Capstone Design Project

Farrell Brook - Shelburne Road Stormwater Retrofit Plan

Community Partner: Jim Pease, VTDEC
Instructors: Prof. John Lens, PE

Katrina Benoit, Andrea Dotolo, Jamie Martell,
Andrew Sampsell, Laura Tracy

Civil and Environmental Engineering
The University of Vermont
Burlington, VT 05405

Date: May 6, 2016

EXECUTIVE SUMMARY

The following project outlines stormwater issues seen in the Farrell Brook watershed located in South Burlington, Vermont, which drains to Lake Champlain. Due to a proposed redesign of the stormwater system in an upstream neighborhood, it is estimated that peak flows downstream will almost triple. Increasing flows along this stream will have adverse effects on stream bank erosion that will further impair the water quality throughout the watershed and in Lake Champlain. Therefore, this project aims to offer various solutions to this problem through means of engineered design and cost analysis.

After a long period of review and discussion regarding the various options available to mitigate this stormwater issue, the approach was narrowed to five alternatives. The first, is a dry detention pond to slow and control flows. The second, is a retrofit retention pond further downstream for improving water quality. A gravel wetland was also suggested as a possibility to treat stormwater from the northern Orchards neighborhood. A hydrodynamic separator could be installed along the stream, which works to trap debris, sediment, and hydrocarbons from the stormwater runoff. Lastly, slope stabilization techniques such as live plantings or bioengineering have been conceptualized for implementation only after all flows upstream have been controlled.

It has been concluded that multiple solutions will be needed to address the water quality and high flow issues in Farrell Brook. Overall, the best option would be a combination of these designs, however, this project is currently capped at a $1,000,000 budget. It is recommended that the detention pond be installed first as it plays the most significant role in lowering peak flows and aids in the effectiveness of other alternatives. The retention pond is also suggested as water quality is a main concern. The gravel wetland and swirl separator would further improve the health of Farrell Brook and subsequently, Lake Champlain, but may be an option only if funding is available. In summary, we recommend the following measures:

1) Implementation of the detention pond to reduce peak flows
2) If funds are available, implementation of the retention pond to meet water quality standards
3) If further water quality improvements are desired, implementation of the gravel wetland, hydrodynamic separator, and/or slope stabilization techniques

Table of Contents

List of Figures

List of Tables

LIMITATIONS

The intent of this report is to present the data collected, evaluations, analysis, design, and opinions of probable cost for the Shelburne Road stormwater retrofit project. The work presented here was performed in a 15-week long project as part of the course, CE 175 Senior Capstone Design instructed by Professor John Lens, P.E. Although we have exercised care while working on all components of this project, the reader should be aware that the work was performed within a short time period and with limited resources. This work was directed and reviewed by Professor Lens, other UVM faculty and external evaluators; however, it has not been formally reviewed by a Professional Engineer. The reader is advised that before using any part of this report, the work presented here must be independently evaluated by a qualified Professional Engineer licensed in Vermont.

1.0 INTRODUCTION

1.1 Overview

From undeveloped forests, to agricultural fields, to the current commercial state of Route 7 in South Burlington, Vermont, this area has seen a long history of development. These changes have significantly affected the geomorphology and hydrology of the land. With the increased development in the last fifty years, the percentage of impervious surfaces along Route 7 has exponentially increased. This has resulted in increased runoff and environmental degradation. Recent studies have shown that about 16% of the phosphorus that enters Lake Champlain sources from developed lands and 20% sources from stream bank erosion in Vermont (Dunlap et. al., 2015). Phosphorus and other nutrients contribute to harmful algae blooms in the lake that threaten aquatic species and inhibit recreation. According to the "2015 State of the Lake Report," Shelburne Bay showed an increasing trend in phosphorus concentration each year. The Clean Water Initiative (Act 64) states that the total maximum daily load needs to be reduced in Shelburne Bay by 21.3% for developed lands and 55.0% for streams. In 2015, the Vermont Clean Water Act (Act 64) was passed to safeguard the public's access to clean and safe water throughout the state. The bill focuses on best management practices for agriculture, reducing polluted runoff from developed land, using "natural infrastructure" to reduce and mitigate stormwater pollution and erosion, and offers support to municipalities and farmers to meet these clean water goals (Vermont, 2015).

The following report involves the investigation, design, and analysis of a stormwater retrofit plan to reduce flows and environmental degradation along Farrell Brook, which intersects a section of U.S. Route 7 in South Burlington, Vermont (*Figure 1*). Farrell Brook starts at the stormwater outlet of a small housing development on the East side of Route 7. It then meanders through developed lands and discharges into Shelburne Bay, a section of Lake Champlain. The project aim, specified by our community partner, Jim Pease, is to enhance the water quality of Farrell Brook. Farrell Brook currently suffers from typical impairments caused by stormwater runoff including channel incision, bank erosion, hydrologic flashiness, and siltation. Primarily, this project involves the implementation of Vermont state stormwater standards, including natural stormwater infrastructure, to help retrofit this small, thoroughly urbanized watershed. The two objectives of this project are (1) reducing flows in the stream channel through the implementation of a detention pond and (2) removing nutrients and sediment with a retention pond further downstream. Further recommendations and designs such as a gravel wetland, a hydrodynamic separator, and slope stabilization techniques will be used in conjunction with the ponds to develop an overall stormwater retrofit plan that will both decrease incoming flows from impervious surfaces and ultimately improve the water quality. Further recommendations should only be used after the detention pond is in place to control the flows.

Team members include Katrina Benoit, Andrea Dotolo, Jamie Martell, Andrew Sampsell and Laura Tracy, civil and environmental engineering seniors at the University of Vermont. Guidance on this project was provided by Jim Pease of the Watershed Management Division, Vermont Department of Environmental Conservation, and Agency of Natural Resources. Background information and data was compiled from several documented sources. Computer software such as HydroCAD, ArcGIS, and AutoCAD was also used to model the study area and help produce stormwater infrastructure designs.

1.2 Project Location and Description

Farrell Brook in South Burlington runs between impervious areas off of Route 7, beneath roads and train tracks, and eventually west through private property before discharging into Shelburne Bay as depicted in *Figure 1* below. The watershed (also depicted in *Figure 1*) encompasses a small neighborhood referred to as “the Orchards”, the Orchard Elementary School on the east side of Shelburne Road, about 1.8 acres of Shelburne Road, a cemetery, L&M Park (SW permit 4835-9010), Farrell Distributors (SW permit 3095-9010), the railroad corridor and a private residential property on the west side.

The brook is considered to be an intermittent stream, because it only possesses visible flow during wet seasons and following precipitation events. The following photos show close ups of the degradation that currently exists and the main areas available for stormwater retrofit designs. *Figure 4* (Location 1-Figure 1) is an example of the current streambank erosion and *Figure 3* (Location 2-Figure 1) shows a decrepit culvert and headwall that connects the northern section of the stream to the lower Farrell Brook. *Figure 5* (Location 3-Figure 1) identifies an existing culvert structure and a site of interest for a stormwater retrofit. Shown in *Figure 2* (Location 4-Figure 1), is a wooded area within the city owned cemetery that may be considered for a gravel wetland design or constructed wetland that would not only aid water quality but also provide an opportunity to aesthetically enhance a public area.

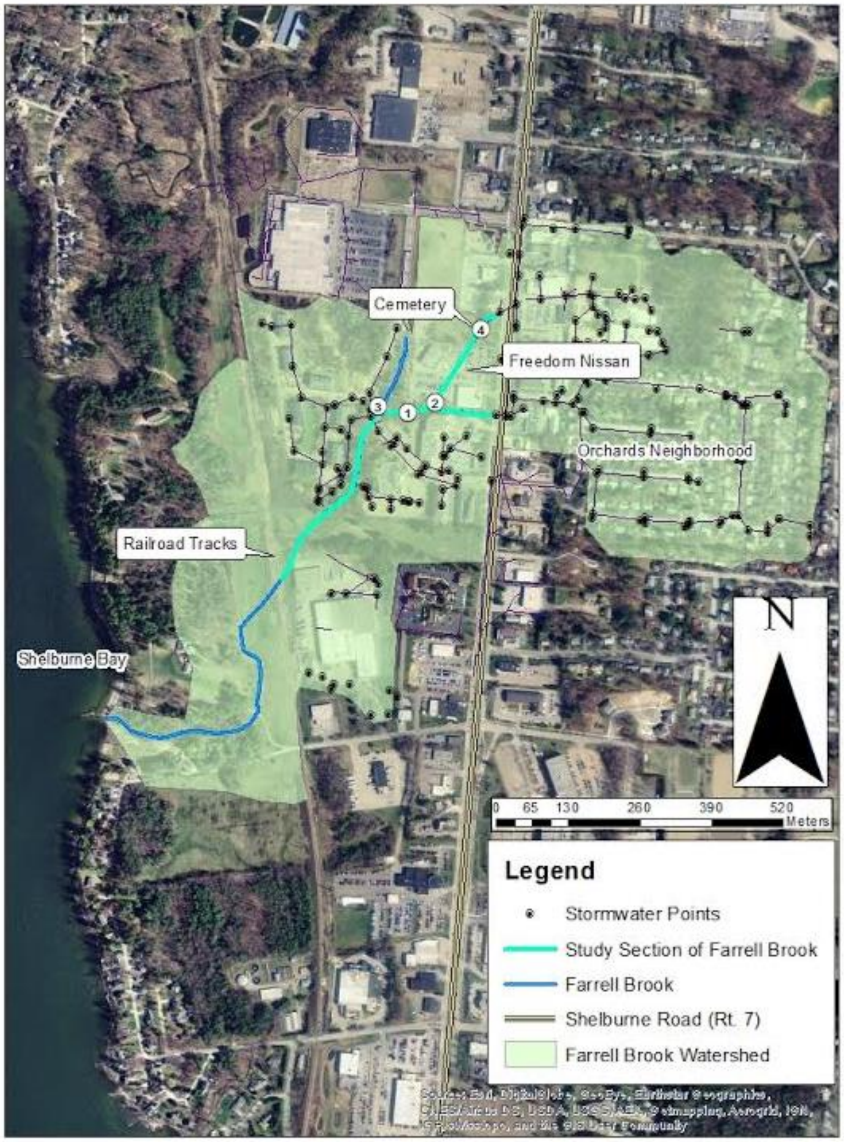


Figure 1: Aerial view of the project study area along Route 7 in South Burlington, VT. Specific features are highlighted for reference and circled numbers identify photo locations below (Location 1-Figure 4, Location 2-Figure 3, Location 3-Figure 5, Location 4-Figure 2)

Figure 2: Wooded area containing Farrell Brook. Historical cemetery is located behind woods

Figure 3: Degraded south end of culvert under Freedom Nissan Parking Lot

Figure 4: Streambank erosion and incision in Farrell Brook

Figure 5: Existing infrastructure on Farrell Brook off of off Fayette Drive

1.3 Main Objectives

- ❖ Reduce volume and discharge rate of flows entering Farrell Brook below Route 7 and before the railroad through the installation of new stormwater management practices, upgrade of existing systems, and repair of eroded or incised stream banks.
- ❖ Ensure that Farrell Brook is managed consistent with the most recent Total Maximum Daily Load (TMDL) for phosphorus entering Lake Champlain by focusing on reducing total phosphorus (TP) and total suspended solids (TSS) to enhance water quality as much as possible.
- ❖ Support the city of South Burlington ordinance that requires all future, new or redeveloped lots to comply with state-of-practice stormwater management practices (Article 12, City of South Burlington Land Use and Development Regulations).
- ❖ Provide a stormwater retrofit plan that is both aesthetically pleasing and educational, as well as beneficial to the health of the environment and the public that live and work in this community.

1.4 Scope of Work

- ❖ Meet with community advisor, define the problem and main objectives
- ❖ Review existing documents including the Stantec report (Gendron and Goyette, 2015) and VT Stormwater Manuals Vol. 1 and Vol. 2
- ❖ Initial site reconnaissance
- ❖ Research & brainstorm solutions, identify main areas of interest
- ❖ Finalize solution conceptions with team
- ❖ Present solutions to community advisor
- ❖ Adjust solutions accordingly to community advisors suggestions
- ❖ Investigate viability of solutions in terms of permitting
- ❖ Investigate theoretical performance of relevant solutions through computer modeling
- ❖ Present options to community advisor
- ❖ Finalize project designs post feedback
- ❖ Present final product and report

2.0 DATA AND BACKGROUND INFORMATION

The existing stormwater piping system in the Orchards has been assessed by Stantec Consulting LTD, an engineering firm in South Burlington, Vermont. Stantec has recommended an increase in stormwater pipe sizes due to problematic roadside ponding and basement flooding that occurs during 1-year storm events in the neighborhood (Gendron and Goyette, 2015). Upsizing the stormwater piping system will reduce flooding in the upstream neighborhood but it will also increase the volume and velocity of discharge entering Farrell Brook and eventually, Lake Champlain. The report provided by Stantec states that peak discharge from the southern outlet to Farrell Brook would increase from 28 cubic-feet per second (cfs) to 73 cfs for a 10-year storm after installing the upsized pipes (Gendron and Goyette, 2015). This is of great concern as the discharge rate from the neighborhood would be almost triple (2.6x) the current state if no flow rate mitigation is provided. Stantec suggested new stormwater management systems that could be implemented to detain the increased flow. Their suggestions include the following: underground detention systems and roadside infiltration systems in the Orchards, and a detention basin downstream.

