The Meanderers

Matt Bedner, Matt de Wit

Final Design Report

5/21/2014
Executive Summary

Team 4, The Meanderers, is a senior design team of the Calvin College engineering program. It is comprised of two members, Matt Bedner and Matt de Wit. Matt Bedner is a senior engineering major in the civil & environmental concentration and is originally from Brighton, Michigan. Matt de Wit is senior engineering major in the civil & environmental concentration and is originally from London, Ontario.

The team is working in conjunction with Calvin College Biology Department organization Plaster Creek Stewards to develop a restoration plan for two sites of one of West Michigan’s most polluted creeks, Plaster Creek. According to a document published by civil engineering consulting firm Fishbeck, Thompson, Carr, and Huber, Inc., the two problems with the creek that damage its ability to comply with the Michigan Department of Environmental Quality’s watershed designated uses standards are erosion during storm events and pollution by sediment and *Escherichia coli* bacteria. These problems can partially be attributed to the rerouting of the creek into straight channels due to land development over the past 100 years. During storm events, flows exceed these channels’ carrying capacity, leading to erosion and flooding. The objective of the team’s restoration plan in combating these problems is to re-introduce meanders into two sections of Plaster Creek to lower flow velocities during storm events and to develop a natural vegetation scheme that will help filter out bacteria and sediment.

Five sites were originally chosen as potential restoration sites from a qualitative analysis on Google Earth and from suggestions by Plaster Creek Stewards program coordinator Mike Ryskamp. Of these five sites, two were chosen to be the focus of this project on the bases of maximizing the impact of restoration on Plaster Creek, minimizing land acquisition costs, and minimizing any negative impacts on the surrounding environment. The two sites are located in Gaines Township, Michigan. One is located on the south side of M-6, just west of Hanna Lake Avenue, and the other is located approximately 2000 feet south of 68th Street, just west of Hanna Lake Avenue.

In the design selection process, three design alternatives other than meanders were considered in order to verify that meanders had the best combination of minimizing flooding of the surrounding land, maximizing pollutant removal, and minimizing negative ecological effects. These alternatives were constructed wetlands, storm detention ponds, and concrete-lined channels. After evaluating these alternatives by these criteria, the team decided that meanders were the best solution.

The meander design for each site was completed using the Rosgen design method and calculations taken from the NRCS National Engineering Handbook. Using the methods described by these two resources the team was able to design a number of new stream channel configurations. These designs were evaluated based on their bankfull flow performance, and one design per site was ultimately selected for storm flow testing. The stream design at the M-6 site was matched to the existing channel flow capacity, and the 68th Street site design was selected from a natural bankfull flow rate measured in the existing channel. These new channel designs resulted in reduced flow velocities, as shown in Figures 1 through 4.
Floodplain design was initiated using recommended guidelines from the Rosgen design handbook. Floodplain width was recommended to be 2.2 times the nominal bankfull width, and the depth was recommended to be twice the nominal depth of the stream. Hydraulic modeling was completed to test the storm capacity of both site’s designs. Through this modeling, it was found that the 68th Street floodplain had enough capacity for a 2-year storm, which is greater than the capacity of the current stream channel. Further testing of larger storms also confirmed that the new stream channel design performed as well as or better than the current channel, with regard to flood water depths. The M-6 site design was originally designed to convey a 100-year flood event; however this design proved to be impractical, and thus the design was changed to the guidelines mentioned above. This change resulted in a floodplain that could contain a storm slightly larger than a 50-year frequency storm. Included in the modeling of the floodplain was the storm conveyance culvert located downstream of the site. It was determined that under 100-year storm events the culvert would not exceed capacity, and water would not overtop the road. The reason for increased storm capacity at the M-6 site, as opposed to the 68th Street site, was the surrounding land. At the M-6 site the surrounding land had been zoned for residential use, and the team decided that maximum flood protection would be a priority for this site. At the 68th street site the surrounding land was zoned for agricultural use, so it was decided that it was not as economically viable to increase the size of the floodplain in this area and instead focus on naturalization and stability of the stream and floodplain.
Sediment data was collected at each site to serve as a base line for calibration of the sediment transport equations and to determine the amount of sediment removal seen in the proposed designs. The results of this data were that the M-6 site has significantly more sediment than the 68th Street site, with Total Suspended Solids concentrations ranging from 67 to 88 mg/L at the M-6 site and 0.2 to 2.8 mg/L at the 68th Street site. From this data it was determined that sediment concentrations at the 68th Street site were not dangerously high, and thus the team decided that sediment control would only be a secondary goal for this site. The proposed design of the M-6 site reduces suspended sediment by 98% within its slow flowing pool sections, depositing nearly 1000 cubic feet of sediment per year. This deposited sediment will need to be excavated periodically throughout the year, so a permanent access road has been included in the site design to provide access for heavy equipment (see Figure 5).

![Figure 5 M-6 Site Overview](image)

![Figure 6 68th Street Site Overview](image)

The final cost estimates of the project are expected to be $250,000 for the M-6 site and $350,000 for the 68th Street Site. The M-6 site will also require a yearly maintenance cost of approximately $15,000 for sediment removal. These costs were calculated using Goldenseal construction estimation software.

Looking forward there are more areas of Plaster Creek that would be excellent candidates for stream restoration projects. More data collected on the watershed would also improve the designs of new stream channels and help to determine the stability and ecology of the watershed.
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Project Background

I. Introduction

Team 4, The Meanderers, of the Senior Design course of the Calvin College Engineering Program is working in conjunction with Calvin College Biology Department organization Plaster Creek Stewards to develop a restoration plan for one of the most polluted creeks in West Michigan, Plaster Creek. This restoration plan includes the re-introduction of meanders and a natural vegetation scheme for two sections of the creek that have been converted into straight channels to accommodate for residential and agricultural development. The most prevalent problems with Plaster Creek are flooding during storm events and pollution by sediment and *Escherichia coli* bacteria. Considering these problems, the team developed two primary objectives for its plan:

1. To lower water velocity in Plaster Creek to reduce flooding and erosion
2. To reduce sediment and *E. coli* in Plaster Creek

The re-introduction of meanders will slow down water flow and thereby address the problems of flooding and erosion. Meanders alone will not, however, substantially reduce sediment and *E. coli* in the creek, so these pollutants must be filtrated out of the water. Introducing natural vegetation along the banks of the stream channel will be the means of this filtration process.

II. A Brief History of Plaster Creek Watershed

Plaster Creek Watershed is a watershed in Kent County, Michigan that drains approximately 58 square miles of land (FTCH). The name “Plaster” comes from a description of the pollution in the creek that would enter as a byproduct from gypsum mining, back when it was an industry in West Michigan. The watershed’s headwaters are in Gaines Township, and the main creek, Plaster Creek, flows through the cities of Kentwood and Grand Rapids and into the Grand River. A map outlining the boundaries of Plaster Creek Watershed is shown in Figure 1.1. The bedrock of the watershed is comprised primarily of shale, sandstone, limestone, and gypsum and was formed from sediments deposited around 345 to 370 million years ago in seas (FTCH). Around 10,000 years ago, glaciers covering the state melted and deposited the next layer of soil, which primarily falls into the Alfalfa soil order (FTCH). Between 1840 and 1870, the lumbering industry became prominent in West Michigan, which at the time was almost entirely covered with hardwood forest (FTCH). Deforested land was converted to farmland, and farming also became prominent in the region (FTCH). In many areas, Plaster Creek needed to be rerouted to accommodate for this change in land use, and in some cases, this rerouting involved converting the creek into a straight channel. Today, the fate of Plaster Creek is threatened with increased urbanization within its watershed – increased urbanization leads to a larger area of impervious surface contributing runoff to the watershed to the point of exceeding its natural capacity.
Figure 1.1 Boundaries of Plaster Creek Watershed (FTCH).
III. Plaster Creek Stewards

Plaster Creek Stewards are, as stated on their website, a collaboration of Calvin College faculty, staff, and students working with local schools, churches, and community partners to restore the health and beauty of Plaster Creek Watershed. Their work can be split into three categories: education, research, and restoration projects. They educate the community with presentations, events, workshops, and opportunities to participate in restoration projects to promote awareness about the needs of the watershed and actions people can take to restore it. Their research is mostly devoted to measuring the water quality of Plaster Creek over time. This data is collected in multiple labs for a Biology Department course entitled *Research Design and Methodology*. The restoration projects involve planting natural vegetation, including trees and wetland plants, along the creek banks to help stabilize the soil, prevent erosion, and filter out runoff entering the creek. The plants planted in the banks of Plaster Creek are grown from seedlings in Calvin College’s greenhouse, located in DeVries Hall. The team hopes that its restoration plan will at least give Plaster Creek Stewards ideas for potential restoration projects.

IV. Calvin College and Senior Design

Calvin College is a four-year liberal arts college in Grand Rapids, Michigan. It is affiliated with the Christian Reformed Church in North America. Students graduating from the Engineering Department at Calvin College earn an ABET-accredited Bachelor of Science in Engineering, with one of four concentrations. These concentrations are mechanical, civil & environmental, electrical & computer, and chemical. Both members of The Meanderers are in the civil & environmental concentration.

Engineering 339-340, *Senior Design Project*, is the capstone course of the Calvin College Engineering major. All senior engineering students enroll in it, and four professors (one from each concentration) teach it. Along with completing a design project, students learn a variety of principles for professional practice, including resume building, management, communication, ethics, and cost estimation.

V. The Team

Matthew Bedner is a senior Calvin College student originally from Brighton, Michigan. He will be graduating in May 2014 with a Bachelor of Science in Engineering with a civil/environmental concentration. Over the past two years, he has worked as an intern, performing such tasks as construction inspections and GIS data updates, for the City of Ann Arbor, Lambert Associates, and Vriesman & Korhorn Civil Engineers. He is also a member of the Calvin College Wind Ensemble as a trumpet player. After graduation Matt will begin employment with Fleis and VandenBrink Engineering in Grand Rapids.

Matthew de Wit was born and raised in London, Ontario. Being a Canadian native he enjoys winter sports such as snowboarding and hockey, but his favorite sport is American Football. For the last two summers Matthew has interned at the London office of Delcan Corporation where he has worked on a number of
drainage related projects including storm sewer design and modeling. The skills and knowledge gained through this internship position will make him an asset to Team 4. Matthew will begin employment with Parsons Corporation after graduation in his home town.

VI. Design Norms

Through the course of this project the team selected a number of design norms that the team decided fit within the goals of the project and that could be used to shape and mold the design to fit within the team’s Christian values. These design norms allowed the team to focus their efforts and look beyond their immediate projects to discern the impact of their design within a larger world.

**Stewardship.** Stewardship is using resources effectively and for the glory of God. This norm will be perhaps the most prevalent norm we will see in our project – our project is focused on designing a stream restoration plan that seeks to reduce pollutants in a creek that is polluted because of the ineffective use of land. If our project solution were to be implemented, it would not only reduce pollutants in the creek but also return the creek to a more natural state.

**Justice.** Justice is respecting the rights of everybody and considering everyone who is and who may become impacted by a design. If our design were to be implemented, it would impact a large number of people. Such people include the members of Plaster Creek Stewards, the farmers who own the property one of our selected sites is on, construction workers performing manual labor to build our design, and anyone who comes in contact with the creek. We will design our restoration plan so that, if implemented, it will help reduce pollutants in the creek, not span over an excessive amount of land, and not be overly complex to build.

**Integrity.** Integrity is the completeness, functionality, ease of use, and ability to promote human values and relationships of a design. One major concern with our design is that the route we choose for the creek is not going remain as it is forever – stream rerouting is a natural process. We must take this complication into consideration when we design the channel, bank, and vegetation scheme. Also, because a large number of people could be impacted by the design if it were to be implemented, it could provide a means for new relationships to form. With the help of the educational programs of Plaster Creek Stewards, farmers, inner-city residents of Grand Rapids, and members of Plaster Creek Stewards could join forces in restoring the watershed.

**Humility.** Humility is recognizing that designs will not always be perfect. Stream restoration is not exact science – most of the equations used to design the various parts of the stream are only correlations based on studies. In effect, when any kind of dimension or size is specified, that specification is only a prediction as to how the stream will best behave. In the team’s design, uncertainty must be incorporated.
VII. WBS and Roles

Matthew Bedner has worked on research for methods in stream rerouting, bank design, channel design, vegetation scheme, and anything else that is relevant. He has also worked on the design of the M-6 site and performed the research for bank stabilization methods, groundwater analysis, stone sizing, and scour calculations. Matthew de Wit has worked on creek flow modeling in Hydrologic Engineering Center’s River Analysis System (HEC-RAS). He has also worked on the design of the 68th St site and drafting on AutoCAD Civil3D. Both team members have also worked together on gathering data from the two sites, making decisions on sites and designs, cost estimating, and writing the reports. The team’s most recent Work Breakdown Structure is shown in Figure 1.2.
Figure 1.2. Team 4’s Current Work Breakdown Structure.
VIII. Habitat Assessment

In 2001, the Michigan Department of Environmental Quality (MDEQ) conducted an assessment of biological community and pollution at four sites on Plaster Creek. A map showing the locations of these sites is in Appendix II. Neither of these assessments involved taking data; rather, they were qualitative assessments that involved simply walking along the banks of the creek. The data sheet for the biological assessment contained a list of a variety of macroinvertebrate species one may find at a creek, and the procedure was simply to count the number of organisms of each species listed found at each site. The data sheet for the pollution assessment contained a list of criteria and specified a scale (for example 1-20, with 1 being the worst and 20 being the best) to evaluate each criterion by, and the procedure was simply to score each criterion by visual observation. Examples of these data sheets are shown in the Appendix II. When the numbers for each data sheet were added, the biological and environmental conditions of each site were rated “excellent,” “good,” “acceptable,” “fair,” or “poor.” In the biological assessment, two sites were rated “acceptable,” and the other two were rated “poor.” In the pollution assessment, two sites were rated “good,” and the other two were rated “fair.” The report explaining the results of these assessments attributed the decline in habitat quality to an increase in impervious surfaces, which contribute runoff and cause flashy flows during storm events, causing increases in erosion and sedimentation (FTCH). Other attributes included log jams and cattle access to the creek near 68th Street (FTCH).

IX. E. coli Assessment

The amount of E. coli in a body of water is used as a parameter to determine whether it is contaminated with fecal matter. Specifically, the MDEQ requires that a body of water not contain greater than, as a 30-day geometric mean, 130 E. coli cells per 100 milliliters of water. In 2001, the MDEQ conducted an assessment for E. coli contamination for ten sites on Plaster Creek and found that the 30-day geometric mean of E. coli cells per 100 milliliters of water was greater than 130. A map showing the locations of these sites and the results of this assessment are shown in Appendix II. The MDEQ also found that E. coli level correlates with precipitation, suggesting that the E. coli cells enter the creek in runoff, probably from farm fields (FTCH).

X. Subwatershed Evaluations

During the development of the Lower Grand River Watershed Management Plan, the twelve subwatersheds of Plaster Creek were evaluated to determine the parts of the watershed that are in the worst condition. A map outlining the subwatersheds of Plaster Creek Watershed is shown in Figure 1.3.
Figure 1.3. Subwatersheds of Plaster Creek Watershed (FTCH).
The criteria used in this assessment were septic system usage, urban and agricultural land area, total maximum daily load (TMDL) reach, *E. coli* concentration, and sediment, phosphorous, and nitrogen load. Septic system usage was determined by calculating the percent of a subwatershed’s area that uses a septic system and not a sanitary sewer. Each subwatershed received a rank of 1-4 based on this percentage, where a 1 corresponds to a small percentage and a 4 corresponds to a high percentage. Urban and agricultural land area was determined by calculating the percent of a subwatershed’s area that is, essentially, not natural. Each subwatershed received a rank of 1-4 in the same manner. TMDL reach is the length of creek where pollutants are suspected of entering, which is determined by GIS analysis or from information in MDEQ’s 2002 Biota and *E. coli* TMDL reports for Plaster Creek. This length was divided by the total length of creek in a particular subwatershed to obtain a percentage, and each subwatershed received a rank of 1-4 in the same manner they were assigned for septic system usage and urban and agricultural land area. *E. coli* concentration was determined by calculating the average *E. coli* concentration (cells/100 mL of water) at a monitoring location in each subwatershed. Each subwatershed received a ranking of 1-12, where a 1 corresponds to the subwatershed with the smallest *E. coli* concentration and a 12 corresponds to the subwatershed with the largest *E. coli* concentration. Sediment, phosphorous, and nitrogen load and the subwatershed rankings for this criterion were determined in the same manner.

The ranks for each subwatershed were then summed so that the subwatershed with the highest ranking was deemed to be in the worst condition. The results of this analysis are shown in Table 1.1.

Table 1.1. Plaster Creek Subwatershed Evaluations (FTCH).
XI. Designated Uses of Plaster Creek Watershed

The State of Michigan has developed water quality standards (WQS) that protect all surface waters in Michigan for eight uses. These uses are listed and defined in Table 1.2.

Table 1.2. State of Michigan Designated Uses for Surface Water (FTCH).

<table>
<thead>
<tr>
<th>Designated Use</th>
<th>General Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agriculture</td>
<td>Livestock watering, irrigation, and crop spraying</td>
</tr>
<tr>
<td>Navigation</td>
<td>Navigation of inland waters</td>
</tr>
<tr>
<td>Warmwater or coldwater fishery</td>
<td>Supports warm water or cold water species</td>
</tr>
<tr>
<td>Other Indigenous aquatic life and wildlife</td>
<td>Supports other indigenous animals, plants, and macroinvertebrates</td>
</tr>
<tr>
<td>Partial body contact recreation</td>
<td>Supports boating, wading, and fishing activities</td>
</tr>
<tr>
<td>Total body contact recreation</td>
<td>Supports swimming activities between May 1 to October 31</td>
</tr>
<tr>
<td>Public water supply</td>
<td>Surface waters meet human cancer and non-cancer values for drinking water</td>
</tr>
<tr>
<td>Industrial water supply</td>
<td>Water utilized in industrial or commercial applications</td>
</tr>
</tbody>
</table>

Each use has a specific set of standards (more specific than those listed in Table 1.2) that must be met. One example of such a set is that for total body contact, the 30-day geometric mean of *E. coli* concentration in a body of water must be less than 130 cells per 100 milliliters of water. Likewise, for partial body contact, the 30-day geometric mean of *E. coli* concentration must be less than 1000 cells per 100 milliliters of water. The effectiveness by which the WQS are met for each use is expressed as “met,” “impaired,” or “threatened.” “Met” means that all available data indicate that all applicable WQS are consistently being met. “Threatened” means that all available data indicate that all applicable WQS are being met, but projected land use changes and/or declining water quality trends suggest that the WQS will not be met in the future. “Impaired” means that physical and analytical data indicate that not all applicable WQS are consistently being met. The effectiveness by which Plaster Creek Watershed meets the WQS for the eight uses is summarized in Table 1.3.

