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North Green Springs Valley Drainage Study
Hydrologic & Hydraulic Analysis

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ABSTRACT

This project was generously provided by Lochmueller Group, a civil engineering firm located in Evansville, Indiana. North Green Springs Valley is a small residential subdivision located in Newburgh, Indiana. Documented as far back as 1974, North Green Springs Valley subdivision and its residents have been experiencing extreme flooding due to inadequate conveyance in the primary drainage channel. A site visit indicated significant debris in the stream and evidence of erosion and water damage. The flooding poses a threat to the residents and their homes. The focus of this project was to complete a professional hydrologic and hydraulic study of the stream. The hydrologic study quantified the magnitude of a 50-year flood event. A hydraulic model of the study reach was used to simulate existing field conditions, including two bridge structures using the United States Army Corps of Engineer's Hydrologic Engineering Center River Analysis System (HEC-RAS). New renditions of the model were created to simulate several design alternatives by altering model input parameters. Based off the new models, several design alternatives were then proposed that restrict the water flow to the banks of the stream and thus reduce flooding risk.

TABLE OF CONTENTS

Acknowledgements ..... **i**

Abstract ..... **ii**

List of Figures ..... **v**

List of Tables ..... **vii**

Introduction to Project **1**

1 Background **1**

1.1 Motivation for study 1

1.2 Resident Questionnaires 2

1.3 Site Visit 3

2 Scope of Work **4**

3 Deliverables **5**

4 ABET Outcome #2 Design Factor Considerations **5**

4.1 Public Health, Safety, & Welfare 5

4.2 Environmental 6

4.3 Economic 6

4.4 Ethical & Professional 6

4.5 Insignificant Factors 7

Hydrologic Study **8**

5 Delineating the Watershed **8**

6 Time of Concentration **9**

7 Runoff Curve Number **11**

8 Peak Discharges **16**

9 Stream Channel Geometry **17**

10 Determining Manning's Roughness Coefficient **18**

11 Hydrologic Study Conclusions **19**

Hydraulic Study **20**

12 Creation of an Existing Conditions Model **20**

12.1 Stream Channel and Floodplain Geometry 20

12.2 Bridge Structures 21

12.3 Blocked Obstructions 22

13 Creation of a Steady Flow File & Analysis 23
14 Evaluation of Existing Conditions Model 24

Primary Causes of Flooding 26

15 1st Cause: Undersized Bridge Structures 26
16 2nd Cause: Geometry of Stream 27

Creation of Design Alternatives 28

17 Alternative #1 – Clearing & Grubbing 28
18 Alternative #2 – Resize Structures & Geometry of Channel 29
19 Alternative #3 – Resize Structures, Geometry of Channel, & Install Flexamat 33

Design Alternative Selection 36

20 Alternative #2 36

Concluding Thoughts 37

Project Teamwork 39

References 41

Appendix 43

Appendix A 44

Appendix B 51

Appendix C 55

Appendix D 56

LIST OF FIGURES

Figure 1: Vicinity Map 1
Figure 2: Flooding Photo Provided by Resident 3
Figure 3: Delineated Watershed 9
Figure 4: Raindrop Flow Path 10
Figure 5: Hydrologic Soil Groups Map 13
Figure 6: 2019 NLCD Land Cover Map 14
Figure 7: Curve Number Map 15
Figure 8: Intensity Duration Frequency (IDF) Curves 16
Figure 9: Horizontal Variation in Roughness 18
Figure 10: Cross Section Geometry 21
Figure 11: Location of Bridge Structures Within Stream Reach 21
Figure 12: Bridge 1 Geometry 22
Figure 13: Bridge 2 Geometry 22
Figure 14: Houses/Building Within Cross Section 23
Figure 15: Existing Conditions Model Output (Cross Section View) 24
Figure 16: Existing Conditions Model Output (Profile Plot) 25
Figure 17: Bridge 1 Existing Condition 26
Figure 18: Alternative #1 Profile Plot 28
Figure 19: Alternative #1 Cross Sectional View 29
Figure 20: Geometry Changes Throughout Stream Reach 30
Figure 21: Alternative #2 Profile Plot 31
Figure 22: Alternative #2 Cross Sectional View 32
Figure 23: Flexamat Erosion Protection 33

Figure 24: Alternative #3 Profile Plot 35
Figure 25: Possible Effects of Detention Pond 38
Figure 26: Time of Concentration Calculations (1) 44
Figure 27: Time of Concentration Calculations (2) 45
Figure 28: NOAA Rainfall Data 46
Figure 29: House/Building Stations & Elevations Along Cross Sections 47
Figure 30: Manning's Roughness Coefficient Calculations (Cross Section 10) 48
Figure 31: Flexamat n-Value Calculations 50

LIST OF TABLES

Table 1: Hydrologic Soil Groups Within Watershed 12

Table 2: NLCD Curve Numbers 14

Table 3: Peak Discharges 17

Table 4: Hydrologic Study Conclusions 19

Table 5: Completion of Project Tasks 39

Table 6: Manning's Roughness Coefficient Notes 49

Table 7: Existing Conditions Model Output Table 51

Table 8: Alternative #1 Model Output Table 52

Table 9: Alternative #2 Model Output Table 53

Table 10: Alternative #3 Model Output Table 54

Table 11: Preliminary Project Schedule 55

Table 12: ABET Outcome #2 - Design Factors Considered 56

INTRODUCTION TO PROJECT

1 BACKGROUND

1.1 MOTIVATION FOR STUDY

Documented as far back as 1974, North Green Springs Valley subdivision and its residents have experienced extreme flooding due to the drainage stream running through the neighborhood. The stream has never been cleared and is filled with overgrown/overturned trees, bushes, and other debris. Historically, flooding has been a reoccurring issue that the residents have continuously battled. This issue causes consistent headaches and worry, while also posing a threat to the residents' homes, their individual safety, and the value of their property. The main motivation of this drainage study was the continuous escalation of severe erosion, water damage, and various other associated issues. Figure 1 shows a vicinity map of the subdivision as well as the drainage stream that runs through it. The subdivision is located just South of State Road 66 and just East of County Road 850 W.

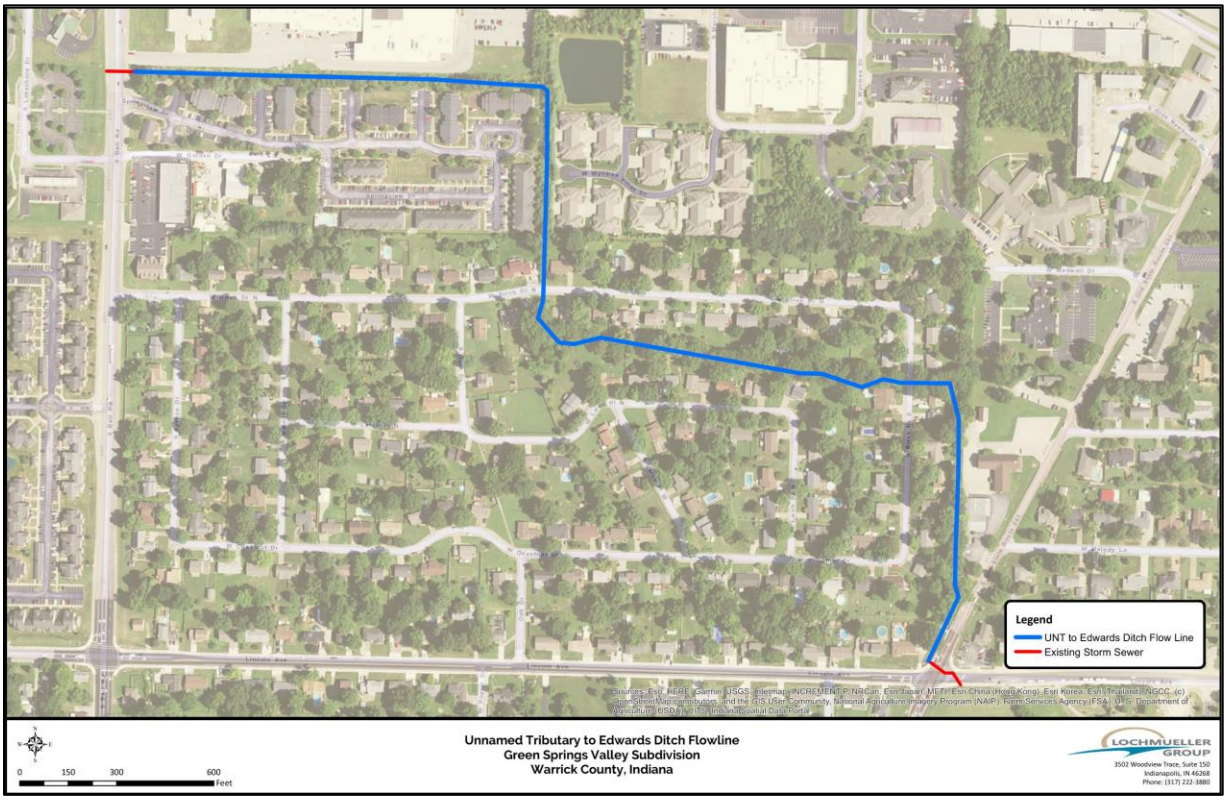

Figure 1: Vicinity Map

This project was generously provided by Lochmueller Group, a civil engineering firm located in Evansville, Indiana. As well as the project itself, Lochmueller also provided a package of various files to assist in the development of the study [1]. Within the package included a LiDAR surface of the subdivision location, structure plan excerpts, local stormwater management ordinances, existing survey data collected from the drainage stream, and resident questionnaires.

1.2 RESIDENT QUESTIONNAIRES

During Lochmueller's initial stage of their study, a questionnaire was sent out to all of the residents in the subdivision. This questionnaire allowed for the residents to voice their opinions and provide information such as locations, field measurements, and photos. Thirteen residents responded to the questionnaire and provided much useful information. Some of the responses from the residents includes verbiage as "stream is not adequate to handle flow" and "this issue will not stop unless someone does something". The following were also useful notes taken from the questionnaires. Additional flow was supposedly "illegally" routed into the stream during construction of Lincoln Avenue and State Road 66 that was routed elsewhere initially. Stream has never been cleared of overgrown/overturn trees, bushes, and other debris. Water will overtop the bridge at East Birch Street with high velocity. Velocity is strong enough to "take grease off bearings underneath pickup truck". One resident had to continuously place sandbags around their garage to prevent water from coming inside, and later sold the property due to the issue. Another resident was injured while trying to remove obstructions from the stream during a flooding event. Some residents responded with no issues at all. This narrowed down which residents in the subdivision were experiencing the flooding and which ones were not. All thirteen of these responses added to a bountiful reason to have this study take place and to have the issue resolved. The residents' voices have been heard. Figure 2 is a photo that one of the residents attached in their response. The severity of the flooding can be seen in this photo.

Figure 2: Flooding Photo Provided by Resident

1.3 SITE VISIT

On February 9, 2023, a site visit occurred. The focus of the site visit was to confirm the stream channel geometry, take pictures and notes for later reference in modeling the stream hydraulics. Because of the resident questionnaire responses, specific locations were marked before the visit to inspect. These locations were where residents were experiencing the most flooding. Within the package given by Lochmueller, thirteen survey cross-sections were provided. The exact location of all thirteen cross-sections were then pinned. During the field visit, all cross-section locations were inspected, and various notes and photos were taken describing the behavior of the stream at those locations. Besides the cross-section locations, other locations such as the three 90-degree bends and two bridges were also thoroughly inspected. Many other stream characteristics such as floodplain features, expansion and contractions, and stream bank features were noted. It was noted that at each of the three 90-degree bends, rip rap lined the outer bank of the stream. Rip rap was also placed in various locations along the stream. This must have been done to areas where the stream was experiencing a high amount of erosion in that location. It was also noted that two

large culverts discharged into the entrance of the stream. One of these culverts continued from a storage detention pond at the First Christion Church (Southeast of the subdivision) and the other continued from the roadside ditch along State Route 261 (running Southwest to Northeast of the subdivision). Only one culvert discharged out of the stream on the Northwest corner of the subdivision. The stream was noted to have a smaller cross-section geometry in the reach between the two bridges. Coincidently, this is where most residents were experiencing flooding. Small amounts of vegetation resided on the banks of the stream in those areas as well. The stream had larger cross-section geometry and more vegetation and small trees in the reaches outside of the two bridge structures.

2 SCOPE OF WORK

The scope of this study is to complete a professional hydrologic and hydraulic analysis on the drainage stream running through the subdivision to prevent the occurrence of flooding. The hydrologic analysis includes calculating an accurate representation of the existing water flow through the stream and the geometric conditions of the stream itself based on given information. This includes delineating the watershed and calculating time of concentration, runoff curve number, peak discharges, and Manning's roughness coefficients for the stream. The geometry of the stream will also be analyzed by the creation of cross sections using survey and LiDAR data. The hydraulic analysis includes the creation of an accurate model which represents the existing conditions of the stream. This includes all stream characteristics such as obstructions (debris/buildings), structures (bridges), and the stream's geometry. An iterative process in calculating time of concentration and peak discharges will refine the model to accurately represent the current conditions of the stream. Once the hydraulics of the existing stream conditions is completed, it will be used to design alternatives will be proposed for the stream. Parameters in the model will then be altered to restrict the water flow to the banks of the stream, therefore eliminating flooding. Design alternatives will then be proposed and evaluated.

3 DELIVERABLES

The deliverables for this project include a complete and professional drainage study for the stream in North Green Springs Valley subdivision. This includes a hydrologic and hydraulic study of the stream. Also included within the deliverables is a working Hydrologic Engineering Center's River Analysis System (HEC-RAS) model that accurately represents the existing conditions of the stream. Included in the model are three design alternatives for the stream that reduce and restrict flooding.

4 ABET OUTCOME #2 DESIGN FACTOR CONSIDERATIONS

Accreditation Board for Engineering and Technology (ABET) Outcome #2 was taken into consideration during the completion of this project. The Outcome states: “An ability to apply engineering design to produce solutions that meet specified needs with consideration of public health, safety, and welfare, as well as global, cultural, social, environmental, economic, ethical and professional factors, and including references for standards.” Many of these design factors were incorporated into this project throughout its completion. Some factors were unable to be met but were taken into consideration. They are described in detail below.

4.1 PUBLIC HEALTH, SAFETY, & WELFARE

The flooding throughout the subdivision posed a threat to the residents’ homes, their individual safety, and the value of their property. It was noted within many of the resident questionaries that some residents were injured during flooding events as they were trying to remove blockage and debris from the stream. During a flooding event, the health and safety of the residents are at risk due to injury or drowning. It was also noted in the resident questionnaires that many residents are trying to sell their properties because they are no longer satisfied with their situation. They are having trouble selling their properties because new homeowners do not want to buy a property that floods multiple times a year. Because of this, the residents’ welfare is also at risk. Due to these issues, the public health, safety, and welfare factors were taken into consideration when providing design alternatives for the stream.

4.2 ENVIRONMENTAL

The flooding caused continuous escalation of severe erosion within the channel, water damage to the residents' homes, and many other associated issues. During the site visit, erosion issues were found within the channel at various locations along the stream reach. When creating design alternatives, not only was the water level in the stream a factor, but the velocity of the water flowing through the channel was also considered. Knowing the velocity of the water flowing through the channel at different locations along the stream reach allowed for an indication whether there would be erosion issues at those locations. Erosion within the stream channel can cause unwanted meandering within the stream as well as soil buildup further downstream. Design changes and implantations were incorporated into the design alternatives to deal with this issue. Due to these issues, the environmental factor was taken into consideration when providing design alternatives for the stream.

4.3 ECONOMIC

As for the deliverables, three design alternatives were provided. The goal of each alternative was to restrict the water level throughout the entire stream reach to the banks of the stream, therefore eliminating the flooding. Two of the design alternatives achieve this goal, but one does not. This alternative does not prevent flooding from happening, but it does retract the water level from the residents' homes, therefore providing more safety to the residents. The reason behind this alternative is that it provides a much cheaper option to lower the water levels through the stream reach, even though it does not prevent flooding from occurring. The other two design alternatives that meet the goal of eliminating the flooding come at a much higher economic cost. Even though they come at a higher cost, the cost was kept to a minimum by only allowing for the minimum design changes to occur. A detailed economic evaluation was not performed, nor was a numerical cost value calculated for the three design alternatives. Cost was considered in defining the three alternatives that will be defined and explored later in this report.