If the city implements the new Orchards pipe system, the pipe crossing Shelburne Road will increase from 36 inches to 48 inches. The stormwater would then run into Farrell Brook and into an area between two commercial lots that Stantec believes is sufficient in size to construct and grade an adequately sized detention pond, the location of which can be seen in *Figure 1* above Location 3. According to Stantec, in order to bring the increased peak flow from the upsized pipes back down to the current (2016) rate, so as not to increase erosion over that which currently exists, the pond would have to detain 81,000 cubic feet of water (Gendron and Goyette, 2015). Underground detention systems were not feasible as there are few areas that have the necessary holding capacity for the volume the pipe system is designed to detain. It was stated in the Stantec report that 6,400 linear feet of 48 inch diameter underground storage pipe would be needed to accommodate the increased volume. Infiltration trenches and rain gardens in the Orchards Neighborhood were also a proposed solution by Stantec, however, most areas within the neighborhood were discounted due to pre-existing ponding issues that indicate poor soils. This was confirmed as the soil classification in that area is hydrologic soil group D which means that the soils have poor infiltration capabilities (Gendron and Goyette, 2015). Several borings were drilled in the Orchards neighborhood by Stantec to confirm the soil classification and it was also determined that a high water table exists in the area (Gendron and Goyette, 2015).

3.0 ANALYSIS OF EXISTING CONDITIONS

Following the review of background information, in the long term, the existing Orchard's drainage system will need to be redesigned to stop the basement flooding and roadside ponding that was discussed in *Section 2.0*. With the redesign and reconstruction of the pipe network the flows and volumes of runoff will increase. Increased peak flows are expected to increase streambank erosion and further contributing to sediment transportation and water quality in Lake Champlain. Therefore, in order to compensate, we concluded that the best choice was to implement downstream stormwater ponds for detaining the excess volume and improving water quality in Farrell Brook and eventually Lake Champlain. Retrofitting stormwater ponds in the area and upgrading existing ponds will achieve this goal. This conclusion was arrived at from the provided stormwater infrastructure assessment produced by Stantec. Additional treatment would be necessary beyond the Stantec proposal to reduce flows to a more natural hydrologic regime and treat water quality.

A major issue identified within the Farrell Brook watershed was that the majority of the land is privately owned, and therefore already developed. The areas which are not developed are small and tend to be the areas immediately surrounding the brook, which consists of steep slopes and wooded areas. The section of brook on the west side of the train tracks is on private property and, at this time, is not an area of interest in terms of implementing primary solutions, although slope stabilization recommendations will be more valuable. Due to the lack of available space it was concluded that in order to achieve the goals of reducing peak discharge and allowing time for sedimentation, it is necessary to combine a series of smaller solutions as opposed to one, large, stormwater pond.

A hydraulic model was completed by Stantec using HydroCAD (Gendron and Goyette, 2015). Their analysis used the proposed upgraded piping system in the Orchards neighborhood to produce hydrographs for 1-inch, 1-year, and 10-year storms at the inlet of their proposed pond. We used ArcGIS to analyze potential solution areas and determine geographic areas that may be used for detaining and treating the required volume of water. Once design areas were determined, each design was sized to treat the watershed above each specific design point. All watershed analysis was completed in ArcGIS. It was then possible to start designing solutions to meet the necessary water quality volume and peak discharge control following the VT Stormwater Manual.

In addition to hydraulic modeling, it was hypothesized that it could be beneficial to look into acquiring water and soil samples from Farrell Brook in order to quantify the contaminant levels in the soil and water. After discussing this plan with Jim Pease and several Environmental Engineering professors here at UVM, it was concluded that in order for the samples to provide an accurate depiction of the site conditions they would need to be collected over a longer period of time than is available for this project.

After visiting the site it was observed that the current peak discharges are causing significant erosion of stream bank soils as seen in *Figure 6*. In certain locations, culverts are severely degraded. As shown in *Figure 3* the culvert has been partially crushed and the headwall is collapsing. In *Figure 7* the inlet to the culvert is completely submerged and buried under leaves and sediment. The existing conditions suggest an ideal solution would involve reducing peak discharges even further, and requiring stabilization of existing stream banks. In the part of Farrell Brook on the west side of Shelburne Road, where the water runs alongside the South Burlington cemetery, there is no clear flow path. The area where the water is flowing is filled with dirt, sediment, mud, and leaves. It will be necessary to perform some maintenance or retrofits to sections of the brook such as these to ensure proper flow of water through the system.

The proposed upgrades by Stantec will eliminate the basement and road-flooding problem the residents of the Orchards neighborhood are currently experiencing (Gendron and Goyette, 2015). By reducing the peak discharge rate and stabilizing the existing profile of the brook, the quality of the water exiting the brook into Shelburne Bay will improve in terms of sediment, bacteria, phosphorous, and other nutrients currently being dislodged during storm events.

Figure 6: Bank Erosion (South side of Freedom Nissan)

Figure 7: Buried culvert pipe filled with leaves and sediment
(Adjacent to the graveyard on the North side of Freedom Nissan)

4.0 NEEDS ASSESSMENT

Taking into account the current conditions of the watershed discussed in *Section 3.0*, it was determined that the site has a desperate need for both improvement of the water quality and detention if the flows from the Orchards neighborhood increase. This project will focus on addressing the water quality of Farrell Brook and determining ways to decrease the flows entering the brook, preventing further erosion, while also treating the water before it is discharged into Shelburne Bay. To address the problem it will be necessary to come up with multiple alternatives. The final solution may contain a variety of best management practices for stormwater. Some solutions may not be able to completely address the problem. The solution that is the most feasible will be the one that addresses the problem to the highest degree and is also cost effective for the client to pursue. This will be done by combining multiple elements of a stormwater system such as detention and retention ponds, hydrodynamic separators, slope stabilization techniques, and constructed wetlands.

5.0 DESIGN ALTERNATIVES

Each design alternative presented in this section is one that is believed to provide a source of benefit to the overall project goals. According to the Vermont Stormwater Management Manual, "effective stormwater management must include both water quality and water quantity controls" (VTANR, 2002). The five design alternatives that are focused on within this report include, (1) construction of a new stormwater extended detention pond, (2) updating an existing pond that will serve as a retention pond to address water quality, (3) constructing a gravel wetlands in the cemetery area, (4) placing a hydrodynamic separator at the discharge location of the Orchards stormwater network across Shelburne Road (Route 7), and (5) the solution of taking 'No Action'. Each alternative will be compared to each other and in this scenario the final solution will be the most cost effective alternative that also meets the goals of this project. The potential locations of the alternatives are shown in Figure *14*.

5.1 Alternative Design 1: Construction of extended stormwater detention pond on the south side of Freedom Nissan near Fayette Drive

In between the southwest corner of Freedom Nissan and Fayette Drive exists a potential location capable of containing the required volume storage. The runoff from the Orchards neighborhood discharges across Shelburne Road into a swale between Freedom Nissan and the Tilt Plaza. The swale would turn into the pond inlet and the stormwater pond would then discharge through a riser outlet structure and then through a culvert underneath Fayette Drive. This pond would be designed for a 10-year storm event and would be required to detain 81,000 cubic-feet, as proposed by Stantec, in order to bring the increased peak flows back to their current values. Construction of this pond will have to consider available area, inlet and outlet elevations, groundwater table and underground utilities. For a new development it is necessary to grade this pond at a 3:1 (H:V) slope and follow design guidance as presented by the Vermont Stormwater Manual. However, since this is a retrofit it may be possible to design around the guidance and requirements and implement other necessary slope stability measures in order to detain the required volume. This pond would serve as overbank flood control to the downstream water quality retention pond. *Figure 8* below is taken from the Vermont Stormwater Manual indicates an example of a dry detention pond.

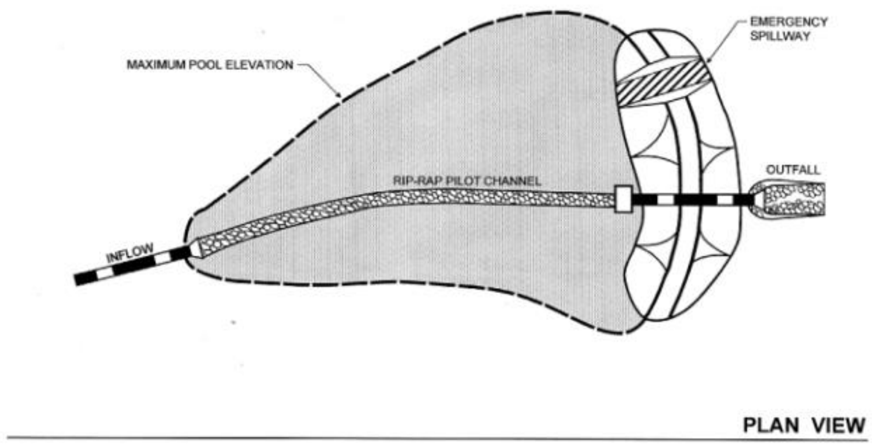


Figure 8: Example of Dry Detention Pond from Vermont Stormwater Management Manual

5.2 Alternative Design 2: Upgrade existing pond on Inn Rd into a stormwater retention pond

This existing pond seen in *Figure 9* is located downstream of proposed Alternative Design 1. It currently covers an approximate area of 8,500ft^2. This pond would be upgraded to provide necessary water quality treatment for Farrell Brook. This site is accessible by Inn Rd and there is no existing forebay or sedimentation pretreatment structure. However, the upstream detention pond would serve as the pretreatment for this pond in order to maximize the area for the water quality volume pool. This retention pond would hold water for a 24 hour period of time to allow for sedimentation to decrease concentrations of phosphorus, nitrogen and total suspended solids. This pond will be designed for the Water Quality Volume (WQv), which is the permanent pool volume, and the Channel Protection Volume (CPv) above the permanent pool. However, it must also be able to safely bypass any storm event over the 1-yr, 24-hr storm without overflowing its banks and flooding the surrounding area (VTANR 2002).

Figure 9: Existing Pond offering potential upgrades

5.3 Alternative 3: Gravel Wetland in Cemetery

Gravel wetlands provide a good green alternative to stormwater ponds as they have higher treatment levels, are safer in design, and more applicable to a wide variety of land types and soils. They work to mimic natural wetlands as they filter, drain, treat, evapotranspire, and slowly discharge stormwater. Beyond their stormwater benefits, gravel wetlands work to provide wildlife habitat, improve air quality, as well as provide educational opportunities and aesthetic benefits. A gravel wetland design is proposed in the South Burlington city cemetery off Route 7 to slow flows and treat stormwater from the north side of the Orchards Community. Represented below in Table *1* is the median data values collected by the Center for Watershed Protection in their 2007 “National Pollutant removal Performance Database Version 3.0”.

Table 1: Comparison of Percent Removal Efficiencies for each applied Best Management Practice (BMP).

Stormwater BMP	%TSS	%TP	%TN	%NO_X	%Cu	%Zn	%Bacteria
Dry Detention Pond	49	20	24	9	29	29	88
Wet Retention Pond	80	52	31	45	57	64	70
Gravel Wetland	72	48	24	67	47	42	78

TSS = Total Suspended Solids
TP = Total Phosphorus
TN = Total Nitrogen
NO_X = Nitrogen as Nitrate (NO_2) and Nitrite (NO_3)
Cu = Copper
Zn = Zinc
Bacteria = Bacteriological Indicators (fecal streptococci, enterococci, fecal coliform, E. coli and total coliform)

5.4 Alternative 4: Hydrodynamic Separator below Shelburne Road

Hydrodynamic separators use swirl concentration or hydrodynamic separation, continuous deflective separation (as shown in Figure *10*) is a combination of hydrodynamic concentration and indirect screening to screen, separate and trap debris, sediment, and hydrocarbons from stormwater runoff. This structure needs to be placed somewhere where it can easily be checked and cleaned. Hydrodynamic separators consume very little land which makes them a good choice for our project given the lack of available space. The Vermont Agency of Natural Resources (VTANR) typically sizes the hydrodynamic separators to the 1-inch, 24-hr storm peak discharge from the watershed upstream of the point of installation. Therefore the size and the overall cost of a hydrodynamic separator would depend on the chosen location. Due to the high discharge rates in this area a high-flow bypass will be needed. The 1-inch storm was used to size the separator. This was completed using Stantec's HydroCAD model. This alternative will be placed where the pipe from the Orchard's neighborhood discharges across Shelburne Road (Figure *14*).

