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Evaluation of Dry Wells and Cisterns for Stormwater Control: Millburn Township, NJ

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Evaluation of Dry Wells and Cisterns for Stormwater Control: Millburn Township, New Jersey

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Abstract

The primary objective of this project was to investigate the effectiveness of the Township of Millburn's use of on-site dry wells to limit stormwater flows into the local drainage system. The objective was to examine this stormwater management alternative applicable for mature urban and suburban communities to reduce stormwater discharges associated with new development and redevelopment. This objective was achieved by collecting and monitoring the performance of dry wells during both short and long-periods. The water quality beneath dry wells and in a storage cistern was also monitored during ten rain events.

There were varying levels of dry well performance in the area, but most were able to completely drain within a few days. However, several had extended periods of standing water that may have been associated with high water tables, poorly draining soils (or partially clogged soils), or detrimental effects from snowmelt on the clays in the soils. The infiltration rates all met the infiltration rate criterion of the state guidelines for stormwater discharges to dry wells (but not the state regulations that only allow roof runoff to be discharged to dry wells and those that prohibit dry well use in areas of shallow water tables). Overall, most of the Millburn dry wells worked well in infiltrating runoff. Although the dry wells provided no significant improvements in water quality for constituents of interest for the infiltrating water, they resulted in reduced mass discharges of flows and pollutants to surface waters and reduced runoff energy, major causes of local erosion problems.

Acknowledgements

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Glossary

AD	Anderson-Darling Statistical Test
ANOVA	Analysis of Variance
ASCE	American Society of Civil Engineers
BDL	Below Detection Limit
BGS	Below Ground Surface
BMP	Best Management Practice
BOD_5	Five Day Biochemical Oxygen Demand
CBOD	Carbonaceous Biochemical Oxygen Demand
CI	Confidence Interval
COD	Chemical Oxygen Demand
COV	Coefficient of Variation
DEM	Digital Elevation Model
EPA	Environmental Protection Agency
ET	Evapotranspiration
ETA	Estimated Time of Arrival
FAO	Food and Agriculture Organization
HDPE	High-density Polyethylene
HNO_3	Nitric Acid
HSG	Hydrologic Soil Group
IDF	Intensity, Duration, Frequency
LDL	Lower Detection Limit
MCL	Maximum Contaminant Limits
MPN	Most Probable Number
N.J.A.C	New Jersey Administrative Code
ND	Not Detected (below detection limits)
NJDEP OPRA	NJ Department of Environmental Protection - Open Public Records Act
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NSQD	National Stormwater Quality Database
NTU	Nephelometric Turbidity Units
PVC	Polyvinyl Chloride
SAR	Sodium Absorption Ratio
SCS	Soil Conservation Service
SMCLs	Secondary Maximum Contaminant Levels
Std Dev	Standard Deviation
TN	Total Nitrogen
TSS	Total Suspended Solids
UDFCD	Urban Drainage and Flood Control District
UDL	Upper Detection Limit
USDA	United States Department of Agriculture
USGS	United States Geological Survey
WinSLAMM	Source Loading and Management Model
WUCOLS	Water Use Classification of Landscape Species

Chapter 1 Executive Summary

Description of Millburn and Its Dry Wells

The Township of Millburn, Essex County, NJ, is located near New York City, and less than 10 miles from Newark International Airport. The 2010 US census indicated the Township had a population of 20,149. Housing costs are very high (According to Wikipedia, Millburn had the highest annual property tax bills in NJ in 2009 at an average of more than $19,000 per year, compared to the statewide average property tax that was $7,300, the highest statewide average in the country). There are about 5,900 detached homes in the Township and about 1,500 have dry wells.

In 1999, the Township of Millburn created an ordinance that required increased runoff from new impervious areas to be directed into seepage pits (dry wells). The purpose of this project was to investigate the effectiveness of this ordinance, specifically to examine the use of dry wells as a technique to redirect surface runoff to the local shallow groundwater. The objective of this approach is to reduce local drainage and erosion problems associated with new development and increased impervious areas of currently developed areas. The slower release of the shallow groundwater to surface streams also better simulates natural hydrologic patterns with reduced in-stream problems associated with increased rapid surface runoff. The Township of Millburn has a stable population where there is little vacant land. All new construction within the community is performed on previously developed plots.

The Millburn Township stormwater regulations (Development Regulations) list dry wells as one option for minimizing increased flows associated with new (and increased) development. They do not include any specific criteria for their use, except for a statement pertaining to a 60 cm (2 ft) blanket of crushed stone surrounding the dry well. Specifically, they do not describe applicable soil characteristics, groundwater conditions, or suitable source waters. The NJ State stormwater regulations also requires the infiltration of excess water above natural conditions associated with development or land modifications (either maintaining the pre-development groundwater recharge or preventing excess surface runoff). The state dry well regulations describe the construction of the dry wells, the acceptable soil conditions (NRCS hydrologic soil groups, HSG, A and B), groundwater conditions (at least 60 cm or 2 ft above seasonal water table), and source waters (roof runoff only).

A dry well is a subsurface infiltration stormwater disposal practice that receives stormwater runoff from surrounding areas for subsurface disposal to shallow groundwater. Most of the dry wells in the Township of Millburn are precast concrete structures (Figure 1-1), with open bottoms resting on 0.6 m (2 ft) crushed stone layers and with 0.6 m (2 ft) of crushed stone surrounding the dry wells. Most of the dry wells receive water directly from roof drain leaders or by storm drain inlets located in driveways or small parking lots. Some also have grated covers and receive surface runoff from the surrounding lawn or paved areas.

Figure 1-1. Peerless Concrete Products, Butler, NJ, supplies the dry wells to many of the sites in Millburn (photo from http://www.peerlessconcrete.com/).

Figure 1-2 shows typical dry well installations. Many of the dry wells are located in landscaped areas and have open covers, allowing surface runoff from the lawns to enter the dry wells, as well as the subsurface piped roof runoff. Some are also located in paved areas, also allowing surface runoff from the driveways to enter along with the roof runoff.

Backyard dry well showing lawn area also as a source.

Backyard dry well showing driveway runoff also as a source.

Figure 1-2. Typical Millburn dry well locations.

Fifteen dry wells were monitored for water levels during periods ranging from 2 months to one year, or by controlled tests using Township water from fire hydrants. Four systems (three dry wells and one cistern) were also monitored for water quality during 10 storms to indicate any differences in water quality directly below the dry well (or at the cistern inlet) compared to deeper depths at least 0.6 m (2 ft) below the bottom of the crushed stone layer, or at least 1.2 m (4 ft) below the bottom of the dry well itself (or in the cistern). Four rain gages were also installed near the dry wells.

The study sites were surveyed to obtain detailed development characteristics that affect the amount of runoff from the different source areas. Soil information was also compiled. Most of the surface soils were of NRCS hydrologic soil group (HSG) C or D category, indicating poor infiltration potential. However, subsurface soils where the dry wells were located were mostly in the HSG A or B categories (glacial deposits) with much improved infiltration potentials. The groundwater in the area may be as shallow as 2.4 to 3 m (8 to 10 ft) below the ground surface in low-lying areas along the river, but otherwise is expected to be greater than 8 m (25 ft) below the ground surface in general.

Project Objectives and Findings

The overall objective of this project was to investigate the effectiveness of Millburn's stormwater management practices that rely on the use of dry wells to limit stormwater discharges into the local drainage system. Millburn has separate sewers and there are concerns about drainage problems developing in areas of new construction. The city has been pleased with the performance of the dry wells. This project quantified their performance and offered suggestions for improved stormwater management in the area.

Both short- and long-term infiltration monitoring was conducted in a selection of Millburn dry wells. There were varying levels of dry well performance in the area, but most were able to completely drain within a few days. However, several had extended periods of standing water that may have been associated with high water tables, poorly draining soils (or partially clogged soils), or detrimental effects from snowmelt. The infiltration rate characteristics were separated into three conditions: 1) HSG A and B surface soils having well-drained HSG A subsurface soils; 2) C and D surface soils and well-drained A and B subsurface soils; and, 3) C and D surface soils and poorly drained subsurface soils with long-term standing water. Even sites having surface C and D soils (not acceptable infiltration sites according to the NJ dry well standards) had much better subsurface conditions where the dry wells were located. The infiltration rates for these conditions were less than for the excellent areas having HSG A and B surface soils, but all met the infiltration rate criterion of the state guidelines.

Water samples were collected at one cistern and three dry well locations during ten rains. The samples were analyzed for nutrients and heavy metals, and selected samples were also tested for pesticides and herbicides. The samples were collected directly below the dry wells (or at the inlet of the cistern) for comparison to water samples collected deeper (at least 1.2 m (4 ft)) beneath the dry wells (and at the outlet of the cistern). Various statistical tests were used to compare the measured water quality to detect any significant differences due to operation of the dry wells. The paired sample sets did not indicate any significant differences for any of the water quality constituents for these samples for the dry wells (ten events in three dry wells). The cistern outlet median total coliform values were greater than the inflow median values, indicating possible re-growth; however, the median *E. coli* and COD cistern outlet values were less than the inflow values for these constituents. These findings indicate that the dry wells did not significantly change the water quality for the monitored stormwater constituents. If the influent stormwater is of good quality, the dry wells can be a safe disposal method. However, the bacteria and lead concentrations exceeded the groundwater disposal criteria for NJ and may require treatment if the aquifer is critical (even though these were the runoff conditions and were not affected either by increases or decreases, by the dry wells).

Dry wells may be a preferred option in cases that are allowed by the NJ dry well disposal regulations for stormwater which limits their use to areas having excellent soils (HSG A or B; although subsurface soils should be considered also), where the groundwater table is below the dry well system (to prevent standing water in the dry wells and very slow infiltration), and to only receive roof runoff water (generally the best quality runoff from a site and the snowmelt from roofs would not be contaminated with deicing salts). The Millburn Township stormwater regulations do not restrict dry well disposal to just roof runoff, but also includes disposal of all excess runoff from new development as one option.

Beneficial uses of the roof runoff for irrigation may be a preferred alternative, and in many cases may be less costly than dry wells, especially considering increasing water utility rates and the desire to conserve highly treated domestic water supplies. Groundwater recharge may be an important objective for an area and dry well use addresses that objective. However, “over-irrigation” (beyond the plants evapotranspiration (ET) deficit needs, but less than would produce direct runoff) also addresses that objective and would also conserve domestic water while offering better groundwater protection than the dry wells.

Rain gardens are another viable alternative for stormwater management in the Millburn area, especially as they provide some groundwater quality protection and can be incorporated into the landscaping plans of homes. They likely require additional maintenance, similar to any garden, but can be located to receive runoff from several source areas on a site, increasing the overall stormwater management level. In some areas, they have even been incorporated along roads, as curb-cut biofilters, resulting in significant overall runoff volume reductions, but with special care to prevent premature clogging and appropriately sized to handle the large flow volumes.

It is important to use alternative stormwater control options when dry well use is restricted, such as with the following conditions:

- poor infiltration capacity of subsurface soil layers
- concerns about premature clogging or other failures due to sediment discharges or saline snowmelt discharges to dry wells
- seasonal or permanent high water tables
- concerns about groundwater contamination potential

Primary Report Questions

The following questions were listed during the development of this project by Millburn Township personnel, in addition to others that were asked during the review of the final report: (see appendix A for a comprehensive list of primary and follow-up questions)

1) Are the implementation activities working?

At most of the monitoring locations, the dry wells drained quickly and completely after rains (within a day or two for full dry wells). However, some locations experienced standing water for extended periods and would be considered to not be working as intended. Basically, more careful site evaluations and design, along with better control of the source waters entering the dry wells, are needed. In critical situations, alternative stormwater controls should be considered.

2) What is the impact of the effectiveness in various soil types?

Originally, it was thought that the surface soil characteristics would have little affect on the performance of the dry wells, as they are subsurface devices and most of the water is percolating from the dry wells at depth and not near the surface. At all of the Millburn test locations, the subsurface soils had better infiltration characteristics compared to the surface soils; in fact, the subsurface soils were all HSG A or B, which meet the State's dry well design standards, even though the surface soils were mostly HSG C or D soils. The measured infiltration rates from all of the dry wells meet the minimum rates specified by the State's design guidance, but there were substantial variations, as noted below (average infiltration rates for typical storm durations):

- A and B surface soils and having well-drained A subsurface soils (190 mm/hr or 7.6 in./hr)
- C and D surface soils and having well-drained A and B subsurface soils (43 mm/hr or 1.7 in./hr)
- C and D surface soils and having poorly-drained subsurface soils with long-term standing water (20 mm/hr or 0.8 in./hr)

Generally, the lowest infiltration rates associated with long-term saturated conditions averaged about 12 mm/hr (0.5 in./hr). Again, all of these rates satisfied the State's design guidance. However, several sites had long-term standing water and never drained completely, while other locations required several weeks to drain (and seldom were the dry periods long enough to allow complete drainage).

Therefore, even though the site conditions met the design guidance, some locations still had standing water. It is likely that seasonal (or possibly long-term) high water tables occurred at some of the locations. The lack of site specific groundwater depth information did not allow this to be verified, but the performance of some of the drain-down curves supports this finding.

In other cases, the rates appeared to vary by season, with some incidences of standing water mostly in the spring and sometimes in the winter. It is thought that soil chemistry changes due to saline snowmelt waters entering the dry wells from non-roof areas were responsible for these periods of reduced performance. De-icing chemicals would likely be heavily applied near home walkways, porches, steps and driveways (but not roofs). If this water was allowed to enter the dry wells and if there was clay in the surrounding soils, sodium adsorption ratio (SAR) imbalances would disperse the clays and cause significant reductions in the infiltration rates. In most cases, excess sodium would be rinsed from soils over a few months, partially restoring the infiltration conditions. However, problems would continue to reoccur with subsequent saline snowmelt discharges to the dry wells.

Therefore, the sites that had sandy surface and subsurface soils (HSG A and B soils) performed the best. It was thought that the other sites would also perform well, having good subsurface soils, but their characteristics were not likely as good as the other locations (even in the same soil category), and the likely presence of small to moderate amounts of clay would be more sensitive to SAR problems.

3) Is it more important to address roof runoff versus other runoff sources such as driveways and patios, etc?

Roof runoff contributes about one-third of the average annual runoff for the Millburn residential areas, a typical value compared to other residential areas in the US. However, the roof runoff only contributes about 11% of the TSS. Other source areas, such as driveways on the private property, are important runoff sources. Streets contribute about one-fourth of the runoff. Patios are not very significant as their runoff is mostly directed to the landscaped areas where it can infiltrate. After the roofs, the driveways should be controlled (such as by using rain gardens near the lower ends of the driveways near the streets, as many are steeply sloped from the house to the roads). Dry wells for driveways (or other paved areas besides roofs) should not be considered due to the much greater sediment load that would likely cause premature failure by clogging.

4) Are there any maintenance requirements needed for the dry wells?

It is difficult to maintain the dry wells as they are buried. The bottoms are open and resting on crushed stone allowing the penetration of silts and clays into the voids. These materials cannot be easily removed. However, leaves and other vegetation debris on top of the crushed stone could possibly be removed without disturbing the rocks. Many of the dry wells have grated openings allowing surface runoff from the surrounding areas to directly flow into the dry wells. During construction, erosion sediment may enter the dry wells which would significantly hinder their performance. As noted, organic matter from the surrounding areas can also directly enter the dry wells through these surface openings. It is recommended that only directly connected roof leaders enter the dry wells (in compliance with the state regulations) and that the dry wells be inspected and superficially cleaned periodically. Leaf filters should also be installed on the roof gutters or down spouts or capture these materials before they are discharged into the dry wells. If needing maintenance to remove the silts and sands from the crushed stone, much of the crushed stone would have to be removed and replaced, a costly option similar to totally rebuilding the dry well. Prevention (such as by only allowing roof runoff to enter the dry wells) is therefore key to long-term satisfactory performance.

5) What is the impact of dry wells on groundwater quality?

Residential roof runoff has few contaminants and should therefore be the preferred source of water directed to the dry wells (although bacteria may be a problem, and other pollutants may periodically be a concern, especially if zinc and copper materials are used on the roofs). The dry wells provided no significant improvements to the quality of the stormwater, based on the limited sampling conducted during this study. If other source waters enter the dry wells (such as from driveways and streets), groundwater contamination would be a much greater concern. The best quality waters in residential areas that should not present a significant groundwater hazard and is most suitable for direct infiltration is roof runoff. However, bacterial and lead concentrations observed below the dry wells frequently exceeded the NJ groundwater disposal limits and some additional control may therefore be needed.

6) Evaluate the ordinance that was created by the Township of Millburn to control erosion and flooding.

The local Millburn Township ordinance should be modified to allow dry well use only in areas already having good stormwater runoff quality (such as would be expected for most roofs), or require suitable pretreatment, such as effective grass filtering. In addition, the local ordinance should also prohibit dry well use in areas having seasonal or permanent high water tables, as those conditions result in long-term standing water in the dry wells. If located in areas having poorly draining subsurface soils, the dry well designs need to be modified (greatly enlarged) to account for the more slowly draining conditions. It is recommended that dry well use be restricted to roof runoff, and alternatives that infiltrate water through surface soils (such as rain gardens) be used to treat driveway and parking lot runoff (or in areas having shallow groundwater). Irrigation of landscaped areas using roof runoff (and pretreated paved area runoff) is also a suitable option that can provide economic benefits to the land owner and should be encouraged by the ordinance.

Infiltration Tests at Millburn Dry Well Installations

Infiltration tests were conducted during two project phases: the first phase filled the dry wells with domestic water from Township fire hydrants and the decreasing water levels were recorded; the second phase used continuous water level monitoring in a fewer number of dry wells during many rains. The infiltration measurements were conducted using continuous recording (10 minute observations) LeveLoggers by Solintest that were installed in the dry wells. The short-term tests were conducted in dry wells throughout the Township to measure the influence of many of the conditions present in the community. These tests were conducted using water from fire hydrants and included filling the dry wells completely. The LeveLoggers were then used to record the drop in water level over time. The long-term tests were conducted in fewer dry wells (based on the number of LeveLoggers available). These were installed for several months to over a year and continuously recorded the water levels in the dry wells every 10 minutes. Close-by rain gages were also used to record local rains associated with these events. These rain and water level data were downloaded by PARS Environmental personnel and uploaded to their ftp website where University of Alabama researchers downloaded the data for analysis.

The first step in the data analyses of the long-term tests was to plot the data as time series. Figure 1-3 is an example time series plot of the water levels recorded over a two month period at 11 Woodfield Dr. showing six separate events (the first peak only shows the dropping water levels from the Oct 13, 2009 event). The infiltration characteristics of the dry well installations were calculated from the recession curves of these individual rain events. The infiltration rates for each ten minute step were calculated based on the drop in water level per time increment, resulting in infiltration rate plots of in./hr vs. time since the peak water level. These are classical infiltration rate plots and statistical analyses were used to calculate infiltration rate equation parameters for two common infiltration equations (Horton and Green-Ampt).

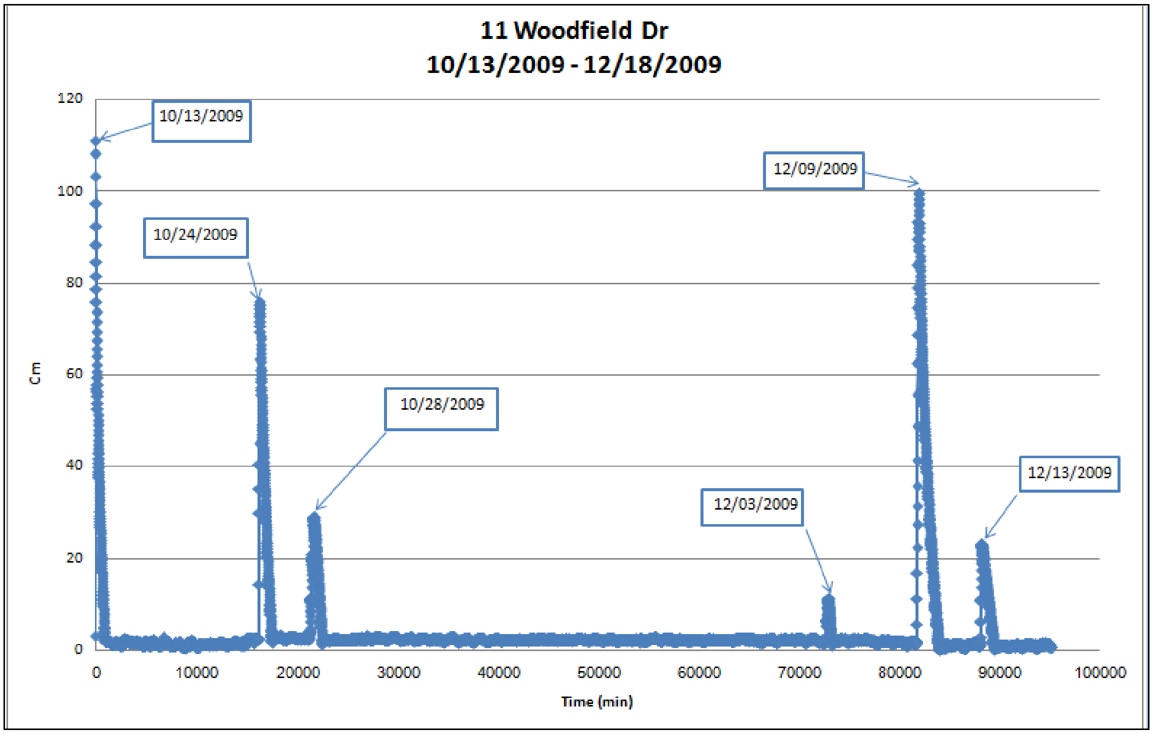


Figure 1-3. Time series example of dry well water levels for a two month period at 11 Woodfield Dr.

Groundwater recharge is a suitable beneficial use of stormwater in many areas as it is used to augment local groundwater resources. This study showed how the dry wells could be very effective in delivering the stormwater to the groundwater. Even though the surface soils were almost all marginal for infiltration options, the relatively shallow dry wells were constructed into subsurface soil layers that had much greater infiltration potentials. However, some of the monitored dry well locations experienced seasonal high groundwater elevations, restricting complete draining of the dry wells after rains. While surface and subsurface soil information is readily available for the Township (and in most other areas of the country), the presence of the shallow water table (or bedrock) is not well known. This makes identifying the most suitable locations for dry wells difficult, as the seasonal groundwater should be at least 4 m (12 ft) below the ground surface (or 60 cm, 2 ft, below the lowest gravel fill layer beneath the dry well: 2 ft of surface cover, 6 ft dry well concrete structure, 2 ft lower gravel layer, and 2 ft of separation above the high seasonal groundwater depth).

Calculating the benefits of the dry wells (including developing sizing requirements) requires the use of an appropriate infiltration equation, preferably as part of a continuous model examining many years of actual rainfall data for a specific area. Two commonly used infiltration equations (Horton and Green-Ampt) were evaluated for their potential use to calculate groundwater recharge at the case study locations in the Township of Millburn, NJ. The fitted graphs and resulting derived equation parameters showed that although the Horton equation usually indicated a better fit to the observed data, the calculated parameters of both infiltration models were not close to values reported in the literature, especially for urban areas. This is likely because the infiltration characteristics in the dry wells were mostly affected by subsurface conditions compared to the literature values that were compared to surface soil characteristics. When the subsurface conditions are used in the comparisons, the observed and literature values are in better (but still not close) agreement. Therefore, locally measured infiltration test data at a scale approaching the size and depth of the final devices should be used for more reliable design guidance, instead of relying on literature values.

Factors Affecting Infiltration Rates

The data analyses of the infiltration data resulted in several interesting conclusions. One of the first issues noted by the field personnel when installing the water level recorders and observing the dry wells over time was that some of the locations experienced periodic (or continuous) long-term standing water in the dry wells, indicating seasonal or permanent high water table conditions, or partially clogged dry wells.

Table 1-1 summarizes the dry well performance observed during the monitoring program, including the length of monitoring, hydrograph behavior, and the presence of standing water (and the percentage of time when the dry well was dry). Figures 1-4 and 1-5 are time series plots of the water levels for the long-term infiltration tests at two dry wells representing a site with poorly draining conditions and another site with rapid drainage conditions. These plots show the water elevations in the dry wells along with the corresponding rain depths as recorded at the nearest rain gage. The rain data indicate the total rain depth and the start and end times; the graphs cover too long of a period to show variable rain intensities during the rains. The times and depths are the most important rain information for these measurements as they relate most closely to the runoff quantity and the dry well water elevations.

Table 1-1. Summary of Infiltration Conditions with Time

	Start date of series	End date of series	# of dry well events	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments
11 Woodfield Dr.	Oct 11, 2009	December 20, 2009	1 hydrant 5 rains (1 small rain missing)	89%	Consistent shape with time	Quickly drained (within a day); No standing water at any time	15 hours total drainage time during hydrant test
15 Marion Dr.	June 17, 2010	August 6, 2010	1 hydrant 5 rains (2 small rains missing)	71%	Consistent shape with time	Several days to drain; No standing water at any time	4.5 days total drainage time during hydrant test
383 Wyoming Ave.	July 16, 2009	October 14, 2009	1 hydrant 6 rains (2 small rains missing)	81%	Consistent shape with time	Several days to drain if full; No standing water at any time	1 day total drainage time during hydrant test
258 Main St.	June 16, 2010	August 5, 2010	5 rains (2 smaller rains missing)	98%	Consistent shape with time	Very rapid drainage time; No standing water at any time	
260 Hartshorn	August 9, 2010	August 1, 2011	Many	10%	Consistent shape with time	Slow drainage time (about a week if full), but dry if given enough time between rains	Clogging or poor soils, not high water table. Possible SAR issues in the Winter and Spring, recovered by mid-summer.
2 Undercliff Rd	July 18, 2009	October 6, 2009	1 hydrant 3 rains	79%	Consistent shape with time	Several days to drain if full; No standing water at any time	10 days total drainage time during hydrant test
87/89 Tennyson	August 10, 2010	August 5, 2011	Many	0%	Consistent shape with time	Very slow drainage time (a couple of weeks); standing water and never dry during this year period	Slow drainage may be due to saturated conditions, never reached stable low water level. If due to SAR, did not recover.

	Start date of series	End date of series	# of dry well events	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments
7 Fox Hill	August 7, 2010	March 23, 2011	Many	2%	Consistent shape with time	Slow drainage time (about a week or two if full), but dry if given enough time between rains	Clogging or poor soils especially in Spring, possibly SAR issues, not high water table
8 So. Beechcroft	July 19, 2009	September 27, 2009	1 hydrant 6 rains	71%	Consistent shape with time for rains, but hydrant test (at end of monitoring period at end of Sept) was very rapid	Quickly drained (within a day or two if full); No standing water at any time	3 hours total drainage time (half full) during hydrant test
142 Fairfield	August 10, 2010	March 4, 2011	many	66%	Somewhat inconsistent shape with time	Quickly drained (within a day or two if full) to poorly drained (a week for moderate rains); Standing water during periods of large and frequent rains	Slowly drained conditions in Spring likely due to saturated conditions, or SAR. Not likely due to high water table
36 Farley Place	June 16, 2010	August 5, 2010	3 rains	97%	Consistent shape with time	Very rapid drainage time; No standing water at any time	

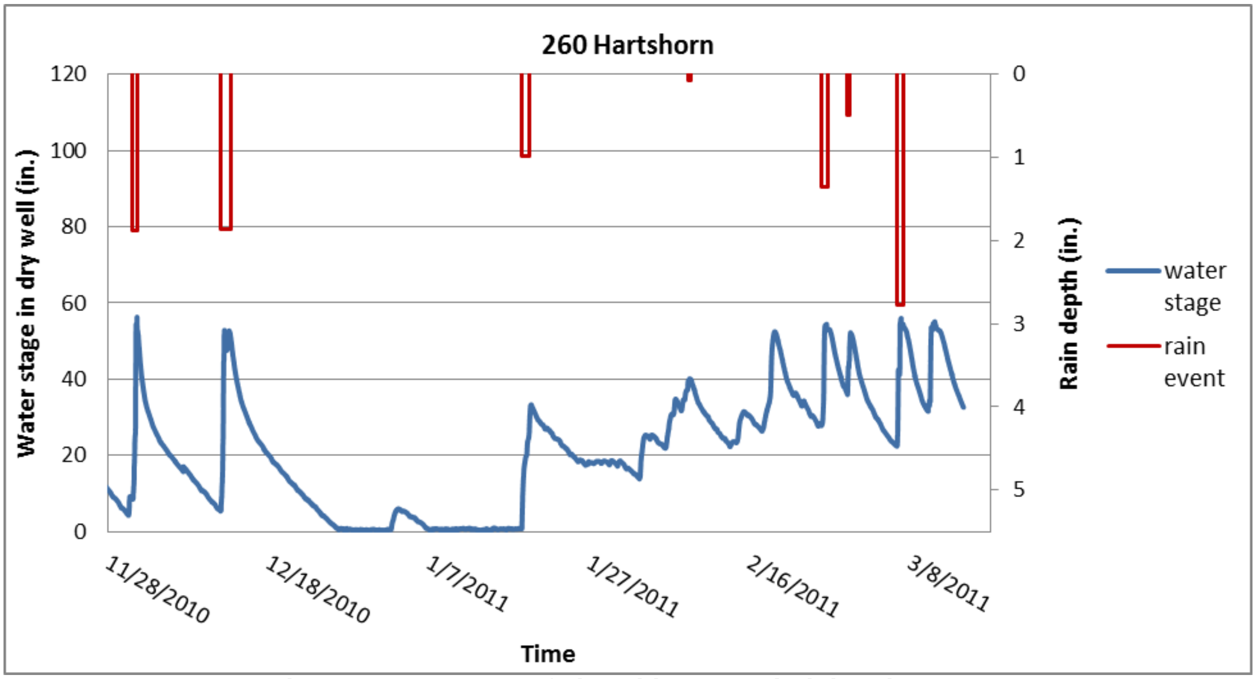


Figure 1-4. Example of site with poorly draining dry well.

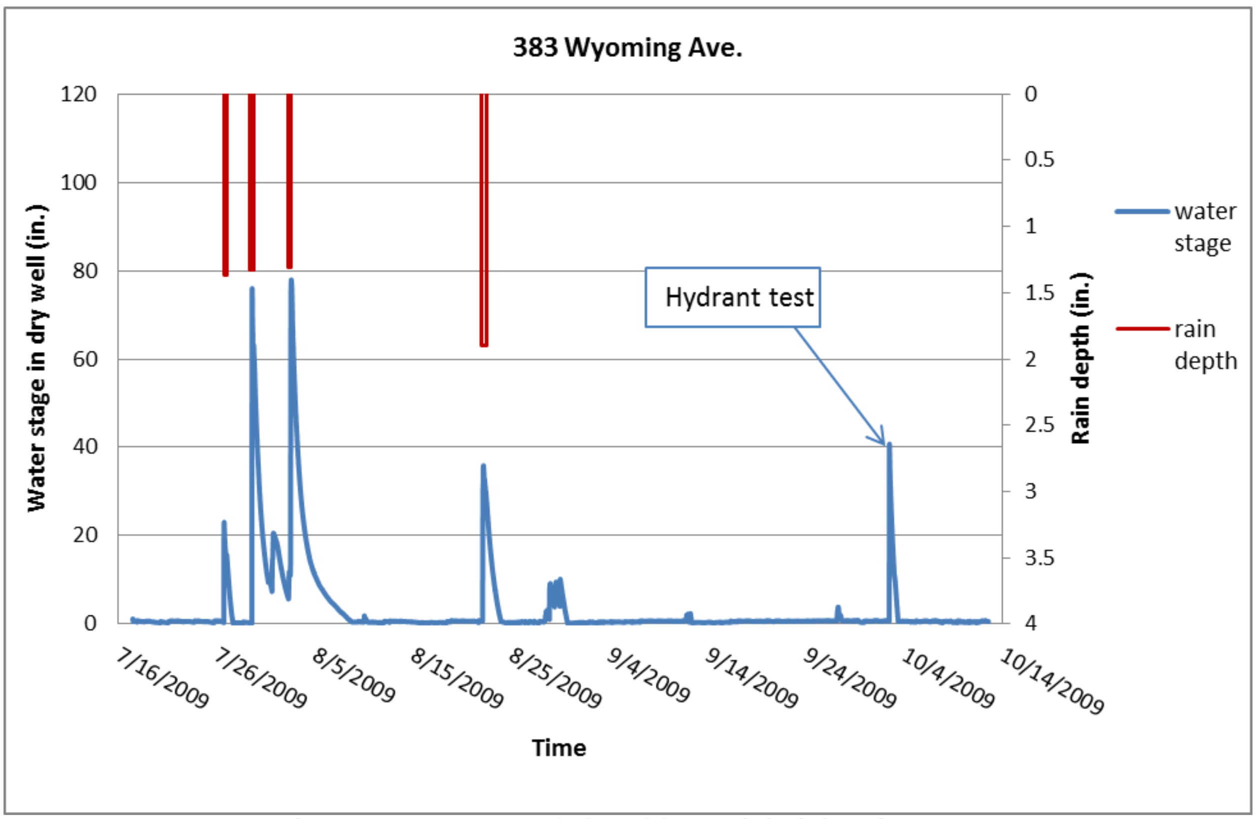


Figure 1-5. Example of site with good draining dry well.

In almost all cases, the general shapes of the recession limbs (water elevation drops with infiltration) are similar for all observations at the same site, including the hydrant tests. However, some changed with time, including several that indicated slower infiltration with more standing water conditions in the winter and spring. This may be due to SAR issues (sodium adsorption ratio) that results in dispersed clays from high sodium content in snowmelt. Normally, snowmelt would not affect these units if only roof runoff is directed to the dry wells. However, if walkway or driveway runoff drains to dry wells, de-icing salts may be in the snowmelt, increasing the SAR and decreasing the infiltration rates.

Standing water was observed in the dry well at 87/89 Tennyson when sufficient time occurred to allow the water to reach a consistent minimum water level of about 0.9 m (3 ft). It is expected that this site very likely has a shallow water table condition. The drainage rates were very slow, so the inter-event periods were not sufficiently long to enable drainage to the stable water level until after about a two week dry period. The slow drainage rate may have been caused by saturated conditions associated with groundwater mounding. Several sites (260 Hartshorn, 7 Fox Hill, and 142 Fairfield) experienced periodic slow- draining conditions, mainly in the spring that could have been associated with SAR problems. The slow infiltration rates could be due to poor soils (with the clays resulting in SAR problems), or saturated soil conditions. The other sites all had rapid drainage rates that were consistent with time.

Another obvious factor affecting the observed infiltration rates was that one or two of the locations had significantly higher infiltration rates than the other sites (all having no standing water issues). These sites were the ones indicated as having the highest surface infiltration rate potentials (even though the infiltration rates of the dry wells were mostly affected by the subsurface soil conditions, which were mapped as being similar A and B conditions for all locations). It is therefore expected that these locations had better subsurface soil conditions compared to the other sites, even though mapped as being similar.

Therefore, the Township of Millburn infiltration rate characteristics were separated into three conditions:

- A and B surface soils having well drained A subsurface soils
- C and D surface soils having well drained A and B subsurface soils
- C and D surface soils having poorly drained A and B subsurface soils with long-term standing water

Table 1-2 compares the observed Horton equation coefficients for the sites having well-drained subsurface soils with equation coefficients that have been reported in the literature. The standing water data are not used in these calculations as most of the observations could not be successfully fitted to the Horton equation. The almost steady infiltration rates (but with substantial variation) were all very low for those conditions and likely represent the f_c (long-term constant rate) conditions only and were therefore included in that parameter category.

Table 1-2. Observed and Reported Horton Equation Coefficients

	f_o (in./hr)	f_c (in./hr)	k (1/min)
Surface A and B soils well drained A subsurface soils (average and COV)	**44.6 (0.53)**	**5.6 (0.2)**	**0.06 (0.22)**
Surface C and D soils well drained A and B subsurface soils (average and COV)	**4.3 (0.64)**	**0.45 (0.85)**	**0.01 (0.63)**
UDFCD (2001) A soils (average)	5.0	1.0	0.04
UDFCD (2001) B soils (average)	4.5	0.6	0.11
UDFCD (2001) C and D soils (average)	3.0	0.5	0.11
Pitt, *et al.* (1999) Clayey, dry and non-compacted (median)	11	3	0.16
Pitt, *et al.* (1999) Clayey, other (median)	2	0.25	0.06
Pitt, *et al*. (1999) Sandy, compacted (median)	5	0.5	0.1
Pitt, *et al.* (1999) Sandy, non-compacted (median)	34	15	0.08
Akan (1993) Sandy soils with little to no vegetation	5		
Akan (1993) Dry loam soils with little to no vegetation	3		
Akan (1993) Dry clay soils with little to no vegetation	1		
Akan (1993) Moist sandy soils with little to no vegetation	1.7		
Akan (1993) Moist loam soils with little to no vegetation	1		
Akan (1993) Moist clay soils with little to no vegetation	0.3		

(1 in./hr = 25.4 mm/hr)
f_o = initial infiltration rate (in./hr)
f_c = final infiltration rate (in./hr)
k = first order rate constant (1/min)

The very large observed f_o value (45 in./hr) for the A and B surface soil sites that are well drained is greater than any of the reported literature values, and only approaches the observations for the non-compacted sandy soil conditions (34 in./hr) observed by Pitt, *et al*. (1999). The subsurface soil conditions affecting the dry well infiltration rates are likely natural with little compaction. Also, the subsurface soils at that location are noted as being sandy loam (A) and stratified gravelly sand to sand to loamy sand (A). The other sites having smaller f_o rates (4.3 in./hr) are described as gravelly sandy loam (A) and fine sandy loam (B) and are similar to many of the reported literature values for sandy soils, with some compaction.

The large f_c value (5.6 in./hr) observed for the well-drained A and B surface soil location is bracketed by the non-compacted clayey and sandy soil conditions (3 and 15 in./hr) reported by Pitt, *et al.* (1999), but is substantially larger than the other reported values. The f_c value observed for the well-drained C and D surface soil site (0.45 in./hr) is similar to the other reported values (0.5 to 1.0 in./hr). The k first-order rate values (0.01 and 0.06 1/min) are similar, but on the low side, of the reported values (0.04 to 0.11 1/min).

In order to most accurately design dry well installations in an area, actual site observations of the expected infiltration rates should be used instead of general literature values. This is especially true for surface infiltration devices (such as rain gardens), where compaction due to construction activities and general urban use will have a much greater effect than on the deeper subsurface soils. Also, all of the sites in this study had improved infiltration characteristics with depth compared to expected surface conditions; in other cases, this may not be true. Criteria based only on surface soil conditions are likely not good predictors of deeper dry well performance. Luckily, county soil surveys do have some subsurface soil information that was found to be generally accurate during this study. Unfortunately, shallow water table conditions are not well known for the area and that characteristic can have a significant detrimental effect on the observed dry well performance.

Water Quality Observations

Water samples were collected at three dry wells and at one cistern during ten rains. The samples were analyzed for nutrients and heavy metals, and selected samples were also tested for pesticides and herbicides. The samples were collected directly below the dry wells (or at the inlet of the cistern) for comparison to water samples collected at least 0.6 m (2 ft) below the 0.6 m (2 ft) gravel layer beneath the dry wells (and in the cistern), for a total subsurface flow path of at least 1.2 m (4 ft) through the crushed stone and subsurface soil (more than the minimum 2 ft separation to the groundwater table as required by the NJ stormwater infiltration regulations). Various statistical tests were used to compare the water quality from the inlet to the outlet locations to detect any significant differences due to operation of the dry wells.

Log-normal probability plots were used to identify the range, randomness, and normality of the data. The log-normal probability plots are shown for inflow vs. cistern, and for the cistern and deep vs. shallow for each sampling site. Figure 1-6 includes example paired log-normal probability plots for one of the sites (135 Tennyson Road, Millburn, NJ 07078) for different parameters including bacteria, nutrients, and COD. For these plots, most of the data are seen to overlap within the limits of the 95% confidence limits, indicating that the data are likely from the same population. Also, the data seem to generally fit a straight line, indicating likely log-normal data distributions, as verified by the Anderson-Darling test statistic.

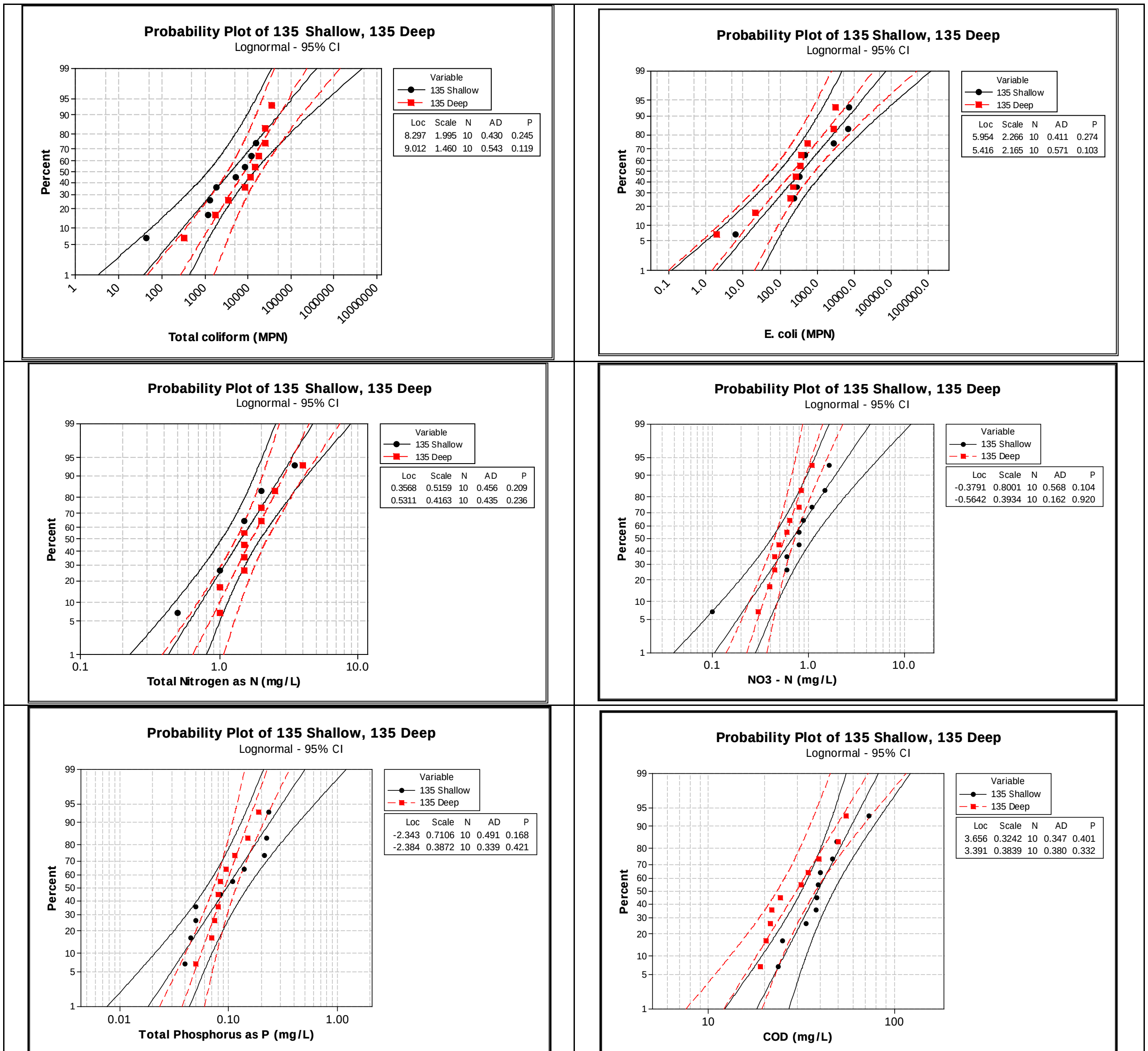


Figure 1-6. Log-normal probability plots for the site located on 135 Tennyson Road (shallow vs. deep)

Table 1-3 shows the output obtained using MINITAB for nonparametric Mann-Whitney comparisons between paired data. Except for the bacteria and COD results for the cistern site, as noted previously, all paired sample sets did not indicate significant differences for these numbers of samples at the 0.05 level for the numbers of sample pairs available.

Table 1-3.Summary of Mann-Whitney Test for Paired Data

Parameter		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Total Coliforms	p-value	0.03	0.40	0.16	0.72
	Significant Difference Observes? (at level of 0.05)	Yes (but cistern median values were larger than the inflow median values)	No	No	No
E. coli	p-value	0.05	0.60	0.69	1
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No
Total Nitrogen as N	p-value	0.86	0.50	0.42	0.64
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
NO_3 plus NO_2-N	p-value	0.14	0.24	0.15	0.77
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
Total Phosphorus as P	p-value	0.77	0.94	0.10	0.27
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
COD	p-value	0.04	0.14	0.40	0.83
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No

Table 1-4 lists the results for the paired sign test (used because of numerous non-detected values) for lead, copper and zinc observations for the cistern and dry well samples. No statistically significant differences were seen between the sample sets for the heavy metals for the numbers of samples available.

Table 1-4. Summary of Paired Sign Test for Metal Analysis

Metal		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Lead	p-value	> 0.06	> 0.06	0.18	> 0.06
	Significant Difference in Medians?	No	No	No	No
Copper	p-value	0.13	*	>0.06	*
	Significant Difference in Medians?	No	*	No	*
Zinc	p-value	0.45	0.45	>0.06	>0.06
	Significant Difference in Medians?	No	No	No	No

***** All the results are below detection limit (BDL), therefore it is not possible to do a paired sign test

Statistical analyses indicated that the differences in water quality between the shallow and the deeper samples were not significant for the number of sample pairs available (p-values were > 0.05). However, significant differences were found (p< 0.05) between the quality of inflow samples and cistern samples for total coliforms (possible re-growth), *E. coli*, and COD (concentration reductions). These findings indicate that the dry wells do not significantly change the water quality for most of the stormwater constituents. If the influent water quality is of good quality, the dry wells can be a safe disposal method for stormwater quality. However, most of the bacteria and lead concentrations exceeded the groundwater disposal criteria for NJ and may require treatment, if the aquifer is critical.

Results and Conclusions

Dry wells may be a preferred option in cases that are allowed by the NJ dry well disposal regulations for stormwater which limits their use to areas having excellent soils (HSG A or B), where the groundwater table is below the dry well system (to prevent standing water in the dry wells and very slow infiltration), and to only receive roof runoff water (generally the best quality runoff from a site and not contaminated with deicing salts). However, the beneficial uses of roof runoff should be the preferred option, and in many cases may be less costly, especially considering increasing water utility rates and the desire to conserve highly treated domestic water supplies. Shallow groundwater recharge may be an important objective for an area, but “over” irrigation (beyond the plants ET deficit needs, but less than would produce direct runoff) would also contribute to that objective, at the same time as conserving water and offering better groundwater protection.

Figure 1-7 is a map showing the general infiltration rate conditions for Millburn. Most of the monitored dry wells were along a ridge between the two main drainages of the Township, with no obvious pattern of high water conditions, except that the high standing water dry wells were located along a line to the southwest along the ridge and are located fairly close to headwaters of streams (high water tables were noted in areas with nearby streams, but that was assumed to be in the larger stream valleys and not at the headwaters). The sites that had high standing water long after the events ended had substantially reduced infiltration rates. In the analyses, these rates were considered to be the constant (final) rates observed, with no initial rate data or first-order decay Horton coefficients used (relatively constant, but very low infiltration rates). Three of the sites had severely degraded infiltration conditions (260 Hartshorn, 87/89 Tennyson, and 7 Fox Hill). These sites all received runoff from the entire property or from multiple impervious areas (and are 1 to 5 years old). It is not known if the source water or groundwater conditions affected the drainage conditions at these sites. Dry wells receiving runoff from all impervious areas would have a greater silt load and likely clog prematurely compared to sites only receiving roof runoff.

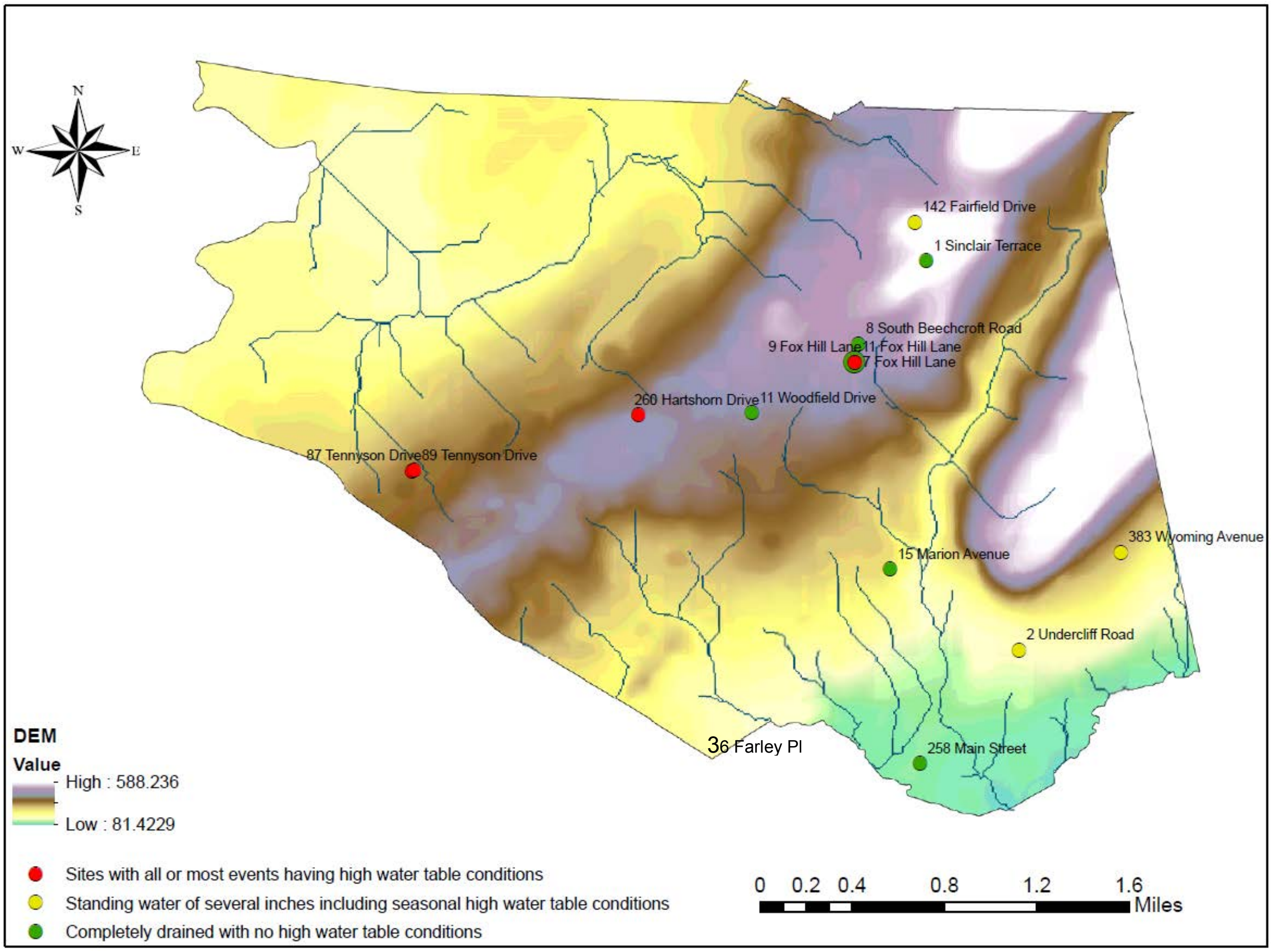


Figure 1-7. Township map showing locations having varying standing water conditions in monitored dry wells.

Table 1-5 compares the observed Horton equation coefficients for the two well-drained categories. The standing water data are not shown on this table as most of the observations could not be successfully fitted to the Horton equation. The almost steady infiltration rates (but with substantial variation) were all very low for those conditions and likely represent the f_c conditions only and were therefore included in that parameter category.

Table 1-5. Observed and Reported Horton Equation Coefficients (average and COV values)

	f_o (in./hr)	f_c (in./hr)	k (1/min)
Surface A and B soils well drained A subsurface soils (average and COV)	**44.6 (0.53)**	**5.6 (0.2)**	**0.06 (0.22)**
Surface C and D soils well drained A and B subsurface soils (average and COV)	**4.3 (0.64)**	**0.45 (0.85)**	**0.01 (0.63)**

(1 in./hr = 25.4 mm/hr)

Even sites having surface C and D soils (not acceptable infiltration sites according to the NJ dry well standards) had much better subsurface conditions where the dry wells were located than the surface conditions. The infiltration rates for these conditions were less than for the excellent areas having A and B surface soils, but all met the infiltration rate criteria of the state guidelines.

Table 1-6 lists the most stringent regulatory levels for groundwater contaminants derived from N.J.A.C. 7:9C (2010), along with the range of observed concentrations for each constituent during these tests. The microbiological and lead concentrations frequently exceeded the groundwater disposal criteria.

Table 1-6. Groundwater Quality Criteria for the State of New Jersey Compared to Observed Water Quality from Dry Wells

Constituent	Groundwater Quality Criterion[1]	Observed Range [1]	Fraction of samples that exceed the criteria
Microbiological criteria[2]	Standards promulgated in the Safe Drinking Water Act Regulations (N.J.A.C. 7:10-1 et seq.)[3]	Total coliform: 1 to 36,294 MPN/100 mL *E. coli*: 1 to 8,469 MPN/100 mL	Total coliform: 63 of 71 samples exceeded the criterion for total coliforms *E. coli*: 45 of 71 samples exceeded the criterion for *E. coli*
Nitrate and Nitrite	10	0.0 to 16.5 (one sample had a concentration of 16.5 mg/L)	1of 71 samples exceeded the criterion for nitrates plus nitrites
Nitrate	10	0.1 to 4.7	0
Phosphorus	n/a	0.02 to 1.36	n/a
COD	n/a	5.0 to 148	n/a
Lead	0.005	BDL to 0.38	33 of 71 samples exceeded the criterion for lead
Copper	1.3	BDL to 1.1	0
Zinc	2.0	BDL to 0.14	0

Constituent	Groundwater Quality Criterion[1]	Observed Range [1]	Fraction of samples that exceed the criteria
2,4-D	0.07	Not Detected	0
2,4,5-TP (Silvex)	0.06	Not Detected	0
2,4,5-T	0.7	Not Detected	0
Aldrin	0.00004	Not Detected	0
Alpha-BHC	0.00002	Not Detected	0
beta-BHC	0.00004	Not Detected	0
delta-BHC	n/a	Not Detected	n/a
gamma-BHC (Lindane)	0.00003	Not Detected	0
alpha-Chlordane	n/a	0.00003	n/a
gamma-Chlordane	n/a	0.00002 to 0.000024	n/a
Dieldrin	0.00003	Not Detected	0
4,4′-DDD	0.0001	Not Detected	0
4,4′-DDE	0.0001	Not Detected	0
4,4′-DDT	0.0001	Not Detected	0
Endrin	0.002	Not Detected	0
Endosulfan sulfate	0.04	Not Detected	0
Endosulfan-I	0.04	0.000032 to 0.000034	0
Heptachlor	0.00005	Not Detected	0
Heptachlor epoxide	0.0002	0.00003 to 0.000035	0
Methoxychlor	0.04	Not Detected	0
Toxaphene	0.002	Not Detected	0

[1] Ground water quality criteria and observed range are expressed as mg/L unless otherwise noted.
[2] Pursuant to prevailing Safe Drinking Water Act Regulations any positive result for fecal coliform is in violation of the MCL and is therefore an exceedance of the ground water quality criteria.
[3] 50 MPN/100 mL

Reference evapotranspiration (ET) rates for the Millburn area range from about 0.4 mm/day (0.015 in./day) during January to about 4 mm/hr (0.16 in./hr) during May through July. The period of maximum ET also corresponds to the period of maximum rainfall in the area, reducing the need for irrigation (and also the sizes of long-term water storage tanks). Therefore, the beneficial use of roof runoff for irrigation is limited if it is used only to meet the irrigation demand. However, irrigation can also be used as a stormwater management option with excess water being used to recharge the shallow groundwater and to meet the increased moisture needs of some heavily watered lawns (such as common Kentucky Bluegrass).

Rain gardens are another viable alternative for stormwater management in the Millburn area, especially as they provide some groundwater quality protection and can be incorporated into the landscaping plan of the site. They likely require additional maintenance; similar to any garden, but they can be placed to receive runoff from several of the source areas on a site, increasing the overall stormwater management level. They have even been incorporated along roads, as curb-cut biofilters, resulting in significant overall runoff volume reductions (but with special care to prevent pre-mature clogging, reduced salt discharges, and appropriately sized to handle the large flow volumes).

Alternative stormwater options should be used when dry well use should be restricted, such as with the following conditions:

- poor infiltration capacity of subsurface soil layers;
- concerns about premature clogging or other failures due to sediment; discharges or snowmelt discharges to dry wells;
- seasonal or permanent high water tables; and,
- concerns about groundwater contamination potential.

Chapter 2 Background of Millburn Stormwater Management using Dry Wells

In 1999, the Township of Millburn created an ordinance that required increased runoff from new impervious areas to be directed into seepage pits (dry wells). The purpose of this project was to investigate the effectiveness of this ordinance, specifically to examine the use of dry wells as a technique to redirect surface runoff to the local shallow groundwater. The objective of this approach is to reduce local drainage and erosion problems associated with new development and increases in impervious areas of currently developed areas. The slower release of the shallow groundwater to surface streams also better simulates natural hydrologic patterns with reduced in-stream problems associated with increased rapid surface runoff. The Township of Millburn has a stable population where there is little vacant land and that all new construction within the community is performed on previously developed plots. This ordinance has impacted over 1500 properties where seepage pits have been installed on both commercial and residential properties.

Description of Millburn Dry Wells

A dry well is a subsurface infiltration stormwater disposal practice that receives stormwater runoff from surrounding areas for subsurface disposal to shallow groundwater. Dry wells reduce the direct discharges of stormwater runoff to surface receiving waters or to downstream stormwater treatment facilities. Figure 2-1 is a schematic of a dry well in Millburn. The dry wells of this study are precast concrete structures (Figure 2-2), with open bottoms resting on 0.6 m (2 ft) crushed stone layers and with 0.6 m (2 ft) of crushed stone surrounding the dry wells. Most of the dry wells receive water directly from roof drain leaders or by storm drain inlets located in driveways or small parking lots. Some also have grated covers and receive surface runoff from the surrounding lawn or paved areas.

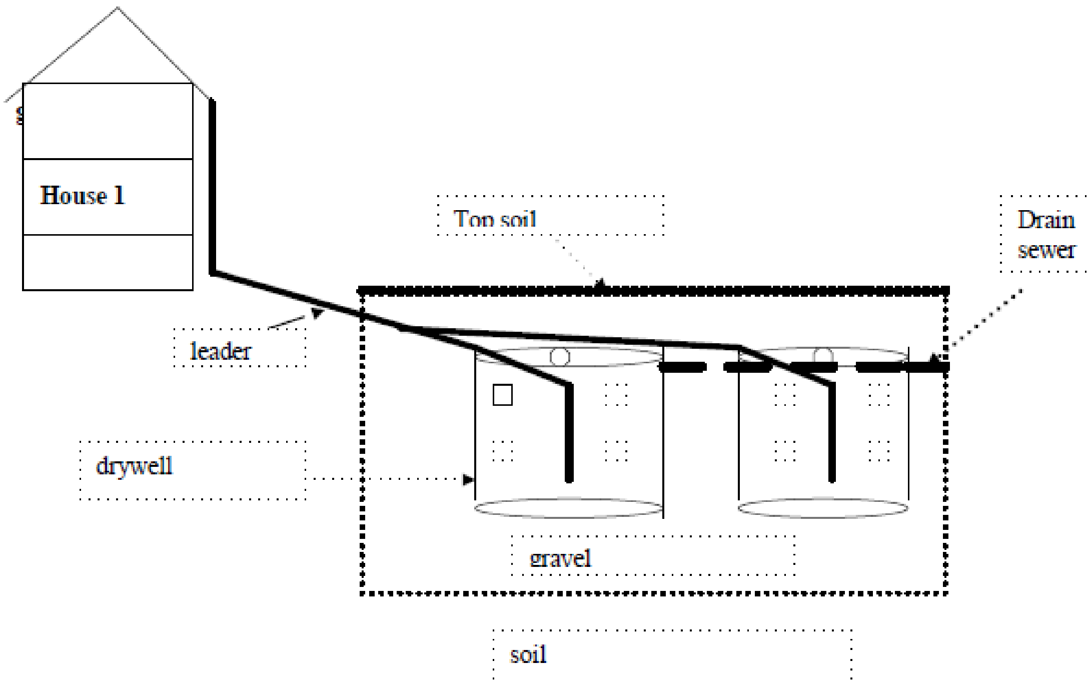


Figure 2-1. Schematic of Millburn Dry Wells

Figure 2-2. Peerless Concrete Products, Butler, NJ supplies the dry wells to many of the sites in Millburn (photo from http://www.peerlessconcrete.com/).

About 85% of the dry wells receive roof runoff only, and about 98% are in residential areas. The general design requirements are to provide 9 m^3 (250 ft^3) of void volume in the dry wells for every 110 m^2 (1,000 ft^2) of impervious drainage area. In most cases, one perforated concrete tank dry well is used for every 110 m^2 (1,000 ft^2) of impervious area. The most common dry wells used are open bottomed precast 1.8 m (6 ft) diameter and 1.8 m (6 ft) deep having numerous holes. These are installed with a 0.6 m (2 ft) blanket of crushed stone on the bottom and around the sides (Figure 2-3). The dry wells also have overflows to the storm drain system. The subgrade soils' permeability rate must be sufficient to drain the runoff in the tank within 72 hr (NJ Stormwater BMP Manual, pg 9.3-1). Each of these dry wells has about 4.8 m^3 (170 ft^3) of void storage, so the average 1.5 dry well per average lot provides about 7.1 m^3 (250 ft^3) of storage, or the amount required for 90 m^2 (1,000 ft^2) of impervious surfaces. Roof leaders enter the dry wells directly, after passing through a small cleanout. In some cases, a water storage tank (non-perforated) is installed upgradient from the dry well with a small pump to provide irrigation water. The lot sizes where the dry wells are used range from 9.1 m (30 ft) by 30 m (100 ft) to 2 acre sites, although most are 1,900 to 2,800 m^2 (20,000 to 30,000 ft^2), with an average of about 1-1/2 dry wells per lot.

Figure 2-3. Installed dry well in Millburn, NJ, showing the surrounding blanket of crushed stone before completion of the backfilling (photo from Mel Singer).

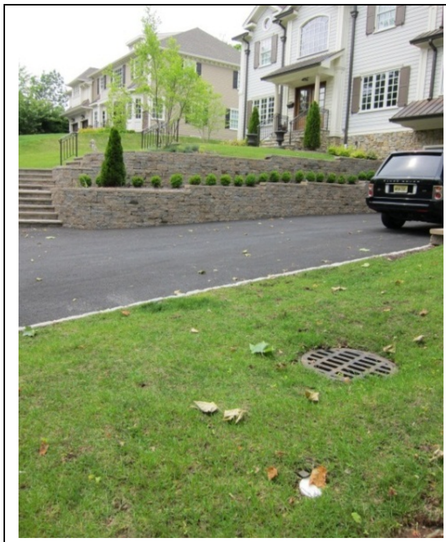
The Township estimates the cost of a typical dry well installation to be about $4,100 (Table 2-1). They have also found that with most of the roof water being directed to the dry wells instead of into residents' plant beds, the dry wells can save homeowners as much as $700 per year in landscape maintenance costs since the roof drain discharge doesn't wash away the mulch and erode topsoil.

Table 2-1. Cost Breakdown for a Typical Dry Well Installation

Dry well (6 ft dia., 6 ft deep)	$1,000
Clean stone (2 ft at bottom and sides)	$680
Excavation (10 ft x 10 ft x 9 ft deep)	$1,200
Manhole Casting	$300
Drain Pipes (sch. 40 PVC)	$900
TOTAL	$4,080

1 ft = 3.05 m

Photographs of dry wells and surrounding areas in Millburn are shown in Figures 2-4 through 2-6. Figure 2-4 shows typical dry wells located in both front and backyards in the area that were considered for the initial controlled infiltration tests using Township fire hydrant water. Some of these locations were not used due to access problems or distance from the fire hydrants. They show that many of the dry wells are located in landscaped areas and have open covers, allowing surface runoff from the lawns to enter the dry wells, as well as the subsurface piped roof runoff. Some are also located in paved areas, also allowing surface runoff from the driveways to enter along with the roof runoff. Figure 2-5 shows the home with the cistern that was included in the water quality monitoring, including the inlet strainer filter to capture leaves from the roof. Figure 2-6 shows two examples of dry wells, one that is draining well and the other with long-term standing water. Both were open to the surface and received runoff from landscaped areas and contained some lawn and leaf debris.



Front yard dry well also receiving lawn area inflows.

Backyard dry well sealed against surface inflows.

Figure 2-4A. Site photographs of dry wells in Millburn.

 Backyard dry well showing lawn area also as a source.	 Backyard dry well on Wyoming St. showing driveway runoff also as a source.
 Front yard dry well only receiving roof runoff.	 Backyard dry well at new home on Parsonage Hill Rd used for water quality monitoring; sealed against surface inflows.

Figure 2-4B. Site photographs of dry wells in Millburn.

Frontyard dry well at new home used for water quality monitoring on Slope Drive open grating receives surface inflows in addition to roof runoff.	Backyard dry well at new home on Tennyson Rd. used for water quality monitoring (monitoring well caps shown), sealed against surface inflows.

Figure 2-4C. Site photographs of dry wells in Millburn.


Landscaping in yard having underground cistern.


Cistern manhole.


Cistern inlet filter.


Cistern irrigation pump.

Figure 2-5. Cistern monitoring location on Minnisink Rd.

Dry well showing crushed stones at bottom with some minor lawn debris.

Dry well with standing water and some leaves.

Figure 2-6. Example dry wells completely draining and with standing water.

Chapter 3 Millburn, NJ, Study Site Descriptions

Millburn, NJ, is a typical affluent suburb in New Jersey in the southwest corner of Essex County and is situated about 21 miles west of New York City. It is a mature community of 6,450 acres (about 10 square miles), with less than 15 per cent of its land vacant. There are approximately 7,195 residential homes with a population of 20,149 (2010 US census). The community has a normal mix of commercial and retail establishments, parks and schools and an upscale shopping mall.

About 5,900 homes are detached single-family units and about 1,500 have dry wells. Some also have water storage tanks before the seepage pits for irrigation use withdrawals. The city has no above ground detention facilities. About 60% of the community water supply is from public wells. The groundwater table is as shallow as 2.4 to 3 m (8 to 10 ft) along the river in town. The soils vary greatly in the community, with large amounts of clayey soils. The following photographs show some of the neighborhoods in Millburn (from Google).

Figure 3-1. Millburn, NJ, high density residential neighborhood (Google)

Figure 3-2. A medium density residential neighborhood in Millburn, NJ (Google)

Site Descriptions

Table 3-1 lists locations of the study sites in the Township of Millburn, Essex County, NJ where dry well water level measurements were obtained for different rain events. All of the study sites are residential buildings with one or two families. Three of the dry wells as well as the cistern were also instrumented with monitoring underdrains for water quality monitoring. Table 3-2 lists water quality sampling locations.

Table 3-1. Infiltration Monitoring Dry Well Locations, Township of Millburn, NJ, 07078

1 Sinclair Terrace
15 Marion Avenue
258 Main Street
36 Farley Place, Short Hills (Linda's Florist)
11 Fox Hill Lane
11 Woodfield Drive
142 Fairfield Drive
2 Undercliff Road
260 Hartshorn Drive
383 Wyoming Avenue
7 Fox Hill Lane
8 South Beechcroft Road
87/89 Tennyson Drive
9 Fox Hill Lane

Table 3-2. Water Quality Monitoring Dry Well and Cistern Locations, Township of Millburn, NJ, 07078

135 Tennyson Road (dry well location)
18 Slope Drive (dry well location)
139 Parsonage Hill Road (dry well location)
79 Minnisink Road (cistern location)

Table 3-3 is a summary of the impervious drainage areas, and the dry well system storage volumes for some of the monitored dry wells, along with the drainage area to storage volume and approximate date of construction. These dates were on the plan drawings and maps. The ages of the dry wells monitored therefore ranged from about 5 years to new. The ratios of the drainage areas to the storage volumes ranged from about 5 to 10 ft^2 per ft^3 (or 17 to 33 m^2 drainage area per m^3 of storage). Unfortunately, this information is not complete for all of the test areas and some of the information is suspect. It was hoped that this information could be compared to the observed performance of the dry wells to indicate any degradation with age or source area treated.

Table 3-3. Dates of Final Construction Drawings, Impervious Drainage Areas, and Dry Well Storage Volumes for Selected Dry Wells Studied

Site Location	Drainage area (ft^2)	Description of Drainage Area	Date (from maps)	Tank Specifications					Ratio of contributing area to dry well volume (ft^2/ft^3)**
				Tank dia. (ft)	Tank depth (ft)	Number of tanks	Volume per tank (ft^3)	Total volume of dry wells (ft^3)	
11 Woodfield Dr	900	Driveway	11/29/2007	6	6	1	170	170	5.3
383 Wyoming Ave.	600	Lawn Area*	6/11/2009	8	6	2	300	600	1.0 (?)*
260 Hartshorn	5,003	Impervious Area	5/6/2009	8	6	3	300	900	5.6
87/89 Tennyson Drive	6,044	Impervious Area	4/18/2005	8	8	2	338	675	9.0
1 Sinclair	1,324	Addition	7/11/2008	7.5	4.5	1	199	199	6.7
11 Fox Hill Lane	3,633	Entire Property	2/21/2008	3.5	7	5	67	337	10.8
7 Fox Hill Lane	3,633	Entire Property	3/18/2008	3.5	7	5	67	337	10.8
8 S Beechcroft	15,000	Water drains from several properties *	8/23/2007	6	5.25	1	148	148	101 (?)*

* Likely errors in source area descriptions
** The local design guidelines require 250 ft^3 of dry well storage (including void space in the crushed stone blanket) for every 1,000 ft^2 impervious drainage area (ratio of 4)
(1 ft = 3.05 m, 1 ft^2 = 0.093 m^2)

Aerial Photos of Study Locations

Figure 3-3 is a large scale map showing the locations of the study areas in the Township of Millburn (www.maps.google.com). The following are aerial photographs for each site as well as some of the dry wells. Appendix A also includes drawings of the dry well installations for each site.

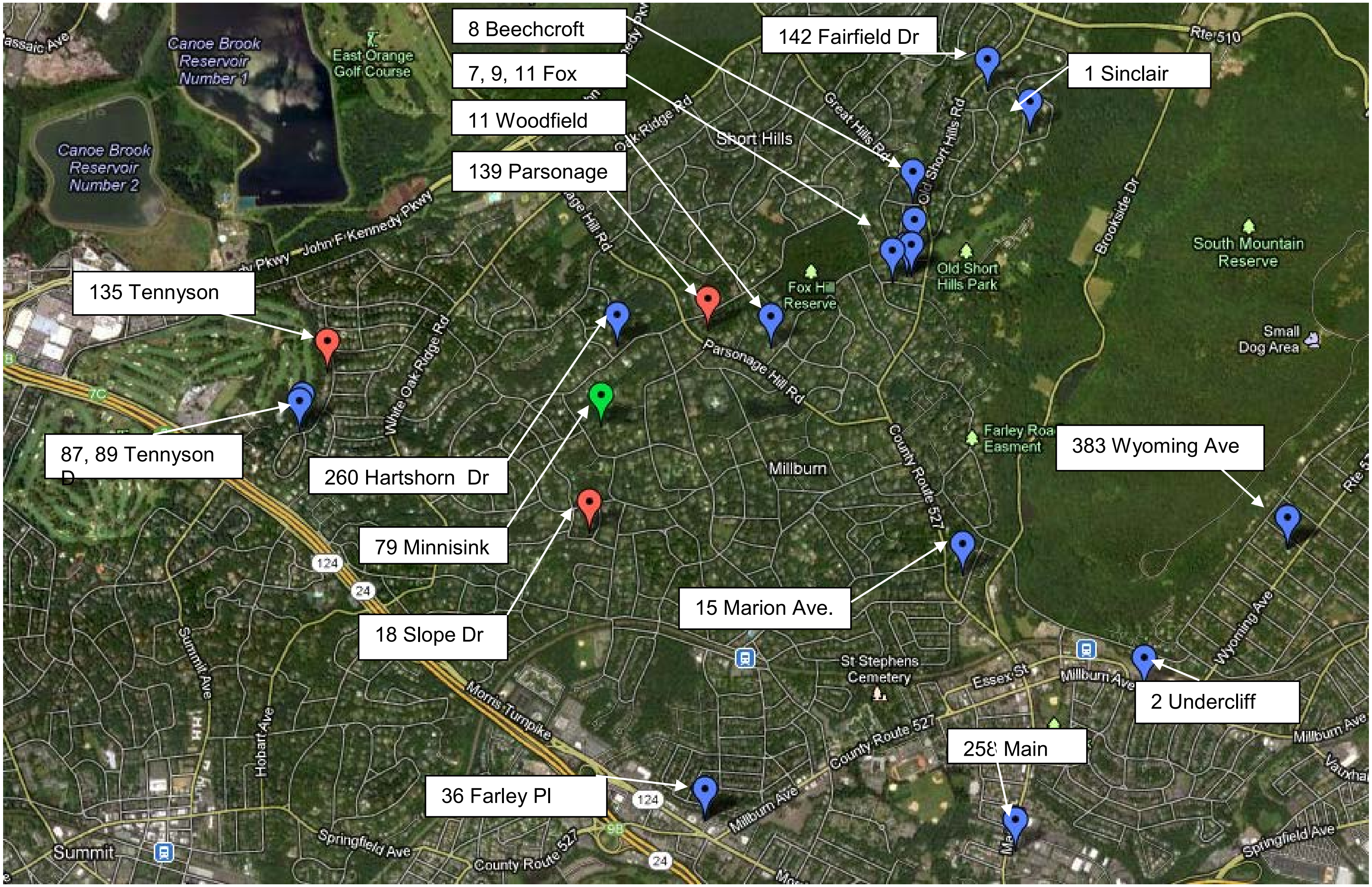


Figure 3-3. Locations of infiltration dry wells (shown with blue icons) and cistern (79 Minnisink, green icon) and water quality monitoring dry wells (shown with red icons)

8 South Beechcroft

Dry well (8 South Beechcroft)

11 Woodfield Dr.

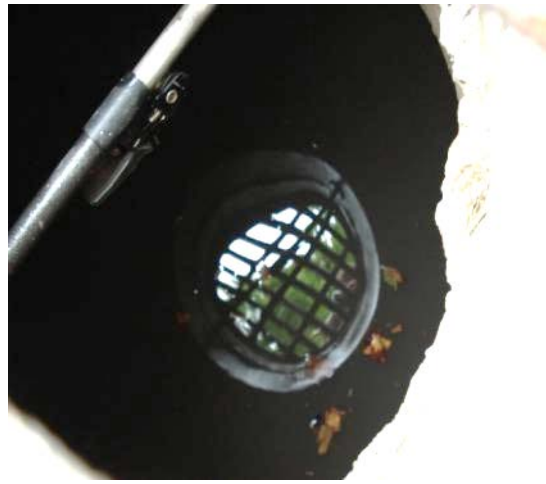
Dry well (11 Woodfield Dr.)

Figure 3-4A Aerial Photos and photos of dry wells for study areas in Millburn, NJ

 11 Fox Hill Lane	 Dry well (11 Fox Hill LN)
 1 Sinclair Terrace	 Dry well with standing water (1 Sinclair Terrace)

Figure 3-4B Aerial Photos and photos of dry wells for study areas in Millburn, NJ

383 Wyoming



Dry well with standing water (383 Wyoming)

258 Main St.

Dry well (258 Main St.)

9 Fox Hill Lane

Dry well with standing water (9 Fox Hill)

Figure 3-4C Aerial Photos and photos of dry wells for study areas in Millburn, NJ

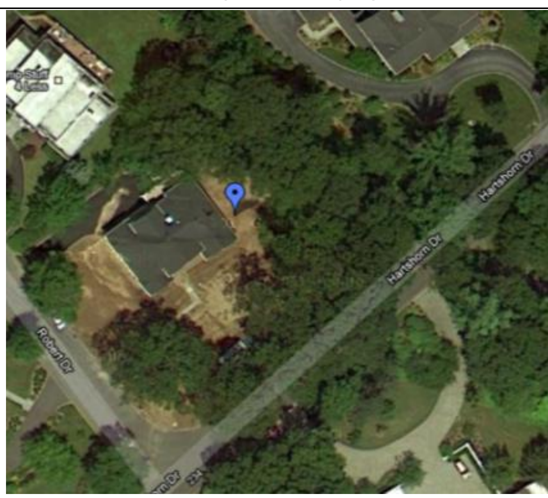
 2 Undercliff Rd	 Dry well (2 Undercliff Rd)
 7 Fox Hill Lane	 15 Marion Avenue
 260 Hartshorn Dr	 18 Slope Dr

Figure 3-4D Aerial Photos and photos of dry wells for study areas in Millburn, NJ (www.maps.google.com)

Land Cover Descriptions of Study Sites

The land covers of the project sites, including roofs, driveways, sidewalks, streets, landscaped areas, patios, etc. are shown in Table 3-4. The percentage of each of these land covers is shown in Table 3-5. These data were calculated from the plan maps for each home obtained by PARS Environmental, Inc. from the Township.

Table 3-4. Land Covers for Study Sites (Area, ft^2)

Monitoring Location	Roofs	Driveways	Parking	Side walks	Street	Landscape	Paved Patio	Rear Walkway and Steps	Shed	Deck	Total	Housing Density (units/acre)
8 South Beechcroft	2,800	2,030	0	384	3,200	21,243	381	40	0	162	30,240	1.4
11 Fox Hill	2,183	1,125	0	50	1,650	11,003	277	0	0	0	16,288	2.7
43 Browning Road S.H	2,376	980	0	110	2,200	10,557	486	0	0	0	16,710	2.6
1 Sinclair terrace	3,216	1,438	0	237	1,900	22,277	0	433	88	0	29,589	1.5
7 Fox Hill	2,435	1,070	0	380	1,800	10,952	369	0	0	0	17,006	2.6
9 Lancer	3,360	2,214	0	448	2,100	14,189	0	537	0	288	23,136	1.9
135 Tennyson Dr	1,096	990	792	274	3,240	12,680	0	0	0	0	19,076	2.3
79 Minnisink Rd	9,150	5,200	3,200	2,600	3,000	24,450	0	0	0	0	47,600	0.9
18 Slope Dr	3,713	2,812	1,406	0	6,000	10,125	0	0	0	0	24,056	1.8
139 Parsonage Hill Rd	4,560	2,246	2,722	272	5,775	18,692	0	0	0	0	34,267	1.3
Monitoring Location	Roofs	Driveways	Parking	Side walks	Street	Landscape	Paved Patio	Rear Walkway and Steps	Shed	Deck	Total	Housing Density (units/acre)
Minimum	1,096	980	0	0	1,650	10,125	0	0	0	0	16,288	0.9
Maximum	9,150	5,200	3,200	2,600	6,000	24,450	486	537	88	288	47,600	2.7
Average	3,489	2,011	812	476	3,087	15,617	151	101	9	45	25,797	1.9
Standard Deviation	2,201	1,292	1,232	761	1,586	5,507	201	204	28	99	9,926	0.6
Coefficient of Variation (COV)	0.6	0.6	1.5	1.6	0.5	0.4	1.3	2.0	3.2	2.2	0.4	0.3

(1 ft^2 = 0.093 m^2)

As shown in Table 3-5, most of land cover is landscaped (62%), while roofs make up about 13% of the areas and streets make up about 12.5% of the areas. The variations of these major areas are relatively small, with the COVs of these three areas all less than 0.5. The housing densities for these ten homes ranged from about 1 to 3 homes per acre, with an average of about 2 homes per acre.

