COLLEGE OF ENGINEERING

design document for

Harvesting Stormwater Runoff for Urban Farm Irrigation

submitted to:
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Team No. 11: GHV Engineers: Stormwater Management Consultants

Title: Harvesting Stormwater for Urban Farm Irrigation

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ABSTRACT

Rainwater control and reuse contributes to a healthier environment, especially in urban regions. A rainwater harvesting system was designed for an urban farm located in the Germantown section of Philadelphia, Pennsylvania. This design includes a transport mechanism to convey captured rainwater from the roof, approximately 2350ft^2, to a constructed wetland system, where it will be treated. The water, treated to EPA non-potable reuse standards, will then be stored or transported by pumping to the adjacent half-acre farm to irrigate the crops. When the system fills, overflow is controlled and directed to the combined sewer system. Research has shown elevated heavy metals concentrations in runoff coming from aging roof structures; these concentrations can be reduced substantially through treatment in a constructed wetland. Onsite treatment of rainwater reduces the farmers' dependency on municipal water resources and usage costs. Additional benefits of this system are reducing the hydraulic load and improving water quality of runoff from the property into Philadelphia's combined sewer system.

EXECUTIVE SUMMARY

In densely populated cities like Philadelphia where much of the area is covered by impervious surfaces (e. g. roads, rooftops, and parking lots), the natural hydrologic cycle is disrupted. Instead of infiltrating, evaporating, or transpiring, runoff is directed into the city's sewer system. Philadelphia has a combined sewer system, used for stormwater discharge as well as municipal waste.

Stormwater runoff can pick up contaminants present on contact surfaces and wash them into the sewer system. Combined sewer systems are known to overflow even in moderately sized storms. These overflows can release untreated stormwater and municipal waste into the nearby streams and rivers.

Our client seeks to reduce his property's environmental impact by collecting and treating the stormwater runoff from the roof of the residence, as well as prevent basement flooding that may cause structural damage. The water would be treated for reuse on the adjacent urban farm. Additionally, a section for excess treated runoff storage will be designed as a wading pool for resident use. Thus, this design addresses issues of stormwater quality and quantity management while supporting sustainable food growth practices.

The system will completely remove the first inch, approximately 1500 gallons of runoff, from entering the sewer. Additionally, it will effectively manage the two year, 24-hour storm, an anticipated 5000 gallons. While the system must be large enough to retain or responsibly redirect rooftop runoff, it shall not exceed 200 ft^2 of the backyard surface area. Infiltration rate in the treatment area will be less than 1.42in/hr. Irrigating water must meet EPA Recommended Limits for Lead and Zinc of less than 5.0 mg/L and and 2.0 mg/L, respectively. In order to prevent increased mosquito and pest populations on site, water will not be left to stagnate for longer than 48 hours. The wetland system will have varying depths, a serpentine flow pattern, and diverse native plant species to attract diverse populations of wildlife. Water level variance in the pond and wetland will not exceed +/- 6 inches. Excess runoff will be mitigated through controlled overflow. For safety, side slopes will not exceed a 3:1 slope, and benches or fences will be included where depths exceed 3 ft. The total cost of construction will not exceed $30,000, and the system will be designed to allow phased construction to fragment construction costs.

To accommodate the volume constraints, the system design includes a 125 ft^3 sedimentation basin, 415 ft^3 constructed wetland, 180 ft^3 natural pool, and a 227ft^3 underground storage cistern. Varying depths accounted for, the system can retain excess runoff up to 467 ft^3. This value exceeds the amount of inflow expected from the first two inches of rain, meeting Pennsylvania BMP recommendations. ensure that no pockets of standing water will be created in the system. A diverse selection of native plants was chosen based on their treatment, aesthetic, and ecological qualities. However, the final plant selection may be adjusted upon results of water testing which will define the pollutant levels in the water. Literature values and modeling using EPA Stormwater Management Model (SWMM) indicate that treated effluent is expected to contain 0.004-0.79 mg/L of Zinc and 0.007-0.1 mg/L of Lead. Hydrologic modeling using SWMM also shows that evaporation will not significantly affect plant life in the wetland and pond.

There is little discussion on constructed wetland systems that treat runoff from less than a 5 acre area. Furthermore, few texts describe systems that both treat and reuse runoff for agriculture and recreation, especially in an urban setting. However, the need to capture and treat stormwater on site, reduce municipal water usage, mitigate runoff impact, and encourage sustainable growth of food crops are growing concerns in Philadelphia and beyond. The success of our system will create a model for research and education of urban stormwater harvesting which appeals to multiple markets in many cities. Residents who wish to add an amenity to their property and reduce their environmental footprint, community leaders who want to beautify their neighborhood and utilize vacant areas to promote local foods, healthy eating and sustainable practices, and municipalities like Philadelphia seeking relief for its

aging infrastructure, water treatment facilities, and the “broken window” effect that unkempt vacant land has on its neighborhoods, can all benefit from the model of this rainwater harvesting system. Our client will benefit in many of these ways as he minimizes his environmental footprint, adds an amenity to his property, and reduces his water usage costs.

Contents

1. PROBLEM

Recently, GHV Engineers: Stormwater Management Consultants were approached a property owner to investigate stormwater runoff solutions for his home and adjacent urban farm. The property is located in the Germantown section of Philadelphia, Pennsylvania. Currently, the runoff from his residence is not properly directed away from the house into the Philadelphia combined sewer system. Stormwater runs uncontrolled from the roof and causes his basement and driveway to flood. The client requested GHV Engineers design a system that captures rainwater from the roof for reuse. The system is designed to redirect and treat this runoff naturally and store it for irrigating crops on the adjacent urban farm. This system should significantly reduce or eliminate flooding on site and reduce the client's dependence on the public water supply to irrigate the farm. This proposal aims to achieve these goals in a manner that lessens this property's hydraulic and pollutant load on the combined sewer system.

After World War II and the development of the Rational Method (a peak flow estimating method), several measures referred to as "modern urban drainage," were engineered and implemented (National Resource Council, 2008). One of these measures was to design a system of catch basins and pipes that would direct stormwater away from properties and to receiving waters as fast as possible. Unfortunately, this quick redirection has led to stream-bank erosion and sedimentation issues in some areas. In older urban centers like Philadelphia, surface stormwater runoff is directed into a combined sewer system. Combined sewers are designed to convey both stormwater runoff and wastewater discharge to a treatment facility. These combined sewers are prone to overflowing even in the most frequent small storms. This overflow releases untreated residential discharge and stormwater runoff into the nearest open body of water. Furthermore, the lack of capacity of older sewer systems to withstand the volume can occasionally cause back up in and on residential properties (National Resource Council, 2008).

In recent years, concern has grown for pollution caused by stormwater runoff. Roof runoff specifically has been found to contain heavy metals including lead, zinc, and copper (Lye, 2009). As the roof weathers, heavy metal particles break free from roofing materials and are carried off the roof by rainwater, mostly during the "first flush", or first fifteen minutes of rainfall (Forster, 1996). Though not specifically targeted for this design, carcinogenic hydrocarbons deposited primarily by automobile exhaust may be flushed from roofs during rain events in some cases(Van Metre, 2003). These types of pollutants are costly to remove in treatment plants. In the case of a combined sewer overflow (CSO), these pollutants reach an open body of water before reaching a proper treatment facility. The system should be designed to treat for these pollutants to standards specified by the United States Environmental Protection Agency (US EPA).

The system designed to capture and reuse this runoff must take into account these pollutants and treat the water to the standards specified in the USEPA's guidelines for agricultural water reuse. Stormwater management has become increasingly more sophisticated and decentralized. For example, low-impact development (LID) has been gaining popularity as it controls as much stormwater onsite as possible. LID is especially beneficial in rapidly developing suburban areas and can be used in redeveloping urban properties (National Resource Council, 2001). These solutions are man-made structures engineered to imitate natural systems by infiltrating, capturing, and retaining stormwater runoff. Retention is mainly achieved through the use of constructed wetlands and wet ponds, while infiltration occurs in rain gardens and trenches (PA DEP, 2006). Many of these methods are known to reduce the levels of heavy metals, phosphorus, nitrates, and hydrocarbons that are washed away with stormwater runoff. Pollutant removal rates of these systems are often high enough to allow discharge into fresh water without harm (Shutes et. al., 1997). These contaminants can all be removed by a waste water treatment facility. However, in the event of a CSO, many contaminants may be released into and pollute surface waters. The discussed runoff treatment options reduce both the actual amount of stormwater reaching the combined sewer and the amount of pollutants in that runoff. Our design also serves a third practical purpose: storing the treated water to irrigate the urban farm and reducing their demand on public water supplies.

The City of Philadelphia has created a plan to update its aging sewer systems, called the “Clean Water, Green City Program (CWGC)”. The program proposes to address combined sewer overflows (CSOs) with two steps. The first is a list of nine “tune-up” criteria that do not require extensive engineering or construction. These will quickly help reduce the hydraulic load on the systems and bodies of water that are prone to receiving overflow. The second requires extensive engineering and reconstruction of existing system components, and will be both time consuming and expensive. The CWGC program divides itself into three component programs that include “land-based” stormwater management techniques to be used to retain water on the property initially contacted by rain through evaporation, transpiration, infiltration and controlled release (Philadelphia Water Department, 2010). These point source solutions are very relevant to the project GHV has been selected to investigate.

Philadelphia is 44 percent impervious (City of Philadelphia, Greenworks, Philadelphia, 2009). Furthermore, residential land use accounts for approximately 30 percent of total land showing that a majority of this impervious area is due to housing footprints (Delaware Valley Regional Planning Comission, 2010). Some of Philadelphia’s watersheds have been manipulated to protect health and safety of humans from sewage dating back to the 1800’s (Levine, 2005). One of these is the watershed that our project is based in, the Tacony-Frankford watershed.

The Pennsylvania Stormwater Best Management Practices (BMPs) Manual Control Guideline 1 recommends that new construction must not increase total runoff volume for all storms less than or equal to the two-year, 24-hour storm event (PA DEP, 2006). If an existing structure’s collection and redirection mechanisms are to be changed, it should be done according to these guidelines. GHV proposes to completely remove at least the first inch of runoff from flow into the sewer, in accordance with Pennsylvania BMP’s Control Guideline 2, and effectively manage runoff from the two-year, 24-hour event (3.22 inches) (PA DEP, 2006). The roof will produce approximately 1460 gallons of rainwater per inch of rain.

Traditionally, capturing rooftop runoff for irrigation purposes is done using only a catch basin (PA DEP, 2006). However, this method ignores the potential for soil contamination due to lead and other heavy metals that may be present in rooftop runoff. To ensure toxin-free water and safe food crops, GHV Engineers suggest a constructed wetland to naturally remove potential contaminants from the runoff before utilizing it. This project is beneficial in utilizing constructed wetlands on a small scale because generally speaking a constructed wetland is used to treat runoff from a 5+ acre area (PA DEP). In this case, the collection area is only the size of the roof, 2350 ft^2. There is a fair amount of information regarding constructed wetland use on a larger scale in runoff treatment. In a 2010 study, levels of copper, lead, and zinc were reduced by a full scale constructed wetland used to treat domestic wastewater (Arroyo, et. al., 2010). Another study done in Brazil showed that heavy metal levels in runoff from an airport runway can be significantly reduced in a full-scale constructed wetland (Calijuri, et. al., 2010). Even refinery wastewater can be treated efficiently for metals removal (Aslam, et. al. 2010).

Furthermore, the property contains a functioning urban farm and can become a benchmark for how single residences may reclaim their rainwater and utilize it. GHV is excited to be on the front lines of engineering sophisticated natural systems for stormwater reuse. Urbanized areas such as Philadelphia cannot easily retrofit existing combined sewer systems. The proposed onsite LID stormwater harvesting system and others like it would be beneficial in reducing the demand on Philadelphia’s aging sewer system.

Locally, Perkiomen Valley High School has constructed one example of system similar to GHV’s design.

Financially, irrigating an urban farm with roof runoff, as opposed to city water, will potentially save thousands of dollars over the lifetime of the system. Mr. Hennesy, using a gauge to measure consumption, estimates water usage to be about 30,000 gallons for the farm over the course of one growing season. GHV aims to be able to drop this number substantially. In addition, the Philadelphia Water Department (PWD) has instituted a plan to charge property owners for their water runoff in order to pay for maintenance of the combined sewer system. Property owners will be charged one rate for total area, and another for impervious area of their properties. Savings from reducing impervious area will likely be a small savings compared to the amount saved by conserving 10,000 gallons of water annually. It does however, add another small financial incentive.

2. DESIGN REQUIREMENTS

Prior to the design of this system, many things needed to be considered that would affect its functionality. The following discussion describes the functional and non-functional design constraints of the system. Functional design constraints are those that, as the name suggests, influence the functional integrity of the system. These are influenced by physical laws, government regulations, best management practice recommendations, and pertinent data. Non-functional constraints do not necessarily affect the way that the system will perform, but reflect important social, environmental, and economic concerns which are indirect gauges of the system's success. These constraints are the backbone of our design and influenced many decisions in our design process, as described in further detail below.

2.1. Functional Design Constraints

As stated, the purpose of this design is harvesting stormwater for irrigation. Thus, the functionality of the design is constrained mostly by the quantity and quality requirements that govern water collection and reuse.

The first quantity control measure was obtained from Pennsylvania's Stormwater Best Management Practices Manual (PA BMP) recommendations. The PA BMP Control Guideline 2 recommends that at least the first inch of runoff from newly constructed impervious surfaces be "permanently removed from the runoff flow, i.e. it shall not be released into the surface Waters of this Commonwealth (PA DEP, 2006)." Although most of our drainage area does not include newly constructed impervious areas, we decided to use this one-inch guideline as a benchmark for our design. According to Luna Leopold's <u>A View of the River</u>, "An inch of runoff means the volume of water covering the drainage basin in question to a depth of 1 inch." With a roof area of 2350 ft^2 and the treatment (including pond) area of 200 ft^2, the first inch of rain results in approximately 1500 gallons to be completely removed through reuse on the farm, infiltration, or evapotranspiration from wetland plants. In order to further ensure responsible stormwater management, we raised the goal for our system to accommodate a the two-year, 24-hour storm. According to data obtained from the National Oceanic and Atmospheric Administration (NOAA), the two year, 24-hour storm in the Philadelphia region yields an estimated 3.22 inches of rainfall (NOAA, 2004). This storm is increasingly being used as a benchmark for stormwater design as a response to the ineffective stormwater management designs of the past. A large majority (over 95 percent) of average annual rainfall occurs in storms under this depth. In a large urban setting like Philadelphia, storms between one and 3.22 inches instantly become runoff that has the potential to damage streams and rivers. During the two-year, 24-hour storm, the roof footprint of the Germantown house produces approximately 4720 gallons of runoff. Adding the prospective area of the wetland, we must design to retain about 5200 gallons of rainwater in the system, and responsibly control overflow.

In addition to ensuring runoff mitigation, the system design requires an appropriate height in the wetland and pond in order to sustain plant life and a comfortable pond depth for residents to wade in. One trait that will help to maintain optimum water depths in the system is a low infiltration rate of water through the soil. Thus, appropriate measures must be taken to reduce infiltration rates, such as installing a synthetic or clay liner to ensure an infiltration rate of less than the suggested 10 micrometers per second (1.42 inches per hour).

Next, proper conveyance of water from treatment to storage tanks was investigated. Separating the treated water helps prevent mixing with untreated water. It also ensures that optimal levels of water can be retained and treated in the wetland system. Therefore, we had to consider the feasibility of transporting water from the system, which in turn influenced various aspects of our design. The location of the treatment area in the backyard, the depth of wetland and pond, and the depth of storage tanks must be considered when selecting the pump(s) used. In other words, the difference in elevation between the storage and farm must be such that that a pump of reasonable flow rate, price, and energy usage can effectively transport water. The storage tank must be 1700 gallons to ensure that the first inch, once treated, can be removed from the original site and stored for reuse. So we must consider that the pump is to transport water from the very bottom of the wetland to the very top of this large storage tank. This volume selection will also dictate the area of the tank itself along with that of the platform that must be constructed to elevate it; the drip irrigation system requires 15-17 PSI in order to properly irrigate throughout the farm, so the pressure due to gravity must equal this.

Finally, we must ensure that treated water is of an acceptable quality for agricultural and recreational reuse. The EPA Recommended Limits for Constituents of Reclaimed Water for Irrigation require that irrigation water contain less than 5.0 mg/L of lead and 2.0 mg/L of zinc (U.S. EPA, 2004), which are the two metals most commonly found on residential asphalt roofs (Schueler, 2000). These limits will dictate the planting plan, necessary detention times in the treatment area, and feasibility of the wading pool component.

Name	Description
Pollutant Reduction	Treated water must meet EPA Recommended Limits for Constituents of Reclaimed Water for Irrigation of less than 5.0 mg/L of lead and 2.0 mg/L of zinc.
Water Management	At least the first inch-approximately 1500 gallons-of each rain event intercepted by the roof must be completely removed from the runoff flow. Further, the system should accommodate the two-year, 24-hour storm of nearly 5200 gallons.
Soil Properties	Infiltration rate in the treatment area must be less than 10 micrometers per second, or 1.42 inches per hour.
Water Level	The wetland and pond will maintain a height of at least 4 feet and mitigate overflow occurrences.
Water Harvesting	Treated water must be delivered at 15-17 psi to the drip-irrigation tubes.

Table 1: Functional Design Constraints

2.2. Non-Functional Design Constraints

The design of our system is further constrained by social and environmental concerns that do not necessarily affect the functionality of the system, but must be considered in order to ensure a system that is safe, socially responsible, environmentally and technically sustainable, and can feasibly be manufactured.

Manufacturability is a major concern as this project is to be implemented on a residential property of finite area and for a client with limited funding. Thus, we are constrained to a treatment area of under 250 ft^2. This limitation will dictate the depth of treatment area and volume of storage tanks necessary to retain the desired amounts of runoff. Furthermore, manufacturability is limited by the amount of funding available for the project. The client's expects to spend under $30,000 on the overall costs of the project. Therefore, the sum of the costs of all the components and installation must fall below this amount. The cost limit set by the client is made with the assumption that he will be able to obtain grants and special programs to help supplement personal funds used. Thus, he has requested that we design the system to be feasibly constructed in phases, so that components can be built as funding becomes available. Also, we wish to design the system to be much less than this total cost, so that the client can cover costs in the

Type	Name	Description
Economic	Cost	The total cost of construction should not exceed $30,000.
Manufacturability	Phased Construction	The system should be designed to promote a phased construction process.
Manufacturability	Size	The treatment area must be large enough to retain or responsibly redirect rooftop runoff, but not exceed a total surface area of 250 ft^2.
Sustainability	Maintenance	The design must include an indicator of the halfway-full point for sediment in the forebay that must be dredged; this will be approximately every 5 to 10 years. Vegetation must be removed and replaced as it dies.
Environmental	Biodiversity and Wildlife	The treatment area must contain diverse native plant species which attract wildlife and also efficiently remove expected pollutants.
Health and Safety	Safety	Side slopes must not exceed 3:1, and benches must be included for depths exceeding 3 ft.
Health and Safety	Pest and Vector Control	The pond will manage pest and vector populations naturally with plants and aquatic life.
Social	Usability	The wading pool, requested for resident use, will be separate from the sedimentation basin.

Table 2: Non-Functional Design Constraints

event that funding is not awarded. After initial construction, maintenance costs related to dredging of the forebay and plant upkeep and replacement must be considered. The forebay must be dredged once it fills halfway with sediment, so an indicator must be part of the design. Also, plants that are dead or no longer functioning properly must be removed and replaced to prevent reintroduction of nutrients and absorbed heavy metals into the system. Plant selection and forebay size will be influenced by this constraint as we attempt to design a system that requires minimal maintenance frequency, contracted labor, and plant costs.

Plants should be selected based on their removal rates of the various pollutants that are expected to be present in roof runoff. Specifically, plants should have a high removal rate for the heavy metals found in asphalt shingle runoff i.e. copper, zinc, and lead (Lye, 2009)(Van Metre and Mahler, 2003). In addition to pollutant removal efficiency and cost, plant selection will also be dictated by environmental sustainability. Pennsylvania BMPs and most modern-day landscape architects and horticulturists insist that wetlands be equipped with plants that are native to the region (PA DEP, 2006). Choosing a diverse population of native plant species allows for the best replication of a natural wetland, which is designed by nature to attract and provide habitat to native wild life and naturally manage pest and vector populations. Native plant selection also helps protect the environment from potentially invasive non-native species (Tallamy, 2009).

The ability of the system to resist the growth of pest populations is an important design consideration. Mosquitoes are annoying pests that also have the ability to spread diseases such as the West Nile Virus, so any action necessary to prevent their accumulation on site must be taken into account. Since mosquitoes are attracted to standing water, Pennsylvania BMPs recommend that water not be left to stagnate for longer than 3 days (PA DEP, 2006). Aside from plant considerations described above, this will influence the mechanical components of the system, such as the necessity of an aeration pump or fountain in periods where little to no rainfall is available to naturally sustain flow in the system.

Several safety concerns arise with the addition of the wading pool, which is an unusual component in most constructed wetland systems. First, we must make certain that sediment that has settled in the treatment area is not disturbed. We must also guarantee that the water quality in the pool is suitable for recreational use. Though this issue was examined in our functional design constraints, a practical (i.e. non-technical) solution that provides further assurance is physically separating the pool from the treatment area. Depending on the makeup and concentration of pollutants in the runoff, this will be done by either a stone or plant barrier. Safety concerns also arise when considering the depth of the system, which might be a hazard to wandering animals or small children. Thus, the deep areas of the forebay and wetland must be blocked from easy entrance from wanderers, and the pond entrance must be carefully designed. As a secondary safety precaution, safety benches must be placed where depths in the area exceed 3 feet, and side slopes shall not exceed 3:1. These precautions will ensure that the pond area is safe for both invited and accidental users.

3. APPROACH

Figure 1: System block diagram illustrating pollutant removal and flow of runoff through the system

3.1. System Development

The stormwater management system was developed at the request of the client in response to their desire to reduce the household's municipal water usage. A water retention and treatment system was designed to treat rainwater runoff from the roof of the residence to be a supplemental supply of irrigation for the urban farm attached to the property.

Before fully developing this system, many factors had to be considered. First, the problems inherent to Philadelphia's combined sewer system, how to equip a residence with adequate gutters and channels to convey rain-water from the catchment area (roof), what plants to use in the wetland system, the feasibility of a natural pool for recreational use, how to store water for irrigation, how to transport water from the wetland/pool system to the irrigation system, and how to maintain acceptable water level during periods of low rainfall.

Figure 1 shows a simplified illustration of the system functions. Upon falling onto the roof of the house, runoff is collected in a gutter system surrounding the perimeter of the roof. Gutters convey runoff into several downspouts, which are equipped with screens at the entrance to prevent clogging later in the system. Each of these downspouts lead directly into an underground piping system. These pipes lead to the entrance of the treatment area, the forebay. It is here that larger particles settle out of the runoff and are detained. Once runoff has travelled through the forebay, it enters the constructed wetland system. The wetland features a serpentine path that slows the flow of water, allows additional particles to settle, and approximates plug flow. As runoff travels through the wetland, suspended particles continue to settle and wetland plants adsorb and transform trace metals and hydrocarbons. Once runoff has reached the outlet of the wetland, it has been treated sufficiently to be used for recreational and agricultural use. From there, it enters a pond that acts as primary water storage and a wading pool for residents. Once it fills, water from the pond will begin to flow into an underground cistern, whose additional storage space is sufficient to store a little over an inch of rain.

