GLMRIS

Great Lakes and Mississippi River Interbasin Study

Aquatic Nuisance Species | Ecosystems | Navigation | Recreation | Flood Risk Management | Water Use

Aquatic Nuisance Species Controls Report

Wabash-Maumee Basin Connection

Fort Wayne, Indiana

APPENDIX F
ENGINEERING

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AQUATIC NUISANCE SPECIES CONTROLS REPORT
WABASH-MAUMEE BASIN CONNECTION STUDY
FORT WAYNE, INDIANA

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SECTION 1
GENERAL

1.1. INTRODUCTION

Based on the U.S. Army Corps of Engineers document Great Lakes and Mississippi River Interbasin Study (GLMRIS), Other Pathways Preliminary Risk Characterization, dated 9 November 2010, thirty-one surface water pathways were identified outside of the Chicago Area Waterway System (CAWS) for which risk characterizations were deemed necessary to evaluate the risk of aquatic nuisance species (ANS) crossing the drainage divide between the Great Lakes and the Mississippi River basins. In order to carry out the risk characterization, teams of experts in hydrology and invasive species were formed. The teams included members of the United States Geological Survey (USGS), United States Fish and Wildlife Service (USFWS), Natural Resources Conservation Service (NRCS), National Oceanic and Atmospheric Administration (NOAA), the Departments for Natural Resources from Minnesota, Wisconsin, Indiana, Ohio, the New York Department of Environmental Conservation, and the Great Lakes Fishery Commission.

Each ANS expert assigned hydrologic risk ratings (Ideal, Very Favorable, Favorable, Possible, Unlikely, or Highly Unlikely) to each of the thirty-one water surface pathways for ANS movement in both directions (into the Great Lakes and out of the Great Lakes.) The Highly Unlikely rating was defined as insignificant risk and was a basis for no further consideration. One site was singled out as the greatest concern, the Eagle Marsh site in Fort Wayne, Indiana. A 2009 Flood Insurance Study indicated that significant backflow from the St. Marys and St. Joseph Rivers at the confluence with the Maumee River occurs through Junk Ditch. The USGS representatives that studied the Eagle Marsh pathway indicated that a 10% frequency storm was likely to provide a water column depth across the drainage divide sufficient for large fish, such as the Asian carp, to traverse.

Eagle Marsh, a 716 acre wetland preserve located on the southwest border of Fort Wayne, Indiana, straddles a natural geographic divide created by the last glacial movement approximately 10,000 years ago. The site is operated by the private Little River Wetlands Project and is co-owned by the Indiana Department of Natural Resources (DNR). The broad wetland marsh extends across the divide and is flanked by two drainage ditches, the Graham-McCulloch Ditch and Junk Ditch. The Graham-McCulloch Ditch drains west into the Little River and eventually the Wabash River near Huntington, while Junk Ditch drains northeast into the St. Marys River and then the Maumee River. Under normal conditions, there is no direct hydraulic link between the Wabash River and the Maumee River. However, the tributaries and drainage ditches near Eagle Marsh provide a potential hydraulic connection under certain flooding situations.

Photo 1.1: Eagle Marsh in Fort Wayne

Asian carp, one of the ANS, have been present in the Wabash River for over fifteen years. Indiana DNR biologists first discovered them in 1996 during a survey at Hovey Lake Fish & Wildlife Area at the southwest tip of Indiana. Since then, Asian carp have been detected in spot locations on the Wabash River as far upstream as the tailwater area below J. Edward Roush Lake in Huntington County, which is downstream of Eagle Marsh. The 91 foot high concrete and earth dam at J. Edward Roush Lake is an impassible barrier to further upstream movement through the Wabash River. However, where the Little River enters the Wabash River downstream of the Roush Lake Dam there are no significant barriers in place to prevent fish movement or other ANS movement through the Little River toward the basin divide at Eagle Marsh.

Asian carp are currently known to exist near the mouth of the Little River, approximately 20 miles southwest from Eagle Marsh. During flooding conditions, there is concern these and other ANS can move upstream through the Little River and the Graham-McCulloch Ditch and cross the divide to the Maumee River Basin, giving them direct access to Lake Erie.

With assistance from the Indiana Department of Natural Resources fisheries biologists, 247 water samples were collected in October 2010 in ditches and streams in the vicinity of Eagle Marsh. The largest sampling to date of Indiana waterways for environmental DNA (eDNA) evidence of Asian carp yielded negative results on either side of Eagle Marsh. Supplemental sampling in the summer of 2011 also yielded negative results.

1.2. OBJECTIVES

The main objectives of Appendix F are to:

- Identify structural barriers which provide a single point of defense to a multi-tiered defense to prevent or reduce the possibilities of the movement of ANS. Work is to be at a screening level of design effort;
- Conceptually illustrate the structural barrier alternatives on plans;
- Identify potential hurdles;
- Explain why other structural alternatives were eliminated from further study; and
- Provide open communication with local stakeholders for their contribution.

1.3. EXISTING TEMPORARY BARRIER

As an immediate preventative measure, in October 2010, the Indiana DNR constructed a temporary 1,177 foot long, eight foot high chain link fence and a supplemental 494 foot debris catch fence at Eagle Marsh. The chain link fence is bolstered by approximately 120 concrete New Jersey type barriers weighing 2.5 tons each. For stability purposes and to prevent undermining, the fence fabric extends 24 inches below grade and is covered with a compacted granular fill. The fence posts were filled with concrete for stability.

Photo 1.2: Indiana DNR fence construction

Photo 1.3: Indiana DNR fence at north abutment

Rip rap stone was placed at the north abutment of the fence at the Graham-McCulloch Ditch earthen embankment and at the south abutment at the Norfolk Southern Railroad embankment. The fence was constructed in accordance with the Wetland Reserve Program Compatible Use Authorization which expires on 24 August 2014. At that time, either the fence will need to be removed or the agreement extended.

1.4. VALUE ENGINEERING STUDY

Value Engineering (VE) is a structured facilitated process that identifies the key functions that must be provided to assure project success and to verify optimum project value. As per ER 11-1-321, change 1 “Army Programs, Value Engineering” U.S. Army Corps of Engineers, dated 01 January 2011, a VE study for this project was conducted 19-21 January 2011 in Louisville, Kentucky and was facilitated by Strategic Value Solutions, Inc.

A multi-disciplinary team comprised primarily of the Project Delivery Team (PDT) from the Army Corps of Engineers and personnel from the Indiana DNR and NRCS attended the three day integrated value based design VE workshop. One of the goals of the workshop was to confirm a full understanding of the project scope by allowing the multi-disciplinary team to participate as a group in an open dialogue to assure open project communication. The second goal was to utilize the team collaboration to brainstorm as many solutions as possible and formulate measures that warranted further investigation with the objective of finding the best solution to meet the project purpose and need. Emphasis was placed on preserving the important ecological and aesthetic significance of the Fort Wayne, Indiana area.

During the creativity phase of the workshop, over 100 measures were generated regardless of feasibility. During the evaluation phase, the VE team selected measures with the most merit for further development. The criteria used for selection were inherent value, benefit, acceptance of the measure, life cycle cost savings, and technical appropriateness of the measure. The measures were narrowed down to twelve and were expanded during the development phase, which consisted of preparing a description of the measure, evaluating advantages and disadvantages, preparing a brief narrative, and making cost comparisons.

The presentation phase primarily consisted of the final written report, however the charrette participants wrapped up the workshop with an informal discussion of the developed ideas and the next steps for the team. The project manager utilized the information learned during the workshop to prepare a briefing to Corps personnel and key stakeholders on the project. Of the twelve measures developed at the workshop, seven were developed into alternatives and are included in this report for further development below. The other five measures were eliminated for various reasons as explained in Section 1.6, Eliminated Structural Measures. Two new alternatives, Alternatives H and I, were generated by the PDT after the VE study concluded and additional survey data was obtained of the area. All nine alternatives are explained in Section 1.5, Structural Alternatives, below.

A copy of the Value Engineering Study dated February 2011 is included as supporting documentation only. Reference Appendix D.

1.5. STRUCTURAL ALTERNATIVES

1.5.1. Construct an I-Wall, Alternative A, (Eagle Marsh, Basin Divide)

The drainage basin divide for the Maumee River basin and the Wabash River basin is located northeast of Eagle Marsh and Engle Road, runs in a north-south direction and is identified and located on the map below. In order to prevent any movement of ANS from one basin to another, Alternative A proposes construction of a permanent structure at the approximate location of the hydraulic basin divide. Reference is made to Alternative A Site Plan CS101 in Appendix G.

Two types of structural barriers were considered: an earthen levee and an I-wall. The advantages and dis-advantages of each barrier type were discussed. The footprint of an earthen levee would be larger than that of an I- wall; therefore, real estate acquisition for construction of an I-wall would be less and would consequently reduce overall project costs.

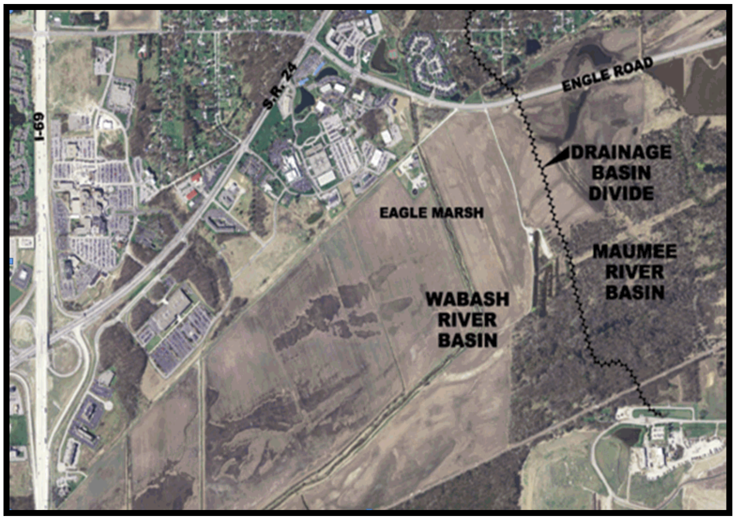


Photo 1.4: Location of the drainage basin divide

The larger footprint of an earthen levee would also impose a greater impact to the Eagle Marsh wetland preserve. In addition, an earthen levee would require importing material from a nearby borrow site which would need to be identified. Transportation costs for material hauling from the borrow site would be high. An earthen levee would also need routine mowing which will increase the long-term operation and maintenance costs of this project. After consideration of these factors, an I-wall is proposed. In an effort to preserve the character of Eagle Marsh and public acceptance of this project, it is also proposed that an environmentally themed concrete form liner be used on the I-wall.

This alternative provides a hydraulic separation between basins and will raise floodwater crest elevations for Junk Ditch, which will affect structures within the area. The I-wall will traverse along the hydraulic divide at an elevation varying from 760 to 762 feet to provide protection from over topping for a 1% annual chance event. Using a low berm with an elevation of 762 feet on the south side of Engle Road, the berm will tie the I-wall into the left abutment of the Engle Road Bridge.

Minimal operation and maintenance will be required for this alternative.

The baseline cost estimate for Alternative A is approximately $14 million.

1.5.2. Construct a Fence and Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative B, (Eagle Marsh, Basin Divide)

Construct a permanent fence east of the location of the existing Indiana DNR fence located in Eagle Marsh. The fence will run in a north-south direction and will include rip rap stone abutments at the Graham-McCulloch Ditch berm and at the Norfolk Southern Railroad embankment. Reference is made to Alternative B Site Plan CS102 in Appendix G. Based upon visual inspection during a site reconnaissance trip in November 2010, it appears the Graham-McCulloch Ditch berm is eroding in several areas and is in poor condition. A portion of the berm would serve as part of the barrier, and thus should be reconstructed in accordance with appropriate USACE standards to protect against failure

Photo 1.5: Left descending bank of Graham-McCulloch Ditch shown on left

during a high water event. From the tie in point with the fence, the berm along the left descending (east) bank of the Graham-McCulloch Ditch is to be demolished and reconstructed to an elevation to prevent overtopping by the 1% annual chance event and will tie into high ground at the wastewater treatment plant. The earthen berm will be rebuilt up to an elevation of 762 along the south side of the wasterwater treatment plant access road to Engle Road, and along Engle Road to the left (looking downstream) abutment for the Engle Road bridge over the Graham-McCulloch Ditch. Most of the existing material in the left berm will be available for reuse. However, additional borrow material will be needed to construct a reliable cross section. Multiple permits and regulations will be required to acquire a borrow site. A sacrificial drift/ice fence will be located on the east side of the primary barrier to prevent or reduce possible damage or clogging of the primary fence.

An option of this alternative would be to use the temporary barrier fence constructed by Indiana DNR, and rebuild the additional section of berm down to the location of the existing fence. Assuming the existing fence is constructed to adequate hydrologic and structural requirements, it likely has a sufficient service life left to be considered for this alternative.

This alternative will require routine maintenance and inspection after each rain event to ensure the fence is not damaged and to remove debris. If damage does occur, the fence will need to be repaired or replaced.

The baseline cost estimate for Alternative B is approximately $3.2 million.

1.5.3. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

For this alternative, a pump station and earthen berm is proposed downstream of the basin divide on the Graham-McCulloch Ditch just downstream of Homestead Road. Reference is made to Alternative C Site Plan CS103 in Appendix G. This alternative consists of constructing a large pump station to control all flow of the Graham-McCulloch. The pump station would pump the flows through the earthen berm, which creates a complete cutoff of the existing Graham-McCulloch flow. The berm would tie into the Norfolk Southern Railroad on the southern end and into Homestead Road on the northern end.

Material for the berm would come from existing loam soils on site excavated for the construction of the pump station. The berm would be compacted to 95% maximum density per ASTM D 698, protected by riprap on the upstream side, and constructed to an elevation exceeding the 1% annual chance event. An overflow spillway section may be constructed within the portion of the berm through the original channel. This spillway would account for extreme flows and keep water levels from overtopping Homestead Road, the railroad, or from causing damage to personal property.

The pump station itself will contain a total of eleven electric pumps. Three pumps will be 150 horsepower, and the other eight 350 horsepower. The intake structure for the pump station is approximately 101 feet by 47 feet. Eight of the pumps will have a 48 inch discharge pipe and the remaining three pumps will have a 30 inch discharge pipe. The discharge pipes will outlet into a 52 foot by 15 foot discharge well. The discharge well will outlet by two 10 feet wide by 6 feet tall box culverts that drain into Graham-McCulloch Ditch, downstream of the pump station and berm. Pumps will be electronically controlled by a float system or other electronic water level indicator software.

Each of the eleven discharge pipes will contain a flap gate within the discharge well. This will be the first obstacle for ANS travelling upstream. The second obstacle will be the pump impeller. ANS travelling downstream that are small enough could potentially pass through the pump impellers and be released downstream.

The pump station intake structure and discharge well will be massive concrete structures capable of supporting the heavy loads and turbulent forces of the pumps and flowing water. Deep foundations are anticipated to support the pump station and potentially the discharge well.

During normal to low flow conditions, it is anticipated one or two of the smaller pumps will need to run to continuously pass flow on the Graham-McCulloch Ditch. Electricity costs for this alternative will be high. The pump station will require regular maintenance and inspection to ensure systems are operating as intended.

The baseline cost estimate for Alternative C is approximately $25 million.

1.5.4. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

This alternative consists of constructing a permeable berm that would pass flow during a potential connection high water condition. Reference is made to Alternative D Site Plan CS105 in Appendix G. A permeable berm is an embankment made up of opened graded rip rap surrounding a perforated pipe system that will capture the water as it passes through the stone. The berm, proposed just upstream of Amber Road, will run north-northeast adjacent to the road and tie into high ground approximately 200 yards east of Amber Road. The southern end of the permeable berm would tie into the embankment of the Norfolk Southern Railroad. The system will drain south to the Graham-McCulloch Ditch and the pipe system with collector channels will empty into the existing channel downstream of the berm.

The berm will contain sluice gates with automated closure mechanisms that will close the gates to a nominal opening height of 3 inches when gages on the Graham-McCulloch Ditch, Junk Ditch and/or within Eagle Marsh near the drainage divide indicate that flow conditions are imminent that could support transfer of ANS. During normal low flow conditions the gates would be open to allow normal drainage of the Graham-McCulloch watershed. Gages at the sluice gate would allow the gate to reopen once adequate head differential across the berm was developed such that velocities through the sluice gate would be unsuitable to support ANS transfer in the upstream direction (toward the Great Lakes watershed). These gages at the structure would also trigger closure of the gate when other scenarios such as backwater flooding or headwater flooding of the Graham-McCulloch Ditch might support movement of ANS upstream. When the velocity of flow through the gates decreases below threshold values preventing ANS transfer, the gates will close and ponded water will be released by infiltration through the permeable berm.

The berm would be designed so that the entire length would be equally submerged during a high water condition. This would allow the berm to pass maximum flows. Water levels are expected to rise on the berm during an event, and would inundate a large area upstream of the berm. Debris and sediment buildup onto and within the berm is a concern for this alternative. A vegetated filter strip should be planted upstream from the permeable berm to filter debris and sediment. It is important to the performance of the system that debris and sediment do not collect on or in the stone matrix of the berm as this could reduce flows through the system. This system would require periodic maintenance and testing of automated systems to assure functionality of the system.

The baseline cost estimate for Alternative D is approximately $7.8 million.

1.5.5. Construct a Fence/Earthen Berm Combination, Alternative E, (Eagle Marsh, Basin Divide)

An earthen berm will be constructed to an elevation exceeding a 4% annual chance event on the Graham-McCulloch Ditch, with a fence on the crown of the berm constructed to an elevation equal to the 1% annual chance event on either watershed. A 4% annual chance storm on the Graham-McCulloch Ditch was selected based on professional judgment to reduce high frequency flooding without increasing water surface elevations on Junk Ditch. This corresponds to approximately a 99% annual chance event on the St. Marys River. Reference is made to Alternative E Site Plan CS107 in Appendix G.

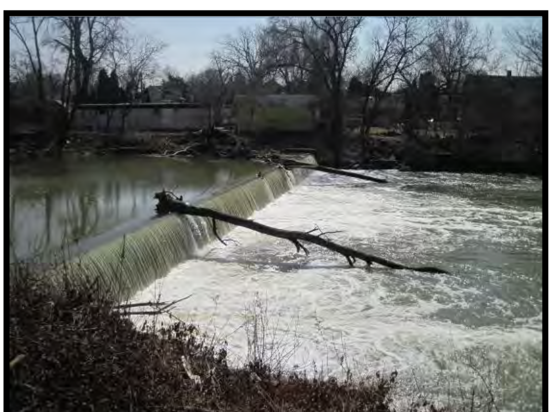
This alternative will serve as a permanent alternative to the temporary fence constructed by Indiana DNR in 2010. This alternative reduces the frequency that flood flows will allow transfer of ANS species but permits flow across the divide during less frequent events. The fence component of the barrier would prevent movement of adult swimming species of ANS. The design of the fence is generally based upon the Indiana DNR fence concept.

The southern portion of the fence/berm alignment follows the general location of the basin divide. At the intersection of the Eagle Marsh access road and the fence/berm alignment, the fence and the berm separate. It is proposed the Eagle Marsh access road would be located on the crown of the berm. The south abutment of the fence/berm combination ties into the Norfolk Southern Railroad embankment. This alternative will require routine maintenance and inspection after each rain event to ensure the fence is not damaged. If damage does occur, the fence will need to be repaired or replaced.

The conceptual cost estimate for Alternative E is approximately $4.2 million.

1.5.6. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

Huntington Dam is a concrete, fixed crest weir located on the Little River (sometimes also referred to as the Little Wabash River) in Huntington, Indiana, at roughly stream mile 2.4, about 20 miles downstream of the drainage divide. The crest of the dam is at approximately elevation 720.6 (NAVD88), nominally providing a six foot head differential at low water. Reference is made to Alternative F Site Plan CS108 in Appendix G.



The weir component of the existing Huntington Dam would provide a hydraulic drop over the structure which would hinder the leaping ability of some of the ANS, such as the Asian carp, during low flow conditions. In conjunction with the vertical drop, a bar screen and supporting superstructure is proposed to be constructed across the Little River.

The bar screen would be angled to allow debris and ice to pass over the top of the screen during high flows and reduce the possibility of blockage of the screen.

Top Photo 1.6: Huntington Dam is estimated to be 150 feet wide
Bottom Photo 1.7: An example of a heavy-duty debris boom

To reduce debris or ice from collecting or damaging the bar screen barrier, a heavy-duty floating boom made for controlling and diverting heavier items, such as logs, would be placed upstream of the existing weir. The floating boom would be placed at a 45 degree angle across Little River and would divert debris to the right descending bank where it would accumulate until removed. Access to Little River can be made easily from East State Street for debris removal.

This alternative will require routine maintenance after each rain event to clean the bar screens and to remove trees and limbs from the debris boom.

The baseline cost estimate for Alternative F is approximately $2.7 million.

1.5.7. Construct Vertical Drop Structures with Telemetered Sluice Gate, Alternative G, (Homestead Road)

For this alternative, a series of vertical drop structures would be constructed integral to a berm upstream of Homestead Road. Reference is made to Alternative G Site Plan CS109 in Appendix G. The drop structures, ten in all, are preliminarily sized as 24-foot diameter circular concrete structures, mimicking the construction of silos commonly used for grain storage. Each drop structure would empty using a 6 foot by 3 foot box culvert traveling through the berm and exiting downstream. A trash fence/jumping fish fence would be placed around the rim of the circular drop structure. This would prevent debris from entering the drop structure from upstream, and prevent fish from jumping out of the drop structure that traveled through the box culvert from downstream. This alternative creates a vertical drop of flow from upstream to downstream, limiting the passage of ANS from downstream to upstream. A sluice gate structure is proposed to be constructed on the Graham-McCulloch Ditch which will be telemetered similar to the concept in Alternative D. The sluice gate will contain an automated closure mechanism that will close such gates to a nominal opening height of 3 inches when gages on the Graham-McCulloch Ditch, Junk Ditch and/or within Eagle Marsh near the drainage divide indicate that flow conditions are imminent that could support transfer of ANS. During normal low flow conditions the gates would be open to allow normal drainage of the Graham-McCulloch watershed.

This alternative will require routine maintenance and inspection after each rain event to ensure debris is not blocking the trash fence on the drop structures and the sluice gate.

The baseline cost estimate for Alternative G is approximately $4.8 million.

1.5.8. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative H, (Eagle Marsh, Basin Divide)

The Graham-McCulloch Ditch at Eagle Marsh is located east of the wastewater treatment plant and passes under the Towpath Trail and wastewater treatment plant access road bridge. Reference is made to Alternative H Site Plan CS111 and CS112 in Appendix G. From there, berms exist on either side of Graham-McCulloch Ditch as it traverses downstream through the Eagle Marsh Area. This section is approximately 8,700 feet long and ties into the Norfolk Southern Corporation Railroad embankment approximately 1,700 feet east of I-69. The existing Indiana DNR fence temporary barrier is tied into the left descending berm of Graham-McCulloch Ditch at Eagle Marsh. From visual inspection of the existing berm in November 2010 and March 2011, it was found to be in deteriorating condition. Unless the berms of the Graham-McCulloch Ditch are repaired in the near future, they will eventually fail. This alternative addresses that aspect and has the potential to provide additional benefits to Eagle Marsh.

Left Photo 1.8: Graham-McCulloch Ditch located east of wastewater treatment plant which is shown on the right in the photo
Right Photo 1.9: Visual inspection of the Graham-McCulloch Ditch reveals voids in the embankment

This alternative proposes to separate the waters from the Maumee and the Wabash River basins by reconstructing the left berm to a higher elevation and to higher construction standards.

This alternative would require demolishing the left descending berm of the Graham-McCulloch Ditch from the water treatment plant access road to the Norfolk Southern Railroad embankment. A new berm will be constructed along the same alignment to prevent overflows of the Graham-McCulloch Ditch into Eagle Marsh during storm events. This new berm will be the hydraulic separation between the basins and would prevent the movement of ANS. In some locations, the new berm will be 2-3 feet higher, and most sections will have a wider footprint than the existing. It is proposed the slopes of the berm will be 3H:1V to accommodate mowing equipment and the berm will have a ten foot crown to facilitate access of operation and maintenance vehicles. The berm would be compacted to 95% maximum dry density per ASTM D 698, and covered with topsoil. The berm itself would be seeded with native grasses, and some type of erosion control measure applied to the berm side slopes and crown following construction. Portions of the berm on the ditch side may need to be lined with rip rap or other means for channel protection.

The Graham-McCulloch Ditch right descending berm would remain as is and would function in the same manner as it does now. Any water which overflows the top of the right descending berm would flow into the north area of Eagle Marsh. A spillway section could be constructed within the right descending berm with this alternative. This spillway would control any overtopping and likely prolong the life of the right descending berm.

Borrow material will need to be acquired for this alternative. While most of the material can be reused from the existing left descending berm, additional material will be needed to build the new berm to a reliable cross section. Regulatory permits would be required for construction of this alternative.

Eliminating overflow of flood waters from the Graham-McCulloch Ditch into storage areas behind the left bank berm will result in increases in peak water surface elevations downstream. The increases in water surface elevations above existing conditions are estimated to be approximately 0.4 foot upstream of I-69 and 0.3 feet upstream of Aboite Road, based upon preliminary modeling. Flowage easements may be necessary for some overbank areas upstream of Aboite Road, but it does not appear any structures are affected. Downstream of Aboite, water surfaces appear to be generally contained within the channel banks.

This alternative will require minimal maintenance and inspection to ensure erosion is not occurring on the berms. Yearly inspections should be anticipated.

The baseline cost estimate for Alternative H is approximately $5.7 million.

1.5.9. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area, Alternative I, (Eagle Marsh, Basin Divide)

This alternative is similar to the previous Alternative H, as it includes reconstruction of the left descending berm. Reference is made to Alternative I Site Plan CS113 and CS114 in Appendix G. The difference is this alternative also includes removal of the right descending berm from the wastewater treatment plant to the point where the Graham-McCulloch Ditch turns at the Norfolk Southern Railroad embankment, a distance of approximately 8,700 feet. Material from the right descending berm would then be used to reconstruct the left descending berm. This would provide a substantial amount of additional material, and would likely delete the need for a borrow site.

Alternative I proposes to separate the waters from the Maumee and the Wabash River basins by reconstructing the left berm to a higher elevation and to higher construction standards. Once the right descending berm is removed, stormwater flows from the Graham-McCulloch will flood the north section of Eagle Marsh with a higher frequency. A multi-cell wetland area is proposed along the previous alignment of the right descending berm of the Graham-McCulloch Ditch. The multi-cell wetland area would begin at the wastewater treatment plant and run approximately 1,800 feet downstream to where the ditch turns and flows in a southwesterly direction. The multi-cell wetland area will pre-treat the urban stormwater from the Graham-McCulloch Ditch by slowing runoff and allowing sediment and attached pollutants to settle and/or be filtered by vegetation.

In Alternative H, water which overtops the right descending berm during a storm event is retained until water levels in the Graham-McCulloch Ditch recede. With Alternative I, as the right descending berm has been removed, there is no water retained by it. This will cause an increase in peak water surface elevations for the Graham-McCulloch Ditch during a storm event that previously would have overtopped the right descending berm, because peak flows are not reduced through detention behind the right bank berm. Anticipated increases over Alternative H elevations are approximately 0.6 foot near I-69, and 0.3 foot downstream at Aboite Road for the 1% annual chance event As discussed above for Alternative H, these increases may require flowage easements upstream of Aboite Road, but do not appear to impact any structures. Downstream of Aboite Road, the water surfaces still appear to remain within the channel banks.

This alternative will require minimal maintenance and inspection to ensure erosion is not occurring on the berms. Yearly inspections will be required.

The baseline cost estimate for Alternative I is approximately $7.2 million.

1.6. ELIMINATED STRUCTURAL MEASURES

As noted in the Value Engineering discussion, five measures were eliminated prior to being developed into alternatives. These five measures are discussed in the following paragraphs along with a few other measures that were initially discussed. These measures are detailed in the following paragraphs including reasoning for elimination.

1.6.1. Create Storage in Both Basins

This measure would require construction of large detention areas within the St. Marys River and Graham-McCulloch Ditch watersheds in order to reduce water surface elevations enough that a hydraulic connection does not occur. Areas along the Junk Ditch corridor and the Graham-McCulloch Ditch upstream of Engle Road were reviewed for potential sites. The Junk Ditch watershed is well-developed with both residential and light industrial and commercial development along both banks. The Graham-McCulloch Ditch watershed is largely residential areas. To significantly reduce the likelihood of a hydraulic connection, i.e., prevent water passing the natural drainage divide for the 1% annual chance event, approximately 990 acre-feet of storage will be required on the Junk Ditch side of the watershed divide and 285 acre-feet of storage on the Graham-McCulloch Ditch side assuming that flow is permitted to be stored in the Eagle Marsh and Fox Island areas without passing the drainage divide. To prevent flow from passing the Graham-McCulloch left bank berm, approximately 585 acre-feet of storage would need to be created.

Based on review of the watershed and an attempt to identify potential detention basin sites, areas large enough to provide adequate storage volumes were not available. Depths of excavation for detention areas would be limited due to the flat topography or expected shallow depths to ground water surface. Therefore, large surface areas would be required to achieve the necessary storage volumes.

This measure was considered not feasible due to the amount of real estate acquisition that would be necessary to achieve the necessary storage volumes. The length of time it would take to acquire the real estate required for this measure was also considered. The real estate process for this amount of acquisition would be lengthy; due to the urgency to prevent movement, this measure was eliminated.

1.6.2. Keep the Temporary Barrier Fence at Eagle Marsh

The current location of the Indiana DNR fence at Eagle Marsh was not without controversy. After all, the fence was constructed in the midst of a wetland restoration project at Eagle Marsh. However, the completion of the temporary barrier was a substantial accomplishment in preventing the movement of ANS, particularly mature Asian carp, across the basin divide until a permanent solution can be designed and constructed.

The incorporation of the existing fence measure was eliminated for multiple reasons. The main reason is due to the condition of the left descending Graham-McCulloch Ditch berm which the north abutment of the fence ties into. As stated earlier in previous alternatives, upon visual inspection, the Graham-McCulloch Ditch berms are in a deteriorated condition, with unknown methods of construction and level of compaction. This raises concerns about the long-term integrity of the Graham-McCulloch Ditch berms. If the left descending berm fails upstream of the current fence during a flood event, the ANS will have a hydraulic path around the fence to migrate between drainage basins. For this reason, a fence/berm combination such as Alternative B is more desirable as it significantly reduces the risk of a berm failure.

Another reason the temporary fence barrier design was eliminated is the barrier not only stops the passage of some ANS, but it also stops native wildlife movement as indicated by local environmental agencies. Additionally, the 2 inch mesh size of the current chain link fence does not prevent the transfer of smaller ANS when submerged.

1.6.3. Electric Dispersal Barrier

An electric dispersal barrier operates by creating an electrical field in the water of the stream/ditch, which either will immobilize the ANS or create sufficient discomfort to deter them from attempting to pass through the area. The electrical field is created by running direct electrical current through steel cables secured to the bottom of the stream/ditch. The electrodes are connected to a raceway, consisting of electrical connections to a control building. Equipment in the control building generates a direct current pulse through the electrodes, creating an electric field in the water. To ensure the barrier is always operational during power outages, back-up generators that automatically activate are essential if a complete or partial power loss occurs.

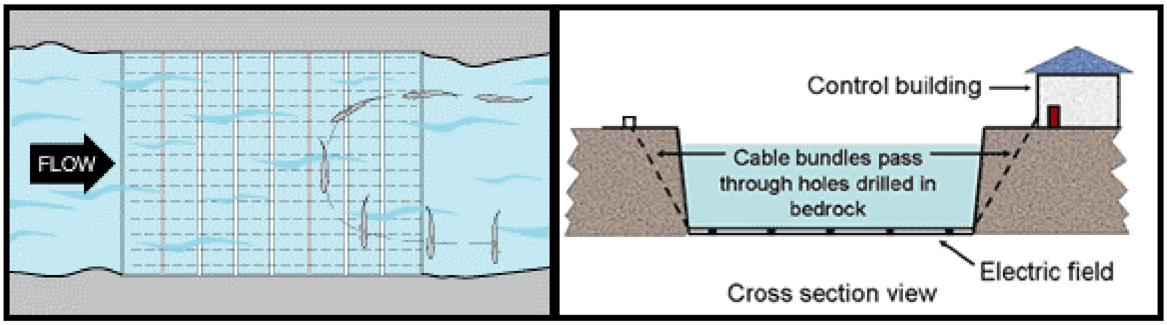


Figure 1.1: Schematic of typical electrical dispersal barrier across waterway. As fish advances into the electric field, they feel discomfort and are unable to advance further. The fish turns and swims in the opposite direction.

There were several reasons why this measure was eliminated. First and foremost were the long-term operation and maintenance requirements for the electric dispersal barrier. Depending on the location of the electrical dispersal barrier, the electric field may or may not be running continuously. But there will be monthly utility bills associated with this barrier which will be significant during months in which rainfall events occur.

Field effectiveness of the electrical barrier is also a concern. In a recent memorandum for record prepared by the Corps of Engineers, Great Lakes and Ohio River Division, dated 24 March 2011, Subject "Operational Protocols for Electrical Barriers on the Chicago Sanitary and Ship Canal: Influence of Electrical Characteristics, Water Conductivity, Fish Behavior, and Water Velocity on Risk for Breach by Small Silver and Bighead Carp" a study has shown that reactions of ANS to electrical exposure are influenced by fish length. Longer fish are repelled earlier in the electric field than short fish. Therefore, there is no guarantee the electrical barrier can prevent movement across the hydraulic basin divide.

Another factor that influenced the elimination of this measure was the evaluation of human safety versus the electric barrier. Eagle Marsh is a popular attraction for outdoor activities; the proximity of the electric barrier could be a danger to hikers and others using this area.

1.6.4. Rotating Drums, Traveling Curtains, Floating Curtains

Rotating drum screens and traveling screens operate very similarly. Both barriers continuously rotate to pass debris over the top of the drum to the downstream side while deterring ANS from migrating past the screen. Continuous exposure to wet weather during extremely low temperatures will cause ice to form on the rotating drums and traveling/floating curtains. When this occurs, they are inoperable and ineffective to preventing the movement of ANS.

Both barriers are good for small, well-defined channels. However, for this particular study, movement of the ANS only occurs when the two drainage basins connect, which is during flooding conditions and water is outside the channel, spread across the wide valley. Therefore, these barriers were determined not to be feasible.

Floating curtains consist of a system of floats attached with a cable that stretch across the channel. Attached to the cable below the water surface are nets, made of nylon or chains. Floating curtains are best used in channels with low velocities and little debris and do not prevent movement of ANS species which may jump over the floating curtain to the other side.

1.6.5. Utilize Local Quarry as a Storage Area

This measure considers the use of one of the existing quarries to store excess floodwaters and prevent a hydraulic connection. Two quarries, owned by Hanson Aggregates, are located southeast and south of Eagle Marsh. The southern quarry on Lower Huntington Road is no longer active and was considered for this measure.

The concept of this measure is that floodwaters from Junk Ditch could be channeled or piped to the quarry, thus reducing floodwaters in the Eagle Marsh area so that there would not be a hydraulic connection. This is similar to the eliminated measure in Section 1.6.1, Create Storage in Both Basins, as previously discussed. The existing quarry pit would serve as the storage area for the Junk Ditch basin.

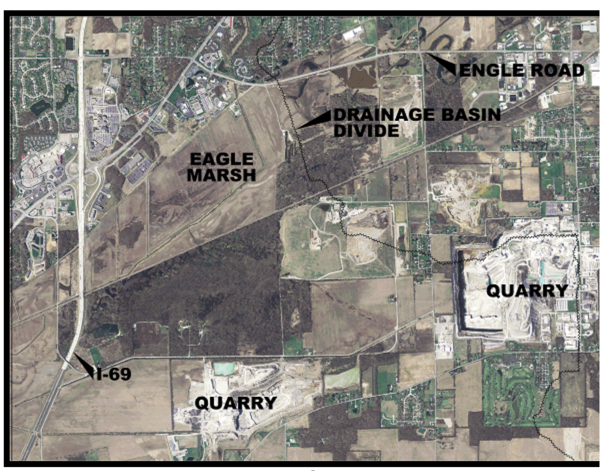


Photo 1.10: Location of quarries in proximity to project area

To use this quarry for storage would require purchasing or acquiring an easement on the quarry itself, as well as multiple acres to construct the channel or pipeline. Extensive earthwork and the creation of over a mile of drainage channel or pipeline to the quarry would also be necessary. Piping the water to the quarry does not appear cost effective. Existing channels are not large enough to convey the volume of water, and are potentially subject to floodwaters from overflow of the Graham-McCulloch Ditch.

Therefore substantial excavation and grading would need to be accomplished. The anticipated operation and maintenance of pumping of the stored water back to the proper watershed after each flood event would also be costly. Therefore, acquisition of this property combined with construction of the drainageway and pumping operations is expected to far exceed the cost of other alternatives.

1.6.6. Bar Screens

Angled bar screens are proposed in Alternative F in Section 1.5.6 above in combination with a vertical drop. This eliminated measure is for vertical bar screens as a standalone measure.

Bar screens are devices which allow unrestricted passage of water and small debris, but prevents passage of ANS of a specified size. The size of the opening on the bar screen can vary depending on the size of the ANS targeted for cross contamination. The bar screen would be attached to a vertical support post which would span the entire width of the stream. Since some of the ANS are excellent jumpers, the barrier height of the bar screen would extend above the water level to prevent movement. Bar screens are often found on intakes to pump stations, and often have a mechanical debris removal system.

Bar screens were eliminated from consideration for several reasons. First, bar screens can dramatically reduce the flow rate depending on the opening sizes. Second, to use bar screens on this project, a large length or area of screen would be required to pass the needed flow. Third, the effect of bar screens is comparable to fencing, yet is a much more expensive option. Finally, debris buildup on the bar screens would be a maintenance issue and would require regular cleanup. Due to these reasons, bar screens are not intended in areas that need to allow a large or constant amount of flow.

1.6.7. Wetlands Storage

This measure is similar to the eliminated measure in Section 1.6.1, Create Storage in Both Basins, as previously discussed. The wetlands storage alternative considers the use of the existing wetland areas at Eagle Marsh and Homestead Road to store excess floodwaters and prevent a hydraulic connection. Based on review of the watershed and the areas of the available existing wetland areas, adequate storage volumes did not appear achievable. Depths of excavation for detention areas would be limited due to the flat topography or expected shallow depths to ground water surface. Therefore, large surface areas would be required to achieve the necessary storage volumes.

This measure was considered not feasible due to the amount of real estate acquisition that would be necessary to achieve the necessary storage volumes. The length of time it would take to acquire the real estate required for this measure would be prohibitive. Due to the real estate costs and the urgency to prevent movement, this measure was eliminated.

1.6.8. Enhance Conveyance of Maumee and Wabash Rivers

This measuree would develop a plan for enhancing conveyance of the existing channels, or creating new channels in order to reduce peak water surface elevations, eliminating or significantly reducing the frequency of a hydraulic connection between the basins. This typically would take the form of channel clearing, channel widening, lowering the thalweg of the channel, or a combination of these measures. Channel improvements in the Maumee River and St. Marys River channels could likely reduce water surface elevations to prevent flow across the left descending bank berm of the Graham-McCulloch Ditch, but real estate acquisition along these rivers in downtown Fort Wayne would likely take significant time. Besides real estate acquisition costs and delays, the Graham-McCulloch Ditch overflows the left descending bank berm in Eagle Marsh for flows greater than a 10% annual chance event or approximately 1300 cubic feet per second (cfs). Expanding the channel to pass additional flows in excess of 1900 cfs for the 1% annual chance event to prevent overtopping of the left bank berm was not deemed feasible.

1.6.9. Removable Fence/Barrier

This measure would require a removable fence/barrier erected only when flood waters threaten to create a hydraulic connection of the Maumee and Wabash Rivers at Eagle Marsh. Once the floodwaters recede, the fence/barrier would be removed and packed neatly away, leaving an unobstructed view of Eagle Marsh. An example of a removable barrier is shown below in Photo 1.11. Vertical supports anchor into the concrete foundation and aluminum planks are stacked between the vertical supports. The aluminum planks weigh approximately five pounds per foot.

Photo 1.11: Example of a removable barrier

Photo 1.12: Concrete wall foundation, only bolts are visible when barrier is down

Manufacturers of the removable wall estimate one person can erect a 50 foot section of 10 foot high wall in about 2.5- 5 man-hours, depending on the difficulty of the site. Therefore, based on the average height and length of the removable wall at the basin divide which is estimated to be approximately six feet high and 4,800 feet long, the removable wall could take up to 288 man-hours to install.

