	Average	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum
Bankfull width (feet)	171.81	153.51	199.33	188.30	160.83	223.35	172.01	147.22	196.80
Bankfull mean riffle depth (feet)	2.32	2.12	2.50	3.03	2.55	3.19	2.32	2.01	2.62
Bankfull max riffle depth (feet)	3.22	2.93	3.36	4.37	3.85	4.93	3.11	2.94	3.27
Max/Mean Riffle Depth Ratio	1.39	1.38	1.34	1.57	1.21	1.93	1.34	1.46	1.25
Riffle Width/Depth Ratio	74.98	65.03	94.02	63.51	50.58	87.59	77.05	56.19	97.91
Riffle Cross-Sectional Area (square feet)	396.04	357.89	423.32	563.64	510.74	664.40	390.99	385.86	396.12
Low Bank Height (feet)	3.55	2.93	4.17	3.77	3.85	5.00	3.39	3.26	3.52
Low Bank Height to max Riffle Depth Ratio	1.12	1.00	1.24	1.01	1.00	1.01	1.10	1.00	1.20
Flood prone width (feet)	327.78	298.50	369.91	328.65	300.13	403.33	294.12	280.09	308.14
Entrenchment Ratio	1.91	1.50	2.41	1.93	1.34	2.51	1.76	1.42	2.09
Riffle Length (feet)	109.81	148.49	169.90	142.80	137.09	148.51	107.32	107.32	107.32
Riffle Length/Bankfull Width Ratio	0.64	0.74	1.11	0.77	0.61	0.92	0.62	0.55	1.34
Riffle Channel Slope (%)	0.77%	0.54%	0.93%	0.36%	0.13%	0.57%	0.28%	0.45%	0.63%
Riffle Water Surface Slope (%)	0.10%	0.00%	0.17%	0.34%	0.08%	0.51%	0.21%	0.24%	0.26%
Reach Bankfull Slope (%)	0.05%	0.05%	0.05%	0.02%	0.02%	0.02%	0.15%	0.15%	0.15%
Riffle Water Surface Slope to Reach Bankfull Slope Ratio	2.04	0.00	3.53	13.34	5.27	21.42	1.43	1.59	1.74
Valley Slope (%)	0.02%	0.02%	0.02%	0.12%	0.21%	0.02%	0.12%	0.12%	0.12%
Pool Slope (%)	0.05%	0.00%	0.09%	0.03%	0.01%	0.04%	0.01%	0.05%	0.09%
Pool Slope to Reach Bankfull Slope Ratio	1.02	0.00	1.78	1.28	0.93	1.63	0.07	0.30	0.61
Maximum Pool Depth (feet)	4.93	4.42	5.31	6.04	5.50	6.57	4.98	4.82	5.23
Pool depth to average bankfull depth ratio	2.13	1.77	2.50	2.15	1.72	2.58	2.20	2.40	2.00
Pool Width (feet)	148.71	115.59	210.35	170.08	101.01	239.14	163.20	132.77	182.50
Pool width to Riffle width ratio	0.87	0.58	1.37	0.97	0.45	1.49	0.95	0.90	0.93
Pool Area (square feet)	453.36	358.91	531.16	522.54	351.05	694.02	523.82	439.68	643.70
Pool area to Riffle area ratio	1.14	0.85	1.48	0.94	0.53	1.36	1.34	1.14	1.63
Pool to Pool Spacing (feet)	536.64	699.63	980.04	916.45	911.37	921.53	414.26	461.21	508.16
Pool to Pool Spacing to bankfull width ratio	3.12	3.51	6.38	4.91	4.08	5.73	2.41	2.34	3.45
Riffle Material Silt/Clay (%)	15.2%			10.9%			4.8%		
Riffle Material Sand (%)	18.2%			13.9%			5.7%		
Riffle Material Gravel (%)	53.5%			69.3%			73.3%		
Riffle Material Cobble (%)	13.1%			5.9%			10.5%		
Riffle Material Boulder (%)	0.0%			0.0%			2.9%		
Riffle Material Bedrock (%)	0.0%			0.0%			2.9%		
Riffle D16 particle size (mm)	0.1			0.2			8.4		
Riffle D35 particle size (mm)	4.6			7.4			17.8		
Riffle D50 particle size (mm)	20.1			19.3			27.3		
Riffle D84 particle size (mm)	58.0			43.1			64.7		
Riffle D100 particle size (mm)	256.0			180.0			Bedrock		

Table 10. Applied Reference Reach Dimensions in TCRA Area 4

Location	Bankfull Top Width (ft)	Bankfull Maximum Riffle Depth (ft)	Bankfull Riffle Flow Area (ft ²)	Entrenchment Ratio
River Mile 45-45.3 (Narrow Valley)	224 - 240	4.8 - 8.5	900 - 1210	1.4 - 2.2
Upstream of River Mile 45.3 (Wide Valley)	225 - 266	4.8 - 7.1	900 - 1210	>2.2

Notes:

1. Narrow valley dimensions calculated from reference reach data dimensionless ratios collected on the Kalamazoo River between 11 Mile Road and Interstate 94 upstream from Battle Creek.
2. Wide valley dimensions calculated from reference reach data dimensionless ratios collected by Stantec on the Kalamazoo River and its tributaries as published in the Ceresco Dam Removal and Kalamazoo River Restoration Construction Drawings Design Report, November 27, 2013.

Created by: EG 9/17/2021

Checked by: LN 7/21/2022

Table 11. Area 4 Future Conditions Bankfull Parameters¹

River Mile	Bankfull Water Surface Elevation (ft)	Flow Area (ft ²)	Top Width (ft)	Maximum Depth (ft)	Width/Depth Ratio ²	Entrenchment Ratio ³	Bank Height Ratio ⁴	Feature ⁵
49.45	666.9	1084	362	4.1	121	1.1	3.1	Riffle
49.35	666.3	924	240	5.6	62	1.6	2.3	
49.23	665.7	880	213	6.7	52	3.1	1.1	
49.12	665	831	208	7.3	52	9.2	1.5	
48.84	663.8	1146	305	5.0	81	1.3	2.1	Riffle
48.67	663.2	1031	199	7.0	38	3.6	1.1	Riffle
48.54	662.9	1036	196	7.7	37	4.2	1.1	
48.41	662.6	1163	269	6.0	62	2.7	1.1	
48.29	662.1	939	230	6.9	56	5.1	1.1	
48.11	661.4	927	174	7.8	33	11.1	1.3	Riffle
47.96	660.9	984	195	8.1	39	11.8	1.2	
47.78	660.3	859	144	10.0	24	11.3	1.3	
47.61	659.7	1036	230	6.4	51	2.8	1.6	
47.47	659.4	1112	183	11.6	30	2.0	1.5	Riffle
47.37	659.2	1052	182	8.1	32	2.5	1.6	
47.27	658.7	884	276	5.1	86	1.5	2.7	
47.14	658.2	1165	236	6.7	48	2.7	1.2	
47.07	657.9	1028	233	5.2	53	2.8	1.7	Riffle
47	657.5	989	205	5.8	43	3.3	1.3	
46.92	657.4	1298	180	8.4	25	6.1	1.2	
46.84	657.1	882	183	5.7	38	6.2	1.7	
46.8	656.9	989	210	5.5	45	6.7	1.6	Riffle
46.67	656.7	1344	166	9.6	21	14.7	1.2	
46.46	656.2	1146	152	9.1	20	15.0	1.6	
46.36	655.5	891	195	5.1	43	1.9	1.6	
46.26	655.3	1372	248	7.0	45	4.8	1.5	Riffle
46.16	654.8	955	222	5.8	52	4.4	1.8	
46.07	654.3	1003	208	5.5	43	4.1	1.9	
46	653.7	795	198	4.9	49	1.4	2.2	
45.89	653.2	1044	205	6.6	40	6.2	1.9	Riffle
45.83	653.1	1242	210	7.8	35	5.7	1.3	
45.77	652.8	1003	226	5.6	51	1.5	2.2	
45.72	652.7	1160	203	6.3	36	1.5	2.0	
45.68	652.6	1337	214	8.9	34	4.8	1.1	Riffle
45.64	652.5	1341	219	8.8	36	5.4	1.3	
45.59	652.3	1258	274	5.9	60	2.1	1.6	
45.56	652.1	1192	235	6.2	46	1.3	2.0	
45.51	652	1558	251	7.8	40	3.3	1.3	Riffle
45.48	651.7	1038	224	5.9	48	2.8	1.0	
45.44	651.6	1263	285	6.1	64	1.9	1.6	
45.41	651.5	1585	312	8.0	61	3.3	0.9	
45.37	651.4	1453	235	8.2	38	4.7	1.0	Riffle
45.35	651.3	1263	266	8.8	56	4.2	1.5	
45.32	651	995	277	6.2	77	1.8	2.1	
45.27	650.6	1143	250	6.7	55	1.5	1.1	
45.24	650.5	1501	265	10.5	47	1.4	0.9	Riffle
45.2	650.2	1036	245	6.3	58	1.3	2.5	
45.16	649.8	1028	275	5.9	73	1.2	2.5	
45.13	649.7	1270	298	5.8	70	1.3	5.7	
45.1	649.5	1145	257	9.7	58	1.6	1.4	Riffle
45.08	649.1	912	266	5.5	77	1.2	6.8	
45.05	648.8	991	208	5.8	44	1.3	6.4	
45.02	648.6	934	209	7.1	47	1.4	1.1	
45	648.4	900	207	7.5	48	1.5	1.1	Riffle
44.99	648.4	1024	210	7.5	43	1.5	1.1	
44.98	648.4	1079	208	8.4	40	1.6	1.1	

River Mile	Bankfull Water Surface Elevation (ft)	Flow Area (ft ²)	Top Width (ft)	Maximum Depth (ft)	Width/Depth Ratio ²	Entrenchment Ratio ³	Bank Height Ratio ⁴	Feature ⁵
44.97	648.4	1245	313	8.7	79	1.1	3.4	Riffle
44.96	648.2	921	238	5.2	62	1.4	1.1	
44.95	648.1	883	217	5.2	54	1.6	1.1	
44.94	648.1	1443	341	6.7	81	1.2	4.2	
44.93	648.1	1922	342	8.7	61	1.2	3.4	
44.89	648.1	2468	303	11.1	37	1.2	3.2	

Notes:

1. All bankfull dimensions calculated at the 1.5-year return interval (bankfull) flow from the future conditions 1-D HEC-RAS model at all model cross sections.
2. The width/depth ratio is the bankfull water surface width divided by the mean depth at bankfull flow.
3. The entrenchment ratio is the bankfull water surface width at two times the maximum bankfull riffle depth (called the flood-prone width) divided by the bankfull width. Entrenchment ratios in this table are not all calculated at riffle features.
4. The bank height ratio is the height of the top of the lowest bank divided by the maximum depth at bankfull flow. Bank height ratios in this table are not all calculated at riffle features.
5. Feature refers to the likely morphological feature present in the main channel.

Created by: EG 9/18/2024

Checked by: LN 9/19/2024

Table 12. 2-D Model Downstream Boundary Rating Curve

Stage (ft)	Flow (cfs)
643.00	0
644.07	400
644.79	800
645.37	1,200
645.88	1,600
646.34	2,000
646.77	2,400
647.16	2,800
647.54	3,200
647.89	3,600
648.23	4,000
648.56	4,400
648.88	4,800
649.18	5,200
649.48	5,600
649.76	6,000
650.04	6,400
650.31	6,800
650.57	7,200
650.83	7,600
651.08	8,000
651.32	8,400
651.56	8,800
651.79	9,200
652.02	9,600
652.24	10,000
652.46	10,400
652.68	10,800
652.90	11,200
653.10	11,600
653.31	12,000
653.46	12,400
653.70	12,800
653.90	13,200
654.09	13,600
654.28	14,000

1. Rating curve developed from 1-D HEC-RAS model output at cross section 44.86.

Created by: EG 6/24/2022

Checked by: LN 7/21/2022

Table 13. Area 4 Stable Particles

River Mile	2D Maximum Velocity (ft/s)	Stable Particle Size Based on 2D Velocity ¹ (in)	2D Maximum Shear Stress (lbs/ft ²)	Stable Particle Size Based on 2D Shear Stress ² (in)	Stable Particle Size ³ (in)
47.27	6.2	2.8	0.45	1.1	2.8
47.14	5.6	2.1	0.37	0.9	2.1
47.07	5.1	1.6	0.97	2.4	2.4
47	5.8	2.3	0.57	1.4	2.3
46.92	6.0	2.4	0.72	1.7	2.4
46.84	6.2	2.7	0.93	2.3	2.7
46.8	6.3	2.8	0.58	1.4	2.8
46.67	6.3	2.7	1.05	2.6	2.7
46.46	8.4	5.4	1.98	4.7	5.4
46.36	8.5	5.8	2.4	5.7	5.8
46.26	7.2	4.0	0.65	1.6	4.0
46.16	6.7	3.4	0.9	2.2	3.4
46.07	6.7	3.4	0.6	1.5	3.4
46	7.1	4.1	1.06	2.6	4.1
45.89	6.7	3.3	0.93	2.3	3.3
45.83	6.3	2.8	0.44	1.1	2.8
45.77	6.2	2.7	0.58	1.4	2.7
45.72	6.2	2.8	0.58	1.4	2.8
45.68	6.0	2.4	1.18	2.8	2.8
45.64	6.3	2.7	1.29	3.1	3.1
45.59	6.5	3.1	2.91	6.9	6.9
45.56	6.9	3.6	3.29	7.7	7.7
45.51	6.3	2.8	1.61	3.8	3.8
45.48	6.3	2.9	2.42	5.7	5.7
45.44	6.3	2.8	1.68	4.0	4.0
45.41	5.8	2.3	0.59	1.5	2.3
45.37	6.0	2.5	0.9	2.2	2.5
45.35	6.8	3.4	1.53	3.6	3.6
45.32	8.3	5.7	4.58	10.8	10.8
45.27	8.5	6.1	2.05	4.9	6.1
45.24	8.1	5.0	1.26	3.0	5.0
45.2	7.0	3.7	1.04	2.5	3.7
45.16	6.8	3.5	0.9	2.2	3.5
45.13	6.5	3.2	0.9	2.2	3.2
45.1	6.0	2.4	0.77	1.9	2.4
45.08	6.5	3.1	0.88	2.1	3.1
45.05	6.8	3.5	0.88	2.1	3.5
45.02	6.8	3.4	0.87	2.1	3.4
45	7.0	3.7	0.79	1.9	3.7
44.99	6.8	3.3	0.55	1.4	3.3
44.98	6.5	3.0	0.45	1.1	3.0
44.97	6.2	2.6	0.55	1.4	2.6
44.96	6.6	3.3	0.89	2.2	3.3
44.95	6.6	3.3	0.89	2.2	3.3
44.94	6.2	2.7	0.78	1.9	2.7
44.93	5.5	2.0	0.59	1.5	2.0
44.89	5.2	1.6	0.27	0.7	1.6

Notes:

1. Stable particles calculated based on 2-D HEC-RAS 100-year velocity using Maynard method (NRCS, 2007) with factor of safety = 1.2.
2. Stable particles calculated based on 2-D HEC-RAS 100-year shear stress and critical shear thresholds from Berenbrock and Tranmer (2008) with factor of safety = 1.2.
3. Stable particle size based on the most conservative particle size from 2D velocity and 2D shear stress.

Created by: EG 9/17/2024

Checked by: LN 9/17/2024



LEGEND:

— Area 3 Cross Sections

— Area 3 Centerline

0 260 520

SCALE, FEET

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Checked by EG/Date: 07/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

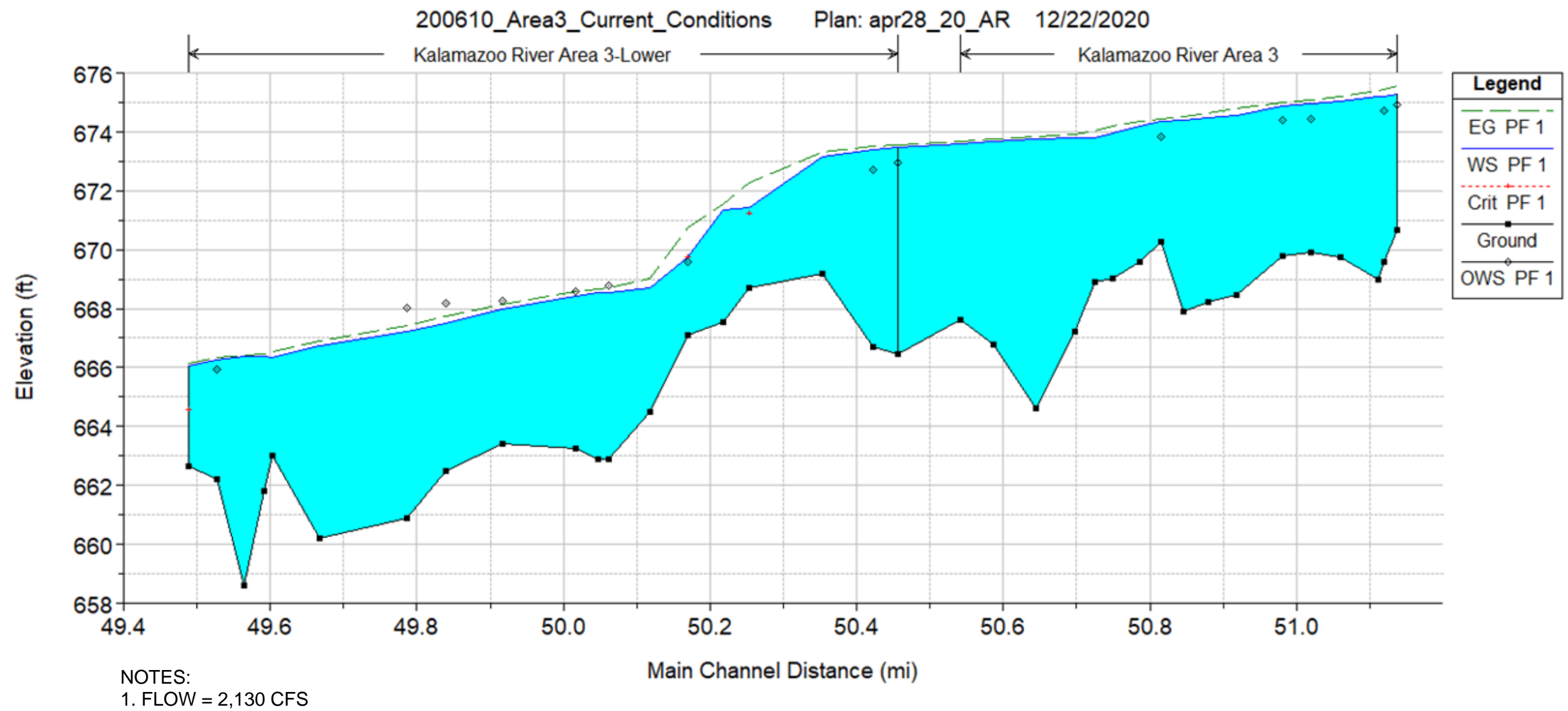
Kalamazoo
river cleanup group
AREAS 2, 3 AND 4 TCRA

Project 2000273

AREA 3 H&H MODEL CROSS
SECTION LOCATIONS

August 2022

Fig. 1



Prepared by LN/Date: 08/19/2021
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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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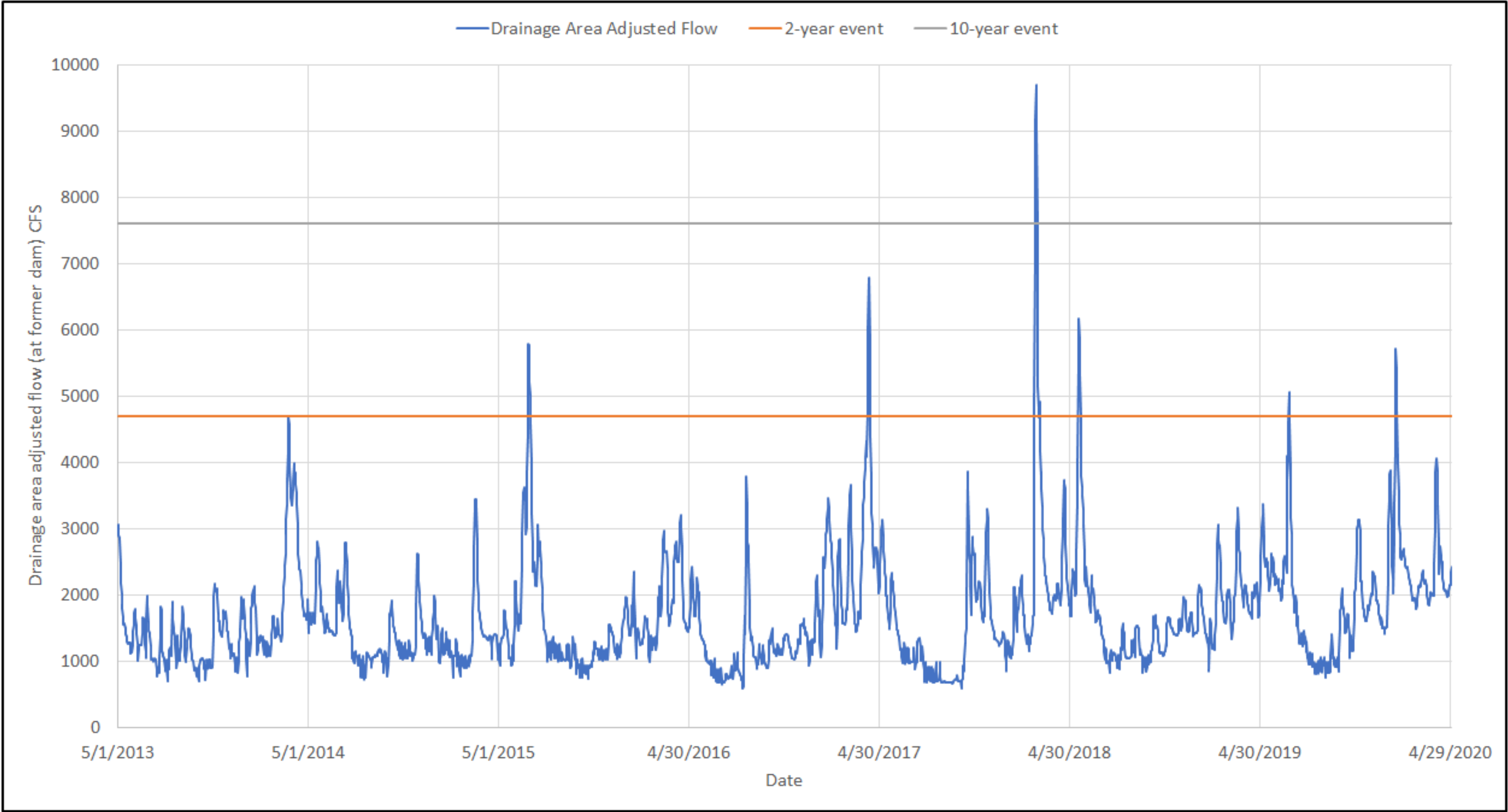


AREA 3 H&H MODEL
CALIBRATION


Project 2000273

August 2022

Fig. 2



Prepared by LN/Date: 08/19/2021
Checked by EG/Date: 07/8/2022


OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	AREA 3 UNSTEADY FLOW HYDROGRAPH WITH RETURN INTERVALS	
		Project 2000273	August 2022

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



- NOTES:
- 1. EXPOSED GLACIAL TILL FEATURES IN AREA 3
 - 2. PHOTOS TAKEN AT APPROXIMATELY RIVER MILE 50.25

Prepared by LN/Date: 08/19/2021
Checked by EG/Date: 07/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		AREA 3 GLACIAL TILL FEATURES
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	August 2022



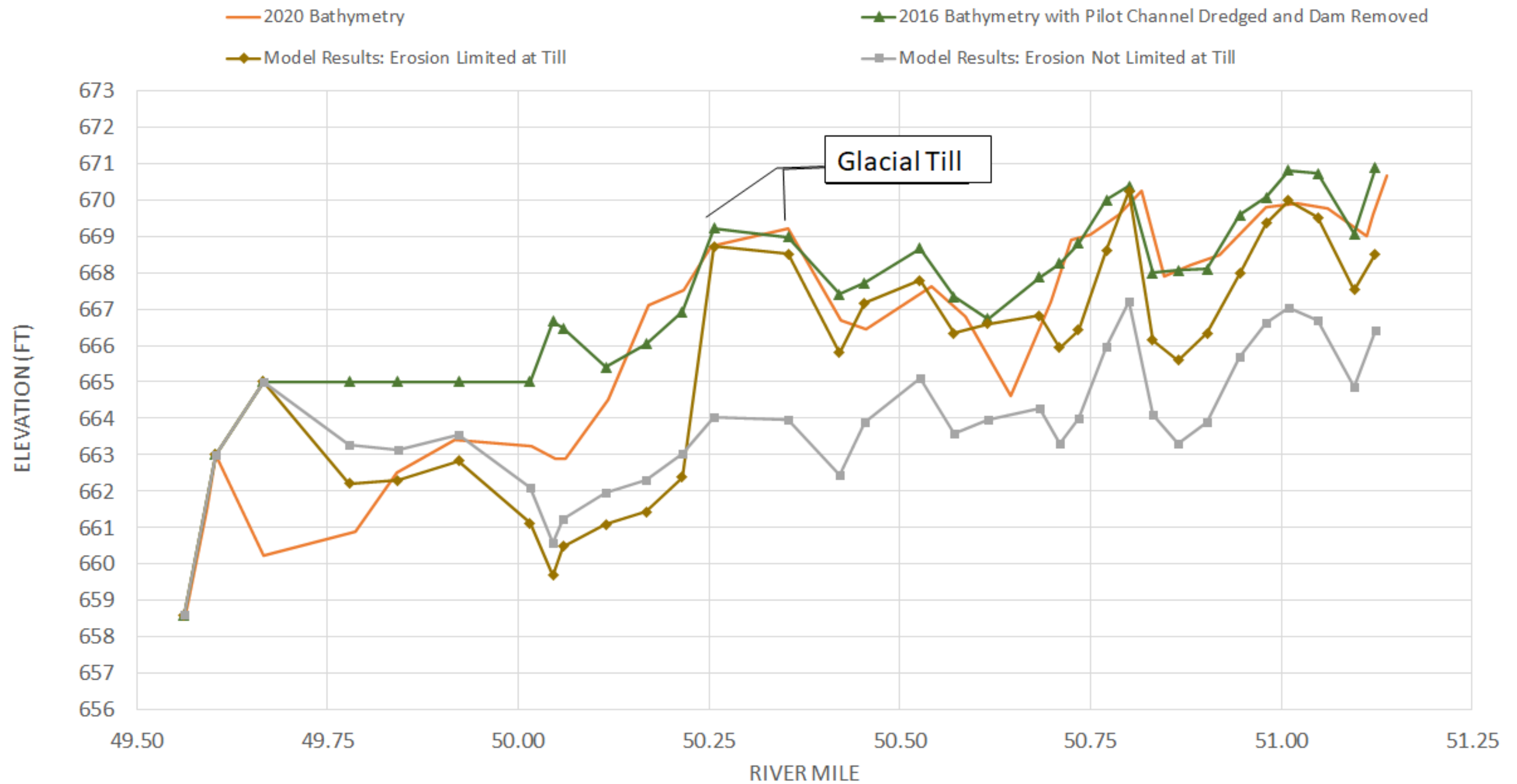
Surface substrate observed in Area 4 at approximately river mile 49.45, Fall 2020



Surface substrate observed in Area 3 at approximately river mile 50.15 ("The Narrows"), Fall 2020



Caddis fly casings observed on surface substrate in Area 3 near river mile 50.15, Fall 2020



Prepared by EG/Date: 08/19/2021
 Checked by LN/Date: 07/8/2022


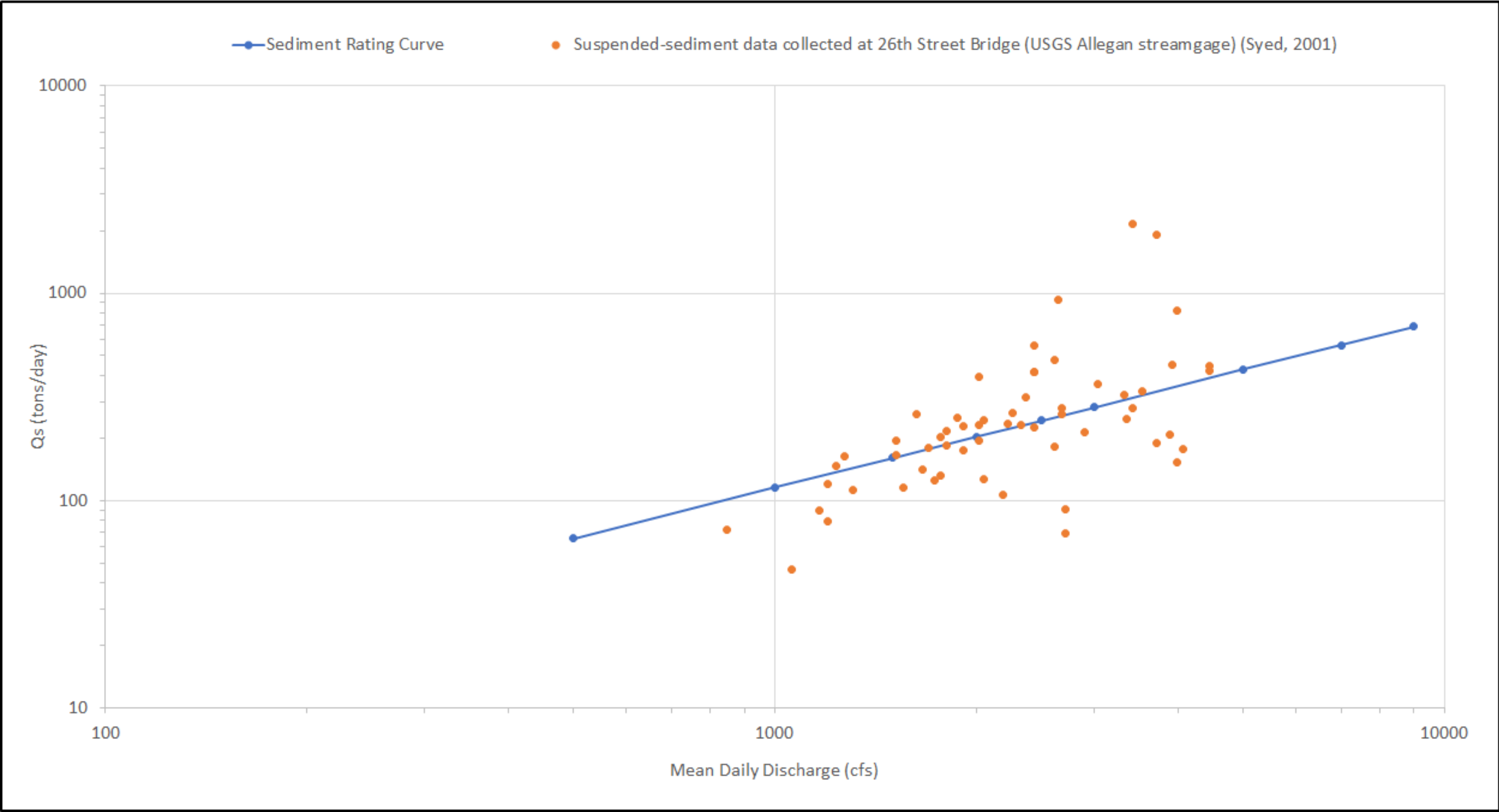
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		AREA 3 SENSITIVITY ANALYSIS ON ERODIBLE LIMITS	
		Project 2000273	August 2022

Fig. 5



Prepared by EG/Date: 08/19/2021
Checked by LN/Date: 07/8/2022


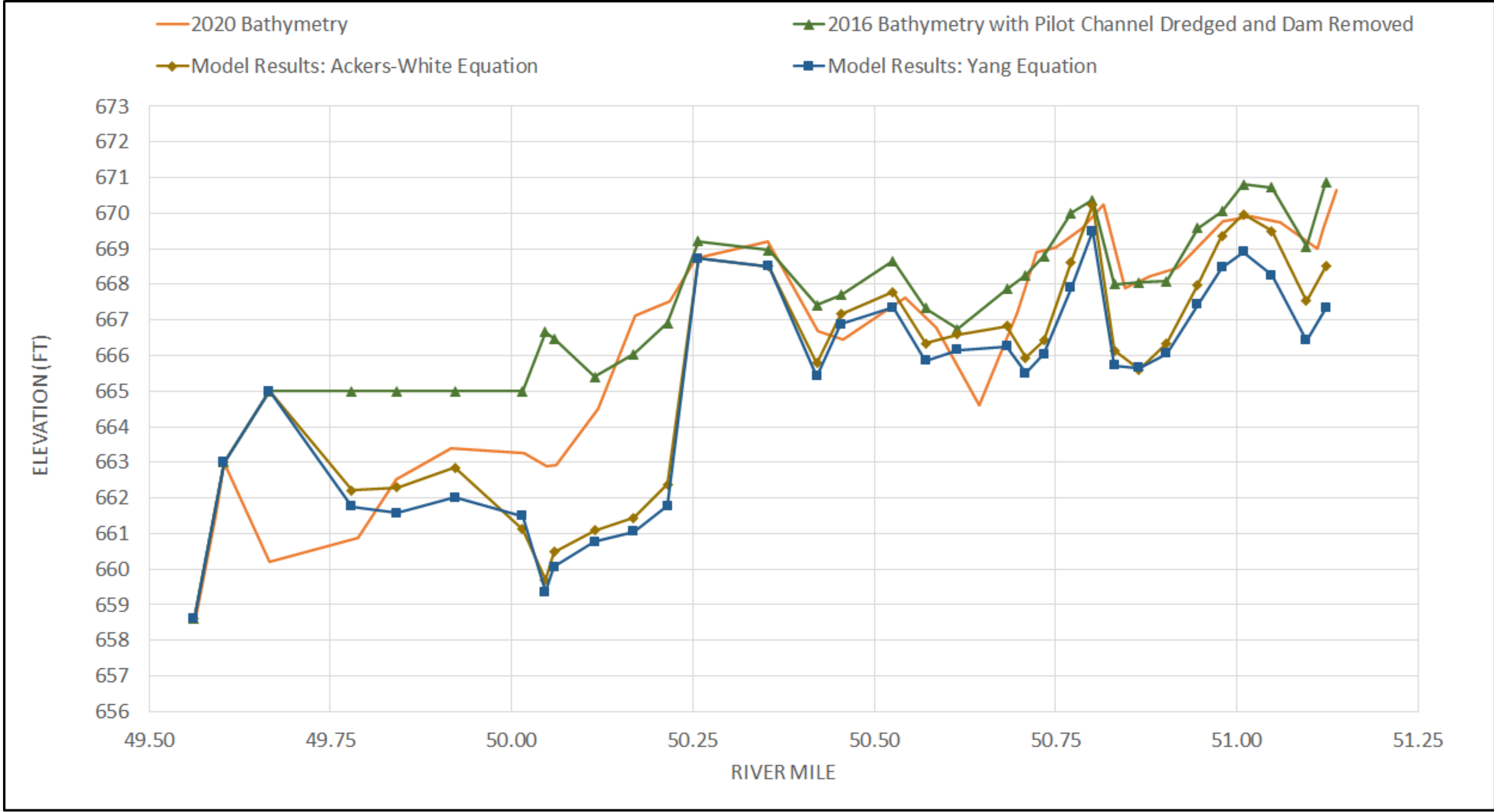
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		SEDIMENT RATING CURVE	
		Project 2000273	August 2022

Fig. 6



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



AREA 3 SEDIMENT
TRANSPORT CALIBRATION

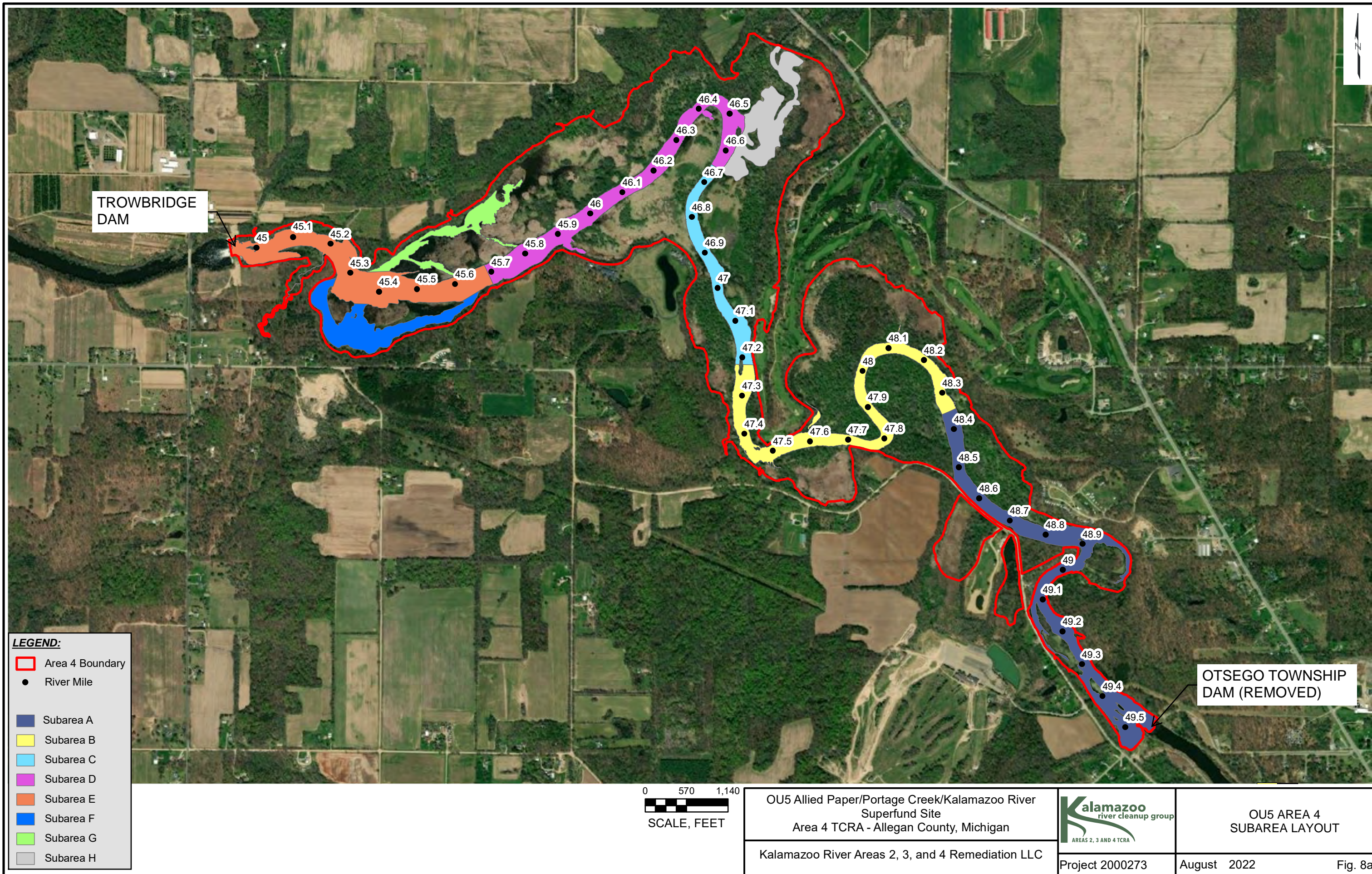
Prepared by EG/Date: 08/19/2021
Checked by LN/Date: 07/8/2022

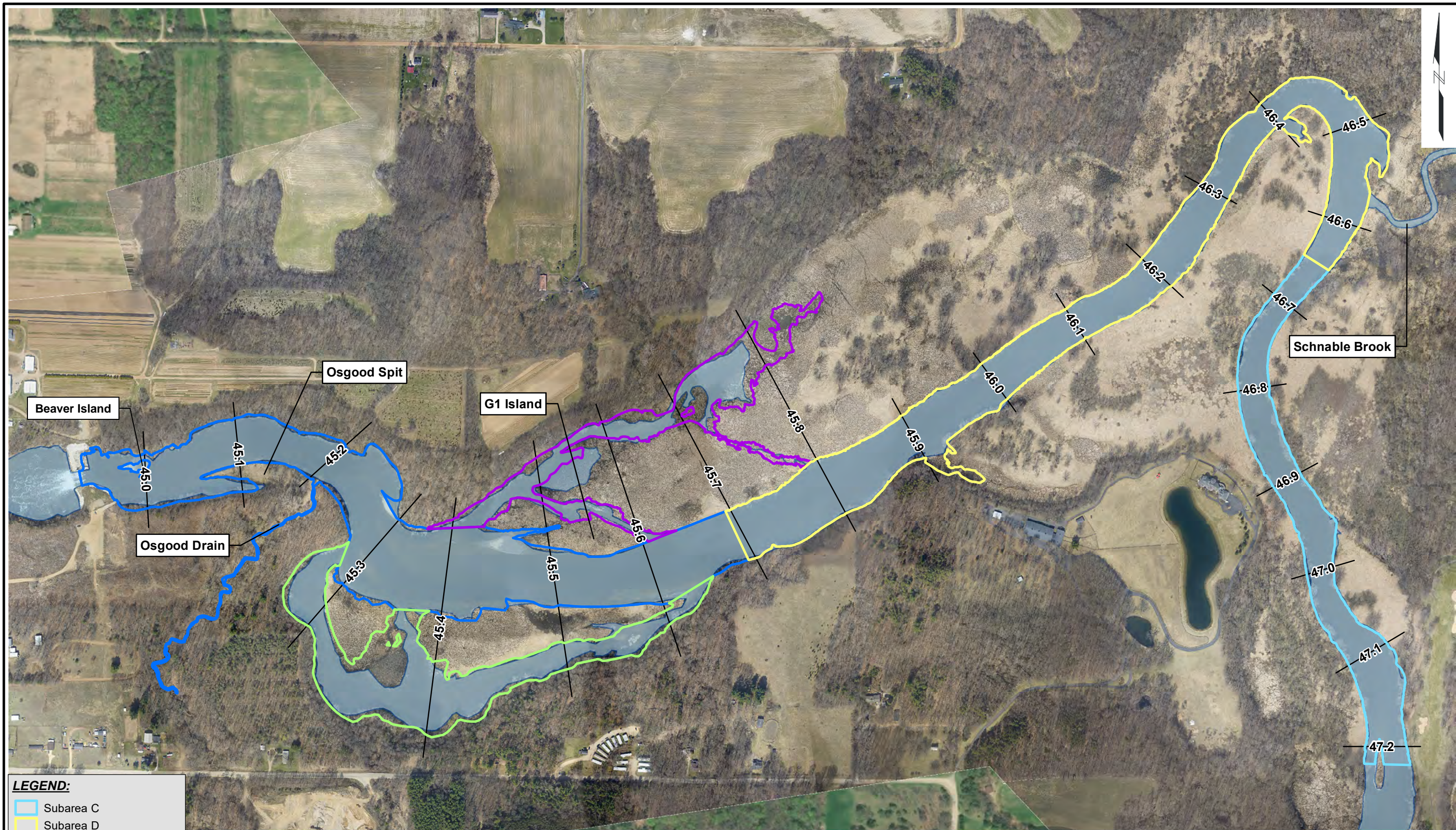
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