4.4 ETHICAL & PROFESSIONAL

Throughout the entirety of this project, it was ensured that an ethical and professional outline was followed. Throughout the process of completion, care was taken in each step, from each calculation to each click of a mouse. All referenced material was extracted from professional and educational resources and standards such as government entities or higher education.

Professional engineering software was implemented throughout the project to complete both hydrologic and hydraulic studies. The residents of the subdivision were the top priority for this project. Decisions on topics such as the safety, health, and economic resources of the residents were highly influential throughout this project. Many ethical decisions upon these issues had to be made with the residents and the health of the stream in mind. Due to these issues, both ethical and professional factors were taken into consideration when providing design alternatives for the stream.

4.5 INSIGNIFICANT FACTORS

The global factor was not considered significant for this project. The flooding through the stream is only present within the North Green Springs Valley subdivision. This is a localized issue that only occurs in the subdivision and only its residents experience it. This project does not have a global effect, nor is it associated with any global issues.

The cultural factor was not considered significant for this project. Culture refers to the way that individuals from within a group of people live their lives, shown through their behaviors, customs, beliefs, norms, traditions, and even language use. Culture does not influence this project, nor is it associated with any cultural issues.

The social factor was not considered significant for this project. Social refers to the way that humans interact and relate to each other. This can incorporate the means of communication, but also considers that categories like race, gender, ethnicity, and class are constructs of society and inform how those individuals communicate. The term “social” also carries with it the different ways that humans organize together to form groups, such as marriages, families, and governments. Social does not influence this project, nor is it associated with any social issues.

HYDROLOGIC STUDY

5 DELINEATING THE WATERSHED

The first step in delineating the watershed was implementing the United States Geological Survey's StreamStats website [2]. This website allowed for an approximate/preliminary watershed delineation for the stream at hand. Within StreamStats, the watershed was modified by snipping and adding land area. This was done by referencing contour lines and common knowledge. The contour lines allowed for an accurate watershed boundary to be created by analyzing which direction water would flow across the land. Conditions such 23.as roadway inlets also allowed for common knowledge to assume that water on the roadway and on the opposing side of the roadway from the watershed would gather into storm sewer pipes and flow underneath the roadway in a different direction away from the stream. A pin was dropped at the outlet of the stream and StreamStats produced a preliminary watershed. Figure 3 shows the final delineated watershed created within after its alterations. This is the total land area that contributes to the stream during a rainfall event. The watershed made up a total area of 221.8 acres. The watershed is represented by the yellow polygon. The stream is represented by the dark blue line flowing Southeast to Northwest through the watershed. The current location of the stream varies from the one depicted in Figure 3. The existing location of the stream is represented by the red line. During the field visit, it was noted that the storage detention pond at First Christian Church and the roadside ditch along State Route 261 contributed to the flow in the stream. These two contributing conditions were factored into determining the watershed boundary. This is why the watershed has a rounded extrusion on its Southeastern corner.

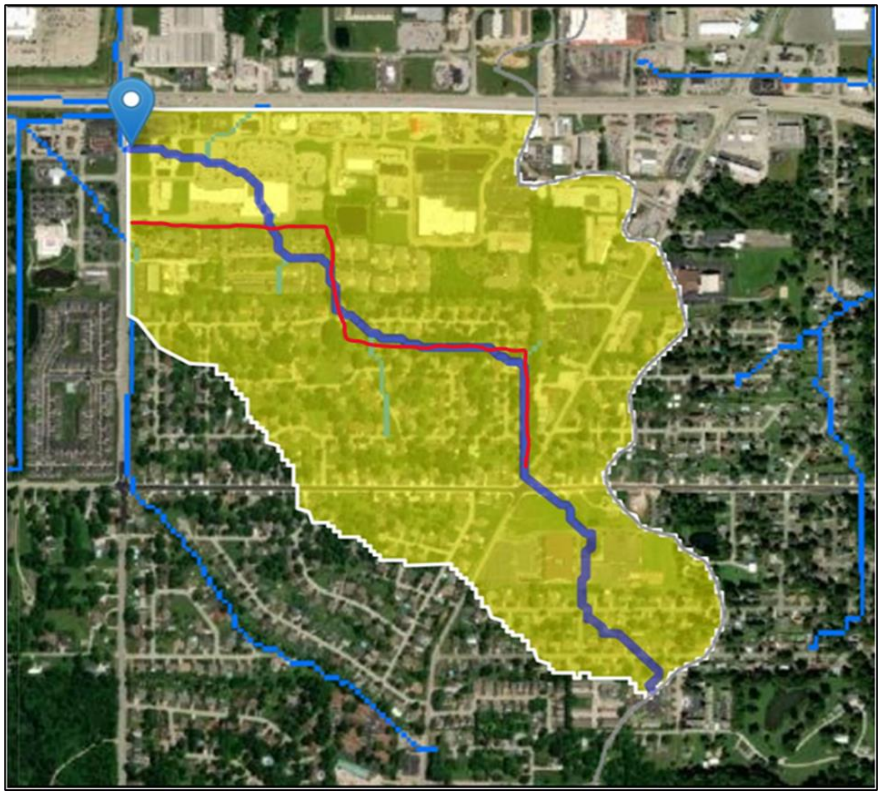

Figure 3: Delineated Watershed

6 TIME OF CONCENTRATION

Time of concentration is defined as the amount of time for water to travel from the hydraulicly most distant point of the watershed to the watershed outlet. This value is used to calculate the peak discharge, or the maximum amount of flow during a specific flow event. The two most common methods of calculating the time of concentration are the National Resources Conservation Method (NRCS) velocity method and the lag method. The NRCS velocity method was utilized for computing the time of concentration for this watershed as the lag method is designed for rural watersheds which does not apply to this project.

The NRCS velocity method divides the flow path into three or more segments in order to sum individual travel times of each type of flow. The methodology and equations for each flow type were found in Chapter 15 of *Part 630 Hydrology National Engineering Handbook* [3]. The path the water travels along the watershed was generated using the flow path network navigation tool within StreamStats and is shown by the red line inside the watershed in Figure 4 below.

Figure 4: Raindrop Flow Path

The first 100 feet of the raindrop path are sheet flow. Sheet flow occurs as the water begins to runoff and contains shallow depths with fluid orderly motion. The time for the water to travel this distance is 1.76 minutes and was calculated using the sheet flow equation in Figure 26 in Appendix A.

Following sheet flow was 992.36 feet of shallow concentrated flow. This flow type is expected to have depths around .1 to .5 feet and no defined channel. In this project the shallow concentrated flow was divided into five sections to account for the varying slopes across the watershed. The slope for each section was determined using the contour lines from the survey data provided by Lochmueller Group. The shallow concentrated flow equation in Figure 26 in Appendix A was used to determine the time for each section and added to find an overall shallow concentrated flow time of 17.4 minutes.

The detention basin located in the lower right portion of the watershed plays a minute role in the overall time of concentration but was still accounted for in this study due to its large size. The time for the water to travel across the detention basin and exit through the culvert is .003

minutes. The equation and calculations can be reviewed under the “Pond Flow” section in Figure 27 in Appendix A.

Finally, this watershed had 5,048 feet of open channel flow. This is the path the water takes from the beginning of the channel to the outlet point. This flow type is associated with deeper depths than sheet and shallow concentrated flow. There are two culverts located along the channel that were accounted for when calculating the channel flow time. The size, material, and velocity of the culverts were utilized to determine an individual time for each culvert then added to the channel flow time found using the channel flow equation in Figure 27 in Appendix A to get a total of 20.3 minutes.

The final overall time of concentration for this watershed is 39.5 minutes.

7 RUNOFF CURVE NUMBER

The runoff curve number describes the amount of water that turns into runoff during a rainfall event. The number is based on an integration of soil and land use data. Soil data was received from the United States Department of Agriculture Web Soil Survey website [4]. The data provided the various hydrologic soil groups present within the watershed along with their locations and areas. Hydrologic soil groups are defined as a rating of A, B, C, or D. Group A indicates a low runoff potential with high infiltration rates and group D indicates a high runoff potential with low infiltration rates. The “Rating” column in Table 1 below shows the hydrologic soil groups present in the watershed. If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. All areas were assumed to be drained; therefore B/D groups were assumed to be B and C/D groups were assumed to be C. The watershed was now broken into hydrologic soil groups of B and C.

Table 1: Hydrologic Soil Groups Within Watershed

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
AfB2	Alford silt loam, 2 to 5 percent slopes, eroded	B	65.5	29.5%
AfC	Alford silt loam, 5 to 10 percent slopes	B	10.6	4.8%
AfC3	Alford silt loam, 5 to 10 percent slopes, severely eroded	B	34.0	15.3%
Bd	Birds silt loam, 0 to 2 percent slopes, frequently flooded	C/D	9.0	4.1%
Ev	Evansville silt loam	B/D	22.9	10.3%
HoB	Hosmer silt loam, 2 to 5 percent slopes	C/D	8.1	3.6%
HoC3	Hosmer silt loam, 5 to 10 percent slopes, severely eroded	C	0.4	0.2%
IvA	Iva silt loam, 0 to 2 percent slopes	B/D	7.8	3.5%
MuA	Muren silt loam, 0 to 2 percent slopes	B/D	4.2	1.9%
MuB2	Muren silt loam, 2 to 6 percent slopes, eroded	B/D	16.8	7.6%
Wa	Wakeland silt loam, 0 to 2 percent slopes, frequently flooded	B/D	42.4	19.1%
Totals for Area of Interest			**221.8**	**100.0%**

ArcMap GIS software [5] was then used to map and organize the data within the watershed. ArcMap allowed for the collection of various land data to be mapped within the watershed. Figure 5 below shows the hydrologic soil groups map created in ArcMap for the watershed. Hydrologic soil groups B was contained in 92.0% of the watershed and hydrologic soil group C was contained in 8.0% of the watershed.

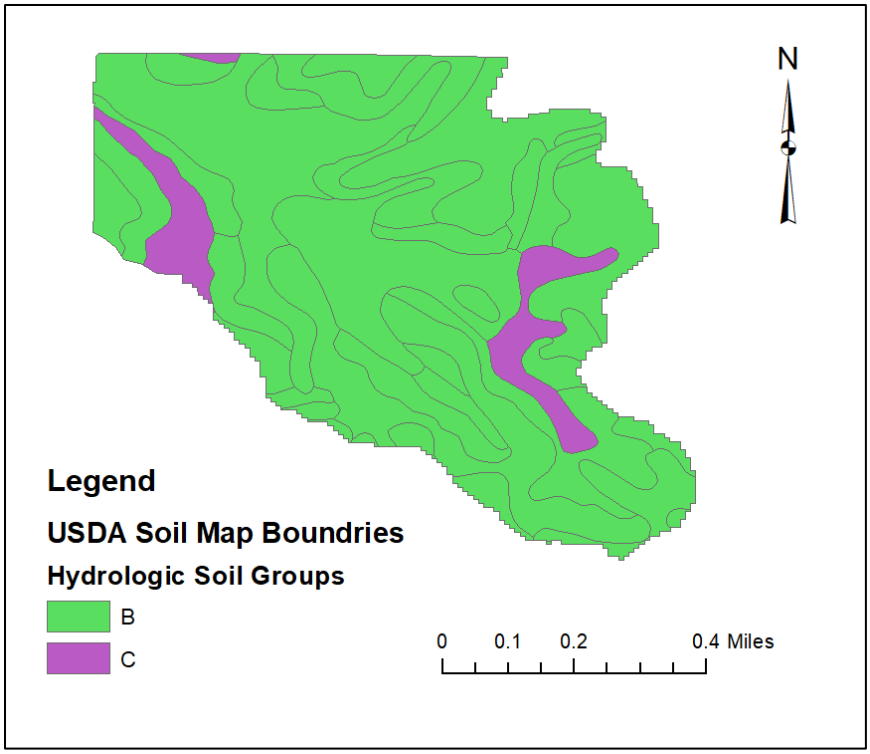

Figure 5: Hydrologic Soil Groups Map

Land cover data was received from the Multi-Resolution Land Characteristics Consortium website [6]. The National Land Cover Database (NLCD) provided land cover data and land cover change at a 30m resolution with a 16-class legend based on a modified Anderson Level II classification system. 2019 National Land Cover Data was chosen as it was the latest data available. Figure 6 shows the land cover map created in ArcMap for the watershed.

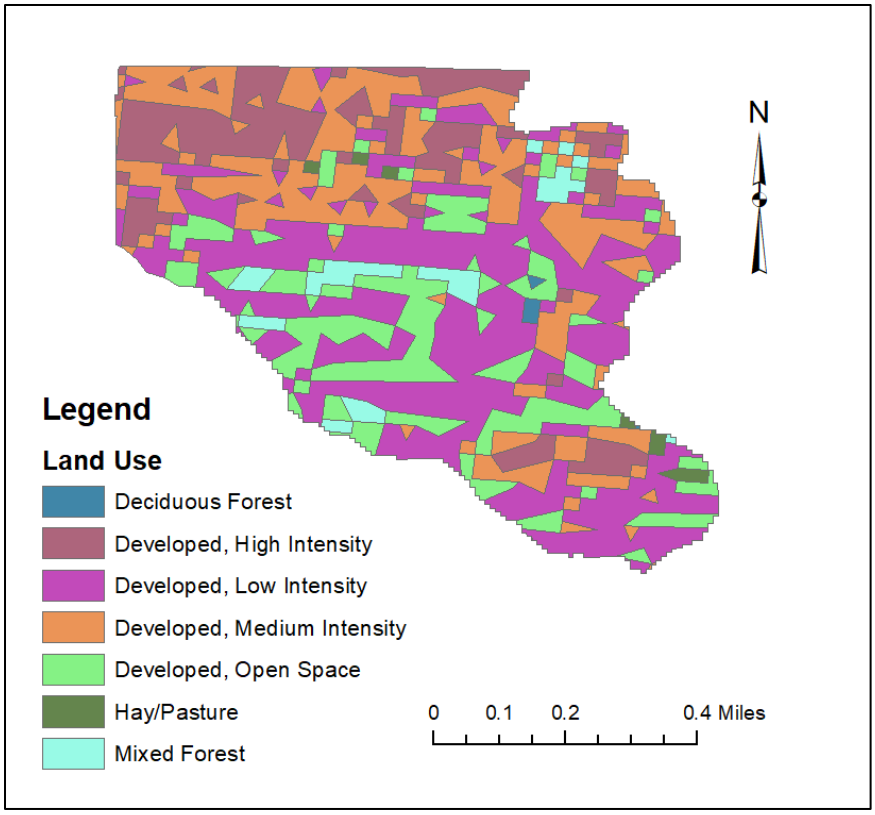

Figure 6: 2019 NLCD Land Cover Map

The hydrologic soil group and land cover maps were used to determine the curve number. A single curve number is described for each specific land cover use and each specific hydrologic soil group. Table 2 below was used to calculate the various curve numbers within the watershed.

Table 2: NLCD Curve Numbers

NLCD LandUse Code	Description	Curve Number			
		A	B	C	D
11	Open water	98	98	98	98
21	Developed, Open Space	72	82	87	89
22	Developed, Low Intensity	77	85	90	92
23	Developed, Medium Intensity	81	88	91	93
24	Developed, High Intensity	98	98	98	98
31	Barren Land	77	86	91	94
42	Evergreen Forest	39.1	48	57	63
43	Mixed Forest	39.1	48	57	63
52	Shrub/Scrub	63	77	85	88
71	Grassland/Herbaceous	68.9	71	81	89
81	Pasture/Hay	68.9	71	81	89
90	Woody Wetlands	87.4	89	90.4	90.7
91	Emergent Herbaceous Wetlands	98	98	98	98

The intersection tool within ArcMap was used to integrate the soil and land use data. Each land cover use and each hydrologic soil group were then selected individually and given their specific curve number value. Figure 7 shows the curve number map created in ArcMap for the watershed.