Based on the accepted hydrograph, as developed by Indiana DNR and reviewed by multiple agencies, the St. Marys River will peak 36 hours after the initial rainfall. For the Graham-McCulloch watershed area, which is considerably smaller than the St. Marys

River watershed, the time to peak is even less. Due to the time of concentration which is estimated to be less than one day, the flashiness of both streams would not allow adequate time to mobilize and erect a temporary barrier. Therefore, this measure was considered not feasible.

1.6.10. Reroute Graham-McCulloch Ditch to Junk Ditch

For any permanent barrier measure that blocks the Graham-McCulloch Ditch, in addition to preventing flows across the divide, it will also block the natural drainage of the Graham-McCulloch watershed, including flows contributed from local groundwater. The Graham-McCulloch Ditch drains an area of approximately twelve square miles upstream of the Towpath Trail (also referred to as the old railroad grade) bridge at the existing wastewater treatment plant. To allow this area to drain without impediment, and thereby reduce the need for a barrier structure to handle the majority of these daily flows, this alternative would intercept the Graham-McCulloch Ditch at Engle Road and create a pathway for flow eastward along Engle Road to near its intersection with Smith Road, where flows would enter Junk Ditch. Two potential alignments were investigated:

1. Construct a channel along the south side of Engle Road to the edge of Little River Wetlands property. To avoid disturbance to existing constructed wetlands area on the United Refuse Company property, this channel would be diverted to the north side of Engle Road via a box culvert and follow eastward adjacent to the roadway to a new confluence with Junk Ditch downstream of the Engle Road bridge near Smith Road.

2. Construct a culvert along the north side of Engle Road to the east side of the property of the existing housing development at Statesmans Way. A culvert would likely be required in order to avoid disturbance of the homes adjacent to Engle Road due to the narrow distance. A channel would then follow along the north side of Engle Road to a junction with Junk Ditch near Smith Road.

This measure was eliminated due to the extremely small slope available between the Graham-McCulloch Ditch at Engle Road and Junk Ditch at Smith Road. In order to handle any significant flows without inundation of Engle Road, the channel would need to be extremely wide and the associated culverts excessively large. A storage basin on the south side of Engle Road was considered to complement these measures, in order to reduce flows and thus reduce the size of the channel, but the basin too would have had to be very large (i.e., past the bend in the Graham-McCulloch Ditch), making draining this area back to the east extremely difficult. Lowering the Junk Ditch thalweg to increase slopes was considered, but this would require channel modification for most of the length of the Junk Ditch, which was considered infeasible due to the length of time required for associated real estate acquisition.

1.6.11. Longest Economical Crest

This concept was proposed based upon the theory that the flow capacity of a weir can be increased by increasing the length of the weir. The weir would serve as a barrier to ANS by creating a vertical water surface differential at the weir, much the same as a drop inlet structure. In order to pass adequate quantities of flow during high flow events, a labyrinthine weir would be constructed to an elevation allowing the 1% annual chance event to pass without increasing water surface elevations in the Junk Ditch and St. Marys River floodplains. The weir would likely be constructed as a concrete and/or sheet pile wall structure. Low flows detained behind this structure would be drained by manual or automated opening of a sluice gate or similar structure. This measure was eliminated because the low topographical relief of the area combined with the high flows of the 1% annual chance event would not create adequate water surface differential across the weir, even with weir lengths several times wider than the valley, without increasing water surface elevations upstream.

1.7. COST ESTIMATE SUMMARY

Table 1.1 gives a summary of structural alternatives and the baseline cost estimates (screening level of design) that prevent or reduce the possibility of movement of ANS.

Alternative	Alternative Description	Location of Proposed Work	Operation and Maintenance*	Baseline Cost Estimate**
A	Construct an I-wall	Eagle Marsh (Basin Divide)	$11k	$14M
B	Construct a Fence and Reconstruct the Left Descending Graham-McCulloch Ditch Berm	Eagle Marsh (Basin Divide)	$18k	$3.2M
C	Construct an Earthen Berm and Pump Station	Homestead Road	$600k	$25M
D	Construct a Permeable Berm with Telemetered Sluice Gates	Amber Road	$22k	$7.8M
E	Construct Fence/Earthen Berm Combination	Eagle Marsh (Basin Divide)	$22k	$4.2M
F	Construct Bar Screen Barrier at Existing Weir	Huntington Dam	$96k	$2.7M
G	Construct Vertical Drop Structures with Telemetered Sluice Gate	Homestead Road	$26k	$4.8M
H	Reconstruct Left Descending Graham-McCulloch Ditch Berm	Eagle Marsh (Basin Divide)	$14k	$5.7M
I	Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area	Eagle Marsh (Basin Divide)	$17k	$7.2M

Table 1.1: Summary of Alternatives and Baseline Cost Estimate

Hydrologic Risk Analysis Ratings are to be determined. Reference Appendix A for additional information.
**Reference Section 9 of this Appendix for the Operation and Maintenance schedules.*
***Reference Section 10 of this Appendix for the Cost Estimate.*

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2.1. REFERENCES

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2.2. GENERAL

The city of Fort Wayne is located in northeastern Indiana in Allen County. The city developed at the junction of the St. Marys River and the St. Joseph River, which together form the Maumee River. The headwaters of the St. Marys River begin in northwestern Ohio in the vicinity of the town of St. Marys, and the river generally drains in a northwesterly direction, crossing into northeastern Indiana, before turning northeasterly near its mouth. The St. Joseph River headwaters are located in southeastern Michigan and flows to the southwest, crossing the northwestern tip of Ohio, and joining the St. Marys River in downtown Fort Wayne. The Maumee River drains from this point in Fort Wayne to the northeast and into northwestern Ohio, to its confluence with Lake Erie in Maumee Bay along the Ohio-Michigan state border. Please also reference Figure 2.1 and the Appendix G drawings.

In addition to this river network, a wide valley known as the Wabash-Erie Channel, formed by glacial melt waters, extends from the west side of Fort Wayne to the southwest. The Wabash-Erie Channel formed over time as the natural outlet path for the relatively young St. Marys and St. Joseph Rivers in the wake of retreating glacier, and later as the outlet for “Glacial Lake Maumee”, prior to glacial retreat and formation of the Maumee River. Also called the St. Marys Overflow, this valley acts as a “relief valve” for floods on the St. Marys River and Maumee River watersheds, allowing high flows to inundate and reverse flow in Junk Ditch, a tributary near the mouth of the St. Marys River, and cross the natural divide into the Wabash River watershed.

The Wabash-Erie Channel valley was attractive to early Native Americans and European settlers in the area as it provided a short 8-mile portage into the headwaters of the Wabash River, affording access not only to the areas of northern Ohio and inland Michigan by way of the St. Marys, St. Josephs, and Maumee Rivers, but much of current-day Indiana, Illinois and other areas along the Ohio and Mississippi Rivers to the south. Being the lowest elevations in western Allen County, the wide, flat valley generally was composed of a series of interconnected wetlands. Over time, this area became popular for agriculture, and man-made channels were constructed in this valley to promote drain

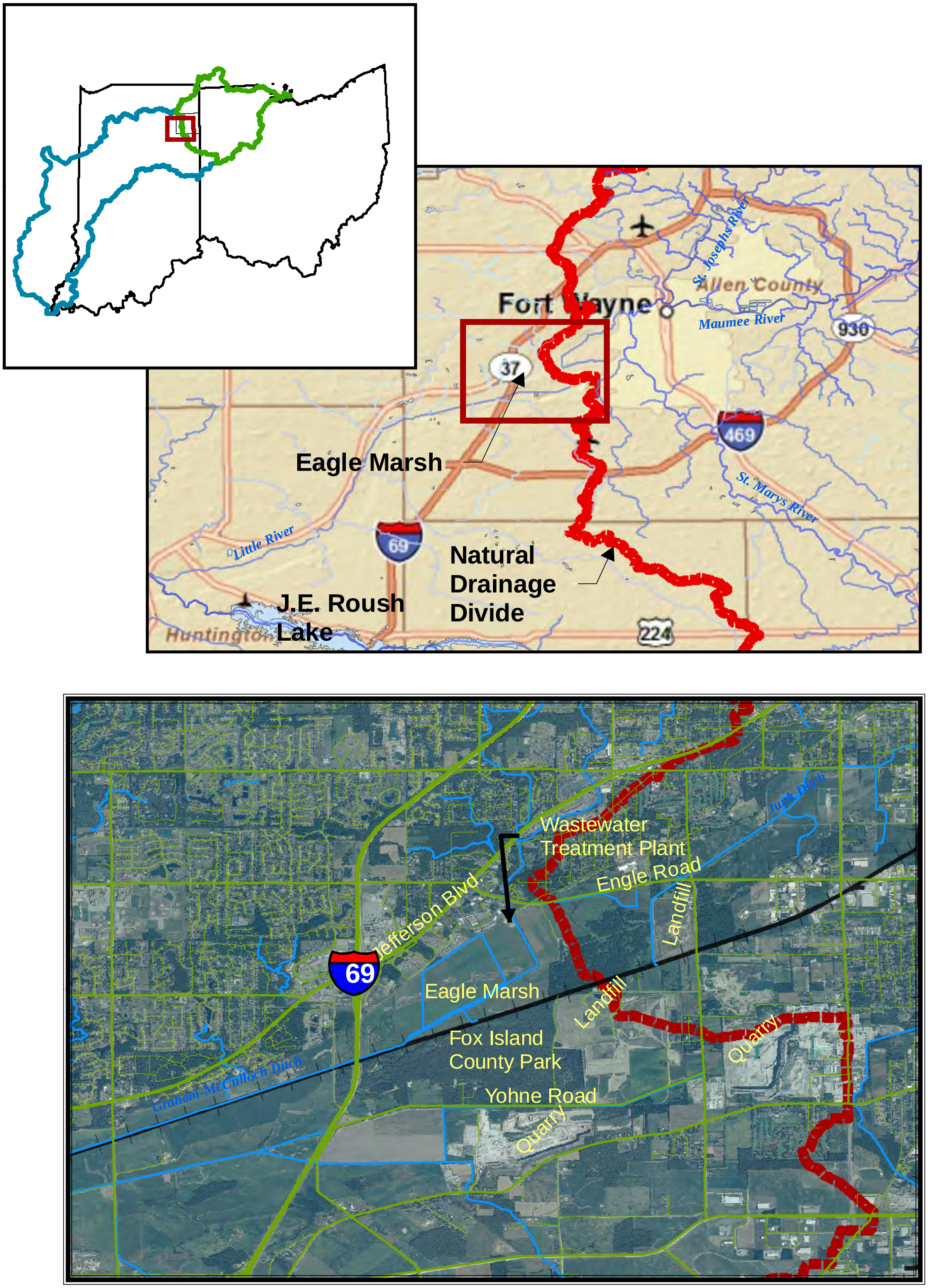


Figure 2.1: Location Map

-age and improve crop production. The stream and upper watershed that was formerly called Cranberry Creek drained into the marsh lands near the current wastewater treatment facility at Engle Road. The Graham-McCulloch Ditch was cut across the marsh from the mouth of Cranberry Creek and the entire watershed was named for this new ditch. Berms isolate the regular floodwaters of the Graham-McCulloch Ditch from the surrounding valley floodplain; these berms were likely constructed from the channel excavation materials and additional soil pushed up from the surrounding valley as it was converted to farm land. Junk Ditch was also constructed through the eastern end of the valley, connecting several isolated meanders with a much smaller existing ditch that drained toward the St. Marys River, effectively reversing the direction of flow for that drainage area and moving the drainage divide from its natural location to a point roughly three miles further west. These channels, combined with systematic tiling of the farm lands, led in large part to the demise of these wetlands within the valley. These former wetlands are gradually being restored through the cooperative efforts of the Little River Wetlands Project, the Natural Resource Conservation Service (NRCS), and the Indiana Department of Natural Resources (IDNR). Eagle Marsh is the first and largest project for this restoration initiative, encompassing approximately 716 acres east of Interstate 69 and south of Engle Road.

Because of the natural connection between the two watersheds by way of the Wabash-Erie Channel valley, the concern is that this location provides adequate hydraulic connection that could support the exchange of invasive species between the Great Lakes and the Mississippi watersheds. As part of this study is to evaluate technologies to prevent the transfer of invasive species, a better understanding of the hydraulic conditions and frequency of flooding was required. This section summarizes the hydrologic and hydraulic analysis performed to date to support this study.

2.3. CLIMATE

Allen County is located within the humid continental climate zone and experiences four distinct seasons per year. Weather patterns in Allen County are influenced by the Great Lakes. The seasonal range of temperature is a daily winter minimum of approximately 20 degrees Fahrenheit (ºF) to a daily summer maximum of approximately 85ºF. Snowfall is prevalent in the winter months, with average cumulative annual snowfall of approximately 33 inches. The average annual precipitation is approximately 36 inches, with annual evapotranspiration of approximately 27 inches.

2.4. PRECIPITATION

Precipitation in the Fort Wayne area is fairly well distributed throughout the year, with the monthly averages ranging from 1.9 inches in August to 4.0 inches in June. Table 2.1 gives the average monthly rainfall based upon the period of 1971-2000 for the Fort Wayne International Airport.

TABLE 2.1
MEAN MONTHLY PRECIPITATION (INCHES)
FOR THE FORT WAYNE, IN AREA

Month	Precipitation (Inches)	Month	Precipitation (Inches)
January	2.1	July	3.6
February	1.9	August	3.6
March	2.9	September	2.8
April	3.5	October	2.6
May	3.8	November	3.0
June	4.0	December	2.8
		Total:	**36.6**

2.5. HISTORIC STORMS AND FLOODS

2.5.1. Discussion of return frequency event terminology

In the past, terminology such as "10-year flood" or "100-year flood" has been used to describe the chances of an event occurring with a statistically-determined peak flow in a given year. This terminology has fallen out of favor because the emphasis is on the period expected to occur between flood events of a given magnitude, and the statistics behind the frequency of the event is lost. For example, it becomes "expected" that the "100-year flood" will only occur once every 100 years, instead of understanding that there is a 1% chance that this event could be equaled or exceeded in any given year. Over time, such a flow may on average occur once every hundred years, but the time between individual events could vary widely. Instead, terminology used in this report will refer to the probability of an event occurring or being exceeded in any given year, e.g., the "1% annual chance event."

2.5.2. Historic Floods

Flash flooding in the vicinity of Fort Wayne occurs nearly annually to some degree, but based upon review of the floods in Allen County from the National Climatic Data Center storm event database (http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms), the majority of these floods occur on minor tributaries with limited areas of damage. Much larger events on the St. Marys watershed have occurred infrequently, with the most significant occurring in 1913, 1982, 1985, and 1991, and 2003. Significant floods have typically been caused by frontal storms of great intensity, long duration and extending over large areas of the St. Marys and St. Joseph Rivers watersheds.

The 1982 flood is generally the benchmark to which most floods are compared, being nearly the flood of record and superceded by less than 0.2' by the 1913 flood on the Maumee River. Various sources report that the 1982 flood event was the product of a 2.9% - 4% annual chance event on the St. Marys River combined with a 0.7% - 1% chance event on the St. Joseph River peaking approximately 3 days later, and was the result of extended rainfall combined with snowmelt. In 1985, a 4% annual chance event on the St. Marys River combined with a 3.3% annual chance event on the St. Joseph River, peaking only approximately 12 hours apart. The 2003 flood event became the flood of record on the upper St. Marys River after nearly 15 inches of rain fell over the watershed, with the majority falling over a four day period. Flooding was widespread, with additional damage noted along the upper Wabash River near Bluffton. Tables 2.2a and 2.2b list the top ten flood stages at the USGS gages upstream and downstream of the city of Fort Wayne.

TABLE 2.2a
HISTORIC PEAK STAGES
MAUMEE RIVER AT FORT WAYNE
(USGS GAGE 04182900)

Date	Gage Height (feet)	Elevation (NAVD88)	Notes
03/26/1913	26.10	755.66	~ 2% annual chance event
03/17/1982	25.93	755.49	~ 1.25% annual chance event
02/27/1985	24.55	754.11	~ 3.3% annual chance event
01/01/1991	23.90	753.46	
03/24/1978	23.76	753.32	
03/13/2009	23.58	753.14	
02/08/2008	22.93	752.49	
03/08/1908	22.50	752.06	
01/14/2005	22.44	752.00	
01/15/1930	22.40	751.96	

* Historical Peak Elevation. Datum of the gage is 729.56 NAVD88.

TABLE 2.2b
HISTORIC PEAK STAGES
ST. MARYS RIVER NEAR FORT WAYNE
(USGS GAGE 04182000, a.k.a MULDOON BRIDGE)

Date	Gage Height (feet)	Elevation (NAVD88)	Notes
07/09/2003	21.20	769.66	
03/14/1982	19.66	768.12	~ 4% annual chance event
02/11/1959	19.42	767.88	
01/14/2005	19.06	767.52	
5/19/1943	18.79	767.25	
3/21/1978	18.39	766.85	
2/16/1950	18.34	766.80	
02/26/1985	18.33	766.79	
01/01/1991	17.92	766.38	
02/09/2008	17.67	766.13	

* Historical Peak Elevation. Datum of the gage is 748.46 NAVD88.

2.6. HYDROLOGY

2.6.1. Existing Gages

Detailed hydrologic models were not developed for the individual watersheds that contribute to the complex hydrology of the Fort Wayne area; rather, existing data was to be used to the extent possible. Several flow and/or elevation gages exist within the Fort Wayne area, but only a few have been in place for enough time to develop sufficient periods of data for analysis. Table 2.3 lists the gages and their periods of records, and Figure 2.1 depicts their locations. Real-time stages and flows are provisional, available for a period of 120 days prior to the most recent reading. As can be seen, the gages are concentrated on the St. Marys and Maumee Rivers. No gages exist on Junk Ditch or the Wabash side of the natural drainage divide in or near the area of interest; the nearest gage on the Wabash side is on the Littler River at Huntington, Indiana.

TABLE 2.3
STREAM GAGES NEAR ST. MARYS OVERFLOW

USGS Gage No.	Gage Name/Location	Gage Type	Period of Record
03324000	Little River near Huntington, IN	Real-Time Stage, Flow; Mean Daily Discharge	4/1944-Present
03323583	Eagle Marsh East Near Fort Wayne, IN	Real-Time Stage	8/2010-Present
03323584	Eagle Marsh West Near Fort Wayne, IN	Real-Time Stage	8/2010-Present
04180500	St. Joseph River near Fort Wayne, IN	Real-Time Stage, Flow; Mean Daily Discharge	8/1941-Present
04182000	St. Marys River near Fort Wayne, IN (Muldoon Bridge)	Real-Time Stage, Flow; Mean Daily Discharge	11/1930-Present
04182769	St. Marys River At Main St. at Fort Wayne, IN	Real-Time Stage, Flow; Mean Daily Discharge	10/2009-Present
04182808	Spy Run Creek near Park Drive at Fort Wayne, IN	Real-Time Stage, Flow; Mean Daily Discharge	5/2008-Present
04182900	Maumee River at Fort Wayne, IN (Anthony Blvd.)	Real-Time Stage; Daily Stage	10/1997-Present
04182950	Maumee River at Coliseum Blvd. at Fort Wayne, IN	Real-Time Stage, Flow; Mean Daily Discharge	12/2003-Present
04183000	Maumee River at New Haven, IN	Real-Time Stage, Flow; Mean Daily Discharge	10/1956-Present

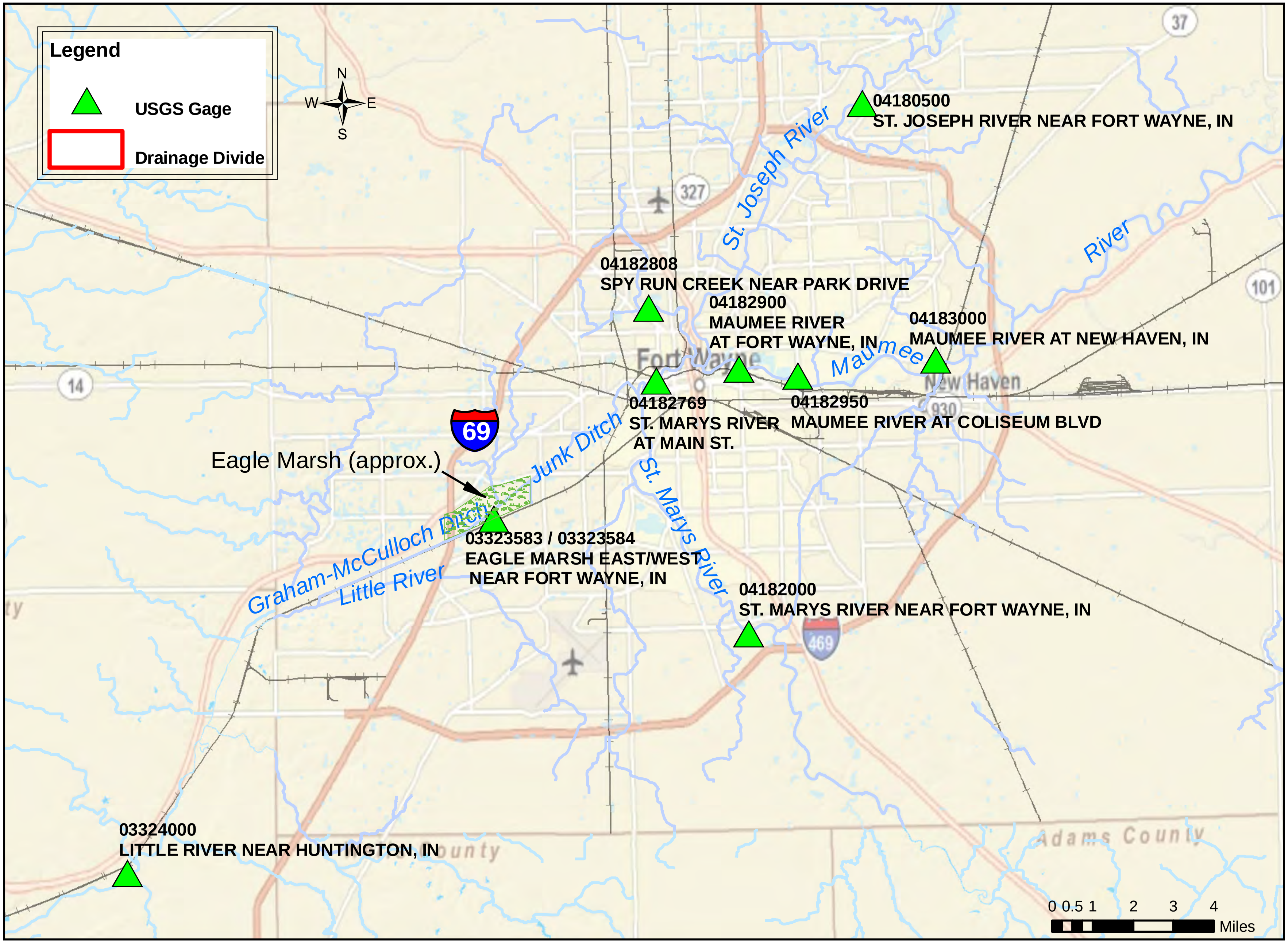


Figure 2.2: Stream Gages Near St. Marys Overflow

2.6.2. Flow Data Used for Hydraulic Modeling

By a Memorandum of Understanding (MOU) of 6 May 1976, the U. S. Soil Conservation Service (now known as the Natural Resources Conservation Service), the U. S. Geological Survey (USGS), the U.S. Army Corps of Engineers (USACE, specifically Louisville, Detroit and Chicago Districts), and the Indiana Department of Natural Resources (IDNR) mutually agreed to coordinate discharge-frequency values for use in water resources investigations and planning activities in the State of Indiana. "Coordinated Discharge" values for only the 10%, 4%, 2%, and 1% annual chance events were determined under this agreement, and were used where available in this study. Flows for other design storm frequency events were interpolated or extrapolated from these values using standard methodologies. Coordinated Discharge graphs for the St. Marys River and the Graham-McCulloch Ditch are included as Figures 2.3a and 2.3b respectively. For the St. Marys River, where an extended period of gage data was available, a partial duration analysis was performed to better establish flows for events more frequent than the 10% annual chance event. The derived discharge-frequency curve for the St. Marys River at the USGS gage is included as Figure 2.4. It should be noted that the Coordinated Discharge values for the Graham-McCulloch Ditch are not specific to this stream, but rather were obtained from the Coordinated Discharges chart "Ungaged Streams in Allen County". Per the MOU, "[w]here adequate stream-flow data is not available, provisional discharge-frequency determinations" will consider USGS Circular 710, *Floods in Indiana: Technical Manual for Estimating Their Magnitude and Frequency*. This circular presents regression equations for estimating flood magnitude and frequency. Due to the absence of any gages on the Graham-McCulloch or any of its tributaries, this was the best existing information available, although regional regression equations are largely discounted as the least accurate of methodologies for determining flow. The accuracy of this information is also questioned given the substantial development that is believed to have occurred in the Graham-McCulloch watershed since Circular 710 (dated 1974) or the Ungaged Streams in Allen County (dated March 1982) was published. A peak discharge value for the 0.2% annual chance event was also published in the 2009 FIS study report. These values were used to derive peak flow values for other design frequency events, as shown in Figure 2.5.

Per files available through IDNR, the hydrograph for the 1% annual chance event was based upon the observed hydrograph for the 1985 flood and the unit hydrograph. It is believed that the 1985 event was chosen instead of the 1982 flood for the basis of this hydrograph because of better available hydrograph information and flow records, and the combination of flooding on the St. Marys and St. Joseph Rivers created a stable backwater for an extended period of time for analysis of Junk Ditch flooding. The hydrograph for the 1985 flood was scaled up to match the 1% annual chance peak flows, as shown in Figure 2.6. This same method of scaling the unit hydrograph was used to create hydrographs for other design frequency events. No information was available to establish a typical hydrograph for floods on the Graham-McCulloch Ditch, so a theoretical hydrograph shape was based upon the St. Marys River unit hydrograph, with the rising and descending limbs of the hydrograph shortened based upon engineering judgment to account for the smaller watershed size (See Figure 2.7).

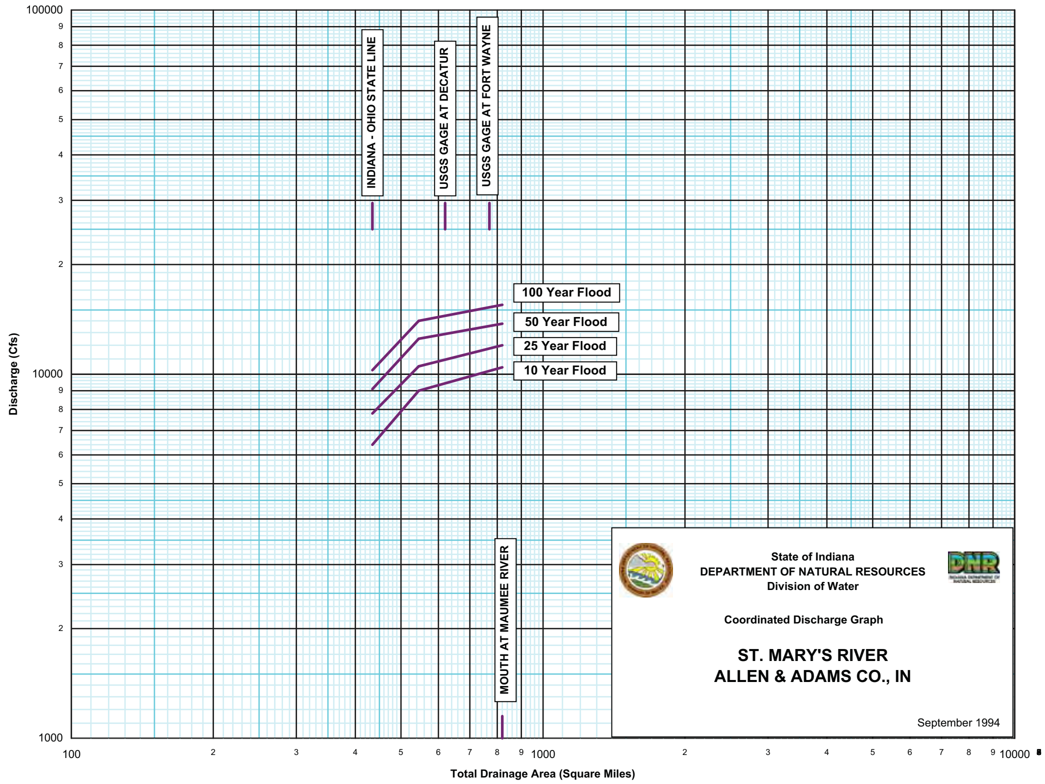


Figure 2.3a: Coordinated Discharges graph for St. Marys River

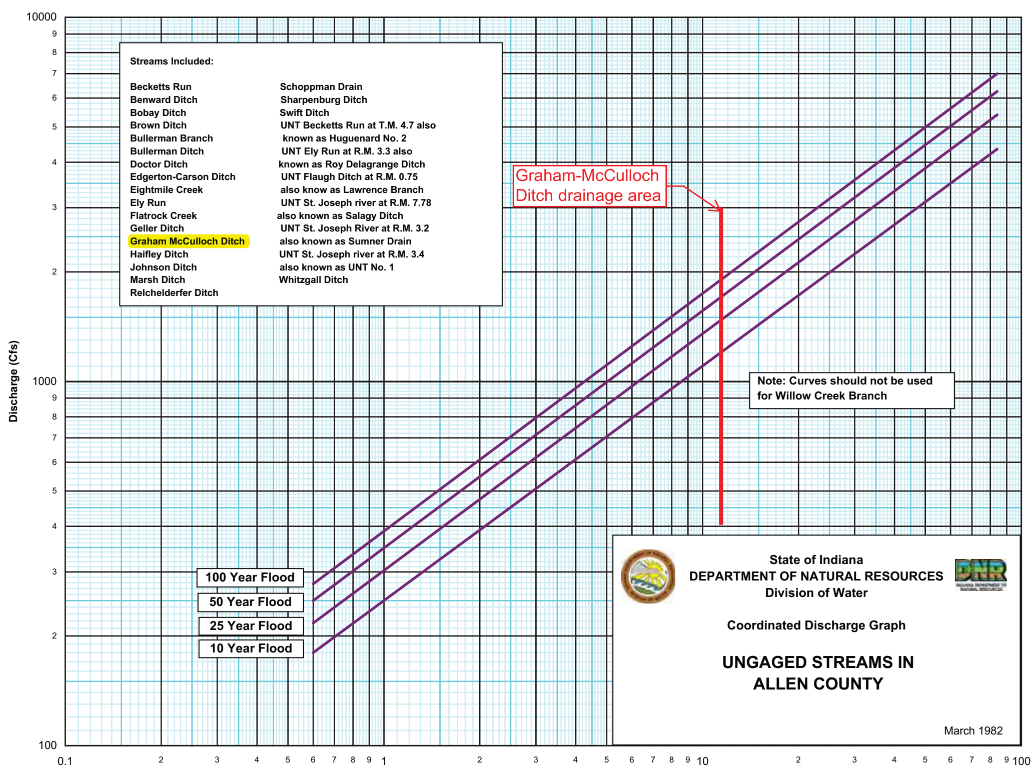


Figure 2.3b: Coordinated Discharges graph for Graham-McCulloch Ditch

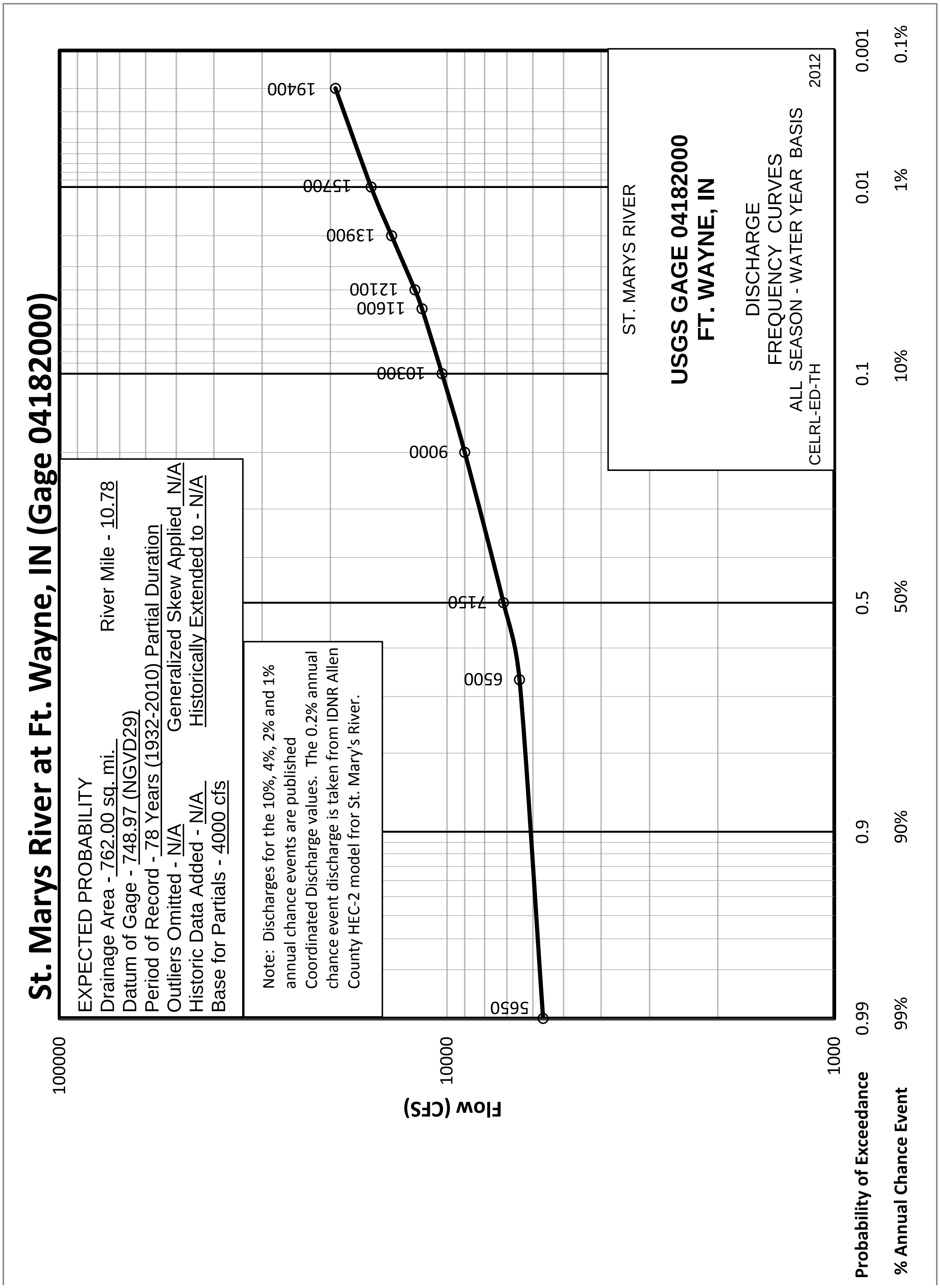


Figure 2.4: Derived flows for St. Marys River at USGS gage

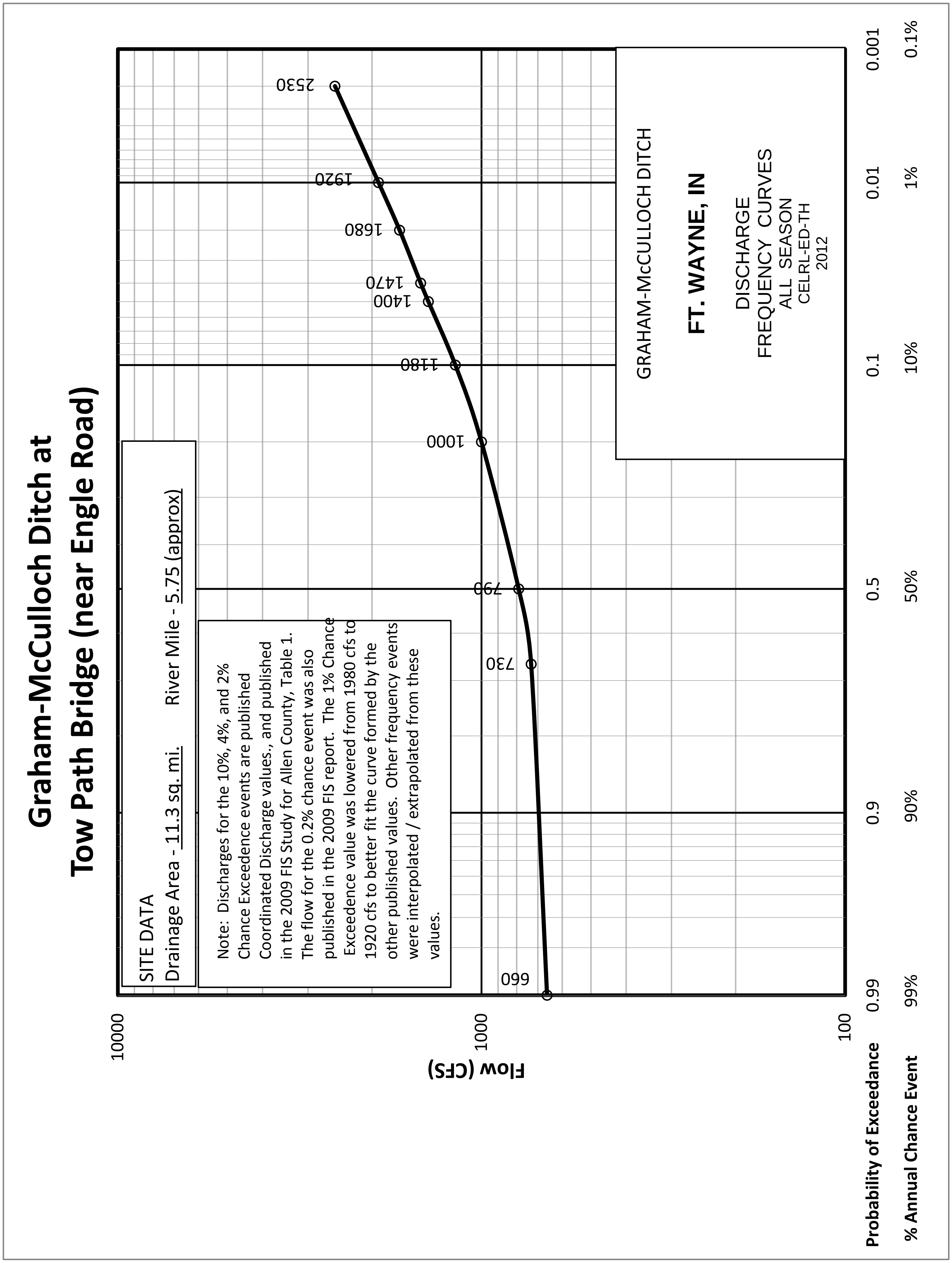


Figure 2.5: Derived flows for Graham-McCulloch Ditch upstream of Eagle Marsh

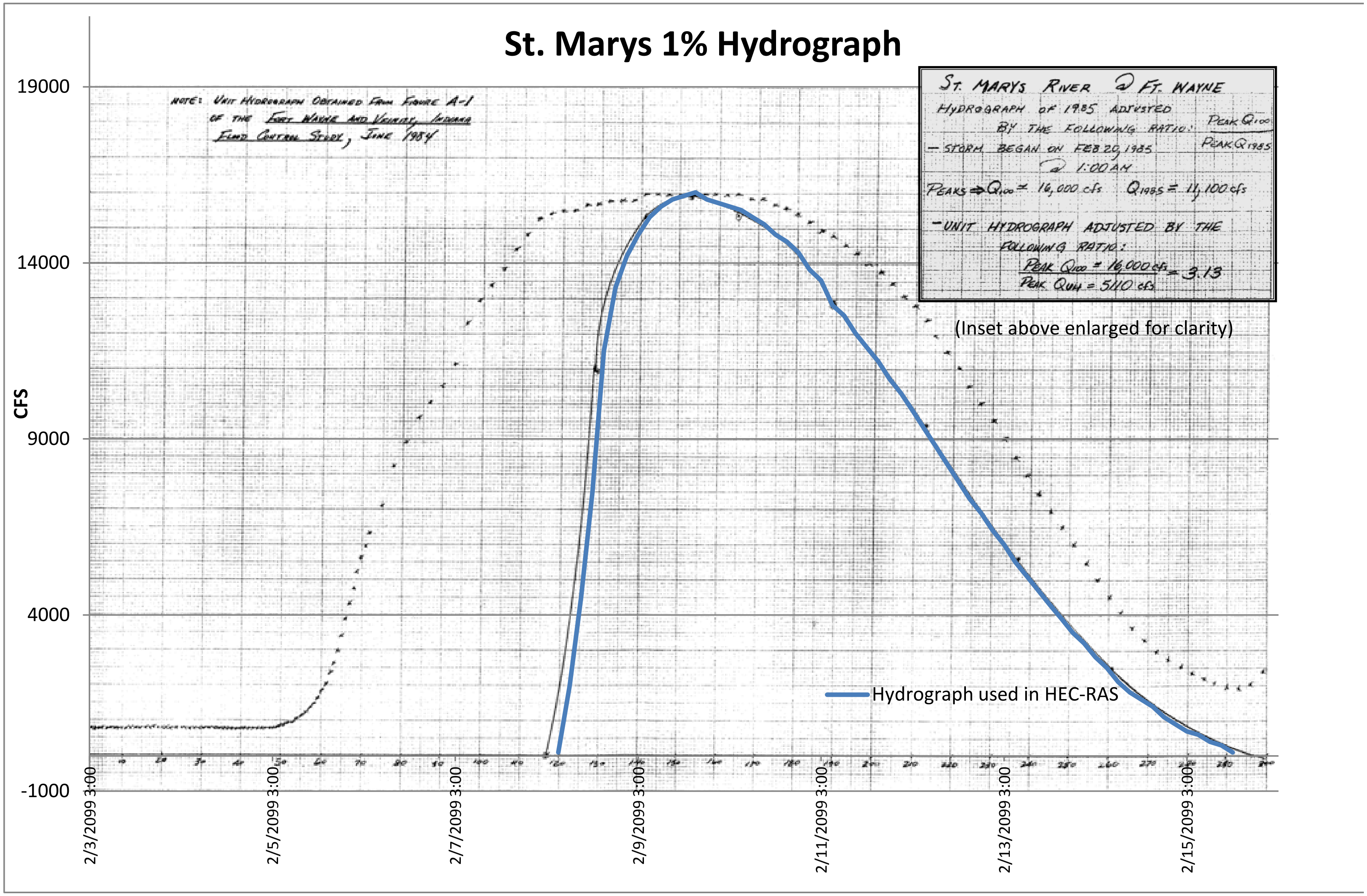


Figure 2.6: Theoretical St. Marys River 1% Annual Chance Event Hydrograph, as modeled per IDNR derivation