Table 3-5. Land Covers for Study Sites (Area, as a percentage)

Monitoring Location	Roofs	Driveways	Parking	Side walks	Street	Landscape	Paved Patio	Rear Walkway and Steps	Shed	Deck	Total
8 South Beechcroft	9.3	6.7	0.0	1.3	10.6	70.2	1.3	0.1	0.0	0.5	100.0
11 Fox Hill	13.4	6.9	0.0	0.3	10.1	67.6	1.7	0.0	0.0	0.0	100.0
43 Browning Road S.H	14.2	5.9	0.0	0.7	13.2	63.2	2.9	0.0	0.0	0.0	100.0
1 Sinclair terrace	10.9	4.9	0.0	0.8	6.4	75.3	0.0	1.5	0.3	0.0	100.0
7 Fox Hill	14.3	6.3	0.0	2.2	10.6	64.4	2.2	0.0	0.0	0.0	100.0
9 Lancer	14.5	9.6	0.0	1.9	9.1	61.3	0.0	2.3	0.0	1.2	100.0
135 Tennyson Dr	5.7	5.2	4.2	1.4	17.0	66.5	0.0	0.0	0.0	0.0	100.0
79 Minnisink Rd	19.2	10.9	6.7	5.5	6.3	51.4	0.0	0.0	0.0	0.0	100.0
18 Slope Dr	15.4	11.7	5.8	0.0	24.9	42.1	0.0	0.0	0.0	0.0	100.0
139 Parsonage Hill Rd	13.3	6.6	7.9	0.8	16.9	54.5	0.0	0.0	0.0	0.0	100.0
Monitoring Location	**Roofs**	**Driveways**	**Parking**	**Side walks**	**Street**	**Landscape**	**Paved Patio**	**Rear Walkway and Steps**	**Shed**	**Deck**	
Minimum	5.7	4.9	0.0	0.0	6.3	42.1	0.0	0.0	0.0	0.0	
Maximum	19.2	11.7	7.9	5.5	24.9	75.3	2.9	2.3	0.3	1.2	
Average	13.0	7.5	2.5	1.5	12.5	61.6	0.8	0.4	0.0	0.2	
Standard Deviation	3.7	2.4	3.3	1.6	5.7	9.8	1.1	0.8	0.1	0.4	
Coefficient of Variation (COV)	0.3	0.3	1.3	1.0	0.5	0.2	1.4	2.1	3.2	2.3	

Features Affecting Water Use

Population, Residences, and Householder Data

Demographic information is needed when evaluating beneficial stormwater use potential for an area. Information concerning population and householder social-economic conditions was obtained from the U.S. Census Bureau, based on the 2000 census for the Millburn Township zip codes 07078 and 07041. Household data was also confirmed by the Township Engineer.

Table 3-6. Summary of Census 2000 Information for Zip Codes 07078 and 07041 (Source: U.S. Census Bureau)

Zip Code	Population	Total Housing Units	Occupied Housing Units	Average Household Size
07078	12,849	4,337	4,256	3.02
07041	6,880	2,809	2,747	2.5
Total	19,729	7,146	7,003	2.81

Soil types

Soil characteristics are also needed when evaluating stormwater infiltration and recharge potential for an area and for designing these control practices. Table 3-7 lists locations of sites where infiltration measurements were made, along with the ID of each as shown on the map (Figure 3-5). Figure 3-5 is a map of surface soil types for the Township of Millburn. The soil spatial and tabular map data were obtained from the Natural Resources Conservation Service (NRCS) Soil Survey for Essex County and imported into ArcMap 10. Most of the sites have "BowtB" soil type (Boonton - Urban land, Boonton substratum complex, terminal moraine). Figure 3-6 shows the Hydrologic Soil Group (HSG) for the surface soils.

Table 3-7. Locations of Infiltration Monitoring Sites and Soil Conditions in Millburn and Short Hills, NJ

Street Address	City	Latitude	Longitude	ID on Map (Fig. 3-5)	Surface Soil Name[1]	Surface Soil HSG[2]
1 Sinclair Terrace	Millburn	40.749	-74.307	1	BowtB	D
15 Marion Avenue	Millburn	40.729	-74.311	2	BowtB	D
258 Main Street	Millburn	40.717	-74.308	3	DuuB	A and D
11 Fox Hill Lane	Millburn	40.743	-74.314	4	BowtB	D
11 Woodfield Drive	Millburn	40.740	-74.322	5	BowtB	D
142 Fairfield Drive	Millburn	40.751	-74.310	6	BowtB	D
2 Undercliff Road	Millburn	40.724	-74.300	7	BowrB	C
260 Hartshorn Drive	Millburn	40.739	-74.331	8	BowtB	D
383 Wyoming Avenue	Millburn	40.730	-74.291	9	BowrB	C
7 Fox Hill Lane	Millburn	40.742	-74.314	10	BowtB	D
79 Minnisink Road	Millburn	40.736	-74.332	11	BowtC	D
8 South Beechcroft Road	Millburn	40.743	-74.314	12	BowtB	D
87/89 Tennyson Drive	Millburn	40.735	-74.350	13 and 14	BowtB	D
9 Fox Hill Lane	Millburn	40.742	-74.315	15	BowtB	D
36 Farley Pl	Short Hills	40.718	-74.326	16	UrbanB	D

[1] Natural Resources Conservation Service (NRCS)
[2] Source: *Soil Survey of Essex County*, New Jersey Report, USDA, NRCS.

Table 3-8 summarizes the surface and subsurface soil characteristics for the Millburn sites using the NRCS on-line soil survey. All the sites have surface soils with hydrologic soil group (HSG) “C” or “D”, except for the Main St. area that has “A” soils. Group “A” soils have a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission. Group “C” Soils have slow infiltration rates when thoroughly wet. These consist mainly of soils having a layer that impedes the downward movement of water or soils of moderately fine to fine texture. Group “D” soils have a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. Group D soils have a very slow rate of water transmission. All of the sites’ subsurface soils shown on Table 3-8 are well drained. The dry wells are usually 2.4 m or 8 ft deep (2 ft of surface cover with a 6 ft tall concrete perforated tank), with another 2.4 m (2 ft) of gravel, so the main infiltration layer is from 0.6 m (2 ft) to about 3.1m (10 ft) below the ground surface. The soil profiles indicate increased infiltration potentials at these deeper soil depths, with all subsurface soils being group A or B from about 2.4 m (2 ft) and deeper, as shown on Figure 3-7, which likely better indicates the potential function of the dry wells compared to the surface soil conditions.

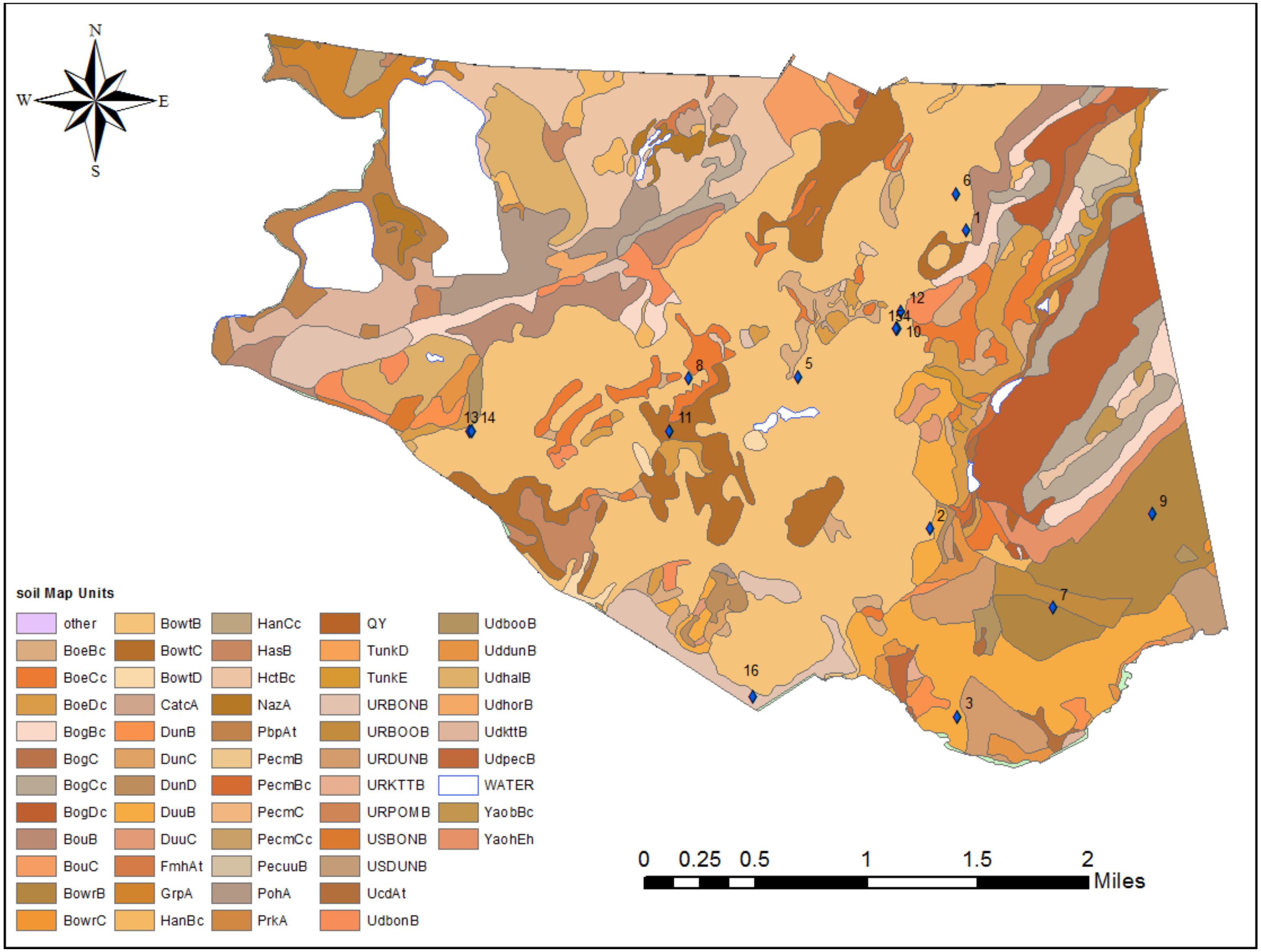


Figure 3-5. Soil Map for the Township of Millburn (NRCS; http://soils.usda.gov/)

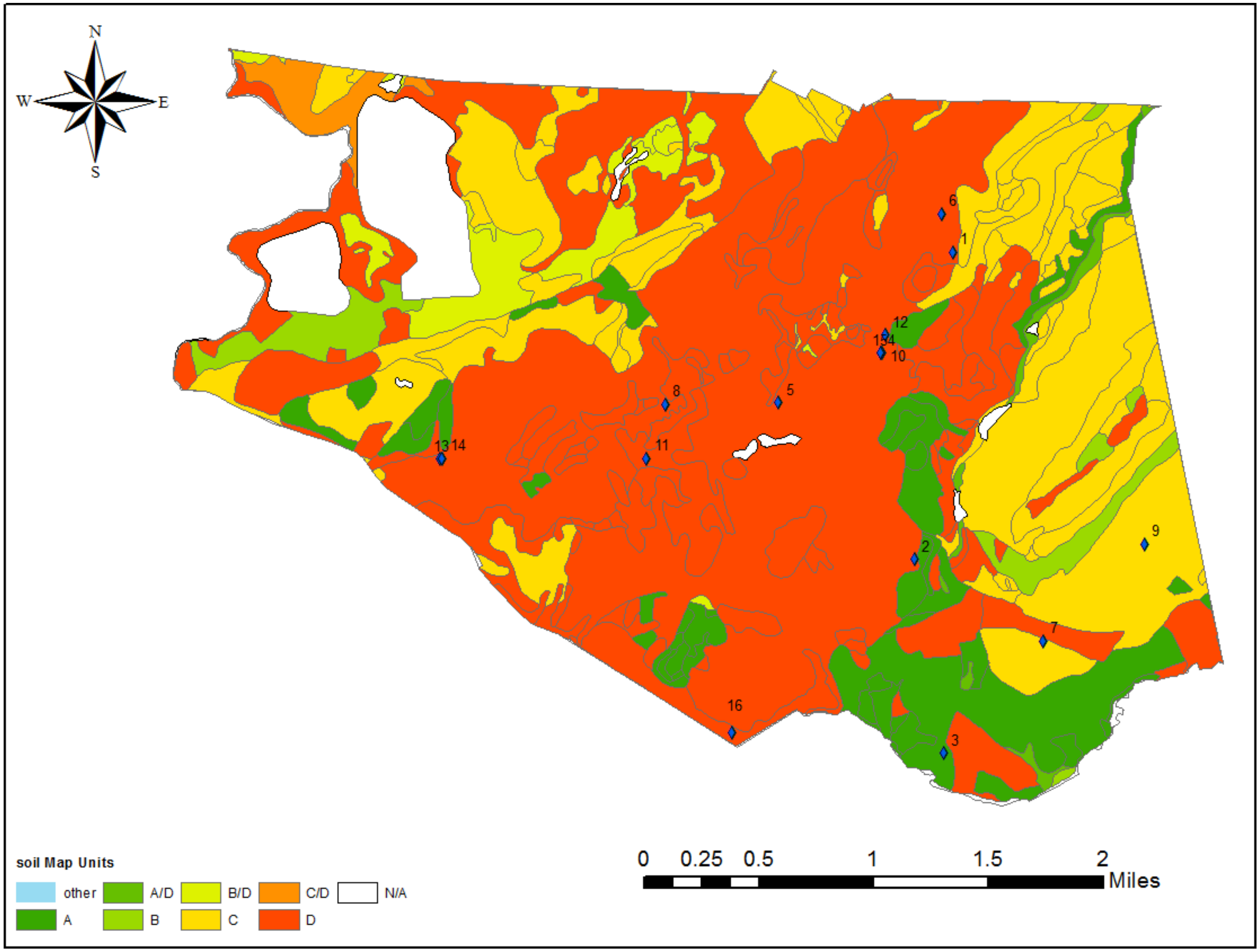


Figure 3-6. Hydrologic Soil Group Index of the Township of Millburn for Surface Soils (NRCS; http://soils.usda.gov/)

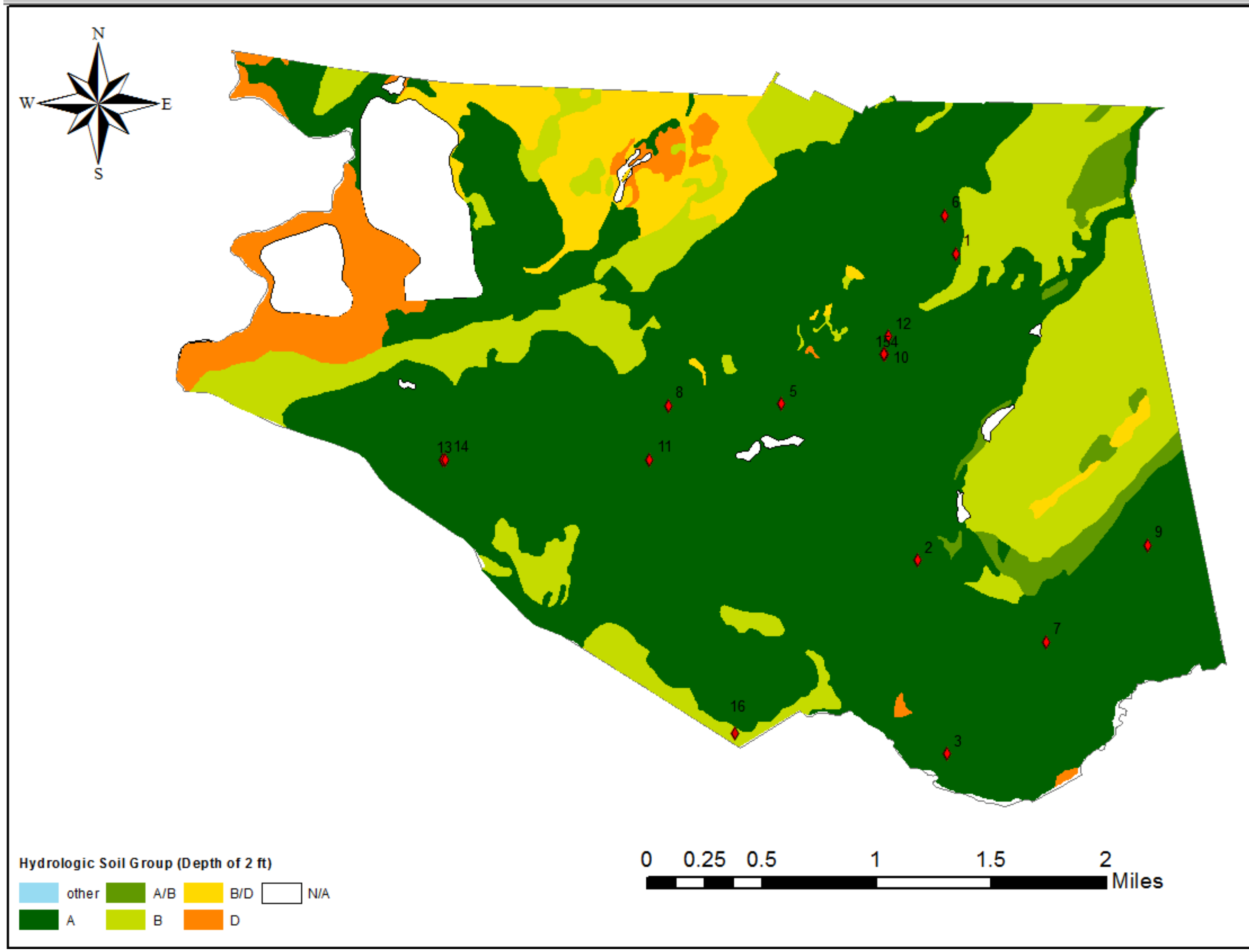


Figure 3-7. Hydrologic Soil Group Index of the Township of Millburn for Shallow Subsurface Soils 2 ft Deep (NRCS; http://soils.usda.gov/)

Table 3-8 Summary of soil characteristics
(Source: http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

Address	Soil Name	Slope (%)	K_{sat}[1]	Drainage class	Typical profile and associated Hydrologic Soil Groups for subsurface soils
383 Wyoming Ave. 90 Chestnut St.	Boonton-Urban land, Boonton substratum complex, red sandstone lowland	3-8	Moderately low to moderately high (0.06 to 0.20 in./hr)	Well drained	0 to1 in.: Slightly decomposed plant (C) 1-3 in.: Silt loam (C) 3-10 in.: Loam (C) 10-27 in.: Gravelly loam (B) 27-67 in.: Gravelly fine sandy loam (A) 67-83 in.: Gravelly sandy loam (A)
258 Main St.	Dunellen sandy loam	3-8	High (1.98 to 5.95 in./hr)	Well drained	0-42 in.: Sandy loam (A) 42-70 in.: Stratified gravelly sand to sand to loamy sand (A)
260 Hartshorn 142 Fairfield 87/89 Tennyson 7 Fox Hill 9 Fox Hill 11 Fox Hill 8 South Beechcroft 2 Undercliff Linda's Flower 15 Marion 11 Woodfield Dr	Boonton - Urban land, Boonton substratum complex, terminal moraine	3-8	Moderately low to moderately high (0.06 to 0.20 in./hr)	Well drained	0 to 1 in.: Highly decomposed plant (D) 1-24 in.: Sandy loam (B) 24-42 in.: Gravelly sandy loam (A) 42-60 in.: Fine sandy loam (B)
9 Lancer	Boonton - Urban land, Boonton substratum complex	8-15	Moderately low to moderately high (0.06 to 0.20 in./hr)	Well drained	0-5 in.: Loam (B/C) 5-30 in.: Silt loam (B) 30-40 in.: Gravelly fine sandy loam (A) 40-47 in.: Fine sandy loam (A) 47-72 in.: Loamy sand (A)
1 Sinclair Terrace	Boonton - Urban land, Boonton substratum complex	0-8	Moderately low to moderately high (0.06 to 0.20 in./hr)	Well drained	0-5 in.: Loam (B/D) 5-30 in.: Silt loam (B) 30-40 in.: Gravelly fine sandy loam (A) 40-47 in.: Fine sandy loam (A) 47-72 in.: Loamy sand (A)
36 Farley Place	Urban land, Boonton substratum	0-8	Moderate to moderately rapid	Well drained	0-12 in.: impervious material (D) 12-47 in.: silt loam (C) 47-72 in.: loamy sand (A)

[1]Capacity of the most limiting layer to transmit water
1 inch = 2.54 cm
Source: http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm

Groundwater Conditions in the Township of Millburn

Unfortunately, groundwater depth conditions are not readily available for the Township of Millburn. The geology in the area is comprised mostly of glacial deposits down to bedrock and bedrock in that area is encountered anywhere between 30 m to 60 m (100 to 200 ft) below the ground surface (bgs). The water table is generally encountered > 9 m (>25 ft) bgs (personnel communication, Michael D. Moore, PG, LSRP, Senior Project Manager, PARS Environmental, Inc., Robbinsville, NJ). The NJDEP OPRA database generally shows depth to water measurements at:

http://datamine2.state.nj.us/DEP_OPRA/OpraMain/categories?category=WS+Well+Permits

However, groundwater depth data were not available for the study area, although other sites in Essex County were included. The groundwater table is reported by the Township to be as shallow as 2.4 to 3 m (8 to 10 ft) along the river in town. Therefore, it is assumed that generally, shallow groundwater conditions are not likely in the study area, except possibly in low-lying areas.

Rainfall Characteristics in Northern New Jersey

Long-term rainfall characteristics for Newark, NJ, were examined as part of this study. As noted elsewhere in this report, four short-term rain gages were installed in the project vicinity to obtain rain information corresponding to the monitoring activities. However, because these were only in use for several months, they do not provide adequate information concerning the expected long-term rain conditions in the area. Therefore, more than 50 years of continuous rain records covering the period from May 1948 through December 1999 from the Newark International Airport (located less than 10 miles from the Township of Millburn) were examined. WinSLAMM was used to combine the individual hourly rain depths supplied by EarthInfo CDRoms (of NOAA data) into rain events. Each rain event was defined as having measureable rainfall (at least 0.025 cm (0.01 in.) of rain) with a preceding interevent dry period of at least 6 hr. Table 3-9 summarizes this rain information for this period.

Table 3-9. Newark, NJ, Rain Characteristics (1948 through 1999)

	Rain depth (in.)	Rain duration (hr)	Average event rain intensity (in./hr)	Preceding interevent dry period (days)
Total per year	44.0	810		331
Minimum	0.01	1	0	0.04
1st percentile	0.01	1	0.01	0.3
10th percentile	0.01	1	0.01	0.4
25th percentile	0.04	2	0.02	0.8
50th percentile (median)	0.18	5	0.03	2.3
75th percentile	0.55	11	0.07	4.4
90th percentile	1.11	18	0.12	7.2
99th percentile	2.81	34	0.35	14.8
Maximum	8.25	80	1.00	33.9
Average	0.42	7.8	0.05	3.2
Standard deviation	0.60	7.8	0.07	3.1
COV	1.4	1.0	1.3	1.0

1 in. = 2.54 cm

Figure 3-8 shows the distribution of the rain depths with time for this 52 year period. The three largest rains (15 to 20 cm, or 6 to 8 in.) are quite distinct from the other events. Figure 3-9 (Pitt 1999) displays probability plots of rain events (by count) and runoff volumes for ten years for the Newark airport. This plot shows that the median rain depth is about 5 mm (0.2 in.), but this rain (and smaller events), only accounts for about 10% of the total annual runoff volumes from typical residential and commercial sites. Most (about 75%) of the annual runoff, and therefore stormwater pollutants, are associated with rains in the range from about 10 mm to 64 mm (0.4 to 2.5 in.), with about another 15% of the runoff associated with rains smaller than 10 mm (0.4 in.) and about 10% of the annual runoff associated with rains larger than about 64 mm (2.5 in.).

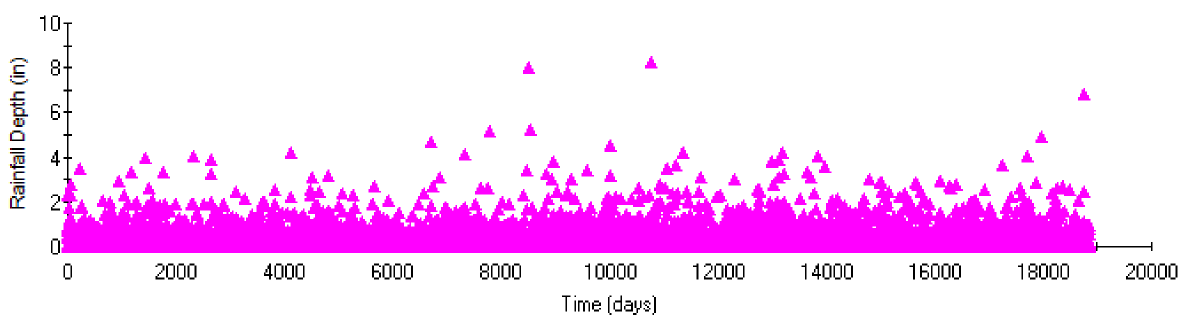


Figure 3-8. Rain depth distribution with time for Newark, NJ, for 1948 through 1999 (WinSLAMM rain file data plot).

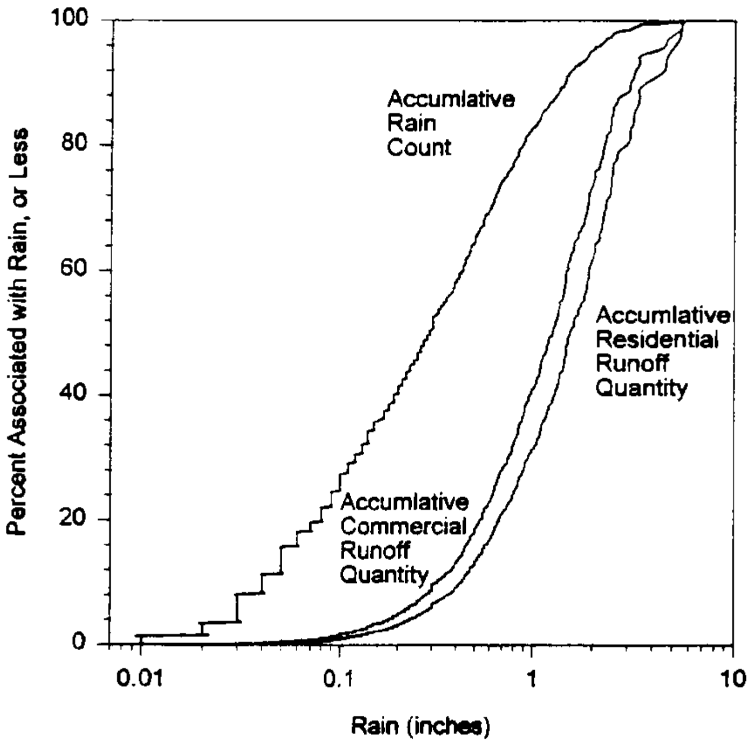


Figure 3-9. Probability distribution of rain events and runoff quantities for Newark, NJ (1982 through 1992) (Pitt 1999).

Typical rain durations (10 to 90th percentiles) range from about 1 to 18 hr, with the median duration being 5 hr. The average total event (average duration of 5 hr) rain intensity is only about 0.8 mm/hr (0.03 in./hr) and rarely exceeds 9 mm/hr (0.35 in./hr), with the recorded maximum only being about 25 mm/hr (1 in./hr). Peak shorter duration rain intensities are much larger, according to the local IDF (intensity, duration, frequency) relationship, shown on Figure 3-10. For a 10 minute time of concentration (typical for a small urban drainage area), the one year frequency rain intensity is about 90 mm/hr (3.5 in./hr), increasing to about 160 mm/hr (6.3 in./hr) for a rain that is expected only once every 100 years (1% probability of occurring in any one year). It is interesting to note that the 25 mm/hr (1 in./hr) maximum intensity for the typical 5 hour duration that was observed at the Newark airport in this 52 year period is expected to be exceeded with about a 2% probability per year, as observed (slightly extrapolated off the chart), indicating good agreement.

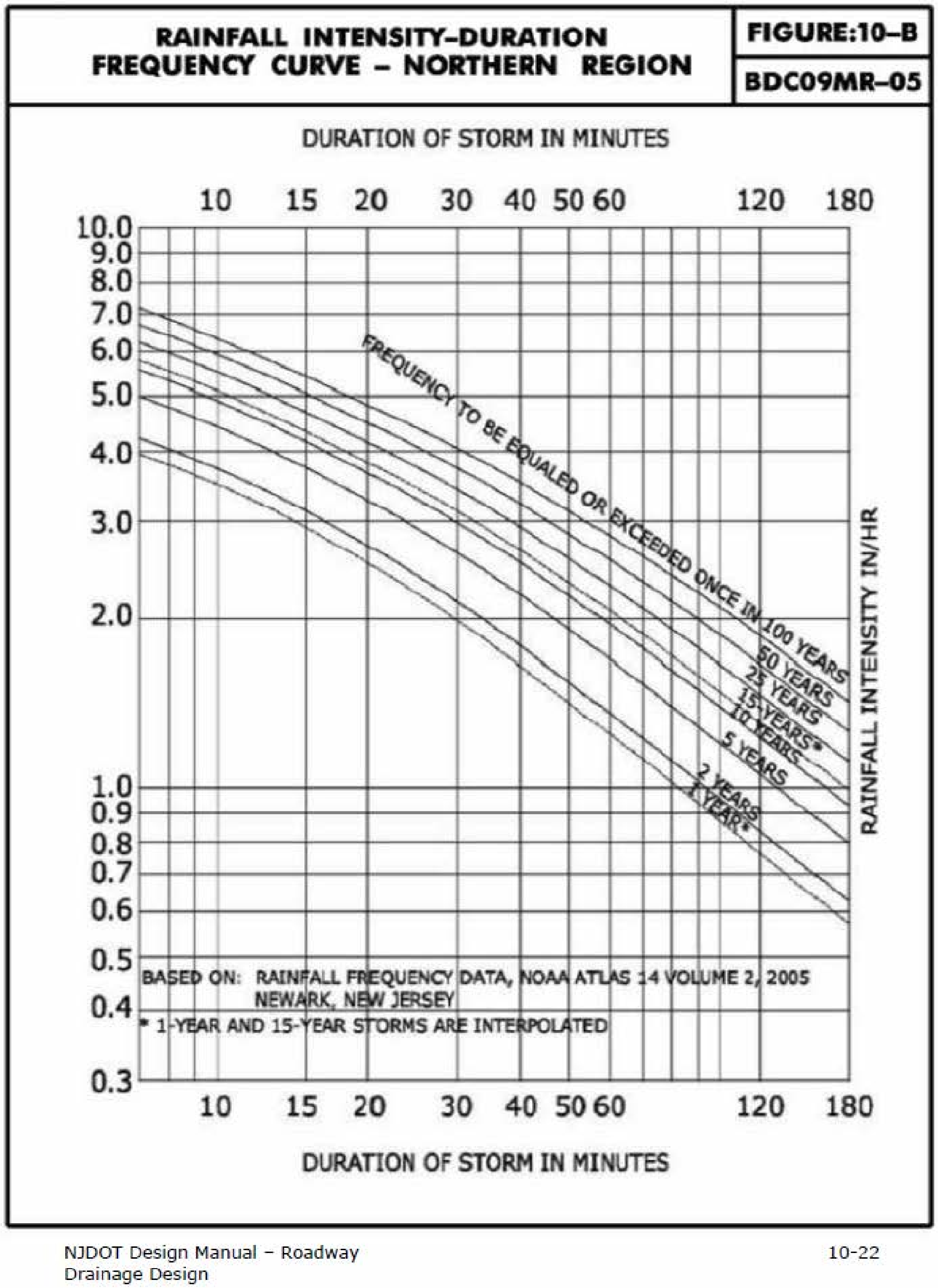


Figure 3-10. Northern New Jersey IDF curve (NJDOT Design Manual: http://www.state.nj.us/transportation/eng/documents/BDC/pdf/DMR-Sec10.pdf)

Table 3-10 lists the rainfall totals that are expected in a 24 hour period with different frequencies for New Jersey counties. The Township of Millburn is located in Essex County, having an 86 mm (3.4 in.) rainfall expected in 24 hr with about a 50% probability in any one year. This 2-year rainfall amount is the design storm depth to be used in the design of groundwater recharge devices to infiltrate excess runoff compared to pre-development conditions.

Table 3-10. New Jersey 24 hour Rainfall Frequency Data (rainfall in in.) (NJDOT Design Manual: http://www.state.nj.us/transportation/eng/documents/BDC/pdf/DMR-Sec10.pdf)

County	Rainfall Frequency Data						
	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Atlantic	2.8	3.3	4.3	5.2	6.5	7.6	8.9
Bergen	2.8	3.3	4.3	5.1	6.3	7.3	8.4
Burlington	2.8	3.4	4.3	5.2	6.4	7.6	8.8
Camden	2.8	3.3	4.3	5.1	6.3	7.3	8.5
Cape May	2.8	3.3	4.2	5.1	6.4	7.5	8.8
Cumberland	2.8	3.3	4.2	5.1	6.4	7.5	8.8
Essex	2.8	3.4	4.4	5.2	6.4	7.5	8.7
Gloucester	2.8	3.3	4.2	5.0	6.2	7.3	8.5
Hudson	2.7	3.3	4.2	5.0	6.2	7.2	8.3
Hunterdon	2.9	3.4	4.3	5.0	6.1	7.0	8.0
Mercer	2.8	3.3	4.2	5.0	6.2	7.2	8.3
Middlesex	2.8	3.3	4.3	5.1	6.4	7.4	8.6
Monmouth	2.9	3.4	4.4	5.2	6.5	7.7	8.9
Morris	3.0	3.5	4.5	5.2	6.3	7.3	8.3
Ocean	3.0	3.4	4.5	5.4	6.7	7.9	9.2
Passaic	3.0	3.5	4.4	5.3	6.5	7.5	8.7
Salem	2.8	3.3	4.2	5.0	6.2	7.3	8.5
Somerset	2.8	3.3	4.3	5.0	6.2	7.2	8.2
Sussex	2.7	3.2	4.0	4.7	5.7	6.6	7.6
Union	2.8	3.4	4.4	5.2	6.4	7.5	8.7
Warren	2.8	3.3	4.2	4.9	5.9	6.8	7.8

1 in. = 25.4 mm

Summary of Site Characteristics

The Township of Millburn, Essex County, NJ, is located near New York City, and less than 10 miles from Newark International Airport. The 2010 US census indicated the Township had a population of 20,149. Housing costs are very high (According to Wikipedia, Millburn had the highest annual property tax bills in New Jersey in 2009 at more than $19,000 per year, compared to the statewide average property tax that was $7,300 which was the highest in the country). There are about 5,900 detached homes in the Township and about 1,500 have dry wells. Fifteen dry wells were monitored for water levels during periods ranging from two months to one year, or by controlled tests using Township water from fire hydrants. Four systems (three dry wells and one cistern) were also monitored for water quality during 10 storms to indicate any difference in water quality directly below the dry wells (or at the cistern inlet) compared to deeper depths at least 0.6 m (2 ft) below the crushed stone layer beneath the dry wells (or in the cistern). Four rain gages were also installed near the dry wells.

The study sites were surveyed to obtain detailed development characteristics that affect the amount of runoff from the different source areas. Soil information was also compiled. Most of the surface soils were of HSG C or D category, indicating poor infiltration potential. However, subsurface soils where the dry wells were located were mostly of HSG A or B category (glacial deposits) with much improved infiltration potentials. The groundwater in the area may be as shallow as 2.4 to 3 m (8 to 10 ft) below the ground surface in low-lying areas along the river, but otherwise is expected to be greater than 8 m (25 ft) below the ground surface in general.

The annual rainfall for the area is about 1,120 mm (44 in.) and the median interevent period is about two days. The median rain depth is about 5 mm (0.2 in.), but this rain (and smaller events), only accounts for about 10% of the expected total annual runoff volumes from typical residential and commercial areas. Most (about 75%) of the annual runoff, and therefore stormwater pollutants, is associated with rains in the range from about 10 to 64 mm (0.4 to 2.5 in.), with about another 15% of the runoff associated with rains smaller than 10 mm (0.4 in.) and about 10% of the annual runoff associated with rains larger than about 64 mm (2.5 in.). The New Jersey requirements for stormwater list the 2-year, 24-hr rainfall amount of 87 mm (3.4 in.) for the design of groundwater recharge devices to infiltrate excess runoff compared to pre-development conditions.

Chapter 4 Millburn Township Stormwater Regulations and New Jersey State Groundwater Disposal and Water Reuse Regulations and other Guidance

This report section summarizes the *Development Regulations and Zoning Ordinance* of the Township of Millburn and the applicable New Jersey Groundwater Disposal standards applicable to dry wells. Also summarized are regulations and guidance pertaining to beneficial uses of stormwater, from New Jersey, and elsewhere.

Millburn Township Stormwater Regulations

The following sections of the *Development Regulations and Zoning Ordinance* of the Township of Millburn, Essex County, NJ (as amended through Dec 14, 2004) are the local stormwater regulations pertaining to new developments in the Township:

"Stormwater Runoff

The provisions of this section apply to all major subdivisions and site plans.
No land area shall be developed by any person such that:

a. The rate of stormwater runoff occurring at the site is increased over what occurs under existing conditions;
b. The drainage of adjacent areas is adversely affected;
c. Soil erosion during and after development is increased over what naturally occurs;
d. Soil absorption and groundwater recharge capacity is adversely affected by the proposed development;
e. The natural drainage pattern is significantly altered.

In order to supplicate as nearly as possible natural drainage conditions, regulations and control of stormwater runoff and erosion for any land area to be developed shall be through onsite stormwater detention and/or ground absorption systems such as:

a. Detention areas; which may be excavated basins, basins created through use of curbs, stabilized earthen berms or dikes, or any other form of grading which serves to temporarily impound and store water;
b. Rooftop storage through temporary impoundment and storage of stormwater on flat or slightly pitched building rooftops by use of drain outlets which restricts the stormwater runoff from the roof surface;
c. Dry wells or leeching basins which control stormwater runoff through ground absorption and temporary storage;

d. Any system of porous media, such as gravel trenches drained by porous wall or perforated pipe, which temporarily stores and dissipates stormwater through ground absorption;
f. Any combination of the above mentioned techniques which limit stormwater runoff from a given site to what presently occurred there.

Stormwater detention facilities shall be designed to contain an amount equal to the increase in volume of runoff which would result from development of any site. The volume of runoff shall be computed on the basis of the total runoff which is produced by the flood of record for the area involved, more specifically, 170 mm (6.6 in.) of rainfall in 7 hr. The system shall be designed to store the SCS Type III 100-yr, 24-hr storm.

Underground storage facilities which are designed to percolate water into the soil should be surrounded by a blanket of crushed stone or gravel which is to be a minimum 24” thick. The stone shall be separated from surrounding soil by an appropriate geotextile fabric to be approved by the Township Engineer.”

This local ordinance includes dry wells as a management option. However, there are no guidelines on how they are to be constructed (except for the 24 in. thick gravel layer surrounding the device). Their performance is to retain the existing runoff rate. Most of the dry wells are constructed in locations of existing development and this requirement is commonly interpreted as pertaining to increased runoff associated with the modifications and expansions of the existing site. If pertaining to new construction, this requirement would then refer to predevelopment conditions. A design storm is described for stormwater detention facilities, but it is not clear if this also affects the dry well designs. Also, there are no requirements pertaining to the source waters that can be directed to the dry wells.

The following section summarizes the New Jersey state regulations pertaining to dry wells that do include various restrictions for their use, specifically soil compatibility, depth to groundwater, and allowable source waters.

New Jersey Groundwater Disposal Criteria for Stormwater

The *New Jersey Stormwater Best Management Practices Manual* (Standard for Dry Wells – Chapter 9.3) includes specific design criteria for dry wells used for the disposal of stormwater. It requires sufficient storage volumes in the dry well to contain the design storm runoff volume without overflow, while the subgrade soils’ permeability rate must be sufficient to drain the stored runoff within 72 hr. Also, the manual requires that the bottom of the dry well (including the lower crushed stone layer) must be at least 2 ft above the seasonal high water table or bedrock and be as level as possible to uniformly distribute runoff infiltration over the subgrade soils. The construction of a dry well must be done without compacting the dry well’s subgrade soils. The *New Jersey Stormwater Best Management Practices Manual* (Standard for Infiltration Basins - Chapter 9.5) further requires that dry wells be used to collect only roof runoff and that the maximum drainage area to a dry well be less than one acre.

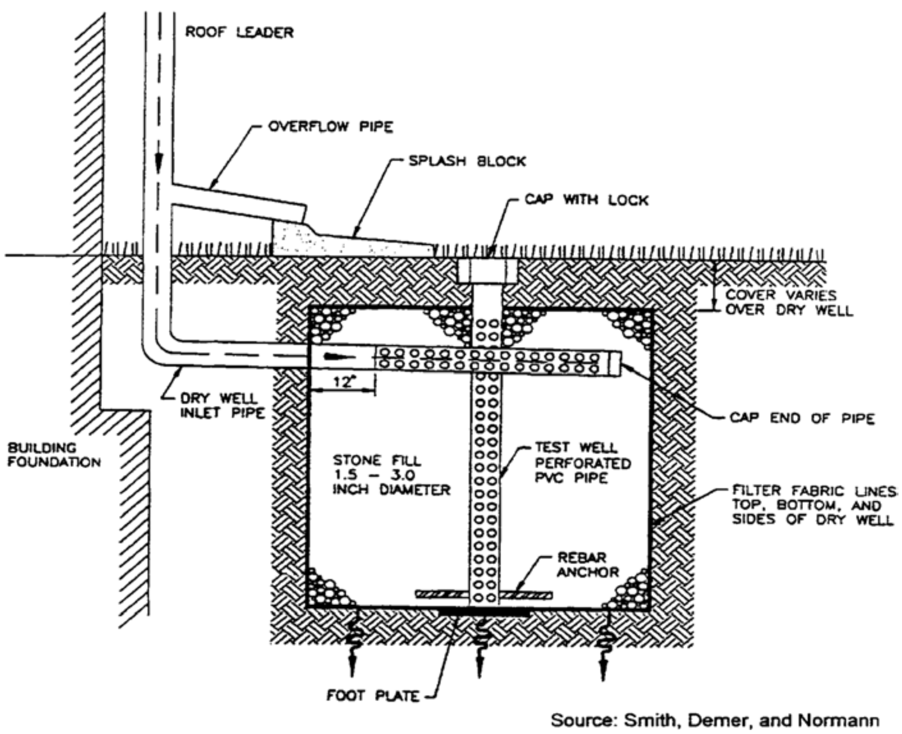
For infiltration purposes, the manual requires Hydrologic Soil Group A and B soils for dry wells designed for storms greater than the groundwater recharge storm. Additional permeability requirements are presented below in Table 4-1. It should be noted that if the dry well does receive runoff and associated pollutants from larger storm events, a minimum permeability rate of 0.5 in./hr (12.7 mm/hr) must be used.

Table 4-1. Minimum Design Permeability Rates for Dry Wells

Maximum Design Storm	Minimum Design Permeability Rate (In./hr)
Groundwater Recharge	0.2
Stormwater Quality	0.5

1 in./hr = 25.4 mm/hr

Figure 4-1 is a generic dry well illustration with its main components labeled from the New Jersey Stormwater Manual. This is an example of a stone filled dry well, while the installations monitored during this project were all perforated concrete vaults with no rock fill.



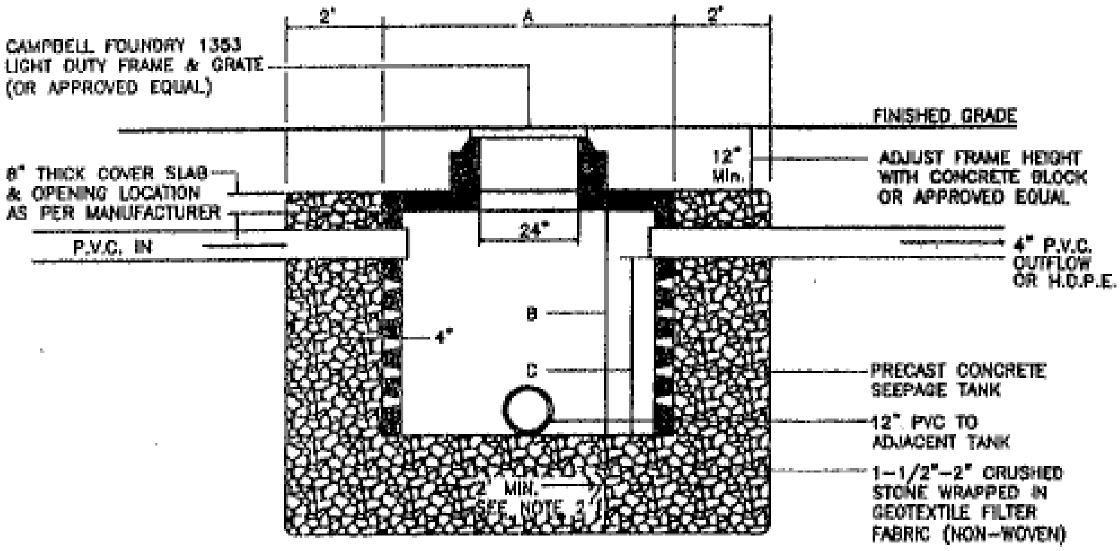
Source: Adapted from Standards for Soil Erosion and Sediment Control in New Jersey

Figure 4-1. Example dry well included in the New Jersey Stormwater Manual.

Specifically, the *Stormwater Management Rules* require that a proposed major land development comply with one of the following two groundwater recharge requirements:

- Requirement 1: That 100% of the site's average annual pre-developed groundwater recharge volume be maintained after development; or
- Requirement 2: That 100% of the difference between the site's pre- and post-development 2-year runoff volumes be infiltrated.

Chapter 6 of the *New Jersey Stormwater Manual* includes guidance and spreadsheets for the calculations of the design storm conditions for a specific site. Figure 4-2 shows a typical dry well used in the Millburn study area along with the sizing values showing how the installation complies with the regulations.



REQUIRED VOLUME OF DETENTION PER TOWNSHIP OF MILLBURN ORDINANCE: 250 CF / 1000 SF OF ADDITIONAL IMPERV. AREA

IMPERV. AREA	x SF/SF	= VOLUME REQ'D
3,701	0.2500	925.25

TANK SPECIFICATIONS						
A	B	C	Shape	#Tanks	Vol/Tank	Volume
8	8	7.25	Round	2	205	410

STONE VOLUME					
L	W	D	Tank	Void	Volume
((10	x 10	x 10)	– 205)	x .40	318

VOLUME OF DETENTION	
Type of Storage	VOLUME
Drywells	410
Stone	636
Pipe	----
TOTAL PROVIDED	1,046

Figure 4-2. Typical dry well used in Millburn study areas and volume calculations.

Beneficial Use Regulations

Many states in the U.S. have regulations and guidelines relating to reuse of waste waters. Currently, there are no federal regulations directly governing water reuse practices in the U.S. During a review by the U.S. EPA (2004), 26 states were found that had adopted regulations regarding the reuse of reclaimed water, 16 states had guidelines or design standards, and eight states had no regulations or guidelines. Some states have developed regulations for water reuse specifying water quality requirements or treatment processes to derive the maximum resource benefits of reclaimed water with respect to public health and protecting the environment. These states have set standards for reclaimed water quality and/or specified minimum treatment requirements. Generally, where unrestricted public exposure is likely in the reuse application, wastewater must be treated to a high degree prior to its application. Where exposure is not likely, however, a lower level of treatment is usually accepted. The most common parameters for which water quality limits are imposed are BOD_5, TSS, turbidity, and total or fecal coliform bacteria counts.

There is a wide range of uses of reclaimed water, but most states do not have regulations that cover them all. Current regulations and guidelines may be divided into the following reuse categories (U.S.EPA, 2004):

- Unrestricted urban reuse – irrigation of areas in which public access is not restricted, such as parks, playgrounds, school yards, and residences; toilet flushing, air conditioning, fire protection, construction, ornamental fountains, and aesthetic impoundments.
- Restricted urban reuse – irrigation of areas in which public access can be controlled, such as golf courses, cemeteries, and highway medians.
- Agricultural reuse on food crops – irrigation of food crops which are intended for direct human consumption, often further classified as to whether the food crop is to be processed or consumed raw.
- Agricultural reuse on non-food crops – irrigation of fodder, fiber, and seed crops, pasture land, commercial nurseries, and sod farms.
- Unrestricted recreational reuse – an impoundment of water in which no limitations are imposed on body-contact water recreation activities.
- Restricted recreational reuse – an impoundment of reclaimed water in which recreation is limited to fishing, boating, and other non-contact recreational activities.

Unrestricted Urban Reuse

Unrestricted urban water reuse involves irrigation of areas in which public access is not restricted, such as parks, playgrounds, school yards, and residences; toilet flushing, air conditioning, fire protection, construction, ornamental fountains, and aesthetic impoundments. This water therefore requires a higher degree of treatment than for restricted uses. In general, all states that specify a treatment process require a minimum of secondary treatment and treatment with disinfection prior to unrestricted urban reuse. These requirements obviously pertain to sanitary wastewaters, with minimal relevance to other waters, such as stormwater. Some states require additional levels of treatment such as oxidation, coagulation, and filtration. Some of the States, such as Texas, do not mention the type of treatment processes required and sets water quality limits on the reclaimed water. Table 4-2 shows the reclaimed water quality and treatment requirements for unrestricted urban reuse for New Jersey.

Table 4-2. Unrestricted Urban Reuse Regulations for New Jersey

Treatment	**Secondary treatment, filtered (chemical addition before filtration)**
BOD_5	Not specified
TSS	5 mg/L (not to be exceed before disinfection)
Turbidity	not to exceed 2 NTU
Coliform Bacteria	Fecal coliforms: - 2.2/100 mL (7-day median) - 14/100 mL (maximum any one sample)

Limits on BOD_5 range from 5 to 30 mg/L. Texas and Georgia require a BOD_5 limit of 5 mg/L while Massachusetts, Nevada, Tennessee and Washington require a BOD_5 limit of 30 mg/L. Some states have different ranges of BOD_5 for different time ranges. For example, North Carolina requires that BOD_5 not exceed 10 mg/L (monthly average), while the daily average of BOD_5 should not exceed 15 mg/L. Some states such as Florida and Ohio specify limits on CBOD which is respectively 20 mg/L and 25 mg/L. Limits on TSS vary from 5 to 30 mg/L. Florida, Georgia, Indiana, and New Jersey require a TSS limit of 5.0 mg/L prior to disinfection, while North Dakota, Tennessee, and Washington require that TSS not exceed 30 mg/L. South Carolina and North Carolina have different limits of TSS for daily and monthly averages. Limits on turbidity range from 1 to 10 NTU, but most of the states require an average turbidity limit of 2 NTU and a not-to-exceed limit of 5 NTU. Average fecal and total coliform limits range from non-detectable to 23 counts per 100 mL. Higher single sample fecal and total coliform limits are allowed in several state regulations. Florida requires that 75% of the fecal coliform samples taken over a 30 day period be below detectable levels, with no single sample in excess of 25 counts per 100 mL, while Massachusetts requires a median of no detectable fecal coliform per 100 mL over continuous seven-day sampling periods, and not to exceed 14 counts per 100 mL in any one sample.

Restricted Urban Reuse

Restricted urban reuse involves: irrigation of areas in which public access can be controlled, such as golf courses, cemeteries, and highway medians. Thus, treatment requirements may not be as strict as for unrestricted urban reuse. Some States impose the same requirements on both unrestricted and restricted urban access reuse, while others adjusted different requirements for the restricted and unrestricted categories. Table 4-3 shows the reclaimed water quality and treatment requirements for restricted urban reuse for New Jersey. The only difference between the New Jersey unrestricted and restricted urban reuse regulations is that the restricted regulations do not contain a specified turbidity limit.

Table 4-3. Restricted Urban Water Reuse Regulations for New Jersey

Treatment	Secondary treatment, filtered (chemical addition before filtration)
BOD_5	Not specified
TSS	5 mg/L (not to exceed before disinfection)
Turbidity	Not specified
Coliform	Fecal: - 2.2/100 mL (7-day median) - 14/100 mL (maximum any one sample)

Limits on BOD_5 range from 5 mg/L to 70 mg/L. Georgia requires a BOD_5 limit of 5 mg/L where Maryland requires a BOD_5 limit of 70 mg/L. Some states have different ranges of BOD_5 for different time ranges. For example South Carolina requires that BOD_5 not to exceed 30 mg/L (monthly average) where the daily average of BOD_5 should not exceed 45 mg/L. Some States such as Ohio specify limits on CBOD which is 40mg/L. Limits on TSS vary from 5 mg/L to 90 mg/L. Georgia and Massachusetts require a TSS limit of 5.0 mg/L prior to disinfection and Maryland requires that TSS not exceed 90 mg/L. South Carolina and North Carolina have different limits of TSS for daily and monthly averages. Limits on turbidity range from 2 to 10 NTU, but most of the states require an average turbidity limit of 2 NTU and a not-to-exceed limit of 5 NTU. Average fecal and total coliform limits range from 2.2 counts per 100 mL to 200 counts per 100 ml. Higher single sample fecal and total coliform limits are allowed in several state regulations.

Criteria that May Affect Irrigation as a Beneficial Use of Stormwater

There are few regulations restricting irrigation use of stormwater, although irrigation of food and fodder crops are included in some of the above described restricted and unrestricted urban water reuse regulations. Existing irrigation regulations focus on public health and restrict bacteria levels in water that may be in contact with the public. However, water quality criteria have been in place for many years recommending water quality levels to prevent damage to the plants themselves. These are mostly for heavy metal concentrations. Several cooperative extension services provide suggested water quality guidelines. Table 4-4 is from the Texas Cooperative Extension Service, for example, that lists specific irrigation water quality guidelines. In many cases, short-term use allows higher concentrations compared to long-term use.

This table also lists potable water drinking water standards (MCLs, or maximum contaminant limits) for reference. Potable uses require that the harvested water be treated to drinking water standards. In many areas, stormwater is a significant water source for the local drinking water supplies. Many states set drinking water levels based on U.S. EPA MCLs; however, testing of the harvested water is based only on the likeliest contaminants. These would be issued typically by the state’s department of health and would be reflected in testing requirements for well water. On this table, the irrigation criteria are less restrictive than the MCLs, with some exceptions, including: chromium, copper, fluoride, and zinc. No drinking water MCLs exist for several of the metals for which irrigation criteria are listed, including: cobalt, lithium, molybdenum, nickel, and vanadium. The copper and zinc are common stormwater contaminants that may hinder irrigation use. In addition, these two metals can be dramatically affected by the use of certain materials commonly used in the construction of storage and delivery facilities (galvanized metal roofs and storage tanks and copper pipes or other plumbing fittings).

Table 4-4. Texas Reuse Water Quality Criteria for Irrigation and EPA Potable Water MCLs (Irrigation: http://lubbock.tamu.edu/irrigate/documents/2074410-B1667.pdf; Drinking Water: http://water.epa.gov/drink/contaminants/upload/mcl-2.pdf)

Constituent	Irrigation Criteria (Texas)			EPA Potable Water MCLs	
	Short-Term Use (mg/L)	Long-Term Use (mg/L)	Remarks	MCLs (M)/ SMCLs (S)	Remarks
Aluminum (Al)	20	5.0	Can cause nonproductivity in acid soils, but soils at pH 5.5 to 8.0 will precipitate Al and eliminate toxicity	0.05 – 2.0 (S)	
Arsenic (As)	2.0	0.10	Toxicity to plants varies widely	0.01 (M)	Circulatory system damage; skin damage; cancer
Beryllium (Be)	0.5	0.10	Toxicity to plants varies widely	0.004 (M)	Internal lesions
Boron (B)	2.0	0.75	Essential to plant growth. Toxic to many sensitive plants at 1 mg/L.		
Cadmium (Cd)	0.05	0.01	Toxic to beans, beets, and turnips at 0.1 mg/L.	0.005 (M)	Kidney damage
Chromium (Cr)	1.0	0.1	Lack of knowledge on plant toxicity.	0.1 (M)	Allergic dermatitis
Cobalt (Co)	5.0	0.05	Toxic to tomatoes at 0.1 mg/L. Tends to be inactivated by neutral and alkaline solutions.		
Copper (Cu)	5.0	0.2	Toxic to many plants at 0.1 to 1.0 mg/L.	1.3 (M)/1.0 (S)	Short-term: Gastrointestinal distress; Long-term: Liver and kidney damage
Fluoride (F^-)	15.0	1.0	Inactivated by neutral to alkaline soils.	4.0 (M)/2.0 (S)	Bone disease
Iron (Fe)	20.0	5.0	Not toxic to plants in aerated soils, but can contribute soil acidification and loss of P and Mo	0.3 (S)	
Lead (Pb)	10.0	5.0	Can inhibit plant cell growth.	0.015 (M)	Children: Physical/mental

Constituent	Irrigation Criteria (Texas)			EPA Potable Water MCLs	
	Short-Term Use (mg/L)	Long-Term Use (mg/L)	Remarks	MCLs (M)/ SMCLs (S)	Remarks
					delays; Adults: Kidney damage
Lithium (Li)	2.5	2.5	Tolerated by most crops up to 5 mg/L; mobile in soils. Toxic at low doses to citrus.		
Manganese (Mn)	10.0	0.2	Toxic to number of crops at low concentrations.	0.05 (S)	

Table 4-4. Texas Reuse Water Quality Criteria for Irrigation and EPA Potable Water MCLs (continued) (Irrigation: http://lubbock.tamu.edu/irrigate/documents/2074410-B1667.pdf; Drinking Water: http://water.epa.gov/drink/contaminants/upload/mcl-2.pdf)

Constituent	Irrigation Criteria (Texas)			EPA Potable Water MCLs	
	Short-Term Use (mg/L)	Long-Term Use (mg/L)	Remarks	MCLs (M)/ SMCLs (S)	Remarks
Mercury				0.002 (M)	Kidney damage
Molybdenum (Mo)	0.05	0.01	Nontoxic at normal concentrations. Toxic to livestock if forage grown in soils with high levels of available Mo.		
Nitrate-N				10.0 (M)	Methemoglobinemia
Nitrite-N				1.0 (M)	
Nickel (Ni)	2.0	0.2	Toxic to number of plants at 0.5 mg/L. Reduced toxicity at neutral to alkaline pH.		
Selenium (Se)	0.02	0.02	Toxic to plants at low concentrations and to livestock if forage grown in soils with added Se.	0.05 (M)	Numbness; Circulatory problems

Constituent	Irrigation Criteria (Texas)			EPA Potable Water MCLs	
	Short-Term Use (mg/L)	Long-Term Use (mg/L)	Remarks	MCLs (M)/ SMCLs (S)	Remarks
Vanadium (V)	1.0	0.1	Toxic to many plants at low concentrations.		
Zinc (Zn)	10.0	2.0	Toxic to many plants at wide concentration variation. Reduced toxicity at increased pH (> 6) and in fine-textured or organic soils.	5.0 (S)	

Original Source of Irrigation Water Quality Standards Data: Rowe, D.R. and I.M. Abdel-Magid. 1995. *Handbook of Wastewater Reclamation and Reuse*. CRC Press, Inc. 550pp.

Treatment Methods to Enhance Stormwater Quality for Beneficial Uses

Off-the-Shelf Treatment Systems

Many small-scale, rapid treatment systems have been developed that could be used to treat stormwater runoff for beneficial uses, including the following (with some examples provided in the following discussion):

- Rainwater harvesting treatment systems
- Aquaculture water treatment systems
- Well water treatment for indoor potable use
- Swimming pool water treatment systems

Rainwater Harvesting Treatment Systems

Rainwater harvesting systems typically are designed to capture relatively-clean runoff from roofs. The website www.harvesth2o.com specializes in information relevant to the rainwater harvesting industry. The focus of many of the treatment systems is nonpotable reuse, such as landscape irrigation.

Rainwater harvesting systems usually consist of piping or gutters and cisterns or water tanks, plus possible filtering or other treatment systems. Storage tanks range in size from 130 liter (35 gallon) "rain barrels" to several thousand gallon underground storage tanks. Depending on the size and visibility of the cistern, tank materials can be plastic (typically opaque HDPE), wood, or galvanized metal. The interior of the tank should be constructed from materials that are relatively unreactive with water, even during long-term storage, and that do not allow light into the system to minimize algae growth (Virginia Rainwater Harvesting Manual, 2009 available at: http://dcr.virginia.gov/documents/stmrainharv.pdf. Screens often are used either at the gutter, in the piping system, or at the entry to the cistern to capture leaves and other large debris. For example, Rainwater Management Solutions (www.rainwatermanagement.com) sells mesh screen filters (having aperture sizes ranging from 280 to 1,000 micrometers) that are placed in the gutter system. Mesh sizes in this range are not likely to provide removal of pollutants other than leaves and other large debris. Meshes that are not cleaned regularly are likely to have a buildup of leaves and, as the leaves degrade, nutrients likely will leach from the leaves and end up in the cistern. The leaching of nutrients into the system from degrading leaves is why rainwater harvesting guidance recommends opaque tanks to prevent algal growth.

Many water harvesting system vendors also sell water purification systems that can be attached to the cistern outlet. These systems usually consist of a membrane filter and a UV disinfection unit. They are very similar, or in some cases identical, to point-of-use drinking water systems used in homes having private wells. The nominal pore size of the filters used in these units can range from < 1 to several hundred micrometers.

Systems such as the SkyHarvester (Watertronics, Inc., at: www.watertronics.com/?gclid=CPPt1KrT7KgCFYXd4Aod5XfuCg#/skyharvester) allow more treatability options to be added to roof runoff harvesting system. SkyHarvester includes filtration components that remove particles >75 µm for drip irrigation systems, plus the system can be used with reverse osmosis or ultrafiltration units. The UrbanGreen Rainwater Harvesting System, sold by Contech Construction Products, Inc. (www.contech-cpi.com/Products/Stormwater-Management/Rainwater-Harvesting.aspx?gclid=CJXpuZjV7KgCFcTd4AodyhZtEw) consists of filtration down to 2-5 µm and disinfection (chlorination or UV light) can be attached to the tank and operated in-line prior to use. Sediment filters can remove solids greater than the pore size opening and those pollutants that are associated with the solids. However, these filters do not remove dissolved pollutants effectively unless a chemically-active media is included. For example, the Contech Downspout Filters contain a treatment media to provide removal of many dissolved constituents, including zinc. The Rainwater Store (http://therainwaterstore.com/index.php/) provides products from several manufacturers, including a range of cartridge inserts for filter units. Some cartridges contain activated carbon to enhance pollutant removal. In general, rainwater harvesting system vendors do not report treatment effectiveness information. However, because these systems are similar to those used in the aquaculture and drinking water industry, their efficiencies and effluent quality can be estimated.

Swimming Pool Water Treatment Systems

Although not typically used to treat stormwater for beneficial uses, the technologies long used for treating swimming pool water (focusing on bacteria levels for safe water contact) could be used for maintaining acceptable water quality in water storage tanks and to meet the bacteria limits in the reuse criteria. Most of these units use a recirculating pump system having a sand filter and a disinfection unit. Systems are now available that use ozone, reverse osmosis, and even chitosan to maintain bacteriological quality, but historically, chlorine (usually added as Trichloro-S-Triazinetrione, Sodium Dichloro-S-Triazinetrione, or calcium hypochlorite) was used. With recirculation, it is possible to maintain good bacteriological conditions in storage tanks, even without maintaining a high chlorine residual (such as required by some of the water reuse standards).

Summary of Millburn Township and New Jersey Groundwater Disposal Regulations and Treatment Options

The above discussions summarize regulations and guidance that may affect the beneficial uses of stormwater, along with specific regulations from Millburn Township and New Jersey pertaining to the use of dry wells for the disposal of stormwater to the subsurface.

The Millburn Township stormwater regulations (in their Development Regulations) list dry wells as one option for minimizing increased flows associated with new (and increased) development. They do not include any specific criteria for their use, except for a statement pertaining to a 60 cm (2 ft) blanket of crushed stone surrounding the dry well. Specifically, they do not describe applicable soil characteristics, groundwater conditions, or suitable source waters. In contrast, while the New Jersey stormwater regulations require the infiltration of excess water above natural conditions associated with development or land modifications (either maintaining the pre-development groundwater recharge or preventing excess surface runoff), they also include additional guidance. The state dry well regulations describe the construction of the dry wells, the acceptable soil conditions (HSG A and B), groundwater conditions (at least 60 cm or 2 ft above seasonal water table), and source waters (roof runoff only).

Most of the reuse regulations available from different regulatory agencies were originally written to pertain to reuse of sanitary wastewaters and do not specifically address stormwater as a source water. There are a few regulations, however, that were specifically prepared to regulate the beneficial uses of stormwater. All of these focus on public health issues and contain restrictive levels of bacteria, typically with lower allowable limits where public access is not well controlled, and with higher allowable limits for water non-contact situations and where access can be well controlled. These bacteria levels will be difficult to meet without further treatment. In addition, irrigation criteria may affect stormwater use for certain plants, especially if galvanized metals or copper is in contact with either the collection, storage, or distribution areas of the rain water harvesting systems. Situations where groundwater recharge is direct with dry wells or injection wells, or other methods providing little treatment, may also result in adverse water quality.

Many people have questioned the application of the fecal indicator bacteria criteria to non-human contamination sources, especially considering the commonly observed very large indicator bacteria levels found in many stormwater flows. However, the EPA recently published (December 2011 *Federal Register*) their new proposed recreational water quality criteria pertaining to bacteria indicators. The following is a quote from that report describing why they feel that the bacteria indicator criteria should apply to all recreational waters, irrespective of contamination source:

“While human sources of fecal contamination are fairly consistent in the potential human health risks posted during recreational exposure, non-human sources of fecal contamination, and thus the potential human health risks, can vary from site-to-site depending on factors such as: the nature of the non-human source(s), the fecal load from the non-human source(s), and the fate and transport characteristics of the fecal contamination from deposition to the point of exposure. Nonhuman fecal sources can contaminate recreational bodies of water via direct fecal loading into the body of water, and indirect contamination can occur via runoff from the land. The fate and transport characteristics of the zoonotic pathogens and FIB present under these conditions can be different (e.g., differences in attachment to particulates or differences in susceptibility to environmental parameters affecting survival). For more information on pathogenic risks from nonhuman sources, see *Review of Zoonotic Pathogens in Ambient Waters* (U.S. EPA, 2009a). EPA did not develop nationally applicable criteria values that adjust for the source of the fecal contamination, for non-human sources. Rather, EPA recommends that States use these nationally applicable criteria in all waters designated for primary contact recreation.”

Some of the stormwater constituents in roof runoff would likely have concentrations greater than the associated numeric criteria. The most potentially problematic constituents (where the exceedences are the greatest), include bacteria, followed by the solids and turbidity values. The metals having the potentially greatest exceedences include cadmium and zinc. Generally, roof runoff has better water quality than other stormwater source areas, with most stormwater source areas (such as parking lots and street runoff, and even landscaped areas) likely exceeding the numeric criteria for: BOD_5, COD, TSS, and fecal coliforms. Therefore, none of the stormwater or source waters would likely be able to meet the numeric criteria for stormwater beneficial uses, with the bacteria being the most problematic, and the solids and turbidity values also being an issue. Roof runoff is the preferred source water for beneficial stormwater uses, but treatment, especially for bacteria, will likely be necessary.

Different materials are used in the collection, drainage, and storage components (such as gutters, pipes and storage tanks) of stormwater beneficial use systems. Some materials can degrade runoff water even with very short contact times, and would be a problem even if used for the collection surface. Other materials, however, require extended exposure periods to degrade the water, such as would be evident in storage tanks. The most significant potential problems are associated with galvanized metal roofs or gutter and tanks, plus copper pipe or other plumbing fixtures used in the systems. These materials can elevate the zinc and copper concentrations to problematic concentrations during rain events, while extended contact, such as storage tanks, can cause very high concentrations.

Treatment of stormwater before most beneficial uses may therefore be needed. For simple irrigation use, bacteria reductions may be necessary, and the prevention of excessive metal concentrations through careful selection of materials. Cistern and water tank storage can reduce most bacteria levels to close to the regulation's numeric values, although some additional treatment may be needed. Roof runoff typically has excessive bacteria levels, especially during the non-winter months and if trees are over the roofs (providing habitat for birds and squirrels). Depending on the water quality of the source stormwater and the intended beneficial use, different water quality treatment options can be examined. There are a number of commercial units available that would be suitable that can reduce the solids, bacteria, and heavy metals in the water before use. Simple storage in cisterns and water tanks may approach the guideline values for roof and yard runoff (most which were developed for treated sanitary wastewater), and measures to minimize scour resuspension of deposited sediments, would likely be sufficient to protect public health. More contaminated source waters may require more sophisticated treatment options.

As noted elsewhere in this report, the Millburn dry wells worked well in infiltrating runoff, except in areas having high water tables, or if poor subsurface soils exist. However, as noted elsewhere in this report, they provided no significant improvement in water quality for constituents of interest. Therefore, the local Millburn Township ordinance should be modified to allow dry well use only in areas already having good water quality (such as would be expected for most roofs), or require suitable pretreatment. In addition, the local ordinance should also prohibit dry well use in areas having seasonal or permanent high water tables, as those conditions result in long-term standing water in the dry wells. If located in areas having poorly draining subsurface soils, their designs need to be modified to account for the more slowly draining conditions. Overall, it is recommended the dry well use be restricted to roof runoff water sources, and alternatives that infiltrate water through surface soils (such as rain gardens) be used to treat driveway and parking lot runoff. Irrigation of landscaped areas using roof runoff (and pretreated paved area runoff) is also a suitable alternative that also provides economic benefits to the land owner.

Chapter 5 Beneficial Uses of Stormwater for Infiltration and Recharge in Millburn, New Jersey

Groundwater Recharge

Infiltrating groundwater through surface soils or infiltration stormwater controls (rain gardens, biofilters, percolation ponds, etc.) or more direct recharging of groundwater using stormwater (dry wells, injection wells, porous pavements, gravel trenches, etc.) are the two mechanisms used to discharge stormwater to the groundwater as a receiving water. The first mechanism is usually focused on removing stormwater from the immediate surface water regime as a stormwater management tool, while the second method is more to recharge local groundwater supplies for future use.

One of the earliest comprehensive reports investigating groundwater recharge was the committee report prepared for the National Research Council (1994; *Ground Water Recharge using Waters of Impaired Quality*). This report contained many international case studies, mostly examining treated sanitary wastewaters, but also some on stormwater. The main focus was groundwater recharge for later beneficial uses, including potable use. The case studies that addressed potable use were mainly associated with soil-aquifer treatment and had substantial subsurface residence times. Short residence times and little aquifer movement of the recharged water would be more similar to a storage tank, with reduced improvements in water quality.

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 may travel through the soils and vadose zone to the groundwater. Source stormwaters from residential areas are not likely to be contaminated with compounds having significant groundwater contaminating potential (with the exception of high salinity snowmelt waters). In contrast, commercial and industrial areas are likely to have greater concentrations of contaminants of concern that may affect the groundwater quality adversely. Therefore, pretreatment of the stormwater before infiltration may 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.* (2010) summarized prior research on potential groundwater contamination. Table 5-1 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 was clayey and/or had a high organic content, then most of the organic compounds would be less mobile 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 5-1. Groundwater Contamination Potential for Stormwater Pollutants Post-Treatment.

Compound Class	Compounds	Surface Infiltration with Minimal Pretreatment (such as rain gardens and swales)*	Surface Infiltration with Sedimentation or Filtration Pretreatment*	Subsurface Injection with Minimal Pretreatment (such as in dry wells and porous pavements)
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

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, prior to 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 may be appropriate in an area with a shallow groundwater table if maintenance is sufficiently frequent to replace contaminated vadose zone soils.

Modified from Pitt, *et al.* 1994

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 pre-treatment may be needed in these other areas.

Infiltration Tests at Millburn Dry Well Installations

Infiltration tests were conducted during two project phases: the first phase filled the dry wells with domestic water from Township fire hydrants and the decreasing water levels were recorded; the second phase used continuous monitoring in a fewer number of dry wells during many rains.

Much information was collected as part of this research project in Millburn to measure actual performance of the dry wells. Both short and long-term infiltration measurements were conducted at many locations. This data were analyzed and are summarized in this report section, with more detailed data included in Appendix B.

The infiltration measurements were conducted using continuously recording (10 minute observations) LeveLoggers by Solintest that were installed in the dry wells. Short-term tests were conducted in many dry wells throughout the Township to measure the influence of many of the conditions present in the community. These tests were conducted using water from fire hydrants and included filling the dry wells completely. The LeveLoggers were then used to measure the drop in water level over time. The long-term tests were conducted in fewer dry wells (based on the number of LeveLoggers available). These were installed for several months to over a year and continuously recorded the water levels in the dry wells every 10 min. Close-by rain gages were also used to record local rains associated with these events. These rain and water level data were downloaded by PARS Environmental personnel and uploaded to their FTP site where University of Alabama researchers downloaded the data for analysis.

The first step in the data analyses was to plot the data as time series. Figure 5-1 is an example time series plot of the water levels recorded over a two month period at 11 Woodfield Dr. showing 6 separate events (the first peak only shows the dropping water levels from the Oct 13, 2009 event). The infiltration characteristics of the dry well installations were calculated from the recession cures of these individual rain events. The infiltration rates for each 10 minute step were calculated based on the drop in water level per increment, resulting in plots of infiltration rates vs. time since the peak water level. These are classical infiltration rate plots and statistical analyses were used to calculate infiltration rate equation parameters for two common infiltration equations (Horton and Green-Ampt).

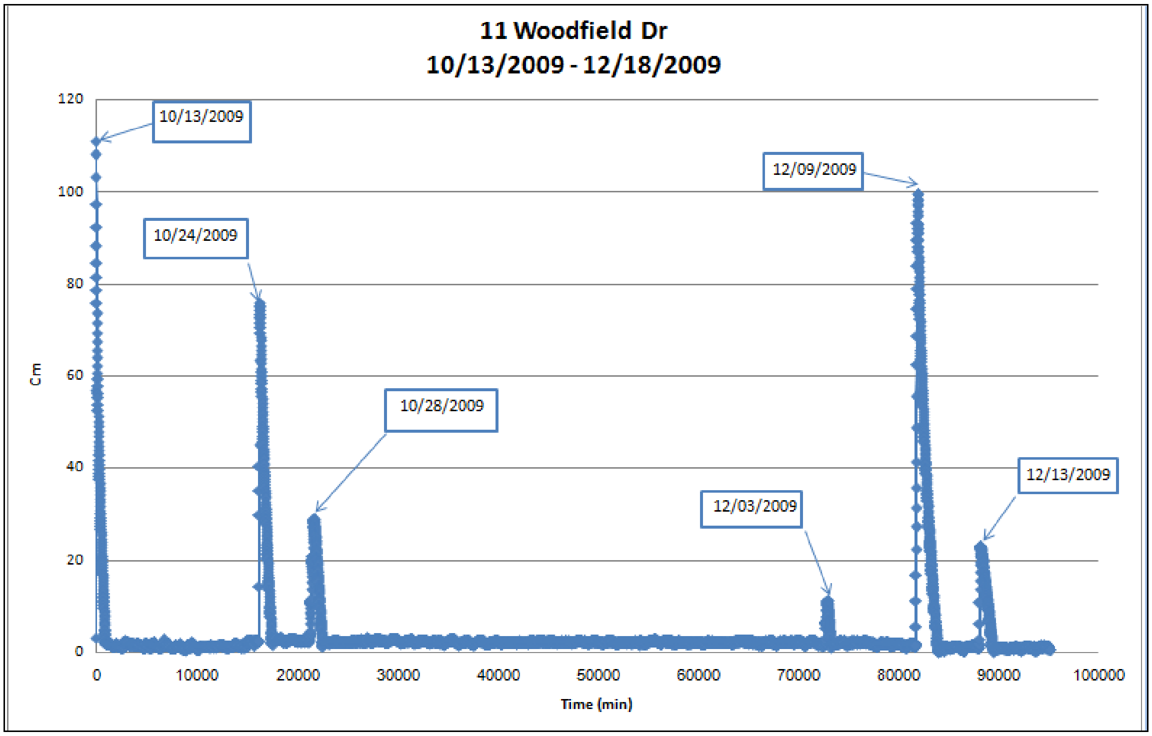


Figure 5-1. Time series example of dry well water levels for a two month period at 11 Woodfield Dr.

The following discussions present and compare these results with the varying site conditions.

Rainfall Measurements

Four rain gages were installed in the study area for this project (HOBO® data logging rain gage data logger). The rain gauges are battery-powered rainfall data collection and recording systems which included a HOBO® Pendant Event data logger integrated into a tipping-bucket rain gauge. Below is a list of the locations of the four rain gages. Figure 5-2 shows photos of three of the rain gages (with some undergoing calibration).

- **R1:** Private house on top of chimney slab at 1 Delwick Lane - Calibrated and launched at 14:00 on 5/22/09.
- **R2:** Roof of Township's maintenance garage on Essex Rd - Calibrated and launched at 12:00 on 5/13/09.
- **R3:** Municipal Par 3 Golf Course on White Oak Ridge Rd - Calibrated and launched at 16:00 on 5/13/09.

- **R4:** Old tennis court at Greenwood Gardens on Old Short Hills Rd – Calibrated and launched at 16:00 on 5/6/09.

	
R2: Roof of Township's maintenance garage (gage being calibrated)	**R2:** Roof of Township's maintenance garage (gage being calibrated)
	
R3: Municipal Par 3 Golf Course (gage being calibrated)	**R4:** Old tennis court at Greenwood Gardens (with top funnel removed)

R2: Millburn Township Garage rooftop rain gage location.

R1: Residential rooftop rain gage location (located near chimney).

Figure 5-2. Photos of rain gages (R1, R2, and R3 are shown during site calibration).

Figure 5-4 shows the locations of the rain gages and the monitoring locations, while Table 5-2 lists the monitoring sites and corresponding closest rain gage locations.

