Whiskey Creek Final Report

Nick Keegstra, Jake Parks, Lukas Vander Linden

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List of Abbreviations

AASHTO – American Association of State Highway and Transportation Officials
BMP – Best Management Practice
BOD – Biochemical Oxygen Demand
EB – East Beltline
EPA – Environmental Protection Agency
FAC – Fine Arts Center
HEC HMS – Hydrologic Engineering Center’s Hydrologic Modeling System
LID – Low Impact Development
NOAA – National Oceanic and Atmospheric Administration
SC Lot – Southwest Commuter’s Lot
SCS – Soil Conservation Service
SEMCOG – Southeast Michigan Council Of Governments
STEPL – Spreadsheet Tool for Estimating Pollutant Loads
SWMM – Stormwater Management Model
TSS – Total Suspended Solids
Executive Summary

This project is a pilot study to demonstrate to communities, developments, and campuses in the West Michigan area that Low Impact Development (LID) stormwater management is a feasible and cost effective form of stormwater management which can increase stream health. To do this, the team analyzed several LID best management practices (BMPs) and determined which were most suitable for the developed campus of Calvin College. Then, sites were selected for each BMP. The Environmental Protection Agency’s Stormwater Management Model (SWMM) software was used to determine the effectiveness of the proposed BMPs on volume reduction of stormwater runoff. Spreadsheet tools were used to evaluate pollution reduction. A detailed map of the Calvin College campus is shown in Figure 1.

![Figure 1: Map of Calvin College.](image)
Pervious pavement parking lots, bioretention basins, vegetated roofs, natural revegetation, and subsurface infiltration beds were the BMPs identified as being feasible on the Calvin campus. Eight sites were identified on campus as potential sites for these BMPs.

It is important that each BMP be of reasonable cost, because many communities, businesses, and campuses may think that LID costs more than traditional stormwater management techniques. The estimated costs are however comparable to the cost of constructing and maintaining a detention basin which is currently a common practice of stormwater management.

The team designed each proposed BMP in detail and determined the effect of each BMP on stormwater runoff quantity and quality. Additionally preconstruction drawings were made for every BMP proposed to be located on campus with an emphasis on the design of the East Beltline Lot (EB Lot). Prof. Randy Van Dragt has possible funding from the Michigan Department of Environmental Quality to construct a stormwater BMP for a drainage problem associated with the EB Lot. The rest of the BMPs are only proposed solutions to demonstrate that LID stormwater management is feasible, so they will not be constructed unless more funding is secured. However, the team hopes that Calvin College and surrounding communities will consider constructing LID BMPs in the future as a result of this study.

**LID**

LID is a new trend in stormwater management. The goal of LID is to manage stormwater in a way so that a site mimics its presettlement hydrology by using design techniques that infiltrate, filter, store, evaporate and detain runoff close to its source. Traditionally, stormwater management systems have been designed to detain flow for a period of time before releasing it into the water system. Only recently has the industry realized that it is not when the stormwater runoff is released that is important, but the volume of stormwater release that is important. Studies have shown that even if stormwater runoff is detained and released over time, natural hydrology can change drastically. Increased runoff volume is also associated with streambank erosion. LID seeks to reduce the total volume of runoff by infiltrating it into the ground, therefore returning land to its natural hydrology.

As Christians we desire to join God in his redemption work until he comes in to wipe away our tears as told in Revelation 21:4. Part of this work is stewardship. Because of this, the team is motivated to remediate the pollution and drainage problems found throughout the Whiskey Creek watershed by restoring the original hydrology through LID BMP implementation.

**Pilot Study**

Team 10’s final report will be a pilot study that demonstrates LID implementation in a developed area. LID is relatively new, so a case study showing its retrofit implementation could be helpful to businesses, institutions, developments, or communities that wish to improve their impact on the environment. Plaster Creek is a debilitated stream running through Kentwood, MI and Grand Rapids, MI. The team will design solutions in such a way that they will be specific to the Calvin Campus, but easily adapted for other sites in the area that drain to Plaster Creek. As part of the pilot study, the team will create typical drawings of each of the selected BMPs, detailed cost estimates, and a discussion of BMP alternatives. This will enable municipalities, businesses, and other colleges to use this project as a good starting point when designing LID stormwater management systems.
**BMP Plan**

In order to increase runoff quality from campus the team performed a campus wide study to determine where BMPs will be most effective. The team did this by highlighting areas that are high in runoff volume, pollution, or both and designing the most cost effective solution for that area. The team only investigated the portions of campus that are in the Whiskey Creek drainage basin, so the northern portion of the campus was not evaluated. The areas that the team highlighted as “problem areas” include: the EB Lot, the dorms, the FAC and surrounding parking lots, and the south west commuter’s lot. The team also selected several BMPs that do not directly address these major problem areas, but still improve hydrology while staying cost effective.

The team selected bioretention for the south portion of the SC Lot, pervious pavement for the northern portion of the SC Lot, pervious pavement for the FAC Lot, subsurface infiltration for the runoff from the science building and north hall, a green roof on top of Hiemenga hall, and on top of the Hekman library. The team also selected natural revegetation for the grassy strip along the west side of the East Beltline, and bioretention just north of the Hekman Library. The East Beltline student parking lot is unique in that it is not only a source of pollution and increased runoff, but is associated with erosion of an ecosystem preserve trail, and was the inspiration for this project. Because of the uniqueness of the EB Lot, the team will design two solutions, bioretention and pervious pavement. A map of proposed BMPs is shown in Figure 2.
Figure 2: Selected BMPs for the Whiskey Creek watershed on Calvin’s Campus.

Southwest Commuter’s Lot

The SC Lot was dealt with by using two different BMPs, because of the inverted islands in the far south of the lot. These inverted islands are designed to capture stormwater runoff by being slightly lower than the parking lot and grading the parking lot in order to direct stormwater towards the islands. Because stormwater flows towards these inverted islands, they collect lots of floatables like leaves and trash, and often get clogged and therefore require regular maintenance. These islands are ideal for bioretention cells, not only because this parking lot is a large source of increased pollution and flow, but also because these islands are already designed to capture stormwater runoff. There will be no need to regrade the parking lot in order to direct flow towards these islands because the parking lot is already designed to do so. Pervious pavement is more feasible for the northern portion of this lot because the lot is not graded so that stormwater is directed towards islands, but is still a source of pollution and increased runoff volumes.
North Hall and Science Building

The roofs of North Hall and the Science Building are unique in that they are large impervious surfaces and generate high amounts of runoff, but release relatively low pollution concentrations. Currently, these roofs drain into French drains that direct the stormwater into the stormsewer system and discharge it into Whiskey Creek under the FAC Lot. The team selected subsurface infiltration to reduce flow and pollution before discharging into Whiskey Creek. Subsurface infiltration was selected because it is especially effective when there are high water volumes but low pollution levels. Also, there is a grassy area just north of the Commons Lawn that would be suitable for subsurface infiltration. Construction could begin early summer, and finish late summer and very few people would be affected by the design because the area would be replanted with turf grass for the fall semester.

FAC Parking Lot

The FAC Lot is similar to the north portion of the SC Lot in that it is a major contributor of pollution and increased flow. Pervious pavement was selected because the lot is surrounded by development, and there is not enough room to design a different solution. Pervious pavement is beneficial because it requires no more land than what is already dedicated to parking, so the development around the lot would not have to be disturbed.

East Beltline Parking Lot

As mentioned earlier, because this lot is directly associated with an erosion problem in the ecosystem, the team will design several solutions. The first being pervious pavement and the second being bioretention. Bioretention is currently more cost effective, but since the lot is in poor condition, the college might have to resurface the lot soon. If they decided to resurface the lot in the next two to three years, pervious pavement will be a very cost effective option because it is relatively inexpensive to construct pervious pavement when the lot will be under construction anyway.

Supplemental BMPs

The team also selected bioretention just north of the library, vegetated roofs for two major buildings on campus, and revegetation along the west side of the East Beltline because all these BMPs would be inexpensive, but also produce a large “kill” of pollution and volume. The bioretention cell would decrease pollution and runoff volume generated on commons lawn. The vegetated roofs would implement cheap green roof tiles to generally decrease runoff throughout the whiskey creek watershed. The revegetation along the East Beltline would help with erosion problems present because of the slope of the grassy patch currently present in this location, but would also slow flow over this strip. Instead of flowing over turf grass, the runoff would be slowed by larger plants, and more runoff would be taken up and captured by the native species.
Design Cost Estimates

Table 1: Estimated Costs of BMPs.

<table>
<thead>
<tr>
<th>BMP</th>
<th>Low Infiltration</th>
<th></th>
<th>High Infiltration</th>
<th></th>
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<tbody>
<tr>
<td></td>
<td>BMP Cost</td>
<td>Runoff Treated (ft³)</td>
<td>Cost/ft³ Infiltrated</td>
<td>BMP Cost</td>
</tr>
<tr>
<td>Bioretention – SC Lot</td>
<td>$90,000</td>
<td>7760</td>
<td>$11.60</td>
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<td>Bioretention - E Beltline</td>
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<td>2620</td>
<td>$15.27</td>
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<tr>
<td>Green Roof - Library</td>
<td>$1,300,000</td>
<td>15040</td>
<td>$86.44</td>
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<tr>
<td>Porous Pavement - E Beltline</td>
<td>$400,000</td>
<td>17810</td>
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<tr>
<td>Porous Pavement - FAC Lot</td>
<td>$500,000</td>
<td>34370</td>
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</tr>
<tr>
<td>Porous Pavement - SC Lot</td>
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<td>49370</td>
<td>$19.24</td>
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<tr>
<td>Subsurface Infiltration - SB, NH</td>
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<td>$8.63</td>
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<tr>
<td>Native Vegetation - E Beltline</td>
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<td>1640</td>
<td>$30.49</td>
<td>$50,000</td>
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</tbody>
</table>

Porous pavement has a greater construction cost than traditional pavement, however, it does not require the additional construction of a storm sewer system. If Calvin plans to resurface the EB Lot soon, it can pay for porous pavement, thereby fixing the paving and washout problems with one solution without needing a separate stormwater management system. This would make porous pavement a feasible option. A large bioretention basin is feasible because its construction would also solve two problems with a single solution. The biology department wants to clear a half acre area of non-native plant species and replace them with Michigan-native species. A bioretention basin would solve that problem and the trail washout problem with a single solution, thereby making it a cost effective option.

Modeling

Three different computer models were created as part of the Whiskey Project. The models include a hydrologic model, a pollution model, and a stormwater runoff model. The hydrologic model was created
using the Environmental Protection Agency’s Stormwater Management Model (EPA SWMM). The pollution model was created using the EPA’s Spreadsheet Tool for Estimating Pollutant Load (STEPL). The runoff volume reduction was calculated using hand calculations.

**SWMM**

To create the SWMM model, it was first necessary to delineate the Whiskey Creek watershed on the Calvin College campus. At the beginning of this project, there was no conclusive knowledge of the Whiskey Creek drainage basin on campus among the faculty and staff the team met with. Team 10 first had to determine where the main channel of Whiskey Creek flowed. After it was determined that Whiskey Creek began in the marshy area on the east side of the nature preserve before flowing under the East Beltline, around the FAC Lot and into the Seminary Pond, the team began delineating the Whiskey Creek basin. Using a topographic map created from an aerial survey taken in 1997 of Calvin College, the team delineated seventeen sub basins across campus. The sub basins were verified by lifting manhole covers to verify that the actual stormsewer system matched the stormsewer system on the map. Figure 3 shows the delineation of the Whiskey Creek watershed.

![Figure 3: Sub basin delineation.](image)

Once sub basins were delineated, data could then be gathered from each sub basin for the SWMM inputs. SWMM calculates runoff by simulating each sub basin as being a rectangle with a certain slope, depression storage, and roughness. The SWMM equations then calculate runoff. After creating the SWMM model, the team verified its accuracy by comparing the runoff values from SWMM with flow measurements taken in the field. This was done by retrieving data from the National Oceanic and Atmospheric Association’s (NOAA) Daily and Hourly Precipitation Browser and recreating the storm during which flow measurements were taken in SWMM. Then, the flow measured in the field was compared to the runoff of values created in SWMM by the real storm data.
Three different SWMM models were created to compare runoff volumes for presettlement, predevelopment, existing conditions, and existing conditions with the LID plan. The results are shown in Figure 4.

![Figure 4: Runoff volumes for a 2-year statistical storm for various conditions of campus.](image)

**Pollution**

Pollution reduction is a secondary effect of LID. The primary goal is to reduce runoff volume, but LID BMPs also improve water runoff quality by reducing the amount of pollution entering streams. The STEPL spreadsheet pollution model was created by recreating the sub basins according to their estimated pollution load and their runoff curve number. The STEPL spreadsheet then calculates the pollution removal by taking into consideration the effective BMP application area and the BMP pollution removal efficiency. Figure 5 shows the estimated pollution load and pollution load reduction due to BMPs.
Figure 5: Percent reductions of various pollutants after implementing the LID Plan.

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Proposed BMPs</th>
<th>Existing BMPs</th>
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</thead>
<tbody>
<tr>
<td>Nitrogen</td>
<td>23</td>
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<tr>
<td>BOD</td>
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</tr>
<tr>
<td>Sediment</td>
<td>23</td>
<td>50</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Percent Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nitrogen</td>
</tr>
<tr>
<td>Phosphates</td>
</tr>
<tr>
<td>BOD</td>
</tr>
<tr>
<td>Sediment</td>
</tr>
</tbody>
</table>

The figure shows the percent reductions of various pollutants after implementing the LID Plan. The proposed BMPs show a higher reduction in Sediment compared to the existing BMPs.
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1 Background image from Bing maps
1. Acknowledgements

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Prof. Randy Van Dragt, Biology, Calvin College
Prof. David Wunder, Engineering, Calvin College

Jake would like to thank his fiancé, Becky Marras, for all of her patience and encouragement. Christ is a sure foundation.
2. Introduction

Calvin College is a Christian liberal arts college in Grand Rapids, Michigan affiliated with the Christian Reformed Church in North America. Figure 6 displays an aerial image of Calvin’s campus. Calvin has an engineering program with approximately 350 students. Part of the engineering curriculum is a senior design project which is completed over the course of the entire senior year. The senior design project is meant to challenge senior engineers to use all skills and abilities they have learned during four years of engineering study on an engineering project which is conducted in a manner similar to projects in industry. This report serves as the final report and pilot study of senior design Team 10, Whiskey Creek.

Figure 6: Aerial View of Calvin College.
2.1. Team 10

Pictured from left to right in Figure 7 are the members of Team 10; Nick Keegstra, Jake Parks, and Lukas Vander Linden.

![Team 10 Image]

Figure 7: Team 10.

Nick is a senior civil/environmental engineering student from Grand Rapids, Michigan. Over the past three years Nick has worked for Calvin College’s physical plant. In his spare time there is nothing he enjoys more than fishing. Nick’s enthusiasm for this project stems from his love for the outdoors which has been instilled in him since childhood. Nick is currently searching for a job in the civil engineering field.

Jake is a senior civil/environmental engineering student from the small town of Carlisle, Ohio. For two summers Jake worked at the Warren County Engineer’s Office. He created cost estimates, mapped the county highway culvert system, and collected data for bridge calculations. After getting married in July, Jake will pursue his passion for ministry by studying at Western Theological Seminary in Holland, Michigan.

Lukas is a senior civil/environmental engineering student from Grand Rapids, Michigan. Last summer he interned at RMD Architects where he created preliminary site layouts, drafted structural details, and did code work. Lukas enjoys sports and has been a member of both the cross country and track teams while at Calvin. He will continue his education next year at the University of Michigan where he will be working toward a Master’s of Science in Civil Engineering.

2.2. Whiskey and Plaster Creeks

Whiskey Creek is a tributary of Plaster Creek, a large watershed in metropolitan Grand Rapids, consisting of approximately 58 square miles of land. The stream’s headwaters are just south of the unincorporated town of Dutton. Plaster Creek is a heavily polluted stream which flows through the cities of Kentwood and Grand Rapids before discharging into the Grand River. Plaster Creek also suffers from streambank
erosion and flooding due to the increase in runoff caused by the continuing development of the southern Grand Rapids metro area.\textsuperscript{iii}

Whiskey Creek begins in a marshy area on the eastern side of Calvin’s ecosystem preserve. Whiskey Creek flows from the ecosystem preserve underneath the East Beltline through a pipe and then flows underneath the FACLot until discharging into a detention basin northeast of the Seminary Pond. The creek then flows from the detention basin to the Seminary Pond, and from there it discharges underneath Burton St. Figure 8 shows the delineated basins and major reaches of Whiskey Creek while Figure 9 gives a schematic for Whiskey Creek where triangles represent drainage areas, and rectangles represent ponds. Figure 10 identifies the names of the major on campus ponds which Whiskey Creek flows through.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{whiskey_creek_map.png}
\caption{Whiskey Creek on Campus. Sub basins are outlined and numbered in red. Major reaches of Whiskey Creek are highlighted and numbered in blue.}
\end{figure}
Figure 9: Whiskey Creek Schematic. Each rectangle in the schematic represents one of the five ponds on Calvin’s campus which Whiskey Creek flows through. Each triangle represents one of the major sub basins identified in Figure 8.

Figure 10: Major Ponds Within Watershed.
2.3. Problem Overview

The Whiskey Creek channel has been heavily altered since the 1960s when Calvin’s current campus was the Knollcrest Farm. Instead of an overland creek freely flowing into the Seminary Pond, much of Whiskey Creek is now piped underneath the campus. Figure 11 shows the Knollcrest Farm before it was developed into the Calvin College campusiv. Note all of the vegetated space and the natural contours of the land.

![Figure 11: Aerial image of the Knollcrest Farm.](image)

The objective of this project is to create as much infiltration as possible so that Calvin’s campus only produces as much runoff as when it was not developed. Calvin’s campus most likely was a forested area in its natural state. Heavily wooded areas can create very little runoff. As a result of campus development, the runoff volume created by the land is dramatically higher than its undeveloped state. Due to large parking lots and lawn care practices, Calvin’s runoff also contains pollution. It was the team’s goal to implement LID BMPs in order to reduce the total runoff volume created by campus and to reduce the pollution load in the runoff.

2.4. Project History

The idea for this project originated from Prof. Randy Van Dragt of the Biology Department. Prof. Van Dragt is one of the professors in charge of the Ecosystem Preserve, a 90ac site on the east side of campus dedicated to preserving native Michigan ecology. Prof. Van Dragt contacted the Calvin Engineering Department about an erosion problem on one of the Ecosystem Preserve trails which is caused by
stormwater runoff. The problem is located where the trail passes by the southeast corner of the EB Lot. During storm events, the entire parking lot drains to the southeast corner of the lot, and washes out the trail due to a triple pipe culvert which is insufficient to convey the runoff from the EB Lot. The EB Lot is also a major source of pollution due to its heavy use. Because no stormwater detention or treatment methods are used to slow the flow of water discharging from the parking lot, all of the pollution is conveyed to Whiskey Creek in the runoff water. Figure 12 is an image of the runoff problem location on the nature trail. Note the exposed length of pipe which is caused by the runoff washing out the trail.

2.5. LID BMPs

It is important to note that LID and BMPs are not the same thing. LID is a specific kind of BMP. BMPs are stormwater management methods that prevent the movement of pollutants, prevent degradation of soil and water resources, and are compatible with the land use. In other words, BMPs are methods of minimizing stormwater pollution level and quantity increases caused by changes in land use. Human development changes the land use of a given area from natural to industrialized, often increasing the imperviousness of the area. The additional impervious surface area found in developed areas increases runoff volume and also increases pollution concentrations in runoff. BMPs seek to minimize the effects of increasing the impervious area.

LID is a specific method of achieving the goals of BMPs. LID is a relatively recent trend in stormwater management that uses infiltration to reduce pollution levels, but also to reduce total discharged volume.
The basic principle of LID is to restore land to its predevelopment hydrology. Restoring natural hydrology is achieved by infiltrating, filtering, storing, evaporating, and detaining runoff close to its source.

2.6. Project Objectives

As Christians, we believe that Christ has invited us to participate in his work of redemption, for it is through Christ that God is reconciling the world to himself (II Corinthians 5:19). Part of this work is stewardship. Because of this, the team is motivated to reduce the total volume of runoff created by Calvin’s campus and to improve the quality of the runoff. Whiskey Creek eventually flows into Plaster Creek which is a degraded urban stream. If all communities within the Plaster Creek watershed implemented LID, then Plaster Creek would be dramatically remediated. This project also serves as a pilot study that outlines how institutions, developments, and communities can implement LID BMPs to reduce the effects of development on runoff volume and quality.

It is the team’s goal to identify which LID BMPs will perform best on campus, and to select locations for these BMPs in order to maximize their impact on runoff volume and quality. The team designed each of the BMPs and included drawings and cost estimates as part of this report. These designs and cost estimates should aid the readers of this report in understanding how BMPs could be implemented on their site or in their community.

This project also serves as a form of information collection. When the team began this project, little information about Whiskey Creek was available, especially on the west side of campus. Therefore, the computer models the team designed, the maps the team collected, and the delineation the team did will all serve as an information base for Whiskey Creek on the Calvin campus.

3. Stormwater Management

Stormwater is precipitation that does not infiltrate into the ground, but rather runs off the surface it impacts. It easily is formed on impervious surfaces such as roofs and roads, and more difficult to generate in fields and forests. Stormwater is a natural phenomenon but it can also pose significant problems in urban areas due to the increased volume of stormwater created by impervious areas and due to the increase pollution in runoff caused by cars, sediment, fertilizers, and fertilizers.