Table 1.3. Statuses of the Eight Designated Uses for Plaster Creek Watershed (FTCH).

<table>
<thead>
<tr>
<th>Level of Priority</th>
<th>Designated Use</th>
<th>Status of Designated Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Other Indigenous aquatic life and wildlife</td>
<td>Impaired</td>
</tr>
<tr>
<td>2</td>
<td>Warmwater fishery</td>
<td>Impaired</td>
</tr>
<tr>
<td>3</td>
<td>Partial body contact recreation</td>
<td>Threatened</td>
</tr>
<tr>
<td>4</td>
<td>Total body contact recreation</td>
<td>Impaired</td>
</tr>
<tr>
<td>5</td>
<td>Agriculture</td>
<td>Met</td>
</tr>
<tr>
<td>6</td>
<td>Public water supply</td>
<td>Not a current use</td>
</tr>
<tr>
<td>7</td>
<td>Industrial water supply</td>
<td>Not a current use</td>
</tr>
<tr>
<td>8</td>
<td>Navigation</td>
<td>Not a current use</td>
</tr>
</tbody>
</table>
The Plaster Creek Steering Committee, a board comprised of city officials, engineers, and professors in the Grand Rapids area with expertise in stream restoration, prioritized the uses as shown in Table 1.3 based on their perceived values and restoration feasibilities. The committee also consulted past and current studies, input from watershed stakeholders, and field observations to identify pollutants adversely impacting the watershed. They then prioritized which ones should be dealt with by the priority level of the designated uses they impact. A summary of their findings is shown in Table 1.4.

Table 1.4. Impacted Designated Uses, Prioritized Pollutants, Prioritized Pollutant Sources, and Pollutant Causes as Determined by the Steering Committee (FTCH).

<table>
<thead>
<tr>
<th>Impacted Designated Uses</th>
<th>Prioritized Pollutants</th>
<th>Prioritized Pollutant Sources</th>
<th>Pollutant Causes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Warm water fishery and other indigenous aquatic life and wildlife</td>
<td>1. Sediment (k)</td>
<td>1. Streambank erosion (k)</td>
<td>Flashy flows (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Storm water outfalls and tile outlets (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Livestock access (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Road/stream crossings (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Log jams (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Off Road Vehicle use (k)</td>
</tr>
<tr>
<td></td>
<td>2. Urban runoff (k)</td>
<td>2. Urban runoff (k)</td>
<td>Fill and guilty erosion (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Agricultural runoff (k)</td>
<td>3. Agricultural runoff (k)</td>
<td>Improper erosion and sediment control measures (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. Construction sites (k)</td>
<td>4. Construction sites (k)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total body contact recreation and partial body contact recreation</td>
<td>2. E. coli (k)</td>
<td>1. Animal waste (k)</td>
<td>Livestock access (k)</td>
</tr>
<tr>
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<td></td>
<td>Manure spreading (s)</td>
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<td>Feedlot runoff (s)</td>
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<td>Wildlife (s)</td>
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<td>Pet waste (s)</td>
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<td></td>
<td>2. Septic systems (s)</td>
<td>2. Septic systems (s)</td>
<td>Improper septic system maintenance (s)</td>
</tr>
<tr>
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<td></td>
<td>3. Sanitary sewer connections (s)</td>
<td>3. Sanitary sewer connections (s)</td>
<td>Faulty connections (s)</td>
</tr>
<tr>
<td>Warm water fishery and other indigenous aquatic life and wildlife</td>
<td>3. Nutrients (k)</td>
<td>1. Lawn inputs (s)</td>
<td>Improper fertilizer management and yard waste disposal (s)</td>
</tr>
<tr>
<td></td>
<td></td>
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<tr>
<td></td>
<td>2. Animal waste (k)</td>
<td>2. Animal waste (k)</td>
<td>Livestock access (k)</td>
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<td></td>
<td></td>
<td>Manure spreading (s)</td>
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<td></td>
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<td>Wildlife (s)</td>
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<td>Pet waste (s)</td>
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<td></td>
<td>3. Septic systems (s)</td>
<td>3. Septic systems (s)</td>
<td>Improper septic system maintenance (s)</td>
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<tr>
<td></td>
<td>4. Sanitary sewer connections (s)</td>
<td>4. Sanitary sewer connections (s)</td>
<td>Faulty connections (s)</td>
</tr>
<tr>
<td>Warm water fishery and other indigenous aquatic life and wildlife</td>
<td>4. Thermal pollution (s)</td>
<td>1. Urban runoff (k)</td>
<td>Impervious surfaces (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Lack of riparian vegetation (k)</td>
<td>2. Lack of riparian vegetation (k)</td>
<td>Removal of riparian vegetation (k)</td>
</tr>
<tr>
<td>Warm water fishery and other indigenous aquatic life and wildlife</td>
<td>5. Toxic substances (s)</td>
<td>1. Urban runoff (k)</td>
<td>Untreated urban runoff (k)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Excessive application of road salt (s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2. Agricultural runoff (s)</td>
<td>2. Agricultural runoff (s)</td>
<td>Improper application of pesticides (s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3. Storm sewer (s)</td>
<td>3. Storm sewer (s)</td>
<td>Illicit dumping (s)</td>
</tr>
</tbody>
</table>

(k) = known; (s) = suspected
XII. Summary

The team can conclude from the MDEQ’s studies on Plaster Creek and the Steering Committee’s pollutant prioritization specifications that if implemented, the team’s restoration plan will address the primary concerns of Plaster Creek Watershed. As Table 1.4 shows, the top two prioritized pollutants are sediment and E. coli, and the second of our two objectives is to reduce these very pollutants. Table 1.4 also shows that the top causes for these pollutants are associated with flashy flows, contaminated runoff, and erosion. Together, the re-introduction of meanders and implementation of a natural vegetation scheme will alleviate these causes.
I. Introduction

The team explored a number of alternative sites for naturalization and construction of meanders through the course of this project. Each alternative site was selected through a qualitative process before a more detailed quantitative analysis was completed. This qualitative process involved scouring aerial photography data from Google Maps and researching property tax data from Kent County. The sites were chosen under two main criteria:

1. The site in question must have an obviously straightened section of stream
2. The site must appear to be on public land or require little property acquisition costs

Under these criteria there were five sites that were considered as ideal locations for design and implementation of meanders and naturalization. These site locations can be seen in Figure 2.1 and are as follows:

1. Little Plaster Creek at Forest Meadows Park
2. Plaster Creek at M-6
3. Plaster Creek Tributary at East Paris, near Steelcase pyramid
4. Plaster Creek south of 68th street near Terra Cotta Drive SE
5. Plaster Creek between 76th street and 84th street

While all but the first site will require some property acquisition these areas are not expected to be more than 5% of total construction cost, and land ownership deals may be reached if property owners are well informed of the benefits of this project.
Figure 2.1 Proposed Site Locations
II. Forest Meadows

This site is part of the Little Plaster Creek watershed and is located along side Forest Meadows Park just upstream of I-96. Figure 2.2 shows this site’s location in detail. Preliminary flow analysis shows that average low flows experienced by this creek section are 2.8 cfs with expected storm flows ranging from 190 cfs to 550 cfs for 10-year to 100-year storms respectively. These flows are lower than expected given that the size of the drainage area is 6.1 mi² at this point and that there is a large amount of urban development surrounding the creek. This is likely due to a number of retention and drainage ponds located in the watershed of this creek. The low flow in this area decreases the attractive nature of this site as lower flows will decrease the positive impact that naturalizing this particular portion of the creek will have.

The Forest Meadows site also has the lowest measured levels of *E. coli* bacteria of any of the proposed sites and the lowest expected sediment levels. These criteria were included as they help quantify the potential environmental benefits of naturalization and construction of meanders in that a greater impact through proper design is deemed possible if sediment and *E. coli* levels are high. Likewise, low levels of both sediment and *E. coli* at a site are deemed to lower the potential impact that can be made through naturalization and lower that site’s ranking in the decision process.

Despite the low flow rates, *E. coli* levels, and sediment levels, Forest Meadows Park remains a very attractive location for naturalization because it is located on public property. The greatest benefit arising from such a location is that given support from the City of Grand Rapids, this site could be developed with little to no property acquisition cost. Another benefit of this site location is the possibility of incorporating the design of the meanders into the existing park and converting the stream into a public amenity. This could increase public support for the project and add to the features of the Forest Meadows Park. Forest Meadows Park also contains a preexisting temporary access road which could be utilized.
during construction of the meanders and naturalization, depending on the state of the road. This feature has the potential of lowering construction cost and allowing for ease of design.

Although this site is the most probable for construction approval, it has not been chosen as a primary design site due to its isolated nature from the other potential sites and the low environmental benefits. The separated nature of this site requires a unique hydrological model to be constructed, which would increase the length of time spent on modeling to an unacceptable duration. Moreover, low flows experienced at this site would complicate design more than what has been considered acceptable.

### III. Plaster Creek Tributary at Steelcase Pyramid

As shown in Figure 2.3, this potential site is located on the property of the former Steelcase pyramid. This land is currently on the market as Steelcase has vacated their building and placed the land for sale. This could simplify land acquisition if the area required for construction could be acquired as a separate parcel from the current or future owners. However, because of the potentially high land cost of the project area, an agreement to improve the land without purchasing may be a better approach.

![Figure 2.3 Plaster Creek Tributary at Steelcase Pyramid](image)

Preliminary flow analysis of this site shows that the drainage area is 2.96 mi² and that storm flows are expected to be 280 cfs for a 10-year storm and 550 cfs for a 100-year storm. No data was found regarding low flow condition, but given the expected storm flows and the drainage area, low flows can be expected to range from 2 cfs to 5 cfs. These flow conditions are similar in quantity to Little Plaster Creek at Forest Meadows Park, so the design considerations are similar. The low flows are deemed to decrease the impact of meanders on flooding and lower the chance of erosion prevention downstream of this site. These factors decrease the need for naturalization of this section of the Plaster Creek Watershed and make this site less attractive.
This particular tributary does have the highest levels of measured *E. coli*, which, as described in Section II, increases the potential effects for naturalization and the attractiveness of this site. Natural vegetation could be used to lower the concentration of bacteria in this section of the creek by creating an area of very low flow to allow for the water to experience long enough contact times with the chosen vegetation so that removal is at least partially effective. This would increase the difficulty of the design but would have greater measurable effects on the environment of the stream downstream of the project area.

This site also represents an excellent potential site for future design and construction of meanders. The primary reason this site was not selected was due to the unknowns surrounding land acquisition. Design and modeling of this site also faces the same challenges as Little Plaster Creek, including that it is an isolated site and would require a unique drainage model to be constructed and calibrated for flow analysis.

**IV. Plaster Creek Upstream of 76th Street**
This section of Plaster Creek has been straightened extensively through the agricultural developments that border this area. Figure 2.4 shows the expected project area within this section of stream. The land surrounding it is privately owned agricultural land, parceled between four different owners. Land acquisition could be difficult as agreements may need to be reached with multiple owners, or a large portion of land from one owner may need to be purchased prior to construction.

Preliminary flow data at this site was not attainable and was approximated from existing data based on the drainage basin size, which is expected to be less than 3 square miles. This lack of background information creates a large challenge because the hydrological model for this section of Plaster Creek will be substantially less accurate without site specific calibration data. For site selection purposes, flows were assumed to be similar to those of the Plaster Creek tributary at the Steelcase pyramid. Although this estimate lacks a high degree of accuracy, it was deemed acceptable for site selection purposes.

Another major deterrent to the selection of this site as a primary design site is the drainage pond located directly downstream. This drainage pond would limit the impacts of this project as it is expected that this feature would properly handle the increased peak storm flows due to the channel straightening within the project boundary. In addition to lowering peak storm flows, this drainage pond would presumably decrease sediment load in the stream by lowering the velocity of water that passes through it and thereby allow for settling of suspended sediment.

This site is the least attractive location for introduction of meanders of the proposed sites. Even though it should not be ruled out as an alternative site because of its drawbacks, it should only be considered after all other previously mentioned sites for development. The effects of naturalization of this section of Plaster Creek are expected to be the lowest because of the drainage pond and this site’s location far upstream in the watershed.

V. Conclusion

Each of these alternative sites was selected due to its good location and the high impact potential from introduction of meanders and naturalization of the stream channel. Although only two of the five sites were selected for design, the remaining three remain good locations for further study and design.
Site Selection and Primary Sites

I. Introduction

Two sites of an original five possible sites were selected for in-depth hydrological modeling and design work. These sites are both located on the main channel of Plaster Creek. One is located at the crossing of M-6, and the other is located upstream of 68th Street, as shown in Figure 2.1. These two sites were ultimately chosen because they fit best within the team’s project goals and were the most feasible pair of sites of the original five.

II. Decision Matrix

A design matrix, Table 3.1, was used to evaluate the five original sites so that a non-biased decision could be reached. This table is ordered with the most critical criteria on the left and decreasing in importance to the right. The most important two factors are the expected storm flows and the land acquisition costs that will be required should this project be implemented.

The evaluation of expected storm flows was completed with data provided by the Michigan Department of Environmental Quality (MDEQ). This data was rated so that sites with larger expected flow rates received better scores in the category of flow. This criterion was tied in having the largest weight because the amount of flow a site receives was deemed to be the primary indicator as to whether a restoration project would impact the creek – the larger the storm flows, the larger the contribution to the state of the creek.

For the criterion of land acquisition, sites with lower expected land acquisition costs scored higher. This criterion was tied in having the largest weight because it was deemed to be the primary determinant of whether a construction project could be implemented, regardless of the expected impact on Plaster Creek. Two sites were expected to have no acquisition costs. Both of these sites resided on public land where it was assumed an agreement with the city or state board could be reached to allow construction on the sites. The other sites were rated based on the taxable land value and extrapolating the cost of the land size required for the project.

The next criterion considered was the expected amount of sediment load in the stream. This criterion received the second-largest weight because as one of the two target pollutants for this restoration project, sediment load is the greatest indicator, after storm flows, as to whether a restoration project would
impact the creek. Sediment load in the stream acts as a pollutant against sunlight penetration which deters aquatic vegetation growth and aquatic life. High sediment load is also a symptom of erosion and can provide evidence of very high stream velocities. The data was extrapolated from the flow data and the expected stream velocities. Sites with higher flow and narrow channels were expected to have high sediment loads, such as Plaster Creek at M-6. This site has the highest expected flow rate and has a narrow stream channel at the upstream end of the selected site. Hence, it was awarded the highest score in this category.

*E. coli* count was given the third-largest weight because it is the second of the two target pollutants, but it was perceived that reducing *E. coli* would not have as great an impact on the creek as would reducing sediment load. Sediment load has a large impact on the entire ecosystem of the watershed, whereas *E. coli* simply makes the water unsafe for human contact. Even so, *E. coli* is a very dangerous pathogen that is common in Plaster Creek Watershed, with strong data for many of the proposed sites. Reduction of *E. coli* in the creek is expected to be possible through natural vegetation areas in the meanders where lower flows, and thus higher contact times, will be possible. The concentration data was taken from the FTCH report on Plaster Creek Watershed. Sites with larger *E. coli* counts were given larger ratings in the *E. coli* category.

The size of the selected site was an important factor as it determined how much stream had been straightened and was available to be rerouted. This factor was again related to the potential impact that development of meanders would have on the stream but not as much so as reducing storm flows and the two target pollutants. The area of Plaster Creek south of 76th street contains large sections of straightened creek linked together by very sharp bends. This area had the greatest project size and thus received the highest score in this category.

Ease of design was based on the scope of design for each site and the complexity of the hydrological model that would need to be constructed for these sites. This criterion is more related to the feasibility of completing a design by the end of the school year than to the impact the design would have on the creek, so it was not given as large a weight as the criteria related to impact on the creek. Sites that were expected to have better data, or were along the same stream path, were expected to be easier to design for. The selected sites were given the highest two scores in this category because they are linked together, and thus the hydrological calculations from the upstream site could be used for the hydrological calculations for the downstream site.

The criterion of downstream impact was included to identify any possible negative effects of the construction of meanders. This criterion was added to identify characteristics about a site that would add challenges to but not completely eliminate the possibility of implementing meanders at the site, so it was not given a large weight. Sites with less identified potential negative impacts received larger ratings. Ecologically significant areas downstream of the proposed site were identified, such as wetlands sustained through flooding of the creek. Plaster Creek south of M-6 has a wetland retention area directly downstream of the project which could be negatively impacted through lower peak storm flows. This wetland area also decreases the necessity of introducing meanders upstream of it because wetland areas themselves reduce peak flows.
The final criterion of pollution sources was included to help qualify the difficulty of lowering pollutants other than the two target pollutants in the creek. Sites that had many different types of pollution sources, such as agricultural, residential trash, and oil and grit from road crossings were given a lower score. This is because these other pollutants may interfere with removal of the target pollutants. The presence of pollutants other than the two target pollutants would pose a challenge but not completely eliminate the possibility of designing meanders at a site, so this criterion was not given a large weight.

### III. Plaster Creek at M-6

As shown in the design matrix the proposed site of Plaster Creek at M-6 was the highest rated site for detailed study and design. Figure 2.1 shows the location of this site within Kent County, and Figure 3.1 shows the Google Earth image of this site. Plaster Creek at M-6 has the highest flows out of all proposed sites with expected storm flows of 651 cfs and 1572 cfs for 10-year and 100-year storms respectively. Base flow has been measured to be 141 cfs or 9% of expected peak design flow. This base flow measurement was recorded on October 31 2013, during a 3-month frequency rainfall event, and is thus slightly greater than the average low flow rate. This base flow rate was used to calibrate a hydrological flow model of the stream and determine a bank full flow rate of 388 cfs. The bank full flow rate correlates approximately with a 1-year frequency storm, and thus any storm greater than this will result in widespread flooding in the surrounding area. This flooding is expected to be controlled though the design of a controlled flood plain that will limit the expanse of flooding surrounding the proposed meanders.
Plaster Creek at M-6 also contains a large sediment load of 76 mg/L. This is due to high upstream velocities experienced through an artificially narrowed section of the creek resulting in excessive erosion. The high velocity of water has eroded the stream bed to a depth of more than 1.5 m during normal flow near the widening of the stream channel. The increase in sediment due to erosion has caused the water to become excessively turbid, greatly lowering the penetration of sunlight and thereby deterring aquatic vegetation growth. In addition to the sediment, the stream channel is heavily clogged with fallen trees and branches. This has caused the channel to maneuver around these obstacles and begin the process of meander formation. The removal of sediment will be the primary focus of design at this site rather than retention of the large flood volumes as this is more practical within the confines of the land available.