Table 5-2. List of Rain Gages Closest to Monitoring Site Locations

Rain Gage	Dry well Locations	ID on Map (Figure 5-4)
R1: 1 Delwick Ln	11 Woodfield Dr	1
R2: 345 Essex St	15 Marion Ave	2
	258 Main St	3
	2 Undercliff Rd	4
	383 Wyoming Ave	5
R3: 335 White Oak Ridge Rd	260 Hartshorn Dr	6
	79 Minnisink Rd	7
	87/89 Tennyson Dr	8 and 9
	36 Farley Pl	16
R4: 274 Old Short Hills Rd	1 Sinclair Terrace	10
	142 Fairfield Dr	11
	8 Beechcroft Rd	12
	7 Fox Hill Ln	13
	9 Fox Hill Ln	14
	11 Fox Hill Ln	15

The rain gages provided information about the start time, end time, duration, depth and average intensity of each rain event. Each rain event is defined as a separate rain event that has at least 6 hr of no rain before and after the recorded rainfall. The rain information corresponding to the infiltration data is summarized for each infiltration event monitored, as shown in Table 5-3 and Figure 5-3. The rainfall graphs and information are presented in Appendix B, along with the infiltration information.

Table 5-3. Example Summary of Rainfall Information (2/25/2011 – R3) (1 in. = 25.4 mm)

Start time		End time		Duration (hr)	Depth (in.)	Average intensity (in./hr)
2/25/2011	0:25	2/25/2011	18:44	18:19	1.36	0.06

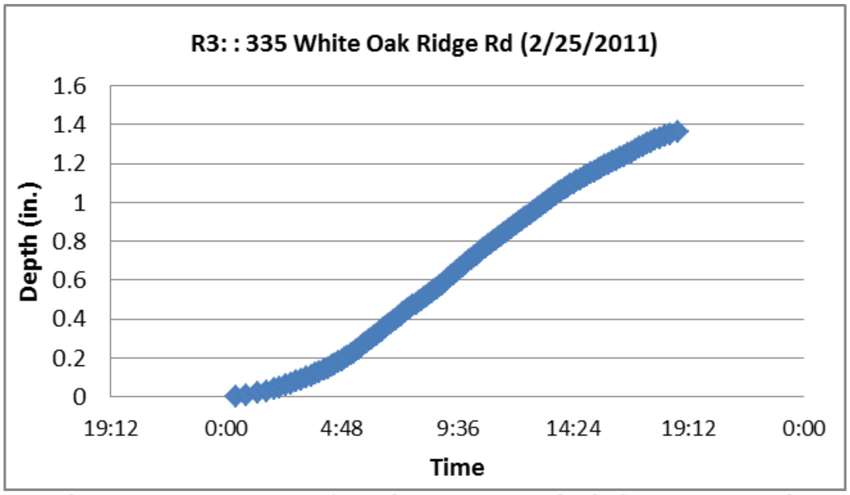


Figure 5-3. Example of a rain event graph. (1 in. = 25.4 mm)

Figure 5-4. Location of dry wells (blue icons), rain gages (yellow icons), and water quality samplers (red icons for dry wells and green icon for cistern).

Infiltration Measurements

The water levels in the dry wells were recorded using Solinist Levelogger Gold and Barologger monitors. The Levelogger Gold is an absolute data logger which measures water levels and temperature. The Levelogger Gold devices use a sensitive piezoresistive silicon pressure transducer packaged in a stainless steel housing. The Levelogger converts the total pressure reading to its corresponding water level equivalent, after correction for changing atmospheric pressure from the Barologgers. The water levels were recorded every 10 min.

Initial infiltration studies were conducted by quickly filling selected dry wells with water from Township fire hydrants and recording the subsequent fall of the water levels. These infiltration studies were performed after at least a 72 hr dry period. The photographs in Figure 5-5 show the process of filling the dry well with the Township fire hydrant water at one of the test sites. Table 5-4 describes the Township water infiltration tests for the seven selected sites.

Figure 5-5. Infiltration studies for a dry well located at 383 Wyoming: rapidly filling the dry well with water from the fire hydrant and recording the fall of water level.

Table 5-4. Test Characteristics for Township Fire Hydrant Infiltration Tests at Seven Dry Wells

Location	Fill Date	Start Fill Time	Stop Fill Time	Total Fill Time (min)	Total Fill Volume from Hydrant (gal)	Fill Rate (gal/min)
1 Sinclair Terrace	7/15/2009	10:40	11:30	50	3,300	66
2 Undercliff Road	10/2/2009	09:07	09:26	19	2,500	132
383 Wyoming	10/2/2009	10:14	10:43	29	2,900	100
8 South Beechcroft	10/2/2009	12:07	12:15	8	900	113
9 Fox Hill Lane	10/2/2009	12:44	13:15	31	2,600	84
11Fox Hill Lane	10/2/2009	13:16	14:00	44	3,400	77
11 Woodfield Road	10/13/2009	10:07	10:30	23	3,600	157

Infiltration Equations

Site soil evaluations included infiltration measurements, along with soil density, texture, and moisture determinations. The water infiltration data can be fitted to soil water infiltration models, such as the Green–Ampt (1911), the Kostiakov (1932), the Horton (1940) and the Philip's (1957) equations. Although various infiltration equations have different mathematical structures and calibration parameters, their estimates are all premised on observed water infiltration data (in./hr as a function of time). The most common Green-Ampt and Horton equations were examined during this project and are briefly described in the following discussions.

Horton Infiltration Equation

One of the most commonly used infiltration equations was developed by Horton (1940). The equation is as follows:

$$f = f_c + (f_o - f_c)e^{-kt} \qquad (1)$$

Where f is the infiltration rate at time t (in./hr), f_o is the initial infiltration rate (in./hr), f_c is the final (constant) infiltration rate (in./hr), and k is first-order rate constant (hr^{-1} or min^{-1}). This equation assumes that the rainfall intensity is greater than the infiltration capacity at all times and that the infiltration rate decreases with time (Bedient and Huber 1992). This is a reasonable assumption for ponded conditions, such as in the dry wells. The capacity of the soil to hold additional water decreases as the time of the storm increases because the pores in the soil become saturated with water. The Horton equation's major drawback is that it does not consider the soil water storage availability after varying amounts of infiltration have occurred, but only considers infiltration as a function of time (Akan 1993). However, integrated forms of the equation can be used that do consider the amount of water added to the soil. It is recommended that f_c, f_o, and k all be obtained through field data, but they are rarely measured locally. Table 5-5 shows commonly used Horton infiltration parameter values, as summarized by Denver's Urban

Drainage and Flood Control District (2001). This summary is for the four NRCS hydrologic soil groups corresponding to HSG sandy (A) to clayey (D) conditions. The coefficient values for C and D soils are the same, with B soils having only slightly increased infiltration rates.

Table 5-5. Horton Infiltration Coefficient Values Typically used in Urban Drainage Projects (Urban Drainage and Flood Control District, UDFCD 2001)

HSG	Initial infiltration rate, f_o (in./hr)	Final infiltration rate, f_c (in./hr)	First-order rate constant (1/hr)	First-order rate constant (1/min)
A	5.0	1.0	2.52	0.04
B	4.5	0.6	6.48	0.11
C	3.0	0.5	6.48	0.11
D	3.0	0.5	6.48	0.11

1 in./hr = 25.4 mm/hr

Akan (1993) presented a somewhat more detailed table for the initial infiltration rates (the other coefficients did not change greatly for the different soil conditions). Akan shows the effects of antecedent moisture and vegetation on these initial infiltration rates.

Table 5-6. Horton parameters (Akan, 1993)

Soil Type	f_o (in./hr)
Sandy soils with little to no vegetation	5
Dry loam soils with little to no vegetation	3
Dry clay soils with little to no vegetation	1
Dry sandy soils with dense vegetation	10
Dry loam soils with dense vegetation	6
Dry clay soils with dense vegetation	2
Moist sandy soils with little to no vegetation	1.7
Moist loam soils with little to no vegetation	1
Moist clay soils with little to no vegetation	0.3
Moist sandy soils with dense vegetation	3.3
Moist loam soils with dense vegetation	2
Moist clay soils with dense vegetation	0.7

1 in./hr = 25.4 mm/hr

Green-Ampt Infiltration Equation

Another common equation for infiltration calculations is by Green-Ampt. The Green-Ampt equation calculates cumulative infiltration as the water flows into a vertical soil profile (Green and Ampt, 1911).

$$f_t = K(\frac{\psi\Delta\theta}{F_t} + 1) \qquad (2)$$

Where: f_t is infiltration rate, cm/hr; ψ is the initial matric potential of the soil (in.); $\Delta\theta$ is the difference of soil water content after infiltration with initial water content (in.^{3}/ in.^{3}); K is hydraulic conductivity (in./hr); and F_t is the cumulative infiltration at time t (in.). This equation requires a linear relationship between f_t and (1/F_t). Table 5-7 shows some typical Green-Ampt equation parameter values suggested by Rawls, *et al.* (1983).

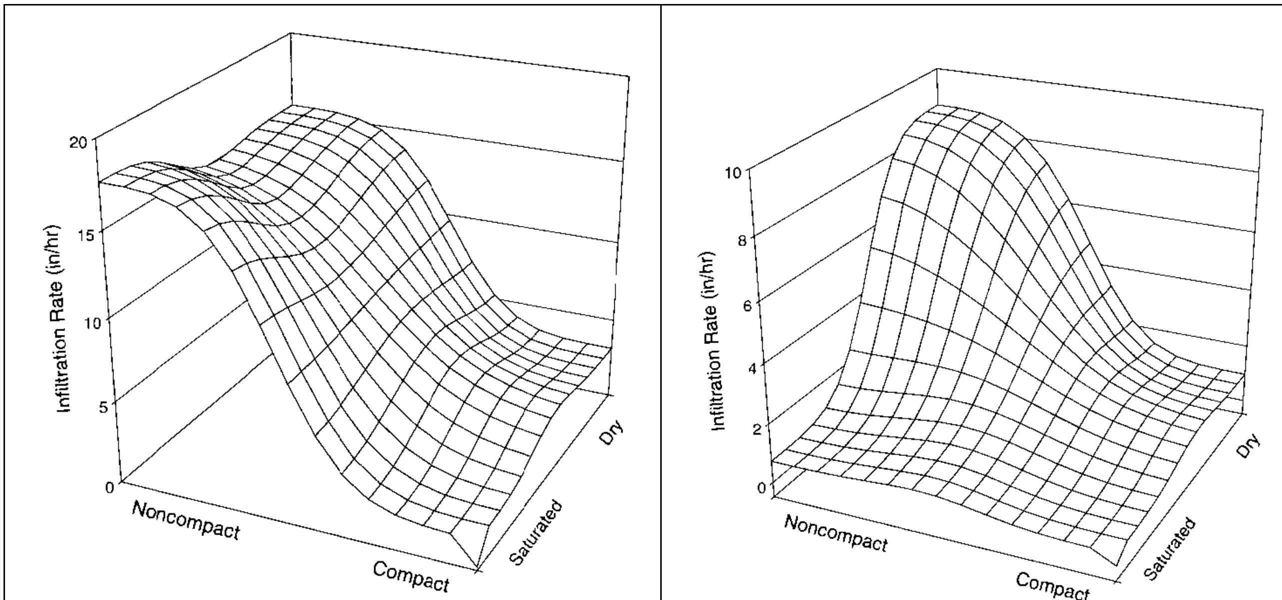
Table 5-7. Green-Ampt parameters (Rawls, *et al.* 1983)

Soil type	Porosity	Effective porosity	Suction head (mm)	Hydraulic conductivity (mm/h)
sand	0.437	0.417	49.5	117.8
loamy sand	0.437	0.401	61.3	29.9
sandy loam	0.453	0.412	110.1	10.9
loam	0.463	0.434	88.9	3.4
silt loam	0.501	0.486	166.8	6.5
sandy clay loam	0.398	0.330	218.5	1.5
clay loam	0.464	0.309	208.8	1.0
silty clay loam	0.471	0.432	273.0	1.0
sandy clay	0.430	0.321	239.0	0.6
silty clay	0.479	0.423	292.2	0.5
clay	0.475	0.385	316.3	0.3

Infiltration as a Function of Soil Texture and Compaction

Hydrologic models must contain a process to address the infiltration of rain water into the soil. The infiltration process in most models is usually dependent on the porosity and moisture content of the soil: in an unsaturated soil, infiltration usually is initially rapid but then declines to a constant value as the soil becomes saturated. Soil infiltration is an issue in urban watershed management due to concerns of groundwater contamination and because poor infiltration conditions after land development, which is one of the causes of increased surface runoff (in addition to increased amounts of impervious surfaces) (Pitt, *et al.* 1994 and 1995). It has been well documented that during urbanization, soils are greatly modified, especially related to soil density. Increased soil compaction results in soils that do not behave in a manner predicted by traditional infiltration models. It is crucial, therefore, that stormwater engineers better understand infiltration in disturbed urban soils. Laboratory and field tests can be used to determine expected infiltration behavior of disturbed urban soils for a specific area.

Since the early 1990s, Pitt, et al. (1999) has conducted a series of laboratory and field tests on soils covering a wide range of soil textures, densities, and stiffness. As shown in Figure 5-6, these field tests highlighted the importance of compaction on the infiltration rate of soils. For sandy soils, minimal effects are seen associated with antecedent moisture conditions compared to soil compaction. For the clayey soils, both the compaction level and antecedent moisture conditions are likely important in determining the infiltration rate. Table 5-8 summarizes the Horton equation coefficients for these urban soils, showing the dramatic effect soil density has on the infiltration characteristics.



Three dimensional plots of infiltration rates for sandy soil conditions.

Three dimensional plots of infiltration rates for clayey soil conditions.

Figure 5-6. Effects of soil moisture and soil compaction on infiltration rates (Pitt, *et al.* 1999).

Table 5-8. Horton Coefficients (Pitt, *et al.* 1999)

Infiltration Parameter	Soil Group	90%	75%	50%	25%	10%
f_o (in./hr)	Clay – Dry Noncompact	42	24	11	7	5
	Clay-Other	7	3.75	2	1	0
	Sand-Compact	42	12	5	1.5	0
	Sand-Noncompact	52	46	34	24	0.25
f_c (in./hr)	Clay – Dry Noncompact	20	12	3	0.75	0.25
	Clay-Other	0.75	0.5	0.25	0	0
	Sand-Compact	5	1.25	0.5	0.25	0
	Sand-Noncompact	24	19	15	9	0
k (1/min)	Clay – Dry Noncompact	0.3	0.22	0.16	0.07	0.05
	Clay-Other	0.18	0.1	0.06	0.03	0
	Sand-Compact	0.28	0.2	0.1	0.05	0.016
	Sand-Noncompact	0.32	0.2	0.08	0.03	0

1 in./hr = 25.4 mm/hr

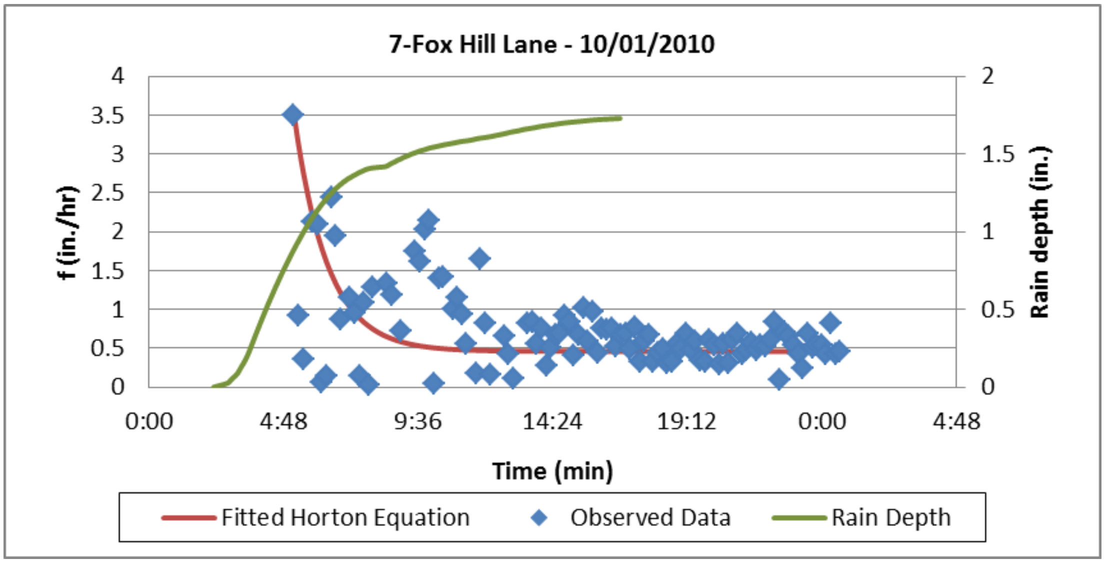
Fitted Horton Equation Parameters for Millburn Dry Well Infiltration Measurements

Fitting Observed data to Horton's Equation

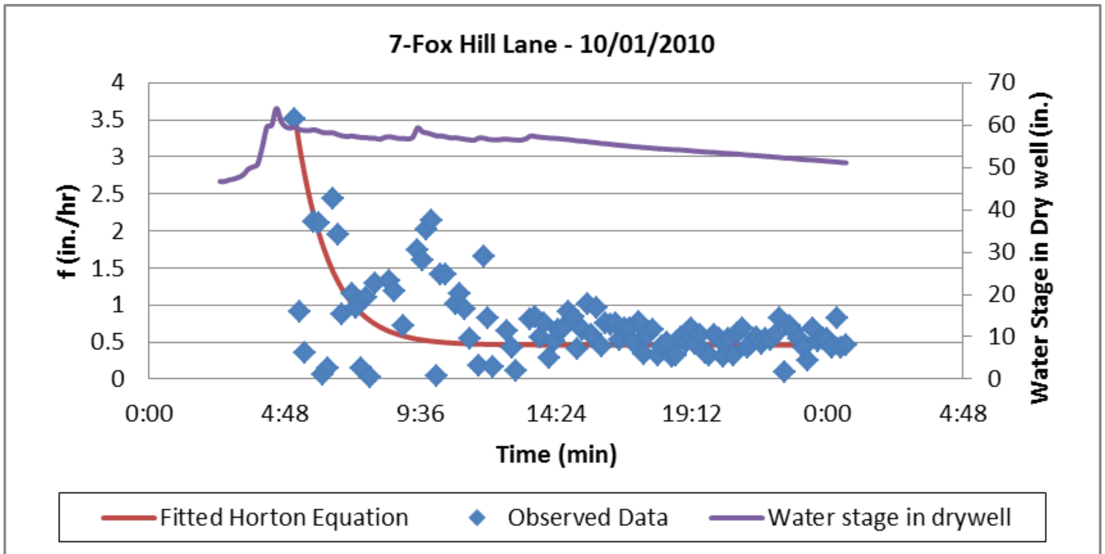
The initial infiltration data analysis was to prepare plots of the observed infiltration data in order to evaluate major trends and groupings of the data. Observed data included water stage in dry wells for every 10 min. The differential values of water stages in a dry well for each event were divided by time to calculate the infiltration rates as a function of time. Data from each site for each event/infiltration test was fitted to the Horton infiltration equation and the equation parameters were derived for f_o, the initial infiltration capacity, f_c, the constant infiltration capacity as t approaches infinity, and k, a soil parameter that controls the rate of decrease of infiltration rate. For some of the sites, the Horton equation was not able to be fitted to the observed data, as little change occurred with time. This typically occurred for narrow ranges of the dry well water depth and when standing water due to shallow water tables. For these conditions, the observed rates most likely corresponded to the f_c values, the saturated infiltration rate (f_o and k were not calculated).

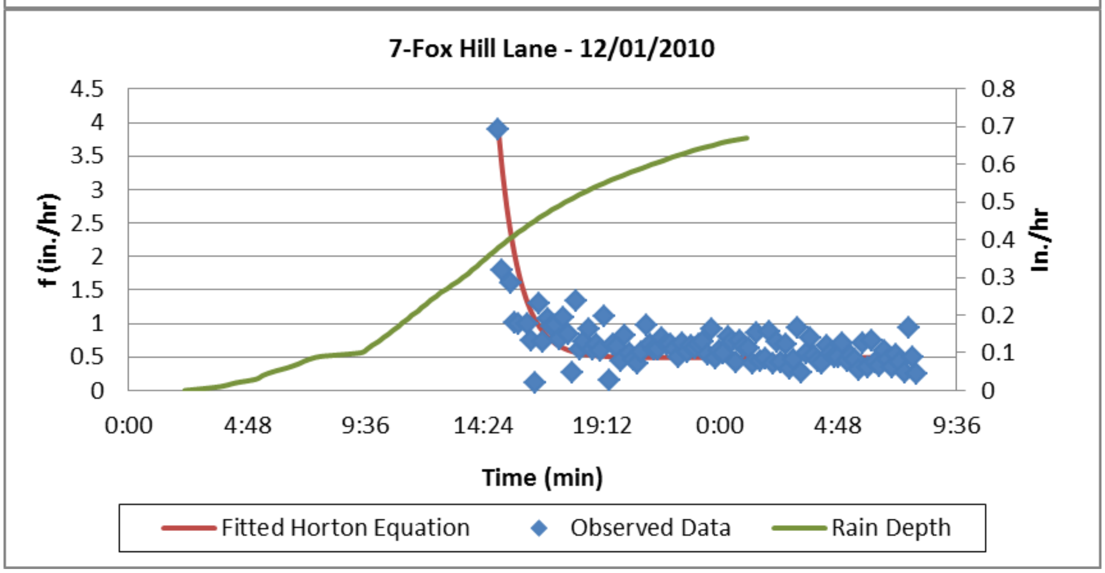
Figure 5-7 shows the observed infiltration rates and the fitted Horton equation parameter values for the dry well located at 7 Fox Hill Ln, Millburn, NJ, as an example. Graphs are for three different actual rain events representing observed data, fitted Horton equations, rain depths, and the water stage in the dry well. The remaining observed data along with fitted Horton graphs for each dry well and each event are presented in Appendix B. Some initial rates were very large, but the rates decreased quickly with time.

Basic statistical analyses, including average, minimum, maximum, standard deviation, and COV are included for all the data, as well as ANOVA test and residual plots for some of the fitted Horton equations in comparison to Green-Ampt equation.



7-Fox Hill Lane - 10/01/2010
f (in./hr)
Rain depth (in.)
Time (min)
4
3.5
3
2.5
2
1.5
1
0.5
0
0:00
4:48
9:36
14:24
19:12
0:00
4:48
2
1.5
1
0.5
0
Fitted Horton Equation
Observed Data
Rain Depth


7-Fox Hill Lane - 10/01/2010
f (in./hr)
Water Stage in Dry well (in.)
Time (min)
Fitted Horton Equation
Observed Data
Water stage in drywell


7-Fox Hill Lane - 12/01/2010
f (in./hr)
In./hr
Time (min)
Fitted Horton Equation
Observed Data
Rain Depth

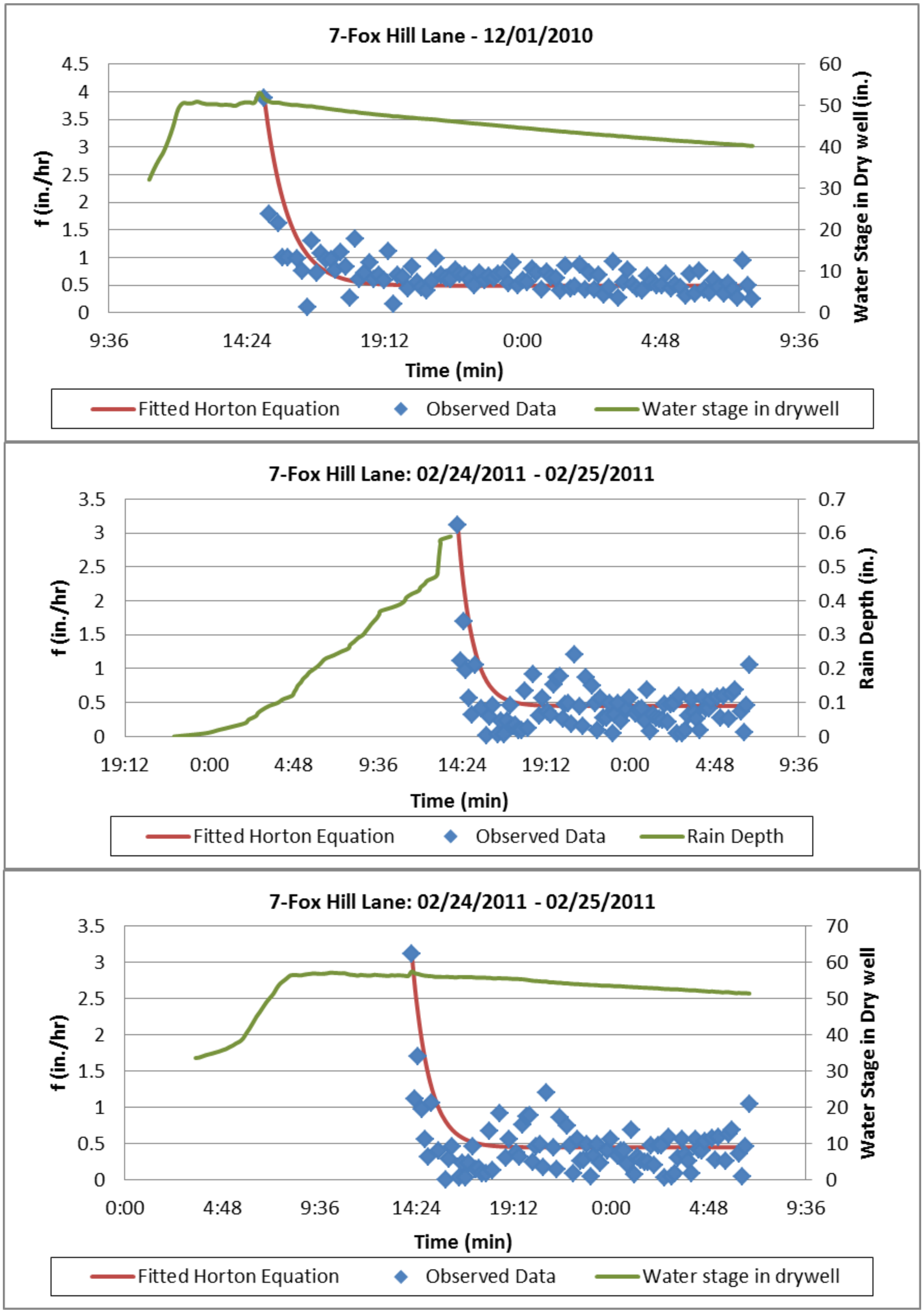


Figure 5-7 Example of observed data, fitted Horton equation, rain depth, and water stage in a dry well for three different rain events in a selected dry well. (1 in./hr = 25.4 mm/hr)

Tables 6a through 6n in Appendix B are a summary of the best-fit Horton equation parameter values based on infiltration tests for some sites at Millburn, NJ, for different rains. Three types of tables are included for the tests:

- **Infiltration study test:** A table summarizing Horton parameters, infiltration study test characteristics and water depth in the dry wells.

- **Infiltration for rain events:** Fitted observed data to Horton's equation resulting for f_o, f_c, and k values.

- **Infiltration for rain events when fitting the observed data to Horton's equation results in $f_o = f_c$ (and k is therefore not applicable):** A table summarizing statistical analysis for $f_o = f_c$, rain characteristics of corresponding rain event, and water depth in the dry wells.

Statistical Groupings of Site Data for Horton Coefficients

Multiple iterations of grouped box and whisker plots and ANOVA tests were used to identify data groupings. The data were not normally distributed so ANOVA based on ranks and Mann-Whitney Rank Sum nonparametric tests were used to calculate the significance that the data did not originate from the same populations.

There were two distinct sets for the f_c data: the 258 Main St location vs. all of the other sites combined. Figure 5-8 shows these two data sets.

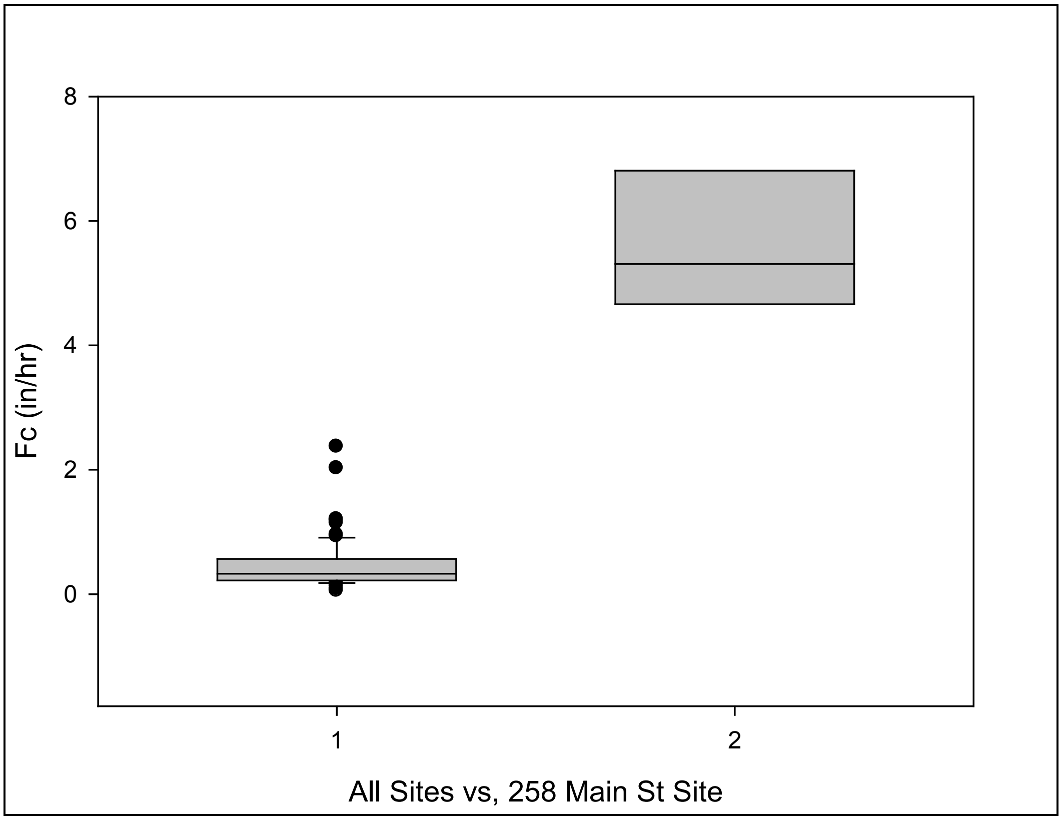


Figure 5-8. Box and whisker plot of f_c data showing two sets of data. (1 in./hr = 25.4 mm/hr)

The results of the final Mann-Whitney Rank Sum test for f_c are shown below:

Normality Test (Shapiro-Wilk) Failed ($P < 0.050$)

Group	N	Missing	Median	25%	75%
Combined	81	0	0.33	0.22	0.568
258 Main	3	0	5.308	4.662	6.808

Mann-Whitney U Statistic= 0.000

T = 249.000; n (small) = 3; n (big) = 81; P = 0.004

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference, with P = 0.004. Tables 5-9 and 5-10 summarize the values and test conditions for these two sets of data.

Table 5-9. f_c Summary Values and Conditions for 258 Main St.

	f_c (in./hr)	Rain Depth (in.)	Max. depth of water in dry well (in.)	Min. depth of water in dry well (in.)
number	3	3	3	3
Minimum	4.66	0.69	22.32	0.11
Maximum	6.81	1.34	54.77	0.67
Average	5.59	1.08	43.57	0.44
Median	5.31	1.22	53.62	0.53
Std Dev	1.10	0.35	18.41	0.29
COV	0.20	0.32	0.42	0.67

1 in. = 25.4 mm

Table 5-10. f_c Summary Values and Conditions for All of the Other Sites

	f_c (in./hr)	Rain Depth (in.)	Max. depth of water in dry well (in.)	Min. depth of water in dry well (in.)
number	81	63	81	81
Minimum	0.05	0.22	6.51	0.00
Maximum	2.37	2.90	93.85	82.98
Average	0.45	1.20	50.45	20.88
Median	0.33	1.15	53.76	10.07
Std Dev	0.38	0.76	22.93	24.15
COV	0.85	0.63	0.45	1.16

1 in. = 25.4 mm

Similar tests were conducted to identify significant groups for the f_o data. Figure 5-9 is the final box and whisker plot, showing the two data groups: 258 Main St, plus 8 So. Beechcroft vs. all the data combined.

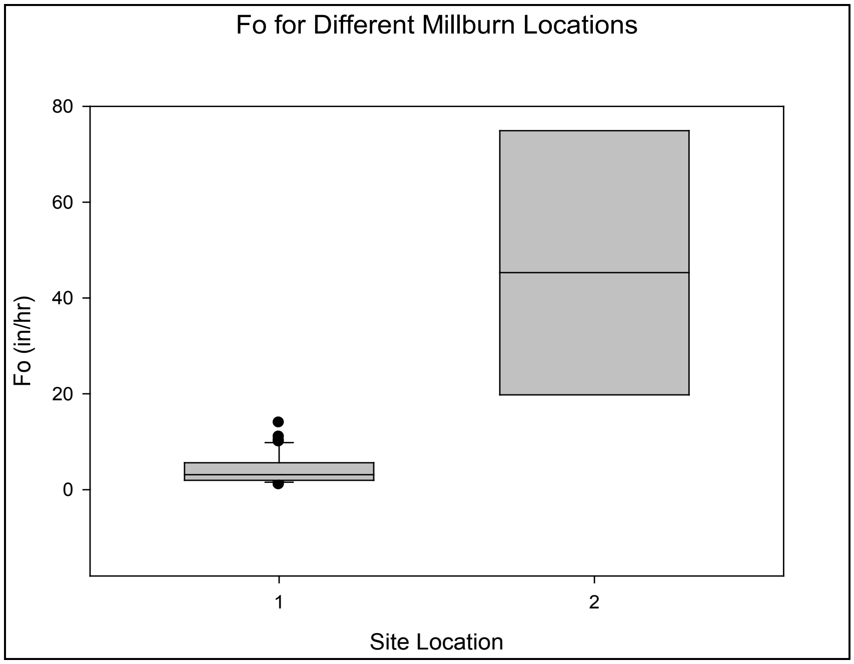


Figure 5-9. Box and whisker plot of f_o data showing two sets of data. (1 in./hr = 25.4 mm/hr)

The results of the final Mann-Whitney Rank Sum test for f_o are shown below:

Normality Test (Shapiro-Wilk) Failed (P < 0.050)

Group	N	Missing	Median	25%	75%
All the rest combined	43	0	3.116	1.941	5.631
258 Main & 8 So. Beechcroft	7	0	45.29	19.78	74.916

Mann-Whitney U Statistic= 0.000

T = 329.000; n (small) = 7; n (big) = 43; 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.

Tables 5-11 and 5-12 summarize the values and test conditions for these two sets of data.

Table 5-11. f_o Summary Values and Conditions for 258 Main St. and 8 So Beechdroft Rd.

	f_o (in./hr)	Rain Depth (in.)	Max. depth of water in dry well (in.)	Min. depth of water in dry well (in.)
number	7	6	7	7
Minimum	16.12	0.52	16.76	0.10
Maximum	75.14	1.71	54.77	1.94
Average	44.55	1.14	38.29	0.54
Median	45.29	1.28	41.29	0.32
Std Dev	23.74	0.45	14.98	0.65
COV	0.53	0.39	0.39	1.21

1 in. = 25.4 mm

Table 5-12. f_o Summary Values and Conditions for All of the Other Sites

	f_o (in./hr)	Rain Depth (in.)	Max. depth of water in dry well (in.)	Min. depth of water in dry well (in.)
number	43	60	77	77
Minimum	1.01	0.22	6.51	0.00
Maximum	13.95	2.90	93.85	82.98
Average	4.34	1.20	51.28	21.93
Median	3.12	1.07	54.45	12.06
Std Dev	3.20	0.77	23.07	24.32
COV	0.74	0.64	0.45	1.11

1 in. = 25.4 mm

Similar tests were conducted to identify significant groups for the k data. Figure 5-10 is the final box and whisker plot, showing the two data groups: 258 Main St vs. all the other data combined.

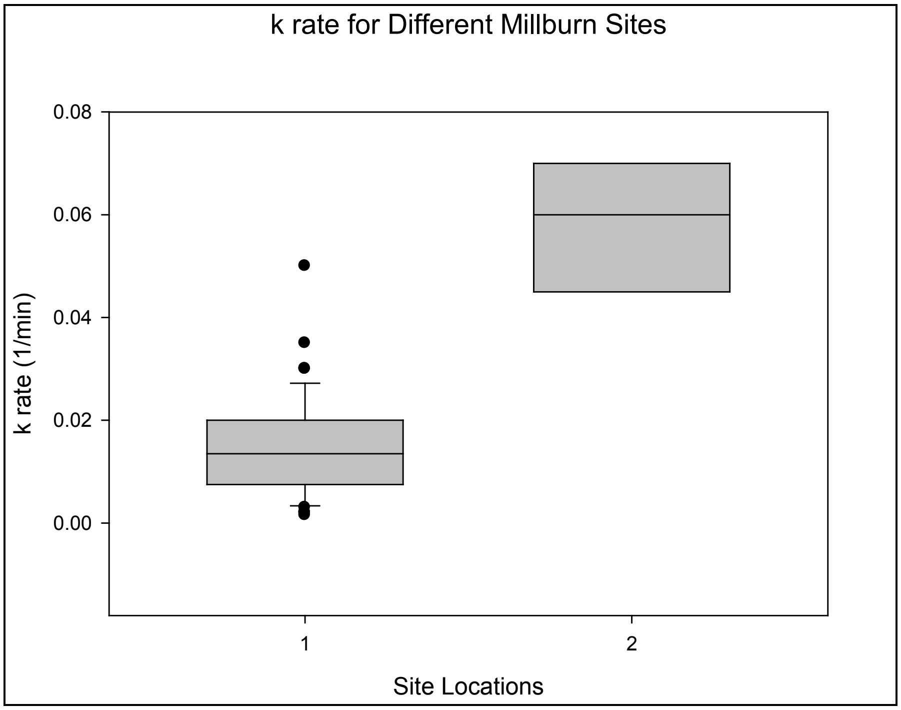


Figure 5-10. Box and whisker plot of k data showing two sets of data.

The results of the final Mann-Whitney Rank Sum test for k are shown below:

Normality Test (Shapiro-Wilk) Failed ($P < 0.050$)

Group	N	Missing	Median	25%	75%
All others combined	46	0	0.0135	0.0075	0.02
258 Main	3	0	0.06	0.045	0.07

Mann-Whitney U Statistic= 1.000

T = 143.000; n (small) = 3; n (big) = 46; P = 0.005

The difference in the median values between the two groups is greater than would be expected by chance; there is a statistically significant difference, with P = 0.005. Tables 5-13 and 5-14 summarize the values and test conditions for these two sets of data.

Table 5-13. k Summary Values and Conditions for 258 Main St.

	k (1/min)	**Rain Depth (in.)**	**Max. depth of water in dry well (in.)**	**Min. depth of water in dry well (in.)**
number	3	3	3	3
Minimum	0.05	0.69	22.32	0.11
Maximum	0.07	1.34	54.77	0.67
Average	0.06	1.08	43.57	0.44
Median	0.06	1.22	53.62	0.53
Std Dev	0.01	0.35	18.41	0.29
COV	0.22	0.32	0.42	0.67

1 in. = 25.4 mm

Table 5-14. k Summary Values and Conditions for All of the Other Sites

	k (1/min)	**Rain Depth (in.)**	**Max. depth of water in dry well (in.)**	**Min. depth of water in dry well (in.)**
number	46	63	81	81
Minimum	0.002	0.22	6.51	0.00
Maximum	0.050	2.90	93.85	82.98
Average	0.014	1.20	50.45	20.88
Median	0.014	1.15	53.76	10.07
Std Dev	0.009	0.76	22.93	24.15
COV	0.666	0.63	0.45	1.16

1 in. = 25.4 mm

Fitting Observed Data to Green-Ampt Equation

The Green-Ampt equation calculates cumulative infiltration assuming water flowing into a vertical soil profile. Figure 5-11 is an example comparison between fitted Horton and Green-Ampt equations for one of the events at a selected dry well, as well as statistical analysis and residual plots. The remaining graphs are in Appendix B.

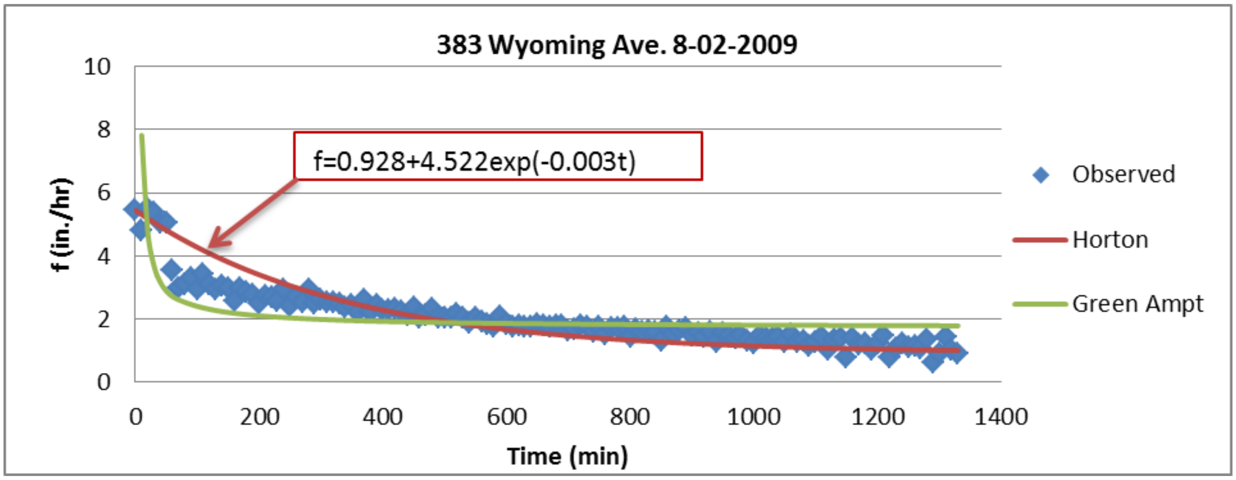


Figure 5-11. An example of fitted obsereved data to Horton equation and Green-Ampt equation (1 in. = 25.4 mm)

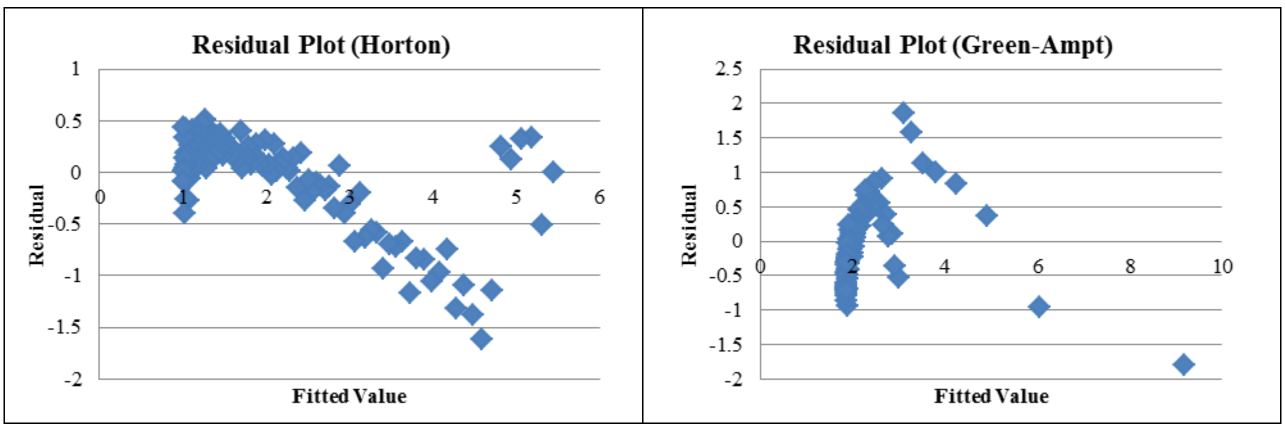


Figure 5-12. Residual Plots for Horton and Green-Ampt fitted values

Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

	df	SS	MS	F	Significance F
Regression	1	51.25879	51.25879	129.5995	2.67E-21
Residual	131	51.81269	0.395517		
Total	132	103.0715			

	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.676322	0.062127	26.98234	2.4E-55	1.553421	1.799223
X Variable 1	5.26193	0.462214	11.38418	2.67E-21	4.34756	6.1763

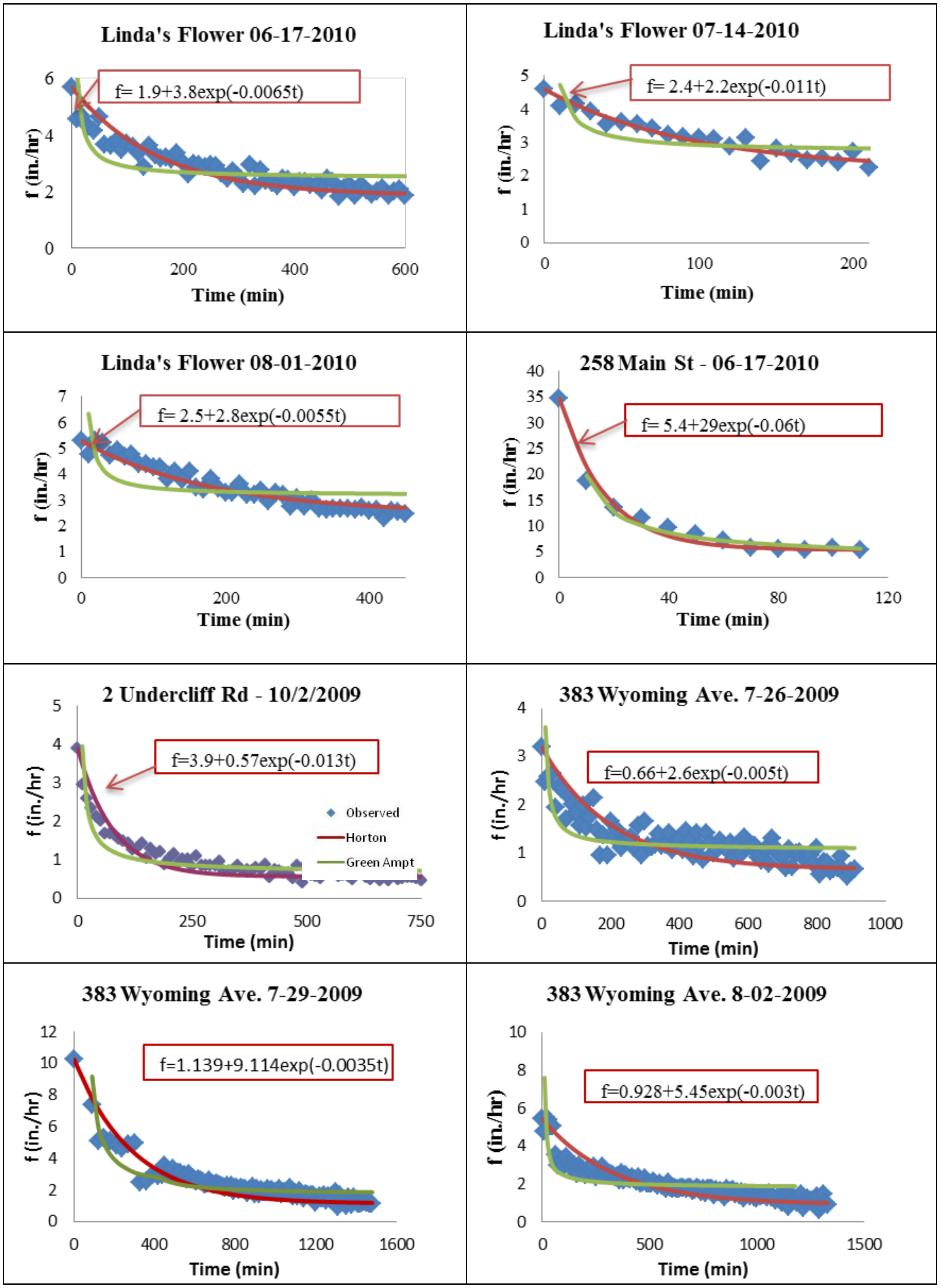


Figure 5-13. Horton and Green-Ampt fitted curves for observed data. (dots: observed data, red line: Horton and green line:Green-Ampt. The Horton equation is written on each graph) (1 in. = 25.4 mm)

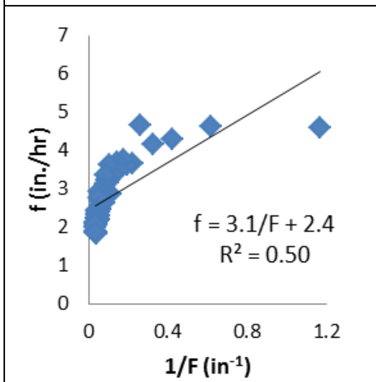
As it is shown in Figure 5-13, the Horton equation usually had a better fit to the data compared to the Green-Ampt equation for the Millburn data. However, for some sites, the Green-Ampt equation was a better fit. As noted previously, a linear relationship between f_t and $(1/F_t)$ is needed to determine the Green-Ampt equation parameters. Figure 5-14 presents the linear regressions of f_t vs $(1/F_t)$ for the monitored sites. The only visually acceptable linear regression is associated with the observations from the 258 Main St. site (the only location that had soils in the A group from the surface to about 1.1 m (3.5 ft) deep). This site also had the best Green-Ampt fitted equation shown in Figure 5-13 as well. In almost all cases, the linear relationship between f_t vs $(1/F_t)$ is unacceptable (except for this one location), making the Horton equation a more suitable tool for calculating expected infiltration for the dry wells.

Table 5-15. Green-Ampt parameters

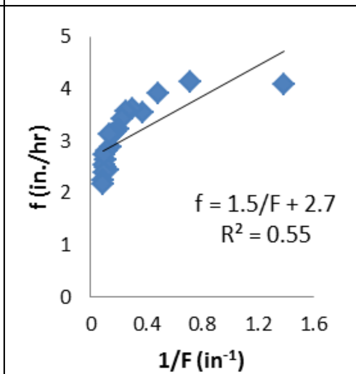
Site Address	Date	Hydraulic conductivity K (in./hr) estimated	Hydraulic conductivity K (in./hr) Rawls et al. (1983)
Linda's Flower	06-17-2010	2.435	0.429
	07-14-2010	2.685	
	08-01-2010	3.131	
258 Main St.	06-17-2010	1.018	1.17
2 Undercliff	10-02-2009	0.557	0.429
383 Wyoming Ave.	7-26-2009	1.039	0.13-0.43

1 in. = 25.4 mm

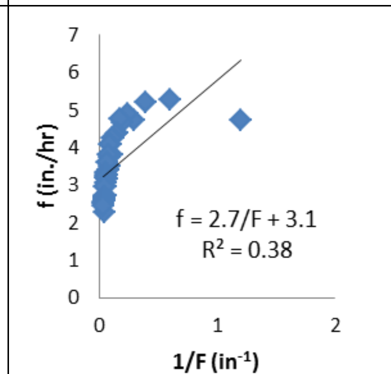
Linda's Flower 06-17-2010



Linda's Flower 07-14-2010



Linda's Flower 08-01-2010



258 Main St - 06-17-2010	2 Undercliff Rd - 10/2/2009	383 Wyoming Ave. 7-26-2009

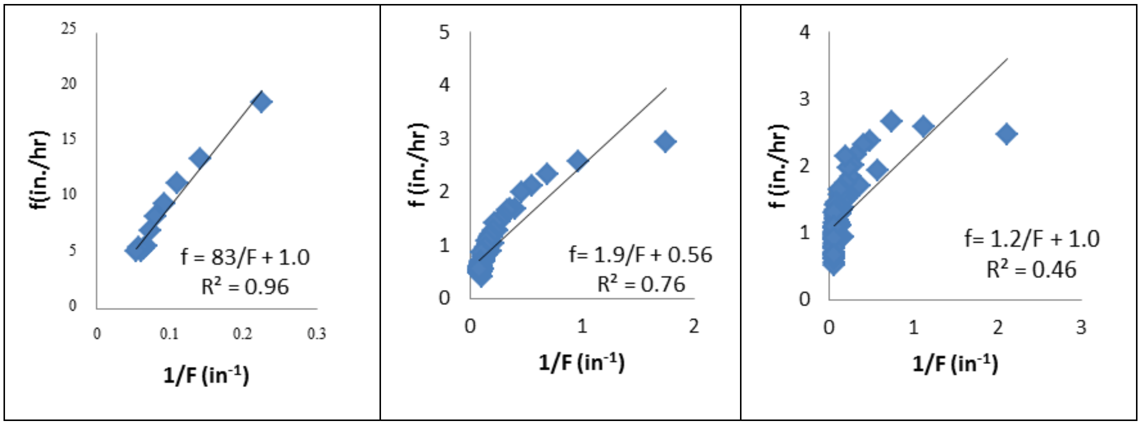


Figure 5-14. Linear regression of ft vs 1/F_t for some sites in Millburn, NJ. (1 in. = 25.4 mm)

Summary of Recharge Observations with Dry Wells

Groundwater recharge is a suitable beneficial use of stormwater in many areas used to augment local groundwater resources. This study showed how the dry wells could be very effective in delivering the stormwater to the shallow groundwaters. Even though the surface soils were almost all marginal for infiltration options, the relatively shallow dry wells were constructed into subsurface soil layers that had much greater infiltration potentials. However, some of the monitored dry well locations experienced seasonal high groundwater elevations, restricting complete draining of the dry wells after rains. While surface and subsurface soil information is readily available for the Township (and in most other areas of the country), the presence of the shallow water table (or bedrock) is not well known. This makes identifying the most suitable locations for dry wells difficult, as the seasonal groundwater should be at least 2.4 m (8 ft) below the ground surface (or 60 cm, 2 ft, below the lowest gravel fill layer beneath the dry well).

Calculating the benefits of the dry wells (including developing sizing requirements) requires the use of an appropriate infiltration equation, preferably as part of a continuous model examining many years of actual rain fall data for a specific area. Two commonly used infiltration models, the Horton and Green-Ampt equations, were evaluated for their potential use to calculate groundwater recharge at the case study locations in the Township of Millburn, NJ. The fitted graphs and resulting derived equation parameters indicate that although the Horton curve is usually a better fit to the observed data, the calculated parameters of both infiltration models are not close to values reported in the literature for urban areas. This is likely because the infiltration characteristics in the dry wells were mostly affected by subsurface conditions compared to the literature values that were compared to the surface soil characteristics. When the subsurface conditions are used in the comparisons, the observed and literature values are in better (but still not close) agreement. Therefore, locally measured infiltration test data at a scale approaching the size and depth of the final devices should be used for more reliable design guidance, instead of rely on literature values.

Factors Affecting Infiltration Rates

The data analyses of the infiltration data indicated several interesting conclusions. One of the first issues noted by the field personnel when installing the level recorders and observing the dry wells over time is that some of the locations experienced periodic (or continuous) standing water in the dry wells, indicating seasonal or permanent high water table conditions, or partially clogged dry wells.

Figures 5-15 and 5-16 are time series plots of the water levels for the long-term infiltration tests at the dry wells and are very informative concerning the trends and overall behavior of the infiltration characteristics at the different sites. The hydrant water tests are shown separately (with expanded time scales), and are also shown on the longer period plots. The plots show the water elevations in the dry wells along with the corresponding rain depths as recorded at the nearest rain gage. In some cases, dry well activity is indicated with no corresponding rainfall. This is likely due to variable (small) rains in the areas that were not recorded at all of the gages. The rain data indicate the total rain depth and the start and end times; the graphs cover too long of a period to show variable rain intensities during the rains. The times and depths are the most important rain information for these measurements, as they relate most closely to the runoff quantity and the dry well water elevations.

In almost all cases, the general shapes of the recession limbs (water elevation drops with infiltration) are similar for the same site, including the hydrant tests. However, some changed with time, including several that indicated slower infiltration with more standing water conditions in the winter and spring. This may be due to SAR issues (sodium adsorption ratio) that results in dispersed clays from high sodium content in snowmelt. Normally, snowmelt would not affect these units if only roof runoff is directed to the dry wells. However, if walkway or driveway runoff drains to dry wells, de-icing chemicals (heavy salt loads) may be in the runoff.

Standing water was observed in the dry well at 87/89 Tennyson when sufficient time occurred to allow the water to reach a consistent minimum water level (about 0.9 m or 3 ft deep). It is expected that this site very likely has a high water table condition. The drainage rates were very slow, so the interevent periods were not sufficiently long to enable drainage to the stable water level until after about a two week dry period. The slow drainage rate may have been caused by saturated conditions.

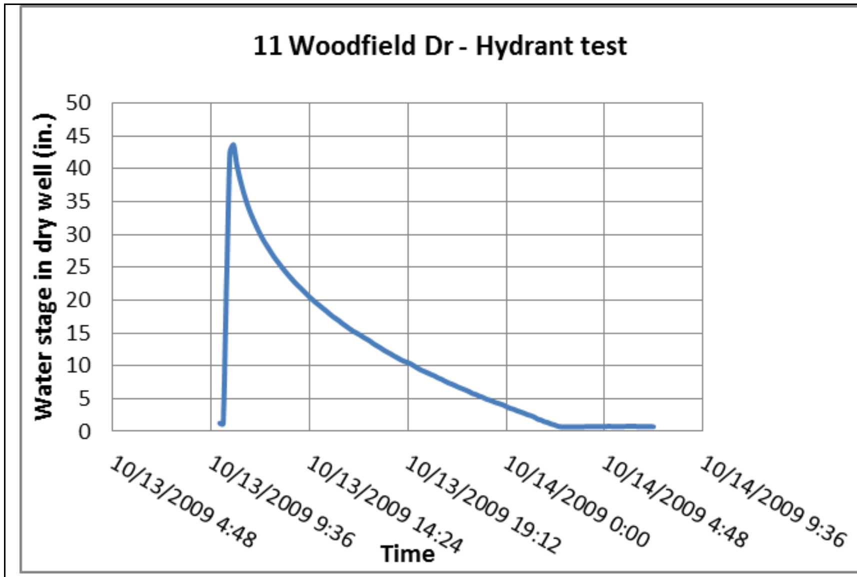
Several sites (260 Hartshorn, 7 Fox Hill, and 142 Fairfield) experienced periodic slowly draining conditions, mainly in the spring that could have been associated with SAR problems. The slow infiltration rates could be due to poor soils (with the clays resulting in SAR problems), or saturated soil conditions.

The other sites all had rapid drainage rates that were consistent with time.

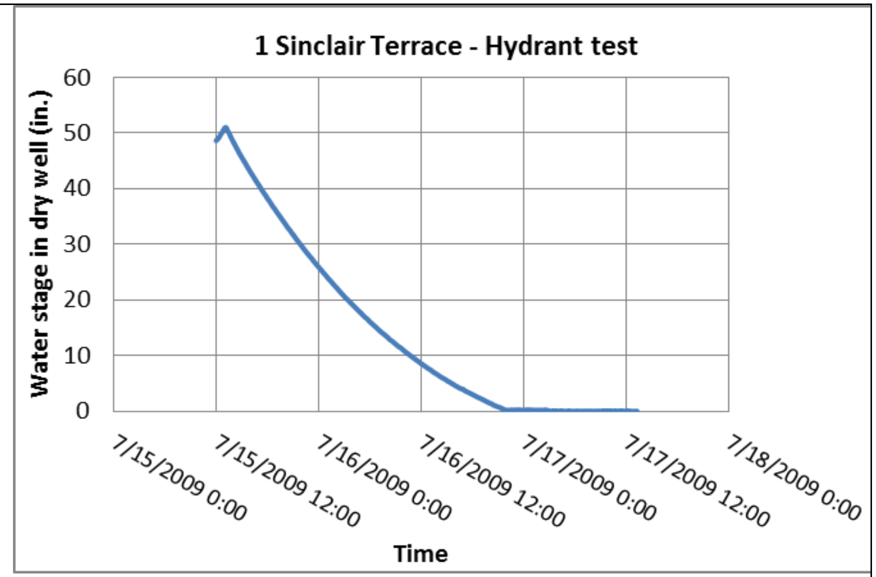
Table 5-16. Summary of Infiltration Conditions with Time

	Start date of series	End date of series	# of dry well events	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments
11 Woodfield Dr.	Oct 11, 2009	Dec 20, 2009	1 hydrant 5 rains (1 small rain missing)	89%	Consistent shape with time	Quickly drained (within a day); No standing water at any time	15 hr total drainage time during hydrant test
15 Marion Dr.	June 17, 2010	August 6, 2010	1 hydrant 5 rains (2 small rains missing)	71%	Consistent shape with time	Several days to drain; No standing water at any time	4.5 days total drainage time during hydrant test
383 Wyoming Ave.	July 16, 2009	October 14, 2009	1 hydrant 6 rains (2 small rains missing)	81%	Consistent shape with time	Several days to drain if full; No standing water at any time	1 day total drainage time during hydrant test
258 Main St.	June 16, 2010	August 5, 2010	5 rains (2 smaller rains missing)	98%	Consistent shape with time	Very rapid drainage time; No standing water at any time	
260 Hartshorn	August 9, 2010	August 1, 2011	Many!	10%	Consistent shape with time	Slow drainage time (about a week if full), but dry if given enough time between rains	Clogging or poor soils, not high water table. Possible SAR issues in the Winter and Spring, recovered by mid-summer.
2 Undercliff Rd	July 18, 2009	October 6, 2009	1 hydrant 3 rains	79%	Consistent shape with time	Several days to drain if full; No standing water at any time	10 days total drainage time during hydrant test
87/89 Tennyson	August 10, 2010	August 5, 2011	Many	0%	Consistent shape with time	Very slow drainage time (a couple of weeks); standing water and never dry during this year period	Slow drainage may be due to saturated conditions, never reached stable low water level. If due to SAR, did not recover.

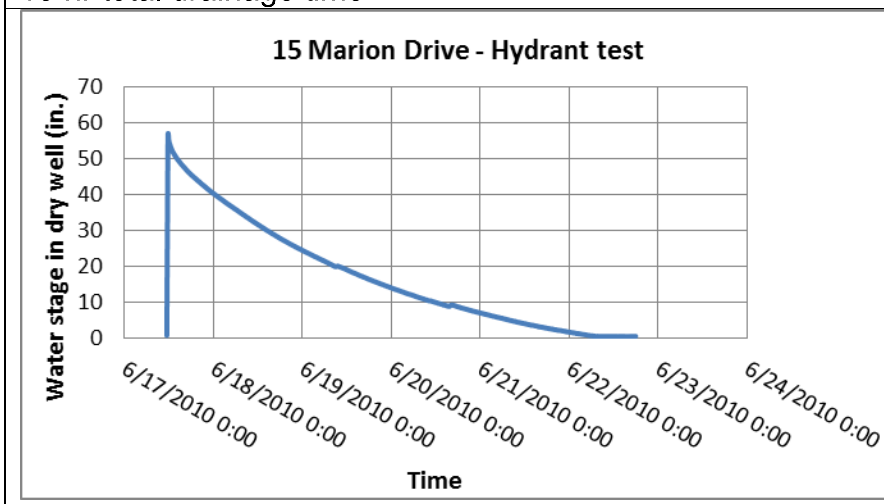
	Start date of series	End date of series	# of dry well events	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments
7 Fox Hill	August 7, 2010	March 23, 2011	Many	2%	Consistent shape with time	Slow drainage time (about a week or two if full), but dry if given enough time between rains	Clogging or poor soils especially in Spring, possibly SAR issues, not high water table
8 So. Beechcroft	July 19, 2009	September 27, 2009	1 hydrant 6 rains	71%	Consistent shape with time for rains, but hydrant test (at end of periods at end of Sept) was very rapid	Quickly drained (within a day or two if full); No standing water at any time	3 hr total drainage time (half full) during hydrant test
142 Fairfield	August 10, 2010	March 4, 2011	many	66%	Somewhat inconsistent shape with time	Quickly drained (within a day or two if full) to poorly drained (a week for moderate rains); Standing water during periods of large and frequent rains	Slowly drained conditions in Spring likely due to saturated conditions, or SAR. Not likely due to high water table
36 Farley Place	June 16, 2010	August 5, 2010	3 rains	97%	Consistent shape with time	Very rapid drainage time; No standing water at any time	



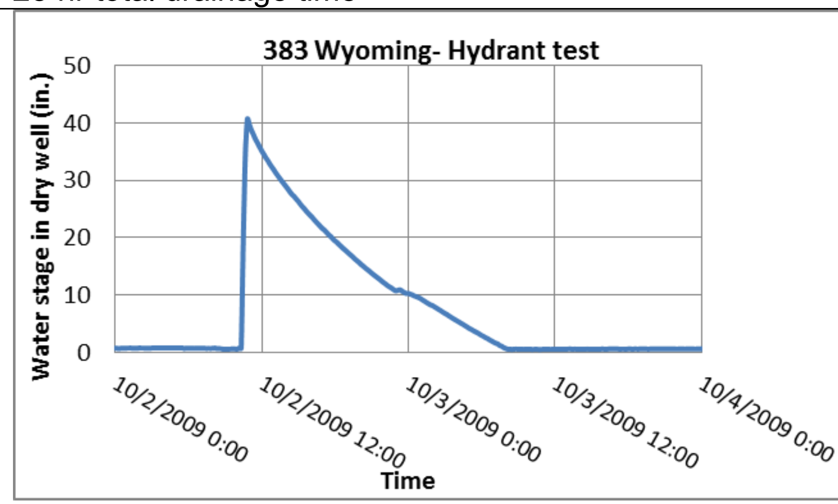
15 hr total drainage time



28 hr total drainage time

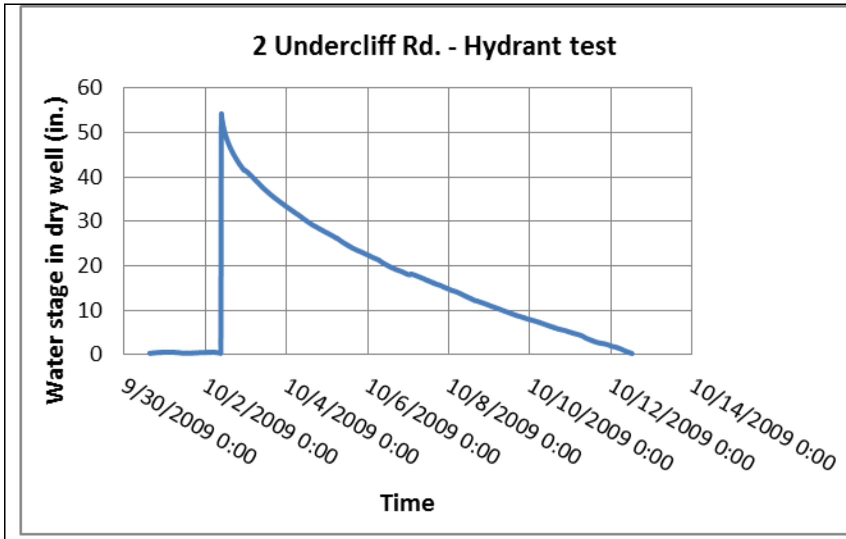


4.5 days total drainage time

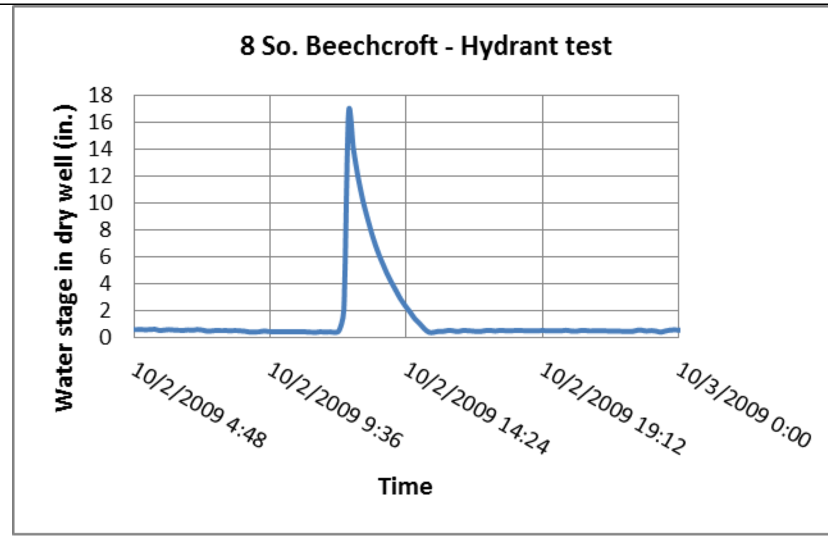


1 day total drainage time

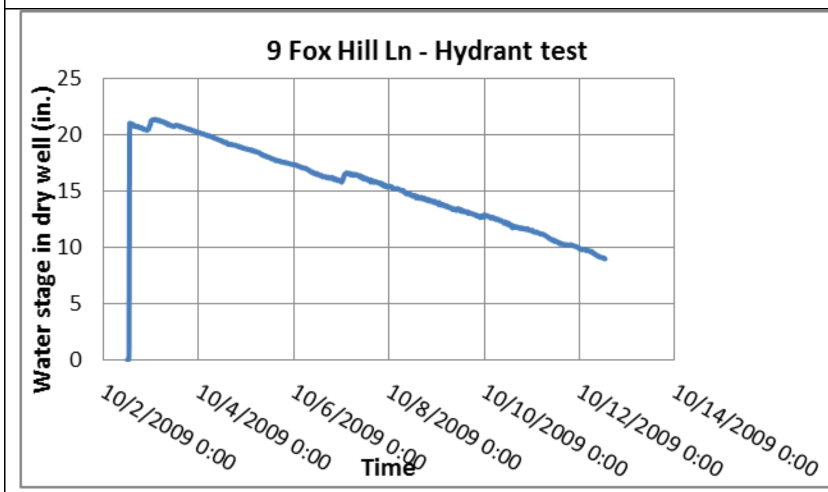
Figure 5-15. Hydrant water test infiltration plots. (1 in. = 25.4 mm)



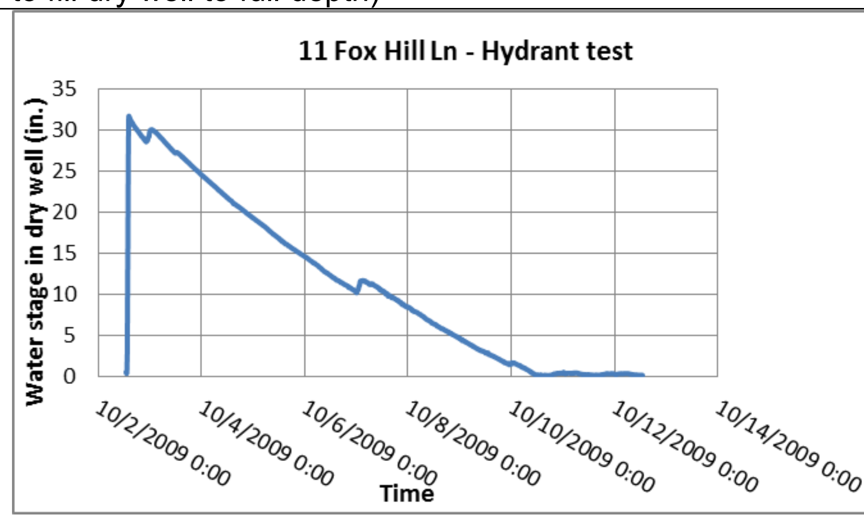
10 days total drainage time



3 hr total drainage time (only half full as too rapid infiltration to fill dry well to full depth)

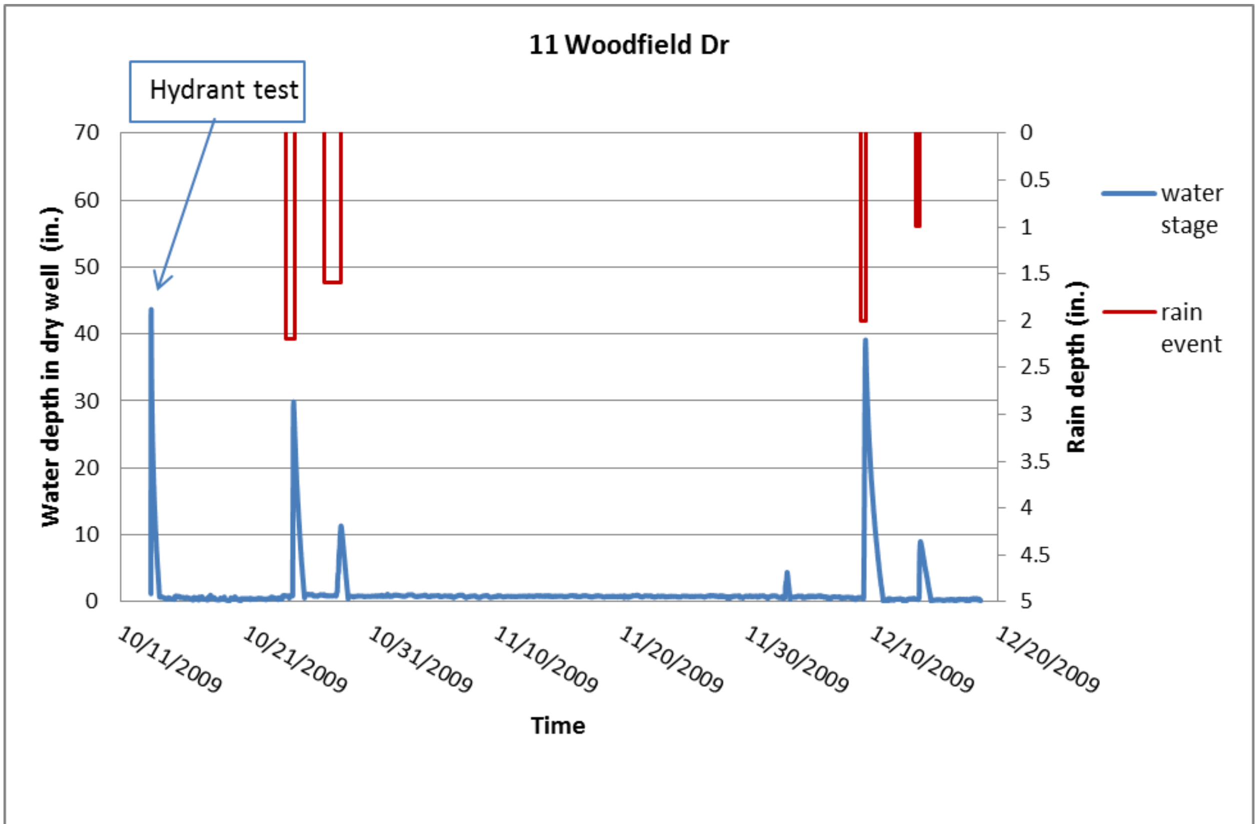


About 20 days total drainage time (extrapolated as rains interrupted test)



8 days total drainage time

Figure 5-15. Hydrant water test infiltration plots (cont.).(1 in. = 25.4 mm)



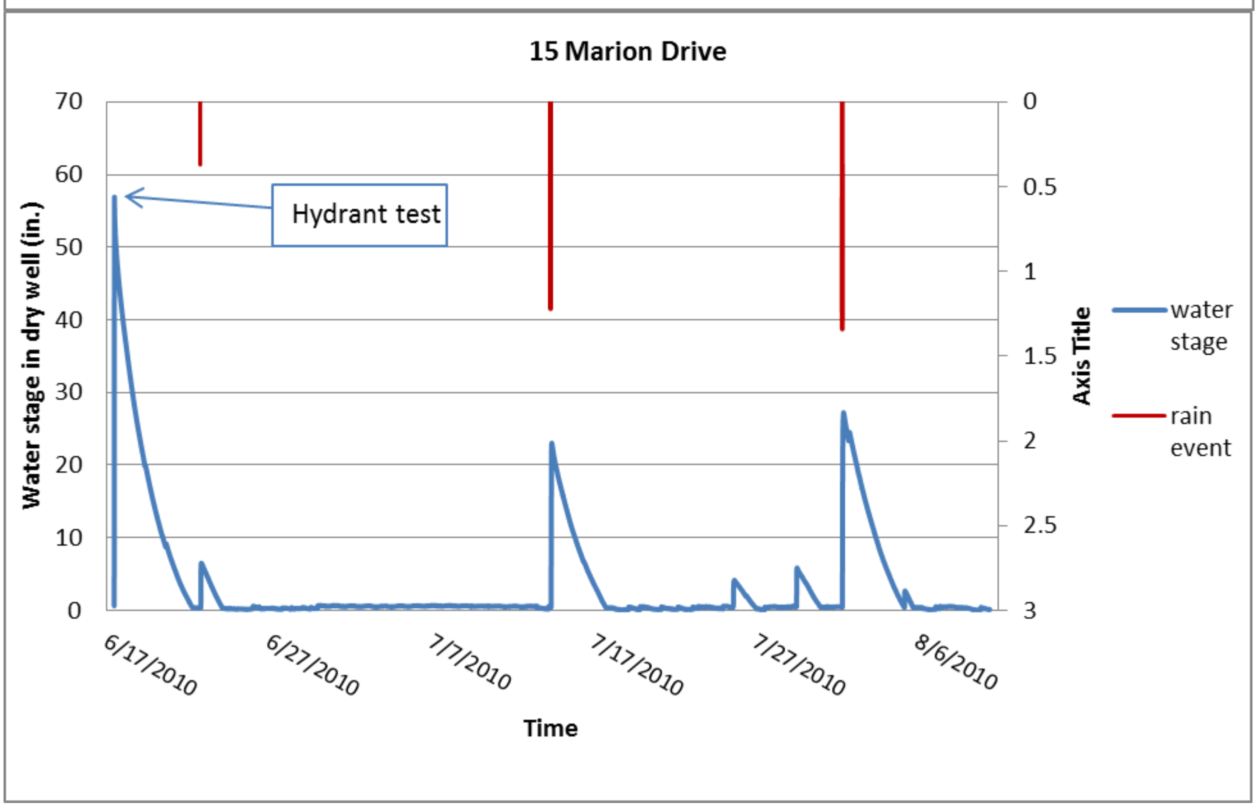
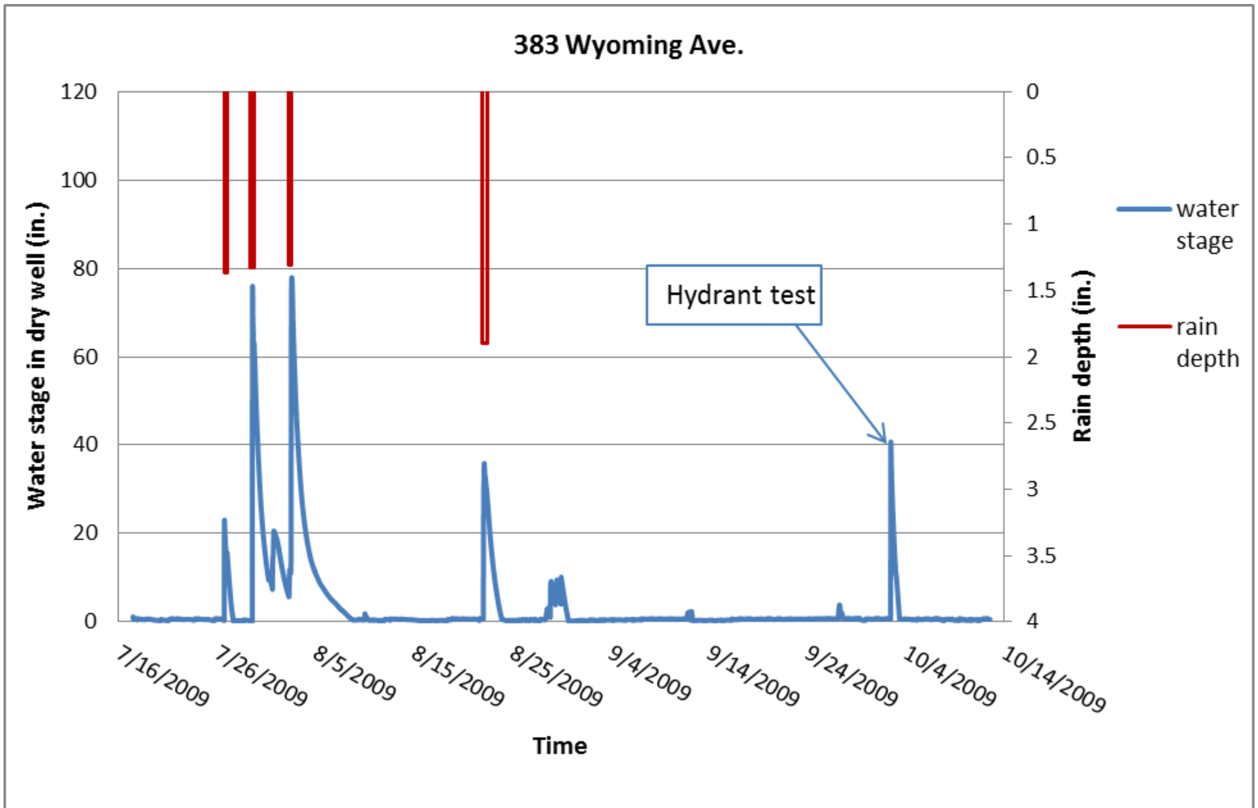