3.1. History

The earliest form of stormwater management was conveyance. Runoff water is unwanted and causes flooding damage to buildings if it is not properly drained. So the first method of stormwater management was to simply route it into channels and/or pipes and connect those conduits to the nearest body of water via the shortest route possible. This worked for a while before intense urbanization. Figure 13 shows the impervious cover in America.
All of that impervious area generates runoff on land that once produced little runoff, especially in areas such as the eastern Midwest, which is relatively flat land that used to be well forested. Farm land and range land also creates more runoff than natural grassland. This dramatic increase in runoff creates flooding problems. After the Clean Water Act was signed into law in 1972, developers and communities began to detain runoff in detention ponds where it could then slowly be released into streams. The main goal of a detention pond is to reduce the peak discharge of runoff. A reduction in peak discharge limits the amount of runoff flowing through a stream at a given time, thereby reducing its floodstage.

Detaining stormwater can help resolve flooding problems, but it does a bad job solving other problems like pollution and streambank erosion. Those problems are associated with the total volume of runoff. A typical detention pond limits peak discharge, but it still allows the same amount of water to drain offsite as before there was a detention basin. LID is the newest movement in stormwater management. Its purpose is to reduce the total volume of runoff and to improve the quality of runoff by using practices that store, filter, infiltrate, evaporate, and detain stormwater close to its source. LID techniques are applicable to many sites considering that they can be customized to meet many requirements. The ultimate goal of LID is for sites to return to their presettlement hydrology. LID is usually achieved through the use of cost effective landscape features known as BMPs. These BMPs strive to replicate the natural water cycle by minimizing the total runoff, controlling the peak discharge rate of runoff, maximizing infiltration, maintaining the base flow of streams, maximizing evapotranspiration, and protecting water quality. Infiltration practices are proven very effective at removing pollutants even more so than other BMPs. That is why the BMP plan associated with this report contains many structural BMPs that implement infiltration.

Infiltration of stormwater is at the heart of most LID BMPs because most precipitation soaks into the ground in unsettled areas, so any BMPs that utilize infiltration greatly aid in restoring natural hydrology.
Infiltrating stormwater recharges groundwater which aids in plant growth and also recharges underground aquifers. It naturally filters out pollutants, preventing them from entering waterways. Because infiltration practices recharge the water table, they also help to balance out the water cycle.

Stormwater management is the job of well-trained engineers and it is up to them to manage our water resources responsibly. One goal that the United States has through the Clean Water Act is “to restore and maintain the chemical, physical, and biological integrity of the Nation’s waters.” This would be an ideal condition for our country and beneficial in innumerable ways. The trend toward LID stormwater management is good because God calls us to be faithful stewards of his creation. That includes the water resources we all depend on.

3.2. Stormwater Problems

Many of the problems that are caused by stormwater affect streams and waterways. Some of these problems include pollution, flooding, sedimentation, erosion, and temperature change in streams. Anyone can observe pollution in streams by standing by the shore and observing the pieces of litter washed up on shore. Most of this litter, or debris, was not there originally but transported through a stormsewer system from parking lots and roadways until it reached that location. But the most subversive pollutants are the ones that cannot be seen. Other forms of pollution besides trash debris include things we cannot see such as nitrates, phosphates, oil, suspended solids, and biomass. Pollutants such as these affect the ecosystem, the stability of the stream, and public health. All of these pollutants build up in developed areas and are washed off the surface during rain events into streams. Plaster Creek has been notorious for being the most polluted stream in Grand Rapids because it drains much of the land within the city and its major suburbs. According to the Department of Environmental Quality’s Plaster Creek Watershed Management Plan, Plaster Creek has an impaired quality of indigenous aquatic life, impaired functionality as a water fishery, is threatened for use in partial body recreation, impaired for use in total body recreation, and only is truly approved for use in agricultural practices.

Flooding is one of the first issues that must be addressed by stormwater management. It can cause extensive property damage and can temporarily make roadways impassable. Flooding in itself can easily be solved by the proper implementation of detention basins throughout a watershed. Flooding is caused by continued development and urbanization. Most likely, 40 years ago, Plaster Creek did not have any flooding problems. Since that time, though, many developments have been constructed in Kentwood and southeast Grand Rapids which have greatly added to the amount of runoff flowing through a channel that is the same size as when it was undeveloped. Presently there are some problems in the Whiskey Creek drainage basin as well. For example there is some minor flooding at 2041 Raybrook. During times of heavy rain the lower parking area fills up with water to the point where it starts to flow into the lower entryway. Reducing the peak flows in Whiskey Creek could prevent many of these occurrences.

Stream channel degradation is another problem that must be addressed by stormwater management. Erosion is the breakdown of a healthy streambank or streambed on a stream or waterway. Erosion in most circumstances is caused by an increase in water flowing down a stream channel as a result of stormwater runoff. It poses a problem to streams because it increases the amount of sediment carried in the flow. Sediment is problematic for streams because it can clog interstitial spaces in streams that could otherwise have served as habitat for aquatic life. If it gets bad enough it can even clog fish gills and throw off the entire food chain of a stream. It also poses a problem because it angers people when property lines shift as a result of erosion. Stream channel degradation is apparent in Plaster Creek. Erosion problems in Ken-O-Sha Park are just one apparent example of degradation in Plaster Creek.
4. Project Management

Project management is essential when executing a yearlong project such as a senior design project. For that reason the team made an effort to apply principals of project management. The team kept track of all time spent working on the project. The team used Microsoft Project 2010 to create a Gantt chart of proposed deadlines and to record actual completion dates. All important resources given to the team from consulting professors and professionals, such as a soil survey map, a map of the Plaster Creek watershed, and an explanation of sub basin delineation were stored in binders for future reference. Additionally, the team used NetStorage extensively to save documents, CAD drawings, pictures, notes, and more because it enabled the whole team to have access to all of these materials anyplace at any time.

Because of the nature of the senior design course, project teams were required to divide work up amongst themselves with little to no supervision from the professors. Team 10 met every Saturday morning to organize, to set deadlines, and to work on the project as a group. Since the team only had three members, decision making was usually done by unanimous decision. Jake Parks provided project management, so he divided the work amongst the team members and set deadlines for the work to be completed. Jake was also responsible for creating the volume model and pollution model. In addition to writing report sections and delineation/modeling work, Nick Keegstra was in charge of bioretention design and green roof design. Lukas Vander Linden’s role involved creating and managing the team’s website, designing pervious pavement BMPs and designing the area for native revegetation.

The project was divided into two parts corresponding to the two semesters of the senior design course. In the fall semester of 2011, the team created a hydrologic model, selected feasible BMPs, and selected possible BMP locations. During the fall semester, the main deliverable was the project feasibility study. In the spring semester of 2012, the team created a pollution model, created a volume model, revised the BMP plan, designed the selected BMPs, and created cost estimates. The flow of work for the spring semester is outlined in Figure 14.

5. Infiltration Rates

One of the key factors involved in LID is the infiltration rate of the soil, or how fast water is allowed to soak into the ground. The infiltration rate affects every design aspect of LID BMPs: from the depth of the reservoir, to the plan view area, to the overflow plan. With a high infiltration rate, BMPs have a smaller plan view area, and therefore lower cost.

![Figure 14: Gantt Chart for second semester](image-url)
One major indicator of infiltration rates is the type of soil present in a given area. Soils can belong to hydrologic soil groups A, B, C, or D where A generally infiltrates the most water and D infiltrates the least. According to a Michigan soil survey map the soil types at Calvin College were not generally good belonging to the hydrologic soil group C. To confirm this data the team decided to conduct an infiltration test.

5.1. Infiltration Test

The team conducted an infiltration test near the southeast corner of the EBL because this area’s infiltration rate will be representative of the infiltration rate for both the pervious pavement solution and the bioretention solution for the EBL. The team could not conduct tests anywhere else on campus because Physical Plant probably would not give permission to test infiltration rates on the academic area of campus. So an average value of 0.32”/hr was assumed for the west side of campus based on the soil types present.

The test, shown in Figure 15, was conducted by digging a hole into the most impervious layer of soil. Then, an impervious barrier was place along the walls of the hole to prevent water from infiltrating through the sides of the hole. The team used a 5 gallon bucket with the bottom cut out as the impervious layer. For soils infiltration rates to be accurate, the soil must be saturated, so the team filled the bucket but did not start testing until the infiltration of the water had stabilized. Had the team started testing as soon as water was placed in the bucket, the initial infiltration rates would have been much higher than the actual infiltration rates because dry soil infiltrates water fast, but as the soil saturates itself infiltration rated decrease and become steadier. The team then collected data for ten hours, and also collected two long term data points the next day.

5.2. Infiltration Testing Results

The LID manual states that infiltrations are ideal if they are above 1 in/hr. Unfortunately for the team, there is a good size layer of mainly clay so the infiltration rates were much lower than ideal. The results of the infiltration test are shown in Figure 16.
Figure 16: Infiltration test results.

As shown above, the infiltration rate for this portion of campus is about 0.21 in/hr. This value is below the minimum infiltration rate allowed for a system with no underdrains, so the team designed underdrains for both solutions for the EB Lot.
6. Feasible LID BMPs

In order to create a BMP plan, various alternatives were identified as possible BMPs which could be implemented in the plan. Not all of these alternatives were applicable for Calvin’s campus, but all of them could be implemented in a developed setting. These feasible BMPs include pervious pavement, bioretention, infiltration trenches, vegetated filter strips, vegetated roofs, vegetated swales, baffle boxes, subsurface infiltration beds, and dry wells.

6.1. Pervious Pavement

Pervious pavement could be implemented across campus. Any parking lot, sidewalk, or courtyard could be renovated as a pervious surface. Because pervious pavement performs well independent of the pollution load, parking lots with a high car concentration will be targeted as possible locations for pervious pavement. The FAC Lot and the SC Lot are two potential locations for pervious pavement because they have a large area and an elevated buildup of pollution due to their frequent use. Figure 17 shows a cross section of pervious pavement.\[14\]

![Figure 17: Pervious pavement cross section.](image-url)
6.2. Bioretention (Rain Gardens)

Rain gardens are lower elevation areas that are planted with native plant species that are specifically selected for their ability to capture and treat stormwater runoff. Bioretention is a method for treating stormwater runoff that lets runoff pool in a vegetated area and then gradually infiltrate. By letting the runoff infiltrate, this method reduces peak discharge, reduces stormwater temperature, and filters out suspended solids and pollutants. Figure 18 shows a typical bioretention basin cross section.\textsuperscript{xiii} The suggested maximum ratio of treated area to treatment area is 5:1 (one fifth of an acre of bioretention is needed for every acre of catchment).\textsuperscript{xiv} The suggested maximum treatment area is one acre.

Bioretention has many different applications, but is somewhat area intensive, making it difficult to implement on a developed college campus. One possible location for bioretention is the FAC Lot islands. This area was selected because it provides lots of small areas that would be ideal for bioretention. Each island located in this parking lot could be redeveloped as a rain garden. This would allow the 5:1 ratio between treated area and treatment area to hold as well as limiting the maximum impervious area for each island to under one acre.
6.3. Infiltration Trench

Infiltration trenches are specifically designed to capture runoff and let it percolate into the surrounding soil. These trenches are usually three to twelve feet in depth and lined with a filter fabric that is filled with gravel. Infiltration trenches can be located in the median of highways, and between parking lot isles because they are thin and can be fitted into many different areas. Infiltration trenches however, require pretreatment to perform to their potential. Pretreatment can vary from grit chambers to filter strips. Figure 19 shows a structural soil infiltration trench; trenches can vary in size and in media grain size.

Figure 19: Example of an infiltration trench.

The SC Lot is a suitable location for an infiltration trench. This site was chosen because it is composed of long driving aisles that would be ideal for infiltration trenches. These trenches could be located along the edge of the parking lot, or between the isles. An infiltration trench could also be constructed bordering Knollcrest Circle between the FAC and the Boer Bennink Dormitory. This length of road has a grassy strip along most of its length. The grass could be replaced with an infiltration trench.
6.4. Vegetated Filter Strip

A vegetated filter strip is a sloped portion of land covered with vegetation which is used to slow runoff, infiltrate water, and remove pollutants. Vegetated filter strips are cost effective, they reduce pollutant loads. Figure 20 shows a typical vegetated filter cross section.

The grassy slope between Knollcrest Circle and the Seminary Pond is a potential site for a vegetated filter strip. This area is already sloped away from the road, so would require little regrading. The only work involved would be in planting vegetation along the hillside and in creating a level spreader for the flow.
6.5. Vegetated Roof

A vegetated roof is a rooftop fitted with a layer of vegetation that decreases runoff volume. The vegetation is usually between 4 and 6 in thick and is separated from the roof of the building with a waterproof material. The benefits of a vegetated roof include runoff volume control, temperature reduction, and aesthetic appeal. Figure 21 shows a schematic of a typical vegetated roof. However, for a vegetated roof to be possible, the roof must have slope of 2:12. They are also rather costly.

Most buildings on Calvin’s campus have a slope less than 2:12, so there are many areas where vegetated roofs may be implemented based on the slope of the roof. Unfortunately, vegetated roofs require the building to be able to support the extra weight. At this time the team does not know which, if any, buildings on Calvin’s campus would be able to support this extra weight.

Figure 21: Vegetated roof.
6.6. Vegetated Swale

A vegetated swale is a shallow earthen stormwater channel that is designed to infiltrate, and filter stormwater. Vegetated swales have a high density of plants which slows and filters stormwater. Figure 22 shows a rendering of a vegetated swale.

Because there are already some daylighted portions of Whiskey Creek, these are obvious choices to implement vegetated swales. The portion of Whiskey Creek between the FAC Lot and the Seminary Pond already has many aspects of a vegetated swale; it is daylighted, gently sloped, and is heavily vegetated with grasses, shrubs and trees. Implementing vegetated swales in this portion of Whiskey Creek would require some work, but would be relatively easy compared with implementing vegetated swales along other portions of the creek.

Figure 22: Vegetated swale.
6.7. Baffle Boxes

There are two major complaints that originate from constructing BMPs; cost and land availability. Baffle boxes remedy both these problems by providing an in-line sediment removal trap that is also cost effective. Baffle boxes have permeable fabrics that filter out trash and a series of concrete weirs which cause sediment to accumulate in the bottom of the box (metals and other suspended solids are removed with the sediment). Figure 23 shows the interior of a baffle box in detail.

Because the Seminary Pond is so close to Burton, there is very little room to implement BMPs with large space requirements. For this reason a baffle box, which have almost no space requirement, would be a reasonable BMP to treat the Seminary Pond water which runs over the controlling weir during storm events.
6.8. Dry Wells

A dry well is a LID infiltration practice that requires no pretreatment. This makes it an ideal BMP for a surface such as a roof which does not accumulate much sediment or particles. Dry wells can come in two forms. They can be large holes cut into the ground and then filled with rock and lined with a geotextile. They can also be large concrete cylinders with holes in the sides and an open bottom to allow for infiltration. Dry wells are designed to have an overdrain in the event of a rain event exceeding the design storm. A typical cross section of a dry well can be seen in Figure 24.

![Figure 24: Typical dry well construction.](image)
6.9. Native Vegetation

Native vegetation is a widely applicable LID BMP that can reduce runoff volume, and reduce pollution loading by increasing uptake, detention storage, and evapotranspiration. The main idea behind native vegetation is to replace turf grass with native species. Turf grass has a small root cluster and small leaves that do not allow it to take up, or detain much rain water. Native plants have both large root clusters and larger leaves to allow for uptake, detention storage, and evapotranspiration. Native plants also improve soil conditions, sequester carbon, and enhance infiltration. Native vegetation can be implemented anywhere there is currently turf grass, but areas like the common’s lawn would be hard to vegetate because it is often used as a recreational area. Areas that would be better suited for native vegetation would be the grassy areas around the daylighted portion of Whiskey creek just east of the Seminary Pond, the grassy patch just west of the Spoelhof College Center, any portion of campus that regularly gets flooded like the area in between the wings of the Heinz and Timmer dorms, and any roadside grassy areas like in between the two roads at the main entrance of the campus. Figure 25 shows natural vegetation and how a landscape can look good while providing an important function.

Figure 25: Natural vegetation
6.10. **Subsurface Infiltration**

A subsurface infiltration basin is a series of large perforated pipes which is placed underground. When water flows into these pipes it infiltrates into the ground via the holes in the pipes. The large diameter of the pipes allows for storage for an entire 2 year statistical storm or for storage of larger rain events. Subsurface infiltration basins are commonly constructed underneath large fields, parking lots, or any other open surface. For that reason, Commons Lawn and the DeVos Field are both potential locations for a subsurface infiltration basin. Figure 26 shows a subsurface infiltration bed in construction.

![Figure 26: Installation of a subsurface infiltration bed](image)

7. **Campus Wide Design Plan**

Ultimately, not every feasible LID BMP that the team evaluated could actually be designed. Designing this many LID BMPs is out of the scope of this project, and some would not be nearly as efficient at removing pollution and reducing volume as others. For this reason five different LID BMPs located at six different places across campus were designed as part of the campus wide design plan. Each LID BMP and location was selected based on land availability, cost, efficiency, and importance of location. Small BMPs receiving water from a small area on campus were avoided because those types of BMPs do not have the same level of impact on runoff quality as a large BMP. Also, BMPs were selected for locations with potentially high pollution levels. Not all LID BMPs that were deemed feasible in the previous section were designed. The campus wide design plan LID BMPs are shown in Figure 27.
Figure 27: Campus wide LID BMP locations.
7.1. Fine Arts Center Parking Lot – Pervious Pavement

The Fine Arts Center Lot is a large lot, therefore it is a major contributor of pollution and it produces large runoff volumes. Pervious pavement would remedy both of these problems at the same time by infiltrating runoff volume into the ground while also filtering out pollution. Although this is a costly LID BMP to implement, it will result in significant volume and pollution reduction. The team selected bioretention in the islands of the FAC Lot as a possible BMP, but this would also require regrading of the lot. Bioretention may be cheaper by itself, but because the college would have to regrade the entire lot, bioretention would be cost prohibitive. A simpler and cheaper option was to design a pervious pavement solution. While this would still require the lot to be redone, it would not carry the extra cost of retrofitting all the islands found in this lot with bioretention cells.

Fine Arts Center Parking Lot Pervious Pavement Design

Like all the BMPs designed on campus, the FAC Lot was designed to infiltrate a 2-year statistical storm, while also providing conveyance for larger storms, such as a 25-year statistical storm. For the FAC Lot, not only will the lot infiltrate water generated on the lot itself, which has 114,000\(\text{ft}^2\) of impervious area, but the lot will also infiltrate the stormwater generated on the roof of the Fine Arts Center, which has 60,000\(\text{ft}^2\) of impervious area. Figure 28: Plan view of the FAC lot pervious pavement design, shows both the lot that the pervious pavement is designed for, and it also shows the FAC roof that will generate and discharge runoff to the pervious pavement.

![Plan view of the FAC lot pervious pavement design](image)

Figure 28: Plan view of the FAC lot pervious pavement design
The 2-year 24-hour storm for West Michigan is 2.37in and the infiltration rate for the west side of campus was estimated to be 0.32in/hr. In order to infiltrate this storm event with the given infiltration rate, the reservoir layer of the pervious pavement must be 8.2in deep. For a 25-year statistical storm, 4.45in of rainfall in 24 hours, the reservoir layer must be 16.2in deep, this number was rounded up to 17in. A typical section cut of the pervious pavement solution is shown in Figure 29.

![Figure 29: Typical section for the FAC parking lot pervious pavement solution.](image)

In the case of the Fine Arts Center, the pervious pavement will be built around the existing stormwater management system of catch basins and manholes in order to provide overflow for the pervious pavement and an acceptable drainage time. Overflow would be needed in the case of a 50-year statistical storm that the pervious pavement is not capable of handling. The runoff volume that the pervious pavement is not capable of infiltrating or detaining would be allowed to flow into the existing catch basins and would then be conveyed off site. A typical overflow detail for the FAC Lot pervious pavement is shown in Figure 30.

![Figure 30: Typical overflow detail for the FAC pervious pavement solution.](image)

According to the LID manual, all BMPs must be capable of handling back-to-back storms, so the maximum time to drain the reservoir layer is 72 hours. Because the infiltration rates on campus may vary, a time of 48 hours was used to determine the amount of runoff that can drain from the reservoir layer. Using 48 hours and the infiltration rate of 0.32in/hr, the maximum depth of the reservoir layer is 19.2in, so the calculated reservoir depth of 16.2in is acceptable. Although our calculated reservoir depth is safe according to drainage times, the existing manhole wall will be retrofitted to allow for drainage above 8in in depth. These perforations will allow for extra drainage around the catch basins and
manholes in the event of a storm above the 2-year statistical limit. These perforations are above the 2-year statistical storm level of the reservoir bed, so the entire 2-year statistical storm will still be infiltrated.

In order to implement pervious pavement in the FAC Lot, the existing pavement will need to be removed and a depth of 21.5in will need to be excavated. The excavation must be done without compacting the bed bottom to allow for desired infiltration rates. The excavation will also need to be done so that the bed bottom is flat to allow for even distribution of the stormwater. Directly on top of the uncompacted bed bottom, a non-woven geotextile should be installed to prevent sediment from mixing with the reservoir layer. A 1'-5” layer of American Association of State Highway Transportation Officials (AASHTO) No. 2 aggregate will be placed on the geotextile. Above the reservoir layer is a 1in choker course comprised of AASHTO No. 57 stone. This layer provides enough void space to pass water, but is also capable of holding up the pervious pavement layer itself. The pervious asphalt layer is to be 2.5in thick and constructed and laid the same way as normal pervious pavement except for the fact that no fine aggregate is added to the asphalt mix. This creates the holes that are necessary to infiltrate water.