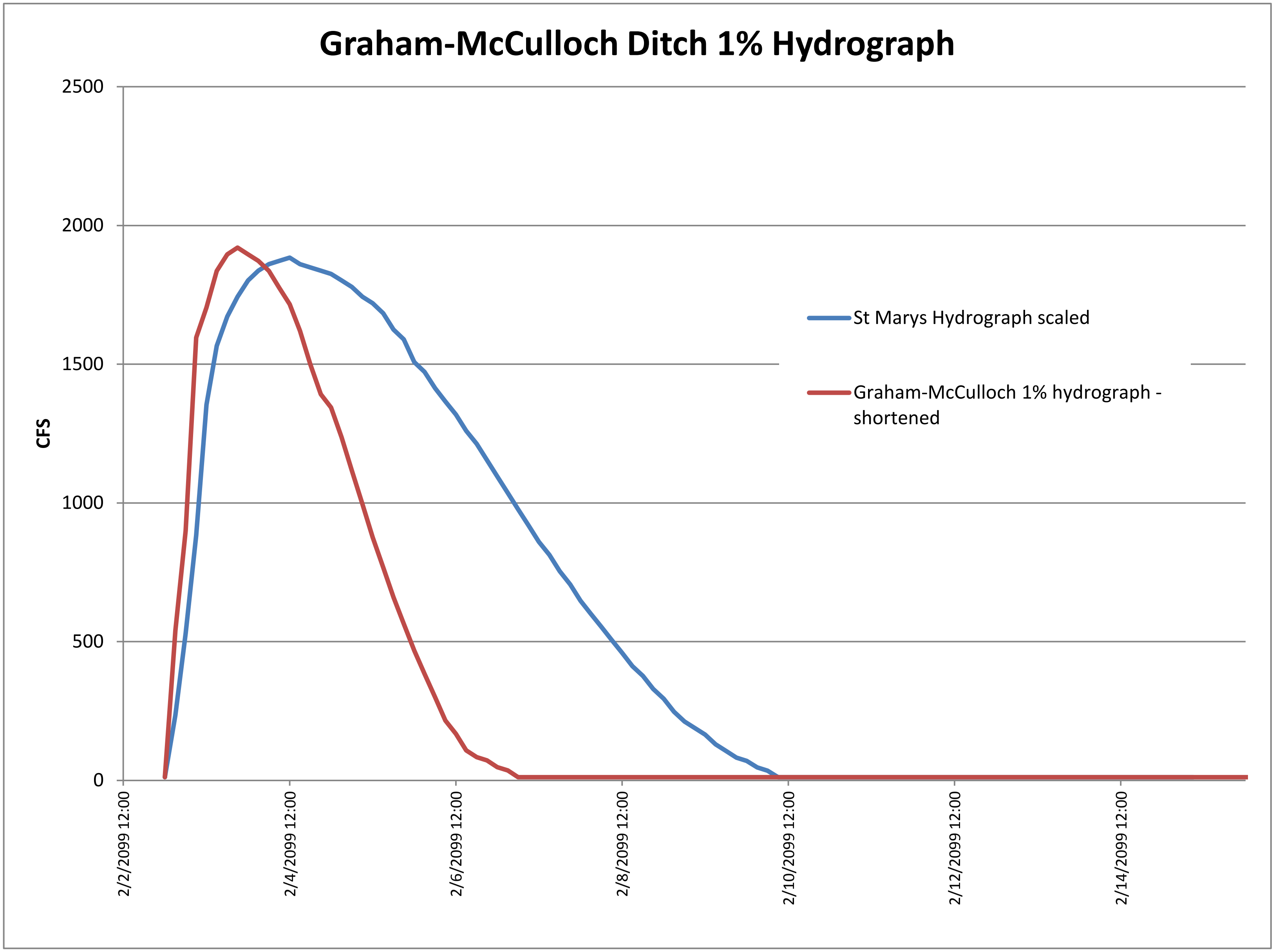


Figure 2.7: Theoretical Graham-McCulloch Ditch 1% Annual Chance Event Hydrograph, as modeled

In the hydraulic model, it was assumed that floods on the St. Marys River, St. Joseph River and the Graham-McCulloch Ditch watersheds will not occur or peak at the same time, due to the significantly different size of the watersheds; the drainage area of the Graham-McCulloch Ditch upstream of Eagle Marsh (near Engle Road) is approximately 12 square miles, whereas the drainage area of the St. Marys River at the confluence of the Junk Ditch is approximately 820 square miles. The St. Joseph River drains approximately 1086 square miles at its mouth. If a large rain event occurred on all of these watersheds, with the event typically progressing from west to east, flooding on the Graham-McCulloch Ditch will peak much sooner than from the same event occurring on the St. Marys and/or St. Joseph watersheds. In the hydraulic model, scenarios were analyzed assuming independent floods on either the St. Marys River or the Graham-McCulloch Ditch. Floods on the St. Josephs River were not modeled because the confluence with the St. Marys River was deemed sufficiently downstream of the mouth of the Junk Ditch that it would not impact backwater elevations as significantly as floods of the same frequency on the St. Marys River.

2.7 HYDRAULICS

2.7.1. Hydraulic Modeling

At the onset of the study, due to the emphasis on use of existing information to the extent possible and the initial accelerated schedule for the study, it was determined that modeling of the St. Marys overflow area would best be accomplished utilizing a HEC-RAS (version 4.1) unsteady flow model. HEC-RAS is a one-dimensional model that can be used to simulate both steady- and unsteady-flow situations. Use of a two-dimensional (2D) model was considered due to the complex flow patterns occurring between the two watersheds, but HEC-RAS was deemed adequate and preferred for the following reasons: 1) the flatness of the area would require a grid size for a 2D model to be small. When combined the large overall area to be modeled, it is expected that this would result in excessively long computation times to analyze a single event or alternative; 2) a number of alternatives were expected to be modeled, each requiring multiple changes to geometry to identify acceptable conditions. Geometry changes are much more difficult to make for each alternative in a 2D grid, requiring additional time for model setup; 3) Existing HEC-2 models were available for the St. Marys, St. Joseph, and Maumee Rivers from IDNR. HEC-RAS is the successor model to HEC-2, therefore it was expected that these models could be imported into HEC-RAS with minimal updates; 4) it was uncertain how typical 2D models would handle the multiple bridges on Junk Ditch that were expected to impact water surface profiles on Junk Ditch and the overflow into the Wabash watershed; and 5) in-house personnel had significant experience with HEC-RAS. A 2-D model would have likely required outsourcing to another District or Center of Expertise, other agency, or engineering firm, requiring significant time for coordination of scopes of work and contract negotiations. For these multiple reasons, modeling of this system with HEC-RAS was deemed the best alternative for this study.

HEC-RAS Model Schematic

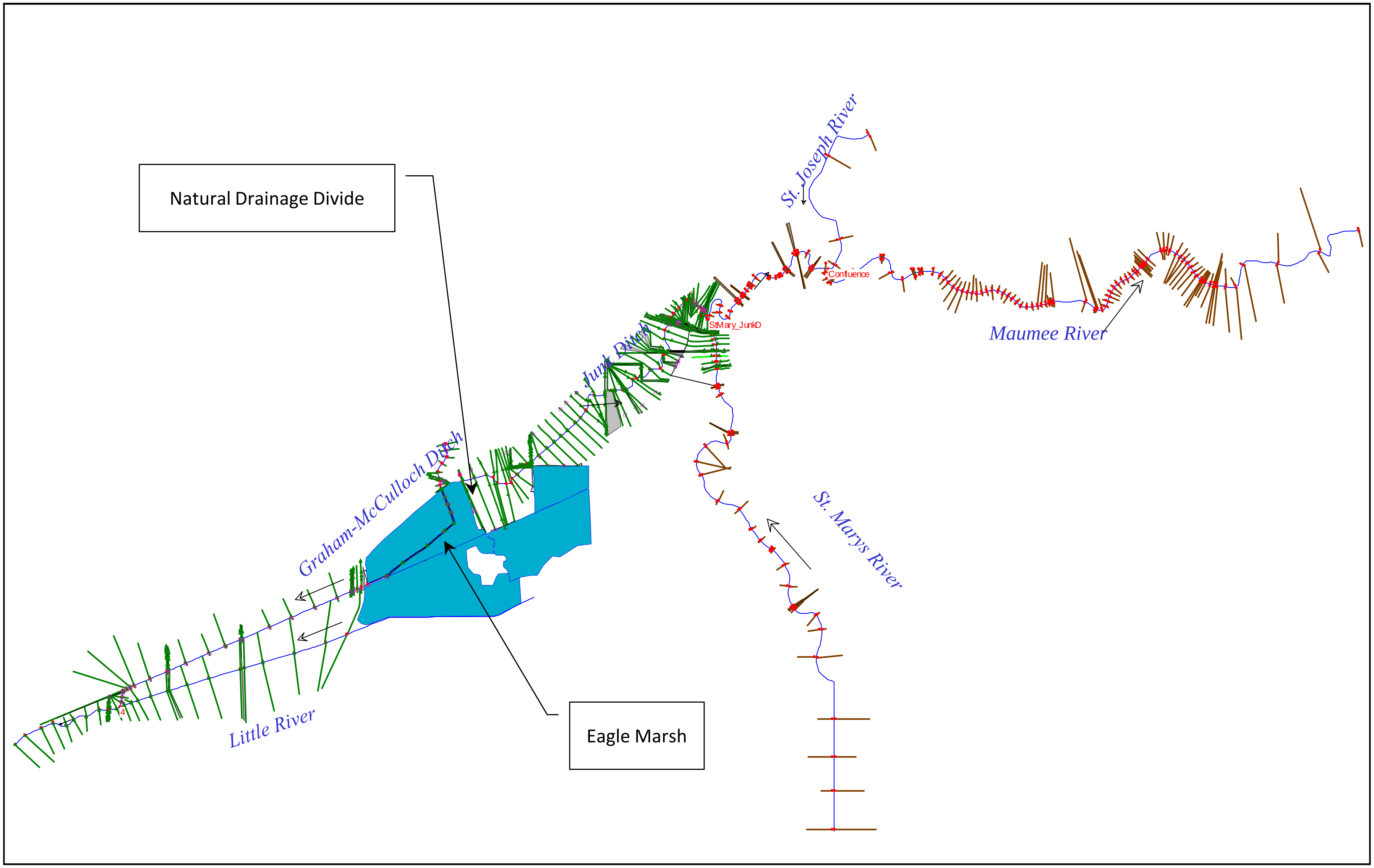


Figure 2.8: HEC-RAS model schematic of St. Marys Overflow and affected streams

The schematic of the stream network represented in the HEC-RAS model is shown in Figure 2.8. The HEC-RAS model that was developed represents the St. Marys River from the USGS gage location at approximately river mile 10.78 to its confluence with the St. Joseph River, and the Maumee River from this confluence to the gage location at river mile 129.08. The cross sections for the St. Marys and Maumee Rivers were imported from existing HEC-2 models developed by IDNR and most recently used in the 2009 Flood Insurance Study (FIS) update for Allen County. Initially, the St. Joseph River HEC-2 model was not available, so minimal required cross sections were cut from the 2009 Allen County LIDAR digital elevation model (DEM) to serve as a place holder representing the location of the St. Joseph confluence. The actual geometry of the St. Joseph River was not deemed crucial for the purposes of this study, but the river was merely included to create the junction point in HEC-RAS so that the St. Marys River and Maumee River stream mileage could be preserved in the model, and to provide a place for flows from the St. Joseph River to be input. The stream centerlines of the St. Marys, St. Joseph, and Maumee Rivers were partially geo-referenced to USGS quad sheets in the HEC-RAS model for visual purposes only, with the exception of the upper end of the St. Marys River beyond the bounds of Fort Wayne West quadrangle, which was not readily available. The data requirements and hydraulic computations for bridges in an HEC-RAS model is different than for HEC-2 models; therefore the bridges had to be re-coded manually after the existing HEC-2 models were imported. The new HEC-RAS model after conversion from HEC-2 produced profiles that closely matched the original HEC-2 calculated water surface elevation profiles. Due to the limited width of the existing HEC-2 model cross sections in reaches above the mouth of the Junk Ditch, it was later determined that the reach between the mouth of the Junk Ditch (St. Marys River mile 2.6) and new river mile 3.285 should be reprogrammed using updated mapping due to questions about the exchange of flow between the St. Marys River and Junk Ditch in the vicinity of Taylor Street; for this section, overbank geometry was cut from the 2009 Allen County LIDAR mapping DEM, and the stream invert ("thalweg") of the channel was approximated by interpolating the elevations and widths from the nearest HEC-2 models cross sections.

No existing geometry data was available for Junk Ditch, the Graham-McCulloch Ditch, or the Little River; therefore HEC-GeoRAS was used to cut overbank geometry from the 2009 LIDAR DEM, which had been augmented with additional survey points obtained from the NRCS survey of the Eagle Marsh property in 2006. As this DEM information stopped at the water surface that occurred at the time of the mapping, thalweg information was surveyed by Louisville District survey crews at most of the bridges on Junk Ditch and the Graham-McCulloch Ditch, and channel inverts were interpolated from this survey information for cross sections located between bridges.

The exchange of flow between the two watersheds is controlled by the elevation of an earthen berm that lines the left descending bank of the Graham-McCulloch Ditch. Please reference Figure 2.9 for a sample cross section of this channel, berm and overbanks. Floods on the St. Marys River and tributaries can cause flow to reverse in Junk Ditch and gradually fill in storage areas along the channel. The Junk Ditch channel will eventually "fill" to the elevation when it reaches a low area or "saddle point" at the head of Junk

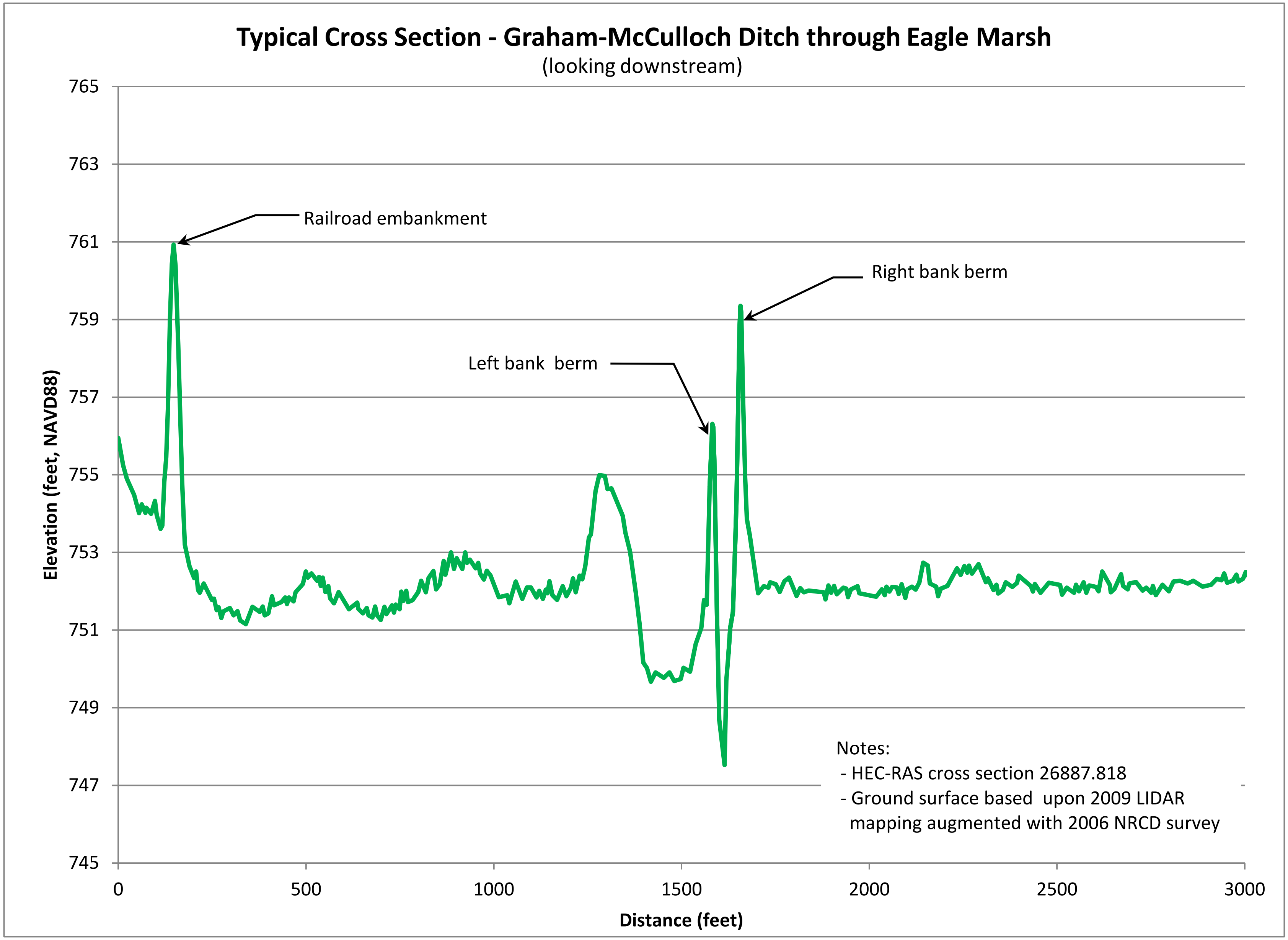


Figure 2.9: Typical Cross Section of Graham-McCulloch Ditch through Eagle Marsh, depicting confining berms on both banks

Ditch, at the east end of Eagle Marsh, which normally serves as the natural watershed divide. The watershed divide then acts as a weir, allowing flow into low areas bounded by the railroad berm to the south and the Graham-McCulloch left bank berm to the north and west, which eventually ties into the railroad berm near the west end of the Eagle Marsh. Conversely, floods on the Graham-McCulloch Watershed must generally overtop the left descending bank berm, fill the low Eagle Marsh south storage (EMSS) areas, and then "overtop" the natural watershed divide to flow into the Junk Ditch. Four known 18" corrugated metal drain tiles exist through the left descending bank berm, which connect the EMSS to the Graham-McCulloch Ditch. One of these pipes to the east of the existing IDNR barrier fence was previously buried in riprap to prevent its use as a conduit for fish species during floods. Due to large head differentials observed during floods in the spring of 2011, the riprap covering the pipe entrance was dislodged due to the high flows through the pipe. At the time of this report, IDNR has stated their plans to plug and fill this pipe in the fall of 2011. Because of the observed poor condition of these pipes, their effectiveness as a conduit for both flow and aquatic species is considered questionable, so the pipes were not included in the HEC-RAS model. Definition of the left bank berm elevations came from survey data by the NRCS from 2006. This data was deemed to be a better representation of the berm elevations than the elevations from the 2009 LIDAR mapping; however, deterioration (e.g., erosion or sloughing) of the left bank berm may have occurred in the interim period, which is thus not captured in the model. In the HEC-RAS model, this berm is a lateral weir structure with a weir coefficient (C_d) of 2.0, a value lower than normal, as defined by engineering judgment, to represent the reduced efficiency of flow in a lateral direction. Due to the lack of gage data on the Graham-McCulloch Ditch, further refinement of this value through calibration could not be performed. In HEC-RAS, the natural watershed divide is also modeled as a lateral weir connected to the last cross section on the Junk Ditch and the EMSS.

Along Junk Ditch and in the vicinity of Eagle Marsh, the railroad and other surface features can act as weirs that allow flow to occur laterally away from the channel, either into isolated areas where storage can occur, or permit the exchange of flow between channels. These lateral weir elevations were taken from the 2009 DEM either using automated tools in HEC-GeoRAS or manually entered from survey information, where available. Storage area elevation-volume curves were computed using HEC-GeoRAS tools within ArcGIS using the 2009 DEM. The existing 4' diameter culvert through the railroad embankment was surveyed and included as the connection between water surface elevations in Eagle Marsh and the Fox Island County Park, which also provides additional storage during floods. There are no apparent connections to the Little River under Yohne Road to the south of the Fox Island County Park.

Initial flows on each stream were set equal to expected low flow values, based upon engineering judgment and preliminary review of flow data, where available. Stability of the numerical model also influenced these minimum values in certain cases. Tables 2.4a and 2.4b summarize the boundary conditions and initial flows for events on the St. Marys River and Graham-McCulloch Ditch respectively. It is assumed that no antecedent storms have reduced the available storage in the storage areas and that they are nearly dry, with the initial ponded elevations set just above the minimum elevation on the elevation-storage curve. The boundary conditions are defined as constant low flows equal to the initial flows for all streams except for the flood event hydrograph which is introduced on either the St. Marys River or Graham-McCulloch Ditch. Please refer to Section 2.6.2 for further discussion of the flow hydrographs used.

TABLE 2.4a
MODEL BOUNDARY CONDITIONS FOR
FLOOD EVENT ON ST. MARYS RIVER

Reach	Reach Location	Boundary Type	Input Flow (cfs)
Graham-McCulloch Ditch	Upstream	Base Flow hydrograph	25
Junk Ditch	Upstream	Base Flow hydrograph	25
Little River	Upstream	Base Flow hydrograph	10
Little River	Downstream	Normal Depth	-
Maumee River	Downstream	Normal Depth	-
St. Joseph River	Upstream	Base Flow hydrograph	2000
St. Marys River	Upstream	Flow Hydrograph	Varies

TABLE 2.4b
MODEL BOUNDARY CONDITIONS FOR
FLOOD EVENT ON GRAHAM-McCULLOCH DITCH

Reach	Reach Location	Boundary Type	Input Flow (cfs)
Graham-McCulloch Ditch	Upstream	Base Flow hydrograph	Varies
Junk Ditch	Upstream	Base Flow hydrograph	25
Little River	Upstream	Base Flow hydrograph	50
Little River	Downstream	Normal Depth	-
Maumee River	Downstream	Normal Depth	-
St. Joseph River	Upstream	Base Flow hydrograph	200
St. Marys River	Upstream	Flow Hydrograph	2000

2.7.2. Calibration

The 1982 flood event was chosen as a calibration event due to the well documented high water mark information collected by IDNR for this significant flood. An additional high water mark located inside the Little River Wetlands Project (LRWP) maintenance barn at Eagle Marsh for the 1982 flood was surveyed by Louisville District survey personnel. There is some conflicting information on the hydrograph used: a review of IDNR files

TABLE 2.—*Summary of peak stage and discharge data*—Continued

[R, river; nr, near; Cr, creek; mi², square miles; ft, feet; ft³/s, cubic feet per second; (ft³/s)/mi², cubic feet per second per square mile; dashes indicate missing or unknown value]

Site number	Station number	Station name	Contributing drainage area (mi²)	Period of record (water years)	Maximum flood previously recorded: Date	Maximum flood previously recorded: Gage height (ft)	Maximum flood previously recorded: Discharge (ft³/s)	Maximum flood previously recorded: Discharge (ft³/s)/mi²	Maximum during 1982 flood: Date	Maximum during 1982 flood: Gage height (ft)	Maximum during 1982 flood: Discharge (ft³/s)	Maximum during 1982 flood: Discharge (ft³/s)/mi²	Recurrence interval (years)
		St. Joseph River Basin—Continued											
40	04100500	Elkhart R at Goshen, Ind.	594	1932–82	4- 4–50	10.15	5,440	9	3–14	11.94	6,180	10	[1]80
41	04101000	St. Joseph R at Elkhart, Ind.	3,370	1948–82	4- 5–50	27.82	18,400	5	3–21	27.91	18,600	6	[1]45
42	04101500	St. Joseph at Niles, Mich.	3,666	1931–82	4- 5–50	15.10	20,200	6	3–21	14.97	19,900	5	[1]45
43	04101800	Dowagiac R at Sumnerville, Mich.	255	1961–82	6–26–68	8.78	1,280	5	3–17	8.33	1,150	5	15
44	04102320	Paw Paw R nr Paw Paw, Mich.	195	1981–82	5–11–81	5.73	1,230	6	3–14	6.26	1,540	8	---
45	04102420	Paw Paw R nr Hartford, Mich.	311	1981–82	2–21–81	9.76	1,760	6	3–17	10.37	2,500	8	---
46	04102500	Paw Paw R at Riverside, Mich.	390	1952–82	3- 9–79	10.11	2,830	7	3–18	10.11	2,650	7	45
		River Raisin Basin											
47	04176000	River Raisin nr Adrian, Mich.	463	1933–38 1954–82	4–30–56	14.87	5,580	12	3–15	15.77	6,660	14	>100
48	04176400	Saline R nr Saline, Mich.	94.6	1966–82	6–26–68	13.37	3,990	42	3–14	11.84	1,990	21	10
49	04176500	River Raisin nr Monroe, Mich.	1,042	1938–82	9- 6–81	10.22	14,500	14	3–15 3–16	11.16 ---	Ice jam 15,300	15	50
		Maumee River Basin											
50	04177720	Fish Creek at Hamilton, Ind.	37.5	1970–82	3–23–78	10.79	497	13	3–17	11.52	603	16	35
51	04178000	St. Joseph R nr Newville, Ind.	610	1947–82	4- 6–50	17.05	9,710	16	3–17	17.96	9,190	15	[1]50
52	04179000	St. Joseph R at Cedarville, Ind.	763	1956–82	3–24–78 5- 1–56	18.62 ---	--- 10,100	13	3–17	21.94	14,500	19	[1]>100
53	04179500	Cedar Cr at Auburn, Ind.	87.3	1943–82	4- 5–50	9.90	1,520	17	3–14	10.63	2,100	24	[1]>100
54	04179510	Cecil Metcalf ditch nr Auburn, Ind.	.78	1973–82	6–13–81	---	90	115	3–14	10.50	140	179	20
55	04180000	Cedar Cr nr Cedarville, Ind.	270	1947–82	4- 5–50	11.67	4,870	18	3–14	12.98	5,340	20	[1]45
56	04181500	St. Marys R at Decatur, Ind.	621	1947–82	2–10–59	24.22	11,300	18	3–14	24.40	10,900	18	[1]20
57	04182000	St. Marys R nr Fort Wayne, Ind.	762	1931–82	2–11–59	19.42	13,600	18	3–14	19.66	12,600	17	[1]25
58	04182590	Harber ditch at Fort Wayne, Ind.	21.9	1965–82	6–13–81	11.67	916	42	3–14	12.25	900	41	10
59	04182900	Maumee R at Fort Wayne, Ind.	1,926	1907–82	3–26–13	26.10	---	---	3–17	25.93	---	---	---
60	04183000	Maumee R at New Haven, Ind.	1,967	1947–82	3–24–78	23.58	22,400	11	3–17	25.49	26,600	14	[1]80
61	04183500	Maumee R at Antwerp, Ohio	2,129	1922–82	5–20–43	20.29	26,200	12	3–17	21.70	26,100	12	[1]40
62	04185000	Tiffin R at Stryker, Ohio	410	1922–28 1941–82	4–25–50	15.45	6,640	16	3–15	18.36	7,760	19	50
63	04186500	Auglaize R nr Fort Jennings, Ohio	332	1922–36 1941–82	1–23–59	20.30	12,000	36	3–13	15.05	5,850	18	<5
64	04187500	Ottawa R at Allentown, Ohio	160	1924–35 1943–82	1–22–59	10.88	7,740	48	3–12	8.70	3,640	23	<5
65	04189000	Blanchard R nr Findlay, Ohio	346	1924–36 1941–82	6–14–81	17.43	13,000	38	3–13	12.35	6,320	18	<5
66	04191500	Auglaize R nr Defiance, Ohio	2,318	1916–82	2–13–59 2–16–50 2–12–59	27.65 --- ---	Ice jam 52,500 52,500	23 23	3–14	27.39	52,300	23	20
67	04192500	Maumee R nr Defiance, Ohio	5,545	1925–36 1939–75 1979–82	2–16–50	13.70	87,100	16	3–15	15.87	104,000	19	>100
68	04193500	Maumee R at Waterville, Ohio	6,330	1900–01 1922–36 1939–82	2–16–50	14.52	94,000	15	3–15	17.18	120,000	19	90
		Kankakee River Basin											
69	05515000	Kankakee R nr North Liberty, Ind.	116	1951–82	6–27–68 6–14–81	9.04 ---	--- 780	7	3–17	9.01	908	8	[1]>100
70	05515400	Kingsbury Cr nr LaPorte, Ind.	3.01	1971–82	7–26–81	6.83	73	24	3–13	6.31	63	21	10
71	05515500	Kankakee R at Davis, Ind.	400	1932–82	7–29–81	12.52	1,580	4	3–17 3–20	12.98 ---	--- 1,920	5	[1]>100
72	05516000	Yellow R nr Bremen, Ind.	131	1955–82	5–15–78	17.68	2,750	21	3–16	15.17	2,800	21	[1]>100

Figure 2.10: Excerpt from USGS Water Supply Paper, 1982 Flood

found a hydrograph plotted for this event, but the peak flows on this hydrograph do not match USGS and other IDNR documentation that the flood was created as a result of a 4% annual chance event on the St. Marys River, followed closely by a 1% annual chance event on the St. Joseph River. Figure 2.10 is taken from the USGS Water Supply Paper on the 1982 flood event. This latter combination of floods was chosen for hydraulic modeling of this event based upon better documentation supporting this scenario and the better results obtained in the model. Figures 2.11 and 2.12 show the water surface profiles for the St. Marys River and Junk Ditch respectively, where high water mark information was well documented. This model closely reproduced these high water marks, particularly on the lower Junk Ditch and St. Marys River. At Eagle Marsh, the computed water surface elevation is approximately 0.6' higher than the observed high water mark in the LRWP barn.

Little additional information was available for validation of the model to an additional storm. To that end, documentation of any flood observations was requested of LRWP personnel. Photos were provided for four different events where the entrance road to the

St. Marys River Profile – 1982 Calibration Event

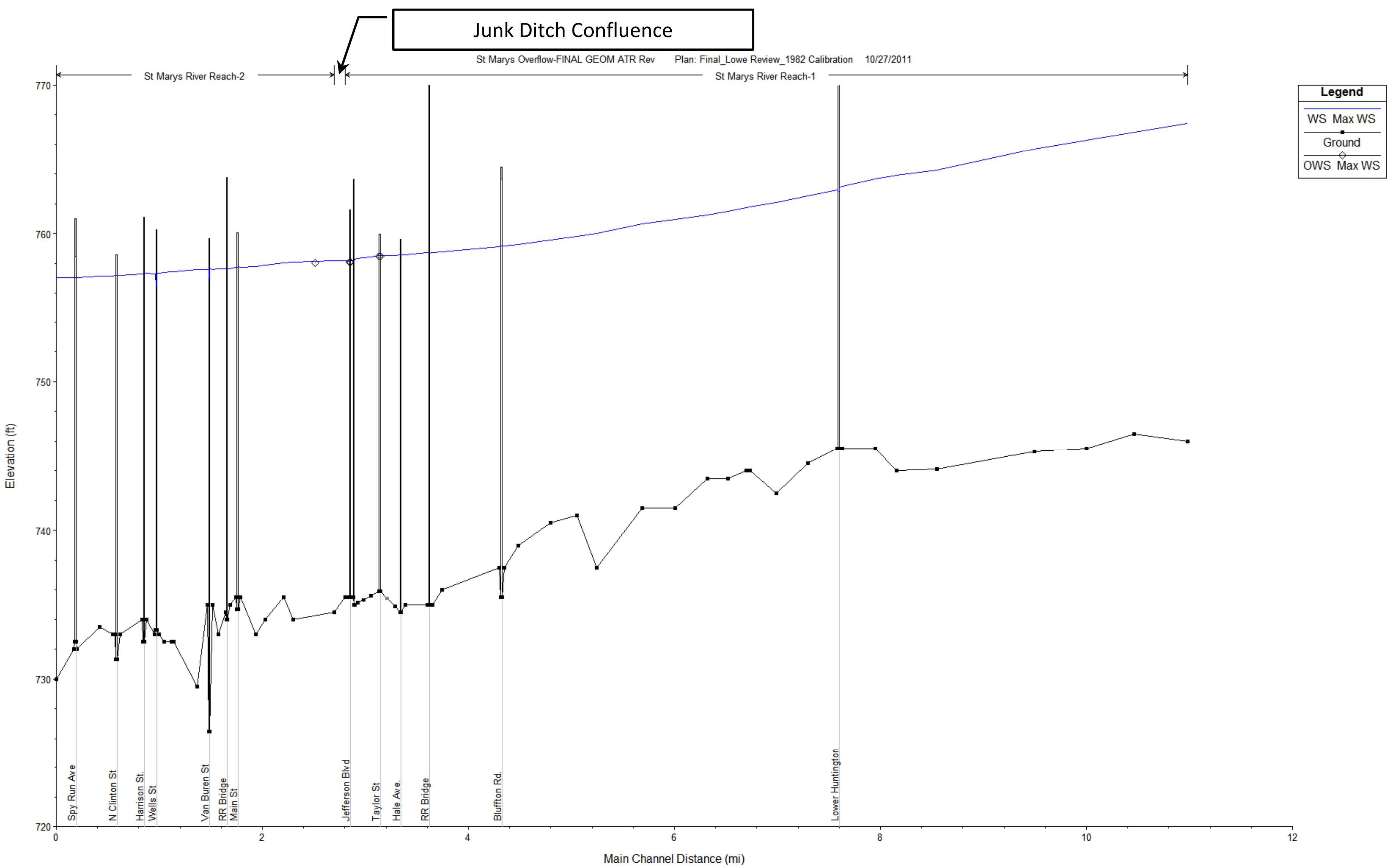


Figure 2.11: Calibration results for 1982 flood event – St. Marys River

Junk Ditch Profile – 1982 Calibration Event

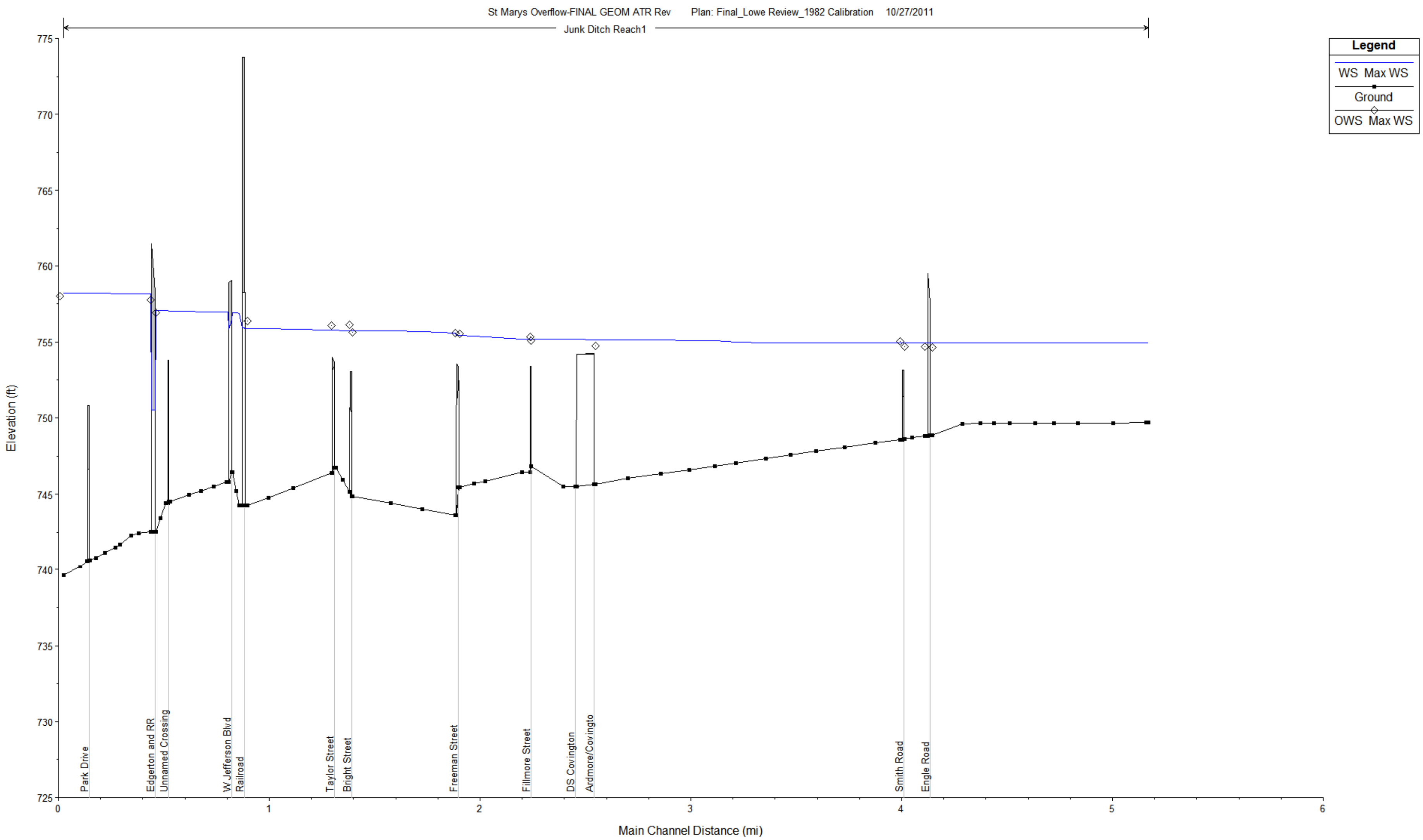


Figure 2.12: Calibration results for 1982 flood event – Junk Ditch

Photo 2.1: Flooding at Eagle Marsh Entrance Road, March 10, 2009. (Photo provided by Betsy Yankowiak, LRWP)

LRWP barn was overtopped and thus prevented access: 21 August 2007; 17 February 2009; 10 March 2009; and 31 March 2010. Based upon review of a new gages on the St. Marys and Maumee Rivers in downtown Fort Wayne, of these events only the 10 March 2009 flood event at Eagle Marsh appeared to be a result of backwater flooding from the St. Marys River (see Photo 2.1). Based upon examination of the DEM, it is estimated that the water surface elevation in the picture is approximately 752.5, referencing NAVD88. Modeling of this event reproduced very closely the peak gage elevation of 753.05 on 13 March 2009 at the Anthony Boulevard gage. At Eagle Marsh, the model predicts a peak elevation of approximately 753.7 several days after the photograph was taken, however it is unknown how water surface elevations reacted at Eagle Marsh during this period. It is also unknown what role, if any, flooding on the Graham-McCulloch may have played during this event; the model does not assume any significant flows on the Graham-McCulloch during this event. Based upon the data available, it is believed that the model generally reproduces the hydraulics of this system for this flood event.

2.7.3. Suggested Model Improvements

As discussed previously, the understanding of flows on the Graham-McCulloch Ditch is limited due to the absence of any gage information and detailed hydrologic analysis performed for this watershed. At a minimum, a detailed hydrologic model such as HEC-HMS should be developed for the watershed, preferably coupled with the installation of one or more flow gages in the vicinity of Eagle Marsh so that the hydrologic model may be calibrated. Ideally two gages would be installed, one upstream of Eagle Marsh to define flows strictly from the Graham-McCulloch headwaters, and a second gage near the I-69 Bridge which, when compared with the first gage upstream, could estimate flows that may be entering or leaving the Graham-McCulloch watershed by crossing the left bank berm. Other configurations of gages may also provide similar information.