Bacterial concentration is also a concern at this site due to a dairy operation directly upstream of this site as well as the large amount of agricultural land that contributes runoff to Plaster Creek Watershed. Reducing the sediment within the stream will help to reduce the bacterial concentrations as this will remove the transport mechanism for the bacteria and allow sunlight to penetrate farther into the water. The direct removal of bacteria is also an important focus at this site as the removal of sediment alone is not expected to lower the concentration of bacteria enough.

IV. Plaster Creek Upstream of 68th Street

The Plaster Creek site at 68th Street is different in many ways from the site at M-6 and will thus involve a different design focus. Figure 3.2 shows the location of this site and an aerial view of this site (courtesy of Google Earth). Qualitative research into this site reveals that the degree of issues present are minor compared to those at the previous site. Flood flow rates were calculated to be 377 cfs and 924 cfs for 10-year and 100-year storms respectively. The base flow measurements for this selected site were taken on October 17, 2013, at a time after 3 days of slightly above average rainfall. The depth of precipitation during these three days did not exceed 0.3 inches and can be assumed to have had negligible effect on the flow rate within the stream. Through cross-section and velocity measurements, the base flow rate within this section of stream was measured to be 11 cfs. The base flow was again used to calibrate the hydrological model, and the bank full flow rate was determined to be 102 cfs, which correlates with a 1-year frequency storm.

The sediment levels within this portion of Plaster Creek are very low — 1.61 mg/L on average. This low concentration is due to the low velocity of water through the many deeper pools that exist within this particular stream reach. Within these deep pools, the flow rate is approximately 0.4 m³/s on average. These pools are caused by the many obstructions such as fallen trees that exist within this reach. Because of the low flow rates at this site, the obstructions have not yet caused channel rerouting, as they have at the M-6 site, but they have caused the channel bottom to rise from sediment being deposited on the upstream side of the obstructions. To accommodate the rise in the channel bed, the flow velocity
increases proportionally with the decrease in channel cross-section, in some cases to the point of reaching supercritical flow. These sections of the stream have a sediment load almost double the average at up to 2.82 mg/L, showing evidence of some erosion.

Due to the low sediment levels and large amount of channel obstructions, the focus of design at this site will be flood control and intelligent meander design. The goal will be to design a channel depth that will be able to convey at least the same amount of flows without flooding or eroding the banks. Bank stabilization will be an important part of the design at this site because the meanders will need to be designed with longevity so that they do not expand into the agricultural land that borders this site. Flooding from minor storms is expected to be contained within a designed flood plain while large storms are not expected to be contained as their volumes exceed what could be feasibly retained.
Existing Sediment Load Analysis

In October 2013, the team went to its sites and collected 500-mL samples of Plaster Creek water. The team collected three samples from each site – one from the north end, one from the middle, and one from the south end. Each sample was filtered through a Millipore AP1504700 filter, which has 1-µm pore spacings. The filters were then dried in an oven at 220°F and weighed. The mass of sediment found in each sample was divided by the sample volume to get the total suspended solids (TSS) concentration. For the 68th St site samples, all 500 mL were filtered because the team anticipated that they did not have enough sediment to clog the filter, but for the M-6 site samples, only 250 mL were used because the team anticipated that they had enough sediment to clog the filter. The results of the analysis are shown in Table 4.1.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Filter No.</th>
<th>Mass of Filter (mg)</th>
<th>Mass of Filter + Solids (mg)</th>
<th>Mass of Solids (mg)</th>
<th>Volume of Water (mL)</th>
<th>TSS (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>68th-S</td>
<td>1</td>
<td>143.4</td>
<td>144.8</td>
<td>1.4</td>
<td>497</td>
<td>2.82</td>
</tr>
<tr>
<td>68th-M</td>
<td>2</td>
<td>142.6</td>
<td>143.5</td>
<td>0.9</td>
<td>498</td>
<td>1.81</td>
</tr>
<tr>
<td>68th-N</td>
<td>3</td>
<td>142.6</td>
<td>142.7</td>
<td>0.1</td>
<td>502</td>
<td>0.20</td>
</tr>
<tr>
<td>M-6-N</td>
<td>4</td>
<td>140.8</td>
<td>157.5</td>
<td>16.7</td>
<td>250</td>
<td>66.80</td>
</tr>
<tr>
<td>M-6-M</td>
<td>5</td>
<td>141.2</td>
<td>159.8</td>
<td>18.6</td>
<td>248</td>
<td>75.00</td>
</tr>
<tr>
<td>M-6-S</td>
<td>6</td>
<td>140.8</td>
<td>162.8</td>
<td>22.0</td>
<td>250</td>
<td>88.00</td>
</tr>
</tbody>
</table>

As Table 4.1 shows, the M-6 site receives a TSS load approximately 40 times greater than the TSS load of the 68th St site. The team hypothesizes that this larger TSS load can be attributed to the higher water velocity, and hence more stream bank erosion, at the M-6 site. However, the larger TSS load can also be attributed to the team having taken samples during a 3-month frequency rain event, as opposed to no rain when the team took the samples at the 68th St site. This data has been fit to a sediment transport model that has allowed for calculations of expected sediment load within the preliminary design stream.
I. Introduction

Four high-level design alternatives were explored during the preliminary design phase of this project. A design matrix was constructed to assist with the selection of the primary design approach and provide a quantitative basis for the team’s decision. This selection process was completed to narrow the team’s focus at the planning stage of the project and was not intended to provide the ultimate design solution for either of the chosen project sites. The four design alternatives are as follows:

1. Constructed Wetland
2. Storm Water Detention Pond
3. Concrete Lined Channel
4. Constructed Meanders

Each of these alternatives has the potential to address the team’s goal of providing a stable solution to slowing storm water flows and providing a stable channel route during times of low flow.

II. Constructed Wetland

Constructed wetlands have been used in storm water treatment applications in many locations. Their effects on both low flow treatment and storage capacity are somewhat limited due to low maximum and minimum water depths. These structures are wetlands that contain specifically chosen vegetation to target specific pollutants for removal. They are typically designed so that a constant water level is maintained during the active months of the year. They are also designed so that during storm events, a
portion of the runoff can be retained within the wetland for treatment and storage. Figure 5.1 shows a typical single-tier wetland cross section. Constructed wetlands have been primarily designed to remove bacterial and pathogenic pollutants in water, and have been proven to be very effective in this application. A constructed wetland, if designed properly, will have little problems with erosion and will reduce the water velocities exiting the system, thus ensuring positive effects on erosion downstream of the site. Despite the high pollution removal expectations, constructed wetlands have not been selected as the primary design method. This is due to the following drawbacks: low effects on peak flow reductions and water detention volumes, difficulty keeping the wetland active during winter months, and large land requirements.

Storm water retention and the lowering of peak flows has been central to this project’s design objective, and there is evidence that constructed wetlands have difficulty handling large flow volumes and high peak flow rates. Although constructed wetlands have been shown to reduce peak flows and offer storage potential for rainfall runoff, the land required for a constructed wetland that could serve as both a practical treatment area and a storm detention area is not available to the team in either site. Table 5.1 shows the expected storm volumes and acreages required for a constructed wetland based on a number of different design depths.

### Table 5.1 Wetland Acreage requirements

<table>
<thead>
<tr>
<th>Depth</th>
<th>M-6 Site</th>
<th>68th Street Site</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum Wetland Acreage</td>
<td>Minimum Wetland Acreage</td>
</tr>
<tr>
<td>10 Year</td>
<td>1 ft</td>
<td>737</td>
</tr>
<tr>
<td></td>
<td>2 ft</td>
<td>369</td>
</tr>
<tr>
<td></td>
<td>3 ft</td>
<td>246</td>
</tr>
<tr>
<td>25 Year</td>
<td>1 ft</td>
<td>1174</td>
</tr>
<tr>
<td></td>
<td>2 ft</td>
<td>587</td>
</tr>
<tr>
<td></td>
<td>3 ft</td>
<td>391</td>
</tr>
<tr>
<td>100 Year</td>
<td>1 ft</td>
<td>2035</td>
</tr>
<tr>
<td></td>
<td>2 ft</td>
<td>1018</td>
</tr>
<tr>
<td></td>
<td>3 ft</td>
<td>678</td>
</tr>
</tbody>
</table>

The other major drawback to a constructed wetland is a direct result of the climate and effects that it will have on the vegetation in the wetland. It is very unlikely that it will be possible to keep the vegetation from entering a dormant state during the summer months. Thus during this time, the wetland will have little to no pollution control abilities, and during the first spring runoff the wetland may not have rejuvenated to an acceptable level to handle pollutant removal.

### III. Storm Water Detention Pond – Dry Pond

Storm water detention ponds have become a common method of reducing and controlling storm runoff. As shown in Figure 4.2, detention ponds are earthen structures designed to store excess storm runoff and release it in a controlled manner through an outlet structure. Their construction and design is relatively simple, and the expected cost is the lowest of the design alternatives. A storm detention pond is also expected to have the greatest effect on flood control within the confined site areas. This design alternative would also require a large amount of land area. A detention pond also has a number of
drawbacks, namely decreased pollution removal capabilities, and stagnant water issues after large storms.

IV. Concrete Channel
A concrete channel would allow for the stream to keep its current shape and provide a structure that would ensure little to no erosion of the stream banks. This option would also include design of a flood plain where it is possible to allow some control of the peak storm flows. A concrete channel would be the simplest design alternative, but not necessarily the least expensive depending on the soil conditions at each site. This design also has the greatest number of drawbacks of the proposed design alternatives. A concrete channel would likely increase the velocity of water and would not allow for any pollution control. It may possible to reduce water velocity and thereby reduce sediment in the stream through the use of weirs or baffles. During storm flows these measures may have negligible effect on reducing the flow velocity, and thus increased erosion downstream of the concrete channel would likely occur. A concrete channel also has the potential drawback of increasing the water temperature during low flow periods, especially if flow control structures such as weirs are used to reduce the water velocity. Concrete heats easily in the sunlight, and lower water velocities in the channel would increase the water’s contact time with the concrete. Stagnant water also heats more easily than flowing water.

V. Constructed Meanders
Meanders offer the most natural solution to reducing erosion, flooding, and pollution levels. Meanders form naturally in all unconfined streams and rivers over time as the outside banks in bends erode and sediment is deposited along the inner edge of the stream as shown in Figure 5.3. By introducing meanders into the selected sites and rerouting the stream through them instead of on the current straight sections, the flow velocity would be reduced. This would decrease the erosion downstream of the chosen sites and would control where future erosion and meander formation would occur within the selected

Figure 5.2 Detention Basin Detail Source: NJDEP (2004)
sites. Due to the lowered flow velocities, pollution removal would be possible through vegetated banks. This would have the added benefit of stabilizing the banks and controlling erosion on site. Peak flows would be controlled with a designed flood plain to reduce the flooding of surrounding land. Even though a flood plain would not be as effective as some of the other design alternatives, it would have less environmental impact. The drawbacks involved with this alternative are reduced flooding control and unknown effects on the sediment levels within the stream; although they are expected to be reduced somewhat due to the reduced flow velocity. This design alternative is also not entirely permanent due to the nature of meanders, which grow and reshape over time. This phenomenon is not expected to be detrimental, however, because it is expected to be contained within the site’s area.

VI. Design Selection and Conclusions

Table 5.2 shows the decision matrix that was used to assist in selecting the primary design method for either site. Meanders was the clear choice through this selection process as it provides the largest amount of benefits and meets the team’s goals. Although implementing meanders will be the focus of the team’s design work, combinations of the other design alternatives with meanders will be explored on a site-by-site basis.
Computer Modeling

I. Introduction

Hydrologic Engineering Center’s River Analysis System (HEC-RAS) is an excellent tool for hydraulic engineering. In this project it was used to calculate the flows experienced in the existing and proposed stream channels and to determine the depth of flow and the sizing of the flood plain to properly control storm water flows ranging up to a 100-year event. This model was also used to determine the velocity of flow within the channel so that sediment load could be properly calculated, allowing for proper sizing of the meanders.

II. Modeling Program Alternatives

There are many programs available for modeling hydrological systems, including Hydrologic Engineering Center’s River Analysis System (HEC-RAS) and Autodesk Storm and Sanitary Analysis (SSA). Each of these programs is capable of open channel conduit modeling with custom cross-section. Both programs were also available for use by the team for no cost, so cost of licensing was not an issue. The differences between these models exist in the outputs provided from the computation engine, the accessibility of the data and input files, and the ease of use for the team.

 Ease of use for the team was the first considered criteria. The team has extensive experience with Autodesk SSA, thus speeding the creation of the model as very little time will need to be spent learning the program. HEC-RAS is an unfamiliar program to the team members and will require that time be spent learning this program so that a high quality model can be produced. Due to wait periods for required data it has been determined that time will be available for the team members to learn this unfamiliar program. External accessibility was an important issue when choosing between the two programs to use. While both programs were available to the team, it was realized that Autodesk SSA may not be available to every reader of this report and project. For those who did not have access to this software, the cost of acquiring a license to this is generally prohibitively high as it is an extension module for Autodesk Civil 3D 2012 and newer. Alternatively HEC-RAS is freely available to anyone in the United States and any country with access to US Army Corps of Engineers website. For those with access, this program can be downloaded and installed within minutes.

 The most important differences between these two programs lies in their methods of calculation as well as the output information generated. Autodesk SSA is based off the EPA-SWMM calculation engine, and thus is similar in both input and output structure. It is capable of producing runoff hydrographs and analyzing the resulting flows through channel and pipe networks. The flows from this model are based solely on average channel velocity and cross-sectional area, and the program lacks a velocity distribution calculation engine and output. These drawbacks lower the accuracy of the model when calculating flows over large flood plains. It is also limited to a uniform channel shape between network nodes, unlike HEC-RAS, which creates a stream channel from multiple cross-sections between nodes. This allows for a more realistic stream cross-section, as the program interpolates the cross-sections between the entered
shapes. HEC-RAS also includes outputs of velocity distribution, three-dimensional flow path view, and flood plain modeling. These outputs will become important for designing the shape and size of the new stream section. One drawback to HEC-RAS is the lack of an internal way to calculate sub basin runoff. Catchment runoff must be calculated through an external program such as Hydrologic Engineering Center’s Hydraulic Modeling System (HEC-HMS). This drawback increases the time required for modeling as a separate computer model will be required to calculate the expected storm runoff at the downstream end of the stream. There are benefits to using a separate runoff modeling program, however. For example, HEC-HMS is a program dedicated to stream catchment modeling and has a quasi-two-dimensional calculation engine which allows for highly accurate calculations of storm runoff.

HEC-RAS has ultimately been chosen because it is deemed superior in modeling rivers and streams to Autodesk SSA under the previously mentioned criteria. HEC-RAS is also deemed superior in output, external accessibility, and time efficiency.

III. Background Data

Before a hydrological model can be constructed, a number of important sources of data must be obtained to determine the feasibility of constructing the model. This background data is not information that will be directly entered into the model; rather, it will support the hydrological model and allow for some inputs to be calculated. The most important piece of data required for the hydrological model was calibration data. This data was provided by the Michigan Department of Environmental Quality in the form of storm flow calculations at different points along the watershed. The flow data is used as the target for preexisting flow calculations in the model to determine the accuracy of both the inputs in the model and the selected calculation method. Once the accuracy of the preexisting conditions have been assured, preliminary designs can be created and tested with confidence.

Another important piece of information required to help support the HEC-RAS model is contour data. High resolution contour data was provided by the Grand Valley Metropolitan Council’s GIS database, known as REGIS, and shows contours elevations at two foot intervals within the project area. This contour data has been used to determine the gradient of the existing stream, the proposed design, and the location of the proposed meanders. Additionally the contours were essential in determining the size of the directly connected upstream drainage area, where flow data from the MDEQ was not possible to obtain, and calculating the flows of these connected drainage areas with HEC-HMS.

HEC-HMS was required to generate these runoff flows because HEC-RAS lacks an internal runoff calculation method. This program uses a quasi-two-dimensional computation method to calculate overland drainage patterns. This means that instead of evaluating based on an assumed land shape, the model is broken down into cells based on highly detailed contour data. This allows for a more realistic calculation process as each cell’s drainage is calculated individually and routed separately. Each of the cells’ expected flows are summed together at the point of interest, allowing for a very realistic output of the stream’s expected flow. These flows are generated in the form of hydrographs that can then be entered directly into the HEC-RAS hydrological model. This allows for easy integration between the two programs, and data can quickly be created or changed in HEC-HMS and entered into HEC-RAS.
IV. Model Inputs and Procedure

There are many data requirements for constructing a hydrological model, most of which are common between different modeling programs. For natural streams custom cross-sections are required to describe the shape of the stream channel. These are measured on site using a stationing line and depth rod. HEC-RAS can be used to interpolate more cross-sections between manually entered ones. This increases the accuracy of the model and limits the number of required measured cross-sections. A Manning’s $n$ value is also required for both the stream channel and the stream banks. This value is selected based on the type of vegetation found within the flood plain and the condition of the stream channel. Finally, the model requires a base flow for calibration purposes. This flow is used to make any minor cross section slope changes and to determine the water surface slope.
I. Design Calculations

The plan-view and profile-view designs of the channels at both sites were based on a procedure of calculations specified by widely-renowned stream restoration expert David Rosgen. This procedure is outlined in chapter 11 of the NRCS National Engineering Handbook, Part 654. Rosgen’s procedure is based on his findings that stable streams, or streams that have little tendency to erode away at the banks or deposit sediment as part of the natural re-routing process, fall into one of eight “types” based on the channel slope, sinuosity (ratio of the channel length to the valley length), width-to-depth ratio (ratio of the bankfull width to the bankfull depth), and entrenchment ratio (ratio of the flood-prone area width to the bankfull width). These dimensions and Rosgen’s stream types are illustrated in Figures 7.1 and 7.2.