Finally, excess runoff is directed into a perforated overflow pipe. This pipe allows some of the overflow to infiltrate into the ground as it travels to the street. Furthermore, a submersible pump is installed in the cistern. This pump serves multiple purposes. First, it provides circulation of water in the system to ensure that water does not stagnate and that water that may become contaminated while in the pond can be retreated. Secondly, it will transport water, when needed, to a drip-flow irrigation system on the farm. This will allow residents to utilize water runoff and minimize their municipal water usage.

3.1.1. Hydrology

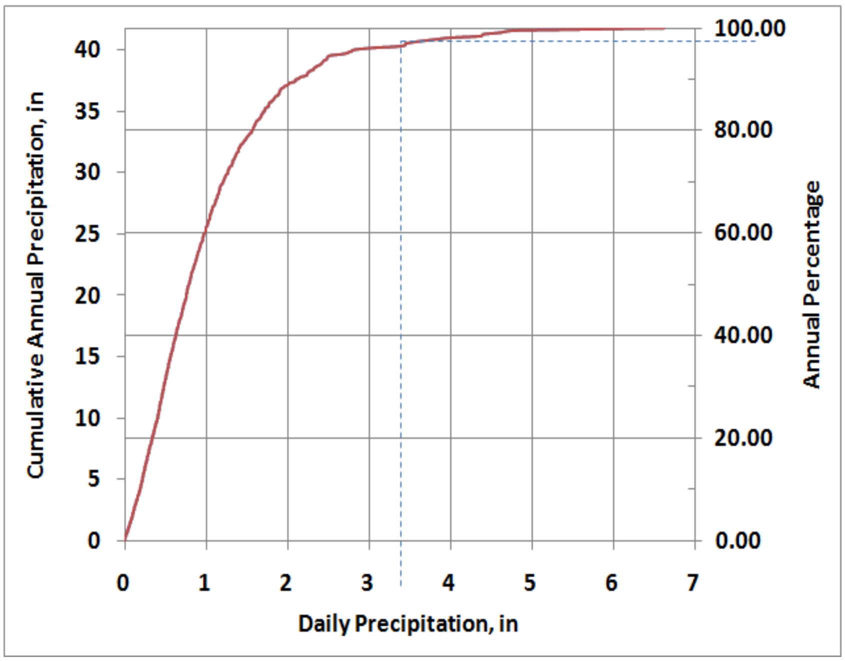


Figure 2: Graph of Daily Precipitation vs. Annual Percentage, compiled using Franklin Institute rainfall. The dashed line shows that about 96 percent of average annual rainfall occurs in storms that are 3.22 inches or less. 3.22 inches represents the two-year, 24-hour storm event for Philadelphia.

The hydrology on the site was a major design consideration for the system. Selection of a design storm and rainfall frequency estimates were necessary in order to ensure the system functions properly. The two-year, 24-hour storm was selected as the design storm for several reasons. First, the PA BMP Manual explains that, although current stormwater management systems are designed to detain large storm events, it is the smaller, more frequent events that must be mitigated to minimize erosion of urban streams and rivers (PA DEP, 2006). Secondly, annual rainfall data from the Franklin Institute from 1971-2001 indicate most rainfall events in a year are equal to or less than the two-year, 24-hour event. This data was sorted from lowest to highest Daily Precipitation values. A running sum of daily rainfall values over 30 years was formed and then divided by the number of days on record to obtain values for Cumulative Daily Precipitation. Then, these values were multiplied by 365 days in a year to obtain Cumulative Annual Precipitation values. Finally, each value along the running sum was divided by the final Cumulative Annual Precipitation value (41.8 inches) to obtain an Annual Percentage for each value. Daily Precipitation values were plotted against Cumulative Annual Precipitation and then against Annual Percentage to form two overlapping curves, which are shown in Figure 2. The blue dotted line illustrates that about 96 percent of the 41.8 inches of precipitation received annually occurs in storms that are 3.22 inches in depth or less. Precipitation frequency estimates from the National Oceanic and Atmospheric Association (NOAA) shown in Figure 3 indicate that the two-year, 24-hour storm for Germantown yields about 3.22 inches (NOAA, 2004).

Precipitation Frequency Estimates (inches)																		
ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.34	0.55	0.69	0.95	1.18	1.41	1.53	1.91	2.31	2.68	3.09	3.43	4.00	4.54	6.13	7.62	9.66	11.56
2	0.41	0.66	0.83	1.14	1.43	1.71	1.86	2.30	2.79	3.22	3.73	4.14	4.79	5.42	7.27	8.98	11.34	13.54
5	0.48	0.77	0.97	1.38	1.77	2.12	2.32	2.86	3.48	4.04	4.68	5.17	5.92	6.60	8.67	10.47	13.03	15.46
10	0.53	0.85	1.07	1.55	2.02	2.44	2.67	3.31	4.06	4.72	5.45	6.00	6.84	7.55	9.76	11.61	14.30	16.88
25	0.59	0.93	1.18	1.75	2.33	2.86	3.12	3.93	4.89	5.70	6.55	7.18	8.16	8.87	11.24	13.11	15.91	18.64
50	0.62	0.99	1.26	1.89	2.56	3.19	3.48	4.42	5.58	6.52	7.45	8.15	9.25	9.93	12.40	14.25	17.10	19.93
100	0.66	1.05	1.32	2.03	2.80	3.51	3.85	4.94	6.32	7.40	8.41	9.17	10.40	11.03	13.56	15.37	18.21	21.12
200	0.69	1.10	1.38	2.15	3.02	3.84	4.21	5.49	7.12	8.35	9.43	10.26	11.62	12.17	14.74	16.47	19.26	22.23
500	0.72	1.15	1.44	2.30	3.29	4.27	4.68	6.23	8.26	9.74	10.88	11.80	13.35	13.75	16.30	17.89	20.57	23.57
1000	0.75	1.18	1.48	2.40	3.50	4.60	5.04	6.82	9.20	10.88	12.06	13.05	14.76	15.00	17.50	18.95	21.49	24.51

Figure 3: Precipitation Frequency Estimates obtained from the National Oceanic and Atmospheric Association (NOAA, 2004)

Another concern was whether or not rainfall was sufficient enough to sustain plant life in the wetland. To be reassured about this, we sought to find the likelihood of filling our system to its minimum water levels in a given month. Taking into consideration design features such as a forebay without the capacity to be drained, as well as natural properties such as the water requirements needed for the most drought-sensitive plants, we determined that a minimum volume of approximately 2500 gallons was needed in the treatment area at any given time. This value was obtained with the following assumptions:

- Minimum level in the forebay = normal operating level = d_f = 5 ft
    - Minimum volume = normal operating volume = v_f = 5ft x 5ft x 5ft = 125 ft^3
- The wetland sits 3.75 feet below ground, and contains a stepped elevation where the deepest zone has a normal operating level of 3 ft and the shallowest holds 6 inches (see Table 7: Area and Volume Variance with Depth in the Constructed Wetland.). At minimum, the wetland must contain enough water to fill the shallowest level halfway, or 3 inches deep. Accounting for the stepped elevation, the minimum volume in the wetland is
    - V_W =211 ft^3
    - V_t = 211 + 125 ft^3 = 336 ft^3 x 7.48 gal/ft^3 = 2513.28 gal ≈ 2500 gallons

This volume of water corresponds to about 1.72 inches of runoff from the rooftop drainage area:

$$\frac{336\ \mathrm{ft}^3}{2350\ \mathrm{ft}^2} = 0.143\mathrm{ft} \times 12\ \mathrm{in/ft} = 1.72\ \mathrm{inches}$$

Thus, it was left to determine what the probability was of 1.72 inches of rain falling on site in any given month. Figure 4 shows a graph developed using Monthly Precipitation Probability data from NOAA (2002). The x-axis reveals the probability, based on frequency in years, of a cumulative precipitation depth which is shown on the y-axis. Months are represented by colored lines. The colored arrows corresponding to each month show the probability that 1.72 inches of rain will fall during that month. These probabilities range from 0.1 in March to about 0.3 probability in November. This means that, in the driest month, 3 out of every 10 years are likely receive 1.72 inches of rain or less, so there is a 70 percent chance that 1.72 inches or more will fall in that month. On average, there is an 83 percent chance of filling the system to minimum levels; in wet months, the probability is close to 90 percent (NOAA. 2002). On the basis of precipitation alone (before evaporation is considered), it appears that the wetland will receive adequate rainfall a majority of the time. Hydrologic modeling using EPA Stormwater Management Model (SWMM) software will solidify this argument, and is discussed in Section 4.2 of this document.

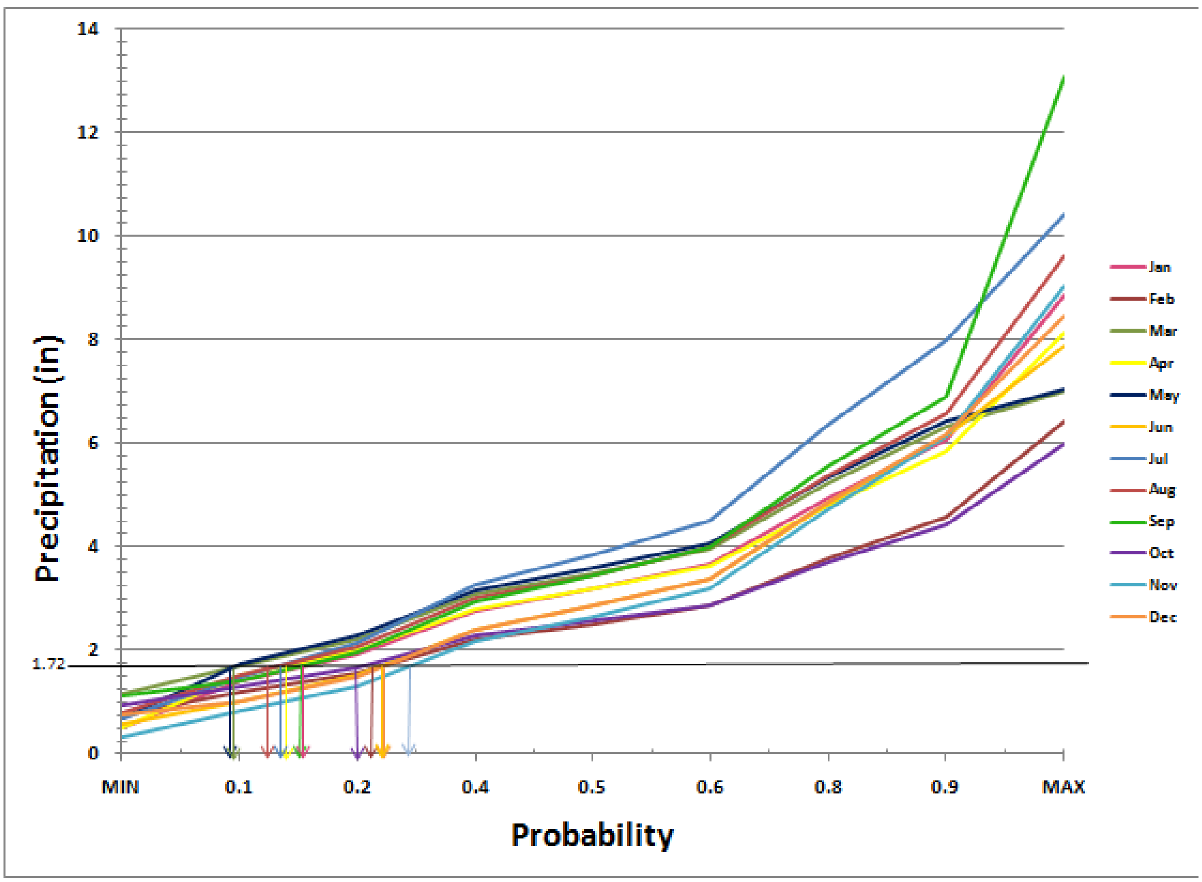


Figure 4: Monthly Precipitation Probabilities, 1971-2000. Arrows indicate the probability of 1.72 inches of rain occurring each month for any given year. Most of the arrows fall under 0.3 probability, suggesting at minimum a 70 percent probability that the area will receive 1.72 inches of rain or more each month.

3.1.2. Pollutants

As specified in Table 1, water used for irrigation must meet US EPA water standards. Though testing of the rooftop runoff for this particular site has yet to be completed, Mendez (2010) reveals seven pollutants present in asphalt roof runoff after a rain event, and the author explicitly advises against reusing runoff collected directly from asphalt shingles. The author also notes that older roofs may be more susceptible to pollutants in runoff (Mendez, 2010). Table 3 shows particulates, organic carbon, and eight different metals found in runoff from asphalt tile roofs both near and far from a busy highway in Austin, Texas (Van Metre and Mahler, 2003).

Sample	Date	Particulates (g/m^2)	As ($\mu g/m^2$)	Cd ($\mu g/m^2$)	Cr ($\mu g/m^2$)	Cu ($\mu g/m^2$)	Hg ($\mu g/m^2$)	Ni ($\mu g/m^2$)	Pb ($\mu g/m^2$)	Zn ($\mu g/m^2$)
ANM	8/30/99	0.54	4.4	2.6	37	55	0.12	18	200	900
ANM	11/22/99	0.99	8.0	3.4	69	91	0.24	30	310	1200
ANM	07/07/00	0.19	1.4	0.8	11	16	0.031	5.4	76	370
AFM	8/30/99	0.14	1.1	0.5	8.7	13	0.032	9.9	53	130
AFM	11/22/99	0.48	4.5	0.9	28	25	0.15	15	150	350
AFM	07/07/00	0.12	0.97	0.32	6.9	7	0.012	3.9	28	120

Table 3: Yields of particulates and particle-bound trace elements from rooftop washoff near Mopac Expressway (ANM) and far from Mopac (AFM), as found in Van Metre and Mahler (2003)

Figure 5 highlights the findings of Chang and Crowley's (1993) research on metal concentrations in asphalt shingle roofs in relation to North Carolina Department of Environment and Natural Resourses (N.C.DENR)fresh water standards for aquatic life and U.S EPA standards for drinking water. Lead (Pb) and Zinc (Zn) levels of 0.056 mg/L and 2.33 mg/L, respectively, were found. While only the lead concentration exceeds U.S EPA standards for drinking water (0.015 mg/l), both lead and zinc levels exceed N.C.DENR

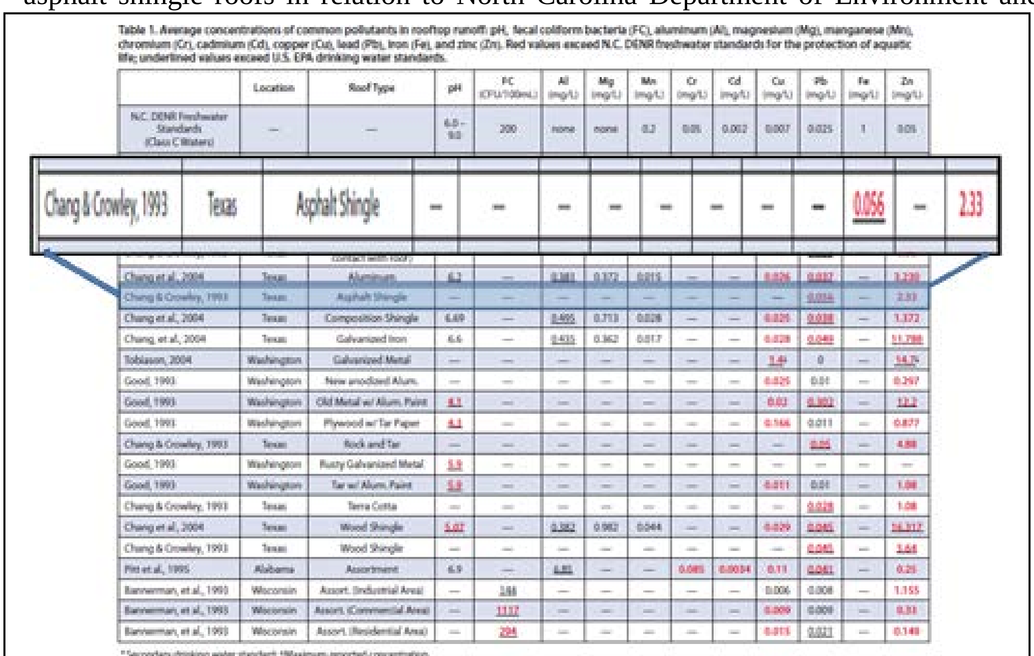


Table 1. Average concentrations of common pollutants in rooftop runoff: pH, fecal coliform bacteria (FC), aluminum (Al), magnesium (Mg), manganese (Mn), chromium (Cr), cadmium (Cd), copper (Cu), lead (Pb), iron (Fe), and zinc (Zn). Red values exceed N.C. DENR freshwater standards for the protection of aquatic life; underlined values exceed U.S. EPA drinking water standards.

	Location	Roof Type	pH	FC (CFU/100mL)	Al (mg/L)	Mg (mg/L)	Mn (mg/L)	Cr (mg/L)	Cd (mg/L)	Cu (mg/L)	Pb (mg/L)	Fe (mg/L)	Zn (mg/L)
N.C. DENR Freshwater Standards (Class C Waters)	—	—	6.0 – 9.0	200	none	none	0.2	0.05	0.002	0.007	0.025	1	0.05
Chang & Crowley, 1993	Texas	Asphalt Shingle	—	—	—	—	—	—	—	—	0.056	—	2.33
Chang et al., 2004	Texas	Aluminum	6.2	—	0.383	0.372	0.015	—	—	0.026	0.037	—	3.220
Chang & Crowley, 1993	Texas	Asphalt Shingle	—	—	—	—	—	—	—	—	0.056	—	2.33
Chang et al., 2004	Texas	Composition Shingle	6.69	—	0.495	0.713	0.028	—	—	0.025	0.038	—	1.372
Chang, et al., 2004	Texas	Galvanized Iron	6.6	—	0.435	0.362	0.017	—	—	0.028	0.048	—	11.798
Tobiason, 2004	Washington	Galvanized Metal	—	—	—	—	—	—	—	1.49	0	—	14.75
Good, 1993	Washington	New anodized Alum.	—	—	—	—	—	—	—	0.025	0.01	—	0.257
Good, 1993	Washington	Old Metal w/ Alum. Paint	4.1	—	—	—	—	—	—	0.02	0.302	—	12.2
Good, 1993	Washington	Plywood w/ Tar Paper	4.1	—	—	—	—	—	—	0.166	0.011	—	0.877
Chang & Crowley, 1993	Texas	Rock and Tar	—	—	—	—	—	—	—	—	0.05	—	4.88
Good, 1993	Washington	Rusty Galvanized Metal	5.9	—	—	—	—	—	—	—	—	—	—
Good, 1993	Washington	Tar w/ Alum. Paint	5.8	—	—	—	—	—	—	0.011	0.01	—	1.08
Chang & Crowley, 1993	Texas	Terra Cotta	—	—	—	—	—	—	—	—	0.028	—	1.08
Chang et al., 2004	Texas	Wood Shingle	5.07	—	0.382	0.982	0.044	—	—	0.029	0.065	—	16.317
Chang & Crowley, 1993	Texas	Wood Shingle	—	—	—	—	—	—	—	—	0.065	—	3.64
Pitt et al., 1995	Alabama	Assortment	6.9	—	4.85	—	—	0.085	0.0034	0.11	0.061	—	0.25
Bannerman, et al., 1993	Wisconsin	Assort. (Industrial Area)	—	144	—	—	—	—	—	0.006	0.008	—	1.155
Bannerman, et al., 1993	Wisconsin	Assort. (Commercial Area)	—	1117	—	—	—	—	—	0.009	0.009	—	0.33
Bannerman, et al., 1993	Wisconsin	Assort. (Residential Area)	—	294	—	—	—	—	—	0.015	0.021	—	0.149

* Secondary drinking water standard; †Maximum reported concentration.

Figure 5: Data chart found in N.C. State University's report in "Urban Waterways," ***Water Quality of Rooftop Runoff*** **(DeBusk, 2010)**

freshwater standards (0.025 and 0.05 mg/l, respectively) and zinc exceeds the EPA standard for irrigation use (2 mg/l). Based on these findings, we believe that water from the roof will need to be tested for metals, total suspended solids, organic matter, atmospheric deposition (pollutants that accumulate on surfaces during dry periods) and bacteria (bird/animal waste). Treatment methods were based upon these findings and are detailed in Section 3.4.3. Upon receiving results of testing, the planting plan and size of the constructed wetland can be modified for either more or less aggressive treatment.

3.2. Catchment Area

The catchment area for the treatment and irrigation system is the roof of the existing property. This area multiplied by rain event depth dictates the volumes of runoff the system will experience, in addition to that directly falling on the system during an event.

Figure 6: The residence providing the catchment area

The size of the catchment area was approximated by taking measurements of the residence during a site visit. The front of the residence is pictured from street level in Figure 6. Furthermore, a survey of the property was conducted. This was done to obtain the layout of the site and to determine elevations of the property (which was relatively flat). The raw calculations of the catchment area amounted to approximately 2350 square feet sheltering the residence. The total roof area was found by simply finding the areas of square sections of the roof and adding them together. Equation 1 was used for each rectangular piece of estimated roof footprint.

$$A = L * W \qquad \textbf{Equation 1}$$

Where A = area of the roof, L = length of roof and W = width of roof.

The type of roof on the house is very similar to a Mansard roof. A Mansard, as can almost be observed in Figure 6, is double sloped. The shallow top slope is barely visible above the very steeply sloped, almost vertical, sides. The sides draining directly into the gutters are the steepest and typically have a pitch ratio of at least 18:12 (rise over run). The pitch of the roof is important because it can affect the behavior of the runoff into the gutters. Therefore, when sizing gutters a roof pitch factor must be determined. Usually this is done by looking in a chart such as that in Table 3. A factor of 1.8 was obtained from the chart provided by Roof Estimator (2006). Multiplying this by our estimated roof area we were able to define the design area to be about 4230 ft^2.

$$DesignArea = (2350\,ft^2)(1.8) = 4230\,ft^2$$

Once the catchment area was defined we were able to calculate volumes that would be produced by any

Pitch Factor Chart

Roof Pitch	Pitch Factor (Gets Multiplied by Total Sqs.)
2/12	1.01
3/12	1.03
4/12	1.05
5/12	1.08
6/12	1.12
7/12	1.16
8/12	1.20
9/12	1.25
10/12	1.30
12/12	1.41
14/12	1.54
16/12	1.67
18/12	1.80
20/12	1.94
22/12	2.09
24/12	2.24

Table 4: Roof Pitch Factor Chart, used to determine design area for gutter sizing.

storm depth. One inch of rainfall on the unfactored catchment area was calculated using Equation 2 to yield approximately 1460 gallons of runoff.

$$V = AD \qquad \textbf{Equation 2}$$

Where V is the volume of runoff produced from the specified rain event of depth D, and A is the area of the catchment (roof) area. So:

$$(2350\,ft^2)(1in)\left(\frac{ft}{12in}\right) = 195.8\,ft^3$$

$$(195.8\,ft^3)\left(\frac{7.48gal}{ft^3}\right) = 1464gal$$

The design storm, with a depth of 3.22 inches, would produce approximately 4700 gallons of runoff using the procedure above.