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Fig. 7





LEGEND:

- Subarea C
- Subarea D
- Subarea E
- Subarea F
- Subarea G
- 1/10th Mile Marker
- Approximate water surface

Prepared by AG/Date: 08/09/2021
 Checked by BB/Date: 08/09/2021



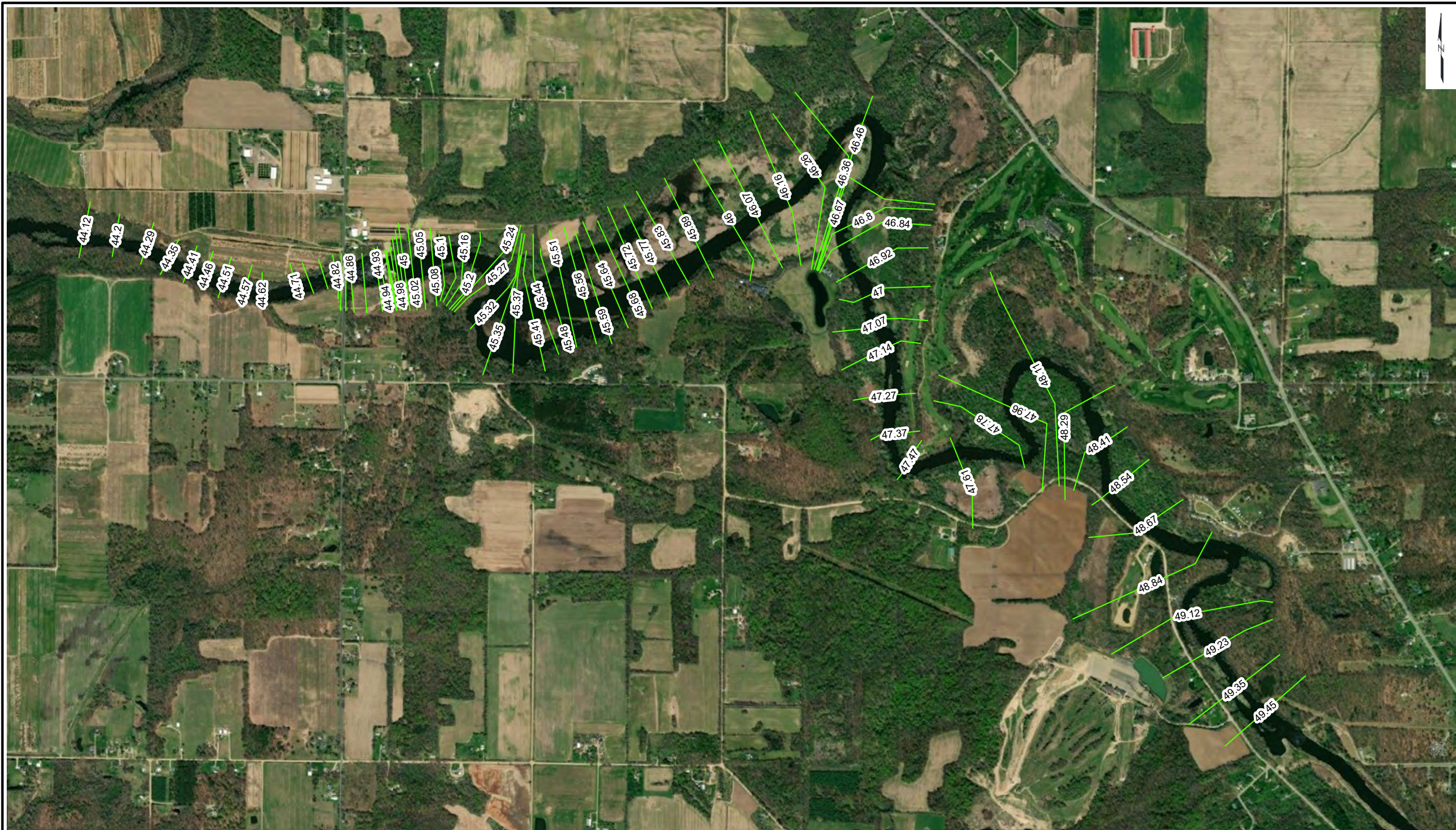
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

Kalamazoo
 river cleanup group
AREAS 2, 3 AND 4 TCRA

Project 2000273

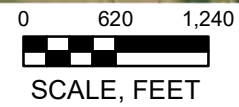
TCRA SUBAREA BOUNDARY MAP
August 2022

Fig. 8b



LEGEND:

Area 4 Cross Sections



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OU5 Allied Paper/Portage Creek/Kalamazoo River
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Area 4 TCRA - Allegan County, Michigan

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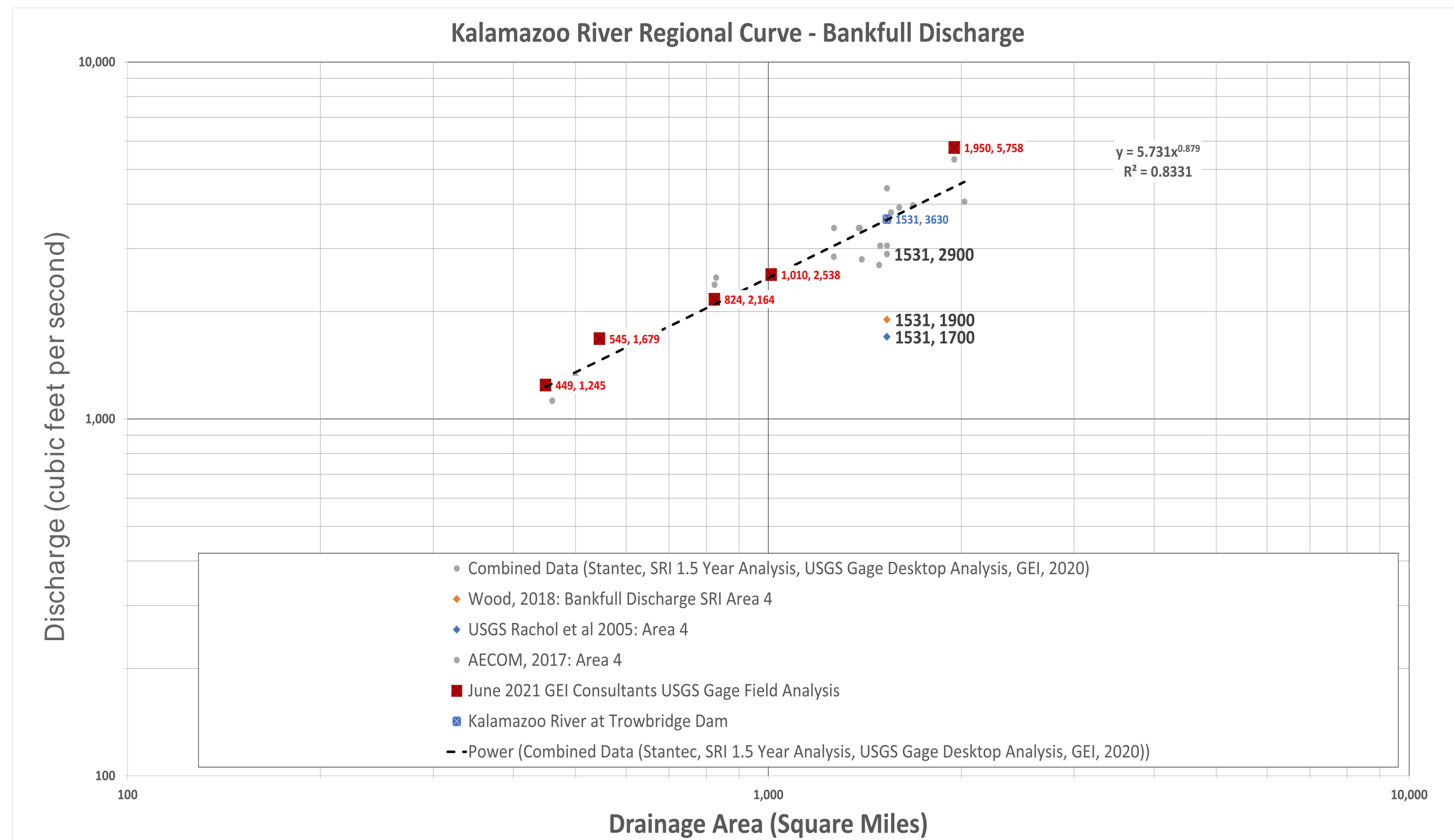
Kalamazoo
river cleanup group
AREAS 2, 3 AND 4 TCRA

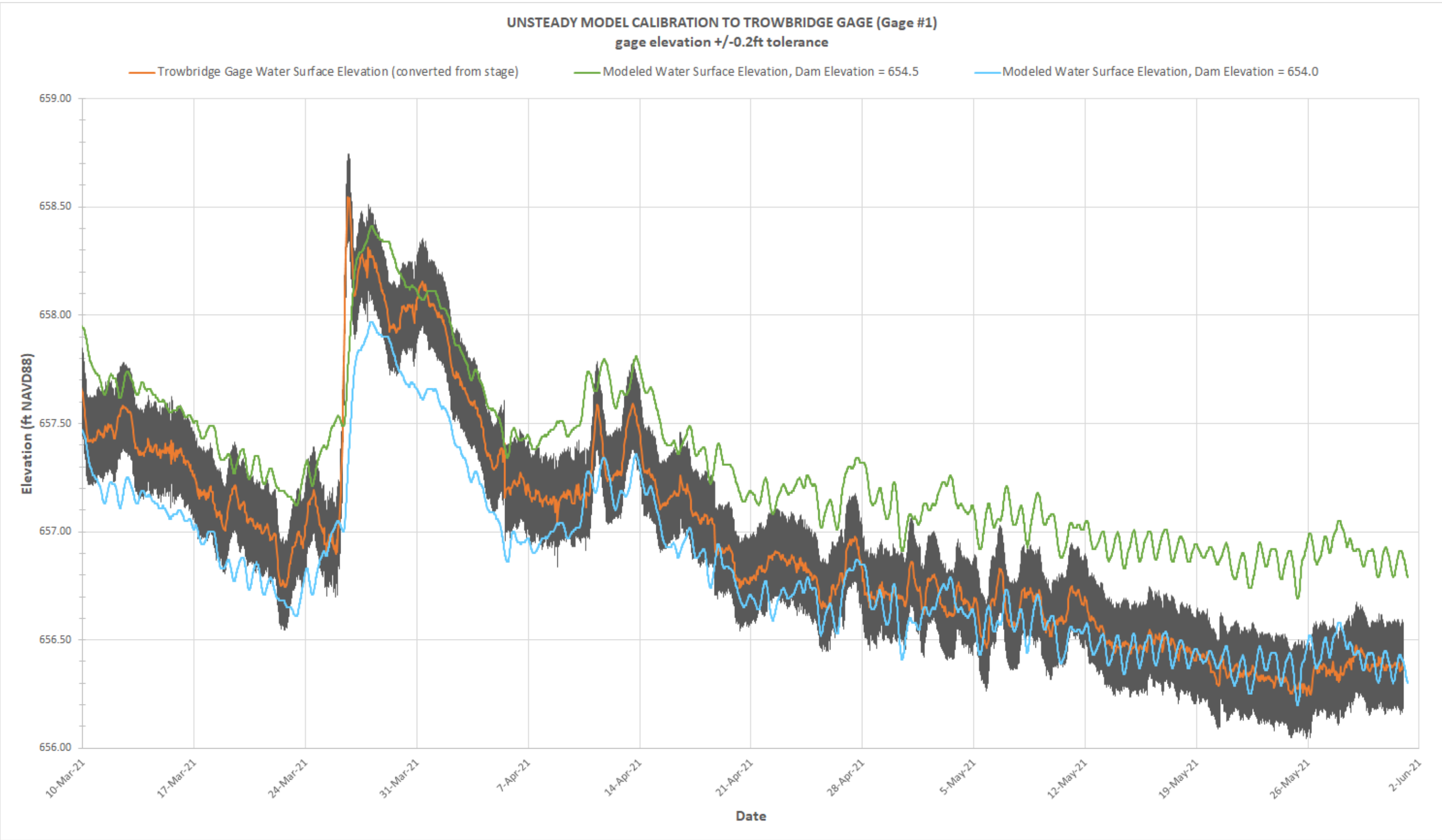
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AREA 4 MODEL CROSS
SECTION LOCATIONS

August 2022

Fig. 9





Prepared by LN/Date: 08/19/2021
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Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

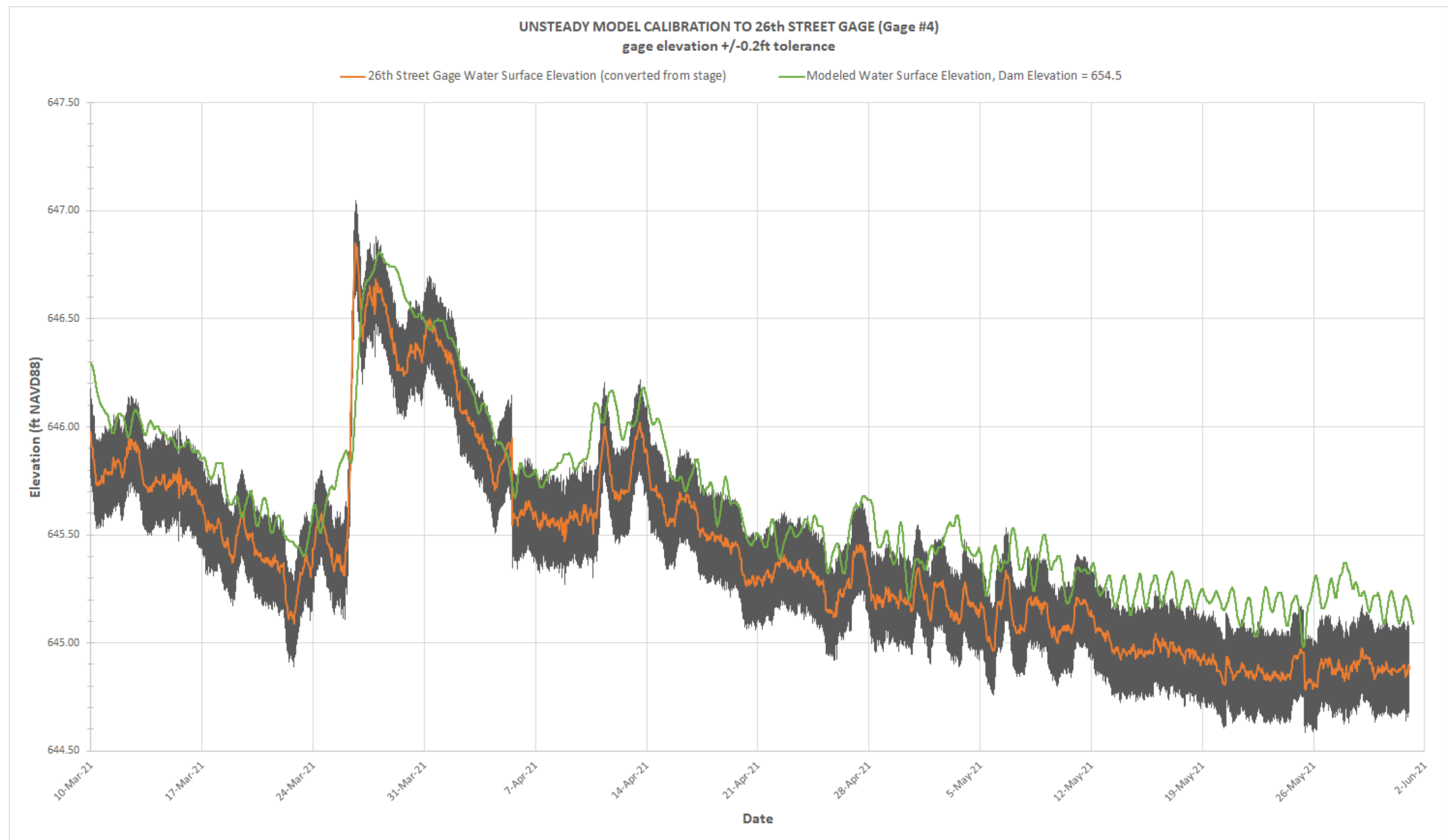


AREA 4 UNSTEADY H&H MODEL
CALIBRATION TROWBRIDGE
GAGE

Project 2000273

August 2022

Fig. 11a



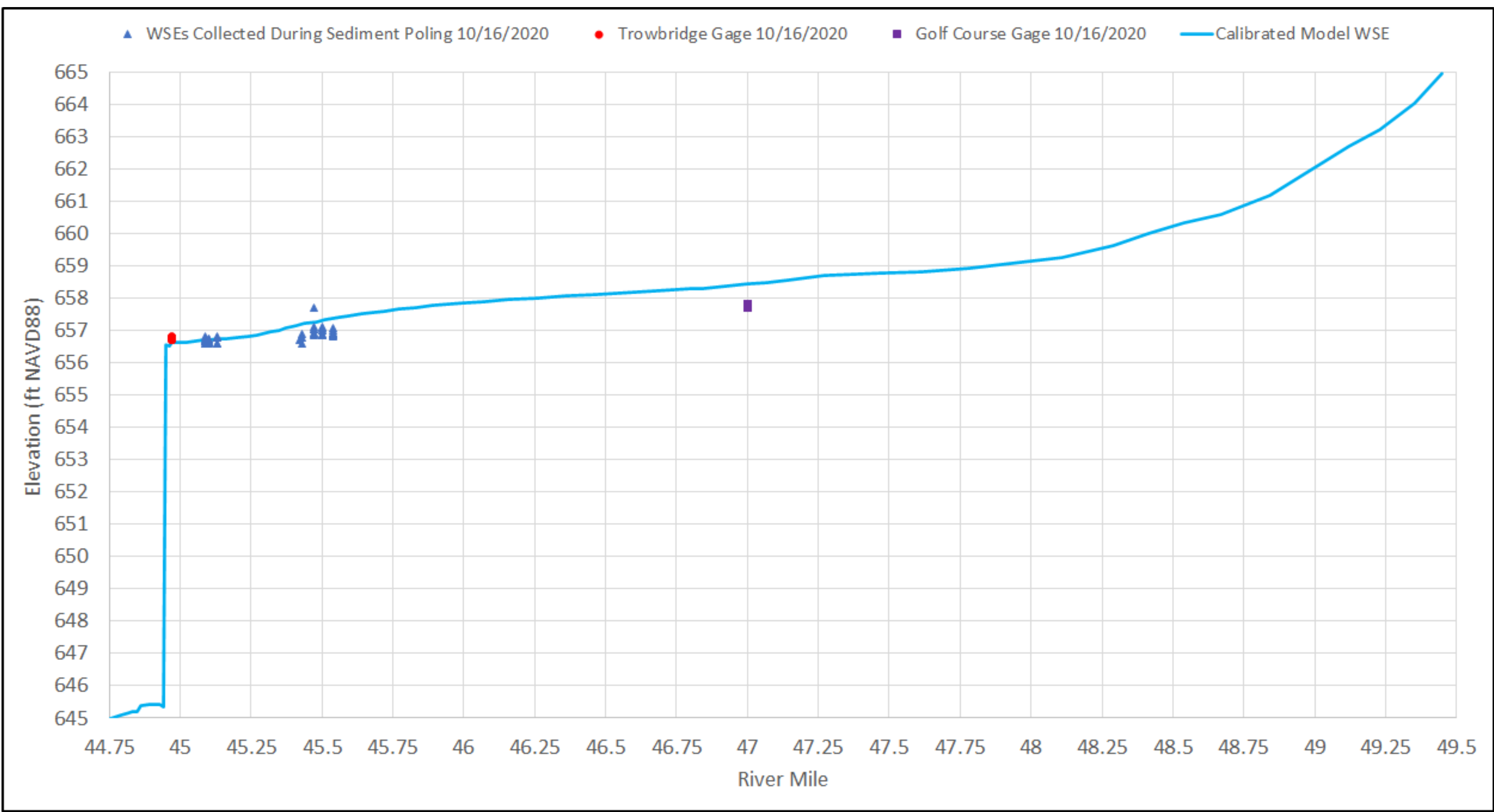
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OU5 Allied Paper/Portage Creek/Kalamazoo River
 Superfund Site
 Area 4 TCRA - Allegan County, Michigan
 Kalamazoo River Areas 2, 3, and 4 Remediation LLC



AREA 4 UNSTEADY H&H
 MODEL CALIBRATION 26TH
 STREET GAGE
 Project 2000273
 August 2022

Fig. 11b



NOTES:
1. FLOW = 1,110 CFS

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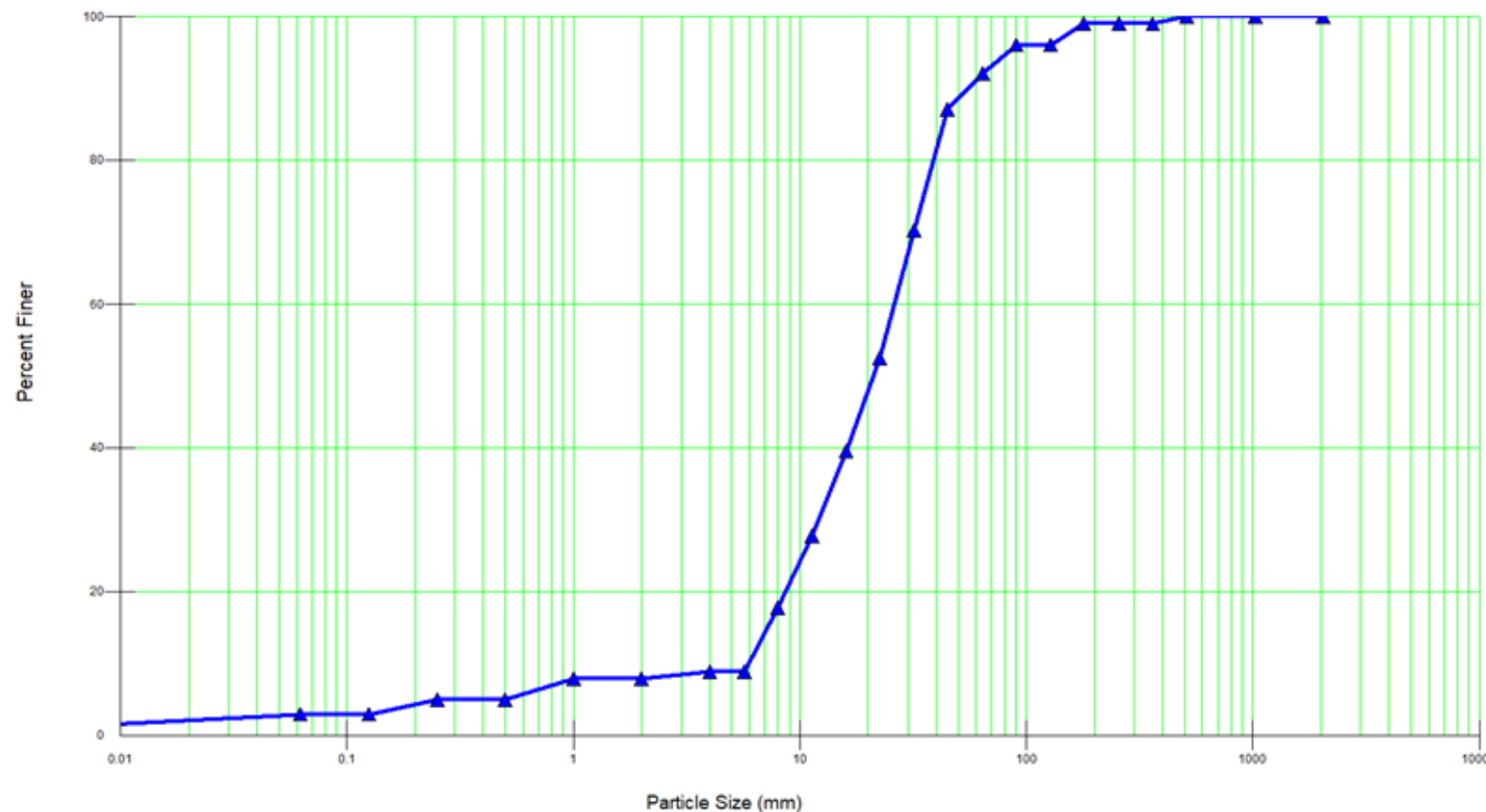
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	AREA 4 1-D MODEL CALIBRATION PROFILE PLOT	
		Project 200273	August 2022

Fig. 12



Size (mm)	TOT #	ITEM %	CUM %
0 - 0.062	3	2.97	2.97
0.062 - 0.125	0	0.00	2.97
0.125 - 0.25	2	1.98	4.95
0.25 - 0.50	0	0.00	4.95
0.50 - 1.0	3	2.97	7.92
1.0 - 2.0	0	0.00	7.92
2.0 - 4.0	1	0.99	8.91
4.0 - 5.7	0	0.00	8.91
5.7 - 8.0	9	8.91	17.82
8.0 - 11.3	10	9.90	27.72
11.3 - 16.0	12	11.88	39.60
16.0 - 22.6	13	12.87	52.48
22.6 - 32.0	18	17.82	70.30
32 - 45	17	16.83	87.13
45 - 64	5	4.95	92.08
64 - 90	4	3.96	96.04
90 - 128	0	0.00	96.04
128 - 180	3	2.97	99.01
180 - 256	0	0.00	99.01
256 - 362	0	0.00	99.01
362 - 512	1	0.99	100.00
512 - 1024	0	0.00	100.00
1024 - 2048	0	0.00	100.00
Bedrock	0	0.00	100.00

Particle Size Analysis

D16 (mm)	7.53
D35 (mm)	14.18
D50 (mm)	21.33
D84 (mm)	42.58
D95 (mm)	83.17
D100 (mm)	511.98
Silt/Clay (%)	2.97
Sand (%)	4.95
Gravel (%)	84.16
Cobble (%)	6.93
Boulder (%)	0.99
Bedrock (%)	0

Total Particles = 101

D50 21.33 mm

NOTES:
1. TRANSECT NOT CONTINUED ON LEFT SIDE OF ISLAND (LOOKING DOWNSTREAM)
DUE TO UNSAFE WADING CONDITIONS

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Checked by EG/Date: 07/8/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

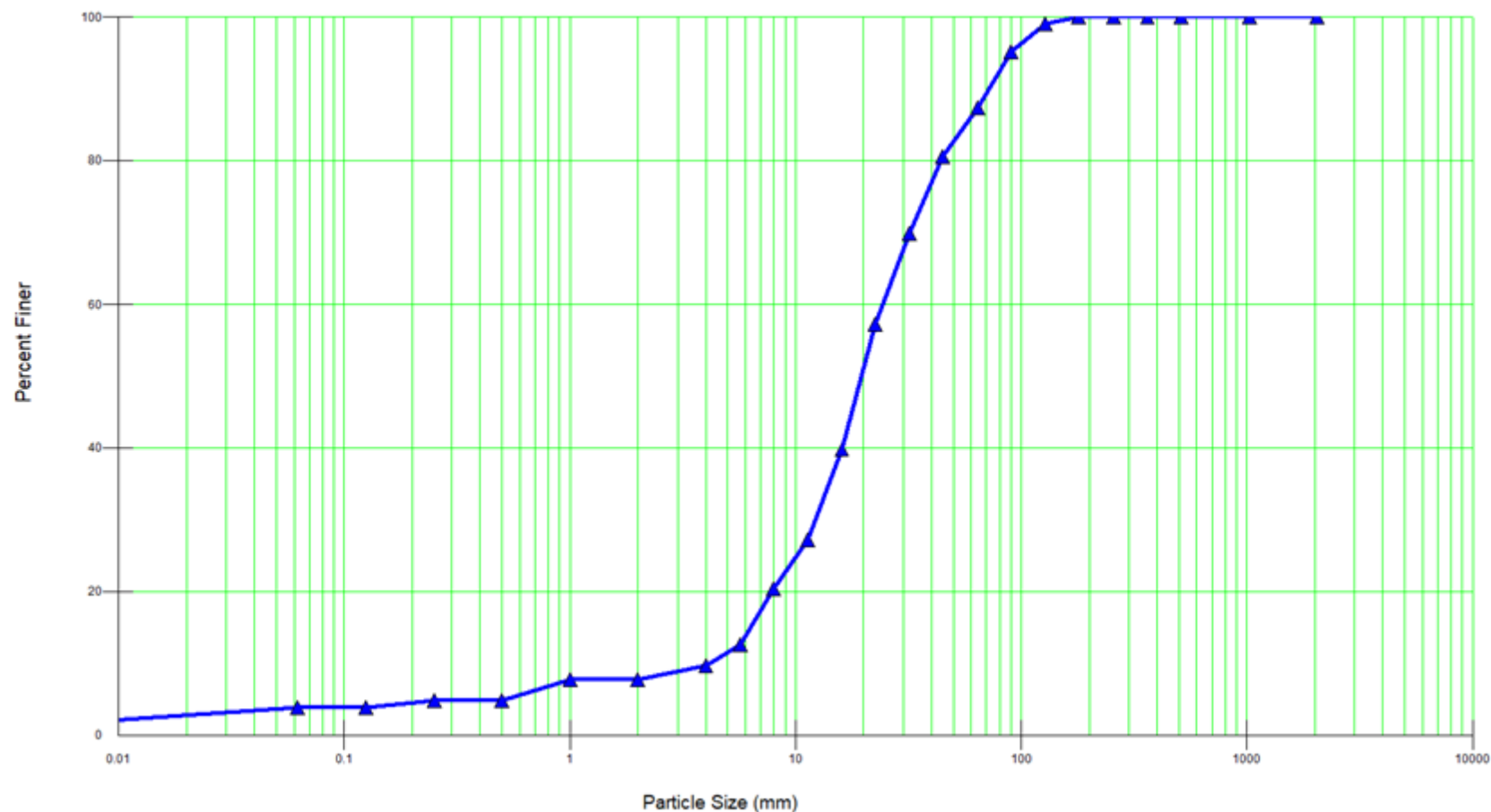


AREA 4 PEBBLE COUNT
NEAR RIVER MILE 49.5

Project 2000273

August 2022

Fig. 13a



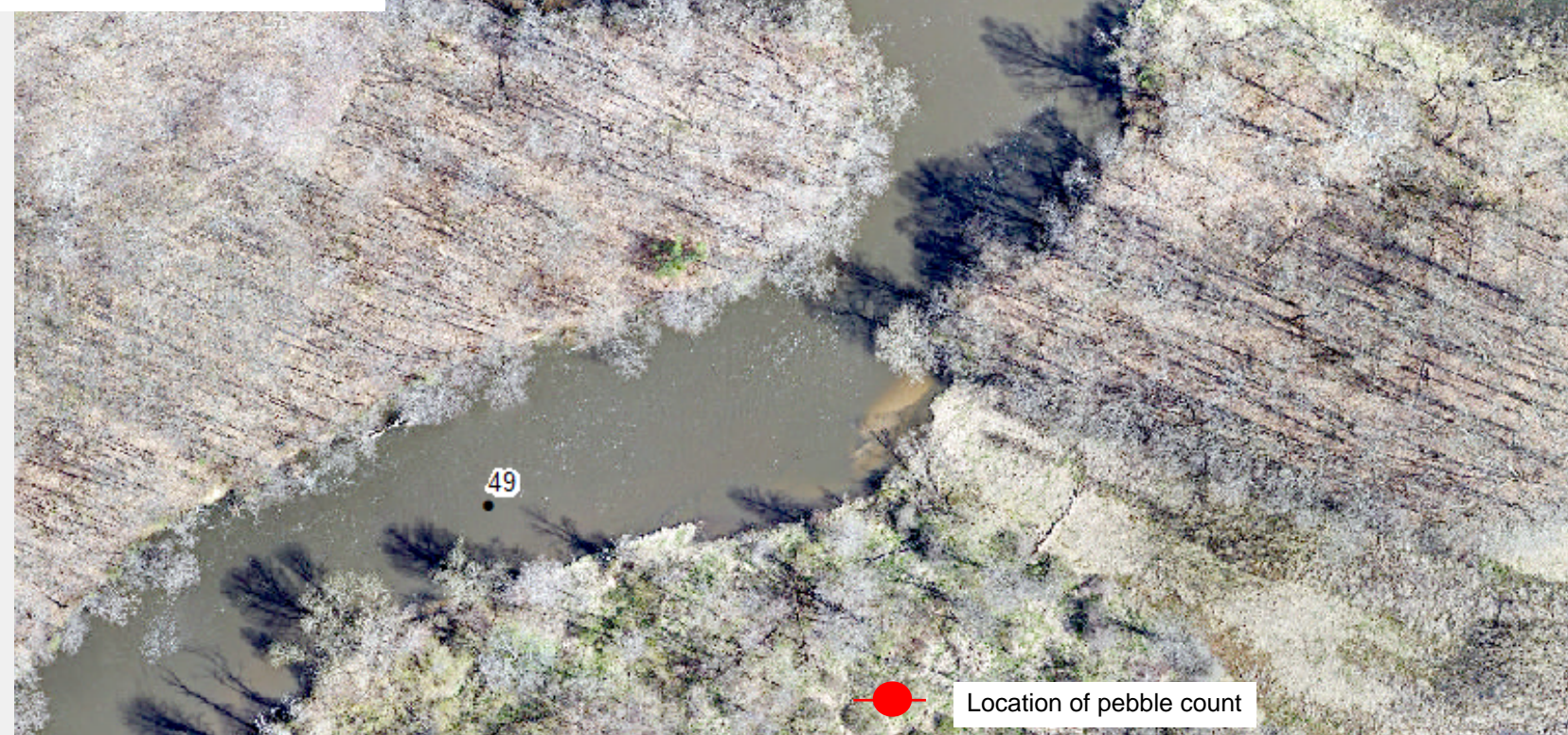
Size (mm)	TOT #	ITEM %	CUM %
0 - 0.062	4	3.88	3.88
0.062 - 0.125	0	0.00	3.88
0.125 - 0.25	1	0.97	4.85
0.25 - 0.50	0	0.00	4.85
0.50 - 1.0	3	2.91	7.77
1.0 - 2.0	0	0.00	7.77
2.0 - 4.0	2	1.94	9.71
4.0 - 5.7	3	2.91	12.62
5.7 - 8.0	8	7.77	20.39
8.0 - 11.3	7	6.80	27.18
11.3 - 16.0	13	12.62	39.81
16.0 - 22.6	18	17.48	57.28
22.6 - 32.0	13	12.62	69.90
32 - 45	11	10.68	80.58
45 - 64	7	6.80	87.38
64 - 90	8	7.77	95.15
90 - 128	4	3.88	99.03
128 - 180	1	0.97	100.00
180 - 256	0	0.00	100.00
256 - 362	0	0.00	100.00
362 - 512	0	0.00	100.00
512 - 1024	0	0.00	100.00
1024 - 2048	0	0.00	100.00
Bedrock	0	0.00	100.00

Particle Size Analysis

D16 (mm)	6.7
D35 (mm)	14.21
D50 (mm)	19.85
D84 (mm)	54.56
D95 (mm)	89.5
D100 (mm)	179.99
Silt/Clay (%)	3.88
Sand (%)	3.89
Gravel (%)	79.61
Cobble (%)	12.62
Boulder (%)	0
Bedrock (%)	0

Total Particles = 103

D50 19.85 mm



Location of pebble count

NOTES:
1. TRANSECT LOCATION CHOSEN FOR SAFEST WADING CONDITIONS. WADING CONDITIONS DOWNSTREAM OF RIVER MILE 48.9 WERE UNSAFE DUE TO HIGH VELOCITIES.