The Taylor Street overflow area which connects of the Junk Ditch and the St. Marys River upstream of the mouth of Junk Ditch, appears to play a significant role in determining the amount of flow that enters the Junk Ditch and utilizes storage within that watershed, thereby affecting the frequency that the Graham-McCulloch left bank berm is overtopped. The current modeling of this area controlled by a lateral weir at the railroad embankment appears to be conservative in that it may allow greater flows in the model to pass to Junk Ditch than may actually occur. Further study and alternative means of modeling this area should be explored.

2.8. BASELINE CONDITIONS

With the model developed to the extent possible given the available information and resources, the model was run for eight standard design flood frequencies: 0.2%, 1%, 2%, 4%, 10%, 20%, 50% and 99% annual chance events. As stated previously, the floods were each run independently on the St. Marys River and the Graham-McCulloch Ditch. Figures 2.13a through 2.13c show the maximum water surface elevation profiles for the St. Marys River, Junk Ditch, and Graham-McCulloch Ditch respectively for these flood events occurring on the St. Marys River. Figures 2.13d through 2.13f depict the same frequency events if the floods occurred on the Graham-McCulloch Ditch. These profiles are considered the baseline conditions to which alternatives for preventing the transfer of aquatic nuisance species will be compared. It should be noted in Figure 2.13c that the water surface profiles on the Graham-McCulloch Ditch are coincident for the 4%, 10%, 20%, 50% and 99% annual chance events on the St. Marys River because the left bank berm is not overtopped until approximately a 3% annual chance event on that watershed. Likewise, the Junk Ditch profiles in Figure 2.13e for the 10%, 20%, 50%, and 99% annual chance events are coincident as a 10% annual chance event on the Graham-McCulloch does not overtop the left bank berm. Figures 2.14a and 2.14b illustrate water surface elevations in the vicinity of the Eagle Marsh for select representative events relative to the Graham-McCulloch Ditch left bank berm. For reference, Figures 2.15a and 2.15b illustrate the approximate depths of flow and extents of inundation in the vicinity of Eagle Marsh for a 1% annual chance event occurring on the St. Marys River or the Graham-McCulloch Ditch. Please note when examining these depth charts that the depths have been broken into classifications for easier interpretation, but this results in a conservative representation; for example, an actual depth of 1.6 feet will appear the same as a depth of 3 feet because they have been classified in the group of depths of 1.5' – 3'.

It is not assumed that a flood on the St. Joseph River will occur coincident with the St. Marys or Graham-McCulloch events. While the St. Marys and St. Joseph watersheds are similar in size, the geographic areas they encompass are significantly separated; the headwaters of each stream are separated by over 100 miles. It is not believed to be likely that a storm of extreme intensity or duration would encompass both watersheds, producing peaks that would reach the confluence simultaneously. Preliminary analysis

Baseline Water Surface Profiles – St. Marys River Flood Events

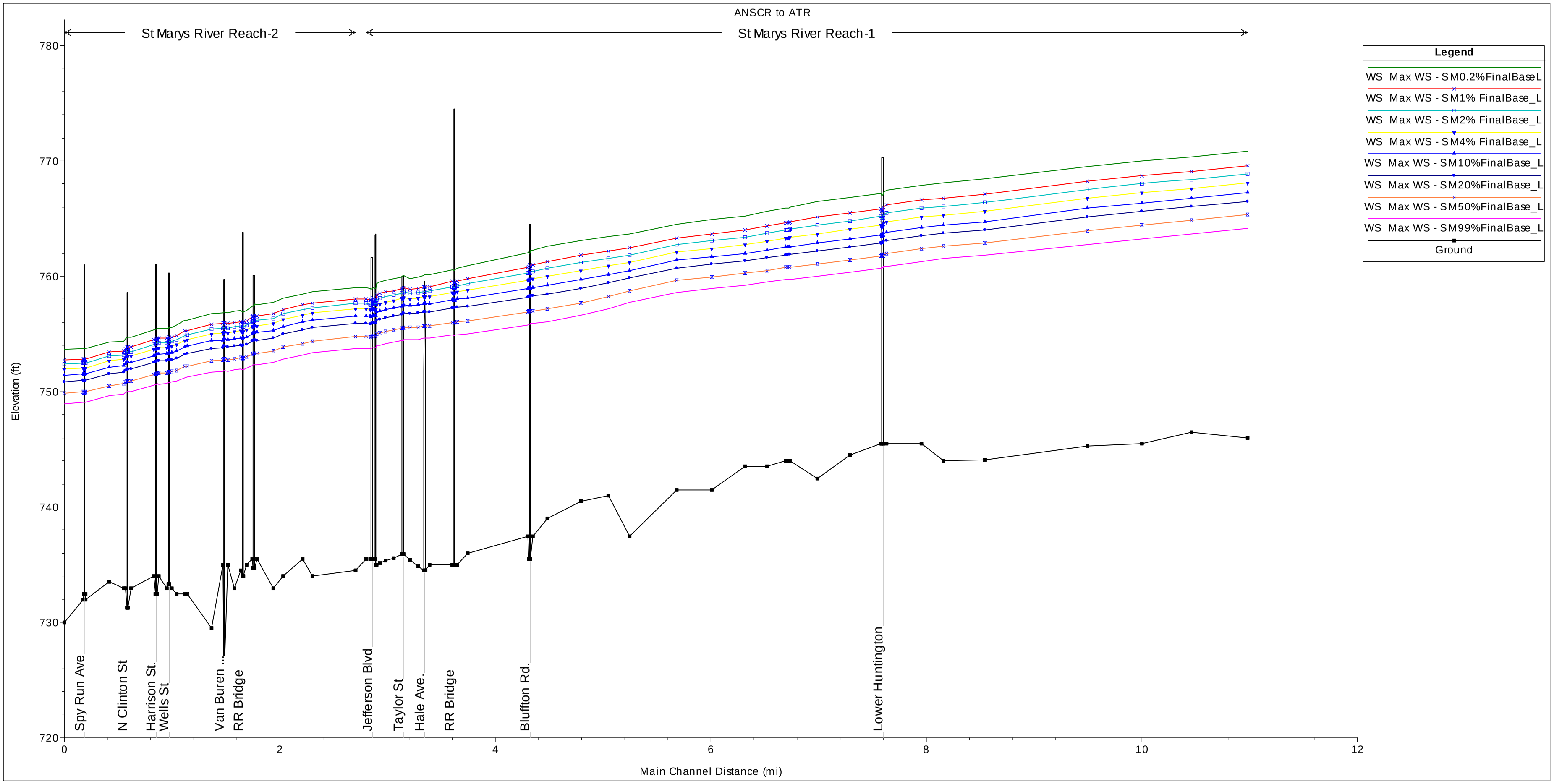


Figure 2.13a: Baseline water surface profiles for floods on the St. Marys River

Baseline Water Surface Profiles – St. Marys River Flood Events

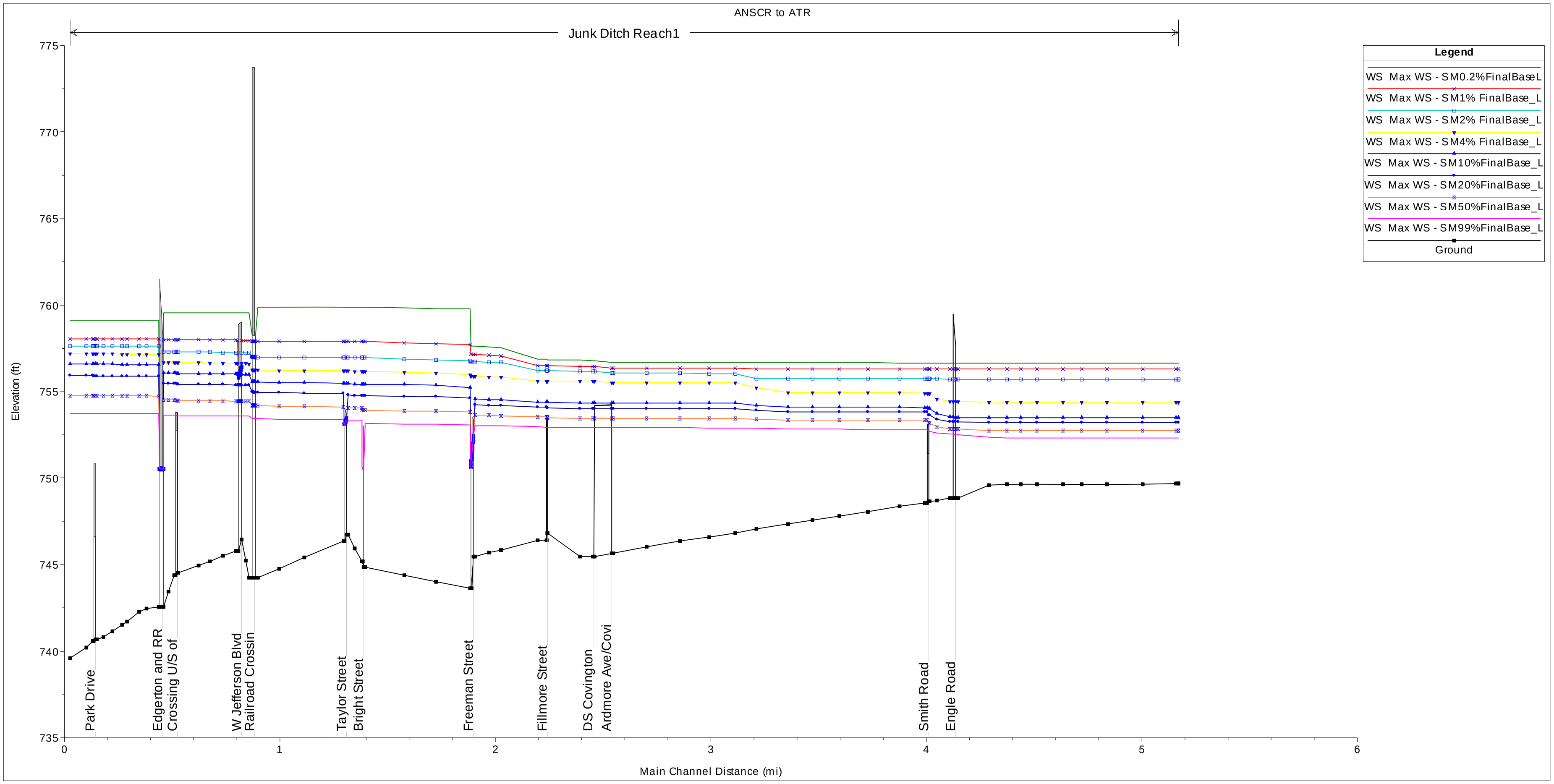


Figure 2.13b: Baseline water surface profiles on Junk Ditch for floods on the St. Marys River

Baseline Water Surface Profiles – St. Marys River Flood Events

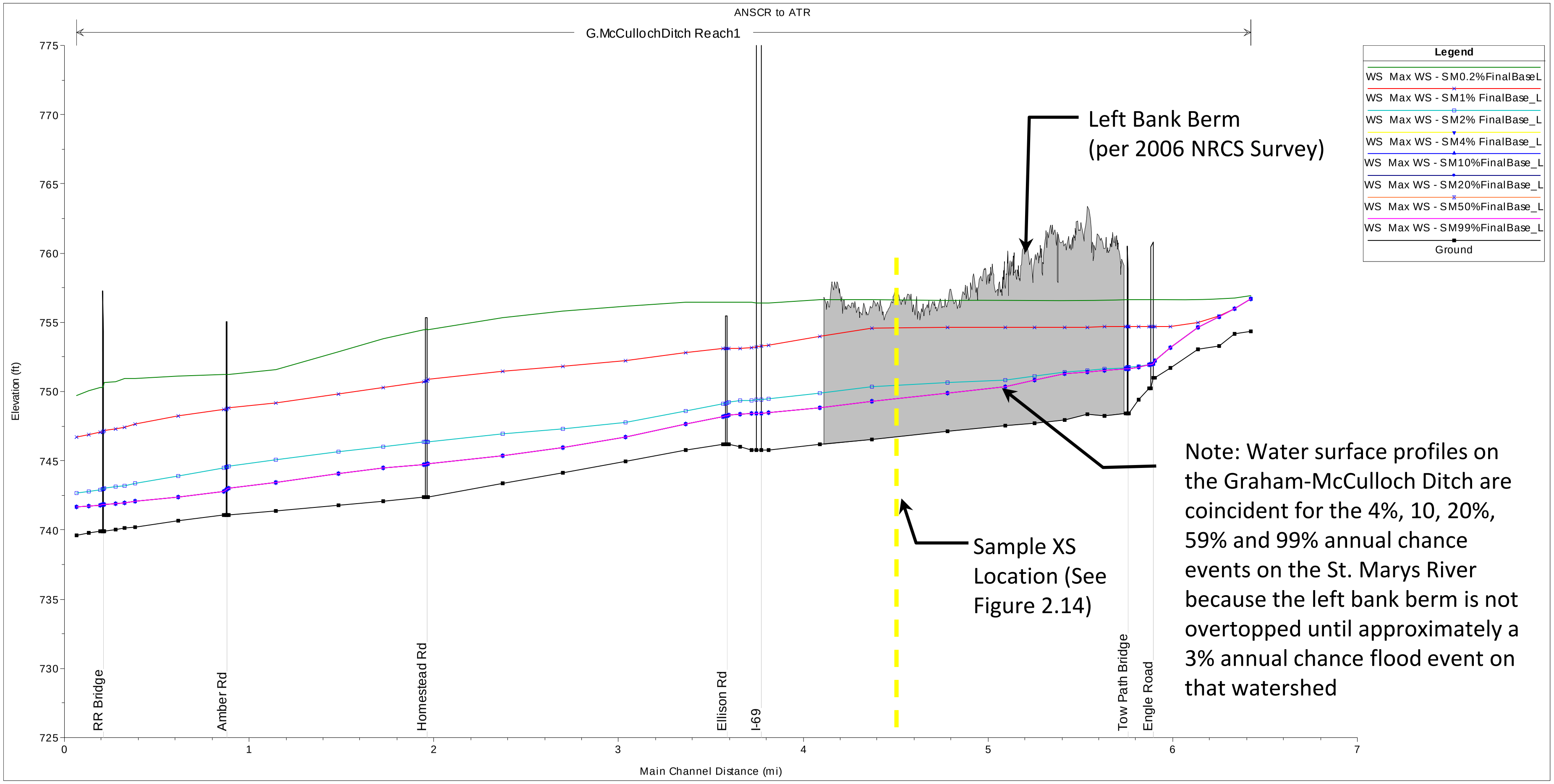


Figure 2.13c: Baseline water surface profiles on Graham-McCulloch Ditch for floods on the St. Marys River

Baseline Water Surface Profiles – Graham-McCulloch Ditch Flood Events

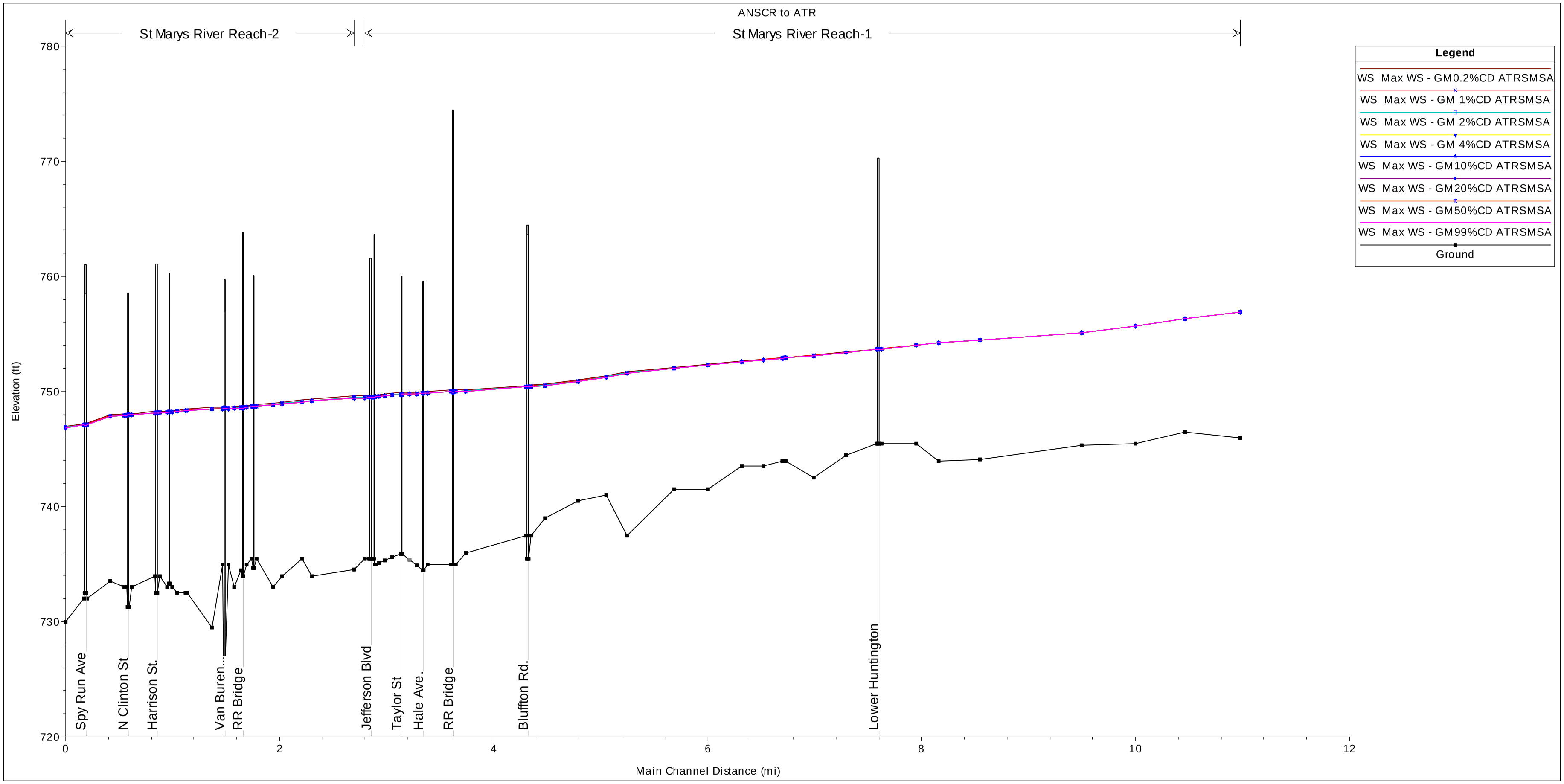


Figure 2.13d: Baseline water surface profiles on the St. Marys River for floods on the Graham-McCulloch Ditch

Baseline Water Surface Profiles – Graham-McCulloch Ditch Flood Events

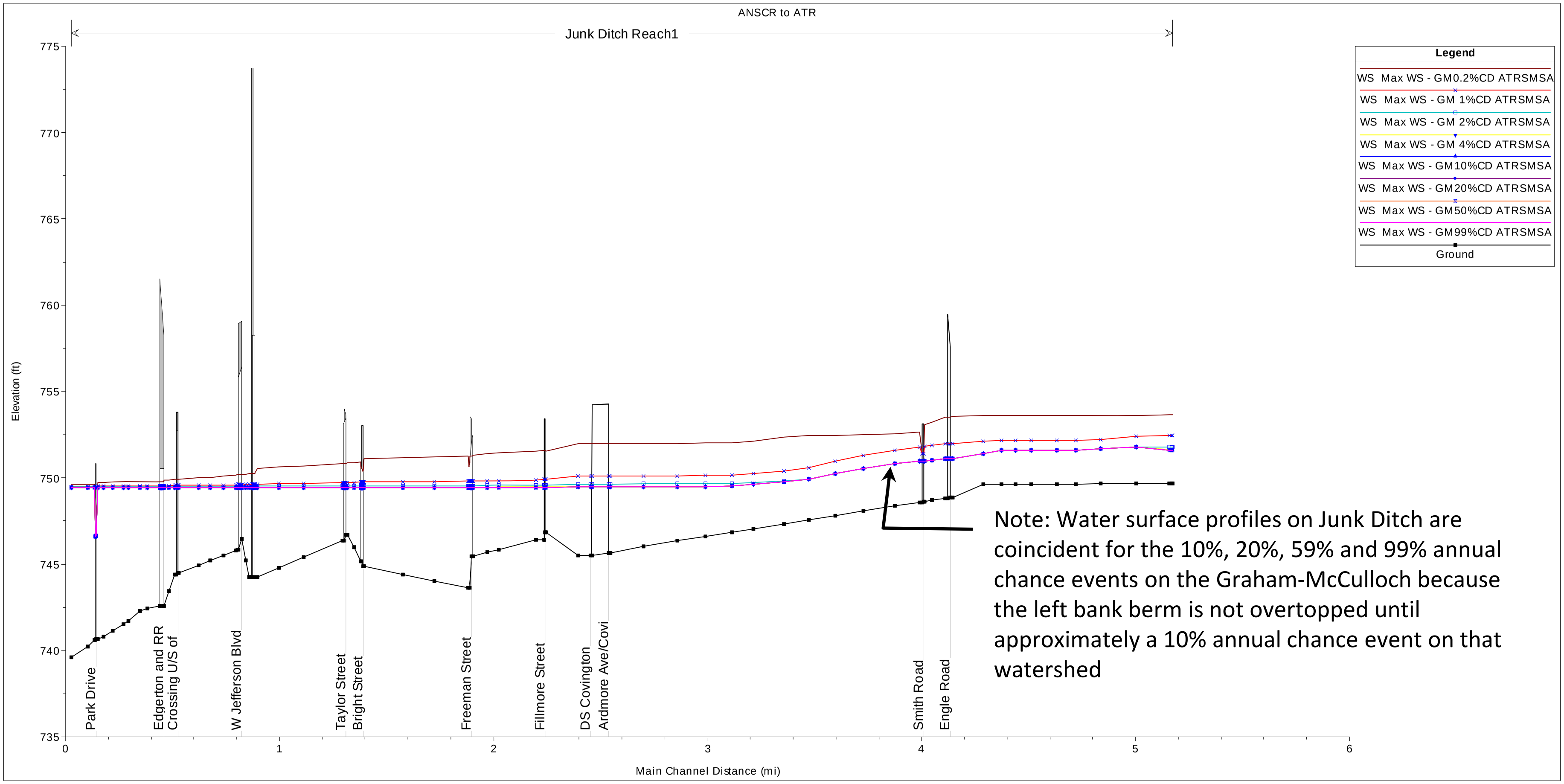


Figure 2.13e: Baseline water surface profiles on Junk Ditch for floods on the Graham-McCulloch Ditch

Baseline Water Surface Profiles – Graham-McCulloch Ditch Flood Events

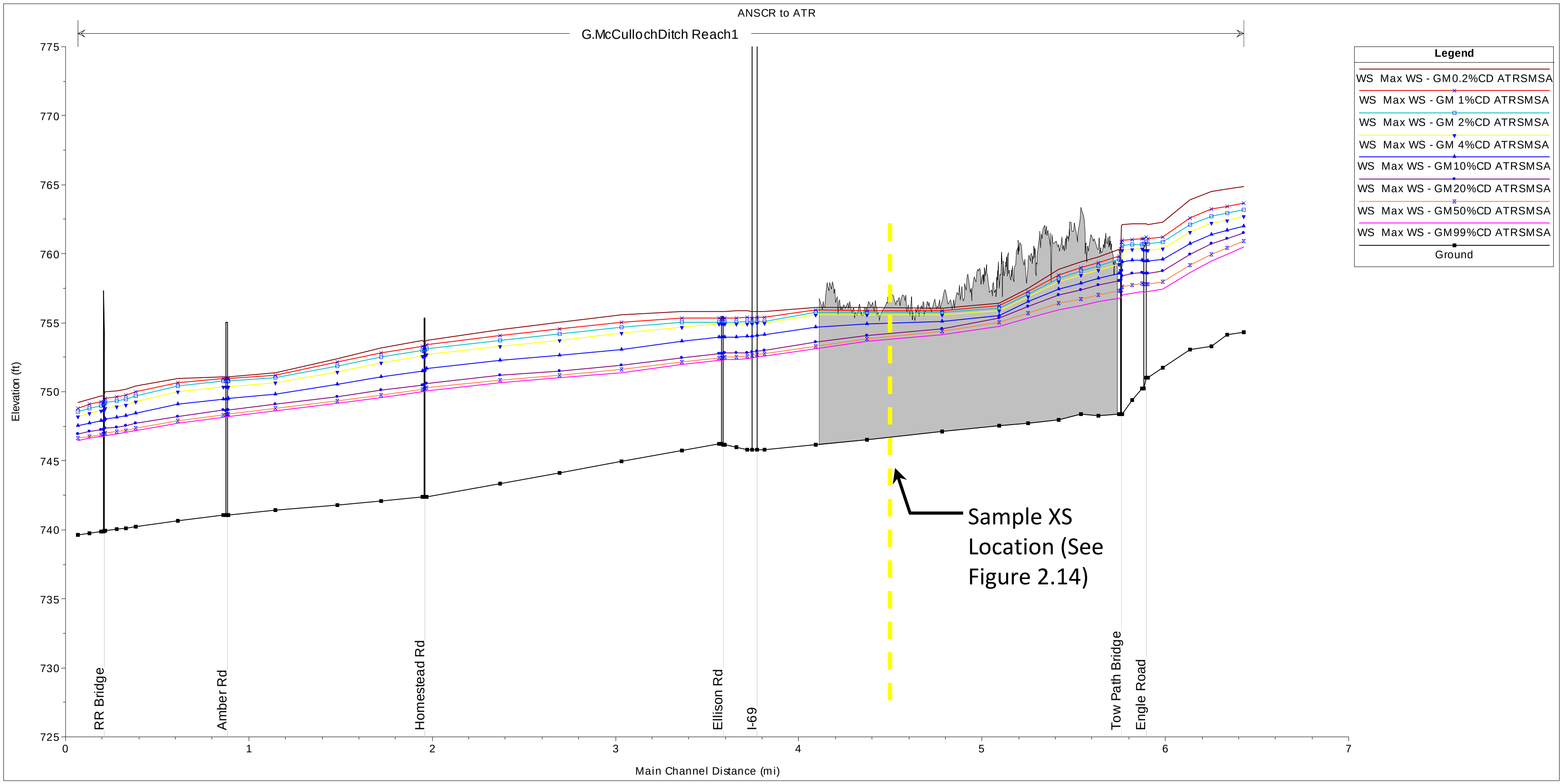
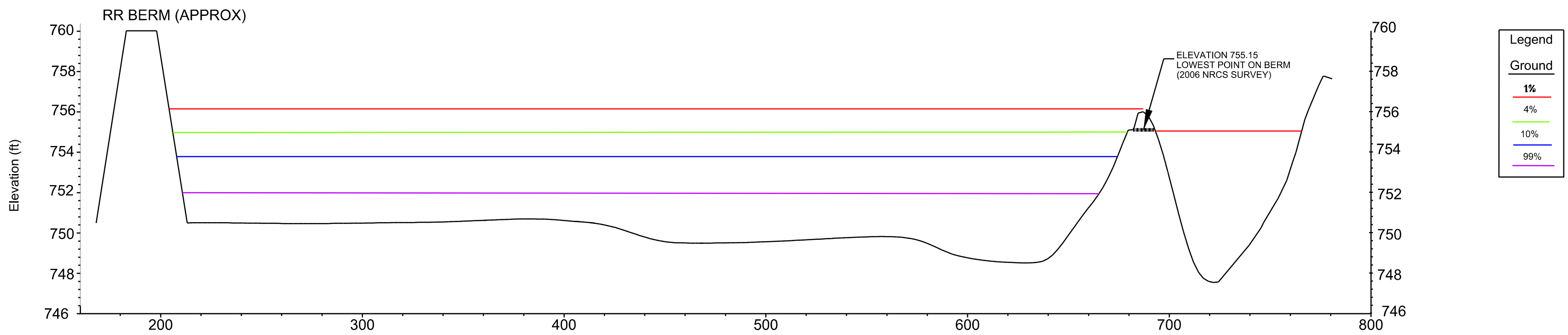
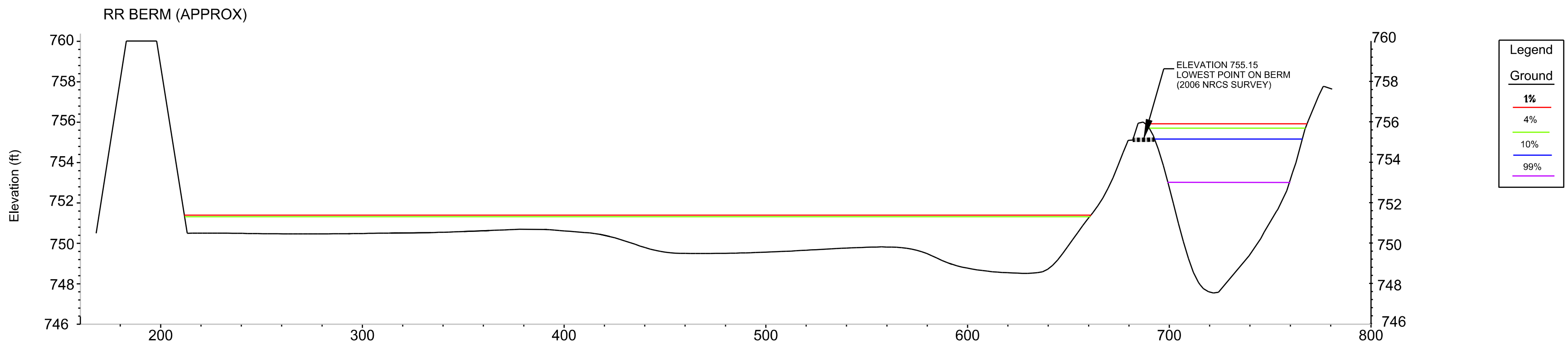


Figure 2.13f: Baseline water surface profiles on Graham-McCulloch Ditch



SAMPLE CROSS SECTION
LEFT OVERBANK GRAHAM-McCULLOCH DITCH
FLOW DEPTH FOR SELECT FLOODS OCCURING
ON ST. MARYS RIVER WATERSHED



SAMPLE CROSS SECTION
LEFT OVERBANK GRAHAM-McCULLOCH DITCH
FLOW DEPTH FOR SELECT FLOODS OCCURING
ON GRAHAM-McCULLOCH DITCH WATERSHED

Figure 2.14a: Sample Cross Section through Eagle Marsh left bank berm and southern storage area

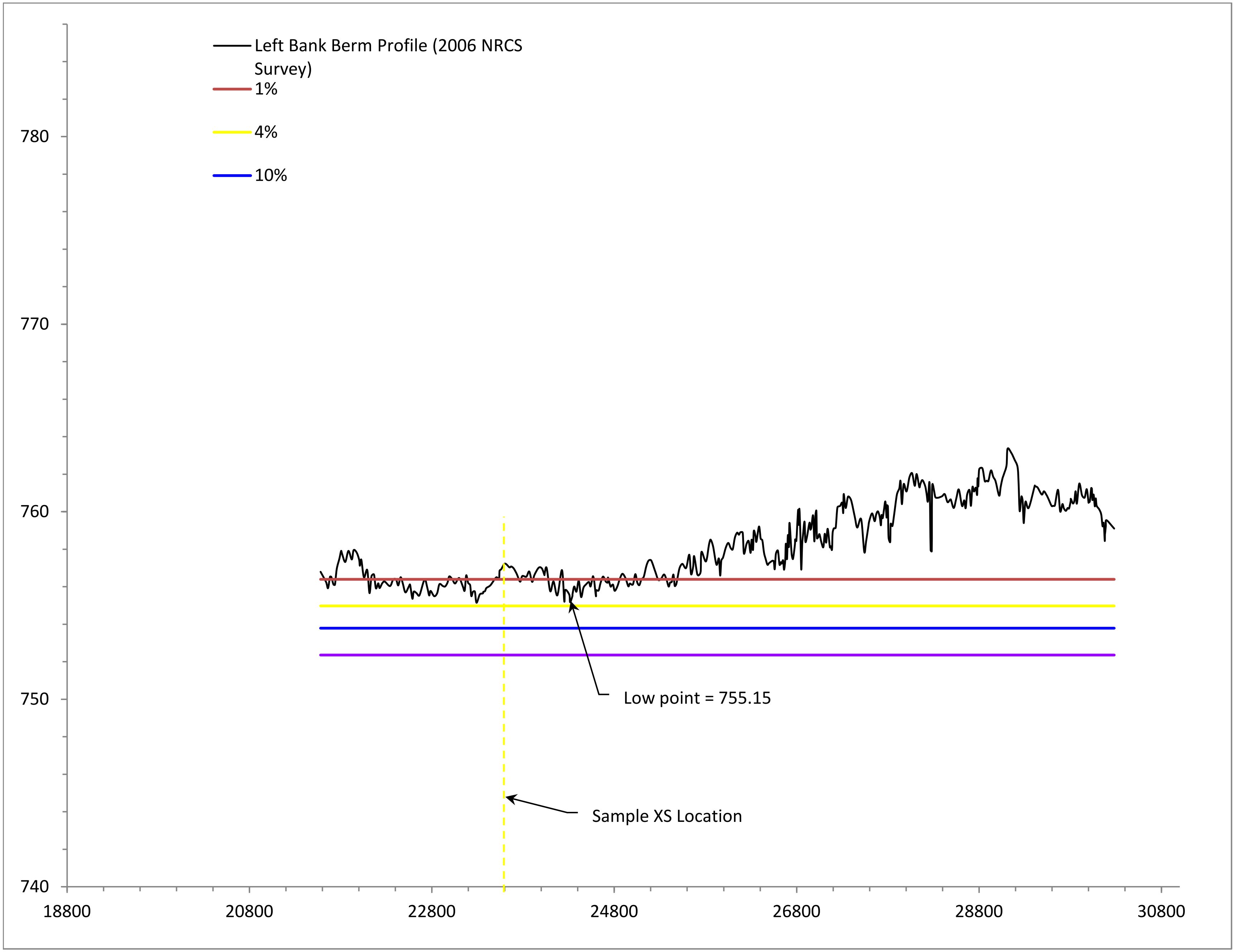


Figure 2.14b: St. Marys Flood water surface elevations behind Graham-McCulloch left bank berm (2006 NRCS survey)

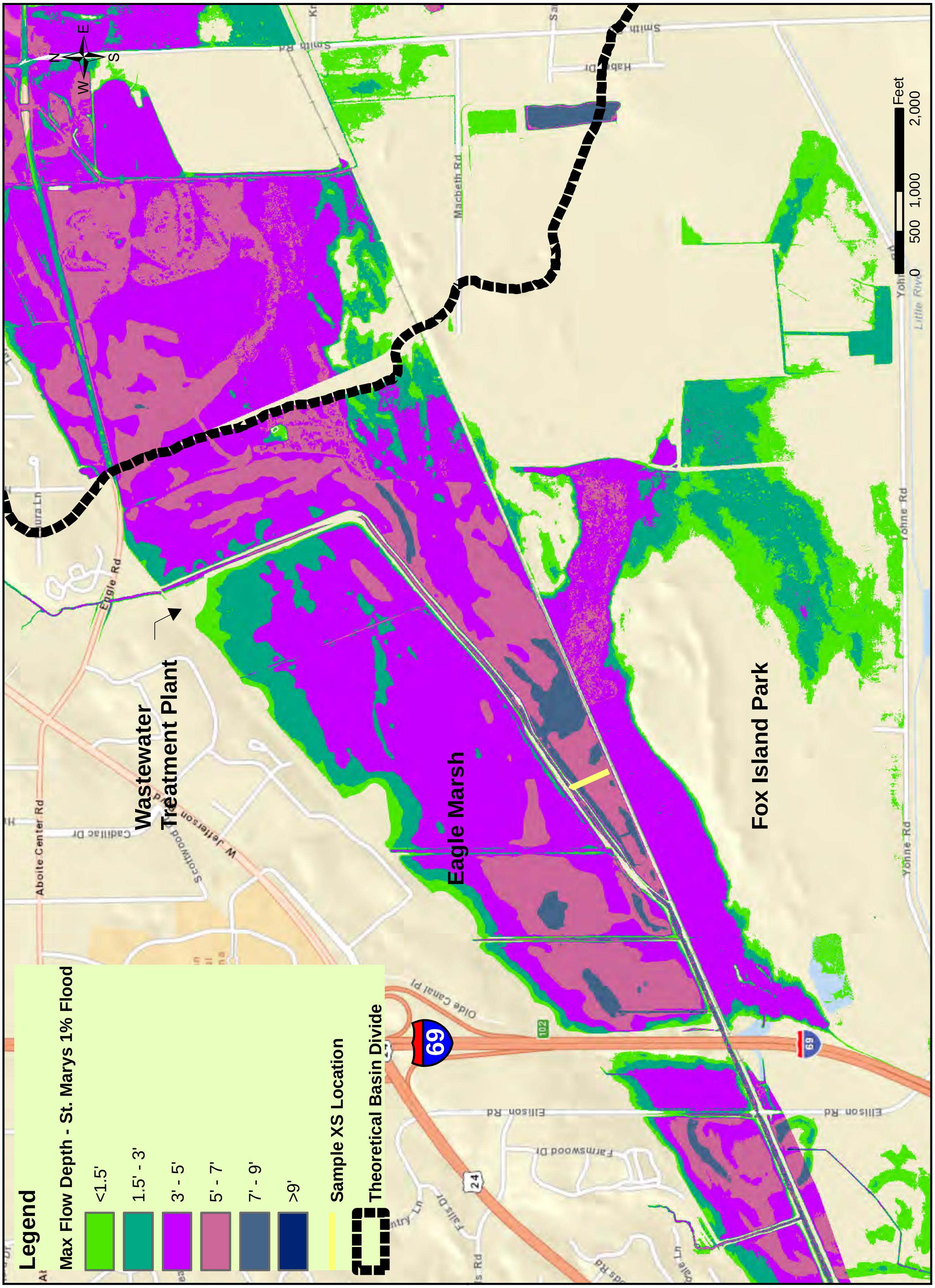


Figure 2.15a: St. Marys River 1% Annual Chance Event Depths at Eagle Marsh

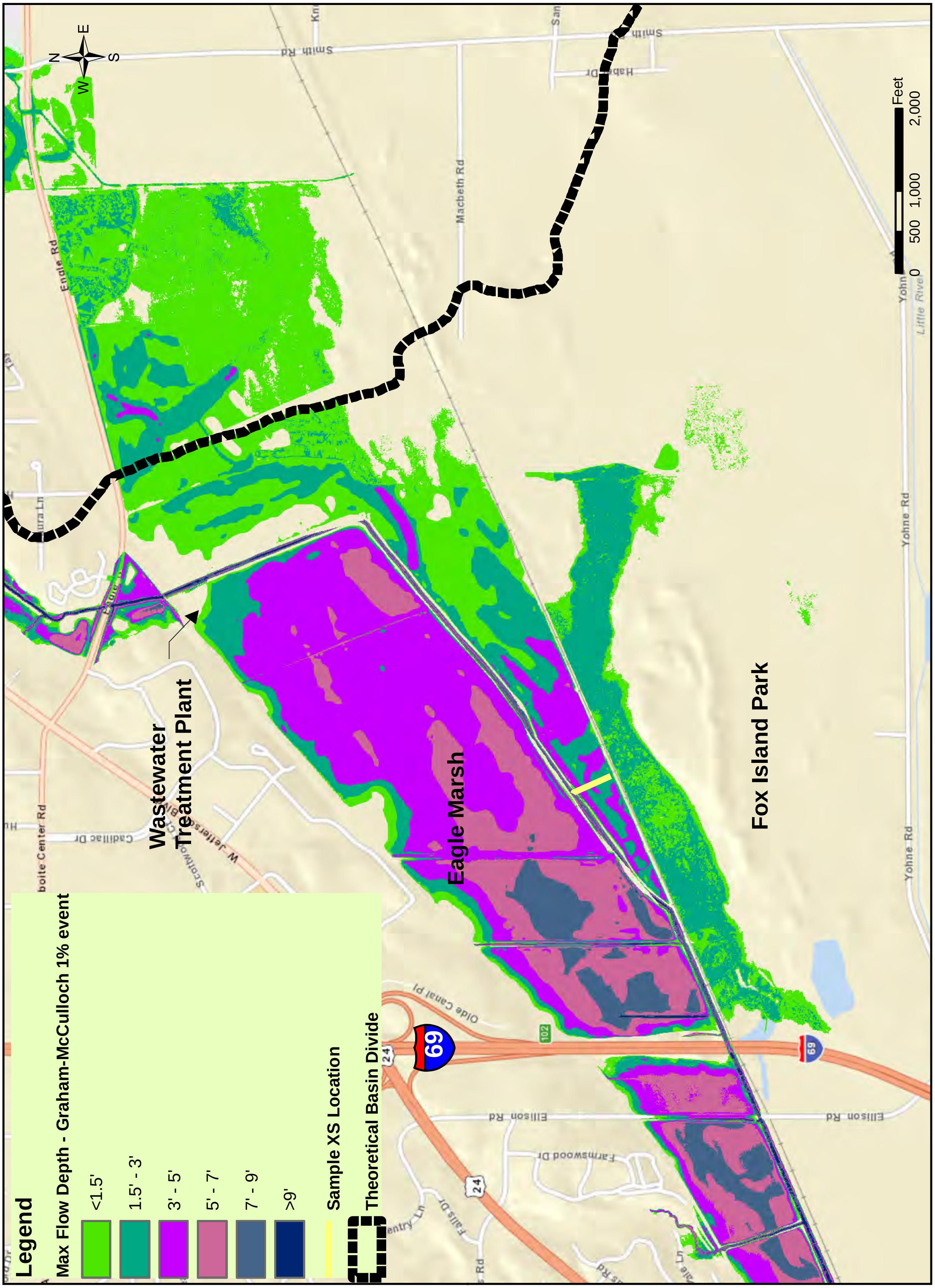


Figure 2.15b: Graham-McCulloch Ditch 1% Annual Chance Event Depths at Eagle Marsh

also showed that backwater effects from flood events on the St. Joseph River do not create significant flooding on Junk Ditch as compared to the same frequency events on the St. Marys River.