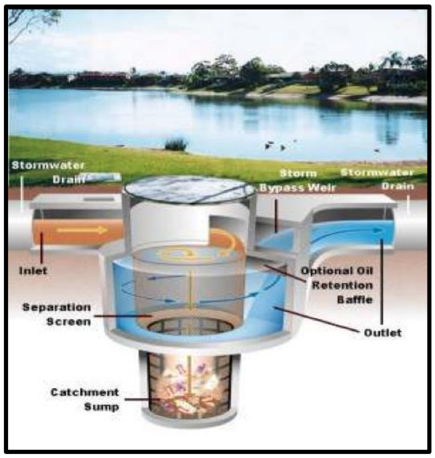
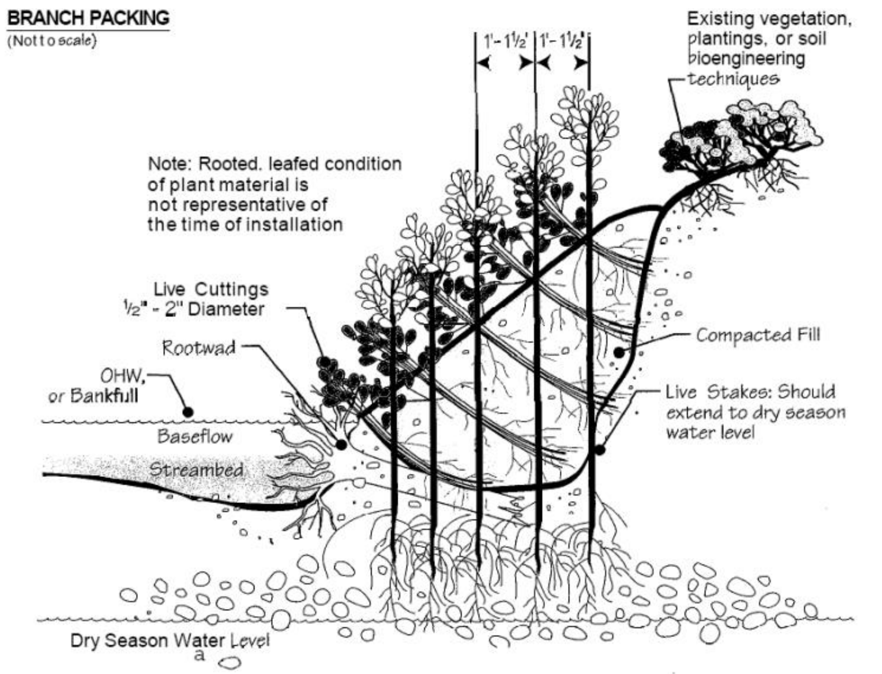


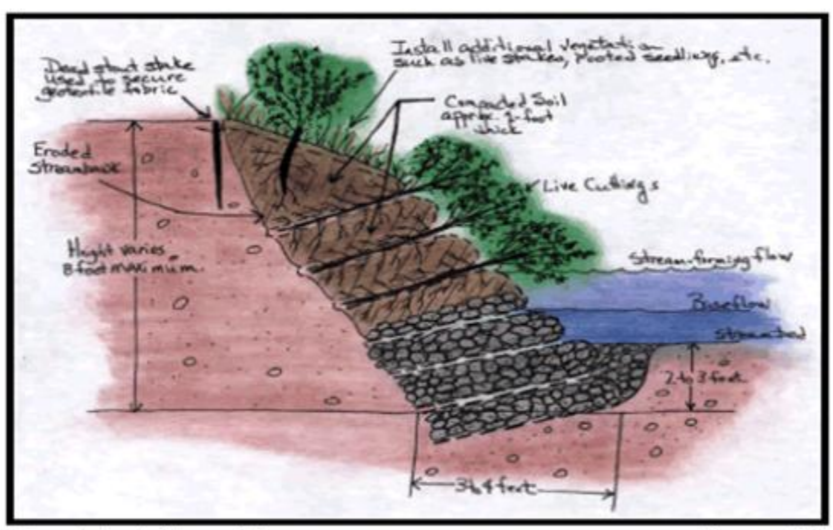
Figure 10: Continuous Deflective Swirl Separator example

5.5 Alternative 5: Slope Stabilization below Railroad

Streambank erosion stabilization structures would decrease the amount of soil being eroded from the brook channel. Erosion of streambanks causes the transportation of sediments and the nutrients attached to the sediments by Farrell Brook and these nutrients are ultimately discharged into Shelburne Bay. However, implementing natural erosion control measures in an unstable brook, stream, or river environment is a risk, and can result in loss of investment. If a significant storm event arises before plants have time to grow and develop their root systems, they may be washed away. It would be difficult to say that any part of Farrell Brook is in a stable enough state to receive such treatment without direct channel measurements, and soil erosion analysis. The current state can be seen in *Figure 12* and Figure 13 below. This option would be of high risk, but of low cost. Stabilization could work after the other stormwater runoff mitigation alternatives are implemented and the hydrology of the 10-year peak flow and channel protection storage volume is more stable. A combination of buffers, bioengineering and toe bank structures designed to withstand the 10-year peak discharge could be installed in the lower reach, further downstream from the engineered ponds. An example of a combination of different bank stabilization techniques can be seen in Figure *11*. A recent study by Stephen J. Burges a professor in civil and environmental engineering at Washington State University, and others, state:

"Aggressive efforts at channel stabilization during the period of active watershed urbanization will probably achieve only limited rehabilitation gains at high and perhaps unnecessary cost, even though bank armoring projects often are constructed in the name of stream-habitat "improvement." Most lowland channels achieve a stable physical form some years or decades following urbanization, with or without human intervention." (Booth, 2001)





Vegetated Geogrids **Figure 32**
adapted from U.S. Department of Agriculture

Figure 11: Examples of bank stabilization using eco-friendly strengthening methods.


Figure 12: Stream banks below railroad tracks though field


Figure 13: Stream entering woods below railroad

5.6 Alternative 5: "No Action"

The "No Action" approach is one that is considered as a comparison to all alternatives. If Farrell Brook is left as it currently stands, environmental degradation of the watershed will continue to occur at more dramatic rates. If the Orchards stormwater system is updated, flows in Farrell Brook are predicted to almost triple and it is expected that erosion and nutrient concentration in the stream may see the same response. Even if the Orchards system is not upsized, the flows through Farrell Brook are causing harmful degradation as they currently exist. With new TMDL regulations by the Environmental Protection Agency (EPA) for Lake Champlain and new state requirements for stormwater control through Act 64, the "No Action" alternative really is not an option. It may even be considered that this alternative has high cost as it would be in violation of state regulations. There may also be property value loss as the land is eroded and the lake further polluted.

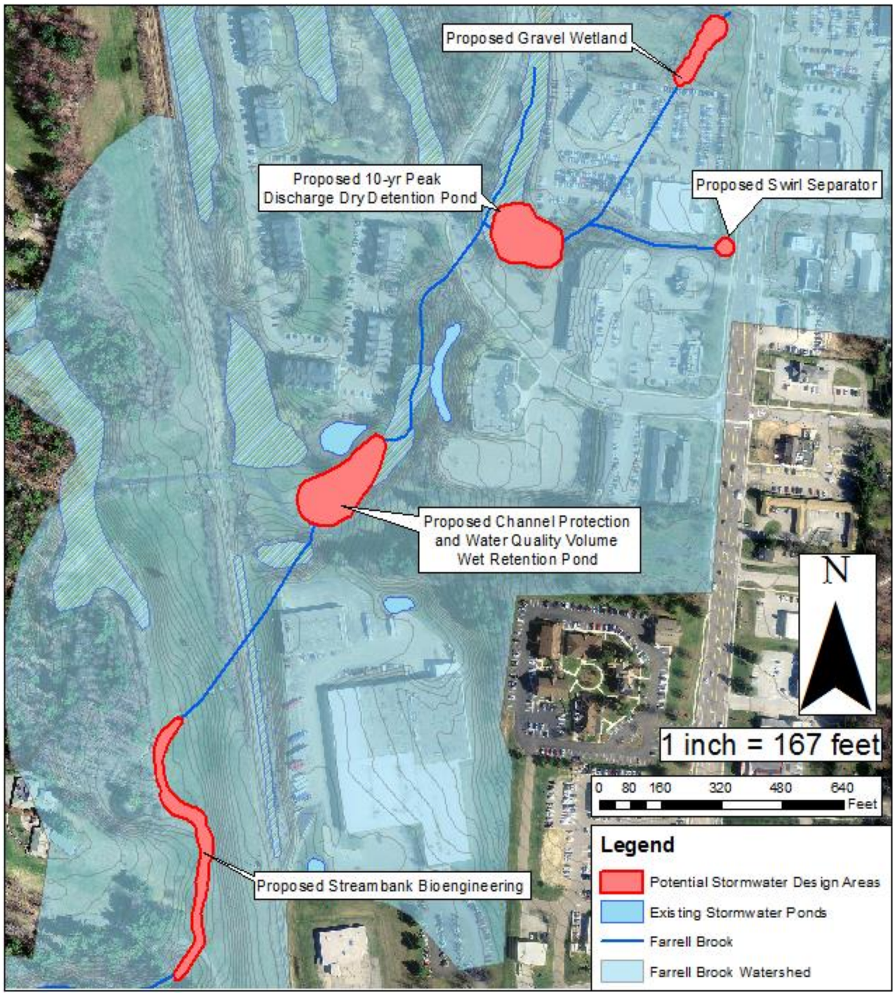


Figure 14: Map of the study with specified alternative design area locations

6.0 SUSTAINABILITY, RISK, UNCERTAINTY, AND LIFE-CYCLE PRINCIPLES

6.1 Sustainability

When a design is suggested the sustainability of that design must always be considered. Any design will have both beneficial and negative impacts on the surrounding communities and the environment. If the new pipe network designed by Stantec is implemented, the negative impacts to the surrounding area must be dealt with. The new pipe system will be beneficial to the Orchards neighborhood by removing the flooding and ponding issues, but the downstream area will then be negatively impacted by increased flows in a stream that is too small to handle them. This is a common issue in stormwater treatment, as most designs deal with the immediate problem but do not consider the downstream impacts. In this case Stantec suggested several alternatives to slow the flows in order to avoid negatively impacting the downstream areas. This project takes this one step further and also looks to improve the water quality while also returning the stream to its "normal" flow rate before it can erode the downstream banks.

Improving the water quality allows for a much more sustainable design. Lake Champlain is currently experiencing an overabundance of phosphorus from many different sources, including agricultural runoff, streambank erosion, and effluent from wastewater treatment plants. The majority of the runoff (66%) comes from Vermont non-wastewater sources, making the improvement of Lake Champlain's health rest heavily on Vermont's landscape (EPA, 2015). Phosphorus is healthy for the lake ecosystem in small quantities, allowing plants to grow and thrive, however, when there is an extreme excess of phosphorus, eutrophication (a decrease in in dissolved oxygen) can occur. Eutrophication is harmful for the fauna and can also cause an increase in harmful algal blooms which contain toxins that are harmful to fish, other aquatic life, and humans. To ensure the lake does not become a toxic environment, the state of Vermont issues TMDL's for phosphorus to control the amount entering the lake. Shelburne Bay alone receives 10.2 metric tons per year of phosphorus from its contributing sources (EPA, 2015). The overall load for Lake Champlain is required to drop from its current load to 570 metric tons per year or 34% of its current state (Smeltzer, 2015). By retrofitting Farrell Brook to include improve water quality, the designs and recommendations included in this report will be a step in meeting the overall water quality standards for Shelburne Bay and, by extension, Lake Champlain.

6.2 Risk

Risk for this design will come from the secondary impacts of the recommended solutions on the environment and public health of the local community. There is always a possibility that an unexpected, high volume, rain event could occur which could overwhelm the stormwater mitigation design and lead to structural failure and increased nutrient pollution in the stream and, subsequently, Shelburne Bay. When stormwater systems are implemented, they have to also take into account the possibility that research will demonstrate that certain systems are not as beneficial to the environment as they were thought to be. The reader should be aware that practices that are accepted standards today may change in the future and this report may need to be updated to reflect changing standards and practices.

6.3 Uncertainty

For this project uncertainty will come from our comprehension of existing data and our analytical interpretation of that data. Measures to decrease this uncertainty included seeking further explanation and aide from our community partner regarding the overall goals of this project. Throughout the process we have scaled back the scope of this project, and narrowed in on a specific area of focus to allow a more feasible final product to take form. When dealing with stormwater runoff and mitigation more specific uncertainties can come from changing rainfall patterns, such as those predicted to occur due to the changing climate. Statistical techniques, through a process called frequency analysis, can be used to estimate the probability of the occurrence of a given precipitation event. The recurrence interval is based on the probability that the given event will be equaled or exceeded in any given year. However, this method of prediction does not guarantee exactly when these storm events will occur; for example, it is possible for a 100-year flood to occur two years in a row. Therefore our design will always have to contend with rainfall variability over time and must be designed and implemented taking this into account. This is true for any stormwater design and while it must be considered in any analysis, it is not always cost effective or practical to design to this level, especially in an area like Farrell Brook where there is limited space.

6.4 Life Cycles

The design life for this project is expected to be 30-years (CNT, 2006). Costs for the project include preliminary engineering, right-of-way acquisition, permitting, construction engineering, final construction, and yearly maintenance. Maintenance requirements for typical stormwater infrastructure are outlined Table *2* below. The Stantec report estimates that the cost of a detention basin off Fayette Dr. would be $120,000 for construction costs alone (Gendron and Goyette, 2015). It was stated by our community partner that a feasible watershed alternative design should be under $1,000,000.