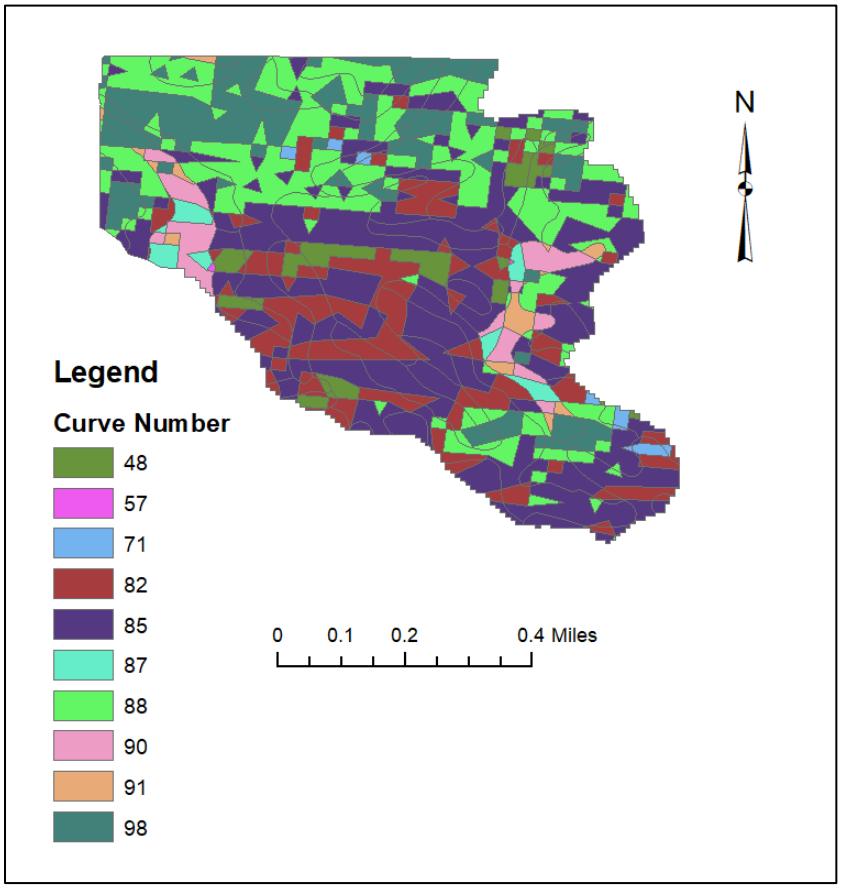

Figure 7: Curve Number Map

The watershed was found to have 10 different curve numbers. A weighted curve number was then calculated based on each curve number's area that it consumed within the watershed. The weighted curve number for the watershed was calculated to be 86. This means that 86% of the rainfall within the watershed will turn into runoff. Therefore, only 14% of the rainfall will be absorbed by the land. The curve number value is high due to the intensity of developed areas throughout the watershed. Most of the watershed is made up of developed subdivisions and neighborhoods. A large portion of the Northern part of the watershed is made up of shopping centers with sizable amounts of impervious areas.

8 PEAK DISCHARGES

Rainfall data was received from the National Oceanic and Atmospheric Administration (NOAA) National Weather Service website [7]. The Precipitation Frequency Data Server (PFDS) was implemented to find rainfall data for the specific study site location. The rainfall data is shown in Figure 28 in Appendix A. From the data provided, an intensity duration frequency (IDF) graph was created and represented all three flood frequency events. The IDF curves are shown in Figure 8. The curves are plotted by duration (minutes) versus rainfall intensity (inches per hour). From an extended version of the IDF curves, 24-hour rainfall intensities were identified for all three storm frequency events. Because of the geographic location of the study site, the rainfall was classified as a Type II rainfall distribution based on the hyetograph. The design flow for the stream was quantified based on a 50-year flooding frequency event. This means, that statistically, every 50 years a flood of this magnitude will flow through the stream. The 50-year frequency event for a Type II rainfall distribution produced precipitation depth of 6.38 inches.

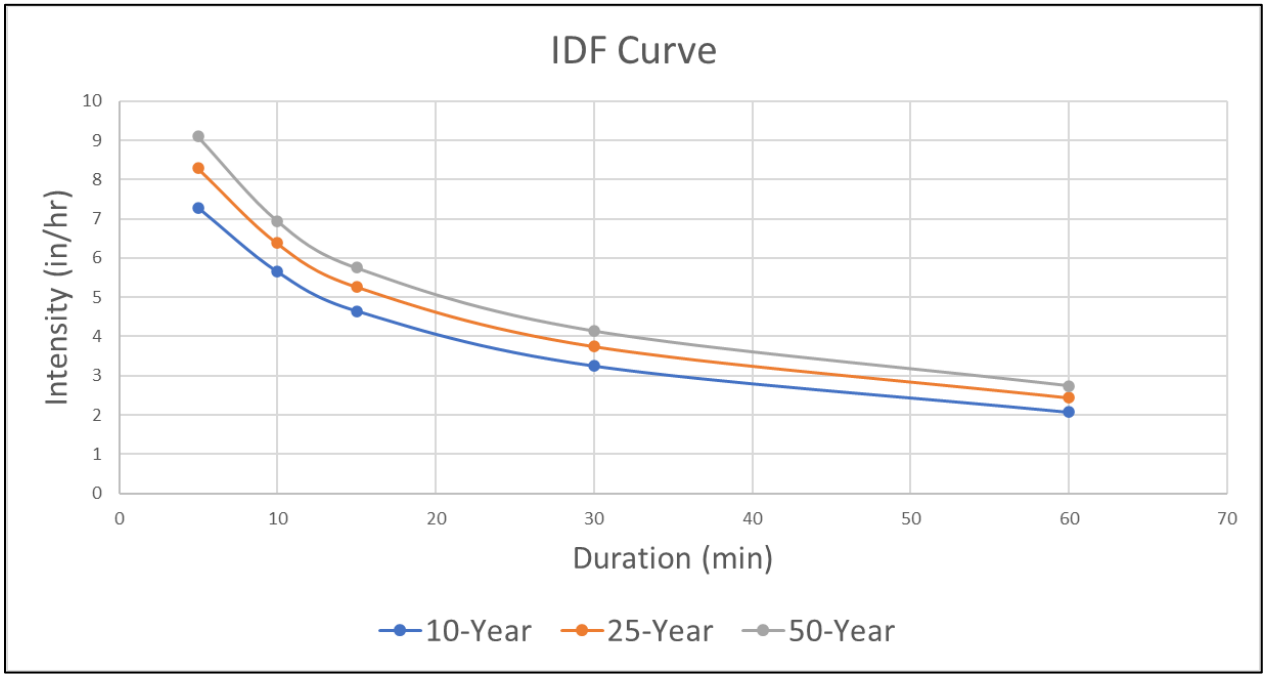

Figure 8: Intensity Duration Frequency (IDF) Curves

Peak discharges were found using Autodesk Civil 3-D's Storm and Sanitary Analysis (SSA) [9]. For this SSA model, two different parameter categories were inputted to complete a successful model, rainfall data and subbasin parameters. Using rainfall depths from the IDF curves to input into SSA, the Huff distribution was used to find peak discharges. The Huff distribution was used

for calculating peak discharges based on historical rainfall data and patterns for central Indiana, specifically. The Huff distribution has four different quartiles which are different time series plots which alter how the depth is distributed over time. Indianapolis Huff Distribution Second Quartile was chosen to be ran through model. The subbasin was created by inputting previously calculated parameters found in the hydrologic study such as the watershed area, curve number and time of concentration. From the completed SSA model, the 50-year peak discharge was found to be 292 cubic feet per second (cfs). This is the discharge that was used to evaluate the stream within the model created in the hydraulic study. 10- and 25-year discharges were provided from a report generated from StreamStats after the completion of delineating the watershed. The 10- and 25-year peak discharges were shown to be 183 cfs and 243 cfs, respectively. It is important to note that only the 50-year flow was calculated within SSA, being that it was the design flow. Table 3 below shows the flows calculated for the stream.

Table 3: Peak Discharges

Peak Discharges		
10-Year	25-Year	50-Year
183 cfs	243 cfs	292 cfs

9 STREAM CHANNEL GEOMETRY

Lochmueller Group surveyed thirteen cross sections of the drainage stream along with mapping the two bridges over the drainage stream. Lochmueller Group also collected other information including spot elevations. Using the survey data and LiDAR data, from OpenTopography [9], a ditch line was constructed in Civil-3D [9]. Due to the survey data being more accurate than the LiDAR data, at any location where there was both survey data and LiDAR data, surveyed data overrode the LiDAR data. From the first cross section surveyed to the last cross section, 3,562.59 feet of channel line are present in the project reach. At each cross section, a polyline was drawn to estimate the survey points of the channel along with the floodplain to create profiles for each cross section. Once the profiles were constructed, a table was generated with Point Numbers, PVI Station, and Elevation.

The measured stations and elevations for the houses and buildings within each cross section can be found in Figure 29 in Appendix A.

10 DETERMINING MANNING'S ROUGHNESS COEFFICIENT

Manning's roughness coefficient quantifies the friction applied to the flow by the channel. This parameter plays an important role in determining energy losses and velocities through each cross section. The *United States Geological Survey's Estimation of Roughness Coefficients* [10] was utilized when determining the roughness coefficients along this channel. Detailed notes were taken during the initial site visit of the varying stream conditions and roughness along each cross section and an example is shown in Table 6 of Appendix A. Specific coefficients were determined using a combination of the notes taken at the site visit and using an aerial along each cross section in Civil 3-D. An example of this is shown in Figure 30 of Appendix A. Figure 9 below shows the horizontal variation of a particular cross section and is reflective of many other cross sections in this project.

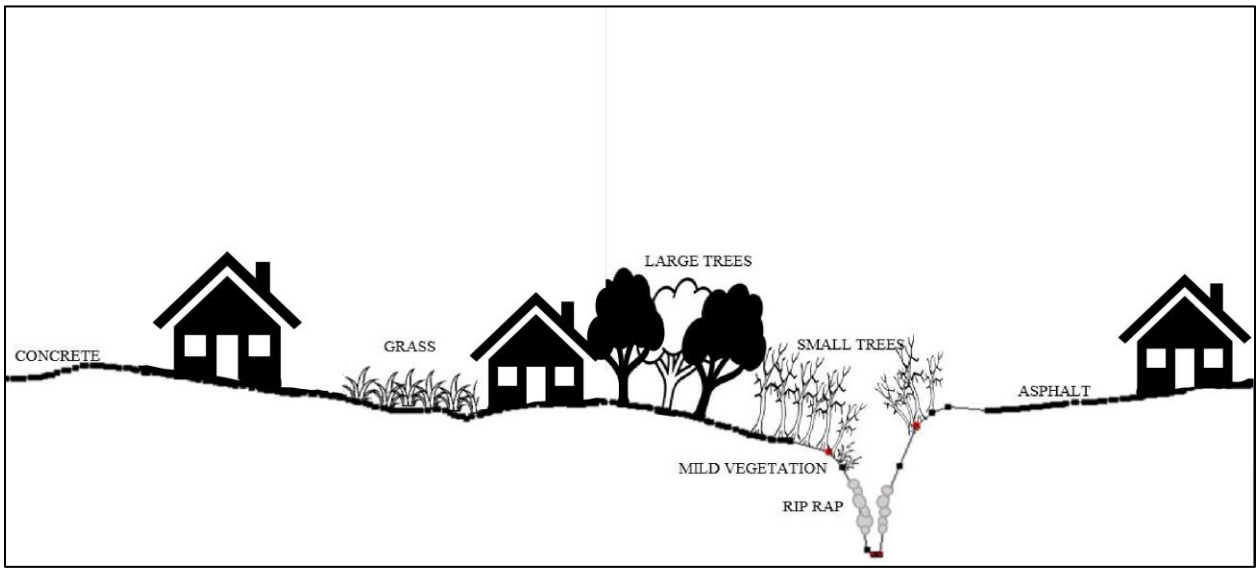

Figure 9: Horizontal Variation in Roughness

11 HYDROLOGIC STUDY CONCLUSIONS

The findings of the hydrologic study are shown below in Table 4. The 50-year discharge of 292 cfs was used in all design alternatives defined in this report.

Table 4: Hydrologic Study Conclusions

Area	Time of Concentration	Curve Number	Peak Discharges		
			10-Year	25-Year	50-Year
221.8 acres	39 min	86	183 cfs	243 cfs	292 cfs

HYDRAULIC STUDY

12 CREATION OF AN EXISTING CONDITIONS MODEL

The use of Hydrologic Engineering Center River Analysis System (HEC-RAS) [11] allowed for an accurate hydraulic model of the stream to be created and performed a one-dimensional steady flow analysis on the stream in its existing condition. This model represented the existing conditions of the stream. This included the stream's geometry, flood plain geometry and structures, as well as the two bridge structures. A steady flow analysis was then run in the model to visualize each of the flooding frequencies at each of the cross sections and along the entire stream reach.

12.1 STREAM CHANNEL AND FLOODPLAIN GEOMETRY

Station versus elevation data created in the hydrologic study were inputted into HEC-RAS for each of the thirteen cross sections. River Stations (RS) for each individual cross section were identified when inputting the reach length to the next cross section immediately downstream. This was the main channel distance between cross sections. Left overbank (LOB) and right overbank (ROB) downstream reach lengths were identified by measuring the distance from the midpoint on the LOB/ROB of the current cross section to the midpoint of the LOB/ROB of the next cross section immediately downstream. The LOB and ROB downstream reach lengths allowed for the model to accurately represent the meandering and bends in the stream. The top of banks were identified using the given survey data from Lochmueller Group. A horizontal variation in roughness values was applied to each of the cross sections. This allowed for the representation of the variation in ground surface throughout the stream channel and floodplain. After the thirteen cross sections were finalized, cross sections were interpolated between the surveyed cross sections to construct a more accurate model that represented more stations along the stream. Interpolated cross sections were created every 100 feet along the channel reach. A total of 32 cross sections were created within the stream reach. The stream reach was identified from RS 0 to RS 3562.59, with RS 0 being the furthermost downstream cross section. Therefore, the model represented a total of channel length of 3,562.59 feet. Figure 10 below shows the geometry created for RS 3236.11 within HEC-RAS. The red points identify the top of bank stations for the left and right side of the stream channel. The horizontal variation in roughness values is represented by the values spanning the top of Figure 10.

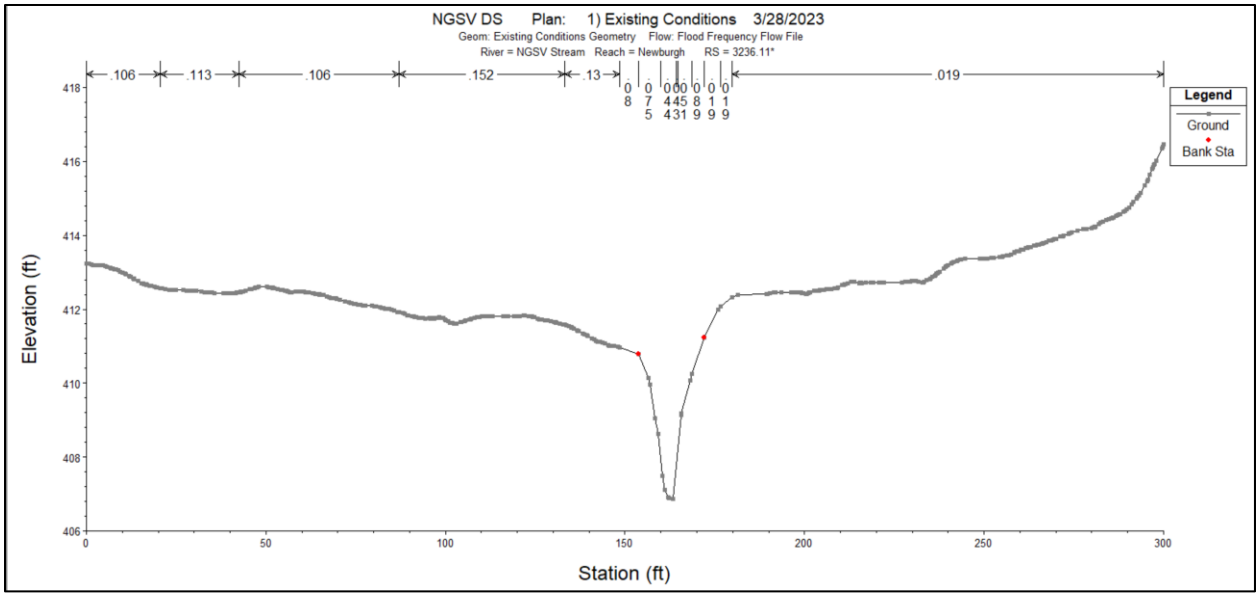

Figure 10: Cross Section Geometry

12.2 BRIDGE STRUCTURES

The two bridge structures located within the stream reach were created within the model. The location of the structures are represented by a red "X" as shown below in Figure 11. The direction of flow is represented by the red arrows. The furthest upstream bridge is designated as "Bridge 1" and the furthest downstream bridge is designated as "Bridge 2".