3.3. Drainage

3.3.1. Gutters and Downspouts

Proper gutters will convey water from the roof to the forebay through downspouts that are directed into sloped and buried PVC pipes. These pipes run into a larger sloped and buried PVC pipe of approximately 50 linear feet. The design of gutters, though difficult to find published literature outside of plumbing codes, is a very integral part of the design of any rainwater conveyance systems. Gutter design is particularly important in this design as the current system does not properly convey water and causes flooding on the property. Safety is one great consideration when determining gutter sizes (McDanal, 2010). Gutters must be able to convey the largest storm event on record, usually the five-minute 100-year storm. Though this storm only has a one in 100 years chance of happening in a given year, design standards still require that it be used for stormwater disposal systems, including gutters and downspouts (City of Philadelphia, 2004).

Information needed to size gutters and downspouts:

- Design area, this is determined from the measured roof area and a pitch factor
- Rainfall intensity recorded for the geographical area
- Gutter length
- Area of roof drained (ft^2) per area of downspout (in^2)

Design Area

As mentioned in Section 3.2, the roof is of a Mansard style with a calculated design area of about 4230 ft^2. There will be a total of two separate gutters draining the roof. Therefore, the design area per gutter is about 2120 ft^2 per gutter.

Rainfall Intensity

As stated, the five-minute 100-year storm is required to be used in sizing gutters. The rainfall intensity of this storm is 9.02in/hr as obtained from NOAA. This was the highest of three confidence levels and chosen in order to provide a conservative design.

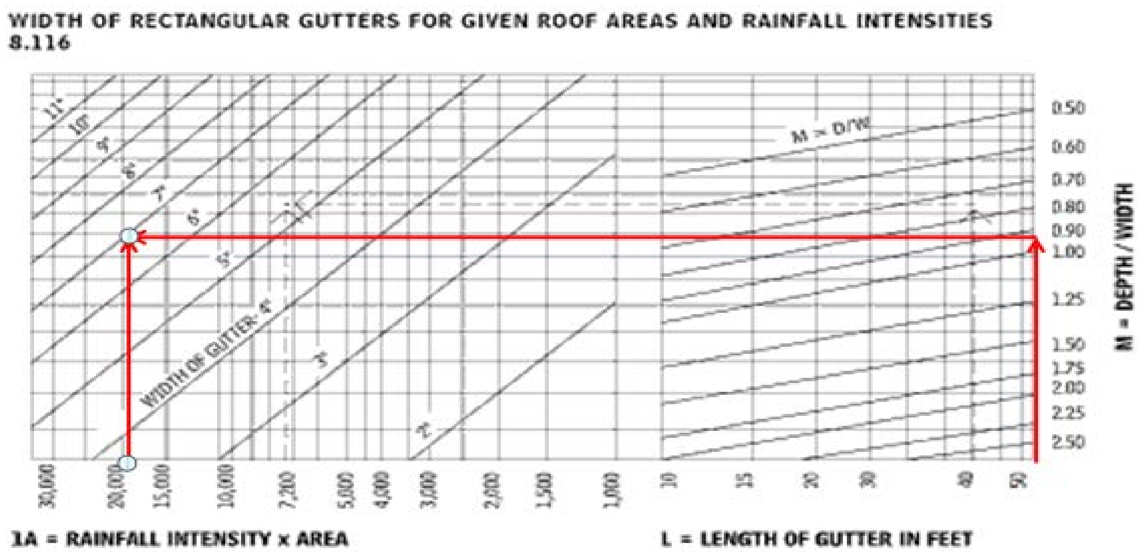


Figure 7: From Architectural Graphics Standards for Residential Construction (AIA, 2010) Chart to determine roof gutter width based off desired depth to width ratio and flow rate due to the entire roof and the IA value.

Gutter Sizing

To size the gutters, the length of gutter necessary to drain the roof was needed. This was obtained by measuring the perimeter of the house at the base of the structure.

Using Figure 7, obtained from "Architectural Graphics Standards for Residential Construction" (AIA, 2010), appropriate gutter sizes were determined. First, a desirable depth to width ratio (M in Equation 3) needed to be chosen. Typical gutter widths have been reported to be between four to eight inches (AIA, 2010). A value of 0.90 was chosen in order to utilize the recommended gutter sizing chart and keep gutters at a reasonable width in comparison to typical gutter widths. Rain intensity multiplied by the design roof area, IA, was the last variable needed to determine gutter width. The five-minute 100-year event of 9.02 in/hr multiplied by the design roof area of 2120ft^2 is approximately19000 in/hr/ft^2. Thus, with M and IA, one can see in Figure 7 that they intersect between 6 and 7 inches width, much closer to 7 inches. This suggests the minimum width for one gutter to drain the required storm intensity. Finally, with the aspect ratio of M = 0.90 and W = 7, algebraically manipulating Equation 3gives the gutter depth to be about 6.3 inches.

$$M = D / W \qquad \textbf{Equation 3}$$

Where M is the ratio chosen by the designer, W is the gutter width determined from the chart in Figure 7 and D is the gutter depth.

Downspout Sizing

Intensity in in/hr lasting 5 min	2’	3’	4’	5’	6’	7’	8’	9’	10’	11’
Sq ft roof/sq in of downspout	600 sf	400 sf	300sf	240 sf	200sf	175 sf	150 sf	130 sf	120 sf	110 sf

Table 5: Downspout drainage area capacity per square inch of spout cross-section based off most intense predicted storm event, *(AIA, 2010)*.

The100-year, 5 min storm event must be continually used for all sizing calculations. Referring to Table 4 obtained from “Architectural Graphics Standards for Residential Construction” (AIA, 2010), the area of roof drained per square inch of downspout area for a five-minute storm intensity of 9 in/hr is $130ft^2$. We will have two downspouts, one per gutter, with each gutter draining 2120 ft^2, as stated in section 3.2. Gutter drainage area divided by the 130 ft^2 suggests $16.3in^2$ in cross-section as the minimum area. This corresponds to about a 4 in diameter pipe needed. We will recommend a plain rectangular downspout of nominal width of 5 in whose actual dimensions will be 3 ¾in. by 4 ¾in. This supplies one gutter with almost $18in^2$ of cross-sectional area, slightly more than the minimum suggests.

Each downspout will drain into its own buried PVC pipe at a two-percent slope over 16 linear feet. Here each pipe will be joined together with a larger, PVC pipe (the main channel). The main channel conveys the roof runoff to the forebay of the constructed wetland. The location of the junction should be 10 linear feet from the base of the back of the house, approximately in a central location. The main channel is discussed in Section 3.3.2

Flow Calculations

The volumetric flow rate, Q, of rainwater in the gutters and channels needed to be determined. This is to ensure that the dimensions calculated will be able to theoretically contain the most intense storm recorded.

The rate of flow entering the gutter was determined to approximately be 26.5 ft^3/min, 200 gal/min or 3.3 gal/s. Calculations for flow will be computed in gal/s. These values were found using the Rational Method (see Equation 4). This method has been in use for over a century in determining peak flow of runoff in small drainage areas (PA DEP, 2006). The runoff coefficient, C, in the equation can be taken to be unity (a value of 1) when determining Q for roof drainage (Graber, 2009). The theoretical capacity flow rate of the gutter was then calculated to be about 13.12 gal/s using the previously determined gutter dimensions and Manning’s equation in volumetric flow rate form Equation 5 for open channel flow (Munson, 2009**)**.

From these values we see that the gutter capacity designed appears to be much larger than that of the roof runoff. Therefore, the gutters should be able to withstand the five-minute 100-year storm event, as required for in design.

$$Q = CiA$$ **Equation 4**

The Rational Method equation, above, shows the flow rate, Q as a function of the runoff coefficient (C), the rainfall intensity (*i*), and the area of overland flow (A). Or, in this case, it is the area of the roof drained by a single gutter.

In the case of our system, the intensity is 9.02in/hr or 0.752ft/hr and the area is 2120ft^2. This calculation yields a runoff rate of 1594ft^3/hr as determined below, corresponding to a value of 3.3 gal/s.

$$Q_{runoff} = (1)(0.752\,ft/hr)(2120\,ft^2) = 1594\,ft^3/hr$$
$$= 1594\,ft^3/hr = 0.443\,ft^3/s * 7.48 gal/\,ft^3 = 3.31 gal/s$$

Velocity of flow in a pipe may be computed using *Manning's Equation* (Munson, 2009):

$$V = \frac{\kappa}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$ **Equation 5**

Where *n* is Manning's coefficient (a value usually found in a table), κ is a conversion factor. Here, κ is 1.49. R is the hydraulic radius in ft and S is the slope of the open channel in ft/ft. See Equation 7 for how to calculate R. Gutters and downspouts were designed to be rectangular, and the underground pipes conveying runoff to the treatment area are circular.

The following is a sample calculation using Manning's equation for finding the velocity, 5.76 ft/s, of flow in the gutters. Since the gutters will be rectangular, R was found to be about 0.188 ft and the slope will be two percent. Therefore, S is 0.02 and n for smooth steel is reported as 0.012 (Munson, 2009). Smooth steel will be the material used for the gutters. If that should change, calculations can easily be adjusted to reflect any changes.

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$
$$= \frac{1.49}{0.012}(0.188\,ft)^{\frac{2}{3}}(0.02)^{\frac{1}{2}}$$
$$= 5.76\,ft/s$$

Volumetric flow rate must be calculated to ensure the rate of flow entering the gutters (as determined by the Rational Method) does not exceed that of the capacity of the gutter. We see now that Q = VA by looking at Manning's equation in volumetric flowrate form, Equation 6.

$$Q = \frac{1.49}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$ **Equation 6**

Since,

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Therefore,

$$
\begin{aligned}
&Q = VA \\
&= (5.76\,ft/s)*(.3061\,ft^2) = 1.76\,ft^3/s * 7.48 gal/ft^3 \\
&= 13.2 gal/s
\end{aligned}
$$

Below, you will find the expressions for the values that are defined by the dimensions of the gutters and channels. These values will be used in the Manning's equation.

$$R_{rectangular} = \frac{A}{P} \rightarrow R_{circular} = \left(\frac{\pi D^2}{4}\right)\left(\frac{1}{\pi D}\right) = \frac{D}{4}$$ **Equation 7**

Here, D is the pipe diameter, if you have a circular component. A is the cross-sectional area for rectangular or circular components, and P is the wetted perimeter. Below is a sample calculation using the gutter dimensions for determining R. The area was determined to be $0.3061 ft^2$ by multiplying the depth (6.3in) times the width (7in).

$$R_{rectangular} = \frac{0.3061\,ft^2}{1.63\,ft} = 0.188\,ft$$

$$P_{gutter} = 2D + W$$ **Equation 8**

Here, D is the depth of gutter and W is the width of gutter.

$$P_{gutter} = 2(6.3in) + (7in) = 19.6in = 1.63\,ft$$

3.3.2. Channel

The downspouts will be connected to a PVC pipe as mentioned in Section 3.3.1 to transport the water efficiently and inexpensively. Initially the system was designed to use a pre-fabricated channel drain. The pre-fabricated drain being an assemblage consisting of an open channel and a grate fitted specifically for it that would have allowed great ease in maintenance. However, for economical purposes, the design was reevaluated to use simple PVC piping. The following describes the design of the main channel based on the most intense rain event, the 100-year, 5min event.

First, the flow rate produced in the channel by the 100-year, 5min event needed to be calculated. Over the length of the gutter into the downspout this came to be about 3.3gal/s. This flow rate helps us decide the minimum size of the channel drain we would need to choose.

Since this single drain will be receiving the runoff from the entire design area, twice as much as either gutter, the diameter must be larger. A similar procedure for determining downspout sizing was used. Downspouts subjected to a 9 in/hr storm event should be able to drain $130 ft^2$ per square inch of downspout cross-sectional area. Therefore, considering the entire roof area the area of $2350 ft^2$, the cross sectional area of the channel should be 32.5 square inches. This would correspond to a pipe that is completely full when considering the predicted 100 year storm. However, in order to utilize Manning's equation for open channel flow, we would want a rain event from that storm to only fill the pipe to its centerline. Therefore, we double the cross-sectional area to assume that would accomplish a half full pipe, which comes to be about 65 in^2 in cross-sectional area. This would then correspond to a pipe diameter of 9in.

$$D = \sqrt{\frac{4A}{\pi}} \quad \textbf{Equation 9}$$

$$D = \sqrt{\frac{(4)(65in^2)}{\pi}} = 9.09in$$

We designed the pipe to be laid a linear distance of 50 feet with a two-percent slope. The vertical distance then drops only one foot to the inlet of the forebay. The flow in this main pipe was determined to be about 13 gal/s, using Equation 6. This is assuming, through our calculations, that the five-minute 100-year storm will not entirely fill the pipe. The hydraulic radius used here was determined to be 0.1875ft using Equation 7.

3.4. Treatment

A constructed wetland system is a natural way to remove pollutants from urban runoff while also limiting volumetric flow into the sewer system during a storm event. Many pollutants removed by a constructed wetland are commonly found in roof runoff. Wetlands are able to treat heavy metals and nutrients with high efficiency (Yeh, 2008)(PA DEP, 2006). Though not specifically intended for this design, wetlands have also been shown to remove carcinogenic hydrocarbons (Fountoulakis et al, 2009). Thus, a combination of constructed wetland, forebay, pond, and storage cistern has been designed to meet the water quality and quantity needs of the client. Figure 7 shows a plan view of the treatment area, covering 396 ft^2.

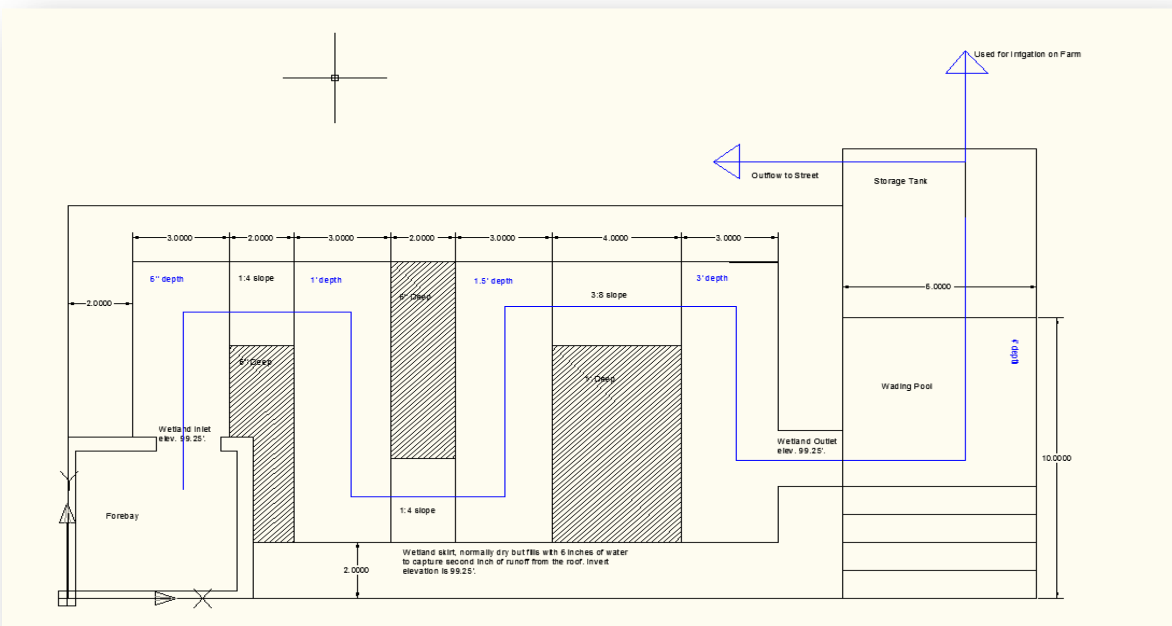


Figure 8: Plan view of system treatment area including 125 ft2 forebay inside of a 336 ft2 wetland, 60 ft2 pond, and 2000 gallon underground storage cistern

The system is designed to retain and treat the first two inches of runoff from the catchment area, as dictated by the constraints. The first inch is diverted to a storage tank while the second inch is retained in the wetland and pool. Everything beyond these first two inches will overflow through a controlled outlet.

Flow throughout the system is driven by gravity. The channels between each component are all at the same elevation and not sloped. This necessitates the buildup of water to the normal operating level in one component before flow into the next can occur. All invert elevations will be given with the assumption that ground level of the relatively flat backyard where the system will be constructed is 100'.

3.4.1. Forebay

The main function of the forebay is to slow water entry into the constructed wetland to avoid erosion in the system and damage to the plants, while also settling out most large particles (PA DEP, 2006)**.** These particles will settle out to the bottom of the five foot deep forebay**.** As “sedimentation is the primary process for the removal of heavy metals from stromwater,” most heavy metal removal will occur in the forebay (Yeh, 2008). The forebay’s function is to remove TSS and heavy metals, while also slowing water to between .3 and .5m/s. If the flow of the water is not slowed, plants in the system may be damaged or the system may experience erosion. Care must also be taken to prevent turbulence in the bottom of the forebay which would allow reentry of particles into the water. Thus, the inlet to the forebay includes stones which will break up the flow.

The dimensions of the forebay are to be 5’x5’x5’, with a volume of 125 ft^3. At peak rate flow for the two-year 24-hour storm, the forebay will have a detention time of 4.81 hours. This is computed using the equations:

$\boldsymbol{Q = CiA}$ **Equation 10**

$.99 * 3.22in * \frac{1ft}{12in} * 2350ft^2 = \frac{624ft^3}{24hr} = \frac{26.0ft^3}{hr}$ **Equation 11**

$$\frac{125ft^3}{\frac{26.0ft^3}{hr}} = 4.81hr$$

Although water testing has not been completed, the smallest particle that will settle out during the detention time of 4.81 hours can be calculated using Stoke’s Law. Because zinc and lead concentrations are of primary concern, the smallest particle that can be settled out is of particular interest. Once we obtain a breakdown of particle size in the metals, we can project what percentage will be removed through settling in the forebay.

Stokes’ law: $v_s = \frac{g(\rho_s - \rho)d^2}{18\mu}$ **Equation 12**

The minimum velocity at which particles settle out may be computed by dividing the depth by the detention time. Stokes’ law may be used to calculate the minimum diameter of the particles that will settle out fully in the forebay. The equation to find diameter then becomes:

$d = \sqrt{\frac{18 v_s \mu}{g(\rho_s - \rho)}}$ **Equation 13**

Temperature of the water entering the system will affect the viscosity and thus the settling velocity. The temperature range in Table 6 shows the minimum size of particles that can be removed in the forebay from season to season. According to these calculations, smaller particles are allowed to settle, and thus more particles will completely settle out, when the water is at a higher temperature.

Detention Time (hr)	height (ft)	metal	Temp (F)	Temp (C)	Viscosity [m(Pa*s)]	Density (kg/m^3)	min Diameter (m)	D (micrometers)
4.81	5	Zn	39.2	4	1.567	2650	0.000392	392
		Zn	50	10	1.307	2650	0.000358	358
min velocity		Zn	70	21	0.998	2650	0.000313	313
1.039501	ft/hr	Pb	39.2	4	1.567	2650	0.000392	392
0.0002888	ft/s	Pb	50	10	1.307	2650	0.000358	358
0.0000881	m/s	Pb	70	21	0.998	2650	0.000313	313

Table 6: Represents diameter of particles that can be removed in the forebay.

The forebay must be dredged whenever sediment builds up to half of its depth. The amount of particles settling out should not be affected by this buildup. This is because even though the retention time is halved, the amount of time necessary for a particle to reach the bottom and settle out is also halved. The forebay is to be built of concrete so it may be dredged without breaking the seal of the pond liner. A marking system will be employed to show how much sediment has built up in the forebay. Dredging is expected to be necessary somewhere between 3 and 7 years (PA DEP, 2006).

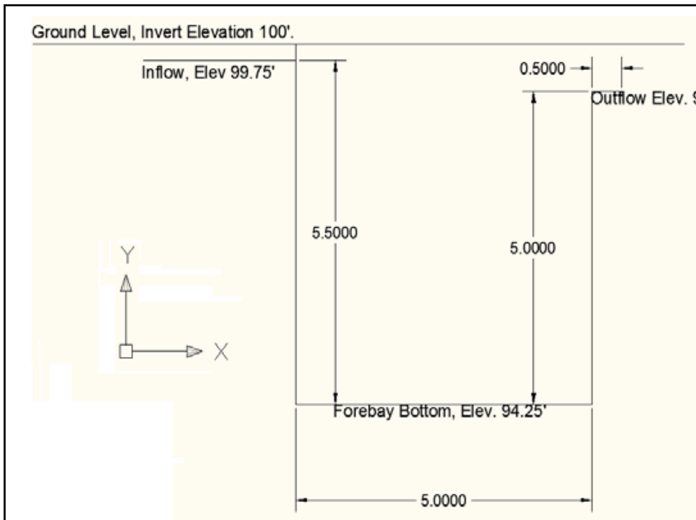


Figure 9: CAD drawing of forebay with inlet and exit heights

The forebay's invert elevation will be at 94.25ft. The inlet to the forebay will be at an invert elevation of 99.75ft, with the outlet at 99.25ft. The normal operating depth of the forebay will be five feet. An additional six inches of water can be stored on top of this normal operating depth after the storage tank fills, giving it a total storage potential of 1028.5 gallons.

The forebay is 5 ft in depth and lacks safety benches in order to keep the surface area small. This creates a potential hazard for children and animals that might be drawn to the small pool. Thus, we recommend that a fence be installed around the perimeter of the constructed wetland. A four-foot high wrought-iron fence is aesthetically pleasing and should suffice to keep the area safe. As there are chickens on the property, the fence posts should be spaced no more than 6 inches apart. We recommend that the fence surround not just the wetland but the entire treatment area, and be separated from the area by about 3 feet on three sides and 5 feet on one side. This will increase the area of the backyard the system occupies to almost double the original size (792 ft^2) of the system. However, 3 feet allows ample walking space and 5 feet encourages the possibility to install benches next to the wetland if the residents wish to utilize the area for passive recreation. We feel that this addition will provide safety in a way that also encourages use and enjoyment of the system. Finally, to facilitate maintenance, the fence gate should be about ten feet wide and placed in front of the forebay so that a backhoe can easily enter and dredge the forebay.

3.4.2. Constructed Wetland

A constructed wetland was chosen for treatment because they have been shown to reduce common runoff pollutant levels while also providing peak rate retention during storm events. Chiefly, the aim of the wetland is to lower pollutant metal concentrations of lead and zinc to below 5.0 mg/L and 2.0 mg/L respectively and also to retain the first inch of runoff to meet our design constraints.