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (1 in. = 25.4 mm)



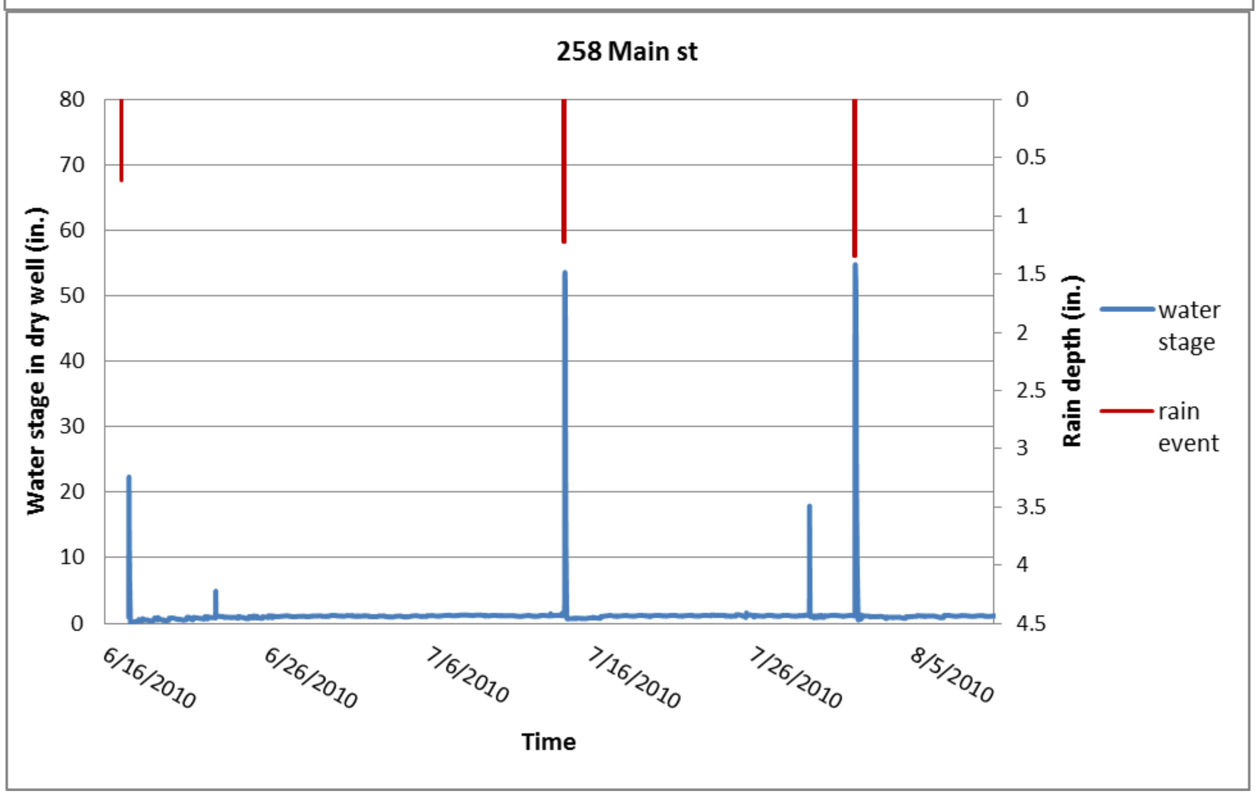
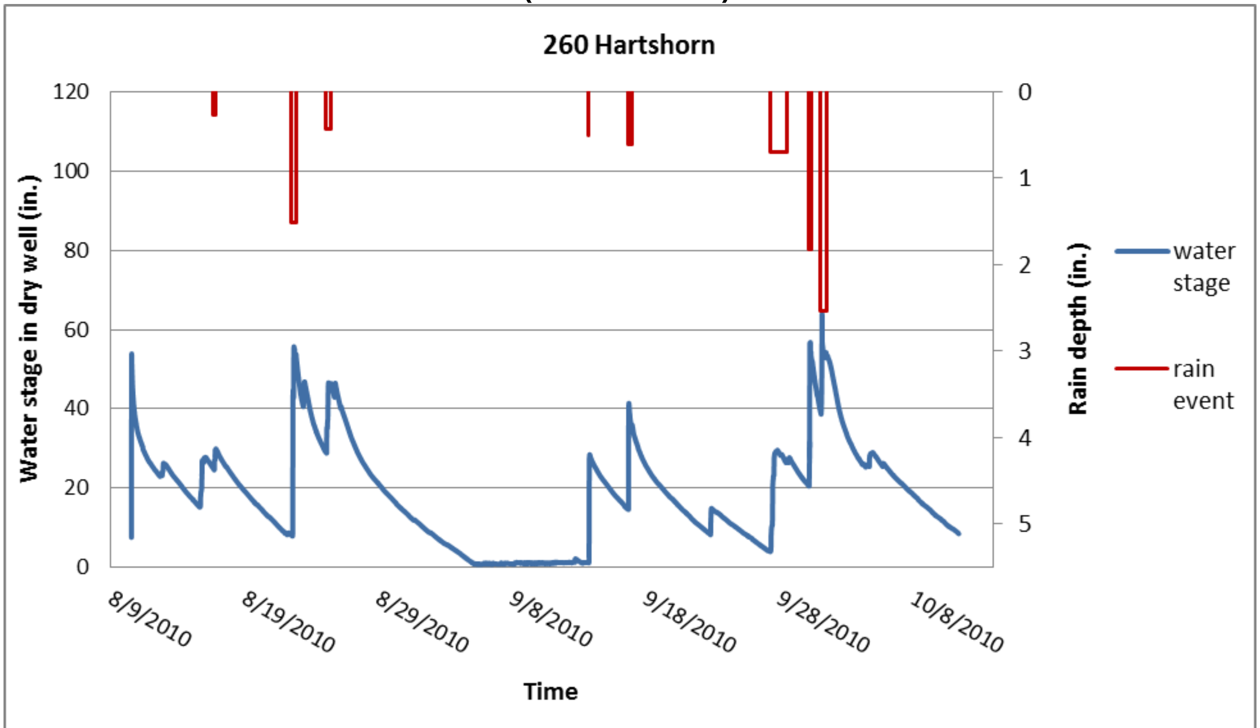
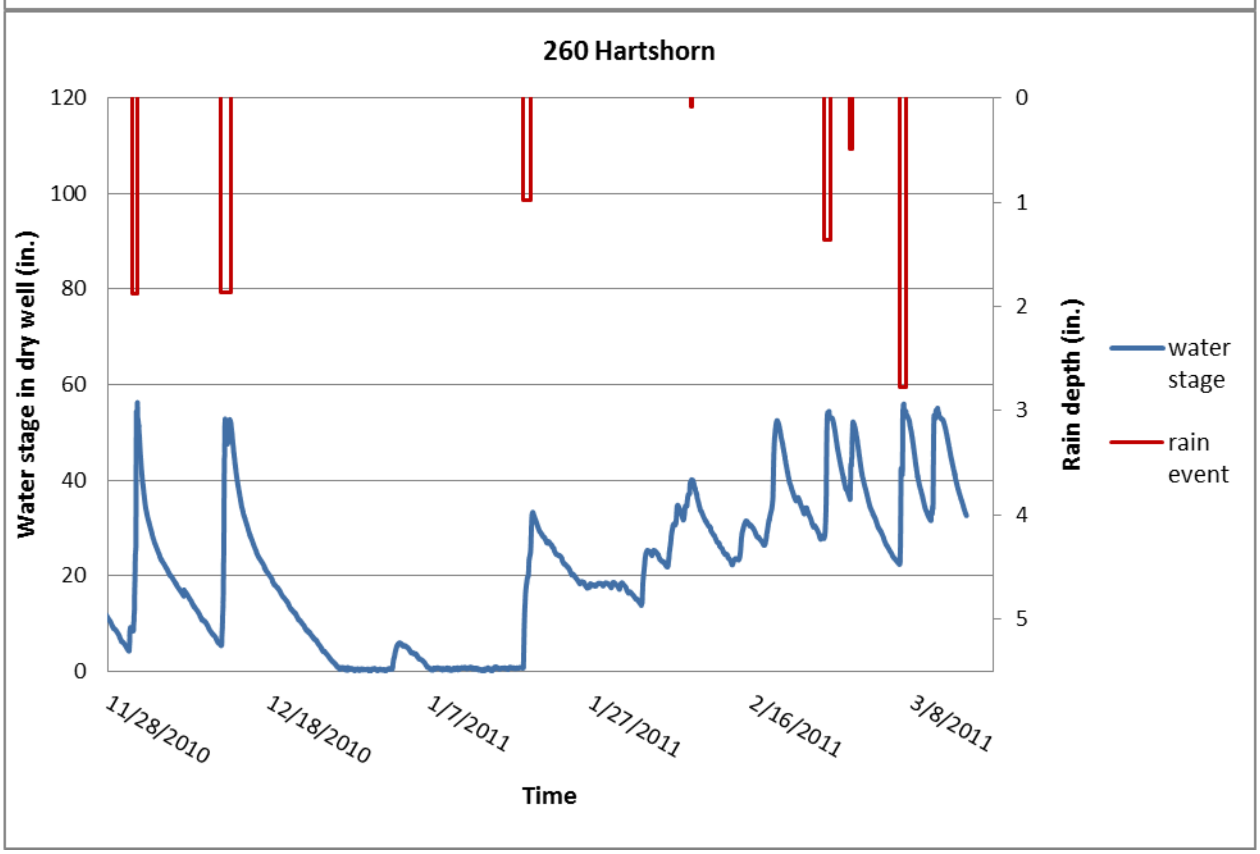


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)

(1 in. = 25.4 mm)

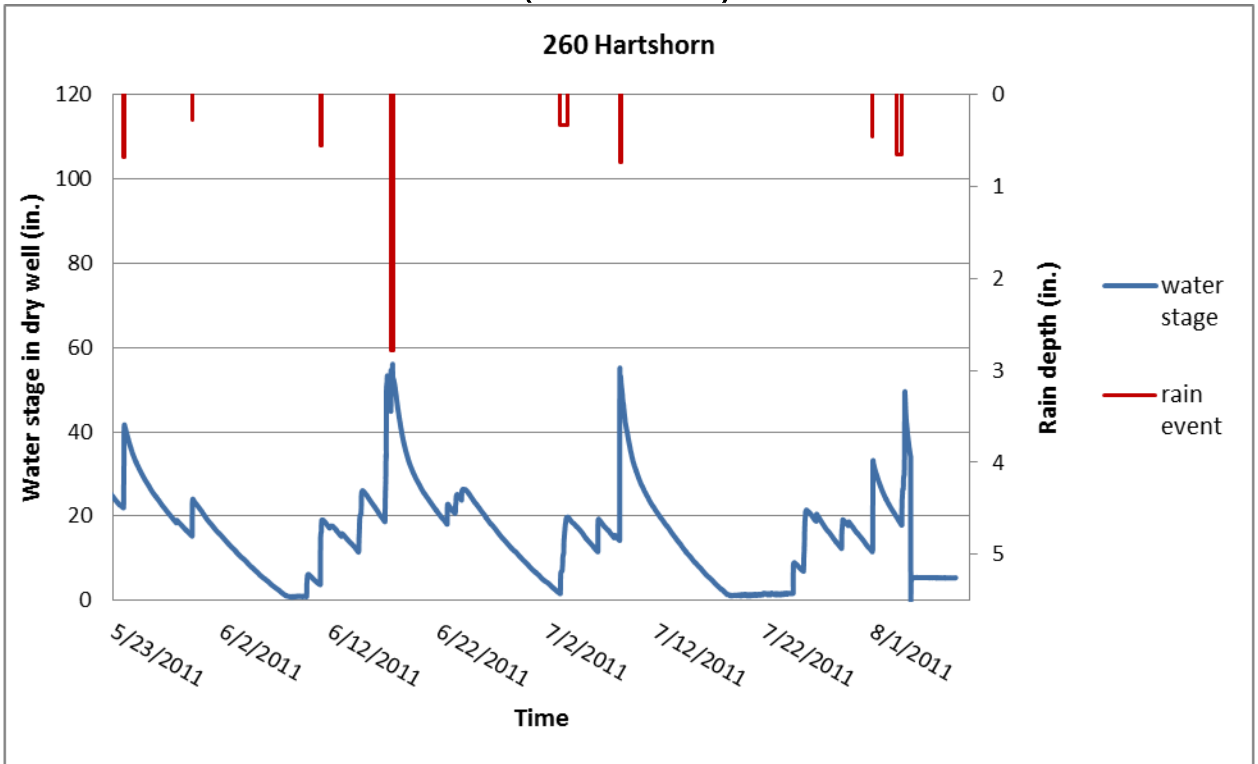


260 Hartshorn
Water stage in dry well (in.)
120
100
80
60
40
20
0
Rain depth (in.)
0
1
2
3
4
5
8/9/2010
8/19/2010
8/29/2010
9/8/2010
9/18/2010
9/28/2010
10/8/2010
Time
water stage
rain event



260 Hartshorn
Water stage in dry well (in.)
120
100
80
60
40
20
0
Rain depth (in.)
0
1
2
3
4
5
11/28/2010
12/18/2010
1/7/2011
1/27/2011
2/16/2011
3/8/2011
Time
water stage
rain event

Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



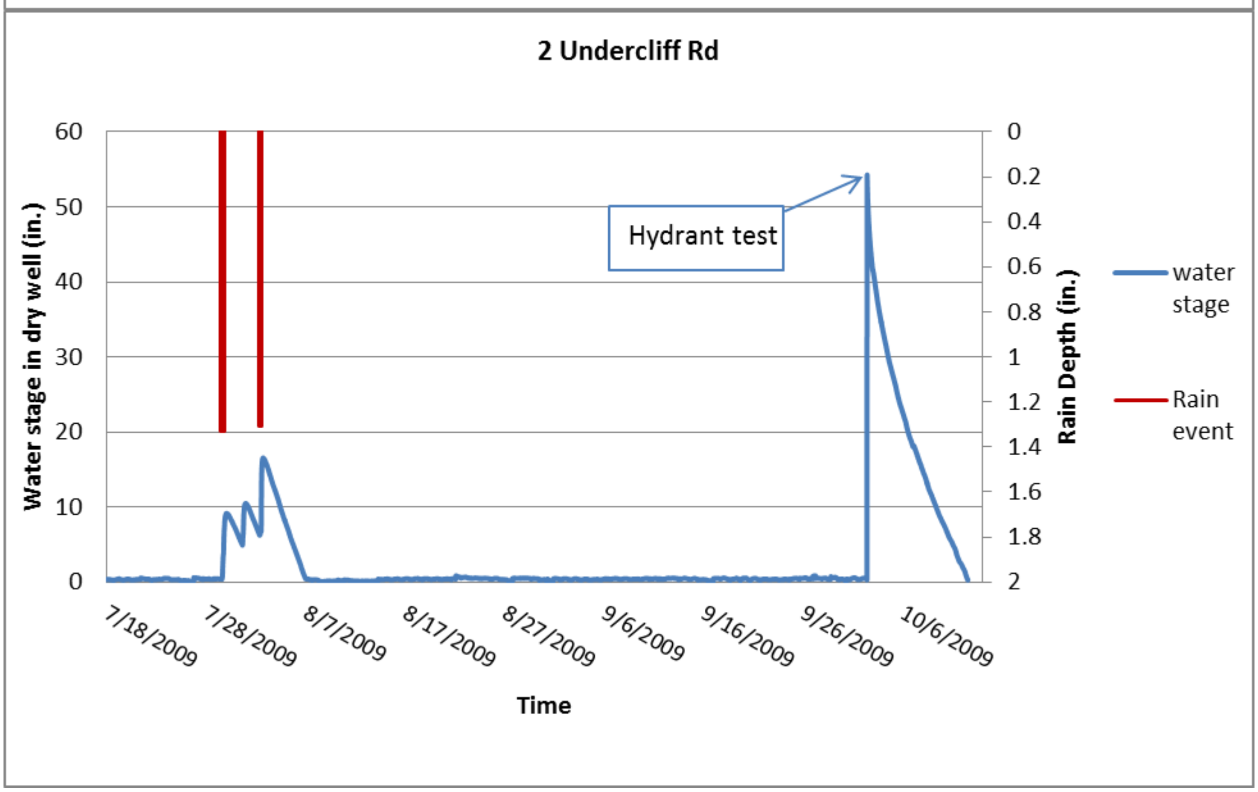
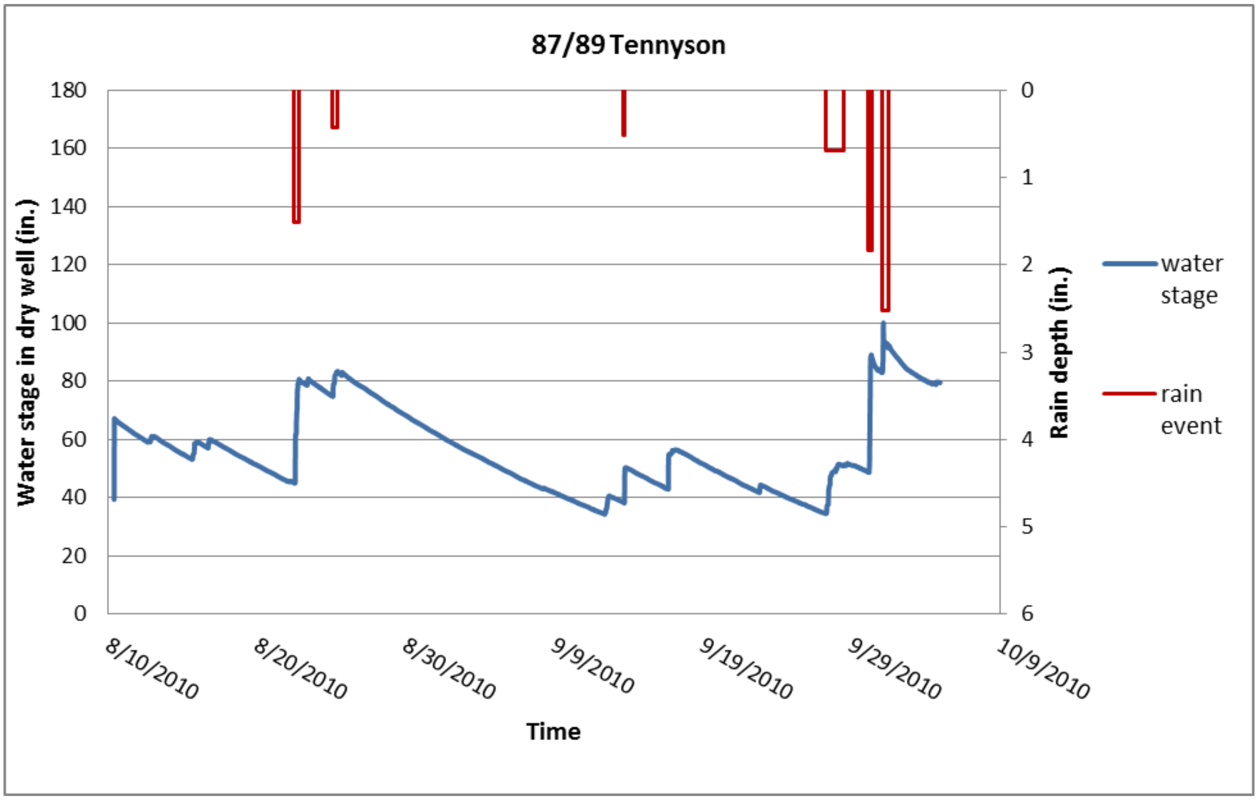


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



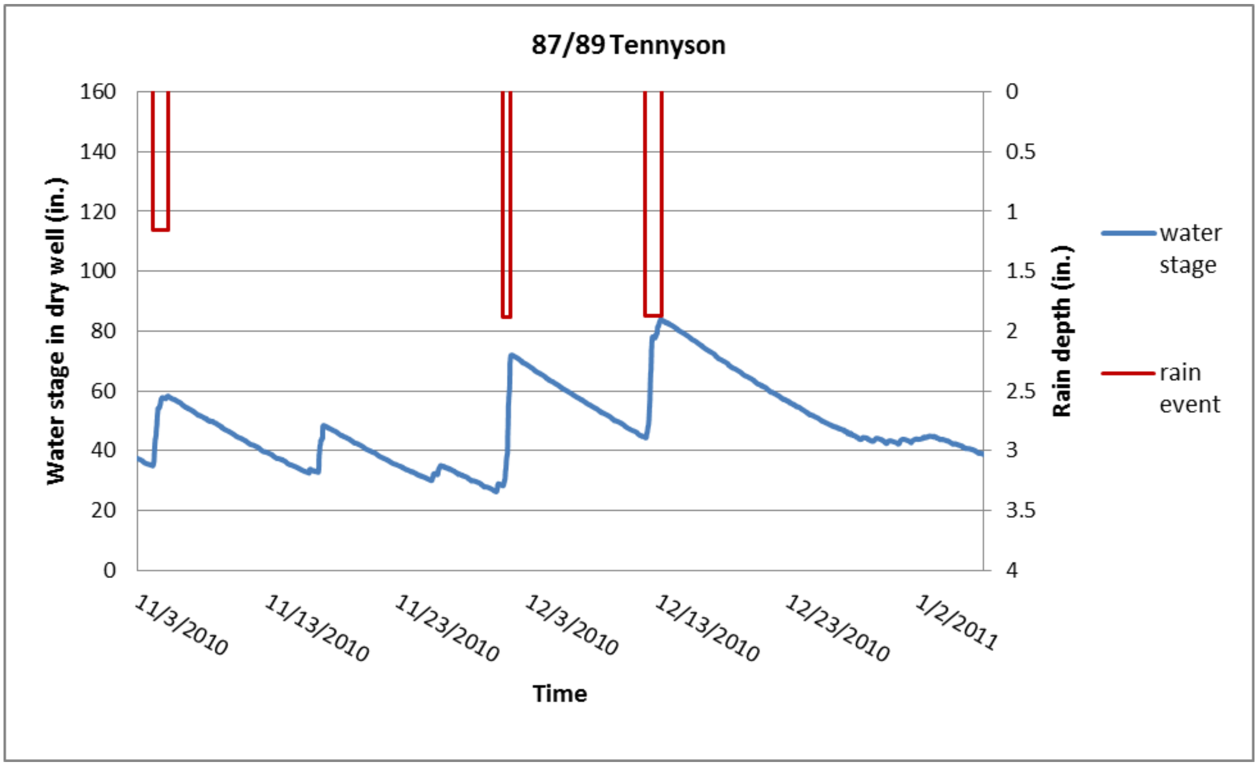
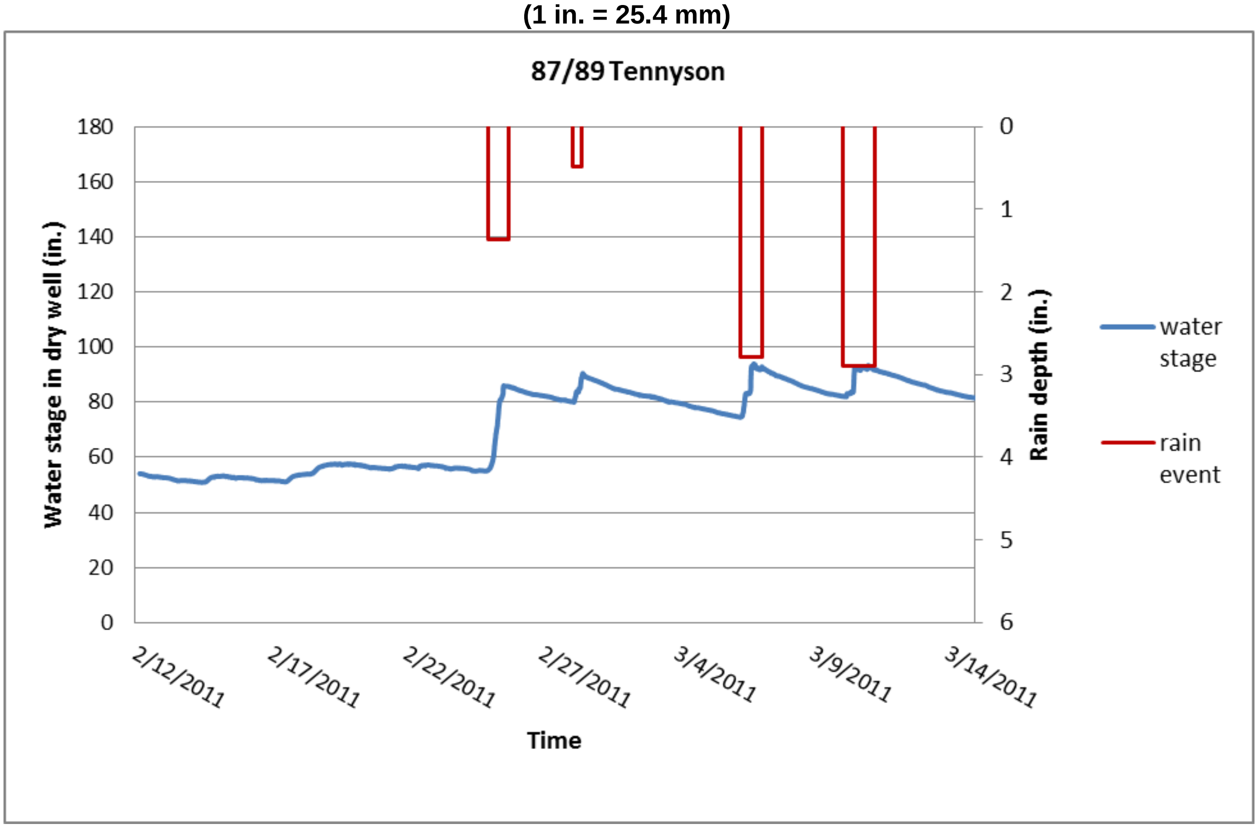


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



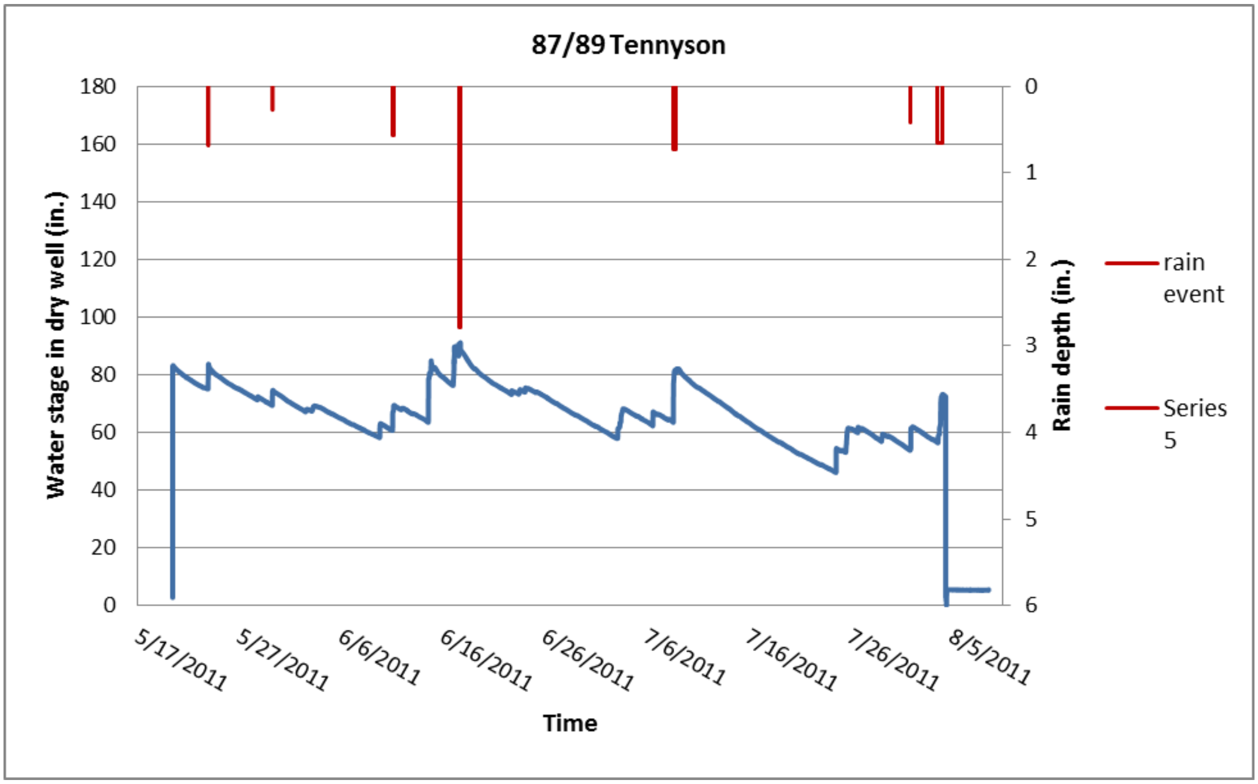
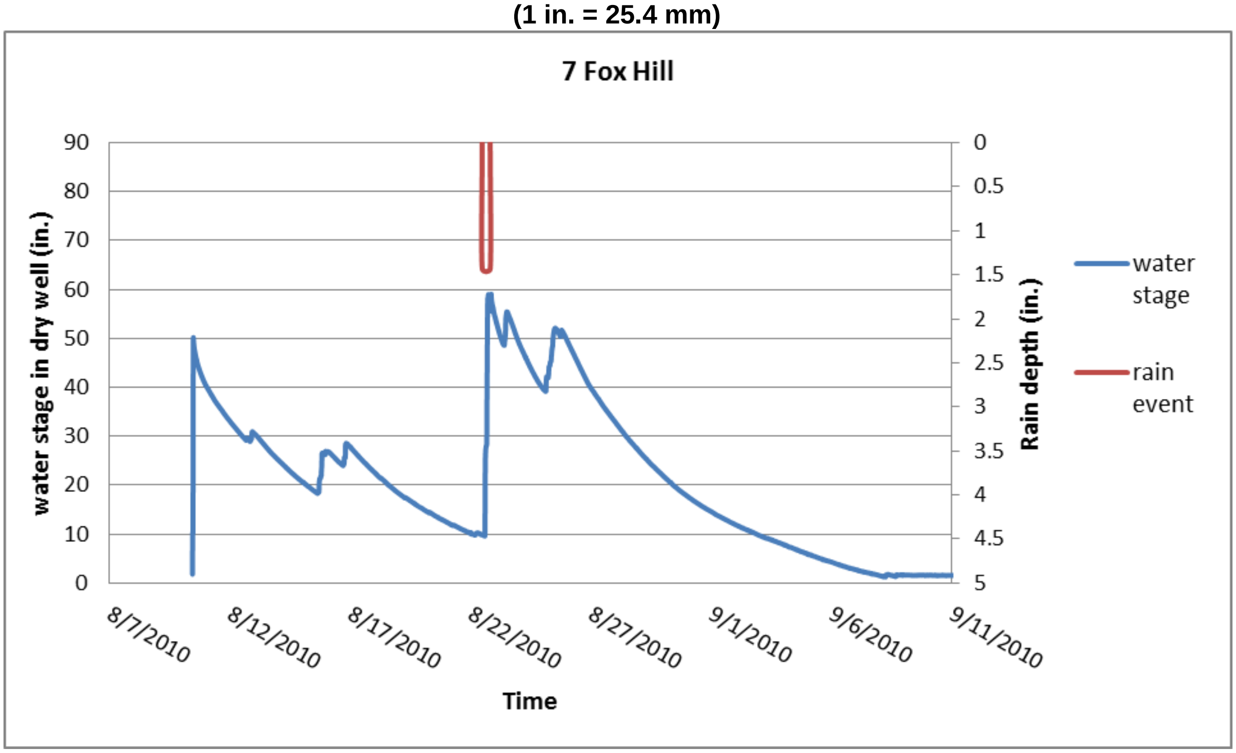


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



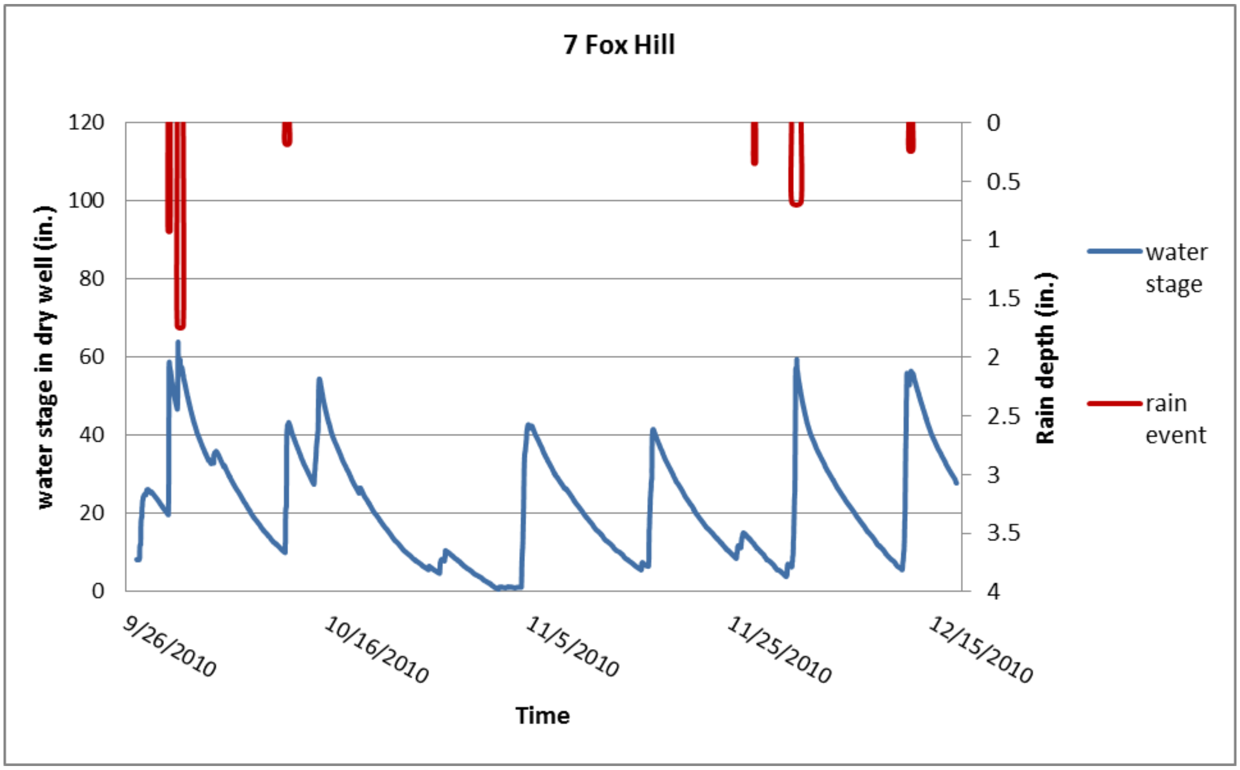
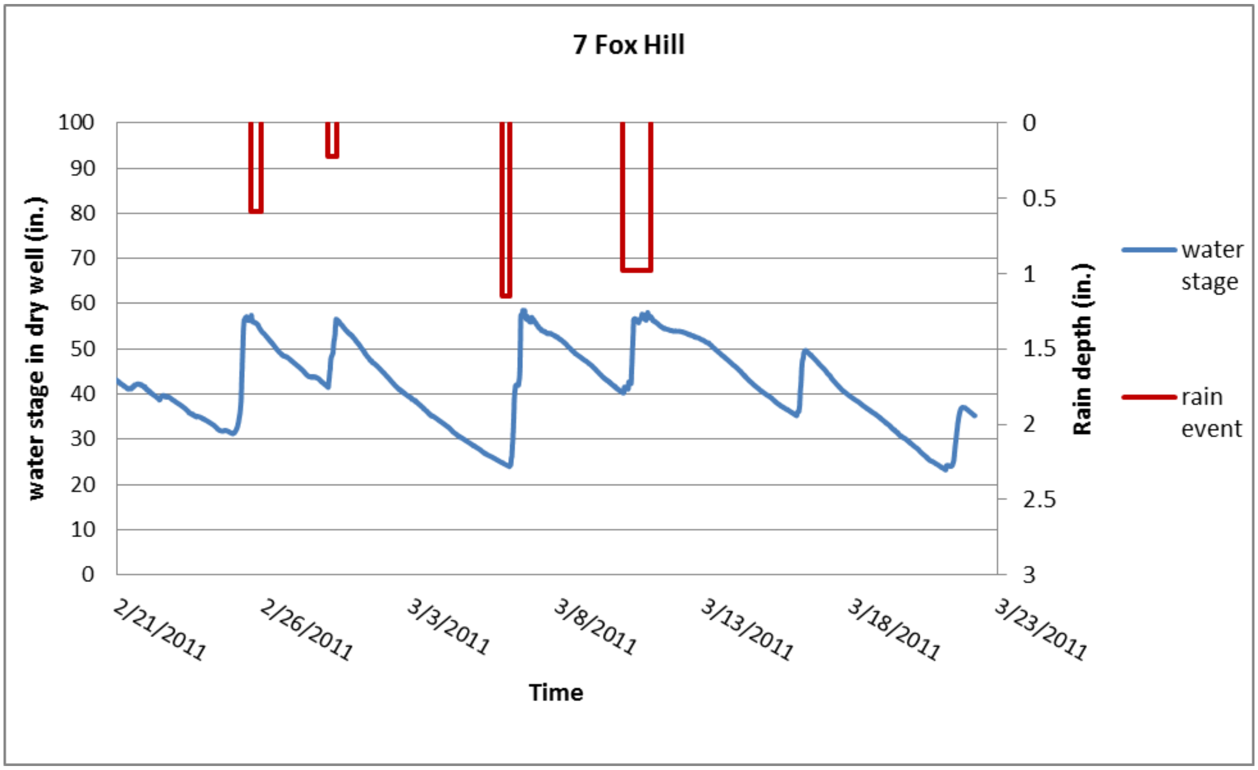


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



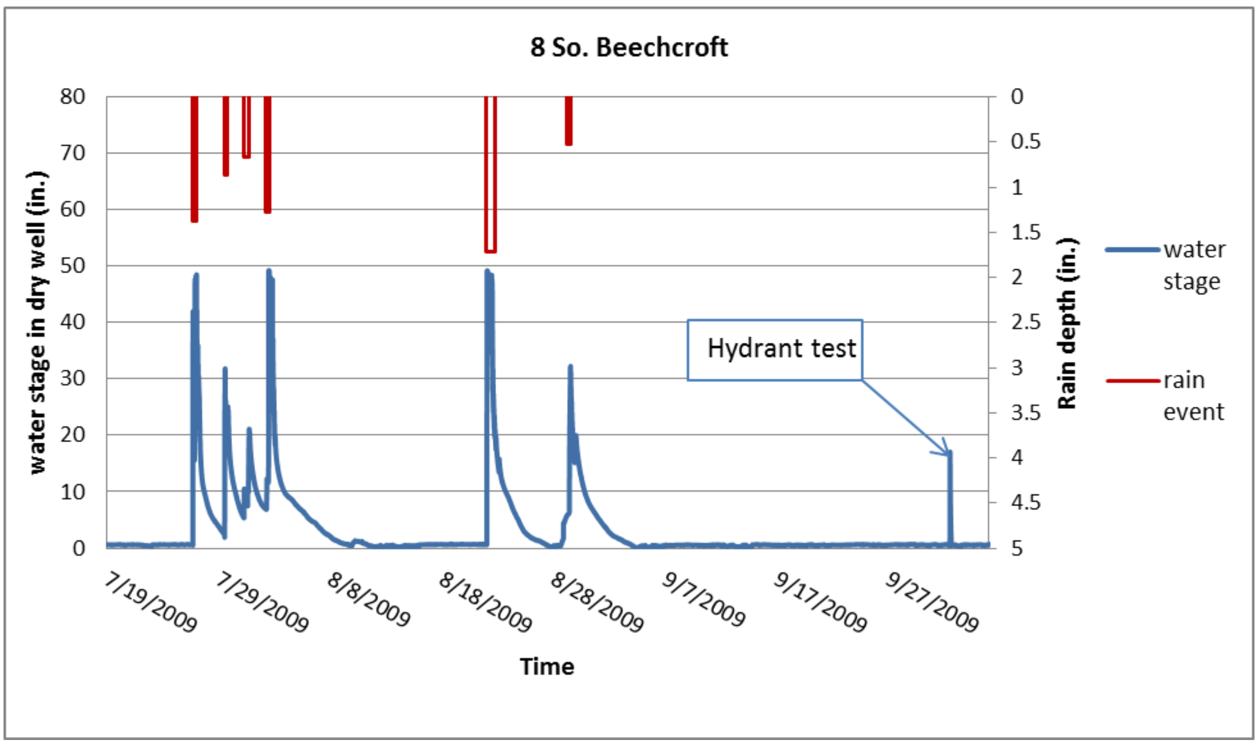
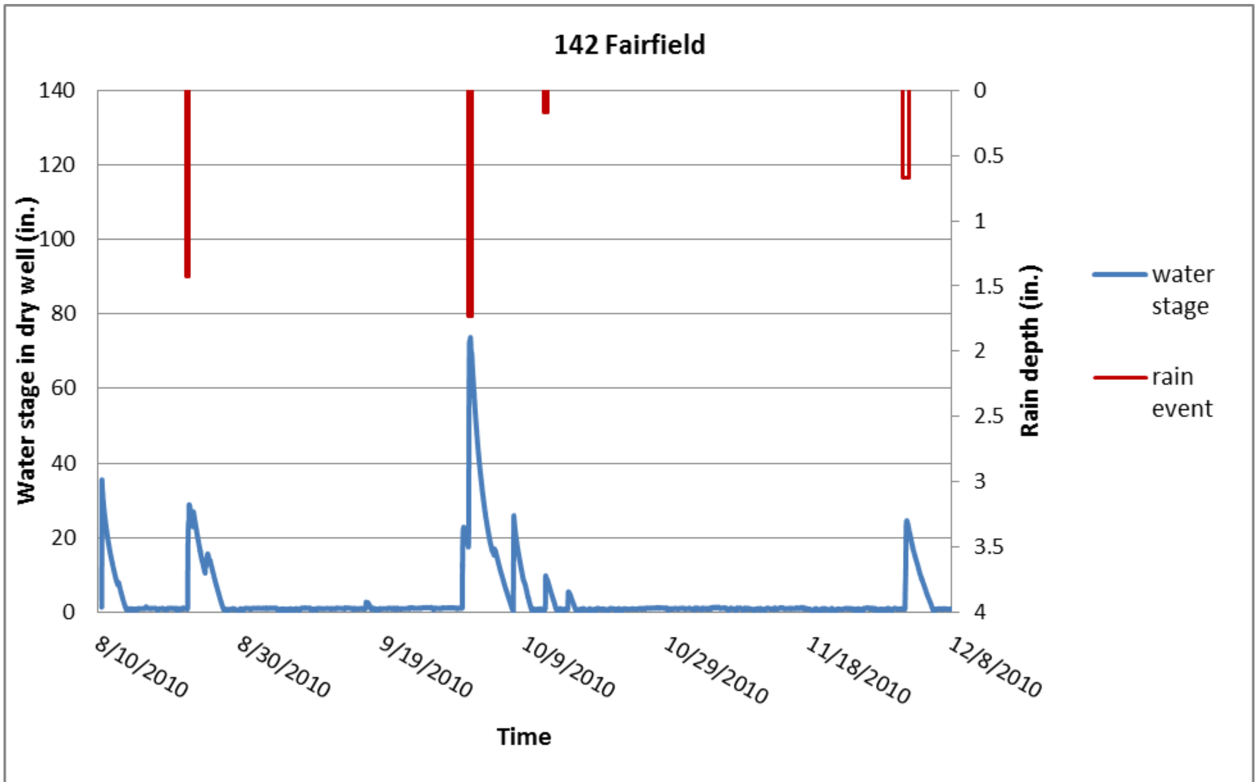


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)



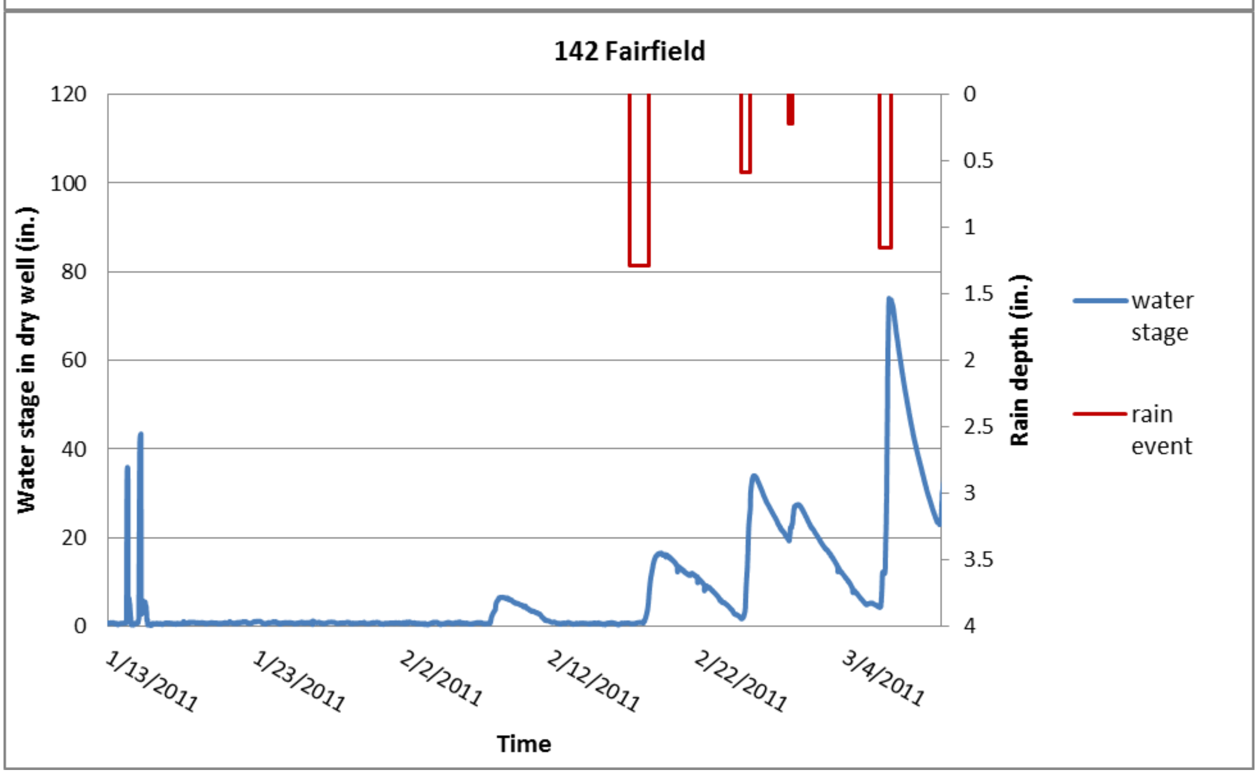


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)

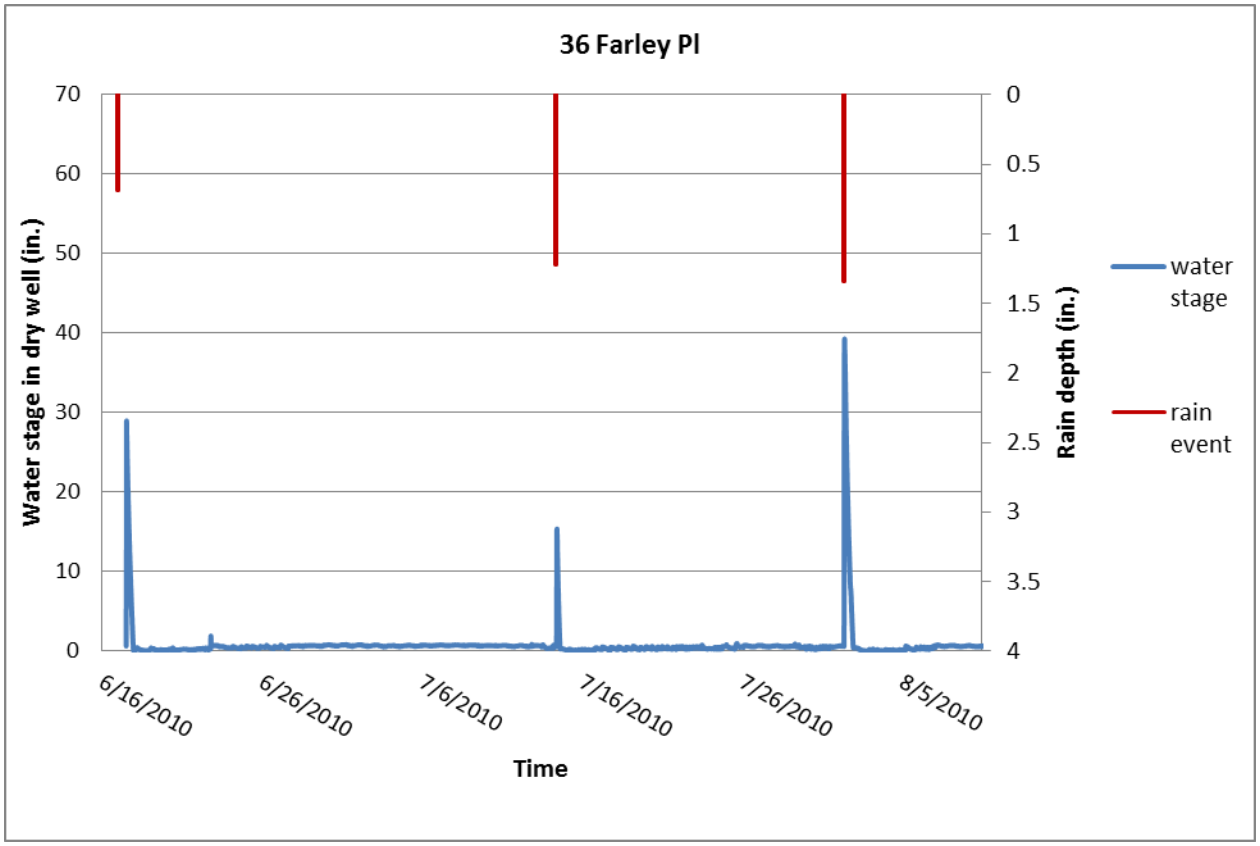


Figure 5-16. Time series plots of the water levels for the long-term infiltration tests at the dry wells (cont.)
(1 in. = 25.4 mm)

The New Jersey dry well disposal regulations for stormwater require that the seasonal water table be no closer than two ft below the bottom of the dry well (and underlying rock storage area) due to expected deceased performance (and increased groundwater contamination potential). Water table information was not readily available and some of the dry wells were apparently constructed in areas having water tables that were too shallow. The following list shows the water table conditions at the dry well monitoring locations:

• Sites having no standing water after the events (completely drained with no apparent high water table conditions):

11 Woodfield Dr
15 Marion
258 Main St
1 Sinclair Terrace (only one observation)
8 South Beechcroft Rd
11 Fox Hill Lane (only one observation)
36 Farley Place

• Sites having a few standing water conditions after the events (standing water of several inches, or more, indicating possible seasonal high water table conditions):

2 Undercliff Rd (one high water condition out of 3 observations; July 29 2009 event)
383 Wyoming Ave (one high water condition out of 5 observations; July 29 2009 event)
142 Fairfield Dr (two high water conditions out of 7 observations; Feb 26, 2011 and March 7, 2011 events)

• Sites with all or most events having high water conditions:

260 Hartshorn Dr (16 of 19 observations had high water conditions; 8/25/10, 5/30/11, and 7/8/11 drained almost completely)
87/89 Tennyson Dr (20 out of 20 observations had high water conditions)
7 Fox Hill Lane (9 of 11 observations had high water conditions; 8/22/10 and 12/13/10 drained almost completely)
9 Fox Hill Lane (only 1 observation)

Figure 5-17 is a map showing these conditions for the Township. Most of the monitored dry wells were along a ridge between the two main drainages of the Township, with no obvious pattern of high water conditions, except that the high standing water dry wells were located along a line to the southwest along the ridge and are located fairly close to headwaters of streams (high water tables were noted in areas with nearby streams, but that was assumed to be in the larger stream valleys and not at the headwaters). The sites that had high standing water long after the events ended had substantially reduced infiltration rates. In the analyses, these rates were considered to be the constant (final) rates observed, with no initial rate data or first-order decay Horton coefficients used (relatively constant, but very low infiltration rates).

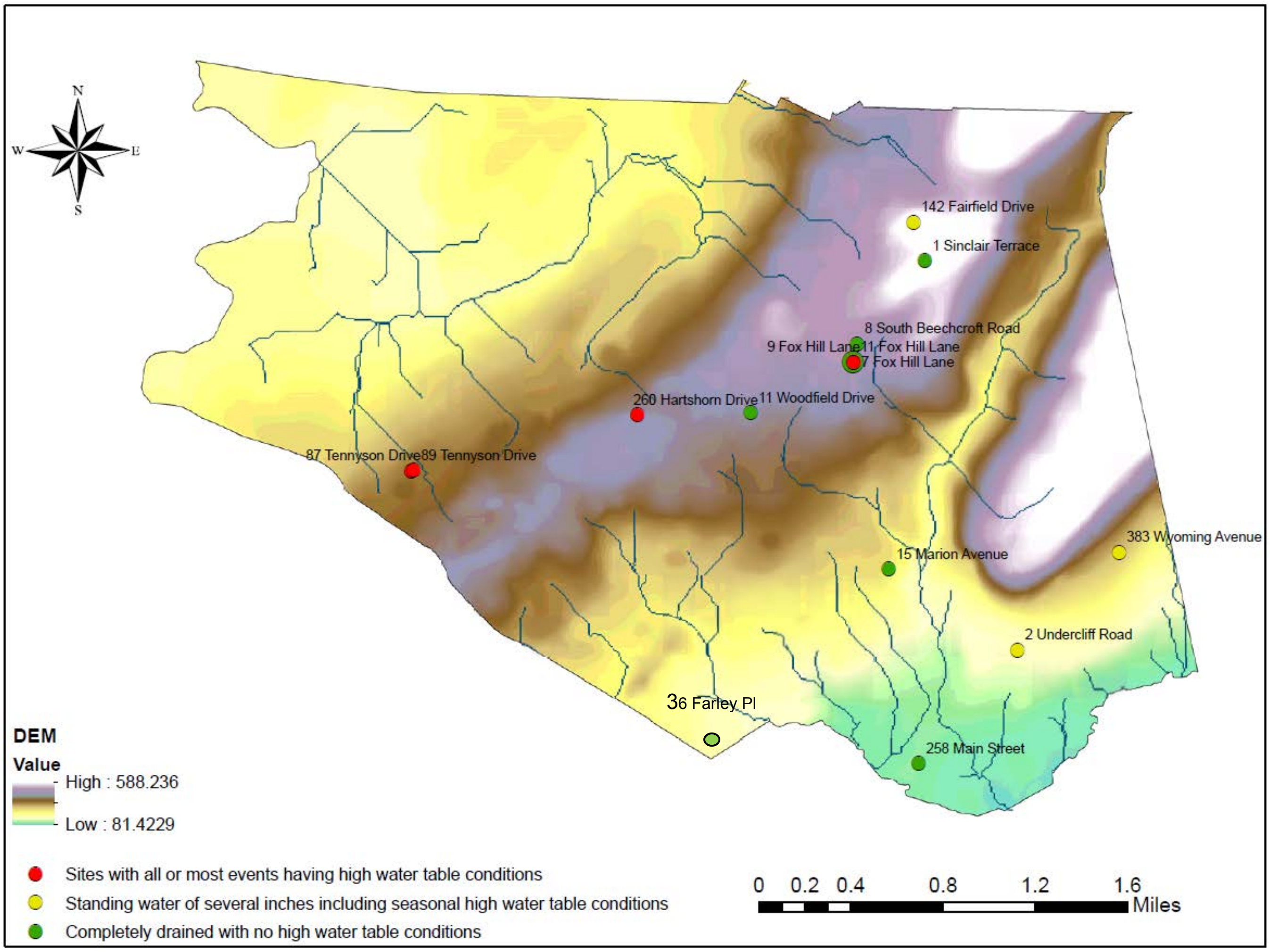


Figure 5-17. Township map showing locations having varying standing water conditions in monitored dry wells.

Another obvious factor affecting the observed infiltration rates was that one or two of the locations had significantly higher infiltration rates than the other sites (all having no standing water issues). These sites were the ones indicated as having the highest surface infiltration rate potentials (even though the infiltration rates of the dry wells were mostly affected by the subsurface soil conditions, which were mapped as being similar A and B conditions for all locations). It is therefore expected that these locations had better subsurface soil conditions compared to the other sites, even though mapped as being similar.

Therefore, the Township of Millburn infiltration rate characteristics were separated into three conditions:

- A and B surface soils and having well drained HSG A subsurface soils
- C and D surface soils and having well drained A and B subsurface soils

• C and D surface soils and having poorly drained A and B subsurface soils with long-term standing water

The infiltration rate conditions for these Township of Millburn situations are presented in Figures 5-18 through 5-20 and Tables 5-17 through 5-19.

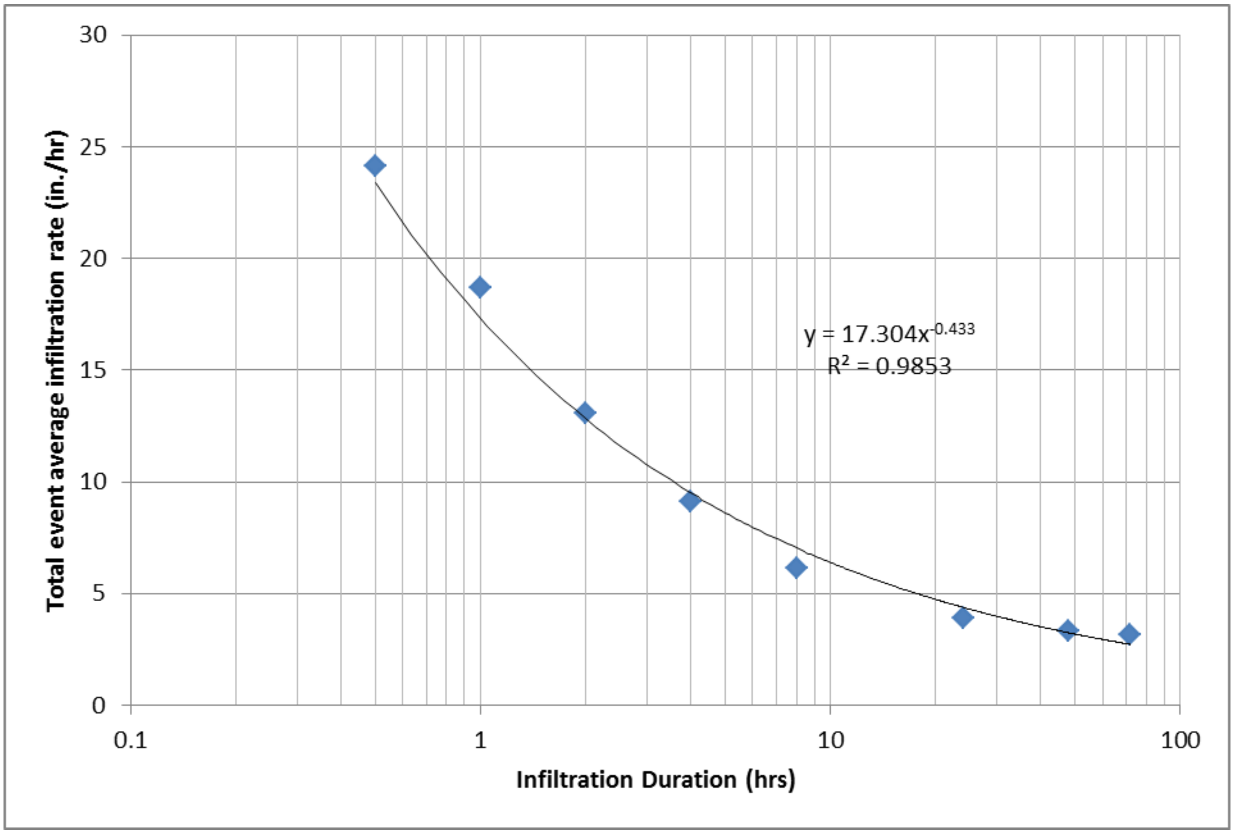


Figure 5-18. Infiltration rates averaged over event durations for A and B surface soils and well-drained A subsurface soils. (1 in./hr = 25.4 mm/hr)

Table 5-17. Infiltration Rates Averaged Over Event Durations for A and B Surface Soils and Well-Drained A Subsurface Soils

duration (hrs)	infiltration rate averaged over duration of event (in./hr)	COV
0.5	24.1	0.3
1	18.7	0.3
2	13.0	0.3
4	9.1	0.3
8	6.1	0.4
24	3.9	0.6
48	3.3	0.8
72	3.1	0.8

(1 in./hr = 25.4 mm/hr)

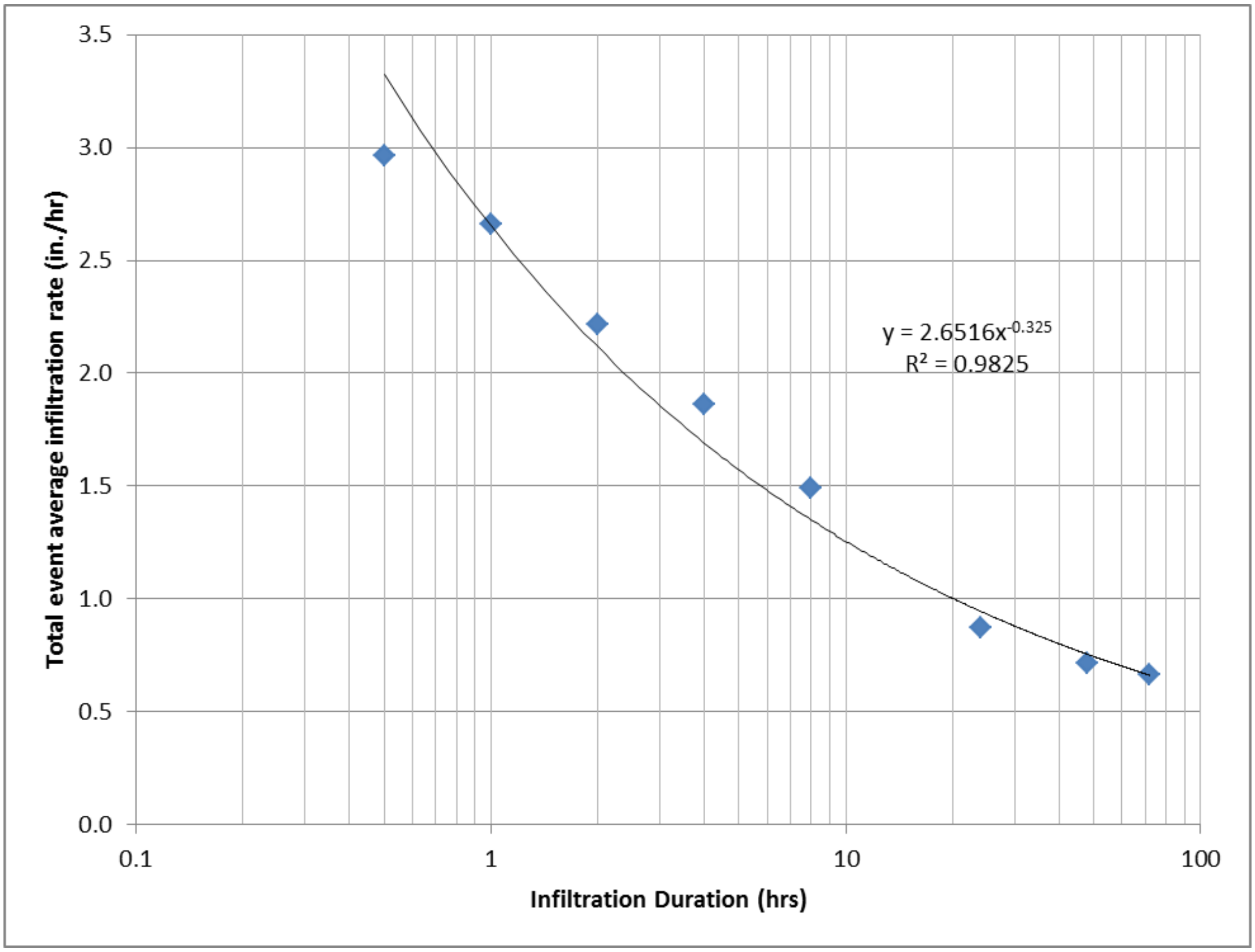


Figure 5-19. Infiltration rates averaged over event durations for C and D surface soils and well-drained A and B subsurface soils. (1 in./hr = 25.4 mm/hr)

Table 5-18. Infiltration Rates Averaged Over Event Durations for C and D Surface Soils and Well-Drained A and B Subsurface Soils

duration (hrs)	infiltration rate averaged over duration of event (in./hr)	COV
0.5	3.0	1.0
1	2.7	1.0
2	2.2	0.9
4	1.9	0.9
8	1.5	0.8
24	0.9	0.6
48	0.7	0.6
72	0.7	0.6

(1 in./hr = 25.4 mm/hr)

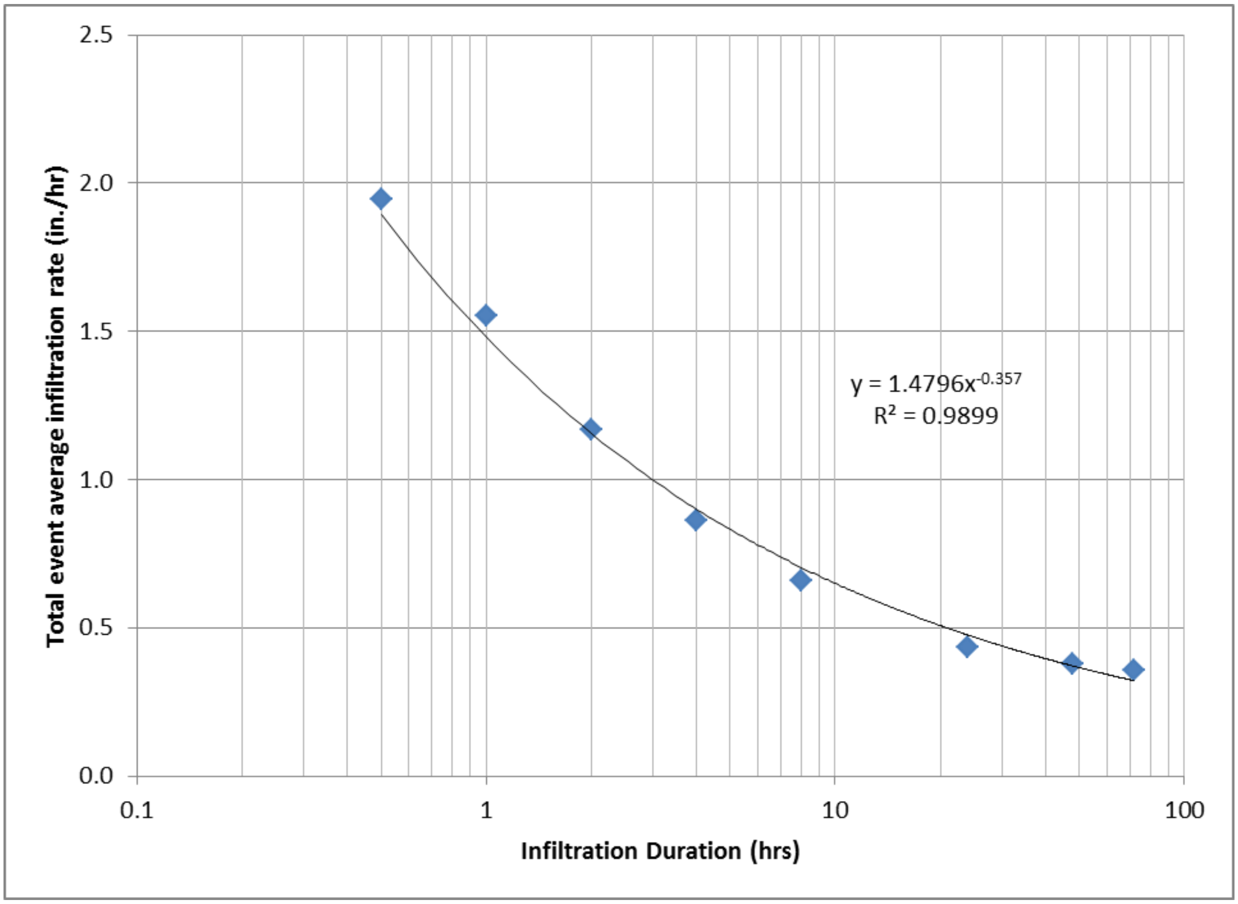


Figure 5-20. Infiltration rates averaged over event durations for C and D surface soils and poorly-drained A and B subsurface soils having extended standing water. (1 in./hr = 25.4 mm/hr)

Table 5-19. Infiltration Rates Averaged Over Event Durations for C and D Surface Soils and Poorly-Drained A and B Subsurface Soils Having Extended Standing Water

duration (hrs)	infiltration rate averaged over duration of event (in./hr)	COV
0.5	1.9	1.2
1	1.6	1.1
2	1.2	1.0
4	0.9	0.9
8	0.7	0.8
24	0.4	0.6
48	0.4	0.6
72	0.4	0.6

(1 in./hr = 25.4 mm/hr)

These figures and tables show that the over-all event infiltration rate decreases as the duration increases. For example, for a typical 5 hour rain period (having about a 6 hour runoff period), the event-averaged infiltration rate would be about 190 mm (7.6 in.) per hour for the best conditions having A or B surface soils and well-drained subsurface A and B soils. This reduces to about 43 mm (1.7 in.) per hour for the C or D surface soils having well-drained subsurface A and B soils. For the condition having standing water and poorly drained subsurface soils, the infiltration rates would be about 20 mm (0.8 in.) per hour. Complete drainage times for the best soil conditions for this event would be several hours, extending to about a day for the intermediate condition, and several days for the condition with standing water. Of course, for the situation having standing water, the "dry" well may never drain completely if the standing water was associated with a high water table. If the standing water observations were due to clogging from debris, the dry wells may eventually drain completely, if enough time occurs between rains.

The New Jersey stormwater regulations require the infiltration of excess water above natural conditions associated with development or land modifications (either maintaining the pre-development groundwater recharge or preventing excess surface runoff), for the 24-hr, 2-year storm, which is about 86 mm (3.4 in.) for Essex County. The dry well regulations describe the construction of the dry wells, the acceptable soil conditions (HSG A and B), groundwater conditions (at least 2 ft or 60 cm above seasonal water table), and source waters (roof runoff only). The minimum design infiltration rate for groundwater recharge is 5 mm/hr (0.2 in./hr) while it is 12 mm/hr (0.5 in./hr) for stormwater quality use. These design standards for total event infiltration rates would not be met only for conditions having standing water for very long event durations. The largest rain that had infiltration measurements during this study was about 74 mm (2.9 in.), close to the design storm value.

Observed Infiltration Coefficient Values Compared to Literature Values

Table 5-20 compares the observed Horton equation coefficients with values that have been reported in the literature. The standing water data are not shown on this table as most of the observations could not be successfully fitted to the Horton equation. The almost steady infiltration rates (but with substantial variation) were all very low for those conditions and likely represent the f_c conditions only and were therefore included in that parameter category.

Table 5-20. Observed and Reported Horton Equation Coefficients

	f_o (in./hr)	f_c (in./hr)	K (1/min)
Surface A and B soils well drained A subsurface soils (average and COV)	**44.6 (0.53)**	**5.6 (0.2)**	**0.06 (0.22)**
Surface C and D soils well drained A and B subsurface soils (average and COV)	**4.3 (0.64)**	**0.45 (0.85)**	**0.01 (0.63)**
UDFCD (2001) A soils (average)	5.0	1.0	0.04
UDFCD (2001) B soils (average)	4.5	0.6	0.11
UDFCD (2001) C and D soils (average)	3.0	0.5	0.11
Pitt, *et al.* (1999) Clayey, dry and non-compacted (median)	11	3	0.16
Pitt, *et al.* (1999) Clayey, other (median)	2	0.25	0.06
Pitt, *et al*. (1999) Sandy, compacted (median)	5	0.5	0.1
Pitt, *et al.* (1999) Sandy, non-compacted (median)	34	15	0.08
Akan (1993) Sandy soils with little to no vegetation	5		
Akan (1993) Dry loam soils with little to no vegetation	3		
Akan (1993) Dry clay soils with little to no vegetation	1		
Akan (1993) Moist sandy soils with little to no vegetation	1.7		
Akan (1993) Moist loam soils with little to no vegetation	1		
Akan (1993) Moist clay soils with little to no vegetation	0.3		

(1 in./hr = 25.4 mm/hr)

The very large observed f_o value (45 in./hr) for the A and B surface soil sites that are well drained is greater than any of the reported literature values, and only approaches the observations for the non-compacted sandy soil conditions (34 in./hr) observed by Pitt, *et al*. (1999). The subsurface soil conditions affecting the dry well infiltration rates are likely natural with little compaction. Also, the subsurface soils at that location are noted as being sandy loam (A) and stratified gravelly sand to sand to loamy sand (A). The other sites having smaller f_o rates (4.3 in./hr) are described as gravelly sandy loam (A) and fine sandy loam (B) and are similar to many of the reported literature values for sandy soils, with some compaction.

The largest f_c value (5.6 in./hr) observed for the well-drained A and B surface soil location is bracketed by the non-compacted clayey and sandy soil conditions (3 and 15 in./hr) reported by Pitt, *et al.* (1999), but is substantially larger than the other reported values. The f_c value observed for the well-drained C and D surface soil site (0.45 in./hr) is similar to the other reported values (0.5 to 1.0 in./hr). The k first-order rate values (0.01 and 0.06 1/min) are similar, but on the low side, of the reported values (0.04 to 0.11 1/min).

In order to most accurately design dry well installations in an area, actual site observations of the expected infiltration rates should be used instead of general literature values. This is especially true for surface infiltration devices (such as rain gardens), where compaction will have a much greater effect than on the deeper subsurface soils. Also, all of the sites in this study had improved infiltration characteristics with depth compared to expected surface conditions; in other cases, this may not be true. Criteria based only on surface soil conditions are likely not good predictors of deeper dry well performance. Luckily, county soil surveys do have some subsurface soil information that was found to be generally accurate during this study. Unfortunately, shallow water table conditions are not well known for the area and that characteristic can have a significant detrimental effect on the observed dry well performance.

Chapter 6 Dry Well Disposal Water Quality Observed in Millburn, NJ

During construction of three new dry wells, shallow and deep monitoring well underdrains were installed to collect percolating water for analyses during ten rain events. The water quality of these samples were compared to water quality criteria to identify potential groundwater contamination issues associated with dry well use in these typical installations and also to identify differences in the water quality as it passed through the dry well gravel layer and at least two feet of underlying soil.

Sampling Locations

Three dry wells in Millburn were instrumented with monitoring well underdrains for sampling, as shown in Figure 6-1. The shallow monitoring well underdrain was constructed directly below the dry well near the surface of the gravel layer and a deeper one was installed at least 0.6 m (2 ft) below the bottom of the gravel layer (the NJ state requirement for closest groundwater). Therefore, the deep monitoring location was at least 1.2 m (4 ft) below the bottom of the dry well. Water samples were manually pumped from these monitoring well underdrains during or immediately after the rains and analyzed for a range of typical stormwater pollutants.

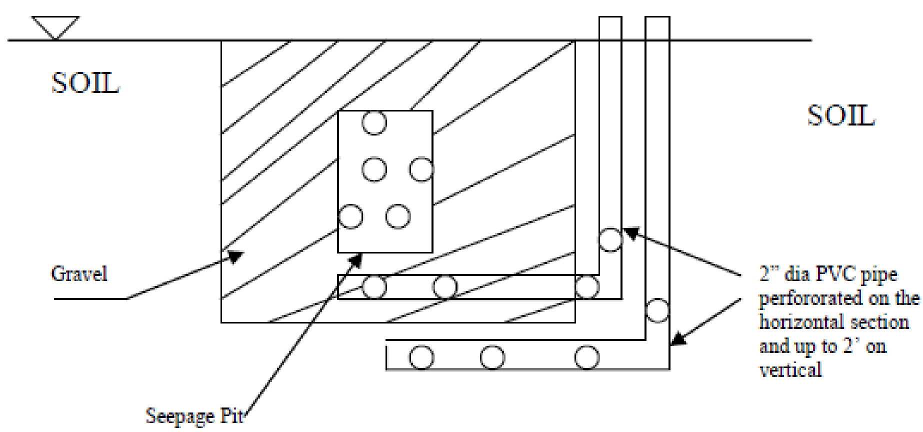


Figure 6-1. PVC Pipe arrangement in dry wells

Table 6-1 lists the addresses of the sampled dry wells in Millburn. Water samples were collected during or after ten storm events from these dry wells involving a total of two samples for each location; one from directly below the dry well ("shallow") and the other from the deeper monitoring well underdrain ("deep"). Samples were also collected from the inlet and outlet locations at an underground water storage cistern at 79 Minnisink Road, Millburn, NJ. Figure 6-2 shows the locations of these sampling locations.