Table 2: Maintenance Requirements for Stormwater Best Management Practices (BMPs)

BMP	**Maintenance Activities**	**Schedule for Maintenance**
Ponds (Dry and Wet) (EPA, 2009)	— Cleaning and removing debris after major storm events (>2" rainfall) — Harvesting of vegetation when a 50% reduction in the original open water surface area occurs — Repairing embankment and side slopes — Repairing control structure	Annually or when needed
	— Removing accumulated sediment from forebays or sediment storage areas when 60% of the original volume has been lost	5-Year Cycle
	— Removing accumulated sediment from main cells of pond once 50% of the original volume has been lost	20-Year Cycle
Hydrodynamic Separator (Contech, 2014)	- Visual inspections	2 times per year
	- Floatables cleanout -Vacuuming	Annually or as needed
Gravel Wetland (Hall, 2009)	-Inspection	1-Year Cycle
	-Sediment Removal	When sediment depth >10cm
Culverts/Pipes (Meegoda and Zou, 2015)	- Check for pipe settling and repair - Check for leaks and repair	3-Year Cycle
	-Cleaning In-Place	Annually or as needed
	-Replacement	Depends on type

The reader should be aware that the costs represented in this report will not delve into the benefits and costs of sustainability. This means that the externality costs of this project will need to be considered without numbers. In the OFA Database, Chapter 10, the fundamental ecosystem services are described as supporting (interactions between abiotic and biotic components), regulating (impacts of eutrophication on water purification, etc.), provisioning (extracted resources of food, water, etc.), and cultural (recreation, aesthetics, etc.) (OFA, 2015). Any impact on one of these services can change the economic development of the lake, such as increased eutrophication creating the need for further purification of the water for drinking purposes and also affecting the aesthetic quality of the lake. Externality costs are very important to consider and should be analyzed along with the direct costs specified above.

7.0 ANALYSES AND DESIGN

Analysis and design of the project was performed using several computer aided programs including AutoCAD, ArcMap, and HydroCAD. AutoCAD was used for the design and development of plans for alternatives one, two, and three which may be reviewed in *Appendix C*. Cut and fill cost estimates for both ponds (alternative one and two) were also calculated using cross-sections drawn in AutoCAD. ArcMap was used for a wide variety of tasks including delineating watersheds and calculating areas, determining impervious area on the site, as well as mapping utilities, soil types, parcels, elevation data, and locating potential design areas. GIS data was collected from the Vermont Center for Geographic Information (VCGI) website, as well as provided by the City of South Burlington, Stantec, and our community consultant Jim Pease. Utility and elevation data was transferred for use in AutoCAD. HydroCAD was then used for watershed flow analysis to determine peak flows for various storm events specific to Chittenden County, VT. Further HydroCAD data was provided by Stantec and their calculated flows were used throughout our analysis. HydroCAD reports may be found in *Appendix D*. All calculations performed for this design may be reviewed in *Appendix B*.

7.1 Alternative Design 1: Detention Pond

The upper pond by Freedom Nissan is proposed to be a dry detention pond. The main purpose of this pond would be to control flows that are discharged from the Orchard's neighborhood and to release them at a slower rate. Stormwater ponds designed for new construction are required to follow the VT Stormwater Manual criteria for design, however this is a retrofit project and is therefore not required to meet all specifications. By creating a site plan in AutoCAD with GIS sourced 2 ft. contour layers, impervious surfaces and spatial imagery, it was found that a pond could fit in the proposed location. However, the area would still lack room for a suggested safety bench of 15 ft. from the normal water edge to the toe of the pond side slope. A fence may be added, allowing this safety bench to decrease to 6 ft., saving space at an additional cost. Another option to consider is extending the area available for construction. This would involve land acquisition from Freedom Nissan and Tilt Plaza to design the pond at a proper volume and 3:1 (H:V) side slopes. This idea poses risks, however, as property owners may be unwilling to sell or the cost to purchase land might be too high to be feasible given the current budget.

The current pond, as designed in AutoCAD, has banks graded at a 3:1 slope with the bottom of the pond occurring at 171 ft. and the maximum water level occurring at 179.5 ft. This allows for half a foot of freeboard at the top of the pond. The downstream berms begins at 180 ft. and is 6 ft. wide before being graded back down to the existing outlet headwall. This provides 0.5 ft. of freeboard above the maximum water level. With the 8.5 ft. water depth of the pond, the volume was calculated based on the average area between 1 ft. contours. Doing this for the entire depth, the volume of the pond was calculated to be 81,750 ft^3.

This dry detention pond design is in the preliminary stages and although it considers grading, volumes, outlet structures, setbacks, and safety benches, there were several educated assumptions made regarding existing conditions, as well as the groundwater table depth and underground utilities. There was no survey performed specifically for this report. All data used was collected from outside resources listed in the first paragraph of *Section 7.0*. From site visits, it is known that currently there is a 36" outlet culvert near the base of the designed detention pond. This culvert runs under a berm and discharges downstream and then drains through a larger 60" culvert that runs beneath Fayette Drive. The exact location of this culvert has not been determined, however, an outlet structure for the pond is designed to directly tie into the existing 36" culvert. It is important to tie the outlet structure into the existing culvert so as not to have to reconstruction the berm and the existing culvert. This would increase construction cost as well as posing the threat of working with underground utilities as there is both water and sewer lines that run through the berm. The proposed grading is seen below in Figure *15*.

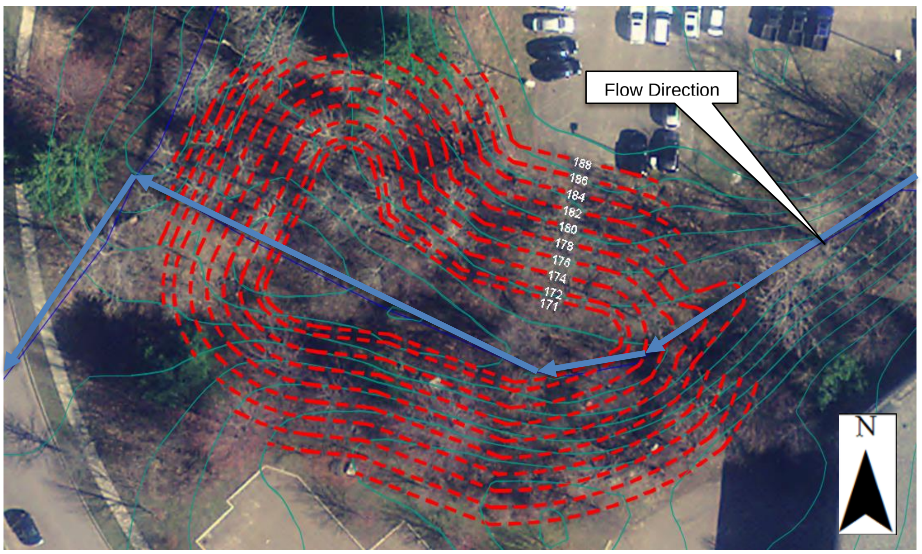


Figure 15: Proposed dry detention pond with grading

The outlet structure for this pond restricts flows to lower than 28 cfs, the pre-development flow rate for the Orchards neighborhood. This is accomplished by channeling water through an 18" orifice of the outlet structure at the bottom elevation of the detention pond. If the pond reaches its maximum volume, water will begin to spill over the top of the outlet structure through a grated opening and continue downstream. This will only occur during large storm events, higher than the 10-yr storm. The 18 in. pipe was determined using an iterative solution of the flow through an orifice equation. These calculations may be reviewed in *Appendix B*. The max flow out of the 18 in. orifice with the pond at its maximum depth was calculated to be 23.7 cfs. This flow would drain the full volume of the pond in approximately one hour. The length of time it takes to drain the pond is less important than the flow at which the water is discharged. Since this is a detention pond the design is focused on controlling flows and not sedimentation time, as will be focused on with the downstream retention pond.

7.1.1 HydroCAD Analysis for Various Land Cover Conditions

Using HydroCAD, peak discharges for the 10-yr storm were modeled for watersheds with various land cover types for comparison. The model parameters used can be seen in Table *3*. The models produced peak flow values for cover types representative of a natural wooded landscape and a less developed area. Orifice sizes for the outlet riser were then calculated based on the acquired peak flows. These were used as comparisons for what the orifice size of the outlet riser would need to be to bring flows in the stream back to a natural condition. More natural flows would then be beneficial for slope stabilization and bioengineering of streambanks. Once a standard orifice size was chosen, the actual peak flow through that orifice was calculated. The results of the HydroCAD flows and the orifice sizes can be seen in Table *4*.

Table 3: Model Parameters used in the HydroCAD Model

Model Parameters	
Flow	10-Year Peak Discharge (24-hr storm)
Methods	SCS TR-20
	UH = SCS
	Weighted Q (Varies based on soil type)*
Total Acreage**	80.6 acres
Flow Length	3,219 feet
Average Slope	3.56%

* The weighted Q method is required in the 2016 Stormwater Manual
** The total watershed area was broken into the four hydrologic soils A through D and appropriate acreages were assigned to each. The HydroCAD reports along with a soils map of the area may be found in Appendix D.

Table 4: HydroCAD model outputs and the calculated orifice sizes needed to approach the output peak flows

Model	**Modeled Peak Discharge**	**Outlet Orifice Size**	**Actual Peak Discharge through Orifice**
Natural*	7.72 cfs	10 inch	7.46 cfs
Residential 1-acre (20% impervious)	21.5 cfs	15 inch	16.6 cfs

* Assuming wooded areas and grasslands, no impervious surfaces.

7.2 Alternative Design 2: Retention Pond

The lower pond off of Inn Road is another key component in this stormwater mitigation design. Currently, a pond exists in the design location. This alternative proposes to retrofit the current pond to hold, and treat the water quality volume, and channel protection volumes mandated by the VT Stormwater Manual. As discussed in previous sections this is an important aspect of this project since water quality in Lake Champlain is a key environmental and public health concern. Since this pond is located directly adjacent to a private road the design also needs to be aesthetically pleasing. Seen below in *Table 5* are the general design requirements for calculating the water quality volume.

Table 5: Required Stormwater Treatment Standards
(From Table 1.1 of VT Stormwater Manual Vol. 1)

Criteria	Sizing Requirement
Water Quality (WQ_v)	90% Rule: $WQ_v = [(P)(Rv)(A)] / 12$ expressed in acre-feet when A has units of acres where: P = 0.9 inches R_v = Runoff Coefficient = [0.05+0.009(I)] I = Impervious Cover (whole number percent) A = Site (area in acres) Note: Minimum WQ_v = 0.2 inches (0.0167 ac-ft.)

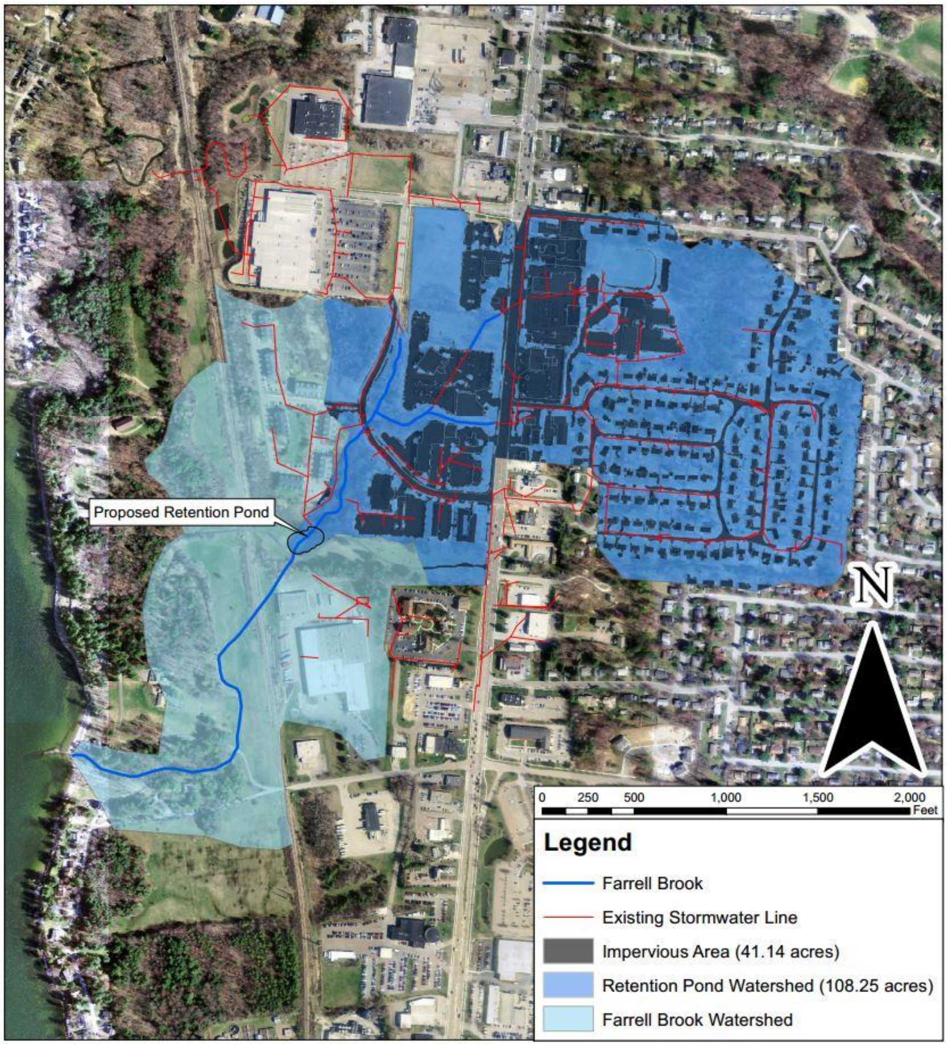


Figure 16: Map depicting the retention pond watershed and impervious area

Referencing *Table 5*, the site area for this pond was taken to be the portion of the watershed above the outlet of the pond. GIS was used to determine this area as well as the amount of impervious cover. The water quality volume was calculated to be 3.18 acre-feet (139,000 ft^3) as can be seen in *Appendix B*. The channel protection volume corresponds to the volume of water falling over the watershed from a 1-year storm event. Due to the complexity of the watershed and the lack of available field data the channel protection volume calculation was derived from the existing HydroCAD model produced by Stantec. Therefore, the channel protection volume was calculated for the portion of the watershed corresponding to the orchards neighborhood. The model, which includes the Orchards neighborhood with upgraded pipe sizes, produces a storage volume for the 1-yr storm event of 33,578 ft^3, as seen in the output hydrograph in Figure *17* below.