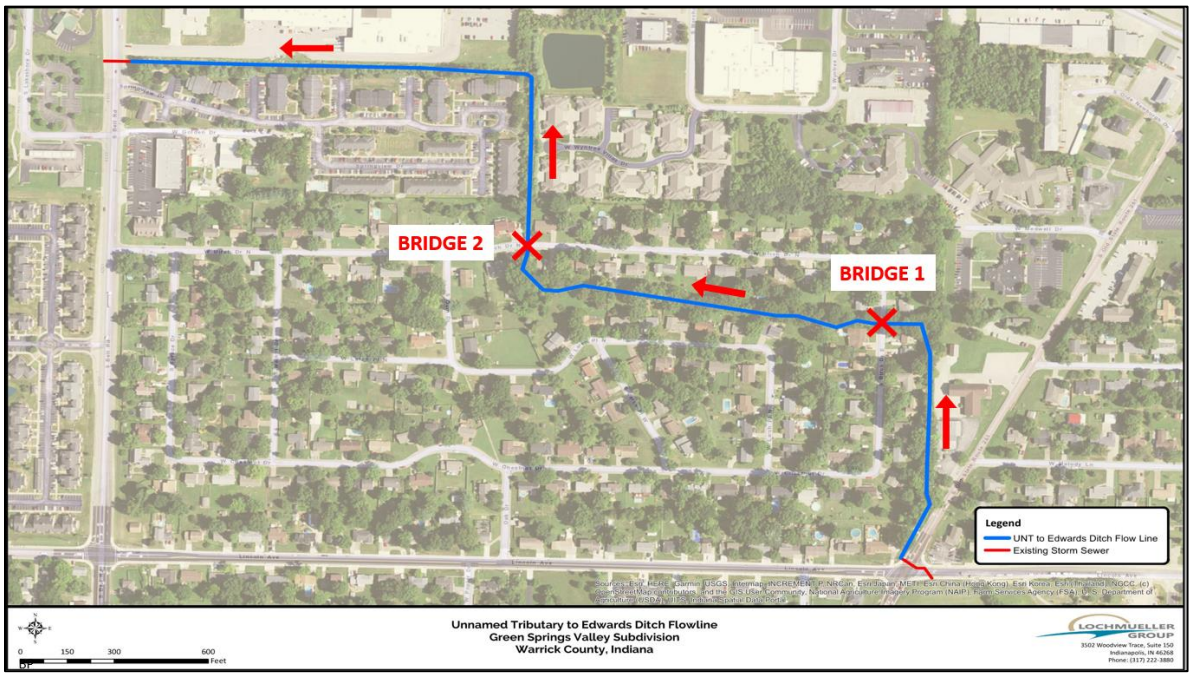

Figure 11: Location of Bridge Structures Within Stream Reach

Through a combination of survey data and field measurements, the geometric make up of each structure was created. The geometry for bridge 1 and bridge 2 are shown below in Figures 12 & 13 respectively. Bridge 1 had a deck thickness of 1.4 feet, width between abutments of 6.8 feet, and channel to bottom of deck height of 2.5 feet. Bridge 2 had a deck thickness of 1.2 feet, width between abutments of 8.1 feet, and channel to bottom of deck height of 3.2 feet.

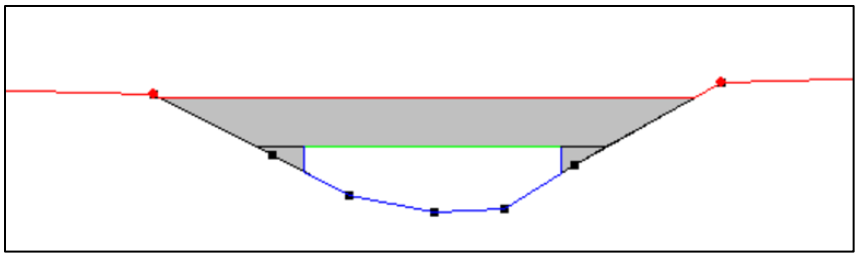

Figure 12: Bridge 1 Geometry

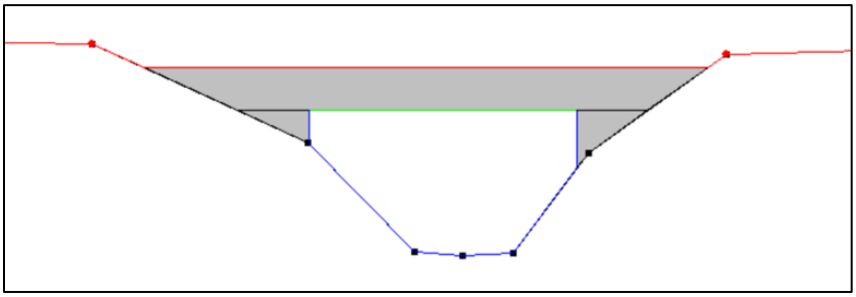

Figure 13: Bridge 2 Geometry

Bridge 1 was created at RS 2890.715 and bridge 2 was created at RS 1546.42. Expansion and contraction coefficients around the bridge structures were chosen from Table 5-2 in the HEC-RAS Hydraulic Reference Manual [12]. A contraction coefficient of 0.3 and expansion coefficient of 0.5 were used. Ineffective flow areas were also modeled around the bridges and applied at the cross sections immediately upstream and downstream of the bridges. This area is defined as the area outside of the contraction just upstream of the bridge and expansion just downstream of the bridge. A 1:1 ratio was used for both contractions and expansions in ineffective flow areas.

12.3 BLOCKED OBSTRUCTIONS

Residential houses and buildings along the stream reach were modeled within respective cross sections to where they appeared. Exact locations, lengths, and elevations of the houses and buildings were found within Civil-3D by measuring along the cross sections in use with the

geolocation tool. They were modeled by creating multiple obstructed areas within the respective cross section. Identifying the locations of houses and buildings within the cross sections allowed for analyzing the model output and how it compares to photos received in the resident questionnaires of flooding levels captured by the residents. The blacked-out stations in Figure 14 below represent the houses and buildings modeled within a cross section.

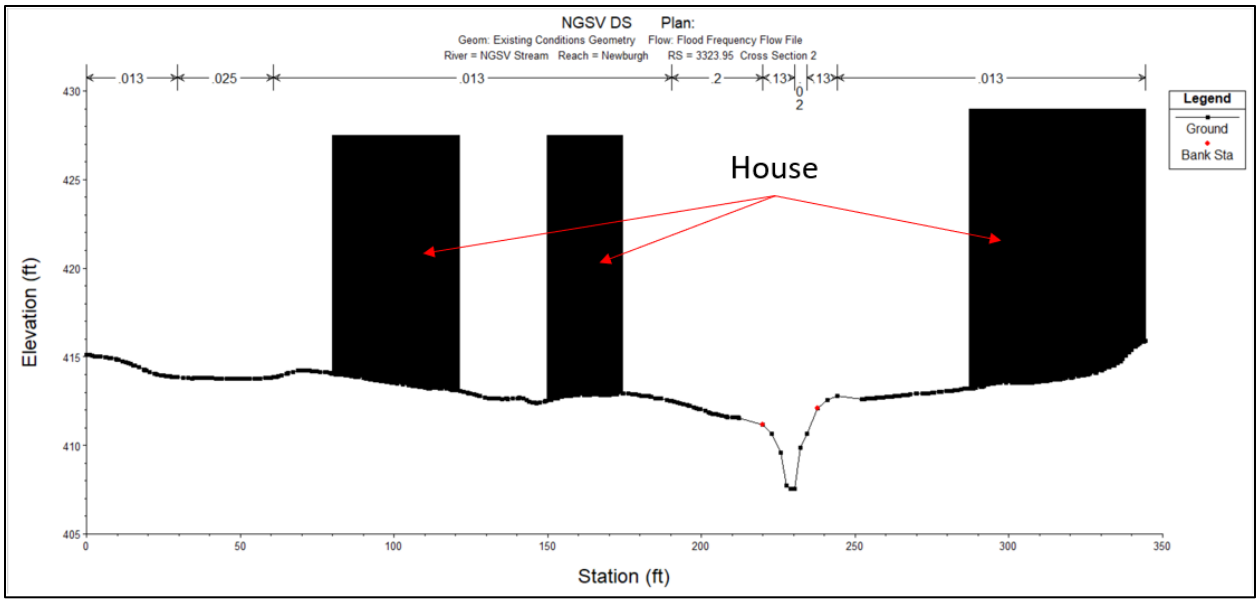

Figure 14: Houses/Building Within Cross Section

Debris in the stream was also represented in the model by inflating the roughness values within the stream channel at respective cross sections. Inflation values were referenced from the United States Geological Survey's *Estimation of Roughness Coefficients* publication [10]. A large debris blockage was identified during the field visit approximately 10 feet upstream of bridge 2. The condition of the debris was described "appreciable" because the obstruction occupies 15 to 50 percent of the cross-sectional area within the channel. Because of this, the roughness values within the channel were inflated by a value of 0.03.

13 CREATION OF A STEADY FLOW FILE & ANALYSIS

A steady flow file was created for the stream reach. The flow file consisted of three profiles. Each profile represented a flow for each flooding frequency (10-, 25-, and 50-year peak discharges). Only the 50-year flow profile was used due to it being the design flow. The steady flow boundary conditions were also created for the upstream and downstream boundaries. The upstream boundary condition was set to critical depth and the downstream boundary conditions

was set to normal depth at a declining slope of 0.002%. This slope was measured in the bottom of the channel bed between the two furthermost downstream cross river stations that were not interpolated (RS 553.20 and RS 0). A plan was created to integrate the existing conditions geometry file and steady flow file. This allowed for the 50-year design flow to be sent through the existing geometry of the stream. The flow regime was set to mixed to represent the flow more accurately throughout the entire stream. Once the geometry and steady flow files were created for the existing conditions of the stream, a steady flow analysis was performed on the stream reach. This allowed the 50-year design flow of 292 cfs to flow through the channel.

14 EVALUATION OF EXISTING CONDITIONS MODEL

The 50-year design flow accurately reflected the flooding within the stream during strong rainfall periods. Figure 15 below shows a cross section view of RS 3323.95 with the top of bank stations referenced. With the water level outside of the top of banks, conditions for flooding were established, as the water is no longer restricted to the stream channel. At this particular river station, the water level is 1.96 feet above the left top of the bank station. The water level also reaches the doorsteps of the surrounding homes in the floodplain.

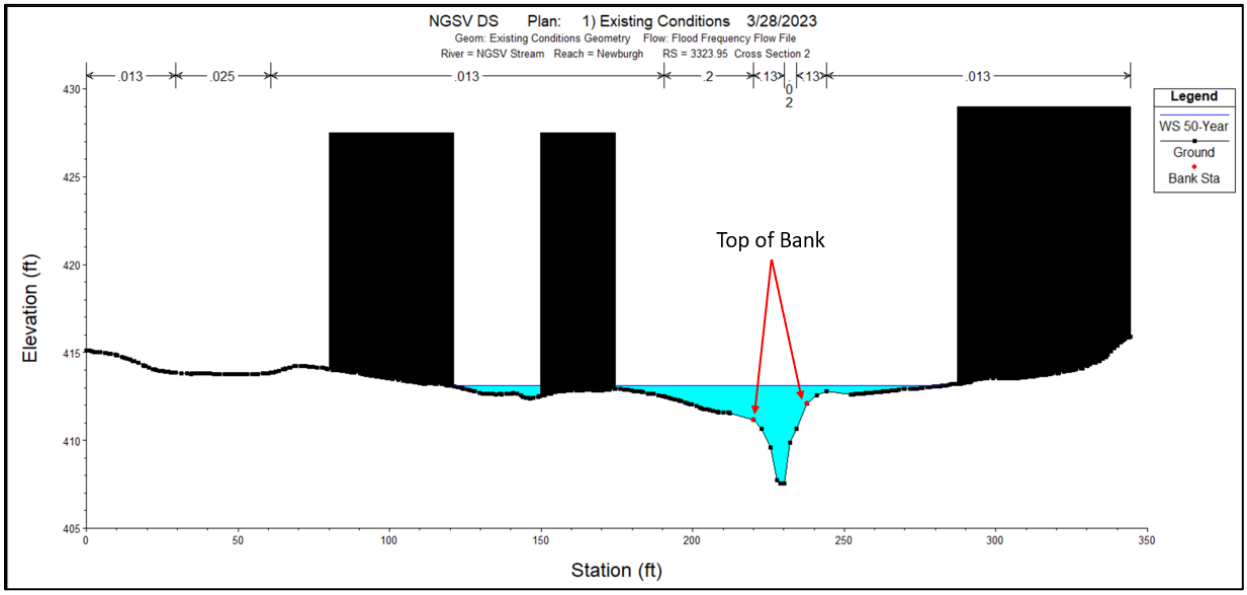

Figure 15: Existing Conditions Model Output (Cross Section View)

Figure 16 below shows a profile plot of the entire stream reach from RS 0 to RS 3562.59. The model accurately represented historical flooding conditions based on photographs sent in with the resident questionnaires. The light blue and purple dashed lines represent the top of bank

stations. The water levels are shown to exceed those elevations for many river stations along the length of the stream, representing flooding. As well as flooding into the floodplain, both bridge structures were overtopped. The bridge structures are represented by the two gray polygons in the reach. Bridge 1 and bridge 2 were overtopped by 1.40 and 1.62 feet, respectively. Because of the overtopping of the bridge structures, weir flow became present at both structures as the water flowed over the top of the bridge deck. Because the water was not able to convey solely through the openings underneath the bridge decks, backwater also became present in the stream. This caused a buildup of water upstream of each structure and therefore caused the water to spill into the floodplain. Water flow velocities are also shown in Figure 16 for various river stations along the stream reach. Locations such as immediately downstream of bridge 1 and the furthermost downstream river station proved to have high velocities in the channel. Erosion control issues come into focus when high velocities like these are present in an open flow channel. These locations were also noted to have erosion issues during the site visit.

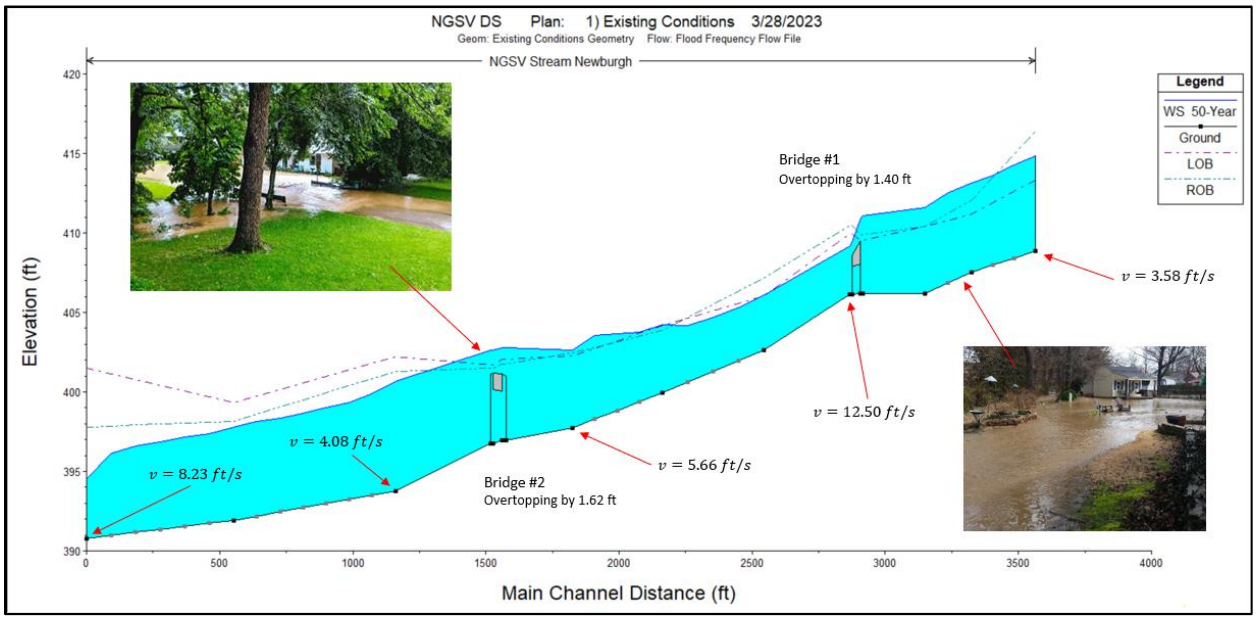

Figure 16: Existing Conditions Model Output (Profile Plot)

The model output table for the existing conditions is shown in Appendix B in Table 7.