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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

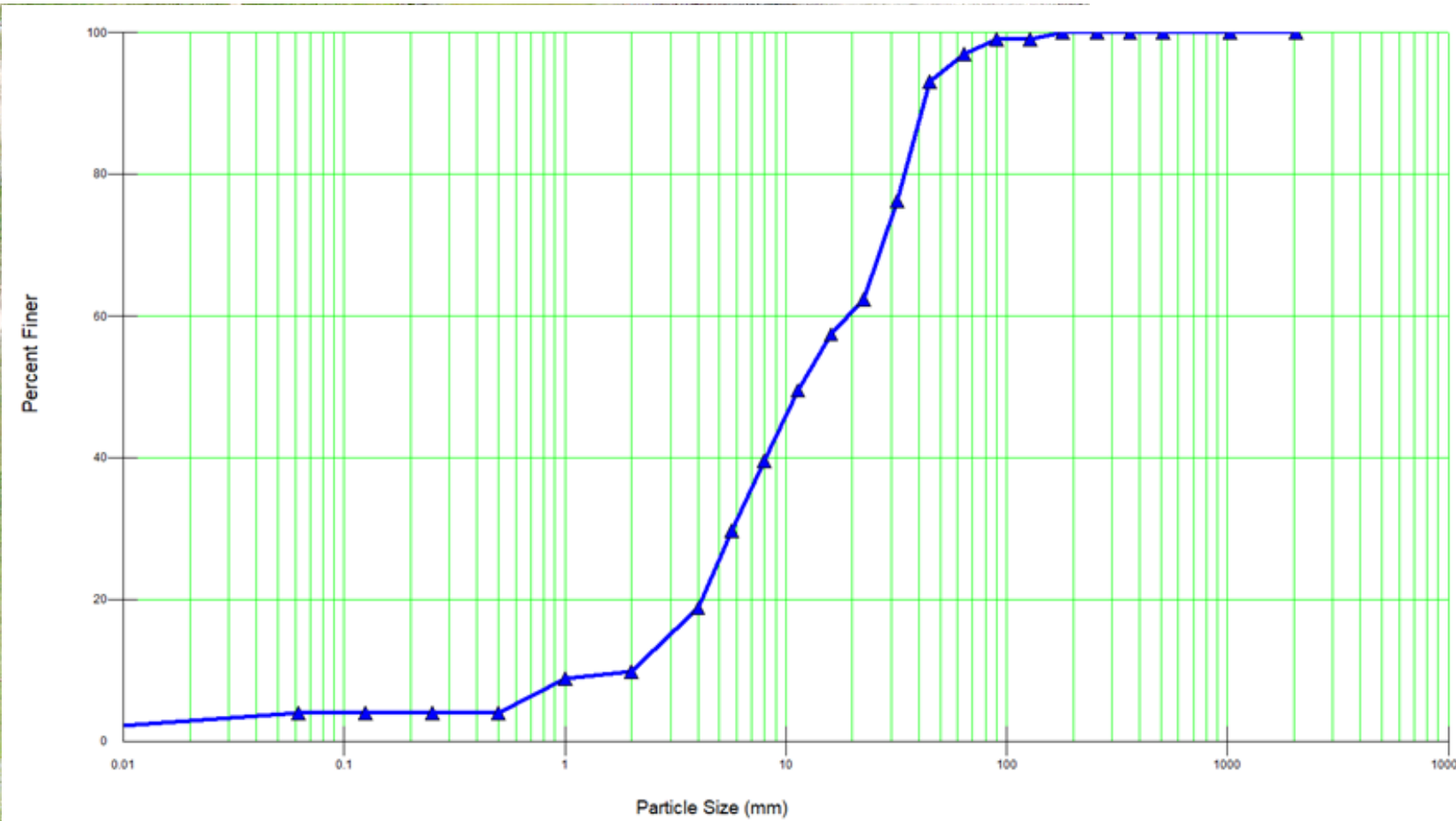


AREA 4 PEBBLE COUNT
NEAR RIVER MILE 48.9

Project 2000273

August 2022

Fig. 13b



Size (mm)	TOT #	ITEM %	CUM %
0 - 0.062	4	3.96	3.96
0.062 - 0.125	0	0.00	3.96
0.125 - 0.25	0	0.00	3.96
0.25 - 0.50	0	0.00	3.96
0.50 - 1.0	5	4.95	8.91
1.0 - 2.0	1	0.99	9.90
2.0 - 4.0	9	8.91	18.81
4.0 - 5.7	11	10.89	29.70
5.7 - 8.0	10	9.90	39.60
8.0 - 11.3	10	9.90	49.50
11.3 - 16.0	8	7.92	57.43
16.0 - 22.6	5	4.95	62.38
22.6 - 32.0	14	13.86	76.24
32 - 45	17	16.83	93.07
45 - 64	4	3.96	97.03
64 - 90	2	1.98	99.01
90 - 128	0	0.00	99.01
128 - 180	1	0.99	100.00
180 - 256	0	0.00	100.00
256 - 362	0	0.00	100.00
362 - 512	0	0.00	100.00
512 - 1024	0	0.00	100.00
1024 - 2048	0	0.00	100.00
Bedrock	0	0.00	100.00

Particle Size Analysis

D16 (mm)	3.37
D35 (mm)	6.93
D50 (mm)	11.6
D84 (mm)	37.99
D95 (mm)	54.26
D100 (mm)	179.99
Silt/Clay (%)	3.96
Sand (%)	5.94
Gravel (%)	87.13
Cobble (%)	2.97
Boulder (%)	0
Bedrock (%)	0

Total Particles = 101
D50 11.6 mm

OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



AREA 4 PEBBLE COUNT
NEAR RIVER MILE 47.3

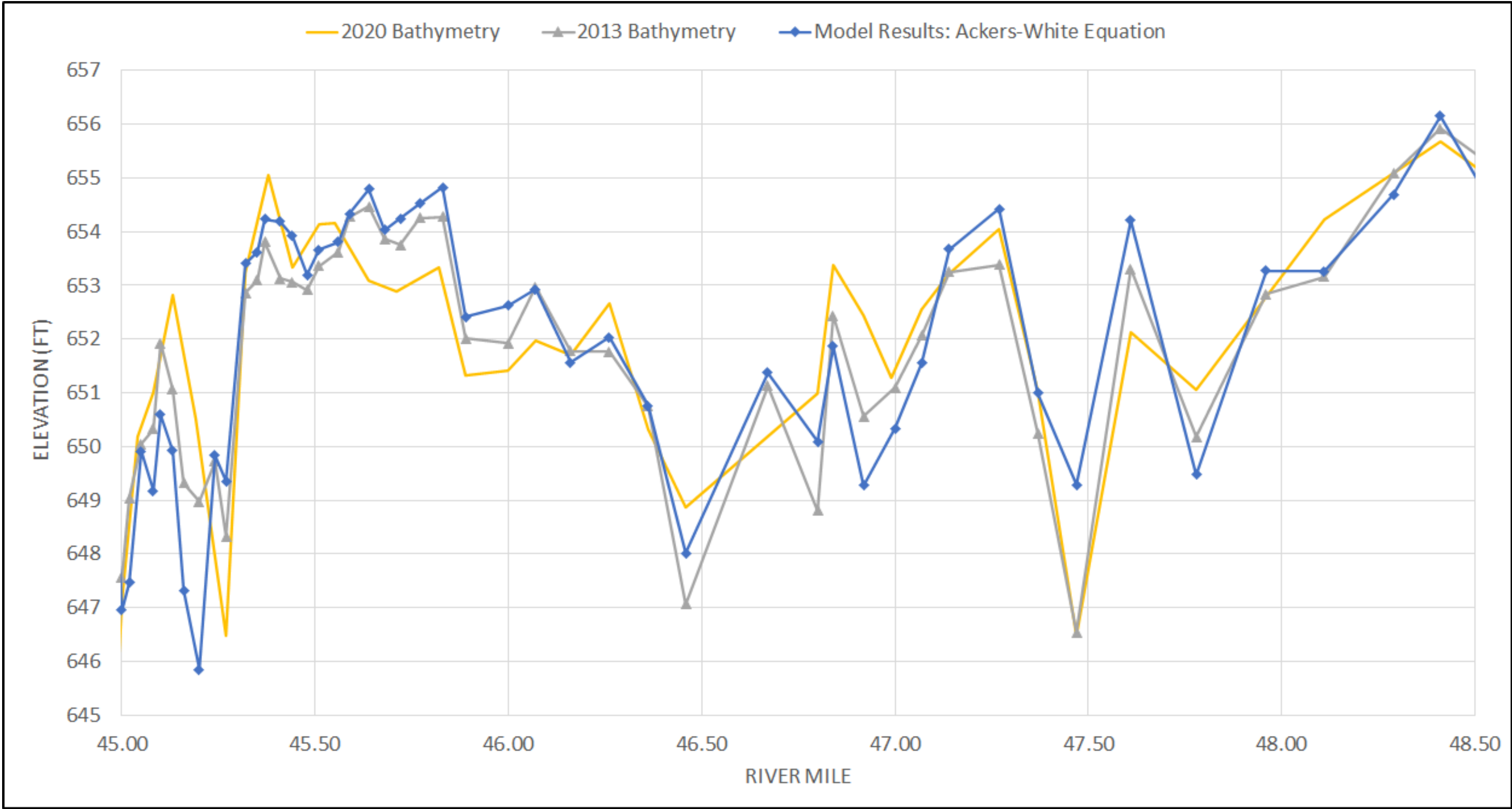
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

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August 2022

Fig. 13c

Prepared by LN/Date: 08/19/2021
Checked by EG/Date: 07/8/2022

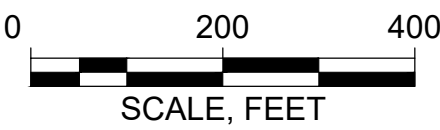
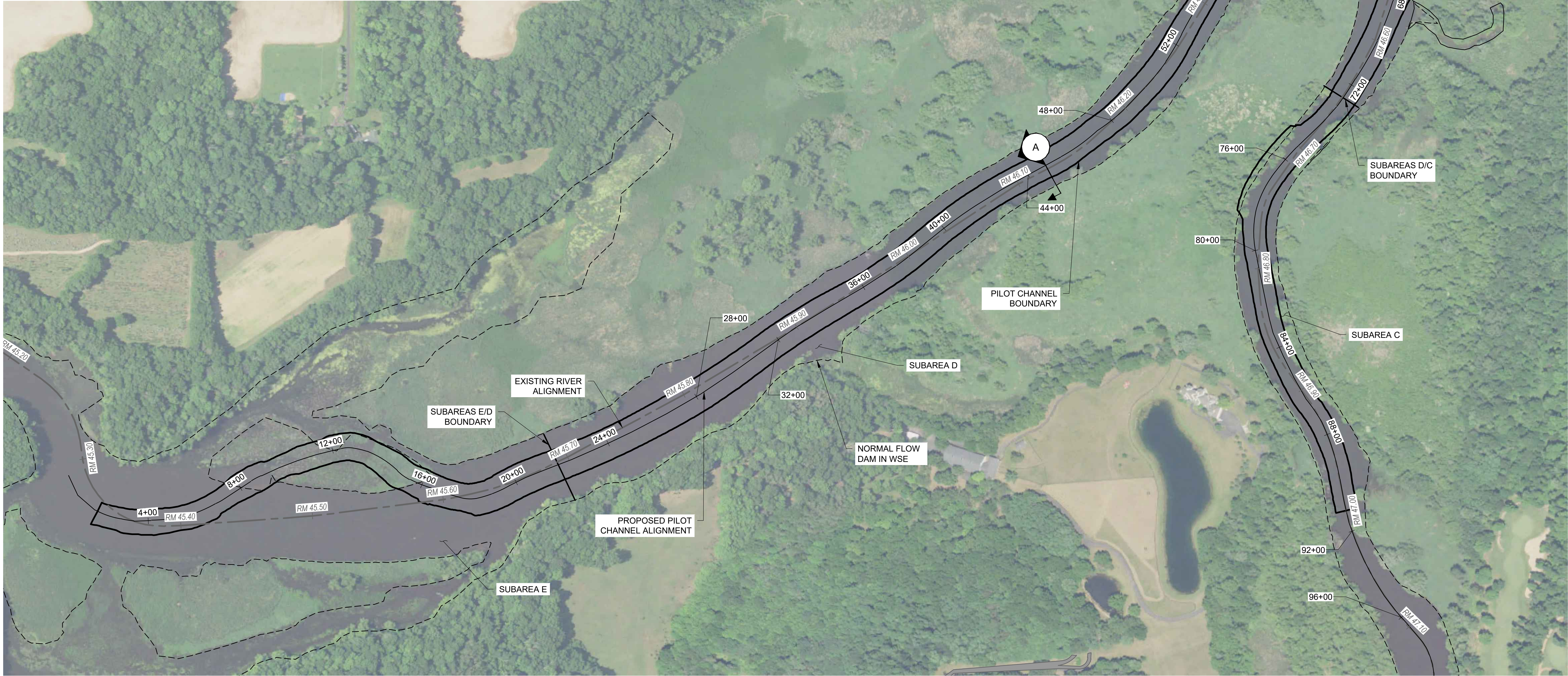
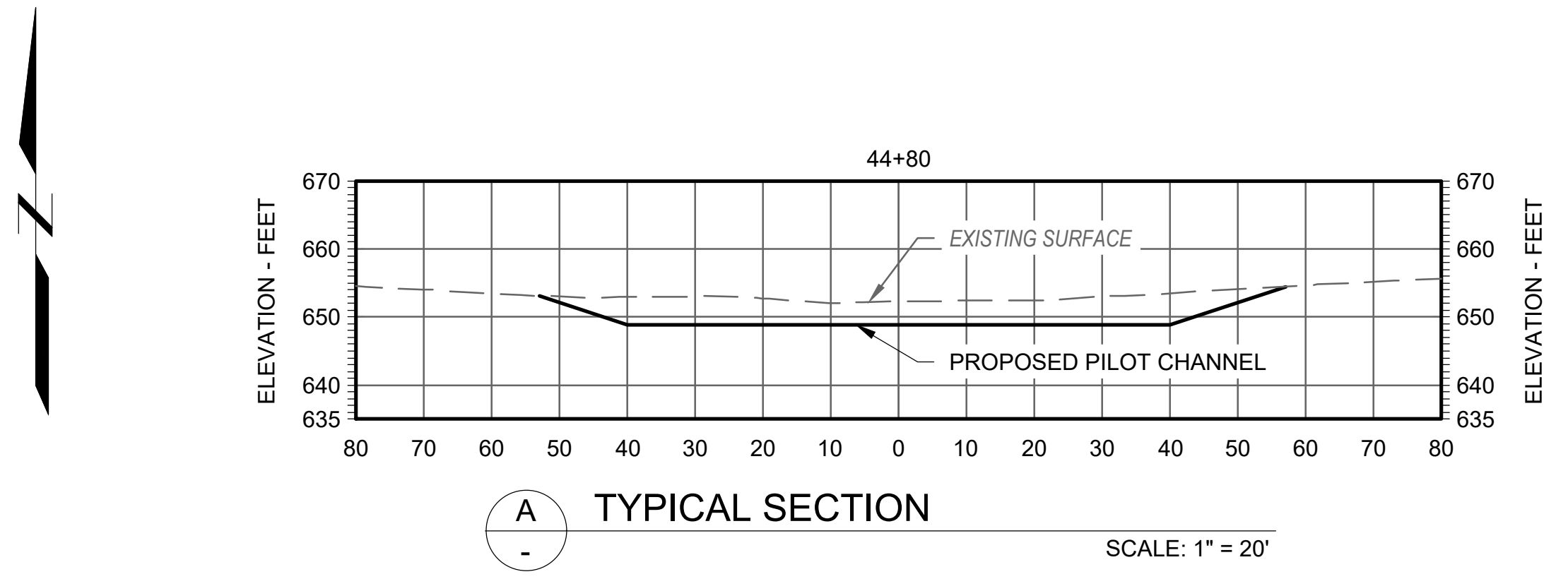


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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

Kalamazoo
river cleanup group
AREAS 2, 3 AND 4 TCRA
Project 2000273

AREA 4 SEDIMENT
CALIBRATION PROFILE
August 2022



<p>Attention:</p> <p>If this scale bar does not measure 1" then drawing is not original scale.</p>		<p>Prepared by GS/Date: 6/8/2022 Checked by EG/Date: 7/8/2022</p>	<p>GEI Consultants GEI CONSULTANTS OF MICHIGAN, P.C. 3065 AKERS MILL ROAD, SUITE 235 ATLANTA, GEORGIA 30339</p>	<p>August 2022</p>	<p>Kalamazoo River Areas 2, 3, and 4 Remediation LLC</p> <p>AREAS 2, 3 AND 4 TCRA</p>	<p>OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan</p>	<p>Fig. 15</p>
					<p>GEI Project 2000273</p>	<p>AREA 4 CURRENT AND PROPOSED PILOT CHANNEL ALIGNMENT</p>	



 PROPOSED, COARSENED RIFFLES

Prepared by EG/Date: 10/4/2024
Checked by LN/Date: 10/4/2024

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Superfund Site
Area 4 TCRA - Allegan County, Michigan
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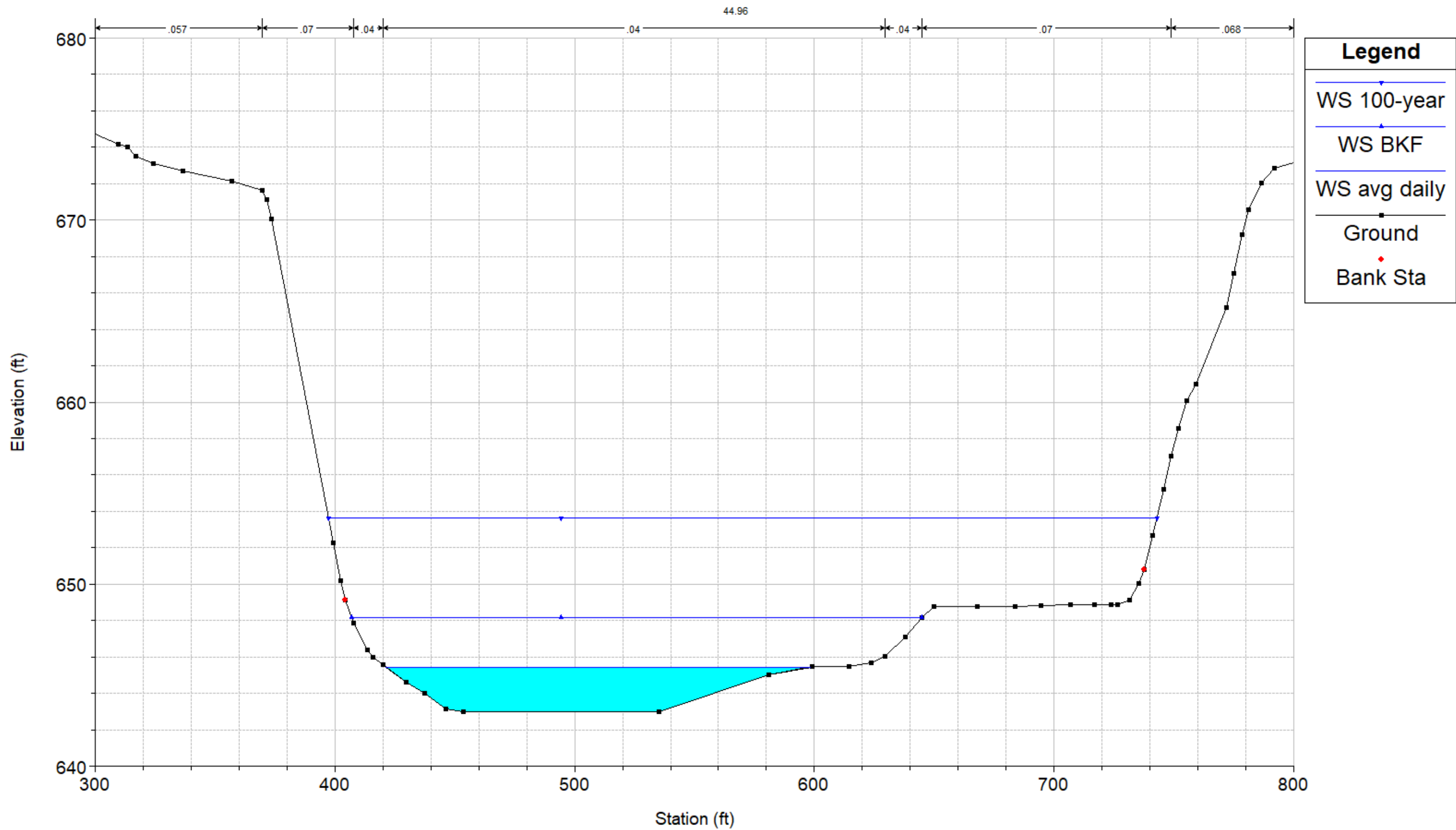


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AREA 4 PLAN VIEW OF
EXPECTED CHANNEL
ALIGNMENT AND PROPOSED,
COARSENED RIFFLES

October 2024

Fig.16



Prepared by EG/Date: 6/23/2022
 Checked by LN/Date: 7/8/2022

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 Superfund Site
 Area 4 TCRA - Allegan County, Michigan

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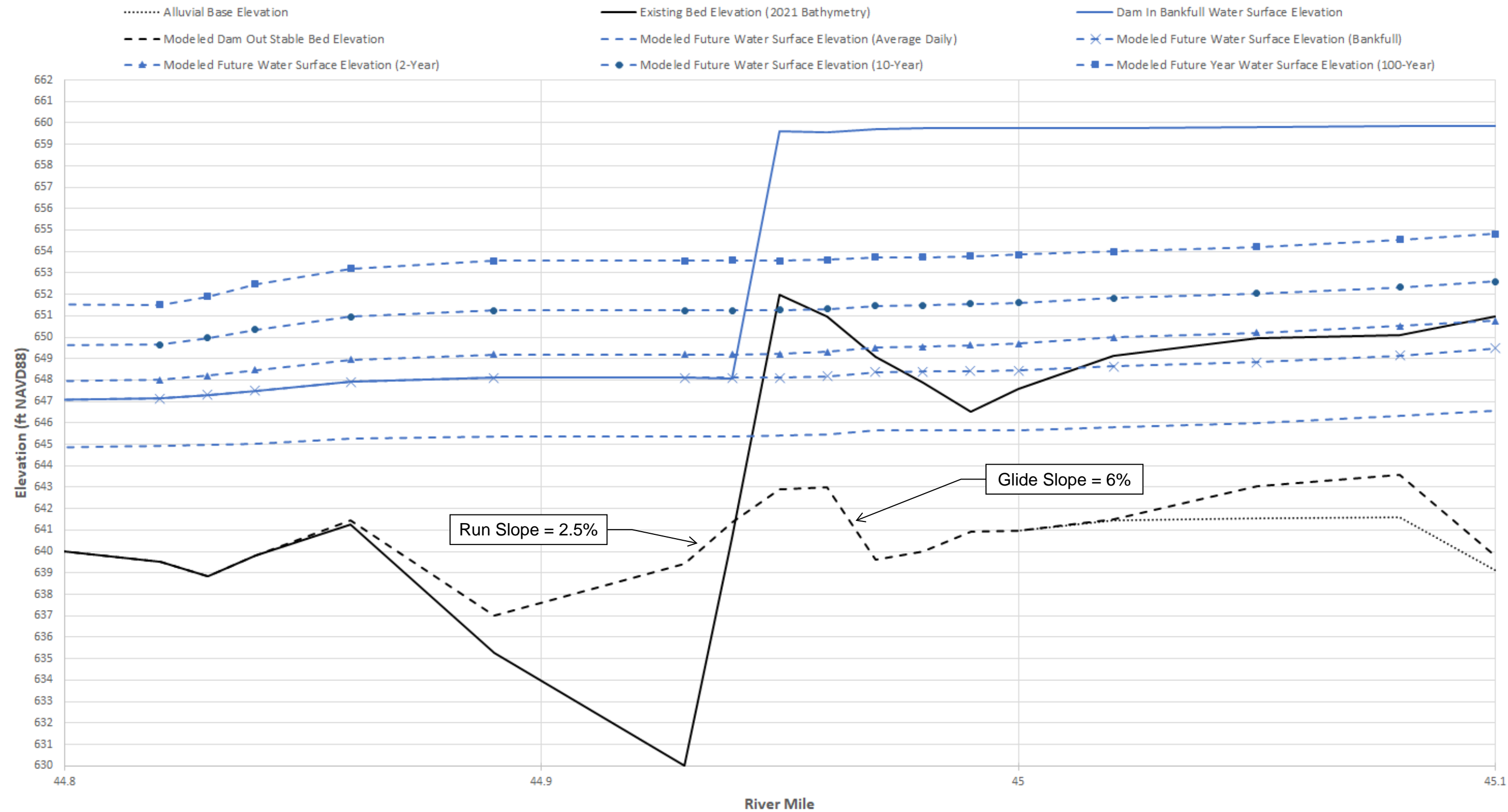
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PROPOSED RIFFLE AT
 TROWBRIDGE DAM
 CROSS-SECTION AT CREST

August 2022


Fig. 17

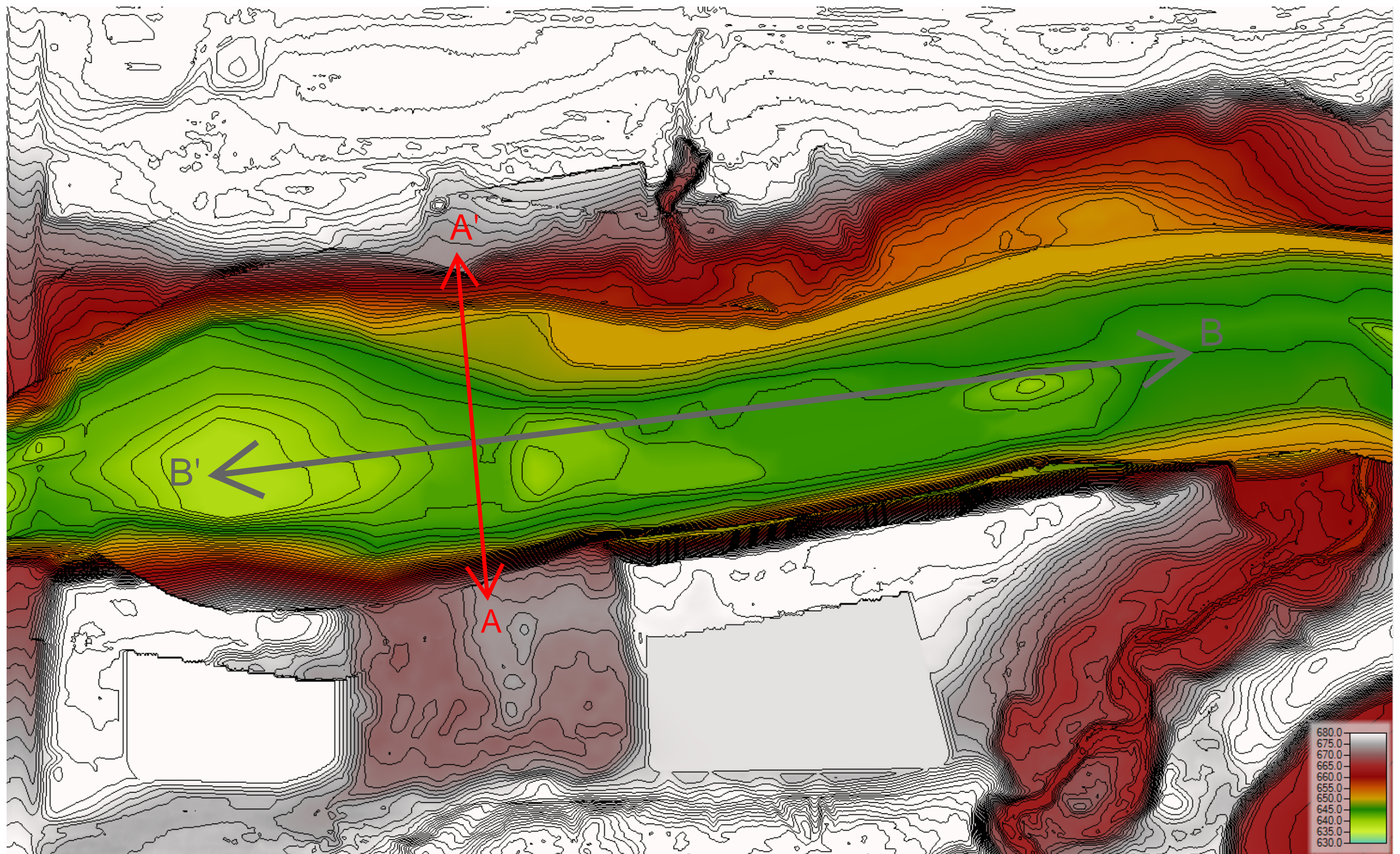
Current and Model Simulated Future Bed and Water Surface Elevations



NOTE: PROFILE LOCATION IS SHOWN ON FIGURE 19 (B-B')

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OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	PROPOSED RIFFLE AT TROWBRIDGE DAM PROFILE
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	August 2022



A \longleftrightarrow A' RIFFLE CREST CROSS SECTION LOCATION (FIG. 17)
 B \longleftrightarrow B' RIFFLE PROFILE LOCATION (FIG. 18)

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 Superfund Site
 Area 4 TCRA - Allegan County, Michigan

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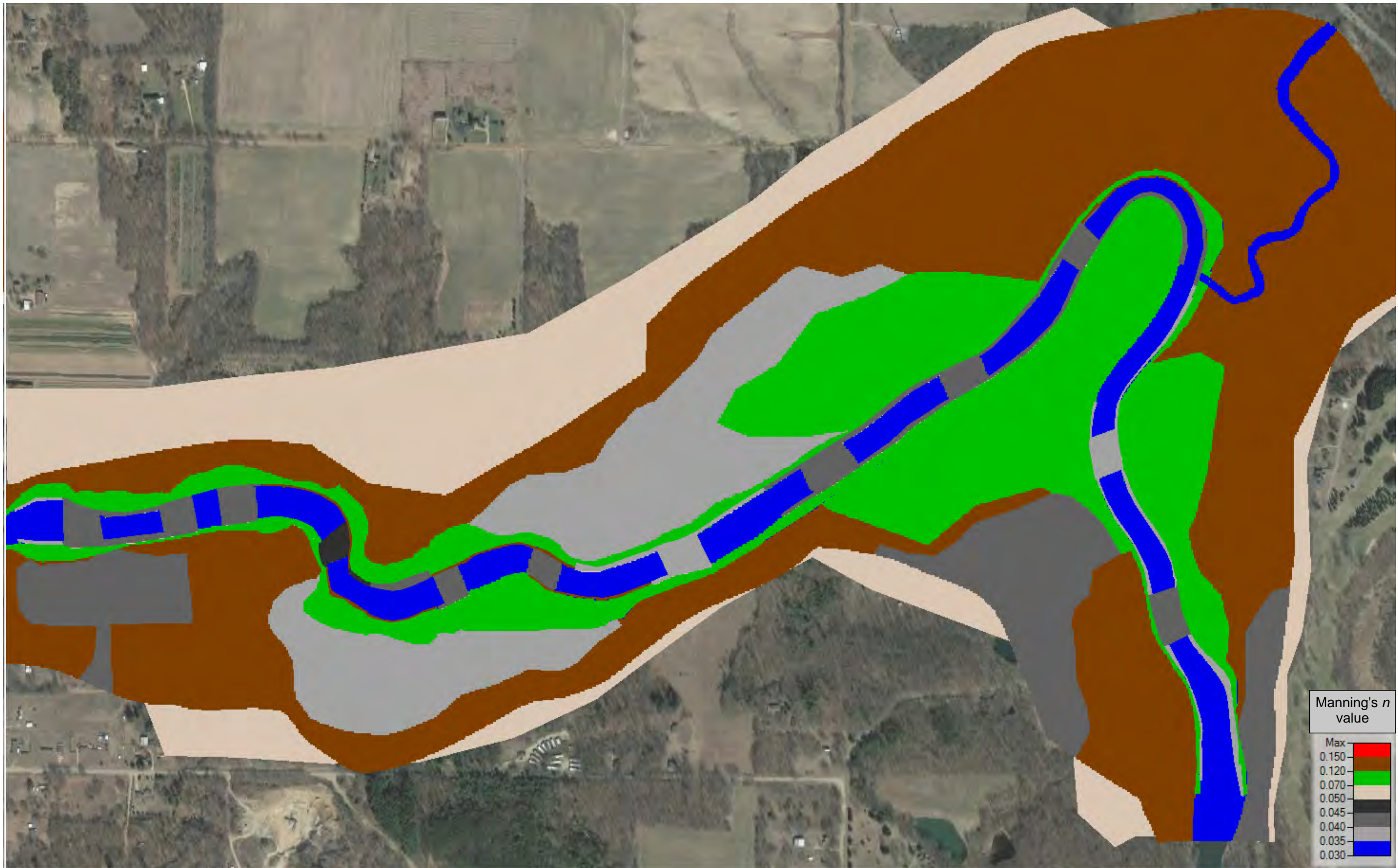


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PROPOSED RIFFLE AT
 TROWBRIDGE DAM
 PLAN VIEW

August 2022

Fig.19



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



AREA 4 2D MODEL
MANNING'S n REGIONS

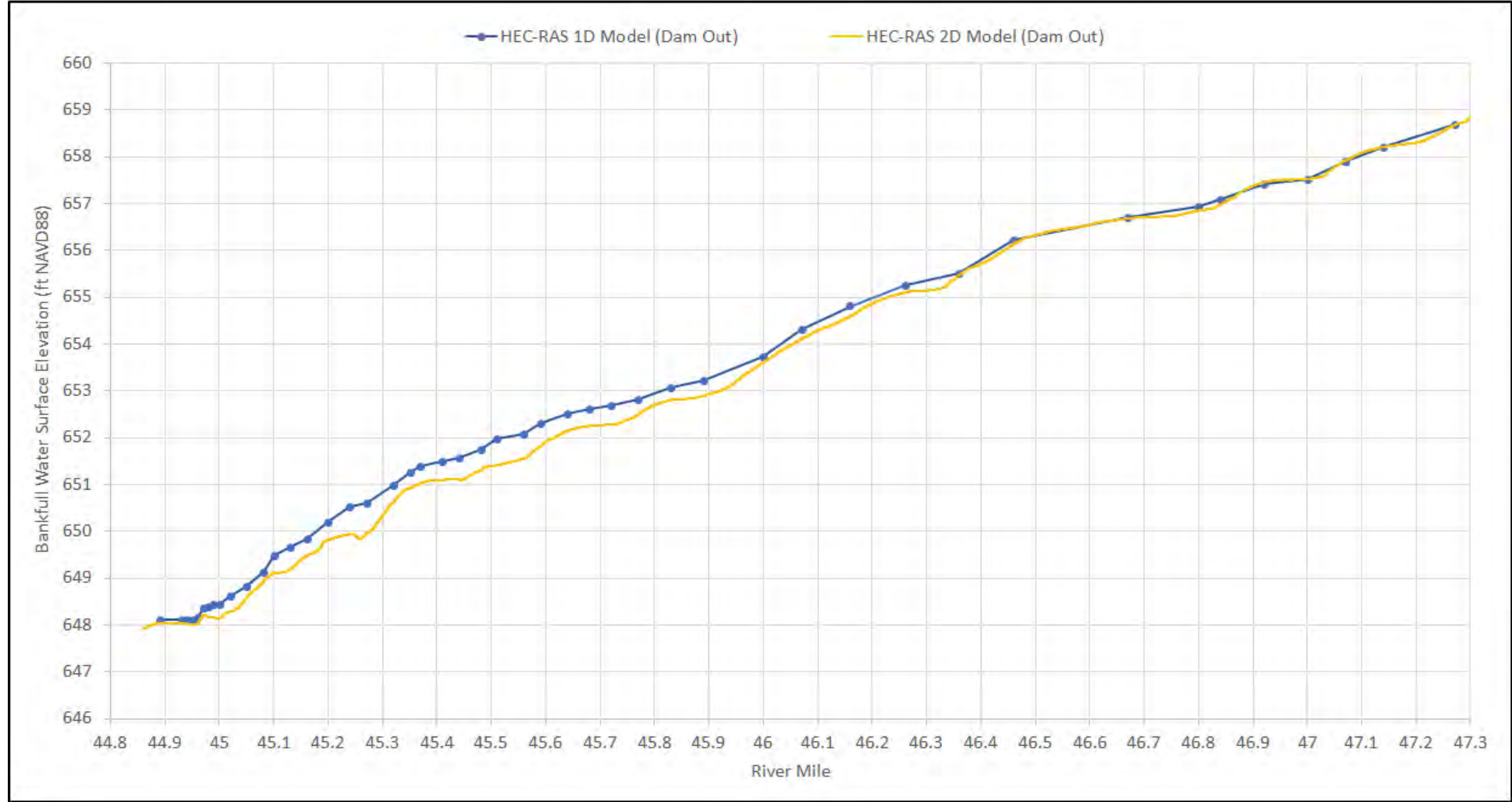
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Fig.20

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NOTES:

1. FLOW= 3,630 CFS

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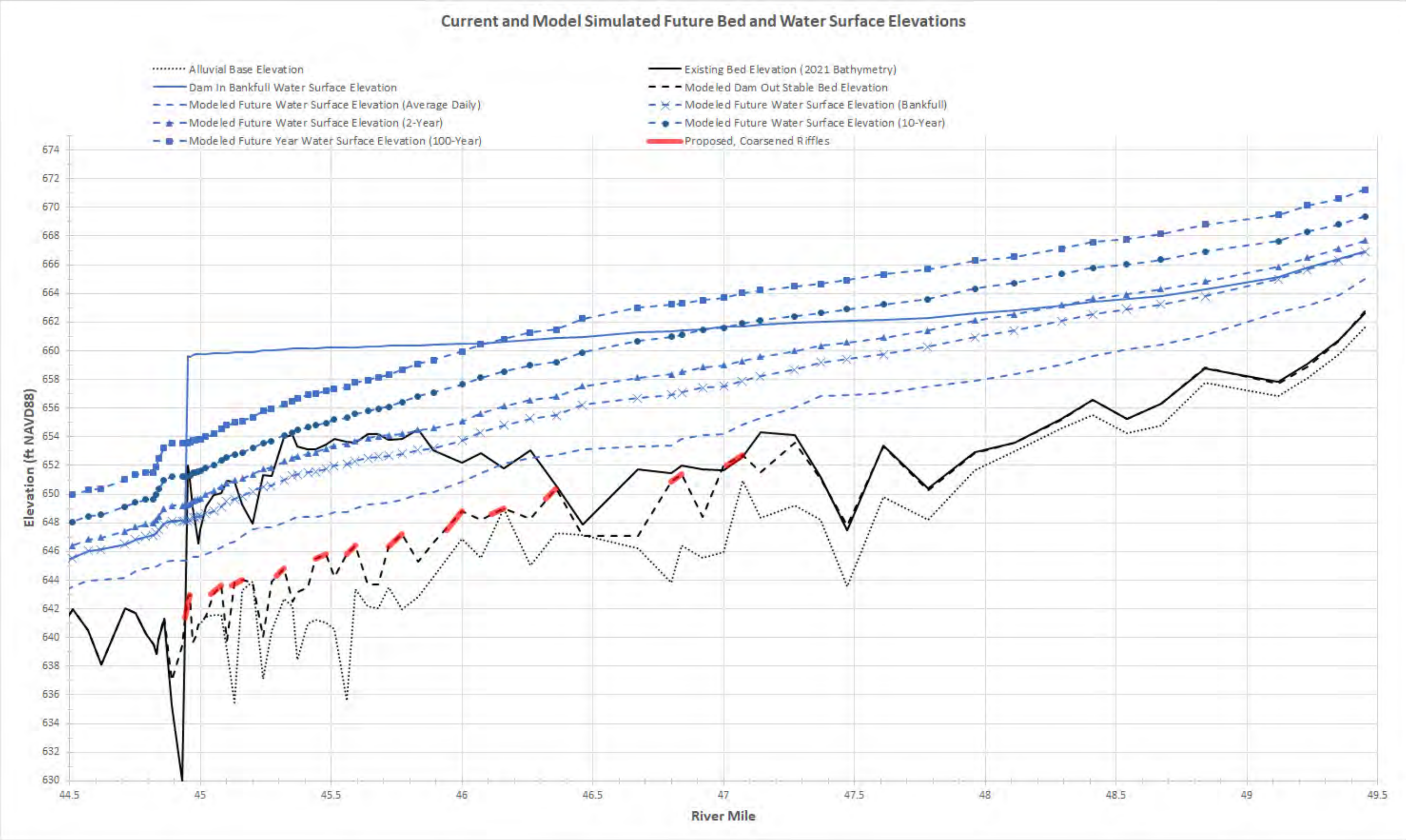
OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	AREA 4 2-D COMPARISON PROFILE PLOT
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 20002732	October 2024

Fig. 21



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



AREA 4 FUTURE
CONDITIONS PROFILE

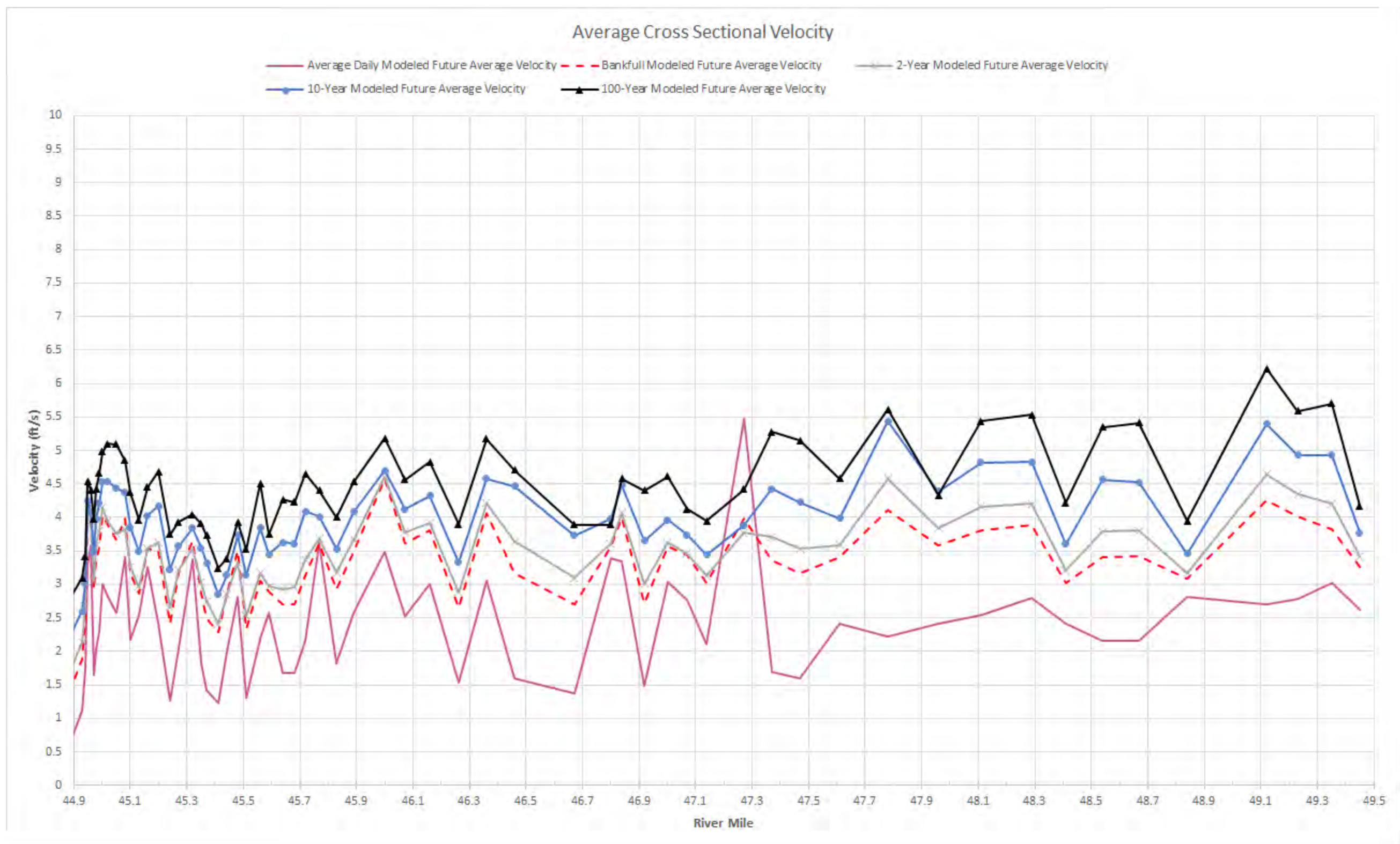
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Fig.22



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



AREA 4 MODELED
VELOCITY PROFILES

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Kalamazoo River Areas 2, 3, and 4 Remediation LLC

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Fig. 23



LEGEND:
Velocity Average Daily Flow (fps)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8
8 +

NOTES:
MAX VELOCITY = 4.2 FPS



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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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Area 4 Plan View of 2-D Velocities
for Average Daily Flow in Modeled
Alignment

Project 2000273

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Fig. 24a



LEGEND:

Velocity Bankfull Flow (fps)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8
8+

NOTES:
MAX VELOCITY = 5.8 FPS



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Area 4 TCRA - Allegan County, Michigan