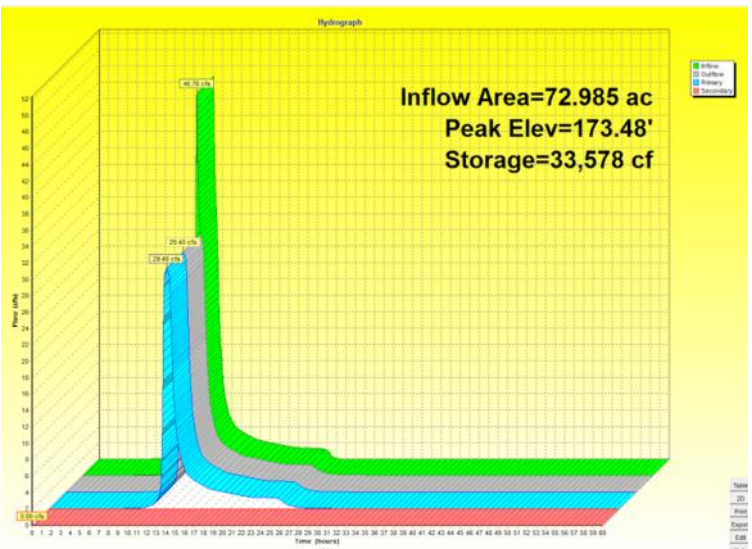


Figure 17: Output hydrograph from Stantec's HydroCAD model for 1-year storm

The pond slopes are graded at a 3:1 (H:V) slope as suggested by the stormwater manual. Further grading was completed to tie the pond back into the existing contours. A grading plan and overview of the pond is shown in Figure *18*. The pond has a bottom elevation of 136 ft. and a top elevation of 146 ft. The maximum water level occurs at 145.6 ft. which allows for 0.4 ft. of freeboard.

The retention ponds consist of water levels for a permanent pool, the water quality volume (WQv), and channel protection volume (CPv). The permanent pool accounts for roughly 50% of the WQv and sits below the inlet and the outlet of the pond and is the constant water level throughout the year. The remaining amount of the WQv sits above the permanent pool and the CPv sits above the WQv.

Table 6 below indicates the water level, the elevation each occurs at the storage at that level and the total cumulative storage for the pond. A cross-section of the retention pond can be seen in *Appendix C: Sheet B3* that illustrates the water levels and their corresponding storage volumes.

Table 6: Retention Pond Water Level Designs

Water Level	**Elevation (ft.)**	**Storage (ft^3)**	**Cumulative Storage (ft^3)**
Permanent Pool	141	70,150	70,150
WQv	144.2	68,850	139,000
CPv	145.6	33,300	172,300

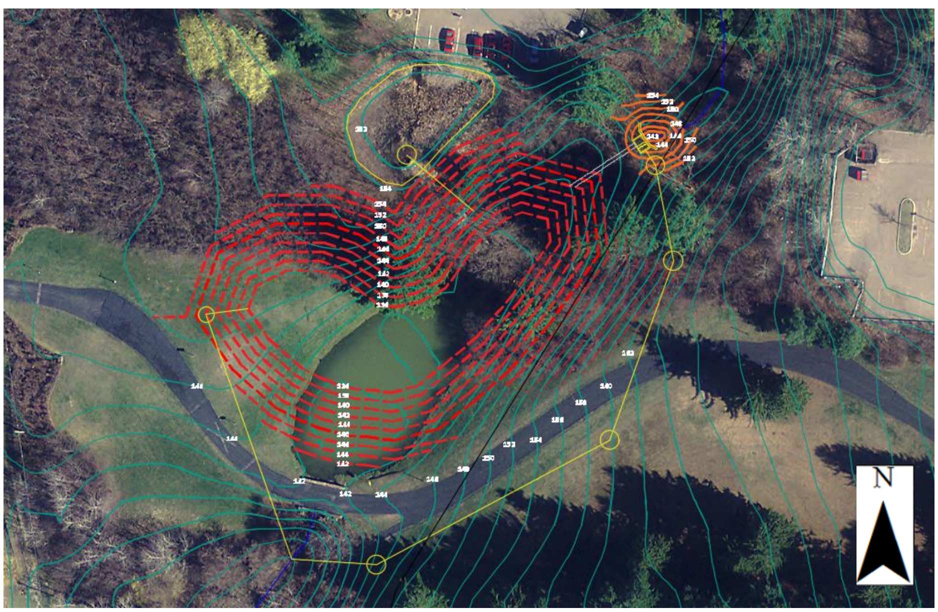


Figure 18: Proposed wet retention pond with grading, showing piping and structure outlines

The proposed outlet structure is located in the southwest end of the pond to maximize the distance between the inlet structure and the outlet for proper flow. The outlet structure consist of a reverse pipe to drain the WQv above the permanent pool and an orifice to drain the CPv. The outlet structure is designed for a 24-hour retention time of the WQv and CPv to allow for sedimentation. It also includes a gated valve at the bottom elevation of the pond to drain the pond for maintenance, as well as an overflow drain at the top of the riser for when the pond reaches max capacity. The discharge from the outlet structure must be piped southeast across the Inn Rd. back into Farrell Brook.

The inlet structure to the pond is designed to let a maximum flow of 14.5 cfs into the pond. All higher flows will be rerouted through a spillway by the use of a flow splitter. The inlet pipe to the inlet structure is sized at 48 in. which will allow for flows higher than the 10-yr storm. Incoming flows are then restricted to a 30 in. outlet from the structure into the pond. Any flows that do not make it through this pipe will be forced over a small wall in the inlet structure which acts as a weir. Additionally the height of the weir is set such that when the pond reaches max capacity the water will begin to spill over the weir even if it is flowing in at a rate less than 14.5 cfs. These excess flows will be discharged to a 48 in. outlet from the structure that is routed around the pond on the east side. This prevents high flows from entering the pond and disrupting the sedimentation process. Due to space restrictions and an existing sewer line in the area, this piping outlet must be routed under the road in a second location. This is not ideal, but the only foreseen option. There is also an existing pond to the north behind this proposed pond that will discharge into this pond although the exact location was not verified. All calculations and design details are available for review in *Appendix B and C*, respectively.

7.3 Alternative Design 3: Gravel Wetlands

Commented [KB1]: Same as above-use illustrations and less words

A gravel wetland may also be considered as a further design element to enhance water quality and reduce the pretreatment requirements of the lower ponds discussed in *Section 7.1* and *7.2* above. As stated in the Vermont Stormwater Manual Vol. 2, gravel wetlands are well-suited for roads and highways as well as commercial areas. They are also well suited for all soil types and may be implemented below the water table. The main purpose of a gravel wetland is to improve water quality. The manual suggests that gravel wetlands remove 83% of TSS and 64% TP. These levels are met as water flows into the pre-sedimentation basin, then the main basin where native wet-tolerant plants slow flow rates and allow for sediment and pollutant settling. Through biological processes in the sediment and the plant roots, nitrogen, phosphorus, heavy metals and bacteria may undergo microbial transformation and uptake. The wetland outlet is therefore sized to slowly release the water quality volume for the design over a 24-hr period.

A gravel wetland was chosen specifically for the cemetery location as there are slower flows coming from the northern Orchard's neighborhood. It would also provide an aesthetic and educational natural environment that would well suit a public cemetery, enhancing the overall atmosphere. Gravel wetlands are also safe, with a permanent pool of only 3", there is no need for fencing. A bridge will need to be constructed from the road side of the cemetery to the grave plots. The wetland would include a variety of Vermont native plants designed to withstand cold climates, shade, high salt levels, and occasional deep flowing water. These plants may include gray sedge, windflower, daylilies, blue-flag iris, spiked lobelia, goldenrod, black-chokeberry, silky dogwood, pussy willow, great rhododendron, and highbush blueberry, among others. There are also several trees already in the area that would be replaced around the outskirts of the wetland, maintaining privacy between the road and the cemetery.

From GIS analysis the water quality volume for the area draining to this design was calculated to be 28,423 ft^3 using the same method as seen in *Table 3* above. This is based off the total watershed area and total impervious area shown in Figure *8*Figure *19* below. The watershed was determined to have 40% impervious cover. These calculations follow those suggested in the stormwater manual and may be reviewed in *Appendix B*. Due to space constraints in the cemetery, the wetland was designed to maximize the available construction space. This size wetland allows for the treatment of one half the water quality volume. All calculations for the pond were then based off a water quality volume of 14,211 ft^3.

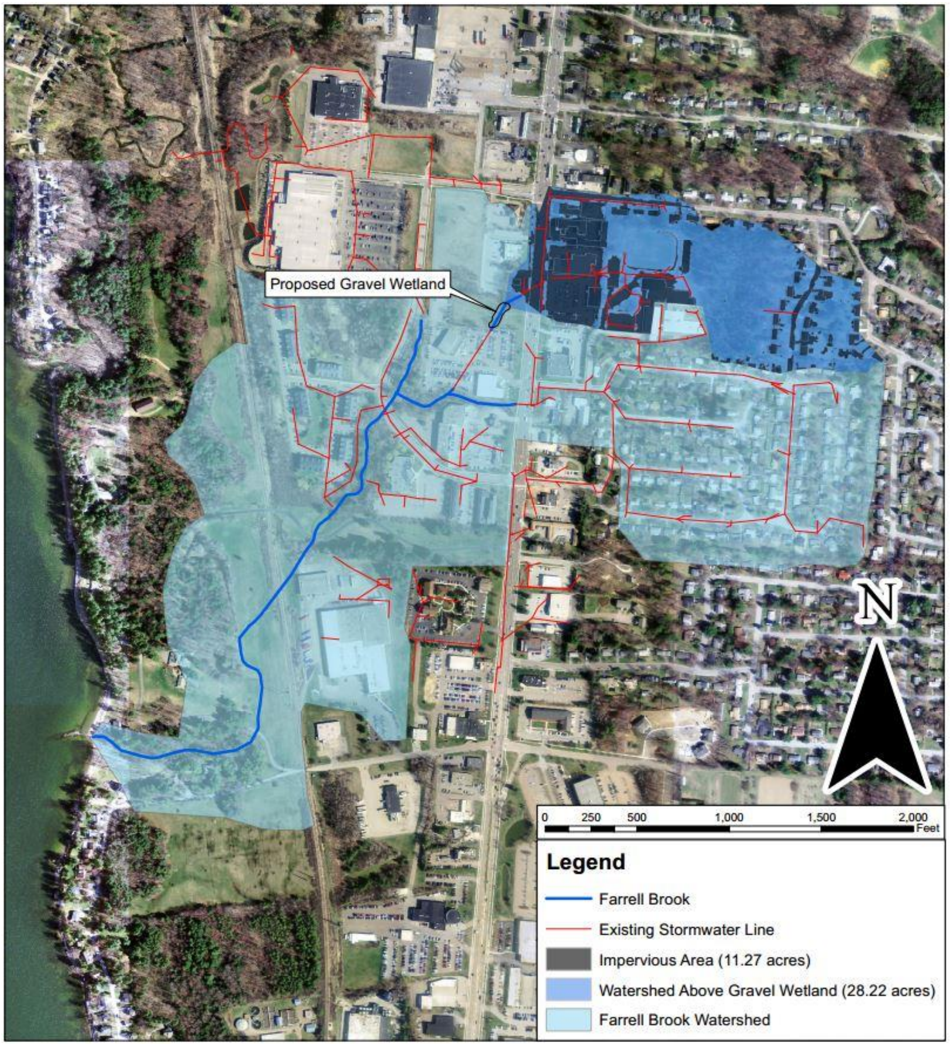


Figure 19: Map depicting the gravel wetland watershed and impervious area

The pre-treatment, forebay, is designed to treat 10% of the water quality volume and is sized to be 20 ft. by 24 ft. with a 3ft. depth. A perforated pipe in the forebay allows water to flow in and through a solid underground pipe to the main wetland bay. The main wetland is sized to be 90% of the water quality volume, and designed with a 40 ft. by 107 ft. by 3 ft. bed with a maximum permanent pool of 3 in. Below the permanent pool is 3 ft. of designed wetland consisting of a 27 in. layer of gravel on the bottom, followed by a 3 in. layer of pea stone, and a 6 in. layer of organic soil. The outlet is at the bottom of the gravel wetland and is a gated valve with a 3.5 in. diameter. This outlet was designed to allow the water quality volume to drain over a 24-hr period. Calculations may be found in *Appendix B*. The outlet structure consists of a riser with the described bottom orifice and a grated top at the maximum height of the pool to allow for overflow during large rain events. An oversized outlet pipe allows water to flow from the riser and tie into the existing 36 in. culvert at the north end of the Freedom Nissan parking lot.