PRIMARY CAUSES OF FLOODING

After evaluating the results from the model, having a site visit, and computing size calculations, two primary causes of flooding came into focus. These are the two outlining causes directly related to the flooding in the stream.

15 1ST CAUSE: UNDERSIZED BRIDGE STRUCTURES

The first primary cause of flooding was related to the bridge structures. Both bridge structures were found to be drastically undersized. The structures ranged from 2.5-3.2 feet in height above the stream channel to 6.8-8.1 feet in width between abutments. The model showed both bridge structures being overtopped and causing backwater in the stream. This proves that the opening underneath the structures are not large enough to adequately convey the amount of water for a 50-year flooding event. The velocities of the water in the channel just downstream of each structure were also shown to have excessive quantities, therefore causing erosion in the channel. This is due to the water flowing underneath the structure and the water flowing over top of the structure meeting and causing a higher velocity. Figure 17 below shows a photograph of bridge 1 (most upstream bridge) in its existing condition.

Figure 17: Bridge 1 Existing Condition

16 2ND CAUSE: GEOMETRY OF STREAM

The second primary cause of flooding was related to the geometry of the stream. Specifically related to stream reach between both bridge structures (RS 1546.42 to RS 2890.715). The severity of flooding and erosion was found to be the greatest throughout this reach. The declining slope in the channel bed for this section was also found to be greater than any other section in the stream. It is believed that during initial construction of the stream, this reach was intended to be smaller in size in order to be able to fit more houses in the subdivision and the slope was made grater to try to send the water downstream quicker to reduce the risk of flooding. This may be a solution for small quantity flows, but for the 50-year design flow this is inadequate.

CREATION OF DESIGN ALTERNATIVES

After an evaluation of the hydraulic study output from HEC-RAS, three design alternatives were created. The goal of each design alternative was to restrict the water flow to the banks of the stream, therefore eliminating the flooding. The design alternatives are described in detail below.

17 ALTERNATIVE #1 – CLEARING & GRUBBING

Alternative #1 includes clearing and grubbing of the stream channel. This includes the removal of all trees, shrubs, debris, and any other obstructions in the channel. After completion of the clearing and grubbing, the channel will be seeded with grass. Straw mats with landscape staples will also be installed to ensure seed stabilization.

Alternative #1 was modeled in HEC-RAS by changing Manning's roughness coefficient at each cross-section along the channel. Figure 18 below shows the profile plot for the entire stream reach with comparison of Alternative #1 to the existing conditions model. The model showed to lower the water level in the floodplain by an average of one foot in various cross sections along the stream reach except for immediately downstream of bridge 1, where the water level was found to increase. Both bridge structures were found to still be overtopped and causing backwater to occur upstream of each structure. The water level overtopping bridge 1 was found to raise by 0.1 feet and the water level overtopping bridge 2 was found to lower by 0.3 feet.

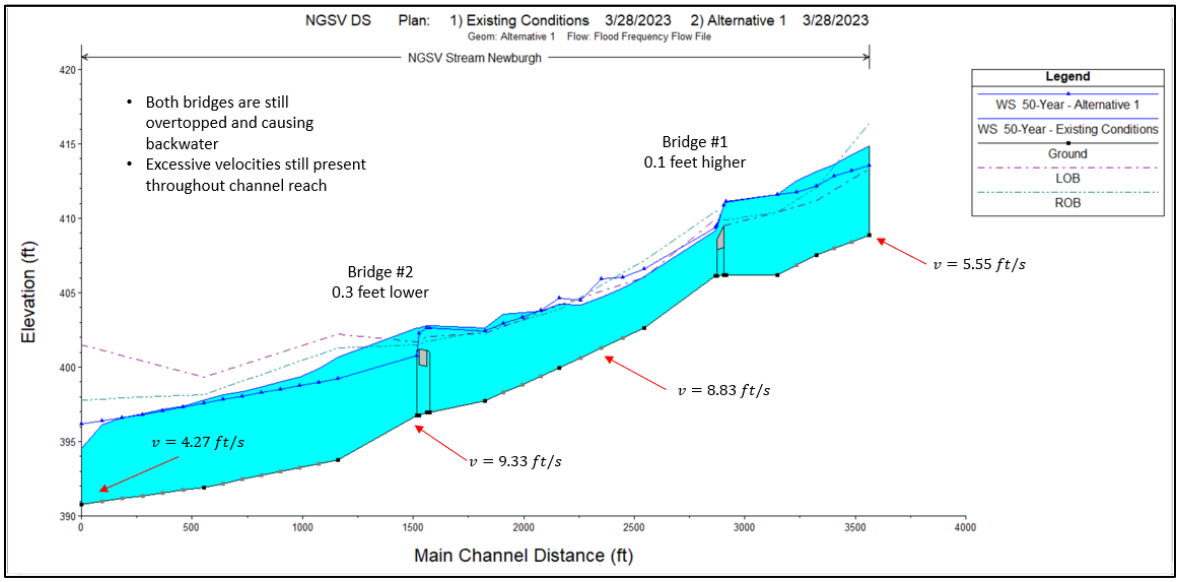

Figure 18: Alternative #1 Profile Plot

Figure 19 below aids in the representation of lowering the flooding levels in the floodplain. The Alternative #1 water level is shown to decrease by 1 foot compared to the existing conditions water level at this particular river station (RS 3323.95). The water level is shown still exceeds the top of bank stations (left and right) and spills into the floodplain but does lower in the floodplain when compared to the existing conditions level.

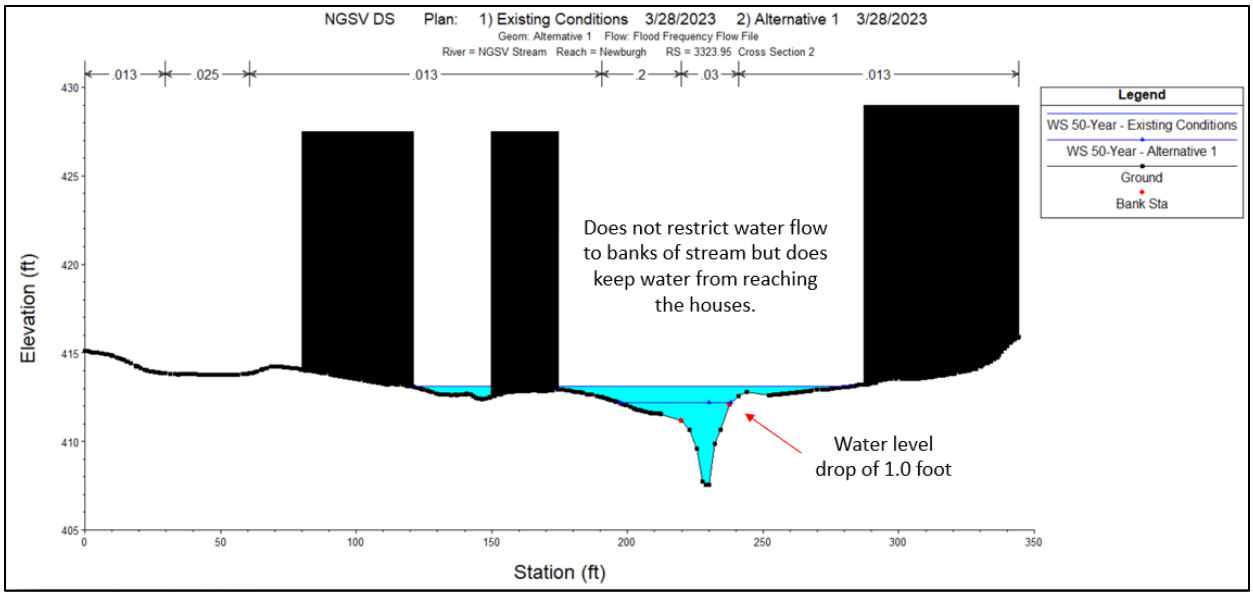

Figure 19: Alternative #1 Cross Sectional View

It is important to note that Alternative #1 does not follow the goal of the design alternatives in regard to reducing the water level to the banks of the stream channel. This alternative was proposed strictly to provide a cheap and quick option to at least lower the flooding levels away from the residents' homes. Flooding will still occur, but at a lower severity level.

The model output table for Alternative #1 is shown in Appendix B in Table 8.

18 ALTERNATIVE #2 – RESIZE STRUCTURES & GEOMETRY OF CHANNEL

Alternative #2 includes a complete redesign of the sizing of the two bridge structures and the geometry of portions of the channel along the stream reach. Both existing structures will be removed and replaced with reinforced concrete box culverts. The box culverts will be 16 feet in span length, 4 feet in rise length, and 12 inches in thickness all around. Each box culvert will be 36 feet in length, making up six 6-foot sections. Wing walls and a head wall will be added to each end of the culverts to provide erosion and roadway protection. Flowable fill will be used to

backfill around each structure as well as soil around each wing wall. All design criteria for the precast concrete box culverts were based off information given at the *American Concrete Products* webpage [13].

The geometry of the stream channel will be modified to a trapezoidal channel with a 12 foot base, 6 foot height, and 2:1 side slopes. The channel will be excavated with this design from RS 1576.42 to RS 2860.715 and RS 2920.715 to RS 3562.59. In order to properly fit the box culverts within the channel, the trapezoidal design will need to be extended to a 16 foot base at each structure location and 30 feet upstream and downstream of each structure to ensure proper expansion/contractions in the channel. The channel will be excavated with this design from RS 1516.42 to RS 1576.42 and RS 2860.716 to RS 2920.715. Allow a minimum of 10 feet to integrate both channel designs. Due to the redesign of the channel geometry, a total of 4,304 cubic yards of earth will need to be excavated from the stream reach. Figure 20 below shows the geometry design locations throughout the stream reach. Both box culverts are located with a red "X". The stream locations highlighted in yellow will be designed with the 12 foot base and the stream locations highlighted in black will be designed with the 16 foot base. The stream locations highlighted in blue will remain in its existing conditions. This portion of the stream reach was found to adequately convey the water for the 50-year design flow.

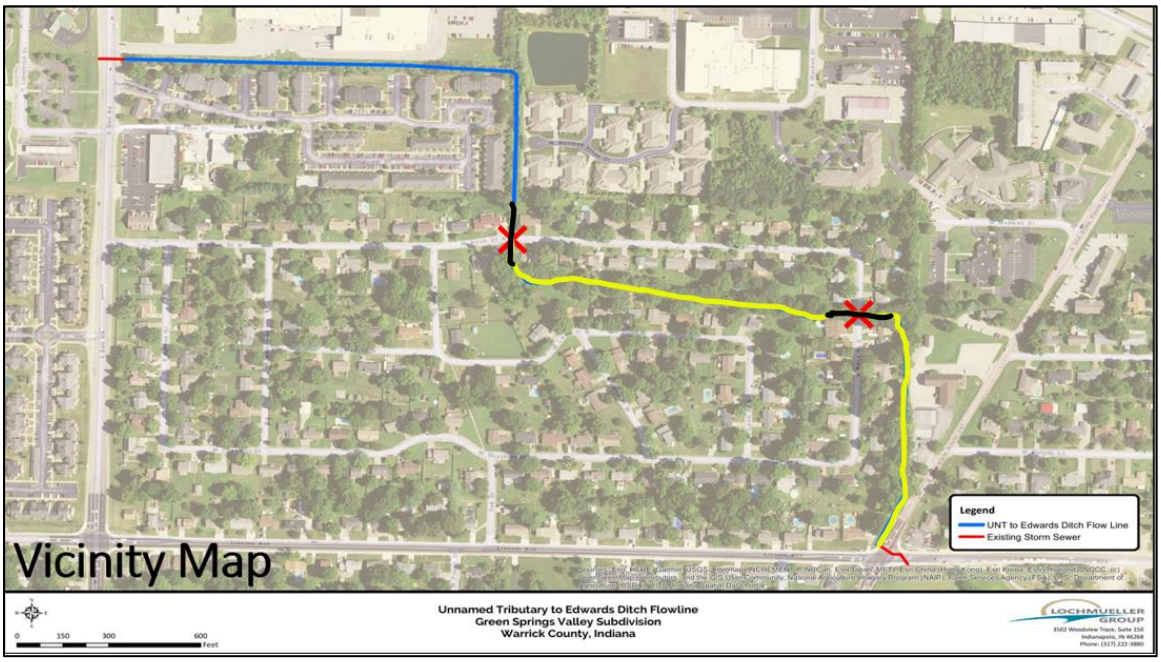

Figure 20: Geometry Changes Throughout Stream Reach

Once excavation and box culvert installation are complete, the channel will be seeded with grass for the incorporation of a grass lined channel. Straw mats with landscape staples will also be installed to ensure seed stabilization. Land acquisition will need to occur due to the proposed 2:1 channel side slope. Due to this slope, the proposed top of bank stations will overreach the existing easements along the channel.

Alternative #2 was modeled in HEC-RAS by changing the model design parameters. Figure 21 below shows the profile plot for the entire stream reach with the incorporation of the changes outlined in Alternative #2. The model shows the water levels throughout the entire stream reach staying within the banks of the channel and out of the floodplain. There is no flooding occurring anywhere in the stream. The water is also properly conveyed through each box culvert while leaving an average of 0.8 feet of clearance between the water lever and the interior top of box for each culvert. Box culvert 1 does not show up on the profile plot because the top of the culvert (12 inch thickness) lies at a higher elevation than the top of bank stations. Due to this, the roadway approaches leading to the structure will need to be raised in elevation by 1.23 feet on the Northern side and 1.63 feet on the Southern side of the structure.

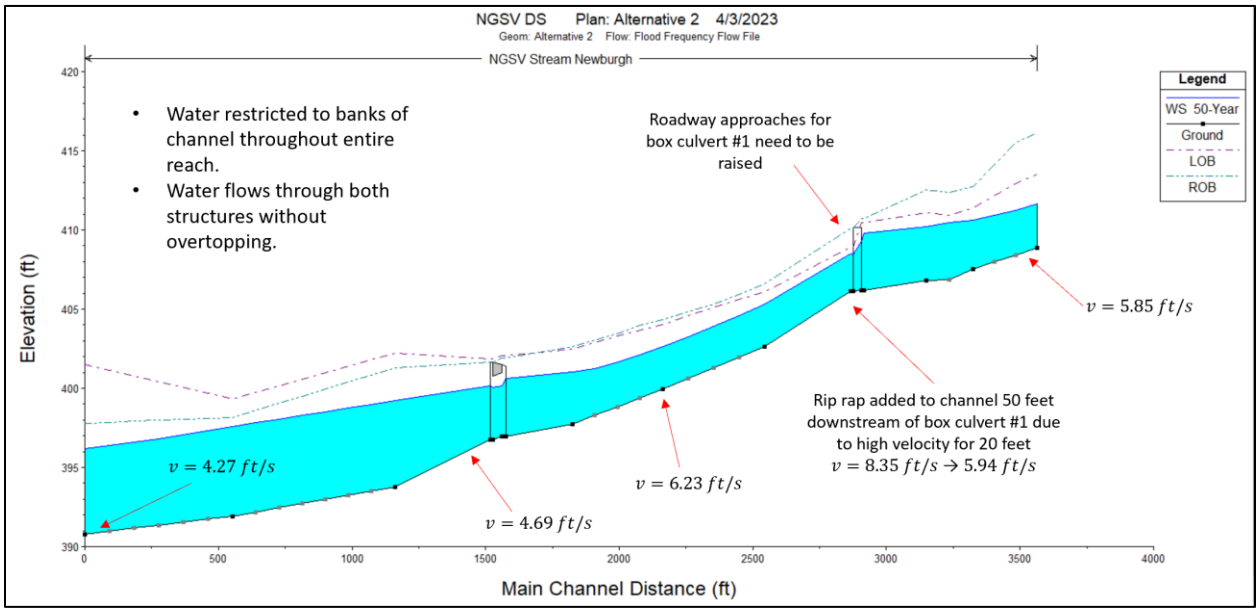

Figure 21: Alternative #2 Profile Plot

The velocities at all river stations were verified to be adequate except for approximately 25 feet downstream of box culvert 1. The velocity at this river station was found to be 8.35 ft/s, which caused alarm for erosion control issues. Therefore, the channel was lined with 6 inch diameter

rip rap for 20 feet in length (from RS 2845.86 to RS 2865.86). This brought the velocity of that section down to 7.80 ft/s as well supplying erosion control for the channel.