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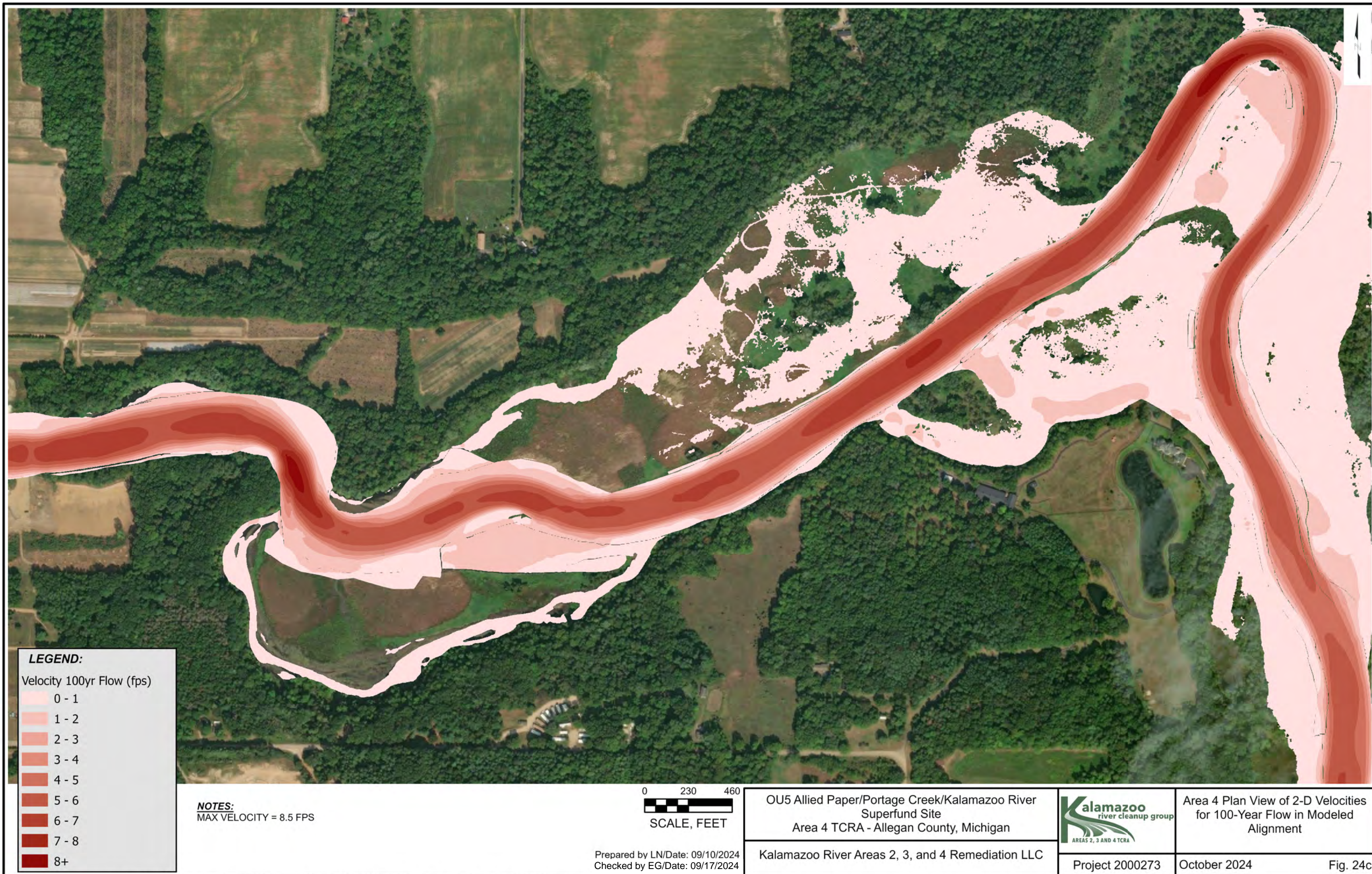


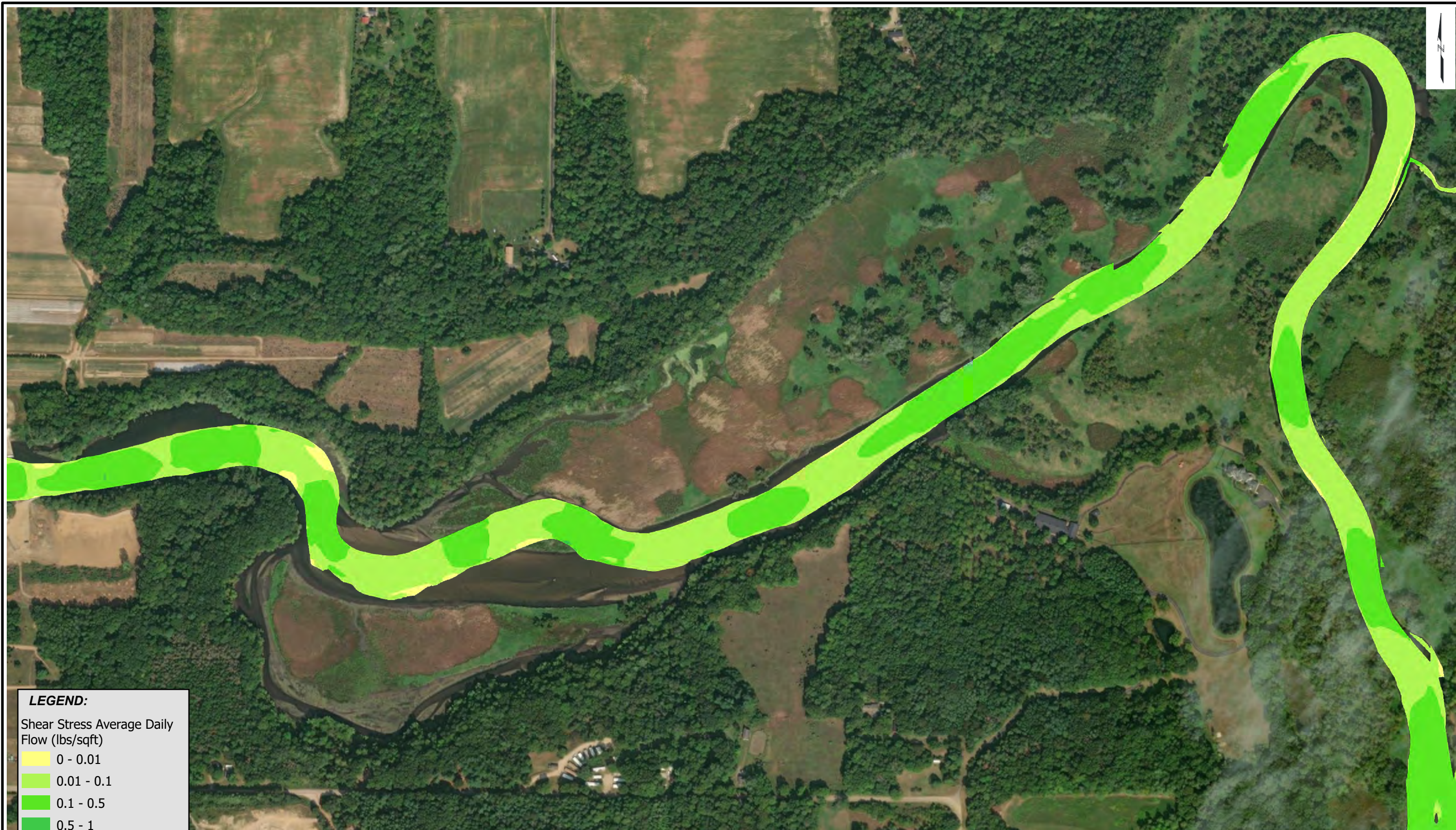
Area 4 Plan View of 2-D Velocities
for Bankfull Flow in Modeled
Alignment

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Fig. 24b

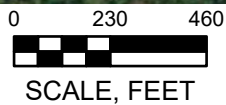




LEGEND:
Shear Stress Average Daily
Flow (lbs/sqft)

0 - 0.01
0.01 - 0.1
0.1 - 0.5
0.5 - 1
1 - 1.5
1.5 - 2
2 - 2.5
2.5+

NOTES:
MAX SHEAR STRESS = 0.7 LBS/SQFT



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Checked by BA/Date: 10/22/2024

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Area 4 Plan View of 2-D Shear
Stress for Average Daily Flow in
Modeled Alignment

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Fig. 25a



LEGEND:

Shear Stress Bankfull Flow
(lb/sqft)



NOTES:
MAX SHEAR STRESS = 1.9 LBS/SQFT



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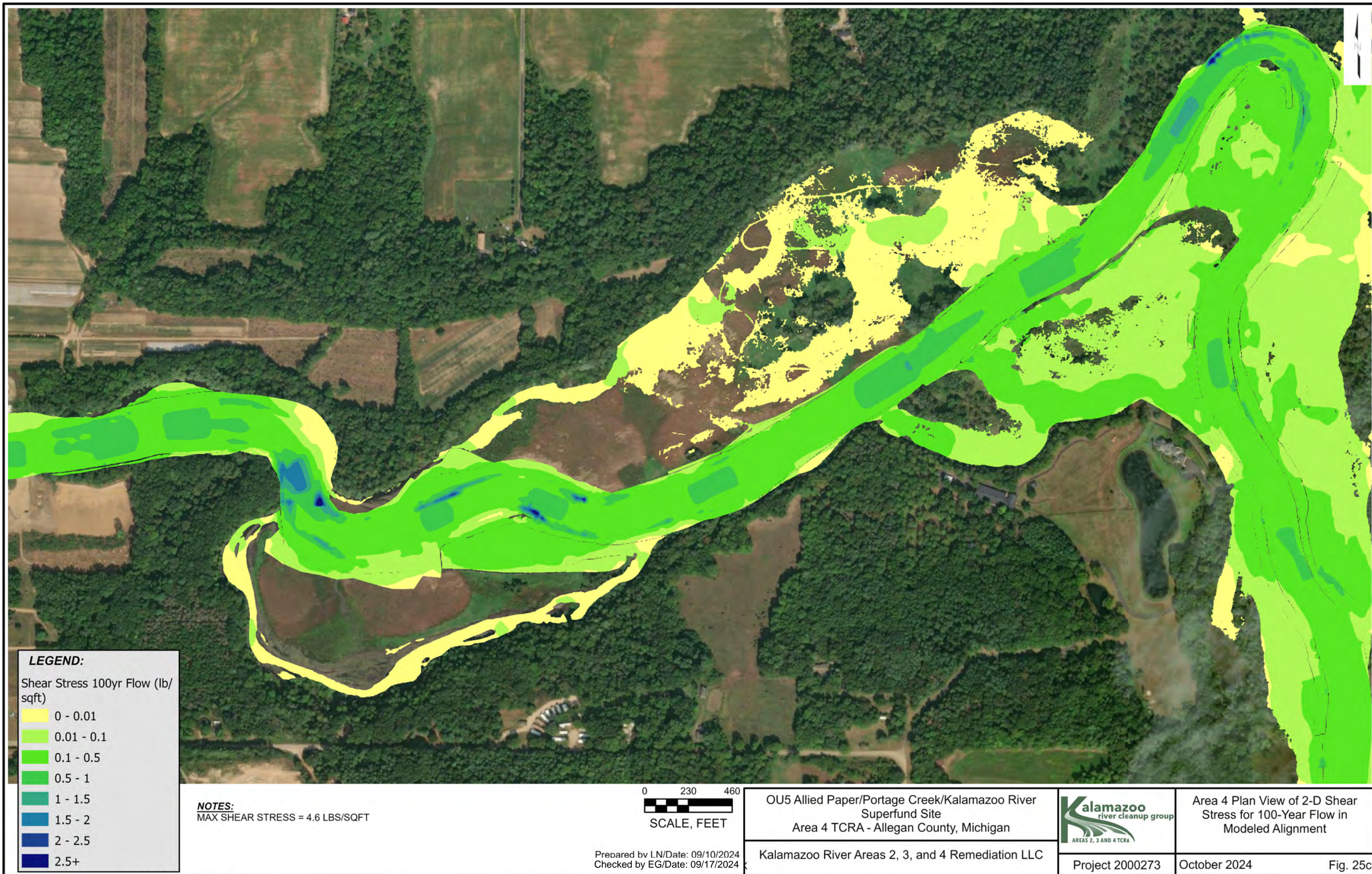


Area 4 Plan View of 2-D Shear
Stress for Bankfull Flow in Modeled
Alignment

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October 2024

Fig. 25b





LEGEND:

Depth Average Daily Flow (ft)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8+



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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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AREA 4 PLAN VIEW OF WATER
DEPTH FOR AVERAGE DAILY
FLOW IN MODELED ALIGNMENT

October 2024

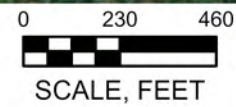
Fig. 26a



LEGEND:

Depth Bankfull Flow (ft)

0 - 1
1 - 2
2 - 3
3 - 4
4 - 5
5 - 6
6 - 7
7 - 8+



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Superfund Site
Area 4 TCRA - Allegan County, Michigan

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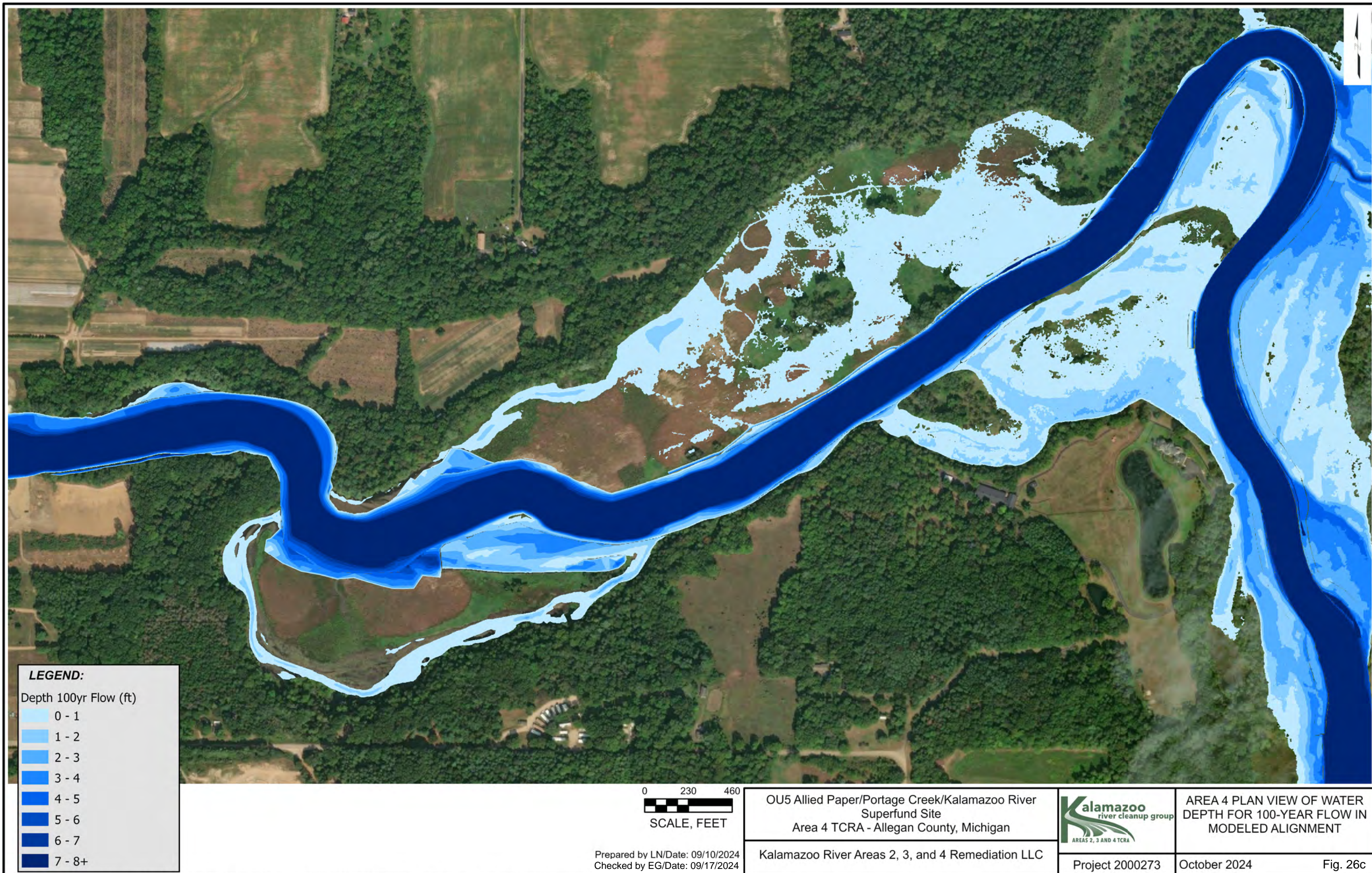


AREA 4 PLAN VIEW OF WATER
DEPTH FOR BANKFULL FLOW IN
MODELED ALIGNMENT

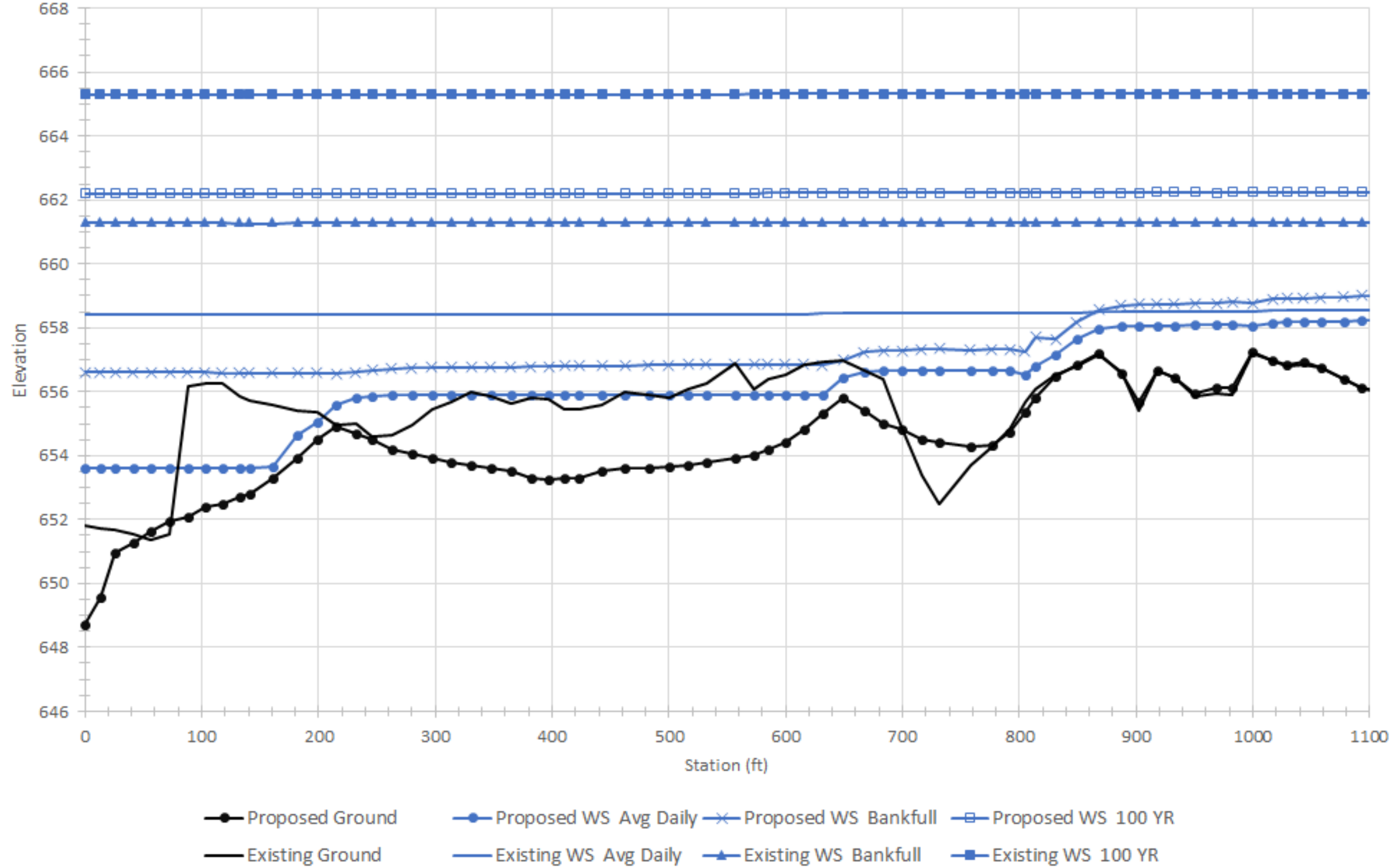
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Fig. 26b



Schnable Brook Existing and Proposed Conditions



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Superfund Site
Area 4 TCRA - Allegan County, Michigan

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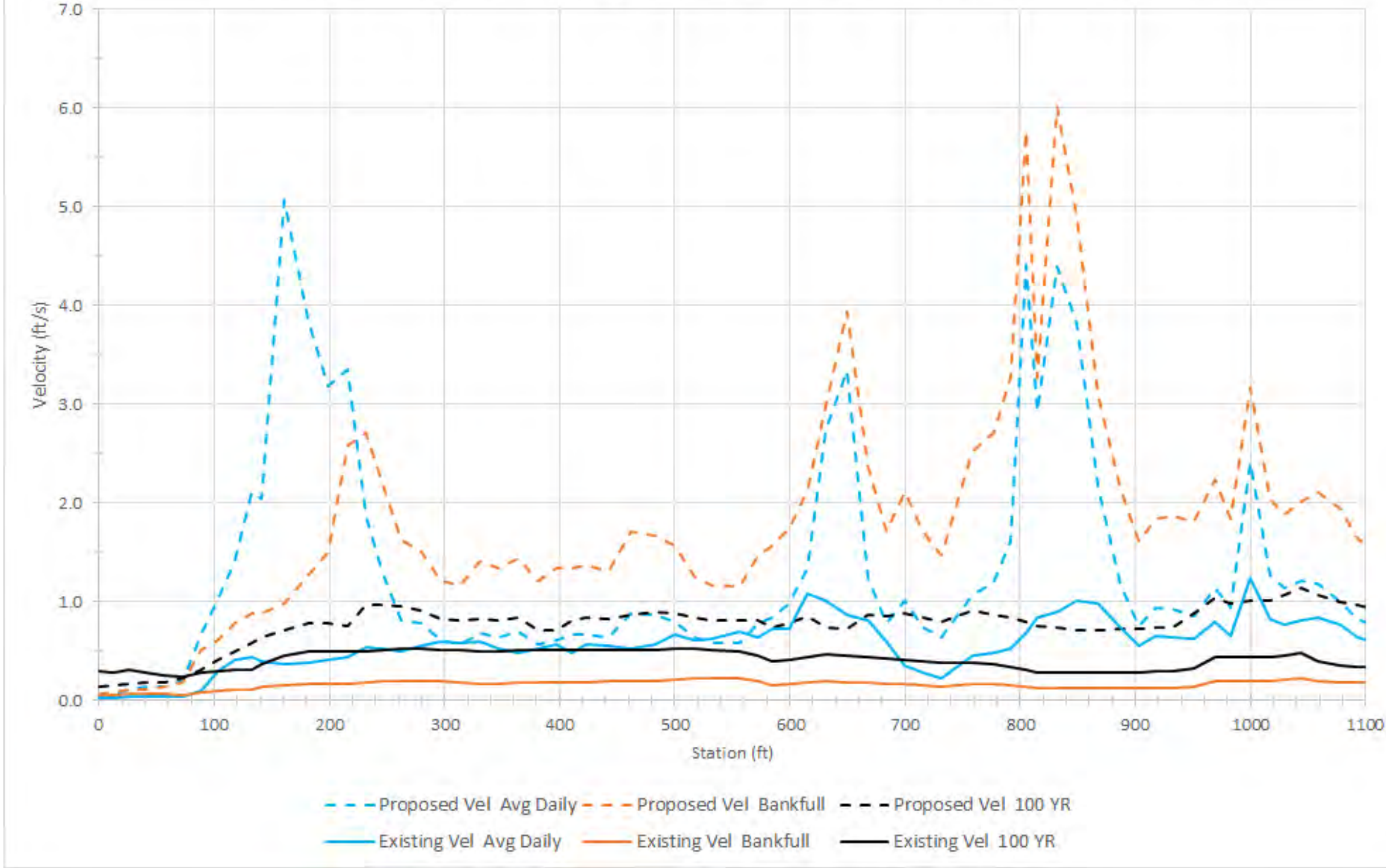
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SCHNABLE BROOK
EXISTING AND PROPOSED
CONDITIONS

October 2024

Fig.27a

Schnable Brook Existing and Proposed Conditions Velocity



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OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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SCHNABLE BROOK
EXISTING AND PROPOSED
CONDITIONS VELOCITY

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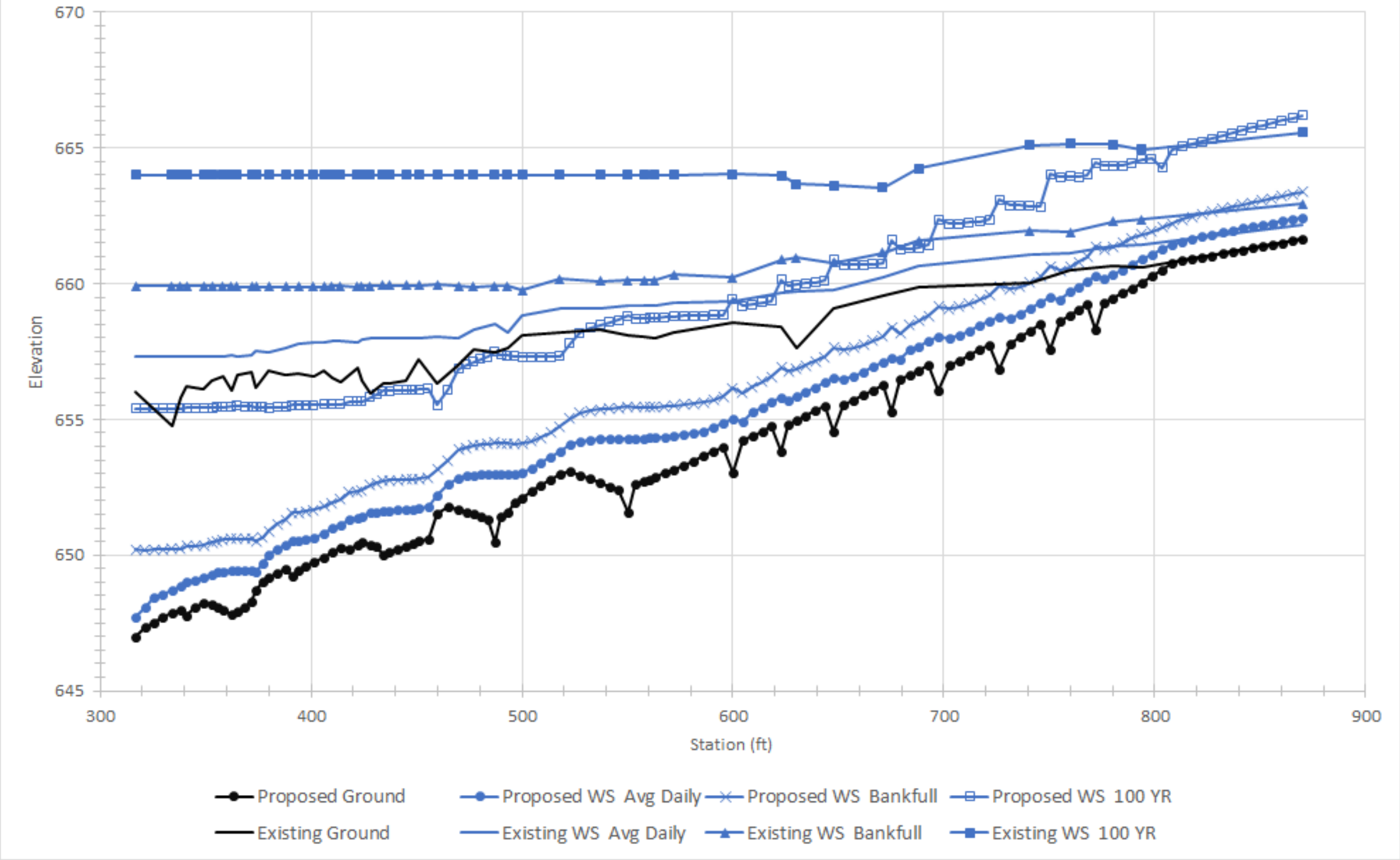
October 2024

Fig.27b



LEGEND: — Schnable Brook ▤ Grade Control Structure	NOTES: 1) Imagery from Seaworks Group, LLC 2) Stationing corresponds to hydraulic model stationing 3) Grade control structures consist of stone with D50 of 3 inches	 SCALE, FEET	OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan	 AREAS 2, 3 AND 4 TCRA	SCHNABLE BROOK GRADE CONTROL STRUCTURES
			Kalamazoo River Areas 2, 3, and 4 Remediation LLC		
Prepared by LN/Date: 07/19/2022 Checked by EG/Date: 07/20/2022			August 2022		

Osgood Drain Existing and Proposed Conditions



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



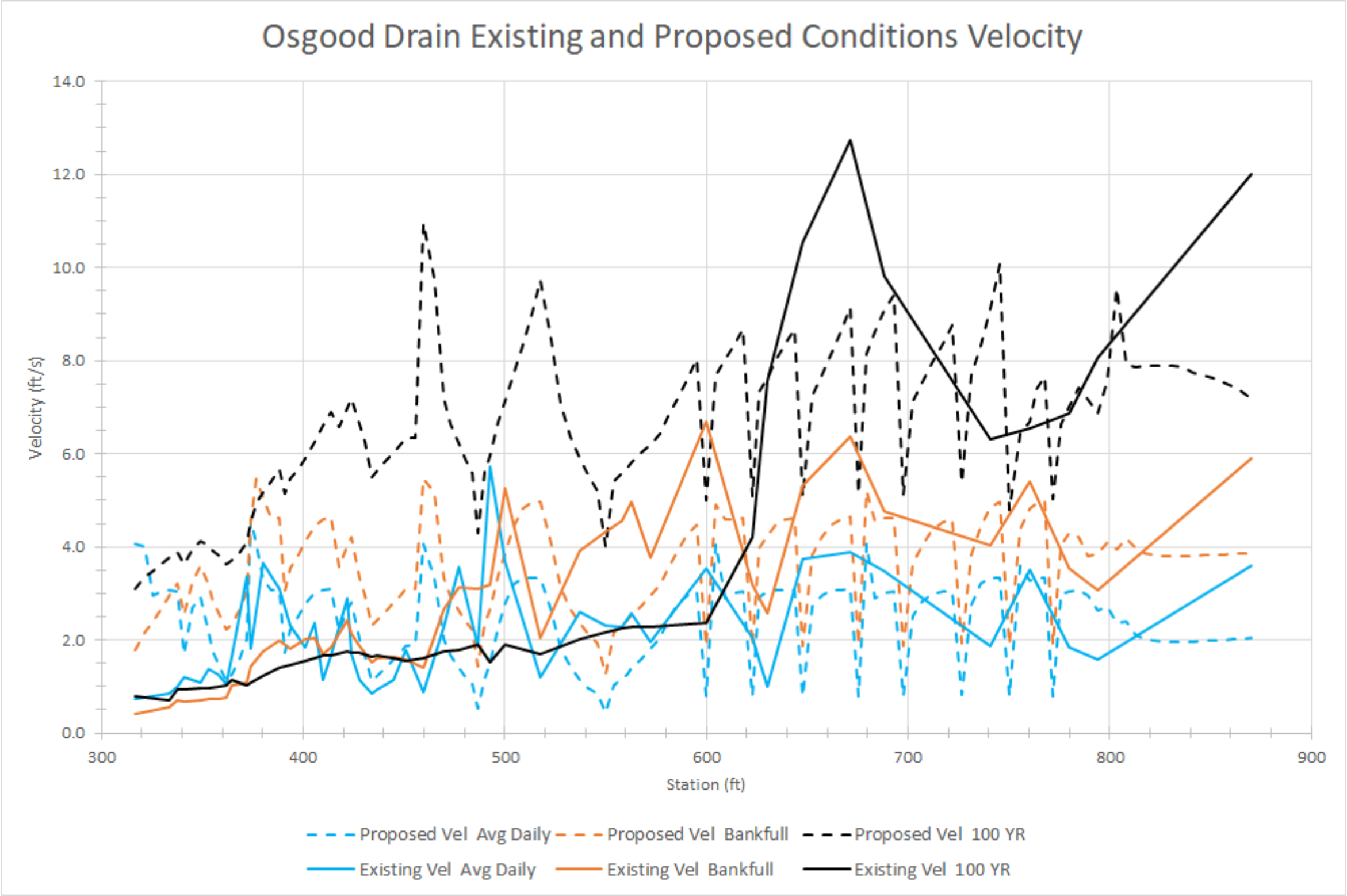
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OSGOOD DRAIN EXISTING
AND PROPOSED
CONDITIONS

August 2022

Fig.28a

Prepared by LN/Date: 6/23/2022
Checked by EG/Date: 7/20/2022



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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AREAS 2, 3 AND 4 TCRA

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OSGOOD DRAIN EXISTING
AND PROPOSED
CONDITIONS VELOCITY
PROFILE


August 2022


Fig.28b

Prepared by LN/Date: 6/23/2022
Checked by EG/Date: 7/20/2022



LEGEND:

 Riffle Grade Control Structures

 Step Grade Control Structures


NOTES:

1) Imagery from Seaworks Group, LLC

2) Step grade control structures consist of stone with D50 of 4-6 inches and small boulders

3) Riffle grade control structures consist of stone with D50 of 4-6 inches

4) Furthest downstream riffle grade control structure will be installed as needed

0 25 50

SCALE, FEET

Prepared by LN/Date: 07/19/2022
Checked by EG/Date: 07/20/2022

OU5 Allied Paper/Portage Creek/Kalamazoo River Superfund Site Area 4 TCRA - Allegan County, Michigan		OSGOOD DRAIN GRADE CONTROL STRUCTURES
Kalamazoo River Areas 2, 3, and 4 Remediation LLC	Project 2000273	August 2022 Fig. 28c

Attachment B

BSTEM Calibration and Application Technical Memorandum

Memorandum

To: Brian Kelly, EPA
From: Roger Hathaway, PE, GEI
Amber Ahles, PMP, GEI
CC: John D. Jolly, NCR
Paul Jansen, GEI
Scott Dierks, GEI
Date: 10/22/2024
Re: BSTEM Calibration and Application Technical Memorandum

Introduction

Bank stability evaluations were performed to support stabilization design in Area 4 using the United States Department of Agriculture Agricultural Research Service Bank Stability and Toe Erosion Model (BSTEM) routine in Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 5.0.7 (USACE, 2019). BSTEM calculates bank failures and toe scour and can be combined with sediment transport routines in HEC-RAS (USACE, 2015).

Bank failures are calculated based on bank geometry and parameters for weight of soil, cohesion, hydrostatic confining forces, and pore water pressure. The bank failure routine iterates through multiple potential failure planes to determine the most likely failure plane, and then calculates a factor of safety.

The factor of safety is the ratio of the resisting forces (cohesion, frictional resistance, and hydrostatic confining force) to the driving forces (hydrostatic uplift and weight of soil). In the BSTEM routine, a bank with a factor of safety >1 will not fail (USACE, 2015). For application of model results to design, a factor of safety >1.3 is generally considered stable.

Toe scour is calculated based on the difference between applied shear stress and the toe material's critical shear stress. Applied shear stress is calculated in the HEC-RAS 1-D hydraulic model. When applied shear stress is greater than the toe material's critical shear stress, the model begins to simulate toe scour. An erodibility rate parameter defines the rate of sediment removal from the toe once scour begins.

Both the bank failure and toe scour BSTEM routines can interact with each other and with the vertical erosion and deposition in the sediment transport routine in HEC-RAS. For example, channel erosion in the sediment transport routine can cause a bank failure due to over-steepening of the bank. Similarly, the failed bank material or the eroded toe material is added to the mass of sediment in the sediment transport routine and can either be deposited or transported downstream.

BSTEM was also applied to the Area 3 one-dimensional (1-D) sediment transport model to evaluate pilot-channel widening to help with the calibration of the sediment transport model for Area 4. For more information on how BSTEM was used in the Area 3 sediment transport

calibration, refer to the Calibration and Application of Hydrologic/Hydraulic and Sediment Transport Models Technical Memorandum (GEI, 2021a).

BSTEM was used to support design in Area 4, so groundwater and soil parameters were incorporated and calibrated to site-specific data. Historical datasets used for modeling included repeat bank surveys in Area 4 conducted by BBL in 2000–2002, United States Geological Survey (USGS) bank stability evaluations upstream in the Kalamazoo River, and a groundwater study in Allegan County (BBL, 2003; Rachol et al., 2005; Hydrosimulatics, 2021). In addition to these historical datasets, GEI collected soil and dynamic cone penetrometer (DCP) data at 16 locations along the riverbanks in Area 4 (GEI, 2021b).

The purpose of these evaluations is to inform design development for the post-dam removal condition in Area 4. This memorandum details the methods used to set up and calibrate the BSTEM model and provides a summary of the results for both dam-in and dam-out conditions.

Modeling Approach

The existing conditions 1-D sediment transport model (GEI, 2021a) for Area 4 was adapted to include the data inputs required for the BSTEM calculations. The model was then calibrated using the repeated bank survey data collected between 2000 and 2002 in Area 4 (BBL, 2003) and USGS gage (04106000 Kalamazoo River at Comstock, Michigan) streamflow data from 2000–2002. Sensitivity analyses were conducted to determine model sensitivity to groundwater, bank stability, and toe-scour parameters. The calibrated model was then applied to the dam-out condition to evaluate bank stability in Area 4 following dam removal. The parameters for BSTEM include:

- Bank geometry.
- Groundwater parameters.
 - Hydraulic conductivity.
 - Reservoir storage.
 - Initial groundwater elevation.
- Bank stability parameters.
 - Friction angle.
 - Cohesion.
 - Saturated unit weight.
 - Matric suction.
- Toe scour parameters.
 - Critical shear stress.
 - Erodibility rate.

Each of these parameters is described in greater detail below.

Calibration to Dam-In Conditions

The BBL erosion pin monitoring study collected repeat bank survey data at five locations in Area 4 with 3 sets of left bank and right bank profiles at each location (BBL, 2003). The study banks were located in subareas A, B, C, and D. The survey was conducted four times over the 2-year period between 2000 and 2002. Based on drainage area-adjusted streamflow data from the USGS gage at Comstock, the monitoring period included a range of flow conditions including four bankfull events (Fig. 1). The results of the BBL bank study are reproduced in Table 1. See Attachment A for the bank survey data.

Dam-Out Simulations

Following model calibration, the calibrated parameters were applied to the dam-out condition. For the dam-out simulations, the 7-year inflow hydrograph from the 1-D sediment transport modeling was used (GEI, 2021a).

Bank Geometry

Dam-In Calibration

GEI's existing conditions 1-D HEC-RAS model bank geometry was modified at five cross sections to match the BBL surveyed bank geometry in 2000 (Table 1). During model calibration, the simulated change in bank geometry was compared to the observed change in bank geometry in the BBL study to evaluate model performance.

BSTEM requires user-defined toe-of-bank and top-of-bank locations at each model cross section to define where the bank failure plane is calculated and where toe scour can occur. The toe station was set at the toe of each bank at the same location as the sediment transport movable bed limits. This allowed the BSTEM routine and sediment transport routine to run simultaneously without double counting erosion or deposition of material. The top-of-bank station was set at the inflection point where the bank slope flattens toward the floodplain.

Dam-Out Simulations

For the dam-out simulations, the post-dredge geometry from the 1-D sediment transport modeling was used (GEI, 2021a), with the same process as above for defining toe of bank and top of bank.

Groundwater Data and Calibration

Dam-In Calibration

Stream and groundwater gage data from USGS and a groundwater study conducted in Allegan County were used to set up and calibrate groundwater in the model (USGS, 2021 and Hydrosimulatics, 2021, respectively). Groundwater within BSTEM is modeled based on water level in the channel (calculated by the hydraulic model), soil hydraulic conductivity (user input), and reservoir storage (user input). The USGS groundwater gage at Morrow (Gage 421614085270801) is adjacent to Morrow Lake and is approximately 3 miles away from the USGS streamflow gage at Comstock (Gage 04106000), which is downstream of Morrow Dam. Comparing data from the two gages showed that groundwater in this area tracks fairly closely with streamflow, indicating a fairly high hydraulic conductivity rate (Fig. 2). The Allegan County Groundwater study confirms this, showing hydraulic conductivity rates in the range of 100–250 feet/day near the Kalamazoo River at Plainwell, Otsego, Trowbridge, and Allegan.

Hydraulic conductivity was set to 135 feet/day in the model based on the Allegan County Groundwater study near Area 4. The storage parameter was varied over multiple runs until the modeled groundwater matched the groundwater response observed at the Morrow gage during the same time period. See the final groundwater calibration in Fig. 2. Note that the calibrated model groundwater level rises and falls at similar rates as the observed groundwater level.

Dam-Out Simulations

For the dam-out simulations, hydraulic conductivity and storage were kept the same as in the calibrated dam-in model. The starting groundwater elevation was varied between 650 feet and 660 feet to evaluate sensitivity to initial groundwater conditions as the reservoir is lowered during construction. Starting groundwater level of 660 feet represents a more rapid drawdown scenario, where the groundwater elevation is higher than the water surface elevation in the river. Starting groundwater level of 650 feet represents a more gradual drawdown scenario, where the groundwater elevation is about the same as the water surface elevation in the river.

Soil Parameters

Dam-In Calibration

Soil parameter values were defined using GEI-collected DCP data from 16 locations along the banks in subareas C, D, and E (GEI, 2021b), and geotechnical data collected by USGS upstream in the Kalamazoo River from approximately river mile 51 to 57 (Rachol et al., 2005) (Table 2 and Attachment B). The upstream banks were assumed to be similar enough to the Area 4 banks that the same parameter ranges could be applied. The model was run using the method of slices calculation routine (USACE, 2016).

DCP testing consisted of dropping a 10-pound weighted hammer a specified distance (22 inches) to drive a conical steel-tip rod into the soil (ASTM D7380). The number of blows required for every 6 inches of penetration was recorded. In conjunction with the DCP testing, a hand auger was used to obtain representative samples of material for visual classification (Unified Soil Classification System) and potential gradation testing. Select intervals were submitted to the GEI Marquette geotechnical laboratory for grain-size analysis (GEI, 2021b).

DCP results are presented in Table 3. Higher blow counts correlate with higher in-situ soil strength. In subareas C and D soils were loose, fine grained, and had low strength. In lower subarea E, soils were a mix of loose sediment and coarser, denser material, with higher strength than in C and D. DCP data was not collected in the subarea F and G banks because soils were too soft.