Figure 7.1 Channel Cross-Section Dimensions

Figure 7.2 Rosgen Stream Types
The first step is to locate a stream reach with channel dimensions similar to those of the project reach, with the exception that this reach (known as the reference reach) is stable and hence has geometric properties corresponding to one of Rosgen’s stream types. The dimensions of the designed project reach are then calculated with the dimensions obtained from the reference reach substituted for the variables in the equations of Rosgen’s procedure.

Because the team did not learn about obtaining the dimensions of a reference reach until the middle of the project, the team estimated the dimensions of the reference reach by first guessing a bankfull depth and then using the ratios of a C-type channel to determine the other profile-view dimensions, namely the bankfull width and the flood-prone area width. A C-type channel corresponds to a channel slope of less than 2%, a sinuosity of 1.2 or greater, a width-to-depth ratio of 12 or greater, and an entrenchment ratio of 2.2 or greater. The team’s industrial consultant, Tom Bennett, had suggested that the project reach be designed to either a C-type or E-type channel (an E-type channel is similar to a C-type channel with the exceptions of the sinuosity being 1.5 or greater and the width-to-depth ratio being less than 12). For both sites, the team chose to design to a C-type channel because even though the majority of Plaster Creek is an E-type channel, a C-type channel better met the team’s goal of slowing flow velocity and controlling flooding during rain events.

Once the guessed profile-view dimensions were determined, the guessed bankfull width was then substituted into equations from chapter 12 of the NRCS National Engineering Handbook, Part 654 to calculate the reference-reach meander wavelength, meander amplitude, riffle spacing, widths at pools and apexes, pool locations, and maximum pool depth (illustrated in Figure 7.3). These equations are from
studies suggesting that these dimensions are directly proportional to bankfull width. Once the reference-reach dimensions were calculated, they were substituted into the equations of Rosgen’s procedure to determine the project reach meander wavelength, meander amplitude, maximum pool depth, flood-prone area width, and channel slope. The project reach dimensions were then entered into HEC-RAS, and a simulation was run with water flowing at the bankfull flow rate, as supplied by the MDEQ. The purpose of this simulation was to verify that water flowing at the bankfull flow rate would flow as deep as the bankfull depth. If bankfull flow was not as deep as the channel’s bankfull depth, the calculation procedure outlined above was repeated until the channel was just the right size for bankfull flow to flow as deep as bankfull depth. All calculations for the channel design are shown in Appendix VI.

II. Channel Design in HEC-RAS

Preparing the Model
Hydraulic modeling was performed with the Hydrologic Engineering Center’s River Analysis System, (HEC-RAS). This program was used in conjunction with AutoCAD Civil 3D. Site alignments were created in AutoCAD and imported into HEC-RAS with the proper ground surface data. The new stream cross-sections were then added to this data to create the stream channel which was to be analyzed. To increase accuracy and reduce the number of cross-sections to be added, manually-entered cross-sections were interpolated for every 10 feet. This allowed for a highly accurate model of both sites to be created.

Design Process
Each model was evaluated based on a number of different criteria. The most important design goal was to achieve the proper bankfull flow that was used in the design calculations, as discussed previously. At the M-6 site the design calculations provided an excellent foundation for design, and little was altered from the recommended cross-section shape. The 68th Street site, in contrast, required some variation in the typical cross-section shape through the reach. These variations were kept within the recommended ranges and were added to better match a natural system. Each river bend was modified as a whole to keep constant design variation between each bend. As an example narrower riffle sections were paired with wider pools to more effectively dissipate the increased channel velocities. Once proper bankfull flows were achieved, an analysis of the Froude number was performed by running the model at different flow conditions. All points of critical and super critical flow were marked, and the reach profile was modified to reduce the flow velocities at these points. This process also involved slight modification to some of the horizontal cross-section when simple modification of the reach profile was inadequate. Critical flows during low flow conditions at the 68th Street site were not able to be reduced to subcritical flow; however, the flow velocities at these points matched current recorded velocities at riffle sections and are not expected to be an issue.

The final design variable involved the flow rate of the designed flood plain. Each site had different goals with regards to the capacity of the floodplain. At the M-6 site, the floodplain was designed to carry a 50-year storm frequency event. This level of capacity was selected to best match the highway culvert at the downstream end of the site. This culvert was included in the hydraulic model so that the effects of the
new stream channel and floodplain could be properly determined. The added benefit of allowing for such a high degree of flood capacity was increased flood safety for the properties adjacent to the current stream channel. The design of the 68th Street stream reach involved a smaller floodplain designed to convey only the current stream channel bankfull flow. The reason for this design decision was that the current stream channel’s calculated bankfull flow is far greater than its natural bank full flow. It was also decided that a flood plain designed to convey the 100-year storm flow would be both impractical and unnecessary as the current land surrounding the stream is zoned for agricultural use. Once the design goals for the stream channel and flood plain were met, the channel data was imported back into AutoCAD for drafting and construction analysis.
M-6 Site

As mentioned in the sediment load analysis, Plaster Creek carries substantially more sediment at the M-6 site than it does at the 68th St site. As such, reducing sediment concentration is a more prevalent focus at the M-6 site than at the 68th St site. The primary means of accomplishing this objective is creating a channel with a greater cross-sectional area and shallower slope to reduce the flow velocity and thereby allow a larger amount of sediment to settle.

I. Channel Geometry

From the channel design process, the team estimated the meander wavelength for the designed channel at the M-6 site to be 700 feet, which is the entire project reach. With the meander wavelength at 700 feet, only one meander will fit in the project reach, and the stream will be entirely re-routed from its current placement. This re-routing will help solve a few problems with the current stream:

- The land parcel at the upstream end of the site has a path for heavy vehicles along the bank of Plaster Creek that has forced the creek to narrow. As a result, the flow velocity increases and has caused the formation of a scour pool at least 6 feet deep at the downstream end of the land parcel. The high-velocity flow is also causing increased erosion of the banks and increased sediment concentration in the water. Re-routing the creek away from this land parcel will allow the creek to flow freely without immediate human impact, leading to decreased erosion and sediment concentrations.
- In its current channel, Plaster Creek flows along the edge of several land parcels, making them highly prone to flooding. Re-routing the creek away from these land parcels will reduce the risk of flooding for them.
- At the upstream end of the site, Plaster Creek makes a bend in the same direction the upstream-end bend would make, allowing for a smoother transition between the natural channel and the designed channel.

From the channel design process, the team found that a channel top width of 53 feet and depth of 4 feet conveys bankfull flow (388 cfs) just under bankfull depth. In the calculations of chapter 12 of NRCS NEH, Part 654, this width corresponds to a meander amplitude of 176 feet, an apex width of 72 feet, a pool width of 60 feet, and a pool depth of 8 feet. In the design, a pool is placed at the downstream end of each bend, except for the bend leading into the culvert, because the downstream end of a bend is where pools naturally form in streams (NRCS NEH, Part 654, chapter 12). A plan-view drawing of the proposed channel at the M-6 site is shown in Figure 8.3. Detailed plan-view and profile-view drawings of the proposed channel are shown in the contract drawings.
II. Bank Stabilization

One concern with the M-6 site is that the outer banks of the channel will erode away. Even though the team has designed the channel so that this erosion will not occur, the calculations used in the channel design process are not perfect, so the team deemed it necessary to design around the uncertainty in the calculations. In other stream restoration projects, stream bank erosion is prevented by constructing structures known as bank stabilization devices. The team researched into some bank stabilization devices and had to decide between several different options.

**Brush spurs.** Brush spurs are structures of brush that extend from the bank to partway across the stream. They serve two purposes – to divert the region of fastest flow away from the outer stream bank and to promote sediment deposition along the outer bank, which helps structurally strengthen the bank. Their construction mostly involves excavation but also includes installing soil anchors (NRCS NEH 654, TS14I).

**Cribwall.** Cribwalls are hollow structures of logs filled with soil, rocks, and live cuttings (chopped off parts of plants that eventually take root in the surrounding soil). Their purpose is to harden the bank and are constructed by the same means brush spurs are constructed (NRCS NEH 654, TS14I).

**Fascines.** Fascines are long bundles of live cuttings bound together by rope. They eventually take root in the bank and thereby harden it and promote sedimentation (NRCS NEH 654, TS14I).

**Joint plantings.** Joint plantings are live cuttings planted all over the stream’s floodplain area, with the floodplain area covered with riprap. This method is especially useful for floodplain areas with steep slopes, where the soil is unstable (NRCS NEH 654, TS14I).

**Vegetated reinforced soil slope (VRSS).** A VRSS is a system of layers of soil, rooted plants, and geotextiles that form the floodplain area of the stream. It is similar to joint plantings in that it helps stabilize the floodplain area, especially ones with steep slopes (NRCS NEH 654, TS14I).

**Brush wattle fence.** Brush wattle fences are rows of live cuttings planted vertically in the ground with string-like materials woven between the cuttings. The plantings eventually take root in the soil, hardening the banks and promoting sediment deposition (NRCS NEH 654, TS14I).

**Stream barbs.** Stream barbs are long, narrow formations of rock that extend from the bank to partway across the stream. They are similar to brush spurs both in structure and in function but differ in that they are made of rocks rather than brush and do not require soil anchors (NRCS NEH 654, TS14H).

The team decided that stream barbs would be the best bank stabilization device to use for this design. Unlike stream barbs, the other devices constructed at the stream bank are made of materials that decay over time and are at risk of not staying fastened to the soil (NRCS NEH 654, TS14H and TS14I). The method of joint plantings is a potentially good alternative for structural stabilization of the floodplain area, but the team deems that the placement of riprap in the floodplain area would denaturalize it and thereby go...
against the team’s design norm of stewardship. A VRSS is also a potentially good alternative for structural stabilization of the floodplain area, but its design is complicated and would be beyond the scope of this project. Stream barbs are simple to construct, easy to design, and carry the dual benefits of both diverting fast flow away from the outer bank and promoting sediment deposition at the outer bank.

The design process of stream barbs is outlined in NRCS NEH 654, TS14H. It is based on two rules: the barbs are not to span more than one quarter the width of the stream, and their angle to the tangent of the outer bank must be in a certain range depending on the ratio of the radius of curvature to the stream width. For the M-6 site, this angle had to be 30° or less, so the team arbitrarily chose 20°. Using these two rules, the barbs’ placement is determined using a geometric process, and their placement at the M-6 site is shown in contract drawings. The sizing of the rocks forming the barbs has a calculation procedure of its own; it is outlined in NRCS NEH 654, TS14C and TS14H. The team calculated the size of these rocks to be approximately 2 inches in diameter, and the calculations are shown in Appendix I. The depth of the barbs below the ground surface is to be as deep as the maximum scour predicted to occur due to flow over the barbs. This scour depth was determined using a calculation procedure outlined in NRCS NEH 654, TS14B and was calculated to be approximately 10 feet. These calculations are shown in Appendix V. In NRCS NEH 654, TS14H, it is specified that the height of the barbs above the channel bottom should one-third the bankfull depth (1.4 ft) at the outer end and one-half the bankfull depth (2 ft) at the bank end of the barbs. It is also specified here that the width of the barbs should be approximately 6 times the diameter of the rocks comprising them (1 ft). A cross-sectional view of a typical stream barb is shown in contract drawings.

III. Groundwater Considerations

Another concern with the M-6 site is groundwater, especially since the M-6 site is in a wetland. The concern with this issue is that if the current stream channel is lower than the water table surrounding it, moving the stream channel could cause areas of the wetland to dry up due to the change in underground flow. The MDEQ’s water table contour map, shown in Figure 8.1, suggests that to the contrary, the water table is actually lowest where the stream channel is to be moved, so if the stream channel were to be moved, the wetland may actually have an increased ability to replenish itself.
IV. Armoring in the Transition

Results of the team’s HEC-RAS simulations (to be discussed later) suggest that a sharp increase in flow velocity will occur in the transition between the existing channel and the proposed channel. This velocity increase could potentially lead to some serious issues in this section, including erosion, bank destabilization, scouring, and increased sediment concentration in the water. To combat these issues, the team has decided to armor this section with 8-inch riprap. The sizing of this riprap was determined from a calculation procedure found in NRCS NEH 654, TS14B, and the team used it based on the assumption that the maximum flow velocity in this section (which occurs during a 25-year storm, 968 cfs) is approximately 7.2 ft/s. The calculations are shown in Appendix IV.

V. Sediment Removal

Results of the team’s HEC-RAS simulations also suggest that at all flow rates, the flow velocity in the designed channel is lowered by approximately 1 ft/s. As shown in Appendix I, the team predicts that this decrease in flow velocity will lead to approximately 1000 cubic feet of sediment being deposited in the project reach per year. Unless this sediment is removed manually, it will eventually constrict the channel and divert flows elsewhere. To provide a means of removing this sediment manually, the team has routed an access road to the M-6 site for excavation machinery, shown in Figure 8.3 and in contract drawings. The geotechnical and structural components of the access road design were deemed to be beyond the scope of this project.
VI. HEC-RAS Results

Once the channel design for the M-6 site was determined, the team used HEC-RAS to compare the flow conditions of the current channel and the proposed channel. Plan-view and water surface profile diagrams showing the features of the current and proposed channels with stationing are shown in Figures 8.2-8.4.

Figure 8.2. Water surface profile of current M-6 site channel. Flow is 131 cfs.

Figure 8.3. Plan view of proposed M-6 site channel.
One of the comparisons of the HEC-RAS simulations pertained to water depth during different storm frequencies and flow rates. A flow rate of 131 cfs, the normal flow rate of Plaster Creek at the M-6 site, was first simulated to verify that the depth in the proposed channel is not too low. As shown in Figures 8.5-8.8, even though the depth in the proposed channel is at 2 feet, as opposed to 4 feet in the current channel, the Froude number is never greater than 1, signifying that flow always remains subcritical.
Figure 8.5. Water depth at 131 cfs in a typical cross-section of the current channel.

Figure 8.6. Water depth at 131 cfs in a riffle section of the proposed channel.
The second flow rate simulated was 388 cfs, the bankfull flow rate. As shown in Figures 8.9-8.10, water flows up to the bank line in both the current channel and the proposed channel.
Since one of the team's design goals was to reduce flooding, flows corresponding to less frequent storm events were simulated. As shown in Figures 8.11-8.16, the proposed channel at the M-6 site meets this goal. The floodplain of the proposed channel is able to contain up to a 50-year storm, whereas the floodplain of the current channel is only able to contain slightly greater than bankfull flow. Also, during a 100-year storm, the floodplain of the proposed channel overflows by 0.5 feet, whereas the floodplain of the current channel overflows by 2 feet.
Figure 8.11. Water depth in the current channel during a 10-year (651 cfs) storm.

Figure 8.12. Water depth in a riffle section of the proposed channel during a 10-year (651 cfs) storm.
Figure 8.13. Water depth in the current channel during a 50-year (1250 cfs) storm.

Figure 8.14. Water depth in a riffle section of the proposed channel during a 50-year (1250 cfs) storm.
Another comparison of the HEC-RAS simulations pertained to water velocity. Water velocity is an important determinant in scouring, erosion, and sediment transport. Scouring and erosion are important to consider in any stream restoration project because if not properly controlled, the land around the stream could be damaged or destroyed. Since reducing sediment is one of the team’s design goals, an overall reduction in water velocity is ideal because slower water velocities lead to greater sediment deposition, even though this deposition must be controlled so that it does not lead to channel obstructions. Figures 8.17 and 8.18 show the velocities in the current channel and the proposed channel during a 25-year storm (968 cfs). This is the storm event at which flow velocities are highest. The proposed channel is located between stations 0 and 950. The culvert and reach just downstream of it is located between stations -200 and 0. The current channel upstream of the site is located between stations 1200 and 1500, and the transition between it and the proposed channel is located between stations 950 and 1200.
According to the HEC-RAS simulation, the average water velocity in the proposed channel is approximately 2.5 ft/s, which is approximately 1.3 ft/s less than the average water velocity of the current channel. The pools are located at stations 400 and 850, where the water velocity is slowest, and the riffle section is located between stations 500 and 800. One potential issue is that the water velocity rises to 7.2 ft/s as it approaches the transition between the current channel and the proposed channel. This sharp increase is an unfortunate consequence of designing the proposed channel with a much greater width-to-depth ratio.
than that of the current channel and could have been prevented had the team designed to an E-type channel, which has a much smaller width-to-depth ratio, instead of a C-type channel. However, the team decided to keep the design because a C-type channel is better able to control flooding than an E-type channel, and the issues of scouring and erosion in the high-velocity sections can be addressed by armoring the channel with riprap, as discussed previously.

Bank erosion is a concern where the water velocity increases from 2.5 ft/s to 3.0 ft/s in the transition from the channel 53 feet in width to the channel 35 feet in width, located between stations 0 and 200. To address this issue, stream barbs, discussed previously, will be installed in the east bank of this section. Channel obstructions caused by sedimentation are a concern in the riffle section because water flow is shallowest here. However, the riffle section is also the location of the bend apex, where the channel is widest, so the team predicts that sedimentation will mainly occur on the inside (east side) of the bend and that flow will be more concentrated to the outside (west side) of the bend. Over time, flow will slightly scour the channel toward the outside of the bend, and sedimentation will decrease the depth of the channel toward the inside of the bend so that the overall cross-sectional area of the channel will not change.

**VII. Conclusion**

The design of the M-6 site proved to be an interesting challenge. Stewardship was taken into consideration in investigating the effect moving the stream channel would have on the surrounding wetland. Caring was taken into consideration in designing a channel that protects the land parcels adjacent to the site from flooding. Humility and integrity were taken into consideration by incorporating uncertainty into the design. In some cases, these norms conflicted with each other – for example, in order to protect the land parcels from flooding, the channel needs to be moved farther into the wetland.
The design of the new stream channel for Plaster Creek at 68th Street was completed using Rosgen stream morphology correlations. Four initial designs were calculated based on different average bankfull widths and flow rates. These designs were evaluated using a flow rate of 36 cfs. This flow rate was determined to be the natural bank full flow rate as measured from the existing channel riffle cross-sections. Through preliminary hydraulic modeling it was determined that a channel with a nominal width of 14 feet at the riffle sections was the ideal size of channel to convey this flow rate. This width was used to determine the size of the pools and riffles, and the average shape of each can been seen in Figure 9.1 and Figure 9.2.