Figure 22 below shows a cross sectional view of the channel at RS 3323.95. The trapezoidal channel can be seen as well as the water level remaining within the banks of the channel. There is no water flow present in the floodplain and the resident's homes are safe due to the fact that there is no flooding occurring.

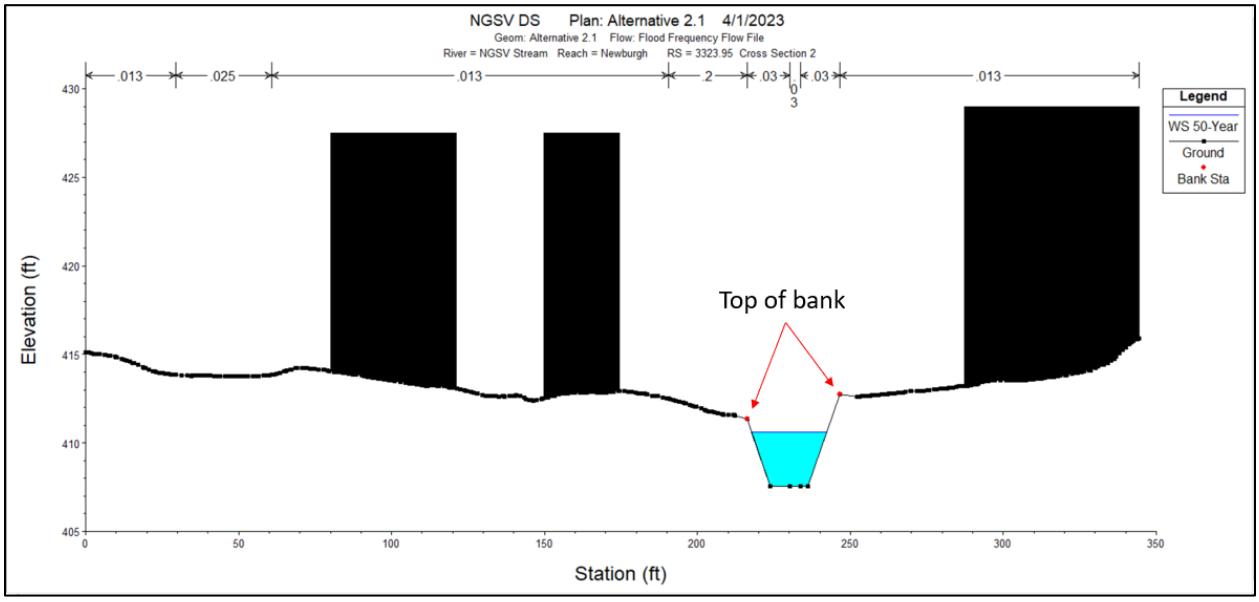

Figure 22: Alternative #2 Cross Sectional View

Alternative #2 was designed in an iterative process. River stations along the stream reach were redesigned to various trapezoidal designs, as well as sizing various box culverts. Designs such as only modifying the geometry around the structures and only modifying the structure sizes were found to be inadequate for the stream to convey the 50-year design flow. The design outlined in Alternative #2 was found to be the minimum change in design to prevent the stream from flooding.

The model output table for Alternative #2 is shown in Appendix B in Table 9.

19 ALTERNATIVE #3 – RESIZE STRUCTURES, GEOMETRY OF CHANNEL, & INSTALL FLEXAMAT

Alternative #3 includes a complete redesign of the sizing of the two bridge structures and the geometry of portions of the channel along the stream reach. Both existing structures will be removed and replaced with reinforced concrete box culverts. These design changes are the same changes made within Alternative #2. The difference between Alternative #2 and #3 is that Alternative #3 includes the installation of Flexamat within the channel. Flexamat (also known as tied concrete block mat) is a vegetated concrete block mat utilized for stabilizing slopes, channels, low water crossings, inlet/outlet protection, and shorelines. It consists of concrete blocks (6.5" x 6.5" with a 2.25" profile) locked together and embedded into a high-strength geogrid. There is 1.5 spacing between the blocks that gives the mat flexibility and allows for controlled vegetation growth. The establishment of perennial vegetation increases infiltration of storm water runoff into the soil, increased removal pollutants found in road and parking lots runoff through filtration. The mat is packaged in rolls, making transporting, and installing Flexamat extremely efficient. Flexamat is designed to control erosion as well as provide stable driving/walking surfaces. This drastically minimizes future stream maintenance. An example of Flexamat installed within a stream is shown below in Figure 23.

Figure 23: Flexamat Erosion Protection

Flexamat was designed for the stream channel per design parameters and information provided by the Flexamat website [14]. The Flexamat roughness value calculations for each individual river station are shown in Figure 31 in Appendix A.

Flexamat will be installed along the entire channel where the channel geometry will be changed. This is from RS 1516.42 to RS 3562.59 (30 feet downstream of box culvert 2 to the furthermost upstream river station). Flexamat will be installed throughout the outlined reach from top of bank to top of bank in cross sectional view. For the installation of Flexamat, it will be rolled out spanning the banks of the channel as well as the bed of the channel. Approximately 8,884 square yards of Flexamat will be installed within the outlined channel reach.

As stated above, Alternative #3 includes the same design changes as Alternative #2, but with the inclusion of Flexamat. The main purpose of this inclusion is to provide erosion control within the channel where it had been excavated to the new trapezoidal design. Alternative #3 was modeled in HEC-RAS by changing the model design parameters. Figure 24 below shows the profile plot for the entire stream reach with comparison of Alternative #3 to Alternative #2.

The model shows the water levels throughout the entire stream reach staying within the banks of the channel and out of the floodplain. There is no flooding occurring anywhere in the stream. The water is also properly conveyed through each box culvert while leaving an average of 0.7 feet of clearance between the water lever and the interior top of box for each culvert. Box culvert 1 does not show up on the profile plot because the top of the culvert (12 inch thickness) lies at a higher elevation than the top of bank stations. Due to this, the roadway approaches leading to the structure will need to be raised in elevation by 1.23 feet on the Northern side and 1.63 feet on the Southern side of the structure.

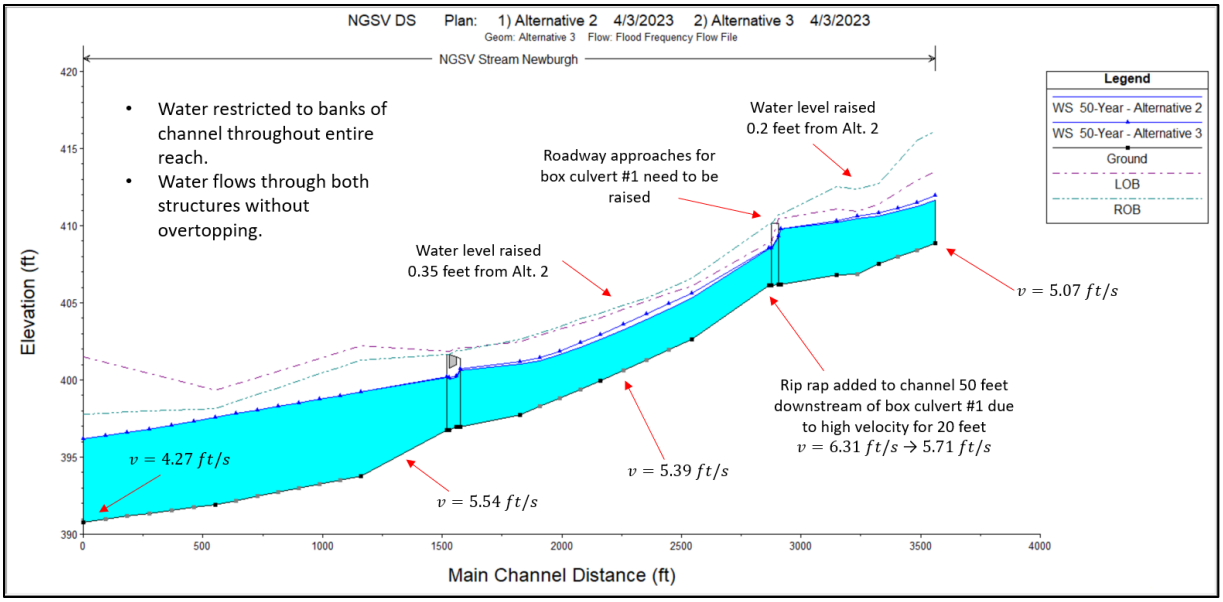

Figure 24: Alternative #3 Profile Plot

Water levels in parts of the stream reach were found to increase from 0.2-0.35 feet in elevation. Even though this occurred, the water remained within the banks of the stream. Because of the installation of Flexamat, the velocities at all river stations were verified to be adequate. Only one mildly excessive velocity was found approximately 25 feet downstream of box culvert 1. The velocity at this river station was found to be 6.31 ft/s, which caused an alarm for erosion control issues. Therefore, the channel was lined with 6-inch diameter rip rap for 20 feet in length (from RS 2845.86 to RS 2865.86). This is the same location of rip rap installation in Alternative #2. This brought the velocity of that section down to 5.71 ft/s as well supplying erosion control for the channel.

The model output table for Alternative #3 is shown in Appendix B in Table 10.

DESIGN ALTERNATIVE SELECTION

20 ALTERNATIVE #2

It is important to note that the 50-year design flow of 292 cfs was used to redesign the stream and both structures to prevent flooding. With that being said, both the 10-year flow of 183 cfs and 25-year flow of 243 cfs also stayed within the banks of the stream for Alternative #2 and #3.

After evaluating all three design alternatives, Alternative #2 was chosen as the most beneficial alternative to eliminate the flooding in the stream. Alternative #2 included the minimum design changes needed to properly convey the 50-year design flow of 292 cfs. The water was restricted to the banks of the stream, therefore eliminating flooding. The water also adequately flowed through both box culverts without overtopping the structures. The velocities found throughout the entire stream reach were found to be acceptable and caused no alarm for erosion issues, except for one location where rip rap was installed to deal with that issue. Even though Alternative #3 provides more erosion control, adding Flexamat installation to the channel does not prove to be substantial enough when looking at the costs of material and installation.

CONCLUDING THOUGHTS

The drainage stream in the North Green Springs Valley subdivision is in dire need of some kind of attention. It was found that the existing stream geometry from the most upstream river station to just downstream of bridge 2 is undersized as well as the bridge structures themselves. It is recommended that at a minimum, Alternative #1 (clearing and grubbing of the stream channel) is completed. This action would at least lower the flooding levels away from the resident's homes. As stated above, Alternative #2 was chosen as the primary choice for the design alternative.

When deriving each alternative, cost was considered in the design changes and an effort was made to keep the cost at a minimum. The exact amount of cost was not calculated for each alternative. It can be said that Alternative #1 is the least expensive option by far in comparison to the other two alternatives. Alternative #2 and #3 cost considerably more. Alternative #3 is the most expensive, being that it is the same as Alternative #2, with the addition of installation of Flexamat.

During the site visit while speaking with one of the residents, they stated that the flooding events occurred more frequently and substantial after the construction of the First Christian Church and the detention pond that lies on its property. This statement was also recorded in many of the resident questionnaires. The site visit confirmed that all of the rainfall around that area drains into the detention basin through storm inlets throughout the church parking lot. From there, the water flows directly into the entrance of the stream through the detention pond's exit structure and various culverts with steep downward slopes. This makes a large impact on the capabilities of the stream to convey its water flow during a heavy rainfall period. The detention pond was not taken into consideration for either the hydrologic or hydraulic studies within this project. It would be highly recommended to analyze the detention pond and its effects on the stream. This could include resizing the detention pond, redesigning the exit structure, or the culverts connecting the detention pond to the stream. Figure 25 below shows some of the possible effects of the detention pond on the stream. The picture in the top left shows the exit structure and the top right picture shows the detention pond. The bottom left picture shows the flow path from the detention pond into the stream. The bottom right picture shows the large parking lot and impervious areas that directly drain into the detention pond.

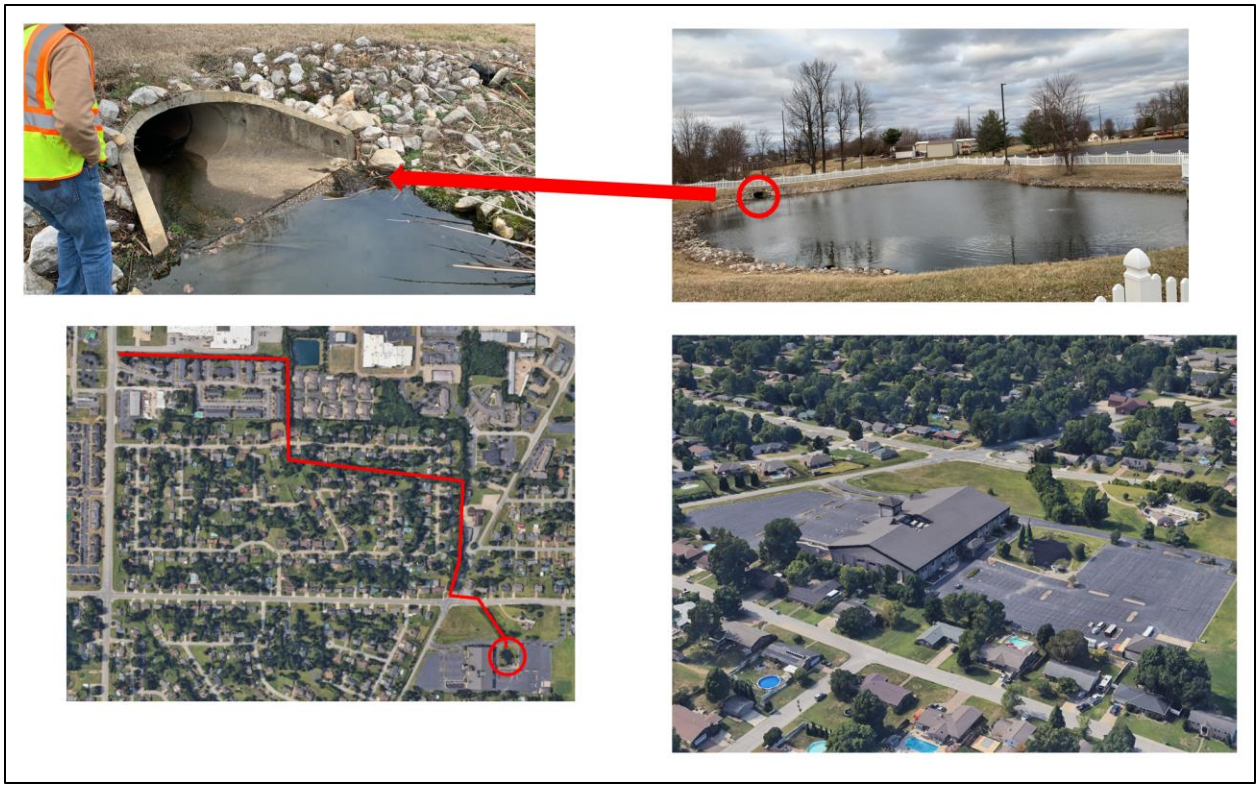

Figure 25: Possible Effects of Detention Pond

PROJECT TEAMWORK

Throughout the entirety of the project, the team showed collaborative teamwork. The project was divided into six phases with various tasks within each phase. The team divided and conquered each of the tasks and allowed for input of the other team members to aid in the competition of said task. The phase and tasks breakdown can be seen in Table 5 below.