Using depth grids and inundation maps like Figures 2.15a and 2.15b, with cooperation from the Maumee River Basin Commission, a preliminary assessment of the existing condition damages were measured, quantifying the number of parcels and structures affected by the 1% annual chance events occurring on each watershed and the classification of the properties affected. The value of the affected properties was also calculated utilizing 2008 Assessment values which were readily available. Please refer to Section 2.3.3 of the main report for a summary of the findings.

2.9. EXISTING FLOOD CONTROL MEASURES

A system of levees and floodwalls has gradually been constructed and improved since the March 1913 flood. In general, the system is intended to protect areas near the lower reaches of Junk Ditch near its mouth, downstream of Main Street; the left bank of the St. Marys River from Junk Ditch to N. Clinton Street and from the Spy Run confluence to the mouth of the St. Marys River; the lower left bank of Spy Run from State Boulevard, to its mouth; the St. Joseph River left bank from near North Anthony Boulevard, and the right bank from State Boulevard to its confluence with the St. Marys River, and the left bank of the Maumee River from the St. Marys and St. Joseph Rivers confluence to a point east of North Anthony Boulevard. Figure 2.16 is extracted from the National Levee Database public map server depicting the locations of these structures. These flood hazard reduction measures are typically constructed to an elevation two feet greater than the1913 flood elevation. Per the 2009 FIS report, not all of these structures meet current FEMA standards for adequate design height, structural stability, and/or adequate operation and maintenance for protection against the 1% annual chance event.

2.10. STRUCTURAL ALTERNATIVES

In the development of the following alternatives, risk and uncertainty in water surface elevations was considered when determining the elevations of barrier components of each alternative. Preliminary calculations were performed based upon the multiple standard frequency events for both the St. Marys River watershed and the Graham-McCulloch Ditch watersheds. The preliminary calculated values were less than the standard 2.0 foot minimum freeboard, so per EC 1110-2-6067, this minimum value was used. These calculations will need to be further refined upon selection of a preferred alternative in accordance with any final design details.

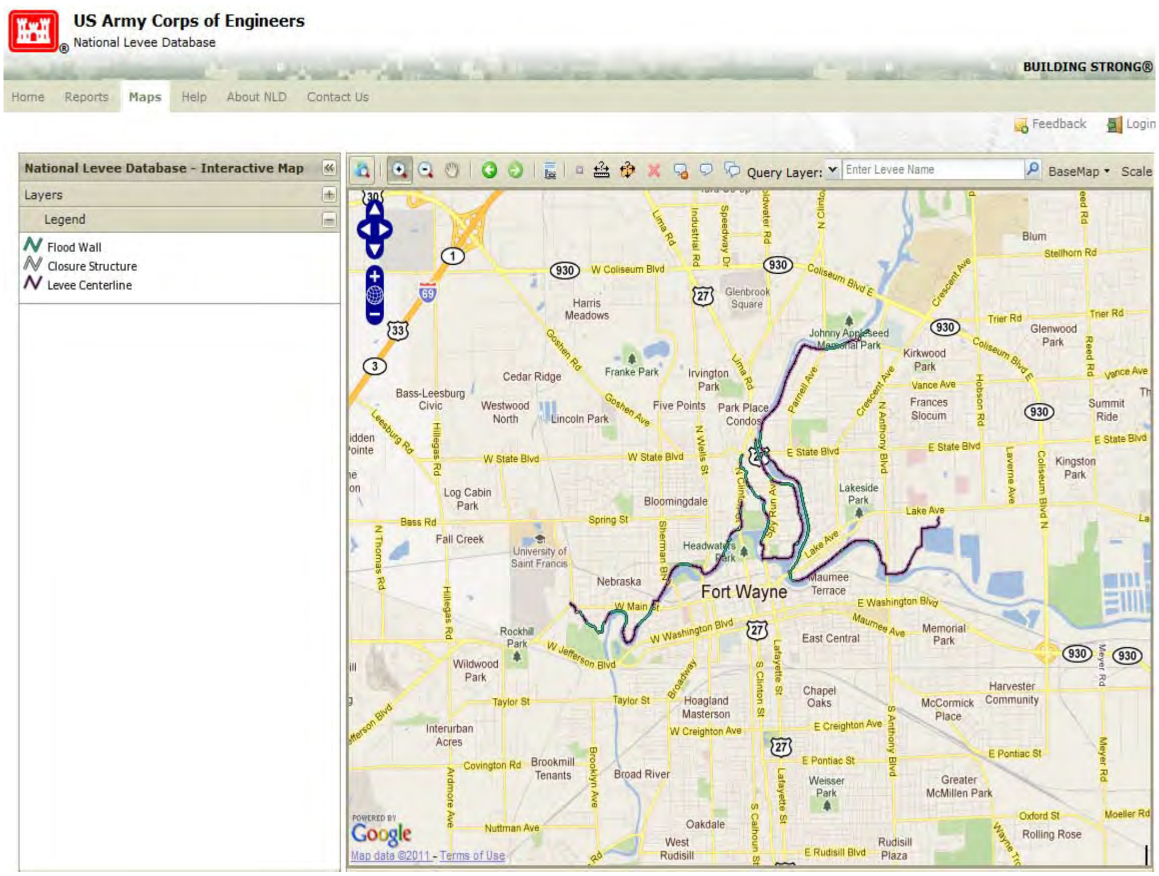


Figure 2.16: National Levee Database levee and floodwall locations in Fort Wayne

Tables 2.5a and 2.5b summarize the changes to calculated water surface elevations at select points of interest within each watershed for the 1% annual chance event on each watershed. Please also refer to Section 3.4.3 for further discussion of each structural alternative, including evaluation of impacts for each concept. As was performed for the existing conditions, preliminary inundation maps and depth grids were compared to 2008 property assessment data for Allen County with the assistance of the Maumee River Basin Commission. For each alternative modeled, it was determined utilizing GIS the number of new parcels and structures that would be affected by increased water surface elevations. The real property values were calculated based on newly inundated structures that fell within these parcels. The property values provided in the main report discussion assume that the entire parcel and structure is completely inundated. In reality, there are places where only the corner of a structure may experience minimal flooding (for example, 1" of water), but in the absence of first floor information and depth-damage curves for these properties, it was assumed that the entire property is completely affected. The real property values reported are very conservative, but are provided as a way to qualitatively evaluate and compare the impacts of each structural alternative relative to potential increased water surface elevations.

TABLE 2.5a

COMPARISON OF CALCULATED WATER SURFACE ELEVATIONS FOR CONSIDERED STRUCTURAL ALTERNATIVES FOR 1% ANNUAL CHANCE EVENT ON THE ST. MARYS RIVER WATERSHED

	Stream	Graham-McCulloch Ditch				Junk Ditch				St. Marys River	
	Bridge	Amber Road		I-69		Ardmore Avenue		W. Jefferson Blvd		Main Street	
Alternative	Description	Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)	
-	Baseline Conditions	748.81		753.28		756.37		757.98		756.61	
		Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)
A	Construct an I-Wall	742.95	-5.86	748.44	-4.84	757.87	1.5	757.98	0	756.55	-0.06
B	Construct a Fence and Reconstruct Left Descending Graham McCulloch Ditch Berm (2)										
C	Construct an Earthen Berm and Pump Station	749.41	0.6	754.37	1.09	756.45	0.08	757.33	-0.65	756.09	-0.52
D	Construct a Permeable Berm with Telemetered Sluice Gates (3)										
E	Construct a Fence/Earthen Berm Combination	748.82	0.01	753.29	0.01	756.36	-0.01	757.98	0	756.61	0
F	Construct Bar Screen Barrier at Existing Weir (4)										
G	Construct Vertical Drop Structure with Telemetered Sluice Gate	749.38	0.57	754.83	1.55	756.45	0.08	757.33	-0.65	756.15	-0.46
H	Reconstruct Left Descending Graham McCulloch Ditch Berm	742.96	-5.85	748.44	-4.84	757.17	0.8	757.98	0	756.61	0
I	Reconstruct Left Descending Graham McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area (5)	742.96	-5.85	748.44	-4.84	757.17	0.8	757.98	0	756.61	0

Notes

(1) All calculated elevations taken upstream of bridge (normal flow direction)

(2) Alternative B was not specifically modeled, as the hydraulic conditions will remain essentially the same as the current (baseline) conditions.

(3) Alternative D was not specifically modeled for this flow condition, as the Graham-McCulloch Ditch 1% annual chance event flows will control.

(4) Alternative F was not specifically modeled, as it was assumed that the constraints of minimizing impacts to upstream water surface elevations upstream to at or near Indiana maximum allowable levels of 0.14 feet increases for 1% chance event will dominate the bar screen design

(5) Alternative I was not specifically modeled for this flow condition, as the hydraulic conditions will remain essentially the same as Alternative H.

TABLE 2.5b

COMPARISON OF CALCULATED WATER SURFACE ELEVATIONS FOR CONSIDERED STRUCTURAL ALTERNATIVES FOR 1% ANNUAL CHANCE EVENT ON THEGRAHAM-McCULLOCH DITCH WATERSHED

	Stream	Graham-McCulloch Ditch				Junk Ditch				St. Marys River	
	Bridge	Amber Road		I-69		Ardmore Avenue		W. Jefferson Blvd		Main Street	
Alternative	Description	Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)		Max. WSEL (ft NAVD88)	
-	Baseline Conditions	750.94		755.35		750.1		749.6		748.77	
		Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)	Calculated WSEL	Increase (+) / Decrease (-) from Baseline (feet)
A	Construct an I-Wall	750.94	0	755.39	0.04	749.69	-0.41	749.51	-0.09	748.75	-0.02
B	Construct a Fence and Reconstruct Left Descending Graham McCulloch Ditch Berm (2)										
C	Construct an Earthen Berm and Pump Station	750.74	-0.2	755.66	0.31	750.27	0.17	749.64	0.04	748.78	0.01
D	Construct a Permeable Berm with Telemetered Sluice Gates	750.88	-0.06	755.63	0.28	750.32	0.22	749.34	-0.26	748.31	-0.46
E	Construct a Fence/Earthen Berm Combination	750.95	0.01	755.46	0.11	750.25	0.15	749.64	0.04	748.78	0.01
F	Construct Bar Screen Barrier at Existing Weir (4)										
G	Construct Vertical Drop Structure with Telemetered Sluice Gate	750.17	-0.77	756.02	0.67	751.32	1.22	749.67	0.07	748.37	-0.4
H	Reconstruct Left Descending Graham McCulloch Ditch Berm	751.1	0.16	755.89	0.54	749.46	-0.64	749.45	-0.15	748.73	-0.04
I	Reconstruct Left Descending Graham McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area	751.2	0.26	756.47	1.12	749.46	-0.64	749.45	-0.15	748.73	-0.04

Notes

(1) All calculated elevations taken upstream of bridge (normal flow direction)

(2) Alternative B was not specifically modeled, as the hydraulic conditions will remain essentially the same as the current (baseline) conditions.

(4) Alternative F was not specifically modeled, as it was assumed that the constraints of minimizing impacts to upstream water surface elevations upstream to at or near Indiana

2.10.1. Construct an I-Wall, Alternative A, (Eagle Marsh, Basin Divide)

Alternative A was developed for reference to demonstrate the impacts of complete hydraulic separation of the St. Marys and Maumee watersheds from the Wabash River watershed, thus preventing the possibility of any aquatic nuisance species from being transferred via this path through a hydraulic connection in either direction. Including preliminary calculations to account for risk and uncertainty in the modeling and available data, it is estimated that a wall with elevation varying from 760 to 762 would provide protection from overtopping (and thereby a hydraulic connection) for a 1% annual chance event. To tie into high ground, a low berm will be constructed to elevation 762 along the south side of Engle Road connecting this I-wall with the left (looking downstream) abutment for the Engle Road bridge over the Graham-McCulloch Ditch. Figure 2.17 shows the impacts to maximum water surface elevations (WSEL) on Junk Ditch for the 1% annual chance event on the St. Marys River, relative to the baseline WSEL for the same event. As can be seen in these figures, the impacts to maximum water surface profiles would be significant and far reaching. For floods on the St. Marys River, hydraulic separation at the drainage boundary does not allow use of storage that is currently occurring in the Eagle Marsh South Storage area (EMSS) east of the Graham-McCulloch left berm, nor in the Fox Island County Park, and does not allow additional flows to escape into the Graham-McCulloch Ditch. Flows thus are held in the Junk Ditch and thus increases water surface levels in that reach. The increases in WSEL for the 1% annual chance event created by this barrier alternative are anticipated to be approximately 1.5 feet on the Junk Ditch for areas between the barrier and Ardmore Avenue, and gradually decreasing as you continue east toward Taylor Street. Likewise, events on the Graham-McCulloch watershed would not be allowed to discharge excess flows into the St. Marys watershed, resulting in water surface increases on the Eagle Marsh south storage area and Fox Island County Park of approximately 0.6 feet and 0.4 feet respectively for the 1% annual chance event. These elevations are still well below the elevations for the 1% annual chance event on the St. Marys River under existing conditions. Water surface profiles for the 1% annual chance Graham-McCulloch event are not expected to change. Indiana floodplain regulations allow a project to increase the WSEL for the 1% annual chance event by a maximum of 0.14 feet.

Alternative A Water Surface Profiles – St. Marys River 1% Annual Chance Event

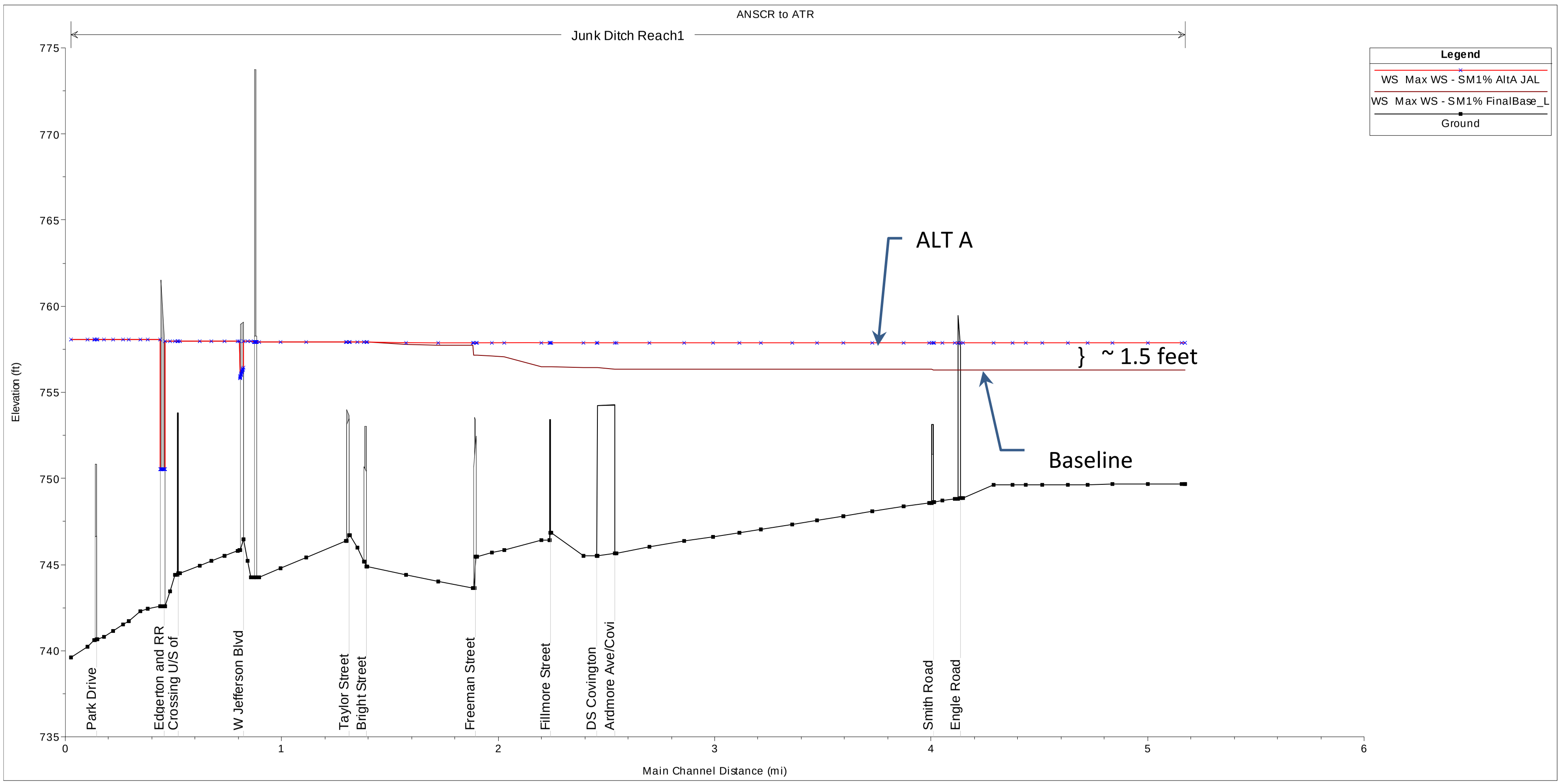


Figure 2.17: Comparison of Baseline and Proposed water surface profiles for Alternative A – Junk Ditch Reach

2.10.2. Construct a Fence and Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative B, (Eagle Marsh, Basin Divide)

As discussed previously in the report, in 2010 IDNR with the cooperation of the NRCS and LRWP constructed a temporary fence between the Graham-McCulloch left berm and the railroad embankment that separates the Eagle Marsh south storage area and the Fox Island County Park. This fence was built primarily to prevent the transfer of adult fish species between basins due to the known presence of Asian carp species in the Wabash River watershed in relatively close proximity to the natural drainage divide. The effectiveness of this fence as a barrier is in part dependent on the integrity of the left bank berm and its ability to separate the Graham-McCulloch ditch from the Eagle Marsh south storage area and thus the watershed divide. As further discussed in Section 4 of Appendix F, the integrity of the left bank berm is deemed to be poor due to poor construction methods, observed slope failures and penetrations by animal burrows and vegetation. The most upstream section of the berm would be reconstructed, starting at the tie-in point with the existing tow path embankment and ending immediately past the westward bend in the Graham-McCulloch Ditch, where the new fence alignment would be constructed. The fence and berm would be constructed to an elevation to prevent overtopping by the 1% annual chance event. The berm would be constructed to an elevation of 762 near the tow-path embankment and sloping to elevation 760 at the fence. To tie into high ground, a low berm will be constructed to elevation 762 along the south side of the wastewater treatment plant access road to Engle Road, and along Engle Road to the left (looking downstream) abutment for the Engle Road bridge over the Graham-McCulloch Ditch. It is anticipated that the remainder of the left bank berm downstream of the fence and reconstructed berm would be left undisturbed. Like the current IDNR fence, this barrier would only prevent the movement of large fish species that can make it to the fence location. To prevent debris buildup and the potential for related swell-heads, the fence would be constructed of standard chain link material, although smaller opening sizes could be investigated. Sacrificial fences would also be positioned on the upstream side of the main barrier fence to catch debris. No modeling specific to this alternative was performed as this condition should not significantly change the hydraulics of the area; this section of berm to be replaced is currently higher in general than other areas, therefore the increase in berm height is not significant.

2.10.3. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

This alternative would consist of raising the roadway embankment at Homestead Road to elevation 758.0 and constructing a pump station and confining berm (also to elevation 758.0) downstream of the roadway. The confining berm and raised roadway would allow some storage of flood waters upstream, with higher flows being pumped through the berm. The pump station is conceptually designed to handle peak flows of approximately 1100 cfs resulting from the 1% annual chance event on the Graham – McCulloch Ditch, in order to prevent increasing the transfer of flows across the left bank berm and subsequently water surface elevations on Junk Ditch. This alternative would primarily be a barrier to the passage of aquatic nuisance species from the Wabash River watershed to the St. Marys River watershed, and has little capability to prevent transfer of ANS from the St. Marys River to the Wabash River watershed. Figures 2.18a and 2.18b show the resulting water surface elevations for this alternative compared to baseline conditions for the 1% annual chance event for the Graham-McCulloch Ditch watershed. The resulting increase in water surface elevations on the Junk Ditch is estimated to be 0.2 feet for the Graham-McCulloch 1% annual chance event. While greater than the allowable increase per IDNR floodplain management regulations of 0.14 feet, it is expected that this difference can be mitigated through further detailed modeling of the pump station features. For comparison, the increased WSEL for this alternative on the Junk Ditch is over 4 feet less than the St. Marys 1% annual chance event.

2.10.4. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

This alternative consists of constructing a permeable berm that would pass flow during a potential connection high water condition. A permeable berm is an embankment made up of opened graded rip rap surrounding a perforated pipe system that will capture the water as it passes through the stone. Please reference Section F4, Geotechnical Engineering, for further discussion of the berm characteristics. The berm is proposed just upstream of Amber Road and ties into high ground to the northeast and into the embankment of the Norfolk Southern Railroad. The system will drain south to the Graham-McCulloch Ditch and the pipe system with collector channels will empty into the existing channel downstream of the berm.

The berm will contain sluice gates with automated closure mechanisms that will close the gates to a nominal opening height of 3 inches when gages on the Graham-McCulloch Ditch, Junk Ditch and/or within Eagle Marsh near the drainage divide indicate that flow conditions are imminent that could support transfer of ANS. Preliminary modeling approximated the 16 gates 5 feet in width and 5 feet in height, although the number and sizes of these gates should be evaluated further to optimize these parameters. During normal low flow conditions, the gates would be open to allow normal drainage of the Graham-McCulloch watershed. Gages at the sluice gate would allow the gate to reopen once adequate head differential across the berm was developed such that velocities through the sluice gate would be unsuitable to support ANS transfer in the upstream direction (toward the Great Lakes watershed). These gages at the structure would also trigger closure of the gate when other scenarios such as backwater flooding or headwater flooding of the Graham-McCulloch ditch might support movement of ANS upstream. Gates could be manipulated individually in order to maximize the amount of flow allowed through the gates while maintaining velocities above minimum acceptable levels. When the velocity of flow through the gates decreases below threshold values preventing ANS transfer, the gates will close and ponded water will be released by infiltration through the permeable berm. Because infiltration through the berm will be slow, this was not modeled in the HEC-RAS analysis of the system, therefore the results are considered

Alternative C Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

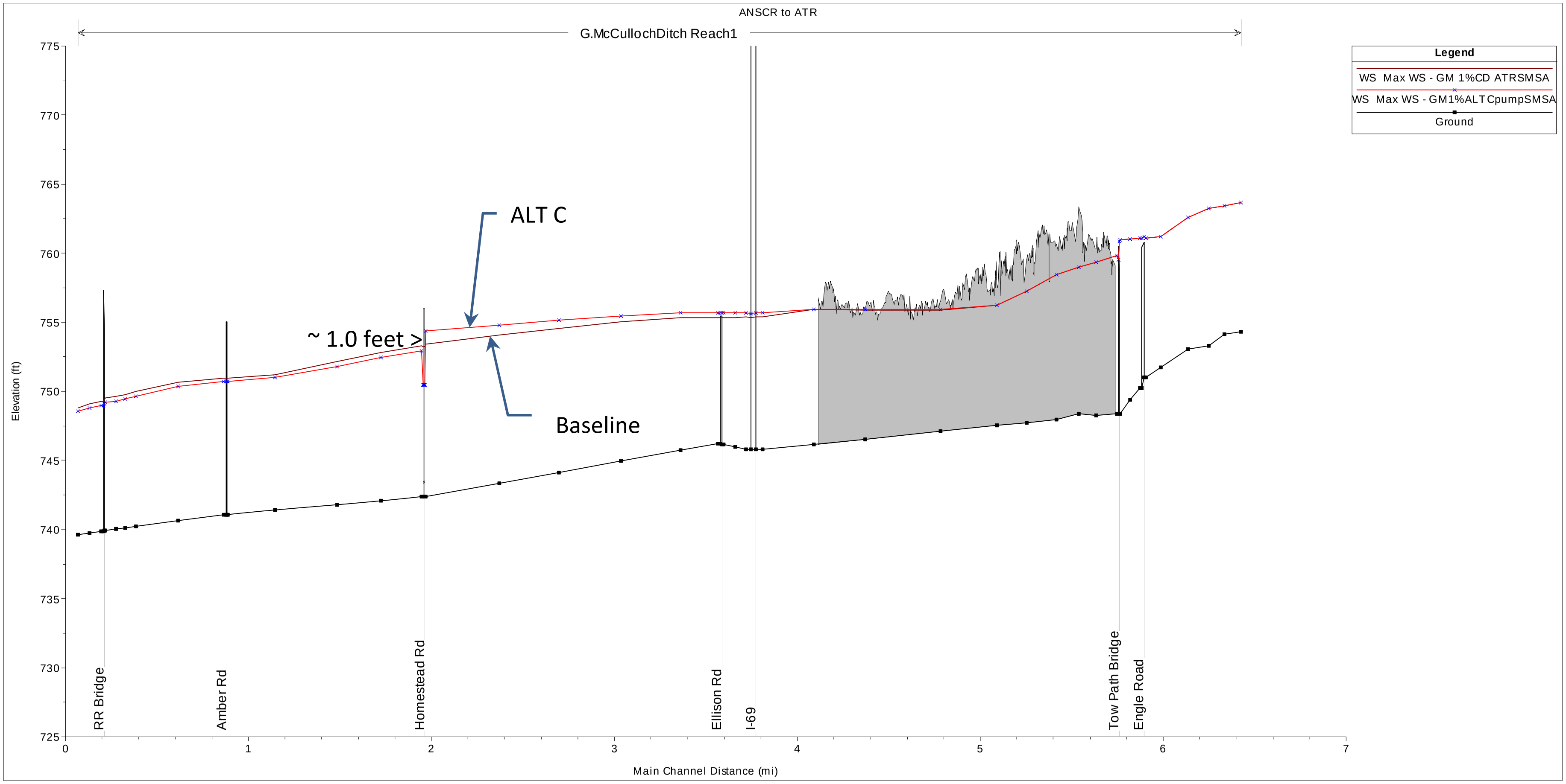


Figure 2.18a: Comparison of Baseline and Proposed water surface profiles for Alternative C – Graham-McCulloch Ditch

Alternative C Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

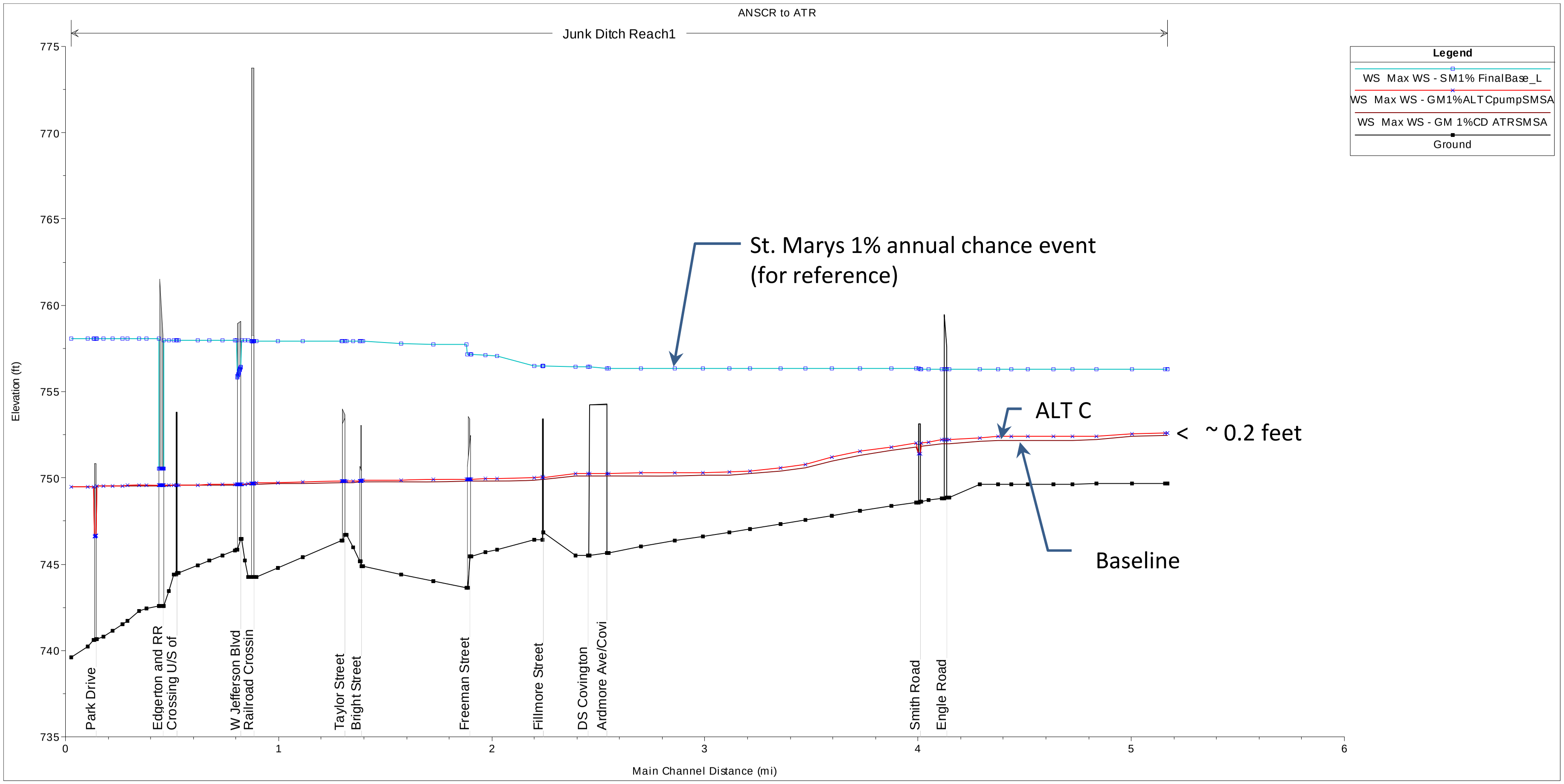


Figure 2.18b: Comparison of Baseline and Proposed water surface profiles for Alternative C – Junk Ditch Reach

conservative. Water surface profiles for this alternative are included as Figures 2.19a and 2.19b. The maximum water surface elevation increases on the Junk Ditch for the 1% annual chance event on the Graham-McCulloch is approximately 0.4 feet, but it is believed that further refinement of the gate operations could reduce this to meet Indiana floodplain limitations of 0.14 feet increase in the 1% annual chance. For reference, this increase is still less than the 1% annual chance elevation for the St. Marys River flood event.

The berm would be designed so that the entire length would be equally submerged during a high water condition. This would allow the berm to pass maximum flows. Water levels are expected to rise on the berm during an event, and would inundate a large area upstream of the berm. Debris and sediment buildup onto and within the berm is a concern for this alternative. A vegetated filter strip should be planted upstream from the permeable berm to filter debris and sediment. It is important to the performance of the system that debris and sediment do not collect on or in the stone matrix of the berm as this could reduce flows through the system. This system would require periodic maintenance and testing of automated systems to assure functionality of the system.

2.10.5. Construct a Fence/Earthen Berm Combination, Alternative E, (Eagle Marsh, Basin Divide)

Alternative E is intended to reduce the annual chance of inter-basin flow by creating a berm to elevation 752.5 that would block flows up to a 4% annual chance event on the Graham-McCulloch Ditch. This event is similar to approximately a 99% annual chance exceedance event on the St. Marys River. A higherberm elevation could not achieve the goal of no induced damages on Junk Ditch. For larger events up to the 1% annual chance event on the St. Marys River, the passage of ANS, primarily larger swimming species, could be blocked by a fence on top of or immediately adjacent to this berm. This alternative is conceptually aligned such that the Eagle Marsh entrance road could be raised to provide better access during frequent flood events and regular inspection of the fence, and allow for ease of removing debris accumulations or fence repairs. Figure 2.20 shows the impacts to the maximum WSEL for the 1% annual chance event on the Junk Ditch for floods on both the Graham-McCulloch Ditch and the St. Marys River. This alternative has no effect on the 1% annual chance event occurring on the St. Marys, and decreases the water surface elevations for the same chance exceedance event on the Graham-McCulloch Ditch. Minor increases of up to 0.15 feet are created on the Graham-McCulloch Ditch by this alternative for the 1% annual chance event on that watershed.

2.10.6. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

This alternative is located to the west of the Eagle Marsh site in the town of Huntington, Indiana, at an existing low head, fixed weir dam across the Little River. A bar screen structure would be constructed immediately downstream of the fixed weir to prevent the movement of ANS past this barrier. A floating boom would be constructed upstream of the dam and angled toward the right descending bank to divert floating debris to a

Alternative D Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

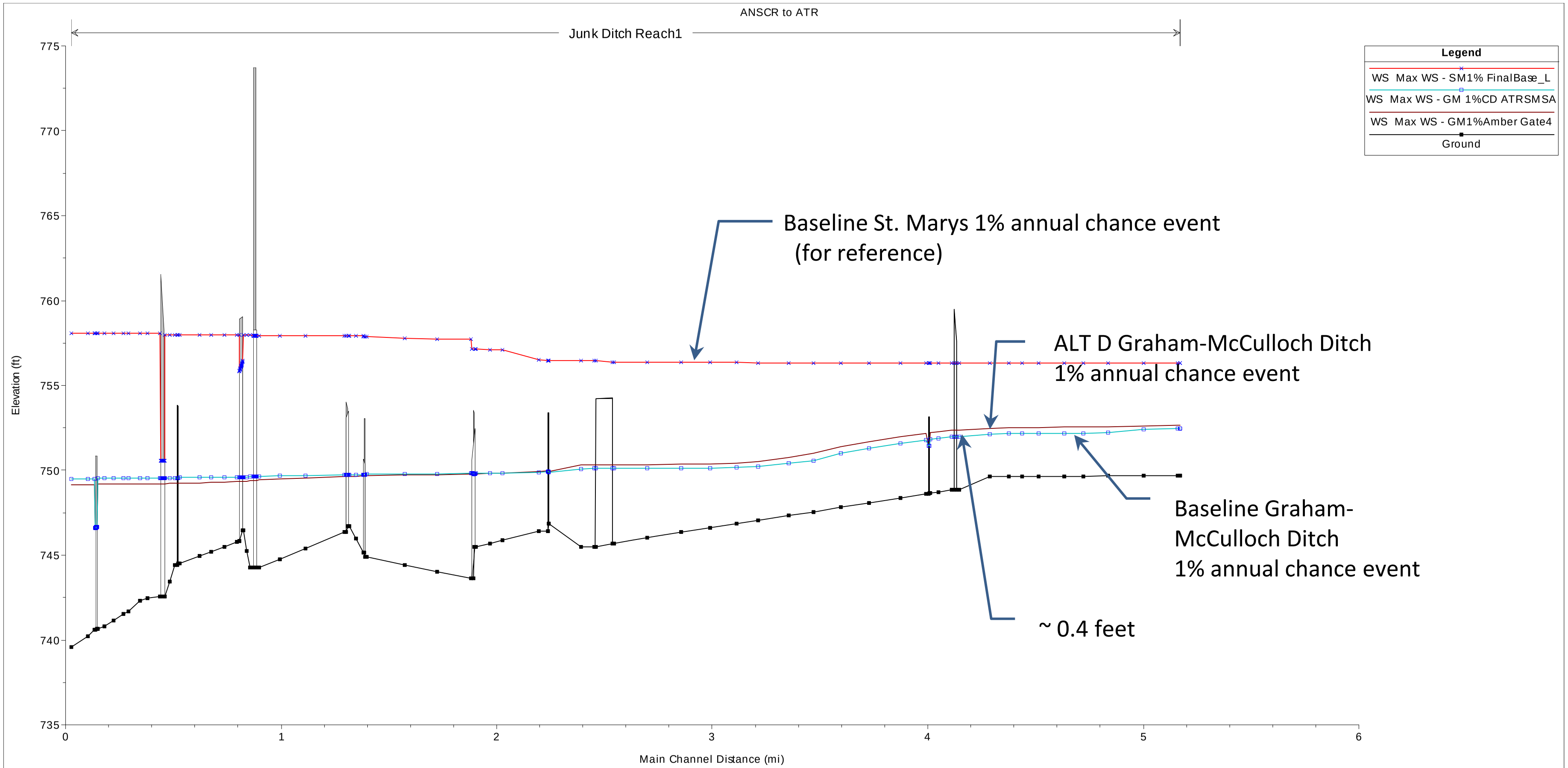


Figure 2.19a: Comparison of Baseline and Proposed water surface profiles for Alternative D – Junk Ditch Reach

Alternative D Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

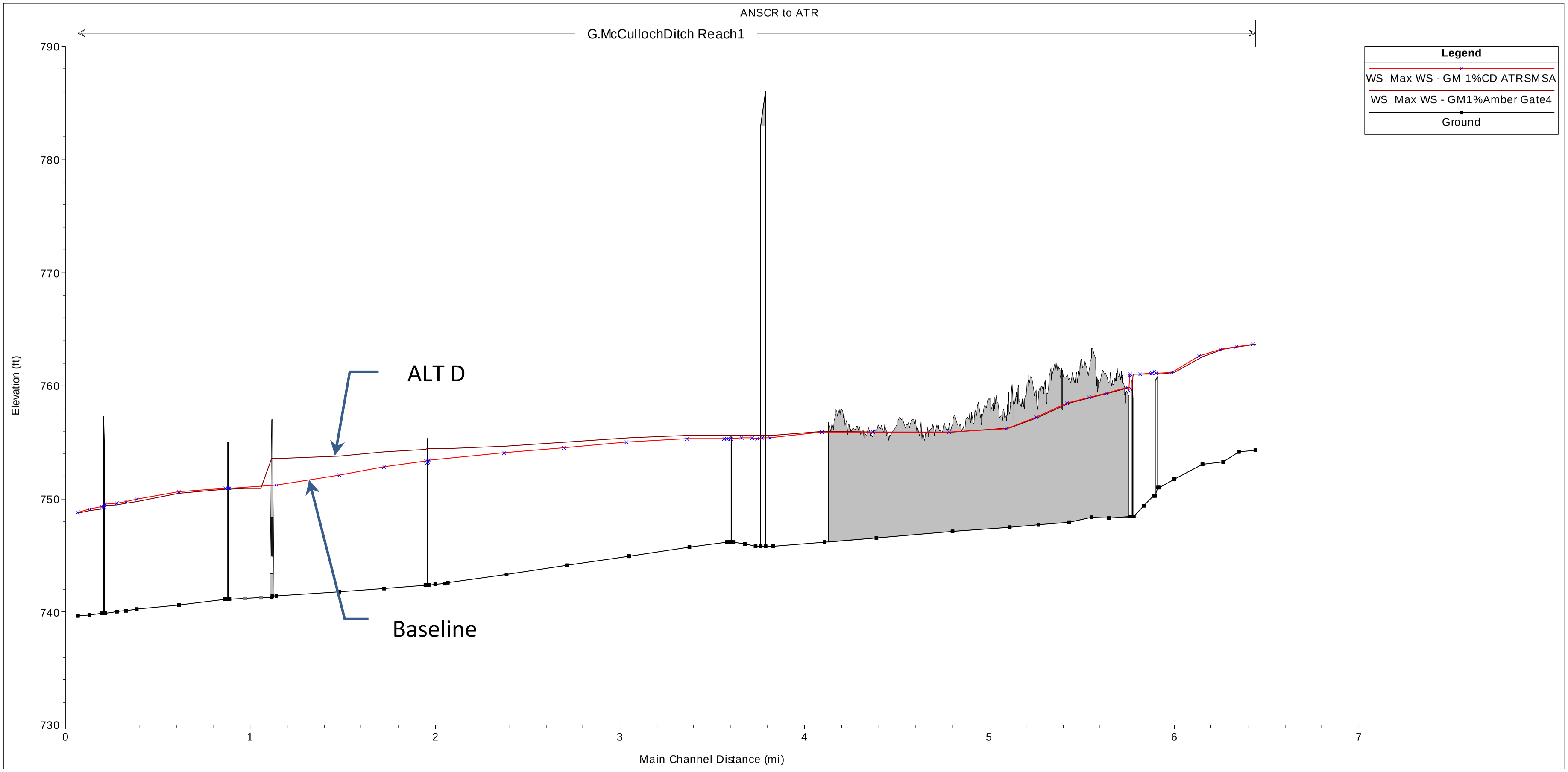


Figure 2.19b: Comparison of Baseline and Proposed water surface profiles for Alternative D – Graham-McCulloch Ditch

Alternative E Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

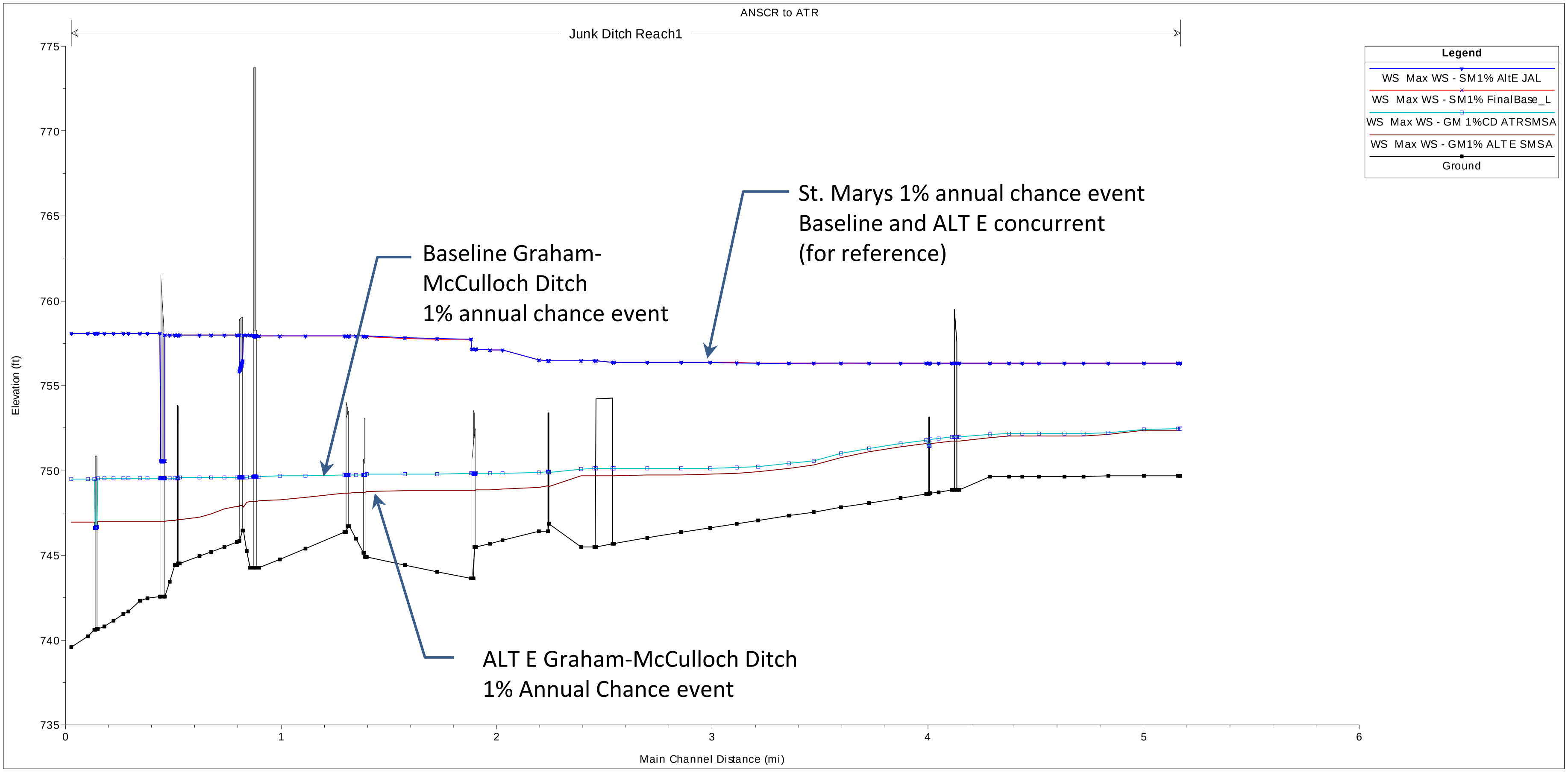


Figure 2.20: Comparison of Baseline and Proposed water surface profiles for Alternative E – Junk Ditch Reach

collection point on that bank. The bar screen would be slanted downstream, both to prevent the ability for certain species of ANS to jump over the screen, and also to allow any floating debris that might get past the floating debris boom to float upward during high flows and potentially be pushed over the bar screen during extreme flood events. This is done to prevent debris from clogging the bar screen and inducing flooding upstream. The top of the screen is set to elevation 727.0 (referencing NAVD88) to equal the 1% annual chance event elevation for the Little River, including preliminary risk and uncertainty considerations. This elevation was determined by conversion of an IDNR HEC-2 model for this reach of the Littler River to HEC-RAS, adjusting elevations to the NAVD88 datum, and incorporating survey information taken at the dam site by Louisville District survey crews. No additional modeling to include the bar screen structure was performed, as the bar screen itself should pose minimal restriction to flows. Piers to support the screen would be small and also should have minimal effect. Additional modeling should be performed in any future design to ensure that the upstream impacts do not exceed Indiana floodplain limitations.