DCP data were used to set the friction angle (ϕ), which is the BSTEM input parameter that measures the friction shear resistance of soil. Higher friction angle correlates with higher soil strength and resistance to bank failure. DCP blow counts were converted to standard penetration resistance, then to relative density and friction angle (ϕ) (Sowers and Hedges, 1966; Perloff and Baron, 1976). Initial runs showed that results were sensitive to ϕ values and to layering, i.e., the results were different when the bank ϕ values were defined in 2-foot layers by depth rather than when an average ϕ value was used for the entire bank. Layered ϕ values were therefore used for the final calibration and the dam-out simulations. The 2-foot interval ϕ values ranged from 32 to 40 in subareas C and D, and from 32 to 41 in lower subarea E. Average values for subareas C and D were applied to the subarea A and B banks for purposes of calibration.

Soil cohesion is a measure of the attractive forces or bonds between particles in the soil, and can increase soil strength; however, it is not present in granular soils. Cohesion in Area 4 was therefore assumed to be 0. Note that the USGS study found cohesion to be 0 in several locations of upper sediments in its BSTEM work (Rachol, et.al, 2005 and Attachment B).

Saturated unit weight, or weight of soil when completed saturated with water, was set at the median value measured by USGS upstream in the Kalamazoo River. Matric suction is a measure of the negative pore water pressure as the soil drains, and because it is difficult to measure directly, it is commonly a calibration parameter. For Area 4, matric suction was set at 25 degrees based on the typical range of 10–30 degrees for most materials (USACE, 2015). The Area 4 BSTEM model was not very sensitive to saturated unit weight or matric suction, so no further adjustments were made to either parameter.

BSTEM calculates toe scour based on 1) critical shear stress, which is the shear at which the toe begins to scour, and 2) erodibility, which is the rate at which sediment is removed once toe scour begins. Toe scour was not observed in the BBL bank study cross sections during the study period 2000–2002. This suggests the bank soils at those cross sections were able to withstand the observed shear values during that time period, including at flows up to and higher than bankfull. Average modeled shear values at those cross sections between 2000–2002 varied between 0.02 and 0.07 pounds per square foot (lbs/ft²) with maximum shear values between 0.03 and 0.21 lbs/ft². Therefore, the modeled critical shear value was varied between 0.1 and 0.2 lbs/ft², which did not result in toe scour at any of the calibration cross sections. The rate of erodibility was set at the median value measured by USGS in Area 3. Without any observed toe erosion in the BBL survey calibration data, the erodibility rate was not adjusted further.

Dam-Out Simulations

For the dam-out simulations, the nearest DCP data were used to define phi values (friction angle) in 2-foot intervals. For locations not near a DCP sample location, the average DCP data for that subarea was used. For subareas F and G, phi was set at 20 degrees to represent very low soil strength. No changes were made to the calibrated values for cohesion, saturated unit weight, or matric suction.

Critical shear value at the toe was set at 2 lbs/ft² for the dam-out condition, which represents some measure of toe protection, such as stone. Some risk of toe erosion as the reservoir is lowered and before toe protection can be put in place is likely. However, the BSTEM routine can in some cases over-predict the amount of toe erosion (USACE, 2015). For this evaluation, the critical shear value was set assuming some level of protection at the toe but no additional protection on the bank, and applied shear stresses were also evaluated. Future evaluations will include additional analysis of toe scour and bank protection during and following the drawdown period with and without designed toe protection.

BSTEM Calibration and Sensitivity Results

Dam-In Calibration

The calibrated model results matched the observed change in banks in the BBL study (Figs. 3a – 3j). The BSTEM model simulated bank failures at the same locations BBL data indicated “sloughing” near river miles 48.41 (right descending bank [RDB]), 47.37 (RDB), 47.07 (left descending bank [LDB]), and 46.8 (LDB) (Table 1, Figs. 3a – 3j). There were also modeled bank failures (smaller, in most cases) at river miles 47.37 (LDB), 46.8 (RDB), and 46 (RDB). The bank failure results were most sensitive to friction angle and bank angle. In general, the modeled bank failures occurred at banks with steeper slopes than 2 horizontal:1 vertical.

Dam-Out Simulations

For the dam-out simulations, the model was sensitive to starting groundwater elevation. In the simulations where groundwater started higher, there were more bank failures than when the groundwater started lower (Table 4). Similar to the dam-in calibration simulations, the dam-out simulations showed some sensitivity to friction angle and bank angle. In general, the factor of safety was lower and the number of bank failures was higher in subareas F and G where ϕ was lowest. At river mile 45, despite high ϕ values, the left bank has a steep slope of 1.2:1 and the resulting factor of safety indicates the bank is only marginally stable. In some cases, starting bank angle was approximately 3:1, and bank failures were induced by bank steepening as the channel bed lowered during the sediment transport simulation.

The dam-out simulations showed no toe erosion with critical shear set at 2 lbs/ft². Modeled average channel shear values were between 0.09 and 1.53 lbs/ft² during the simulation period, with maximum shear values between 0.19 to 2.6 lbs/ft². Shear stress of 0.09 lbs/ft² can mobilize fine gravels, whereas shear stress of 2.6 lbs/ft² can mobilize coarse cobbles (Fischenich, 2001). This suggests some risk of toe erosion at locations where the toe is not protected. Future evaluations will include analyzing two-dimensional shear along the banks and potential toe scour during the drawdown period, along with the impacts of targeted bank and toe protection measures.

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Tables

1. Summary of Bank Calibration Data
2. BSTEM Parameters
3. Dynamic Cone Penetrometer Testing Results
4. Representative BSTEM Factors of Safety

Figures

1. Streamflow During Calibration Period
2. Groundwater Calibration
3. Bank Failure Calibration

Attachments

Attachment A – Area 4 Bank Survey Data (BBL, 2003)

Attachment B – Kalamazoo River Geotechnical Data (Rachol et al., 2005)

Table 1. Summary of Bank Calibration Data

BBL Location¹	Left Bank BBL Results¹	Right Bank BBL Results¹	Approximate River Mile	Nearest GEI Model Cross Section²	Nearest DCP Sample Location
EP-99	Minimal change	Sloughing and intermittent erosion/deposition	48.45	48.41	NA
EP-101	Erosion and deposition	Bank sloughing and erosion	47.4	47.37	NA
EP-102	Erosion and sloughing	Minimal change and erosion	47.1	47.07	47.07 (DCP 1&2)
EP-103	Deposition and sloughing	Minimal change and deposition	46.75	46.8	46.8 (DCP 3&4)
EP-106	Erosion	Minimal change	46	46	45.89 (DCP 9&10)

Notes:

1. From BBL Erosion Pin Monitoring Study (BBL, 2003).
2. GEI 1-D HEC-RAS model cross section location

DCP= dynamic cone penetrometer.

Created by: EG 9/17/2021

Checked by: LAP 9/24/2021

Table 2. BSTEM Parameters

Parameter	Category	USGS Minimum ¹	USGS Median ¹	USGS Maximum ¹	Area 4 Calibrated Model Value	Area 4 Calibrated Model Source
Saturated Unit Weight (lb/ft ³)	Bank Stability	47	94	115	94	USGS median value (Rachol et al., 2005)
Friction angle (degrees)		10.9	23.5	40.6	Varies 20-41 ²	GEI collected DCP data
Cohesion (lb/ft ²)		0	44	285	0	Conservative assumption
Matrix Suction Phi b (degrees)		--	--	--	25	HEC RAS manual (USACE, 2015), calibrated value
Critical Shear Stress (lb/ft ²)	Toe Scour	0.004	0.05	1.5	0.2	Set high to prevent over scouring
Erodibility (ft ³ /lb-s)		0.00045	0.0014	0.037	0.0014	USGS median value (Rachol et al., 2005)
Saturated Hydraulic Conductivity (ft/day)	Groundwater				135	Allegan County Study (Hydrosimulatics, 2021)
Reservoir Storage Width (ft)					500	Calibrated value

Notes:

1. Rachol et al., 2005. See Attachment B for all data

2. See results table for phi values at each bank

BSTEM = Bank Stability and Toe Erosion Model; lb/ft³ = pounds per cubic foot; lb/ft² = pounds per square foot; ft³/lb-s = cubic feet per pound-second; ft/day = feet per day; ft = feet; USGS = US Geological Survey; DCP = dynamic cone penetrometer; HEC RAS = Hydrologic Engineering Center's River Analysis System.

Created by: EG 9/17/2021

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Table 3. Dynamic Cone Penetrometer Testing Results
Kalamazoo River OU5 Areas 2, 3, and 4 Remediation LLC
Area 4 TCRA



Location ID	Test Date	Location	Surface Elevation (feet)	Depth (inches)	DCP Blow Counts	Soil Texture from Geotechnical Analysis (USCS) ¹
4G-DCP-1	5/11/2021	LDB - Subarea C at approximate RM 47.01	660.85	0-6	9	
				6-12	4	
				12-18	5	
				18-24	6	
				24-30	8	
				30-36	14	
				36-42	13	
				42-48	11	
				48-54	11	
				54-60	12	
				60-66	21	SM
				66-72	24	
				72-78	22	
				78-84	21	
4G-DCP-2	5/11/2021	RDB - Subarea C at approximate RM 47.01	660.19	0-6	3	
				6-12	5	
				12-18	8	
				18-24	7	
				24-30	4	
				30-36	3	
				36-42	3	
				42-48	7	
				48-54	9	SM
				54-60	17	
				60-66	43	
4G-DCP-3	5/11/2021	LDB - Subarea C at approximate RM 46.74	660.07	0-6	4	
				6-12	4	
				12-18	8	
				18-24	5	
				24-30	4	
				30-36	5	
				36-42	4	
				42-48	6	
				48-54	9	ML
				54-60	15	
				60-66	39	
4G-DCP-4	5/11/2021	RDB - Subarea C at approximate RM 46.74	659.18	0-6	1	
				6-12	1	
				12-18	2	
				18-24	7	
				24-30	10	
				30-36	9	
				36-42	9	
				42-48	8	
				48-54	7	
				54-60	9	
				60-66	11	
				66-72	14	
				72-78	12	
				78-84	17	
				84-90	16	
				90-96	17	

Table 3. Dynamic Cone Penetrometer Testing Results
Kalamazoo River OU5 Areas 2, 3, and 4 Remediation LLC
Area 4 TCRA



Location ID	Test Date	Location	Surface Elevation (feet)	Depth (inches)	DCP Blow Counts	Soil Texture from Geotechnical Analysis (USCS) ¹
4G-DCP-5	5/11/2021	RDB (outer bend) - Subarea D at approximate RM 46.44	660.31	0-6	9	SP
				6-12	16	
				12-18	16	
				18-24	16	
				24-30	15	
				30-36	15	
				36-42	17	
				42-48	15	
				48-54	19	
				54-60	23	
				60-66	19	
				66-72	22	
4G-DCP-6	5/11/2021	LDB (inner bend) - Subarea D at approximate RM 46.44	659.71	0-6	2	SP-SM
				6-12	1	
				12-18	WOH	
				18-24	WOH	
				24-30	WOH	
				30-36	WOH	
				36-42	2	
				42-48	5	
				48-54	7	
				54-60	8	
				60-66	7	
				66-72	7	
				72-78	15	
				78-84	7	
				84-90	8	
				90-96	8	
				96-102	11	
				102-108	14	
4G-DCP-7	5/12/2012	RDB - Subarea D at approximate RM 46.21	659.15	0-6	1	
				6-12	1	
				12-18	1	
				18-24	1	
				24-30	4	
				30-36	5	
				36-42	3	
				42-48	3	
				48-54	4	
				54-60	7	
				60-66	5	
				66-72	10	
				72-78	12	
				78-84	12	
				84-90	13	

Table 3. Dynamic Cone Penetrometer Testing Results
Kalamazoo River OU5 Areas 2, 3, and 4 Remediation LLC
Area 4 TCRA



Location ID	Test Date	Location	Surface Elevation (feet)	Depth (inches)	DCP Blow Counts	Soil Texture from Geotechnical Analysis (USCS) ¹
4G-DCP-8	5/12/2021	LDB - Subarea D at approximate RM 46.21	660.76	0-6	6	
				6-12	8	
				12-18	10	
				18-24	11	
				24-30	8	
				30-36	6	
				36-42	7	
				42-48	11	
				48-54	11	
				54-60	10	
				60-66	20	SM
				66-72	18	
4G-DCP-9	5/12/2021	LDB - Subarea D at approximate RM 45.87	660.00	0-6	5	
				6-12	6	
				12-18	6	
				18-24	7	
				24-30	9	
				30-36	8	
				36-42	5	
				42-48	7	
				48-54	18	
				54-60	10	
				60-66	6	SP
				66-72	6	
				72-78	8	
				78-84	11	
				84-90	13	
4G-DCP-10	5/12/2021	RDB - Subarea D at approximate RM 45.87	659.37	0-6	3	
				6-12	3	
				12-18	5	
				18-24	4	
				24-30	4	
				30-36	5	
				36-42	4	ML
				42-48	7	
				48-54	9	
				54-60	10	
				60-66	8	
				66-72	7	
				72-78	14	
				78-84	16	
				84-90	21	
				90-96	20	

Table 3. Dynamic Cone Penetrometer Testing Results
Kalamazoo River OU5 Areas 2, 3, and 4 Remediation LLC
Area 4 TCRA

Location ID	Test Date	Location	Surface Elevation (feet)	Depth (inches)	DCP Blow Counts	Soil Texture from Geotechnical Analysis (USCS) ¹
4G-DCP-11	5/12/2021	LDB -Subarea E at approximate RM 45.21 (immediately upstream of the confluence with Osgood Drain)	657.70	0-6	2	
				6-12	1	
				12-18	1	
				18-24	2	
				24-30	4	
				30-36	1	
				36-42	1	
				42-48	2	
				48-54	4	
				54-60	1	
				60-66	2	SM
				66-72	6	
				72-78	3	
				78-84	8	
				84-90	10	
4G-DCP-12	5/12/2021	RDB (outer bend) - Subarea E at approximate RM 45.18	659.38	0-6	11	
				6-12	8	
				12-18	4	
				18-24	1	
				24-30	1	
				30-36	2	
				36-42	4	SM
				42-48	9	
				48-54	9	
				54-60	10	
				60-66	8	
				66-72	5	
				72-78	26	
				78-84	15	
				84-90	10	
4G-DCP-13	5/12/2021	RDB - Subarea E at approximate RM 45.05 (immediately upstream of Trowbridge Dam)	658.15	0-6	1	
				6-12	4	
				12-18	4	SP-SM
				18-24	6	
				24-30	2	
				30-36	14	
				36-42	5	SM
				42-48	8	
				48-54	23	SM
				54-60	56	

Table 3. Dynamic Cone Penetrometer Testing Results
Kalamazoo River OU5 Areas 2, 3, and 4 Remediation LLC
Area 4 TCRA



Location ID	Test Date	Location	Surface Elevation (feet)	Depth (inches)	DCP Blow Counts	Soil Texture from Geotechnical Analysis (USCS) ¹
4G-DCP-14	5/12/2021	Downstream point of Osgood Spit - Subarea E at approximate RM 45.08	656.67	0-6	2	SP-SM
				6-12	4	
				12-18	3	
				18-24	2	
				24-30	3	
				30-36	3	
				36-42	2	
				42-48	5	
				48-54	6	
				54-60	9	
				60-66	7	
				66-72	10	
				72-78	9	
				78-84	14	
				84-90	17	
				90-96	19	
				96-102	22	
4G-DCP-15	5/13/2021	LDB immediately below Trowbridge Dam	645.92	0-6	7	GW
				6-12	5	
				12-18	27	
				18-24	28	
				24-30	33	
				30-36	36	
				36-42	35	
				42-48	42	
4G-DCP-16	5/13/2021	RDB immediately downstream of Trowbridge Dam	646.49	48-54	48	GP
				0-6	5	
				6-12	9	
				12-18	9	
				18-24	7	
				24-30	9	
				30-36	5	
				36-42	9	
				42-48	13	
				48-54	33	
				54-60	28	
				60-66	31	
				66-72	37	

Notes:

DCP = Dynamic Cone Penetrometer consisting of a 10 pound hammer falling 22 inches, driving a 1-1/8 inch conical tip.

Blows = number of hammer blows required to drive 6 inches. WOH = weight of hammer.

¹ See GEI, 2021b for gradations

USCS = Unified Soil Classification System; OH = organic clay; OL = organic silt; SP = poorly graded sand; SM = silty sand;

SW = well-graded sand; ML = silt; CL = lean clay.SP-SM = poorly graded sand with silt; GW = well-graded gravel; graded gravel.

Prepared by/Date: JLE 8/17/2021

Checked by/Date: AWH 8/20/2021

Revised by/Date: TLW 9/1/2021

Checked by/Date: DHM 9/2/2021

Table 4. Representative BSTEM Factors of Safety (Pre-design with no bank treatments applied)

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
C	47.14	Left	0.9:1	35 to 38	1.5 to 2.5	1.5 to 3
C	47.14	Right	3.4:1	35 to 38	5.3 to 1.00E+08	5.4 to 1.00E+08
C	47.07	Left	3.2:1	35 to 40	2.6	2.4 to 3.1
C	47.07	Right	3.8:1	34 to 39	2.8	2.8
C	47.00	Left	3.6:1	35 to 40	2.7	2.5 to 2.9
C	47.00	Right	3.4:1	34 to 39	2.8	1.00E+08
C	46.92	Left	4.3:1	35 to 38	1.00E+08	1.00E+08
C	46.92	Right	3.4:1	35 to 38	2 to 14	2.7 to 25.5
C	46.84	Left	3.4:1	35 to 38	2.6	2.5 to 2.9
C	46.84	Right	2.6:1	35 to 38	3.6 to 4	3.5 to 4.4
C	46.8	Left	2.9:1	34 to 39	2.0	2.0
C	46.8	Right	4.8:1	33 to 39	0.6 to 3.3	0.6 to 2.8
C	46.67	Left	3:1	34 to 39	2.2	2.2
C	46.67	Right	2.6:1	33 to 39	5.0 to 1.00E+08	5.0 to 1.00E+08
D	46.46	Left	5.9:1	32 to 37	3.9	3.8 to 4.0
D	46.46	Right	3.7:1	39 to 40	3.5 to 5.5	4.0 to 6.4
D	46.36	Left	3:1	35 to 38	1.00E+08	1.00E+08
D	46.36	Right	2.9:1	35 to 38	2.2	2.1 to 2.2
D	46.26	Left	1.1:1	36 to 39	1.00E+08	1.00E+08
D	46.26	Right	3.1:1	32 to 38	2.0	2.0
D	46.16	Left	2.9:1	36 to 39	2.3	2.3
D	46.16	Right	3.1:1	32 to 38	0.3 to 2.7	2.1
D	46.07	Left	4.3:1	35 to 38	1.1 to 3.6	3.0 to 3.6
D	46.07	Right	3.3:1	35 to 38	2.3	2.3
D	46.00	Left	2.9:1	35 to 38	0.5 to 2.0	2.0
D	46.00	Right	3.2:1	34 to 40	0.5 to 2.6	2.2
D	45.89	Left	3.7:1	35 to 38	5.5 to 10	5 to 16.5
Upper E / F / G	45.89	Right	2.9:1	20	1.0 to 1.1	1.1
D	45.83	Left	2.6:1	35 to 38	1.8 to 1.0E+08	1.8 to 6
Upper E / F / G	45.83	Right	5:1	20	1.9 to 3.2	4 to 25
D	45.77	Left	1.8:1	35 to 38	5.5 to 1.00E+08	5.5 to 8
Upper E / F / G	45.77	Right	4:1	20	2 to 2.7	2 to 3.7
D	45.72	Left	2.3:1	35 to 38	9 to 15	6.5 to 19
Upper E / F / G	45.72	Right	3.3:1	20	0 to 1.00E+08	2.8 to 3

Table 4. Factors of Safety

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
Upper E / F / G	45.68	Left	2:1	35 to 38	2.9 to 1.00E+08	2.9 to 15
Upper E / F / G	45.68	Right	4.2:1	20	0 to 1.00E+08	0 to 2.1
Upper E / F / G	45.64	Left	25:1	35 to 38	1.00E+08	1.00E+08
Upper E / F / G	45.64	Right	3.1:1	20	0 to 1.00E+08	1.2 to 3.5
Upper E / F / G	45.59	Left	25:1	20	1.00E+08	1.00E+08
Upper E / F / G	45.59	Right	3.8:1	20	0 to 1.00E+08	3.4 to 4.2
Upper E / F / G	45.56	Left	7.7:1	20	0.5 to 1.00E+08	0.5 to 1.0E+08
Upper E / F / G	45.56	Right	4.0:1	20	0 to 1.00E+08	1.8 to 5.3
Upper E / F / G	45.51	Left	3.6:1	20	0 to 1.00E+08	0 to 4
Upper E / F / G	45.51	Right	3:1	20	0 to 1.00E+08	1.8 to 4.1
Upper E / F / G	45.48	Left	5.3:1	20	0 to 2.8	1.5 to 2.6
Upper E / F / G	45.48	Right	2.4:1	20	0 to 1.00E+08	1.3 to 2
Upper E / F / G	45.44	Left	5.9:1	20	1.00E+08	1.00E+08
Upper E / F / G	45.44	Right	7.1:1	20	3.9 to 1.00E+08	4.9 to 1.00E+08
Upper E / F / G	45.41	Left	7.1:1	20	3.3	2.8
Upper E / F / G	45.41	Right	4.8:1	33 to 41	6.9 to 8.3	7 to 9.4
Upper E / F / G	45.37	Left	9.1:1	20	7 to 1.00E+08	7 to 1.00E+08
Upper E / F / G	45.37	Right	4:1	33 to 41	1.00E+08	1.00E+08
Upper E / F / G	45.35	Left	4.3:1	20	1.5 to 2.9	2.8
Upper E / F / G	45.35	Right	14.3:1	33 to 41	0.7 to 35	27 to 41
Upper E / F / G	45.32	Left	3.7:1	20	0 to 1.00E+08	6.5 to 11.5
Upper E / F / G	45.32	Right	4.3:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.27	Left	1.2:1	33 to 41	2.1 to 90	2.3
Lower E	45.27	Right	0.8:1	33 to 41	5.5 to 10.2	7.6 to 10.2
Lower E	45.24	Left	1.3:1	33 to 41	1.7 to 2.5	1.8 to 2.2
Lower E	45.24	Right	3.2:1	33 to 41	9 to 15	12 to 17.5
Lower E	45.2	Left	7.7:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.2	Right	5.9:1	33 to 41	0.96 to 24	18.5 to 24
Lower E	45.16	Left	1.3:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.16	Right	5.9:1	34 to 39	1.00E+08	1.00E+08
Lower E	45.13	Left	2.9:1	33 to 41	3.7 to 5.4	4.4 to 5.3
Lower E	45.13	Right	0.1:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.1	Left	1.6:1	33 to 41	2 to 10.6	2.2 to 2.7
Lower E	45.1	Right	1.6:1	33 to 41	1.00E+08	1.00E+08
Lower E	45.08	Left	1.8:1	33 to 41	1.4 to 9.1	1.4 to 1.6
Lower E	45.08	Right	4.0:1	33 to 41	9.9 to 1.00E+08	14.5 to 18.5

Table 4. Factors of Safety

Subarea	River Mile	Bank (Left or Right)	Bank Slope ¹ (H:V)	Friction Angle ϕ (degrees)	Factor of Safety ² Starting Groundwater Elevation: 660 ft	Factor of Safety ² Starting Groundwater Elevation: 650 ft
Lower E	45.05	Left	1.2:1	33 to 41	1.3 to 6.5	1.5 to 1.6
Lower E	45.05	Right	3.4:1	32 to 36	15.6 to 1.00E+08	21 to 27
Lower E	45.02	Left	2:1	33 to 41	1.5 to 9.8	1.6
Lower E	45.02	Right	3.2:1	33 to 41	3.5 to 4.9	4.2 to 4.9
Lower E	45.00	Left	1.2:1	33 to 41	1 to 9.7	1.1 to 1.5
Lower E	45.00	Right	3.3:1	33 to 41	7.8 to 24.4	10.8 to 14.2

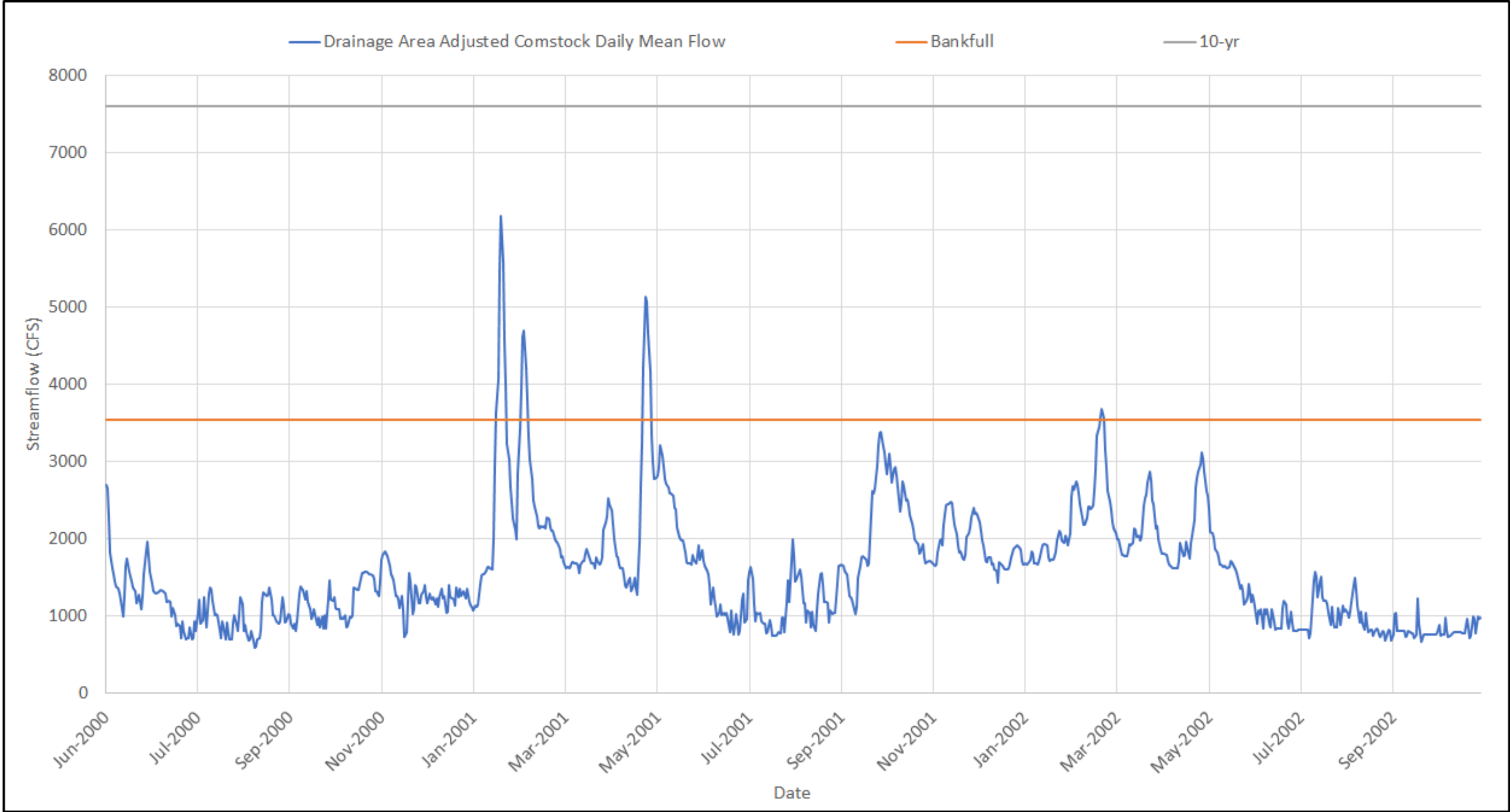
Notes:

1. Bank Slope measured at beginning of simulation.
2. Factor of Safety presented as range of values during the 7-year 1-D HEC-RAS simulation. Factor of Safety < 1 is unstable; 1-1.3 is marginally stable; >1.3 is stable.
3. At Factor of Safety values < 1 the model will simulate a bank failure. The Factor of Safety is often higher after a bank failure due to the resulting shallower bank slope.

BSTEM = Bank Stability and Toe Erosion Model; H = horizontal; V = vertical; ft = feet.

Created by: EG 9/17/2021

Checked by: LAP 9/24/2021



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan



STREAMFLOW DURING
CALIBRATION PERIOD

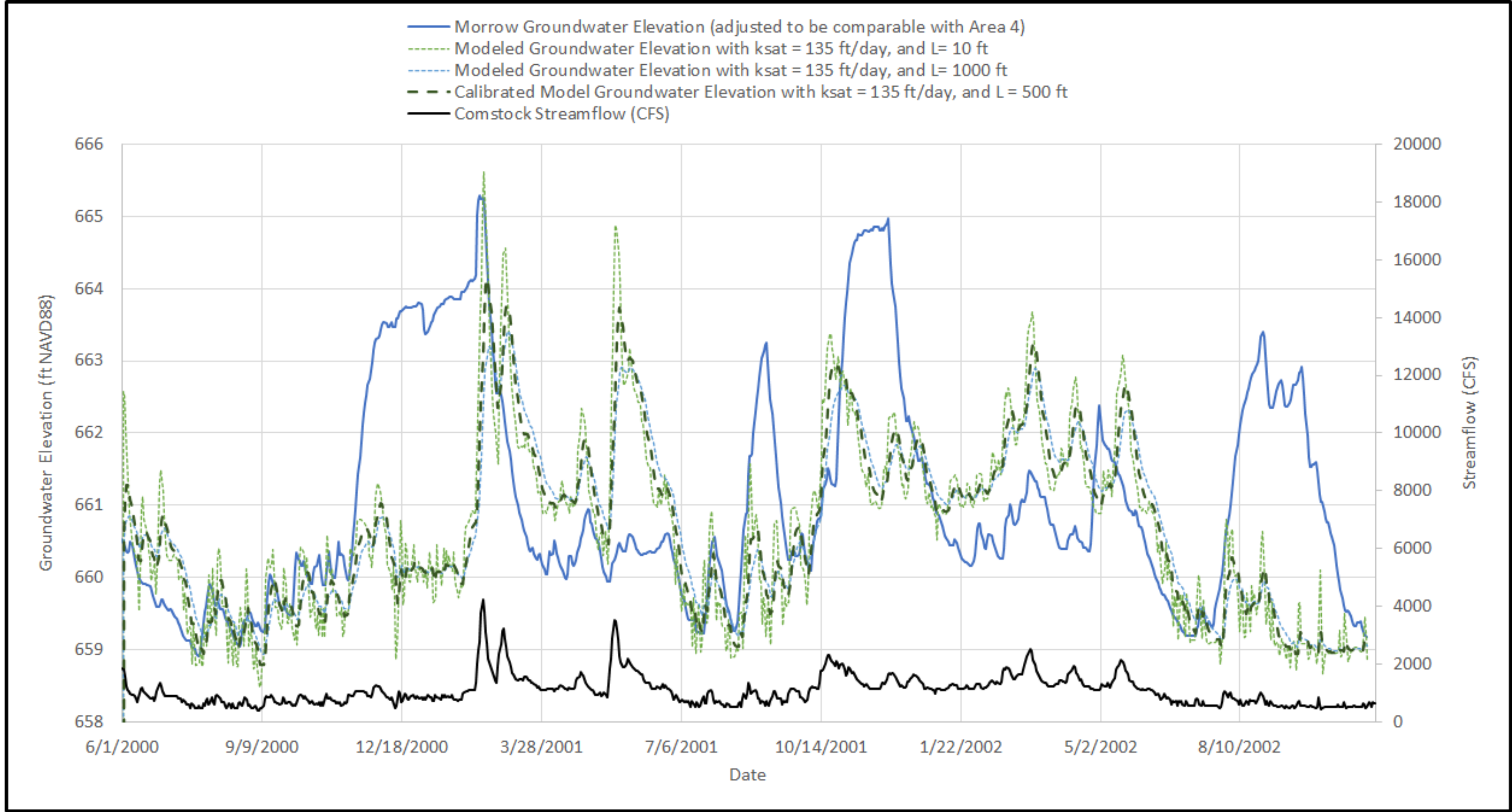
Kalamazoo River Areas 2, 3, and 4 Remediation LLC

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September 2021

Fig. 1

Prepared by XX/Date: 08/19/2021
Checked by XX/Date: 08/19/2021



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
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Kalamazoo River Areas 2, 3, and 4 Remediation LLC

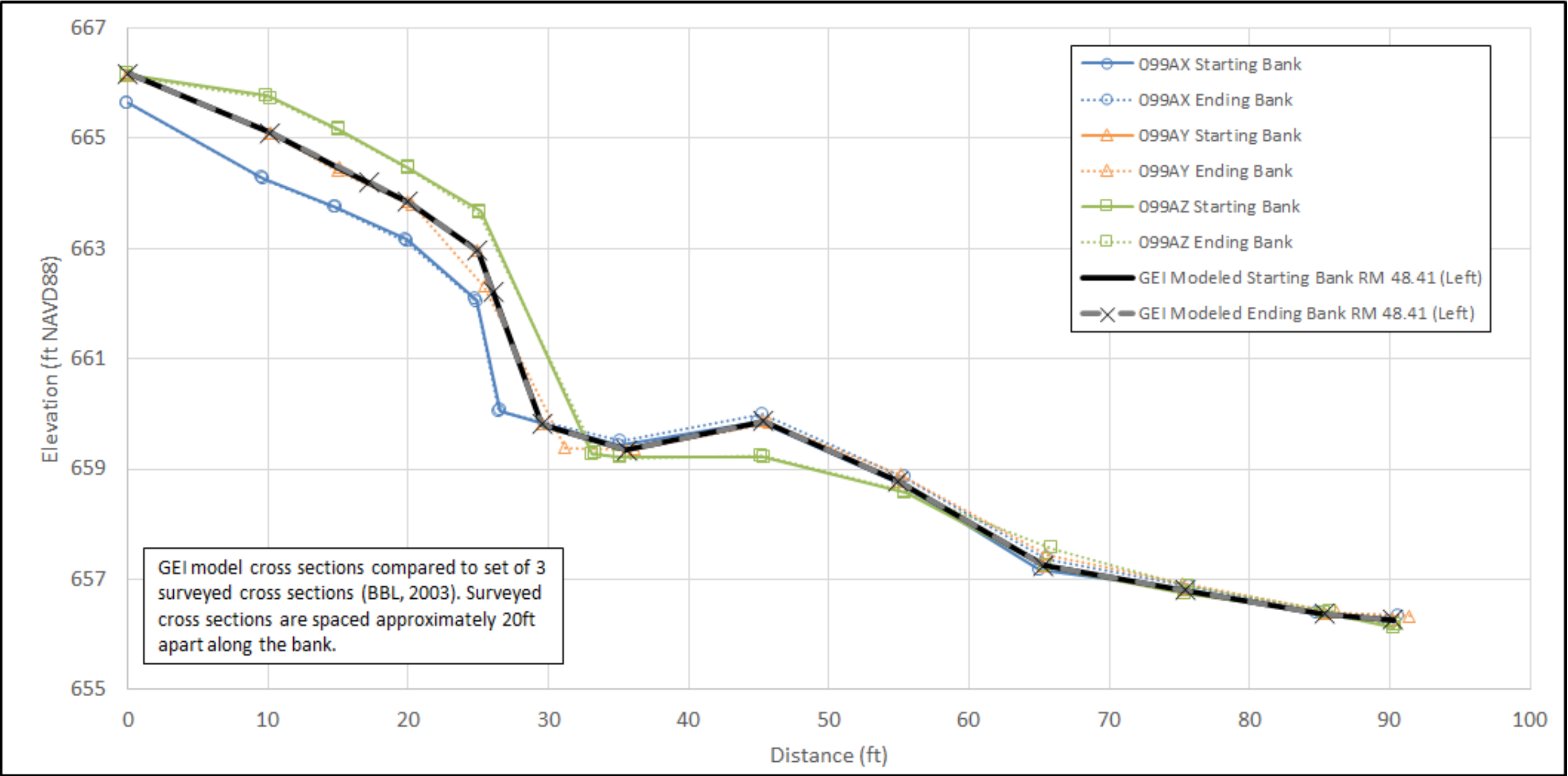


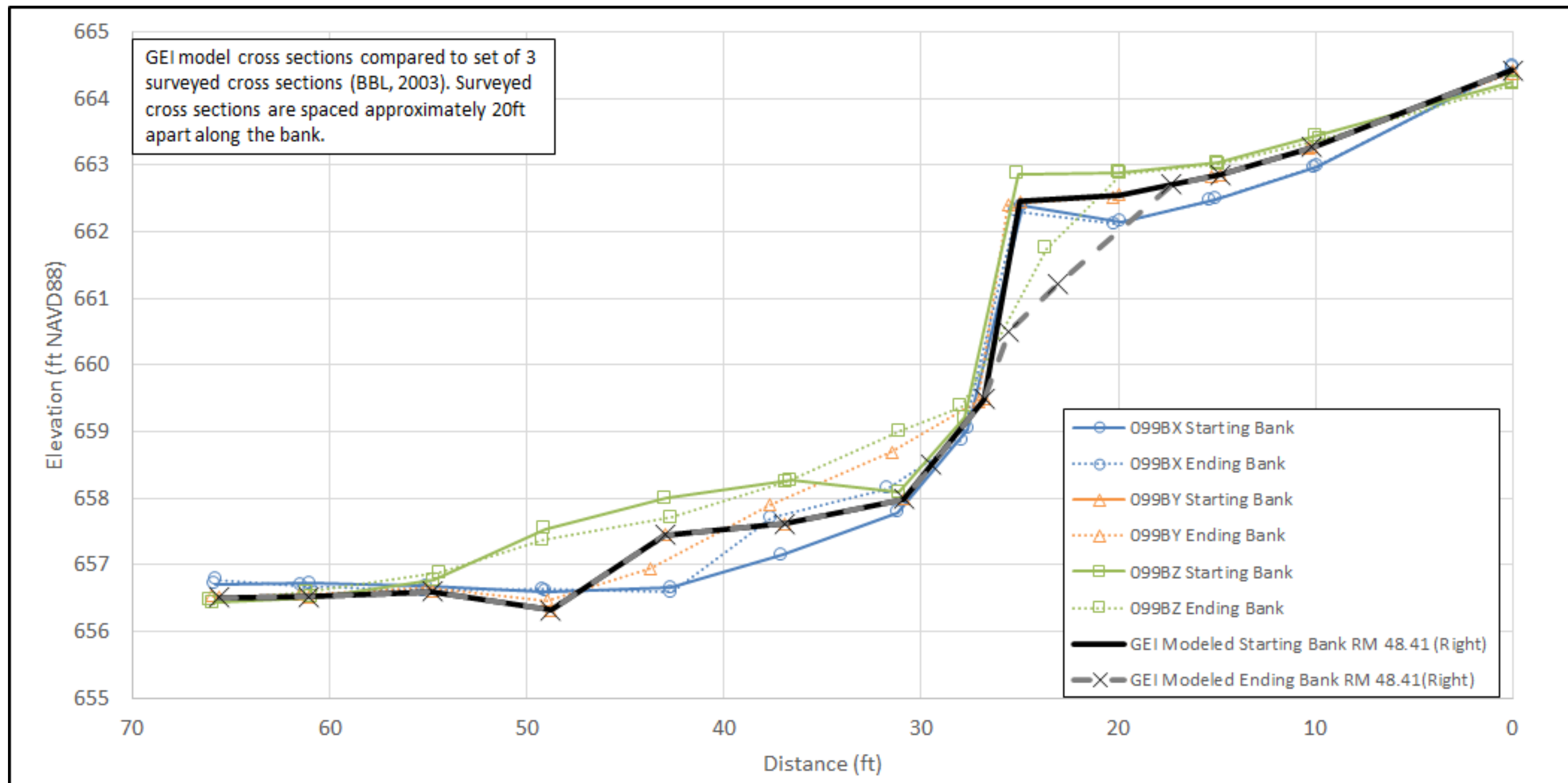
GROUNDWATER
CALIBRATION

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September 2021

Fig. 2





OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

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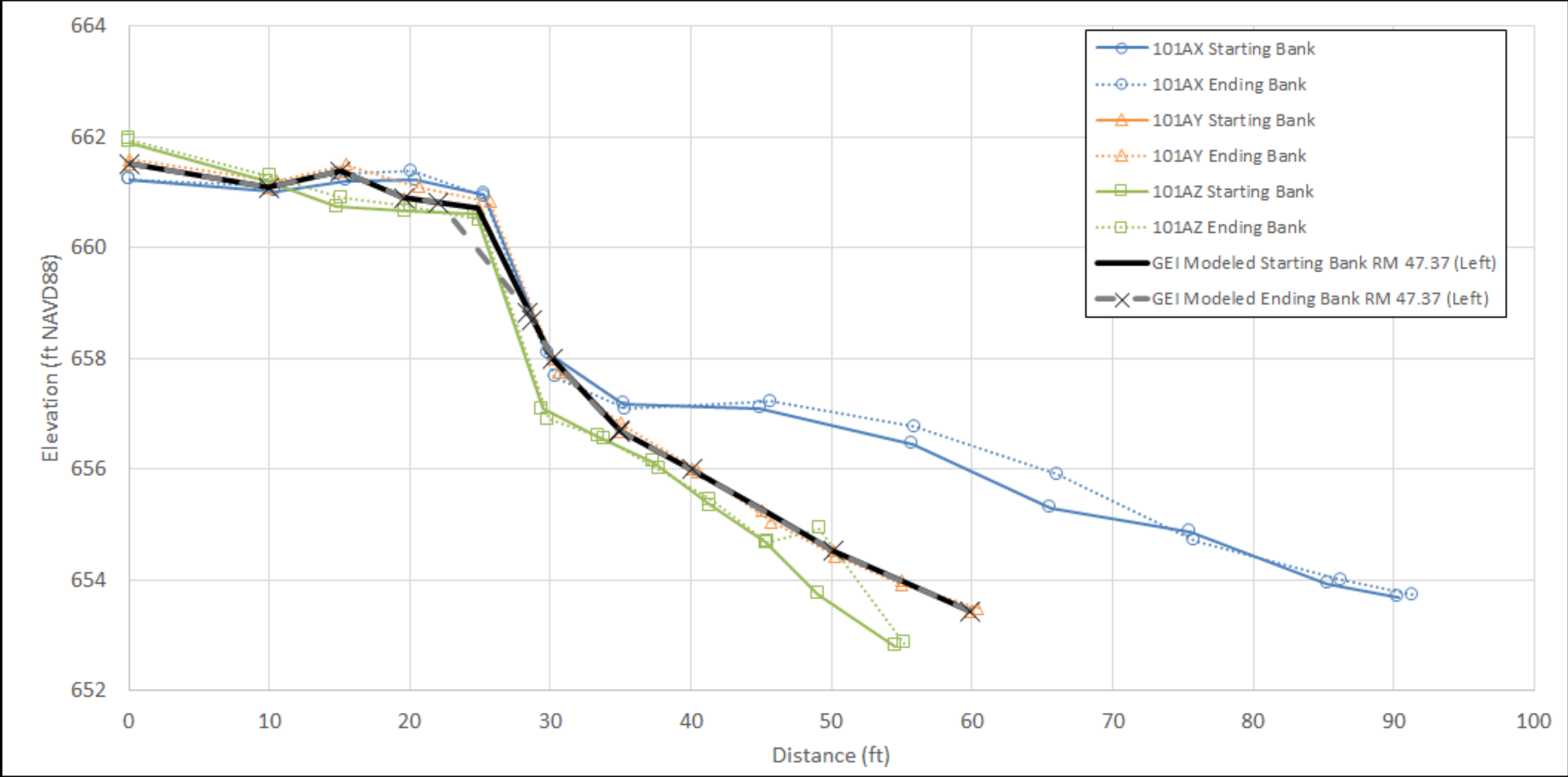