Different conditions in constructed wetlands influence removal rates of heavy metals. Smaller experimental systems tend to show higher removal rates, over 90% in some cases (Mungur et al, 1997). However, other data from full scale operational constructed wetlands shows slightly lower removal rates of zinc and lead, at 78.3% and 62.2% respectively (Kropfelova et al, 2009). It is these lower removal values which will be used to project the efficiency of treatment in our constructed wetland. Metals are removed through settling, adherence to soil, and uptake by plants. Different varieties of plants exhibit different removal rates of specific heavy metals.

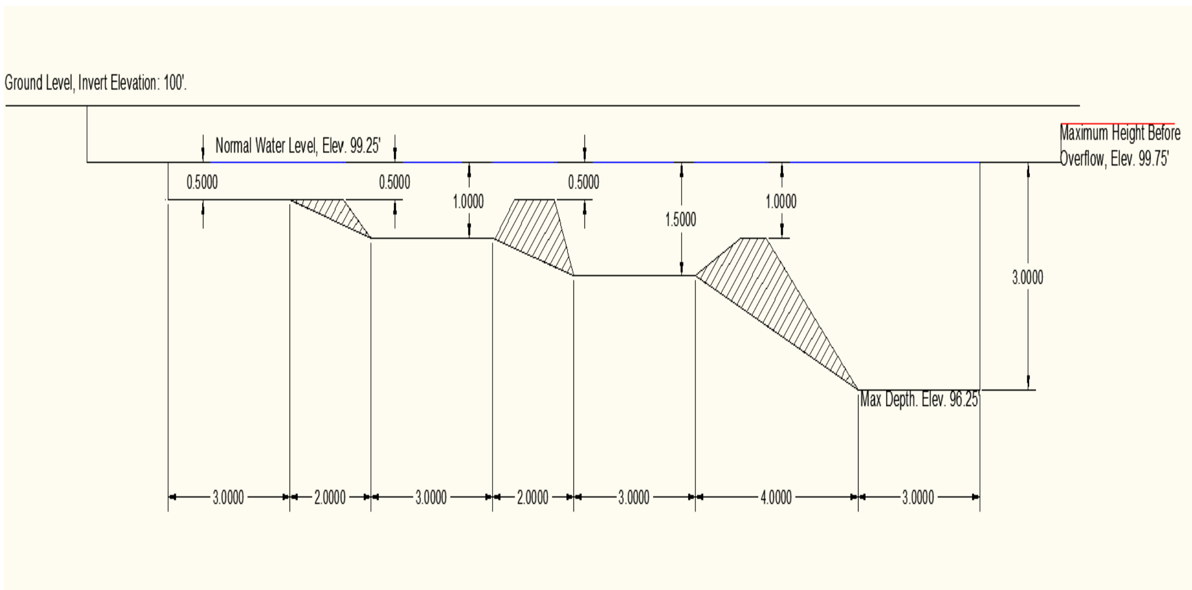


Figure 10: CAD drawing of constructed wetland showing the dimensions of the system. The shaded regions are built up sections of soil to allow varying plant growth in the system. See Figure 11 for a plan view of the wetland.

The design was formulated with the 200 ft^2 constraint in mind. The wetland dimensions would be 10'x20' in that case. However, assuming the treatment area is at capacity, the rise in water level due to a one-inch storm would be too great and could potentially impact marsh plant life. Ideally the water level would not rise much more than six inches from its normal operating level. At 200 ft^2 with a 60 ft^2 pond, water level would rise about 12.7 inches.

$$\frac{V_{roof\ runoff}+V_{rainfall\ on\ wetland}}{A_{wetland}} = \Delta \boldsymbol{h} \qquad \textbf{Equation 14}$$

$$195.8ft^3\ (from\ roof) +\ 200ft^2 * 1\frac{ft}{12\ in} * 1in = \frac{212.5ft^3}{200ft^2} = 1.06ft * \frac{12in}{ft} = 12.7in$$

The rise in water level should be no more than six inches at full capacity because most Zones 3 and 4 plants listed in Table 8 cannot survive a larger rise in water level. Therefore a skirt two feet in width was added to the perimeter of the wetland. This skirt will be dry at normal operating depth and planted with draught-tolerant grasses. This makes the new wetland dimensions 14'x24', and the new area 336ft^2. Adding the pond's 60ft^2 surface area gives a total square footage of 396 ft^2, just under double our expected area. This is roughly 17% of the drainage area of the roof. Typically a constructed wetland will be between 3-5% of the area it drains from (PA DEP, 2006). The need for a much larger percentage is due to 100% of the drainage area being impervious, which is not typical of most wetland drainage areas. The rise in water level required to retain the second inch of rainfall in this system is 6.93 inches before controlled overflow.

$$195.8ft^3\ (from\ roof) +\ 396ft^2 * 1\frac{ft}{12\ in} * 1in = \frac{228.8ft^3}{396ft^2} = .578ft * \frac{12in}{ft} = 6.93in$$

The wetland portion of this system is to have depths varying from six inches to three feet at normal operating height. See Table **7** for area and volume variation with depth. There are to be elevated high and low marsh zones in the deeper portions of the wetland to promote a serpentine flow pattern. A serpentine flow pattern allows the flow to be approximated as plug flow, maximizing treatment time and minimizing mixing of treated and untreated water. These zones also provide varying depths throughout the wetland, promoting varied plant life. The total volume retained when water is at "normal operating level" is 415 ft^3.

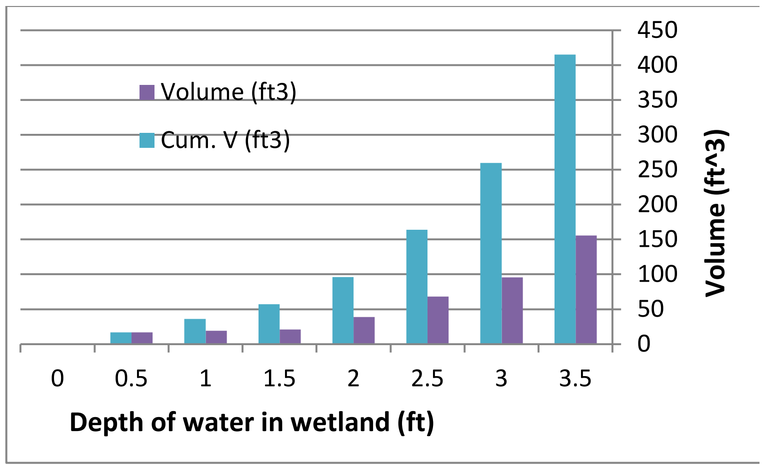


Figure 11: Staged and Cumulative Volume Variance with Depth in the Constructed Wetland

In reality the wetland has been designed with a six inch rise in water level in mind. This is slightly less than the 6.93 inches required to retain the entire second inch of runoff. The forebay and wetland together will retain an additional 168ft^3 (1250gal) of water when the water level rises six inches. This will be lower than the 194 ft^3 which would be removed if the water level were to rise 6.93 inches. The solution for this will be to capture more of the runoff initially with an underground storage tank.

Invert Elev. (ft)	Elev from bottom of wetland (ft)	Surface Area (ft^2)	Volume (ft^3)	Cum. Vol. (ft^3)
99.75	3.5	311	155.5	415
99.25	3	191	95.5	259.5
98.75	2.5	136	68	164
98.25	2	78	39	96
97.75	1.5	42	21	57
97.25	1	38	19	36
96.75	0.5	34	17	17
96.25	0	30	0	0

Table 7: Area and Volume Variance with Depth in the Constructed Wetland.

Although the treatment system nearly doubled in size, from 200 to 396 square feet, it still only renders 5.2% of the backyard unusable. The total square footage of the backyard is 7665 ft^2.

Both the inflow and outflow of the system are situated at an invert elevation of 99.25'. This allows water to fill the system to its normal operating level before the cistern begins to fill. Once the cistern is full, the 6 inches of extra space will be utilized, and once it is used overflow through a perforated pipe will occur.

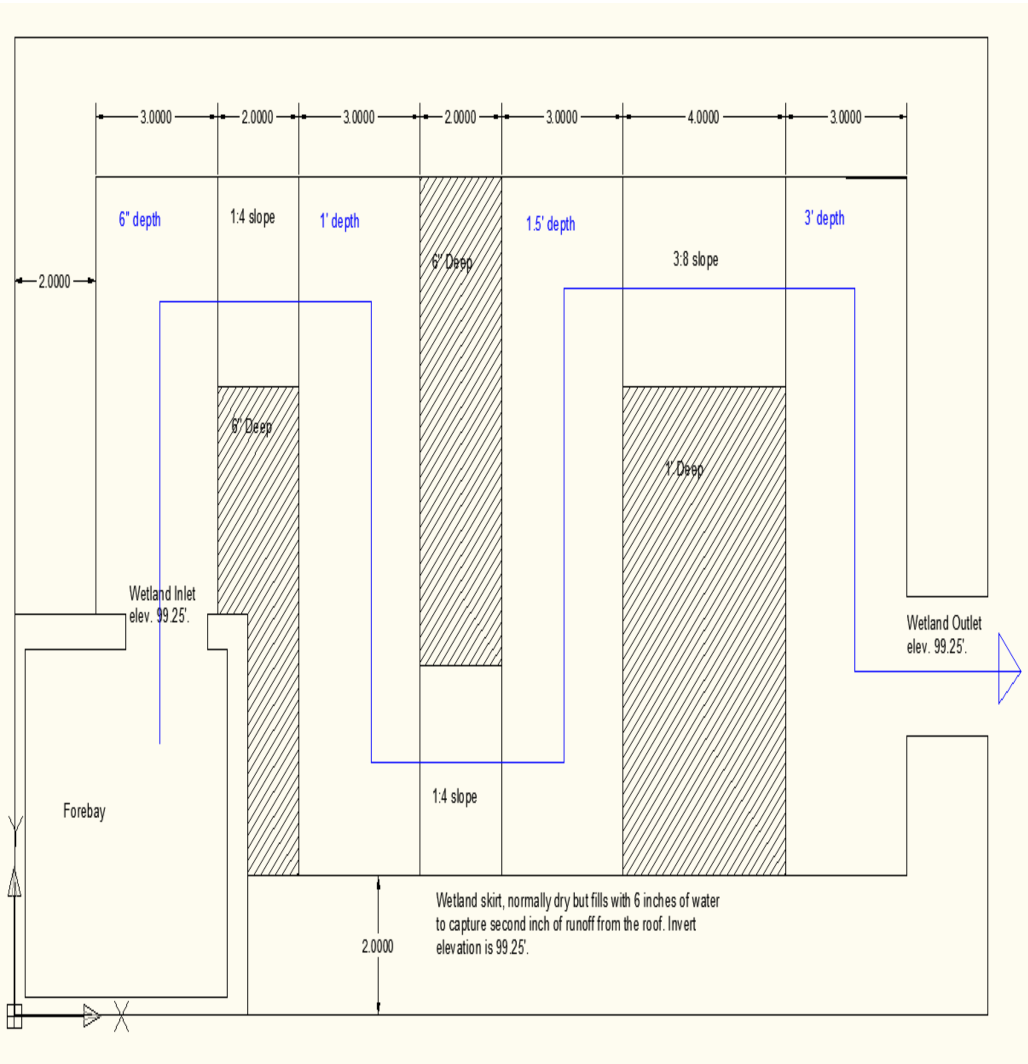


Figure 12: Plan view of wetland. Shaded regions are high and low marsh zones to allow varied plant growth as well as promote a serpentine flow path in the wetland.

A summary of all the water stored in the treatment system is as follows:

System Component	Initial Volume	Max Volume	**Total Volume that can be captured (assuming initial conditions): 1188-720 = 468 ft^3 = 3500 gallons**
Forebay	125ft^3	137.5 ft^3	
Wetland	415ft^3	570.5 ft^3	
Pond	180 ft^3	210 ft^3	
Cistern	0 ft^3	270 ft^3	

Table 8: Volume Retention In Each Treatment Component

3.4.3. Plant Selection

Plant selection for the constructed wetland is shown in Table 8, which includes the hydrologic zone and water depth, reason selected, estimated removal efficiency, and wetland indicator symbol for each plant. The basis for these parameters is explained in detail below.

Aside from settling, pollutant removal by plants is the primary means of water treatment in this system. Research shows that different kinds of plants can efficiently remove pollutants, including metals and hydrocarbons, from runoff (Cheng, 2002). Floating plants store metals into their own biomass, while emergent plants have been shown to “immobilize” pollutants into belowground biomass (Marchand, et. al, 2010). Emergent plants are also effective in removing organic matter in the same manner. Marchand et al. (2010) reported removal efficiencies for pondweed (about 66% for zinc and 79% for lead), bulrush (about 81% for zinc, and 87% for lead), and cattails (84%- 99% for zinc and 89%-99% for lead). These can be compared in Table 8. Adams (1992) states that rush removes heavy metals, including, cobalt, copper, manganese, nickel and zinc, and reiterates that bulrush are known to remove bacteria, oil, organics and nutrients and are some of the most aggressive water purifying plants in wetlands.

Using the minimum of these expected removal rates given by Marchand et al. (2010), along with the expected runoff pollutant levels outlined in section 3.1.2, we can estimate the concentrations of some of the pollutants in the effluent:

$$[\boldsymbol{X}]_{\boldsymbol{i}} - ([\boldsymbol{X}]_{\boldsymbol{i}} \times \boldsymbol{\eta}) = [\boldsymbol{X}]_{\boldsymbol{f}} \qquad \textbf{Equation 15}$$

Where $[\boldsymbol{X}]_{\boldsymbol{i}}$ and $[\boldsymbol{X}]_{\boldsymbol{f}}$are the initial and final concentrations of the given pollutants, respectively, and η is the minimum expected removal efficiency of the plant. Thus,

$$[\boldsymbol{Pb}]_{\boldsymbol{f}} = \mathbf{0.056}\frac{\boldsymbol{mg}}{\boldsymbol{l}} - \left(\mathbf{0.056}\frac{\boldsymbol{mg}}{\boldsymbol{l}} \times \mathbf{.79}\right) = \mathbf{0.012}\frac{\boldsymbol{mg}}{\boldsymbol{l}}$$

$$[\boldsymbol{Zn}]_{\boldsymbol{f}} = \mathbf{2.33}\frac{\boldsymbol{mg}}{\boldsymbol{l}} - \left(\mathbf{2.33}\frac{\boldsymbol{mg}}{\boldsymbol{l}} \times \mathbf{.66}\right) = \mathbf{0.79}\frac{\boldsymbol{mg}}{\boldsymbol{l}}$$

$$[\boldsymbol{Hg}]_{\boldsymbol{f}} = \mathbf{0.22}\frac{\boldsymbol{mg}}{\boldsymbol{l}} - \left(\mathbf{0.22}\frac{\boldsymbol{mg}}{\boldsymbol{l}} \times \mathbf{.5561}\right) = \mathbf{0.098}\frac{\boldsymbol{mg}}{\boldsymbol{l}}$$

indicating that utilization of the selected plants will more than ensure that effluent reused in the pond and irrigation system will meet federal standards for irrigation (5 mg/L for Pb and 2 mg/L for Zn), and in some cases surpasses the standard for fresh water (0.025 mg/L for Pb and 0.05 mg/L for Zn) and even drinking water (0.015 for Pb).

Zone	Depth	Plant Selected	Reason Selected	Estimated Removal Efficiency			Wetland Indicator Symbol
				Zn	Hg	Pb	
Zone 1 "Open Water"	1'-3'	Water Lilly	Aesthetics, Wetland indicator status				OBL
	1'-6'	Pondweed	Not many long-leaf need to be planted, as they spread quickly, wetland-indicator status	66		79	OBL
Zone 2 "Shallow Terrace– Aquatic Bench"	0.5'-1'	Bul rush (soft-stem)	Is a "grass-like" plant type; is an "aggressive" colonizer and has high pollutant removal properties	81.09	55.61	87.26	OBL
		Rush (soft)	Is a "grass-like" plant type, Tolerates occasional dry conditions, shown to have good nutrient uptake properties	✓			FACW+
		Broadleaf Cattail	High metal tolerance and removal	84-99	--	89-99	OBL
Zone 3 "Low – Marsh" BMP "Fringe" Area	0'-0.5'	New York Ironweed	Aesthetics, a tall plant with purple flowers, attracts butterflies				FACW+
		Blue Joint Reed Grass					FACW+
		Tussoc Sedge	Persists during winter, attracts songbirds				FACW
Zone 4 "High – Marsh" BMP Fringe Saturated Periodically Inundated	0'	Switchgrass	Is a seasonal grass and tolerates wet/dry conditions				FAC

Table 9: Description of plants selected for use in the constructed wetland.

Although some plants are particularly known for their ability to tolerate and treat metal-laden waters, most sources suggest that the type of plant (Mungur, 1997) or even the diversity of plant selection (Marchand 2010) are not as relevant to removal efficiency as is plant density (i.e. quantity or biomass of plants in a given area). However, selection of diverse and native plant species and replication of natural wetlands is recommended practice (PA DEP, 2006). Native species are preferred for their likelihood to attract native insects and animals, and their selection is deemed more responsible than selection of foreign, potentially invasive species. Diversity of selection is encouraged because it helps the wetland resist pests and invasive species and attract a more diverse population of wildlife (Davis, 1994). And finally, replication of natural wetlands is important to help ensure the strength and resilience of the constructed wetland (PA DEP, 2006). To facilitate the selection process, he PA BMP Manual (2006) provides a list of plants native to Pennsylvania, along with their “wetland indicator status”. The indicator symbol signifies the probability of finding an individual plant in a natural wetland setting. As a guide (USDA, n.d.):

OBL	Obligate Wetland	Occurs almost always (estimated probability 99%) under natural conditions in wetlands.
FACW	Facultative Wetland	Usually occurs in wetlands (estimated probability 67%-99%), but occasionally found in non-wetlands.
FAC	Facultative	Equally likely to occur in wetlands or non-wetlands (estimated probability 34%-66%).
FACU	Facultative Upland	Usually occurs in non-wetlands (estimated probability 67%-99%), but occasionally found on wetlands (estimated probability 1%-33%).

A “+” or “-” after the indicator symbol simply indicates that the plant is more or less often found in wetlands than others with the same symbol. The Hydrologic Zones identified in Table 8 are yet another classification by which wetland plants are identified. Along with indicating the depths of water that a particular plant can tolerate, the Zones assist the planner in choosing the correct plants for various expected depths.

Based on these considerations, we expect the design to meet our design criteria of minimum pollutant removal, biodiversity, and pest and vector control.

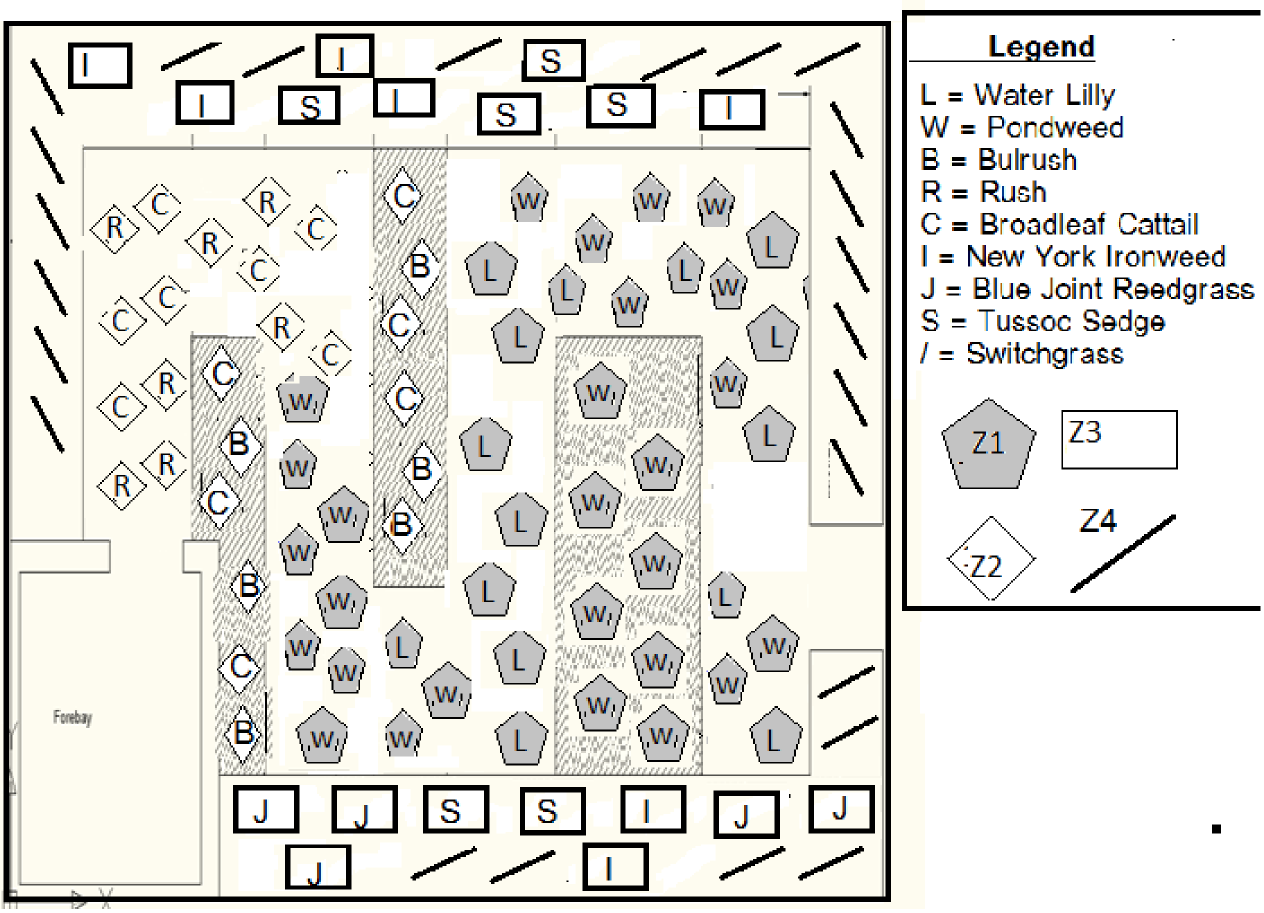


Figure 13: Planting Plan for the Constructed Wetland

Figure 12 shows a suggested planting plan for the constructed wetland based on the plants selected and their maximum depth tolerances (See Figure 7 for corresponding wetland depths). The most hardy and contamination-resistant plants were placed towards the entrance of the wetland in order to protect the more sensitive floating plants at the exit. Following a general rule of thumb, plants were spaced 18” apart and plants of the same species were clustered to replicate natural plant patterns. The drought-resistant grasses were placed around the perimeter as this area will be the last to receive runoff.

3.4.4. Wading Pool

The wading pool requested by our client provides both an amenity and more storage for the runoff collected. The pool, like the wetland, will be capable of retaining six inches of water in addition to its normal operating level. The normal operating depth of the pool is four feet. Figure 14 and Table 9 show variation of area and volume in the pool.