2.10.7. Construct Vertical Drop Structure with Telemetered Sluice Gate, Alternative G, (Homestead Road)

Alternative G relies on the creation of a water surface elevation differential across a barrier to prevent the movement of ANS from the Wabash River basin into the Great Lakes basin. Conceptually located at Homestead Road, a berm would be constructed east (upstream) of Homestead Road. A portion of the berm would be permeable, similar to that described in Alternative D. To pass additional flows, vertical drop structures would act similar to control structures commonly used in lakes, ponds, or detention basins. Low flows would be passed though a sluice gate structure in the main channel of Graham-McCulloch Ditch. The gate would be controlled by gages in a similar fashion to Alternative D, closing when flows increased and threatened to create a hydraulic connection between basins. After the sluice gate closed, the water surface elevation immediately upstream of the barrier would increase and overtop the drop structures. In the model, culverts are used to determine the capacity of flow that the drop structures need to pass past the barrier structure to minimize the impacts to water surface profiles at the left bank berm barrier for the 1% annual chance event for the Graham-McCulloch Ditch. The number and size of the drop structures were then determined for this flow. The drop structures are conceptually 24-foot diameter structures constructed to elevation 754.0, with a debris rack extending to elevation 757.0. Circular structures were conceptually used due to standard computation methods and the diameter sized to be similar to grain silos to facilitate construction. Standard weir and orifice flow equations were used for analysis of the drop structure. It was determined that ten structures of this size would be required. Each drop structure would pass flows through the berm to a receiving channel between the berm and the Homestead Road embankment by way of a concrete box culvert, conceptually sized at 6 feet wide by 3 feet tall. Figures 2.21a and 2.21b depicts the impacts to the maximum WSEL for this alternative for the Graham-McCulloch Ditch 1% annual chance event. Even though the increase to WSEL elevation is less than 0.2 feet at the left bank berm, significant increases of approximately 1 foot or

Alternative G Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

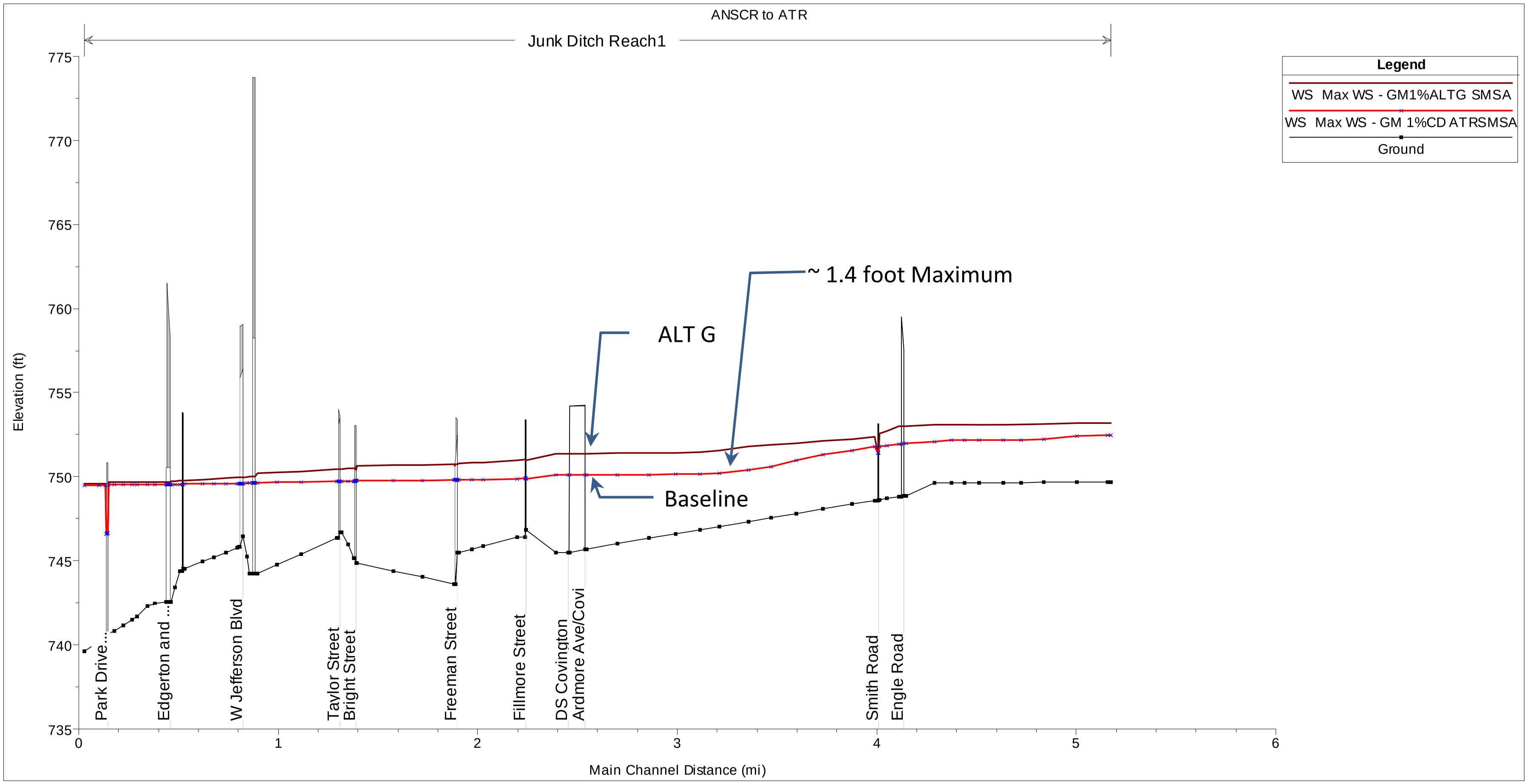


Figure 2.21a: Comparison of Baseline and Proposed water surface profiles for Alternative G – Junk Ditch Reach

Alternative G Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

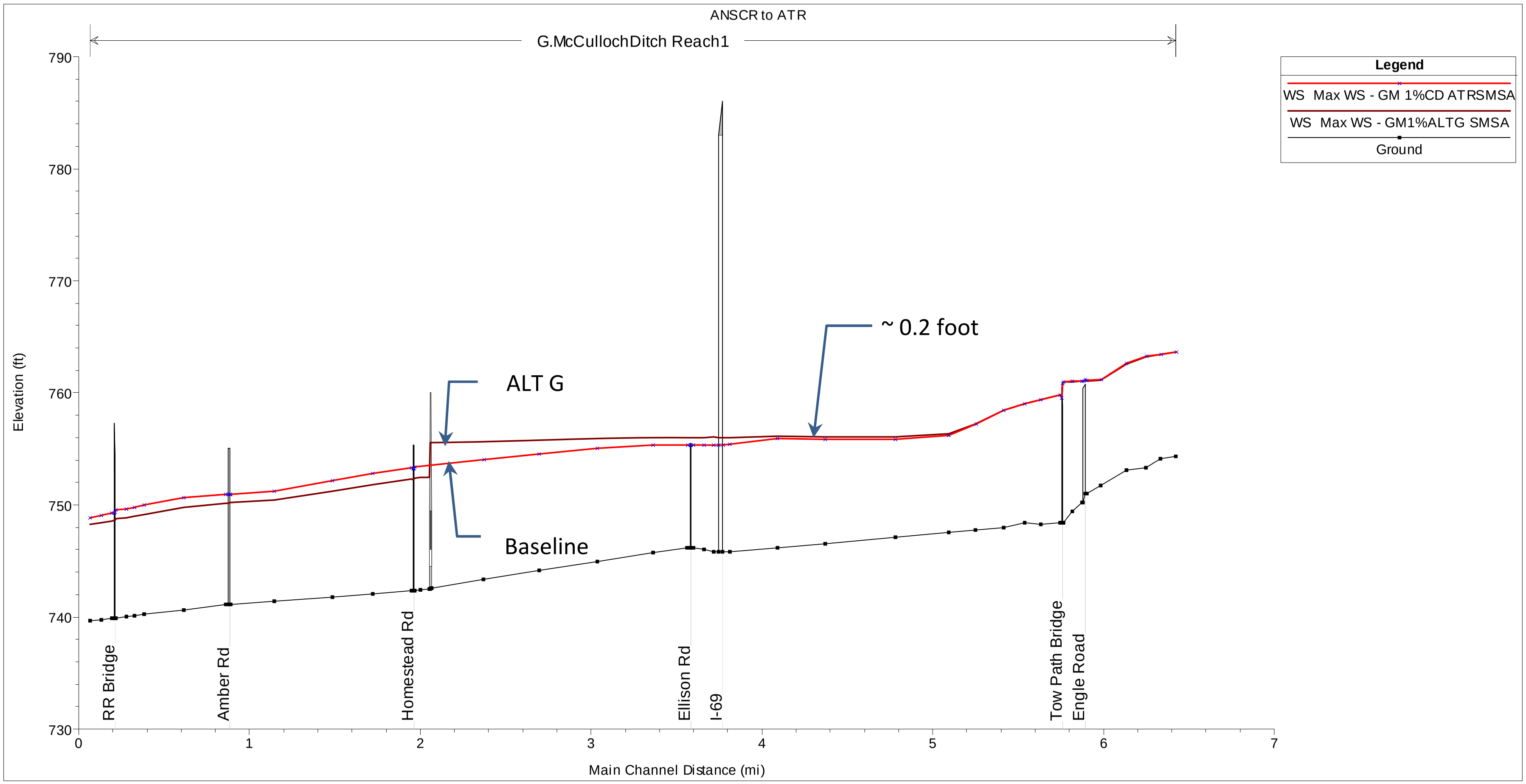


Figure 2.21b: Comparison of Baseline and Proposed water surface profiles for Alternative G – Graham-McCulloch Ditch

more are observed on the Junk Ditch. Even with the number of these structures, due to the flatness of this area and the significant flows that must be transferred across the barrier, the head differential at the barrier during the maximum water surface elevation is only approximately 3 feet.

2.10.8. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative H, (Eagle Marsh, Basin Divide)

As discussed for Alternative B, the Graham-McCulloch Ditch left bank berm is deemed to be in poor condition and currently is overtopped by approximately a 10% annual chance event on the Graham-McCulloch Ditch watershed and approximately a 3% annual chance event on the St. Marys River. This alternative proposes to reconstruct the berm to an elevation that would prevent overtopping by floods on either watershed, and thus create a barrier to ANS movement, up to the 1% annual chance event. Based upon preliminary calculations to account for risk and uncertainty in the design, the elevation of the berm would transition from elevation 762.0 at the existing tow path embankment to elevation 759.0 at the tie-in to the railroad embankment. The berm would be continued along the south side of the WWTP access Road and Engle Road to connect to high ground at the Engle Road Bridge over the Graham-McCulloch Ditch, as previously discussedin Alternative B. All existing drain tiles currently penetrating the existing berm would be removed. This berm would only increase in elevation by approximately 4 feet at the lowest point of the existing berm, and increase on average approximately 2 feet.

Similar to Alternative A, the I-wall barrier at the natural watershed boundary, a barrier completely blocking the exchange of flow will impact water surface elevations in the area exceeding the permissible increases by Indiana floodplain regulations. Figures 2.22a and 2.22b illustrate the impacts to the 1% annual chance elevations with respect to the baseline conditions. Increases to the Graham-McCulloch Ditch 1% annual chance event were approximately 0.5 feet near I-69, and increases of 0.3 feet were observed on the Little River to the end of the model. Increases of up to 0.9 feet were noted on the western end of the Junk Ditch, but no increases were computed east of Freeman Street. It was noted during modeling of this alternative that the Fox Island County Park storage area was not being fully utilized, likely due to the single 4-foot diameter culvert through the railroad embankment that links the area to the Eagle Marsh south storage area. The impacts could potentially be mitigated to a small degree by increasing the number of pipes through the railroad embankment and should be investigated further if this alternative is developed.

2.10.9. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area, Alternative I, (Eagle Marsh, Basin Divide)

Alternative I is similar in concept to Alternative H, with the additional features of removing the Graham-McCulloch Ditch right bank berm and the construction of multi-cell wetland areas along the right bank, conceptually in the upper reach near the wastewater treatment area. Removing the right bank berm has the benefit of providing a nearby source of material for reconstruction of the left bank berm. It has also been expressed by Little River Wetlands Project and NRCS personnel that ideally they would desire to have the Eagle Marsh area inundated more frequently, but the water quality of the Graham-McCulloch Ditch is generally poor due to the fact that it is largely comprised of urban runoff. Construction of the multi-cell wetland would attempt to pre-treat the water and thus improve the water quality before it enters Eagle Marsh.

The northern storage area of Eagle Marsh created by the right bank berm currently functions much like a "side-saddle" detention basin, such that at higher water surface elevations, water overtops a weir structure (the right bank berm) and enters a storage area having a limited means for flow to drain out, reducing peak flows and releasing the detained water over a longer period of time after the peak has passed. As such, removing the right bank berm, which functions as the controlling weir, causes the area to no longer function as a detention area. As a result, peak water surface elevations increase downstream of the Eagle Marsh Area, as seen in Figure 2.23. The 1.0 foot increase in the 1% annual chance event shown by the model may be slightly conservative (i.e., the increase may be less); as with the left bank berm, pipes that drain the north storage area were not included in the model due to a lack of good information about them. The profile of the right bank berm was primarily developed from the 2009 LIDAR information, with the exception of the uppermost adjacent to the wastewater treatment plant, which was surveyed by Louisville District personnel. The LIDAR data was the best available information, but it was observed when compiling the model data that the NRCS survey of the left bank berm resulted in higher elevations in general than the profile cut from the 2009 LIDAR DEM; it may thus be true that the right bank berm is higher than depicted in the model, but in general, by visual inspection at the site, it is confirmed that the right bank berm is generally lower in elevation than the left bank berm. Despite these issues, it is believed that the model generally represents the decreased functionality of the right overbank as a significant means of storing flood water.

Alternative H Water Surface Profiles – St. Marys River 1% Annual Chance Event

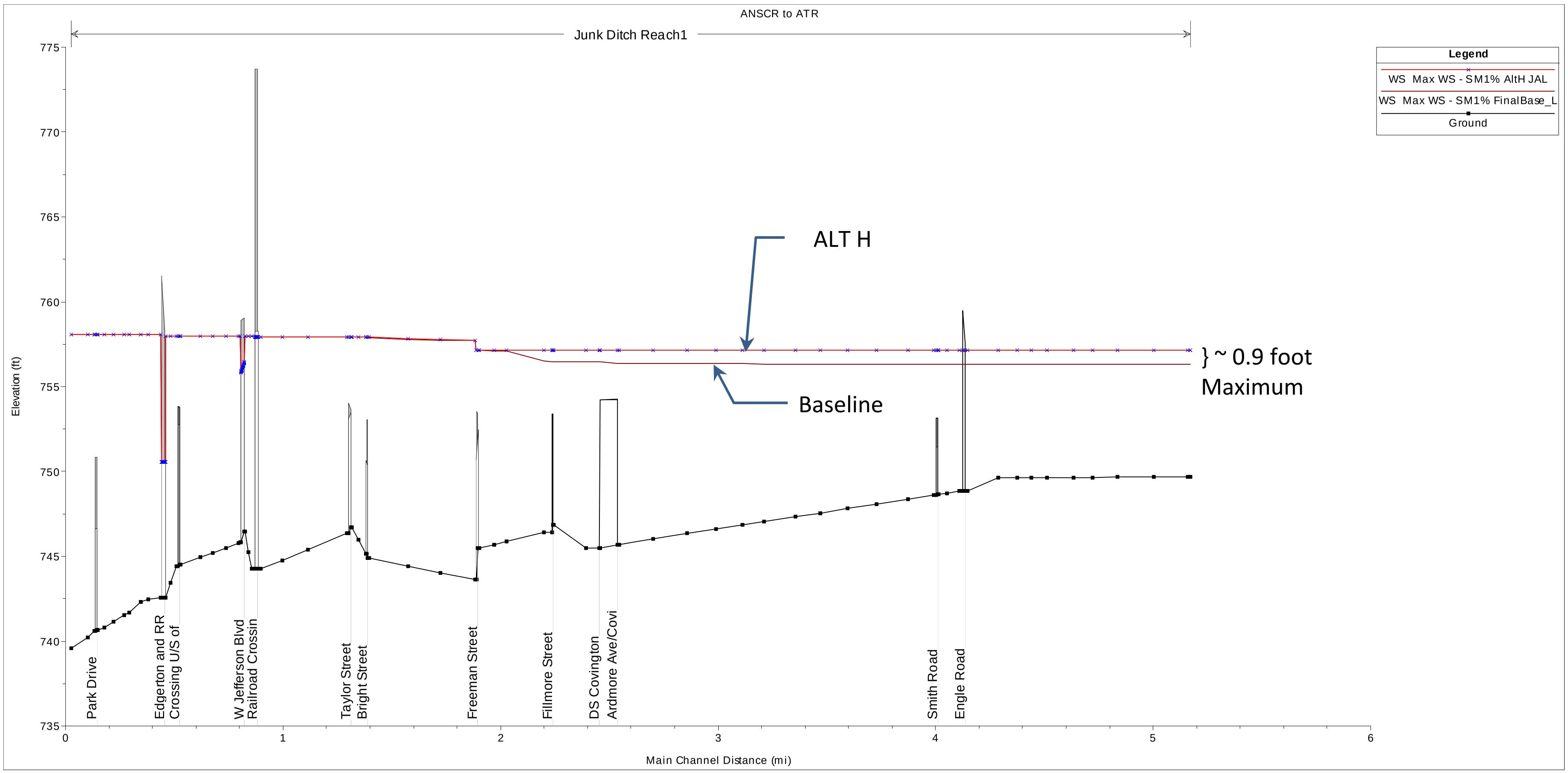


Figure 2.22a: Comparison of Baseline and Proposed water surface profiles for Alternative H – Junk Ditch Reach

Alternative H Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

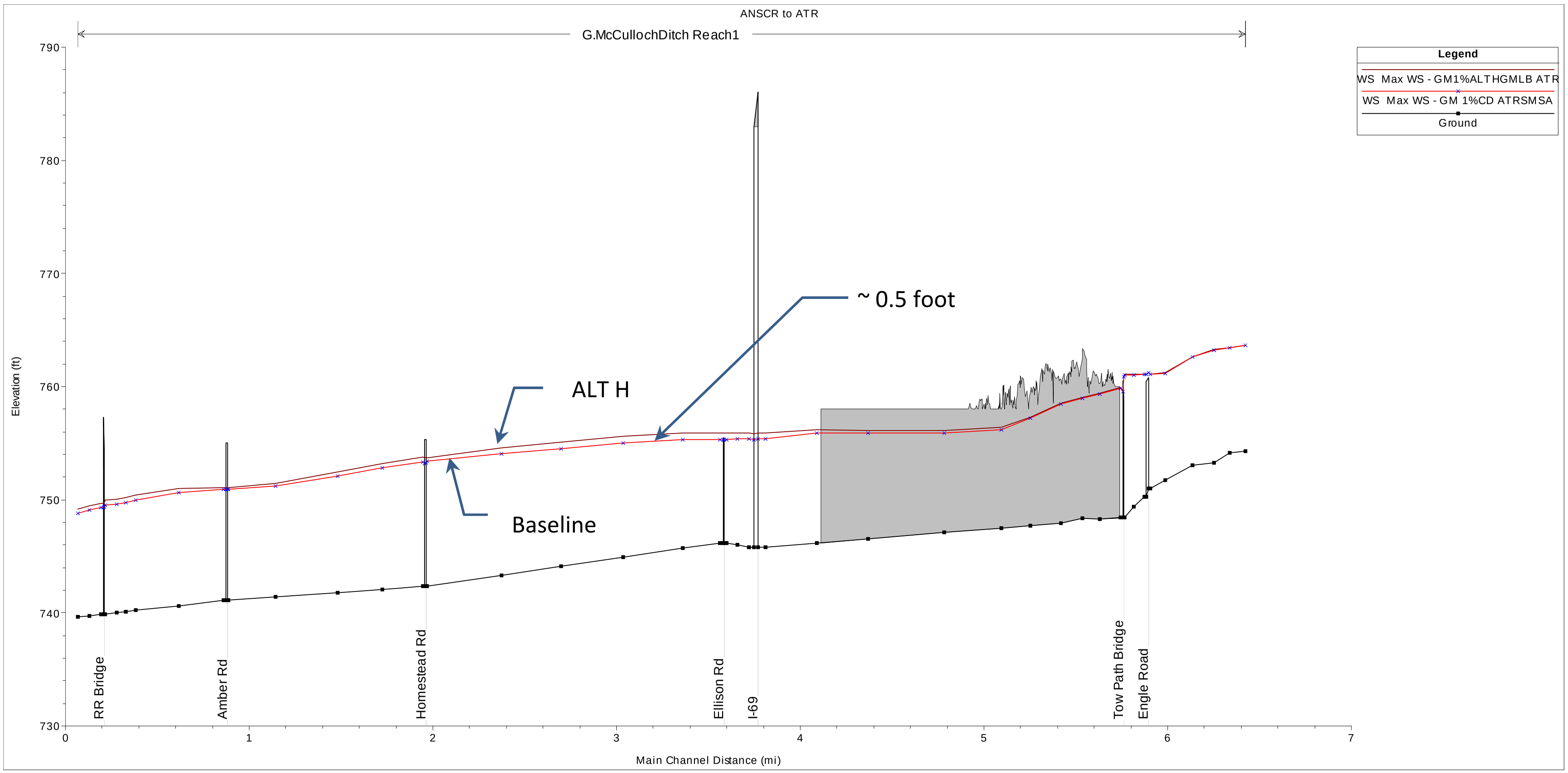


Figure 2.22b: Comparison of Baseline and Proposed water surface profiles for Alternative H – Graham-McCulloch Ditch

Alternative I Water Surface Profiles – Graham-McCulloch Ditch 1% Annual Chance Event

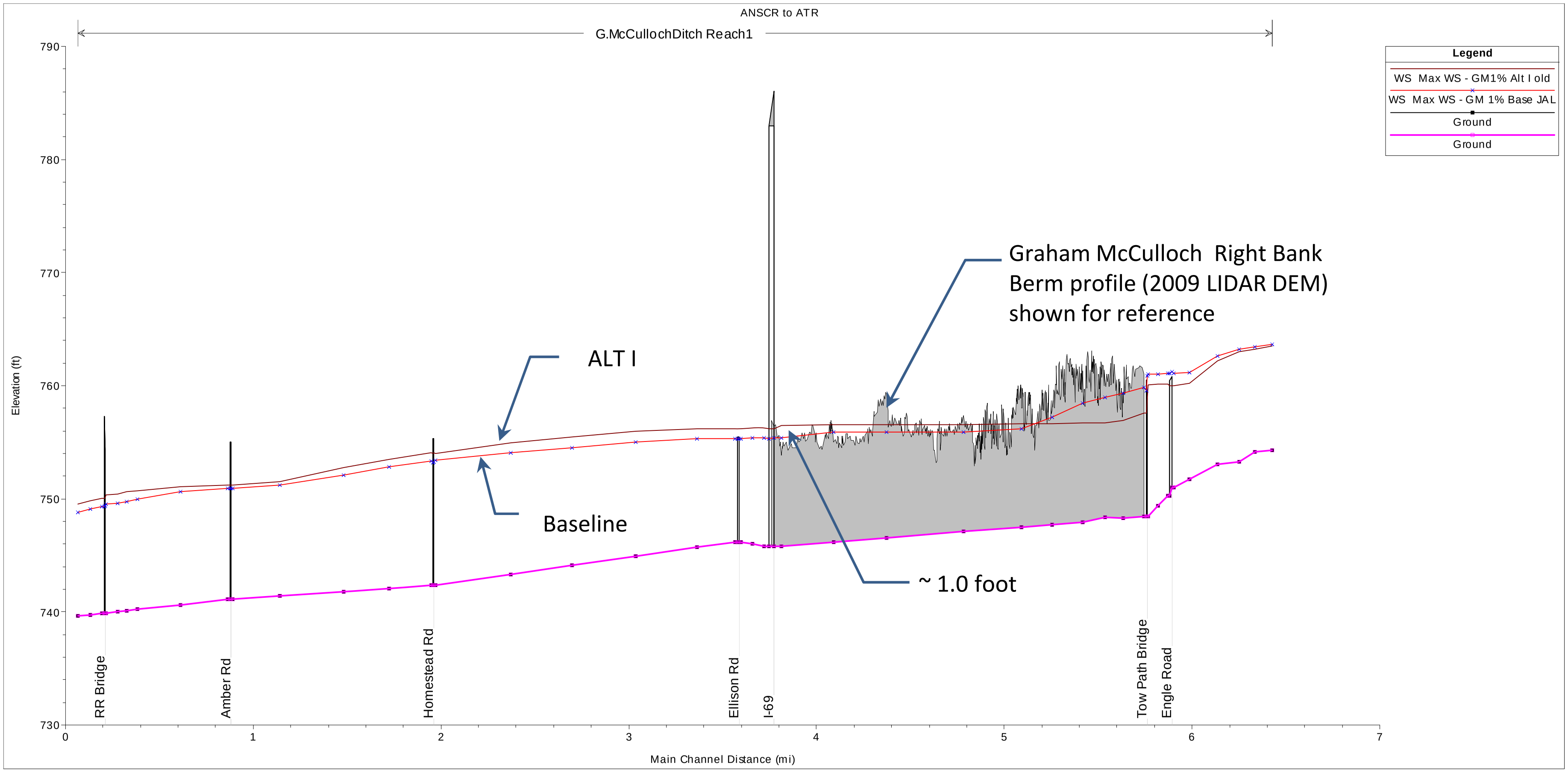


Figure 2.23: Comparison of Baseline and Proposed water surface profiles for Alternative I – Graham-McCulloch Ditch

AQUATIC NUISANCE SPECIES CONTROLS REPORT
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SURVEY AND MAPPING REQUIREMENTS

3.1. EXISTING SURVEY AND MAPPING

The Corps of Engineers obtained various types of data from several sources to be used for the development of the plans for Fort Wayne, Indiana. The Allen County GIS Department provided utility information such as locations of water mains and structures, storm and sanitary sewers and street centerlines. Power and telephone utilities information were not readily available.

The United States Geological Survey (USGS) provided a digital elevation model (DEM) for all of Allen County, Indiana. The DEM was generated from Light Detection and Ranging (LIDAR) data points collected by the Allen County GIS Department in September 2009. The LIDAR data has a vertical precision of 0.116 RMSE (Root Mean Square Error), and horizontal precision of +/- 3.25-ft. The grid cell size of the DEM that was generated from the LIDAR data was 2.5-ft, in an ESRI grid format. Two-ft contours were generated from the DEM for the entire study area in Fort Wayne. All terrain mapping used the horizontal datum NAD83, Indiana State Plane East coordinate system, and a vertical datum of NAVD88.

The National Resources Conservation Service (NRCS) provided surveyed 2-ft contours and spot elevations for the Eagle Marsh area, and surveyed top of berm elevations along the Graham-McCulloch Ditch. This data was used to update the 2.5-ft DEM to create a more precise terrain surface. The data supplied used the horizontal datum NAD83, Indiana State Plane East coordinate system, and a vertical datum of NAVD88.

Aerial photography was downloaded from the Indiana Spatial Data Portal, 2005 Indiana Map Natural Color Orthophotography with 1-ft resolution, and 2008 National Agriculture Imagery Program (NAIP) with 1-meter resolution.

Additional field survey work was required in order to obtain bridge information along Graham-McCulloch and Junk Ditches. This survey work was done by the Corps of Engineers, Louisville District in-house survey team. The bridge information surveyed included cross sections just upstream and/or downstream of the bridge, road profile, low chord, thalweg elevation, width of opening(s), number and size of any piers. The data was surveyed using the horizontal datum NAD83, Indiana State Plane East coordinate system, and a vertical datum of NAVD88.

3.2. FUTURE SURVEY AND MAPPING

Once an alternative is selected, then associated site(s) will need a detailed topographic survey, in accordance with USACE EM 1110-1-1005, Topographic Surveying**.** This reference is available at the following Internet Address http://140.194.76.129/publications/eng-manuals/em1110-1-1005/toc.htm

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4.1. REFERENCES

1) *Wabash-Erie Channel Hydrogeology.* Tony Fleming, LPG (Indiana), January 1996

2) State of Indiana Department of Natural Resources, Division of Water. *Water Resource Availability in the Maumee River Basin, Indiana,* 1996

3) EM 1110-2-1913 "*Design and Construction of Levees*" U.S. Army Corps of Engineers, dated 30 April 2000

4) EM 1110-2-2100 "Stability Analysis of Concrete Structures" U.S. Army Corps of Engineers, dated 01 December 2005

5) "Technical Note-Dual Wall HDPE Perforation Patterns". ADS Pipe, January 2010

6) "WSUD Engineering Procedures: Stormwater". Melbourne Water. CSIRO Publishing, 2005

7) *Foundation Design*. Teng, W.C. Prentice Hall, Englewood Cliffs, NJ. 1969

4.2. GEOLOGICAL CONDITIONS

4.2.1. Regional and Site Geology

The channel which extends from Fort Wayne to Huntington is referred to as the Wabash-Erie Channel. Bedrock in the area is composed of lower to mid-Devonian limestone, dolomite, and gypsum of the Traverse and Detroit River formations. These units overlie Silurian age limestone and dolomite. Several glacier events have occurred in the area. The most recent, known as the Late Wisconsin Stage, gives us the current formations. The Trafalgar formation is a dense glacial till, ranging from 35-100 ft thick in and near the Wabash Erie Channel and overlies the bedrock surface. The second formation is represented by the clayey till highlands know as the Lagro Formation which composes most of the walls of the Wabash Erie Channel. The channel was an active glacial drainage way throughout the late Wisconsin glaciation. A spillway event occurred from glacial melt water when the glacier in the Maumee basin surged, sending a large amount of water down the Wabash Erie Channel, eroding out the Lagro Formation. This event is referred to as the Maumee Torrent. The channel later filled with silt, clay, and sand from river sediment (St. Marys and St. Joseph Rivers flowed through the channel at this time to the Wabash basin) and later organic sediments as it became a wetland due to the Maumee River migrating north and intercepting the St. Joseph and St. Marys Rivers, creating a slack water area in the Channel. A generic profile is shown in Figure 4.1.

Proposed Project sites will generally lie in the silt, sand and clay loam areas of the channel bottom with the potential for organic muck.

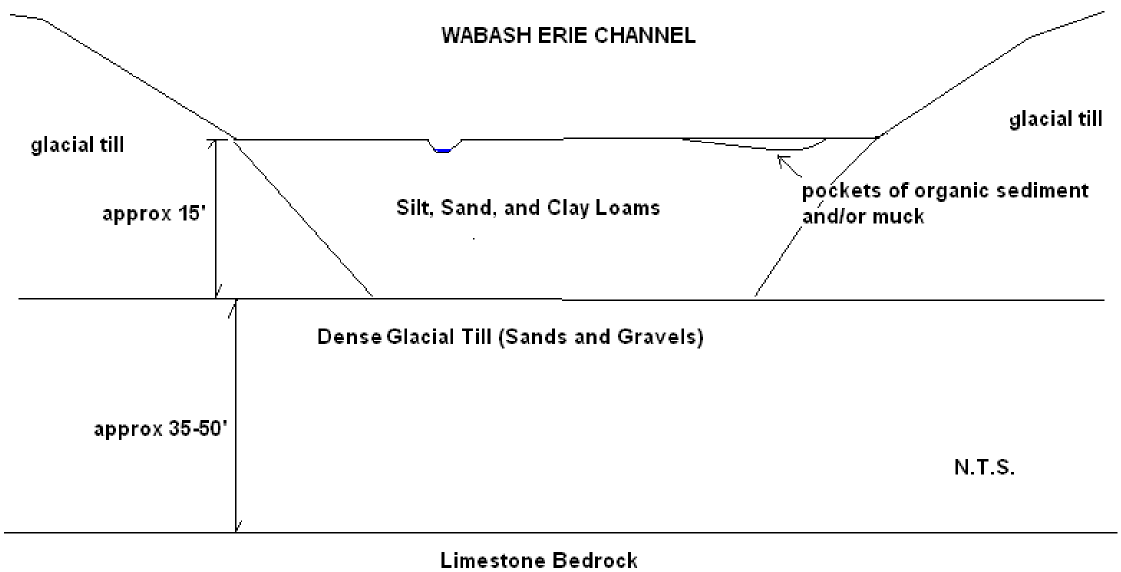


Figure 4.1: General Geologic Cross Section

4.2.2. Seismic Setting

The project site is located in a fairly quiescent region on the periphery of the New Madrid and Western Ohio (Shelby County) region which has the potential to influence ground motions. A site-specific study to determine ground motion parameters was beyond the scope of this report.

The approximate center of the site is found at latitude 41° 1’ 49” N and longitude 85° 14’ 6” W. For this study, the maximum considered earthquake (MCE) ground motions were represented by the spectral response accelerations for 2 percent probability of exceedance (PE) in 50 years, obtained from the 2009 International Building Code published by the United States Geological Survey (USGS). Using the USGS Java Ground Motion Parameter Calculator - Version 5.1.0**,** the spectral response acceleration parameter Ss (0.2 second period) is 0.149*g*, and the spectral response acceleration parameter S_1 at a period of one second is 0.060*g*.

Due to the high groundwater levels and loose sands and gravels present in the area, the upper soils of the Wabash-Erie Canal likely contain the physical properties required for liquefaction. However, due to the generally quiescent regional seismic activity, it is unlikely that the required ground movement will occur in order to induce liquefaction. A site specific liquefaction study was beyond the scope of this report.

4.2.3. Ground Water and Hydrology

The water table is shallow and often at the surface of the Wabash Erie Channel. The channel floor is the lowest elevation of the surrounding area. Wetlands such as Eagle Marsh exist within the basin, and much of the basin is groundwater fed. The limestone bedrock is the primary aquifer system in the area. This bedrock is often in direct contact with the sands and gravels of the Trafalgar formation, creating a large, transmissive groundwater system in the region. Limestone quarries in the area have diminished the groundwater levels in their surrounding area. Typical groundwater levels downstream likely reflect water levels in the Graham-McCulloch Ditch and Little River drainages. It is anticipated that excavations for foundations will encounter groundwater during construction, and that a pumping/groundwater control plan will be required.

4.3. SITE CONDITIONS

A site reconnaissance was conducted on 2 November 2010 by a geotechnical engineer to observe and document surface conditions at the project site. The information gathered was used to help interpret the surface data and to detect conditions which could affect design and construction recommendations.

The exact project site is not yet known as it is dependent on the selected alternative. The Eagle Marsh area is a potential location for multiple alternatives. The marsh is located just south of Engle Road, east of I-69, and bounded to the south by the railroad. At the time of the reconnaissance, the project site was a wetland area which had been experiencing drought conditions.

The surface cover generally consisted of grasses and aquatic vegetation. The topography of the area is relatively flat with a slight rise to the northwest, and the two berms on either side of the Graham-McCulloch Ditch protrude up dissecting the site from northeast to southwest. Other shallow drainage channels exist on the north side of the ditch. Several shallow ponds are found throughout the site. The Graham-McCulloch Ditch was a shallow creek at the time of the reconnaissance. A sewage treatment plant exists at the northeast corner of the site, and the only structure on site is the Little River Wetland Conservancy shed located on the east end of the property, accessible from Engle Road. The Indiana DNR fence crosses the southern half of the site from north to south. The only visible utilities are a large electrical transmission line which runs down the center of the site from northeast to southwest.