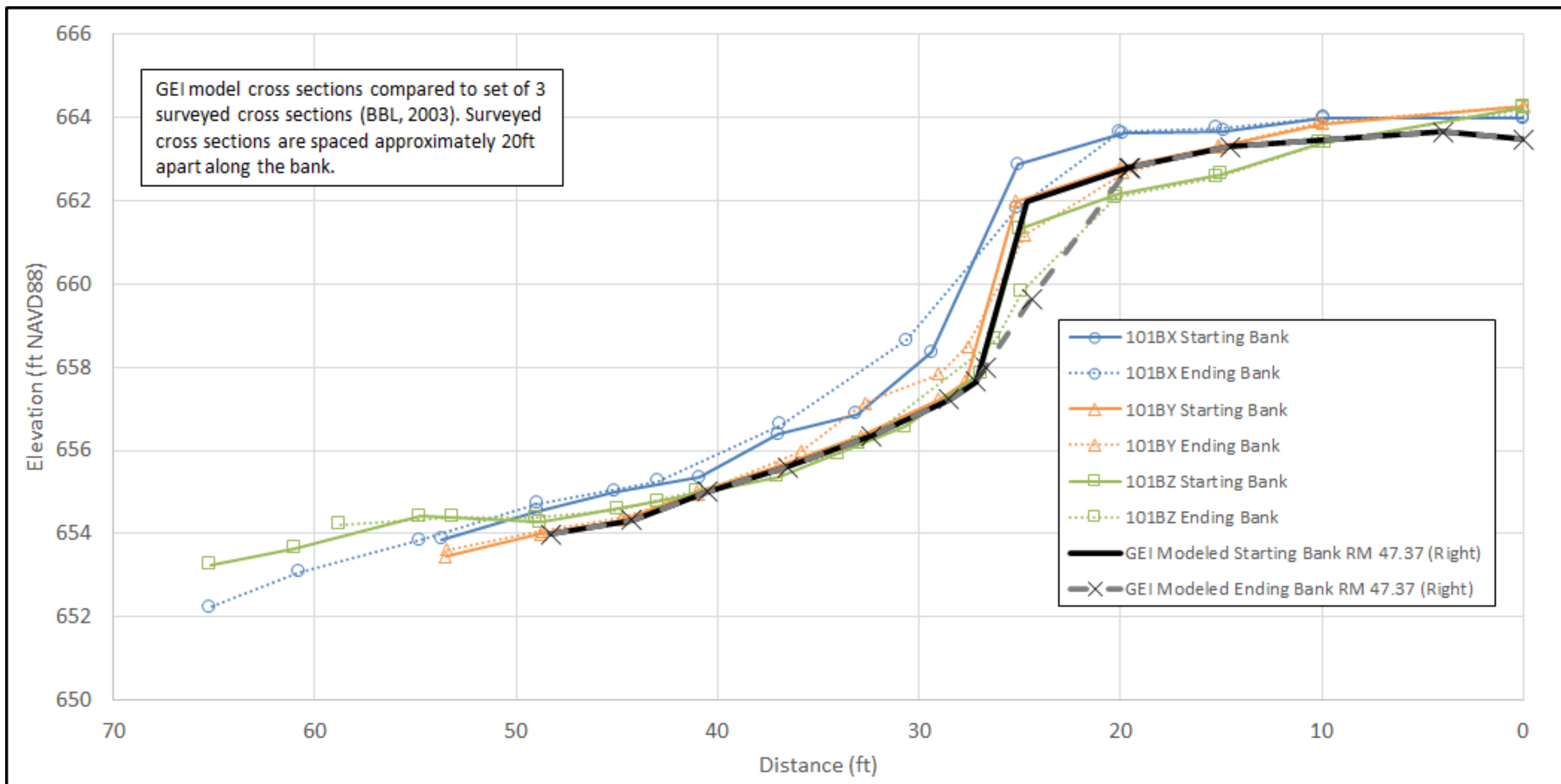
BANK FAILURE
CALIBRATION

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Fig. 3b





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Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

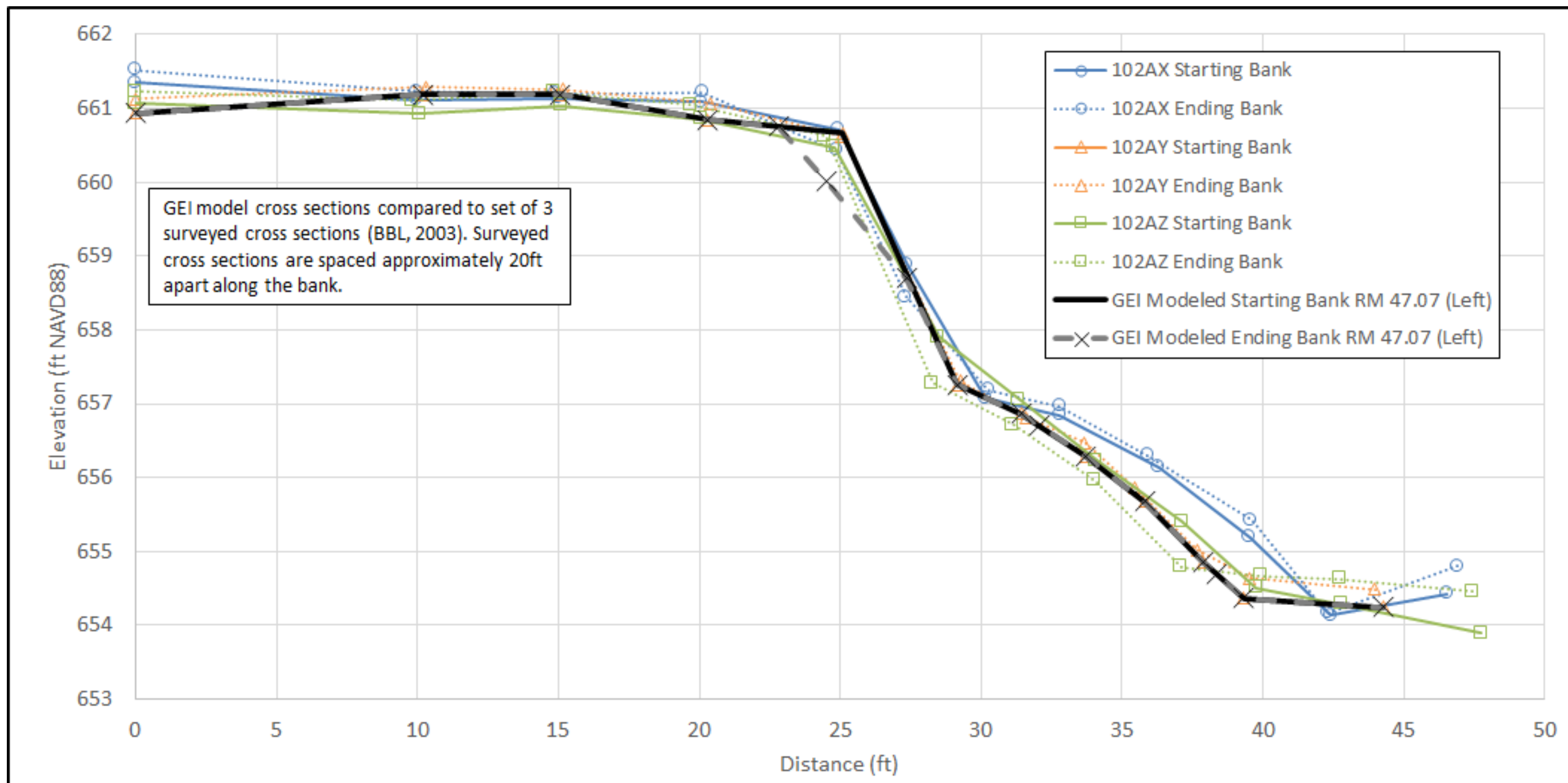


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BANK FAILURE
CALIBRATION

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Fig. 3d



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
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Kalamazoo River Areas 2, 3, and 4 Remediation LLC

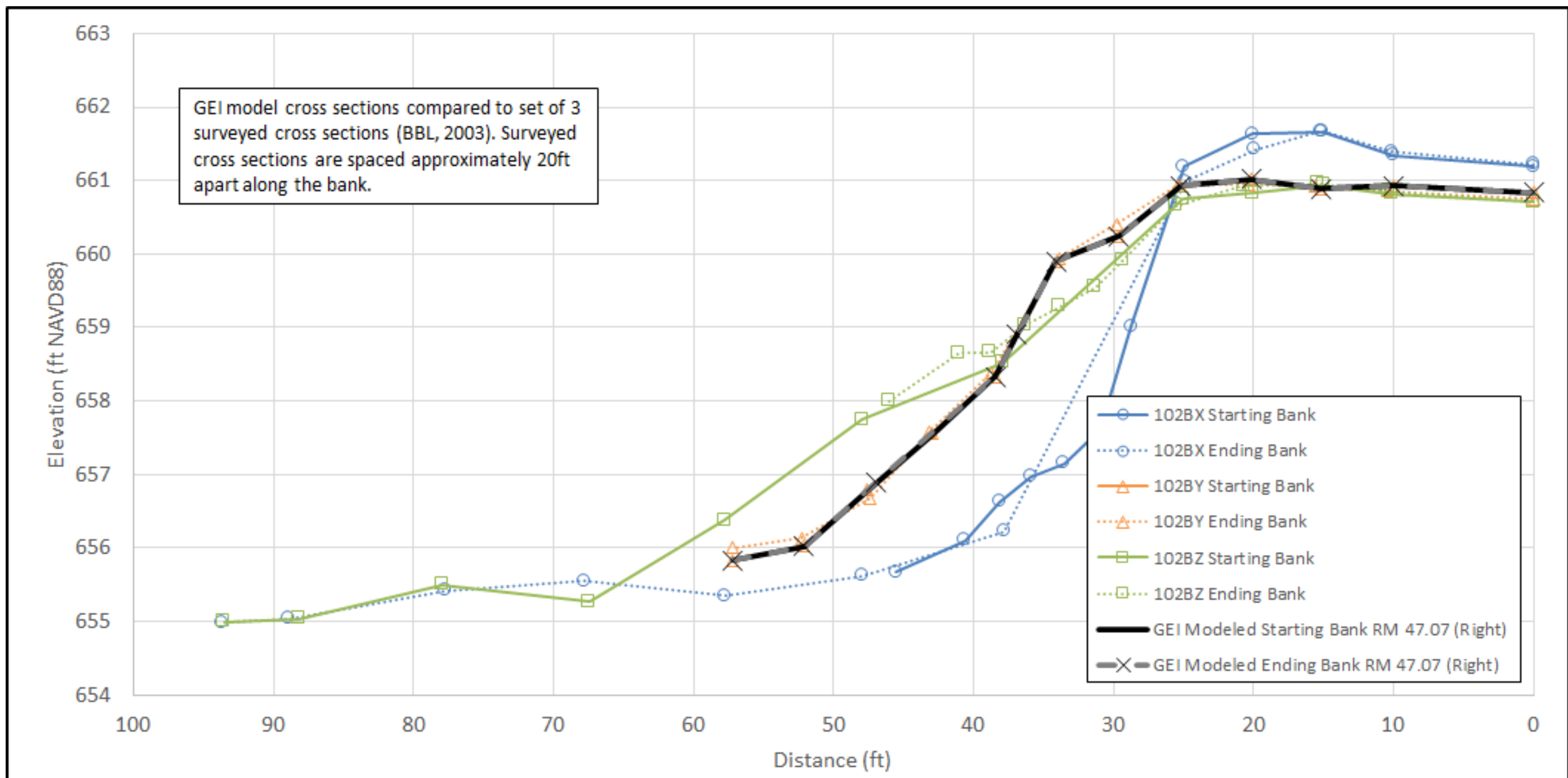


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BANK FAILURE
CALIBRATION

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Fig. 3e



OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC

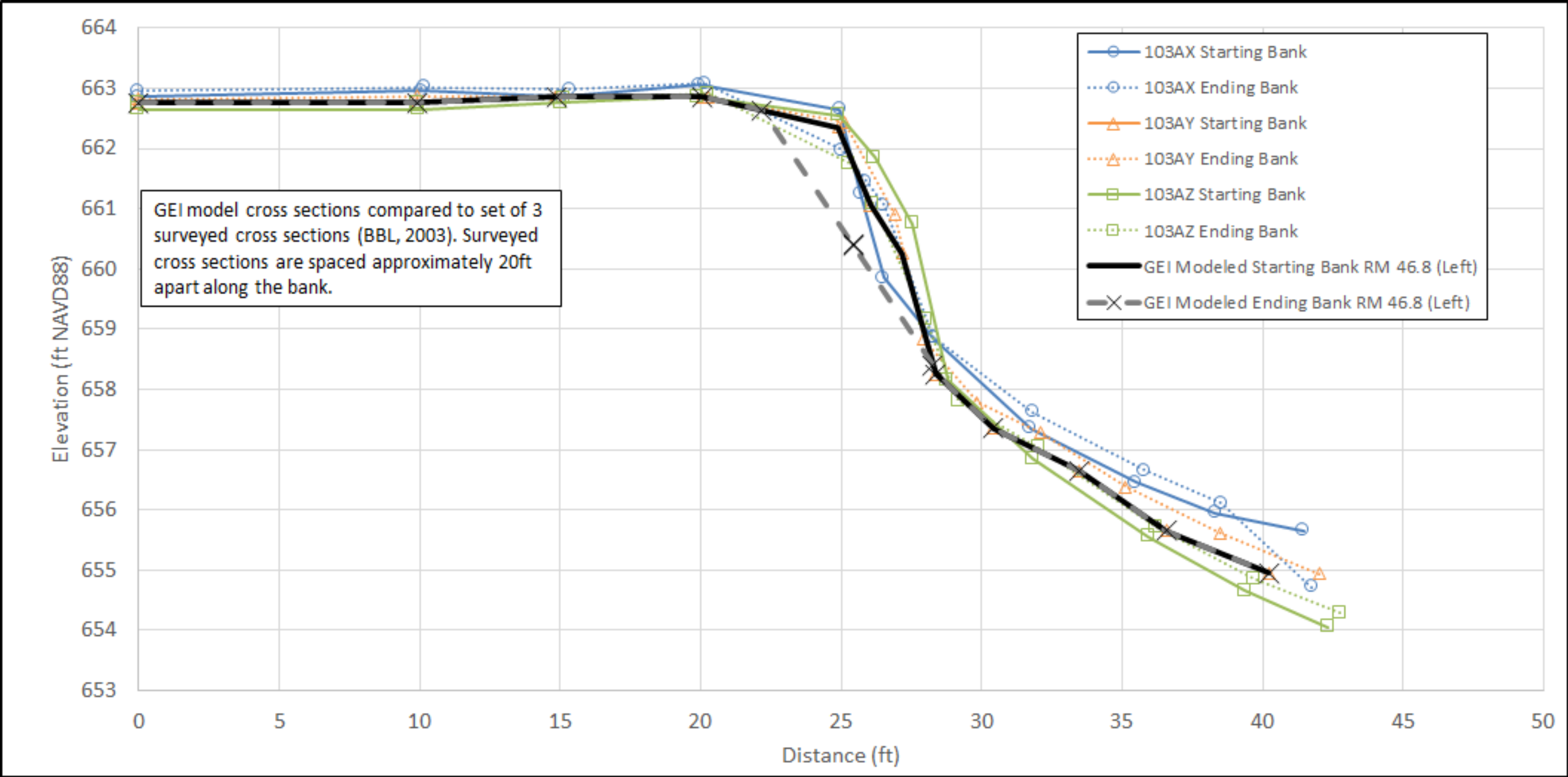


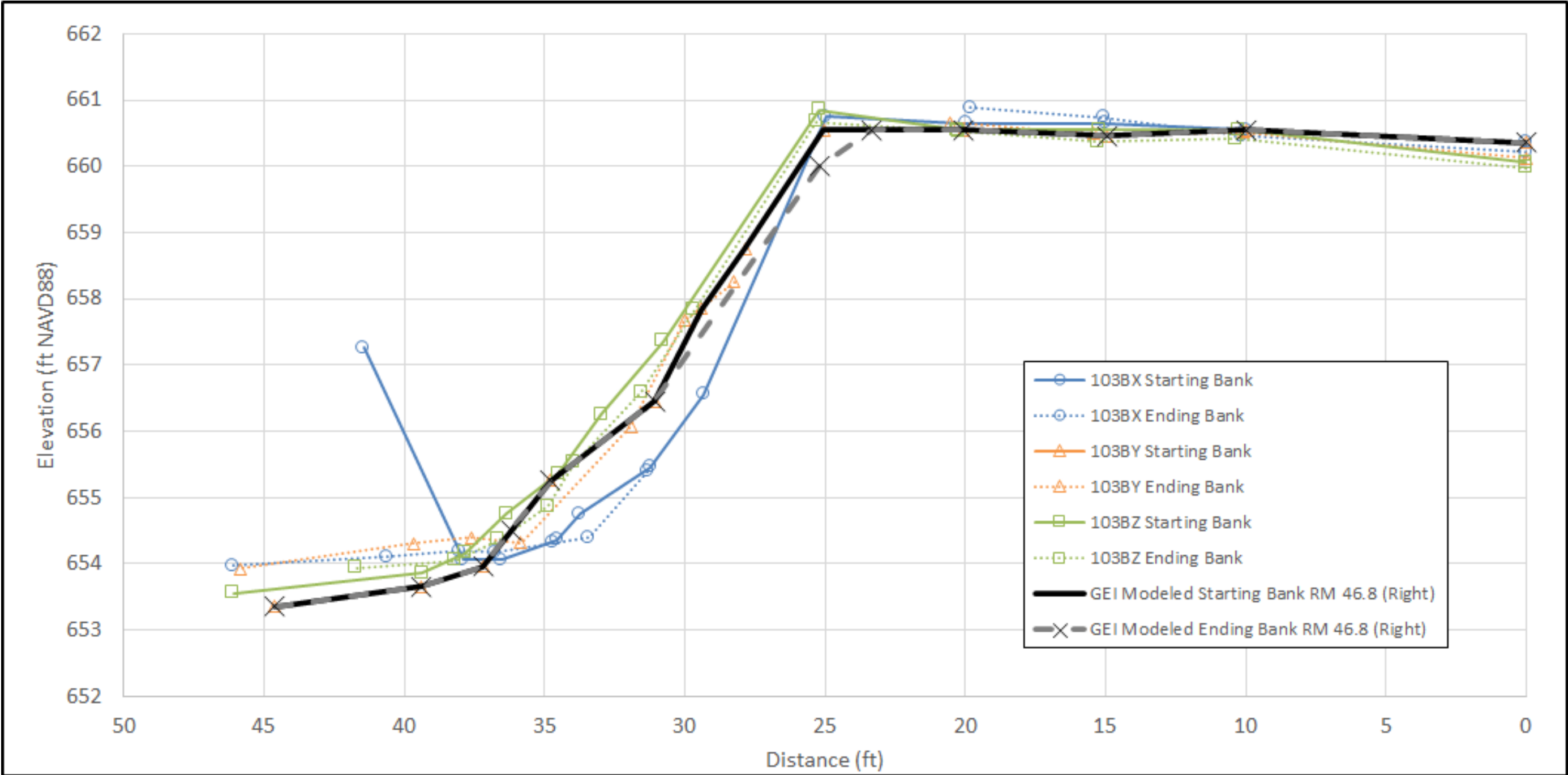
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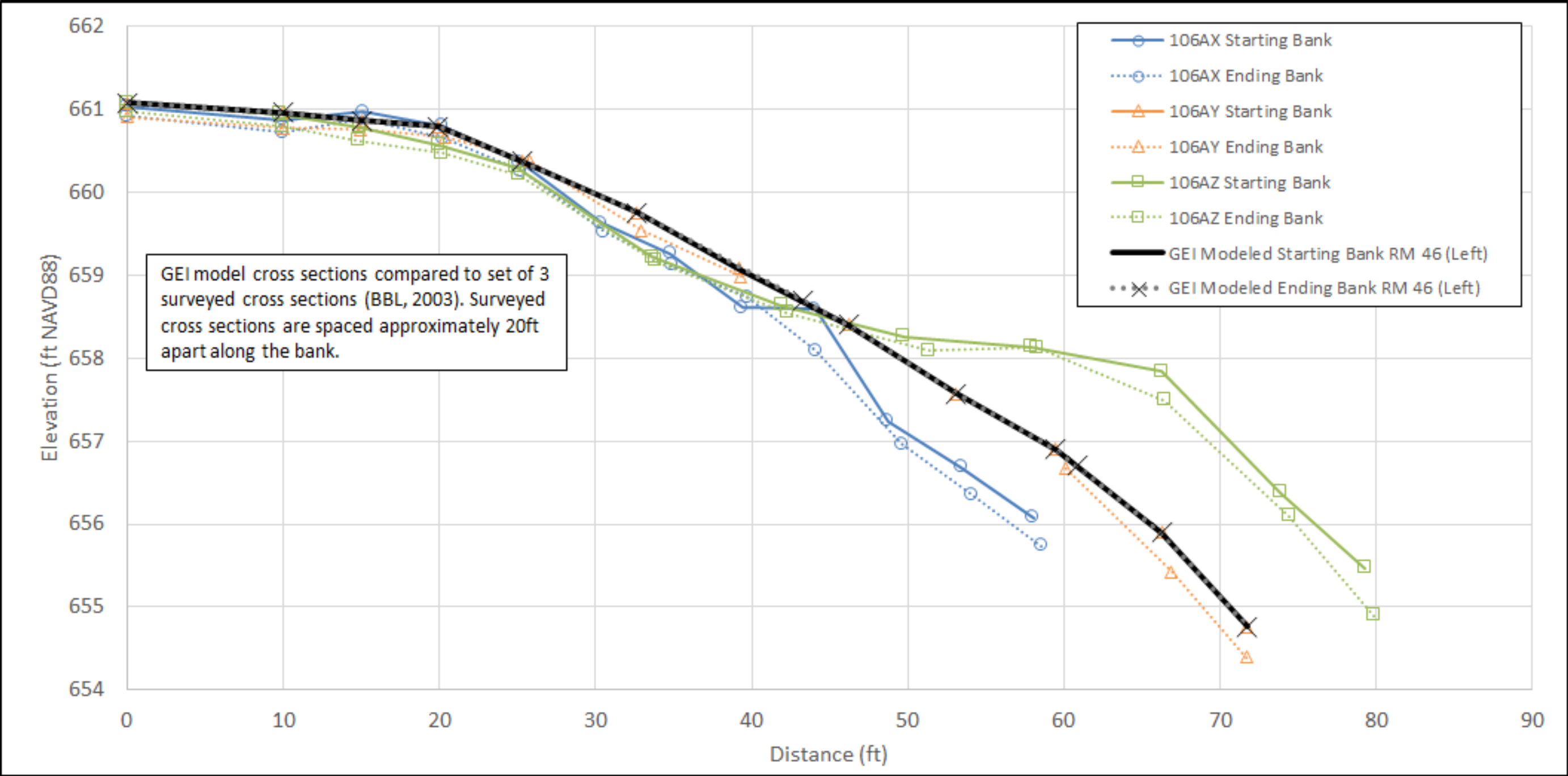
Project 2000273

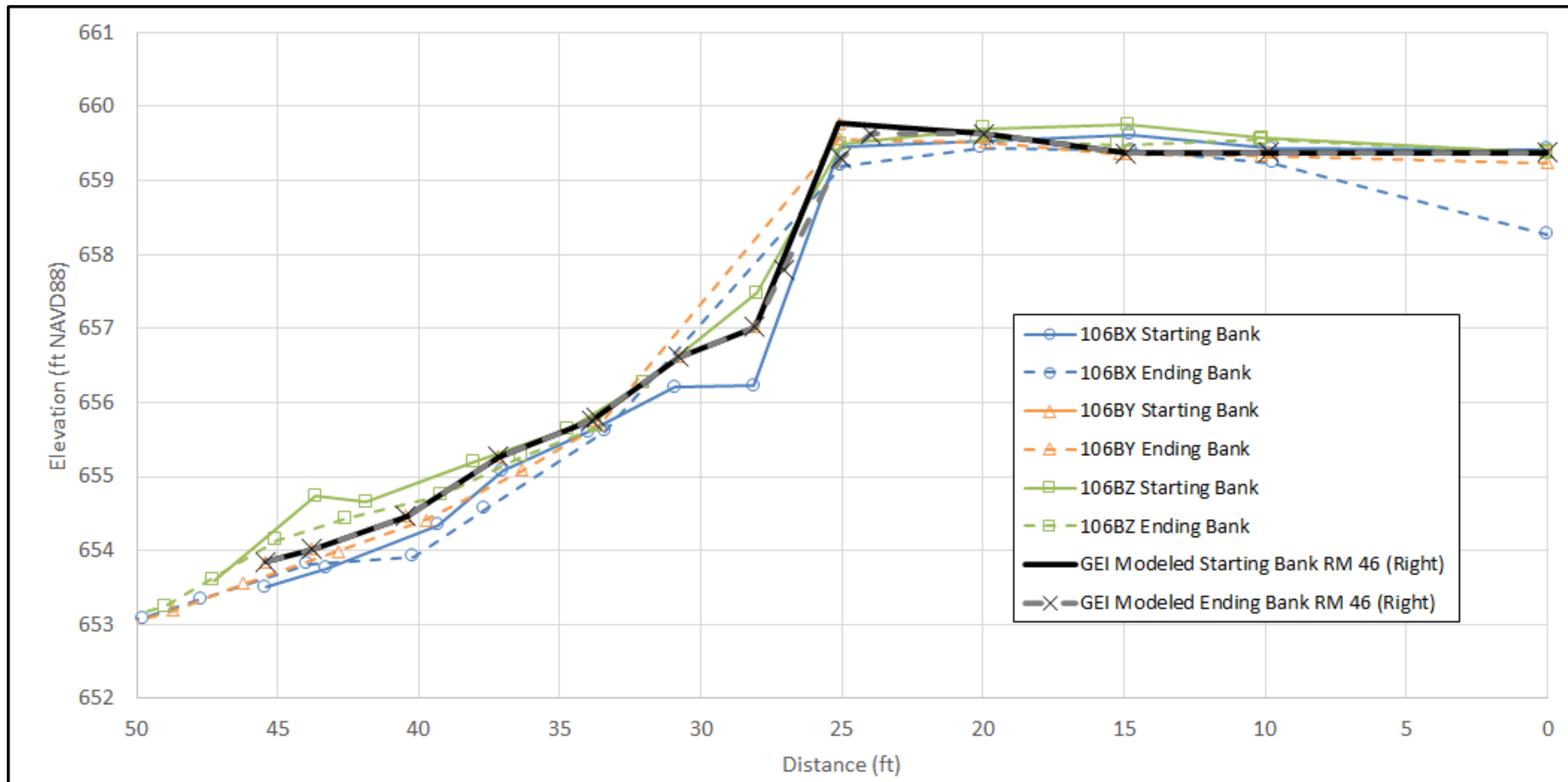
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Fig. 3f









OU5 Allied Paper/Portage Creek/Kalamazoo River
Superfund Site
Area 4 TCRA - Allegan County, Michigan

Kalamazoo River Areas 2, 3, and 4 Remediation LLC



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BANK FAILURE
CALIBRATION

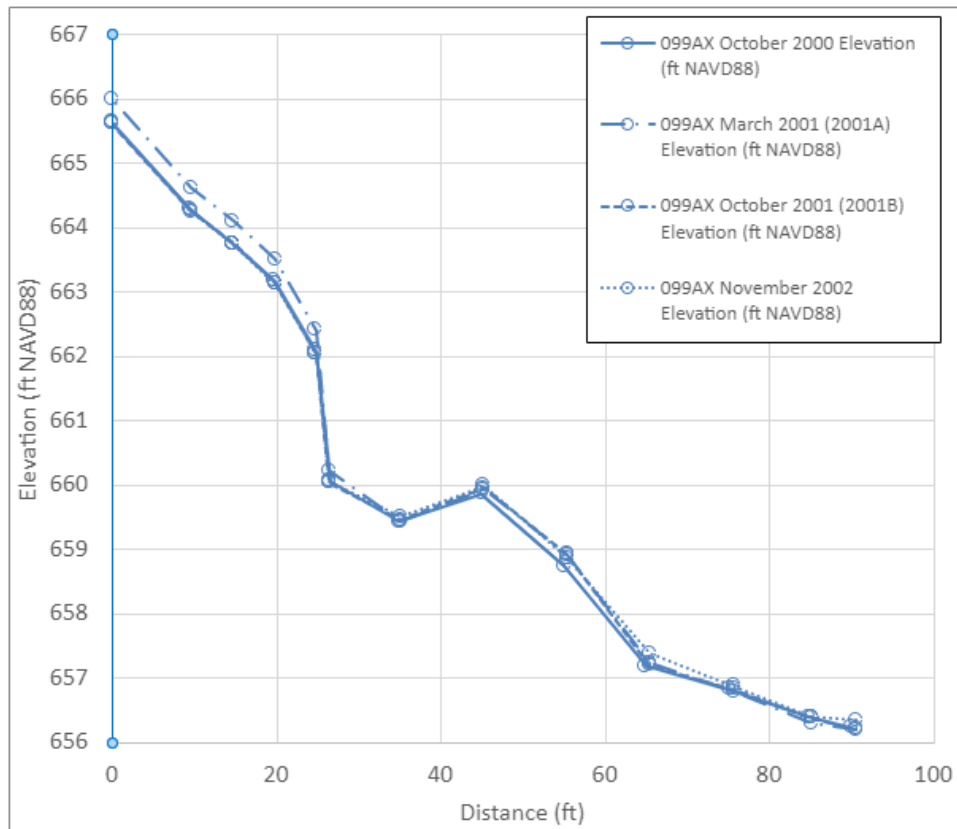
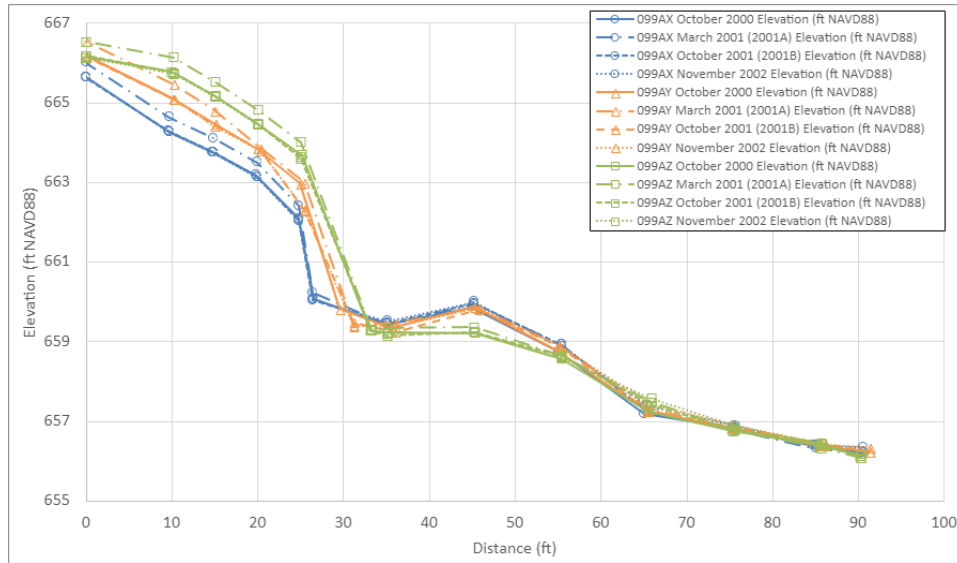
September 2021

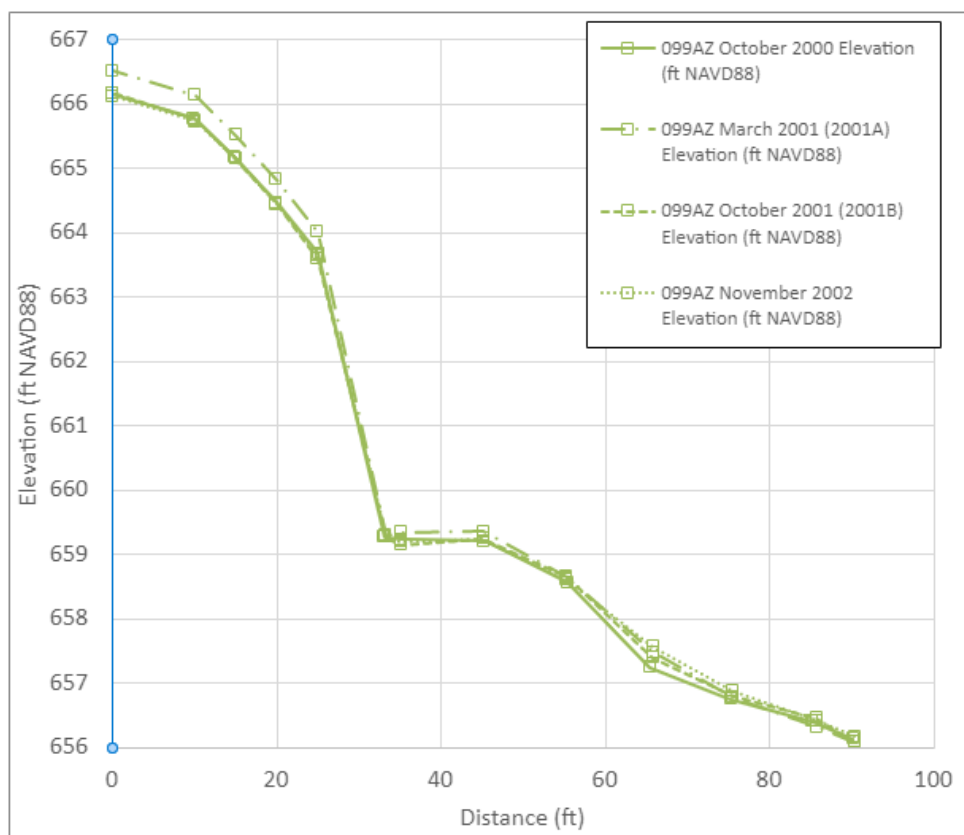
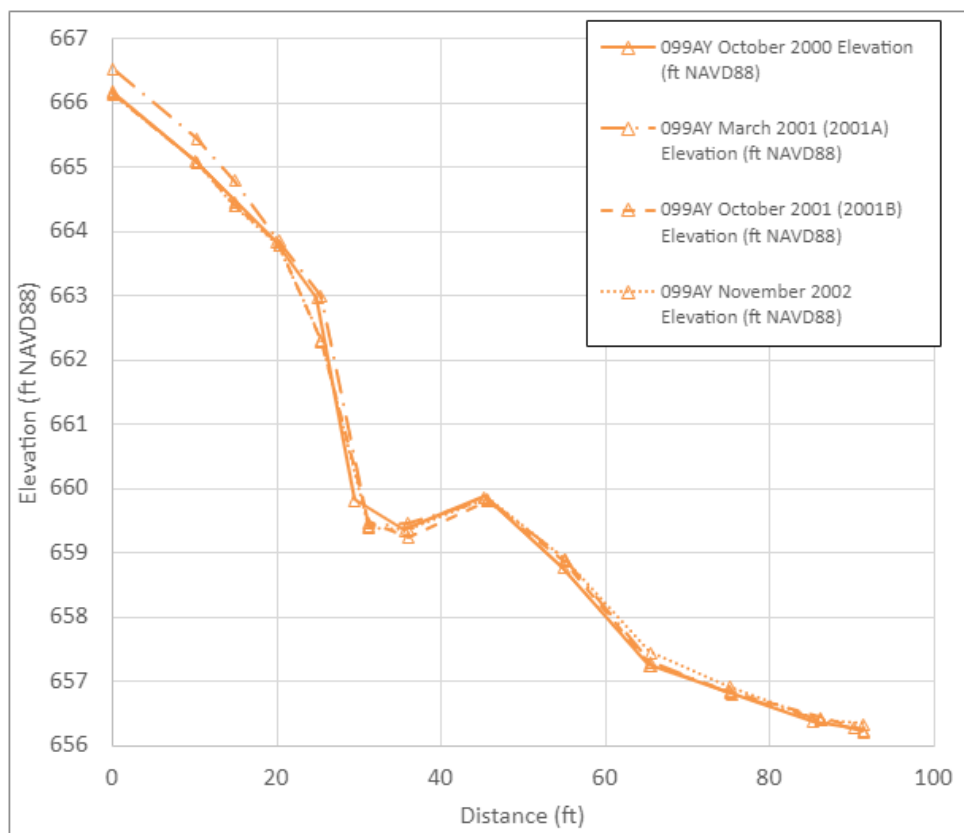
Fig. 3j

Attachment A: Bank Survey Data

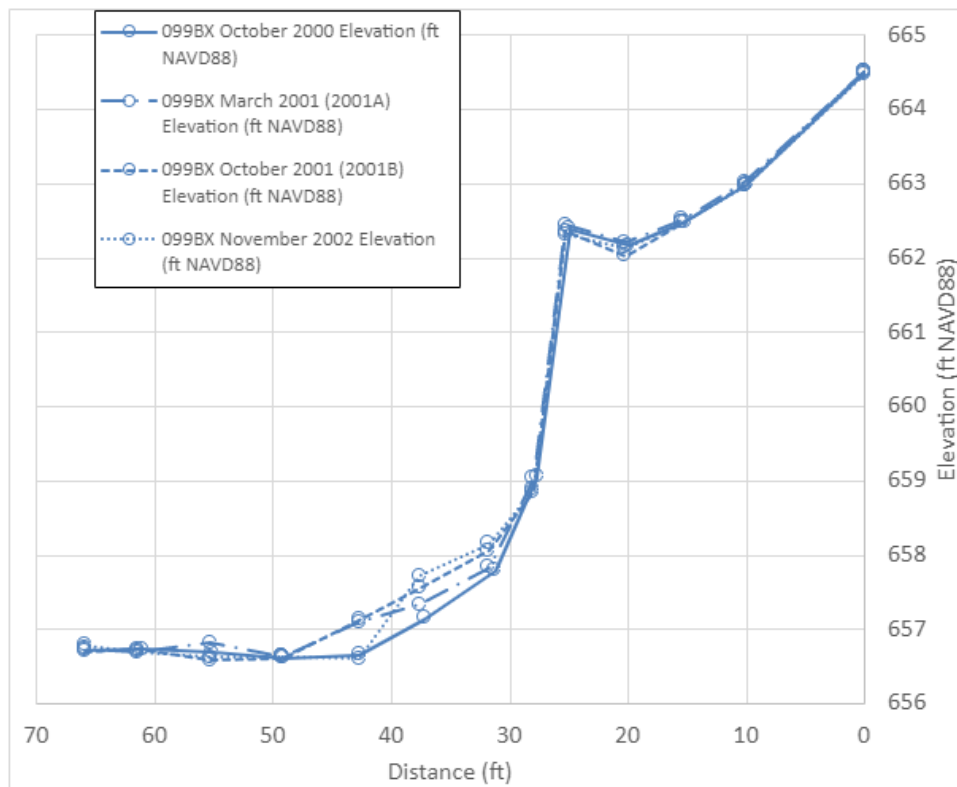
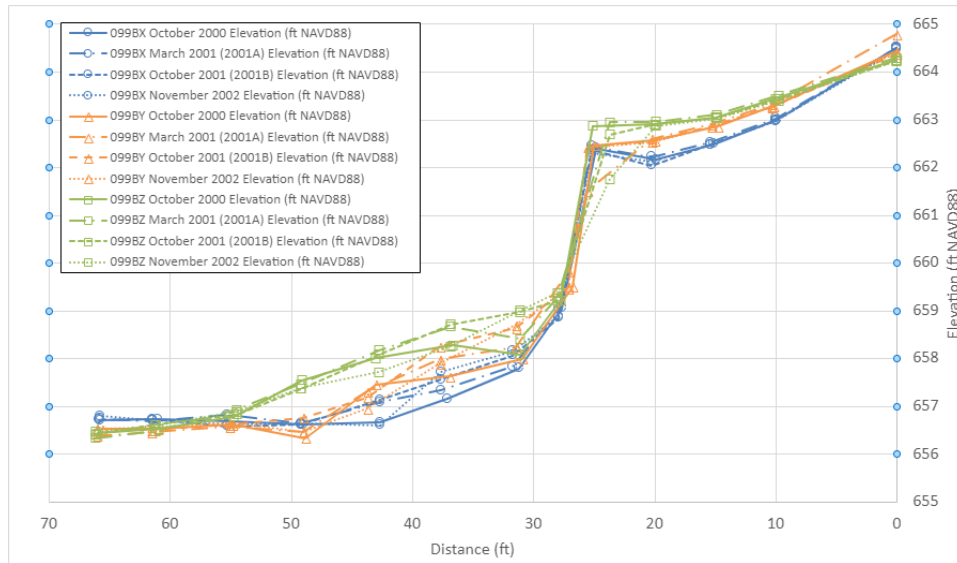
Reference: Bank survey data reproduced from BBL (2003). "Attachment A, Erosion Pin Monitoring Data." Kalamazoo River Study Group Allied Paper Inc./Portage Creek/Kalamazoo River Superfund Site. March 2003.