In keeping with the requirements outlined in Table 2, benches must be included for depths exceeding three feet. Stairs 1’x1’ are included to ensure that accidental users of the pool, including small children, will be able to climb out.

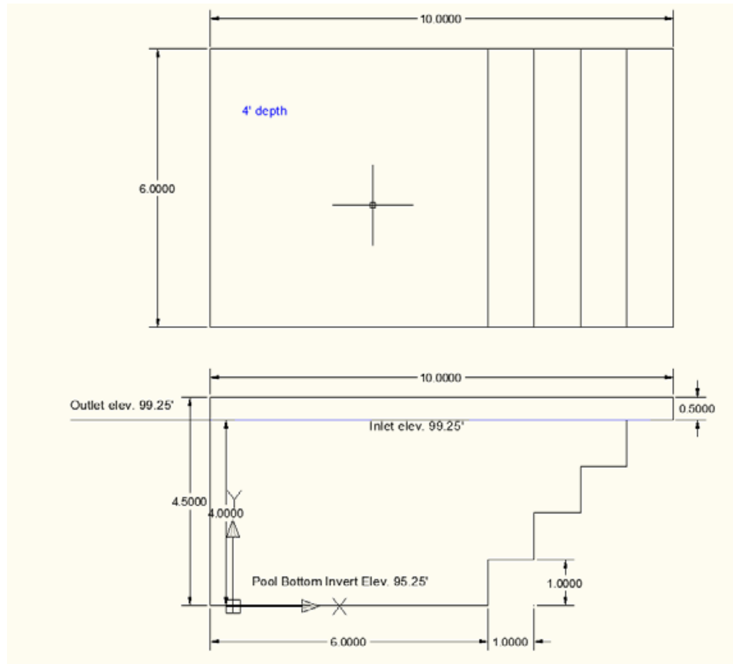


Figure 14: CAD drawings of Pool Dimensions.

At normal operating level, 180ft^3 of water will be stored in the wading pool. An additional 30ft^3 can be stored in the pool when the depth rises from its normal 4ft level to 4.5ft. The amount of extra water that can be stored above normal operating level

Depth (ft)	Invert Elev.	Surface Area (ft^2)	Volume (ft^3)	Cum. V. (ft^3)
0	99.75	60	30	210
0.5	99.25	54	27	180
1.0	98.75	54	27	153
1.5	98.25	48	24	126
2.0	97.75	48	24	102
2.5	97.25	42	21	78
3.0	96.75	42	21	57
3.5	96.25	36	18	36
4.0	95.75	36	18	18
4.5	95.25	36	0	0

Table 10: Volume variation in the wading pool

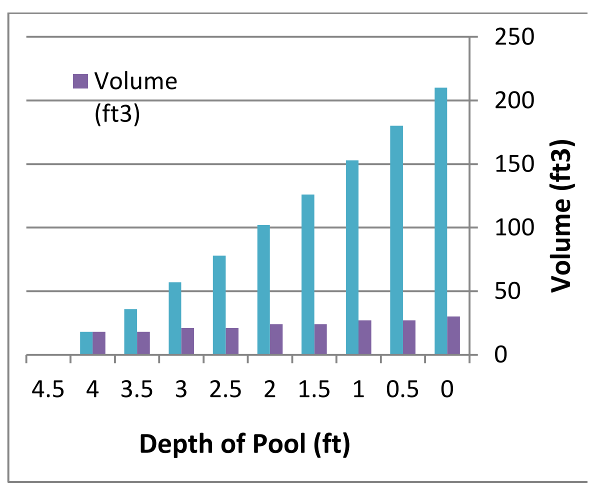


Figure 15: Volume variation in the wading pool by depth.

through the entire system (forebay, wetland, and pool) is described below in Equation 16.

A represents the area of the entire system, and h represents the maximum rise in water level.

$$\boldsymbol{Ah = V}$$ **Equation 16**

$$396ft^2 * .5ft = 198ft^3$$

The 198ft^3 of extra storage (forebay + wetland + pond) is not enough to store the entire second inch of rain in the system. One inch of rain produces a volume of 229ft^3 in the treatment system. A larger storage tank will be used to ensure the first two inches are captured.

3.4.5. Constructed Wetland Maintenance

The PA BMP manual provides the following guidelines for maintenance of the wetland:

> *"Constructed Wetland must have a maintenance plan and privately owned facilities should have an easement, deed restriction, or other legal measure to prevent neglect or removal. During the first growing season, vegetation should be inspected every 2 to 3 weeks. During the first 2 years, CWs should be inspected at least 4 times per year and after major storms (greater than 2 inches in 24 hours. Inspections should assess the vegetation, erosion, flow channelization, bank stability, inlet/outlet conditions, and sediment/debris accumulation. Problems should be corrected as soon as possible. Wetland and buffer vegetation may require support – watering, weeding, mulching, replanting, etc. – during the first 3 years. Undesirable species should be removed and desirable replacements planted if necessary.*
>
> *Once established, properly designed and installed Constructed Wetland should require little maintenance. They should be inspected at least semiannually and after major storms as well as rapid ice breakup. Vegetation should maintain at least 85 percent cover of the emergent vegetation zone. Annual harvesting of vegetation may increase the nutrient removal of CWs; it should generally be done in the summer so that there is adequate regrowth before winter. Care should be taken to minimize disturbance, especially of bottom sediments, during harvesting. The potential disturbance from harvesting may outweigh its benefits unless the CW receives a particularly high nutrient load or discharges to a nutrient sensitive waterbody. Sediment should be removed from the forebay before it occupies 50 percent of the forebay, typically every 3 to 7 years,"* (PA DEP, 2006).

In general, after plant life in the wetland is established, minimal maintenance is required. If a plant dies it should be replaced. Replacing plants before they die can increase pollutant removal, but agitating the sediment at the bottom of the wetland may release metals that have settled or sorbed into the soil.

3.5. Conveyance and Irrigation

3.5.1. Pump

A pump was selected to transport treated water to the farm. The sump pump was placed at the bottom of the cistern and connected to an outflow pipe. The outflow pipe serves to transport treated water vertically upwards and out of the ground and connect to the drip irrigation system currently in place. The irrigation system,

Aqua-Traxx® EAXxx0850, has the following requirements:

- Average Length: 2500 ft
- Maximum Length: 4000 ft
- Pressure Requirement:
  15-17 PSI
- Flow Requirement:
  0.69 gpm/100ft

Thus, the requirements for appropriate pump selection were calculated as follows:

Max Flow:

$$\frac{0.69\,GPM}{100\,ft\,hosing} \times \frac{4\,hoses}{bed} \times \frac{100\,ft}{hose} \times 10\,beds = \mathbf{27.6\,GPM}$$

Pressure:

$$Total\ Dynamic\ Head\ (TDH, ft) = Static\ Head + Pressure\ Head + Friction\ Head$$

Where Static Head is the physical elevation change, or:

$$9.75\,ft\,(below\ ground) + 2\,ft\,(elevation\ rise\ from\ system\ to\ farm)$$

Pressure Head is the desired pressure at the outlet, or:

$$15\,PSI \times \frac{2.33\,ft}{PSI} = 34.95\,Ft$$

And Friction Head is 0.8 ft (Engineering Toolbox, 2011). Therefore,

$$TDH = 11.75ft + 34.95ft + 0.8ft = \mathbf{47.5\,ft}$$

Figure 15 shows the pump curve for the Tsurumi LB-800 Sump Pump, which was selected for its ability to achieve approximately 27.5 GPM at 47.5 feet of head.

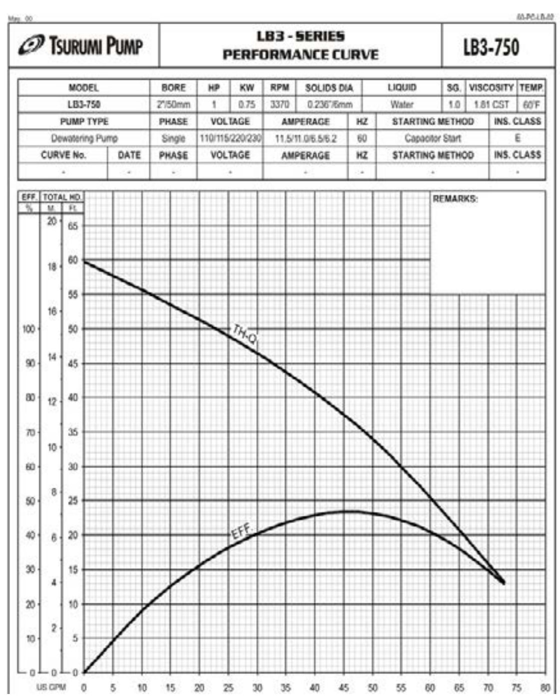


Figure 16: Pump Curve for Tsurumi LB-800 Sump Pump. 1 H.P. 1/220V with a 2" discharge

4. EVALUATION

In order to validate the performance claims of the water treatment system designed for this project, a plan for testing and verification of results must be developed. The Technology Acceptance Reciprocity Parternship (TARP Tier II) describes in detail the protocol to validating a stormwater BMP. However, physical construction of this BMP is outside the scope of this project. Thus, the tests described below are simply used to identify the contaminants entering the system for the sake of completeness. To evaluate our system design, we will use the EPA Storm Water Management Model (SWMM). SWMM will simulate the hydrology and pollutant removal capabilities of our design using rainfall data obtained from the Franklin Institute and NOAA. Specifications of both the water quality and SWMM tests are described in detail below.

4.1. Test Specification

The sections below describe the tests methods and procedures needed to test the quality of the water entering the designed system from the roof of the residence. The tests selected adhere to, at least, the minimum type of tests set by the TARP *Protocol for Stormwater BMP Demonstrations*. Tests beyond the minimum to be conducted provide a thorough spectrum of data and results to be analyzed to prove the system performance claims made throughout this document, to the best of our knowledge and sought after advice.

4.1.1. Rainwater Runoff Quality Testing

4.1.1.1 Total Suspended Solids – Standard Methods – Method 2540

The purpose of acquiring total suspended solids (TSS) data from the runoff off the roof of the house is to predict the concentration of TSS entering the forebay. After this test is completed the expected size of particles can also be determined. Determining the size of particles assists in calculating an appropriate detention time for runoff in the forebay.

This method was retrieved from the *Standards Method for the Examination of Water and Wastewater* organization and is to be used for Total Suspended Solids (TSS) testing.

PREPARATION

Before samples could be collected, containers had to be prepared to capture and transport samples. At least one liter per TSS test is recommended. Two one liter clear glass containers were thoroughly washed using DI water and cleansing solution. Then the glassware was filled with acid to further prepare the glassware for such sampling. Two were prepared in order to capture the first flush and another sample 15 to 20 minutes into the storm event we were able to catch.

Storm Event Date: Sunday, December 12, 2010
Date TSS Testing: Tuesday, December 21, 2010

EQUIPMENT AND MATERIALS

- Millpore AP40 Glass Fiber Filters for TCLP, USEPA Method 1311 (courtesy of Nicole Khan)
- Oven

- Scale
- Filtration device
- Magnetic Mixer

IMPLEMENTATION OF TSS TEST

First, glass fiber filters had to be prepared. They were hand cut to fit the piece they would lay on in the filtering device. Six filters were made, three for each sample as it is suggested to have triplicates in order to get a mean value. One-by-one the disks were placed in the filtering device and “washed” with reagent (DI water) with three successive 20mL portions of reagent. Vacuum was applied during this process and kept on after the washings until visible traces of water had been removed. Each disk was placed in its own, numbered, inert aluminum weighing dish for support and protection. After the washing process each dish and disk was weighed together then put in the oven at 105 C for an hour then desiccated until constant weight was found. Then, for water testing, the samples were magnetically mixed to keep the sample homogeneous while pipetting a measured volume onto the prepared filter in the filtration device. The pipetted volumes are recorded for each filter. After each filter had enough rainwater passed through it to visibly alter the color of the white filter it was then rinsed with four 10mL volumes of reagent with vacuum applied and left with suction for three minutes after last rinse to assure great enough water removal by vacuum. Then the samples were weighed in their respective trays and put in the oven for at least an hour at 105 C and then desiccated and weighed again until constant weight was reached.

RAW DATA

	Filter No.	Filter Weight after Oven + Desiccator, mg (B)		Weight after filtering, mg	Weight after Drying + Cooling, mg (A)	Rain Sample Volume, mL	mg TSS/L
Sample I	1	1748		1768	1768	350	57.14286
	2	1841		1861	1861	400	50
	3	1864		1866	1866	400	5
Sample II	4	1867		1871	1869	500	4
	5	1840		1844	1845	500	10
	6	1704		1706	1706	500	4

Table 11: Raw Data from First Runoff Collection

CALCULATIONS

mg/L of TSS = [(A – B)*1000mL/L]/(mL of Sample Volume Passed)

4.2. Stormwater Management Model (SWMM)

The EPA Storm Water Management Model (SWMM) Software, Version 5.0, was used to test the system. This program, developed specifically for modeling stormwater runoff, is capable of analyzing both the hydrology and water quality of our system.

SWMM allows users to identify subcatchment areas where rainwater and pollutants will run off. Pipes, channels, and pumps for conveyance may be modeled. Treatment and storage units where water is contained and treated may also be identified. The Rain Gage feature allows the user to input variable rainfall data so that the model may be viewed for either a short or long duration. This was important given we wanted to test the same system for the 2-year, 24-hour storm and also with 30 years of rainfall data.

Figure 16 shows a screenshot of the system as it was laid out by GHV Engineers in SWMM. The five subcatchment areas displayed in Figure 16 represent the portions of the roof, forebay, wetland, and pond that capture rainfall. Storage units were also included for the forebay, wetland, and pond in addition to the actual storage cistern. These serve to describe the volume distributions and pollutant removal qualities of each component. The reason for depicting the treatment areas as both subcatchments and storage units is to “tell” the model that the treatment areas are not only storing and treating runoff, but actually capturing rainfall directly on their open surface areas. This is a necessary adjustment because inputs for subcatchments do not include pollutant removal, and storage areas are assumed by the software to be closed units. Outfalls were included in two places: one where the overflow will outfall into the street, and one where the irrigation system on the farm begins. Conduits include the gutters and downspouts of the house, pipes connecting downspouts to the forebay, the open channels between components of the treatment area and storage unit and pipes to the pump and outfalls. The inputs for the various elements of the system are shown in APPENDIX E. Once the model was built, a simulation of the two-year, 24-hour storm was run and a status report of the simulation was obtained. From the status report, we sought to ensure that the system would function the way it was designed. This is explained in detail in Section 4.2.

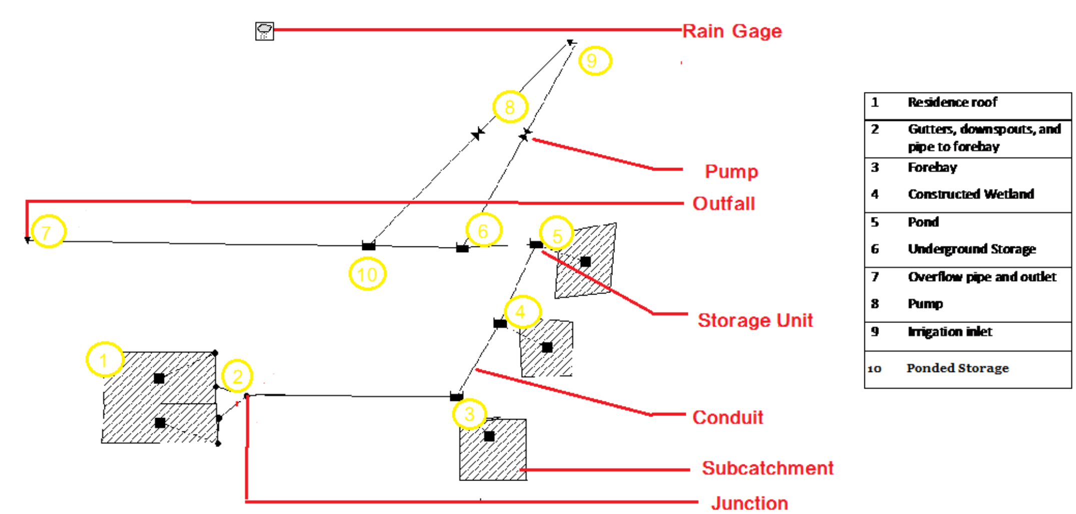


Figure 17: Drawing of Treatment System as It Appears in SWMM.

4.2.1. Test Certification – 2-year, 24-hour storm

The hydrology status report as given for the two-year, 24-hour storm event by SWMM is provided in APPENDIX F. The Node Depth Summary was examined first to ensure that the hydrology through each node—that is, the pipe junctions, treatment areas, and storage cistern—were performing as our design intended. Maximum depths in the cistern, forebay, wetland, and pond are recorded as 9.3, 5.01, 3.01, and 4.01 respectively. These accurately describe the depth from the bottom of each component to its normal operating level. Maximum HGL are given as 99.3, 99.2, 99.26, and 99.26 respectively. Keeping in mind that a ground elevation of 100 was used, these numbers illustrate that water does not rise past the normal operating level of each component, which indicates no overflow will occur. According to our design, water should begin to fill the cistern once the forebay, wetland, and pond reach their normal operating levels of 9 inches below ground level. From the time lapse of profile graphs in Figure 17, we can see that this is in fact occurring. However, with an inlet pipe at this same depth and a higher outlet pipe, the cistern should allow water to begin ponding in the treatment area and fill it to maximum levels (slightly below ground level) before it begins releasing water to the outfall. Clearly, since the report states that maximum levels do not surpass the normal operating levels, this does not seem to be occurring. It appears that the cistern fills to its capacity and begins outflow while keeping the treatment area at its normal operating level. This is not only an incorrect portrayal of the system's function, but also seemingly impossible. The only apparent remedy to this glitch was to add a fifth, "imaginary", storage unit (U6) which models the storage of the second inch of water through ponding on the treatment area from (-9) inches to ground level. With the addition of U6, the system appears to function as it was intended, with the cistern filling up (Time of Max Occurrence) after about 9 hours and overflow to street beginning after about 15 hours. Furthermore, no nodes were surcharged or flooded; the storage volume of each component follows the design volumes, and no conduits were surcharged.

Flow Routing Continuity	Volume acre-feet	Volume 10^6 gal
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.015	0.005
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	0.006	0.002
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.016	0.005
Final Stored Volume	0.026	0.008
Continuity Error (%)	0.000	

Figure 18: Inflows and Outflows of the Treatment System During the 2-year, 24-hour Storm Event

Figure 17 shows that our projected values for amount stored in the wetland match up with the values we calculated in Table 8.

The peak rate flow from the property was increased by the addition of the system. This is due to the increased impervious area created by installing the system. However, there is a significant delay from the time rainfall begins to the occurrence of runoff out from the system, as shown in Figure 3. The graph in Figure 3 illustrates an increase in flow rate out of the system from about 3.2 to 3.7 gpm, however the outflow does not occur until nearly 15 hours after the start of the storm.

Figure 4 illustrates the depths of each component of the system during the 2-year, 24-hour storm. As expected, the depths of the forebay, wetland, and pond stayed stable. The cistern fills first, then ponding occurs, with overflow finally occurring at around the 16 hour mark. No overflow occurs during the 2-year, 24-hour storm as noted in the status report in APPENDIX F.

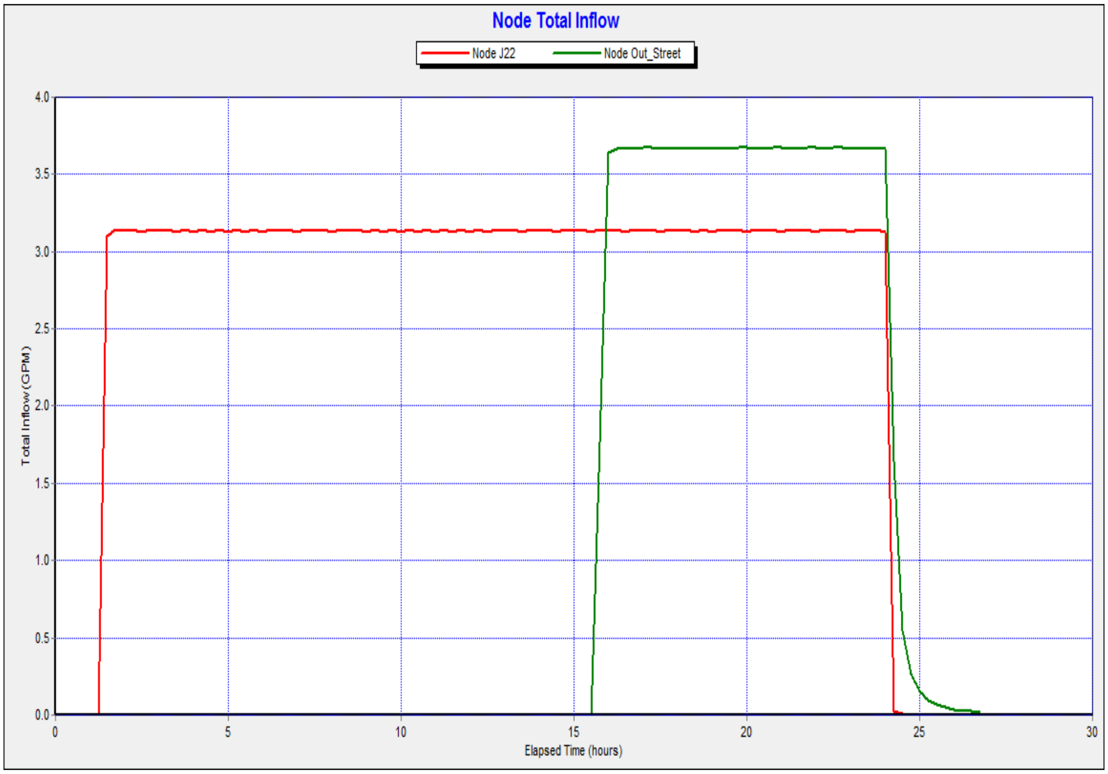


Figure 20: Peak Rate Flow for the 2-year, 24-hour Storm.