A grading plan and a general overview of the wetland is represented in Figure *20* below. Banks within the wetland, where water may pond, were graded to 3:1 (H:V) slopes and a 15 ft. boundary of 6:1 (H:V) slopes was graded around the perimeter of the wetland as suggested by the manual. Due to the 15 ft. boundary, shallow slopes, and low permanent pool this wetland does not require fencing. Although this design is feasible, it was not designed to the same level of detail as the previous two ponds. This wetland serves as another option for stormwater retrofit, but was not intended as the focus of this project.

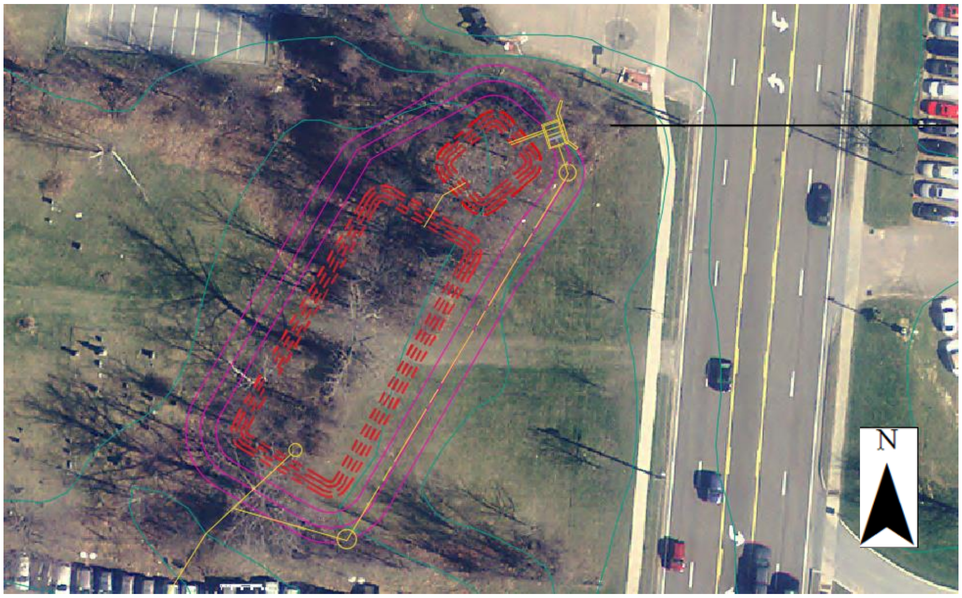


Figure 20: Proposed gravel wetland with grading

7.4 Design Alternative 4: Hydrodynamic Separator

A hydrodynamic separator is proposed immediately below where Farrell Brook crosses Shelburne Road. Using HydroCAD, the 1 inch storm, inflow was calculated to be 9.1 cfs for the upstream drainage area. The following types of hydrodynamic separators that can meet this need are listed below in *Table 7*, along with their costs and general design criteria. It was recommended by Contech (the manufacturer of the Vortechs and the Continuous Deflective Separator, CDS) that the CDS might not have a large enough internal bypass to handle larger flows coming out of the Orchards neighborhood and that the Vortechs unit would be a better choice for that reason, therefore we recommend choosing the Vortechs model. A layout of the swirl separator can be seen in Figure *21* below. The location is shown in Figure *14*.

Table 7: Applicable Hydrodynamic Separators

Hydrodynamic Separator Brand	Model	Unit Cost	Removal Rates	Min. Rim to Invert Depth	Sediment Storage	Oil/Floatables Storage
Downstream Defender	10-ft	$42,000	-90% of particles larger than 150	5.0ft	8.70CY	1050gal
High Eff. CDS	5653-10	$67,650	-100% Trash and particles greater than 0.048in. -100% Floatables	6.0ft	8.7CY	1309gal
Vortechs	7000	$57,125	-80% of TSS	3.0ft	4.0CY	N/A

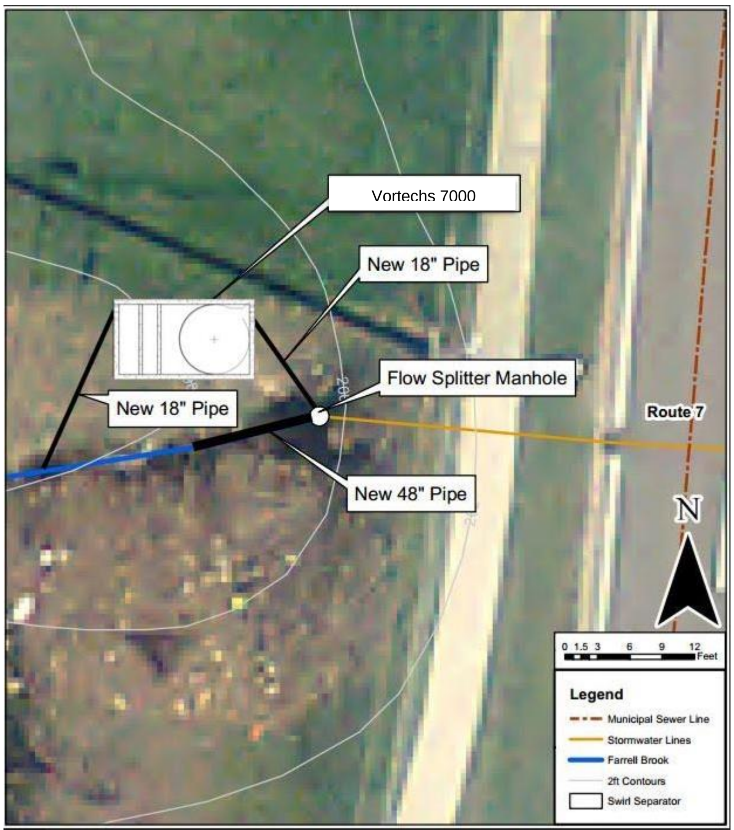


Figure 21: Layout of Vortechs Unit below Route 7.

7.5 Design Alternative 5: Slope Stabilization

Existing stream channel slope stabilization may be implemented to reduce environmental degradation in the watershed. This effort would only be applicable if flows in the stream are restored to a more natural state. The following is a conceptual design for practices that may be considered after some upstream mitigation has been put in place in the stream.

Bank shaping would be required in conjunction with other methods. This involves the removal of soil to reduce the slope of very steep banks to a more stable angle (3:1). This would be beneficial in some areas along Farrell Brook where steep, heavily eroding slopes are an evident issue. Bank shaping allows for other stabilization techniques to be implemented more successfully if the existing slope is stable. Live planting techniques make use of natural sources for stream stabilization (Figure *22*). By using trees, shrubs, and other vegetation to stabilize banks toe protection, upper bank protection, and runoff control would be provided. Stakes or erosion control matting may be required during root establishment to ensure proper planting. Branch packing can drastically affect runoff levels when live branch cuttings are incorporated into compacted soil along the banks.

There are a wide variety of bioengineering techniques (Figure *23*) including vegetated geogrids. These consist of alternating layers of live branch cuttings and compacted soil layers wrapped in geotextile fabric. Vegetated geogrids have a high cost but help rebuild and vegetate eroded banks and can be installed for steeper slopes along bends of a stream. Brush mattresses are a stabilization technique that involves live branch cuttings covering an entire stream bank and secured in place. This will provide immediate complete cover and long-term stabilization. Another method is called tree revetments, which involves placing rows of cut trees anchored to the bank, mainly offering toe protection. Coconut fiber rolls are flexible logs made from coconut hull fibers that are staked at the toe of a bank. This technique can be used to trap sediment and encourage native plant growth along the stream. Good native species for buffer planting, stakes or mattresses are native willows, Silver Maple and Red Osier Dogwood (Tennessee Valley Authority).

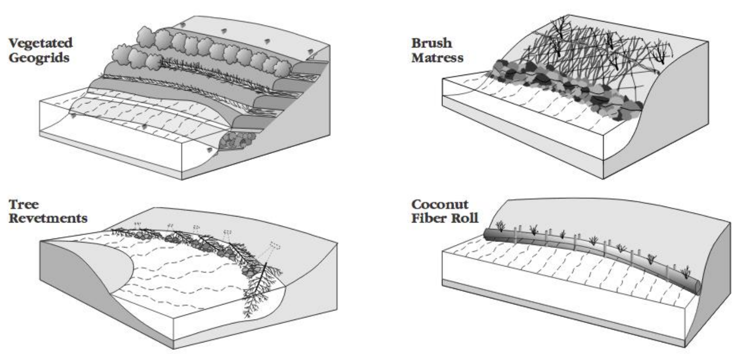


Figure 22: Visual representation of live planting stabilization techniques.

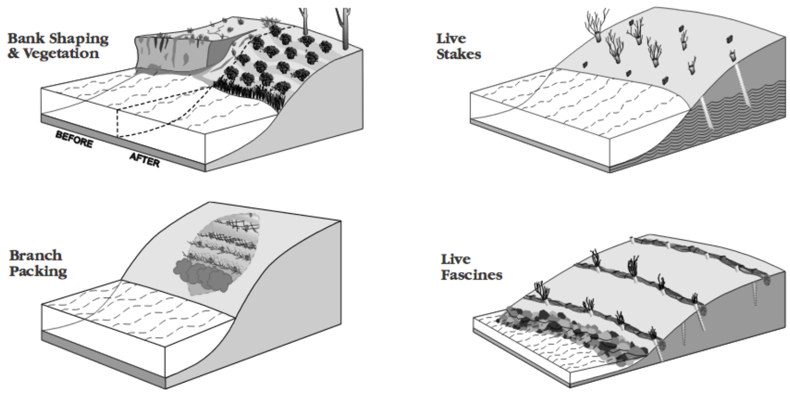


Figure 23: Visual representation of bioengineered stabilization techniques.

8.0 PERMITTING AND RIGHT OF WAY

8.1 Permitting

Permitting is required in projects such as this one due to the sensitive nature of the environment. While Farrell Brook appears to be a stream, the upper reach above the railroad tracks is not considered as such due to it not being perennial. This allows Farrell Brook to be exempted from requiring a stream channel alteration permit, however, an example of a general stream alteration permit is included in *Appendix E* for reference. The area surrounding the brook contains several Class 3 wetlands. If the project area is too close to a wetland then it will be necessary to contact the Wetlands Office to see if a permit will be required as specified under *Title 10 V.S.A, Chapter 37, Section 905(b)*. The areas where both ponds were designed does encroach on these wetland areas so wetland permits from the Department of Environmental Conservation (DEC) and the Army Corps of Engineers will be further discussed. A stormwater construction permit may also be necessary if our project disturbs over one acre. Another possible permit is an Act 250 permit amendment for the adjacent landowners. The partially completed permit may be found in *Appendix E*. To ascertain whether there is already an existing Act 250 permit for the area, or whether a permit is needed, the Act 250 office will need to be contacted. The general permits that may be needed were researched and links to them are included in *Appendix E*. A breakdown of permits and their costs are shown in Table *9* below.

Figure 24 shows the locations of both existing and pending permits in the area surrounding the stream. The corresponding existing stormwater permits for these locations are listed in Table *8* below with the full permits shown in *Appendix F*. L&M Park currently has an Act 250 shown in Table *8* permit which could be amended to encompass this project. The site plans for this project will need to be submitted to the South Burlington Design Review Board before any further design can be completed. In addition, certification that the designs proposed are compliant with the Vermont Stormwater Management Manual will need to be attained. Stormwater treatment worksheets and analysis guidance provided by the state of Vermont will help aide in compliance assurance.

Table 8: Existing Stormwater Permits

Name	**Permit Number**	**Status**
Farrell Distributing	3095-9010.R	Issued
Farrell Distributing	3095-INDS	Issued
L & M Park	4835-9010	Issued
Southland Plaza	5579-9010	Issued
L & M Park	Act 250 ID: 4C0877	Issued

Table 9: Permitting Fees

Permit Type	Administrative Fees	Other Applicable Fees
Wetland Permit	$240	-Wetland Impact Fee: $0.75 per sq. ft. -Wetland Buffer Impact Fee: $0.25 per sq. ft. -Application Fee: $50 (If impact fees < $50)
*Act 250	N/A	-Application Fee: $6.65 per $1000 of construction costs -Other Fee: $100 per lot -Other Fee: $0.75 per $1000 of construction costs for ANR
Stormwater Construction Permit	$240	-GCP 3-9020 Low Risk < 5 acres: $100

* Municipal and state projects are exempt from Act 250 fees

8.2 Right of Way

New construction always impacts the surrounding area, so it is necessary to keep track of which properties will be impacted by the design. A table of parcels within 100 feet of Farrell Brook are attached in *Appendix I*. In particular, the properties that will be involved the most are Freedom Nissan, L&M Park, and the Farrell Property. These properties are shown in Figure *25*. According to the City of South Burlington Land Development Regulations, South Burlington requires a site plan review and approval if there is significant changes to a parcel of land. Therefore it would be required to ascertain approval from Freedom Nissan, L&M Park, and the Farrell Property in regards to project implementation for the retention and detention ponds as well as any slope stabilization methods used. Since the gravel wetland in the cemetery and the swirl separator are located in public right of ways site plan approval would not be necessary since these projects would be installed in collaboration with the city, ultimately having South Burlington own and maintain them upon completion of construction.