These sizing of the pools and riffles vary slightly throughout the stream based on velocity requirements and to better match the variation of a natural system. On average the pools have a width of 17 feet and a depth of 1.8 feet. A total of 11 bends, or 5 meanders, were incorporated into the site design, allowing for a large number of pool and riffle systems. This helps to keep a more constant water surface profile and keep a consistent flood plain width.

The final design process was completed in HEC-RAS through an iterative process. The stream profile was manipulated, and after each change the design was tested to check the water velocity through each feature of the design. By this means, the team was able to find the right balance of transition slope and cross-section size for each meander bend. The ultimate goal of this procedure was to assess points of supercritical flow and take steps to reduce the water velocity at these points. There were a number of designs that were successful in reducing the water velocity, the best of which is found in the final design. By increasing the length of the riffle section, the water surface was kept at a more consistent elevation, thereby increasing the depth through the riffles and lowering the velocity. Figure 9.3 shows a plot of the water surface profile at bankfull conditions.
Figure 9.3 Water Surface Profile - Bankfull Conditions
The proposed stream does not significantly reduce the flow velocity of the current bankfull flow. Figures 9.4 and 9.5 show the velocity profile of the stream when under a flow rate of 36 cfs. It has been determined that due to the higher peaks in water velocity through the riffle sections in the proposed design, gravel beds consisting of 0.5-inch gravel will be required to prevent erosion. This facet of the design conforms to natural stream features, as natural riffle beds are made up of gravel beds as smaller sediment is eroded away. Data collected from the current stream channel riffle section confirms that the new stream channel riffle velocity is within stable bounds.

Figure 9.4 Existing Stream - new bankfull Flow (36cfs)
The proposed channel design is significantly more stable than the current design at higher flows due to the floodplain connectivity. Under old bankfull flow conditions of 104 cfs, the flow rate across the entire channel and floodplain is 0.25 cfs lower on average within the proposed channel. Figures 9.6 and 9.7 show the velocities of the old stream and the proposed stream. By increasing the floodplain connectivity, storms that produce more than 36 cfs of flow will not cause a great deal of erosion, as the water will be carried through the floodplain, which will contain a mix of vegetation designed to provide the maximum amount of soil stabilization.
Figure 9.6 Existing Stream Bankfull Flow (131 cfs), Existing Channel

Figure 9.7 Existing Stream Bankfull Flow (131 cfs), Proposed Channel
The reduction in size of the stream channel and the minimal increase in floodplain capacity has resulted in little change in flood control for storms larger than 2-year frequency. While it is possible to increase the size of the floodplain by increasing the height of the banks, such a solution would not be natural and would go against the team’s design norm of stewardship. Due to the current land use surrounding the stream, the team decided that flood control was not as important of a design consideration at this site. The immediate surrounding land is currently zoned for agricultural use, and there is a forested buffer on either side of the new site that will provide a small degree of flood and erosion control.
control. Simulations of storms as large a 100-year frequency storms show (see Figures 9.8 and 9.9) that the flood depths are no worse than those of the current stream.

Vegetation

By the recommendation of the industrial consultant, the team has decided to vegetate the stream banks and floodplain area by spreading a special seed mixture meant specifically for stream restoration projects. This seed mixture would be purchased from a company called Cardno JFNew. The seed mixture is called Emergent Wetland, and a list of species in the mixture, the quantities of species, and cost information can be found in Appendix III. This same information, as well as information about their other seed mixtures, can be found in their catalog at www.cardnojfnew.com. Once the plants from the seed mixture grow to maturity, other plants from the surrounding land will be able to germinate on the stream bank and floodplain area so that the same trees and shrubs as those found in the current stream’s bank and floodplain area will eventually be present in the project reach. This method of vegetation eliminates the complication of worrying about invasive species and other habitat impairments caused by choosing the wrong plants.

One optional means of protecting the seeds after dispersing them throughout the project reach is a technique called crimping. Crimping is a procedure in which straw is planted vertically into the ground in rows parallel to the stream, with 2 to 3 feet between each row. With these rows of straw in place, the soil is temporarily stabilized, and the seeds cannot be carried into the stream by runoff as easily. The procedure is outlined in NRCS NEH 654, TS14I.

Figure 10.1. Crimping. Photo credit: NRCS NEH 654, Figure TS14I-37.
Construction Implementation Plan

I. Construction Phasing

The construction of each site shall begin with the construction of an access road and the erection of work signs at the major connecting roads and entrance to the site. At the M-6 site, the temporary road will require the clearing of marked trees to allow the access road to be constructed. The access road at the M-6 site will be constructed to provide permeated access to the site throughout the year. Once a road has been established, staging areas shall be erected and protected by construction fencing. Equipment and materials required for construction are not to be stored outside the designated staging areas. Silt fencing will be required along the banks of the current stream, as marked in the engineering drawings, to protect from erosion. Once silt fencing has been placed per contractor administrator’s approval, excavation of the site will begin. The channels are to be excavated, and any excess material is to be removed off site immediately. A levy is to remain at both the downstream and upstream end of the proposed channel to block flow from the current stream. The designated construction staging areas will be used to temporarily store fill that is to remain on site. The contractor will compact the slopes and bed of the channel to achieve proper channel shape as specified in the contract drawings. In locations where supplementary bed material is required, specifically within the riffle sections and the rip-rap armored sections, the contractor will refer to the specials included with the contractor drawings that specify the size and depth of said material. Once the channel and floodplain have been excavated, the contractor will spread the designated seed mix within the corridor specified in the contract drawings. Watering of the site will be required if there is unacceptable natural rainfall expected, per contract administrator’s discretion. Once the vegetation has been established, which should occur 2-3 weeks after seeding, the downstream levy is to be excavated to allow water to fill the channel, per contract administrator’s approval. Once the water has stabilized within the channel, the upstream levy will be excavated to allow the stream flow to enter the new channel. This work is to be completed at a time when the flow within the stream is at an acceptably low rate. This flow rate is expected to be less than 50 cfs but is subject to contract administrator’s approval. Once the new channel has been fully connected, the old channel will be partially backfilled at both the upstream and downstream connection points; the contract drawings include contours that show the expected post-construction ground surface. The contractor will re-seed the areas of the site that have been degraded by construction equipment and materials. Note that the field located west of the 68th Street site will not need to be seeded. The temporary access road at the 68th Street site will be removed and the land returned to its existing state. Post-construction review is to be completed after the first major storm and after 1 year has passed to determine the stability of the new stream reach and the vitality of the vegetation. This is to be completed by the contractor and the contract administrator.
Project Costs

Final costing for the project sites was completed using Goldenseal construction estimation software. This software allows for a comprehensive estimate of job items including materials, labour, and administration fees. By using this software, the team was able to arrive at a more complete estimate which included basic engineering services and a 15% contractor profit margin. All quantities were calculated using the current drawings produced in AutoCAD Civil 3D. Volumes of cut and fill were calculated by comparing the proposed stream channel surfaces with the current ground surface. All other measurements, such as fencing lengths, ground clearing area, and road lengths were calculated based on the proposed design. The final cost of the M-6 Project site was $250,000, which does not include permitting cost and meeting cost for said permits. The final cost of the 68th Street site was $350,000, which includes a land acquisition cost of $90,000 but also does not include permitting costs. Full cost breakdowns, including expected materials and labour, are included in Appendix VII.
Future Project Suggestions

The team would like to suggest some areas of Plaster Creek Watershed for a future stream restoration senior design project. One of these areas is a tributary that flows west from Shadyside Park in Gaines Township and joins Plaster Creek between the 68th St site and the 68th St culvert. As can be seen via satellite imagery, this tributary has some bends that are badly eroding its banks and passes by livestock farms without sanitary protection. Efforts have been made to restore this stream at one site located in Shadyside Park, but other parts of the stream are in need of restoration as well. Another area is a tributary that runs through industrial areas near the interchange between M-6 and Broadmoor Ave, including past the Steelcase pyramid, and joins Plaster Creek just upstream of the M-6 site. According to Fishbeck, Thompson, Carr & Huber, this tributary contains more *E. coli* than does any other part of the watershed. This tributary was also a potential restoration site suggested by Plaster Creek Stewards.

The team encourages any team interested in stream restoration to contact either Professor David Warners or Mike Ryskamp for project ideas. Both play leadership roles in Plaster Creek Stewards, were very forthcoming in suggesting project ideas, and were excellent to work with. The organization itself is very active in developing small restoration projects for Plaster Creek and is always interested in new ideas for restoring the health of it. The team also recommends that teams consult the NRCS National Engineering Handbook, Part 654, chapters 11, 12, and TS14, as well as *Applied River Morphology* by David Rosgen.
Acknowledgements

The Meanderers would like to thank the following people for helping us with our project this semester:

- Professor David Wunder for guiding us as we progressed with our project
- Professor Robert Hoeksema for giving us recommendations for our project at the onset and for helping us with HEC-RAS issues
- Professor David Warners for introducing us to Plaster Creek Stewards and potential projects
- Tom Bennett for guiding us in his expertise with stream restoration projects
- Susi Greiner at the MDEQ for providing us with storm flow information and delineations of the watershed
- Mike Ryskamp for giving us direction in developing project goals
- Professor Ralph Stearley for referring us to texts about stream morphology
- Jason VanHorn and Jason Moore for helping us gather GIS data
- Claire Schwartz for giving us design guidance
- Tom Timmermans for giving us advice about wetland base flow
References

- Plaster Creek Stewards. http://www.calvin.edu/admin/provost/pcw/
Appendix I – Sediment Equations

M-6 Proposed

\[ \rho := \frac{1000 \text{ kg}}{\text{m}^3} \quad \theta_c := 0.6 \quad U := \frac{3 \text{ m}}{\text{s}} \quad h := 0.88r \quad z := 0.88r \quad d_{50} := 0.1 \text{ mm} \]

\[ \rho_s := \frac{2750 \text{ kg}}{\text{m}^3} \quad k := 0.4 \quad v := 10^{-6} \frac{\text{m}^2}{\text{s}} \]

\[ k_s := d_{50} \times 100 = 0.1 \text{ m} \quad S := \frac{\rho_s}{\rho} = 2.75 \quad S_a := \frac{d_{50} \times (S - 1) \times g \times d_{50}}{4v} = 1.036 \]

Friction Calculations

\[ \tau_{bp} := 0.5 \rho \left[ \frac{0.06}{\left( \log \left( \frac{12h}{2.5d_{50}} + 10 \right) \right)^2} \right] \cdot U^2 = 0.126 \text{ Pa} \]

\[ \tau_{b} := 0.5 \rho \left[ \frac{0.06}{\left( \log \left( \frac{12h}{k_s} + 10 \right) \right)^2} \right] \cdot U^2 = 0.295 \text{ Pa} \]

\[ \frac{\tau_{bp}}{\rho} = 0.017 \frac{\text{m}}{\text{s}} \]

Bed Load Calculations

Kalinske - Frijlink

\[ q_{B1} := 2 \cdot d_{50} \left( \frac{\tau_{b}}{\rho} \right)^{0.5} \cdot \exp \left[ -0.27 \frac{(S - 1) \cdot d_{50} \cdot g}{\tau_{bp}} \right] = 8.738 \times 10^{-8} \frac{\text{m}^3}{\text{s}} \]

\[ \theta := \left( \frac{\tau_{bp}}{\rho} \right) \frac{1}{(S - 1) \cdot g \cdot d_{50}} = 0.074 \]

Meyer - Peter

\[ \Phi B2 := 8 \cdot (\theta - \theta_c)^{1.5} = -0.017i \]

\[ q_{B2} := \Phi B2 \cdot d_{50} \cdot [(S - 1) \cdot g \cdot d_{50}]^{0.5} = -7.007 \times 10^{-8} \frac{\text{m}^3}{\text{s}} \]

Einstien - Brown

\[ K_{\text{Ein}} := \left[ \frac{2}{3} + \frac{36v^2}{(S - 1) \cdot g \cdot d_{50}^3} \right]^{0.5} - \left[ \frac{36v^2}{(S - 1) \cdot g \cdot d_{50}^3} \right]^{0.5} \]

Calcs by: Matt de Wit
Checked by: Matt Bedner
\( \Phi B3 := 40 \cdot K \cdot 0.3^3 \)

\( qB3 := \Phi B3 \cdot d50 \cdot [(S - 1) \cdot g \cdot d50]^{0.5} = 1.412 \times 10^{-8} \text{ m}^2 / \text{s} \)

**Suspended Sediment Load**

\[
\omega s := \frac{\left( \frac{36v}{d50} \right)^2 + 7.5(S - 1) \cdot g \cdot d50}{2.8} = 6.233 \times 10^{-3} \text{ m/s}
\]

\[
za := \frac{\omega s}{k \cdot uA} = 0.907 \quad A := \frac{ks}{h} \quad B := \frac{z}{h}
\]

\[
I_1 := 0.216 \left( \frac{A}{1 - A} \right)^{za} \int_{A}^{1} \left( \frac{1 - B}{B} \right)^{za} dB = 0.93
\]

\[
I_2 := 0.216 \left( \frac{A}{1 - A} \right)^{za} \int_{A}^{1} \left( \frac{1 - B}{B} \right)^{za} \cdot \ln(B) dB = -2.282
\]

\[
qs := 1.83qB3 \left( \left[ I_1 \cdot \ln \left( \frac{h}{0.033ks} \right) + I_2 \right] \right) = 1.305 \times 10^{-7} \text{ m}^2 / \text{s}
\]

\( qT := qs + qB3 \)

\( \text{sed} := qT \cdot 12.8m = 1.852 \times 10^{-6} \text{ m}^3 / \text{s} \)

\( \text{sedannum} := \text{sed} \cdot 365 \cdot 24 \cdot 60 \cdot 60 = 58.392 \text{ m}^3 / \text{s} \)

\( \text{sedrate} := \text{sed} \cdot ps = 5.092 \times 10^{-3} \text{ kg/s} \)

\( \text{conc} := \frac{\text{sedrate}}{3709 \text{ L/s}} = 1.373 \times 10^{-3} \frac{\text{ kg}}{\text{ m}^3} \)

Calcs by: Matt de Wit

Checked by: Matt Bedner
M-6 Existing

\[ \rho := \frac{1000 \text{ kg}}{\text{m}^3} \quad \theta_c := .03 \quad U := 1 \text{ m/s} \quad h := .95r \quad z := .95r \quad d_{50} := .6 \text{mm} \]

\[ \rho_s := \frac{2750 \text{ kg}}{\text{m}^3} \quad k := 0.2 \quad v := 10^{-6} \text{ m}^2/\text{s} \]

\[ k_s := d_{50} \cdot 100 = 0.06 \text{m} \quad S := \frac{\rho_s}{\rho} = 2.75 \]

\[ S_a := \frac{d_{50} \cdot [(S - 1) \cdot g \cdot d_{50}]^{0.5}}{4 \cdot v} = 15.221 \]

**Friction Calculations**

\[ \tau_{bp} := 0.5 \rho \left[ \frac{0.06}{\left( \log \left( \frac{12h}{2.5d_{50}} \right) \right)^2} \right] \cdot U^2 = 1.992 \text{ Pa} \]

\[ \tau_b := 0.5 \rho \left[ \frac{0.06}{\left( \log \left( \frac{12h}{k_s} \right) \right)^2} \right] \cdot U^2 = 5.777 \text{ Pa} \]

\[ u_a := \left( \frac{\tau_b}{\rho} \right)^{0.5} = 0.076 \text{ m/s} \]

**Bed Load Calculations**

**Kalinske - Frijlink**

\[ qB1 := 2 \cdot d_{50} \left( \frac{\tau_b}{\rho} \right)^{0.5} \cdot \exp \left[ -0.27 \frac{(S - 1) \cdot d_{50} \cdot g}{\tau_{bp}} \right] = 2.259 \times 10^{-5} \text{ m}^2/\text{s} \]

\[ \theta := \frac{\tau_{bp}}{(S - 1) \cdot g \cdot d_{50}} = 0.193 \]

**Meyer - Peter**

\[ \Phi B2 := 8 \cdot (\theta - \theta_c)^{1.5} = 0.514 \]

\[ qB2 := \Phi B2 \cdot d_{50} \cdot [(S - 1) \cdot g \cdot d_{50}]^{0.5} = 3.13 \times 10^{-5} \text{ m}^2/\text{s} \]
Einstein - Brown

\[ K := \left[ \frac{2}{3} + \frac{36v^2}{(S - 1) \cdot \text{g} \cdot d_{50}^3} \right]^{0.5} - \left[ \frac{36v^2}{(S - 1) \cdot \text{g} \cdot d_{50}^3} \right]^{0.5} = 0.724 \]

\[ \Phi B3 := 40 \cdot K \cdot \theta^3 \]

\[ qB3 := \Phi B3 \cdot d_{50} \cdot [(S - 1) \cdot \text{g} \cdot d_{50}]^{0.5} = 1.276 \times 10^{-5} \text{ m}^2 / \text{s} \]

Suspension Sediment Load

\[ \omega_s := \frac{\left( \frac{36v}{d_{50}} \right)^2 + 7.5(S - 1) \cdot \text{g} \cdot d_{50}}{2.8} \cdot \left[ \frac{-36v}{d_{50}} \right]^{0.5} = 0.08 \text{ m/s} \]

\[ za := \frac{\omega_s}{k \cdot \text{ua}} = 2.635 \quad A := \frac{ks}{h} \quad B := \frac{z}{h} \]

\[ I_1 := 0.216 \left( \frac{A}{(1 - A)^z} \right) \int_1^{(1 - B)^{za}} \left( \frac{1 - B}{B} \right) \cdot \text{dB} = 0.109 \]

\[ I_2 := 0.216 \left( \frac{A}{(1 - A)^z} \right) \int_1^{(1 - B)^{za}} \left( \frac{1 - B}{B} \right)^{za} \cdot \text{ln(B)} \cdot \text{dB} = -0.252 \]

\[ qs := 1.83qB2 \left( \left( I_1 \cdot \text{ln} \left( \frac{h}{0.033ks} \right) + I_2 \right) \right) = 2.417 \times 10^{-5} \text{ m}^2 / \text{s} \]

\[ \text{sed} := qs \cdot 4.3 \cdot h = 1.136 \times 10^{-4} \text{ m}^3 / \text{s} \]

\[ \text{sedrate} := \text{sed} \cdot \rho_s = 0.312 \frac{\text{kg}}{\text{s}} \]

\[ \text{conc} := \frac{\text{sedrate}}{3990L} = 0.078 \frac{\text{kg}}{\text{m}^3} \]

Calcs by: Matt de Wit
Checked by: Matt Bedner
Appendix II – FTCH Habitat and *E. coli* Assessments

Figure A.2.1. Locations of the habitat assessments.
### Figure A.2.2. Macroinvertebrate evaluation results.