4.4. STRUCTURAL ALTERNATIVES

4.4.1. Construct an I-Wall, Alternative A, (Eagle Marsh, Basin Divide)

This alternative consists of constructing a concrete I-wall founded on sheet piling, which would dissect the divide located near Engle Road. The I-wall would tie into high ground just above Engle Road, and will head south and tie into the railroad ballast. Anticipated soil conditions in this area are loose, sandy loam soils. Sheet piling is driven to a predetermined depth based on multiple factors, including the shear properties of the soil, unit weight of the soil, the height (H) of the wall, design loads, and the bending moment of the piling. From *(7) Teng*, the approximate penetration depth for cantilever sheet piling in loose sand is 1.5H. A thorough wall design including a soil investigation should take place in the design stage.

4.4.2. Construct a Fence and Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative B, (Eagle Marsh, Basin Divide)

This alternative will consist of reconstructing a portion of the Graham-McCulloch Ditch berm and tying it into a fence which would cross the divide and tie into the railroad ballast.

The existing berms on both sides of the Graham-McCulloch Ditch appear to be composed of excavations from the initial construction of the ditch, and potentially from additional cleanout thereafter. NRCS personnel have indicated that the previous landowner who farmed the area had fill material brought in to build up the berm. It is not known to what extent or where the borrow material came from. The crowns of the berms are undulating with variable widths, and the footprints are nonlinear, suggesting the material was dumped adjacent to the ditch. Several low spots appear to have been previously overtopped from Graham-McCulloch Ditch flows. Animal burrows and trees are present throughout (Photo 4.1). Several gravity drainage structures cross through the berms, most consisting of CMP piping. In order to ensure satisfactory performance of the section of berm used in this or other alternatives, it is recommended the berm be reconstructed in order to ensure adequate materials, compaction, and factors of safety are met. Depending on the results of design phase geotechnical investigations, there is the potential the berm may generally be allowed to remain in place and be added to with additional material. However, based on surface conditions and the likelihood of variable materials, this seems unlikely.

Photo 4.1: Graham-McCulloch Ditch and Associated Berms

It is anticipated that the majority of the existing berm will be acceptable material for the reconstruction of the berm. Additional material will likely be needed, which can be obtained from other portions of nearby berms or an offsite borrow area. Potential borrow areas were not identified in this study and would need to be procured in the design stage. Acceptable materials for the berm construction generally depend on the shear strength and permeability of the soil. Typically clayey sands, lean clays, and silts (SC, CL, and ML) are acceptable due to their low permeability and moderate shear strengths. The dimension of the berm will depend on slope and seepage calculations, but would likely consist of a 10 ft crown width with three horizontal to one vertical (3H:1V) side slopes. The berm should be constructed an additional six inches above the design elevation to account for post-construction settlement. Compaction of flood control structures is typically required at 95% of the maximum density per ASTM D 698.

4.4.3. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

This alternative consists of creating an earthen berm to cutoff the Graham-McCulloch Ditch near Homestead Road. A pump station would be constructed to pump from upstream to downstream.

The pump station will be a large structure with moderate loading. It will house a total of 11 large pumps with associated valves, piping, overhead crane, and controls. Additional to the pump station, a large discharge well will be required. A deep foundation system such as steel H-piling is anticipated for these structures in order to control potential settlement issues.

The earthen berm will be constructed of nearby excavated materials and will be similar to the berm of the previous alternative. The berm would be required to cut off the Graham-McCulloch Ditch just downstream of Homestead Road and tie into the railroad embankment. Sheet piling may be required through this area. Erosion protection may be required on the upstream face.

4.4.4. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

The permeable berm alternative is based on the theory of allowing floodwater to pass through the berm, yet restricting ANS. Gradation of the stone and perforations of the piping would be considered the restrictive sizes that would prevent ANS from transferring through the berm. Typical perforation sizes for larger HDPE piping is 10mm. Combined with a bedding stone size of INDOT #2, the 10mm should be considered the restrictive opening size. The amount of water that the berm can pass was estimated for this alternative. Based on seepage calculations and gradation permeability analyses, the estimated maximum flowrate through the berm used was 0.2 cfs per ft of berm. Seepage modeling of the proposed berm cross section was performed to gauge the effectiveness of the structure. The addition of piping to intercept and accelerate the movement of water through the berm was developed based on these results. A spreadsheet was created to determine the flow rates through the pipe perforations, which is considered to be the limiting factor for the flowrate.

Hydraulic modeling of the berm is recommended in the design phase to more accurately determine the design flowrate. It may be an option to special order perforated piping which can contain additional perforations to help pass as much flow as possible. Perforated feeder pipes protruding upstream from the main berm could also add to the flowrate by providing additional infiltration.

The telemetered sluice gates will likely be required in order to pass an additional amount of flow to prevent the berm from overtopping. Anticipated foundation types for the sluice gates are spread footing or mat foundation. A larger sluice gate or series of sluice gates will be installed in the main channel of the Graham-McCulloch Ditch in this alternative. Dewatering and/or pump-around during construction will be required.

Table 4.1 contains the flowrate calculations for the perforated piping. Figure 4.2 labels the piping perforation pattern, and Figure 4.3 contains the general berm cross section seepage model.

Permeable Berm Alternative

04/07/11

Reference: WSUD Engineering Procedures: Stormwater. Melbourne Water 2005, Section 5.3.5.1

Perforations inflow Check

ADS Pipe 36" Corrugated (3 ea) Circular slot 10mm dia., 12 rows of perf. , 2 holes at every 45 deg. *water level at top of pipe

perforation config type H AASHTO Class II

g(cm/s^2)	A (cm^2)	C	B	h (ft)*	h (cm)	Q (cm^3/s)	Q (cfs)		# orifices/ft		Total Q in
981	0.785	0.6	2								
				6	182.88	141.06638	0.004982		4		0.019927
				5.56	169.4688	135.79547	0.004796		6		0.028773
				4.5	137.16	122.16707	0.004314		6		0.025886
				3.44	104.8512	106.81374	0.003772		6		0.022633
				3	91.44	99.748993	0.003523		6		0.021136
				2.56	78.0288	92.144173	0.003254		2		0.006508
				1.5	45.72	70.53319	0.002491		2		0.004982
				0.44	13.4112	38.200956	0.001349		2		0.002698
				0	0	0	0		2		0

Perforated Pipe
Capacity
per Bentley Flowmaster
provided by K. Lampkin

Total
0.132542 cfs/ft

slope	pipe dia (in)			full discharge (cfs)
1.00%	60			282
0.50%	60			199
1.00%	36			96
0.50%	36			68

Table 4.1: Perforation Flow Rate Calculations

Figure 4.2: Perforation Pattern

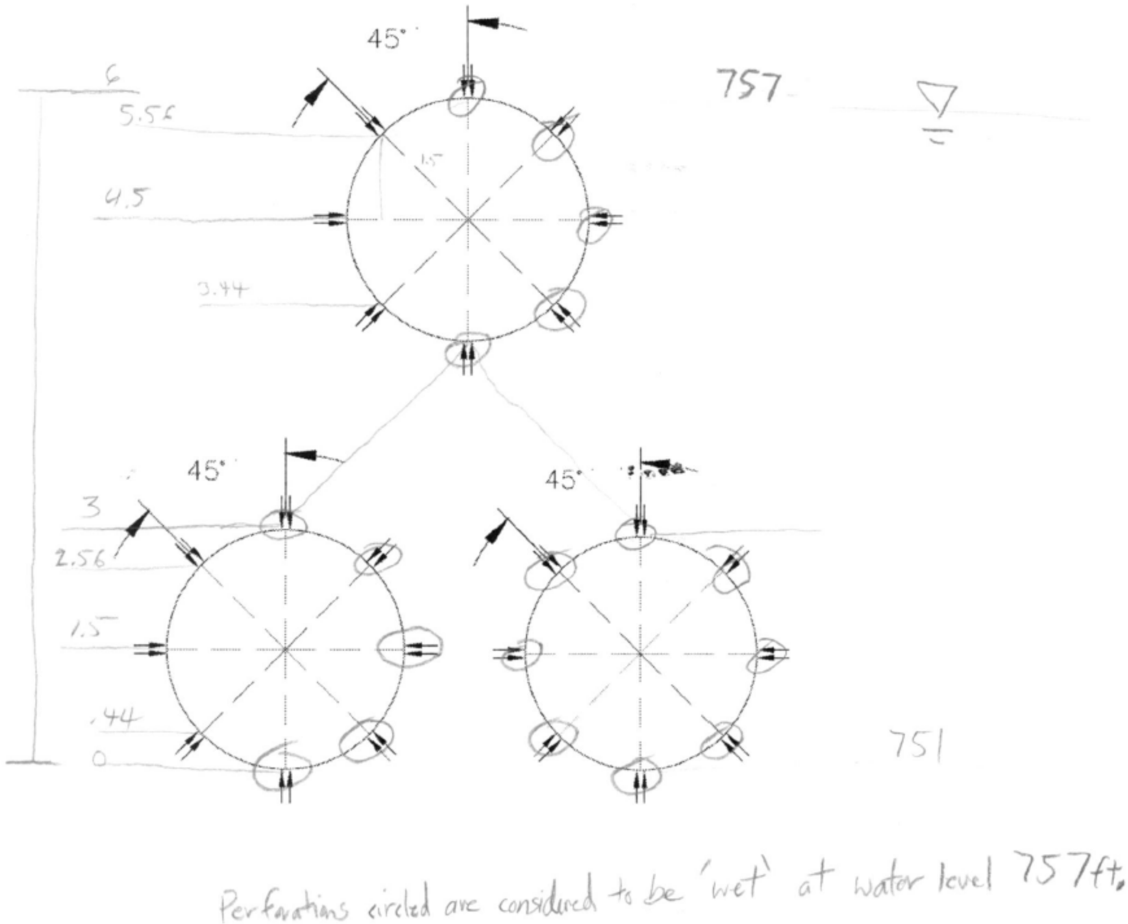
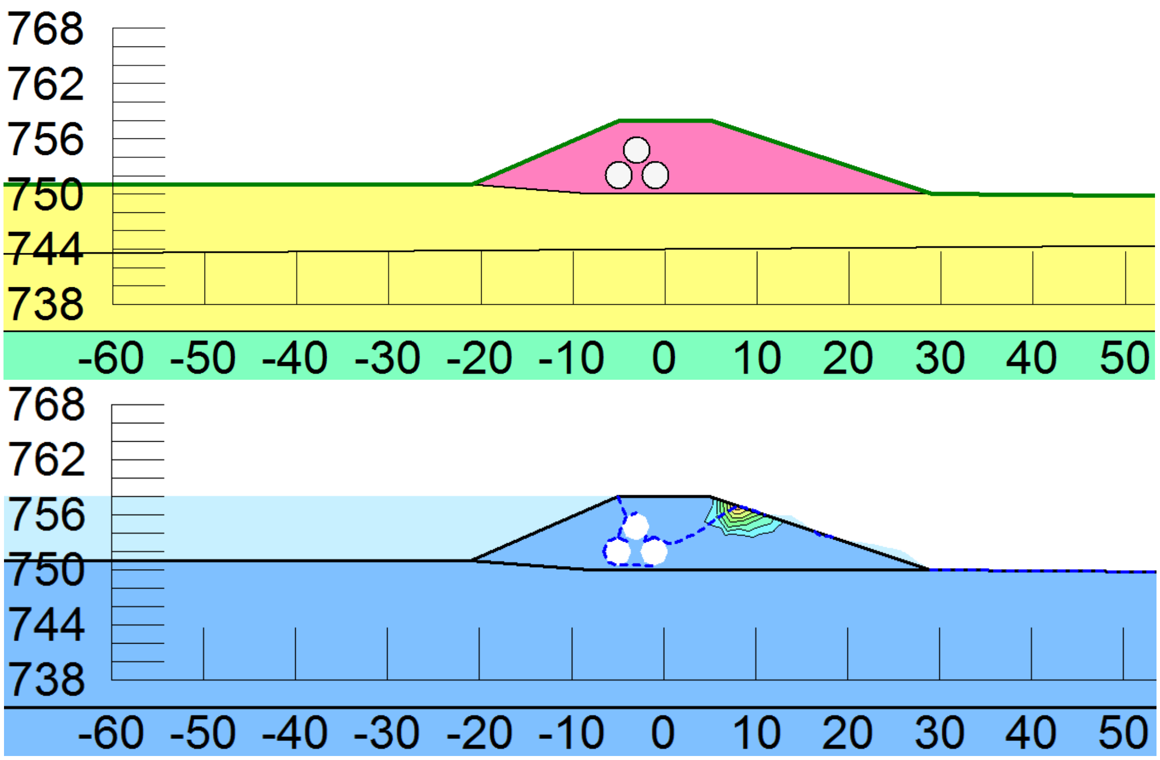


Figure 4.3: Berm Cross Section Seepage Model



4.4.5. Construct a Fence/Earthen Berm Combination, Alternative E, (Eagle Marsh, Basin Divide)

This alternative generally consists of a fence constructed on top of a shortened berm. In theory, only large flood events would overtop the berm, which would then flow through the fence. Erosion protection will need to be in place for this alternative to be effective. Grid type geotextiles, stone protection, or cellular concrete blocks are perceived for use. The erosion protection would be located on the crown, down slope, and a few feet out from the toe, as determined in the design stage.

A borrow site would be needed as the berm would be constructed from borrow material. The width of the berm would likely require the ability to carry a vehicle along its length, in addition to the fence for a significant portion. The berm would generally have 3H:1V side slopes, constructed from compacted clay materials, and would be seeded with grasses. Grade control would be extremely important for this alternative. As the water begins to spill over the berm and through the fence, it is essential that the flow rate be relatively constant over the length of the berm and that flow does not concentrate in any one area. Concentrated flow could lead to a breach of the berm from scour. A concrete sill should be considered along the top of the berm during design.

4.4.6. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

This alternative consists of constructing a bar screen barrier along the top of the Huntington Dam located in Huntington, Indiana. A debris boom would also be required.

The bar screen would be connected through a series of concrete piers constructed downstream adjacent to the dam. These piers would likely be founded on bedrock which is exposed in some locations within the channel (Photo 4.2). The foundation type would be a spread foundation bearing on rock with the potential for rock anchors to help resist lateral loads.

The debris boom cable would need to be anchored on either side of the channel. A concrete dead weight poured on rock is anticipated.

Photo 4.2: Exposed Bedrock at Huntington Dam

4.4.7. Construct Vertical Drop Structures with Telemetered Sluice Gate, Alternative G, (Homestead Road)

The vertical drop structure alternative consists of ten 24 ft diameter circular intake structures tied to box culverts to pass the water through the berm. These structures will be lightly loaded. The anticipated foundation type is a concrete mat foundation. Uplift forces during a submerged condition will need to be considered during design.

The adjacent berm which provides the cutoff and through which the ten box culverts travel will need to be constructed from borrow material. A borrow site will need to be established during the design phase. The berm will likely require erosion protection on the upstream side to protect from wavewash during high water. This berm will be similar in construction to other earthen berms in the previous alternatives.

A sluice gate structure is proposed within the berm section in the Graham-McCulloch Ditch. This sluice gate is anticipated to be founded on a mat or spread footing foundation.

4.4.8. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative H, (Eagle Marsh, Basin Divide)

This alternative provides a permanent cutoff for a major flood event by reconstructing the Graham-McCulloch Ditch left descending berm to engineering standards and a precise elevation. The size of the berm will be dependent on soil properties and seepage/slope stability analyses, but would likely consist of a 10 ft crown width with 3H:1V side slopes. A borrow area will be required for this alternative. The amount of borrow material will depend on the amount of material currently within the existing berm, and also on the percentage of that material which is acceptable for use as fill. As the Graham-McCulloch Ditch was dug onsite and the spoils deposited to create the berm on either side, it is anticipated the berm will mainly consist of the same loam material which is found in the area. However, portions of the berm may be unusable due to organic content or unsatisfactory fill.

Erosion protection on the Graham-McCulloch Ditch side of the berm will likely be required at certain intervals due to high velocity flows within the ditch. The berm should be 'overbuilt' by approximately 6 inches in height to account for post-construction settlement of the berm. More overbuild may be necessary depending on the height and location of the berm and should be analyzed during the design stage.

4.4.9. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area, Alternative I, (Eagle Marsh, Basin Divide)

This alternative will reconstruct the left descending berm the same as the previous alternative, but it assumes the use of the right descending berm as additional borrow material and will likely not require a borrow site. The wetland area would be constructed in the place of the right descending berm. Temporary stream crossings across the Graham-McCulloch Ditch will be needed to transport the borrow material.

4.5. FURTHER STUDIES, FIELD WORK, TESTS AND ANALYSES AFTER THIS REPORT

Additional borings will be required during the next phase to better characterize the soil properties in the area of the final alternative. A geotechnical engineer should perform a soil exploration of the area and develop testing to characterize the soils based on the selected alternative. Seepage and slope stability analyses would be required to evaluate all critical cross-sections of a berm. Updated survey data for all critical cross-sections would be required. Bearing strength and settlement analyses would have to be completed for select alternatives. Further investigation will also be required for potential sources of borrow materials to determine their quantity and suitability.

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5.1. REFERENCES

1) U.S. Army Corps of Engineers document Great Lakes and Mississippi River Interbasin Study (GLMRIS), Other Pathways Preliminary Risk Characterization, dated 9 November 2010

2) Digital elevation model (DEM) from the United States Geological Survey (USGS) for Allen County, Indiana

3) Eagle Marsh mapping from the National Resources Conservation Service (NRCS)

4) ER 1110-2-1150 "*Engineering and Design for Civil Works Projects*" U.S. Army Corps of Engineers, dated 31 August 1999

5) EM 1110-2-1913 "*Design and Construction of Levees*" U.S. Army Corps of Engineers, dated 30 April 2000

6) ETL 1110-2-571, *"Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures"* U.S. Army Corps of Engineers, dated 10 April 2009

7) Roadside Design Guide, American Association of State Highway and Transportation Officials (AASHTO), 4th Edition 2011

8) Policy on Geometric Design of Highways and Streets, AASHTO, 5th Edition 2004

5.2. UTILITY AND ROAD RELOCATION

Utility information for water, storm and sanitary sewers were provided by the Allen County GIS Department. Power and telephone utilities were not readily available and were obtained by field reconnaissance performed by various Project Delivery Team members. Google Maps identified road names. Preliminary utility and road relocations are shown on the plan sheets, which are located in Appendix G, Sheets CS101 through CS114.

Utility and road relocations (as per Engineering Federal Acquisition Regulation Supplement (EFARS), Appendix Q, Section 73-000 et. seq.) are applicable to utilities and roads which involve relocation, abandonment, vacation, or an alteration of an existing utility or road. The term relocation also includes the raising, lowering, altering, adjusting, or protecting a facility, as well as changing its location.

Attorney's reports of compensability will be prepared during the design stage to determine who has a compensable interest and is empowered to contract with the United States of America for utility and road relocations.

5.3. RAILROAD

A single-track railway exists on an embankment located on the southern border of Eagle Marsh and runs in a northeasterly direction. Through phone calls and emails, Norfolk Southern Railroad Corporation has confirmed they operate trains on the railroad through the study area. They have provided right-of-way and track mapping dated 30 June 1919 of their facilities. The mapping has been updated to include the I-69 bridge crossing constructed in 1966 and pipe replacements along the rail alignment. It appears the right-of-way through the study area varies from a minimum of 44 feet to 72 feet on each side of the rail centerline depending on the location.

Photo 5.1: (Left) Single-track railway borders Eagle Marsh, which is located on the left. Photo taken looking northeast direction.
Photo 5.2: (Right) The railway embankment is shown on the left and the south abutment of the temporary fence barrier is shown on right. Photo taken looking in a southwest direction.

Eight of the nine alternatives discussed in this report tie into the existing north side of the railroad embankment, which is also where the railroad communication signals are believed to be located. During design, the exact location of the communication signals will need to be verified with railroad officials. It is not anticipated there will be any disruption to rail traffic during construction of any of the alternatives. However, Norfolk

Southern Railroad Corporation will likely require flagmen to be present during construction activities located with their right-of-way.

Construction activities for this project that are located within the Norfolk Southern Railroad Corporation right of way are applicable to EFARS (Engineering Federal Acquisition Regulation Supplement), Appendix Q, Section 73-000 et. seq. and will require a relocations contract with the railroad.

Attorney's reports of compensability will be prepared during the design stage to determine who has a compensable interest and is empowered to contract with the United States of America for railroad relocations.

5.4. STAGING AREAS

Staging areas are necessary to give the contractor sufficient room to temporarily store construction equipment and/or materials used to construct the project. For this report, the staging areas are located adjacent to the proposed work as much as possible. Keeping the staging areas close to the construction area will reduce the length of transport of construction equipment and materials through public and private properties and will likely reduce real estate costs for the project.

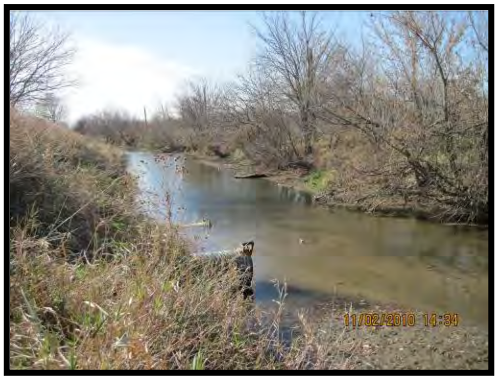
The staging areas are located and identified on the plan sheets for each alternative. The site plans are located in Appendix G, Sheets CS101 through CS114.

5.5. EXISTING PIPES LOCATED WITHIN STUDY AREA

During the November 2010 field reconnaissance trip, the design team discovered multiple abandoned culverts in the Eagle Marsh area, which may allow the passage of ANS during a storm event. The culverts shown in the photos below are located in the left descending berm of the Graham-McCulloch Ditch in Eagle Marsh.

Photo 5.3 and 5.4: (Left and Right) Partially buried and abandoned culverts located in Graham-McCulloch Ditch may allow ANS movement.

Based on discussions with the Indiana DNR, previous efforts to cover these culvert openings with rip rap have failed due to high head differentials experienced during floods on the Graham-McCulloch in the spring of 2011. Indiana DNR plans to plug and cap the culverts that allow movement of ANS to the east side of the existing temporary barrier fence.



Depending on the alternative selected for this project, all culverts located in the left descending bank of the Graham-McCulloch Ditch will be removed in their entirety or filled with expansive grout. Further investigation of these existing pipes will be carried out during the design stage.

Photo 5.5: (Left) Existing pipes in the Graham-McCulloch Ditch will need to be evaluated during design stage.

5.6. REAL ESTATE

Right-of-way was determined based on location of the proposed work, staging areas, and future access for operation and maintenance of the project. Most of the proposed work will be located on private property. Areas needed for construction, staging, or access have been identified on the plans and designated with the appropriate estate. All construction and permanent access points originate from City and State owned roads where possible, which will reduce land acquisition costs. For operation and maintenance purposes, a minimum of twenty feet is reserved on each side of the proposed structure and/or alignment. Additional areas are to be provided at the end of alignments for vehicle turn-around. This access corridor must be free of obstructions to assure adequate access by personnel and equipment for surveillance, inspection, maintenance, and monitoring.

In some instances where the road may act as part of the barrier to prevent movement of the ANS, right-of-way may be carried across the road to ensure it is not altered in a way that would enable ANS transfer.

Flowage easements will be required for a majority of the alternatives. Further investigations will be completed during the design stage of the project. Additional information regarding flowage easements is explained in the Real Estate Plan, Appendix E.

Real Estate plans have been included in Appendix G, Sheets RW101 through RW114.

5.7. STRUCTURAL ALTERNATIVES

5.7.1. Construct an I-Wall, Alternative A, (Eagle Marsh, Basin Divide)

Alternative A is proposed to construct about 4,448 linear feet of sheet pile I-wall at the approximate location of the hydraulic basin divide. The top of wall is proposed to be at elevation 762.0 at Engle Road and sloping downward to elevation 760 at a point near the existing barn, and continuing at elevation 760 to its south termination. This elevation is based on hydraulic models of the area to prevent the movement of ANS. Additional hydraulic information is located in Section 2, Hydrology and Hydraulics, of this report. The I-wall height varies from about three to nine feet dependent on the existing ground elevation along the alignment.

The south end of the alignment ties into high ground at the Norfolk Southern Railroad embankment. To minimize the disturbance to the railroad, the I-wall will stop at the toe of the railroad embankment. Indiana revetment riprap will be placed between the end of the I-wall and the railroad embankment to prevent passage of ANS. The I-wall will run northward which will require about 2,000 linear feet of clearing and grubbing through a wooded area of Eagle Marsh. The I-wall alignment will then tie into the Engle Road embankment on the south and north sides of the road.

To avoid a gate or sand bag closure across Engle Road during storm events, a berm will be constructed at elevation 762.0 along the south side of Engle Road connecting the I-wall to high ground at the left bank abutment of the Engle Road bridge over the Graham McCulloch Ditch. Raising the road was considered in lieu of this berm, but due to the impacts on new trail areas and adjacent marsh areas on the north side of Engle Road, constructing this berm would be less intrusive. Raising the road would also have significant costs for maintaining traffic on this high volume road during construction.

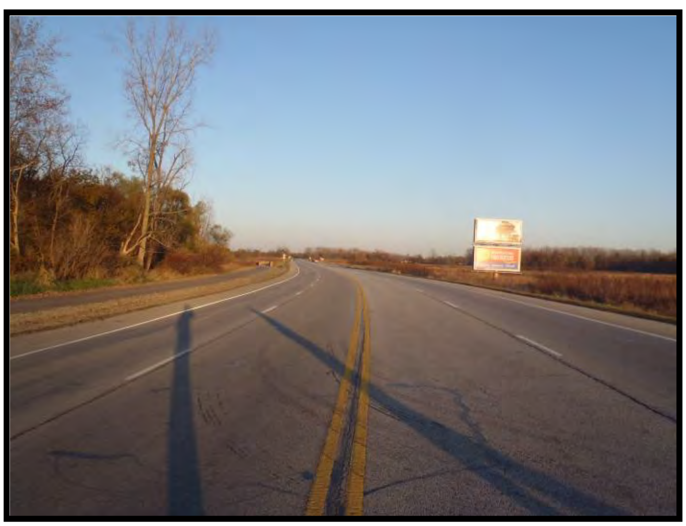


Photo 5.6: (Left) Engle Road in the vicinity of the Alternative A I-wall.

Vehicular or pedestrian gates were not provided due to the flashiness of the storms in the study area. In addition, the team concluded gates would only increase the operation and maintenance costs associated with this project which would be counterintuitive based on our objectives as defined in Section 1, General.

During the initial visit to Eagle Marsh in November 2010, design team members met with the Little Rivers Wetlands Project (LRWP) board who operates Eagle Marsh. They were concerned the existing temporary fence barrier prevents the migration of terrestrial animals such as deer. Therefore, three animal crossing ramps were provided in Eagle Marsh to provide crossing of the I-wall. The animal crossings consist of a 3:1 compacted fill with a geogrid surface on both sides of the I-wall. Reference is made to Miscellaneous Details, Sheet CZ001 in Appendix G.

High-voltage overhead electric transmission lines run through the wooded area across Eagle Marsh. Any relocation of these lines would be cost prohibitive since they likely are part of the Eastern Interconnection power grid. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the I-wall to be constructed. However, this will need to be verified during final design if this alternative is selected. Electric lines are also located on high ground north of Engle Road. These lines will unlikely be affected, but during construction the contractor will need to use caution when working in the area. It appears a utility relocations contract will not be necessary.

The site plan for Alternative A is located on plan sheet CS101 in Appendix G.

5.7.2. Construct a Fence and Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative B, (Eagle Marsh, Basin Divide)

Alternative B is proposed to construct about 1,774 linear feet of permanent chain-link fence and about 2,782 feet of earthen berm. The proposed fence will be located east of the Indiana DNR temporary fence in Eagle Marsh. The top of the fence is proposed to be at elevation 760.0 and is based on hydraulic models of the area to prevent the movement of ANS. Additional hydraulic information is located in Section 2, Hydrology and Hydraulics, of this report. The fence height varies from seven to nine feet dependent on the existing ground elevation along the alignment.

The south end of the alignment ties into high ground at the Norfolk Southern Railroad embankment west of the wooded area to avoid clearing and grubbing activities. To minimize the disturbance to the railroad, the fence will stop at the toe of the railroad embankment. Indiana revetment rip rap will be placed between the end of the fence and the railroad embankment to prevent passage of ANS. The fence alignment runs north where it ties into the left descending (east) bank of the Graham-McCulloch Ditch.

Approximately 2,782 linear feet of the existing left descending bank of the Graham-McCulloch Ditch will be demolished and rebuilt. The reconstructed berm will begin at the fence tie-in location and end at wastewater treatment plant (WWTP) access road. New berm will be constructed along the south side of the WWTP road to Engle Road, and continue along the south side of Engle Road to high ground at the left bank abutment of the Engle Road Bridge over Graham McCulloch Ditch. Additional information on the condition of the existing Graham-McCulloch Ditch berms is located in Section 1, General, and Section 4, Geotechnical.

The top of crown of the new earthen berm will be at elevation 760 at its tie in to the fence, and tapering up to elevation 762 at the WWTP road, and contining at elevation 762 to its termination. The berms will have 3:1 side slopes. The design team recommends a minimum ten-foot wide crown be constructed on the berm for LRWP maintenance vehicles. This detail can be discussed further and decided upon during the design stage of the project. However, for cost estimating purposes, the crown width will be ten feet. A shorter alignment of the new berm that would follow the left bank of Graham-McCulloch Ditch in lieu of the longer route following the roadways was considered; the longer route was selected as it would allow the roadway to the WWTP to be adjusted without modification to the existing bridge there, and should have less impact to tow path trail improvements in the area.

High-voltage overhead electric transmission lines cross the Graham-McCulloch Ditch north of the fence tie-in location in Eagle Marsh. Any relocation of these lines would be cost prohibitive since they likely are part of the Eastern Interconnection power grid. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the reconstruction of the Graham-McCulloch Ditch. However, this will need to be verified during final design if this alternative is selected.

Photo 5.7: (left) Photo taken from Towpath Trail bridge looking downstream at Graham-McCulloch Ditch. The wastewater treatment plant is on right.
Photo 5.8: (right) Towpath Trail (before 2011 improvement) looking northeast. The wastewater treatment plant is on the right.

The wastewater treatment plant, which is located on the right descending bank of the Graham-McCulloch Ditch, will not be affected by this alternative. The Graham-McCulloch Ditch berm ties into the wastewater treatment access road. Between Engle road and the Tow Path Trail, the access road is a low volume, two-lane gravel road. In 2011, this road was asphalted from a point near the bridge crossing the Graham-McCulloch Ditch and continuing further to the southwest as part of a larger Fort Wayne Parks Towpath Trail improvement project, for the purposes of recreational bicycling and hiking access. It is anticipated that the wastewater treatment plant access portion of this road will be briefly closed during the demolition and reconstruction of the berm adjacent to and crossing this road. Due to recent construction in the vicinity of Engle Road for the Towpath Trail and trailhead parking area, if this alternative is selected for further design, additional surveying will be required to determine the final termination point of the berm in the vicinity of Engle Road. A road relocation contract will likely be required for modifications to the wastewater treatment access road. In addition, approximately 95 linear feet of Eagle Marsh access road, which is a one-lane gravel road, will also need to be relocated as part of this alternative.

The site plan for Alternative B is located on plan sheet CS102 in Appendix G.

5.7.3. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

Alternative C is proposed to construct an earthen berm which prevents the Graham-McCulloch from freely flowing downstream. The earthen berm would divert flow to a pump station which would act as a filter to reduce the likelihood of ANS movement further downstream. The earthen berm and pump station would be located west of Homestead Road near the Norfolk Southern Railroad Corporation at-grade intersection. Additional pump station information is located in Sections 7 and 8, Electrical and Mechanical Engineering, of this report.

The purpose of the pump station is to handle peak flows from the 1% annual chance event on the Graham-McCulloch Ditch. The pump station will be configured utilizing eleven "can sytle" pumps. Eight of the pumps will have an approximate capacity of 130 CFS and the remaining three pumps will be approximately 50 CFS. Each discharge pipe will have a gravity type flap gate to prevent back flow.

Photo 5.9 (left): Homestead Road is located on right. Photo is looking north

About 175 linear feet of earthen berm would be constructed to an elevation of 758.0. The south abutment of the earthen berm would tie into the Norfolk Southern Railroad embankment and the north abutment would tie into Homestead Road.

To prevent water from overtopping Homestead Road, it would need to be raised in place to an approximate elevation of 758.0. Road work will begin north of the bridge which is estimated to be at an approximate elevation of 758.0. Approximately 726 linear feet of Homestead Road is to be altered. Since ponding of water east of Homestead Road may occur during storm events, Indiana revetment rip rap is to be placed on the upstream road embankment to prevent erosion. A road relocation contract will be required.

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The farmer's access on each side of Homestead Road will be maintained. The existing ditch which lies west of Homestead Road (see Photo 5.9) would be diverted downstream of the earthen berm and pump station.

About 1,055 line feet of power and telephone lines and four joint-use utility poles will need to be adjusted on existing alignment. This work will be completed by the owning utility companies. Buried fiber optic cable, located adjacent to the road, will be protected and not be disturbed during construction activities. A utility relocation contract will be required.

The site plan for Alternative C is located on plan sheets CS103 and CS104 in Appendix G.

5.7.4. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

Alternative D is proposed to construct about 2,113 linear feet of permeable berm that allows water to pass through during high water events but restricts ANS. The berm would be made of open graded rip rap material and would be located east of Amber Road. Gradation of the stone and perforations of the piping would be considered the restrictive sizes that would prevent ANS from passing through the berm. See Section 4, Geotechnical Engineering, of this report for further description of the berm composition. The south abutment of the permeable berm would tie into the Norfolk Southern Railroad embankment, cross the Graham-McCulloch Ditch and tie into high ground. The alignment of the berm is approximately parallel with Amber Road.

On the Graham-McCulloch Ditch, a telemetered sluice gate with automated closure mechanisms will be constructed. During storm events when ANS transfer is imminent, the sluice gates would close blocking normal drainage of the Graham-McCulloch Ditch. Three gaging stations are proposed; one on Junk Ditch (Engle Road) and two on Graham-McCulloch Ditch (Eagle Marsh and Ellison Road). Additional hydraulic information about telemetered sluice gates is located in Section 2, Hydrology and Hydraulics, of this report.

Photo 5.10: (left) Alternative at Amber Road, looking north.

Flow that normally passes through the Graham-McCulloch Ditch would be redirected to pass through the permeable berm. To pass maximum flow during storm events, islands extending from the berm will be constructed. Ponding of the water may occur upstream of the permeable berm. Additional hydraulic analysis will be required during the design stage to determine the ponding areas and associated flowage easements needed for this alternative.

To reduce the amount of debris that may build-up on the permeable berm, it is proposed a vegetative filter strip be constructed 200-300 feet upstream. The filter strip is proposed to be a 75-foot wide by 1,711 long area of permanent vegetation that will consist of a combination of native, grasses, trees and shrubs.

A twelve-foot gravel access road will be constructed from Amber Road to the project area. Total length of the gravel access road is estimated to be approximately 200 linear feet. Utilities are not located within the project area. A utilities or road relocation contract will not be required for this alternative.

The site plan for Alternative D is located on plan sheets CS105 and CS106 in Appendix G.

5.7.5. Construct a Fence/Earthen Berm Combination, Alternative E, (Eagle Marsh, Basin Divide)

Alternative E is proposed to construct about 4,825 linear feet of permanent chain-link fence and earthen berm combination at the approximate location of the hydraulic basin divide in Eagle Marsh. The top of fence and earthen berm are proposed to be at elevations 762.0 and 753.0 respectively and are based on hydraulic models of the area to prevent the movement of ANS. Additional hydraulic information is located in Section 2, Hydrology and Hydraulics, of this report. The fence height varies from two to nine feet and is dependent on the existing ground elevation along the alignment.

The south end of the alignment ties into high ground at the Norfolk Southern Railroad embankment. To minimize the disturbance to the railroad, the fence will stop at the toe of the railroad embankment. Indiana revetment rip rap will be placed between the end of the fence and the railroad embankment to prevent passage of ANS. The fence will run northward for about 1,200 linear feet until approximate ground elevation 753.0 where the fence/earthen berm combination will begin. The alignment will continue making a slight curve left until it meets the existing Eagle Marsh access road. It is estimated about 2.1 acres of clearing and grubbing along 2,280 linear feet of fence alignment will be required for this alternative.

The existing Eagle Marsh gravel access road will be raised in place about one foot and will serve as an earthen berm for high frequency flooding. An equipment gate will be constructed where the fence crosses the access road. The fence will continue north and be located approximately 250 feet west of the access road.

To avoid a gate or sand bag closure across Engle Road during storm events, a berm will be constructed along the south side of Engle Road to elevation 762 to connect this fence to high ground at the left bank abutment of the Engle Road bridge over the Graham-McCulloch Ditch. This would avoid significant maintenance of traffic costs associated with raising Egngle Road, a high-volume road, which was considered in lieu of this berm.

During the initial visit to Eagle Marsh in November 2010, design team members met with the Little Rivers Wetlands Project (LRWP) board that operates Eagle Marsh. They were concerned the existing temporary fence barrier prevents the migration of terrestrial animals such as deer. Therefore, pedestrian and equipment gates will be provided along the fence alignment in Eagle Marsh. Reference is made to Fence Details, Sheet CZ002 in Appendix G.

High-voltage overhead electric transmission lines run through the wooded area across Eagle Marsh. Any relocation of these lines would be cost prohibitive since they likely are part of the Eastern Interconnection power grid. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the I-wall to be constructed. However, this will need to be verified during final design if this alternative is selected. Electric lines are also located on high ground north of Engle Road. These lines will unlikely be affected but the contractor will need to use caution when working in the area. It appears a utility relocations contract will not be necessary.

The site plan for Alternative E is located on plan sheet CS107 in Appendix G.

5.7.6. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

Alternative F is proposed to construct a bar screen across the Huntington Dam on the Little River in Huntington, Indiana. The bar screen structure would be constructed downstream of the existing weir to prevent movement of ANS past the barrier. The bar screen would be slanted downstream to prevent any ANS to jump over the bar screen. The angle of the bar screen would allow any floating debris that may pass the floating debris boom to float upward during high flows and be pushed over the bar screen. The work will include construction of about 327 linear feet of gravel access road off East State Street and about 320 linear feet of gravel access road off W. Riverside Drive.

High-voltage overhead electric transmission lines are located on the right descending bank of the Little River. Any relocation of these lines would be cost prohibitive. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the bar screen barrier construction. However, equipment needed for debris removal may have clearance issues at this location and will need to be verified during final design.

The site plan for Alternative F is located on plan sheet CS108 in Appendix G.

5.7.7. Construct Vertical Drop Structures with Telemetered Sluice Gate, Alternative G, (Homestead Road)

Alternative G is proposed to construct a total of ten 24 foot diameter weir inlets upstream of Homestead Road, which will not be effected by this alternative. In conjunction with the vertical drop structures, about 750 linear feet of permeable berm will be constructed.

Vertical drop structures would act similar to control structures used in lakes, ponds, or detention basins. Low flows would pass through a sluice gate structure in the main channel of the Graham-McCulloch Ditch. The gate would be controlled by gages closing when flows increased and threatened to create a hydraulic connection between basins. After the sluice gate closed, the water surface elevation immediately upstream of the barrier would increase and overtop the drop structures. Each drop structure would pass flows through the berm to a flat-bottom ditch adjacent to Homestead Road by a concrete box culvert. The flat-bottom ditch empties into the Graham-McCulloch Ditch.

An access road from Homestead Road will be necessary for permanent access to the project. Overhead utility lines are located on the downstream side of Homestead Road and will not be affected. It is anticipated that a relocations contract with the Norfolk Southern Railroad Corporation would be necessary to tie into the railroad embankment.

The site plan for Alternative F is located on plan sheets CS109 and CS110 in Appendix G

5.7.8. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative H, (Eagle Marsh, Basin Divide)

Alternative H is proposed to reconstruct about 8,700 linear feet of earthen berm. The left descending berm of the Graham-McCulloch Ditch would be demolished from the south end of the alignment where it ties into high ground at the Norfolk Southern Railroad to the north end at the wastewater treatment access road. The reconstructed berm will generally be constructed in the same location as the existing berm.

The top of crown of the new earthen berm will range from 761.0 at the wastewater treatment access road to 759.0 at the Norfolk Southern Railroad. Due to recent construction in the vicinity of Engle Road for the Towpath Trail and trailhead parking area, if this alternative is selected for further design, additional surveying will be required to determine the final termination point of the berm in the vicinity of Engle Road. The side slopes of the berm will be 3H:1V. The design team recommends a minimum ten-foot wide crown be constructed on the berm for LRWP maintenance vehicles. The berm detail will be discussed further and decided upon during the design stage of the project. However, for cost estimating purposes, the crown width will be ten feet.