Table 6-1. Water Quality Monitoring Locations

Dry Well and Cistern Sampling Site Locations
135 Tennyson Road, Millburn, NJ, 07078 (dry well location)
18 Slope Drive, Millburn, NJ, 07078 (dry well location)
139 Parsonage Hill Road, Millburn, N,J 07078 (dry well location)
79 Minnisink Road, Millburn, NJ, 07078 (cistern location)

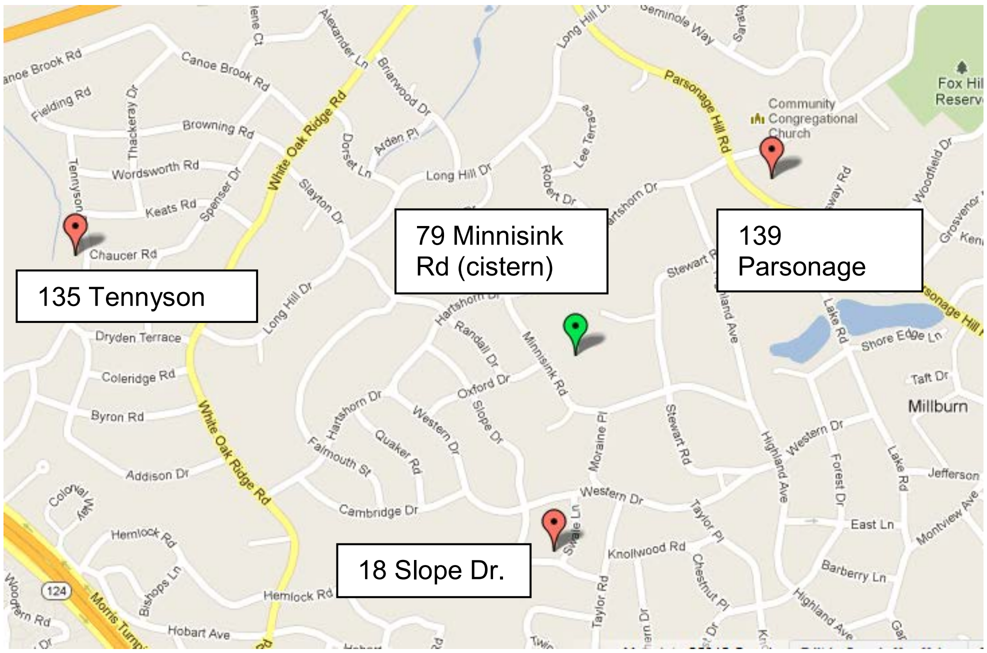


Figure 6-2. Locations of Water Quality Sampling Sites in Millburn, NJ.

Table 6-2 lists the rain depths for the ten monitored rains. A storm is defined as a rain event producing at least 2.5 mm (0.1 in.) of total rainfall and produces sufficient flow for collection of samples for analysis. A rain free period of 3 hr was used to define separate rain events. A water volume of 2 L was collected during each sampling event at each sampling location.

Table 6-2. Rain Depths for Monitored Events

Date	Rain Depth
10/20/2010	0.10 in.*
7/29/2011	0.15 in.*
8/5/2011	0.14 in.*
08/10/2011	0.12 in.*
08/16/2011	0.15 in.
08/17/2011	0.20 in.
08/18/2011	0.10 in.
08/22/2011	0.50 in.
08/25/2011	0.25 in.
08/28/2011**	9 in.

*The data from these rains was obtained from http://www.wunderground.com/ while the other rains were obtained from on-site rain gages.
**Hurricane Irene rain began about 3:00 pm on 08/27/2011 and finished at about 10:00 am on 08/28/2011, producing record rainfall for the area.
(1 in. = 25.4 mm)

Sampling Procedure

The samples obtained after each event were packaged, cooled, and transported by PARS personnel to the University of Alabama laboratory for analyses. The samples were preserved and stored in a sample storage refrigerator according to requirements described in Table 6-3 upon arrival to the laboratory until analysis. HDPE containers were used for all samples and subsamples received in the laboratory. Each water sample was analyzed for bacteria (total coliform and *E. coli*), total nitrogen, total phosphorus, and chemical oxygen demand using the standard procedures listed in Table 6-3. Also, each analysis for each sample was duplicated.

For bacteria analyses (using IDEXX QuantiTray methods) all analyses were conducted within 24 hr of sampling. This delay is longer than the desired 6 hrs holding time for bacteria analyses but were used to indicate approximate bacteria levels; based on prior storage tests, the observed bacteria levels were likely about half the levels that may be expected with fresher samples. Therefore, if the results were greater than the criteria after 24 hr holding times, they are very likely also greater than if the samples were analyzed within the shorter holding time. If the results were very low, they would probably also be low if analyzed earlier. Samples for metal analyses were all preserved by acidification to a pH less than 2 using ultra-pure HNO_3 and then stored at 4°C before analyses at the outside laboratory (Stillbrook Environmental Laboratories, Fairfield, AL). For all the other parameters, sample preservation was according to recommended methods as shown on Table 6-3.

Table 6-3. Summary Table of Standard Methods, Procedures, and Quality Assurance

Parameter	Analytical Method Number and Name	Preservation Method/Maximum Holding Time	Sample Volume and Sample Container	Processing Summary	Detection Limit	Accuracy (%)	Precision (%)	Completeness (%)
Total nitrogen (as N)	Persulfate digestion method (HACH 10071)	Cool 4^{o}C/48hrs or preservation with H_2SO_4 to a pH of <2 required and cool 4C; holding time 28 days	100 mL HDPE, acid washed with HCl	Analyses performed on whole sample	1 mg/L	80-120	<10	80
Nitrate plus nitrite (as N)	SM 4500-NO_3-D&E Cd reduction (HACH 8039)	Cool 4^{o}C/48hr or preservation with H_2SO_4 to a pH of <2 required if samples will not be analyzed within a 48 holding time	100 mL HDPE, acid washed with HCl	Analyses performed on whole sample	0.002 mg/L**	80-120	<10	80
Total phosphorus (as P)	SM 4500-P-E Colorimetric, Ascorbic Acid, Single Reagent, EPA 365.2 (HACH 8190)	Cool 4^{o}C/48hrs or preservation with H_2SO_4 to a pH of <2 required and cool 4C; holding time 28 days	100 mL glass, rinsed with HCl, then distilled water	Analyses performed on whole sample	0.020 mg/L	80-120	<10	80
COD (as C)	Mercury free dichromate digestion (HACH 800)	Cool 4^{o}C/48hr or preservation with H_2SO_4 to a pH of <2 required if samples will not be analyzed within a 48 holding time	100 mL HDPE, acid washed with HCl	80-120	<10	80	<10	80
Pesticides	SW846-8081A	Cool 4^{o}C/7days	2L sample volume, amber glass with Teflon liner, pre-cleaned		0.5 µg/kg	80-120	<10	80
Herbicides	SW846-8081A	Cool 4^{o}C/7days	2L sample volume, amber glass with Teflon liner, pre-cleaned		0.5 µg/kg	80-120	<10	80
Heavy Metals (Cu, Pb, Zn)	ICP	Acidification to pH 1-2 (ultra-pure HNO_3)/6 months	100 mL HDPE		5 µg/L for Pb; 20 µg/L for Cu and Zn	85-115	<15	80

Results of Dry Well and Cistern Water Sample Analyses

All samples were analyzed in duplicate for each analytical run. The following discussion is a summary of the results for each measured parameter. Only samples from three of the locations were available for the first event, and cistern samples were not obtained during the second and fourth events. Therefore, seven to ten samples were available from each sampling location.

Bacteria

Tables 6-4 and 6-5 summarize the total coliform and *E. coli* levels. The upper detection limit (UDL) of this method is 2,419.2 MPN/100 mL and the lower detection limit (LDL) is 1 MPN/100 mL for both indicator organisms. After completion of the first two rounds of sampling, it was observed that most bacteria levels exceeded the UDL (even with the 24 hr maximum delay that was longer than the desired standard 6 hr holding time). Therefore, one of the samples per site was diluted 10 times to increase the UDL to 24,192 MPN/100 mL. For some samples, 20 times dilution was applied to increase the UDL to 48,384 MPN/100 mL. As can be seen from Table 6-4, wide ranges of bacteria levels were detected for all events at all locations. The geometric means for the total coliform results were 17 to 15,106 MPN/100 mL, while the geometric means for the *E coli* results (setting half of the detection limits for nd values) were 4 to 358 MPN/100 mL. The cistern related sample bacteria levels were generally lower than for the dry well samples. The last event listed in Tables 6-4 and 6-5 has the highest values of bacteria observed and occurred during the record 9 inch rainfall of Hurricane Irene. Total coliform levels were higher in the cistern than the inflow and generally the deep locations in each site had higher values of total coliform, possibly indicating some regrowth in the gravel layer.

Table 6-4. Summary of Sampling Results for Total Coliform Bacteria (MPN/100 mL)

Date	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2010		83	1,317	1,785				
7/29/2011			8,469	3,629	404	8,469	352	8,469
8/5/2011	10	34	1,909	18,147	199	18,147	352	364
8/10/2011			15,971	15,174	14,711	36,294	12,351	13,122
8/16/2011	36	2,024	36,294	36,294	36,294	31,961	36,294	31,961
8/17/2011	11	25,406	5,316	8,480	11,672	25,406	18,539	5,562
8/18/2011	11	1,483	12,409	11,672	25,406	18,539	14,207	9,374
8/22/2011	1	2,609	1,210	25,406	7,698	15,346	15,346	8,911
8/25/2011*	1	374	43	332	620	1,251	2,021	1,589
8/28/2011	9,374	11,672	25,406	25,406	18,539	25,406	18,539	25,406
Basic Statistics								
Number	7	8	10	10	9	9	9	9
Average	1,349	5,461	10,834	14,632	12,838	20,091	13,111	11,640
Min	1	34	43	332	199	1,251	352	364
Max	9,374	25,406	36,294	36,294	36,294	36,294	36,294	31,961
Median	11	1,753	6,892	13,423	11,672	18,539	14,207	8,911
Geometric Mean	17	1,119	4,010	8,199	4,703	15,106	5,881	6,615
St Dev	3,539	8,915	12,040	11,791	12,428	11,060	11,466	10,562
COV	2.6	1.6	1.1	0.8	1.0	0.6	0.9	0.9

* This sample was received in the laboratory two days after sampling and the measured bacteria levels are therefore likely lower than expected compared to fresh samples.

Table 6-5. Summary of Sampling Results for *E. coli* (MPN/100 mL)

Date	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2010		<22	<22	<22				
7/29/2011			6,752	268	<2	2	252	8,469
8/5/2011	3	3	471	550	10	15	252	162
8/10/2011			239	227	222	57	154	136
8/16/2011	<20	287	7,183	360	57	36	215	180
8/17/2011	<1	22	352	195	85	125	106	46
8/18/2011	<1	14	281	381	20	58	42	39
8/22/2011	<1	269	343	3,128	41	124	70	62
8/25/2011*	<1	4	6	2	2	25	6	24
8/28/2011	5,087	5,562	2,817	2,716	70	68	99	1,593
Basic Statistics								
Number	2	7	9	9	8	9	9	9
Average	2,545	880	2,049	870	63	57	133	1,190
Min	<1	3	6	2	2	2	6	24
Max	5,087	5,562	7,183	3,128	222	125	252	8,469
Median	<1	22	352	360	49	57	106	136
Geometric Mean	4	45	358	210	22	36	90	174
St Dev	3,595	2,068	2,914	1,178	70	44	91	2,775
COV	1.4	2.4	1.4	1.4	1.1	0.8	0.7	2.3

* This sample was received in the laboratory two days after sampling and the measured bacteria levels are therefore likely lower than expected compared to fresh samples.

Nutrients

Tables 6-6 to 6-8 summarize the observed concentrations for the nutrients total nitrogen, NO_3 plus NO_2, and total phosphorus. The total nitrogen as N varied from zero (ND, reported as zero by this test method) to 16.5 mg/L. The NO_3 plus NO_2 concentrations ranged between 0.2 to 3.2 mg/L. The total phosphorus concentrations ranged from 0.02 to 1.36 mg/L. The median values for most of the locations were about the same for shallow or deep samples and for inflow or cistern samples, except for one of the sites in which the deep samples had higher TN median values than for the shallow samples. As shown later with the statistical tests, there were no significant differences between the shallow and deep sample nutrient concentrations, based on the number of samples available.

Table 6-6. Summary of Sampling Results for Total Nitrogen as N (mg/L) (* Standard solution)

	Location									
Date	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep	1* mg/L	10* mg/L
10/20/2010		1	1	2						
7/29/2011			1.5	2.5	2	4	0.5	3.5	1.5	10.5
8/5/2011	6	2	0.5	1.5	0	3	1.5	1	0.5	8
8/10/2011			1	1	1	5.5	1.5	1.5	1	9.5
8/16/2011	1.5	1	1.5	1	1	1.5	0	1.5	0.5	8
8/17/2011	1	7	1.5	1.5	3.5	6.5	1.5	0.5	1	9.5
8/18/2011	1	1	1.5	4	2	1	4.5	1.5	0.5	9
8/22/2011	0.5	0.5	2	1.5	2.5	1.5	1.5	6	0.5	9
8/25/2011	1.5	3.5	2	1.5	16.5	3.5	6.5	3.5	1	10
8/28/2011	3	3	3.5	2	3	2	1.5	2	1	10
Basic Statistics										
Number	7	8	10	10	9	9	9	9	9	9
Average	2.1	2.4	1.6	1.9	3.5	3.2	2.1	2.3	0.8	9.3
Min	0.5	0.5	0.5	1.0	0.0	1.0	0.0	0.5	0.5	8.0
Max	6.0	7.0	3.5	4.0	16.5	6.5	6.5	6.0	1.5	10.5
Median	1.5	1.5	1.5	1.5	2.0	3.0	1.5	1.5	1.0	9.5
St Dev	1.9	2.2	0.8	0.9	5.0	1.9	2.1	1.7	0.4	0.9
COV	0.9	0.9	0.5	0.5	1.4	0.6	1.0	0.7	0.4	0.1

* These are QA samples at 1 and 10 mg/L as N.

Table 6-7. Summary of Sampling Results for NO_3 plus NO_2 as N (mg/L)

	Location							
Date	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2010		0.9	0.8	1.1				
7/29/2011			0.9	0.8	0.9	1.4	0.8	1.1
8/5/2011	2.5	0.8	0.8	0.5	0.8	4.7	1	2
8/10/2011			1.65	0.85	1.7	2.05	1.95	0.5
8/16/2011	3.15	1.9	1.5	0.45	0.8	0.8	1.1	3.2
8/17/2011	Error*	0.55	1.1	0.65	1.55	Error	Error	0.25
8/18/2011	0.8	0.45	0.6	0.45	0.5	0.7	0.3	0.65
8/22/2011	0.75	0.3	0.6	0.6	0.7	0.75	0.45	0.7
8/25/2011	0.8	0.4	0.1	0.3	0.6	1.2	0.6	0.4
8/28/2011	0.6	0.7	0.4	0.4	0.2	1	0.2	0.5
Basic Statistics								
Number	6	8	10	10	9	8	8	9
Average	1.43	0.75	0.85	0.61	0.86	1.58	0.80	1.03
Min	0.60	0.30	0.10	0.30	0.20	0.70	0.20	0.25
Max	3.15	1.90	1.65	1.10	1.70	4.70	1.95	3.20
Median	0.8	0.6	0.8	0.6	0.8	1.1	0.7	0.7
St Dev	1.1	0.5	0.5	0.2	0.5	1.3	0.6	1.0
COV	0.8	0.7	0.6	0.4	0.6	0.8	0.7	0.9

* The error occurred because the samples were muddy that interfered with the test results.

Table 6-8. Summary of Sampling Results for Total Phosphorus as P (mg/L)

Data										
	Location									
Date	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep	1* mg/L	3* mg/L
10/20/2010		0.05	0.235	0.15						
7/29/2011			0.04	0.05	0.09	0.35	0.095	0.42	0.335	0.985
8/5/2011	0.39	0.49	0.085	0.075	0.17	1.1	0.125	0.43	0.33	0.975
8/10/2011			0.215	0.085	0.67	1.36	0.465	0.125	0.335	0.98
8/16/2011	0.06	0.07	0.225	0.095	0.12	0.105	0.165	0.135	0.34	0.99
8/17/2011	0.02	0.14	0.14	0.115	0.085	0.145	0.135	0.085	0.335	0.975
8/18/2011	0.08	0.05	0.05	0.19	0.2	0.2	0.12	0.1	0.33	0.975
8/22/2011	0.035	0.04	0.045	0.08	0.12	0.205	0.125	0.075	0.335	0.975
8/25/2011	0.19	0.52	0.05	0.08	0.14	0.16	0.13	0.11	0.325	0.98
8/28/2011	0.18	0.12	0.11	0.07	0.28	0.33	0.22	0.11	0.325	0.98
Basic Statistics										
Number	7	8	10	10	9	9	9	9	9	9
Average	0.14	0.19	0.12	0.10	0.21	0.44	0.18	0.18	0.33	0.98
Min	0.02	0.04	0.04	0.05	0.09	0.11	0.10	0.08	0.33	0.98
Max	0.39	0.52	0.24	0.19	0.67	1.36	0.47	0.43	0.34	0.99
Median	0.08	0.10	0.10	0.08	0.14	0.21	0.13	0.11	0.34	0.98
St Dev	0.13	0.20	0.08	0.04	0.18	0.46	0.11	0.14	0.01	0.01
COV	1.0	1.1	0.7	0.4	0.9	1.0	0.7	0.8	0.0	0.0

* Standard solution (P as PO_4, multiply by 3.1 for standard solution concentrations as P, like the other data)

Chemical Oxygen Demand (COD)

The COD concentration in different locations and for various storm events ranged from 5.0 to 148 mg/L. Also, as shown later, the statistical analyses did not indicate any significant differences between the shallow and deep samples for any location (or inflow or cistern samples), for the number of samples available.

Table 6-9. Summary of Sampling Results for COD (mg/L)

Date	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2010		11.3	23.7	34.5				
7/29/2011			39	22	19	9	34	55.5
8/5/2011	50.5	40.5	38	31.5	20	52	50	51
8/10/2011			46.5	19	55	148	39.5	51
8/16/2011	30	17	73	21.5	51	29.5	49	33
8/17/2011	19.5	10	49	50	62.5	131	45.5	27
8/18/2011	17	5	38.5	55	29.5	36.5	44	48
8/22/2011	26	9	25	39.5	19.5	24	36	35.5
8/25/2011	51	29	33.5	24.5	55	75	50.5	41.5
8/28/2011	22.5	15	40	20.5	18.5	25	36	45.5
Basic Statistics								
Number	7	8	10	10	9	9	9	9
Average	30.9	17.1	40.6	31.8	36.7	58.8	42.7	43.1
Min	17.0	5.0	23.7	19.0	18.5	9.0	34.0	27.0
Max	51.0	40.5	73.0	55.0	62.5	147.5	50.5	55.5
Median	26.0	13.2	38.8	28.0	29.5	36.5	44.0	45.5
St Dev	14.2	11.9	13.9	12.8	18.7	49.5	6.5	9.6
COV	0.5	0.7	0.3	0.4	0.5	0.8	0.2	0.2

Metals

Total forms of lead, copper and zinc were analyzed for each sample. The detection limits for lead, copper, and zinc were 0.005 mg/L, 0.02 mg/L and 0.02 mg/L, respectively. There were many below detection limit (BDL) values in the results. The maximum observed concentration for lead was 0.38 mg/L which occurred in a deep sample under a dry well. The maximum concentration of copper was 1.1 mg/L which occurred in a cistern influent sample. The concentrations of zinc in all samples ranged from BDL to 0.14 mg/L for the different storm events. The statistical analyses did not detect any significant differences between any of the paired heavy metal values, based on the number of samples available.

Table 6-10. Summary of Sampling Results for Lead (mg/L)

Date	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2011		BDL	0.013	0.015				
7/29/2011			BDL	BDL	0.031	0.058	0.009	0.381
8/5/2011	BDL	BDL	BDL	BDL	0.009	0.038	BDL	BDL
8/10/2011			0.014	0.012	0.186	0.282	0.012	BDL
8/16/2011	BDL	BDL	BDL	BDL	0.011	0.038	0.006	BDL
8/17/2011	0.005	0.089	BDL	0.027	0.314	0.291	0.013	BDL
8/18/2011	0.007	BDL	BDL	0.028	0.031	0.06	BDL	BDL
8/22/2011	BDL	BDL	BDL	BDL	0.025	0.029	BDL	BDL
8/25/2011	0.007	0.005	BDL	BDL	0.025	0.022	BDL	BDL
8/28/2011	BDL	0.007	BDL	BDL	0.01	0.011	BDL	BDL
Basic Statistics								
Number	3	3	2	4	9	9	4	1
Average	0.0063	0.034	0.014	0.021	0.071	0.092	0.01	0.38
Min	BDL	BDL	BDL	BDL	0.009	0.011	BDL	BDL
Max	0.007	0.089	0.014	0.028	0.31	0.29	0.013	0.38
Median	BDL	BDL	BDL	BDL	0.025	0.038	BDL	BDL
St Dev	0.0011	0.048	0.00070	0.0081	0.11	0.11	0.0032	NA
COV	0.18	1.4	0.052	0.40	1.57	1.2	0.32	NA

Note: Detection Limit = 0.005 mg/L

Table 6-11. Summary of Sampling Results for Copper (mg/L)

Data	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2011		0.22	BDL	BDL				
7/29/2011			BDL	BDL	0.02	BDL	BDL	0.1
8/5/2011	0.86	0.05	BDL	BDL	BDL	BDL	BDL	BDL
8/10/2011			BDL	BDL	0.03	0.06	BDL	BDL
8/16/2011	0.82	0.12	BDL	BDL	BDL	BDL	BDL	BDL
8/17/2011	0.61	1.13	BDL	BDL	0.04	0.05	BDL	BDL
8/18/2011	0.61	0.06	BDL	BDL	BDL	BDL	BDL	BDL
8/22/2011	0.51	0.16	BDL	BDL	BDL	BDL	BDL	BDL
8/25/2011	1.05	0.21	BDL	BDL	BDL	BDL	BDL	BDL
8/28/2011	0.22	0.13	BDL	BDL	BDL	BDL	BDL	BDL
Basic Statistics								
Number	7	8	10	10	3	2	10	1
Average	0.67	0.26	NA	NA	0.03	0.055	NA	0.1
Min	0.22	0.05	NA	NA	BDL	BDL	NA	BDL
Max	1.05	1.13	NA	NA	0.04	0.06	NA	0.1
Median	0.61	0.14	NA	NA	BDL	BDL	NA	BDL
St Dev	0.27	0.36	NA	NA	0.01	0.0070	NA	NA
COV	0.40	1.4	NA	NA	0.33	0.13	NA	NA

Note: Detection Limit = 0.02 mg/L

Table 6-12. Summary of Sampling Results for Zinc (mg/L)

Date	Location							
	79 Inflow	79 Cistern	135 Shallow	135 Deep	18 Shallow	18 Deep	139 Shallow	139 Deep
10/20/2011		0.08	0.02	0.03				
7/29/2011			0.14	0.04	BDL*	0.03	0.02	0.11
8/5/2011	0.14	0.03	0.04	0.07	BDL	BDL	BDL	BDL
8/10/2011			0.05	0.05	0.04	0.04	BDL	BDL
8/16/2011	0.13	0.02	0.06	0.12	BDL	BDL	0.04	0.02
8/17/2011	0.1	0.13	BDL	0.06	0.05	0.05	0.02	BDL
8/18/2011	0.08	0.02	BDL	0.03	BDL	BDL	BDL	BDL
8/22/2011	0.06	0.03	BDL	BDL	BDL	BDL	BDL	BDL
8/25/2011	0.13	0.04	BDL	BDL	BDL	BDL	BDL	BDL
8/28/2011	BDL	0.02	BDL	BDL	BDL	BDL	BDL	BDL
Basic Statistics								
Number	6	8	5	7	2	3	3	2
Average	0.11	0.046	0.062	0.057	0.045	0.04	0.027	0.065
Min	BDL	0.02	BDL	0.03	BDL	BDL	BDL	BDL
Max	0.14	0.13	0.14	0.12	0.05	0.05	0.04	0.11
Median	0.12	0.03	0.05	0.05	BDL	BDL	BDL	BDL
St Dev	0.032	0.039	0.046	0.031	0.0070	0.01	0.012	0.064
COV	0.30	0.85	0.74	0.55	0.16	0.25	0.43	0.98

Note: Detection Limit = 0.02 mg/L
* BDL = Below Detection Limit

Herbicides and Pesticides

Table 6-13 shows the results for the pesticide and herbicide analyses for three samples obtained during the October 20 2011 event. Only these three samples were analyzed for these analytes due to their costs. None of these constituents were found in the cistern sample, while alpha and gamma chlordane, endosulfan-I, and heptachlor epoxide were found in both of the two dry well samples.

Table 6-13. Summary of Sampling Results for Herbicides and Pesticides

		10/20/2010		
		135 Shallow	135 Deep	Cistern
Herbicides (µg/L)	2,4-D	ND*	ND	ND
	2,4,5-TP (Silvex)	ND	ND	ND
	2,4,5-T	ND	ND	ND
Pesticides (µg/L)	Aldrin	ND	ND	ND
	Alpha-BHC	ND	ND	ND
	beta-BHC	ND	ND	ND
	delta-BHC	ND	ND	ND
	gamma-BHC (Lindane)	ND	ND	ND
	alpha-Chlordane	0.03	0.03	ND
	gamma-Chlordane	0.02	0.024	ND
	Dieldrin	ND	ND	ND
	4,4′-DDD	ND	ND	ND
	4,4′-DDE	ND	ND	ND
	4,4′-DDT	ND	ND	ND
	Endrin	ND	ND	ND
	Endosulfan sulfate	ND	ND	ND
	Endrin aldehyde	ND	ND	ND
	Endrin ketone	ND	ND	ND
	Endosulfan-I	0.032	0.034	ND
	Endosulfan-II	ND	ND	ND
	Heptachlor	ND	ND	ND
	Heptachlor epoxide	0.03	0.035	ND
	Methoxychlor	ND	ND	ND
	Toxaphene	ND	ND	ND

* ND = Not Detected (below detection limits)

Statistical Analyses and Discussion

A number of complementary statistical analyses of the water quality data were conducted using MINITAB and MS-Excel software:

- Group box plots. These plots compare the ranges of observed concentrations between different sampling locations. These plots enable a rapid, visual comparison of data from different sampling locations. If a median value (the central line in the box) is above or below an adjacent box (the 25th and 75th percentiles), the data sets are likely significantly different. These plots also indicate the data symmetry and the presence of unusually high or low concentration values. In many cases, the concentration axis is plotted with a log scale. These plots can include data from many locations on the same figure for easy overall comparisons.

• Paired line plots. These exploratory data analysis plots compare paired data from single sites. The paired concentrations are connected with lines so trends between the two locations can be readily seen. The goal is to have parallel lines, or converging lines, indicating consistent differences. If many of the connecting lines cross with no pattern being obvious, then the sets of data are likely not correlated.

• Time series plots. These are also exploratory data analyses plots and indicate if concentrations vary with time. These are most appropriate for relatively long time periods of data observations, or to observe repeating trends (such as seasonal or other time series variations).

• Log-normal probability plots. These plots show a single (or few) set(s) of data on a single plot. The data is ranked and scored for probability. The probability axis has a distorted, but symmetrical, scale that results in a straight line of the concentration values if the data are normally distributed. The concentration scale can be plotted with a log scale to indicate log-normal probability. Two sets of data plotted on the same figure, especially with a fitted best fit line with confidence limits, are easy to compare to indicate if they are from the same population.

• Anderson-Darling (AD) p test for normality. This statistical test complements the probability plots by indicating if the data are significantly different from the fitted normal distribution. If the calculated AD p test statistic is smaller than 0.05, the data are significantly different from a normal distribution. If the p-value is larger than 0.05, insufficient data are available to indicate they are different and the observed data are usually assumed to be normally distributed (especially if the p-value is relatively large).

• Mann-Whitney comparison tests. If the data are not normally distributed, then nonparametric statistical tests are needed, compared to the more commonly used parametric tests. In this data evaluation, it was desired to compare paired sets of observations (incoming water vs. cistern water; shallow vs. deep observation well underdrain water). The Mann-Whitney test is a nonparametric test for paired data (simultaneous observations from both sampling locations) that considers the actual observation values (and not just relative values as in the less powerful Sign Test).

• Paired Sign Test (metals only). This is the simplest nonparametric paired sample comparison test that can be used if there are many non-detectable observations, as long as the other observation of the pair is detectable. This test compares the relatively magnitude of the values only and not the specific values. For these data, this test was used to examine the heavy metal data as many of those observations were non-detectable and the Mann-Whitney test could therefore not be used with these data.

The following sections discuss these analyses and provide some of the output examples, with the remaining output information presented in Appendix C.

Group Box Plots

Figures 6-3 to 6-8 shows the group box plots for each measured parameter including bacteria, nutrients, and COD. Too many of the heavy metals results were not detected and could not be effectively plotted. These plots show the data for all of the sites and (non-metal) constituents. There are no apparent visual trends between any of the paired data.

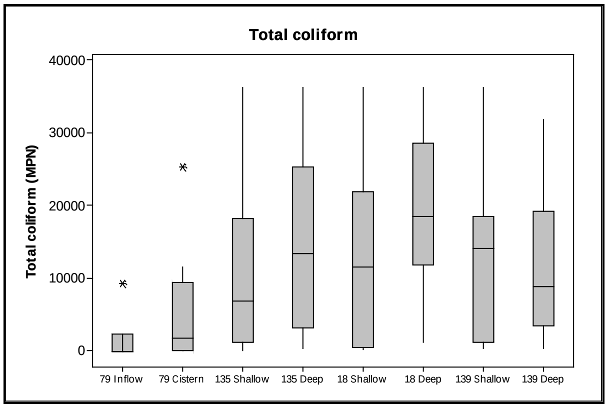


Figure 6-3. Group Box Plot for Total coliform

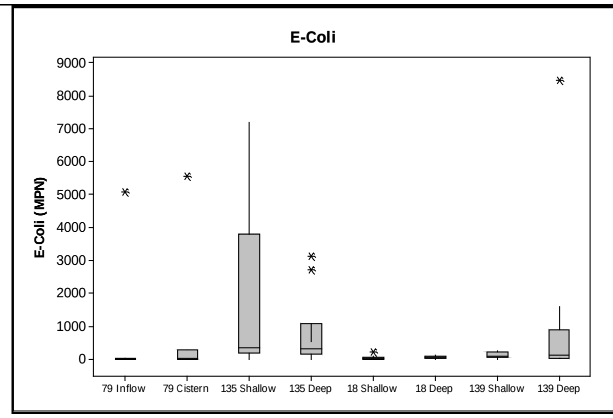


Figure 6-4. Group Box Plot for E. coli

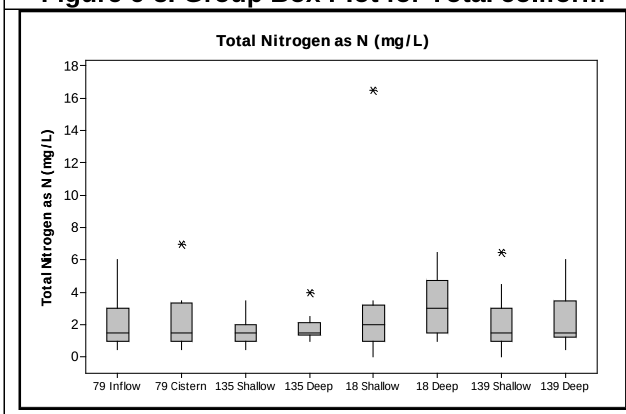


Figure 6-5. Group Box Plot for Total Nitrogen as N

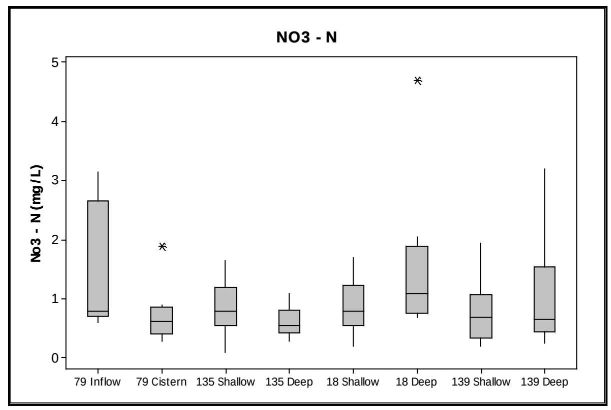


Figure 6-6. Group Box Plot for NO3 and NO2 as N

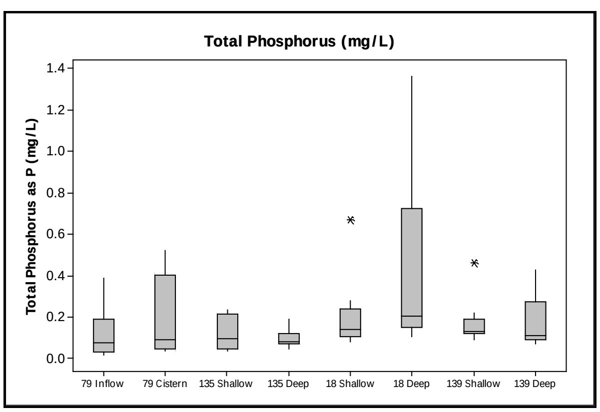


Figure 6-7. Group Box Plot for Total Phosphorus as N

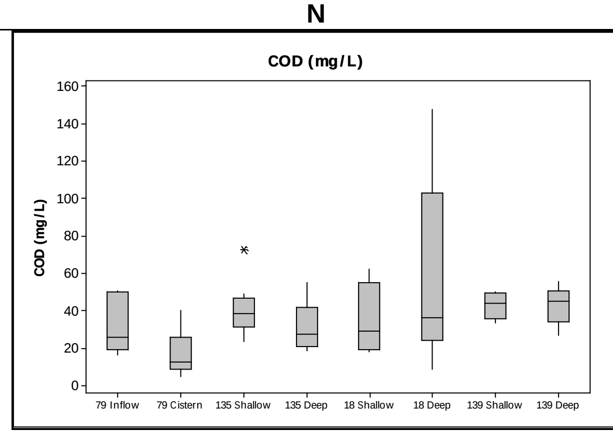
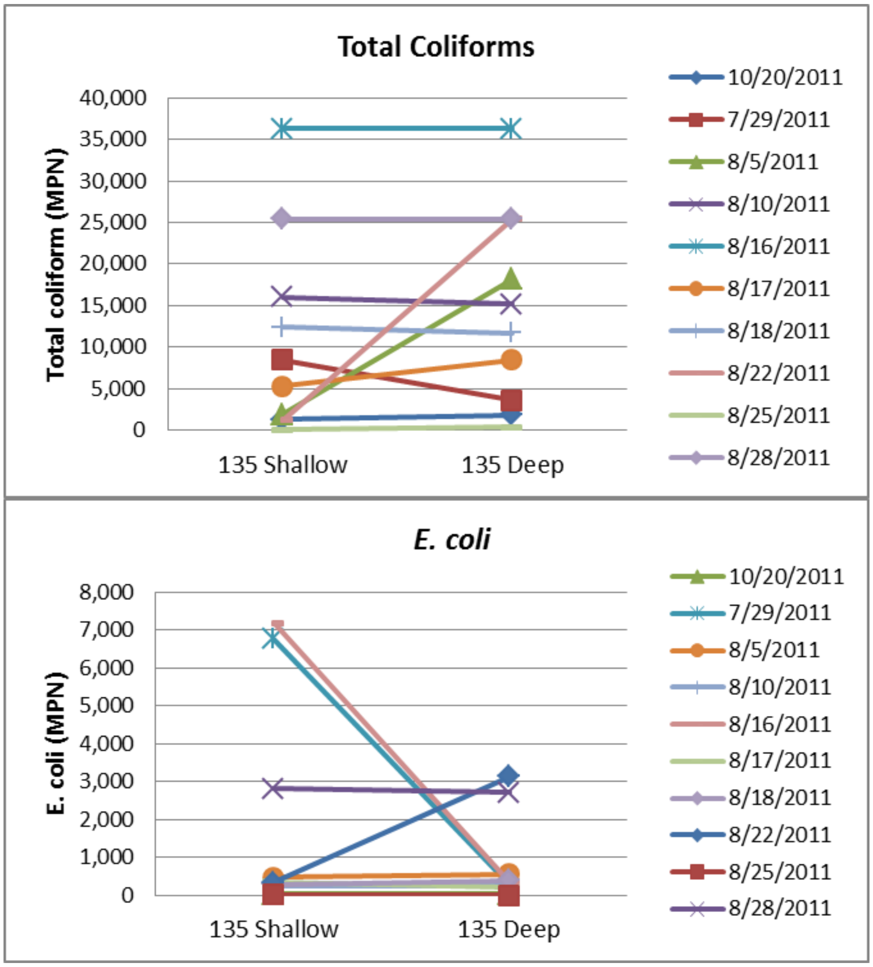


Figure 6-8. Group Box Plot for COD

Paired Line Plots

Figure 6-9 shows the paired line graphs for shallow vs. deep sampling locations for one of the dry well sites (135 Tennyson Road, Millburn, NJ 07078). The remaining sets of plots are shown in Appendix C. As shown on Figure 6-9, the concentration values vary with no consistent pattern: in some cases shallow samples may have higher bacteria levels or nutrient levels as well as COD levels, while during other storms, the deep samples may experience higher values.



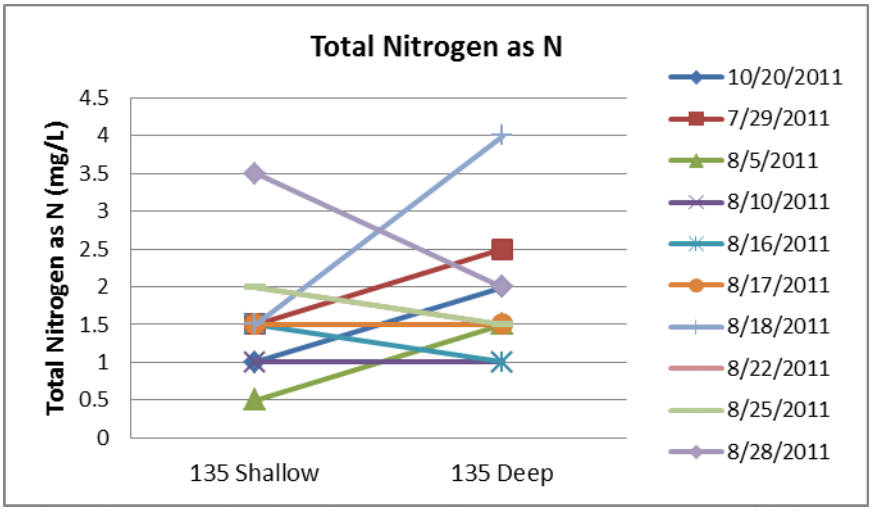
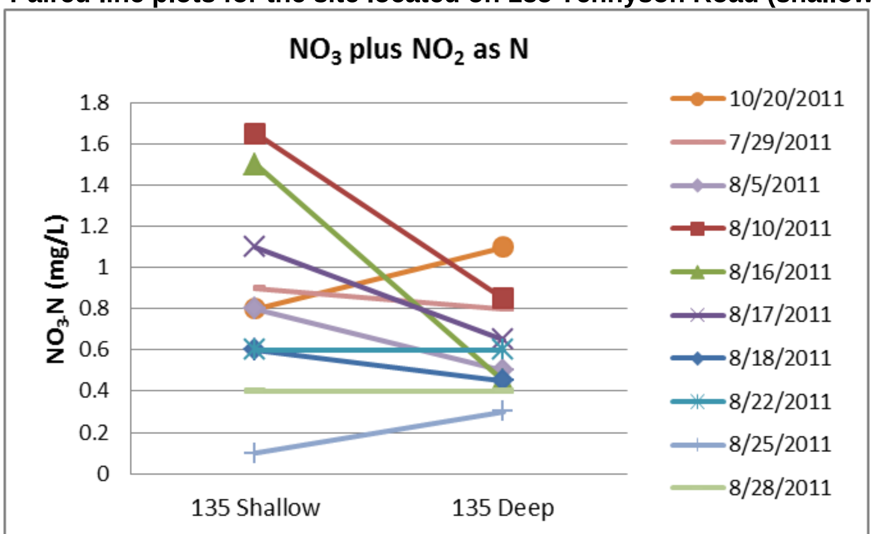
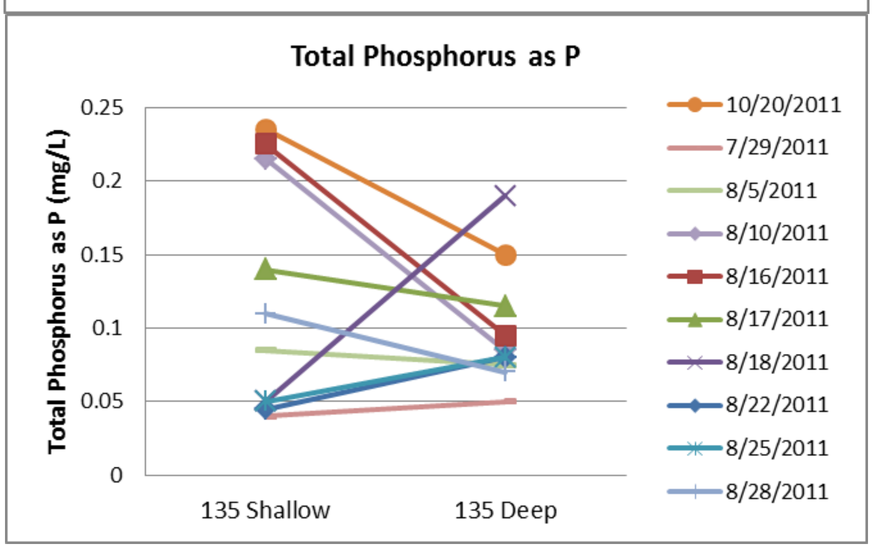


Figure 6-9 Paired line plots for the site located on 135 Tennyson Road (shallow vs. deep)





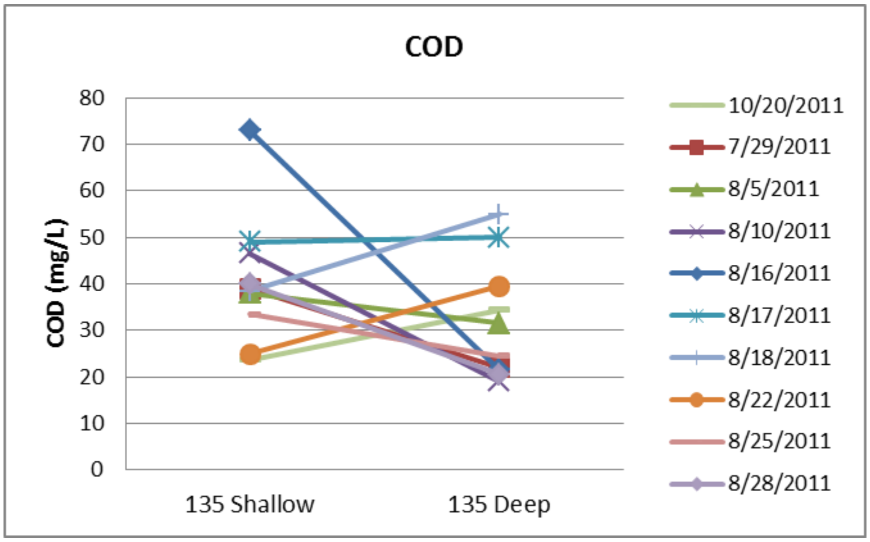
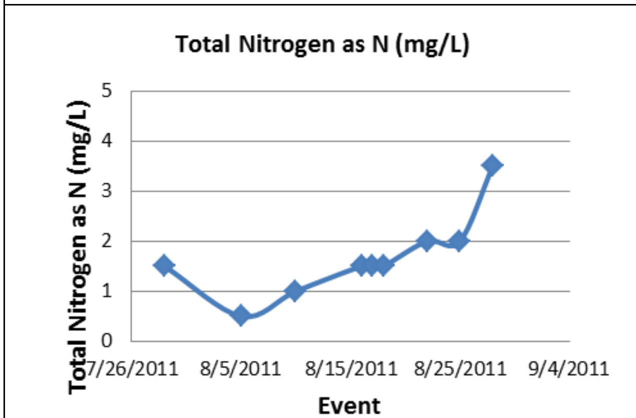
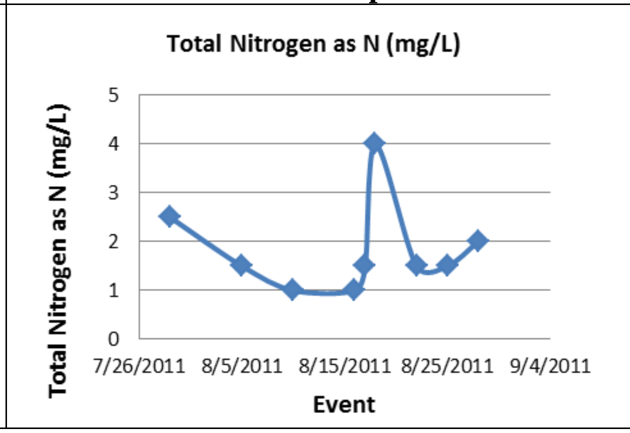
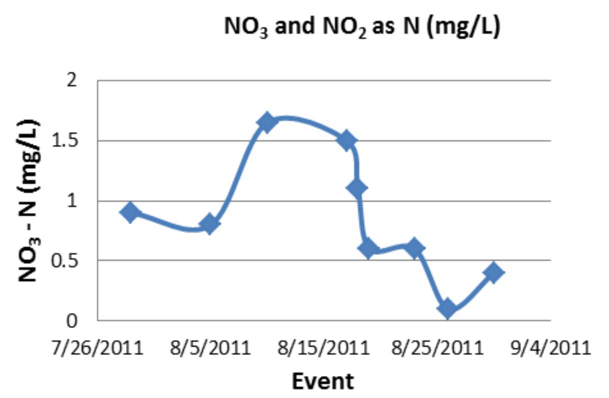


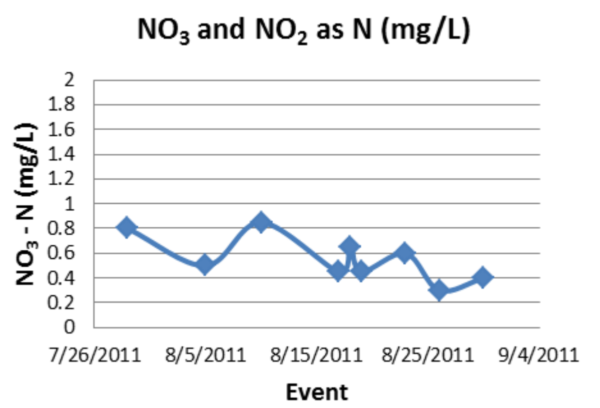
Figure 6-9. Paired line plots for the site located on 135 Tennyson Road (shallow vs. deep) (continued)

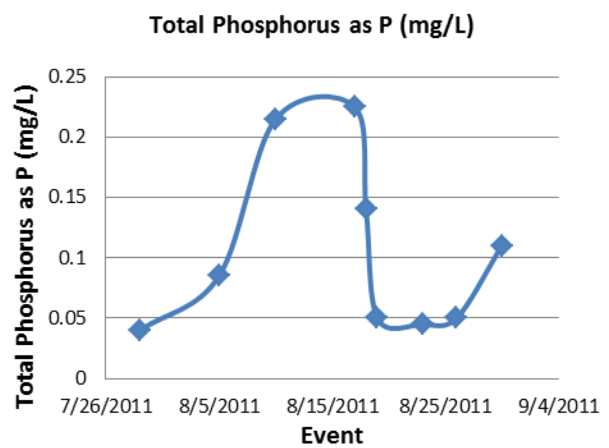
Time Series Plots

Figure 6-10 shows time series plots for one of the dry well locations as an example (135 Tennyson Road, Millburn, NJ 07078). The remaining set of time series plots is shown in Appendix C. These are for relatively short periods (barely more than one month), so obvious repeating trends are not expected.

135 Shallow	135 Deep
	







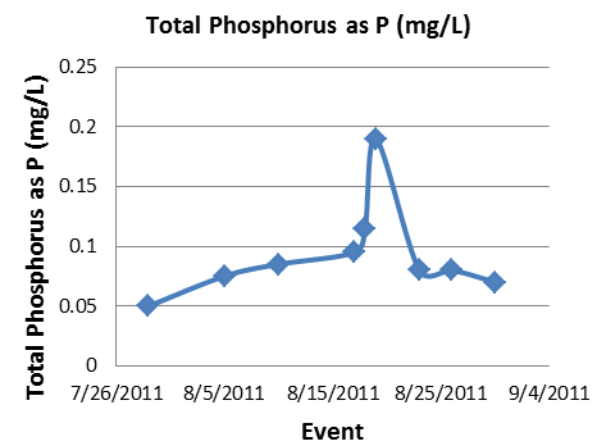
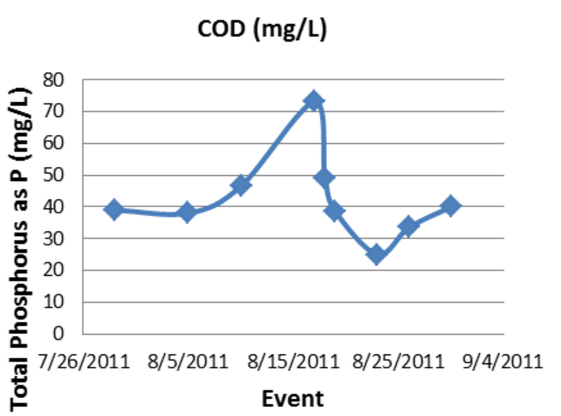


Figure 6-10 Time series plots for the site located on 135 Tennyson Road (shallow vs. deep)



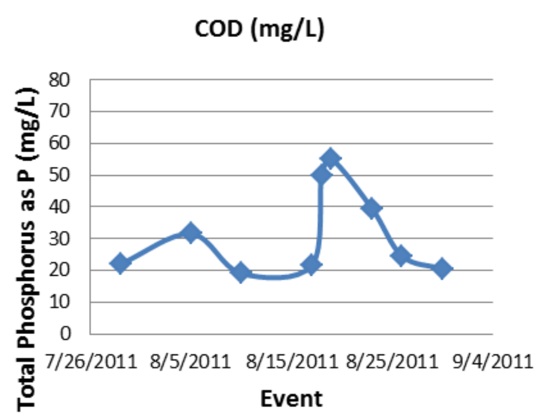
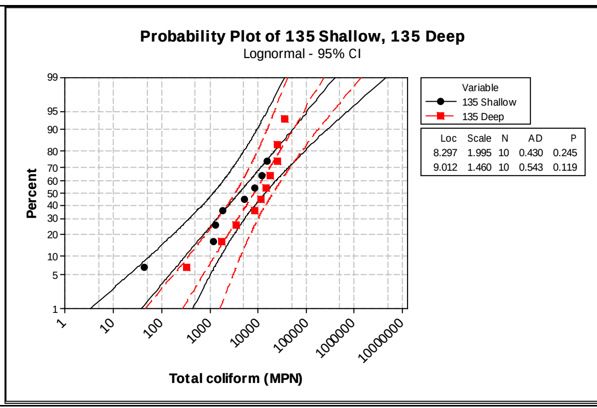


Figure 6-10 Time series plots for the site located on 135 Tennyson Road (shallow vs. deep)(continued)

Log-normal Probability Plots and Anderson-Darling Test Statistic

Log-normal probability plots were used to identify range, randomness, and normality of the data and to determine what type of statistical comparison tests can be used. The

Anderson-Darling (AD) statistical test was also conducted as part of these Minitab plots. In the AD test, the null hypothesis is that data follow a normal distribution (log-normal for these data as the data are plotted after log transformations). If the p-value is not less than the chosen level of 0.05, there is insufficient evidence to reject the null hypothesis, therefore the data fit the normal distribution. On the other hand, if the p-value is less than the chosen level of 0.05, the hypothesis would be rejected, thus the data do not follow a normal distribution. In this study, the log-normal probability plots are shown for inflow vs. cistern, for the cistern and deep vs. shallow for each sampling site. Figure 6-11 shows example paired log-normal probability plots for one of the sites (135 Tennyson Road, Millburn, NJ 07078) for different parameters including bacteria, nutrients, and COD. The remaining sets of plots are shown in Appendix C. For these plots, most of the data are seen to overlap within the limits of the 95% confidence limits, indicating that the data are likely from the same population. Also, the data seem to generally fit a straight line, indicating likely log-normal data distributions. The Anderson-Darling test statistics are used to quantify if the data are log-normally distributed.



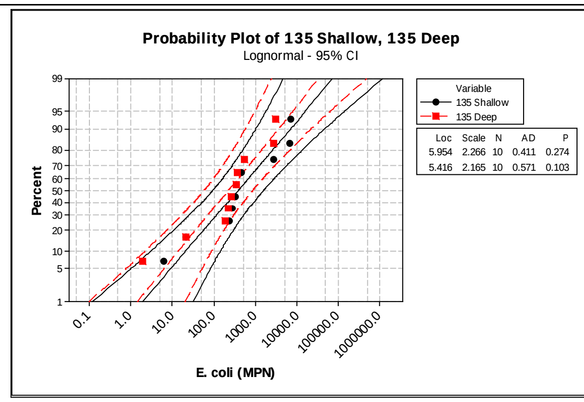
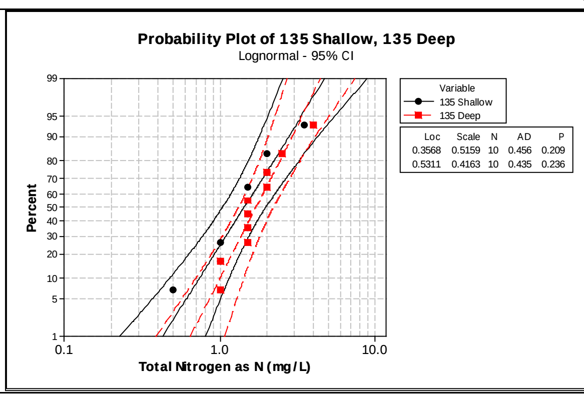
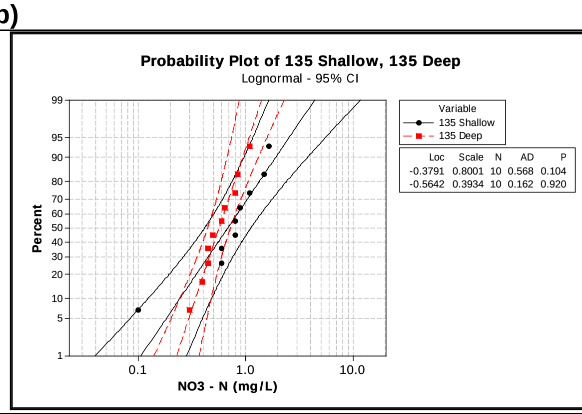
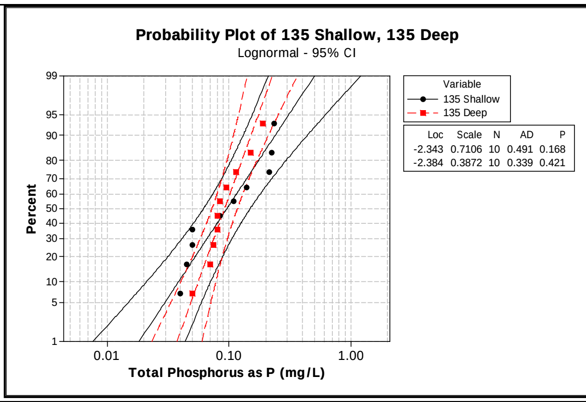


Figure 6-11 Log-normal probability plots for the site located on 135 Tennyson Road (shallow vs. deep)







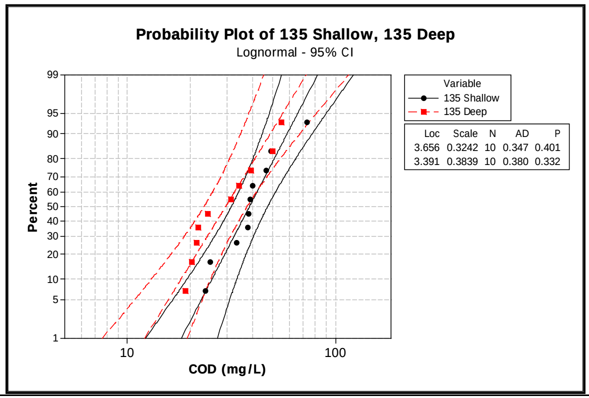


Figure 6-11 Log-normal probability plots for the site located on 135 Tennyson Road (shallow vs. deep)(continued)

Table 6-14 represents a summary of Anderson-Darling p-values for all parameters at all sampling sites. The highlighted values less than 0.05 represent conditions when the hypothesis would be rejected (these data do not follow a log-normal distribution). Due to the presence of many below detection limit (BDL) values for the heavy metal analyses, it is not possible to fit a log-normal distribution to the metal results.

Table 6-14. Summary of Anderson-Darling p-values.

Location	Parameter					
	Total coliform	*E. coli*	Total Nitrogen as N	NO_3 plus NO_2 as N	Total Phosphorus as P	COD
79 Inflow	0.047*	<0.005	0.567	0.673	0.885	0.373
79 Cistern	0.815	0.378	0.513	0.202	0.157	0.88
135 Shallow	0.245	0.274	0.456	0.568	0.168	0.401
135 Deep	0.119	0.103	0.236	0.162	0.421	0.332
18 Shallow	0.049	0.353	<0.005	0.482	0.253	0.023
18 Deep	0.013	0.135	0.213	0.44	0.158	0.771
139 Shallow	0.013	0.09	<0.005	0.136	0.018	0.262
139 Deep	0.165	0.11	0.589	0.003	0.011	0.409

* high-lighted conditions indicate data sets that were significantly different from normal distributions

Mann Whitney Test

The Mann-Whitney test, also called the rank sum test, is a nonparametric test that compares two unpaired groups. Nonparametric tests are preferred when the values are not normally distributed, or the distribution is unknown or mixed (as in this case). The

Mann Whitney test was performed using MINITAB to test if the shallow samples have significantly higher or lower concentrations than the deep values (same comparison test for inflow vs. cistern). This test performs a hypothesis test of the equality of the two population medians and calculates the corresponding point estimate and confidence interval. The probability of these two medians being the same (within the confidence interval) is then calculated. The p-value is used to evaluate the test results: if the populations really have the same median, what is the chance that random sampling would result in medians as far apart (or more so) as observed in this set of observations? If the number of samples is small, the Mann-Whitney test has little power. In fact, if the total sample size is seven or less, the Mann-Whitney test will always give a p-value greater than 0.05 no matter how much the groups differ (in this study there are only seven to ten samples at each location, so a larger critical p-value may be suitable). However, the p values calculated for these data are all much larger than 0.05, except as noted.

p-values less than, or equal to, a level of 0.05 are usually used to signify a significant difference (indicating an error of 1 out of 20 cases). An assumption for the Mann-Whitney test is that the data are independent random samples from two populations that have the same shape (distribution). To make sure that the populations have the same shape, over-laying probability plots were made for the two pairs of data in the previous probability plots. In all the cases, the straight lines were very close to each other and the bandwidths were quite similar. Therefore, the distributions can be reasonably assumed to be the same shape, and the samples from the same population. Table 6-15 shows the output obtained using MINITAB for comparison between paired data. Except for the bacteria and COD results for the cistern site, all paired sample sets did not indicate significant differences for these numbers of samples at the 0.05 level. The cistern median total coliform values were greater than the inflow median values, indicating possible re-growth; however, the median *E. coli* and COD cistern values were less than the inflow values for these constituents.

Table 6-15.Summary of Mann-Whitney Test for Paired Data

Parameter		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Total Coliforms	p-value	0.03	0.40	0.16	0.72
	Significant Difference Observes? (at level of 0.05)	Yes (but cistern median values were larger than the inflow median values)	No	No	No
E. coli	p-value	0.05	0.60	0.69	1
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No
Total	p-value	0.86	0.50	0.42	0.64

Parameter		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Nitrogen as N	Significant Difference Observes? (at level of 0.05)	No	No	No	No
NO_3 plus NO_2 -N	p-value	0.14	0.24	0.15	0.77
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
Total Phosphorus as P	p-value	0.77	0.94	0.10	0.27
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
COD	p-value	0.04	0.14	0.40	0.83
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No

Paired Sign Test for Metal Analyses

Due to the presence of large numbers of below-detection concentration values for the metal analyses, a simple paired sign test was used to compare each paired set of data. In the paired sign test, the null hypothesis is that the population medians are similar. In each pair of observations, a comparison was made to determine if there is an increase from the shallow sample to the deep sample or if there was a decrease. The advantage of the sign test is that if one part of the pair of data is not detected, while the other is, it is still possible to determine which is larger. However, if both data parts in the pair are not detected, it is not possible to determine which is larger and that pair is ignored in the calculations. If the calculated p-value is less than 0.05, then the null hypothesis will be rejected and the data are assumed to originate from different sample populations. Table 6-16 lists the results for the paired sign test for lead, copper and zinc data from the cistern and dry well samples. No statistically significant differences are seen between the sample sets for the heavy metals for the numbers of samples available.

Table 6-16. Summary of Paired Sign Test for Metal analysis

Metal		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Lead	p-value	> 0.06	> 0.06	0.18	> 0.06
	Significant Difference in Medians?	No	No	No	No
Copper	p-value	0.13	*	>0.06	*
	Significant Difference in Medians?	No	*	No	*
Zinc	p-value	0.45	0.45	>0.06	>0.06
	Significant Difference in Medians?	No	No	No	No

***** All the results are BDL, therefore it is not possible to do a paired sign test

Comparisons of Observed Water Quality to New Jersey Groundwater Disposal Criteria

Table 6-17 lists the most stringent regulatory levels for groundwater contaminants derived from N.J.A.C. 7:9C (2010), along with the range of observed concentrations for each constituent during these tests. Clearly, the microbiological and lead concentrations frequently exceeded the groundwater criteria.

Table 6-17. Groundwater Quality Criteria for the State of New Jersey Compared to Observed Water Quality from Dry Wells

Constituent	Groundwater Quality Criterion[1]	Observed Range [1]	Fraction of samples that exceed the criteria
Microbiological criteria[2]	Standards promulgated in the Safe Drinking Water Act Regulations (N.J.A.C. 7:10-1 et seq.)[3]	Total coliform: 1 to 36,294 MPN/100 mL *E. coli*: 1 to 8,469 MPN/100 mL	Total coliform: 63 of 71 samples exceeded the criterion for total coliforms *E. coli*: 45 of 71 samples exceeded the criterion for *E. coli*
Nitrate and Nitrite	10	0.0 to 16.5 (one sample had a concentration of 16.5 mg/L)	1of 71 samples exceeded the criterion for nitrates plus nitrites
Nitrate	10	0.1 to 4.7	0
Phosphorus		0.02 to 1.36	
COD		5.0 to 148	
Lead	0.005	BDL to 0.38	33 of 71 samples exceeded the criterion for lead
Copper	1.3	BDL to 1.1	0
Zinc	2.0	BDL to 0.14	0
2,4-D	0.07	Not Detected	0
2,4,5-TP (Silvex)	0.06	Not Detected	0
2,4,5-T	0.7	Not Detected	0
Aldrin	0.00004	Not Detected	0
Alpha-BHC	0.00002	Not Detected	0
beta-BHC	0.00004	Not Detected	0
delta-BHC		Not Detected	0
gamma-BHC (Lindane)	0.00003	Not Detected	0
alpha-Chlordane		0.00003	0
gamma-Chlordane		0.00002 to 0.000024	0
Dieldrin	0.00003	Not Detected	0
4,4′-DDD	0.0001	Not Detected	0
4,4′-DDE	0.0001	Not Detected	0
4,4′-DDT	0.0001	Not Detected	0
Endrin	0.002	Not Detected	0
Endosulfan sulfate	0.04	Not Detected	0
Endrin aldehyde		Not Detected	0
Endrin ketone		Not Detected	0
Endosulfan-I	0.04	0.000032 to 0.000034	0
Endosulfan-II	0.04	Not Detected	0
Heptachlor	0.00005	Not Detected	0
Heptachlor epoxide	0.0002	0.00003 to 0.000035	0
Methoxychlor	0.04	Not Detected	0
Toxaphene	0.002	Not Detected	0

[1] Groundwater quality criteria and observed range are expressed as milligrams per liter (mg/L) unless otherwise noted. [2] Pursuant to prevailing Safe Drinking Water Act Regulations any positive result for fecal

coliform is in violation of the MCL and is therefore an exceedance of the ground water quality criteria. [3] 50 MPN/100 mL

Summary of Water Quality Observations

Shallow and deep samples beneath three dry wells and samples at the inflow and in the cistern during ten storm events were analyzed for: total coliforms, *E. coli*, total nitrogen, NO_3 plus NO_2, total phosphorus, COD, lead, copper, and zinc. Three samples were also analyzed for pesticides and herbicides. Statistical analyses indicated that the differences in water quality between the shallow and the deep samples were not significant (p-values were > 0.05). However, significant differences were found ($p < 0.05$) between the quality of inflow samples and cistern samples for total coliforms (possible re-growth), *E. coli*, and COD.

These findings indicate that the dry wells did not significantly change any of the water quality concentrations for the stormwater constituents observed. If the influent water quality is of good quality, the dry wells can be a safe disposal method for stormwater quality. However, the bacteria and lead concentrations exceeded the groundwater disposal criteria for New Jersey and may require treatment, if the aquifer is critical.

Chapter 7 Alternative Stormwater Management Options for Millburn, New Jersey

Approach for Examining Alternative Stormwater Management Options for Millburn

The use of the dry wells in Millburn was in response to problems due to increased flows associated with increasing impervious areas during development and building expansions. The Millburn dry well regulations were therefore developed to require the use of dry wells to infiltrate these increased flows by specifying required dry well storage volumes for new impervious areas. The New Jersey state dry well regulations pertaining to stormwater control also include several guidelines, specifically relating to NRCS soil characteristics (A or B soils needed and associated minimum 5 or 12 mm/hr (0.2 or 0.5 in./hr) infiltration rates) and depth to the seasonal water table or bedrock (at least 2 ft below the infiltration system) that restrict their use. These regulations also restrict the source waters for infiltration to be roof runoff only. The 2-year runoff volumes are also to be used for the calculations. Depending on their applicability and intended use, some water quality criteria may also apply to the stormwater management options. Fecal indicator bacteria are likely of the most concern, while some heavy metals and nutrients may also be of interest.

When evaluating the Millburn dry wells during this study, most appeared to be operating well, with rapid infiltration and little standing water. However, several dry wells had standing water problems, possibly associated with elevated water tables. In addition, the water quality analyses did not indicate any significant improvement in the water quality of the runoff water while being discharged through the dry wells. Therefore, several alternatives were investigated that may offer some benefits for problematic conditions. Described in this report section include:

- Brief summary of the WinSLAMM model. This tool was used to examine the stormwater management alternatives for Millburn, using regionally calibrated conditions.

- Millburn area rainfall characteristics. Specifically, which rain conditions are responsible for most of the runoff from the area? Newark International Airport (located within 10 miles of Millburn) long-term rain information was used for these analyses for the 1948 through 1999 period.

- Sources of runoff water for Millburn residential land uses. This analysis was conducted to calculate the likely source contributions from all surface areas in the land use. Specifically, how does treatment of the roof runoff contribute to overall stormwater flow reductions? Should other sources areas also be considered for treatment?

• Dry well design alternatives. Specifically, the use of multiple dry wells, or shallower dry wells, was examined. The multiple dry wells may be needed in areas of marginal soils, while the shallower systems may be useful in areas having high water tables.

• Irrigation beneficial use of roof runoff. This was examined based on the local rainfall pattern, the regional evapotranspiration rates, and the irrigation limits of the landscaping plants. Cisterns used by themselves and in conjunction with dry wells were examined. Some homeowners are currently using cisterns in the area as a cost-effective alternative to the dry wells, especially considering the very high summer domestic water bills associated with landscaping irrigation.

• Rain garden use. The performance of rain gardens was examined as a function of the soil infiltration rates and rain garden size for local conditions. The advantage of rain gardens is that they are shallower and may not interact with the high water tables in some areas (although they would still be deep enough to penetrate through the more restrictive surface soils in this area). In addition, the media used in the rain gardens would provide some treatment of the infiltrating stormwater, especially compared to dry wells. As noted previously, lead and bacteria levels in the infiltrating water in and beneath the monitored dry wells frequently exceeded the New Jersey groundwater discharge criteria. Both of these constituents would be much more likely to be significantly reduced through infiltration using soils and other media, as in rain gardens, instead of the crushed stone as in the dry wells.

WinSLAMM Background Information

WinSLAMM (Pitt and Voorhees 1995) was developed to evaluate stormwater runoff volume and pollutant loadings in urban areas using continuous small storm hydrology relationships, in contrast to single event hydrology methods that have been traditionally used for much larger drainage design events. WinSLAMM determines the runoff based on local rain records and calculates runoff volumes and pollutant loadings from each individual source area within each land use category for each rain. Examples of source areas include: roofs, streets, small landscaped areas, large landscaped areas, sidewalks, and parking lots.

WinSLAMM is unique in many aspects. One of the most important aspects is its ability to consider many stormwater controls (affecting source areas, drainage systems, and outfalls) together, for a long series of rains. Another is its ability to accurately describe a drainage area in sufficient detail for water quality investigations, but without requiring a great deal of superfluous information that field studies have shown to be of little value in accurately predicting discharge results. WinSLAMM also applies stochastic analysis procedures to more accurately represent actual uncertainty in model input parameters in order to better predict the actual range of outfall conditions (especially pollutant concentrations). However, the main reason WinSLAMM was developed was because of errors contained in many existing urban runoff models. These errors in calculating pollutant washoff from streets and runoff volume calculations during small and intermediate storms were obvious when comparing actual field measurements to the solutions obtained from algorithms used in other available models.

Stormwater Controls in WinSLAMM and Calculation Processes

WinSLAMM was used to examine a series of stormwater control practices, including dry wells and water tanks for stormwater irrigation for residential land use conditions observed in Millburn. The model evaluates the practices through engineering calculations of the unit processes based on the actual design and size of the controls specified and determines how effectively these practices remove runoff volume and pollutants.

WinSLAMM does not use a percent imperviousness or a curve number to general runoff volume or pollutant loadings. The model applies runoff coefficients to each "source area" within 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, etc., 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 rainfall 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 contained in the range of rains specified 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 based on the specific source areas in that modeled area. The model replicates the physical processes occurring within the stormwater control.

The model's output is comprehensive and customizable, and typically includes:

1. Runoff volume, pollutant loadings and event mean concentrations (EMCs) for a period of record and/or for each event.
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 can be found at www.winslamm.com. For this project, the parameter files were calibrated using regional East Coast MS4 monitoring data as contained in the National Stormwater Quality Database (NSQD), available at:
http://www.unix.eng.ua.edu/~rpitt/Research/ms4/mainms4.shtml

Regional Rainfall and Runoff Distributions and Sources of Stormwater Discharges

The model can use any length of rainfall record as determined by the user, from single rainfall events to several decades of rains. The rainfall file used in the calculations for Newark International Airport (the closest long-term rainfall station to Millburn, at less than ten miles away) was developed from hourly data obtained from EarthInfo CDROMs, using the 51 years from 1948 through 1999, as shown on Figure 7-1. This period contained 5,401 rains, with an average depth of 11 mm (0.42 in.) and a maximum depth of 210 mm (8.25 in.). Hurricane Irene monitored during this study period resulted in more than 9 in. of rainfall, a historical record for the area.

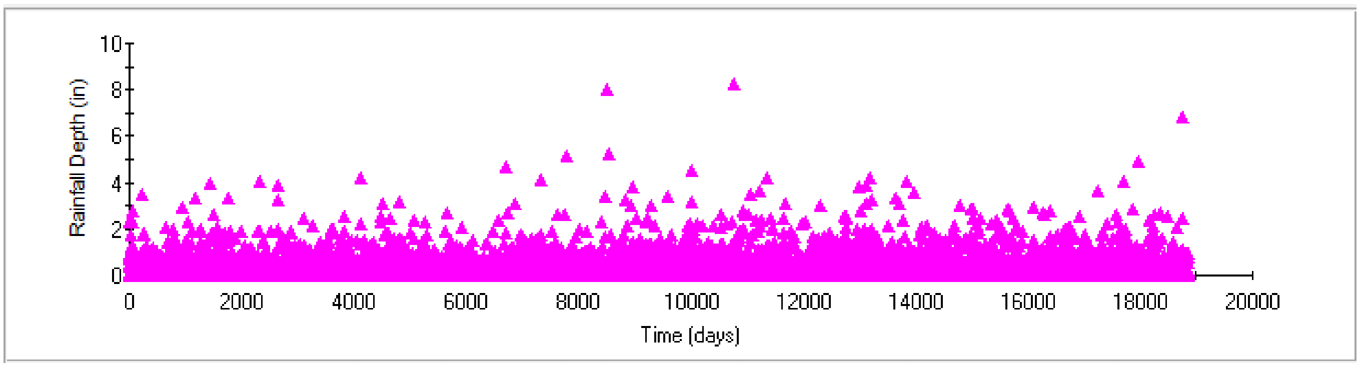


Figure 7-1. Long-Term Rain Depths for Individual Newark, NJ, Rains. (1948 – 1999). (1 in. = 25.4 mm)

Figure 7-2 shows that the regional stormwater runoff is heavily influenced by the small to intermediate rains (data for the region shown for Newark, NJ). Almost all of the runoff is associated with rains between about 7.6 to 76 mm (0.3 to 3 in.), the events for which WinSLAMM is optimized. The 2-year rain depth for Essex County is about 86 mm (3.4 in.); rains of this event and smaller are responsible for about 90% of the total annual runoff volumes. This rain depth is the design event for dry wells based on the New Jersey dry well design criteria. The larger, rare drainage design events generally contribute a very small portion of the typical year's runoff.

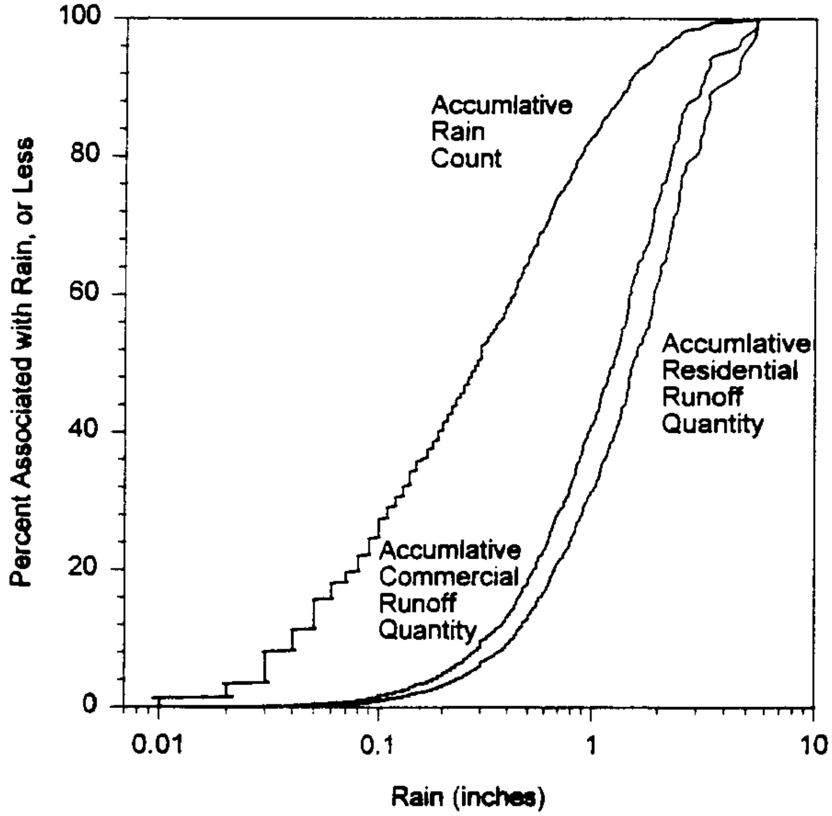


Figure 7-2. Newark, NJ, Rain and Runoff Distributions. (1982 through 1992 rains). (1 in. = 25.4 mm)

Sources of Runoff from Different Source Areas

WinSLAMM version 10 was used to calculate the relative source contributions of runoff volume and particulate solids for the typical residential areas in Millburn. Tables 7-3 through 7-5 and Figures 7-3 and 7-4 include the source area contributions for the Millburn residential stormwater. Table 7-1 shows the areas associated with each surface, with a few minor source areas not included. Almost all of the impervious areas are directly connected to the drainage system, with a few flat roofs (sheds), backyard walkways, and decks and patios draining to pervious areas and are therefore disconnected.

Table 7-1. Source Areas in Millburn Residential Land Use (average of investigated sites)

	Roofs (directly connected pitched)	Paved Parking (directly connected)	Driveways (directly connected)	Sidewalks (directly connected)	Street Areas (intermediate texture, 35 ft wide)	Landscaped Areas (silty soil)	Land Use Total
Percentage Area:	13.5%	3.4%	7.8%	1.7%	11.8%	60.8%	100

1 ft = 0.3048 m

Table 7-2 summarizes the associated runoff volume contributions for these areas, and Table 7-3 shows the particulate solids contributions for these areas, using the 1952 through 1999 rain series from the Newark International Airport. The landscaped area is the largest single land cover, at about 61%, but only contributes about 12% of the runoff volume and 24% of the particulate solids contributions over this long time series. The directly connected roofs are the single largest runoff contributor (at 33%), with the streets the next most important contributor (at 27%). Driveways contribute a surprisingly large portion of the runoff (at 18%). Figure 7-3 shows the runoff volume contributions for different rain depths. This plot indicates the importance of the different source areas as the rain depth changes. For the smallest rains, the directly connected roofs and the streets are the most important sources, as typical. However, for the larger rains (greater than about 51 mm (2 in.)), the landscaped areas contribute about 25% of the total runoff volume. For stormwater controls only affecting the roofs, the maximum outfall runoff reduction would therefore be about 33%. If driveways and parking areas could also be controlled on site, the maximum outfall benefit could increase to about 60%. It is difficult to reduce street runoff on private property, especially in an area having relatively steep front yards, as in the Millburn area, and runoff from landscaping areas can only be reduced by enhancing the soil structure (which may be possible during construction, but difficult after construction).

