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Modeling of Green Infrastructure Components and Large Scale Test and Control Watersheds

at Kansas City, Missouri

Prepared for
USEPA Office of Research and Development
Urban Watershed Management Branch

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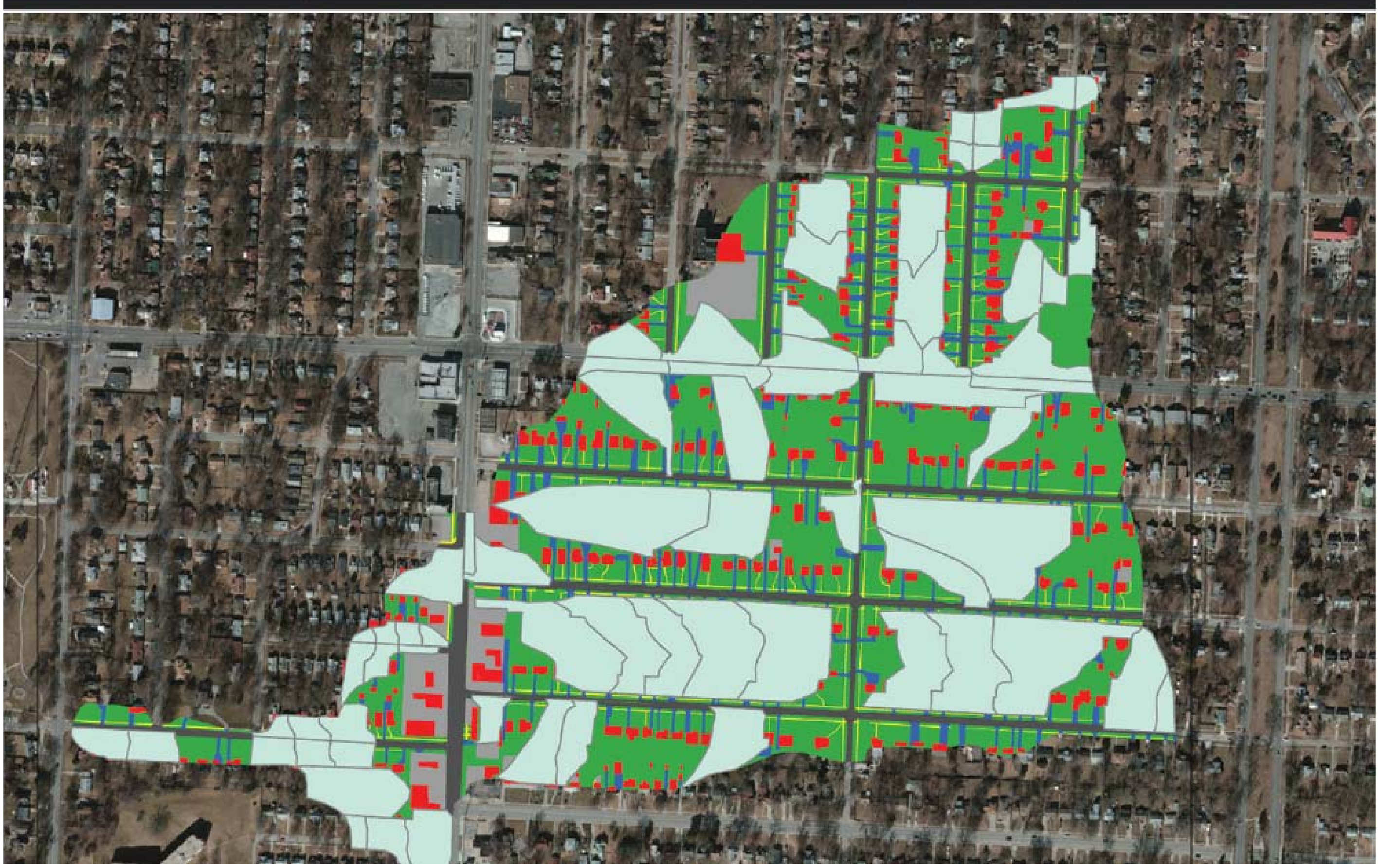

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Acknowledgements

The results presented in this report are a small portion of the full project activities associated with the National Demonstration of Advanced Drainage Concepts Using Green Solutions for CSO Control Project in Kansas City, funded by the USEPA Office of Research and Development, Urban Watershed Management Branch. The direction and support of the EPA Project Officers, Michelle Simon and Richard Field, are gratefully acknowledged. Other project participants, including Tetra Tech personnel (Dustin Bambic, Jason Wright, and Scott Struck, plus others), University of Missouri, Kansas City faculty and student researchers (under the direction of Dr. Deb O'Bannon), University of Alabama graduate students, the Kansas City Water Services Department, Dr. Michael Ports, and Mid-America Regional Council (MARC), were all instrumental in planning and carrying out this project and are also gratefully acknowledged.

1. Introduction and Summary

Green infrastructure includes practices and site-design techniques that store, infiltrate, evaporate, or detain stormwater runoff and in so doing, control the timing and volume of stormwater discharges from impervious surfaces (e.g., streets, building roofs, and parking lots) to the stormwater collection systems. EPA's Office of Research and Development has the goal to provide detailed guidance and information on methodologies for selection, placement, and cost effectiveness and to document the benefits of green infrastructure applications in urban watersheds for new development, redevelopment, and retrofit situations.

The Kansas City Water Services Department (KCWSD) provides wastewater collection and treatment services for approximately 650,000 people, located within the City and in 27 tributary or "satellite" communities. This demonstration project was developed to demonstrate the application of green infrastructure for combined sewer overflow (CSO) control in the Middle Blue River. KCWSD completed construction of a 100-acre retrofit of an aging neighborhood that has included sewer rehabilitation and implementation of over 100 green infrastructure (GI) solutions. This project is one of the largest in the United States and provides a unique opportunity for USEPA ORD to quantify the benefits of GI solutions on large scales (overall pilot project area) and small scales (individual GI solutions) and meet its GI-related goals.

This report describes efforts to develop a watershed model (using WinSLAMM—the Source Loading and Management Model) for this area using the pre-construction flow and water quality data. The pre- and post-construction flow monitoring has facilitated quantification of the benefits of the upland stormwater controls, and served as the basis for watershed model development. WinSLAMM was used to calculate the stormwater contributions to the combined sewerage system during wet weather by providing a time series of flows and water quality conditions, for various types of upland controls. The study test (pilot) area is a 100-acre subcatchment. This sewershed is mostly medium-density residential areas, with some commercial and institutional land uses. An adjacent 80-acre subcatchment was also monitored as a control watershed, with no stormwater controls, for comparison.

The project contractor was Tetra Tech, Inc., and associated subcontractors were the University of Alabama (UA), University of Missouri–Kansas City (UMKC), Mid-America Regional Council (MARC), and Michael Ports and Associates. Critical project leveraging and cooperation was provided by the Kansas City Water Services Department and EPA Region 7.

Project Summary

The following summary is compiled from the end of section summaries, plus most of the conclusion section of this report.

Overview of Watershed Model Development

Model calibration requires detailed information pertaining to the areas where monitored data have been collected to compare to the modeled predictions. For this project, calibration of the WinSLAMM model was conducted in several steps:

- Regional calibration using water quality and flow data from the National Stormwater Quality Database (NSQD). This information was used to update and compare the original model calibrations that were mostly associated with Wisconsin and Alabama source data. The regional NSQD data, along with additional more recent Nebraska data, enabled significant amounts of data to be examined for the main land use categories for this geographical area in the US. The Kansas City area is in the central U.S. region, and those data were used for this step of the calibration process.
- Detailed land development characteristics were obtained for the study area, along with site soil infiltration measurements, by UMKC project teams. This allowed the model calibration based on these critical site characteristics to be included. Long-term continuous rain data were also used during the analyses to minimize the effects of any unusual conditions, along with the actual monitored rains.
- Site-specific rainfall and runoff data were obtained during five years of monitoring (from 2009 through 2013) in the test/pilot and control watersheds in the Marlborough study area of Kansas City. Being a combined sewer system, the measured wet weather flows were adjusted by having the expected concurrent dry weather sanitary sewage flows (from adjacent dry period monitoring periods) subtracted from the combined sewer flows. The final hydrograph separation analyses were conducted by the University of Alabama project team. These flow data were used to verify the regional and site calibration conditions. The site development characteristics for the test and control watershed were used, along with the actual rain history during the flow monitoring period, to show how closely the calibrated model predicted the runoff characteristics that were monitored.
- As data became available, additional calibration verifications of the model were made. As an example, the two demonstration rain gardens have two to three years of flow data available (no overflows occurred during the monitoring period, with complete infiltration). Those observations were also used to verify the modeled expected performance of these controls. The major data source available includes the complete area before and after the green infrastructure (GI) components were constructed (mostly composed of curb-cut biofilters and porous pavement). Eight of the GI components were constructed that enabled localized monitoring to supplement the large-scale system monitoring.

Summary of WinSLAMM Description and Use for GI Projects

Over the years, WinSLAMM has been extensively revised and expanded and now includes a wide range of capabilities, including its ability to evaluate stormwater management options using a long series of rain events, especially important for evaluating combined sewer and GI issues. The effectiveness of the control practices in WinSLAMM are calculated on the basis of the actual sizing and other attributes of the devices, the source area or outfall location characteristics, and the calculated runoff characteristics. The model does a complete mass balance and routing of water volume and particulate mass, considering the combined effects of all controls. Hydraulic and particle size routing occurs for each device individually, and serial effects of multiple devices are now accurately considered in version 10 of WinSLAMM.

WinSLAMM conducts a continuous water mass balance for every storm in the study period at various scales. As an example, for rain barrels, water tanks or cisterns, used for harvesting roof runoff for later irrigation or other beneficial uses, the model fills the available storage during rains. Between rains, the storage tank is drained according to the water withdrawal use for each month. If the tank is almost full from a preceding close rain (and not enough time was available to drain the storage tank), excess water from the event would be discharged to the drainage system after the tank fills. Curb-cut rain biofilters and cascading swale systems along a street allow the site runoff to infiltrate. If the runoff volume is greater than the capacity of the biofilters, the excessive water is discharged into the drainage system, or possibly additional downgradient controls. When evaluated together, the cisterns treat the roof runoff first, with the excess water discharged to curb-cut biofilters for infiltration. The continuous simulation drains the devices between events according to the interevent conditions.

The first step in setting up a WinSLAMM analysis is to identify the rain and the calibrated parameter files to be used. The rain file describes the series of rains to be considered in the analysis. Initially, the 10 years of Kansas City rains from 1990 through 1999 were used for basic analyses. This ten year period had 920 rains that ranged from 0.01 to 3.79 inches (in.), with an average total annual rainfall of about 35 to 40 in. Land development characteristics describing local site conditions of the study area are used by WinSLAMM to calculate expected runoff characteristics. One of the important features of WinSLAMM is to calculate the sources of the flows and pollutants of interest for the study area under different rain conditions.

Summary of Site Characteristics Used in Stormwater Quality Modeling

Land development information corresponding to the different land uses in an area is needed as an initial step in investigating stormwater management options for an area. The Marlborough study (pilot) area in Kansas City is mostly a medium-density residential area, constructed before 1960, with a small amount of strip commercial area along Troost Ave., and a small portion of a school. Detailed inventories were made of each of the approximately 600 homes in the area by graduate students from UMKC. Table 1 shows the breakdown of the surface areas in the medium density residential area portion of the test (pilot) watershed. The values shown on this table are the percentages of each subarea of the whole area, while the values shown in parentheses are the breakdown within a single subarea. For example, directly connected roofs make up about 1.87% of the complete 100 acre site, and represent about 15% of all roofs.

Table 1. Medium-density residential area site characteristics (%)

	Roofs	Driveways	Sidewalks	Parking/storage	Streets	Landscaped	Isolated	Total
Impervious								
directly connected	1.87 (15%*)	4.12 (46%)	1.15 (46%)	1.59	9.35			18.07
disconnected	10.57 (85%)	4.03 (45%)	1.34 (54%)					15.95
Pervious								
unpaved (gravel, severely compacted)		0.81 (9%)						0.81
landscaped						65.13		65.13
isolated (swimming pools)							0.05	0.05
Total residential area	12.44	8.95	2.49	1.59	9.35	65.13	0.05	100.00

* percentage of total subarea in this category; for example, 15% of all roofs are directly connected and 85% of all roofs are disconnected.

Detailed site information is needed for stormwater management evaluations. Only about 15% of the residential roofs are directly connected in the test (pilot) area. If all were assumed to be directly connected, large errors in the roof runoff contribution calculations would occur. Similarly, if roof runoff stormwater controls were located at all roofs, those located where the roofs were already disconnected would provide much lower additional benefits in decreasing the area's runoff quantity.

In addition to the site surveys, site soils tests were also conducted for the area by the UMKC graduate students. Small-scale infiltrometers were used to measure infiltration rates in the disturbed urban soils of the test watershed area. The most precise measurements of infiltration, and which should be used in areas where large-scale infiltration units are being designed, should rely on full-scale tests. These are typically large trenches or boreholes, constructed to penetrate the depths of soil that the final units will use for infiltration, and use large volumes of water over extended periods. In the Kansas City study area, the constructed rain gardens and curb-cut biofilters have undergone full-scale inundation tests after construction to supplement the smaller scale tests. In addition, the rate of infiltration during the actual rains was also measured to obtain actual rates for the area and designs used. Figure 1 shows the measured infiltration rates from the small-scale tests in the test area.

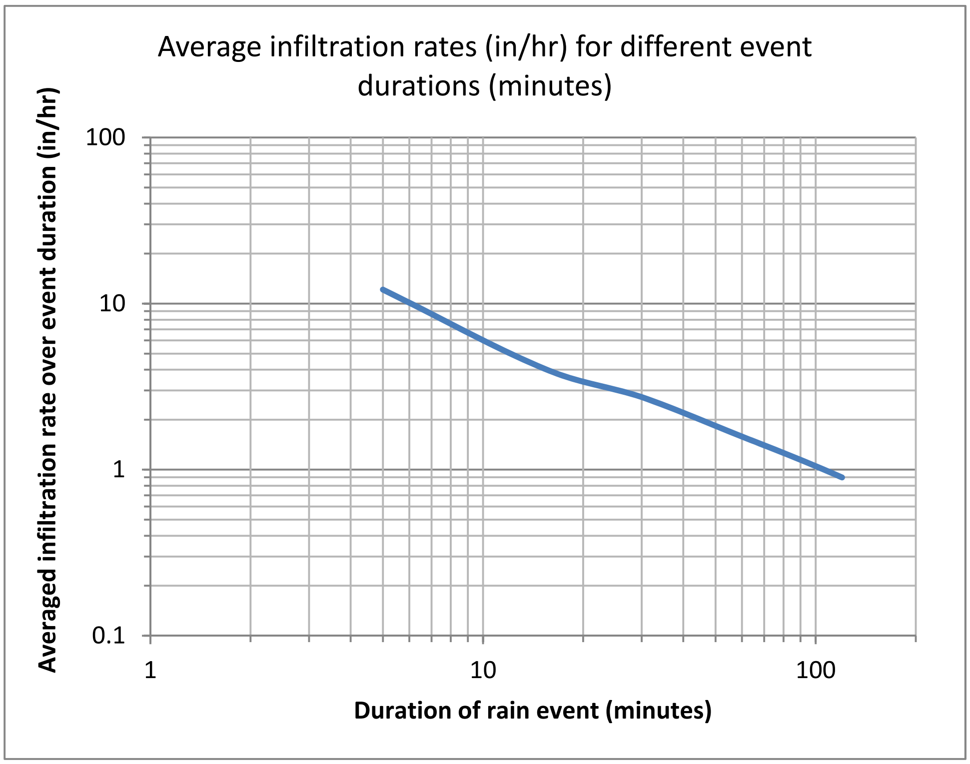


Figure 1. Duration-infiltration rates for surface soils.

Figure 1 indicates that the infiltration rate would be between 1 and 10 inches per hour (in/hr) for rains that lasted up to about two hours, with likely decreasing infiltration rates for the long rains of interest for the critical combined sewer overflow (CSO) event design storm. Initial modeling efforts supporting the GI designs assumed an infiltration rate of about 0.3 in/hr. Deeper soil profiles indicated that this might be too large. Therefore, for the shallow rain gardens, an infiltration rate of 0.2 in/hr was used by the initial designers. However, actual infiltration measurements in the constructed biofilters after saturated conditions indicated system infiltration rates are generally between 0.5 and 2.5 in/hr, while modeling indicates that the subsurface infiltration rates in the native soils are likely close to 1 in/hr. Subsurface infiltration in areas of biofiltration device construction can be higher than surface rates because of typical decreased amounts of clays and reduced compaction. If care is taken to minimize compaction during construction, these higher rates might be preserved. The extended monitoring period will help verify the actual soil infiltration conditions.

Summary of System-wide Observations and Model Calibration

Runoff monitoring was conducted in the combined sewer system at several locations in the test and control watersheds. Events were monitored after the sewer was rehabilitated, and these data were used as a new baseline condition. WinSLAMM evaluated the test (pilot) and control watershed conditions during the two monitoring periods to verify the rainfall-runoff calibration based on site development characteristics and the actual rains monitored.

Figure 2 compares the pre and post-construction Rv values for the test/pilot watershed as monitored at UMKC01. The post-construction Rv values are apparently smaller than the pre-construction Rv values, as expected. The combined sewer was relined near the end of the baseline monitoring phase before the construction of the stormwater controls. Statistical tests of the flows after the relining compared to before relining did not identify any significant differences for the number of samples available. There was an apparent increase in flows after relining (possibly associated with reduced exfiltration of combined sewage), but the few flow observations after relining did not allow an effective statistical comparison. Comparisons of pre and post-construction Rv values were also made for different rain categories which also showed decreasing flows after construction of the stormwater controls, especially for the small rains where the biofilters would be expected to be most effective. The RV differences for the two smallest rain categories were significantly different (40 and 33% runoff volume reductions for <0.5 inches and 0.5 to 1.5 inches rains respectively), but the largest rain category (>1.5 inches) did not have significant differences (observed at 13%) for the number of observations available. Table 2 summarizes the overall reduction in flows observed in the test/pilot watershed which were calculated to be about 32% on a flow-weighted basis and were highly significant ($p < 0.001$).

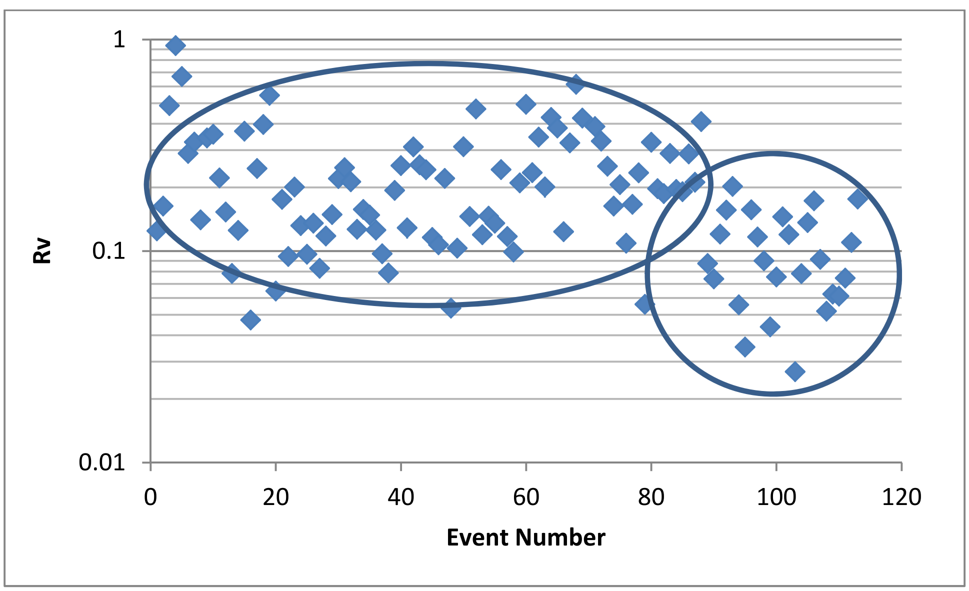


Figure 2. Volumetric runoff coefficients at UNKC01 before and after GI construction.

Table 2. Summary of Statistical Comparisons for Before and After GI Facility Construction at UMKC01

Monitoring period	Dates corresponding to monitoring period	Number of monitored storms in each monitoring period	Flow-Weighted Rv values	% change compared to "initial baseline and after relining (combined)" (p from Mann-Whitney Rank-Sum test)
Initial baseline and after Relining	(03/23/09 – 06/16/10) and (02/24/11 – 03/19/11)	75	0.26	n/a
After construction	04/07/13 – 10/31/2013	37	0.18	32.3% decrease ($p < 0.001$)

Figure 3 is a scatterplot showing the observed versus the modeled test (pilot) watershed area total flows for each of the events during the baseline period. As shown, these are all close to the line of equivalent values.

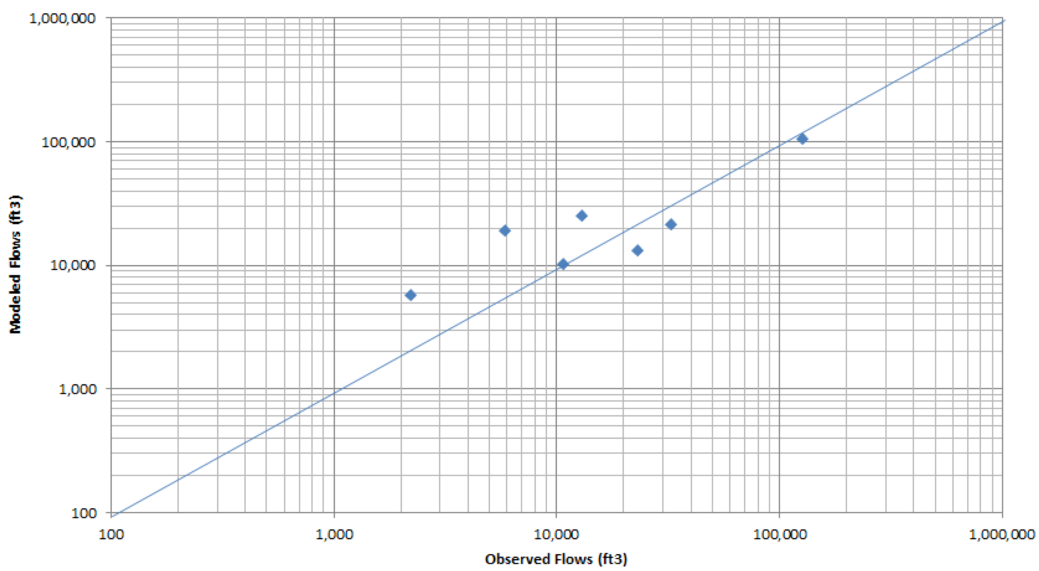


Figure 3. Observed versus modeled flows during baseline conditions.

Summary of Biofilter Measurements during Rain Events

A tremendous amount of information was collected during this project, ranging from drainage area characteristics to runoff and flow monitoring data. The extended construction period resulted in about 70 events (combined for all eight biofilters) that were monitored at the inlets and outlets. The infiltration rates in the biofilters were monitored during actual rains by measuring the rate of drop of the ponded water after the rains ended. Statistical analyses identified two distinct groups of these data, as shown in the following list and group box and whisker plot (Figure 4).

- Low rates: average 0.70 in/hr; range 0.19 to 1.9
- Normal rates: average 2.7 in/hr; range 0.10 to 7.2

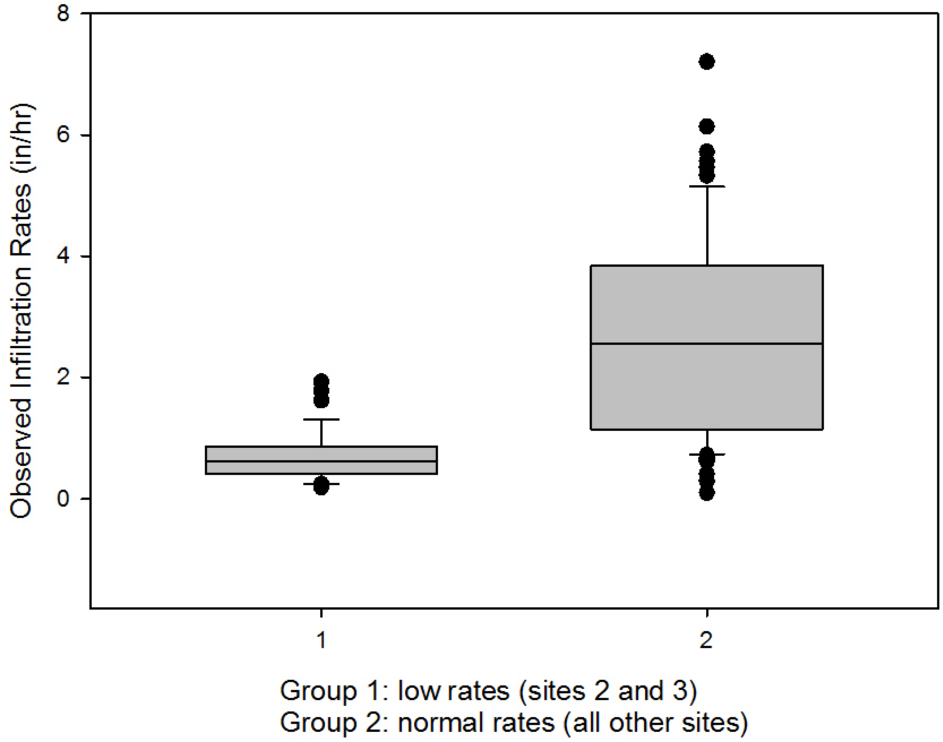


Figure 4. Measured infiltration rates in biofilters during actual rains.

The average time to ponding for each of the eight curb-side biofilters after the rain started ranged from 0.15 to 0.5 hour, with the fast group starting ponding in about 0.16 hrs and the slow group starting in about 0.3 hrs . The maximum depth of ponding was also separated into two categories, as shown below:

- Shallow: average of 3.4 in., range of 0.72 to 12 in.
- Deep: average of 6.3, range of 0.60 to 13 in

Figure 5 is a group box and whisker plot showing these two combined sets of data for maximum depth of ponding.

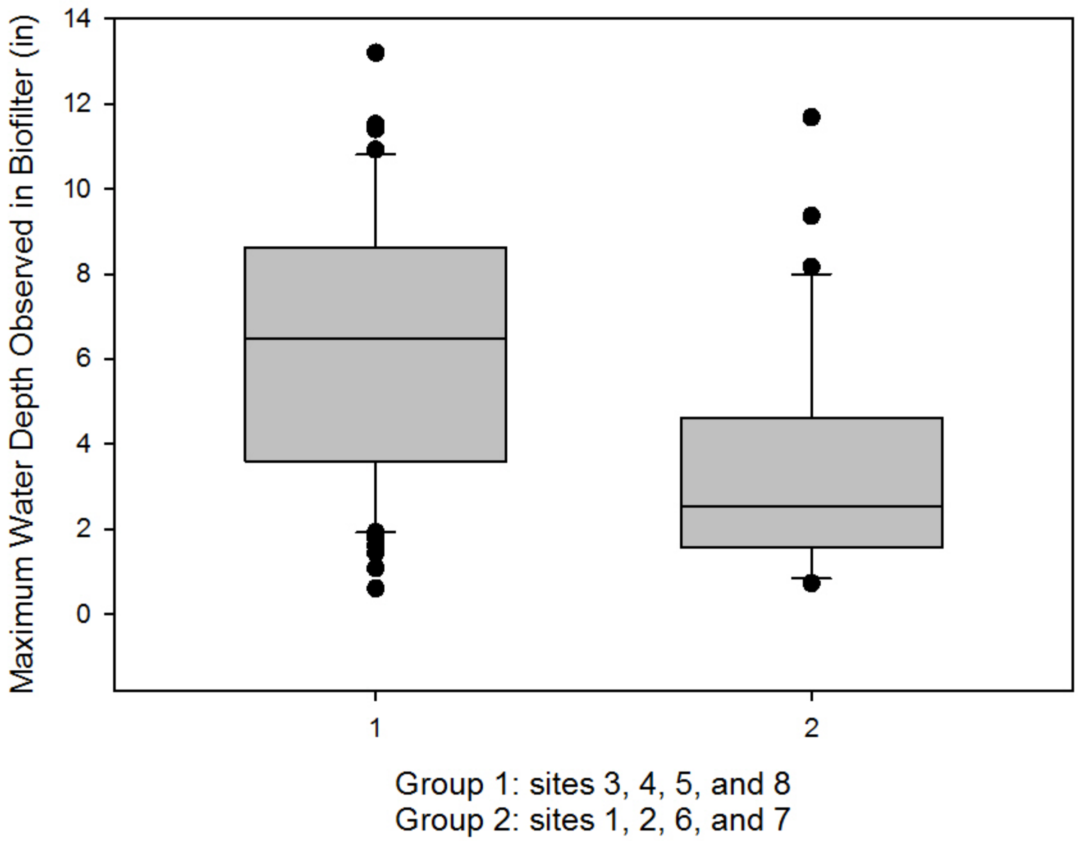


Figure 5. Maximum ponding depth observed in biofilters during actual rains.

Laboratory column tests were conducted to investigate the biofilter media used at the Kansas City sites. Columns were constructed to measure the infiltration rates as a function of compaction (and therefore density). The density of the media column with hand compaction was 1.00 grams per cubic centimeter (g/cm^3); the density of the standard proctor media column was 1.13 g/cm^3, and the density of the modified proctor column was 1.12 g/cm^3. The soil media has a median particle size (D_{50}) of about 1.9 millimeters (mm) and a very high uniformity coefficient (C_u) of 39. The porosity of the media for the hand compaction columns was 0.36, 0.15 for the standard proctor compaction columns, and 0.25 for the modified compaction columns.

Infiltration data for different test trials were fitted to the Horton equation by using multiple nonlinear regressions to estimate f_c (the saturated soil infiltration rate), f_o (the initial rate), and k (the rate coefficient), using the observed data. The saturated rates were of greatest interest as they would apply during most of the biofilter operation during events. The estimated infiltration rates of the saturated media ranged from 0.4 to 0.8 in/hr for the hand compaction tests (initial rates were about 0.75 to 3 in/hr), and 0.4 to 0.9 in/hr for the standard proctor compaction tests, and 0.03 to 0.33 in/hr for the modified proctor compaction tests. Only the severe modified compaction level significantly affected the infiltration rates. More than 90% of the media is larger than 100 micrometers (µm), with appreciable fractions clearly in the coarse sand category, resulting in a relatively robust media with minimal compaction potential. Media with large amounts of sand do not compact as much as media having more fines because of the structural support of the sand grains. Figure 6 contains example plots of laboratory infiltration measurements fitted to the Horton equation for the hand compaction (least dense) tests.

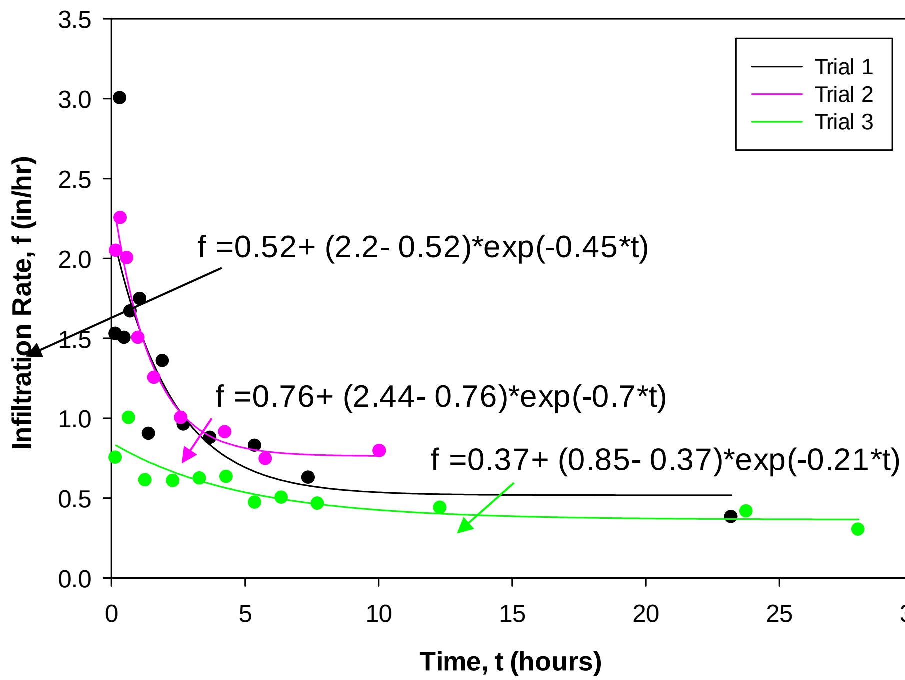
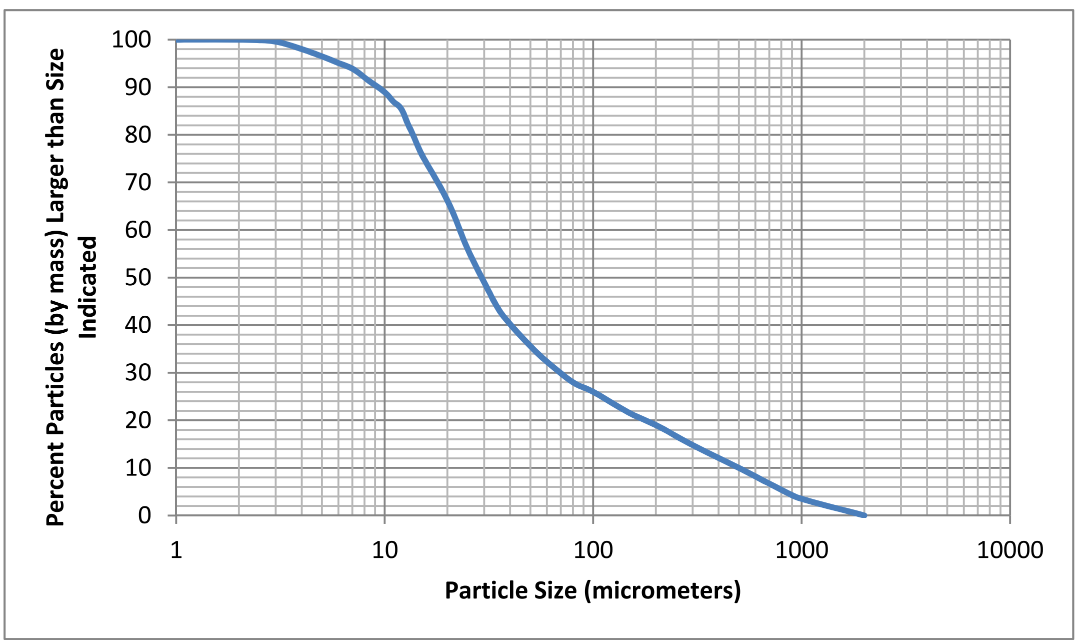


Figure 6. Kansas City biofilter media infiltration rates during column tests for hand compacted density.

Samples were also collected of inflowing water entering the biofilters for analyses. Figure 7 is a particle size distribution (PSD) plot for the 20 influent samples. The median particle size (by mass) is about 30 µm, and about 25% were larger than 100 µm. The observed median size is typical for stormwater gutter/inlet samples, but it is larger than would be expected at a stormwater outfall (the larger particles are subjected to deposition in the drainage system).



Fiugre 7. Particle size distribution for curb-cut influent stormwater samples.

The stir plate and pipette total suspended solids (TSS) method has been shown to have the highest yield and most consistent results compared to the suspended solids concentration (SSC) values as standards. The shake and pour method shows reduced values compared to the pipette and SSC methods. The relationship between the shake and pour TSS and stir plate and pipette TSS values are consistent but with about a 25% bias with the shake and pour results being less. The stir plate and pipette TSS values were about 7% less than the SSC values, as shown on Figure 8.

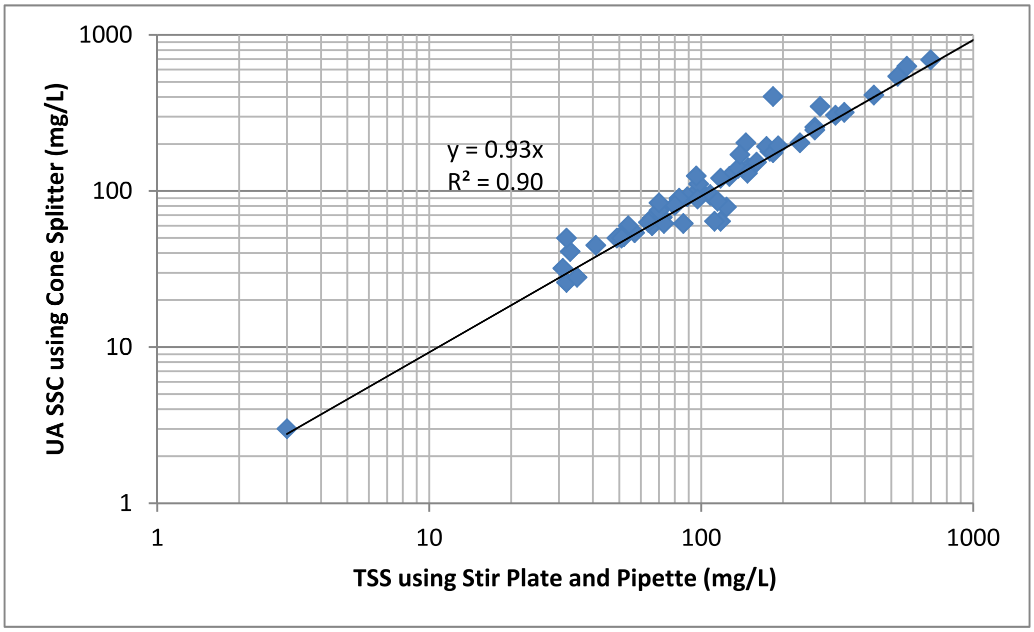


Figure 8. TSS by shake and pour versus TSS by stirring and pipetting.

The average SSC concentrations for the sites ranged from 129 to 205 mg/L, with an overall average of 174 mg/L. The overall observed SSC concentrations ranged from 26 to 711 mg/L, resulting in a COV value of about 1.0 (typical for stormwater observations). There were no significant differences in the SSC concentrations between the different locations based on the Kruskal-Wallis one way analysis of variance on ranks statistical test for the number of observations available.

Summary of Monitored and Modeled Performance of Stormwater Control Practices

The Kansas City GI demonstration project site is unique because a very large portion of the test (pilot) area receives direct treatment from many separate stormwater control devices, and the large area is being monitored to demonstrate the actual flow reductions. However, as in all retrofit installations, stormwater controls could not be placed to treat all the flows from the entire watershed area because of interferences from existing infrastructure, large trees, and surface drainage paths. Figure 9 is a map showing the subareas having stormwater control before being discharged into the combined sewer. The blanked-out areas drain into the combined sewers directly without any surface infiltration or retention control. Some areas are treated by multiple control units, with overflows from upgradient devices flowing into downgradient controls.

Figure 9. Areas receiving surface stormwater control before being discharged into the combined sewer.

The total impervious area for the area being treated is about 45%; the total impervious area for the untreated area is about 37%, indicating greater flows from the treated areas than indicated than indicated if based only on the total subareas. The calculations and modeling efforts determine the maximum amounts of stormwater control possible, reflecting the different land development characteristics in the treated and untreated subareas, and shows the sensitivity of the native soil conditions on biofilter performance.

Figure 10 compares the modeled to the monitored events that occurred after the majority of the site construction was completed. The model used a native soil infiltration rate of 1 in/hr below the biofilters, which results in reasonable predictions as shown in this figure. Lower native infiltration rates (as used in the initial design calculations) resulted in significantly increased biofilter overflow discharges, resulting in poor fits of the data.

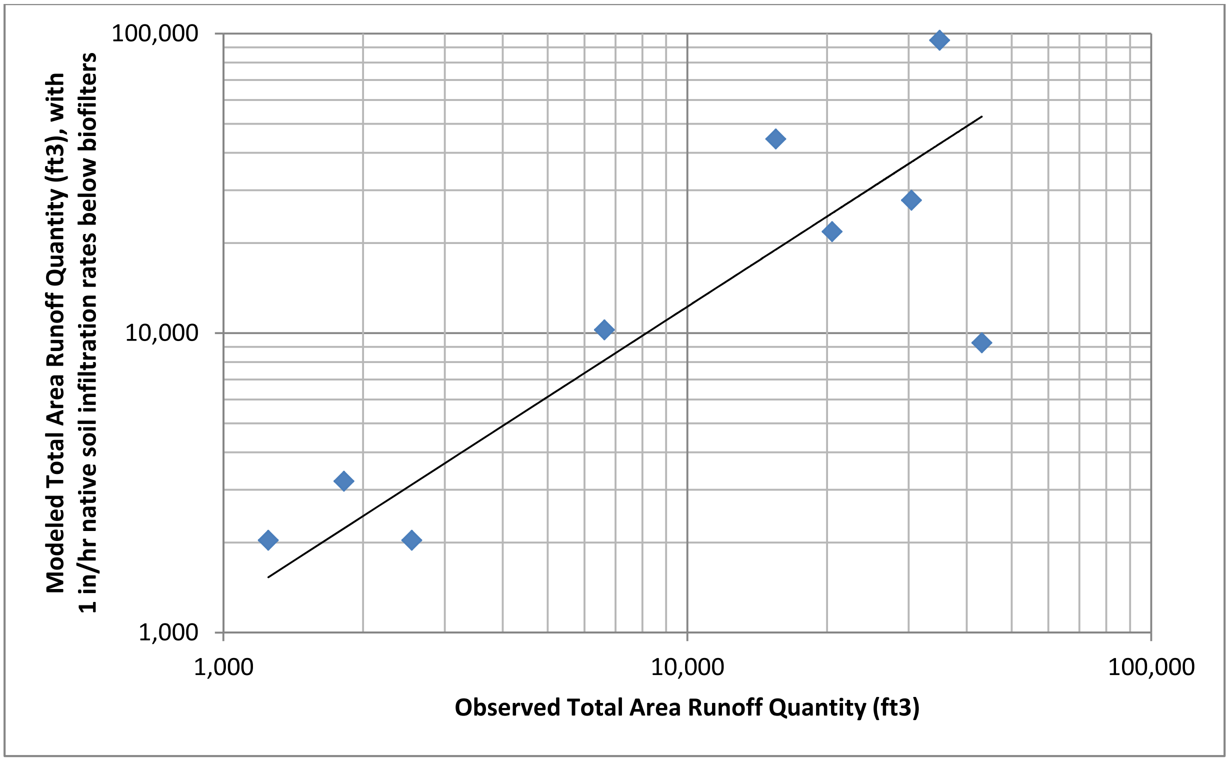
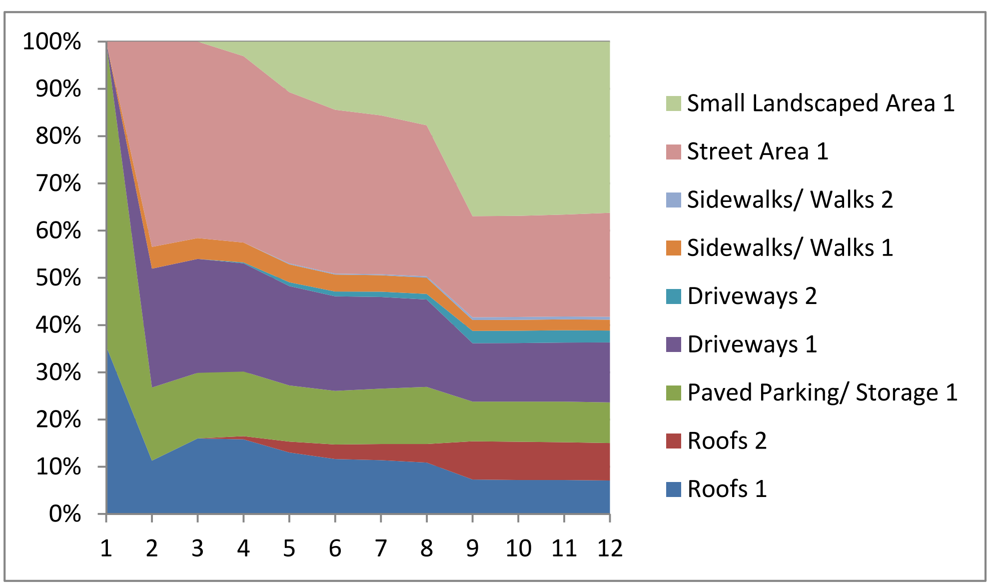


Figure 10. Modeled versus observed flows in the test (pilot) area after construction of stormwater controls.

One of the main features of WinSLAMM is its ability to calculate these source contributions for varying rain conditions. Figure 11 illustrates the source contributions for the test (pilot) area without stormwater controls, for rains ranging from 0.01 to 4 in. The sources of flows (and pollutants) vary with the rain characteristics, but the directly connected areas are most important for the small- and intermediate-sized rains, with pervious contributions becoming more important as the rains increase in size.



Event number on graphs	Rain depth (in)
1	0.01
2	0.05
3	0.10
4	0.25
5	0.50
6	0.75
7	1.00
8	1.50
9	2.00
10	2.50
11	3.00
12	4.00

Figure 11. Sources of runoff volume during different rain events (no control practices).

Table 3 summarizes the characteristics for each category of stormwater control used in the test (pilot) area, including the number of each device type and the average areas being treated by each type of control. The device areas as a percentage of drainage area are also shown and range from about 1.5 to 2% for the biofilters to 9% for the bioswale. The porous pavement sidewalks treat 100% of the sidewalk areas because they do not receive runon from adjacent areas.

Table 3. Summary of the stormwater controls constructed in the test (pilot) watershed

Design plan component	Structural description	Number of this type of stormwater control units in test (pilot) area	Drainage area to device area ratio	Device as a % of the drainage area	Drainage area for each unit (ac)	Total area treated by these devices (ac)
Bioretention	Bioretention without curb extension	24	61.8	1.6%	0.40	9.6
	Curb extensions with bioretention	28	66.1	1.5%	0.40	11.2
	Shallow bioretention	5	61.8	1.6%	0.40	2.0
Bioswale	Vegetated swale infiltrates to background soil	1	11.2	8.9%	0.50	0.5
Cascade	Terraced bioretention cells in series	5	53.0	1.9%	0.40	2.0
Porous sidewalk or pavement	With underdrain	18	1.0	100.0%	0.015	0.3
	With underground storage cubes	5	1.0	99.9%	0.015	0.1
Rain garden	Rain garden without curb extension	64	35.8	2.8%	0.40	25.6
	Curb extensions with rain gardens	8	66.0	1.5%	0.40	3.2
	Total number of control units (w/o porous pvt):	135			Total area treated:	54.4
	Total area treated (acres):	54.4				
	Area per unit:	0.40				

The calculated runoff volume reductions range from 86 to 100% for a 4-year continuous simulation period (September 2008 through October 2012) corresponding to much of the site total monitoring period. The predicted maximum water depths in the biofilters ranged from about 2 to 5 in., similar to the water depths observed. The maximum ponding times for the biofilters ranged from about 60 to 90 hours. Only a single event in the 4 years of simulation had a holding time longer than 3 days, the typical criterion for mosquito control. Only about one-third of the events likely have any surface or underdrain discharges, and these amounts would be very small compared to the untreated volumes.

Summary of Performance Production Functions for the Design and Analysis of Stormwater Management Controls

The first stormwater control that should be considered in an area is disconnecting the currently directly connected impervious areas, such as roofs and paved parking lots. The directly connected roofs in the test area contribute only about 5.8% of the total area flows, whereas the much greater area of disconnected roofs contribute about 7.2% of the annual runoff from the whole 100-acre area. The current flow contributions of all roofs in the area total about 13%. If all the roofs were directly connected, they would contribute about 31% of the total area runoff, and the runoff from the total area would increase by about 25%, a significant increase. In contrast, if the currently directly connected roofs were disconnected through a downspout disconnection program, the total roof contribution would decrease to about 9%, and the total area runoff would decrease by about 5%. Because about 85% of the existing roofs in the area are already disconnected, the benefits of controlling the remaining directly connected roofs are, therefore, limited. Directly connected roofs in the study area contribute about 4.5 times the amount of runoff per unit area as the disconnected roofs. This indicates that about 78% of the annual runoff from the disconnected roofs is infiltrated as it passes over previous areas on the way to the drainage system. Therefore, it is much less cost-effective to use roof runoff controls for the runoff from the disconnected roofs compared to runoff controls for the directly connected roofs. The benefits of disconnecting currently connected paved parking or storage areas are similar to the benefits shown above for roofs.

Private rain gardens for controlling roof runoff are being used in the residential areas in the test (pilot) area. As runoff enters the device, water infiltrates through the engineered soil or media (or natural soil, as in a rain garden). If the entering rain cannot all be infiltrated through the surface layer, the water ponds. If the ponding becomes deep, it can overflow through the broad-crested weir, or other surface outlet. The percolating water moves down through the device until it reaches the bottom and intercepts the native soil. If the native soil infiltration rate is greater than the percolation water rate, there is no subsurface ponding; if the native soil infiltration rate is slower than the percolation water rate, ponding occurs. As shown in Figure 12, as the rain garden size increases in relationship to the roof area, less water is discharged to the collection system. About 90% of the long-term runoff would be infiltrated for a rain garden that is about 20% of the roof area (similar to the monitored roof runoff rain gardens in this study).

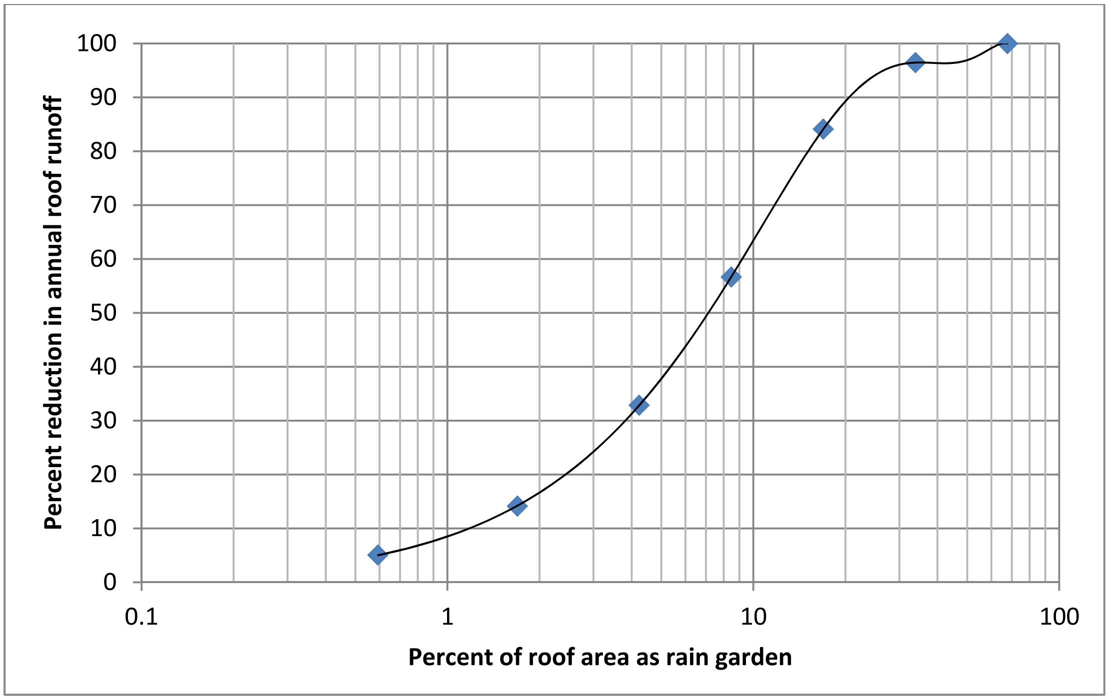


Figure 12. Percentage reduction in annual roof runoff with rain gardens.

Rain gardens 20% of the roof area would also provide about 90% runoff reductions from the directly connected roofs during the 1.4-in regulatory design storm *D*.

Biofilter performance is based on the characteristics of the flow entering the device, the infiltration rate into the native soil, the filtering capacity and infiltration rate of the engineered media fill if used, the amount of rock fill storage, the size of the device and the outlet structures for the device. WinSLAMM was used with the calibration files prepared for the Kansas City demonstration project to examine alternative biofilter and bioinfiltration device designs for the residential test (pilot) area. Four infiltration rates for the native subsurface soil were examined: 0.2, 0.5, 1.0, and 2.5 in/hr (corresponding to sandy silt loam, loam, sandy loam, and loamy sand soils, respectively). The lowest rate (0.2 in/hr) was the assumed early infiltration rate used by the design consultants for the original designs. Site surface soil measurements in the test watershed indicated 1 in/hr, or greater, infiltration rates for rains lasting 2 hours or less. Site measurements of the biofilters during storms indicated infiltration rates of the media and device at 1.8 in/hr, and modeling indicated likely subsurface rates of about 1 in/hr (or greater) to result in the observed performance during the rains (almost complete infiltration with very little overflow or subsurface underdrain discharges). The use of gravel storage is important for only the low infiltration rate conditions: once the infiltration rate is about 1 in/hr, or more, this additional storage is not needed, as far as benefiting the long-term infiltration conditions. As shown in Figure 13, for the low infiltration rates, the use of underdrains degrades the performance of the biofilters because the underdrains discharge subsurface ponding water before it can completely infiltrate (but underdrains do decrease surface ponding, a desired objective). The use of a slow underdrain (as indicated here by the SmartDrain™), results in an intermediate effect, while also decreasing periods of long surface ponding. As with the gravel storage, underdrains have very little effect on performance when the native subsurface native infiltration rate is about 1 in/hr, or greater.

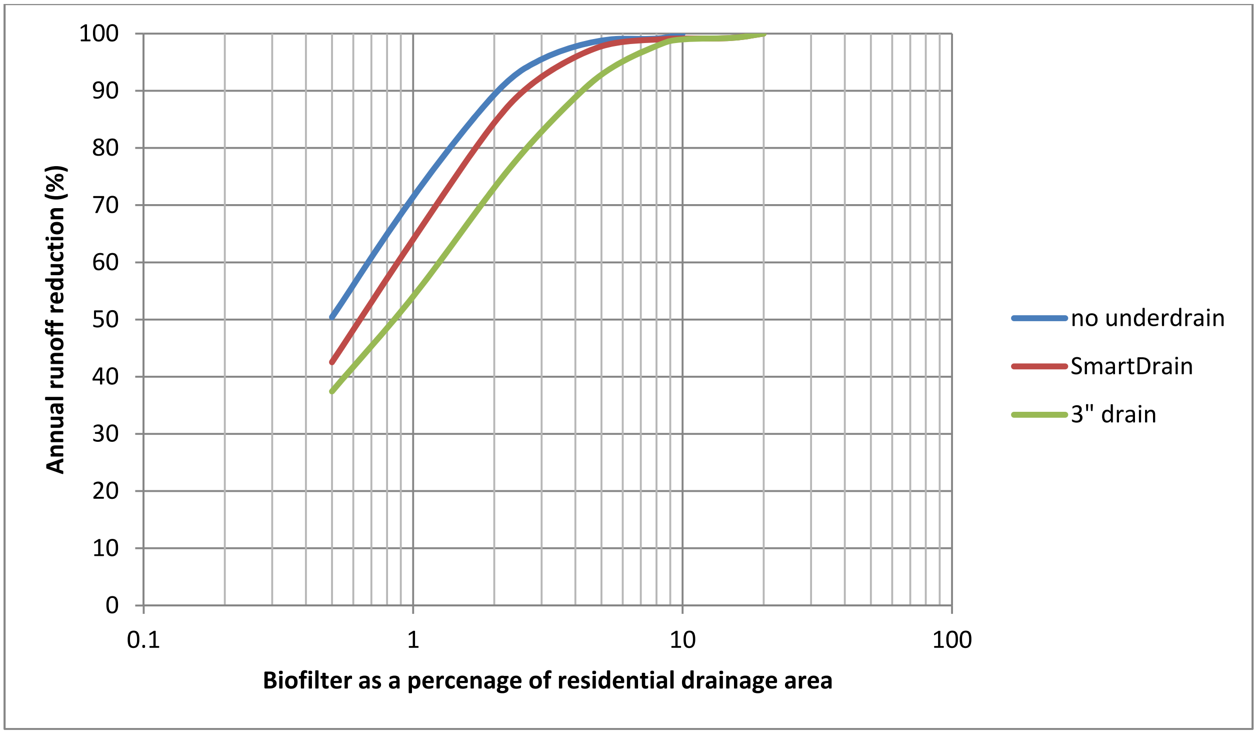


Figure 13. Effects of underdrains in biofilters on annual runoff reductions for subsurface native soil infiltration rates of 0.5 in/hr.

Biofilter media is likely to fail, resulting in very low infiltration rates with rapid and excessive particulate solids loadings. Generally, particulate loads of between 10 and 25 kilograms per square meter (kg/m^2) might lead to significantly reduced infiltration. A planted biofilter is likely to be able to incorporate this additional material into the soil as healthy plants can keep the infiltration rates at a desired level, if this accumulative load occurs over at least 10 years. However, if this load occurs in just a few years, it is likely to overwhelm the system, resulting in premature clogging. This is more of a problem for small biofilters receiving runoff having high particulate solids concentrations, such as parking lots where space is limited. Pretreatment using grass filters or swales can reduce these problems. For this study area, if the biofilters are at least 1 to 3 percent of the residential drainage area, the particulate loading is not likely to be a problem. The biofilters and bioinfiltration devices in the test (pilot) area are about 1.5 to 2% of the residential drainage areas. For the 1 in/hr subsurface infiltration rate, this size of treatment device is expected to provide about a 90% reduction in the annual flows for the areas treated, with very little overflows. The SmartDrain™ installations are expected to have <10% of the annual flows being captured by this underdrain, with the remaining flows infiltrated. These calculated conditions are all similar to the observed conditions during the brief monitoring period.

The WinSLAMM porous pavement control in version 10 has full routing calculations associated with subsurface porous media storage and also allows runon from adjacent paved areas that do not have porous pavement. Table 4 summarizes the calculated performance of porous pavements located at paved parking/storage areas. The given underlying soil is a loam soil. A conventional 3-in. perforated pipe underdrain was also assumed. As indicated, even the smallest area examined (25% of the area as porous pavement) had very good runoff volume reductions. The porous pavement was cleaned every year, restoring much of the lost surface infiltration rate capacity in this example. If the area was not cleaned, clogging would be expected in about 8 years, based on field experience. Care needs to be taken to prevent runon of stormwater having high particulate solids loads, or excessive leaf debris on the porous pavement because both conditions can result in premature failure. Porous pavements are also not recommended for areas having substantial traffic or receiving other more highly contaminated runoff (especially snowmelt in areas using deicing chemicals) to reduce groundwater contamination potential. Sidewalks and walkways, along with residential driveways are the most suitable areas for porous pavement installations.

Table 4. Porous pavement performance (paved parking and storage area; loam soil; 3-in underdrains every 20 ft.)

Porous pvt as a % of paved parking area	Rv	Volume reduction (%)	Expected habitat conditions	TSS (mg/L)	Solids discharged (lbs/yr)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.75	n/a	poor	130	812	0.21	13	21	1.3
25%	0.06	92	good	130	60	0.21	0.98	21	0.098
50%	0.05	93	good	130	58	0.32	0.94	12	0.093
100%	0.05	93	good	130	58	0.21	0.94	21	0.093

Note: Cu = copper; Rv = volumetric runoff coefficient, the ratio of runoff to rain volume; TP = total phosphorus; TSS = total suspended solids

Grass filters have broad, shallow flows. WinSLAMM calculations for grass filters are based on extensive pilot-scale and field measurements of grass swales and filters. Table 5 summarizes the performance of grass filters for controlling runoff from 2 acres of an impervious area. As the grass filters become steep, they lose some of their performance because of the faster flowing water reducing the effective infiltration rates.

Table 5. Grass filter performance for different soils and slopes

Description	Rv	% runoff volume reduction	TSS (mg/L)	Solids yield (lbs/yr)	% solids yield reduction	Peak runoff rate (cfs)	% peak runoff rate reduction	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
base conditions, no controls	0.55		100	1040		4.6		0.28	29	17	1.7
grass filter 0.5% slope	0.17	69	91	300	71	2.6	43	0.27	8.7	16	0.52
grass filter 2 to 25% slopes	0.22	60	90	376	64	3.5	24	0.26	11	16	0.67

Note: cfs = cubic feet per second; Cu = Copper; Rv = Volumetric runoff coefficient; TP = total phosphorus; TSS = total suspended solids

Grass swales are evaluated in WinSLAMM with the same general processes as for grass filters, except that concentration flows occur. Table 6 summarizes the performance of a swale for two different soil conditions. As expected, the swale water volume and pollutant reduction performance is better for the loam soil than for the silty soil.

Table 6. Grass swale performance

Description	Rv	% runoff volume reduc.	Expected habitat conditions	TSS (mg/L)	% solids yield reduc.	Solids yield (lbs/yr)	Peak runoff rate (cfs)	% peak runoff rate reduc.	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
base conditions, no controls	0.55		poor	100		1040	4.6		0.28	29	17	1.7
silty soil	0.33	40	poor	86	92	535	4.4	4	0.25	16	16	0.98
loam soil	0.16	71	fair	87	92	263	2.9	37	0.26	7.8	16	0.47

Note: cfs = cubic feet per second; Cu = Copper; Rv = Volumetric runoff coefficient; TP = total phosphorus; TSS = total suspended solids

Benefits associated with stormwater use for irrigation and other on-site uses can be calculated on the basis of site specific information. Irrigation of land on the homeowner's property was considered the beneficial use of most interest. Rain barrel/water cistern effectiveness is related to supplemental irrigation and how that matches the rainfall deficit (evapotranspiration [ET], minus rainfall) for each season. The continuous simulations used a typical one-year rain series and average monthly ET values for varying amounts of roof runoff storage. Figure 14 shows the expected roof runoff reductions for different storage tank volumes. One 35-gallon rain barrel is expected to reduce the total annual directly connected roof runoff by about 24%, if the water use could be closely regulated to match the irrigation requirements, such as with an automated irrigation system with soil moisture sensors (not likely to be used in conjunction with a few rain barrels, but more likely with a large tank that can be pressurized). If four rain barrels were used for each house, such as one at each corner of a house receiving runoff from separate roof downspouts, the total annual roof runoff volume reductions from the roofs could be as high as about 40%. A small water storage tank about 5 ft in diameter and 6 ft tall could result in about 75% total annual runoff reductions from directly connected roofs; a larger 10-ft diameter tank that is 6 ft tall could approach complete roof runoff control. The 5-ft diameter tank is also expected to provide almost complete control of runoff from the regulatory design storm *D*. These calculations are very sensitive to location as the rainfall deficit varies greatly throughout the country. The central part of the United States (including Kansas City) has a relatively large rainfall deficit with rainfall occurring at relatively optimal times for enhanced beneficial uses of roof runoff. Other areas of the county are not as suitable for this control.

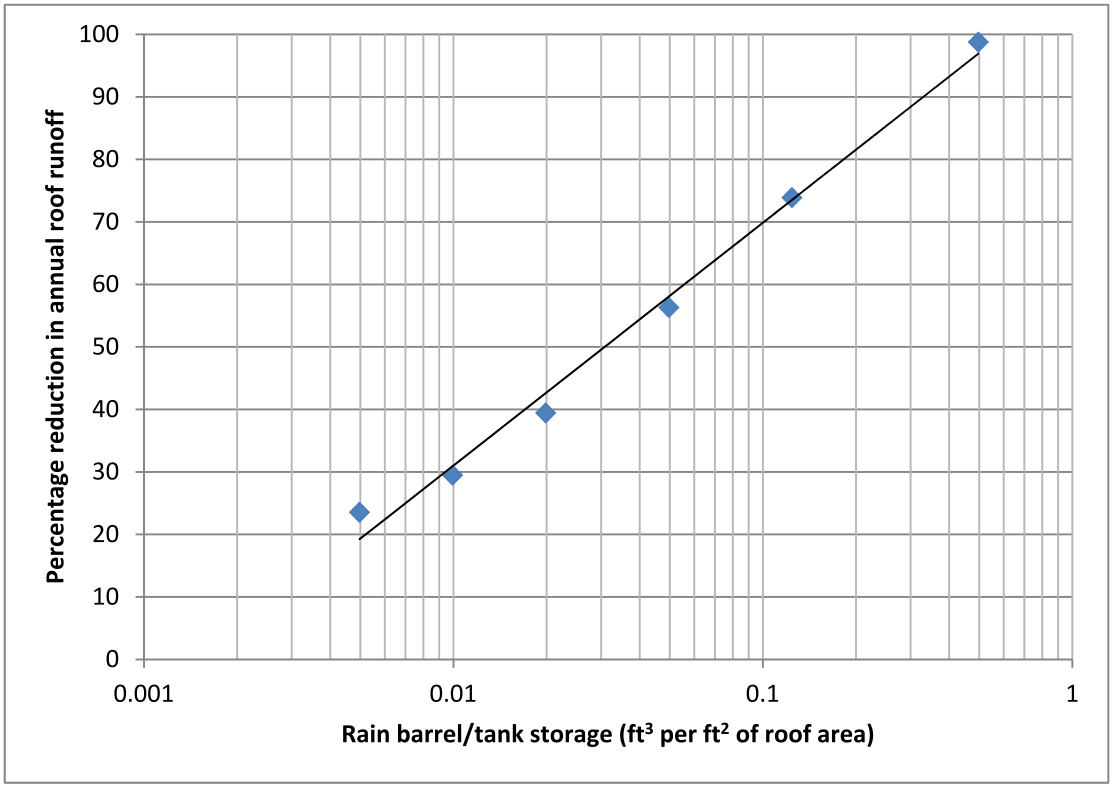


Figure 14. Percentage reduction in roof runoff with irrigation of landscaped areas in Kansas City.

For maximum use of the roof runoff to decrease runoff volumes, it is desired to irrigate at the highest rate possible, without causing harm to the plants. For a healthy lawn, total water applied (including rain) is generally about 25 mm (1 in.) of water per week, or 100 mm (4 in.) per month. Excessive watering is harmful to plants, so indiscriminate over-watering is to be avoided. Some plants can accommodate additional water. As an example, Kentucky bluegrass, the most common lawn plant in the United States, needs about 64 mm/week (2.5 in/week), or more, during the heat of the summer and should receive some moisture during the winter.

The biofilter option in WinSLAMM can be configured to represent green roofs. Basically, the green roof area is used as the area of the biofilter and no natural infiltration allowed. The only outlets include the required broad crested weir for surface overflows, underdrains, and ET. Partial roof coverage can be modeled by using a smaller area for the biofilter to represent the area dedicated to green roof processes. Table 7 summarizes the calculated performance of a green roof system for different roof coverages. The concentrations are similar for all scenarios because almost all the water is filtered by the roof media, with little being discharged to the surface overflows. The available ET resulted in about 25% reductions in runoff volume reductions. If more surface storage is provided in the green roof design and if more efficient plants are used, it is likely that these runoff volume reductions could be about double the reductions shown here.

Table 7. Calculated green roof performance

Green roof as a % of flat roof area (3-in conventional underdrains every 20 ft)	Rv	Volume reductions (%)	TSS (mg/L)	Solids discharged (lbs/yr)	Peak runoff rate (cfs)	Peak rate reductions (%)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.8	n/a	33	55	0.76	n/a	0.22	3.6	11	0.18
25%	0.71	11	24	35	0.57	25	0.17	2.4	9.8	0.14
50%	0.66	18	24	33	0.45	41	0.16	2.2	9.7	0.13
100%	0.6	25	24	29	0.38	50	0.16	2	9.7	0.12

Note: cfs = cubic feet per second; Cu = Copper; Rv = volumetric runoff coefficient; TP = total phosphorus; TSS = total suspended solids

Summary of Decision Analysis Methods to Assist in the Selection of Stormwater Control Programs

Stormwater quality models can produce copious amounts of information for large numbers of alternative management programs that contain a wide variety of individual stormwater control practices, as described by Pitt and Clark (2008). In most cases, just a few of the values are sufficient for quick comparisons. These include the overall percent runoff and particulate solids reductions, the final Rv and runoff volume, and the resulting particulate solids yields and concentrations. WinSLAMM also calculates the life-cycle costs and the expected habitat conditions of the receiving waters to be compared, in addition to flow-duration information. The use of decision analysis procedures, based on methods developed by Keeney and Raiffa (1976) with the WinSLAMM batch processor allows semi-automatic formal evaluations of alternative stormwater control programs considering multiple conflicting objectives.

This decision analysis approach has the flexibility of allowing for variable levels of analytical depth, depending on the problem requirements. The preliminary level of defining the problem explicitly in terms of attributes often serves to make the most preferred alternatives clear. The next level of analysis might consist of a first-cut assessment and ranking. Several different utility function curve types can be used with a simple additive model. Spreadsheet calculations with such a model are easily performed, making it possible to conduct several decision analysis evaluations using different tradeoffs, representing different viewpoints. It is possible there will be a small set of options that everyone agrees are the best choices. Also, this procedure documents the process for later discussion and review. Sensitivity analyses can also be conducted to identify the most significant factors that affect the decisions. The deepest level of analysis can use all the analytical information one collects, such as probabilistic forecasts for each of the alternatives and the preferences of experts over the range of individual attributes. Monte Carlo options available in WinSLAMM can also be used that consider the uncertainties in the calculated attributes for each option.

Therefore, decision analysis has several important advantages. It is very explicit in specifying tradeoffs, objectives, alternatives, and sensitivity of changes to the results. It is theoretically sound in its treatment of tradeoffs and uncertainty. Other methods ignore uncertainty and often rank attributes in importance without regard to their ranges in the problem. This decision analysis procedure can be implemented flexibly with varying degrees of analytical depth, depending on the requirements of the problem and the available resources.

Conclusions

WinSLAMM has been undergoing development and changes since the mid-1970s and now includes a wide range of options. Over the years, periodic major upgrades have occurred to take advantage of advancing computer capabilities and knowledge gained through stormwater research, and to respond to requests by users.

The expected major sources of runoff from the test area vary for different rain depth categories. A detailed land survey found that most of the homes in the test watershed already have disconnected roofs (85% of all roof areas) and that the total roof areas account for 13% of the total study area. The directly connected roofs, which make up only 2% of the study area, contribute 6% of the total annual flows. The disconnected roofs, which constitute 11% of the area, contribute 7% of the total flows. Thus complete control of the runoff from the directly connected roofs would reduce the total area runoff by only a very small amount, less than can be reliably detected by monitoring the total runoff from the area. The modeling calculations illustrate the different effects of using rain gardens, rain barrels or tanks, or simple disconnections of the directly connected roofs. The results are presented on the basis of the effects for the directly connected roofs alone; if calculated for the entire drainage area, the contribution would be less than 5%. If all the roofs were directly connected, they would then contribute 30% of the annual flows, and the outfall consequences for the whole area from these roof controls would be substantially larger.

Performance plots were prepared comparing the size of rain gardens to the roof areas to result in expected roof runoff flow reductions. Rain gardens that are 20% of the roof areas are expected to result in about 90% reductions of the total annual flow compared to directly connected roofs. This rain garden size is about 200 ft^2/house (about 20 m^2/house) which could, for example, be composed of several smaller rain gardens each located at a downspout. Reductions of 50% in the total annual flows could be obtained if the total rain garden area per house was 7% of the roof area.

Rain barrel effectiveness is related to the need for supplemental irrigation and how that matches the rains for each season, or the use of water resistant plants. The continuous simulations used a typical one-year series of typical rains and average monthly ET values for varying amounts of roof runoff storage. A single 35-gal (133 L) rain barrel is expected to reduce the total annual runoff by 24% from the directly connected roofs, if the water use can be closely regulated to match the irrigation requirements. If four rain barrels were used (such as one on each corner of a house and receiving runoff from separate roof downspouts), the total annual volume reductions could be as high as 40%. Larger storage quantities result in increased usage but likely require larger water tanks. A small tank with a 5-ft (1.5 m) diameter and 6 ft (1.8 m) high is expected to result in 75% total annual runoff reductions; a larger, 10-ft (3 m) diameter tank that is 6 ft (1.8 m) tall would approach complete roof runoff control.
The use of rain barrels and rain gardens together at a home is more effective than using either method alone: the rain barrels would overflow into the rain gardens, so their irrigation use is not quite as critical. To obtain reductions of 90% in the total annual runoff, it is necessary to have at least one rain garden/house, unless the number of rain barrels is more than 25 (or one small water tank)/house. In such a case, the rain gardens can be reduced to 80 ft^2/house (7 m^2/house).

The best combination of control options is not necessarily obvious. The CSO control program must meet permit requirements, which specify certain amounts of upland storage in the watershed. Other elements, including costs, aesthetics, improvements to streetside infrastructure, and other potential benefits, must also be considered in a decision analysis framework. Caution is needed when comparing the amount of site runoff storage provided by these upland controls to the total storage goals to meet the objective of the CSO control program (288,000 gal). As an example, storage provided at directly connected roofs needs to be discounted by a factor of about 1.4 because not all the storage is available during all rains, and because their drainage is influenced by low infiltration rates through the native soils, compared to flow controls directly connected to the combined sewers. In addition, the curbcut biofilters also have access to almost all the flows in the area, so their storage volumes are more effectively used. More significantly, if storage was provided at roofs that are already disconnected, their storage volumes would need to be discounted by a factor of 4.5 when compared to the total site storage goals, because of the existing infiltration already occurring from the disconnected roofs.

Cost-effective designs of biofilters for the area can be identified by examining the production functions provided in this report. For slowly infiltrating native subsoils (less than 1 in/hr), the use of additional subsurface storage and restricted underdrains can be very beneficial. For higher rate soils, these features have minimal benefit on performance. The biofilters being about 1.5 to 2% of the drainage area in the residential area are expected to provide about 90% long-term reductions in stormwater runoff to the combined sewer for the areas treated. However, only about half of the test (pilot) watershed received runoff control, so the maximum overall runoff volume reduction benefit can only be about 40 to 50% (compared to the median measured runoff reduction during the monitoring period of about 40%). Subsurface drainage water from the biofilters undergo substantial retention (several hours) which would benefit peak combined sewer flows, but the volume affected is relatively small.

Considerations that Affect Use of Different Stormwater Controls

Certain site conditions could restrict the applicability of some of these controls. The following comments are mostly summarized from Pitt et al. (2008a) and from preliminary research reported by others at recent technical conferences.

Sodium Adsorption Ratio (SAR)

The SAR can radically degrade the performance of an infiltration device, especially when clays are present in the media or underlying soils. Media or soils with an excess of sodium ions, compared to calcium and magnesium ions, remain in a dispersed condition, and are almost impermeable to rain or applied water. A *dispersed* soil is extremely sticky when wet, tends to crust, and becomes very hard and cloddy when dry. Water infiltration is therefore severely restricted. Dispersion caused by sodium can result in poor physical soil conditions and water and air do not readily move through the soil. An SAR value of 15 or greater indicates that an excess of sodium will be adsorbed by the soil clay particles. This can cause the soil to be hard and cloddy when dry, to crust badly, and to take water very slowly. SAR values near 5 can also cause problems, depending on the type of clay present. Montmorillonite, vermiculite, illite and mica-derived clays are more sensitive to sodium than other clays. Additions of gypsum (calcium sulfate) to the soil can be used to free the sodium and allow it to be leached from the soil in some situations, but recent laboratory tests with biofilter media at UA indicate minimal improvement.

The SAR is calculated by using the concentrations of sodium, calcium, and magnesium (in meq) in the following formula:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{(Ca^{+2} + Mg^{+2})}{2}}}$$

SAR has been documented to be causing premature failures of biofiltration devices in northern communities, such as several in the Madison, Wisconsin, area documented by University of Wisconsin soil science student projects. These failures occur when snowmelt water is allowed to enter a biofilter that has clay in the soil mixture. To minimize this failure potential, the following are recommended:

1. Do not allow snowmelt water to enter a biofilter unit. As an example, roof runoff likely has little salt and SAR problems seldom occur for roof runoff rain gardens, even in areas having large amounts of clay in the soil. However, if driveway or walkway runoff waters affected by saline deicing chemicals are discharged to these devices, problems can occur. The largest problem is associated with curb-cut biofilters or parking lot biofilters in areas with snowmelt entering these devices, especially if clay is present in the engineered backfill soil/media.
2. The biofilter media should not have any clay. It appears that even a small percent of clay in the media can cause a problem, but little information is available on the tolerable clay content of biofilter soils. Some biofilter guidance documents recommend an appreciable clay content to slow the water infiltration rate (and therefore increase the hydraulic detention time in the system) to improve pollutant capture. Instead of clay used to control the infiltration rates, restrictive underdrains, such as the SmartDrain™, should be used. Guidance documents recommending fines in the biofilter mixture are usually from areas having mild climates with little or no snowmelt (and deicing chemical use).
3. The most robust engineered soil mixtures used in biofilters tend to be mixtures of sand and an organic material (such as compost, if nutrient leaching is not a concern, or Canadian peat for a more stable material having little nutrient leaching potential). Other mixtures of biofilter media can be used targeting specific pollutants, but these are usually expensive and likely only appropriate for special applications.
4. If a suitable soil mixture not having clay (should be less than 3% based on preliminary information), and if snowmelt water will affect the system, biofilters should not be used in the area if high levels of control are needed. As noted above, rain gardens receiving only roof runoff might be suitable in most situations because of the absence of excessive sodium in the runoff water.

The Kansas City biofilter media was tested and has about 10% clays. This amount of clay may affect the infiltration rates during snowmelt periods and for some time after if sodium accumulates in the biofilter from snowmelt.

Clogging of Infiltration Devices

The designs of infiltration devices need to be checked for their clogging potential. For example, a relatively small and highly efficient biofilter (especially in an area having a high native infiltrating rate) could capture a large amount of sediment. Having a small surface area, this sediment would accumulate rapidly over the area, possibly reaching a critical clogging load early in its design lifetime. Therefore, the clogging potential can be calculated on the basis of the predicted annual discharge of suspended solids to the biofiltration device and the desired media replacement interval. Infiltration and bioretention devices can show significantly reduced infiltration rates after about 2 to 5 lb/ft^2 (10 to 25 kg/m^2) of particulate solids have been loaded (Clark 1996, 2000; Urbonas 1999). Deeply rooted vegetation and a healthy soil structure can extend the actual life much longer. However, abuse (especially compaction and excessive siltation) can significantly reduce the life of the system. If this critical load accumulates relatively slowly (taking about 10 or more years to reach this total load) and if healthy vegetation with deep roots are present, the infiltration rate might not significantly degrade because of the plant's activities in incorporating the imported sediment into the soil column. If this critical load accumulates in just a few years, or if healthy vegetation is not present, the premature failure from clogging could occur. Therefore, relatively large surface areas might be necessary in areas having large sediment content in the runoff, or suitable pretreatment to reduce the sediment load before entering the biofilter or infiltration device would be necessary.

For some of the calculated Kansas City biofilter size options, the sediment loading rates are high (mostly because of treatment of relatively large areas compared to the size of the biofilters), which could result in premature failure if the minimum sizes were used according to infiltration goals alone. Therefore, a larger area might actually be needed to prevent premature failure because of clogging. The following considerations apply to infiltration/biofiltration devices to minimize clogging failure:

1. Use a sufficient infiltration area to enable at least 10 years before the critical sediment loading (10 to 25 kg/m^2) occurs and maintain a healthy, deep-rooted plant community to incorporate the sediment into the soil horizon.
2. Use pretreatment to reduce the sediment load entering a biofilter to reduce the TSS concentrations to match the desired maintenance or clogging interval. Using a grass filter/grass swale before a biofilter can significantly reduce the loading to the device, extending the operational life.

The characteristics for the Kansas City biofilters in the test area indicate that most are likely sufficiently sized to result in minimal clogging potential (several years at least to reach 10 kg/m^2, and >10 years to reach 25 kg/m^2 total loading). However, there might be a desire to reduce the sizes appreciably during future construction to reduce costs, which could result in early failure.

Groundwater Contamination Potential and Over-Irrigation

The potential for infiltrating stormwater to contaminate groundwater is dependent on the concentrations of the contaminants in the infiltrating stormwater and how effective those contaminants might travel through the soils and vadose zone to the groundwater. Source stormwater from residential areas are not likely to be contaminated with compounds having significant groundwater contaminating potential (with the exception of high salinity snowmelt). In contrast, commercial and industrial areas are likely to have greater concentrations of contaminants of concern that might adversely affect the groundwater. Therefore, pretreatment of the stormwater before infiltration might be necessary, or treatment media can be used in a biofilter or as a soil amendment to hinder the migration of the stormwater contaminants of concern to the groundwater. Again, these concerns are usually more of a problem in industrial and commercial areas than in residential areas.

Pitt et al. (2010a) summarized prior research on potential groundwater contamination. Table 8 can be used for initial estimates of contamination potential of stormwater affecting groundwater. This table includes likely worst case mobility conditions using sandy soils having low organic content. If the soil is clayey or has a high organic content (or both), most of the organic compounds would have less mobility than shown. The abundance and filterable fraction information is generally applicable for warm-weather stormwater runoff at residential and commercial area outfalls. The concentrations and detection frequencies would likely be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas), with greater groundwater contamination potential.

Therefore, groundwater contamination potential of infiltrating stormwater can be reduced by:

1) Careful placement of the infiltrating devices and selection of the source waters. Most residential stormwater is not highly contaminated with the problematic contaminants, except for chlorides associated with snowmelt.
2) Commercial and industrial area stormwater would likely need pretreatment of reduce the potential of groundwater contamination associated with stormwater. The use of specialized media in the biofilter, or external pretreatment might be needed in these other areas.

The Kansas City test area is expected to have minimal groundwater contamination potential because it has relatively uncontaminated stormwater, and the soil has appreciable clay. However, snowmelt salts could be a problem if deicing salt use is not restricted in the area.

Table 8. Groundwater contamination potential for stormwater pollutants post-treatment

Compound class	Compounds	Subsurface injection with minimal pretreatment	Surface infiltration with sedimentation (along with sorption, if possible)*	Surface infiltration and no pretreatment*
Nutrients	Nitrates	Low/moderate	Low/moderate	Low/moderate
Pesticides	2,4-D	Low	Low	Low
	γ-BHC (lindane)	Moderate	Low	Moderate
	Atrazine	Low	Low	Low
	Chlordane	Moderate	Low	Moderate
	Diazinon	Low	Low	Low
Other organics	VOCs	Low	Low	Low
	1,3-dichlorobenzene	Low	Low	**High**
	Benzo(a) anthracene	Moderate	Low	Moderate
	Bis (2-ethyl-hexyl) phthalate	Moderate	Low	Moderate
	Fluoranthene	Moderate	Moderate	**High**
	Naphthalene	Low	Low	Low
	Phenanthrene	Moderate	Low	Moderate
	Pyrene	Moderate	Moderate	**High**
Pathogens	Enteroviruses	**High**	**High**	**High**
	Shigella	Low/moderate	Low/moderate	**High**
	P. aeruginosa	Low/moderate	Low/moderate	**High**
	Protozoa	Low	Low	**High**
Heavy metals	Cadmium	Low	Low	Low
	Chromium	Low/moderate	Low	Moderate
	Lead	Low	Low	Moderate
	Zinc	Low	Low	**High**
Salts	Chloride	**High**	**High**	**High**

Source: Modified from Pitt et al. 1994

Note: Overall contamination potential (the combination of the subfactors of mobility, abundance, and filterable fraction) is the critical influencing factor in determining whether to use infiltration at a site. The ranking of these three subfactors in assessing contamination potential depends of the type of treatment planned, if any, before infiltration.

* Even for those compounds with low contamination potential from surface infiltration, the depth to the groundwater must be considered if it is shallow (1 m or less in a sandy soil). Infiltration might be appropriate in an area with a shallow groundwater table if maintenance is sufficiently frequent to replace contaminated vadose zone soils.

Retrofitting and Availability of Land

Most of the control options being used in GI approaches to minimize combined sewer problems are retrofitted in existing urban areas. Their increased costs and availability of land can be detrimental in developing highly effective control programs. The selection and construction of stormwater controls at the time of development (rather than retrofits) is usually much more cost-effective and can provide a higher level of control. However, many controls can be retrofitted into existing areas. Practices that can usually be easily retrofitted get the most attention in stormwater management program in existing areas. Table 9 summarizes some of the problems associated with different stormwater retrofitting options in combined sewer areas.

Table 9. Retrofitting problems for different stormwater management options

Controls	Ability to retrofit	Land requirements
Roof Runoff Controls		
Rain Gardens	Easy in areas having landscaping	Part of landscaping area
Disconnections	Suitable only if the adjacent pervious area is adequate (mild slope and long travel path)	Part of landscaping area
Rain Barrels and Water Tanks	Easy, if placed close to a building, or underground large tanks	Supplements landscaping irrigation, no land requirements
Pavement Controls		
Disconnections	Suitable only if adjacent pervious area is adequate (mild slope and long travel path)	Most large, paved areas are not adjacent to suitable large turf areas, except for schools; no additional land requirements, but land is needed.
Biofiltration/bioinfiltration	Easy if one can rebuild parking lot islands as bioinfiltration areas; perimeter areas also possible (especially good if existing stormwater drainage system can be used to easily collect overflows)	Part of landscaped islands in parking areas, along parking area perimeters, or sacrifice some existing parking areas.
Porous Pavement	Difficult as a retrofit must replace complete pavement system; possible if during rebuilding effort	Uses parking area
Street Side Drainage Controls		
Grass Swales	Difficult to retrofit. Suitable if existing swales are to be rebuilt.	Part of street right of way
Curb-cut Biofilters	Difficult to retrofit, but much easier than simple swales. Usually built to work with existing drainage system. Can do extensions into parking lanes/shoulders to increase areas.	Part of street right of way, but can be major nuisance during construction and can consume street side parking. Can be used to rebuild street edge and improve aesthetics.

The range of difficulties and land requirements varies, mostly depending on available opportunities. In some communities, extensive retrofitting is occurring, including installing curb-cut biofilters, during scheduled street improvement projects. These can also be installed during scheduled repaving and sidewalk repairs that usually occur in many areas every few decades. Rain gardens are usually installed by the homeowners with no cost to the city. Many areas have organized efforts encouraging these, for example. Redevelopment and new construction periods are the most suitable times for installing many of these controls to have the least interferences with residents and for the least costs.

2. Description of WinSLAMM, the Source Loading and Management Model

WinSLAMM was developed starting in the mid-1970s as part of early EPA-funded street cleaning and receiving water projects in San Jose (Pitt 1979) and Coyote Creek, California (Pitt and Bozeman 1982). The primary purpose of the model is to identify sources of urban stormwater pollutants and to evaluate the efficiency of control practices. During the mid-1980s, the model was expanded to include more management options beyond street cleaning. The Nationwide Urban Runoff Program (NURP) projects (USEPA 1983) provided a large data set for models, specifically, the Alameda County, California (Pitt and Shawley 1982); Bellevue, Washington (Pitt and Bissonnette 1984); and Milwaukee, Wisconsin (Bannerman et al. 1983) projects were used in major expansions of WinSLAMM. Research funded by the Ontario Ministry of the Environment (Ottawa) (Pitt 1987) and the Toronto Area Watershed Management Strategy study in the Humber River (Pitt and McLean 1986) also provided much information on bacteria sources in urban areas. During the mid-1980s, the model started to be used by the Wisconsin Department of Natural Resources (DNR) in its Priority Watershed Program (Pitt 1986). The first Windows version of the model was developed in 1995, and version 10 was recently released. The model is continuously being updated according to user needs and new research (recent and current support from Stormwater Management Authority of Jefferson County, Alabama; the Tennessee Valley Authority, Economic Development group; Wisconsin DNR; U.S. Geological Survey (USGS); Contech Stormwater Solutions; and Hydro-International, for example). Version 10 includes drag and drop watershed elements and more complete flow and particle size routing components, enabling more accurate serial evaluations of stormwater controls in complex arrangements.

Over the years, WinSLAMM has been extensively revised and expanded and now includes a wide range of capabilities. The following lists several important model features:

- The model can evaluate a long series of rain events, usually 1 to 10 years of typical rains are used, but as many decades of rain data can also be evaluated (limitations due to available memory and complexity of the model).
- The model is based on actual field data. Street dirt accumulation and washoff equations and direct runoff from paved surfaces during all rains are used, for example, based on many thousands of actual measurements.
- The effects of compacted urban soils are also considered.
- Uncertainties of many modeling parameters are represented by built-in Monte Carlo components.
- Costs of control practices can be directly calculated and considered in model runs.
- Runoff flow-duration probability distributions and associated receiving water biological conditions are calculated on the basis of site conditions and the control measures being used.
- The model can be interfaced with several other models for more detailed drainage system and receiving water evaluations.

Prior descriptions of WinSLAMM have been presented during Engineering Foundation conferences and in the Urban Water Modeling Conference series, along with other publications (Pitt 1986, 1997, 1999; Pitt and Voorhees

2002 for example). The model website (http://www.winslamm.com/) also contains further model descriptions and references.

The effectiveness of the control practices in WinSLAMM are calculated using the actual sizing and other attributes of the devices, the source area or outfall location characteristics, and the calculated runoff characteristics. The model does a complete mass balance and routing of water volume and particulate mass, considering the combined effects of all controls. Hydraulic and particle size routing occurs for each device individually, and serial effects of multiple devices are accurately considered in version 10 of the model. The effects of the sedimentation controls are calculated using modified Puls hydraulic routing with surface overflow rate particulate routing. The performance of wet ponds has been verified by extensive monitoring of several locations (Wisconsin DNR and USGS, with extensive documentation at http://unix.eng.ua.edu/~rpitt/SLAMMDETPOND/WinDetpond/WinDETPOND%20user%20guide%20and%20documentation.pdf). The infiltration and biofiltration devices use a combination of hydraulic routing with infiltration and evaporation losses, plus any pumped withdrawals, and have been verified using both small- and large-scale field tests conducted by the USGS (Selbig and Bannerman 2008; Selbig and Balster 2010) and the Kansas City EPA demonstration monitoring (Pitt and Voorhees 2010; Struck 2009), for example. ET losses are also included in the performance calculations. Underdrain filtering is based on extensive tests of media filtration (Pitt et al. 2010b; Sileshi 2013). Grass swale performance is calculated on the basis of extensive laboratory and outdoor testing of particulate trapping of shallow flowing water and infiltration losses (Kirby et al. 2005; Johnson et al. 2003; Nara and Pitt 2005). Porous pavement performance is calculated on the basis of infiltration losses and clogging effects. Street cleaning and catchbasin benefits are based on extensive EPA research and newer updated research that have examined modern equipment. Hydrodynamic devices are based on the basic sedimentation processes and have also been verified by extensive tests conducted by the USGS and the Wisconsin DNR, plus continued tests at UA.

As noted, WinSLAMM conducts a continuous water mass balance for every storm in the study period. As an example, for rain barrels, water tanks or cisterns, capturing roof runoff, the model fills the available storage during rains. Between rains, the storage tank is drained according to the water demands for each month. If the tank is almost full from a preceding close rain (and not enough time was available to drain the storage tank), excess water from the event would be discharged to the drainage system after the tank fills. Curb-cut biofilters along a street are basically a cascading swale system where the site runoff is allowed to infiltrate. If the runoff volume is greater than the capacity of the rain gardens, the excessive water is discharged into the drainage system, or possibly additional downgradient controls. When evaluated together, the cisterns treat the roof runoff first, but the excess water is discharged to the curb-cut rain gardens for infiltration. The continuous simulation drains the devices between events, depending on the interevent conditions.

Basic Model Setup for Site Characteristics

The first step in setting up a WinSLAMM analysis is to identify the rain and the calibrated parameter files to be used, as shown in Figure 15. The rain file describes the series of rains to be considered in the analysis. In this example shown below, the Kansas City rain file was selected, as shown in Figure 16. The 10 years of rain data from 1990 through 1999 are selected from the complete series. During this period, 920 rains occurred that were 0.01 in., or larger. The largest rain observed in this period was 3.79 in. WinSLAMM has a utility that creates rain files from National Oceanic and Atmospheric Administration data sources. EarthInfo (Santa Monica, California) CDs of these data are most convenient, for example, having many decades of rainfall records from throughout the United

States. Figure 15 also shows several other selections for the calibrated parameter files. These describe the rainfall-runoff relationships for the different source areas for the different land uses. These relationships are based on the small storm hydrology concepts described by Pitt (1987) and summarized in a chapter in the urban water systems modeling monograph series (Pitt 1999). The pollutant probability distribution files and the particulate solids concentration files are based on field data, also summarized by Pitt et al. (2005a, 2005b) in chapters published in the urban water systems modeling monograph series. These files contain probability distributions of the expected particulate-bound pollutant concentrations and the filtered pollutant concentrations for the different source areas. Monte Carlo sampling methods can be optionally used to randomly vary these characteristics for different events, as observed during field monitoring. The street dirt accumulation and washoff mechanisms are specifically modeled, as described by Pitt (1987; Pitt et al. 2005c). Delivery functions are used to describe deposition and transport of the particulates through the storm drainage systems and are again based on field observations.

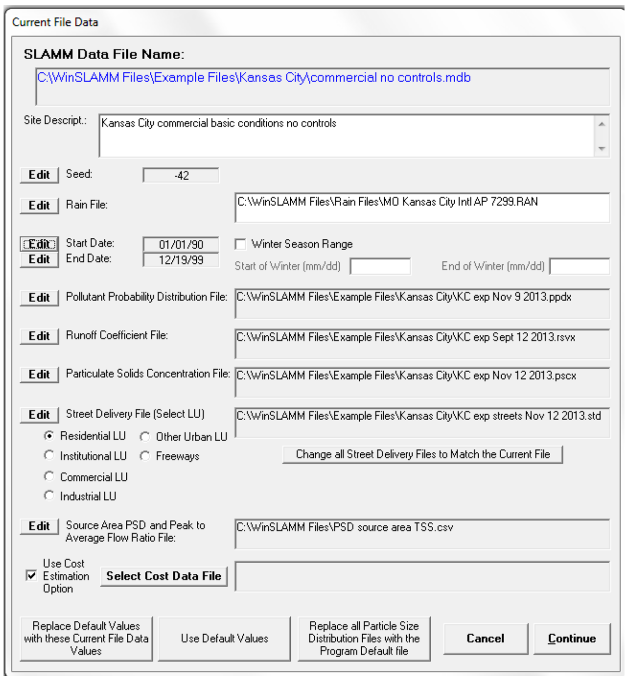


Figure 15. Example parameter files selection.

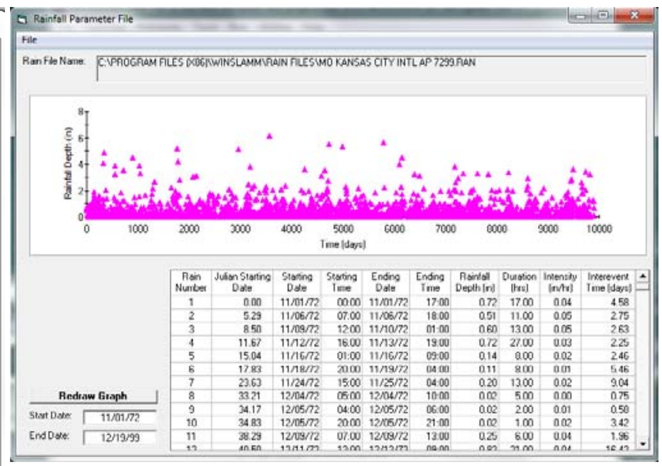


Figure 16. Scatterplot of Kansas City, Missouri, rain file.

Land development characteristics describing local site conditions of the study area are used by WinSLAMM to calculate expected runoff characteristics. Figure 17 is a screenshot for entered site conditions for the commercial example being used in this demonstration along with the drag and drop map now used in version 10 of the model; Figure 18 contains screenshots describing the five source areas used in this example. It has two roof area types—one paved parking area, and two landscaped areas. The soils are described as silty in texture with normal compaction representing typical urban activities. If adversely compacted, the moderately and severely compacted options can be selected (Pitt et al. 2009). Bochis et al. (2008) describe land use patterns and development characteristics, including the procedures used to collect that needed information.

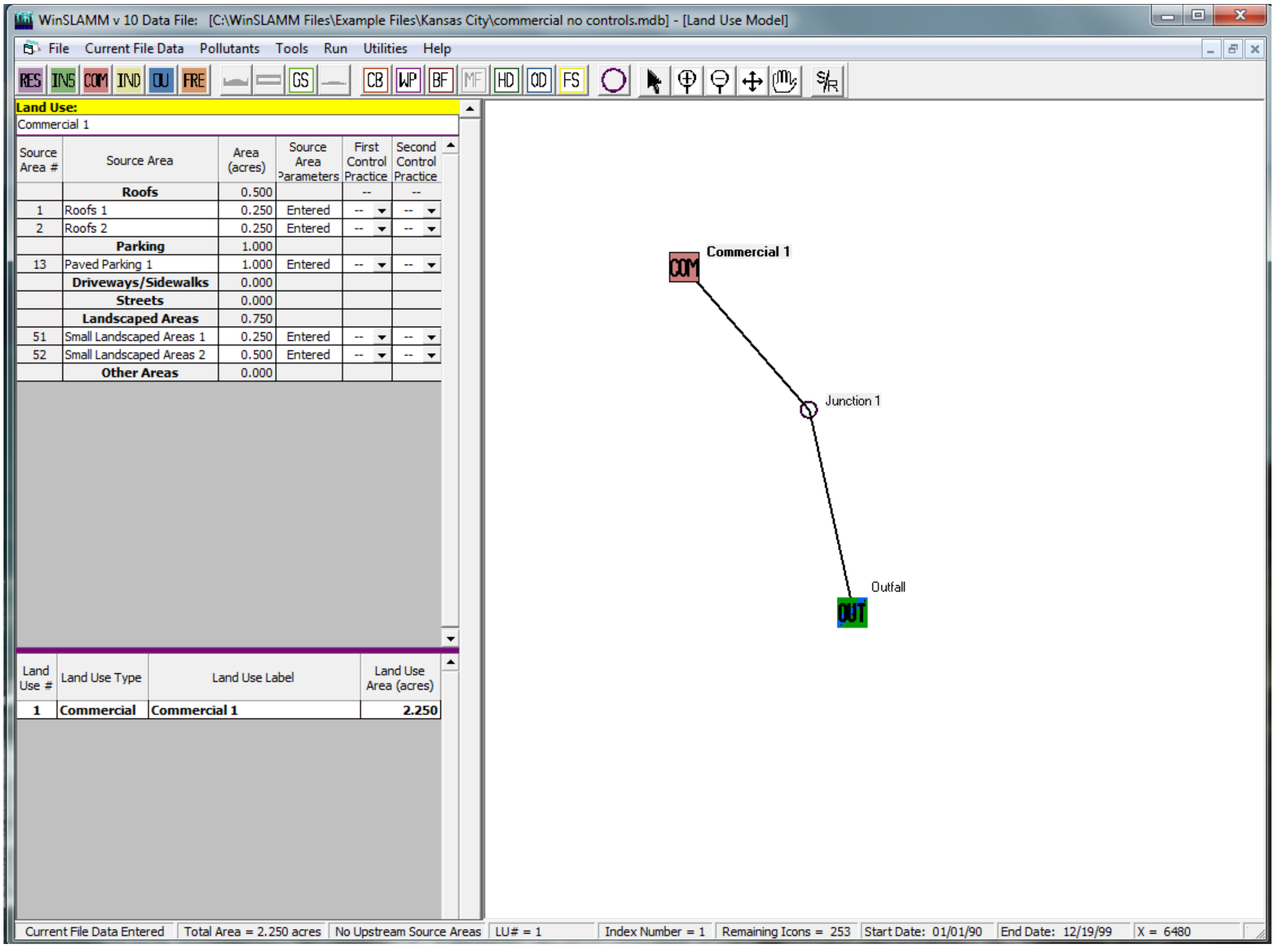
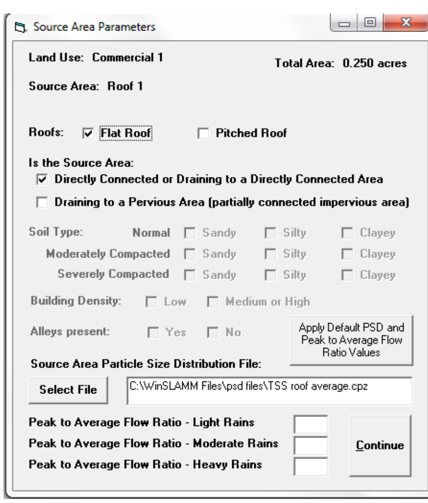
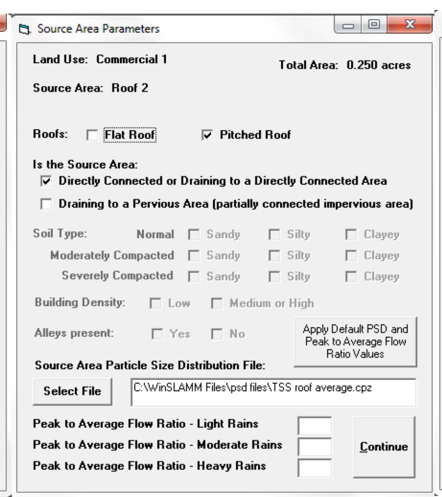


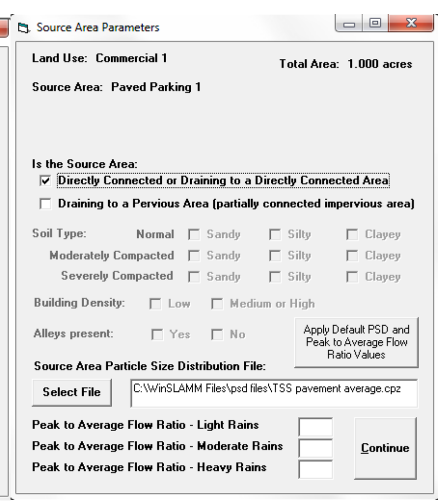
Figure 17. Input screen and drag and drop layout map for commercial example.



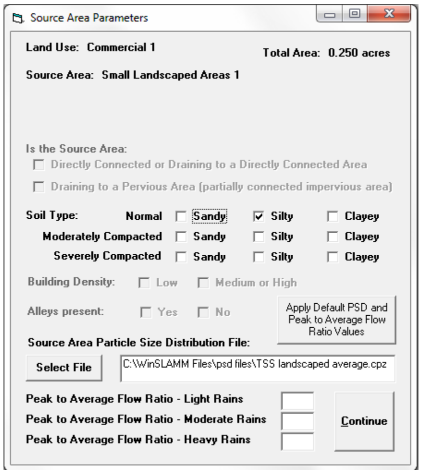
Roof 1 (directly connected flat roof)



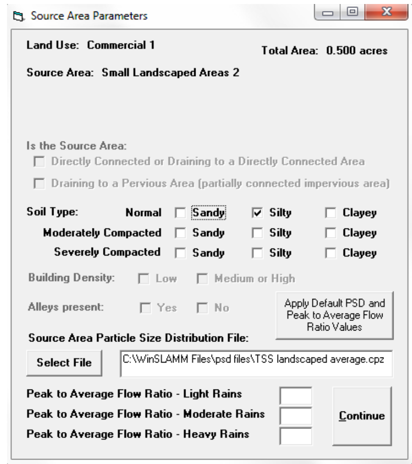
Roof 2 (directly connected pitched roof)



Paved parking/storage area 1 (directly connected)



Small landscaped area 1 (filter strip area)



Small landscaped area 2 (other pervious areas)

Figure 18. Source area characteristics for the example problem.

Figure 19 shows the pollutant selection form. The pollutants shown are those that are included in the pollutant probability distribution file and are calibrated for the area of interest. In this example, particulate solids (SSC or TSS, depending on the laboratory method used in the monitoring activities; for this file, TSS are used), total phosphorus, and total copper have been selected as examples. As noted, it is possible to select the particulate-bound or dissolved forms of the pollutants separately, or the total concentrations. Special studies have focused on urban area bacteria and for polycyclic aromatic hydrocarbons (PAHs), for example, and those constituents can be described in the pollutant probability distribution file and then selected in this form.

Figure 20 illustrates the form that can be used to select the main output formatting desired. If not selected, option 1 is used, which gives a brief summary of the calculated results along with data for all events. The summary output form (always shown) presents the basic information. After the calculations and when viewing the output summary form, it is possible to view the other output forms by having the data reformatted, if desired, without having to rerun the model scenario.

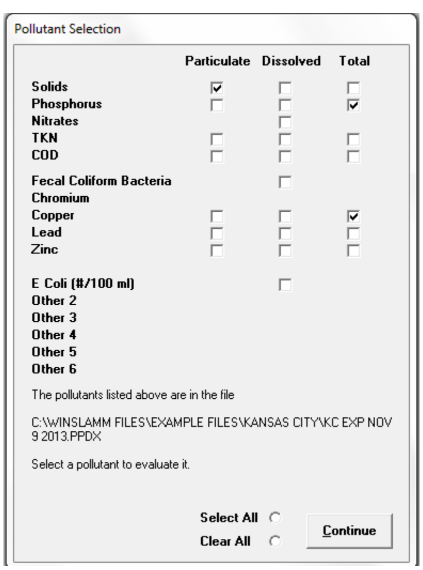


Figure 19. Selection of pollutants to be evaluated.

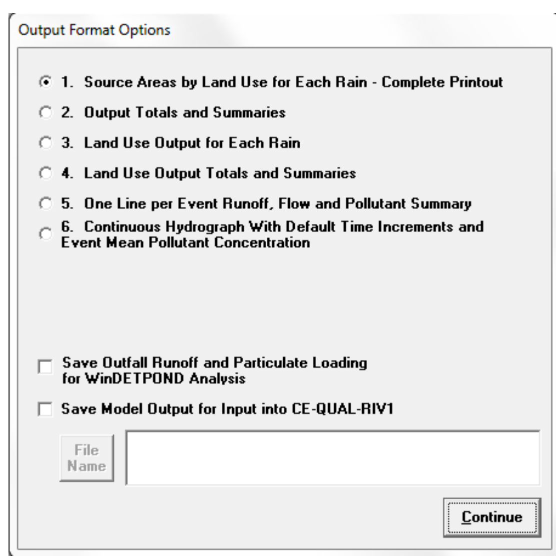


Figure 20. Selection of output formats.

Base Analyses with No Stormwater Controls

When this basic information is entered in the model, the model scenario is executed and the results are presented in different ways. Figure 21 is the summary output screen that is always displayed when the model run is completed. This screen shows runoff quantity, TSS, and pollutant conditions. If selected, different costs associated with described stormwater controls are also shown, along with expected receiving water habitat conditions (based on the Center for Watershed Protection's Impervious Cover Model). This form also has a selection to show the flow-duration curves for the base conditions and with the stormwater controls for the area, as shown in Figure 22. This base example has no stormwater controls, so the two plots are identical. It is also possible to see these data in much higher resolution by selecting any of the tabs across the top of the form.

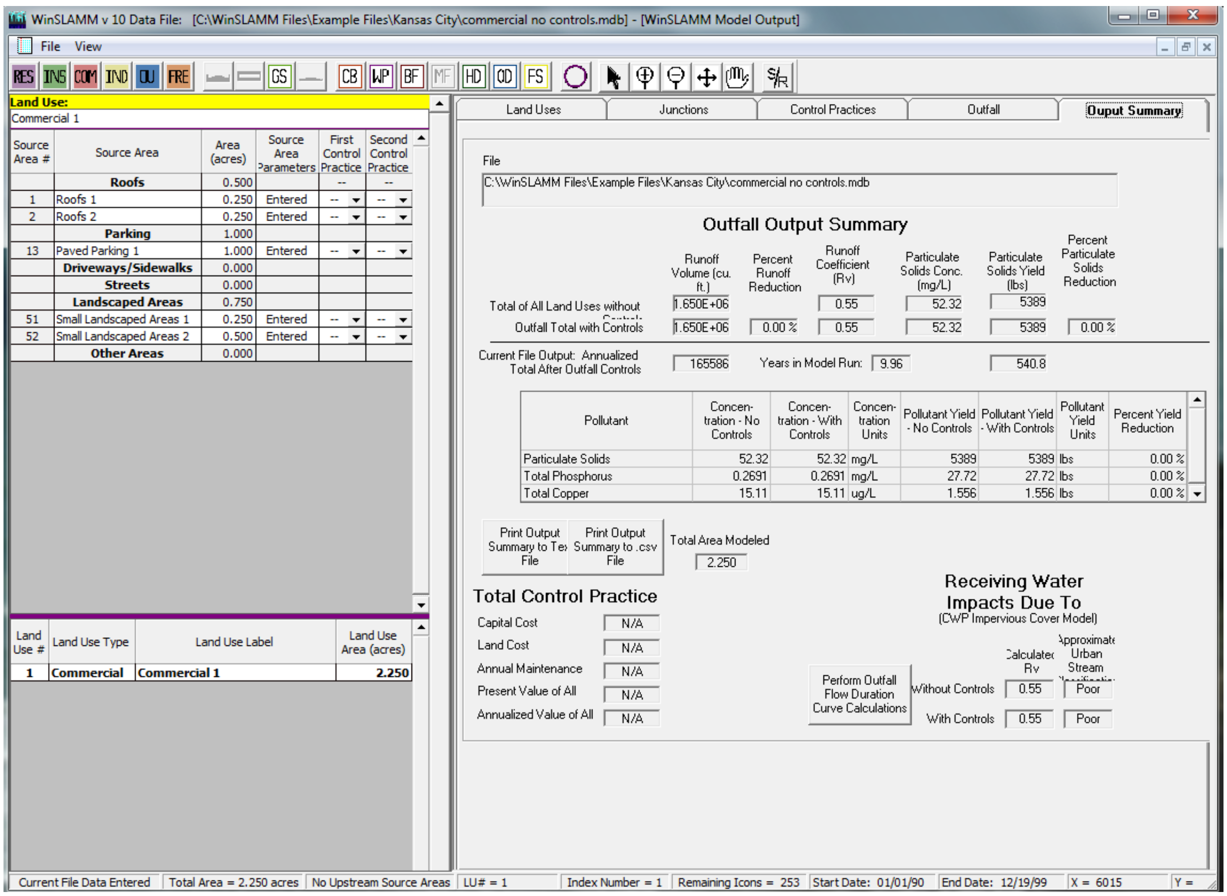


Figure 21. Basic Summary Screen.

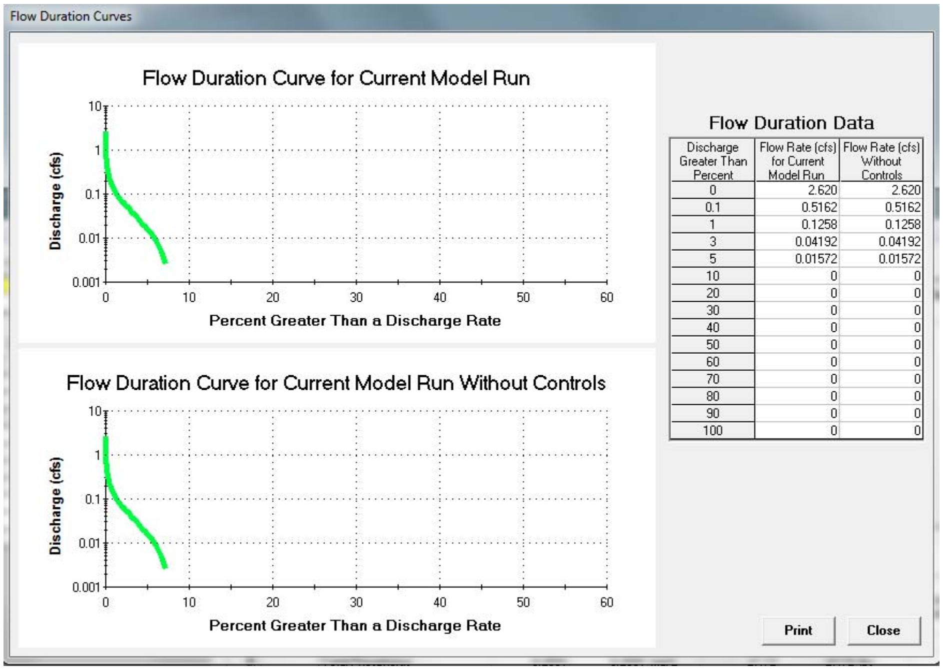
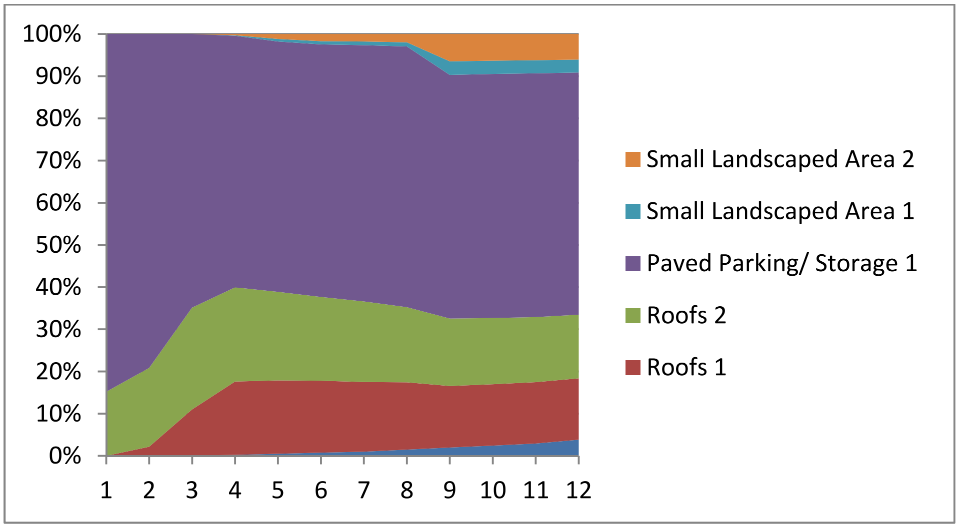


Figure 22. Base flow duration plot.

Sources of Pollutants of Interest

One of the important uses of WinSLAMM is to calculate the sources of the flows and pollutants of interest for the study area under different rain conditions. Figure 23 is a simple area plot created in Excel from imported values from WinSLAMM. The rain file used for this analysis contains only 12 events, ranging from 0.01 to 4.0 in.

This plot is for runoff volume sources and indicates that the large paved parking/storage area is the major runoff source for all events (from about 85% in the smallest rains to about 55% in the largest rains). The runoff contributions from the roofs combined range from about 15 to 35%, while the landscaped areas start to contribute flows after only about 0.25 in. and reach their maximum contributions after 2.0 in., approaching about 10% of the total flows from the area. This type of plot can be created for each of the constituents selected in the model run and indicate locations for the most effective source controls, or if the sources are too diverse, if outfall or drainage system controls should be stressed. For this example, it is not surprising that the paved parking/storage areas should receive the most attention, followed by the directly connected roofs.



Rain #	Rain depth (in)
1	0.01
2	0.05
3	0.10
4	0.25
5	0.50
6	0.75
7	1.0
8	1.5
9	2.0
10	2.5
11	3.0
12	4.0

Key for rain numbers on area contribution plot.

Figure 23. Flow sources for different rains.

Summary of WinSLAMM Description and use for GI Projects

Over the years, WinSLAMM has been extensively revised and expanded and now includes a wide range of capabilities, including its ability to evaluate stormwater management options using a long series of rain event data, especially important for evaluating combined sewer issues and GI issues. The effectiveness of the control practices in WinSLAMM are calculated on the basis of the actual sizing and other attributes of the devices, the source area or outfall location characteristics, and the calculated runoff characteristics. The model does a complete mass balance and routing of water volume and particulate mass, considering the combined effects of all controls. Hydraulic and particle size routing occurs for each device individually, and serial effects of multiple devices are now calculated in version 10.

WinSLAMM conducts a continuous water mass balance for every storm in the study period. As an example, for rain barrels, water tanks or cisterns, for harvesting roof runoff for later irrigation or other beneficial uses, the model fills the available storage during rains. Between rains, the storage tank is drained according to the water withdrawal use for each month. If the tank is almost full from a preceding close rain (and not enough time was available to drain the storage tank), excess water from the event would be discharged to the drainage system after the tank fills. Curb-cut rain gardens/biofilters along a street are basically a cascading swale system where the site runoff is allowed to infiltrate. If the runoff volume is greater than the capacity of the rain gardens, the excessive water is discharged into the drainage system, or possibly additional downgradient controls. When evaluated together, the cisterns treat the roof runoff first, but the excess water is discharged to the curb-cut biofilters for infiltration. The continuous simulation drains the devices between events, according to the interevent conditions. The first step in setting up a WinSLAMM analysis is to identify the rain and the calibrated parameter files to be used. The rain file describes the series of rains to be considered in the analysis. The 10 years of Kansas City rains from 1990 through 1999 had 920 rains that ranged from 0.01 to 3.79 in., with an average total annual rainfall of about 35 to 40 in. Land development characteristics describing local site conditions of the study area are used by WinSLAMM to calculate expected runoff characteristics. One of the important features of WinSLAMM is to calculate the sources of the flows and pollutants of interest for the study area under different rain conditions.

3. Test and Control Area Land Use Development Characteristics

Land development information corresponding to the different land uses is needed as an initial step in investigating stormwater quality for an area. This is especially true when modeling expected stormwater characteristics under a variety of conditions. Detailed land use characteristics for a wide variety of land uses are available from several stormwater research projects. These available data were used in conjunction with the detailed, house-by-house surveys conducted in the study area. These data were used in conjunction with the site soils infiltration and density measurements also conducted in the test area.

The Marlborough study area in Kansas City is mostly a medium-density residential area, constructed before 1960, with a small amount of strip commercial area along Troost Avenue and a small portion of a school. UMKC graduate students made detailed inventories of each of the approximately 600 homes in the area by. These data, along with initial modeling results, have been summarized in publications and conferences (Pitt and Voorhees 2009, 2011).

Land Characteristics Survey in Kansas City Test Watershed

In many areas, detailed aerial coverage with GIS data sets are becoming available, showing and quantifying the finer elements of an area. Figure 24 is an example GIS map from Kansas City, Missouri, showing parts of the study area. This high-resolution GIS data shows all the main elements, but field surveys were still needed to verify the drainage pattern for each impervious element in the test watershed and to identify many other site elements used in stormwater quality modeling.

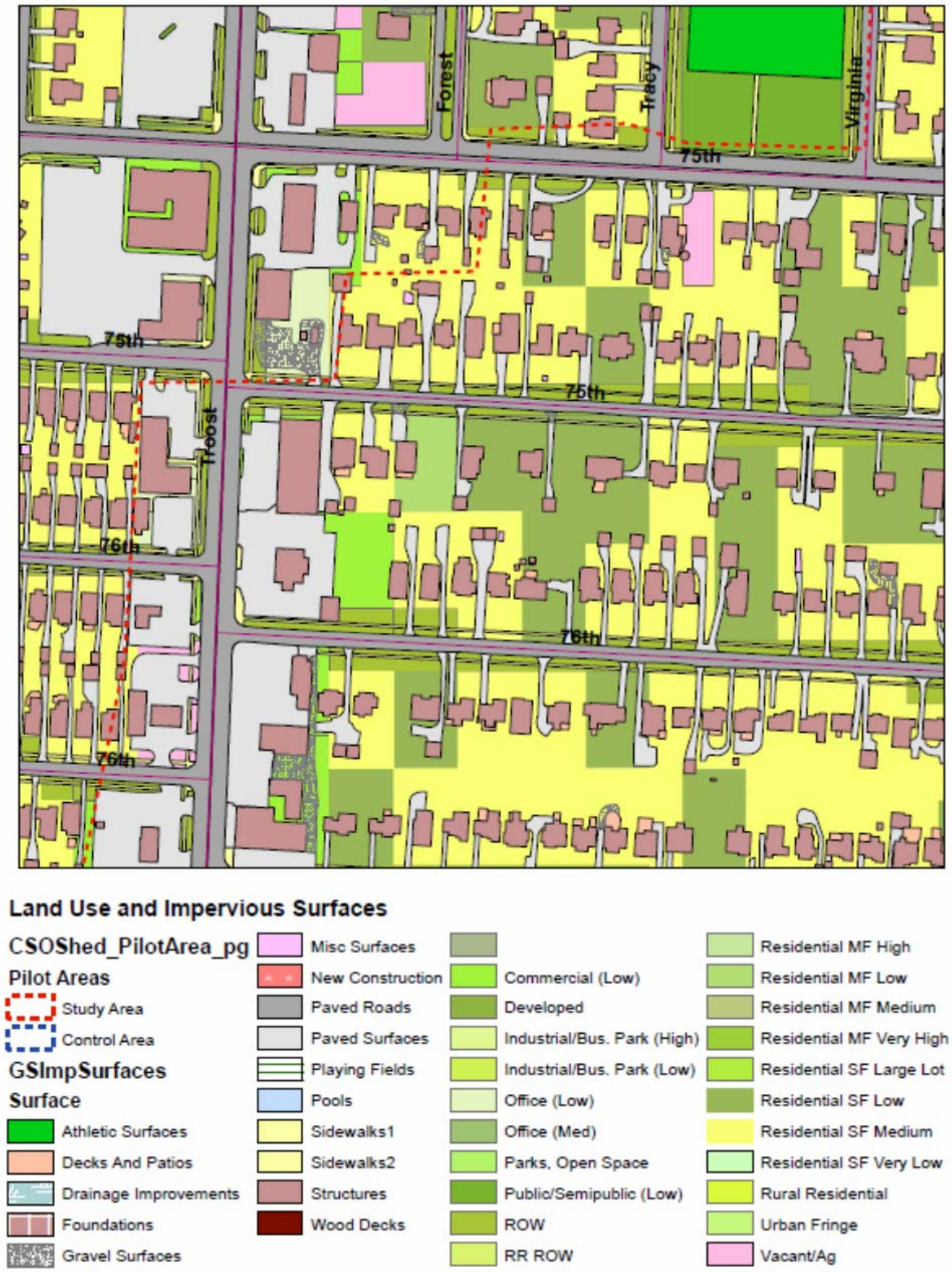


Figure 24. Detailed GIS coverage showing land cover components of different land uses in the Kansas City test watershed.

Dr. Deb O'Bannon and her graduate students at UMKC conducted a detailed survey of the development characteristics in the study area. This information was used in conjunction with the overall GIS information describing each land element to identify the specifics needed for the continuous modeling. They surveyed 576 homes in the 100-acre area (90.6 acres was residential). The housing density is therefore about 6.4 homes per acre. Tables 10 and 11 show the original GIS information for the test watershed, from Kansas City, Missouri, city sources and the detailed site data after categorizing by the site data information. The values shown on Table 11 are the percentages of each subarea of the whole area, while the values shown in parentheses are the breakdown within a single subarea. For example, directly connected roofs make up about 1.87% of the complete 100 acre site, and represent about 15% of all roofs.

Table 10. Original GIS measurements by Kansas City, Missouri, for the test watershed

	Decks and patios	Gravel surfaces	Paved roads	Paved parking/ storage	Sidewalks	Roofs	Pools	Pervious areas	Sum
All Commercial:									
acres	0.00	0.14	1.92	3.41	0.24	1.36	0.00	1.25	8.32
%	0.00	1.68	23.10	40.93	2.87	16.37	0.00	15.06	100.00
All Office									
acres	0.00	0.00	0.00	0.26	0.03	0.17	0.00	0.11	0.58
%	0.00	0.00	0.00	45.86	5.80	29.72	0.00	18.63	100.00
All Institutional									
acres	0.00	0.00	0.31	0.01	0.04	0.00	0.00	0.19	0.56
%	0.00	0.00	56.07	2.59	6.36	0.00	0.00	34.98	100.00
All Residential									
acres	0.94	0.25	8.08	8.17	2.03	11.72	0.02	59.35	90.56
%	1.04	0.27	8.93	9.02	2.24	12.94	0.02	65.54	100.00
All Combined									
acres	0.94	0.39	10.32	11.85	2.34	13.25	0.02	60.91	100.02
%	0.94	0.39	10.32	11.85	2.34	13.25	0.02	60.89	100.00

Table 11. Medium-density residential areas (%)

	Roofs	Driveways	Sidewalks	Parking/ storage	Streets	Landscaped	Isolated	Total
Impervious								
directly connected	1.87 (15%)	4.12 (46%)	1.15 (46%)	1.59	9.35			18.07
disconnected	10.57 (85%)	4.03 (45%)	1.34 (54%)					15.95
Pervious								
unpaved (gravel, severely compacted)		0.81 (9%)						0.81
landscaped						65.13		65.13
isolated (swimming pools)							0.05	0.05
Total residential area	12.44	8.95	2.49	1.59	9.35	65.13	0.05	100.00

Even though the major categories for the site agreed when the GIS information and the site surveys were compared, the site surveys were able to distinguish the different categories of pervious surfaces and to quantify how much of the impervious areas were directly connected to the drainage system. This addition information has dramatic effects on the actual stormwater quality and quantity, especially for the small and intermediate storms that produce most of the annual runoff, and even for the 1.4-in. design storm used for the CSO evaluations. As an example, only about 15% of the residential roofs are directly connected. If all were assumed to be connected, large errors in the roof runoff contribution calculations would occur. Similarly, if roof runoff stormwater controls were located at all roofs, those located where the roofs were already disconnected would have much lower additional benefits in decreasing the area's runoff quantity. Therefore, even though the detailed GIS information is very helpful, the area still needed site surveys. An *Area Description* field sheet is used to record important characteristics of the homogeneous land use areas during the field surveys (Figure 25).

Location: Site number:
Date: Time:
Photo numbers:
<u>Land-use and industrial activity</u>:
Residential: low medium high density single family
multiple family
trailer parks
high rise apartments
Income level: low medium high
Age of development: <1960 1960-1980 1980-2000 >2000
Institutional: school church hospital other (type):
Commercial: strip shopping center/mall downtown hotel offices
Industrial: light medium heavy (manufacturing) describe:
Open space: undeveloped park golf cemetery
Other: freeway utility ROW railroad ROW other:
<u>Maintenance of building</u>: excellent moderate poor
<u>Heights of buildings</u>: 1 2 3 4+ stories
<u>Roof drains</u>: % underground % gutter % impervious % pervious
<u>Roof types</u>: flat composition shingle wood shingle metal other:
<u>Sediment source nearby</u>? No Yes (describe):
<u>Treated wood near drainage system or directly connected pavement</u>? No telephone poles fence other:
<u>Landscaping near road or directly connected impervious surfaces</u>:
Quantity: none some much
Type: deciduous evergreen lawn
Maintenance: excessive adequate poor
Leafs on street: none some much
<u>Topography</u>:
Street slope: flat (<2%) medium (2-5%) steep (>5%)
Land slope (next to street): flat (<2%) medium (2-5%) steep (>5%)
<u>Traffic speed</u>: <25mph 25-40mph >40mph
<u>Traffic density</u>: light moderate heavy
<u>Parking density</u>: none light (20 to 50%) moderate (50 to 80%) heavy (>80%)
<u>Width of street</u>: number of parking lanes:
number of driving lanes:
<u>Condition of street</u>: good fair poor
<u>Texture of street</u>: smooth intermediate rough very rough
<u>Pavement material</u>: asphalt concrete unpaved
<u>Driveways:</u> paved unpaved
Condition: good fair poor
Texture: smooth intermediate rough
<u>Gutter material</u>: grass swale lined ditch concrete asphalt
Condition: good fair poor
Street/gutter interface: smooth fair uneven
<u>Litter loadings near street</u>: clean fair dirty
<u>Parking/storage areas</u> (describe):
Condition of pavement: good fair poor
Texture of pavement: smooth intermediate rough unpaved
Directly connected to drainage: yes no
<u>Other paved areas</u> (such as alleys and playgrounds), describe:
Condition: good fair poor
Texture: smooth intermediate rough
Directly connected to drainage: yes no
<u>Other notes/comments</u>:

Figure 25. Area description field sheet.

Infiltration Rate Monitoring

In addition to the site surveys described above, site-specific soils information is also needed for the area. Disturbed urban soils have infiltration rates that are usually substantially less than the assumed rates according to county soil maps. For the Kansas City project, small-scale infiltrometers (Figure 26) were used to measure infiltration rates in the disturbed urban soils of the test watershed area, as shown in the photograph below. Using several of these units simultaneously and in relatively close proximity also enables measurements of variability to be determined. Any standard or small double-ring infiltrometer likely overestimates the actual infiltration rates for a site. The relatively small areas being tested, even with the larger traditional units, have substantial edge effects, especially if the area's soils are not saturated. Also, double-ring infiltrometer measurements do not use large amounts of water that would be needed to cause groundwater mounding, and then saturated flow conditions, with resultant highly reduced infiltration rates. The most precise measurements of infiltration, and which should be used in areas where large infiltration units are being designed, should rely on full-scale tests. These are typically large trenches or boreholes, constructed to penetrate the depths of soil that the final units will use for infiltration, and use large volumes of water over extended periods. For small stormwater biofiltration units, this approach is usually not warranted, while it would be for infiltration galleries that are critical for drainage in enclosed areas. In the Kansas City study area, the constructed rain gardens and curb-cut biofilters have undergone full-scale inundation tests to supplement the smaller scale tests. In addition, infiltration rates during the monitored rains were also measured to obtain actual rates for the areas and designs used.

Figure 26. Set of three Turf-Tec infiltrometers for infiltration measurements in pre-development soils.

Infiltration rates are strongly affected by the soil density. In fact, for sandy soils, Pitt et al. (1999, 2008b) show that soil density has a greater effect on infiltration rates than soil moisture; for clayey soils, soil density has about the same effect on infiltration as does soil moisture. Unfortunately, most stormwater models effectively track soil moisture, but they ignore soil density. It is important to also measure soil density with the infiltration rates.

WinSLAMM has a Monte Carlo component that can describe the highly variable infiltration rates actually observed. Infiltration rates were monitored at several locations near the streets throughout the project area by the UMKC students. Figure 27 shows the average infiltration responses from three sets of measurements at six locations, representing 18 individual infiltration tests. Initial infiltration rates were several in/hr, but the instantaneous rates were reduced to about 1 in/hr after about one hour.

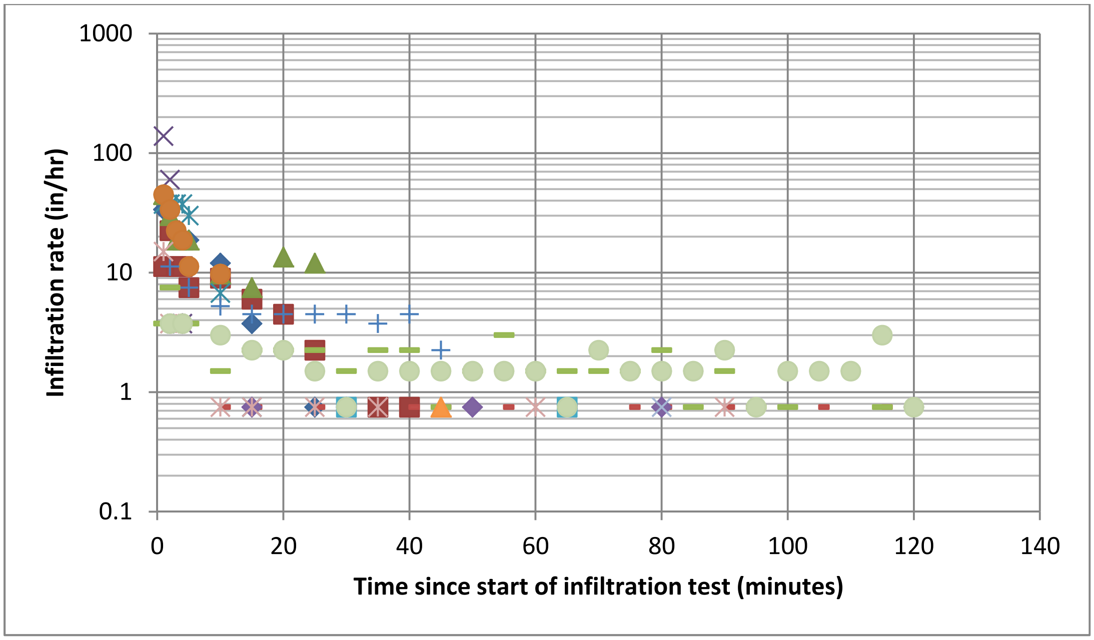


Figure 27. Example infiltration plot in area soils.

Table 12 shows the observed infiltration rates, averaged for different event durations (in/hr), and plotted in Figure 28.

Table 12. Infiltration characteristics for area soils

	5-min event	15-min event	30-min event	60-min event	90-min event	120-min event
Average	12.15	4.12	2.73	1.58	1.15	0.90
St dev	20.42	6.28	5.04	3.79	3.17	2.78
COV	1.68	1.52	1.84	2.39	2.76	3.10
Min	0.00	0.00	0.00	0.00	0.00	0.00
Max	138.75	30.00	30.00	30.00	30.00	30.00

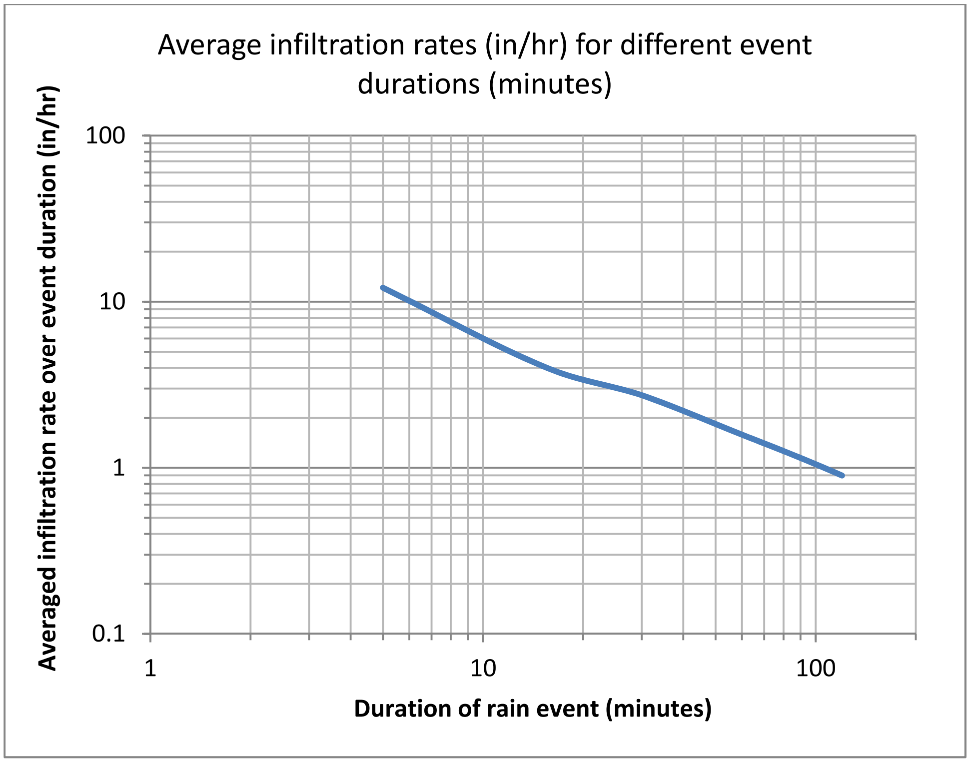


Figure 28. Decreasing infiltration rates as the rain event duration increases.

This graph indicates that the infiltration rate would be between 1 and 10 in/hr for rains that lasted up to about 2 hours, with likely decreasing infiltration rates for the long rains of interest for the critical CSO event design storm. Initial modeling efforts assumed infiltration rates of about 0.3 in/hr, but later measurements and deeper soil profiles indicated that this might be too large for the site. Therefore, for the shallow rain gardens considered during the initial analysis, infiltration rates of 0.2 in/hr were used. However, actual infiltration measurements in the biofilters after saturated conditions indicated system infiltration rates were generally between 1 and 4 in/hr, while modeling indicates that the subsurface infiltration rates are likely close to 1 in/hr. Subsurface infiltration in areas of biofiltration device construction can be higher than surface rates because of typical decreased amounts of clays and fewer compacted conditions (Pitt and Talebi 2012). If care is taken to minimize compaction during construction, these higher rates could be preserved.

Summary of Site Characteristics Used in Stormwater Quality Modeling

Land development information corresponding to the land uses in an area is needed as an initial step in investigating stormwater management options for an area. The Marlborough study (pilot) area in Kansas City is mostly a medium-density residential area, constructed before 1960, with a small amount of strip commercial area along Troost Avenue and a small portion of a school. UMKC graduate students made detailed inventories of each of the approximately 600 homes in the area.

Detailed site information is needed for stormwater management evaluations. Only about 15% of the residential roofs are directly connected in the test (pilot) area. If all were assumed to be directly connected, large errors in the roof runoff contribution calculations would occur. Similarly, if roof runoff stormwater controls were located at all roofs, those located where the roofs were already disconnected would provide much lower additional benefits in decreasing the area's runoff quantity.

In addition to the site surveys, the UMKC students conducted site-specific soils surveys for the area. Small-scale infiltrometers were used to measure infiltration rates in the disturbed urban soils of the test watershed area. The most precise measurements of infiltration, and which should be used in areas where large-scale infiltration units are being designed, should rely on full-scale tests. These are typically large trenches or boreholes, constructed to penetrate the depths of soil that the final units will use for infiltration, and use large volumes of water over extended periods. In the Kansas City study area, the constructed rain gardens and curb-cut biofilters have undergone full-scale inundation tests after construction to supplement the smaller scale tests. In addition, the rate of infiltration during the actual rains was also measured to obtain actual rates for the area and designs used. Figure 29 shows the measured infiltration rates from the small-scale tests in the test area.

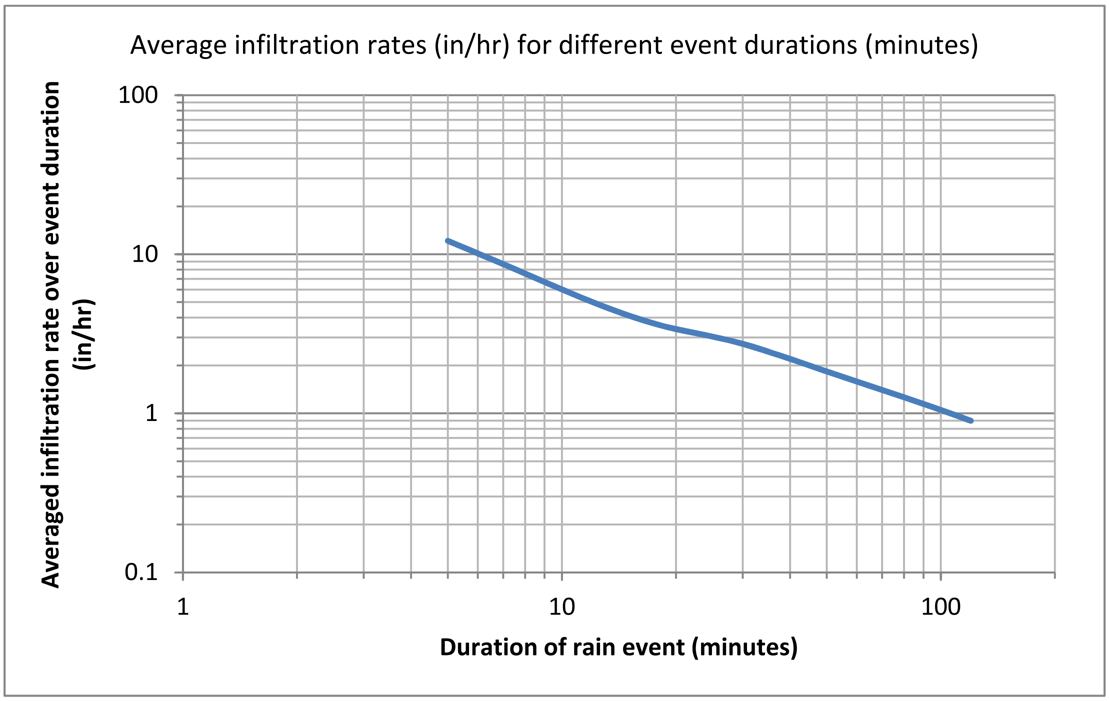


Figure 29. Duration-infiltration rates for surface soils.

Figure 29 indicates that the infiltration rate would be between 1 and 10 in/hr for rains that lasted up to about 2 hours, with likely decreasing infiltration rates for the long rains of interest for the critical CSO event design storm. Initial modeling efforts supporting the GI designs assumed infiltration rates of about 0.3 in/hr. Deeper soil profiles indicated that this might be too large. Therefore, for the shallow rain gardens, an infiltration rate of 0.2 in/hr was used by the initial designers. However, actual infiltration measurements in the constructed biofilters after saturated conditions indicated system infiltration rates are generally between 1 and 4 in/hr (average of 2.5 in/hr), while modeling indicates that the subsurface infiltration rates in the native soils are likely close to 1 in/hr. Two of the monitored biofilters had reduced infiltration rates of about 0.6 in/hr, likely due to compacted conditions during their construction. Subsurface infiltration in areas of biofiltration device construction can be higher than surface rates because of typical decreased amounts of clays and reduced compaction. If care is taken to minimize compaction during construction, these higher rates could be preserved. The extended monitoring period will help verify the actual soil infiltration conditions.

4. Large-Scale Calibration of WinSLAMM: Modeled Stormwater Characteristics Compared to Observed Data

Monitoring Locations in Test and Control Watersheds

Runoff monitoring was conducted in the combined sewer system at several locations in the test and control watersheds. This sampling arrangement enabled flows to be separated for the test (pilot) and the control watersheds. Nine complete events were monitored in the area in 2009, and six events were monitored in 2010. These initial data were used to do the verification of the WinSLAMM runoff calculations. Because sewer rehabilitation was occurring during this period in the test watershed, only the control area data were used for these analyses. Additional events were monitored after the sewer was rehabilitated, and these data were compared to the flows observed before re-lining. Construction of the stormwater controls started after the re-lining, with the final seven events from April 1 to the first part of June 2012 representing built conditions with the stormwater controls for the first study period. These analyses do not include events after these June events because of lag times in data summaries. The project continued to collect data through 2013 and further data analyses were conducted with the complete data set. A total of 75 events were used for the pre-construction baseline conditions and 37 events were used for the post-construction conditions.

As noted previously, the detailed land development and land use information for the test and control watersheds enabled the verification of the water quantity portion of WinSLAMM using the site rainfall and runoff data. Figures 30 and 31 show the test and control watershed boundaries and the locations of the flow monitoring stations. Monitoring stations UMKC02a, 02b, and 3 measured flows from the control watershed, while station UMKC01 measured the flows from the test (pilot) watershed alone.

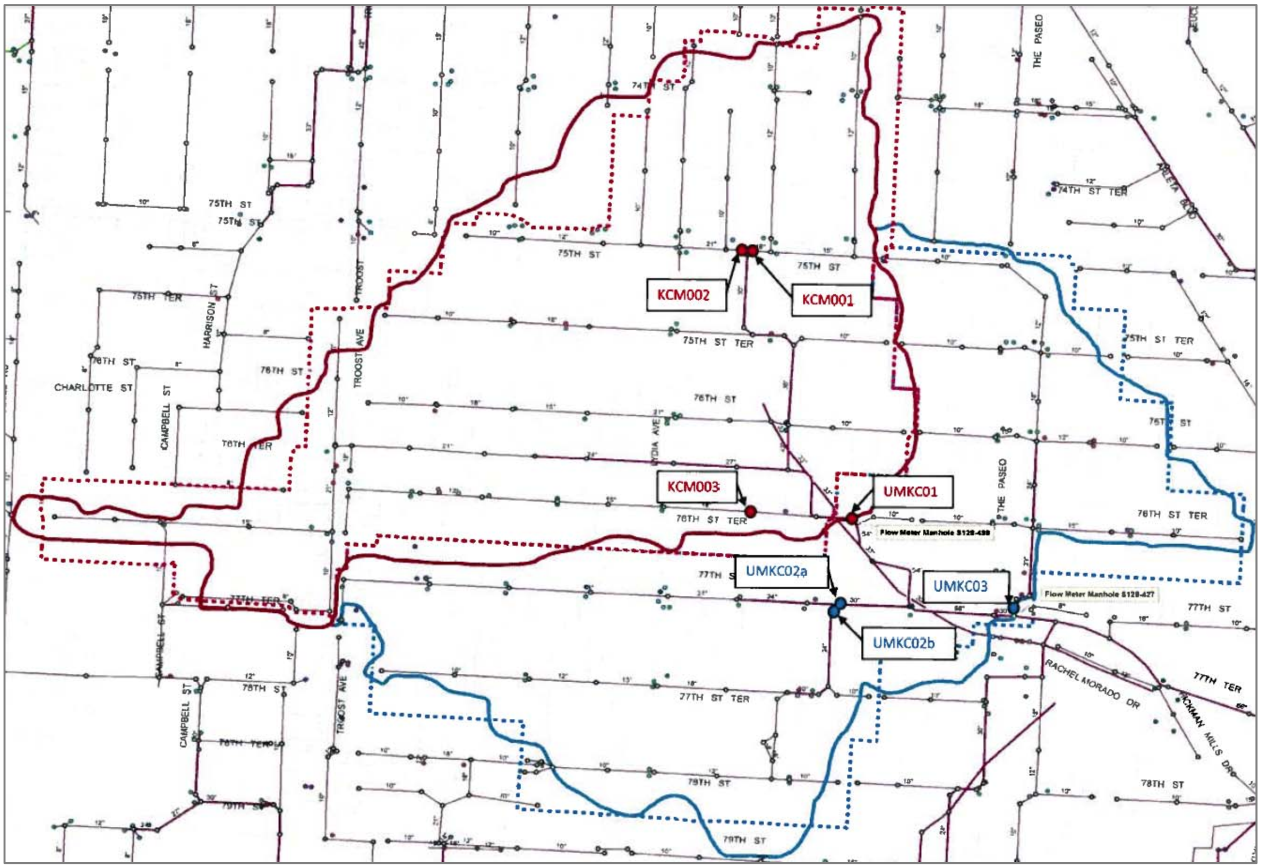


Figure 30. Test (100 acres) and control (86 acres) watersheds in Marlborough area of Kansas City, Missouri.

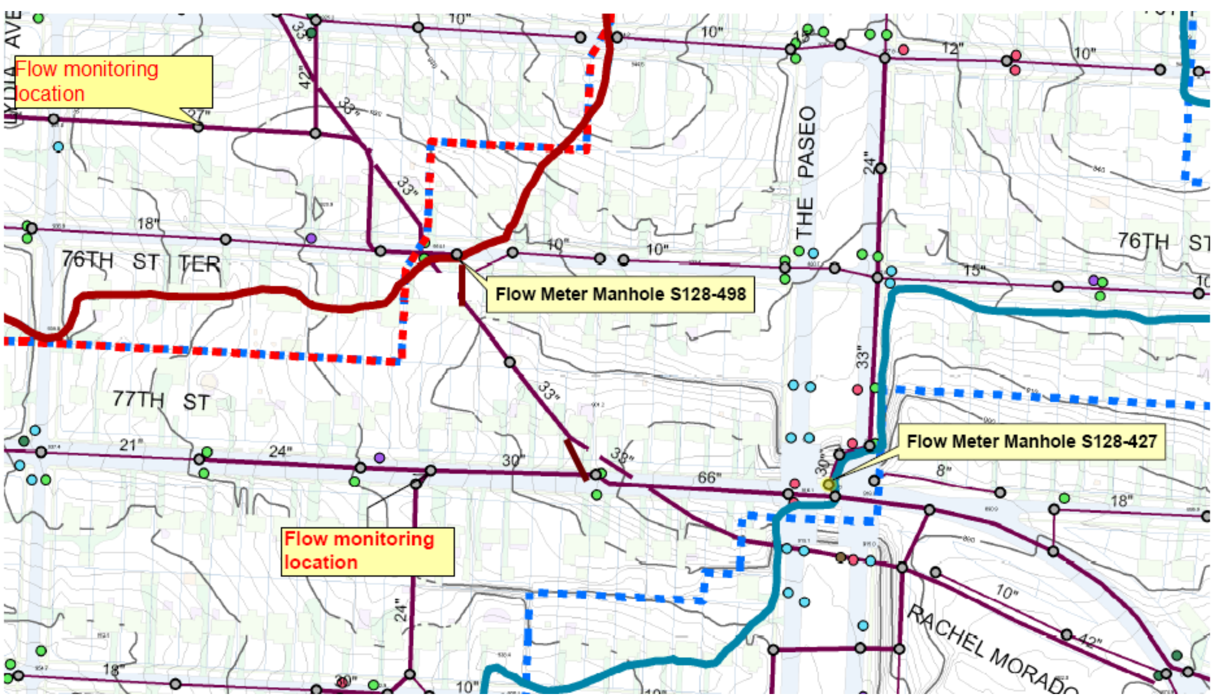


Figure 31. Flow monitoring locations at test and control area boundaries.

Table 13 summarizes the four flow monitoring locations established for this project as shown in the above figures. The combined sewers ranged from 24 to 42 inches in diameter. The areas were re-evaluated during the project using as-built sewer maps and some site surveys using GIS tools. The UMKC1 meter location measured flows from the test (or pilot) watershed where the GI controls were constructed, while UMKC2a, 2b, and 3 monitored portions of the control watershed.

Table 13. Flow Monitoring Locations in Combined Sewers for Test and Control Watersheds

GI Project Location	Area Type	Location	Design Station	Date Installed	Drainage Area (acres)	Pipe Diameter at Monitoring Location
UMKC1 flow meter	Test/Pilot	Near 1461 E 76th Terrace	S128-498	11/7/08	99.7	42”
UMKC2a flow meter	Control	Near 1451 E. 77th St.	S128-422	11/7/08	41.4	30”
UMKC2b flow meter	Control	Near 1451 E. 77th St.	S128-420	11/7/08	27.6	24”
UMKC3 flow meter	Control	77th St & Paseo Overpass	S128-426	11/7/08	17.6	30”

Each station includes an ISCO 2150 area-velocity sensor. According to ISCO: “The 2150 Flow Module uses continuous wave Doppler technology to measure mean velocity. The sensor transmits a continuous ultrasonic wave, then measures the frequency shift of returned echoes reflected by air bubbles or particles in the flow. The 2150’s “smart” area velocity probe is built on digital electronics, so the analog level is digitized in the sensor itself to overcome electromagnetic interference. The probe is also factory-calibrated for 10-foot (3 meter) span at different temperatures. This built-in calibration eliminates drift in the level signal, providing long-term level stability that reduces recalibration frequency and completely eliminates span recalibration.” ISCO further states that this sensor can measure shallow flow is small pipes as the low-profile velocity sensor minimizes flow stream obstructions and senses velocity in flows down to 1 inch in depth. Table 14 lists some of the level and velocity measurement specifications for the ISCO 2150 sensor. The stated range includes minimum depths of about 0.4 inches with a 0.12 in accuracy, and long term stability of about 0.3 in per year. The velocity range is from -5 to 20 ft/sec (includes adverse flows) with a stated accuracy of about 0.1 ft/sec for low velocities.

Table 14. Level and Velocity Measurement Specifications for ISCO 2150 Sensor

Level Measurement	
Method:	Submerged pressure transducer mounted in the flow stream
Transducer Type:	Differential linear integrated circuit pressure transducer
Range:	(standard) 0.033 to 10 ft (0.010 to 3.05 m); (optional) up to 30 ft (9.15 m).
Maximum Allowable Level:	34 ft (10.5 m)
Accuracy:	±0.01 ft from 0.033 to 10 ft, (±0.003 m from 0.01 to 3.05 m)
Long-Term Stability:	±0.023 ft/yr (±0.007 m/yr)
Compensated Range:	32° to 122°F (0° to 50°C)
Velocity Measurement	
Method:	Doppler ultrasonic, frequency 500 kHz
Typical Minimum Depth:	0.08 ft (25 mm)
Range:	-5 to +20 ft/s (-1.5 to +6.1 m/s)
Accuracy:	(in water with uniform velocity profile, speed of sound = 4850 ft/s, for indicated velocity range); ±0.1 ft/s from -5 to 5 ft/s (±0.03 m/s from -1.5 to +1.5 m/s); ±2% of reading from 5 to 20 ft/s (1.5 to 6.1 m/s)

Rain Gauges near the Monitoring Locations

Data from seven rain gauges reported by Johnson County Kansas Regional Weather (www.stormwatch.com) near the study area during the monitoring period were examined as part of the project QA/QC activities. Figure 32shows the locations of these stations which were: sensor ID 1800 (Allied Signal @ Indian Creek), sensor ID 2400 (Allied Signal @ Blue River), sensor ID 2420 (85th @ Blue River), sensor ID 2790 (92nd and Ward Parkway Trib to Indian Creek), sensor ID 5050 (Lee Blvd @ Dykes Branch), sensor ID 5100 (Brooklyn PS), and sensor ID 5110 (75th Terrace and Troost). Rain gauges 1800 and 2400 have a lot of missing data over the study period, and only three (2420, 5100, and 5050) had rain depth data available for the whole study period from 2009. The sensor ID 2420 rain gauge was used for these analyses as it was closest to the flow monitors and had continuous information.

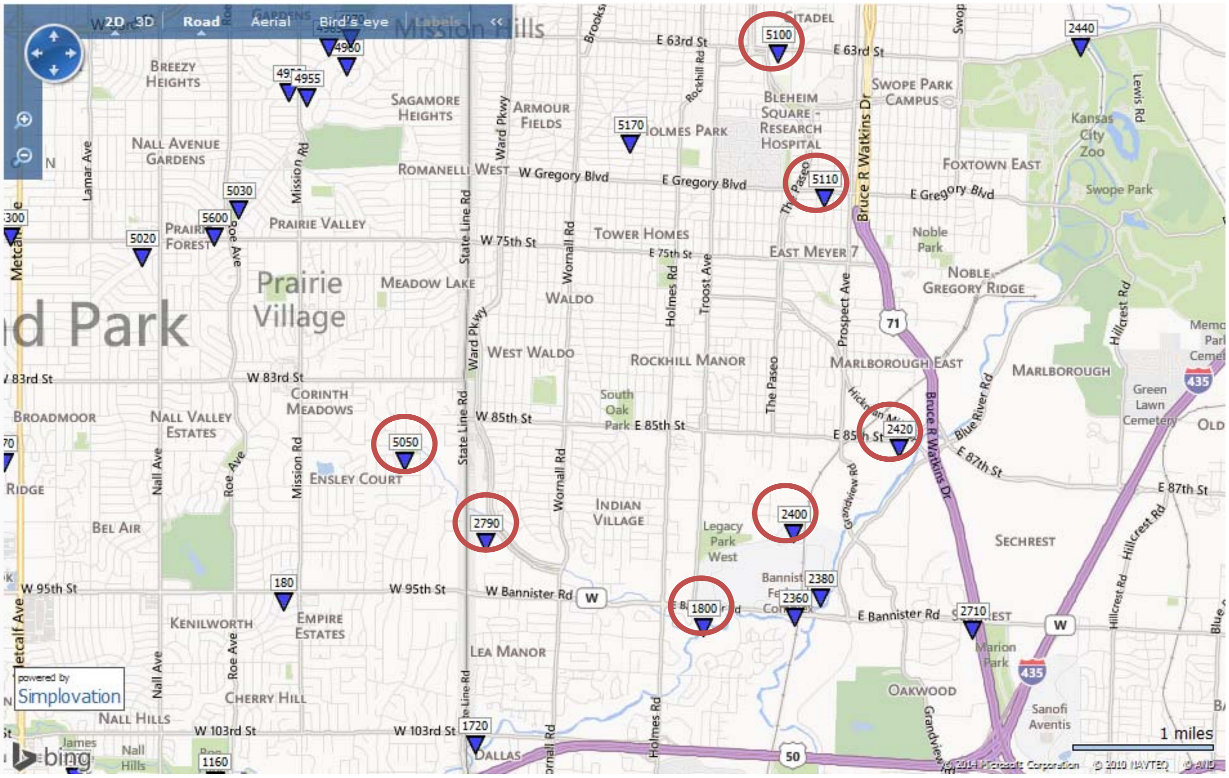


Figure 32. Locations of rain gauges near the study area (source: www.stormwatch.com).

Runoff and Rainfall Observations in Test and Control Watersheds

Large amounts of data were collected as part of the Kansas City Green Infrastructure project. This chapter discusses the large-scale monitoring in the combined sewers, while Chapter 5 discusses the small-scale monitoring at the curb-side biofilters, and Chapter 6 discusses complimentary laboratory pilot-scale tests on the biofilter media. The project was designed to include these multiple scales of monitoring which would be complementary and to some extend redundant. This chapter describes the large-scale monitoring in adjacent pilot (test) and control areas served by combined sewers. Monitoring at these four monitoring locations (described previously) started before the construction of the GI facilities in order to obtain baseline information. The pilot watershed combined sewer drainage system was also relined to reduce infiltration and exfiltration near the end of this baseline period. Monitoring was also conducted during and after the construction of the GI facilities. Much information was obtained during the “initial” baseline period (before the relining) at all of the monitoring locations. However, few events were monitored after the relining and during the construction period. The real time rainfall and runoff data from combined sewer systems that are affected by green infrastructure (GI) stormwater controls in upstream areas were analyzed for different phases, including an initial baseline before relining, after relining, and after the construction of the stormwater controls. The Rv values (the ratio of runoff to rainfall depths) of pre- and post-construction conditions were compared to measure the benefits of the integrated GI stormwater controls at the large watershed scales.

Table 15 shows the dates associated with each project monitoring period. The initial baseline period extended for about 15 months and 69 events were monitored before the sewer relining which lasted for about seven months. The period after the re-lining included about seven events before the construction of the GI facilities began. The monitoring after the GI facility construction began in April of 2013 and lasted for about six months, with 37 events being monitored.

Table 15. Monitoring Periods in Test/Pilot and Control Area Watersheds

Monitoring period	Dates corresponding to monitoring period	Number of monitored storms in each monitoring period*
Initial baseline	03/23/09 – 06/19/10	69 events
After re-lining	01/22/11 – 03/19/11	7 events
After construction (after April 1, 2012)	04/07/13 – 10/30/13	37 events

* there are gaps in the flow record (such as during the sewer re-lining in the test watershed, and flow monitoring equipment failures) so not all rains had flow data available for these analyses

Quality Assurance and Quality Control Examinations of Monitored Runoff and Rainfall Data

The monitored stage-discharge relationship in the combined sewer was plotted and compared to basic plots based on Manning's equation as an important QA/QC process. These data were obtained from an area-velocity sensor that reports the discharge (flow) directly, using a calculated flow cross-sectional area based on the stage value multiplied by the measured velocity value. Figure 33 shows the stage-discharge at UMKC01 (downstream of the 100 ac pilot study area). This figure was plotted using the separately recorded stage and flow data. As shown on this figure, three Manning's roughness coefficient "n" values were used to account for the varying n values with depth and the observed stage-discharge relationship (basic Camp's curve relationships) (0.0082 to 0.012). This plot shows three regions of data observations. The "main sequence" includes almost all of the data and was fitted using reasonable n roughness values that slightly varied with depth. Most of the data inside area 1 were observed during 6/2009 and 6/2010 (occurring during the "before construction" period). The reduced discharge values for these stage observations were therefore deemed incorrect for unknown reasons. The stage values inside area 2 represent surcharged conditions, being greater than the 42'' pipe diameter. These six surcharged pressure recorded stage values were therefore re-adjusted to 42". The stage observations in area 2 were therefore changed to 42" and full-flowing discharge values were assigned for these data. The stage values for the observations in area 1 were also applied to the Manning's equation with the calibrated n roughness values. In all, only about 3% of the measured flows were modified at UMKC01. Figure 34 shows the final set of stage-discharge values for all observations at this monitoring location.

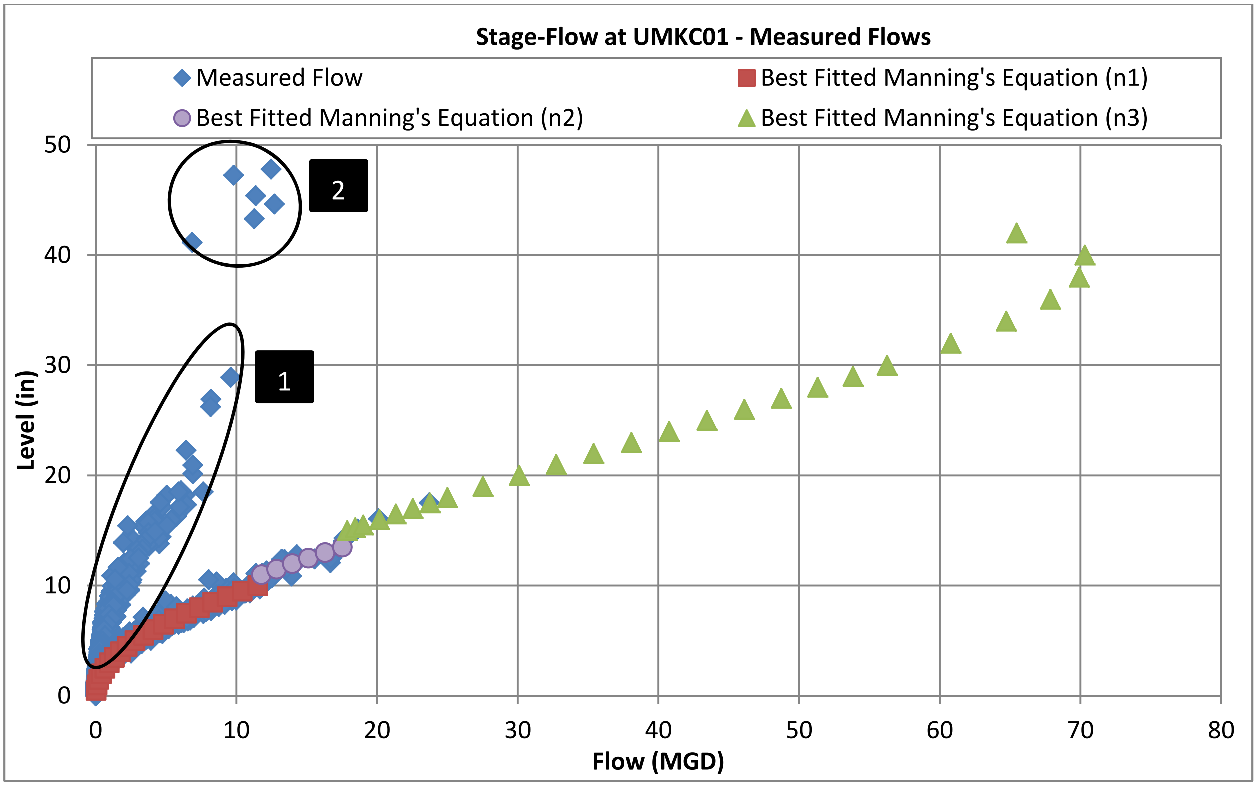


Figure 33. Stage-discharge relationship at UMKC01 for measured flows

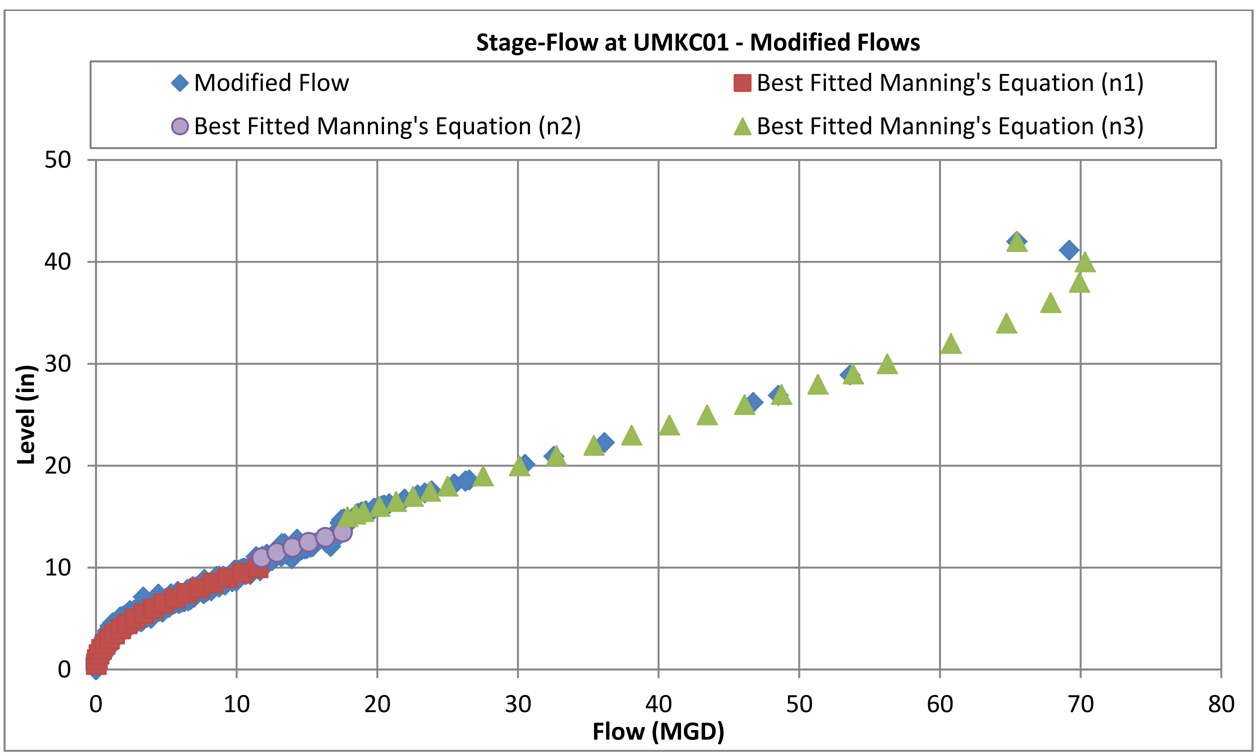


Figure 34. Modified Stage-discharge relationship at UMKC01

Similarly, the stage-discharge relationships were plotted and studied for the observed data from the UMKC02a monitoring location (part of the control watershed area). Different Manning's n values were also fitted to the observed flows that varies with the stage values (ranging from n = 0.012 to 0.019). As shown in Figure 35, the measured flow data are more scattered at UMKC02a than at UMKC01, requiring about 80% of data being modified using the calibrated Manning's equation. Figure 36 shows the final set of observations. Figure 35 represents stage-discharge at UMKC02a for measured flows.

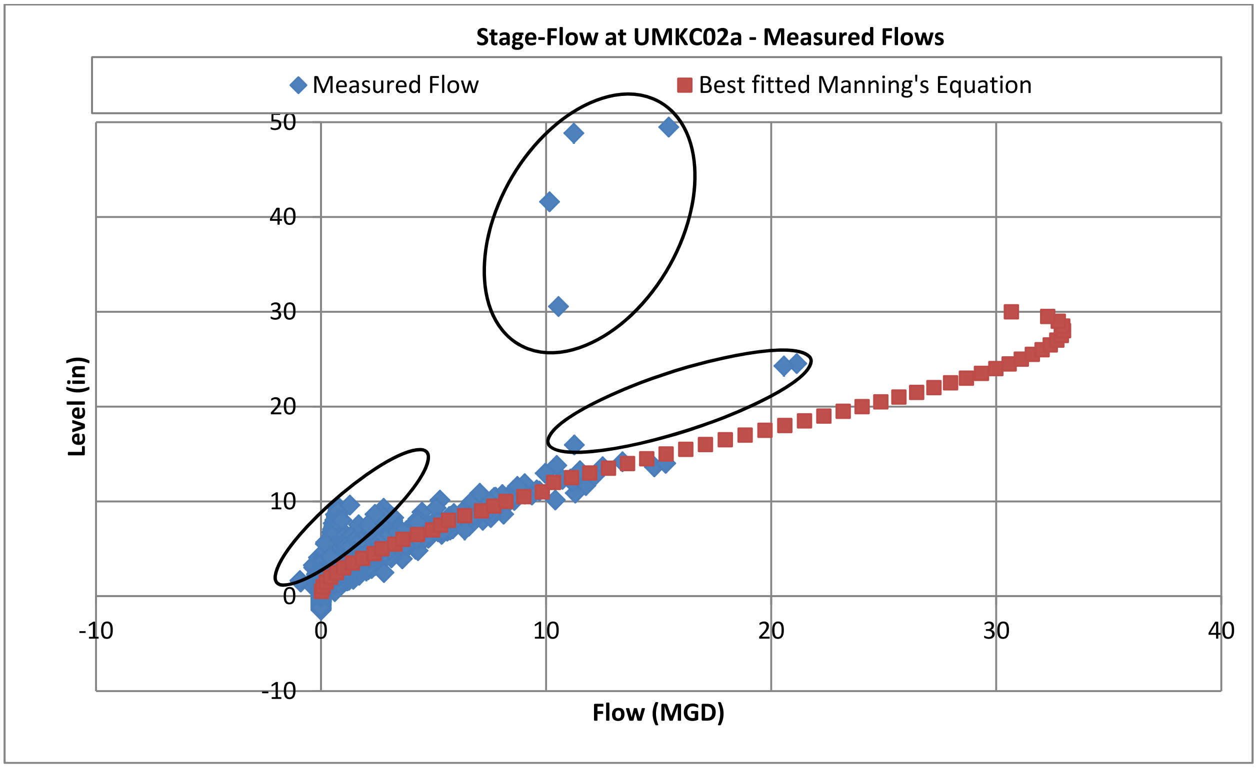


Figure 35. Stage-discharge relationship at UMKC02a for measured flows

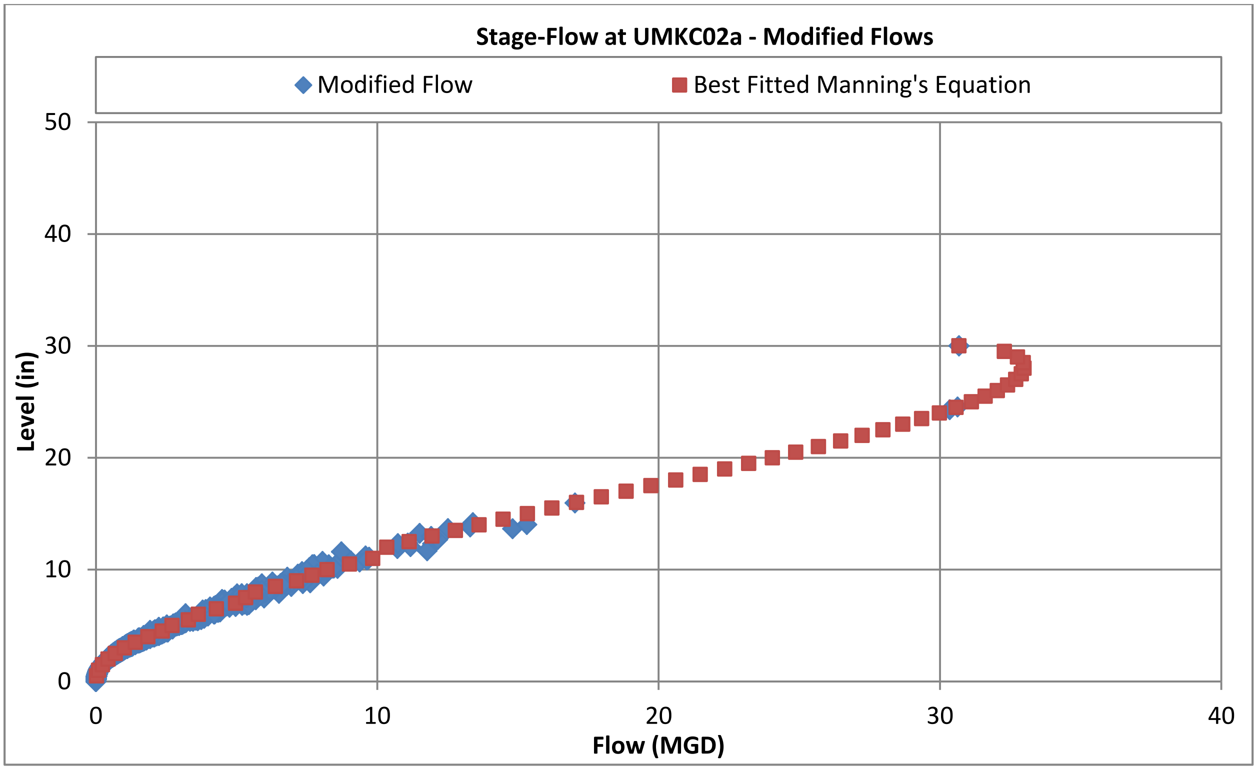


Figure 36. Modified Stage-discharge relationship at UMKC02a

Figures 37 and 38 are similar stage-discharge plots for the two other control area monitoring locations: UMKC02b and UMKC03. As seen on both of these plots, there is a great deal of uncertainty in the plots and all stage values observed were quite small (most less than about 4 inches at UMKC2b in a 24 inch pipe and a maximum stage of about 2.5 inches at UMKC03 in a 30 inch pipe). It was not possible to perform a suitable Manning's n calibration with these data. It is very challenging to measure flows accurately at low stages, especially when the pipes are relatively large.