The wastewater treatment plant, which is located on the right descending bank of the Graham-McCulloch Ditch, will not be affected by this alternative.

Photo 5.11: (left) Location where Graham-McCulloch Ditch and the railroad meet in Eagle Marsh.

The reconstruction of the left descending berm at the wastewater treatment plant access road, which is a low volume, two-lane gravel road, will likely require a road relocation contract. It is anticipated that this access road will be briefly closed during the demolition and reconstruction of the berm adjacent to the road.

High-voltage overhead electric transmission lines cross the Graham-McCulloch Ditch at the 90 degree bend of the ditch in Eagle Marsh. Any relocation of these lines would be cost prohibitive since they likely are part of the Eastern Interconnection power grid. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the reconstruction of the Graham-McCulloch Ditch. However, this will need to be verified during final design if this alternative is selected. Utility relocations are not expected.

A relocations contract with the Norfolk Southern Railroad Corporation would be necessary to tie into the railroad embankment.

The site plan for Alternative H is located on plan sheets CS111 and CS112 in Appendix G.

5.7.9. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area, Alternative I, (Eagle Marsh, Basin Divide)

Alternative I is proposed to reconstruct about 8,700 linear feet of earthen berm. The left descending berm of the Graham-McCulloch Ditch and most of the right descending berm would be demolished from the south end of the alignment where it ties into high ground at the Norfolk Southern Railroad to the north end at the wastewater treatment access road. The left descending berm would be reconstructed and would be in about the same location as the existing berm.

The top of crown of the new earthen berm will range from 761.0 at the wastewater treatment access road to 759.0 at the Norfolk Southern Railroad. Due to recent construction in the vicinity of Engle Road for the Towpath Trail and trailhead parking area, if this alternative is selected for further design, additional surveying will be required to determine the final termination point of the berm in the vicinity of Engle Road.

The side slopes will be 3H:1V. The design team recommends a minimum ten-foot wide crown be constructed on the berm for LRWP maintenance vehicles. The berm detail will be discussed further and decided upon during the design stage of the project; however, for cost estimating purposes, the crown width will be ten feet.

The wastewater treatment plant currently has a berm system around the plant to protect it from flooding. In addition, it appears that a portion of the right descending bank of the Graham-McCulloch Ditch has been reconstructed as part of their berm system. Based on survey data, the average height of the berm system around the plant is at elevation 762.0. Reference is made to Photo 5.13, which shows the right descending bank adjacent to the plant. The bank/ berm appear to be well maintained and securely gated. Therefore, it is assumed the right descending bank adjacent to the plant will remain in place. Further investigation will be required during the design stage if this alternative is selected for construction. The construction of this alternative will not affect the wastewater treatment plant.

The reconstruction of the left descending berm at the wastewater treatment access road, which is a low volume, two-lane gravel road, will likely require a road relocation contract. It is anticipated that the wastewater treatment access road will be briefly closed during the demolition and reconstruction of the berm adjacent to the road.

High-voltage overhead electric transmission lines cross the Graham-McCulloch Ditch at the 90 degree bend of the ditch in Eagle Marsh. Any relocation of these lines would be cost prohibitive since they likely are part of the Eastern Interconnection power grid. From preliminary site reconnaissance, the low wire elevation appears to be high enough for the reconstruction of the Graham-McCulloch Ditch. However, this will need to be verified during final design if this alternative is selected. Utility relocations are not expected.

Photo 5.12: (left) An earthen berm surrounds the wastewater treatment plant to protect it from flooding.
Photo 5.13: (right) The wastewater treatment plant utilizes a portion of the right descending bank of the Graham-McCulloch Ditch as part of their berm system to protect the plant from flooding.

A relocations contract with the Norfolk Southern Railroad Corporation would be necessary to tie into the railroad embankment.

The site plan for Alternative I is located on plan sheets CS113 and CS114 in Appendix G.

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6.1. REFERENCES

1) ACI 318-08, "*Building Code Requirements for Structural Concrete and Commentary*" American Concrete Institute

2) ACI 350-06, "*Environmental Engineering for Concrete Structures*" American Concrete Institute

3) ASCE 7-05, "*Minimum Design Loads for Buildings and Other Structures*" American Society of Civil Engineers

4) EM 1100-2-2100, "*Stability Analysis of Concrete Structure*" U.S. Army Corps of Engineers, dated 01 December 2005

5) EM 1100-2-6053, "*Engineering and Design - Earthquake Design and Evaluation of Concrete Hydraulic Structures*" U.S. Army Corps of Engineers, dated May 2007

6) ER 1110-2-1806, "*Earthquake Design and Evaluation for Civil Works Projects*" U.S. Army Corps of Engineers, dated 31 July 1995

7) REMR-EM-6, "*Mechanical Properties and Corrosion Behavior of Stainless Steels for Locks, Dams, and Hydroelectric Plant Application*" published by Dr. Ashok Kumar, US Army Construction Engineering Research Laboratories (1988)

8) REMR-EM-11, "*Coatings for Use on Wet or Damp Steel Surfaces*", published by Alfred D. Beitelman, US Army Construction Engineering Research Laboratories (1997)

9) UFC 3-301-01, "*Unified Facilities Criteria – Structural Engineering*" US Army Corps of Engineers/Naval Facilities Engineering Command/Air Force Civil Engineer Support Agency.

6.2. STRUCTURAL ALTERNATIVES

6.2.1 Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

Alternative C involves the construction of a pump station at Homestead Road near the Graham-McCulloch Ditch and Norfolk Southern Railroad Corporation. The alternative includes options for two configurations of pump stations. Either pump station should consist of reinforced, cast-in-place concrete, which includes the pump bay, walls and building floor slab, and pipe inlets/outlets. Foundations should be founded to the frost penetration depth and that depth used in the stability analyses. Deep foundations may be required based on the existing soil conditions. The superstructure building should be constructed of reinforced concrete masonry unit (CMU) block or concrete with a roof support system of precast concrete deck planks. Architectural features will include the roof system and water proofing. Either pump plant configuration includes a structural steel trash rack, assumed to be fabricated from steel bars a minimum of 3-inches deep, spaced no further than 2-inches center-to-center, and a coating system that is capable of being submerged for extended periods of time (comply to REMR-EM-6 or REMR-EM-11 for stainless steel selection or coatings). Conceptual sketches are included shown on Alternative C Site Plan CS103 in Appendix G.

6.2.1.1. Design Criteria

Screening level designs for the pump plant configurations have been completed. The functional layouts were developed by the project mechanical engineer. Computations of the structures' stability were utilized to develop wall and slab thicknesses. Estimates of the concrete and other material quantities have been completed. During design, the pertinent criteria of all applicable publications need to be met.

6.2.1.2. Stability and Strength Requirements

The criteria used to analyze the pump plant stability are based on EM1110-2-2100, *Stability Analysis of Concrete Structures* and EM1110-2-3104, *Structural and Architectural Design of Pumping Stations*. The proposed pump plants were designed per the design flood case, which by definition, is an unusual load condition and the soil information was considered to be "ordinary". The required Factors of Safety for Sliding and Flotation are outlined in Tables 6.1 and 6.2. These tables are excerpted from Chapter 3 of EM 1100-2-2100. The load cases to be analyzed are outlined in detail in Table 6.3.

TABLE 6.1
REQUIRED FACTORS OF SAFETY FOR SLIDING - CRITICAL STRUCTURES

Load Condition Categories Site Information Category	Usual	Unusual	Extreme
Well Defined	1.7	1.3	1.1
Ordinary	2.0	1.5	1.1
For preliminary seismic analysis without detailed site-specific ground motion	-	1.7	1.3
Limited	-	-	-

TABLE 6.2
REQUIRED FACTORS OF SAFETY FOR FLOTATION – ALL STRUCTURES

Load Condition Categories Site Information Category	Usual	Unusual	Extreme
All Categories	1.3	1.2	1.1

Appendix B ("Loading Conditions and Loading-Condition Classification") from EM 1110-2-2100 includes the following description of the load cases to be analyzed for pump plant structures.

(a) Loading Condition 4-4a – Construction Condition.
- Pumping station complete with and without backfill in place.
- No water loads.

(b) Loading Condition 4-4b - Normal Operating Condition.
- Plant operating to discharge routine local floods over a range of exterior flood levels with a maximum 2-year return period.

(c) Loading Condition 4-4e - Maximum Design Flood.
- Maximum water level outside protection line.
- Minimum pumping level inside.

(d) Loading Condition 4-4f - Maximum Pump Thrust.
- Maximum operating floods both inside and outside protection line.
- Maximum pump thrust.

(e) Loading Condition 4-4g - Maintenance Conditions.
- Maximum design water level inside.
- One, more, or all intake bays unwatered.

(f) Loading Condition 4-4j - Pumping Plant Inundated.
- Maximum flood levels inside and outside protection line.
- Pumping plant inoperative.
- Foundation drains inoperative.

• Protection line intact.

(g) Loading Condition 4-4k - Coincident Pool + OBE.
- Coincident pool
- OBE in most critical direction.

(h) Loading Condition 4-4l- Coincident Pool + MDE.
- Coincident pool
- MDE in most critical direction.

TABLE 6.3
PUMPING PLANT LOADING -CONDITION CLASSIFICATION
STRUCTURE TYPE: PUMPING PLANTS, EM 1110-2-3104

Load Case	Loading Description	Classification
4-4a	Construction Condition	UN
4-4b	Normal Operating Condition	U
4-4e	Maximum Design Flood (MDF)	U/UN/E
4-4f	Maximum Pump Thrust	U/UN/E
4-4g	Maintenance Condition	UN
4-4j	Pumping Plant Inundated	E
4-4k	Coincident Pool + Operating Basis Earthquake (OBE)	UN
4-4l	Coincident Pool + Maximum Design Earthquake (MDE)	E

It is noted that the stability requirements based on EM 1110-2-3104 are more stringent than those in the newer EM 1110-2-2100; however the Transmittal letter accompanying EM 1110-2-2100 states the newer document was developed to “provide adequate safety factors for all types of structures and loading conditions, while reducing excess conservatism for infrequent loadings of short duration. This will result in project cost savings when compared to some structures designed using previous criteria. Stability criteria in other manuals are being revised to be consistent with this manual. In the interim, where there are conflicting stability criteria, the provisions of this manual shall govern.”

The proposed optional layouts have been designed to a screening level without a sheet pile cutoff or toe drains. Global stability is achieved by increasing the structural mass to resist external loads. A high water table was assumed to the ground surface on the protected or resisting side of the plant, even though it was also assumed that the pump bay was only filled to the minimum pump elevation. Thus the analyses are conservative.

6.2.1.3. Seismic Stability and Frost Penetration

A seismic analysis should be performed for the proposed pump stations. Seismic forces should be developed using pseudo statically-derived accelerations. The resulting seismic forces will then be compared to the flood-induced water and/or soil forces applied to the pump plant structures. As seismic loads are not imposed simultaneously with the flood loads, a determination should be made as to whether or not the seismic loads control the stability or structural analysis of the pump plant. Although the seismic load condition is not expected to be a controlling case, it should still be checked.

When designing the foundation of the pump stations, resistance to frost upheaval should be considered. The foundation should be founded to a depth of 49 inches below grade to account for frost penetration, per UFC 3-301-01. Frost depth was estimated by the nearest location listed in the UFC, Grissom Air Reserve Base (ARB). Given that Huntington is more north of the ARB, frost penetration could be slightly higher, but the variation inherent due to this assumption is negligible for cost estimating purposes. Detailed investigation will need to be performed in order to obtain a more accurate value during design.

6.2.2. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

The Huntington Dam is a six foot high concrete dam structure on the Little River in Huntington, Indiana. The dam is approximately 20 miles west of Eagle Marsh, and it is presently unknown as to who owns or has authority over the structure. The current condition of the structure has not been verified, but it should be noted that water is undercutting the dam at about mid-channel (see Photo 6.1). Ideally, the construction/design of the physical barrier should only minimally impact the existing structure, if at all. Any construction that modifies the existing dam would hold the customer liable in a failure event.

Photo 6.1: Huntington Dam during Summer Flow

Photo 6.2: Overhead View of Huntington Dam

6.2.2.1. Description of Alternative

The concept behind Alternative F is to create a physical barrier adjacent to the existing dam to prevent the invasive species from swimming upstream during normal flow conditions as well as up to that of the 1% annual chance flood. Top of dam elevations have been surveyed at a rough average of 720.65 (NAVD88), and the 1% annual chance event elevation with risk and uncertainty is calculated to approximately 727. Estimating from a surveyed bank point upstream of the structure, it is assumed that the river bottom elevation downstream is 710. As opposed to placing the physical barrier in the vertical position, it is desired to slope the barrier up to the required height to aid in maintenance of the barrier. The barrier itself should be similar in fashion to the trash rack of a pump station inlet channel, steel bars spaced a desired length in the longitudinal direction with a transverse bar every two to three feet. The "barrier rack" will then be supported by either a frame or concrete pier spaced as necessary to allow the barrier rack to clear span between supports. For more details on the barrier and supports, see corresponding Sections 6.2.2.2 and 6.2.2.3, respectively.

6.2.2.2. Barrier Configuration

The physical barrier is intended to perform in a manner similar to that of a trash rack in a pump station inlet channel. Given that fact, it would be beneficial to mimic the configuration of a typical trash rack. See Figure 6.1 for a typical trash rack configuration out of a Local Flood Protection Project's "As-Built" drawings.

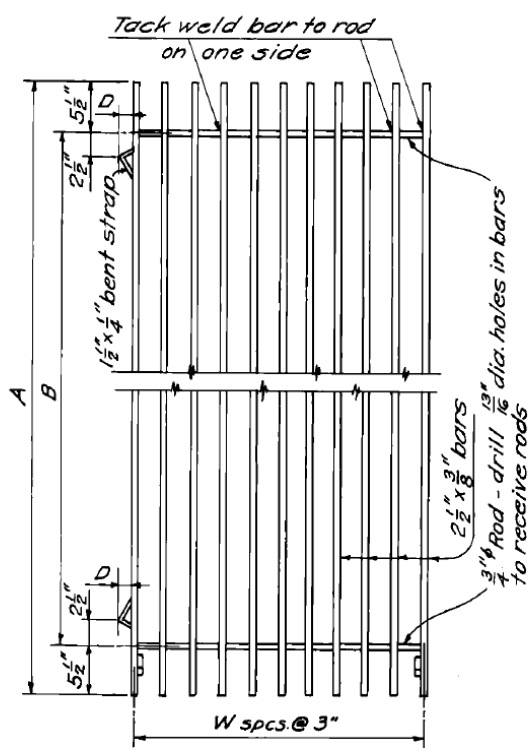

Figure 6.1: Typical Trash Rack Configuration

During design, a desired spacing of the longitudinal bars should be selected. The customer should decide on the spacing to ensure a balance between the species size (and possibly maturity) which are required to be blocked and the extent and degree of stream borne trash that could inhibit flow requiring frequent rack maintenance cleaning. In addition to those considerations, a few environmental factors should also be taken into account prior to selecting the spacing.

The barrier rack should be capable of withstanding an impact force from a large diameter log. Judging from photos taken on a site visit in March of 2010, it appears that trees of diameters in magnitudes of one to two feet regularly pass over the structure, see Photo 6.3. The spacing of the bars needs to adequately resist such an impact force.

Another design factor that could affect the spacing of the bars are concerns with ice accumulation. According Figure 10-2 of ASCE 7-08, Huntington has a one inch 50-year mean recurrence interval uniform ice thickness due to freezing rain. Chapter 10 further explains that this value should be applied radially to any member unprotected from the elements. A one inch thick coating of ice would reduce the effective spacing on the barrier rack by two inches during periods of freezing temperatures. To avoid any

Photo 6.3: Typical Large Logs that Impact Dam

disturbances to the hydraulics of the stream, the minimum spacing of the bars should be 2 ½ inches, larger if possible. If a smaller spacing is required to block certain aquatic nuisance species, a heating element would need to be implemented to ensure substantial amounts of ice does not build up on the barrier rack.

Given that the structure is constructed over an active river and the barrier is to be designed to provide protection during the 1% annual chance flood event, careful considerations need to be taken to extend the life of the structure due to exposure to moisture and submerged conditions. Selection of material/coating of the barrier rack should be based on either REMR-EM-6 or REMR-EM-11 from the Construction Engineering Research Laboratories within the Army Corps of Engineers. It is recommended that galvanized steel be avoided in this application. Galvanization, although primarily intended for members in high moisture conditions, does not necessarily perform well in submerged cases. Stainless steel or a waterproof coating should be applied per the referenced REMRs.

6.2.2.3. Pier Supports

The support for the physical barrier should be concrete piers, but a steel frame could also be used. Concrete would be more beneficial since it will require less maintenance in this hydraulic condition, but a steel frame could possibly be cheaper to construct depending on the design. For cost estimating purposes, a concrete pier system will be assumed.

The number of concrete piers will be based on the maximum allowable clear span designed for the barrier rack. Clear spans of existing trash racks at Local Flood Protection Project (LFPP) pump stations are typically 15-25 feet apart; piers should be similarly spaced. The length of the physical barrier was estimated by measuring the extents of the 1% annual chance event elevations (see Figure 7.5-1), which equates to approximately 225 feet. This would require 9-15 concrete piers depending on the spacing selected. Piers should be placed as close to the Huntington Dam as possible without impacting the structure. Piers should be founded on a spread footing bearing on rock of a designed thickness and width.

Figure 6.2: Limits of 1% Annual Chance Event, (Indiana DNR, 2004)

A design case that should be considered when designing the foundations of the concrete piers is a low flow winter condition. The water levels could get low enough on the downstream side to expose the river bed during the winter months. In this event, frost upheaval could pose an issue to the foundations of the pier structures. The force of frost upheaval should be considered during the foundation design.

Dimensions of each pier support will be determined by the slope of the barrier desired. In Table 6.4, the variations of length based on height and slope of the piers and barrier rack are listed. The minimum height that the top of protection needs to be designed to is the 1% annual chance event elevation including additional height for risk and uncertainty. The height of the piers is calculated from that value, which is 727, then subtracting the estimated height of the river bottom, 710. Therefore, the top of protection required is 17 feet, which is approximately 6.5 feet higher than the top of dam. The barrier rack would be the same height as the Huntington Dam at the most upstream end (720.65), and then slope up to the required top of protection at the downstream end. The 27 feet tall option is included in the table in case the top of protection needs to be extended to account for the aquatic nuisance species' ability to jump above water. The Environmental Protection Agency (EPA) reports that Silver Carp, a species of Asian carp, can jump up to 10 feet into the air. It may not be feasible to design the barrier dam to that height, but the dimensions are included in case that is a desired option.

TABLE 6.4
LENGTHS OF PIERS AND BARRIER RACK

	Length of Concrete Pier		Length of Barrier Rack	
	100 Year Elevation	100 Year Including Jumping Ability	100 Year Elevation	100 Year Including Jumping Ability
	Top of Protection: 17 feet	Top of Protection: 27 feet	Top of Protection: 17 feet	Top of Protection: 27 feet
1 on 12	76.20	196.20	76.46	196.88
2 on 12	38.10	98.10	38.63	99.45
3 on 12	25.40	65.40	26.18	67.41
4 on 12	19.05	49.05	20.08	51.70
5 on 12	15.24	39.24	16.51	42.51
6 on 12	12.70	32.70	14.20	36.56
7 on 12	10.89	28.03	12.60	32.45
8 on 12	9.53	24.53	11.45	29.48
9 on 12	8.47	21.80	10.58	27.25

A low sloped barrier may perform better in attempting to contain the aquatic nuisance species. The species may not easily see the edge of the structure, and therefore may not try to jump it. Although, the smaller the slope, the more maintenance would be required. Shallow slopes will more than likely accumulate large amounts of debris, and would have to be cleared on a regular basis. During design, the necessity of an anchorage or clamping system of the barrier rack to the concrete piers should also be investigated. Reference is made to Alternative F Site Plan CS108 in Appendix G.

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SECTION 7
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7.1. REFERENCES

1) EM 1110-2-3102 "*General Principle of Pumping Station Design and Layout*" U.S. Army Corps of Engineers, dated 28 February 1995

2) EM 1110-2-3105, "Mechanical and Electrical Design of Pumping Stations" U.S. Army Corps of Engineers, dated 30 November 1999

7.2. STRUCTURAL ALTERNATIVES

7.2.1. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

A conceptual one-line power distribution diagram was developed for the proposed pump station for Alternative C as described in Appendix F Section 1.5.3. One line diagram utilized a system voltage of 4160 volts. A lighting transformer integral to the switch gear was shown at 240 volts. Design included new medium voltage switchgear with motor starters with reduced voltage soft start capability, and a lighting panel board for general building loads. A budgetary cost estimate was requested from an application engineer at the electrical manufacturer Eaton. Initial estimates were roughly $530,000 for all power distribution equipment. Due to the size of the pump and the simplicity of the necessary control equipment, (all pumps will be manually activated and shut off) a conservative estimate for the control system is $5,000.

The required station power was calculated to be roughly 4 megavolt-ampere (MVA). This number is extremely high and a cost for utility procurement was not determined due to the infeasibility of the required power amount. Power one line diagram was designed to 4160 volts.

7.2.2. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

A conceptual plan was developed for the permeable berm with telemetry-activated sluice gates. This plan includes actuated gates, which are controlled from three telemetry systems at different locations in close proximity to the site. Two of the telemetry systems shall be located at other locations. The final shall be local to the site.

7.2.3. Construct Vertical Drop Structures with Telemetered Sluice Gate, Alternative G, (Homestead Road)

A conceptual plan was developed for the vertical drop structures with a telemetry-activated sluice gate. This plan includes an actuated gate, which is controlled from three telemetry systems at different locations in close proximity to the site. Two of the telemetry systems shall be located at other locations. The final shall be local to the site.

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SECTION 8
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8.1. REFERENCES

1) EM 1110-2-3102 "*General Principle of Pumping Station Design and Layout*" U.S. Army Corps of Engineers, dated 28 February 1995

2) EM 1110-2-3105, "Mechanical and Electrical Design of Pumping Stations" U.S. Army Corps of Engineers, dated 30 November 1999

8.2. STRUCTURAL ALTERNATIVES

8.2.1. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

Two different pump station configurations have been preliminarily investigated. The pump station is to be located at the corner of Graham-McCulloch Ditch and Homestead Road in Fort Wayne, Indiana. The "prevent" option utilizes grinder pumps and the other option utilizes submersible "can style" pumps. The station will have a minimum pumping capacity of 50 CFS and a maximum pumping capacity 1200 CFS as determined by the Hydrology and Hydraulics Section.

The first alternative is a grinder pump station configuration. Investigation into grinder pumps was requested due to their ability to grind small foreign objects that may enter the pump station. This also reduces the risk of movement of aquatic nuisance species. Due to a grinder pump having inherently smaller pumping capacities, this configuration would require in excess of 75 grinder pumps depending on the pump manufacturer and exact impeller selection. Additionally, this configuration would require a massive and complex pump station structure to house the pumps and an extensive operation and maintenance schedule. Based on this, no further investigation was conducted into a grinder pump configuration.

The second alternative is a "can style" submersible pump configuration. A "can style" pump was selected versus a typical vertical column pump because the pumps can be removed from pump station floor thus reducing confined space entry. The downfall to a submersible pump or vertical column pump configuration is that there is no guarantee of preventing movement of aquatic nuisance species. Since the pump intake and impellers are larger there is an increased risk that some aquatic nuisance species may pass through the pump. The pump station features are depicted below. These features are rough and for cost estimating purposes only. If the pump station option is selected then a detailed design will be implemented.

(1) Size and Number of Pumps: This pump station configuration utilizes eleven “can style” pumps for the primary flow and one submersible pump for dewater the sump if maintenance is required. Eight of the primary pumps will have an approximate capacity of 130 CFS while the remaining three pumps will be approximately 50 CFS for typical daily inflow. Each “can style” pump will have a steel pump tube to house the pump and a formed suction intake (FSI). The dewater pump will only be installed by a rail system when dewatering is necessary. This reduces the chance of the pump becoming silted in when it is not being operated.

(2) Pump Discharge Pipes: Each 130 CFS pump will utilize a 48-inch discharge pipe and each 50 CFS pump will utilize a 30-inch discharge pipe. The discharge pipes will run from the sump well through the earthen berm and then terminate back into Graham-McCulloch Ditch. Each discharge pipe will utilize gravity type flap gates to prevent backflow into the pump sump. Another option would be having the discharge pipes terminate at a discharge well that would then direct all the water back into McCulloch Ditch by a large gravity pipe. Each discharge pipe will have a gravity type flap gate in the discharge well. No matter which option is selected each discharge pipe will have a gate valve as a secondary means of closure. Additionally, each discharge pipe may require an air vent depending on whether high points exist.

(3) Sluice Gate/Maintenance Bulkhead: A sluice gate or maintenance bulkhead structure will be located at the inlet of the pump station as a means to stop inflow for maintenance and repairs.

(4) Miscellaneous Features: A manually operated trash rack will be provided and designed to meet the requirements of EM 1110-2-3102. Ladders will be provided to access sumps for maintenance.

(5) Please see Structural Section for pump station structure.

(6) Final Design and pump selection will be based on cost and maintenance of the new equipment. The design will meet all of the requirements of the Corps of Engineers Engineering Manuals (EM) and Industry Standards.

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SECTION 9
OPERATION AND MAINTENANCE

9.1. OPERATION AND MAINTENANCE SCHEDULE

For the purpose of this study, preliminary operation and maintenance (O&M) considerations were developed for each alternative. This section provides a list of anticipated cost items for future operations, maintenance, replacement, and repairs to be incurred over a 50-year period of analysis for each of the nine structural alternatives. The mean annual O&M cost estimate was generated for each alternative, based on the 50 year period of operation.

9.2. STRUCTURAL ALTERNATIVES

9.2.1. Construct an I-Wall, Alternative A, (Eagle Marsh, Basin Divide)

- Repair/replace joint material of I-wall monoliths – every 15 years
- Annual inspection

The estimated annual O&M cost for Alternative A is $11,000.

9.2.2. Construct a Fence and Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative B, (Eagle Marsh, Basin Divide)

- Inspect fence after flooding event for possible damage and debris
- Remove debris from secondary debris fence – after each high water event
- Mowing – twice annually
- Replace metal fence material, every 10-20 years
- Replace fence posts, every 20-30 years
- Annual inspection

The estimated annual O&M cost for Alternative B is $18,000.

9.2.3. Construct an Earthen Berm and Pump Station, Alternative C, (Homestead Road)

- Mowing – twice annually
- Large electric utility bill - monthly
- Control system maintenance-as needed
- Crane load testing and maintenance-yearly
- Recondition pump motor– variable, assumed 1 motor every year starting in 5 years for the 50 cfs pumps, 1 motor every year starting in 20 years for the 130 cfs pumps
- Recondition pump components- variable
- Weekly inspection/maintenance of low-flow condition pumps
- Megger testing- annually
- Clean trash racks – monthly and/or after major storm events
- Pump preventive maintenance and lubrication – yearly
- Annual inspection

The estimated annual O&M cost for Alternative C is $600,000.

9.2.4. Construct a Permeable Berm with Telemetered Sluice Gates, Alternative D, (Amber Road)

- Repair erosion/scour areas following high water event
- Inspect berm after high water event to check for possible sediment and debris build up
- Inspect silt fence after every high water event
- Clean sediment from piping – every 10 years
- Debris removal from vegetative filer – annually
- Excavate sediment accumulation of berm – every ten years
- Electrical and communication utility charges for the telemetered sluice gate(s)
- Maintenance of telemetry components
- Annual inspection

The estimated annual O&M cost for Alternative D is $22,000.

9.2.5. Construct a Fence/Earthen Berm Combination, Alternative E, (Eagle Marsh, Basin Divide)

- Debris removal from fence after high water event
- Inspect fence and berm after high water event for possible damage
- Replace metal fence material, every 10-20 years
- Replace fence posts, every 20-30 years
- Annual Inspection

- Mowing – twice annually

The estimated annual O&M cost for Alternative E is $44,000.

9.2.6. Construct Bar Screen Barrier at Existing Weir, Alternative F, (Huntington Dam)

- Debris removal and disposal from debris boom – twice monthly from January – June, monthly from July – December.
- Damaged bar screens from debris will need to be removed and repaired/replaced as needed.
- Debris removal from bar screens – seasonally
- Annual inspection

The estimated annual O&M cost for Alternative F is $96,000.

9.2.7. Construct Vertical Drop Structures with Telemetered Sluice Gates, Alternative G, (Homestead Road)

- Clean debris from trash fence atop the intake structures after each high water event
- Clean debris from sluice gates - seasonally
- Electrical and communication utility charges for the telemetered sluice gate(s)
- Maintenance of telemetry components
- Annual inspection

The estimated annual O&M cost for Alternative G is $26,000.

9.2.8. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Alternative H, (Eagle Marsh, Basin Divide)

- Mowing – twice annually
- Annual inspection

The estimated annual O&M cost for Alternative H is $14,000.

9.2.9. Reconstruct Left Descending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area, Alternative I, (Eagle Marsh, Basin Divide)

- Mowing – twice annually
- Annual inspection

The estimated annual O&M cost for Alternative I is $17,000.

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SECTION 10
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10.1. REFERENCES

1.) ER 1110-2-1302 *"Engineering and Design- Civil Works Cost Engineering"* U.S. Army Corps of Engineers, dated 15 September 2008

2.) *"Agency Technical Review Guidance for Cost Engineering Products"* U.S. Army Corps of Engineers Directory of Expertise for Civil Works Cost Engineering, dated May 2011

10.2. BASIS OF ESTIMATE

The alternatives estimate is based on the design indicated in the report dated February 2012 and depicted on the site plans in Appendix G. This document is considered to be at screening level and given the parameters this estimate shall be considered a Class 4.

WBS: The estimate is organized per the Civil Works Work Breakdown Structure (WBS) by feature.

Acquisition Strategy: Per discussion with the PDT this project is projected to be awarded by "Full and Open Competitive Bidding" process.

Contractor Hierarchy: The estimator assumes different contractor hierarchies due to the range in structural alternatives. With alternatives that include earthen berms as the major work feature, then the Prime contractor is assumed to self-perform the earthwork and subcontract items like fencing and seeding. On alternatives where a concrete structure is the major feature of work, it was assumed that the Prime contractor would perform concrete work and subcontract most other activities.

Construction Methods: The estimator's approach for the mass excavation work items included dozers, hydraulic excavators, and off-road articulating trucks due to the digging depths required and relatively small foot prints of the berms. With the berms only having a 10' crown width and a required six foot deep inspection trench, it was assumed that an excavator and dozer combination was best suited for this task. The work would commence with an excavator demolishing any existing berm and loading off-road trucks that would dump the material in stockpiles in the work limits. The inspection trench would then be performed using again an excavator dumping the material along side the sloped backed trench. Since this area is a marsh, dewatering will likely be required when digging just a few feet under the existing grade. A dozer will then back fill the trench with compaction from a riding sheepsfoot type roller compactor. The berm will then be constructed by either on-site fill from demolition or off-site fill. On-site fill operations will include an excavator loading the off-road trucks at various stock fill locations. A dozer will then shape the berm placement from the dumped fill. The shaped fill will then be compacted by the means of a riding sheepsfoot type compactor.

Other work items were assumed by using common construction practices. Specifically, the sheet pile in Alternative "A" would be performed by a crane handled hammer and the concrete component would be placed by the means of pumping. The fence in Alternative "B" would be performed first by trenching the footprint, then installing the fence by traditional methods, then backfilling the trench with sand, and setting precast concrete barriers against posts. A subcontractor quote was obtained for this item. Excavation methods described above would be utilized in the pump station Alternative "C". The construction would be performed like any other vertical structure. Alternative "D" would utilize a loader and excavator to embank the aggregate around the perforated pipes that make up the permeable dam. Alternative "E" would be approached by both excavation and fencing methods previously mentioned. Alternative "F" involves constructing concrete piers and bar screens directly adjacent to the downstream side of an existing low-height dam. The estimator assumed a two phase cofferdam consisting of sheet pile panels to create a dry working area. The concrete piers would then be cast-in-place with ready mix being unloaded into a placing bucket maneuvered by a crane to the piers. Alternative "G-I" were approached by excavation methods described above.

10.3. PRICING

Quotes were received for significant or specialized materials and noted throughout the baseline estimate. Aggregate pricing used throughout all alternatives was obtained from Hanson Aggregate (Mr. Rick Hullinger (260) 438-2403). Concrete pricing was obtained from Erie Haven based on the assumption of a 4000 psi mix throughout several alternatives (Mr. Rick Vorndran (260) 760-1269). Sheet pile pricing was obtained from L.B. Foster (Mr. Rich Fifield (800) 824-6166) for the I-wall in Alternative "A". A quote for material and installation was received from R&C Fence (Mr. Don Roop Jr. (260) 478-7667) for fencing found in Alternatives B and E. Quotes were also received for the pumping station pumps and electrical switchgear through the help of the respective engineers.

Crew and production rates used in the alternatives estimate were from the cost book or derived by the estimator. Production rates different from the cost book for mass excavation were derived by the estimator using the Caterpillar Handbook, 39th Edition.

Cost sources: 2010 Cost Book - English. Updated material costs per quotes received and/or production rates as appropriate per estimator's judgment.

Wage Rates: As indicated by the Davis-Bacon Act, Allen County, Indiana decision number IN00006 02/17/2012.

Equipment: The 2009 Equipment Ownership and Operating Expense database Region 2. Updated fuel costs to reflect current market conditions.

Material: The 2010 Cost Book – English (Effective Pricing Date Jan 2010). Updated material costs per quotes received for the major and various items as noted in the estimate costs items.

All costs identified Feature 01 - Lands and Damages are total costs to owner, provided by Corps of Engineers, Louisville District, Real Estate Division. Costs include a 25% contingency. No contractor assigned.

Project Planning, Engineering, and Design (PED) are assessed at 12% of estimated construction cost consisting of Feature 02, Feature 04, Feature 11, and Feature 13. Basis comes from recent Olmsted out-year contracts effort and Paducah Feasibility Report.

Construction Management: 7.5% was used for Construction Division oversight during construction and is based on the recent projects.

10.4. MARKUPS

Price Level Adjustment: For the cost items based on the 2010 Cost Book database, an "inflation" adjustment of 6.3% was applied based on Engineering News Record (ENR) values (Oct11/Jan 10). This markup was not applied to cost items that were supported by current vendor pricing. An escalation rate of 12% was also applied to the MCACES estimate to capture escalation from the estimate pricing level of FY 2012 Q1 to the midpoint of construction of FY 2016 Q2. The four year average of the ENR Building Cost Index is nearly 3% whereas Civil Works Construction Cost Index System (CWCCIS) was running over 4% per year.

Contingencies: The contingencies were developed based on other historical project contingencies. This includes the 19.7% recently experienced with the Olmsted PACR and 19% for the Feasibility Report for the Paducah, KY Flood Protection Rehabilitation Project (FY10). This report is still a precursor to the Feasibility and should be expected to have higher risks. Therefore, 25% was assessed to Alternatives A, B, E, H, and I with the remaining alternatives having 30%. The exception being that Alternative C includes building a pumping station on the Graham-McCulloch ditch. Due to significant power requirements, rough layout, and lacking reconnaissance it was assessed at a 50% contingency. The "*Agency Technical Review Guidance for Cost Engineering Products*"

Directory of Expertise for Civil Works Cost Engineering, dated May 2011 was referenced for the level of contingencies required at this effort.

10.5. SCHEDULE

The estimator's projected project schedule is as follows:
Start of PED: Oct 2013;
Midpoint of PED: Jun 2014;
RTA: Feb 2015;
Start of Construction: Apr 2015;
Midpoint of Construction: Jan 2016;
Completion of Construction: Oct 2016

10.6. ALTERNATIVES ESTIMATE

The alternatives estimate summarized at the account level is provided on the following pages:

U.S. ARMY CORPS OF ENGINEERS

Louisville District

Alternatives Estimate
Aquatic Nuisance Species Controls Report
Wasbash-Maumee Basin Connection
Fort Wayne, Indiana
February-12

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
A		Construct Concrete I-Wall w/ Sheet Pile					
	1	Real Estate	$119,000	$12,495	25%	Included	$131,495
	11	Levees & Floodwalls	$8,313,839	$997,661	25%	$2,327,875	$11,639,375
	30	Planning, Engineering, and Design	$997,661	$74,825	25%	$268,121	$1,340,607
	31	Construction Management	$623,538	$74,825	25%	$174,591	$872,953
						Alternative A- Estimated Total Cost	**$14,000,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
B		Construct a Fence and Reconstruct the Left Descending Graham-McCulloch Ditch Berm					
	1	Real Estate	$189,000	$19,845	25%	Included	$208,845
	11	Levees & Floodwalls	$1,745,360	$209,443	25%	$488,701	$2,443,504
	30	Planning, Engineering, and Design	$209,443	$15,708	25%	$56,288	$281,439
	31	Construction Management	$130,902	$15,708	25%	$36,653	$183,263
						Alternative B- Estimated Total Cost	**$3,200,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
C		Construct an Earthen berm and Pump Station					
	1	Real Estate	$34,000	$3,570	25%	Included	$37,570
	13	Pumping Plant	$12,469,143	$1,496,297	50%	$6,982,720	$20,948,160
	30	Planning, Engineering, and Design	$1,496,297	$112,222	50%	$804,260	$2,412,779
	31	Construction Management	$935,186	$112,222	50%	$523,704	$1,571,112
					Alternative C- Estimated Total Cost		**$25,000,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
D		Construct a Permeable Berm with Telemetered Sluice Gates					
	1	Real Estate	$157,000	$16,485	25%	Included	$173,485
	4	Dams	$4,349,972	$521,997	30%	$1,461,591	$6,333,559
	30	Planning, Engineering, and Design	$521,997	$39,150	30%	$168,344	$729,490
	31	Construction Management	$326,248	$39,150	30%	$109,619	$475,017
					Alternative D- Estimated Total Cost		**$7,800,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
E		Construct Fence/Earthen Berm Combination					
	1	Real Estate	$202,000	$21,210	25%	Included	$223,210
	11	Levees & Floodwall	$2,370,849	$284,502	25%	$663,838	$3,319,189
	30	Planning, Engineering, and Design	$284,502	$21,338	25%	$76,460	$382,299
	31	Construction Management	$177,814	$21,338	25%	$49,788	$248,939
					Alternative E- Estimated Total Cost		**$4,200,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
F		Construct Bar Screen Barrier at Existing Weir					
	1	Real Estate	$85,000	$8,925	25%	Included	$93,925
	6	Fish and Wildlife Facilities	$1,447,586	$173,710	30%	$486,389	$2,107,685
	30	Planning, Engineering, and Design	$173,710	$13,028	30%	$56,022	$242,760
	31	Construction Management	$108,569	$13,028	30%	$36,479	$158,076
					Alternative F- Estimated Total Cost		**$2,700,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
G		Construct Vertical Drop Structures with Telemetered Sluice Gate					
	1	Real Estate	$25,000	$2,625	25%	Included	$27,625
	4	Dams	$2,726,711	$327,205	30%	$916,175	$3,970,091
	30	Planning, Engineering, and Design	$327,205	$24,540	30%	$105,524	$457,269
	31	Construction Management	$204,503	$24,540	30%	$68,713	$297,757
					Alternative G- Estimated Total Cost		**$4,800,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Totals
H		Reconstruct Left Decending Graham-McCulloch Ditch Berm					
	1	Real Estate	$230,000	$24,150	25%	Included	$254,150
	11	Levees & Floodwalls	$3,234,560	$388,147	25%	$905,677	$4,528,384
	30	Planning, Engineering, and Design	$388,147	$29,111	25%	$104,315	$521,573
	31	Construction Management	$242,592	$29,111	25%	$67,926	$339,629
					Alternative H- Estimated Total Cost		**$5,700,000**

Alternative	WBS Account	Description	Cost	Escalation	Contingecy %	Contingency Amount	Total Alternative Estimate
I		Reconstruct Left Decending Graham-McCulloch Ditch Berm, Demolish Right Descending Berm, and Construct Multi-Cell Wetland Area					
	1	Real Estate	$310,000	$32,550	25%	Included	$342,550
	11	Levees & Floodwalls	$4,084,199	$490,104	25%	$1,143,576	$5,717,879
	30	Planning, Engineering, and Design	$490,104	$36,758	25%	$131,715	$658,577
	31	Construction Management	$306,315	$36,758	25%	$85,768	$428,841
				Alterantive I- Estimated Total Cost			**$7,200,000**

Backup File: X:\ED-M-C\Civil\FY12\114597 - Ft. Wayne Interbasin Study - Asian Carp\FINAL REVISED ATR DOCS - FEB 2012\Estimate

Prepared By:________________________ Checked By:_____________________________ Approved By:____________________________________

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