Table 7-2. Runoff Volume Sources (%) for Millburn Residential Area (Newark 1952-1999 rain series)

	Rain Total (in.)	Roofs (directly connected pitched)	Paved Parking (directly connected)	Driveways (directly connected)	Sidewalks (directly connected)	Street Area (intermediate texture, 35 ft wide)	Landscaped Area (silty soil)	Land Use Totals
Minimum:	0.01	25.8	6.2	10.4	2.3	15.8	0.1	100
Maximum:	6.8	74.1	25.9	23.8	5.2	36.3	25	100
Fl Wt Ave:	0.45	33.1	6.7	17.5	3.8	26.7	11.9	100

(1 in. = 25.4 mm, 1 ft = 0.3048 m)

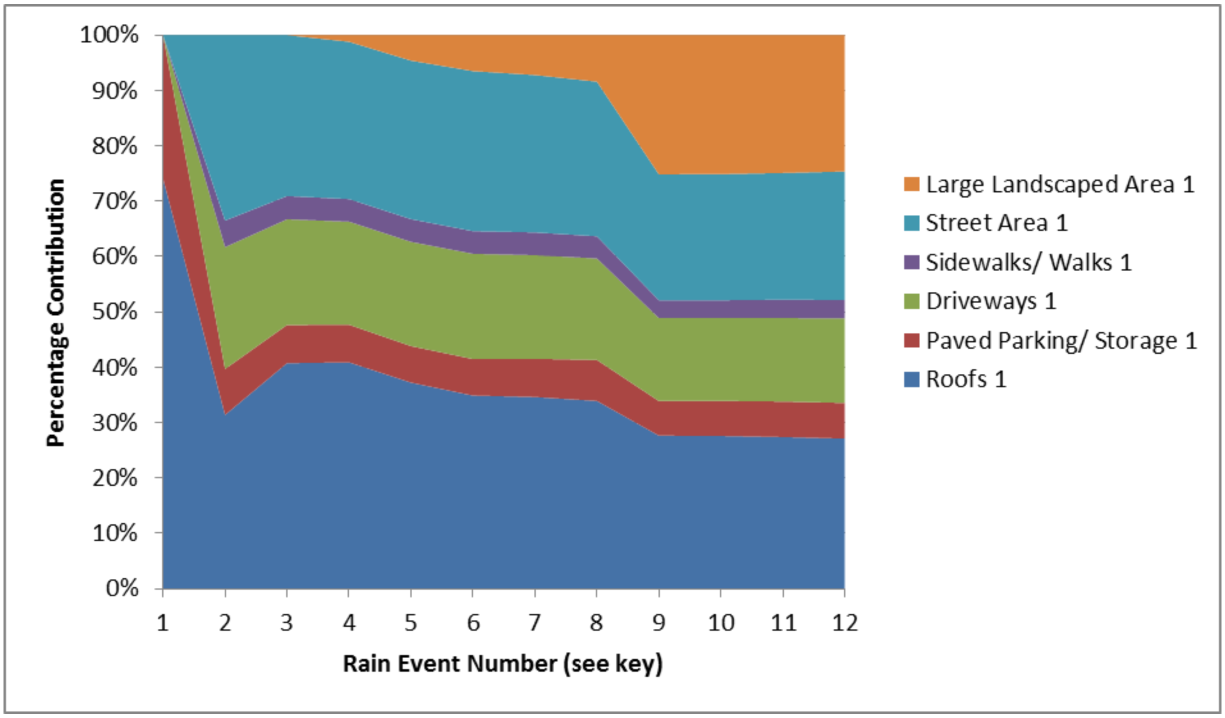


Figure 7-3. Runoff volume source contributions for different rain events for Millburn, NJ (Rain key: 1: 0.01 in., 2: 0.05 in.; 3: 0.1 in.; 4: 0.25 in.; 5: 0.5 in.; 6: 0.75 in.; 7: 1 in.; 8: 1.5 in.; 9: 2 in.; 10: 2.5 in.; 11: 3 in.; 12: 4 in.).

Table 7-3 and Figure 7-4 show similar data for particulate solids (TSS). The contributions from the different areas are different from the runoff volume sources. Specifically, roof runoff has a much lower TSS concentration than other areas. For this area, the following source area sheetflow TSS concentrations were typically modeled: roof runoff <50 mg/L, driveway and street runoff about 100 to 150 mg/L, and landscaping runoff about 200 mg/L. Therefore, roofs are only expected to contribute about 11% of the long-term averaged TSS contributions, while the streets, landscaped areas, and driveways each contribute about 25 to 30%. Figure 7-4 shows that landscaped areas contribute almost half of the TSS for the largest rains, but contribute little, until at least 25 mm (1 inch) rains.

Table 7-3. Particulate Solids Sources (%) for Millburn Residential Area (Newark 1952-1999 rain series)

	Rain Total (in.)	Roofs (directly connected pitched)	Paved Parking (directly connected)	Driveways (directly connected)	Sidewalks (directly connected)	Street Area (intermediate texture, 35 ft wide)	Landscaped Area (silty soil)	Land Use Totals
Minimum:	0.01	6.5	4.2	7.9	0.8	4.7	0.1	100
Maximum:	6.8	44.8	55.2	43.2	4.6	69.7	52.5	100
Fl Wt Ave:	0.45	11	7.8	24.2	2.6	29.9	24.3	100

(1 in. = 25.4 mm, 1 ft = 0.305 m)

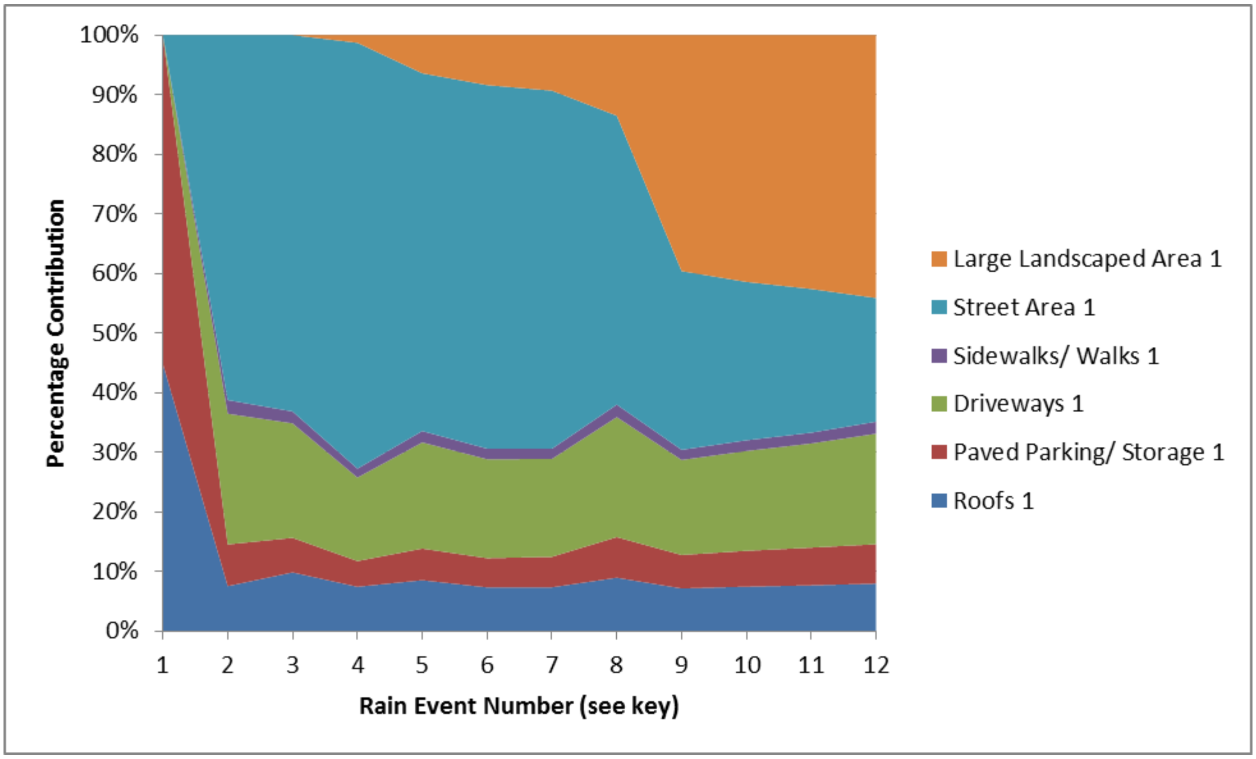


Figure 7-4. Particulate solids mass source contributions for different rain events for Millburn, NJ (Rain key: 1: 0.01 in., 2: 0.05 in.; 3: 0.1 in.; 4: 0.25 in.; 5: 0.5 in.; 6: 0.75 in.; 7: 1 in.; 8: 1.5 in.; 9: 2 in.; 10: 2.5 in.; 11: 3 in.; 12: 4 in.).

Dry Well Analyses for Millburn Residential Areas

WinSLAMM version 10 was used to calculate the effects of many dry well options. The standard units are 1.8 m (6 ft) diameter, 1.8 m (6 ft) deep on top of 0.6 m (2 ft) of crushed stone and 0.6 m (2 ft) of crushed stone around the sides. This is therefore equivalent to 2.4 m (8 ft) diameter (5.6 m^2 or 51 ft^2) area, and 2.3 m (6.9 ft) depth, with 12.9 m^3 (350 ft^3) of volume storage per unit. Figure 7-5 is a screen shot of the basic input screen used for the dry wells (a very simplified version of the biofilter control practice).

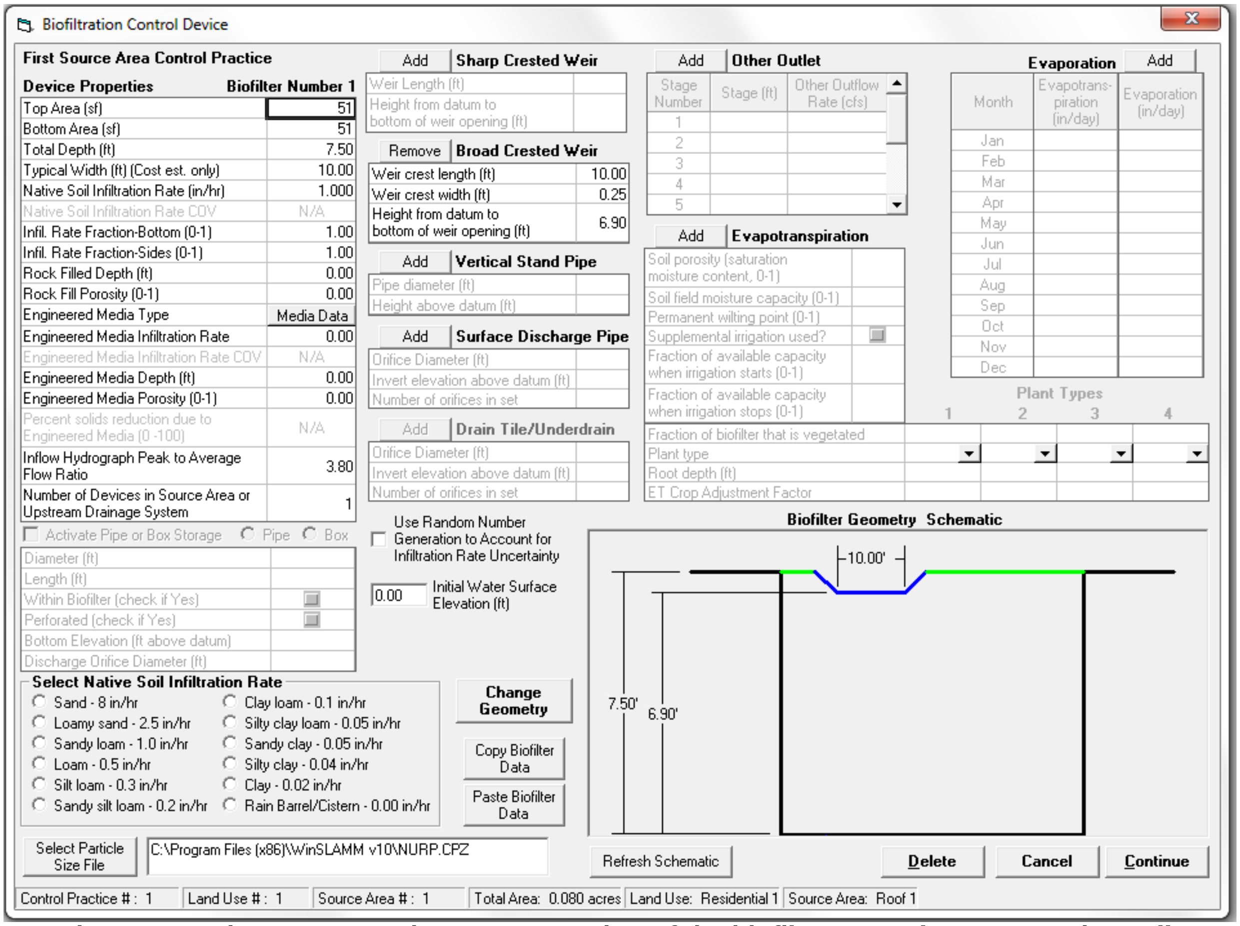


Figure 7-5. WinSLAMM version 10 screen shot of the biofilter control setup as a dry well.

The analyses examined a range of subsurface soil infiltration rates, ranging from 2.5 to 130 mm/hr (0.1 to 5 in./hr). One, two, and three dry wells per residential lot were also examined. Figures 7-6 and 7-7 are production function plots showing the annual runoff volume reductions associated with using 1, 2, or 3 dry wells for the different infiltration rates. Figure 7-6 is for the roof runoff volume reductions and shows that 90% reductions in the annual roof runoff is expected to be infiltrated for about 1.5 dry wells per lot (the typical average for Millburn) whenever the infiltration rate is at least about 7.6 mm/hr (0.3 in./hr), slightly more than the minimum 5.1 mm/hr (0.2 in./hr) criterion in the state dry well guidance. The 12 mm/hr (0.5 in./hr) state criterion would result in roof runoff infiltration of close to 95% of the annual roof runoff. The 11 mm/hr (0.45 in./hr) f_c (constant Horton rate after saturation) observed for the C and D surface soils (well-drained subsurface conditions) would result in similar annual roof runoff losses. With two dry wells per lot, the annual roof runoff reductions are about 90% for infiltration rates as low as 2.5 mm/hr (0.1 in./hr).

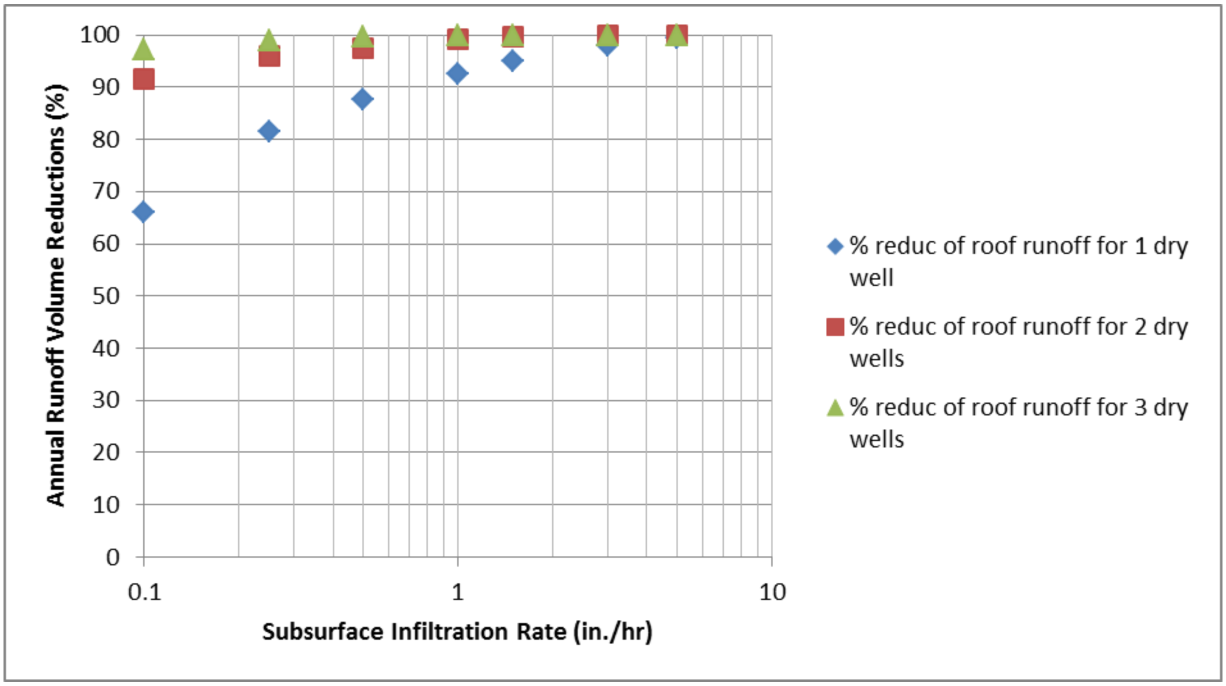


Figure 7-6. Roof runoff volume reductions using dry wells in Millburn, NJ. (1 in./hr = 25.4 mm/hr)

Figure 7-7 is a similar plot, but for outfall (total residential area) runoff volume reductions associated with the roof runoff dry wells. The long-term roof runoff percentage of the total area runoff is 33% (only 11% for TSS). Therefore, this plot shows values about one-third of the roof runoff control plot: The maximum benefit for the whole area would therefore be limited to about 33% runoff volume control.

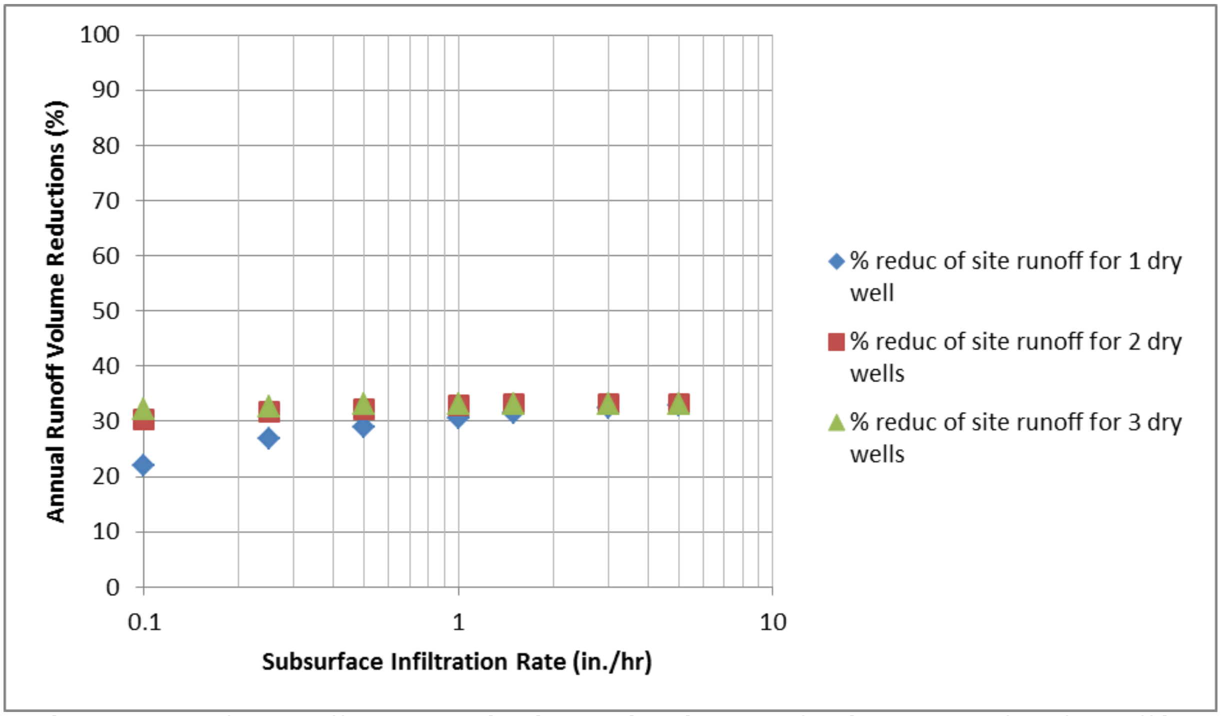


Figure 7-7. Outfall runoff volume reductions using dry wells for the control of roof runoff in Millburn, NJ.
(1 in./hr = 25.4 mm/hr)

The expected modeled performance of the dry wells is similar to the observed level of performance; they function well and completely drain the inflowing water with little overflow. However, two problems occur: slow drainage times for some dry wells and standing water in others. Figure 7-8 is a plot of the drainage times needed for dry wells at 1.8 m (6 ft) and 0.9 m (3 ft) deep. The State dry well criterion of 72 hr for complete drainage is met for full 1.8 m (6 ft) depth dry wells with at least 2.5 mm/hr (1 in./hr) infiltration rates. In order to allow this criterion to be met in areas having lower infiltration rates, it may be necessary to use shallower dry wells. As an example, a 0.9 m (3 ft) deep dry well full of water would require at least 12 mm/hr (0.5 in./hr) infiltration rates to meet the 72 hour criterion. As noted above for the long-term continuous simulations, this drainage time is not needed to ensure good performance by having the dry wells empty before the next rain. However, aquatic insect pests (mainly mosquitoes) can be a problem with standing water after several days of quiescent conditions. Luckily, the dry wells do not reach maximum depth for every rain (unless they were greatly undersized and filled frequently). Therefore, the maximum drainage time should only infrequently occur.

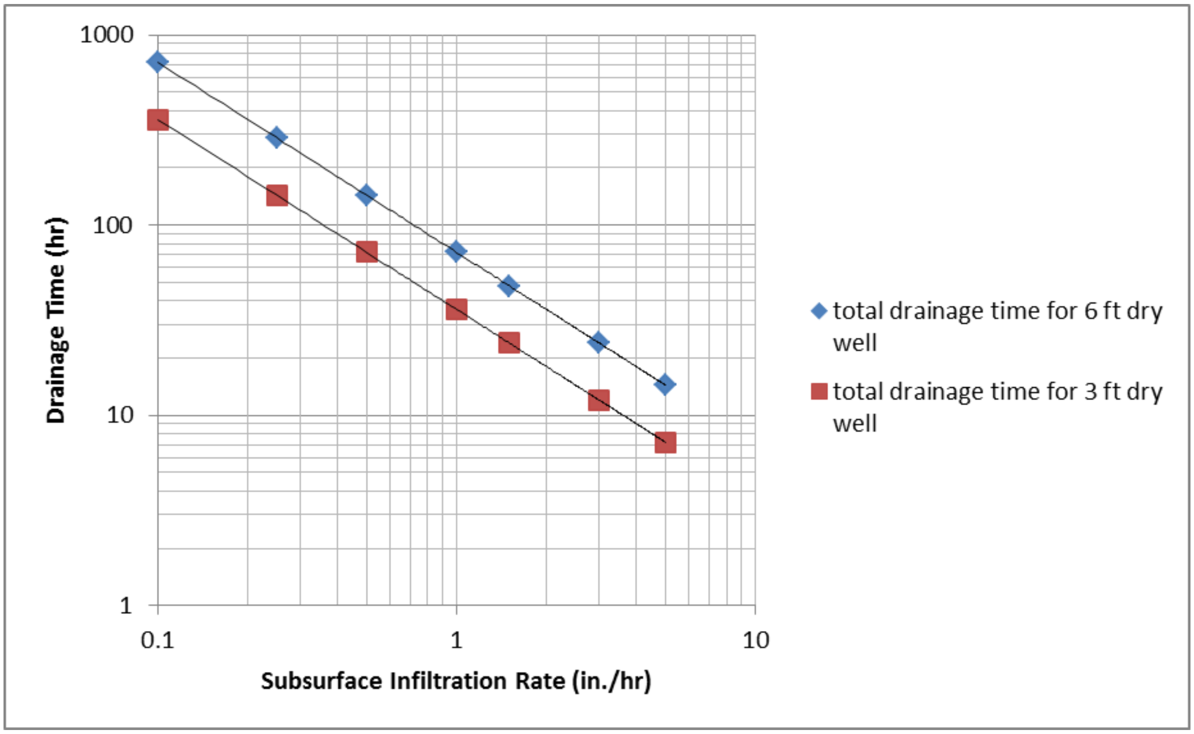


Figure 7-8. Drainage times required for full dry wells 6 ft and 3 ft deep for different infiltration rates. (1 in./hr = 25.4 mm/hr, , 1 ft = 0.3048 m)

If the water table is high and enters the bottom of the dry well, long periods of standing water may then occur, causing nuisance conditions. The use of multiple shallower dry wells may therefore be an option to lessen the likelihood of the high seasonal water table entering the dry well. Fewer hours of standing water will also occur in the shallower dry well for the probable slower infiltration rates in the low lying areas. Figure 7-9 is a plot comparing the performance of two shallower 0.9 m (3 ft) deep dry wells compared to a single 1.8 m (6 ft) deep dry well. The total storage volume is the same for both options, but the shallower dry wells offer greater performance for the same infiltration rates (especially at low rates). This is due to the shallower dry wells having a larger infiltration area and an overall faster rate of drainage, resulting in the shallower dry wells having more usable storage volume for more of the rains compared to the single deeper dry well.

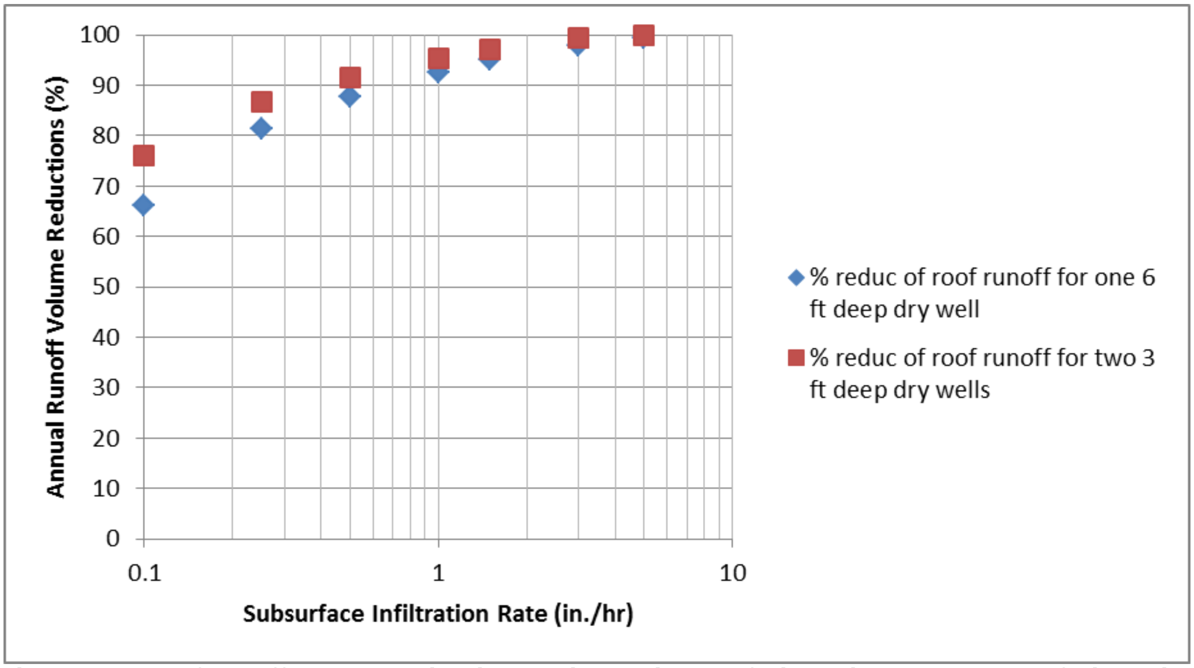


Figure 7-9. Roof runoff volume reductions using a single 6 ft deep dry well or two 3 ft deep dry wells. (1 in./hr = 25.4 mm/hr, , 1 ft = 0.3048 m)

Another issue that needs to be considered is the clogging potential of the dry wells. Figure 7-10 is a plot indicating the time before sediment loads that may cause significantly decreased infiltration in the dry wells (10 to 25 kg/m^2 (2.1 to 5.1 lb/ft^2) corresponding to about 4 to 10 mm (0.15 to 0.4 in.) sediment depth). As shown, these sediment loads from the roof runoff may occur in as little as about 4 years to more than 10 years for infiltration rates greater than 6.4 mm/hr (0.25 in./hr). There is substantial storage in the void space of the crushed stone beneath the dry well (about 250 mm), so this sediment accumulation may cause lateral flow near the bottom of the stone layer before fully restricting the infiltration. However, if additional sediment or debris enters the dry wells (such as from surface flows from eroding areas and even from landscaped areas during heavy rains), clogging may be responsible for premature reduced performance.

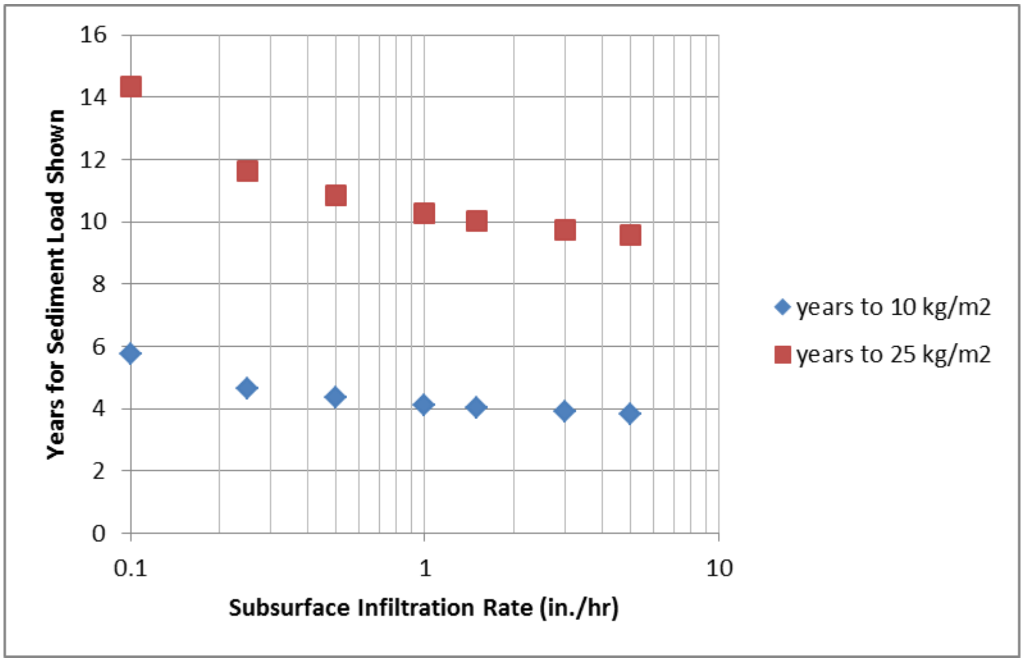


Figure 7-10. Clogging potential of dry wells in Millburn, NJ. (1 in./hr = 25.4 mm/hr)

Stormwater used for Irrigation of Landscaped Areas in Millburn

Millburn, New Jersey Water Use

Population and water use changes with time affect future stormwater management. As an example, these estimates are both needed when comparing opportunities for beneficial uses of stormwater in residential areas. In the U.S., information concerning population and household social-economic conditions is available from the U.S. Census Bureau by zip code. Millburn, NJ, zip codes and population conditions are shown in Table 7-4.

Table 7-4. Summary of Census 2000 Information for Millburn, NJ, Zip Codes 07078 and 07041. (U.S. Census Bureau)

Zip Code	Population	Total Housing Units	Occupied Housing Units	Average Household Size
07078	12,849	4,337	4,256	3.02
07041	6,880	2,809	2,747	2.5
Total	19,729	7,146	7,003	2.81

Domestic water use information is also available from the USGS ("Water use in the United States," available at: http://water.usgs.gov/watuse/), by county. The water use values are available for domestic uses and for several dates in recent years. Figure 7-

11 is a plot of how these domestic water use values have changed in Essex County (containing Millburn, NJ), which has ranged between 64 and 84 gal/person/day during the time from 1985 to 2005, with the most recent rates being the lowest shown.

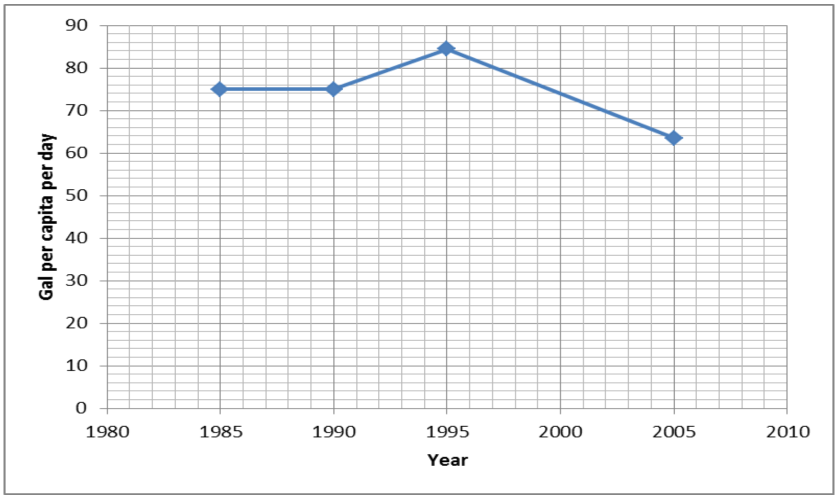


Figure 7-11. Essex County NJ daily per capita Water Use.

The Urban Water Budget and Stormwater Reuse in U.S. Residential Areas

It is possible to determine the fraction of the irrigation water and toilet flushing water that can be supplied by roof runoff. For example, the following lists example inside household water use (no irrigation):

- bathing 42%
- laundry 11%
- kitchen sink 15%
- dishwasher 8%
- bath sinks 12%
- toilet flushing 12%

This example household is for a working family with a child in school; the bathing water use was therefore relatively high, while the toilet flushing water use was relatively low, as the household residents are away from home much of the day. There were also wide variations in water use for different days of the week, with weekday water use (especially toilet flushing and laundry) being substantially less than for weekend water use. The household water use was relatively constant throughout the year and averaged about 340 L/c/day, liters per capita per day (90 gal/capita/day, gpcd), ranging from 290 to 400 L/C/day (77 to 106 gpcd). Outside irrigation water use during the dry months averaged about 190 Liters per day (50 gal/day), or 200 L/day (for a 0.5 acre landscaped area) above the inside water uses listed above. Landscape irrigation may occur for about 2 months at this level of use in this area.

Table 7-5 is from a study by Aquacraft, Inc. and the American Water Works Association Research Foundation for 12 different study sites (1999). The typical household in the United States uses about 59% of its total water use for outdoor usage and 35% for indoor usage (leakage and unknown uses make up the remaining 6%).

Table 7-5 Breakdown of residential water usage in the United States (Source: http://www.aquacraft.com/Publications/resident.htm)

Fixture/End Use	Indoor use percent	Total use percent
Toilet	30.9%	10.8%
Clothes washer	25.1%	8.7%
Shower	19.4%	6.8%
Faucet	18.2%	6.3%
Other domestic	2.7%	0.9%
Bath	2.0%	0.7%
Dishwasher	1.7%	0.6%
Indoor Total	**100.0%**	**34.8%**
Leak	NA	5.5%
Unknown	NA	1.0%
Outdoor	NA	58.7%
TOTAL	**NA**	**100.0%**

The following is a summary of the monthly rainfall pattern for Millburn, NJ. The total rainfall for Millburn is almost 1270 mm (50 in.) (slightly more than the NJ average of about 1219 mm (48 in.) per year), ranging from about 76 to 130 mm (3 to 5 in.) per month (most occurring from April thru July).

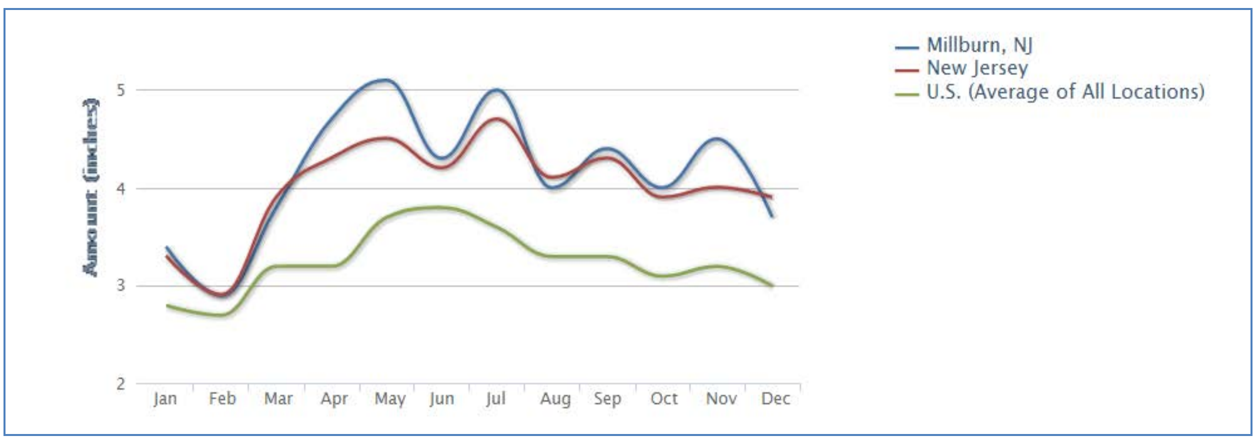


Figure 7-12. A summary of the monthly rainfall pattern for Millburn Data from: http://www.usa.com/millburn-nj-weather.htm#HistoricalPrecipitation (1 in. = 25.4 mm)

The estimated roof runoff for a typical 200 m^2 (2,000 ft^2), 1- ½ level, house (roof area of about 120 m^2 (1300 ft^2)) would be about 150 m^3 (40,000 gal) per year, for this area having about 50 in. of rain a year. The total water use for this household could be about 400 m^3 (100,000 gal) per year, with the amount used for toilet flushing being about 45 m^3 (12,000 gal), with another 10 m^3 (3,000 gal) used for landscaping irrigation. For this example, the roof runoff would supply almost three times the amount of water needed for toilet flushing and landscape irrigation. None of the other household water uses would be suitable for supply by roof runoff due to health and safety considerations. The rainfall varies between about 76 to 130 mm (3 to 5 in.) per month, with a rain occurring about twice a week on the average. Rains occurring only once every two weeks can occur during the most unusual conditions (the driest months when landscaping irrigation is most needed). Therefore, a simple estimate for required roof runoff storage would be two weeks for average toilet flushing (1.7 m^3 or 450 gal), plus two weeks for maximum landscaping irrigation (3 m^3 or 700 gal). A total storage tank of 4.7 m^3 (1250 gal) (a typical septic tank size) would therefore be needed. Of course, a factor-of-safety multiplier can be applied, depending on the availability of alternative water sources.

For a 0.5 acre residential lot, the annual stormwater generated would be about 650 m^3 (170,000 gal) per year. The roof would produce about 25% of this total, pavement would produce another 25%, and the landscaped area would produce about 50% of this total. Therefore, the amount of stormwater used on-site for toilet flushing and irrigation of landscaped areas would be only about 10% of the total generated. Therefore, most of the runoff would still have to be infiltrated on-site, or safely conveyed and discharged.

Calculating the Benefits of Rainwater Harvesting Systems and Evapotranspiration Rates

The following discussion presents a method to evaluate or determine the needed size of water tanks needed to optimize the beneficial uses of stormwater. Irrigation of land on the homeowner's property was considered the beneficial use of most interest. Production function curves were prepared for the Millburn, NJ area, showing the relationship between water tank size and roof runoff beneficial use.

Benefits associated with stormwater use for irrigation and other on-site uses can be calculated based on 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 describes how WinSLAMM, the Source Loading and Management Model (Pitt 1997), was used to calculate the production functions that can be used to size storage water tanks to maximize irrigation use for residential locations in Millburn.

On the average, each person uses approximately 240 Liters (64 gal) of water per day in New Jersey. A significant amount is used each day to maintain outdoor landscaping. With a little planning, the amount of water required for landscaping can be significantly reduced. The amount of water required for landscaping depends on the type and location of the plants (i.e., lawn, annual flowers, trees and shrubs).

Essex County is near to the New Middlesex Co., NJ and Ringwood, NJ evapotranspiration reference (ET_o) stations. Table 7-6 describes the average ET_o for these stations per month, with the average values used for these calculations for Essex County.

Table 7-6 Average ET_o by Month for New Middlesex and Ringwood, New Jersey

	New Middlesex County New Jersey (in./day)	Ringwood New Jersey (in./day)	Average ET_o (in./day)
January	0.02	0.01	0.015
February	0.03	0.03	0.03
March	0.09	0.12	0.105
April	0.14	0.12	0.13
May	0.17	0.14	0.155
June	0.17	0.14	0.155
July	0.18	0.13	0.155
August	0.16	0.11	0.135
September	0.14	0.10	0.12
October	0.10	0.13	0.115
November	0.09	0.11	0.10
December	0.04	0.05	0.045

(1 in./day = 25.4 mm/day)

Irrigation Water Use

Tables 7-7 and 7-8 and Figures 7-13 and 7-14 are calculated supplemental irrigation requirements for Millburn residential areas. These areas have roofs that are about 325 m^2 (3,500 ft^2) (13.5% of the land use) and landscaped areas about 1,440 m^2 (15,500 ft^2) (61% of the land use), with a relatively large roof to landscaped area ratio of about 0.23 (large homes and small lots). Table 7-7 and Figure 7-13 show the irrigation needs that can be considered the minimum amount by minimally meeting the area evapotranspiration requirements (assuming all of the rainfall contributes to soil moisture, which is true for rains less than about 25 mm (1 inch) in depth, but some of the rain flows to the storm drainage system for larger rains, as shown earlier, with no crop adjustment factor). The monthly rainfall compared to the monthly ET is shown in Figure 7-13 and illustrates how supplemental irrigation would be needed in the summer months, as expected. Table 7-7 shows these calculations, including the monthly irrigation needs in gal per day per house. This rate would be used for minimally meeting the ET needs without 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 goal, maximum utilization of the roof runoff is desired.

Table 7-7. Irrigation Needs to Satisfy Evapotranspiration Requirements for Essex County, NJ

	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 acres	0	0	0	63	256	577	188	47	0	160	0	0	39,200 gal/year

(1 in./mo = 25.4 mm/mo)

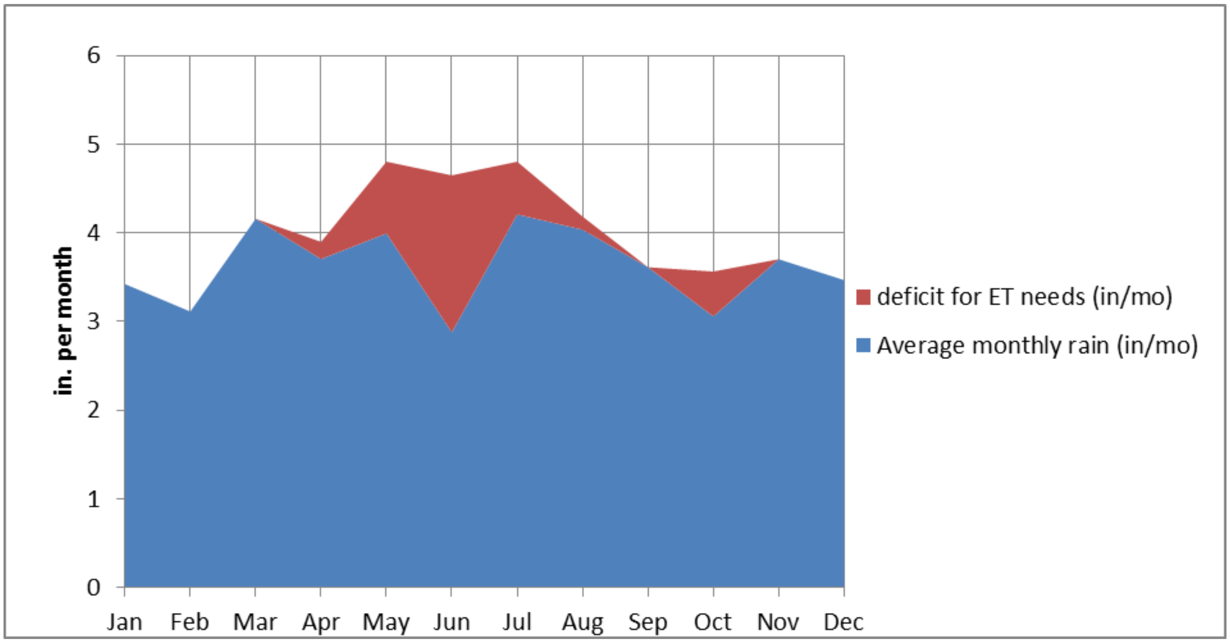


Figure 7-13. Plot of supplemental irrigation needs to match evapotranspiration deficit for Essex County, NJ. (1 in./mo = 25.4 mm/mo)

For maximum use of the roof runoff, it is desired to irrigate at the highest rate possible, without causing harm to the plants, or adverse groundwater elevation increases (mounding). Therefore, Table 7-8 and Figure 7-14 show an alternative 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 US, 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. Table 7-8 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 1,200 m^3 (318,000 gal) per year per home. This is about eight times the amount needed to "barely" satisfy the ET requirements noted above. However, the roofs in the study area are only expected to produce about 340 m^3 (90,000 gal) 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 may be possible to use runoff from other areas, besides the roofs, for supplemental irrigation.

Table 7-8. Irrigation Needs to Satisfy Heavily Irrigated Lawn for Essex County, NJ

	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 (gal/day/house) 0.36 acres	0	0	0	96	1263	1669	1826	1880	2081	1558	96	0	318,000 gal/year

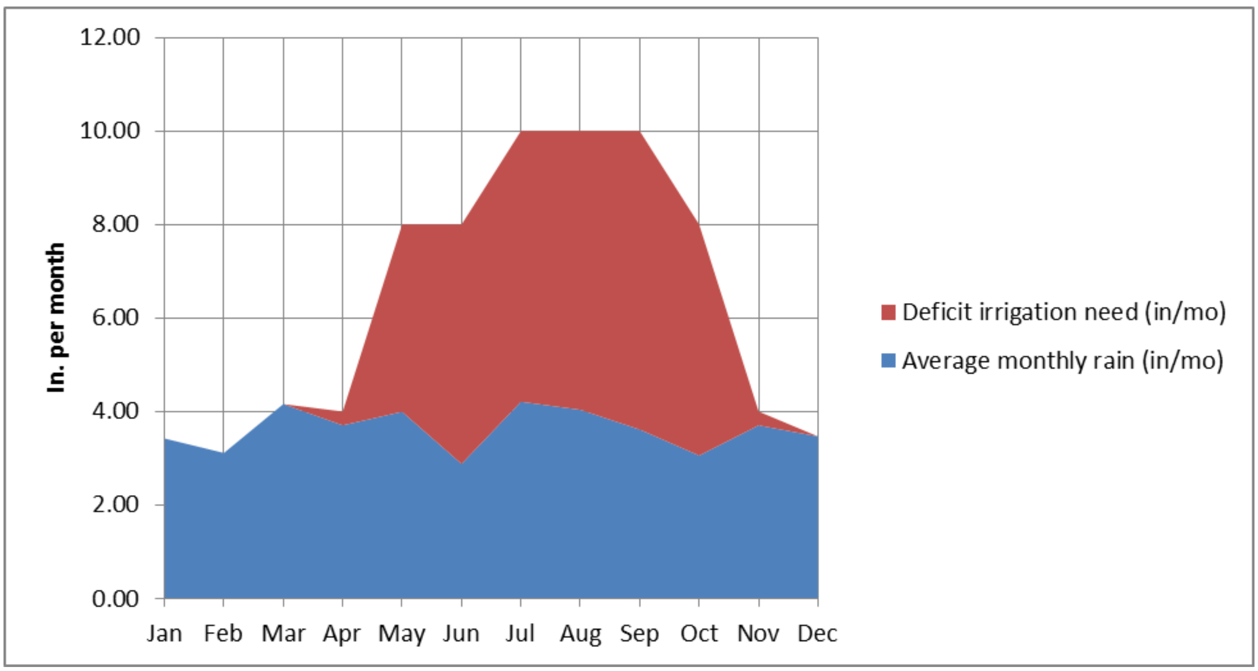


Figure 7-14. Plot of supplemental irrigation needs to match heavily watered lawn (0.5 to 2.5 in./week) deficit for Essex County, NJ. (1 in./mo = 25.4 mm/mo)

The following discussions examine the effective use of this water for beneficial irrigation use and the needed water storage tank (cistern) volumes for Millburn, NJ.

Roof Harvesting and Water Tank Sizes

The monthly infiltration amounts in the landscaped areas, assuming silty soils, were calculated using continuous WinSLAMM simulations for typical Millburn residential areas. For the initial calculations, those values were subtracted from the monthly evapotranspiration (ET) values to obtain the monthly moisture deficits per month, and the daily deficits per house per day. These were the needed irrigation requirements in order to meet the ET deficit values compared to the typical rainfall that naturally infiltrates into the landscaped areas. This would be considered the minimum irrigation water needed and is a conservative value. As noted above, it is possible to disposal of excess stormwater to pervious areas with minimal harm to the vegetation (the main concept applied to the use of bioretention and rain garden facilities).

These minimum monthly irrigation needs were used in the model examining different storage volumes for water tanks or cisterns. The long term simulations examined the actual rain series to determine how much water can be stored in the tanks from the roof runoff and used for irrigation. Small storage volumes result in the tanks not being able to store significant amounts of the roof runoff (most of the roof runoff would overflow the tanks) and would be drained quickly after the rain. Large storage volumes would have excess storage capacity and would seldom overflow, but the irrigation needs may not be able to use all of the water. There is usually an optimal storage volume that is associated with maximum amounts of roof runoff use. As noted, in most cases, more water can be used for irrigation than indicated just by matching the ET deficits, so these tank sizes and the irrigation needs can be considered the minimum values.

Figure 7-15 is the input screen in WinSLAMM, version 10, used to enter the information for a continuous analysis for beneficial uses for different water storage tank characteristics. Figure 7-16 contains plots of the roof runoff reductions vs. roof runoff storage tank volumes for the Newark rain conditions and for silty soil conditions, the most common surface soil found in the Millburn study area for storage tank sizes ranging from very small 0.003 to very large 0.9 m (3 ft) of storage (volumes expressed as the depth over the roof area (3,500 ft^2); a 0.3 m (1 ft) storage volume corresponds to about 100 m^3 (3,500 ft^3) of storage for this example, or two large tanks about 3 m (10 ft) deep and 4.6 m (15 ft) in diameter). The 0.005 ft roof top storage volume corresponds to a total tank storage volume of about 0.5 m^3 (130 gal), or about four typical 35 gallon rain barrels. As noted previously, the outfall runoff reduction benefits are about one-third of the direct roof runoff reductions.

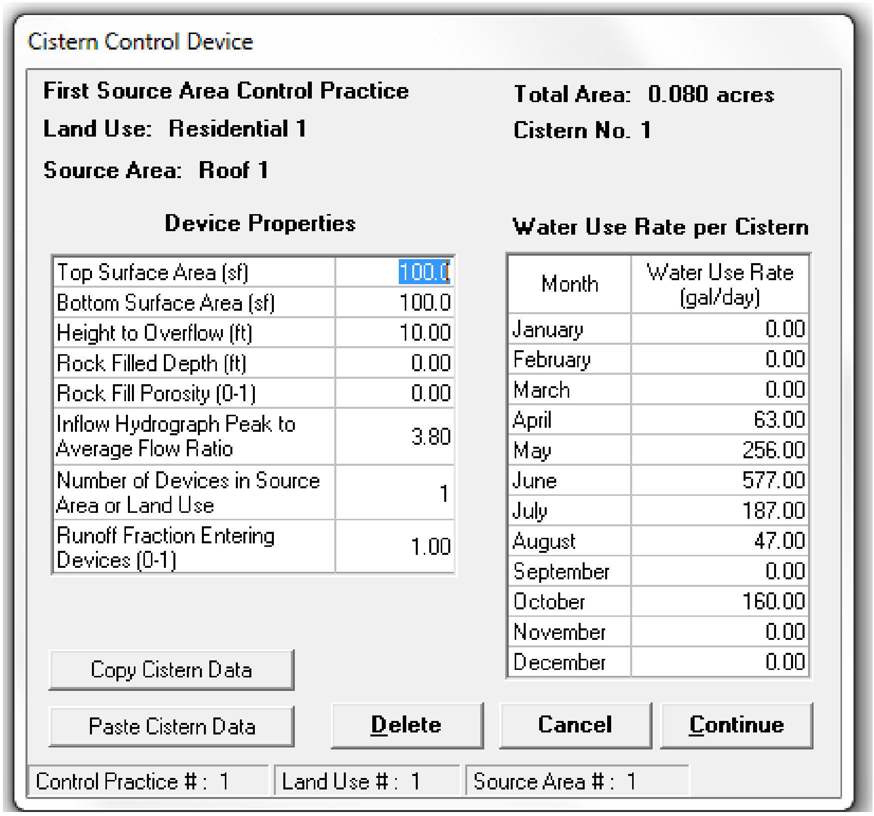


Figure 7-15. WinSLAMM, version 10, input screen for water tanks/cisterns.

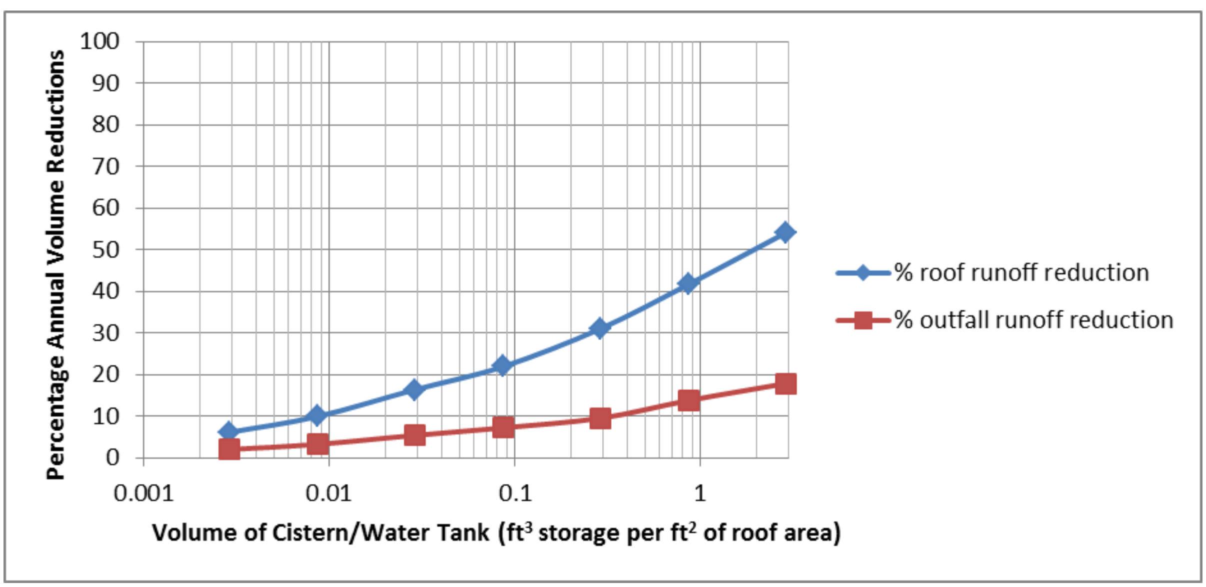


Figure 7-16. Roof runoff and water tank storage production function for Millburn Township residential areas (typical silty soil conditions).

Similar analyses for sandy soil areas result in lower levels of performance (about 50% for 0.6 m (2 ft) of storage) compared to the clayey and silty soils (about 60% for 0.6 m or 2 ft of storage) because more of the rainfall falling directly on the sandy landscaped areas contribute to soil moisture, resulting in less of an irrigation demand to match the ET deficits. Table 7-9 summarizes the results of these calculations for silty soil conditions for different areas of the US (Pitt and Talebi 2011). The Central and Great Lakes areas have the highest potential level of control because the ET demands best match the rain distributions. The East Coast, Southeast, and Southwest regions all have moderate levels of control due to poorer matches of ET and rainfall, or greater amounts of rainfall. The Northwest region has the poorest level of potential control, and large storage tanks are not likely to be very effective due to small ET-infiltration deficits.

Table 7-9. Roof Runoff Harvesting Benefits for Regional Conditions (Medium Density Residential Land Uses, silty soil conditions) (Pitt and Talebi 2011)

Region	total roof area (% of total residential area)	landscaped area (% or total residential area)	representative city for rain fall and ET values	study period annual rain fall (average in. per year) (1995 to 2000)	roof runoff control (%) for 0.025 ft^3 storage/ft^2 roof area (about 5 rain barrels per 1,000 ft^2 roof)	roof runoff control (%) for 0.25 ft^3 storage/ft^2 roof area (3 ft high by 6 ft diameter tank per 1,000 ft^2 roof)	roof runoff control (%) for 1.0 ft^3 storage/ft^2 roof area (two 6 ft high by 10 ft diameter tanks per 1,000 ft^2 roof)
Central	18.1	62.5	Kansas City, MO	33.5	40%	78%	90%
East Coast	15.9	54.5	Newark, NJ	53.0	24%	33%	42%
Southeast	8.8	81.1	Birmingham, AL	49.8	34%	41%	42%
Southwest	15.4	61.2	Los Angeles, CA	16.7	35%	44%	48%
Northwest	15.4	61.2	Seattle, WA	41.7	16%	16%	19%
Great Lakes	15.0	57.5	Madison, WI	28.7	46%	68%	72%

(1 ft = 0.305 m, 1 ft^2 = 0.093 m^2, 1 ft^3 = 0.028 m^3)

The ratios of roof areas to landscaped areas for medium density land uses range from 0.11 to 0.29 (average of 0.25); these ratios for low density land uses range from 0.05 to 0.23 (most at 0.11); while the ratios for strip commercial areas range from 1.8 to 4.0 (most at 2.3). Low density residential area irrigation uses would therefore have a greater potential benefit compared to the medium density areas, while the strip commercial areas would have much poorer potential benefits due to the lack of landscaped areas to irrigate and the relatively large roof areas contributing flows.

Figure 7-17 is a similar plot compared to Figure 7-16 but shows the irrigation needs to meet the maximum moisture needs of a heavily watered Kentucky Bluegrass lawn. The runoff reductions are much greater and reach 100% of the roof runoff (and 33% of the whole area runoff), but only for very large storage volumes.

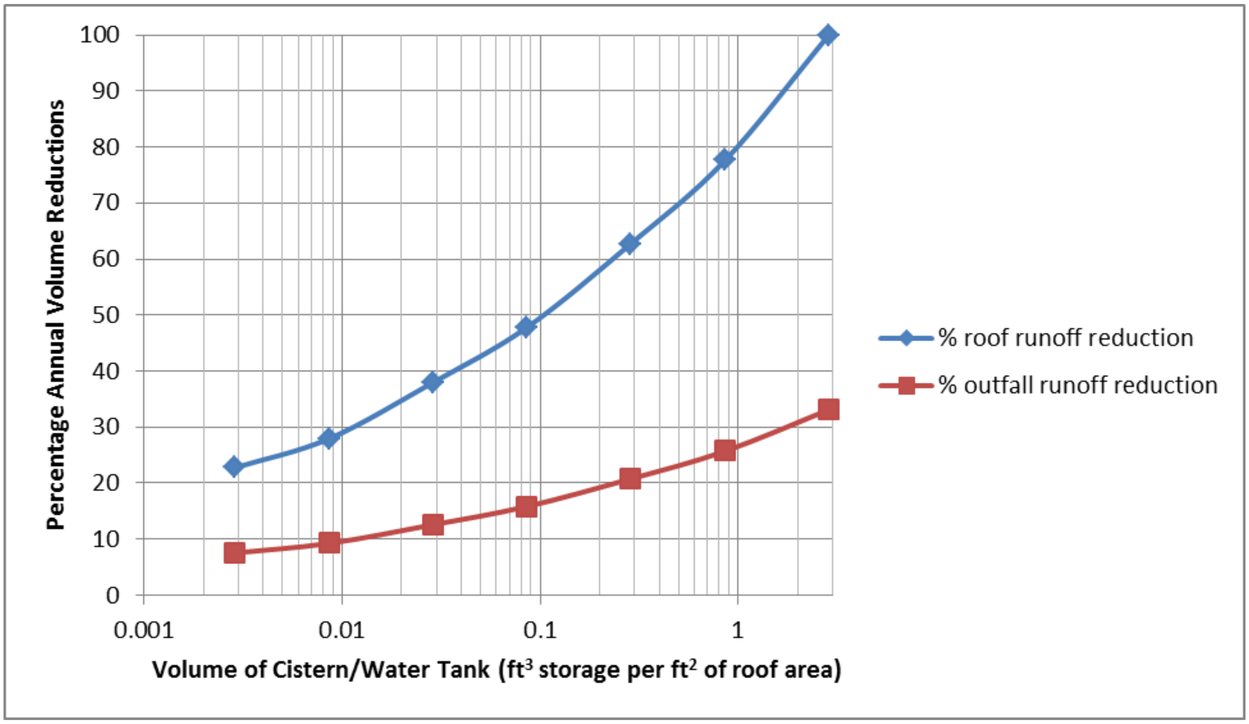


Figure 7-17. Water storage tank benefits for supplemental irrigation to meet heavily irrigated lawn deficits (Millburn, NJ). (1 ft^2 = 0.092 m^2)

A storage volume of 7.6 cm (0.25 ft, or 6,500 gal or a storage tank about 3 m (10 ft) high and 3 m (10 ft) in diameter) would therefore result in a roof runoff reduction ranging from 30 to 60%, depending on the irrigation rate actually used (from the minimum ET needs to the heavily irrigated lawn needs). This is much less than was shown possible by using the current dry well installations, but also results in significant domestic water savings.

Use of Cisterns for Irrigation of Roof Runoff in Conjunction with Dry Wells

It may be feasible to use a dry well in conjunction with a water storage tank. The water tank would be located next to the building and water withdrawn for beneficial irrigation use. Overflowing water from the storage tank would then be directed to a dry well. WinSLAMM, version 10, was used to calculate the simultaneous benefits of these controls operating together at Millburn residential areas. Figure 7-18 is a plot of the resulting reduction in roof runoff (total area reductions would be about one-third of these values). Subsurface infiltration rates of 6.4 to 76 mm/hr (0.25 to 3 in./hr) were examined in conjunction with storage tanks having 2.8 m^3 (100 ft^3) storage (0.03 ft^3/ft^2 of roof area), 8.5 m^3 (300 ft^3) (0.09 ft^3/ft^2), 1,000 ft^3 or 28.32 m^3 (0.26 ft^3/ft^2), and 3,000 ft^3 or 84.95 m^3 (0.86 ft^3/ft^2), along with no dry wells or no cisterns.

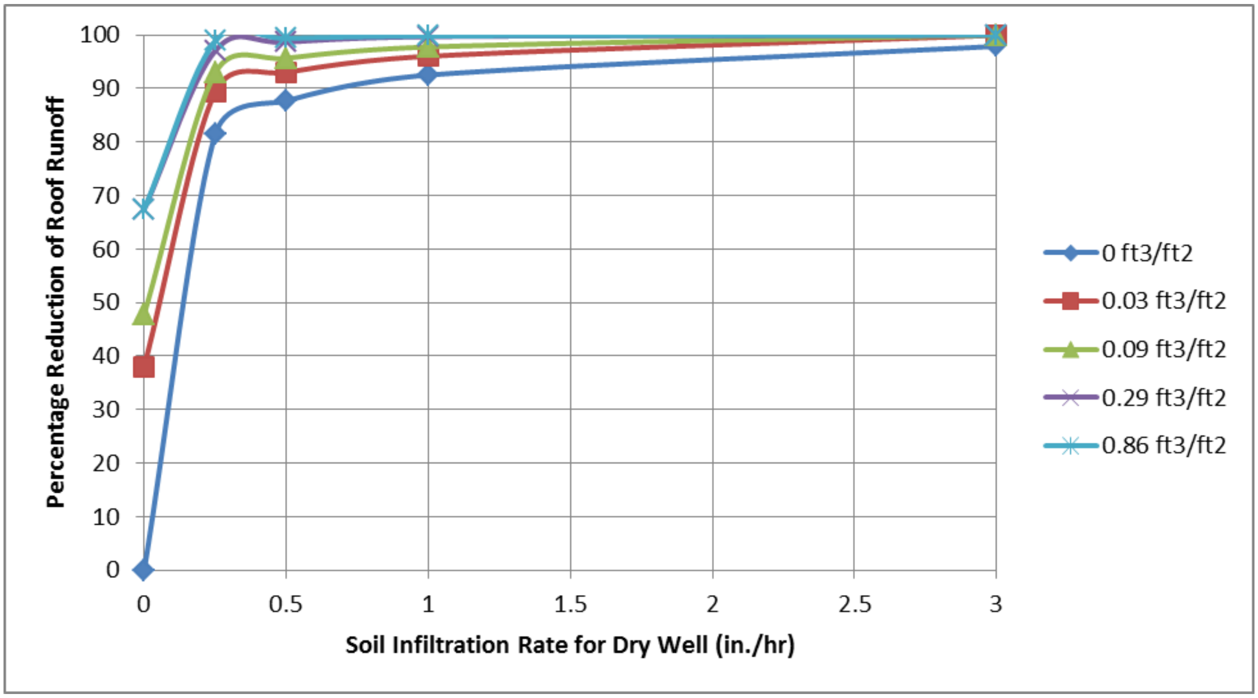


Figure 7-18. Production functions for cisterns and dry wells in residential areas, Millburn, NJ. (1 in./hr = 25.4 mm/hr)

As shown, the use of dry wells is much more effective than cistern use alone. Basically, maximum beneficial use of the roof runoff for irrigation does not require much storage at this location due to the closely matched irrigation needs and the rainfall pattern. With the conservative irrigation pattern (to only match the ET deficit after infiltration of the rainwater which would prevent any seepage of water to the subsurface), the maximum roof runoff irrigation use only reduces the study period runoff from from 26,000 m^3 to 21,000 m^3 (930,000 ft^3 to 750,000 ft^3) (about 20%). However, it is not unreasonable to over-irrigate the pervious areas for significantly increased runoff reductions as part of a stormwater management strategy (in contrast to restricting irrigation as part of a water conservation activity), as noted. The irrigation water applied that is excessive compared to the ET requirements would contribute to shallow groundwater recharge, typically a desirable benefit, as shown on this figure.

This analysis is a bit misleading because it implies that dry wells are much more effective stormwater runoff volume controls. However, dry wells also provide little, if any, protection to groundwater quality. In contrast, irrigation, and other surface stormwater applications to pervious areas, can provide significant pollutant reductions by treatment as it passes through the surface soils (treatment is provided by particulate trapping by filtration in the soil column and some dissolved pollutant reductions are due to ion exchange and sorption that can occur in surface soils that have a greater organic content than the subsurface layers).

The use of water storage tanks and irrigation also reduces the amount of sediment discharged to the dry wells, significantly lengthening the time before critical sediment loads would occur causing decreased infiltration. The decreased discharges of sediment to the dry wells are about one-third to one-half with concurrent use of cisterns.

Rain Gardens used in Millburn Residential Areas

A popular stormwater control for roof runoff is the use of rain gardens. These are relatively small planted areas (usually prairie plants having deep roots, but can be plain turf grass with almost the same performance) located near buildings that receive runoff from the roof. They are excavated to several feet deep and refilled with media mixtures, such as sandy soil, possibly having some organic amendments. The media selection can be based on water quality treatment objectives. There is a surface impoundment allowing water to pool for short periods before infiltration. Excess water is allowed to overflow to the adjacent lawn area. Rain gardens typically do not include underdrains (which tend to significantly short-circuit the infiltrating water). Rain gardens can be easily integrated into the landscaping around a home, but do require maintenance.

Figure 7-19 is an input screen for a rain garden in WinSLAMM, version 10. In these calculations, basic 11 m^2 (120 ft^2) rain gardens excavated to 0.9 m (3 ft) deep and back-filled 60 cm (2 ft), with a 230 mm (9 inch) surface ponding depth were used. The only outlet is a surface overflow located on the low end of the rain garden.

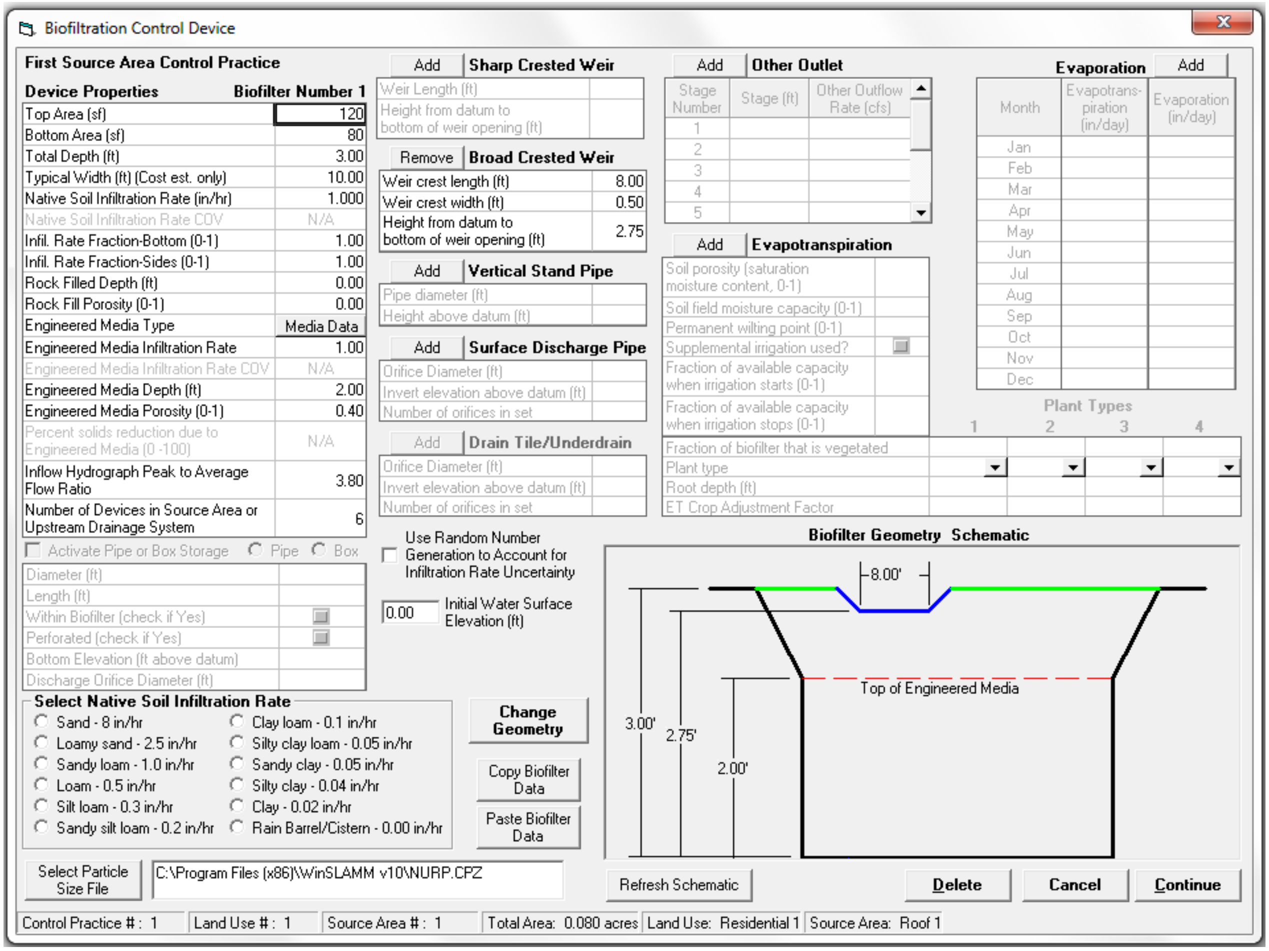


Figure 7-19. Input screen for rain gardens in WinSLAMM, version 10.

Figure 7-20 is a set of production functions for rain gardens treating runoff from impervious areas in Millburn. The plot is normalized showing the total rain garden area as a percentage of the contributing impervious (roof or paved area) area. Production functions are shown for the natural subsurface infiltration rates at the bottom of the rain garden, ranging from 6.4 to 76 mm/hr (0.25 to 3 in./hr). Rain gardens sized to be approximately 10% of the contributing impervious area are expected to infiltrate from 90 to 95% of the runoff, for all soils in the range of these infiltration rates. Rain gardens at the more common size of about 3% of the contributing area are only expected to infiltrate about 50 to 70% of the annual runoff. Rain gardens can work quite well for a range of soils, including those with relatively large amounts of clay, if they are sufficiently sized and the soils are not compacted.

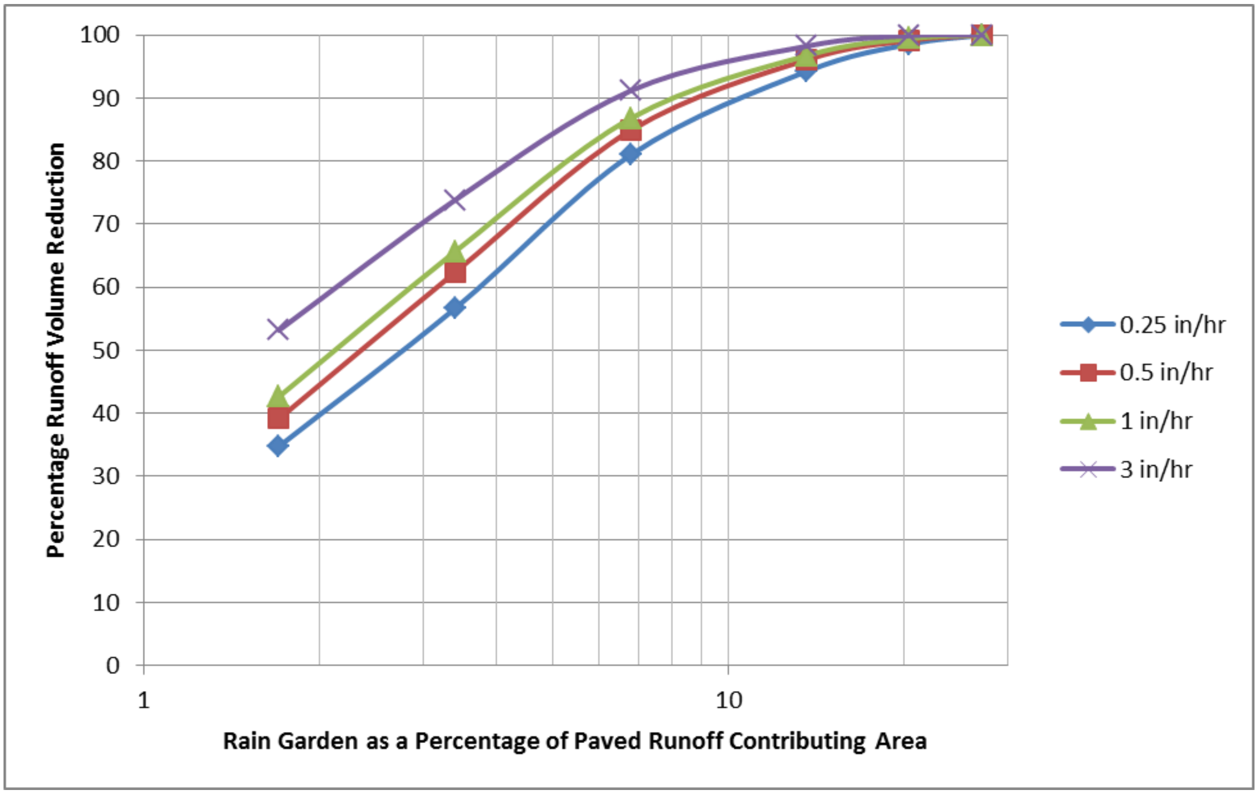


Figure 7-20. Rain garden performance for impervious areas for different soil infiltration rates, Millburn, NJ. (1 in./hr = 25.4 mm/hr)

Figure 7-21 shows the expected times before clogging of rain gardens receiving runoff from roofs, for rain gardens about 7% of the roof area (two 11 m^2 (120 ft^2) rain gardens for a 330 m^2 (3,500 ft^2) residential roof in Millburn). These critical times are quite long, being much longer than the minimum ten years for well-planted rain gardens. The plants are expected to incorporate the incoming sediment at this rate and their roots are able to keep sealing layers of silts and clays form forming. If the expected clogging period was only several years (such as for smaller rain gardens, of with more contaminated inflowing water), then premature clogging may occur. As an example, if the rain garden received runoff from roads or driveways, the critical clogging period may be about one-fourth as long (35 mg/L TSS for roofs vs. about 150 mg/L TSS for these other paved areas), resulting in five to ten year critical clogging periods that may cause problems.

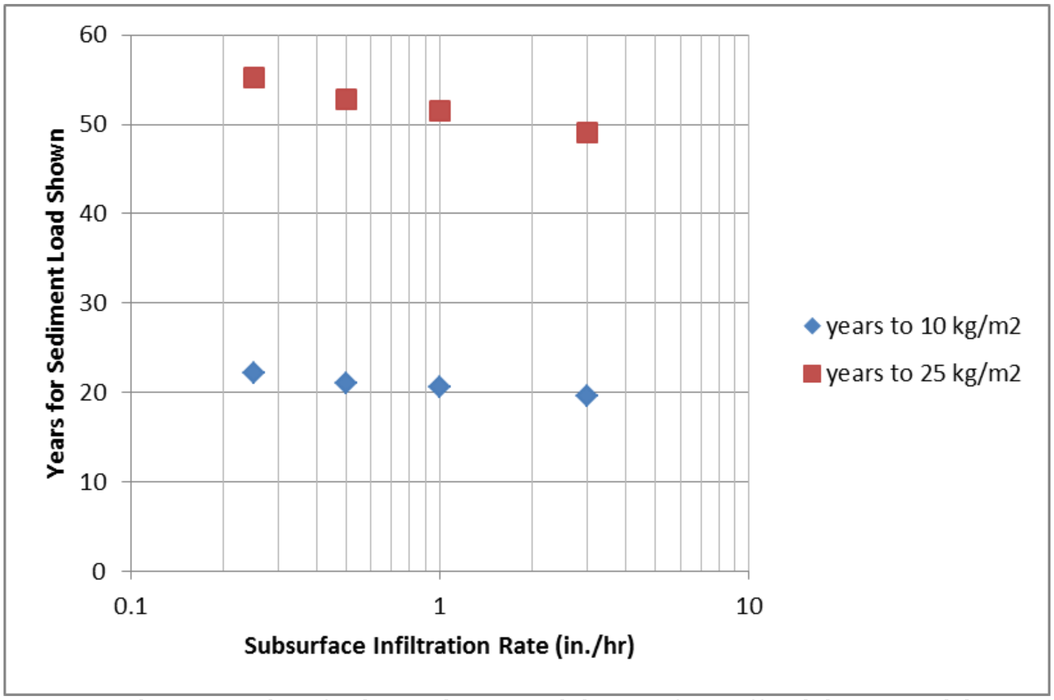


Figure 7-21. Clogging potential of rain gardens receiving roof runoff, Millburn, NJ (about 7% of the roof area). (1 in./hr = 25.4 mm/hr)

Summary of Stormwater Management Alternatives for Millburn

Dry wells may be a preferred option in cases that are allowed by the New Jersey dry well disposal regulations for stormwater which limits their use to areas having excellent soils (HSG A or B; although subsurface soils where the dry well is located should also be considered), where the groundwater table is below the dry well system (to prevent standing water in the dry wells and very slow infiltration), and to only receive roof runoff water (generally the best quality runoff from a site and the snowmelt from roofs would not be contaminated with deicing salts). However, irrigation beneficial uses of the roof runoff should be the preferred option, and in many cases may be less costly, especially considering increasing water utility rates and the desire to conserve highly treated domestic water supplies. Shallow groundwater recharge may be an important objective for an area, but “over” irrigation (beyond the plants ET deficit needs, but less than would produce direct runoff) would also contribute to that objective, at the same time as conserving water and offering better groundwater protection.

Reference evapotranspiration for the Millburn area ranges from about 0.04 mm/day (0.015 in./day) during January to about 4 mm/hr (0.16 in./hr) during May through July. The period of maximum ET also corresponds to the period of maximum rainfall in the area, reducing the need for irrigation (and also the sizes of long-term water tanks). Therefore, the beneficial use of roof runoff for irrigation is limited if it is only to meet the irrigation demand. However, irrigation can also be used as a stormwater management option with excess water being used to recharge the shallow groundwater and to meet the increased moisture needs of some heavily watered lawns (such as common Kentucky Bluegrass).