<table>
<thead>
<tr>
<th>METRIC</th>
<th>STATION 1</th>
<th>STATION 2</th>
<th>STATION 3</th>
<th>STATION 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL NUMBER OF TAXA</td>
<td>7</td>
<td>10</td>
<td>7</td>
<td>13</td>
</tr>
<tr>
<td>NUMBER OF MAYFLY TAXA</td>
<td>0</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
</tr>
<tr>
<td>NUMBER OF CADISFLY TAXA</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>NUMBER OF STONEFLY TAXA</td>
<td>0</td>
<td>-1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>PERCENT MAYFLY COMP.</td>
<td>0.00</td>
<td>4.39</td>
<td>0.00</td>
<td>4.50</td>
</tr>
<tr>
<td>PERCENT CADISFLY COMP.</td>
<td>21.51</td>
<td>17.54</td>
<td>2.63</td>
<td>0.90</td>
</tr>
<tr>
<td>PERCENT CONTR. DOM. TAXON</td>
<td>32.28</td>
<td>0</td>
<td>26.32</td>
<td>39.47</td>
</tr>
<tr>
<td>PERCENT ISOPOD, SNAIL, LEECH</td>
<td>23.65</td>
<td>28.07</td>
<td>26.32</td>
<td>5.41</td>
</tr>
<tr>
<td>PERCENT SURF. AIR BREATHERS</td>
<td>0.00</td>
<td>1.75</td>
<td>1.32</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**TOTAL SCORE**: -5  -4  -7  -4

**MACROINV. COMMUNITY RATING**: POOR  ACCEPT.  POOR  ACCEPT.

### Figure A.2.3. Pollution evaluation results.

<table>
<thead>
<tr>
<th>HABITAT METRIC (MAXIMUM SCORE)</th>
<th>Plaster Cr. u/s Godfrey Ave.</th>
<th>Plaster Cr. u/s Eastern Ave.</th>
<th>Plaster Cr. 68th St</th>
<th>- Little Plaster Creek</th>
<th>East Paris Ave.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Avail. Cover (20):</td>
<td>15</td>
<td>5</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Embeddedness (20):</td>
<td>6</td>
<td>6</td>
<td>5</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Velocity/Depth (20):</td>
<td>11</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Flow Stability (10):</td>
<td>8</td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Bottom Depos. (10):</td>
<td>12</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Run-off Bands (15):</td>
<td>15</td>
<td>4</td>
<td>2</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Bank Stability (10):</td>
<td>2</td>
<td>2</td>
<td>3</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Stability (10):</td>
<td>6</td>
<td>3</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Stream Cover (10):</td>
<td>6</td>
<td>8</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

**TOTAL SCORE (135):** 81 43 38 48

**HABITAT RATING:** GOOD  FAIR  FAIR  FAIR

Weather: Sunny  Sunny  Sunny  Sunny
Air Temperature (Deg. F): 74  80  85  80
Water Temperature (Deg. F): 67  70  74  69
Ave. Stream Width (Feet): 20  25  20  12
Ave. Stream Depth (Feet): 1  2  1  0.5
Surface Velocity (Ft./Sec): 1  0.5  0.5  0.5
Estimated Flow (CFS): 20  25  10  3
Stream Modifications: cattle access
Non-Point Plants (%): 0  0  0  0
Report Number: MDEQ/SWQ-01/107
STORED No.: 410628  410629  410631  410630

**County Code:** 41  41  41  41
**County Name:** Kent  Kent  Kent  Kent
**TRS:** 06N12W02  06N11W17  05N11W10  05N11W12
**Latitude (dd):** 42.9359  42.93936  42.84102  42.9006
**Longitude (dd):** -85.68748  -85.64974  -85.50867  -85.55
**Ecoregion:** SNMTP  SNMTP  SNMTP  SNMTP
**Stream Type:** Warmwater  Warmwater  Warmwater  Warmwater
**USGS Basin Code:** 4005006  4050006  4050006  4050006
**USGS Basin Name:** Lower Grand R.  Lower Grand R.  Lower Grand R.  Lower Grand R.
**Comments:**
Plaster Creek is impaired due to excessive solids and runoff loadings from a variety of sources including agriculture in the upper reaches, suburban development in the upper/middle reaches and urbanization in the lower reaches. Major WQ problems are related to elevated E. coli, suspended solids and extremes in flow regimes (highs and lows) that also impair the biota of Plaster Creek.
Figure A.2.4. Locations of the *E. coli* assessments.

Figure A.2.5. Results of the *E. coli* assessments.
Appendix III – Cardno JFNew Emergent Wetland Seed Mixture

<table>
<thead>
<tr>
<th>Botanical Name</th>
<th>Common Name</th>
<th>PLS Dounces/Acre</th>
<th>Seeds/Oz</th>
<th>Seeds/SQ FT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Permanent Grasses/Sedges/Rushes:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carex comosa</td>
<td>Bristly Sedge</td>
<td>2.50</td>
<td>4183</td>
<td>2.36</td>
</tr>
<tr>
<td>Carex lacustris</td>
<td>Common Lake Sedge</td>
<td>0.25</td>
<td>26000</td>
<td>0.15</td>
</tr>
<tr>
<td>Carex lunda</td>
<td>Bottlebrush Sedge</td>
<td>4.00</td>
<td>12000</td>
<td>1.10</td>
</tr>
<tr>
<td>Carex vulpinoidea</td>
<td>Brown Fox Sedge</td>
<td>6.00</td>
<td>125000</td>
<td>17.22</td>
</tr>
<tr>
<td>Eleocharis ovata</td>
<td>Blunt Spike Rush</td>
<td>1.00</td>
<td>95000</td>
<td>2.18</td>
</tr>
<tr>
<td>Leersia argyrosides</td>
<td>Rice Cut Grass</td>
<td>3.00</td>
<td>94500</td>
<td>6.51</td>
</tr>
<tr>
<td>Juncus effusus</td>
<td>Common Rush</td>
<td>1.00</td>
<td>281000</td>
<td>6.45</td>
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<tr>
<td>Scirpus acutus</td>
<td>Hard-stemmed Bulrush</td>
<td>2.50</td>
<td>20000</td>
<td>1.15</td>
</tr>
<tr>
<td>Scirpus pungens</td>
<td>Chairmaker’s Rush</td>
<td>4.00</td>
<td>125000</td>
<td>11.48</td>
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<tr>
<td>Scirpus validus</td>
<td>Great Bulrush</td>
<td>6.00</td>
<td>37813</td>
<td>5.21</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>30.25</td>
<td></td>
<td>53.81</td>
</tr>
</tbody>
</table>

| **Temporary Cover:** |                              |                  |          |             |
| Avena sativa         | Common Oat                   | 360.00           | 8125     | 67.15       |
| Lolium multiflorum   | Annual Rye                   | 100.00           | 14188    | 32.57       |
| **Total**            |                              | 460.00           |          | 99.72       |

| **Forbs:** |                              |                  |          |             |
| Alcinos calamus      | Sweet Flag                  | 0.50             | 7000     | 0.08        |
| Alllisma spp.        | Water Plantain [Various Mix]| 2.00             | 70175    | 3.22        |
| Asclepias incarnata  | Swamp Milkweed              | 1.50             | 4540     | 0.16        |
| Castaloanthus occidentalis | Buttonbush | 0.50             | 12500    | 0.14        |
| Decodon verticillatus| Swamp Loosestrife           | 0.50             | 40250    | 0.46        |
| Euaptionum maculatum | Spotted Joe-Pye Weed        | 0.50             | 78125    | 0.90        |
| Hibiscus spp.        | Rosemallow [Various Mix]    | 3.00             | 2188     | 0.15        |
| Iris virginica       | Blue Flag                   | 6.00             | 1400     | 0.19        |
| Lobelia cardinalis   | Cardinal Flower             | 0.25             | 437500   | 2.51        |
| Lobelia spathulata   | Great Blue Lobelia          | 1.50             | 520000   | 17.91       |
| Lythrum americanus   | Common Water Herehound      | 0.25             | 225000   | 1.35        |
| Milium effusus       | Monkey Flower               | 1.00             | 203500   | 6.51        |
| Pellanda virginica   | Arrow Arum                  | 16.00            | 42       | 0.02        |
| Penthorum sedexides  | Ditch Stonecrop             | 0.50             | 36063    | 0.41        |
| Polygonum spp.       | Pinkweed [Various Mix]      | 0.50             | 4063     | 0.05        |
| Pontederia cordata   | Pickerel Weed               | 10.00            | 1250     | 0.29        |
| Sagittaria latifolia | Common Arrowhead            | 2.00             | 56700    | 2.60        |
| Sparganium americanum| American Bur Reed           | 1.00             | 975      | 0.02        |
| Sparganium eurycarpum| Common Bur Reed             | 4.00             | 596      | 0.05        |
| Verbena hastata      | Blue Vervain                | 1.00             | 125000   | 2.87        |
| **Total**            |                              | 52.50            |          | 36.94       |

| **Sold in 1 Acre Increments** |                              |                  |          |             |
| 1 or More Acre       | 12 Acre                      | 14 Acre          |          |             |
| $1250                | $750                         | $400             |          |             |

| **Mix Statistics** |                              |                  |          |             |
| Native Component    | PLS lbs/Acre | PLS Seeds/Acre | PLS Seeds/Sq Ft | % of Native Mix |
| Forbs               | 3.28         | 1609,322      | 36.94     | 40.71%      |
| Grasses             | 1.83         | 2,343,836     | 53.61     | 53.23%      |
| **Total Natives**   | 5.17         | 3,953,157     | 90.75     | 100.00%     |
| Cover               | 23.75        | 4,343,800     | 99.72     |             |
| **Totals**          | 33.92        | 8,296,957     | 190.47    |             |
Appendix IV – Stone Sizing

Stone Sizing, NRCS NEH 654, TS14C

Isbash Method

Velocities taken from HEC-RAS results for 25-year storm

\[
V_1 := \frac{7.23}{\text{s}} \quad \text{For transition between current and proposed channel} \quad V_2 := \frac{2.6}{\text{s}} \quad \text{Average velocity throughout designed channel}
\]

\[
\gamma_s := 165 \frac{\text{lb}}{\text{ft}^3} \quad \text{Density of stones}
\]

\[
\gamma_w := 62.4 \frac{\text{lb}}{\text{ft}^3} \quad \text{Density of water}
\]

\[
\mu := 0.00002034 \frac{\text{lb}}{\text{ft} \cdot \text{s}} \quad \text{Dynamic viscosity of water}
\]

\[
D := 7.85 \text{ft} \quad \text{Hydraulic diameter of channel, assuming it is a rectangle}
\]

\[
\text{Re}_1 := \frac{\gamma_w \cdot V_1 \cdot D}{\mu} = 1.741 \times 10^8 \text{ Re}_2 := \frac{\gamma_w \cdot V_2 \cdot D}{\mu}
\]

\[
C := 0.8 \quad \text{Coefficient for high turbulence}
\]

\[
D_{50\text{trans}} := \left( \frac{V_1}{C} \right)^2 \left[ \frac{1}{2 \cdot g \cdot \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right)} \right] = 8.016 \text{ in} \quad \text{Average stone size in transition zone}
\]

\[
D_{50\text{avg}} := \left( \frac{V_2}{C} \right)^2 \left[ \frac{1}{2 \cdot g \cdot \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right)} \right] = 1.037 \text{ in} \quad \text{Average stone size in designed channel}
\]

Calcs by: Matt Bedner
Checked by: Matt de Wit
National Cooperative Highway Research Program Report 108

\[ R := 1.96 \text{ft} \] Hydraulic radius of channel, assuming it is square

\[ S_e := 0.0066 \] Energy slope of designed channel

\[ D_{50\text{emp}} := 0.25 \frac{\gamma w}{S_e} \frac{R}{\text{ft}} = 0.02 \]

\[ D_{50\text{NCHRPR}} = 12 \cdot D_{50\text{emp}} \cdot \text{in} = 0.245 \text{ir} \]

USACE - Maynord Method

\[ FS := 1.5 \]

\[ C_s := 0.37 \]

\[ C_T := 1.5 \]

\[ R_c := 248 \text{ft} \] Radius of curvature

\[ \theta := 36.8^\circ \] Channel side angle in degrees

\[ \phi := 45^\circ \] Angle of repose

\[ W_{\text{trans}} := 32 \text{ft} \]

\[ W_{\text{bend}} := 72 \text{ft} \]

\[ d_{\text{trans}} := 6.53 \text{ft} \]

\[ d_{\text{bend}} := 4.24 \text{ft} \]

\[ K_1 := \sqrt{1 - \frac{\sin(\theta)^2}{\sin(\phi)^2}} = 0.499 \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
C_{\text{vtrans}} := 1.283 - 0.21\log\left(\frac{R_c}{W_{\text{trans}}}\right) \quad C_{\text{vbend}} := 1.283 - 0.21\log\left(\frac{R_c}{W_{\text{bend}}}\right)

30 percentile stone diameter for transition zone

D_{30\text{mmtrans}} := \text{FS} \cdot C_s \cdot C_{\text{vtrans}} \cdot C_T \cdot d_{\text{trans}} \left(\frac{\gamma_w}{\gamma_s - \gamma_w} \cdot \frac{V_1}{\sqrt{K_1 \cdot g \cdot d_{\text{trans}}}}\right)^{2.5} = 16.437\text{in}

30 percentile stone diameter for designed channel

D_{30\text{mmbend}} := \text{FS} \cdot C_s \cdot C_{\text{vbend}} \cdot C_T \cdot d_{\text{bend}} \left(\frac{\gamma_w}{\gamma_s - \gamma_w} \cdot \frac{V_2}{\sqrt{K_1 \cdot g \cdot d_{\text{bend}}}}\right)^{2.5} = 1.511\text{in}

US Bureau of Reclamation Method

D_{50\text{BRtrans}} := 0.01 \left(\frac{V_1}{\text{ft/s}}\right)^{2.44} \cdot \text{ft} = 14.979\text{in} \quad 50\text{ percentile stone diameter for transition zone}

D_{50\text{BRavg}} := 0.01 \left(\frac{V_2}{\text{ft/s}}\right)^{2.44} \cdot \text{ft} = 1.235\text{in} \quad 50\text{ percentile stone diameter for designed channel}

Calcs by: Matt Bedner
Checked by: Matt de Wit
Tillatoba Model Study

\[
D_{50T\, \text{MS}_{\text{trans}}} = 0.00116 \left( \frac{V_1}{\text{ft/s}} \right)^3 \frac{\text{ft}}{\text{d}_{\text{trans}}} = 2.059\text{in}
\]

\[
D_{50T\, \text{MS}_{\text{avg}}} = 0.00116 \left( \frac{V_2}{\text{ft/s}} \right)^3 \frac{\text{ft}}{\text{d}_{\text{bend}}} = 0.119\text{in}
\]

Calcs by: Matt Bedner
Checked by: Matt de Wit
USACE Steep Slope Riprap Design

\[ C_{SSRD} = 1.4 \]

\[ W_{b\text{trans}} := 18.38 \text{ft} \quad W_{b\text{avg}} := 59 \text{ft} \]

\[ Q := 968 \frac{\text{ft}^3}{\text{s}} \]

\[ q_{\text{trans}} := \frac{Q}{W_{b\text{trans}}} \quad q_{\text{avg}} := \frac{Q}{W_{b\text{avg}}} \]

\[ D_{30\text{SSRD trans}} := \frac{1.95 S_e 0.555 \left(C_{SSRD q_{\text{trans}}} \right)^{\frac{2}{3}}}{\frac{1}{3} \left(\frac{1}{g}\right)} = 2.34 \text{in} \]

\[ D_{30\text{SSRD avg}} := \frac{1.95 S_e 0.555 \left(C_{SSRD q_{\text{avg}}} \right)^{\frac{2}{3}}}{\frac{1}{3} \left(\frac{1}{g}\right)} = 1.075 \text{in} \]

USACE Habitat Boulder Design

\[ SG := 2.6 \]

\[ D_{\text{minHBD trans}} := \frac{18 d_{\text{trans}} S_e}{(SG - 1)} = 0.57 \text{in} \]

\[ D_{\text{minHBD avg}} := \frac{18 d_{\text{bend}} S_e}{(SG - 1)} = 0.37 \text{in} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Abt and Johnson

\[ D_{50AJtrans} := \left( \frac{q_{trans}}{\frac{ft^2}{s}} \right)^{0.56} \cdot S_e^{0.43} \cdot 5.23\text{ in} = 2.075\text{ in} \]

\[ D_{50AJavg} := \left( \frac{q_{avg}}{\frac{ft^2}{s}} \right)^{0.56} \cdot S_e^{0.43} \cdot 5.23\text{ in} = 1.08\text{ in} \]

ABS Rock Chutes

\[ D_{50ABStrans} := 12 \left( 1.923 \frac{q_{trans}}{\frac{ft^2}{s}} \cdot S_e^{1.5} \right)^{0.529} \cdot \text{in} = 0.417\text{ in} \]

\[ D_{50ABSavg} := 12 \left( 1.923 \frac{q_{avg}}{\frac{ft^2}{s}} \cdot S_e^{1.5} \right)^{0.529} \cdot \text{in} = 0.225\text{ in} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
California Department of Transportation RSP

\[ r := 76 \quad \text{Angle of repose (degrees)} \]

\[ a := 36.8' \quad \text{Outside slope face angle (degrees)} \]

\[
W_{\text{CDOTtrans}} := \frac{\left( \frac{V_1}{\frac{\text{ft}}{s}} \right)^6 \cdot \text{SG}}{(SG - 1)^3 \sin \left( (r - a) \cdot \frac{\pi}{180} \right)} \cdot \text{lb}
\]

\[
D_{\text{CDOTtrans}} := 2 \left( \frac{3 \cdot W_{\text{CDOTtrans}}}{4 \pi \cdot \gamma_s} \right)^{\frac{1}{3}} = 5.91 \text{ in}
\]

\[
W_{\text{CDOTavg}} := \frac{\left( \frac{V_2}{\frac{\text{ft}}{s}} \right)^6 \cdot \text{SG}}{(SG - 1)^3 \sin \left( (r - a) \cdot \frac{\pi}{180} \right)} \cdot \text{lb}
\]

\[
D_{\text{CDOTavg}} := 2 \left( \frac{3 \cdot W_{\text{CDOTavg}}}{4 \pi \cdot \gamma_s} \right)^{\frac{1}{3}} = 0.764 \text{ in}
\]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Far West States - Lane's Method