7.2. North Section of the Southwest Commuters Lot – Pervious Pavement

The Southwest Commuter’s Lot (SC Lot) is very similar to the FAC parking lot in that it is a very large parking lot. This lot also has islands that may be feasible for bioretention except that the lot would need to be completely regraded to redirect water towards these islands instead of into the stormwater system.

Southwest Commuter’s Lot Pervious Pavement Design

Because the SC Lot is very similar to the FAC parking lot, the designs are very similar as well. The same storms and infiltration rates were used to design the depth of the reservoir layer, but the SC Lot would not need to infiltrate extra runoff like the FAC parking lot. Because there are no large structures located around the SC Lot, there will be no added runoff volume that the pervious pavement solution would have to infiltrate. Figure 31 shows the area to be paved with pervious pavement, about 250,000 ft².
The necessary reservoir lot depth of this lot was calculated to be 5.1in for a 2-year storm, and 10in for a 25-year storm. The LID manual recommends that no pervious pavement solution be shallower than 1ft, so the depth of the reservoir layer was increased to 1ft. This also provides extra detention for larger storm events. Because the infiltration rate is 0.32in/hr, the allowable depth of the reservoir layer based on the maximum drainage time is 19.2in, so the 1ft depth is acceptable, and will drain within 48 hours. A typical section is shown in Figure 32, and a typical overflow detail is shown in Figure 33.

**Figure 31**: Plan view of the southwest commuter’s lot pervious pavement design.

**Figure 32**: Typical section for the southwest commuter’s lot pervious pavement solution.
The materials required for the SC Lot pervious pavement are almost identical to the FAC Lot pervious pavement design. Excavation must be done about 16” deep and uniformly graded with a zero percent slope to allow for even drainage. A nonwoven geotextile must act as a barrier between the uncompacted bed bottom and the reservoir layer. For the SC Lot, a depth of 1’ is acceptable for the AASHTO No. 2 aggregate reservoir layer. This shallower depth is acceptable because no extra water has to be taken into consideration; no roof runoff is draining to the pervious pavement. The choker course and pervious pavement layer is the same as the FAC lot; 1” AASHTO No. 57 aggregate and 2.5” pervious asphalt.

7.3. South Section of the Southwest Commuter’s Lot – Bioretention

The south section of the SC Lot was designed differently because the lot is not graded in a traditional manner. The southernmost islands in this lot are inverted islands, meaning that they are designed to collect stormwater. These islands are retrofitted with storm drains so that this collected stormwater can be directed into the storm sewer system. These inverted islands are major problems for physical plant because they often collect debris and clog. These islands require frequent maintenance, and are expensive to maintain. When clogging occurs during a rain event, flooding is common. Flooding can be extreme enough to reduce the effectiveness of the lot, or to render this portion of the lot unusable. Figure 34 shows the clogging that can be caused by these inverted islands.

Figure 33: Typical overflow detail for the southwest commuter’s lot pervious pavement solution.
Unlike the north portion of this lot, the south portion is ideal for retrofitting these inverted islands into bioretention cells. Since the lot is already graded to direct water towards the islands, all that would need to be done is to retrofit the islands. This would require constructing a gravel reservoir below a planted surface along with minor modifications of the current stormwater system.

**Southwest Commuter’s Lot Bioretention Design**

The southern end of the SC Lot is unique in that it has inverted islands. The pavement here is sloped to direct runoff to flow into these inverted islands where a proposed bioretention area is located. The soils are type C with an assumed drainage of 0.32in/hr. A safety factor of 2 was used in the design of these bioretention areas so the infiltration rate the final product was designed for is 0.16in/hr. The layout encompasses all of the space within the existing 3 islands in the parking area plus a little bit more area so that there is enough room for infiltration within 72 hours. The proposed layout for bioretention in this spot is shown in Figure 35.
Two of the existing islands in the parking lot have catch basin inlets. Therefore two inlets were incorporated into the bioretention design as overdrains. The manholes themselves may need to be resealed so that no water gets into them from the soil adjacent to it. The bioretention areas would take up a total of 8,100ft$^2$ of the SC Lot and be excavated down to an elevation of 759ft. A total of 53,800ft$^3$ would need to be removed and replaced for this area. An underdrain would be necessary because of poor infiltration rates and should be placed at the top of the infiltration layer. 2.75ft of well sorted rock and gravel should be used for an infiltration layer off the bottom of the excavated area and 4ft of sandy soil should be used as a planting medium on top. Hardy vegetation should be planted on top of the bioretention area. A maximum ponding depth of 6in is designed into this bioretention area because the overdrain is 6in above the ground elevation in the bioretention area. In this area’s current configuration no water would be expelled into the stormsewer network that the overdrains are connected to unless a rain event exceeding a 2 year storm were to occur. The materials that make up this cell are a gravel infiltration reservoir and a sandy soil growth layer. These layers are shown in a representative cross section of this bioretention basin in Figure 36.
Figure 36: Representative cross section of SC Lot bioretention basin

7.4. North of the Library – Bioretention

Although this is a smaller application of LID BMPs, it is still highly efficient. The entire commons lawn drains southeast towards the library and gets stagnant around this low spot. Bioretention would use the lands natural slope to collect runoff from commons lawn. This bioretention cell would also allow for Calvin to educate students, and visitors about the green initiative.

Commons Lawn Bioretention Design

In order to treat the excess water generated by the Commons Lawn, a bioretention basin is proposed in place of a storm sewer inlet. The layout of this area is show in Figure 37. The total area that this bioretention basin would take up is 3,250ft² and it would fit in between the existing paths as shown in Figure 37. There would need to be grading done to make the area flat allow stormwater to spread evenly across its surface. The bioretention basin would be a total of 6.65ft deep and require about 21,650ft³ of excavation. A rock infiltration bed of 2.65ft would be on the bottom with an underdrain and 4ft of sandy soil would be on top of that. A new manhole would not be required but the underdrain should tie in with the manhole southeast of the bioretention area to allow for drainage and for maintenance of the underdrain. A cross section detailing the layers of this design are shown in Figure 38.
Figure 37: Proposed layout for commons lawn bioretention

B-B Cross Section

Figure 38: Commons lawn representative bioretention section
7.5. East Beltline Strip– Native Revegetation

Any rain that falls on the strip of turf grass between the East Beltline and Knollcrest Circle runs off very quickly due to the slope of the land. Turf grass does not allow for much infiltration and when it is coupled with a steep slope, almost all the water runs off. This water eventually flows onto Knollcrest Circle and into the stormsewer system.

*East Beltline Strip Native Revegetation Design*

Native revegetation is the simplest BMP that the team designed. The only calculations that were done were to find the plant density based on a given separation. Plants are to be planted with 5ft spacing between plants, this gives a density of $0.09\text{plants/ft}^2$. This plant density holds for all the area being replanted, shown in Figure 39. The area being planted is 3.3ac. This area and the calculated density of plants result in 12750 total plants required.
The southernmost portion of the native revegetation solution is located within a depression in the topography where Whiskey Creek flows through an open channel. Because of this open channel, different plants are required for this location because of different depths to the water table. Figure 40 shows the locations of different kinds of plants. The darker areas represent areas to be planted with vegetation that is capable of growing closer to the water table, while lighter areas are areas that can be plated with vegetation that can grow higher above the water table.
7.6. Hiemenga Hall and Hekman Library – Vegetated Roof

Vegetated roofs are especially applicable on Calvin’s campus due to all the flat roofs on campus. The team evaluated all flat roofs on campus including the Science Building, North Hall, Devries Hall, Hiemenga Hall, and the Hekman Library. Devries and North Halls were rejected due to a study done that rejects the possibility of constructing a third floor on these buildings, so Hiemenga Hall and the Hekman Library were selected to consolidate construction to one, continuous building. It is out of the scope of this project to verify the structural capabilities of these buildings.

Green Roof Design

An extensive vegetated roof, or green roof, is proposed to be installed on top of the Hekman Library and Hiemenga Hall building complex. Extensive meaning that it requires little to no maintenance once it has been constructed. Its proposed design would utilize a sedum mat system. Sedum is a type of plant that is
drought resistant, holds a lot of water, and overall makes good vegetation to be placed in conjunction with a vegetated roof. This mat system uses premade and pre-grown mats of a specific size and shape that can be placed on most flat roof systems without the need for structural reinforcing. It would be capable of not only detaining a lot of stormwater in its growing medium, but it would also transpire it back into the atmosphere. A picture of layout of this green roof is shown in Figure 41.

Figure 41: Green roof layout

A green roof would involve making sure the roof is water tight and root resistant. For the purposes of this report the roof is considered to be watertight, root resistant, and structurally sound already. The reasons behind this decision are because flat roofs are typically built to handle heavier loads than angled roofs and roofs built in this era were constructed extra conservatively. The decision to use a sedum mat system would also significantly decrease the amount of labor needed to install it and keep the overall cost to a minimum. The benefits of a green roof at this location would be a significant reduction in the amount of stormwater flowing off these roofs. Roofs generate stormwater runoff proportional to their size and the total surface area that this green roof would cover is 74,130ft². Figure 42 shows a cross section of an extensive green roof.
Subsurface infiltration was selected to infiltrate the runoff from the roofs of North Hall and the Science Building. This area is a major source of stormwater runoff, but what is unique about this runoff is that it is generally not very polluted. Any runoff that is generated on rooftops is relatively clean, especially if the roof is above the canopy of trees, like the Science Building roof is. Subsurface infiltration was specifically selected because it is very efficient with runoff that is of high quality. This BMP is also easily hidden. Because the majority of the solution is underground, the above landscape can be maintained like a normal plot of land. For this reason the infiltration bed was located just east of the North Hall as shown in Figure 43. This allows the construction site to be replanted with turf grass after construction is complete. Construction of this BMP could begin in late May, and grass could be growing before school starts again in the fall. This solution is ideal because the runoff generated by these roofs is managed with French drains that could easily be retrofitted to flow towards our BMP. Because of this existing stormwater management system, an overflow system would be simple to construct as well because the existing stormsewer could be used to convey water downstream.
Figure 43: Plan view of contributing area and treatment area for subsurface infiltration.

Science Building and North Hall Subsurface Infiltration Design

The roofs of the Science Building and North Hall have a total area of 44,000 ft² and produce a total runoff volume of 8,690 ft³ for a 2-year 24-hour storm event. The design of this area started by determining the depth of water that could be drained in 72 hours; this depth was based on the infiltration rate of the soil, 0.32 in/hr, and was calculated to be 0.96 ft. This 0.96 ft depth is a depth of pure water that the soil can infiltrate in a 72 hour period, so the void ratio of the reservoir layer, 0.4, is used to determine the depth of the reservoir layer. The total depth of the reservoir layer was calculated to be 28.8 in. As shown in Figure 44, the reservoir layer is 29 in deep. This reservoir is then covered with topsoil that can be used for planting.
Figure 44: Typical section for subsurface infiltration.

A typical outlet detail is shown in Figure 45. The overflow outlet is placed just above the reservoir layer to allow the entire volume generated in a 2-year storm to infiltrate, while providing drainage for any larger storms. This overflow outlet should be connected to the existing stormwater system for conveyance downstream.

Figure 45: Outlet detail for subsurface infiltration.

Based on the allowable depth and the required volume storage, the plan view area necessary was calculated to be 9,052 ft². This required area is completely contained in the area enclosed by existing paths shown in Figure 46.
Figure 46 also shows the underdrain layout. Runoff generated on the roofs is collected in the French drains and is conveyed to the manhole on the northern side of the subsurface infiltration basin. This runoff then flows into the 18in diameter perforated PVC that traverses the northern side of the subsurface infiltration bed. After the runoff flows through the 18in pipe, it flows through 12in diameter perforated PVC pipes that spread the water out throughout the entire bed. These 1ft diameter pipes are separated by 10ft from center to center, and distribute the runoff throughout the system to allow for even infiltration.
7.8. Rejected LID BMPs

The team had originally selected several LID BMP sites that the team no longer considers viable options. These LID BMPs were all rejected for a number of reasons, and might still be feasible if the college was willing to put more time and effort into design and construction. The team rejected them because they would create extra work, not be as efficient, and cost more than the BMPs outlined above.

7.8.1. Southwest Commuter’s lot – Infiltration Trench

The infiltration trench located in the Southwest Commuter’s Lot (SC Lot) was suggested based partially on the shape of the lot itself. The lot is very narrow, and infiltration trenches would be able to fit with ease along, or within the lot as shown in Figure 47. The design was ultimately rejected because infiltration trenches that deal with stormwater usually have some form of pre-treatment. The SC Lot is surrounded by trees that drop debris on the lot. The team fears that this debris would clog the infiltration trenches in the same way it clogs the inverted islands present in the southern portion of the lot.

Figure 47: Location of the proposed infiltration trench.
7.8.2. North Seminary Ditch – Vegetated Filter Strip

The vegetated filter strip along Knollcrest Circle just north of the seminary ditch was proposed to deal with the runoff from Knollcrest Circle itself and the runoff from the sloped turf grass it would replace. This design is shown in Figure 48, but was rejected because very little of the runoff from Knollcrest Circle would reach this LID BMP. Also, there would not be a high volume generated on the sloped turf grass, and what runoff would be generated would be relatively pollution free. This runoff would also be directed into the vegetated area in the ditch, this would slow the runoff and allow some of it to infiltrate into the ground before discharging the remainder into the Seminary Pond.

Figure 48: Location of the proposed vegetated filter strip.
7.8.3. Seminary Pond Outlet – Baffle Box

A baffle box is still feasible for use on campus, but could be placed in many different locations. The location at the outlet of the Seminary Pond, shown in Figure 49, was rejected because it did not treat stormwater before it entered the Seminary Pond. A pond acts as a natural sedimentation basin, reducing the amount of pollution in the runoff, a baffle box would be more effective before entering a pond, not downstream of a pond. One benefit of increasing the quality of the stormwater discharged into Whiskey Creek is that it would increase the quality and appearance of the water present in the Seminary Pond. A better location for a baffle box would be before the split in the 36in pipe under the FAC Lot, or before discharge into the seminary pond. A solution involving a baffle box under the FAC Lot was not designed because pervious pavement will result in a much bigger effect on stormwater volume and quality. If the college decides not to pursue pervious pavement in this location, a baffle box would be the next best LID BMP for this area. A baffle box at the outlet of the Seminary Pond was also rejected because there is very little room in this area, and flow of Whiskey Creek is not confined to a pipe. The application of a baffle box is much easier when the flow of water is restricted to a pipe.

![Figure 49: Location of the proposed baffle box.](image-url)
7.8.4. Dorms – Dry Wells

Dry wells were originally selected to infiltrate the runoff from the dorms for two reasons. First, the roofs create about 17,000ft$^3$ of high quality runoff in a 2yr statistical storm. This is a significant amount of runoff which has been added to the runoff of campus since Calvin developed the Knollcrest Farm and it can easily be infiltrated with limited pretreatment due to its quality. Second, most of the Calvin dorms are drained by French drains. A French drain is a gravel filled trench that has a perforated pipe in its bottom. Water is allowed to run straight off of the roof and into the trench. Some of the water infiltrates into the ground, but most of it enters the pipe and flows downstream. These pipes can easily be connected to a pipe flowing to a dry well, making the installation of the dry wells simple since extensive reworking of the drainage systems will not have to be done. The dry wells were to be located near the French drain bordering each side of each dorm as shown in Figure 50, but area requirements were too great to implement this design.

Figure 50: Location of dry wells in dorms.
Dorm Dry Well Design

A typical Calvin dorm has a plan view area of approximately 24,000 ft$^2$. Divide by four to account for the number of trenches present for each dorm (one trench for each side of the two dorm wings) and each dry well has 6,000 ft$^2$ of impervious surface feeding into it. This results in a total runoff volume of 1,190 ft$^3$. Assuming the soil in the dorms area has an infiltration rate of 0.12 in/hr, the dry wells can drain up to 0.72 ft of water in 72 hr. Unfortunately, this design requires 1650 ft$^2$ of plan view area for each existing French drain to infiltrate 1,190 ft$^3$ of runoff. Each dorm has about four French drains, so this area is too large to be feasible around the dorms. This design was eliminated from our BMP plan, however, this is still a feasible BMP for other campuses and businesses that may not have as strict space requirements.

7.9. BMP Performance

Two major BMP performance concerns are winter effectiveness and pollutant removal efficiency. Because Calvin College is located in West Michigan which has a longer, colder winter than most of the United States, it is important that all selected BMPs are capable of performing during winter. According to the Southeast Michigan Council of Governments’ (SEMCOG) LID manual, all of the selected BMPs have a winter performance rating of medium or high. In an area with a high concentration of paved surfaces, suspended solids removal is important because of the oil and grease that is picked up by stormwater runoff as the runoff flows over parking lots and roads. Removing suspended solids from stormwater often will also remove other pollutants because dissolved pollutants will often adsorb to the surface of suspended solids. Therefore, total suspended solids (TSS) removal efficiency is the most important water quality indicator for the selected BMPs. All of the selected BMPs also have a total suspended solids removal efficiency rating of medium or high. Table 2 displays the winter performance and TSS removal efficiencies of each of the selected BMPs.

Table 2: BMP winter performance and TSS removal.

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<tr>
<td>Native Revegetation</td>
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<td>High</td>
</tr>
<tr>
<td>Pervious Pavement</td>
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<td>High</td>
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<tr>
<td>Subsurface Infiltration</td>
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<td>High</td>
</tr>
<tr>
<td>Green Roof</td>
<td>Medium</td>
<td>Medium</td>
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8. East Beltline Parking Lot Problem

While the team looked within the entire Whiskey Creek watershed and found locations where BMPs can be implemented, the EB Lot washout problem was specifically addressed because it was the inspiration for this project. Also, the Biology Department could possibly get funding from the Michigan Department of Environmental Quality to implement a design solution to this problem.

Randy Van Dragt of the Biology Department came to the Engineering Department with an erosion problem in the Ecosystem Preserve on the east side of campus. The EB Lot is located on the northeast edge of the Ecosystem Preserve and is the source of large amounts of stormwater runoff. The entire lot is graded so that all of its runoff drains off the southeast corner of the parking lot. A map showing the EB Lot and the surrounding landscape is shown in Figure 51.
Figure 51: East Beltine Lot and Surrounding Area.

This is a relatively new problem that only arose when the lot was regraded. Originally, the lot was graded in such a way as to direct all the stormwater directly east and into the Ecosystem Preserve. There was no problem with this system until the trees just to the east of lot started growing rapidly, cracking the pavement, and dropping debris onto the lot causing damage to parked cars, and increased maintenance costs. Figure 52 shows the flow of water and the location of the affected trees. This rapid growth and large amounts of falling debris were attributed to the extra water these trees were getting due to the grading of the EB Lot. Because of the EB Lot’s grading, these trees could not only get water from their own area of influence, but would also get water from the EB Lot, leading to increased growth.
Eventually, the college decided to regrade the EB Lot to restrict the amount of water that these trees were getting. Instead of regrading the entire lot, the college decided to only regrade a strip on the east side of the lot. This strip was in poor condition because of the root systems of the trees so it needed to be replaced regardless of the college’s desire to regrade the lot. The college decided to grade this strip so that any water that fell on it would be directed west, opposite of the original direction. This grading technique prevented the trees from receiving too much water, which slowed their growth, and sent water to the southeast corner of the EB Lot. Figure 53 outlines the new grading and flow of water.

Although this grading technique was beneficial because it reduced water flowing towards the trees, it created other problems. Instead of spreading the flow along the entire east edge of the lot, the flow was now concentrated onto a single point on the southeast corner. This concentrated flow would cause erosion, but would not be a large problem except that an Ecosystem Preserve path travels directly through the area of effect of the stormwater runoff. This path is washed out and eroded by almost any storm event.

Figure 52: Original flow of water on the EBL
because of the speed and volume of stormwater runoff being discharged at this point. The Biology Department would not only like to solve path erosion problem, but would also like to replant about an acre and a half of land in this same area that is home to several invasive species. Figure 53 also shows the discharge point of the runoff and the invasive species location.

![Figure 53: Current flow of water on the EB Lot.](image)

9. **East Beltline Lot Alternatives**

The team designed two different solutions to the EB Lot problem, including pervious pavement, and a bioretention cell and detailed two other solutions including a redesigned conduit, and a reinforced walkway. The team designed two solutions so that a well thought out decision could be made.
9.1. Pervious Pavement

The first alternative that the team designed for solving the EB Lot trail washout problem was a pervious pavement solution. Pervious pavement is ideal for this lot because there is no existing stormwater management system in place, so no retrofitting of the existing system will have to be done. This area is also ideal because it always has a high concentration of cars, but is not a high-speed area, so the pavement will hold its structural integrity for a long time and not require extensive maintenance. A major expense in the construction of pervious pavement is demolishing the existing lot, so by implementing pervious pavement at a time when the college would need to replace the lot, the college could realize significant savings.