Rain gardens are another viable alternative for stormwater management in the Millburn area, especially as they provide some groundwater quality protection and can be incorporated into the landscaping plan of the site. They likely require additional maintenance; similar to any garden, but they can be placed to receive runoff from several of the sources areas on a site, increasing the overall stormwater management level. They have even been incorporated along roads, as curb-cut biofilters, resulting in significant overall runoff volume reductions (but with special care to prevent pre-mature clogging, reduced salt discharges, and appropriately sized to handle the large flow volumes).

It is obviously viable to use alternative stormwater options when dry well use should be restricted, such as with the following conditions:

- poor infiltration capacity of subsurface soil layers
- concerns about premature clogging or other failures due to sediment discharges or snowmelt discharges to dry wells
- seasonal or permanent high water tables
- concerns about groundwater contamination potential.

Chapter 8 Conclusions and Recommendations

Dry Well Performance Observations

The main purpose of this dry well study was to assist the Township of Millburn in understanding the effectiveness of the Town Dry Well Ordinance and to determine whether it should be modified and/or eliminated. The controversy arose due to complaints by home owners and contractors due to the costs of the dry wells and that the soil conditions around many of the dry wells prevented them from draining well, with water remaining in the dry wells several days after a rain event.

A recent tour of the community by members of the project team highlighted that in areas where the dry wells have been installed, the roof runoff from the properties is substantially less, or non-existent, compared to sites where no dry wells are installed. The lawns were greener and there was little or no evident erosion of the top soil. The sites where the dry wells have not been installed exhibited flooding, extended water ponding, and erosion of topsoil. The Township has several soil types and the lower permeability of the clayey soils impacted the ability of the dry wells to drain quickly. During the site investigations, the following observations were made:

- Dry wells with clay soils drain slowly. The Township has therefore modified the dry well requirements in areas having clay soils by requiring a deeper trench and filling it with more gravel. For example, one owner dug a 30 ft deep trench to capture more runoff at one location.
- On properties without dry wells, the soil is eroded and runoff coming down the driveways is intense. The water ponded on the lawns.
- There is a substantial savings of water that can be realized with the installation of cisterns, allowing the beneficial use of the captured water for landscaping irrigation. Cisterns combined with dry wells offer an effective means to maximize the effectiveness of stormwater treatment, especially in poorly drained soils.

Infiltration Rates and Drainage Times

Table 8-1 is a summary of the observed site conditions and infiltration characteristics at some of the monitoring locations. This table shows that there were varying levels of dry well performance in the area, but most were able to completely drain within a few days. However, several had extended periods of standing water that may have been associated with high water tables, poorly draining soils (or partially clogged soils), or detrimental effects from snowmelt on the clays in the soils.

Table 8-1. Summary of Infiltration Conditions at Several of the Test Locations

	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments	Approximate age of dry well system (years)	Ratio of contributing area to dry well volume (ft^2/ft^3)
11 Woodfield Dr.	89%	Consistent shape with time	Quickly drained (within a day); No standing water at any time	15 hr total drainage time during hydrant test	3 years	5.3 (driveway)
15 Marion Dr.	71%	Consistent shape with time	Several days to drain; No standing water at any time	4.5 days total drainage time during hydrant test	n/a	n/a
383 Wyoming Ave.	81%	Consistent shape with time	Several days to drain if full; No standing water at any time	1 day total drainage time during hydrant test	1 year	1.0 (likely incorrect) (lawn area)
258 Main St.	98%	Consistent shape with time	Very rapid drainage time; No standing water at any time		n/a	n/a
260 Hartshorn	10% (severely degraded)	Consistent shape with time	Slow drainage time (about a week if full), but dry if given enough time between rains	Clogging or poor soils, not high water table. Possible SAR issues in the Winter and Spring, recovered by mid-summer.	1 year	5.6 (impervious areas)
2 Undercliff Rd	79%	Consistent shape with time	Several days to drain if full; No standing water at any time	10 days total drainage time during hydrant test	n/a	n/a
87/89 Tennyson	0% (severely degraded)	Consistent shape with time	Very slow drainage time (a couple of weeks); standing water and never dry	Slow drainage may be due to saturated conditions, never reached stable low water level. If due to SAR, did not recover.	5 years	9.0 (impervious areas)
7 Fox Hill	2% (severely degraded)	Consistent shape with time	Slow drainage time (about a week or two if full), but dry if given enough time between rains	Clogging or poor soils especially in Spring, possibly SAR issues, not high water table	2 years	10.8 (entire property)

	% of time dry well was dry	Consistent shape with time?	Standing water after events?	Other comments	Approximate age of dry well system (years)	Ratio of contributing area to dry well volume (ft^2/ft^3)
8 So. Beechcroft	71%	Consistent shape with time for rains, but hydrant test (at end of periods at end of Sept) was very rapid	Quickly drained (within a day or two if full); No standing water at any time	3 hr total drainage time (half full) during hydrant test	3 years	101 (likely incorrect source areas) (several properties)
142 Fairfield	66%	Somewhat inconsistent shape with time	Quickly drained (within a day or two if full) to poorly drained (a week for moderate rains); Standing water during periods of large and frequent rains	Slowly drained conditions in Spring likely due to saturated conditions, or SAR. Not likely due to high water table	n/a	n/a
36 Farley Place	97%	Consistent shape with time	Very rapid drainage time; No standing water at any time		n/a	n/a

Figure 8-1 is a map showing these general conditions for Millburn. Most of the monitored dry wells were along a ridge between the two main drainages of the Township, with no obvious pattern of high water conditions, except that the high standing water dry wells were located along a line to the southwest along the ridge and are located fairly close to headwaters of streams (high water tables were noted in areas with nearby streams, but that was assumed to be in the larger stream valleys and not at the headwaters). The sites that had high standing water long after the events ended had substantially reduced infiltration rates. In the analyses, these rates were considered to be the constant (final) rates observed, with no initial rate data or first-order decay Horton coefficients used (relatively constant, but very low infiltration rates). Three of the sites shown in Table 8-1 had severely degraded infiltration conditions (260 Hartshorn, 87/89 Tennyson, and 7 Fox Hill). These sites all received runoff from the entire property or from multiple impervious areas (from 1 to 5 years old). It is not known if the source water or groundwater conditions affected the drainage conditions at these sites. Dry wells receiving runoff from all impervious areas would have a greater silt load and likely clog prematurely compared to sites only receiving roof runoff.

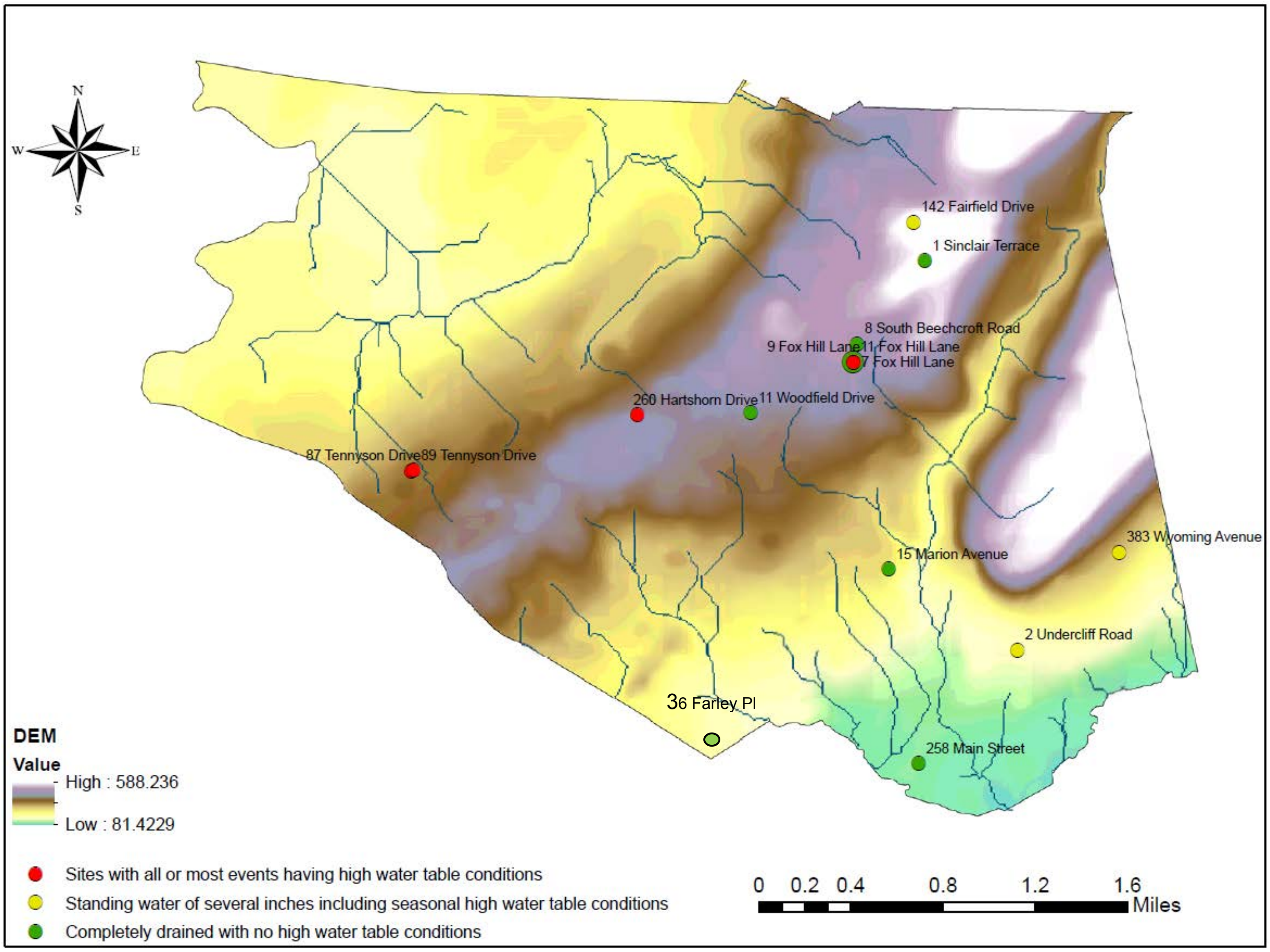


Figure 8-1. Township map showing locations having varying standing water conditions in monitored dry wells.

The infiltration rate characteristics were separated into three conditions:

- A and B surface soils and having well drained HSG A subsurface soils
- C and D surface soils and having well drained A and B subsurface soils
- C and D surface soils and having poorly drained subsurface soils with long-term standing water

Table 8-2 compares the observed Horton equation coefficients for the well-drained categories. The standing water data are not shown on this table as most of the observations could not be successfully fitted to the Horton equation. The almost steady infiltration rates (but with substantial variation) were all very low for those conditions and likely represent the f_c conditions only and were therefore included in that parameter category.

Table 8-2. Observed and Reported Horton Equation Coefficients

	f_o (in./hr)	f_c (in./hr)	k (1/min)
Surface A and B soils well drained A subsurface soils (average and COV)	**44.6 (0.53)**	**5.6 (0.2)**	**0.06 (0.22)**
Surface C and D soils well drained A and B subsurface soils (average and COV)	**4.3 (0.64)**	**0.45 (0.85)**	**0.01 (0.63)**

(1 in./hr = 25.4 mm/hr)

Even sites having surface C and D soils (not acceptable infiltration sites according to the New Jersey dry well standards) all had much better subsurface conditions where the dry wells were located than the surface conditions. The infiltration rates for these conditions were less than for the excellent areas having A and B surface soils, but all met the infiltration rate criteria of the state guidelines.

Dry Well Water Quality Observations

Water samples were collected at three dry wells and at one cistern during ten rains. The samples were analyzed for nutrients and heavy metals, and selected samples were also tested for pesticides and herbicides. The samples were collected directly below the dry wells (or at the inlet of the cistern) for comparison to water samples collected at least 0.6 m (2 ft) below the 0.6 m (2 ft) gravel layer beneath the dry wells (and in the cistern), for a total subsurface flow path of at least 1.2 m (4 ft) through the crushed stone and subsurface soil. Various statistical tests were used to compare the water quality from the inlet to the outlet locations to detect any significant differences due to operation of the dry wells.

Log-normal probability plots were used to identify range, randomness, and normality of the data and to determine what type of statistical comparison tests can be used. Figure 8-2 shows example paired log-normal probability plots for one of the sites (135 Tennyson Road, Millburn, NJ 07078) for different parameters including bacteria, nutrients, and COD. For these plots, most of the data are seen to overlap within the limits of the 95% confidence limits, indicating that the data are likely from the same population (no significant differences detected based on the number of samples available).

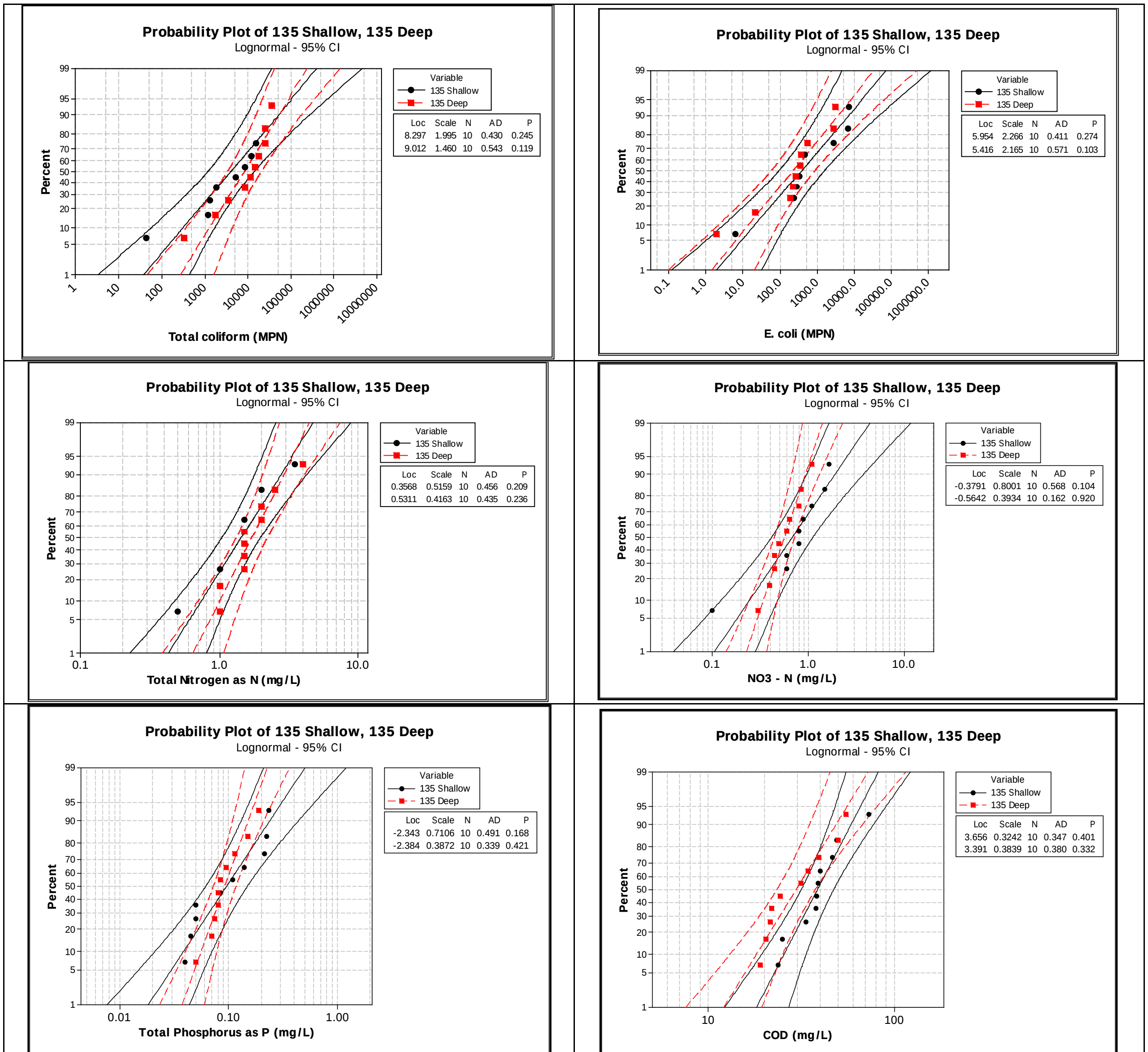


Figure 8-2. Log-normal probability plots for dry well samples located at 135 Tennyson Road (shallow vs. deep).

The Mann Whitney test was performed using MINITAB to test if the shallow samples have significantly higher or lower concentrations than the deep values (same comparison test for inflow vs. cistern).

Table 8-3 shows the Mann Whitney test results for comparison between the paired data. Except for the bacteria and COD results for the cistern site, all paired sample sets did not indicate significant differences for these numbers of samples. The cistern median total coliform values were greater than the inflow median values, indicating possible re-growth; however, the median *E. coli* and COD cistern values were less than the inflow values for these constituents.

Table 8-3. Summary of Mann-Whitney Test for Paired Data

	Parameter	79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Total Coliform bacteria	p-value	0.033	0.40	0.16	0.72
	Significant Difference Observes? (at level of 0.05)	Yes (but cistern median values were larger than the inflow median values)	No	No	No
E. coli bacteria	p-value	0.047	0.60	0.69	1
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No
Total Nitrogen as N	p-value	0.86	0.50	0.42	0.64
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
NO_3-N as N	p-value	0.14	0.24	0.15	0.77
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
Total Phosphorus as P	p-value	0.77	0.94	0.10	0.278
	Significant Difference Observes? (at level of 0.05)	No	No	No	No
COD	p-value	0.037	0.14	0.40	0.83
	Significant Difference Observes? (at level of 0.05)	Yes (cistern median values significantly less than the inflow median values)	No	No	No

Due to a large number of below-detection values for the metal analyses, a simple paired sign test was used to compare each paired set of data. Table 8-4 lists the results for the paired sign test for lead, copper and zinc data from the cistern and dry well samples. No statistically significant differences are seen between the sample sets for the heavy metals for the numbers of samples available.

Table 8-4. Summary of Paired Sign Test for Metals Analyses

Metal		79 Inflow vs. 79 Cistern	135 Shallow vs. 135 Deep	18 Shallow vs. 18 Deep	139 Shallow vs. 139 Deep
Lead	p-value	> 0.06	> 0.06	0.18	> 0.06
	Significant Difference in Medians?	No	No	No	No
Copper	p-value	0.13	*	>0.06	*
	Significant Difference in Medians?	No	*	No	*
Zinc	p-value	0.45	0.45	>0.06	>0.06
	Significant Difference in Medians?	No	No	No	No

***** All the results are BDL, therefore it is not possible to do a paired sign test

Comparisons of Observed Water Quality to New Jersey Groundwater Disposal Criteria

Table 8-5 lists the most stringent regulatory levels for groundwater contaminants derived from N.J.A.C. 7:9C (2010), along with the range of observed concentrations for each constituent during these tests. The microbiological and lead concentrations frequently exceeded the groundwater criteria.

Table 8-5. Groundwater Quality Criteria for the state of New Jersey compared to observed water quality from dry wells

Constituent	Groundwater Quality Criterion[1]	Observed Range [1]	Fraction of samples that exceed the criteria
Microbiological criteria[2]	Standards promulgated in the Safe Drinking Water Act Regulations (N.J.A.C. 7:10-1 et seq.)[3]	Total coliform: 1 to 36,294 MPN/100 mL *E. coli*: 1 to 8,469 MPN/100 mL	Total coliform: 63 of 71 samples exceeded the criterion for total coliforms *E. coli*: 45 of 71 samples exceeded the criterion for *E. coli*
Nitrate and Nitrite	10	0.0 to 16.5 (one sample had a concentration of 16.5 mg/L)	1of 71 samples exceeded the criterion for nitrates plus nitrites
Nitrate	10	0.1 to 4.7	0
Phosphorus		0.02 to 1.36	n/a
COD		5.0 to 148	n/a
Lead	0.005	BDL to 0.38	33 of 71 samples exceeded the criterion for lead
Copper	1.3	BDL to 1.1	0
Zinc	2.0	BDL to 0.14	0
2,4-D	0.07	Not Detected	0
2,4,5-TP (Silvex)	0.06	Not Detected	0

Constituent	Groundwater Quality Criterion[1]	Observed Range [1]	Fraction of samples that exceed the criteria
2,4,5-T	0.7	Not Detected	0
Aldrin	0.00004	Not Detected	0
Alpha-BHC	0.00002	Not Detected	0
beta-BHC	0.00004	Not Detected	0
delta-BHC		Not Detected	0
gamma-BHC (Lindane)	0.00003	Not Detected	0
alpha-Chlordane		0.00003	0
gamma-Chlordane		0.00002 to 0.000024	0
Dieldrin	0.00003	Not Detected	0
4,4'-DDD	0.0001	Not Detected	0
4,4'-DDE	0.0001	Not Detected	0
4,4'-DDT	0.0001	Not Detected	0
Endrin	0.002	Not Detected	0
Endosulfan sulfate	0.04	Not Detected	0
Endrin aldehyde		Not Detected	0
Endrin ketone		Not Detected	0
Endosulfan-I	0.04	0.000032 to 0.000034	0
Heptachlor	0.00005	Not Detected	0
Heptachlor epoxide	0.0002	0.00003 to 0.000035	0
Methoxychlor	0.04	Not Detected	0
Toxaphene	0.002	Not Detected	0

[1] Ground water quality criteria and observed range are expressed as miligrams per liter (mg/L) unless otherwise noted.
[2] Pursuant to prevailing Safe Drinking Water Act Regulations any positive result for fecal coliform is in violation of the MCL and is therefore an exceedance of the ground water quality criteria.
[3] 50 MPN/100 mL

Statistical analyses indicated that the differences in water quality between the shallow and the deep samples were not significant (p-values were > 0.05). However, significant differences were found ($p< 0.05$) between the quality of inflow samples and cistern samples for total coliforms (possible re-growth), *E. coli*, and COD. These findings indicate that the dry wells do not significantly change the water quality for most of the stormwater constituents. If the influent water quality is of good quality, the dry wells can be a safe disposal method for stormwater quality. However, the bacteria and lead concentrations exceeded the groundwater disposal criteria for New Jersey and may require treatment, if the aquifer is critical.

Summary of Alternative Stormwater Management Options for Millburn

Dry wells may be a preferred option in cases that are allowed by the New Jersey dry well disposal regulations for stormwater which limits their use to areas having excellent soils (HSG A or B; although subsurface soils where the dry well is located should be the consideration), where the groundwater table is below the dry well system (to prevent standing water in the dry wells and very slow infiltration), and to only receive roof runoff water (generally the best quality runoff from a site and the snowmelt from roofs would not be contaminated with deicing salts). However, irrigation beneficial uses of the roof runoff should be the preferred option (possibly used in conjunction with the dry wells), and in many cases may be less costly, especially considering increasing water utility rates and the desire to conserve highly treated domestic water supplies. Shallow groundwater recharge may be an important objective for an area, but “over” irrigation (beyond the plants ET deficit needs, but less than would produce direct runoff) would also contribute to that objective, at the same time as conserving water and offering better groundwater protection.

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It is obviously viable to use alternative stormwater options when dry well use should be restricted, such as with the following conditions:

- poor infiltration capacity of subsurface soil layers
- concerns about premature clogging or other failures due to sediment discharges or snowmelt discharges to dry wells
- seasonal or permanent high water tables
- concerns about groundwater contamination potential.

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Appendix A. Primary and Follow-up Report Questions

Primary Report Questions

The Township concludes that the use of dry wells on private property where the increased runoff originates eliminates the cost to the municipal government, while only slightly increasing the cost to the property owner. Other objectives include the recharge of the shallow groundwater, increased water conservation, decreased volumes and/or delaying the delivery the stormwater entering the municipal drainage system, and decreases of impacts on local streams and rivers. Observations by Township personnel indicate that this strategy has reduced both flooding and soil erosion within the Township. Because of the lower municipal costs associated with the implementation of this approach, Federal, State and local agencies are asking:

- Are the implementation activities working?
- What is the impact of the effectiveness in various soil types?
- Is it more important to address roof runoff, versus other runoff sources such as driveways and patios, etc.?

In addition to these basic questions, other issues that the Township would like addressed by this study include:

1. Are there any maintenance requirements needed for the dry wells?
2. What is the life cycle of the dry wells?
3. Do the efficiencies of the dry wells change with time, and are there any differences in their effectiveness in different soil types over time?
4. What is the impact to long term maintenance requirements for the storm sewers?
5. What are the impacts on the water table and on the local water supply?
6. What is their impact on groundwater quality?
7. Is erosion of the top soil reduced by directing the stormwater runoff to the dry wells versus letting the stormwater runoff drain across properties?
8. Does directing stormwater runoff to the dry wells filter and improve the quality of the stormwater?

Answers to these questions (presented below) and more are needed to support the development of watershed implementation plans, to motivate stormwater control practice implementation by stakeholders, and to ensure the vitality of the cost share programs to retrofit properties in areas of extreme soil erosion promoting water conservation.

1) Are the implementation activities working?

At most of the monitoring locations, the dry wells are draining quickly and completely after rains (within a day or two for full dry wells). However, some locations experienced standing water for extended periods and would be considered to not be working as intended. The likely reasons for these failures are discussed in some of the following question responses. Basically, more careful site evaluations and design, along with better control of the source waters entering the dry wells, are needed. In critical situations, alternative stormwater controls should be considered.

2) What is the impact of the effectiveness in various soil types?

Originally, it was thought that the surface soil characteristics would have little effect on the performance of the dry wells, as they are subsurface devices and most of the water is percolating from the dry wells at depth and not near the surface. At all of the Millburn test locations, the subsurface soils had better infiltration characteristics compared to the surface soils; in fact, the subsurface soils were all HSG A or B, which meet the State's dry well design standards, even though the surface soils were mostly HSG C or D soils. The measured infiltration rates from all of the dry wells meet the minimum rates specified by the State's design guidance, but there were substantial variations, as noted below (average infiltration rates for typical storm durations):

- A and B surface soils and having well-drained HSG A subsurface soils (190 mm/hr or 7.6 in./hr)
- C and D surface soils and having well-drained A and B subsurface soils (43 mm/hr or 1.7 in./hr)
- C and D surface soils and having poorly-drained subsurface soils with long-term standing water (20 mm/hr or 0.8 in./hr)

Generally, the lowest infiltration rates associated with long-term saturated conditions averaged about 12 mm/hr (0.5 in./hr). Again, all of these rates satisfied the State's design guidance. However, several sites had long-term standing water and never drained completely, while other locations required several weeks to drain (and seldom were the dry periods long enough to allow complete drainage).

Therefore, even though the site conditions met the design guidance, some locations still had standing water. It is likely that seasonal (or possibly long-term) high water tables occurred at some of the locations. The lack of site specific groundwater elevation information did not allow this to be verified, but the performance of some of the drain-down curves supports this finding.

In other cases, the rates appeared to vary by season, with some incidences of standing water. These mostly occurred in the spring and sometimes in the winter. It is thought that soil chemistry changes due to snowmelt waters entering the dry wells from non-roof areas were responsible for these observations. De-icing chemicals would likely be heavily applied near home walkways, porches, steps and driveways (but not roofs!). If this water was allowed to enter the dry wells and if there was some clay in the surrounding soils, sodium adsorption ratio (SAR) imbalances would disperse the clays and cause significant reductions in the infiltration rates. In most cases, excess sodium would be rinsed from soils over a few months, partially restoring the infiltration conditions. However, problems would continue to reoccur with subsequent saline snowmelt discharges to the dry wells.

Therefore, the sites that had sandy surface and subsurface soils (HSG A and B soils) performed the best. It was thought that the other sites would also perform well having good subsurface soils, but their characteristics were not likely as good as the other locations (even in the same soil category), and the likely presence of small to moderate amounts of clay would be more sensitive to SAR problems.

3) Is it more important to address roof runoff versus other runoff sources such as driveways and patios, etc?

Roof runoff contributes about one-third of the average annual runoff for the Millburn residential areas, a typical value compared to other residential areas in the US. However, the roof runoff only contributes about 11% of the TSS. Other source areas, such as driveways on private property are significant contributors to runoff. Streets also make up about one-fourth of the runoff. Patios are not very significant as their runoff is mostly already directly to the landscaped areas where most of it can infiltrate. After roofs, driveways should be controlled (such as by using rain gardens near the lower ends of the driveways near the streets, as many are steeply sloped from the house to the roads). Dry wells for driveways (or other paved areas besides roofs) should not be considered due to the much greater sediment load that would likely cause premature failure by clogging.

In addition to these basic questions, other issues that the Township wanted to be addressed by this study were:

4) Are there any maintenance requirements needed for the dry wells?

It is difficult to maintain the dry wells as they are buried. The bottoms are open and resting on crushed stone allowing the penetration of silts and clays into the voids. These materials cannot be easily removed. However, leaves and other vegetation debris on top of the crushed stone could be removed without disturbing the rocks. Many of the dry wells have grated openings allowing surface runoff from the surrounding areas to directly flow into the dry wells. During construction, erosion sediment may enter the dry wells which would significantly hinder their performance. As noted, organic matter from the surrounding areas can also directly enter the dry wells through these surface openings. It is recommended that only directly connected roof leaders enter the dry wells (in compliance with the state regulations) and that the dry wells be inspected and superficially cleaned periodically. Leaf filters should also be installed on the roof gutters or leaders or capture these material before they are discharged into the dry well. If needing maintenance to remove the silts and sands from the crushed stone, all of the crushed stone would have to be removed and replaced, a costly option similar to totally rebuilding the dry well. Prevention is therefore key to long-term satisfactory performance.

5) What is the life cycle of the dry wells?

If only receiving roof runoff, the dry wells should function for years (at least five to ten years, likely much longer). However, if sediment is allowed to enter them, their life can be shortened considerably. There is little data for older dry wells in the area, so their full expected life cannot be accurately estimated, although there is some indications of past failed dry wells that have been replaced since they have started to be used in the area.

6) Do the efficiencies of the dry wells change with time, and are there any differences in their effectiveness in different soil types over time?

Again, the short time of use of the Millburn dry wells does not allow any analysis of their performance with time. The sites investigated during this study did identify several problems in areas having marginal surface soils and high standing water. It is not known if these issues changed with time, but more careful site evaluations and the use of alternative stormwater controls in areas having questionable conditions would result in better functioning stormwater management for the Township.

7) What is the impact to long term maintenance requirements for the storm sewers?

The decreases in runoff volumes likely have little effect on the maintenance of the storm sewers (roof runoff has little sediment, for example). Decreased flows could reduce the flushing actions in the storm sewers, but this detrimental effect is not expected to be significant for the storm sewers (maximum 30% reductions in overall flows associated with complete roof runoff removal, but only a portion of the roofs in the area are being controlled in the Township). However, slower rates of increases of runoff with increased development will result in decreased need for expansion of the storm sewers.

8) What are the impacts on the water table and on the local water supply?

Increased recharge of the shallow groundwater will result in a rising of the water table. However, this will be most significant very close to the dry wells and is not expected to be widespread. If all increased stormwaters in the Township were infiltrated, then this effect could be widespread. If dry wells were located near buildings, there is the potential for seepage of water into basements. However, the dry wells studied were mostly located large distances from the buildings. The bigger concern is the potential effect of high water tables on the dry wells themselves, with resulting greatly reduced infiltration capacities.

9) What is their impact on groundwater quality?

Roof runoff has few contaminants and should therefore be the preferred source of water directed to the dry wells (although bacteria may be a problem, and other pollutants may periodically be a concern, especially if zinc and copper materials are used on the roofs). The dry wells had no significant benefit on the quality of the stormwater, based on the limited sampling conducted during this study. If other source waters enter the dry wells (such as driveways and streets), groundwater contamination would be a much greater concern. The best quality waters in residential areas and most suitable for direct infiltration is roof runoff.

10) Is erosion of the top soil reduced by directing the stormwater runoff to the dry wells versus letting the stormwater runoff drain across properties?

Complete infiltration of the roof runoff results in about a 30% reduction in the runoff volume (and flow rate), which results in reductions in energy of the flowing water and erosion. Concentrated runoff from roof downspouts or outlets near bare soil near the streets causes erosion of soils. With infiltration, this is obviously reduced.

11) Does directing stormwater runoff to the dry wells filter and improve the quality of the stormwater?

No, the dry wells provide no significant water quality benefit. Because the roof runoff is the least contaminated water in the area, removing this component actually will cause a small increase in the concentrations of the stormwater pollutants from the whole area. The mass discharges of the pollutants will decrease however, and as noted above, there is a possible decrease in erosion hot spots that would result in decreased TSS levels in the site runoff.

Follow-up Questions Pertaining to the use of Millburn Township Dry Wells

The following are follow-up questions that were asked during the review of the draft report. Summary answers follow each question that are based on information obtained during the research of the Millburn Township dry wells. Some of these questions directly pertain to the scope of the work of the project and can be answered based on the project activities, while others are new, follow-up issues associated with the report conclusions and are beyond the scope of the current project. Some of these questions also cover the same issues as the above questions.

1) Evaluate the ordinance that was created by the Township of Millburn to control erosion and flooding.

A discussion of the local ordinance is in Section 4. The local Millburn Township ordinance should be modified to allow dry well use only in areas already having good stormwater quality (such as would be expected for most roofs), or require suitable pretreatment, such as effective grass filtering. In addition, the local ordinance should also prohibit dry well use in areas having seasonal or permanent high water tables, as those conditions result in long-term standing water in the dry wells. If located in areas having poorly draining subsurface soils, the dry well designs need to be modified (greatly enlarged) to account for the more slower draining conditions. It is recommended that dry well use be restricted to roof runoff, and alternatives that infiltrate water through surface soils (such as rain gardens) be used to treat driveway and parking lot runoff (or in areas having shallow groundwater). Irrigation of landscaped areas using roof runoff (and pretreated paved area runoff) is also a suitable option that also provides economic benefits to the land owner and should be encouraged by the ordinance.

2) Observe if the existing dry wells are working and whether a long-term maintenance program is valid.

As noted in Section 5, the dry wells are "working" in that most are capable of removing significant fractions of the stormwater runoff. However, they are not working well when one considers the lack of water quality treatment (as discussed in Section 6), or potential long-term problems due to clogging when runoff from areas besides roofs is directed to the dry wells. Maintenance will require replacement of most of the crushed stone in the bottom of the dry wells and trapped silt. This will be needed frequently in locations where runoff from eroding areas and most impervious areas is discharged to the dry wells. Periodic inspection programs can be used to identify dry wells with standing water. These will need maintenance, unless the standing water is associated with high water tables.

3) Will the use of the stormwater model, determine the full reduction of stormwater flow and the impact on local streams and the drainage system?

The WinSLAMM model calibrated for the area was used to examine existing areas, as described in Section 7, as part of the investigations of alternative management options. Unless controls are used in existing areas to treat existing flow sources, there can be no reduction in flows and impacts. With treatment of flows from newly developed areas, the treated increment will result in less severe increases in flows.

4) Use actual field data to determine if there are any improvements in water quality due to the installation of the dry wells.

As discussed in Section 6, there were no observed improvements in water quality in the stormwater after passing through the dry wells, the gravel blankets, and at least 2 ft of soil. As noted in that section, overall reductions in flows and flow energies associated with increased infiltration will decrease channel erosion rates and pollutant transport compared to conditions with no treatment. However, since the dry wells are not retrofitted to treat existing flows, there will be no change in existing channel conditions; the rate of degradation should decrease, however, if infiltration controls are used to partially infiltrate new flows.

5) Can the existing design of the dry well systems be improved to maximize their effectiveness, such as in areas where the soil characteristics are poor should the depth the dry well be increased, or whether cisterns should be recommended over dry wells or should a system of a combination of cisterns and dry wells be used?

Dry wells used in areas having marginal soils need to be enlarged to correspond to the reduced infiltration rates. Areas that discharge to dry wells should be restricted to roof runoff. Alternatives are discussed in Section 7 and include cisterns and irrigation beneficial uses, and rain gardens for other impervious areas. Shallow (but increased surface areas) dry wells can be used in some areas with shallow groundwaters, but that may be an unreliable option compared to the others described.

6) Is it a good idea to recharge the water from the lawn and/or tee driveways? Should the water from these areas be filtered?

These areas should be treated using rain gardens in preference to dry wells, as discussed in Section 7. Infiltrating the water from impervious areas through surface soils traps many of the stormwater pollutants to protect the groundwater and can handle the silt load from these source waters much better than dry wells. Deeper soils (such as those deep under the dry wells) have much less treatment abilities and the silts are retained in the voids of the crushed stones, eventually clogging the dry well bottoms. Rain gardens should be sized to be 5 to 20% of the impervious drainage area, depending on the soil characteristics. Pretreatment of the runoff ("filtering") is discussed in Section 4 and is much more than simple straining of leaves and large debris (though that should be worthwhile). Removal of silts requires substantial pretreatment to preserve dry well performance and to protect groundwater quality.

7) Should the inlet of the dry wells have some type of filtering system to increase the longevity of the dry wells or cisterns?

See #6 above. Section 4 discusses pretreatment methods for cistern use. Removal of leaves and other material will be needed and only roof runoff should be directed to the storage cisterns. Any pretreatment, including the removal of leaves and other debris, will increase the longevity of the dry wells.

8) Should the roof drainage be separated from the other runoff?

Definitely, as discussed in Section 4, roof runoff has the least contaminants that will clog dry wells and cause potential groundwater contamination. The NJ State groundwater disposal regulations restrict the flows to dry wells to only roof runoff. The local ordinance needs to be made in compliance with the state regulations. The other site flows should be treated with rain gardens or reused through storage in cisterns.

9) Are there other alternative designs that could be considered?

This has been addressed previously (enlarge for poor soils, shallower for shallow groundwaters, only roof runoff).

10) How would you recommend the ordinance be modified?

This has been addressed previously (only roof runoff to dry wells, encourage beneficial uses for irrigation and rain gardens for other site area infiltration).

11) Are there any seasonal variations that could be used to maximize the operations of the dry wells?

No snowmelt should be allowed to enter dry wells if contaminated with deicing materials (another reason to prohibit driveway and other impervious area runoff from being discharged to infiltration areas).

12) Are there any proposed changes in the type of vegetation that would improve stormwater retention?

This was not directly addressed in this report, as vegetation is not a component of dry wells. Vegetation issues were discussed in relation to evapotranspiration (ET) in Section 7 and in Appendix D of the report. Plants requiring large amounts of water can be encouraged in order to better utilize the runoff from sites in order to keep the cistern tanks smaller. They should also be able to withstand periods of no irrigation. For use in areas of increased infiltration (such as in rain gardens), local agencies usually have a plant list that works well for stormwater management (usually native plants with deep roots). However, regular sod can also be used in rain gardens with substantial benefits, usually requiring less specialized landscaping maintenance.

13) With the average reconstruction of homes estimated between 1.5 to 2 percent for the next ten years, what are the anticipated reductions in stormwater runoff and the anticipated improvement in water quality?

Unfortunately, unless all of the additional runoff associated with the new developments is infiltrated to match natural conditions, there will be an increase in runoff and a decrease in water quality. Only retrofitted infiltration and other stormwater controls in existing developed areas can decrease current levels of degradation. However, if one compares future development conditions with and without runoff controls, then enhanced infiltration can have significant benefits compared to future conditions without controls, but there would still be degradation compared to current conditions. Section 7 shows that the current Millburn residential landscaped areas are the largest single land cover, at about 61%, but only contribute about 12% of the runoff volume and 24% of the particulate solids contributions over a long time series. The directly connected roofs are the single largest runoff contributor (at 33%), with the streets the next most important contributor (at 27%). Driveways contribute a surprisingly large portion of the runoff (at 18%). For stormwater controls only affecting the roofs, the maximum outfall runoff reduction would therefore be about 33%. If driveways and parking areas could also be controlled on site, the maximum outfall benefit could increase to about 60%. It is difficult to reduce street runoff on private property, especially in an area having relatively steep front yards, as in the Millburn area, and runoff from landscaping areas can only be reduced by enhancing the soil structure (which may be possible during construction, but difficult after construction).

14) What is the cost comparison to treating stormwater with dry wells versus a large municipal project?

Comparative cost analyses were not within the scope of this project. Local dry well costs are reported in Section 2 (about $4,000 per dry well). Section 7 also discusses irrigation use cost savings. Large municipal projects for stormwater management can include the components of the drainage infrastructure and regional stormwater controls. The incremental costs associated with larger drainage systems if no on-site infiltration controls are used may not be as important as the costs associated with increased flooding damage in low lying areas. The dry wells (and other on-site controls) are more effective for small and intermediate sized rain events, with some benefits for the large drainage events. The EPA has numerous reports describing direct and indirect stormwater management costs and benefits, especially the supporting information included in the *Federal Register* for new stormwater regulations.

15) What are the potential savings in water consumption with the use of cisterns and what would be the average savings to the resident in annual water bills versus the added costs of a cistern system over dry well system?

As noted above, detailed cost and benefit analyses were not part of this project. However, Section 7 discusses some of the economic features of cistern storage and beneficial uses of stormwater. Some of the homes in Millburn have very large water utility charges during the summer for landscaping irrigation (approaching $1,000 per month). The large cistern and irrigation systems used for these large homes are costly (about $25,000 to $50,000), but there is a significant positive payback after several years. In areas where the water costs and the water needs are less, the payback may not be as rapid.

16) Are there any draw backs in raising the water table by installing the dry wells?

Raising the water table has mixed effects in an area. In many urban locations, the water tables are depressed compared to natural conditions due to increased runoff of rainfall and decreased infiltration. Dry wells (and other infiltration practices) may raise the water table in an area, but the effects are usually localized. Some concerns are expressed due to increased basement flooding if infiltration occurs near buildings. In areas of existing high water tables, the performance of infiltration devices may be hindered. Saturated flow conditions are orders of magnitude less than typical infiltration rates and dramatically decrease the performance of dry wells if mounding intersects the infiltration zone under the dry wells. Groundwater mounding below infiltration devices should therefore be evaluated for an area, but is seldom an issue for relatively deep water tables (about 10 ft or more). Regional water table elevation increases can occur over an area if stormwater infiltration is widespread. Again, problems would occur in areas currently experiencing shallow groundwaters. Infiltration should not be encouraged in areas of shallow groundwaters. Most regulations prohibit infiltration unless the water table is at least 3 ft below the bottom of the infiltration device.

17) What are the economic benefits in reducing the amount of flooding and erosion after the installation of the dry wells? Did the model show the improvements?

The evaluation of regional flooding and erosion economic issues was beyond the scope of this project. Section 7 examined alternative stormwater management options on-site and for the region, but cost estimates are not provided.

18) How can the model be used as a tool for Millburn and the surrounding communities as a model in mature urban settings to treat stormwater?

WinSLAMM was calibrated for the eastern US based on municipal NPDES information as provided in the National Stormwater Quality Database. Site-specific development conditions for Millburn were obtained from Township data sources, aerial photographs, and maps. The model was used to calculate the benefits and limitations of many different stormwater management options. The calibrated model and associated files also can be used in surrounding communities to evaluate many stormwater options. The evaluation of the Millburn dry wells is also expected to be applicable to the surrounding areas, but site-specific soils, groundwater conditions, development characteristics, and topographic conditions need to be considered.

In order to most accurately design dry well installations in an area, actual site observations of the expected infiltration rates should be used instead of general literature values. This is especially true for surface infiltration devices (such as rain gardens), where compaction will have a much greater effect than on the deeper subsurface soils. Also, all of the sites in this study had improved infiltration characteristics with depth compared to expected surface conditions; in other cases, this may not be true. Criteria based only on surface soil conditions are likely not good predictors of deeper dry well performance. Luckily, county soil surveys do have some subsurface soil information that was found to be generally accurate during this study. Unfortunately, shallow water table conditions are not well known for the Millburn area and that characteristic can have a significant detrimental effect on dry well performance.

Appendix B. Descriptions of Millburn, NJ, Study Sites

Plans and Topographic Maps

This appendix contains several example site plans showing the residential area development characteristics and dry well calculations.