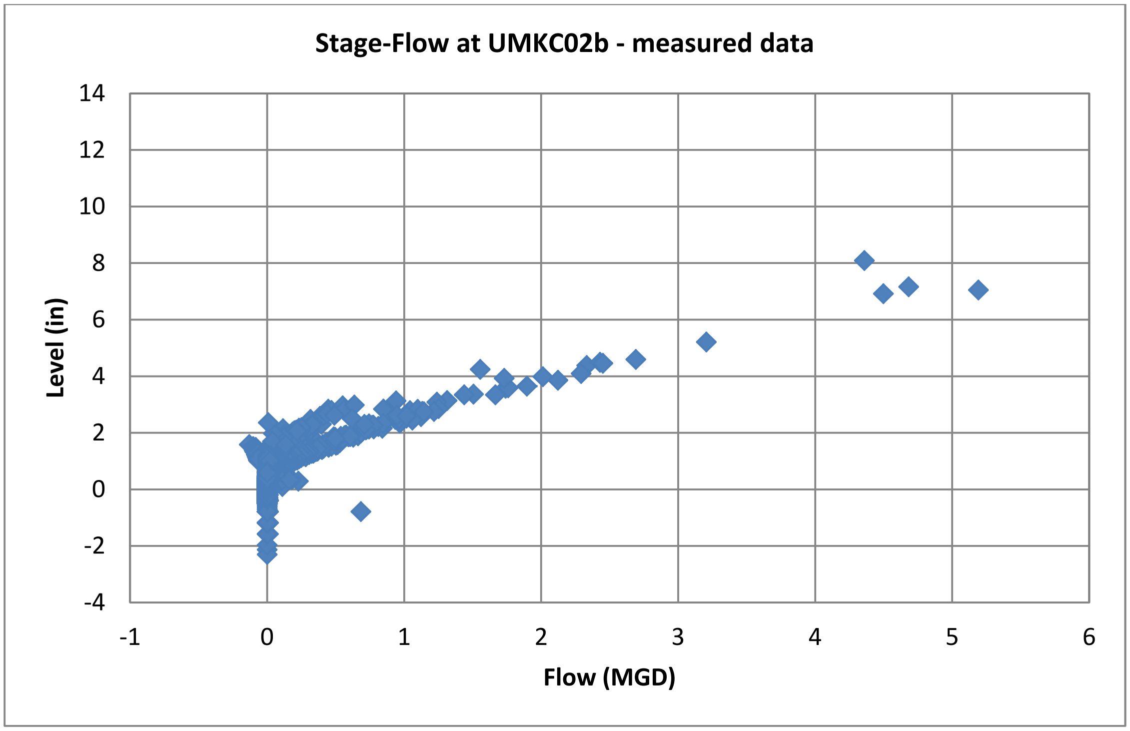


Figure 37. Stage-discharge relationship at UMKC02b for measured flows.

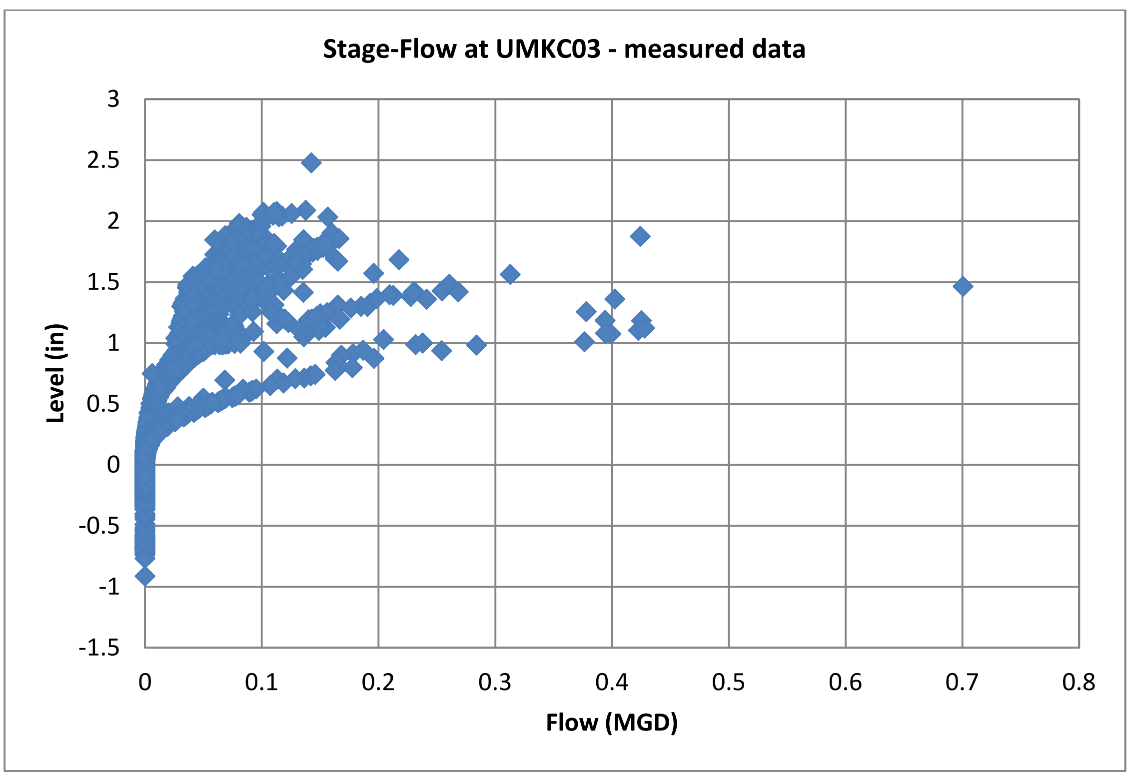


Figure 38. Stage-discharge relationship at UMKC03 for measured flows.

Besides examining these monitored stage and discharge values, runoff volumes for each event for these four locations were also examined as part of the QA/QC process. As noted previously, the land use and soil characteristics, along with detailed land development characteristics, in the test (pilot) and control areas were very similar, but with varying drainage areas. Therefore, the unit area runoff characteristics (usually expressed as the volumetric runoff coefficient, or Rv, the ratio of the runoff depth to the rainfall depth) should be similar for the test and control areas before any GI construction (or relining) and the control areas should have similar Rv values during all test phases as no watershed changes occurred in those areas during the study. The following lists the flow-weighted Rv values for the different areas:

- UMKC01 (test/pilot watershed), before relining and pre-construction: 0.26
- UMKC02a, all monitoring periods combined: 0.50
- UMKC02b, all monitoring periods combined: 0.08
- UMKC03, all monitoring periods combined: 0.18

The Rv values varied greatly during conditions when they should have been similar. This was especially evident for UMKC02a being very high and UMKC02b being very low. The expected Rv value for all of these areas for the land use, development, and soil conditions should have been in the range of about 0.25 to 0.40. The Rv values are calculated based on the total runoff volume, the total rain depth, and the watershed area. The watershed areas obviously did not change during the monitoring period, and the runoff and rains were monitored for each event. The rain data was verified by examining the data from the surrounding rain gauges for all monitored events. There were no significant or obvious differences between the gauges having available rainfall information for the monitored events. The drainage area maps showing the "as-built" combined sewers had several areas of confusion that were not able to be clarified by field surveys. However, any errors in the effective drainage areas would not cause any trends in the Rv values with time for the area not affected by site construction. Therefore, Rv trends were examined for all of these areas. The calculated Rv values for UMKC02b and UMKC03 were highly variable, while UMKC02a were less so, and UMKC01 more consistent (for each monitoring phase). The flow data from UMKC02b and UMKC03 were therefore judged not reliable for these analyses. The flow data from UMKC01 and UMKC2a were further evaluated (with the resulting event data for these two locations presented in Appendix D).

After the basic discharge values were examined and modified as necessary, the next processing step involved studying and understanding the dry weather sewage flows in the combined sewers. The time series analyses for all dry weekdays within each month had similar trends, as illustrated in Figure 39, from a Run Chart analysis in Minitab. Therefore, for each month, all flow data patterns for the dry weekdays were combined and their average was used as the base flow of weekdays. Similarly, the average flow patterns for all dry weekends were used as the base flow for weekends for each month. This resulted in two base flow patterns for each month for each watershed. These dry weather flow patterns were subtracted from the combined flows during wet weather to result in the direct runoff associated with each rain event, as illustrated in Figures 40 and 41. The direct runoff information was then used, along with the rain data for each event, to calculate the total runoff volumes and other characteristics at each monitoring location, as presented in Appendix D.

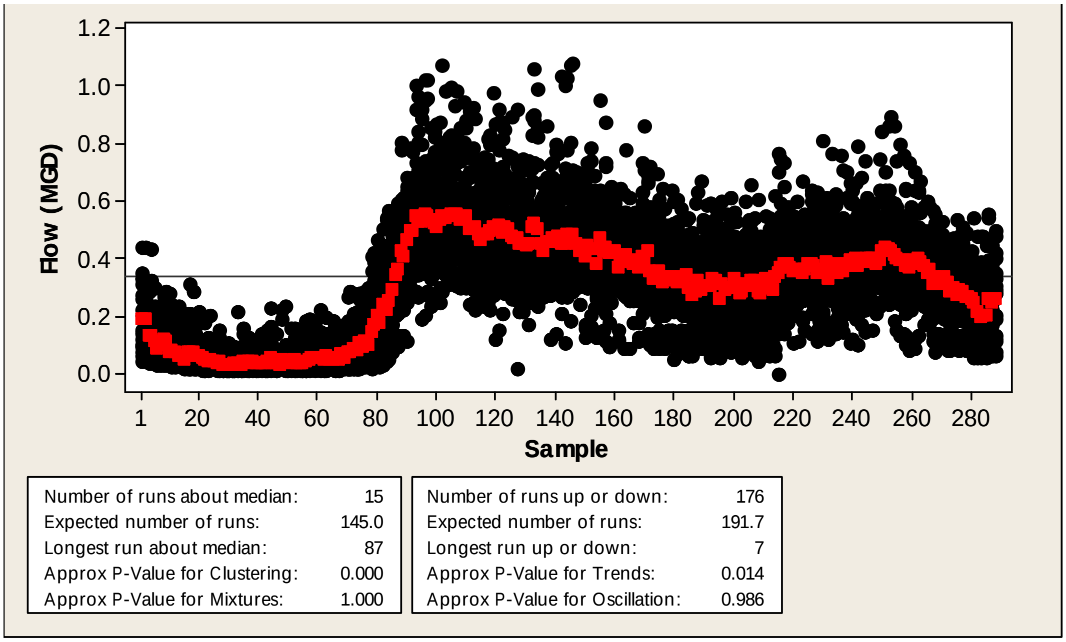


Figure 39. Example dry weather flow pattern for dry weekdays in one month.

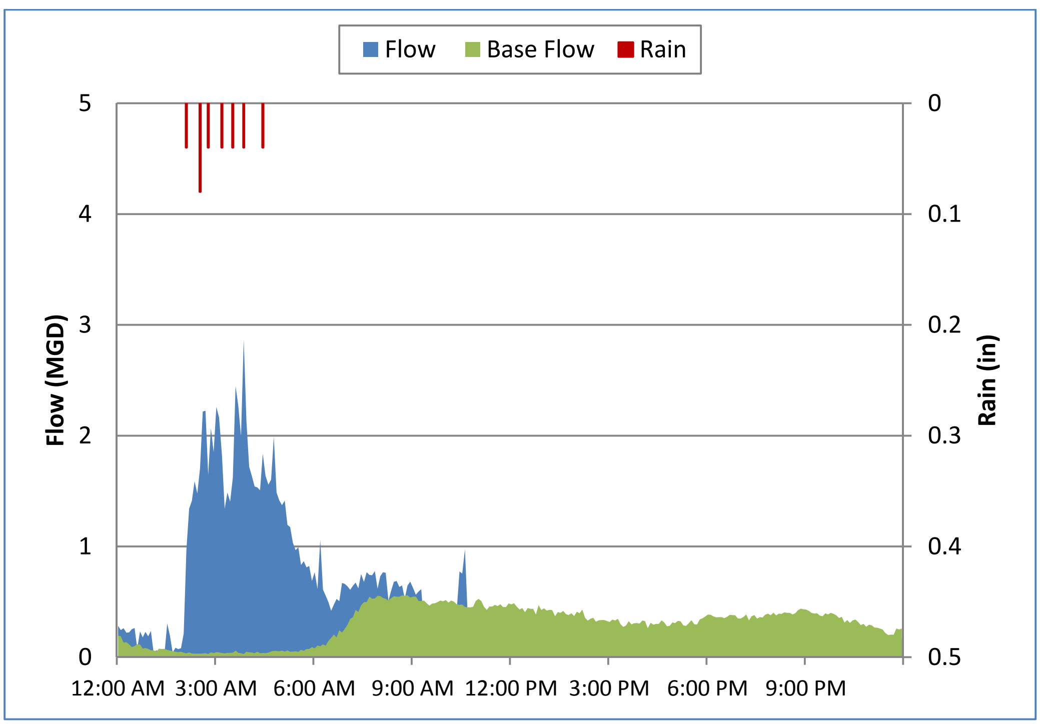


Figure 40. Observed combined wet weather flow.

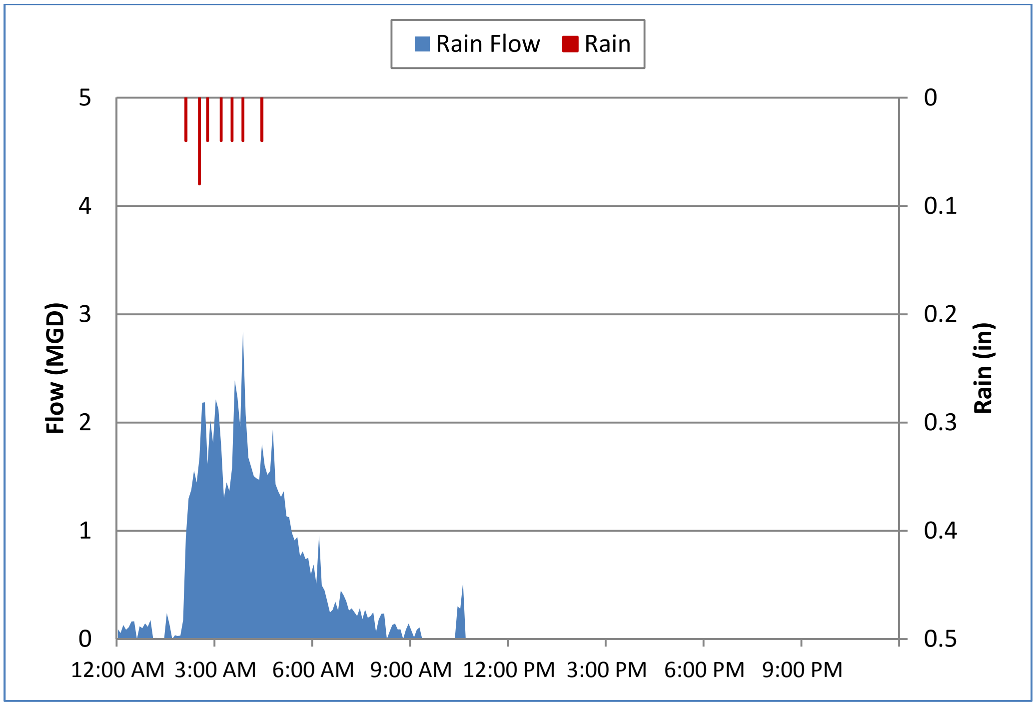


Figure 41. Direct runoff after dry weather baseflow subtracted.

Rainfall and Runoff Evaluations at Test and Control Watersheds

Appendix D contains the flow data observed during the different monitoring periods at the monitoring locations. These tables contain the observed values and the calculated rain and flow parameters based on the observed data. The raw flow data represent both the dry and wet weather flows together in the monitored combined sewers. However, because we are interested in the wet weather flows, the flow values in the wet weather flow tables have had the dry weather sanitary sewage flows subtracted. The dry weather flow pattern (showing the diurnal flow fluctuations that vary by day of the week and time of day) were subtracted from the combined flows to result in the separate rainfall-runoff contributions. These data were also used in the model calibration efforts.

Table 16 summarizes the observed rain and runoff characteristics at the UMKC01 monitoring location for the test/pilot watershed, before the green infrastructure facility construction, while Table 17 summarizes similar information for the monitoring period after the construction of the stormwater controls. Figure 42 illustrates how the Rv (the volumetric runoff coefficient, the runoff to rainfall depth ratio) varied with time during the monitoring activities and Figure 43 is a plot of the rainfall vs. runoff depths. The variability of the Rv values are quite large, but do illustrate how the values dropped during the after construction period. Rv values vary by rain depth, with smaller values associated with smaller rains (a smaller fraction of the rain occurs as runoff for small rains with larger portions of initial abstractions and infiltration losses). Figure 44 is a box and whisker plot showing the range of Rv values for each monitoring period. Even with the variability, it is obvious that the after GI facility construction runoff responses are smaller than the before runoff responses for the same rain conditions. Tables 18, 19, and 20 are Kruskal-Wallis one way analysis of variance on means tests comparing the different Rv values for the different conditions. These tests confirm that there were no statistically significant differences between the Rv values for the initial baseline vs. the after relining periods for the number of observations available (results affected by the few data available after the relining and before the construction began). The combined baseline conditions (initial baseline plus after relining periods) are significantly larger than the after GI facility construction (P <0.001).

Table 16. Rain and Flow Characteristics at UMKC01 Test/Pilot Area before GI Construction

	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)	Flow dur. (hrs)	Total pipeflow discharge volume (ft^3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
count	74	76	76	75	76	75	75	75	75	75	75	75	75
sum		949.6	58.53			3772.0	5,584,861	15.39					
minimum	0.6	1.0	0.12	0.47	0.01	2.4	3,430	0.01	0.2	0.1	0.3	0.05	0.4
maximum	36.6	64.5	3.50	1.90	0.67	2232.8	780,680	2.15	106.7	5.3	41.5	0.94	179.8
average	6.3	12.5	0.77	0.89	0.10	50.3	74,465	0.21	9.2	0.8	10.1	0.24	5.1
median	4.0	9.6	0.49	0.95	0.07	17.9	40,157	0.11	3.9	0.5	8.1	0.20	2.0
standard deviation	7.2	11.4	0.74	0.42	0.11	255.7	114,061	0.31	15.0	0.9	7.5	0.16	20.5
COV	1.1	0.9	1.0	0.5	1.1	5.1	1.5	1.5	1.6	1.2	0.7	0.67	4.0

Table 17. Rain and Flow Characteristics at UMKC01 Test/Pilot Area after GI Construction

	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)	Flow dur. (hrs)	Total pipeflow discharge volume (ft^3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
count	36	37	37	37	37	37	37	37	37	37	37	37	37
sum		393.1	30.29			706.1	1,960,646	5.40					
minimum	0.6	0.3	0.12	0.47	0.01	0.8	1,155	0.00	0.1	0.0	2.2	0.03	0.2
maximum	21.8	49.0	3.43	2.85	0.56	58.8	219,477	0.60	36.4	3.8	48.7	0.41	8.0
average	5.3	10.6	0.82	0.97	0.14	19.1	52,990	0.15	6.7	0.7	10.6	0.14	2.7
median	3.8	5.2	0.79	0.95	0.09	15.8	23,841	0.07	3.1	0.4	9.6	0.12	2.2
standard deviation	5.1	11.1	0.70	0.57	0.15	12.9	62,642	0.17	8.1	0.8	8.6	0.09	1.6
COV	1.0	1.0	0.9	0.6	1.0	0.7	1.2	1.2	1.2	1.2	0.8	0.62	0.6

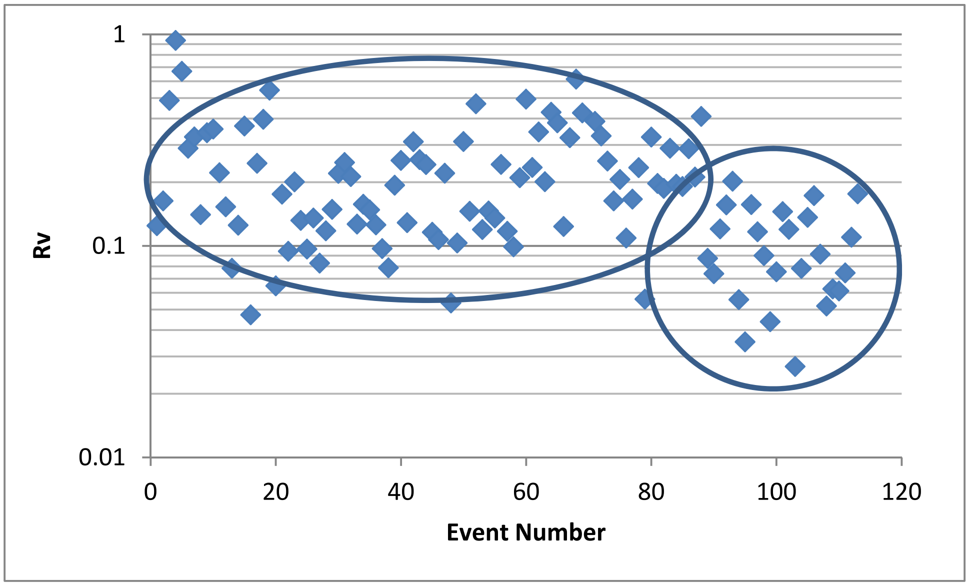


Figure 42. Volumetric runoff coefficients at UNKC01 before and after GI construction.

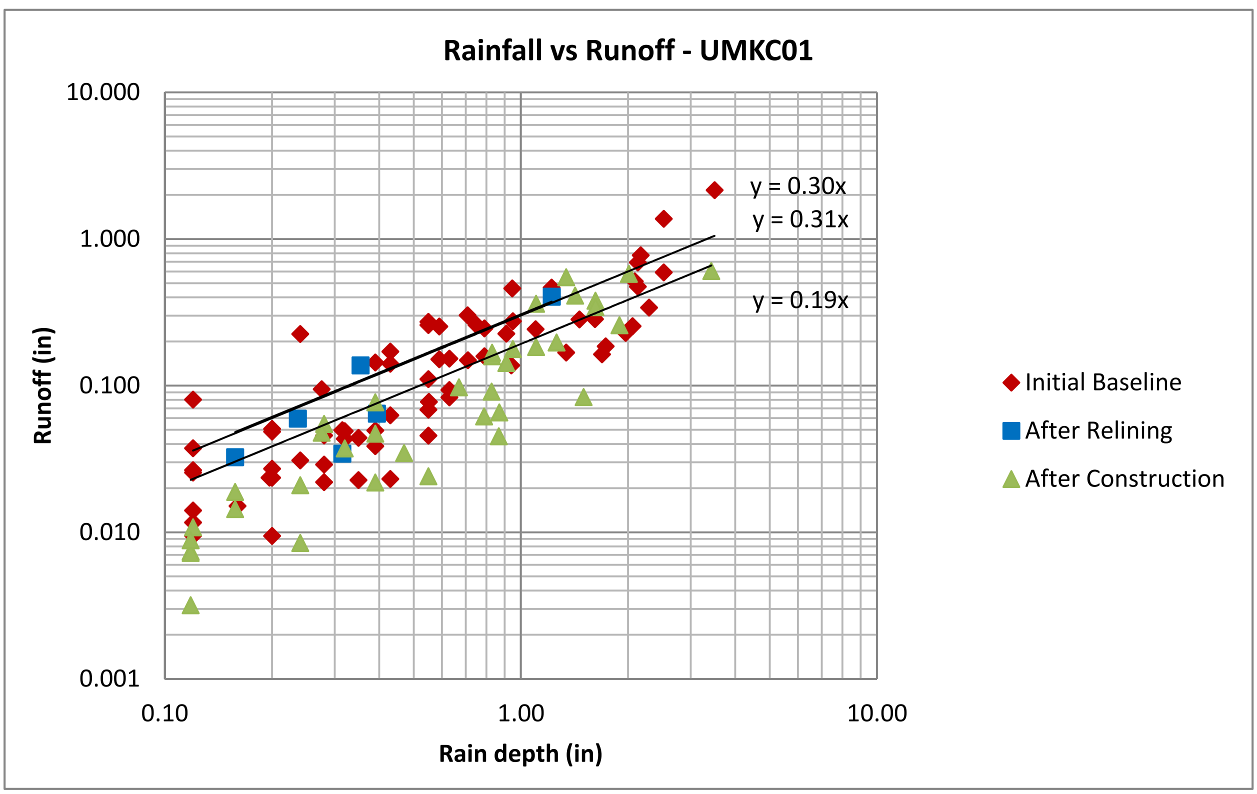


Figure 43. Rain vs. runoff plots for UMKC01 during different monitoring periods.

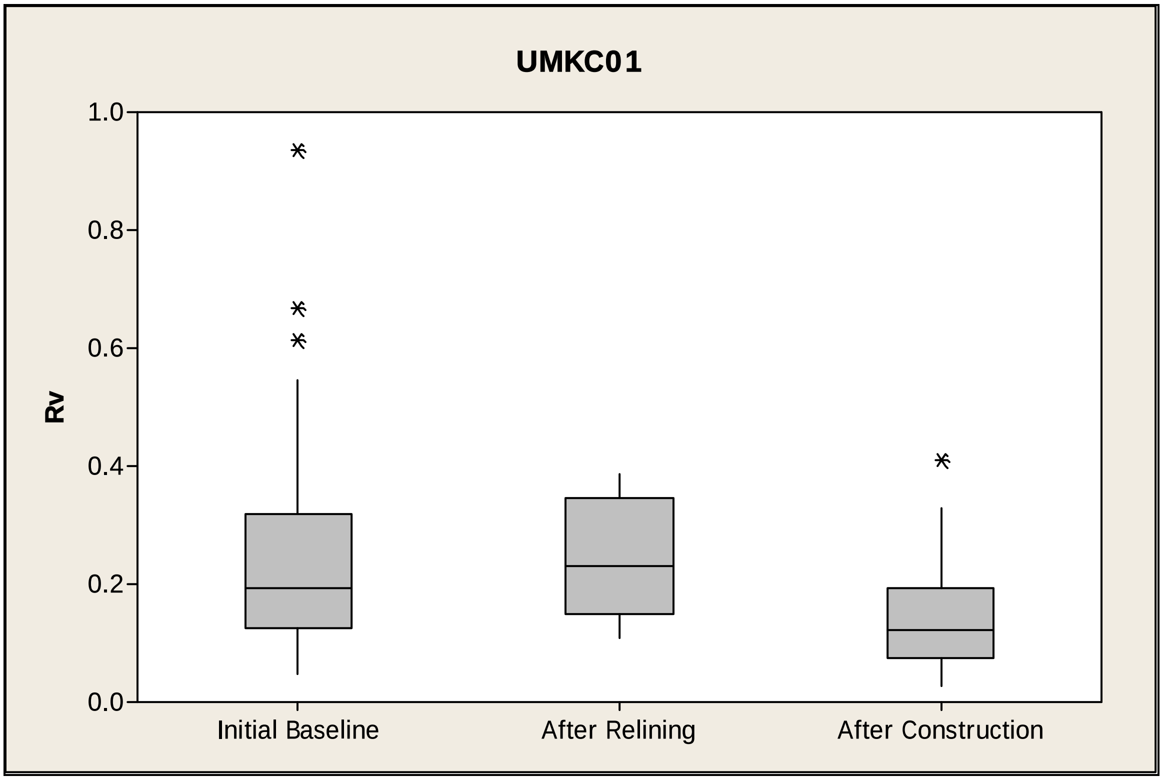


Figure 44. Rv values at UMKC01 during different monitoring periods.

Table 18. Kruskal-Wallis Statistical Tests Comparing Rv Values for Different Monitoring Periods at UMKC01

Group	N	Missing	Median	25%	75%
Initial Baseline	69	0	0.19	0.12	0.315
After Relining	6	0	0.23	0.148	0.345
After Construction	37	0	0.12	0.0723	0.195
H = 12.288 with 2 degrees of freedom. (P = 0.002)					

The differences in the median values among the treatment groups are greater than would be expected by chance; there is a statistically significant difference (P = 0.002).

Table 19. Kruskal-Wallis Statistical Tests Comparing Rv Values for Initial Baseline vs. After Relining Monitoring Periods at UMKC01

Group	N	Missing	Median	25%	75%
Initial Baseline	69	0	0.19	0.12	0.315
After Relining	6	0	0.23	0.148	0.345
Mann-Whitney U Statistic= 173.500					
T = 261.500 n(small)= 6 n(big)= 69 (P = 0.519)					

The difference in the median values between the two groups is not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.519)

Table 20. Kruskal-Wallis Statistical Tests Comparing Rv Values for Baseline vs. After GI Facility Construction Monitoring Periods at UMKC01

Group	N	Missing	Median	25%	75%
InitialBaseline & AfterRelining	75	0	0.2	0.12	0.32
After Construction	37	0	0.12	0.0723	0.195
Mann-Whitney U Statistic= 832.000					
T = 1535.000 n(small)= 37 n(big)= 75 (P = <0.001)					

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = <0.001)

Figure 45 is a plot of the Rv values at UMKC01 comparing the before and after Rv values for different rain depths, while Tables 21, 22, and 23 summarize the Mann-Whitney rank sum test results for these comparisons. The RV differences for the two smallest rain categories are shown to be significantly different (which were at 40 and 33% respectively), but the largest rain category did not show significant differences (at 13% reductions) for the number of observations available. Table 24 summarizes the overall Rv comparison tests showing that the 32% decrease in Rv value observed after the GI facility construction was highly significant.

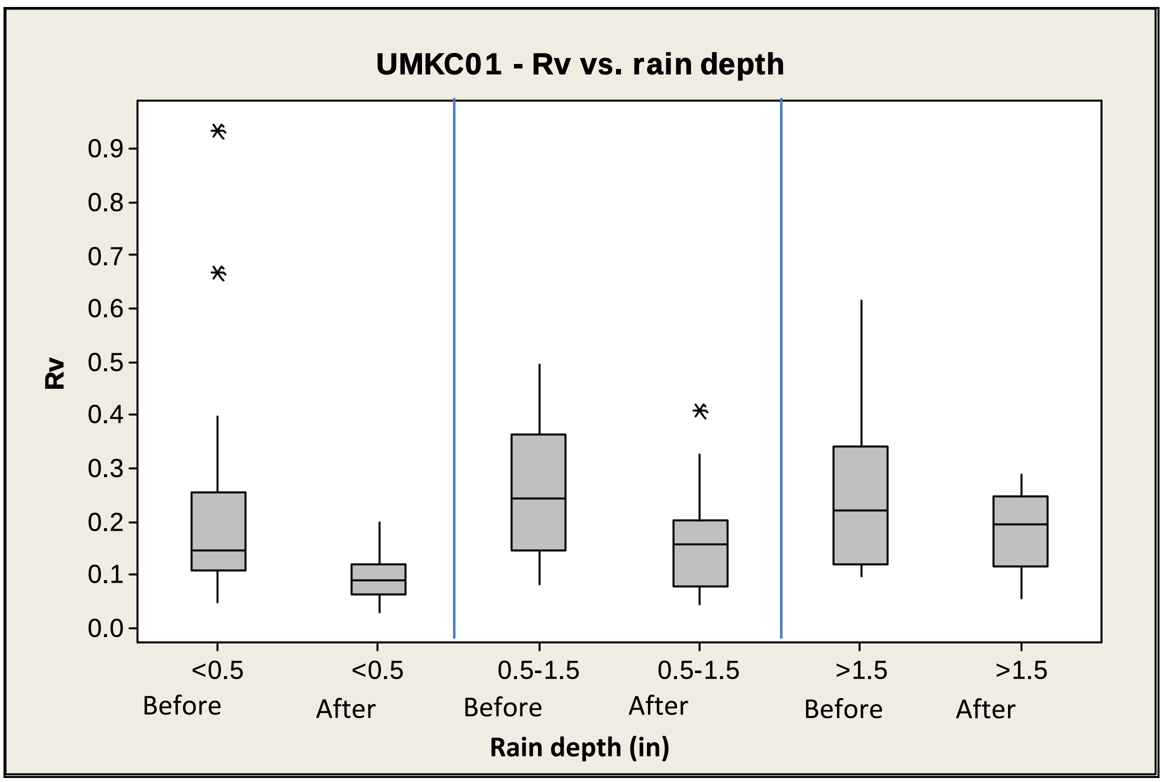


Figure 45. Comparisons of Rv Values at UMKC01 for Before and After GI Facility Construction Monitoring Periods.

Table 21. Mann-Whitney Comparison Tests of Before and After Construction Rv Values at UMKC01 for Rains <0.5 inches

Group	N	Missing	Median	25%	75%
Initial baseline and after relining	37	0	0.15	0.105	0.25
After construction	16	0	0.09	0.06	0.12
Mann-Whitney U Statistic= 133.500					
T = 269.500 n(small)= 16 n(big)= 37 (P = 0.002)					

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = 0.002).

Table 22. Mann-Whitney Comparison Tests of Before and After Construction Rv Values at UMKC01 for Rains between 0.5 and 1.5 inches

Group	N	Missing	Median	25%	75%
Initial baseline and after relining	25	0	0.24	0.15	0.365
After construction	15	0	0.16	0.08	0.2
Mann-Whitney U Statistic= 106.000					
T = 226.000 n(small)= 15 n(big)= 25 (P = 0.023)					

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = 0.023).

Table 23. Mann-Whitney Comparison Tests of Before and After Construction Rv Values at UMKC01 for Rains Larger than 1.5 inches

Group	N	Missing	Median	25%	75%
Initial Baseline and after relining	13	0	0.22	0.12	0.34
After construction	6	0	0.195	0.12	0.245
Mann-Whitney U Statistic= 32.000					
T = 53.000 n(small)= 6 n(big)= 13 (P = 0.568)					

The difference in the median values between the two groups is not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.568).

Table 24. Summary of Statistical Comparisons for Before and After GI Facility Construction at UMKC01

Monitoring period	Dates corresponding to monitoring period	Number of monitored storms in each monitoring period	Flow-Weighted Rv values	% change compared to "initial baseline and after relining (combined)" (p from Mann-Whitney Rank-Sum test)
Initial baseline and after Relining	(03/23/09 – 06/16/10) and (02/24/11 – 03/19/11)	75	0.26	n/a
After construction	04/07/13 – 10/31/2013	37	0.18	32.3% decrease ($p < 0.001$)

Tables 25 and 26 summarize the observed rain and flow conditions for the UMKC02a control area location for all monitoring phases combined. Since there were no changes in the control area during the monitoring period, the runoff characteristics were hypothesized to be similar during the complete monitoring period. Figure 46 is a scatterplot of the monitored rain vs. runoff depth values at this control location, while Figure 47 is a box and whisker plot of the Rv values for the different monitoring periods. It appears that the runoff responses do not differ greatly for the different monitoring periods as the data regions generally overlap. Table 27 shows the results of the Kruskal-Wallis one way analysis of variance on ranks test for the Rv values for these monitoring phases and indicates that no significant differences were found for the number of data observations available. Statistical tests were also conducted comparing the different phases by rain depth; none of those non-parametric Mann Whitney tests indicated any significant differences for the number of observations available at this control area monitoring location. Table 28 is a summary of some of these test results.

Table 25. Rain and Flow Characteristics at UMKC02a Control Area during All Monitoring Phases Combined

	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)	Flow dur. (hrs)	Total pipeflow discharge volume (ft^3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
count	112	115	115	114	115	114	114	114	114	114	114	114	114
sum	703.2	1380.4	91.6			2290.6	3,193,275	48.87					
minimum	0.6	0.3	0.1	0.47	0.01	0.3	397	0.01	0.1	0.0	0.3	0.04	0.1
maximum	36.6	64.5	3.5	2.85	0.67	74.3	292,467	4.48	47.3	3.6	55.0	3.12	21.7
average	6.3	12.0	0.8	0.95	0.11	20.1	28,011	0.43	3.8	0.4	12.6	0.45	2.7
median	4.1	8.2	0.6	0.95	0.07	16.0	12,289	0.19	1.5	0.2	10.4	0.33	1.9
standard deviation	6.8	11.2	0.7	0.52	0.12	13.1	48,752	0.75	5.8	0.7	9.9	0.46	2.7
COV	1.1	0.9	0.9	0.5	1.1	0.7	1.7	1.7	1.5	1.7	0.8	1.0	1.0

Table 26. Rain, Runoff and Rv Monitored Values at UMKC02a Control Location during Different Monitoring Periods

	Initial Baseline			After Relining			After Construction		
	Rain Depth (in)	Runoff Depth (in)	Flow-Weighted Rv	Rain Depth (in)	Runoff Depth (in)	Flow-Weighted Rv	Rain Depth (in)	Runoff Depth (in)	Flow-Weighted Rv
Count	71	71	71	6	6	6	37	37	37
Sum, or flow-weighted	58.50	32.99	0.56	2.68	1.15	0.43	30.29	14.73	0.49
Average	0.82	0.46	0.49	0.45	0.19	0.37	0.82	0.40	0.38
Median	0.55	0.22	0.35	0.33	0.09	0.38	0.79	0.13	0.21

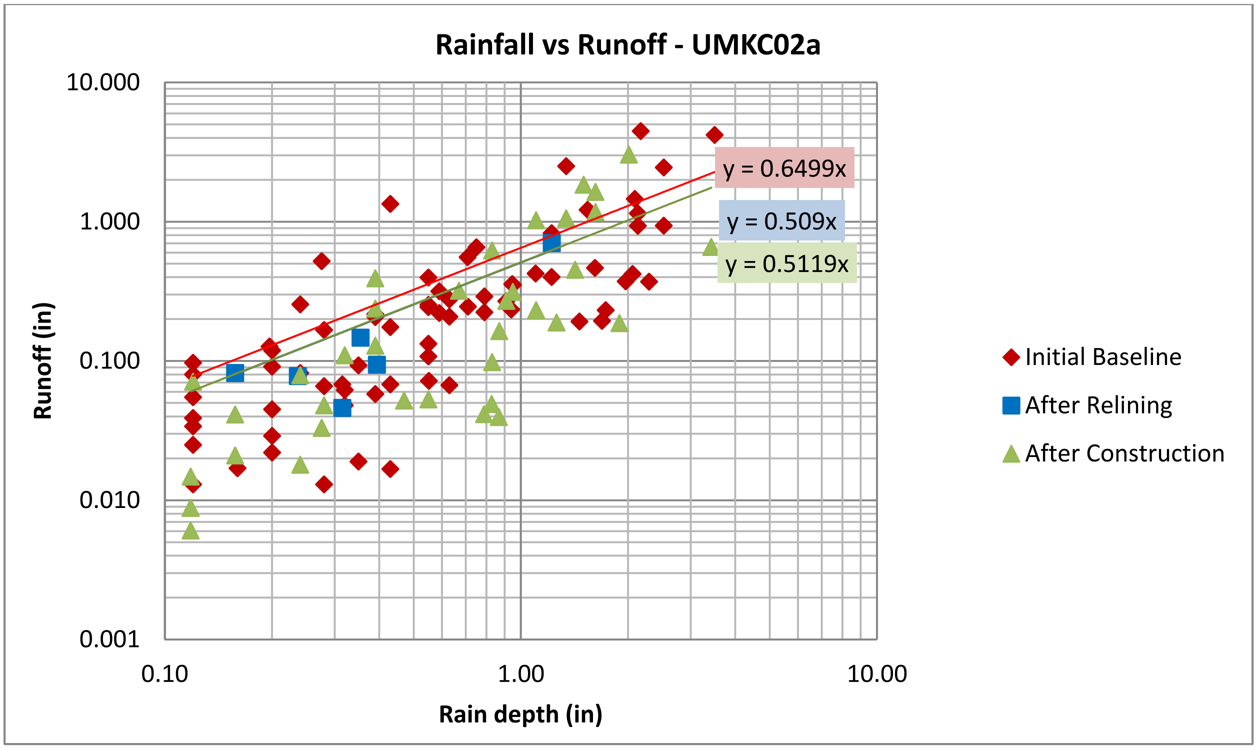


Figure 46. Scatterplot of monitored rain vs. runoff conditions at UMKC02a control location for different monitoring periods.

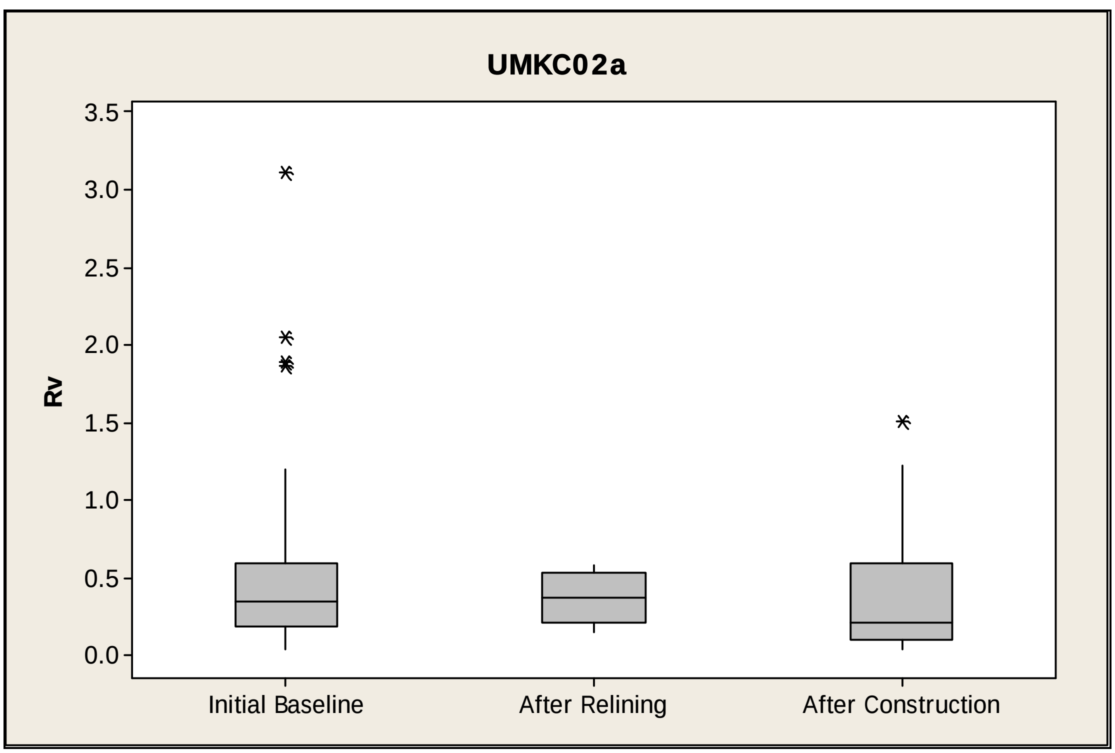


Figure 47. Box and whisker plot of monitored Rv values at UMKC02a control area for different monitoring phases.

Table 27. Kruskal-Wallis One Way Analysis of Variance on Ranks Test for Rv Values from Different Monitoring Phases at UMKC02a Control Location

Group	N	Missing	Median	25%	75%
Initial Baseline	71	0	0.35	0.19	0.6
After Relining	6	0	0.375	0.218	0.535
After Construction	37	0	0.21	0.105	0.6
H = 3.547 with 2 degrees of freedom. (P = 0.170)					

The differences in the median values among the treatment groups are not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.170)

Table 28. Summary Statistics Comparing Rv Values for Different Monitoring Periods at UMKC02a Control Location

Monitoring period corresponding to changes in UMKC01; no modifications were made to UMKC2a during the study period	Dates corresponding to monitoring period	Number of monitored storms in each monitoring period	Flow-Weighted Rv values	% change compared to "initial baseline and after relining (combined)" (p from Mann-Whitney Rank-Sum test)
Initial baseline and After Relining period for UMKC01	(03/23/09 – 06/16/10) and (02/24/11 – 03/19/11)	75	0.56	n/a
After construction period for UMKC01	04/07/13 – 10/31/13	37	0.49	no significant difference (*p =0.28*)

Initially, the monitoring plan was to simultaneously track the Rv values in the test/pilot watershed represented by monitoring at UMKC01 with the Rv values in the control watershed represented by runoff volumes at UMKC02a plus UMKC02b plus UMKC03. The ratio of the Rv values for each rain for the test/pilot watershed to the control watershed was expected to change as the GI facilities were constructed. However, as noted above, there were problems with the control area flow sensors due to a high degree of equipment failure and large uncertainities in the observations (most likely due to the generally low flow depths in large pipes at the monitoring locations). In addition, the volumetric runoff values (Rv) were suspect (also likely due to watershed area questions). The control area sensor at UMKC02a appeared to provide better results that at UMKC02b and UMKC03 and was therefore further examined. Figure 48 is a time series plot of the ratio of the test/pilot area Rv values to the control area Rv values (UMKC01/UMKC02a), with the pre-construction and post-construction events indicated. These is no apparent decrease in this ratio after the construction for the general range of the observations, but more of the post-construction Rv values appear to be somewhat less that the pre-construction values. The box and whisker plot in Figure 49 indicates a couple of very large pre-construction Rv values, but otherwise, the post-construction Rv ratios have a larger 25 to 75 percentile range (represented by the height of the box). Statistical tests were also used to compare these data sets, as shown in the Table 29 results for the Mann-Whitney rank sum test. The average of the Rv ratios was somewhat less for post-construction data, but the differences were not significant (P = 0.66) for the amount of data available. Additional Mann-Whitney rank sum nonparametric statistical tests were conducted examining all combinations of the Rv ratios for all rain categories with no statistically significant differences identified for the number of observations available. The variabilities in the ratios for each group were very large likely due to the high variable runoff measurements obtained at the UMKC02a control location. Table 30 summarizes the test results.

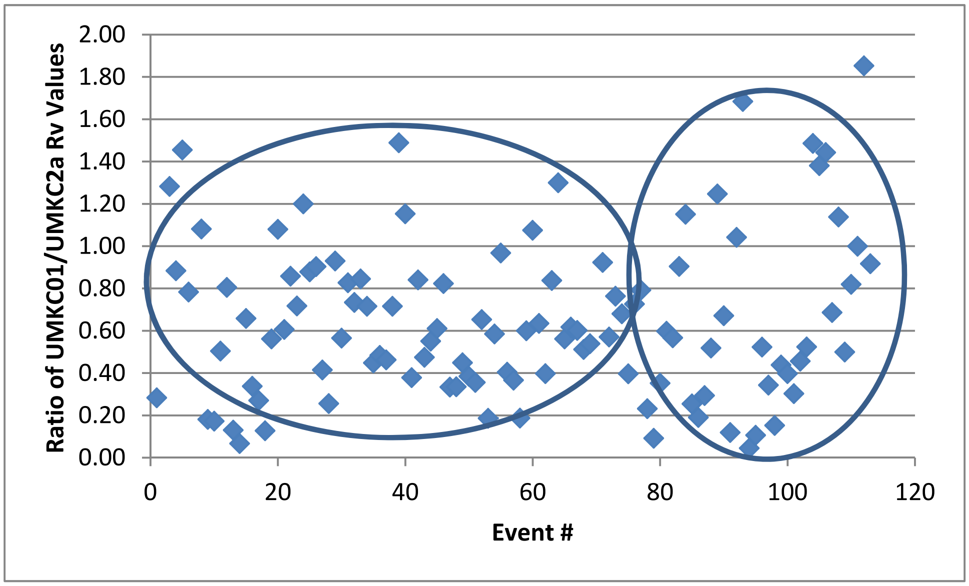


Figure 48. Time series plot of UMKC01/UMKC02a (test/control) Rv ratios for different monitoring periods (two large values removed).

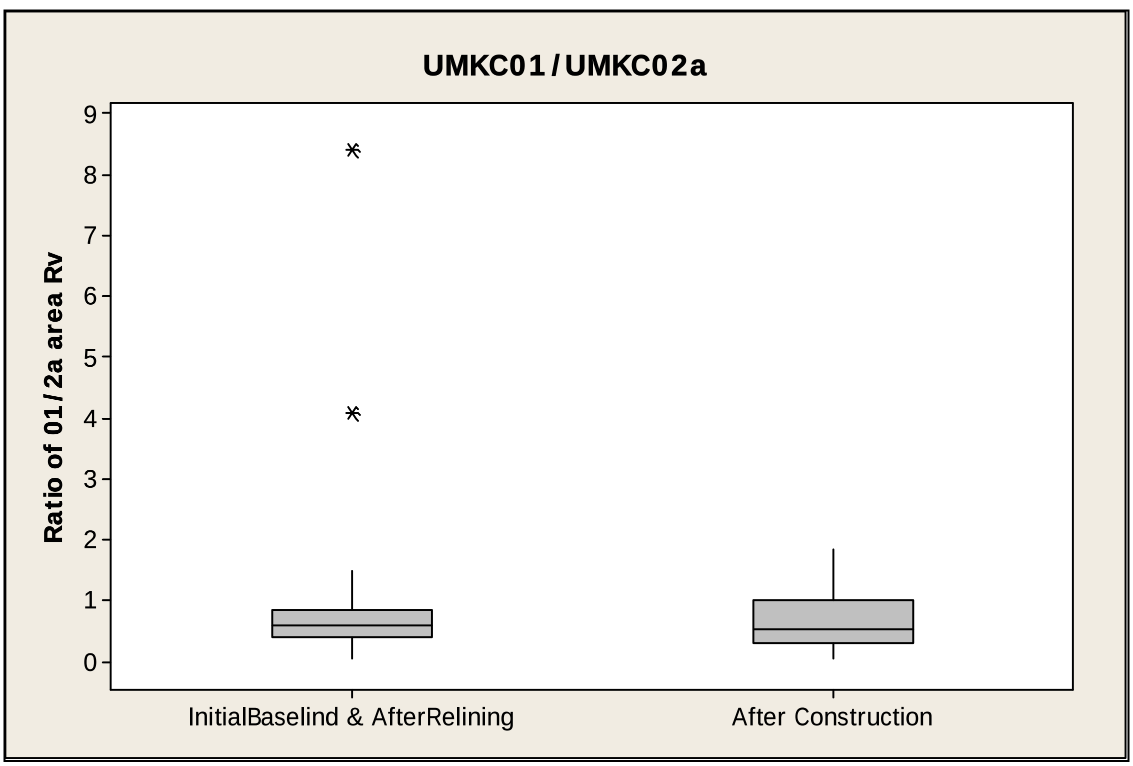


Figure 49. Ratio of UMKC01 to UMKC02a Rv values during monitoring phases.

Table 29. Mann-Whitney Rank Sum Test Comparing Ratios of UMKC01/UMKC02a Rv Values

Group	N	Missing	Median	25%	75%
InitialBaseline & After Relinig	75	0	0.606	0.397	0.845
After Construction	37	0	0.524	0.298	1.021
Mann-Whitney U Statistic= 1316.000					
T = 2019.000 n(small)= 37 n(big)= 75 (P = 0.661)					

The difference in the median values between the two groups is not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.661)

Table 30. Summary of Test Statistics Comparing Ratios of UMKC01/UMKC02a Rv Values for Different Site Conditions

Monitoring period corresponding to changes in UMKC01; no modifications were made to UMKC2a during the study period	Dates corresponding to monitoring period	Number of monitored storms in each monitoring period	Ratio of Flow-Weighted Rv values (01/2a)	% change compared to "initial baseline and after relining (combined)" (p from Mann-Whitney Rank-Sum test)
Initial baseline and After Relining period for UMKC01	(03/23/09 – 06/16/10) and (02/24/11 – 03/19/11)	75	0.46	n/a
After construction period for UMKC01	04/07/13 –10/31/13	37	0.37	no significant difference (*p =0.661*)

Green Infrastructure Effects on Peak Discharge Rates

Green infrastructure facilities are also expected to reduce the peak discharge rates. Figure 50 is a box and whisker plot comparing the pre-construction to the post-construction peak runoff rates at the test/pilot monitoring location at UMKC01 for different peak rain intensities. All of the rain gauge data indicated peak rain intensities in one of these four categories due to the number of rain gauge tips that occurred in a 5 minute period. The 0.47 in/hr rate corresponds to 1 tip of 0.04 inches in 5 minutes for example, 0.95 in/hr corresponds to 2 tips per 5 minutes, 1.42 in/hr corresponds to 3 tips per 5 minutes, and 1.9 in/hr corresponds to 4 tips per hour (the maximum observed). This plot indicates that the pre-construction range of peak flow rates for each of these peak rain intensity categories was larger than for the post-construction range, but statistical tests did not indicate any significant differences in the groups for the number of observations available.

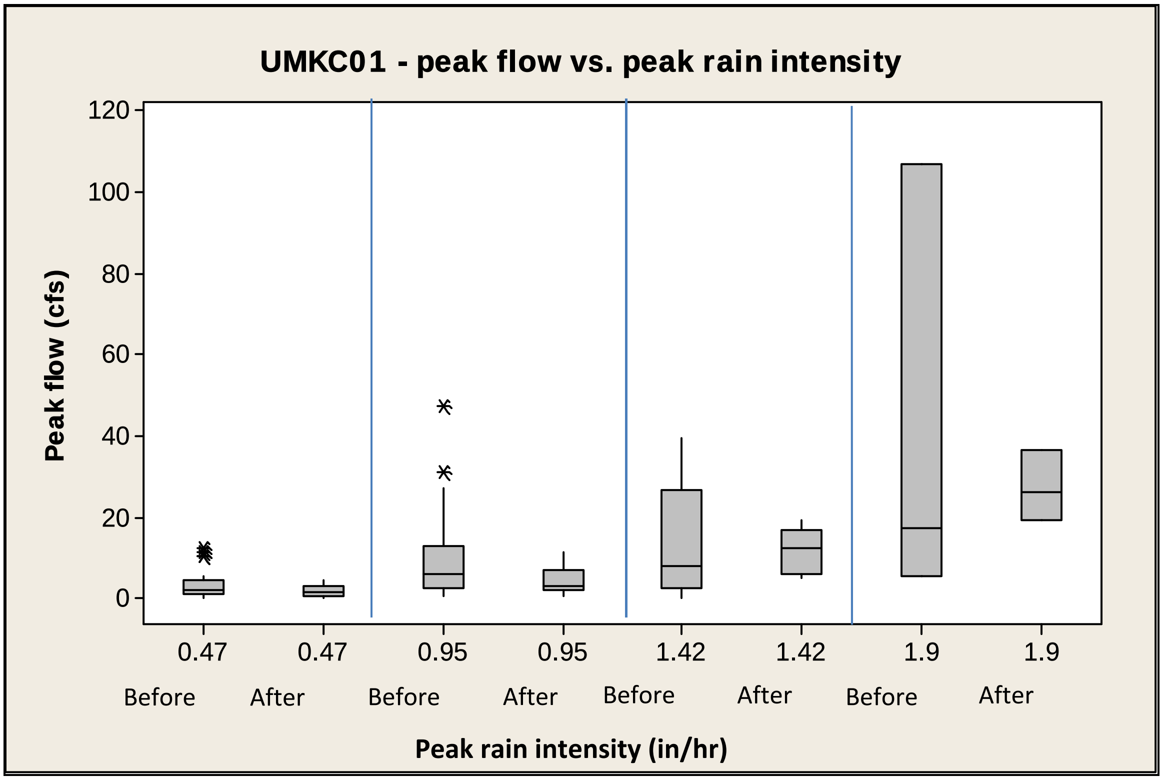


Figure 50. Box and whisker plots of peak flow rates before and after GI facility construction at UMKC01 test/pilot area for different rain intensity categories.

Model Calibration using Site Monitoring Data

Table 31 lists the seven events that were observed in the test (pilot) watershed, after the re-lining was completed and before the construction of the stormwater controls that were used for calibrating WinSLAMM runoff volumes. Also shown are the modeled runoff volume values and the ratio comparing the observed to the modeled flow values.

Table 31. Rain data with observed and modeled flow characteristics after re-lining of the combined sewer and before the construction of the stormwater controls (final baseline conditions).

Event #	Rain start date	Rain start time	Rain end date	Rain end time	Total rain (in)	Total pipe flow discharge volume (ft^3)	modeled runoff (ft^3)	ratio of flows (obs/modeled)
119	1/22/2011	12:20	1/23/2011	3:40	0.12	2,246	6,021	0.37
120	2/24/2011	9:00	2/25/2011	3:00	0.35	33,011	21,124	1.56
121	2/26/2011	13:50	2/28/2011	8:20	1.22	129,497	103,676	1.25
122	3/4/2011	11:10	3/5/2011	1:40	0.24	23,412	12,694	1.84
123	3/8/2011	8:10	3/9/2011	1:10	0.39	13,056	24,597	0.53
124	3/13/2011	23:00	3/15/2011	0:25	0.20	10,708	10,035	1.07
125	3/19/2011	14:30	3/20/2011	4:15	0.32	5,900	18,662	0.32
				Sum:	2.84	217,830	196,809	Ratio of sums: 1.11

For these seven monitored events, the sum of the observed flows was about 11% greater than the sum of the modeled flows. Figure 51 is a scatterplot showing the observed versus the modeled total flows for each of these seven events. As shown, these are all close to the line of equivalent values.

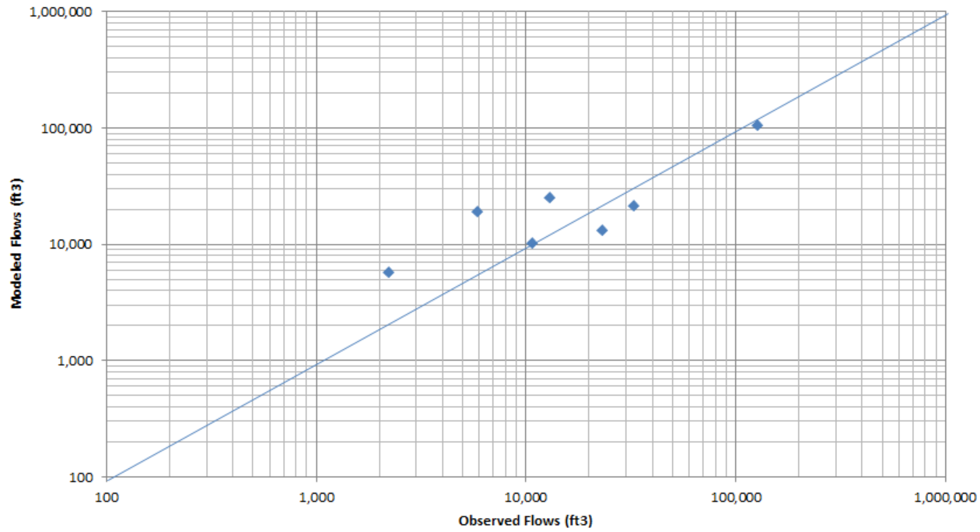


Figure 51. Observed versus modeled flows during final baseline conditions (after re-lining)

Figure 52 is a box plot that compares the single event observed flows to the modeled flows. The boxes substantially overlap, but the observed flows are much more variable than the modeled flows.

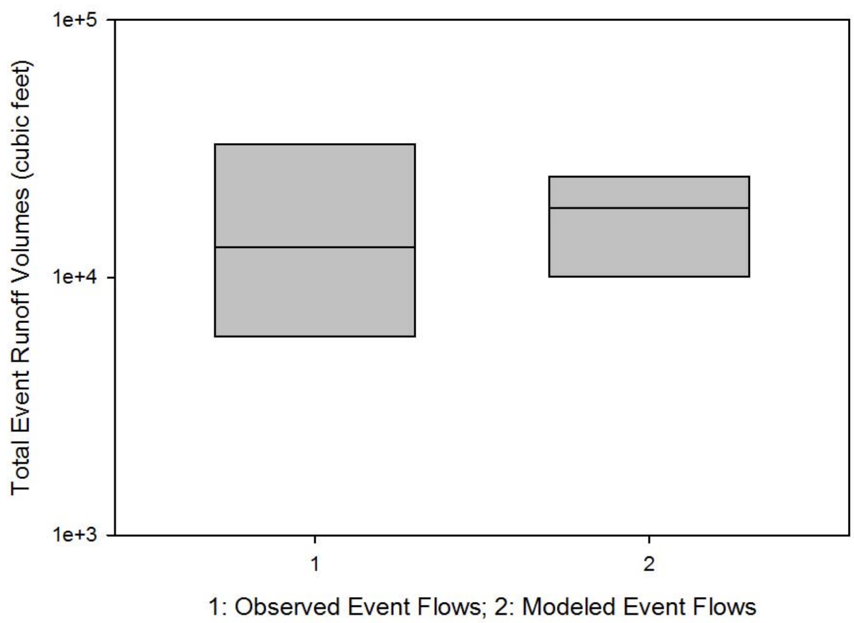


Figure 52. Variabilities of runoff volumes observed and modeled.

The Mann-Whitney Rank Sum Test (using SigmaPlot ver 11) was used to compare the observed with the modeled runoff volumes. The seven pairs of data were not sufficient to detect a significant difference in the two sets of runoff volumes:

Group	N	Missing	Median	25%	75%
obs flows	7	0	13,056	5,900	33,011
modeled flows	7	0	18,662	10,035	24,597

Mann-Whitney U Statistic= 23.000
T = 51.000; n (small) = 7; n (big) = 7; P (est.) = 0.898; P (exact) = 0.90

The difference in the median values between the two groups is not great enough to exclude the possibility that the difference is because of random sampling variability; there is not a statistically significant difference (p = 0.90).

Variability and Uncertainty with WinSLAMM Modeling

WinSLAMM contains various Monte Carlo components that enable uncertainly to be evaluated during the model runs. These are available for the infiltration rates for the various infiltration and biofiltration devices and for the pollutant concentrations. During field investigations, these model parameters have been recognized as having the greatest variabilities that are not explained by the model. The Monte Carlo elements are described by probability distributions, with average and coefficient of variation values (COV) provided, and assumes log-normal distributions of the actual values. If these uncertainty options are selected, the model randomly selects a value of the parameter from this distribution for each rain event. The long-term simulations therefore result in calculated concentrations and loadings of the constituents and the runoff volumes that vary in a similar manner as observed during monitoring. For the calculations in this report, when different options are being directly compared, the Monte Carlo option was not used because that could affect the average ordering of the different options. However, several different scenarios were repeatedly analyzed and the different concentrations and loads were examined to estimate the likely variability in the relative model outcomes. The absolute errors are described above with the calibration and verification discussions. As noted, the flow calculations might have a low to moderate bias by underreporting the expected runoff quantities.

Table 32 summarizes these Monte Carlo results by showing the groups of constituents associated with different ranges of variability and uncertainty. As an example, when calibrated, WinSLAMM is able to predict the runoff volumes and particulate solids loads more accurately than the other constituents. With COV values (the relative standard deviations compared to the average values) of about 5% of the average values, the 95% confidence range of these constituents would be within about 10% of the average (for normal distributions, about 95% of the data are obtained within ±2 times the standard deviation values). However, for zinc concentrations, the 95% confidence interval is about ±20 to 30% of the average values. The bacteria data has an even wider range for the confidence interval, as expected (±60 to 70% for *Escherichia coli* and even wider for fecal coliforms). The relative runoff volume (the primary stormwater characteristic of interest in the Kansas City project) and TSS mass load reduction predictions for the alternative stormwater control programs are expected to be more precise.

Table 32. Expected modeling variability

COV (standard deviation as a percentage of average concentration)	
< 10%	runoff volume Rv total and filterable total Kjeldahl nitrogen (TKN) TSS total and filterable copper total and filterable lead nitrates
10 to 15%	total and filterable zinc total and filterable chemical oxygen demand (COD) TDS
30 to 35%	*E. coli* bacteria total and filterable phosphorus
65%	fecal coliform bacteria

Summary of System-wide Monitoring Observations and Model Calibration

Runoff monitoring was conducted in the combined sewer system at several locations in the test and control watersheds. Events were monitored after the sewer was rehabilitated, and these data were used as a new baseline condition. WinSLAMM evaluated the test (pilot) and control watershed conditions during the two monitoring periods (post re-lining, as the new baseline versus after construction of controls) to verify the rainfall-runoff calibration based on site development characteristics and the actual rains monitored.

Figure 53 compares the pre and post-construction Rv values for the test/pilot watershed as monitored at UMKC01. The post-construction Rv values are apparently smaller than the pre-construction Rv values, as expected. Comparisons of pre and post-construction Rv values were also made for different rain categories which had similar apparent trends, especially for the small rains. The RV differences for the two smallest rain categories were significantly different (which were at 40 and 33% for <0.5 inches and 0.5 to 1.5 inches respectively), but the largest rain category (>1.5 inches) did not have significant differences (at 13%) for the number of observations available. Biofilters remove larger fractions of flows from smaller events based on their storage capacity and other design features. Table 33 summarizes the overall reduction in flows observed in the test/pilot watershed which were calculated to be about 32% on a flow-weighted basis and were highly significant (p <0.001).

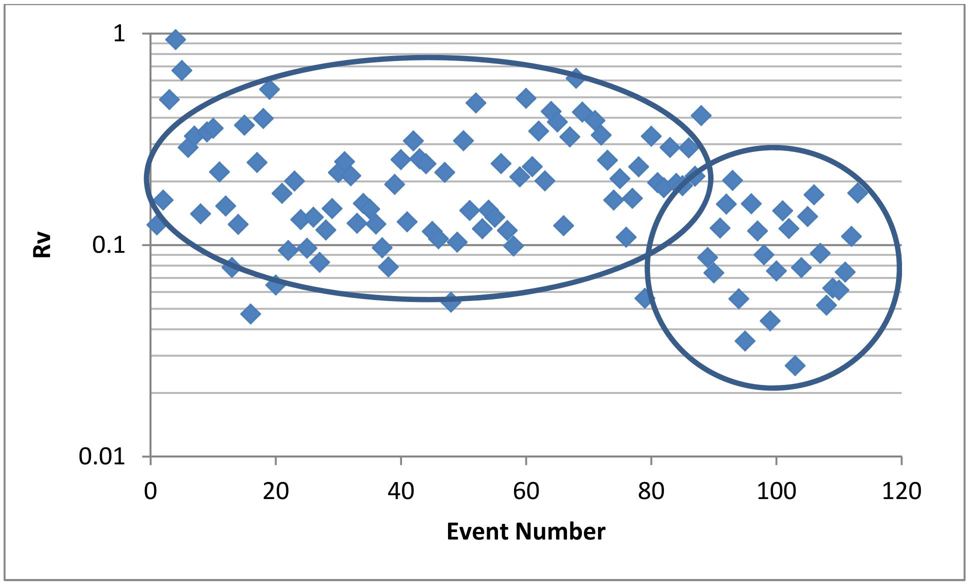


Figure 53. Volumetric runoff coefficients at UNKC01 before and after GI construction.

Table 33. Summary of Statistical Comparisons for Before and After GI Facility Construction at UMKC01

Monitoring period	**Dates corresponding to monitoring period**	**Number of monitored storms in each monitoring period**	**Flow-Weighted Rv values**	**% change compared to "initial baseline and after relining (combined)" (p from Mann-Whitney Rank-Sum test)**
Initial baseline and after Relining	(03/23/09 – 06/16/10) and (02/24/11 – 03/19/11)	75	0.26	n/a
After construction	04/07/13 – 10/31/2013	37	0.18	32.3% decrease (p <0.001)

Figure 54 is a scatterplot showing the observed versus the modeled test (pilot) watershed area total flows for each of the events during the after re-lining baseline period. As shown, these are all close to the line of equivalent values.

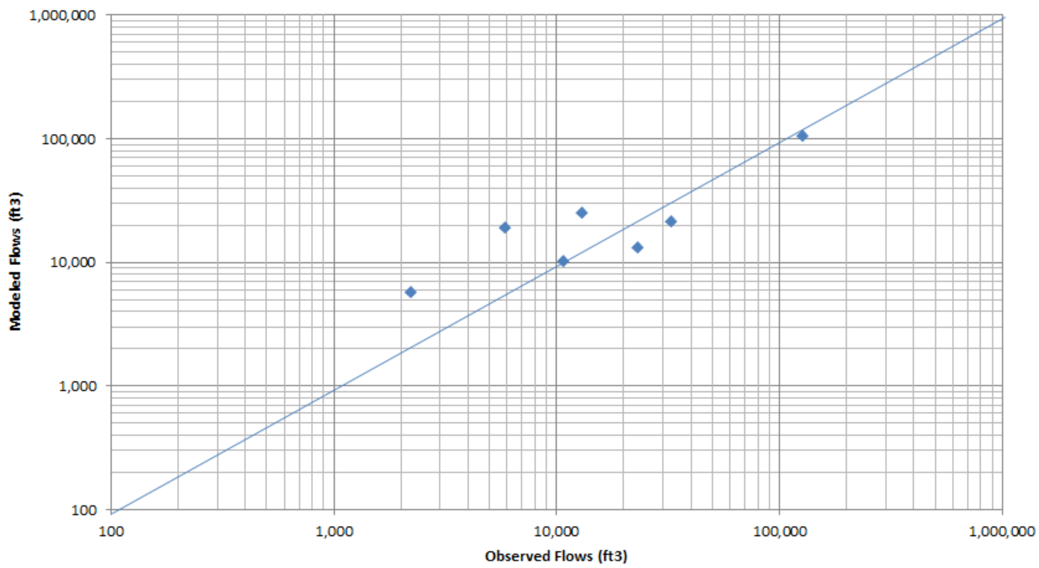


Figure 54. Observed versus modeled flows during final baseline conditions (after re-lining).

5. Small-Scale Drainage Areas, Performance Monitoring during Rain Events, and Associated Model Calibration Factors for Stormwater Controls

The objective of this chapter is to summarize the drainage area characteristics and monitored performance of the GI stormwater control devices. For each of the devices, areas for different urban surfaces (including rooftops, streets, landscaped areas, sidewalks, driveways, and parking lots) have been measured using aerial photos and site visits, plus GIS shapefile layers. This information, along with the attributes of the designs of each control, was used as input for the WinSLAMM model. Table 34 lists the information sources that were used to obtain the information described in this chapter and in Appendix A.

Table 34. Sources of small-scale drainage area information

Document/Material	Source
100% design plans and street side topographic info.	https://sites.tetratech.com/projects/100-KCADC/default.aspx
Subwatershed shapefile	Mr. John Riverson, Tetra Tech (from Sustain KC maps)
Sewer network shapefile	Mr. John Riverson, Tetra Tech (from Sustain KC maps)
Stormwater controls shapefile	Mr. John Riverson (TT) and https://sites.tetratech.com/projects/100-KCADC/default.aspx
Bing aerial maps	Basemap available in ArcMap 10
Listing of locations and sampling equipment	Table supplied by Dr. Deb O'Bannon, UMKC
USGS topo maps (10 ft contours)	Basemap available in ArcMap 10
Topographic maps (1 ft) jpgs	Project map supplied by Dr. Deb O'Bannon, UMKC
"Monitoring water balance of a rain garden by installation of flow monitoring devices on a residential property." Thesis by Jason Nall, UMKC.	https://sites.tetratech.com/projects/100-KCADC/default.aspx
Site photos	Robert Pitt – Site visit on October 25 and 26, 2012

Table 35 is a list of the ten monitoring station locations in the test (pilot) watershed prepared by UMKC researchers. Figure 55 shows these locations on the map of the test area. They were mostly along East 76th Street and East 76th Terrace. Detailed site information is contained in Appendix A, including subarea drainages for each area draining to each stormwater control being monitored (including the land surface breakdowns). Example designs for each type of stormwater control being monitored are included in Appendix B. Appendix C contains detailed information concerning the observed infiltration rates in each of the stormwater controls. The information presented in these three appendices was then used to calibrate WinSLAMM for the site-specific conditions. The following summaries in this section focus on the infiltration rates observed during the monitored events.

Table 35. Locations of Monitoring Stations

No.	Stormwater control type	Address	Design station
1	Curb Extension Biofilter	1324 E 76th St.	19+79.61
2	Curb Extension Biofilter	1325 E 76th St.	19+79.61
3	Curb Extension Biofilter	1419 E 76th Terr.	26+51.65
4	Curb-Cut Biofilter	1612 E 76th St.	31+31.12
5	Curb-Cut Biofilter	1336 E 76th St.	21+29.95
6	Site abandoned due to theft of monitoring equipment		
7	Shallow Curb-Cut Biofilter w/ Smart Drain	1140 E 76th Terr.	15+37.75
8	Shallow Curb-Cut Biofilter w/ Smart Drain	1222 E 76th St.	16+28.15
9	Cascading swale biofilter	1112 E 76th Terr.	12+18.80
10	Private rain garden	1312 E. 79th St.	Mrs. Thomas
11	Private rain garden	1505 E. 76th St.	Mrs. Moss

Source: UMKC

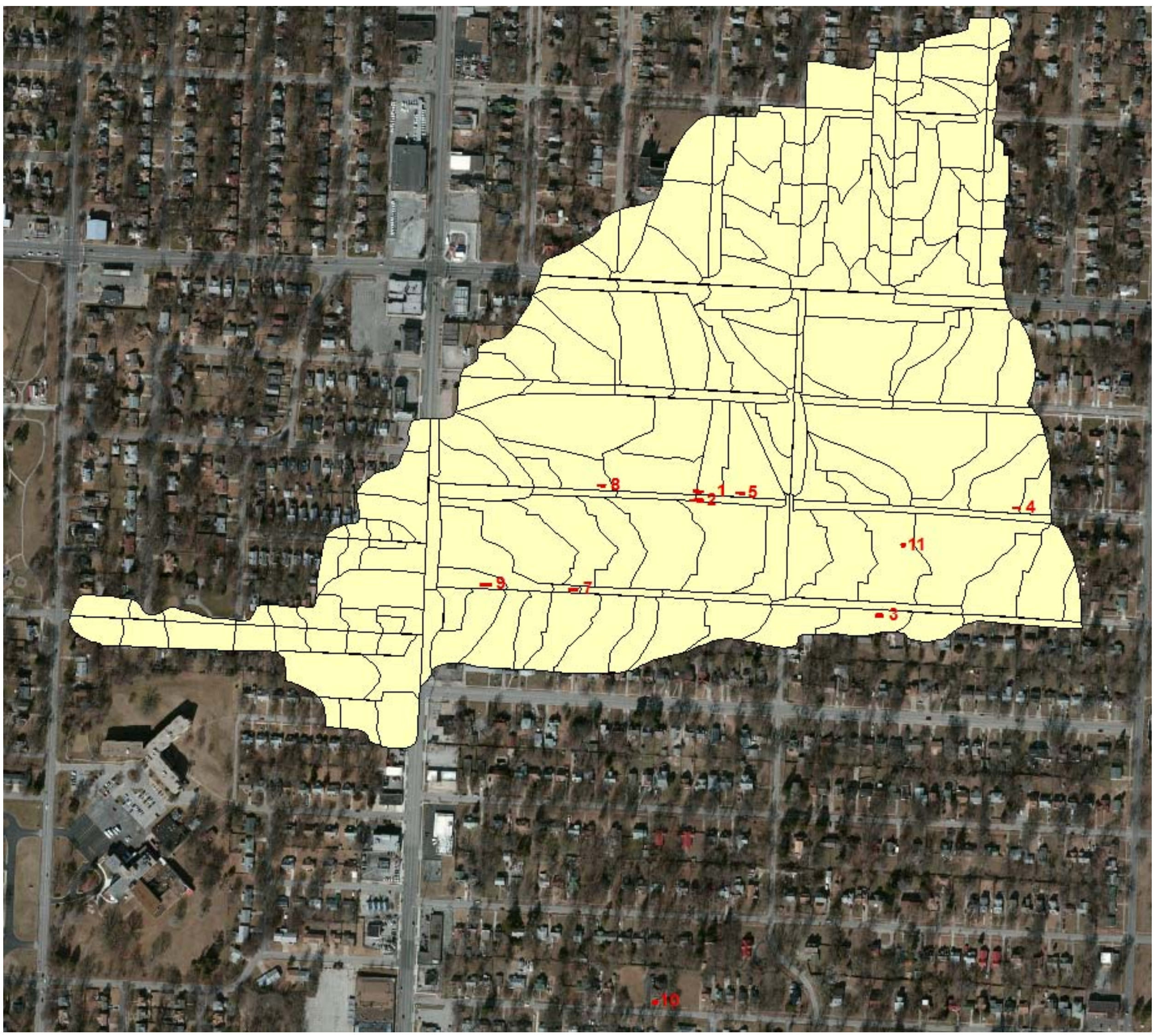
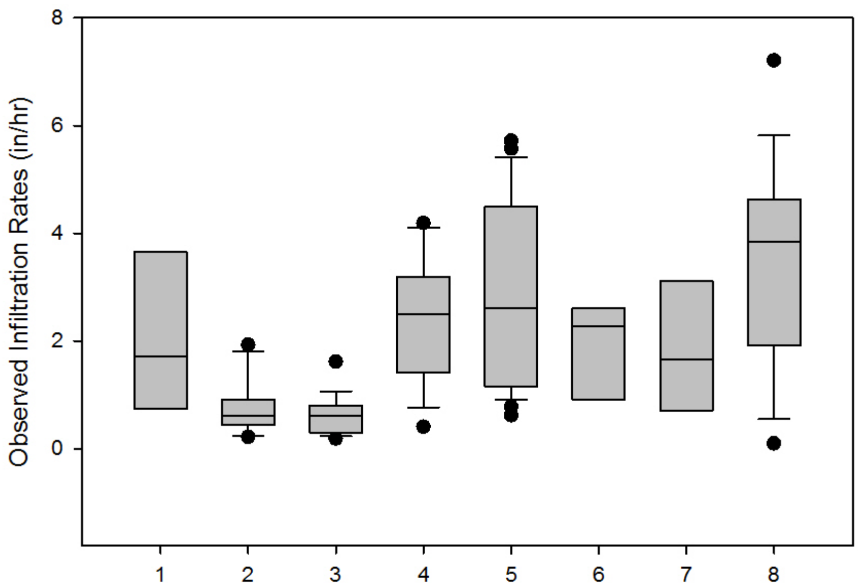


Figure 55. Location of stormwater controls monitored in test (pilot) watershed.

Infiltration Rates in Monitored Biofilters

Tables in Appendix C list the infiltration rates calculated using the monitored data obtained during each rains. As shown in Appendix C, plots of the water depths in the biofilters were used to identify recession limbs of the infiltration periods as recorded from the water level recorders in the biofilters. In some cases, runoff was still entering the devices during the infiltration period. The basic infiltration rates were all very consistent for a recession limb, with no decreasing rate with time. This indicates that the systems were already saturated, and the rates represent the lowest values occurring. If measured during inflowing conditions, the rates were listed as greater than the calculated rates.

Figure 56 is a SigmaPlot (ver 11) box and whisker plot that compares the infiltration rates observed at the eight different biofilter installations. There were 5 to 26 observations at each site, for about 110 total separate infiltration observations. Kruskal-Wallis statistical analyses indicated that at least one of the sites was significantly different (p = <0.05) from the others.



site 1: 1324 E. 76th St. curb-extension biofilter; site 2: 1325 E. 76th St. curb-extension biofilter;
site 3: 1419 E. 76th Terrace curb-extension biofilter; site 4: 1612 E. 76th St. curb-cut biofilter;
site 5: 1336 E. 76th St. curb-cut biofilter; site 6: 1140 E. 76th Terrace shallow curb-cut biofilter with SmartDrain;
site 7: 1222 E. 76th St. shallow curb-cut biofilter with SmartDrain; site 8: 1112 E. 76th Terrace cascading swale biofilter

Figure 56. Plot of Observed Infiltration Rates at Monitored Biofilters during Rain Events

Table 36. Summary of Infiltration Rate Observations at Each Monitoring Location during Rain Events (in/hr)

Monitoring Location	Number of Infilt. Rate Obs.	Avg. Infilt. Rate	Std. Dev.
1324 E 76th St	8	2.3	2.0
1325 E 76th St	17	0.78	0.48
1419 E 76th Terrace	17	0.64	0.35
1612 E 76th St	14	2.4	1.2
1336 E 76th St	26	2.9	1.7
1140 E 76th Terrace SD	5	1.9	0.89
1222 E 76th St SD	6	1.8	1.2
1112 E 76th Terrace	17	3.5	1.9

The following box and whisker plot and Mann-Whitney Rank Sum test results compares the infiltration rates at the two locations having significantly slower infiltration rates (1325 E. 76th St. and 1419 E. 76th Terrace, both curb-extension biofilters) than the other locations. The biofilter media at these two locations were likely compacted during construction, and/or the media contained more fines than the other locations.

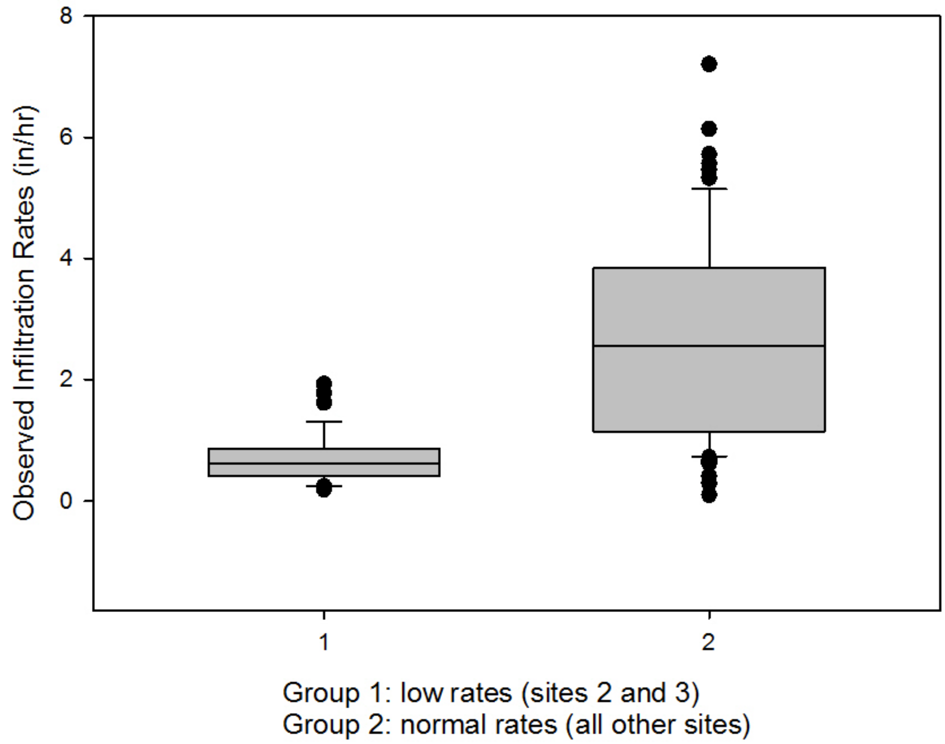


Figure 57. Site Groupings of Infiltration Rates Observed during Rain Events

Table 37. Mann-Whitney Rank Sum Test to Compare Two Groups of Infiltration Rate Site Conditions (in/hr)

Group	N	Missing	Median	25%	75%
slow 1325 and 1419	34	0	0.62	0.41	0.86
fast all others	76	0	2.56	1.14	3.85

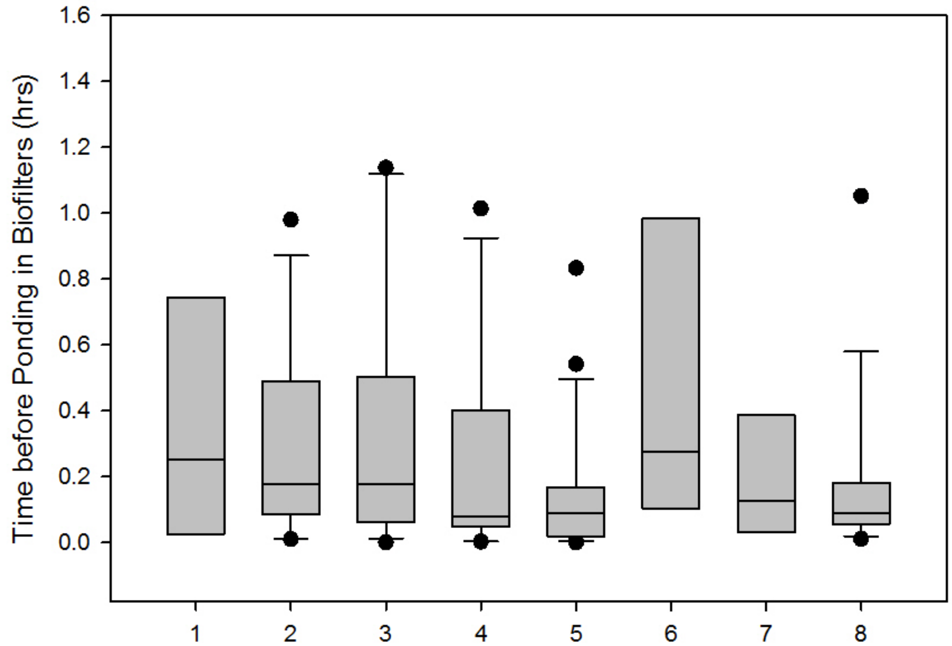
Mann-Whitney U Statistic= 263.000

T = 858.000; n (small) = 34; n (big) = 76; P = <0.001

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = <0.001).

Runoff Duration before Ponding in Biofilters

A similar analysis was conducted to investigate the time since the beginning of flow entering the biofilters to the beginning of ponding. The total amount of rain or runoff before ponding might be a more useful measure, but those data were not available. The time before ponding was obtained from the inflow hydrograph and ponding depth measurements presented in Appendix C. Figure 58 is a box and whisker plot showing the ranges and percentiles of these durations before ponding for each of the eight monitored biofilters.



site 1: 1324 E. 76th St. curb-extension biofilter; site 2: 1325 E. 76th St. curb-extension biolfilter;
site 3: 1419 E. 76th Terrace curb-extension biofilter; site 4: 1612 E. 76th St. curb-cut biofilter;
site 5: 1336 E. 76th St. curb-cut biofilter; site 6: 1140 E. 76th Terrace shallow curb-cut biofilter with SmartDrain;
site 7: 1222 E. 76th St. shallow curb-cut biofilter with SmartDrain; site 8: 1112 E. 76th Terrace cascading swale biofilter

Figure 58. Plots of Time to Ponding after Start of Rain for Each Monitored Biofilter Site

Table 38. Summary of Time to Ponding after Start of Rain at Monitored Biofilters (hrs)

Monitoring Location	# of Events	Avg. Time to Ponding	Std Dev
1324 E 76th St	8	0.38	0.40
1325 E 76th St	17	0.30	0.30
1419 E 76th Terr	19	0.30	0.36
1612 E 76th St	14	0.25	0.32
1336 E 76th St	26	0.15	0.21
1140 E 76th Terr SD	5	0.49	0.55
1222 E 76th St SD	6	0.20	0.20
1112 E 76th Terr	17	0.18	0.25

The following box and whisker plot and Mann-Whitney statistical test results summarize the time to ponding for these two significantly different groups of data. The biofilters at 1336 E 76th St and 1112 E 76th Terrace had the shortest ponding periods (about 5 minutes), and were also the two sites with the highest average infiltration rates (possibly the opposite of what would be expected, but may be due to other site characteristics). The other sites had average ponding times of about 10 minutes since the start of the observed runoff.

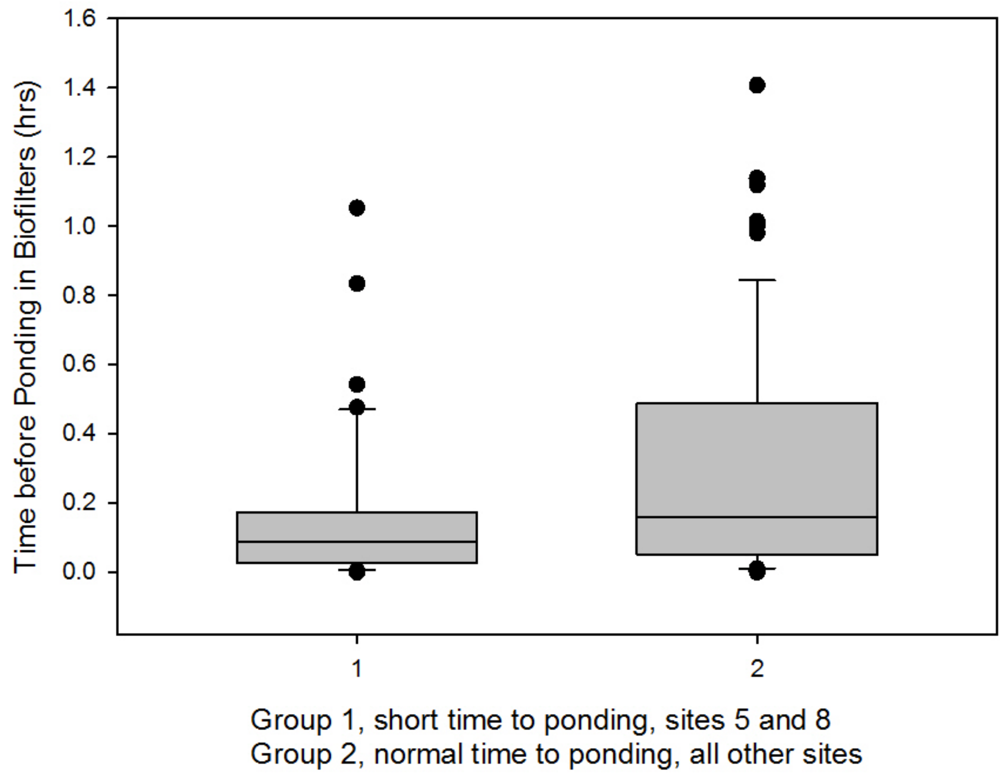


Figure 59. Short and Long Time to Ponding Groups at Monitored Biofilters

Table 39. Mann-Whitney Rank Sum Test to Compare Two Groups of Time to Ponding at Monitored Sites (hrs)

Group	N	Median	25%	75%
short 5 and 8	43	0.088	0.026	0.17
long 1, 2, 3, 4, 6, 7	69	0.16	0.051	0.49

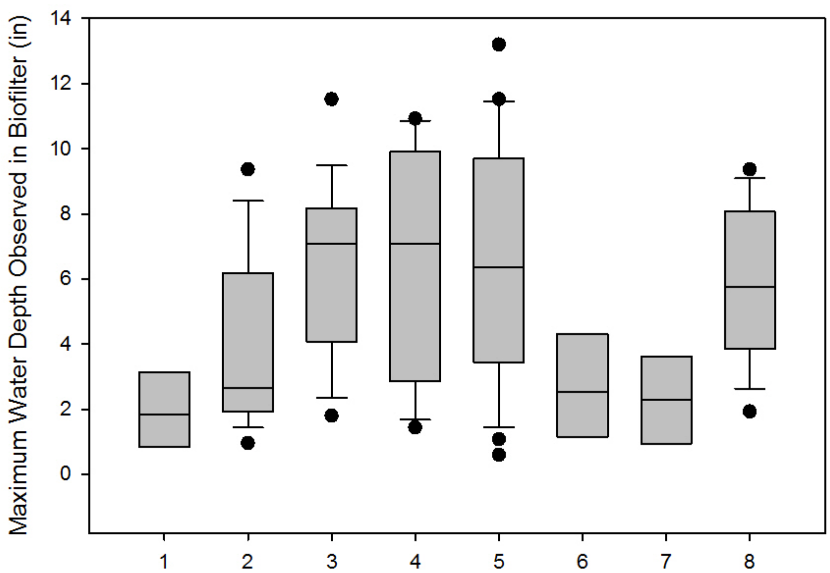
Mann-Whitney U Statistic= 1139.500

T = 2085.500; n (small) = 43; n (big) = 69; P = 0.040

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = 0.040).

Maximum Water Depth Observed in Biofilters

The maximum depth observed in the biofilters was also obtained for each monitored event in each of the biofilters and examined using similar procedures as described above. Figure 60 is a box and whisker plot showing the median and ranges for each of the eight sites.



site 1: 1324 E. 76th St. curb-extension biofilter; site 2: 1325 E. 76th St. curb-extension biofilter
site 3: 1419 E. 76th Terrace curb-extension biofilter; site 4: 1612 E. 76th St. curb-cut biofilter
site 5: 1336 E. 76th St. curb-cut biofilter; site 6: 1140 E. 76th Terrace curb-cut biofilter with SmartDrain
site 7: 1222 E. 76th St. curb-cut biofilter with SmartDrain; site 8: 1112 E. 76th Terrace cascading swale biofliter

Figure 60. Plots of Maximum Water Depth Observed in Monitored Biofilters during Rains

Table 40. Summary of Maximum Water Depth Observed at Monitored Biofilters (in)

Monitored Location	# of Obs.	Avg. of Max. Depth	Std Dev
1324 E 76th St	8	3.0	3.6
1325 E 76th St	17	4.1	2.7
1419 E 76th Terrace	19	6.4	2.6
1612 E 76th St	14	6.5	3.4
1336 E 76th St	26	6.6	3.7
1140 E 76th Terrace SD	5	2.7	1.8
1222 E 76th St SD	6	2.4	1.4
1112 E 76th Terrace	17	5.8	2.3

The following box and whisker plot and Mann-Whitney Rank Sum statistical test result summarizes the maximum depths observed for two groups of sites that were found to be significantly different. The median depth for the sites having the deepest standing water depths was about 6.5 inches, while it was only about 2.5 inches for the other sites. With a typical infiltration rate of 2.5 inches per hour, this standing water would be expected to be completely infiltrated within a few hours after the rain ended, much less than typical 24 to 72 hr maximum requirements to prevent nuisance conditions.

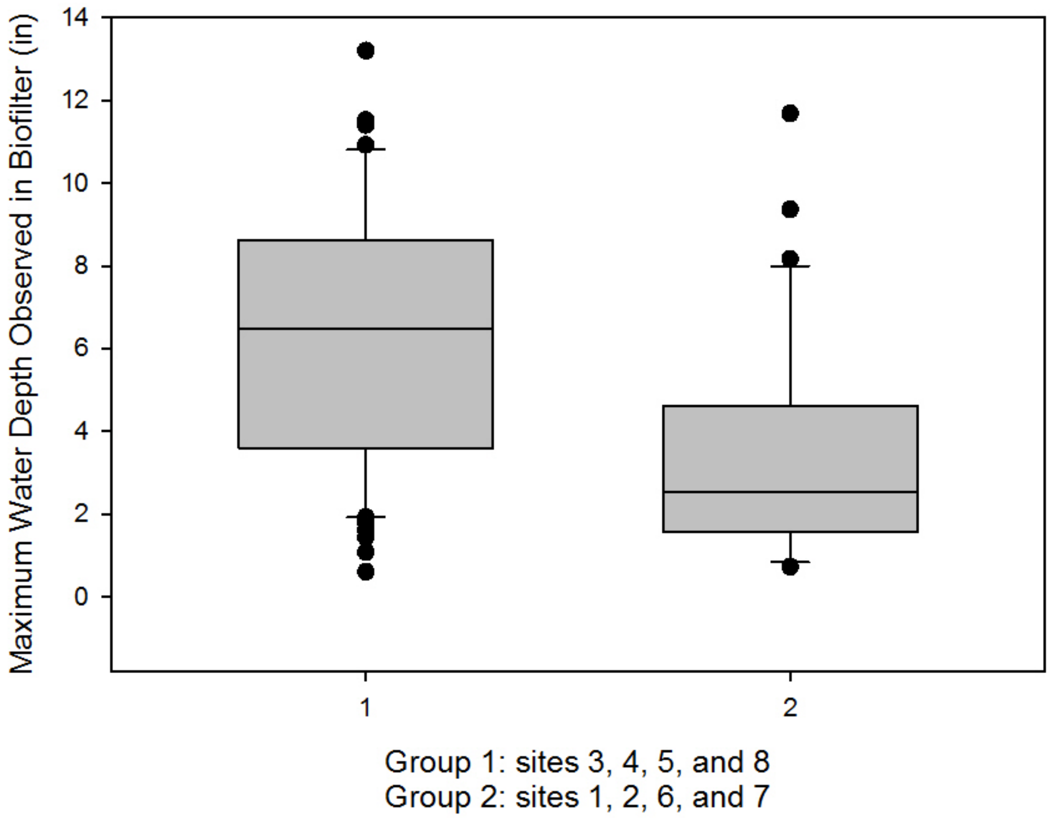


Figure 61. Deep and Shallow Maximum Water Depth Groups for Monitored Biofilters

Table 41. Mann-Whitney Rank Sum Test to Compare Two Groups of Maximum Ponding Depths at Monitored Sites (in)

Group	N	Missing	Median	25%	75%
deep 3, 4, 5, 8	76	0	6.5	3.6	8.6
low and medium	36	0	2.5	1.6	4.6

Mann-Whitney U Statistic= 606.500

T = 1272.500; n (small) = 36; n (big) = 76; P = <0.001

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference (P = <0.001).

Laboratory Column Tests of Infiltration Rates as a Function of Compaction

The effects of different compaction levels on the infiltration rates through the Kansas City soil media were examined during laboratory column testing in the UA Environmental Engineering Laboratory, as part of ongoing dissertation research by Redahegn Sileshi focusing on biofiltration media and underdrain systems (Sileshi 2013). Figure 62 shows photographs of the media, illustrating its heterogeneous nature.

Figure 62. Media samples obtained from Kansas City biofilters.

Four-in. (100 mm) diameter PVC pipe (Charlotte Pipe TrueFit 100 mm PVC Schedule 40 Foam-Core Pipe) purchased from a local building supply store in Tuscaloosa, Alabama, was used to construct the columns for these tests. The columns were filled with about 2 in (5 cm) of cleaned pea gravel purchased from a local supplier. To separate the gravel layer from the media layer, a permeable fiberglass screen was placed over the gravel layer and then filled with the soil media. The media layer was about 1.5 ft (0.5 m) thick. The bottom of the columns had a fiberglass window screen secured to contain the media as shown in Figure 63.

Figure 63. Lab column construction for flow test using Kansas City soil media: bottom of the columns secured with a fiberglass window screen, mixed soil media, and soil compaction.

Three levels of compaction levels were tested. The tests were compacted by hand, standard proctor, and modified proctor methods. Both standard and modified proctor compactions follow ASTM standard (D 1140-54). The standard proctor compaction hammer is 24.4 kN and has a drop height of 300 mm. The modified proctor hammer is 44.5 kN and has a drop height of 460 mm. For the standard proctor setup, the hammer is dropped on the test soil 25 times on each of three soil layers, while for the modified proctor test, the heavier hammer was also dropped 25 times, but on each of five soil layers and using the heavier hammer. The modified proctor test therefore results in a much more compacted soil and usually reflects the most compacted soil observed in the field. The hand compaction is done by gently hand pressing the media material to place it into the test cylinder with as little compaction as possible, with no voids or channels. The hand compacted soil columns therefore have the least amount of compaction. The densities were directly determined by measuring the weights and volume of the media material added to each column. The density of the media column with hand compaction was 1.00 g/cm^3, the density of the standard proctor media column was 1.13 g/cm^3, and the density for the modified proctor media column was 1.12 g/cm^3.

The media in the Kansas City biofilters was composed of 30% planting soil, 20% organic compost, and 50% sand (APWA/MARC 2012). The soil media had a relatively large median particle size (D_{50}) of about 1.9 mm and a very large uniformity coefficient (C_u) of 39, as shown in the soil's particle size distribution plot (Figure 64).

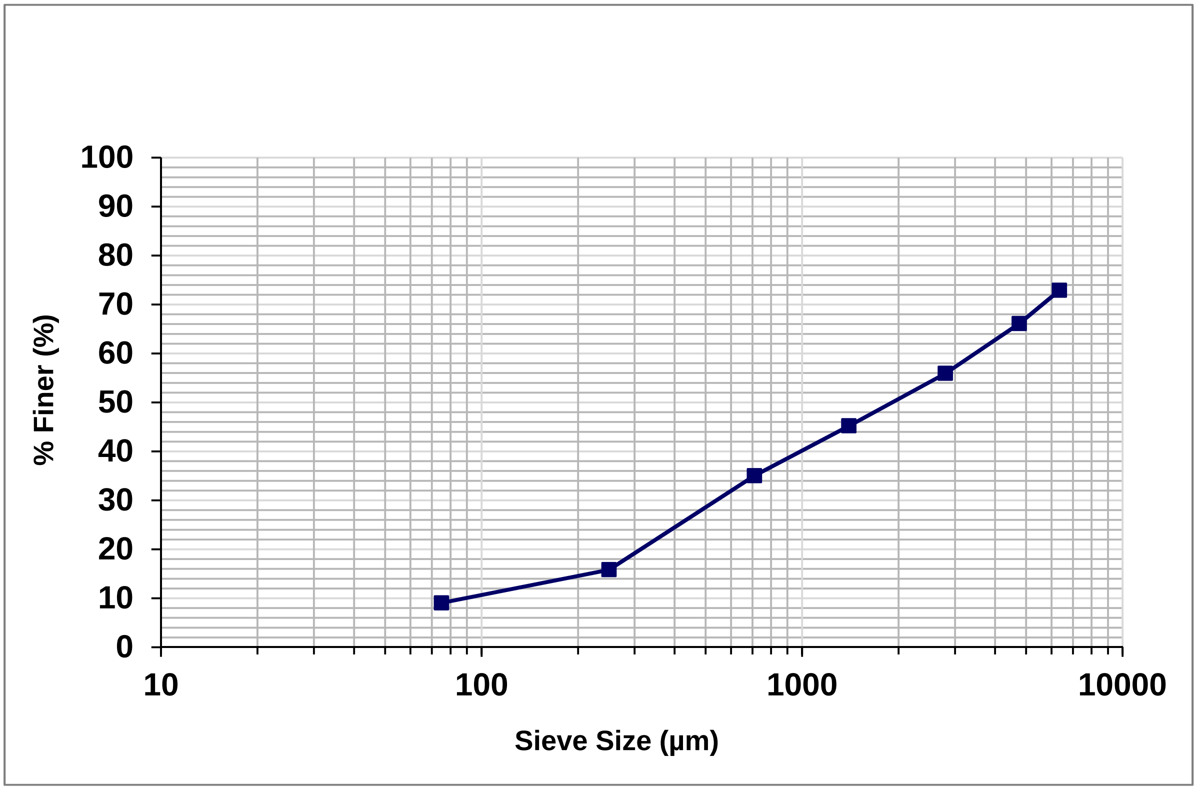


Figure 64. Particle size distribution of Kansas City soil media used during lab compaction test.

Composite samples of treatment media from several Kansas City biofilters were analyzed by Auburn University's Soil Testing Laboratory. Summaries of the biofilter media texture and soils laboratory reports are shown in Tables 42 and 43. The media had a clay content of 10% (higher than desired for biofilter media). According to the AU laboratory tests, the median size of the media was 2,000 um (2 mm) which was the same as the UA laboratory results reported above.

Organic matter improves soil structure and soil tilth, and helps to provide a favorable medium for plant growth. Soils with large amounts of clay generally require large amounts of organic matter. The organic matter content of the Kansas City biofilter media was 15%, indicating the soil is in good condition. The media had CEC and pH values of 0.83 meq/100g (low for biofilter media) and 7.4 respectively. As described by the Auburn University Soil Testing Laboratory (Mitchell and Huluka 2011), these characteristics vary from soil to soil, with sandy soils generally having CEC values ranging from 0 to 4.6 meq/100g and loam soils having CEC values ranging from 4.6 to 9.0 meq/100g. The sodium content of soil affects soil texture and pH, and high sodium (in the presence of low calcium and magnesium, as defined by the sodium adsorption ratio, or SAR) can cause severe flow restrictions if affected by snowmelt waters. The biofilter media obtained from the biofilter sites had a SAR value of about 1.5, well below typical problematic levels of about 5 and greater.

Table 42. Summary of Biofilter Media Texture Report.

Biofilter Media from	Sand (%)	Silt (%)	Clay (%)	Textural Class
Kansas City	58.8	31.3	10.0	Sandy Loam

Table 43. Kansas City Biofilter Media Constituent Concentrations

Calcium (Ca) (mg/kg)	5,903
Potassium (K) (mg/kg)	304
Magnesium (Mg) (mg/kg)	266
Phosphorus (P) (mg/kg)	69
Aluminum (Al) (mg/kg)	3
Arsenic (As) (mg/kg)	1.0
Boron (B) (mg/kg)	1
Barium (Ba) (mg/kg)	14
Cadmium (Cd) (mg/kg)	<0.1
Chromium (Cr) (mg/kg)	<0.1
Copper (Cu) (mg/kg)	0
Iron (Fe) (mg/kg)	1
Manganese (Mn) (mg/kg)	13
Molybdenum (Mo) (mg/kg)	<0.1
Sodium (Na) (mg/kg)	155
Nickel (Ni) (mg/kg)	0.3
Lead (Pb) (mg/kg)	<0.1
Zinc (Zn) (mg/kg)	1
Total Phosphorus (P) (mg/kg)	1009
Nitrogen (N) (%)	0.74
Carbon (C) (%)	8.59
Sulfur (S) (%)	0.18
Organic Matter (OM) (%)	14.8
Sodium Adsorption Ratio (SAR)	1.50
Moisture (%)	44.4
pH	7.41
H_2O availability (cm^3/cm^3)	0.12
Cation Exchange Capacity (CEC) (meq/100g)	0.83

Laboratory Measurement of Porosity of Kansas City Soil Media

Porosity (ϕ) is the portion of the soil's volume that is not occupied by solid material. The pore volume of the soil media was determined from the volume of water needed to saturate the media in the columns. To keep water from coming out of the soil columns during the porosity measurements, a seal was formed using plastic sheeting sealed with duct tape on the inside, wet mat secured using screw-type radiator hose clamps on the outside and bottom of the columns. The bottoms of the columns were placed in buckets so that when the seals were lifted up, the water flowed into the buckets (Figure 65).

The volume of the void in the 2-in pea gravel placed in the bottom of each column was subtracted from the total void volume of a water-saturated soil and gravel layer in the columns to obtain the void in soil media alone. The porosity of the soil media alone for the hand compacted media column was 0.36, 0.15 for the standard proctor compaction tests, and 0.25 for the modified proctor tests.

Figure 65. Laboratory column setup for porosity and infiltration measurements

Laboratory Infiltration Results

The infiltration rates through the soil media were measured in each column using municipal tap water. The surface ponding depths in the columns ranged from 11 to 14 in (28 - 36 cm). Infiltration rates in the soil media were determined by measuring the rates of the water level drops with time until apparent steady state rates were observed.

Observed infiltration data for different test trials were fitted to the Horton infiltration equation by using multiple nonlinear regressions to estimate f_c (the saturated soil infiltration rate), f_o (the initial rate), and k (the rate coefficient). The saturated rates were of greatest interest as they would apply during most of the event durations. The infiltration rates of the saturated media ranged from 0.4 to 0.8 in/hr for the hand compaction tests, 0.4 to 0.9 in/hr for the standard proctor compaction tests, 0.03 to 0.33 in/hr for the modified proctor compaction tests. The COV of the laboratory infiltration rates through the soil media were 0.36, 0.41, and 1.1 for hand compaction, standard proctor, and modified proctor compaction tests, respectively. Figures 66, 67a, and 67b are plots of the data and the derived Horton equations with fitted curves for the different test trials, comparing different compaction conditions. Previous researches indicated that soil compaction has a significant on the infiltration rates (Gregory et al. 2006; Pitt et al. 2008b; Thompson et al. 2008; Sileshi et al. 2012a, 2012b); however the effect of soil compaction on the infiltration rates for the Kansas City media was not observed, except for the modified proctor compaction tests.

The following are the infiltration rates measured in the field during the actual rains. The low rate category corresponds to the laboratory observations during the hand and standard proctor column tests. The high rate measurements are likely associated with media having a more uniform or larger particle size characteristic (or both), or infiltration rates before saturation of the media. As noted on the particle size distribution plot, more than 90% of the media is larger than 100 µm, with appreciable fractions clearly in the coarse sand category. Media with large amounts of sand do not compact as much as media having more fines, because of the structural support of the sand grains. However, these materials usually have greater infiltration rates than measured during these column tests. The organic and the clay content of the Kansas City media were large which would reduce the effective typical pore sizes, resulting in lower infiltration rates. The uniformity coefficient was also quite large which also adversely affects the infiltration rates, and the high clay content would make the media more susceptible to compaction.

- Low: average 0.70 in/hr; range 0.19 to 1.9
- High: average 2.7 in/hr; range 0.10 to 7.2

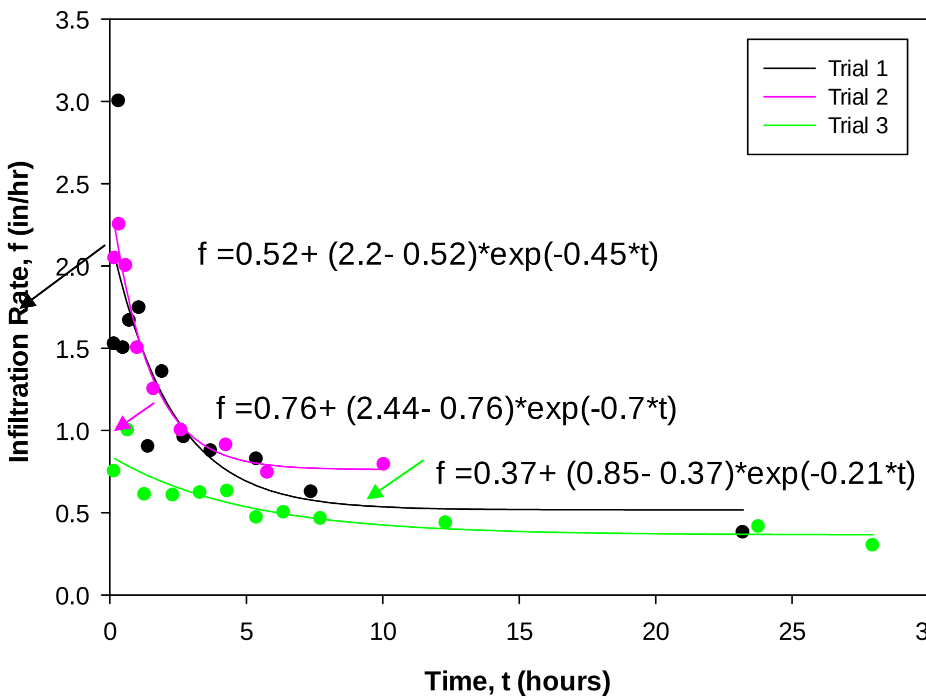


Figure 66. Laboratory infiltration measurements fitted with Horton equations: hand compaction tests for Kansas City biofilter media.

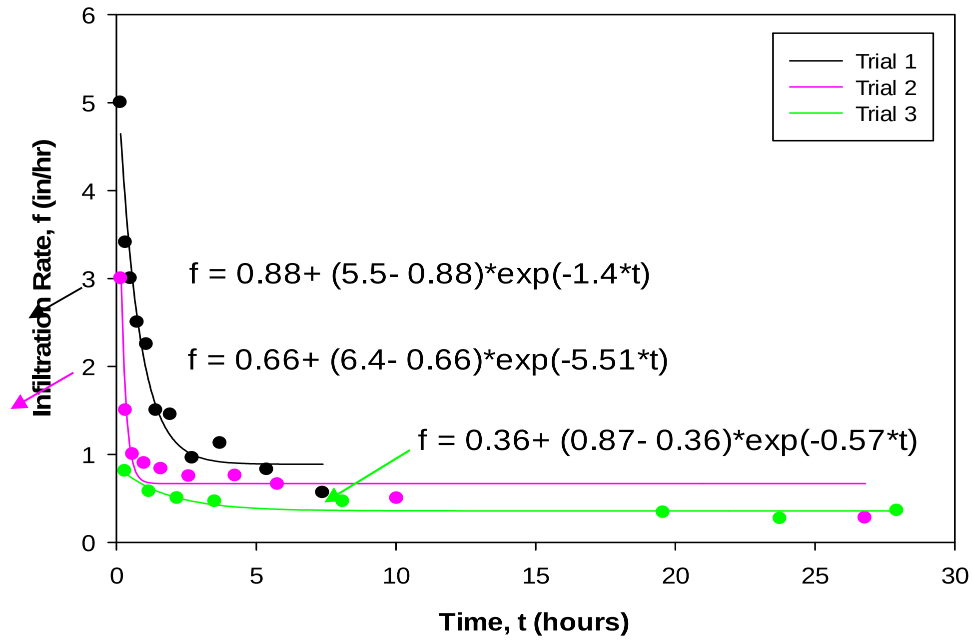


Figure 67a. Laboratory infiltration measurements fitted with Horton equations: standard proctor compaction test for Kansas City biofilter media.

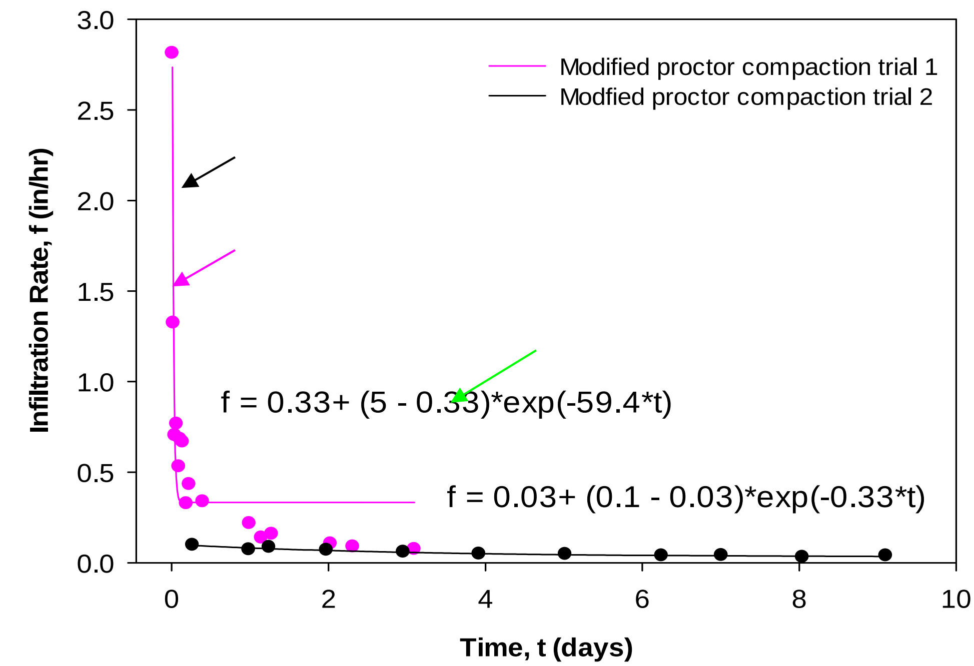


Figure 67b. Laboratory infiltration measurements fitted with Horton equations: modified proctor compaction test for Kansas City biofilter media.

Influent Water Quality to Curb-Side Biofilters

The UMKC team sampled water coming into the biofilters and discharged by underdrains or overflows. When sufficient sample volumes were available, the UA team also analyzed the samples for TSS, SSC, and PSD. UA analyzed 49 influent, 4 overflow, and 6 underdrain samples. For the other events, there were no underdrain or overflow samples, with almost the entire study period runoff being infiltrated by the biofilters. The methods used were ASTM, EPA, USGS, or *Standard Methods* for TSS and SCC that have been described and compared by Clark and Siu 2008; Clark and Pitt 2008; and Clark et al. 2008.

Figure 68 is a PSD plot for the influent samples. The median particle size (by mass) is about 30 µm, and about 25% were larger than 100 µm. Table 44 lists the variability for each particle size range. The COV (the standard deviation divided by the mean, COV) is much smaller for the larger particles than for the small particles.

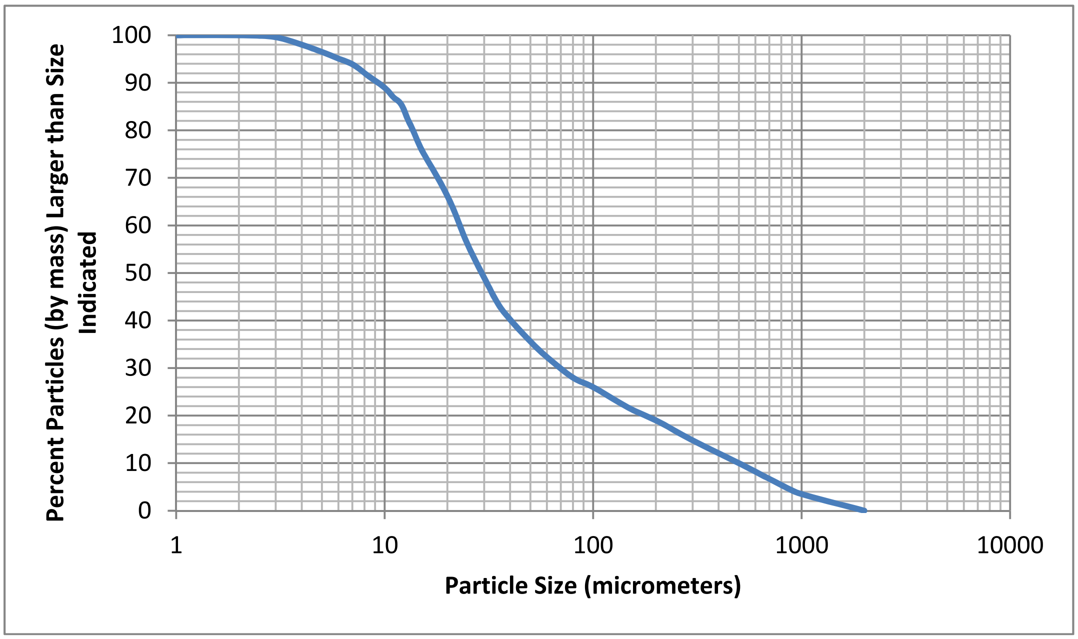


Figure 68. Particle size distributions of water entering the monitored biofilters.

Table 44. Accumulative mass percentage (%) (summary for 20 influent samples)

Particle size (µm)	Average	Min	Max	St dev	COV	Median
< 0.45	0.00	0.00	0.00	0.00	n/a	0.00
< 3	0.41	0.05	1.64	0.39	0.94	0.27
< 12	14.53	2.70	32.36	9.28	0.64	12.02
< 30	50.96	18.03	77.46	15.19	0.30	51.36
< 60	67.68	25.31	86.95	16.03	0.24	71.16
< 120	75.58	36.40	91.71	14.37	0.19	77.66
< 250	78.59	44.45	94.44	13.39	0.17	79.95
< 1,180	100.00	100.00	100.00	0.00	0.00	100.00

The TSS samples were analyzed using both stir plates/pipetting and shake and pour methods; the SCC was determined by subsampling using a cone splitter. The stir plate and pipette method has been shown to have the highest yield and most consistent results compared to the SCC value, as also shown by prior studies (Clark and Siu 2008; Clark and Pitt 2008; Clark et al. 2008). The shake and pour method had reduced values compared to the pipette and SSC methods. The pipette and SSC methods appear similar.

The average shake and pour TSS concentration of the 59 influent samples was 133 mg/L, the stir plate-pipette average TSS concentration was 160 mg/L, while the SSC average concentration was 158 mg/L. Simple paired t-tests identified significant differences between the TSS values obtained when the shake and pour results were compared to the stir plate – pipette results ($p < 0.001$) and the SSC results ($p = 0.0002$), while no significant differences were detected for the stir plate – pipette vs. SSC results ($p = 0.40$) for the number of samples available.

Figures 69 and 70 are scatterplots comparing the stir plate and pipetting TSS results with the SSC results, along with the two TSS methods as analyzed in the UA Environmental Engineering Lab. The stir plate and pipetting TSS values are consistently very close to the SSC values, with an overall bias of less than 10%. The relationship between the shake and pour TSS and stir plate and pipette TSS values are consistent, but with about a 22% bias, with the shake and pour results being less.

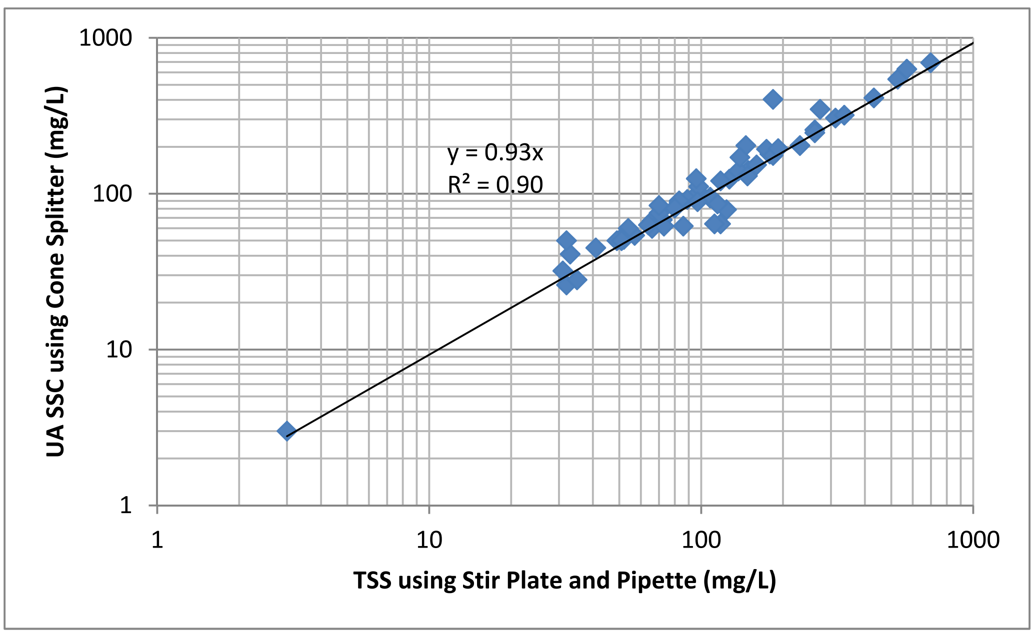


Figure 69. TSS by stirring pipetting versus SSC with cone splitter.

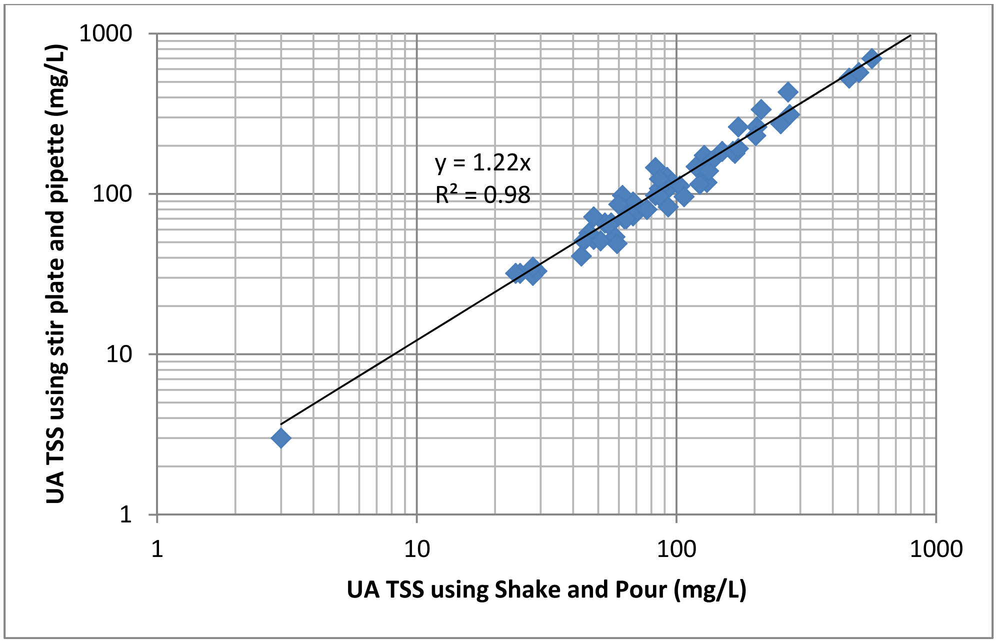


Figure 70. TSS by shaking and pouring versus TSS by stirring and pipetting.

SSC Biofilter Inlet Concentrations for Monitored Locations

As noted, 59 biofilter inlet samples were available for SSC, TSS, and particle size analyses at the University of Alabama environmental engineering laboratories. These samples were collected from five of the eight monitored locations. Three of the locations did not have sufficient sample volumes collected for these analyses. Table 45 lists the influent SSC concentrations and summary statistics for these five locations for which data are available. Figure 71 is a box and whisker plot comparing the monitored SSC inlet concentrations observed at each of these five locations. Table 46 summarizes the results of a non-parametric Kruskal-Wallis one-way ANOVA rank test indicating that no significance differences in SSC concentrations at each site could be detected for the number of data observations available (5 to 14). As indicated on Table 45, the average values for the sites ranged from 129 to 205 mg/L, with an overall average of 174 mg/L. The overall observed SSC concentrations ranged from 26 to 711 mg/L, resulting in a COV value of about 1.0 (typical for stormwater observations).

Table 45. Kansas City Curb-Cut Biofilter Particulate Inlet Concentrations

1222 E 76th Street		
sample #	Date and Time	SSC (mg/L)
8	26-07-2012	102
13	31-08-2012	26
14	31-08-2012	92
30	11-11-2012	28
40	4/17/13 11:01 to 4/18/13 12:01	140
45	4/23/13 03:00 to 4/23/13 08:54	64
31	4/7/13 19:49 to 4/7/13 21:22	630
38	4/9/13 22:00 to 4/10/13 10:00	193
52	5/2/13 02:00 to 5/2/13 22:00	63
55	5/27/13 08:24 to 5/27/13 10:22	412
62	5/30/13 01:18 to 5/30/13 11:32	204

63	5/30/13 01:39 to 5/30/13 02:39	75
67	6/5/13 10:02 to 6/5/13 11:26	171
70	6/9/13 00:25 to 6/9/13 03:05	86
	count	14
	average	163
	min	26
	max	630
	st dev	167
	COV	1.02
1324 E 76th Street		
sample #	Date and Time	SSC (mg/L)
6	21-06-2012	543
9	26-07-2012	180
23	14-10-2012	60
29	11-11-2012	51
42	4/17/13 10:29 to 4/17/13 21:19	64
46	4/23/13 00:40 to 4/23/13 05:53	90
33	4/7/13 19:47 to 4/7/13 20:54	711
36	4/9/13 21:43 to 4/10/13 03:42	306
49	5/2/13 02:00 to 5/2/13 04:52	153
56	5/27/13 08:25 to 5/27/13 09:44	176
60	5/29/13 22:57 to 5/30/13 02:19	50
64	6/4/13 10:57 to 6/5/13 11:47	79
	count	12
	average	205
	min	50
	max	711
	st dev	214
	COV	1.04
1325 E 76th Street		
sample #	Date and Time	SSC (mg/L)
5	21-06-2012	693
7	26-07-2012	88
12	31-08-2012	67
22	17-10-2012	41
28	11-11-2012	32
41	4/17/13 10:22 to 4/18/13 01:02	319
44	4/22/13 16:28 to 4/23/13 06:00	62
39	4/9/13 21:37 to 4/10/13 04:39	403
50	5/2/13 01:58 to 5/2/13 16:32	246
57	5/27/13 08:21 to 5/27/13 09:49	130
65	6/4/13 10:55 to 6/5/13 12:36	125
	count	11
	average	201
	min	32
	max	693
	st dev	203
	COV	1.01
1419 E 76th Terr		

sample #	Date and Time	SSC (mg/L)
4	21-06-2012	124
10	26-07-2012	121
24	17-10-2012	111
26	11-11-2012	257
43	4/17/13 11:02 to 4/18/13 12:18	54
	count	5
	average	133
	min	54
	max	257
	st dev	75
	COV	0.56
1425 E 76th Terr		
sample #	Date and Time	SSC (mg/L)
47	4/23/13 00:37 to 4/23/13 14:38	60
32	4/7/13 19:49 to 4/7/13 22:06	348
35	4/9/13 21:58 to 4/10/13 04:49	141
51	5/2/13 02:00 to 5/2/13 16:44	62
58	5/27/13 08:22 to 5/27/13 10:08	195
61	5/29/13 22:56 to 5/30/13 11:26	50
66	6/4/13 10:59 to 6/5/13 11:34	45
	count	7
	average	129
	min	45
	max	348
	st dev	112
	COV	0.87
all sites combined		SSC (mg/L)
	count	49
	average	174
	median	111
	min	26
	max	711
	stdev	172
	COV	0.99

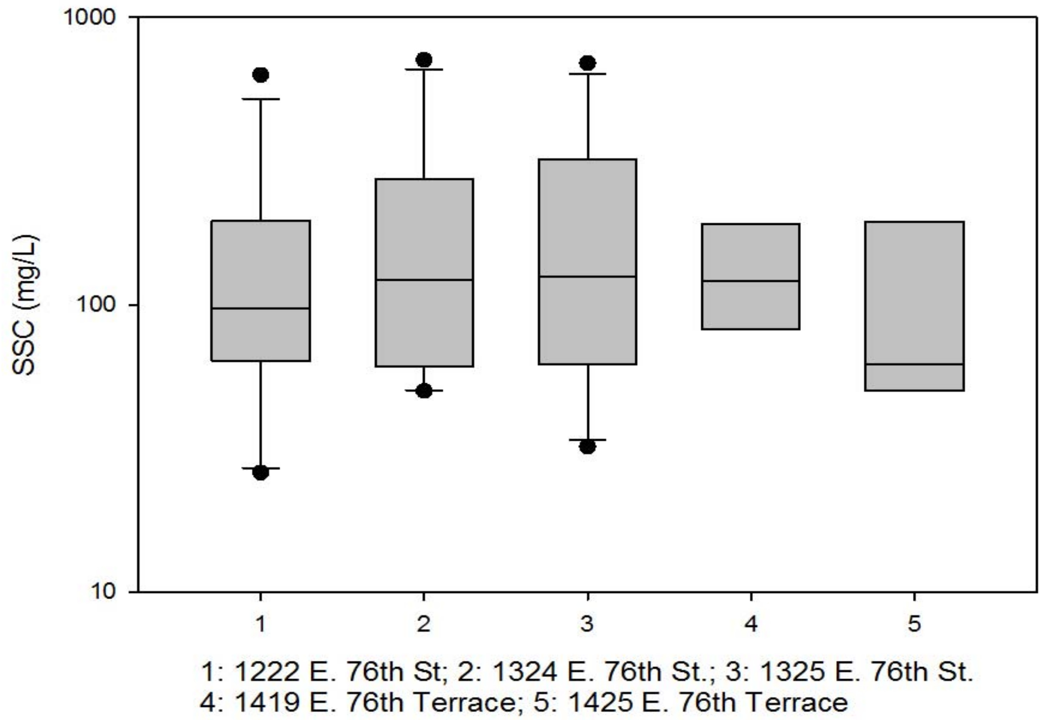


Figure 71. SSC concentrations at biofilter inlet monitoring locations.

Table 46. Kruskal-Wallis One Way Analysis of Variance on Ranks for Biofilter Inlet SSC Concentrations

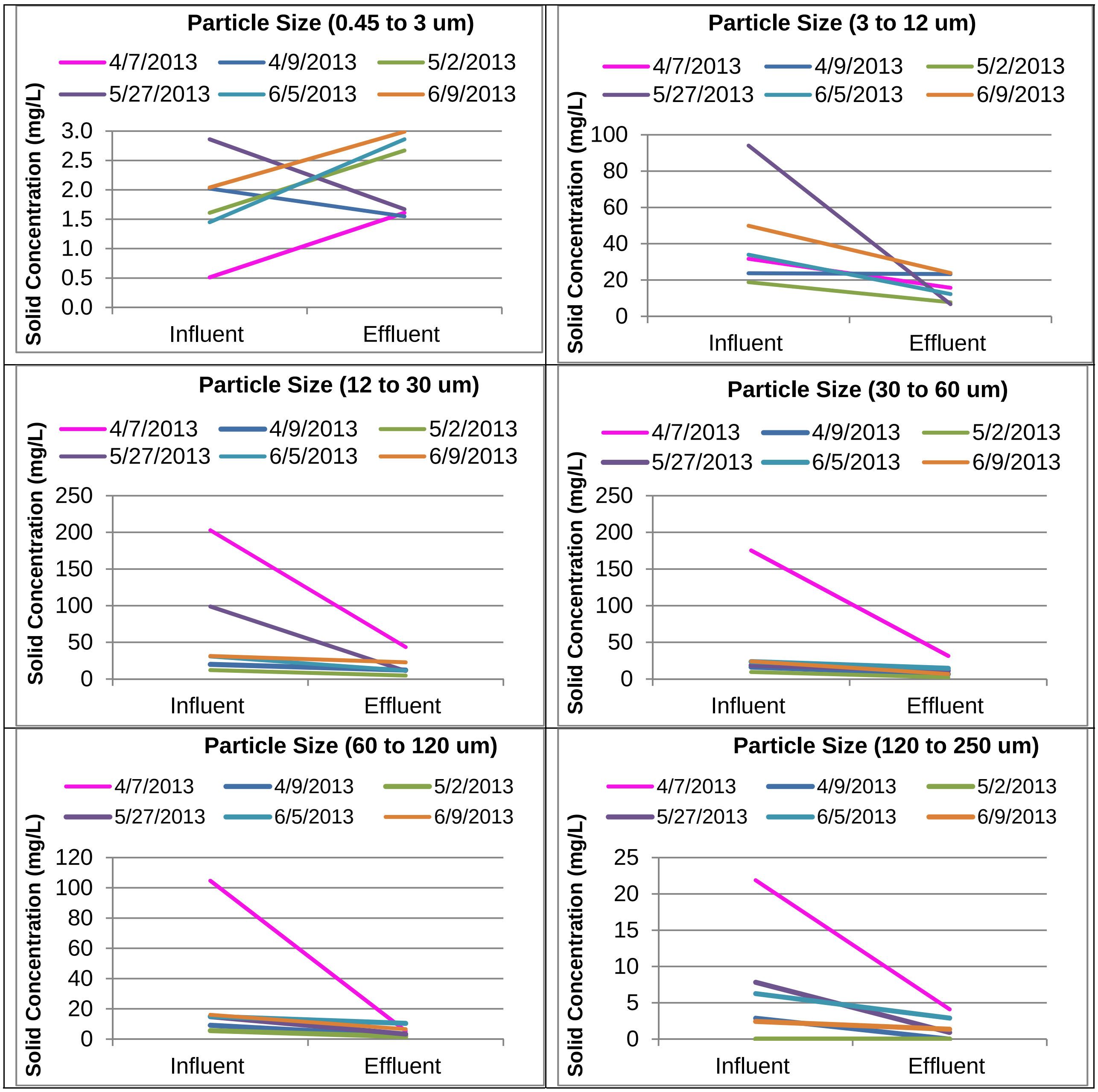
Group	N	Median	25%	75%
1222	14	97	64	196
1324	12	122	61	275
1325	11	125	62	319
1419	5	121	83	191
1425	7	62	50	195

H = 0.827 with 4 degrees of freedom (P = 0.935)

The differences in the median values among the treatment groups are not great enough to exclude the possibility that the difference is due to random sampling variability; there is not a statistically significant difference (P = 0.935).

Particulate Capture in Curb-Side Biofilters

The overall influent SSC concentrations for the 49 influent samples averaged 173 mg/L (26 to 711 mg/L range). Paired t-test statistical tests indicated that the differences in the influent and underdrain SSC concentrations for the 6 events that had underdrain samples (259 vs. 77 mg/L average) were significant (p = 0.048), while the differences in the influent and overflow SSC concentrations for the 4 events that had overflows (126 vs. 89 mg/L) were not statistically significant (p = 0.33) for the few paired samples available. Therefore, complete pollutant removal from the surface flows occurred for more than 80% of the events (49 out of 59), with the biofilters trapping most of the particulates. Figure 72 and Table 47 illustrate the particulate removal performance of the curb-side biofilters using the data from the six events that had both influent and effluent samples. The effluent samples were collected from the SmartDrains after the stormwater passed through the biofilter media. Except for the smallest particle size (0.45 to 3 µm), the effluent concentrations were significantly smaller than the influent sample concentrations (using the nonparametric sign test). It is expected that the small particle size range was adversely affected by washing of fines from the biofilter media as the stormwater passed through the media. Table 47 shows that the concentrations of these very small particles were very small compared to the other particle sizes and therefore did not have an important adverse effect on the overall performance. The average particulate removals ranged from about 50 to 75%, with the SSC overall removal averaging 65%. The underdrain (effluent) concentrations averaged 66 mg/L.



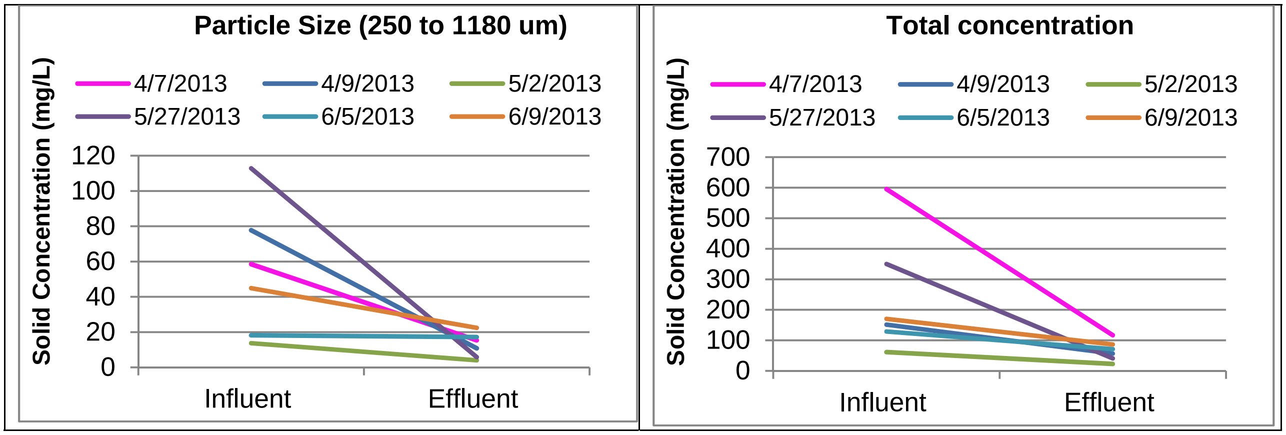


Figure 72. Influent and Underdrain Particulate Concentrations by Particle Size

Table 47. Average Influent and Underdrain Particulate Concentrations and Removals

Size Range	Influent	Underdrain	% Reduction
0.45 to 3 µm	1.7	2.2	-60
3 to 12 µm	42.0	14.9	53
12 to 30 µm	65.9	17.5	60
30 to 60 µm	18.4	8.2	57
60 to 120 µm	27.5	4.9	68
120 to 250 µm	6.9	1.5	74
250 to 1800 µm	54.3	12.6	64
>1180 µm	0.0	0.0	n/a
total	242.8	66.0	65

Summary of Biofilter Measurements during Rain Events

A tremendous amount of information was collected during this project, ranging from drainage area characteristics to runoff and flow monitoring data at different scales, locations, and project phases. The infiltration rates in the biofilters were monitored during actual rains by measuring the rate of drop of the ponded water during large rains. Statistical analyses identified two distinct groups of these data, as shown in the following list and group box and whisker plot (Figure 73).

- Low rates: average 0.70 in/hr; range 0.19 to 1.9
- Normal rates: average 2.7 in/hr; range 0.10 to 7.2

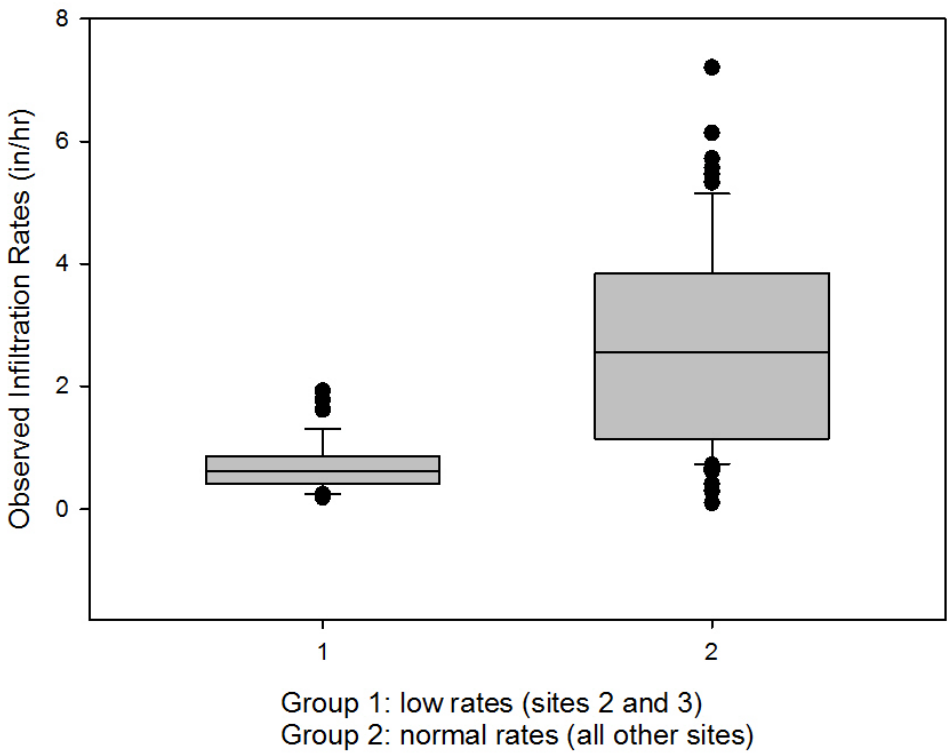


Figure 73. Measured infiltration rates in biofilters during actual rains.

The average time to ponding for each of the eight curb-side biofilters after the rain started ranged from 0.15 to 0.5 hr, with the fast group starting ponding in about 0.16 hrs and the slow group starting in about 0.3 hrs. The maximum depth of ponding was also separated into two categories, as shown below:

- Shallow: average of 3.4 in., range of 0.72 to 12 in.
- Deep: average of 6.3, range of 0.60 to 13 in

Figure 74 is a group box and whisker plot showing these two combined sets of data for maximum depth of ponding.

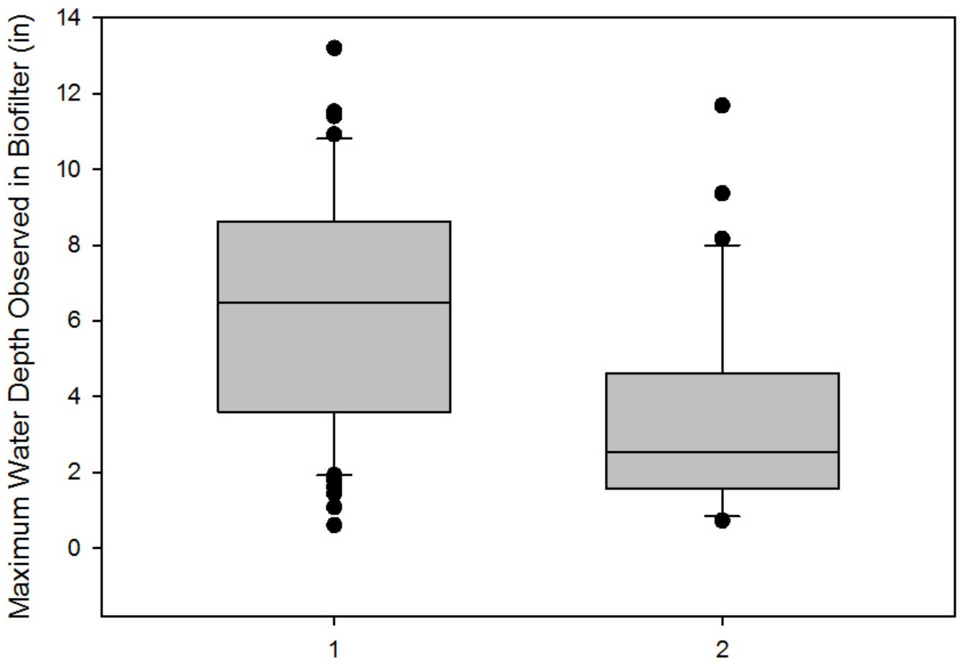


Figure 74. Maximum ponding depth observed in biofilters during actual rains.

Laboratory column tests were conducted to investigate the biofilter media used at the Kansas City sites. Columns were constructed to measure the infiltration rates as a function of compaction (and therefore density). The density of the media column with hand compaction was 1.00 g/cm^3; the density of the standard proctor media column was 1.13 g/cm^3, and the density for the modified proctor media column was 1.12 g/cm^3. The soil media has a median particle size (D_{50}) of about 1.9 mm and a very high uniformity coefficient (C_u) of 39. The porosity of the media for the hand compaction columns was 0.36, 0.15 for the standard proctor compaction columns, and 0.25 for the modified proctor compaction columns.

Infiltration data for different test trials were fitted to the Horton equation by using multiple nonlinear regressions to estimate f_c (the saturated soil infiltration rate), f_o (the initial rate), and k (the rate coefficient), using the observed data. The saturated rates were of greatest interest as they would apply during most of the operation during events. The estimated infiltration rates of the saturated media ranged from 0.4 to 0.8 in/hr for the hand compaction tests (initial rates were about 0.75 to 3 in/hr), 0.4 to 0.9 in/hr for the standard proctor compaction tests, and 0.03 to 0.33 in/hr for the modified proctor compaction tests. Only the modified compaction level significantly affected the infiltration rates. More than 90% of the media is larger than 100 µm, with appreciable fractions clearly in the coarse sand category, resulting in a relatively robust media with minimal compaction potential. Media with large amounts of sand do not compact as much as media having more fines because of the structural support of the sand grains. Figure 75 contains example plots of the laboratory infiltration measurements fitted to the Horton equation for the hand compaction (least dense) tests.

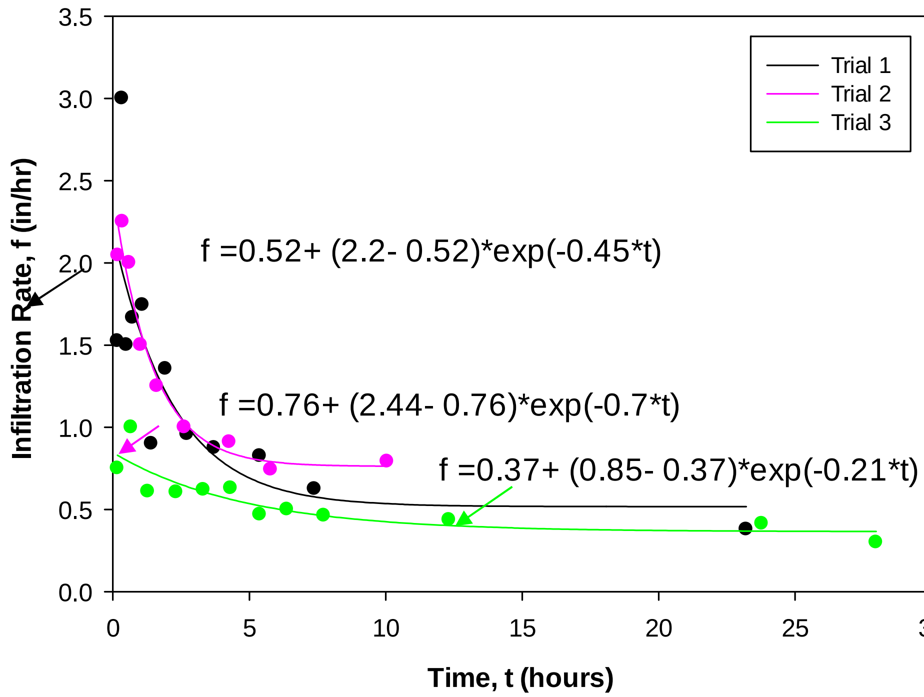


Figure 75. Kansas City biofilter media infiltration rates during column tests for hand compacted density.

Samples were also collected of inflowing water entering the biofilters for analyses. Figure 76 is a PSD plot showing a median particle size (by mass) of about 30 µm, with about 25% larger than 100 µm. The observed median size is typical for stormwater gutter/inlet samples but is larger than would be expected at a stormwater outfall (the larger particles are subjected to deposition in the drainage system).

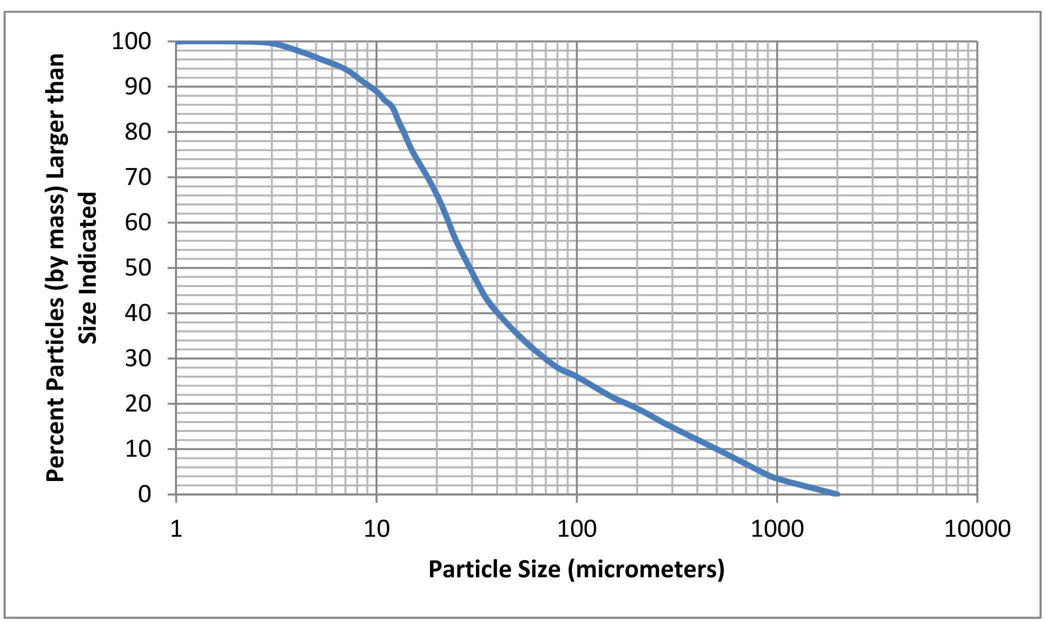


Figure 76. Particle size distribution for curb-cut influent stormwater samples.

The stir plate and pipette TSS method has been shown to have the highest yield and most consistent results compared to the SCC values as standards. The shake and pour method shows reduced values compared to the pipette and SSC methods. The relationship between the stir plate and pipette TSS values and the SSC values are consistent with <10% bias, as shown in Figure 77.

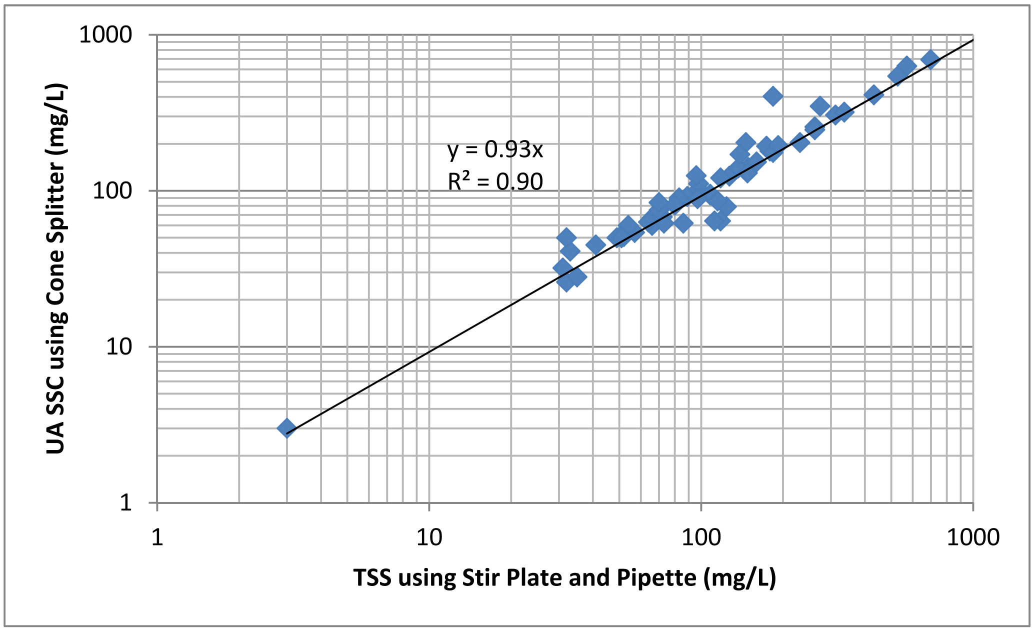


Figure 77. TSS by stir plate and pipetting vs. SSC.

The average SSC concentrations for the sites ranged from 129 to 205 mg/L, with an overall average of 174 mg/L. The overall observed SSC concentrations ranged from 26 to 711 mg/L, resulting in a COV value of about 1.0 (typical for stormwater observations). There were no significant differences in the SSC concentrations between the different locations based on the Kruskal-Wallis one way analysis of variance on ranks statistical test for the number of observations available.

6. Evaluation of Performance of Stormwater Control Practices

Characteristics of Areas Treated and Not Treated by Stormwater Controls

One of the important steps in urban stormwater quality modeling is to quantify the drainage area characteristics. The Kansas City GI demonstration site is unique because a very large portion of the test (pilot) area receives direct treatment from many separate stormwater control devices. However, as in all retrofit installations, stormwater controls could not be placed to treat the complete watershed area. Hindrances to installations of stormwater controls in established urban areas are mature trees that need to be protected, right-of-way restrictions and utility interferences, and other attributes such as the presence of driveways. The micro drainages resulting from original site grading at the time of initial construction seldom allows efficient installations of retrofitted controls compared to stormwater controls installed at the time of new construction. Figure 78 is a map showing the test (pilot) watershed with all major source area components.

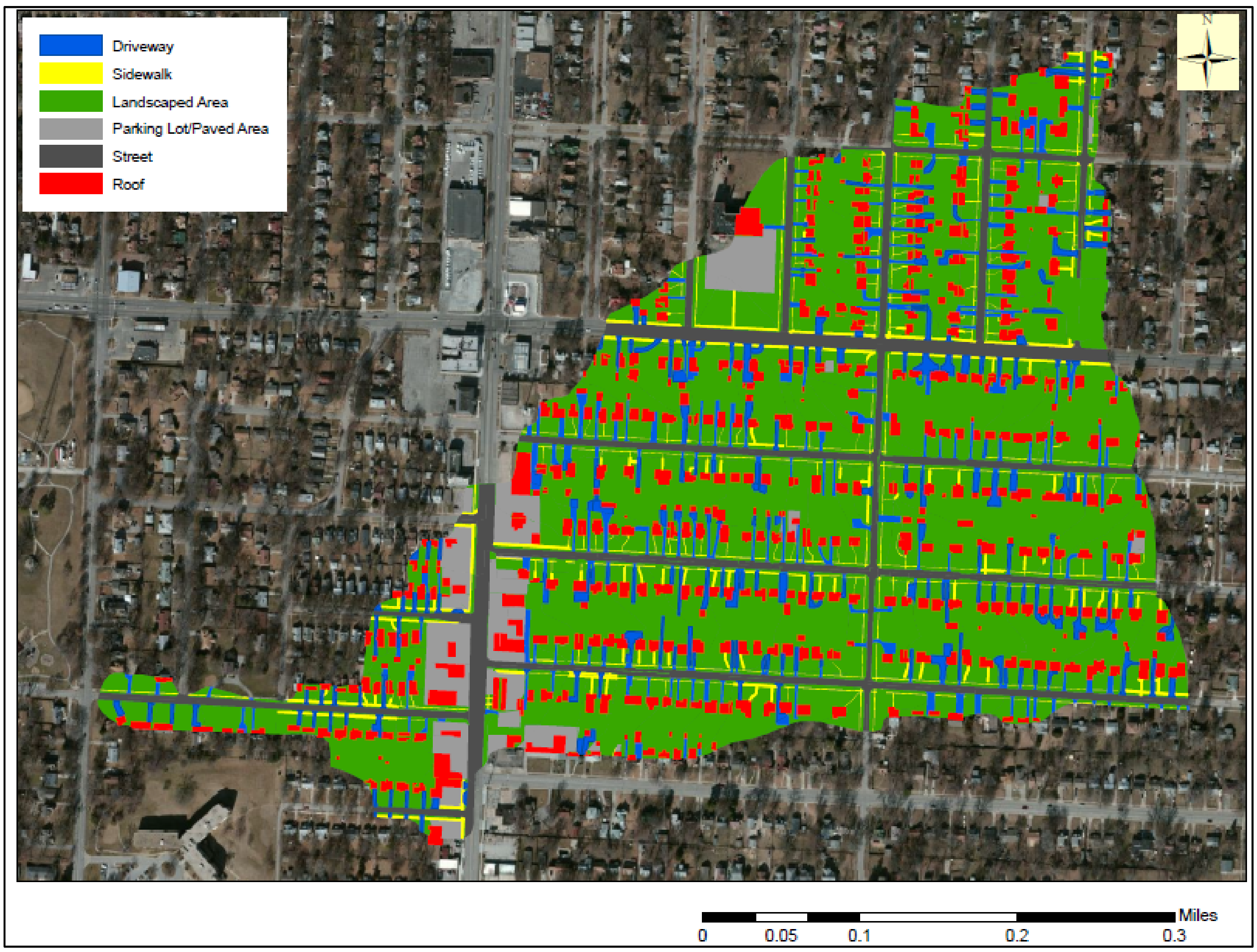


Figure 78. Map of test (pilot) area showing main surface characteristics.

Figure 79 is a similar map, but with only the details for the areas having stormwater control shown. The blanked-out areas drain into the combined sewer without any control. Some of the treated area's runoff flows some distance along the curbs and gutters before it enters the stormwater control practices. In addition, other areas are treated by multiple control units, as previously shown, with overflows from upgradient devices flowing into downgradient controls. This figure includes both the direct and the indirectly treated areas, with the untreated areas flowing directly into the combined sewers without any treatment indicated as blanked out.

Figure 79. Map of test (pilot) area showing surface characteristics of areas receiving stormwater treatment.

Figure 80 is a map showing the surface characteristics of the areas not being treated by any of the stormwater control devices before their runoff enters the combined sewer.

Figure 80. Map of test (pilot) area showing surface characteristics of areas not receiving stormwater treatment.

Figure 81 shows the areas associated with the subdrainage areas for each green infrastructure facility in the test/pilot watershed.

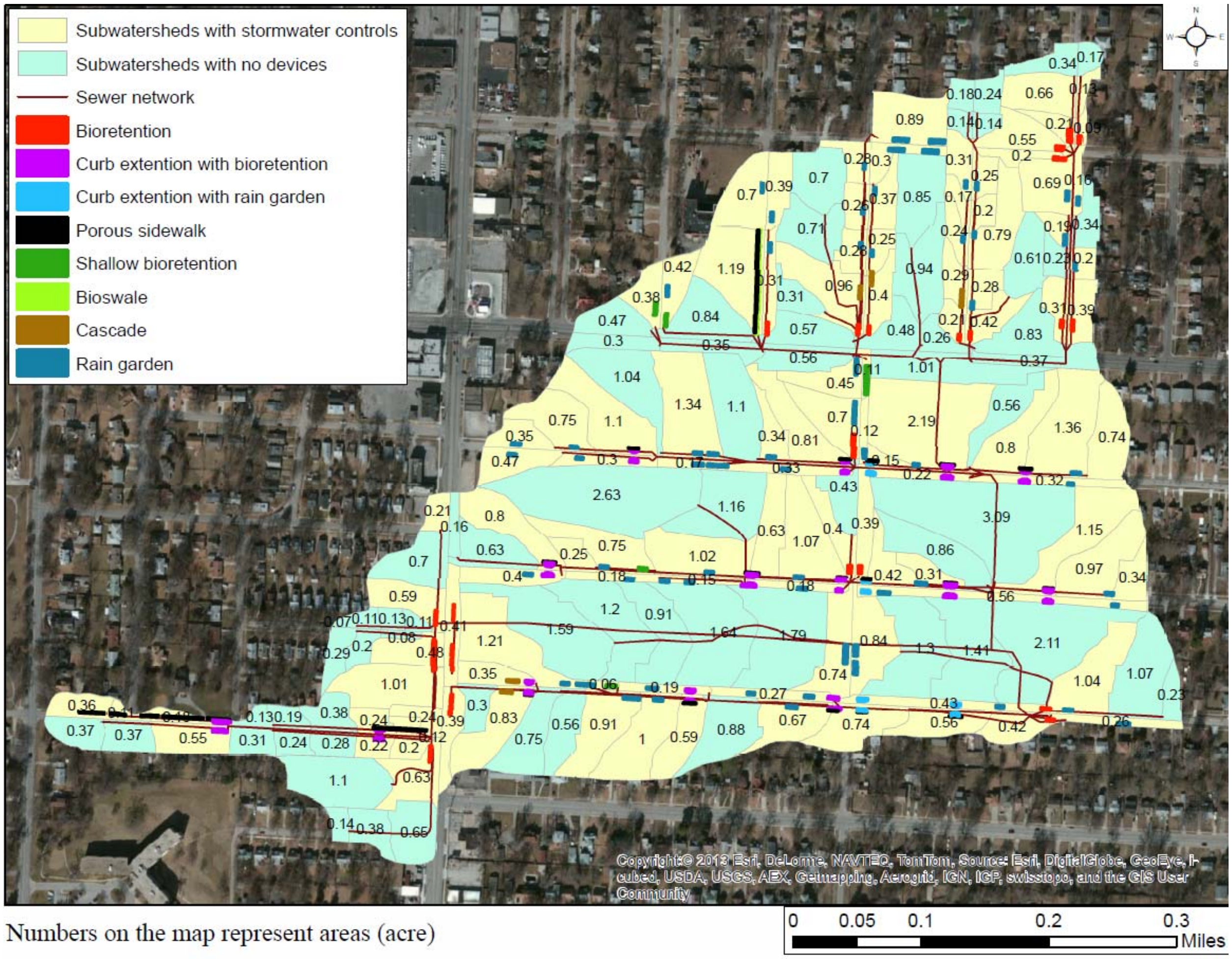


Figure 81. Map of test/pilot watershed showing subdrainage areas for each GI facility and areas not being treated.

Table 48 summarizes the source areas for each of the controlled and uncontrolled subareas in the test (pilot) watershed. About 45% of the complete watershed does not receive any control and drains directly into the combined sewer, and about 55% of the area is treated. Therefore, the absolute upper limit of control is about 55%, assuming both subareas have identical source area makeups. However, the following table and associated maps indicate that the areas being treated are generally closer to the streets (including sidewalks, most of the driveways, and many of the roofs). The untreated areas have a greater portion of landscaped areas that drain through yard drains directly into the combined sewer system.

Table 48. Site characteristics for areas receiving stormwater treatment and other areas

Land component	Areas in subwatersheds with no stormwater controls		Areas in subwatersheds with stormwater controls		Total area (ac)
	Area (acres)	Percentage	Area (acres)	Percentage	
roofs - directly connected	1.11	2.40%	1.05	1.9%	2.16
roofs - drain to landscaped	6.29	13.7%	5.95	10.9%	12.24
driveway - directly connected	2.00	4.40%	2.30	4.2%	4.30
driveways - drain to perv	2.00	4.40%	2.30	4.2%	4.30
sidewalk - directly connected	0.38	0.80%	0.97	1.8%	1.35
sidewalks - to perv	0.45	1.00%	1.13	2.1%	1.58
Parking lot/ Paved area - directly connected	1.40	3.1%	3.40	6.3%	4.80
Streets - directly connected	3.50	7.6%	7.30	13.4%	10.80
Landscaped area - pervious area	28.70	62.6%	30.00	55.1%	58.70
Total area	45.83	100.0%	54.40	100.0%	100.23

Table 49 summarizes the impervious areas that are directly connected or that flow to pervious areas, or are the pervious areas (landscaped areas). The breakdown of the directly and indirectly connected impervious areas was estimated based on the full area land use monitoring. The total impervious area for the area being treated is about 45%, while the total impervious area for the untreated area is about 37%. The calculations and modeling in the following section determine the maximum amount of control possible, and shows the sensitivity of the native soil conditions on biofilter performance.

Table 49. Impervious and pervious areas in subareas receiving stormwater treatment and other areas

Land component	Areas in subwatersheds with no stormwater control		Areas in subwatersheds with stormwater controls	
	Area (acres)	Percent of subarea	Area (acres)	Percent of subarea
Impervious, directly connected	8.09	17.7%	15.02	27.6%
Impervious, draining to pervious areas	9.04	19.7%	9.38	17.2%
Pervious areas	28.70	62.6%	30.00	55.2%
Total area:	45.83	100.0%	54.40	100.0%

Designs and Service Areas for Stormwater Controls in the Test (Pilot) Area

Before the modeling of the area, it was necessary to determine the different types (and number) of each type of stormwater control, and their design attributes, along with the drainage area characteristics for each type of control practice.

Figure 82 shows the layout for the 100-acre pilot study area with the locations of all of the types of stormwater controls. There are 158 individual surface features, along with 21 supplemental underground storage pipe systems. A list of the different surface and subsurface structural components are summarized in Table 50. The schematic drawings of stormwater controls are also cross-referenced in Table 50 for each of the unique design plan component categories. Table 51 summarizes typical sizes for each type of stormwater control, based on reviewing several examples from the 100% design drawings.

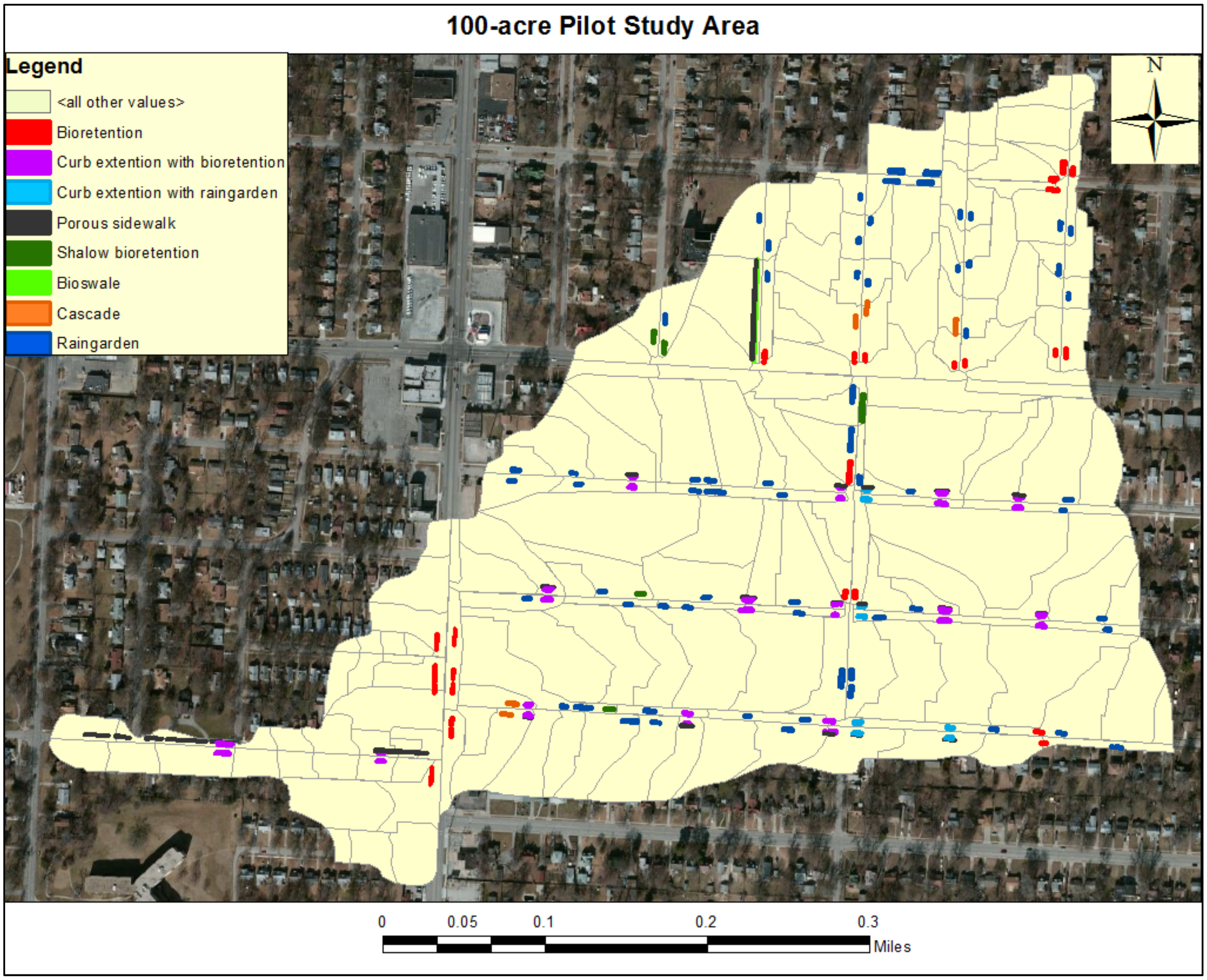


Figure 82. Stormwater controls in the 100-acre test (pilot) study area (source: Tetra Tech).

Table 50. Summary of stormwater control design plan components

Design plan component	Structural description	Number of this type of stormwater control	Figure reference*
Bioretention	Bioretention without curb extension	24	Figure 83
	Curb extensions with bioretention	28	
	Shallow bioretention	5	
Bioswale	Vegetated swale infiltrates to background soil	1	Figure 84
Cascade	Terraced bioretention cells in series	5	Figures 85 and 86
Porous sidewalk or pavement	With underdrain	18	Figure 87
	With underground storage cubes	5	
Rain garden	Rain garden without curb extension	64	Figure 88
	Curb extensions with rain gardens	8	
Below grade storage	Retains stormwater control overflow and underdrain outflow from selected bioretention cells or porous pavement	21	Figure 89

Source: SUSTAIN report, 2011

* Source: 100% design plans and near-street topographic info.

Table 51. Typical sizes of different types of stormwater controls used in the test (pilot) area

Stormwater control type	Examples	Top area (ft^2)	Bottom area (ft^2)	Ponding depth	Total depth to bottom of device	Material
Cascade	1	423.41	105.58	8"–12"	> 16"–20"	Topsoil planting mix on side slopes, engineered soil mix 8-in. min depth on bottom.
	2	316.96	106.73			
	3	290.73	48.16			
	4	283.1	74.12			
Bioswale	1	1,948.86		12"	> 20"	Native soil amended with 3-in. compost, rototilled 8-in. min
Porous Sidewalk	1	1,640.42		Figure 87	Figure 87	Figure 87
	2	650.1				
	3	277.62				
	4	362.86				
	5	544.15				
	6	391.02				
Bioretention	1	194.21	34.12	12"	> 20"	3-in. hardwood mulch on top, topsoil planting mix on side slopes, engineered soil mix 8-in. min depth on bottom.
	2	240.6	28.77			
	3	301.37	31.85			
	4	337.5	55.28			
	5	335.89	53.5			
Curb extension with bioretention	1	383.03	98	12"	24"	Engineering soil mix
	2	169.35	56.32			
	3	238.68	85.24			
Curb extension with rain garden	1	237.01	123.96	12"	24"	Engineering soil mix
	2	265.43	115.98			
	3	279.54	112.9			
	4	275.87	97.63			
Rain garden	1	468.93	247.07	6"	> 17"	3-in. hardwood mulch on top, native soil amended with 3-in. compose, rototilled 8-in. min depth
	2	743.55	463			
	3	514.74	219.77			
	4	282.43	71.3			
	5	422.9	240			

Figures 83 through 89 are example construction drawings from the 100% design plans representing the various stormwater control designs constructed in the test (pilot) area, referenced in Table 50.

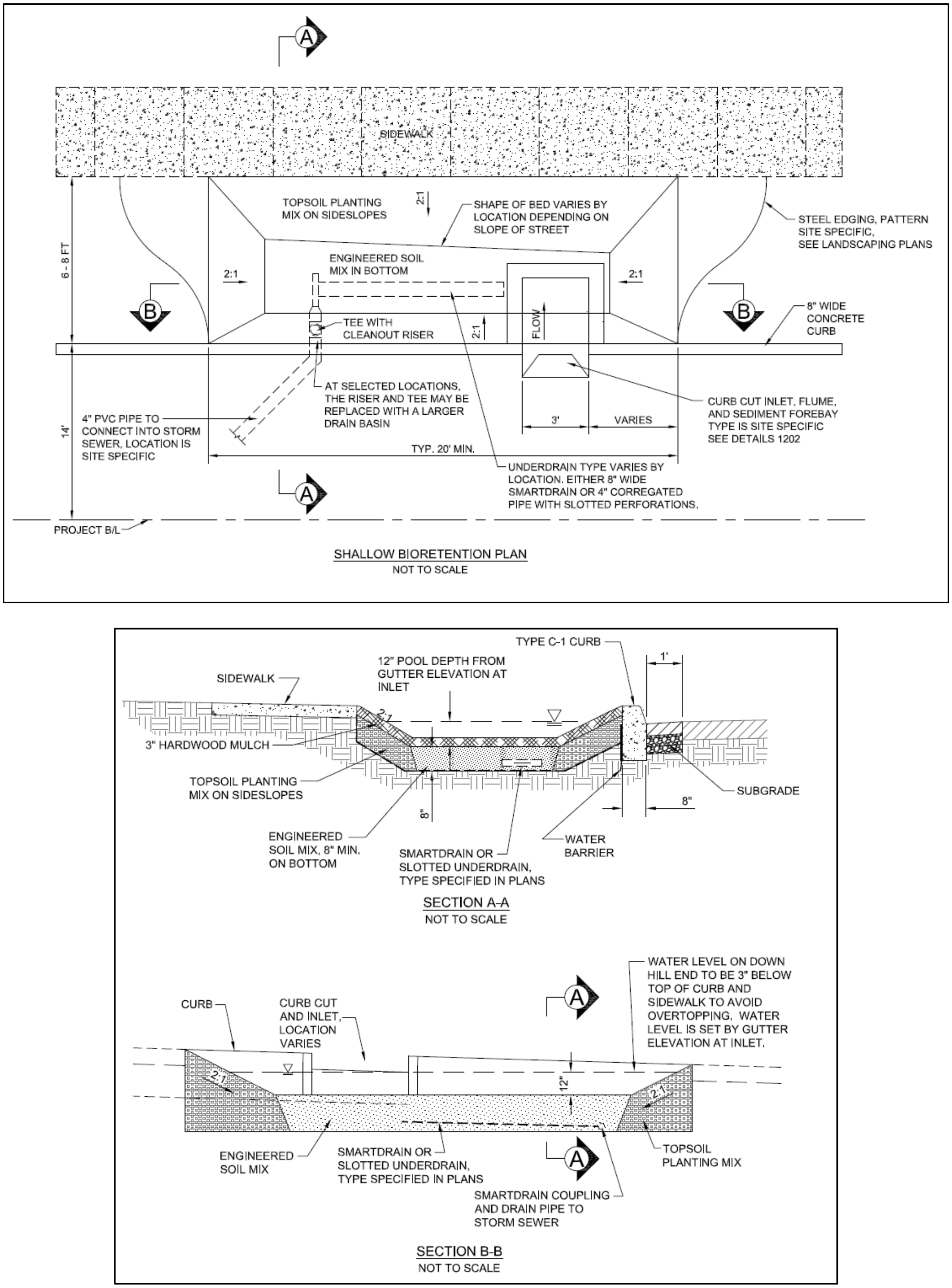


Figure 83. Shallow bioretention device typical details for residential streets.