East Beltline Pervious Pavement Design

The design of pervious pavement for the EB Lot is slightly different than the design for the rest of the pervious pavement designs on campus because the soils are much worse for infiltration. Soils present in this area have about 0.21in/hr instead of 0.32in/hr like the rest of campus. Because of this low infiltration rate, underdrains were required to achieve the necessary drainage time of 72 hours. The plan view is shown in Figure 54 and a typical section is shown in Figure 55.
Figure 54: Plan view of the EBL pervious pavement solution.

Figure 55: Typical section for the EBL pervious pavement solution.
The same storm events were used for calculation, but because the infiltration rates are slightly lower, the reservoir depths are slightly higher. The necessary reservoir depth for fully infiltrating a 2-year storm is 5.4in so a depth of 6in was used. The necessary reservoir layer depth for a 25-year storm detention is 10.6in. This number is not feasible because underdrains are required to be surrounded by 6in of gravel on all sides to ensure proper drainage so the reservoir layer depth was increased to 18in to facilitate a 6in underdrain with 6in of cover below and above the underdrain.

The underdrain was sized at a 6in diameter perforated PVC because this is the minimum recommended pipe diameter for underdrains of pervious pavement. This size of underdrain is also capable of completely conveying a 5-year 24-hour storm event, but also aids in the peak mitigation of larger storm events such as a 25-year storm. The pervious pavement system was designed with two underdrains to facilitate uniform drainage and to provide redundancy in the event that one of the underdrains fails. Both underdrains should be laid at a slope of 0.5% towards the outlet detail. An outlet detail is shown in Figure 56.

![Figure 56: Manhole detail for the EBL pervious pavement solution.](image)

This outlet is located in the southeast corner of the lot. This location was chosen because of the lots current grading scheme. Currently, the lot is graded to drain all water towards this corner, and a waterway is already present that would facilitate the drainage of large storm events, so this outlet pipe would direct water into an already existing waterway. The outlet pipe shown in Figure 56 would emerge on the south side of the nature trail, so the trail washout problem would be solved even when a large storm event occurs.

Like the rest of the pervious pavement solutions on campus, this pervious pavement lot requires a level, uncompacted bed bottom covered by a geotextile. This design does include some compacted subgrade directly under the manhole, but this is the only area that should be compacted. The same reservoir media, choker course, and pervious pavement material should be used in this solution as the other solutions on campus. The only difference is that now the reservoir layer should be 18in deep.

### 9.2. Bioretention

One alternative to solve the drainage and trail washout problem with the EB Lot is to install a bioretention basin where all the water drains from the parking lot. This alternative would be a LID solution for the stormwater because it would prevent much of the water that is generated by rain events from being
conveyed to Whiskey Creek. It would not only reduce the volume of stormwater runoff, but also the pollution load in Whiskey Creek.

The bioretention area would be fairly large. In fact it is more than double the recommended size for such a BMP. It is larger than average because there are few constraints on how large it can be built into the nature preserve area and because it serves as an instrument in killing off nonnative plant species. Because of this a 27,400 square foot area of the ecosystem preserve is proposed to be converted to a bioretention area allowing native species to return. A map of this BMP’s location is in Figure 57.

Figure 57: Map of E Beltline lot and the proposed bioretention with topographic changes.

The bioretention area would be oblong in shape; 290ft at its longest and 110ft at its widest. There would need to be excavation done in the area to an elevation of 769.25 ft. It would have a berm on all sides of it which would be 6in high at the lowest. Underdrains would also be needed to discharge excess water due to slow infiltration rates. These underdrains would need to be 6 in in diameter and should be laid north to south at an elevation of 771.5ft. Additionally, grading would need to be done to ensure that the underdrains slopes gently southward and exits into a new manhole. The manhole would outlet via a 1ft diameter pipe to the southeast of the bioretention are where it would tie in with an existing natural waterway. This pipe would only experience significant flow if it so happens that a rainfall event occurred with a statistical likelihood of greater than two years. The first fill layer for the excavated area would be well sorted rocks and gravel with a porosity of 0.45. It should fill the area to a depth of 1.75ft. A perforated underdrain pipe would be laid at this layer and be shrouded in bio filter fabric along with the entire infiltration reservoir. The next layer would be 1.0ft of the same gravel and cover the underdrain
completely. This extensive rock layer will serve as an infiltration reservoir. An overdrain at the new manhole should be utilized to accommodate intense unforeseen flows and to allow for maintenance of the underdrain. The overdrain inlet should be 6 inches above the ground elevation of the bioretention area which is 775.5 ft on average. The overdrain should spill directly into the manhole connecting with the underdrains and discharge to the established waterway. Above the rock and gravel area there should be 4 ft of sandy soil that could accommodate all kinds of natural vegetation. The soil mixture should be 70% sand and 30% compost which would give it a porosity of about 0.45. Additionally rip rap should be lain near the inlet to the bioretention area so that the water that enters the area does not erode away the soil. Measures must be taken to ensure that water will distribute evenly across the bioretention area. This would likely involve grading the area so that water flows north immediately after entering the riprap at the entrance of the bioretention area. A typical cross section for this bioretention basin is shown in Figure 58.

![B-B Cross Section](image)

**Figure 58:** Representative cross section for E Beltline bioretention basin

The eroded nature trail would need to be rebuilt into a boardwalk where it crosses the bioretention area and be built up high enough so that it does not get flooded or compact the soils in the bioretention area. Safeguards would also need to be in place for the area to make sure that none of the soil is compacted during construction or the area may lose its effectiveness in infiltrating water. According to the LID Manual for Michigan the soil in the area should not experience more than 4 pounds per square inch during construction.

This design is based on information gathered from Prof. Randy Van Dragt and from infiltration testing. The team leaned from Prof Van Dragt that there are many invasive species located just east of this parking lot that he would like to see gone. The team performed an infiltration test to see what the infiltration was actually like in this specific spot. This infiltration test of the area showed that the infiltration rate was low, around 0.5ft/day and that is one of the reasons this bioretention area is so large. This is consistent with Michigan soil survey maps. These maps show that in this area there is a lot of Perrinton loam which is in hydrologic soil group C. This bioretention basin is designed with a safety factor of 2, which likely also means that this area can accommodate and infiltrate more water than a 2 year statistical storm. Also, this design does not fulfill Kent County’s suggestion of detaining a 25 year
storm, however, because this BMP would attenuate the hydrograph and reduce the peak discharge, it was
determined that construction of this BMP is acceptable.

The vegetation to be located on top of this BMP serves as an agent of transpiration and of absorbing
pollutants. Key plants to consider are ones in the LID manual for Michigan with large root systems and
low maintenance requirements. The soil and plants are designed to filter out pollutants and infiltrate
stormwater into the ground. Therefore many processes are taking place in order to clean up and reduce the
stormwater in this parking lot assuming a bioretention basin as a solution.

10. Volume Model

In order to determine the impact that the proposed BMP plan would have on the total volume of runoff
produced by Calvin’s development, some kind of model was needed. Instead of doing all runoff
calculations by hand, a computer model was deemed as desirable due to the complexity of modeling
stormwater on a campus that is both urban and undeveloped.

10.1. SWMM

Two major stormwater modeling software packages were identified: the Hydrologic Engineering Center’s
Hydrologic Modeling System (HEC HMS) and EPA SWMM. HEC HMS can model open channels,
culverts, and spillways but cannot model an urban storm sewer network. EPA SWMM can model any
aspect of a storm sewer network, including pipes, manholes, detention ponds, swales, weirs, etc. EPA
SWMM is not as accurate as HEC HMS in modeling undeveloped areas, but it does have the capabilities
to model them. There are approximately 92 undeveloped acres on Calvin’s campus that are within the
Whiskey Creek basin. The remaining 156 acres are developed. Since most of Calvin’s campus is
urbanized, it is preferable to have a more accurate model for the developed areas of campus, therefore,
EPA SWMM was selected as the modeling software package.

EPA SWMM does not use the rational method or the Soil Conservation Service (SCS) curve method for
calculating runoff, rather, SWMM has its own method of determining runoff. For each sub basin in
SWMM, the user must input the slope, width, and area. This is a method of approximating each irregular
sub basin as an ideal rectangle which collects water into a central rivlet. This idealized basin allows
SWMM to calculate runoff using its own equations. Figure 59 below shows a schematic of SWMM’s
method of approximation.xxxx
10.2. Delineation

A challenge in using EPA SWMM is dividing the drainage basin into smaller sub basins. The only available stormsewer map of Calvin College does not show any apparent drainage divide across campus. According to the map, Calvin’s stormsewer pipes are all interconnected. Therefore, it was the team’s responsibility to determine the drainage divide between Whiskey Creek (to the south) and Rush Creek (to the north). It was also the team’s responsibility to determine the flow path of Whiskey Creek. Everyone the team talked to seemed to have an idea where Whiskey Creek might flow, but no one could say for certain.

The drainage basin was delineated in two steps. First, the stormsewer map was analyzed to determine the location of important pipes, culverts, and manholes. Then, the team physically inspected manholes as necessary to verify or correct the stormsewer network map. This process was conducted to make the major drainage basin divide between Whiskey Creek and Rush Creek, and then it was repeated to divide the Whiskey Creek basin into smaller sub basins. Figure 8 in section 2.2 displays the sub basins identified for the EPA SWMM model.

10.3. Model Set Up

The EPA SWMM model conduit and node configuration is shown in Figure 60 and Figure 61 and detailed in Table 3. Note the sub basin, node, and conduit labeling. The sub basin numbers are the same as the sub basin numbers in Figure 60.
Figure 60: SWMM model conduit IDs.

Figure 61: SWMM model node IDs.
Conduits 1, 2, 3, and 22 are open rectangular channels which are meant to model the streams flowing through the nature preserve. Conduits 19 and 20 are the weir and orifice controls for the Gainey Athletic Fields detention pond. All other conduits are circular, concrete pipes with the exception of conduit 13 which is a vegetated swale. Conduit 15 is the v-notch weir which controls the water level of the seminary pond. Nodes 01, 02, 03, 04, 012, and 015, and 026 are ponds. Nodes 017, 020, 021, 33, and 34 are outfalls. The remaining nodes are manholes. Reference Table 3 for a detailed description of conduit and node types.

**Table 3: SWMM model conduit and node types.**

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<td>Weir</td>
<td>14</td>
<td>Pond</td>
</tr>
<tr>
<td>15</td>
<td>Circular Concrete Pipe</td>
<td>15</td>
<td>Manhole</td>
</tr>
<tr>
<td>16</td>
<td>Circular Concrete Pipe</td>
<td>16</td>
<td>Manhole</td>
</tr>
<tr>
<td>17</td>
<td>Circular Concrete Pipe</td>
<td>17</td>
<td>Outfall</td>
</tr>
<tr>
<td>18</td>
<td>Circular Concrete Pipe</td>
<td>18</td>
<td>Manhole</td>
</tr>
<tr>
<td>19</td>
<td>Orifice</td>
<td>19</td>
<td>Manhole</td>
</tr>
<tr>
<td>20</td>
<td>Weir</td>
<td>20</td>
<td>Outfall</td>
</tr>
<tr>
<td>21</td>
<td>Circular Concrete Pipe</td>
<td>21</td>
<td>Outfall</td>
</tr>
<tr>
<td>22</td>
<td>Rectangular Channel</td>
<td>22</td>
<td>Outfall</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>Outfall</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>Manhole</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>Manhole</td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>Detention Pond</td>
</tr>
</tbody>
</table>

The basin was modeled with four main outfalls because Calvin’s campus drains underneath Burton St. at four different locations. When the runoff leaves campus it joins the City of Kentwood stormsewer lines underneath Burton St. Therefore, to not overcomplicate the project, four outfalls were established instead of one. Please note that outfall 34 is the sequestration pond. All runoff from subbasin 21 flows to this pond and is allowed to infiltrate into the ground.
For each sub basin, an area, width of characteristic rectangle, and slope are needed as inputs. Therefore, for each sub basin, a rectangle with an area equal to that of the sub basin was fitted on top of the sub basin. From these rectangles, the width is equal to the length of the side perpendicular to the path of flow. The slope for each rectangle was determined by analyzing the topographic map and deciding upon an average starting elevation and an average ending elevation for each sub basin. The model also needs a value for the percent of the total area which is impervious. To find this value, the area of roads, buildings and parking lots was calculated for each sub basin. Then, 10 percent of the remaining area was assumed to be taken up with walkways, adding to the total percent impervious area. Table 4 lists the properties of each characteristic rectangle.

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Area (ac)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Start Elevation (ft)</th>
<th>End Elevation (ft)</th>
<th>Elevation Difference (ft)</th>
<th>Slope (%)</th>
<th>Area Impervious (ac)</th>
<th>% Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.4</td>
<td>460</td>
<td>701</td>
<td>784</td>
<td>781</td>
<td>3</td>
<td>0.4</td>
<td>3.0</td>
<td>41</td>
</tr>
<tr>
<td>2</td>
<td>1.1</td>
<td>320</td>
<td>150</td>
<td>781</td>
<td>779</td>
<td>2</td>
<td>1.3</td>
<td>1.1</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>3.6</td>
<td>380</td>
<td>413</td>
<td>780</td>
<td>767</td>
<td>13</td>
<td>3.2</td>
<td>0.4</td>
<td>11</td>
</tr>
<tr>
<td>4</td>
<td>18.8</td>
<td>590</td>
<td>1388</td>
<td>800</td>
<td>772</td>
<td>28</td>
<td>2.0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>50.7</td>
<td>1525</td>
<td>1448</td>
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<td>1.7</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>29.3</td>
<td>870</td>
<td>1467</td>
<td>780</td>
<td>765</td>
<td>15</td>
<td>1.0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>2.1</td>
<td>240</td>
<td>381</td>
<td>780</td>
<td>776</td>
<td>4</td>
<td>1.0</td>
<td>2.1</td>
<td>100</td>
</tr>
<tr>
<td>8</td>
<td>0.2</td>
<td>70</td>
<td>124</td>
<td>778</td>
<td>776</td>
<td>2</td>
<td>1.6</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>15.4</td>
<td>740</td>
<td>907</td>
<td>781</td>
<td>769</td>
<td>12</td>
<td>1.3</td>
<td>6.5</td>
<td>42</td>
</tr>
<tr>
<td>10</td>
<td>17.9</td>
<td>730</td>
<td>1068</td>
<td>772</td>
<td>768</td>
<td>4</td>
<td>0.4</td>
<td>13.9</td>
<td>78</td>
</tr>
<tr>
<td>11</td>
<td>5.9</td>
<td>700</td>
<td>367</td>
<td>780</td>
<td>768</td>
<td>12</td>
<td>3.3</td>
<td>2.1</td>
<td>36</td>
</tr>
<tr>
<td>12</td>
<td>17.9</td>
<td>480</td>
<td>1624</td>
<td>779</td>
<td>769</td>
<td>10</td>
<td>0.6</td>
<td>14.1</td>
<td>79</td>
</tr>
<tr>
<td>13</td>
<td>2.0</td>
<td>300</td>
<td>290</td>
<td>768</td>
<td>761</td>
<td>7</td>
<td>2.4</td>
<td>2.0</td>
<td>100</td>
</tr>
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<td>14</td>
<td>2.6</td>
<td>260</td>
<td>436</td>
<td>780</td>
<td>769</td>
<td>11</td>
<td>2.5</td>
<td>1.9</td>
<td>73</td>
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<td>9.5</td>
<td>320</td>
<td>1293</td>
<td>786</td>
<td>768</td>
<td>18</td>
<td>1.4</td>
<td>2.6</td>
<td>27</td>
</tr>
<tr>
<td>16</td>
<td>18.8</td>
<td>1054</td>
<td>777</td>
<td>790</td>
<td>760</td>
<td>30</td>
<td>3.9</td>
<td>6.0</td>
<td>32</td>
</tr>
<tr>
<td>17</td>
<td>9.6</td>
<td>580</td>
<td>721</td>
<td>780</td>
<td>758</td>
<td>22</td>
<td>3.1</td>
<td>2.3</td>
<td>24</td>
</tr>
<tr>
<td>18</td>
<td>30.4</td>
<td>1181</td>
<td>1121</td>
<td>795</td>
<td>782</td>
<td>13</td>
<td>1.2</td>
<td>5.8</td>
<td>19</td>
</tr>
<tr>
<td>19</td>
<td>2.2</td>
<td>201</td>
<td>476</td>
<td>780</td>
<td>770</td>
<td>10</td>
<td>2.1</td>
<td>1.1</td>
<td>50</td>
</tr>
<tr>
<td>20</td>
<td>1.9</td>
<td>363</td>
<td>228</td>
<td>786</td>
<td>780</td>
<td>6</td>
<td>2.6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>21</td>
<td>27.4</td>
<td>830</td>
<td>1438</td>
<td>790</td>
<td>775</td>
<td>15</td>
<td>1</td>
<td>8.5</td>
<td>31</td>
</tr>
</tbody>
</table>

The other critical parameter in the EPA SWMM model is the infiltration method. The SCS curve number method was employed instead of the Horton or Green-Ampt methods because of the team’s familiarity with its use in modeling. A soil survey map was created for Calvin College, and then each soil type it listed was categorized into a hydrologic soil group. All other curve number values were taken directly from a table. Table 5 lists the curve number values for each sub basin.
Table 5: Sub basin curve numbers.

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Zoning District</th>
<th>Impervious Area (%)</th>
<th>CN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Residential 1/4acre</td>
<td>41</td>
<td>83.0</td>
</tr>
<tr>
<td>2</td>
<td>Parking Lot</td>
<td>100</td>
<td>98.0</td>
</tr>
<tr>
<td>3</td>
<td>Residential 2 acre</td>
<td>11</td>
<td>77.0</td>
</tr>
<tr>
<td>4</td>
<td>Woods - Good</td>
<td>0</td>
<td>70.0</td>
</tr>
<tr>
<td>5</td>
<td>Woods/Meadow</td>
<td>0</td>
<td>70.5</td>
</tr>
<tr>
<td>6</td>
<td>Woods - Good</td>
<td>0</td>
<td>70.0</td>
</tr>
<tr>
<td>7</td>
<td>Parking Lot</td>
<td>100</td>
<td>98.0</td>
</tr>
<tr>
<td>8</td>
<td>Woods - Good</td>
<td>0</td>
<td>70.0</td>
</tr>
<tr>
<td>9</td>
<td>Residential 1/4acre</td>
<td>42</td>
<td>83.0</td>
</tr>
<tr>
<td>10</td>
<td>Commercial</td>
<td>78</td>
<td>92.5</td>
</tr>
<tr>
<td>11</td>
<td>Residential 1/4acre</td>
<td>36</td>
<td>83.0</td>
</tr>
<tr>
<td>12</td>
<td>Commercial</td>
<td>79</td>
<td>92.5</td>
</tr>
<tr>
<td>13</td>
<td>Parking Lot</td>
<td>100</td>
<td>98.0</td>
</tr>
<tr>
<td>14</td>
<td>Industrial</td>
<td>73</td>
<td>91.0</td>
</tr>
<tr>
<td>15</td>
<td>Residential 1/2acre</td>
<td>27</td>
<td>80.5</td>
</tr>
<tr>
<td>16</td>
<td>Residential 1/3acre</td>
<td>32</td>
<td>81.5</td>
</tr>
<tr>
<td>17</td>
<td>Residential 1/2acre</td>
<td>24</td>
<td>80.0</td>
</tr>
<tr>
<td>18</td>
<td>Residential 2 acre</td>
<td>19</td>
<td>79.0</td>
</tr>
<tr>
<td>19</td>
<td>Custom</td>
<td>50</td>
<td>86.5</td>
</tr>
<tr>
<td>20</td>
<td>Woods - Fair</td>
<td>0</td>
<td>73.0</td>
</tr>
<tr>
<td>21</td>
<td>Residential 1/3acre</td>
<td>31</td>
<td>81</td>
</tr>
</tbody>
</table>

The model also required values for depression storage\textsuperscript{xxxviii xxxix}, Manning’s roughness coefficient\textsuperscript{xl}, and percent of impervious area without depression storage\textsuperscript{xli}. These values were taken from the resources cited and applied to the model.

In addition to sub basin parameters, parameters were required for each conduit, pond, and manhole. The main branch of Whiskey Creek begins at an elevation of 780’ at the research marsh and eventually outlets underneath Burton St. at an elevation of 756’, traveling a total distance of 4,500’. Figure 62 displays the elevations and distances of different sections of the main branch.
10.4. Model Verification

On March 12\textsuperscript{th}, 2012, it rained on campus in the morning and in the early evening. The team measured flow rates across campus in order to verify the accuracy of the SWMM model. Flow measurements were taken at the outlet of Whiskey Pond, the outlet to the swale south of the FAC Lot and the outlet of the Seminary Pond. The flow measurements were taken by measuring the cross sectional area of the flow through each channel or pipe. Then a piece of tissue paper was placed into the centerline of the flow and the time it took to travel a specific distance was recorded for 3 or 4 trials. The flow was calculated by multiplying the average velocity by the cross sectional area. Table 6 shows the results of the flow measurements.

<table>
<thead>
<tr>
<th>Time</th>
<th>Location</th>
<th>Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:16pm</td>
<td>Whiskey Pond Outlet</td>
<td>1.40</td>
</tr>
<tr>
<td>7:28pm</td>
<td>East FAC Outlet</td>
<td>1.57</td>
</tr>
<tr>
<td>7:39pm</td>
<td>West FAC Outlet</td>
<td>0.27</td>
</tr>
<tr>
<td>8:06pm</td>
<td>Seminary Pond Outlet</td>
<td>3.59</td>
</tr>
</tbody>
</table>

In the SWMM model, real hourly precipitation data was retrieved from NOAA and the actual storm from March 12\textsuperscript{th}, 2012 was designed in SWMM. At 7:30pm in the modeled storm, SWMM calculated the outputs in Table 7 for the locations where the flow was measured.
Table 7: SWMM Model Flow Rates for March 12th, 2012 Storm.