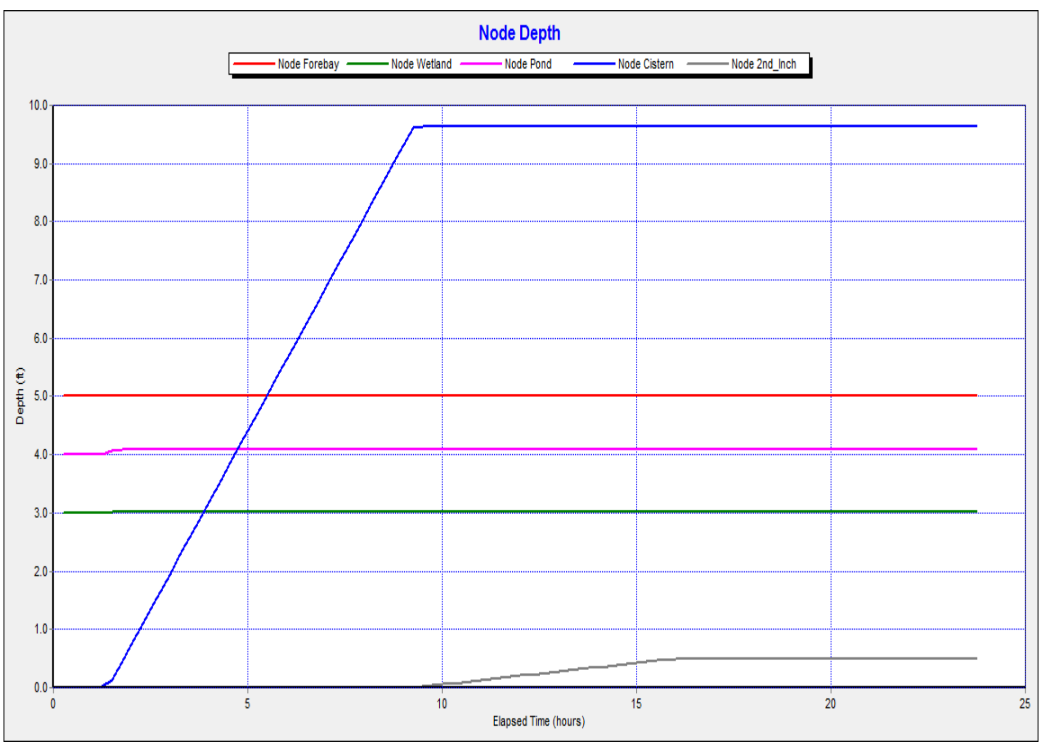
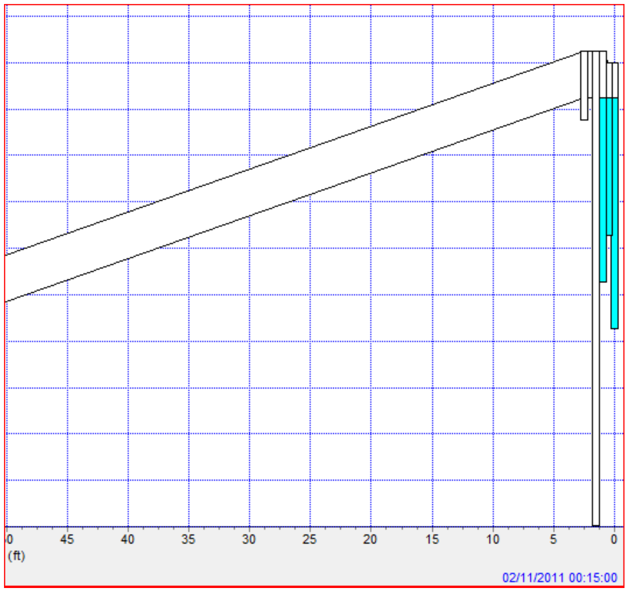
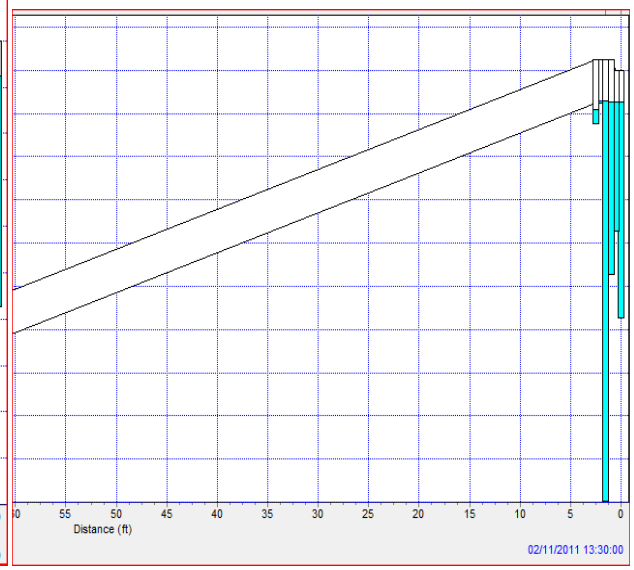


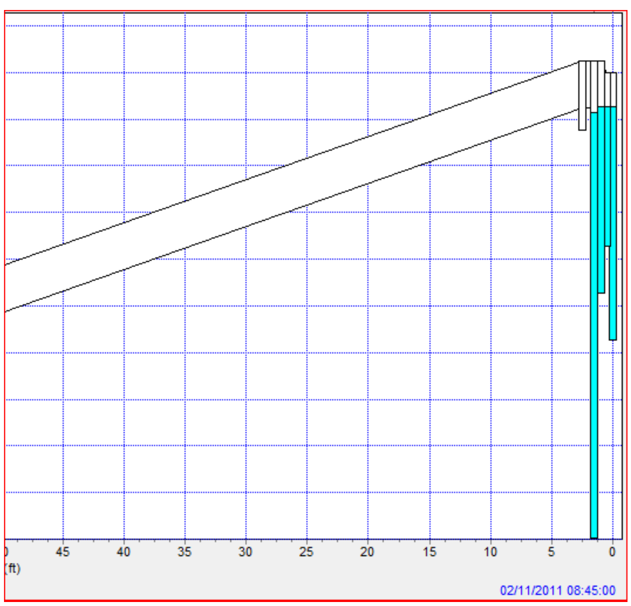
Figure 19: Depth Variance in the system During the 2-year, 24-hour storm. The cistern is denoted by the blue line, The gray is the ponding occurring in the treatment area.



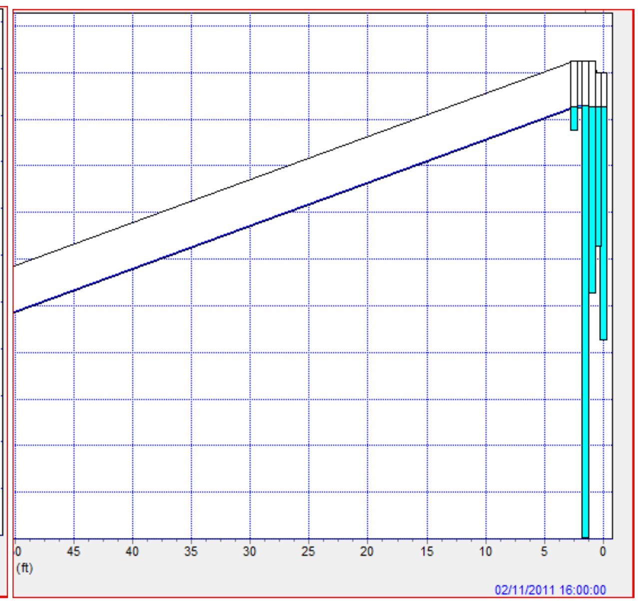
(a) System after 15 minutes; Cistern is empty



(c) System after 13h30m. Ponding has begun



(b) System after 8h45m; Cistern is full



(d) System after 16 hours; Outfall to street has begun

Figure 21 (a-d): Profile graph of flow through treatment area throughout the two-year, 24-hour storm event

4.2.2. Test Certification - Long Term (30 year) Behavior of System

In order to test how the system performed under regular day to day conditions, we imported 30 years of rainfall data from the National Oceanic and Atmospheric Administration (NOAA). Given that the water will be used for irrigation, we needed to develop rules governing when water would be pumped to the farm. Our client informed us that usually watering begins 3 days after a rain event, and roughly 500 gallons of water per day are used. Due to SWMM's complex pump control rules input, we manually added every day over the course of 30 years where it hadn't rained in 3 days. Two pumps were necessary in the model, one to pump out the ponded area first, the second to pump out the cistern.

```
OR SIMULATION DATE = 10/20/2001
OR SIMULATION DATE = 10/21/2001
OR SIMULATION DATE = 10/26/2001
OR SIMULATION DATE = 10/27/2001
OR SIMULATION DATE = 10/31/2001
AND SIMULATION CLOCKTIME < 00:18:00
AND NODE 2ND_INCH DEPTH > 0
THEN OUTLET OU1 SETTING = 1
ELSE OUTLET OU1 SETTING = 0

OR SIMULATION DATE = 10/20/2001
OR SIMULATION DATE = 10/21/2001
OR SIMULATION DATE = 10/26/2001
OR SIMULATION DATE = 10/27/2001
OR SIMULATION DATE = 10/31/2001
AND SIMULATION CLOCKTIME < 00:18:00
AND NODE 2ND_INCH DEPTH = 0
THEN OUTLET OU2 SETTING = 1
ELSE OUTLET OU2 SETTING = 0
```

Figure 23: Pieces of Pump Control Rules: Rule 1 tells SWMM when to pump out the ponded storage. Rule 2 shows when to pump out the cistern. Note that the pump controlling the cistern is dependent upon the ponded storage being empty before turning on.

;year	;Time	;Daily Precipitation, in
1/01/1971	0:00	0.47
1/02/1971	0:00	0
1/03/1971	0:00	0
1/04/1971	0:00	0.83
1/05/1971	0:00	0.26
1/06/1971	0:00	0
1/07/1971	0:00	0
1/08/1971	0:00	0
1/09/1971	0:00	0
1/10/1971	0:00	0
1/11/1971	0:00	0
1/12/1971	0:00	0
1/13/1971	0:00	0.06
1/14/1971	0:00	0.11

Figure 24: Sample of Rainfall Data Input

****
Outfall Loading Summary
****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow GPM	Max. Flow GPM	Total Volume 10^6 gal
Out_Street	0.03	7.79	68.77	1.368
Out1	0.00	27.40	27.60	0.502
System	0.01	35.18	68.77	1.871

Figure 22: Outflow to Street and to Farm

Figure 6 shows that nearly 27% of all runoff is infiltrated on the farm. Over 30 years, the amount of flow into Philadelphia's sewer system is reduced 500,000 gallons.

It is important to consider the depth variance with time over the course of the life of the system. If water level remains 6'' higher or lower than normal for an extended period of time, it could negatively impact plant life in the wetland. Figure 9 shows depth variance over 10 years of the system's life. Note that the gray line representing depth of the ponded area remains at its full capacity of 6'' for long periods of time. Because this could negatively impact plant life, a new planting plan was developed to account for this unexpected behavior.

Figure 10 shows depth change over the course of one year. It is evident that the ponded area is drained before the cistern when the pump is active, as intended. Water is not used for irrigation in the winter months, therefore the water level of the ponded area will rise and stay at maximum capacity during that time. Notice in the summer months, evaporation affects depth in the wetland. This evaporation must be considered when determining whether or not the wetland will need to be watered in dry months.

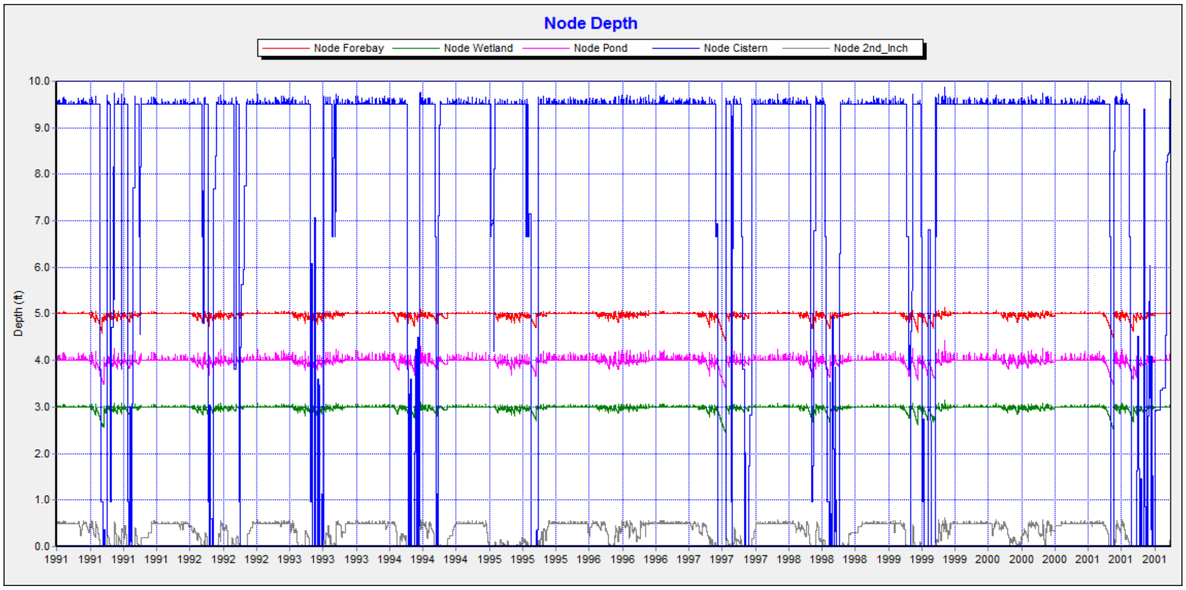


Figure 25: SWMM Output for Depths of Each Treatment Component Over 10 Years: Blue represents the cistern depth, gray the ponded storage.

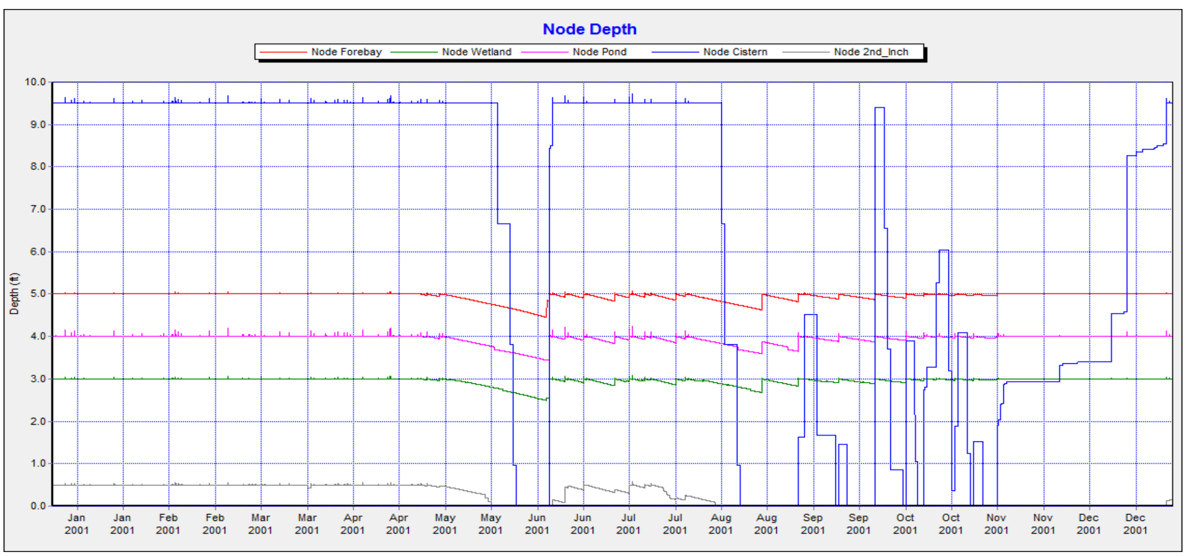


Figure 26: Depth Variance Throughout One Year

Evaporation is not a major concern about the functionality of the wetland, as shown in Figure 11. The depth in the wetland must remain above 2.5ft for plants to remain healthy. As the graph shows, approximately 99% of the time the depth is above the 2.5ft required. Evaporation should not impact the system design in any way.

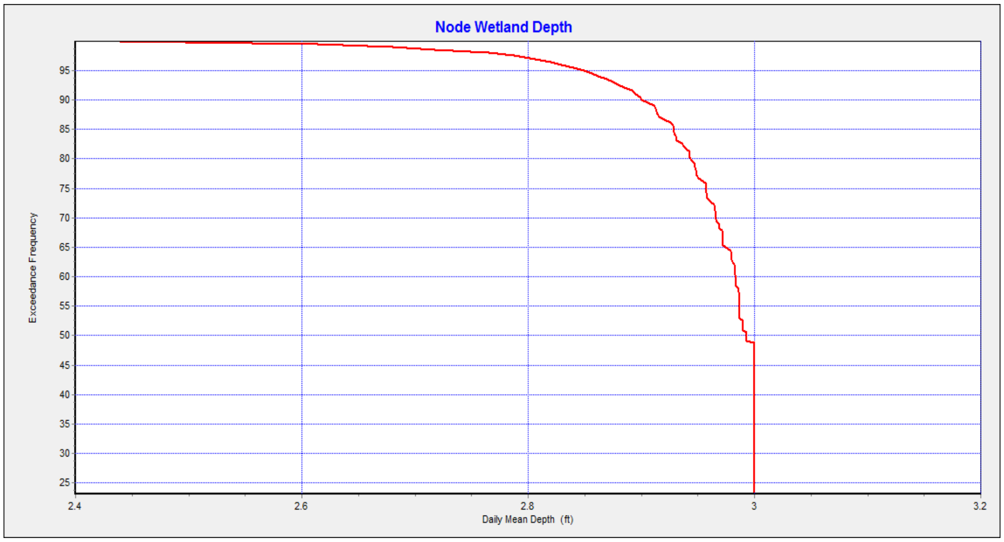


Figure 27: Percentage of Time Wetland Depth is At Least 2.5ft.

In summary, the only part of the system design which behaved differently than expected was the water level. Correcting the planting plan was necessary however, it was the only design component changed after testing the system.

4.3 Test Certification – Water Quality Simulation

EPA SWMM will be used to verify the pollutant removal qualities of the system. This will be done by specifying pollutant concentrations expected from rooftop runoff. Then, we will input functions for pollutant removal into each storage area. These functions are based on the pollutant removal efficiencies found in literature.

In order for SWMM to simulate how pollutants are removed through the system as runoff occurs, pollutant build up and wash off functions were input for the catchment surface. Treatment functions were included for the treatment components of the system.

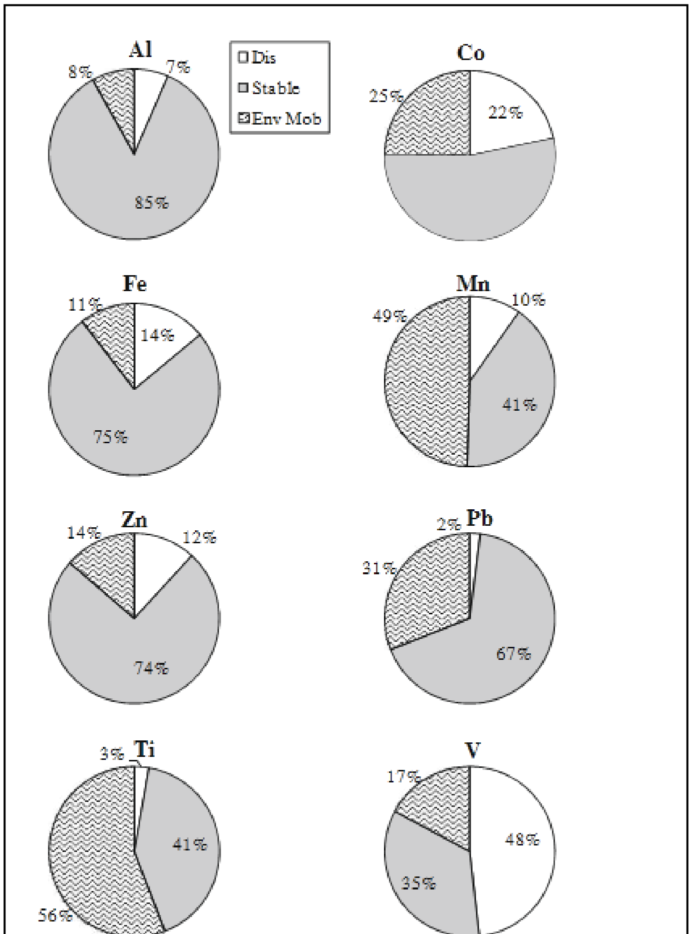


Figure 28: Percent of dissolved, environmentally mobile, and stable fractions of selected metals in run off, *Characteristics and environmental mobility of trace elements in urban runoff.*

Research indicated exponential functions would best simulate build up, wash-off and treatment in the real world (Gironás, J, 2009). When pollutants collect on surfaces exposed to the atmosphere, they do so somewhat rapidly for a time and then taper off, reaching a maximum build limit.

Included in these calculations were co-fraction values. Zinc and Lead are found both dissolved in the runoff and attached to TSS particles. Through research, the fractions of these heavy metals attached to TSS was determined. Furthermore, a great deal of metals removal happens as total suspended solids are settled out. This occurs because the particulate portion of metals is believed to adhere (or “aggregate”) to dust and dirt build up on a catchment during an event that creates runoff (Joshi, 2010). Figure 27 shows percentages of the states of a metal in runoff of an urban area. Zinc’s environmentally mobile value was the only value taken from this figure.

To represent the above, a metal can be named as a co-pollutant and its value is the entered as a co-fraction. Lead was named as 0.25 co-fraction of co-pollutant TSS in μg/L, TSS is in mg/L (Gironás, J, 2009). Zinc was given the co-fraction value of 0.14 of TSS (Joshi, 2010). These co-fractional values were obtained from literature reporting on urban pollutant buildups. These values are not site-specific as we were unable to collect and test water quality properly and in the time frame of our project. It is very strongly recommended that water quality testing be properly conducted before the physical system is installed so adjustments to plant life or detention times can be made. Water quality testing should also be checked after the system is in place in order to verify how efficient treatment is occuring. Testing the water quality of the real system (actual tangible system) is beyond the scope of this projects time-frame.

There are several processes that occur over the system that need to be mathematically expressed in SWMM in order to simulate how these pollutants move through the design. These expressions will then show how the system will treat the runoff. The processes are pollutant buildup, washoff and treatment. First, how the pollutants would "build up" on the catchment area (the roof) must be defined. An exponential expression was used for all of these processes because these rates decrease with time (U.S. EPA, 2009). Therefore, it is not be reasonable to use a linear relationship.

The exponential function used in SWMM is as follows:

$$B = C_1(1 - e^{-C_2 t})$$ **Equation 10**

Here B is the buildup rate as a function of time, t, C_1 is maximum build-up and C_2 is the buildup rate constant. C_1 and C_2 were taken to be typical values reported by the SWMM Applications Manual for an urban residential property, 0.13 (lb/curb*ft) and 0.5 (1/day), respectively.

Next, follows the exponential equation for wash-off:

$$W = C_1 q^{C_2} B$$ **Equation 11**

Where W is the wash-off rate, determined by the buildup rate on the catchment. A new C_1 and C_2, wash-off exponent and wash-off coefficient are defined here and q is the run off rate per unit area at time t, determined internally by SWMM. Typical urban values found in the SWMM Applications Manual for C_1 and C_2 are 40 and 2.2 respectively. The C_2 value was determined by the sediment transport theory as reported in the SWMM applications manual. The more difficult value to determine is C_1 as it can vary in order of magnitude from location to location. However, it has been reported that in urban areas the order of magnitude varies much less (U.S. EPA, 2009).