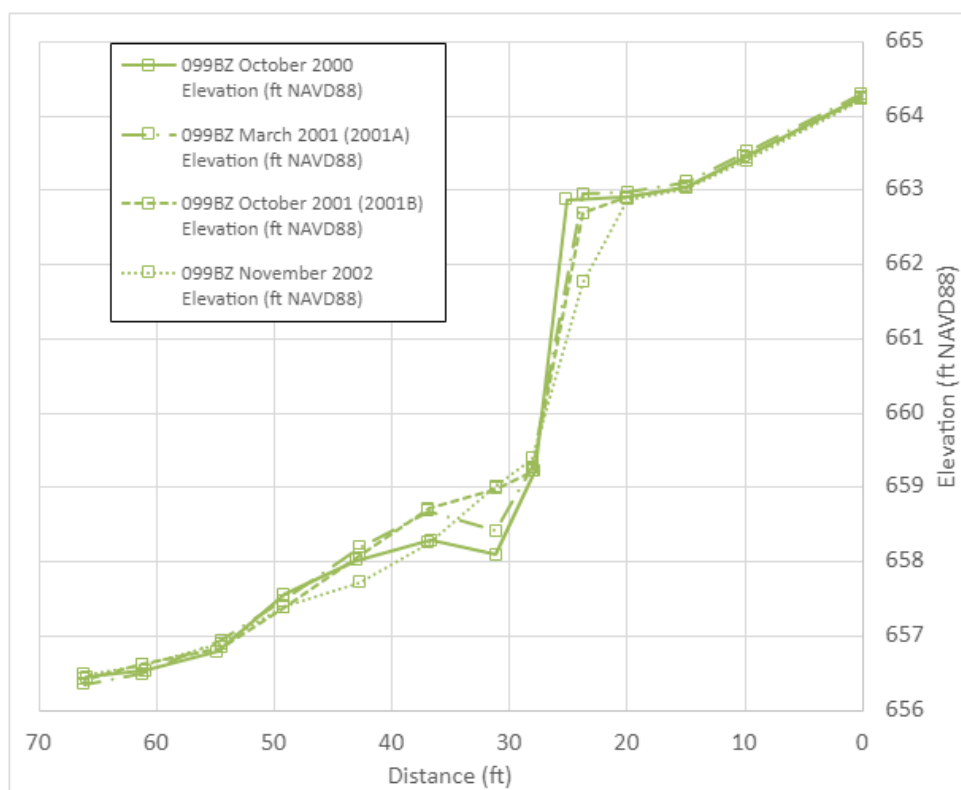
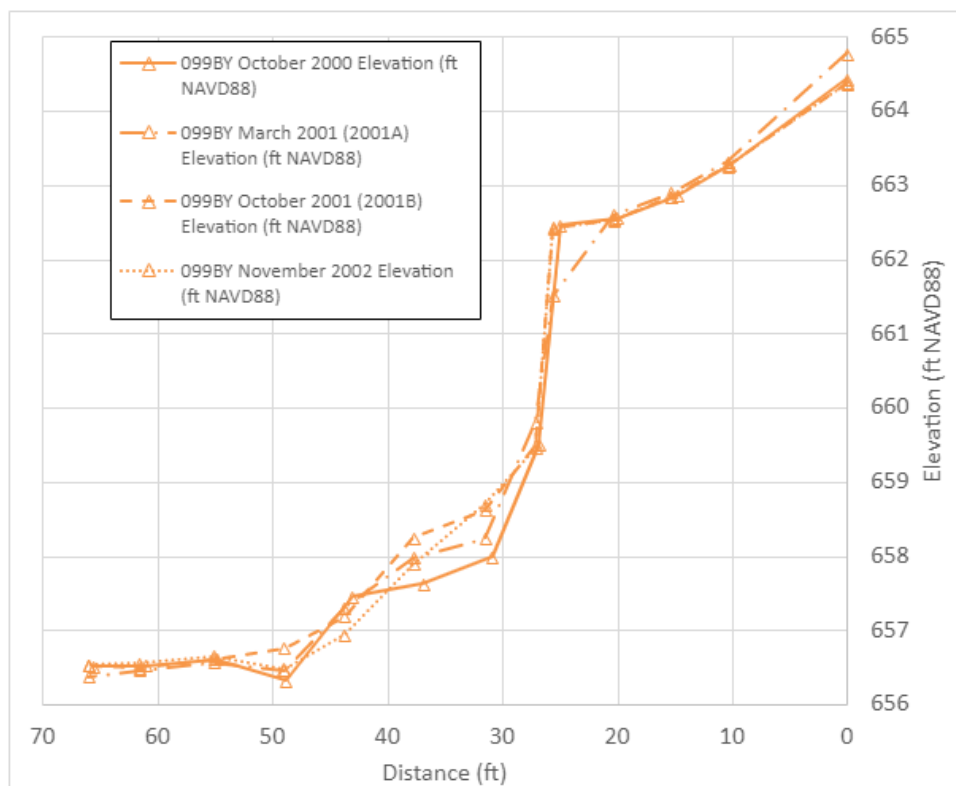
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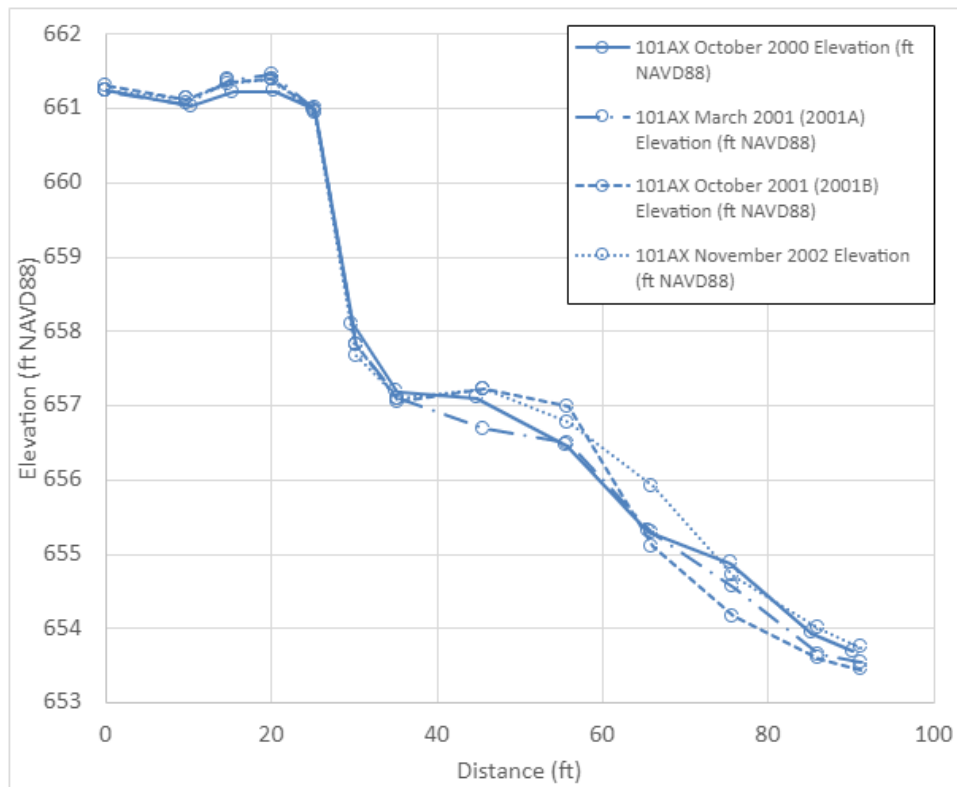
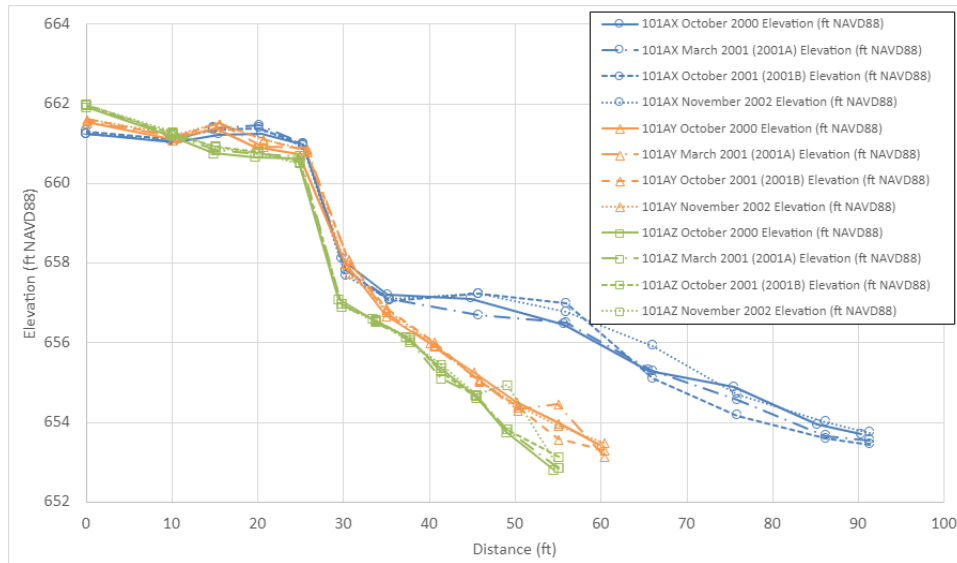


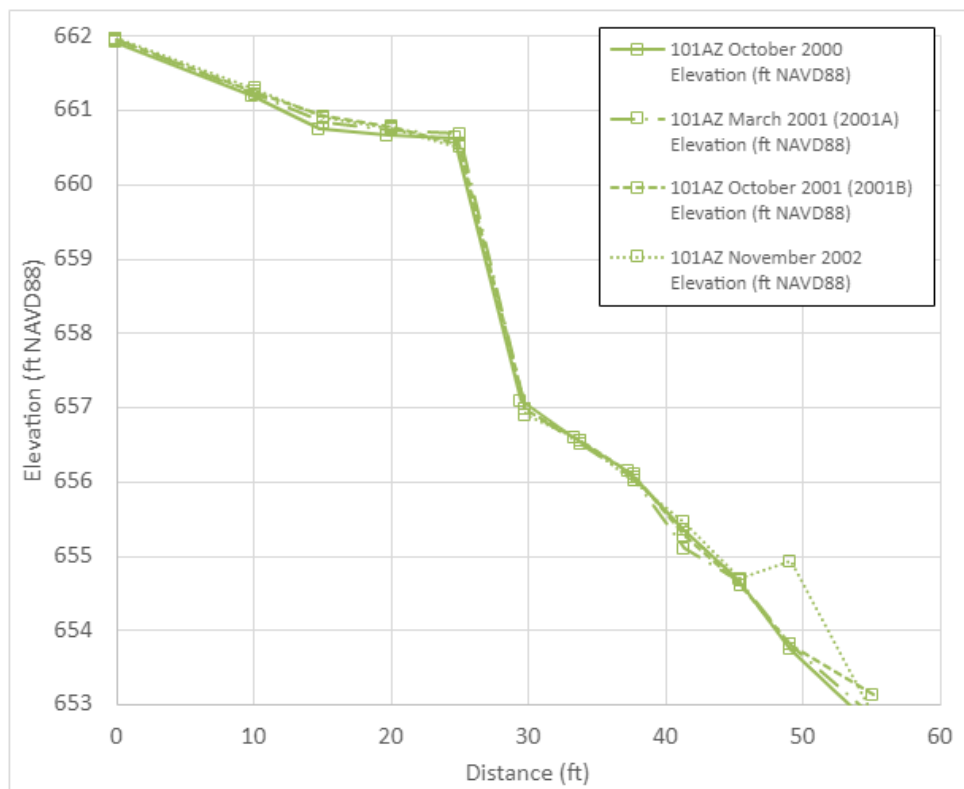
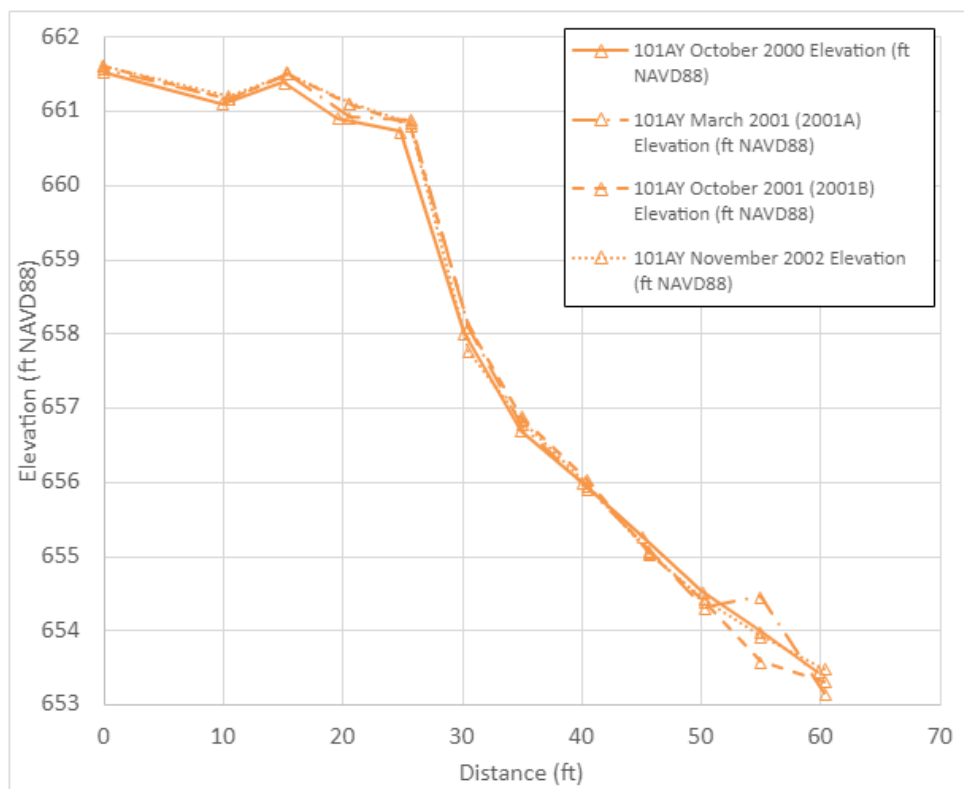
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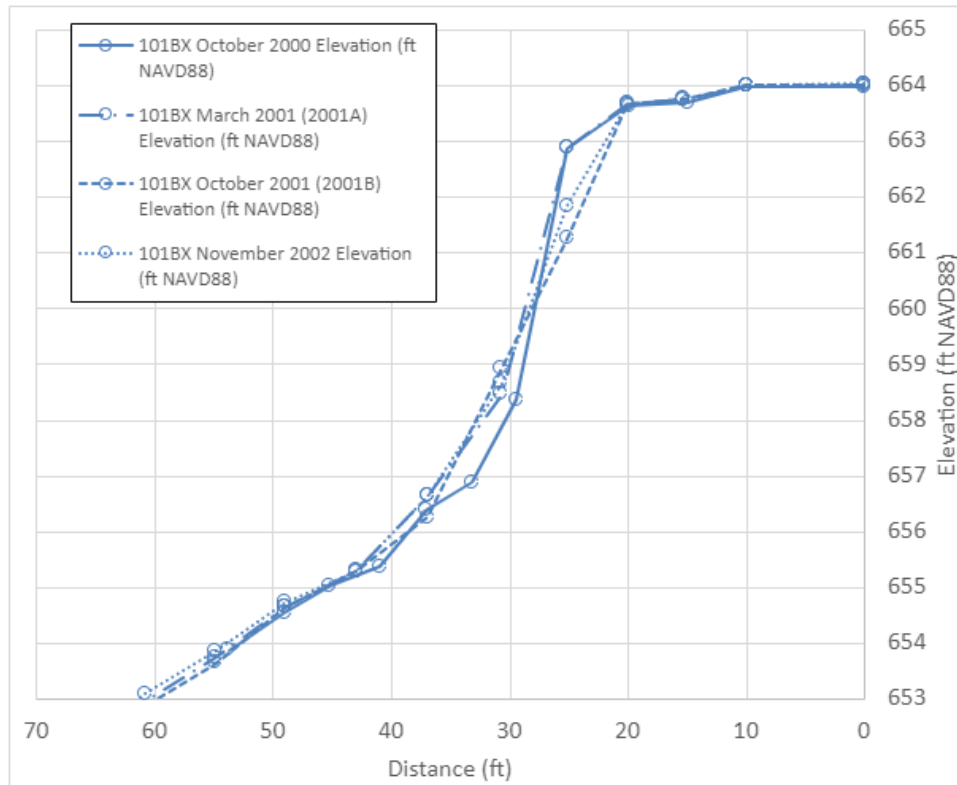
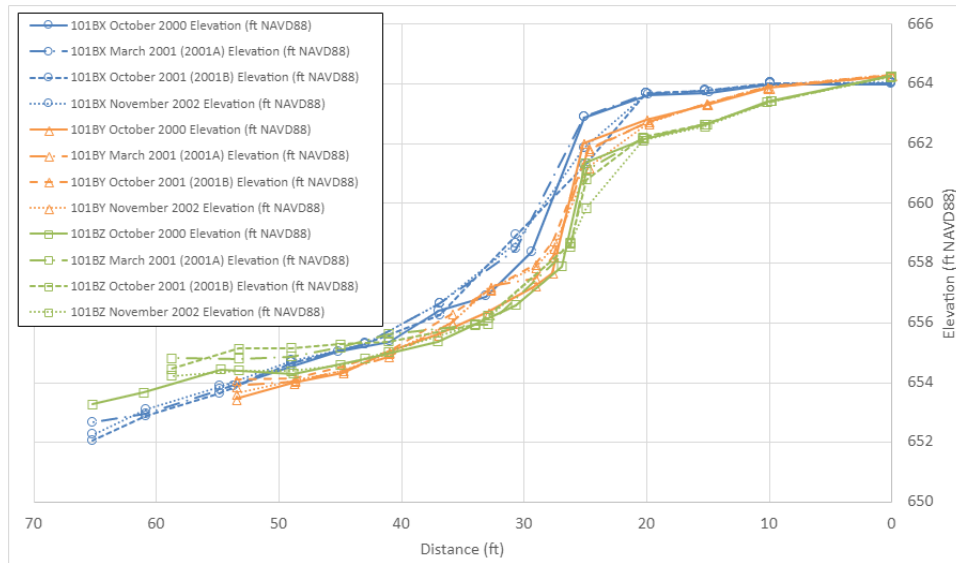


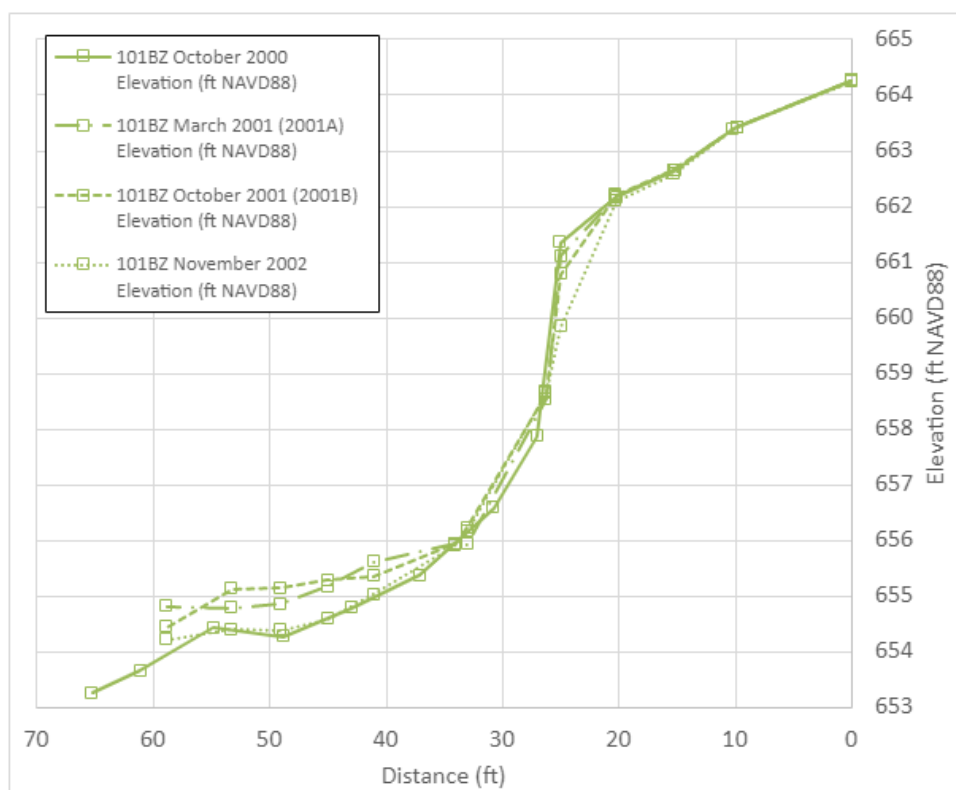
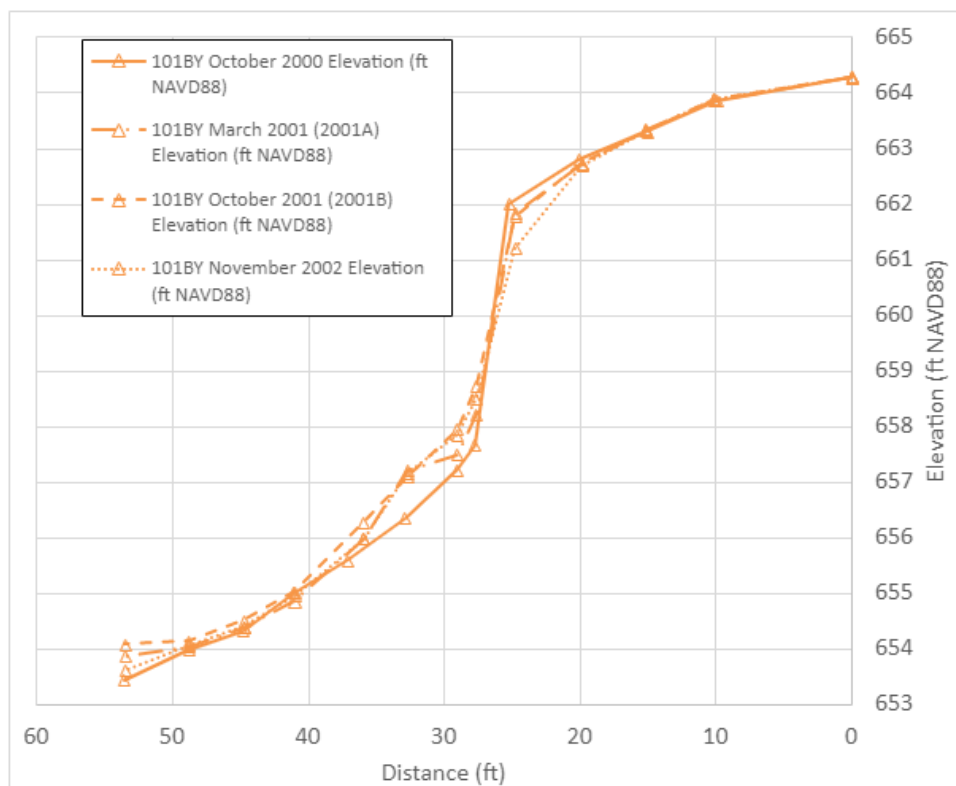
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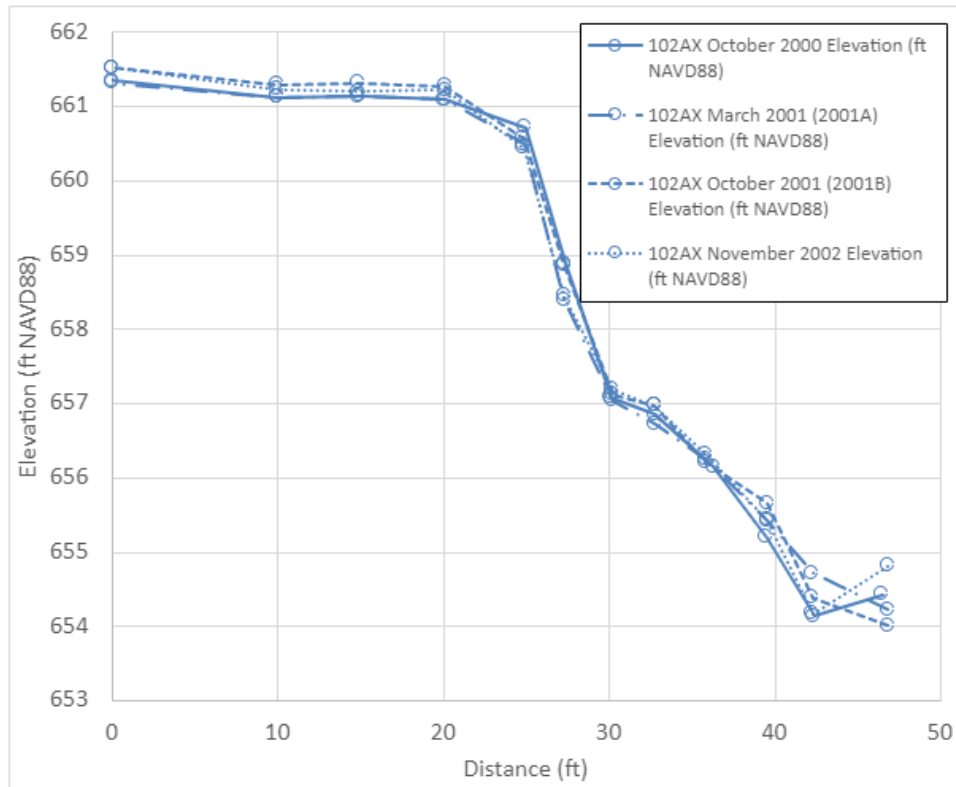
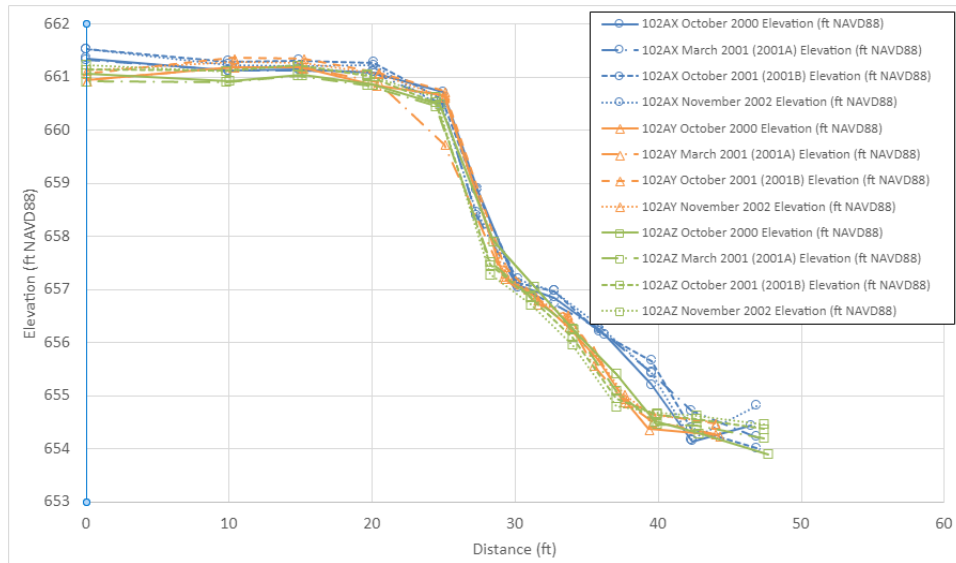


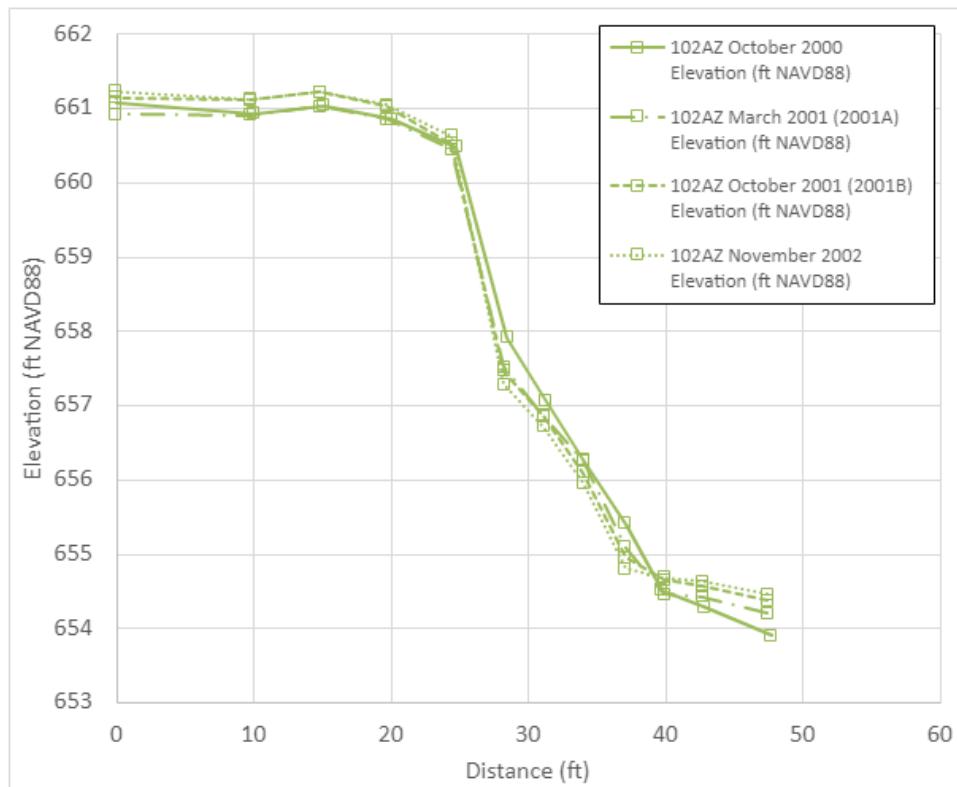
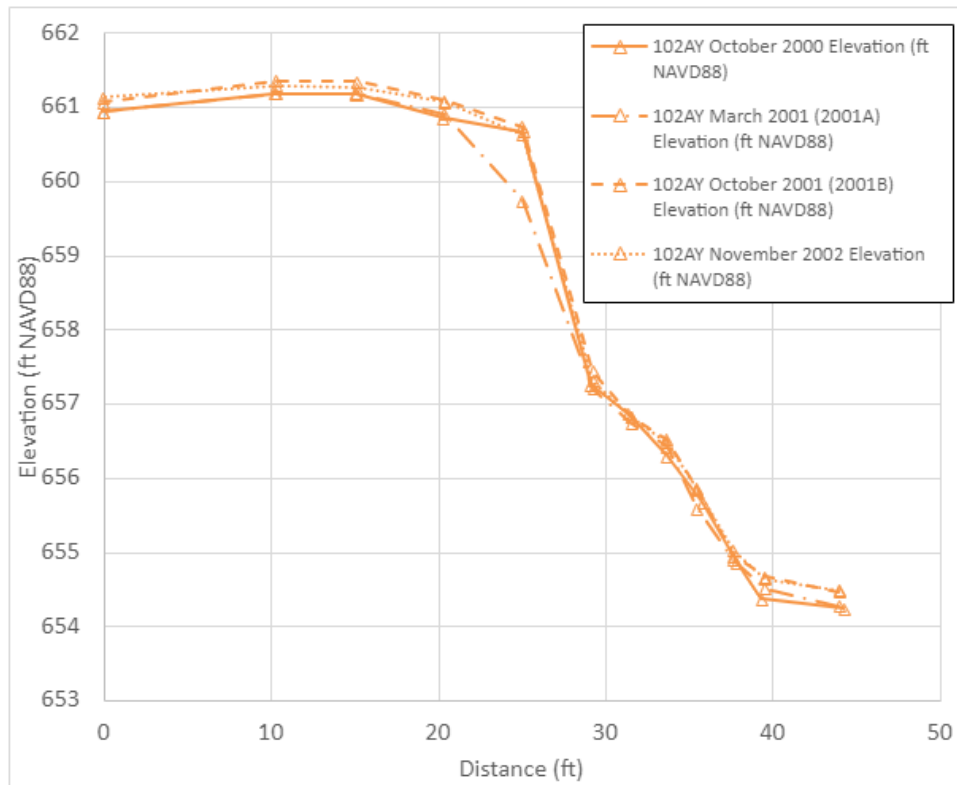
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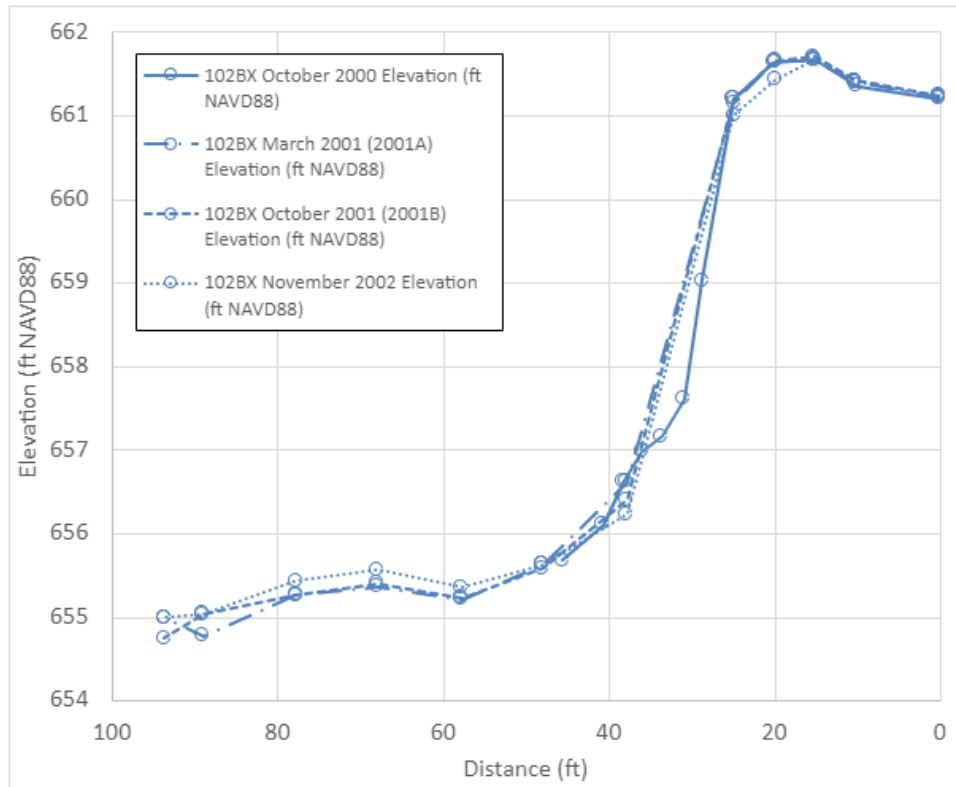
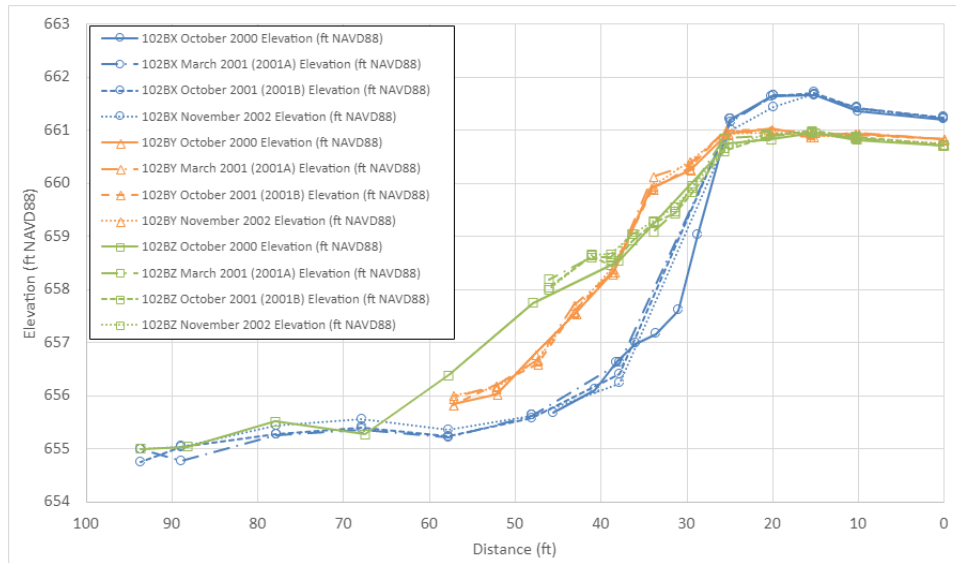