<table>
<thead>
<tr>
<th>Location</th>
<th>Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Whiskey Pond Outlet</td>
<td>0.15</td>
</tr>
<tr>
<td>FAC Outlet</td>
<td>0.45</td>
</tr>
<tr>
<td>Seminary Pond Outlet</td>
<td>1.72</td>
</tr>
</tbody>
</table>

The flow measurement that was most important to verify the SWMM model was the FAC outlet flow. Most of the developed portion of the western side of campus drains to Whiskey Creek before it outlets to the swale south of the FAC parking lot. Since the west side of campus is where all of the BMPs except one will be implemented, the flow measurement at the FAC outlet will provide a good indication of whether or not the model is accurate.

According to the field measurements, the FAC outlet flow on March 12th was 1.84cfs. According to the storm modeled in SWMM, the FAC outlet flow at 7:30pm was 0.45cfs. Since these values are not even on the same order of magnitude there is an obvious discrepancy. The difference between the values can be accounted for in the runoff coming from the east side of campus. It is difficult to model undeveloped land in SWMM and consequently, the mostly undeveloped portion of the Whiskey Creek drainage basin on the east side of campus proved to be the most challenging part to model.

According to the field measurements, the Whiskey Pond outlet flow was 1.40cfs, the SWMM model calculated it to be 0.15cfs. That’s a difference of 1.25cfs, which accounts for most of the 1.39cfs difference between the measured and modeled FAC outlet flow rates. If the flow coming from the east side of campus is removed from each of the FAC outlet flow rates, then the measured flow rate would be 0.44cfs and the modeled flow rate would be 0.30cfs. These values are clearly on the same order of magnitude. Therefore, they verify that SWMM model is an accurate simulation of runoff routing in the Whiskey Creek basin.

10.5. Runoff Volume Reduction

Since the primary goal of LID stormwater management is to reduce the total volume of runoff created by developed areas, it was important that the team calculate how much runoff could be reduced if the BMP plan were implemented on campus.

Since four of the five selected BMPs are infiltration based BMPs which would capture and infiltrate all of the runoff generated for a two year statistical storm, the volume reduction was calculated by hand using the runoff curve number. Because most of the BMPs are designed for impervious surfaces the volume reduction is close to the total volume of rainfall for all the areas draining to BMPs. Two additional models were estimated using SWMM, one for Knoller Crest Farm conditions (pre-development conditions) and one for pre-settlement conditions. Results are shown in Figure 63: Runoff volumes for a 2-year 24-hour storm for different cases.
The model with the BMP plan creates a 15% reduction in total runoff volume. That is a total difference of 140,000ft³ of runoff. The runoff volume reduction is even more dramatic when the runoff created by the Knollcrest Farm is included. When Calvin purchased the land on which it is currently situated, it was already a farm. According to an estimated SWMM model of the Whiskey Creek watershed as farmland, 740,000ft³ was created by Calvin’s current property before any development was done. This means that the BMP plan would reduce the runoff created by Calvin’s development by 64%. With an expanded BMP plan, it could be possible to attain pre-development conditions with LID. The ultimate goal of LID, though, is to attain presettlement hydrologic conditions. However, as evident by the 540,000ft³ disparity between the LID plan and presettlement conditions, retrofitting LID in a developed site is an unfeasible way to restore presettlement hydrology.

11. Pollution Model

LID BMPs also help to improve water quality. Because water is being infiltrated and removed from the system, the total load of pollution carried in runoff is reduced and the concentration of pollution is typically reduced. Therefore, the team decided to model the affects that the proposed LID BMP plan would have on pollution.

11.1. STEPL

The EPA has created a free pollution modeling tool named the Spreadsheet Tool for Estimating Pollutant Load. STEPL was designed to not only estimating runoff pollution loads, but to also include the pollution...
reduction resulting from LID BMPs. Included in the STEPL spreadsheet are to spreadsheets which allow for the user to input BMPs used and their effective areas. Because STEPL was designed with LID in mind, it was chosen as the pollution modeling tool.

STEPL calculates pollutant loads by first calculating the annual runoff of each sub basin using the SCS Curve Number method. After estimating the volume of runoff, STEPL then multiplies the volume by pollutant concentration values which are taken from three sources. To calculate pollutant reduction, STEPL multiplies the total pollutant load of each sub basin by a BMP removal efficiency value to estimate how much pollution is removed from the runoff due to BMPs.

11.2. Modeling Difficulties

The difficulty in using STEPL is that the pollution concentration factors and the SCS curve numbers are both tied to land use. Therefore, the model can be created with land use types which have the correct curve number and therefore the correct runoff values but incorrect pollutant load rates, or the model can be created with land use types which have the correct pollutant load rates, but incorrect runoff volumes. To correct this problem, the STEPL spreadsheet was modified so the SCS curve numbers were entered independent of the land use, allowing for an accurate volume of runoff to be calculated. So, the “Institutional” land use was used because it provides stormwater pollutant concentrations similar to those found on a college campus.

The second challenge the team faced was modeling existing BMPs which are in the main line of Whiskey Creek in combination with our proposed BMPs. Wets ponds such as Whiskey Pond and the Seminary Pond are natural BMPs because they act as sedimentation tanks. There is one dry detention pond and one vegetated swale already on campus, these also remove pollutants from runoff. To model the reductions of the proposed LID plan and the existing BMPs, the STEPL spreadsheet was modified BMPs for subcatchments and BMPs in the main stream channel could be modeled in series.

11.3. Results

The STEPL model calculated the total pollutant loads and the reductions due to proposed and existing BMPs to be the values given in Table 8 and visualized in Figure 64: Pollution removals for Nitrogen, Phosphates, BOD, and Sediment.
Table 8: Total pollutant loads.

<table>
<thead>
<tr>
<th></th>
<th>Nitrates (lb/yr)</th>
<th>Phosphates (lb/yr)</th>
<th>Biological Oxygen Demand (lb/yr)</th>
<th>Total Suspended Solids (t/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual Pollutant Load</td>
<td>711</td>
<td>127</td>
<td>2781</td>
<td>17.7</td>
</tr>
<tr>
<td>Proposed BMP Reduction</td>
<td>166</td>
<td>24</td>
<td>11</td>
<td>4.1</td>
</tr>
<tr>
<td>Existing BMP Reduction</td>
<td>191</td>
<td>49</td>
<td>285</td>
<td>8.8</td>
</tr>
<tr>
<td>Total Reduction</td>
<td>357</td>
<td>72</td>
<td>296</td>
<td>12.9</td>
</tr>
<tr>
<td>Resulting Pollutant Load</td>
<td>354</td>
<td>55</td>
<td>2485</td>
<td>4.8</td>
</tr>
</tbody>
</table>

The pollutant that was reduced by the greatest amount was TSS. That is a result of the wet ponds and the pervious pavement BMPs, which have high TSS removal efficiencies. Biological Oxygen Demand (BOD) was reduced the least. It is uncertain why that is, because the removal of Nitrogen and Phosphates should show a stronger correlation with BOD since many Nitrates and Phosphates are organic materials.
This discrepancy is probably a result of the estimated pollutant concentrations in runoff. These estimates are taken from STEPL and they are not specific to the conditions of Calvin College. In order to have better pollutant concentrations, it would be necessary to test the pollutant concentrations in Calvin’s stormwater over the period of a year or two years. What can be said for certain about the STEPL modeling results, though, is that LID BMPs are effective at improving runoff quality, and when they are combined with existing ponds, water quality can be dramatically improved.

12. Cost Analysis

The final cost of the proposed BMP plan is 3.3 million dollars if bioretention for the EB Lot is selected or 3.4 million dollars if porous pavement is selected for the EB Lot. The overall cost of the project is slightly higher than would be in many other locations in Michigan due to the low infiltration rates associated with the underlying soils. The cost is also raised because the team chose to study some more expensive BMPs like green roofs and pervious pavement. Per cubic foot of water infiltrated, the most cost effective BMP to construct is subsurface infiltration. This option works well in areas with poor infiltration and is easily incorporated into drainage areas that have an open area to put an infiltration bed in. The costs of these BMPs are show in Table 9 for low infiltration rates associated with hydrologic soil type C soils and the better infiltration rates associated with type B soils. Hopefully the cost of the BMPs for higher infiltration rates will help the readers of this report to understand the cost of LID BMPs if they have higher infiltration rates on their site.

### Table 9: Cost of individual BMPs

<table>
<thead>
<tr>
<th>BMP</th>
<th>Low Infiltration</th>
<th>High Infiltration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BMP Cost</td>
<td>Runoff Treated (ft³)</td>
</tr>
<tr>
<td>Bioretention – SC Lot</td>
<td>$90,000</td>
<td>7760</td>
</tr>
<tr>
<td>Bioretention - EB Beltline</td>
<td>$320,000</td>
<td>21360</td>
</tr>
<tr>
<td>Bioretention - Commons Lawn</td>
<td>$40,000</td>
<td>2620</td>
</tr>
<tr>
<td>Green Roof - Library</td>
<td>$1,300,000</td>
<td>15040</td>
</tr>
<tr>
<td>Porous Pavement - EB Beltline</td>
<td>$400,000</td>
<td>17810</td>
</tr>
<tr>
<td>Porous Pavement - FAC Lot</td>
<td>$500,000</td>
<td>34370</td>
</tr>
<tr>
<td>Porous Pavement – SC Lot</td>
<td>$950,000</td>
<td>49370</td>
</tr>
<tr>
<td>Subsurface Infiltration - SB,NH</td>
<td>$75,000</td>
<td>8690</td>
</tr>
<tr>
<td>Native Vegetation - EB Beltline</td>
<td>$50,000</td>
<td>1640</td>
</tr>
<tr>
<td><strong>Total W/ E Beltline</strong></td>
<td>$3,300,000</td>
<td></td>
</tr>
<tr>
<td><strong>Bioretention</strong></td>
<td>$3,400,000</td>
<td></td>
</tr>
</tbody>
</table>

The cheapest BMP in this plan is the bioretention cell for Commons Lawn and the most expensive BMP is the green roof for Hekman Library and Hiemenga Hall. With better infiltration rates the cost of some of these BMPs would be reduced as well as their plan view area. The cost of pervious pavement and the green roof would not change too much but their costs could be mitigated by constructing the BMPs when
either the roof of the library gets repaired or the parking lots get resurfaced. The cost of infiltrating water in the bioretention basin designed for the SC Lot is reduced to 28% of what it would cost if the hydrologic soil type were change from type C to type D soils. For type B soils it would be the most cost effective BMP for reducing runoff and pollution.

Better infiltration rates do not proportionally decrease the cost of all BMPs. The cost of native revegetation and the cost of a green roof would not change at all if it were designed to treat a 2 year storm, and the cost of pervious pavement would be reduced only about 13%. However the cost of bioretention would be reduced significantly. About 73% of the cost would be dropped by using bioretention in an area with type B soil instead of type C.

13. Conclusions

The total cost of the BMP plan is between 3.3 and 3.4 million dollars. This is a very feasible projected cost because the BMP plan can be implemented slowly over time whenever general improvements are made to campus. Eventually the SC, FAC, and EB Lots will have to be resurfaced. Instead of resurfacing them with traditional pavement, they could be resurfaced with pervious pavement. Native revegetation is an inexpensive BMP which could easily be implemented. Calvin will eventually make improvements to Hiemenga Hall, and when it happens, the roof could be retrofitted with a green roof. Of course, the overall cost of the proposed BMP plan could be driven down even more if only subsurface infiltration was used all across campus for infiltration purposes, but the goal of this pilot study was to demonstrate the effectiveness of various LID BMPs, not just one.

By implementing the BMP plan over time, the impact of the cost will be lessened. It will also serve to create an understanding that any construction or redevelopment done on campus in the future should be done in a way that seeks to restore God’s creation to a healthy condition. Any existing developments in the Plaster Creek watershed can easily integrate a BMP plan into their improvement or redevelopment plans.

As clearly shown by the design of the BMP plan, green roofs, bioretention, vegetation, dry wells, and pervious pavement are all suitable BMPs for retrofit. These five BMPs are easily adaptable to any developed site because they do not require any pretreatment and they Bioretention can be implemented in parking lot islands, or to the side of any paved surface. Subsurface infiltration basins connect to any roof drain system. Vegetation can be implemented anywhere there is green space, and done well, it can actually have aesthetic benefits. Green roofs can be retrofitted onto any flat roof because most flat roofs can bear the extra load due to their strong design, and they can be added to angled roofs, though green roofs are not quite as effective on an incline.

In locations with soils that have higher infiltration rates, the cost effectiveness of the BMPs will be increased. Either the BMPs could be designed smaller and cost less, or for the same price, the BMPs could be designed to infiltrate more runoff.

BMPs are feasible for redevelopment and retrofit as discussed above, but they are even more feasible for new construction. If BMPs are implemented in the right way, they can be used to eliminate altogether the need for a traditional stormsewer network and detention basin. This will greatly mitigate the cost of LID stormwater management. They are also even more feasible for new construction because BMPs could be designed without constraint to fit on an existing development. This allows for more extensive use of BMPs such as infiltration trenches, vegetated swales, and bioretention.
LID is feasible for retrofit into a developed site; it must simply be approached in a creative way. BMP selection and design must take into account, runoff volume and quality impact, cost, aesthetics, soils, and available space. While the presettlement hydrology of a site might be unattainable, LID BMPs could still have a significant impact on runoff quality and volume and thereby help to remediate pollution and erosion problems present in urban waterways. Every site is different, so hopefully this report helps you to think of how LID can be retrofitted into your business, community, or institution.


South Carolina Department of Health and Environmental Control


Plaster Creek Watershed Management Plan.


SEMCOG LID Manual. Pg. 131-146


Minnesota Urban Small Sites BMP Manual. 3-196 - 3-179.


SEMCOG LID Manual. Pg. 289-299


SEMCOG LID Manual. Pg. 301-314

SEMCOG LID Manual. Pg. 315-327


SEMCOG LID Manual Pg. 199


SEMCOG LID Manual Pg. 231


SEMCOG LID Manual Pg. 233

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**Table of Appendices**

I. Mathcad Calculations for BMP designs ................................................................. II

II. Design cost estimates using RSMeans 2009 unit costs ................................... XXVI

III. Modeling calculations .......................................................................................... XXXI
Appendix I: Mathcad calculations for BMP designs

Commons Lawn Bioretention Design

Reservoir Depth for a 2-year Storm

\[ CN := 74 \]
\[ So := \frac{1000}{CN} - 1C \]
\[ Q_v := \frac{(2.37 - 0.2So)^2}{(2.37 + 0.8So)} \]
\[ Q_v = 0.537 \]
\[ Total_{Runoff} := \frac{Q_v}{12} \text{BasinArea} \]
\[ Total_{Runoff} = 2.622 \times 10^3 \]

\[ Total_{Runoff} = 3117 \text{ft}^3 \]
\[ Rate_{Infiltration} := 0.32 \text{in/hr} \]
\[ Safety_{Factor} := 2 \]
\[ Rate_{Infiltration}^{Corrected} := \frac{Rate_{Infiltration}}{Safety_{Factor}} \]
\[ Rate_{Infiltration}^{Corrected} = 0.16 \text{in/hr} \]
\[ DesignDrainTime := 3 \text{day} \]
\[ Ratio_{Void} := 0.45 \]
\[ Depth_{Reservoir} := \frac{Rate_{Infiltration}^{Corrected}}{Safety_{Factor}} \cdot \frac{DesignDrainTime}{Ratio_{Void}} \]
\[ Depth_{Reservoir} = 2.133 \text{ft} \]
Misc Designs

Underdrain_Height := Depth_{Reservoir}

Underdrain_Height = 2.133ft

Underdrain_Diameter := 0.5ft

RockLayerHeight := Underdrain_Diameter + Depth_{Reservoir}

RockLayerHeight = 2.633ft

SoilFilterHeight := 4ft

TotalExcavatedDepth := SoilFilterHeight + RockLayerHeight

TotalExcavatedDepth = 6.633ft

OverdrainHeight := 6\text{in}

Surface Area of Bioretention

\[
\text{SurfaceArea} := \frac{\text{Total_Runoff}}{\text{Ratio}_{\text{Void}} \cdot \text{Depth}_{\text{Reservoir}}}
\]

SurfaceArea = 3.247 \times 10^3 \cdot \text{ft}^2
Green Roof Design

Reservoir Depth for a 2-year Storm

Curve number method of determining stormwater runoff.

\[ \text{CN} := 10 \]

\[ \text{BasinArea} := 7613 \text{ft}^2 \]

\[ \text{So} := \frac{1000}{\text{CN}} - 1 \]

\[ Q_v := \frac{(2.37 - 0.2\text{So})^2}{(2.37 + 0.8\text{So})} \]

\[ Q_v = 2.37 \]

\[ \text{GreenRoofArea} := 7413 \text{ft}^2 \]

\[ \text{Total}_{\text{Runoff}} = \frac{Q_v}{12} \cdot \text{BasinArea} \]

\[ \text{Total}_{\text{Runoff}} = 1.504 \times 10^4 \]

\[ \text{Equivalent}Q_v := \frac{\text{BasinArea}}{\text{GreenRoofArea}} \cdot Q_v \]

\[ \text{Equivalent}Q_v = 2.434 \]

\[ \text{Total}_{\text{Runoff}} := 11940 \text{ft}^3 \]

Runoff volume for a 2-year statistical storm for Grand Rapids, Michigan.

\[ \text{DesignDrainTime} := 3 \text{day} \]

Time to train the reservoir layer.

\[ \text{Ratio}_{\text{Void}} := 0.45 \]

Void ratio for the reservoir layer.

\[ \text{Depth}_{\text{SoilReservoir}} := \frac{\text{Equivalent}Q_v}{\text{Ratio}_{\text{Void}}} \]

\[ \text{Depth}_{\text{SoilReservoir}} = 5.409 \]

Reservoir depth to hold rainfall.
West Lot Bioretention Design

Reservoir Depth for a 2-year Storm

\[
CN := 98 \quad \text{BasinArea} := 4348 \text{ft}^2
\]

\[
So := \frac{1000}{CN} - 10 \quad \text{Curve number method of determing stormwater runoff.}
\]

\[
Q_v := \frac{(2.37 - 0.2So)^2}{(2.37 + 0.8So)}
\]

\[
Q_v = 2.142
\]

\[
\text{Total_Runoff}_{\text{Calc}} := \frac{Q_v}{12} \cdot \text{BasinArea}
\]

\[
\text{Total_Runoff}_{\text{Calc}} = 7.76 \times 10^3
\]

\[
\text{Total_Runoff} := 7760 \text{ft}^3
\]

\[
\text{Rate}_{\text{Infiltration}} := 0.32 \text{ in/hr}
\]

\[
\text{Safety}\_\text{Factor} := 2
\]

\[
\text{Rate}_{\text{Infiltration\_Corrected}} := \frac{\text{Rate}_{\text{Infiltration}}}{\text{Safety}_{\text{Factor}}}
\]

\[
\text{Rate}_{\text{Infiltration\_Corrected}} = 0.16 \text{ in/hr}
\]

\[
\text{DesignDrainTime} := 3 \text{day}
\]

\[
\text{Ratio}_{\text{Void}} := 0.45
\]

\[
\text{Depth}_{\text{Reservoir}} := \frac{\text{Rate}_{\text{Infiltration\_Corrected}} \cdot \text{DesignDrainTime}}{\text{Ratio}_{\text{Void}}}
\]

\[
\text{Depth}_{\text{Reservoir}} = 2.133 \text{ ft}
\]

Runoff volume for a 2-year statistical storm for Grand Rapids, Michigan.

Infiltration rate assumed from Michigan soil map.

Infiltration rate corrected for the factor of safety.

Time to train the reservoir layer.

Void ratio for the reservoir layer.