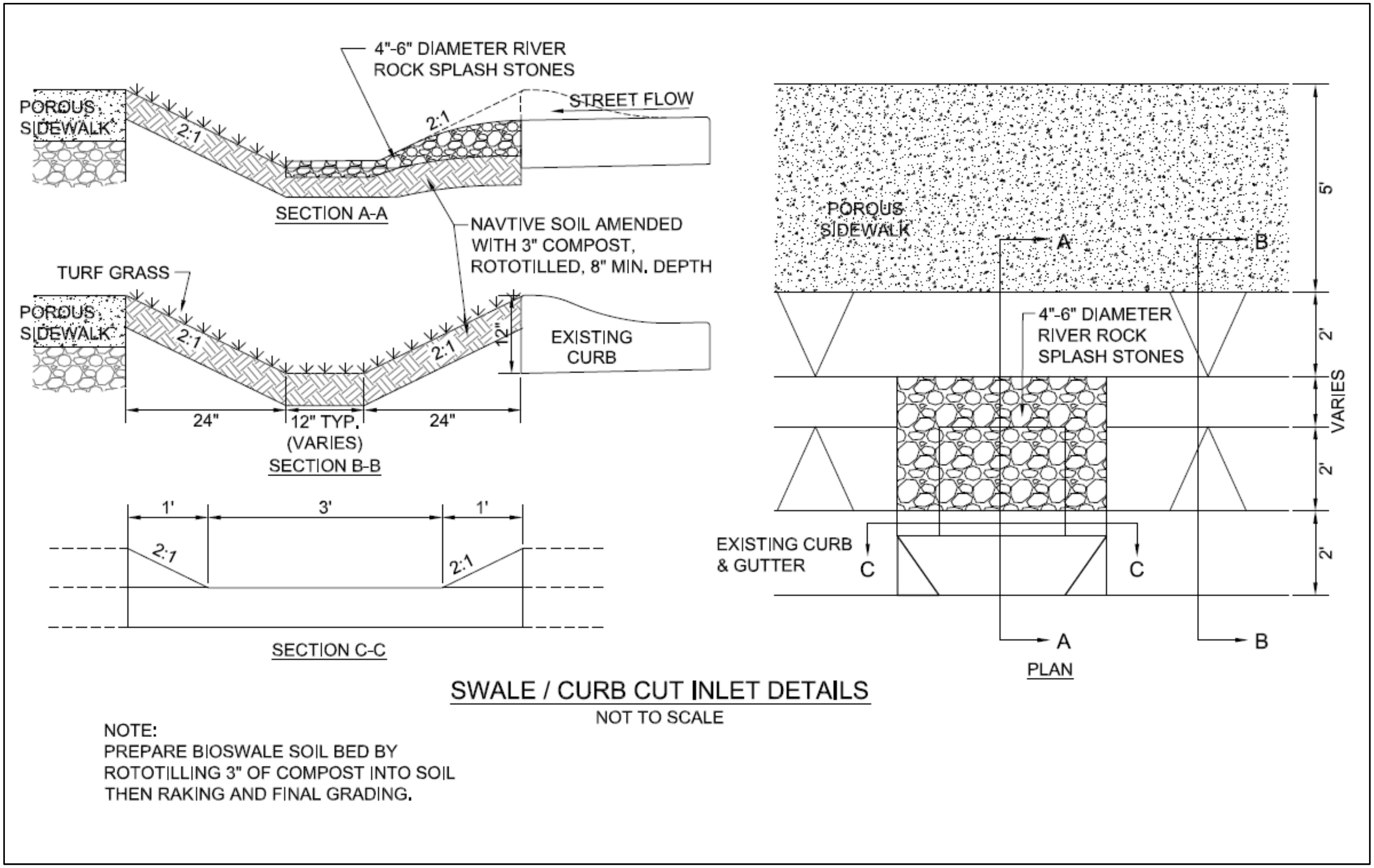


Figure 84. Bioswale typical details for residential streets.

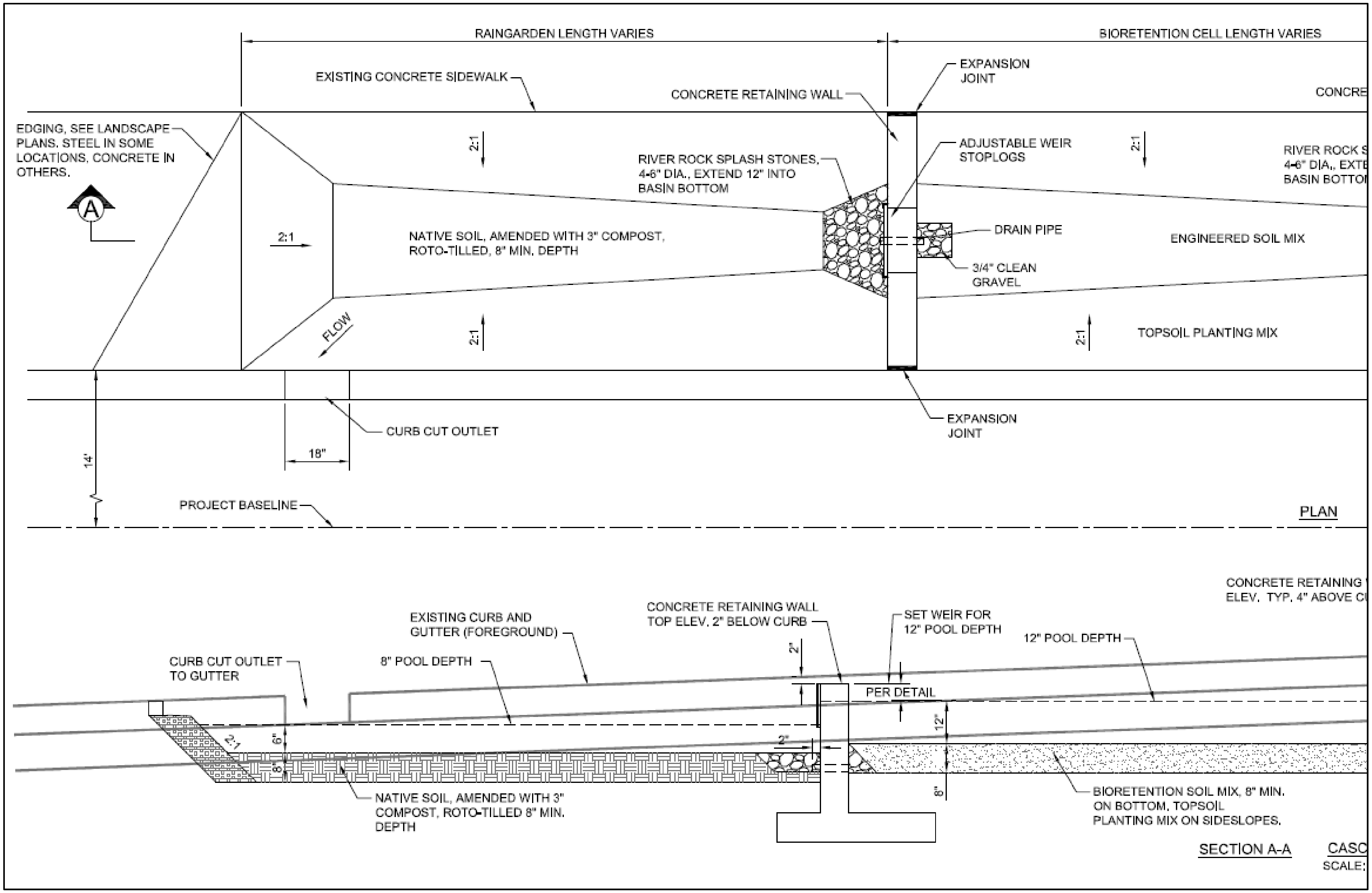


Figure 85. Cascade rain garden typical details for residential streets.

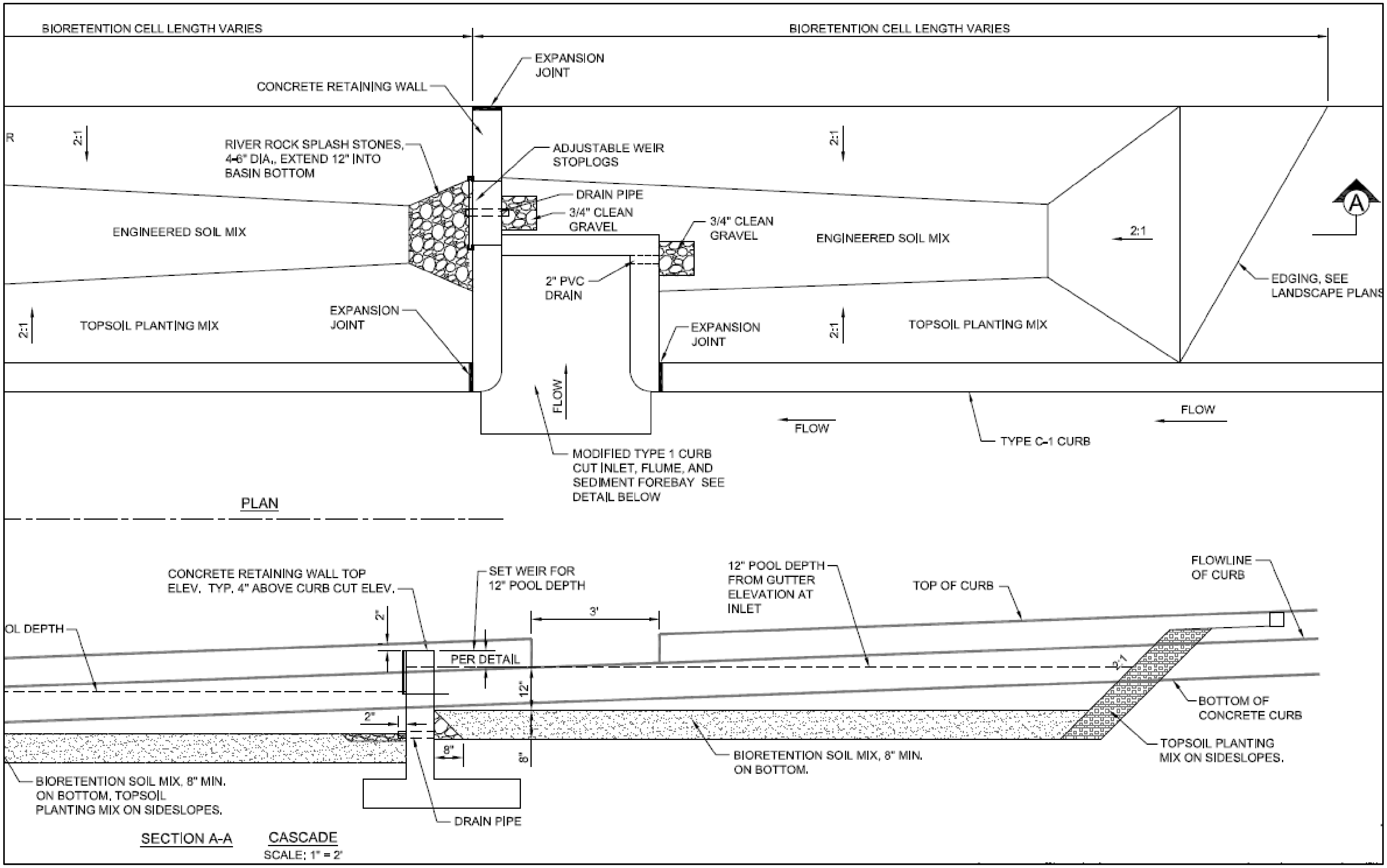


Figure 86. Cascade rain garden typical details for residential streets (continued).

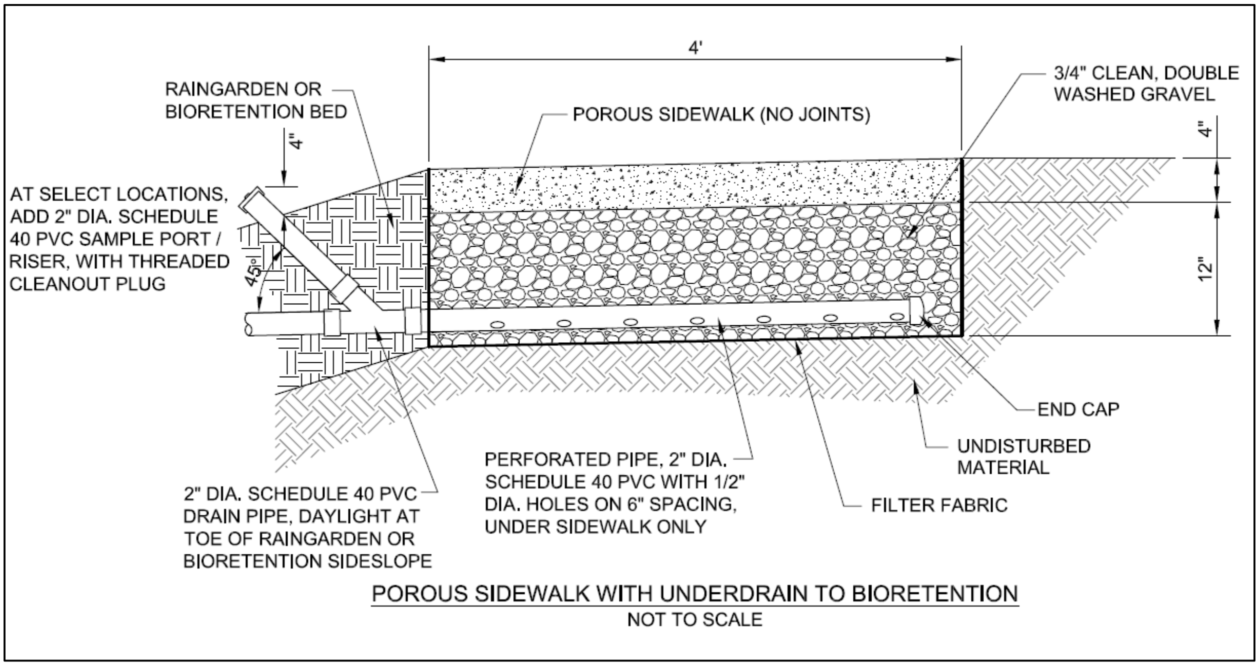


Figure 87. Porous sidewalk typical details for residential streets.

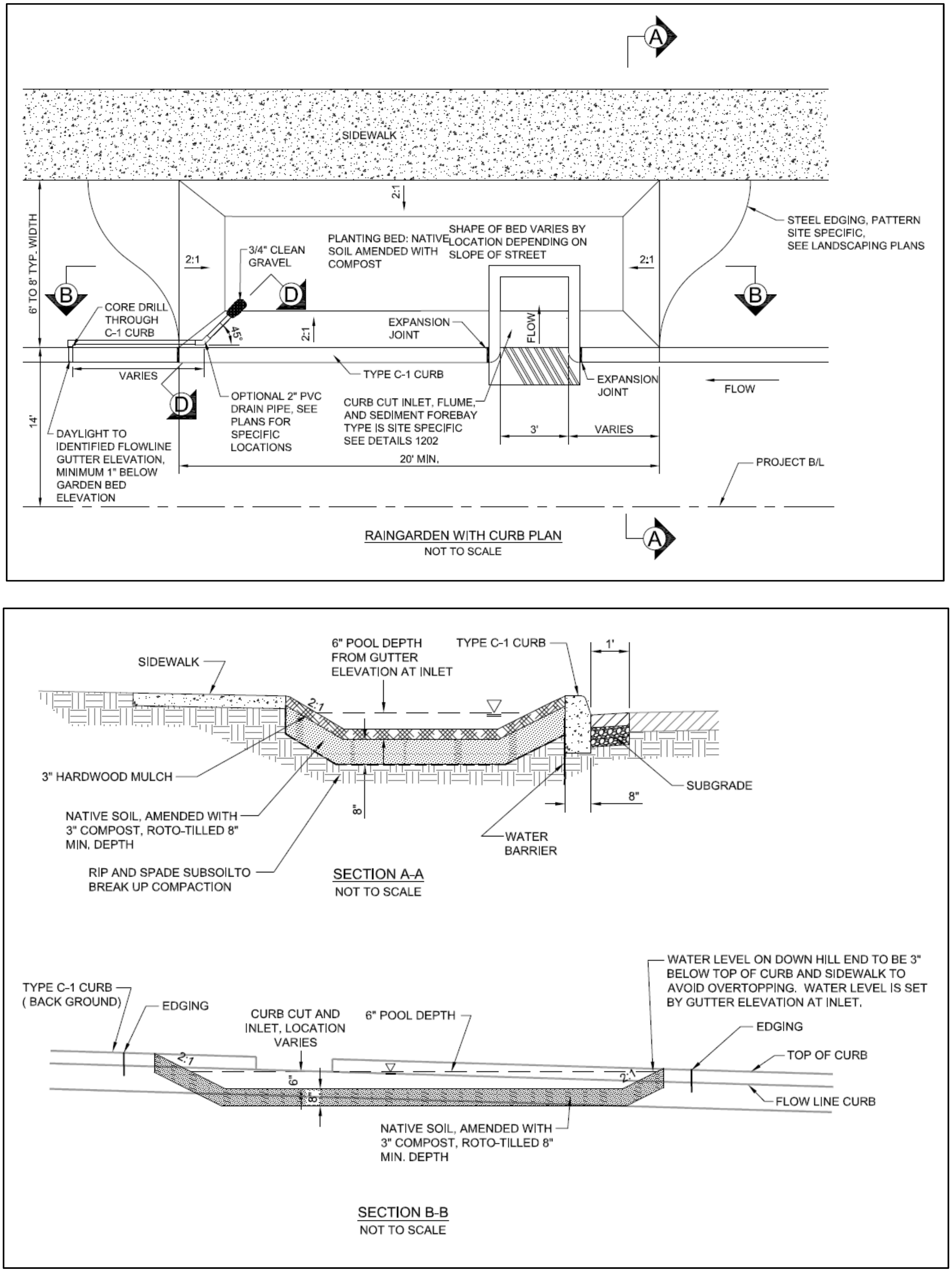


Figure 88. Rain garden typical details for residential streets.

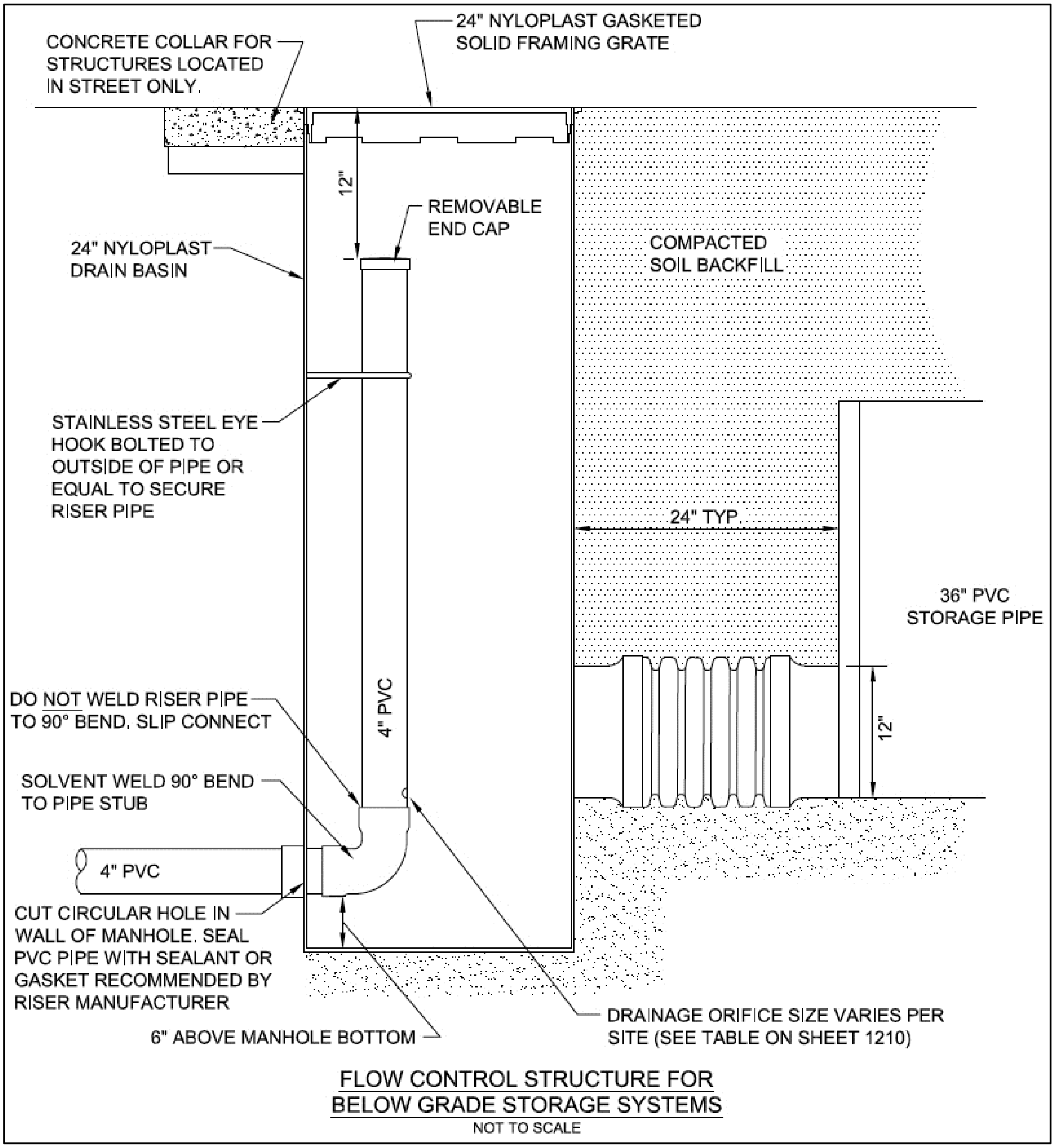


Figure 89. Below grade storage system typical details for residential streets.

Modeling of Test (Pilot) Watershed Area with Stormwater Controls Compared to Observed Flows

Table 52 lists the initial monitored events in 2012 that occurred after the majority of the site construction was completed, including the observed and calculated runoff for the complete area. The model found the native soil infiltration rate was about 1 in/hr below the biofilters which resulted in the best model predictions. Lower infiltration rates, such as used during the design phase, significantly decreased the calculated discharges, resulting in poor fits of the monitored data.

Table 52. Events after construction of stormwater controls in pilot watershed

Rain start date	Rain start time	Rain end date	Rain end time	Total rain (in)	Observed total pipe flow discharge volume (ft^3)	Modeled with controls (1 in/hr)
4/4/2012	8:45:00 PM	4/5/2012	9:10:00 AM	0.18	1,818	3,204
4/12/2012	3:20:00 PM	4/13/2012	4:15:00 AM	0.12	2,546	2,034
4/27/2012	8:40:00 PM	4/28/2012	8:40:00 AM	0.12	1,249	2,034
4/28/2012	10:45:00 PM	4/30/2012	7:50:00 AM	0.75	20,505	21,820
5/1/2012	1:40:00 AM	5/1/2012	10:30:00 PM	0.43	6,626	10,260
5/6/2012	10:05:00 AM	5/7/2012	8:55:00 PM	1.85	34,962	95,046
5/24/2012	8:35:00 PM	5/25/2012	8:10:00 PM	0.40	43,119	9,283
6/11/2012	2:50:00 AM	6/11/2012	7:35:00 PM	1.22	15,514	44,473
6/21/2012	1:20:00 AM	6/21/2012	9:00:00 PM	0.91	30,410	27,777

Figure 90 compares the predicted with the observed total runoff volumes for the complete test (pilot) watershed for the first nine events after biofilter construction.

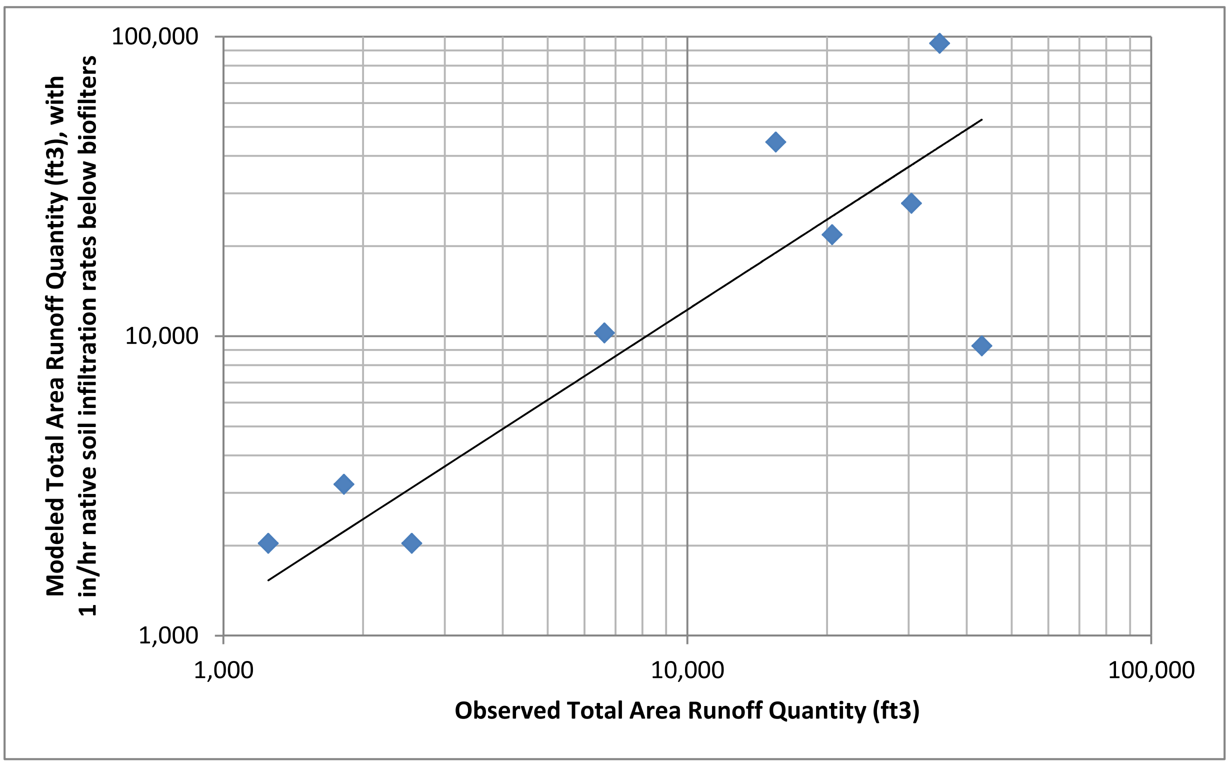
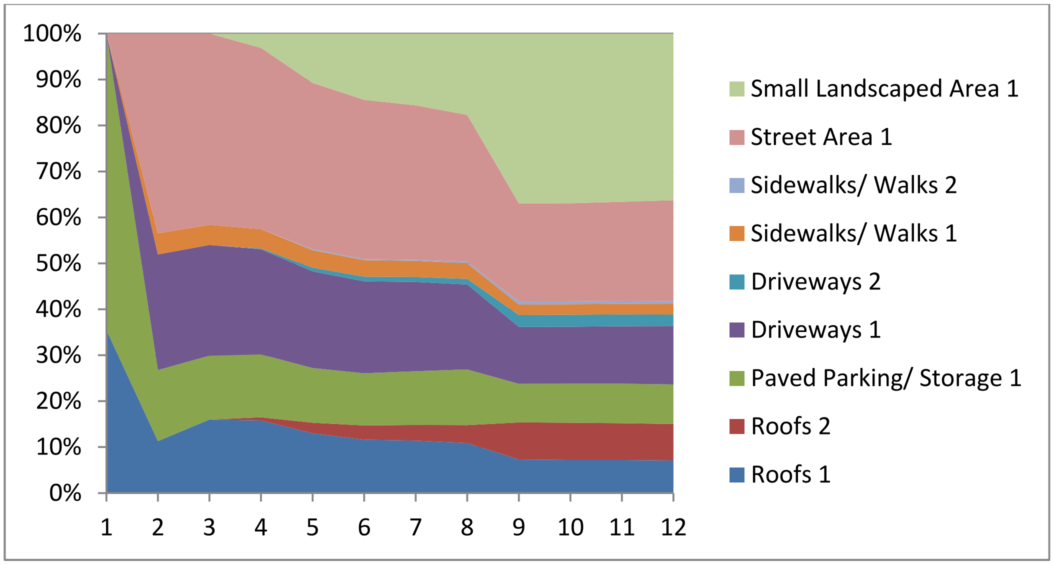


Figure 90. Observed and calculated flows after biofilter construction.

ANOVA analysis of the regression equation indicated a significant equation (p = 0.014) and a significant slope term (p = 0.012). The slope coefficient is 1.22, with a 95% confidence range of 0.36 to 2.1. The additional monitoring at the large scale enables more precise fits of the data and confirms the expected performance of the stormwater controls.

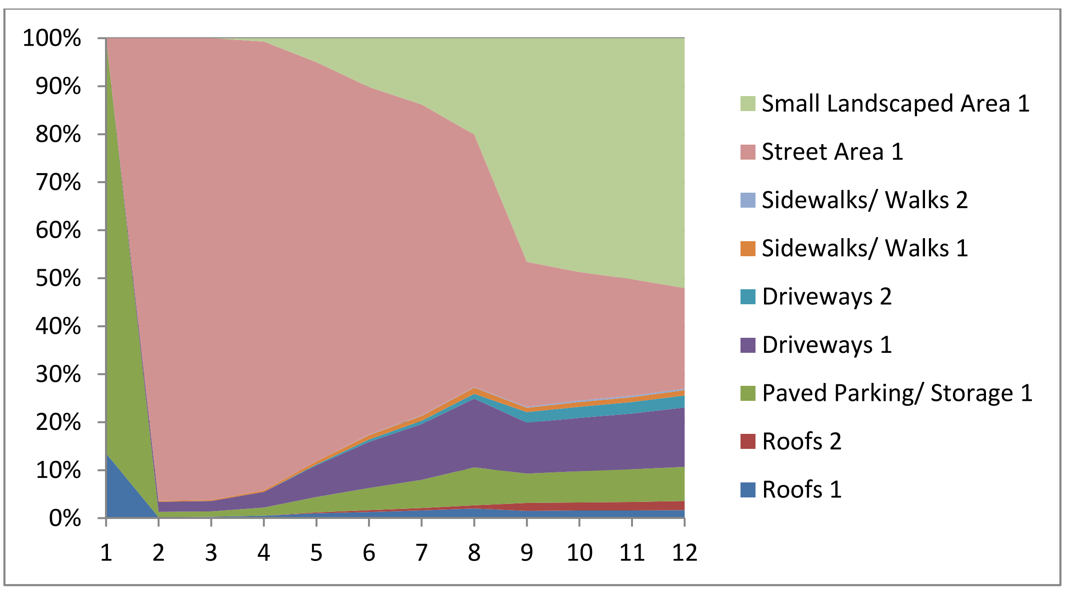
Sources of Flows and Particulates in Untreated Watershed

Before a stormwater management plan is selected for an area, knowing the sources of the flows and pollutants of concern is very helpful. One of the main features of WinSLAMM is its ability to calculate these source contributions for varying rain conditions. The plots shown in Figures 91 and 92 illustrate these source contributions for the test (pilot) area without (before) stormwater controls, for rains ranging from 0.01 to 4 in.



Event number on graphs	Rain depth (in)
1	0.01
2	0.05
3	0.10
4	0.25
5	0.50
6	0.75
7	1.00
8	1.50
9	2.00
10	2.50
11	3.00

Figure 91. Sources of runoff volume during different rain events (no control practices).



Event number on graphs	Rain depth (in)
1	0.01
2	0.05
3	0.10
4	0.25
5	0.50
6	0.75
7	1.00
8	1.50
9	2.00
10	2.50
11	3.00
12	4.00

Figure 92. Sources of particulate solids during different rain events (no control practices).

Table 53 summarizes the major flow and particulate flows for 0.5, 1.0, and 3.0 in rains. As expected, the directly connected impervious areas are responsible for most of these contributions, but landscaped areas become important flow and particulate solids contributions for the largest rains expected in Kansas City.

Table 53. Major source areas contributing runoff and particulate solids

Rain depth (in)	Runoff volume	Particulate solids
0.5	Street areas (36%) Driveways, directly connected (21%) Paved parking areas, directly connected (12%) Small landscaped areas (11%)	Street areas (83%)
1.0	Street areas (32%) Driveways, directly connected (19%) Small landscaped areas (18%) Paved parking areas, directly connected (12%)	Street areas (53%) Small landscaped areas (20%) Driveways, directly connected (14%)
3.0	Small landscaped areas (37%) Street areas (22%) Driveways, directly connected (13%)	Small landscaped areas (50%) Street areas (24%) Driveways, directly connected (12%)

Use of Stormwater Controls in Test (Pilot) Area

Table 54 summarizes the characteristics for each category of stormwater control used in the test (pilot) area, including the number of each device and the expected areas being treated by each unit. The device areas as a percentage of drainage area are also shown, and range from about 1.5% for the biofilters to 9% for the bioswale.

Table 54. Sizes and drainage area characteristics of subareas treated by stormwater controls

Design plan component	Structural description	Number of this type of stormwater control units in test (pilot) area	Drainage area to device area ratio	Device as a % of the drainage area	Drainage area for each unit (ac)	Total area treated by each device type (ac)
Bioretention	Bioretention without curb extension	24	61.8	1.6	0.40	9.6
	Curb extensions with bioretention	28	66.1	1.5	0.40	11.2
	Shallow bioretention	5	61.8	1.6	0.40	2.0
Bioswale	Vegetated swale infiltrates to background soil	1	11.2	8.9	0.50	0.5
Cascade	Terraced bioretention cells in series	5	53.0	1.9	0.40	2.0
Porous sidewalk or pavement	With underdrain	18	1.0	100.0	0.015	0.3
	With underground storage cubes	5	1.0	99.9	0.015	0.1
Rain garden	Rain garden without curb extension	64	35.8	2.8	0.40	25.6
	Curb extensions with rain gardens	8	66.0	1.5	0.40	3.2
	total number of control units (w/o porous pvt)	135			total area treated	54.4
	total area treated (acres)	54.4				
	area per unit	0.40				

Tables 55 through 62 summarize the sizes and other design characteristics for each of these categories of stormwater controls that were used in modeling the total system. Tables are also shown indicating the surface areas being treated by each stormwater device. The percentage components for each category are the same as the entire area average.

Table 55. Modeling characteristics for some of the bioretention areas

	Bioretention subareas		Top area (ft^2)	Bottom area (ft^2)	Total depth (ft)	Typical width (ft)	Native soil infilt rate (in/hr)
res1	Bioretention	Bioretention without curb extension	282	41	5	10	1.0
res2	Bioretention	Curb extensions with bioretention	264	80	5	10	1.0
res3	Bioretention	Shallow bioretention	282	41	2	10	1.0
res4	Cascade	Terraced bioretention cells in series	329	84	2	10	1.0
res5	Rain garden	Rain garden without curb extension	487	248	3.5	10	1.0
res6	Rain garden	Curb extensions with rain gardens	264	113	3.5	10	1.0

Table 55. Modeling characteristics for some of the bioretention areas (cont.)

	Bioretention subareas		Rate fraction for sides	Rock filled depth (ft)	Rock filled porosity	Satur. water content (porosity) %	Field capacity, %	Permanent wilting point, %	Media infilt rate (in/hr)
res1	Bioretention	Bioretention without curb extension	1	2.5	0.4	43.4	21.8	4.6	1.8
res2	Bioretention	Curb extensions with bioretention	1	2.5	0.4	43.4	21.8	4.6	1.8
res3	Bioretention	Shallow bioretention	1	0	n/a	43.4	21.8	4.6	1.8
res4	Cascade	Terraced bioretention cells in series	1	0	n/a	43.4	21.8	4.6	1.8
res5	Rain garden	Rain garden without curb extension	1	1	0.4	43.4	21.8	4.6	1.8
res6	Rain garden	Curb extensions with rain gardens	1	1	0.4	43.4	21.8	4.6	1.8

Table 55. Modeling characteristics for some of the bioretention areas (cont.)

	Bioretention subareas		Eng. media depth (ft)	Inflow hydrograph peak to avg flow rate	Number of devices in source area of this type	Weir crest length (ft)	Weir crest width (ft)	Height from datum to bottom of weir opening (ft)	Prairie plants coverage	Annuals coverage
res1	Bioretention	Bioretention without curb extension	1.5	3.8	24	8	1	4.75	0.75	0.25
res2	Bioretention	Curb extensions with bioretention	1.5	3.8	28	8	1	4.75	0.75	0.25
res3	Bioretention	Shallow bioretention	1	3.8	5	8	1	1.75	0.75	0.25
res4	Cascade	Terraced bioretention cells in series	1	3.8	5	8	1	1.75	0.75	0.25
res5	Rain garden	Rain garden without curb extension	1.5	3.8	64	8	1	3.25	0.75	0.25
res6	Rain garden	Curb extensions with rain gardens	1.5	3.8	8	8	1	3.25	0.75	0.25

Table 56. ET rates for Kansas City biofiltration devices (in/day)

Bioretention subareas			Jan	Feb	Mar	Apr	May	Jun	Jly	Aug	Sept	Oct	Nov	Dec
res1	Bioretention	Bioretention without curb extension	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05
res2	Bioretention	Curb extensions with bioretention	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05
res3	Bioretention	Shallow bioretention	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05
res4	Cascade	Terraced bioretention cells in series	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05
res5	Rain garden	Rain garden without curb extension	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05
res6	Rain garden	Curb extensions with rain gardens	0.05	0.1	0.1	0.15	0.2	0.2	0.25	0.25	0.2	0.1	0.05	0.05

Table 57. Modeling characteristics for some of the porous pavement areas

Porous Pavement subareas			Porous pvt area (acres)	Inflow hydro peak/avg ratio	Pavement thickness (in)	Pavement porosity	Aggreg bedding thickness (in)	Aggreg bedding porosity	Aggreg base reser thickness (in)	Aggreg base porosity
res7	Porous sidewalk or pavement	With underdrain	0.015	3.8	3	0.4	3	0.4	12	0.45
res8	Porous sidewalk or pavement	With underground storage cubes	0.015	3.8	3	0.4	3	0.4	36	0.95

Table 57. Modeling characteristics for some of the porous pavement areas (cont.)

Porous Pavement subareas			Perforated underdrain D (in)	Underdrain invert elev (in)	Number of underdrains	Subgrade seepage rate (in/hr)	Por pvt initial infilt rate (in/hr)	% after 3 yrs	% after 5 yrs	Total clogging (yrs)	% restored with cleaning	Cleaning frequency
res7	Porous sidewalk or pavement	With underdrain	3	8	1	1	40	80	50	10	75%	1/yr
res8	Porous sidewalk or pavement	With underground storage cubes	n/a	n/a	n/a	1	40	80	50	10	75%	1/yr

Table 58. Modeling characteristics for some swale drained areas

Swale subareas			Fraction of area served by swales	Swale density (ft/ac)	Bottom width (ft)	Swale side slope H/1V	Long slope V/1H	Retardance factor	Grass height (in)	Dynamic infilt rate (in/hr)	Swale depth (ft)
res9	Bioswale	Vegetated swale infiltrates to background soil	100%	MDR land use value	3	3	0.02	D	4	0.5	3

Table 59. Drainage areas to different types of bioretention areas

Bioretention subareas			Roofs1 (directly connected)	Roofs2 (to pervious areas)	Pvdpark1 (directly connected)	Drvy1 (directly connected)	Drvy2 (to pervious areas)	Sidwlks1 (directly connected)	Sidwlks (to pervious areas)	Streets1	Small landscp	Total Area (acres)
res1	Bioretention	Bioretention without curb extension	0.182	1.046	0.605	0.403	0.403	0.173	0.202	1.286	5.299	9.6
res2	Bioretention	Curb extensions with bioretention	0.213	1.221	0.706	0.470	0.470	0.202	0.235	1.501	6.182	11.2
res3	Bioretention	Shallow bioretention	0.038	0.218	0.126	0.084	0.084	0.036	0.042	0.268	1.104	2.0
res4	Cascade	Terraced bioretention cells in series	0.038	0.218	0.126	0.084	0.084	0.036	0.042	0.268	1.104	2.0
res5	Rain garden	Rain garden without curb extension	0.486	2.790	1.613	1.075	1.075	0.461	0.538	3.430	14.131	25.6
res6	Rain garden	Curb extensions with rain gardens	0.061	0.349	0.202	0.134	0.134	0.058	0.067	0.429	1.766	3.2

Table 60. Drainage areas to porous pavements

Porous Pavement subareas			Roofs1 (directly connected)	Roofs2 (to pervious areas)	Pvdpark1 (directly connected)	Drvy1 (directly connected)	Drvy2 (to pervious areas)	Sidwlks1 (directly connected)	Sidwlks (to pervious areas)	Streets1	Small landscp	Total Area (acres)
res7	Porous sidewalk or pavement	With underdrain	0.006	0.033	0.019	0.013	0.013	0.005	0.006	0.040	0.166	0.3
res8	Porous sidewalk or pavement	With underground storage cubes	0.002	0.011	0.006	0.004	0.004	0.002	0.002	0.013	0.055	0.1

Table 61. Drainage areas to swales

	Swale subarea		Roofs1 (directly connected)	Roofs2 (to pervious areas)	Pvdpark1 (directly connected)	Drvy1 (directly connected)	Drvy2 (to pervious areas)	Sidwlks1 (directly connected)	Sidwlks (to pervious areas)	Streets1	Small landscp	Total Area (acres)
res9	Bioswale	Vegetated swale infiltrates to background soil	0.010	0.055	0.032	0.021	0.021	0.009	0.011	0.067	0.276	0.5

Table 62. Drainage areas not treated by stormwater controls

		Roofs1 (directly connected)	Roofs2 (to pervious areas)	Pvdpark1 (directly connected)	Drvy1 (directly connected)	Drvy2 (to pervious areas)	Sidwlks1 (directly connected)	Sidwlks (to pervious areas)	Streets1	Small landscp	Total Area (acres)
res10	no controls	1.099	6.275	1.420	2.015	2.015	0.366	0.458	3.481	28.671	45.8

Tables 63 and 64 summarize the calculated runoff conditions entering the stormwater controls, along with the expected removals for each type of device. The runoff volume reductions range from 86 to 100% for a 4-year continuous simulation period (the same period and events included in the monitoring period). The predicted maximum water depths in the biofilters range from about 2 to 5 in, similar to the water depths observed. The maximum ponding times for the biofilters range from about 60 to 90 hours. Only a single event in the 4 years of simulation had a holding time longer than 3 days, the typical criterion for mosquito control. Only about one-third of the events might have any surface or underdrain discharges, and these amounts would be small compared to the treated volumes.

Table 63. Calculated stormwater control performance

Control practice no.	Control practice type	Control practice name or location	Total inflow volume (ft^3)	Total outflow volume (ft^3)	Percent volume reduction	Total influent load (lbs)	Total effluent load (lbs)	Percent load reduction	Flow weighted influent conc (mg/l)	Flow weighted effluent conc (mg/L)	Percent conc. reduction
1	Biofilter	DS Biofilters # 1	1,234,000	259,759	79%	16,138	2,248	86%	210	138.6	34%
2	Biofilter	DS Biofilters # 2	1,440,000	229,535	84%	18,844	1,943	90%	210	135.6	35%
3	Biofilter	DS Biofilters # 3	257,173	56,361	78%	3,358	488	85%	209	138.7	34%
4	Biofilter	DS Biofilters # 4	257,173	36,807	86%	3,493	314	91%	218	136.8	37%
5	Biofilter	DS Biofilters # 5	3,292,000	72,824	98%	43,059	602	99%	210	132.5	37%
6	Biofilter	DS Biofilters # 6	411,738	51,201	88%	5,384	429	92%	210	134.1	36%
7	Grass Swales	DS Grass Swales # 1	64,704	12,950	80%	845	74	91%	209	91.12	56%
8	Porous Pavement	SA Device, LU# 7 ,SA# 31	2,635	0	100%	12	0	100%	75	0	100%
9	Porous Pavement	SA Device, LU# 7 ,SA# 32	258	0	100%	1	0	100%	75	0	100%
10	Porous Pavement	SA Device, LU# 8 ,SA# 31	753	0	100%	4	0	100%	75	0	100%
11	Porous Pavement	SA Device, LU# 8 ,SA# 32	74	0	100%	0	0	100%	75	0	100%

Table 63. Calculated stormwater control performance (cont.)

Control practice no.	Control practice type	Control practice name or location	Influent median part. Size (microns)	Effluent (surface overflow) median part. Size (microns)	Maximum stage (ft)	Hydraulic volume out (cf)	Maximum surface ponding time (hrs)	Maximum subsurface ponding time (hrs)
1	Biofilter	DS Biofilters # 1	29.31	29.31	4.77	10,718	92	90
2	Biofilter	DS Biofilters # 2	29.31	29.31	4.77	8,091	87	87
3	Biofilter	DS Biofilters # 3	29.31	29.31	1.77	11,174	92	91
4	Biofilter	DS Biofilters # 4	29.31	29.31	1.77	7,292	86	87
5	Biofilter	DS Biofilters # 5	29.31	29.31	3.27	1,191	57	77
6	Biofilter	DS Biofilters # 6	29.31	29.31	3.27	6,284	83	85

Table 63. Calculated stormwater control performance (cont.)

Control practice no.	Control practice type	Control practice name or location	Volume infiltrated (cf)	Underdrain discharge Vol. (cf)	Evapo- transpir. vol. (cf)	Surface discharge bypass vol. (cf)	Surface ponding events >72 hrs (count)	Runoff producing events/ total rains
1	Biofilter	DS Biofilters # 1	40,152	0	350	10,666	1	68/190
2	Biofilter	DS Biofilters # 2	42,468	0	683	8,004	1	83/190
3	Biofilter	DS Biofilters # 3	39,781	0	341	11,123	1	56/190
4	Biofilter	DS Biofilters # 4	43,720	0	718	7,203	1	37/190
5	Biofilter	DS Biofilters # 5	47,937	0	2,116	1,038	0	88/190
6	Biofilter	DS Biofilters # 6	44,017	0	964	6,176	1	42/190
7	Grass Swales	DS Grass Swales # 1	0					0/190
8	Porous Pavement	SA Device, LU# 7 ,SA# 31	2,635	0				0/190
9	Porous Pavement	SA Device, LU# 7 ,SA# 32	258	0				0/190
10	Porous Pavement	SA Device, LU# 8 ,SA# 31	753	0				0/190
11	Porous Pavement	SA Device, LU# 8 ,SA# 32	74	0				0/190

Table 64. Calculated stormwater conditions for treated and untreated areas

	Area (acres)	Area as a % of total area	Runoff volume (ft^3/year)	Rv	Partic solids (mg/L)	Part. solids yield (lb/yr)	Part. solids yield (lb/ac/yr)	% flow of total area	% part. solids of total area	% Flow reductions compared to untreated conditions	% Part.solids reductions compared to untreated conditions
Total Site Conditions, before controls	100.30		2,802,000	0.23	204	35,677	356	n/a	n/a	n/a	n/a
Untreated site area	45.80	45.7%	1,097,000	0.20	195	13,356	292	39.2%	37.4%	n/a	n/a
Area to be treated	54.50	54.3%	1,704,000	0.26	210	22,321	410	60.8%	62.6%	n/a	n/a
Total site conditions, after controls	100.30		1,284,000	0.11	187	14,998	150	n/a	n/a	54.2%	58.0%
Untreated site area	45.80		1,097,000	0.20	195	13,356	292	85.4%	89.1%	0.0	0.0
Treated area with controls	54.50		186,714	0.03	141	1,642	141	14.5%	10.9%	89.0%	92.6%

The following report sections are summaries of how these stormwater controls are modeled and how they can be sized to provide the desired benefits of a stormwater management program.

Summary of Monitored and Modeled Performance of Stormwater Control Practices

The Kansas City GI demonstration project site is unique because a very large portion of the test (pilot) area receives direct treatment from many separate stormwater control devices, and the large area is being monitored to demonstrate the actual flow reductions. However, as in all retrofit installations, stormwater controls could not be placed to treat all the flows from the entire watershed area because of interferences from existing infrastructure, large trees, and surface drainage paths. The map in Figure 93 shows the subareas having stormwater control before being discharged into the combined sewer. The blanked-out areas drain into the combined sewers directly without any surface infiltration or retention control. Some areas are treated by multiple control units, with overflows from upgradient devices flowing into downgradient controls.

Figure 93. Areas receiving surface stormwater control before being discharged into the combined sewer.

The total impervious area for the area being treated is about 45%; the total impervious area for the untreated area is about 37%, indicating greater flows from the treated areas than indicated than indicated if based only on the total subareas. The calculations and modeling efforts determine the maximum amounts of stormwater control possible, reflecting the different land development characteristics in the treated and untreated subareas and showing the sensitivity of the native soil conditions on biofilter performance.

Figure 94 compares the modeled to the monitored events that occurred for the first nine events after the majority of the site construction was completed. The model used a native soil infiltration rate of 1 in/hr below the biofilters, which results in reasonable predictions as shown in the figure. Lower native soil infiltration rates (as used in the initial design calculations) resulted in significantly decreased calculated discharges, resulting in poor fits of the data.

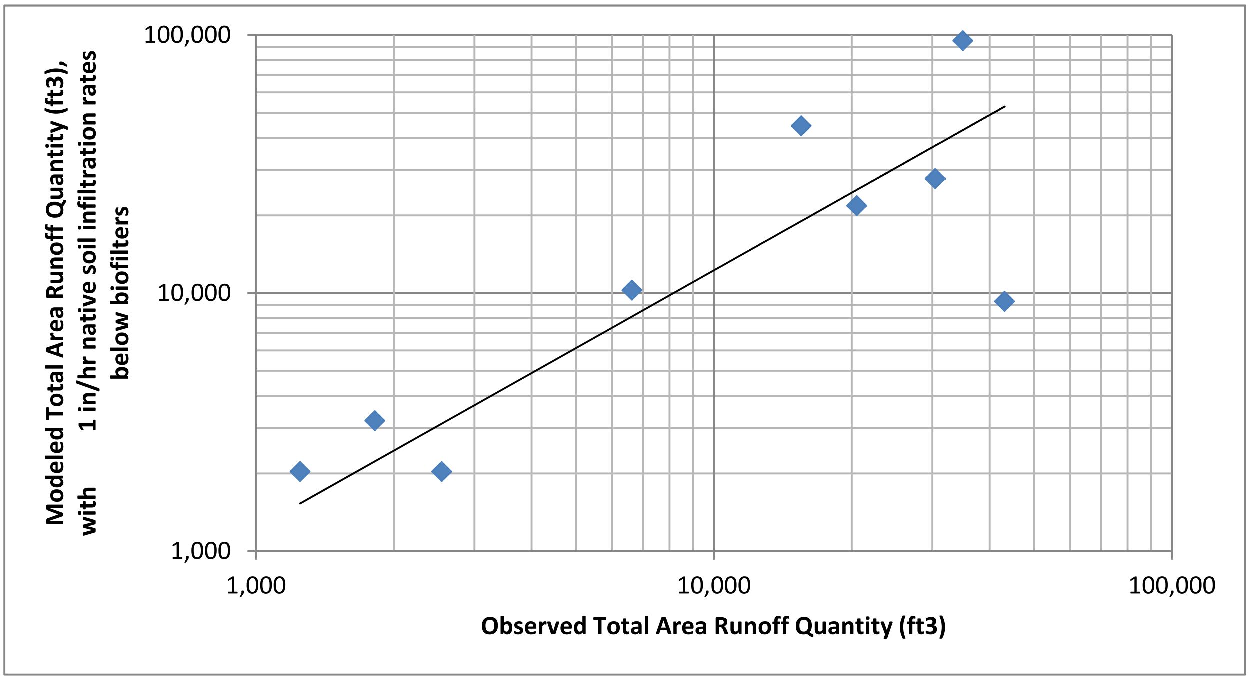
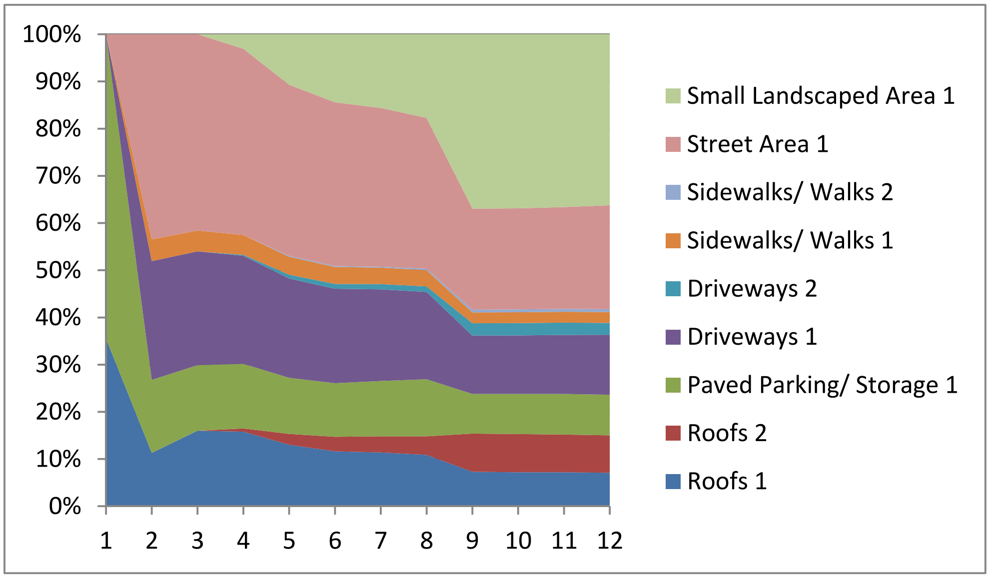


Figure 94. Modeled versus observed flows in the test (pilot) area after construction of stormwater controls.

One of the main features of WinSLAMM is its ability to calculate these source contributions for varying rain conditions. Figure 95 illustrates the source contributions for the test (pilot) area without stormwater controls, for rains ranging from 0.01 to 4 in. The sources of flows (and pollutants) vary with the rain characteristics, but the directly connected areas are most important for the small- and intermediate-sized rains, with pervious contributions becoming more important as rains increase in size.



Event number on graphs	Rain depth (in)
1	0.01
2	0.05
3	0.10
4	0.25
5	0.50
6	0.75
7	1.00
8	1.50
9	2.00
10	2.50
11	3.00
12	4.00

Figure 95. Sources of runoff volume during different rain events (no control practices).

Table 65 summarizes the characteristics for each category of stormwater control used in the test (pilot) area, including the number of each device type and the average areas being treated by each type of control. The device areas as a percentage of drainage area are also shown and range from about 1.5 to 2% for the biofilters to 9% for the bioswale. The porous pavement sidewalks treat 100% of the sidewalk areas because they do not receive runon from adjacent areas.

Table 65. Summary of the stormwater controls constructed in the test (pilot) watershed

Design plan component	Structural description	Number of this type of stormwater control units in test (pilot) area	Drainage area to device area ratio	Device as a % of the drainage area	Drainage area for each unit (ac)	Total area treated by these devices (ac)
Bioretention	Bioretention without curb extension	24	61.8	1.6%	0.40	9.6
	Curb extensions with bioretention	28	66.1	1.5%	0.40	11.2
	Shallow bioretention	5	61.8	1.6%	0.40	2.0
Bioswale	Vegetated swale infiltrates to background soil	1	11.2	8.9%	0.50	0.5
Cascade	Terraced bioretention cells in series	5	53.0	1.9%	0.40	2.0
Porous sidewalk or pavement	With underdrain	18	1.0	100.0%	0.015	0.3
	With underground storage cubes	5	1.0	99.9%	0.015	0.1

Design plan component	Structural description	Number of this type of stormwater control units in test (pilot) area	Drainage area to device area ratio	Device as a % of the drainage area	Drainage area for each unit (ac)	Total area treated by these devices (ac)
Rain garden	Rain garden without curb extension	64	35.8	2.8%	0.40	25.6
	Curb extensions with rain gardens	8	66.0	1.5%	0.40	3.2
	Total number of control units (w/o porous pvt)	135			Total area treated	54.4
	Total area treated (acres)	54.4				
	Area per unit	0.40				

The calculated runoff volume reductions range from 86 to 100% for a 4-year continuous simulation period (September 2008 through October 2012). The predicted maximum water depths in the biofilters ranged from about 2 to 5 in, similar to the water depths observed. The maximum ponding times for the biofilters ranged from about 60 to 90 hours. Only a single event in the 4 years of simulation had a holding time longer than 3 days, the typical criterion for mosquito control. Only about one-third of the events likely have any surface or underdrain discharges, and these amounts would be very small compared to the untreated volumes.

7. Stormwater Control Production Functions

WinSLAMM was used to examine a series of stormwater control practices, including rain barrels and water tanks for stormwater irrigation, pavement and roof disconnections, roof rain gardens, infiltration/biofiltration in parking lots and as curb-cut biofilters, street cleaning, grass swales, porous pavement, and selected combinations of these practices for the Kansas City regional land use conditions. The model evaluates the practices through engineering calculations of the unit processes on the basis of the actual design and size of the controls specified, and it determines how effectively the practices remove runoff volume and pollutants.

WinSLAMM does not use a percent imperviousness value or a curve number to generate runoff volume or pollutant loadings. The model applies runoff coefficients to each *source area* in a land use category. Each source area has a different runoff coefficient equation based on factors such as slope, type and condition of surface, soil properties, and such, and calculates the runoff expected for each rain. The runoff coefficients were developed using monitoring data from typical examples of each site type under a broad range of conditions. The runoff coefficients are continuously updated as new research data become available.

For each rain in a data set, WinSLAMM calculates the runoff volume and pollutant load for each source area. The model then sums the loads from the source areas to generate a land use or drainage basin subtotal load. The model continues this process for the entire rain series included in the rain file. It is important to note that WinSLAMM does not apply a *unit load* to a land use. Each rainfall produces a unique load from a modeled area on the basis of the specific source areas in that modeled area.

The model replicates the physical processes occurring in the practice. For example, for a wet detention pond, the model incorporates the following information for each rain event:

1. Runoff hydrograph, pollution load, and sediment particle size distribution from the drainage basin to the pond
2. Pond geometry (depth, area)
3. Hydraulics of the outlet structure
4. Particle settling time and velocity in the pond based on retention time

Stokes Law and Newton's settling equations are used in conjunction with conventional surface overflow rate calculations and modified Puls-storage indication hydraulic routing methods to determine the sediment amounts and characteristics that are trapped in the pond. Again, it is important to note that the model does not apply *default* percent efficiency values to a control practice. Each rainfall is analyzed, and the pollutant control effectiveness varies according to each rainfall and the pond's antecedent condition.
The model's output is comprehensive and customizable, and typically includes

1. Runoff volume, pollutant loadings and EMCs for a period of record or for each event, or both
2. The above data pre- and post- for each stormwater management practice
3. Removal by particle size from stormwater management practices applying particle settling
4. Other results can be selected related to flow-duration relationships for the study area, impervious cover model expected biological receiving water conditions, and life-cycle costs of the controls

A full explanation of the model's capabilities, calibration, functions, and applications is at www.winslamm.com. For this project, the parameter files were calibrated using the local Kansas City monitoring data, supplemented by additional information from regional data from the NSQD, available at http://www.unix.eng.ua.edu/~rpitt/Research/ms4/mainms4.shtml.

Pavement and Roof Disconnections

The first stormwater control that should be considered in an area is disconnecting the directly connected impervious areas, such as roofs and paved parking lots. WinSLAMM can evaluate disconnections in different ways. The most direct way to evaluate disconnections of impervious areas is by changing the source area parameter characteristic from directly connected (or draining to a directly connected area) to draining to a pervious area (partially connected impervious area), as shown in Figure 96 for moderately compacted silty soils. If the area has clayey soils, the building density is also needed, and if it is a medium- or high-density area, the presence of alleys also needs to be known. This process is based on extensive monitoring of residential and commercial sites that ranged from completely connected to completed disconnected with varying density and soil conditions (Pitt 1987). Table 66 shows the results of these disconnections, showing excellent control when all areas are disconnected. For example, to obtain good receiving water habitat conditions, all the roofs and the parking areas must be disconnected in this example. As expected from observing the flow source area plot, disconnecting only a portion of these impervious areas has limited benefits. It is noted that the concentrations of the pollutants increase with increasing roof disconnections because the better quality roof runoff is being infiltrated and not diluting the runoff from the paved parking/storage area. However, the mass discharges all decrease with increased disconnections.

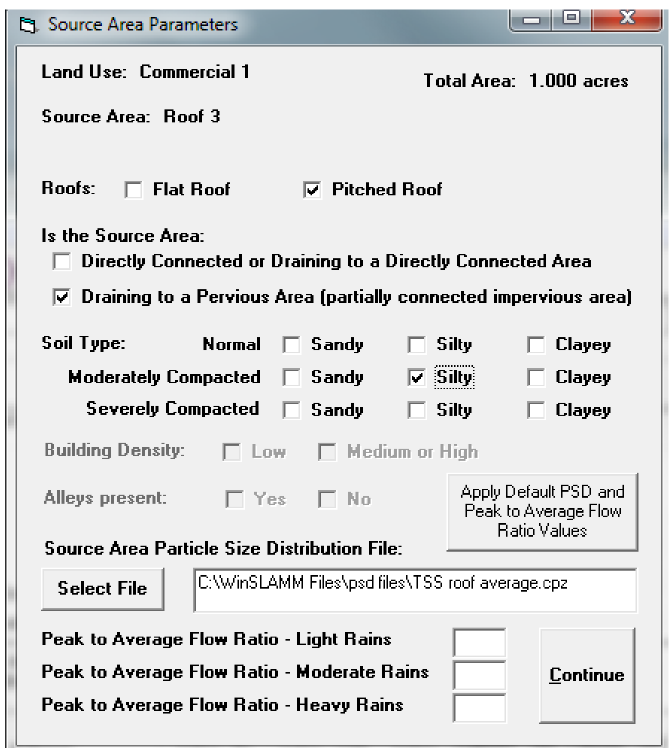


Figure 96. Disconnection of pitched roof to silty soil.

Table 66. Effectiveness of disconnecting impervious areas in 2.25-acre commercial site over 10 years

Description	Rv	Expected habitat conditions	TSS (mg/L)	solids yield (lbs/yr)	peak runoff rate (cfs)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
Base conditions, no controls	0.55	Poor	100	1,040	4.6	0.28	29	17	1.7
Flat roof disconnections	0.47	Poor	112	990	3.8	0.29	26	18	1.6
Pitched roof disconnections	0.46	Poor	115	980	3.7	0.29	25	18	1.6
Both roof disconnections	0.38	Poor	132	930	3.0	0.31	22	20	1.4
Parking lot disconnections	0.25	Poor	66	309	1.9	0.36	17	12	0.56
All roofs and parking area disconnections	0.08	Good	140	200	0.72	0.67	9.8	15	0.21

Rain gardens, rain barrel/tanks, and disconnection of roof runoff are controls being used on private property in the residential areas in the Kansas City Marlborough GI test (pilot) area. Their maximum benefit is therefore dependent on the amount of runoff that is contributed from the source areas where they would be located. These controls receive runoff from the roofs. Table 67 shows that the directly connected roofs contribute only about 5.8%, but the much greater area of disconnected roofs contribute about 7.2% of the annual runoff from the entire 100-acre area. The current flow contributions of all roofs in the area total about 13%. If all the roofs were directly connected, the roofs would contribute about 31% of the total area runoff, and the runoff from the total area would increase by about 25%, a significant increase. In contrast, if the directly connected roofs were disconnected through a downspout disconnection program, the total roof contribution would decrease to about 9%, and the total area runoff would decrease by about 5%. Because about 85% of the existing roofs in the area are already disconnected, the benefits of controlling the remaining directly connected roofs are therefore limited.

Table 67. Effectiveness of roof area disconnections.

	Roof 1 areas (directly connected) (1.87 acres)	Roof 2 areas (disconnected) (10.57 acres)	Land use total (100 acres)	Whole area Rv
Base conditions (ft^3/year)	257,200	319,200	4,449,000	0.30
% contributions	5.8%	7.2%		
% roof contributions	13.0%			
if all roofs connected (ft^3/year)	257,200	1,458,000	5,588,000	0.38
% contributions	4.6%	26.1%		
% roof contributions	30.7%			
if all roofs disconnected (ft^3/year)	56,340	319,200	4,248,000	0.29
% contributions	1.3%	7.5%		
% roof contributions	8.8%			

Table 68 shows that directly connected roofs in the study area contribute about 4.5 times the amount of runoff per unit area as the disconnected roofs. This indicates that about 78% of the annual runoff from the disconnected roofs is infiltrated as it passes over pervious areas on the way to the drainage system. Therefore, it is much less cost-effective to use roof runoff controls for the runoff from the disconnected roofs compared to runoff controls for the directly connected roofs. If an infiltration or beneficial use control is used to control runoff from disconnected roofs, they would have to be about 4.5 times larger than if used for runoff control from directly connected roofs, to have the same benefit on the overall discharge volume from the area.

Table 68. Disconnected and directly connected roof runoff differences

	Area (acres)	Annual runoff (ft^3)	Runoff contributions to outfall per roof area (ft^3/acre/year)
Roof 1 areas (directly connected)	1.87	257,200	137,500
Roof 2 areas (disconnected)	10.57	319,200	30,200
Ratio of disconnected to directed connected	5.65	1.24	0.220

The benefits of disconnecting connected paved parking or storage areas are similar to the benefits shown above for roofs. However, disconnecting these areas as part of a retrofit program is likely to be difficult because extensive re-grading would be needed, or at least a suitable adjacent undeveloped or landscaped area downgradient of the paved area would be needed. No such areas are available in the test area, for example, and are expected to be rare. In redevelopments and in new developments, this might be a more suitable option. However, the use of biofilters to infiltrate the runoff at directly connected paved areas is likely a much more suitable option.

Roof Runoff Rain Gardens

Private rain gardens for controlling roof runoff are being used in the residential areas in the Kansas City CSO GI demonstration project test (pilot) area. The performance of these devices is affected by several unit processes, which are modeled in WinSLAMM. Modified puls hydraulic routing, with surface overflow calculations, are the basic processes modeled. However, several layers in the rain garden (or biofilter) must be considered. As runoff enters the device, water infiltrates through the engineered soil or media (or natural soil, in a rain garden). If the entering rain cannot all be infiltrated through the surface layer, the water ponds. If the ponding becomes deep, it can overflow through the broad-crested weir or other surface outlet. The percolating water moves down through the device until it reaches the bottom and intercepts the native soil. If the native soil infiltration rate is greater than the percolation water rate, no subsurface ponding occurs; if the native soil infiltration rate is slower than the percolation water rate, ponding occurs. This ponding can build up to the surface of the device and add to the surface ponding. If an underdrain is present (usually with a subsurface storage layer), the subsurface ponding will be intercepted by the drain which then discharges it to the surface water, but later in the event (or directly to the combined sewer system).

With the water percolating through the engineered soil or other fill, particulates and particulate-bound pollutants are trapped by the media through filtering actions. Therefore, the underdrain water usually has a lower particulate solids content that the surface waters entering the device. The calculations are sensitive to the amount of the different media used as fill (or the native soil) and its characteristics (especially its porosity and percolation rate; and if ET is used, the wilting point). The hydraulic routing uses the sum of the void volumes in the device to determine the effluent hydrograph, while the different infiltration/percolation rates affect the internal ponding. The stage-discharge relationships of the outlet devices are all modeled using conventional hydraulic processes. The ET loss calculations are based on the changing water content in the root zone at each time increment, and the ET adjustment factors for the mixture of plants in the device (Pitt et al. 2008a).

Figure 97 is the main WinSLAMM input screen used for rain gardens. This is a general format that is also used for other infiltration devices, including biofilters and bioinfiltration devices. This form includes the geometry of the device and material placed in the device. Most simple rain gardens do not have any special media, using only soils, nor do they have underdrains, so only some of the form is used. In this example, a loam soil is used in the rain garden, and the subsurface native soil is assumed to be a sandy loam having long-term infiltration rates of about

1.0 in/hr. As indicated, it is possible to also incorporate a Monte Carlo routine to better represent the variable infiltration rates that any individual unit has. All the devices using this input screen require a hydraulic overflow, described as a broad crested weir. For these devices, evaporation of water from any pooled standing water above the soil and ET losses associated with plants installed in the rain garden, are also added as outlet devices. The engineered soil media characteristics screen is shown in Figure 98, as an example.

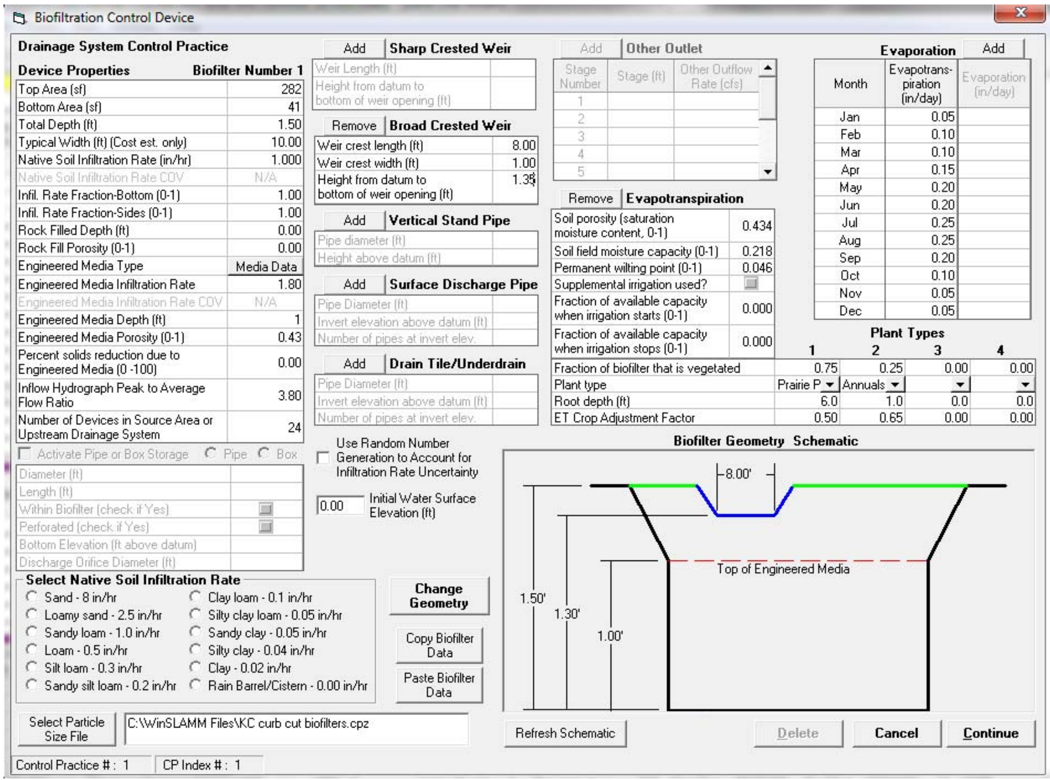


Figure 97. Rain garden input screen.

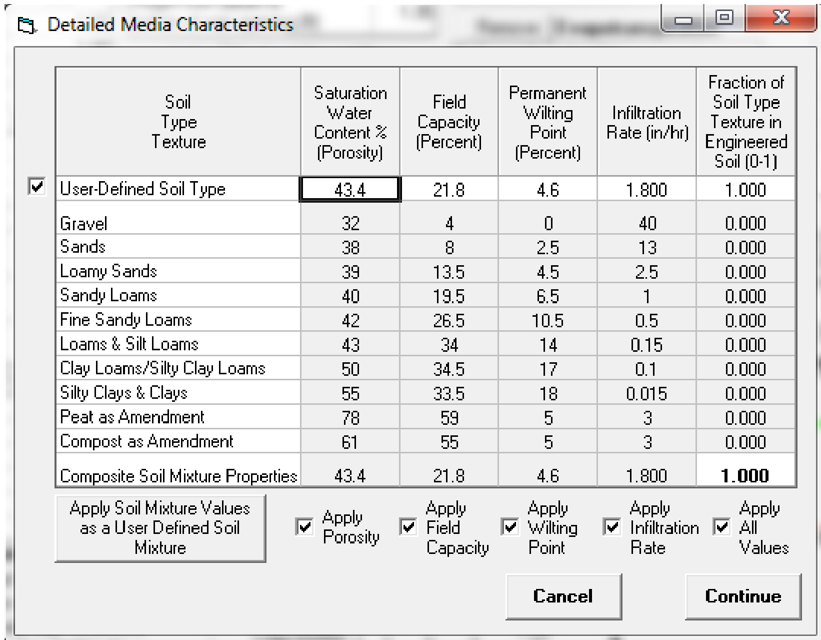


Figure 98. Detailed media characteristics for rain gardens.

The performance of a rain garden for controlling runoff from directly connected pitched roofs is summarized in Table 69 and Figure 99. As a rain garden increases in size in relationship to the roof area, less water is discharged to the storm or combined sewer. About 80% of the long-term runoff would be infiltrated for a rain garden that is about 20% of the roof area for these conditions. The concentrations all remain the same, because there is no underdrain or subsurface collection of filtered water; the water quality of the water discharged through the surface overflow weir is assumed to be the same as the incoming water. However, the mass discharges are decreased as the runoff volume decreases. The roof runoff has relatively low TSS concentrations, and the life of the rain gardens shown here would be very long, with very little clogging potential (clogging of biofilters occur with accumulative solids loadings of about 10 to 25 kg/m^2). The peak flow rate reductions are also substantial; about 64% reductions of the uncontrolled peak flow rate for rain gardens that are about 20% of the roof area.

Table 69. Rain garden performance for directly connected pitched roofs

Rain garden as a % of contributing roof area	Estimated habitat conditions	TSS (mg/L)	Peak runoff rate (cfs)	Peak flow rate reduction (%)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
	Poor	33	0.87	0	0.22	4.2	11	0.21
2%	Poor	33	0.78	10	0.22	3.4	11	0.17
5%	Poor	33	0.67	23	0.22	2.6	11	0.13
10%	Poor	33	0.47	46	0.22	1.6	11	0.08
15%	Poor	33	0.34	61	0.22	1	11	0.05
20%	Fair	33	0.31	64	0.22	0.59	11	0.029
25%	Good	33	0.28	68	0.22	0.35	11	0.017
30%	Good	33	0.22	75	0.22	0.19	11	0.0095
40%	Good	33	0.15	83	0.22	0.039	11	0.0019
50%	Good	33	0.079	91	0.22	0.01	11	0.00045

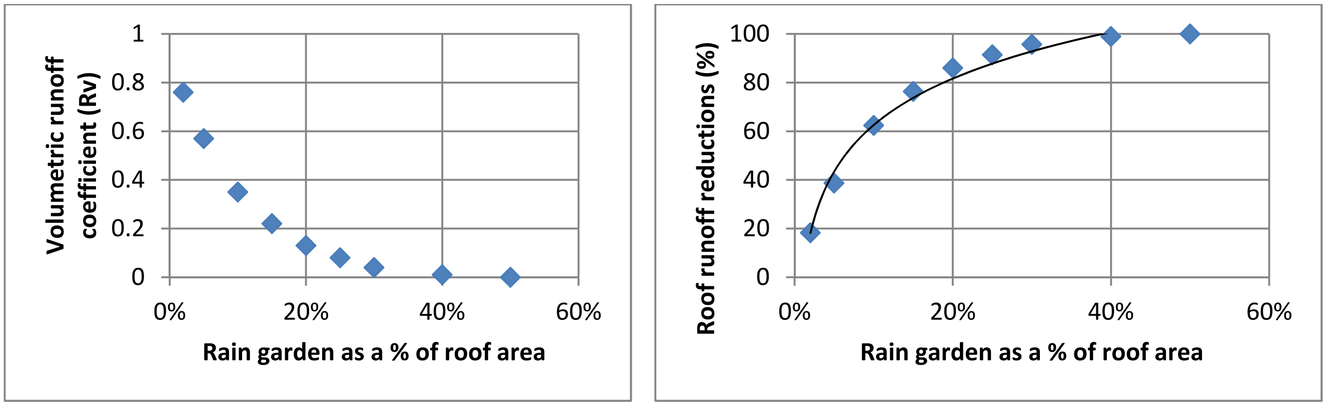


Figure 99. Calculated roof runoff rain garden performance as a function of size.

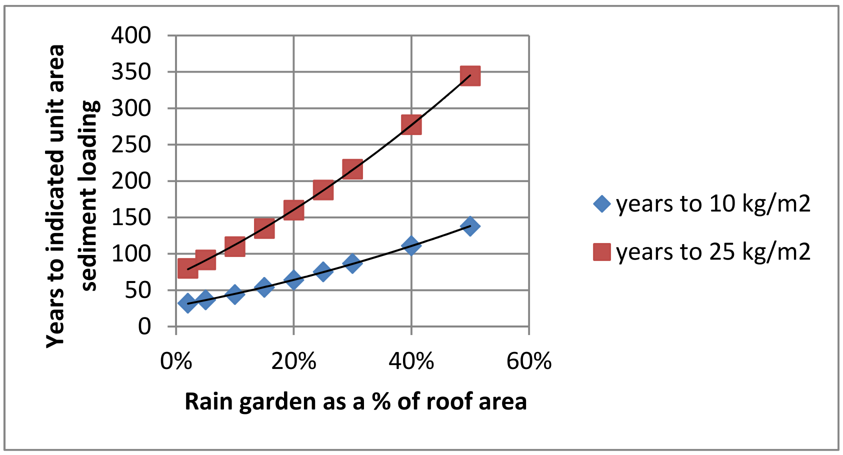


Figure 99. Calculated roof runoff rain garden performance as a function of size (cont.).

Another example is for rain gardens having a top surface area of 160 ft^2, being about 10 by 16 ft in area. It is excavated to an overall depth of 3 ft, with 2 ft backfilled with a loam soil. The top 1 ft of surface is left open to provide surface storage 9 in deep (with a 3 inch overflow weir opening). A native soil infiltration rate of 0.2 in/hr was used in these calculations, while the loam soil fill had only a 0.15 in/hr infiltration rate. The only outlet used (besides the natural infiltration) is a surface overflow along one edge of the rain garden. One of these rain gardens per house represents about 17% of the typical roof area in the study area.

Table 70 and Figure 100 summarize the continuous modeling results for several different sizes and numbers of rain gardens, per house, according to the 1990 rain year (the year that was selected as being representative of the long-term rain record for Kansas City). As noted above, disconnected roofs already experience substantial runoff reductions (about 78%) in the study area, even when low infiltration rates are assumed. Rain gardens sized to be about 13% of the roof areas would be equivalent to the current benefits of disconnected roof drainage. This corresponds to a rain garden having about 120 ft^2 of surface area per house, with the rain garden overflow then flowing directly to the drainage system.

Table 70. Numbers and sizes of rain gardens to provide specific roof runoff flow benefits

# rain gardens per house	ft^2 of rain gardens per house	% of roof area as rain garden	% reduction in roof runoff	Total number of rain gardens if usage rate applied to all 576 homes	Total storage in rain gardens if applied to all 576 homes (ft^3)	Total storage in rain gardens if all 576 homes used them (gal)	Total storage in rain gardens if only used for 86 directly connected roofs (gal)
0	0	0%	0%	0	0	0	0
0.035	5.6	1%	5%	20	2,460	18,400	2,760
0.1	16	2%	14%	58	7,030	52,600	7,890
0.25	40	4%	33%	144	17,600	131,500	19,700
0.5	80	8%	57%	288	35,140	263,000	39,400
1	160	17%	84%	576	70,300	526,000	78,900
2	320	34%	96%	1150	140,500	1,052,000	158,000
4	640	68%	100%	2300	281,100	2,104,000	316,000

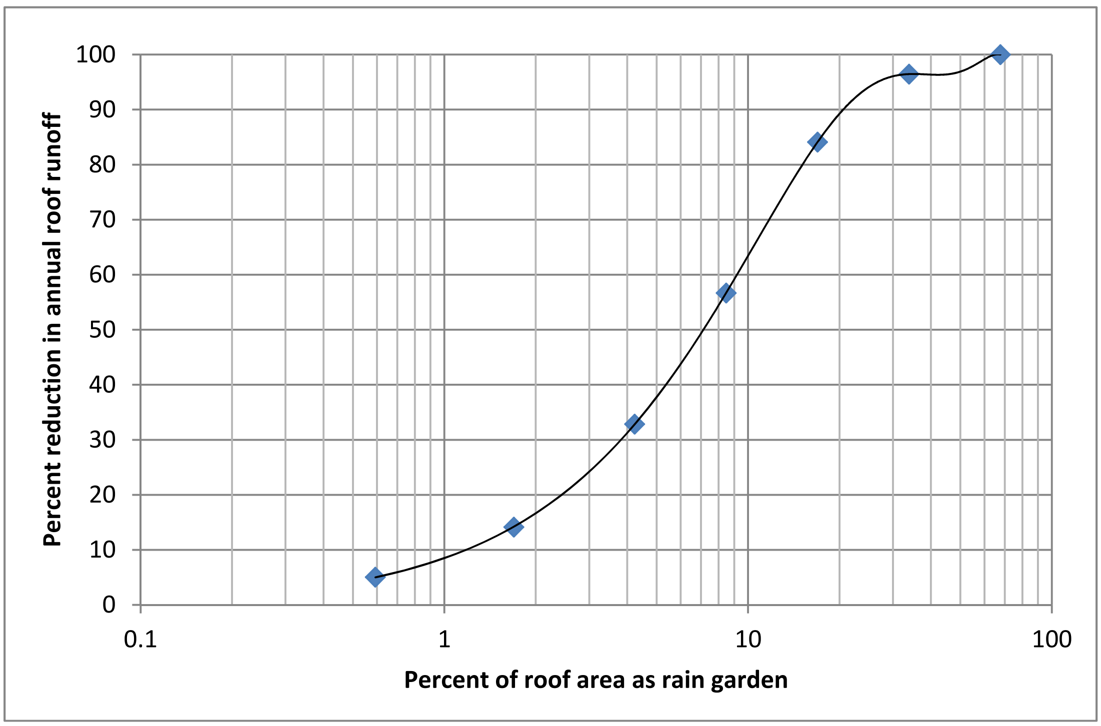


Figure 100. Production function for rain garden use for control of total annual roof runoff volume.

The continuous simulations examined all 98 rain events that occurred in the typical 1990 rain year. The six rains closest to 1.4-in total depth (the critical event for the local CSO consent decree) for this year are shown in Tables 71 and 72. During this year, three rains were also larger than 1.4 in: 3.23, 3.11, and 2.18 in. The six rains close to 1.4 in ranged in depth from 1.21 to 1.76 in and had durations ranging from 8 to 28 hours. Antecedent dry periods ranged from 8 hours to about 4 days, and the total rain depth that occurred in the week before these rains ranged from 0.02 to 1.24 in.

Table 71. Large rains close to 1.4-inch design storm *D*

Date	Rainfall (in)	Event duration (hrs)	Average rain intensity (in/hr)	Prior event interevent period (days)	Prior event rain depths for at least a week before (in, and its prior interevent periods in days)	Total rain fall in week before event (in)
3/14/1990	1.28	28	0.05	0.33	0.14 (0.67); 0.52 (1.1); 0.08 (0.25); 0.19 (3.0)	0.93
4/26/1990	1.76	26	0.07	0.92	0.03(5.0); 0.01 (5.0)	0.04
6/6/1990	1.22	8	0.15	3.8	0.01 (3.1); 0.01 (5.2)	0.02
6/8/1990	1.22	12	0.1	2.1	1.22 (3.8); 0.01 (3.1); 0.01 (5.2)	1.24
7/21/1990	1.67	13	0.13	0.58	0.39 (0.33); 0.08 (6.5)	0.47
10/2/1990	1.21	15	0.08	3	0.12 (8.5)	0.12
average	1.39	17	0.10			0.47
standard deviation	0.25	8.1	0.038			0.51
COV	0.18	0.48	0.38			1.1

The storage provided in the rain gardens is somewhat larger than the amount of runoff removed during the design storm *D* that is 1.4 in. Continuous simulations of this one year's rains considers antecedent conditions in the rain garden, specifically, some of the storage capacity might not be available because some of the water from a prior event might not have completely drained. This is especially true in areas of poorly draining soils. The total drainage time in this general rain garden design is about 4 days, with about 1.5 to 2 days needed to drain the maximum ponding on the surface of the rain garden. Any rain that occurs before the rain garden can completely drain will increase the overall drainage time needed and reduce the amount of effective storage available for a subsequent event.

Table 72. Roof runoff volumes for large rains close to 1.4-in design storm *D*

Date	Rainfall (in)	Base conditions, total runoff (ft^3/100 ac)	Base conditions, Rv at outfall	Directly con. roof (ft^3 86 of 576 homes)
3/14/1990	1.28	151,000	0.32	8,497
4/26/1990	1.76	227,000	0.35	11,739
6/6/1990	1.22	143,000	0.32	8,098
6/8/1990	1.22	143,000	0.32	8,098
7/21/1990	1.67	211,000	0.35	11,113
10/2/1990	1.21	141,000	0.32	8,032
average	1.39	169,000	0.33	9,260
standard deviation	0.25	38,800	0.015	1,700
COV	0.18	0.23	0.047	0.18

For up to one rain garden per house (17% of roof area), the storage provided is about 30 to 40% greater than the actual amount of runoff removed during storms that are close to the 1.4-in depth. This additional storage volume is related to the typical antecedent conditions before these rains, especially assuming the low infiltration rates used in this example. When the desired level of performance increases, this over-design volume also increases. When two rain gardens are used per house (totaling 34% of the roof area), the actual storage in the rain garden is about 2.3 times the volume removed, and when the rain garden usage is further increased to four per house (64% of roof area), the actual storage is about 4.6 times the roof runoff removed. This is evidenced by the non-linear plot shown below, which flattens out considerably for the largest removal rates. Using two rain gardens per house results in complete removal of the runoff from directly connected roofs from the drainage system during this 1.4-in site design storm, so that is the practical upper limit when considering only the design storm regulatory objectives. When the number of rain gardens is increased above one, the rain gardens do not always fill completely during all the rains in this size category. However, additional rain garden area could be used to increase the total amount of runoff reduction when the complete annual rain series is considered, as shown above. Using two rain gardens per house provides 100% control of the regulatory design storm, and it results in an expected 96% reduction in the total annual runoff from the directly connected roofs, as shown in Figure 101.

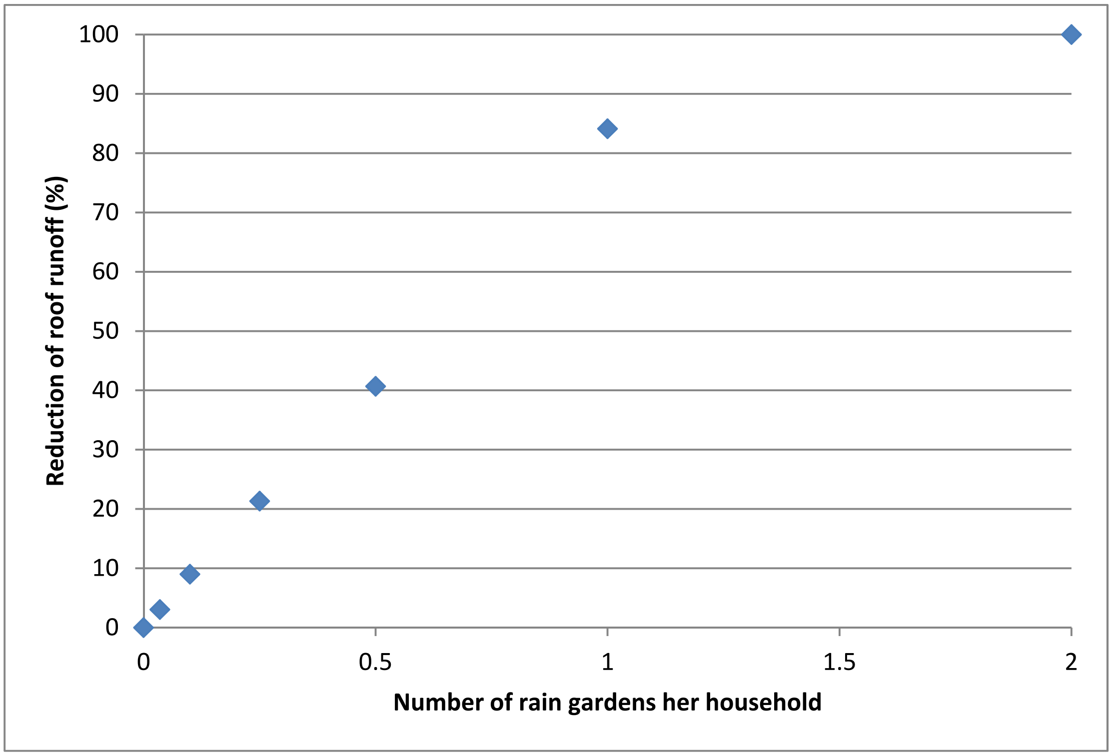


Figure 101. Performance function of roof runoff rain garden use and 1.4-in design storm *D* used for regulations.

A goal of reducing 90% of the runoff from directly connected roofs in the study area would require rain gardens that are about 20% of the roof areas, or a total area of slightly less than 200 ft^2 per house. This would also provide about 90% runoff reductions from the directly connected roofs during the 1.4-in regulatory design storm *D.* In most cases, this area would be made of two to four separate smaller rain gardens per house, depending on the locations of the roof gutter downspouts. With a peaked roof that all drains to one end of the house, two would be needed (each about 100 ft^2 of area), but for a more common peaked roof that drains to each corner separately, four separate smaller rain gardens would be needed (each about 50 ft^2 of area).

Curb-Cut Biofilters

Biofilter performance is based on the characteristics of the flow entering the device, the infiltration rate into the native soil, the filtering capacity and infiltration rate of the engineered media fill if used, the amount of rock fill storage, the size of the device and the outlet structures for the device. Pollutant filtering by the engineered media (usually containing amendments) is based on the engineered media type and the particle size distribution of the inflowing water, or the user can directly enter the percent reduction from filtering that is directed by a regulatory agency. If the engineered media flow rate is lower than the flow rates entering the device, the engineered media will affect the device performance by forcing the excess water to bypass the device through surface discharges if the storage capacity above the engineered media is inadequate.

The device operation is modeled using the Modified Puls Storage-Indication method and is analyzed differently depending on whether a rock and engineered media layer is in the model. The model simulates the inflow and outflow hydrographs using a time interval selected by the user (typically 6 minutes), although this interval is reduced automatically by the program if the simulation approaches becoming unstable.

The complex triangular inflow hydrograph is divided into the selected time intervals, which are routed to the surface of the biofilter. The biofilter is evaluated in two basic sections: the aboveground section (or above the engineered media) and the belowground section (below the surface of the engineered media). If there is a rock and engineered media layer, the available surface outflow devices include broad (required) and sharp crested weirs, vertical stand pipe, evaporation/ET, and flow through the engineered media.

As water enters the device, all flow is routed to the belowground section of the device as long as the engineered media infiltration rate is greater than the inflowing water rate. As the inflow rate increases, the aboveground storage begins to fill once the inflow rate exceeds the engineered media infiltration rate. If the inflow rate is high enough and the excess runoff volume exceeds the available storage, the water begins to discharge from the device through the aboveground surface outflow devices. As water enters the belowground section of the device, it discharges through the native media and, as the bottom section fills, through the underdrain (if used). All water that flows through the underdrain is assumed to be filtered by the engineered media. The filtering performance changes based on the type of engineered media and varies by the particle size, which also affects the minimum effluent concentration. If the water level in the belowground section of the device reaches the top of the engineered media layer, infiltration from the surface layer into the belowground layer is not possible until the water level in the belowground section is below the top of the engineered media layer. If there are no rock and engineered media layers, flow into the native soil is considered to be an outflow: there is no belowground section, and all treatment by the device is assumed to be through volume loss by infiltration into the native soil.
Biofilters can be used as control devices in individual source areas or as a part of the drainage system. To model biofilters, the geometry and other characteristics of the biofilter are described, or of a typical biofilter if modeling a set of biofilters for, say, roofs or parking lot source areas. The number of biofilters to be modeled in the source area is also entered on the form. The model divides the total source area runoff volume by the number of biofilters in the source area, creates a complex triangular hydrograph for that representative flow fraction that is then routed through that biofilter. It then multiplies the resulting losses by the number of biofilters for the total source area.

Biofilter Data Entry

Figure 102 is the data entry form used for biofilters and related stormwater controls.

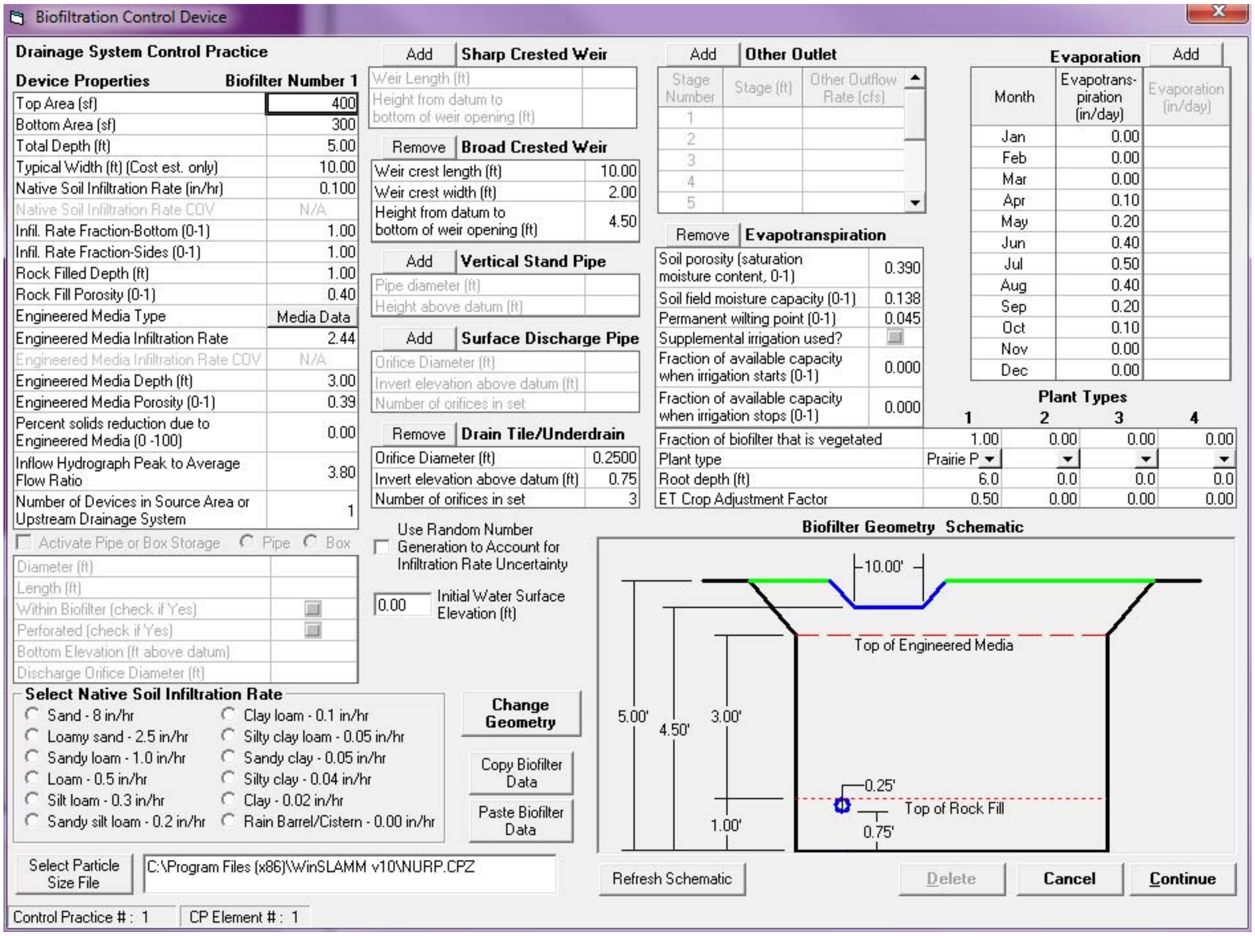


Figure 102. Basis data entry screen for biofilters and bioinfiltration stormwater controls.

The bottom of the biofilter has a datum of zero. To describe the biofilter, the following information is entered.

Device Geometry

- Top Area (square feet): Enter the top area of the biofilter
- Bottom Area (square feet): Enter the bottom area of the biofilter
- Total Depth (feet): Enter the depth of the biofilter.
- Typical Width (ft): If you intend to perform a cost analysis of the biofilter practices listed in the .mdb file, you must enter the typical biofilter width (ft) of a biofilter system you are modeling. This value is not used for a hydraulic or water quality analysis; it is relevant only for the cost analysis.
- Native Soil Infiltration Rate (in/hr): Enter the infiltration rate or select a typical infiltration rate based on soil type from the provided list in the lower left-hand corner of the window. The native soil infiltration rate value, based on a large number of tests performed by Pitt is supplied if you select the typical seepage rate provided by the model.
- Native Soil Infiltration Rate COV (Coefficient of Variation): If you want to consider the typical variabilities in the infiltration rates, select the “Use Random Number Generation to Account for Uncertainty in Infiltration Rate” checkbox and then accept or enter another seepage rate COV value in the cell below the native soil infiltration rate. This is optional and uses a Monte Carlo simulation built into the model. If selected, the

infiltration rates are randomly varied for each event based on a log-normal probability distribution of actual measured infiltration rate variabilities.

- Infiltration Rate Fraction - Bottom (0-1): Enter the seepage rate multiplier for bottom flow (from 0 to 1) to reduce the seepage rate through the bottom of the biofilter. This option can be useful if you want to evaluate the effects of clogging on the bottom of the device.
- Infiltration Rate Fraction - Side (0-1): Enter the seepage rate multiplier for side flow (from 0 to 1) to reduce the seepage rate through either the sides of the biofilter. This option can be useful if you want to evaluate the effects of clogging on the bottom of the device or ignore the benefits of seepage out of the sides of the device, as assumed by some regulatory agencies.
- Rock Filled Depth (ft): This is the depth of biofilter that is rock filled. This must be less than or equal to the biofilter depth, and may be zero if there is no rock fill. Water is assumed to flow through the rock storage layer very quickly.
- Rock Fill Porosity: Enter the fraction of rock fill that is voids as a value from zero to one. If you have both rock fill and engineered soil, the model calculates and uses the weighted average of the two porosity values to determine the benefits of this subsurface storage. If you are using an underdrain, a rock storage layer is usually required.
- Engineered Media Type. If the device has an engineered soil layer, the program enters an infiltration rate depending on the type of engineered media, based on extensive media tests in laboratory columns and in the field. Select the 'Media Data' button to enter media type information including the media porosity, infiltration rate, field moisture capacity and permanent wilting point.
- Engineered Media Infiltration Rate (in/hr): If you have selected a specific engineered media type, the program enters a measured infiltration rate for that media, or if you selected a user defined media type, you may enter your own engineered media infiltration rate.
- Engineered Media Depth (ft). This must be less than or equal to the biofilter depth, and may be zero if there is no engineered media fill.
- Engineered Media Porosity (0-1): This is the fraction of engineered media that is voids - enter the porosity of the engineered media as a value from zero to one. If you have both rock fill and engineered media, the model calculates and uses the weighted average of the two porosity values.
- Percent Solids Reduction Due to Engineered Media. If you want to enter a percent solids reduction value from engineered media if permitted to do so by the regulatory agency or because you have suitable data, select "User-Defined" as the engineered media type in the Detailed Soil Characteristics form. If you select any other engineered media type, the program calculates the percent reduction based on that media type.
- Inflow Hydrograph Peak Flow to Average Flow Ratio. This value is used to determine the shape of the complex triangular unit hydrograph that is routed through the device. A typical value of the peak to average flow ratio is 3.8, based on monitoring many urban areas (Pitt, et al. 2012). However, short duration events in small areas may have larger ratios and similarly, long duration events in large areas may have smaller ratios. WinDETPOND can evaluate any inflow hydrograph shape that you enter. In version 10, it is recommended that the option to use the hydrograph from upgradient areas and controls be used instead of resetting this value to 3.8.
- Number of Devices in the Source Area or Upstream Drainage System. The model divides the runoff volume by the number of biofilters in the source area or land use, creates a complex triangular hydrograph that it routes through that biofilter, and then multiplies the resulting losses by the number of biofilters to apply the results to the source area.
- Particle Size Distribution File. The particle size distribution of the particulates in the runoff affects the percent solids reduction of the engineered media layer. The program has pre-defined percent solids

reductions for selected particle size distributions. If you have a user-defined engineered media type, then you do not need to enter a particle size distribution file. If you select the 'Route Hydrographs and Particle Sizes Between Control Devices' checkbox in Program Options/Default Model Options, the program uses the default particle size distribution file for all source areas. The particle size distribution entering the control device is modified by whatever practices are upstream of the control practice. If the practice is the most upstream practice, the default particle size distribution is used.

- Pipe or Box Storage is not activated.

Typical Kansas City curb-cut rain gardens along the street were assumed to be simple excavations 20 ft (6.1 m) long and 5 ft (1.5 m) wide in the terrace between the sidewalk and the street, but most of the curb-cut rain gardens installed in the test area are about 2 to 4 times this size. The following example calculations are still valid, as long as the *unit* rain gardens are 100 ft^2 of area and the actual components are sized accordingly. Their depth was limited to 1 ft (0.3 m) to decrease uneven steep slopes and other hazardous conditions. It is assumed that the subsoil would be loosened after the excavation, and a minimum amount of organic material would be added to the soil. There are less than 6 mi (9.6 km) of street side drainage systems in the 100 acre (40.5 ha) test watershed. Therefore, a maximum of 1,500 small street side rain gardens (150,000 ft^2 total rain garden area) was assumed to be possible in the area. However, a more reasonable maximum number would be 750 (75,000 ft^2 total rain garden area) because of the presence of large trees and other interferences.

Figure 103 is a plot of the percentage of the typical annual runoff amount that can be infiltrated by the curb-cut rain gardens on the basis of the number of units used and with no other controls in the area. With a maximum 1,500 units possible (total of 150,000 ft^2, or 3.4% of the 100 acres being treated), up to 80% of the annual runoff can be infiltrated. With 400 units (total of 40,000 ft^2, or less than 1% of the 100 acres being treated), 40% of the annual flows would be diverted from the combined sewers.

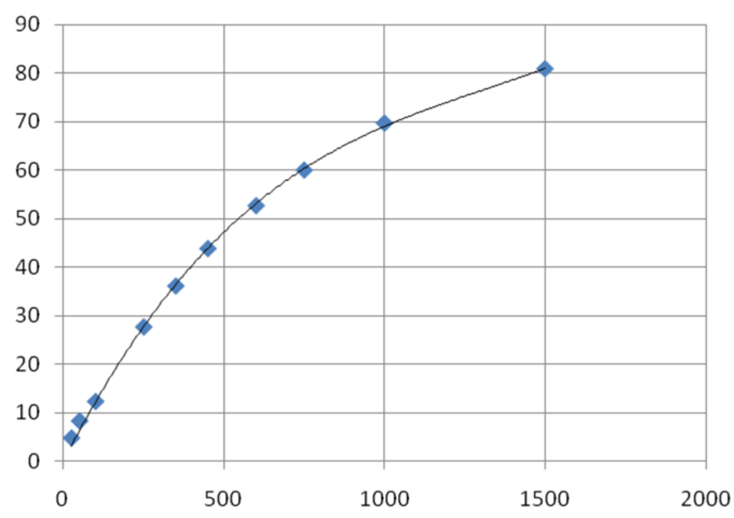


Figure 103. Annual runoff volume reduction (%) for typical rain year (1990) for different numbers of simple curbcut rain gardens (100 ft^2 each) per 100-acre watershed.

Figure 104 shows the durations of flows at different rates for several different curb-cut rain garden applications. The maximum peak flow for the typical rain year is expected to be more than 25 ft^3 (708 L)/sec and less than 30 ft^3

(850 L)/sec for this area. The use of 600 rain gardens is likely to reduce the flow rates that occur for 0.1% of the annual hours (about 5 h/y to 10 h/y) to half the value if uncontrolled.

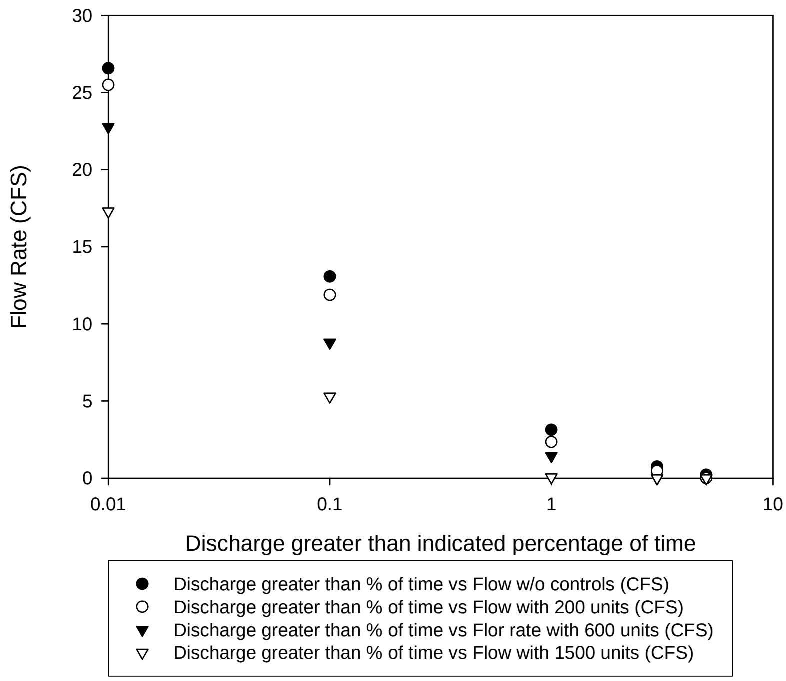
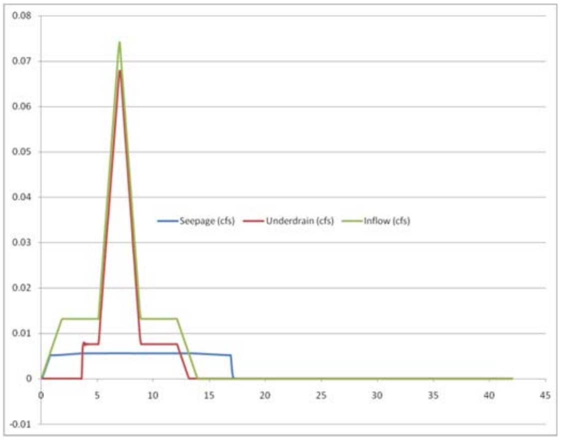


Figure 104. Durations of flows (percentage of time) for different numbers of simple curb-cut rain gardens.

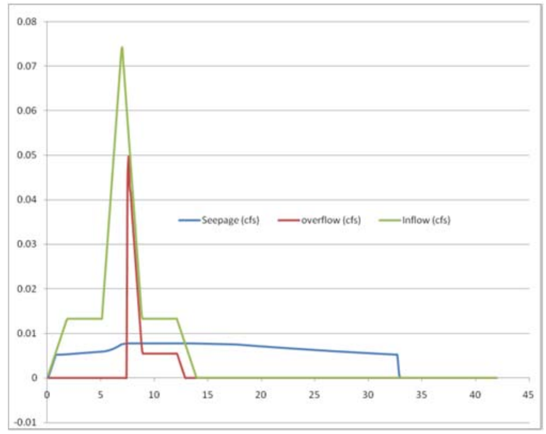
Use of Underdrains in Biofilters

The treatment of stormwater by biofilters is dependent on the hydraulic residence time in the device for some critical pollutants. The effective use of biofilters for controlling stormwater in combined sewered areas is also related to residence time because it is desired to retain the water before discharge to the drainage system to reduce the peak flows to the treatment plant. This section describes the results from a series of tests conducted by Redahegn Sileshi, while a Ph.D. student at UA, to determine the hydraulic characteristics of sand-based filter media (having a variety of particles sizes representing a range of median particle sizes and uniformity coefficients) during pilot-scale trench tests (Sileshi et al. 2012a, 2012b, and 2013). The drainage rate in biofiltration devices is usually controlled using an underdrain that is restricted with a small orifice or other flow-moderating component. These frequently fail because the orifices are usually very small (less than 10 mm) and are prone to clogging. A series of tests are also being conducted using a newly developed foundation drain material (SmartDrain™), which offers promise as a low-flow control device with minimal clogging potential. A pilot-scale biofilter comprised of a trough 3 m long and having a cross section of 0.6 x 0.6 m is being used to test the variables affecting the drainage characteristics of the underdrain material (such as length, slope, hydraulic head, and type of sand media). Tests are also being conducted to determine the clogging potential of this drainage material. This report describes the initial tests that have investigated the basic hydraulic properties and the clogging potential of this drain material.

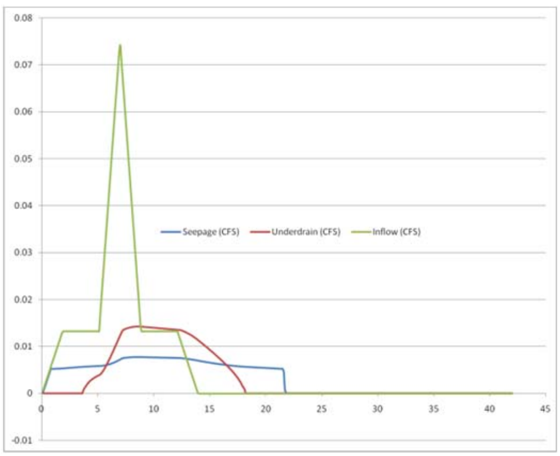
Figure 105 is an example showing the effects of a small bioretention facility and different underdrain options. Depending on the objectives (peak flow reduction, infiltration, or filtering of the water), different underdrain options can be selected. Sizing the controls can also be evaluated using the model based on both short-term and long-term rain records for the area.



Conventional underdrain



No underdrain



Restricted underdrain

Example Biofilter Performance and Design using WinSLAMM

0.75-in rain, 12 hour duration, with complex inflow hydrograph from 1 acre of pavement; the biofilter surface area corresponds to 2.2% of the paved area, with natural loam soil (0.5 in/hr infilt. rate) and 2 ft of modified fill soil for water treatment and to protect groundwater.

Conventional underdrains allow significant short-circuiting and less infiltration and little peak flow attenuation.

Figure 105. Initial design evaluation of alternative bioretention facility designs.

A typical biofilter that is 1 m deep, 1.5 m wide and 5 m long would require about 8 hours to drain using the SmartDrain^{TM} material as the underdrain. This is a substantial residence time in the media to optimize contaminant removal and provides significant retention of the stormwater before being discharged to a combined sewer system. In addition, this slow drainage time allows infiltration into the native underlying soil, with reduced short-circuiting to the underdrain.

The smart drain has many micro channels in an 8-in width, as shown in Figure 106. The micro channel inlet area composes over 20% of the active drainage surface of the belt/ribbon.

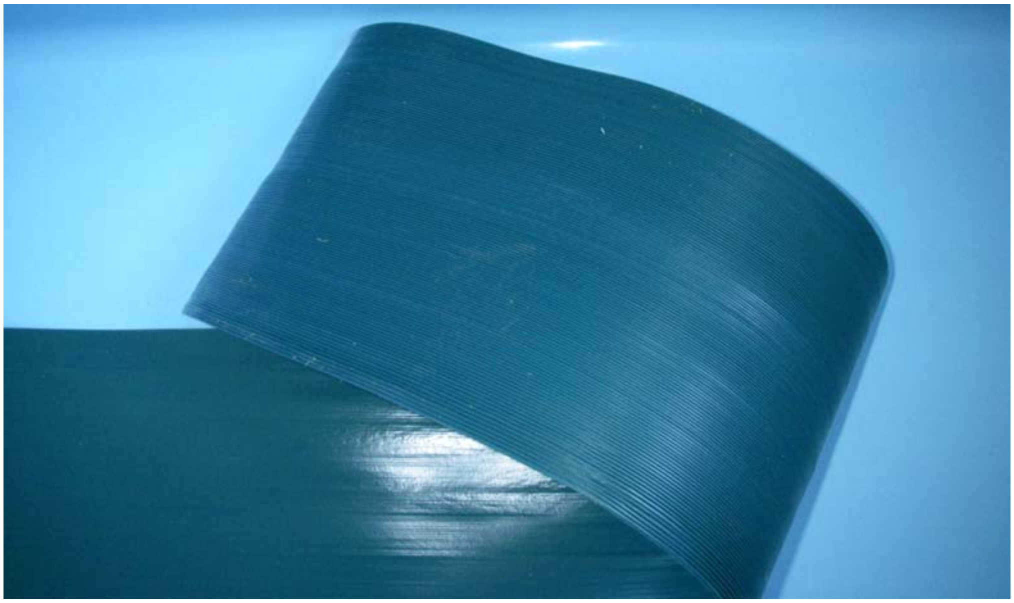


Figure 106. Close-up photograph of SmartDrain™ material showing the microchannels on the underside of the 8-in-wide strip.

The controlled tests investigated the drainage characteristics of the SmartDrain™ material under a range of typical biofilter conditions. A sand filter media purchased locally was used for the pilot-scale test setup to measure the hydraulic characteristics of the drainage material. The particle size distributions of the sand filter media, and the US Silica Sil-Co-Sil 250 ground silica material that is being used in the clogging tests, are shown in Figure 107.

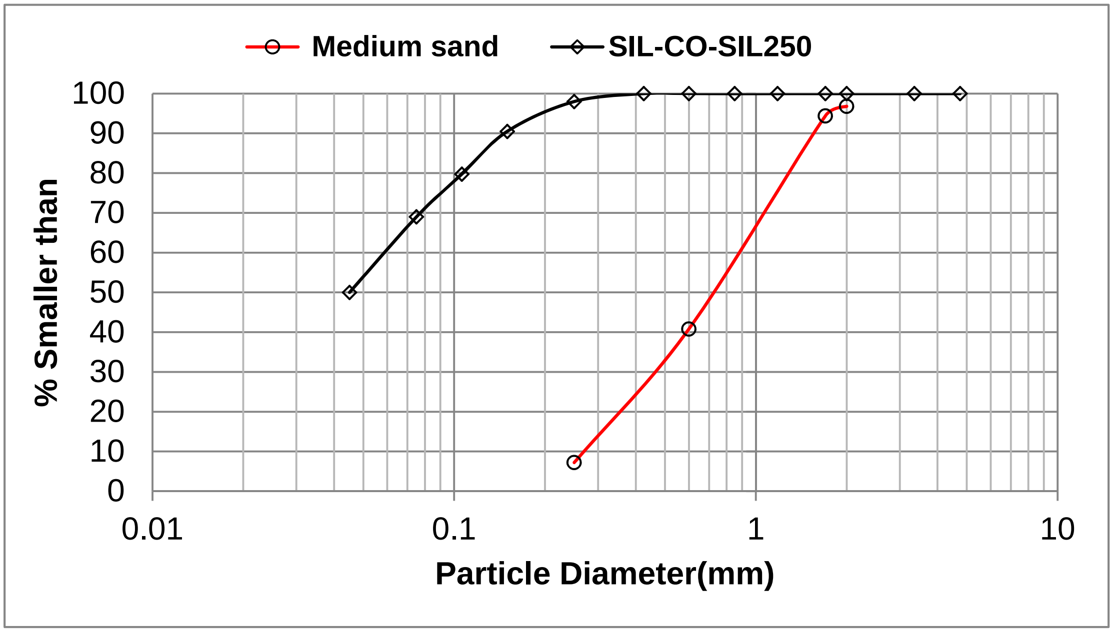


Figure 107. Particle size distribution for medium-sized sand and SIL-CO-SIL250.

Underdrain Testing Procedure

The experimental apparatus for the pilot-scale biofilter tests consisted of a fiberglass trough 3 m long having a 0.6 x 0.6 m cross section. The outlet end of the SmartDrain™ was inserted into a slit cut in the PVC collection pipe and secured with screws and silica sealant (Figures 108 and 109); the sealant is used only on the top smooth surface of the SmartDrain™ material and not on the bottom, which would clog the channels. The SmartDrain™ material is installed with the micro channels on the underside of the strip between two layers of coarse sand, each about 4 in thick. The SmartDrain™ directs the collected water into the PVC pipe, with a several inch drop to enhance siphoning action. The PVC collector pipe used was 2 in (5 cm) in diameter and was placed 1 in (2.5 cm) above the trough bottom. A hole was drilled through the side of the trough for an extension of this pipe outside the trough to allow sampling of the drainage water and to measure the flow rates. During the tests, the trough was initially filled with water to a maximum head of 22 in (56 cm) above the center of the pipe. A hydraulic jack and blocks were used to change the slope of the tank. Different lengths of the SmartDrain™ were tested for a range of slopes. Each test was also repeated several times, and regression analyses were conducted to obtain equation coefficients for the stage versus head relationships for these different conditions.

Figure 108. SmartDrain™ installation procedures in the trough.

The second testing phase examined the clogging potential of the SmartDrain™. Sil-Co-Sil 250, having a median particle size of about 45 µm, was mixed with the test water for the clogging tests. Figure 108 shows the tall, lined box that was used to verify the head versus discharge relationships for deeper water and was used for the clogging tests. This Formica-lined plywood box is 3 ft (90 cm) by 2.8 ft (85 cm) in cross-sectional area and 4 ft (120 cm) tall. The box was filled with tap water to produce a maximum head of 4 ft (120 cm) above the filter, and Sil-Co-Sil 250 was added to the water to provide a concentration of 1 g/L (1,000 mg/L). The box was then drained and flow measurements taken. These clogging tests were continually conducted to result in a high accumulation of the test particulates to measure degradation in performance with increasing loading.

Figure 109. SmartDrain™ installation for the clogging test in the tall box.

Effects of Slope, Lengths, and Sediment Load on the Drainage Characteristics of the SmartDrain™

Five replicates for each of five different lengths of the SmartDrain™ [9.4 ft (2.9 m), 7.1 ft (2.2 m), 5.1 ft (1.6 m), 3.1 ft (0.95 m) and 1.1 ft (0.34 m)] were tested. Two different lengths of the SmartDrain™ (9.4 and 7.1 ft) were tested for five different slopes (0, 3, 6, 9, and 12%) and the remaining three lengths of the SmartDrain™ (5.1, 3.1 and 1.1 ft) were tested for three different slopes (0, 3, and 12%). Flow rate measurements were manually obtained at 25- to 30-minute intervals until the water was completely drained from the pilot-scale biofilters. The flows were measured by timing how long it took to fill a 0.5-L graduated cylinder. Linear regression analyses were used to determine the intercept and slope terms of the head versus discharge relationships. The p values of the estimated coefficients were used to determine if the coefficients were significant ($p < 0.05$). All five lengths tested for the given slopes showed statistically significant slope coefficients ($p < 0.05$), while many of the intercept terms were not found to be significant. Stage-discharge relationships (Figure 111) reflect that the slope of the SmartDrain™ has no significant effect on the effluent flow rates.

Reductions in the outflow rate relationships of the filter media were not observed during the clogging tests (having a total load of more than 30 kg/m^2 onto the filter area). We would normally expect *complete* clogging (to less than about 1 m/day flow rates) after many repeated tests on the same media when a resulting total surface loading of about 10 to 20 kg/m^2 of sediment has been loaded to the filter area.

Influent and effluent turbidity measurements were also taken at 25- to 30-minute intervals at the same time as the flow rate measurements until the water completely drained from the tank. The turbidity (NTUs) values decreased with decreasing head of water in the tank (and effluent flow rate). The initial turbidity levels were about 1,000 NTU in the tank at the beginning of the test. The initial effluent water turbidity values were similar at the beginning of the tests, but significantly decreased as the tests progressed and with flow rates decreases.

Algal fouling of the SmartDrain™ material were also examined by allowing nutrient loaded test water to stagnate in the test tank for extended periods and then conducting flow rate measurements. The pilot-scale biofilter was used for these tests to verify the stage-discharge relationships under adverse algal conditions. During these biofouling tests, the tank was filled with tap water to produce a maximum head of 4 ft (1.2 m). The tank was left open to the sun for several weeks to promote algae growth. Two different algal species collected from a pond on the UA campus and from the Black Warrior River in Tuscaloosa, Alabama, were added to the test water. Miracle-Gro 12-4-8, an all-purpose liquid fertilizer, was also added to increase the algae growth rate in the biofilter tank (Figure 110). Seven biofouling trials were conducted at various algal growth stages in the device, with several weeks between each drainage test. The ponded depth of the test water in the tank for the first five trials was 4 ft (1.2 m), and was reduced to 1.4 ft (0.41 m) for the last two trials to encourage algal growths near the filter sand surface and along the drainage ribbon. At the end of each biofouling test period, the test water was drained, resulting in seven stage-discharge relationships.

Figure 110. Algae in the test tank during the biofouling tests.

Figure 111 summarizes the results of different SmartDrain™ tests under the test conditions. The SmartDrain™ functions similar to a very small orifice of 0.10 to 0.25 in. (2.5 to 6 mm) for all of the tests.

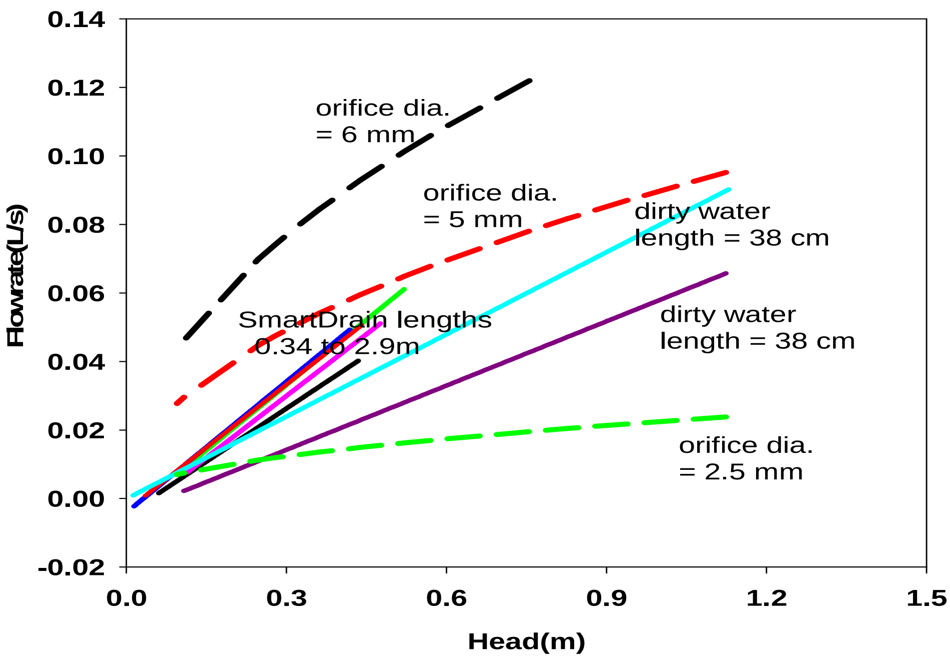


Figure 111. Stage-discharge relationships for various test conditions for the SmartDrain^{TM}.

Production Functions of Curb-Cut Biofilters Using WinSLAMM

WinSLAMM was used with the calibration files prepared for the Kansas City demonstration project to examine alternative biofilter and bioinfiltration device designs for the residential test (pilot) area. Four infiltration rates for the native subsurface soil were examined: 0.2, 0.5, 1.0, and 2.5 in/hr (corresponding to sandy silt loam, loam, sandy loam, and loamy sand, respectively). The lowest rate (0.2 in/hr) was the assumed early infiltration rate used by the design consultants for the original designs. Site surface soil measurements in the test watershed indicated 1 in/hr, or greater, infiltration rates for rains lasting 2 hours or less. Site measurements of the biofilters during storms indicated infiltration rates of the media and device at 1.8 in/hr, and modeling indicated likely subsurface rates of about 1 in/hr to result in the observed performance during the rains (almost complete infiltration with very little overflow or subsurface underdrain discharges). Other features investigated included using alternative underdrain conditions (no underdrain, conventional 3-in perforated pipe underdrain, or a SmartDrain^{TM}), and with gravel storage for the underdrains and with and without the gravel for no underdrain.

The detailed calculation summaries are in Appendix E, and plots and shorter summaries are in this section. The main objectives were to identify how these alternative designs affected performance. Performance was indicated for various sizes of the devices (expressed as a percentage of the test area residential land use), ranging from 0.5 % of the drainage area as the biofilter size to maximum sizes that resulted in 100% runoff infiltration. The main performance measures summarized here are percentage of the annual flows infiltrated (or lost because of ET), number of events having 3 days or more of standing water (the typical stormwater criteria to prevent mosquito problems), the percentage of the annual flows being filtered by the media and then discharged to the combined sewer (and subjected to about a 4-hr delay because of the residence time in the media, benefiting the resultant peak flow rate in the combined sewer), and the potential useful life before clogging can occur. WinSLAMM calculates many other attributes for these devices, but these were selected as the most relevant for this project.

Descriptions of Alternatives Examined

The model used locally measured rainfall starting in September 2008, through October 2012 to correspond to the time when much of the sewer flow monitoring was conducted for this project. During this 4-year period, the average annual rain was about 33 in, and about 46 rains per year occurred (from 0.11 to 3.98 in. each). Figure 112 shows the pattern of these rains with time.

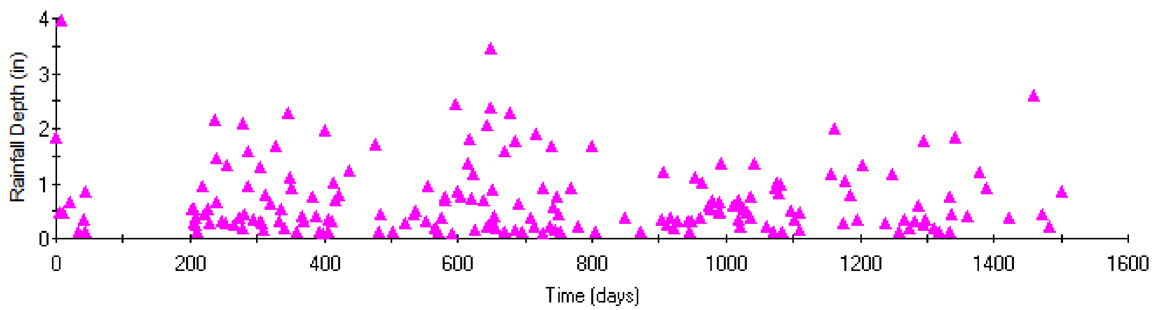


Figure 112. September 2, 2008, to October 12, 2012 rains monitored in the Kansas City GI test area during the demonstration project monitoring period.

Figures 113 through 117 are screenshots showing the four basic setups corresponding to the underdrain conditions for the various biofilters installed. The areas of the biofilter devices and the subsurface infiltration rates were changed for each tested condition. The top area was calculated according to the percentage of the residential drainage area (a unit acre was evaluated). The bottom areas were half of the top areas. The depths of the devices were 2.5 ft if no gravel storage was used, or 5 ft with gravel storage. The media layer was 1.5 ft thick, and its characteristics, shown below were from the analyzed site media used. The media infiltration rate was 1.8 in/hr with a porosity of 0.43. Gravel storage was 2.5 ft thick and had a porosity of 0.4. The ET monthly values are described in another section of this report and were obtained from the closest complete ET monitoring station. The broad crested weir provided a controlled surface discharge location, resulting in about 9 in. of surface pond storage before the overflow. The underdrains examined included a conventional 3-in. perforated pipe. The SmartDrain™ underdrain (with an equivalent orifice of 0.25 in.) was also placed at the same depth. The underdrains were placed 2 ft off the biofilter bottom to provide substantial storage during larger or intense rains.

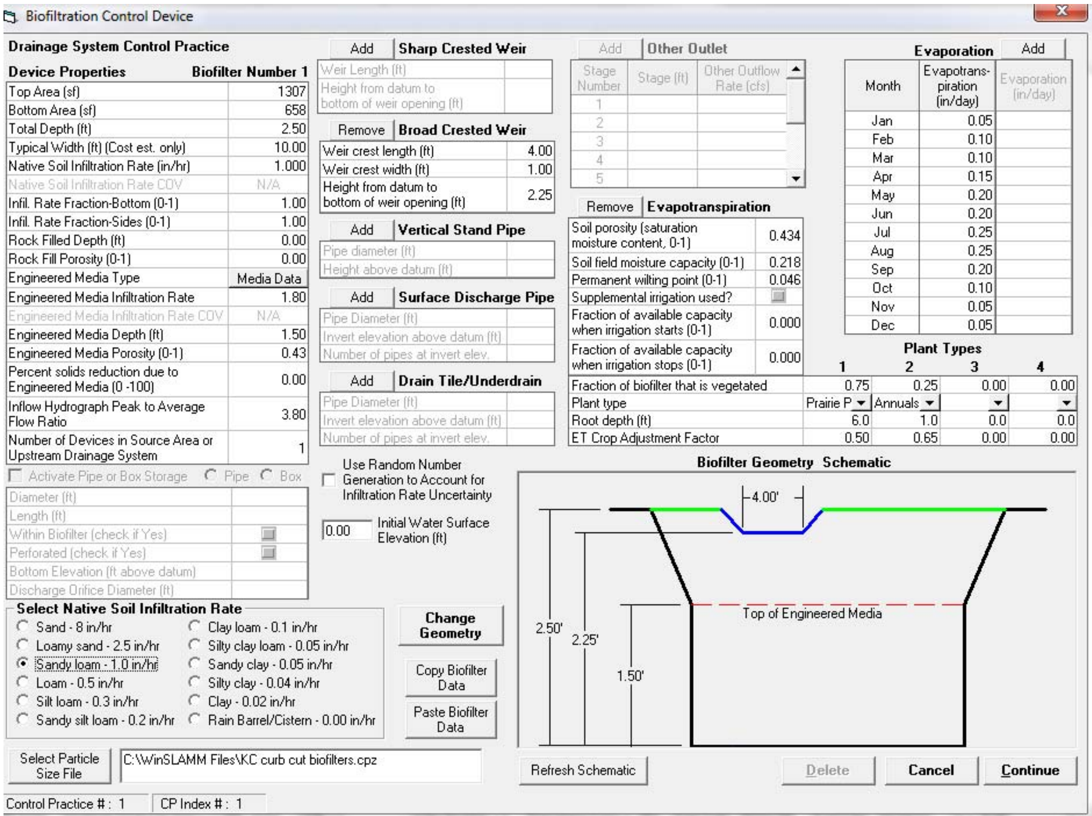


Figure 113. Bioinfiltration device, no underdrain, and no gravel storage.

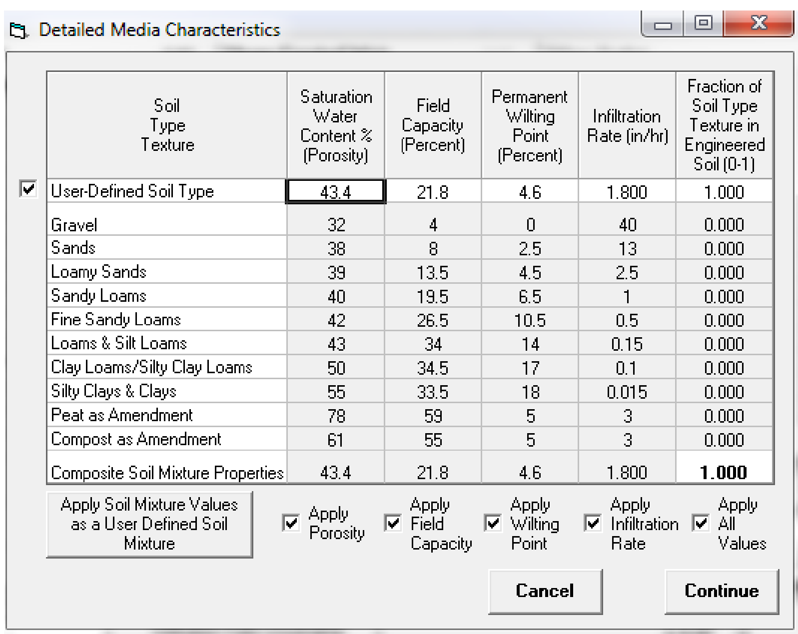


Figure 114. Media characteristics used in the test (pilot) biofilters and bioinfiltration devices.

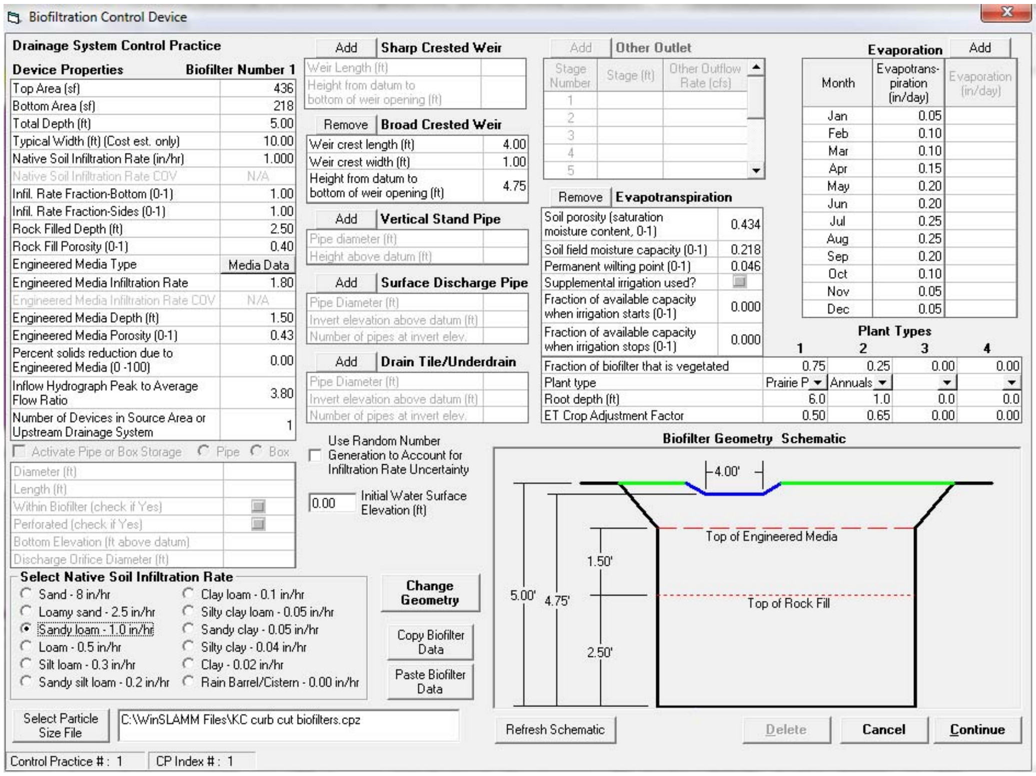


Figure 115. Bioinfiltration device with no underdrain but with gravel storage.

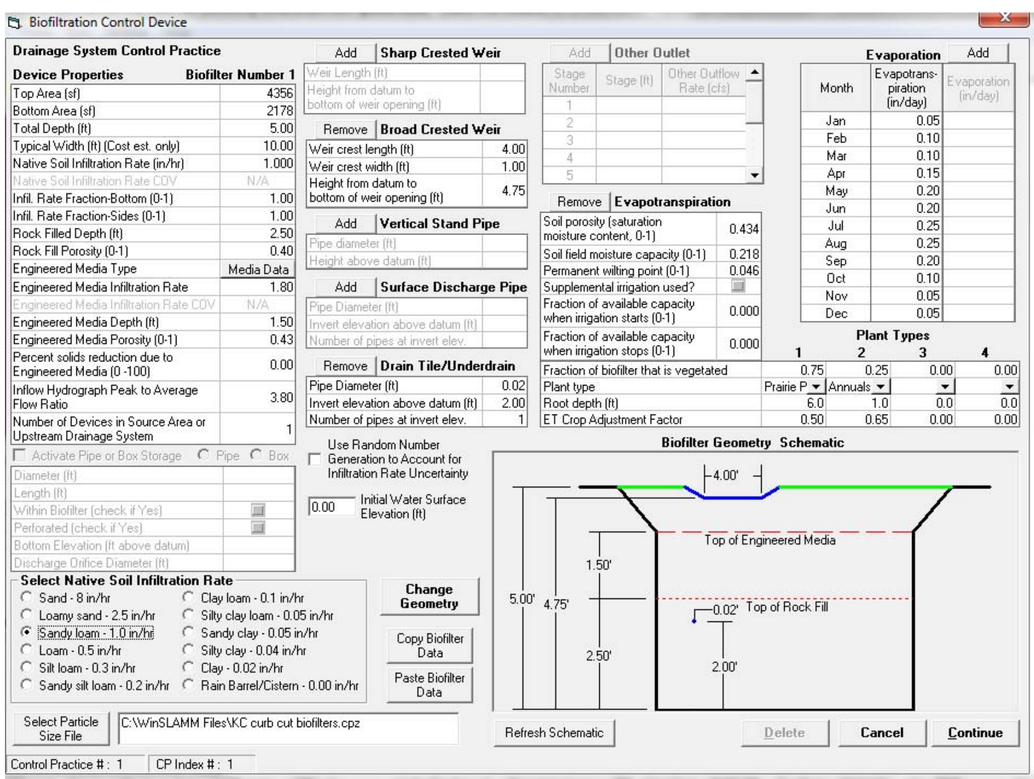


Figure 116. Biofilter with SmartDrain™ underdrain with gravel storage.

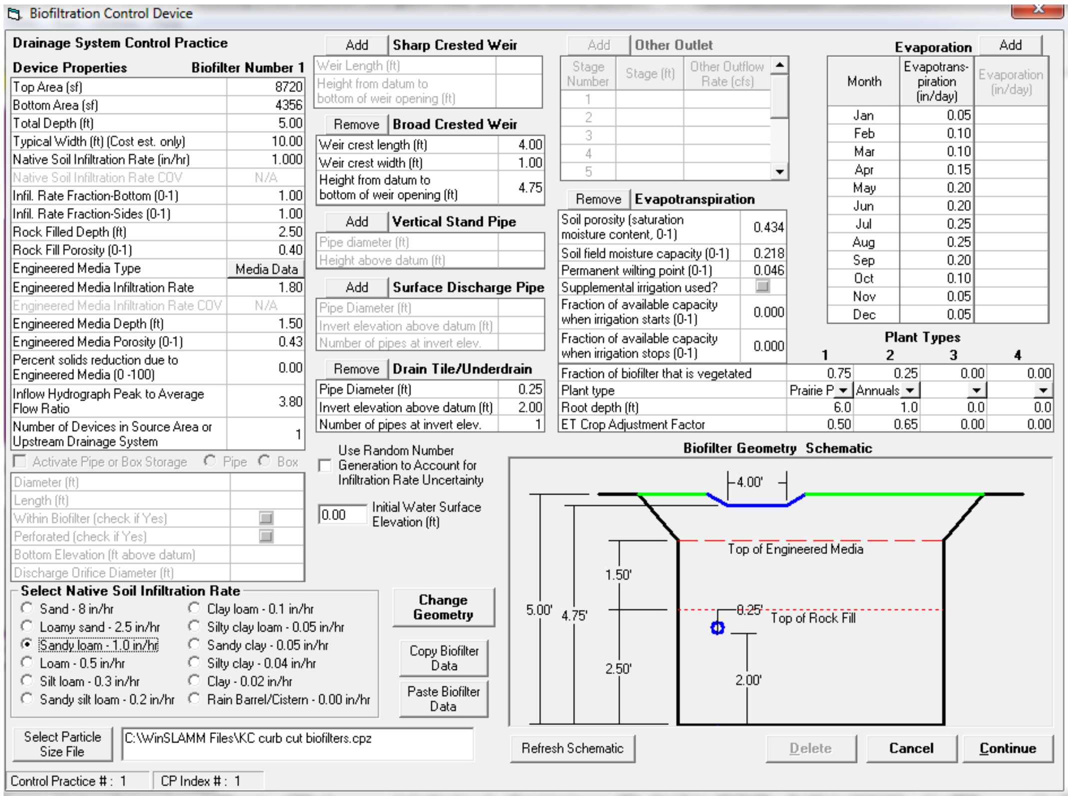
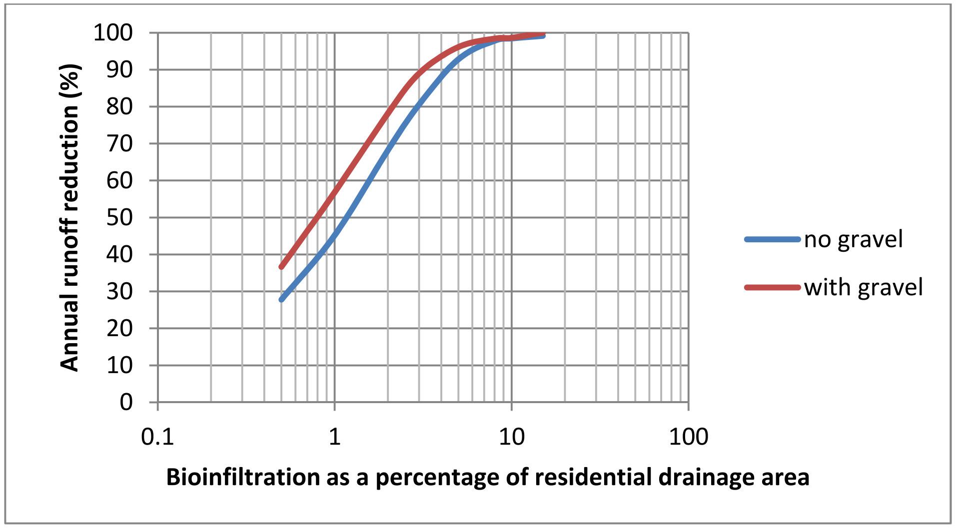


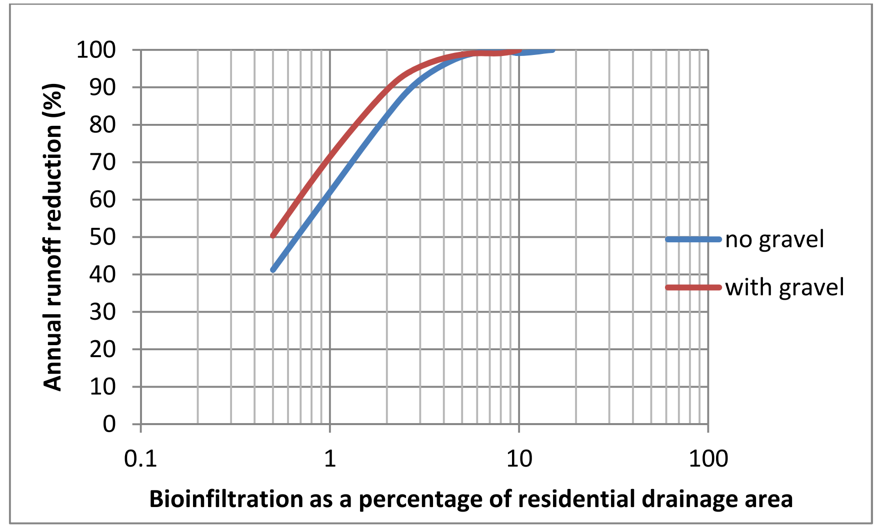
Figure 117. Biofilter with conventional 3-in. underdrain and gravel storage.

Figures 118 through 122 are the production function plots for the conditions examined, followed by summary Table 73. The first four plots compare the no underdrain condition, with and without gravel storage, for the three subsurface native soil infiltration rates. As noted, the use of the gravel storage is important for only the low infiltration rate conditions: once the infiltration rate is about 1 in/hr or larger, this additional storage is not needed, as far as benefiting the long-term infiltration performance.

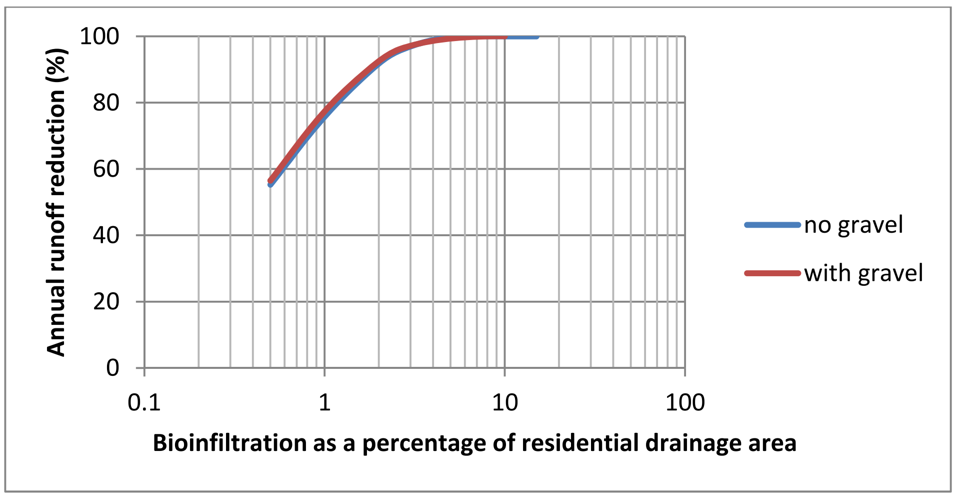
The next four plots show the effects of the underdrains for the infiltration rates. For the low infiltration rates, using underdrains degrades the performance of the biofilters because the underdrains discharge subsurface ponding water before it can completely infiltrate. The use of a slow underdrain (as indicated here by the SmartDrain^{TM}), results in an intermediate effect on infiltration and with decreasing durations of surface ponding. As with the gravel storage, underdrains have very little effect on performance when the native subsurface native infiltration rate is about 1 in/hr or greater.



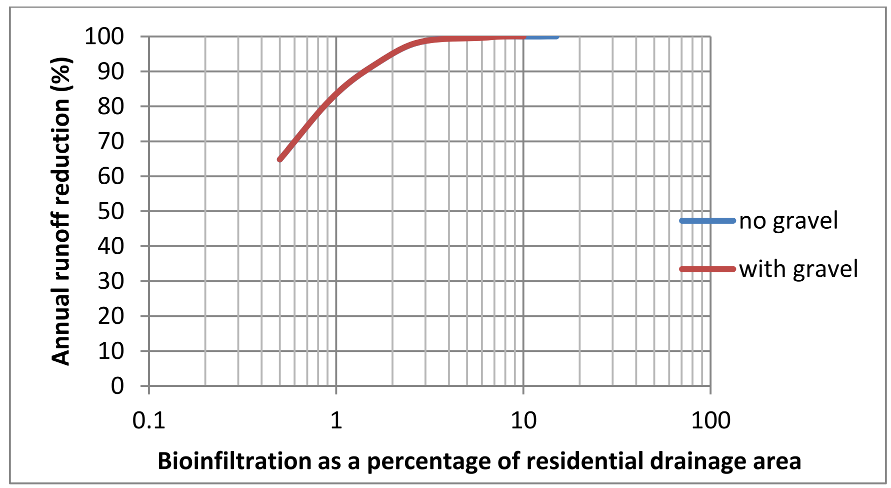
0.2 in/hr infilt. rate



0.5 in/hr infilt. rate



1.0 in/hr infilt. rate



2.5 in/hr infilt. rate

Figure 118. No underdrain alternatives, with varying native soil infiltration rates and with and without gravel storage.

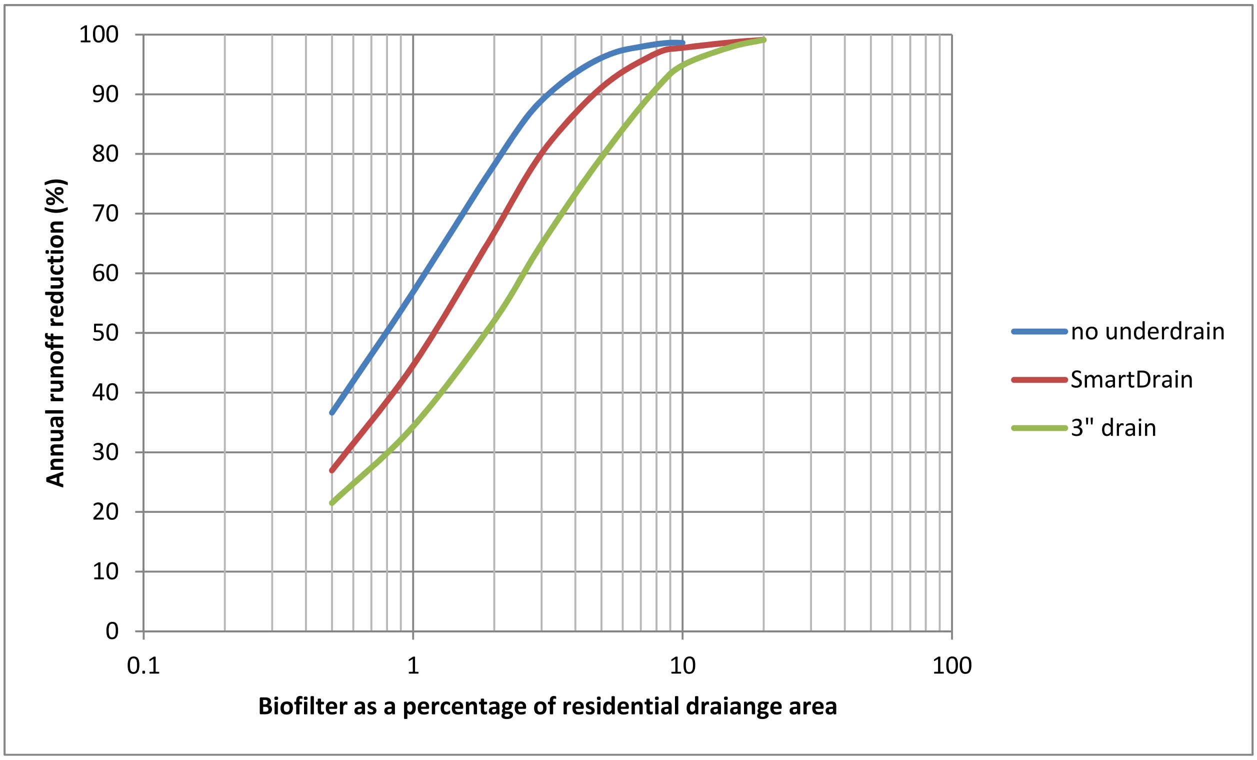


Figure 119. Use of underdrains in soils having 0.2 in/hr native subsurface infiltration rates.

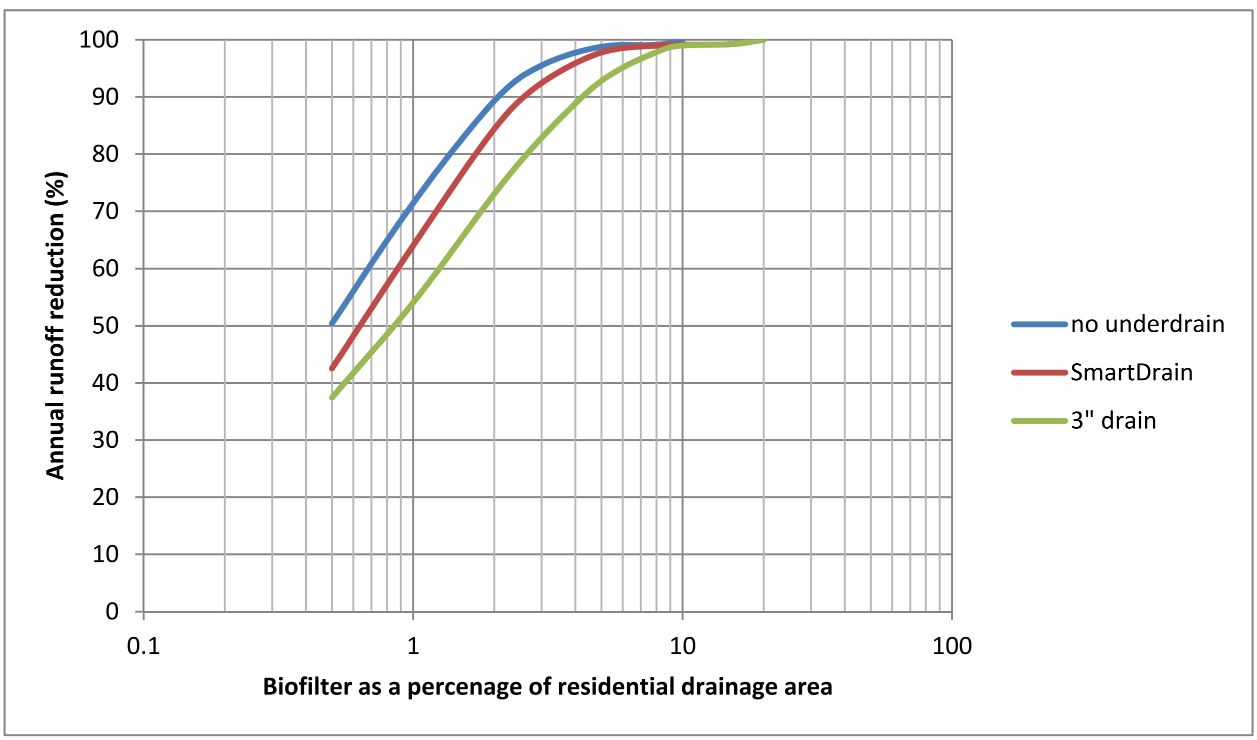


Figure 120. Use of underdrains in soils having 0.5 in/hr native subsurface infiltration rates.

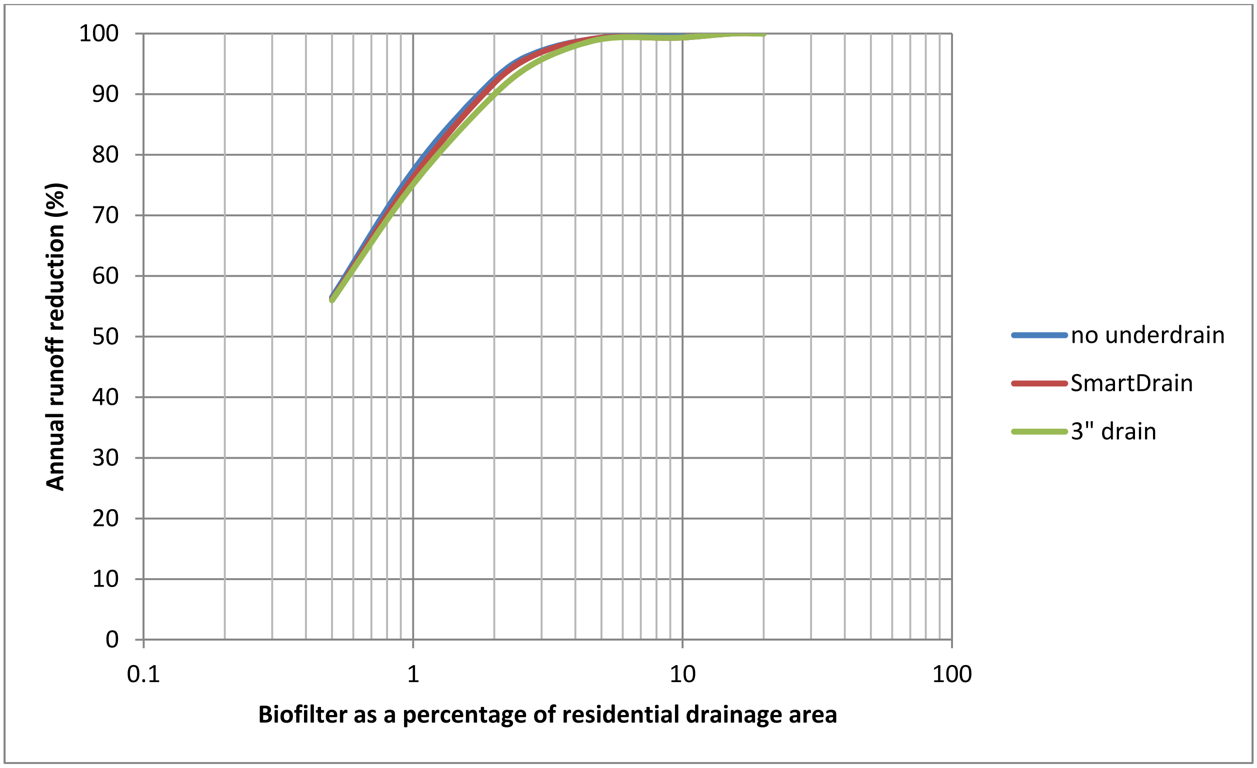


Figure 121. Use of underdrains in soils having 1.0 in/hr native subsurface infiltration rates.

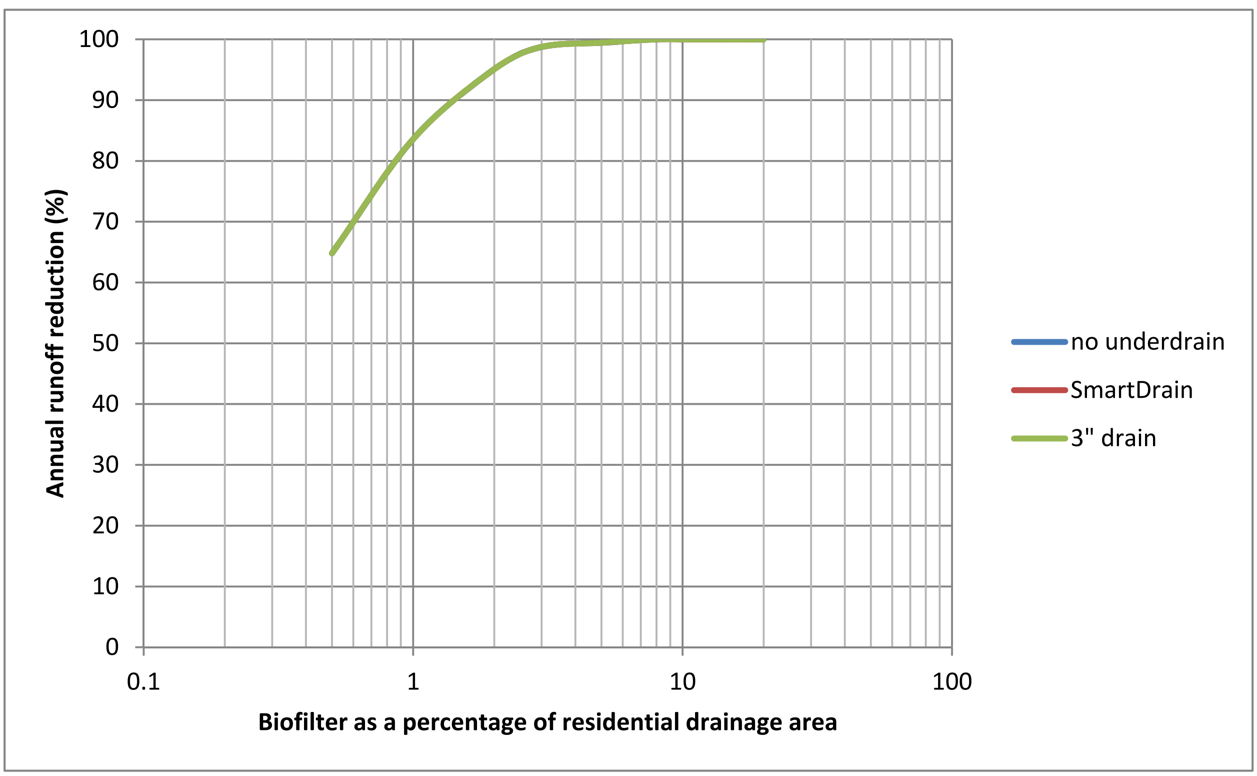


Figure 122. Use of underdrains in soils having 2.5 in/hr native subsurface infiltration rates.

Figure 123 is a plot indicating the clogging potential for the biofilters. Biofilter media is likely to fail resulting in very low infiltration rates with rapid and excessive particulate solids loadings. Generally, particulate loads of between 10 and 25 kg/m^2 could be indicative of significantly reduced infiltration. With a planted biofilter in good condition, and if this accumulative load occurs over at least 10 years, the biofilter is likely to be able to incorporate this additional material into the soil, and the plants can help retain the infiltration rate at a desired level (but with reduced surface storage volume). However, if this load occurs within just a few years, it is likely to overwhelm the system, resulting in premature clogging. This is more of a problem for small biofilters receiving runoff having high particulate solids concentrations, such as parking lots where space is limited for larger biofilters. Pretreatment using grass filters or swales can reduce these problems. For this study area, if the biofilters are at least 1 to 3% of the residential drainage area, the particulate loading is not likely to be a problem.

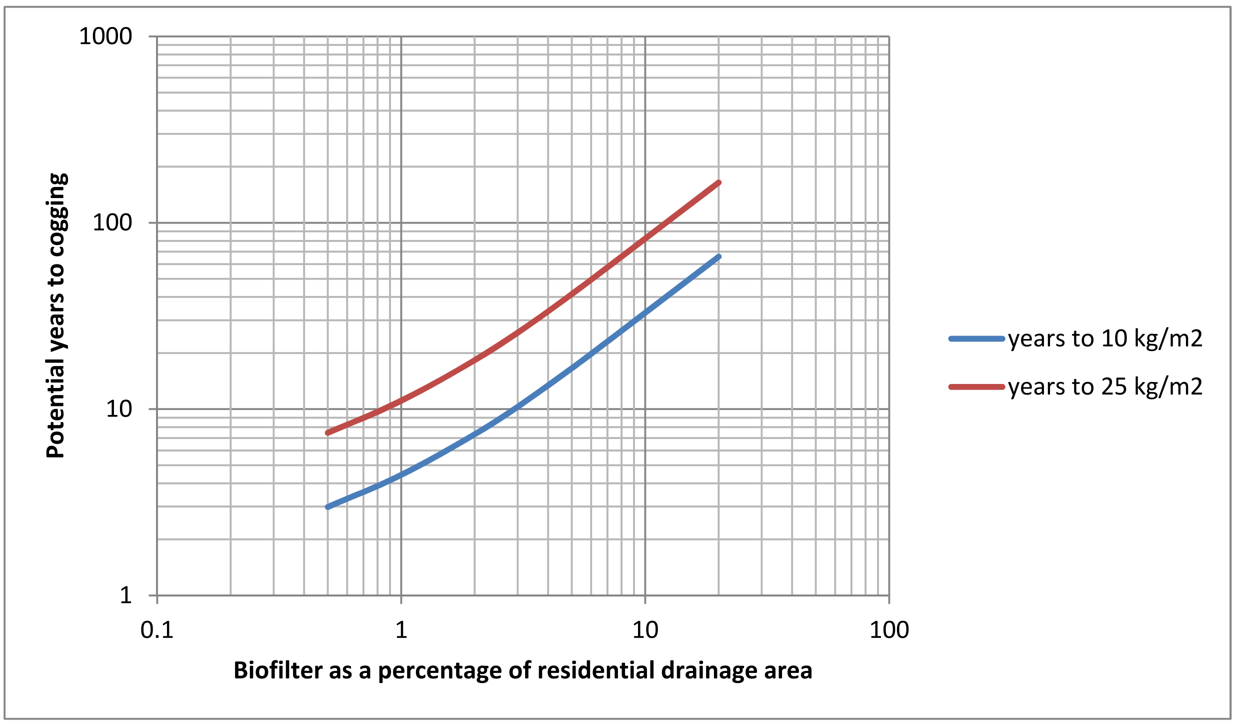


Figure 123. Clogging potential for biofilters in test (pilot) area.

Table 73 summarizes some of the features shown in Appendix E and from the above plots. Performance levels of 75, 90, and 95% reductions of surface runoff are indicated for the four infiltration rates and four underdrain options. The biofilters and bioinfiltration devices in the test (pilot) area are about 1.5 to 2% of the residential drainage areas. For the 1 in/hr subsurface infiltration rate, these sizes of the biofilters are expected to provide about a 90% reduction in the annual flows for the areas treated, with very little overflows. The SmartDrain™ installation is expected to have <10% of the annual flows being captured by this underdrain. These calculated conditions are all similar to the observed conditions during the brief monitoring period.

Table 73. Summary of performance of biofilter size, use of underdrains, and subsurface soil infiltration rates on desired performance objectives

		No underdrain, no gravel storage				No underdrain, with gravel storage			
Native soil subsurface infiltration rate (in/hr)	Annual flow removal goal (%)	Size as a % of residential drainage area	Days ponding ≥ 3 days per 4 yrs	% of flows with extended retention time (4 hrs)	Potential years to clogging (10 to 25 kg/m^2)	Size as a % of residential drainage area	Days ponding ≥ 3 days per 4 yrs	% of flows with extended retention time (4 hrs)	Potential years to clogging (10 to 25 kg/m^2)
0.2	75%	2.4%	12	0%	10 to 25	1.5%	10	0%	7 to 14
	90%	4.3%	6	0%	15 to 35	3%	5	0%	12 to 30
	95%	6%	2	0%	20 to 50	4.3%	2	0%	15 to 35
0.5	75%	1.6%	1	0%	6 to 16	1.2%	1	0%	5 to 12
	90%	2.7%	1	0%	9 to 25	2%	1	0%	8 to 19
	95%	3.6%	1	0%	14 to 35	3%	1	0%	10 to 25
1.0	75%	0.9%	1	0%	4 to 11	0.9%	1	0%	4 to 11
	90%	1.7%	1	0%	7 to 17	1.7%	1	0%	7 to 17
	95%	2.3%	1	0%	10 to 25	2.3%	1	0%	10 to 25
2.5	75%	0.7%	1	0%	3 to8	0.7%	1	0%	3 to8
	90%	1.3%	1	0%	5 to 15	1.3%	1	0%	5 to 15
	95%	2%	1	0%	7 to 17	2%	1	0%	7 to 17

Table 73. Summary of performance of biofilter size, use of underdrains, and subsurface soil infiltration rates on desired performance objectives (cont.)

		SmartDrain				Conventional 3-in. drain pipe			
Native soil subsurface infiltration rate (in/hr)	Annual flow removal goal (%)	Size as a % of residential drainage area	Days ponding ≥ 3 days per 4 yrs	% of flows with extended retention time (4 hrs)	Potential years to clogging (10 to 25 kg/m2)	Size as a % of residential drainage area	Days ponding ≥ 3 days per 4 yrs	% of flows with extended retention time (4 hrs)	Potential years to clogging (10 to 25 kg/m2)
0.2	75%	2.6%	0	10%	9 to 22	4.2%	0	25%	13 to 32
	90%	4.5%	0	6%	15 to 40	7.8%	0	7%	26 to 66
	95%	6.5%	0	2%	25 to 60	10%	0	4%	35 to 85
0.5	75%	1.4%	1	7%	6 to 14	2.2%	1	17%	8 to 20
	90%	2.4%	1	4%	9 to 22	4.2%	1	10%	15 to 35
	95%	3%	1	3%	15 to 35	6%	1	5%	25 to 60
1.0	75%	0.8%	1	1%	4 to 9	0.8%	1	2%	4 to 9
	90%	2%	1	1%	7 to16	2%	1	3%	7 to16
	95%	2.3%	1	0.5%	9 to 24	2.3%	1	2%	9 to 24
2.5	75%	0.6%	1	0%	3 to 7	0.6%	1	0%	3 to 7
	90%	1.4%	1	0%	4 to 12	1.4%	1	0%	4 to 12
	95%	2%	1	0%	7to17	2%	1	0%	7 to17

Porous Pavement

The WinSLAMM porous pavement control in version 10 has full routing calculations associated with subsurface pond storage, and it allows runon from adjacent paved areas that do not have porous pavement. The *outlet* options for porous pavements include subgrade seepage and an optional underdrain, which is modeled as an orifice. The porous pavement control device has a surface seepage rate that limits the amount of runoff that can enter the storage system. The seepage rate is usually much larger than the rain intensity, so this would be unusual, except if it is significantly reduced by clogging or if substantial runon occurs from adjacent paved areas. This surface seepage rate is reduced to account for clogging with time, while the surface seepage rate can be partially restored with cleaning at a stated cleaning frequency. The runoff volume reaching the porous pavement surface is equal to the rainfall volume directly falling on the porous pavement, plus runoff volume from any runon from the adjacent paved areas. The porous pavement surface can be paver blocks, porous concrete, porous asphalt, or any other porous surface, including reinforced turf. Porous pavements are usually installed over a subsurface storage layer that can dramatically increase the infiltration performance of the device, while reinforced turf does not have subsurface storage.

Porous pavements are typically used at paved parking and storage areas, paved playgrounds, paved driveways, or paved walkways. They should be used in relatively clean areas (walkways or driveways or other surfaces that receive little traffic, for example), to minimize groundwater contamination potential and premature clogging and failure. Porous pavements direct the infiltrating water to subsurface soil layers, usually at a depth where the soils have little organic matter that tend to sorb pollutants. Salts used for ice control in northern areas are also problematic when considering infiltrating stormwater. Consider biofiltration devices to infiltrate water from more contaminated sites because they can use amended soils to help trap contaminants before infiltration, or use other appropriate pre-treatment before infiltration, and are easier to restore. No common pretreatment device is suitable for removing salts, however, so minimal use of deicing chemicals is the preferred control option.

It is necessary to describe the geometry and other characteristics of a typical porous pavement surface, as shown in Figure 124. The model computes the runoff volume, equal to the rainfall volume plus any runon, and then creates a complex triangular hydrograph (the flow duration equals the rain duration) that it routes through that porous pavement system.

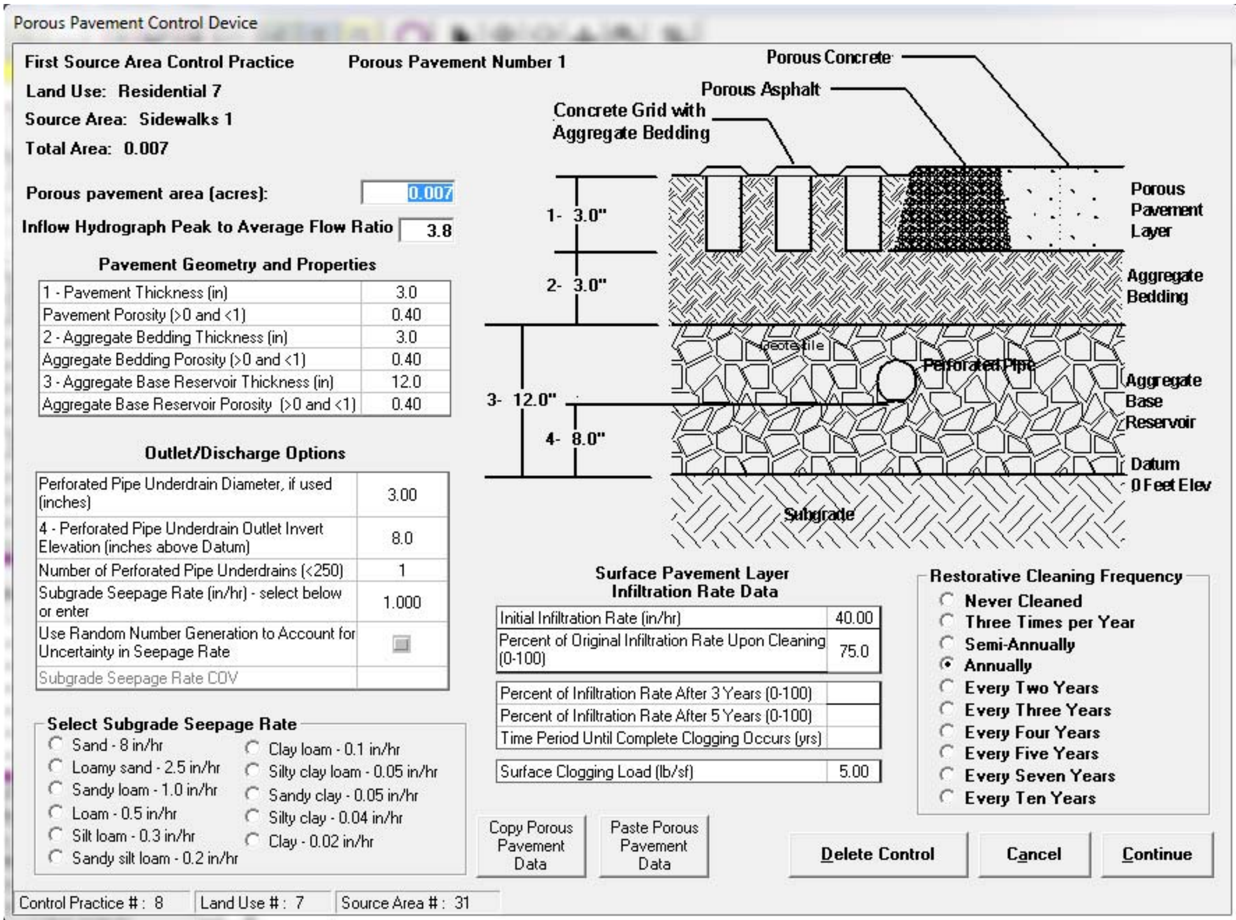


Figure 124. Porous pavement main input screen.

Table 74 summarizes the calculated performance of porous pavements located at paved parking/storage areas. The given underlying soil is a loam soil. A conventional 3-in. perforated pipe underdrain was also used. As indicated, even the smallest area examined (25% of the area as porous pavement) had very good runoff volume reductions for this example. The porous pavement was cleaned every year, restoring much of the lost surface infiltration rate capacity in this example. If the area was not cleaned, clogging would be expected in about 8 years, based on field experience.

Table 74. Porous pavement performance (paved parking and storage area; loam soil; 3-in underdrains placed 20 ft apart)

Porous pvt as a % of paved parking area	Rv	Volume reduction (%)	Expected habitat conditions	TSS (mg/L)	Solids discharged (lbs/yr)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.75	n/a	Poor	130	812	0.21	13	21	1.3
25%	0.06	92%	Good	130	60	0.21	0.98	21	0.098
50%	0.05	93%	Good	130	58	0.32	0.94	12	0.093
100%	0.05	93%	Good	130	58	0.21	0.94	21	0.093

Grass Filters

Grass filters have broad, shallow flows. WinSLAMM calculations for grass filters are based on extensive pilot-scale and field measurements of grass swales and filters conducted for the Alabama Department of Transportation. This model determines the flow conditions for every calculation increment, including flow velocity and depth. Special shallow Manning's n values are used according to shallow sheetflow measurements. Sediment transport is calculated for each narrow particle size range using their sedimentation rate, depth of flow, and length of flow. Scour is also considered, along with equilibrium concentrations. The pilot-scale tests were confirmed during full-scale tests during actual rains.

The grass filter and grass swale controls calculate pollutant and runoff volume reductions. The model determines the runoff volume reduction by calculating the infiltration loss for each time step. The particulate reduction is based on the settling frequency of the particles entering the grassed area and the height of the grass relative to the flow depth. The grass "filters" the runoff using the settling frequency and the length of the flow path. The algorithms used to determine the Manning's n values were developed from the master's thesis by Jason Kirby Kirby, et al. 2005) as part of a WERF-supported research project (Johnson, et al. 2003). The particle trapping algorithms were based on the master's thesis research conducted by Yukio Nara (Nara, et al. 2006), supported by the University Transportation Center for Alabama (Nara and Pitt 2005).