Finally, the expression for treatment must be defined. The forebay and wetland were named for treatment in the model. Sedimentation occurs in the forebay and, to a lesser extent, in the wetland.A BMP efficiency is named for the wetland because the primary function of the wetland is to take up pollutants via vegetation. The equation used is as follows:

$$C_{t+\Delta T} = C^* + (C_t - C^*)e^{-(k/d)\Delta t}$$ **Equation 12**

The concentration remaining after treatment is $C_{t+\Delta T}$. C* is present as it is assumed there is some amount of TSS that is non-settleable (Gironas, 2009). k is determined from design specifications and d is the water depth. C* is assumed to be 20mg/L for the purposes of this model as per the SWMM applications manual.

$$k = \frac{\bar{d}\ln(1 - removal\_efficiency)}{hours}$$

Here $\bar{d}$ is some representative value of depth during an event in a particular treatment component (U.S. EPA, 2009). For the forebay, the representative depth is 6.52 inches or 0.54 feet. We call this representative because during a rain event this is the depth on top of the forebay that will be flowing from forebay to wetland. The time component input was the calculated detention time of the forebay, 4.81 hours. The removal efficiency was input as 70% (0.70 in the equation). The k value for the wetland was determined to be about 0.14. The k value for the wetland was found to be 0.11using the designed detention time of 9.42 hours and the same representative depth as the forebay.

Results

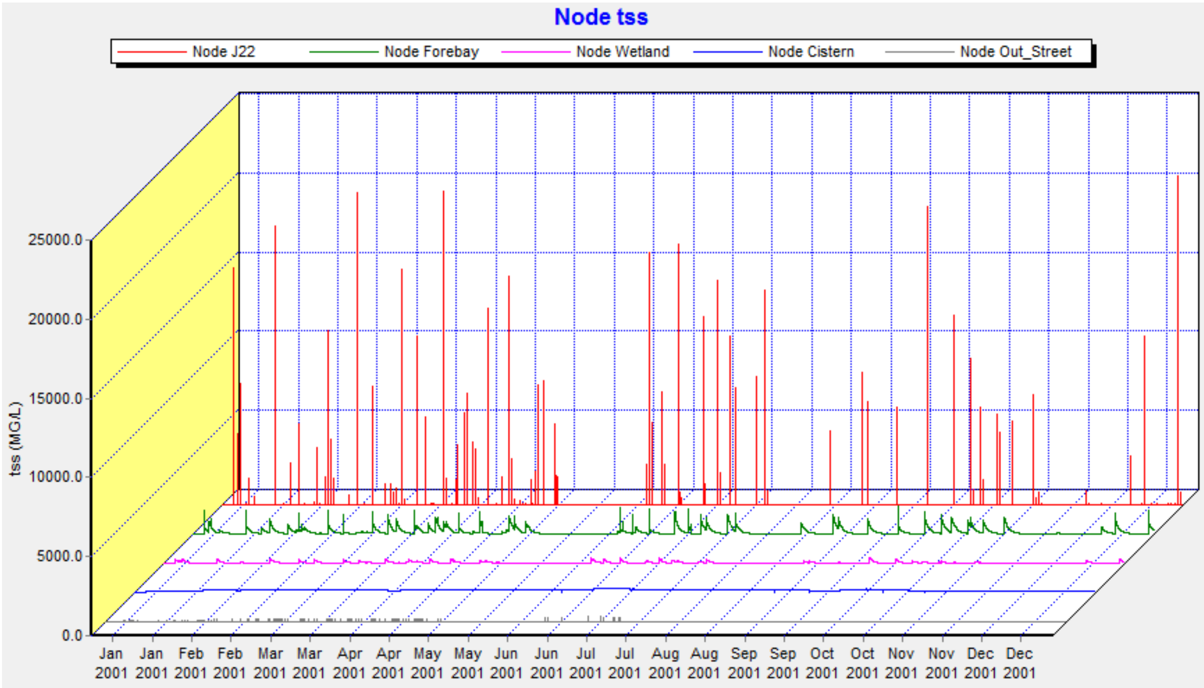


Figure 29: SWMM graphical nodal pollutant load comparison of data for 2001 from using rain data from the Franklin Institute. Shows decrease in pollutant loading entering main channel and exiting system.

Figure 28 is a representation of the decrease in pollutant loading through the designed system. Selected were the data for the year 2001. This was the most recent data available from our 30 year data collection, which had also been used to determine the rainfall behavior during a typical year in this region. The components meant for settling and treatment were graphed with the inlet and outlet nodes of the system as a reference point to show that TSS had decreased from the roof through the system. The red line in the back shows the pollutant loading in the junction, at the beginning of the main channel, of the roof's downspouts that were routed underground. The blue line in the front shows the loading to the street when outfall occurs. The green (second from back) shows that of the inlet of the forebay and the pink (second from front) for the wetland.

It is visually obvious that the pollutant levels decrease as rainwater moves through the system from roof to outfall (outfall is if "overflow" to street occurs). Therefore, we would expect the system to act accordingly when physically integrated on site. There is reason to believe that we have over-estimated values in relation to the real behavior of pollutant build up this site should see. Therefore, the real system when tested is not expected to exceed values we report through SWWM. Furthermore, Figure 29 shows the pollutant loading through the system during the 2year-24 hour storm.

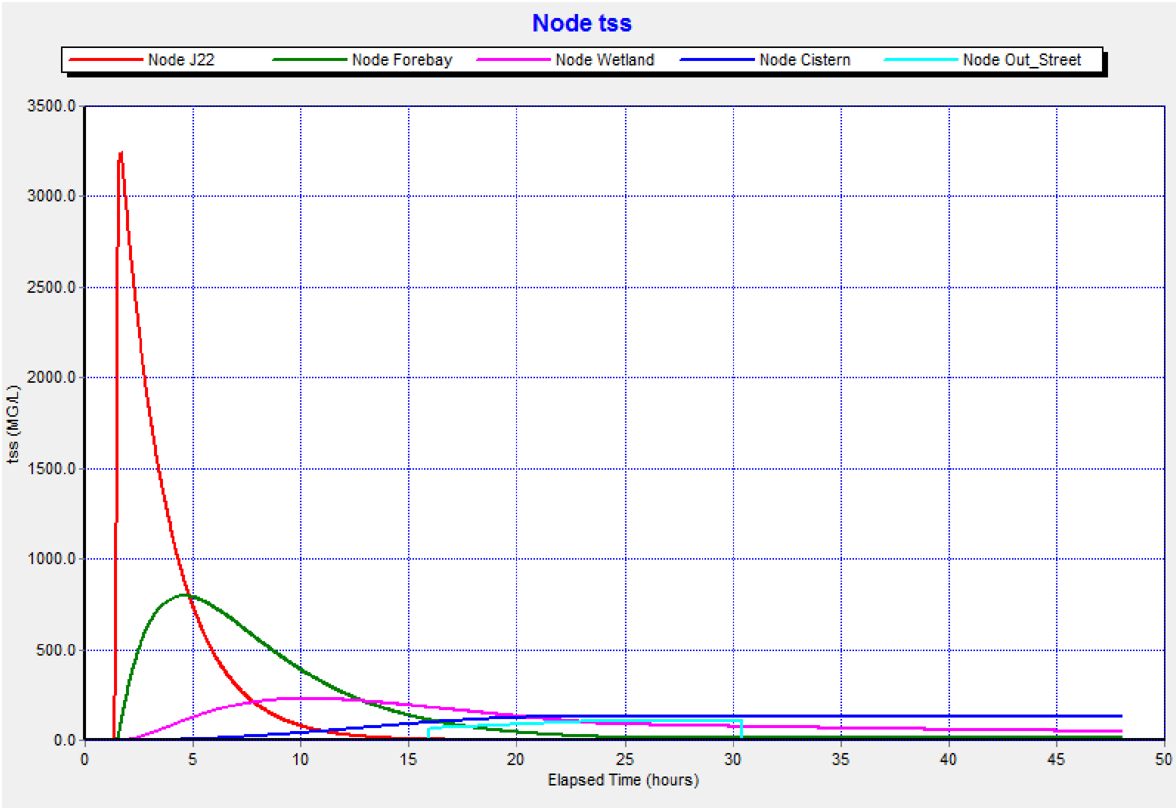


Figure 30: 2year-24 hour pollutant loading through the main channel junction, forebay, wetland, cistern and street.

From the SWMM report we can verify the removal of pollutants through the system.

```
***********************
Outfall Loading Summary
***********************
```

Outfall Node	Flow Freq. Pcnt.	Avg. Flow GPM	Max. Flow GPM	Total Volume 10^6 gal	Total zn lbs	Total pb lbs	Total tss lbs
Out_Street	0.96	8.46	191.85	1.368	1.689	3.017	1915.240
Out1	0.11	27.49	27.60	0.502	0.750	1.340	880.943
System	0.53	35.95	191.85	1.871	2.440	4.357	2796.182

Figure 31: SWMM 5.0 system status results for pollutants reaching the street (above) and the cistern (below) over the course of 30 years.

Figure 30 shows that SWMM reports 1.689lb and 3.017lb of Zinc and Lead, respectively, to the street which flows to the sewers. Also, 0.750lb and 1.340lb of Zinc and Lead, respectively, that will be in the cistern that is pumped to the farm. These correspond to more understandable values of 0.146mg/L and 0.0.264mg/L of Zinc and Lead to the street and 0.0657mg/L and 0.117mg/L of Zinc and Lead in the irrigation water. The values were converted to mg/L of the pollutants by converting pounds (1lb = 453.59g, about and 1g = 1000mg) to milligrams, dividing by the "Total Volume in 10^6 gal" and then multiplying by the conversion of gallons to liters (1gal = 3.785L). These values are well below those recommended by the U.S. EPA for reclaimed water and almost meet those for freshwater. These values can be compared in Table 12 below.

	EPA Irrigation Standard	EPA Freshwater Standard	Irrigation Water from System, SWMM 5.0	Water to Street, SWMM 5.0
Zinc	2.0 mg/L	.05 mg/L	0.0657mg/L	0.146mg/L
Lead	5 mg/L	.025 mg/L	0.117mg/L	0.264mg/L

Table 12: Comparison of EPA standards of irrigation and freshwater against the SWWM system report

Figure 31and Figure 32 below show the SWMM results give an overall picture of pollutant build up and inflow and outflow of the system over 30 years. The inflow and outflow results are especially important because they show that external outflow of pollutants was greatly decreased by the treatment system.

```
**************************          zn          pb         tss
Runoff Quality Continuity          lbs         lbs         lbs
**************************  ----------  ----------  ----------
Initial Buildup ..........       0.000       0.000      12.353
Surface Buildup ..........       2.529       4.516   18056.997
Wet Deposition ...........       0.000       0.000       0.000
Sweeping Removal .........       0.000       0.000       0.000
Infiltration Loss ........       0.000       0.000       0.000
BMP Removal ..............       0.000       0.000       0.000
Surface Runoff ...........       2.529       4.515   18061.496
Remaining Buildup ........       0.000       0.000       7.349
Continuity Error (%) .....       0.000       0.000       0.003
```

Figure 32: SWMM 5.0 Status report of pollutant buildup on catchment area (roof) for 30-year rainfall data collected from Franklin Institute.

```
**************************          zn          pb         tss
Quality Routing Continuity         lbs         lbs         lbs
**************************  ----------  ----------  ----------
Dry Weather Inflow .......       0.000       0.000       0.000
Wet Weather Inflow .......       2.335       4.169   16676.478
Groundwater Inflow .......       0.000       0.000       0.000
RDII Inflow ..............       0.000       0.000       0.000
External Inflow ..........       0.000       0.000       0.000
Internal Flooding ........       0.000       0.000       0.000
External Outflow .........       2.502       4.468    2863.708
Mass Reacted .............       0.000       0.000   15087.451
Initial Stored Mass ......       0.000       0.000       0.000
Final Stored Mass ........       0.018       0.032       7.185
Continuity Error (%) .....      -7.929      -7.929      -7.687
```

Figure 33: SWMM 5.0 Status report of runoff quality for 30-year rainfall data collected from Franklin Institute.

The SWMM pollutant modeling presented can represent real-world removal, because it shows that through each treatment component there is a reduction in the pollutant concentrations on the bases of our design inputs. However, these pollutant values are very theoretical due to the insufficient amount of water quality testing results acquired. We are aware that there may be an additional amount of metals in the water due to dissolved quantities. So, we were only able to speculate on these amounts found in research. Even though it is known that the plants selected readily remove metals from water. Therefore, it is again sternly advised that water quality testing from the roof be done by a reputable laboratory to gain insight into site-specific pollutants the designed system will be expected to have to treat for. The final design can be further altered depending on the results and desires of the client.

4.3. Cost-Benefit Analysis

4.3.1. Initial Costs

Component	Unit Cost	Total Cost
Gutters		
Gutters	$2/ft	$428
Conveyance		
Hand Excavation	$24.12/yd³	$25.08
PVC Piping	$12.77/ft	$638.50
Treatment Area		
Excavation	$10.67/yd³ + equipment	$405 + equipment
Concrete	$70/yd³	$868.5
Bentonite Liner	$15/25 ft²	$195
Plants:		
14 Water Lilly	$1.41	$19.74
27 Pondweed	$1.38	$37.26
13 cattail	$1.38	$17.94
7 soft rush	$1.38	$9.66
6 bulrush	$1.38	$8.28
5 Reedgrass	$0.80	$4.00
6 Sedge	$1.41	$8.46
7 Irongrass	$1.09	$7.63
24 switch grass	$1.09	$26.16
	Total:	$146.54
Irrigation		
Storage Tank	$864.99/tank	$865
Tsurumi LB-800 Sump Pump	$430/pump	$430
	Total:	$3570

Table 13: Initial Costs of Building Constructed Wetland

Table 11 shows the estimated costs of installation of the system. The total estimated cost is about $3570. In addition to upfront costs, maintenance and usage costs and savings from reduced water usage were calculated:

- Electricity needed to run pump (See Pump section of the approach for estimation of pump usage time) :

$$\frac{\$0.0254}{kwh} \times 0.75\,kw \times 18\frac{min}{day} \times \frac{1\,hr}{60min} \times 37\frac{days}{year} = \frac{\$0.21}{year}$$

- Maintenance, based on dredging once every 5 years as suggested by the PA BMP manual (PA DEP, 2006) :
  - Dredging: $Backhoe\ rental\ (2hrs) = \$177\ \div 5\ years = \frac{\$35.4}{year}$
- Savings (Based on water used in the irrigation pipe over 31 years – see Figure 21:

$$V_{out\ 1} = 0.502E^6\ \div 31\ years = 16{,}000\frac{gal}{year} = 2000\frac{cf}{year} = 20\frac{ccf}{year} \times \frac{\$5.12}{ccf} = \frac{\$100}{year}$$

It would take over 50 years to make back the initial investment into the pond, not including how many plants will die and need replacement. Though sustainable and green, the cost of this project is prohibitive. If the farm were larger, allowing more water usage, the time until return on investment would be much smaller. Though not economically feasible for this specific project, there is no reason to believe that such a system would not yield a decent return for a larger farm.

$$\frac{\$3570}{\frac{\$100}{yr} - \frac{\$35.40}{yr} - \frac{\$0.19}{yr}} = 54.4\ years$$

5. SUMMARY AND FUTURE WORK

During the course of Senior Design, GHV Engineers was able to produce a design document detailing the benefits of a constructed wetland used to treat water for irrigation on an urban farm. Such a system is capable of removing a sizable percentage of water and pollutants from loading the city's sewer system, while reducing dependence on city water for irrigation. Ultimately, we expect the owners of the urban farm will employ a similar system to the one designed, likely with more of a focus on the pool.

Everyone working on an assignment did not work very well. Having different people specialize in different areas tended to produce better results than team member having a hand in each assignment. Editing other team members work tended to produce better results than editing one's own work. It is easy to miss errors in something you wrote on your own, so having a different person proofread a section produced more helpful comments and edits. For example, having two group members working on the SWMM model was difficult in that certain things were input differently and two separate files were created. Ultimately, one group member was assigned the hydrological aspect of modeling the system, and another group member worked on the water quality aspect of the model. This approach ensured that we had one team expert for each aspect of the project, but all team members were quite knowledgeable about the entire project.

Putting assignments together the night before they are due did not work out well in Senior Design. Giving a longer period of time to look everything over to ensure there were no formatting errors and everything was cross-referenced properly was extremely helpful. It also considerably lowered the amount of sleep lost by the team.

If we were to do the project over, I feel more attention would be paid to the approach in the beginning of the year. A solid approach results in a very strong project overall. If the way you are going to go about solving a problem is developed first, it makes the whole process much easier. We solely focused on the assignment that was due that week in SDI, but in SDII we paid much more attention to the overarching goals. In SDII, we were able to work much more efficiently, and this was reflected in the quality of our work. We had clear goals and our own deadlines to meet and these were very helpful in making regular progress on the project.

Specific to this project, things we could have done to optimize the design would include additional storage space to allow more savings through irrigation. Instead of a pipe flowing to the street, a vegetated swale could infiltrate additional water to reduce the load of the property on the sewer system even further.

It is unfortunate that the amount of savings generated by the system is not adequate to make the constructed wetland system a viable economic investment for this particular property. Low Impact Development systems such as the one designed are highly beneficial, but not cost effective on such a small scale.

6. ACKNOWLEDGEMENTS

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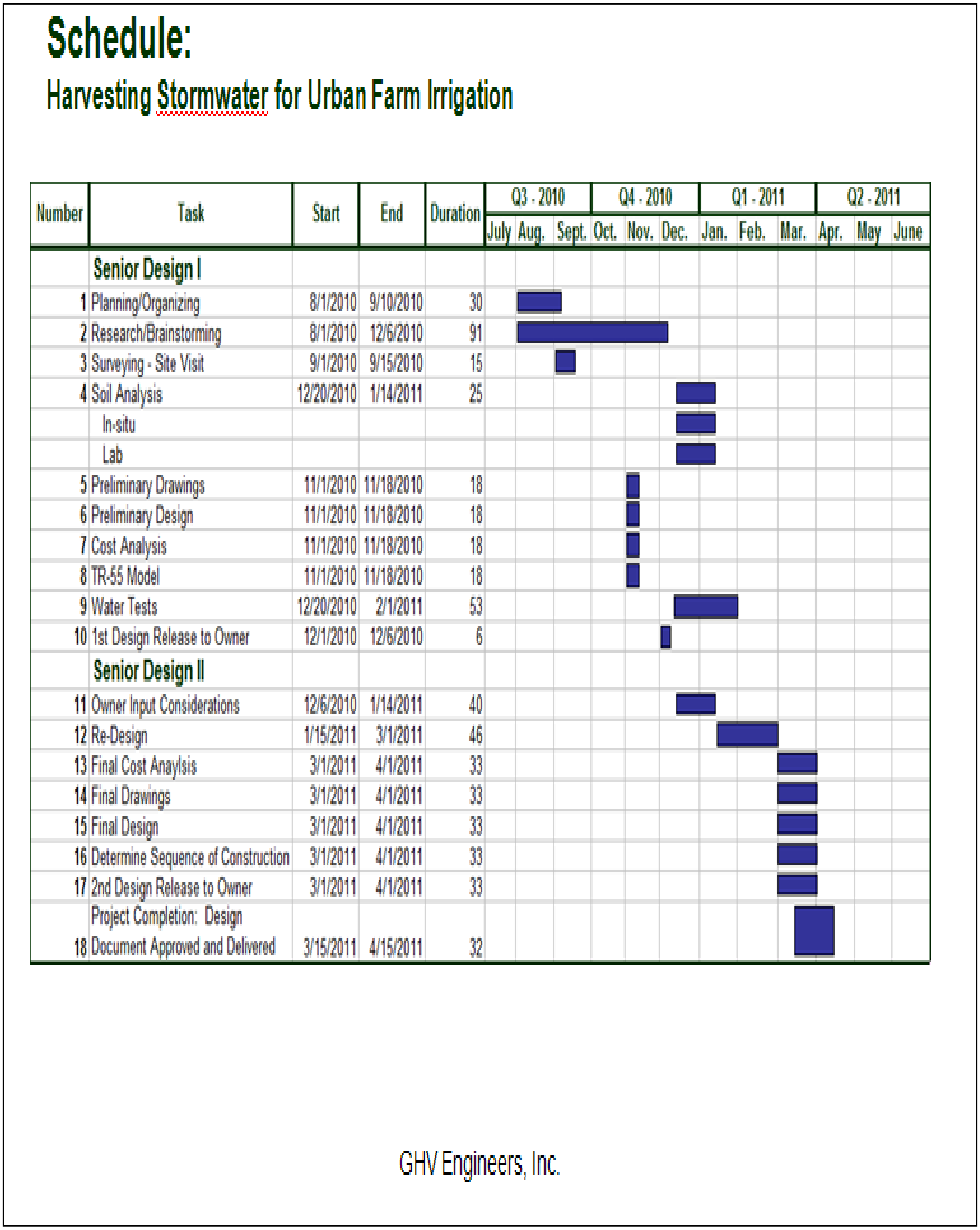
APPENDIX A: WETLAND SPECIFICATIONS (PA BMP MANUAL)

Specifications:

The following specifications are provided for information purposes only. These specifications include information on acceptable materials for typical applications, but are by no means exclusive or limiting.

1. **Excavation**
   a. The area to be used for the CWs should be excavated to the required depth below the desired bottom elevation to accommodate any required impermeable liner, organic matter, and/or planting soil.
   b. The compaction of the subgrade and/or the installation of any impermeable liners will follow immediately.
2. **Subsoil Preparation**
   a. Subsoil shall be free from hard clods, stiff clay, hardpan, ashes, slag, construction debris, petroleum hydrocarbons, or other undesirable material. Subsoil must not be delivered in a frozen or muddy state.
   b. Scarify the subsoil to a depth of 8 to 10 inches with a disk, rototiller, or similar equipment.
   c. Roll the subsoil under optimum moisture conditions to a dense seal layer with four to six passes of a sheepsfoot roller or equivalent. The compacted seal layer shall be at least 8 inches thick.
3. **Impermeable Liner**
   a. If necessary, install impermeable liner in accordance with manufacturer's guidelines.
   b. Place a minimum 12 inches of subsoil on top of impermeable liner in addition to planting soil.
4. **Planting Soil (Topsoil)**
   a. See Local Specifications for general Planting Soil requirements.
   b. Use a minimum of 12 inches of topsoil in marsh areas of the Wetland. If natural topsoil from the site is to be used it must have at least 8 percent organic carbon content (by weight) in the A-horizon for sandy soils and 12% for other soil types.
   c. If planting soil is being imported it should be made up of equivalent proportions of organic and mineral materials.
   d. Lime should not be added to planting soil unless absolutely necessary as it may encourage the propagation of invasive species.
   e. The final elevations and hydrology of the wetland zones should be evaluated prior to planting to determine if grading or planting changes are required.
5. **Vegetation**
   a. Plant Lists for Constructed Wetlands can be found in Appendix B. No substitutions of specified plants will be accepted without prior approval of the designer. Planting locations shall be based on the Planting Plan and directed in the field by a qualified wetland ecologist.
   b. All wetland plant stock shall exhibit live buds or shoots. All plant stock shall be turgid, firm, and resilient. Internodes of rhizomes may be flexible and not necessarily rigid. Soft or mushy stock shall be rejected. The stock shall be free of deleterious insect infestation, disease and defects such as knots, sun-scald, injuries, abrasions, or disfigurement that could adversely affect the survival or performance of the plants.
   c. All stock shall be free from invasive or nuisance plants or seeds such as those listed in Appendix B.
   d. During all phases of the work, including transport and onsite handling, the plant materials shall be carefully handled and packed to prevent injuries and desiccation. During transit and onsite handling, the plant material shall be kept from freezing and shall be kept covered, moist, cool, out of the weather, and out of the wind and sun. Plants shall be watered to maintain moist soil and/or plant conditions until accepted.
   e. Plants not meeting these specifications or damaged during handling, loading, and unloading will be rejected.
   f. Detailed planting specifications can be found in Appendix B.
6. **Outlet Control Structure**
   a. Outlet control structures shall be constructed of non-corrodible material.
   b. Outlets shall be resistant to clogging by debris, sediment, floatables, plant material, or ice.
   c. Materials shall comply with applicable specifications (PennDOT or AASHTO, latest edition)

APPENDIX B: SCHEDULE



Schedule:

Harvesting Stormwater for Urban Farm Irrigation

Number	Task	Start	End	Duration
	Senior Design I			
1	Planning/Organizing	8/1/2010	9/10/2010	30
2	Research/Brainstorming	8/1/2010	12/6/2010	91
3	Surveying - Site Visit	9/1/2010	9/15/2010	15
4	Soil Analysis	12/20/2010	1/14/2011	25
	In-situ			
	Lab			
5	Preliminary Drawings	11/1/2010	11/18/2010	18
6	Preliminary Design	11/1/2010	11/18/2010	18
7	Cost Analysis	11/1/2010	11/18/2010	18
8	TR-55 Model	11/1/2010	11/18/2010	18
9	Water Tests	12/20/2010	2/1/2011	53
10	1st Design Release to Owner	12/1/2010	12/6/2010	6
	Senior Design II			
11	Owner Input Considerations	12/6/2010	1/14/2011	40
12	Re-Design	1/15/2011	3/1/2011	46
13	Final Cost Anaylsis	3/1/2011	4/1/2011	33
14	Final Drawings	3/1/2011	4/1/2011	33
15	Final Design	3/1/2011	4/1/2011	33
16	Determine Sequence of Construction	3/1/2011	4/1/2011	33
17	2nd Design Release to Owner	3/1/2011	4/1/2011	33
18	Project Completion: Design Document Approved and Delivered	3/15/2011	4/15/2011	32

GHV Engineers, Inc.