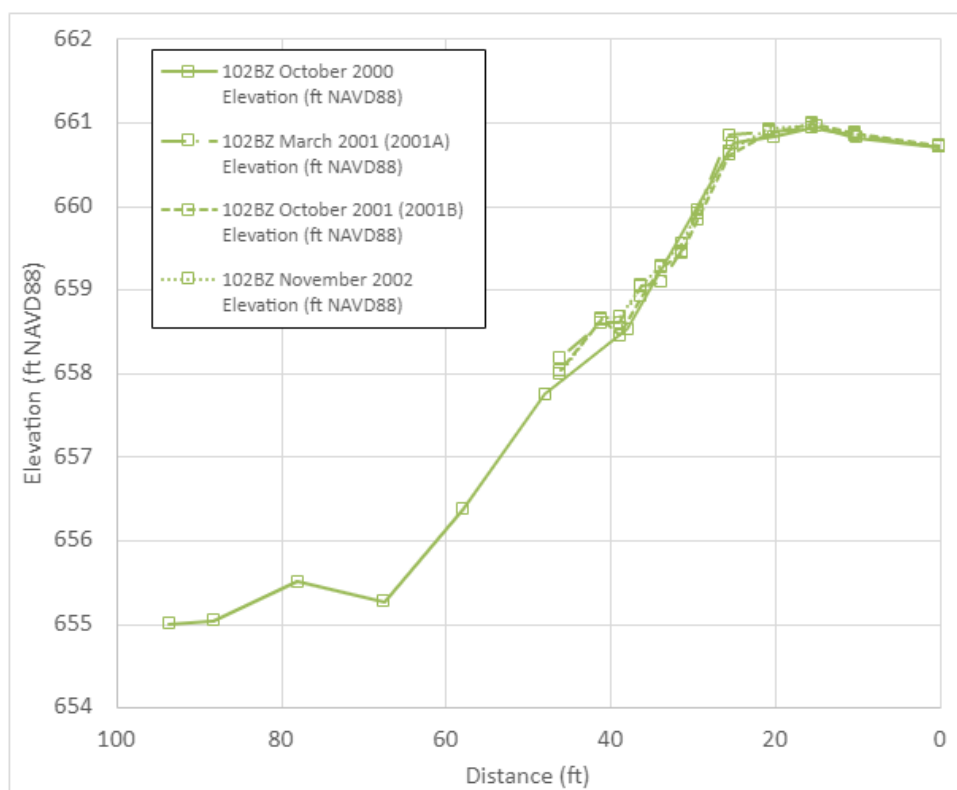
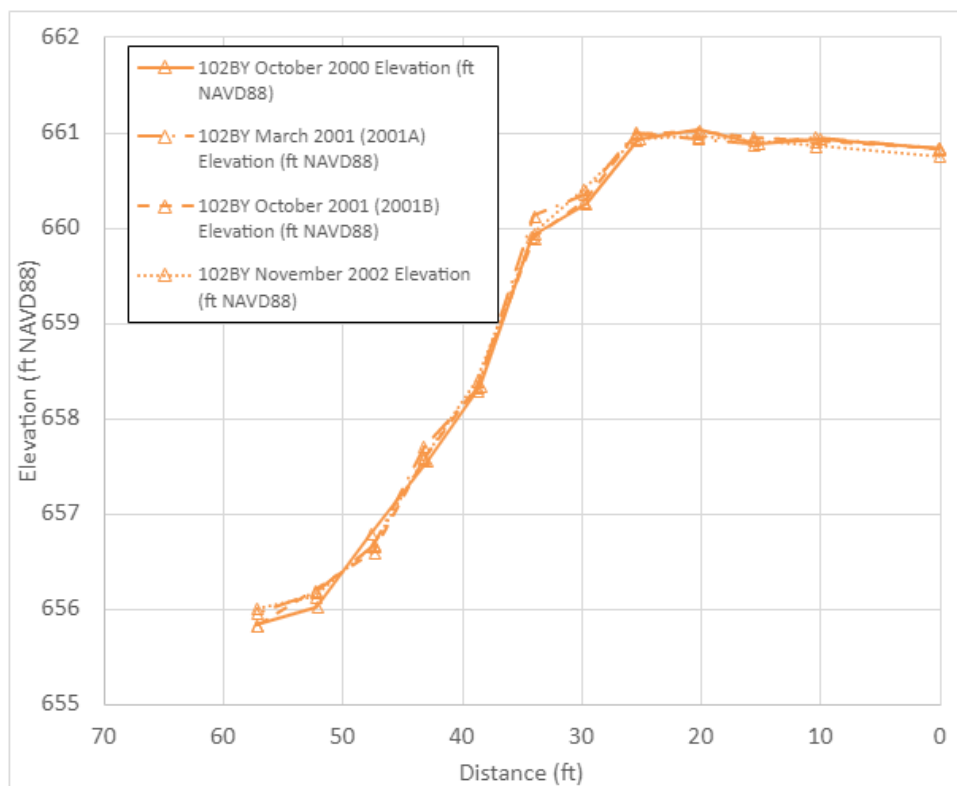
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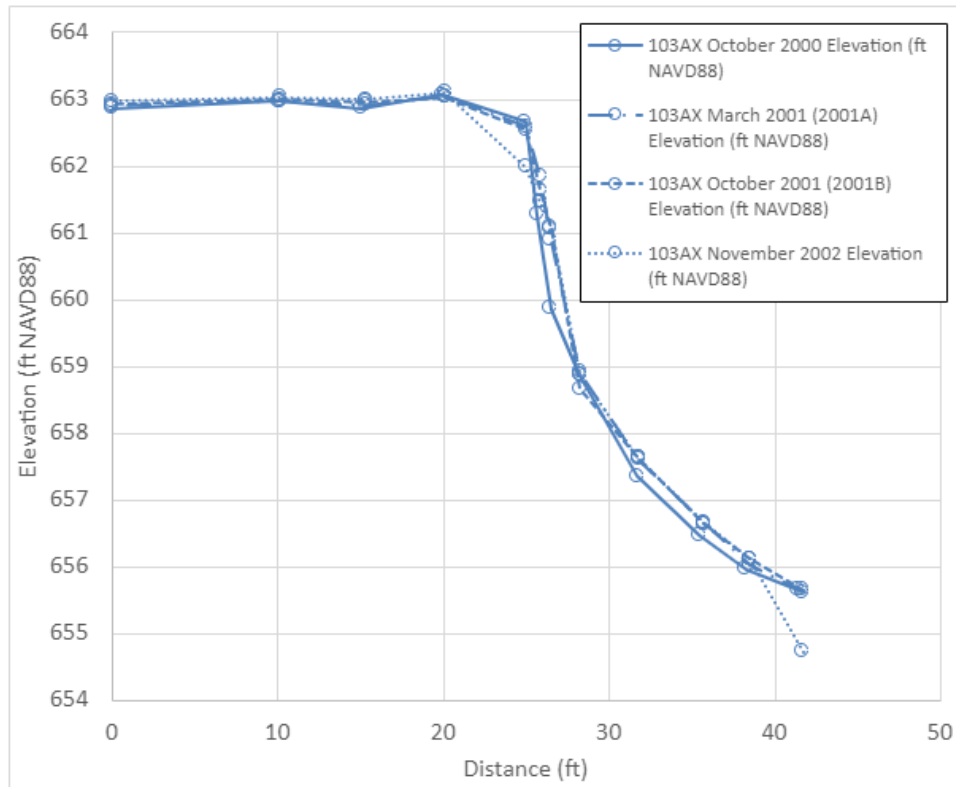
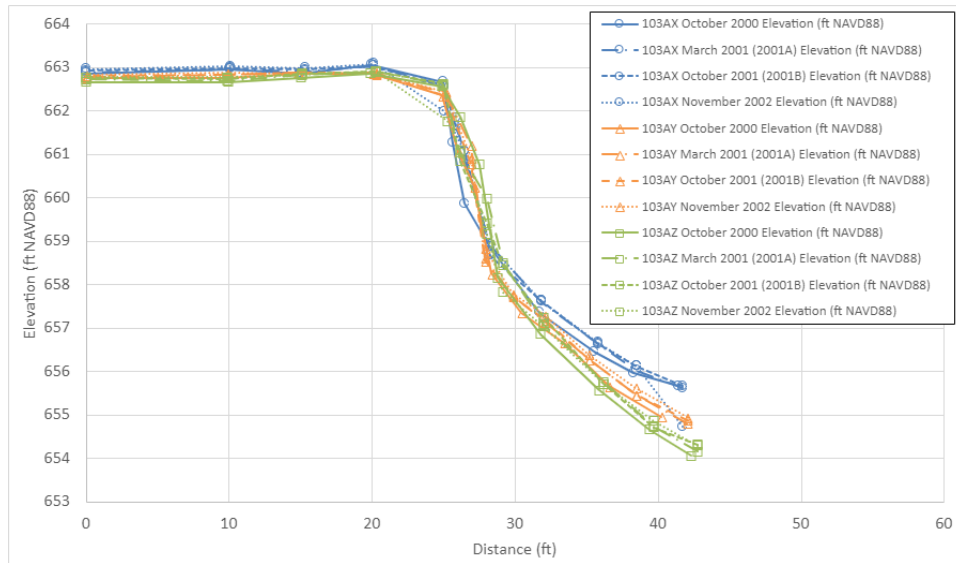


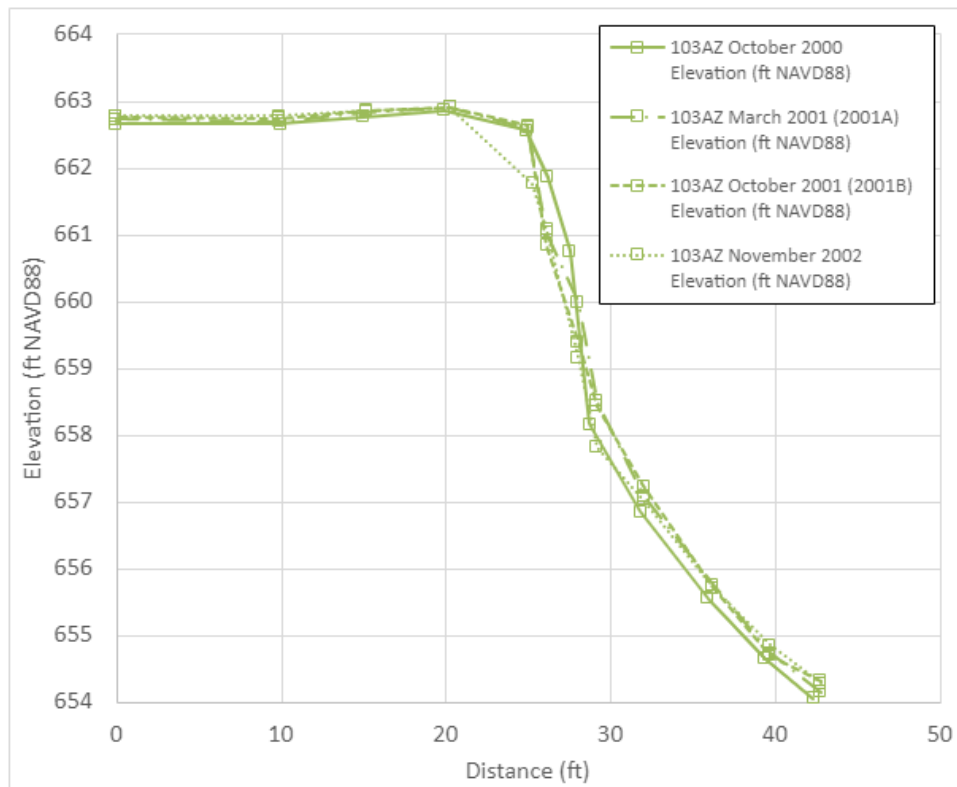
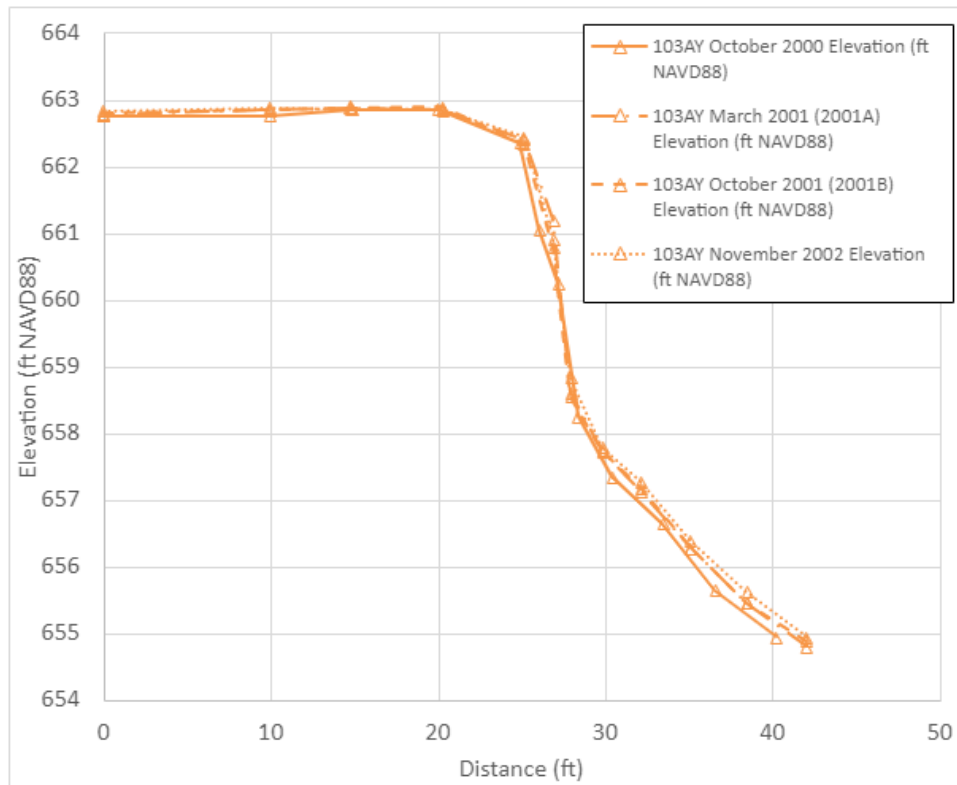
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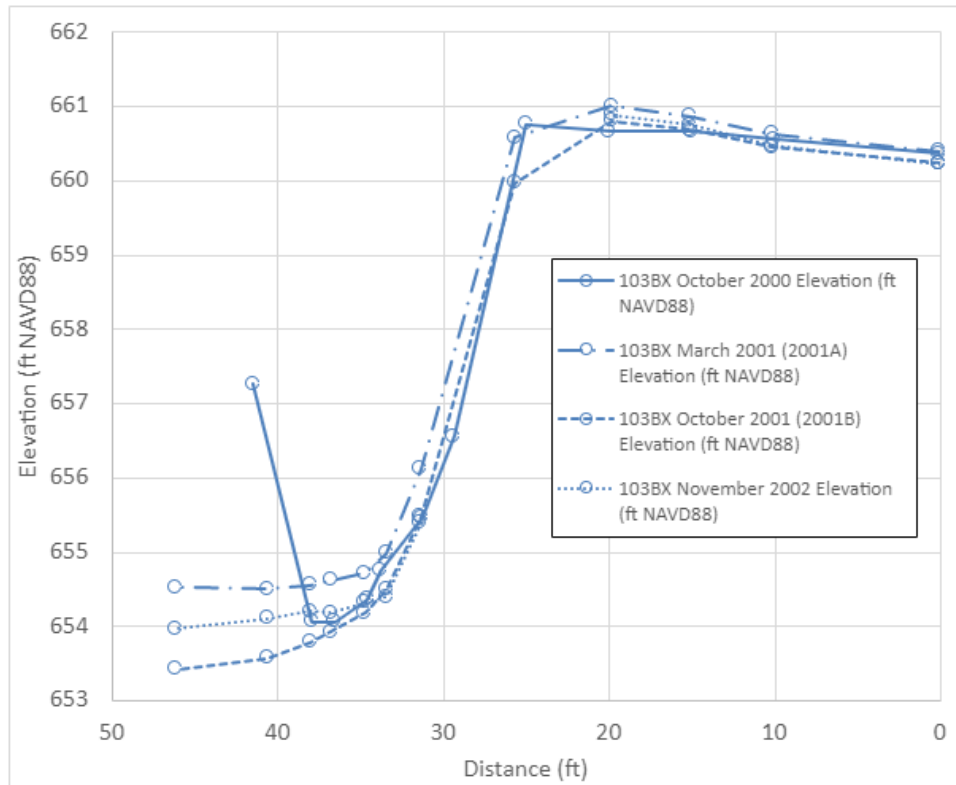
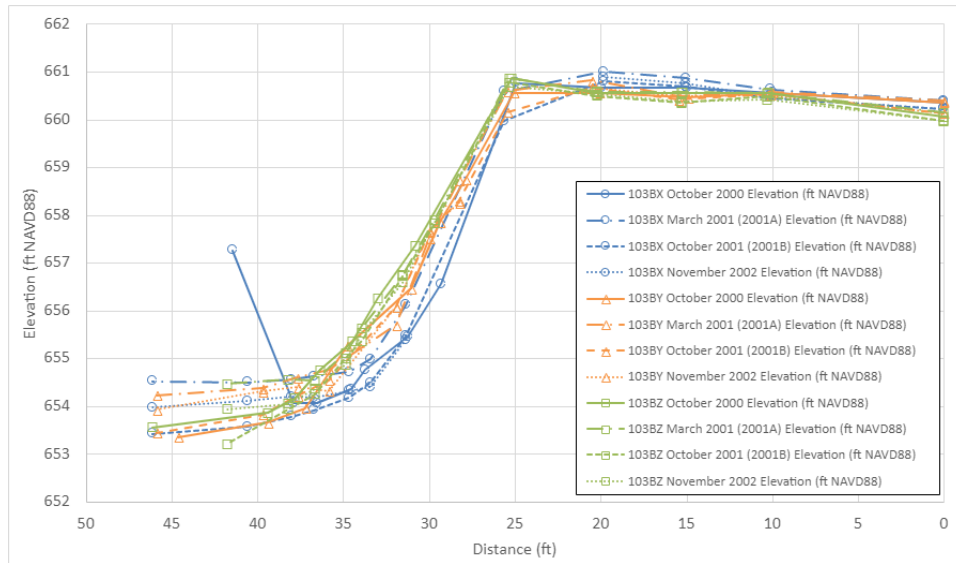


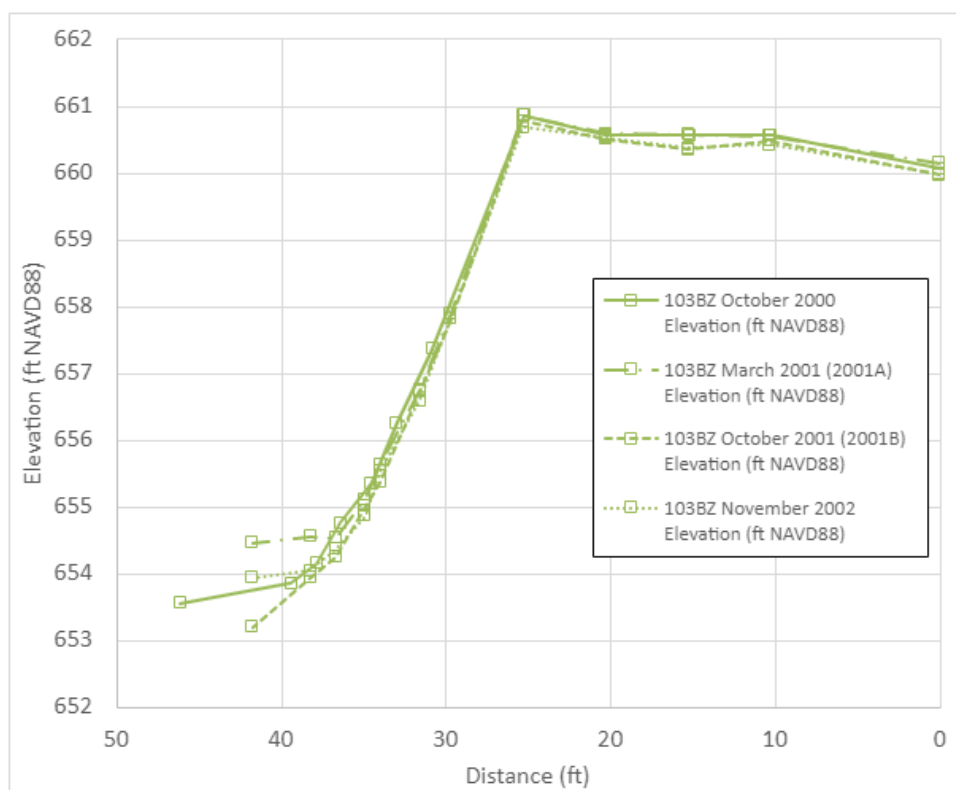
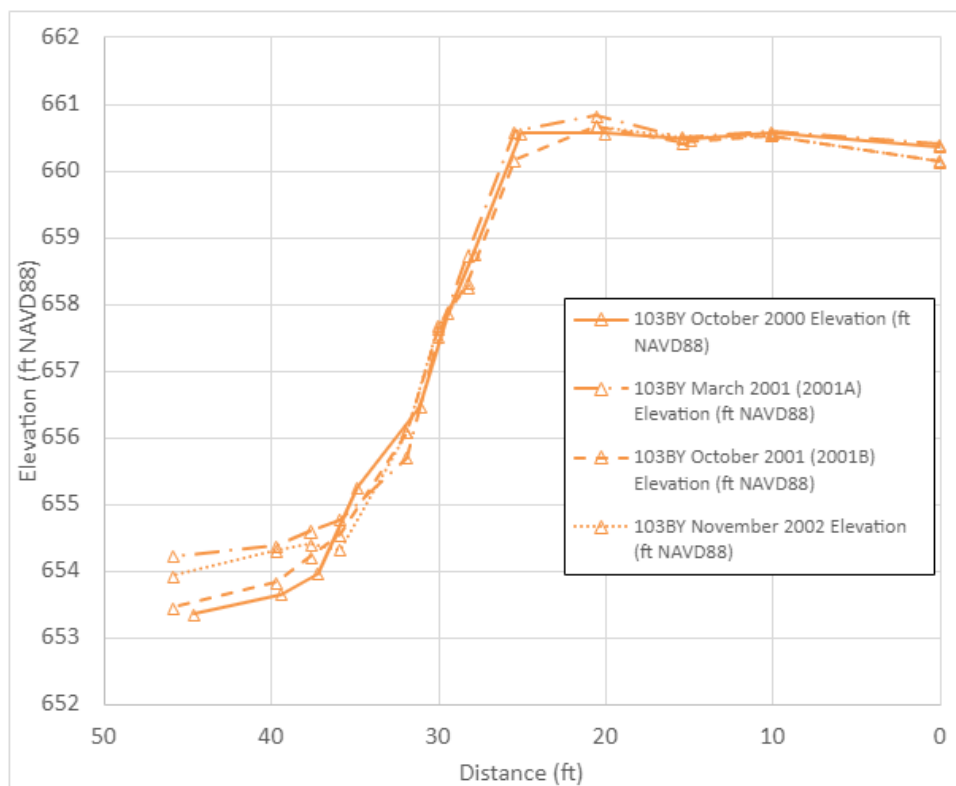
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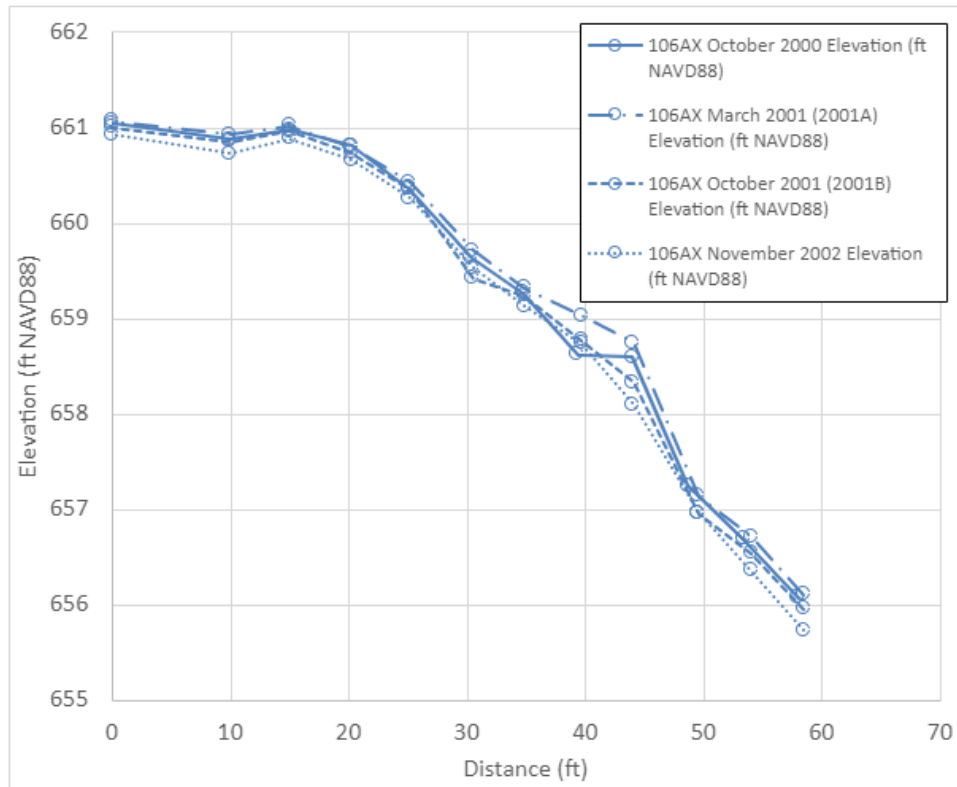
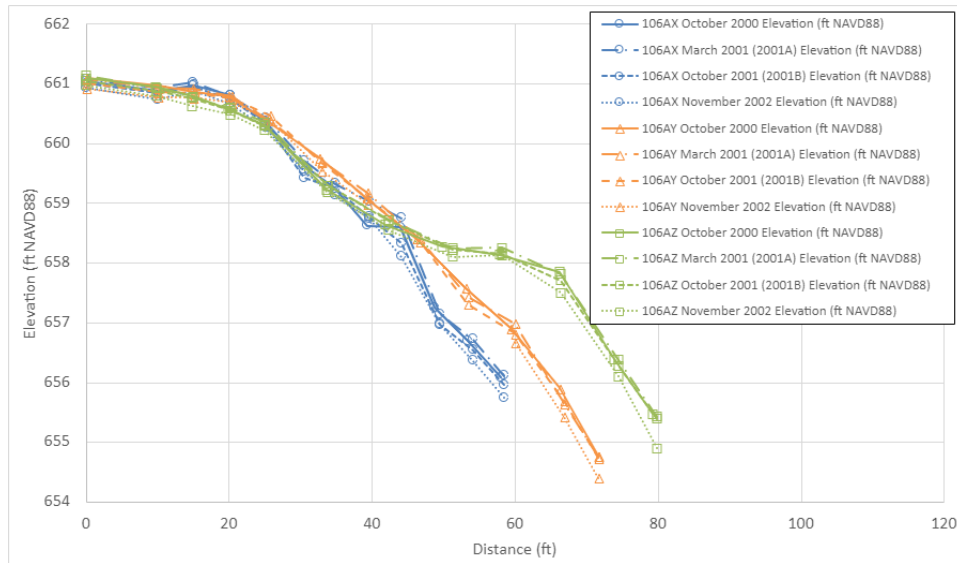


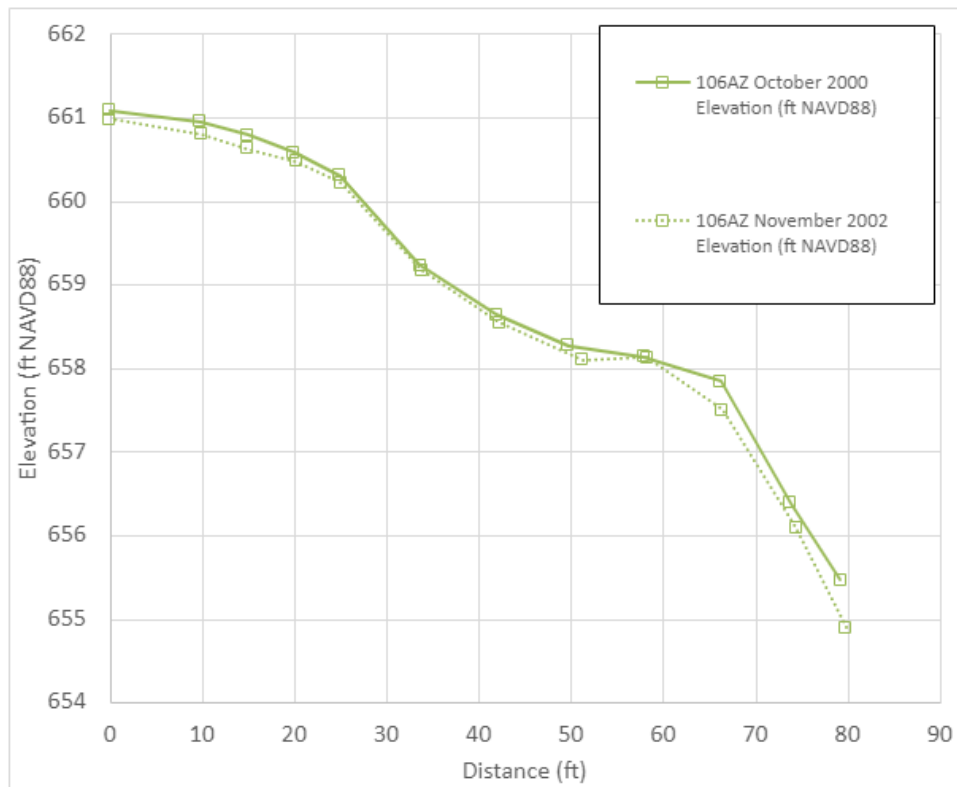
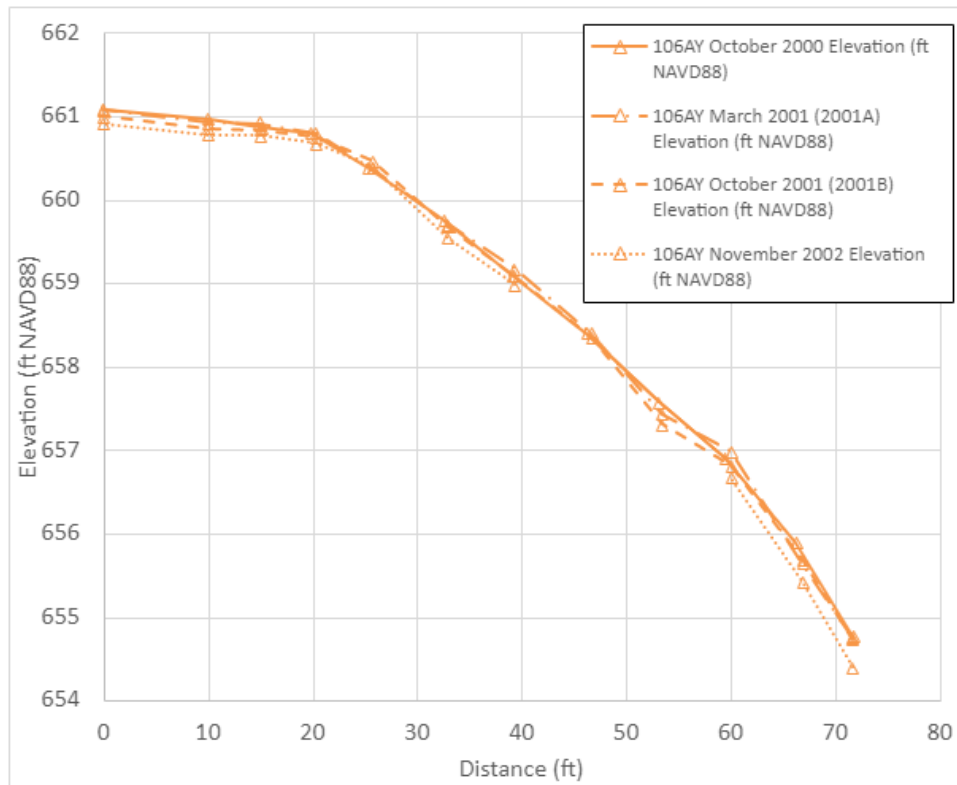
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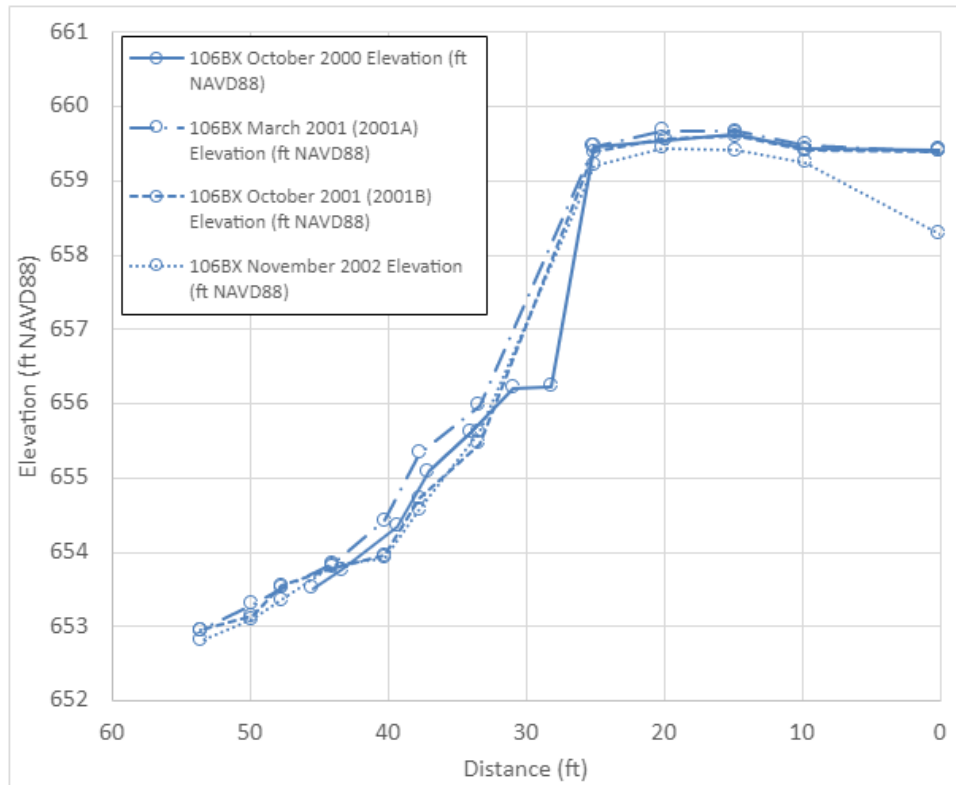
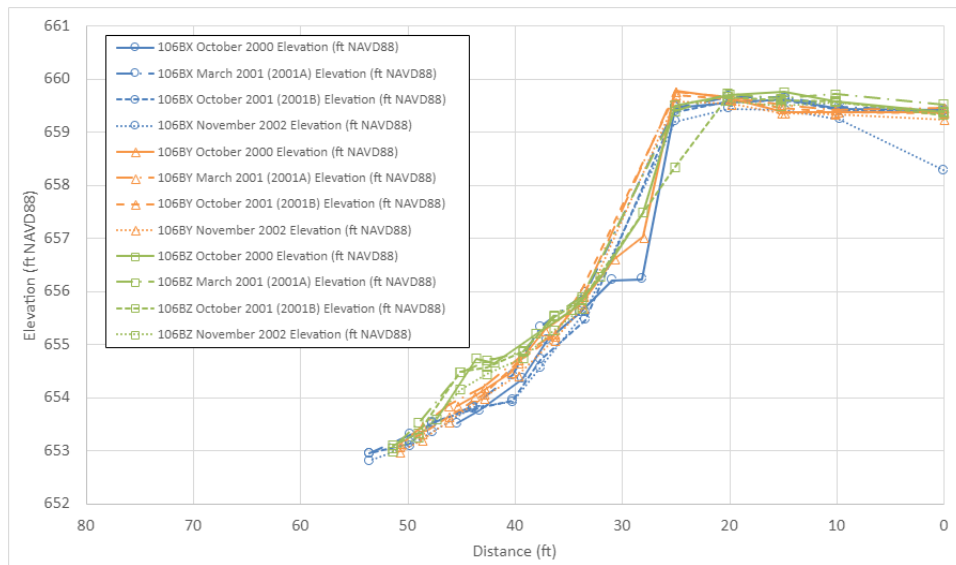


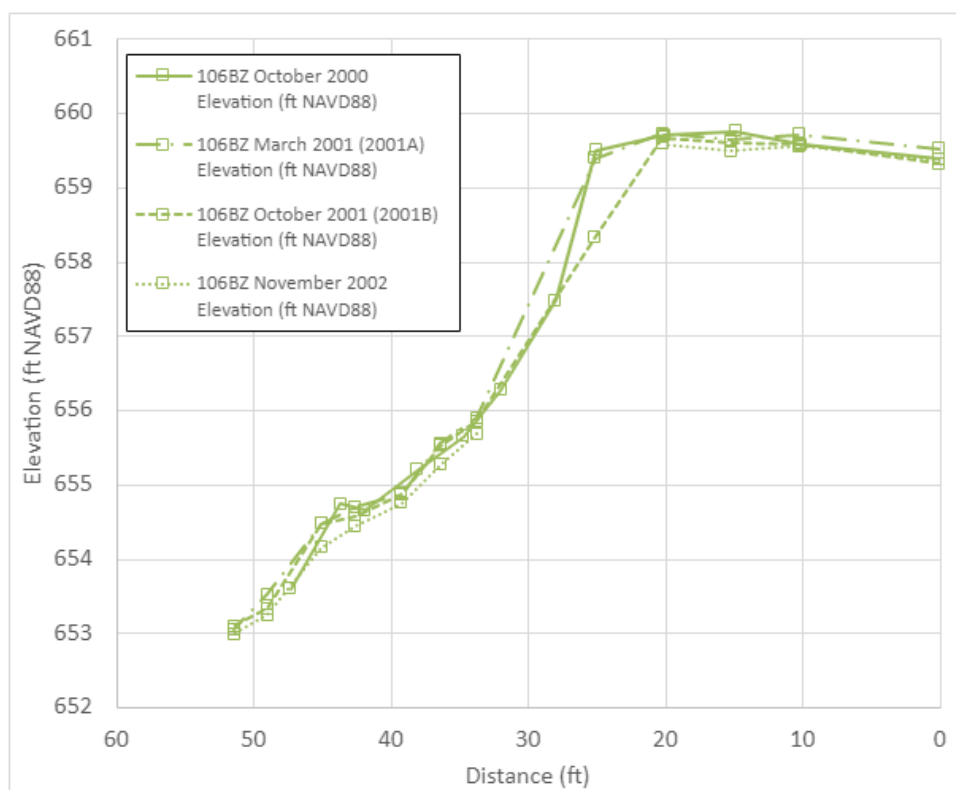
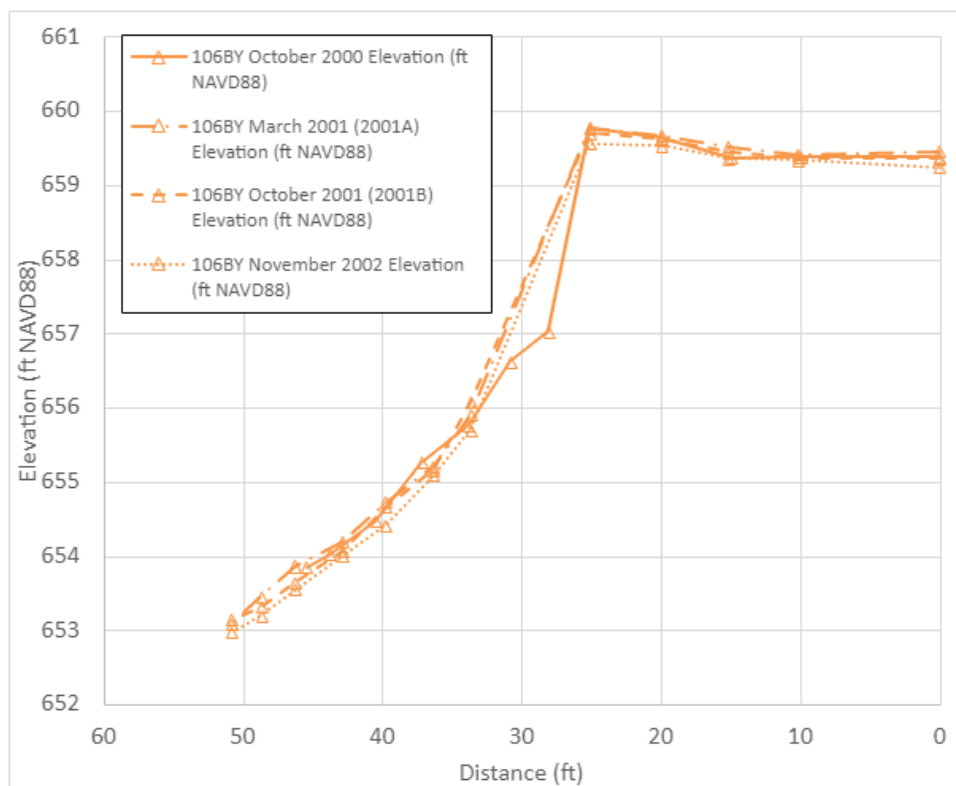
106A





106B





Attachment B: USGS Data

References:

BBL (2003). "Attachment A, Erosion Pin Monitoring Data." Kalamazoo River Study Group Allied Paper Inc./Portage Creek/Kalamazoo River Superfund Site. March 2003.

Rachol C M, Fitzpatrick FA, Rossi T (2005). "Historical and Simulated Changes in Channel Characteristics

Appendix B. Geotechnical parameters from field tests of streambanks for the Kalamazoo river from Plainwell to Otsego, Michigan. Streambank locations within study reach are shown on Figure 11. [*], denotes locations where the average of multiple field tests were presented; Abbreviations: c_a , apparent cohesion; c' , cohesion; ϕ' , internal friction angle; γ , saturated unit weight; τ_c , critical shear stress; k , erodibility; kPa, kilo-Pascals; kN/m³, kilo-Newton per cubic meter; Pa, Pascal; cm³/N-s, cubic centimeter per Newton per second; --, no data]

River mile	Bank Layer properties						Bank Toe Properties	
	Layer from top of bank	Depth of layer from top down to river bottom (m)	c_a (kPa)	c' (kPa)	ϕ' , in degrees	γ (kN/m ³)	τ_c (Pa)	k (cm ³ /N-s)
56.67	1	0-5.0	5.0	.0	24.2	16.0		
56.67	2	5.1-6.6	4.6	4.3	21.9	10.6	12.2	.69
56.29	1	0-1.53	2.2	.8	40.6	9.1		
56.29	2	1.54-4.6	3.9	1.4	27.9	16.0	11.3	.13
56.03	1	0-5.5	1.9	.0	25.7	15.6		
56.03	2	5.5-7.0	9.8	8.2	21.2	18.0	2.0	.07
55.87	1	0-3.0	1.9	.0	25.7	18.0		
55.87	2	3.1-5.1	9.8	8.2	21.2	12.7	9.8	1.71
55.72	1	0-5.6	2.7	.0	27.5	16.4		
55.72	2	5.61-8.6	4.9	4.0	23.8	10.1	18.0	.44
55.62	1	0-4.5	6.2	2.1	23.2	16.7		
55.62	2	4.51-8.5	6.1	4.0	18.7	11.3	7.9	1.14
55.45	1	0-3.7	3.7	1.9	23.5	15.0	2.0	.07
55.3	1	0-2.2	.0	.0	30.0	*15.9		
55.3	2	2.3-5.8	6.7	6.1	20.4	11.6	24.0	1.63
55.2	1	0-1.7	3.7	1.9	23.5	15.7		
55.2	2	1.71-2.7	.0	.0	35.0	*15.9		
55.2	3	2.71-6.2	10.2	10.7	14.6	16.9	72.2	.85
55.1	1	0-4.93	10.2	4.5	16.7	14.9		
55.1	2	4.94-7.5	8.8	7.8	12.5	12.9	4.7	5.79
55.01	1	0-4.1	5.5	3.3	11.2	15.2		
55.01	2	4.2-6.6	16.0	13.7	26.6	12.5	7.8	1.24
54.68-L	1	0-0.42	.4	.1	29.1	12.9		
54.68-L	2	0.43-2.2	.0	.0	30.0	*15.9	--	--
54.68-R	1	0-0.50	2.2	2.1	31.4	12.7		
54.68-R	2	0.51-1.20	6.8	5.7	35.0	11.5	2.0	.07
54.61-L	1	0-1.9	8.9	7.6	19.8	15.0	--	--
54.61-R	1	0-0.82	2.3	2.1	10.9	13.8	.23	.20
54.54	1	0-2.6	.3	.0	19.1	12.5	.22	.21
54.46	1	0-2.4	5.2	4.4	24.0	12.5	2.0	.07
54.12	1	0-3.9	6.3	6.2	16.9	9.7	2.0	.07
53.73	1	0-1.7	3.6	3.2	19.4	16.2	2.0	.07
53.19	1	0-4.0	2.3	.0	10.9	7.4	2.6	5.27
52.59	1	0-5.2	3.7	3.4	29.3	14.8	.2	.22
52.25	1	0-4.1	.2	.0	11.2	15.8	--	--
51.83	1	0-9.8	3.6	1.6	21.3	17.2	1.1	.10
51.43	1	0-5.5	3.6	1.6	21.3	17.2		
51.43	2	5.6-8.3	4.7	.0	30.0	9.3	4.4	.47
51.23	1	0-11.58	0.0	0.0	29.7	14.6	--	--