Runoff volume is reduced by the dynamic infiltration rate of the swales for each 6-minute time step of the hydrograph. The flow and the geometry are used to determine Manning's n to iteratively determine the depth of flow in the swale for each time step, using traditional VR-n curves that were extended by Kirby (Kirby, et al. 2005) to address the smaller flows found in roadside grass swales and filters. Using the calculated depth of flow for each time increment, the model calculates the wetted perimeter (using the swale cross-sectional shape), which is then multiplied by the total flow length to determine the area used to infiltrate the runoff. Details for these calculations are available by selecting the "Hydraulics Detailed Output File" checkbox from the "Detailed Output Options" listing under "Program Options." The event-by-event summary detailed output is available by selecting the "Hydraulics and Concentration by Event" checkbox from the Detailed Output Options listing. These comma-separated tabular files are created when the model is executed and can be reviewed using a spreadsheet after importing the files.

Figure 125 is the WinSLAMM basic input screen used for grass filters. Table 75 summarizes the performance of the grass filters for controlling the runoff from 2 acres of impervious areas. As the grass filters become steep, they lose some of their performance because of the faster flowing has a greater equilibrium capacity associated with its carry capacity and the faster flowing water has reduced effective infiltration rates compared to ponded water. Version 10 uses a direct calculation of the hydraulics for grass filter strips as for grass swales, but with modified turbulent induced length restrictions. An upcoming model release will use Muskingum channel routing to more effectively calculate the flowing water conditions in the grass filters (and swales).

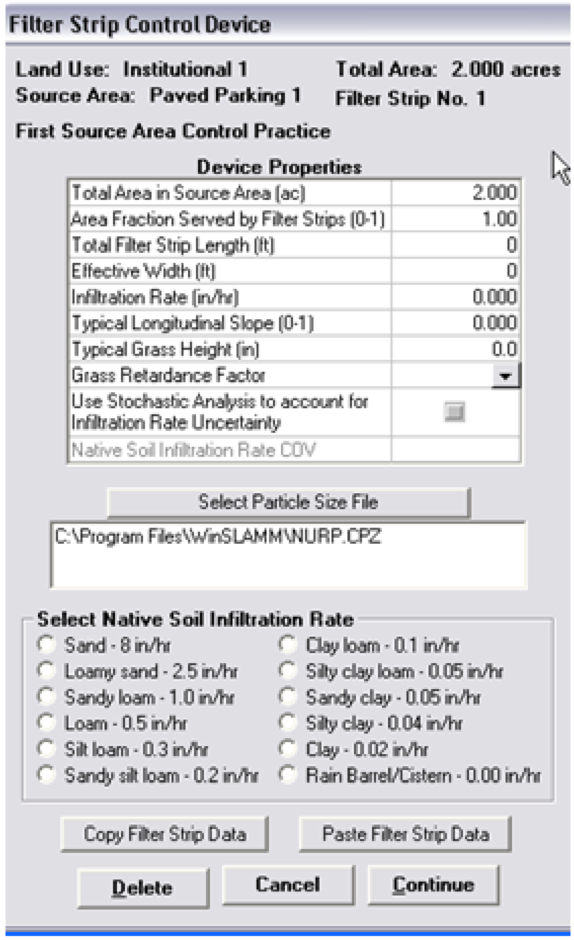


Figure 125. Grass filter strip form in Version 10.

Table 75. Grass filter performance for different soils and slopes

Description	Rv	% runoff volume reduction	TSS (mg/L)	Solids yield (lbs/yr)	% solids yield reduction	Peak runoff rate (cfs)	% peak runoff rate reduction	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
Base conditions, no controls	0.55		100	1,040		4.6		0.28	29	17	1.7
Grass filter 0.5% slope	0.17	69%	91	300	71%	2.6	43%	0.27	8.7	16	0.52
Grass filter 2 to 25% slopes	0.22	60%	90	376	64%	3.5	24%	0.26	11	16	0.67

Grass Swales

Grass swales are evaluated using the same general process as described previously for grass filters. As summarized, these procedures are based on extensive laboratory and field tests and calculate swale performance through infiltration mechanisms and sedimentation of many discrete particles sizes. The data entry form is shown in Figure

126. Table 76 summarizes the performance of a swale for two soil conditions. As expected, the swale water volume and pollutant reduction performance is better for the loam soil than for the silty soil.

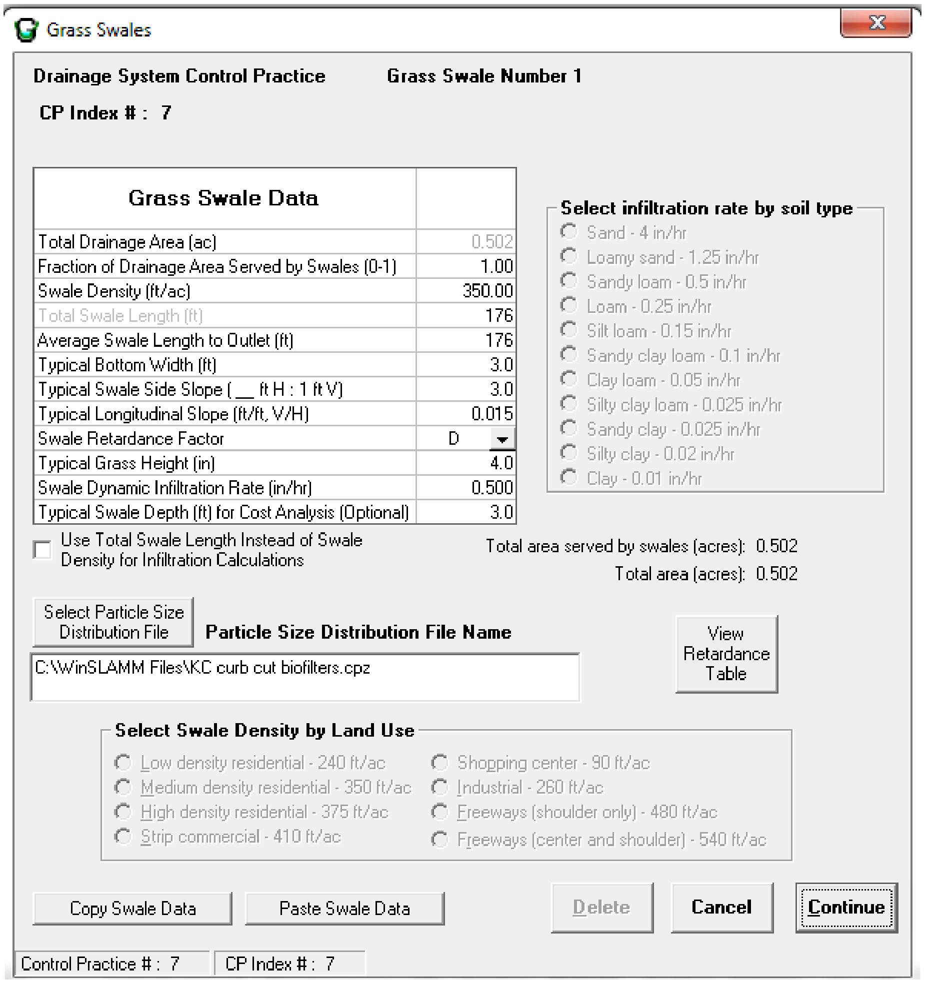


Figure 126. Grass swale input screen.

Table 76. Grass swale performance

Descscription	Rv	% runoff volume reduc.	Expected habitat conditions	TSS (mg/L)	solids yield (lbs/yr)	% solids yield reduc.	peak runoff rate (cfs)	% peak runoff rate reduc.	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
base conditions, no controls	0.55		Poor	100	1,040		4.6		0.28	29	17	1.7
silty soil	0.33	40%	Poor	86	535	92%	4.4	4%	0.25	16	16	0.98
loam soil	0.16	71%	Fair	87	263	92%	2.9	37%	0.26	7.8	16	0.47

Cisterns and Water Storage Tanks

This section describes a method to evaluate or size water storage tanks needed to optimize the beneficial uses of stormwater. Much of this material was previously summarized in the recent WERF report on nonpotable beneficial uses of stormwater (Pitt, et al. 2011b). Irrigation of land on the homeowner's property was considered the beneficial use of most interest. Production function curves were prepared showing the relationship between water tank size and roof runoff beneficial use for the Kansas City study area.

Calculating the Benefits of Rainwater Harvesting Systems

Benefits associated with stormwater use for irrigation and other on-site uses can be calculated using site-specific information. Specifically, source area characteristics describing where the flows will originate and how the water will be used, are needed. In the most direct case, this information is used in conjunction with the local rainfall information and storage tank sizes to determine how much of the water needs can be satisfied with the stormwater, and how the stormwater discharges can be reduced. The following section describes how WinSLAMM can be used to calculate production functions that can be used to size storage water tanks to maximize irrigation use for residential locations in Kansas City, Missouri.

Regional Rainfall and Runoff Distributions Affecting Roof Runoff Harvesting

The model can use any length of rainfall record as determined by the user, from single rainfall events to several decades of rains, depending on the complexity of the study area and the available computer memory. The rainfall file used in these calculations for Kansas City were developed from hourly data obtained from EarthInfo CD ROMs, using the 27 years from 1972 through 1999, as shown in Figure 127. This period contains 2,537 rains, with an average depth of 0.40 in. and a maximum of 6.19 in.

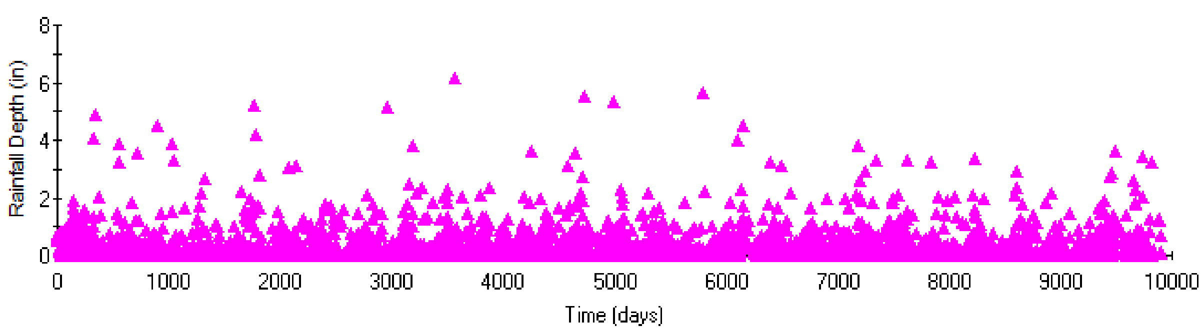


Figure 127. Long-term rain depths for individual Kansas City, Missouri, rains (1972–1999).

Figure 128 shows that the regional stormwater runoff is heavily influenced by the small to intermediate rains (data for the region shown for St. Louis, Missouri). Almost all of the runoff is associated with rains between 0.3 to 2 in., the events for which WinSLAMM is optimized. The rare drainage design events generally comprise a very small portion of the typical year's runoff. The 1.4-in. event used in Kansas City for the original sizing of distributed storage systems is close to the rain depth associated with the median runoff depth.

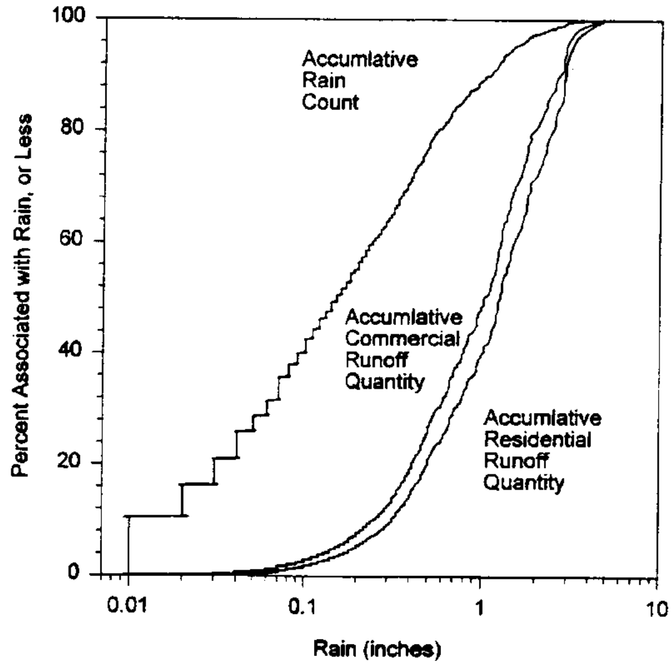


Figure 128. St. Louis, Missouri, rain and runoff distributions (1984–1992 rains).

The land development characteristics and the evaluation of flow and pollutant sources in the area determine the maximum effectiveness of different types of controls. The land survey found that most of the homes in the test watershed already have disconnected roofs (85% of all roof areas) and that the total roof areas compose about 13% of the total area. The land survey also found that about 65% of the area is landscaped, with most being in turf grass in poor to good condition. This information was used in conjunction with regional ET data to calculate the amount of supplemental irrigation needed to meet the ET requirements of typical turf grass, considering the long-term rainfall patterns. Most of the supplemental irrigation would be needed in July and August, whereas excess rainfall occurs in October through December (compared to ET requirements during these relatively dormant months). Soil infiltration monitoring in the area, along with soil profile surveys, has indicated relatively poorly draining soil in the test area for the larger rains. Surface infiltration rates during several-hour rains can have infiltration rates of about 1 in/hr or greater, but these rates continue to decrease with increasing rain depths. For conservative modeling calculations, a soil infiltration rate of 0.2 in/hr was used.

The expected major sources of runoff from the test area vary for different rain depth categories. Directly connected impervious areas are the major runoff sources only for rains that are less than about 0.25 in. The large landscaped areas contribute about half of the runoff for rains larger than about 0.5 in. The directly connected roofs, which make up only about 2% of the study area, contribute about 6% of the total annual flows. The disconnected roofs, which compose about 11 percent of the area, contribute about 7% of the total flows. If all roofs were directly connected, they would compose about 31 percent of the annual total runoff flows, most of which could be eliminated through the use of cisterns/water tanks and irrigation.

Rain barrel/water cistern effectiveness is related to the need for supplemental irrigation and how that matches the rains for each season. The continuous simulations used a typical one-year rain series and average monthly ET values for varying amounts of roof runoff storage. One 35-gallon rain barrel is expected to reduce the total annual directly connected roof runoff by about 24%, if the water use could be closely regulated to match the irrigation requirements, such as with an automated irrigation system with soil moisture sensors (not likely to be used in conjunction with a few rain barrels, but more likely with a large tank than can be pressurizes). If four rain barrels were used (such as one at each corner of a house receiving runoff from separate roof downspouts), the total annual roof runoff volume reductions from the roofs could be as high as about 40%. Larger storage quantities result in increased beneficial use but likely require larger water tanks instead of large numbers of rain barrels. Water use from one water tank is also easier to control through soil moisture sensors and can be integrated with landscaping irrigation systems for almost automatic operation. A small water storage tank about 5 ft in diameter and 6 ft high is expected to result in about 75% total annual runoff reductions from directly connected roofs; a larger, 10-ft diameter and 6-ft tall tank could approach complete roof runoff control for this area. The 5-ft diameter tank is also expected to provide almost complete control of runoff from the regulatory design storm *D.* The use of rain barrels and rain gardens together at a home is more robust than using either method alone: the rain barrels would overflow into the rain gardens, so their irrigation use is not quite as critical. In order to obtain reductions of about 90% in the total annual roof runoff, it is necessary to have at least one rain garden per house, unless the number of rain barrels exceeds about 25 (or 1 small water tank) per house.

Simple disconnections of the currently directly connected roofs can provide significant reductions in the annual flows from the roofs for low cost. A reduction of about 80% is expected in the total flows with disconnections, even with the site’s clayey soils, with most occurring during small rains, and the benefits decreasing as the rains increase in depth. This flow volume reduction is enhanced because of the relatively small roof areas and large landscaped areas, which provide long flow paths. With steep slopes and poor grass, this reduction will be less.

Caution is needed when comparing the amount of site runoff storage provided by these upland controls to the total storage goals to meet the objectives of the CSO control program (288,000 gallons storage required). For example, storage provided at directly connected roofs need to be discounted by factor of 1.3 to 1.4 because not all of the storage is available during all rains, and their drainage is controlled by low infiltration rates through the native soils, compared to flow controls directly connected to the combined sewers. In contrast, curb-cut biofilters have access to almost all the flows in the area, so their storage volumes are more effectively used. More significantly, if storage was provided at roofs that are already disconnected, their storage volumes would need to be discounted by about 4.5 times when compared to the total site storage goals because of the existing infiltration occurring with the disconnected roof runoff.

Water Harvesting Potential in Kansas City

The water harvesting potential for water tank use was calculated on the basis of supplemental irrigation requirements for the basic landscaped areas. The irrigation needs were determined to be the amount of water needed to satisfy the ET requirements of typical turf grasses, after the normal rainfall (a conservative calculation because only a portion of the rainfall contributes to soil moisture).

Table 77 summarizes the monthly average rainfall for the 1973 through 1999 period at the Kansas City airport, a 26-year continuous rain record. The average total annual rainfall is typically about 37.5 in., with most falling in the spring to early fall. A much smaller fraction of the annual rain occurs in December through February.

Table 77. 1973 through 1999 Kansas City Airport monthly rain depth totals (inches)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Average	1.13	1.24	2.54	3.48	5.41	4.27	4.15	3.63	4.63	3.32	2.08	1.60	37.49
COV	0.68	0.57	0.66	0.61	0.54	0.48	0.85	0.67	0.75	0.81	0.59	0.83	0.25
Minimum	0.02	0.20	0.32	0.34	1.18	1.73	0.25	0.65	0.57	0.00	0.00	0.00	21.60
Maximum	2.81	2.72	9.08	8.43	12.41	8.67	15.47	9.58	11.11	10.16	5.12	5.42	55.26

The total landscaped area in the 100-acre residential land use area is 65.1 acres, and with 576 homes, each has about 4,925 ft^2 of landscaped area that might be irrigated. Tables 78 and 79 along with Figures 129 through 131 show the monthly ET requirements of typical turf grasses for a monitoring station near Kansas City (Ottawa, Kansas, at a University of Kansas field station). The total annual ET is about 52 in. a year, and the annual total rainfall is about 37 in. a year, resulting in a rainfall deficit of about 15 in. per year.

Table 78. Monthly irrigation requirements

	In/day ET*	ET (in/month)	Rainfall (in/month)	Irrigation deficit (in/month)	Irrigation deficit (gal/day/house)
Jan	0.05	1.55	1.13	0.42	42
Feb	0.10	2.83	1.24	1.59	172
Mar	0.10	3.10	2.54	0.56	55
Apr	0.15	4.50	3.48	1.02	104
May	0.20	6.20	5.41	0.79	78
Jun	0.20	6.00	4.27	1.73	177
Jul	0.25	7.75	4.15	3.60	357
Aug	0.25	7.75	3.63	4.12	408
Sep	0.20	6.00	4.63	1.37	140
Oct	0.10	3.10	3.32	excess rain	0
Nov	0.05	1.50	2.08	excess rain	0
Dec	0.05	1.55	1.60	excess rain	0

* These ET values are for eastern Kansas (Ottawa, Kansas) and are for typical turf grasses.

Table 79. Monthly irrigation per household

Month	Irrigation needs per month (gal/house)	Irrigation needs per month (ft^3/house)	Irrigation needs per month (ft depth/house)	Supplemental irrigation needs per month (inches depth/month)	Supplemental irrigation needs per month (inches depth/week)
Jan	1,302	174	0.04	0.42	0.10
Feb	4,859	650	0.13	1.58	0.39
Mar	1,705	228	0.05	0.56	0.13
Apr	3,120	417	0.08	1.02	0.24
May	2,418	323	0.07	0.79	0.18
Jun	5,310	710	0.14	1.73	0.40
Jul	11,067	1,480	0.30	3.60	0.81
Aug	12,648	1,691	0.34	4.12	0.93
Sep	4,200	561	0.11	1.37	0.32
Oct	0	0	0.00	0.00	0.00
Nov	0	0	0.00	0.00	0.00
Dec	0	0	0.00	0.00	0.00
Totals:	46,629	6,234	1.27	15.19	

Figures 129 through 131 plot the monthly ET, rainfall, and supplemental irrigation needs. Most of the supplemental irrigation is needed in July and August, whereas there is an excess of rainfall in October through December, and therefore no supplemental irrigation is needed in those months.

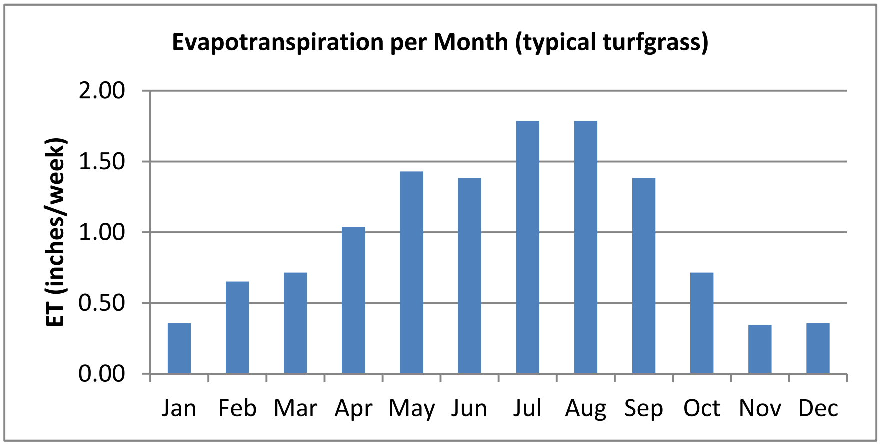


Figure 129. ET by month.

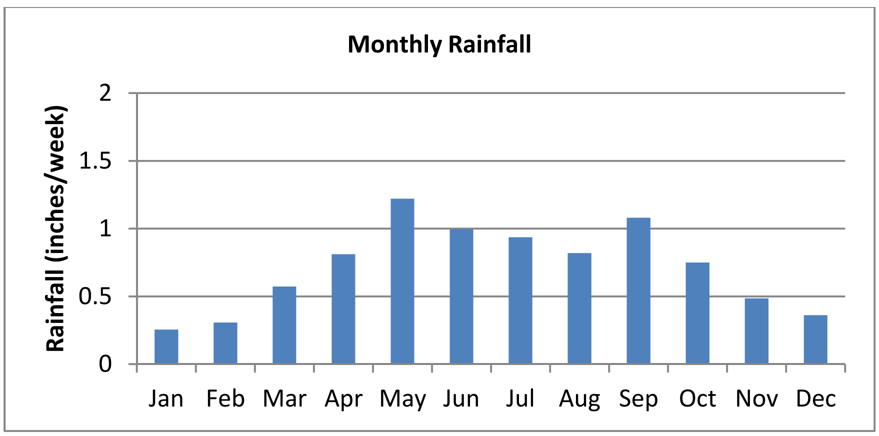


Figure 130. Monthly rainfall.

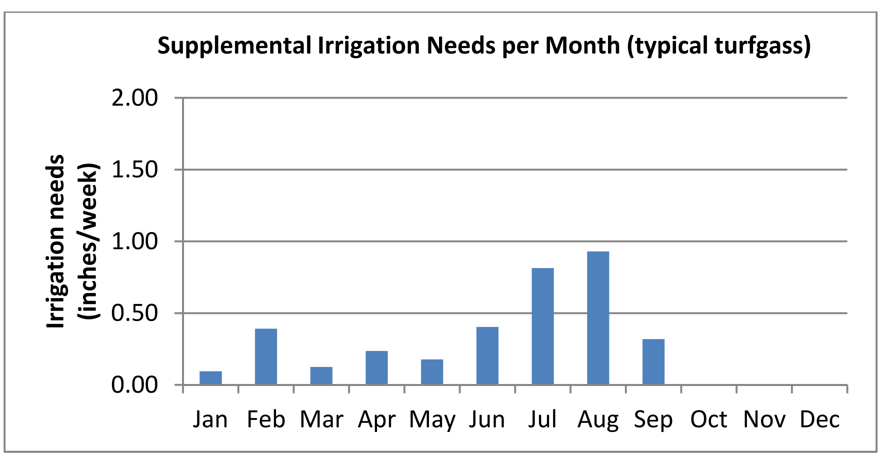


Figure 131. Monthly supplemental irrigation requirements to meet ET.

The total amount of rainfall harvesting potential for irrigation (to match the ET) is about 46,600 gallons (6,230 ft^3) per household per year. With 4,925 ft^2 of landscaped area per household, the annual irrigation requirement is about 1.3 ft, or 15 in., or an average of about one-half inch of water applied per week during the 9 months when there is an irrigation need. With 576 homes in the watershed, this totals about 27 million gallons (3.6 million ft^3) per year for the 100-acre project area. Continuous simulations are used to see how much of this can actually be used according to the interevent conditions and rain patterns compared to the water need patterns and water storage volumes. It is also possible to use a greater amount of this water for irrigation for certain plants. These irrigation values are for typical turf grasses. Any additional irrigation would not be used by the plants but would be infiltrated into the soil. As noted, the long-term infiltration rates available through the soils at the project site are low.

Rain Barrels and Water Tanks for Roof Runoff Harvesting

Rain barrels are a very simple method for collecting roof runoff for beneficial uses. In these analyses, irrigation of typical turf grass landscaping around the homes in the study area is the use being examined. This irrigation requirement was described previously and is the additional water needed to supplement the long-term monthly average rainfall to match the ET requirements for the area. As shown in these analyses, small rain barrels provide limited direct benefits, so larger water tanks were also considered. Also, to be most beneficial, these calculations assume that the irrigation rates are controlled by soil moisture conditions to match the ET requirements closely. This level of control is usually most effectively achieved with one large storage tank connected to an automatic irrigation system. Numerous smaller rain barrels are more difficult to optimally control.

For these calculations, each rain barrel is assumed to have 35 gallons of storage capacity (4.7 ft^3). Each roof has an average area of 945 ft^2 and receives a total of 3,100 ft^3 of rainfall. As noted above, these analyses are for the directly connected roofs in the area, which are only about 15% of the total roof area in the study watershed.

Figure 132 is the input screen used for rain barrels or cisterns in WinSLAMM version 10. The only discharges is the required monthly water use requirements (such as irrigation demands).

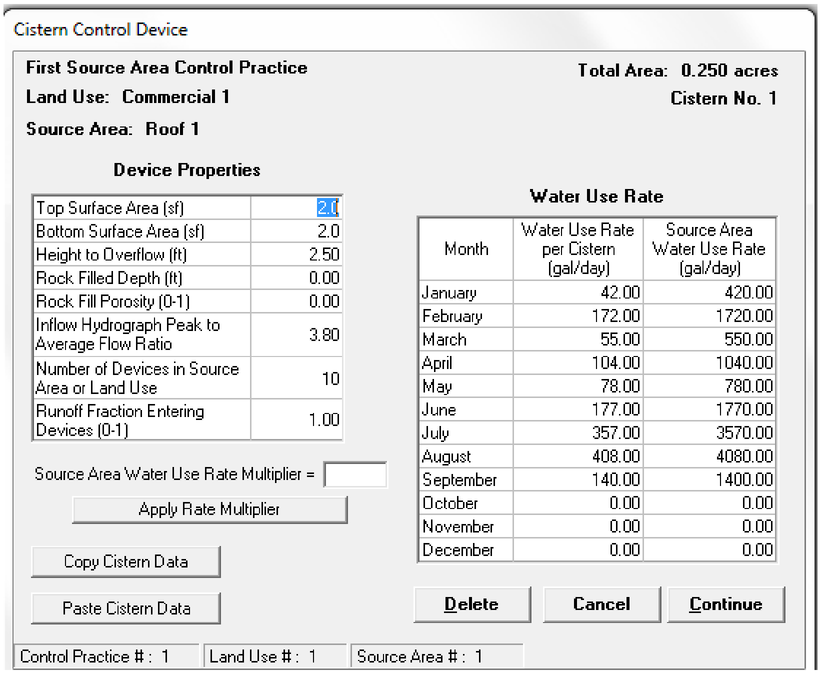


Figure 132. Cistern/water tank WinSLAMM input screen.

Tables 80 and 81 and Figure 133 summarize the benefits of storage and irrigation use of runoff collected from directly connected roofs. The use of one rain barrel is expected to provide about a 24% reduction in roof runoff through irrigation to match ET. To match the benefits of disconnection of connected downspouts (about 78% reductions), about 25 rain barrels would be needed. Twenty-five rain barrels correspond to a total storage quantity about equal to 0.12 ft (1.4 in.). The level of maximum performance for roof runoff storage in Kansas City is relatively high compared to other US locations because the excess rainfall occurs during times of the greatest ET needs (with some winter months not having ET needs). More importantly, the landscaped areas that can be irrigated are relatively large when compared to the small roof areas. Together, these result in substantial maximum potential benefits associated with irrigation beneficial uses.

Table 80. Roof runoff storage needs for beneficial use objectives

# of rain 35 gal. barrels per house	Rain barrel storage per house (ft^3)	Rain barrel storage per house (ft^3) per roof area (ft^2, or ft depth over the roof)	Total annual roof runoff for 86 houses (ft^3)	Total annual roof runoff per house (ft^3)	Rv for roof area	% reduction in roof runoff
0	0	0	257,200	2,990	0.97	0
1	4.7	0.0050	196,700	2,290	0.74	24
4	19	0.020	155,800	1,810	0.58	39
10	47	0.050	112,400	1,310	0.42	56
100	470	0.50	3,160	37	0.01	99

1 ft^3 = 28 liters

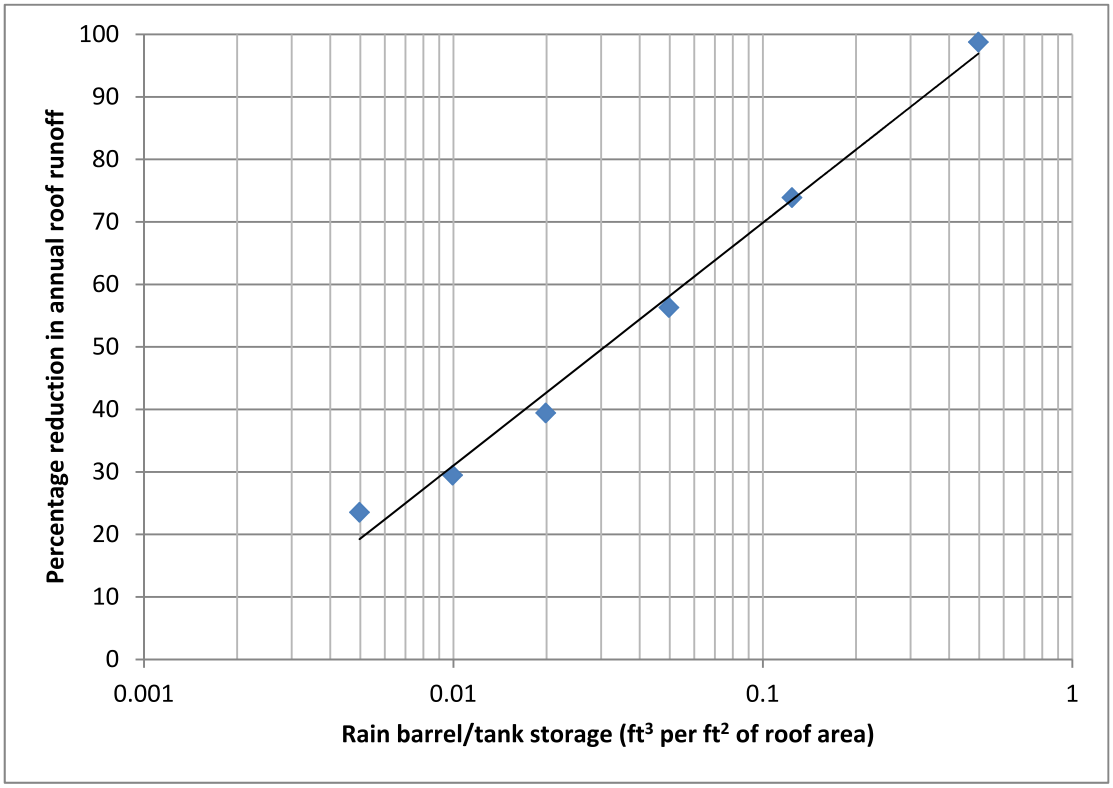


Figure 133. Irrigation storage requirements production function.

As the storage volume increases, it likely becomes impractical to meet the total storage volume with small rain barrels. Table 81 shows the equivalent size of larger water tanks or cisterns when the number of rain barrels is more than four. As an example, a moderately sized water tank 5 ft in diameter and 6 ft tall has a similar storage capacity as 25 rain barrels, and if the 6-ft tall tank was expanded to 10 ft in diameter, this larger tank would have a similar capacity as 100 rain barrels.

The use of about 25 rain barrels, or a small tank 5 ft in diameter and 6 ft tall, is the recommended amount of storage for the directly connected roofs in the study area. This would provide about 74% reductions in the total annual runoff discharges, and almost complete control for the 1.4-in. regulatory design storm *D.*

Table 81. Rain barrels and water tank equivalents

Storage per house (ft depth over the roof)	Storage per house having 945 ft^2 roof area (ft^3 and gallons)	Reduction in roof runoff for 1.4-in. rain (%)	Reduction in annual roof runoff (%)	# of 35-gallon rain barrels	Tank height size required if 5 ft diameter (ft)	Tank height size required if 10 ft diameter (ft)
0	0 (0)	0	0	0	0	0
0.0050	4.7 (35)	16	24	1*	0.24	0.060
0.010	9.4 (70)	19	29	2	0.45	0.12
0.020	19 (140)	27	39	4	0.96	0.24
0.050	47 (350)	46	56	10	2.4	0.60
0.12	118 (880)	96	74	25	6.0	1.5
0.50	470 (3,500)	100	99	100	24	6.0

*the yellow high-lighted cells are the most reasonable alternatives for these performance levels

Example Alternative Irrigation Water Use Calculations

Tables 82 and 83 and Figures 134 and 135 are calculated supplemental irrigation requirements for residential areas in Millburn, New Jersey, an area having very challenging conditions for using stormwater to match local ET requirements (Pitt and Talebi 2012). These areas have roofs that are about 325 m^2 in area (3,500 ft^2) corresponding to about 13.5% of the land use, and landscaped areas about 1,440 m^2 (15,500 ft^2) corresponding to about 61% of the land use, with a relatively high roof to landscaped area ratio of about 0.23 (large homes and small lots). Table 82 and Figure 134 show the irrigation needs that can be considered the minimum amount by barely meeting the landscaped area ET requirements (assuming all rainfall contributes to soil moisture, which is true for rains less than about 25 mm (1 in.) in depth, but some of the rain flows to the storm drainage system for larger rains. The monthly rainfall compared to the monthly ET is shown in Figure 134 and illustrates how supplemental irrigation would be needed in the summer months, as expected. Table 82 shows the monthly irrigation needs in gallons per day per house. This rate would be used for barely meeting the ET needs with excessive irrigation. Excessive irrigation water would result in runoff (if applied at a rate greater than the infiltration rate of the surface soils) and recharge of the shallow groundwater. For a water conservation program, this irrigation amount is usually the target. However, for a stormwater management and combined sewage control goal, maximum use of the roof runoff is desired.

Table 82. Irrigation needs to satisfy ET requirements for Essex County, New Jersey

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total Annual
Average monthly rain (in/mo)	3.42	3.11	4.16	3.71	3.99	2.88	4.21	4.04	3.61	3.06	3.70	3.47	43.37
Average monthly ET (in/mo)	0.47	0.85	3.26	3.90	4.81	4.65	4.81	4.19	3.60	3.57	3.00	1.40	38.47
deficit for ET needs (in/mo)	0.00	0.00	0.00	0.19	0.81	1.77	0.60	0.15	0.00	0.51	0.00	0.00	4.03
Deficit ET needed (gal/day/house) 0.36 acre	0	0	0	63	256	577	188	47	0	160	0	0	39,200 gal/year

Source: Pitt and Talebi 2012
(1 in/mo = 25 mm/mo)

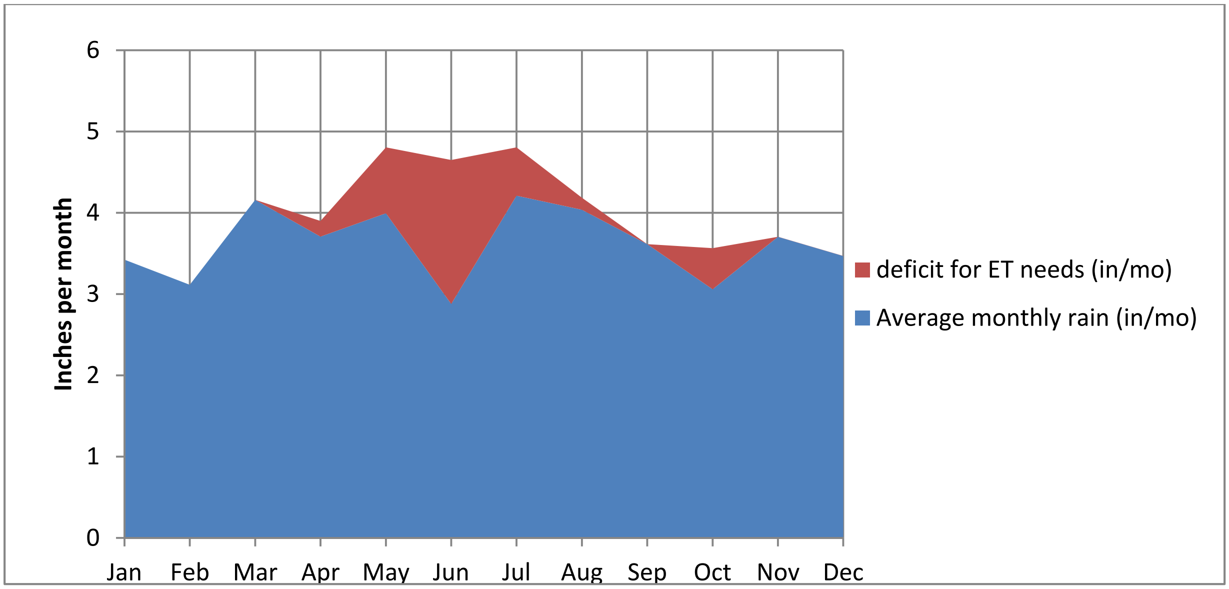


Figure 134. Plot of supplemental irrigation needs to match ET deficit for Essex County, New Jersey. (1 in/mo = 25 mm/mo).

For maximum use of the roof runoff to decrease runoff volume discharges, it is desired to irrigate at the highest rate possible, without causing harm to the plants. Therefore, Table 83 and Figure 135 show an alternative calculation corresponding to a possible maximum use of the roof runoff. For a *healthy* lawn, total water applied (including rain) is generally about 25 mm (1 in.) of water per week, or 100 mm (4 in.) per month. Excessive watering is harmful to plants, so indiscriminate over-watering is to be avoided. Some plants can accommodate additional water. As an example, Kentucky bluegrass, the most common lawn plant in the United States, needs about 64 mm/week (2.5 in/week), or more, during the heat of the summer and should also receive some moisture during the winter. Table 83 therefore calculates supplemental irrigation for 12 mm (0.5 in.) per week in the dormant season and up to 64 mm/week (2.5 in./week) in the hot months. Natural rains are expected to meet the cold season moisture requirements. The total irrigation needs for this moisture series is about 318,000 gallons (1,200 m^3) per year per home. This is about eight times the amount needed to barely satisfy the ET requirements noted before. However, the roofs in the Millburn study area are expected to produce about 90,000 gallons (340 m^3) of roof runoff per year, or less than a third of the bluegrass needs but more than twice the needs for the ET deficit. Therefore, it is possible to use runoff from other areas, besides the roofs, for supplemental irrigation.

Table 83. Irrigation needs to satisfy heavily irrigated lawn for Essex County, New Jersey

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total annual
Average monthly rain (in/mo)	3.42	3.11	4.16	3.71	3.99	2.88	4.21	4.04	3.61	3.06	3.70	3.47	43.37
Lawn moisture needs (in/mo)	2.00	2.00	4.00	4.00	8.00	8.00	10.00	10.00	10.00	8.00	4.00	2.00	72.00
Deficit irrigation need (in/mo)	0.00	0.00	0.00	0.29	4.01	5.12	5.79	5.96	6.39	4.94	0.30	0.00	32.80
Deficit irrigation needed (gallons/day/house) 0.36 acre	0	0	0	96	1,263	1,669	1,826	1,880	2,081	1,558	96	0	318,000 gal/year

Source: Pitt and Talebi 2012

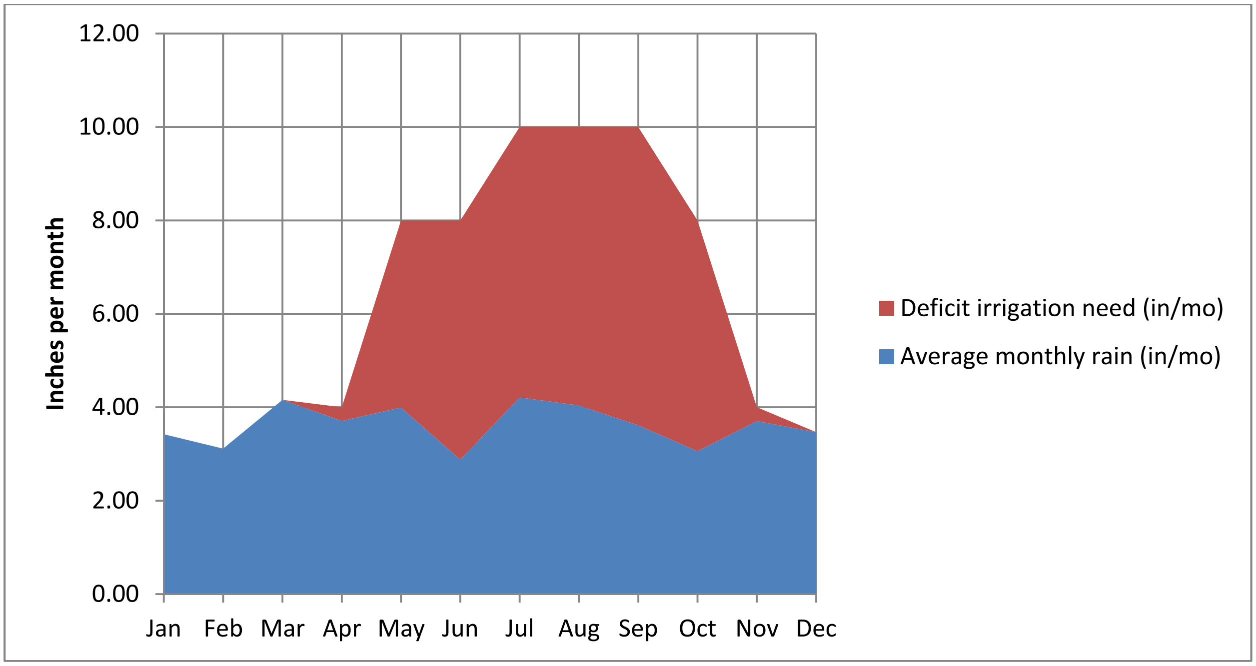


Figure 135. Plot of supplemental irrigation needs to match heavily watered lawn (0.5 to 2.5 in./week) deficit for Essex County, New Jersey (1 in/mo = 25 mm/mo).

Green Roofs

As noted above for the description of the biofilter calculations, the biofilter device can be configured to represent green roofs, as illustrated in Figure 136. In an upcoming WinSLAMM version, a separate screen will be provided for these devices. Basically, the green roof area is used as the area of the biofilter, and no natural infiltration is allowed. The only outlets include the required broad crested weir for surface overflows, underdrains, and ET. Partial roof coverage can be modeled by using a smaller area for the "biofilter" to represent the area dedicated to green roof processes.

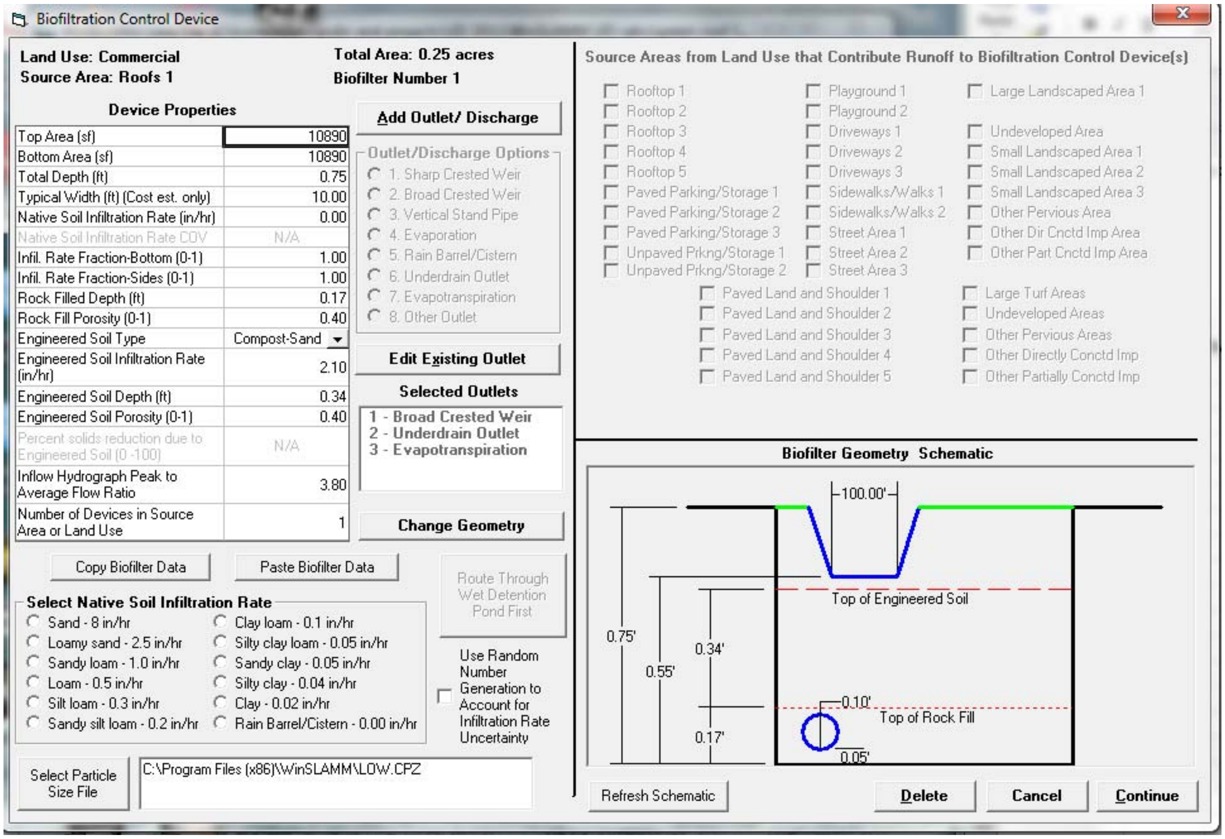


Figure 136. Green roof main input screen.

Table 84 summarizes the calculated performance of the specified green roof system, for different roof coverages. The concentrations are similar for all scenarios because almost all of the water is filtered by the roof media, with little being discharged to the surface overflows. The available ET resulted in about 25% reductions in runoff volume discharges. If more surface storage was provided in the green roof design and if more efficient plants were used, it is likely that these runoff volume reductions could be about double the reductions shown in this example.

Table 84. Calculated green roof performance

Green roof as a % of flat roof area (3-in conventional underdrains every 20 ft)	Rv	Volume reductions (%)	TSS (mg/L)	Solids discharged (lbs/yr)	Peak runoff rate (cfs)	Peak rate reductions (%)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.8	n/a	33	55	0.76	n/a	0.22	3.6	11	0.18
25%	0.71	11	24	35	0.57	25	0.17	2.4	9.8	0.14
50%	0.66	18	24	33	0.45	41	0.16	2.2	9.7	0.13
100%	0.6	25	24	29	0.38	50	0.16	2	9.7	0.12

Summary of Performance Production Functions for the Design and Analysis of Stormwater Management Controls

The first stormwater control that should be considered in an area is disconnecting the directly connected impervious areas, such as roofs and paved parking lots. The directly connected roofs in the test area contribute about 5.8% of the total area flows, whereas the much greater area of disconnected roofs contribute about 7.2% of the annual runoff from the entire 100-acre area. The current flow contributions of all roofs in the area total about 13%. If all the roofs were directly connected, the roofs would contribute about 31% of the total area runoff, and the runoff from the total area would increase by about 25%, a significant increase. In contrast, if the directly connected roofs were disconnected through a downspout disconnection program, the total roof contribution would decrease to about 9%, and the total area runoff would decrease by about 5%. Because about 85% of the roofs in the area are already disconnected, the benefits of controlling the remaining directly connected roofs are limited. Directly connected roofs in the study area contribute about 4.5 times the amount of runoff per unit area as the disconnected roofs. This indicates that about 78% of the annual runoff from the disconnected roofs is infiltrated as it passes over previous areas on the way to the drainage system. Therefore, it is much less cost-effective to use roof runoff controls for the runoff from the disconnected roofs compared to runoff controls for the directly connected roofs. The benefits of disconnecting connected paved parking or storage areas are similar to the benefits shown above for roofs.

Private rain gardens for controlling roof runoff are being used in the residential areas in the test (pilot) area. As runoff enters the device, water infiltrates through the engineered soil or media (or natural soil, as in a rain garden). If the entering rain cannot all be infiltrated through the surface layer, the water ponds. If the ponding becomes deep, it can overflow through the broad-crested weir or other surface outlet. The percolating water moves down through the device until it reaches the bottom and intercepts the native soil. If the native soil infiltration rate is greater than the percolation water rate, no subsurface ponding occurs; if the native soil infiltration rate is slower than the percolation water rate, ponding occurs. As shown in Figure 137, as the rain garden size increases in relation to the roof area, less water is discharged to the collection system. About 90% of the long-term runoff would be infiltrated for a rain garden that is about 20% of the roof area (similar to the monitored roof runoff rain gardens in this study):

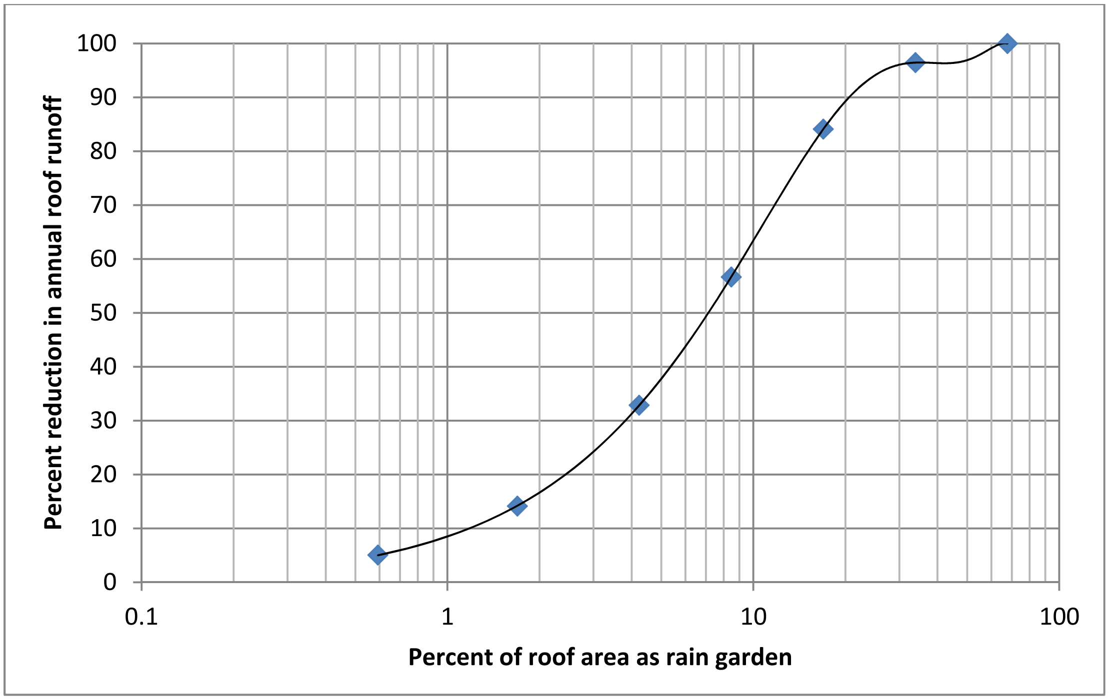


Figure 137. Percentage reduction in annual roof runoff with rain gardens.

Rain gardens 20% of the roof area would also provide about 90% runoff reductions from the directly connected roofs during the 1.4-in. regulatory design storm *D.*

Biofilter performance is based on the characteristics of the flow entering the device, the infiltration rate into the native soil, the filtering capacity and infiltration rate of the engineered media fill if used, the amount of rock fill storage, the size of the device, and the outlet structures for the device. WinSLAMM was used with the calibration files prepared for the Kansas City demonstration project to examine alternative biofilter and bioinfiltration device designs for the residential test (pilot) area. Four infiltration rates for the native subsurface soil were examined: 0.2, 0.5, 1.0, and 2.5 in/hr (corresponding to sandy silt loam, loam, sandy loam, and loamy sand soils, respectively). The lowest rate (0.2 in/hr) was the assumed early infiltration rate used by the design consultants for the original designs. Site surface soil measurements in the test watershed indicated 1 in/hr, or greater, infiltration rates for rains lasting 2 hours or less. Site measurements of the biofilters during storms indicated infiltration rates of the media and device at 1.8 in/hr, and modeling indicated likely subsurface rates of about 1 in/hr (or greater) to result in the observed performance during the rains (almost complete infiltration with very little overflow or subsurface underdrain discharges). The use of gravel storage is important for only the low infiltration rate conditions: once the infiltration rate is about 1 in/hr, or more, this additional storage is not needed, as far as benefiting the long-term infiltration conditions. As shown in Figure 138, for the low infiltration rates, the use of underdrains degrades the performance of the biofilters because the underdrains discharge subsurface ponding water before it can completely infiltrate (but underdrains decrease surface ponding, a desired objective). The use of a slow underdrain (as indicated here by the SmartDrain™), results in an intermediate effect, while also decreasing long periods of surface ponding. As with the gravel storage, underdrains have very little effect on performance when the native subsurface native infiltration rate is about 1 in/hr, or greater.

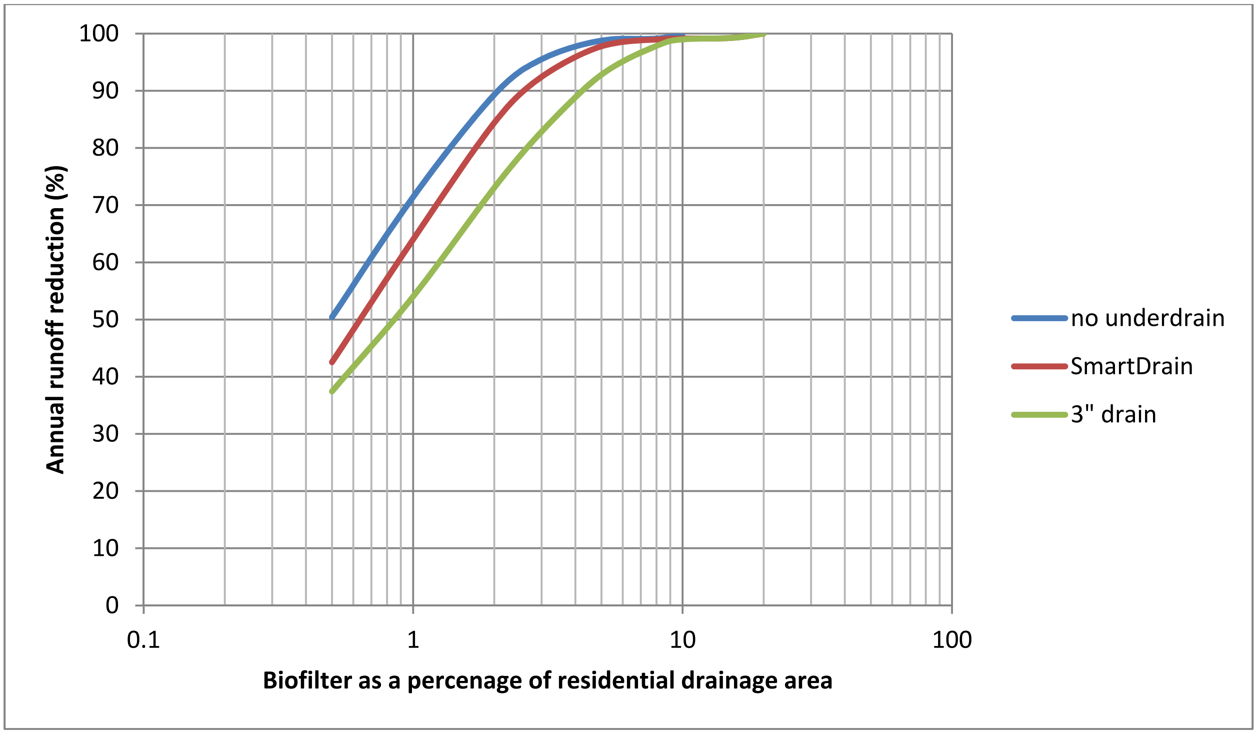


Figure 138. Effects of underdrains in biofilters on annual runoff reductions for subsurface native soil infiltration rate of 0.5 in/hr.

Biofilter media is likely to fail resulting in very low infiltration rates with rapid and excessive particulate solids loadings. Generally, particulate loads of between 10 and 25 kg/m^2 might lead to significantly reduced infiltration. A planted biofilter is likely to be able to incorporate this additional material into the soil as healthy plants can keep the infiltration rates at a desired level, if this accumulative load occurs over at least 10 years. However, if this load occurs within just a few years, it is likely to overwhelm the system, resulting in premature clogging. This is more of a problem for small biofilters receiving runoff having high particulate solids concentrations, such as parking lots where space is limited. Pretreatment using grass filters or swales can reduce these problems. For this study area, if the biofilters are at least 1 to 3% of the residential drainage area, the particulate loading is not likely to be a problem. The biofilters and bioinfiltration devices in the test (pilot) area are about 1.5 to 2% of the residential drainage areas. For the 1 in/hr subsurface infiltration rate, this size of treatment device is expected to provide about a 90% reduction in the annual flows for the areas treated, with very little overflows. The SmartDrain™ installation is expected to have <10% of the annual flows being captured by this underdrain. These calculated conditions are all similar to the observed conditions during the brief monitoring period.

The WinSLAMM porous pavement control in version 10 has full routing calculations associated with subsurface porous media storage and allows runon from adjacent areas. Table 85 summarizes the calculated performance of porous pavement located at paved parking/storage areas. The given underlying soil is a loam soil. A conventional 3-in. perforated pipe underdrain was also assumed. As indicated, even the smallest area examined (25% of the area as porous pavement) had very good runoff volume reductions. The porous pavement was cleaned every year, restoring much of the lost surface infiltration rate capacity in this example. If the area is not cleaned, clogging would be expected in about 8 years, based on field experience. Care needs to be taken to prevent runon of stormwater having high particulate solids loads, or excessive leaf debris on the porous pavement, as both conditions can result in premature failure. Porous pavements are also not recommended for areas having substantial traffic or receiving other more highly contaminated runoff (especially snowmelt in areas using deicing chemicals) to reduce groundwater contamination potential. Sidewalks and walkways, along with residential driveways are the most suitable areas for porous pavement installations.

Table 85. Porous pavement performance (paved parking and storage area; loam soil; 3-in underdrains every 20 ft.)

Porous pvt as a % of paved parking area	Rv	Volume reduction (%)	Expected habitat conditions	TSS (mg/L)	Solids discharged (lbs/yr)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.75	n/a	Poor	130	812	0.21	13	21	1.3
25%	0.06	92%	Good	130	60	0.21	0.98	21	0.098
50%	0.05	93%	Good	130	58	0.32	0.94	12	0.093
100%	0.05	93%	Good	130	58	0.21	0.94	21	0.093

Grass filters have broad, shallow flows. WinSLAMM calculations for grass filters are based on extensive pilot-scale and field measurements of grass swales and filters. Table 86 summarizes the performance of grass filters for controlling runoff from 2 acres of impervious area. As the grass filters become steep, they lose some of their performance because of the faster flowing water reducing the effective infiltration rates.

Table 86. Grass filter performance for different soils and slopes

Description	Rv	% runoff volume reduction	TSS (mg/L)	Solids yield (lbs/yr)	% solids yield reduction	Peak runoff rate (cfs)	% peak runoff rate reduction	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
base conditions, no controls	0.55		100	1040		4.6		0.28	29	17	1.7
grass filter 0.5% slope	0.17	69%	91	300	71%	2.6	43%	0.27	8.7	16	0.52
grass filter 2 to 25% slopes	0.22	60%	90	376	64%	3.5	24%	0.26	11	16	0.67

Grass swales are evaluated in WinSLAMM with the same general processes as for grass filters, except that concentration flows occur. Table 87 summarizes the performance of a swale for two soil conditions. As expected, the swale water volume and pollutant reduction performance is better for the loam soil than for the silty soil.

Table 87. Grass swale performance

Description	Rv	% runoff volume reduc.	Expected habitat conditions	TSS (mg/L)	% solids yield reduc.	Solids yield (lbs/yr)	Peak runoff rate (cfs)	% peak runoff rate reduc.	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
base conditions, no controls	0.55		poor	100		1040	4.6		0.28	29	17	1.7
silty soil	0.33	40%	poor	86	92%	535	4.4	4%	0.25	16	16	0.98
loam soil	0.16	71%	fair	87	92%	263	2.9	37%	0.26	7.8	16	0.47

Benefits associated with stormwater use for irrigation and other on-site uses can be calculated on the basis of site-specific information. Irrigation of land on the homeowner's property was considered the beneficial use of most interest. Rain barrel/water cistern effectiveness is related to supplemental irrigation and how that matches the rainfall deficit (ET minus rainfall) for each season. The continuous simulations used a typical one-year rain series and average monthly ET values for varying amounts of roof runoff storage. Figure 139shows the expected roof runoff reductions for different storage tank volumes. One 35-gallon rain barrel is expected to reduce the total annual directly connected roof runoff by about 24%, if the water use could be closely regulated to match the irrigation requirements, such as with an automated irrigation system with soil moisture sensors (not likely to be used in conjunction with a few rain barrels, but more likely with a large tank than can be pressurized). If four rain barrels were used for each house, such as one at each corner of a house receiving runoff from separate roof downspouts, the total annual roof runoff volume reductions from the roofs could be as high as about 40%. A small water storage tank about 5 ft in diameter and 6 ft in height could result in about 75% total annual runoff reductions from directly connected roofs; a larger, 10-ft-diameter tank that is 6 ft tall could approach complete roof runoff control. The 5-ft-diameter tank is also expected to provide almost complete control of runoff from the regulatory design storm *D.* These calculations are very sensitive to location as the rainfall deficit varies greatly throughout the country. The central part of the United States (including Kansas City) has a relatively large rainfall deficit with rainfall occurring at relatively optimal times for enhanced beneficial uses of roof runoff. Other areas of the county are not as suitable for this control.

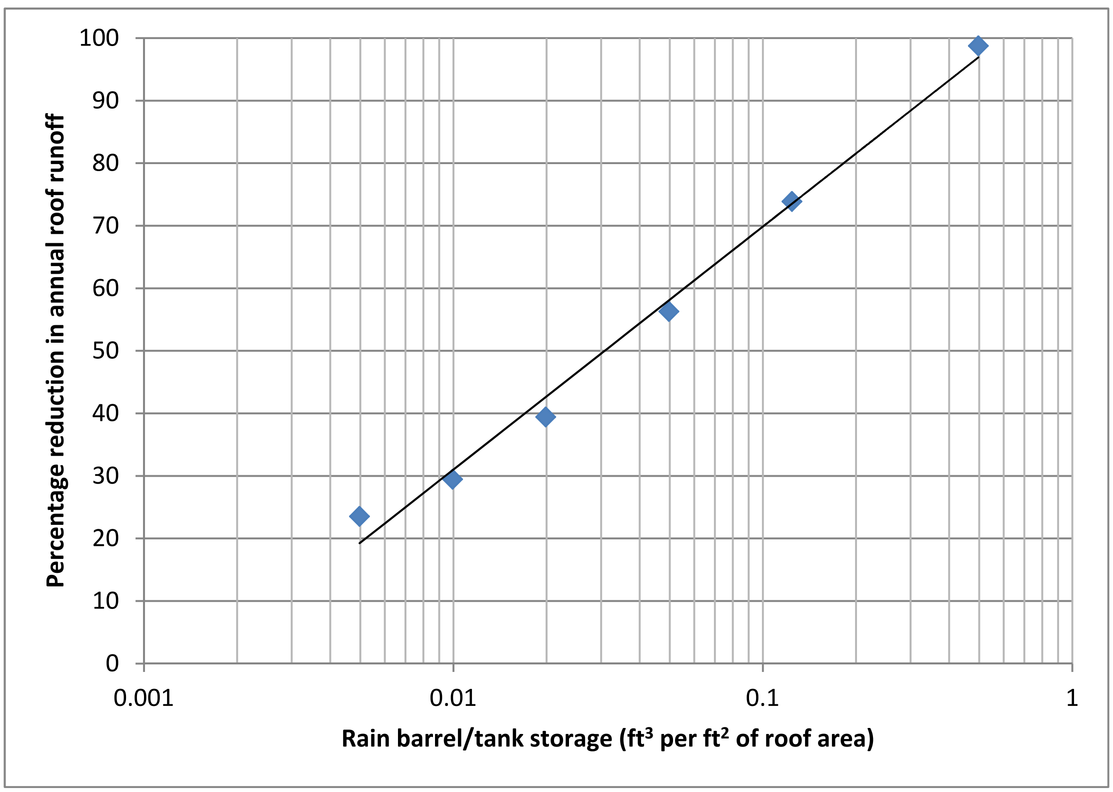


Figure 139. Percentage reduction in roof runoff with irrigation of landscaped areas in Kansas City.

For maximum use of the roof runoff to decrease runoff volumes to a combined sewer, it is desired to irrigate at the highest rate possible, without causing harm to the plants. For a *healthy* lawn, total water applied (including rain) is generally about 25 mm (1 in.) of water per week, or 100 mm (4 in.) per month. Excessive watering is harmful to plants, so indiscriminate over-watering is to be avoided. Some plants can accommodate additional water. As an example, Kentucky Bluegrass, the most common lawn plant in the United States, needs about 64 mm/week (2.5 in/week), or more, during the heat of the summer and should receive some moisture during the winter.

The biofilter option in WinSLAMM can be configured to represent green roofs. Basically, the green roof area is used as the area of the biofilter and no natural infiltration allowed. The only outlets include the required broad crested weir for surface overflows, underdrains, and ET. Partial roof coverage can be modeled by using a smaller area for the *biofilter* to represent the area dedicated to green roof processes. Table 88 summarizes the calculated performance of a green roof system for different roof coverages. The concentrations are similar for all scenarios because almost all of the water is filtered by the roof media, with little being discharged to the surface overflows. The available ET resulted in about 25% reductions in runoff volume reductions. If more surface storage was provided in the green roof design and if more efficient plants were used, it is likely that these runoff volume reductions could be about double the reductions shown here.

Table 88. Calculated green roof performance

Green roof as a % of flat roof area (3-in conventional underdrains every 20 ft)	Rv	Volume reductions (%)	TSS (mg/L)	Solids discharged (lbs/yr)	Peak runoff rate (cfs)	Peak rate reductions (%)	TP (mg/L)	TP load (lbs)	Cu (µg/L)	Cu load (lbs)
none	0.8	n/a	33	55	0.76	n/a	0.22	3.6	11	0.18
25%	0.71	11%	24	35	0.57	25%	0.17	2.4	9.8	0.14
50%	0.66	18%	24	33	0.45	41%	0.16	2.2	9.7	0.13
100%	0.6	25%	24	29	0.38	50%	0.16	2	9.7	0.12

8. Economic and Decision Analyses using WinSLAMM

The cost analyses in WinSLAMM can be used to automatically calculate the capital, maintenance and operation, and financing costs for stormwater control programs being examined. This information can be used with the model batch processor to develop cost-benefit curves for the different control options. The cost information is entered in the model using the set of forms as shown in Figure 140. Figure 141 shows the cities that have inflation data already in the model (including Kansas City). Besides the unit cost rates that are already available, it is possible to enter more specific local cost data, based on site costs.

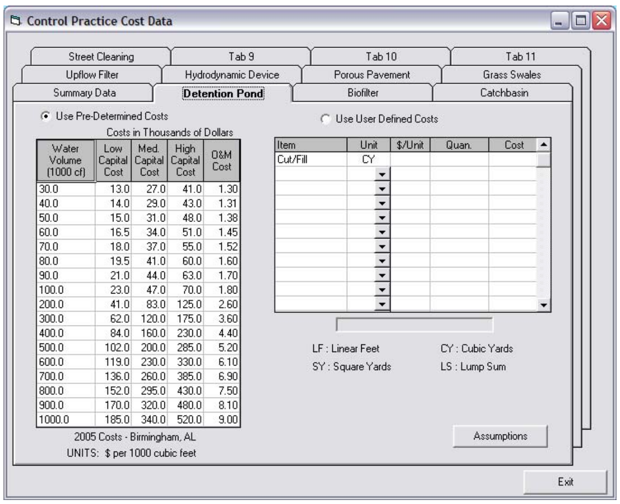


Figure 140. Basic economic analyses input screen

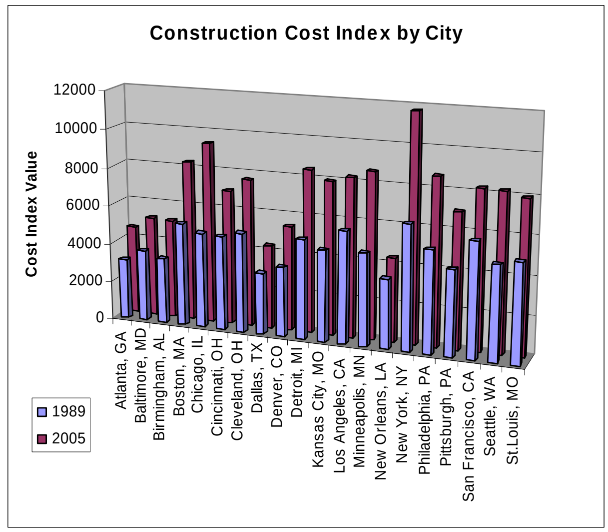


Figure 141. U.S. cities already in the economic model

An example of a performance production function that can be used in conjunction with the economic analyses is illustrated in Figure 142 which is a plot of the percentage of the typical annual runoff amount that can be infiltrated by curb-cut rain gardens, based on the number of units used. With 1,500 units possible in this area, up to about 80% of the annual runoff could be infiltrated. With 400 units, about 40% of the annual flows would be diverted from the combined sewers. Figure 143 plots some preliminary cost estimates for these devices (this estimate does not consider aesthetic landscaping, only basic excavation and simple curb cuts). The basic total capital cost for these very small devices is expected to be about $1,000 each, and the annualized total cost to be about $150 each. Again, the actual costs are likely to be greater because of the planting and plant maintenance. Figure 144 shows the durations of flows at different rates for several different curb-cut rain garden applications. The maximum peak flow for the typical rain year is expected to be between 25 and 30 cfs for this area. The use of 600 rain gardens is also likely to reduce the peak flow rates that occur about 5 to 10 hours a year to about half of the flow rates that would occur if uncontrolled.

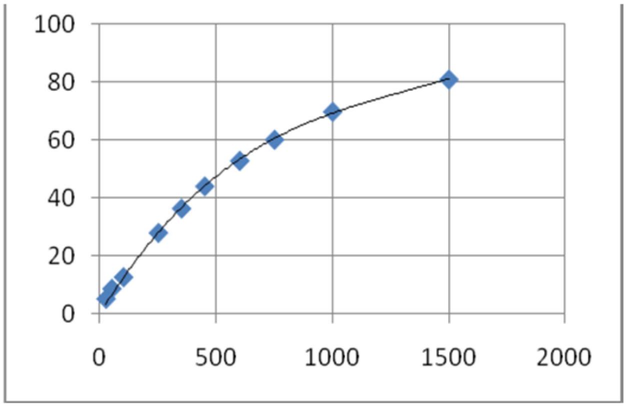


Figure 142. Annual runoff volume reduction (%) for typical rain year (1990) for different numbers of simple curb-cut rain gardens per 100-acre watershed.

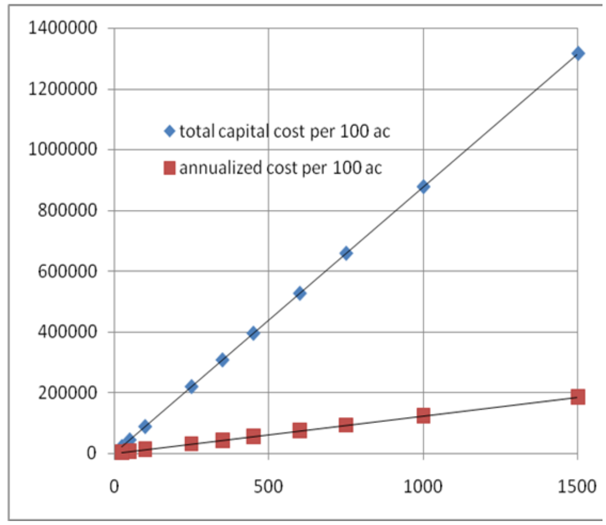


Figure 143. Total capital costs and total annualized costs for different numbers of simple curb-cut rain gardens per 100-acre watershed.

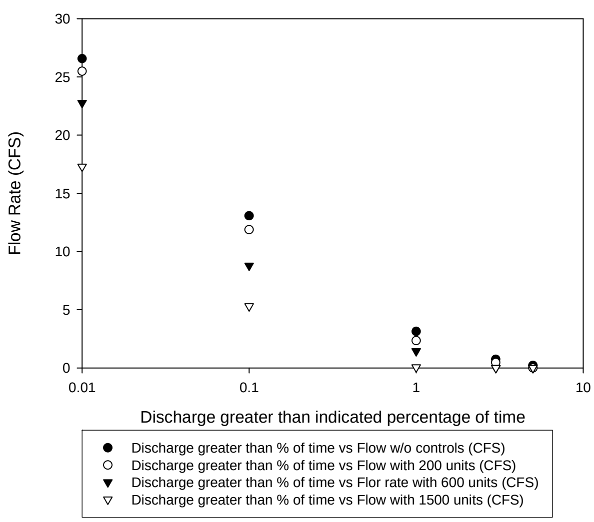


Figure 144. Durations of flows (% of time) for different numbers of simple curb-cut rain gardens.

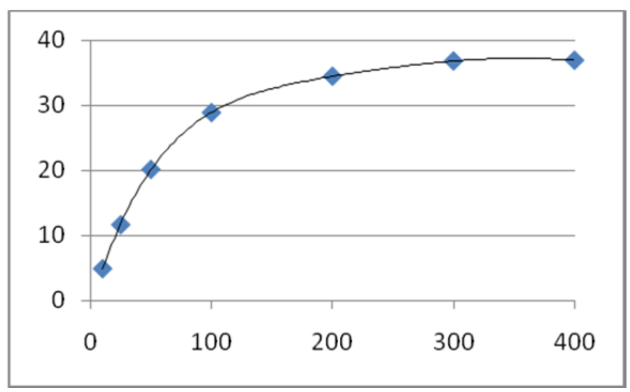


Figure 145. Percentage reduction of annual flows with 10 ft diameter x 5 ft tall cisterns (numbers per 100 acres)

Figure 145 is a plot of the annual roof runoff removals that would occur for different numbers of large cisterns in the area. The maximum control that is expected is about 35%, as that is the fraction of the annual flow that is expected to originate from the roofs. This level of control would occur with about 200 large cisterns in the 100-acre area. Very small rain barrels would have very little benefits in reducing the annual discharges to the combined sewer.

Table 89 shows the expected level of control for various combinations of large cisterns and curb-side rain gardens. The largest level of control expected is about 90% of the annual runoff, but that would require a maximum application of these controls. However, levels of runoff reduction of about 75% could be achieved with a more reasonable effort (about 500 rain gardens and 250 cisterns, or 1,000 rain gardens and 50 cisterns). The expected cost of this high level of control is likely to be more than $1million for the 100 acres, just for these components. Controls established at the time of development can be much less and, in many cases, can be less than conventional development options.

Table 89. Approximate annual flow reductions (%) for combinations of large cisterns and simple curb-side rain gardens, per 100 acres

	0 rain gardens	100 rain gardens	500 rain gardens	1,000 rain gardens	1,500 rain gardens
0 cisterns	0%	12%	47%	70%	81%
25 cisterns	12%	23%	52%	73%	82%
50 cisterns	20%	32%	58%	76%	83%
100 cisterns	29%	40%	66%	80%	85%
250 cisterns	36%	47%	73%	86%	90%
600 cisterns	37%	48%	74%	87%	91%

Using WinSLAMM Decision Analyses to Select an Urban Runoff Control Program

Decision analysis techniques can be used to guide the selection of an urban runoff control program. Decision analysis is a systematic procedure that enables one to study the tradeoffs among multiple and usually conflicting program objectives. An alternative procedure is to separately determine the programs necessary to meet each objective and to use the least costly program that satisfies all the identified critical objectives. This is an acceptable procedure some of the time, but it might not result in the most cost-effective program, especially when multiple objectives need to be considered.

Decision analysis optimizes the partial fulfillment of all the objectives. It translates these into their relative worth to the decision maker or other interested parties. This section describes the types of output information calculated by WinSLAMM and how it can be used in decision analysis procedures of varying complexities.

As in most models, there is a great deal of information calculated by WinSLAMM during an analysis of stormwater management alternatives. In most cases, values presented on the main WinSLAMM summary screen are sufficient for most comparisons. These include the overall percent runoff and particulate solids reductions, the final Rv and runoff volume, and the resulting particulate solids and pollutant yields and concentrations. In addition, life cycle costs (including lost opportunity, capital, land, operation, and maintenance costs) and the expected habitat conditions of the receiving waters is also available for evaluation, in addition to flow-duration information. Cost data included in the model were obtained from several studies, including those by APWA 1992; Brown and Schueler 1997; Frank 1989; Heaney et al. 2002; Muthukrishnan et al. 2006; Sample, et al. 2003; SEWRPC 1991; Wiegand et al. 1986; and Wossink and Hunt 2003. The batch processor in WinSLAMM is frequently used to automatically examine all the land use and stormwater control options for a relatively large area, such as for citywide analysis, especially when used in conjunction with GIS data.

Figure 146 is a screenshot of the main batch processor screen that is used to select the standard land use files for an area being examined, along with the areas, and soils. This screen is also used to select a set of files that can be run in batch mode to compare multiple stormwater controls for the same site, as described later. In that configuration, the first file listed is the base condition that is compared to the other files. Alternative analyses are also usually conducted to examine different stormwater control practices.

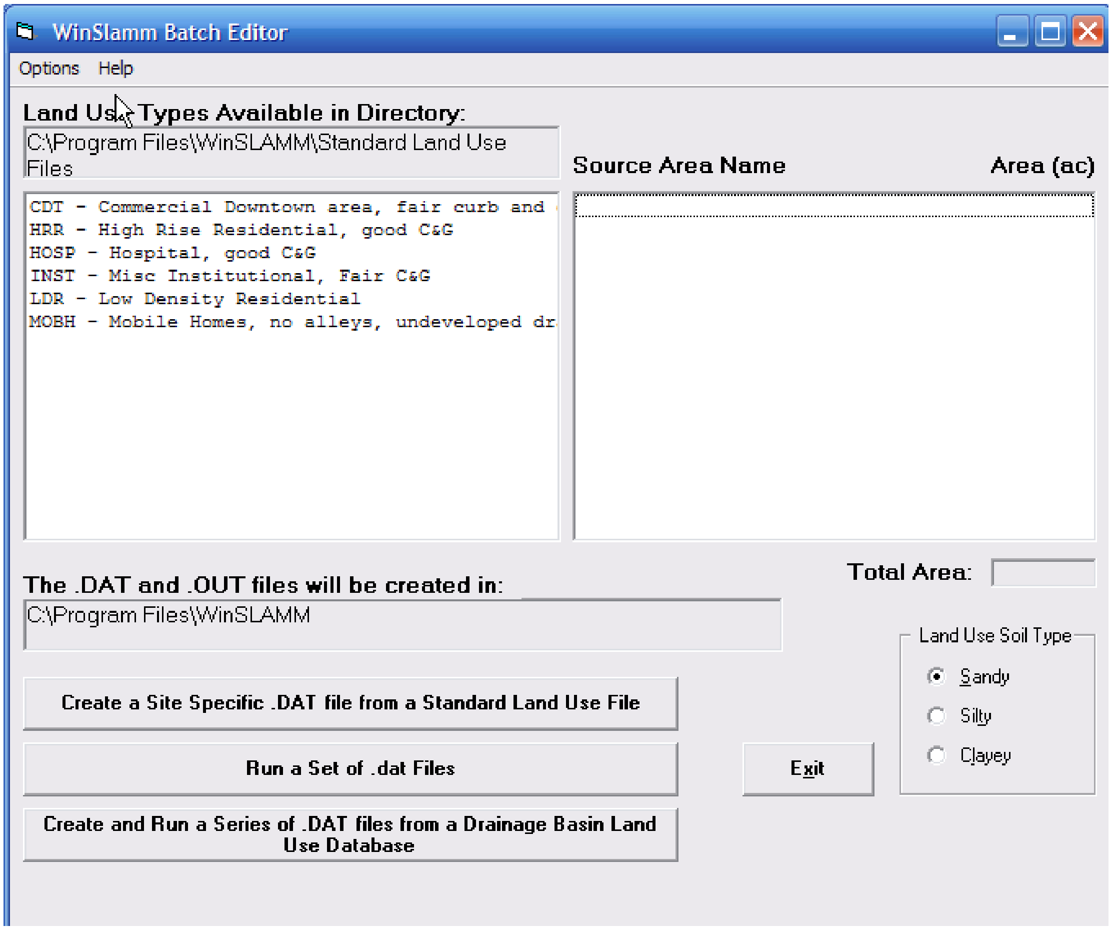


Figure 146. WinSLAMM Batch Editor setup screen.

Recent enhancements to WinSLAMM allow the batch processor to be used to enable comparisons of different stormwater control programs for a single site. As noted above, many stormwater factors are calculated for each analysis, and a stormwater manager might have difficulty comparing the different alternatives. Tables 90 and 91 are summarized from the expanded csv output file (showing only a few of the calculated factors, as an example), comparing eight alternative stormwater management programs to a base condition that was calculated with the WinSLAMM batch processor. The alternatives and the full analyses for this example are shown later in this section. The different stormwater management programs considered in this example are: grass swales, two wet detention ponds, biofilters, plus combinations of these controls. WinSLAMM can evaluate many other alternative controls, and combinations, but this is shown as only a short example of the output table.

Table 90. Attributes of several different stormwater management programs

Stormwater treatment option	Part. phos yield (lbs/yr)	Volum. runoff coeff. (Rv) (est. bio. cond.)	% of time flow > 1 cfs	% of time flow > 10 cfs	SS conc. (mg/L)	Part. P conc. (mg/L)	Zn conc. (µg/L)
Base, No Controls	174	0.29 (poor)	4.5	0.3	204	0.50	359
Option 1 Pond	25	0.29 (poor)	4	0.05	30	0.073	128
Option 2 Grass Swale	79	0.15 (fair)	2	0.1	178	0.43	390
Option 3 Site Biofilter	172	0.14 (fair)	2	0.2	408	1.0	696
Option 4 Small pond	41	0.29 (poor)	4	0.2	48	0.12	151
Option 5 Pond and grass swale	10	0.15 (fair)	2	0	23	0.057	203
Option 6 Pond, swale, biofilter	5.5	0.06 (good)	0.5	0	29	0.073	386
Option 7 Small pond and swale	17	0.15 (fair)	2	0.05	39	0.095	220
Option 8 Small pond, swale and biofilter	10	0.07 (good)	0.8	0	53	0.13	390

Table 91. Additional attributes of several different stormwater management programs

Stormwater treatment option	Annual total sw treat. cost ($/yr)	Annual addit. drain. system cost ($/yr)	Total annual cost ($/yr)	Land needs for SW mgt (acres)	Runoff volume (cf/yr)	Part. solids yield (lbs/yr)	Reduc. in SS yield (%)
Base, No Controls	0	64,230	64,230	0	5,600,000	71,375	n/a
Option 1 Pond	19,134	64,230	83,364	4.5	5,507,000	10,192	86
Option 2 Grass Swale	3,158	26,850	30,008	0	2,926,000	32,231	55
Option 3 Site Biofilter	32,330	37,380	69,710	0	2,705,000	68,890	1
Option 4 Small pond	10,209	64,230	74,439	2.3	5,557,000	19,552	73
Option 5 Pond and grass swale	22,292	26,850	49,142	4.5	2,844,000	4,133	94
Option 6 Pond, swale, biofilter	54,622	0	54,622	4.5	1,203,000	2,183	97
Option 7 Small pond and swale	13,367	26,850	40,217	2.3	2,887,000	6,937	90
Option 8 Small pond, swale and biofilter	45,698	0	45,698	2.3	1,253,000	4,125	94

Table 91 also shows the additional conventional drainage system costs for each option, including the costs associated with a conventional storm drainage system from external calculations. If at least 80% particulate solids reductions are needed (a typical goal for some programs, including those in Massachusetts and Wisconsin for new developments), several options would meet this goal, as shown in the last column. Option 7, the use of grass swales plus a small wet detention pond, is the least costly of these acceptable options. This option also has the benefit of significant runoff volume reductions, compared to the base condition, although the options adding the biofilters with the swales produce even less runoff.

The above example illustrates a relatively straightforward approach in selecting the *best* stormwater control program. However, it might be desirable to also consider other attributes associated with the different options. The following discussion is based on material originally presented by Pitt (1979) and is a hypothetical example application of a decision analysis procedure that considers conflicting and multiple objectives applied to selecting a street cleaning program as part of a stormwater management plan.

Decision Analysis with Multiple Conflicting Objectives

The following is a hypothetical example with fictional values that illustrates the basic elements of decision analysis to select a preferred street cleaning program from a list of alternatives (updated from an earlier discussion presentation by Pitt 1979). The objectives of such a program might include maximizing air, water, and aesthetic quality and minimizing the noise and cost of street cleaning operations. Unfortunately, some objectives (such as cost and environmental quality) tend to conflict with each other. The decision makers must choose the alternative that makes the best tradeoffs among the competing objectives.

The techniques of decision analysis, as described by Keeney and Raiffa (1976), are used to aid in the selection process. This historical reference contains detailed discussions on decision analysis theory and should be consulted for further information. This method uses utility curves and tradeoff values between the different attributes. The utility curves should be based on data and not reflect personal attitudes or objectives, while the tradeoffs between the attributes reflect different viewpoints. This decision analysis method is therefore a powerful tool that can be used to compare the rankings of alternative stormwater management programs for different groups. In many cases, final rankings might be similar among the interested parties, although their specific reasons vary. Most importantly, this tool also completely documents the decision-making process, enabling full disclosure. This feature is probably more important for site selection projects for power plants than for small public works projects, but this level of documentation is still critical when public policy and taxes are concerned.

The detail and depth of understanding needed to fully use this decision analysis methodology forces the user to acquire a deeper understanding of the problem being solved. Multiple experts are usually needed to develop the utility curves, but they can be used for similar projects in the same region sharing similar problems and objectives. The tradeoffs are dependent on the mix of decision makers and stakeholders involved in the process and are expected to change with time. The depth of knowledge obtained and full documentation always is a positive aspect of these methods, but the required resources to fully implement the system can be an insurmountable obstacle to smaller communities. However, sensitivity analyses can be used to focus resources only on those aspects of greatest importance.

The first step in applying decision analysis techniques consists of defining the alternatives and quantitative measures (attributes) for the objectives. How well each alternative achieves the objective is also determined. In this hypothetical example, five example attributes were chosen to reflect widely different considerations in deciding which street cleaning program to select. These attributes, their units of measurement, and the associated ranges are shown in Table 92.

Table 92. Decision analysis attributes, measures, and ranges of values

			Range of values	
	Attribute description	**Units of measurement**	**Best**	**Worst**
1.	Aesthetics (residual loading)	lb/curb-mile	68	525
2.	Annual cost	$/curb-mile/year	350	3,600
3.	Air quality (particulates)	$\mu g/m^3$	100	200
4.	Water quality (suspended solids)	mg/L	200	1,500
5.	Noise Level	dB_A	65	82

The second step consists in describing each alternative in terms of the attributes defined in step one. The value of each attribute for each of the alternatives must be determined. The attribute levels may be described either in terms of probabilistic forecasts, where uncertainties are quantified, or by point estimates representing the level expected for each attribute. In this example, five alternative street cleaning programs are considered, and point estimates are made for each attribute. The street cleaning programs consist of combinations of equipment types and their frequencies of use. These alternatives are defined in Table 93. Point estimates, for illustrative purposes, are used for this example and summarized in Table 94, which shows that all attributes, except cost, are better than, or equal for alternative two.

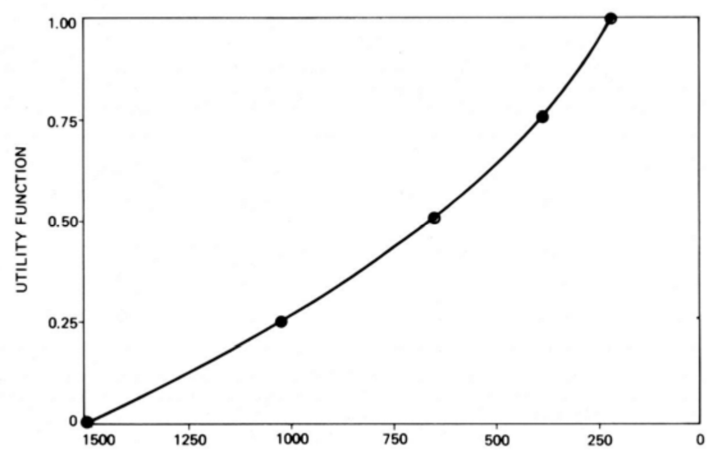
Table 93. Definition of alternatives

Alternative description	
1	Conventional mechanical street cleaner, one pass every week
2	Conventional mechanical street cleaner, one pass every weekday
3	Vacuum street cleaner, one pass every week
4	Street flusher, one pass every week
5	Conventional mechanical street cleaner followed by a flusher, one pass every week

Table 94. Estimated attribute levels for each alternative (fictional)

Alternative	**Aesthetics (lb total solids/curb-mile)**	**Annual cost ($/curb-mile/year)**	**Air quality (µg susp partic/m³)**	**Water quality (mg TSS/L)**	**Noise level (dB_A/pass)**
1	340	700	200	1,000	65
2	68	3,600	120	200	65
3	470	700	150	1,400	70
4	525	350	200	1,500	80
5	150	1,000	150	400	82

The third step consists of quantifying the preference and tradeoffs for the various attribute levels. The concepts of utility theory provide a consistent scale to quantify how much one gives up when choosing one attribute over another. Utility curves are first assessed for the individual attributes. These curves quantify the preferences that exist for the total range of each attribute. They also quantify attitudes toward risk. This is important when alternatives yield uncertain consequences. The curves are theoretically defined from a series of questions that determine points on each of the utility curves. The most preferred point is defined as having a utility value of 1.00 and the least preferred point a utility value of 0.00. The utility assessments establish where the intermediate points fall on the utility scale. An example of a utility function for a water quality attribute is shown in Figure 147. Each of the other attributes can be assessed on a similar curve.



Source: Pitt 1979

Figure 147. Example utility function for a water quality attribute (TSS, mg/L).

The formal development of a utility curve can be determined through a series of questions. In many cases, the shape of the utility curve can be reasonably determined through direct knowledge of the attribute. In other cases, it is suitable to assume a linear relationship between the maximum and minimum attribute levels. The utility curves are technology-based and reflect how different levels of an attribute relate to other levels of the same attribute. As an example, further degradation of a receiving water is unlikely after the dissolved oxygen levels reach anaerobic conditions, but increasing stress occurs as that level is approached. This information can be used to determine the shape of the utility curve. In the example of cost, spending twice as much is probably twice as bad reflecting a straight-line relationship between cost and utility.

The questions that can be used to define the individual attribute utility curves consist of asking the decision maker to choose one of two possible situations. In this example, one situation is uncertain and describes a 50-50 chance for a successful outcome of one of the two possible levels of the attribute; the second situation occurs with certainty and consists of achieving a specified level of the attribute. The level of the attribute in the second situation is somewhere between the two equally possible levels of the first situation. The utility assessment for each point on the curve is determined by the attribute level in the second situation, where the decision maker is indifferent to the choice of the two situations. Because, at the point of indifference, each choice is equally acceptable, the expected utility values of the two situations must be equal, and a point of the utility curve can be established.

Consider for example a situation with a 50-50 chance of achieving water quality at either 1,500 or 200 mg TSS/L. What level of water quality (if known with certainty) would be equally preferable to the uncertain situation above? After a series of trial choices, it was determined that a water quality level of 650 mg TSS/L would be indifferent to the uncertain situation. Again, this would be based on knowledge of the attribute, such as how the risk varies for different concentrations, such as how the toxicity response varied for different conditions during controlled toxicity tests. Thus, the utility of a water quality level of 650 mg/L TSS must equal the expected utility of the uncertain situation with a 50-50 chance of achieving either 1,500 or 200 mg/L TSS. Because the utility values of 1,500 and 200 mg/L are known to be 0.00 and 1.00, respectively, the expected utility of the first situation can be calculated as 0.5 (0) + 0.5 (1.00) = 0.5. Therefore, the utility value of 650 mg/L must equal 0.5. This point is plotted in Figure 147. Similar questions can be used to define the other points shown in Figure 147.

The tradeoffs that exist among the attributes are established next. While the utility curves should be based on scientific knowledge, the tradeoffs should reflect the different attitudes of the different interested parties. Different tradeoffs will result in possibly different final rankings for the different street cleaning programs for the different groups. Determining the tradeoffs is done by first ranking the attributes in order of importance. The tradeoffs result in values given to each attribute, such that the sums of the values equal one. The simplest approach is to request the decision makers to rank the attributes and arbitrarily assign tradeoffs such that the tradeoff values equal one.

The rank order and tradeoff values can be theoretically established by answering questions like the following: "Given that all attributes are at their worst levels, which attribute would one first move to its best level?" The question is repeated to determine which attribute would next be moved to its best level. This process is continued until the complete rank order of the attributes is established. In this example, the following rank order of the attributes was established:

- Water Quality
- Annual Cost
- Air Quality
- Aesthetics
- Noise Level

The tradeoffs among attributes are addressed next. This can be done by considering the choice between two possible situations for a pair of attributes. Both situations are certain but consist of different levels for the pair of attributes. The levels for the pair of attributes are in the form of *worst, best* compared with *?, worst.* The unknown attribute level is established after repeated trials until the decision maker is indifferent to the two situations. Considering the water quality/annual cost attribute pair, the two situations would be "1500 mg/L, $350" and "?, $3600." In this situation, we are determining how much people would expect the water quality to improve with an increase in cost. In this hypothetical example, if the water quality were 650 mg/L, the second situation would be indifferent to the first situation. Similar questions were asked for other pairs of attributes, determining how much the attribute level was expected to improve with increasing cost. These hypothetical results are summarized below, using the notation (≅) to indicate indifference.

- (Water quality, annual cost) = (1500 mg/L, $350) ≅ (650 mg/L, $3600)
- (Annual cost, noise level) = ($3600, 65 dbA/pass) ≅ ($3000, 82 dBA/pass)
- (Annual cost, aesthetics) = ($3600, 68 lb/mile) ≅ ($3000, 525 lb/mile)
- (Annul cost, air quality) = ($3600, 100 μg/m^3) ≅ ($1500, 200 μg/m^3)

The above information concerning the preferences for achieving levels for the attributes can be used to establish a multiattribute utility function. A multiattribute utility function is a mathematical expression that summarizes attribute utility functions and the tradeoffs between the attributes. The mathematical form of the multiattribute utility function is established by verifying several reasonable assumptions regarding preferences. To illustrate, an additive multiattribute utility function is represented as follows:

$$u(x_1, x_2, x_3, x_4, x_5) = \sum_{i=1}^{5} k_i v_i(x_i) \qquad (8.1)$$

where

x_i	=	the level of the *i*th (i-1,5) attributes
$u_i(x_i)$	=	the utility of the *i*th individual attribute
v	=	the multiattribute utility
k_i	=	tradeoff constant for *i*th attribute, and

$$\sum_{i=1}^{5} k_i = 1$$

The tradeoff constants in Equation 8.1, k_i, are calculated on the basis of the individual attribute utility functions and indifference points for pairs of attributes. These individual tradeoff constants can be calculated as shown below, on the basis of the equivalent pairings from the preceding questions. Although the utility functions actually assessed would normally be used to illustrate this example, it is assumed that each of the individual attribute utility functions is linear in this example. Keeney and Raiffa (1976) illustrate many other examples for these calculations for other conditions.

The multiattribute utility values for assessed points of indifference between pairs of attributes must be equal because they are equally preferable. Holding all attributes not considered in the pair tradeoffs at their worst level so that their utility value is zero, the k_i values (where the subscript *i* is for each attribute) in Equation 8.1 can be calculated. The ratio between the tradeoff constants for any two attributes (such as k_2/k_4, the ratio of the cost and water quality tradeoff constants) is therefore equal to the utility value of the attributes that is the denominator for this worst-case comparison.

As an example, the water quality attribute value of 650 mg/L TSS relates to the worst case cost attribute value of $3,600. The corresponding utility value for this water quality attribute value is 0.65, the ratio between the cost and water quality tradeoff constant (k_2/k_4). The following relationships show the ratios of the other tradeoff values:

$$\frac{k_2}{k_4} = u_4(650 mg/L) = 0.65 \qquad (8.2)$$

$$\frac{k_5}{k_2} = u_2(\$3000) = 0.23 \qquad (8.3)$$

$$\frac{k_1}{k_2} = u_2(\$3000) = 0.23 \qquad (8.4)$$

$$\frac{k_3}{k_2} = u_2(\$1500) = 0.46 \qquad (8.5)$$

Using Equation 8.2,

$$\sum_{i=1}^{5} k_i = (0.23 + 1.00 + 0.46 + 1.54 + 0.23)k_2 = 1 \qquad (8.6)$$

$k_2 = 0.29$ for the annual cost attribute (8.7)

Therefore,

$k_1 = 0.07$ the aesthetics attribute (8.8)
$k_3 = 0.13$ for the air quality attribute (8.9)
$k_4 = 0.42$ for the water quality attribute (8.10)
$k_5 = 0.07$ for the noise level attribute (8.11)

The above tradeoff constant values, the individual attribute utility functions, and the original equation completely define the multiattribute utility function.

The fourth step consists of synthesizing the information. The multiattribute preferences, when combined with the attribute levels associated with each alternative, allow a ranking of the five alternative street cleaning program alternatives. The estimated attribute levels for each alternative shown in Table 95 and the individual attribute utility functions are used to determine ui (xi) for each alternative.

Table 95. Individual attribute utility values for each alternative

Alternatives	Aesthetics	Annual cost	Air quality	Water quality	Noise level
1	0.40	0.90	0	0.38	1.00
2	1.00	0	0.80	1.00	1.00
3	0.12	0.90	0.50	0.08	0.71
4	0	1.00	0	0	0.12
5	0.82	0.80	0.50	0.85	0

The information given in Table 95 is then substituted into Equation 8.1 to define the multiattribute utility associated with each alternative. These utility values provide the basis for determining the rank order of the alternatives and the degree to which one alternative is preferred over another. The utility values associated with each alternative are shown in Table 96.

Table 96. Utility of each alternative

Alternative	Utility
1	0.52
2	0.66
3	0.42
4	0.30
5	0.72

The most preferred alternative is that with the highest utility value. For this example, Table 96 reveals that alternative five (conventional mechanical street cleaner followed by a flusher, every five days) is the most preferred alternative. This is followed closely by alternative two (conventional mechanical street cleaner, one pass every day). The least desirable was alternative four (flusher, one pass every five days). Again, this is a hypothetical example used to illustrate a procedure that can be used for this type of decision analysis approach; the values used are fictional as are the results of this hypothetical analysis.

Obviously, changes in preferences for the attributes or estimated attribute levels associated with each alternative could alter the order of preference for the alternatives. The decision analysis methodology summarized here would allow such changes to be rapidly investigated by a sensitivity analysis of the rank order of alternatives. For example, if the tradeoff between annual cost and water quality were changed so that the annual cost is somewhat more important than in the previous tradeoff, alternatives one and two can become equally preferred, but alternative five is still the most preferred. Also, new attributes may be added to the analysis and the alternatives ranked again.

Example Decision Analysis Application with Extended WinSLAMM Data Output

The above example was prepared some time ago when stormwater modeling techniques were still in their infancy, and environmental regulations, especially for stormwater, were not well developed (and when we were very optimistic concerning the benefits of street cleaning). It is now possible, such as with the recent enhancements made to WinSLAMM, to more completely evaluate different stormwater management options that consider a wide variety of conflicting objectives. The following example is based on a recent project and illustrates the procedure from the above discussions (Pitt and Voorhees 2007).

Attribute Levels Associated with Different Stormwater Management Programs

WinSLAMM generates a great deal of information when stormwater management options are evaluated, as previously described. New revisions to the batch processor option in the model make it possible to summarize many of the important attributes in a simple spreadsheet format. The site and corresponding stormwater management options for this example are described below. All costs are in U.S. dollars.

Descriptions of Site and Alternative Stormwater Controls

This example site is a new industrial park in northern Alabama that is about 98 acres (40 ha) in area, comprising about 33.8 acres (13.7 ha) of industrial land, 60.2 acres (24.5 ha) of open space land, and 4.6 acres (1.9 ha) of buffers surrounding sinkholes. There are 13 industrial lots in this subarea, each about 2.6 acres (1.1 ha) in area. The following list shows the estimated total surface covers for these 98 acres:

- Roofs: 18.4 acres (7.5 ha)
- Paved parking: 2.3 acres (0.9 ha)
- Streets (1.27 curb-miles): 3.1 acres (1.3 ha)
- Small landscaped areas (B, or sandy-loam soils, but assumed silty soils because of compaction): 10.0 acres (4.1 ha)
- Large undeveloped area (B or sandy-loam soils, but assumed silty soils because of compaction): 60.2 acres
- Isolated areas (sinkholes): 4.6 acres

The stormwater control options examined in this subarea included the following:

Conventional storm drainage system elements:

The base conditions (associated with the *Base Conditions, No Controls* option) have conventional curb and gutters with concrete storm drainage pipes, and the roofs and paved parking areas are directly connected to the storm drainage system. The conventional drainage system for base conditions were sized using conventional stormwater drainage system methods (SWMM), and were composed of: 5,200 ft (1,585 m) of 18 in. (460 mm) and 3,360 ft (1,024 m) of 36 in. (910 mm) storm drainage pipe, plus 39 on-site and 45 public street inlets. The estimated costs for these conventional storm drainage elements are from RS Means (2006 publication, 2005 basis) and are \$19 per ft (304 mm) for 18 in. and \$72 per ft for 36-in. reinforced concrete pipe. Excavation and backfilling costs add \$6/yd^3. The inlets are \$3,000 each.

The on-site drainage elements are needed whenever the site biofilter-swale option is not being used:
5,200 ft of 18-in. concrete pipe (buried in a 5-ft [1.5 m] deep trench) at \$25/ft = \$130,000
39 inlets = \$117,000.

Total on-site drainage costs: \$247,000 (1996 costs) x 1.2 = \$296,400 (2005 costs, based on ENR index).
In addition, it is assumed that annual maintenance costs for these drainage elements will be 1% of the total capital costs for each year = \$2,960/y (2005 costs).

The roadside drainage elements are needed whenever the regional swale option is not being used:
3,360 ft of 36-in. concrete pipe (buried in an 8-ft [2.4 m] deep trench) at \$80/ft = \$268,800
25 inlets = \$75,000.

Total roadside drainage costs: \$343,800 (1996 costs) x 1.2 = \$412,560 (2005 costs, based on ENR index). In addition, it is assumed that annual maintenance costs for these drainage elements will be 1% of the total capital costs for each year = \$4,130/y (2005 costs).

These initial costs must be converted to annualized costs. The following is based on the procedures outlined by Narayanan and Pitt (2005) and is the same procedure used in WinSLAMM for calculating the costs of the stormwater controls.

Annual on-site drainage costs:
Interest rate on debt capital = 5%
Project financing period = 20 years
Capital cost of project = \$296,400 (2005)
Annual maintenance cost = \$2,960/year (2005)

$$\text{Annual value of present amount} = \frac{i(1+i)^N}{(1+i)^N - 1}$$

$$\text{Annual value of present amount (or) annual value multiplier} = \frac{0.05(1+0.05)^{20}}{(1+0.05)^{20} - 1} = 0.0806$$

Annualized value of all costs = Annualized value of (total capital cost of project) + annual maintenance and operation cost.

= 0.0806 * (\$296,400) + \$2,960 = \$26,850 per year

Annual roadside drainage costs:

Interest rate on debt capital = 5%
Project financing period = 20 years
Capital cost of project = $412,560 (2005)
Annual maintenance cost = $4,130/year (2005)
Annualized value of all costs = Annualized value of (total capital cost of project) + annual maintenance and operation cost = 0.0806 * ($412,560) + $4,130 = $37,380 per year

On-site biofilter swales:

These small drainage swales, included in options 3, 6, and 8, collect the on-site water from the roofs and paved areas and direct it to the large natural swales. These have the following general characteristics: 200 ft (61 m) long, with 10 ft (3.1 m) bottom widths, 3 to 1 (H to V) side slopes (or less), and 2 inches (51 mm) per hour infiltration rates. One of these will be used at each of the 13 sites on the site. These swales will end at the back property lines with level spreaders (broad-crested weirs) to create sheetflow toward the large drainage swale.

When modeling the site biofilters, the following dimensions were used:

Top area: 4,400 ft^2
Bottom area: 2,000 ft^2
Depth: 2 ft
Seepage rate: 2 in/hr
Peak to average flow ratio: 3.8
Typical width for cost purposes: 10 ft
Number of biofilters: 13 (one per site)
All roofs and all paved parking/storage areas drained to the biofilters

The level spreader at the end of the biofilter was modeled assuming a broad-crested weir having a crest length of 12 ft , a crest width of 10 ft, and the height from the datum to bottom of opening was 1 ft. Table 97 shows the evaporation rates used for this example analyses.

Table 97. Example monthly average evaporation rates (in/day)

January	0.01
February	0.03
March	0.06
April	0.08
May	0.12
June	0.25
July	0.25
August	0.15
September	0.08
October	0.06
November	0.03
December	0.01

Large regional drainage swale:

Options 2, 5, 6, 7, and 8 include a natural drainage swale in this subarea that will collect the sheetflows from the bioretention swales from each site and direct the excess water to the ponds. This swale is about 1,700 feet long, on about a 2.6% slope, and is 50 ft wide. It has 3 to 1 (H to V) side slopes, or less, and 1 in/hr infiltration rates. The bottom of the swale will be deep vibratory cultivated during proper moisture conditions to increase the infiltration rate, if compacted. This swale also has limestone check dams every 100 ft to add alkalinity to the water and to encourage infiltration. The vegetation in the drainage will be native grasses having deep roots and be mowed to about 6 in., or higher. Any cut grass will be left in place to act as a mulch that will help preserve infiltration rates. The swale will also have a natural buffer on each side at least 50 ft wide.

When modeling this large, regional swale, the model used a swale density of 29 ft/ac with 57 acres served by the swales, resulting in a total swale length of 1,653 ft. The drainage system is composed of 58% grass swales and 42% undeveloped roadside. The infiltration rate in the swale was 1 in/hr. The swale bottom width was 50 ft, with 3H:1V side slopes. The longitudinal slope was 0.026 ft/ft, and Manning's n roughness coefficient was 0.024. For the cost analysis, the typical swale depth was assumed to be 1 ft.

Wet detention pond:

Options 1, 4, 5, 6, 7 and 8 include a wet detention pond across the main road next to the southern property boundary. The regional swale will direct excess water into the pond far from the discharge point. The pond is a wet pond having the approximate dimensions and depths shown in Table 98.

Table 98. Wet detention pond size and elevation characteristics

Pond elevation (ft)	Pond area (acres)
1	0.15
2	0.25
3	0.5
4	0.75
5	1.0 (normal pool elevation, and invert elevation of 30^{o} v-notch weir)
6	1.5
7	2
8	2.5 (invert elevation of flood flow broad-crested weir). Normal maximum elevation during one and two year rains.
9	3.0 (approximate maximum pond elevation, or as determined based on flood flow analysis). Additional storage and emergency spillway may be needed to accommodate flows in excess of the design flood flow.

The pond storage between 5 and 9 feet is about 8 acre-ft. If additional storage is needed for flood control, either the pond can be enlarged, or an additional dry pond can be located immediately north of the road crossing of the drainageway upstream of the wet pond.

The normal pool elevation of the pond is at 5 ft, about 4 ft below the ground elevation, with an overall pond excavation of 9 ft. The pond is created by a combination of excavation and a downstream embankment. Accessible forebays are located near each of the flow entrance locations to encourage pre-settling of larger sediment in restricted areas. A safety ledge 6–12 in. underwater also extends out 3–10 ft around the pond perimeter and is planted with a thick stand of emerging vegetation to restrict access to deep water. The edge of the pond along the water is also planted with appropriate vegetation as a barrier. Perimeter plantings also discourage nuisance geese populations. A boardwalk extends through this perimeter vegetation at selected locations for access for demonstration purposes. This boardwalk is also connected with the path system through the industrial park that connects other points of interest for recreational use by site workers.

When modeling the pond, the particle size distribution was assumed to have a median particle size of about 20 µm, with 90% of the particles (by mass) less than 250 µm in diameter. A 4-ft-high 30^o v-notch weir 5 ft off the pond bottom was used for water quality control. The emergency spillway was a 50-ft-long broad-crested weir, having a 3 ft width, with 1 ft of freeboard. The same evaporation rates used for the biofilters were also used for the ponds.

Calculated Performance of Stormwater Control Options

A typical Huntsville rain year (1976) was used in this analysis. This year had 102 recorded rains ranging from 0.01 to 3.70 in. The total rain recorded was 53.4 in and the average rain depth was 0.52 in.

Utility Functions for and Tradeoffs between the Different Attributes

The utility functions and tradeoffs between the different attributes are highly dependent on the local goals and regulations that need to be addressed in a stormwater management program. The following discussion describes several alternative goals for a hypothetical situation and how the attributes for each option can be evaluated.

Single Absolute Goal/Limit at Least Cost

In some cases, a watershed analysis might have been completed that recognizes the critical pollutants and set removal goals. This would especially be relevant for areas attempting to address retrofitting stormwater controls in areas already developed. For new developments, some areas might require an 80% reduction in suspended solids, compared to traditional development. If this was the case, the utility functions for particulate solids would be easily defined as being zero for outcomes that do not meet the reduction goal, and one for outcomes that do meet the reduction goal. The ranking of the options would simply be based on examining only those options that meet this simple goal, possibly by cost of implementation. In this example, outcomes for eight stormwater control programs made up of combinations of the different stormwater controls are shown on Table 99.

Table 99. Suspended solids reduction goals and costs (values in italics meet the numeric criterion of 80% TSS goals)

Stormwater treatment option	Total annual cost ($/y)	Reduction in SS Yield (%)	Meet 80% particulate solids reduction goal?	Rank based on annual cost
Option 1, Pond	83,364	*86*	Yes	5
Option 2, Regional Swale	30,008	55	No	n/a
Option 3, Site Biofilter	69,710	1	No	n/a
Option 4, Half-sized pond	74,439	73	No	n/a
Option 5 Pond and reg. swale	49,142	*94*	Yes	3
Option 6 Pond, reg. swale and biofilter	54,622	*97*	Yes	4
Option 7 Small pond and reg. swale	40,217	*90*	Yes	1
Option 8 Small pond, reg. swale and biofilter	45,698	*94*	Yes	2

Therefore, using a small pond in conjunction with a regional swale would be the cheapest option to meet the reduction goal of 80% particulate solids removal. The most costly option to meet the particulate solids removal goal is to use a pond with a conventional storm drainage system, at about twice the expected annual cost. In this example, no other attributes of the different stormwater management options are considered. This solution simply meets the single goal at the least cost. In fact, it exceeds the goal (90% TSS removals exceeding the 80% minimum goal). It would therefore be worthwhile to examine slightly smaller ponds that will more closely meet the single target, with some additional cost savings for the pond construction. The simple ranking method shown in this example would also apply for any other situation where there is a single goal that must be met at the least total cost.

Several Absolute Goals/Limits

When more than one absolute goal is required to be met, the analysis becomes only slightly more complex. It is still relatively simple with absolute goals; the first step is to filter out the options that do not meet all the required goals. This situation can occur when water quality numeric standards must be met. As an example, assume that the hypothetical effluent concentration limits shown in Table 100 must be met. The attribute table shows only the flow-weighted concentrations. If standards need to be met for all rains with a specific recurrence probability, those concentrations can be summarized from the probability distributions of outfall concentrations that WinSLAMM can calculate.

Table 100. Options and specific criteria (values in italics meet numeric criteria)

	Total annual cost ($/y)	SS conc. (mg/L)	Part. P conc. (mg/L)	Zn conc. (µg/L)	Meets all numeric standards?	Rank based on annual cost
Hypothetical Numeric Limits:		< 50 mg/L	< 0.2 mg/L	< 400 µg/L		
Option 1-Pond	83,364	*30*	*0.073*	*128*	Yes	6
Option 2 Regional Swale	30,008	178	0.43	*390*	No	n/a
Option 3 Site Biofilter	69,710	408	1.0	696	No	n/a
Option 4 Half-sized pond	74,439	*48*	*0.12*	*151*	Yes	5
Option 5 Pond and reg. swale	49,142	*23*	*0.057*	*203*	Yes	3
Option 6 Pond, reg. swale and biofilter	54,622	*29*	*0.073*	*386*	Yes	4
Option 7 Small pond and reg. swale	40,217	*39*	*0.095*	*220*	Yes	1
Option 8 Small pond, reg. swale and biofilter	45,698	53	*0.13*	*390*	Yes	2

Again, simple filtering enables the suitable options to be identified, and these can be ranked on the basis of their annual cost to identify the least costly option that meets the applicable numeric standards (option 7 again is the least costly option that meets all three hypothetical goals).

Combinations of Goals/Limits

Things get more complicated as the goals become more involved. In such situations, a more formal decision analysis approach might be worthwhile, possibly as described previously following the Keeney and Raiffa (1976) methods. The goals can be separated into classes:

(i) Specific criteria or limits that must be met. As in the above examples, it is possible to simply filter out (remove) the options that do not meet all the absolutely required criteria. If the options remaining are too few, or otherwise not very satisfying, it might be desirable to continue to explore additional options. The above examples considered combinations of only three types of stormwater control devices, for example. Many others could also be explored. If the options that meet the absolute criteria look interesting and encouraging, it is possible to continue to the next steps. Options 1, 5, 6, 7, and 8 are the five remaining options, after the specific criteria listed above are met.

(ii) Goals that are not absolute. In such a case, utility curves and tradeoffs can be developed for the remaining attributes. The above example includes attributes of several types:

- Costs
- Land requirements
- Runoff volume (volumes, habitat responses, and rates)
- Particulate solids (reductions, yields and concentrations)
- Particulate phosphorus (concentrations)
- Total zinc (concentrations)

In this example, the particulate solids reductions, suspended solids concentrations, particulate phosphorus concentrations, and total zinc concentrations are assumed to have absolute criteria, and only those options that meet them will be further considered. This leaves the attributes, shown in Table 101, that need tradeoffs and utility curves. The rankings and tradeoffs shown on Table 101 were selected for the attributes on the basis of their assumed importance for this project site. These tradeoffs could be expected to vary for different decision makers and other interested parties. Separate analyses can therefore be conducted for each different set of tradeoffs, resulting in slightly different, but hopefully similar, rankings of the options. As noted above, these tradeoffs can be mathematically determined, basically by determining the expected improvements in each attribute for a specific increase in expenditures, and then by solving the set of simultaneous equations. They can also be rather arbitrarily selected, as in this example, by assigning the rankings and values to each attribute so the resultant tradeoff values are summed to equal 1.0.

Table 101. Ranges of attributes for pre-screened options

Attribute	Range of attribute value for acceptable options	Attribute ranks for selection (after absolute goals are met)	Tradeoffs between remaining attributes
Total annual cost ($/year)	$40,217 to $83,364	2	0.20
Land needs (acres)	2.3 to 4.5	5	0.08
Rv	0.06 to 0.29	1	0.30
% of time flow > 1 cfs	0.5% to 4 %	7	0.05
% of time flow > 10 cfs	0% to 0.05 %	3	0.18
Particulate solids yield (lbs/y)	2,183 to 10,192	6	0.07
Part. Phosphorus yield (lbs/y)	5.5 to 25	4	0.12
			Sum = 1.0

The utility curve values for these attributes are shown below. For the flow rates and volumetric runoff coefficients, site conditions and local receiving waters enabled groupings of the attribute values into categories having specific utility values. The best categories were intended to protect the receiving water aquatic habitat by minimizing sediment scour and stream enlargement, whereas the poorest categories would be associated with conventional development practices that frequently are associated with severe receiving water problems. The flow rate groupings are very specific to the site, based on local hydrology and hydrologic calculations; the Rv groupings might be more generally applicable. The other utility curves (for cost, phosphorus yield, land needs, and particulate solids yields) are simple straight line relationships, with the best attribute values obtained for the different options assigned a value of 1.0, and the worst attribute values obtained assigned a value of 0.0. Intermediate values are simply interpolated between these extreme values.

- Volumetric runoff coefficient (Rv) as an indicator of habitat quality and aquatic biology stress:

Attribute value	Expected habitat condition	Utility value
< 0.1	Good	1.0
0.1–0.25	Fair	0.75
0.26–0.50	Poor	0.25
0.51–1.0	Very poor	0

- Total annual cost: straight line, with \$83,364 = 0 and \$40,217 = 1.0.
- % of time flow > 10 cfs:

% of time flow > 10 cfs	Utility value
< 0.05	1.0
0.05–1	0.75
1.1–2.5	0.25
> 2.5	0

- Part. phosphorus yield (lbs/y): straight line, with 25 lbs/y = 0 and 5.5 lbs/y = 1.0
- Land needs (acres): straight line, with 4.5 acres = 0 and 2.3 acres = 1.0
- Particulate solids yield (lbs/y): straight line, with 10,192 lbs/y = 0 and 2,183 lbs/y = 1.0
- % of time flow >1 cfs:

% of time flow > 1 cfs	Utility value
< 1	1.0
1–3	0.75
3.1–10	0.25
> 10	0

Calculation of Utilities and Ranking of Alternative Stormwater Management Programs

At this site, most of the particulate solids originate from the undeveloped areas, so the site biofilters have minimal benefits on reducing the overall particulate solids discharges. Also, the site biofilters infiltrate water having much lower particulate concentrations compared to the undeveloped areas (to minimize clogging), so the resulting outfall concentrations actually increase. The regional swale and detention ponds treat all the site water, so they have a much larger benefit on the particulate solids.

Tables 102 and 103 show the calculated utility factors for each option, along with the sums of the factors and the overall ranking of the options. Option 8, the small pond with the regional swale and the on-site biofilter swale was ranked significantly ahead of the other options. Options 5 (large pond and regional swale) and 7 (small pond and regional swale) ranked next and were basically tied. Option 1, the large pond alone, ranked far below the other options. The factors are calculated by multiplying the utilities by the tradeoff values. As an example, for Option 5, the cost tradeoff was 0.20 and the cost utility was 0.79, and the calculated cost factor is therefore 0.20 x 0.79 = 0.158. The sum of factors is the sum of the individual factors for all attributes for each option. The ranks are based on the sum of factors, with the largest sum of factors ranked 1.

Table 102. Utility and tradeoffs for different options

Stormwater control option	Volumetric runoff coefficient (Rv)	Rv utility	% of time flow > 1 cfs	Mod flow utility	% of time flow > 10 cfs	High flow utility
Tradeoff Value	0.30	0.30	0.05	0.05	0.18	0.18
Option 1 Pond	0.29	0.25	4	0.25	0.05	0.75
Option 5 Pond and reg. swale	0.15	0.75	2	0.75	0	1.0
Option 6 Pond, reg. swale and biofilter	0.06	1.0	0.5	1.0	0	1.0
Option 7 Small pond and reg. swale	0.15	0.75	2	0.75	0.05	0.75
Option 8 Small pond, reg. swale and biofilter	0.07	1.0	0.8	1.0	0	1.0

Table 102. Utility and tradeoffs for different options (continued)

Stormwater control option	Total annual cost ($/yr)	Cost utility	Land needs for SW mgt (acres)	Land utility	Part. solids yield (lbs/yr)	Part. solids utility	Part. phos. yield (lbs/yr)	Phos. utility
Tradeoff value	0.20	0.20	0.08	0.08	0.07	0.07	0.12	0.12
Option 1 Pond	83,364	0	4.5	0	10,192	0	25	0
Option 5 Pond and reg. swale	49,142	0.79	4.5	0	4,133	0.76	10	0.77
Option 6 Pond, reg. swale and biofilter	54,622	0.67	4.5	0	2,183	1.0	5.5	1.0
Option 7 Small pond and reg. swale	40,217	1	2.3	1	6,937	0.41	17	0.41
Option 8 Small pond, reg. swale and biofilter	45,698	0.87	2.3	1	4,125	0.76	10	0.77

Table 103. Calculations of ranks for different stormwater management options

Stormwater control option	Rv utility	Rv factor	Mod flow utility	Mod flow factor	High flow utility	High flow factor	Sum of factors	Overall rank
Tradeoff value	0.30		0.05		0.18			
Option 1 Pond	0.25	0.075	0.25	0.0125	0.75	0.135	0.2225	5
Option 5 Pond and reg. swale	0.75	0.225	0.75	0.0375	1.0	0.18	0.7455	4
Option 6 Pond, reg. swale and biofilter	1.0	0.30	1.0	0.05	1.0	0.18	0.8540	2
Option 7 Small pond and reg. swale	0.75	0.225	0.75	0.0375	0.75	0.135	0.7555	3
Option 8 Small pond, reg. swale and biofilter	1.0	0.30	1.0	0.05	1.0	0.18	0.9290	1

Table 103. Calculations of ranks for different stormwater management options (continued)

Stormwater Control Option	Cost utility	Cost factor	Land utility	Land factor	Part. utility	Part. factor	Phos. utility	Phos factor
Tradeoff value	0.20		0.08		0.07		0.12	
Option 1 Pond	0	0	0	0	0	0	0	0
Option 5 Pond and reg. swale	0.79	0.158	0	0	0.76	0.053	0.77	0.092
Option 6 Pond, reg. swale and biofilter	0.67	0.134	0	0	1.0	0.07	1.0	0.12
Option 7 Small pond and reg. swale	1	0.20	1	0.08	0.41	0.029	0.41	0.049
Option 8 Small pond, reg. swale and biofilter	0.87	0.174	1	0.08	0.76	0.053	0.77	0.092

Summary of Decision Analysis Methods to Assist in the Selection of Stormwater Control Programs

Stormwater quality models can produce copious amounts of information for large numbers of alternative management programs that contain a wide variety of individual stormwater control practices, as described by Pitt and Clark (2008). In most cases, just a few of the values are sufficient for quick comparisons. These include the overall percent runoff and particulate solids reductions, the final Rv and runoff volume, and the resulting particulate solids yields and concentrations. WinSLAMM also calculates the life-cycle costs and the expected habitat conditions of the receiving waters to be compared, in addition to flow-duration information. The use of decision analysis procedures, based on methods developed by Keeney and Raiffa (1976) with the WinSLAMM batch processor allows semi-automatic formal evaluations of alternative stormwater control programs considering multiple conflicting objectives.

This decision analysis approach has the flexibility of allowing for variable levels of analytical depth, depending on the problem requirements. The preliminary level of defining the problem explicitly in terms of attributes often serves to make the most preferred alternatives clear. The next level of analysis might consist of a first-cut assessment and ranking. Several different utility function curve types can be used with a simple additive model. Spreadsheet calculations with such a model are easily performed, making it possible to conduct several decision analysis evaluations using different tradeoffs, representing different viewpoints. It is possible there will be a small set of options that everyone agrees are the best choices. Also, this procedure documents the process for later discussion and review. Sensitivity analyses can also be conducted to identify the most significant factors that affect the decisions. The deepest level of analysis can use all the analytical information one collects, such as probabilistic forecasts for each of the alternatives and the preferences of experts over the range of individual attributes. Monte Carlo options available in WinSLAMM can also be used that consider the uncertainties in the calculated attributes for each option.

Therefore, decision analysis has several important advantages. It is very explicit in specifying tradeoffs, objectives, alternatives, and sensitivity of changes to the results. It is theoretically sound in its treatment of tradeoffs and uncertainty. Other methods ignore uncertainty and often rank attributes in importance without regard to their ranges in the problem. This decision analysis procedure can be implemented flexibly with varying degrees of analytical depth, depending on the requirements of the problem and the available resources.

9. Conclusions

Major Project Findings

Much information was collected during this demonstration project monitoring the benefits of green infrastructure on combined sewer flows. The following briefly summarizes some of the major findings.

Monitored Benefits of Green Infrastructure Facilities

A total of 75 events were examined for the pre-construction baseline conditions and 37 events were examined for the post-construction conditions. Figure 148 compares the pre and post-construction Rv values for the test/pilot watershed as monitored at UMKC01. The post-construction Rv values are apparently smaller than the pre-construction Rv values, as expected. Comparisons of pre and post-construction Rv values were also made for different rain categories which had similar apparent trends, especially for the small rains. The RV differences for the two smallest rain categories were significantly smaller (which were at 40 and 33% reductions for <0.5 inches and 0.5 to 1.5 inches respectively), but the largest rain category (>1.5 inches) did not have significant differences (at 13%) for the number of observations available. The overall reduction in flows observed in the test/pilot watershed were calculated to be about 32% on a flow-weighted basis and were highly significant ($p < 0.001$). The annual ruoff volume directed to the green infrastructure facilities was calculated to be about half of the total test/pilot area annual runoff. The uncontrolled runoff was associated with areas that could not be directed to the controls or where control could not be constructed (mainly on private property or due to infrastructure interferences). The maximum control that could have been caused by the stormwater controls was therefore about 50%. The detected 32% was likely associated with inaccuracies in the flow monitoring at the challenging locations, or due to under-sized facilities in some locations.

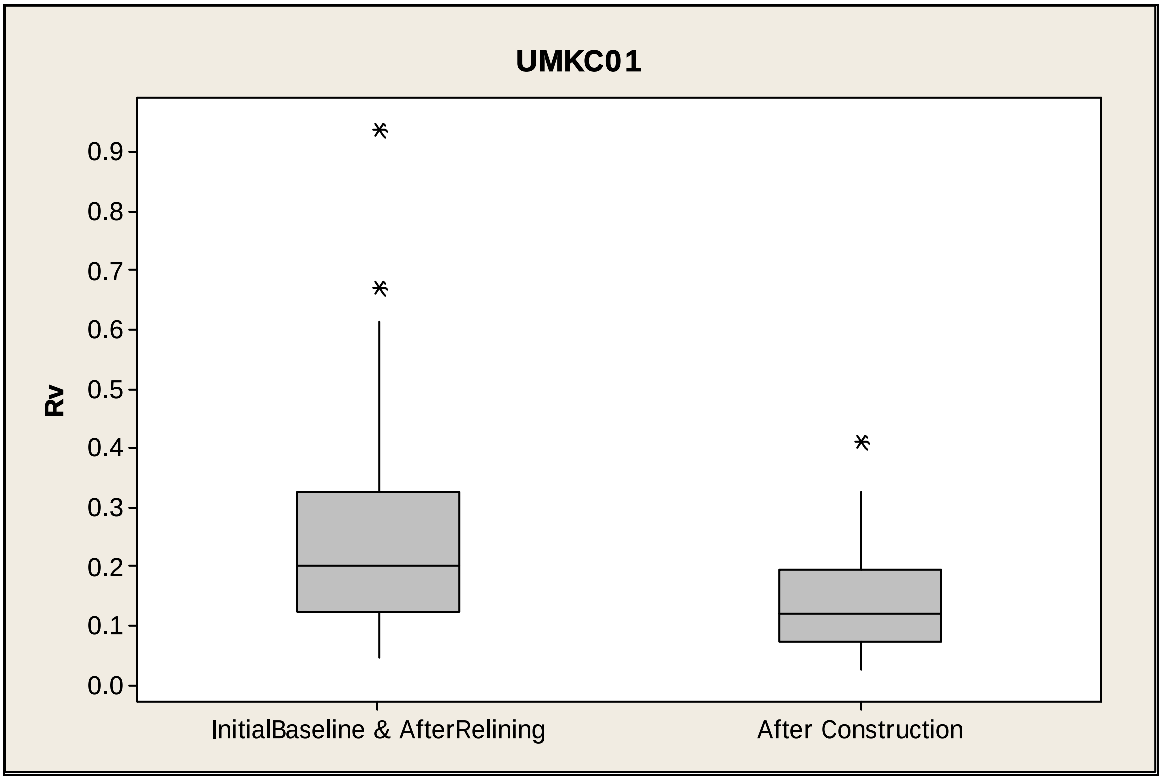


Figure 148. Volumetric runoff coefficients at UNKC01 before and after GI construction.

In addition to the large-scale test and control area flow monitoring, eight curb-side biofilters were also monitored along with two rain gardens. The infiltration rates in these devices were about 0.6 in/hr for several that had poorer performing media, while the others had median infiltration rates of about 2.5 in/hr (all measured in real-time during actual rains). The maximum ponding depths in the biofilters were about 2.5 inches or 6.5 inches, depending on the data grouping. The average inlet SSC concentrations ranged from about 130 to 200 mg/L for the eight curb-side biofilters that were monitored, with a median particle size of about 30 µm. During the post-construction monitoring period, 49 influent samples were collected, while only 4 samples were available for overflowing effluent water; the other events all completely infiltrated the inflowing water. Six samples were also available from the SmartDrain underdrain at two installations. The samples from the underdrains indicated that the SSC concentrations under the biofilter media were about 65% less than the influent SSC concentrations. The reductions would have been greater if the fines in the biofilter media were not rinsed into the water during the storms. About 80% of the events had complete control due to infiltration. The overflowing effluent water had no statistically significant SSC reductions. During monitoring of two roof runoff rain gardens over 1 to 2 years, no overflowing runoff was ever detected, indicating 100% reductions for these controls.

Modeling the Benefits of Green Infrastructure Facilities and Resulting Production Functions

WinSLAMM was calibrated using the small-scale and large-scale monitoring results and then used to produce production functions for various stormwater controls for the Kansas City area. WinSLAMM has been undergoing development and changes since the mid-1970s and now includes a wide range of options. Over the years, periodic major upgrades have occurred to take advantage of advancing computer capabilities and knowledge gained through stormwater research, and to respond to requests by users.

The expected major sources of runoff from the test area vary for different rain depth categories. A detailed land survey found that most of the homes in the test watershed already have disconnected roofs (85% of all roof areas) and that the total roof areas account for 13% of the total study area. The directly connected roofs, which make up only 2% of the study area, contribute 6% of the total annual flows. The disconnected roofs, which constitute 11% of the area, contribute 7% of the total flows. Thus, complete control of the runoff from the directly connected roofs would reduce the total area runoff by only a very small amount, less than can be reliably detected by monitoring the total runoff from the area. The modeling calculations illustrate the different effects of using rain gardens, rain barrels or tanks, or simple disconnections of the directly connected roofs. The results are presented on the basis of the effects for the directly connected roofs alone; if calculated for the whole drainage area, the contribution would be less than 5%. If all the roofs were directly connected, they would then contribute 30% of the annual flows, and the outfall consequences for the entire area from these roof controls would be substantially larger.

Performance plots were prepared comparing the size of rain gardens to the roof areas to result in expected roof runoff flow reductions. Rain gardens that are 20% of the roof areas are expected to result in about 90% reductions of the total annual flow compared to directly connected roofs. This rain garden size is about 200 ft^2/house (about 20 m^2/house) which could, for example, be composed of several smaller rain gardens each located at a downspout. Reductions of 50% in the total annual flows could be obtained if the total rain garden area per house was 7% of the roof area.

Rain barrel effectiveness is related to the need for supplemental irrigation and how that matches the rains for each season, or the use of water-resistant plants. The continuous simulations used a typical 1-year rain series and average monthly ET values for varying amounts of roof runoff storage. One 35-gal (133 L) rain barrel is expected to reduce the total annual runoff by 24% from the directly connected roofs, if the water use can be closely regulated to match the irrigation requirements. If four rain barrels were used (such as one on each corner of a house and receiving runoff from separate roof downspouts), the total annual roof volume reductions could be as high as 40%. Larger storage quantities result in increased usage but likely require larger water tanks. A small tank 5 ft (1.5 m) diameter and 6 ft (1.8 m) high is expected to result in 75% total annual runoff reductions, while a larger 10 ft (3 m) diameter tank 6 ft (1.8 m) tall would approach complete roof runoff control.

Using rain barrels and rain gardens together at a home is more effective than using either method alone: the rain barrels would overflow into the rain gardens, so their irrigation use is not quite as critical. To obtain reductions of 90% in the total annual runoff, it is necessary to have at least one rain garden/house, unless the number of rain barrels more than 25 (or one small water tank)/house. In such a case, the rain gardens can be reduced to 80 ft^2/house (7 m^2/house).

The Use of Decision Analysis to Select the Most Appropriate Stormwater Control Program

The best combination of control options is not necessarily obvious. The CSO control program must meet permit requirements, which specify certain amounts of upland storage in the watershed. Other elements, including costs, aesthetics, improvements to streetside infrastructure, and other potential benefits, must also need to be considered in a decision analysis framework. Caution is needed when comparing the amount of site runoff storage provided by these upland controls to the total storage goals to meet the objective of the CSO control program (288,000 gal). As an example, storage provided at directly connected roofs needs to be discounted by a factor of about 1.4 because not all the storage is available during all rains, and because their drainage is influenced by low infiltration rates through the native soils, compared to flow controls directly connected to the combined sewers. In addition, the curb-cut biofilters also have access to almost all the flows in the area, so their storage volumes are more effectively used. More significantly, if storage was provided at roofs that are already disconnected, their storage volumes would need to be discounted by a factor of 4.5 when compared to the total site storage goals because of the existing infiltration already occurring from the disconnected roofs.

Cost-effective designs of biofilters for the area can be identified by examining the production functions provided in this report. For slowly infiltrating native subsoils (less than 1 in/hr), the use of additional subsurface storage and restricted underdrains can be very beneficial. For higher rate soils, these features have minimal benefit on performance. The biofilters being about 1.5 to 2% of the drainage area in the residential area are expected to provide about 90% long-term reductions in stormwater runoff to the combined sewer for the areas treated. However, only about half of the test (pilot) watershed received runoff control, so the overall runoff volume reduction benefit is expected to be about 40 to 50%. Subsurface drainage water from the biofilters undergoes substantial retention (several hours) which would benefit peak combined sewer flows, but the volume affected is relatively small.

Recommendations for Effective Monitoring of Green Infrastructure Effects

Monitoring urban runoff flows, especially in combined sewer systems presents many challenges, especially retrofitted into an existing development. During this project, several issues became apparent that caused greater uncertainty in some of the monitoring data than expected. The following lists some recommendations that should be considered in monitoring for determining the benefits of green infrastructure controls that addresses some of these confounding issues:

• Excellent as-built sewerage system data is needed. Over the years, remedial construction occurred in the test and control areas to address local drainage problems. Typically, these included the use of yard drains constructed in the backyards in low-laying areas that collected runoff for extended periods. These yard drains were extended to the combined sewers. Other changes may have been made to provide parallel drainage lines in the area. It was not clear where these were connected to the monitored drainage network or if they were even still in operation. The sewer relining project that occurred at the end of the initial baseline monitoring also affected the drainage characteristics of the area. These can have major effects on the calculated drainage area above the monitoring locations and the sewage flow characteristics. Better as-built maps or detailed surveys are necessary to provide the most accurate analyses of the data.

• Low flows are especially challenging to monitor, especially if the water depths are very small in relation to the pipe diameters, as occurred at some of the monitoring locations during this project. Initial calculations for expected flow conditions may have been affected by watershed area uncertainties and the flows were smaller in the control area than expected. The use of sensors that are more accurate for low flows (such as acoustical depth sensors mounted on the pipe crowns) could be used along with the area-velocity sensors that had pressure depth sensors to provide redundancy. Since the flows were all low at the control area monitoring locations, it would have been feasible to install flumes in the pipes for more accurate discharge measurements without causing significant flow restrictions during peak flow conditions. Some cities install temporary plywood v-notch weirs in manholes where the flow capacity is very large without restricting peak flows.

• Some of the flow monitoring instruments experienced pre-mature and frequent failures that went undetected. This may have been due to the monitored combined sewage flows resulting in debris on the sensors. Frequent data collections and reviews, along with the availability of replacement (or redundant) sensors would decrease data gaps.

• Numbers of events to be monitored need to be adequate to detect the expected flow reductions considering the variability in the data and the data quality objectives (confidence and power). The data collected during this project had greater variability than expected, but the multiple monitoring locations and scales, along with the extended post-construction monitoring period allowed effective analyses and conclusions for the most important project objectives.

• The excellent cooperation and participation of the Kansas City Water Services Department (KCWSD) and personnel from other agencies was critical for the successful completion of this project. A high level of involvement and cooperation, as received, is very important for these types of complex projects.

Considerations That Affect Use of Different Stormwater Controls

Certain site conditions could restrict the applicability of some of these controls. The following comments are mostly summarized from Pitt et al. (2008a) and from preliminary research reported by others at recent technical conferences.

Sodium Adsorption Ratio (SAR)

The SAR can radically degrade the performance of an infiltration device, especially when clays are in the media or underlying soils. Media or soils with an excess of sodium ions, compared to calcium and magnesium ions, remain in a dispersed condition, and are almost impermeable to rain or applied water. A *dispersed* soil is extremely sticky when wet, tends to crust, and becomes very hard and cloddy when dry. Water infiltration is therefore severely restricted. Dispersion caused by sodium can result in poor physical soil conditions and water and air do not readily move through the soil. An SAR value of 15 or greater indicates that an excess of sodium will be adsorbed by the soil clay particles. This can cause the soil to be hard and cloddy when dry, to crust badly, and to take water very slowly. SAR values near 5 can also cause problems, depending on the type of clay present. Montmorillonite, vermiculite, illite, and mica-derived clays are more sensitive to sodium than other clays. Additions of gypsum (calcium sulfate) to the soil can be used to free the sodium and allow it to be leached from the soil in some situations, but recent laboratory tests with biofilter media at UA indicate minimal improvement.

The SAR is calculated by using the concentrations of sodium, calcium, and magnesium (in meq) in the following formula:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{(Ca^{+2} + Mg^{+2})}{2}}}$$

SAR has been documented to be causing premature failures of biofiltration devices in northern communities, such as several in the Madison, Wisconsin, area documented by University of Wisconsin soil science student projects. These failures occur when snowmelt water is allowed to enter a biofilter that has clay in the soil mixture. To minimize this failure potential, the following are recommended:

1. Do not allow snowmelt water to enter a biofilter unit. As an example, roof runoff likely has little salt and SAR problems seldom occur for roof runoff rain gardens, even in areas having large amounts of clay in the soil. However, if driveway or walkway runoff waters affected by saline deicing chemicals are discharged to these devices, problems can occur. The largest problem is associated with curb-cut biofilters or parking lot

biofilters in areas with snowmelt entering these devices, especially if clay is in the engineered backfill soil/media.

2. The biofilter media should not have any clay. It appears that even a small percent of clay in the media can cause a problem, but little information is available on the tolerable clay content of biofilter soils. Some biofilter guidance documents recommend an appreciable clay content to slow the water infiltration rate (and therefore increase the hydraulic detention time in the system) to improve pollutant capture. Instead of clay used to control the infiltration rates, restrictive underdrains, such as the SmartDrain^{TM}, should be used. Guidance documents recommending fines in the biofilter mixture are usually from areas having mild climates with little or no snowmelt (and deicing chemical use).
3. The most robust engineered soil mixtures used in biofilters tend to be mixtures of sand and an organic material (such as compost, if nutrient leaching is not a concern, or Canadian peat for a more stable material having little nutrient leaching potential). Other mixtures of biofilter media can be used targeting specific pollutants, but these are usually expensive and likely only appropriate for special applications.
4. If a suitable soil mixture not having clay (should be less than 3% based on preliminary information), and if snowmelt water will affect the system, biofilters should not be used in the area. As noted above, rain gardens receiving only roof runoff might be suitable in most situations because of the absence of excessive sodium in the runoff water.

The Kansas City biofilter media has a clay content of about 10% which is on the high side. This amount of clay may affect the infiltration rates during snowmelt periods and for some time after if sodium accumulations occur.

Clogging of Infiltration Devices

The designs of infiltration devices need to be checked on the basis of their clogging potential. For example, a relatively small and highly efficient biofilter (especially in an area having a high native infiltrating rate) could capture a large amount of sediment. Having a small surface area, this sediment would accumulate rapidly over the area, possibly reaching a critical clogging load early in its design lifetime. Therefore, the clogging potential can be calculated according to the predicted annual discharge of suspended solids to the biofiltration device and the desired media replacement interval. Infiltration and bioretention devices might show significantly reduced infiltration rates after about 2 to 5 lb/ft^2 (10 to 25 kg/m^2) of particulate solids have been loaded (Clark 1996, 2000; Urbonas 1999). Deeply rooted vegetation and a healthy soil structure can extend the actual life much longer. However, abuse (especially compaction and excessive siltation) can significantly reduce the life of the system. If this critical load accumulates relatively slowly (taking about 10 or more years to reach this total load) and if healthy vegetation with deep roots are present, the infiltration rate might not significantly degrade because of the plant's activities in incorporating the imported sediment into the soil column. If this critical load accumulates in just a few years, or if healthy vegetation is not present, the premature failure from clogging can occur. Therefore, relatively large surface areas might be necessary in areas having large sediment content in the runoff, or suitable pretreatment to reduce the sediment load before entering the biofilter or infiltration device would be necessary.

For some of the calculated Kansas City biofilter size options, the sediment loading rates are high (mostly because of treatment of relatively large areas compared to the size of the biofilters), which could result in premature failure if the minimum sizes were used according to infiltration goals alone. Therefore, a larger area might actually be needed to prevent premature failure from clogging. The following considerations apply to infiltration/biofiltration devices to minimize clogging failure:

1. Use a sufficient infiltration area to enable at least 10 years before the critical sediment loading (10 to 25 kg/m^2) occurs, and maintain a healthy, deep-rooted plant community to incorporate the sediment into the soil horizon.
2. Use pretreatment to reduce the sediment load entering a biofilter to reduce the TSS concentrations to match the desired maintenance or clogging interval. Using a grass filter/grass swale before a biofilter can significantly reduce the loading to the device, extending the operational life.

The characteristics for the Kansas City biofilters in the test area indicate that most are likely sufficiently sized to result in minimal clogging potential. However, there might be a desire to reduce the sizes appreciably during future construction to reduce costs, which could result in early failure.

Groundwater Contamination Potential and Over-Irrigation

The potential for infiltrating stormwater to contaminate groundwater is dependent on the concentrations of the contaminants in the infiltrating stormwater and how effective those contaminants might travel through the soils and vadose zone to the groundwater. Source stormwater from residential areas are not likely to be contaminated with compounds having significant groundwater contaminating potential (with the exception of high-salinity snowmelt). In contrast, commercial and industrial areas are likely to have greater concentrations of contaminants of concern that might adversely affect the groundwater. Therefore, pretreatment of the stormwater before infiltration might be necessary, or treatment media can be used in a biofilter or as a soil amendment to hinder the migration of the stormwater contaminants of concern to the groundwater. Again, these concerns are usually more of a problem in industrial and commercial areas than in residential areas.

Pitt et al. (2010a) summarized prior research on potential groundwater contamination. Table 104 can be used for initial estimates of contamination potential of stormwater affecting groundwater. This table includes likely worst-case mobility conditions using sandy soils having low organic content. If the soil is clayey or has a high organic content, or both, most of the organic compounds would have less mobility than shown. The abundance and filterable fraction information is generally applicable for warm-weather stormwater runoff at residential and commercial area outfalls. The concentrations and detection frequencies would likely be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas), with greater groundwater contamination potential.

Table 104. Groundwater contamination potential for stormwater pollutants post-treatment

Compound class	Compounds	Subsurface injection with minimal pretreatment	Surface infiltration with sedimentation*	Surface infiltration and no pretreatment*
Nutrients	Nitrates	Low/moderate	Low/moderate	Low/moderate
Pesticides	2,4-D	Low	Low	Low
	γ-BHC (lindane)	Moderate	Low	Moderate
	Atrazine	Low	Low	Low
	Chlordane	Moderate	Low	Moderate
	Diazinon	Low	Low	Low
Other organics	VOCs	Low	Low	Low
	1,3-dichlorobenzene	Low	Low	**High**
	Benzo(a) anthracene	Moderate	Low	Moderate
	Bis (2-ethyl-hexyl) phthalate	Moderate	Low?	Moderate
	Fluoranthene	Moderate	Moderate	**High**
	Naphthalene	Low	Low	Low
	Phenanthrene	Moderate	Low	Moderate
	Pyrene	Moderate	Moderate	**High**
Pathogens	Enteroviruses	**High**	**High**	**High**
	Shigella	Low/moderate	Low/moderate	**High**
	P. aeruginosa	Low/moderate	Low/moderate	**High**
	Protozoa	Low	Low	**High**
Heavy metals	Cadmium	Low	Low	Low
	Chromium	Low/moderate	Low	Moderate
	Lead	Low	Low	Moderate
	Zinc	Low	Low	**High**
Salts	Chloride	**High**	**High**	**High**

Source: modified from Pitt et al. 1994

Notes: Overall contamination potential (the combination of the subfactors of mobility, abundance, and filterable fraction) is the critical influencing factor in determining whether to use infiltration at a site. The ranking of these three subfactors in assessing contamination potential depends of the type of treatment planned, if any, before infiltration.

* Even for those compounds with low contamination potential from surface infiltration, the depth to the groundwater must be considered if it is shallow (1 m or less in a sandy soil). Infiltration might be appropriate in an area with a shallow groundwater table if maintenance is sufficiently frequent to replace contaminated vadose zone soils.

Therefore, groundwater contamination potential of infiltrating stormwater can be reduced by:

1. Careful placement of the infiltrating devices and selection of the source waters. Most residential stormwater is not highly contaminated with the problematic contaminants, except for chlorides associated with snowmelt.
2. Commercial and industrial area stormwater would likely need pretreatment of reduce the potential of groundwater contamination associated with stormwater. The use of specialized media in the biofilter, or external pretreatment might be needed in these other areas.

The Kansas City test area is expected to have minimal groundwater contamination potential because it has relatively uncontaminated stormwater, and the soil has appreciable clay. However, snowmelt salts could be a problem if deicing salt use is not restricted.

Retrofitting and Availability of Land

Most of the control options being used in GI approaches to minimize combined sewer problems are retrofitted in existing urban areas. Their increased costs and availability of land can be detrimental in developing highly effective control programs. The selection and construction of stormwater controls at the time of development (rather than retrofits) is usually much more cost-effective and can provide a higher level of control. However, many controls can be retrofitted into existing areas. Practices that can usually be easily retrofitted get the most attention in stormwater management program in existing areas. Table 105 summarizes some of the problems associated with different stormwater retrofitting options in combined sewer areas.

Table 105. Retrofitting problems for different stormwater management options

Controls	Ability to retrofit	Land requirements
Roof Runoff Controls		
Rain Gardens	Easy in areas having landscaping	Part of landscaping area
Disconnections	Suitable only if the adjacent pervious area is adequate (mild slope and long travel path)	Part of landscaping area
Rain Barrels and Water Tanks	Easy if placed close to a building or underground large tanks	Supplements landscaping irrigation, no land requirements
Pavement Controls		
Disconnections	Suitable only if the adjacent pervious area is adequate (mild slope and long travel path)	Most large paved areas are not adjacent to suitable large turf areas, except for schools; no additional land requirements, but land is needed.
Biofiltration/bioinfiltration	Easy if one can rebuild parking lot islands as bioinfiltration areas; perimeter areas also possible (especially good if existing stormwater drainage system can be used to easily collect overflows)	Part of landscaped islands in parking areas, along parking area perimeters, or sacrifice some existing parking areas.
Porous Pavement	Difficult as a retrofit; must replace complete pavement system; possible if during rebuilding effort	Uses parking area
Street Side Drainage Controls		
Grass Swales	Difficult to retrofit. Suitable if existing swales are to be rebuilt.	Part of street right of way
Curb-Cut Biofilters	Difficult to retrofit, but much easier than simple swales. Usually build to work with existing drainage system. Can do extensions into parking lanes/shoulders to increase areas.	Part of street right of way, but can be major nuisance during construction and can consume street side parking. Can be used to rebuild street edge and improve aesthetics.

The range of difficulties and land requirements varies, mostly depending on available opportunities. In some communities, extensive retrofitting is occurring, including installing curb-cut biofilters, during scheduled street improvement projects. These can also be installed during scheduled repaving and sidewalk repairs that usually occur in many areas every few decades. Rain gardens are usually installed by the homeowners with no cost to the city. Many areas have organized efforts encouraging these, for example. Redevelopment and new construction periods are the most suitable times for installing many of these controls to have the least interferences with residents and for the least costs.