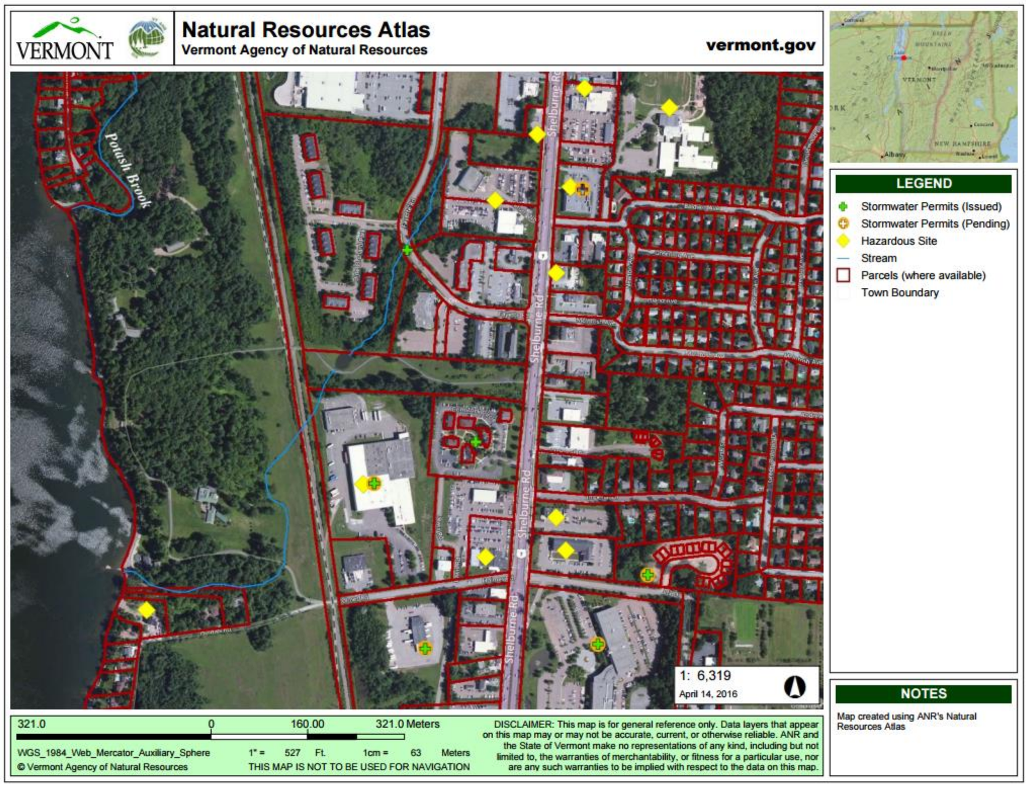


Figure 24: Map showing parcels, hazardous waste sites, and existing and pending stormwater permits.

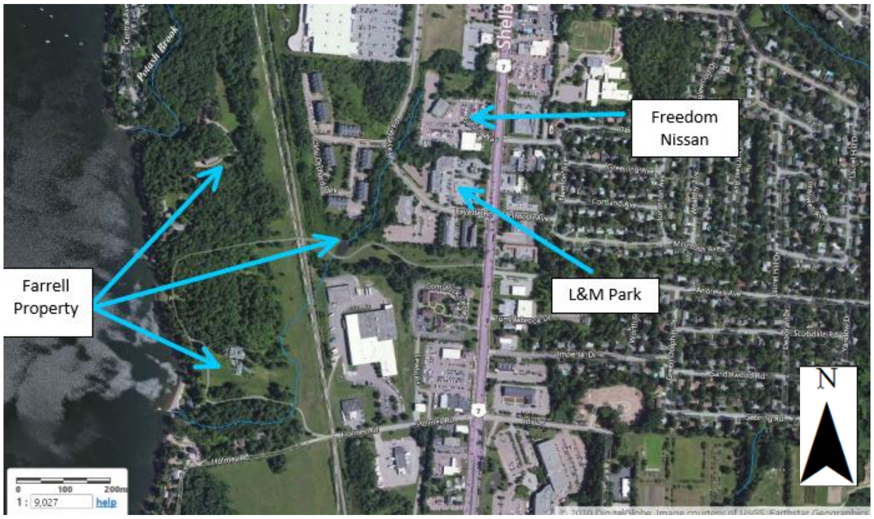


Figure 25: Location of most impacted properties

9.0 COST ESTIMATES

The estimated costs are summarized in Table *10*. In *Section 9.1* through *Section 9.4* the details for each cost are described. In Table *11* the total costs are listed. Construction costs remain under budget if the lower estimate is used, both for all alternatives combined and for just the ponds. If the higher estimate is used both alternatives are over budget. We recommend constructing the detention pond first and, if money is available the retention pond should also be built. The gravel wetland and swirl separator could be implemented to further improve water quality but are not as important as the detention and retention ponds.

Table 10: Overall Costs for Each Alternatives*

Design Alternative	**Cost Breakdown**	**Overall Cost in Present Worth (Design Life of 30 Years)**
Detention Pond	Maintenance	$500,000
	Construction (20% Contingency)	$173,000-$395,000
Retention Pond	Maintenance	$590,000
	Construction (20% Contingency)	$368,000-$832,000
Gravel Wetland	Maintenance	$257,000
	Construction (20% Contingency)	$141,000
Vortechs	Maintenance	$28,000
	Installation and Model 7000 Unit (40% Contingency)	$67,000
*Construction costs include all pre-construction costs (permitting, engineering, design, etc.)		

Table 11: Total cost comparison (30-year design life) for implementation of either all alternatives or only the ponds

Combination of Alternatives	Lower Estimate	Higher Estimate
Total Construction Costs (If all alternatives are implemented)	$749,000	$1,435,000
Total Maintenance Costs (If all alternatives are implemented)	$1,375,000	
Total Overall Costs (If all alternatives are implemented)	**$2,124,000**	**$2,810,000**
Total Construction Costs (If only the ponds are implemented)	$541,000	$1,227,000
Total Maintenance Costs (If only the ponds are implemented)	$1,090,000	
Total Overall Costs (If only the ponds are implemented)	**$1,631,000**	**$2,317,000**

9.1 Detention and Retention Pond System

This alternative consists of putting a detention pond upstream and a retention pond downstream. A detention pond to retain the required 81,000 ft^3 costs approximately $120,000 to construct (Gendron and Goyette, 2015). This cost does not include any other aspects of building the pond such as permitting and engineering. In Table *12* the cost estimate for the detention pond from Stantec is used to extrapolate a value for the retention pond. The maintenance costs and pre-construction costs (design, engineering, permitting, etc.) are shown as well. The total cost (including construction, maintenance, and pre-construction costs) over the 30-year design life is also shown. Table *13* shows an alternative method which results in total costs that are about 1.4 (detention pond) and 2.3 (retention pond) times higher than those extrapolated from Stantec's estimate. The two costs were used together to give an approximate range for the cost of building each of the two ponds.

Table 12: Cost Estimates Extrapolated from Stantec's Detention Pond Cost Estimate (Inflation was accounted for in maintenance costs)

Item	Stantec Construction Estimate (Retention Pond was estimated from detention pond)	Pre-Construction Costs (20% of construction costs)*	Maintenance Costs in 2016 Dollars (Time Value of Money=2.5%)	Total Construction Costs	Construction with 20% Contingency Budget Added	Total Cost over 30-Years
Detention Pond (Retrofit)	$120,000.00	$24,000	$501,000	$144,000.00	$173,000	**$645,000**
Retention Pond (Retrofit)	$255,000	$51,000	$590,000	$306,000	$368,000	**$896,000**
*(King and Hagan, 2011)						

Table 13: Costs Extrapolated from Urban Watershed Retrofit Practices (Schueler et al, 2007)

Item	Base Construction Cost per ft3 (Corrected from 2007 dollars to 2016 dollars)	Volume	Pre-Construction Costs (20% of construction costs)*	Maintenance Costs in 2016 Dollars (Time Value of Money = 2.5%)	Base Construction Cost	Construction with 20% Contingency Budget Added	Total Cost over 30-Years
Detention Pond (Retrofit)	$3.45	81,750	$56,000	$501,000	$282,000	$395,000	**$896,000**
Retention Pond (Retrofit)	$3.45	172,277	$119,000	$590,000	$594,000	$832,000	**$1,400,000**
*(King and Hagan, 2011)							

9.2 Hydrodynamic Separator

The unit costs for the different types of hydrodynamic separators are listed in *Table 7* in *Section 7.4*. Based on recommendations from Contech, the Vortechs unit was chosen as the hydrodynamic separator to be used in this project. The unit costs $25,750. The maintenance and installation costs can be seen below in Table *14* with adjustments for inflation included from the Bureau of Labor Statistics Inflation Calculator. The total installation costs, including the cost for the unit will be about $67,000. The values in the table below (excluding the unit cost and the maintenance cost) are from a 2007 Farrell St. Stormwater Project in South Burlington. To compensate for their having used a smaller Vortechs model and different site parameters, a 40% contingency was added to the cost. The Vortechs will need to be vacuumed every year which will cost around $1,300 per event (Jones et al, 2002).

Table 14: Maintenance and Installation Requirements for Vortechs Unit*

Item	**Units**	**Amount**	**Cost**	**Item Amount**
Unclassified Excavation	CY	100	$15	$1,500
Earth Borrow	CY	70	$15	$1,050
Unsuitable Soil Excavation	CY	5	$20	$100
Vortechs Model 7000	LS			$25,750
Connection of 10" Conc. Pipe	LS			$500
Connection of 6" PE Pipe	LS			$500
6" PE Pipe	LF	30	$30	$900
12" CPEP	LF	150	$35	$5,250
Catch Basin	Each	2	$2,000	$4,000
Relocating Existing Utilities	LS	N/A		$0
Crushed Stone	CY	4	$25	$100
New Pavement	Ton	12	$75	$900
Stone Fill. Type IV	CY	8	$60	$480
Geotextile Under Stone Fill	SY	15	$6	$90
Top Soil	CY	12	$45	$540
Maintenance and Protection of Traffic	LS			$1,500
Clearing and Grubbing	LS			$200
Erosion and Sediment Control	LS			$1,500
Landscaping	LS			$200
Maintenance (Vacuuming)**	Event		$1,300	$1,300
Total Installation				**$45,060**
Say				**$45,000**
Correction for Inflation* **				**$48,000**
40% Contingency				**$67,000**

*Estimated using the City of South Burlington's Farrell St Stormwater Project (Project was for a smaller model therefore contingency is high to compensate)
** Not included in installation costs
***(2007) Inflation corrected using CPI Inflation Calculator, Vortechs Unit was already in 2016 dollars so was not included in this correction

9.3 Gravel Wetland

The maintenance and operations costs for the gravel wetland are shown in Table *15* below. The total construction costs including labor and engineering were calculated using a general cost estimate of $8.31 per cubic foot and the wetland volume of about 142,000 cubic feet (EPA, 2009). The total values for maintenance, construction, and the overall cost can be seen in Table *16* below.

Table 15: Maintenance Costs (2009 dollars) for Gravel Wetland

Sources	**Assumptions**	**Item**	**Unit Price**	**Units**	**Maintenance Schedule (Years)**	**Amount Needed**	**Total Cost**
EPA 2009	when 60% volume is lost	Removing Sediment from Forebay	$60	CY	5	13.33CY	$800
EPA 2009	When 50% volume is lost	Removing Sediment from Main Cells	$7,600	Event	20	-	$7,600
EPA 2009		Clearing Dead Plants	$7,900	Acre	1	0.013 Acres	$102
EPA 2009		Removal of Trash	$350	Event	1	-	$350

Table 16: Total Costs for Gravel Wetland

Total Costs (Present Worth)	**Costs***
Total Maintenance Costs (EPA, 2009)	$257,000
Total Construction Costs (EPA, 2009)	$141,000
Overall Total Costs	**$398,000**
* Maintenance costs were adjusted from 2009 dollars to 2016 dollars	

9.4 Slope Stabilization

The cost displayed in Table *17* represents average cost for each stabilization technique based on the entire length of stream west of the railroad tracks, which is around 1,820 feet. The next cost represents only 820 feet of stream stabilization applications, which is approximately the length of the bends in the stream in the field before the start of the forested area that are more susceptible to effects of erosion (Figure 26). These values will be doubled to account for both stream banks. Evidence of recent soil disturbance and root exposure makes it clear erosion is a problem specifically in these areas. Using the costs below we would recommend using vegetation to stabilize the banks as it is fairly inexpensive and meets all three protection parameters listed in the table. The bends should be the focus as they are where the majority of the damage is occurring, but if time and money are available the full length of the bank in the area highlighted in Figure 26 could also be stabilized to further decrease sediment entering the brook.