Table 5: Completion of Project Tasks

Phase/Task	Completed By
Phase 1: Review of Project Files	
Background Review of Information from Lochmueller	All
Review Questionnaire Results	All
Review Survey & LiDAR Data	All
Review Stormwater Drainage Ordinants	All
Site Visit	All
Phase 2: Hydrologic Study	-
Delineate Watershed Boundary	Cade Prechtel
Calcualte Runoff Curve Number	Cade Prechtel
Calculate Time of Concentration	Brooklyn Pittman
Calculate Peak Discharges (10-, 25-, & 50-Years)	Cade Prechtel
Develop Hydrologic Model for Watershed in SSA	Cole Cooper
Create Channel & Floodplain Geometry	Cole Cooper
Determine Manning's Roughness Coefficients	Brooklyn Pittman
Phase 3: Hydraulic Study (HEC-RAS)	-
Create a model with existing conditions geometry	Cade Prechtel
Incorperate Horizontal Variation of Roughness Values	Cade Prechtel
Incorperate Existing Debris in Channel	Cade Prechtel
Incorperate Expansions/Contractions	Cade Prechtel
Incorperate Blocked Obstructions (Houses/Buildings)	Cade Prechtel
Incorperate Both Bridge Structures in Model	Cade Prechtel
Incorperate Ineffective Flow Areas Around Structures	Cade Prechtel
Finalize Existing Conditions Model	Cade Prechtel
Evaluate Existing Conditons Model with 50-Year Design Flow	Cade Prechtel
Phase 4: Creation of Design Alternatives	-
Creation of Design Alternative #1	Cade Prechtel
Creation of Design Alternative #2	Cade Prechtel
Creation of Design Alternative #3	Cade Prechtel
Phase 5: Evaluation of Design Alternatives	-
Evaluation of Design Alternaitve #1	All
Evaluation of Design Alternaitve #2	All
Evaluation of Design Alternaitve #3	All
Phase 6: Project Deliverables	-
Final Design Presentation	All
Final Design Poster	All
Final Design Report	All

Cole Johnson (employee at Lochmueller Group who provided this project) served as the team's industry liaison for the project. One meeting occurred between the project team and the liaison. The meeting took place via video teleconference. The purpose of this meeting was for the liaison to explain how to create cross section profiles using the LiDAR surface within Autodesk Civil-3D. The team was experiencing difficulties with this task prior to this meeting. The information provided by the liaison allowed the team to complete this task and continue with the advancement of the project.

Dr. Jason Hill, Ph.D., P.E. served as the team's project advisor. The team met with the project advisor approximately 3-4 hours weekly to continue the advancement of the project and to provide updates throughout the project's stages. Many of these meetings were scheduled on a weekly basis, but others arose as needed. The project advisor aided in providing knowledge and insight to the team.

It is also noted that the team was able to acquire knowledge from two new software programs. These two programs included ArcMap GIS and HEC-RAS. None of the team members have used these programs prior to this project. Many hours were spent learning the ins and outs of both programs in order to complete this project in a professional manner. Besides the two new software programs, there were many skills that the team learned throughout the development of the project. Because of these newly acquired knowledge, the team can carry these skills and apply them in future endeavors.

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APPENDIX

Appendix A: Calculations & Data

Appendix B: HEC-RAS Model Output Tables

Appendix C: Preliminary Project Schedule

Appendix D: ABET Outcome 2, Design Factor Considerations

APPENDIX A

Figure 26: Time of Concentration Calculations (1)

Figure 27: Time of Concentration Calculations (2)

Point precipitation frequency estimates (inches/hour)
NOAA Atlas 14 Volume 2 Version 3
Data type: Precipitation intensity
Time series type: Partial duration
Project area: Ohio River Basin
Location name (ESRI Maps): Newburgh, Indiana, USA
Station Name: NEWBURGH
Latitude: 37.9736°
Longitude: -87.4047°
Elevation (USGS): 397.82 ft

data imported from: https://hdsc.nws.noaa.gov/hdsc/pfds/

PRECIPITATION FREQUENCY ESTIMATES

by duration for ARI (years):	1	2	5	10	25	50	100	200	500	1000
5-min:	4.63	5.47	6.48	7.28	8.29	9.1	9.89	10.7	11.8	12.6
10-min:	3.62	4.28	5.05	5.65	6.38	6.95	7.49	8.05	8.75	9.28
15-min:	2.96	3.51	4.16	4.65	5.27	5.75	6.22	6.68	7.29	7.72
30-min:	1.97	2.36	2.86	3.25	3.75	4.14	4.53	4.93	5.45	5.85
60-min:	1.21	1.45	1.8	2.07	2.44	2.74	3.04	3.36	3.79	4.13
2-hr:	0.728	0.881	1.1	1.28	1.52	1.71	1.91	2.12	2.41	2.64
3-hr:	0.52	0.629	0.787	0.915	1.09	1.24	1.39	1.55	1.78	1.96
6-hr:	0.319	0.385	0.48	0.558	0.668	0.758	0.854	0.954	1.1	1.21
12-hr:	0.188	0.227	0.282	0.326	0.389	0.44	0.494	0.55	0.63	0.694
24-hr:	0.113	0.136	0.17	0.197	0.235	0.266	0.299	0.333	0.382	0.421
2-day:	0.068	0.081	0.101	0.117	0.141	0.161	0.182	0.204	0.236	0.263
3-day:	0.048	0.058	0.072	0.083	0.1	0.114	0.129	0.146	0.169	0.188
4-day:	0.038	0.046	0.057	0.066	0.08	0.091	0.103	0.116	0.135	0.151
7-day:	0.026	0.031	0.038	0.044	0.053	0.06	0.068	0.077	0.09	0.1
10-day:	0.02	0.024	0.03	0.035	0.041	0.047	0.053	0.06	0.07	0.077
20-day:	0.014	0.016	0.02	0.022	0.026	0.029	0.032	0.035	0.039	0.043
30-day:	0.011	0.013	0.016	0.018	0.021	0.023	0.025	0.027	0.03	0.032
45-day:	0.01	0.011	0.013	0.015	0.017	0.018	0.02	0.021	0.023	0.025
60-day:	0.009	0.01	0.012	0.013	0.015	0.016	0.017	0.018	0.02	0.021

Date/time (GMT): Fri Jan 13 02:19:33 2023
pyRunTime: 0.018620967865

Figure 28: NOAA Rainfall Data

Stations along Cross Sections						
	Left Over Bank			Right Over Bank		
XSEC 1						
XSEC 2	80	to	121.2	287.17	to	344.31
	149.8	to	174.4			
XSEC 3	0	to	37.63	148.56	to	175.56
XSEC 4						
XSEC 5	191	to	245			
XSEC 6	209.5	to	236.5	421.24	to	459.34
XSEC 7	25.7	to	80.7	244.74	to	273.74
XSEC 8				340.12	to	367.92
				398.82	to	457.32
				485.72	to	548.92
XSEC 9						
XSEC 10	0	to	9.7	688.15	to	721.15
	54.5	to	105.4	745.85	to	808.05
	136.4	to	187.5	833.65	to	890.75
	245.6	to	299.6			
XSEC 11	110.8	to	247.4	1044.52	to	1092.92
	396.6	to	532.1	1146.02	to	1173.72
	641.1	to	703.5			
XSEC 12	36.4	to	95.4	842.95	to	947.55
	189.1	to	228.6			
XSEC 13				485.69	to	579.09

Elevations along Cross Sections						
	Left Over Bank			Right Over Bank		
XSEC 1						
XSEC 2	427.41	to	427.74	428.22	to	430.9
	427.41		427.74			
XSEC 3	426.36	to	426.09	427.23	to	427.3
XSEC 4						
XSEC 5	424.65	to	425.01			
XSEC 6	422.75	to	421.8	424.41	to	424.51
XSEC 7	420.64	to	419.94	419.85	to	421.64
XSEC 8				418.93	to	418.7
				418.74	to	419.51
				420.36	to	421.69
XSEC 9						
XSEC 10	420.34	to	417.89	418.81	to	418.75
	417.61	to	417.61	418.74	to	418.98
	417.61	to	418	419.84	to	420.95
	418	to	418.59			
XSEC 11	419.71	to	420.5	417.36	to	418.19
	420.5	to	419.15	420.11	to	423.74
	420.5	to	419.15			
XSEC 12	417.34	to	418.09	415.67	to	416.1
	419.34	to	415.55			
XSEC 13				413.5	to	411.9

Figure 29: House/Building Stations & Elevations Along Cross Sections

Station	n	Notes
0	0.025	Grass-Maintained Yards
80		House
125	0.025	Grass- Maintained yards
150		House
210	0.013	Concrete driveway
275		House
410	0.013	Concrete driveway
440	0.2	Large trees
450	0.13	Left of Bank: Large rip rap
455	0.07	Ditch Line: Rip rap fallen into ditch, mud/soil
460	0.13	Right of Bank: Large rip rap, small trees
500	0.025	Grass-Maintained yard
540	0.013	Concrete Driveway
770	0.025	Grass- Maintained yards
780		House
890		House

Figure 30: Manning's Roughness Coefficient Calculations (Cross Section 10)

Table 6: Manning's Roughness Coefficient Notes

Cross Section Number (s)	Notes
1,2,3	Large Rip Rap lining both left and right of bank. Very mild vegetation along channel left of bank. Trees throughout entire cross sections left of bank. Trees beginning towards end of cross section one and remaining through entirety of cross sections 2 and 3 right of bank. Two culverts present at beginning of cross section one
4,5	Large Rip Rap lining left and right of bank entirely through cross section four, stopping mid way through cross section 5. Exposed soil right of bank. Trees present left of bank and right of bank. Bridge 1 passes over these cross sections.
6,7,8	Cross sections not accessible due to private property.
9,10	Large Rip Rap lining both left and right of bank. Heavy trees present left of bank. Light/sparce trees present right of bank. Little to no exposed soil on either bank. Bridge 2 passes over these cross sections. Debris/blockage is present downstream of the bridge.
11	No Rip Rap left or right of bank. Large amounts of exposed soil right of bank. Trees present in both left and right of bank and many branches have fallen into the stream. Gravel present within the channel. Channel begins to narrow here (compared to cross sections 12&13).
12,13	No Rip Rap left or right of bank. Heavy trees present left of bank with dead vegetation/grass covering a majority right of bank. Mild exposed soil right of bank.

Flexamat Channel n-Values

Station	Channel Elev.	Water Elev.	Depth of Water	n-Value
3562.59	408.85	411.64	2.79	0.038
3483.04*	408.41	411.25	2.84	0.037
3403.50*	407.98	410.92	2.94	0.035
3323.95	407.54	410.62	3.08	0.035
3236.11*	406.86	410.48	3.62	0.034
3148.26	406.82	410.22	3.4	0.035
2915.57	406.18	409.77	3.59	0.034
2890.715	Bridge			
2865.86	406.13	408.5	2.37	0.039
2544.39	402.64	405.29	2.65	0.038
2448.69*	401.97	404.61	2.64	0.038
2352.99*	401.3	403.93	2.63	0.038
2257.29*	400.62	403.27	2.65	0.038
2161.59	399.95	402.65	2.7	0.037
2077.33*	399.4	402.11	2.71	0.037
1993.08*	398.84	401.63	2.79	0.036
1908.82*	398.29	401.25	2.96	0.035
1824.56	397.73	401.02	3.29	0.034
1574.66	396.98	400.62	3.64	0.033
1546.42	Bridge			
1518.18	396.76	400.14	3.38	0.034
1160.11	393.76	399.2	5.44	0.03
1073.40*	393.5	398.97	5.47	0.03
986.70*	393.23	398.74	5.51	0.03
899.99*	392.97	398.51	5.54	0.03
813.29*	392.71	398.29	5.58	0.03
726.58*	392.45	398.06	5.61	0.03
639.88*	392.18	397.82	5.64	0.03
553.17	391.92	397.57	5.65	0.03
460.98*	391.73	397.3	5.57	0.03
368.78*	391.54	397.04	5.5	0.03
276.59*	391.35	396.79	5.44	0.03
184.39*	391.16	396.59	5.43	0.03
92.19*	390.97	396.38	5.41	0.03
0	390.78	396.2	5.42	0.03

Average Water Depth = 4.06

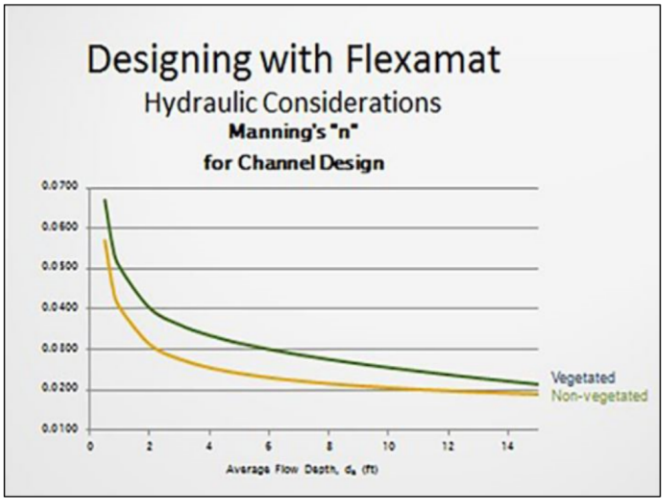

Figure 31: Flexamat n-Value Calculations

APPENDIX B

Table 7: Existing Conditions Model Output Table

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Newburgh	3562.59	50-Year	292.50	408.85	414.85	412.45	415.04	0.007639	3.58	103.12	45.96	0.31
Newburgh	3483.04*	50-Year	292.50	408.41	414.29		414.44	0.007692	3.39	126.25	79.72	0.31
Newburgh	3403.50*	50-Year	292.50	407.98	413.63		413.77	0.010410	3.38	123.75	103.53	0.32
Newburgh	3323.95	50-Year	292.50	407.54	413.14		413.22	0.005599	2.09	133.31	138.08	0.20
Newburgh	3236.11*	50-Year	292.50	406.86	412.53		412.67	0.005282	3.41	142.12	171.94	0.33
Newburgh	3148.26	50-Year	292.50	406.17	411.58		411.85	0.016683	4.42	100.78	74.59	0.45
Newburgh	2915.57	50-Year	292.50	406.18	411.10	409.97	411.36	0.000810	4.50	165.38	185.37	0.43
Newburgh	2890.715		Bridge									
Newburgh	2865.86	50-Year	292.50	406.13	409.19	409.26	411.59	0.175875	12.50	24.23	192.64	1.59
Newburgh	2544.39	50-Year	292.50	402.64	406.06	405.84	407.05	0.004257	7.99	36.65	17.34	0.88
Newburgh	2448.69*	50-Year	292.50	401.97	405.33	405.33	406.57	0.004529	8.92	32.81	13.44	1.01
Newburgh	2352.99*	50-Year	292.50	401.30	404.71	404.86	406.13	0.004206	9.56	30.59	12.68	1.09
Newburgh	2257.29*	50-Year	292.50	400.62	404.17	404.39	405.72	0.004067	9.99	29.27	12.04	1.13
Newburgh	2161.59	50-Year	292.50	399.95	404.24	404.60	405.15	0.003697	7.89	60.16	112.42	0.82
Newburgh	2077.33*	50-Year	292.50	399.40	403.76	403.33	404.57	0.004587	7.27	51.52	106.00	0.78
Newburgh	1993.08*	50-Year	292.50	398.84	403.67	402.80	404.06	0.004736	5.31	93.98	189.32	0.55
Newburgh	1908.82*	50-Year	292.50	398.29	403.56	402.33	403.69	0.003151	3.44	152.51	236.18	0.35
Newburgh	1824.56	50-Year	292.50	397.73	402.60	401.88	403.10	0.028979	5.66	55.04	61.32	0.66
Newburgh	1574.66	50-Year	292.50	396.98	402.78	400.96	402.78	0.000071	0.24	520.30	613.14	0.02
Newburgh	1546.42		Bridge									
Newburgh	1518.18	50-Year	292.50	396.76	402.62	400.99	402.69	0.003838	1.69	148.49	190.02	0.16
Newburgh	1160.11	50-Year	292.50	393.76	400.68		400.81	0.007584	2.88	101.43	30.76	0.28
Newburgh	1073.40*	50-Year	292.50	393.50	399.88		400.06	0.009539	3.39	86.24	28.08	0.34
Newburgh	986.70*	50-Year	292.50	393.23	399.33		399.55	0.003987	3.72	78.60	26.78	0.38
Newburgh	899.99*	50-Year	292.50	392.97	399.00		399.22	0.003485	3.81	76.72	26.77	0.40
Newburgh	813.29*	50-Year	292.50	392.71	398.66		398.90	0.003876	3.92	74.66	26.72	0.41
Newburgh	726.58*	50-Year	292.50	392.45	398.36		398.60	0.003063	4.00	73.11	26.84	0.43
Newburgh	639.88*	50-Year	292.50	392.18	398.14		398.38	0.002144	3.96	73.91	27.49	0.43
Newburgh	553.17	50-Year	292.50	391.92	397.80		398.06	0.007816	4.08	72.29	39.88	0.45
Newburgh	460.98*	50-Year	292.50	391.73	397.38		397.68	0.002458	4.43	66.02	25.10	0.48
Newburgh	368.78*	50-Year	292.50	391.54	397.14		397.45	0.002724	4.41	66.25	24.35	0.47
Newburgh	276.59*	50-Year	292.50	391.35	396.87		397.18	0.003044	4.45	65.79	23.67	0.47
Newburgh	184.39*	50-Year	292.50	391.16	396.61		396.89	0.003113	4.33	69.69	45.36	0.45
Newburgh	92.19*	50-Year	292.50	390.97	396.13	394.83	396.49	0.006325	4.81	60.84	22.01	0.51
Newburgh	0	50-Year	292.50	390.78	394.56	394.56	395.61	0.012832	8.23	35.53	17.15	1.01