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APPENDICES

Modeling of Green Infrastructure Components and Large-Scale Test and Control Watersheds at Kansas City, MO

Contents

Appendix A: Monitored Biofilter Site Descriptions

1. Curb Extension with BR - 1324 E 76th St.

Figure 1 – Aerial photos, topo map, and urban classifications for device #1

Location	Urban Classification	Area (ac)	Note
Figure 1-b	Driveway	0.04524	There is no overflow from upstream as shown in Figure 1-b.
	Landscaped area	0.246	
	Roof	0.07541	
	Sidewalk	0.01603	
	Street	0.03869	
Total area (ac)		0.42137	
Figure 1-c	Driveway	0.12188	Overflow from device#1-u1 as shown in figure 1-c.
	Landscaped area	0.29362	
	Roof	0.14325	
	Sidewalk	0.03706	
	Street	0.06726	
Total area (ac)		0.66307	

1324 E 76th St #1 (sheet 305 for as-built details); no underdrains

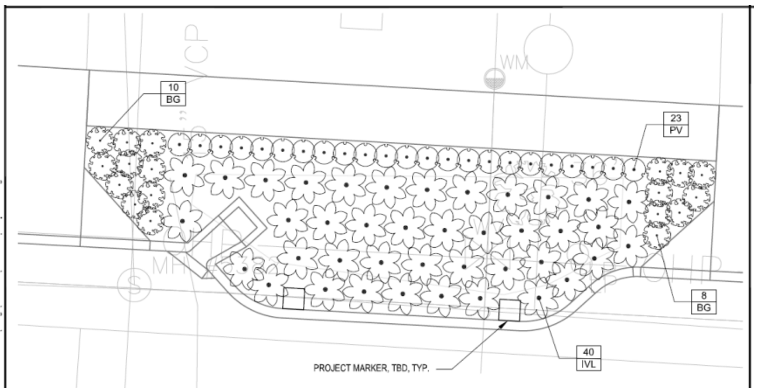
Only received flows from W along E 76th St (from driveway up)

2 samplers and 2 level recorders (inlet and bottom of garden)

Two inlet samples from small event in morning of Oct 25, 2012

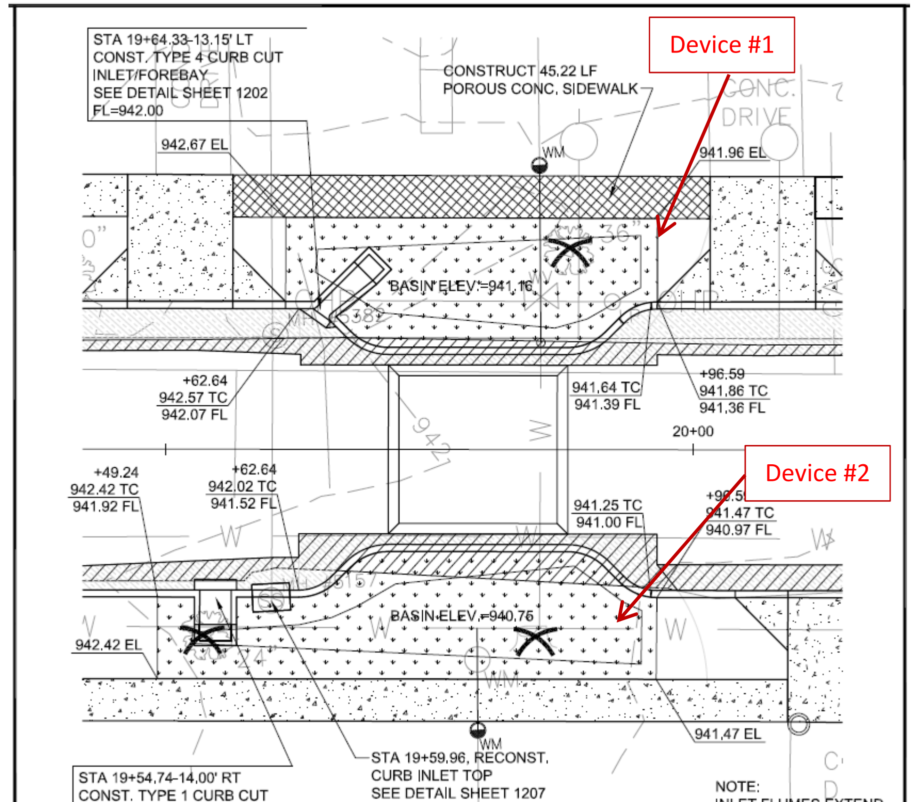
 Leaves washed into inlet		
	 Porous concrete alongside of rain garden collects yard runoff to garden	

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
<1960	Single	Adequate	2	50	50	0	Composite Shingle	Yes	1 pole	Much	Lawn	Poor	0	100	Unpaved	Poor	Rough	Poor



DETAIL 3A: CURB EXTENSION PLANTING PLAN, STA. 19+79.61 L

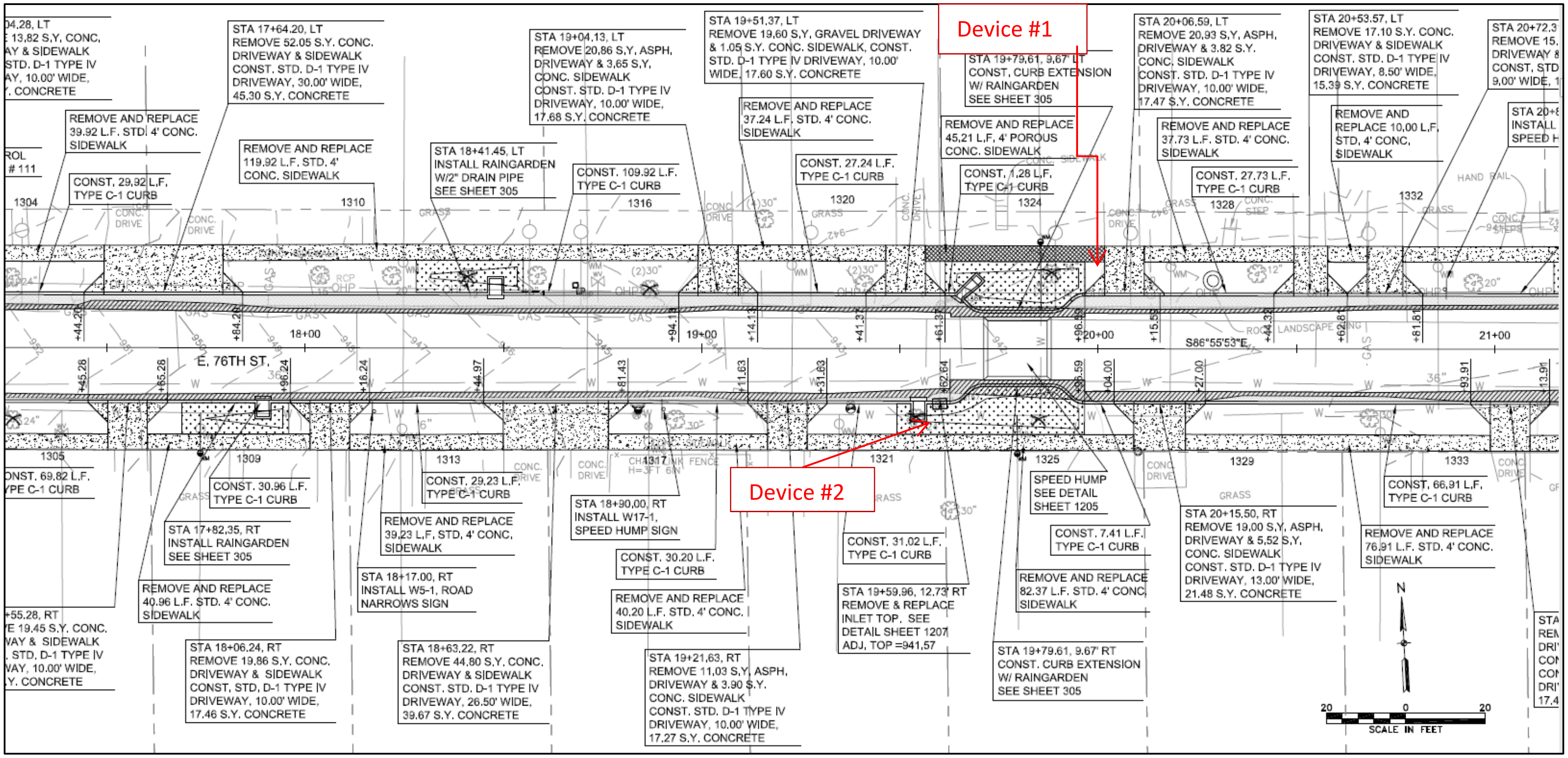
SCALE: 1" = 5'



CURB EXTENSION WITH RAINGARDEN

DETAIL 9 : STA 19+79.61 - SURFACE

SCALE: 1" = 10'


Device #1
Device #2
E. 76TH ST.
STA 19+79.61, 9.67' LT
CONST. CURB EXTENSION
W/ RAINGARDEN
SEE SHEET 305
STA 19+79.61, 9.67' RT
CONST. CURB EXTENSION
W/ RAINGARDEN
SEE SHEET 305
SPEED HUMP
SEE DETAIL
SHEET 1205
STA 18+90.00, RT
INSTALL W17-1,
SPEED HUMP SIGN
STA 18+17.00, RT
INSTALL W5-1, ROAD
NARROWS SIGN
STA 18+41.45, LT
INSTALL RAINGARDEN
W/2" DRAIN PIPE
SEE SHEET 305
STA 17+82.35, RT
INSTALL RAINGARDEN
SEE SHEET 305
STA 19+59.96, 12.73' RT
REMOVE & REPLACE
INLET TOP. SEE
DETAIL SHEET 1207
ADJ. TOP =941.57
SCALE IN FEET

2. Curb Extension with Bioretention - 1325 E 76th St.

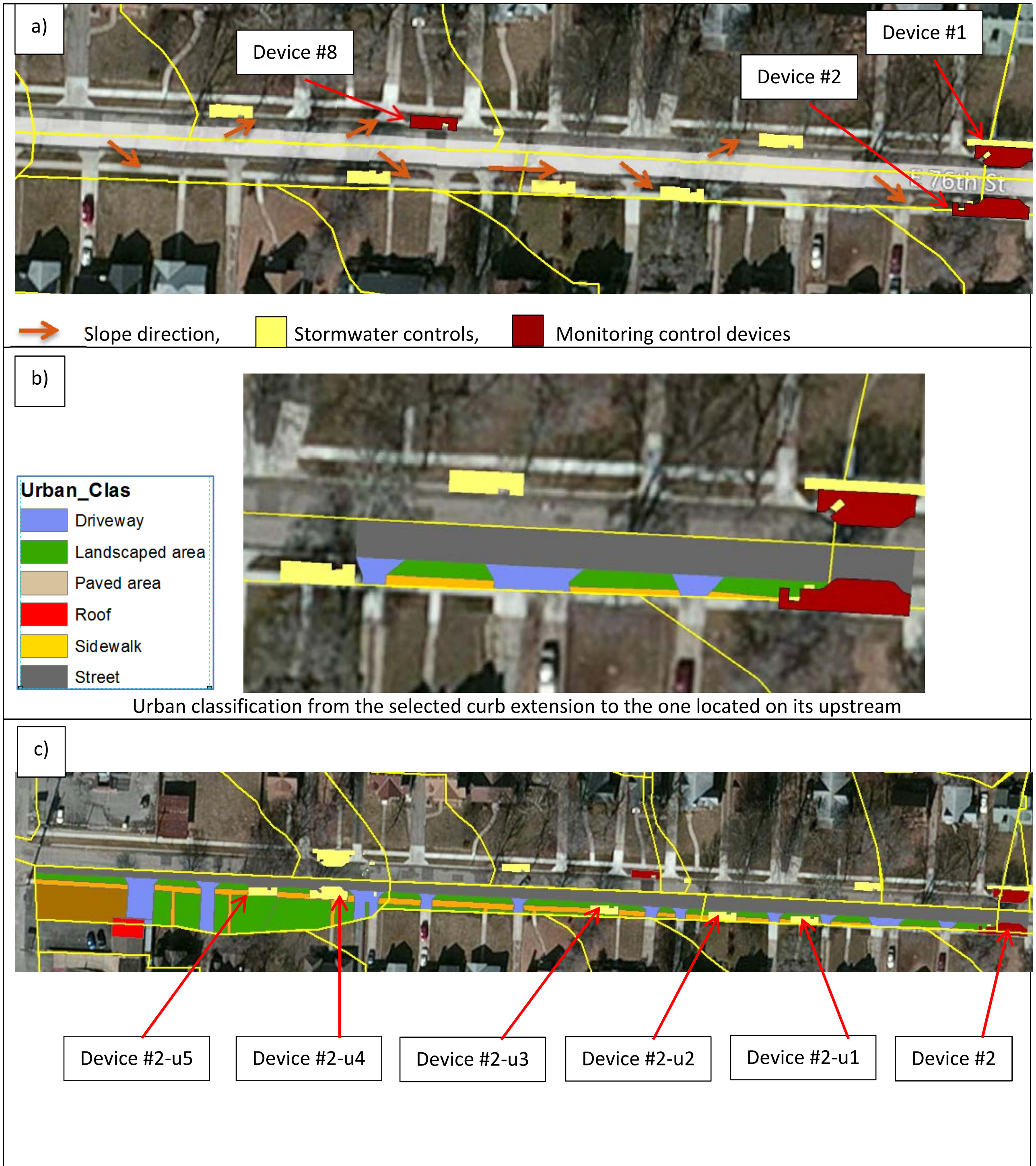


Figure 2 – Aerial photos, topo map, and urban classifications for device #2

Location	Urban Classification	Area (ac)	Note/Assumption
From downstream of device#2-u1 to device#2	Driveway	0.01137	There is no overflow from upstream.
	Sidewalk	0.00636	
	Landscaped area	0.01570	
	Street	0.06257	
Total Area (ac)		0.09600	
From downstream of device#2-u2 to device#2	Driveway	0.01420	Overflow from device#2-u1
	Sidewalk	0.01108	
	Landscaped area	0.02292	
	Street	0.08827	
Total Area (ac)		0.13647	
From downstream of device#2-u3 to device#2	Driveway	0.02054	Overflow from device#2-u1 and device#2-u2
	Sidewalk	0.01933	
	Landscaped area	0.03378	
	Street	0.12520	
Total Area (ac)		0.19885	
From upstream of device#2-u3 to device#2	Driveway	0.04243	Overflow from device#2-u1, device#2-u2, and device#2-u3
	Sidewalk	0.08235	
	Landscaped area	0.04561	
	Street	0.1925	
Total Area (ac)		0.36289	
From upstream of device#2-u4 to device#2	Driveway	0.04243	Overflow from device#2-u1, device#2-u2, device#2-u3, and device#2-u4
	Landscaped area	0.12921	
	Sidewalk	0.05392	
	Street	0.2079	
Total Area (ac)		0.43346	
From upstream of device#2-u5 to device#2	Driveway	0.08275	Overflow from device#2-u1, device#2-u2, device#2-u3, device#2-u4, and device#2-u5
	Landscaped area	0.22758	
	Parking lot	0.066	
	Roof	0.01274	
	Sidewalk	0.08054	
	Street	0.25172	
Total Area (ac)		0.72133	

1325 E 76th St #2 (sheet 305 for as-built details); no underdrains

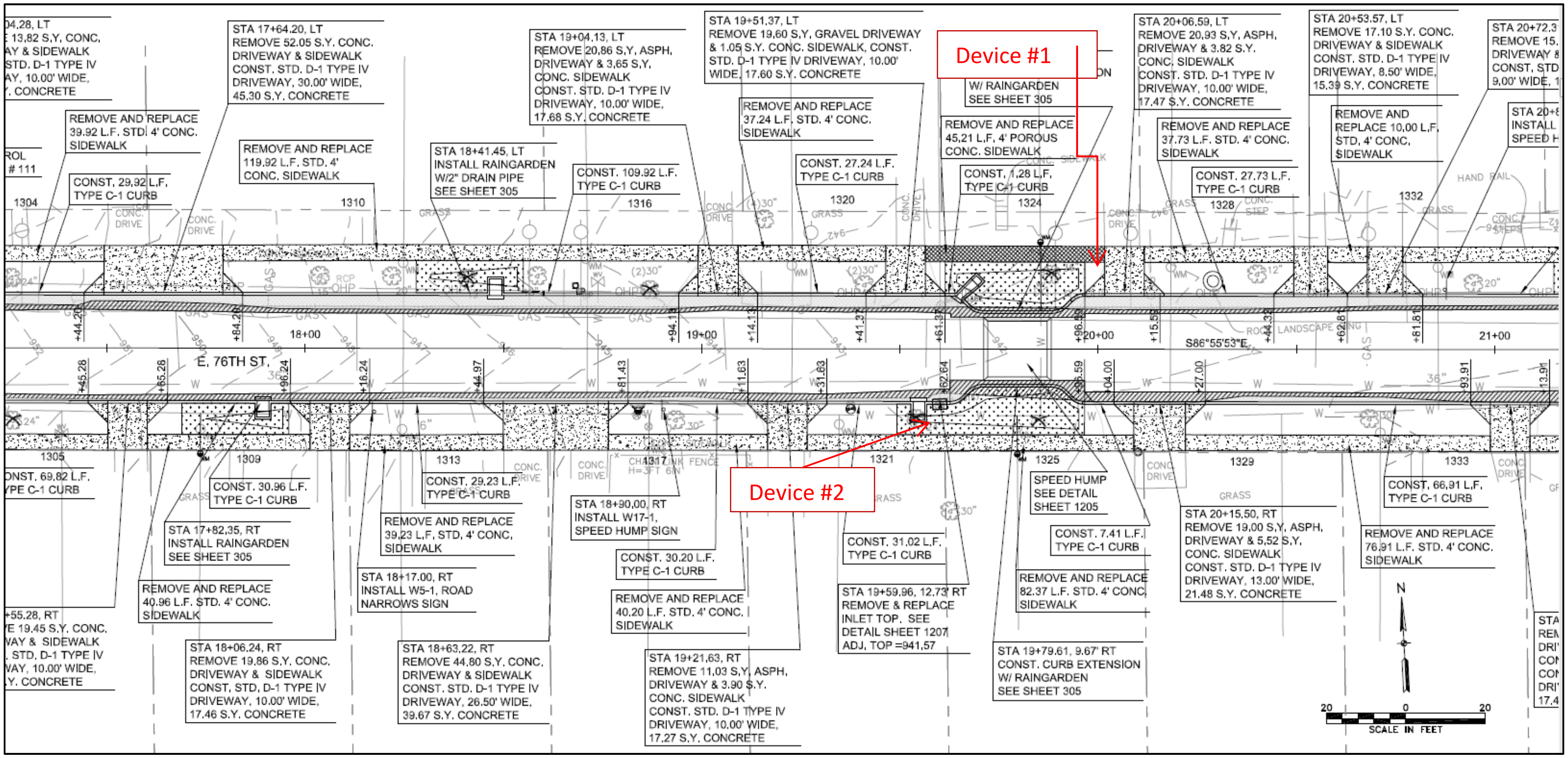
Drains from street centerline to far side of sidewalk to centerline of Troost

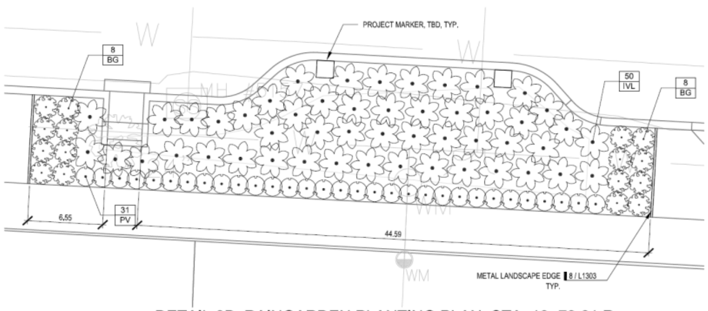
Looking upgradient towards Troost (most of lawns and homes slope south away from this location)

2 inlet samples from small rain in morning of Oct 25, 2012

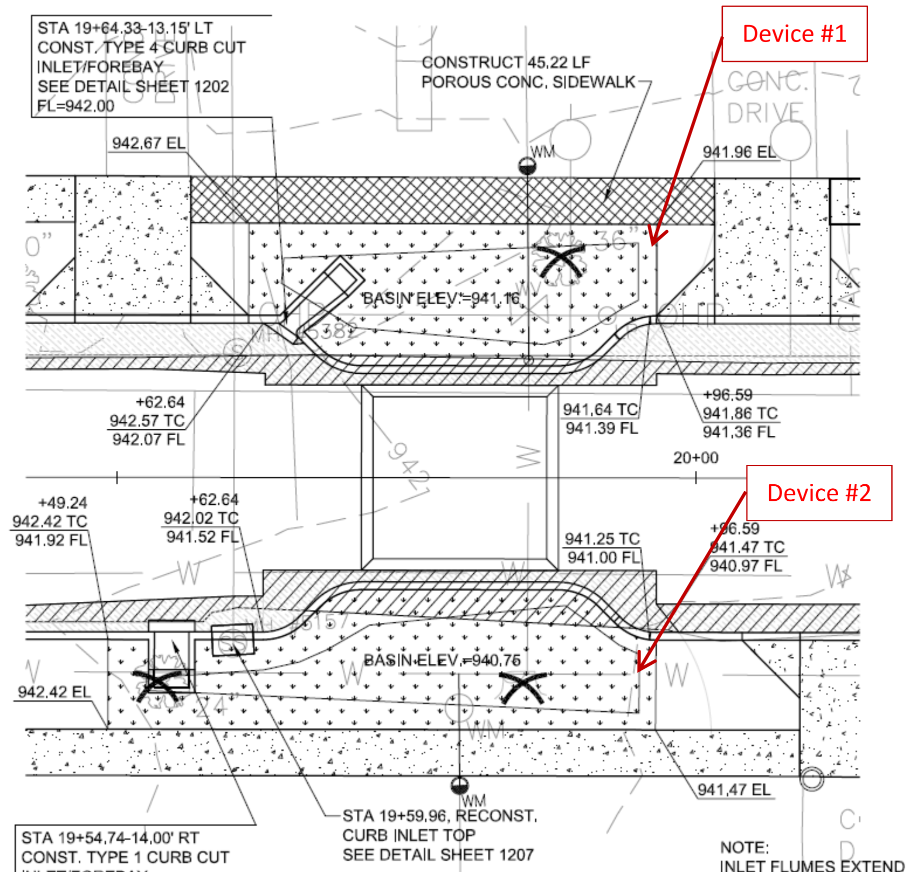
		

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
<1960	Single	Poor	1	100	0	0	Composite Shingle	Yes	No	Much	Lawn/Dec.	Poor	0	0	Unpaved	Poor	Rough	Poor





DETAIL 3B: RAINGARDEN PLANTING PLAN, STA. 19+79.61 R
SCALE: 1" = 5'



CURB EXTENSION WITH RAINGARDEN
DETAIL 9 : STA 19+79.61 - SURFACE
SCALE: 1" = 10'

3. Curb Extension with BR - 1419 E 76th Terr.

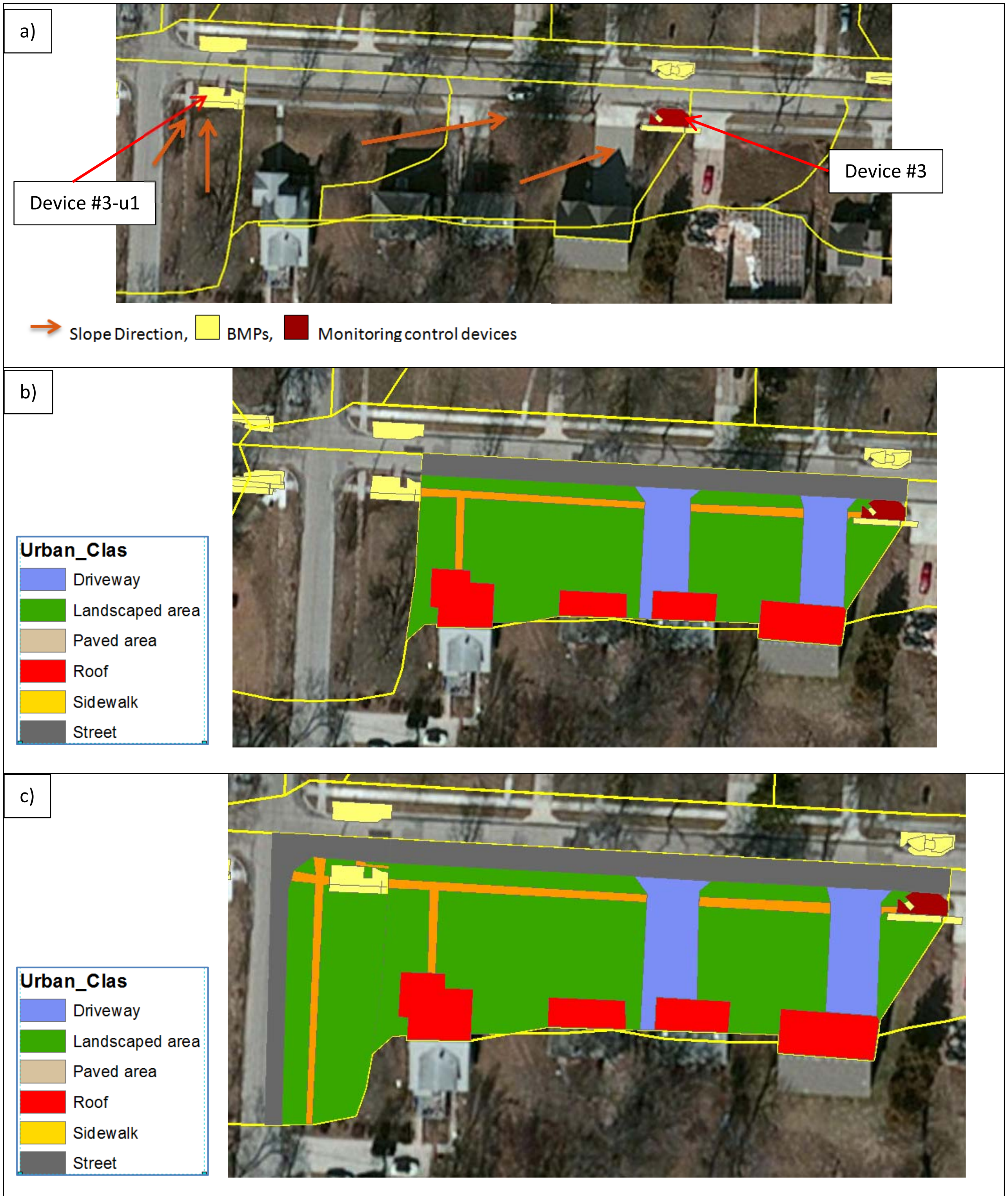
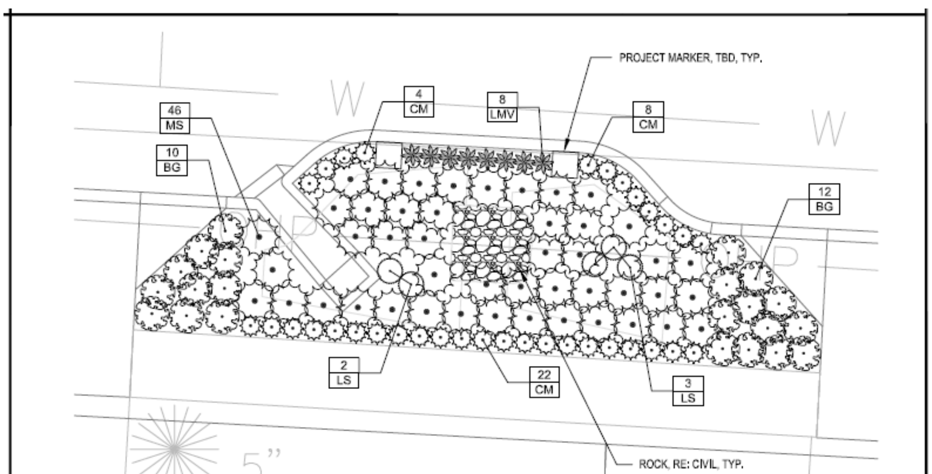


Figure 3 – Aerial photos, topo map, and urban classifications for device #3

Location	Urban Classification	Area (ac)	Note
Figure 3-b	Driveway	0.0875	There is no overflow from upstream as shown in Figure 3-b.
	Landscaped area	0.3388	
	Roof	0.0856	
	Sidewalk	0.0295	
	Street	0.0885	
Total area (ac)		0.6299	
Figure 3-c	Driveway	0.0875	Overflow from device#3-u1 as shown in figure 3-c.
	Landscaped area	0.4678	
	Roof	0.0856	
	Sidewalk	0.0462	
	Street	0.1376	
		0.8247	



DETAIL 3B: CURB EXTENSION PLANTING PLAN, STA. 26+51.65 R

SCALE: 1" = 5'

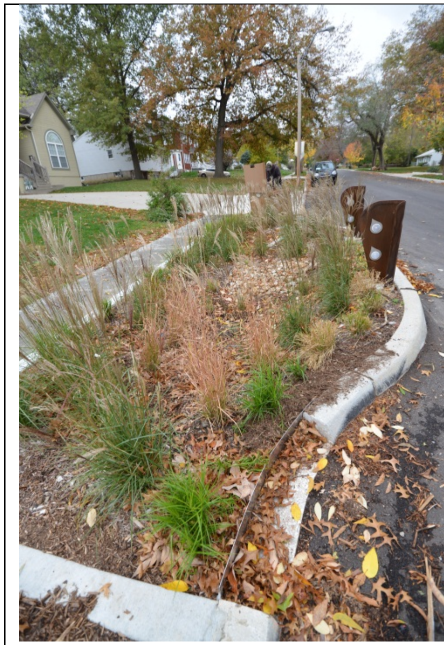
1419 E 76th Terrace #3 (sheet 207 for as-built details); no underdrains; reported to not drain well

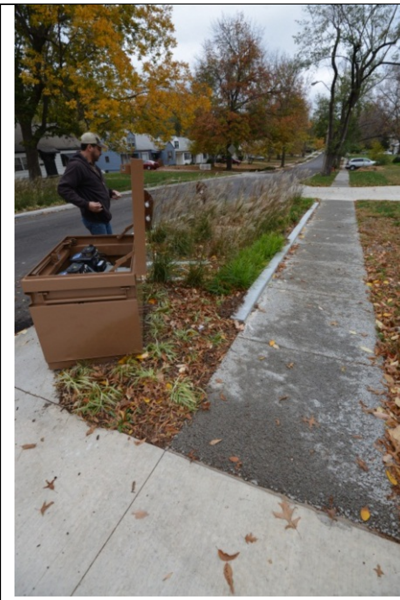
Downgradient (towards east)

Upgradient (towards west to Lydia); drains from center of lots to Lydia

Showing bottom edge of drainage area

4 inlet samples from small rain of morning of Oct 25, 2012 (initial sample contains more sediment)

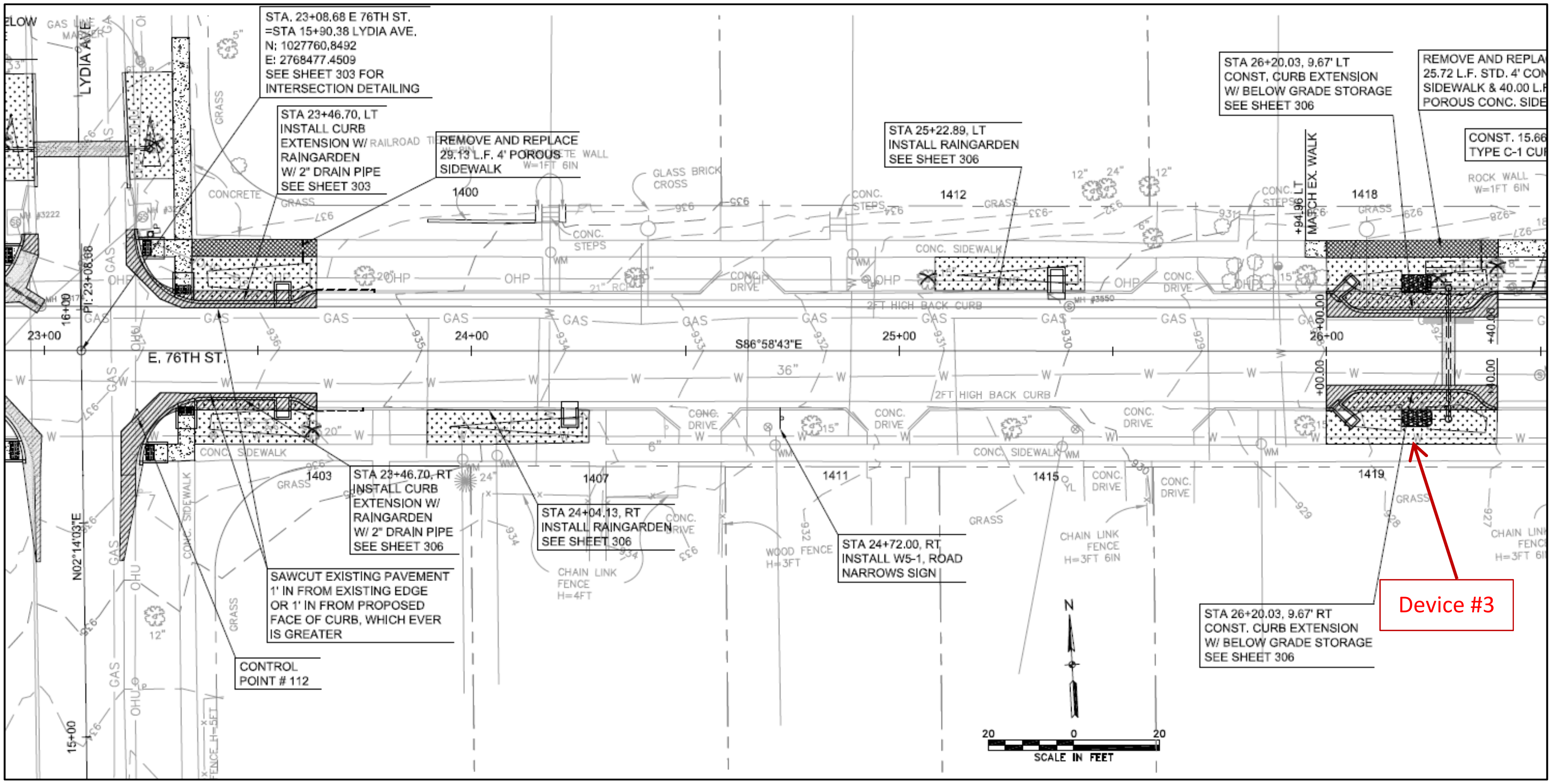


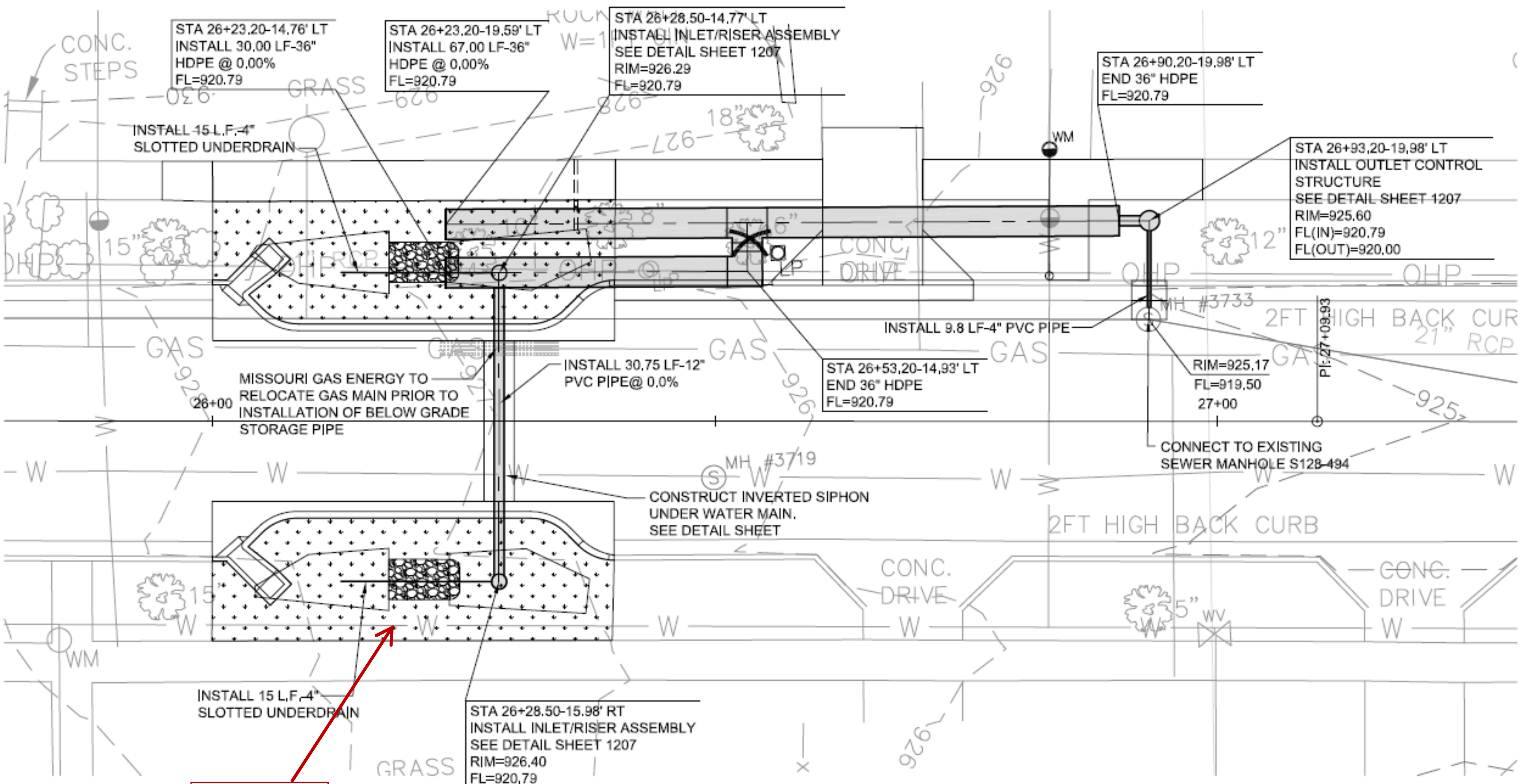


Porous concrete sidewalks all along street from Lydia to monitored rain garden

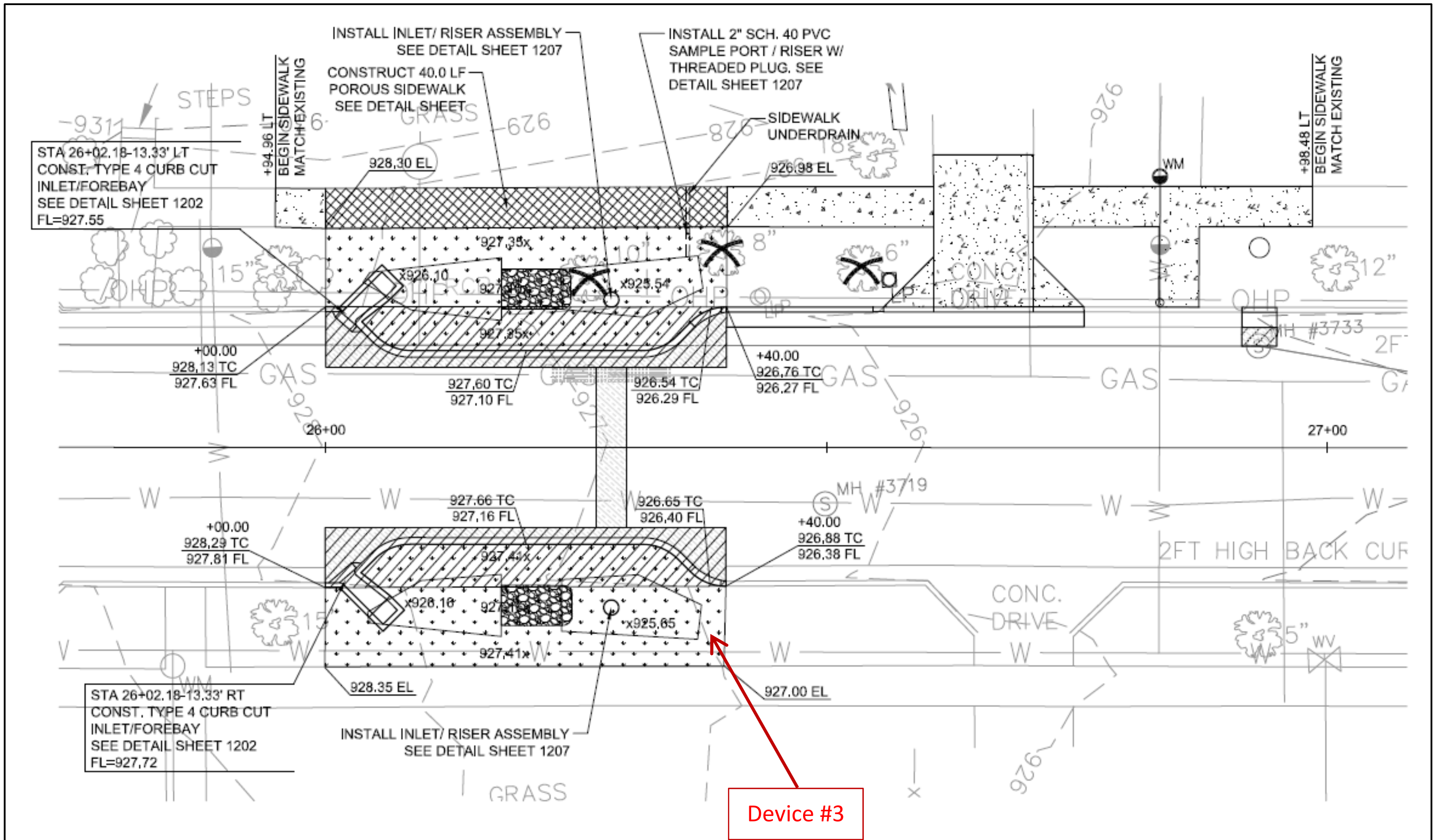
Corner of Lydia to E 76th Terrace (upper end of drainage)

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
>2000	Single	Excellent	2	100	0	0	Composite Shingle	No	1 pole	Much	Lawn	Adequate	75	100	Paved	Good	Smooth	Good





CURB EXTENSION WITH BELOW GRADE STORAGE
DETAIL 15A : STA 26+20.03 – SUBSURFACE
SCALE: 1" = 10'



CURB EXTENSION WITH BELOW GRADE STORAGE

DETAIL 15 : STA 26+20.03 - SURFACE

SCALE: 1" = 10'

4. Rain Garden Extension - 1612 E 76th St.

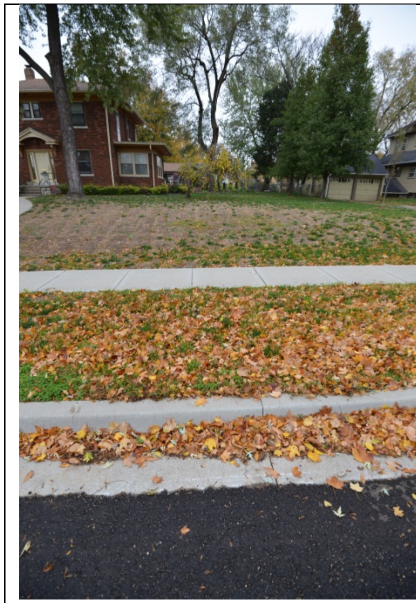
Figure 4 – Aerial photos, topo map, and urban classifications for device #4

Location	Urban Classification	Area (ac)	Note/Assumption
Figure 4-b	Driveway	0.03175	Figure 4-b.
	Landscaped area	0.33922	
	Paved area	0.05197	
	Roof	0.09569	
	Sidewalk	0.02906	
	Street	0.04938	
Total Area (ac)		0.59707	

1612 E 76th St. #4 (sheet 307 for as-built details); no underdrains

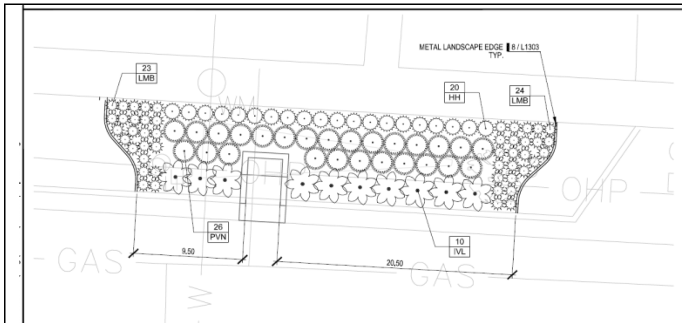
No samplers but two level recorders (inlet and bottom of garden) towards East (upgradient)

Towards West (also upgradient) (treated wood pole in rain garden)


Drainage break to east (flows to left to device from edge of house)

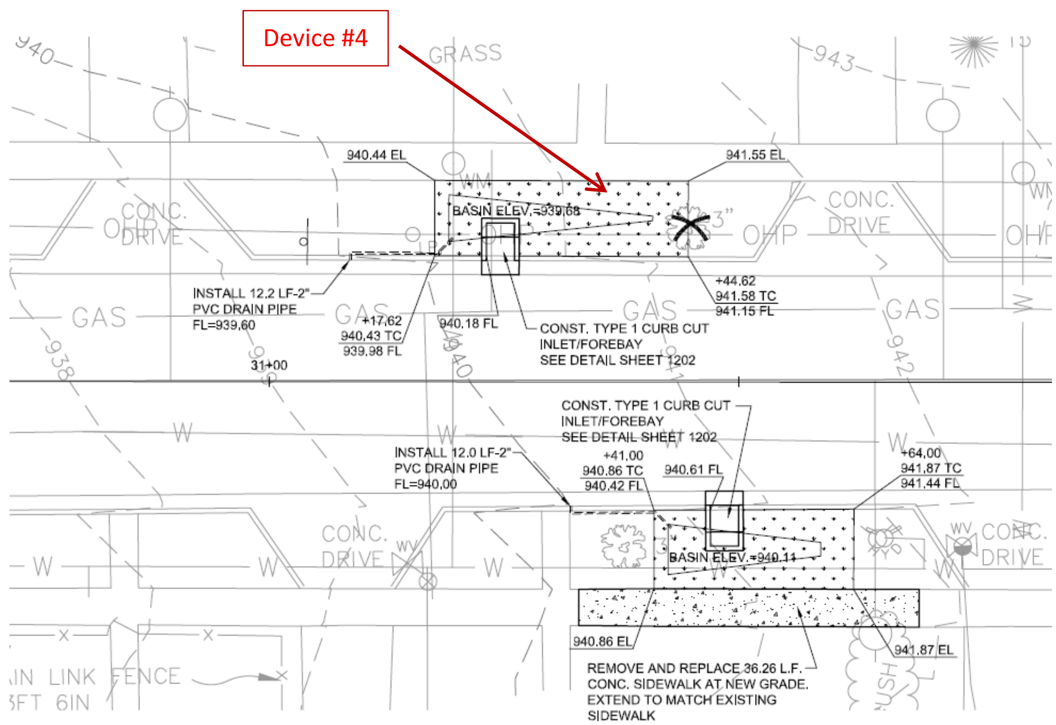

Level sensor recorder in bottom of rain garden

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
<1960	Single	Poor	2	100	0	0	Composite Shingle	No	1 pole	Much	Lawn/Dec.	Good	100	100	Paved	Good	Smooth	Good



DETAIL 2: RAINGARDEN PLANTING PLAN, STA. 31+31.12

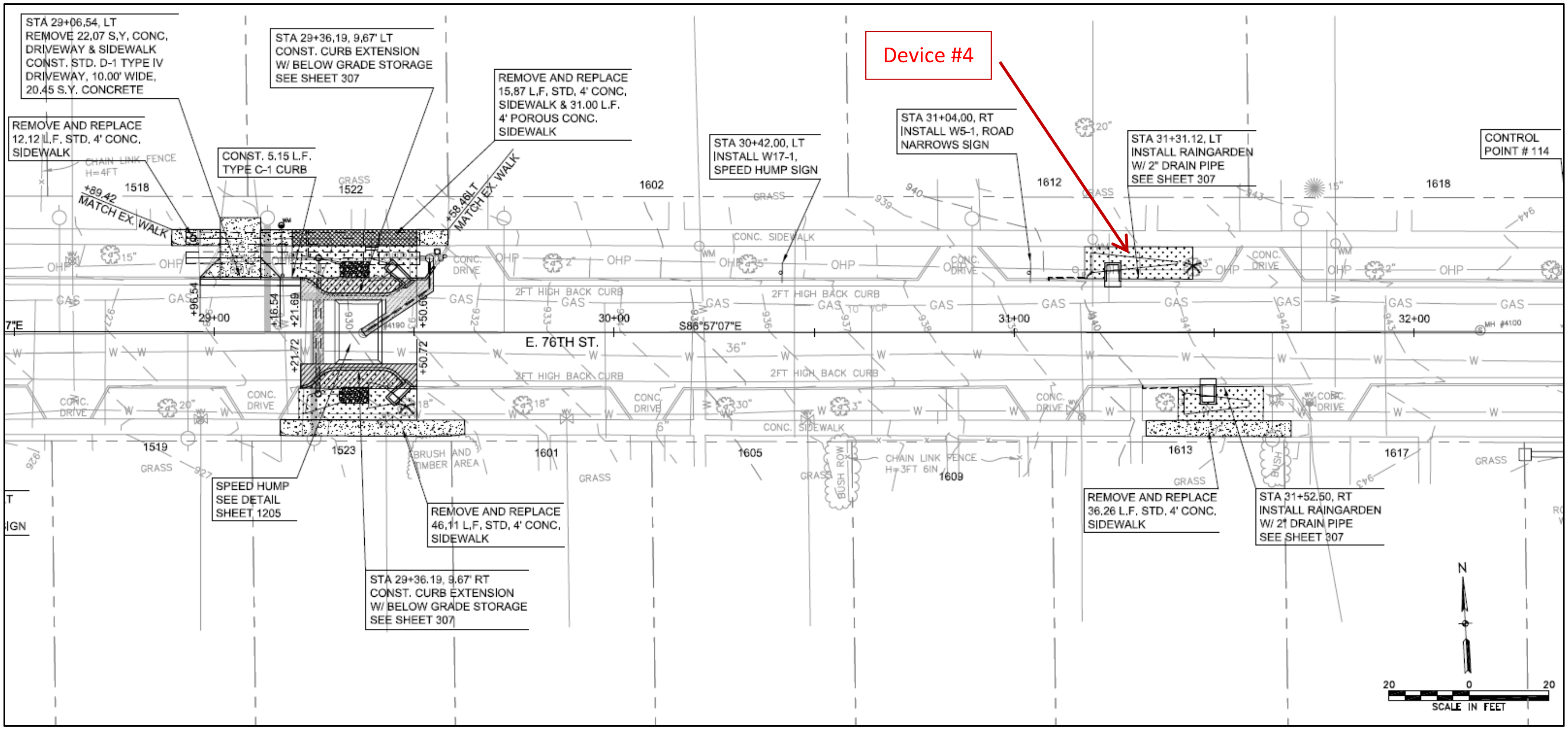
SCALE: 1" = 5'



RAINGARDEN W/ 2" DRAIN PIPE

DETAIL 17 : STA 31+31.12 & 31+52.50 - SURFACE

SCALE: 1" = 10'



Device #4
STA 29+06.54, LT
REMOVE 22.07 S.Y. CONC.
DRIVEWAY & SIDEWALK
CONST. STD. D-1 TYPE IV
DRIVEWAY, 10.00' WIDE,
20.45 S.Y. CONCRETE
REMOVE AND REPLACE
12.12 L.F. STD. 4' CONC.
SIDEWALK
CONST. 5.15 L.F.
TYPE C-1 CURB
STA 29+36.19, 9.67' LT
CONST. CURB EXTENSION
W/ BELOW GRADE STORAGE
SEE SHEET 307
REMOVE AND REPLACE
15.87 L.F. STD. 4' CONC.
SIDEWALK & 31.00 L.F.
4' POROUS CONC.
SIDEWALK
STA 30+42.00, LT
INSTALL W17-1,
SPEED HUMP SIGN
STA 31+04.00, RT
INSTALL W5-1, ROAD
NARROWS SIGN
STA 31+31.12, LT
INSTALL RAINGARDEN
W/ 2" DRAIN PIPE
SEE SHEET 307
CONTROL
POINT # 114
E. 76TH ST.
S86°57'07"E
29+00
30+00
31+00
32+00
2FT HIGH BACK CURB
CONC. SIDEWALK
CONC. DRIVE
GAS
OHP
GRASS
SPEED HUMP
SEE DETAIL
SHEET 1205
REMOVE AND REPLACE
46.11 L.F. STD. 4' CONC.
SIDEWALK
STA 29+36.19, 9.67' RT
CONST. CURB EXTENSION
W/ BELOW GRADE STORAGE
SEE SHEET 307
REMOVE AND REPLACE
36.26 L.F. STD. 4' CONC.
SIDEWALK
STA 31+52.50, RT
INSTALL RAINGARDEN
W/ 2" DRAIN PIPE
SEE SHEET 307
CHAIN LINK FENCE
H=3FT 6IN
BRUSH AND
TIMBER AREA
N
20 0 20
SCALE IN FEET

5. Rain Garden Extension - 1336 E 76th St.

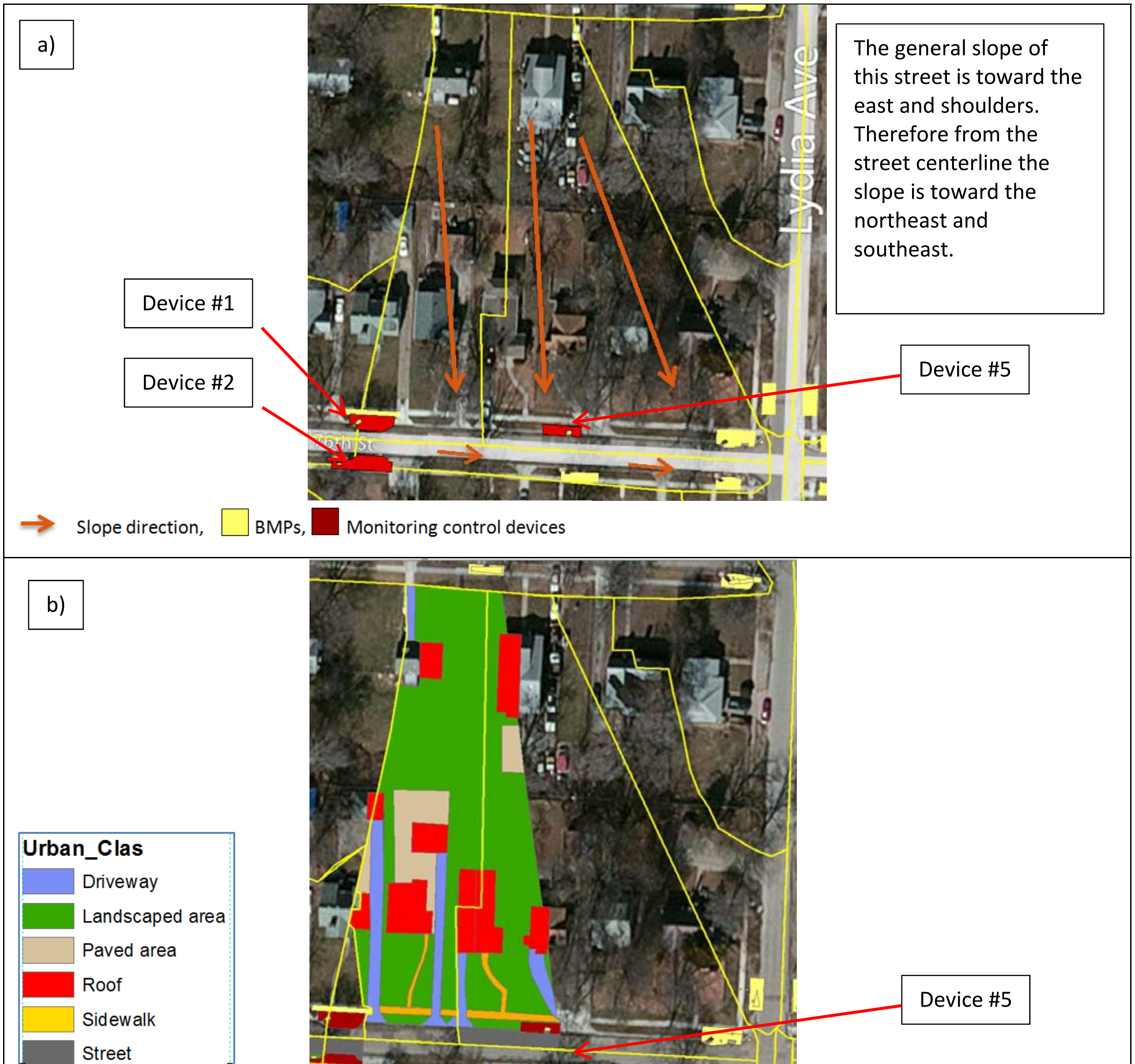


Figure 5 – Aerial photos, topo map, and urban classifications for device #5

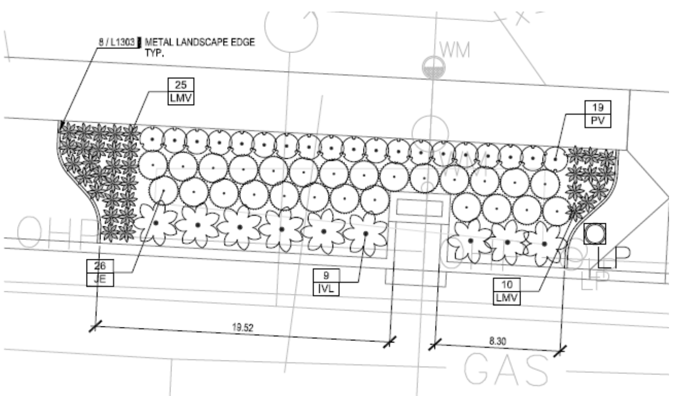
Location	Urban Classification	Area (ac)	Note/Assumption
Figure 5-b	Driveway	0.08434	
	Landscaped area	0.55210	
	Paved area	0.07051	
	Roof	0.14523	
	Sidewalk	0.02434	
	Street	0.02202	
Total Area (ac)		0.89854	

1336 E 76th St #5 (sheet 305 for as-built details); no underdrains

 No samplers, 2 level recorders (inlet and bottom of rain garden)	
	 Upgradient from rain garden

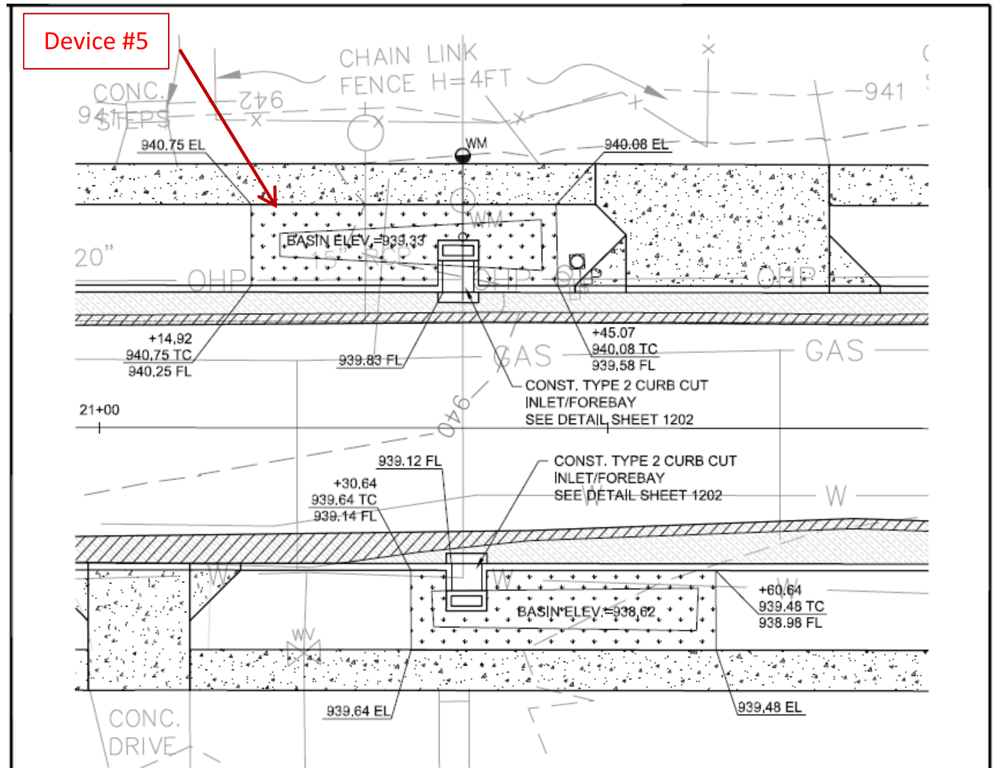
	

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
<1960	Single	Good	2	50	50	0	Composite Shingle	Yes	1 pole	Much	Lawn/Dec.	Adequate	0	100	Paved	Fair	Smooth	Poor



DETAIL 4: RAINGARDEN PLANTING PLAN, STA. 21+29.95

SCALE: 1" = 5'



RAINGARDEN

DETAIL 10 : STA 21+29.95 & 21+45.88 - SURFACE

SCALE: 1" = 10'

6. Site #6 was abandoned and is not being monitored

7. Shallow Bioretention Device w/ Smart Drain - 1140 E 76th Terr.

Figure 7 – Aerial photos, topo map, and urban classifications for device #7

Location	Urban Classification	Area (ac)	Note/Assumption
Figure 7-b	Driveway	0.00482	
	Landscaped area	0.00318	
	Sidewalk	0.00067	
	Street	0.01596	
Total area (ac)		0.02462	

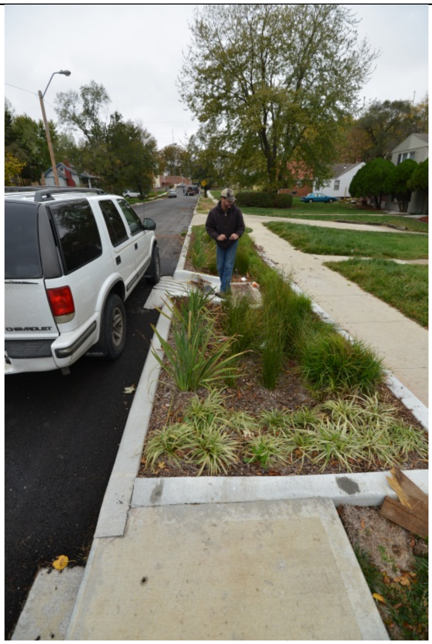
1140 E 76th Terrace #7 (sheet 205 for as-built details); Smart Drains

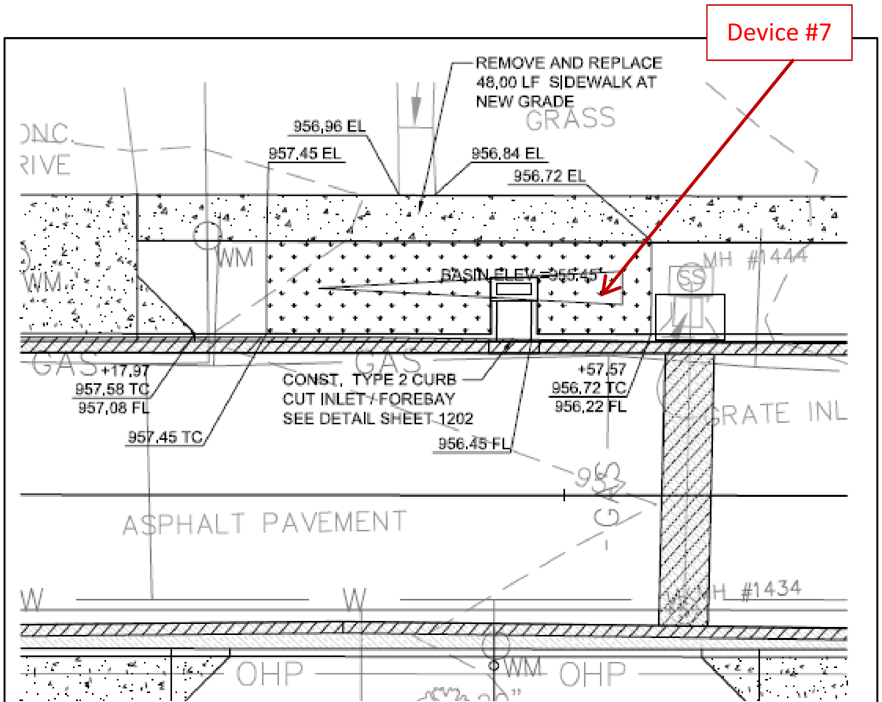
Towards E showing sloping driveway from rain garden; only half of street and a bit of yard to system (near top of street slope)

Very small drainage area; large inlet right below rain garden

Yard slopes away from rain garden; sidewalk edge to street center

Driveway slopes away from rain garden towards yard inlets

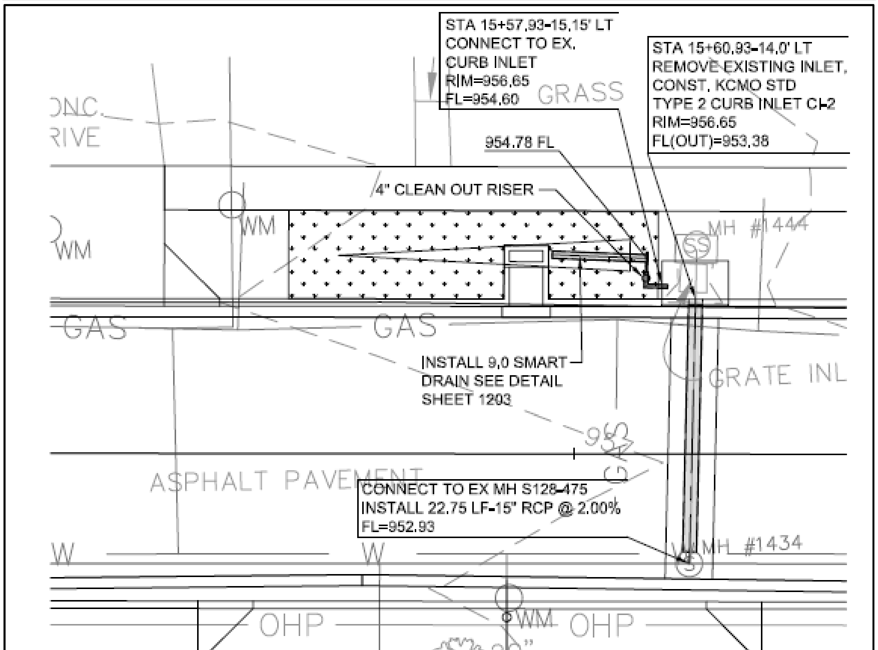
 No samplers, but 2 level recorders at inlet and bottom of rain garden		



SHALLOW BIORETENTION

DETAIL 4 : STA 15+37.75 - SURFACE

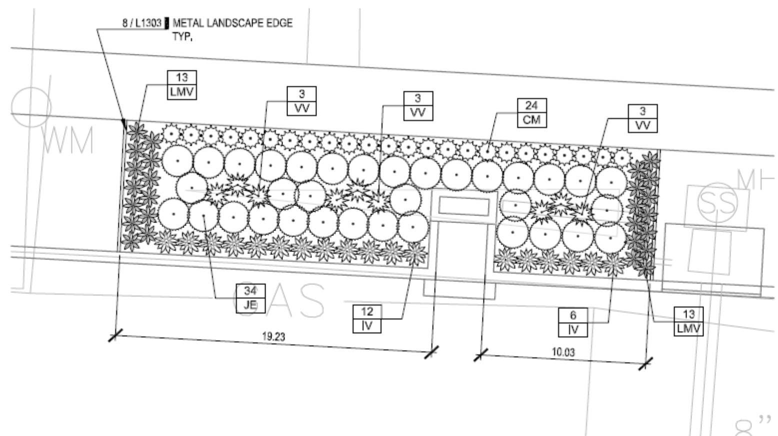
SCALE: 1" = 10'



SHALLOW BIORETENTION

DETAIL 4A : STA 15+37.75 - SUBSURFACE

SCALE: 1" = 10'



DETAIL 6: SHALLOW BIO-RETENTION PLANTING PLAN, STA. 15+37.75

SCALE: 1" = 5'

8. Rain Garden w/ Smart Drain - 1222 E 76th St.

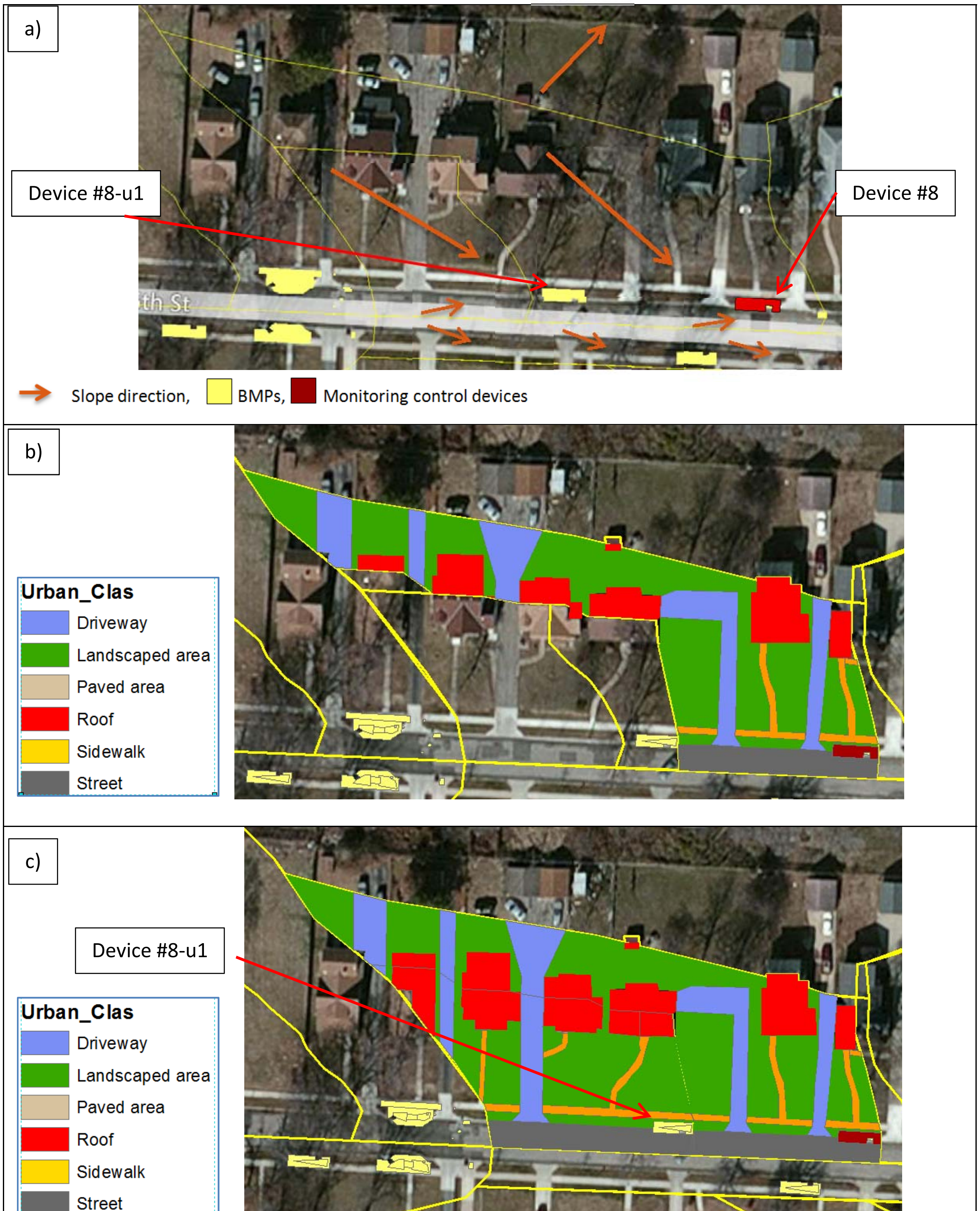


Figure 8 – Aerial photos, topo map, and urban classifications for device #8

Location	Urban Classification	Area (ac)	Note/Assumption
Figure 8-b	Driveway	0.12166	There is no overflow from upstream, as shown in Figure 8-b.
	Landscaped area	0.30035	
	Roof	0.09538	
	Sidewalk	0.02348	
	Street	0.0442	
Total area (ac)		0.5851	
Figure 8-c	Driveway	0.17259	As shown in Figure 8-c, there is an overflow from device# 8-u1.
	Landscaped area	0.48239	
	Roof	0.17459	
	Sidewalk	0.05274	
	Street	0.10525	
Total area (ac)		0.98756	

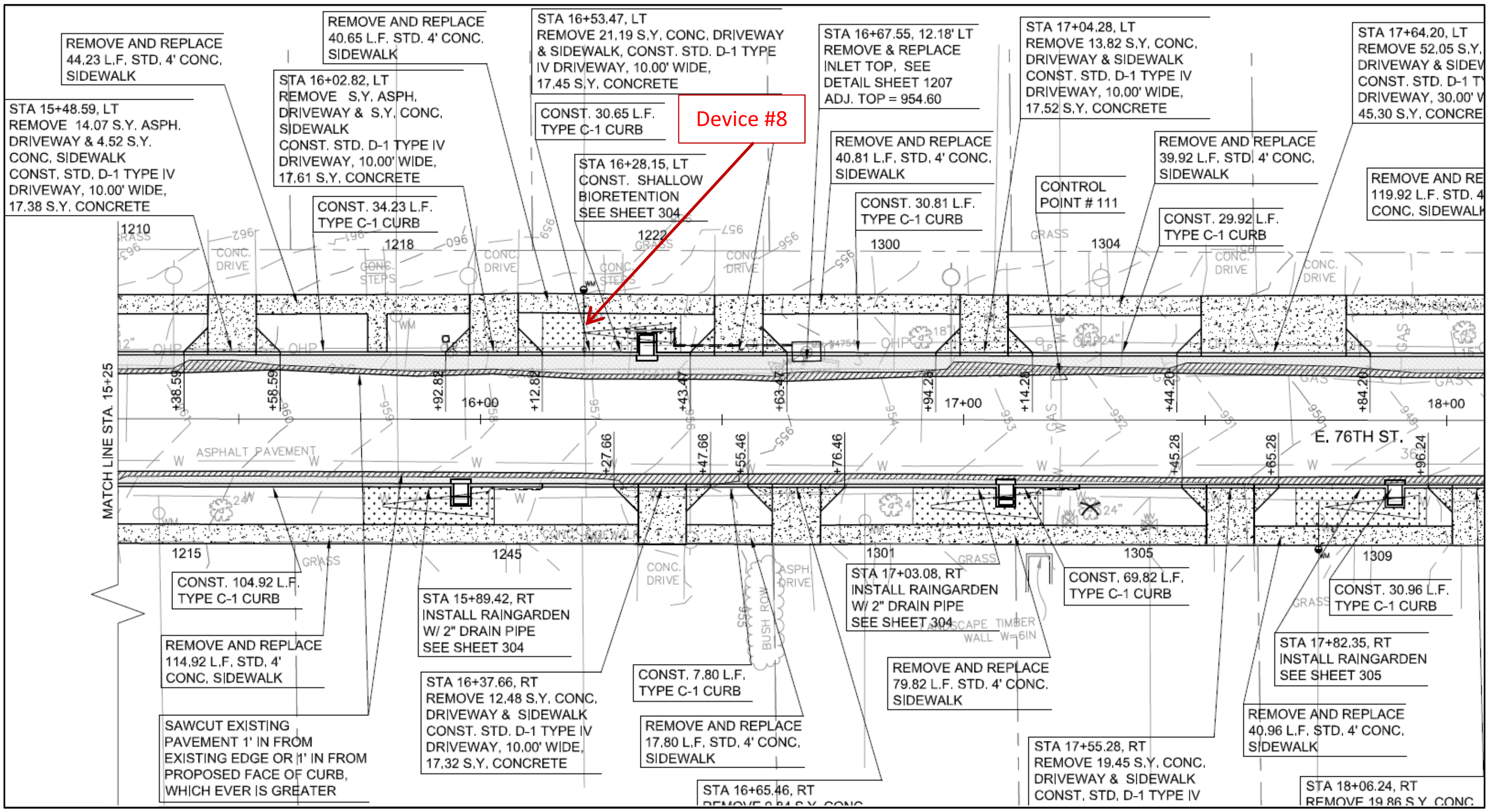
1222 E 76th St #8 (sheet 304 for as-built details); Smart Drains

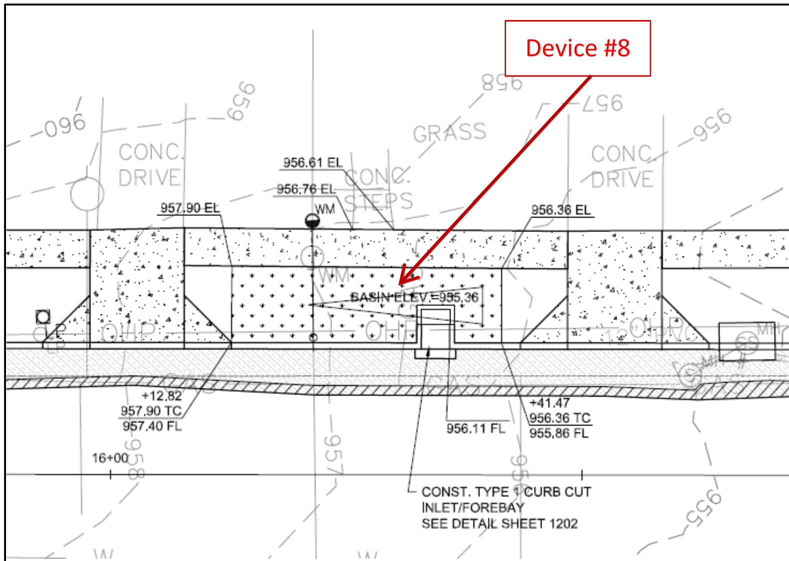
2 samplers and 2 level recorders (inlet and smartdrain underdrain)

E edge of drainage area slopes away from rain garden (no house or driveway)

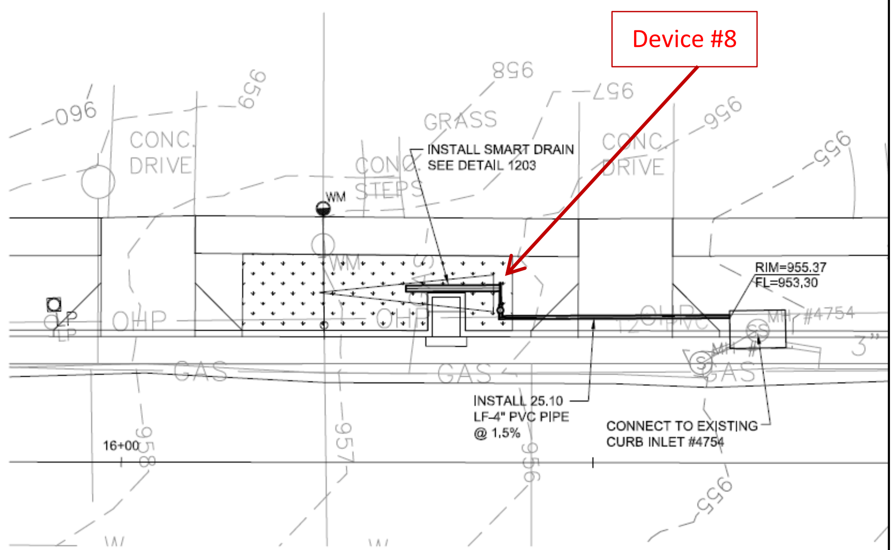
Upgradient rain garden and signage

Date of Construction	Dwelling Type	Building Maintenance	Building Height	Pervious %	Impervious %	Underground %	Roof Type	Sediment Source	Treated Wood	Landscaping Quantity	Landscaping Type	Landscaping Mainten.	% Connected Sidewalk	% Connected Driveway	Driveway Type	Driveway Condition	Driveway Texture	Stormwater Control Potential
<1960	Single	Good	2	50	50	0	Composite Shingle	No	No	Much	Lawn/Dec.	Adequate	100	100	Paved	Good	Smooth	Possible

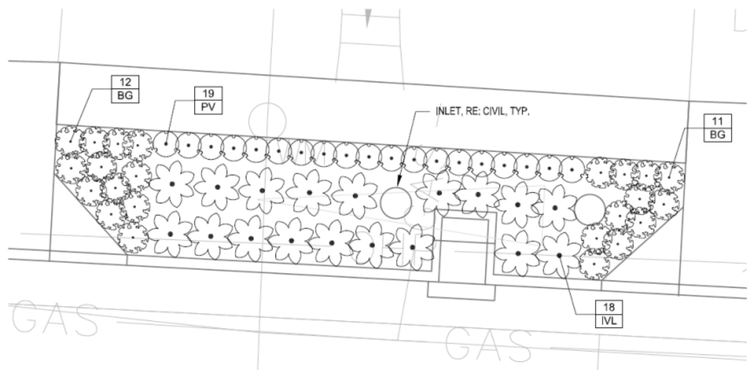




SHALLOW BIORETENTION
DETAIL 5 : STA 16+28.15 - SURFACE
SCALE: 1" = 10'



SHALLOW BIORETENTION
DETAIL 5A : STA 16+28.15 - SUBSURFACE
SCALE: 1" = 10'



DETAIL 5: BIO-RETENTION PLANTING PLAN, STA. 16+28.15
SCALE: 1" = 5'

9. Cascade - 1112 E 76th Terr.

Figure 9 – Aerial photos, topo map, and urban classifications for device #9

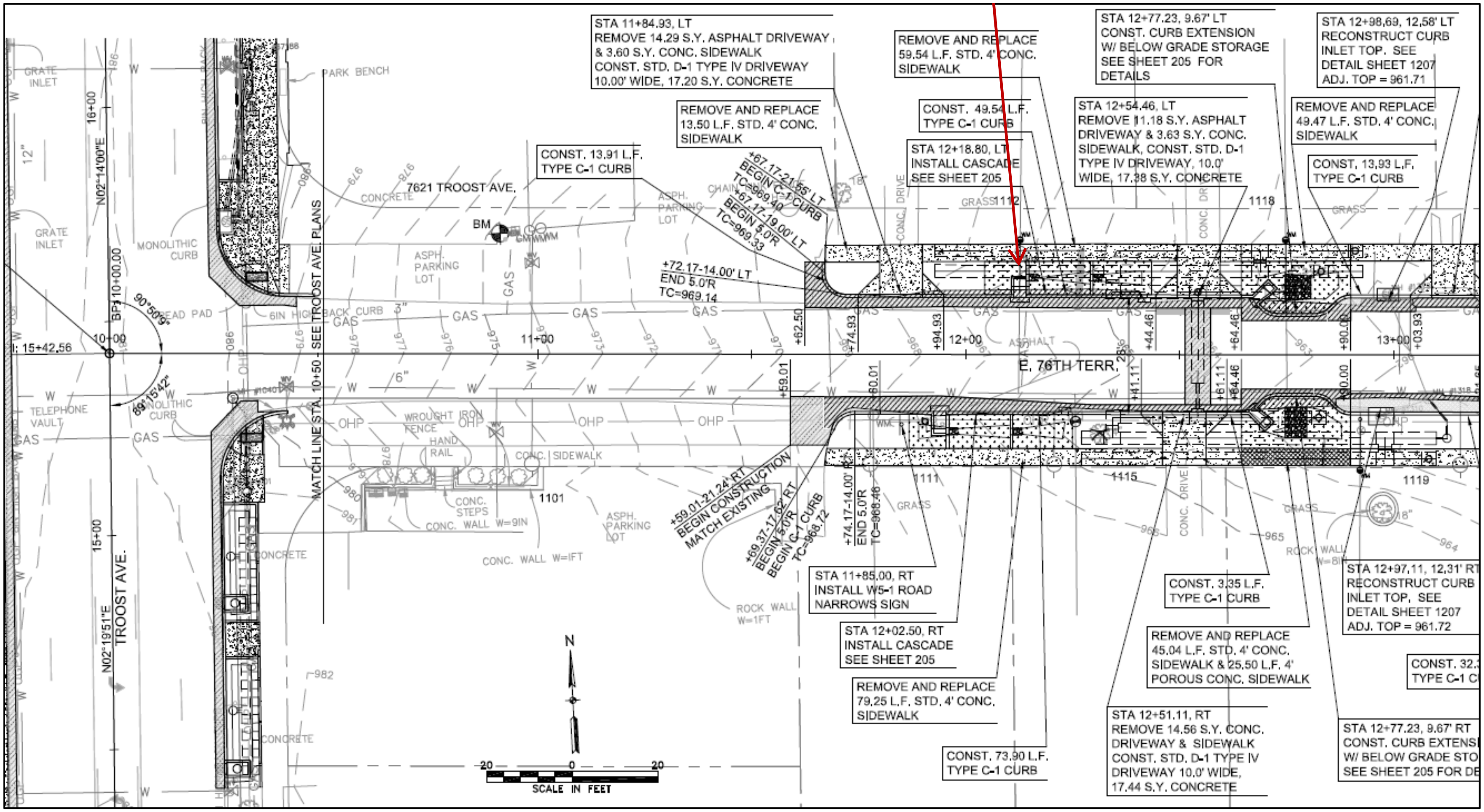
Location	Urban Classification	Area (ac)	Note/Assumption
Figure 9-b	Driveway	0.0392	There is no overflow from upstream, as shown in Figure 9-b.
	Landscaped area	0.0337	
	Parking lot	0.0639	
	Roof	0.0958	
	Sidewalk	0.0101	
	Street	0.0505	
Total area (ac)		0.2931	

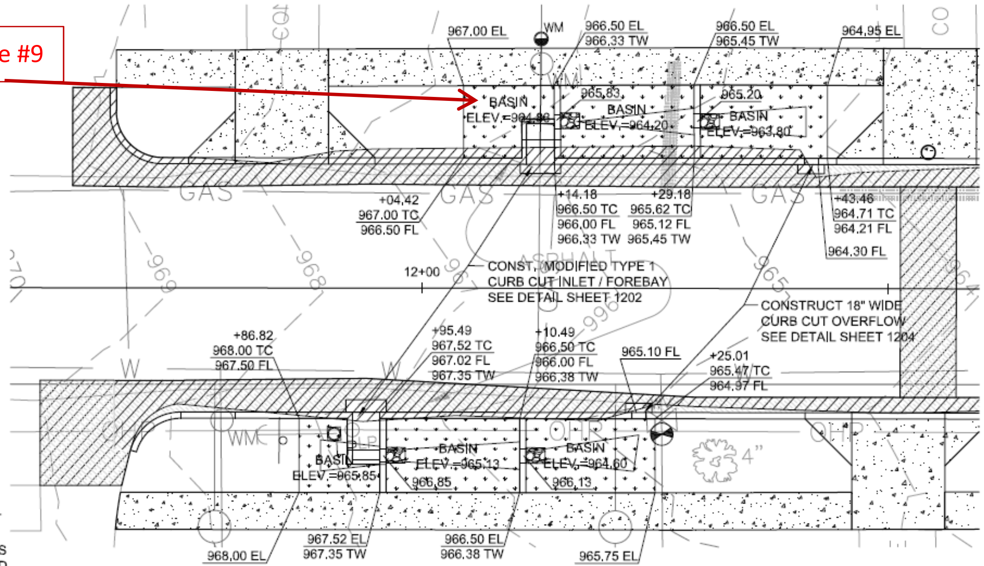
1112 E 76th Terrace #9 (sheet 205 for as-built details); cascading swale (but upper weir set high so runoff bypasses other cells), no underdrains

W towards Troost and two businesses that drain to this device

Towards cascade from next downgradient rain garden and drain inlet

Device #9


STA 11+84.93, LT
REMOVE 14.29 S.Y. ASPHALT DRIVEWAY & 3.60 S.Y. CONC. SIDEWALK
CONST. STD. D-1 TYPE IV DRIVEWAY 10.00' WIDE, 17.20 S.Y. CONCRETE
REMOVE AND REPLACE 59.54 L.F. STD. 4' CONC. SIDEWALK
STA 12+77.23, 9.67' LT
CONST. CURB EXTENSION W/ BELOW GRADE STORAGE SEE SHEET 205 FOR DETAILS
STA 12+98.69, 12.58' LT
RECONSTRUCT CURB INLET TOP. SEE DETAIL SHEET 1207 ADJ. TOP = 961.71
REMOVE AND REPLACE 13.50 L.F. STD. 4' CONC. SIDEWALK
CONST. 49.54 L.F. TYPE C-1 CURB
STA 12+54.46, LT
REMOVE 11.18 S.Y. ASPHALT DRIVEWAY & 3.63 S.Y. CONC. SIDEWALK, CONST. STD. D-1 TYPE IV DRIVEWAY, 10.0' WIDE, 17.38 S.Y. CONCRETE
REMOVE AND REPLACE 49.47 L.F. STD. 4' CONC. SIDEWALK
CONST. 13.91 L.F. TYPE C-1 CURB
STA 12+18.80, LT
INSTALL CASCADE SEE SHEET 205
CONST. 13.93 L.F. TYPE C-1 CURB
7621 TROOST AVE.
BM
+67.21-21.55' LT BEGIN C-1 CURB TC=969.40
+67.17-19.00' LT BEGIN 5.0'R TC=969.33
+72.17-14.00' LT END 5.0'R TC=969.14
MATCH LINE STA 10+50 - SEE TROOST AVE. PLANS
TROOST AVE.
N02°14'00"E
N02°19'51"E
BP 10+00.00
E. 76TH TERR.
+59.01-21.24' RT BEGIN CONSTRUCTION MATCH EXISTING
+69.37-17.62' RT BEGIN 5.0'R BEGIN C-1 CURB TC=968.72
+74.17-14.00' RT END 5.0'R TC=968.48
STA 11+85.00, RT
INSTALL W5-1 ROAD NARROWS SIGN
STA 12+02.50, RT
INSTALL CASCADE SEE SHEET 205
REMOVE AND REPLACE 79.25 L.F. STD. 4' CONC. SIDEWALK
CONST. 73.90 L.F. TYPE C-1 CURB
CONST. 3.35 L.F. TYPE C-1 CURB
REMOVE AND REPLACE 45.04 L.F. STD. 4' CONC. SIDEWALK & 25.50 L.F. 4' POROUS CONC. SIDEWALK
STA 12+51.11, RT
REMOVE 14.56 S.Y. CONC. DRIVEWAY & SIDEWALK CONST. STD. D-1 TYPE IV DRIVEWAY 10.0' WIDE, 17.44 S.Y. CONCRETE
STA 12+97.11, 12.31' RT
RECONSTRUCT CURB INLET TOP, SEE DETAIL SHEET 1207 ADJ. TOP = 961.72
CONST. 32.3
TYPE C-1 C
STA 12+77.23, 9.67' RT
CONST. CURB EXTENSI
W/ BELOW GRADE STO
SEE SHEET 205 FOR DE
SCALE IN FEET

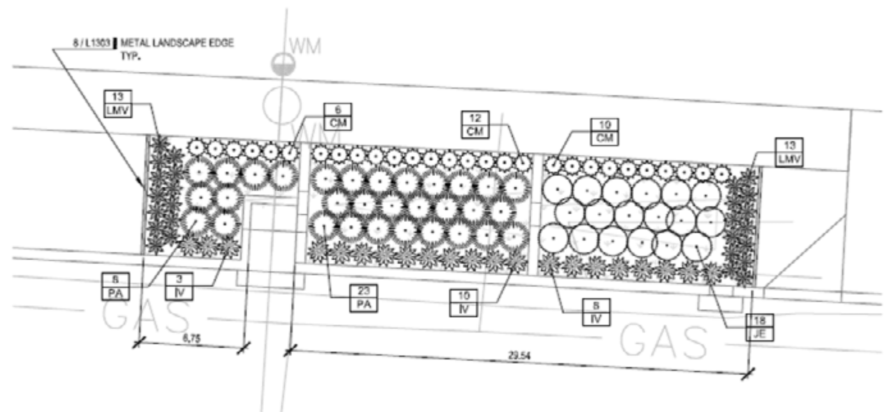


NOTE:
INLET FLUMES EXTEND FARTHER THAN NORMAL AT THIS LOCATION.

NOTE:
CONSTRUCT CURB EXTENSIONS WITH BELOW GRADE STORAGE AT STATIONS 12+77.23 BEFORE CONSTRUCTING THESE CASCADES AS THERE WILL BE PIPES LOCATED BELOW THE CASCADES.

DETAIL 1 : STA 12+02.50 & 12+18.80 - SURFACE

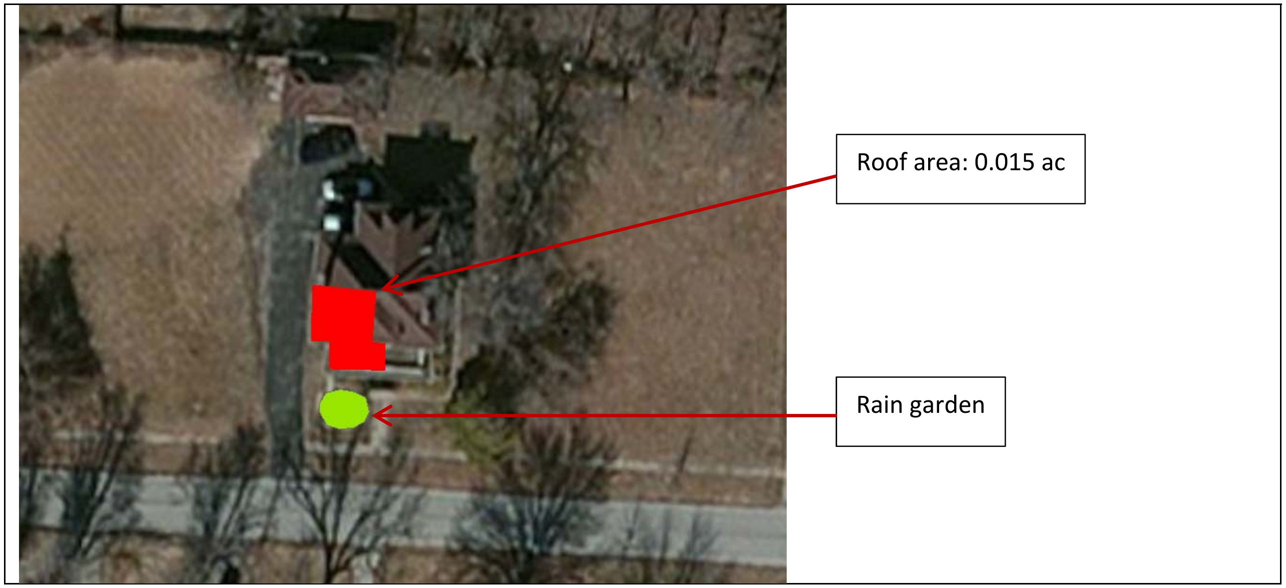
SCALE: 1" = 10'



DETAIL 2: CASCADE PLANTING PLAN, STA. 12+18.80

SCALE: 1" = 5'

10. Private rain garden - 1312 E. 79th St. - Mrs. Thomas



1312 E 79th St #10; Mrs. Thomas Rain Garden (no details; two level recorders, inlet and bottom of rain garden)


Roof drains from half of front and half of side of home


Typical street without rain gardens

11. 1505 E 76th St, #11; Mrs. Moss rain garden (no details); level recorders for inlet and bottom of garden)

	
 Side of house; rain garden in rear	

Appendix B: Details of typical stormwater controls in test area

Device at Site No.	Top area (sq ft)	Bottom area (sq ft)	Pool depth	Material
1	422.9	240		
2	513.5	228.5		
3	341.5	160.5		
4	200.86	59.72	6’’	3’’ hardwood mulch on top, Native soil amended with 3’’ compose, roto-tilled 8’’ min depth
5	222.35	93.5	6’’	
7	247.06	37.9	12’’	3’’ hardwood mulch on top, Topsoil planting mix on side slopes, Engineering soil mix 8’’ min depth on bottom.
8	284	36.28	6’’	3’’ hardwood mulch on top, Native soil amended with 3’’ compose, roto-tilled 8’’ min depth
9	290.73	48.16	12’’	Topsoil planting mix on side slopes, Engineering soil mix 8’’ min depth on bottom.

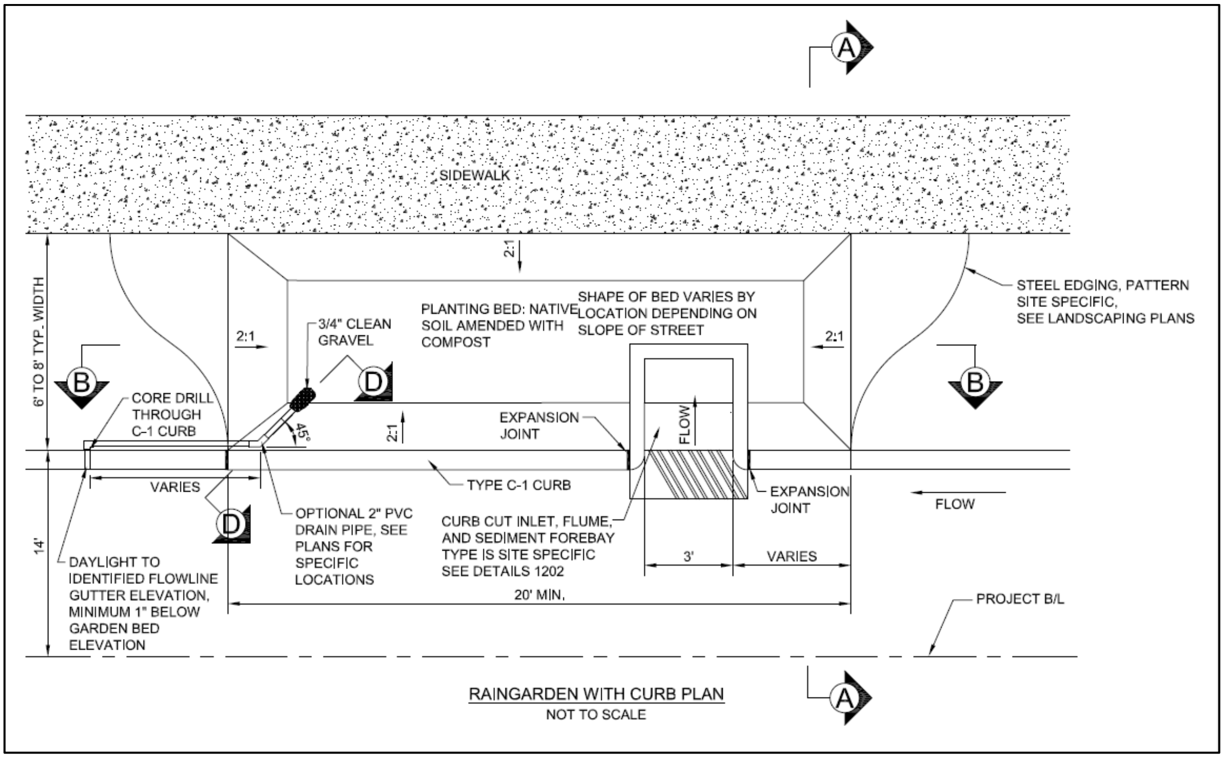
Subsurface layer properties for applicable stormwater control layers (Source Table 2-10 of the “Report on Enhanced Framework (SUSTAIN) and Field Applications for Placement of BMPs in Urban Watersheds”)

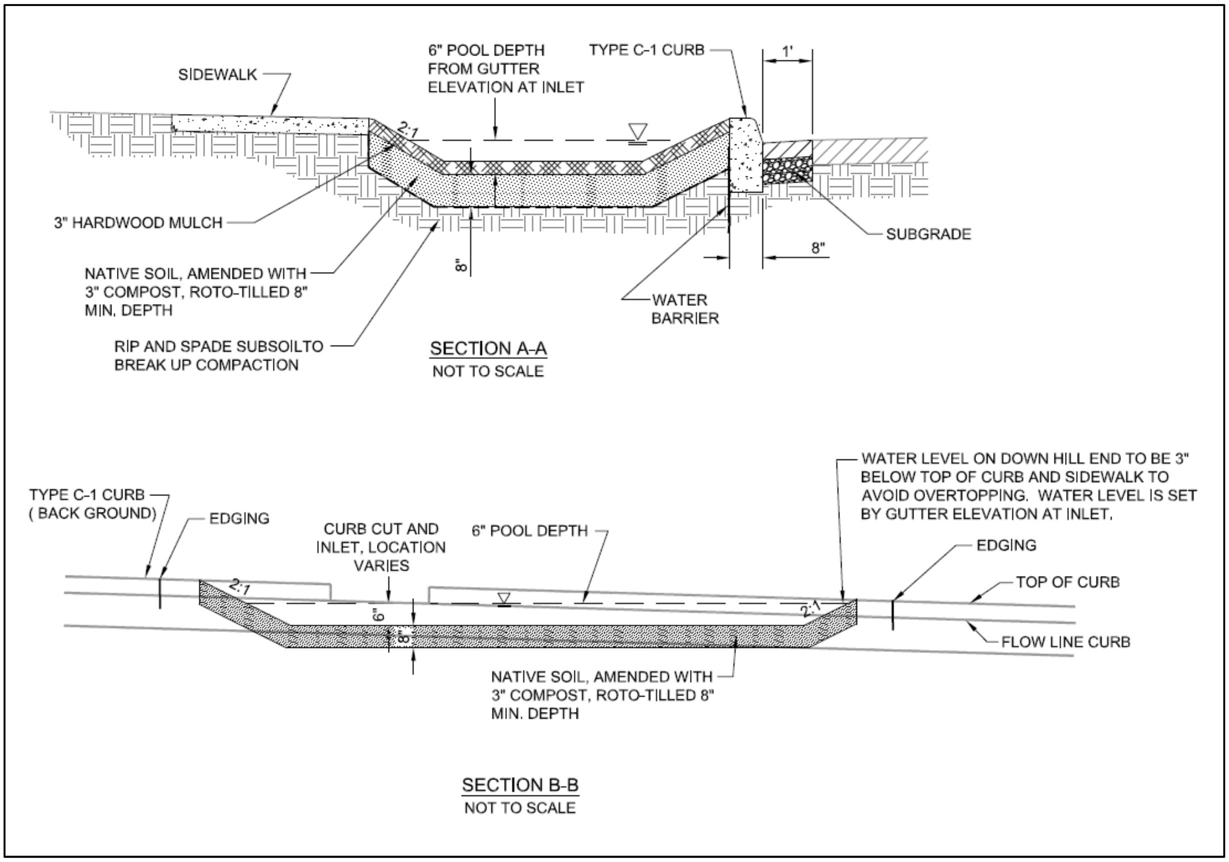
Soil layer	Property	Value	Units
Engineered soil media	Porosity	0.4	--
	Field capacity	0.3	--
	Wilting point	0.1	--
	Holtan vegetation parameter	0.6	--
	Saturated infiltration rate	2	in./hr
Underdrain layer	Void fraction	0.4	--
Native background soil	Saturated infiltration rate	0.1	in./hr

Private rain garden design dimensions and specifications. (Source Table 2-11 of the “Report on Enhanced Framework (SUSTAIN) and Field Applications for Placement of BMPs in Urban Watersheds”)

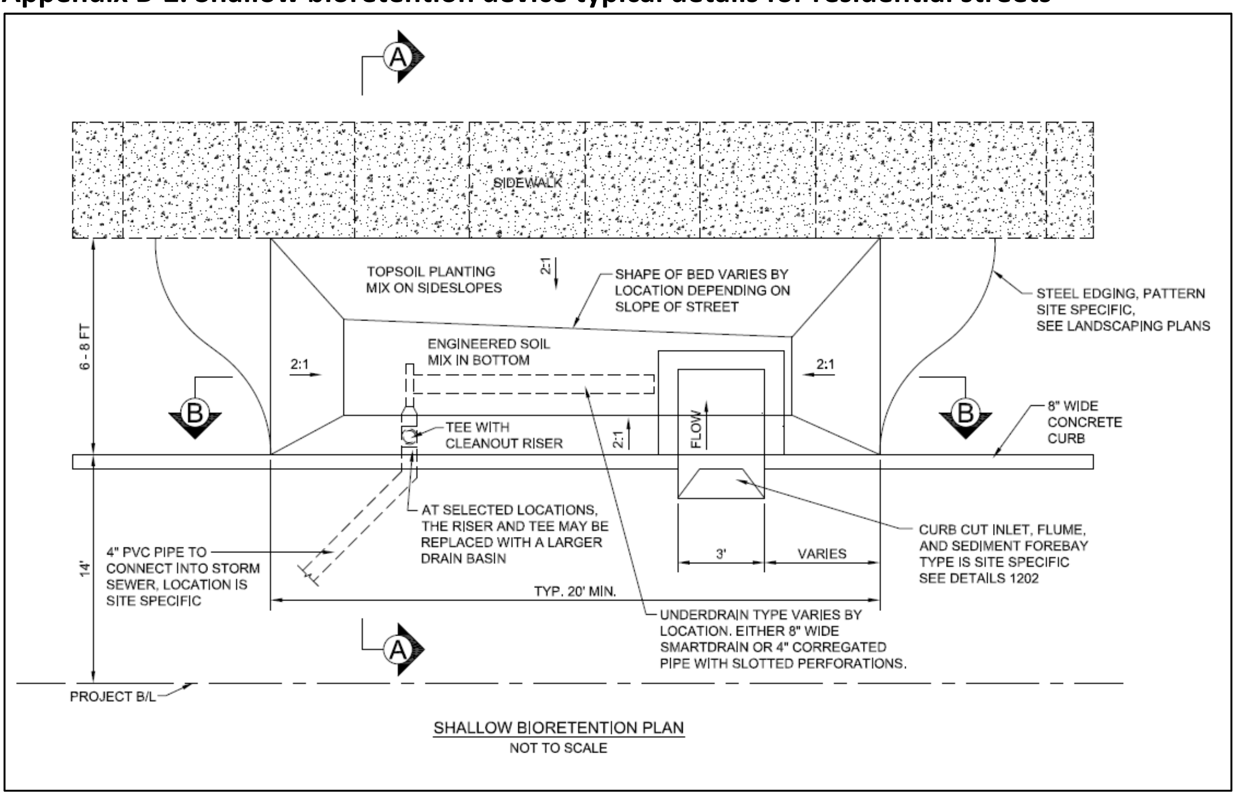
BMP categories	BMP dimensions				Outlet type
	Surface area	Ponding (ft)	Soil media (ft)	Underdrain	
Rain garden	200 sq ft per house (1,000 sq ft roof)	1	2	No underdrain	Weir
Influent flow monitoring device	35-gallon tank with orifice on standpipe				Weir and orifice

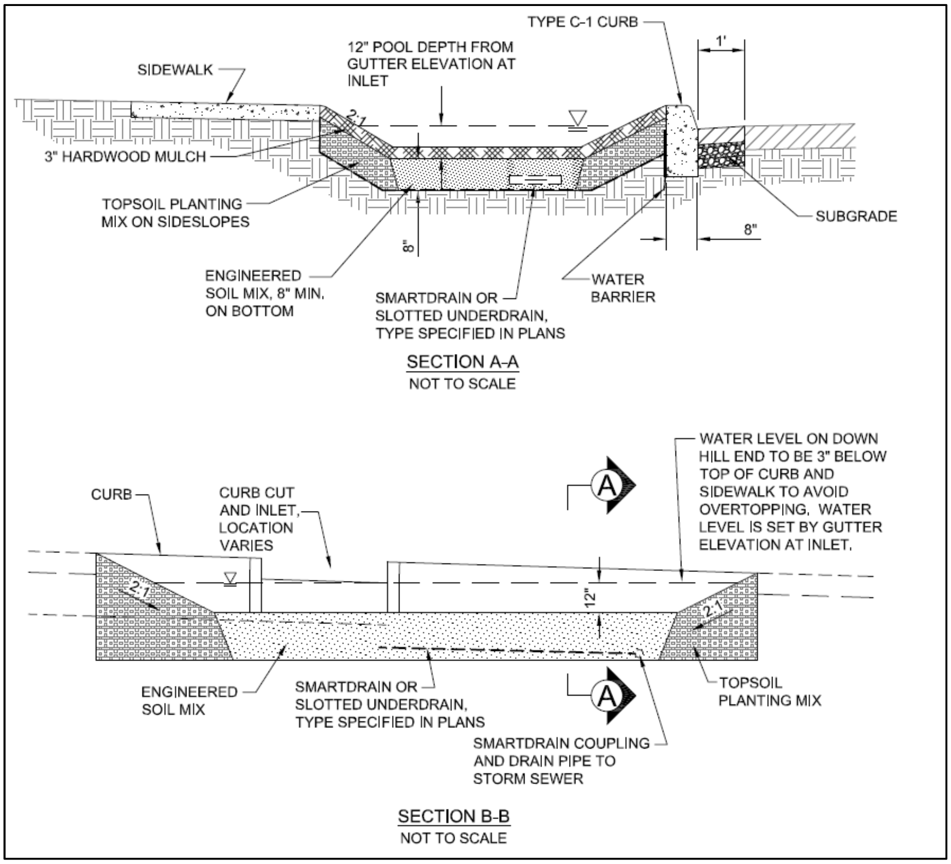
Appendix B-1. Rain garden typical details for residential streets



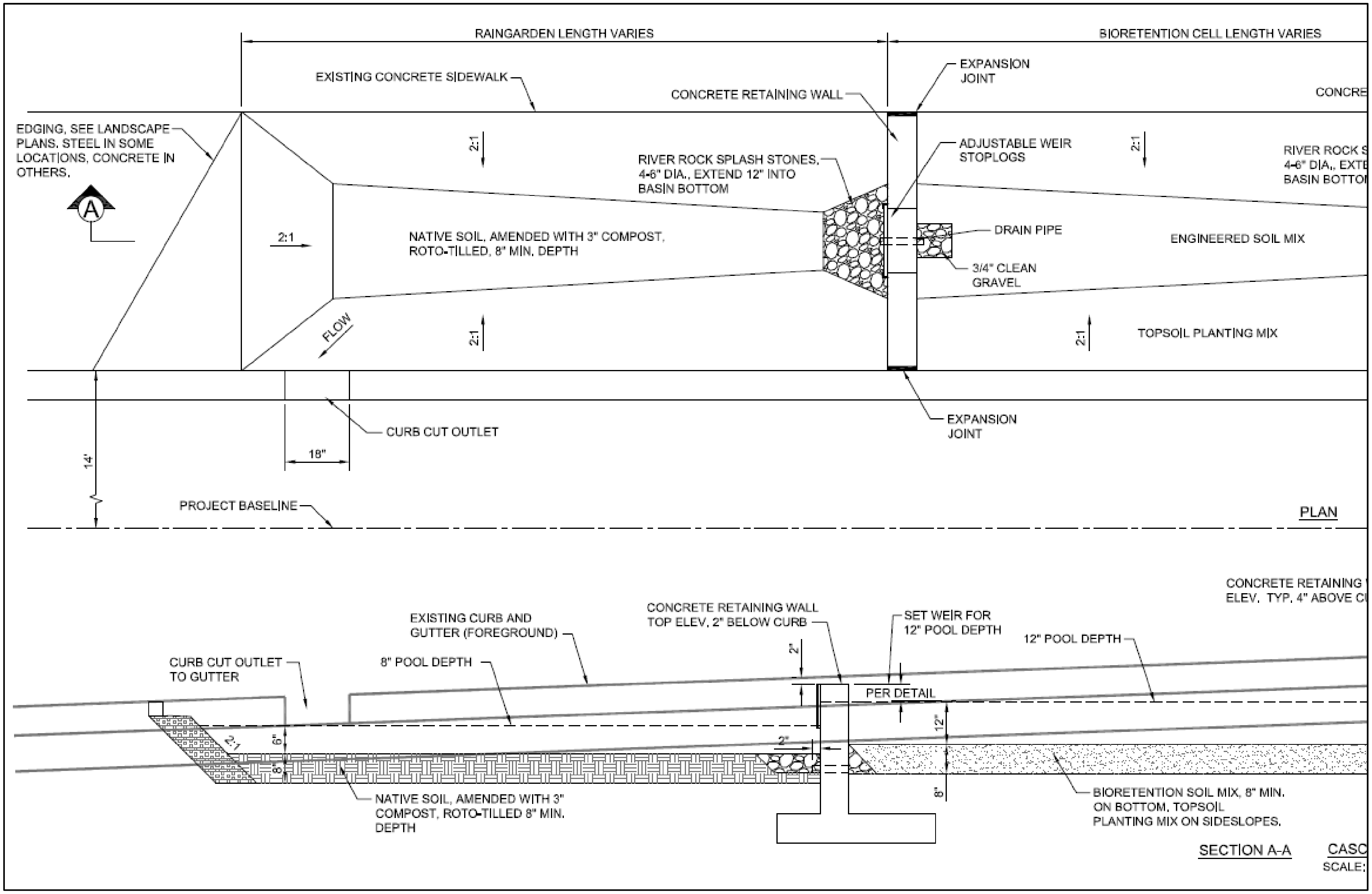


Appendix B-2. Shallow bioretention device typical details for residential streets

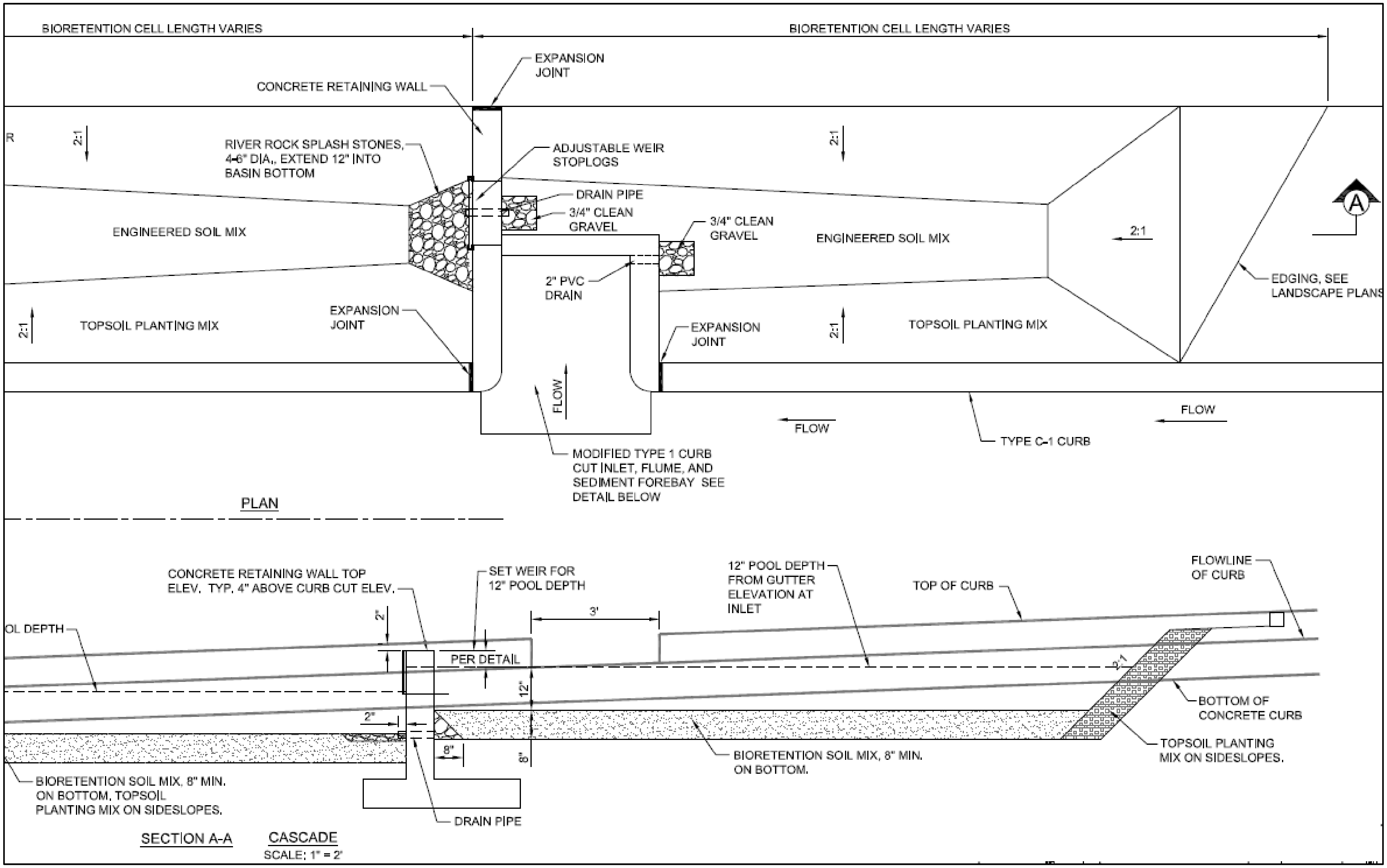



TYPE C-1 CURB
1'
12" POOL DEPTH FROM GUTTER ELEVATION AT INLET
SIDEWALK
2:1
3" HARDWOOD MULCH
TOPSOIL PLANTING MIX ON SIDESLOPES
SUBGRADE
8"
8"
ENGINEERED SOIL MIX, 8" MIN, ON BOTTOM
SMARTDRAIN OR SLOTTED UNDERDRAIN, TYPE SPECIFIED IN PLANS
WATER BARRIER
SECTION A-A
NOT TO SCALE
WATER LEVEL ON DOWN HILL END TO BE 3" BELOW TOP OF CURB AND SIDEWALK TO AVOID OVERTOPPING. WATER LEVEL IS SET BY GUTTER ELEVATION AT INLET.
CURB
CURB CUT AND INLET, LOCATION VARIES
A
2:1
12"
2:1
ENGINEERED SOIL MIX
SMARTDRAIN OR SLOTTED UNDERDRAIN, TYPE SPECIFIED IN PLANS
A
TOPSOIL PLANTING MIX
SMARTDRAIN COUPLING AND DRAIN PIPE TO STORM SEWER
SECTION B-B
NOT TO SCALE

Appendix B-3. Cascade rain garden typical details for residential streets

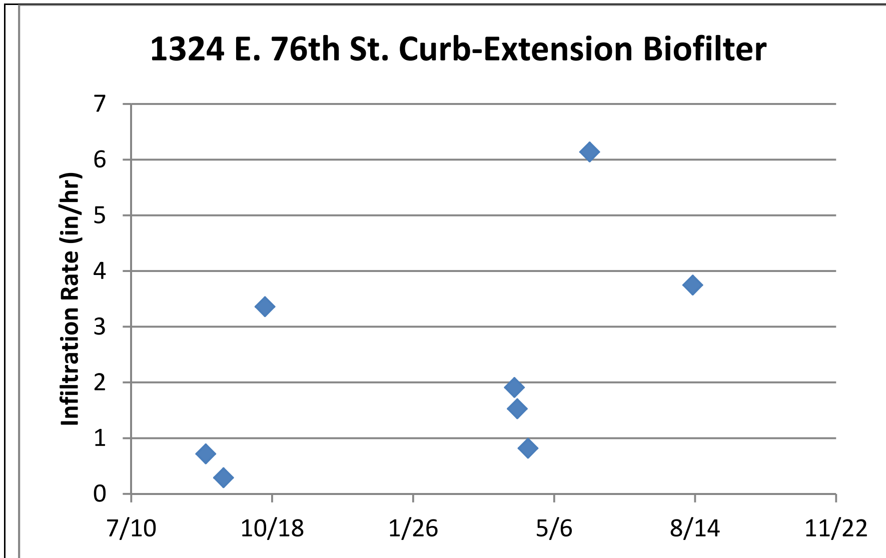


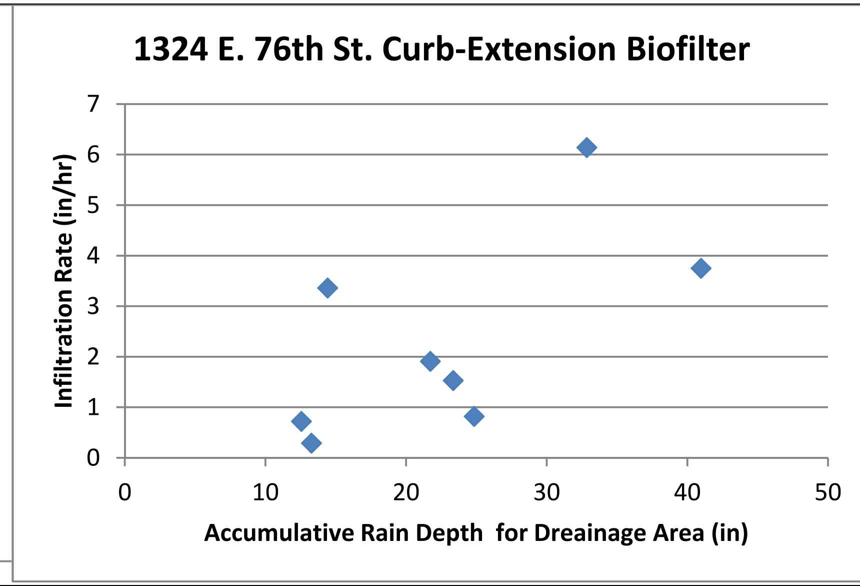
Appendix B-3. Cascade rain garden typical details for residential streets (continued)

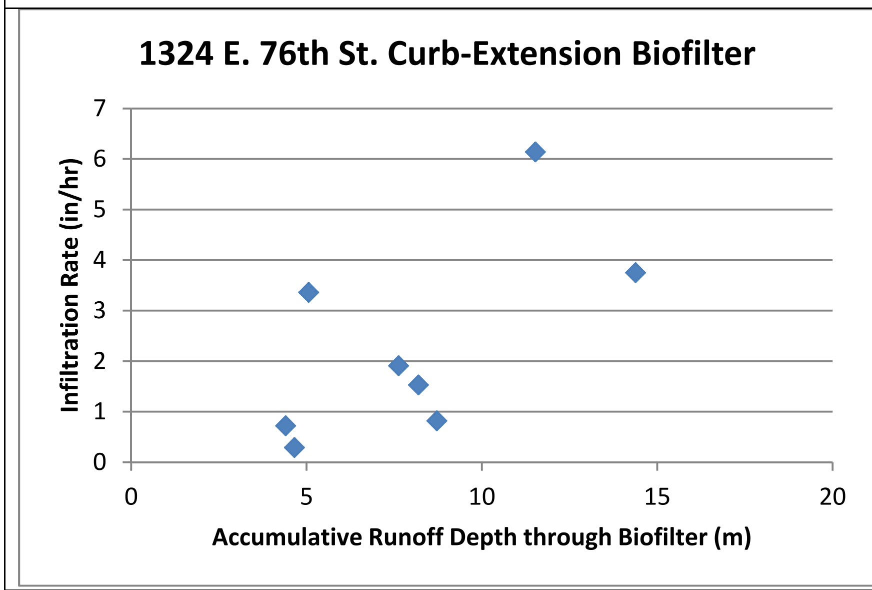


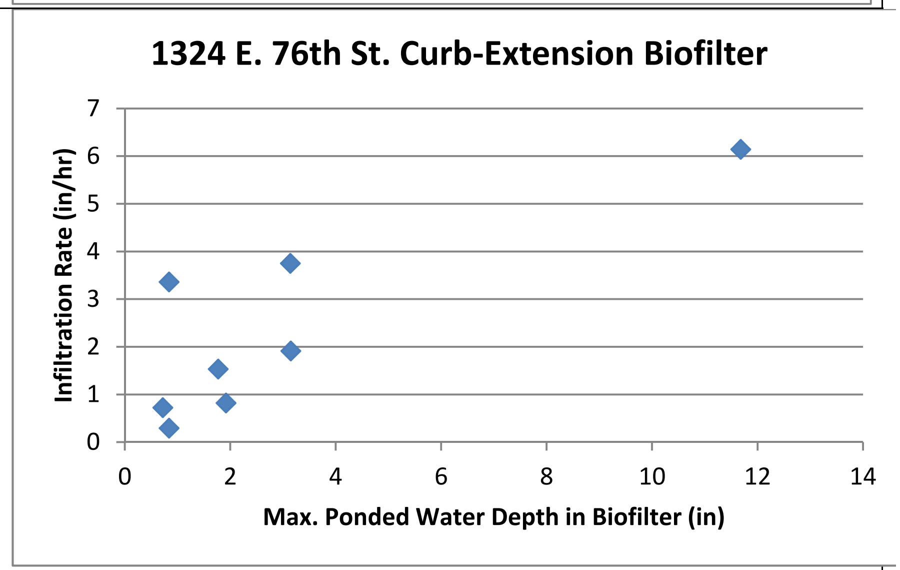
Appendix C: Measured Infiltration Rates in Biofilters

1324 E. 76th St. curb-extension biofilter													
biofilter top area: 264 ft2 (24.2 m2)													
drainage area: 0.42 ac (0.17 ha); biofilter is 1.4% of drainage area													
Rv: 0.2; avg SSC: 205 mg/L													
years of monitoring: 1.35; 4.6 years to reach 10 kg/m2; 11.4 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/20/12 7:05 PM	6/21/12 11:55 AM	16:50	1.03	916	0.08	0.08				8.87	75.4	3.1	0.6
7/25/12 11:15 AM	7/26/12 12:25 PM	1:10	0.49	338	0.03	0.06				9.36	79.5	3.3	0.7
8/31/12 10:20 PM	8/31/12 8:15 PM	21:55	2.61	4000	0.35	0.13	0.7	0.72	18:00	12.56	106.7	4.4	0.9
9/13/12 12:00 PM	9/14/12 10:45 AM	22:45	0.43	2101	0.18	0.43	0.3	0.84	17:30	13.27	112.8	4.7	1.0
10/12/12 9:00 PM	10/14/12 9:40 AM	12:40	0.86	2778	0.24	0.28	3.4	0.84	0:00	14.42	122.5	5.1	1.0
4/7/13 7:52 PM	4/8/13 2:25 AM	6:33	1.1	3556	0.31	0.28	1.9	3.15	1:13	21.73	184.6	7.6	1.6
4/9/13 9:56 PM	4/10/13 6:30 PM	20:34	1.62	3151	0.28	0.17	1.5	1.77	1:01	23.35	198.4	8.2	1.7
4/17/13 11:00 AM	4/18/13 8:41 PM	9:41	1.1	3339	0.29	0.27	0.8	1.92	10:51	24.84	211.1	8.7	1.8
5/31/13 5:26 AM	5/31/13 9:21 AM	3:55	1.34	5710	0.50	0.37	6.1	11.68	0:28	32.85	279.1	11.5	2.4
8/12/13 7:07 AM	8/12/13 10:18 AM	3:11	0.67	N/A	N/A	N/A	3.8	3.14	0:00	40.98	348.2	14.4	2.9
average						0.23	2.3	3.01					
minimum						0.06	0.3	0.72					
maximum						0.43	6.1	11.68					
st dev						0.13	2.0	3.64					
COV						0.56	0.9	1.21					


1324 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
7/10
10/18
1/26
5/6
8/14
11/22


1324 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
10
20
30
40
50
Accumulative Rain Depth for Dreainage Area (in)


1324 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
5
10
15
20
Accumulative Runoff Depth through Biofilter (m)


1324 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
14
Max. Ponded Water Depth in Biofilter (in)

1325 E 76th St. curb-extension biofilter

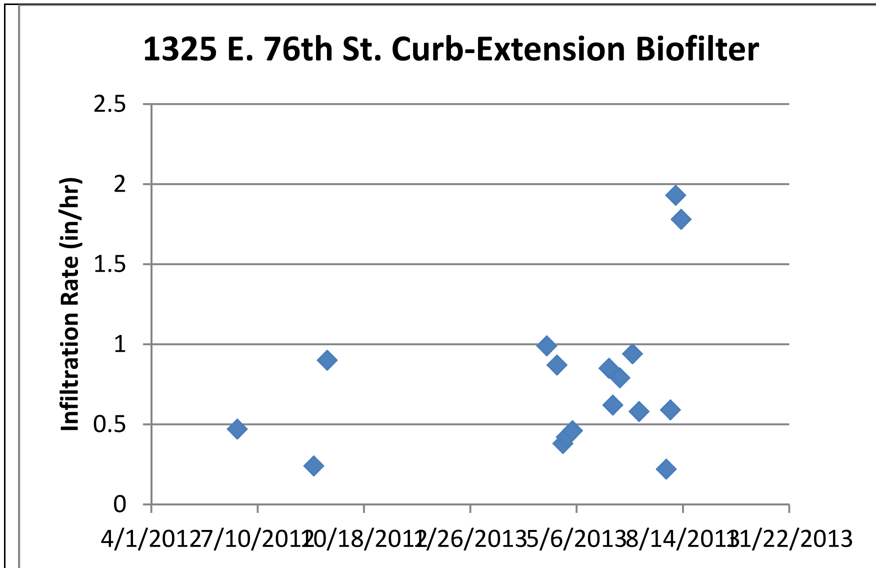
biofilter top area: 264 ft2 (24.2 m2)

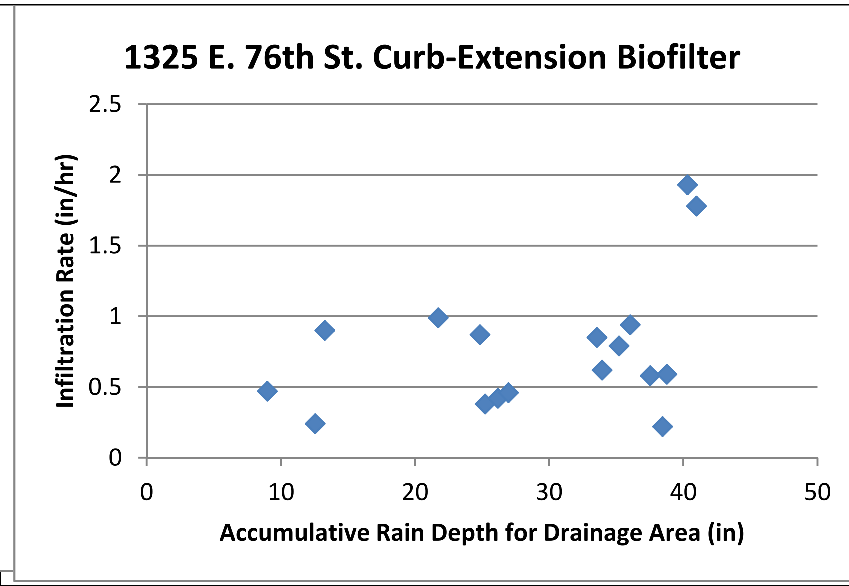
drainage area: 0.096 ac (0.039 ha); biofilter is 6.3% of drainage area

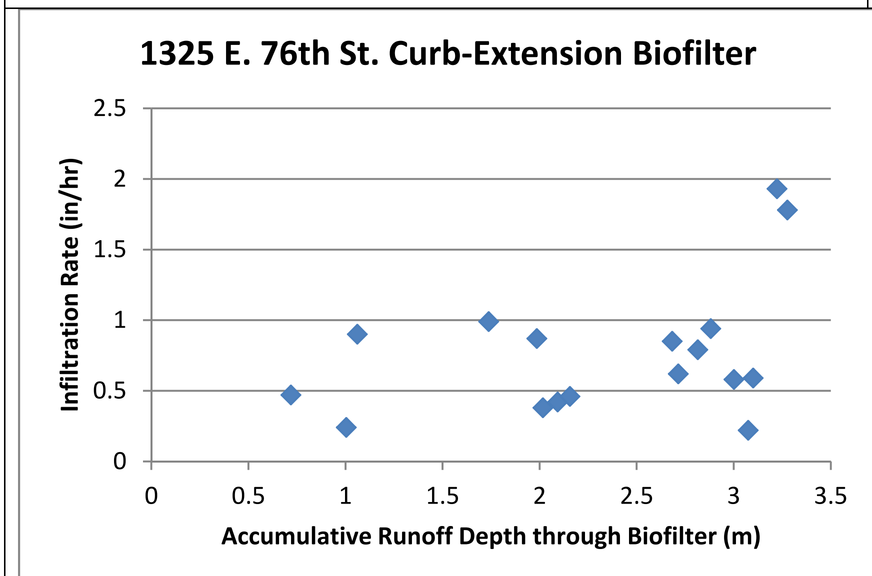
Rv: 0.2; avg SSC: 200 mg/L

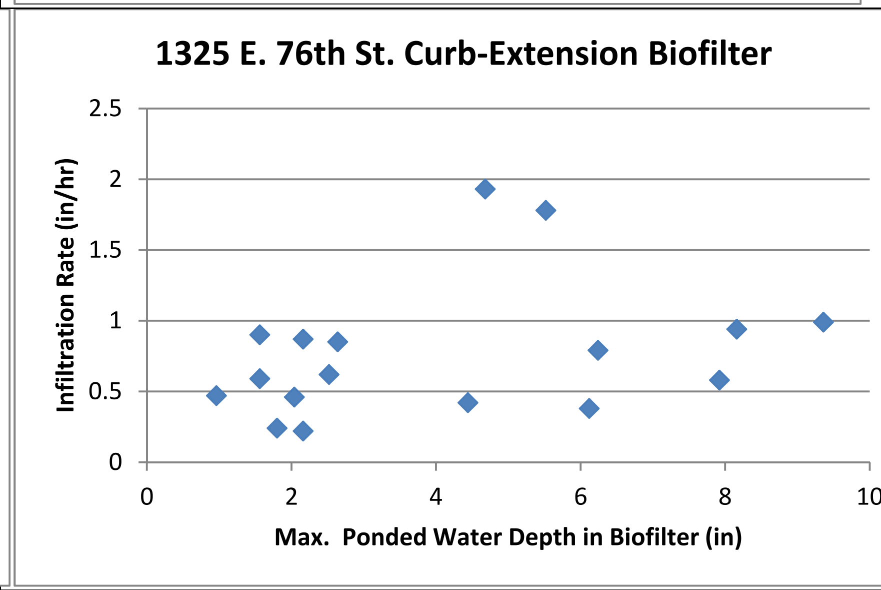
years of monitoring: 1.35; 21 years to reach 10 kg/m2; 52 years to reach 25 kg/m2

Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/10/12 10:05 PM	6/11/12 1:20 PM	15:15	0.8	346.5	0.13	0.17				7.96	15.4	0.6	0.1
6/20/12 7:10 PM	6/21/12 11:55 AM	16:45	1.03	1370	0.53	0.51	0.5	0.96	9:45	8.99	17.4	0.7	0.1
7/25/12 5:00 PM	7/26/12 12:30 PM	19:30	0.49	328	0.13	0.26				9.48	18.4	0.8	0.2
8/31/12 7:00 PM	8/31/12 6:15 PM	23:15	2.61	4870	1.87	0.72	0.2	1.80	20:15	12.56	24.3	1.0	0.2
9/13/12 11:10 AM	9/14/12 10:55 AM	23:45	0.43	5954	2.28	5.31	0.9	1.56	0:15	13.27	25.7	1.1	0.2
10/12/12 7:00 PM	10/14/12 2:20 AM	7:20	0.86	1553	0.60	0.69				14.42	27.9	1.2	0.2
4/7/13 7:52 PM	4/8/13 2:25 AM	6:33	1.1	2452	0.94	0.86	1.0	9.36	5:25	21.73	42.1	1.7	0.3
4/17/13 11:00 AM	4/18/13 8:41 PM	9:41	1.1	3947	1.51	1.38	0.9	2.16	12:03	24.84	48.1	2.0	0.4
4/23/13 1:34 AM	4/23/13 10:53 AM	9:19	0.39	5082	1.95	5.00	0.4	6.12	4:14	25.23	48.8	2.0	0.4
4/26/13 4:19 AM	4/27/13 11:02 AM	6:43	0.95	7504	2.88	3.03	0.4	4.44	23:31	26.18	50.7	2.1	0.4
5/2/13 3:08 AM	5/4/13 4:11 AM	1:03	1.42	2471	0.95	0.67	0.5	2.04	12:10	26.97	52.2	2.2	0.4
6/5/13 9:44 AM	6/5/13 12:47 PM	3:03	0.47	632	0.24	0.52	0.9	2.64	3:05	33.56	65.0	2.7	0.5
6/9/13 12:54 AM	6/9/13 3:53 AM	2:59	0.39	540	0.21	0.53	0.6	2.52	1:55	33.95	65.7	2.7	0.5
6/15/13 3:50 PM	6/15/13 10:05 PM	6:15	1.26	N/A	N/A	N/A	0.8	6.24	0:14	35.21	68.2	2.8	0.6
6/27/13 11:46 AM	6/28/13 12:49 AM	13:03	0.83	N/A	N/A	N/A	0.9	8.16	11:18	36.04	69.8	2.9	0.6
7/3/13 5:54 PM	7/3/13 9:36 PM	3:42	1.5	N/A	N/A	N/A	0.6	7.92	2:10	37.54	72.7	3.0	0.6
7/29/13 6:27 AM	7/30/13 9:41 AM	3:14	0.91	N/A	N/A	N/A	0.2	2.16	11:26	38.45	74.4	3.1	0.6
8/2/13 3:46 AM	8/2/13 8:10 AM	4:24	0.32	N/A	N/A	N/A	0.6	1.56	2:07	38.77	75.1	3.1	0.6
8/7/13 4:13 AM	8/7/13 8:45 AM	4:32	0.87	N/A	N/A	N/A	1.9	4.68	2:25	40.31	78.0	3.2	0.6
8/12/13 7:07 AM	8/12/13 10:18 AM	3:11	0.67	N/A	N/A	N/A	1.8	5.52	0:16	40.98	79.3	3.3	0.7
average						1.51	0.8	4.11					
minimum						0.17	0.2	0.96					
maximum						5.31	1.9	9.36					
st dev						1.77	0.5	2.67					
COV						1.17	0.6	0.65					

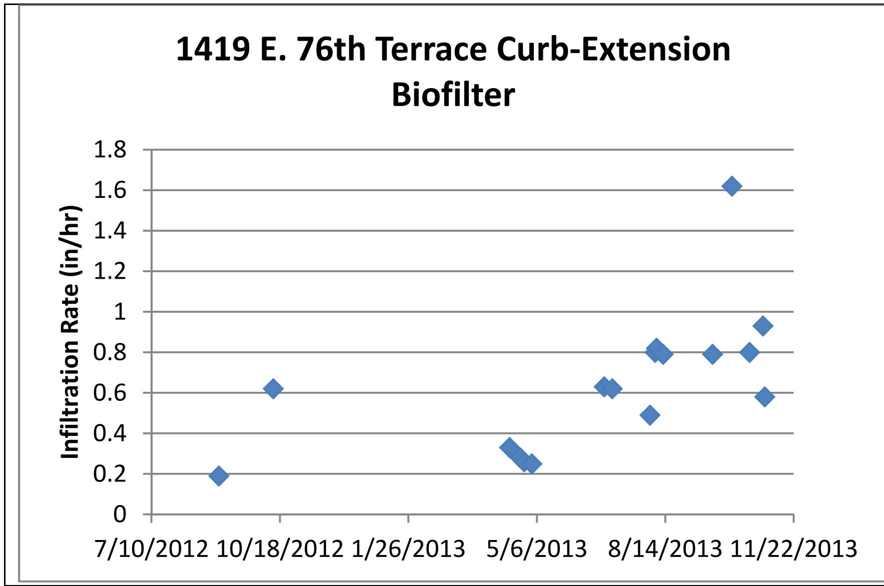

1325 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
2.5
2
1.5
1
0.5
0
4/1/2012
7/10/2012
10/18/2012
1/26/2013
5/6/2013
8/14/2013
11/22/2013

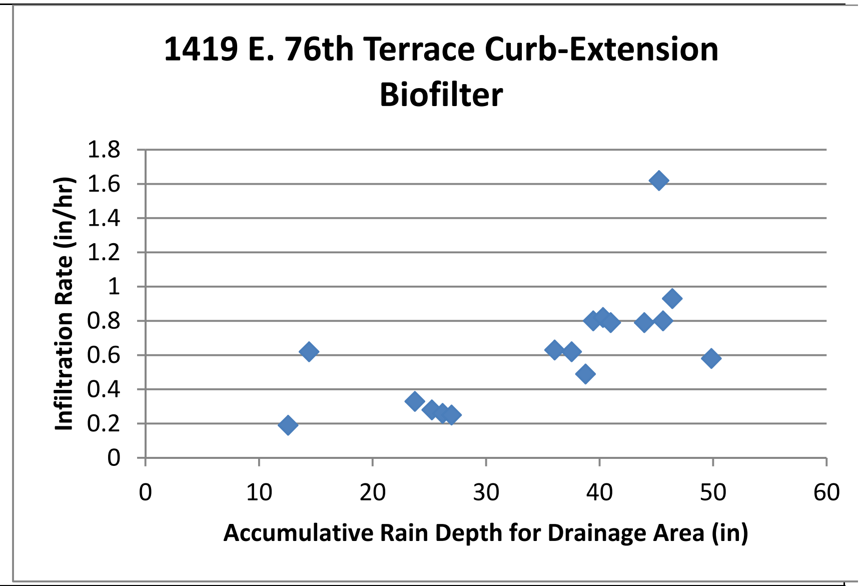

1325 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
2.5
2
1.5
1
0.5
0
0
10
20
30
40
50
Accumulative Rain Depth for Drainage Area (in)

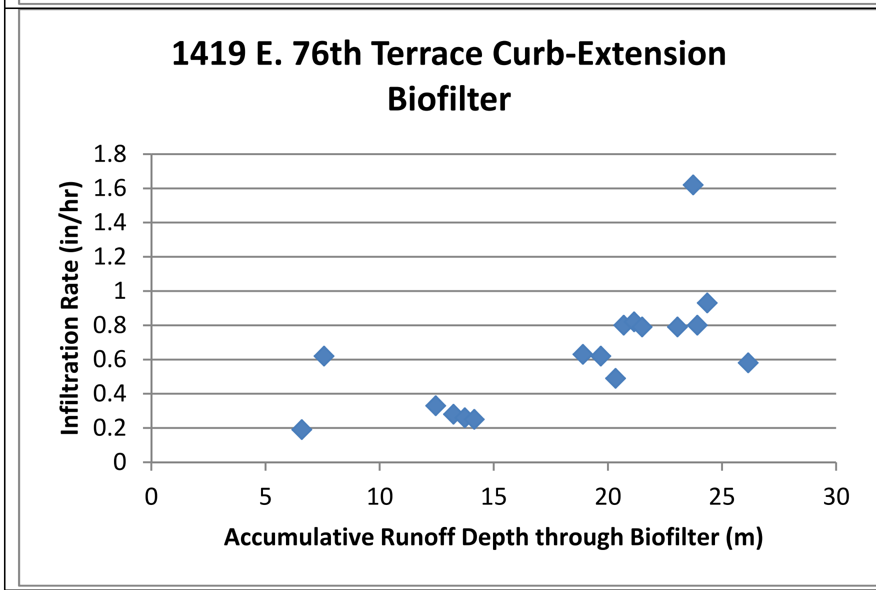

1325 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3
3.5
Accumulative Runoff Depth through Biofilter (m)

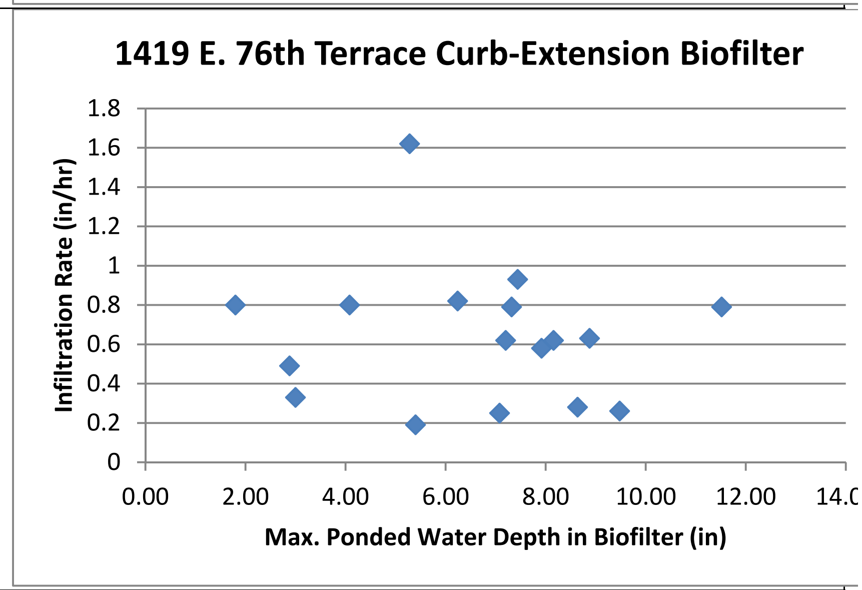

1325 E. 76th St. Curb-Extension Biofilter
Infiltration Rate (in/hr)
2.5
2
1.5
1
0.5
0
0
2
4
6
8
10
Max. Ponded Water Depth in Biofilter (in)

1419 E 76th Terr. curb-extension biofilter													
biofilter top area: 264 ft2 (24.2 m2)													
drainage area: 0.63 ac (0.25 ha); biofilter is 1.0% of drainage area													
Rv: 0.2; avg SSC: 133 mg/L													
years of monitoring: 1.6; 4.7 years to reach 10 kg/m2; 12 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/21/12 12:55 AM	6/21/12 11:25 AM	10:30	1.03	232	0.01	0.01				8.99	114.2	4.7	0.6
7/25/12 5:55 PM	7/26/12 8:30 AM	12:35	0.49	103	0.01	0.01		2.35	7:30	9.48	120.4	5.0	0.7
8/31/12 11:00 AM	8/31/12 5:00 PM	6:00	2.61	1940	0.11	0.04	0.2	5.40	4:15	12.56	159.5	6.6	0.9
9/13/12 2:10 PM	9/14/12 10:10 AM	20:00	0.43	3987	0.23	0.54		6.50	2:30	13.27	168.6	7.0	0.9
9/26/12 2:25 AM	9/26/12 04:30:00 AM	2:05	0.23	583	0.03	0.15				13.51	171.6	7.1	0.9
10/12/12 9:00 PM	10/13/12 10:15 PM	1:15	0.86	3487	0.20	0.24	0.6	7.20	4:55	14.42	183.2	7.6	1.0
4/14/13 7:02 PM	4/15/13 7:08 AM	12:06	0.39	N/A	N/A	N/A	0.3	3.00	13:51	23.74	301.6	12.5	1.7
4/23/13 1:34 AM	4/23/13 10:53 AM	9:19	0.39	1281	0.07	0.19	0.3	8.64	7:19	25.23	320.5	13.2	1.8
4/26/13 4:19 AM	4/27/13 11:02 AM	6:43	0.95	1550	0.09	0.10	0.3	9.48	3:19	26.18	332.5	13.7	1.8
5/2/13 3:08 AM	5/4/13 4:11 AM	1:03	1.42	1400	0.08	0.06	0.3	7.08	2:51	26.97	342.6	14.1	1.9
6/27/13 11:46 AM	6/28/13 12:49 AM	13:03	0.83	N/A	N/A	N/A	0.6	8.88	12:03	36.04	457.8	18.9	2.5
7/3/13 5:54 PM	7/3/13 9:36 PM	3:42	1.5	N/A	N/A	N/A	0.6	8.16	1:11	37.54	476.8	19.7	2.6
8/2/13 3:46 AM	8/2/13 8:10 AM	4:24	0.32	N/A	N/A	N/A	0.5	2.88	1:53	38.77	492.5	20.3	2.7
8/6/13 3:24 AM	8/6/13 4:59 AM	1:35	0.55	N/A	N/A	N/A	0.8	4.08	1:30	39.44	501.0	20.7	2.8
8/7/13 4:13 AM	8/7/13 8:45 AM	4:32	0.87	N/A	N/A	N/A	0.8	6.24	2:41	40.31	512.0	21.1	2.8
8/12/13 7:07 AM	8/12/13 10:18 AM	3:11	0.67	N/A	N/A	N/A	0.8	7.32	0:17	40.98	520.5	21.5	2.9
9/19/13 7:00 PM	9/19/13 10:23 PM	3:23	1.89	N/A	N/A	N/A	0.8	11.52	0:25	43.94	558.1	23.1	3.1
10/4/13 10:36 PM	10/5/13 3:48 AM	5:12	0.87	N/A	N/A	N/A	1.6	5.28	1:34	45.24	574.7	23.7	3.2
10/18/13 2:03 PM	10/18/13 5:05 PM	3:02	0.12	N/A	N/A	N/A	0.8	1.80	0:00	45.5898	579.1	23.9	3.2
10/29/13 2:52 AM	10/29/13 9:41 AM	6:49	0.83	N/A	N/A	N/A	0.9	7.44	4:19	46.4172	589.6	24.4	3.2
10/30/13 12:09 PM	10/31/13 11:46 AM	23:37	3.43	N/A	N/A	N/A	0.6	7.92	17:49	49.845	633.1	26.1	3.5
average						0.15	0.6	6.38					
minimum						0.01	0.2	1.80					
maximum						0.54	1.6	11.52					
st dev						0.17	0.3	2.63					
COV						1.12	0.5	0.41					



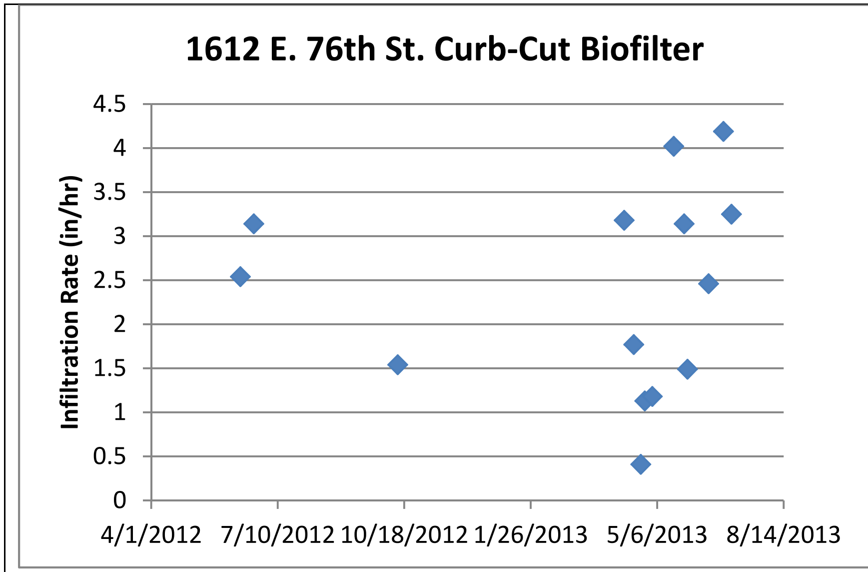


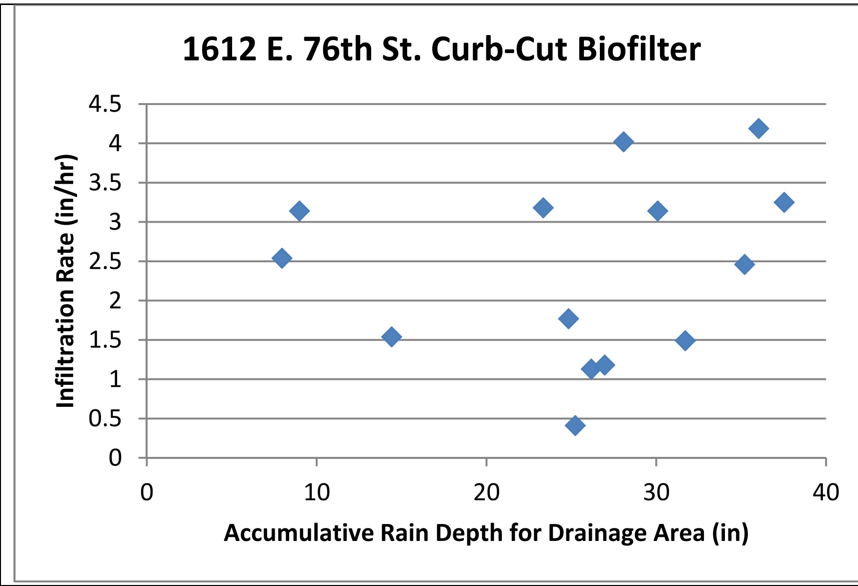


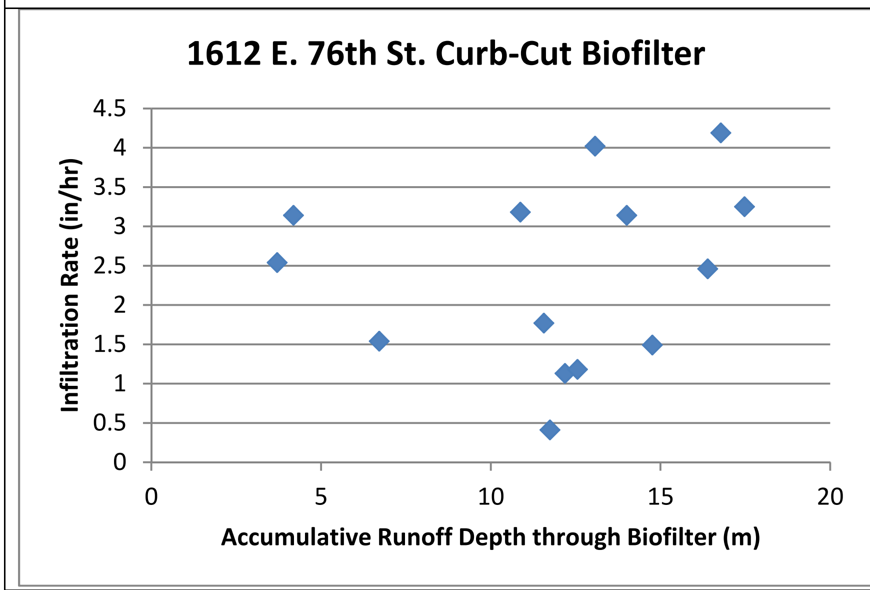


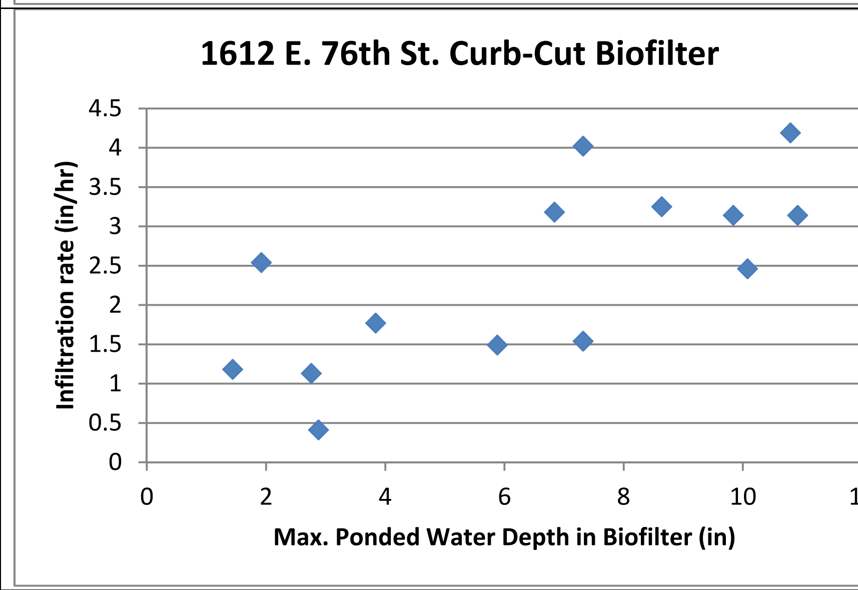
1612 E 76th St. curb-cut biofilter
biofilter top area: 282 ft2 (25.9 m2)
drainage area: 0.60 ac (0.24 ha); biofilter is 1.1% of drainage area

Rv: 0.2; avg SSC: 166 mg/L													
years of monitoring: 1.25; 4.3 years to reach 10 kg/m2; 11 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/10/12 9:45 AM	6/11/12 10:00 AM	0:15	0.8	1.1	0.00	0.00	2.5	1.92	20:00	7.96	95.8	3.7	0.6
6/21/12 12:12 AM	6/21/12 12:02 PM	11:50	1.03	1061	0.07	0.06	3.1	9.84	1:45	8.99	108.2	4.2	0.7
8/31/12 11:00 AM	9/1/12 3:00 PM	4:00	5.6	1194	0.07	0.01				12.56	151.2	5.8	1.0
9/13/12 2:40 PM	9/13/12 8:25 PM	5:45	0.43	40	0.00	0.01				13.27	159.8	6.2	1.0
9/26/12 2:55 AM	9/26/12 5:30 AM	2:35	0.23	30	0.00	0.01				13.51	162.7	6.3	1.0
10/12/12 9:00 PM	10/13/12 9:05 PM	0:05	0.86	754	0.05	0.05	1.5	7.32	9:00	14.42	173.6	6.7	1.1
4/9/13 9:56 PM	4/10/13 6:30 PM	20:34	1.62	7539	0.47	0.29	3.2	6.84	1:13	23.35	281.1	10.9	1.8
4/17/13 11:00 AM	4/18/13 8:41 PM	9:41	1.1	N/A	N/A	N/A	1.8	3.84	0:58	24.84	299.1	11.6	1.9
4/23/13 1:34 AM	4/23/13 10:53 AM	9:19	0.39	1837	0.11	0.29	0.4	2.88	4:50	25.23	303.8	11.7	1.9
4/26/13 4:19 AM	4/27/13 11:02 AM	6:43	0.95	1220	0.08	0.08	1.1	2.76	0:20	26.18	315.2	12.2	2.0
5/2/13 3:08 AM	5/4/13 4:11 AM	1:03	1.42	N/A	N/A	N/A	1.2	1.44	2:01	26.97	324.7	12.6	2.1
5/19/13 2:21 AM	5/20/13 1:44 AM	23:23	0.83	N/A	N/A	N/A	4.0	7.32	1:18	28.08	338.1	13.1	2.2
5/27/13 8:31 AM	5/27/13 12:35 PM	4:04	2.01	12126	0.75	0.37	3.1	10.92	1:40	30.09	362.3	14.0	2.3
5/29/13 10:53 PM	5/30/13 3:17 PM	14:52	1.62	8855	0.55	0.34	1.5	5.88	3:48	31.71	381.8	14.8	2.5
6/15/13 3:50 PM	6/15/13 10:05 PM	6:15	1.26	N/A	N/A	N/A	2.5	10.08	0:06	35.21	423.9	16.4	2.7
6/27/13 11:46 AM	6/28/13 12:49 AM	13:03	0.83	N/A	N/A	N/A	4.2	10.80	11:26	36.04	433.9	16.8	2.8
7/3/13 5:54 PM	7/3/13 9:36 PM	3:42	1.5	N/A	N/A	N/A	3.3	8.64	0:03	37.54	452.0	17.5	2.9
average						0.14	2.39	6.46					
minimum						0.00	0.41	1.44					
maximum						0.37	4.19	10.92					
st dev						0.15	1.15	3.39					
COV						1.09	0.48	0.52					



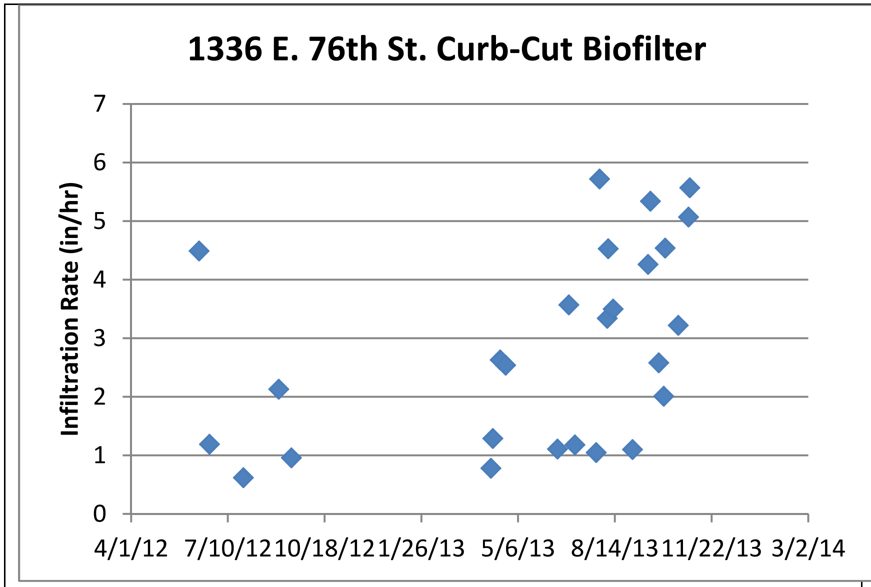


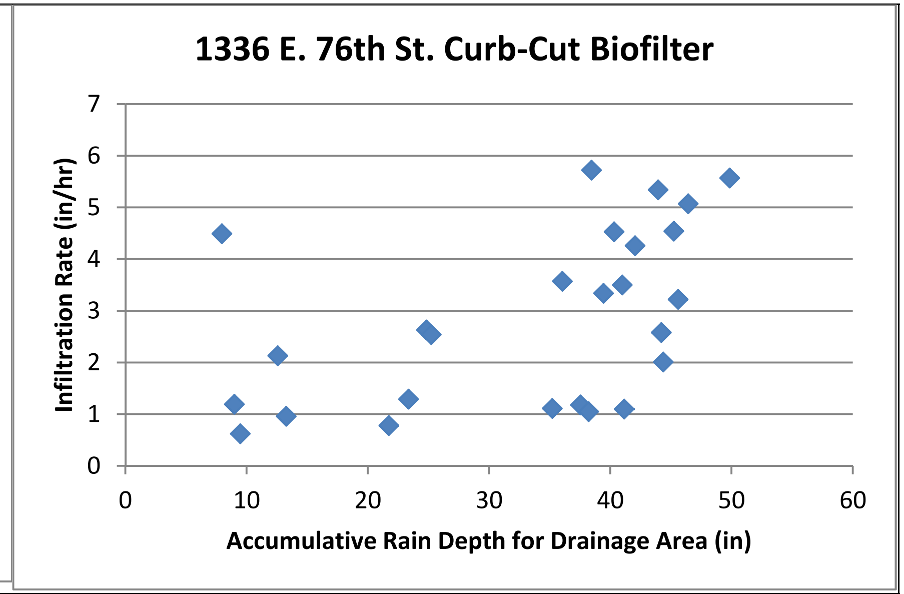


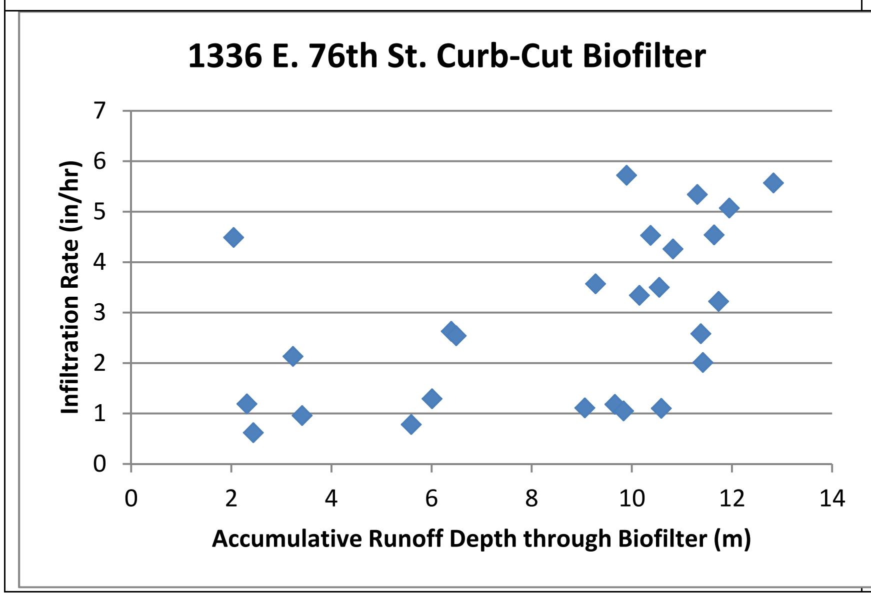


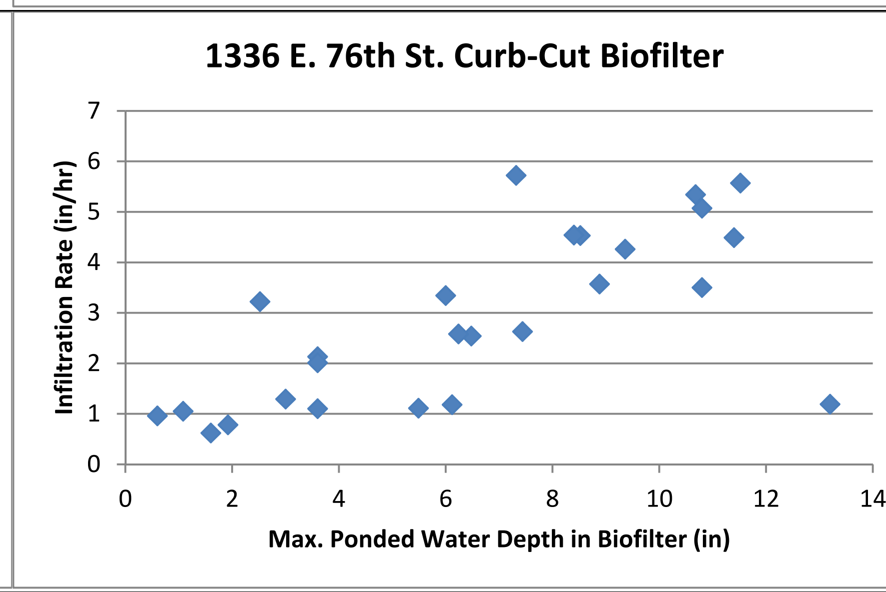
1336 E 76th St. curb-cut biofilter
biofilter top area: 282 ft2 (25.9 m2)
drainage area: 0.33 ac (0.13 ha); biofilter is 2.0% of drainage area

Rv: 0.2; avg SSC: 166 mg/L													
years of monitoring: 1.6; 7.5 years to reach 10 kg/m2; 19 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
5/29/12 5:17 AM	5/30/12 6:07 PM	12:47	0.29	289	0.03	0.11				6.74	44.9	1.7	0.3
6/10/12 7:02 AM	6/11/12 3:02 PM	8:00	0.8	14.7	0.00	0.00	4.5	11.40	20:00	7.96	53.0	2.0	0.3
6/21/12 12:17 AM	6/21/12 12:02 PM	11:45	1.03	3884	0.43	0.42	1.2	13.20	1:00	8.99	59.8	2.3	0.4
7/26/12 1:31 AM	7/26/12 12:16 PM	10:45	0.49	75	0.01	0.02	0.6	1.60	4:00	9.48	63.1	2.4	0.4
8/31/12 3:47 PM	9/2/12 11:02 AM	19:15	5.6	6877	0.77	0.14	2.1	3.60	9:30	12.56	83.6	3.2	0.5
9/13/12 2:37 PM	9/14/12 10:47 AM	20:45	0.43	1692	0.19	0.44	1.0	0.60	2:00	13.27	88.3	3.4	0.6
9/26/12 3:02 AM	9/26/12 7:22 AM	4:20	0.23	156	0.02	0.08				13.51	89.9	3.5	0.6
10/11/12 5:32 PM	10/14/12 10:47 AM	17:15	0.86	293	0.03	0.04				14.42	96.0	3.7	0.6
4/7/13 7:52 PM	4/8/13 2:25 AM	6:33	1.1	502	0.06	0.05	0.8	1.92	13:00	21.73	144.6	5.6	0.9
4/9/13 9:56 PM	4/10/13 6:30 PM	20:34	1.62	8419	0.94	0.58	1.3	3.00	0:57	23.35	155.4	6.0	1.0
4/17/13 11:00 AM	4/18/13 8:41 PM	9:41	1.1	N/A	N/A	N/A	2.6	7.44	0:00	24.84	165.4	6.4	1.1
4/23/13 1:34 AM	4/23/13 10:53 AM	9:19	0.39	N/A	N/A	N/A	2.5	6.48	4:04	25.23	167.9	6.5	1.1
6/15/13 3:50 PM	6/15/13 10:05 PM	6:15	1.26	6748	0.75	0.60	1.1	5.49	0:16	35.21	234.4	9.1	1.5
6/27/13 11:46 AM	6/28/13 12:49 AM	13:03	0.83	5041	0.56	0.68	3.6	8.88	11:25	36.04	239.9	9.3	1.5
7/3/13 5:54 PM	7/3/13 9:36 PM	3:42	1.5	N/A	N/A	N/A	1.2	6.12	2:17	37.54	249.9	9.7	1.6
7/25/13 5:00 PM	7/26/13 11:00 AM	18:00	0.24	N/A	N/A	N/A	1.1	1.08	2:15	38.21	254.4	9.8	1.6
7/29/13 6:27 AM	7/30/13 9:41 AM	3:14	0.91	N/A	N/A	N/A	5.7	7.32	8:03	38.45	255.9	9.9	1.6
8/6/13 3:24 AM	8/6/13 4:59 AM	1:35	0.55	N/A	N/A	N/A	3.3	6.00	0:37	39.44	262.5	10.2	1.7
8/7/13 4:13 AM	8/7/13 8:45 AM	4:32	0.87	N/A	N/A	N/A	4.5	8.52	2:48	40.31	268.3	10.4	1.7
8/12/13 7:07 AM	8/12/13 10:18 AM	3:11	0.67	N/A	N/A	N/A	3.5	10.80	0:24	40.98	272.8	10.5	1.8
9/1/13 7:42 AM	9/1/13 9:02 AM	1:20	0.16	N/A	N/A	N/A	1.1	3.60	0:05	41.1376	273.8	10.6	1.8
9/17/13 7:04 AM	9/17/13 3:14 PM	8:10	0.79	N/A	N/A	N/A	4.3	9.36	3:14	42.0438	279.9	10.8	1.8
9/19/13 7:00 PM	9/19/13 10:23 PM	3:23	1.89	N/A	N/A	N/A	5.3	10.68	2:49	43.935	292.5	11.3	1.9
9/28/13 8:29 AM	9/28/13 11:29 AM	3:00	0.28	N/A	N/A	N/A	2.6	6.24	0:51	44.2108	294.3	11.4	1.9
10/3/13 11:04 AM	10/3/13 11:24 AM	0:20	0.16	N/A	N/A	N/A	2.0	3.60	0:02	44.3684	295.3	11.4	1.9
10/4/13 10:36 PM	10/5/13 3:48 AM	5:12	0.87	N/A	N/A	N/A	4.5	8.40	0:30	45.2352	301.1	11.6	1.9
10/18/13 2:03 PM	10/18/13 5:05 PM	3:02	0.12	N/A	N/A	N/A	3.2	2.52	0:04	45.5898	303.5	11.7	1.9
10/29/13 2:52 AM	10/29/13 9:41 AM	6:49	0.83	N/A	N/A	N/A	5.1	10.80	3:45	46.4172	309.0	11.9	2.0
10/30/13 12:09 PM	10/31/13 11:46 AM	23:37	3.43	N/A	N/A	N/A	5.6	11.52	0:29	49.845	331.8	12.8	2.1
average						0.26	2.9	6.55					
minimum						0.00	0.6	0.60					
maximum						0.68	5.7	13.20					
st dev						0.26	1.7	3.66					
COV						0.99	0.6	0.56					

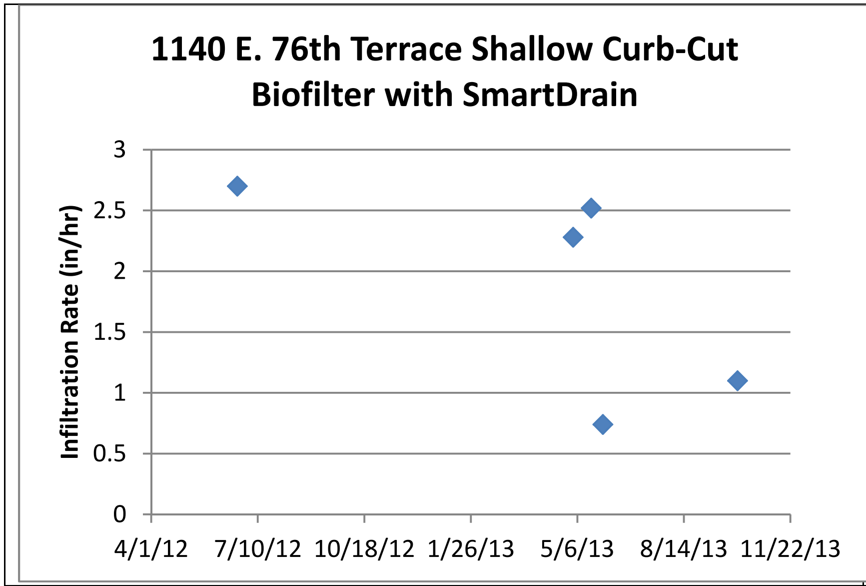

1336 E. 76th St. Curb-Cut Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
4/1/12
7/10/12
10/18/12
1/26/13
5/6/13
8/14/13
11/22/13
3/2/14

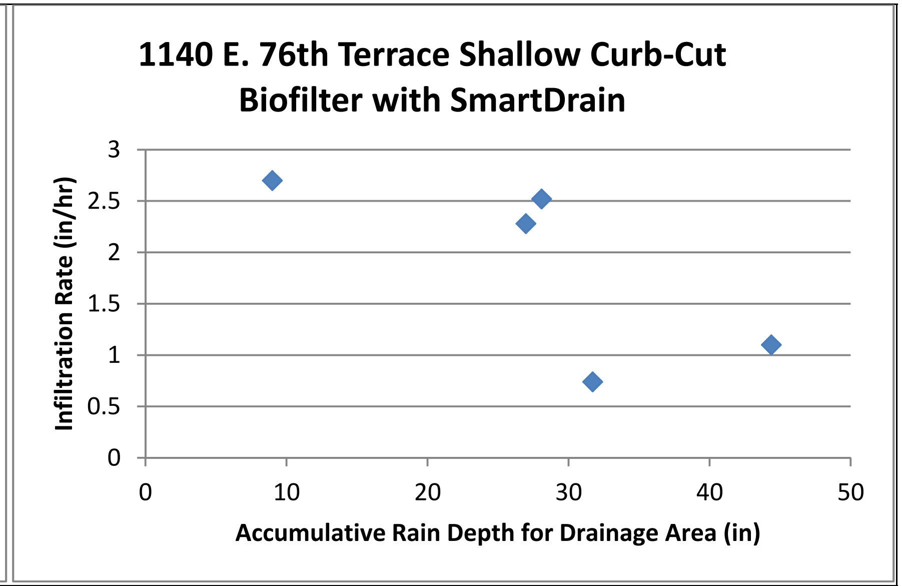

1336 E. 76th St. Curb-Cut Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
10
20
30
40
50
60
Accumulative Rain Depth for Drainage Area (in)

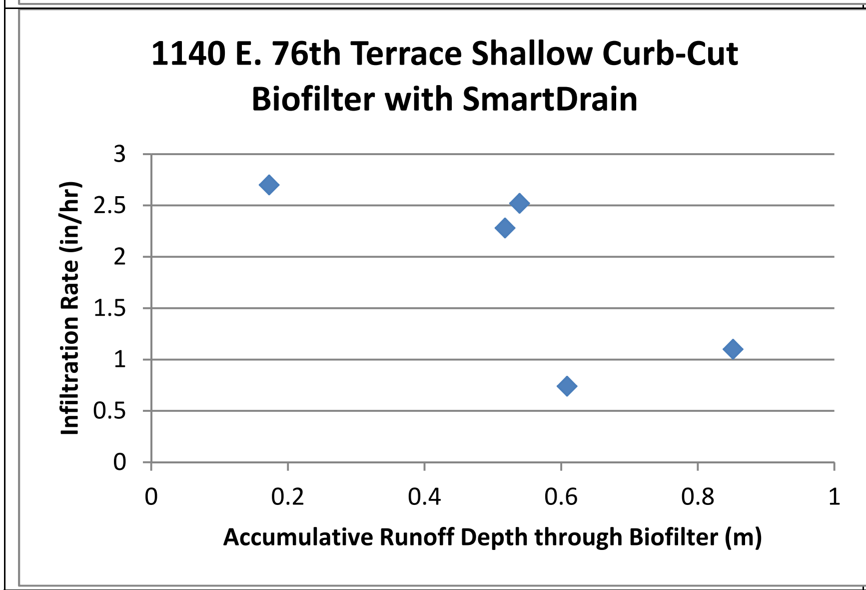

1336 E. 76th St. Curb-Cut Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
14
Accumulative Runoff Depth through Biofilter (m)

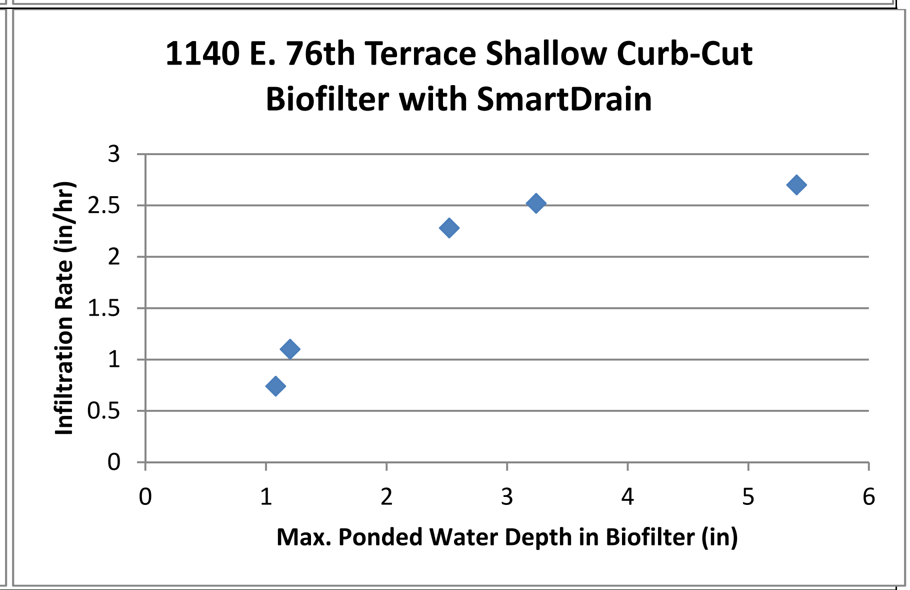

1336 E. 76th St. Curb-Cut Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
14
Max. Ponded Water Depth in Biofilter (in)

1140 E 76th Terr. Shallow curb-cut biofilter with SmartDrain													
biofilter top area: 282 ft2 (25.9 m2)													
drainage area: 0.025 ac (0.010 ha); biofilter is 26% of drainage area													
Rv: 0.2; avg SSC: 166 mg/L													
years of monitoring: 1.6; >100 years to reach 10 kg/m2; >100 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/11/12 2:03 AM	6/11/12 12:03 PM	10:00	0.8	18.2	0.03	0.03				7.96	4.0	0.2	0.0
6/20/12 7:22 PM	6/21/12 11:32 AM	16:10	1.03	14203	21.25	20.63	2.7	5.40	6:35	8.99	4.5	0.2	0.0
8/31/12 11:02 AM	9/1/12 6:02 PM	7:00	5.6	46827	70.05	12.51				12.56	6.2	0.2	0.0
9/13/12 2:37 PM	9/14/12 10:34 AM	20:00	0.43	N/A	N/A	N/A				13.27	6.6	0.3	0.0
9/25/12 11:54 PM	9/26/12 8:34 AM	8:40	0.23	869	1.30	5.65				13.51	6.7	0.3	0.0
10/12/12 8:59 PM	10/12/12 10:39 PM	1:40	0.86	536	0.80	0.93				14.42	7.2	0.3	0.0
5/2/13 3:08 AM	5/4/13 4:11 AM	1:03	1.42	785	1.17	0.83	2.3	2.52	13:30	26.97	13.4	0.5	0.1
5/19/13 2:21 AM	5/20/13 1:44 AM	23:23	0.83	N/A	N/A	N/A	2.5	3.24	4:48	28.08	13.9	0.5	0.1
5/29/13 10:53 PM	5/30/13 3:17 PM	16:24	1.62	N/A	N/A	N/A	0.7	1.08	9:46	31.71	15.7	0.6	0.1
10/3/13 11:04 AM	10/3/13 11:24 AM	0:20	0.16	N/A	N/A	N/A	1.1	1.20	0:07	44.37	22.0	0.9	0.1
average						6.76	1.9	2.69					
minimum						0.03	0.7	1.08					
maximum						20.63	2.7	5.40					
st dev						8.26	0.9	1.77					
COV						1.22	0.5	0.66					

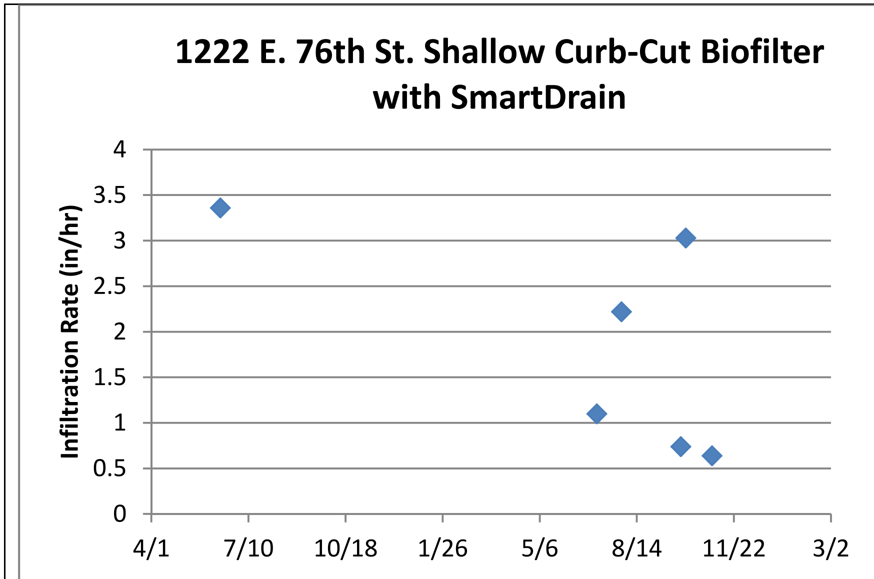

1140 E. 76th Terrace Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
4/1/12
7/10/12
10/18/12
1/26/13
5/6/13
8/14/13
11/22/13

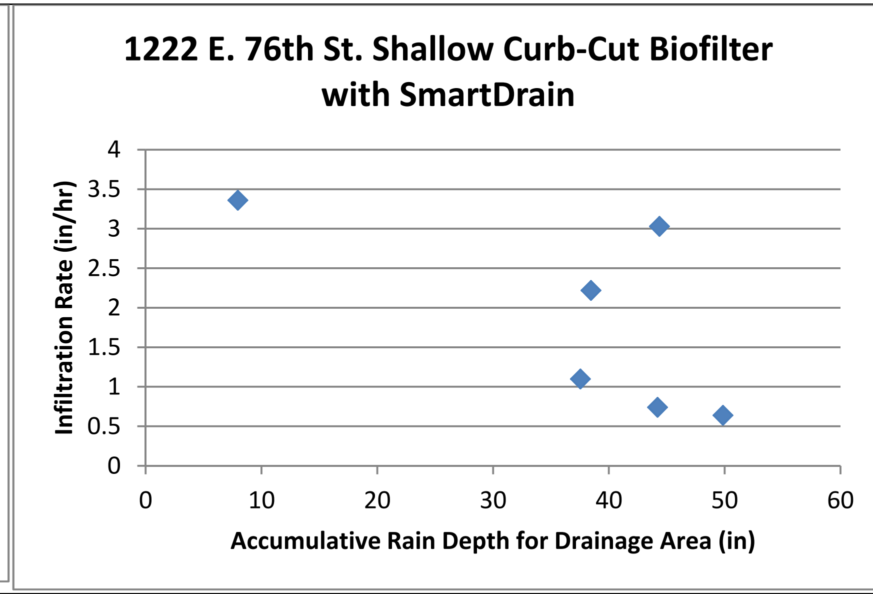

1140 E. 76th Terrace Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
0
10
20
30
40
50
Accumulative Rain Depth for Drainage Area (in)

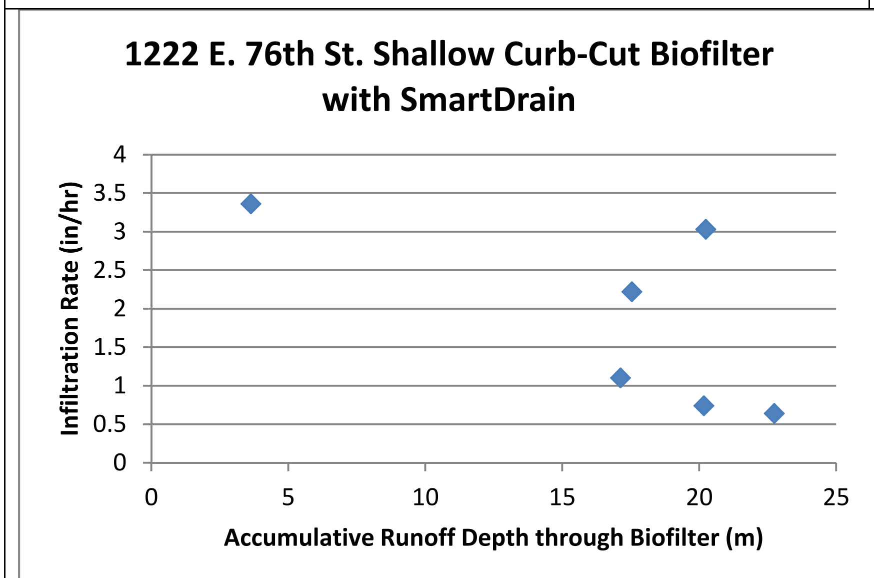

1140 E. 76th Terrace Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
0
0.2
0.4
0.6
0.8
1
Accumulative Runoff Depth through Biofilter (m)

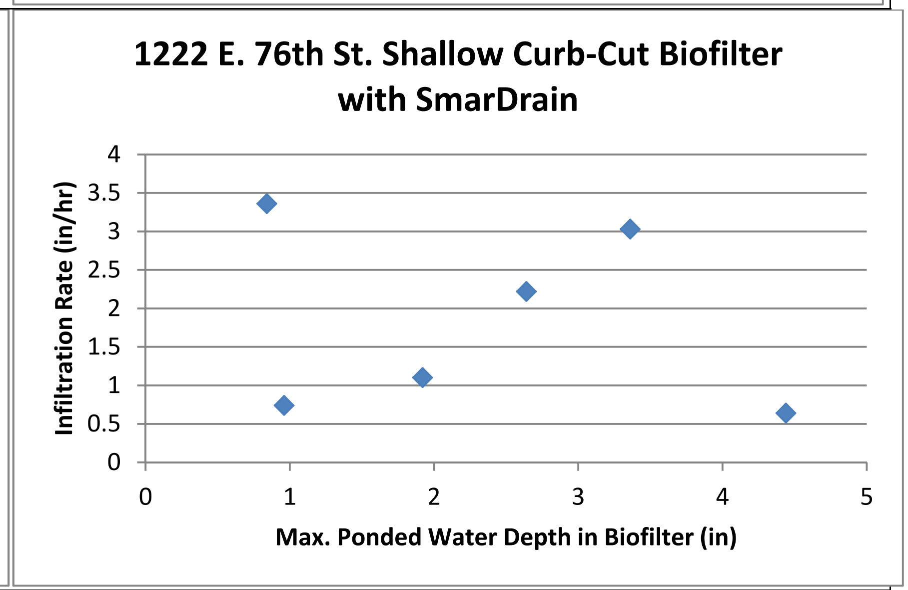

1140 E. 76th Terrace Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
0
1
2
3
4
5
6
Max. Ponded Water Depth in Biofilter (in)

1222 E 76th St. Shallow curb-cut biofilter with SmartDrain													
biofilter top area: 282 ft2 (25.9 m2)													
drainage area: 0.59 ac (0.24 ha); biofilter is 1.1% of drainage area													
Rv: 0.2; avg SSC: 163 mg/L													
years of monitoring: 1.6; 4.3 years to reach 10 kg/m2; 11 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
6/11/12 2:05 AM	6/11/12 6:50 AM	4:55	0.8	6.7	0.00	0.00	3.4	0.84	3:40	7.96	93.9	3.6	0.6
7/25/12 6:00 PM	7/26/12 5:00 AM	11:00	0.49	82	0.01	0.01				9.48	111.9	4.3	0.7
8/31/12 11:35 AM	8/31/12 11:00 PM	11:25	2.61	1492	0.09	0.04				12.56	148.2	5.7	0.9
9/13/12 2:30 PM	9/13/12 8:30 PM	6:00	0.43	762	0.05	0.11				13.27	156.6	6.1	1.0
9/26/12 2:00 AM	9/26/12 9:15 AM	7:15	0.23	527	0.03	0.14				13.51	159.4	6.2	1.0
10/13/12 12:30 AM	10/13/12 10:00 PM	21:30	0.86	547	0.03	0.04				14.42	170.1	6.6	1.1
7/3/13 5:54 PM	7/3/13 9:36 PM	3:42	1.5	N/A	N/A	N/A	1.1	1.92	2:23	37.54	442.9	17.1	2.8
7/29/13 6:27 AM	7/30/13 9:41 AM	3:14	0.91	N/A	N/A	N/A	2.2	2.64	8:08	38.45	453.7	17.5	2.9
9/28/13 8:29 AM	9/28/13 11:29 AM	3:00	0.28	N/A	N/A	N/A	0.7	0.96	0:51	44.2108	521.6	20.2	3.3
10/3/13 11:04 AM	10/3/13 11:24 AM	0:20	0.16	N/A	N/A	N/A	3.0	3.36	0:20	44.37	523.5	20.2	3.3
10/30/13 12:09 PM	10/31/13 11:46 AM	23:35	3.43	N/A	N/A	N/A	0.6	4.44	12:41	49.85	588.2	22.7	3.7
average						0.06	1.8	2.36					
minimum						0.00	0.6	0.84					
maximum						0.14	3.4	4.44					
st dev						0.06	1.2	1.40					
COV						1.01	0.6	0.60					

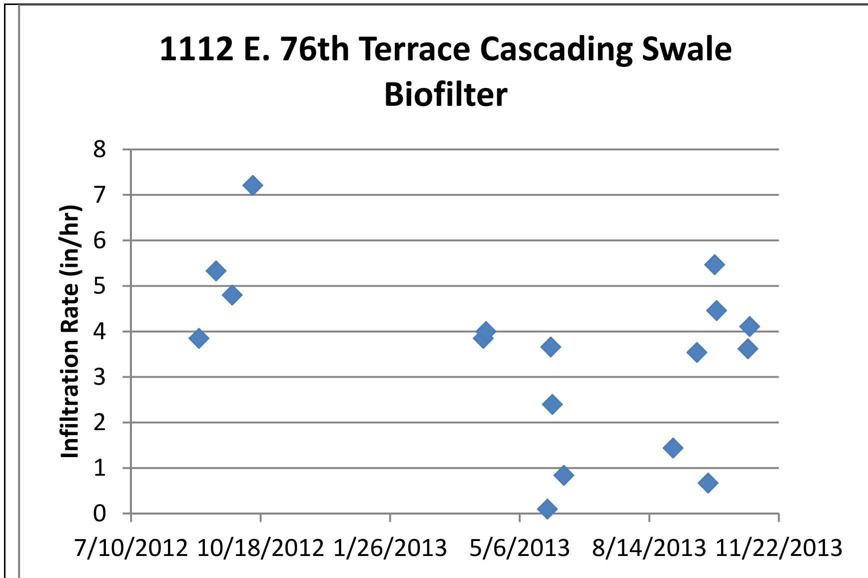

1222 E. 76th St. Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
3.5
4
4/1
7/10
10/18
1/26
5/6
8/14
11/22
3/2

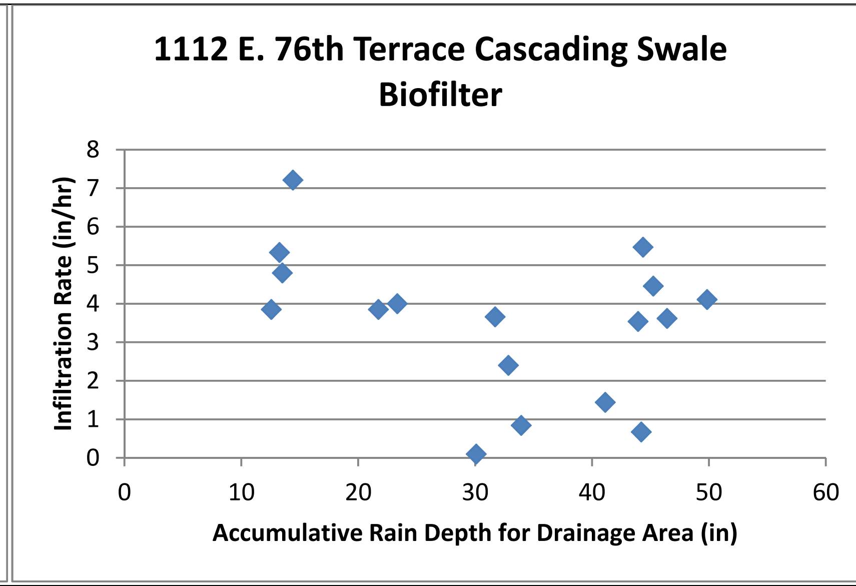

1222 E. 76th St. Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
10
20
30
40
50
60
Accumulative Rain Depth for Drainage Area (in)

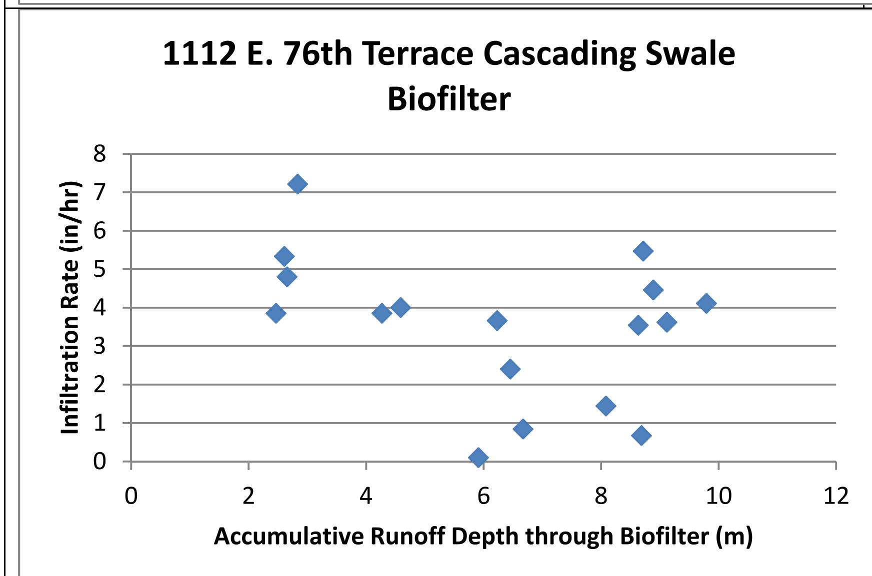

1222 E. 76th St. Shallow Curb-Cut Biofilter with SmartDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
5
10
15
20
25
Accumulative Runoff Depth through Biofilter (m)

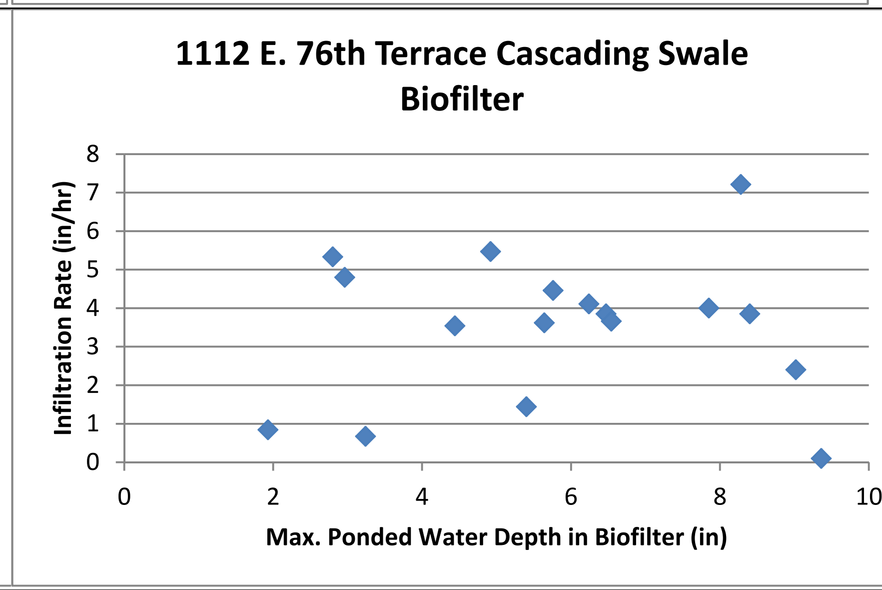

1222 E. 76th St. Shallow Curb-Cut Biofilter with SmarDrain
Infiltration Rate (in/hr)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
1
2
3
4
5
Max. Ponded Water Depth in Biofilter (in)

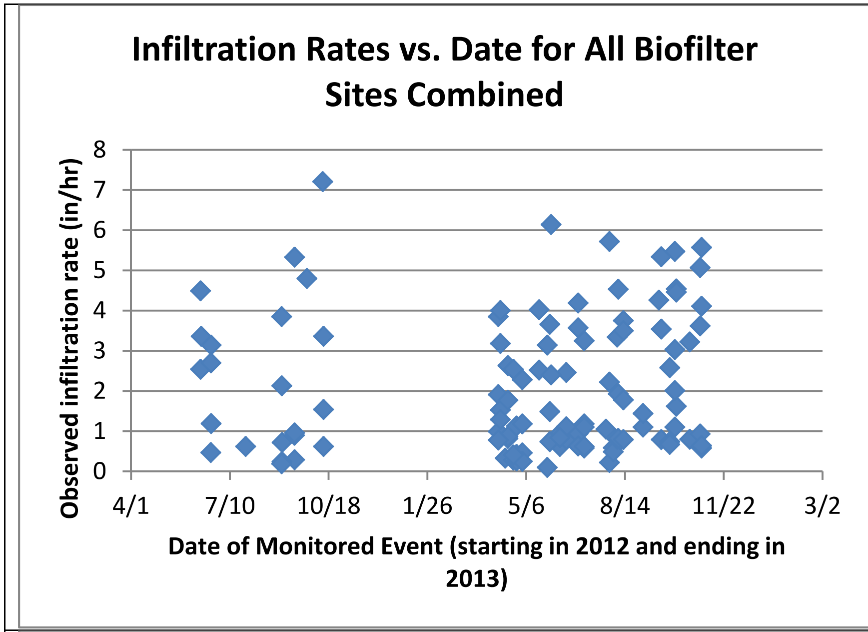
1112 E 76th Terr. Cascading swale biofilters													
biofilter top area: 328 ft2 (30.1 m2)													
drainage area: 0.29 ac (0.12 ha); biofilter is 2.6% of drainage area													
Rv: 0.2; avg SSC: 166 mg/L													
years of monitoring: 1.6; 10 years to reach 10 kg/m2; 25 years to reach 25 kg/m2													
Start Time	End Time	Event Duration (hr:min)	Rainfall Depth (in.)	Total volume of inflow (gal)	Total runoff depth (in)	Rv: Runoff/ rainfall	f (in/hr)	Max Water Depth in Biofilter (in)	Time Duration before Ponding Occurred (hr:min)	Accum. rain depth since April 1, 2012 (in)	Accum. runoff to biofilter (m3)	Accum. runoff (m thru biofilter)	Accum. sediment to biofilter (kg/m2)
8/31/12 11:08 AM	9/1/12 3:08 PM	4:00	5.6	8533	1.07	0.19	3.9	6.47	4:15	12.56	74.2	2.5	0.4
9/13/12 2:08 PM	9/14/12 10:23 AM	20:15	0.43	2098	0.26	0.61	5.3	2.80	1:45	13.27	78.4	2.6	0.4
9/26/12 2:08 AM	9/26/12 4:53 AM	2:45	0.23	261	0.03	0.14	4.8	2.96	2:15	13.51	79.9	2.7	0.4
10/12/12 12:03 AM	10/14/12 7:48 AM	7:45	0.86	1197	0.15	0.17	7.2	8.28	1:15	14.42	85.2	2.8	0.5
4/7/13 7:52 PM	4/8/13 2:25 AM	6:33	1.1	11502	1.45	1.31	3.9	8.40	2:01	21.73	128.4	4.3	0.7
4/9/13 9:56 PM	4/10/13 6:30 PM	20:34	1.62	10047	1.26	0.78	4.0	7.85	2:01	23.35	138.0	4.6	0.8
5/27/13 8:31 AM	5/27/13 12:35 PM	4:04	2.01	26303	3.31	1.64	0.1	9.36	2:05	30.09	177.8	5.9	1.0
5/29/13 10:53 PM	5/30/13 3:17 PM	16:24	1.62	15060	1.89	1.17	3.7	6.54	4:28	31.71	187.4	6.2	1.0
5/31/13 5:26 AM	5/31/13 9:21 AM	3:55	1.34	N/A	N/A	N/A	2.4	9.02	0:55	32.85	194.2	6.5	1.1
6/9/13 12:54 AM	6/9/13 3:53 AM	2:59	0.39	686	0.09	0.22	0.8	1.93	5:41	33.95	200.7	6.7	1.1
9/1/13 7:42 AM	9/1/13 9:02 AM	1:20	0.16	N/A	N/A	N/A	1.4	5.40	0:15	41.14	243.2	8.1	1.3
9/19/13 7:00 PM	9/19/13 10:23 PM	3:23	1.89	N/A	N/A	N/A	3.5	4.44	2:43	43.94	259.7	8.6	1.4
9/28/13 8:29 AM	9/28/13 11:29 AM	3:00	0.28	N/A	N/A	N/A	0.7	3.24	0:29	44.2108	261.3	8.7	1.4
10/3/13 11:04 AM	10/3/13 11:24 AM	0:20	0.16	N/A	N/A	N/A	5.5	4.92	0:39	44.37	262.2	8.7	1.4
10/4/13 10:36 PM	10/5/13 3:48 AM	5:12	0.87	N/A	N/A	N/A	4.5	5.76	2:07	45.24	267.4	8.9	1.5
10/29/13 2:52 AM	10/29/13 9:41 AM	6:49	0.83	N/A	N/A	N/A	3.6	5.64	4:07	46.4172	274.3	9.1	1.5
10/30/13 12:09 PM	10/31/13 11:46 AM	23:35	3.43	N/A	N/A	N/A	4.1	6.24	11:05	49.845	294.6	9.8	1.6
average						0.69	3.5	5.84					
minimum						0.14	0.1	1.93					
maximum						1.64	7.2	9.36					
st dev						0.57	1.9	2.27					
COV						0.82	0.5	0.39					

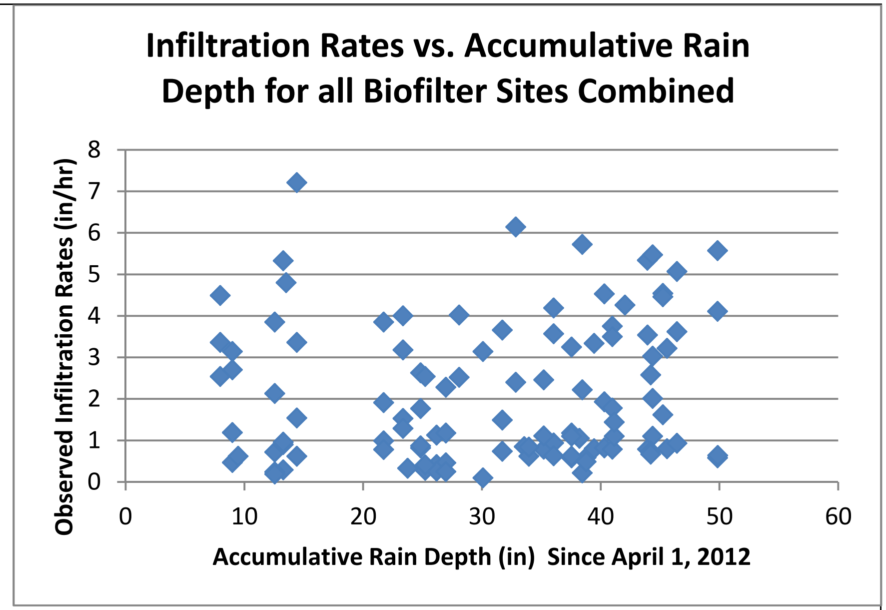

1112 E. 76th Terrace Cascading Swale Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
8
7/10/2012
10/18/2012
1/26/2013
5/6/2013
8/14/2013
11/22/2013

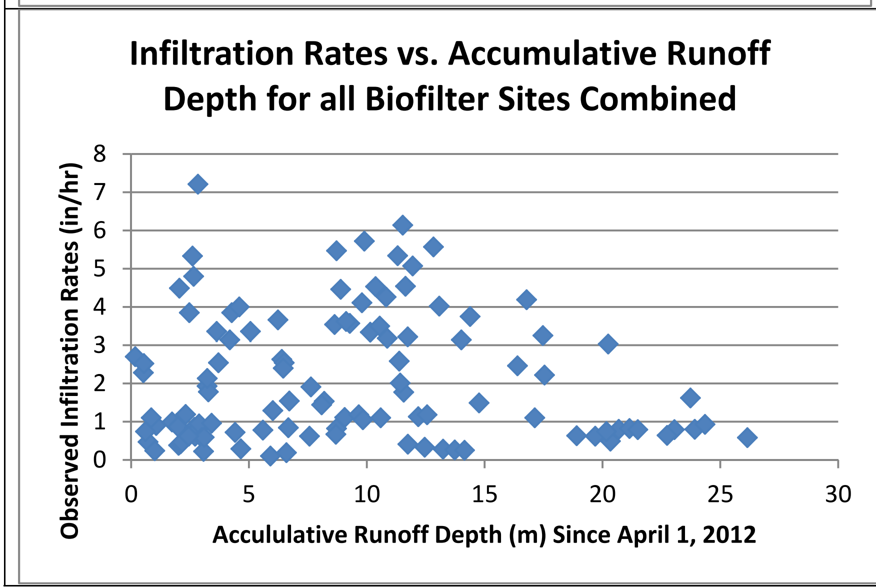

1112 E. 76th Terrace Cascading Swale Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
8
0
10
20
30
40
50
60
Accumulative Rain Depth for Drainage Area (in)

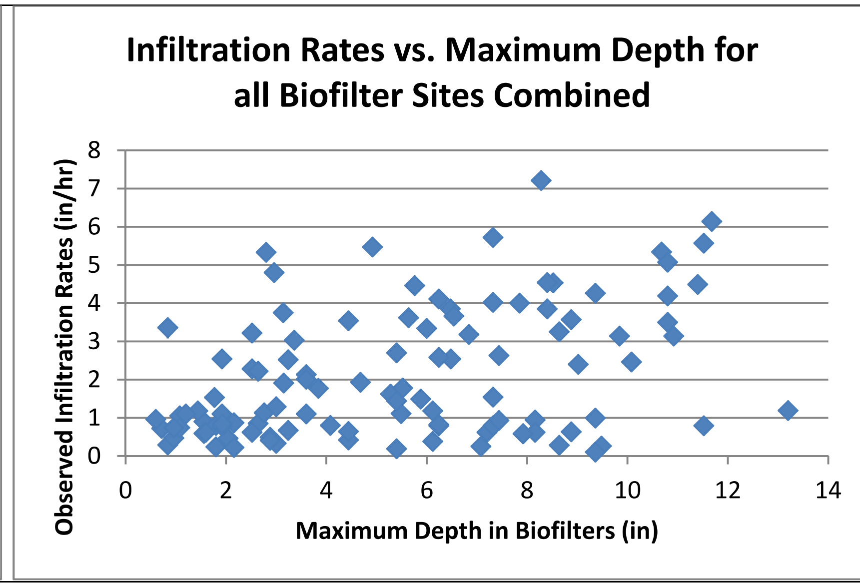

1112 E. 76th Terrace Cascading Swale Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
8
0
2
4
6
8
10
12
Accumulative Runoff Depth through Biofilter (m)

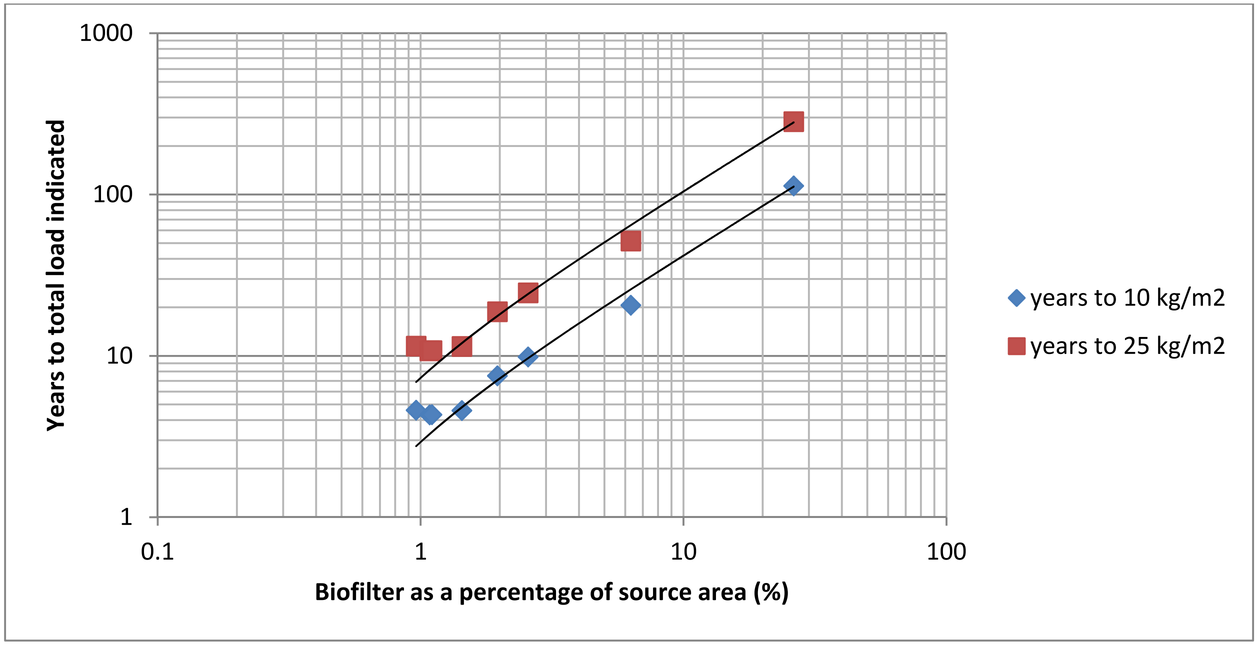

1112 E. 76th Terrace Cascading Swale Biofilter
Infiltration Rate (in/hr)
0
1
2
3
4
5
6
7
8
0
2
4
6
8
10
Max. Ponded Water Depth in Biofilter (in)


Infiltration Rates vs. Date for All Biofilter Sites Combined
Observed infiltration rate (in/hr)
0
1
2
3
4
5
6
7
8
4/1
7/10
10/18
1/26
5/6
8/14
11/22
3/2
Date of Monitored Event (starting in 2012 and ending in 2013)

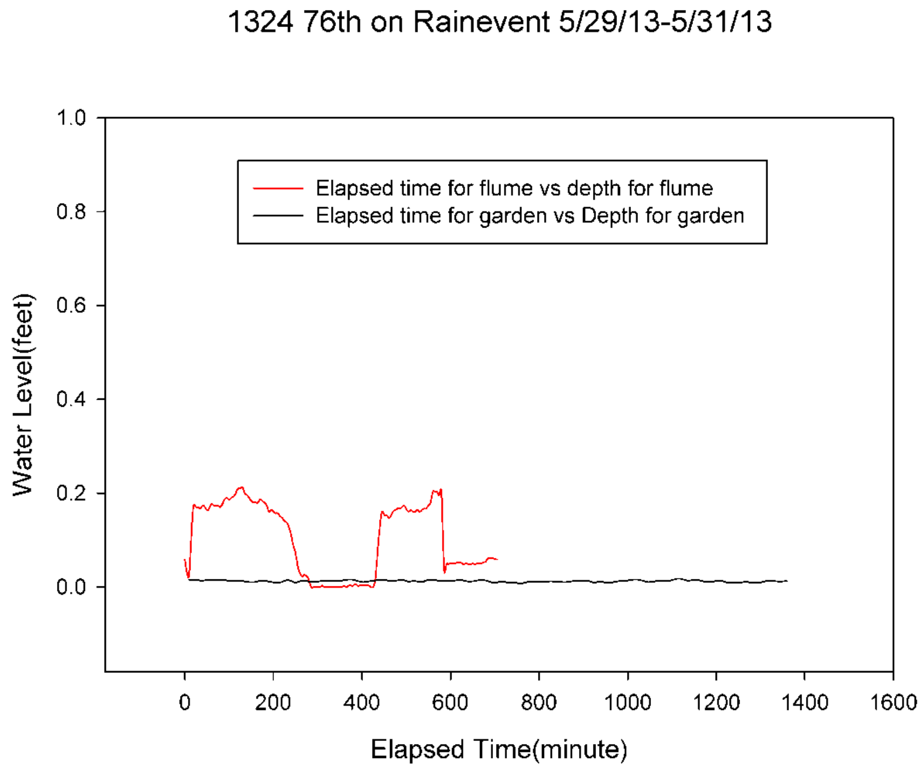

Infiltration Rates vs. Accumulative Rain Depth for all Biofilter Sites Combined
Observed Infiltration Rates (in/hr)
0
1
2
3
4
5
6
7
8
0
10
20
30
40
50
60
Accumulative Rain Depth (in) Since April 1, 2012

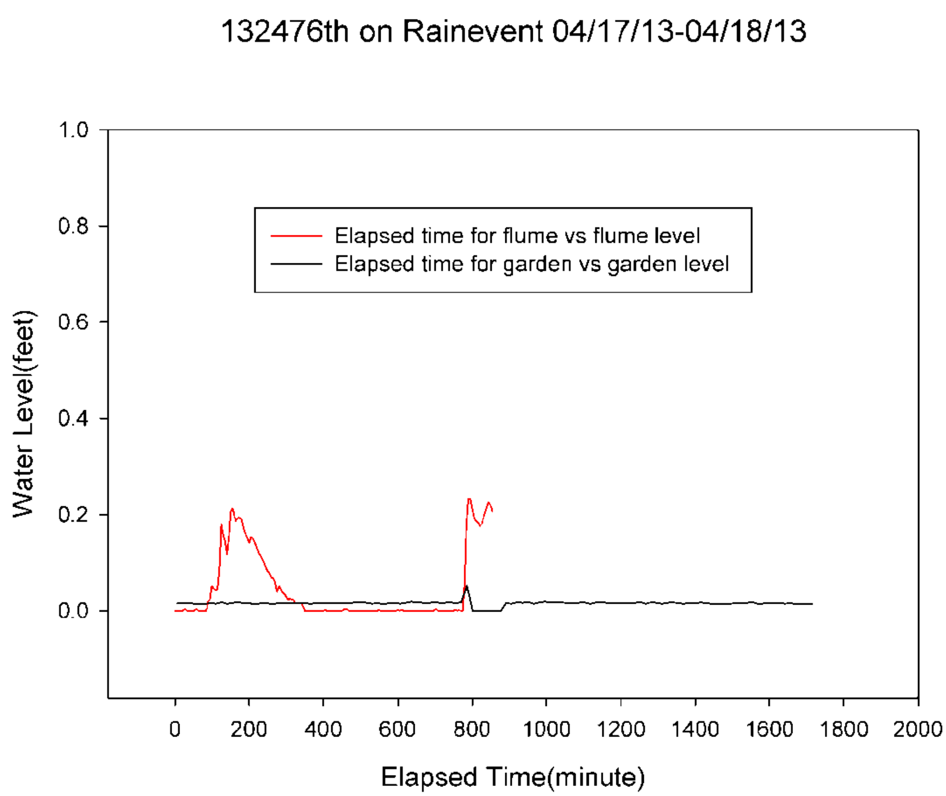

Infiltration Rates vs. Accumulative Runoff Depth for all Biofilter Sites Combined
Observed Infiltration Rates (in/hr)
0
1
2
3
4
5
6
7
8
0
5
10
15
20
25
30
Acculultative Runoff Depth (m) Since April 1, 2012


Infiltration Rates vs. Maximum Depth for all Biofilter Sites Combined
Observed Infiltration Rates (in/hr)
0
1
2
3
4
5
6
7
8
0
2
4
6
8
10
12
14
Maximum Depth in Biofilters (in)

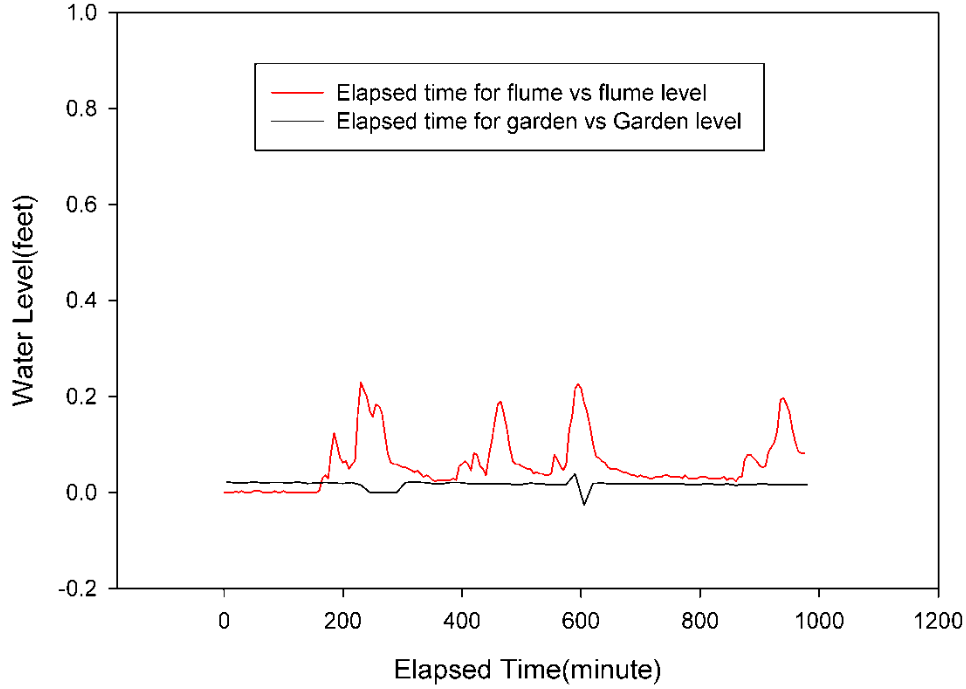

1000
100
10
1
0.1
1
10
100
Years to total load indicated
Biofilter as a percentage of source area (%)
years to 10 kg/m2
years to 25 kg/m2

1- Curb Extension Biofilter - 1324 E 76th St.