APPENDIX C: SWMM 2/24 MODELLING INPUTS

Subcatchment Name	Roof 1	Roof 2	Forebay	Wetland	Pond
Outlet	J15	J16	Forebay	Wetland	Pond
Area (acres)	0.026974	0.026974	0.005739	0.0071396	0.001374
Width (ft)	25	25	5	10	4
% slope	100	100	0.5	1	1
% imperv	99	99	100	100	100
N-Imperv	0.011	0.011	0	0	0
N-Perv	0.1	0.1	0	0	0
Dstore-Imperv	0.05	0.05	0	0	0
Dstore-Perv	0.05	0.05	0	0	0

Storage Unit Name	Forebay	Wetland	Pond	Cistern	U6
Treatment	NO	NO	NO	NO	NO
Invert Elevation (ft)	94.25	96.25	95.25	90	98.75
Max Depth (ft)	5.5	3.5	4.5	10	0.75
Initial Depth (ft)	5	3	4	0	0
Ponded Area (ft2)	25	311	60	24	396
Evap. Factor	1	1	1	0	1
Infiltration	NO	NO	NO	NO	NO
Storage Curve	Tabular	Tabular	Tabular	Tabular	Tabular
Curve Name	Forebay	Wetland	Pond	Cistern	2nd_inch

Junction Name	J15	J16	J18	J21	J22
Invert Elevation	120	120	99	99	98.68
Max Depth	0	0	0	0	0
Initial Depth	0	0	0	0	0
Surcharge Depth	0	0	0	0	0
Ponded Area	0	0	0	0	0

Conduit Name	Downspout 1	Downspout 1	Pipe 1	Pipe 2
Inlet Node	J15	J15	J18	J18
Outlet Node	J18	J18	J22	J22
Shape	Rectangular closed	Rectangular closed	Circular	Circular
Max Depth (ft)	0.396	0.396	0.396	0.396
Bottom Width	0.3125	0.3125	n/a	n/a
Length (ft)	23	23	16	16
Roughness	0.013	0.013	0.013	0.013
Inlet Offset (ft)	0	0	0	0
Outlet Offset (ft)	0	0	0	0

P2F	F_W	W_P	P_C	C23	C24
J22	Forebay	Wetland	Pond	Cistern	U6
Forebay	Wetland	Pond	Cistern	U6	Out1
Circular	Rectangular Open	Rectangular_Open	Circular	Circular	Circular
0.75	0.75	0.75	1	1	1
n/a	2	2	n/a	n/a	n/a
50	0.5	0.5	0.5	1	100
0.015	0.012	0.012	0.013	0.01	0.01
0	5	3	4	9.25	0.5
3.43	3	4	9	0.5	0

APPENDIX D: SWMM STATUS REPORT (2YR/24HR)

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.021)
--------------------------------------------------------------


*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... GPM
Process Models:
  Rainfall/Runoff ........ YES
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... YES
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... KINWAVE
Starting Date ............ JAN-01-2001 00:00:00
Ending Date .............. DEC-31-2001 23:45:00
Antecedent Dry Days ...... 6.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 01:00:00
Routing Time Step ........ 30.00 sec

WARNING 04: minimum elevation drop used for Conduit F_W
WARNING 04: minimum elevation drop used for Conduit W_P
WARNING 04: minimum elevation drop used for Conduit P_C
WARNING 04: minimum elevation drop used for Conduit C1
```

```
**************************        Volume         Depth
Runoff Quantity Continuity     acre-feet        inches
**************************     ---------       -------
Total Precipitation ......         0.016         2.990
Evaporation Loss .........         0.000         0.042
Infiltration Loss ........         0.000         0.026
Surface Runoff ...........         0.015         2.923
Final Surface Storage ....         0.000         0.000
Continuity Error (%) .....         0.000


**************************            zn            pb           tss
Runoff Quality Continuity            lbs           lbs           lbs
**************************    ----------    ----------    ----------
Initial Buildup ..........         0.000         0.000        12.353
Surface Buildup ..........         0.002         0.003        13.017
Wet Deposition ...........         0.000         0.000         0.000
Sweeping Removal .........         0.000         0.000         0.000
Infiltration Loss ........         0.000         0.000         0.000
BMP Removal ..............         0.000         0.000         0.000
Surface Runoff ...........         0.002         0.003        12.370
Remaining Buildup ........         0.000         0.000        13.000
Continuity Error (%) .....         0.000         0.000         0.000


**************************        Volume        Volume
Flow Routing Continuity        acre-feet      10^6 gal
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.015         0.005
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.012         0.004
Internal Outflow .........         0.000         0.000
Storage Losses ...........         0.015         0.005
Initial Stored Volume ....         0.016         0.005
Final Stored Volume ......         0.004         0.001
Continuity Error (%) .....        -0.120
```

```
***************************       zn          pb          tss
Quality Routing Continuity       lbs         lbs          lbs
***************************   ----------  ----------  ----------
Dry Weather Inflow .......        0.000       0.000       0.000
Wet Weather Inflow .......        0.002       0.003      12.338
Groundwater Inflow .......        0.000       0.000       0.000
RDII Inflow ..............        0.000       0.000       0.000
External Inflow ..........        0.000       0.000       0.000
Internal Flooding ........        0.000       0.000       0.000
External Outflow .........        0.001       0.002       4.841
Mass Reacted .............        0.000       0.000       5.928
Initial Stored Mass ......        0.000       0.000       0.000
Final Stored Mass ........        0.001       0.002       1.894
Continuity Error (%) .....       -3.208      -3.208      -2.640


*********************************
Highest Flow Instability Indexes
*********************************
All links are stable.


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :    30.00 sec
Average Time Step           :    30.00 sec
Maximum Time Step           :    30.00 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     1.00


***************************
Subcatchment Runoff Summary
***************************

--------------------------------------------------------------------------------------------------------
                            Total      Total      Total      Total      Total        Total     Peak  Runoff
                           Precip      Runon       Evap      Infil     Runoff       Runoff   Runoff   Coeff
Subcatchment                   in         in         in         in         in    10^6 gal      GPM
--------------------------------------------------------------------------------------------------------
Roof1                        2.99       0.00       0.05       0.03       2.91         0.00     1.57   0.974
Roof2                        2.99       0.00       0.05       0.03       2.91         0.00     1.57   0.974
S9                           2.99       0.00       0.00       0.00       2.99         0.00     0.03   1.000
S10                          2.99       0.00       0.00       0.00       2.99         0.00     0.42   1.000
S11                          2.99       0.00       0.00       0.00       2.99         0.00     0.08   1.000
```

```
****************************
Subcatchment Washoff Summary
****************************

-----------------------------------------------------------------
                                    zn             pb            tss
Subcatchment                       lbs            lbs            lbs
-----------------------------------------------------------------
Roof1                            0.001          0.002          6.185
Roof2                            0.001          0.002          6.185
S9                               0.000          0.000          0.000
S10                              0.000          0.000          0.000
S11                              0.000          0.000          0.000
-----------------------------------------------------------------
System                           0.002          0.003         12.370


******************
Node Depth Summary
******************

-----------------------------------------------------------------------
                                Average  Maximum  Maximum  Time of Max
                                  Depth    Depth      HGL   Occurrence
Node               Type            Feet     Feet     Feet  days hr:min
-----------------------------------------------------------------------
J15                JUNCTION        0.00     0.00   120.00     0  03:05
J16                JUNCTION        0.00     0.00   120.00     0  03:05
J18                JUNCTION        0.00     0.03    99.03     0  03:04
J21                JUNCTION        0.00     0.03    99.03     0  03:04
J22                JUNCTION        0.00     0.03    98.71     0  03:03
Out_Street         OUTFALL         0.00     0.02    90.02     0  19:33
Out1               OUTFALL         0.00     0.00   105.00     0  00:00
Cistern            STORAGE         3.72     9.55    99.30     0  09:14
Forebay            STORAGE         3.81     5.01    99.26     0  02:50
Wetland            STORAGE         1.77     3.01    99.26     0  02:51
Pond               STORAGE         2.80     4.06    99.31     0  02:52
2nd_Inch           STORAGE         0.16     0.52    99.27     0  19:32
```

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow GPM	Maximum Total Inflow GPM	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 gal	Total Inflow Volume 10^6 gal
J15	JUNCTION	1.57	1.57	0 03:05	0.002	0.002
J16	JUNCTION	1.57	1.57	0 03:05	0.002	0.002
J18	JUNCTION	0.00	1.57	0 03:06	0.000	0.002
J21	JUNCTION	0.00	1.57	0 03:06	0.000	0.002
J22	JUNCTION	0.00	3.14	0 03:05	0.000	0.004
Out_Street	OUTFALL	0.00	3.68	0 19:33	0.000	0.002
Out1	OUTFALL	0.00	27.60	83 00:00	0.000	0.002
Cistern	STORAGE	0.00	3.68	0 02:53	0.000	0.005
Forebay	STORAGE	0.03	3.18	0 03:04	0.000	0.005
Wetland	STORAGE	0.42	3.60	0 02:51	0.001	0.008
Pond	STORAGE	0.08	3.68	0 02:51	0.000	0.007
2nd_Inch	STORAGE	0.00	3.78	0 09:15	0.000	0.003

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 ft3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 ft3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow GPM
Cistern	0.088	36	0	0.226	93	0 09:14	27.60
Forebay	0.092	69	9	0.122	91	0 02:50	3.18
Wetland	0.247	66	30	0.360	96	0 02:51	3.60
Pond	0.150	66	15	0.211	93	0 02:52	3.68
2nd_Inch	0.062	21	29	0.205	69	0 19:32	27.60

****
Outfall Loading Summary
****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow GPM	Max. Flow GPM	Total Volume 10^6 gal	Total zn lbs	Total pb lbs	Total tss lbs
Out_Street	0.10	3.42	3.68	0.002	0.000	0.000	1.339
Out1	0.02	27.35	27.60	0.002	0.001	0.001	3.483
System	0.06	30.78	27.60	0.004	0.001	0.002	4.821

****
Link Flow Summary
****

Link	Type	Maximum |Flow| GPM	Time of Max Occurrence days hr:min	Maximum |Veloc| ft/sec	Max/ Full Flow	Max/ Full Depth
Downspout2	CONDUIT	1.57	0 03:06	0.00	0.00	0.01
pipe2	CONDUIT	1.57	0 03:05	1.04	0.01	0.06
downspout1	CONDUIT	1.57	0 03:06	0.00	0.00	0.01
F_W	CONDUIT	3.18	0 02:51	0.29	0.00	0.02
P2F	CONDUIT	3.14	0 03:04	1.08	0.00	0.04
W_P	CONDUIT	3.60	0 02:51	0.31	0.00	0.02
P_C	CONDUIT	3.68	0 02:53	0.42	0.01	0.06
pipe1	CONDUIT	1.57	0 03:05	1.04	0.01	0.06
C1	CONDUIT	3.78	0 09:15	0.50	0.01	0.05
C2	CONDUIT	3.68	0 19:33	2.44	0.00	0.02
ou2	DUMMY	27.60	142 00:00			
ou1	DUMMY	27.60	83 00:00			

```
*************************
Conduit Surcharge Summary
*************************

No conduits were surcharged.


Analysis begun on:  Mon Apr 25 18:55:29 2011
Analysis ended on:  Mon Apr 25 18:56:29 2011
```

APPENDIX E: SWMM STATUS REPORT (31YR)

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.021)
--------------------------------------------------------------


*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... GPM
Process Models:
  Rainfall/Runoff ........ YES
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... YES
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... KINWAVE
Starting Date ............ JAN-01-1971 00:00:00
Ending Date .............. DEC-31-2001 23:45:00
Antecedent Dry Days ...... 6.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 01:00:00
Routing Time Step ........ 30.00 sec

WARNING 09: time series interval greater than recording interval for Rain Gage Gage1
WARNING 04: minimum elevation drop used for Conduit F_W
WARNING 04: minimum elevation drop used for Conduit W_P
WARNING 04: minimum elevation drop used for Conduit P_C
WARNING 04: minimum elevation drop used for Conduit C1

**************************        Volume         Depth
Runoff Quantity Continuity     acre-feet        inches
**************************     ---------       -------
Total Precipitation ......         6.819      1298.070
Evaporation Loss .........         0.436        83.009
Infiltration Loss ........         0.051         9.738
Surface Runoff ...........         6.418      1221.783
Final Surface Storage ....         0.000         0.042
Continuity Error (%) .....        -1.271
```

```
**************************             zn            pb            tss
Runoff Quality Continuity             lbs           lbs            lbs
**************************     ----------    ----------     ----------
Initial Buildup ..........          0.000         0.000         12.353
Surface Buildup ..........          2.529         4.516      18056.997
Wet Deposition ...........          0.000         0.000          0.000
Sweeping Removal .........          0.000         0.000          0.000
Infiltration Loss ........          0.000         0.000          0.000
BMP Removal ..............          0.000         0.000          0.000
Surface Runoff ...........          2.529         4.515      18061.496
Remaining Buildup ........          0.000         0.000          7.349
Continuity Error (%) .....          0.000         0.000          0.003


**************************         Volume        Volume
Flow Routing Continuity         acre-feet      10^6 gal
**************************     ----------    ----------
Dry Weather Inflow .......          0.000         0.000
Wet Weather Inflow .......          6.418         2.091
Groundwater Inflow .......          0.000         0.000
RDII Inflow ..............          0.000         0.000
External Inflow ..........          0.000         0.000
External Outflow .........          5.741         1.871
Internal Outflow .........          0.000         0.000
Storage Losses ...........          0.629         0.205
Initial Stored Volume ....          0.016         0.005
Final Stored Volume ......          0.024         0.008
Continuity Error (%) .....          0.626


**************************             zn            pb            tss
Quality Routing Continuity            lbs           lbs            lbs
**************************     ----------    ----------     ----------
Dry Weather Inflow .......          0.000         0.000          0.000
Wet Weather Inflow .......          2.335         4.169      16676.478
Groundwater Inflow .......          0.000         0.000          0.000
RDII Inflow ..............          0.000         0.000          0.000
External Inflow ..........          0.000         0.000          0.000
Internal Flooding ........          0.000         0.000          0.000
External Outflow .........          2.502         4.468       2863.708
Mass Reacted .............          0.000         0.000      15087.451
Initial Stored Mass ......          0.000         0.000          0.000
Final Stored Mass ........          0.018         0.032          7.185
Continuity Error (%) .....         -7.929        -7.929         -7.687
```

```
********************************
Highest Flow Instability Indexes
********************************
All links are stable.


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :    30.00 sec
Average Time Step           :    30.00 sec
Maximum Time Step           :    30.00 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     1.00


***************************
Subcatchment Runoff Summary
***************************

----------------------------------------------------------------------------------------------------------
                           Total      Total      Total      Total      Total      Total     Peak   Runoff
                          Precip      Runon       Evap      Infil     Runoff     Runoff   Runoff    Coeff
Subcatchment                  in         in         in         in         in    10^6 gal     GPM
----------------------------------------------------------------------------------------------------------
Roof1                    1298.07       0.00      94.21      11.38    1211.71       0.89    80.79    0.933
Roof2                    1298.07       0.00      94.21      11.38    1211.71       0.89    80.79    0.933
S9                       1298.07       0.00      16.51       0.00    1281.56       0.02     1.72    0.987
S10                      1298.07       0.00      16.51       0.00    1281.56       0.25    21.41    0.987
S11                      1298.07       0.00      16.51       0.00    1281.56       0.05     4.12    0.987


****************************
Subcatchment Washoff Summary
****************************

--------------------------------------------------------------
                                 zn           pb          tss
Subcatchment                    lbs          lbs          lbs
--------------------------------------------------------------
Roof1                         1.264        2.258     9030.748
Roof2                         1.264        2.258     9030.748
S9                            0.000        0.000        0.000
S10                           0.000        0.000        0.000
S11                           0.000        0.000        0.000
--------------------------------------------------------------
System                        2.529        4.515    18061.496
```

```
******************
Node Depth Summary
******************

--------------------------------------------------------------------
                               Average  Maximum  Maximum  Time of Max
                                 Depth    Depth      HGL   Occurrence
Node                 Type         Feet     Feet     Feet  days hr:min
--------------------------------------------------------------------
J15                  JUNCTION     0.00     0.04   120.04  10507  00:55
J16                  JUNCTION     0.00     0.04   120.04  10507  00:55
J18                  JUNCTION     0.00     0.18    99.18  10507  00:55
J21                  JUNCTION     0.00     0.18    99.18  10507  00:55
J22                  JUNCTION     0.00     0.21    98.89  10507  00:05
Out_Street           OUTFALL      0.00     0.12    90.12  10507  00:13
Out1                 OUTFALL      0.00     0.00   105.00      0  00:00
Cistern              STORAGE      8.34     9.87    99.62  10507  00:07
Forebay              STORAGE      4.97     5.14    99.39  10507  00:05
Wetland              STORAGE      2.97     3.15    99.40  10507  00:07
Pond                 STORAGE      3.97     4.42    99.67  10507  00:07
2nd_Inch             STORAGE      0.32     0.62    99.37  10507  00:13


*******************
Node Inflow Summary
*******************

-------------------------------------------------------------------------------------
                                Maximum  Maximum                  Lateral       Total
                                Lateral    Total  Time of Max      Inflow      Inflow
                                 Inflow   Inflow   Occurrence      Volume      Volume
Node                 Type           GPM      GPM  days hr:min    10^6 gal    10^6 gal
-------------------------------------------------------------------------------------
J15                  JUNCTION     80.79    80.79  10507  00:55      0.888       0.888
J16                  JUNCTION     80.79    80.79  10507  00:55      0.888       0.888
J18                  JUNCTION      0.00    80.79  10507  00:55      0.000       0.887
J21                  JUNCTION      0.00    80.79  10507  00:55      0.000       0.887
J22                  JUNCTION      0.00   162.04  10507  00:05      0.000       1.774
Out_Street           OUTFALL       0.00   191.85  10507  00:13      0.000       1.368
Out1                 OUTFALL       0.00    27.60     85  00:00     0.000       0.502
Cistern              STORAGE       0.00   189.34  10507  00:07      0.000       2.021
Forebay              STORAGE       1.72   163.53  10507  00:05      0.020       1.795
Wetland              STORAGE      21.41   188.37  10507  00:05      0.248       2.027
Pond                 STORAGE       4.12   189.97  10507  00:07      0.048       2.048
2nd_Inch             STORAGE       0.00   189.53  10507  00:07      0.000       1.816
```

```
**********************
Node Surcharge Summary
**********************

No nodes were surcharged.


*********************
Node Flooding Summary
*********************

No nodes were flooded.


**********************
Storage Volume Summary
**********************
```

Storage Unit	Average Volume 1000 ft3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 ft3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow GPM
Cistern	0.197	81	0	0.234	96	10507 00:07	190.02
Forebay	0.121	90	1	0.125	93	10507 00:05	167.04
Wetland	0.358	95	1	0.365	97	10507 00:07	185.86
Pond	0.208	91	1	0.225	99	10507 00:07	189.29
2nd_Inch	0.126	42	8	0.245	83	10507 00:12	191.97

```
***********************
Outfall Loading Summary
***********************
```

Outfall Node	Flow Freq. Pcnt.	Avg. Flow GPM	Max. Flow GPM	Total Volume 10^6 gal	Total zn lbs	Total pb lbs	Total tss lbs
Out_Street	0.96	8.46	191.85	1.368	1.689	3.017	1915.240
Out1	0.11	27.49	27.60	0.502	0.750	1.340	880.943
System	0.53	35.95	191.85	1.871	2.440	4.357	2796.182

```
********************
Link Flow Summary
********************
```

Link	Type	Maximum |Flow| GPM	Time of Max Occurrence days hr:min	Maximum |Veloc| ft/sec	Max/ Full Flow	Max/ Full Depth
Downspout2	CONDUIT	80.79	10507 00:55	16.15	0.04	0.09
pipe2	CONDUIT	81.02	10507 00:05	3.32	0.42	0.45
downspout1	CONDUIT	80.79	10507 00:55	16.15	0.04	0.09
F_W	CONDUIT	166.95	10507 00:05	1.35	0.08	0.18
P2F	CONDUIT	161.81	10507 00:05	3.46	0.18	0.29
W_P	CONDUIT	185.85	10507 00:07	1.41	0.09	0.20
P_C	CONDUIT	189.34	10507 00:07	1.33	0.37	0.42
pipe1	CONDUIT	81.02	10507 00:05	3.32	0.42	0.45
C1	CONDUIT	189.53	10507 00:07	1.61	0.29	0.37
C2	CONDUIT	191.85	10507 00:13	8.11	0.03	0.12
ou2	DUMMY	27.60	88 00:00			
ou1	DUMMY	27.60	85 00:00			

```
*************************
Conduit Surcharge Summary
*************************

No conduits were surcharged.


Analysis begun on:  Mon Apr 25 17:46:28 2011
Analysis ended on:  Mon Apr 25 18:19:38 2011
```