From Figure TS14C-8

\[ C_{\text{LANE}} := 0.6 \]

\[ K_{\text{LANE}} := 0.5 \]

\[ D_{75\text{LANE}_{\text{trans}}} := \frac{3.5}{C_{\text{LANE}} K_{\text{LANE}}} \frac{\gamma_w}{\text{lb/ft}^3} \frac{d_{\text{trans}}}{\text{ft}} \cdot S_e \cdot \text{in} = 3.049 \text{in} \]

\[ D_{75\text{LANE}_{\text{avg}}} := \frac{3.5}{C_{\text{LANE}} K_{\text{LANE}}} \frac{\gamma_w}{\text{lb/ft}^3} \frac{d_{\text{bend}}}{\text{ft}} \cdot S_e \cdot \text{in} = 1.98 \text{in} \]

Stream Barb Stone Sizing, TS14H

\[ D_{50\text{barb}} := 2.1 \text{in} \]

\[ D_{100\text{barb}} := 2 D_{50\text{barb}} \]

\[ D_{\text{minbarb}} := 0.75 \text{in} \]

\[ H_{\text{outerend}} := \frac{1}{3} d_{\text{bend}} = 1.413 \text{ft} \]

\[ H_{\text{bankend}} := \frac{1}{2} d_{\text{bend}} = 2.12 \text{ft} \]

\[ S := 0.42.5 d_{\text{bend}} = 4.24 \text{ft} \quad \text{Depth of bed key} \]

\[ 1 \text{ in is assumed to be the } \text{d}_{50} \text{ of regular stones} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Figure A.4.1. Stone sizing for the transition and riffle zones using different methods.
Appendix V – Scour Calculations

Scour Calculations, NRCS NEH 654, TS14B

\[ y_1 := 2 \]  
Flow depth (ft). Assumed to be average, not bankfull

\[ S_e := 0.00066 \]  
Energy slope

\[ \Delta S_g := 1.6^c \]  
Relative submerged density of sediment

\[ D_{50} := 0.001968 \]  
50th percentile diameter of sediment particles (ft)

\[ v := 0.0000105 \]  
Kinematic viscosity of water (ft^2/s)

\[ U_{\text{star}} := \sqrt{32.2 y_1 S_e} = 0.207 \]  
Shear velocity (ft/s)

\[ \text{Re} := \frac{U_{\text{star}} D_{50}}{v} = 38.782 \]  
Reynolds number

\[ K := 27 \]  
From Table TS14B-4

\[ a := 0.8^c \]  

\[ b := -0.1^c \]  
Calcs by: Matt Bedner
Checked by: Matt de Wit

\[ D_x := K \left( \frac{y_1 S_e}{\Delta S_g} \right)^a \left( \frac{U_{\text{star}}}{v} \right)^b = 0.015 \]  
Smallest armor particle size (ft)

\[ e := \frac{0.245 + 0.0864}{0.140} = 0.401 \]  
Porosity of bed material. 0.6 is \( D_{50} \) in mm

\[ P_x := 0.1 \]  
Percentage of particles equal to or coarser than \( D_x \)
Equilibrium Slope Method

\[
\tau_c := \frac{0.035 \text{ lb}}{\text{ft}^2}
\]

\[
\gamma_w := \frac{62.4 \text{ lb}}{\text{ft}^3}
\]

\[
L_c := 950 \text{ ft}
\]

\[
S_{\text{design}} := 0.00066
\]

\[
S_{eq} := \frac{\tau_c}{\gamma_w y_1 \text{ ft}}
\]

\[
z_{ad} := L_c \left( S_{\text{design}} - S_{eq} \right) = 0.367 \text{ ft}
\]

From Table TS14B-9. Our site has 75 ppm sediment.

Density of water

Stream reach length

Stream reach slope

Equilibrium slope

Vertical degradation of upstream end

Calcs by: Matt Bedner
Checked by: Matt de Wit
General Scour

\[ K_{\text{mean}} := 1.4 \]  

\[ K_{\text{max}} := 6.4 \]  

\[ z_{\text{mean}} := K_{\text{mean}} D_{50}^{0.11} = 2.818 \text{ ft} \]  

Scour, mean estimate

\[ z_{\text{max}} := K_{\text{max}} D_{50}^{0.11} = 12.899 \text{ ft} \]  

Scour, max estimate

\[ D_{50\text{mm}} := 0.6 \]  

Bankfull discharge

\[ Q_d := 388 \]  

Bankfull width

\[ W := 53 \]  

From Table TS14B-8

\[ K_{\text{lacey}} := 0.195 \]  

\[ K_{\text{blench}} := 0.53 \]  

\[ a_{\text{lacey}} := \frac{1}{3} \]  

\[ a_{\text{blench}} := \frac{2}{3} \]  

\[ b_{\text{lacey}} := 0 \]  

\[ b_{\text{blench}} := -\frac{2}{3} \]  

\[ c_{\text{lacey}} := -\frac{1}{6} \]  

\[ c_{\text{blench}} := -0.109 \]  

\[ z_{\text{lacey}} := K_{\text{lacey}} Q_d^{a_{\text{lacey}}} W^{b_{\text{lacey}}} D_{50\text{mm}}^{c_{\text{lacey}}} = 1.549 \text{ ft} \]  

Two estimates for scour

\[ z_{\text{blench}} := K_{\text{blench}} Q_d^{a_{\text{blench}}} W^{b_{\text{blench}}} D_{50\text{mm}}^{c_{\text{blench}}} = 2.113 \text{ ft} \]  

Calcs by: Matt Bedner
Checked by: Matt de Wit
Live-bed and Clear-Water Scour

\( \text{W}_{b1} := 42 \text{ft} \)  
Bottom width of upstream section

\( \text{W}_{b2} := 35 \text{ft} \)  
Bottom width of contracted section

\( a_{\text{livebed}} := 0.6 \)  
From Table TS14B-10

\[ y_2 := y_1 \left( \frac{\text{W}_{b1}}{\text{W}_{b2}} \right)^a \]

\( z_{\text{livebed}} := y_2 - y_1 = 0.34 \)  
Live-bed scour (ft)

\( K_{\text{clearwater}} := 0.007 \)

\[ y_{\text{clearwater}} := \left[ \frac{K_{\text{clearwater}} \cdot Q_d^2}{2 \left( 1.25D_{50} \right)^3 \left( \frac{\text{W}_{b2}}{\text{ft}} \right)^2} \right]^{\frac{3}{7}} \]

\( z_{\text{clearwater}} := y_{\text{clearwater}} - y_1 = 3.434 \)  
Clearwater scour (ft)

Calcs by: Matt Bedner  
Checked by: Matt de Wit
Bend scour

\[ W_{\text{rifle}} := 53 \text{ft} \]

\[ R_c := 248 \text{ft} \]

\[ y_{\text{bend}} := y_1 \left[ 1.5 + 4.5 \left( \frac{W_{\text{rifle}}}{R_c} \right) \right] \]

\[ z_{\text{bend}} := y_{\text{bend}} - y_1 = 2.923 \]

Radius of curvature

Bend scour (ft)

Calcs by: Matt Bedner
Checked by: Matt de Wit
Will dunes form?

\[
\rho := 62.4
\]

Density of water

\[
u := 1.25
\]

Average velocity at M-6 site

\[
D_{90} := 0.02624
\]

i.e. 8mm

\[
D_{\text{star}} := D_{50} \left( \frac{\Delta S \cdot 32.2}{u^2} \right)^{\frac{1}{3}} = 15.414
\]

If greater than 10, dunes will form

\[
\tau_{\text{ssstar}} := \frac{\rho \cdot 32.2 u^2}{\left( 18 \log \left( \frac{12 R}{3D_{90}} \right) \right)^2}
\]

\[
\theta := \frac{0.24}{D_{\text{star}}} + 0.055 \left( 1 - \exp \left( -0.02 D_{\text{star}} \right) \right)
\]

\[
\tau_{\text{cstar}} := 10^3 \cdot D_{50}
\]

If between 3 and 15, dunes will form

\[
T_{ts} := \frac{\tau_{\text{ssstar}} - \tau_{\text{cstar}}}{\tau_{\text{cstar}}} = 255.795
\]

Dunes will NOT form.

Calcs by: Matt Bedner
Checked by: Matt de Wit
Scour from stream barbs

\[ L_c := 1 \text{ ft} \]
Width of stream barbs extend into (ft)

\[ a_{spur} := 0.5 \]
Corresponds to \( L_c / y_1 \) ratio between 1 and 25

\[ b_{\text{perp}} := -0.781 \]
\[ b_{\text{oblique}} := 0 \]

\[ K_{1a} := 2 \]
a being not submerged
\[ K_{1b} := 1.41 \]
b being submerged

\[ K_{2a} := 17.106 \]
a being perpendicular
\[ K_{2b} := 12.11 \]
b being oblique

\[ z_{spura} := y_1 K_{1a} \left( \frac{L_c}{y_1} \right)^{a_{spur}} = 8.944 \]
Scour depth w/ barb submerged (ft)

\[ z_{spurb} := y_1 K_{1b} \left( \frac{L_c}{y_1} \right)^{a_{spur}} = 6.306 \]
Scour depth w/ barb not submerged (ft)

\[ V_{\text{scouraa}} := z_{spura} \cdot 3 \cdot K_{2a} \left( \frac{L_c}{y_1} \right)^{b_{\text{perp}}} \]
Scour length w/ barb perpendicular, submerged (ft)

\[ V_{\text{scourab}} := z_{spura} \cdot 3 \cdot K_{2b} \left( \frac{L_c}{y_1} \right)^{b_{\text{oblique}}} \]

\[ L_{\text{scouraa}} := \frac{V_{\text{scouraa}}}{z_{spura} \cdot L_c} = 38.935 \]
Scour length w/ barb perpendicular, submerged (ft)

\[ L_{\text{scourab}} := \frac{V_{\text{scourab}}}{z_{spura} \cdot L_c} = 96.88 \]
Scour length w/ barb oblique, submerged (ft)

\[ V_{\text{scourba}} := z_{spurb} \cdot 3 \cdot K_{2a} \left( \frac{L_c}{y_1} \right)^{b_{\text{perp}}} \]
Scour length w/ barb perpendicular, not submerged (ft)

\[ V_{\text{scourbb}} := z_{spurb} \cdot 3 \cdot K_{2b} \left( \frac{L_c}{y_1} \right)^{b_{\text{oblique}}} \]

\[ L_{\text{scourba}} := \frac{V_{\text{scourba}}}{z_{spurb} \cdot L_c} = 19.352 \]
Scour length w/ barb oblique, not submerged (ft)

\[ L_{\text{scourbb}} := \frac{V_{\text{scourbb}}}{z_{spurb} \cdot L_c} = 48.152 \]

\[ z_{\text{spura}} = \text{not submerged} \]
aa = not submerged, perpendicular

\[ z_{\text{spurb}} = \text{submerged} \]
ab = not submerged, oblique

All lengths in ft

Calcs by: Matt Bedner
Checked by: Matt de Wit
Appendix VI – Channel Geometry

Hydraulic Geometry Calculations from NRCS NEH, Part 654, Chapter 12 – M-6 Site

Meander Wavelength

Channel Width in any consistent units of measurement

\[ W := 53 \text{ ft} \]

Lower 95% Confidence Limit Meander Wavelength, Thorne and Soar

\[ \lambda_{\text{min}} := 11.26W \]
\[ \lambda_{\text{min}} = 596.78\text{ft} \]

Upper 95% Confidence Limit Meander Wavelength, Thorne and Soar

\[ \lambda_{\text{max}} := 12.47W \]
\[ \lambda_{\text{max}} = 660.91\text{ft} \]

Leopold and Wolman

\[ \lambda_1 := 10.9 \left( \frac{W}{\text{ft}} \right)^{1.01} \cdot \text{ft} \]
\[ \lambda_1 = 601.098\text{ft} \]

Inglis

\[ \lambda_2 := 6.06 \left( \frac{W}{\text{ft}} \right)^{0.99} \cdot \text{ft} \]
\[ \lambda_2 = 308.678\text{ft} \]

Yalin

\[ \lambda_3 := 6W \]
\[ \lambda_3 = 318\text{ft} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Layout and Sine-Generated Curve

Channel Sinuosity  Minimum sinuosity as prescribed by Rosgen

Meander Arc Length (will be equal to $P \times$ meander wavelength)
Meander wavelength set to 700 ft

Maximum Angle a Path Makes with the Mean Longitudinal Axis (See Figure 12-6)

Riffles

Riffle Spacing, Min Estimate, Hey and Thorne

Riffle Spacing, Avg Estimate, Hey and Thorne

Riffle Spacing, Max Estimate, Hey and Thorne

4 is for Channels with steeper slopes, 10 is for channels with more gradual slopes.

Riffle Width, Min Estimate, Hey and Thorne

Riffle Width, Avg Estimate, Hey and Thorne

Riffle Width, Max Estimate, Hey and Thorne

Widths at Apexes and Pools

Get from Table 12-5

Get from Table 12-5

Width at Apex, Lower Limit of 90% Confidence Interval

Width at Apex, Average Estimate

Width at Apex, Upper Limit of 90% Confidence Interval

Calcs by: Matt Bedner
Checked by: Matt de Wit
\[ a_p := 1.1' \]
\[ u_p := 0.0' \]
Get from Table 12-6

\[ W_{pmin} := (a_p - u_p)W \]
Width at Pool, Lower Limit of 90% Confidence Interval

\[ W_{pavg} := a_pW \]
Width at Pool, Average Estimate

\[ W_{pmax} := (a_p + u_p)W \]
Width at Pool, Upper Limit of 90% Confidence Interval

\[ W_{pmin} = 56.18\text{ft} \]

\[ W_{pavg} = 60.95\text{ft} \]

\[ W_{pmax} = 65.72\text{ft} \]

\[ Pool Location \]
Ratio of curvilinear distance between bend apex and maximum scour location (Z.ap) and curvilinear distance between bend apex and downstream inflection point (Z.ai). 0.36 is the 50% ratio among streams studied in 1981 Red River hydrographic survey. See Figure 12-17.

\[ k := 0.36 \]

\[ Z_{ai} := \frac{M}{2} \]

\[ Z_{ap} := kZ_{ai} \]

\[ Z := Z_{ai} + Z_{ap} \]
Location of pool in feet downstream from stream inflection point

\[ Z_{ai} = 420\text{ft} \]

\[ Z_{ap} = 151.2\text{ft} \]

\[ Z = 571.2\text{ft} \]

\[ Pool Depth \]
Mean Depth of Stream

\[ d_m := 4\text{ft} \]

\[ R_c := \frac{M}{4\omega} \]
Radius of Curvature of Stream

\[ R_c = 233.815\text{ft} \]

\[ \Delta x := 2R_c\sin(\omega) = 365.768\text{ft} \]

\[ \Delta y := 2\left( R_c - R_c\cos(\omega) \right) = 176.268\text{ft} \]
Meander Amplitude

Calcs by: Matt Bedner
Checked by: Matt de Wit
widthtodepth := \frac{W}{d_m} \quad \text{Width-to-Depth Ratio of Stream}
\quad \text{widthtodepth} = 13.25

\begin{align*}
d_{\text{pmin}} := d_m \left( 2.14 - 0.19 \ln \left( \frac{R_c}{W} \right) \right) \\
\text{Pool Depth, Min Estimate; should be used for widthtodepth < 60} \\
\quad \text{d}_{\text{pmin}} = 7.432 \text{ft}
\end{align*}

\begin{align*}
d_{\text{pmax}} := d_m \left( 2.98 - 0.54 \ln \left( \frac{R_c}{W} \right) \right) \\
\text{Pool Depth, Max Estimate; should be used for widthtodepth > or = 60} \\
\quad \text{d}_{\text{pmax}} = 8.714 \text{ft}
\end{align*}

\begin{align*}
d_{\text{psafe}} := d_m \left[ 1.5 + 4.5 \left( \frac{W}{R_c} \right) \right] \\
\text{Pool Depth, "Safe" Estimate} \\
\quad \text{d}_{\text{psafe}} = 10.08 \text{ft}
\end{align*}

\text{Bankline Migration Rate and Apex Movement Rate}

\begin{align*}
A_{\text{B2andC}} := 0.115 \left( \frac{W}{\text{ft}} \right)^{0.6669} \frac{\text{ft}}{\text{yr}} \\
\text{Apex Movement, C and B2 Type Streams, ft/yr} \\
\quad A_{\text{B2andC}} = 1.624 \frac{\text{ft}}{\text{yr}}
\end{align*}

\begin{align*}
A_{\text{B2}} := 0.0143 \left( \frac{W}{\text{ft}} \right)^{0.9834} \frac{\text{ft}}{\text{yr}} \\
\text{Apex Movement, B2 Type Streams, ft/yr} \\
\quad A_{\text{B2}} = 0.71 \frac{\text{ft}}{\text{yr}}
\end{align*}

\begin{align*}
A_{\text{C}} := 0.3965 \left( \frac{W}{\text{ft}} \right)^{0.4747} \frac{\text{ft}}{\text{yr}} \\
\text{Apex Movement, C Type Streams, ft/yr} \\
\quad A_{\text{C}} = 2.611 \frac{\text{ft}}{\text{yr}}
\end{align*}

Calcs by: Matt Bedner
Checked by: Matt de Wit
Step 7

\[ W_{bkfref} := 50 \text{ft} \]
Bankfull Width of Reference Reach
Width from Modified Cross Section worksheet increased by 8 ft

\[ d_{bkfref} := 4 \text{ft} \]
Bankfull Depth of Reference Reach
Depth to keep same cross-section but maintain width-to-depth ratio of 12

\[ L_{mref} := 600 \text{ft} \]
Meander Wavelength of Reference Reach
From hydraulic geometry worksheet

\[ W_{bthr} := \]
Belt Width of Reference Reach

\[ R_{cref} := 233.81 \text{ft} \]
Radius of Curvature of Reference Reach
From Hydraulic Geometry worksheet

\[ d_{mbkfref} := 7.526 \text{ft} \]
Maximum Bankfull Riffle Depth of Reference Reach
From hydraulic geometry worksheet

\[ d_{mbkfpref} := 7.526 \text{ft} \]
Maximum Bankfull Pool Depth of Reference Reach

\[ S_{gref} := \]
Depth of Midpoint of Glide in Reference Reach

\[ S_{runref} := \]
Depth of Midpoint of Run in Reference Reach

\[ S_{pref} := \]
Slope of Pool in Reference Reach

\[ S_{gref} := \]
Slope of Glide in Reference Reach

\[ S_{runref} := \]
Slope of Run in Reference Reach

\[ S_{ref} := 0.000 \]
Slope of Water Surface in Reference Reach
Measured from REGIS’s 2-foot contours. 2-ft elevation change between bridge in Leisure Creek and upstream end of site.