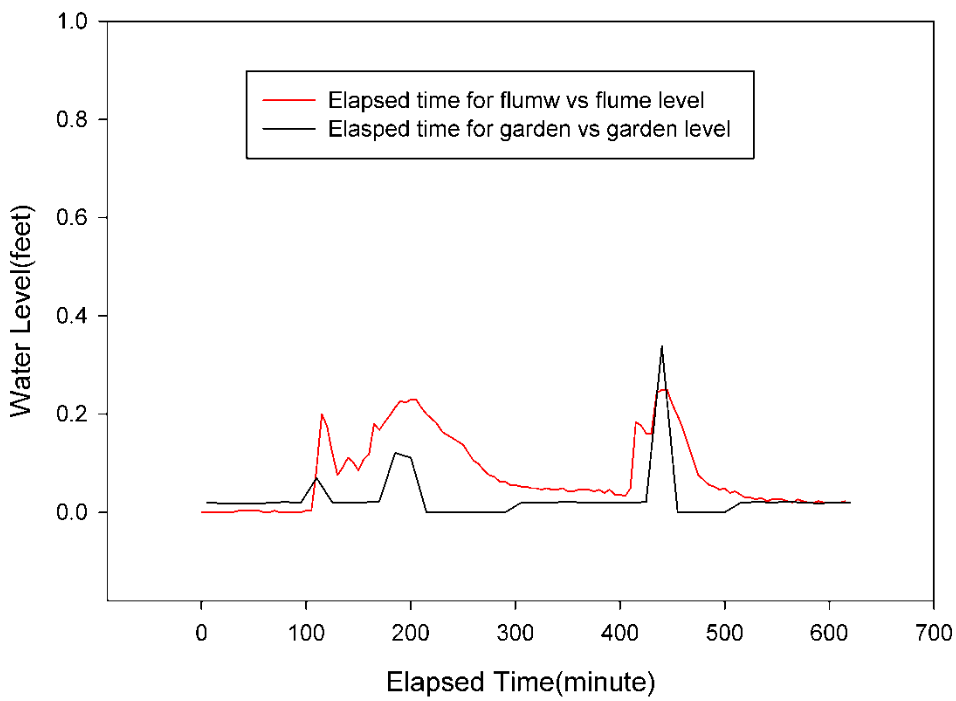




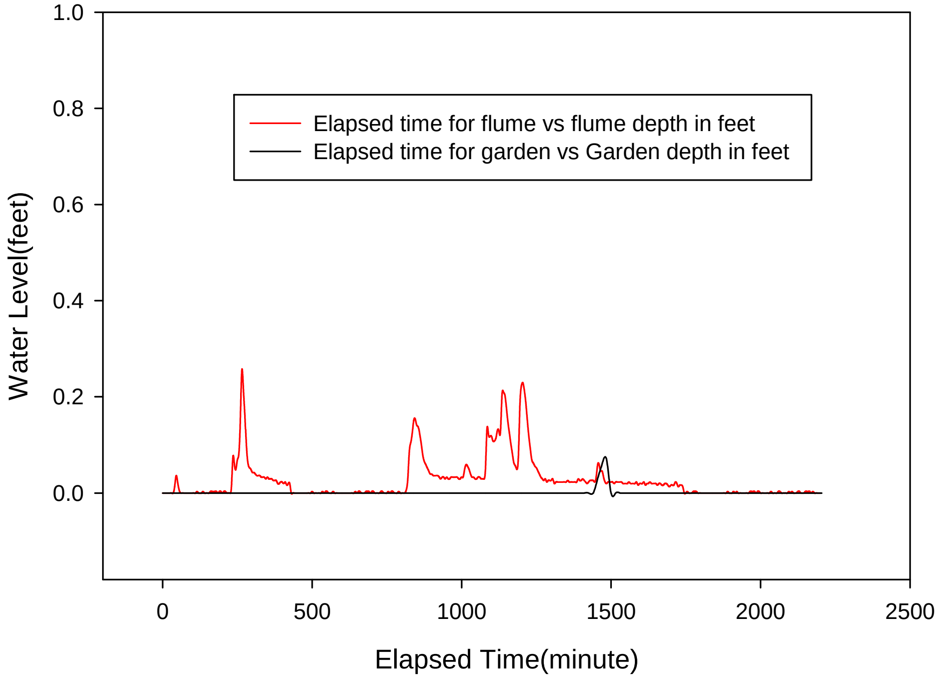
1324 76th on Rainevent 04/09/13--04/10/13



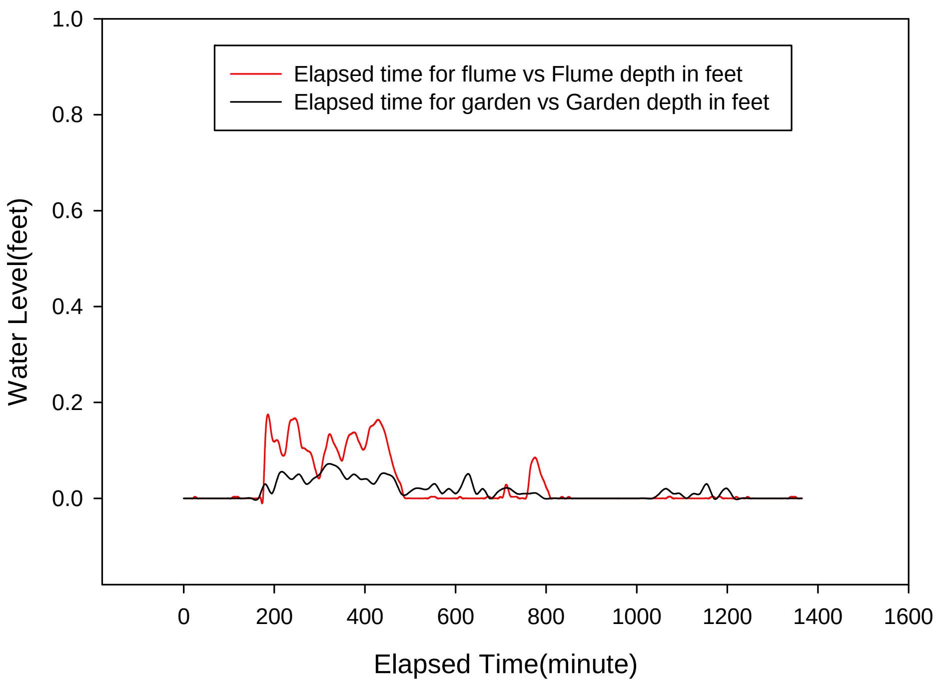
1324 76th on Rainevent 04/07/13



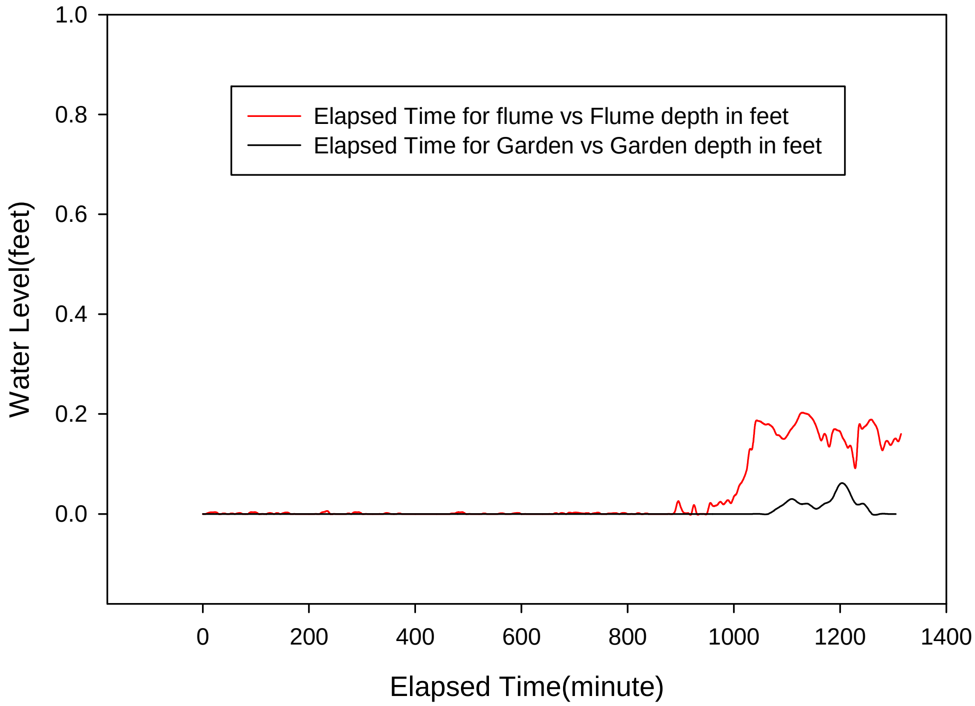
1324 76th Raingarden on Rainevent 10/13/2012



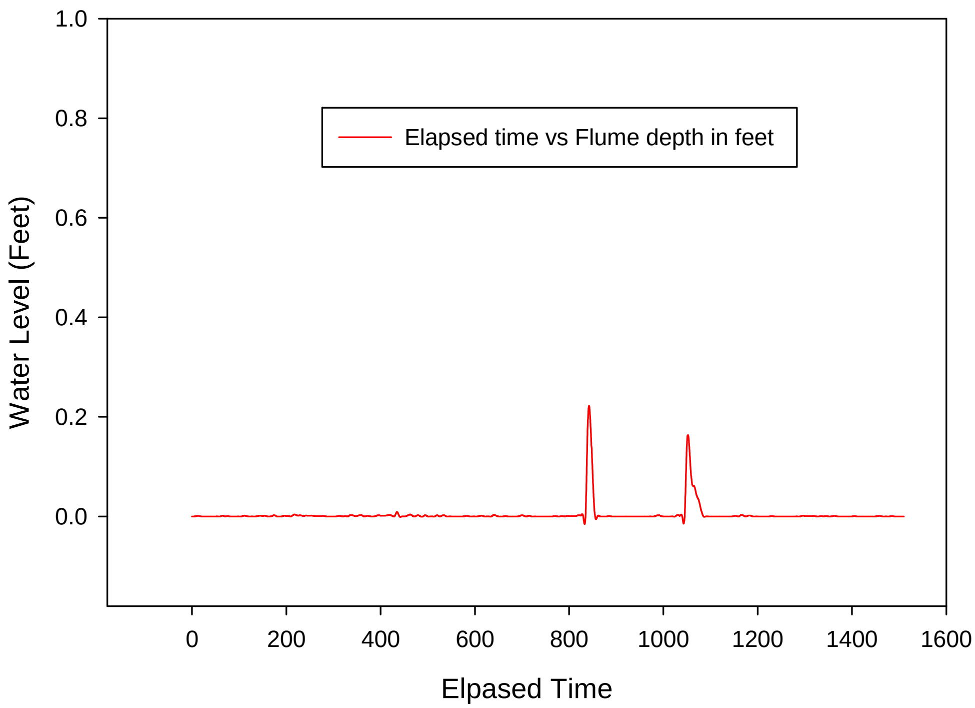
1324 76th Raingarden on Rainevent 9/13/2012



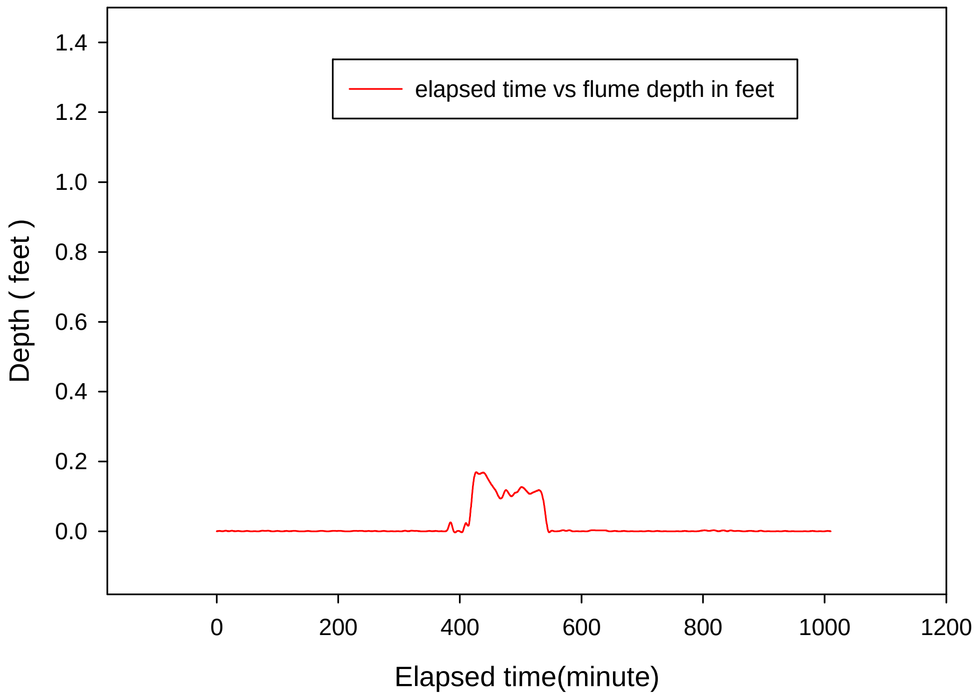
1324 76th Raingarden on Rainevent 8/31/2012



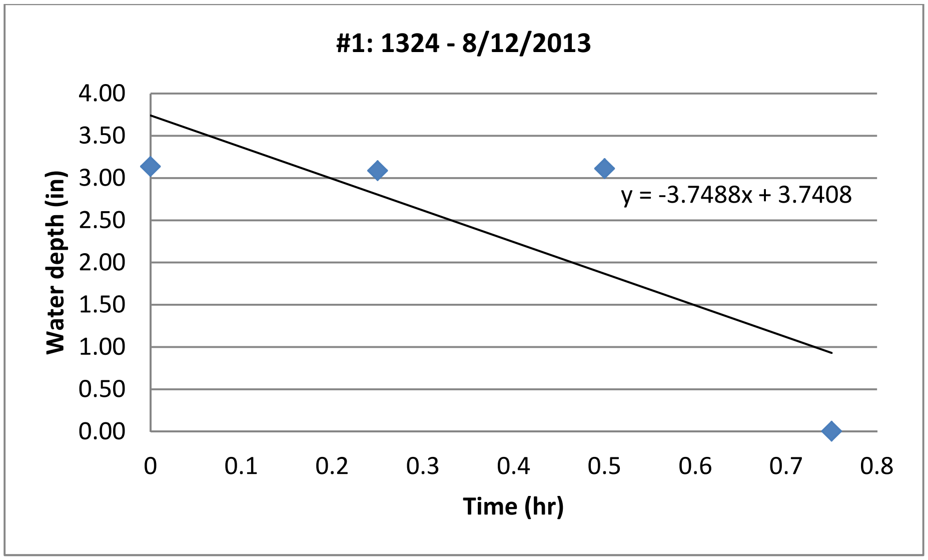
1324 76th Raingarden on Rainevent 7/26/2012

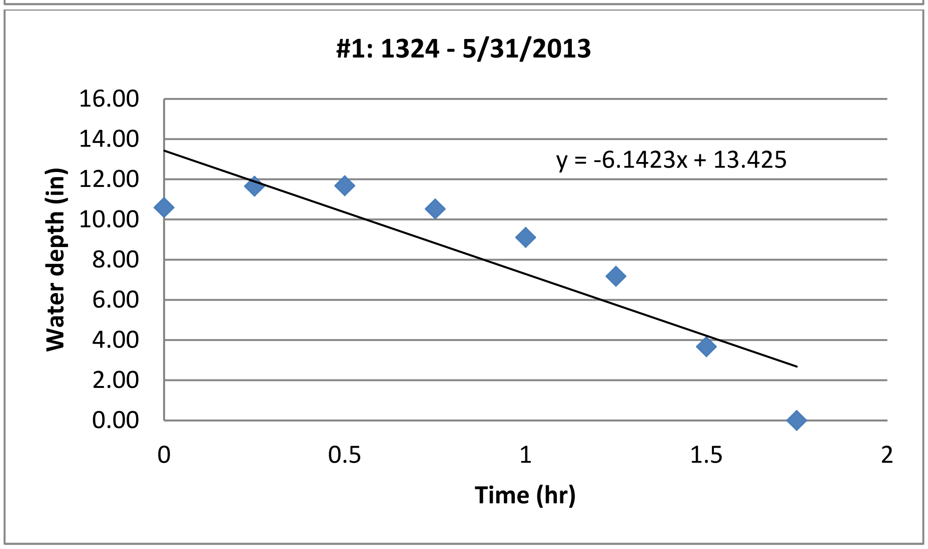


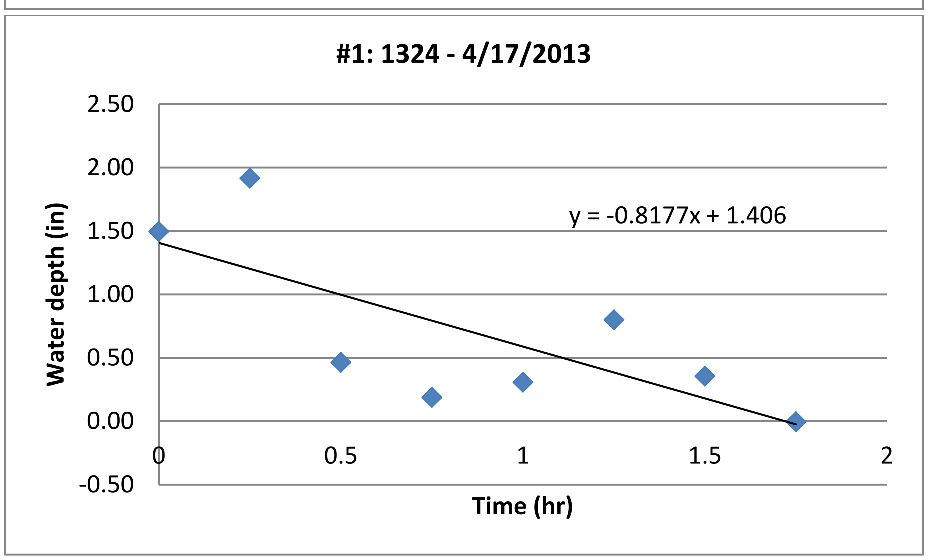
1324 76th Raingarden on Rainevent 6/21/2012

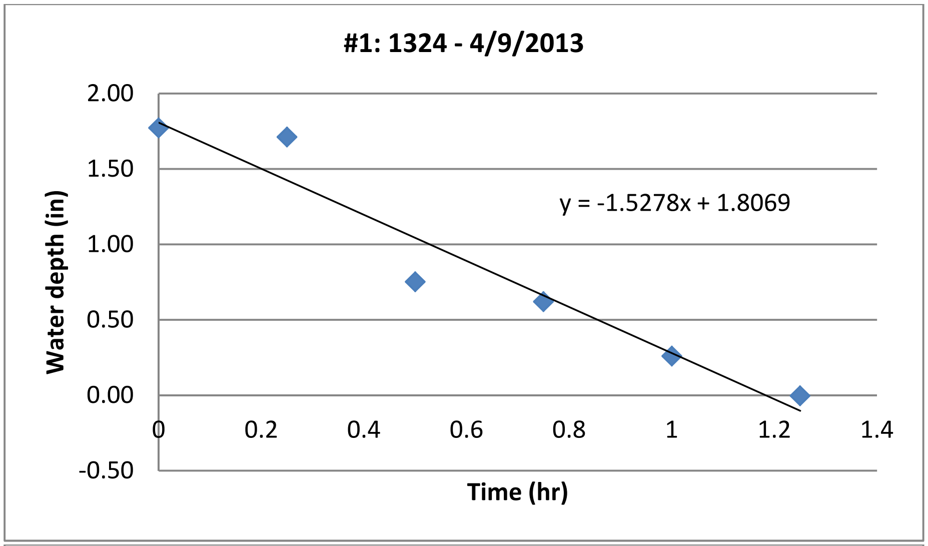


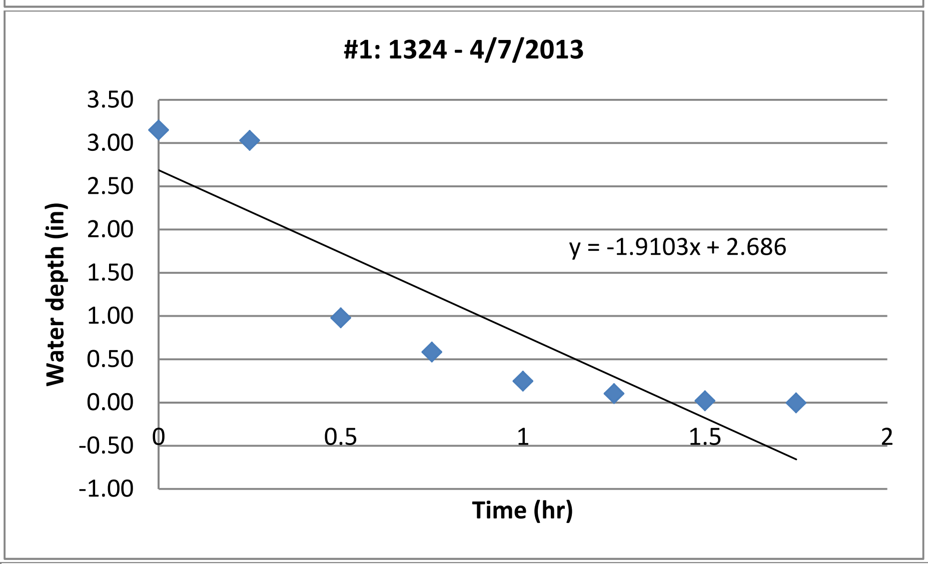
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
0.67	8/12/2013 7:07:00	8/12/2013 10:18:00	3:11	3.14	0	3.75
1.34	5/31/2013 5:26:00	5/31/2013 9:21:00	3:55	11.68	0:28	6.14
1.10	4/17/2013 11:00:00	4/18/2013 20:41:00	33:41	1.92	10:51	0.82
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	1.77	1:01	1.53
1.10	4/7/2013 19:52:00	4/8/2013 2:25:00	6:33	3.15	1:13	1.91
0.86	10/12/2012 21:00:00	10/14/2012 09:40:00	36:40	0.84	24:00	>3.36
0.43	9/13/2012 12:00:00	9/14/2012 10:45:00	22:45	0.84	17:30	0.288
					18:45	1.44
					20:00	0.96
2.61	8/31/2012 22:20:00	8/31/2012 20:15:00	21:55	0.72	18:00	>0.72
0.49	7/25/2012 11:15:00	7/26/2012 12:25:00	25:10	0		
1.03	6/20/2012 19:05:00	6/21/2012 11:55:00	16:50	0		

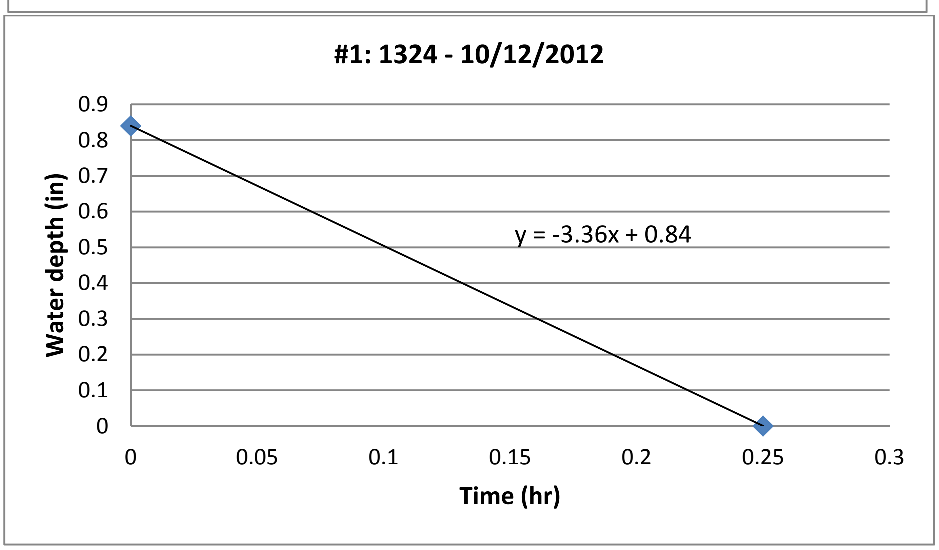

#1: 1324 - 8/12/2013
Water depth (in)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Time (hr)
y = -3.7488x + 3.7408

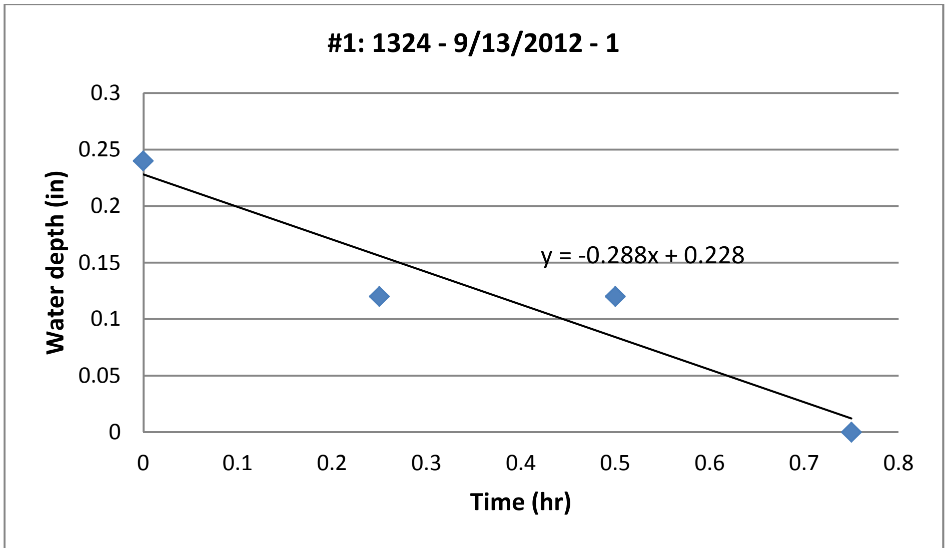

#1: 1324 - 5/31/2013
Water depth (in)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
0.5
1
1.5
2
Time (hr)
y = -6.1423x + 13.425

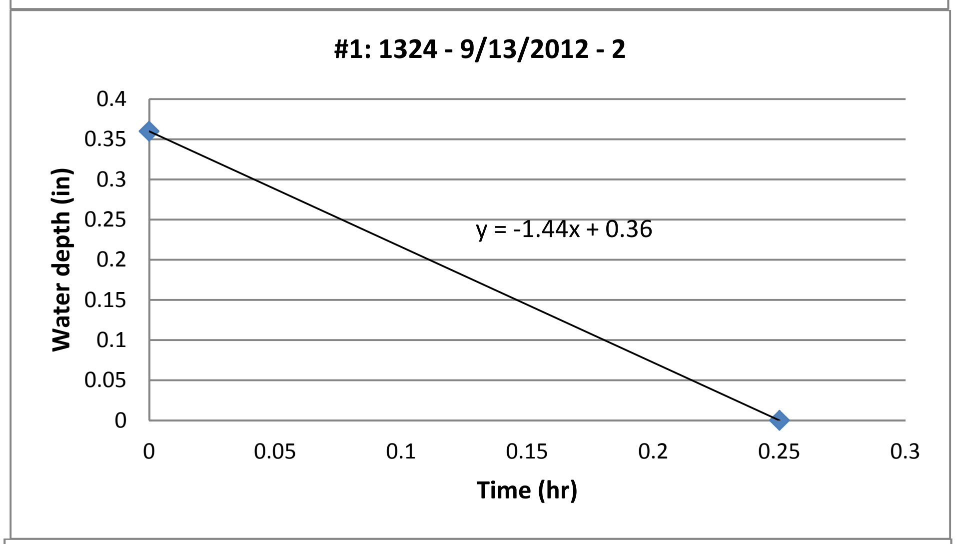

#1: 1324 - 4/17/2013
Water depth (in)
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
0
0.5
1
1.5
2
Time (hr)
y = -0.8177x + 1.406

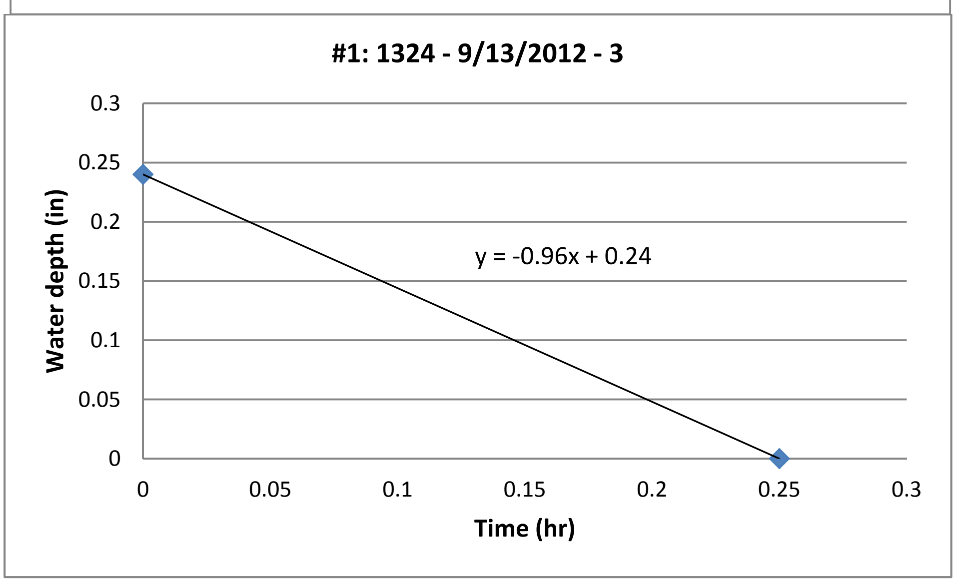

#1: 1324 - 4/9/2013
y = -1.5278x + 1.8069
Water depth (in)
Time (hr)
2.00
1.50
1.00
0.50
0.00
-0.50
0
0.2
0.4
0.6
0.8
1
1.2
1.4

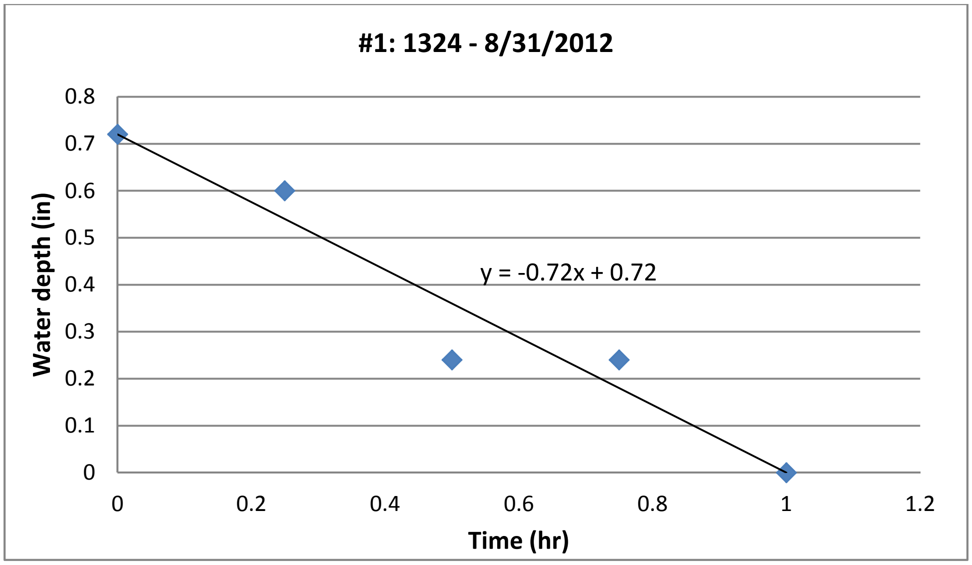

#1: 1324 - 4/7/2013
y = -1.9103x + 2.686
Water depth (in)
Time (hr)
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
0
0.5
1
1.5
2


#1: 1324 - 10/12/2012
y = -3.36x + 0.84
Water depth (in)
Time (hr)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.05
0.1
0.15
0.2
0.25
0.3

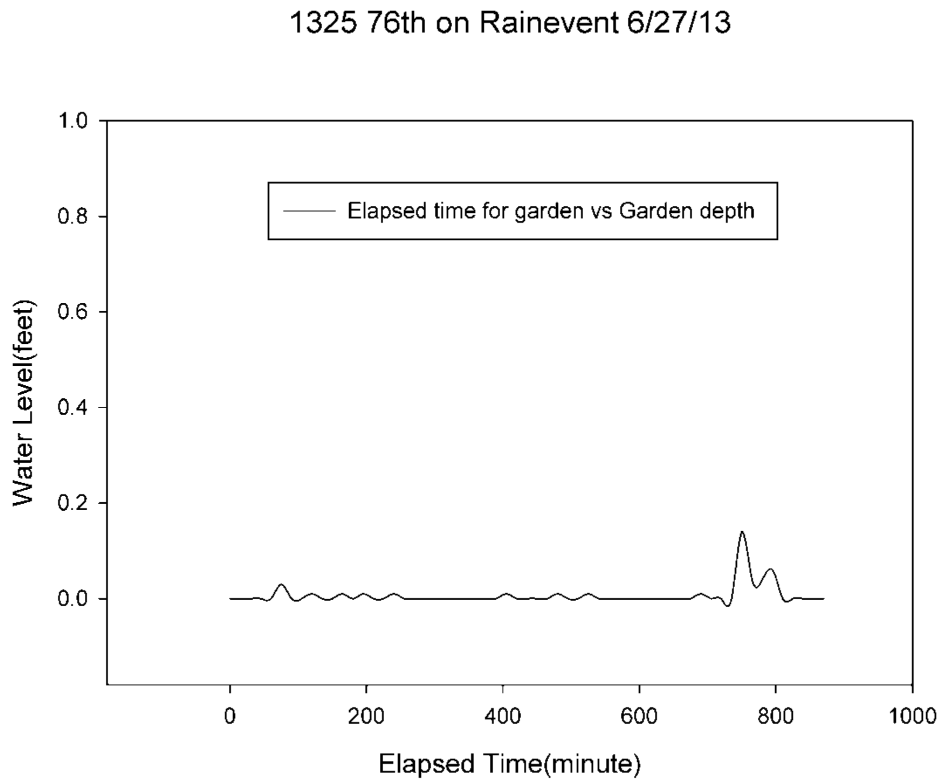

#1: 1324 - 9/13/2012 - 1
Water depth (in)
0.3
0.25
0.2
0.15
0.1
0.05
0
y = -0.288x + 0.228
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Time (hr)

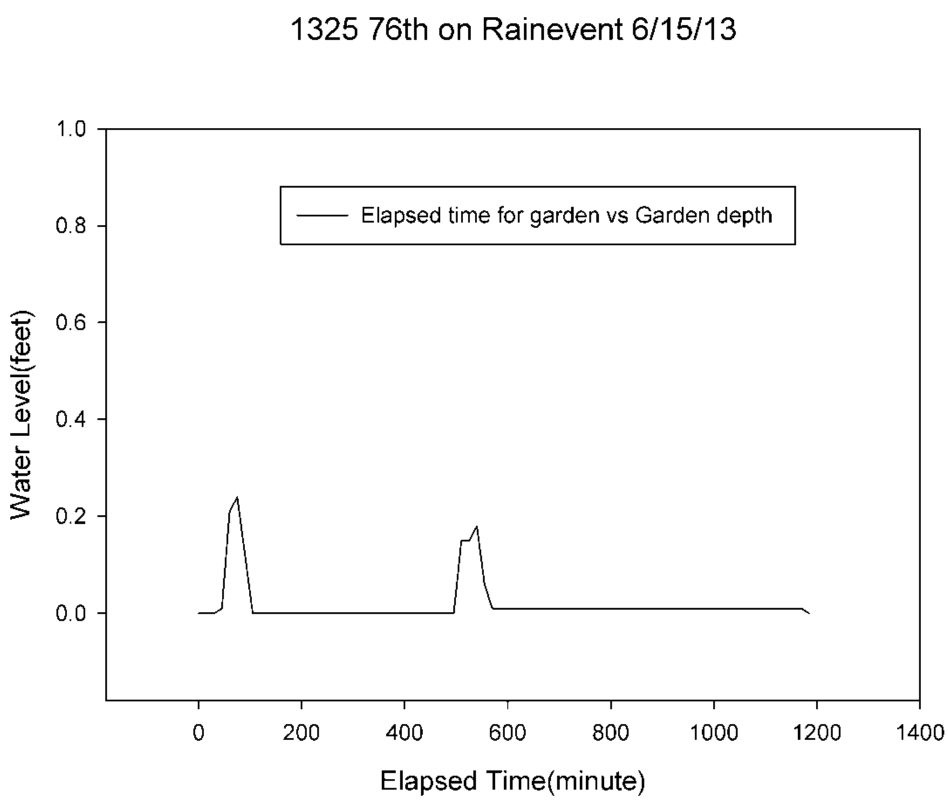

#1: 1324 - 9/13/2012 - 2
Water depth (in)
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
y = -1.44x + 0.36
0
0.05
0.1
0.15
0.2
0.25
0.3
Time (hr)

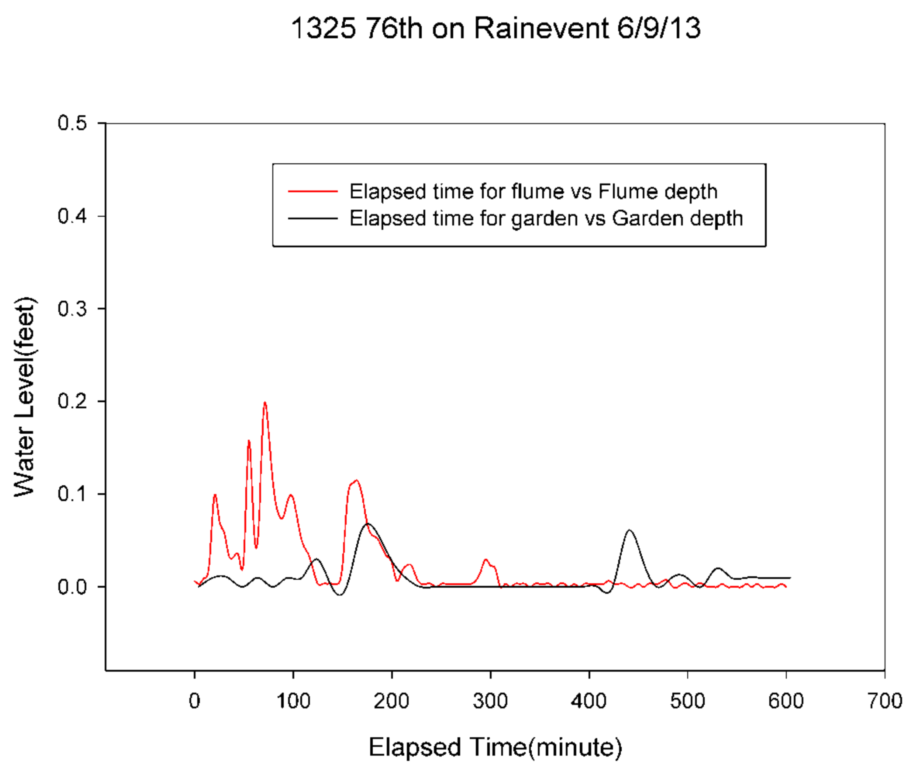

#1: 1324 - 9/13/2012 - 3
Water depth (in)
0.3
0.25
0.2
0.15
0.1
0.05
0
y = -0.96x + 0.24
0
0.05
0.1
0.15
0.2
0.25
0.3
Time (hr)

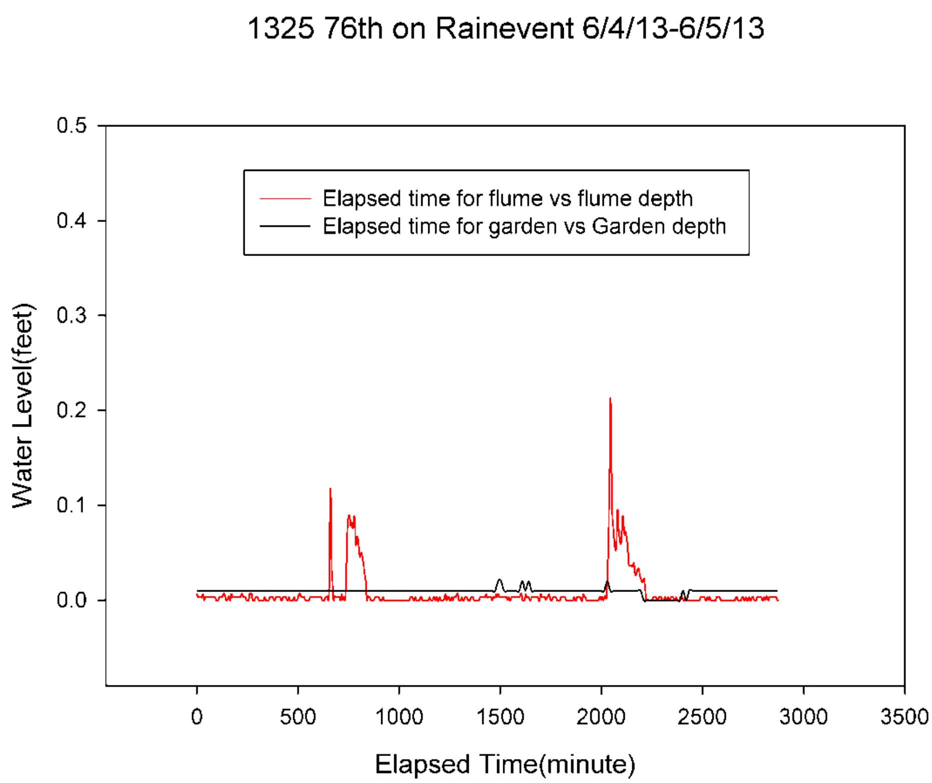

#1: 1324 - 8/31/2012
Water depth (in)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
y = -0.72x + 0.72
0
0.2
0.4
0.6
0.8
1
1.2
Time (hr)

2- Curb Extension Biofilter - 1325 E 76th St.

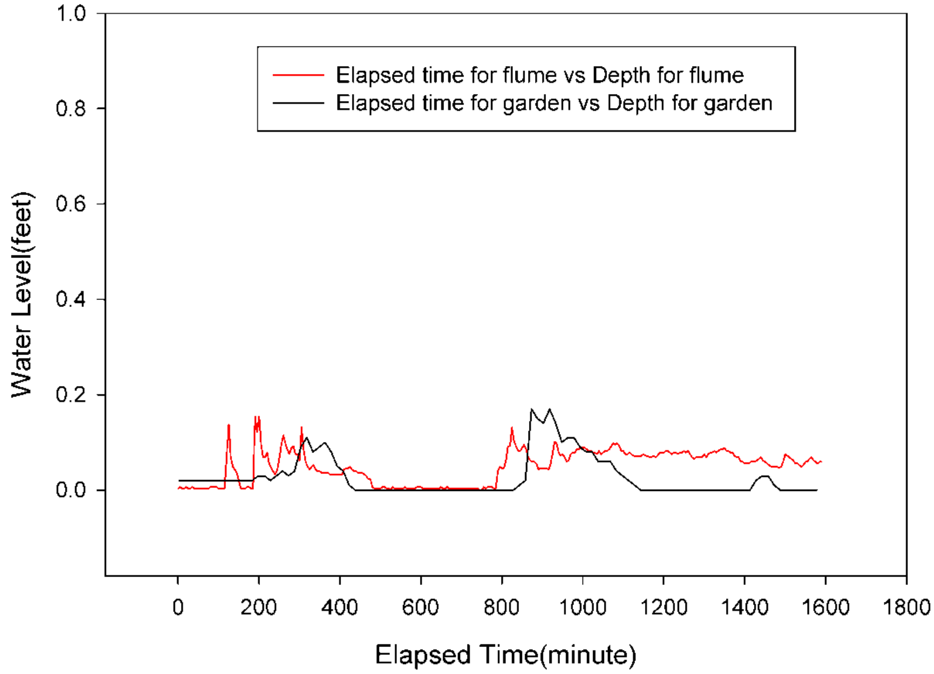




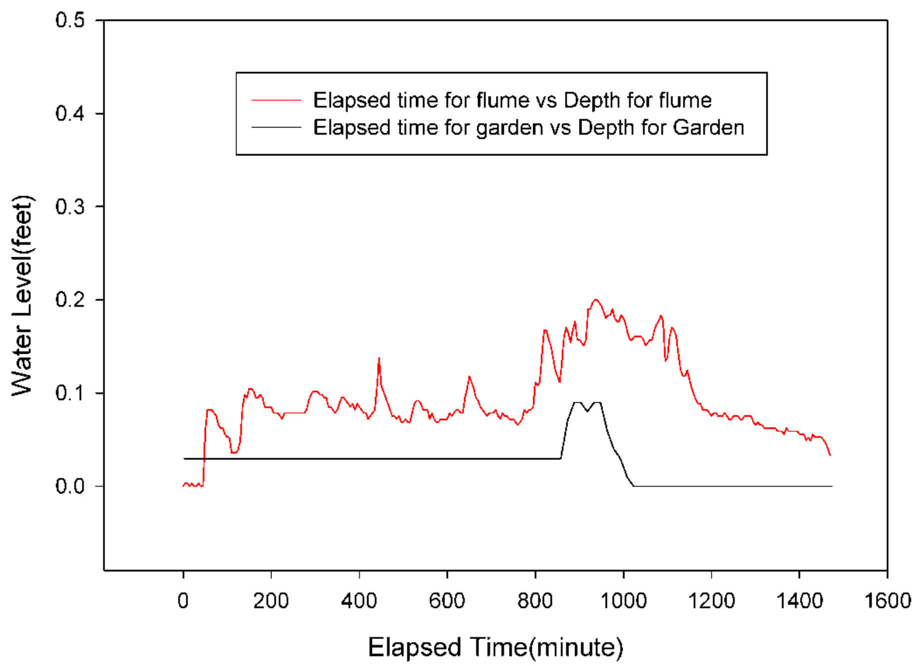

1325 76th on Rainevent 6/9/13
Elapsed time for flume vs Flume depth
Elapsed time for garden vs Garden depth
Water Level(feet)
0.5
0.4
0.3
0.2
0.1
0.0
0
100
200
300
400
500
600
700
Elapsed Time(minute)


1325 76th on Rainevent 6/4/13-6/5/13
Elapsed time for flume vs flume depth
Elapsed time for garden vs Garden depth
Water Level(feet)
0.5
0.4
0.3
0.2
0.1
0.0
0
500
1000
1500
2000
2500
3000
3500
Elapsed Time(minute)

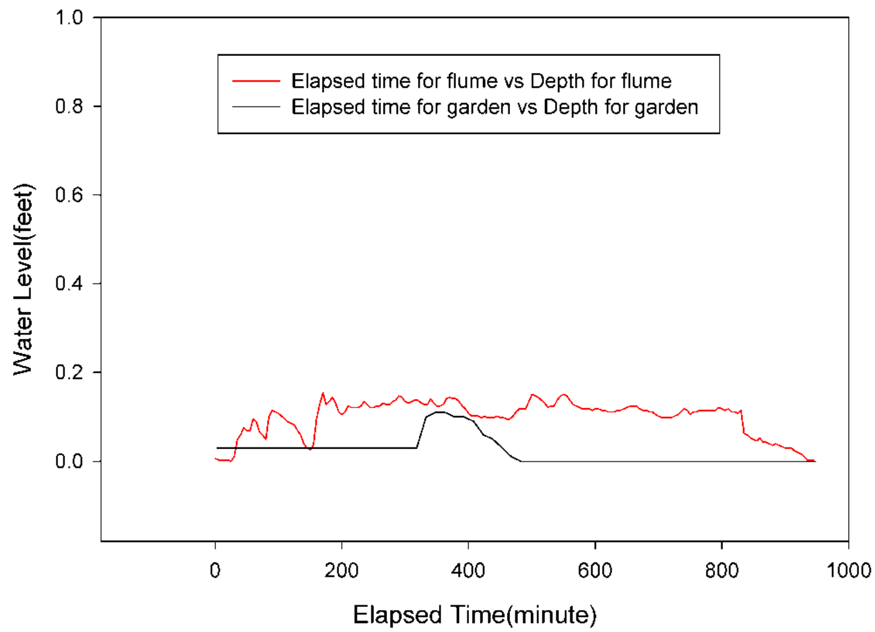
1325 76th on Rainevent 5/2/13-5/3/13



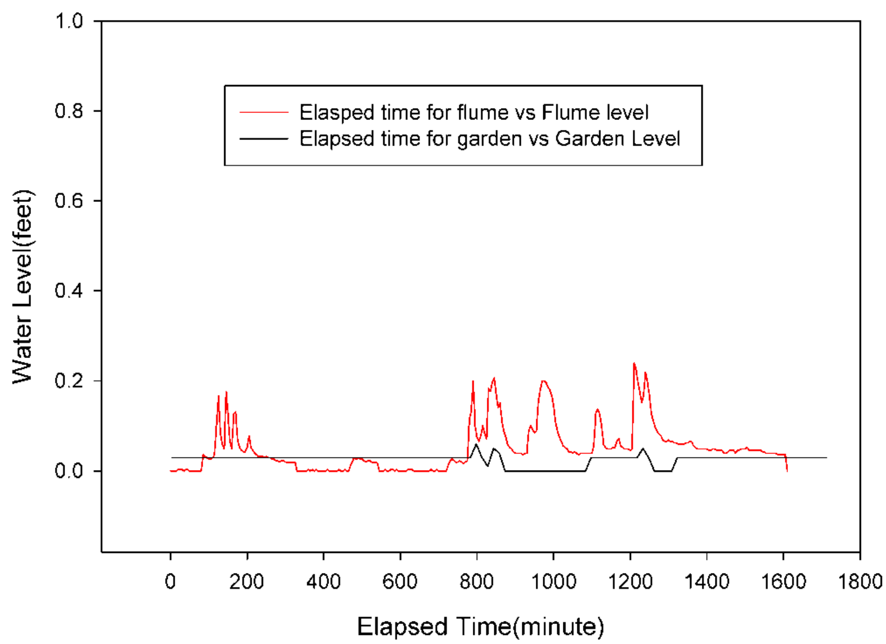
1325 76th on Rainevent 04/26/13-04/27/13



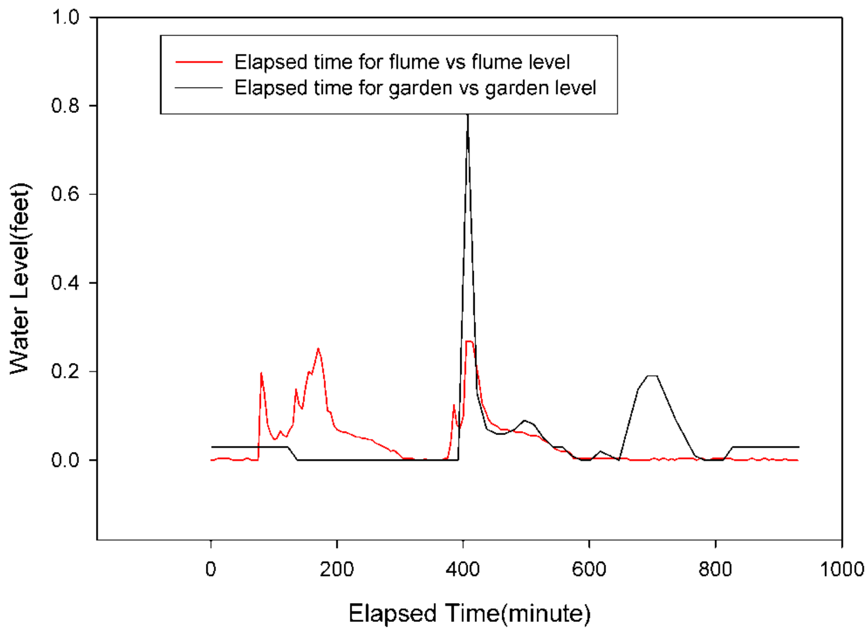
1325 76th on Rainevent 04/23/13



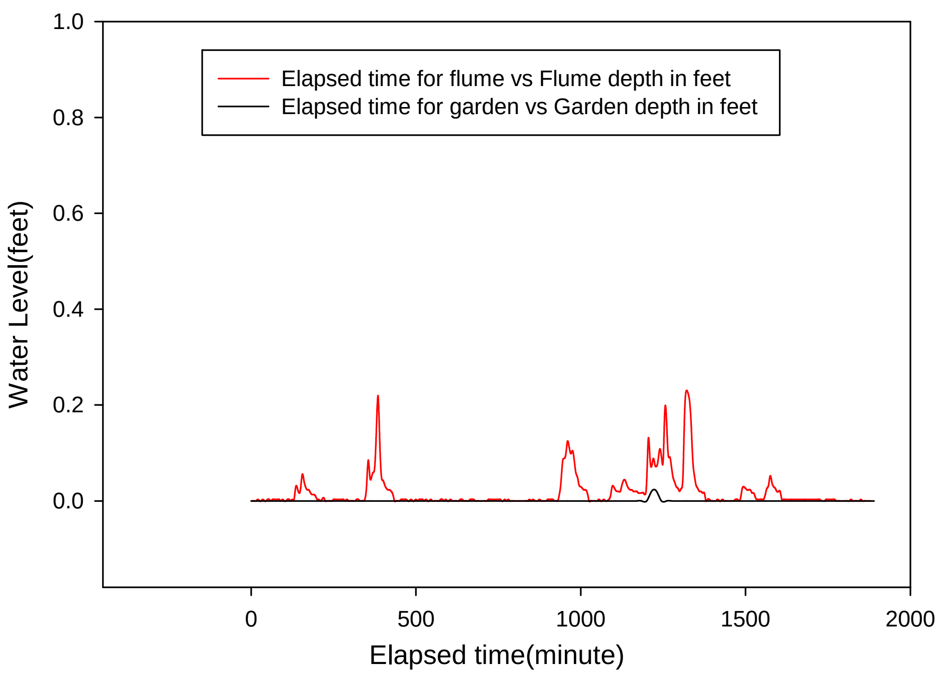
1325 76th on Rainevent 04/17/13-04/18/13



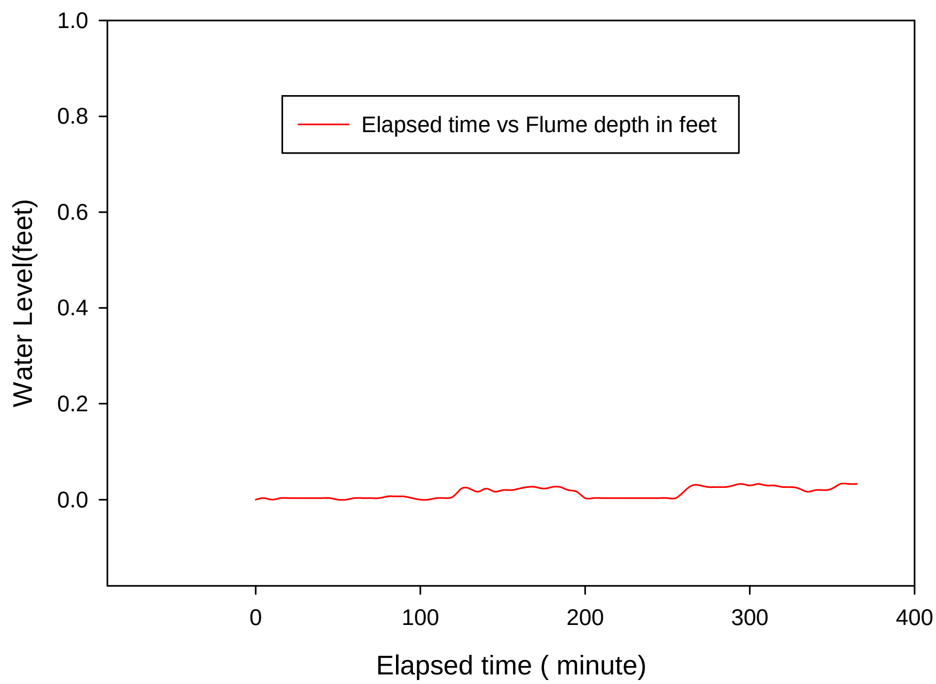
1325 76th on Rainevent 04/07/13



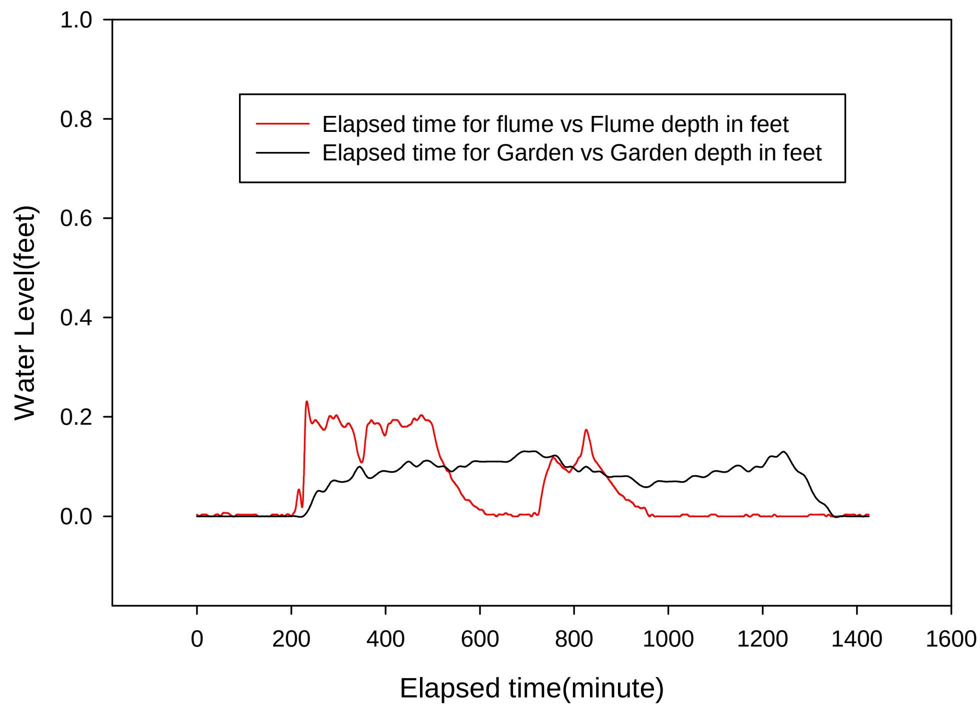
1325 76th Raingarden on Rainevent 10/13/2012



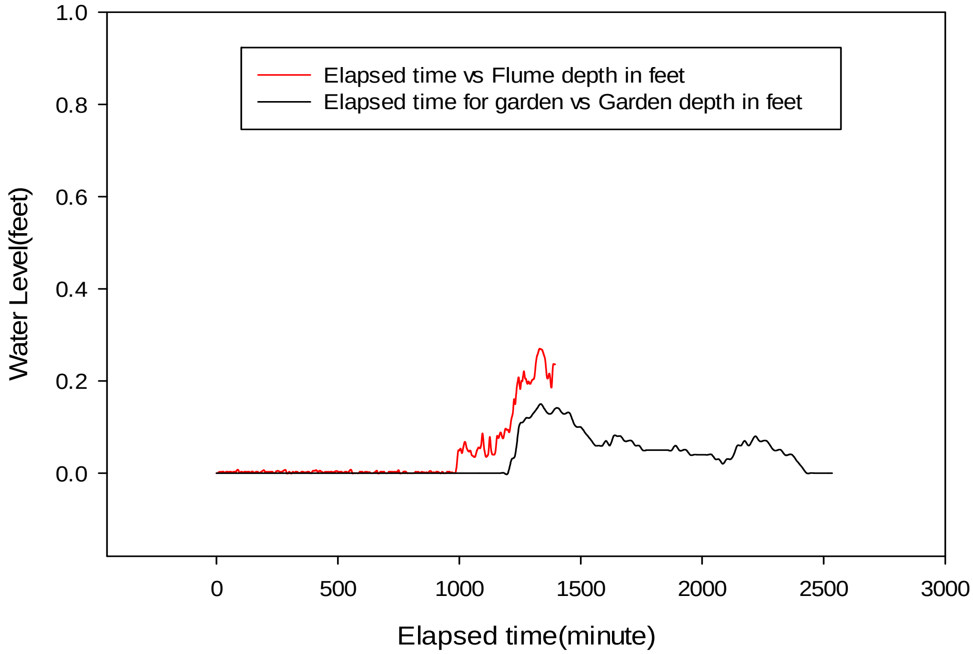
1325 76th Raingarden on Rainevent 9/26/2012



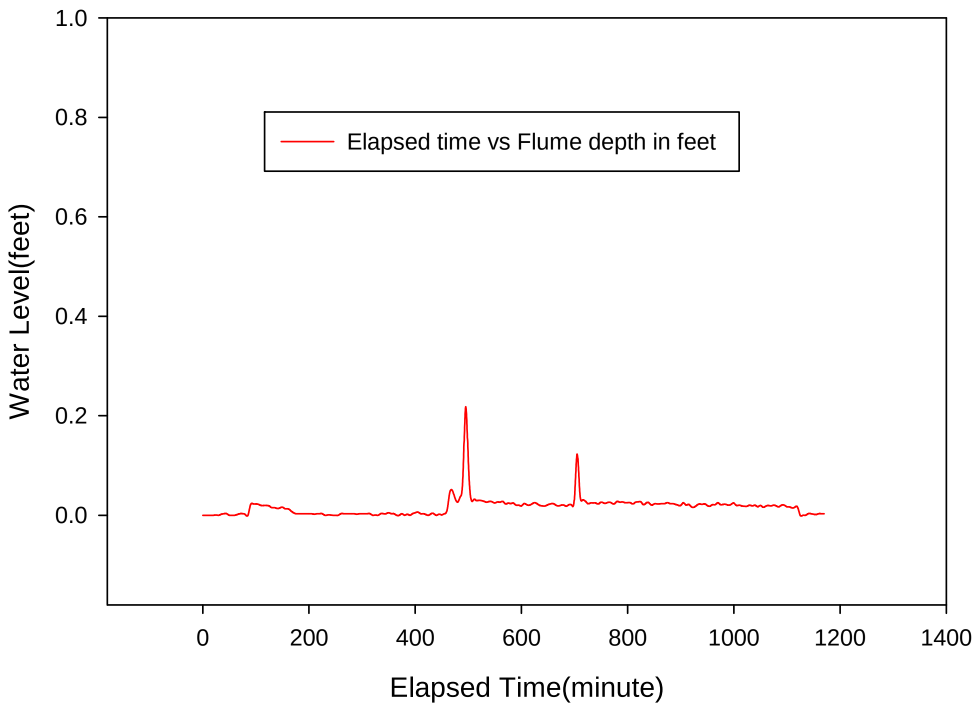
1325 76th Raingarden on Rainevent 9/13/2012



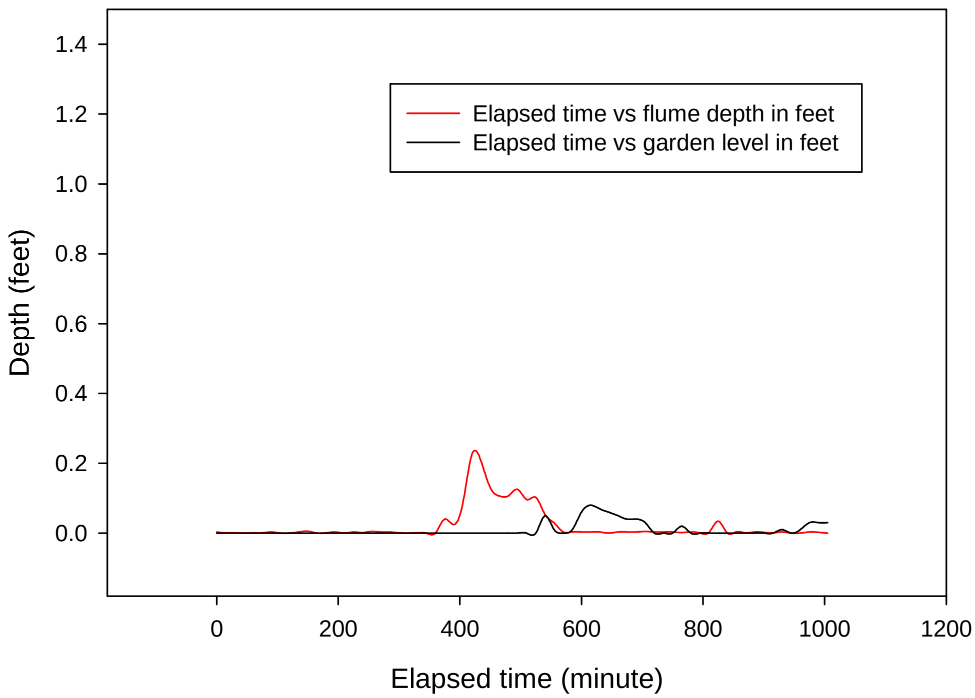
1325 76th Flume on Rainevent 08/31/2012



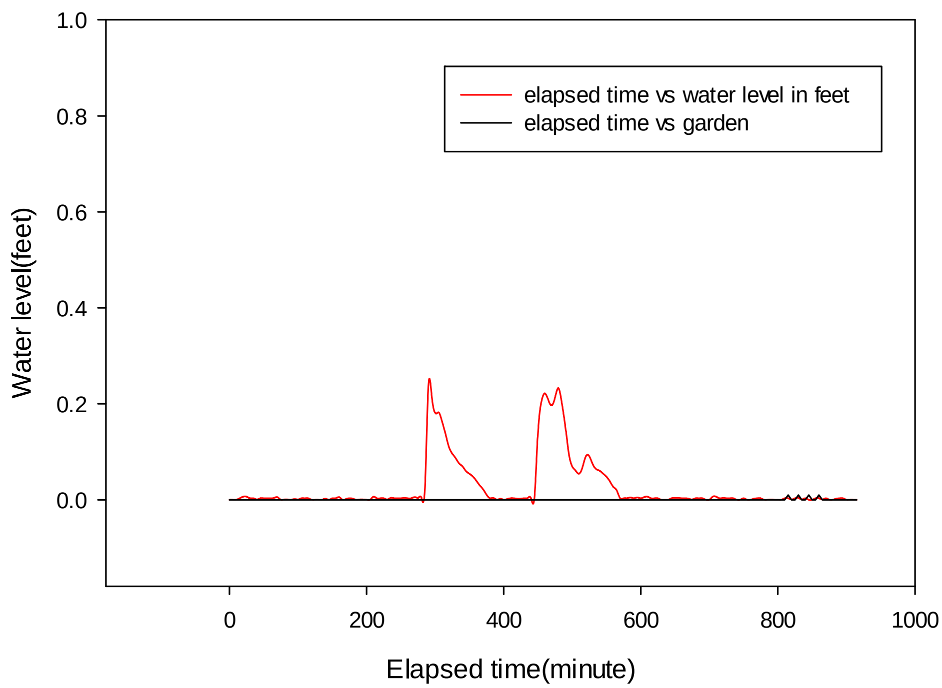
1325 76th Raingarden on Rainevent 7/26/2012



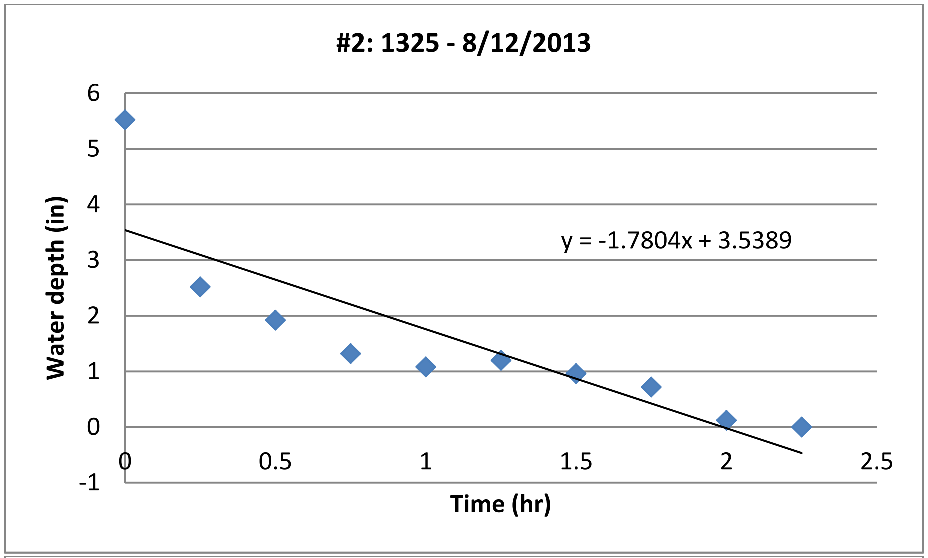
1325 76th Raingarden on Rainevent 6/21/2012

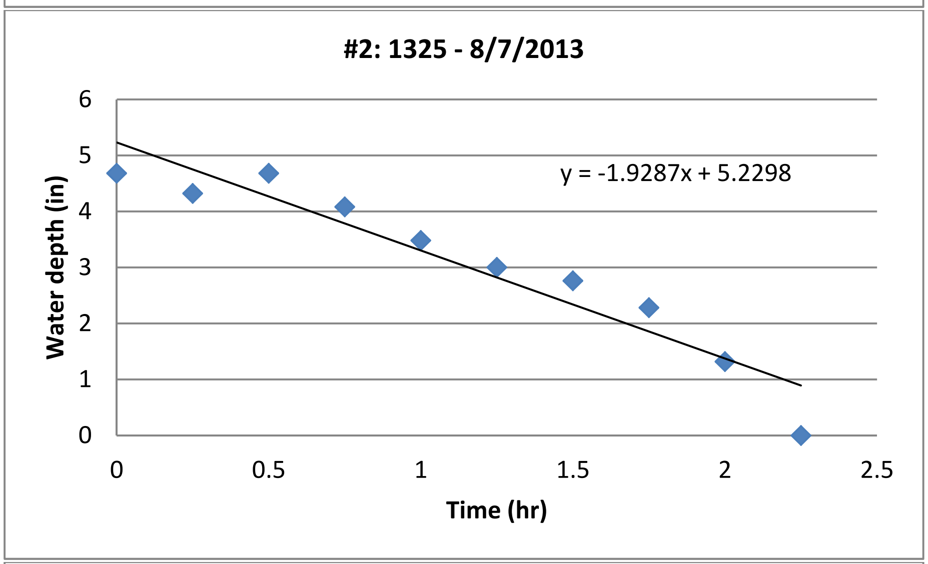


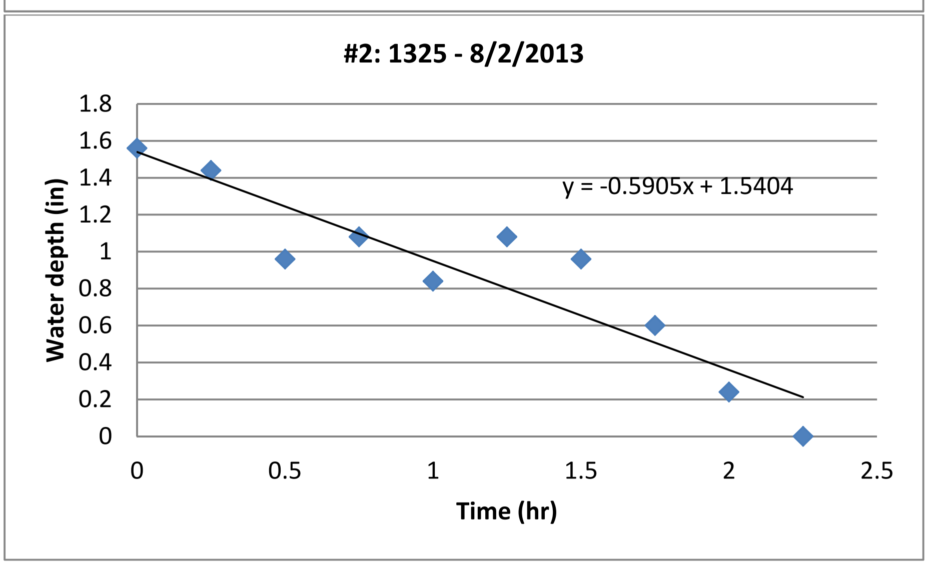
1325 76th Raingarden on Rainevent 06/11/2012

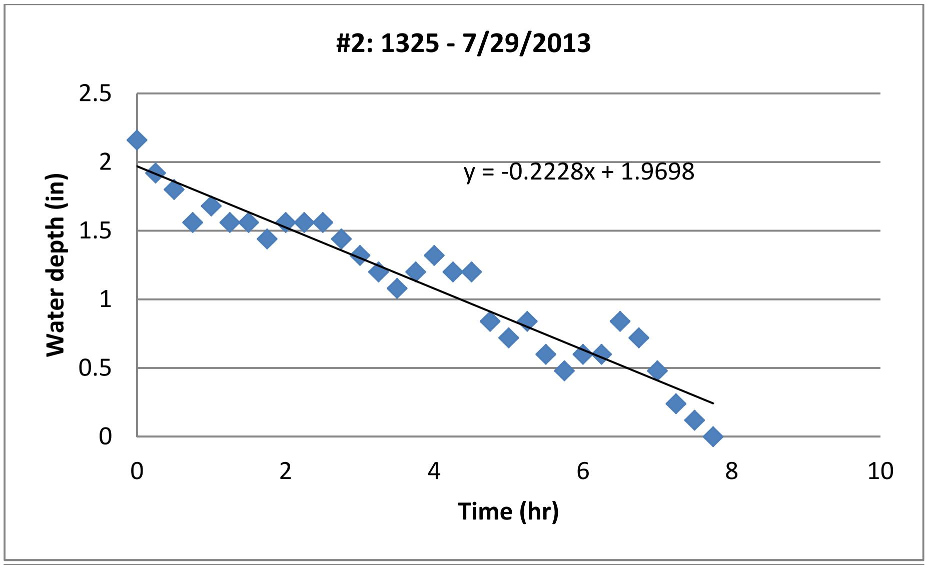


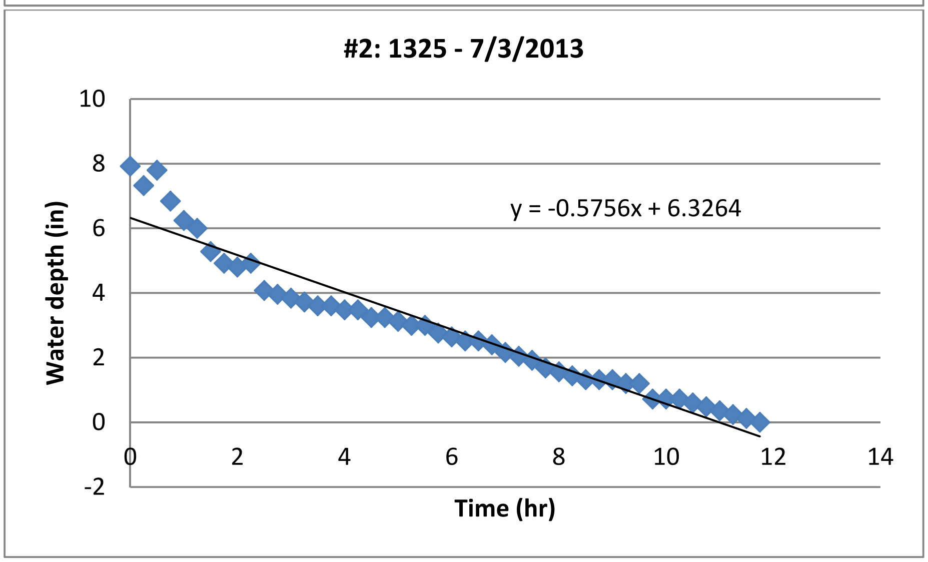
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
0.67	8/12/2013 7:07:00	8/12/2013 10:18:00	3:11	5.52	0:16	1.78
0.87	8/7/2013 4:13:00	8/7/2013 8:45:00	4:32	4.68	2:25	1.93
0.32	8/2/2013 3:46:00	8/2/2013 8:10:00	4:24	1.56	2:07	0.59
0.91	7/29/2013 6:27:00	7/30/2013 9:41:00	27:14	2.16	11:26	0.22
1.5	7/3/2013 17:54:00	7/3/2013 21:36:00	3:42	7.92	2:10	0.58
0.83	6/27/2013 11:46:00	6/28/2013 0:49:00	13:03	8.16	11:18	0.94
1.26	6/15/2013 15:50:00	6/15/2013 22:05:00	6:15	6.24	0:14	0.79
0.39	6/9/2013 00:54:00	6/9/2013 3:53:00	2:59	2.52	1:55	0.62
0.47	6/5/2013 9:44:00	6/5/2013 12:47:00	3:03	2.64	3:05	0.85
1.42	5/2/2013 3:08:00	5/4/2013 4:11:00	49:03	2.04	12:10	0.46
0.95	4/26/2013 4:19:00	4/27/2013 11:02:00	30:43	4.44	23:31	0.42
0.39	4/23/2013 1:34:00	4/23/2013 10:53:00	9:19	6.12	4:14	0.38
1.10	4/17/2013 11:00:00	4/18/2013 20:41:00	33:41	2.16	12:03	0.87
1.10	4/7/2013 19:52:00	4/8/2013 2:25:00	6:33	9.36	5:25	0.99
0.86	10/12/2012 21:00:00	10/13/2012 22:15:00	25:15	0.24		
0.23	9/26/2012 02:25:00	9/26/2012* 04:30:00	02:05	0		
0.43	9/13/2012 14:10:00	9/14/2012 10:10:00	20:00	1.56	0:15	0.9
2.61	8/31/2012 11:00:00	8/31/2012 17:00:00	06:00	1.8	20:15	0.24
0.49	7/25/2012 17:55:00	7/26/2012 08:30:00	12:35	0		
1.03	6/21/2012 00:55:00	6/21/2012 11:25:00	10:30	0.96	9:45	0.47
0.8	6/10/2012 22:05:00	6/11/2012 13:20:00	15:15	0.12		

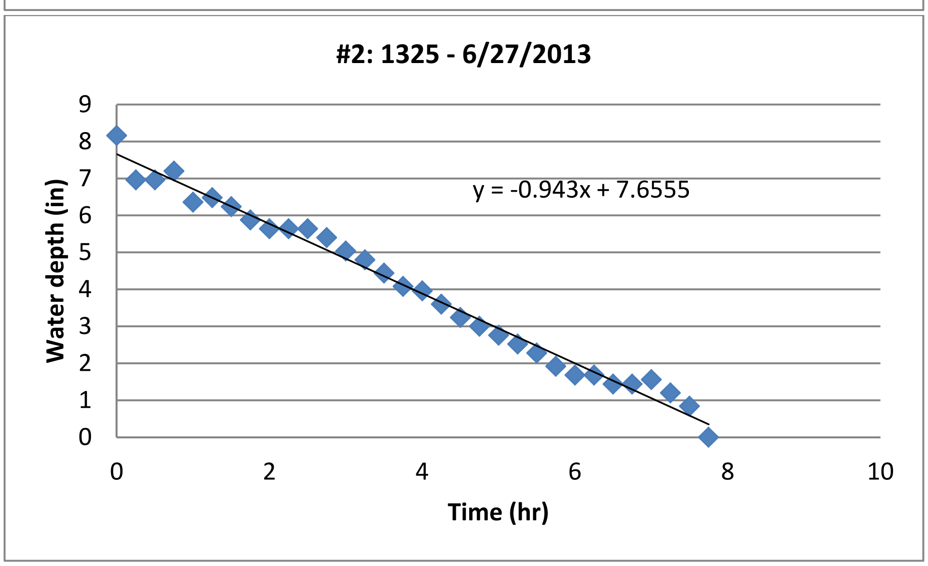

#2: 1325 - 8/12/2013
Water depth (in)
Time (hr)
y = -1.7804x + 3.5389
6
5
4
3
2
1
0
-1
0
0.5
1
1.5
2
2.5

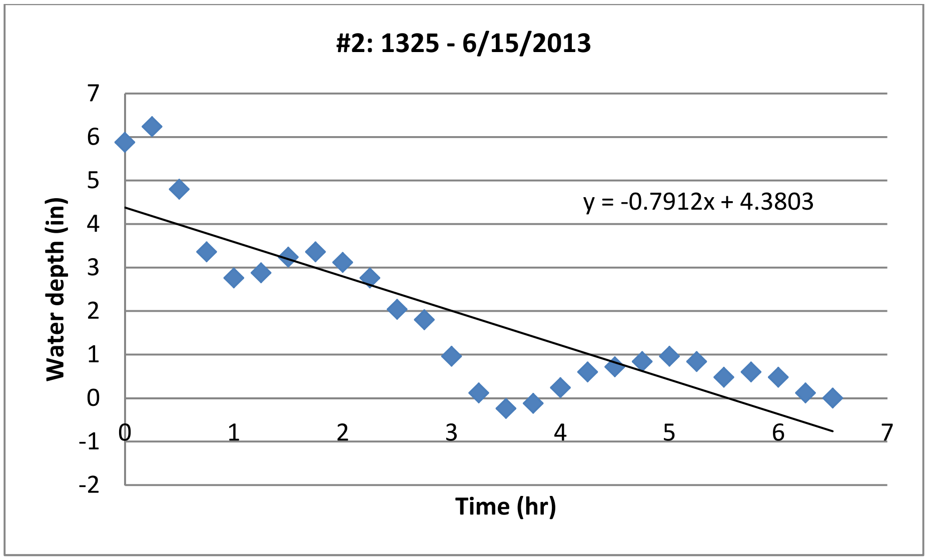

#2: 1325 - 8/7/2013
Water depth (in)
Time (hr)
y = -1.9287x + 5.2298
6
5
4
3
2
1
0
0
0.5
1
1.5
2
2.5

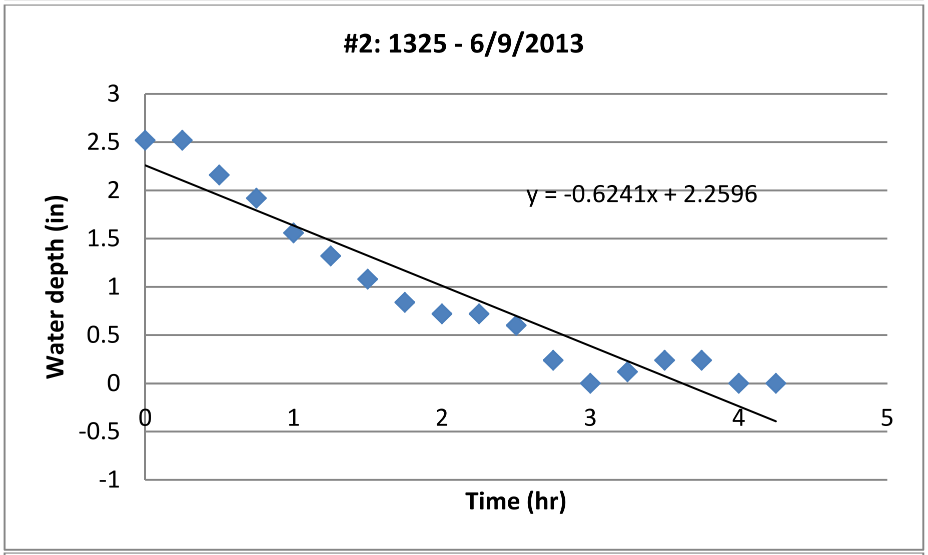

#2: 1325 - 8/2/2013
Water depth (in)
Time (hr)
y = -0.5905x + 1.5404
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5

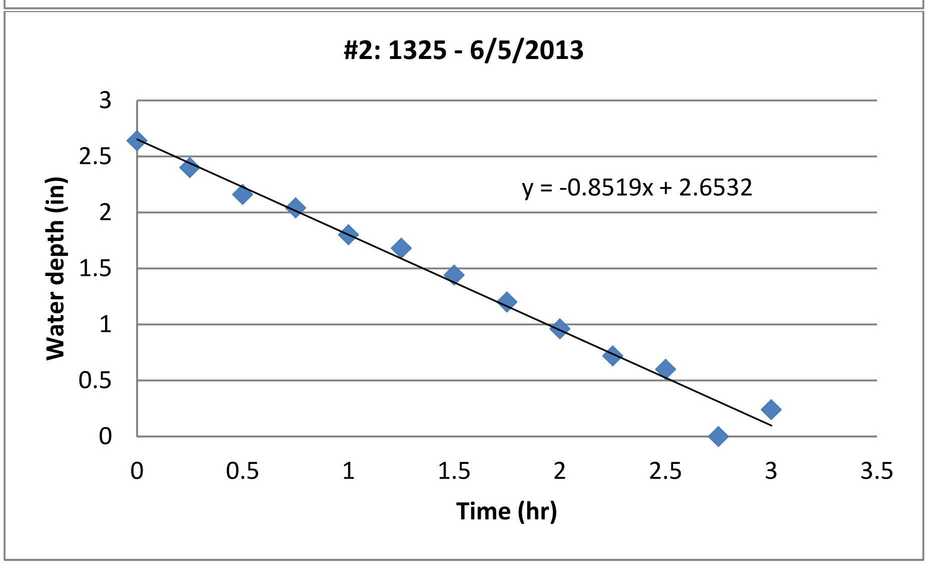

#2: 1325 - 7/29/2013
y = -0.2228x + 1.9698
Water depth (in)
Time (hr)
2.5
2
1.5
1
0.5
0
0
2
4
6
8
10

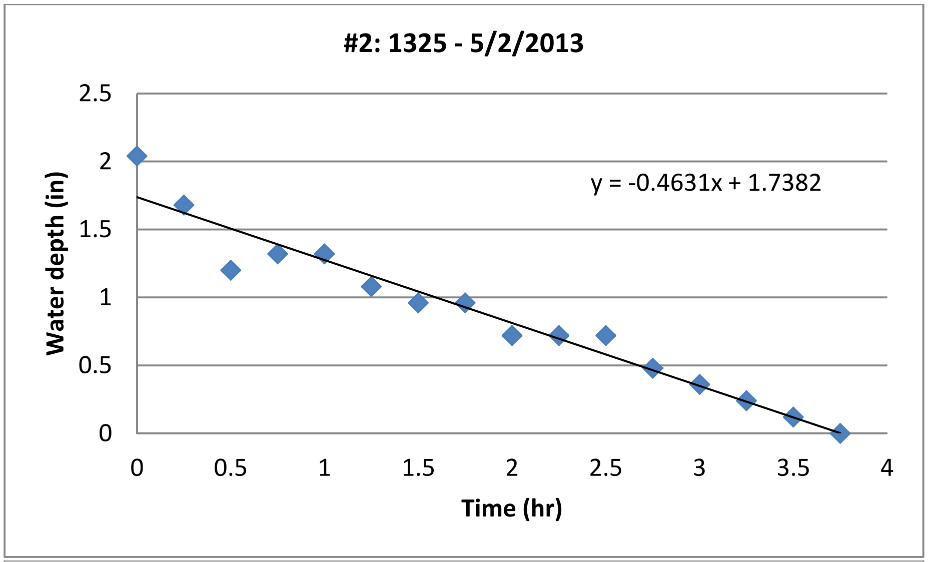

#2: 1325 - 7/3/2013
y = -0.5756x + 6.3264
Water depth (in)
Time (hr)
10
8
6
4
2
0
-2
0
2
4
6
8
10
12
14

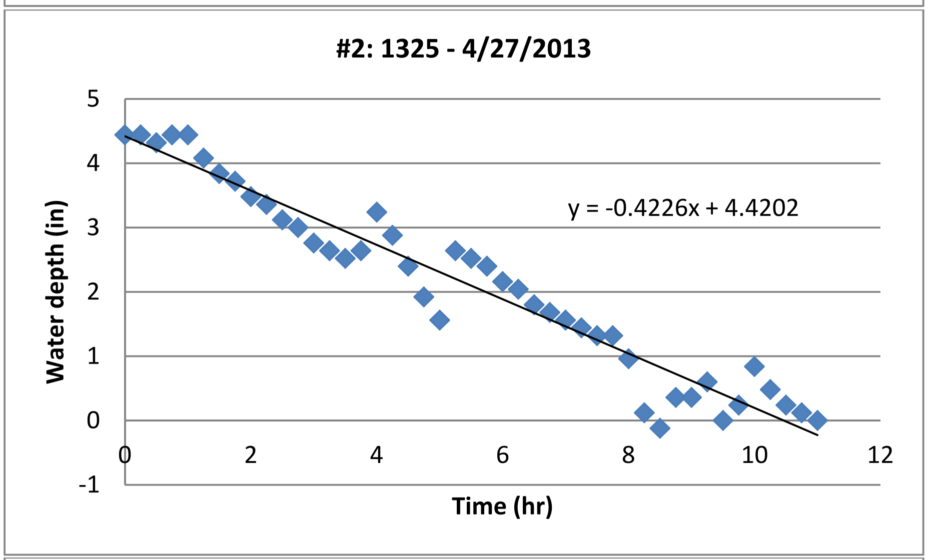

#2: 1325 - 6/27/2013
y = -0.943x + 7.6555
Water depth (in)
Time (hr)
9
8
7
6
5
4
3
2
1
0
0
2
4
6
8
10

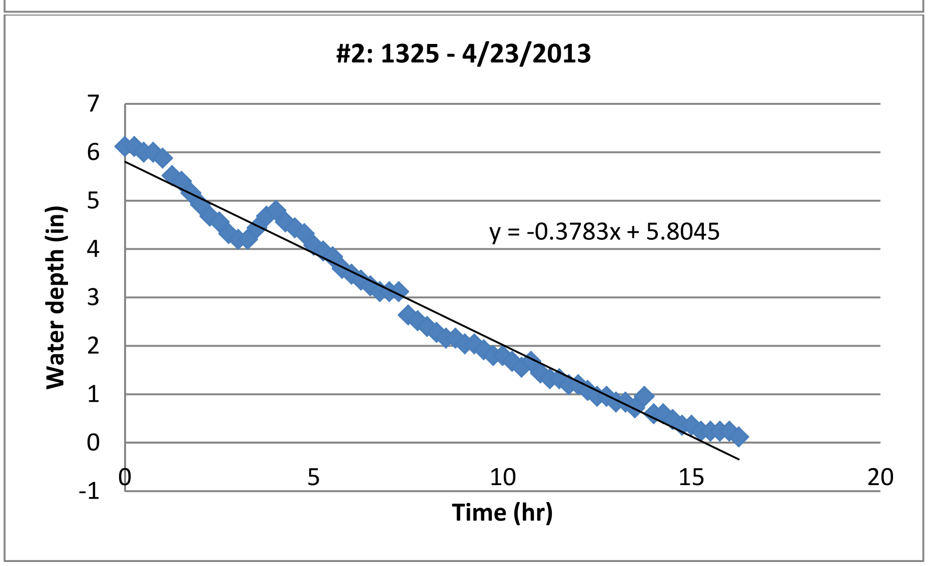

#2: 1325 - 6/15/2013
y = -0.7912x + 4.3803
Water depth (in)
Time (hr)

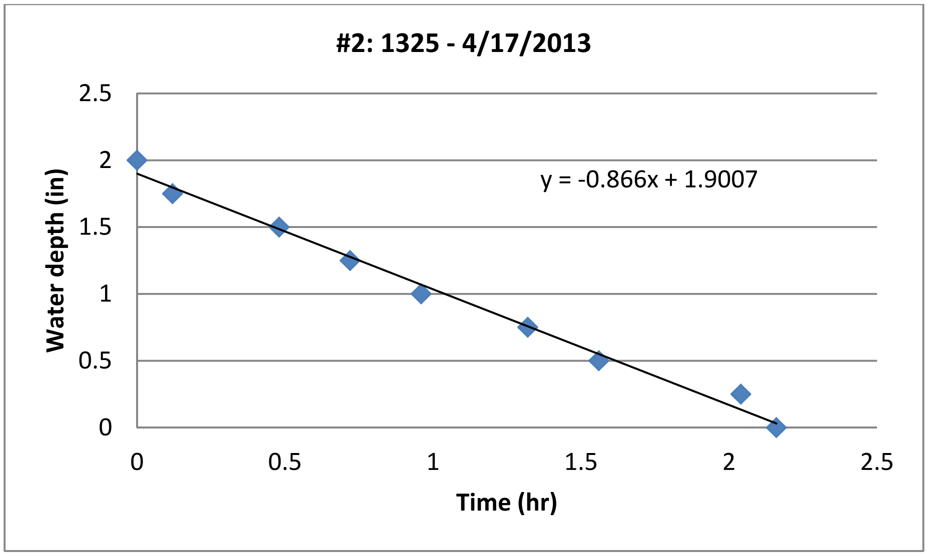

#2: 1325 - 6/9/2013
y = -0.6241x + 2.2596
Water depth (in)
Time (hr)

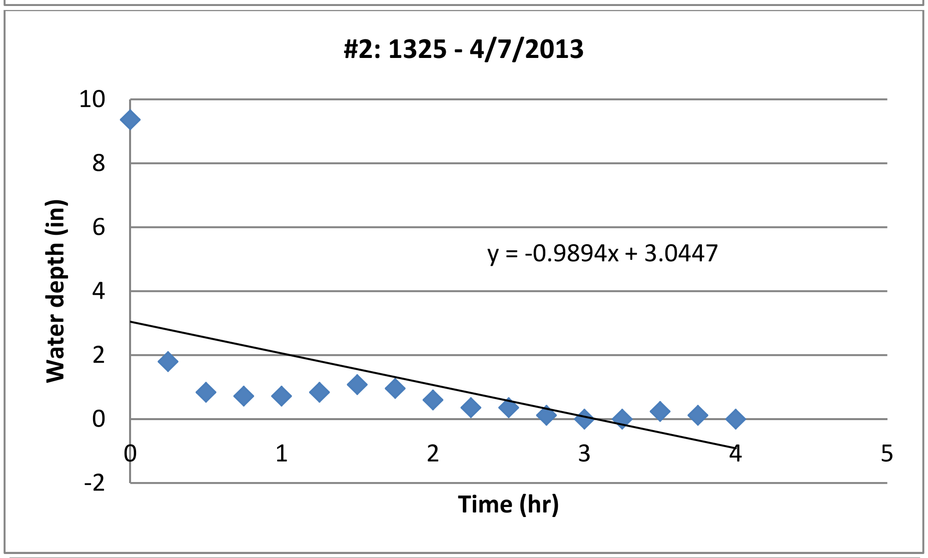

#2: 1325 - 6/5/2013
y = -0.8519x + 2.6532
Water depth (in)
Time (hr)

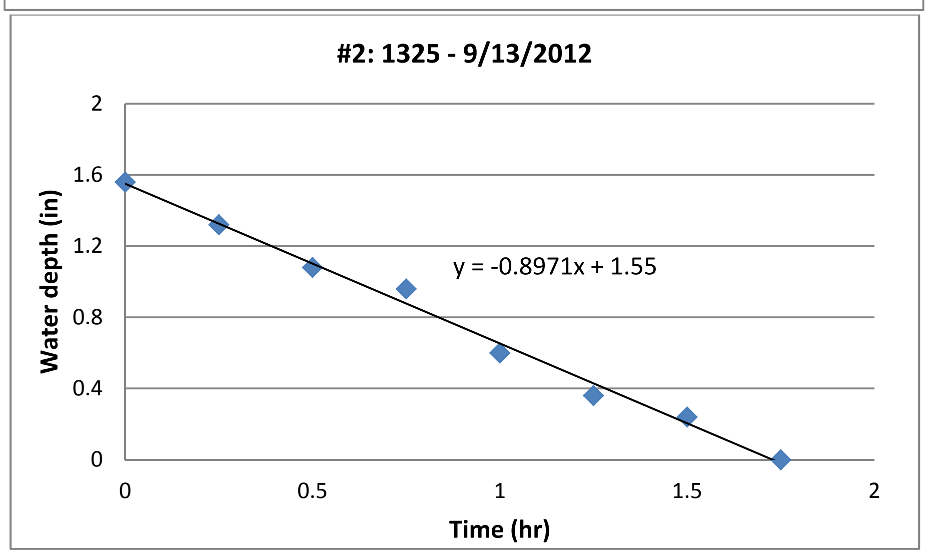

#2: 1325 - 5/2/2013
y = -0.4631x + 1.7382
Water depth (in)
Time (hr)

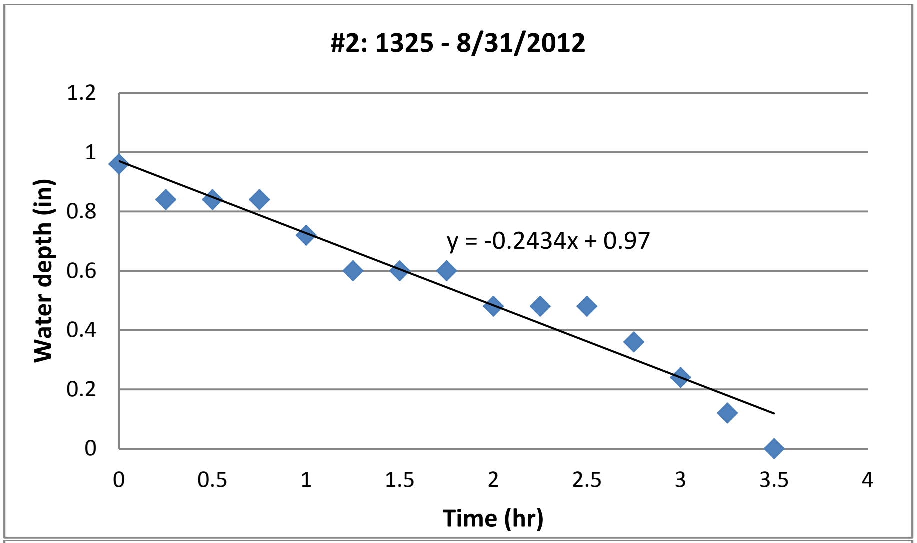

#2: 1325 - 4/27/2013
y = -0.4226x + 4.4202
Water depth (in)
Time (hr)

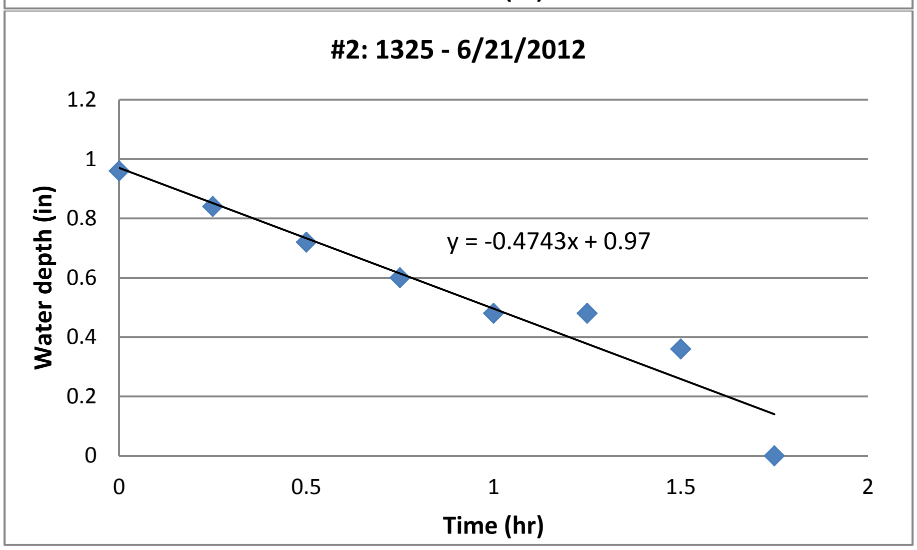

#2: 1325 - 4/23/2013
y = -0.3783x + 5.8045
Water depth (in)
Time (hr)


#2: 1325 - 4/17/2013
2.5
2
1.5
1
0.5
0
Water depth (in)
y = -0.866x + 1.9007
0
0.5
1
1.5
2
2.5
Time (hr)


#2: 1325 - 4/7/2013
10
8
6
4
2
0
-2
Water depth (in)
y = -0.9894x + 3.0447
0
1
2
3
4
5
Time (hr)

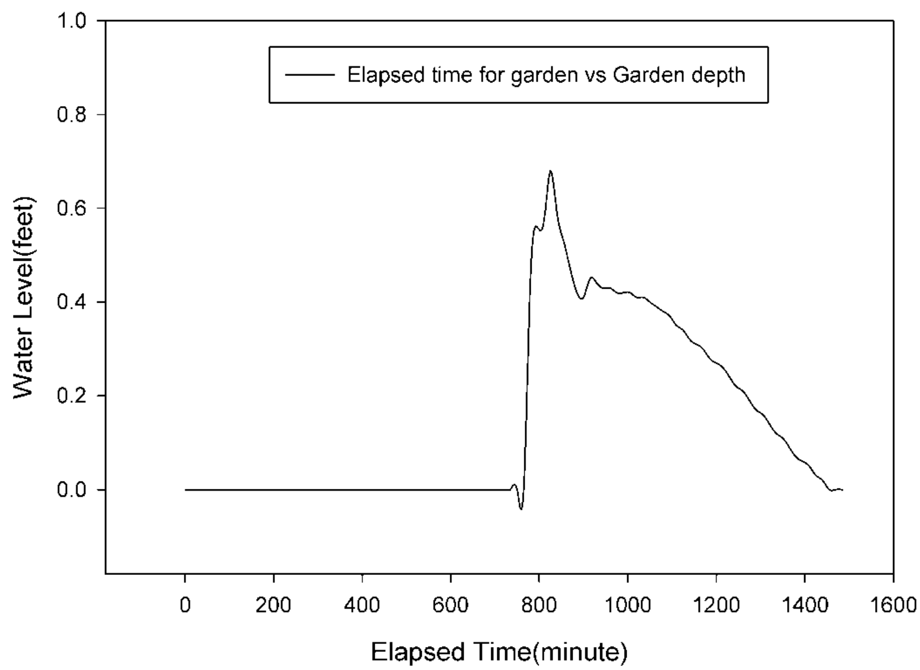

#2: 1325 - 9/13/2012
2
1.6
1.2
0.8
0.4
0
Water depth (in)
y = -0.8971x + 1.55
0
0.5
1
1.5
2
Time (hr)


#2: 1325 - 8/31/2012
Water depth (in)
Time (hr)
y = -0.2434x + 0.97
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4

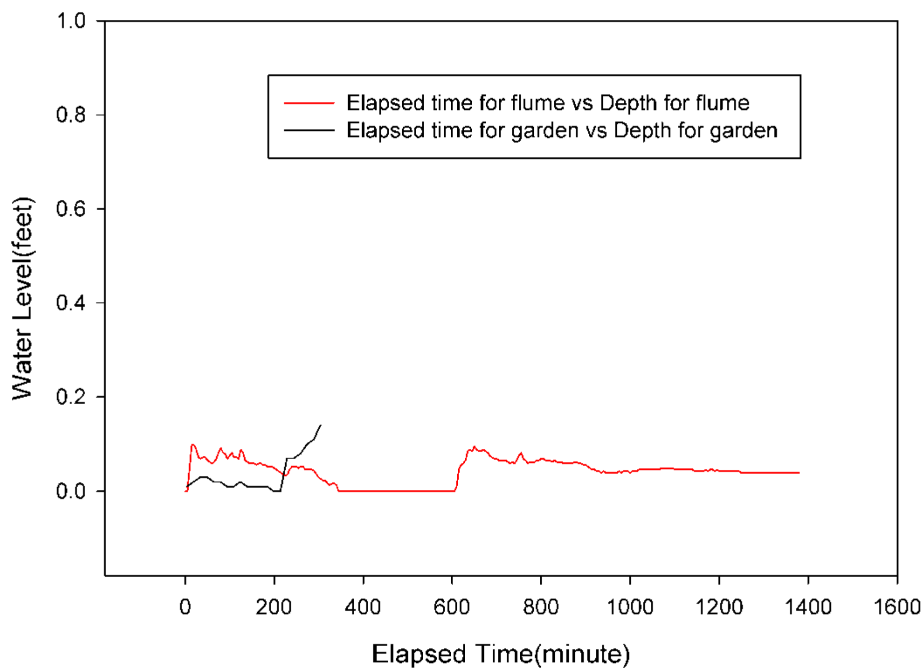

#2: 1325 - 6/21/2012
Water depth (in)
Time (hr)
y = -0.4743x + 0.97
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2

3- Curb Extension Biofilter - 1419 E 76th Terr.

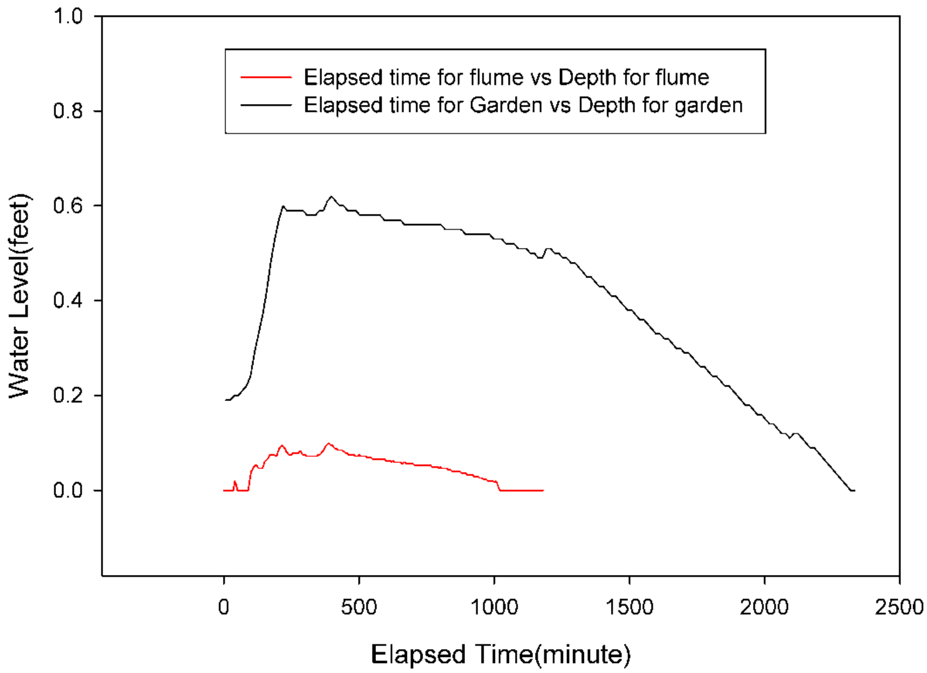
1419 76th terr on Rainevent 6/27/13



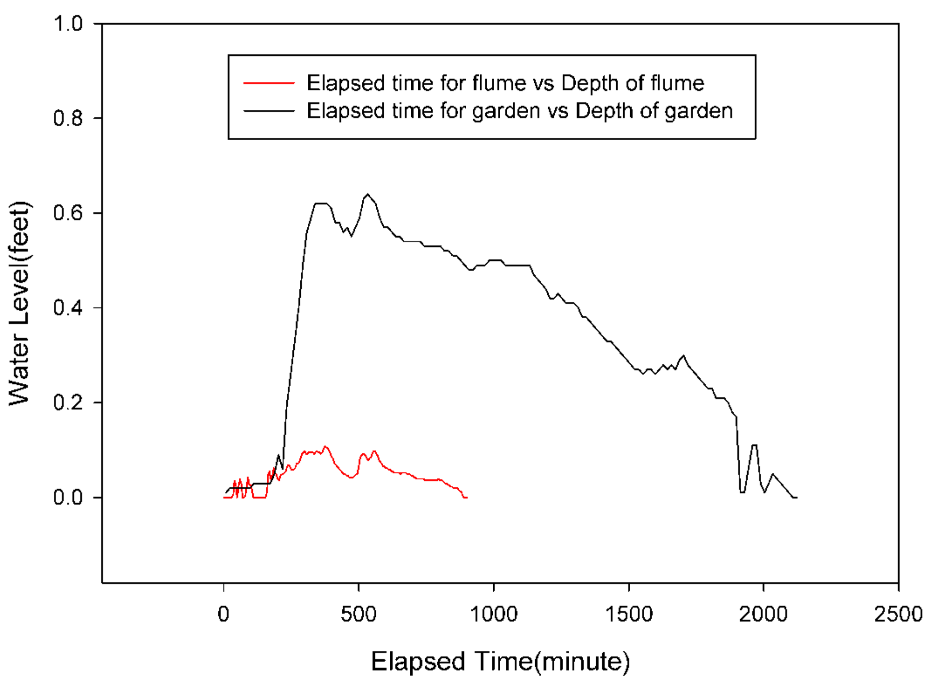
1419 76th terr on Rainevent 5/2/13-5/3/13



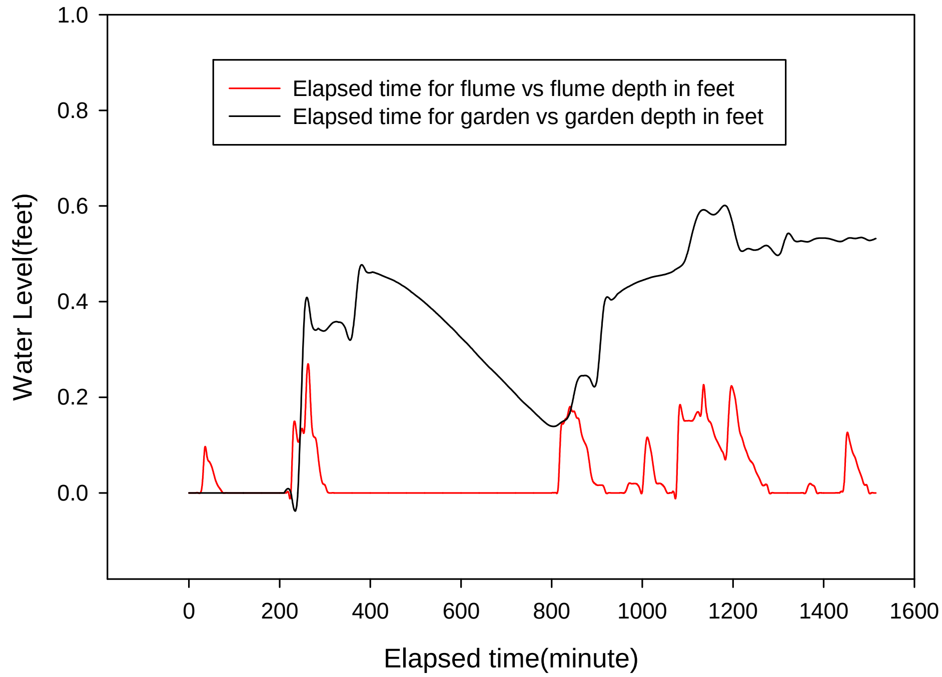
1419 76th terr on Rainevent 04/26/13-04/27/13



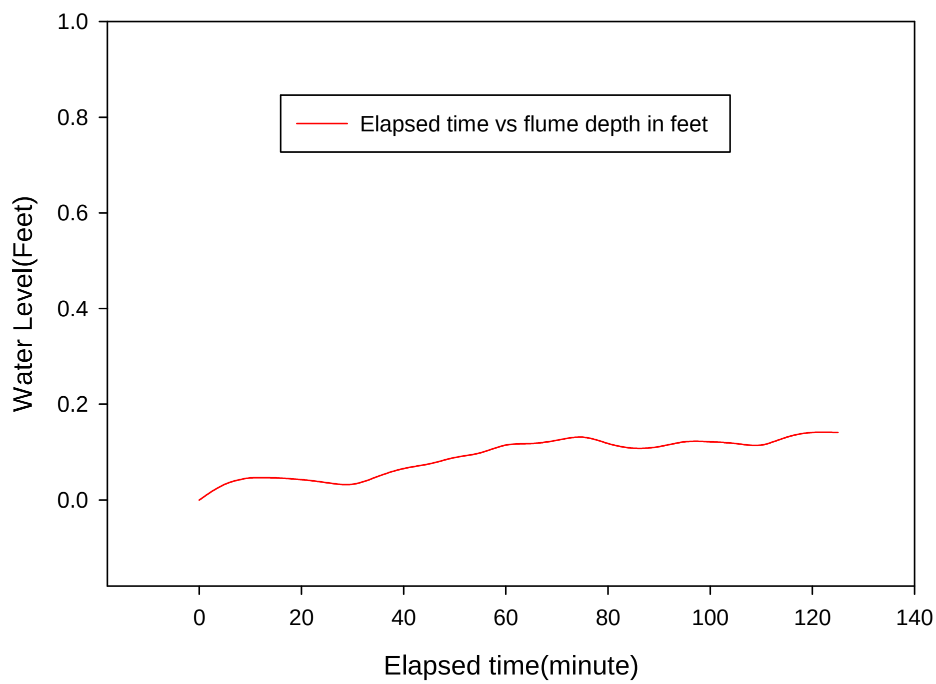
1425 76th terr on Rainevent 04/23/13



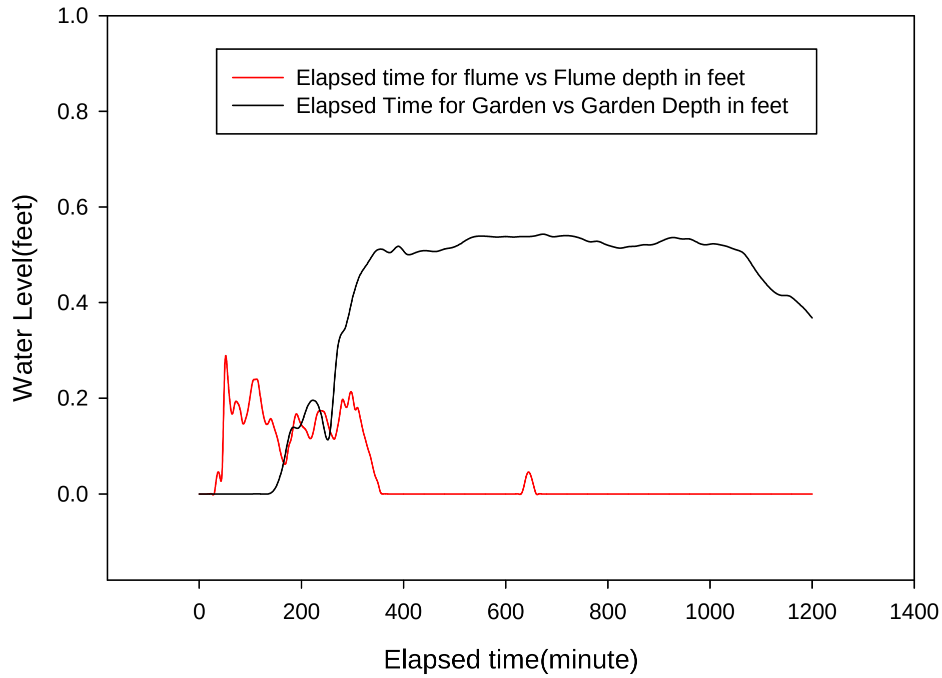
1419 76th terr Raingarden on Rainevent 10/13/2012



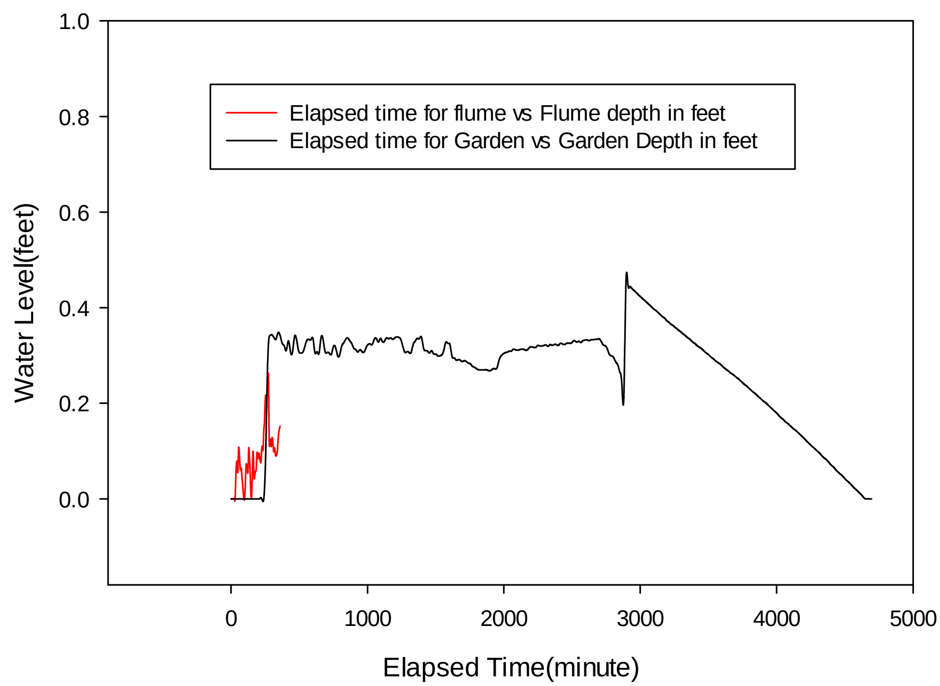
1419 76th terr Raingarden on Rainevent 9/26/2012



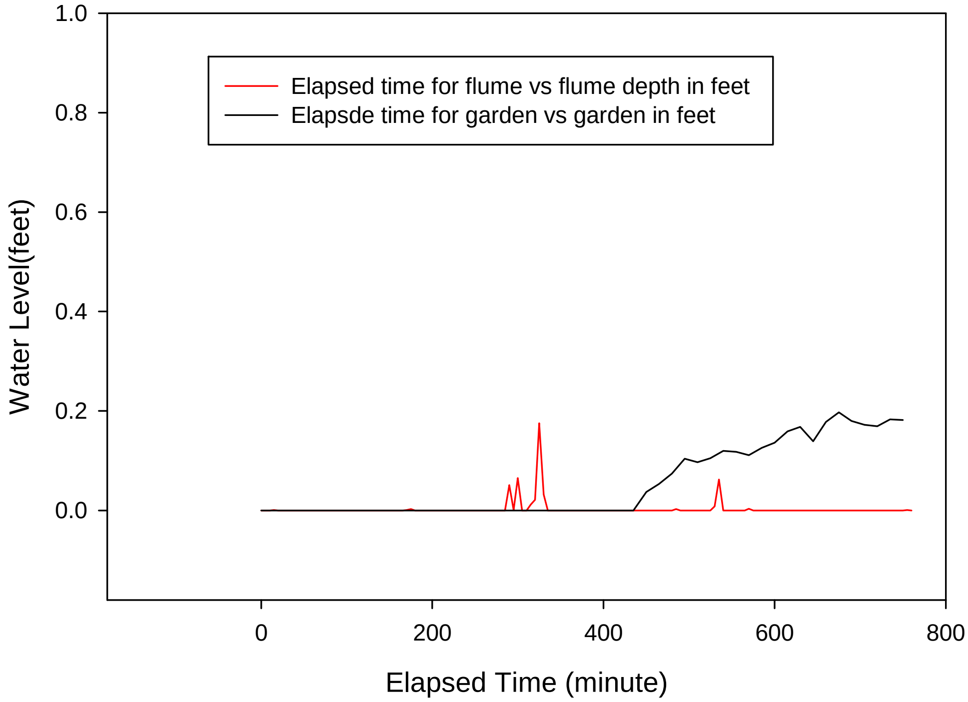
1419 76th terr Raingarden on Rainevent 9/13/2012



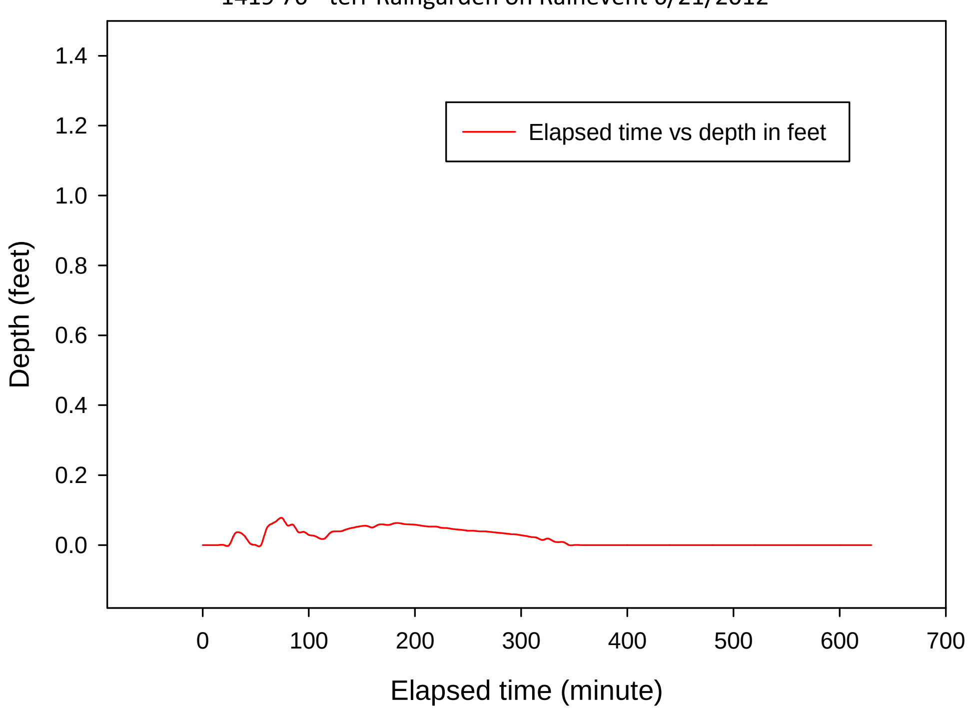
1419 76th Terr Flume on Rainevent 08/31/2012



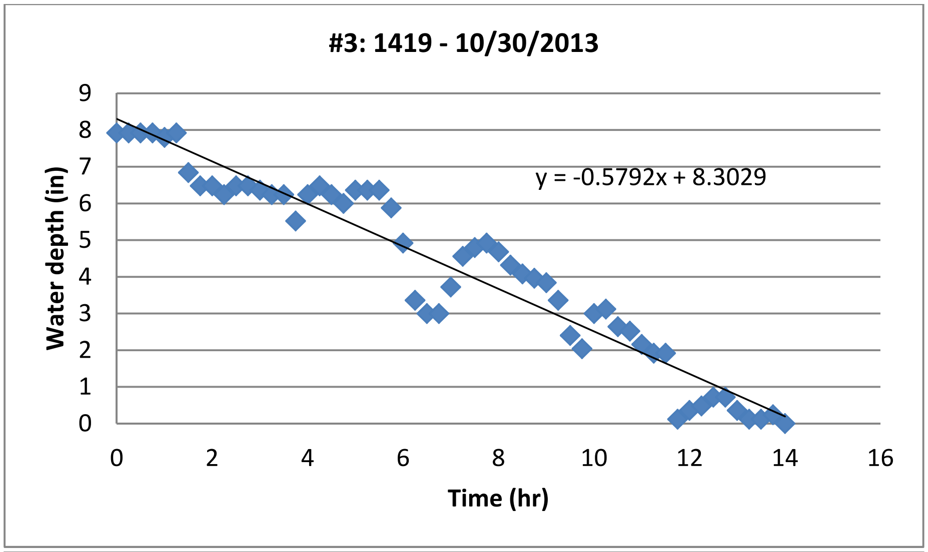
1419 76th terr Raingarden on Rainevent 7/26/2012



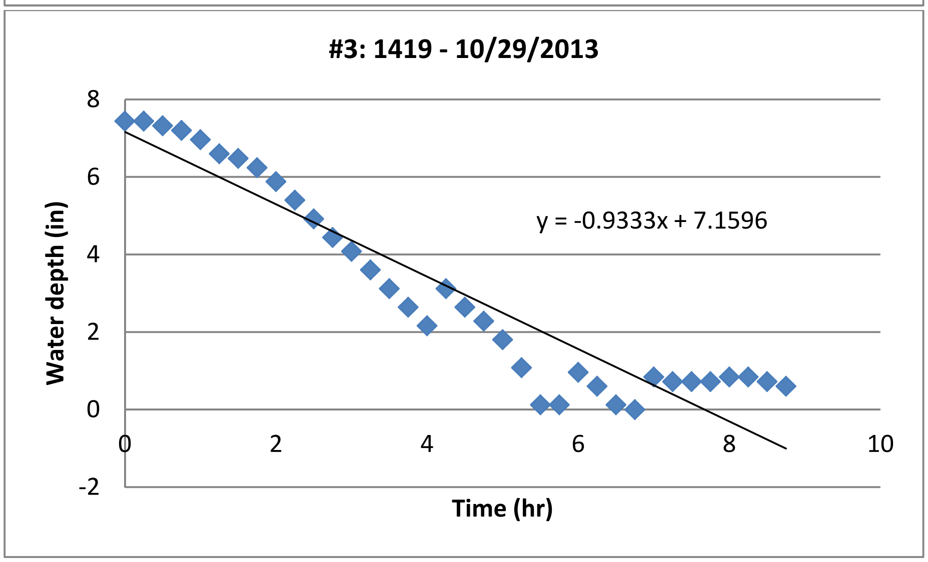
1419 76th terr Raingarden on Rainevent 6/21/2012



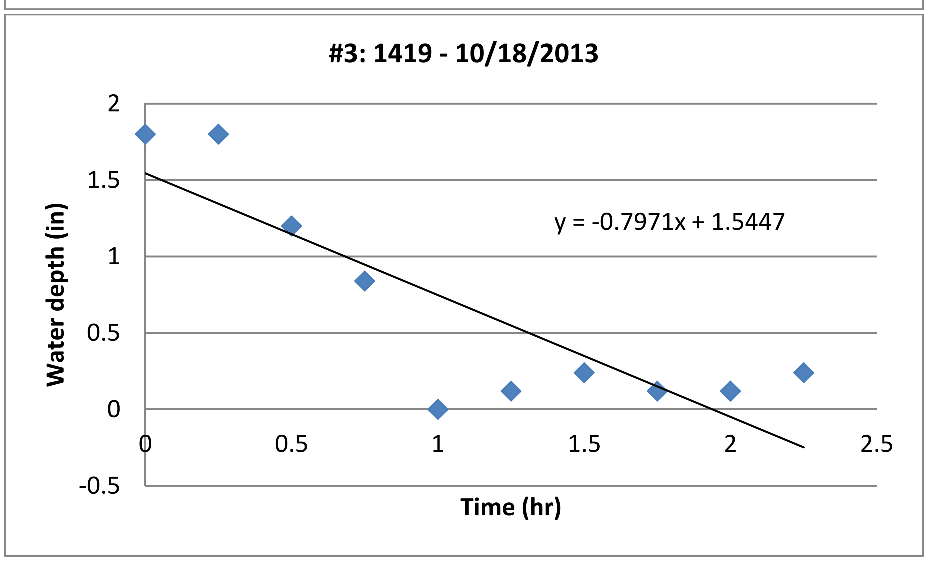
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Total volume of inflow (gal)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:37		7.92	17:49	0.58
0.83	10/29/2013 2:52:00	10/29/2013 9:41:00	6:49		7.44	4:19	0.93
0.12	10/18/2013 14:03:00	10/18/2013 17:05:00	3:02		1.8	0	0.80
0.87	10/4/2013 22:36:00	10/5/2013 3:48:00	5:12		5.28	1:34	1.62
1.89	9/19/2013 19:00:00	9/19/2013 22:23:00	3:23		11.52	0:25	0.79
0.67	8/12/2013 7:07:00	8/12/2013 10:18:00	3:11		7.32	0:17	0.79
0.87	8/7/2013 4:13:00	8/7/2013 8:45:00	4:32		6.24	2:41	0.82
0.55	8/6/2013 3:24:00	8/6/2013 4:59:00	1:35		4.08	1:30	0.80
0.32	8/2/2013 3:46:00	8/2/2013 8:10:00	4:24		2.88	1:53	0.49
1.5	7/3/2013 17:54:00	7/3/2013 21:36:00	3:42		8.16	1:11	0.62
0.83	6/27/2013 11:46:00	6/28/2013 0:49:00	13:03		8.88	12:03	0.63
1.42	5/2/2013 3:08:00	5/4/2013 4:11:00	49:03		7.08	26:51	0.25
0.95	4/26/2013 4:19:00	4/27/2013 11:02:00	30:43		9.48	27:19	0.26
0.39	4/23/2013 1:34:00	4/23/2013 10:53:00	9:19		8.64	7:19	0.28
0.39	4/14/2013 19:02:00	4/15/2013 7:08:00	12:06		2.16	28:51	0.72
0.39	4/14/2013 19:02:00	4/15/2013 7:08:00	12:06		3.0	13:51	0.33
0.86	10/12/2012 21:00:00	10/13/2012 22:15:00	25:15	3487	7.2	4:55	0.62
0.23	9/26/2012 02:25:00	9/26/2012* 04:30:00	02:05	583	0		
0.43	9/13/2012 14:10:00	9/14/2012 10:10:00	20:00	3987	6.5	2:30	More water depth is needed
2.61	8/31/2012 11:00:00	8/31/2012 17:00:00	06:00	1940	5.4	4:15	0.19
0.49	7/25/2012 17:55:00	7/26/2012 08:30:00	12:35	103	2.35	7:30	More water depth is needed
1.03	6/21/2012 00:55:00	6/21/2012 11:25:00	10:30	232	0		



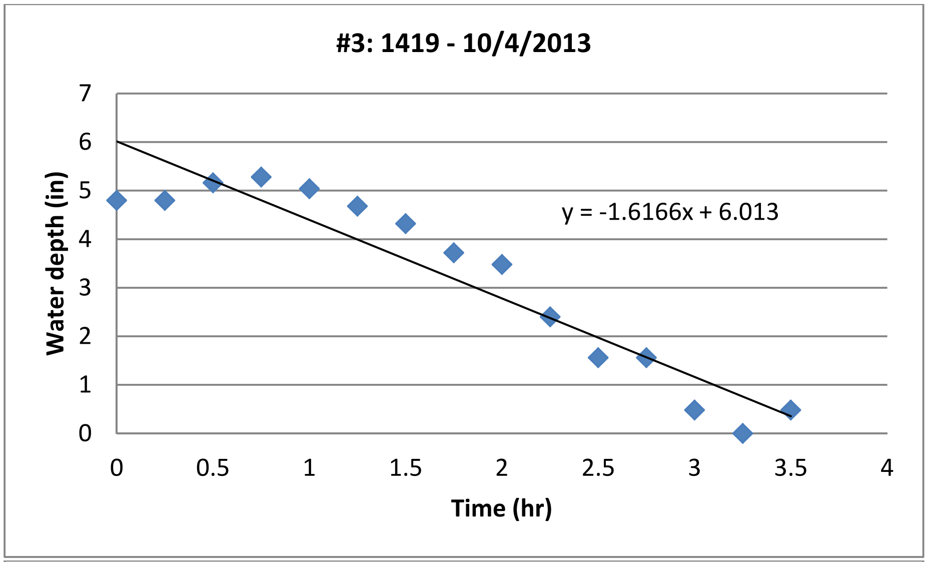
#3: 1419 - 10/30/2013
y = -0.5792x + 8.3029
Water depth (in)
Time (hr)
9
8
7
6
5
4
3
2
1
0
0
2
4
6
8
10
12
14
16

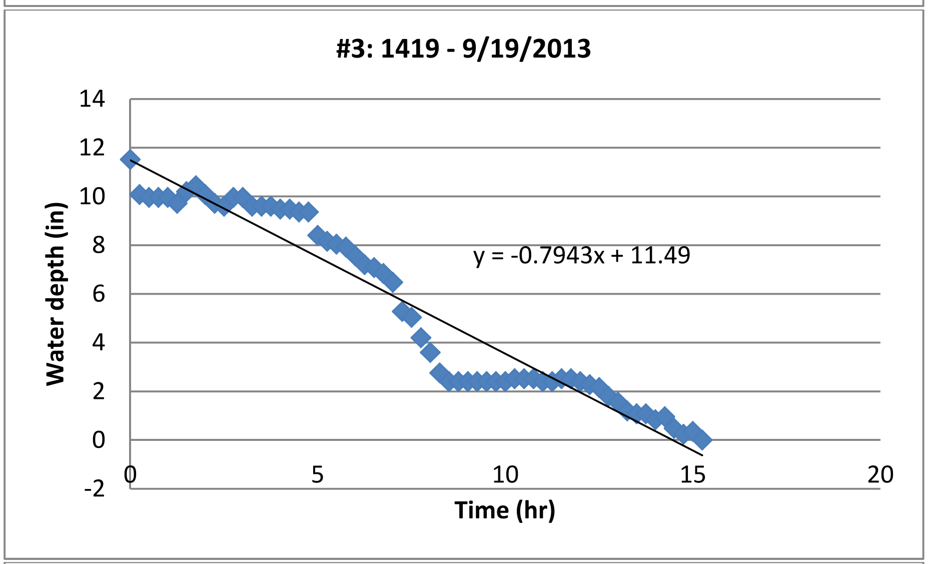


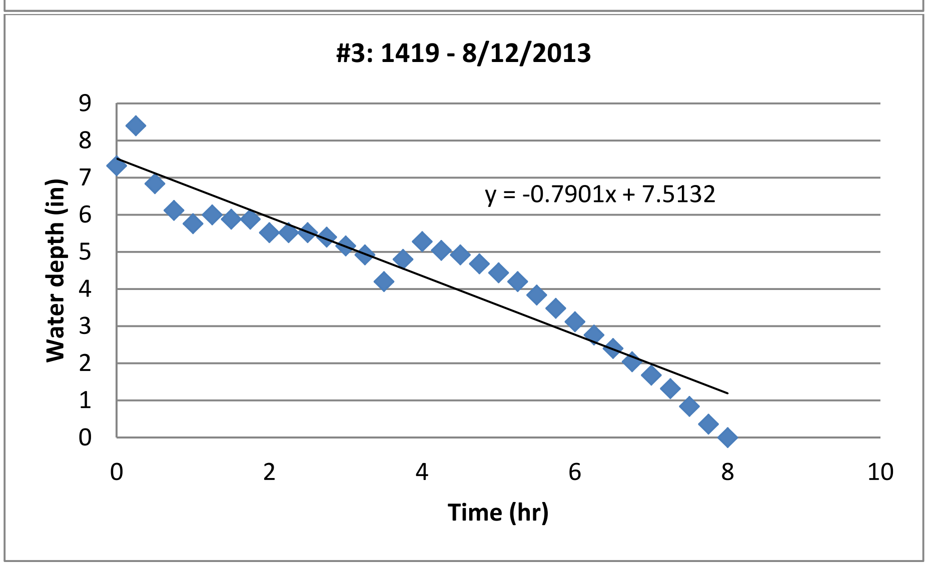
#3: 1419 - 10/29/2013
y = -0.9333x + 7.1596
Water depth (in)
Time (hr)
8
6
4
2
0
-2
0
2
4
6
8
10

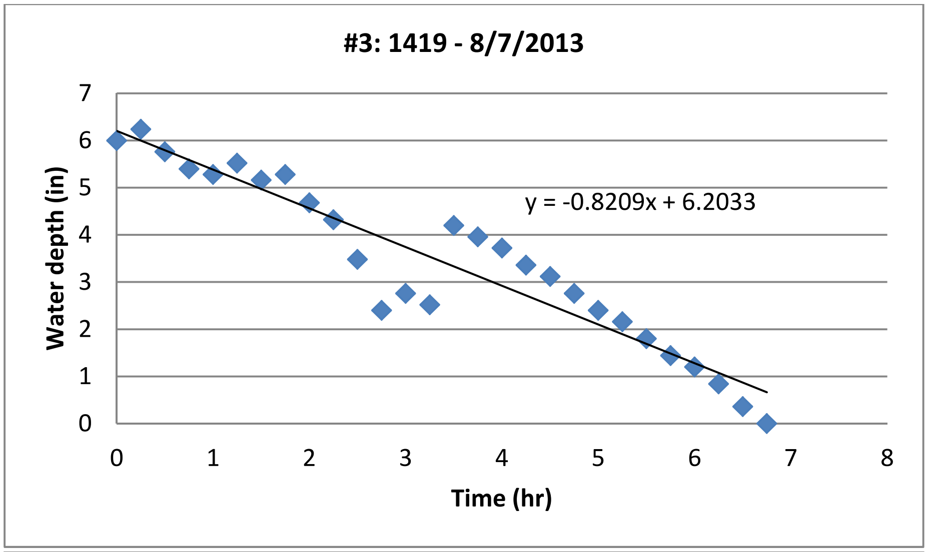


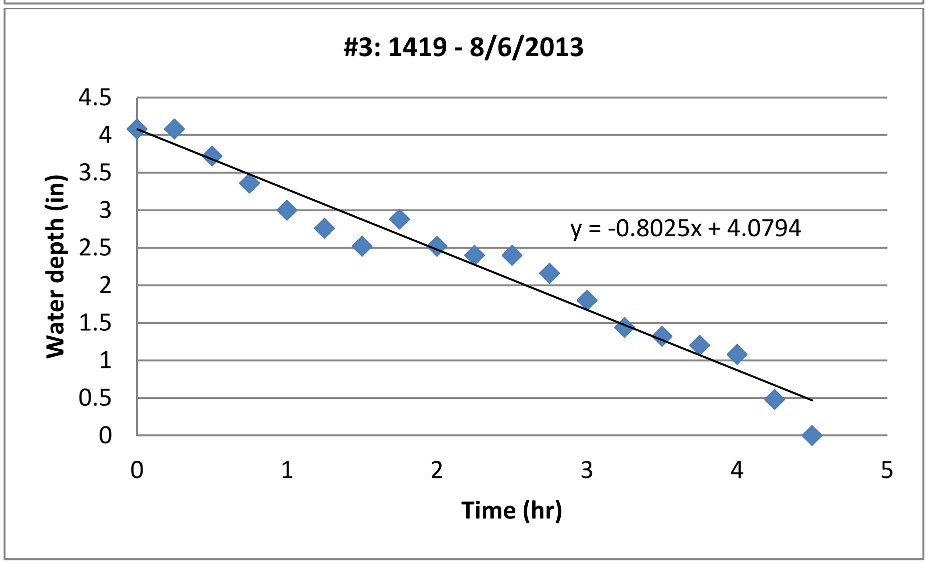
#3: 1419 - 10/18/2013
y = -0.7971x + 1.5447
Water depth (in)
Time (hr)
2
1.5
1
0.5
0
-0.5
0
0.5
1
1.5
2
2.5

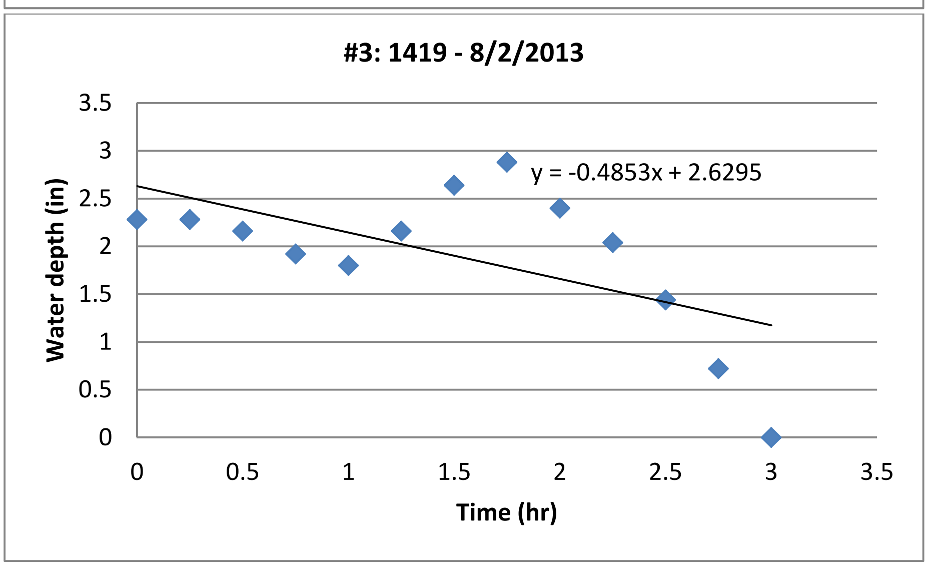

#3: 1419 - 10/4/2013
Water depth (in)
Time (hr)
y = -1.6166x + 6.013
0
1
2
3
4
5
6
7
0
0.5
1
1.5
2
2.5
3
3.5
4

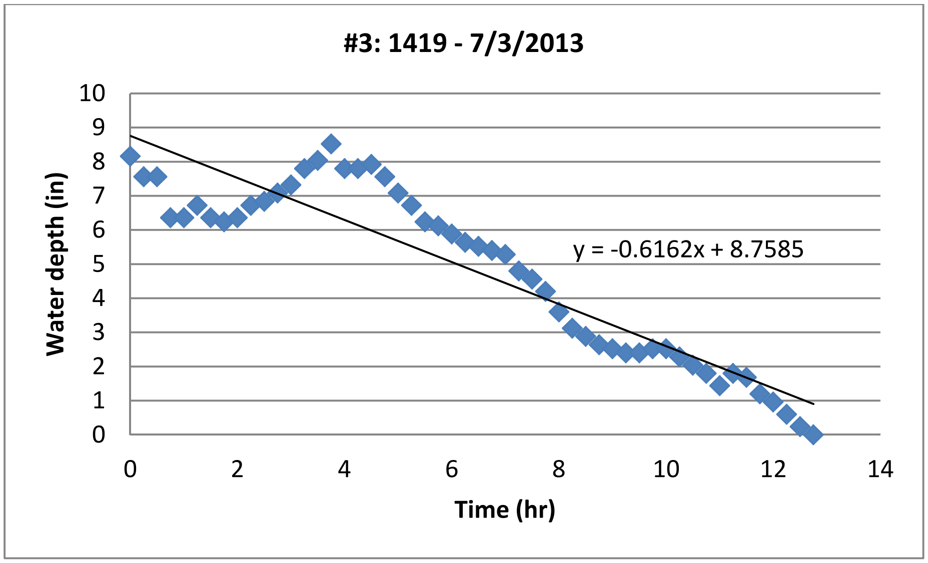

#3: 1419 - 9/19/2013
Water depth (in)
Time (hr)
y = -0.7943x + 11.49
-2
0
2
4
6
8
10
12
14
0
5
10
15
20

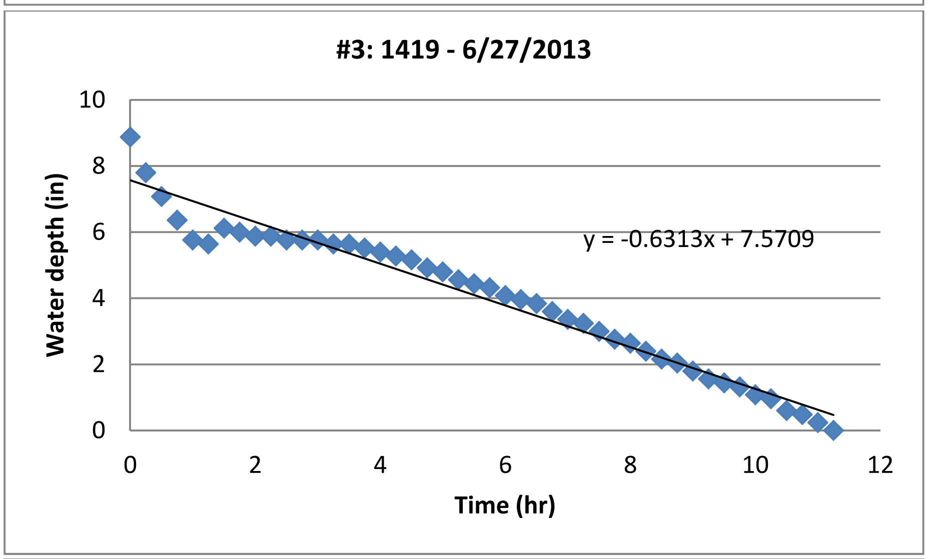

#3: 1419 - 8/12/2013
Water depth (in)
Time (hr)
y = -0.7901x + 7.5132
0
1
2
3
4
5
6
7
8
9
0
2
4
6
8
10

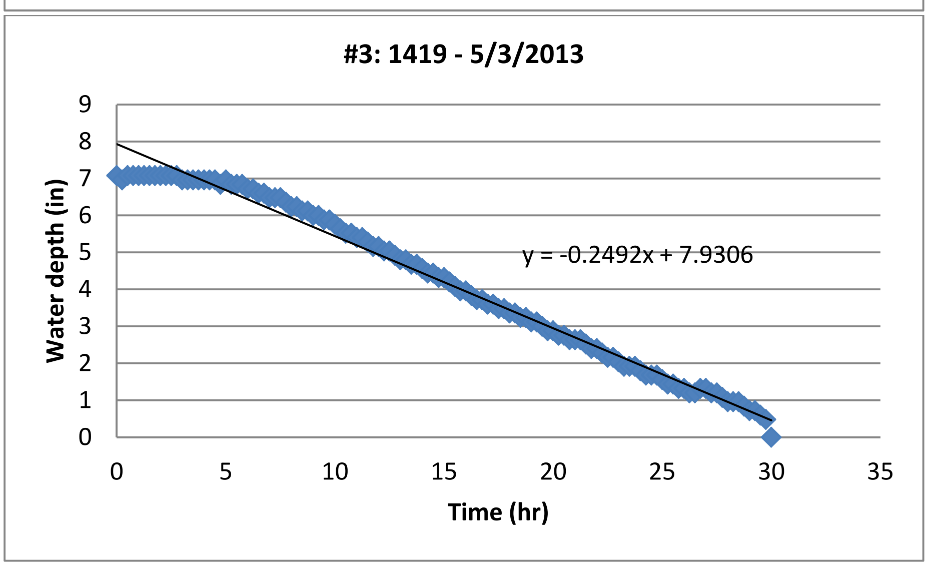

#3: 1419 - 8/7/2013
Water depth (in)
Time (hr)
y = -0.8209x + 6.2033

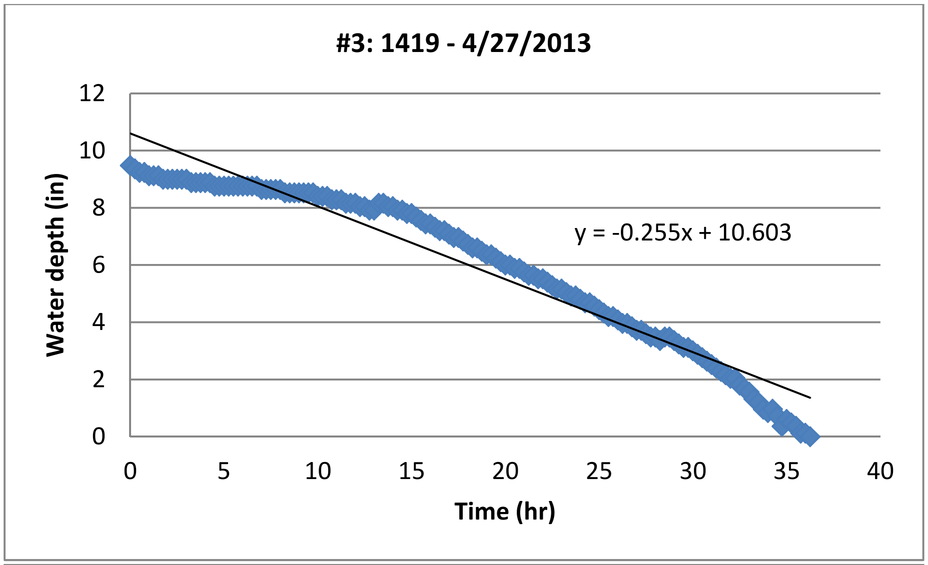

#3: 1419 - 8/6/2013
Water depth (in)
Time (hr)
y = -0.8025x + 4.0794

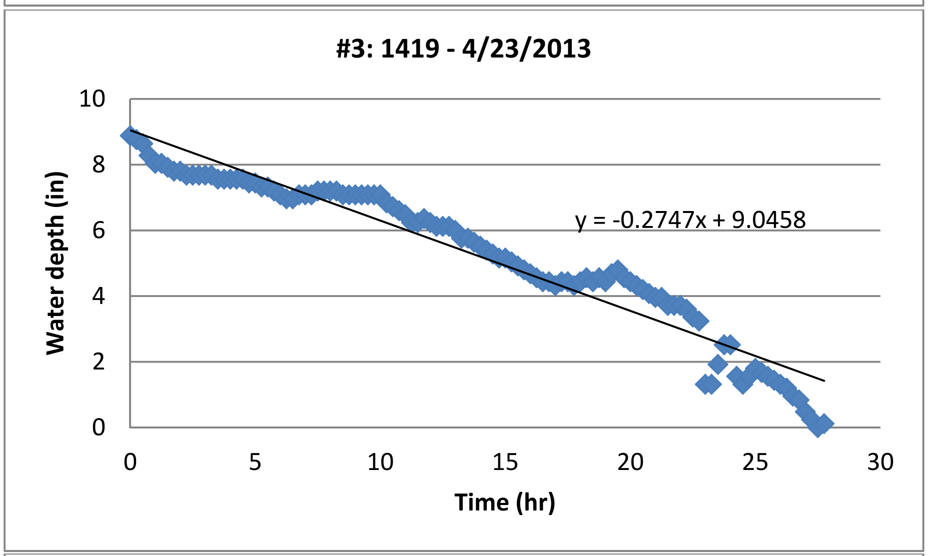

#3: 1419 - 8/2/2013
Water depth (in)
Time (hr)
y = -0.4853x + 2.6295

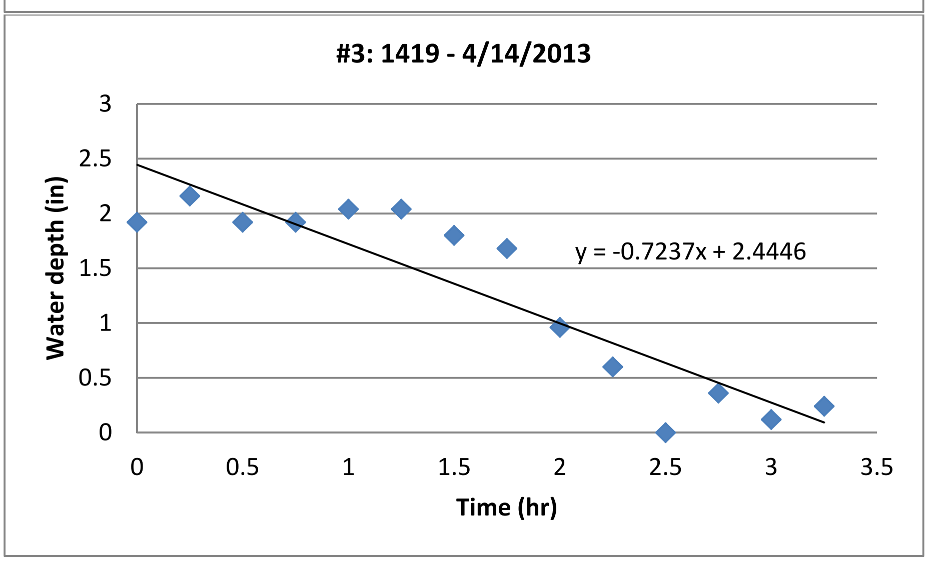

#3: 1419 - 7/3/2013
Water depth (in)
Time (hr)
y = -0.6162x + 8.7585
0
1
2
3
4
5
6
7
8
9
10
0
2
4
6
8
10
12
14

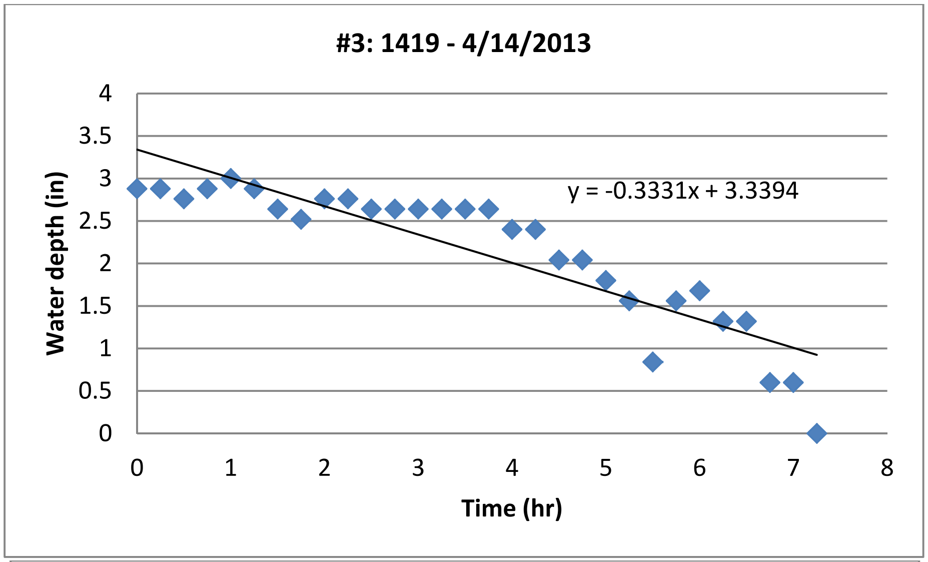

#3: 1419 - 6/27/2013
Water depth (in)
Time (hr)
y = -0.6313x + 7.5709
0
2
4
6
8
10
0
2
4
6
8
10
12

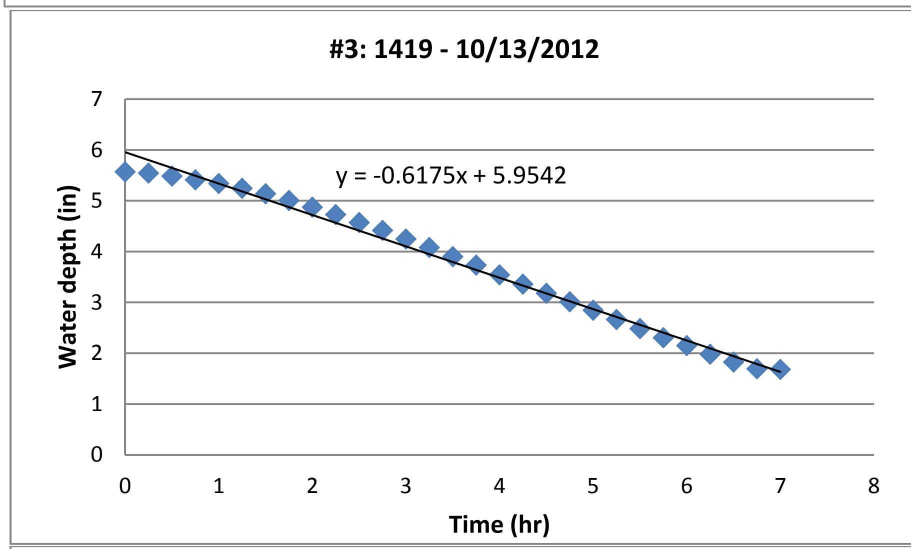

#3: 1419 - 5/3/2013
Water depth (in)
Time (hr)
y = -0.2492x + 7.9306
0
1
2
3
4
5
6
7
8
9
0
5
10
15
20
25
30
35

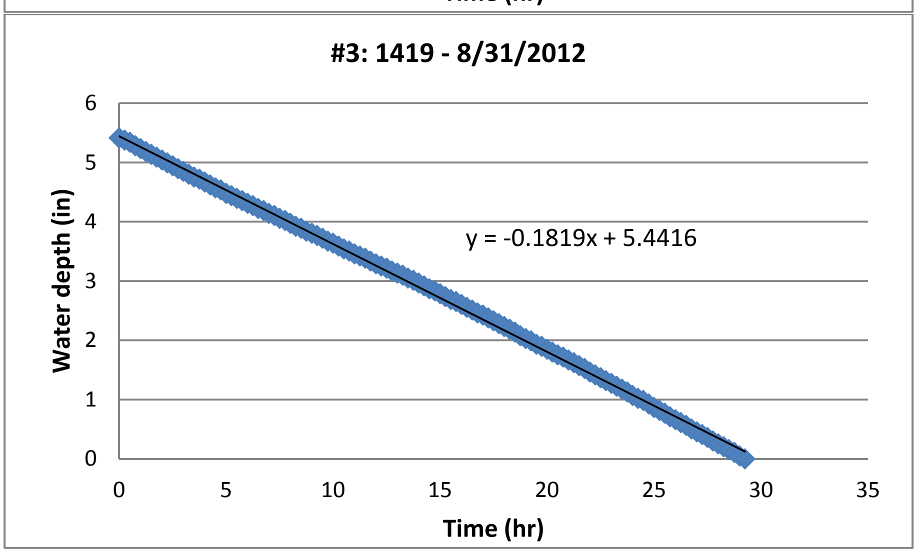

#3: 1419 - 4/27/2013
Water depth (in)
Time (hr)
y = -0.255x + 10.603
12
10
8
6
4
2
0
0
5
10
15
20
25
30
35
40


#3: 1419 - 4/23/2013
Water depth (in)
Time (hr)
y = -0.2747x + 9.0458
10
8
6
4
2
0
0
5
10
15
20
25
30

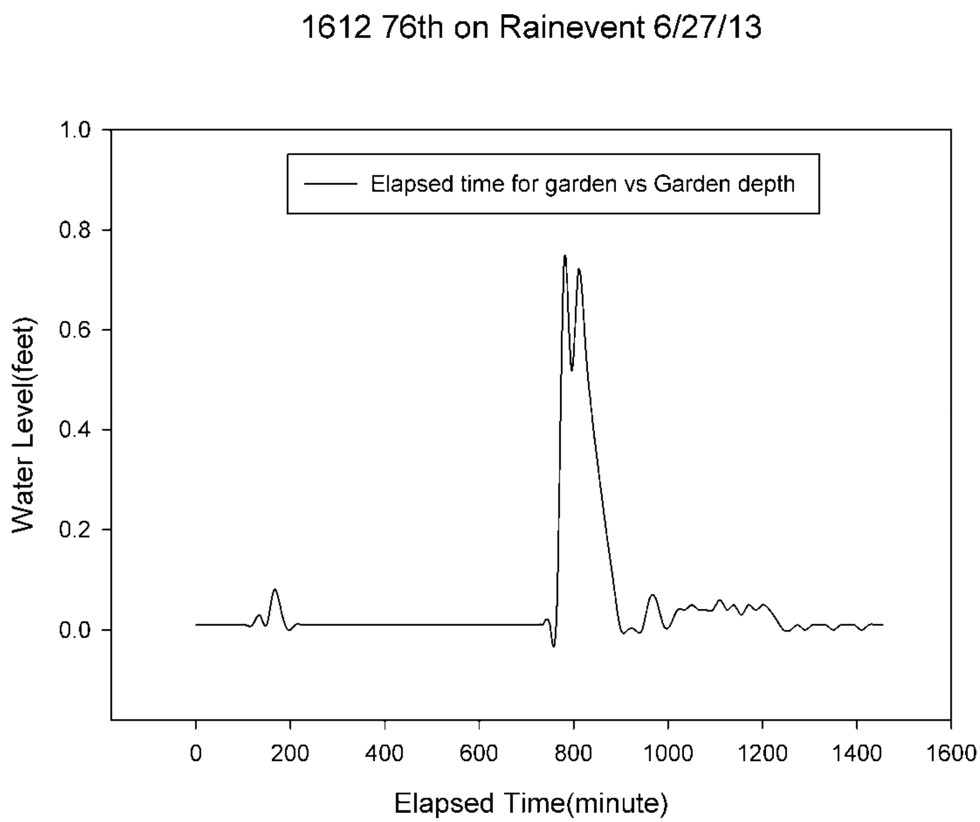

#3: 1419 - 4/14/2013
Water depth (in)
Time (hr)
y = -0.7237x + 2.4446
3
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3
3.5


#3: 1419 - 4/14/2013
Water depth (in)
Time (hr)
y = -0.3331x + 3.3394
0
0.5
1
1.5
2
2.5
3
3.5
4
0
1
2
3
4
5
6
7
8

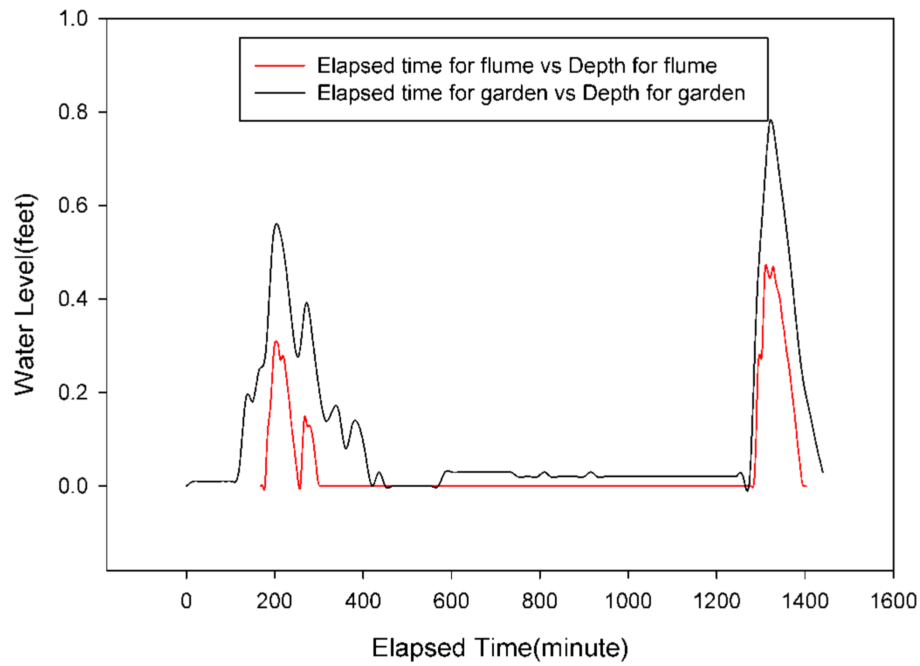

#3: 1419 - 10/13/2012
Water depth (in)
Time (hr)
y = -0.6175x + 5.9542
0
1
2
3
4
5
6
7
0
1
2
3
4
5
6
7
8


#3: 1419 - 8/31/2012
Water depth (in)
Time (hr)
y = -0.1819x + 5.4416
0
1
2
3
4
5
6
0
5
10
15
20
25
30
35

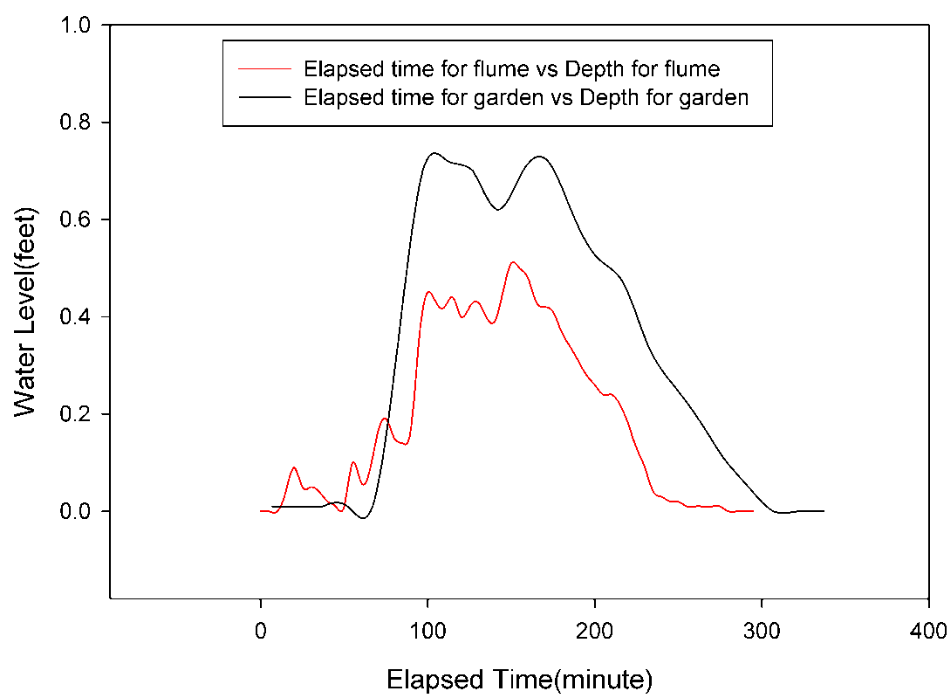
4- Curb-Cut Biofilter - 1612 E 76th St.