Table 8: Alternative #1 Model Output Table

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Newburgh	3562.59	50-Year	292.50	408.85	413.53	412.47	414.01	0.003603	5.55	53.63	26.62	0.57
Newburgh	3483.04*	50-Year	292.50	408.41	413.20	412.24	413.71	0.003895	5.73	54.54	39.15	0.59
Newburgh	3403.50*	50-Year	292.50	407.98	412.84	412.06	413.38	0.004422	5.94	58.56	44.24	0.63
Newburgh	3323.95	50-Year	292.50	407.54	412.17	411.88	412.91	0.007556	6.94	53.33	41.18	0.80
Newburgh	3236.11*	50-Year	292.50	406.86	411.75	411.26	412.30	0.005160	6.08	60.63	58.24	0.68
Newburgh	3148.26	50-Year	292.50	406.17	411.58		411.92	0.002643	4.85	100.16	74.49	0.50
Newburgh	2915.57	50-Year	292.50	406.18	411.15	410.00	411.40	0.001773	4.47	173.23	189.74	0.42
Newburgh	2890.715		Bridge									
Newburgh	2865.86	50-Year	292.50	406.13	409.37	409.37	409.56	0.004384	4.76	108.73	213.76	0.59
Newburgh	2544.39	50-Year	292.50	402.64	406.59	405.83	407.12	0.004433	6.02	56.05	63.17	0.61
Newburgh	2448.69*	50-Year	292.50	401.97	406.04	405.34	406.64	0.005032	6.40	54.23	68.36	0.65
Newburgh	2352.99*	50-Year	292.50	401.30	405.93	404.86	406.22	0.002466	4.87	107.25	139.53	0.46
Newburgh	2257.29*	50-Year	292.50	400.62	404.48	404.40	405.69	0.011574	8.83	33.12	12.57	0.96
Newburgh	2161.59	50-Year	292.50	399.95	404.66	404.57	404.93	0.002743	4.96	120.34	154.07	0.48
Newburgh	2077.33*	50-Year	292.50	399.40	403.82	403.33	404.56	0.006405	7.01	58.22	119.07	0.74
Newburgh	1993.08*	50-Year	292.50	398.84	403.36	402.80	404.03	0.005947	6.59	49.84	75.25	0.72
Newburgh	1908.82*	50-Year	292.50	398.29	402.91	402.33	403.52	0.005710	6.22	48.83	43.46	0.71
Newburgh	1824.56	50-Year	292.50	397.73	402.40	401.87	403.00	0.006504	6.23	47.13	24.86	0.75
Newburgh	1574.66	50-Year	292.50	396.98	402.62	400.96	402.63	0.000083	0.88	428.06	586.78	0.09
Newburgh	1546.42		Bridge									
Newburgh	1518.18	50-Year	292.50	396.76	400.75	400.99	402.10	0.016567	9.33	31.34	15.18	1.15
Newburgh	1160.11	50-Year	292.50	393.76	399.20	397.41	399.54	0.002780	4.69	62.36	22.07	0.49
Newburgh	1073.40*	50-Year	292.50	393.50	398.97		399.30	0.002723	4.64	63.04	22.66	0.49
Newburgh	986.70*	50-Year	292.50	393.23	398.74		399.07	0.002676	4.59	63.77	23.22	0.49
Newburgh	899.99*	50-Year	292.50	392.97	398.51		398.83	0.002652	4.54	64.44	23.83	0.49
Newburgh	813.29*	50-Year	292.50	392.71	398.29		398.60	0.002655	4.50	64.96	24.40	0.49
Newburgh	726.58*	50-Year	292.50	392.45	398.06		398.37	0.002682	4.47	65.37	24.98	0.49
Newburgh	639.88*	50-Year	292.50	392.18	397.82		398.13	0.002748	4.46	65.52	25.49	0.49
Newburgh	553.17	50-Year	292.50	391.92	397.57		397.89	0.002900	4.50	65.01	25.89	0.50
Newburgh	460.98*	50-Year	292.50	391.73	397.30		397.62	0.002847	4.58	63.92	24.62	0.50
Newburgh	368.78*	50-Year	292.50	391.54	397.04		397.36	0.002731	4.59	63.69	23.82	0.50
Newburgh	276.59*	50-Year	292.50	391.35	396.79		397.12	0.002597	4.58	63.93	23.31	0.49
Newburgh	184.39*	50-Year	292.50	391.16	396.59		396.89	0.002267	4.38	69.13	44.89	0.46
Newburgh	92.19*	50-Year	292.50	390.97	396.38	394.83	396.68	0.002203	4.40	66.51	22.98	0.46
Newburgh	0	50-Year	292.50	390.78	396.20	394.56	396.48	0.002001	4.27	68.44	23.00	0.44

Table 9: Alternative #2 Model Output Table

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Newburgh	3562.59	50-Year	292.50	408.85	411.64	411.12	412.18	0.005470	5.85	50.03	23.49	0.71
Newburgh	3483.04*	50-Year	292.50	408.41	411.25		411.75	0.005075	5.69	51.44	23.81	0.68
Newburgh	3403.50*	50-Year	292.50	407.98	410.92		411.36	0.004301	5.35	54.69	24.51	0.63
Newburgh	3323.95	50-Year	292.50	407.54	410.62		411.03	0.003742	5.10	57.30	24.75	0.59
Newburgh	3236.11*	50-Year	292.50	406.86	410.48		410.75	0.002063	4.13	70.86	26.80	0.45
Newburgh	3148.26	50-Year	292.50	406.82	410.22		410.54	0.002646	4.52	64.78	25.85	0.50
Newburgh	2915.57	50-Year	292.50	406.18	409.77	408.18	409.96	0.001866	3.51	83.28	32.39	0.39
Newburgh	2890.715		Bridge									
Newburgh	2865.86	50-Year	292.50	406.13	408.50	408.13	409.05	0.014241	5.94	49.25	25.49	0.75
Newburgh	2544.39	50-Year	292.50	402.64	405.29		405.92	0.006985	6.39	45.75	22.59	0.79
Newburgh	2448.69*	50-Year	292.50	401.97	404.61		405.25	0.007007	6.40	45.70	22.58	0.79
Newburgh	2352.99*	50-Year	292.50	401.30	403.93		404.58	0.007090	6.43	45.51	22.55	0.80
Newburgh	2257.29*	50-Year	292.50	400.62	403.27		403.90	0.006939	6.38	45.86	22.61	0.79
Newburgh	2161.59	50-Year	292.50	399.95	402.65		403.25	0.006494	6.23	46.95	22.80	0.77
Newburgh	2077.33*	50-Year	292.50	399.40	402.11		402.71	0.006408	6.20	47.16	22.83	0.76
Newburgh	1993.08*	50-Year	292.50	398.84	401.63		402.18	0.005755	5.97	49.00	23.16	0.72
Newburgh	1908.82*	50-Year	292.50	398.29	401.25		401.73	0.004577	5.50	53.15	23.86	0.65
Newburgh	1824.56	50-Year	292.50	397.73	401.02		401.38	0.003092	4.78	61.15	25.16	0.54
Newburgh	1574.66	50-Year	292.50	396.98	400.62	398.96	400.80	0.001344	3.45	84.66	30.54	0.37
Newburgh	1546.42		Bridge									
Newburgh	1518.18	50-Year	292.50	396.76	400.14	398.76	400.36	0.001769	3.81	76.83	29.51	0.42
Newburgh	1160.11	50-Year	292.50	393.76	399.20		399.54	0.002780	4.69	62.36	22.07	0.49
Newburgh	1073.40*	50-Year	292.50	393.50	398.97		399.30	0.002723	4.64	63.04	22.66	0.49
Newburgh	986.70*	50-Year	292.50	393.23	398.74		399.07	0.002676	4.59	63.76	23.22	0.49
Newburgh	899.99*	50-Year	292.50	392.97	398.51		398.83	0.002652	4.54	64.44	23.83	0.49
Newburgh	813.29*	50-Year	292.50	392.71	398.29		398.60	0.002655	4.50	64.96	24.40	0.49
Newburgh	726.58*	50-Year	292.50	392.45	398.06		398.37	0.002682	4.47	65.37	24.98	0.49
Newburgh	639.88*	50-Year	292.50	392.18	397.82		398.13	0.002748	4.46	65.52	25.49	0.49
Newburgh	553.17	50-Year	292.50	391.92	397.57		397.89	0.002900	4.50	65.01	25.89	0.50
Newburgh	460.98*	50-Year	292.50	391.73	397.30		397.62	0.002847	4.58	63.92	24.62	0.50
Newburgh	368.78*	50-Year	292.50	391.54	397.04		397.36	0.002731	4.59	63.69	23.82	0.50
Newburgh	276.59*	50-Year	292.50	391.35	396.79		397.12	0.002597	4.58	63.93	23.31	0.49
Newburgh	184.39*	50-Year	292.50	391.16	396.59		396.89	0.002267	4.38	69.13	44.89	0.46
Newburgh	92.19*	50-Year	292.50	390.97	396.38	394.83	396.68	0.002204	4.40	66.51	22.98	0.46
Newburgh	0	50-Year	292.50	390.78	396.20	394.56	396.48	0.002001	4.27	68.44	23.00	0.44

Table 10: Alternative #3 Model Output Table

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Newburgh	3562.59	50-Year	292.50	408.85	411.96	411.12	412.36	0.005880	5.07	57.69	24.76	0.59
Newburgh	3483.04*	50-Year	292.50	408.41	411.51		411.91	0.005547	5.06	57.86	24.87	0.58
Newburgh	3403.50*	50-Year	292.50	407.98	411.14		411.50	0.004474	4.86	60.18	25.39	0.56
Newburgh	3323.95	50-Year	292.50	407.54	410.81		411.16	0.004094	4.72	61.95	25.49	0.53
Newburgh	3236.11*	50-Year	292.50	406.86	410.62		410.86	0.002299	3.92	74.58	27.35	0.42
Newburgh	3148.26	50-Year	292.50	406.82	410.33		410.62	0.003213	4.33	67.49	26.27	0.48
Newburgh	2915.57	50-Year	292.50	406.18	409.81	408.18	410.00	0.001897	3.45	84.69	34.00	0.38
Newburgh	2890.715		Bridge									
Newburgh	2865.86	50-Year	292.50	406.13	408.58	408.13	409.09	0.012722	5.71	51.20	25.80	0.71
Newburgh	2544.39	50-Year	292.50	402.64	405.64		406.10	0.006998	5.41	54.08	24.02	0.64
Newburgh	2448.69*	50-Year	292.50	401.97	404.97		405.43	0.007019	5.41	54.02	24.01	0.64
Newburgh	2352.99*	50-Year	292.50	401.30	404.29		404.75	0.007105	5.44	53.79	23.98	0.64
Newburgh	2257.29*	50-Year	292.50	400.62	403.61		404.07	0.007123	5.44	53.74	23.96	0.64
Newburgh	2161.59	50-Year	292.50	399.95	402.96		403.41	0.006560	5.39	54.30	24.05	0.63
Newburgh	2077.33*	50-Year	292.50	399.40	402.40		402.86	0.006660	5.42	54.00	24.00	0.64
Newburgh	1993.08*	50-Year	292.50	398.84	401.88		402.32	0.006017	5.33	54.91	24.16	0.62
Newburgh	1908.82*	50-Year	292.50	398.29	401.45		401.85	0.004880	5.04	57.99	24.66	0.58
Newburgh	1824.56	50-Year	292.50	397.73	401.17		401.49	0.003345	4.50	65.03	25.77	0.50
Newburgh	1574.66	50-Year	292.50	396.98	400.71	398.96	400.88	0.001483	3.34	87.47	30.91	0.35
Newburgh	1546.42		Bridge									
Newburgh	1518.18	50-Year	292.50	396.76	400.22	398.76	400.44	0.002066	3.68	79.45	29.86	0.40
Newburgh	1160.11	50-Year	292.50	393.76	399.20		399.54	0.002780	4.69	62.36	22.07	0.49
Newburgh	1073.40*	50-Year	292.50	393.50	398.97		399.30	0.002723	4.64	63.04	22.66	0.49
Newburgh	986.70*	50-Year	292.50	393.23	398.74		399.07	0.002676	4.59	63.76	23.22	0.49
Newburgh	899.99*	50-Year	292.50	392.97	398.51		398.83	0.002652	4.54	64.44	23.83	0.49
Newburgh	813.29*	50-Year	292.50	392.71	398.29		398.60	0.002655	4.50	64.96	24.40	0.49
Newburgh	726.58*	50-Year	292.50	392.45	398.06		398.37	0.002682	4.47	65.37	24.98	0.49
Newburgh	639.88*	50-Year	292.50	392.18	397.82		398.13	0.002748	4.46	65.52	25.49	0.49
Newburgh	553.17	50-Year	292.50	391.92	397.57		397.89	0.002900	4.50	65.01	25.89	0.50
Newburgh	460.98*	50-Year	292.50	391.73	397.30		397.62	0.002847	4.58	63.92	24.62	0.50
Newburgh	368.78*	50-Year	292.50	391.54	397.04		397.36	0.002731	4.59	63.69	23.82	0.50
Newburgh	276.59*	50-Year	292.50	391.35	396.79		397.12	0.002597	4.58	63.93	23.31	0.49
Newburgh	184.39*	50-Year	292.50	391.16	396.59		396.89	0.002267	4.38	69.13	44.89	0.46
Newburgh	92.19*	50-Year	292.50	390.97	396.38	394.83	396.68	0.002203	4.40	66.51	22.98	0.46
Newburgh	0	50-Year	292.50	390.78	396.20	394.56	396.48	0.002001	4.27	68.44	23.00	0.44

APPENDIX C

Table 11: Preliminary Project Schedule

Task	Due Date
Background review of information from Lochmueller	1/20/23
Review questionnaire results	1/20/23
Review survey data	1/20/23
Review stormwater drainage ordinates – Warrick County, Indiana	1/20/23
Delineate watershed boundary and sub-watersheds	1/25/23
Determine watershed time of concentration	2/3/23
Determine runoff curve number	2/3/23
Process survey data to get cross-sections of drainage channel	2/3/23
Compute peak discharges	2/3/23
Project site visit	2/10/23
Develop hydrologic model for watershed	2/17/23
Use hydrologic model to predict flood magnitudes for various frequency events	2/24/23
Incorporate hydraulic structures into model	3/3/23
Evaluate model performance using discharges previously estimated	3/17/23
Finalize model (existing conditions)	3/24/23
Identify relevant design alternatives	3/31/23
Model post implementation alternatives	4/12/23
Finalize design alternatives	4/19/23

APPENDIX D

Table 12: ABET Outcome #2 - Design Factors Considered

Design Factor	Page number, or reason not applicable
Public Health Safety, and Welfare	Page Number 5
Global	Not Applicable - Page Number 7
Cultural	Not Applicable - Page Number 7
Social	Not Applicable - Page Number 7
Environmental	Page Number 6
Economic	Page Number 6
Ethical & Professional	Page Number 6
Reference for Standards	Page Number 41-42