Calcs by: Matt Bedner
Checked by: Matt de Wit
Step 10

\[ DA_{\text{project}} := 35 \text{m}^2 \]

Drainage Area of Project Reach
Estimated from regional curves in Rosgen doc

Step 11

\[ A_{\text{bkf}} := 225 \text{ft}^2 \]

Bankfull Cross-Sectional Area
Estimated from regional curves in Rosgen doc

Step 12

\[ \text{Width to Depth Ratio of Stable Stream} = 12.5 \]

\[ \frac{W_{\text{bkf}}}{d_{\text{bkf}}} \]

Width to Depth Ratio of Stable Stream

\[ \frac{W_{\text{bkf}}}{d_{\text{bkf}}} = 12.5 \]

Width of Restoration Reach

\[ W_{\text{bkf}} = 53.033 \text{ft} \]

Step 14

\[ \frac{d_{\text{bkf}}}{W_{\text{bkf}}} = \left( \frac{W_{\text{bkf}}}{d_{\text{bkf}}} \right)^2 \cdot A_{\text{bkf}} \]

Mean Depth of Restoration Reach

\[ d_{\text{bkf}} = 4.243 \text{ft} \]

Step 15

\[ \frac{L_{\text{m}}}{W_{\text{bkf}}} \]

Meander Length Ratio of Restoration Reach

\[ L_{\text{m}} = 636.396 \text{ft} \]

Meander Wavelength of Restoration Reach

\[ L_{\text{m}} = 636.396 \text{ft} \]

Step 16

\[ \frac{W_{\text{blt}}}{W_{\text{bkf}}} \]

Meander Width Ratio of Restoration Reach

\[ W_{\text{blt}} := MWR_{\text{ref}} \cdot W_{\text{bkf}} \]

Belt Width of Restoration Reach

Calcs by: Matt Bedner
Checked by: Matt de Wit
Step 17
\[ R_c := \left( \frac{R_{cref}}{W_{b kf}} \right) \cdot W_{b kf} \]
Radius of Curvature of Restoration Reach
\[ R_c = 247.998 \text{ft} \]

Step 19
- \( SL := 840\text{ft} \)  
  Length of Designed Stream 
  Adjusted to give a sinuosity of 1.2
- \( VL := 700\text{ft} \)  
  Linear Length of Design Stretch 
  Measured on Google Earth
- \( k_{des} := \frac{SL}{VL} \)  
  Sinuosity of Designed Stream

Step 20
- \( DE := 0.5\text{ft} \)  
  Difference in Water Surface Elevation between Bed Features Along Valley Length 
  Equal to 0.0008*700 ft
- \( S_{val} := \frac{DE}{VL} \)  
  Valley Slope

Step 21
\[ S := \frac{S_{val}}{k_{des}} \]
Designed Stream Average Water Surface Slope
\[ S = 6.667 \times 10^{-4} \]

Step 28
- \( d_{mbkf} := \left( \frac{d_{mbkfref}}{d_{bkfref}} \right) \cdot d_{bkf} \)  
  Depth of Maximum Riffle Bankfull of Designed Stream
\[ d_{mbkf} = 7.983 \text{ft} \]

Step 29
- \( W_{fpa} := 120\text{ft} \)  
  Width of Flood-Prone Area (from cross-sectional view) at an elevation of twice the maximum bankfull depth 
  Adjusted to get \( ER = 2.2 \)
- \( ER := \frac{W_{fpa}}{W_{bkf}} \)  
  Entrenchment Ratio of Designed Stream
\[ ER = 2.263 \]

Step 31
\[ d_{mbkf} := \left( \frac{d_{mbkfrefp}}{d_{bkfref}} \right) \cdot d_{bkf} \]
Depth of Maximum Pool Bankfull of Designed Stream
\[ d_{mbkf} = 7.983 \text{ft} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Hydraulic Geometry Calculations from NRCS NEH, Part 654, Chapter 12 - 68th St Site

Meander Wavelength

\[ W := 14 \text{ ft} \]

Channel Width in any consistent units of measurement

\[ \lambda_{\text{min}} := 11.26W \]

Lower 95% Confidence Limit Meander Wavelength, Thorne and Soar

\[ \lambda_{\text{min}} = 157.64 \text{ ft} \]

\[ \lambda_{\text{max}} := 12.47W \]

Upper 95% Confidence Limit Meander Wavelength, Thorne and Soar

\[ \lambda_{\text{max}} = 174.58 \text{ ft} \]

\[ \lambda_1 := 10.9 \left( \frac{W}{\text{ft}} \right)^{1.01} \cdot \text{ft} \]

Leopold and Wolman

\[ \lambda_1 = 156.681 \text{ ft} \]

\[ \lambda_2 := 6.06 \left( \frac{W}{\text{ft}} \right)^{0.99} \cdot \text{ft} \]

Inglis

\[ \lambda_2 = 82.63 \text{ ft} \]

\[ \lambda_3 := 6W \]

Yalin

\[ \lambda_3 = 84 \text{ ft} \]

\[ Q_{\text{bf}} := 33 \]

Bankfull Discharge Estimated from regional curves in Rosgen doc

\[ \lambda_4 := 30Q_{\text{bf}}^{0.5} \cdot \text{ft} \]

Dury

\[ \lambda_4 = 172.337 \text{ ft} \]

\[ \lambda_5 := 8.2Q_{\text{bf}}^{0.62} \cdot \text{ft} \]

Carlston

\[ \lambda_5 = 71.663 \text{ ft} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Layout and Sine-Generated Curve

\[ P := 1.2 \]

Channel Sinuosity

Minimum sinuosity as prescribed by Rosgen

\[ M := 200 \text{ ft} \]

Meander Arc Length (will be equal to \( P \cdot \text{meander wavelength} \))

Meander wavelength set to 167 ft

\[ \omega := 2.2 \left( \frac{P - 1}{P} \right)^{0.5} \]

Maximum Angle a Path Makes with the Mean Longitudinal Axis (See Figure 12-6)

\[ \omega = 0.898 \]

Riffles

\[ Z_{\text{min}} := 4 \text{ W} \]

Riffle Spacing, Min Estimate, Hey and Thorne

\[ Z_{\text{avg}} := 6.3 \text{ W} \]

Riffle Spacing, Avg Estimate, Hey and Thorne

\[ Z_{\text{max}} := 10 \text{ W} \]

Riffle Spacing, Max Estimate, Hey and Thorne

4 is for Channels with steeper slopes, 10 is for channels with more gradual slopes.

\[ \text{RW}_{\text{min}} := 0.75 \text{ W} \]

Riffle Width, Min Estimate, Hey and Thorne

\[ \text{RW}_{\text{avg}} := 1.034 \text{ W} \]

Riffle Width, Avg Estimate, Hey and Thorne

\[ \text{RW}_{\text{max}} := 1.5 \text{ W} \]

Riffle Width, Max Estimate, Hey and Thorne

4 is for Channels with steeper slopes, 10 is for channels with more gradual slopes.

Widths at Apexes and Pools

\[ a_a := 1.3' \]

Get from Table 12-5

\[ u_a := 0.0' \]

Get from Table 12-5

\[ W_{\text{amin}} := (a_a - u_a) \text{ W} \]

Width at Apex, Lower Limit of 90% Confidence Interval

\[ W_{\text{avg}} := a_a \text{ W} \]

Width at Apex, Average Estimate

\[ W_{\text{amax}} := (a_a + u_a) \text{ W} \]

Width at Apex, Upper Limit of 90% Confidence Interval

Calcs by: Matt Bedner
Checked by: Matt de Wit
a_p := 1.1' 
Get from Table 12-6

u_p := 0.05  
Get from Table 12-6

W_{p_{min}} := (a_p - u_p)W  
Width at Pool, Lower Limit of 90% Confidence Interval  
W_{p_{min}} = 14.84\text{ft}

W_{p_{avg}} := a_p\cdot W  
Width at Pool, Average Estimate  
W_{p_{avg}} = 16.1\text{ft}

W_{p_{max}} := (a_p + u_p)W  
Width at Pool, Upper Limit of 90% Confidence Interval  
W_{p_{max}} = 17.36\text{ft}

Pool Location

k := 0.36

Z_{ai} := \frac{M}{2}

Z_{ap} := kZ_{ai}

Z := Z_{ai} + Z_{ap}  
Location of pool in feet downstream from stream inflection point  
Z = 136\text{ft}

Pool Depth

d_m := 1.1\text{ft}  
Mean Depth of Stream

R_c := \frac{M}{4\omega}  
Radius of Curvature of Stream

R_c = 55.67\text{ft}

\Delta x := 2R_c\cdot\sin(\omega) = 87.088\text{ft}

\Delta y := 2\left(R_c - R_c\cdot\cos(\omega)\right) = 41.969\text{ft}  
Meander Amplitude

Calcs by: Matt Bedner
Checked by: Matt de Wit
\[
\text{widthtodepth} := \frac{W}{d_m}
\]

**Width-to-Depth Ratio of Stream**

\[
\text{widthtodepth} = 12.174
\]

\[
d_{p\text{min}} := d_m \left(2.14 - 0.19 \ln \left(\frac{R_c}{W}\right)\right)
\]

Pool Depth, Min Estimate; should be used for \(\text{widthtodepth} < 60\)

\[
d_{p\text{min}} = 2.159 \text{ ft}
\]

\[
d_{p\text{max}} := d_m \left(2.98 - 0.54 \ln \left(\frac{R_c}{W}\right)\right)
\]

Pool Depth, Max Estimate; should be used for \(\text{widthtodepth} \geq 60\)

\[
d_{p\text{max}} = 2.57 \text{ ft}
\]

\[
d_{p\text{safe}} := d_m \left[1.5 + 4.5 \left(\frac{W}{R_c}\right)\right]
\]

Pool Depth, "Safe" Estimate

\[
d_{p\text{safe}} = 3.026 \text{ ft}
\]

**Bankline Migration Rate and Apex Movement Rate**

\[
Q_{\text{mam}} := .3
\]

Mean Annual Flow. MUST BE IN \(\text{m}^3/\text{s}\).

\[
B := 4.2
\]

Equal to \(W\) but in meters

\[
E := \left(\frac{Q_{\text{mam}}}{246} - 0.572 \frac{B^{1.83}}{\text{yr}}\right) \cdot \frac{\text{m}}{\text{yr}}
\]

Bankline Migration Rate, \(\text{m}/\text{yr}\)

\[
E = 0.378 \frac{\text{ft}}{\text{yr}}
\]

\[
A_{B2\text{and}C} := 0.115 \left(\frac{W}{\text{ft}}\right)^{0.6669} \cdot \frac{\text{ft}}{\text{yr}}
\]

Apex Movement, C and B2 Type Streams, \(\text{ft}/\text{yr}\)

\[
A_{B2\text{and}C} = 0.668 \frac{\text{ft}}{\text{yr}}
\]

\[
A_{B2} := 0.0143 \left(\frac{W}{\text{ft}}\right)^{0.9834} \cdot \frac{\text{ft}}{\text{yr}}
\]

Apex Movement, B2 Type Streams, \(\text{ft}/\text{yr}\)

\[
A_{B2} = 0.192 \frac{\text{ft}}{\text{yr}}
\]

\[
A_C := 0.3965 \left(\frac{W}{\text{ft}}\right)^{0.4747} \cdot \frac{\text{ft}}{\text{yr}}
\]

Apex Movement, C Type Streams, \(\text{ft}/\text{yr}\)

\[
A_C = 1.388 \frac{\text{ft}}{\text{yr}}
\]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Rosgen Design Method, NRCS NEH 654, ch 11, 68th St Site

Step 7

\[ W_{bkr} := 14 \text{ ft} \]  
Bankfull Width of Reference Reach
From regional curves in Rosgen doc and meets minimum width-to-depth of 12

\[ d_{bkr} := 1.15 \text{ ft} \]  
Bankfull Depth of Reference Reach
Depth required to keep the same cross-sectional area but maintain a width-to-depth ratio of 12

\[ L_{m} := 167 \text{ ft} \]  
Meander Wavelength of Reference Reach
Calculated by plugging in 30 ft for the width in the hydraulic geometry worksheet and taking the average of \( \lambda_{\min} \) and \( \lambda_{\max} \)

\[ W_{blt} := \]  
Belt Width of Reference Reach

\[ R_{cref} := 55.67 \text{ ft} \]  
Radius of Curvature of Reference Reach
From hydraulic geometry worksheet

\[ d_{mbkref} := .38 \text{ ft} \]  
Maximum Bankfull Ripple Depth of Reference Reach
Parameter not used in the M-6 calculations

\[ d_{mbkrefp} := 2.16 \text{ ft} \]  
Maximum Bankfull Pool Depth of Reference Reach
From hydraulic geometry worksheet

\[ d_{pref} := \]  
Depth of Midpoint of Glide in Reference Reach

\[ d_{runref} := \]  
Depth of Midpoint of Run in Reference Reach

\[ S_{pref} := \]  
Slope of Pool in Reference Reach

\[ S_{gref} := \]  
Slope of Glide in Reference Reach

\[ S_{runref} := \]  
Slope of Run in Reference Reach

\[ S_{ref} := \]  
Slope of Water Surface in Reference Reach

Calcs by: Matt Bedner
Checked by: Matt de Wit
Step 10
\[ DA_{\text{project}} := 0.6 \text{mi}^2 \]

Drainage Area of Project Reach
From regional curves in Rosgen doc; although could be anywhere from 0.4 to 0.9

Step 11
\[ A_{\text{bkf}} := 16 \text{ft}^2 \]

Bankfull Cross-Sectional Area
From regional curves in Rosgen doc

Step 12
\[ \text{Width to depth ratio of stable stream} := \frac{W_{\text{bkf ref}}}{d_{\text{bkf ref}}} \]

Width to Depth Ratio of Stable Stream
\[ \text{Width to depth ratio of stable stream} = 12.174 \]

\[ W_{\text{bkf}} := \left[ \left( \frac{W_{\text{bkf ref}}}{d_{\text{bkf ref}}} \right) \cdot A_{\text{bkf}} \right]^{\frac{1}{2}} \]

Width of Restoration Reach
\[ W_{\text{bkf}} = 13.956 \text{ft} \]

Step 13
\[ d_{\text{bkf}} := \frac{W_{\text{bkf}}}{\left( \frac{W_{\text{bkf ref}}}{d_{\text{bkf ref}}} \right)} \]

Mean Depth of Restoration Reach
\[ d_{\text{bkf}} = 1.146 \text{ft} \]

Step 15
\[ \text{MLR}_{\text{ref}} := \frac{L_{\text{m ref}}}{W_{\text{bkf ref}}} \]

Meander Length Ratio of Restoration Reach
\[ \text{MLR}_{\text{ref}} \cdot W_{\text{bkf}} \]

Meander Wavelength of Restoration Reach
\[ L_{\text{m}} = 166.481 \text{ft} \]

Step 16
\[ \text{MWR}_{\text{ref}} := \frac{W_{\text{blt ref}}}{W_{\text{bkf ref}}} \]

Meander Width Ratio of Restoration Reach
\[ \text{MWR}_{\text{ref}} \cdot W_{\text{bkf}} \]

Belt Width of Restoration Reach

Calcs by: Matt Bedner
Checked by: Matt de Wit
Step 17
\[ R_c := \left( \frac{R_{\text{ref}}}{W_{\text{bkf ref}}} \right) \cdot W_{\text{bkf}} \]
Radius of Curvature of Restoration Reach
\[ R_c = 55.497\text{ft} \]

Step 19
\[ \text{SL} := 1080\text{ft} \]
Length of Designed Stream
Adjusted to get a sinuosity of 1.2
\[ \text{VL} := 900\text{ft} \]
Linear Length of Design Stretch
Measured in Civil 3D
\[ k_{\text{des}} := \frac{\text{SL}}{\text{VL}} \]
Sinuosity of Designed Stream

Step 20
\[ \text{DE} := 1 \]
Difference in Water Surface Elevation between Bed Features Along Valley Length
\[ S_{\text{val}} := \frac{\text{DE}}{\text{VL}} \]
Valley Slope

Step 21
\[ S := \frac{S_{\text{val}}}{k_{\text{des}}} \]
Designed Stream Average Water Surface Slope

Step 28
\[ d_{\text{mbk f}} := \left( \frac{d_{\text{mbk f ref}}}{d_{\text{bkf ref}}} \right) \cdot d_{\text{bkf}} \]
Depth of Maximum Riffle Bankfull of Designed Stream
\[ d_{\text{mbk f}} = 0.379\text{ft} \]

Step 29
\[ W_{\text{fpa}} := 3\text{ft} \]
Width of Flood-Prone Area (from cross-sectional view) at an elevation of twice the maximum bankfull depth
Adjusted to get an ER of 2.2
\[ \text{ER} := \frac{W_{\text{fpa}}}{W_{\text{bkf}}} \]
Entrenchment Ratio of Designed Stream
\[ \text{ER} = 2.221 \]

Step 31
\[ d_{\text{mbk fp}} := \left( \frac{d_{\text{mbk fp ref}}}{d_{\text{bkf ref}}} \right) \cdot d_{\text{bkf}} \]
Depth of Maximum Pool Bankfull of Designed Stream
\[ d_{\text{mbk fp}} = 2.153\text{ft} \]

Calcs by: Matt Bedner
Checked by: Matt de Wit
Appendix VII – Costing
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