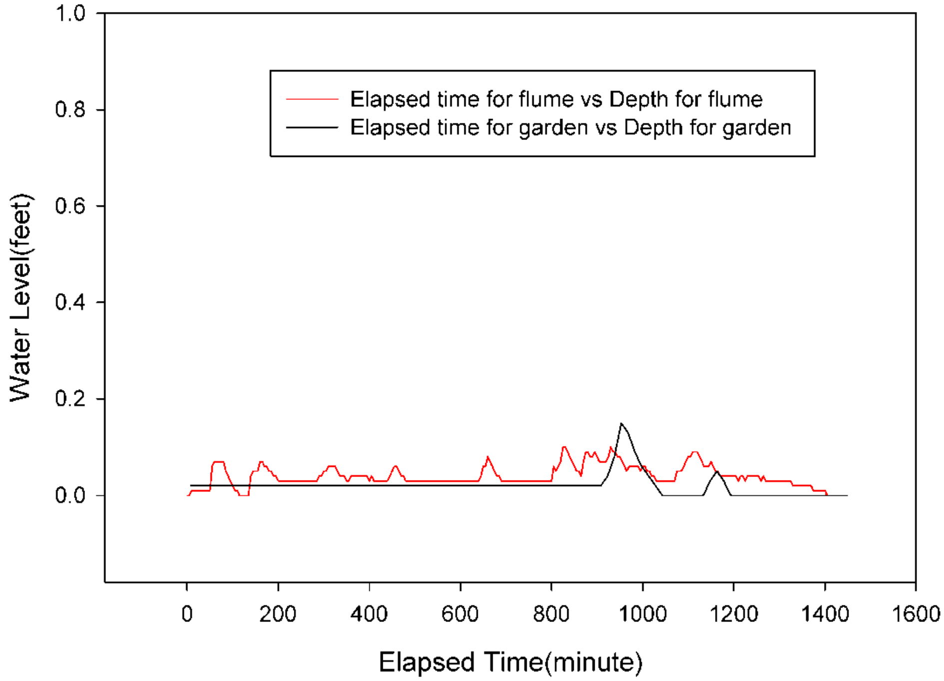
1612 76th on Rainevent 5/29/13-5/31/13



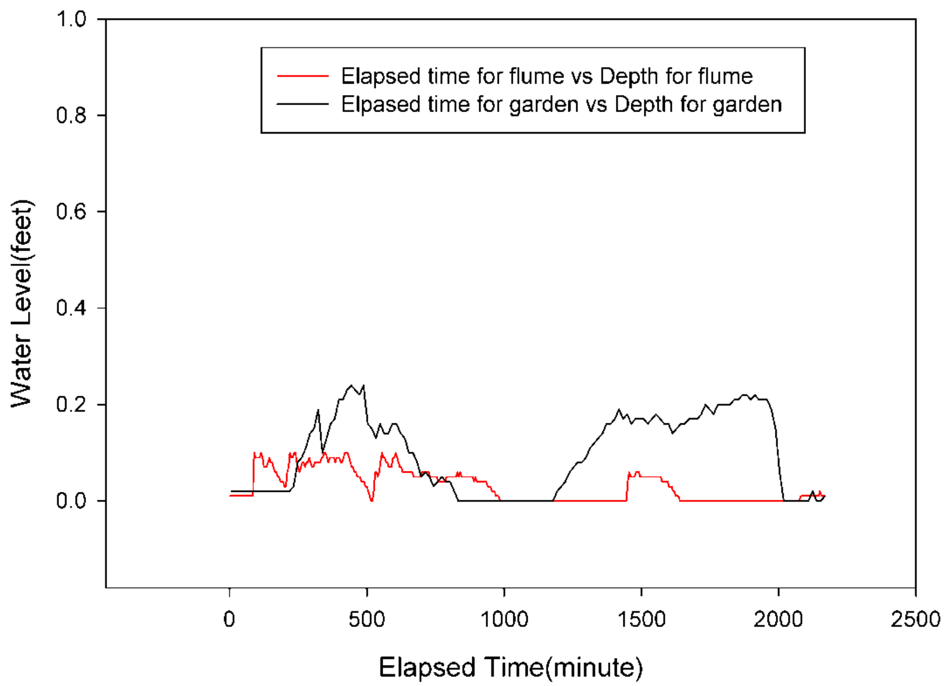
1612 76th on Rainevent 5/27/13-5/28/13



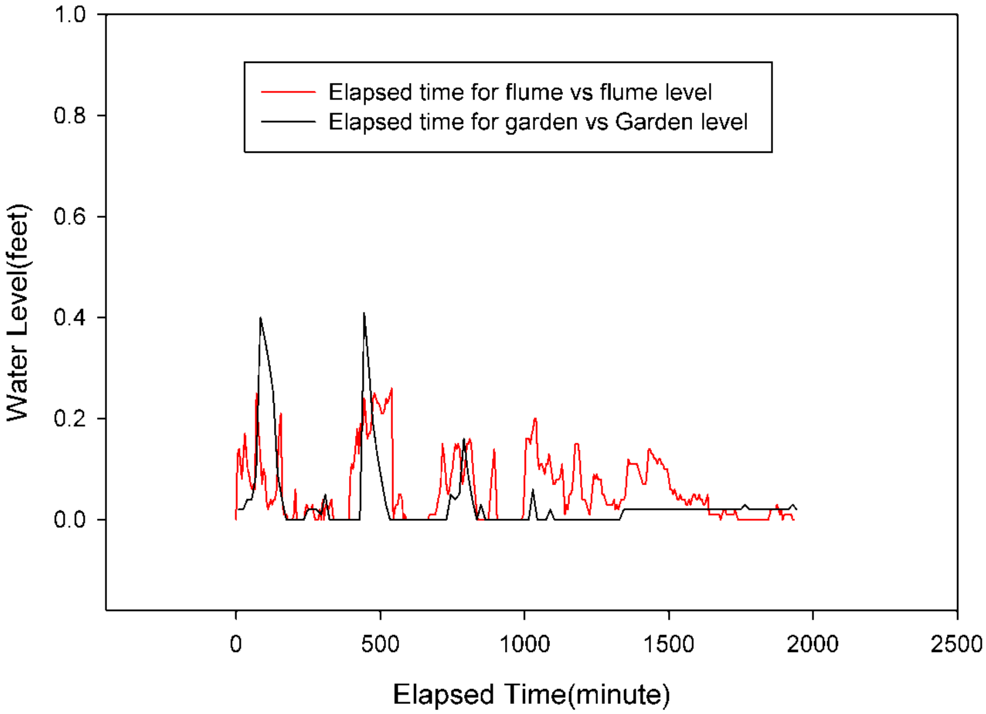
1612 76th on Rainevent 04/26/13-04/27/13



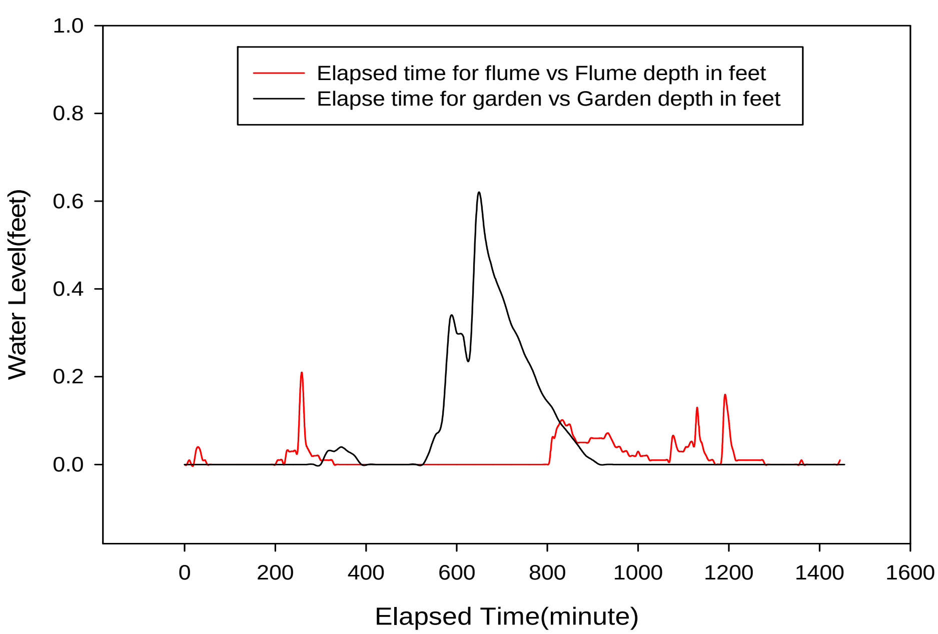
1612 76th on Rainevent 04/22/13-04/2313

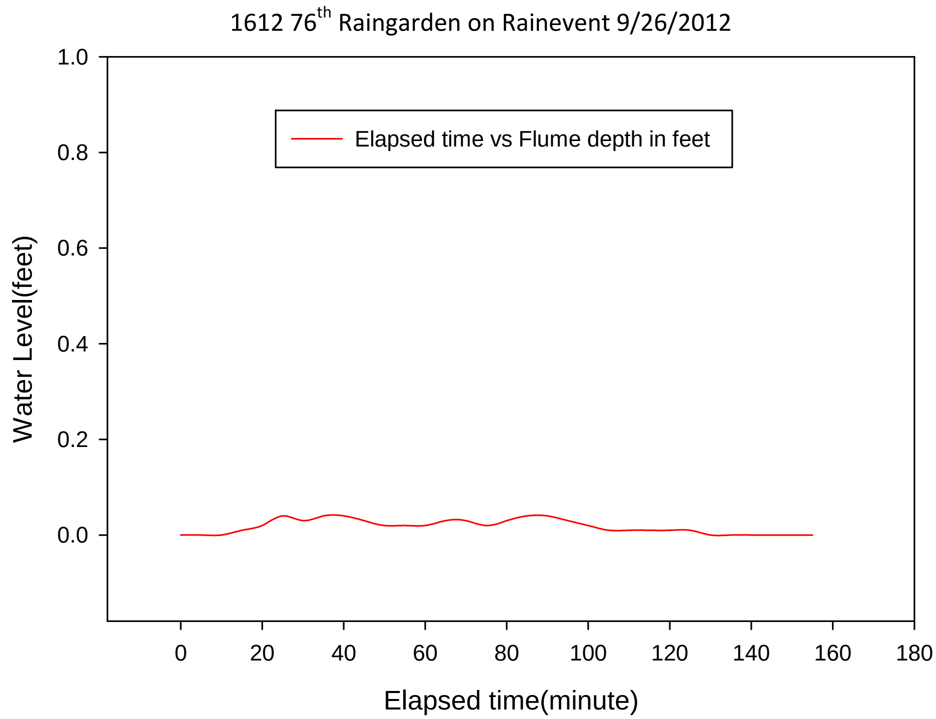


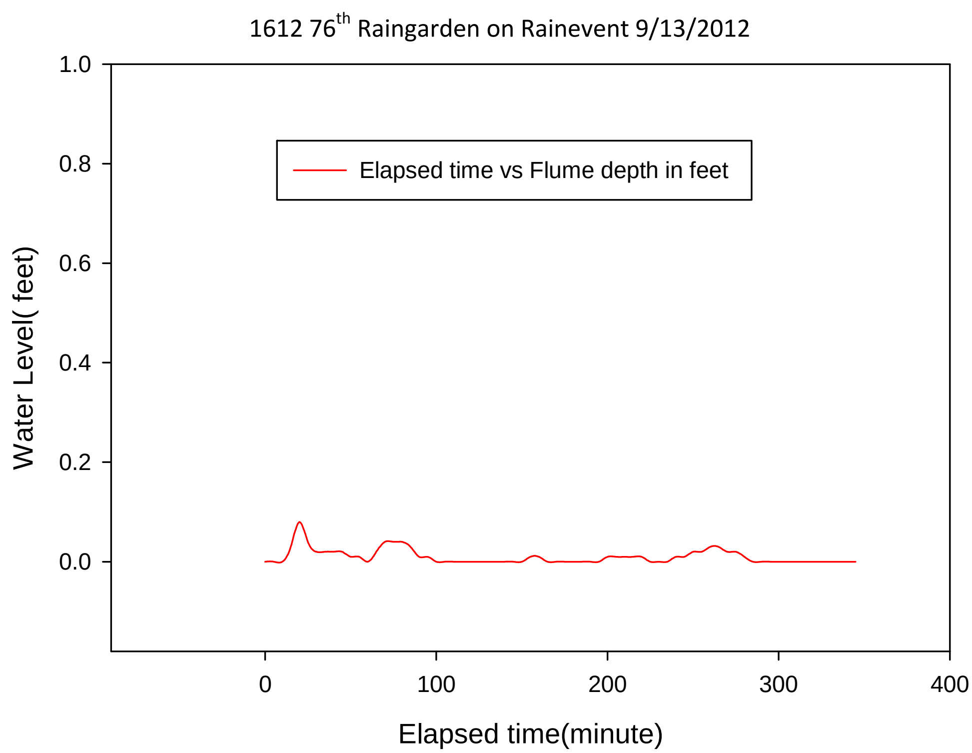
1612 76th on Rainevent 04/09/13--04/10/13



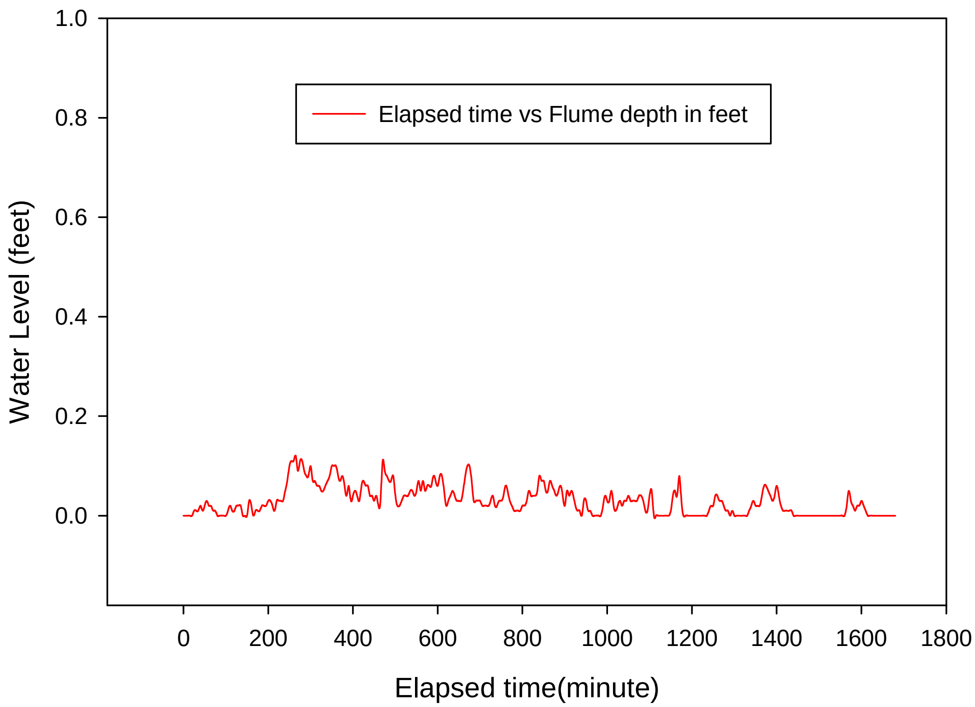
1612 76th Raingarden on Rainevent 10/12/2012



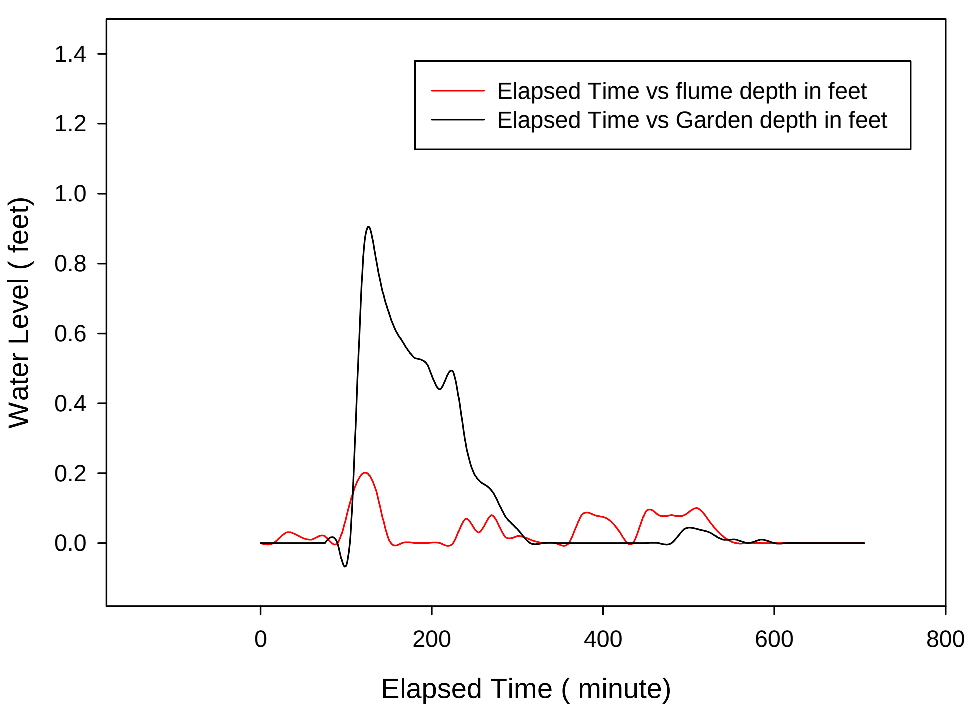

1612 76th Raingarden on Rainevent 9/26/2012
Elapsed time vs Flume depth in feet
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
20
40
60
80
100
120
140
160
180
Elapsed time(minute)

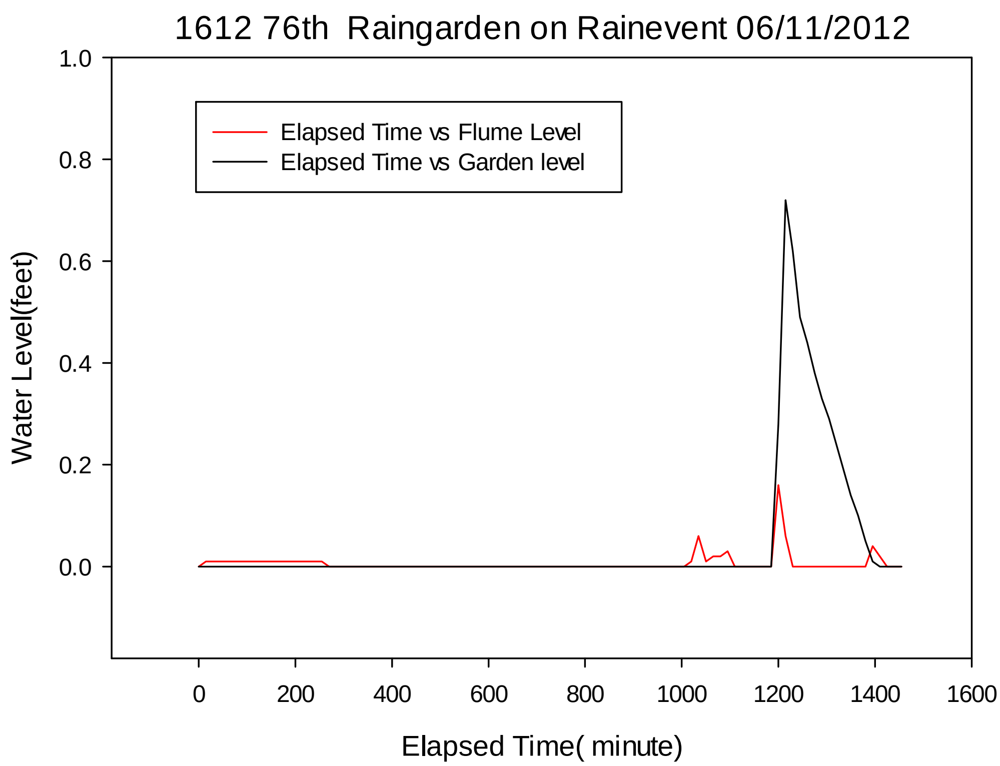

1612 76th Raingarden on Rainevent 9/13/2012
Elapsed time vs Flume depth in feet
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
100
200
300
400
Elapsed time(minute)

1612 76th Raingarden on Rainevent 9/1/2012



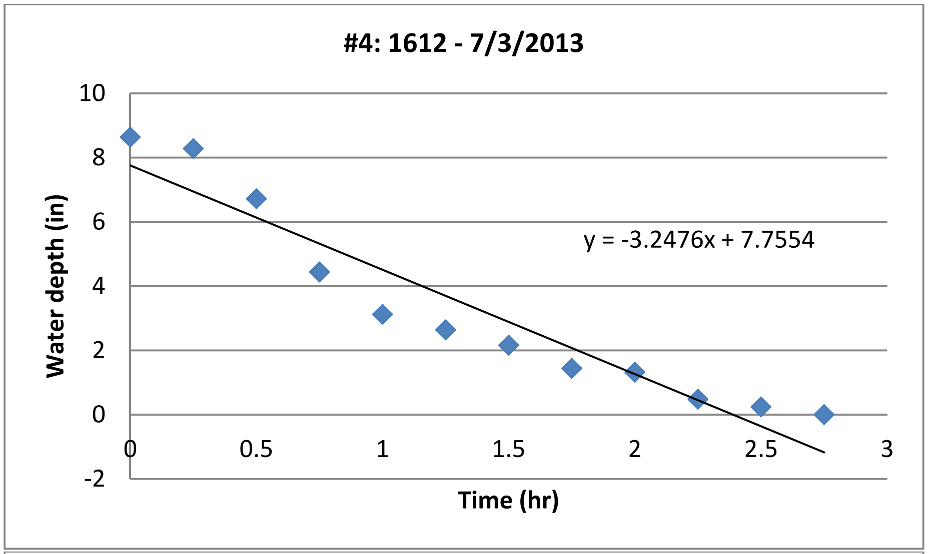
1612 76th Raingarden on Rainevent 6/21/2012

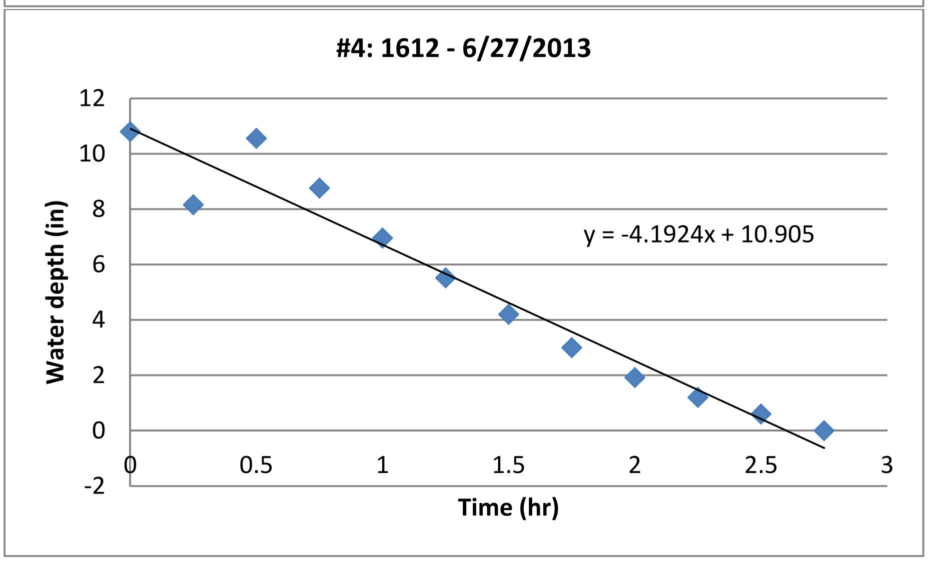


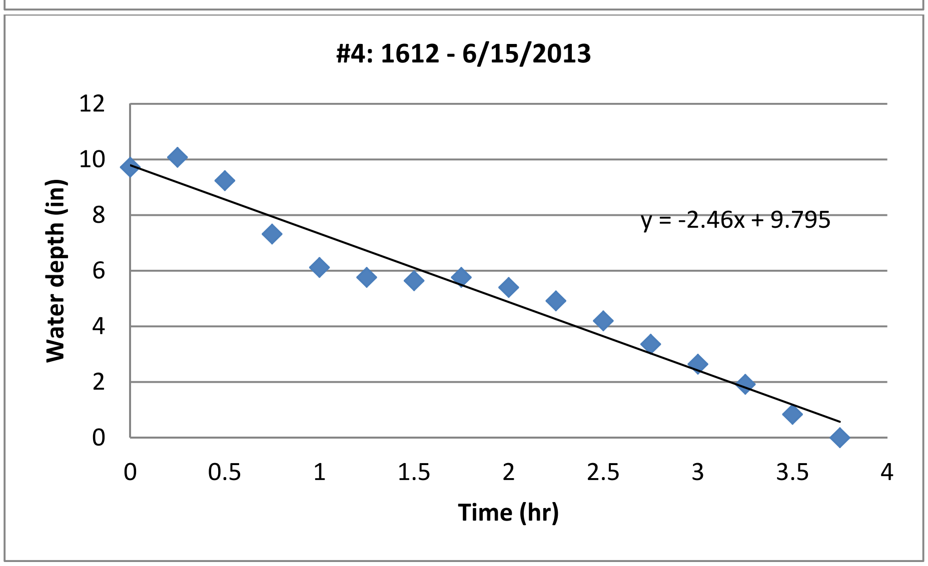


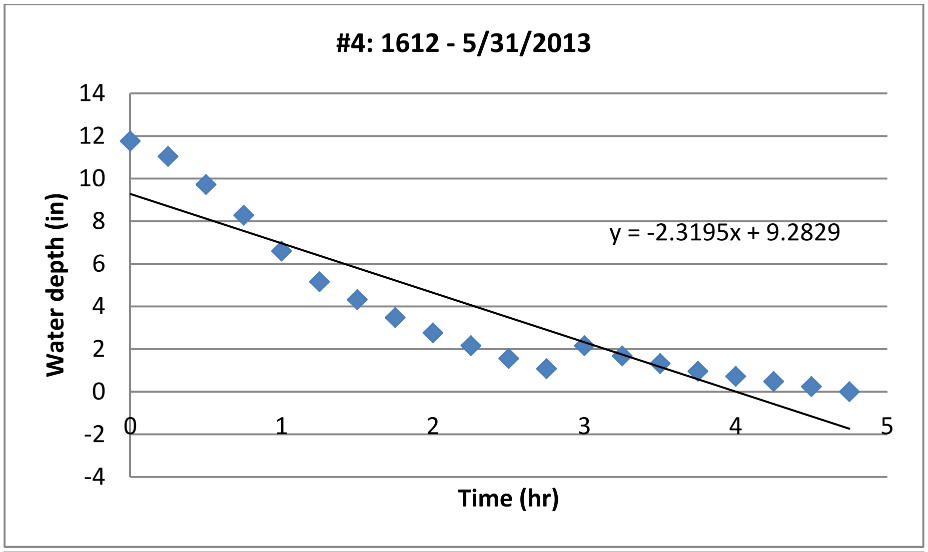
1612 76th Raingarden on Rainevent 06/11/2012
Elapsed Time vs Flume Level
Elapsed Time vs Garden level
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
200
400
600
800
1000
1200
1400
1600
Elapsed Time(minute)

Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
1.5	7/3/2013 17:54:00	7/3/2013 21:36:00	3:42	8.64	0:03	3.25
0.83	6/27/2013 11:46:00	6/28/2013 0:49:00	13:03	10.8	11:26	4.19
1.26	6/15/2013 15:50:00	6/15/2013 22:05:00	6:15	10.08	0:06	2.46
1.34	5/31/2013 5:26:00	5/31/2013 9:21:00	3:55	11.76	0:30	2.32
1.62	5/29/2013 22:53:00	5/30/2013 15:17:00	1.62	9	12:18	1.35
1.62	5/29/2013 22:53:00	5/30/2013 15:17:00	1.62	5.88	3:48	1.49
2.01	5/27/2013 8:31:00	5/27/2013 12:35:00	4:04	10.92	1:40	3.14
0.83	5/19/2013 2:21:00	5/20/2013 1:44:00	23:23	7.32	1:18	4.02
1.42	5/2/2013 3:08:00	5/4/2013 4:11:00	49:03	1.44	2:01	1.18
0.95	4/26/2013 4:19:00	4/27/2013 11:02:00	30:43	2.76	24:20	1.13
0.39	4/23/2013 1:34:00	4/23/2013 10:53:00	9:19	2.88	4:50	0.41
1.10	4/17/2013 11:00:00	4/18/2013 20:41:00	33:41	3.84	0:58	1.77
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	4.68	12:43	1.08
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	7.68	6:58	2.59
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	6.84	1:13	3.18
0.86	10/12/2012 21:00:54	10/13/2012 21:05:54	24:05	7.32	9:00	1.54
0.23	9/26/2012 02:55:54	9/26/2012 05:30:54	02:35	0		
0.43	9/13/2012 14:40:54	9/13/2012 20:25:54	05:45	0		
5.60	8/31/2012 11:00:54	9/1/2012 15:00:54	28:00	0		
1.03	6/21/2012 00:12:33	6/21/2012 12:02:33	11:50	9.84	1:45	>3.14
0.8	6/10/2012 09:45:52	6/11/2012 10:00:52	24:15	1.92	20:00	2.54

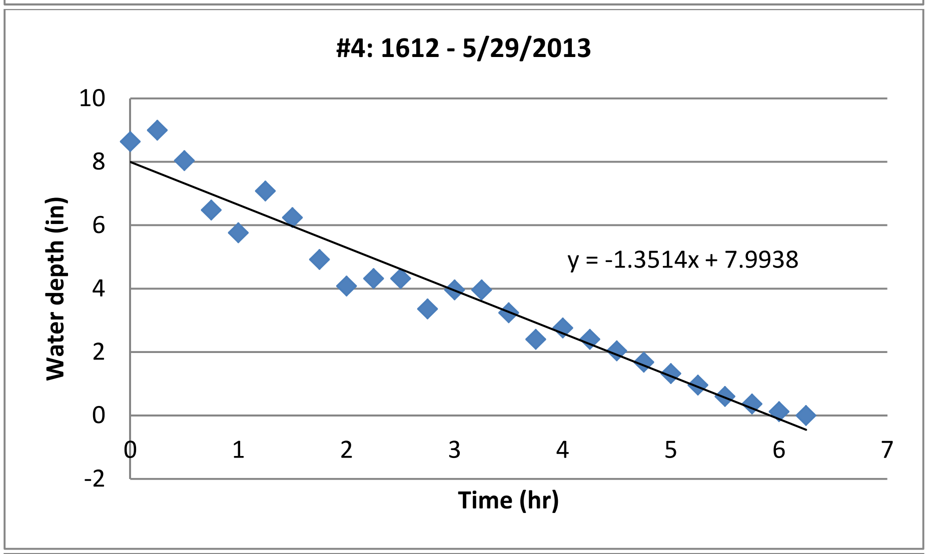

#4: 1612 - 7/3/2013
Water depth (in)
Time (hr)
y = -3.2476x + 7.7554
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5
3


#4: 1612 - 6/27/2013
Water depth (in)
Time (hr)
y = -4.1924x + 10.905
12
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5
3

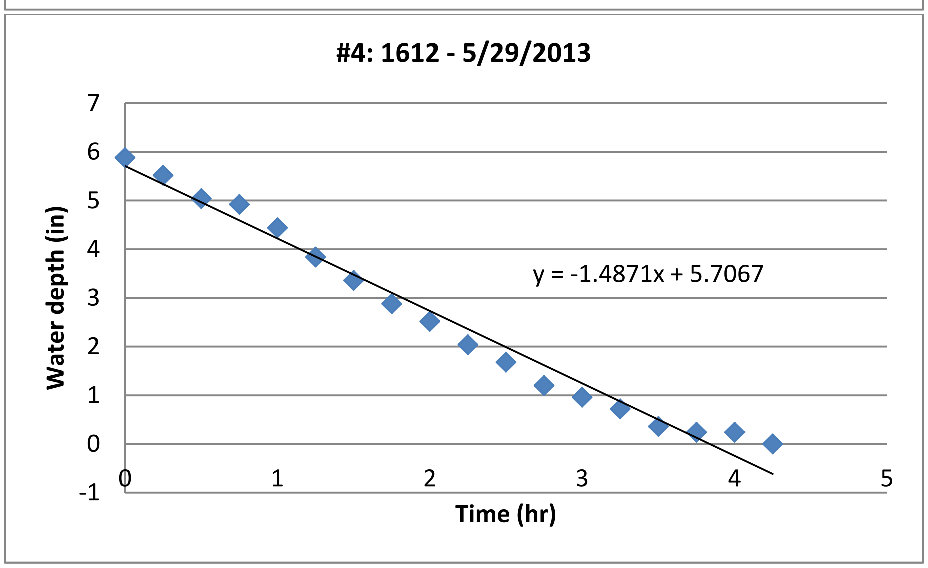

#4: 1612 - 6/15/2013
Water depth (in)
Time (hr)
y = -2.46x + 9.795
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5
3
3.5
4



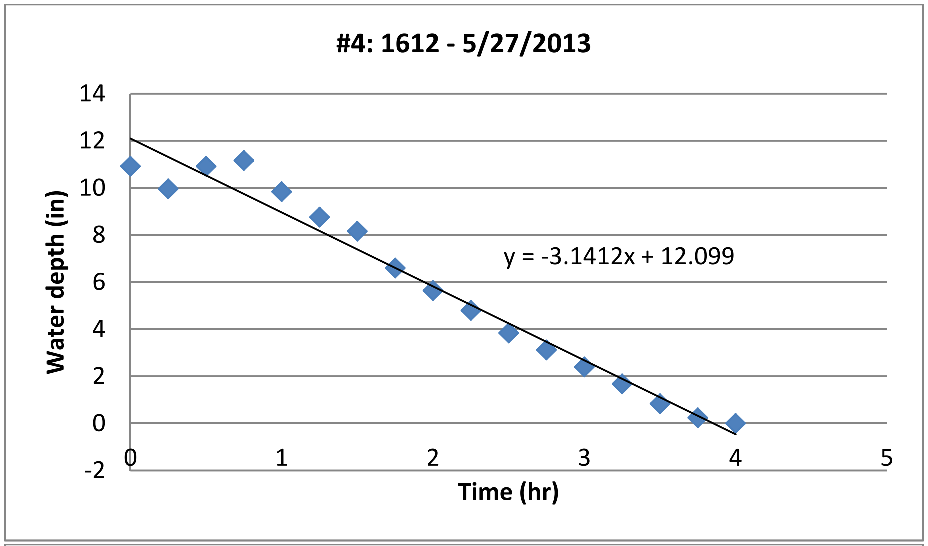
#4: 1612 - 5/31/2013
Water depth (in)
14
12
10
8
6
4
2
0
-2
-4
0
1
2
3
4
5
y = -2.3195x + 9.2829
Time (hr)

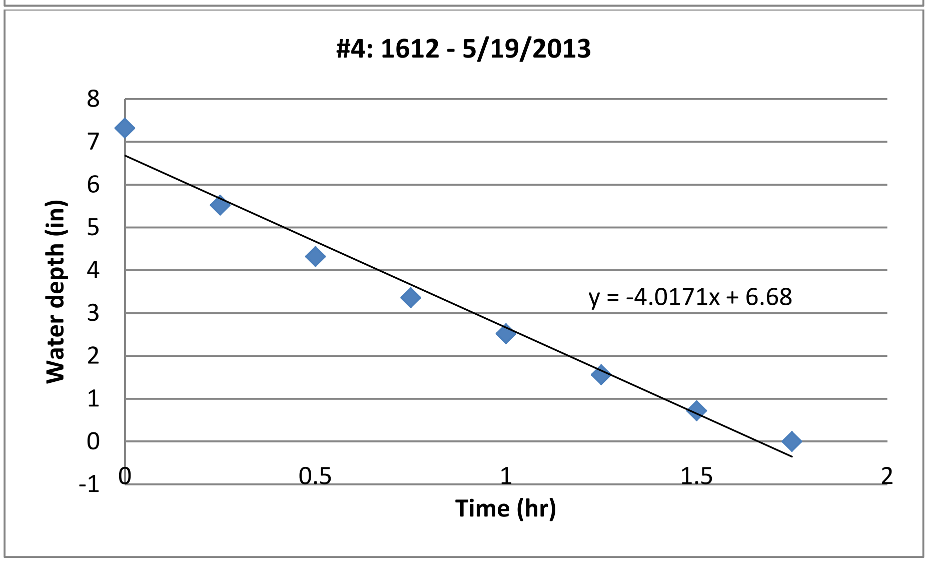


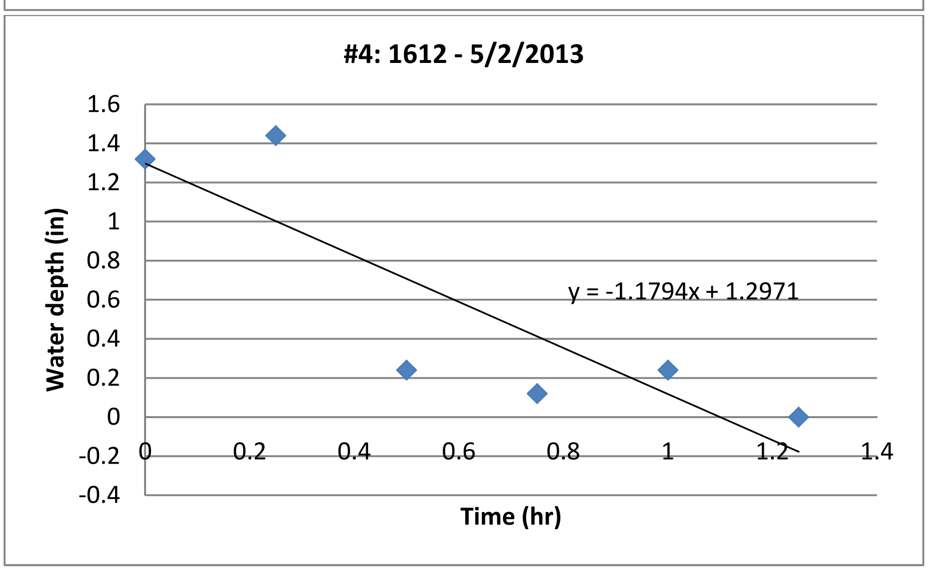
#4: 1612 - 5/29/2013
Water depth (in)
10
8
6
4
2
0
-2
0
1
2
3
4
5
6
7
y = -1.3514x + 7.9938
Time (hr)

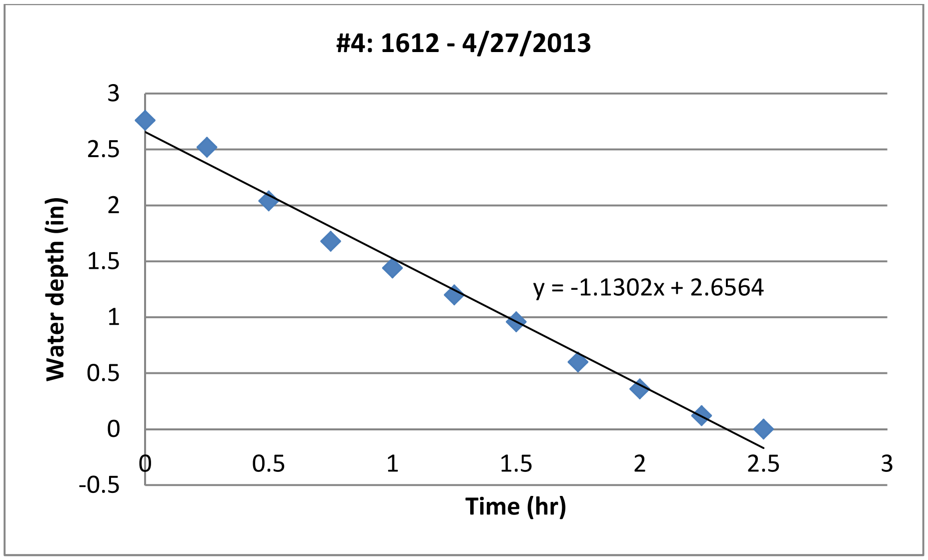


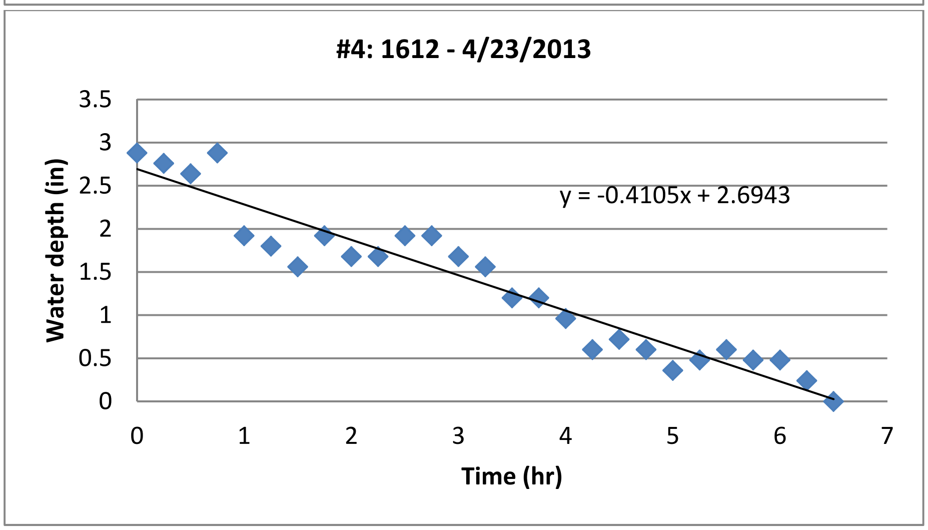
#4: 1612 - 5/29/2013
Water depth (in)
7
6
5
4
3
2
1
0
-1
0
1
2
3
4
5
y = -1.4871x + 5.7067
Time (hr)

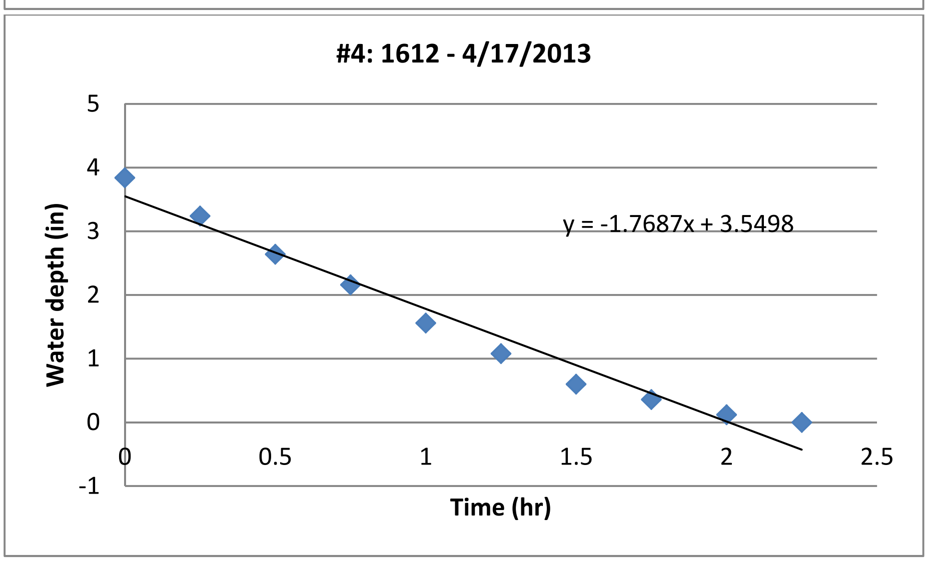

#4: 1612 - 5/27/2013
Water depth (in)
Time (hr)
y = -3.1412x + 12.099
14
12
10
8
6
4
2
0
-2
0
1
2
3
4
5

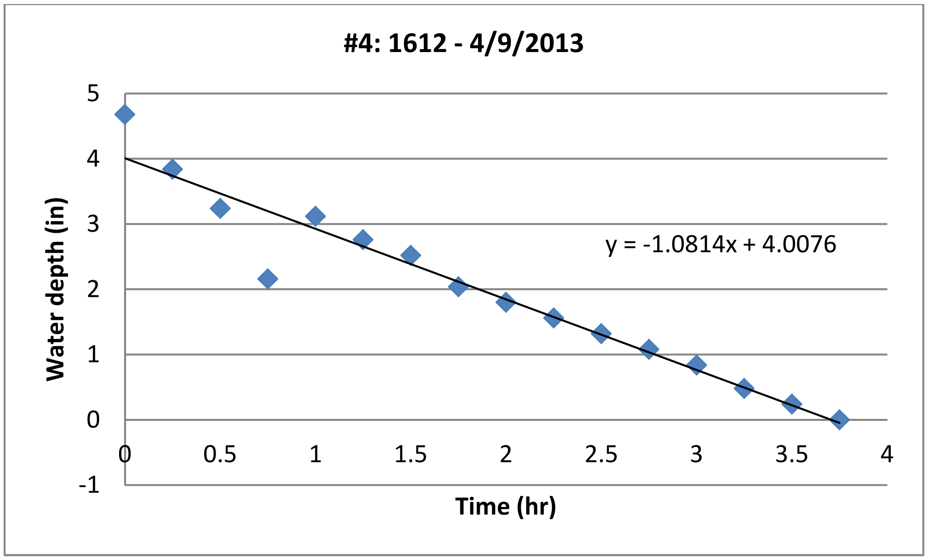

#4: 1612 - 5/19/2013
Water depth (in)
Time (hr)
y = -4.0171x + 6.68
8
7
6
5
4
3
2
1
0
-1
0
0.5
1
1.5
2

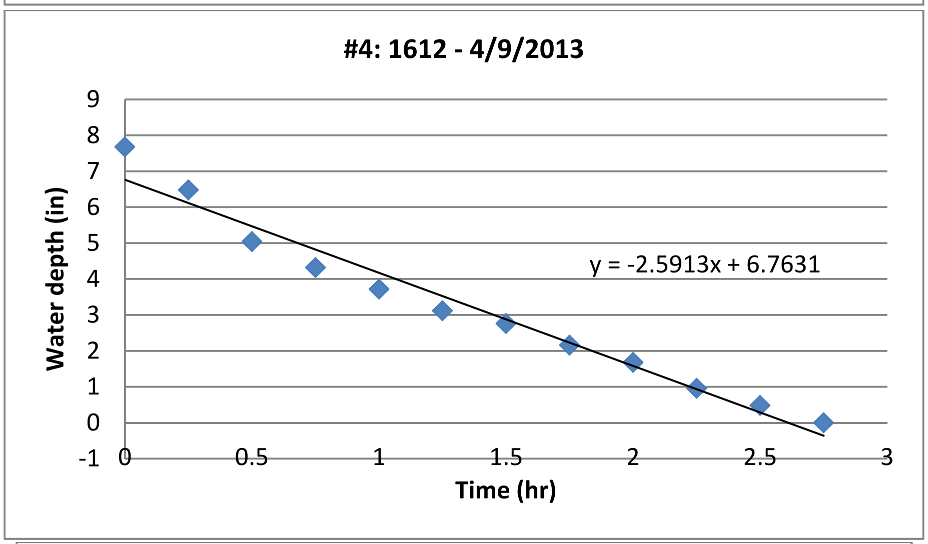

#4: 1612 - 5/2/2013
Water depth (in)
Time (hr)
y = -1.1794x + 1.2971
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
0
0.2
0.4
0.6
0.8
1
1.2
1.4

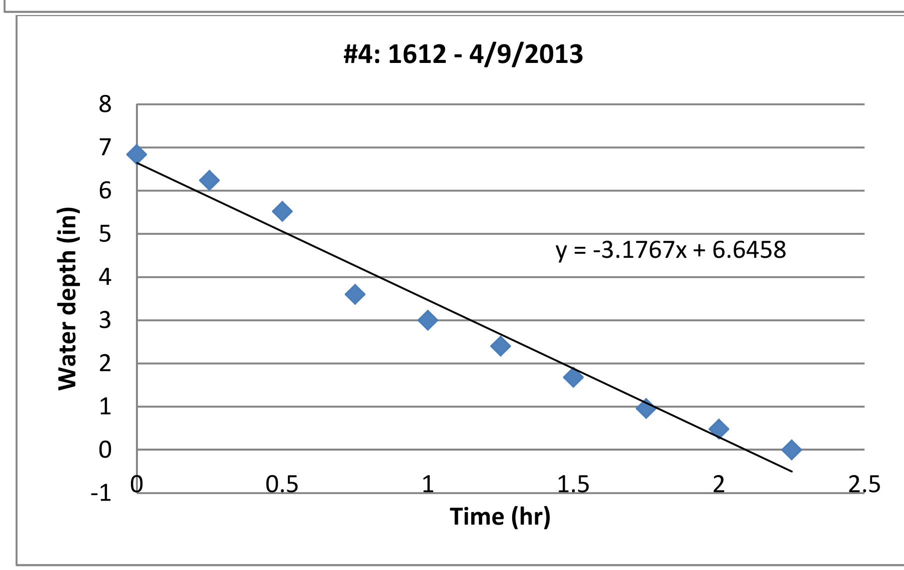

#4: 1612 - 4/27/2013
Water depth (in)
Time (hr)
y = -1.1302x + 2.6564
3
2.5
2
1.5
1
0.5
0
-0.5
0
0.5
1
1.5
2
2.5
3

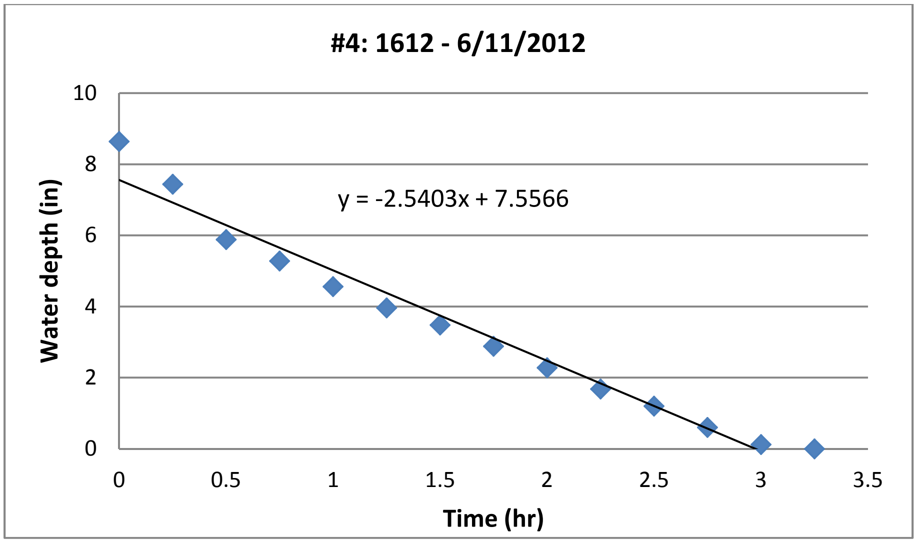

#4: 1612 - 4/23/2013
Water depth (in)
Time (hr)
y = -0.4105x + 2.6943
3.5
3
2.5
2
1.5
1
0.5
0
0
1
2
3
4
5
6
7


#4: 1612 - 4/17/2013
Water depth (in)
Time (hr)
y = -1.7687x + 3.5498
5
4
3
2
1
0
-1
0
0.5
1
1.5
2
2.5


#4: 1612 - 4/9/2013
Water depth (in)
5
4
3
2
1
0
-1
0
0.5
1
1.5
2
2.5
3
3.5
4
y = -1.0814x + 4.0076
Time (hr)

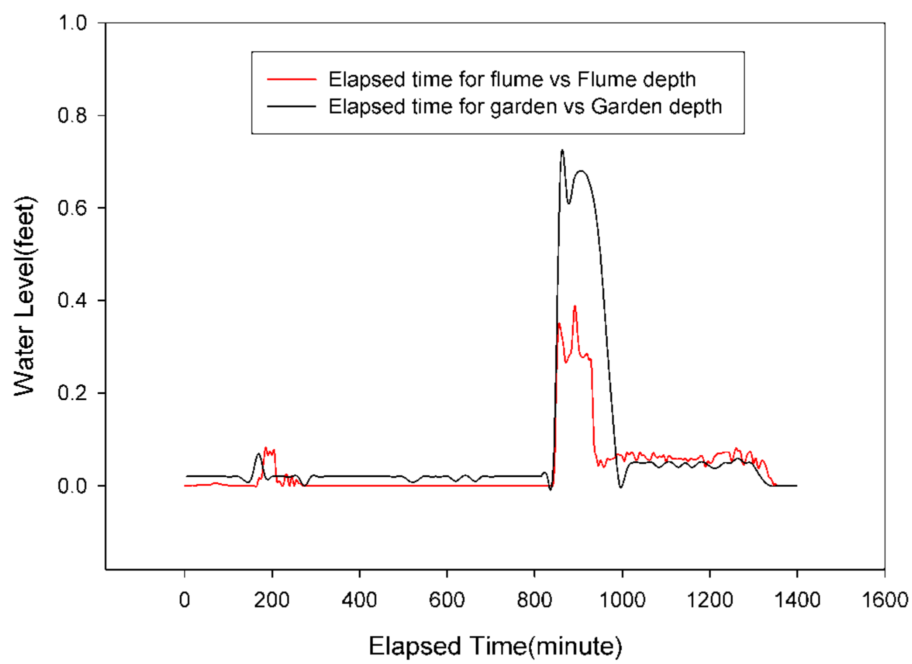

#4: 1612 - 4/9/2013
Water depth (in)
9
8
7
6
5
4
3
2
1
0
-1
0
0.5
1
1.5
2
2.5
3
y = -2.5913x + 6.7631
Time (hr)


#4: 1612 - 4/9/2013
Water depth (in)
8
7
6
5
4
3
2
1
0
-1
0
0.5
1
1.5
2
2.5
y = -3.1767x + 6.6458
Time (hr)

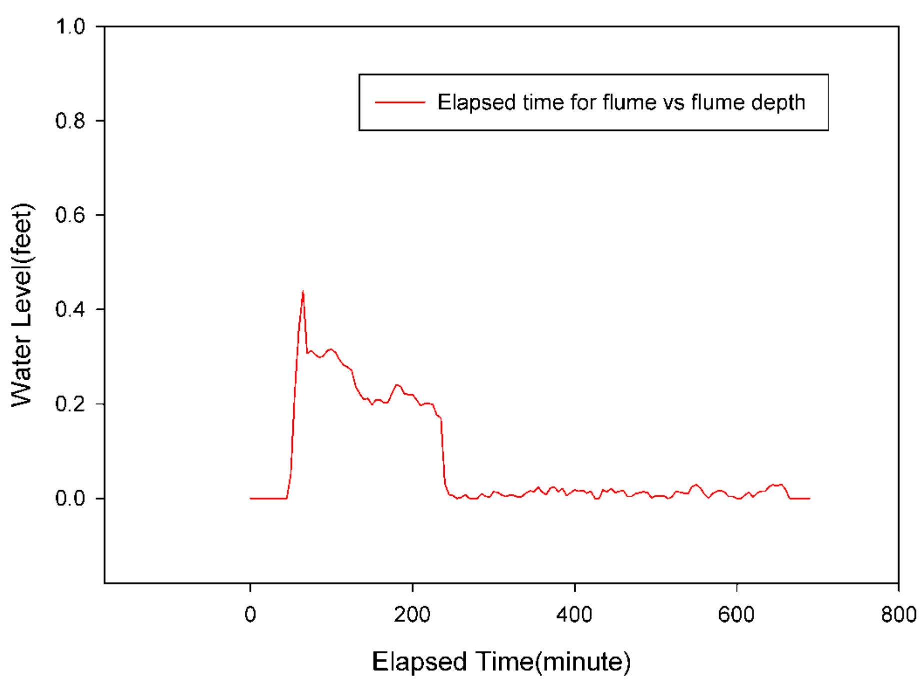

#4: 1612 - 6/11/2012
Water depth (in)
y = -2.5403x + 7.5566
Time (hr)
0
2
4
6
8
10
0
0.5
1
1.5
2
2.5
3
3.5

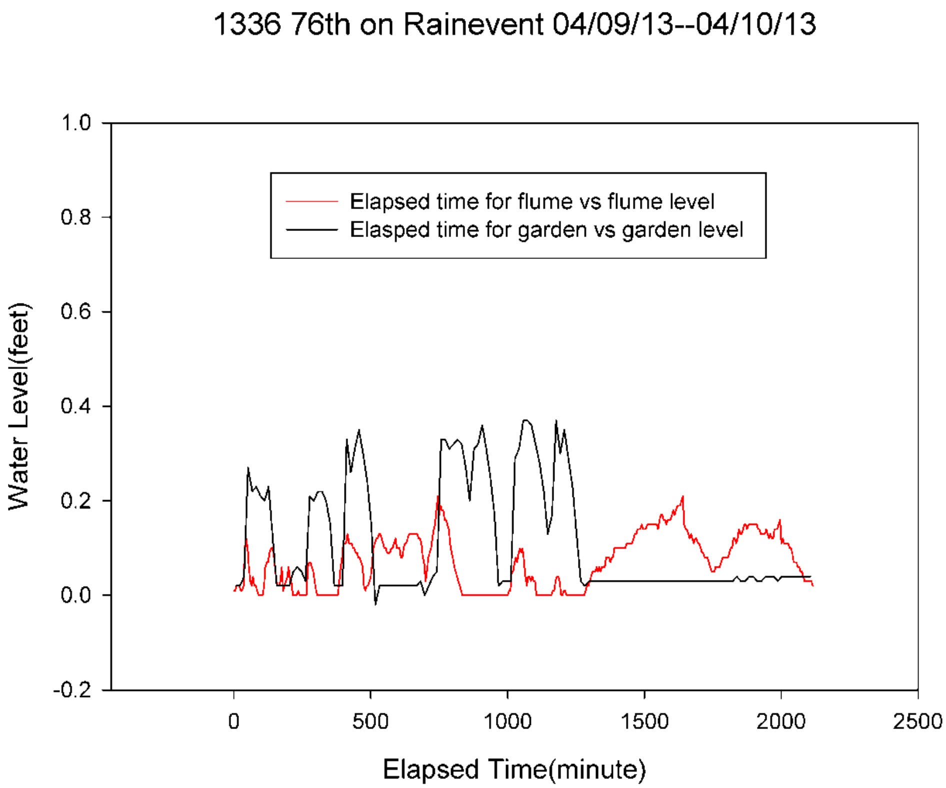
5- Curb-Cut Biofilter - 1336 E 76th St.

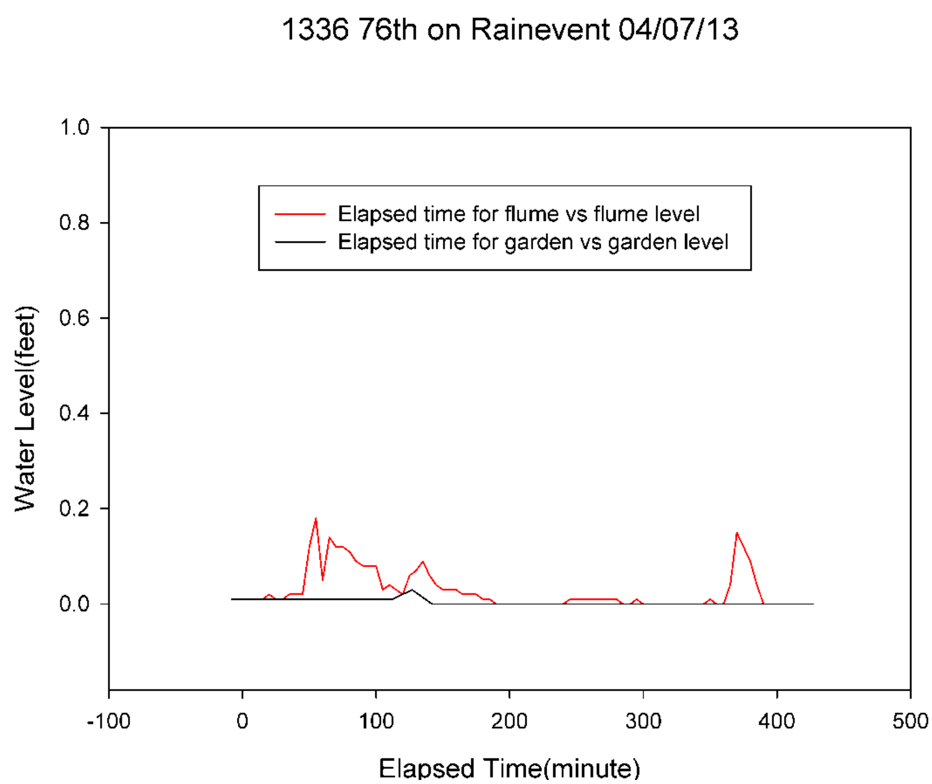
1336 76th on Rainevent 6/27/13



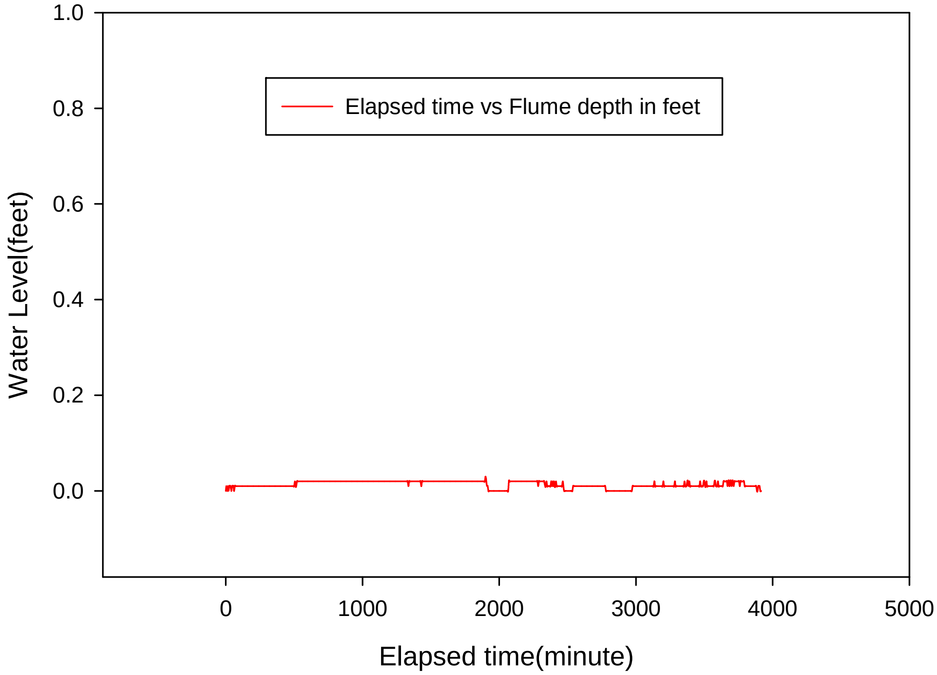
1336 76th on Rainevent 6/15/13



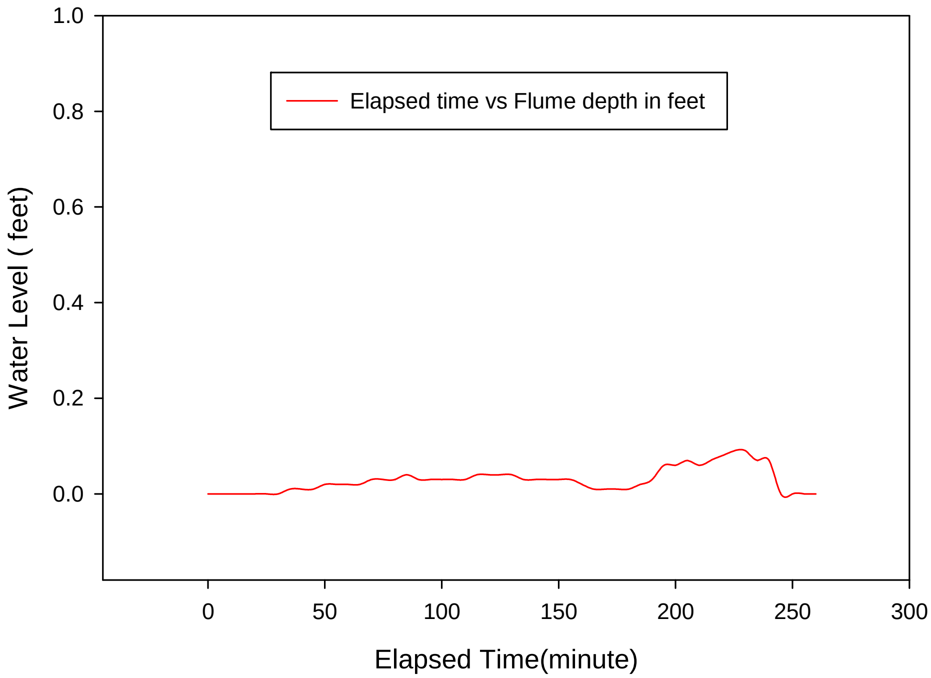

1336 76th on Rainevent 04/09/13--04/10/13
Elapsed time for flume vs flume level
Elasped time for garden vs garden level
Water Level(feet)
Elapsed Time(minute)
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
0
500
1000
1500
2000
2500


1336 76th on Rainevent 04/07/13
Elapsed time for flume vs flume level
Elapsed time for garden vs garden level
Water Level(feet)
Elapsed Time(minute)
1.0
0.8
0.6
0.4
0.2
0.0
-100
0
100
200
300
400
500

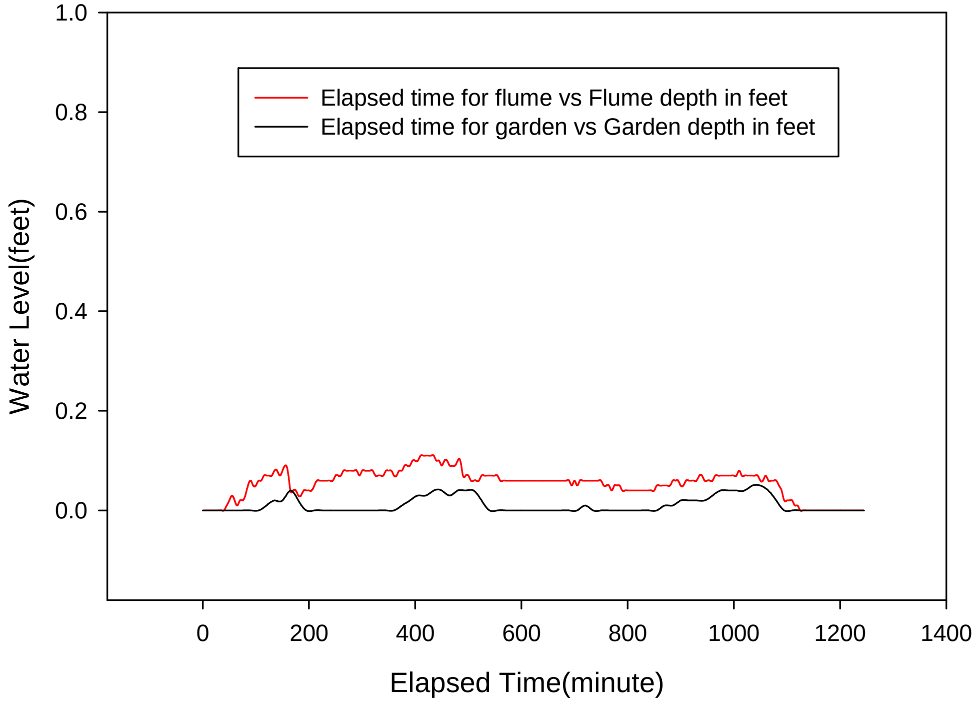
1336 76th Raingarden on Rainevent 10/13/2012



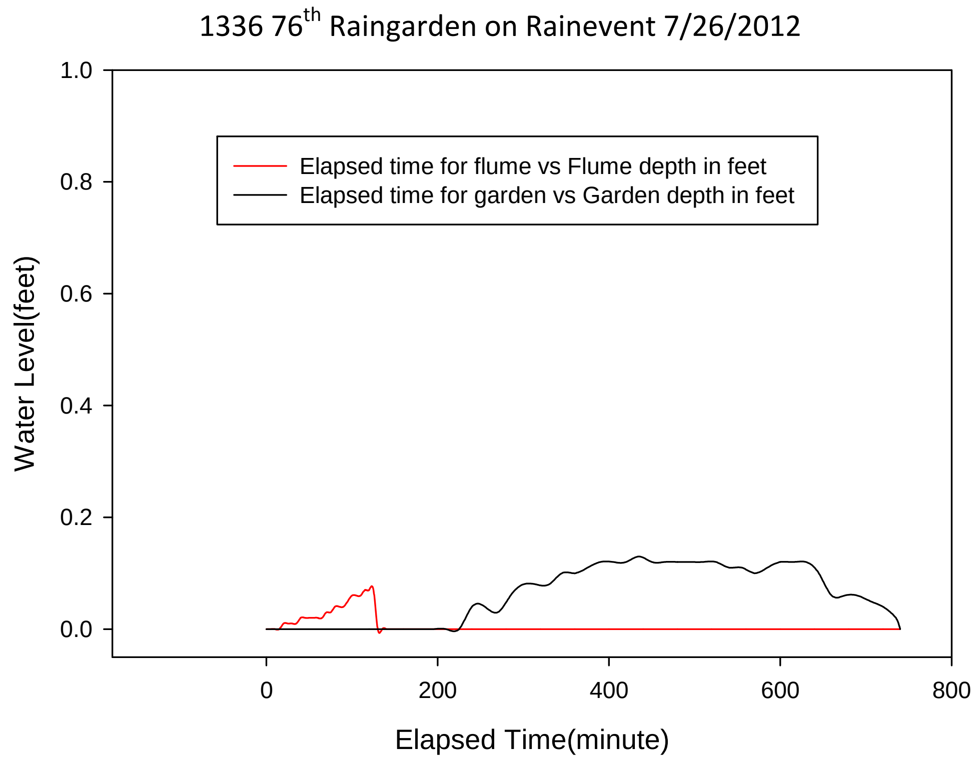
1336 76th Raingarden on Rainevent 9/26/2012

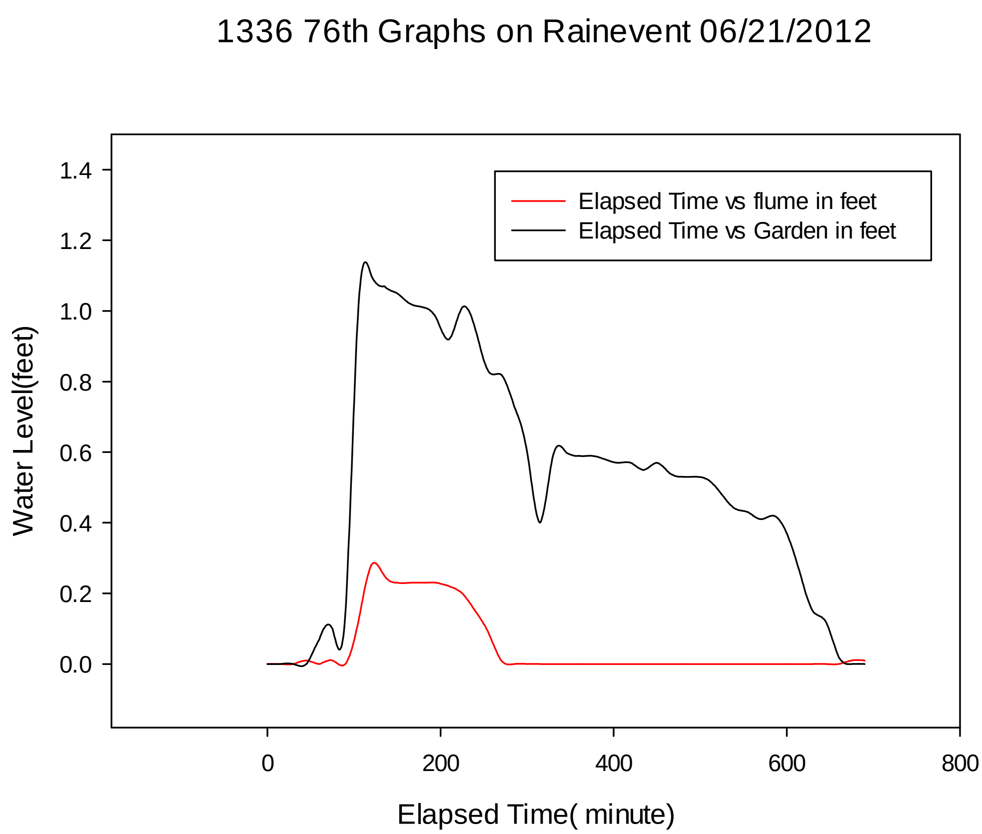


1336 76th Raingarden on Rainevent 9/13/2012

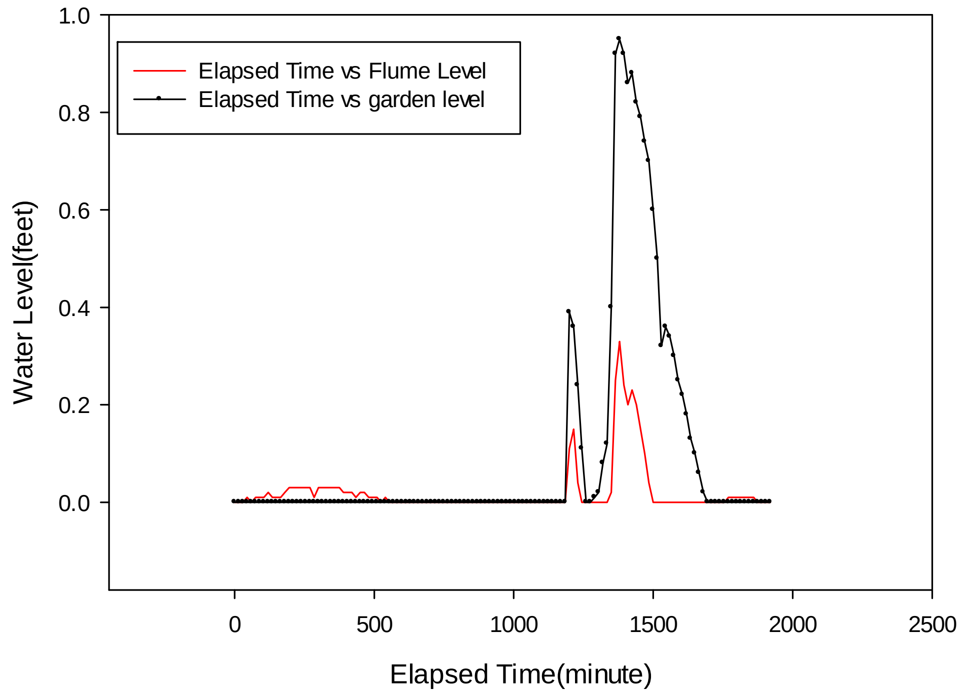


1336 76th Raingarden on Rainevent 9/1/2012


1336 76th Raingarden on Rainevent 7/26/2012
Elapsed time for flume vs Flume depth in feet
Elapsed time for garden vs Garden depth in feet
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
200
400
600
800
Elapsed Time(minute)

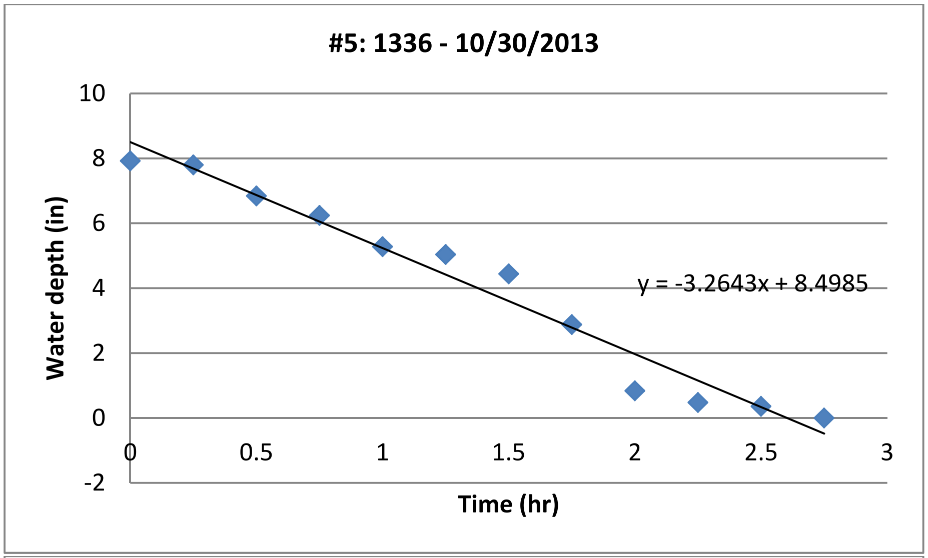

1336 76th Graphs on Rainevent 06/21/2012
Elapsed Time vs flume in feet
Elapsed Time vs Garden in feet
Water Level(feet)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
200
400
600
800
Elapsed Time(minute)

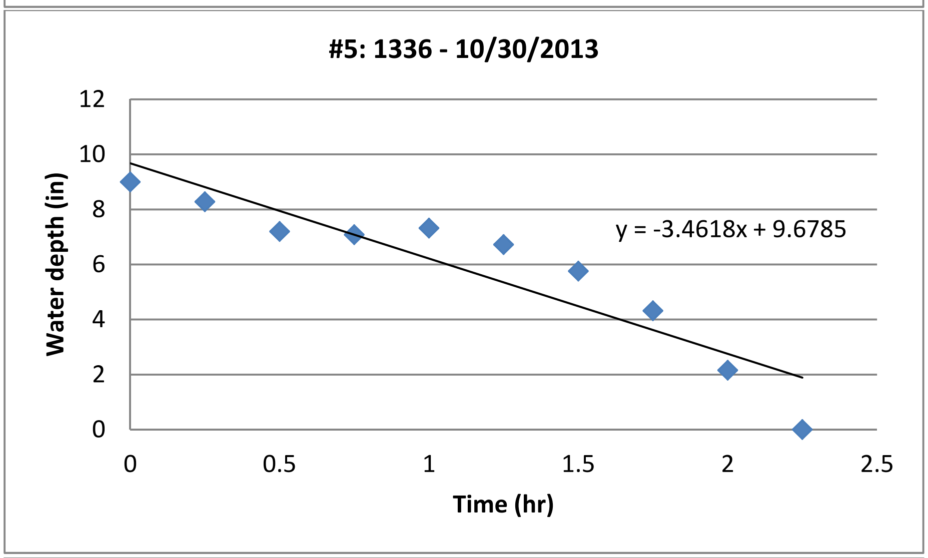
1336 76th Raingarden on Rainevent 06/11/2012

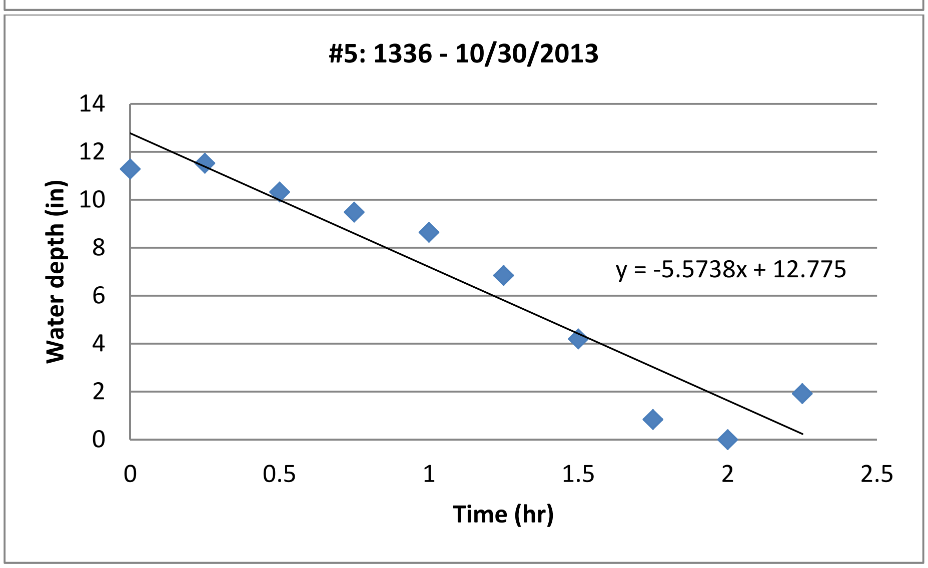


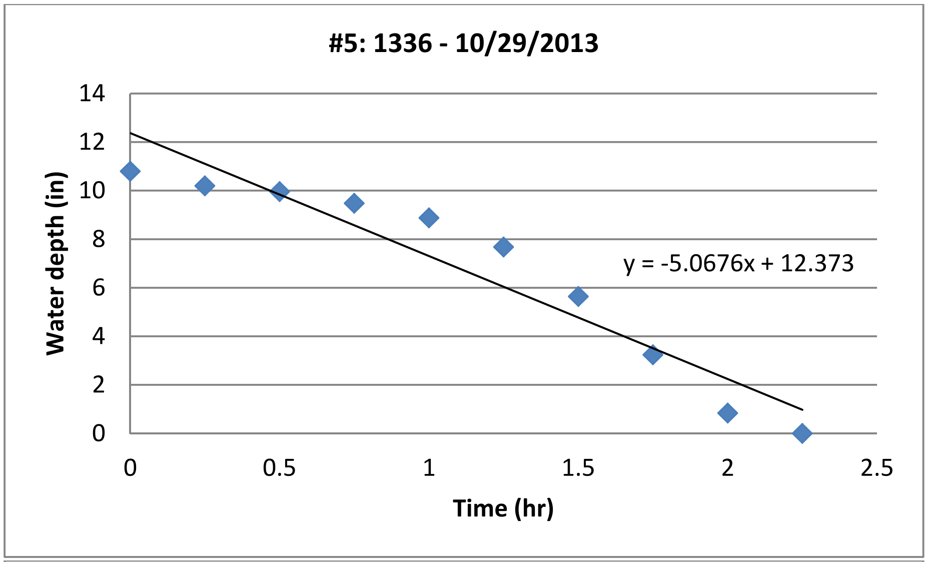
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:37	7.92	18:44	3.26
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:37	9	3:44	3.46
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:37	11.52	0:29	5.57
0.83	10/29/2013 2:52:00	10/29/2013 9:41:00	6:49	10.8	3:45	5.07
0.12	10/18/2013 14:03:00	10/18/2013 17:05:00	3:02	2.52	0:04	3.22
0.87	10/4/2013 22:36:00	10/5/2013 3:48:00	5:12	8.4	0:30	4.54
0.16	10/3/2013 11:04:00	10/3/2013 11:24:00	0:20	3.6	0:02	2.01
0.28	9/28/2013 8:29:00	9/28/2013 11:29	3:00	6.24	0:51	2.58
1.89	9/19/2013 19:00:00	9/19/2013 22:23:00	3:23	10.68	2:49	5.34
0.79	9/17/2013 7:04:00	9/17/2013 15:14:00	8:10	9.36	3:14	4.26
0.16	9/1/2013 07:42:00	9/1/2013 09:02:00	1:20	3.6	0:05	1.1

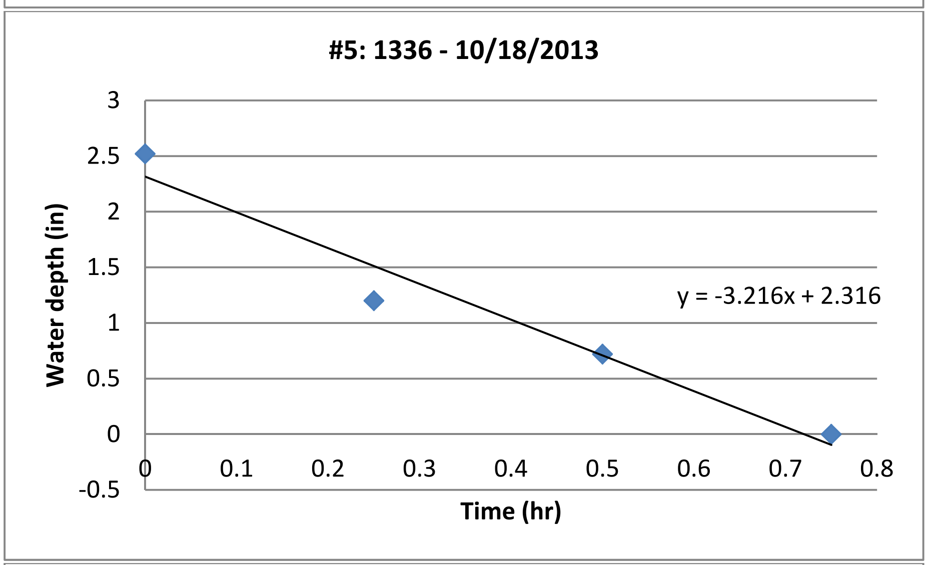
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
0.67	8/12/2013 7:07:00	8/12/2013 10:18:00	3:11	10.8	0:24	3.5
0.87	8/7/2013 4:13:00	8/7/2013 8:45:00	4:32	8.52	2:48	4.53
0.55	8/6/2013 3:24:00	8/6/2013 4:59:00	1:35	6.0	0:37	3.34
0.91	7/29/2013 6:27:00	7/30/2013 9:41:00	27:14	7.32	8:03	5.72
0.24	7/25/2013 17:00:00	7/26/2013 11:00:00	18:00	1.08	2:15	1.05
1.5	7/3/2013 17:54:00	7/3/2013 21:36:00	3:42	6.12	2:17	1.18
0.83	6/27/2013 11:46:00	6/28/2013 0:49:00	13:03	8.88	11:25	3.57
1.26	6/15/2013 15:50:00	6/15/2013 22:05:00	6:15	5.49	0:16	1.11
0.39	4/23/2013 1:34:00	4/23/2013 10:53:00	9:19	6.48	4:04	2.54
1.10	4/17/2013 11:00:00	4/18/2013 20:41:00	33:41	5.16	18:23	1.83
1.10	4/17/2013 11:00:00	4/18/2013 20:41:00	33:41	7.44	0	2.63
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	4.08	14:57	2.92
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	3	0:57	1.29
1.10	4/7/2013 19:52:00	4/8/2013 2:25:00	6:33	1.92	13:00	0.78
0.86	10/11/2012 17:32:07	10/14/2012 10:47:07	65:15	0		
0.23	9/26/2012 03:02:07	9/26/2012 07:22:07	04:20	0		
0.43	9/13/2012 14:37:17	9/14/2012 10:47:07	20:45	0.6	2:00	>0.96
					6:15	>0.96
					14:30	>0.82
5.60	8/31/2012 15:47:07	9/2/2012 11:02:07	43:15	3.6	9:30	>2.13
					30:15	0.82
0.49	7/26/2012 01:31:19	7/26/2012 12:16:19	10:45	1.6	4:00	0.62
1.03	6/21/2012 00:17:19	6/21/2012 12:02:19	11:45	13.2	1:00	1.19
0.8	6/10/2012 07:02:19	6/11/2012 15:02:19	32:00	11.4	20:00	>4.94
					21:30	>2.38

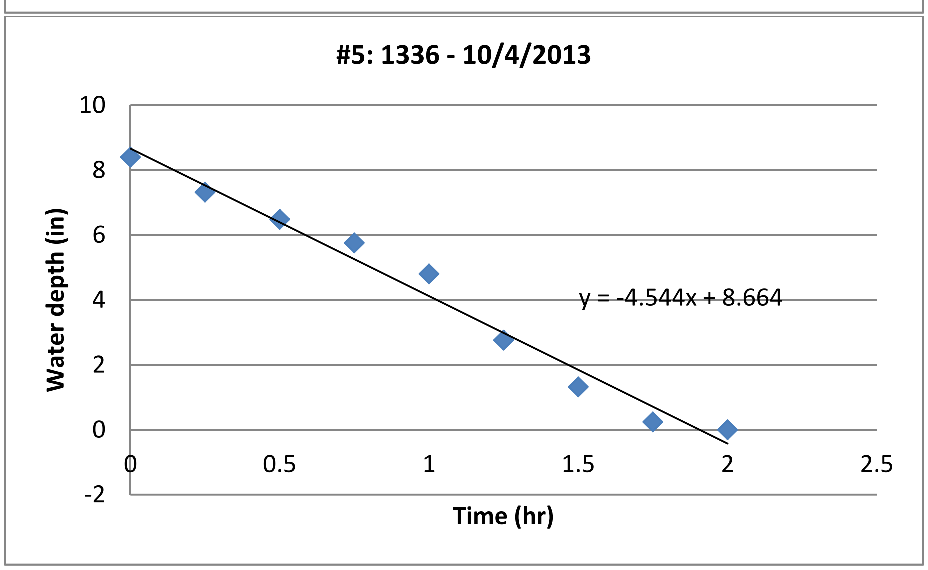

#5: 1336 - 10/30/2013
Water depth (in)
Time (hr)
y = -3.2643x + 8.4985
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5
3

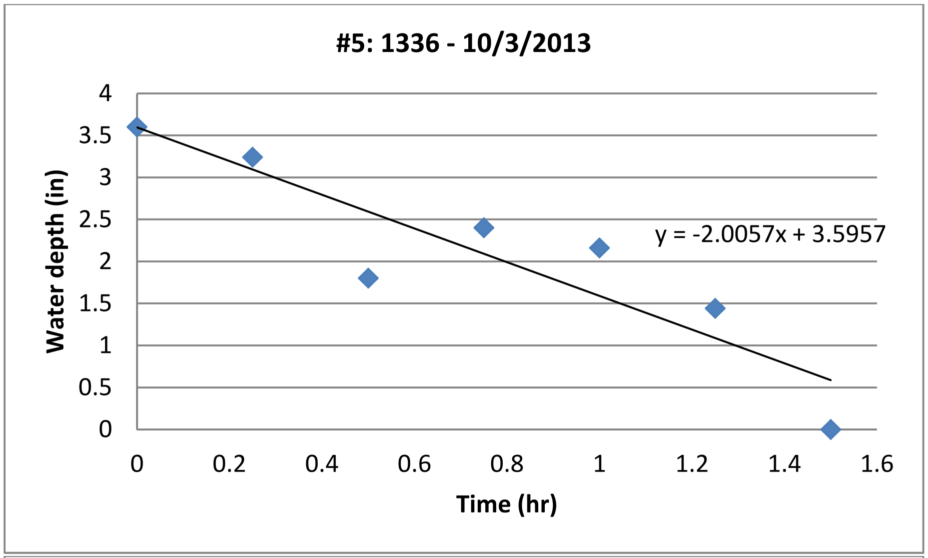

#5: 1336 - 10/30/2013
Water depth (in)
Time (hr)
y = -3.4618x + 9.6785
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5

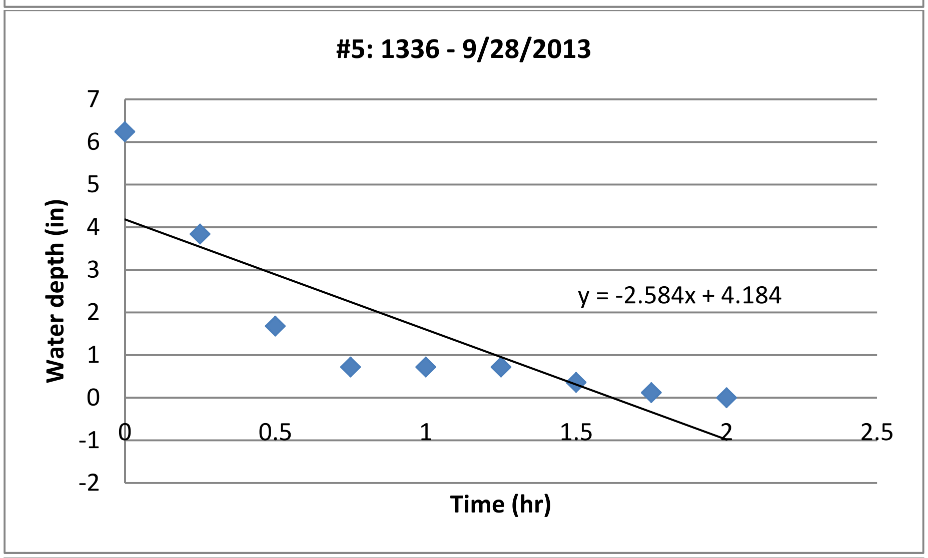

#5: 1336 - 10/30/2013
Water depth (in)
Time (hr)
y = -5.5738x + 12.775
14
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5

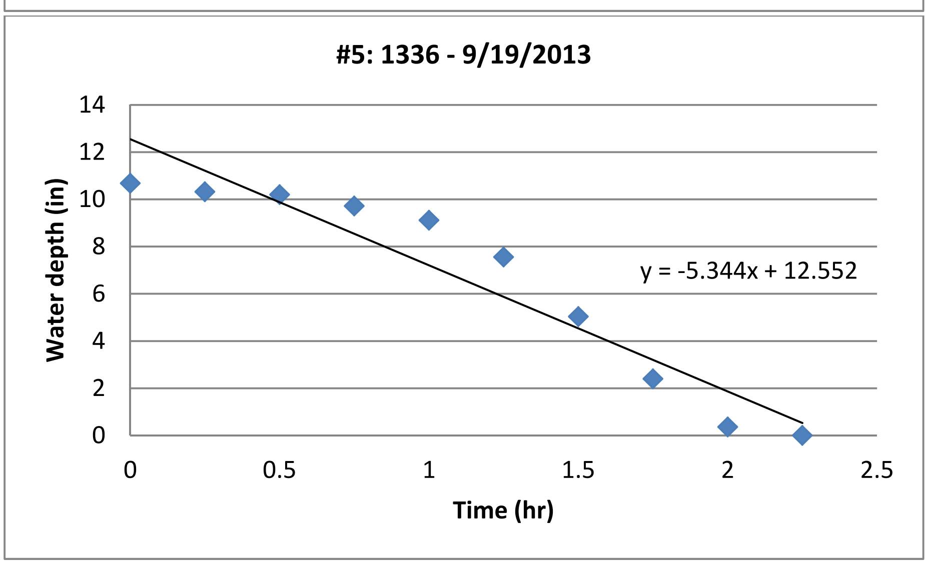

#5: 1336 - 10/29/2013
Water depth (in)
Time (hr)
y = -5.0676x + 12.373
14
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5

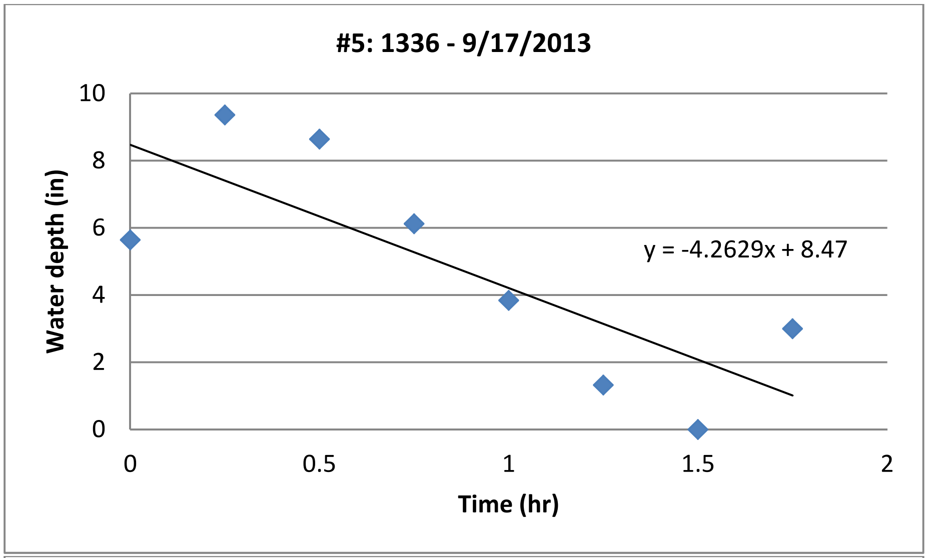

#5: 1336 - 10/18/2013
Water depth (in)
Time (hr)
y = -3.216x + 2.316
3
2.5
2
1.5
1
0.5
0
-0.5
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8

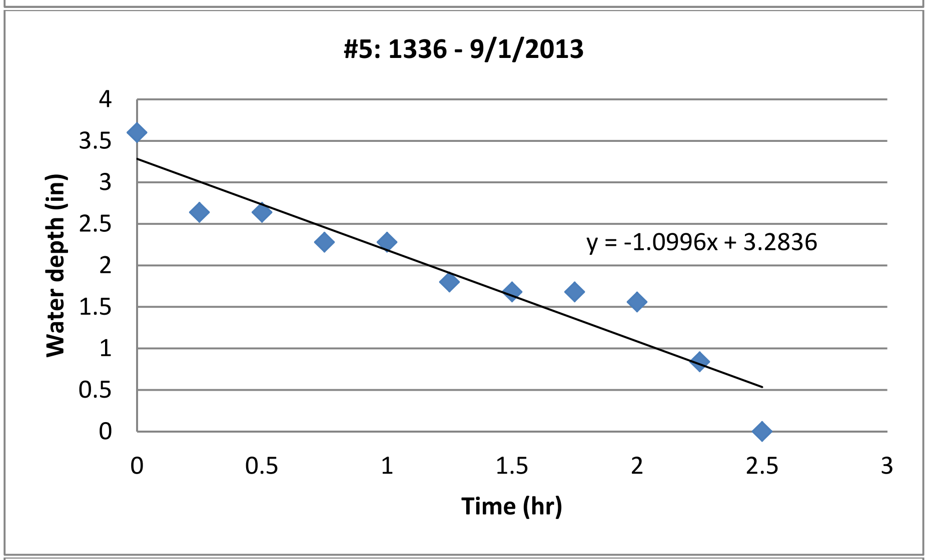

#5: 1336 - 10/4/2013
Water depth (in)
Time (hr)
y = -4.544x + 8.664
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5

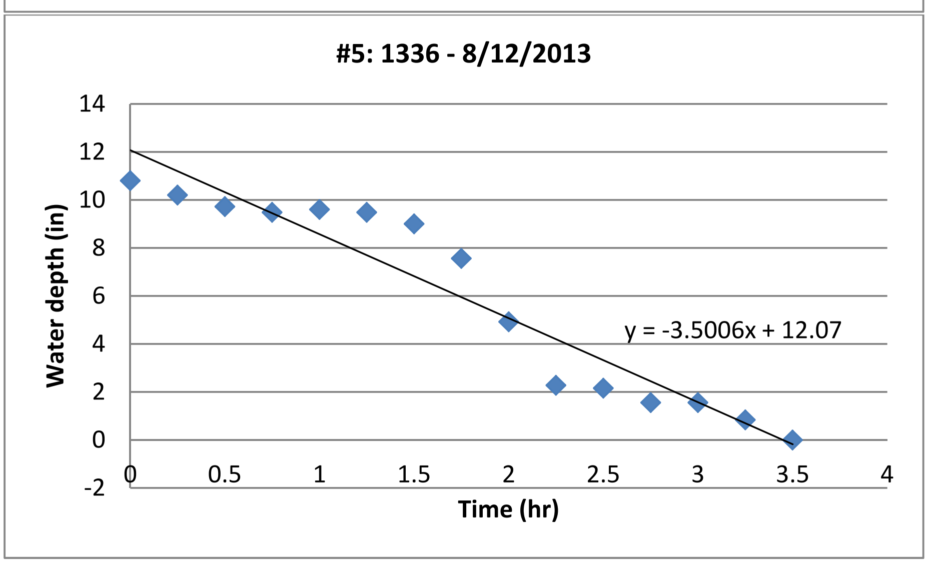

#5: 1336 - 10/3/2013
Water depth (in)
Time (hr)
y = -2.0057x + 3.5957
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6

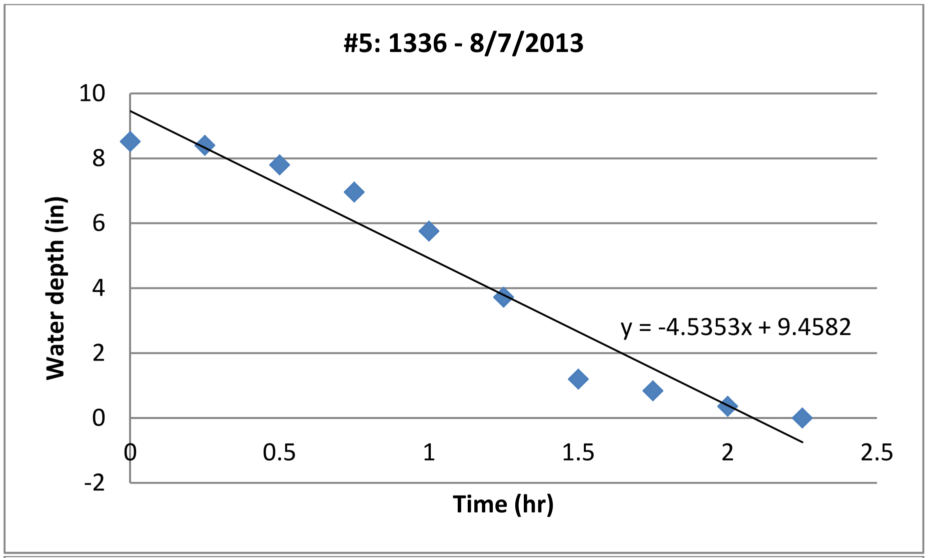

#5: 1336 - 9/28/2013
Water depth (in)
Time (hr)
y = -2.584x + 4.184
7
6
5
4
3
2
1
0
-1
-2
0
0.5
1
1.5
2
2.5

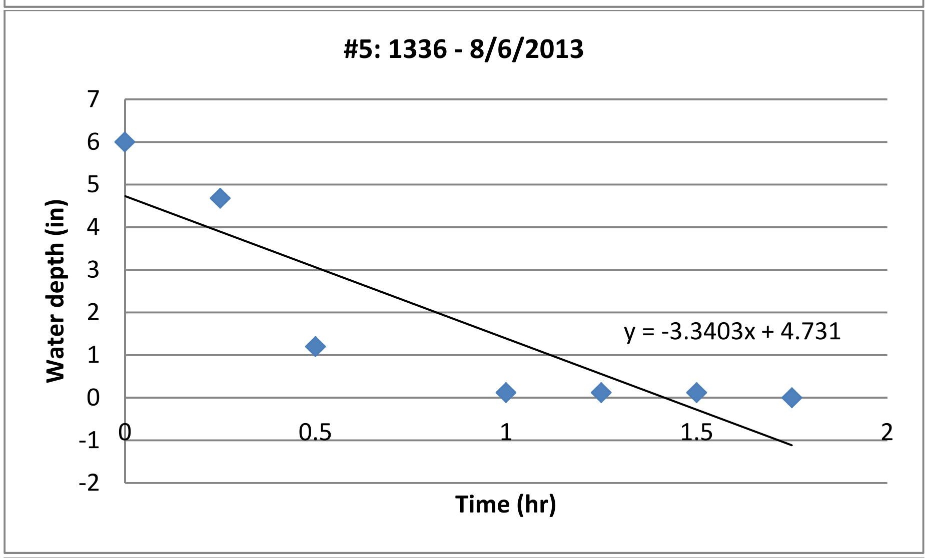

#5: 1336 - 9/19/2013
Water depth (in)
Time (hr)
y = -5.344x + 12.552
14
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5

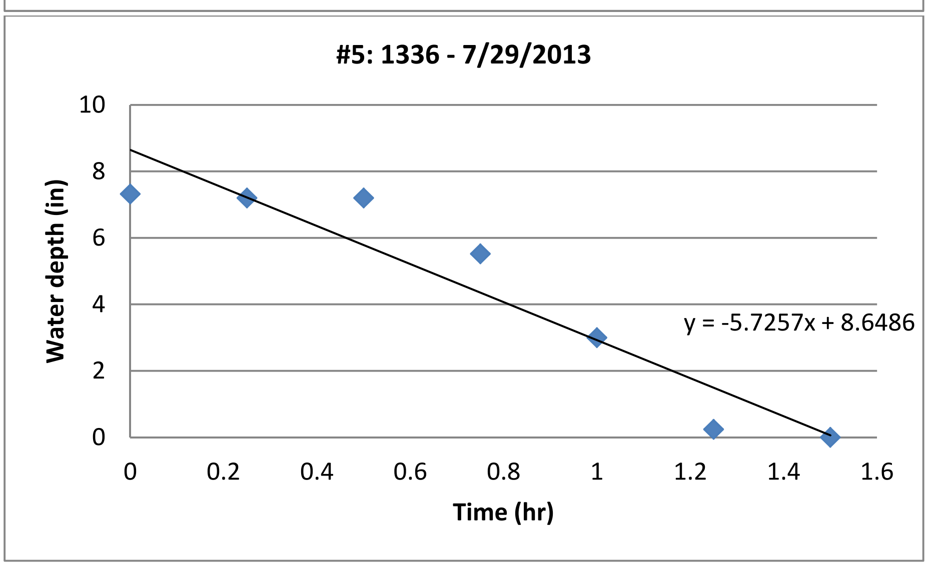

#5: 1336 - 9/17/2013
Water depth (in)
Time (hr)
y = -4.2629x + 8.47
10
8
6
4
2
0
0
0.5
1
1.5
2

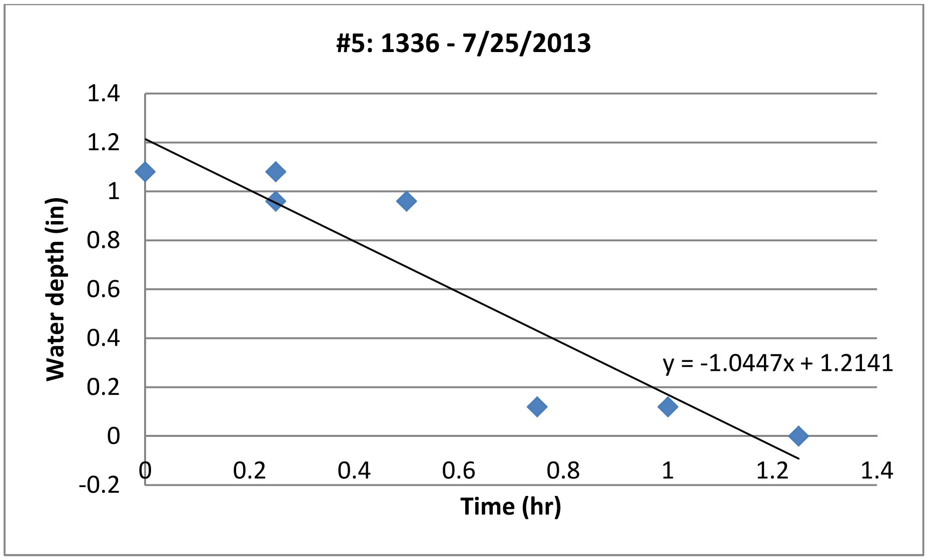

#5: 1336 - 9/1/2013
Water depth (in)
Time (hr)
y = -1.0996x + 3.2836
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2
2.5
3

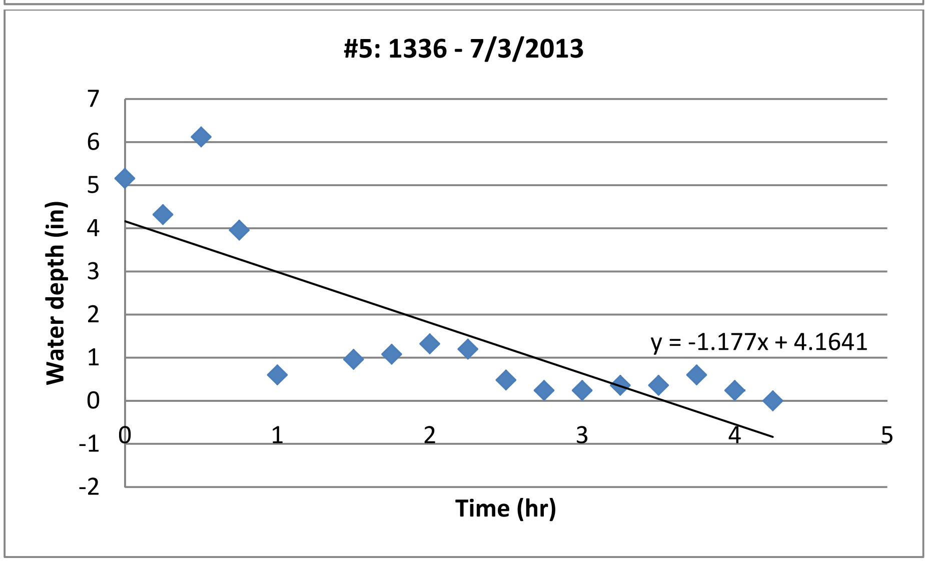

#5: 1336 - 8/12/2013
Water depth (in)
Time (hr)
y = -3.5006x + 12.07
14
12
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5
3
3.5
4

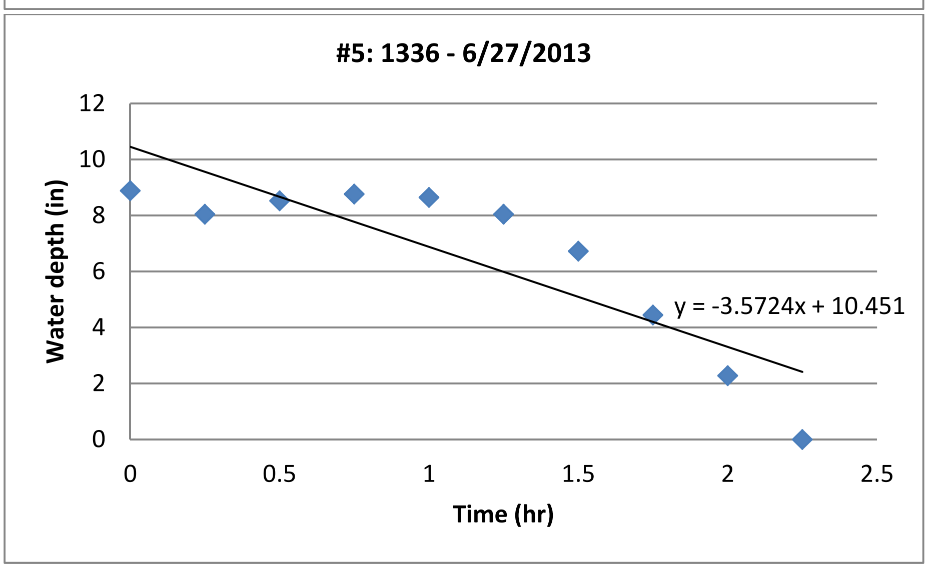

#5: 1336 - 8/7/2013
Water depth (in)
Time (hr)
y = -4.5353x + 9.4582
10
8
6
4
2
0
-2
0
0.5
1
1.5
2
2.5

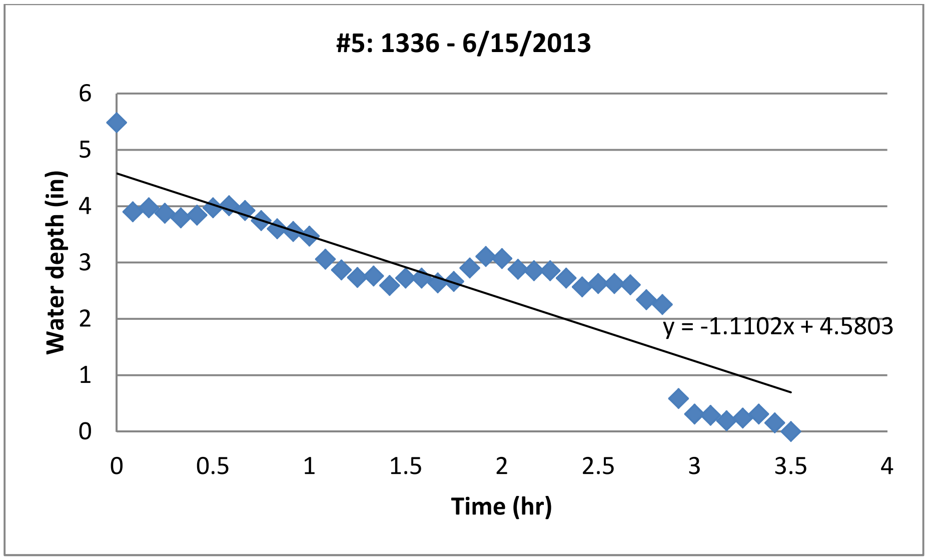

#5: 1336 - 8/6/2013
Water depth (in)
Time (hr)
y = -3.3403x + 4.731
7
6
5
4
3
2
1
0
-1
-2
0
0.5
1
1.5
2

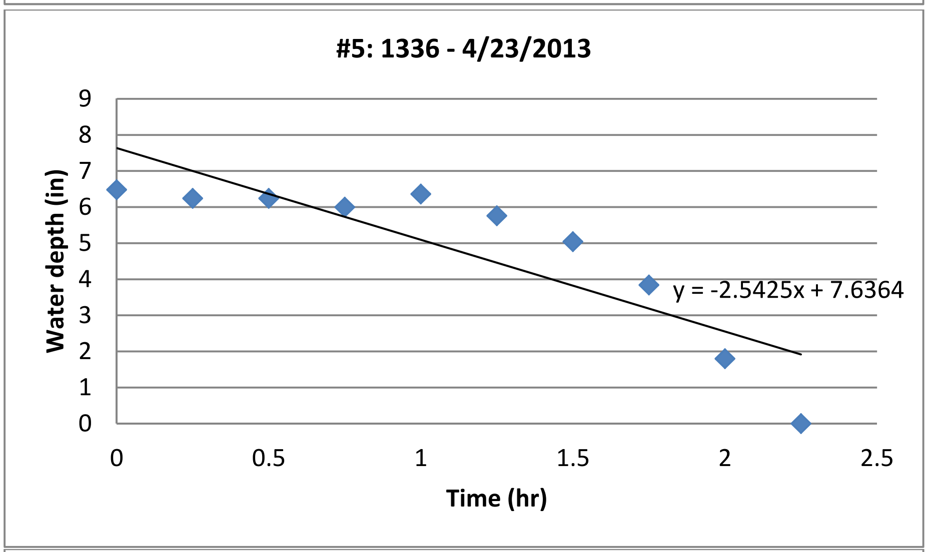

#5: 1336 - 7/29/2013
Water depth (in)
Time (hr)
y = -5.7257x + 8.6486
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6

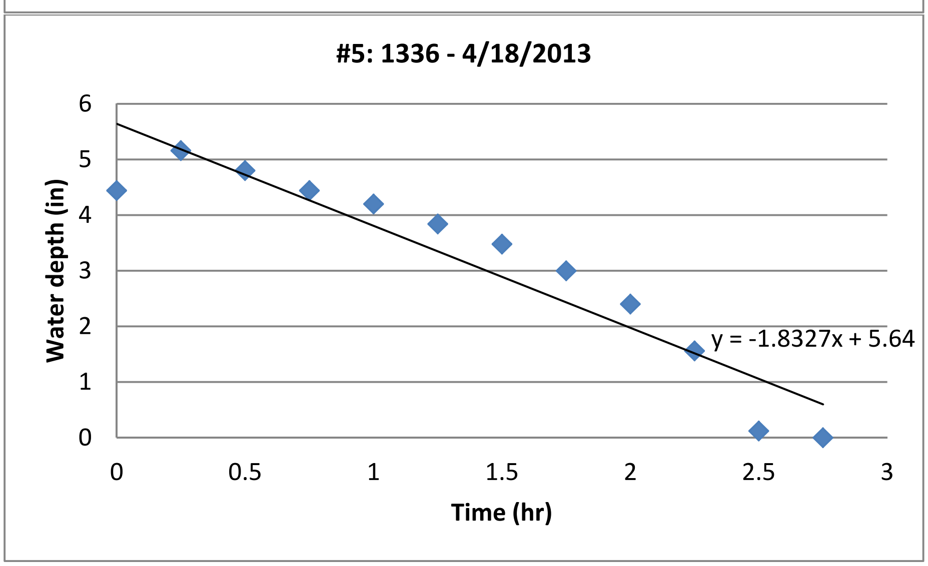

#5: 1336 - 7/25/2013
Water depth (in)
Time (hr)
y = -1.0447x + 1.2141
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
1.4

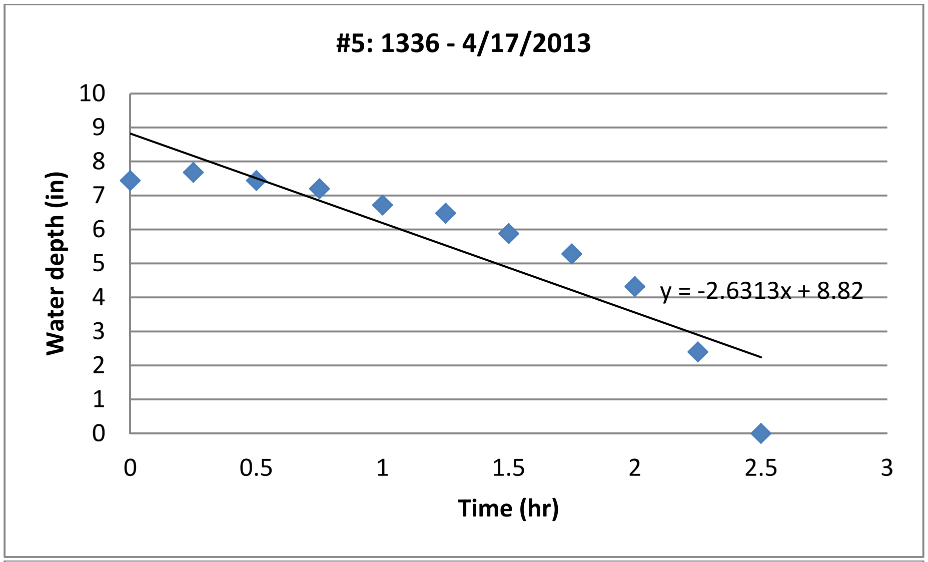

#5: 1336 - 7/3/2013
Water depth (in)
Time (hr)
y = -1.177x + 4.1641
7
6
5
4
3
2
1
0
-1
-2
0
1
2
3
4
5

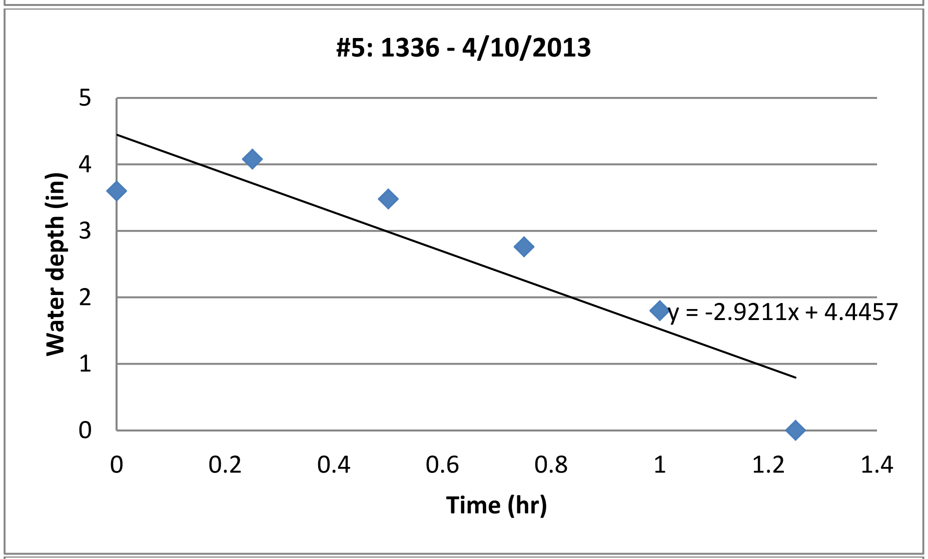

#5: 1336 - 6/27/2013
Water depth (in)
Time (hr)
y = -3.5724x + 10.451
12
10
8
6
4
2
0
0
0.5
1
1.5
2
2.5

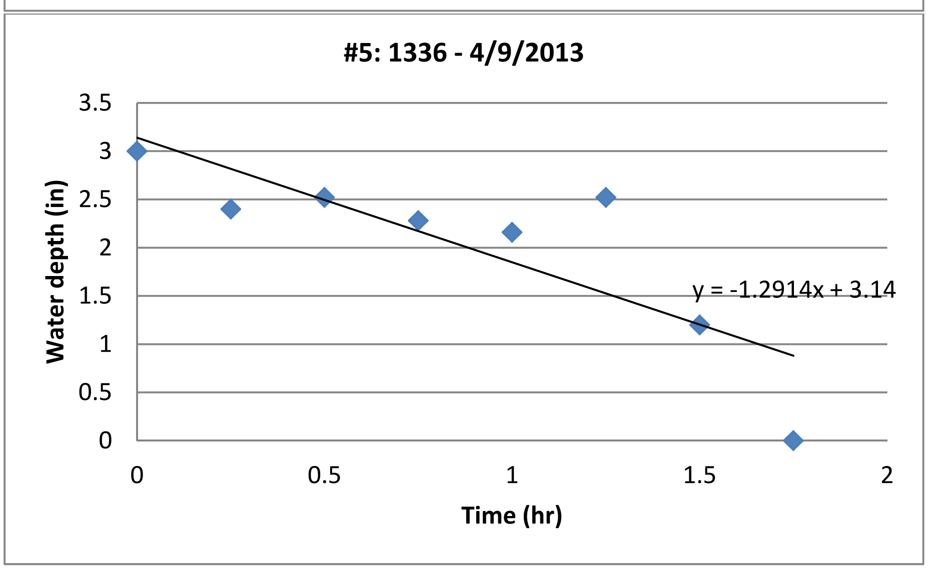

#5: 1336 - 6/15/2013
Water depth (in)
Time (hr)
y = -1.1102x + 4.5803
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5
3
3.5
4

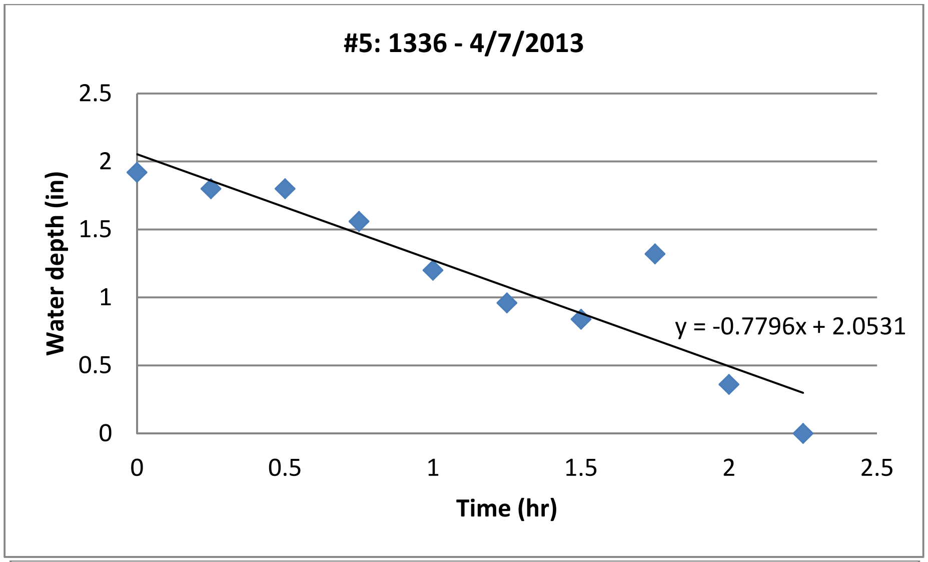

#5: 1336 - 4/23/2013
Water depth (in)
Time (hr)
y = -2.5425x + 7.6364
0
1
2
3
4
5
6
7
8
9
0
0.5
1
1.5
2
2.5


#5: 1336 - 4/18/2013
Water depth (in)
Time (hr)
y = -1.8327x + 5.64
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5
3

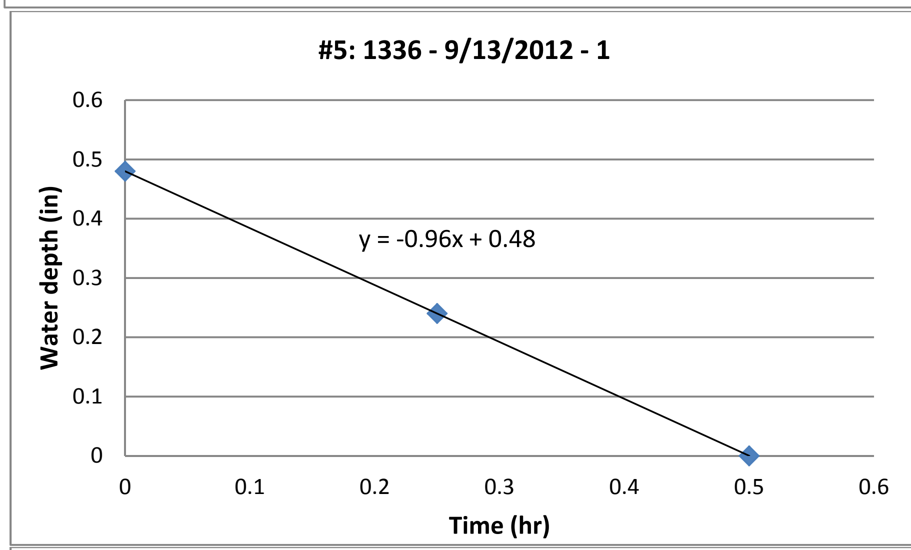

#5: 1336 - 4/17/2013
Water depth (in)
Time (hr)
y = -2.6313x + 8.82


#5: 1336 - 4/10/2013
Water depth (in)
Time (hr)
y = -2.9211x + 4.4457

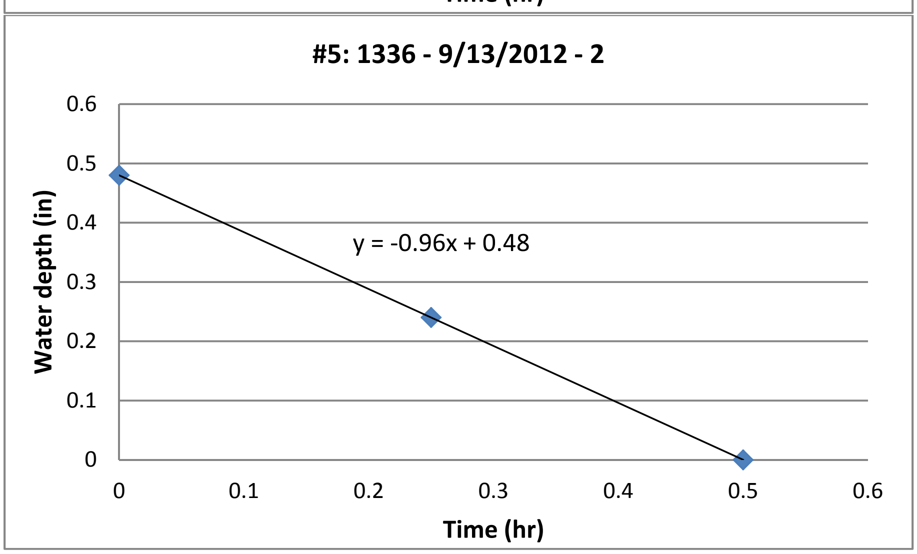

#5: 1336 - 4/9/2013
Water depth (in)
Time (hr)
y = -1.2914x + 3.14



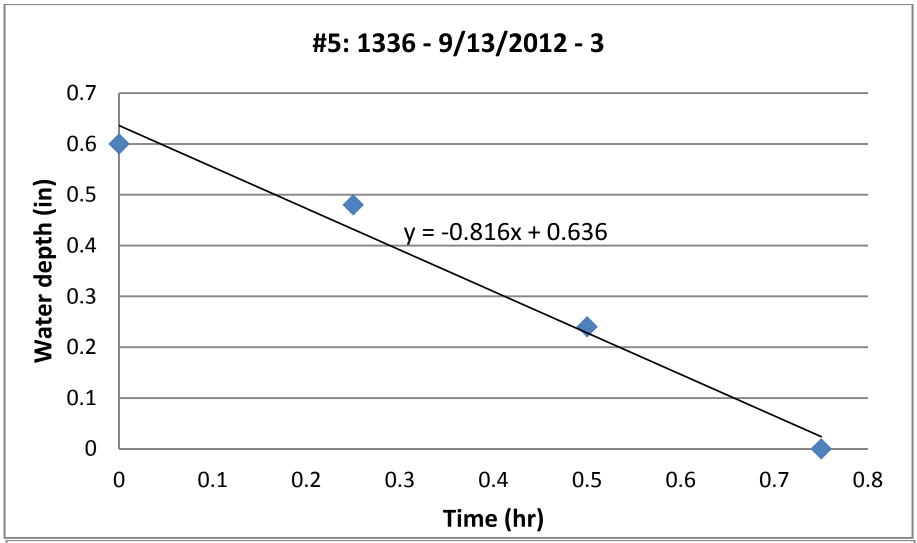
#5: 1336 - 4/7/2013
Water depth (in)
Time (hr)
y = -0.7796x + 2.0531

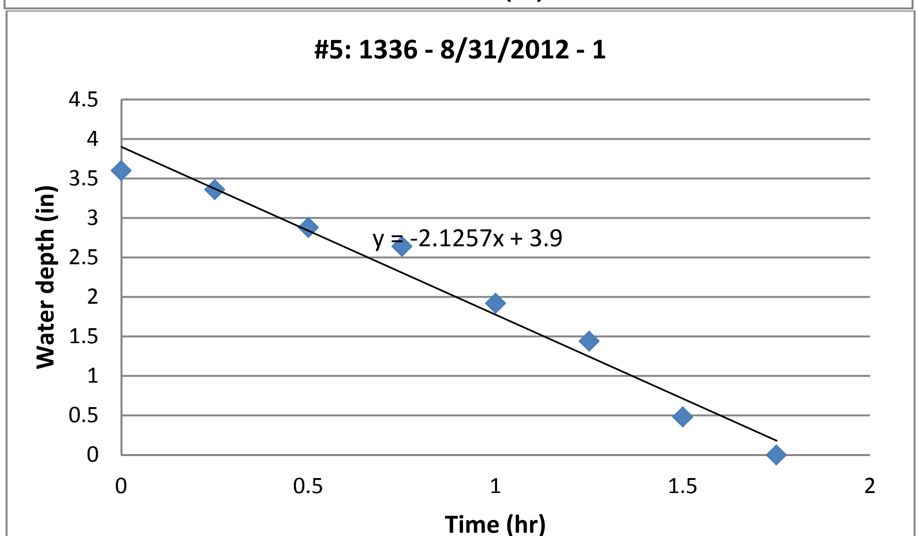


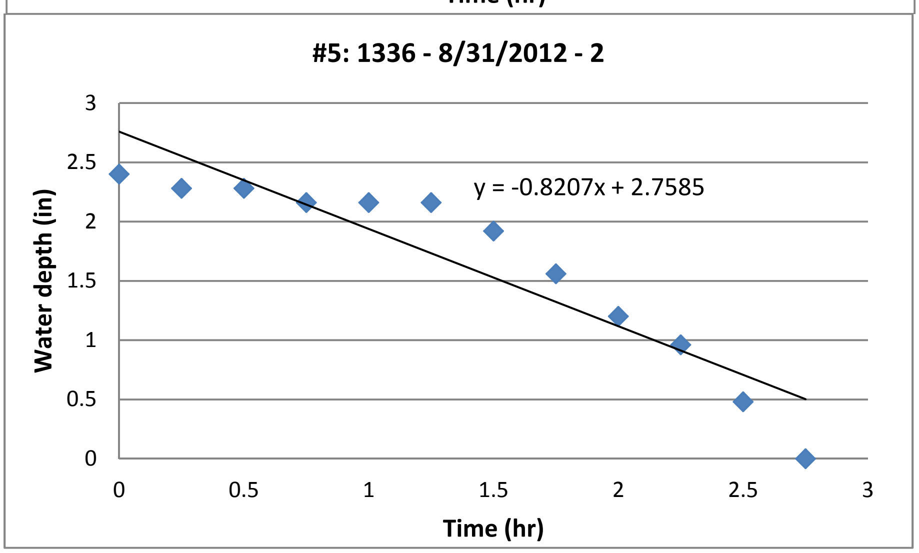
#5: 1336 - 9/13/2012 - 1
Water depth (in)
Time (hr)
y = -0.96x + 0.48

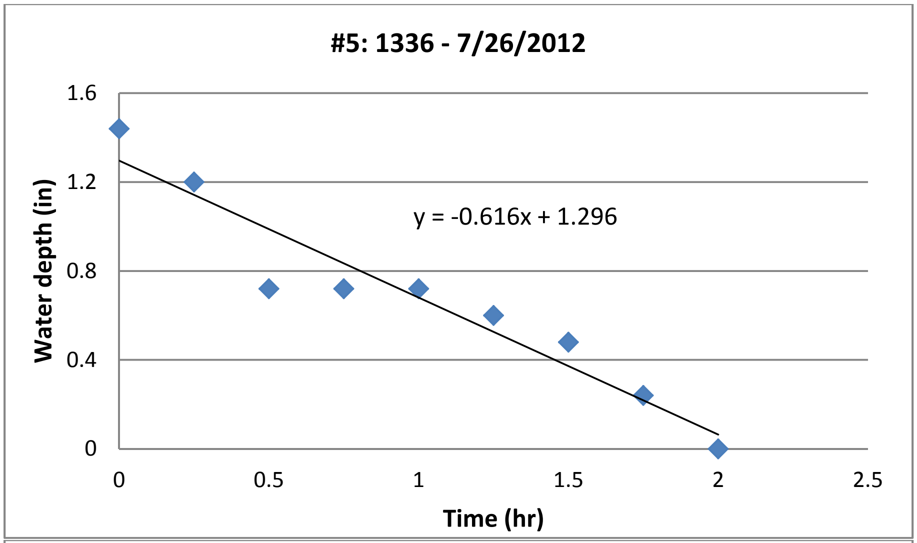


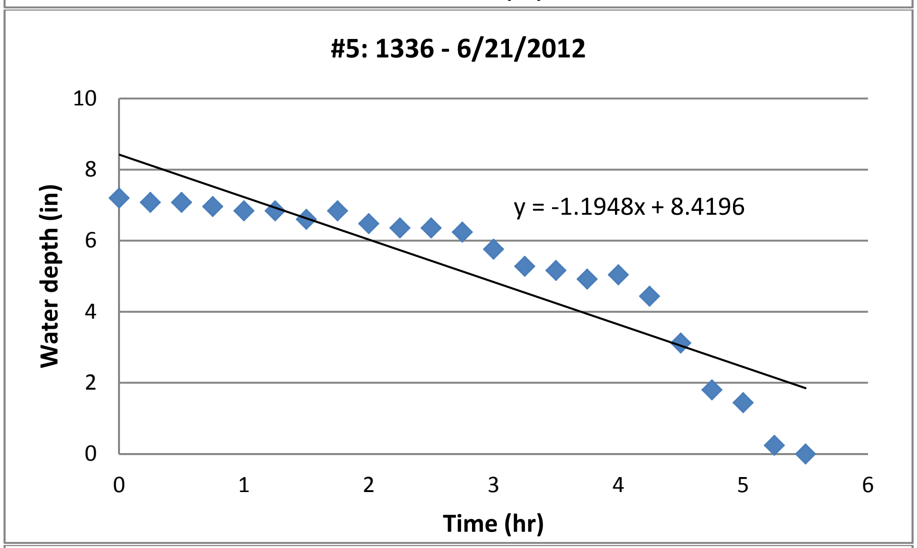
#5: 1336 - 9/13/2012 - 2
Water depth (in)
Time (hr)
y = -0.96x + 0.48

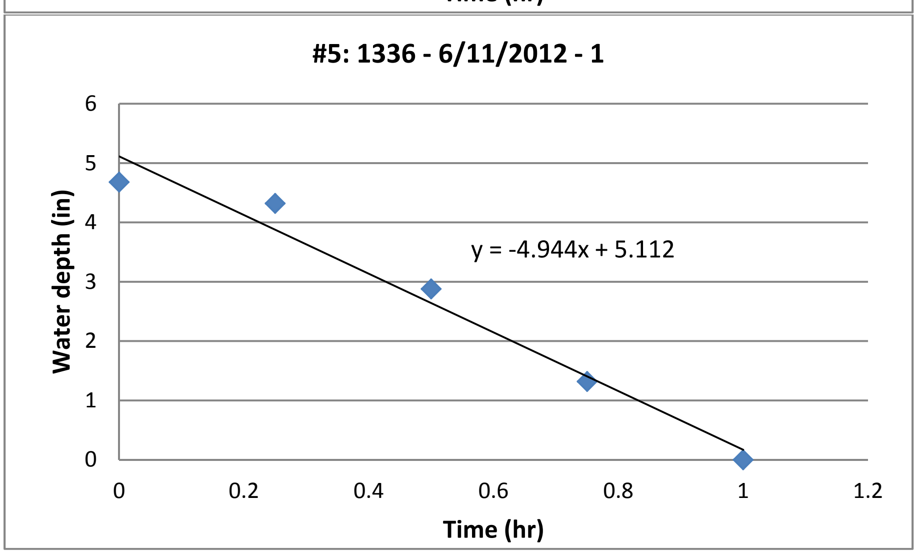

#5: 1336 - 9/13/2012 - 3
Water depth (in)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
y = -0.816x + 0.636
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Time (hr)

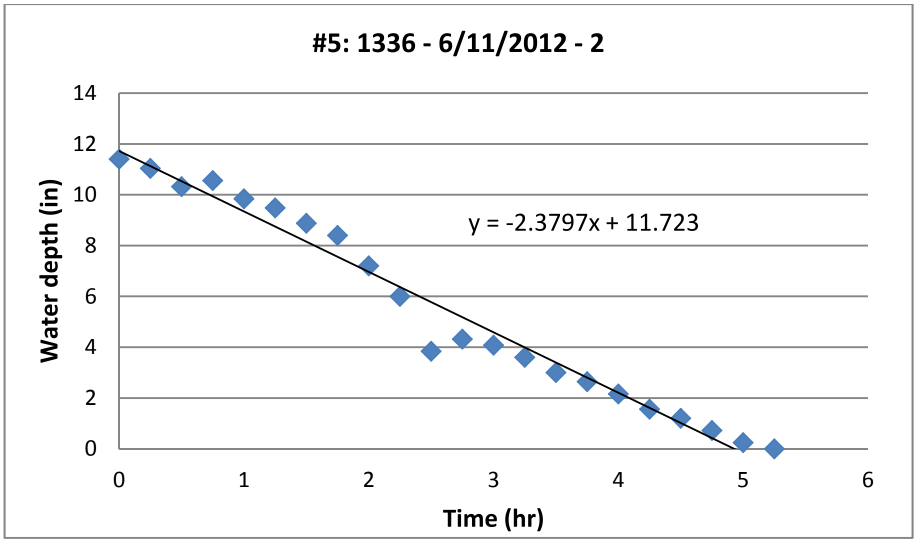

#5: 1336 - 8/31/2012 - 1
Water depth (in)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
y = -2.1257x + 3.9
0
0.5
1
1.5
2
Time (hr)


#5: 1336 - 8/31/2012 - 2
Water depth (in)
3
2.5
2
1.5
1
0.5
0
y = -0.8207x + 2.7585
0
0.5
1
1.5
2
2.5
3
Time (hr)

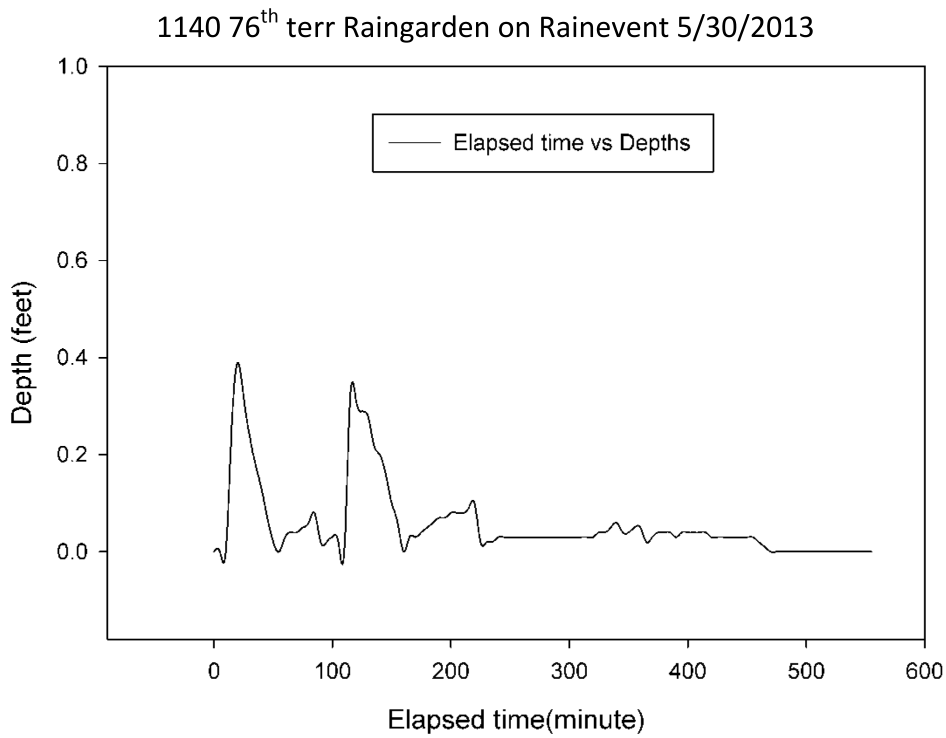

#5: 1336 - 7/26/2012
Water depth (in)
Time (hr)
y = -0.616x + 1.296
1.6
1.2
0.8
0.4
0
0
0.5
1
1.5
2
2.5

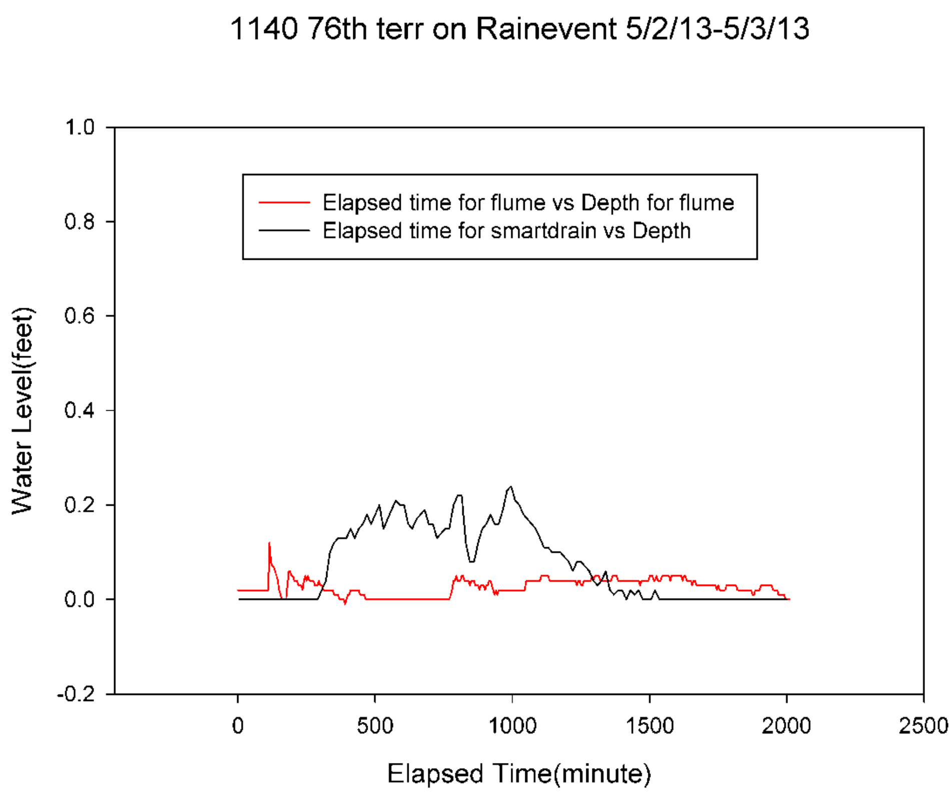

#5: 1336 - 6/21/2012
Water depth (in)
Time (hr)
y = -1.1948x + 8.4196
10
8
6
4
2
0
0
1
2
3
4
5
6


#5: 1336 - 6/11/2012 - 1
Water depth (in)
Time (hr)
y = -4.944x + 5.112
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
1.2

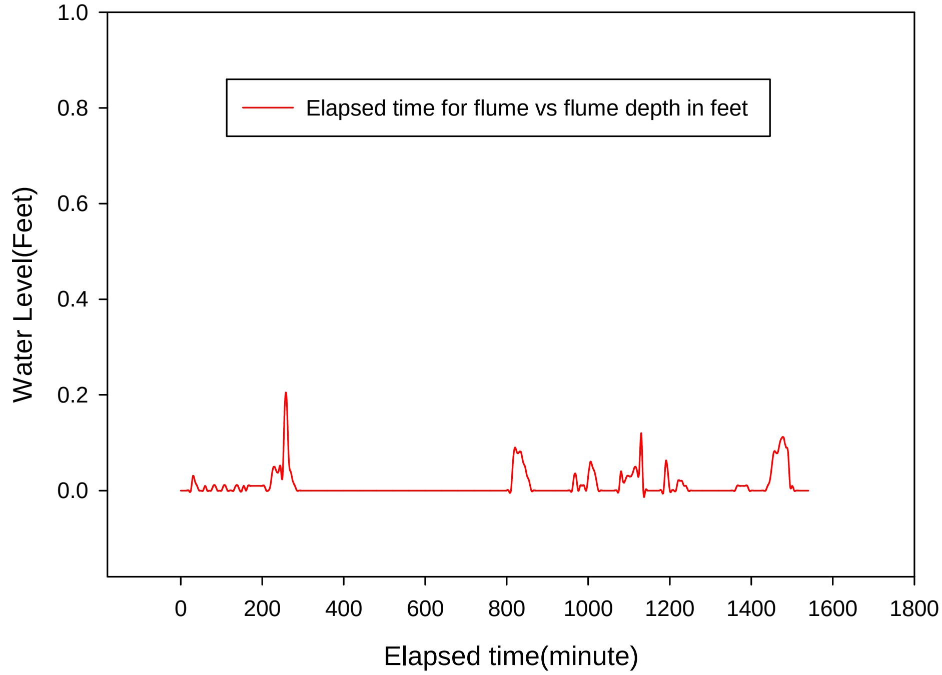

#5: 1336 - 6/11/2012 - 2
y = -2.3797x + 11.723
Water depth (in)
Time (hr)
0
2
4
6
8
10
12
14
0
1
2
3
4
5
6

7- Shallow Curb-Cut Biofilter with SmartDrain - 1140 E 76th Terr.

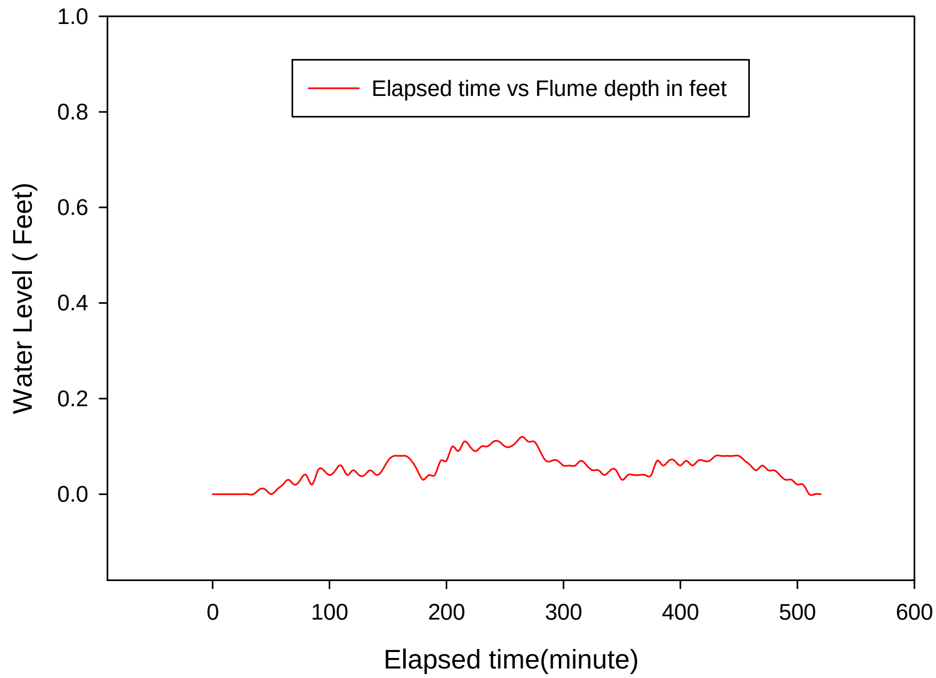




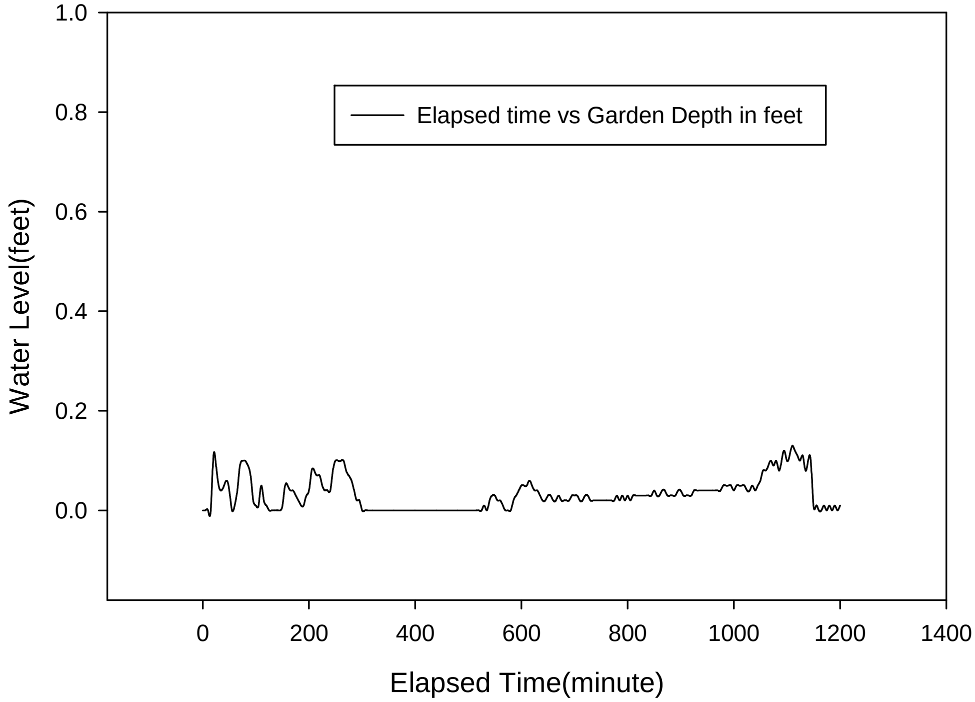
1140 76th terr Raingarden on Rainevent 10/13/2012



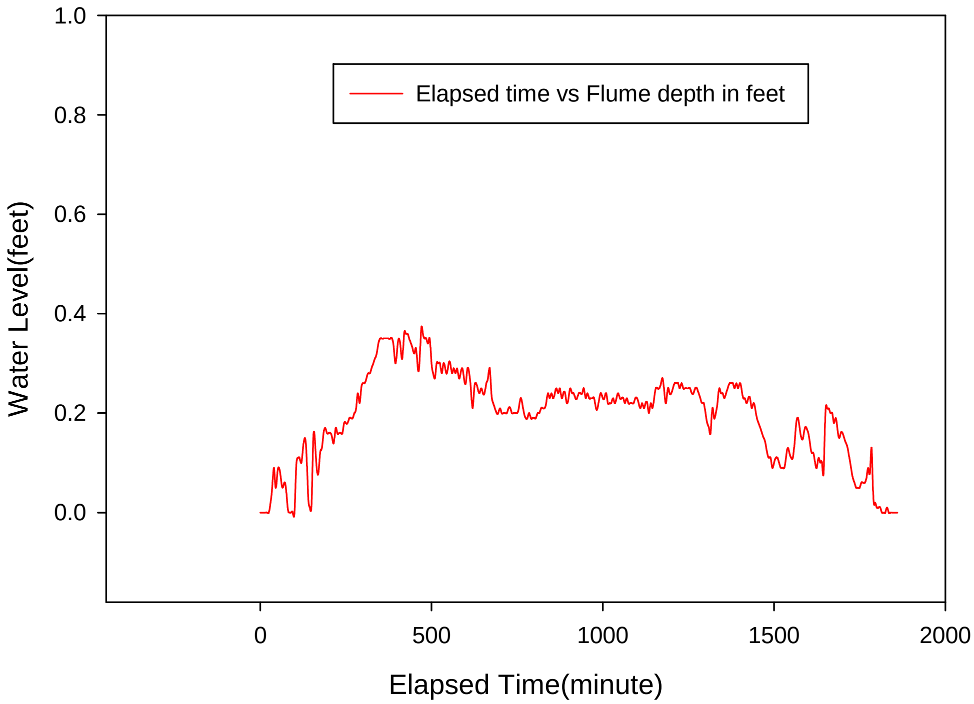
1140 76th terr Raingarden on Rainevent 9/26/2012

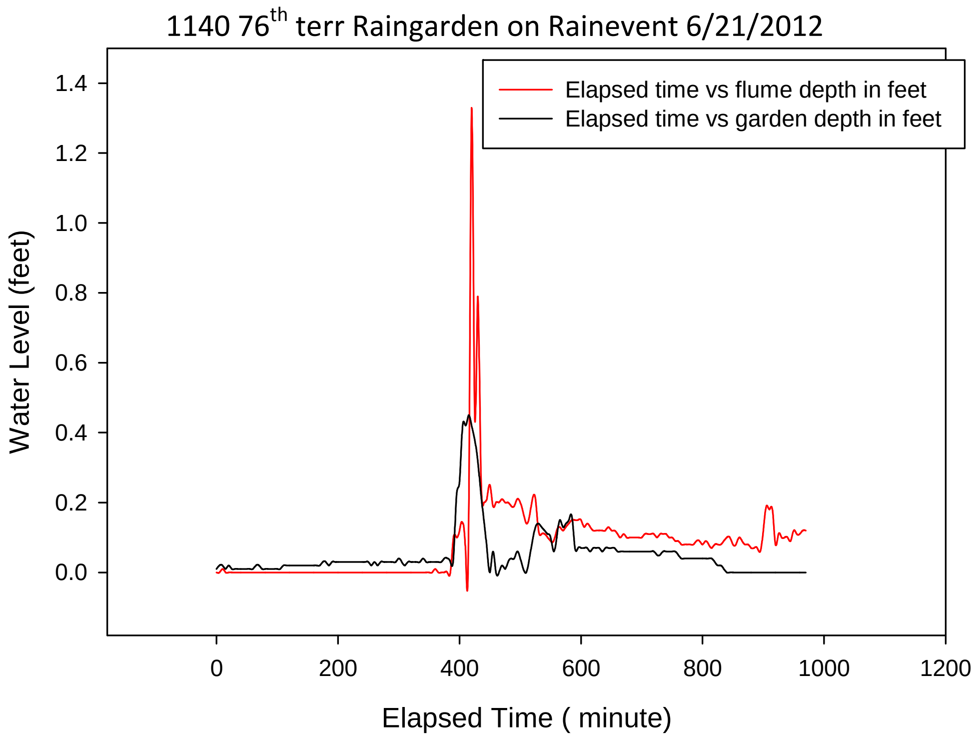


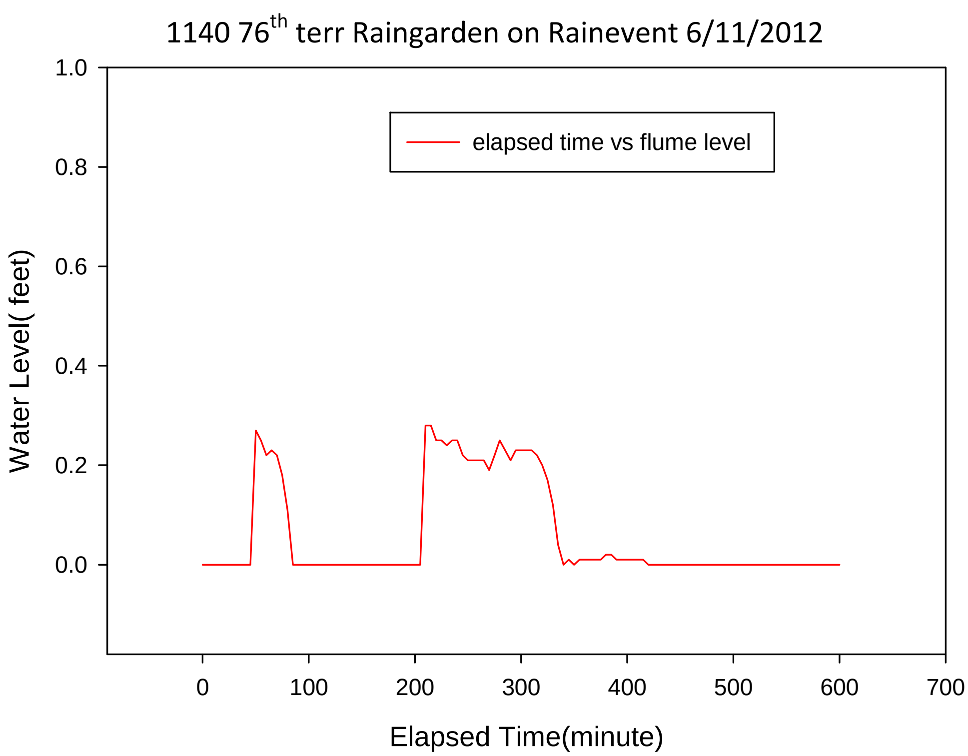
1140 76th terr Raingarden on Rainevent 9/13/2012



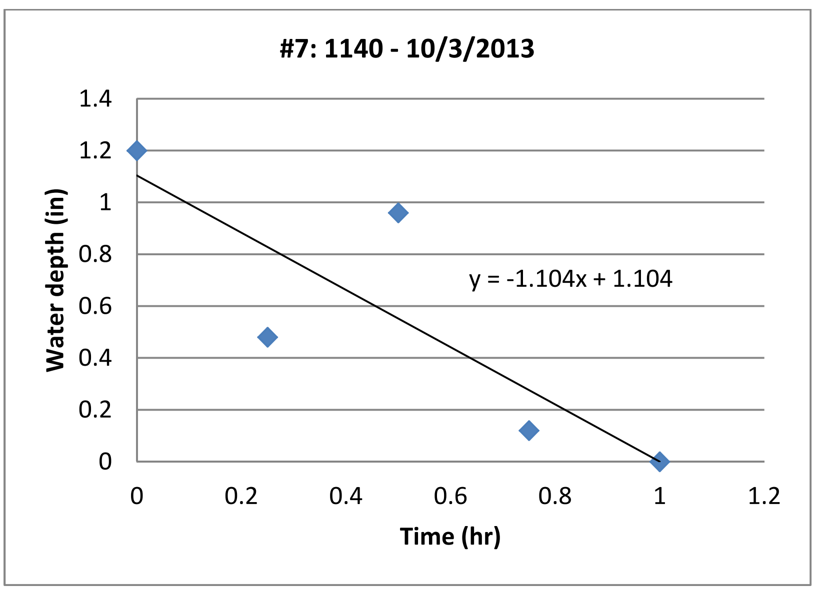
1140 76th terr Raingarden on Rainevent 9/1/2012

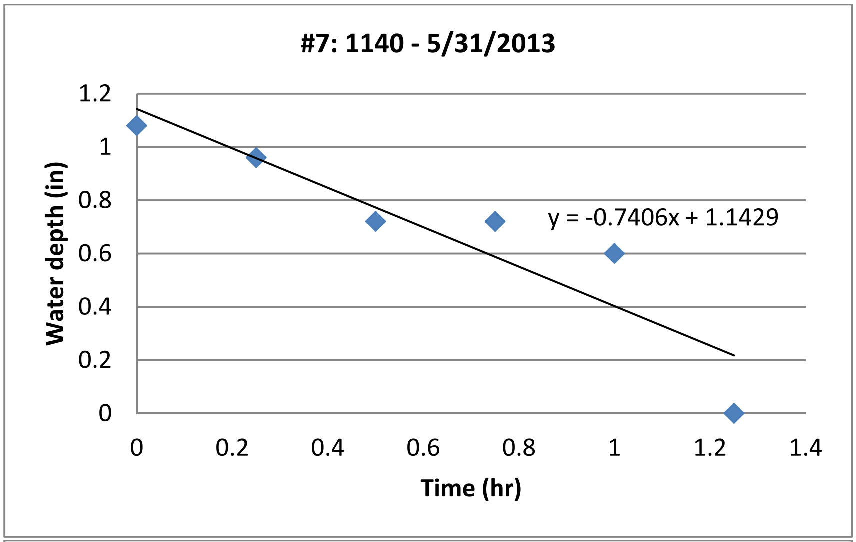


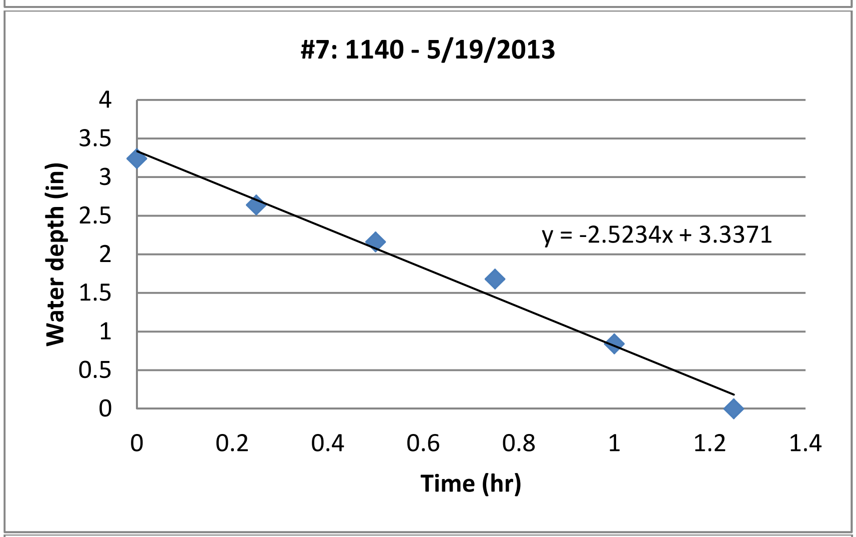

1140 76th terr Raingarden on Rainevent 6/21/2012
Elapsed time vs flume depth in feet
Elapsed time vs garden depth in feet
Water Level (feet)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
200
400
600
800
1000
1200
Elapsed Time (minute)

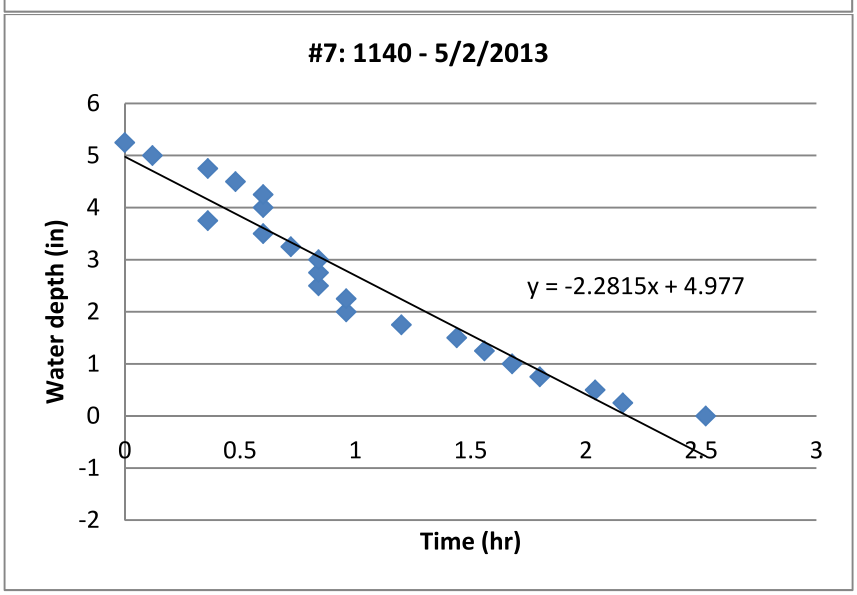

1140 76th terr Raingarden on Rainevent 6/11/2012
elapsed time vs flume level
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
100
200
300
400
500
600
700
Elapsed Time(minute)

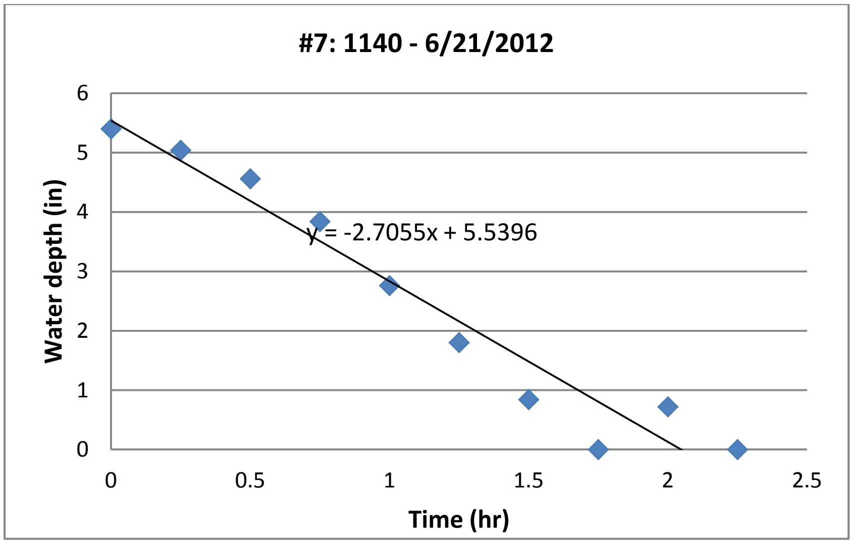
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
0.16	10/3/2013 11:04:00	10/3/2013 11:24:00	0:20	1.2	0:07	1.1
1.62	5/29/2013 22:53:00	5/30/2013 15:17:00	16:24	1.08	33:46	0.74
0.83	5/19/2013 2:21:00	5/20/2013 1:44:00	23:23	3.24	4:48	2.52
1.42	5/2/2013 03:08:00	5/4/2013 04:11:00	49:03	2.52	13:30	2.28
0.86	10/12/2012 20:59:34	10/12/2012 22:39:34	25:40	0		
0.23	9/25/2012 23:54:34	9/26/2012 08:34:34	08:40	0		
0.43	9/13/2012 14:37:17	9/14/2012 10:34:34	20:00	1.56	No Flume data	No Flume data
5.60	8/31/2012 11:02:17	9/1/2012 18:02:17	31:00	0		
1.03	6/20/2012 19:22:24	6/21/2012 11:32:24	16:10	5.4	6:35	>2.7
0.8	6/11/2012 02:03:12	6/11/2012 12:03:12	10:00	0		




#7: 1140 - 5/31/2013
Water depth (in)
Time (hr)
y = -0.7406x + 1.1429

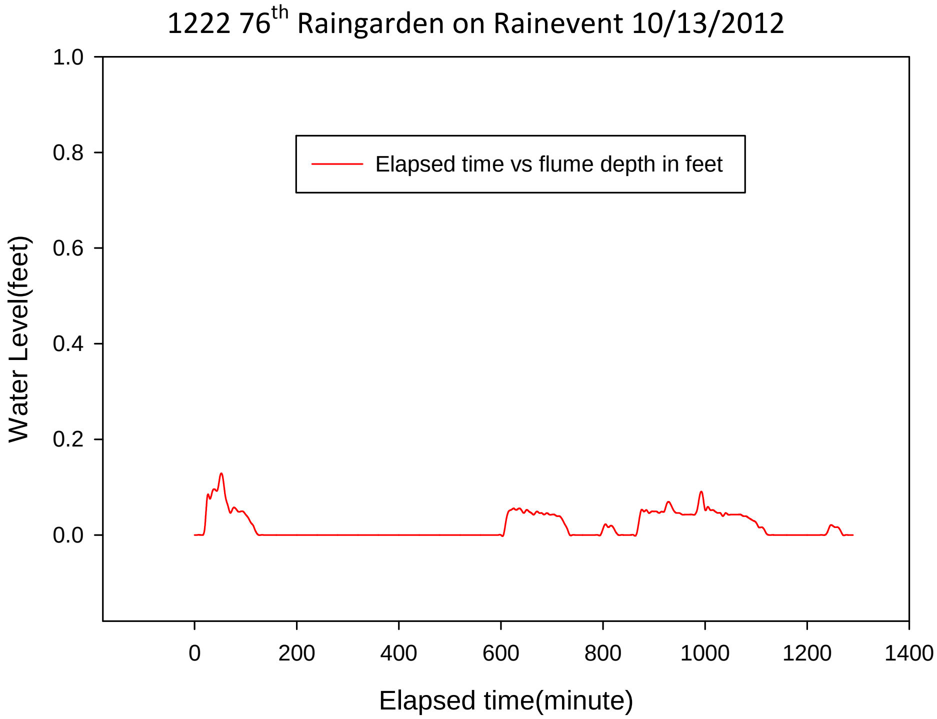

#7: 1140 - 5/19/2013
Water depth (in)
Time (hr)
y = -2.5234x + 3.3371

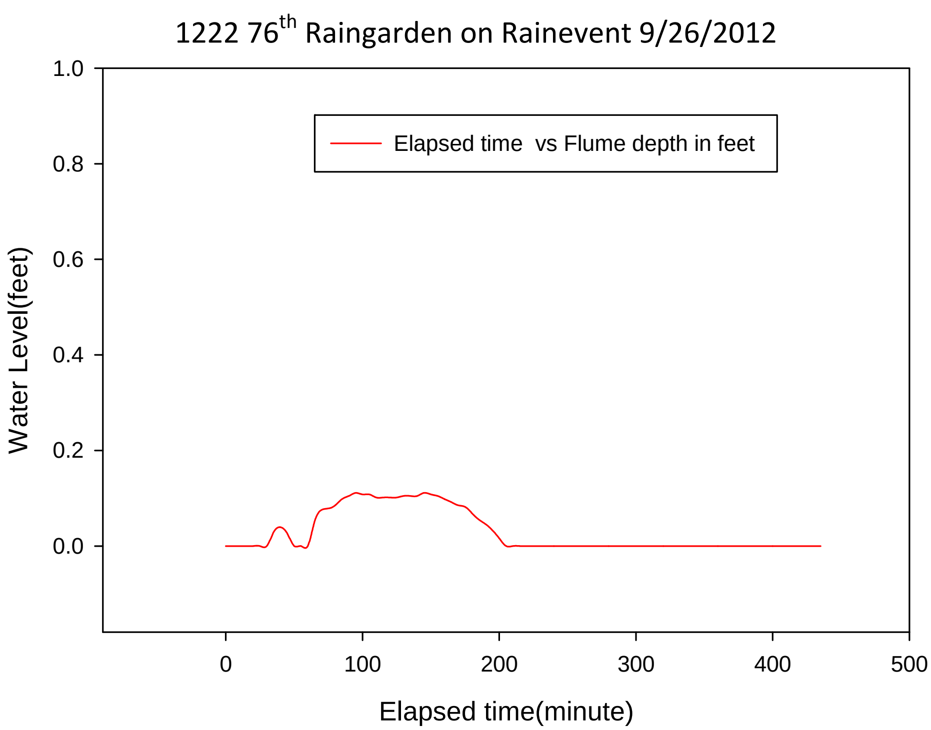

#7: 1140 - 5/2/2013
Water depth (in)
Time (hr)
y = -2.2815x + 4.977

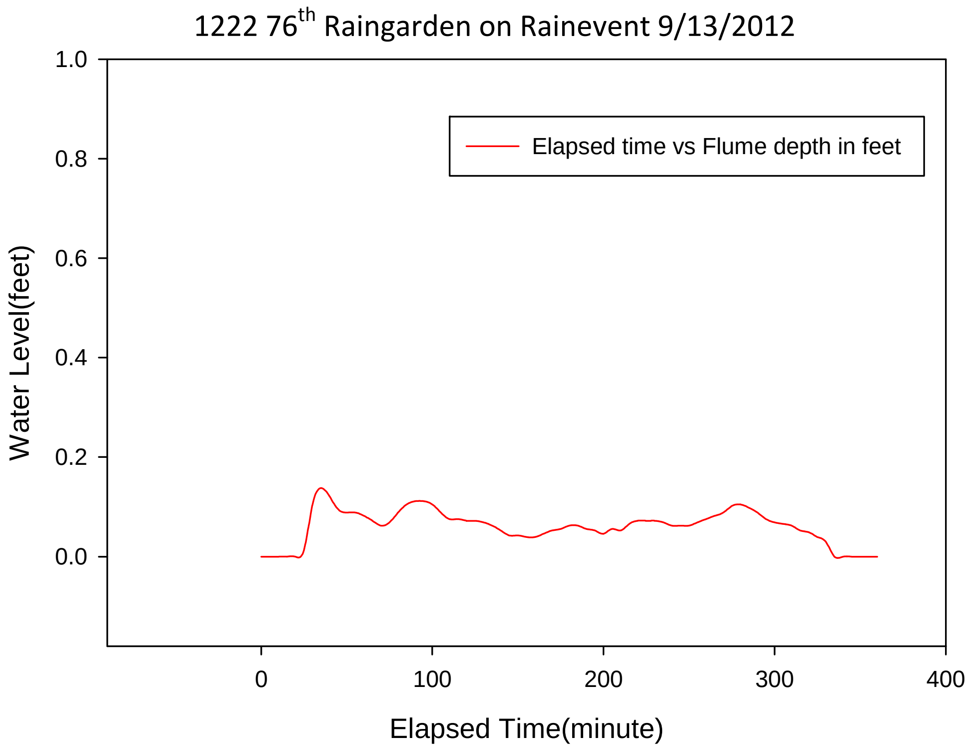


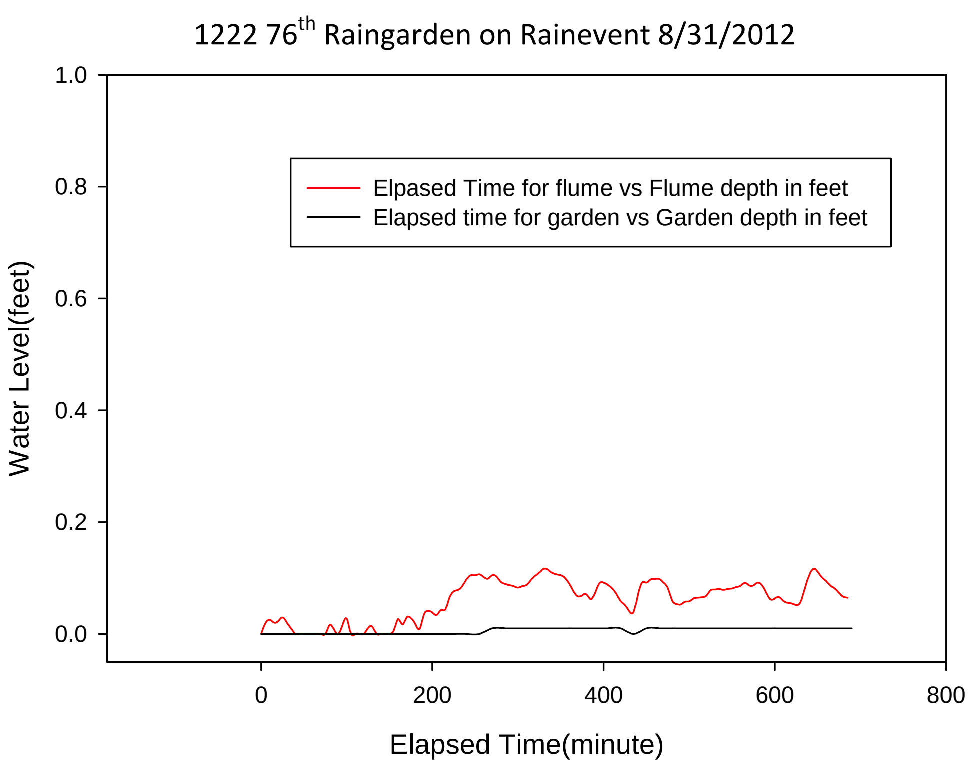
#7: 1140 - 6/21/2012
y = -2.7055x + 5.5396
Water depth (in)
Time (hr)
0
1
2
3
4
5
6
0
0.5
1
1.5
2
2.5

8- Shallow Curb-Cut Biofilter with SmartDrain - 1222 E 76th St.