Reservoir depth to hold rainfall.
**Misc Designs**

Underdrain\_Height := Depth\_Reservoir

Underdrain\_Height = 2.133ft

Underdrain\_Diameter := 0.5ft

RockLayerHeight := Underdrain\_Diameter + Depth\_Reservoir

RockLayerHeight = 2.633ft

SoilFilterHeight := 4ft

TotalExcavatedDepth := SoilFilterHeight + RockLayerHeight

TotalExcavatedDepth = 6.633ft

OverdrainHeight := 6in

**Surface Area of Bioretention**

\[
\text{SurfaceArea} := \frac{\text{Total\_Runoff}}{\text{Ratio\_Void} \cdot \text{Depth\_Reservoir}}
\]

SurfaceArea = \(8.083 \times 10^3\) ft\(^2\)
Underdrain Free Bioretention Design

Reservoir Depth for a 2-year Storm

\[
\text{Total\_Runoff} := 2620\text{ft}^3
\]

Runoff volume for a 2-year statistical storm for Grand Rapids, Michigan.

\[
\text{OverdrainHeight} := 6\text{in}
\]

Assumed infiltration rate based on soil map of Michigan.

\[
\text{Rate\_Infiltration} := \frac{1\text{in}}{\text{hr}}
\]

Infiltration rate corrected for the factor of safety.

\[
\text{Safety\_Factor} := 2
\]

Time to train the reservoir layer.

\[
\text{Rate\_Infiltration\_Corrected} := \frac{\text{Rate\_Infiltration}}{\text{Safety\_Factor}}
\]

Void ratio for the reservoir layer.

\[
\text{Depth\_Reservoir} := \frac{\text{Rate\_Infiltration\_Corrected}}{\text{Ratio\_Void}} \cdot \frac{\text{Design\_Drain\_Time}}{\text{Ratio\_Void}} - \frac{\text{Overdrain\_Height}}{\text{Ratio\_Void}}
\]

Reservoir depth to hold rainfall. Ponding area not included. Includes gravel infiltration layer and planting layer.

\[
\text{Depth\_Reservoir} = 5.556\text{ft}
\]
Misc Designs

No Underdrain

SoilFilterHeight := 3 ft

Typical value for soil layer height

RockLayerHeight := DepthReservoir - SoilFilterHeight

Total height of rock layer accompanying an underdrain

RockLayerHeight = 2.556 ft

TotalExcavatedDepth := SoilFilterHeight + RockLayerHeight

TotalExcavatedDepth = 5.556 ft

Surface Area of Bioretention

SurfaceArea := \frac{\text{Total Runoff} - \text{RatioVoid} \cdot \text{DepthReservoir} + \text{OverdrainHeight}}{\text{RatioVoid} \cdot \text{DepthReservoir}}

SurfaceArea = 873.333 ft^2
East Beltline Bioretention Design

Reservoir Depth for a 2-year Storm

\[ CN := 98 \]
\[ So := \frac{1000}{CN} - 1 \]
\[ Q_v := \frac{(2.37 - 0.2So)^2}{(2.37 + 0.8So)} \]
\[ Q_v = 2.142 \]
\[ Total_{Runoff Calc} := \frac{Q_v}{12} \times \text{BasinArea} \]
\[ Total_{Runoff} := 2135 \text{ ft}^3 \]
\[ Rate_{Infiltration} := 0.26 \text{ in/hr} \]
\[ Safety_Factor := 2 \]
\[ Rate_{Infiltration Corrected} := \frac{Rate_{Infiltration}}{Safety_Factor} \]
\[ Rate_{Infiltration Corrected} = 0.13 \text{ in/hr} \]
\[ DesignDrainTime := 3 \text{ days} \]
\[ Ratio_{Void} := 0.45 \]
\[ Depth_{Reservoir} := \frac{Rate_{Infiltration Corrected} \times DesignDrainTime}{Ratio_{Void}} \]
\[ Depth_{Reservoir} = 1.733 \text{ ft} \]

Curve number method of determining stormwater runoff.

Total runoff was determined by adding the volume of runoff generated by the BMP itself and the total runoff volume of the basin. Then the runoff volume of the BMP itself was determined by iterating it with the total runoff volume generated by the surface area of BMP that Mathcad generates.

Runoff volume for a 2-year statistical storm for Grand Rapids, Michigan.

Infiltration rate tested in the field.

Infiltration rate corrected for the factor of safety.

Time to train the reservoir layer.

Void ratio for the reservoir layer.

Reservoir depth to hold rainfall.
Misc Designs

Underdrain\_Height := Depth_{Reservoir}

Underdrain\_Height = 1.733\text{ft}

Underdrain\_Diameter := 0.5\text{ft}

RockLayer\_Height := Underdrain\_Diameter + Depth_{Reservoir}

RockLayer\_Height = 2.233\text{ft}

SoilFilter\_Height := 4\text{ft}

TotalExcavated\_Depth := SoilFilter\_Height + RockLayer\_Height

TotalExcavated\_Depth = 6.233\text{ft}

Overdrain\_Height := 6\text{in}

Surface Area of Bioretention

\[
\text{SurfaceArea} := \frac{\text{Total\_Runoff}}{\text{Ratio\_Void} \cdot \text{Depth}_{\text{Reservoir}}}
\]

SurfaceArea = 2.738 \times 10^4 \text{ ft}^2
East Beltline Pervious Pavement Design

Reservoir Depth for a 2-year Storm

\[ \text{Depth}_{\text{Rainfall}} := 2.37 \text{in} \]

Typical rainfall depth for a 2-year statistical storm for Grand Rapids, Michigan.

\[ \text{Depth}_{\text{Rainfall}} = 0.197 \text{ft} \]

Infiltration rate tested in the field.

\[ \text{Rate}_{\text{Infiltration}} := 0.21 \text{ in/hr} \]

Infiltration rate corrected for the factor of safety.

\[ \text{Safety}_{\text{Factor}} := 2 \]

\[ \text{Rate}_{\text{Infiltration,C}} := \frac{\text{Rate}_{\text{Infiltration}}}{\text{Safety}_{\text{Factor}}} \]

The time it takes to fill the reservoir layer (2 hours is a typical value).

\[ \text{Time}_{\text{Reservoir,Fill}} := 2 \text{hr} = 0.083 \text{day} \]

Void ratio for the reservoir layer (typically No. 57 stone).

\[ \text{Ratio}_{\text{Void}} := 0.4 \]

\[ \text{Depth}_{\text{Reservoir}} = \frac{\text{Depth}_{\text{Rainfall}} - \left[ \left( \text{Rate}_{\text{Infiltration,C}} \right) \cdot \text{Time}_{\text{Reservoir,Fill}} \right]}{\text{Ratio}_{\text{Void}}} \]  

Minimum reservoir depth to hold rainfall event depth.

Equation 7.1

\[ \text{Depth}_{\text{Reservoir}} = 5.4 \text{ in} \]

\[ \text{Time}_{\text{Drain,Max}} := 2 \text{day} \]

Time to train the reservoir layer.

\[ \text{Depth}_{\text{Reservoir,Max}} := \left( \text{Rate}_{\text{Infiltration,C}} \right) \cdot \frac{\text{Time}_{\text{Drain,Max}}}{\text{Ratio}_{\text{Void}}} \]

Maximum reservoir depth.

Equation 7.2
Underdrain Calculations for a 5-year Storm

Hydraulic conductivity. Gravel is typically 17,000 ft/day, however, the permeable pavement reservoir layer will drain slower as the water level decreases. Because of this property, the conductivity is conservatively estimated at 100 ft/day.

\[ k := 100 \, \text{ft day} = 4.167 \, \text{ft hr} \]

Slope of the underdrain.

\[ M := 0.00' \]

Outflow through the underdrain per outlet pipe, assumed to be 6” pipe.

\[ \text{Flow}_{\text{Underdrain}} := k \cdot M = 0.25 \, \text{in hr} \]

Depth of rainfall present for a 5 year 24 hour storm attained from Bulletin 17.

\[ \text{Depth}_{\text{Rainfall 5year}} := 3.0 \, \text{in 24hr} \]

Minimum number of underdrains required.

\[ \text{Number}_{\text{Underdrain}} := \frac{\text{Depth}_{\text{Rainfall 5year}}}{\text{Flow}_{\text{Underdrain}}} \]

\[ \text{Number}_{\text{Underdrain}} = 0.5 \]
Reservoir Depth for a 25-year Storm

Depth_{Rainfall25} := 4.45\text{in}  

Typical rainfall depth for a 2-year statistical storm for Grand Rapids, Michigan.

Depth_{Rainfall25} = 0.371\text{ft}

Rate_{Infiltration} = 0.21 \frac{\text{in}}{\text{hr}}

Infiltration rate tested in the field. See TABLE BLAH BLAH

Safety\_Factor = 2

Rate_{Infiltration\_C} = 0.105 \frac{\text{in}}{\text{hr}}

Infiltration rate corrected for the factor of safety.

Time_{Reservoir\_Fill} = 0.083\text{day}

The time it takes to fill the reservoir layer (2 hours is a typical value).

Ratio_{Void} = 0.4

Void ratio for the reservoir layer (typically No. 57 stone).

\[
\text{Depth}_{\text{Reservoir25}} := \left[\text{Depth}_{\text{Rainfall25}} - \left(\frac{\text{Rate}_{\text{Infiltration\_C}}}{\text{Ratio}_{\text{Void}}}\right) \cdot \text{Time}_{\text{Reservoir\_Fill}}\right] / \text{Ratio}_{\text{Void}}
\]

Equation 7.1

\text{Depth}_{\text{Reservoir25}} = 10.6\text{in}

Minimum reservoir depth to hold rainfall event depth.

Time_{Drain\_Max} = 2\text{day}

Time to train the reservoir layer.

\[
\text{Depth}_{\text{Reservoir\_max25}} := \left(\frac{\text{Rate}_{\text{Infiltration\_C}}}{\text{Ratio}_{\text{Void}}}\right) \cdot \frac{\text{Time}_{\text{Drain\_Max}}}{\text{Ratio}_{\text{Void}}}
\]

Equation 7.2

\text{Depth}_{\text{Reservoir\_max25}} = 12.6\text{in}

Maximum reservoir depth.

Volume Calculations

\text{Area}_{\text{lot}} := 90200\text{ft}^2

\text{Volume} := \text{Depth}_{\text{Rainfall}} \cdot \text{Area}_{\text{lot}}

\text{Volume} = 1.781 \times 10^4 \text{ft}^3
Subsurface Infiltration Design Calculations

\[ p := 2.3\text{in} \]

\[ A_{\text{roof}} := 44000\text{ft}^2 = 1.01\text{acre} \]

\[ V_{\text{reqd}} := A_{\text{roof}} \cdot p \]

\[ V_{\text{reqd}} = 8.69 \times 10^3 \text{ft}^3 \]

\[ i := 0.32\frac{\text{in}}{\text{hr}} \]

\[ \text{Safety} := 2 \]

\[ i_C := \frac{i}{\text{Safety}} \]

\[ i_C = 0.16\frac{\text{in}}{\text{hr}} \]

\[ t := 72\text{hr} \]

\[ d_{72\text{hr}} := i_C \cdot t \]

\[ d_{72\text{hr}} = 0.96\text{ft} \]

\[ A_{\text{reqd}} := \frac{V_{\text{reqd}}}{d_{72\text{hr}}} \]

\[ A_{\text{reqd}} = 9.052 \times 10^3 \cdot \text{ft}^2 \]

\[ A_{\text{min}} := \frac{A_{\text{roof}}}{5} \]
\[ A_{\text{min}} = 8.8 \times 10^3 \text{ ft}^2 \]

Minimum required area based on the LID manual.

\[ \text{Ratio}_V := 0.4 \]

Void ratio for No. 57 stone reservoir layer.

\[
\frac{d_{\text{bed}}}{d_{72\text{hr}}} = \text{Ratio}_V
\]

\[ d_{\text{bed}} = 28.8 \text{ in} \]

depth of reservoir layer.
Southwest Lot Pervious Pavement Design

Reservoir Depth for a 2-year Storm

Depth_{Rainfall} := 2.37\text{in}

Depth_{Rainfall} = 0.197\text{ft}

Rate_{Infiltration} := \frac{0.32}{\text{hr}}

Typical rainfall depth for a 2-year statistical storm for Grand Rapids, Michigan.

Infiltration rate tested in the field.

Safety\_Factor := 2

Rate_{Infiltration\_C} := \frac{\text{Rate}_{Infiltration}}{\text{Safety\_Factor}}

Infiltration rate corrected for the factor of safety.

Rate_{Infiltration\_C} = 0.16 \text{in/hr}

Time_{Reservoir\_Fill} := 2\text{hr} = 0.083\text{day}

The time it takes to fill the reservoir layer (2 hours is a typical value).

Ratio_{Void} := 0.4

Void ratio for the reservoir layer (typically No. 57 stone).

\[
\text{Depth}_{Reservoir} := \frac{\text{Depth}_{Rainfall} - \left( \left( \text{Rate}_{Infiltration\_C} \right) \cdot \text{Time}_{Reservoir\_Fill} \right) \cdot \text{Ratio}_{Void}}{\text{Ratio}_{Void}} \quad \text{Equation 7.1}
\]

\[
\text{Depth}_{Reservoir} = 5.125\text{in}
\]

Minimum reservoir depth to hold rainfall event depth.

Time_{Drain\_Max} := 2\text{day}

Time to train the reservoir layer.

\[
\text{Depth}_{Reservoir\_max} := \left( \frac{\text{Rate}_{Infiltration\_C}}{\text{Ratio}_{Void}} \right) \cdot \text{Time}_{Drain\_Max} \quad \text{Equation 7.2}
\]

\[
\text{Depth}_{Reservoir\_max} = 19.2\text{in}
\]

Maximum reservoir depth.
Hydraulic conductivity. Gravel is typically 17,000 ft/day, however, the permeable pavement reservoir layer will drain slower as the water level decreases. Because of this property, the conductivity is conservatively estimated at 100 ft/day.

Slope of the underdrain.

Outflow through the underdrain per outlet pipe, assumed to be 6” pipe.

Depth of rainfall present for a 5 year 24 hour storm attained from Bulletin 17.

Minimum number of underdrains required.
## Reservoir Depth for a 25-year Storm

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth_{Rainfall_{25}}</td>
<td>4.45 in</td>
<td>Typical rainfall depth for a 2-year statistical storm for Grand Rapids, Michigan.</td>
</tr>
<tr>
<td>Depth_{Rainfall_{25}}</td>
<td>0.371 ft</td>
<td></td>
</tr>
<tr>
<td>Rate_{Infiltration}</td>
<td>0.32 in/hr</td>
<td>Infiltration rate tested in the field.</td>
</tr>
<tr>
<td>Safety_{Factor}</td>
<td>2</td>
<td>Infiltration rate corrected for the factor of safety.</td>
</tr>
<tr>
<td>Rate_{Infiltration_{C}}</td>
<td>0.16 in/hr</td>
<td></td>
</tr>
<tr>
<td>Time_{Reservoir_{Fill}}</td>
<td>0.083 day</td>
<td>The time it takes to fill the reservoir layer (2 hours is a typical value).</td>
</tr>
<tr>
<td>Ratio_{Void}</td>
<td>0.4</td>
<td>Void ratio for the reservoir layer (typically No. 57 stone).</td>
</tr>
</tbody>
</table>

\[
\text{Depth}_{\text{Reservoir_{25}}} := \left[ \frac{\text{Depth}_{\text{Rainfall_{25}}} - \left( \text{Rate}_{\text{Infiltration_{C}}} \times \text{Time}_{\text{Reservoir_{Fill}}} \right)}{\text{Ratio}_{\text{Void}}} \right] \quad \text{Equation 7.1}
\]

\[
\text{Depth}_{\text{Reservoir_{25}}} = 10.325 \text{ in}
\]

\[
\text{Time}_{\text{Drain_{Max}}} = 2 \text{ day}
\]

\[
\text{Depth}_{\text{Reservoir_{max25}}} := \left( \text{Rate}_{\text{Infiltration_{C}}} \times \frac{\text{Time}_{\text{Drain_{Max}}}}{\text{Ratio}_{\text{Void}}} \right) \quad \text{Equation 7.2}
\]

\[
\text{Depth}_{\text{Reservoir_{max25}}} = 19.2 \text{ in}
\]

Minimum reservoir depth to hold rainfall event depth. Time to train the reservoir layer. Maximum reservoir depth.
V-Trench Design

Difference in rainfall depths between 25-year and a 50-year 24 hour storms.

\[
\text{Depth}_{\text{rain_v}} := 0.82\text{in}
\]

\[
\text{Area}_{\text{Lot}} := 250000\text{ft}^2
\]

Extra rainfall volume generated by the roof and pavement area for a 50-year storm. This volume needs to be discharged safely over a 24 hour storm period.

\[
\text{Volume}_{\text{rain_v}} := \text{Depth}_{\text{rain_v}} \cdot (\text{Area}_{\text{Lot}})
\]

\[
\text{Volume}_{\text{rain_v}} = 1.278 \times 10^5 \cdot \text{gal}
\]

Flowrate for a 50-year storm.

\[
\text{Flowrate}_{\text{storm}} := \frac{\text{Volume}_{\text{rain_v}}}{24\text{hr}}
\]

\[
\text{Flowrate}_{\text{storm}} = 0.198 \frac{\text{ft}^3}{\text{s}}
\]

Flowrate needed to be discharged for a 50-year storm.

\[
\eta := 0.01/t
\]

Mannings roughness coefficient for straight uniform clean cut earth.

\[
y := 0.4\text{ft}
\]

Channel side slope (1:z slope).

\[
\text{Area}_{\text{trench}} := z \cdot y^2
\]

Depth of channel.

\[
\text{R}_{\text{trench}} := \frac{z \cdot y}{2 \sqrt{1 + z^2}}
\]

Area of channel cross section.

\[
S_0 := .0005\frac{\text{ft}}{\text{ft}}
\]

Hydraulic radius of channel.

\[
\text{Slope}_{\text{channel}} := .0005\frac{\text{ft}}{\text{ft}}
\]

Slope of channel.

\[
Q := \frac{1.49}{\eta} \cdot \text{Area}_{\text{trench}} \cdot \text{R}_{\text{trench}} \cdot \left(\frac{2}{3}\cdot 1\cdot \frac{1}{2}\cdot S_0\right)
\]

Maximum flowrate allowed in channel.

\[
Q = 0.384 \frac{\text{ft}^3}{\text{s}}
\]
Native Revegetation Design

Areas

\[ \text{Area}_{2.0} := 3700 \text{ft}^2 \]
\[ \text{Area}_{0.2} := 1750 \text{ft}^2 \]
\[ \text{Area}_{2.4} := 1200 \text{ft}^2 \]
\[ \text{Area}_{4.18} := 20500 \text{ft}^2 \]
\[ \text{Area}_{18} := 38000 \text{ft}^2 + 5000 \text{ft}^2 \]
\[ \text{Area}_{\text{total}} := \text{Area}_{2.0} + \text{Area}_{0.2} + \text{Area}_{2.4} + \text{Area}_{4.18} + \text{Area}_{18} \]
\[ \text{Area}_{\text{total}} = 1.417 \times 10^5 \text{ ft}^2 \]
\[ \text{Area}_{\text{total}} = 3.253 \text{ acre} \]

Planting Requirements

Distance between := 5ft

\[ \text{Density} := \frac{9}{(2 \cdot \text{Distance between})^2} \]
\[ \text{Density} = 0.09 \frac{1}{\text{ft}^2} \]

Number plants := Area_{\text{total}} \cdot \text{Density} \]

\[ \text{Number}_{\text{plants}} = 1.275 \times 10^4 \]
\[
\begin{align*}
\text{Number}_{2.0} & := \text{Area}_{2.0} \cdot \text{Density} = 333 \\
\text{Number}_{0.2} & := \text{Area}_{0.2} \cdot \text{Density} = 1.575 \times 10^3 \\
\text{Number}_{2.4} & := \text{Area}_{2.4} \cdot \text{Density} = 1.08 \times 10^3 \\
\text{Number}_{4.18} & := \text{Area}_{4.18} \cdot \text{Density} = 1.845 \times 10^3 \\
\text{Number}_{18} & := \text{Area}_{18} \cdot \text{Density} = 7.92 \times 10^3
\end{align*}
\]
FAC Parking Lot Pervious Pavement Design

Reservoir Depth for a 2-year Storm

\[
\text{Depth}_{\text{Rainfall}} := 2.37 \text{in}
\]

\[
\text{Depth}_{\text{Rainfall}} = 0.197 \text{ft}
\]

\[
\text{Rate}_{\text{Infiltration}} := 0.32 \frac{\text{in}}{\text{hr}}
\]

\[
\text{Safety}_{\text{Factor}} := 2
\]

\[
\text{Rate}_{\text{Infiltration\_C}} := \frac{\text{Rate}_{\text{Infiltration}}}{\text{Safety}_{\text{Factor}}}
\]

\[
\text{Rate}_{\text{Infiltration\_C}} = 0.16 \frac{\text{in}}{\text{hr}}
\]

\[
\text{Time}_{\text{Reservoir\_Fill}} := 2 \text{hr} = 0.083 \text{day}
\]

\[
\text{Ratio}_{\text{Void}} := 0.4
\]

\[
\text{Area}_{\text{Lot}} := 114000 \text{ft}^2 = 2.617 \text{acre}
\]

\[
\text{Area}_{\text{Roof}} := 60000 \text{ft}^2 = 1.377 \text{acre}
\]

\[
\text{Depth}_{\text{Reservoir}} := \frac{\text{Depth}_{\text{Rainfall}} \left(1 + \frac{\text{Area}_{\text{Roof}}}{\text{Area}_{\text{Lot}}}\right) - \left[\left(\text{Rate}_{\text{Infiltration\_C}} \cdot \text{Time}_{\text{Reservoir\_Fill}}\right) \cdot \text{Ratio}_{\text{Void}}\right]}{\text{Ratio}_{\text{Void}}}
\]

Equation 7.1
**Minimum reservoir depth to hold rainfall event depth.**

Minimum reservoir depth to hold rainfall event depth.

**Time to train the reservoir layer.**

Time to train the reservoir layer.

**Equation 7.2**

**Maximum reservoir depth.**

Maximum reservoir depth.

---

**Reservoir Depth for a 25-year Storm**

**Depth Rainfall25 := 4.45in**

Typical rainfall depth for a 25-year statistical storm for Grand Rapids, Michigan.

**Depth Rainfall25 = 0.371ft**

Infiltration rate for the west portion of campus.

**Rate Infiltration = 0.32in/hr**

Infiltration rate for the west portion of campus.

**Safety Factor = 2**

Infiltration rate corrected for the factor of safety.

**Rate Infiltration_C = 0.16in/hr**

Infiltration rate corrected for the factor of safety.

**Time Reservoir_Fill = 0.083 day**

The time it takes to fill the reservoir layer (2 hours is a typical value).