Table 17: Cost Breakdown for Slope Stabilization

Stabilization Techniques	**Toe Protection**	**Upper Bank Protection**	**Runoff Control**	**Equipment**	**Unit Price ($/LF) (2016 dollars)**	**Total Price for Full Length (3640ft)**	**Total Price for Bends (1640ft)**
Vegetated Geogrids (Leech, 1997)	X	X	X	Hand tools	$148	$539,000	$243,000
Brush Mattress (Leech, 1997)		X	X	Hand tools	$59	$215,000	$96,760
Tree Revetments (Cedar, 2012)	X			Hand tools or light power machinery	$12	$44,000	$20,000
Coconut Fiber Rolls (Murphy, 1996)	X			Hand tools	$70	$255,000	$115,000
Vegetation (Murphy, 1996)	X	X	X	Hand tools or light power machinery	$15	$55,000	$25,000
Live Stakes (Murphy, 1996)		X		Hand tools, extensive labor required	$23	$84,000	$38,000
Branch Packing (F.X. Browne, 2003)		X	X	Hand tools	$26	$95,000	$43,000
Live Fascines (Leech, 1997)		X	X	Hand tools	$7	$25,000	$11,000

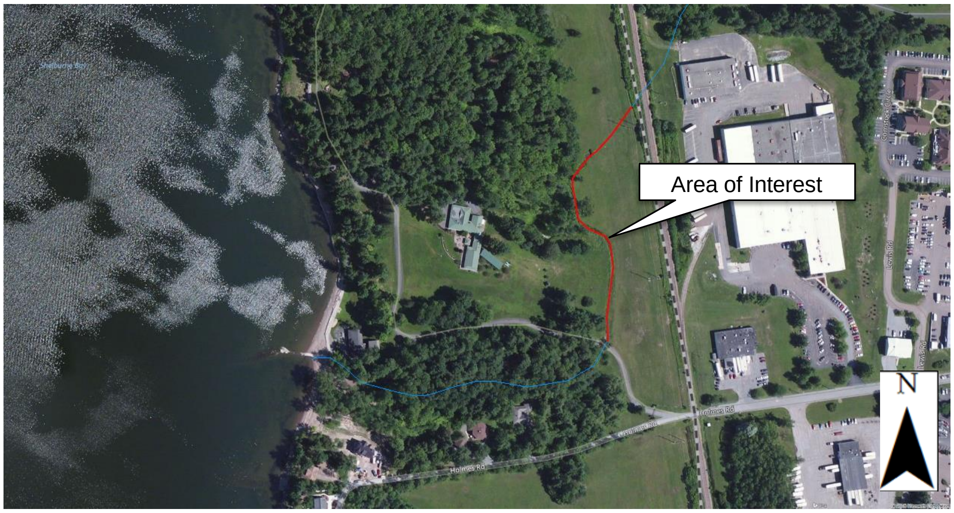


Figure 26: Area of potentially higher levels of erosion along Farrell Brook.

9.5 No Action Alternative

While the no action alternative does not have any construction or design costs, there are many external costs associated with ignoring the issues associated with Farrell Brook. As discussed in *Section 6.4*, there are many externality costs associated with not improving the quality of the water entering Lake Champlain (OFA, 2015). While these cannot be easily represented using traditional cost estimating techniques, the reader should be aware of the impacts of failing to treat the water quality in any stream entering the lake. If the flows in the brook were allowed to increase and no water quality treatment was provided, there would be a significant increase in stream bank erosion. Increased erosion will then lead to an increase in sediment transportation and an increase in TSS, phosphorus and nitrogen entering Shelburne Bay. Lake Champlain is already experiencing the negative effects of excess nutrients and, if it is at all possible, further contamination of the lake should be avoided. Increased erosion can also lead to property damage and the costs due to loss of recreation use. Businesses along Lake Champlain receive much of their income because of the attractiveness of the lake and the ability of their customers to recreate in the area. If that income was to lessen due to the lake becoming unusable for recreation, many businesses would be negatively impacted.

10.0 CONCLUSIONS AND RECOMMENDATIONS

Upon considering all design alternatives and their associated costs, it is recommended that several steps should be followed in order to retrofit this watershed. The detention pond is the key component in that it is required to decrease peak flows. If the discharge from the Orchards neighborhood is increased this pond will return it to the current flow rate, allowing for downstream water quality improvements to be made, such as implementing the retention pond at Inn Road. If the flows from the neighborhood are not increased, then the detention pond could be used to bring flows to a more natural flow rate to ensure the stability of the stream banks are not further compromised.

As seen below in Table *18*, the phosphorus load from the watershed with no impervious areas (natural state) is only about 30 lbs., much lower then what is currently being experienced. *Table 18*, also outlines the removal efficiency of each stormwater best management practice as they stand alone. These percentages were determined based off knowledge of the efficiencies stated by the stormwater manual, the study presented in *Table 1*, and discussion with our community partner Jim Pease, VTDEC. Lowering the flows is a crucial first step in an effort to return the phosphorus load to a more natural state (what it was before urban development changed the landscape). Once the flows have been lowered by implementing the detention pond, the retention pond and stream stabilization can be considered as options to lower the phosphorus load further in the watershed and, by extension, Shelburne Bay. The retention pond and stream stabilization may not stand alone without the implementation of the detention pond to decrease overbank flood flows. The gravel wetland and swirl separator lie upstream of the detention pond location and may be implemented on their own although they are not expected to have as significant a contribution to the overall health of Farrell Brook. That said, they would definitely be a first step towards enhancing water quality.

Table 18: Comparison of calculated phosphorus loadings from the Farrell Brook watershed above each designed best management practice.

Best Management Practice	Watershed Above BMP (Acres)	Impervious Area (Acres)	Percent Impervious	Phosphorus Loading (lbs.)*	BMP Removal Rate	BMP Phosphorus Reduction (lbs.)
Gravel Wetland	21	10.3	51	38.07	24%**	9.1
Swirl Separator	59.64	22.88	38	104.45	15%	15.7
Detention Pond	80.64	32.18	40	139	20%	27.8
Retention Pond	108.25	41.13	38	140	52%	72.8
Entire Natural Watershed* **	176.41	0	0	30		

*Phosphorus Loading Calculations were based off the "Simple Method Pollutant Loading Calculation Worksheet-Phosphorus" from the VT DEC website (Appendix H). These calculations are based off the VT annual precipitation at the Burlington International Airport of 36.82 in (NOAA) and an assumed average annual phosphorus concentration for developed lands of .44 mg/l.

**Typical efficiency would be 48% removal. The proposed gravel wetland only treats half the water quality volume and therefore the efficiency of this BMP would decrease to 24%.

***For the natural watershed, land cover type was assumed to be a forest/meadow combination.
Ideally, the stormwater infrastructure would be able to treat the expected loads to restore the loading back to its natural state. Using the removal efficiencies from Table 13 above, we can estimate the amount of phosphorus removed by each BMP as an effort to work towards the goal of a "natural" watershed with no development.

The best option would be a combination of the proposed designs. Due to the very high flows coming from the Orchard's neighborhood and the limited space, one alternative alone will not fix the problem. Together, the proposed alternatives are designed to remove 79% of phosphorus in Farrell Brook. The flow of removal through each alternative is outlined in *Table 19* below. However, the dry detention pond is of most importance as it provides the greatest effort at decreasing flows, the priority of this project. Ideally, the detention and retention pond could be implemented to meet both goals of decreasing flows and treating water quality. The swirl separator, and or gravel wetland provide further enhancement to water quality but alone do not provide a significant fix. Another consideration is the auxiliary benefits from each alternative. For instance, the gravel wetland and retention pond provide wildlife habitat, urban green space and a water quality education demonstration for the public.

Table 19: Calculation for phosphorus removal with all alternatives combined

Best Management Practice	Phosphorus Loading (lbs.)*	BMP Removal Rate	BMP Phosphorus Reduction (lbs.)	Treatment Train Net Load (lbs.)
Gravel Wetland	38.07	24%**	9.1	29
Swirl Separator	104.45	15%	15.7	88.8
Detention Pond	29 + 88.8 = 117.8	20%	23.6	94.2
Retention Pond	94.2	52%	49	45.2
Entire Watershed	**197.18**	**79%**	**155.8**	**41.4**
Entire Natural Watershed* **	**29.99**			

Like most projects, the real limiting factor is economic feasibility. However, stormwater plans will eventually be required by the state for any preexisting development over three acres. There is also the possibility that Stantec's report is not implemented, and pipes are not upsized in the Orchards neighborhood for a number of years.

Due to a lack of time and resources, and our level of knowledge, this report's recommendations are not ready for construction and all designs would need to be fully checked and redesigned by a Professional Engineer. However, this report provides a good basis for stormwater retrofit options. We hope that this project serves as a foundation for the type of restoration that Farrell Brook may one day receive. This project will serve to enhance the water quality of the highly valued Lake Champlain.

ACKNOWLEDGMENTS

We would like to acknowledge our community partner Jim Pease and the time he put into assisting our team and ensuring we stayed on the correct path. We greatly appreciate his guidance throughout the project and the many resources with which he provided. We would also like to acknowledge the help provided by Stantec Consulting in supporting our project and directing us in the correct direction. We are extremely thankful for the generosity of the members of the University of Vermont and surrounding community. We would also like to thank our course advisor, John Lens, for his assistance to our project throughout the semester.

REFERENCES

1) Booth, Derek B. *Urban Stream Rehabilitation in the Pacific Northwest: Physical, Biological, and Social Considerations.* Rep. Seattle: U of Washington, 2001. pdf.
2) CNT. (2006). "National Stormwater Management Calculator." Green Values. CNT, 2006. Web. 18 Apr. 2016. http://greenvalues.cnt.org/national/cost_detail.php
3) Contech. (2014) "CDS Guide Operation, Design, Performance and Maintenance. "Contech Engineered Solution. Contech, 2014. Web. 17 Apr. 2016.
4) Dunlap, F., Mimeault, M., and Smeltzer, E. (2015). *2015 State of the Lake Report, Phosphorus.* Lake Champlain Basin Program. Vermont and New York.
5) EPA. (1999). "Storm Water Technology Fact Sheet Hydrodynamic Separators". United States Environmental Protection Agency. Sept. 1999. Web. 14 Apr. 2016.
6) EPA. (2009) "Stormwater Wet Pond and Wetland Management Guidebook." Stormwater Wet Pond and Wetland Management Guidebook, February 2009. United States Environmental Protection Agency. EPA, Feb. 2009. Web.
7) "Frequently Asked Questions: Act 250." Natural Resources Board: Frequently Asked Questions. Natural Resources Board. Web. 22 Mar. 2016.
8) Galford, Gillian, Dr. "Impacts of Climate Change in Vermont." *Vermont Climate Assessment*. University of Vermont, 2016. Web. 11 Feb. 2016.
9) Gendron, C., and Goyette, G. (2015). *South Burlington Multi-Site Stormwater Infrastructure Assessment Site No. 3 - Orchards Neighborhood and Streets* (Final Report). Stantec Consulting Ltd. South Burlington, VT.
10) Hall, Gregg. (2009). "UNHSC Subsurface Gravel Wetland Design Specifications." University of New Hampshire. University of New Hampshire Stormwater Center (UNHSC), 2009. Web. 17 Apr. 2016.
11) Jones, Jillian E.; Bisbee, G. Dana; and Stewart, Harry T. (2002), "Innovative Stormwater Treatment Technologies: Best Management Practices Manual" (2002). PREP Publications.
12) King, Dennis, and Patrick Hagan. (2011). "Costs of Stormwater Management Practices In Maryland Counties." Ref. No. [UMCES] CBL 11-043 (2011). Metropolitan

Washington Council of Governments. University of Maryland Center for Environmental Science (UMCES), 10 Oct. 2011. Web. 20 Apr. 2016.

13) OFA (2015). "Sustainable Economic Development in the Lake Champlain Basin. "Lake Champlain Opportunities for Action Database. Lake Champlain Basin Program, 2015. Web. 17 Apr. 2016.

14) U.S. Environmental Protection Agency Region 1, New England Boston, MA. (2015). "Phosphorus TMDLs for Vermont Segments of Lake Champlain." *Phosphorus TMDLs for Vermont Segments of Lake Champlain, United States Environmental Protection Agency*. 14 Aug. 2015. Web. 12 Apr. 2016.

15) Vermont Agency of Natural Resources. (2002). *The Vermont Stormwater Management Manual,* 5th ed., Vermont.

16) Vermont Clean Water Initiative (Act 64). *Clean Water Vermont - DEC Watershed Management Division*. State of Vermont, 2015. Web. 11 Feb. 2016.

17) Vermont.Agency of Natural Resources. Watershed Management Division. VT DEC Stormwater Program Fee Summary: Effective 7/1/2015. Web.

18) Vermont.Agency of Natural Resources. Watershed Management Division. Wetland Permit Application Fees Starting July 1, 2015. Web.

19) "Vermont Stormwater Section – Stormwater Advisory Group (SWAG). "Vermont Stormwater Section – Stormwater Advisory Group (SWAG). Vermont Watershed Management Division, Web. 22 Mar. 2016

20) NVRC. (2007). Northern Virginia Regional Commission. "Maintaining Stormwater Systems." NVRC. NVRC, 2007. Web. 3 May 2016.

21) Schueler, Thomas R., Jennifer Zielinski, David Hirschman, and Michael Novotney. (2007). Urban Stormwater Retrofit Practices. Ellicott City, MD: Center for Watershed Protection, 2007. Staunton Virginia. Center for Watershed Protection, Aug. 2007. Web. 3 May 2016.

22) Leech, James. Allen, Hollis. (1997). "Bioengineering for Streambank Erosion Control". *US Army Corps of Engineers*. 1997. Web. 16 Apr. 2016.

23) "Trees Jetties/ Root Wad/ Tree Revetment". (2012). *Cedar River Stabilization Project*. Web. 16 Apr. 2016.

24) Lake Wallenpaupack Homeowner Streambank and Shoreline Stabilization Handbook. (2003). *F.X. Browne, Inc*. Web. 16 Apr. 2016.
25) Murphy, Michael. Dreher, Dennis. “Shoreline Stabilization: Bioengineering Alternatives”. (1996). *Illinois Environmental Protection Agency*. Web. 16 Apr. 2016.
26) “Using Stabilization Techniques to Control Erosion and Protect Property”. *Tennessee Valley Authority*. Web. 16 Apr. 2016.