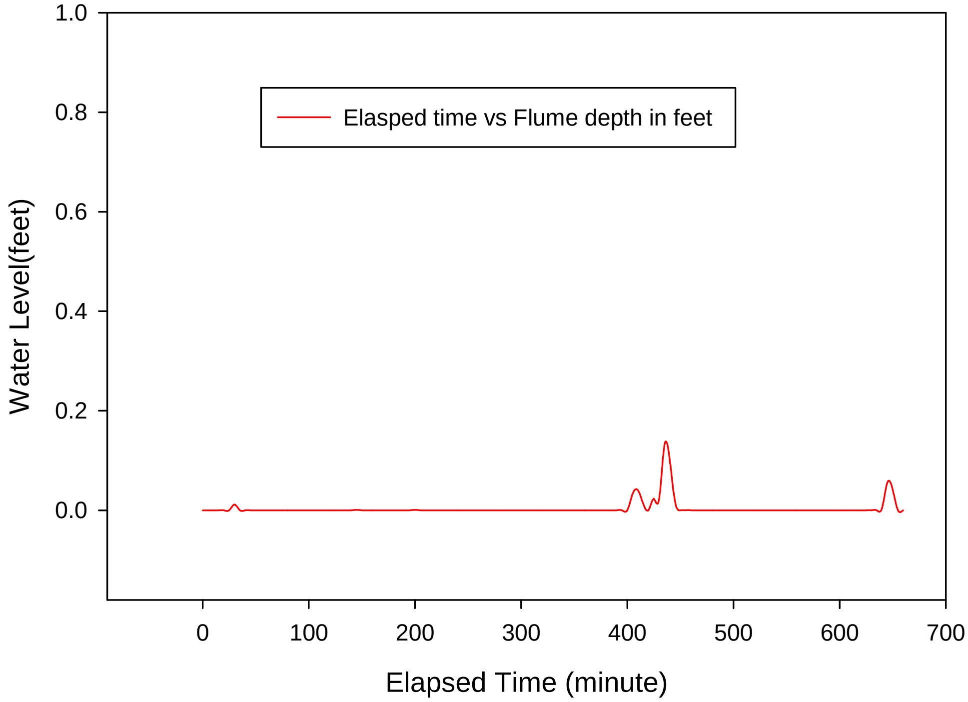




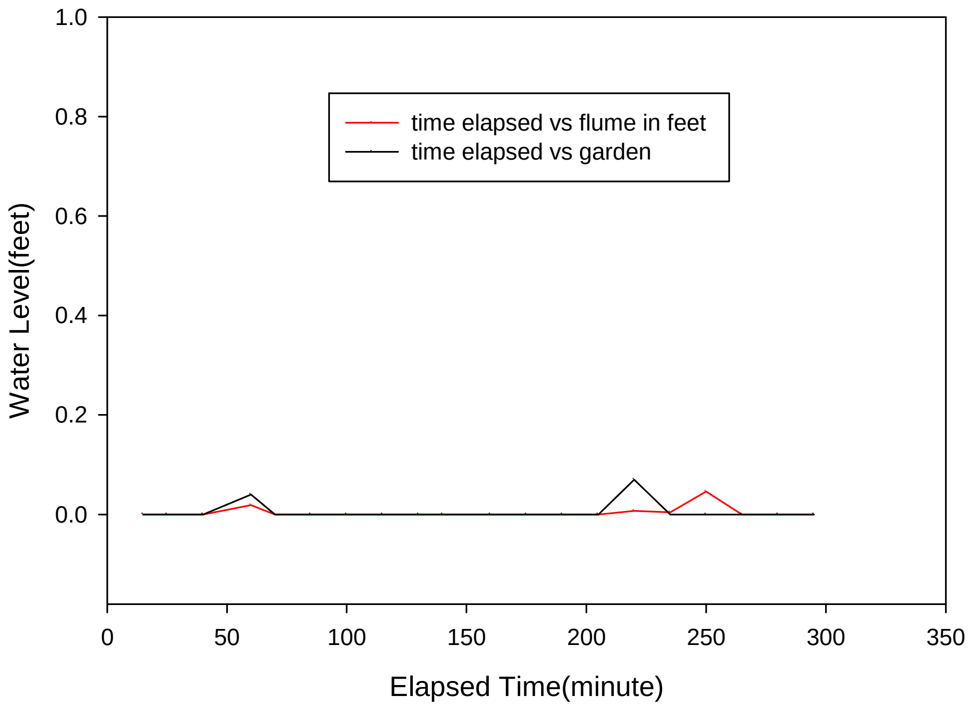

1222 76th Raingarden on Rainevent 9/13/2012
Elapsed time vs Flume depth in feet
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
100
200
300
400
Elapsed Time(minute)


1222 76th Raingarden on Rainevent 8/31/2012
Elpased Time for flume vs Flume depth in feet
Elapsed time for garden vs Garden depth in feet
Water Level(feet)
1.0
0.8
0.6
0.4
0.2
0.0
0
200
400
600
800
Elapsed Time(minute)

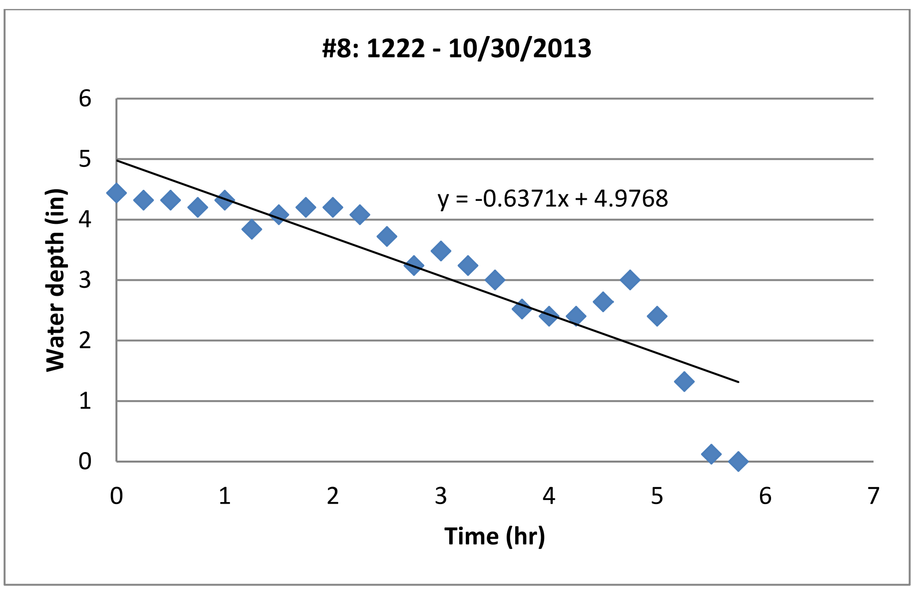
1222 76th Raingarden on Rainevent 7/26/2012

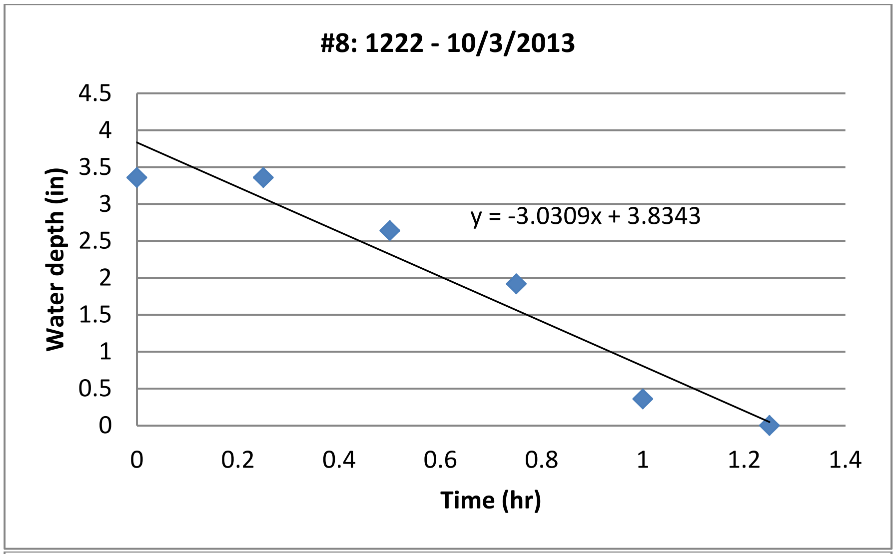


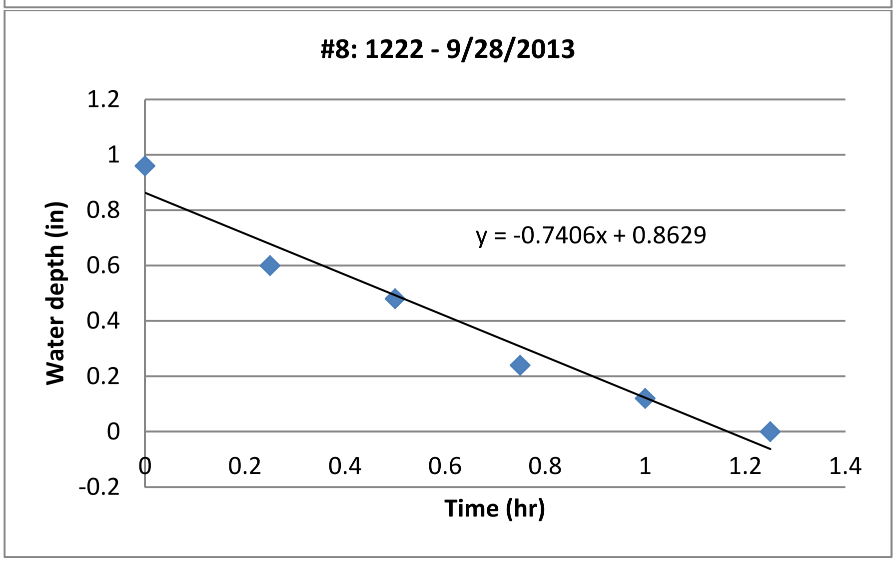
1222 76th Raingarden on Rainevent 6/11/2012

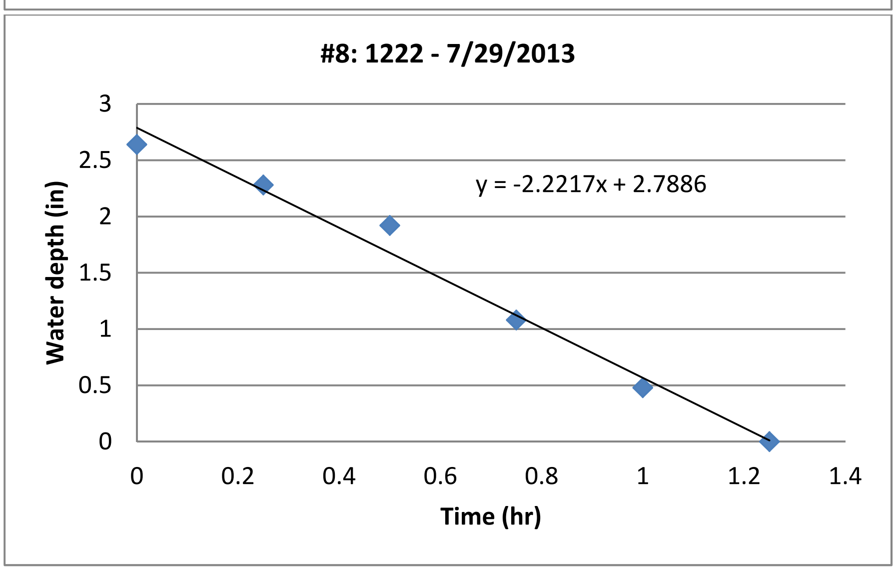


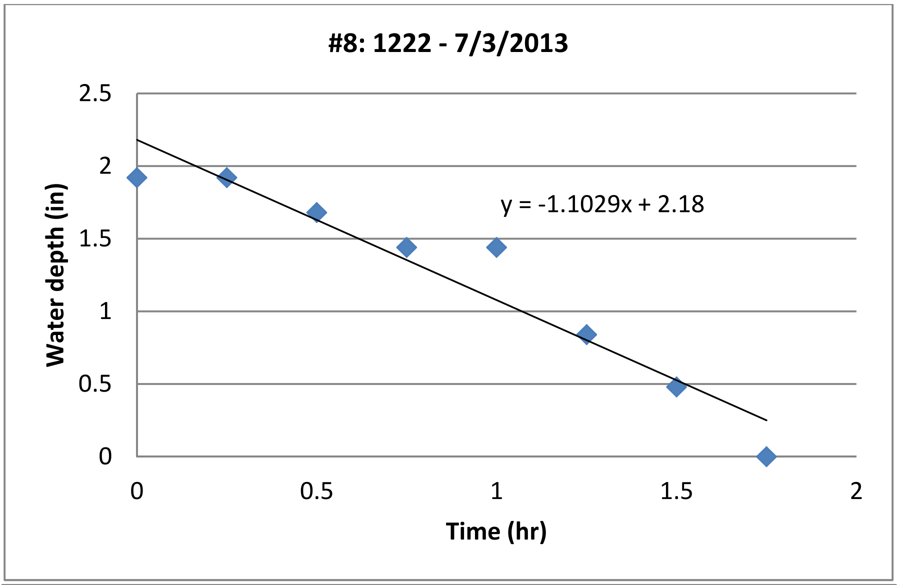
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:35	4.44	12:41	0.64
0.16	10/3/2013 11:04:00	10/3/2013 11:24:00	0:20	3.36	0:20	3.03
0.28	9/28/2013 8:29:00	9/28/2013 11:29	3:00	0.96	0:51	0.74
0.91	7/29/2013 6:27:00	7/30/2013 9:41:00	27:14	2.64	8:08	2.22
1.5	7/3/2013 17:54:00	7/3/2013 21:36:00	3:42	1.92	2:23	1.10
0.86	10/13/2012 00:30:00	10/13/2012 22:00:00	21:30	0		
0.23	9/26/2012 02:00:00	9/26/2012 09:15:00	07:15	0		
0.43	9/13/2012 14:30:00	9/13/2012 20:30:00	06:00	0		
2.61	8/31/2012 11:35:00	8/31/2012 23:00:00	11:25	0.12		
0.49	7/25/2012 18:00:00	7/26/2012 05:00:00	11:00	0		
0.8	6/11/2012 02:05:00	6/11/2012 06:50:00	04:55	0.84	3:40	3.36

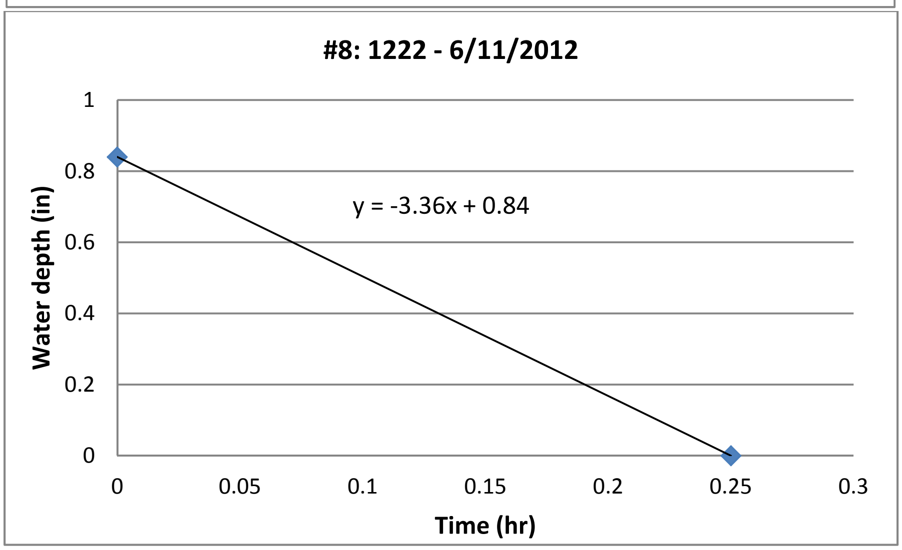



#8: 1222 - 10/3/2013
y = -3.0309x + 3.8343
Water depth (in)
Time (hr)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4

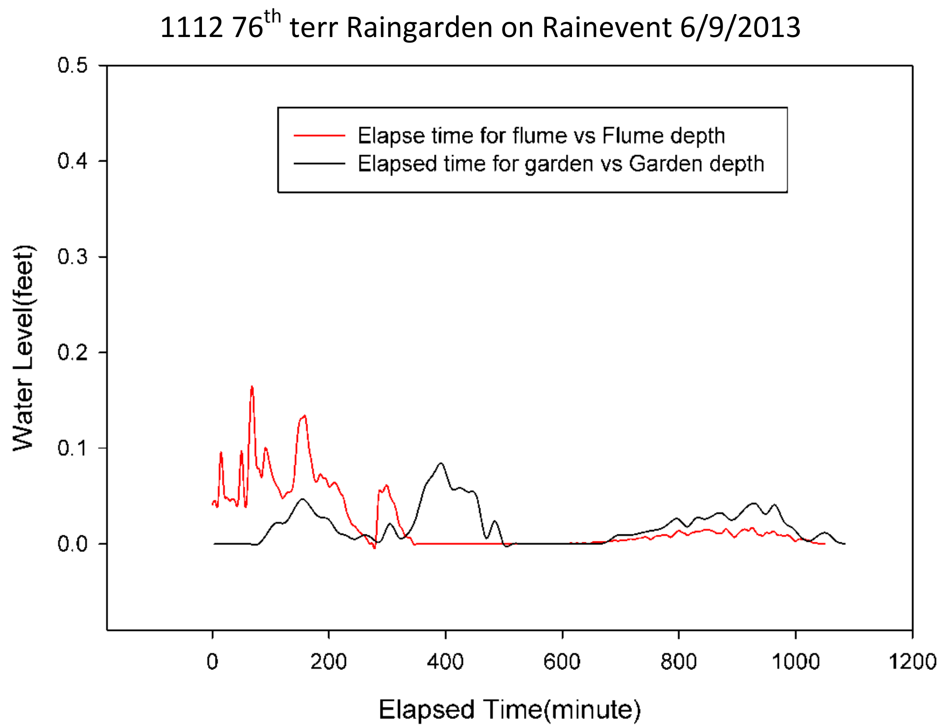

#8: 1222 - 9/28/2013
y = -0.7406x + 0.8629
Water depth (in)
Time (hr)
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
1.4

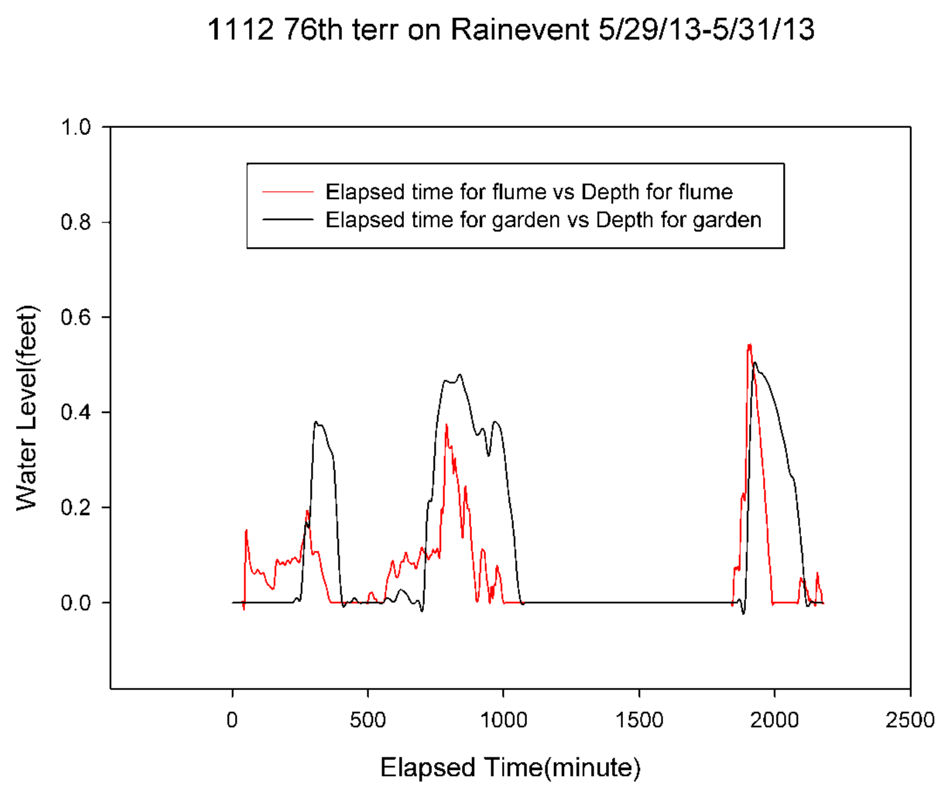

#8: 1222 - 7/29/2013
y = -2.2217x + 2.7886
Water depth (in)
Time (hr)
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4


#8: 1222 - 7/3/2013
Water depth (in)
Time (hr)
y = -1.1029x + 2.18
2.5
2
1.5
1
0.5
0
0
0.5
1
1.5
2

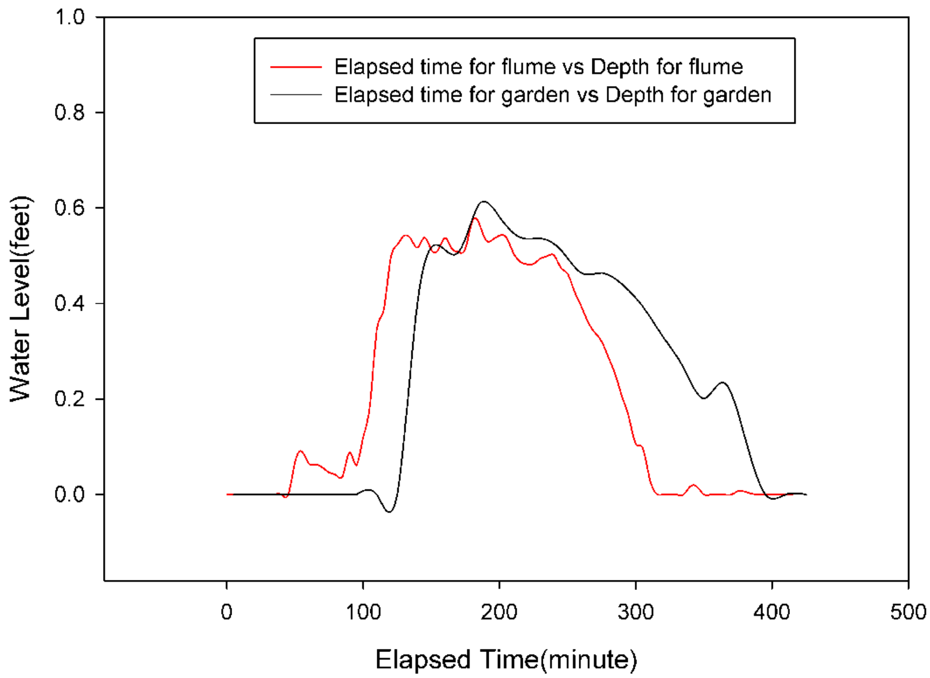

#8: 1222 - 6/11/2012
Water depth (in)
Time (hr)
y = -3.36x + 0.84
1
0.8
0.6
0.4
0.2
0
0
0.05
0.1
0.15
0.2
0.25
0.3

9- Cascading Swale Biofilter - 1112 E 76th Terr.

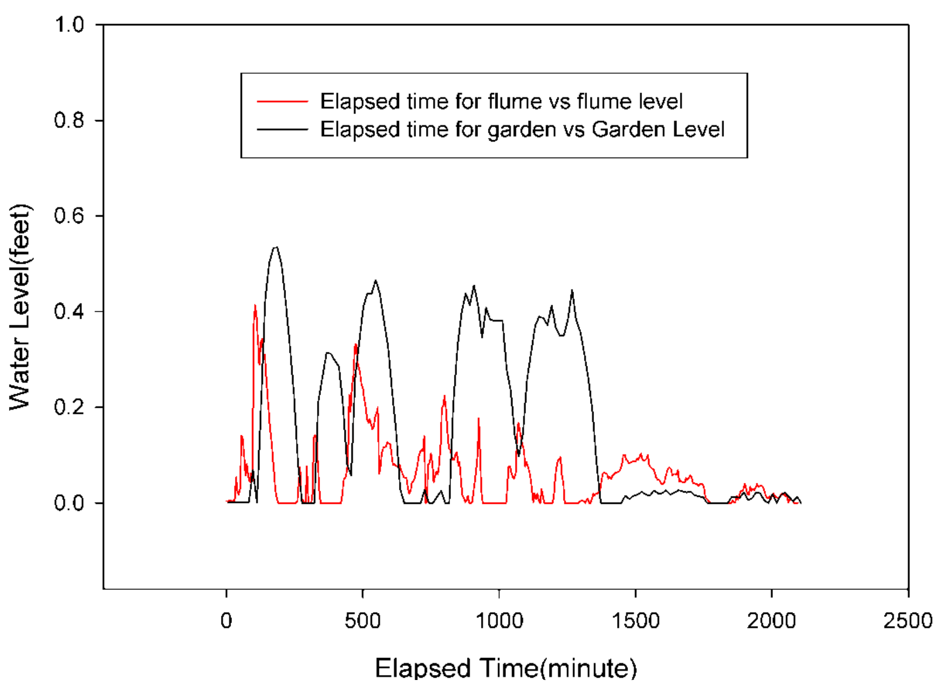




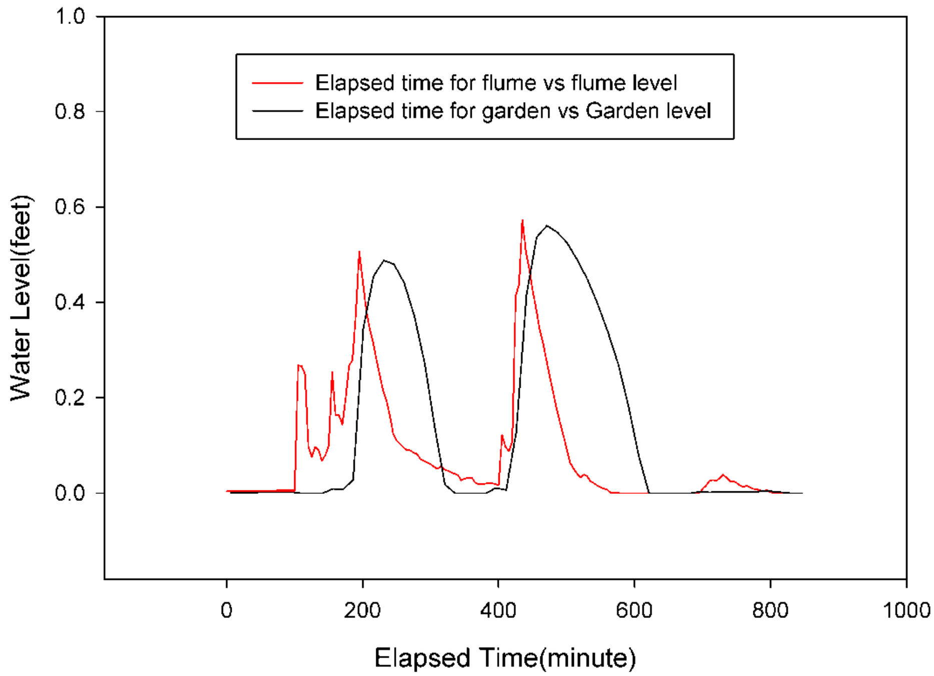
1112 76th terr on Rainevent 5/27/13-5/28/13



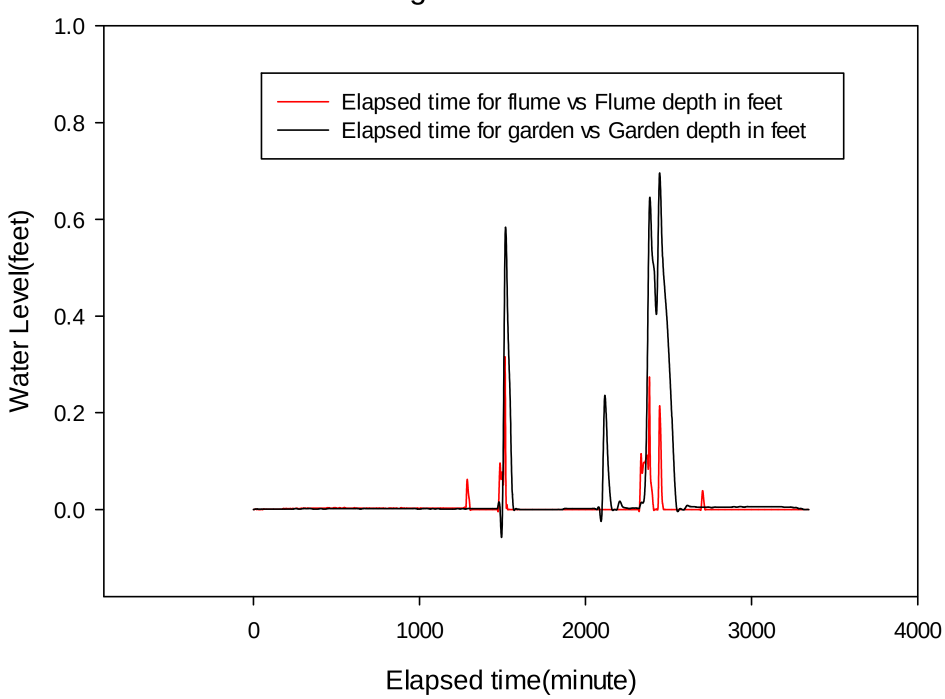
1112 76th terr on Rainevent 04/09/13--04/10/13



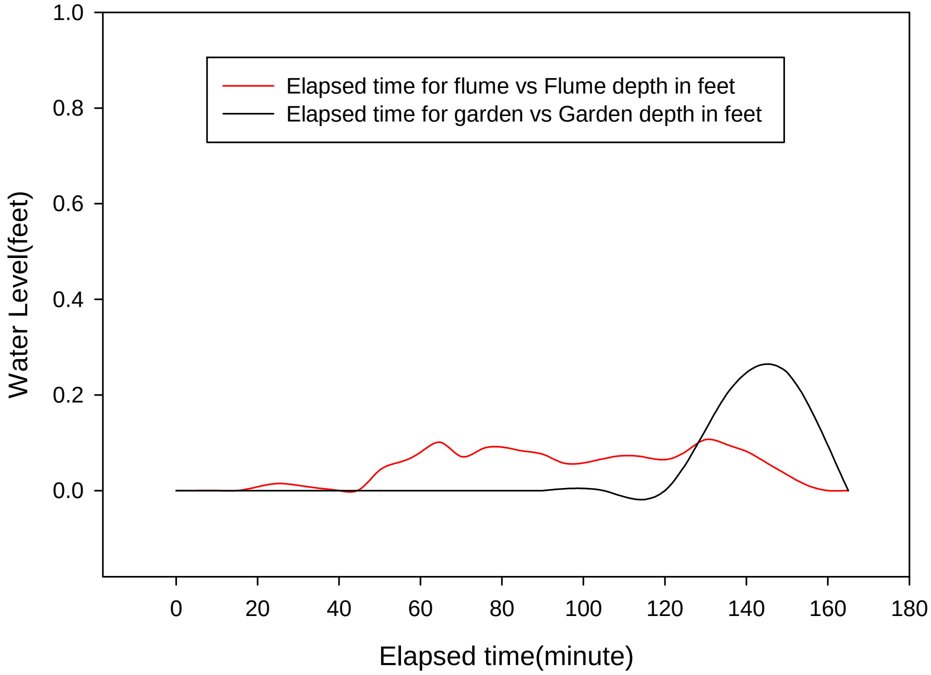
1112 76th terr on Rainevent 04/07/13



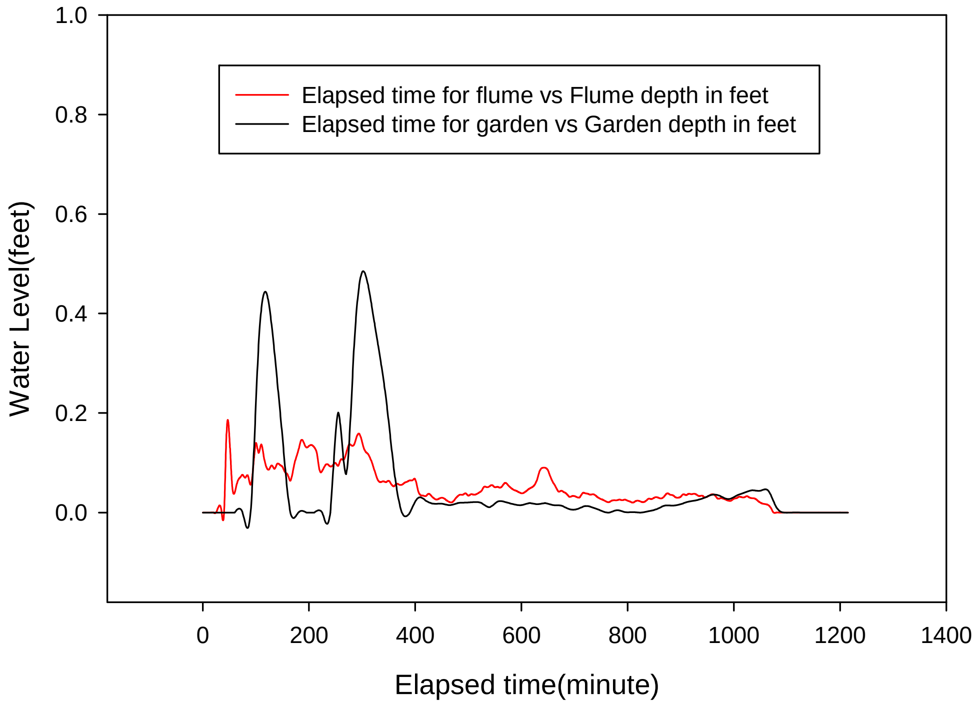
1112 76 terr Raingarden on Rainevent 10/13/2012



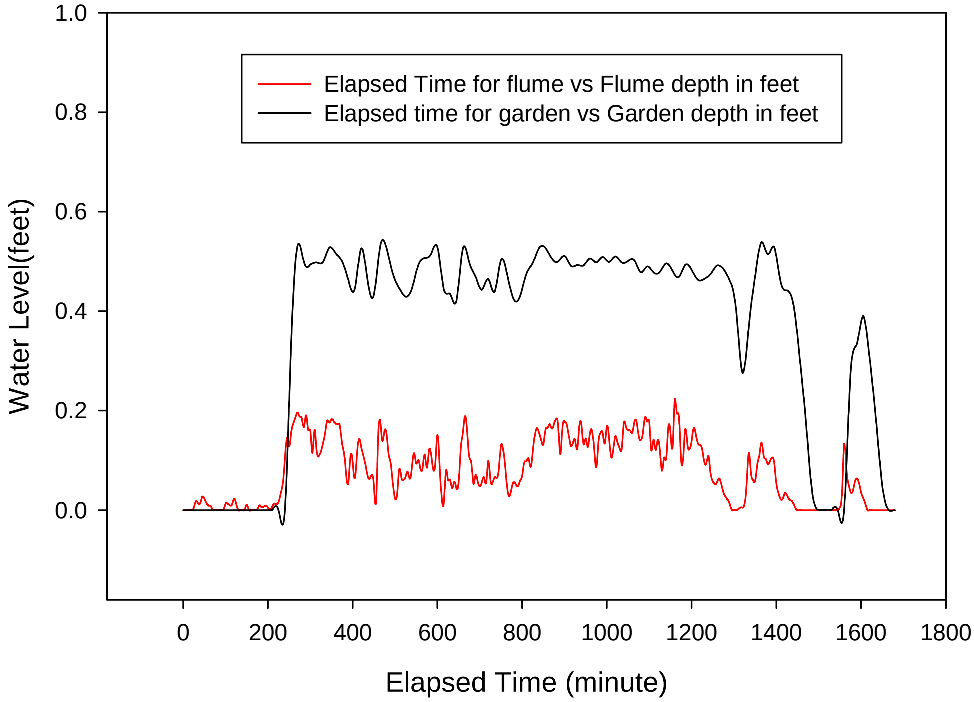
1112 76th terr Raingarden on Rainevent 9/26/2012



1112 76th terr Raingarden on Rainevent 9/13/2012

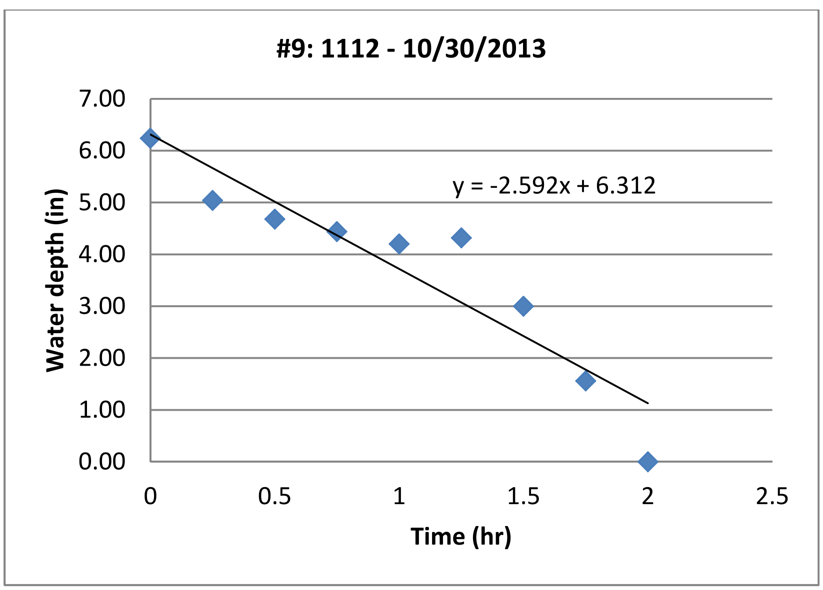


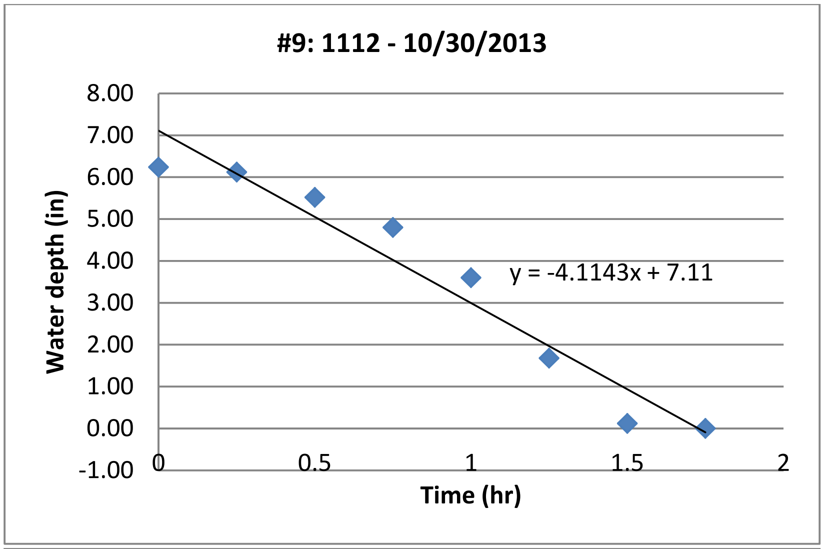
1112 76th terr Raingarden on Rainevent 9/1/2012



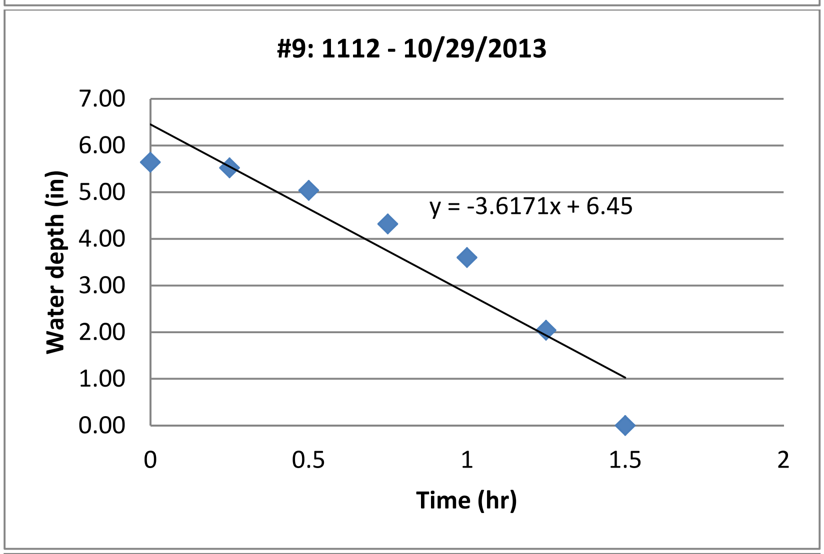
Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:35	6.24	19:05	2.59
3.43	10/30/2013 12:09:00	10/31/2013 11:46:00	23:35	6.24	11:05	4.11
0.83	10/29/2013 2:52:00	10/29/2013 9:41:00	6:49	5.64	4:07	3.62
0.87	10/4/2013 22:36:00	10/5/2013 3:48:00	5:12	5.76	2:07	4.46
0.16	10/3/2013 11:04:00	10/3/2013 11:24:00	00:20	4.92	0:39	5.47
0.28	9/28/2013 8:29:00	9/28/2013 11:29:00	3:00	3.24	0:29	0.67
1.89	9/19/2013 19:00:00	9/19/2013 22:23:00	3:23	4.44	2:43	3.54
0.16	9/1/2013 07:42:00	9/1/2013 09:02:00	1:20	5.40	0:15	1.44
0.39	6/9/2013 00:54:00	6/9/2013 03:53:00	2:59	1.93	5:41	0.84
1.34	5/31/2013 5:26:00	5/31/2013 9:21:00	3:55	9.02	0:55	2.40
1.62	5/29/2013 22:53:00	5/30/2013 15:17:00	16:24	6.54	4:28	3.66
2.01	5/27/2013 8:31:00	5/27/2013 12:35:00	4:04	9.36	2:05	2:23

Rainfall Depth (in.)	Start Time	End Time	Event Duration (hr:min)	Max Water Depth in Garden (in)	Time Duration before Ponding Occurred (hr:min)	f (in/hr)
1.62	4/9/2013 21:56:00	4/10/2013 18:30:00	20:34	7.85	2:01	4.00
1.10	4/7/2013 19:52:00	4/8/2013 2:25:00	6:33	8.40	2:01	3.85
0.86	10/12/2012 00:03:23	10/14/2012 07:48:23	55:45	8.28	25:15	>7.21
					35:15	>3.76
					39:00	>4.69
0.23	9/26/2012 02:08:23	9/26/2012 04:53:23	02:45	2.96	2:15	>4.8
0.43	9/13/2012 14:08:23	9/14/2012 10:23:23	20:15	2.8	1:45	>5.33
					4:15	>4.9
5.60	8/31/2012 11:08:23	9/1/2012 15:08:23	28:00	6.47	4:15	3.85
					26:15	4.99

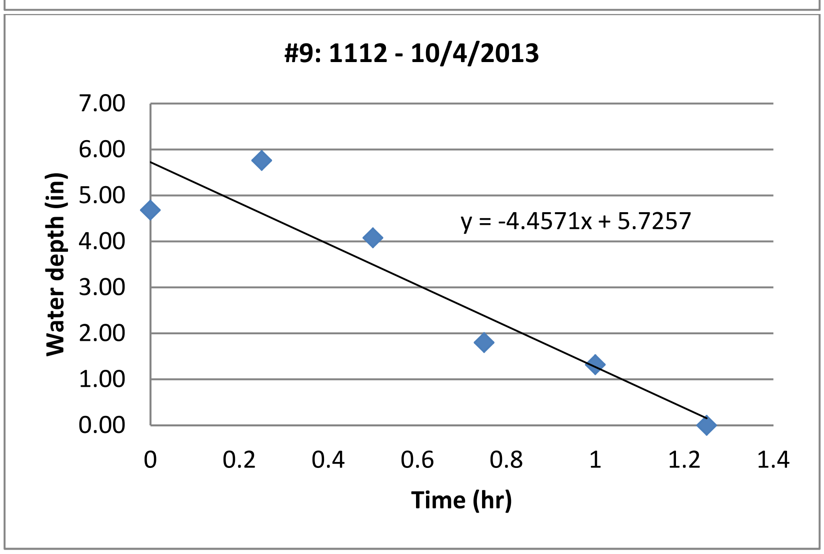




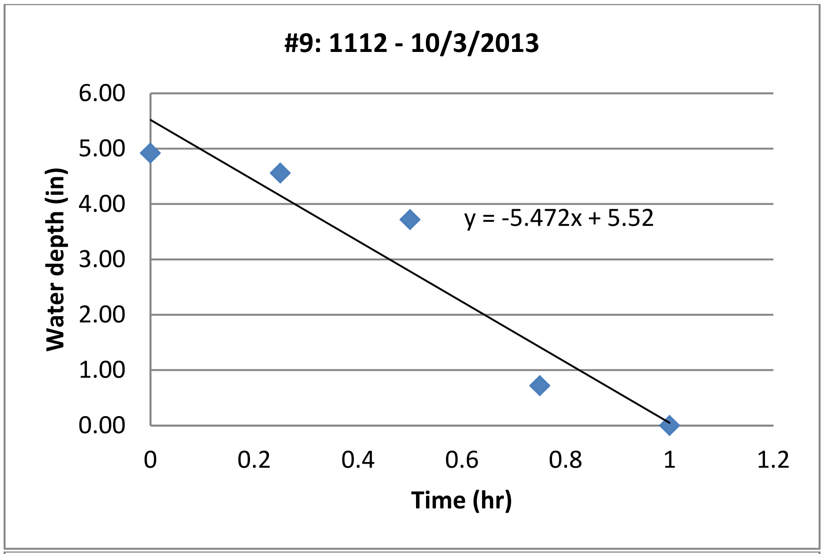
#9: 1112 - 10/30/2013
Water depth (in)
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
0
0.5
1
1.5
2
Time (hr)
y = -4.1143x + 7.11

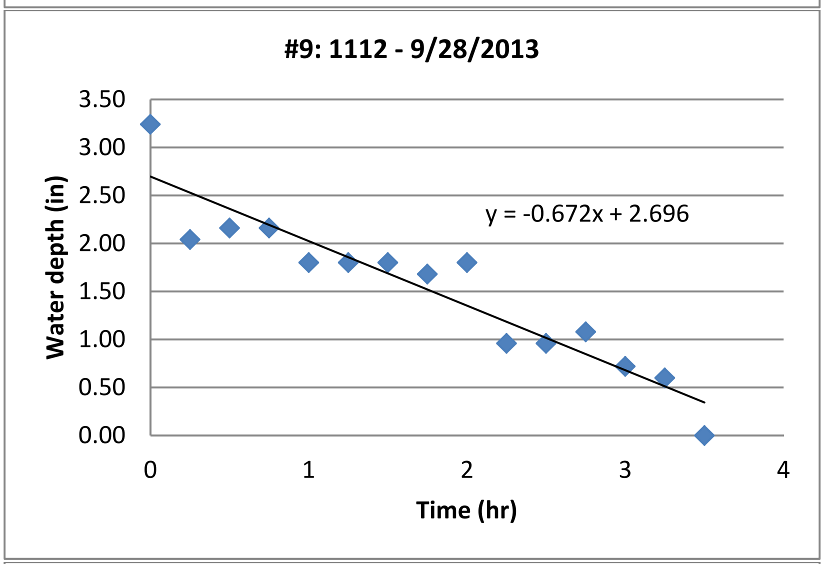


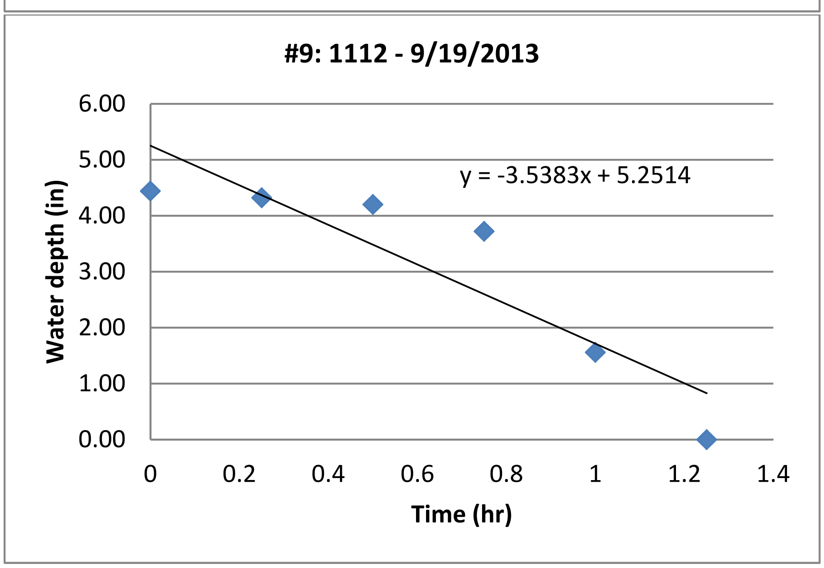
#9: 1112 - 10/29/2013
Water depth (in)
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
0.5
1
1.5
2
Time (hr)
y = -3.6171x + 6.45

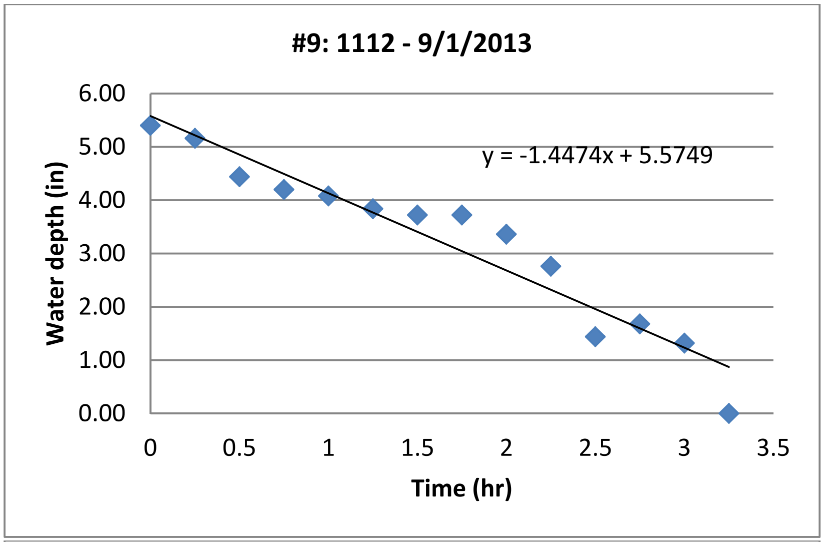


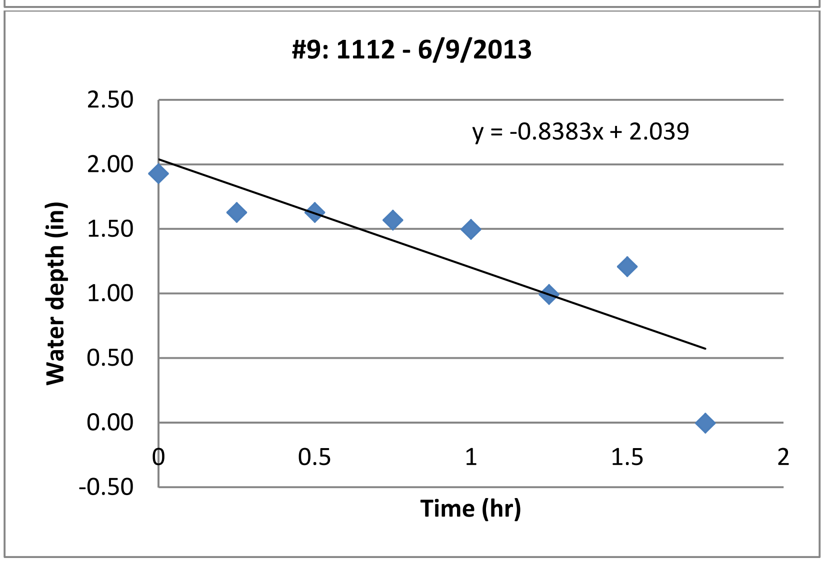
#9: 1112 - 10/4/2013
Water depth (in)
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Time (hr)
y = -4.4571x + 5.7257

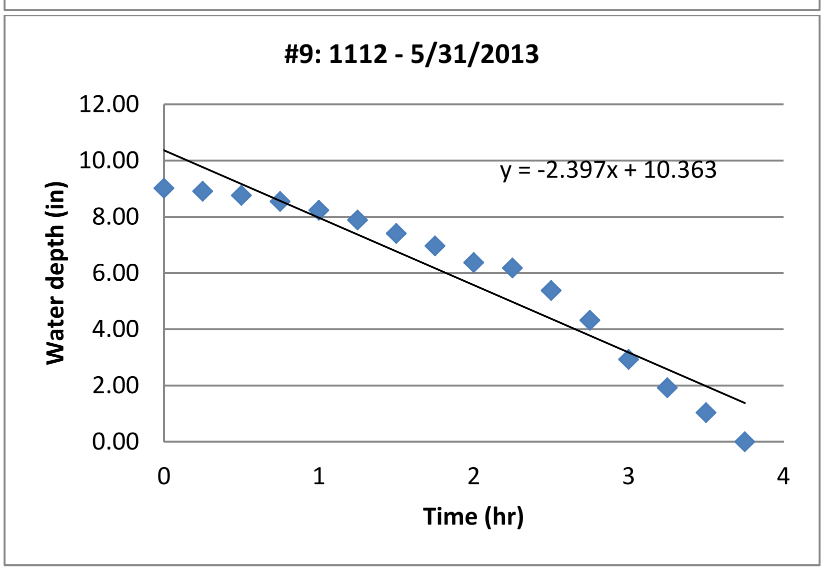

#9: 1112 - 10/3/2013
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Water depth (in)
y = -5.472x + 5.52
0
0.2
0.4
0.6
0.8
1
1.2
Time (hr)

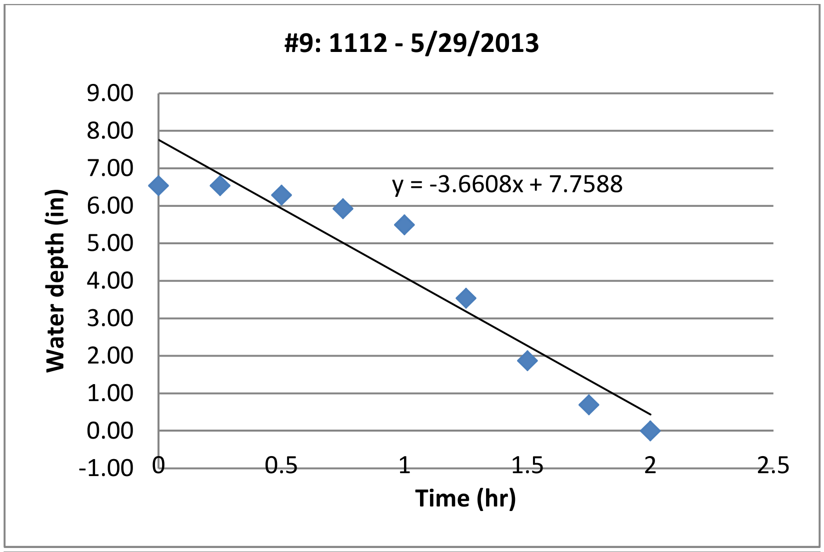

#9: 1112 - 9/28/2013
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Water depth (in)
y = -0.672x + 2.696
0
1
2
3
4
Time (hr)

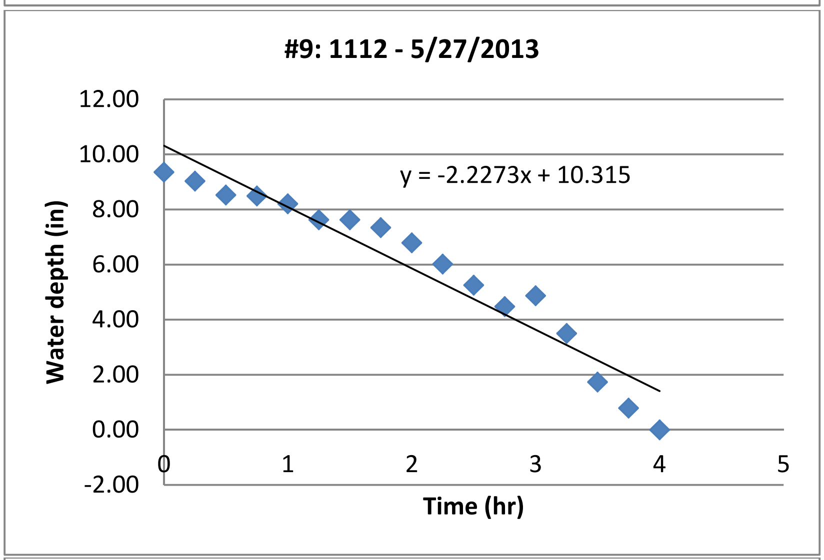

#9: 1112 - 9/19/2013
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Water depth (in)
y = -3.5383x + 5.2514
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Time (hr)

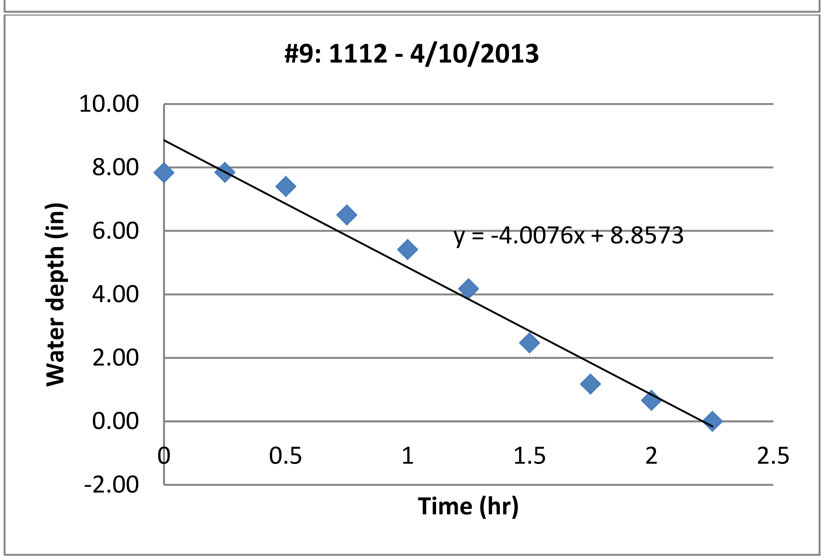

#9: 1112 - 9/1/2013
y = -1.4474x + 5.5749
Water depth (in)
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
0.5
1
1.5
2
2.5
3
3.5
Time (hr)

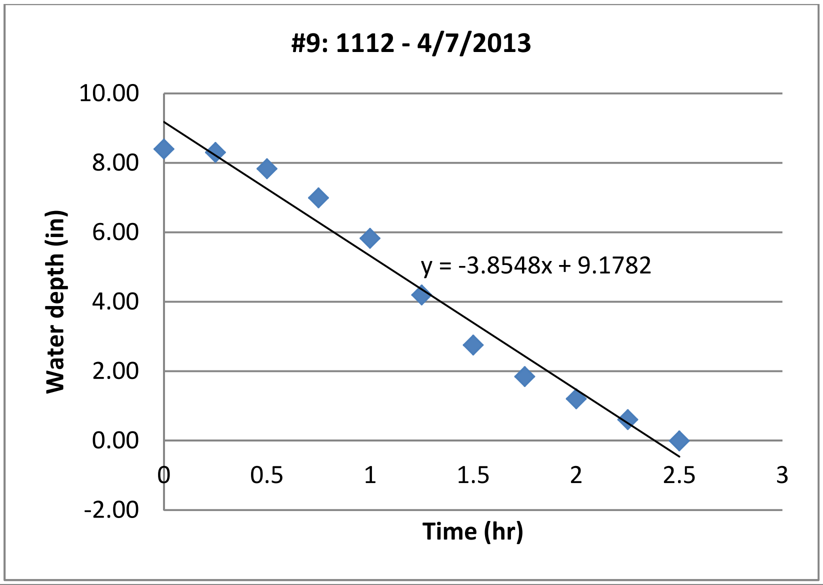

#9: 1112 - 6/9/2013
y = -0.8383x + 2.039
Water depth (in)
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
0
0.5
1
1.5
2
Time (hr)

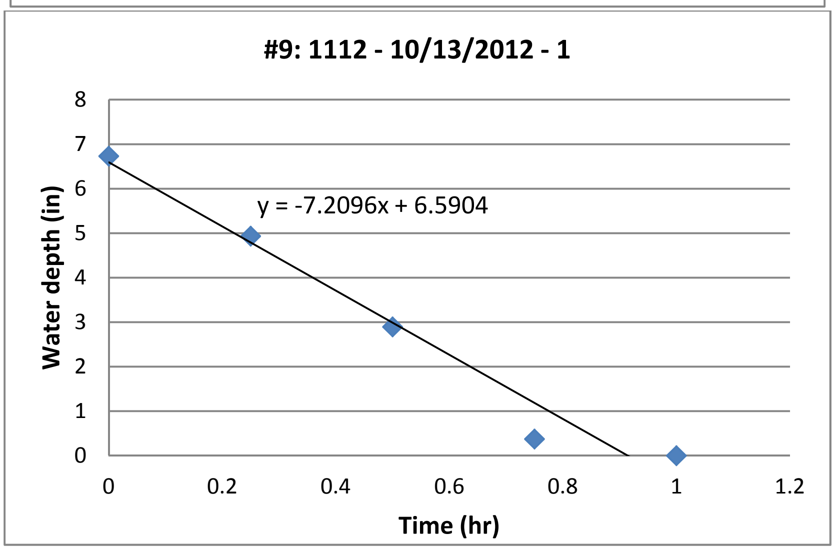

#9: 1112 - 5/31/2013
y = -2.397x + 10.363
Water depth (in)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
1
2
3
4
Time (hr)

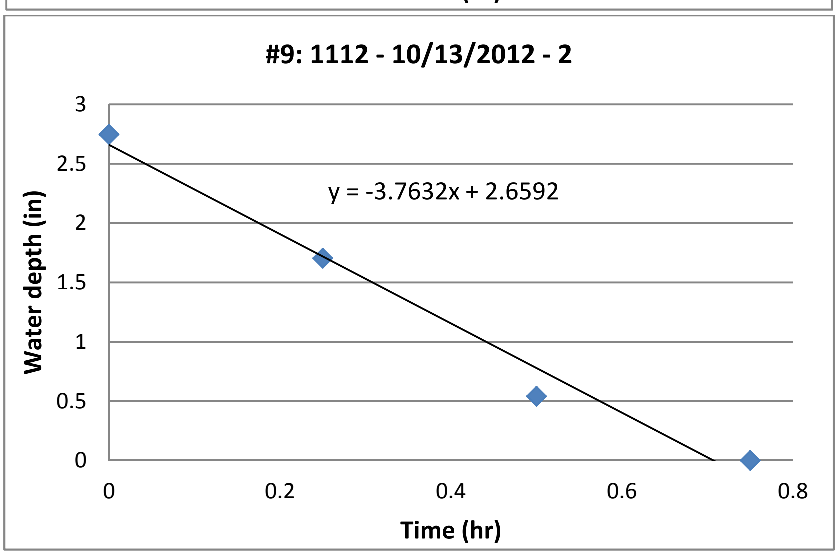

#9: 1112 - 5/29/2013
Water depth (in)
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
y = -3.6608x + 7.7588
0
0.5
1
1.5
2
2.5
Time (hr)

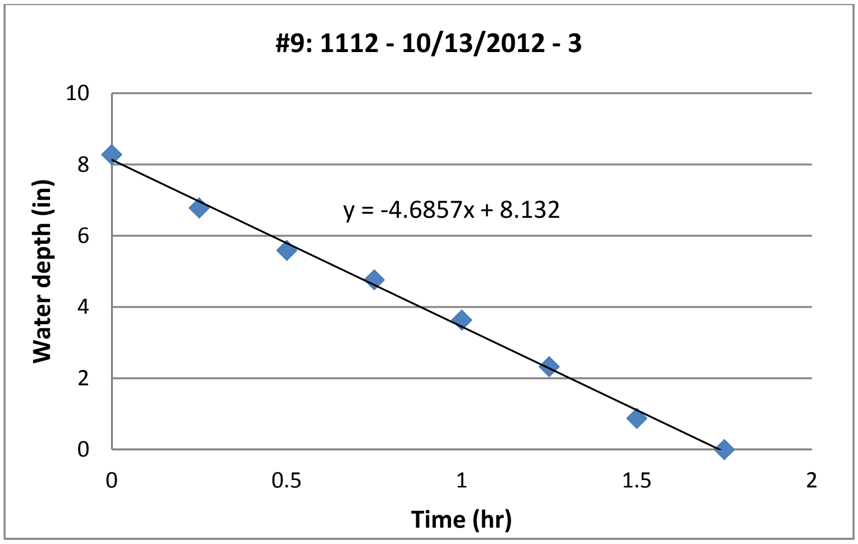

#9: 1112 - 5/27/2013
Water depth (in)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
y = -2.2273x + 10.315
0
1
2
3
4
5
Time (hr)

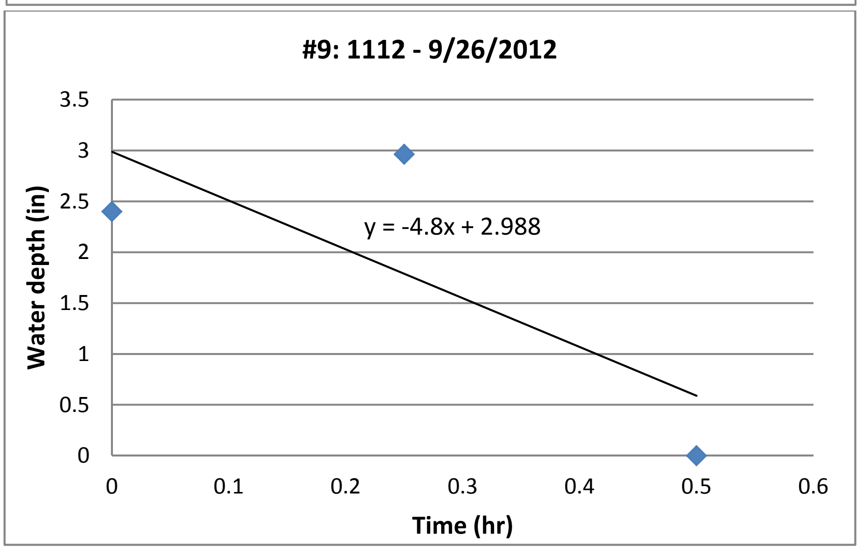

#9: 1112 - 4/10/2013
Water depth (in)
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
y = -4.0076x + 8.8573
0
0.5
1
1.5
2
2.5
Time (hr)

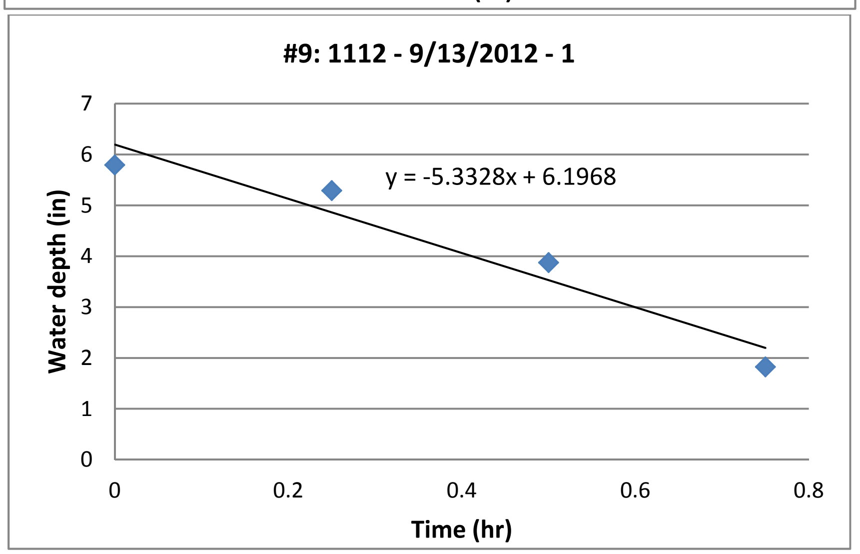

#9: 1112 - 4/7/2013
y = -3.8548x + 9.1782
Water depth (in)
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
0
0.5
1
1.5
2
2.5
3
Time (hr)

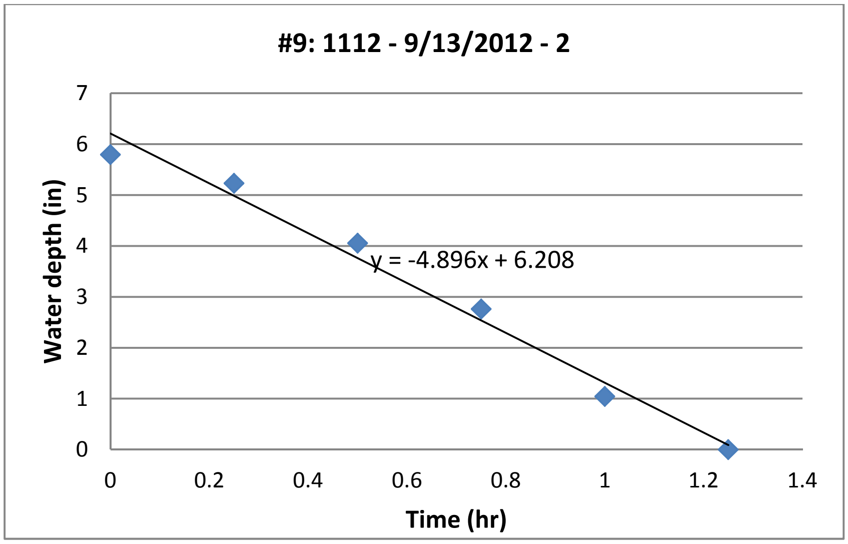

#9: 1112 - 10/13/2012 - 1
y = -7.2096x + 6.5904
Water depth (in)
8
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
1.2
Time (hr)

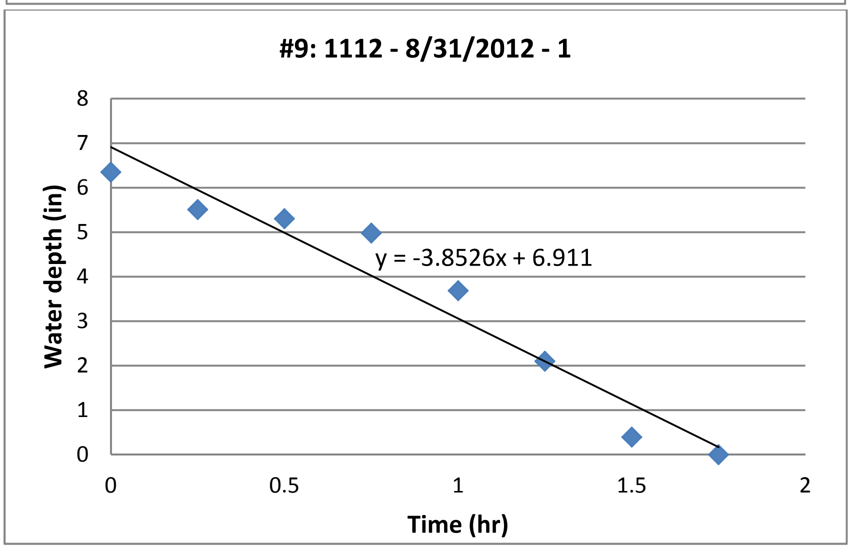

#9: 1112 - 10/13/2012 - 2
y = -3.7632x + 2.6592
Water depth (in)
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
Time (hr)

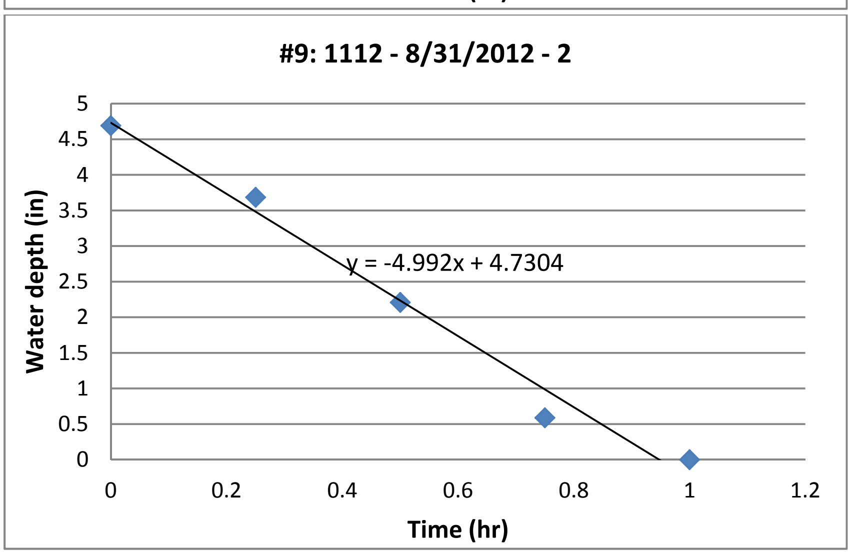

#9: 1112 - 10/13/2012 - 3
Water depth (in)
Time (hr)
y = -4.6857x + 8.132
10
8
6
4
2
0
0
0.5
1
1.5
2


#9: 1112 - 9/26/2012
Water depth (in)
Time (hr)
y = -4.8x + 2.988
3.5
3
2.5
2
1.5
1
0.5
0
0
0.1
0.2
0.3
0.4
0.5
0.6


#9: 1112 - 9/13/2012 - 1
Water depth (in)
Time (hr)
y = -5.3328x + 6.1968
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8


#9: 1112 - 9/13/2012 - 2
Water depth (in)
Time (hr)
y = -4.896x + 6.208
0
1
2
3
4
5
6
7
0
0.2
0.4
0.6
0.8
1
1.2
1.4


#9: 1112 - 8/31/2012 - 1
Water depth (in)
Time (hr)
y = -3.8526x + 6.911
0
1
2
3
4
5
6
7
8
0
0.5
1
1.5
2


#9: 1112 - 8/31/2012 - 2
Water depth (in)
Time (hr)
y = -4.992x + 4.7304
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0
0.2
0.4
0.6
0.8
1
1.2

Appendix D: Large-Scale Combined Sewer Monitoring Data (based on Tetra Tech Compilations from KCMO and UMKC Data)

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Test (Pilot) Area UMKC01 Monitoring Location before Construction of Green Infrastructure Controls (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
Initial Baseline	1	3/23/2009	21:14	3/24/2009	6:57		9.72	0.55	0.47	0.06
	2	3/26/2009	19:55	3/27/2009	2:20	2.54	6.42	0.28	0.95	0.04
	3	3/28/2009	9:19	3/29/2009	15:14	1.29	29.92	0.95	0.95	0.03
	4	3/30/2009	23:23	3/31/2009	1:25	1.34	2.03	0.24	0.95	0.12
	5	4/2/2009	5:53	4/2/2009	12:42	2.19	6.82	0.12	0.47	0.02
	6	4/9/2009	20:05	4/10/2009	14:16	7.31	18.18	0.95	1.42	0.05
	7	4/12/2009	14:07	4/13/2009	8:59	1.99	18.87	0.43	0.47	0.02
	8	4/18/2009	7:17	4/18/2009	12:44	4.93	5.45	0.55	0.47	0.10
	9	4/19/2009	3:15	4/19/2009	6:49	0.60	3.57	0.28	0.47	0.08
	10	4/26/2009	22:48	4/27/2009	13:06	7.67	14.30	2.17	1.42	0.15
	11	4/29/2009	14:51	4/30/2009	21:14	2.07	30.38	2.13	0.95	0.07
	12	5/8/2009	6:00	5/8/2009	9:16	7.37	3.27	0.32	0.47	0.10
	13	5/13/2009	18:07	5/13/2009	19:32	5.37	1.42	0.28	0.47	0.20
	14	5/15/2009	17:14	5/15/2009	22:00	1.90	4.77	1.34	1.90	0.28
	15	6/2/2009	13:12	6/2/2009	19:47	17.63	6.58	0.39	0.47	0.06
	16	6/8/2009	2:38	6/8/2009	3:59	5.29	1.35	0.20	0.95	0.15
	17	6/9/2009	10:37	6/9/2009	23:02	1.28	12.42	2.09	1.42	0.17
	18	6/11/2009	2:42	6/11/2009	5:13	1.15	2.52	0.43	1.42	0.17
	19	6/15/2009	2:38	6/16/2009	7:15	3.89	28.62	2.52	1.42	0.09
	20	6/23/2009	23:33	6/24/2009	1:23	7.68	1.83	0.35	0.95	0.19
	21	7/3/2009	8:24	7/4/2009	6:17	9.29	21.88	1.62	1.42	0.07
	22	7/10/2009	5:13	7/10/2009	6:20	5.96	1.12	0.16	0.47	0.14
	23	7/12/2009	7:58	7/12/2009	18:48	2.07	10.83	0.79	0.95	0.07
	24	7/20/2009	16:47	7/21/2009	3:39	7.92	10.87	0.63	0.95	0.06
	25	7/27/2009	21:38	7/28/2009	16:41	6.75	19.05	1.69	0.95	0.09
	26	8/1/2009	4:02	8/1/2009	6:48	3.47	2.77	0.32	0.95	0.12
	27	8/4/2009	5:59	8/4/2009	8:43	2.97	2.73	0.55	0.47	0.20
	28	8/10/2009	1:19	8/10/2009	3:55	5.69	2.60	0.20	0.47	0.08
	29	8/15/2009	19:42	8/16/2009	10:50	5.66	15.13	2.29	0.95	0.15
	30	8/17/2009	7:39	8/17/2009	12:57	0.87	5.30	1.10	1.42	0.21
	31	8/19/2009	7:00	8/20/2009	1:05	1.75	18.08	0.91	0.95	0.05
	32	8/27/2009	1:31	8/27/2009	6:49	7.02	5.30	0.12	0.47	0.02
	33	9/4/2009	12:04	9/5/2009	6:06	8.22	18.03	0.39	0.47	0.02

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
	34	9/8/2009	17:16	9/9/2009	18:08	3.47	24.87	0.32	0.47	0.01
	35	9/21/2009	10:37	9/21/2009	23:14	11.69	12.62	0.63	0.47	0.05
	36	9/26/2009	1:42	9/26/2009	11:08	4.10	9.43	0.35	0.47	0.04
	37	10/1/2009	4:27	10/1/2009	7:49	4.72	3.37	0.12	0.47	0.04
	38	10/6/2009	2:40	10/6/2009	4:18	4.79	1.63	0.12	0.47	0.07
	39	10/8/2009	0:38	10/8/2009	21:16	1.85	20.63	1.46	0.95	0.07
	40	10/13/2009	17:56	10/14/2009	0:19	4.86	6.38	0.20	0.47	0.03
	41	10/20/2009	5:27	10/20/2009	6:41	6.21	1.23	0.24	0.95	0.19
	42	10/21/2009	15:36	10/22/2009	14:06	1.37	22.50	0.79	0.47	0.04
	43	10/25/2009	14:11	10/26/2009	0:47	3.00	10.60	0.59	0.47	0.06
	44	10/29/2009	6:01	10/29/2009	18:30	3.22	12.48	0.63	0.95	0.05
	45	11/14/2009	23:22	11/17/2009	15:50	16.20	64.47	1.97	1.42	0.03
	46	12/22/2009	21:27	12/24/2009	12:59	35.23	39.53	1.73	1.42	0.04
	47	12/28/2009	4:32	12/28/2009	14:16	3.65	9.73	0.12	0.47	0.01
	48	12/30/2009	6:01	12/30/2009	19:35	1.66	13.57	0.43	1.42	0.03
	49	2/5/2010	9:55	2/6/2010	4:05	36.60	18.17	0.28	0.47	0.02
	50	2/7/2010	19:35	2/8/2010	13:33	1.65	17.97	0.12	0.47	0.01
	51	2/19/2010	6:56	2/19/2010	17:21	10.72	10.42	0.43	0.47	0.04
	52	2/21/2010	5:32	2/22/2010	13:46	1.51	32.23	0.55	1.42	0.02
	53	3/8/2010	20:41	3/9/2010	14:54	14.29	18.22	0.20	0.95	0.01
	54	3/10/2010	18:53	3/11/2010	3:20	1.17	8.45	0.94	1.42	0.11
	55	3/21/2010	12:21	3/21/2010	15:34	10.38	3.22	0.20	1.42	0.06
	56	3/24/2010	12:55	3/24/2010	21:15	2.89	8.33	0.20	0.47	0.02
	57	3/27/2010	8:04	3/27/2010	11:20	2.45	3.27	0.12	0.95	0.04
	58	4/2/2010	9:51	4/2/2010	13:47	5.94	3.93	0.39	1.90	0.10
	59	4/5/2010	7:28	4/5/2010	9:24	2.74	1.93	0.71	0.95	0.37
	60	4/6/2010	20:04	4/7/2010	0:49	1.44	4.75	0.55	0.95	0.12
	61	4/22/2010	10:20	4/23/2010	8:34	15.40	22.23	2.52	1.42	0.11
	62	4/24/2010	11:39	4/25/2010	11:36	1.13	23.95	0.75	0.47	0.03
	63	4/30/2010	7:03	4/30/2010	14:15	4.81	7.20	0.55	0.47	0.08
	64	5/15/2010	7:21	5/16/2010	11:01	14.71	27.67	0.59	0.95	0.02
	65	5/19/2010	12:20	5/20/2010	20:02	3.05	31.70	1.22	0.95	0.04
	66	6/8/2010	8:30	6/9/2010	12:30	18.52	28.00	2.06	0.95	0.07
	67	6/12/2010	10:05	6/12/2010	13:40	2.90	3.58	2.13	0.95	0.59
	68	6/13/2010	7:15	6/14/2010	12:45	0.73	29.50	3.50	1.90	0.12
	69	6/16/2010	18:23	6/16/2010	19:26	2.23	1.05	0.71	1.42	0.67

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
After Relining	70	1/22/2011	12:20	1/22/2011	15:40		3.33	0.12		0.04
	71	2/24/2011	9:00	2/24/2011	15:00	32.58	6.00	0.35	0.47	0.06
	72	2/26/2011	13:50	2/27/2011	20:20	2.15	16.67	1.22	1.42	0.07
	73	3/4/2011	11:10	3/4/2011	13:40	5.08	2.50	0.24	0.95	0.09
	74	3/8/2011	8:10	3/8/2011	13:10	3.65	5.00	0.39	0.47	0.08
	75	3/13/2011	23:00	3/14/2011	12:25	5.07	13.42	0.16	0.47	0.01
	76	3/19/2011	14:30	3/19/2011	16:15	5.69	1.75	0.32	0.95	0.18
	count					74	76	76	75	76
	sum						950	58.53		
	minimum					0.60	1.05	0.12	0.47	0.01
	maximum					36.60	64.47	3.50	1.90	0.67
	average					6.31	12.49	0.77	0.89	0.10
	median					4.00	9.57	0.49	0.95	0.07
	standard deviation					7.23	11.39	0.74	0.42	0.11
	COV					1.15	0.91	0.96	0.47	1.15

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Test (Pilot) Area UMKC01 Monitoring Location before Construction of Green Infrastructure Controls (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
1	3/23/2009	21:40	3/24/2009	17:00	19.33	24,910	0.069	3.81	0.34	0.29	0.12	1.99
2	3/26/2009	12:25	3/27/2009	8:45	20.33	16,603	0.046	1.02	0.25	4.07	0.16	3.17
3	3/28/2009	9:20	3/30/2009	3:00	41.67	167,283	0.461	3.95	0.94	4.20	0.49	1.39
4	3/30/2009	23:20	3/31/2009	13:25	14.08	81,600	0.225	4.95	1.65	3.00	0.94	6.93
5	4/2/2009	5:50	4/2/2009	23:55	18.08	29,159	0.080	0.92	0.45	2.05	0.67	2.65
6	4/9/2009	20:20	4/10/2009	23:55	27.58	99,913	0.275	2.78	0.51	5.45	0.29	1.52
7	4/12/2009	14:05	4/13/2009	17:55	27.83	51,172	0.141	1.46	0.46	3.21	0.33	1.48
8	4/18/2009	7:00	4/19/2009	0:45	17.75	28,145	0.078	5.31	0.44	12.07	0.14	3.26
9	4/19/2009	3:25	4/19/2009	19:00	15.58	34,316	0.095	5.70	0.61	9.34	0.34	4.37
10	4/26/2009	22:55	4/29/2009	2:15	51.33	281,137	0.774	10.66	1.52	7.02	0.36	3.59
11	4/29/2009	14:50	4/30/2009	9:15	18.42	171,573	0.473	12.83	1.54	8.31	0.22	0.61
12	5/8/2009	5:30	5/8/2009	21:15	15.75	17,772	0.049	2.08	0.31	6.71	0.15	4.82
13	5/13/2009	18:10	5/13/2009	23:55	5.75	7,971	0.022	10.63	0.70	15.19	0.08	4.06
14	5/15/2009	17:15	5/16/2009	10:00	16.75	60,959	0.168	17.14	1.05	16.32	0.13	3.51

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
15	6/2/2009	13:25	6/3/2009	7:00	17.58	52,209	0.144	12.57	0.91	13.80	0.37	2.67
16	6/8/2009	2:25	6/8/2009	4:50	2.42	3,430	0.009	1.79	0.38	4.69	0.05	1.79
17	6/9/2009	11:20	9/10/2009	12:05	2232.75	186,479	0.514	39.29	2.09	18.80	0.25	179.82
18	6/11/2009	2:40	6/11/2009	17:00	14.33	62,008	0.171	22.01	1.19	18.50	0.40	5.70
19	6/15/2009	5:10	6/16/2009	19:15	38.08	498,449	1.373	32.10	3.63	8.84	0.54	1.33
20	6/23/2009	23:20	6/24/2009	13:20	14.00	8,229	0.023	2.85	0.16	17.81	0.06	7.64
21	7/3/2009	8:25	7/4/2009	18:15	33.83	103,103	0.284	2.59	0.32	8.03	0.18	1.55
22	7/10/2009	5:05	7/10/2009	8:45	3.67	5,482	0.015	3.25	0.40	8.13	0.09	3.28
23	7/12/2009	7:55	7/13/2009	6:00	22.08	57,620	0.159	31.16	0.75	41.55	0.20	2.04
24	7/20/2009	16:30	7/21/2009	14:50	22.33	30,191	0.083	3.56	0.39	9.12	0.13	2.06
25	7/27/2009	21:40	7/29/2009	4:40	31.00	59,273	0.163	15.02	0.53	28.34	0.10	1.63
26	8/1/2009	4:00	8/1/2009	18:45	14.75	15,742	0.043	2.34	0.29	8.07	0.14	5.33
27	8/4/2009	6:00	8/4/2009	15:50	9.83	16,568	0.046	10.29	0.43	23.93	0.08	3.60
28	8/10/2009	1:40	8/10/2009	15:30	13.83	8,562	0.024	4.77	0.17	28.03	0.12	5.32
29	8/15/2009	19:50	8/16/2009	22:50	27.00	123,747	0.341	13.20	1.22	10.82	0.15	1.78
30	8/17/2009	7:05	8/18/2009	1:00	17.92	88,033	0.243	28.15	1.36	20.70	0.22	3.38
31	8/19/2009	7:00	8/20/2009	12:00	29.00	81,969	0.226	12.04	0.76	15.84	0.25	1.60
32	8/27/2009	1:55	8/27/2009	15:00	13.08	9,272	0.026	0.70	0.19	3.68	0.21	2.47
33	9/4/2009	10:00	9/5/2009	5:50	19.83	17,945	0.049	2.79	0.25	11.16	0.13	1.10
34	9/8/2009	15:15	9/9/2009	22:00	30.75	18,026	0.050	1.92	0.15	12.80	0.16	1.24
35	9/21/2009	10:15	9/22/2009	7:00	20.75	33,845	0.093	2.71	0.45	6.01	0.15	1.64
36	9/26/2009	0:30	9/26/2009	17:45	17.25	15,980	0.044	3.89	0.26	14.97	0.13	1.83
37	10/1/2009	3:45	10/1/2009	17:30	13.75	4,233	0.012	0.58	0.08	7.25	0.10	4.08
38	10/6/2009	2:25	10/6/2009	10:00	7.58	3,432	0.009	1.00	0.16	6.25	0.08	4.64
39	10/8/2009	2:20	10/9/2009	9:00	30.67	102,567	0.283	7.94	0.86	9.23	0.19	1.49
40	10/13/2009	17:25	10/14/2009	12:00	18.58	18,413	0.051	2.36	0.30	7.87	0.25	2.91
41	10/20/2009	5:30	10/20/2009	12:40	7.17	11,219	0.031	5.86	0.40	14.65	0.13	5.81
42	10/21/2009	15:45	10/23/2009	2:25	34.67	89,206	0.246	2.49	0.57	4.37	0.31	1.54
43	10/25/2009	14:00	10/26/2009	2:10	12.17	54,858	0.151	4.35	0.90	4.84	0.26	1.15
44	10/29/2009	5:50	10/29/2009	11:25	5.58	55,455	0.153	6.80	0.62	10.97	0.24	0.45
45	11/14/2009	22:30	11/18/2009	5:00	78.50	83,045	0.229	1.37	0.29	4.72	0.12	1.22
46	12/23/2009	0:00	12/24/2009	23:00	47.00	67,248	0.185	4.68	0.43	10.88	0.11	1.19
47	12/28/2009	4:35	12/28/2009	22:15	17.67	9,612	0.026	0.28	0.13	2.10	0.22	1.82
48	12/30/2009	5:55	12/30/2009	23:55	18.00	8,385	0.023	0.22	0.12	1.83	0.05	1.33
49	2/5/2010	10:25	2/5/2010	23:55	13.50	10,497	0.029	0.33	0.09	3.67	0.10	0.74
50	2/7/2010	11:25	2/7/2010	0:55	13.50	13,563	0.037	0.33	0.13	2.54	0.31	0.75

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
51	2/19/2010	6:55	2/20/2010	1:25	18.50	22,778	0.063	1.55	0.34	4.57	0.15	1.78
52	2/21/2010	5:15	2/22/2010	23:50	42.58	93,882	0.259	2.66	0.77	3.44	0.47	1.32
53	3/8/2010	20:45	3/9/2010	4:55	8.17	8,546	0.024	0.95	0.10	9.50	0.12	0.45
54	3/10/2010	19:00	3/11/2010	14:20	19.33	49,894	0.137	6.13	0.68	9.02	0.15	2.29
55	3/21/2010	12:00	3/22/2010	1:40	13.67	9,836	0.027	0.44	0.21	2.10	0.14	4.25
56	3/24/2010	12:00	3/25/2010	9:00	21.00	17,628	0.049	0.91	0.23	3.96	0.24	2.52
57	3/27/2010	7:20	3/27/2010	22:00	14.67	5,113	0.014	0.78	0.19	4.11	0.12	4.49
58	4/2/2010	9:55	4/2/2010	20:00	10.08	14,035	0.039	5.53	0.39	14.30	0.10	2.56
59	4/5/2010	7:30	4/5/2010	22:00	14.50	54,142	0.149	18.91	1.03	18.36	0.21	7.50
60	4/6/2010	20:10	4/7/2010	11:40	15.50	98,750	0.272	9.47	1.76	5.38	0.49	3.26
61	4/22/2010	10:25	4/23/2010	19:35	33.17	214,933	0.592	10.01	1.78	5.63	0.23	1.49
62	4/24/2010	11:15	4/25/2010	20:30	33.25	94,125	0.259	2.07	0.78	2.65	0.35	1.39
63	4/30/2010	6:30	4/30/2010	23:50	17.33	40,157	0.111	11.39	0.64	17.78	0.20	2.41
64	5/15/2010	7:15	5/16/2010	20:00	36.75	91,855	0.253	2.44	0.63	3.87	0.43	1.33
65	5/19/2010	12:20	5/20/2010	7:00	18.67	169,204	0.466	9.44	1.07	8.82	0.38	0.59
66	6/8/2010	8:50	6/9/2010	8:20	23.50	92,351	0.254	27.25	1.09	25.00	0.12	0.84
67	6/12/2010	10:15	6/13/2010	1:40	15.42	251,228	0.692	47.13	4.68	10.07	0.32	4.30
68	6/13/2010	7:10	6/15/2010	0:05	40.92	780,680	2.151	106.67	5.29	20.16	0.61	1.39
69	6/16/2010	18:25	6/17/2010	6:40	12.25	109,484	0.302	29.04	2.45	11.85	0.43	11.67
70												
71	2/24/2011	9:00	2/25/2011	3:00	9.00	49,932	0.138	1.98	0.73	2.72	0.39	1.50
72	2/26/2011	13:50	2/28/2011	8:00	28.33	146,655	0.404	16.83	0.88	19.12	0.33	1.70
73	3/4/2011	11:15	3/4/2011	22:45	11.50	21,625	0.060	6.14	0.47	13.06	0.25	4.60
74	3/8/2011	8:10	3/9/2011	1:10	8.83	23,355	0.064	2.06	0.41	5.02	0.16	1.77
75	3/13/2011	22:30	3/14/2011	20:15	21.75	11,802	0.033	0.66	0.12	5.50	0.21	1.62
76	3/19/2011	13:30	3/19/2011	22:10	8.67	12,464	0.034	2.44	0.38	6.42	0.11	4.95
	count				75	75	75	75	75	75	75	75
	sum				3,772	5,584,861	15.39					
	minimum				2.42	3,430	0.01	0.22	0.08	0.29	0.05	0.45
	maximum				2232.75	780,680	2.15	106.67	5.29	41.55	0.94	179.82
	average				50.29	74,465	0.21	9.25	0.79	10.06	0.24	5.11
	median				17.92	40,157	0.11	3.89	0.46	8.13	0.20	1.99
	standard deviation				255.71	114,061	0.31	15.03	0.93	7.51	0.16	20.54
	COV				5.08	1.53	1.53	1.62	1.17	0.75	0.67	4.02

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Test (Pilot) Area UMKC01 Monitoring Location after Construction of Green Infrastructure Controls (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
After Construction	77	4/7/2013	19:52	4/8/2013	2:25		6.55	1.10	1.42	0.17
	78	4/9/2013	21:56	4/10/2013	18:30	1.62	20.57	1.62	1.42	0.08
	79	4/14/2013	19:02	4/15/2013	7:08	4.02	12.10	0.39	0.47	0.03
	80	4/17/2013	11:00	4/18/2013	20:41	2.16	33.68	1.10	0.47	0.03
	81	4/23/2013	1:34	4/23/2013	10:53	4.20	9.32	0.39	0.47	0.04
	82	4/26/2013	4:19	4/27/2013	11:02	2.73	30.72	0.95	0.95	0.03
	83	5/2/2013	3:08	5/4/2013	4:11	4.67	49.05	1.42	0.47	0.03
	84	5/8/2013	7:21	5/9/2013	2:36	4.13	19.25	0.28	0.47	0.01
	85	5/19/2013	2:21	5/20/2013	1:44	9.99	23.38	0.83	1.42	0.04
	86	5/27/2013	8:31	5/27/2013	12:35	7.28	4.07	2.01	1.90	0.49
	87	5/29/2013	22:53	5/30/2013	15:17	2.43	16.40	1.62	1.42	0.10
	88	5/31/2013	5:26	5/31/2013	9:21	0.59	3.92	1.34	1.90	0.34
	89	6/4/2013	10:55	6/4/2013	14:19	4.07	3.40	0.24	0.47	0.07
	90	6/5/2013	9:44	6/5/2013	12:47	0.81	3.05	0.47	0.47	0.15
	91	6/9/2013	0:54	6/9/2013	3:53	3.50	2.98	0.39	0.95	0.13
	92	6/15/2013	15:50	6/15/2013	22:05	6.50	6.25	1.26	1.42	0.20
	93	6/27/2013	11:46	6/28/2013	0:49	11.57	13.05	0.83	1.90	0.06
	94	7/3/2013	17:54	7/3/2013	21:36	5.71	3.70	1.50	0.95	0.41
	95	7/25/2013	17:00	7/26/2013	11:00	21.81	18.00	0.24	0.47	0.01
	96	7/29/2013	6:27	7/30/2013	9:41	2.81	27.23	0.91	0.95	0.03
	97	8/2/2013	3:46	8/2/2013	8:10	2.75	4.40	0.32	0.47	0.07
	98	8/4/2013	11:38	8/4/2013	13:58	2.14	2.33	0.12	0.47	0.05
	99	8/6/2013	3:24	8/6/2013	4:59	1.56	1.58	0.55	1.42	0.35
	100	8/7/2013	4:13	8/7/2013	8:45	0.97	4.53	0.87	0.95	0.19
	101	8/12/2013	7:07	8/12/2013	10:18	4.93	3.18	0.67	1.42	0.21
	102	9/1/2013	7:42	9/1/2013	9:02	19.89	1.33	0.16	0.95	0.12
	103	9/16/2013	2:13	9/16/2013	4:10	14.72	1.95	0.12	0.95	0.06
	104	9/17/2013	7:04	9/17/2013	15:14	1.12	8.17	0.79	0.95	0.10
	105	9/19/2013	19:00	9/19/2013	22:23	2.16	3.38	1.89	2.85	0.56
	106	9/28/2013	8:29	9/28/2013	11:29	8.42	3.00	0.28	0.47	0.09
	107	10/3/2013	11:04	10/3/2013	11:24	4.98	0.33	0.16	0.47	0.47
	108	10/4/2013	22:36	10/5/2013	3:48	1.47	5.20	0.87	0.47	0.17
	109	10/11/2013	23:08	10/12/2013	2:29	6.81	3.35	0.12	0.47	0.04
	110	10/14/2013	15:10	10/15/2013	1:23	2.53	10.22	0.12	0.47	0.01

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
	111	10/18/2013	14:03	10/18/2013	17:05	3.53	3.03	0.12	0.47	0.04
	112	10/29/2013	2:52	10/29/2013	9:41	10.41	6.82	0.83	0.95	0.12
	113	10/30/2013	12:09	10/31/2013	11:46	1.10	23.62	3.43	1.42	0.15
	count					36	37	37	37	37
	sum						393	30.29		
	minimum					0.59	0.33	0.12	0.47	0.01
	maximum					21.81	49.05	3.43	2.85	0.56
	average					5.28	10.62	0.82	0.97	0.14
	median					3.78	5.20	0.79	0.95	0.09
	standard deviation					5.06	11.12	0.70	0.57	0.15
	COV					0.96	1.05	0.86	0.59	1.03

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Test (Pilot) Area UMKC01 Monitoring Location after Construction of Green Infrastructure Controls (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
77	4/7/2013	19:50	4/8/2013	9:45	13.92	66,609	0.183	19.33	1.44	13.42	0.17	2.12
78	4/9/2013	21:45	4/11/2013	6:30	32.75	137,860	0.380	13.28	1.19	11.16	0.23	1.59
79	4/14/2013	19:15	4/15/2013	17:20	22.08	7,939	0.022	1.26	0.13	9.69	0.06	1.83
80	4/17/2013	11:00	4/19/2013	8:20	45.33	131,126	0.361	4.72	0.80	5.90	0.33	1.35
81	4/23/2013	1:30	4/23/2013	22:00	20.50	27,952	0.077	0.96	0.43	2.23	0.20	2.20
82	4/26/2013	4:25	4/27/2013	23:00	42.58	64,499	0.178	1.51	0.43	3.51	0.19	1.39
83	5/2/2013	2:10	5/4/2013	13:00	58.83	149,222	0.411	1.51	0.70	2.16	0.29	1.20
84	5/8/2013	7:20	5/9/2013	13:00	29.67	19,890	0.055	2.45	0.21	11.67	0.20	1.54
85	5/19/2013	3:20	5/20/2013	13:00	33.67	57,570	0.159	8.01	0.47	17.04	0.19	1.44
86	5/27/2013	8:35	5/27/2013	23:50	15.25	210,046	0.579	36.40	3.81	9.55	0.29	3.75
87	5/29/2013	23:05	5/31/2013	3:00	27.92	124,377	0.343	5.54	1.28	4.33	0.21	1.70
88	5/31/2013	5:00	5/31/2013	21:15	16.25	199,276	0.549	26.37	3.55	7.43	0.41	4.15
89	6/4/2013	10:45	6/4/2013	21:05	10.33	7,607	0.021	0.84	0.20	4.20	0.09	3.04
90	6/5/2013	9:00	6/5/2013	22:50	13.83	12,604	0.035	2.47	0.26	9.50	0.07	4.54
91	6/9/2013	0:30	6/9/2013	15:05	14.58	17,051	0.047	3.14	0.32	9.81	0.12	4.89

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
92	6/15/2013	15:55	6/16/2013	9:00	17.08	71,491	0.197	12.57	1.16	10.84	0.16	2.73
93	6/27/2013	11:50	6/28/2013	11:00	23.17	60,903	0.168	19.07	0.73	26.12	0.20	1.78
94	7/3/2013	17:55	7/4/2013	6:55	13.00	30,333	0.084	11.24	0.64	17.56	0.06	3.51
95	7/25/2013	17:05	7/26/2013	13:15	20.17	3,066	0.008	0.05	0.01	5.00	0.04	1.12
96	7/29/2013	6:30	7/30/2013	20:00	37.50	51,803	0.143	7.40	0.38	19.47	0.16	1.38
97	8/2/2013	3:00	8/2/2013	20:30	0.83	13,547	0.037	1.23	0.21	5.86	0.12	0.19
98	8/4/2013	11:40	8/4/2013	19:20	7.67	3,923	0.011	0.58	0.14	4.14	0.09	3.29
99	8/6/2013	3:45	8/6/2013	6:45	3.00	8,752	0.024	4.96	0.79	6.28	0.04	1.89
100	8/7/2013	4:20	8/7/2013	13:45	9.42	23,841	0.066	6.87	0.70	9.81	0.08	2.08
101	8/12/2013	7:10	8/12/2013	20:40	13.50	35,350	0.097	11.82	0.76	15.55	0.15	4.24
102	9/1/2013	7:40	9/1/2013	16:05	8.42	6,846	0.019	3.54	0.50	7.08	0.12	6.31
103	9/16/2013	0:30	9/16/2013	6:05	5.58	1,155	0.003	0.39	0.06	6.50	0.03	2.86
104	9/17/2013	8:05	9/17/2013	23:55	15.83	22,378	0.062	2.27	0.39	5.82	0.08	1.94
105	9/19/2013	19:00	9/20/2013	12:00	17.00	93,701	0.258	2.92	0.06	48.67	0.14	5.02
106	9/28/2013	8:05	9/28/2013	23:50	15.75	17,316	0.048	3.02	0.30	10.07	0.17	5.25
107	10/3/2013	11:00	10/3/2013	13:40	2.67	5,226	0.014	3.37	0.53	6.36	0.09	8.00
108	10/4/2013	22:00	10/5/2013	9:35	11.58	16,374	0.045	4.54	0.39	11.64	0.05	2.23
109	10/11/2013	23:00	10/12/2013	7:45	8.75	2,687	0.007	1.92	0.08	24.00	0.06	2.61
110	10/14/2013	15:30	10/15/2013	6:30	15.00	2,628	0.007	0.48	0.05	9.60	0.06	1.47
111	10/18/2013	14:00	10/18/2013	22:15	8.25	3,198	0.009	0.53	0.11	4.82	0.07	2.72
112	10/29/2013	3:05	10/29/2013	22:00	18.92	33,023	0.091	2.11	0.48	4.40	0.11	2.78
113	10/30/2013	12:15	10/31/2013	23:45	35.50	219,477	0.605	18.02	1.71	10.54	0.18	1.50
	count				37	37	37	37	37	37	37	37
	sum				706	1,960,646	5.40					
	minimum				0.83	1,155	0.00	0.05	0.01	2.16	0.03	0.19
	maximum				58.83	219,477	0.60	36.40	3.81	48.67	0.41	8.00
	average				19.08	52,990	0.15	6.67	0.69	10.59	0.14	2.75
	median				15.75	23,841	0.07	3.14	0.43	9.55	0.12	2.20
	standard deviation				12.91	62,642	0.17	8.15	0.83	8.56	0.09	1.64
	COV				0.68	1	1.18	1.22	1.22	0.81	0.62	0.60

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Control Area UMKC02a Monitoring Location (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
Initial Baseline	1	3/23/2009	21:14	3/24/2009	6:57		9.72	0.55	0.47	0.06
	2	3/26/2009	19:55	3/27/2009	2:20	2.54	6.42	0.28	0.95	0.04
	3	3/28/2009	9:19	3/29/2009	15:14	1.29	29.92	0.95	0.95	0.03
	4	3/30/2009	23:23	3/31/2009	1:25	1.34	2.03	0.24	0.95	0.12
	5	4/2/2009	5:53	4/2/2009	12:42	2.19	6.82	0.12	0.47	0.02
	6	4/9/2009	20:05	4/10/2009	14:16	7.31	18.18	0.95	1.42	0.05
	7	4/12/2009	14:07	4/13/2009	8:59	1.99	18.87	0.43	0.47	0.02
	8	4/18/2009	7:17	4/18/2009	12:44	4.93	5.45	0.55	0.47	0.10
	9	4/19/2009	3:15	4/19/2009	6:49	0.60	3.57	0.28	0.47	0.08
	10	4/26/2009	22:48	4/27/2009	13:06	7.67	14.30	2.17	1.42	0.15
	11	4/29/2009	14:51	4/30/2009	21:14	2.07	30.38	2.13	0.95	0.07
	12	5/8/2009	6:00	5/8/2009	9:16	7.37	3.27	0.32	0.47	0.10
	13	5/13/2009	18:07	5/13/2009	19:32	5.37	1.42	0.28	0.47	0.20
	14	5/15/2009	17:14	5/15/2009	22:00	1.90	4.77	1.34	1.90	0.28
	15	6/2/2009	13:12	6/2/2009	19:47	17.63	6.58	0.39	0.47	0.06
	16	6/8/2009	2:38	6/8/2009	3:59	5.29	1.35	0.20	0.95	0.15
	17	6/9/2009	10:37	6/9/2009	23:02	1.28	12.42	2.09	1.42	0.17
	18	6/11/2009	2:42	6/11/2009	5:13	1.15	2.52	0.43	1.42	0.17
	19	6/15/2009	2:38	6/16/2009	7:15	3.89	28.62	2.52	1.42	0.09
	20	6/23/2009	23:33	6/24/2009	1:23	7.68	1.83	0.35	0.95	0.19
	21	7/3/2009	8:24	7/4/2009	6:17	9.29	21.88	1.62	1.42	0.07
	22	7/10/2009	5:13	7/10/2009	6:20	5.96	1.12	0.16	0.47	0.14
	23	7/12/2009	7:58	7/12/2009	18:48	2.07	10.83	0.79	0.95	0.07
	24	7/20/2009	16:47	7/21/2009	3:39	7.92	10.87	0.63	0.95	0.06
	25	7/27/2009	21:38	7/28/2009	16:41	6.75	19.05	1.69	0.95	0.09
	26	8/1/2009	4:02	8/1/2009	6:48	3.47	2.77	0.32	0.95	0.12
	27	8/4/2009	5:59	8/4/2009	8:43	2.97	2.73	0.55	0.47	0.20
	28	8/10/2009	1:19	8/10/2009	3:55	5.69	2.60	0.20	0.47	0.08
	29	8/15/2009	19:42	8/16/2009	10:50	5.66	15.13	2.29	0.95	0.15
	30	8/17/2009	7:39	8/17/2009	12:57	0.87	5.30	1.10	1.42	0.21
	31	8/19/2009	7:00	8/20/2009	1:05	1.75	18.08	0.91	0.95	0.05
	32	8/27/2009	1:31	8/27/2009	6:49	7.02	5.30	0.12	0.47	0.02
	33	9/4/2009	12:04	9/5/2009	6:06	8.22	18.03	0.39	0.47	0.02
	34	9/8/2009	17:16	9/9/2009	18:08	3.47	24.87	0.32	0.47	0.01

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
	35	**9/21/2009**	**10:37**	**9/21/2009**	**23:14**	**11.69**	**12.62**	**0.63**	**0.47**	**0.05**
	36	**9/26/2009**	**1:42**	**9/26/2009**	**11:08**	**4.10**	**9.43**	**0.35**	**0.47**	**0.04**
	37	**10/1/2009**	**4:27**	**10/1/2009**	**7:49**	**4.72**	**3.37**	**0.12**	**0.47**	**0.04**
	38	**10/6/2009**	**2:40**	**10/6/2009**	**4:18**	**4.79**	**1.63**	**0.12**	**0.47**	**0.07**
	39	**10/8/2009**	**0:38**	**10/8/2009**	**21:16**	**1.85**	**20.63**	**1.46**	**0.95**	**0.07**
	40	**10/13/2009**	**17:56**	**10/14/2009**	**0:19**	**4.86**	**6.38**	**0.20**	**0.47**	**0.03**
	41	**10/20/2009**	**5:27**	**10/20/2009**	**6:41**	**6.21**	**1.23**	**0.24**	**0.95**	**0.19**
	42	**10/21/2009**	**15:36**	**10/22/2009**	**14:06**	**1.37**	**22.50**	**0.79**	**0.47**	**0.04**
	43	**10/25/2009**	**14:11**	**10/26/2009**	**0:47**	**3.00**	**10.60**	**0.59**	**0.47**	**0.06**
	44	**10/29/2009**	**6:01**	**10/29/2009**	**18:30**	**3.22**	**12.48**	**0.63**	**0.95**	**0.05**
	45	**11/14/2009**	**23:22**	**11/17/2009**	**15:50**	**16.20**	**64.47**	**1.97**	**1.42**	**0.03**
	46	**12/22/2009**	**21:27**	**12/24/2009**	**12:59**	**35.23**	**39.53**	**1.73**	**1.42**	**0.04**
	47	**12/28/2009**	**4:32**	**12/28/2009**	**14:16**	**3.65**	**9.73**	**0.12**	**0.47**	**0.01**
	48	**12/30/2009**	**6:01**	**12/30/2009**	**19:35**	**1.66**	**13.57**	**0.43**	**1.42**	**0.03**
	49	**2/5/2010**	**9:55**	**2/6/2010**	**4:05**	**36.60**	**18.17**	**0.28**	**0.47**	**0.02**
	50	**2/7/2010**	**19:35**	**2/8/2010**	**13:33**	**1.65**	**17.97**	**0.12**	**0.47**	**0.01**
	51	**2/19/2010**	**6:56**	**2/19/2010**	**17:21**	**10.72**	**10.42**	**0.43**	**0.47**	**0.04**
	52	**2/21/2010**	**5:32**	**2/22/2010**	**13:46**	**1.51**	**32.23**	**0.55**	**1.42**	**0.02**
	53	**3/8/2010**	**20:41**	**3/9/2010**	**14:54**	**14.29**	**18.22**	**0.20**	**0.95**	**0.01**
	54	**3/10/2010**	**18:53**	**3/11/2010**	**3:20**	**1.17**	**8.45**	**0.94**	**1.42**	**0.11**
	55	**3/21/2010**	**12:21**	**3/21/2010**	**15:34**	**10.38**	**3.22**	**0.20**	**1.42**	**0.06**
	56	**3/24/2010**	**12:55**	**3/24/2010**	**21:15**	**2.89**	**8.33**	**0.20**	**0.47**	**0.02**
	57	**3/27/2010**	**8:04**	**3/27/2010**	**11:20**	**2.45**	**3.27**	**0.12**	**0.95**	**0.04**
	58	**4/2/2010**	**9:51**	**4/2/2010**	**13:47**	**5.94**	**3.93**	**0.39**	**1.90**	**0.10**
	59	**4/5/2010**	**7:28**	**4/5/2010**	**9:24**	**2.74**	**1.93**	**0.71**	**0.95**	**0.37**
	60	**4/6/2010**	**20:04**	**4/7/2010**	**0:49**	**1.44**	**4.75**	**0.55**	**0.95**	**0.12**
	61	**4/22/2010**	**10:20**	**4/23/2010**	**8:34**	**15.40**	**22.23**	**2.52**	**1.42**	**0.11**
	62	**4/24/2010**	**11:39**	**4/25/2010**	**11:36**	**1.13**	**23.95**	**0.75**	**0.47**	**0.03**
	63	**4/30/2010**	**7:03**	**4/30/2010**	**14:15**	**4.81**	**7.20**	**0.55**	**0.47**	**0.08**
	64	**5/10/2010**	**10:09**	**5/11/2010**	**2:21**	**10.25**	**18.45**	**1.22**	**2.36**	**0.07**
	65	**5/12/2010**	**17:09**	**5/13/2010**	**9:17**	**1.81**	**19.30**	**1.54**	**2.83**	**0.08**
	66	**5/15/2010**	**7:21**	**5/16/2010**	**11:01**	**14.71**	**27.67**	**0.59**	**0.95**	**0.02**
	67	**5/19/2010**	**12:20**	**5/20/2010**	**20:02**	**3.05**	**31.70**	**1.22**	**0.95**	**0.04**
	68	**6/8/2010**	**8:30**	**6/9/2010**	**12:30**	**18.52**	**28.00**	**2.06**	**0.95**	**0.07**
	69	**6/12/2010**	**10:05**	**6/12/2010**	**13:40**	**2.90**	**3.58**	**2.13**	**0.95**	**0.59**
	70	**6/13/2010**	**7:15**	**6/14/2010**	**12:45**	**0.73**	**29.50**	**3.50**	**1.90**	**0.12**

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
	71	6/16/2010	18:23	6/16/2010	19:26	2.23	1.05	0.71	1.42	0.67
After Relining	72	1/22/2011	12:20	1/22/2011	15:40		3.33	0.12		0.04
	73	2/24/2011	9:00	2/24/2011	15:00	32.58	6.00	0.35	0.47	0.06
	74	2/26/2011	13:50	2/27/2011	20:20	2.15	16.67	1.22	1.42	0.07
	75	3/4/2011	11:10	3/4/2011	13:40	5.08	2.50	0.24	0.95	0.09
	76	3/8/2011	8:10	3/8/2011	13:10	3.65	5.00	0.39	0.47	0.08
	77	3/13/2011	23:00	3/14/2011	12:25	5.07	13.42	0.16	0.47	0.01
	78	3/19/2011	14:30	3/19/2011	16:15	5.69	1.75	0.32	0.95	0.18
After Construction	79	4/7/2013	19:52	4/8/2013	2:25		6.55	1.10	1.42	0.17
	80	4/9/2013	21:56	4/10/2013	18:30	1.62	20.57	1.62	1.42	0.08
	81	4/14/2013	19:02	4/15/2013	7:08	4.02	12.10	0.39	0.47	0.03
	82	4/17/2013	11:00	4/18/2013	20:41	2.16	33.68	1.10	0.47	0.03
	83	4/23/2013	1:34	4/23/2013	10:53	4.20	9.32	0.39	0.47	0.04
	84	4/26/2013	4:19	4/27/2013	11:02	2.73	30.72	0.95	0.95	0.03
	85	5/2/2013	3:08	5/4/2013	4:11	4.67	49.05	1.42	0.47	0.03
	86	5/8/2013	7:21	5/9/2013	2:36	4.13	19.25	0.28	0.47	0.01
	87	5/19/2013	2:21	5/20/2013	1:44	9.99	23.38	0.83	1.42	0.04
	88	5/27/2013	8:31	5/27/2013	12:35	7.28	4.07	2.01	1.90	0.49
	89	5/29/2013	22:53	5/30/2013	15:17	2.43	16.40	1.62	1.42	0.10
	90	5/31/2013	5:26	5/31/2013	9:21	0.59	3.92	1.34	1.90	0.34
	91	6/4/2013	10:55	6/4/2013	14:19	4.07	3.40	0.24	0.47	0.07
	92	6/5/2013	9:44	6/5/2013	12:47	0.81	3.05	0.47	0.47	0.15
	93	6/9/2013	0:54	6/9/2013	3:53	3.50	2.98	0.39	0.95	0.13
	94	6/15/2013	15:50	6/15/2013	22:05	6.50	6.25	1.26	1.42	0.20
	95	6/27/2013	11:46	6/28/2013	0:49	11.57	13.05	0.83	1.90	0.06
	96	7/3/2013	17:54	7/3/2013	21:36	5.71	3.70	1.50	0.95	0.41
	97	7/25/2013	17:00	7/26/2013	11:00	21.81	18.00	0.24	0.47	0.01
	98	7/29/2013	6:27	7/30/2013	9:41	25.37	27.23	0.91	0.95	0.03
	99	8/2/2013	3:46	8/2/2013	8:10	6.70	4.40	0.32	0.47	0.07
	100	8/4/2013	11:38	8/4/2013	13:58	5.08	2.33	0.12	0.47	0.05
	101	8/6/2013	3:24	8/6/2013	4:59	3.80	1.58	0.55	1.42	0.35
	102	8/7/2013	4:13	8/7/2013	8:45	2.59	4.53	0.87	0.95	0.19
	103	8/12/2013	7:07	8/12/2013	10:18	6.09	3.18	0.67	1.42	0.21
	104	9/1/2013	7:42	9/1/2013	9:02	19.89	1.33	0.16	0.95	0.12
	105	9/16/2013	2:13	9/16/2013	4:10	14.72	1.95	0.12	0.95	0.06
	106	9/17/2013	7:04	9/17/2013	15:14	1.12	8.17	0.79	0.95	0.10

	Event #	Rain start date	Rain start time	Rain end date	Rain end time	Antecedent dry days	Rain dur. (hrs)	Total rain (in)	5-minute peak rain intensity (in/hr)	Avg rain int. (in/hr)
	107	9/19/2013	19:00	9/19/2013	22:23	2.16	3.38	1.89	2.85	0.56
	108	9/28/2013	8:29	9/28/2013	11:29	8.42	3.00	0.28	0.47	0.09
	109	10/3/2013	11:04	10/3/2013	11:24	4.98	0.33	0.16	0.47	0.47
	110	10/4/2013	22:36	10/5/2013	3:48	1.47	5.20	0.87	0.47	0.17
	111	10/11/2013	23:08	10/12/2013	2:29	6.81	3.35	0.12	0.47	0.04
	112	10/14/2013	15:10	10/15/2013	1:23	2.53	10.22	0.12	0.47	0.01
	113	10/18/2013	14:03	10/18/2013	17:05	3.53	3.03	0.12	0.47	0.04
	114	10/29/2013	2:52	10/29/2013	9:41	10.41	6.82	0.83	0.95	0.12
	115	10/30/2013	12:09	10/31/2013	11:46	1.10	23.62	3.43	1.42	0.15
		count				112	115	115	114	115
		sum				703	1,380	92		
		minimum				0.59	0.33	0.12	0.47	0.01
		maximum				36.60	64.47	3.50	2.85	0.67
		average				6.28	12.00	0.80	0.95	0.11
		median				4.12	8.17	0.55	0.95	0.07
		standard deviation				6.76	11.22	0.72	0.52	0.12
		COV				1.08	0.94	0.91	0.55	1.11

Large-Scale Rainfall and Runoff Monitoring from Combined Sewer System for Control Area UMKC02a Monitoring Location (raw data from KCMO, UMKC, and Tetra Tech, calculations by UA)

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
1	3/23/2009	21:00	3/24/2009	13:00	16.00	15,945	0.244	1.56	0.28	0.29	0.44	1.65
2	3/26/2009	20:10	3/27/2009	6:10	10.00	822	0.013	0.37	0.02	18.50	0.04	1.56
3	3/28/2009	8:05	3/30/2009	14:20	54.25	23,335	0.357	2.32	0.21	11.05	0.38	1.81
4	3/30/2009	23:30	3/31/2009	13:00	13.50	16,690	0.255	0.79	0.35	0.47	1.06	6.64
5	4/2/2009	5:35	4/2/2009	23:50	18.25	3,625	0.055	0.20	0.05	4.00	0.46	2.68
6	4/9/2009	19:25	4/10/2009	20:35	25.17	23,021	0.352	0.96	0.25	3.84	0.37	1.38
7	4/12/2009	11:50	4/13/2009	20:00	32.17	1,096	0.017	0.12	0.01	12.00	0.04	1.70
8	4/18/2009	6:55	4/19/2009	23:45	40.83	4,733	0.072	1.15	0.05	23.00	0.13	7.49
9	4/19/2009	3:15	4/19/2009	18:45	15.50	34,052	0.521	2.26	0.62	3.65	1.89	4.35

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
10	4/26/2009	22:50	4/28/2009	1:00	26.17	292,467	4.476	9.13	3.09	2.95	2.06	1.83
11	4/29/2009	14:50	4/30/2009	23:55	33.08	61,072	0.935	5.76	0.51	11.29	0.44	1.09
12	5/8/2009	6:10	5/8/2009	13:50	7.67	4,067	0.062	1.31	0.15	8.73	0.19	2.35
13	5/13/2009	18:10	5/13/2009	21:00	2.83	10,902	0.167	4.52	1.04	4.35	0.60	2.00
14	5/15/2009	17:15	5/16/2009	9:00	15.75	163,805	2.507	11.18	2.87	3.90	1.87	3.30
15	6/2/2009	13:10	6/2/2009	19:35	6.42	14,196	0.217	6.12	0.61	10.03	0.56	0.97
16	6/8/2009	1:45	6/8/2009	5:45	4.00	1,415	0.022	1.24	0.10	12.40	0.14	2.96
17	6/9/2009	10:30	6/9/2009	18:15	7.75	95,197	1.457	20.39	3.38	6.03	0.91	0.62
18	6/11/2009	2:35	6/11/2009	17:15	14.67	87,774	1.343	10.86	1.65	6.58	3.12	5.83
19	6/15/2009	5:10	6/16/2009	18:35	37.42	160,447	2.456	17.89	1.14	15.69	0.97	1.31
20	6/23/2009	23:55	6/24/2009	6:45	6.83	1,261	0.019	1.37	0.05	27.40	0.06	3.73
21	7/3/2009	8:20	7/4/2009	8:15	23.92	30,474	0.466	5.91	0.35	16.89	0.29	1.09
22	7/10/2009	5:00	7/10/2009	8:00	3.00	1,115	0.017	1.29	0.10	12.90	0.11	2.69
23	7/12/2009	7:45	7/13/2009	6:45	23.00	14,639	0.224	9.90	0.18	55.00	0.28	2.12
24	7/20/2009	16:45	7/21/2009	3:15	10.50	4,353	0.067	1.11	0.11	10.09	0.11	0.97
25	7/27/2009	21:35	7/28/2009	17:40	20.08	12,699	0.194	6.52	0.17	38.35	0.11	1.05
26	8/1/2009	3:35	8/1/2009	6:30	2.92	3,150	0.048	0.83	0.29	2.86	0.15	1.05
27	8/4/2009	6:00	8/4/2009	20:45	14.75	7,089	0.108	3.71	0.13	28.54	0.20	5.40
28	8/10/2009	1:20	8/10/2009	15:55	14.58	5,976	0.091	1.74	0.11	15.82	0.46	5.61
29	8/15/2009	19:45	8/16/2009	10:50	15.08	24,261	0.371	4.73	0.44	10.75	0.16	1.00
30	8/17/2009	7:35	8/17/2009	23:55	16.33	27,760	0.425	9.70	0.47	20.64	0.39	3.08
31	8/19/2009	7:00	8/20/2009	13:05	30.08	17,584	0.269	3.59	0.16	22.44	0.30	1.66
32	8/27/2009	1:25	8/27/2009	12:50	11.42	2,244	0.034	0.16	0.05	3.20	0.29	2.15
33	9/4/2009	12:00	9/5/2009	18:15	30.25	3,816	0.058	0.80	0.03	23.82	0.15	1.68
34	9/8/2009	17:00	9/9/2009	17:45	24.75	4,452	0.068	0.96	0.05	19.20	0.22	1.00
35	9/21/2009	10:00	9/22/2009	11:15	25.25	13,576	0.208	1.91	0.15	12.73	0.33	2.00
36	9/26/2009	1:50	9/26/2009	23:00	21.17	6,049	0.093	1.43	0.08	17.88	0.26	2.24
37	10/1/2009	4:50	10/1/2009	19:50	15.00	1,636	0.025	0.38	0.03	12.67	0.21	4.46
38	10/6/2009	1:00	10/6/2009	4:20	3.33	826	0.013	0.36	0.07	5.14	0.11	2.04
39	10/8/2009	0:55	10/9/2009	4:15	27.33	12,524	0.192	3.41	0.13	26.23	0.13	1.32
40	10/13/2009	16:00	10/14/2009	2:10	10.17	2,917	0.045	1.01	0.09	11.22	0.22	1.59
41	10/20/2009	5:30	10/20/2009	18:40	13.17	5,349	0.082	2.83	0.11	25.73	0.34	10.68
42	10/21/2009	15:15	10/23/2009	2:05	34.83	18,988	0.291	1.17	0.15	7.80	0.37	1.55
43	10/25/2009	14:00	10/26/2009	12:45	22.75	20,720	0.317	1.53	0.25	6.12	0.54	2.15
44	10/29/2009	6:00	10/30/2009	6:30	24.50	18,121	0.277	2.65	0.20	13.25	0.44	1.96

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
45	11/15/2009	1:30	11/18/2009	3:50	74.33	24,369	0.373	0.85	0.09	9.44	0.19	1.15
46	12/22/2009	21:30	12/24/2009	23:45	50.25	15,127	0.232	1.41	0.08	17.63	0.13	1.27
47	12/28/2009	4:05	12/29/2009	1:55	21.83	5,211	0.080	0.18	0.07	2.57	0.66	2.24
48	12/30/2009	6:00	12/31/2009	7:00	25.00	4,448	0.068	0.13	0.05	2.60	0.16	1.84
49	2/5/2010	9:55	2/6/2010	7:40	21.75	4,292	0.066	0.20	0.05	4.00	0.23	1.20
50	2/7/2010	19:45	2/8/2010	23:00	27.25	6,355	0.097	0.15	0.06	2.50	0.81	1.52
51	2/19/2010	6:40	2/19/2010	20:40	14.00	11,432	0.175	0.93	0.22	4.23	0.41	1.34
52	2/21/2010	5:15	2/22/2010	23:50	42.58	25,973	0.398	1.04	0.17	6.12	0.72	1.32
53	3/8/2010	20:35	3/10/2010	2:55	30.33	8,276	0.127	0.85	0.08	10.63	0.64	1.67
54	3/10/2010	18:55	3/11/2010	15:15	20.33	15,274	0.234	2.51	0.20	12.55	0.25	2.41
55	3/21/2010	11:45	3/21/2010	23:50	12.08	1,865	0.029	0.24	0.05	4.80	0.14	3.76
56	3/24/2010	11:10	3/25/2010	8:55	21.75	7,802	0.119	0.39	0.10	3.90	0.60	2.61
57	3/27/2010	7:15	3/27/2010	15:05	7.83	2,518	0.039	0.27	0.04	6.75	0.32	2.40
58	4/2/2010	9:55	4/2/2010	23:55	14.00	13,507	0.207	6.34	0.27	23.48	0.53	3.56
59	4/5/2010	7:30	4/5/2010	20:45	13.25	16,089	0.246	11.23	0.42	26.74	0.35	6.85
60	4/6/2010	20:05	4/7/2010	12:00	15.92	16,506	0.253	3.04	0.25	12.16	0.46	3.35
61	4/22/2010	10:20	4/23/2010	20:30	34.17	61,201	0.937	3.46	0.50	6.92	0.37	1.54
62	4/24/2010	11:35	4/25/2010	23:35	36.00	42,831	0.656	0.91	0.33	2.76	0.87	1.50
63	4/30/2010	7:30	4/30/2010	20:15	12.75	8,719	0.133	3.52	0.15	23.47	0.24	1.77
64	5/10/2010	9:55	5/11/2010	12:00	28.33	26,199	0.401	8.71	0.33	26.39	0.33	1.54
65	5/12/2010	17:00	5/13/2010	21:20	31.50	79,652	1.219	12.40	0.78	15.90	0.79	1.63
66	5/15/2010	7:20	5/16/2010	23:00	39.67	14,422	0.221	0.61	0.09	6.78	0.37	1.43
67	5/19/2010	12:30	5/20/2010	8:00	19.50	54,087	0.828	4.15	0.34	12.21	0.68	0.62
68	6/8/2010	8:30	6/9/2010	20:40	36.17	27,560	0.422	7.97	0.26	30.65	0.20	1.29
69	6/12/2010	10:10	6/13/2010	1:40	15.50	75,083	1.149	11.93	1.34	8.90	0.54	4.33
70	6/13/2010	7:15	6/14/2010	23:55	40.67	274,075	4.195	47.29	1.87	25.29	1.20	1.38
71	6/16/2010	18:25	6/17/2010	7:30	13.08	36,391	0.557	7.49	0.76	9.86	0.79	12.46
72												
73	2/24/2011	9:00	2/25/2011	19:05	25.08	9,617	0.147	1.19	0.26	4.58	0.42	4.18
74	2/27/2011	8:35	2/28/2011	11:00	26.42	45,905	0.703	5.69	0.47	12.11	0.58	1.59
75	3/4/2011	11:10	3/4/2011	17:40	6.50	5,069	0.078	2.64	0.21	12.57	0.33	2.60
76	3/8/2011	8:10	3/9/2011	1:10	8.83	6,114	0.094	1.01	0.10	10.10	0.24	1.77
77	3/13/2011	22:30	3/14/2011	23:45	25.25	5,370	0.082	0.33	0.05	6.60	0.52	1.88
78	3/19/2011	13:30	3/20/2011	4:15	0.25	3,013	0.046	0.62	0.05	12.40	0.15	0.14
79	4/7/2013	19:45	4/8/2013	12:25	16.67	15,187	0.232	5.80	0.25	23.20	0.21	2.54

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
80	4/9/2013	21:55	4/10/2013	6:30	8.58	107,090	1.639	6.11	0.91	6.71	1.01	0.42
81	4/14/2013	19:00	4/15/2013	19:10	24.17	15,546	0.238	1.23	0.18	6.83	0.61	2.00
82	4/17/2013	11:00	4/19/2013	8:35	45.58	66,975	1.025	4.61	0.41	11.24	0.93	1.35
83	4/23/2013	1:30	4/23/2013	20:50	19.33	8,444	0.129	0.46	0.12	3.83	0.33	2.08
84	4/26/2013	4:25	4/27/2013	23:05	42.67	20,436	0.313	0.95	0.13	7.31	0.33	1.39
85	5/2/2013	3:00	5/4/2013	13:45	58.75	29,465	0.451	0.60	0.14	4.29	0.32	1.20
86	5/8/2013	23:25	5/9/2013	9:05	9.67	3,154	0.048	1.21	0.09	13.44	0.17	0.50
87	5/19/2013	2:20	5/20/2013	13:40	35.33	40,650	0.622	5.05	0.32	15.78	0.75	1.51
88	5/27/2013	8:20	5/27/2013	23:55	15.58	198,341	3.036	20.41	3.57	5.72	1.51	3.83
89	5/29/2013	22:45	5/31/2013	3:15	28.50	76,739	1.174	3.35	0.75	4.47	0.72	1.74
90	5/31/2013	5:15	5/31/2013	20:40	15.42	69,172	1.059	9.71	1.19	8.16	0.79	3.94
91	6/4/2013	10:40	6/4/2013	21:40	11.00	1,164	0.018	0.35	0.03	11.67	0.07	3.24
92	6/5/2013	9:50	6/5/2013	16:10	6.33	3,383	0.052	1.86	0.14	13.29	0.11	2.08
93	6/9/2013	0:55	6/9/2013	15:45	14.83	25,635	0.392	1.40	0.49	2.86	1.01	4.97
94	6/15/2013	15:45	6/16/2013	9:30	17.75	12,366	0.189	6.42	0.20	32.10	0.15	2.84
95	6/27/2013	11:35	6/28/2013	0:25	12.83	6,406	0.098	5.73	0.14	40.93	0.12	0.98
96	7/3/2013	17:35	7/4/2013	9:35	16.00	120,340	1.842	3.62	1.91	1.90	1.23	4.32
97	7/25/2013	17:00	7/26/2013	17:50	24.83	5,138	0.079	0.20	0.06	3.33	0.33	1.38
98	7/29/2013	6:30	7/30/2013	21:50	39.33	17,615	0.270	4.62	0.12	38.50	0.30	1.44
99	8/2/2013	3:45	8/2/2013	19:20	0.83	7,202	0.110	0.33	0.12	2.75	0.34	0.19
100	8/4/2013	11:40	8/4/2013	23:30	11.83	4,611	0.071	0.27	0.11	2.45	0.59	5.07
101	8/6/2013	3:25	8/6/2013	16:05	12.67	3,489	0.053	0.24	0.08	3.00	0.10	8.00
102	8/7/2013	4:20	8/7/2013	15:40	11.33	10,735	0.164	2.42	0.26	9.31	0.19	2.50
103	8/12/2013	5:00	8/12/2013	18:25	13.42	20,839	0.319	2.55	0.51	5.00	0.48	4.21
104	9/1/2013	5:45	9/1/2013	9:00	3.25	2,701	0.041	1.99	0.44	4.52	0.26	2.44
105	9/15/2013	20:00	9/16/2013	3:55	7.92	397	0.006	0.07	0.01	7.00	0.05	4.06
106	9/17/2013	6:40	9/17/2013	12:00	5.33	2,711	0.041	0.42	0.12	3.50	0.05	0.65
107	9/19/2013	17:20	9/20/2013	4:55	11.58	12,213	0.187	7.78	0.30	25.93	0.10	3.42
108	9/28/2013	6:45	9/28/2013	23:50	17.08	2,160	0.033	0.70	0.03	23.33	0.12	5.69
109	10/3/2013	6:00	10/3/2013	13:15	7.25	1,370	0.021	0.99	0.05	19.80	0.13	21.75
110	10/4/2013	21:10	10/5/2013	6:05	8.92	2,590	0.040	1.66	0.08	20.75	0.05	1.71
111	10/11/2013	17:00	10/12/2013	2:15	9.25	968	0.015	0.78	0.09	8.67	0.13	2.76
112	10/14/2013	14:00	10/15/2013	3:40	13.67	577	0.009	0.18	0.01	18.00	0.07	1.34
113	10/18/2013	10:30	10/18/2013	23:40	13.17	576	0.009	0.21	0.01	21.00	0.07	4.34
114	10/29/2013	1:40	10/29/2013	8:55	7.25	3,208	0.049	0.41	0.08	5.13	0.06	1.06

Event #	Pipeflow start date	Pipeflow start time	Pipeflow end date	Flow end time	Flow dur. (hrs)	Total pipeflow discharge volume (ft3)	Total disch. (in)	5-minute Peak flow disch. rate (CFS)	5-minute Avg flow disch. rate (CFS)	Peak/avg pipeflow rate ratio	Rv (Runoff Depth/Rain Depth)	flow/ rain dur. ratio
115	10/30/2013	10:50	10/31/2013	21:00	34.17	43,090	0.659	7.22	0.32	22.56	0.19	1.45
					114	114	114	114	114	114	114	114
					2,291	3,193,275	49					
					0.25	397	0.01	0.07	0.01	0.29	0.04	0.14
					74.33	292,467	4.48	47.29	3.57	55.00	3.12	21.75
					20.09	28,011	0.43	3.76	0.39	12.64	0.45	2.72
					15.96	12,289	0.19	1.48	0.15	10.37	0.33	1.86
					13.10	48,752	0.75	5.81	0.65	9.87	0.46	2.67
					0.65	1.74	1.74	1.55	1.69	0.78	1.03	0.98

Appendix E: Residential Area Production Function Calculations using WinSLAMM

Production Functions for 0.2 in/hr Native Soil Infiltration Rate

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
no	no	0.5	0.2	144,209	39,128	0	929	103,558	17	143,615	1.00	40,057	27.8	27.1	0.0	0.6	71.8	20.3	35	1.70	5.87	14.67
no	no	1.0	0.2	144,209	63,298	0	1,858	78,397	17	143,553	1.00	65,156	45.2	43.9	0.0	1.3	54.4	40.5	56	1.39	7.22	18.04
no	no	2.0	0.2	144,209	94,694	0	3,717	45,110	14	143,521	1.00	98,411	68.2	65.7	0.0	2.6	31.3	81.0	85	1.05	9.56	23.89
no	no	3.0	0.2	144,209	110,596	0	5,609	27,334	11	143,539	1.00	116,205	80.6	76.7	0.0	3.9	19.0	121.5	100	0.82	12.14	30.35
no	no	5.0	0.2	144,209	124,593	0	9,283	9,415	4	143,291	0.99	133,876	92.8	86.4	0.0	6.4	6.5	202.5	115	0.57	17.56	43.91
no	no	8.0	0.2	144,209	126,184	0	14,858	2,046	0	143,088	0.99	141,042	97.8	87.5	0.0	10.3	1.4	324.0	121	0.37	26.67	66.68
no	no	10.0	0.2	144,209	123,408	0	18,566	750	0	142,724	0.99	141,974	98.5	85.6	0.0	12.9	0.5	405.0	122	0.30	33.12	82.80
no	no	15.0	0.2	144,209	115,063	0	27,926	0	0	142,989	0.99	142,989	99.2	79.8	0.0	19.4	0.0	607.5	123	0.20	49.33	123.32
yes	no	0.5	0.2	144,209	51,914	0	929	90,593	8	143,436	0.99	52,843	36.6	36.0	0.0	0.6	62.8	20.3	46	2.25	4.45	11.12
yes	no	1.0	0.2	144,209	80,117	0	1,858	61,149	11	143,124	0.99	81,975	56.8	55.6	0.0	1.3	42.4	40.5	71	1.74	5.74	14.34
yes	no	2.0	0.2	144,209	109,021	0	3,717	30,225	9	142,963	0.99	112,738	78.2	75.6	0.0	2.6	21.0	81.0	97	1.20	8.34	20.86
yes	no	3.0	0.2	144,209	122,711	0	5,609	14,618	5	142,938	0.99	128,320	89.0	85.1	0.0	3.9	10.1	121.5	111	0.91	10.99	27.48
yes	no	5.0	0.2	144,209	129,344	0	9,283	3,972	0	142,599	0.99	138,627	96.1	89.7	0.0	6.4	2.8	202.5	119	0.59	16.96	42.40
yes	no	8.0	0.2	144,209	127,031	0	14,858	375	0	142,264	0.99	141,889	98.4	88.1	0.0	10.3	0.3	324.0	122	0.38	26.51	66.28

Production Functions for 0.2 in/hr Native Soil Infiltration Rate (cont.)

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
yes	no	10.0	0.2	144,209	123,648	0	18,566	0	0	142,214	0.99	142,214	98.6	85.7	0.0	12.9	0.0	405.0	122	0.30	33.07	82.66
yes	yes, 3"	0.5	0.2	144,209	30,083	42,169	929	70,252	1	143,433	0.99	31,012	21.5	20.9	29.2	0.6	48.7	20.3	63	3.11	3.21	8.03
yes	yes, 3"	1.0	0.2	144,209	47,629	55,438	1,858	38,303	1	143,228	0.99	49,487	34.3	33.0	38.4	1.3	26.6	40.5	90	2.23	4.48	11.20
yes	yes, 3"	2.0	0.2	144,209	71,322	55,216	3,717	12,677	1	142,932	0.99	75,039	52.0	49.5	38.3	2.6	8.8	81.0	112	1.39	7.22	18.05
yes	yes, 3"	3.0	0.2	144,209	88,024	44,292	5,609	4,786	0	142,711	0.99	93,633	64.9	61.0	30.7	3.9	3.3	121.5	119	0.98	10.23	25.57
yes	yes, 3"	5.0	0.2	144,209	105,288	27,872	9,283	6	0	142,449	0.99	114,571	79.4	73.0	19.3	6.4	0.0	202.5	123	0.61	16.51	41.26
yes	yes, 3"	8.0	0.2	144,209	116,366	11,103	14,858	0	0	142,327	0.99	131,224	91.0	80.7	7.7	10.3	0.0	324.0	123	0.38	26.43	66.08
yes	yes, 3"	10.0	0.2	144,209	118,235	5,467	18,566	0	0	142,268	0.99	136,801	94.9	82.0	3.8	12.9	0.0	405.0	123	0.30	33.05	82.63
yes	yes, 3"	15.0	0.2	144,209	113,226	1,361	27,926	0	0	142,513	0.99	141,152	97.9	78.5	0.9	19.4	0.0	607.5	123	0.20	49.49	123.73
yes	yes, 3"	20.0	0.2	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	99.1	73.4	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48
yes	yes*	0.5	0.2	144,209	37,945	29,098	929	72,606	1	140,578	0.97	38,874	27.0	26.3	20.2	0.6	50.3	20.3	59	2.89	3.46	8.65
yes	yes*	1.0	0.2	144,209	62,414	26,575	1,858	46,825	1	137,672	0.95	64,272	44.6	43.3	18.4	1.3	32.5	40.5	78	1.93	5.18	12.94
yes	yes*	2.0	0.2	144,209	92,686	19,346	3,717	22,454	1	138,203	0.96	96,403	66.8	64.3	13.4	2.6	15.6	81.0	100	1.23	8.13	20.31
yes	yes*	3.0	0.2	144,209	109,911	13,543	5,609	10,609	0	139,672	0.97	115,520	80.1	76.2	9.4	3.9	7.4	121.5	111	0.91	10.93	27.33
yes	yes*	5.0	0.2	144,209	122,141	7,011	9,283	2,833	0	141,268	0.98	131,424	91.1	84.7	4.9	6.4	2.0	202.5	119	0.59	16.98	42.46
yes	yes*	8.0	0.2	144,209	124,848	2,219	14,858	31	0	141,956	0.98	139,706	96.9	86.6	1.5	10.3	0.0	324.0	122	0.38	26.51	66.26
yes	yes*	10.0	0.2	144,209	122,427	1,082	18,566	0	0	142,075	0.99	140,993	97.8	84.9	0.8	12.9	0.0	405.0	122	0.30	33.10	82.74
yes	yes*	15.0	0.2	144,209	114,457	173	27,849	0	0	142,479	0.99	142,306	98.7	79.4	0.1	19.3	0.0	607.5	123	0.20	49.51	123.76
yes	yes*	20.0	0.2	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	99.1	73.4	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48

*** smartdrain**

Production Functions for 0.5 in/hr Native Soil Infiltration Rate

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
no	no	0.5	0.5	144,209	58,565	0	929	84,162	3	143,656	1.00	59,494	41.3	40.6	0.0	0.6	58.4	20.3	51	2.53	3.95	9.88
no	no	1.0	0.5	144,209	87,551	0	1,858	54,229	2	143,638	1.00	89,409	62.0	60.7	0.0	1.3	37.6	40.5	77	1.90	5.26	13.15
no	no	2.0	0.5	144,209	115,104	0	3,717	25,116	1	143,937	1.00	118,821	82.4	79.8	0.0	2.6	17.4	81.0	102	1.26	7.91	19.79
no	no	3.0	0.5	144,209	127,607	0	5,609	11,084	0	144,300	1.00	133,216	92.4	88.5	0.0	3.9	7.7	121.5	115	0.94	10.59	26.47
no	no	5.0	0.5	144,209	132,382	0	9,283	1,898	0	143,563	1.00	141,665	98.2	91.8	0.0	6.4	1.3	202.5	122	0.60	16.60	41.49
no	no	8.0	0.5	144,209	129,159	0	14,858	0	0	144,017	1.00	144,017	99.9	89.6	0.0	10.3	0.0	324.0	124	0.38	26.12	65.30
no	no	10.0	0.5	144,209	124,431	0	18,566	0	0	142,997	0.99	142,997	99.2	86.3	0.0	12.9	0.0	405.0	123	0.30	32.88	82.21
no	no	15.0	0.5	144,209	115,063	0	27,926	0	0	142,989	0.99	142,989	100.0	80.6	0.0	19.4	0.0	607.5	123	0.20	49.33	123.32
yes	no	0.5	0.5	144,209	71,747	0	929	70,837	1	143,513	1.00	72,676	50.4	49.8	0.0	0.6	49.1	20.3	63	3.09	3.24	8.09
yes	no	1.0	0.5	144,209	101,168	0	1,858	40,768	0	143,794	1.00	103,026	71.4	70.2	0.0	1.3	28.3	40.5	89	2.19	4.56	11.41
yes	no	2.0	0.5	144,209	125,070	0	3,717	14,672	1	143,459	0.99	128,787	89.3	86.7	0.0	2.6	10.2	81.0	111	1.37	7.30	18.26
yes	no	3.0	0.5	144,209	132,145	0	5,609	5,430	0	143,184	0.99	137,754	95.5	91.6	0.0	3.9	3.8	121.5	119	0.98	10.24	25.60
yes	no	5.0	0.5	144,209	133,182	0	9,283	615	0	143,080	0.99	142,465	98.8	92.4	0.0	6.4	0.4	202.5	123	0.61	16.50	41.26
yes	no	8.0	0.5	144,209	128,075	0	14,858	0	0	142,933	0.99	142,933	99.1	88.8	0.0	10.3	0.0	324.0	123	0.38	26.32	65.80
yes	no	10.0	0.5	144,209	123,648	0	18,566	0	0	142,214	0.99	142,214	100.0	87.1	0.0	12.9	0.0	405.0	122	0.30	33.07	82.66

Production Functions for 0.5 in/hr Native Soil Infiltration Rate (cont.)

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
yes	yes, 3"	0.5	0.5	144,209	53,052	22,470	929	67,046	1	143,497	1.00	53,981	37.4	36.8	15.6	0.6	46.5	20.3	66	3.25	3.08	7.69
yes	yes, 3"	1.0	0.5	144,209	76,036	29,782	1,858	35,661	1	143,337	0.99	77,894	54.0	52.7	20.7	1.3	24.7	40.5	93	2.29	4.37	10.92
yes	yes, 3"	2.0	0.5	144,209	101,592	26,338	3,717	11,552	1	143,199	0.99	105,309	73.0	70.4	18.3	2.6	8.0	81.0	113	1.40	7.14	17.86
yes	yes, 3"	3.0	0.5	144,209	113,880	19,499	5,609	4,071	0	143,059	0.99	119,489	82.9	79.0	13.5	3.9	2.8	121.5	120	0.99	10.15	25.37
yes	yes, 3"	5.0	0.5	144,209	124,568	9,117	9,283	0	0	142,968	0.99	133,851	92.8	86.4	6.3	6.4	0.0	202.5	123	0.61	16.45	41.11
yes	yes, 3"	8.0	0.5	144,209	126,251	1,839	14,858	0	0	142,948	0.99	141,109	97.9	87.5	1.3	10.3	0.0	324.0	123	0.38	26.32	65.79
yes	yes, 3"	10.0	0.5	144,209	124,197	216	18,566	0	0	142,979	0.99	142,763	99.0	86.1	0.1	12.9	0.0	405.0	123	0.30	32.89	82.22
yes	yes, 3"	15.0	0.5	144,209	115,167	0	27,926	0	0	143,093	0.99	143,093	99.2	79.9	0.0	19.4	0.0	607.5	123	0.20	49.29	123.23
yes	yes, 3"	20.0	0.5	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48
yes	yes*	0.5	0.5	144,209	60,400	15,028	929	67,045	1	143,402	0.99	61,329	42.5	41.9	10.4	0.6	46.5	20.3	66	3.25	3.08	7.70
yes	yes*	1.0	0.5	144,209	90,447	13,383	1,858	36,496	1	142,184	0.99	92,305	64.0	62.7	9.3	1.3	25.3	40.5	91	2.25	4.45	11.12
yes	yes*	2.0	0.5	144,209	117,974	7,421	3,717	12,760	1	141,872	0.98	121,691	84.4	81.8	5.1	2.6	8.8	81.0	111	1.37	7.28	18.21
yes	yes*	3.0	0.5	144,209	127,691	4,128	5,609	4,963	0	142,391	0.99	133,300	92.4	88.5	2.9	3.9	3.4	121.5	118	0.97	10.26	25.66
yes	yes*	5.0	0.5	144,209	131,736	1,481	9,283	283	0	142,783	0.99	141,019	97.8	91.4	1.0	6.4	0.2	202.5	123	0.61	16.50	41.25
yes	yes*	8.0	0.5	144,209	127,844	236	14,858	0	0	142,938	0.99	142,702	99.0	88.7	0.2	10.3	0.0	324.0	123	0.38	26.32	65.80
yes	yes*	10.0	0.5	144,209	124,396	15	18,566	0	0	142,977	0.99	142,962	99.1	86.3	0.0	12.9	0.0	405.0	123	0.30	32.89	82.22
yes	yes*	15.0	0.5	144,209	115,235	0	27,849	0	0	143,084	0.99	143,084	99.2	79.9	0.0	19.3	0.0	607.5	123	0.20	49.30	123.24
yes	yes*	20.0	0.5	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48

*** smartdrain**

Production Functions for 1.0 in/hr Native Soil Infiltration Rate

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
no	no	0.5	1	144,209	78,731	0	929	64,026	1	143,686	1.00	79,660	55.2	54.6	0.0	0.6	44.4	20.3	69	3.39	2.95	7.38
no	no	1.0	1	144,209	107,421	0	1,858	34,555	1	143,834	1.00	109,279	75.8	74.5	0.0	1.3	24.0	40.5	94	2.32	4.30	10.76
no	no	2.0	1	144,209	128,621	0	3,717	11,234	1	143,572	1.00	132,338	91.8	89.2	0.0	2.6	7.8	81.0	114	1.41	7.11	17.77
no	no	3.0	1	144,209	134,173	0	5,609	3,574	0	143,356	0.99	139,782	96.9	93.0	0.0	3.9	2.5	121.5	120	0.99	10.09	25.23
no	no	5.0	1	144,209	134,694	0	9,283	0	0	143,977	1.00	143,977	99.8	93.4	0.0	6.4	0.0	202.5	124	0.61	16.33	40.83
no	no	8.0	1	144,209	129,159	0	14,858	0	0	144,017	1.00	144,017	100.0	89.7	0.0	10.3	0.0	324.0	124	0.38	26.12	65.30
no	no	10.0	1	144,209	124,431	0	18,566	0	0	142,997	0.99	142,997	100.0	87.1	0.0	12.9	0.0	405.0	123	0.30	32.88	82.21
no	no	15.0	1	144,209	115,063	0	27,926	0	0	142,989	0.99	142,989	100.0	80.6	0.0	19.4	0.0	607.5	123	0.20	49.33	123.32
yes	no	0.5	1	144,209	80,506	0	929	62,080	1	143,515	1.00	81,435	56.5	55.8	0.0	0.6	43.0	20.3	70	3.46	2.89	7.22
yes	no	1.0	1	144,209	109,738	0	1,858	31,801	1	143,397	0.99	111,596	77.4	76.1	0.0	1.3	22.1	40.5	96	2.37	4.21	10.53
yes	no	2.0	1	144,209	129,662	0	3,717	9,911	1	143,290	0.99	133,379	92.5	89.9	0.0	2.6	6.9	81.0	115	1.42	7.05	17.63
yes	no	3.0	1	144,209	134,543	0	5,609	3,087	0	143,239	0.99	140,152	97.2	93.3	0.0	3.9	2.1	121.5	121	0.99	10.07	25.16
yes	no	5.0	1	144,209	133,940	0	9,282	0	0	143,222	0.99	143,222	99.3	92.9	0.0	6.4	0.0	202.5	123	0.61	16.42	41.04
yes	no	8.0	1	144,209	128,075	0	14,858	0	0	142,933	0.99	142,933	100.0	89.7	0.0	10.3	0.0	324.0	123	0.38	26.32	65.80
yes	no	10.0	1	144,209	123,648	0	18,566	0	0	142,214	0.99	142,214	100.0	87.1	0.0	12.9	0.0	405.0	122	0.30	33.07	82.66

Production Functions for 1.0 in/hr Native Soil Infiltration Rate (cont.)

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
yes	yes, 3"	0.5	1	144,209	79,822	689	929	62,080	1	143,520	1.00	80,751	56.0	55.4	0.5	0.6	43.0	20.3	70	3.46	2.89	7.22
yes	yes, 3"	1.0	1	144,209	106,482	3,282	1,858	31,801	1	143,423	0.99	108,340	75.1	73.8	2.3	1.3	22.1	40.5	96	2.37	4.21	10.53
yes	yes, 3"	2.0	1	144,209	125,954	3,736	3,716	9,911	1	143,317	0.99	129,670	89.9	87.3	2.6	2.6	6.9	81.0	115	1.42	7.05	17.62
yes	yes, 3"	3.0	1	144,209	132,420	2,141	5,608	3,087	0	143,256	0.99	138,028	95.7	91.8	1.5	3.9	2.1	121.5	121	0.99	10.06	25.16
yes	yes, 3"	5.0	1	144,209	133,618	324	9,282	0	0	143,224	0.99	142,900	99.1	92.7	0.2	6.4	0.0	202.5	123	0.61	16.42	41.04
yes	yes, 3"	8.0	1	144,209	128,379	0	14,856	0	0	143,235	0.99	143,235	99.3	89.0	0.0	10.3	0.0	324.0	123	0.38	26.26	65.66
yes	yes, 3"	10.0	1	144,209	124,689	0	18,564	0	0	143,253	0.99	143,253	99.3	86.5	0.0	12.9	0.0	405.0	123	0.30	32.83	82.06
yes	yes, 3"	15.0	1	144,209	115,167	0	27,926	0	0	143,093	0.99	143,093	100.0	80.6	0.0	19.4	0.0	607.5	123	0.20	49.29	123.23
yes	yes, 3"	20.0	1	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48
yes	yes*	0.5	1	144,209	79,955	552	929	62,080	1	143,516	1.00	80,884	56.1	55.4	0.4	0.6	43.0	20.3	70	3.46	2.89	7.22
yes	yes*	1.0	1	144,209	108,100	1,639	1,858	31,801	1	143,398	0.99	109,958	76.2	75.0	1.1	1.3	22.1	40.5	96	2.37	4.21	10.53
yes	yes*	2.0	1	144,209	128,672	993	3,716	9,911	1	143,292	0.99	132,388	91.8	89.2	0.7	2.6	6.9	81.0	115	1.42	7.05	17.63
yes	yes*	3.0	1	144,209	134,179	363	5,608	3,087	0	143,237	0.99	139,787	96.9	93.0	0.3	3.9	2.1	121.5	121	0.99	10.07	25.16
yes	yes*	5.0	1	144,209	133,917	22	9,282	0	0	143,221	0.99	143,199	99.3	92.9	0.0	6.4	0.0	202.5	123	0.61	16.42	41.04
yes	yes*	8.0	1	144,209	128,379	0	14,858	0	0	143,237	0.99	143,237	99.3	89.0	0.0	10.3	0.0	324.0	123	0.38	26.26	65.66
yes	yes*	10.0	1	144,209	124,689	0	18,566	0	0	143,255	0.99	143,255	99.3	86.5	0.0	12.9	0.0	405.0	123	0.30	32.82	82.06
yes	yes*	15.0	1	144,209	115,235	0	27,849	0	0	143,084	0.99	143,084	100.0	80.7	0.0	19.3	0.0	607.5	123	0.20	49.30	123.24
yes	yes*	20.0	1	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48

*** smartdrain**

Production Functions for 2.5 in/hr Native Soil Infiltration Rate

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
no	no	0.5	2.5	144,209	92,527	0	929	50,096	1	143,552	1.00	93,456	64.8	64.2	0.0	0.6	34.7	20.3	80	3.97	2.52	6.29
no	no	1.0	2.5	144,209	118,652	0	1,858	22,991	1	143,501	1.00	120,510	83.6	82.3	0.0	1.3	15.9	40.5	104	2.56	3.90	9.76
no	no	2.0	2.5	144,209	133,353	0	3,716	6,374	1	143,443	0.99	137,069	95.0	92.5	0.0	2.6	4.4	81.0	118	1.46	6.86	17.15
no	no	3.0	2.5	144,209	136,751	0	5,607	1,052	0	143,410	0.99	142,358	98.7	94.8	0.0	3.9	0.7	121.5	123	1.01	9.91	24.77
no	no	5.0	2.5	144,209	134,694	0	9,283	0	0	143,977	1.00	143,977	100.0	93.6	0.0	6.4	0.0	202.5	124	0.61	16.33	40.83
no	no	8.0	2.5	144,209	129,159	0	14,858	0	0	144,017	1.00	144,017	100.0	89.7	0.0	10.3	0.0	324.0	124	0.38	26.12	65.30
no	no	10.0	2.5	144,209	124,431	0	18,566	0	0	142,997	0.99	142,997	100.0	87.1	0.0	12.9	0.0	405.0	123	0.30	32.88	82.21
no	no	15.0	2.5	144,209	115,063	0	27,926	0	0	142,989	0.99	142,989	100.0	80.6	0.0	19.4	0.0	607.5	123	0.20	49.33	123.32
yes	no	0.5	2.5	144,209	92,540	0	929	50,096	1	143,565	1.00	93,469	64.8	64.2	0.0	0.6	34.7	20.3	81	3.98	2.52	6.29
yes	no	1.0	2.5	144,209	118,670	0	1,858	22,991	1	143,519	1.00	120,528	83.6	82.3	0.0	1.3	15.9	40.5	104	2.56	3.90	9.75
yes	no	2.0	2.5	144,209	133,376	0	3,717	6,374	1	143,467	0.99	137,093	95.1	92.5	0.0	2.6	4.4	81.0	118	1.46	6.86	17.15
yes	no	3.0	2.5	144,209	136,774	0	5,609	1,052	0	143,435	0.99	142,383	98.7	94.8	0.0	3.9	0.7	121.5	123	1.01	9.91	24.77
yes	no	5.0	2.5	144,209	134,122	0	9,281	0	0	143,403	0.99	143,403	99.4	93.0	0.0	6.4	0.0	202.5	124	0.61	16.40	40.99
yes	no	8.0	2.5	144,209	128,075	0	14,858	0	0	142,933	0.99	142,933	100.0	89.7	0.0	10.3	0.0	324.0	123	0.38	26.32	65.80
yes	no	10.0	2.5	144,209	123,648	0	18,566	0	0	142,214	0.99	142,214	100.0	87.1	0.0	12.9	0.0	405.0	122	0.30	33.07	82.66

Production Functions for 2.5 in/hr Native Soil Infiltration Rate (cont.)

gravel layer?	underdrain?	surface area %	infilt rate, in/hr	total inflow CF	infilt vol CF	underdrain CF	ET CF	surface bypass CF	#>3days ponding in 4.11 yrs)	sum out CF	out/in ratio	sum infilt plus ET	% captured by biofilt	% vol reduction to infilt	% to underdrain	% as ET	% surface bypass	surface area m2	part. solids kg/yr (125 mg/L and 4.11 yrs)	part. Solids kg/m2/yr	years to 10 kg/m2	years to 25 kg/m2
yes	yes, 3"	0.5	2.5	144,209	92,540	0	929	50,096	1	143,565	1.00	93,469	64.8	64.2	0.0	0.6	34.7	20.3	81	3.98	2.52	6.29
yes	yes, 3"	1.0	2.5	144,209	118,670	0	1,858	22,991	1	143,519	1.00	120,528	83.6	82.3	0.0	1.3	15.9	40.5	104	2.56	3.90	9.75
yes	yes, 3"	2.0	2.5	144,209	133,376	0	3,716	6,374	1	143,466	0.99	137,092	95.1	92.5	0.0	2.6	4.4	81.0	118	1.46	6.86	17.15
yes	yes, 3"	3.0	2.5	144,209	136,774	0	5,607	1,052	0	143,433	0.99	142,381	98.7	94.8	0.0	3.9	0.7	121.5	123	1.01	9.91	24.77
yes	yes, 3"	5.0	2.5	144,209	134,122	0	9,281	0	0	143,403	0.99	143,403	99.4	93.0	0.0	6.4	0.0	202.5	124	0.61	16.40	40.99
yes	yes, 3"	8.0	2.5	144,209	128,379	0	14,856	0	0	143,235	0.99	143,235	100.0	89.7	0.0	10.3	0.0	324.0	123	0.38	26.26	65.66
yes	yes, 3"	10.0	2.5	144,209	124,689	0	18,564	0	0	143,253	0.99	143,253	100.0	87.1	0.0	12.9	0.0	405.0	123	0.30	32.83	82.06
yes	yes, 3"	15.0	2.5	144,209	115,167	0	27,926	0	0	143,093	0.99	143,093	100.0	80.6	0.0	19.4	0.0	607.5	123	0.20	49.29	123.23
yes	yes, 3"	20.0	2.5	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48
yes	yes*	0.5	2.5	144,209	92,540	0	929	50,096	1	143,565	1.00	93,469	64.8	64.2	0.0	0.6	34.7	20.3	81	3.98	2.52	6.29
yes	yes*	1.0	2.5	144,209	118,670	0	1,858	22,991	1	143,519	1.00	120,528	83.6	82.3	0.0	1.3	15.9	40.5	104	2.56	3.90	9.75
yes	yes*	2.0	2.5	144,209	133,376	0	3,716	6,374	1	143,466	0.99	137,092	95.1	92.5	0.0	2.6	4.4	81.0	118	1.46	6.86	17.15
yes	yes*	3.0	2.5	144,209	136,774	0	5,607	1,052	0	143,433	0.99	142,381	98.7	94.8	0.0	3.9	0.7	121.5	123	1.01	9.91	24.77
yes	yes*	5.0	2.5	144,209	134,122	0	9,281	0	0	143,403	0.99	143,403	99.4	93.0	0.0	6.4	0.0	202.5	124	0.61	16.40	40.99
yes	yes*	8.0	2.5	144,209	128,379	0	14,858	0	0	143,237	0.99	143,237	100.0	89.7	0.0	10.3	0.0	324.0	123	0.38	26.26	65.66
yes	yes*	10.0	2.5	144,209	124,776	0	18,566	0	0	143,342	0.99	143,342	100.0	87.1	0.0	12.9	0.0	405.0	123	0.30	32.80	82.01
yes	yes*	15.0	2.5	144,209	115,235	0	27,849	0	0	143,084	0.99	143,084	100.0	80.7	0.0	19.3	0.0	607.5	123	0.20	49.30	123.24
yes	yes*	20.0	2.5	144,209	105,809	0	37,133	0	0	142,942	0.99	142,942	100.0	74.3	0.0	25.7	0.0	810.0	123	0.15	65.79	164.48

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