**Ratio Void = 0.4**

Void ratio for the reservoir layer (typically No. 57 stone).

**Equation 7.1**

**Depth Reservoir25 :=**

Minimum reservoir depth to hold rainfall event depth.

\[
\text{Depth Reservoir25} := \frac{\text{Depth Rainfall25} \left(1 + \frac{\text{Area Roof}}{\text{Area Lot}}\right) - \left[\left(\text{Rate Infiltration}_C\right) \cdot \text{Time Reservoir Fill}\right]}{\text{Ratio Void}}
\]

Minimum reservoir depth to hold rainfall event depth.
Time to train the reservoir layer.

\[
\text{Time}_{\text{Drain Max}} = 2 \text{ day}
\]

\[
\text{Depth}_{\text{Reservoir max 25}} = (\text{Rate}_{\text{Infiltration C}}) \cdot \frac{\text{Time}_{\text{Drain Max}}}{\text{Ratio Void}}
\]  \hspace{1cm} \text{Equation 7.2}

\[
\text{Depth}_{\text{Reservoir max 25}} = 19.2 \text{ in}
\]

**V-Trench Design**

\[
\text{Depth}_{\text{rain v}} := 0.82 \text{ in}
\]

\[
\text{Area}_{\text{Lot}} = 1.14 \times 10^5 \text{ ft}^2
\]

\[
\text{Area}_{\text{Roof}} = 6 \times 10^4 \text{ ft}^2
\]

\[
\text{Volume}_{\text{rain v}} := \text{Depth}_{\text{rain v}} \cdot (\text{Area}_{\text{Lot}} + \text{Area}_{\text{Roof}})
\]

\[
\text{Volume}_{\text{rain v}} = 8.894 \times 10^4 \text{ gal}
\]

\[
\text{Flowrate}_{\text{storm}} := \frac{\text{Volume}_{\text{rain v}}}{24 \text{ hr}}
\]

\[
\text{Flowrate}_{\text{storm}} = 0.138 \text{ ft}^3 \text{ s}^{-1}
\]

\[
\eta := 0.01 \text{ ft}
\]

\[
z := 2
\]

\[
y := 0.5 \text{ ft}
\]

\[
\text{Area}_{\text{trench}} := z \cdot y^2
\]

\[
R_{\text{trench}} := \frac{z \cdot y}{2 \sqrt{1 + z^2}}
\]

\[
S_0 := 0.0005 \text{ ft}^{-1}
\]

\[
Q := \frac{1.49}{\eta} \cdot \text{Area}_{\text{trench}} \cdot R_{\text{trench}} \cdot \frac{2}{3} \cdot \frac{1}{S_0}
\]

\[
Q = 0.384 \text{ ft}^3 \text{ s}^{-1}
\]

Maximum reservoir depth.

Difference in rainfall depths between 25-year and a 50-year 24 hour storms.

Extra rainfall volume generated by the roof and pavement area for a 50-year storm. This volume needs to be discharged safely over a 24 hour storm period.

Flowrate needed to be discharged for a 50-year storm.

Mannings roughness coefficient for straight uniform clean cut earth.

Channel side slope (1:z slope).

Depth of channel.

Area of channel cross section.

Hydraulic radius of channel.

Slope of channel.

Maximum flowrate allowed in channel.
Volume Calculations

Volume := Depth_{\text{Rainfall}} \cdot (\text{Area}_{\text{Lot}} + \text{Area}_{\text{Roof}})

Volume = 3.437 \times 10^4 \text{ ft}^3
Appendix II: Design cost estimates using RSMeans 2009 unit costs

Commons Lawn Bioretention

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Cost</th>
<th>Unit</th>
<th># of Units</th>
<th>Total</th>
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East Beltline Bioretention

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Green Roof for Library and Hiemenga Hall

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West Lot Bioretention

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## Porous Pavement for West Lot

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## Porous Pavement for East Beltline Lot

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Porous Pavement for CFAC Lot

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<th># of Units</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>5.75</td>
<td>CY</td>
<td>7389</td>
<td>$42,486.11</td>
</tr>
<tr>
<td>Gravel crushed fill 1-1/2&quot;</td>
<td>21.5</td>
<td>LCY</td>
<td>6333</td>
<td>$136,166.67</td>
</tr>
<tr>
<td>Pervious asphalt 2.5&quot; deep</td>
<td>1</td>
<td>SF</td>
<td>114000</td>
<td>$114,000.00</td>
</tr>
<tr>
<td>Stabilization fabric</td>
<td>1.77</td>
<td>SY</td>
<td>12667</td>
<td>$22,420.00</td>
</tr>
<tr>
<td>Pavement demolition 3&quot; depth</td>
<td>4.81</td>
<td>SY</td>
<td>12667</td>
<td>$60,926.67</td>
</tr>
<tr>
<td>Bituminous Concrete Curbs</td>
<td>3.25</td>
<td>LF</td>
<td>5000</td>
<td>$16,250.00</td>
</tr>
<tr>
<td>Grading</td>
<td></td>
<td></td>
<td></td>
<td>$10,000.00</td>
</tr>
<tr>
<td>4% construction management</td>
<td></td>
<td></td>
<td></td>
<td>$16,089.98</td>
</tr>
<tr>
<td>3% inflation for 3 years</td>
<td></td>
<td></td>
<td></td>
<td>$37,650.55</td>
</tr>
<tr>
<td>10% contingency</td>
<td></td>
<td></td>
<td></td>
<td>$45,599.00</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$501,588.97</td>
</tr>
</tbody>
</table>

Natural Revegetation Along East Beltline

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Cost</th>
<th>Unit</th>
<th># of Units</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>9&quot;-12&quot; Plants</td>
<td>2.5</td>
<td>Ea</td>
<td>12750</td>
<td>$31,875.00</td>
</tr>
<tr>
<td>Removing sod</td>
<td>0.53</td>
<td>SY</td>
<td>15744</td>
<td>$8,344.56</td>
</tr>
<tr>
<td>4% construction management</td>
<td></td>
<td></td>
<td></td>
<td>$1,608.78</td>
</tr>
<tr>
<td>3% inflation for 3 years</td>
<td></td>
<td></td>
<td></td>
<td>$3,764.55</td>
</tr>
<tr>
<td>10% contingency</td>
<td></td>
<td></td>
<td></td>
<td>$4,559.29</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>$50,152.18</td>
</tr>
</tbody>
</table>
### Subsurface Infiltration for North Hall and Science Building Runoff

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Cost</th>
<th>Unit</th>
<th># of Units</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>5.75</td>
<td>CY</td>
<td>1367</td>
<td>$7,858.33</td>
</tr>
<tr>
<td>Gravel crushed fill 1-1/2&quot;</td>
<td>21.5</td>
<td>LCY</td>
<td>1000</td>
<td>$21,500.00</td>
</tr>
<tr>
<td>Soil fill</td>
<td>32.5</td>
<td>CY</td>
<td>400</td>
<td>$13,000.00</td>
</tr>
<tr>
<td>12&quot; perforated PVC</td>
<td>19.3</td>
<td>13LF</td>
<td>75</td>
<td>$1,447.50</td>
</tr>
<tr>
<td>18&quot; Concrete Pipe</td>
<td>29</td>
<td>LF</td>
<td>140</td>
<td>$4,060.00</td>
</tr>
<tr>
<td>Manhole</td>
<td>3850</td>
<td>Ea</td>
<td>1</td>
<td>$3,850.00</td>
</tr>
<tr>
<td>Stabilization fabric</td>
<td>1.77</td>
<td>SY</td>
<td>1200</td>
<td>$2,124.00</td>
</tr>
<tr>
<td>Grass Replanting</td>
<td>0.48</td>
<td>SY</td>
<td>1200</td>
<td>$576.00</td>
</tr>
<tr>
<td>Grading</td>
<td></td>
<td></td>
<td></td>
<td>$3,000.00</td>
</tr>
<tr>
<td>4% construction management</td>
<td></td>
<td></td>
<td></td>
<td>$2,296.63</td>
</tr>
<tr>
<td>3% inflation for 3 years</td>
<td></td>
<td></td>
<td></td>
<td>$5,374.12</td>
</tr>
<tr>
<td>10% contingency</td>
<td></td>
<td></td>
<td></td>
<td>$6,508.66</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>$71,595.25</strong></td>
</tr>
</tbody>
</table>
Appendix III: Modeling calculations

Flow Rate Calculations

Whiskey Pond Outlet  
7:16pm on 3/12/2012

Whiskey Pond Outlet Cross Section

Channel width.

$$W_{WP} := 45\text{in}$$

$$W_{4WP} := \frac{W_{WP}}{4}$$

$$W_{4WP} = 11.25\text{in}$$

One fourth of the channel width, used to calculate partial areas of the cross section.

Cross section depths.

$$d_{1WP} := 6\text{in}$$
$$d_{2WP} := 8\text{in}$$
$$d_{3WP} := 6\text{in}$$

Area of triangle on left end.

$$a_{1WP} := W_{4WP} \frac{d_{1WP}}{2}$$

Area of triangle on right end.

$$a_{2WP} := W_{4WP} \frac{d_{3WP}}{2}$$
a_{3WP} := W_{4WP}(d_{2WP} - d_{1WP})

da_{4WP} := W_{4WP}d_{1WP}²

A_{WP} := a_{1WP} + a_{2WP} + a_{3WP} + a_{4WP}

A_{WP} = 1.562 \text{ft}²

L_{WP} := 3\text{ft}

t_{1WP} := 2.86s

t_{2WP} := 3.50s

t_{3WP} := 3.35s

t_{4WP} := 3.70s

\bar{t}_{WP} := \frac{(t_{1WP} + t_{2WP} + t_{3WP} + t_{4WP})}{4}

\bar{t}_{WP} = 3.353s

V_{WP} := \frac{L_{WP}}{\bar{t}_{WP}}

V_{WP} = 0.895 \text{ft/s}

Q_{WP} := V_{WP}A_{WP}

Q_{WP} = 1.398 \text{ft}³\text{/s}
The flow rate through the seminary pond outlet was also measured in open channel flow, therefore, the flow rate calculations follow the same method used above to calculate the flow rate through the Whiskey Pond outlet.

![Seminary Pond Outlet Cross Section](image)

\[
W_{chSO} := 26\text{in}
\]

\[
W_{4SO} := \frac{W_{chSO}}{4}
\]

\[
d_{1SO} := 1\text{lin}
\]

\[
d_{2SO} := 16\text{in}
\]

\[
d_{3SO} := 14\text{in}
\]

\[
a_{1SO} := W_{4SO} \frac{d_{1SO}}{2}
\]

\[
a_{2SO} := W_{4SO} \frac{d_{3SO}}{2}
\]

\[
a_{3SO} := W_{4SO} \frac{(d_{2SO} - d_{1SO})}{2}
\]

\[
a_{4SO} := W_{4SO} \frac{(d_{2SO} - d_{3SO})}{2}
\]

\[
a_{5SO} := W_{4SO} d_{1SO}
\]

\[
a_{6SO} := W_{4SO} d_{3SO}
\]

\[
A_{SO} := a_{1SO} + a_{2SO} + a_{3SO} + a_{4SO} + a_{5SO} + a_{6SO}
\]

\[
A_{SO} = 1.85\text{ft}^2
\]
\[ L_{SO} := 3\text{ft} \]
\[ t_{1SO} := 1.67s \]
\[ t_{2SO} := 1.46s \]
\[ t_{3SO} := 1.60s \]
\[ t_{4SO} := 1.46s \]

\[ t_{SO} := \frac{(t_{1SO} + t_{2SO} + t_{3SO} + t_{4SO})}{4} \]

\[ t_{SO} = 1.548s \]

\[ V_{SO} := \frac{L_{SO}}{t_{SO}} \]

\[ V_{SO} = 1.939 \, \frac{\text{ft}}{s} \]

\[ Q_{SO} := V_{SO} \cdot A_{SO} \]

\[ Q_{SO} = 3.588 \, \frac{\text{ft}^3}{s} \]
The time measurements and velocity calculations for the other three flow tests were conducted in the same manner. The main difference was in how the area of the flow was calculated, because these were pipe flows, not open channel flows. The basic process followed to calculate the segment area of a circle is as follows: 1. Calculate the area of the circle sector the encompasses the circle segment. 2. Calculate the area of the triangle inside the circle segment which is not part of the flow segment. 3. Subtract the area of the triangle from the area of the sector to get the area of the segment.

\[
D_{SI} := 30 \text{ in} \quad \quad \quad R_{SI} := \frac{D_{SI}}{2}
\]

\[
d_{SI} := 8 \text{ in}
\]

\[
h_{SI} := R_{SI} - d_{SI} \quad \quad \quad h_{SI} = 7 \text{ in}
\]

\[
\theta_{SI} := 2 \cdot \cos \left( \frac{h_{SI}}{R_{SI}} \right) \quad \quad \quad \theta_{SI} = 124.364 \text{ deg}
\]

\[
A_{secSI} := \frac{124.4}{360} \cdot \pi \cdot R_{SI}^2 \quad \quad \quad A_{secSI} = 1.696 \text{ ft}^2
\]
\[ b_{SI} := \left( R_{SI}^2 - h_{SI}^2 \right)^{\frac{1}{2}} \]

Base of triangle which is not part of flow segment.

\[ b_{SI} = 1.106 \text{ft} \]

\[ A_{triSI} := h_{SI} \cdot b_{SI} \]

Area of triangle.

\[ A_{triSI} = 0.645 \text{ft}^2 \]

\[ A_{SI} := A_{secSI} - A_{triSI} \]

Cross sectional area of flow.

\[ A_{SI} = 1.051 \text{ft}^2 \]

\[ L_{SI} := 2 \text{ft} \]

\[ t_{SI1} := 0.34s \]

\[ t_{SI2} := 0.30s \]

\[ t_{SI3} := 0.24s \]

\[ t_{SI} := \frac{(t_{SI1} + t_{SI2} + t_{SI3})}{3} \]

\[ t_{SI} = 0.293s \]

\[ V_{SI} := \frac{L_{SI}}{t_{SI}} \]

\[ V_{SI} = 6.818 \frac{\text{ft}}{s} \]

\[ Q_{SI} := A_{SI} \cdot V_{SI} \]

\[ Q_{SI} = 7.168 \frac{\text{ft}^3}{s} \]
\[ \begin{align*}
D_{EF} & := 36 \text{in} & R_{EF} & := \frac{D_{EF}}{2} \\
\therefore d_{EF} & := 17 \text{in} \\
h_{EF} & := R_{EF} - d_{EF} \\
& \quad \Rightarrow h_{EF} = 1 \text{-in} \\
\theta_{EF} & := 2 \cdot \arccos \left( \frac{h_{EF}}{R_{EF}} \right) \\
& \quad \Rightarrow \theta_{EF} = 173.631 \text{deg} \\
A_{secEF} & := R_{EF}^2 \cdot \frac{\left( \theta_{EF} - \sin(\theta_{EF}) \right)}{2} \\
& \quad \Rightarrow A_{secEF} = 3.284 \text{ft}^2
\end{align*} \]
\[ \begin{align*}
\text{b}_{\text{EF}} & := \left( \text{R}_{\text{EF}}^2 - \text{h}_{\text{EF}}^2 \right)^{\left( \frac{1}{2} \right)} \\
\text{b}_{\text{EF}} & = 1.498\text{ft} \\
\text{A}_{\text{triEF}} & := \text{h}_{\text{EF}} \text{b}_{\text{EF}} \\
\text{A}_{\text{triEF}} & = 0.125\text{ft}^2 \\
\text{A}_{\text{EF}} & := \text{A}_{\text{secEF}} - \text{A}_{\text{triEF}} \\
\text{A}_{\text{EF}} & = 3.16\text{ft}^2 \\
\text{L}_{\text{EF}} & := 2\text{ft} \\
\text{t}_{\text{EF1}} & := 3.92\text{s} \\
\text{t}_{\text{EF2}} & := 3.98\text{s} \\
\text{t}_{\text{EF3}} & := 4.05\text{s} \\
\text{t}_{\text{EF4}} & := 4.12\text{s} \\
\text{t}_{\text{EF}} & := \frac{\left( \text{t}_{\text{EF1}} + \text{t}_{\text{EF2}} + \text{t}_{\text{EF3}} + \text{t}_{\text{EF4}} \right)}{4} \\
\text{t}_{\text{EF}} & = 4.018\text{s} \\
\text{V}_{\text{EF}} & := \frac{\text{L}_{\text{EF}}}{\text{t}_{\text{EF}}} \\
\text{V}_{\text{EF}} & = 0.498\text{ft/s} \\
\text{Q}_{\text{EF}} & := \text{A}_{\text{EF}} \text{V}_{\text{EF}} \\
\text{Q}_{\text{EF}} & = 1.573\text{ft}^3/\text{s} 
\end{align*} \]
The flow through the West FAC Outlet was calculated in the same manner as the East FAC Outlet, except the circle segment was not the cross sectional area of flow. For the West FAC Outlet, the circle segment is the area of the pipe which does not have any flow. Therefore, the area of flow is the area of the circle minus the area of the circle segment.

\[
D_{WF} := 30 \text{in} \quad R_{WF} := \frac{D_{WF}}{2}
\]

\[
d_{WF} := 21 \text{in}
\]

\[
d_{WF2} := D_{WF} - d_{WF}
\]

\[
h_{WF} := R_{WF} - d_{WF2}
\]

\[
h_{WF} = 6 \text{in}
\]

\[
\theta_{WF} := 2 \cdot \cos \left( \frac{h_{WF}}{R_{WF}} \right)
\]

\[
\theta_{WF} = 132.844 \text{deg}
\]
\[ A_{\text{secWF}} := R_{\text{WF}}^2 \cdot \frac{\left( \theta_{\text{WF}} - \sin(\theta_{\text{WF}}) \right)}{2} \]
\[ A_{\text{secWF}} = 1.239 \text{ft}^2 \]
\[ b_{\text{WF}} := \left( R_{\text{WF}}^2 - h_{\text{WF}}^2 \right)^{\frac{1}{2}} \]
\[ b_{\text{WF}} = 1.146 \text{ft} \]
\[ A_{\text{triWF}} := h_{\text{WF}} \cdot b_{\text{WF}} \]
\[ A_{\text{triWF}} = 0.573 \text{ft}^2 \]
\[ A_{\text{segWF}} := A_{\text{secWF}} - A_{\text{triWF}} \]
\[ A_{\text{segWF}} = 0.666 \text{ft}^2 \]
\[ A_{\text{WF}} := R_{\text{WF}}^2 \cdot \pi - A_{\text{segWF}} \]
\[ A_{\text{WF}} = 4.243 \text{ft}^2 \]
\[ L_{\text{WF}} := 2 \text{ft} \]
\[ t_{\text{WF1}} := 35 \text{s} \]
\[ t_{\text{WF2}} := 30 \text{s} \]
\[ t_{\text{WF3}} := 31 \text{s} \]
\[ t_{\text{WF}} := \frac{\left( t_{\text{WF1}} + t_{\text{WF2}} + t_{\text{WF3}} \right)}{3} \]
\[ t_{\text{WF}} = 32 \text{s} \]
The accuracy of the volume model was determined by comparing the flow entering the swale north of the seminary minus the flow leaving Whiskey Pond.

\[ V_{WF} := \frac{L_{WF}}{t_{WF}} \]

\[ V_{WF} = 0.063 \text{ ft/s} \]

\[ Q_{WF} := A_{WF} \cdot V_{WF} \]

\[ Q_{WF} = 0.265 \text{ ft}^3 \text{s}^{-1} \]

The value of 0.44 ft^3/s is on the same order of magnitude as the value calculated by the SWMM model (0.30 ft^3/s).
Basic Volume Model Hand Calculations

The goal of this appendix is to demonstrate through simple hand calculations that the volume model is working properly. SWMM uses complex math to calculate runoff and to route the runoff through the storm sewer network. The Newton-Rhapson method for solving roots is used to solve a non-linear differential equation with one solution. This calculation is iterated by the specified routing time step over the entire duration of the storm. For the purposes of modeling volume reduction, though, the total volume of runoff is the desired value, not the flow rate of runoff, so this sheet will go through a series of simple hand calculations that verify that the SWMM results for total runoff volume are accurate.

First, the SCS curve number method is used to calculate the total runoff depth for pervious areas.

\[
P := 2.37 \text{in} \quad \text{2yr 24hr Storm Cumulative Precipitation.}
\]

\[
\begin{array}{c}
83 \\
98 \\
77 \\
70 \\
70.5 \\
70 \\
70.5 \\
83 \\
93 \\
83 \\
93 \\
98 \\
91 \\
81 \\
82 \\
80 \\
79 \\
86.5 \\
73 \\
\end{array}
\]

\(\text{CN} := \quad \text{Curve numbers for sub basins.}\)
The potential maximum retention for each sub basin is calculated by using the runoff curve number.

\[ S = \left( \frac{1000}{CN} - 10 \right) \text{ in} \]

<table>
<thead>
<tr>
<th>CN</th>
<th>S</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.048</td>
</tr>
<tr>
<td>1</td>
<td>0.204</td>
</tr>
<tr>
<td>2</td>
<td>2.987</td>
</tr>
<tr>
<td>3</td>
<td>4.286</td>
</tr>
<tr>
<td>4</td>
<td>4.184</td>
</tr>
<tr>
<td>5</td>
<td>4.286</td>
</tr>
<tr>
<td>6</td>
<td>0.204</td>
</tr>
<tr>
<td>7</td>
<td>4.286</td>
</tr>
<tr>
<td>8</td>
<td>2.048</td>
</tr>
<tr>
<td>9</td>
<td>0.753</td>
</tr>
<tr>
<td>10</td>
<td>2.048</td>
</tr>
<tr>
<td>11</td>
<td>0.753</td>
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<tr>
<td>12</td>
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<tr>
<td>13</td>
<td>0.989</td>
</tr>
<tr>
<td>14</td>
<td>2.346</td>
</tr>
<tr>
<td>15</td>
<td>2.195</td>
</tr>
<tr>
<td>16</td>
<td>2.5</td>
</tr>
<tr>
<td>17</td>
<td>2.658</td>
</tr>
<tr>
<td>18</td>
<td>1.561</td>
</tr>
<tr>
<td>19</td>
<td>3.699</td>
</tr>
</tbody>
</table>
Initial abstractions are calculated as a function of the potential maximum retention. Initial abstraction is the depth of rainfall that is absorbed by soil without generating any runoff.

\[ I_a := 0.2 \cdot S \]

<table>
<thead>
<tr>
<th></th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
<th>19</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.41</td>
<td>0.041</td>
<td>0.597</td>
<td>0.857</td>
<td>0.837</td>
<td>0.857</td>
<td>0.041</td>
<td>0.857</td>
<td>0.41</td>
<td>0.151</td>
<td>0.41</td>
<td>0.151</td>
<td>0.041</td>
<td>0.198</td>
<td>0.469</td>
<td>0.439</td>
<td>0.5</td>
<td>0.532</td>
<td>0.312</td>
<td>0.74</td>
</tr>
</tbody>
</table>

\[ I_a \text{ in} \]
Rainfall excess is calculated as a function of total precipitation, initial abstraction, and potential maximum retention. Rainfall excess is the depth of runoff that is created as a result of the total rainfall depth.

\[
P_e = \frac{(P - I_a)^2}{P - I_a + S}
\]

<table>
<thead>
<tr>
<th>P_e (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.959</td>
</tr>
<tr>
<td>2.142</td>
</tr>
<tr>
<td>0.66</td>
</tr>
<tr>
<td>0.395</td>
</tr>
<tr>
<td>0.411</td>
</tr>
<tr>
<td>0.395</td>
</tr>
<tr>
<td>2.142</td>
</tr>
<tr>
<td>0.395</td>
</tr>
<tr>
<td>0.959</td>
</tr>
<tr>
<td>1.657</td>
</tr>
<tr>
<td>0.959</td>
</tr>
<tr>
<td>1.657</td>
</tr>
<tr>
<td>2.142</td>
</tr>
<tr>
<td>1.493</td>
</tr>
<tr>
<td>0.851</td>
</tr>
<tr>
<td>0.904</td>
</tr>
<tr>
<td>0.8</td>
</tr>
<tr>
<td>0.752</td>
</tr>
<tr>
<td>1.17</td>
</tr>
<tr>
<td>0.499</td>
</tr>
</tbody>
</table>
The area, $A$, of all the sub basins and the percent impervious area, $\%_{\text{impv}}$, of all the sub basins are taken from the volume model.

$$\begin{bmatrix}
7.4 \\
1.1 \\
3.6 \\
18.8 \\
50.7 \\
29.3 \\
2.1 \\
0.2 \\
15.4 \\
17.9 \\
5.9 \\
17.9 \\
2.0 \\
2.6 \\
9.5 \\
18.8 \\
9.6 \\
30.4 \\
2.2 \\
1.9
\end{bmatrix} \text{ acre}$$

$$\begin{bmatrix}
0.41 \\
1.00 \\
0.11 \\
0.00 \\
0.00 \\
0.00 \\
1.00 \\
0.00 \\
0.42 \\
0.78 \\
0.36 \\
0.79 \\
1.00 \\
0.73 \\
0.27 \\
0.32 \\
0.24 \\
0.19 \\
0.50 \\
0.25
\end{bmatrix} \%_{\text{impv}}$$
Impervious area is calculated by multiplying area by the percent area impervious.
\( \Lambda_{\text{perv}} := A - \Lambda_{\text{impv}} \)

Pervious area is the difference between the sub basin area and impervious area.

<table>
<thead>
<tr>
<th>( n )</th>
<th>( \Lambda_{\text{perv}} ) (acre)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>4.37</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>3.2</td>
</tr>
<tr>
<td>3</td>
<td>18.8</td>
</tr>
<tr>
<td>4</td>
<td>50.7</td>
</tr>
<tr>
<td>5</td>
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</tr>
<tr>
<td>6</td>
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</tr>
<tr>
<td>7</td>
<td>0.2</td>
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<tr>
<td>8</td>
<td>8.93</td>
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<tr>
<td>9</td>
<td>3.94</td>
</tr>
<tr>
<td>10</td>
<td>3.78</td>
</tr>
<tr>
<td>11</td>
<td>3.76</td>
</tr>
<tr>
<td>12</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>0.7</td>
</tr>
<tr>
<td>14</td>
<td>6.93</td>
</tr>
<tr>
<td>15</td>
<td>12.78</td>
</tr>
<tr>
<td>16</td>
<td>7.3</td>
</tr>
<tr>
<td>17</td>
<td>24.62</td>
</tr>
<tr>
<td>18</td>
<td>1.1</td>
</tr>
<tr>
<td>19</td>
<td>1.42</td>
</tr>
</tbody>
</table>

XLVIII
For the purpose of checking the volume model, all precipitation that lands on impervious area is assumed to turn into runoff. Pervious area is assumed to generate runoff according to the SCS curve number method (precipitation excess, $P_e$).

\[ V := P_e \cdot A_{perv} + P \cdot A_{impv} \]

The SWMM volume model calculated the total runoff from a two year storm to be 960,000 ft$^3$. That is just 2% greater than the total runoff value calculated using the simple calculations on this sheet, 980,000 ft$^3$. The difference between the two values exists because the SWMM model includes surface roughness, slope, routing, and other variables in its calculations.

\[ V := \begin{pmatrix} 41270 \\ 9474 \\ 11110 \\ 26970 \\ 75650 \\ 42000 \\ 18080 \\ 287 \\ 86760 \\ 143800 \\ 31400 \\ 144300 \\ 17210 \\ 20150 \\ 43520 \\ 93730 \\ 40980 \\ 116900 \\ 14120 \\ 6716 \end{pmatrix} \]

\[ V_{\text{total}} := 980000 \text{ ft}^3 \]
Pollution Model Verification

The EPA’s Spreadsheet Tool for Estimating Pollutant Loads (STEPL), calculates annual pollutant loads in two steps. First, the annual runoff is calculated by the curve number method. Second, the annual runoff volume is multiplied by standard pollutant concentration values, resulting in annual pollutant loads (lb/yr). This sheet calculates the nitrogen load and reduction for sub basin 7 in order to illustrate how STEPL works.

acre  

Area of sub basin

\[ \text{area} \]

CN\(_7\) := 98  

Basin curve number

\[ \text{CN}_7 \]

RD := 132.1  

Rain days per year

\[ \text{RD} \]

R\(_c\) := 0.349  

Rain days correction factor, to remove events below 5mm.

\[ \text{R}_c \]

P\(_{\text{avg}}\) := 0.629in  

Average precipitation per event

\[ \text{P}_{\text{avg}} \]

\[ S := \left( \frac{1000}{\text{CN}_7} \right) - 1 \]  

Potential maximum retention.

\[ S \]

P\(_{\text{ravg}}\) := \( \left( \frac{\text{P}_{\text{avg}} - \text{I}_a \times S}{\text{P}_{\text{avg}} + 0.8S} \right)^2 \)  

Runoff for average rain event, calculated using the curve number method.

\[ \text{P}_{\text{ravg}} \]

P\(_{\text{ravg}}\) = 0.499 in

\[ \text{P}_{\text{ravg}} = 0.499 \text{ in} \]

\[ V_{yr} := \text{RD} \times \text{R}_c \times \frac{\text{P}_{\text{ravg}}}{12} \]  

Annual runoff volume is calculated by multiplying the runoff for an average event by area and by the corrected number of rain events per year.

\[ V_{yr} \]

\[ V_{yr} = 4.05 \text{ac} \times \text{ft} \]
\[ C_N := \frac{3 \text{ mg}}{\text{ L}} \]  Concentration of given pollutant for given land use.

\[ L_N := C_N \cdot V_{yr} \]  Annual pollutant load for sub basin without BMPs.

\[ L_N = 33.04 \text{ lb} \]  

\[ E_r := 0.85 \]  Removal efficiency of given BMP for given pollutant.

\[ A_{BMP} := 2.1 \text{ acre} \]  Effective area of the given BMP.

\[ A_{BMP\%} := \frac{A_{BMP}}{A_7} \]  Percentage of basin area draining to BMP.

\[ L_{\text{red}} := L_N \cdot E_r \cdot A_{BMP\%} \]  Annual pollutant load reduction. Found by multiplying the total pollutant load by the effective area percent by the BMP removal efficiency.

\[ L_{\text{red}} = 28.084 \text{ lb} \]
Peak Flow Mitigation Hand Calcs

Kalamazoo County's Green Calculator was used to demonstrate that the BMP plan would not increase the peak discharge of a 25yr storm. Since the Green Calculator is proprietary, the method for estimating peak discharge outlined in the Michigan Department of Environmental Quality's *Computing Flood Discharges for Small Ungaged Watersheds*.

\[ V := K \cdot S^{0.5} \]

Estimate of velocity through components of the stream network. \( S \) is slope and \( K \) is a coefficient depending on component type.

\[ K_{\text{sheetflow}} := 0.48 \]

Coefficient for sheetflow and small tributary flow.

\[ K_{\text{smalltrib}} := 2.1 \]

\[ S_{\text{sheetflow}} := 0.66 \]
\[ S_{w1} := 0.557 \]
\[ S_{w2} := 1.182 \]
\[ S_{w3} := 0.577 \]
\[ S_{w4} := 0.190 \]
\[ S_{w5} := 0.284 \]

Slopes of the 6 different stream components used in Green Calculator.

\[ V_{\text{sheetflow}} := K_{\text{sheetflow}} \cdot S_{\text{sheetflow}}^{0.5} \]

\[ V_{\text{sheetflow}} = 0.392 \]

\[ V_{w1} := K_{\text{smalltrib}}^{0.5} \]
\[ V_{w1} = 1.567 \text{ fps} \]

\[ V_{w2} := K_{\text{smalltrib}}^{0.5} \]
\[ V_{w2} = 2.283 \text{ fps} \]

\[ V_{w3} := K_{\text{smalltrib}}^{0.5} \]
\[ V_{w3} = 1.567 \text{ fps} \]

\[ V_{w4} := K_{\text{smalltrib}}^{0.5} \]
\[ V_{w4} = 0.915 \text{ fps} \]

\[ V_{w5} := K_{\text{smalltrib}}^{0.5} \]
\[ V_{w5} = 1.119 \text{ fps} \]

Velocity estimates for the stream components.
Length of each stream component.

\[
L_{\text{sheetflow}} := 300 
\quad \quad \quad \quad \quad \quad L_{w1} := 883 \text{ ft} 
\quad \quad \quad \quad \quad \quad L_{w2} := 846 \text{ ft}
\]

Time of concentration for each stream component.

\[
T_{\text{sheetflow}} := \frac{L_{\text{sheetflow}}}{V_{\text{sheetflow}} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{\text{sheetflow}} = 0.213
\]

\[
T_{w1} := \frac{L_{w1}}{V_{w1} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{w1} = 0.156 \text{ hr}
\]

\[
T_{w2} := \frac{L_{w2}}{V_{w2} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{w2} = 0.103 \text{ hr}
\]

\[
T_{w3} := \frac{L_{w3}}{V_{w3} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{w3} = 0.461 \text{ hr}
\]

\[
T_{w4} := \frac{L_{w4}}{V_{w4} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{w4} = 0.558 \text{ hr}
\]

\[
T_{w5} := \frac{L_{w5}}{V_{w5} \cdot 3600} 
\quad \quad \quad \quad \quad \quad T_{w5} = 0.279 \text{ hr}
\]

\[
T_c := T_{\text{sheetflow}} + T_{w1} + T_{w2} + T_{w3} + T_{w4} + T_{w5}
\quad \quad \quad \quad \quad \quad T_c = 1.77 \text{ hr}
\]

\[
q_p := 238.6 T_c^{-0.82}
\]

\[
DA := 0.3864 \quad \quad SRO := 2.40 \quad \quad POND := 0.75
\]

\[
Q := q_p \cdot SRO \cdot DA \cdot POND 
\quad \quad \quad \quad \quad \quad Q = 109.81 \text{ ft}^3
\]
\[ T_c := 1.31 \]

\[ q_p := 238.6 T_c - 0.82 \]

\[ Q := q_p \cdot SRO \cdot DA \cdot POND \quad Q = 140.55 \text{cfs} \]
Green Calculator
Low Impact Development Hydrologic Analysis Tool
Version 2.0

Notes:
1. After opening spreadsheet you may be required to enable use of programmed macro. Look for security warning above.
2. Data is entered in yellow cells. Green cells allow selection of items from pull down menus.
3. This spreadsheet is intended for all areas contributing to a single treatment train on the site. A site with multiple (parallel) treatment trains requires one spreadsheet for each treatment train.

1. Project Description

<table>
<thead>
<tr>
<th>Name: Existing Conditions to BMP Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
</tr>
<tr>
<td>Date</td>
</tr>
<tr>
<td>Engineer                                    Jacob Parks</td>
</tr>
<tr>
<td>Michigan County                             Kalamazoo</td>
</tr>
<tr>
<td>Local Unit                                  Calvin College</td>
</tr>
<tr>
<td>Description                                 Proposed BMP Plan</td>
</tr>
<tr>
<td>Redevelopment credit?                       Yes</td>
</tr>
</tbody>
</table>

2. Time of Concentration

<table>
<thead>
<tr>
<th>Existing and pre-development (ft^2)</th>
<th>User Spec.</th>
<th>User Spec.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Developed (w/ retention BMPs)(ft^2)</td>
<td>0.00</td>
<td>1.31</td>
</tr>
</tbody>
</table>

3. Area Summary

<table>
<thead>
<tr>
<th>Area (ft^2)</th>
<th>Total area contributing to treatment train</th>
</tr>
</thead>
<tbody>
<tr>
<td>Disturbed area</td>
<td>247.30 ft^2</td>
</tr>
<tr>
<td>Protected area</td>
<td>0.00 ft^2</td>
</tr>
</tbody>
</table>

4. Runoff from Disturbed and Minimally Disturbed Areas

4a. Existing

<table>
<thead>
<tr>
<th>Land Use</th>
<th>HSG</th>
<th>Area (acre)</th>
<th>Units</th>
<th>SCS ON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impervious (paved, sod, concrete, etc.)</td>
<td></td>
<td>68.90</td>
<td>acre</td>
<td>88</td>
</tr>
<tr>
<td>Open lawns, parks, etc. - good</td>
<td>C</td>
<td>80.20</td>
<td>acre</td>
<td>74</td>
</tr>
<tr>
<td>Woods - good</td>
<td>C</td>
<td>102.20</td>
<td>acre</td>
<td>70</td>
</tr>
</tbody>
</table>

4b. Pro-development - Existing

<table>
<thead>
<tr>
<th>Land Use</th>
<th>HSG</th>
<th>Area (acre)</th>
<th>Units</th>
<th>SCS ON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impervious (paved, sod, concrete, etc.)</td>
<td></td>
<td>68.90</td>
<td>acre</td>
<td>88</td>
</tr>
<tr>
<td>Open lawns, parks, etc. - good</td>
<td>C</td>
<td>80.20</td>
<td>acre</td>
<td>74</td>
</tr>
<tr>
<td>Woods - good</td>
<td>C</td>
<td>102.20</td>
<td>acre</td>
<td>70</td>
</tr>
</tbody>
</table>

4c. Developed

<table>
<thead>
<tr>
<th>Land Use</th>
<th>HSG</th>
<th>Area (acre)</th>
<th>Units</th>
<th>SCS ON</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impervious (includes pervious pavement and grass roof)</td>
<td></td>
<td>68.90</td>
<td>acre</td>
<td>88</td>
</tr>
<tr>
<td>Minimal &amp; no disturbance areas; Oobe space - good</td>
<td>C</td>
<td>25.11</td>
<td>acre</td>
<td>74</td>
</tr>
<tr>
<td>Minimal &amp; no disturbance areas; Woods - good</td>
<td>C</td>
<td>102.20</td>
<td>acre</td>
<td>70</td>
</tr>
<tr>
<td>Native vegetation; Meadow</td>
<td>C</td>
<td>3.21</td>
<td>acre</td>
<td>71</td>
</tr>
<tr>
<td>BMP surface (excluding grass roof or pervious pavement)</td>
<td>C</td>
<td>3.68</td>
<td>acre</td>
<td>71</td>
</tr>
</tbody>
</table>

Kalamazoo County Drain Commissioner
201 West Kalamazoo Avenue, Rm. 202
Kalamazoo, Michigan 49007
Existing Conditions to BMP Conditions:is

LV
**Green Calculator**

### 5. Water Quality Volume

- **Directly connected impervious area (ac):** 0.00 ac
- **Directly connected disturbed pervious area (ac):** 0.00 ac

**Minimum required Water Quality Volume:** 0.00 ac-ft

### 6. Stream Protection Volume

- **2-year, 24-hour runoff volume for proposed development condition:** 18.45 ac-ft
- **5-year, 24-hour runoff volume for existing condition:** 18.45 ac-ft

**Minimum required Stream Protection Volume:** 0.00 ac-ft

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bioretention / Rain Garden (no underdrain)</td>
<td>A, V, &amp; i</td>
<td>10,830</td>
<td>11,628</td>
<td>0.00</td>
<td>0.27</td>
</tr>
<tr>
<td>Infiltration practices</td>
<td>A, V, &amp; i</td>
<td>10,774</td>
<td>8,706</td>
<td>0.00</td>
<td>0.20</td>
</tr>
<tr>
<td>Pervious Pavement (no underdrain)</td>
<td>A, V, &amp; i</td>
<td>85,200</td>
<td>101,555</td>
<td>0.00</td>
<td>2.06</td>
</tr>
<tr>
<td>Vegetated Roofs</td>
<td>A &amp; V</td>
<td>74,130</td>
<td>16,036</td>
<td>0.00</td>
<td>0.34</td>
</tr>
</tbody>
</table>

**Total Stream Protection Volume credit [ac-ft]:** 2.86

**Adequate Stream Protection Volume?** YES

**Percentage of Stream Protection Volume met by retention:** 0%

**Average site CN for developed condition:** 79

**Site area requiring extended detention [ac]:** 247.30

**Extended detention volume [ac-ft]:** N.A.

**Extended detention release rate [cfs]:** N.A.

### 7. Detention Storage

**Design storm: 2-year, 5-year**

- **Developed peak discharge [cfs]:** 177.66 cfs
- **Standard allowable discharge [cfs] - 0.1% of 5-year:** 37.10 cfs
- **Other allowable discharge [cfs]:** No
- **Detention storage required?** No
- **Desired peak discharge [cfs]:** 0.00 cfs
- **Calculated peak discharge [cfs]:** 0.00 cfs

**Required detention volume:** 0.00 ac-ft

**Notes:**
Green Calculator
8. Hydrographs

Note: This hydrograph plot does not include the impact of extended detention.

Rainfall

<table>
<thead>
<tr>
<th>Rainfall Frequency</th>
<th>20-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall depth (inches)</td>
<td>4.40</td>
</tr>
</tbody>
</table>

Pre-development (Existing) Conditions

<table>
<thead>
<tr>
<th>Parous Curve Number</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average runoff [in]</td>
<td>2.41</td>
</tr>
<tr>
<td>Peak discharge [cfs]</td>
<td>177.50</td>
</tr>
<tr>
<td>Runoff volume [ac-ft]</td>
<td>49.63</td>
</tr>
</tbody>
</table>

Existing Conditions

<table>
<thead>
<tr>
<th>Parous Curve Number</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average runoff [in]</td>
<td>2.41</td>
</tr>
<tr>
<td>Peak discharge [cfs]</td>
<td>177.53</td>
</tr>
<tr>
<td>Runoff volume [ac-ft]</td>
<td>49.63</td>
</tr>
</tbody>
</table>

Developed (with non-structural BMP credits)

<table>
<thead>
<tr>
<th>Parous Curve Number</th>
<th>7.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average runoff [in]</td>
<td>2.40</td>
</tr>
<tr>
<td>Runoff volume [ac-ft]</td>
<td>49.56</td>
</tr>
</tbody>
</table>

Developed with all BMPs

<table>
<thead>
<tr>
<th>Volume stored [ac-ft]</th>
<th>3.14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume released [ac-ft]</td>
<td>46.41</td>
</tr>
<tr>
<td>Peak discharge [cfs]</td>
<td>177.46</td>
</tr>
</tbody>
</table>

Disclaimer: This Excel spreadsheet is furnished by the Kalamazoo County Drain Commissioner and FTC&H for the convenience of the recipient to show compliance with the Kalamazoo County Site Development Rules.

Any additional conclusions or information obtained or derived from this spreadsheet program will be at the user's sole risk.

Kalamazoo County Drain Commissioner
201 West Kalamazoo Avenue, Rm. 202
Kalamazoo, Michigan 49007

Existing Conditions to BMP Conditions is

115 Arboratum Dr SE
Grand Rapids MI 49546