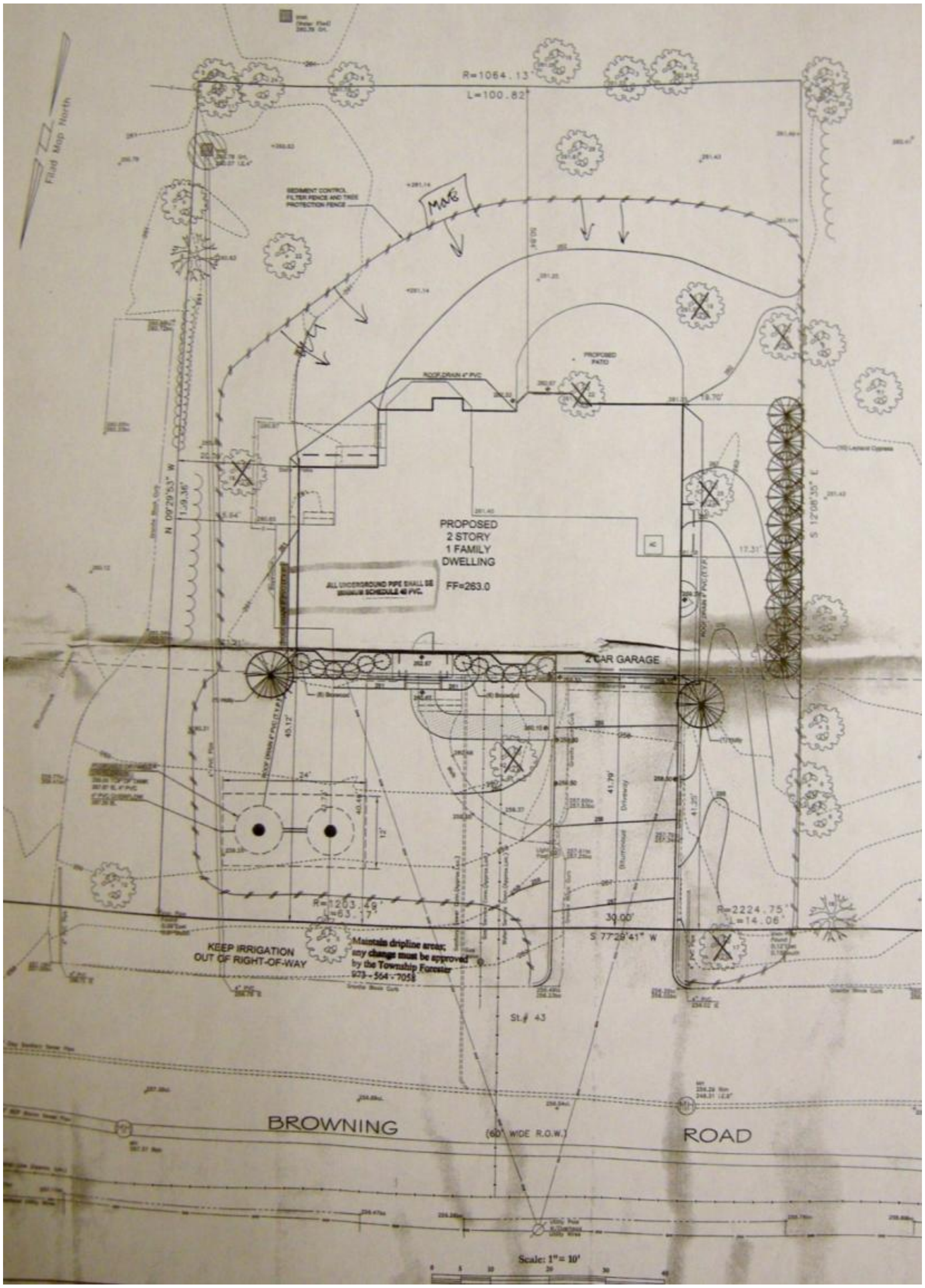


Figure B-1. 43 Browning Road S.H

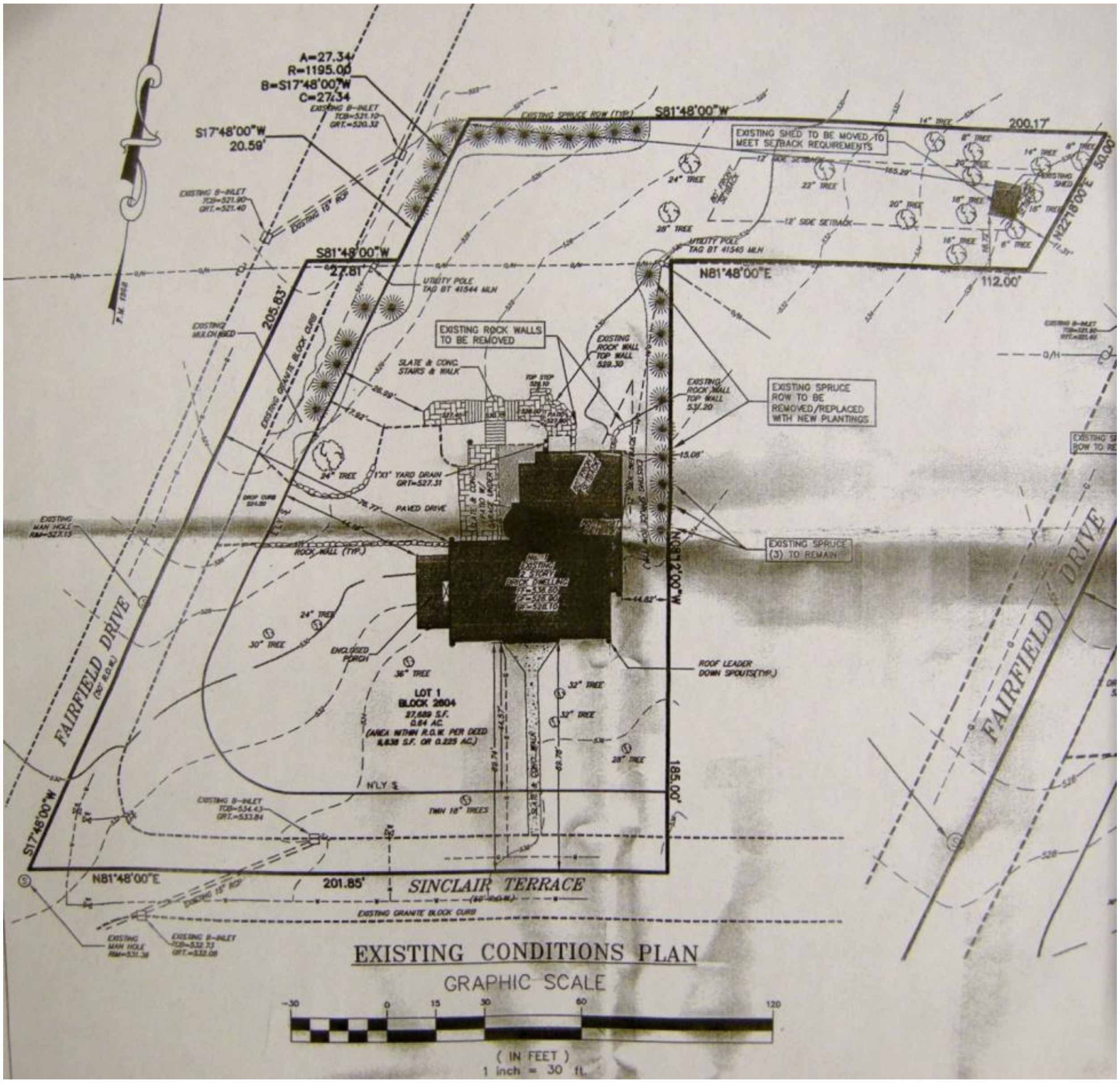


Figure B-2. 1 Sinclair Terrace

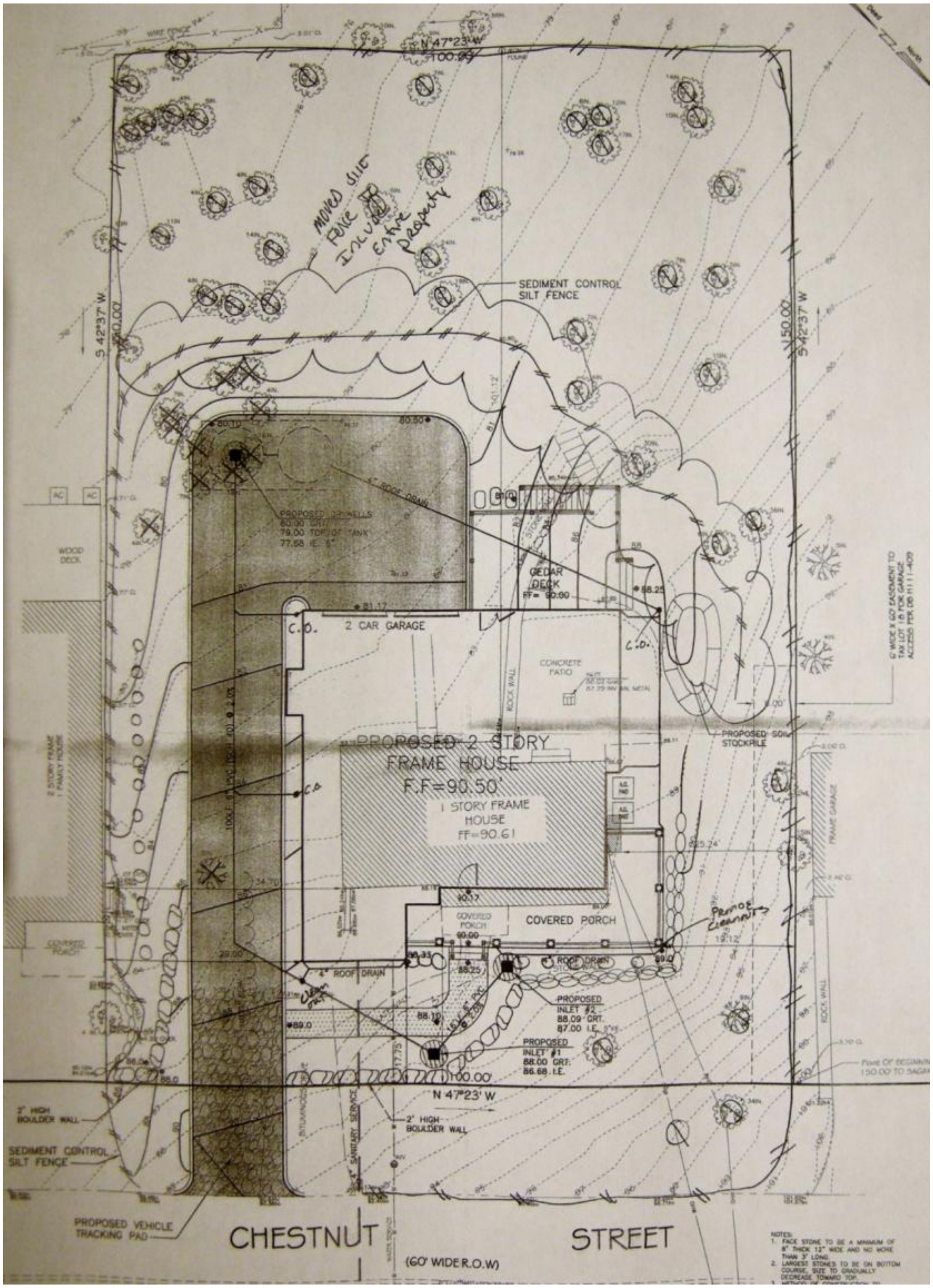


Figure B-3. 90 Chestnut St

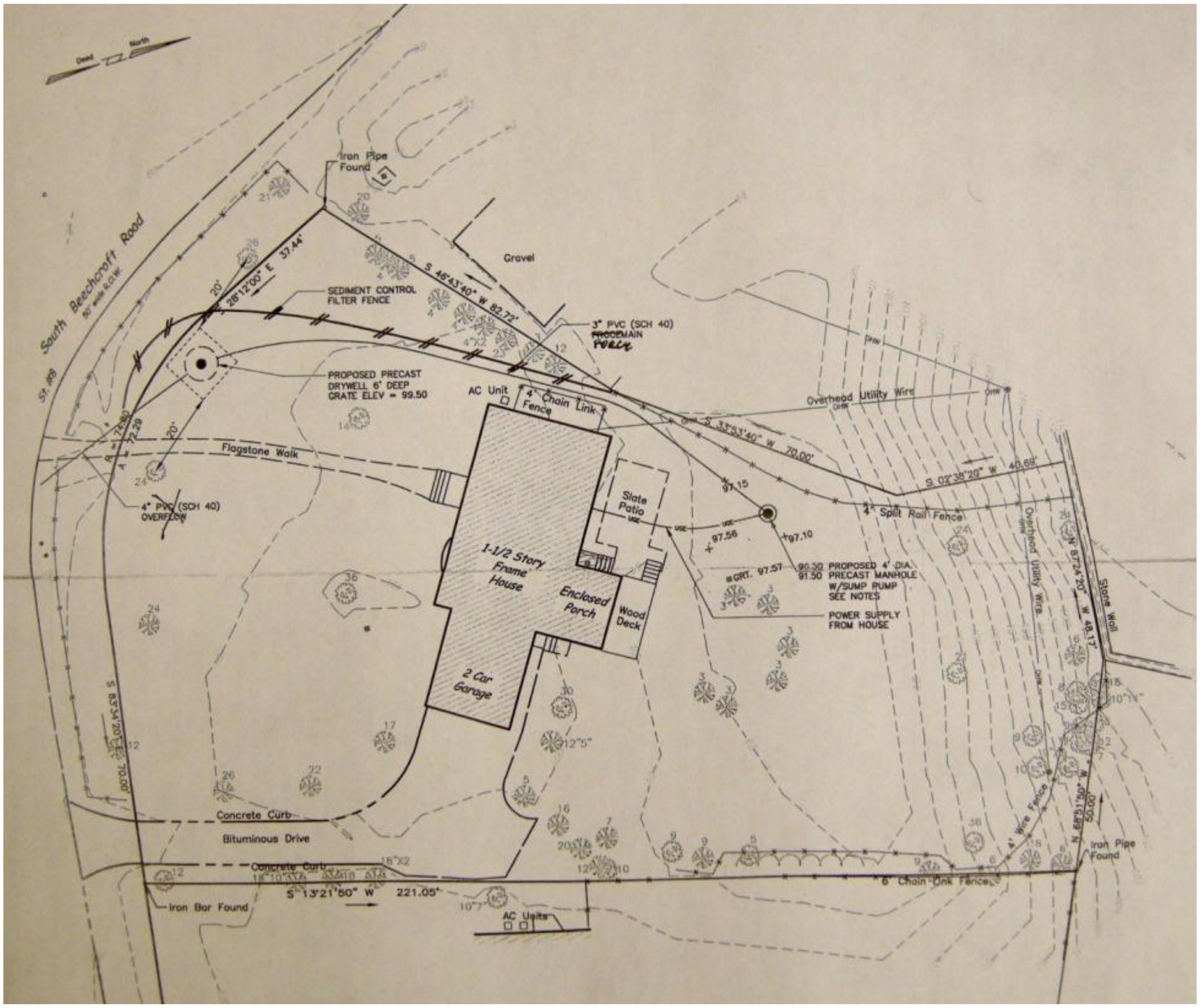


Figure B-4. **8 South Beechcroft**

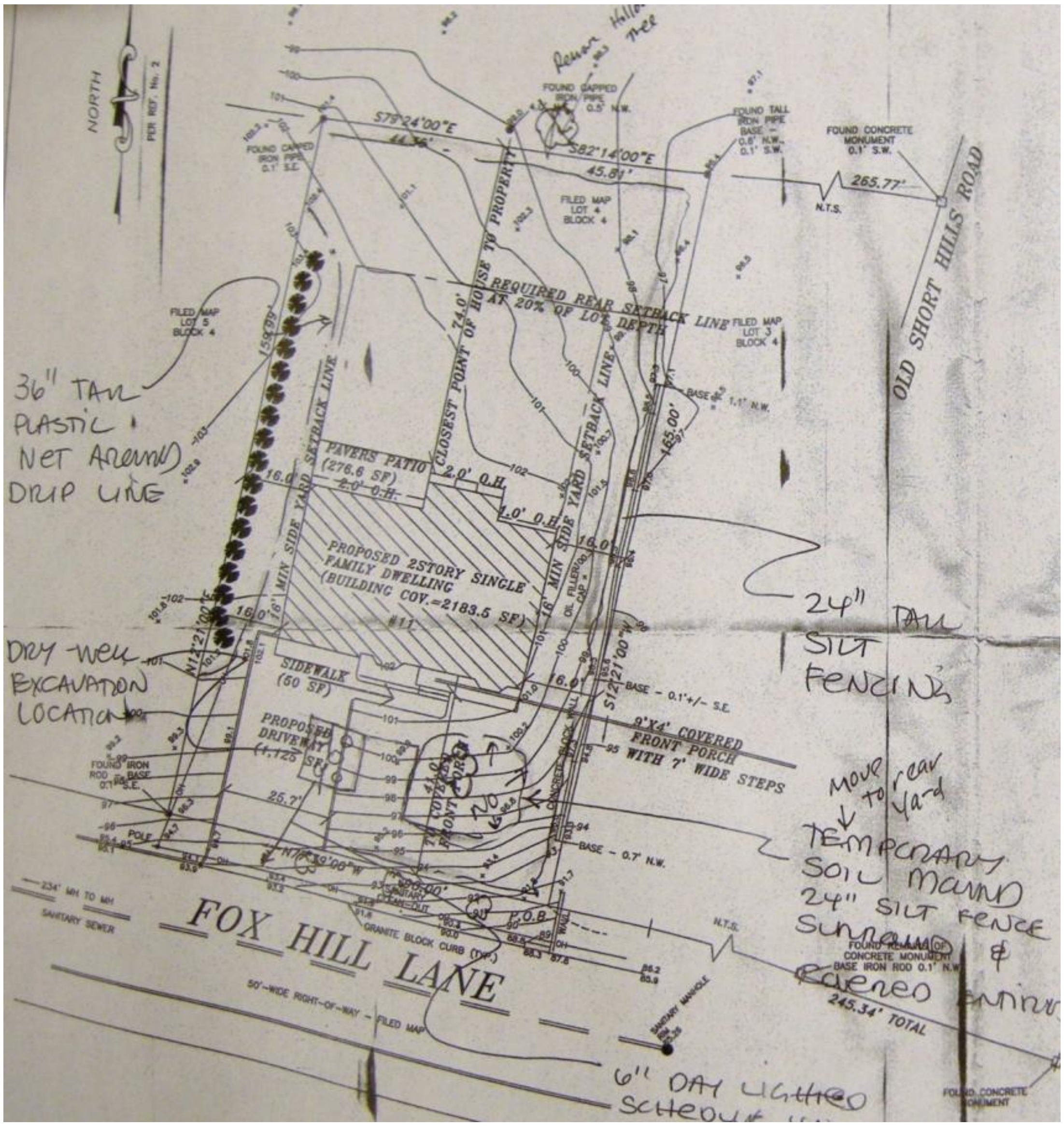


Figure B-5. 11 Fox Hill Lane

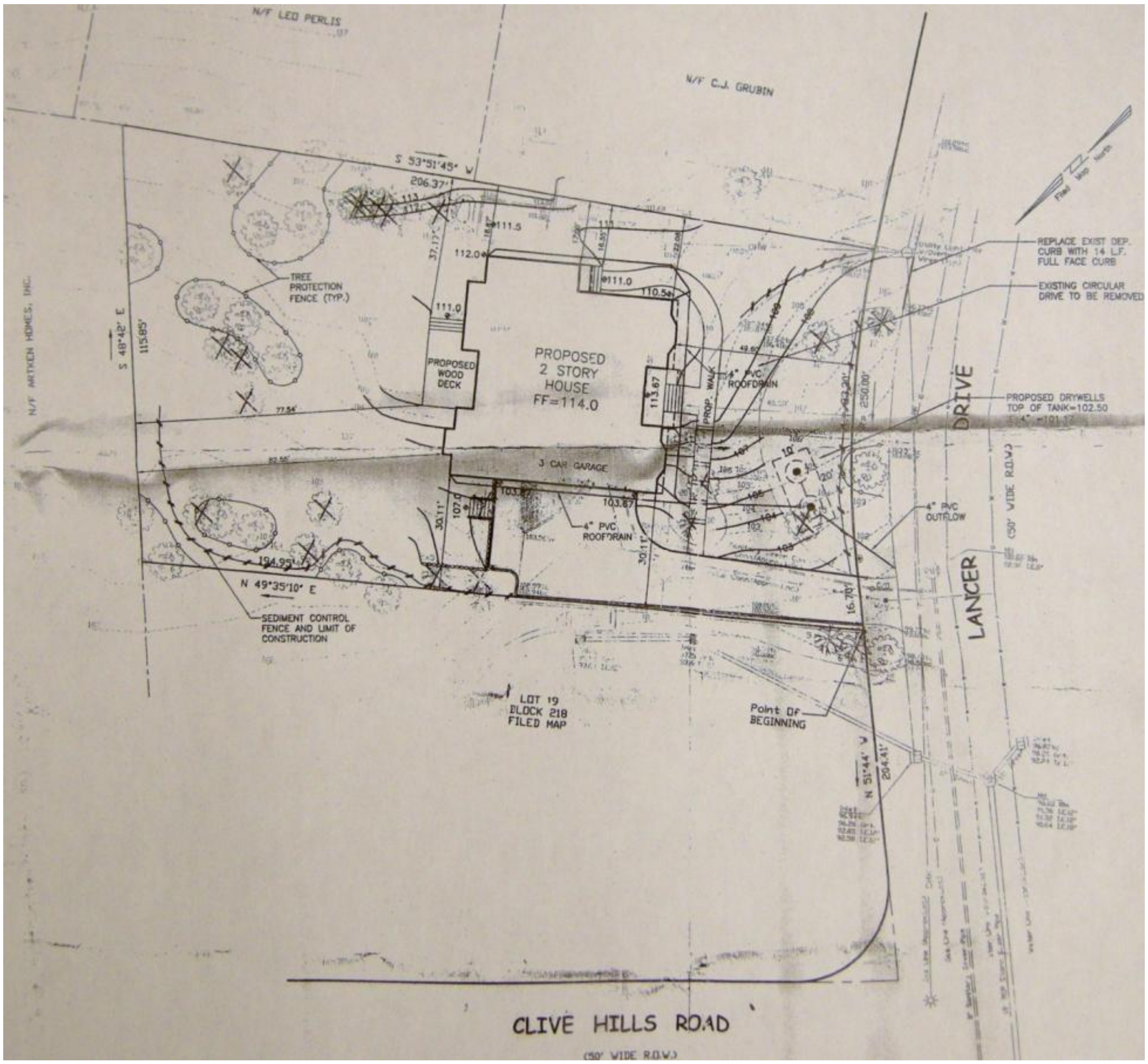


Figure B-6. **9 Lancer**

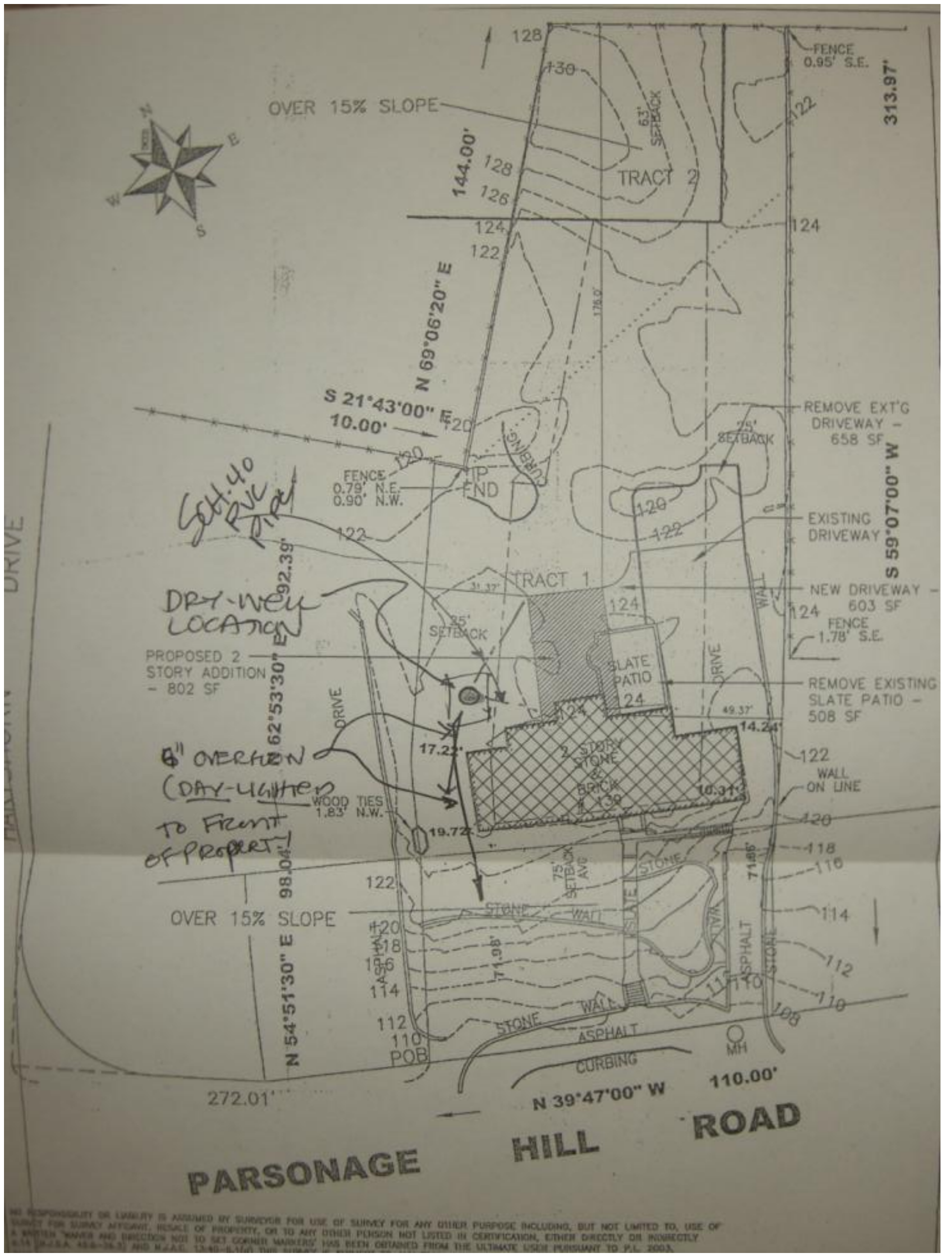


Figure B-7. 139 Parsonage Hill Rd

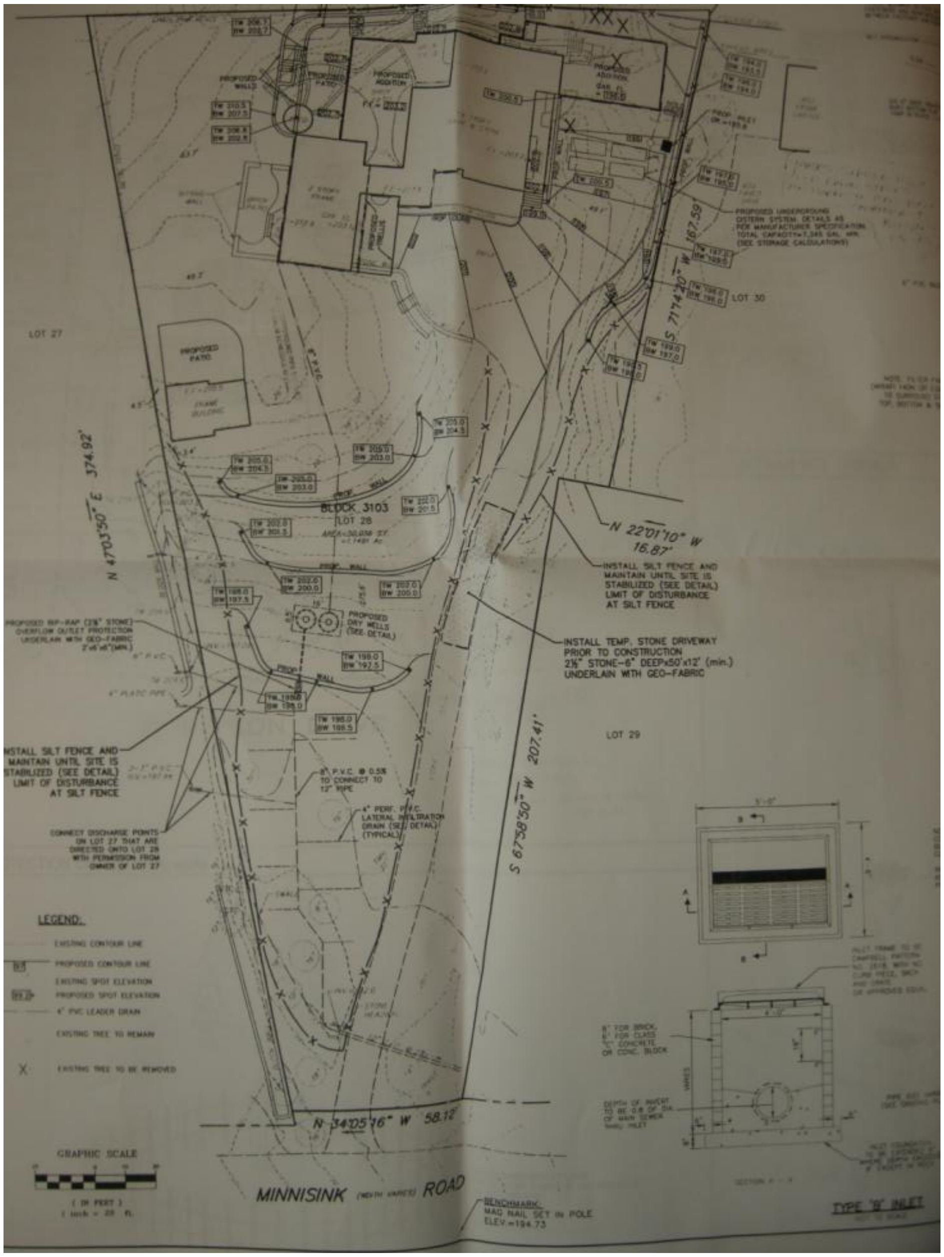


Figure B-8. 79 Minnisink Rd

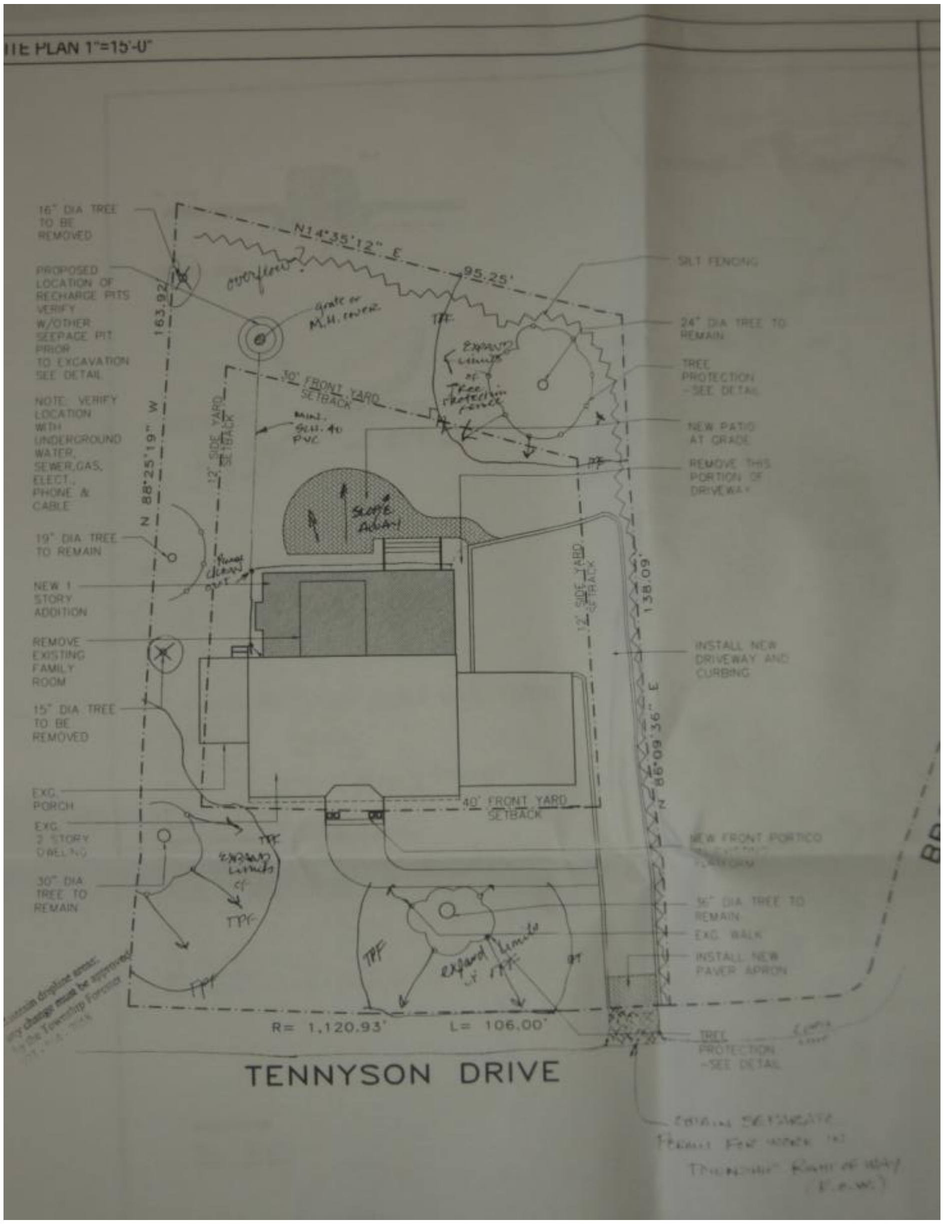


Figure B-9. 135 Tennyson Drive

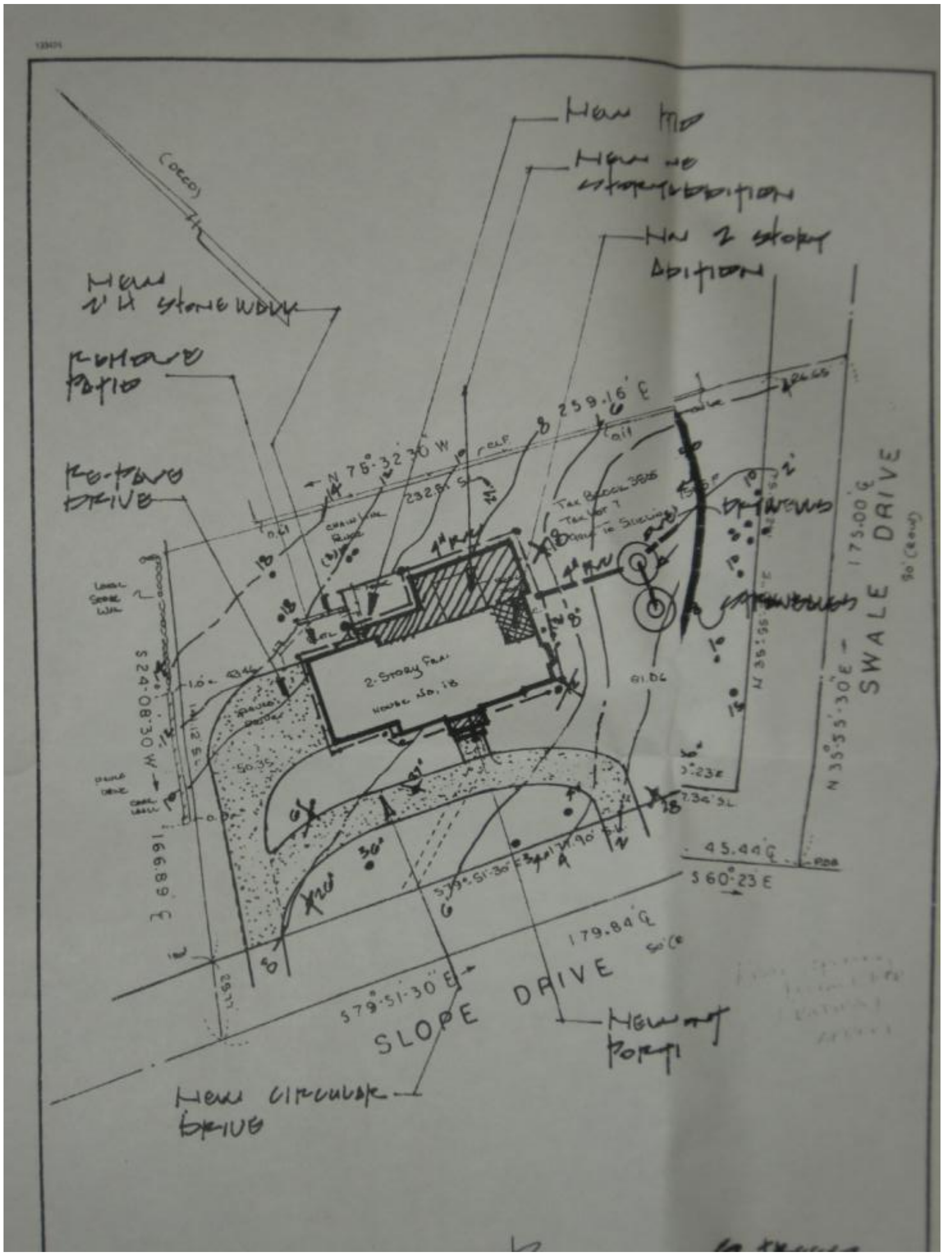


Figure B-10. 18 Slope Drive

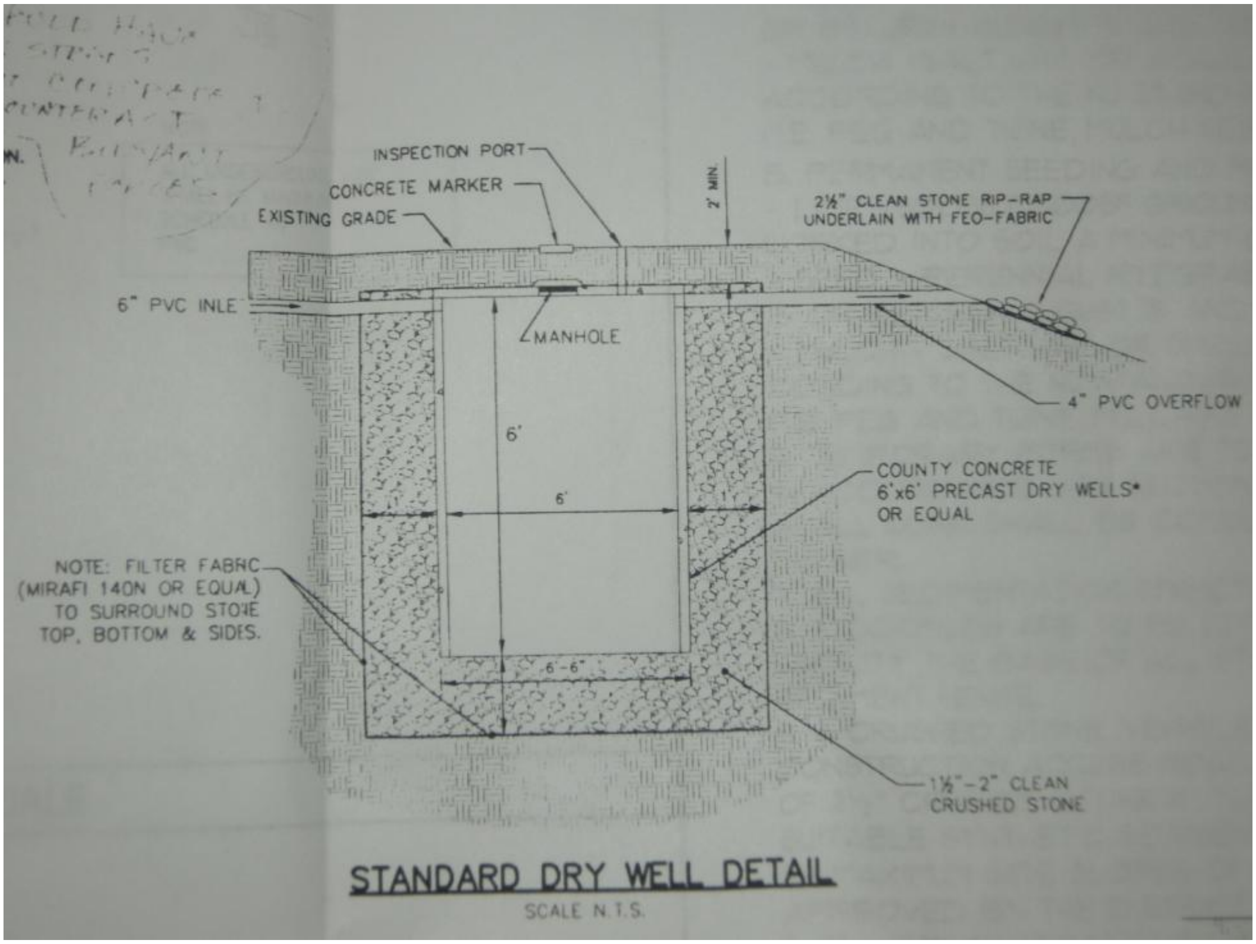


Figure B-11. Details of dry well - 79 Minnisink Rd

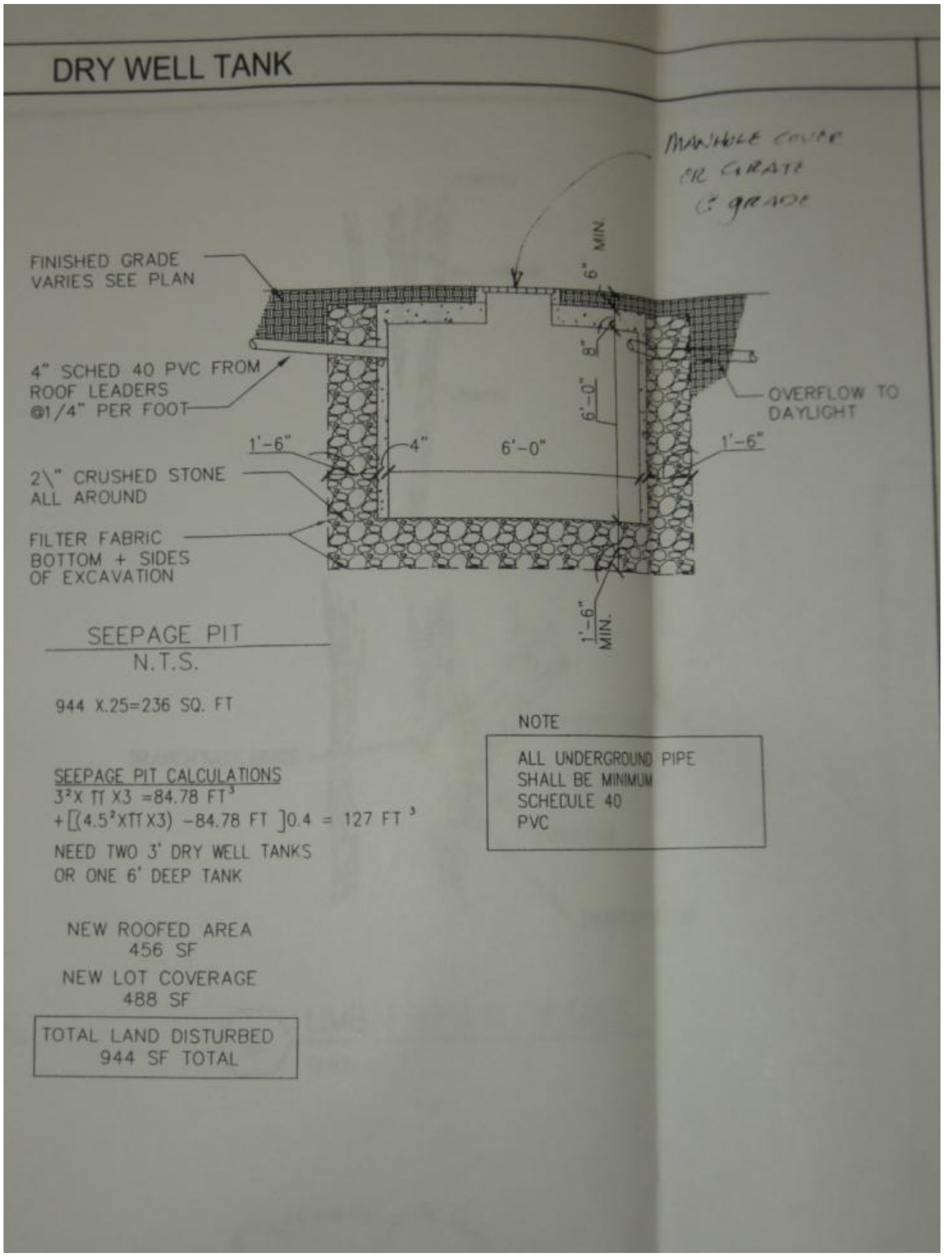


Figure B-12. Details of dry well – 135 Tennyson Drive

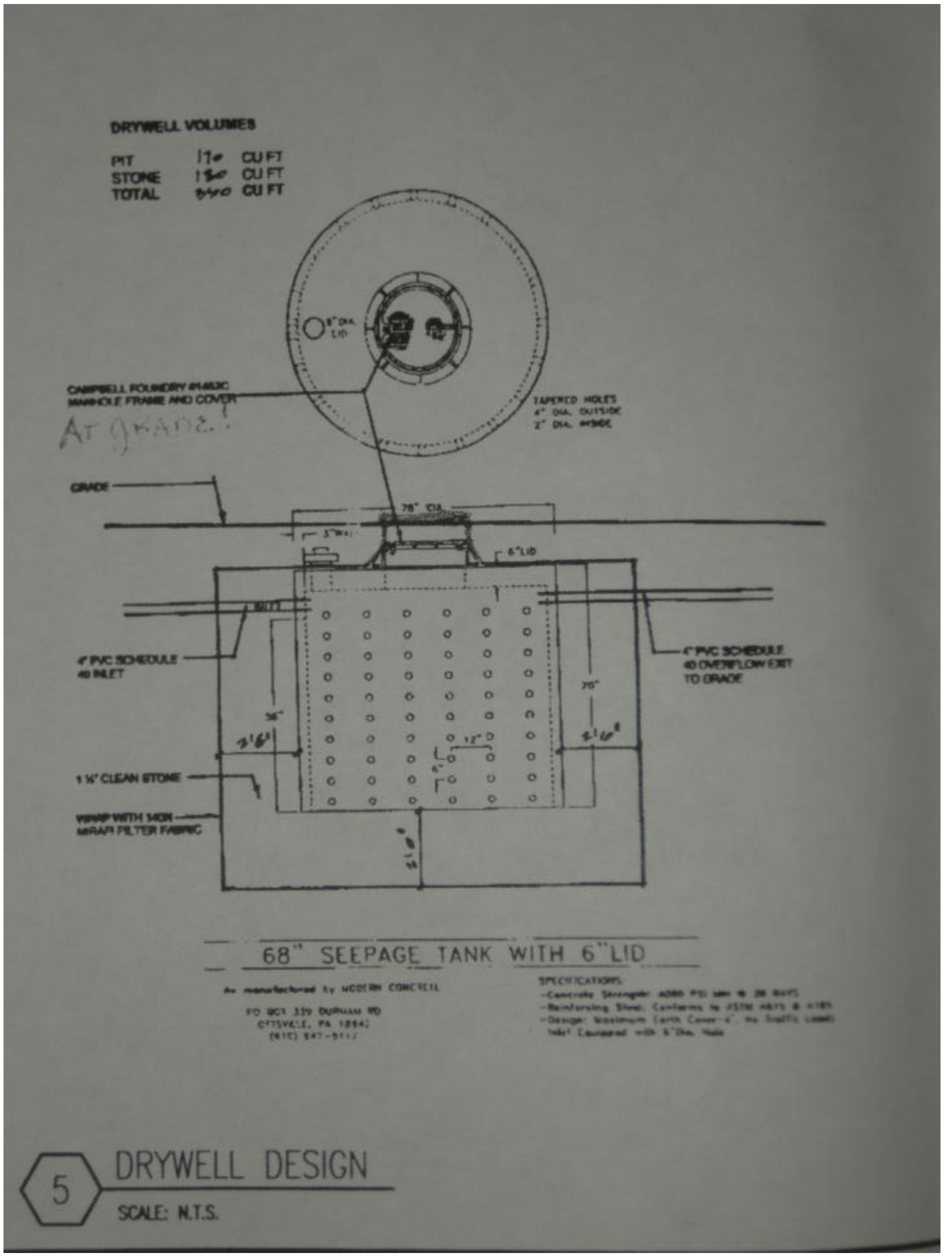


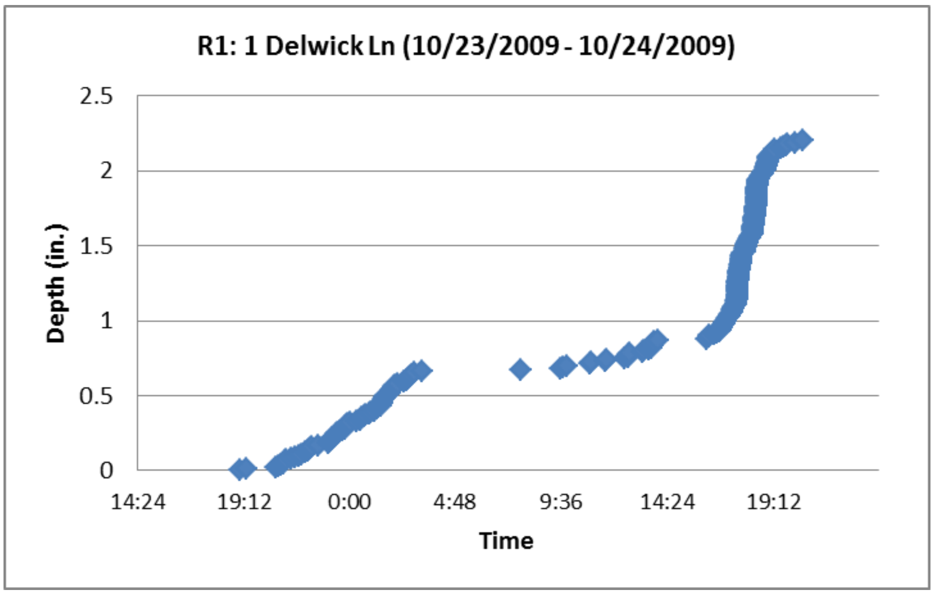
Figure B-13. Dry well (18 Slope Drive)

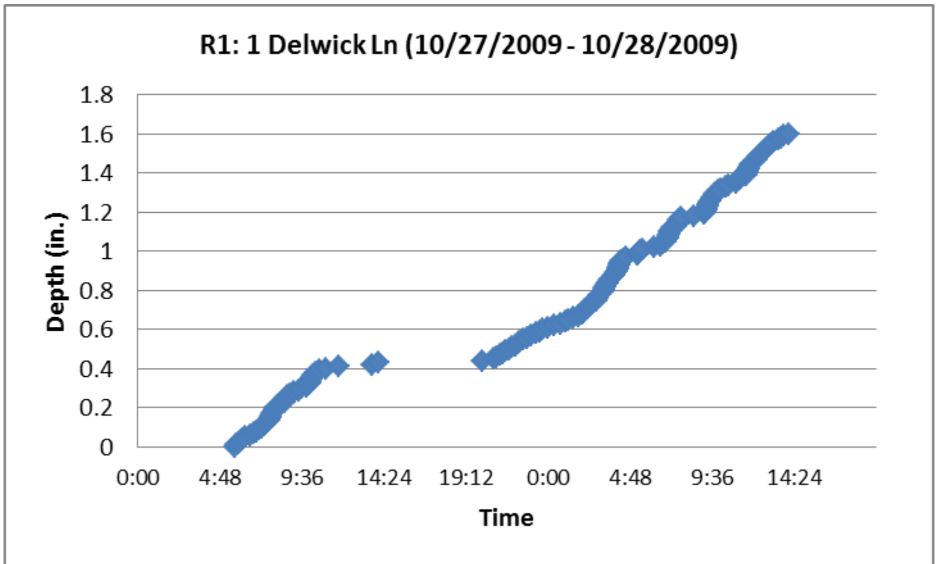
Appendix C. Soils and Infiltration Measurements at Millburn Dry Well Study Locations

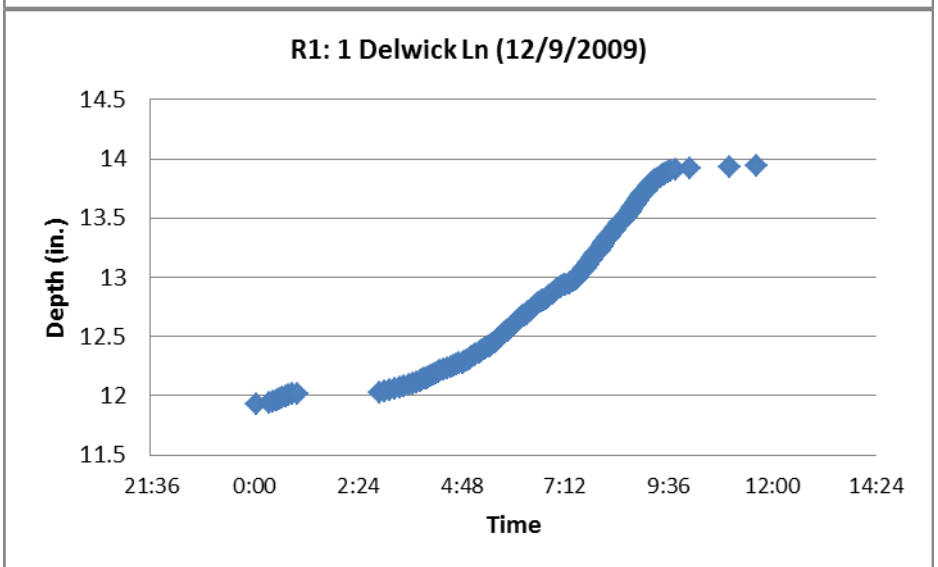
Rain Gage Data and Analysis

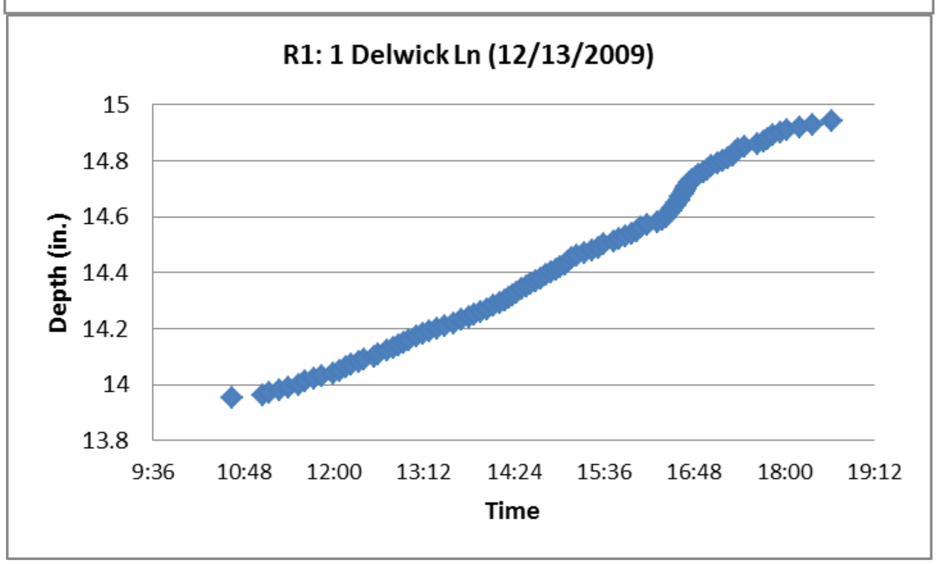
R1: Mel Singer's house on top of chimney slab at 1 Delwick Ln - Calibrated and launched at 14:00 on 5/22/09 by HDB

Start time		End time		Duration (hr)	Depth (in.)	Average intensity (in./hr)
10/23/2009	19:01	10/24/2009	20:34	25:33	2.20	0.09
10/27/2009	5:40	10/28/2009	14:04	32:24	1.60	0.05
12/9/2009	0:03	12/9/2009	11:38	11:35	2.01	0.17
12/13/2009	10:39	12/13/2009	18:38	7:59	0.99	0.12



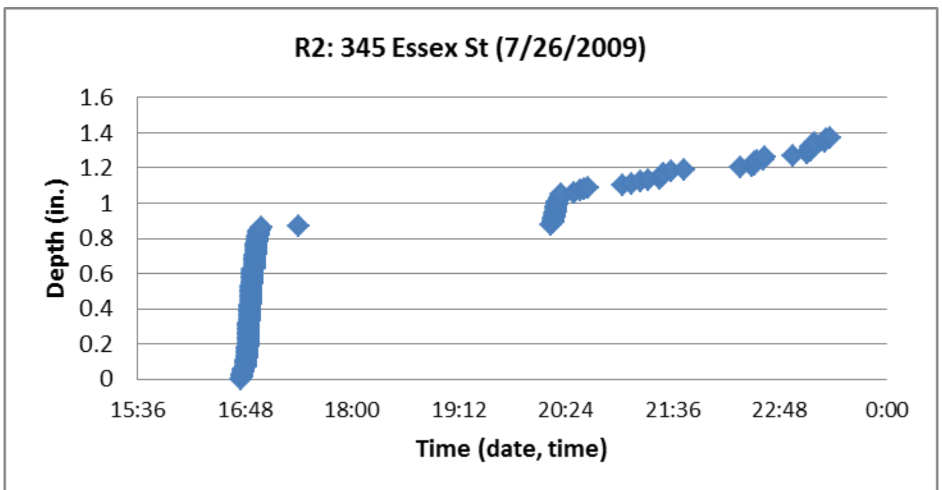

R1: 1 Delwick Ln (10/27/2009 - 10/28/2009)
Depth (in.)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0:00
4:48
9:36
14:24
19:12
0:00
4:48
9:36
14:24
Time

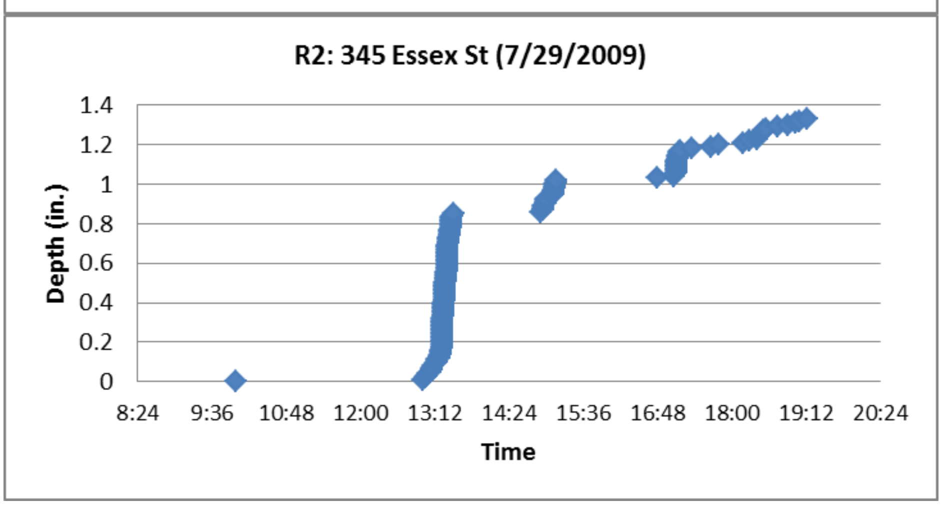

R1: 1 Delwick Ln (12/9/2009)
Depth (in.)
14.5
14
13.5
13
12.5
12
11.5
21:36
0:00
2:24
4:48
7:12
9:36
12:00
14:24
Time

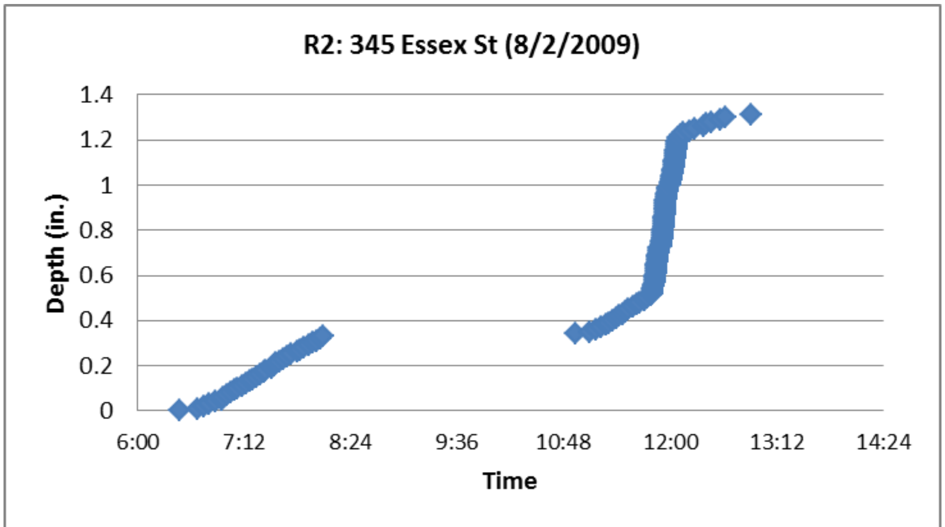

R1: 1 Delwick Ln (12/13/2009)
Depth (in.)
15
14.8
14.6
14.4
14.2
14
13.8
9:36
10:48
12:00
13:12
14:24
15:36
16:48
18:00
19:12
Time

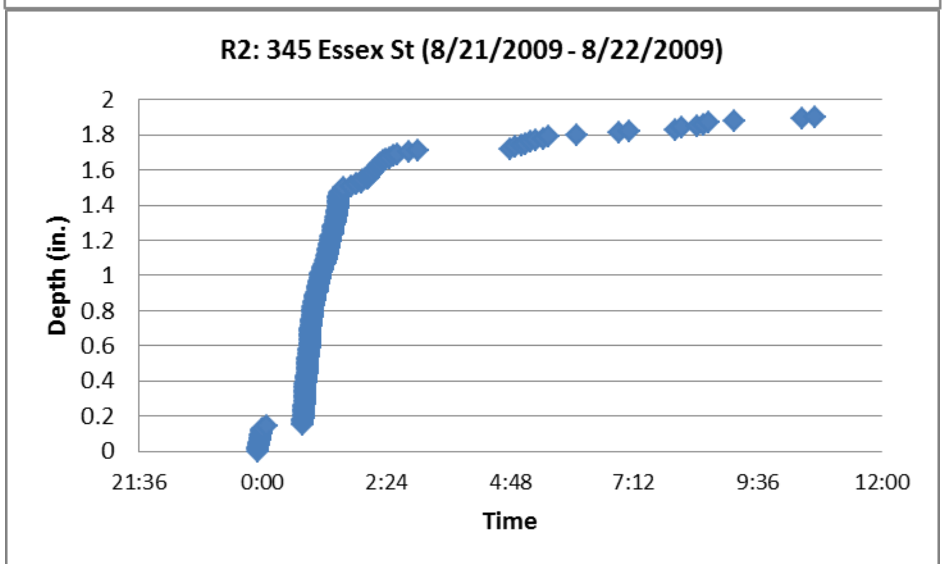
R2: Roof of Township's maintenance garage on Essex Rd - Calibrated and launched at 12:00 on 5/13/09 by HDB

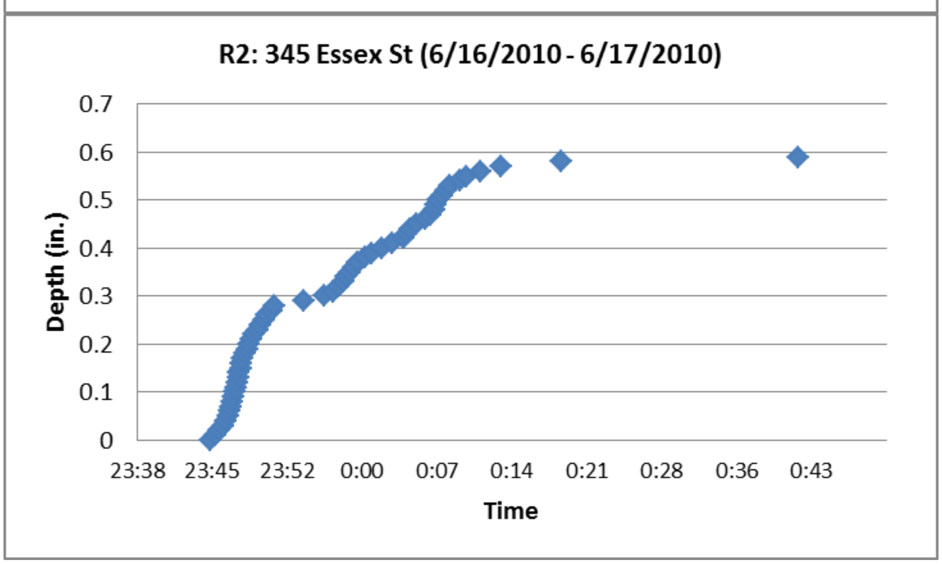
Start time		End time		Duration (hr)	Depth (in.)	Average intensity (in./hr)
7/26/2009	16:46	7/26/2009	23:22	6:36	1.37	0.21
7/29/2009	10:00	7/29/2009	19:13	9:13	1.33	0.14
8/2/2009	6:29	8/2/2009	12:55	6:26	1.31	0.20
8/21/2009	23:54	8/22/2009	10:43	10:49	1.90	0.18
6/16/2010	23:45	6/17/2010	0:41	0:56	0.69	0.74
6/22/2010	18:33	6/22/2010	18:54	0:21	0.37	1.06
7/14/2010	9:02	7/14/2010	10:04	1:02	1.22	1.18
8/1/2010	8:21	8/1/2010	9:54	1:33	1.34	0.86
7/8/2011	16:02	7/8/2011	20:46	4:44	0.73	0.15
8/1/2011	0:25	8/1/2011	0:42	0:17	0.46	1.62
8/3/2011	16:28	8/4/2011	4:02	11:34	0.65	0.06

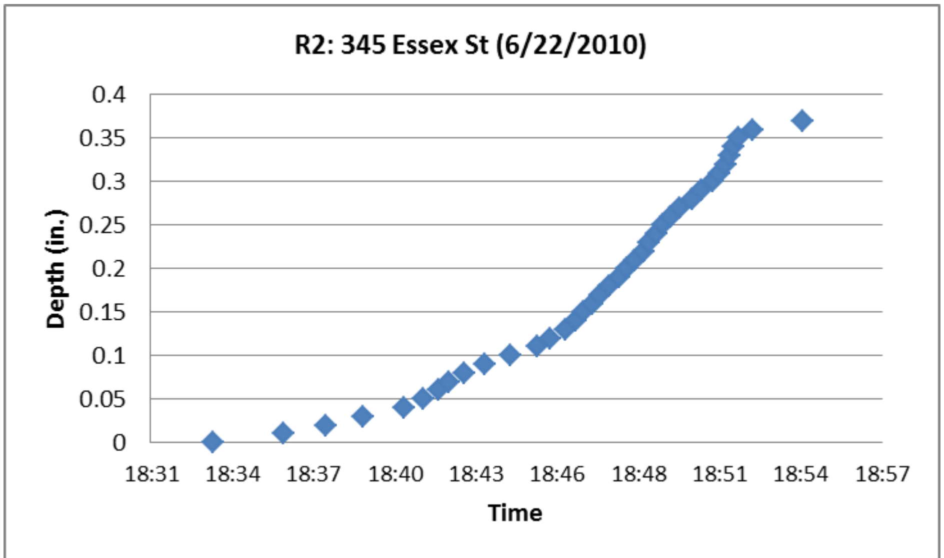


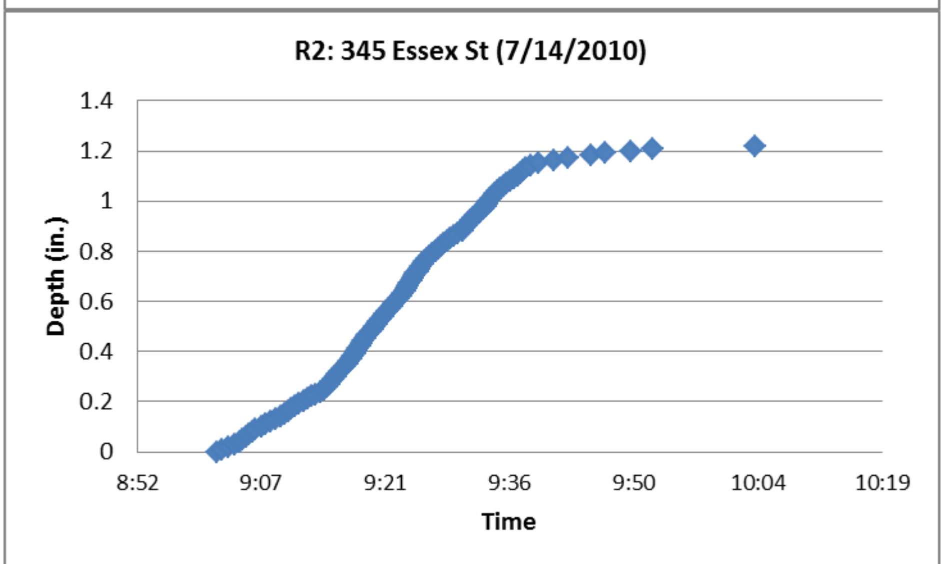


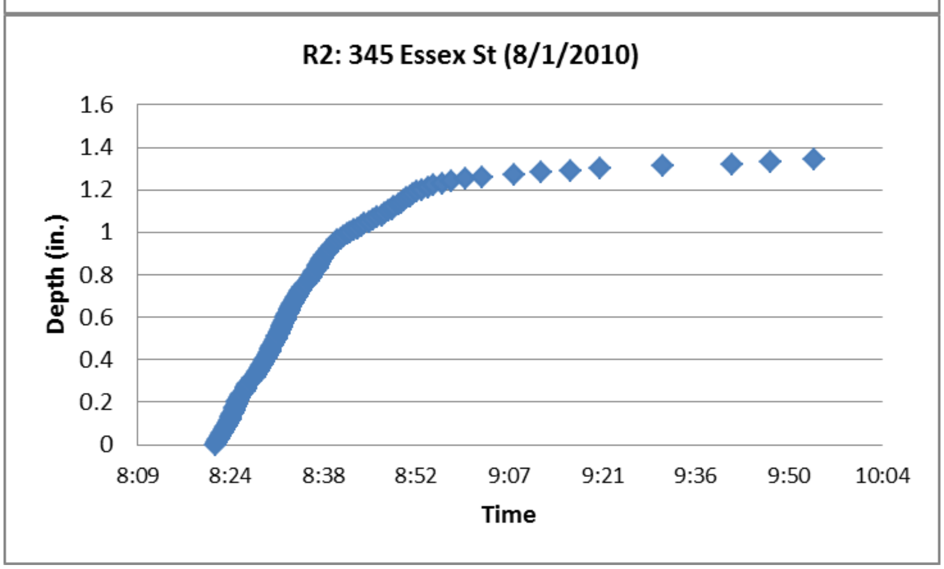

R2: 345 Essex St (8/2/2009)
Depth (in.)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
6:00
7:12
8:24
9:36
10:48
12:00
13:12
14:24
Time

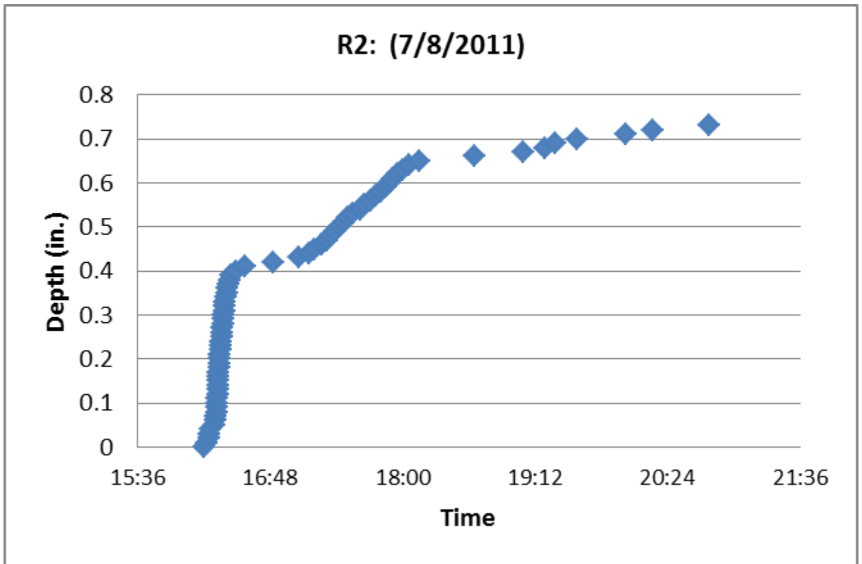

R2: 345 Essex St (8/21/2009 - 8/22/2009)
Depth (in.)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
21:36
0:00
2:24
4:48
7:12
9:36
12:00
Time

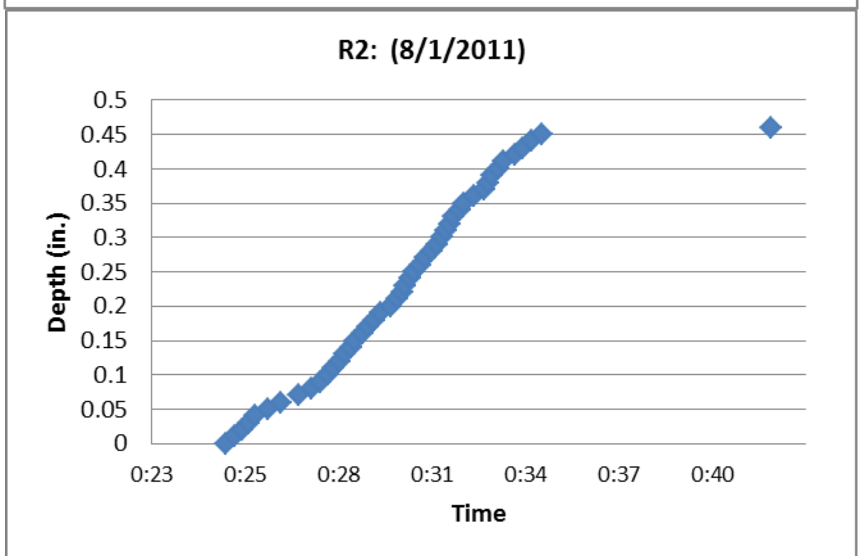

R2: 345 Essex St (6/16/2010 - 6/17/2010)
Depth (in.)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
23:38
23:45
23:52
0:00
0:07
0:14
0:21
0:28
0:36
0:43
Time

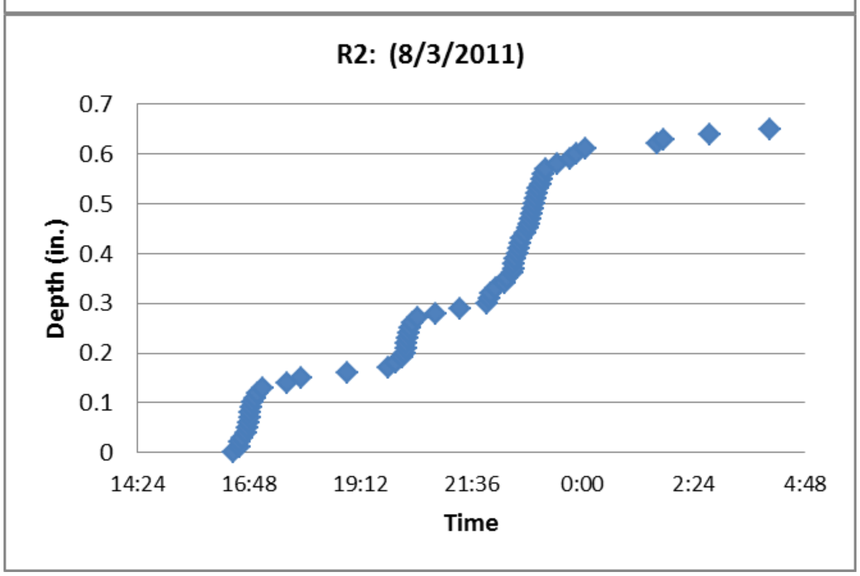

R2: 345 Essex St (6/22/2010)
Depth (in.)
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
18:31
18:34
18:37
18:40
18:43
18:46
18:48
18:51
18:54
18:57
Time


R2: 345 Essex St (7/14/2010)
Depth (in.)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
8:52
9:07
9:21
9:36
9:50
10:04
10:19
Time


R2: 345 Essex St (8/1/2010)
Depth (in.)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
8:09
8:24
8:38
8:52
9:07
9:21
9:36
9:50
10:04
Time

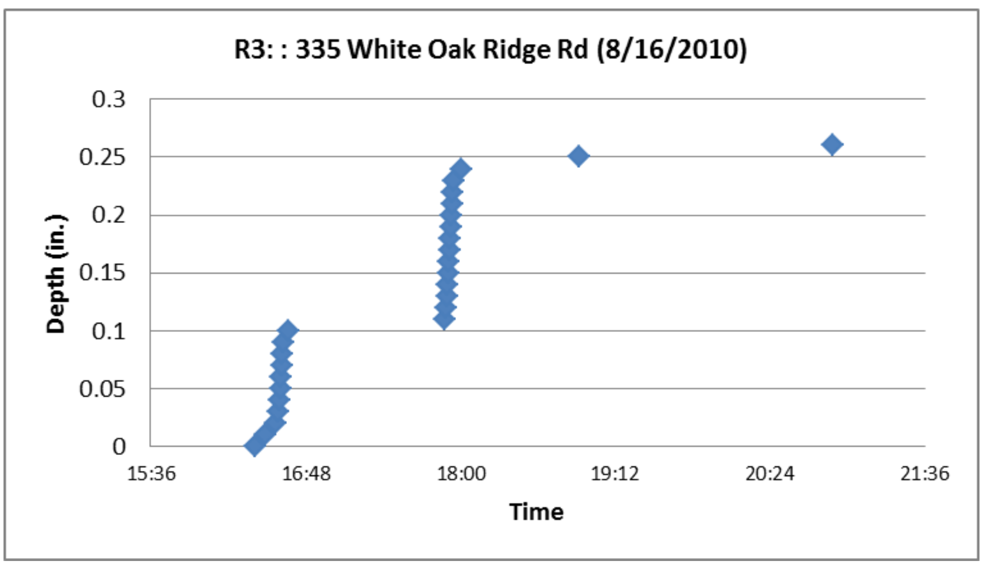

R2: (7/8/2011)
Depth (in.)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
15:36
16:48
18:00
19:12
20:24
21:36
Time

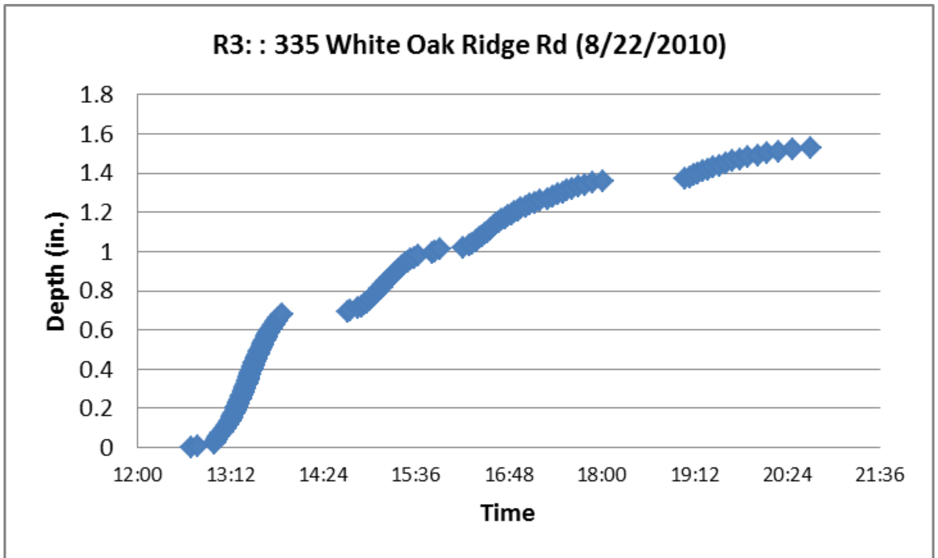

R2: (8/1/2011)
Depth (in.)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0:23
0:25
0:28
0:31
0:34
0:37
0:40
Time

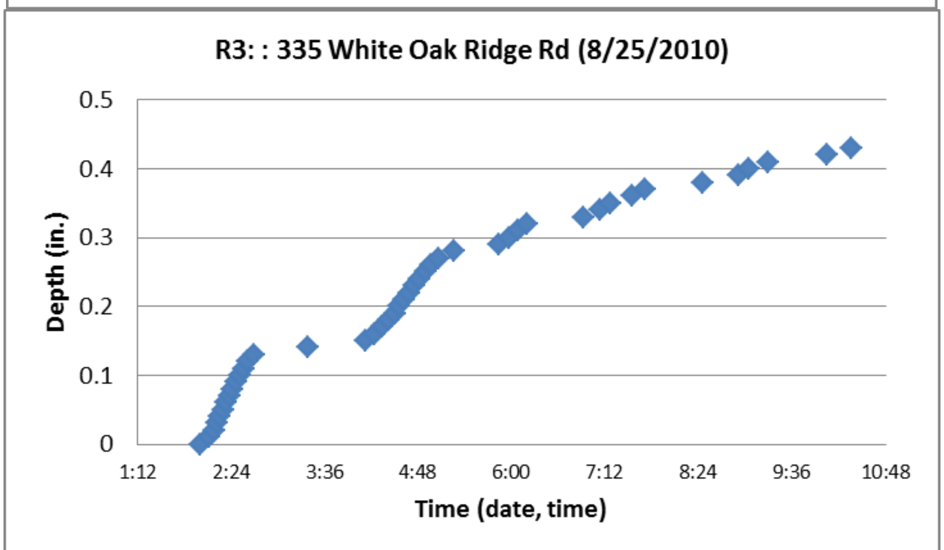

R2: (8/3/2011)
Depth (in.)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
14:24
16:48
19:12
21:36
0:00
2:24
4:48
Time

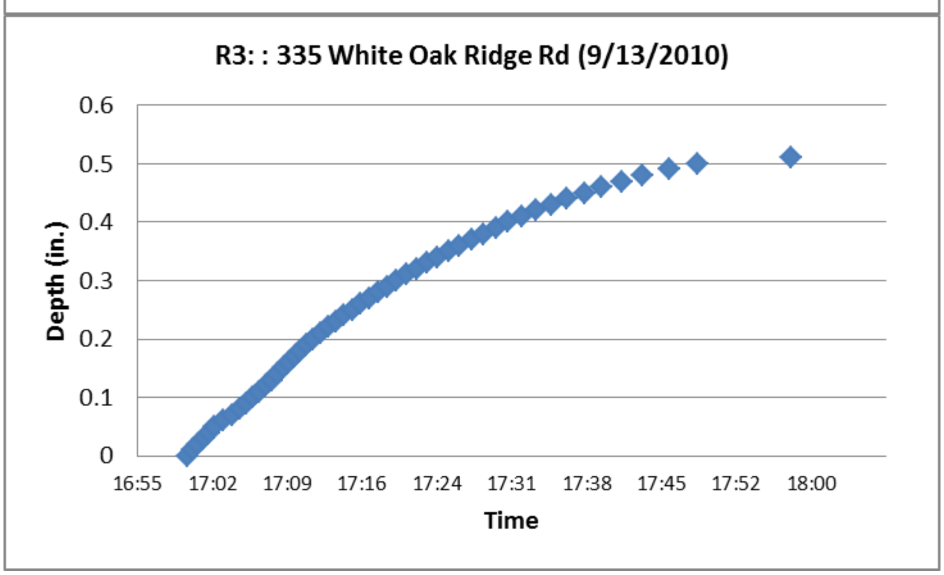
R3: Municipal Par 3 Golf Course on White Oak Ridge Rd - Calibrated and launched at 16:00 on 5/13/09 by HDB

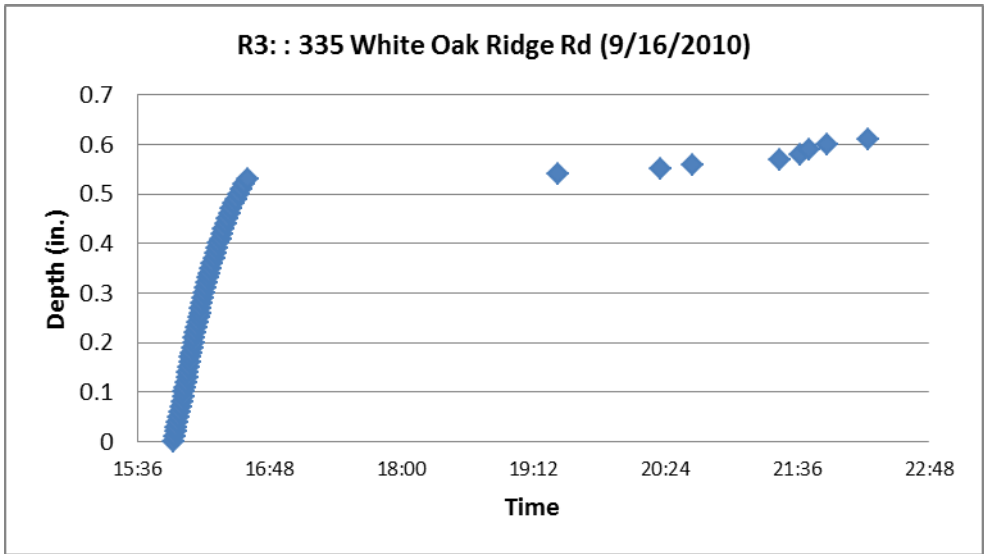
Start time		End time		Duration (hr)	Depth (in.)	Average intensity (in./hr)
8/16/2010	16:24	8/16/2010	20:53	4:29	0.26	0.06
8/22/2010	12:41	8/22/2010	20:42	8:01	1.53	0.19
8/25/2010	2:00	8/25/2010	10:21	8:21	0.43	0.05
9/13/2010	17:00	9/13/2010	17:58	0:58	0.51	0.53
9/16/2010	15:55	9/16/2010	22:14	6:19	0.61	0.10
9/27/2010	7:40	9/28/2010	12:36	28:56	0.69	0.02
9/30/2010	4:14	9/30/2010	10:01	5:47	1.83	0.32
10/1/2010	1:41	10/1/2010	13:05	11:24	2.53	0.22
10/11/2010	18:29	10/11/2010	23:57	5:28	0.71	0.13
11/4/2010	3:26	11/5/2010	7:35	28:09	1.16	0.04
12/1/2010	1:05	12/1/2010	15:07	14:02	1.88	0.13
12/12/2010	0:57	12/13/2010	6:51	29:54	1.87	0.06
2/25/2011	0:25	2/25/2011	18:44	18:19	1.36	0.06
2/28/2011	3:50	2/28/2011	11:30	7:40	0.49	0.06
3/6/2011	7:55	3/7/2011	3:29	19:34	2.78	0.14
3/10/2011	2:47	3/11/2011	8:05	29:18	2.90	0.10
5/23/2011	22:19	5/23/2011	23:17	0:58	0.68	0.70
5/30/2011	6:07	5/30/2011	6:41	0:34	0.27	0.48
6/11/2011	1:26	6/11/2011	5:29	4:03	0.56	0.14
6/17/2011	13:45	6/17/2011	18:22	4:37	2.78	0.62
7/3/2011	4:46	7/3/2011	21:23	16:37	0.33	0.02

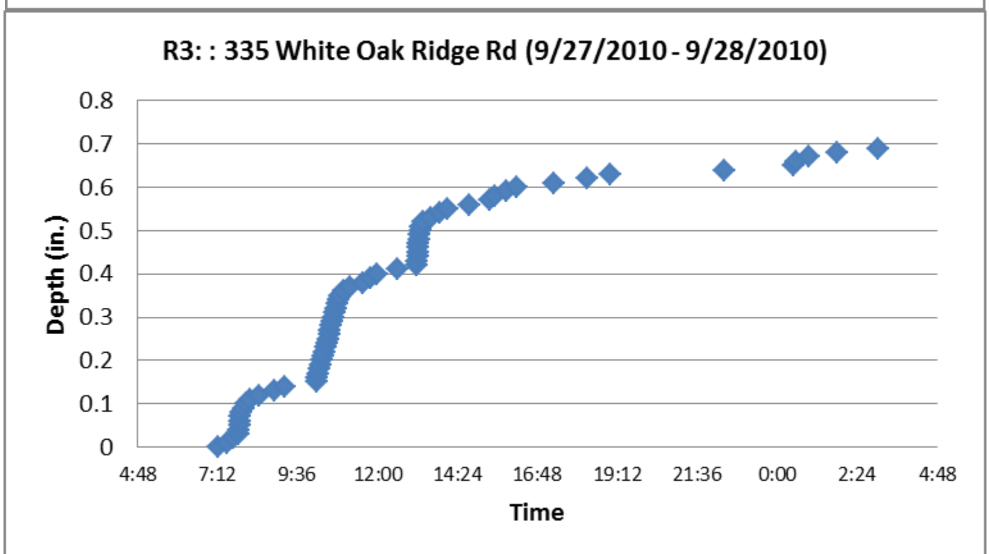


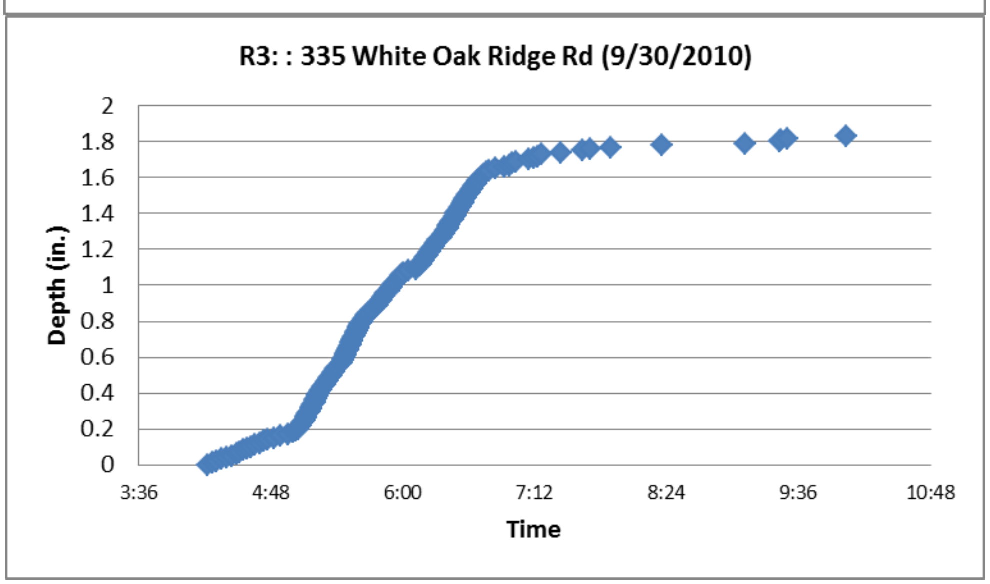

R3: : 335 White Oak Ridge Rd (8/22/2010)
Depth (in.)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
12:00
13:12
14:24
15:36
16:48
18:00
19:12
20:24
21:36
Time

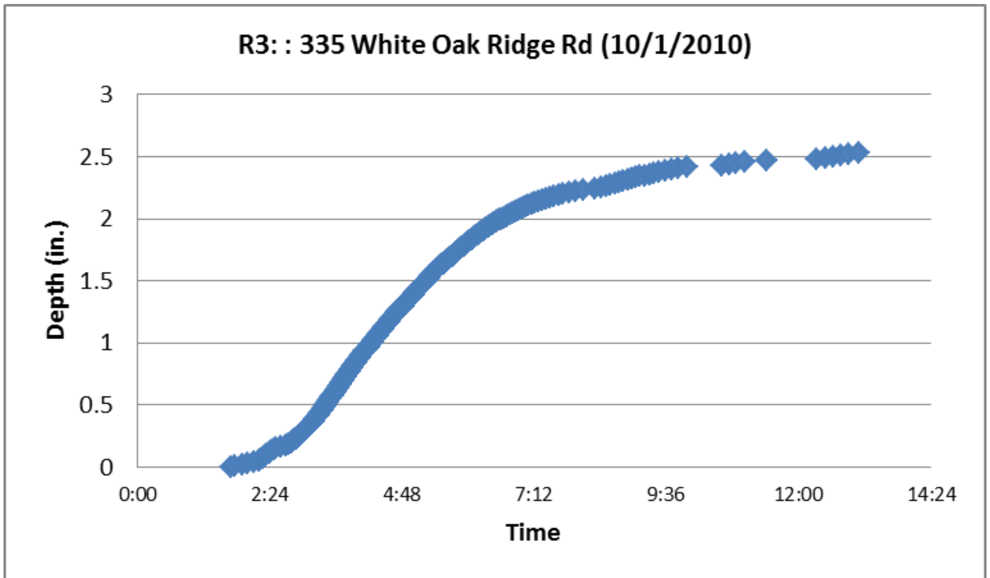

R3: : 335 White Oak Ridge Rd (8/25/2010)
Depth (in.)
0.5
0.4
0.3
0.2
0.1
0
1:12
2:24
3:36
4:48
6:00
7:12
8:24
9:36
10:48
Time (date, time)

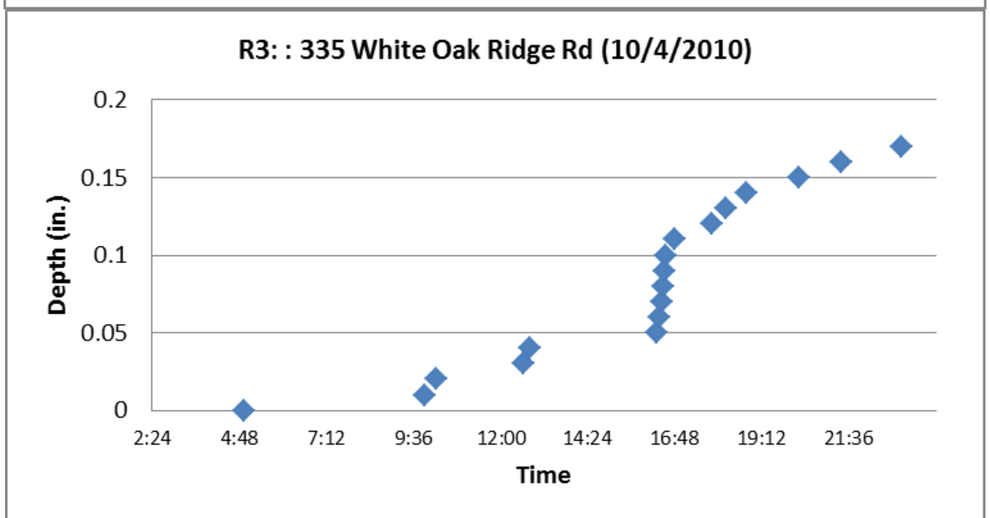

R3: : 335 White Oak Ridge Rd (9/13/2010)
Depth (in.)
0.6
0.5
0.4
0.3
0.2
0.1
0
16:55
17:02
17:09
17:16
17:24
17:31
17:38
17:45
17:52
18:00
Time

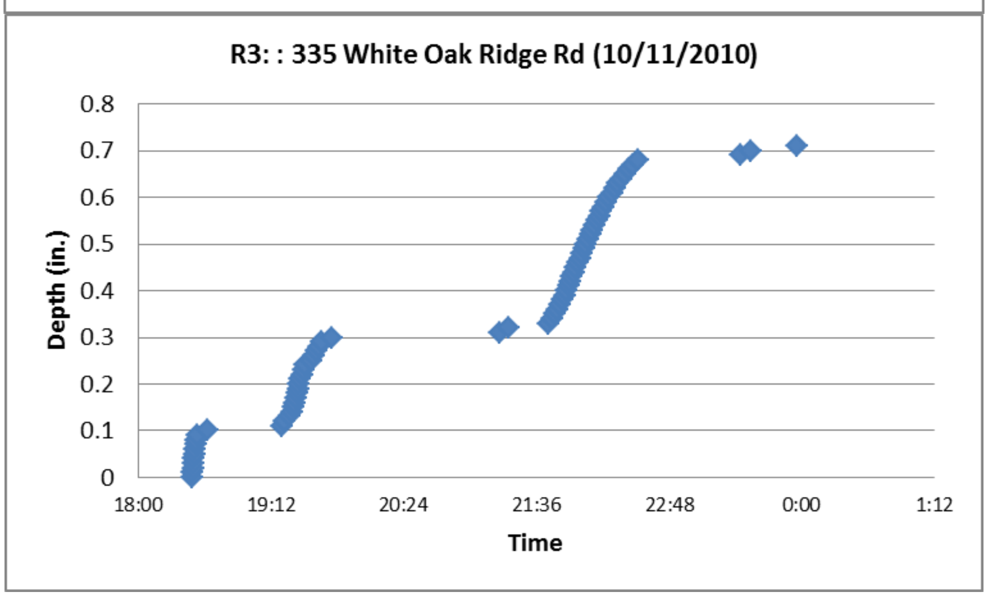

R3: : 335 White Oak Ridge Rd (9/16/2010)
Depth (in.)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
15:36
16:48
18:00
19:12
20:24
21:36
22:48
Time

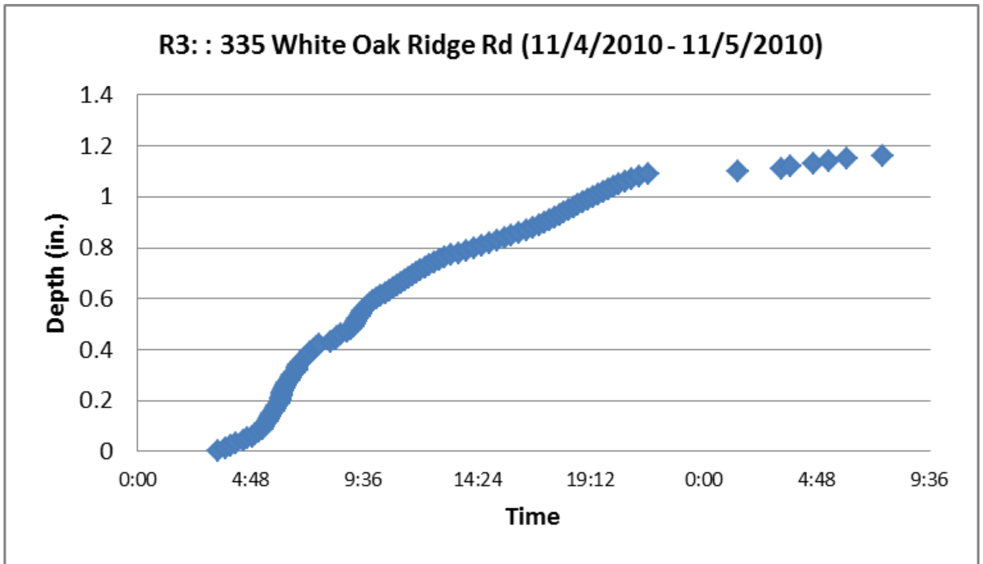

R3: : 335 White Oak Ridge Rd (9/27/2010 - 9/28/2010)
Depth (in.)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
4:48
7:12
9:36
12:00
14:24
16:48
19:12
21:36
0:00
2:24
4:48
Time

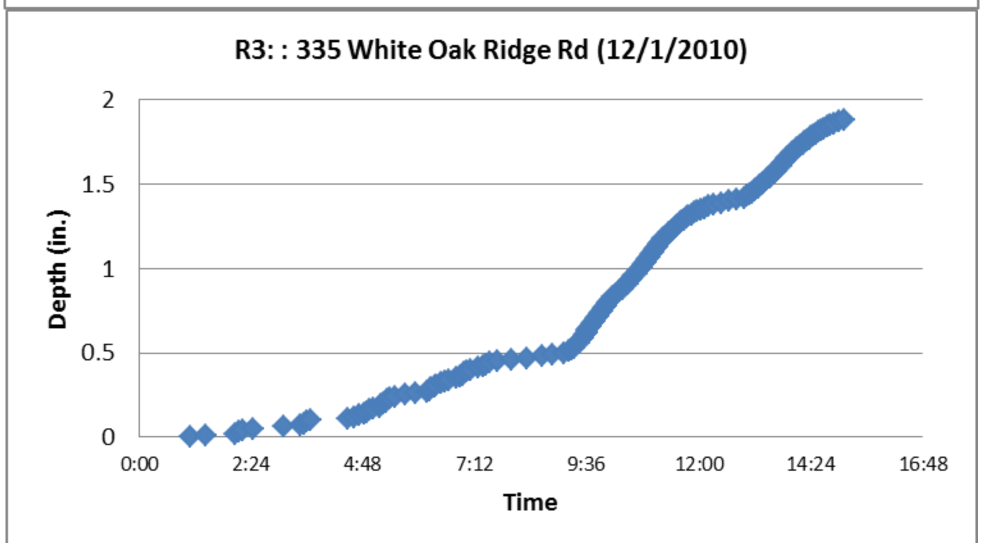

R3: : 335 White Oak Ridge Rd (9/30/2010)
Depth (in.)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3:36
4:48
6:00
7:12
8:24
9:36
10:48
Time

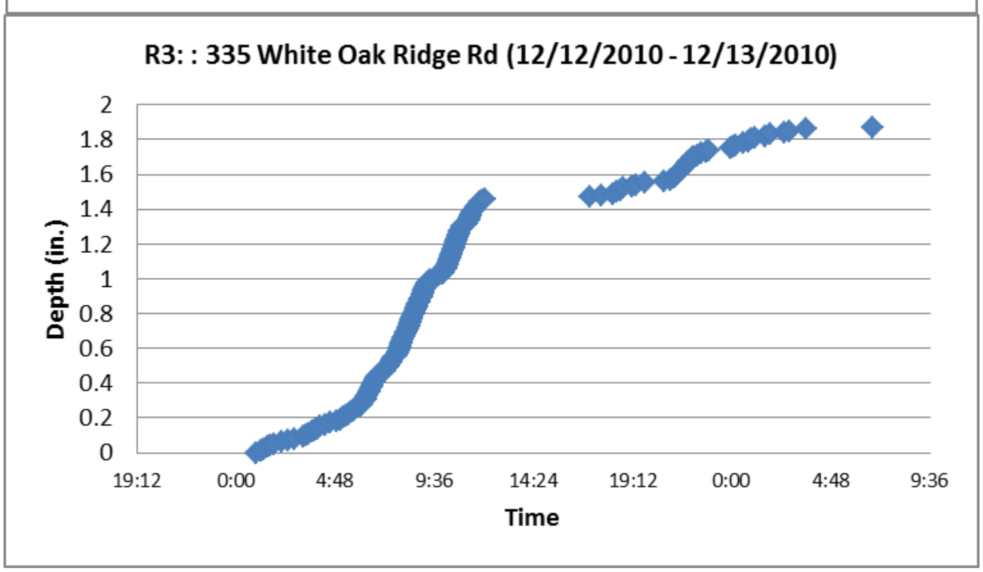

R3: : 335 White Oak Ridge Rd (10/1/2010)
Depth (in.)
3
2.5
2
1.5
1
0.5
0
0:00
2:24
4:48
7:12
9:36
12:00
14:24
Time

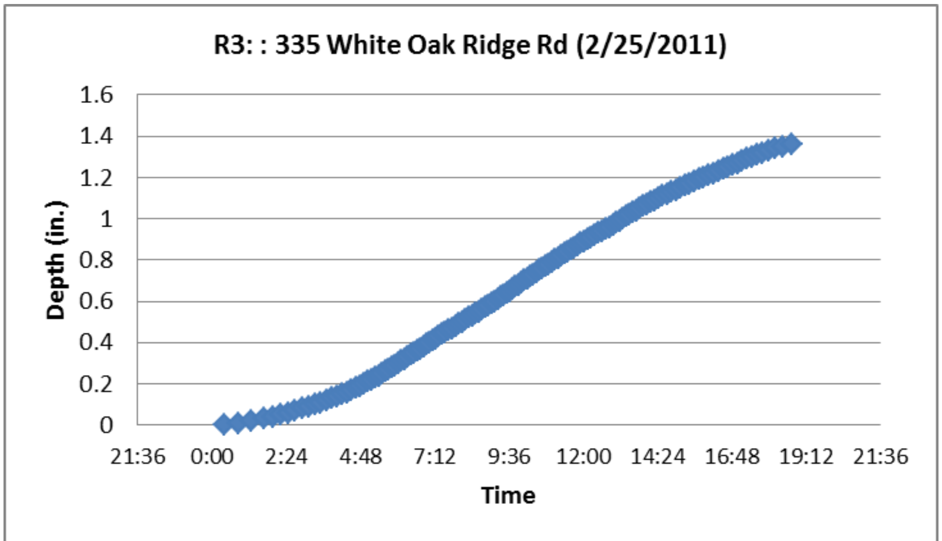

R3: : 335 White Oak Ridge Rd (10/4/2010)
Depth (in.)
0.2
0.15
0.1
0.05
0
2:24
4:48
7:12
9:36
12:00
14:24
16:48
19:12
21:36
Time

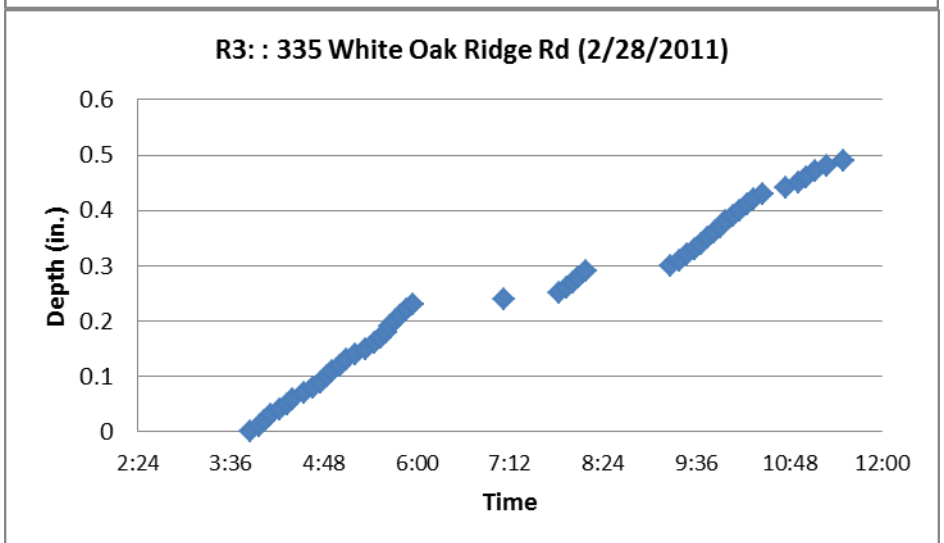

R3: : 335 White Oak Ridge Rd (10/11/2010)
Depth (in.)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
18:00
19:12
20:24
21:36
22:48
0:00
1:12
Time

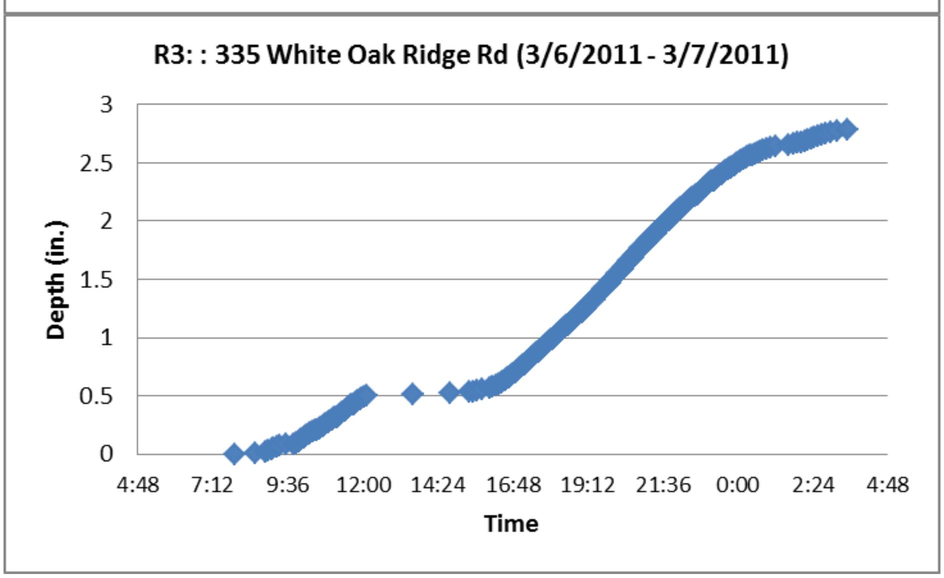

R3: : 335 White Oak Ridge Rd (11/4/2010 - 11/5/2010)
Depth (in.)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0:00
4:48
9:36
14:24
19:12
0:00
4:48
9:36
Time

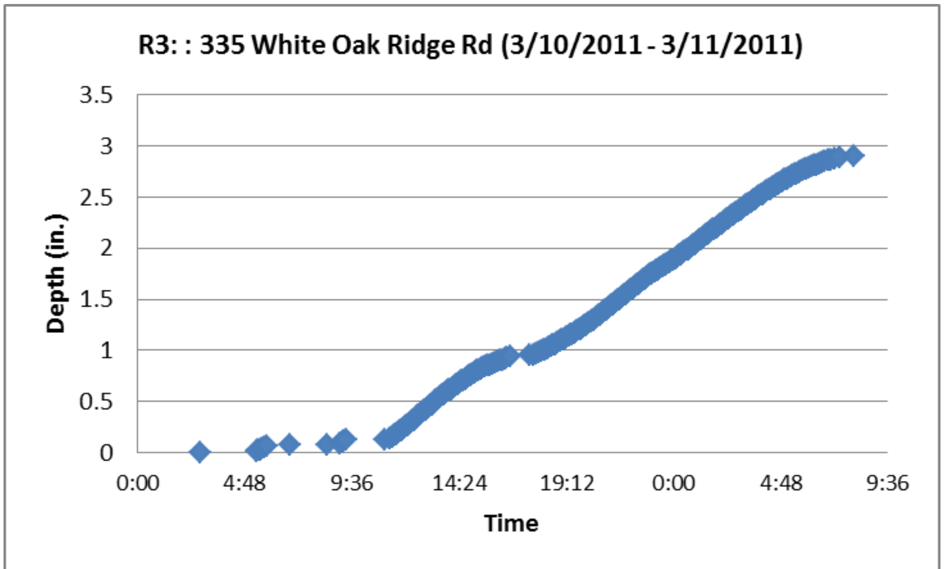

R3: : 335 White Oak Ridge Rd (12/1/2010)
Depth (in.)
2
1.5
1
0.5
0
0:00
2:24
4:48
7:12
9:36
12:00
14:24
16:48
Time

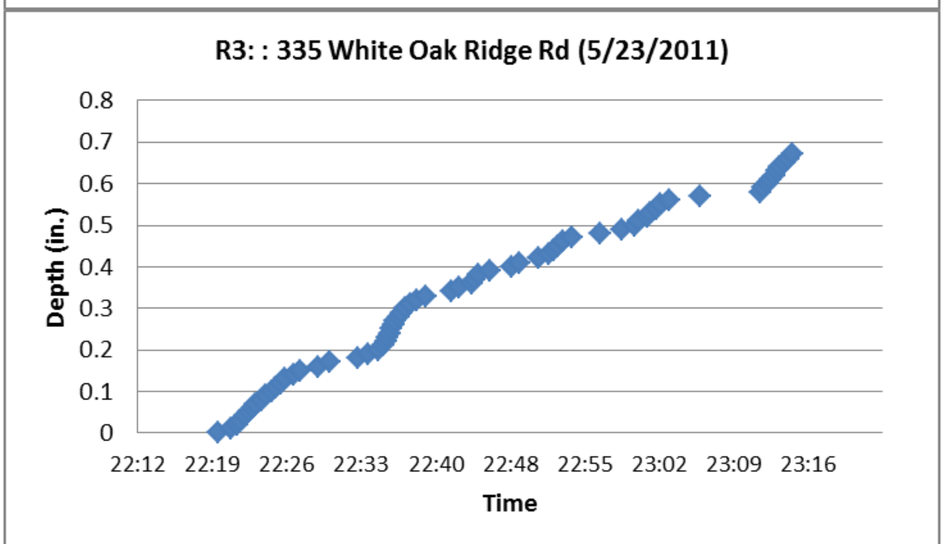

R3: : 335 White Oak Ridge Rd (12/12/2010 - 12/13/2010)
Depth (in.)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
19:12
0:00
4:48
9:36
14:24
19:12
0:00
4:48
9:36
Time

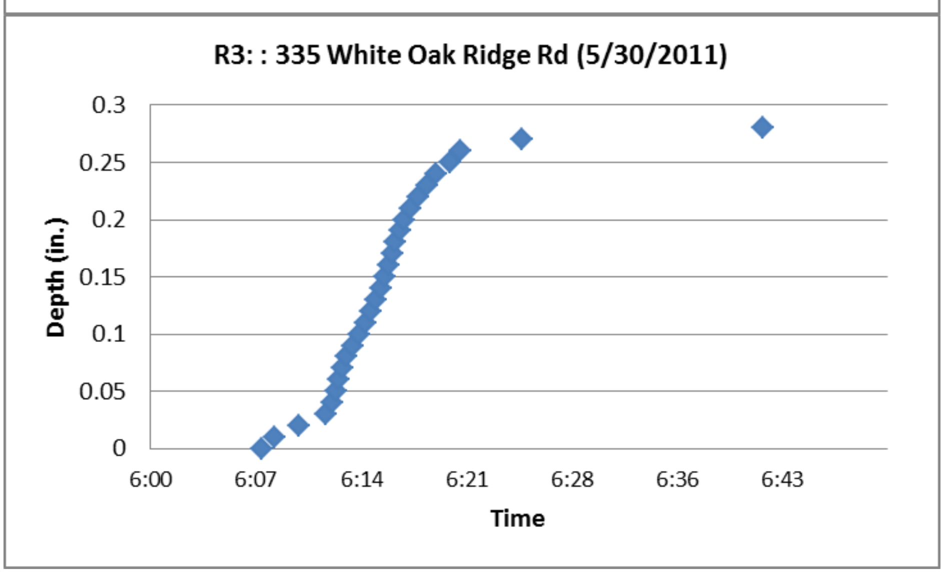

R3: : 335 White Oak Ridge Rd (2/25/2011)
Depth (in.)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
21:36
0:00
2:24
4:48
7:12
9:36
12:00
14:24
16:48
19:12
21:36
Time

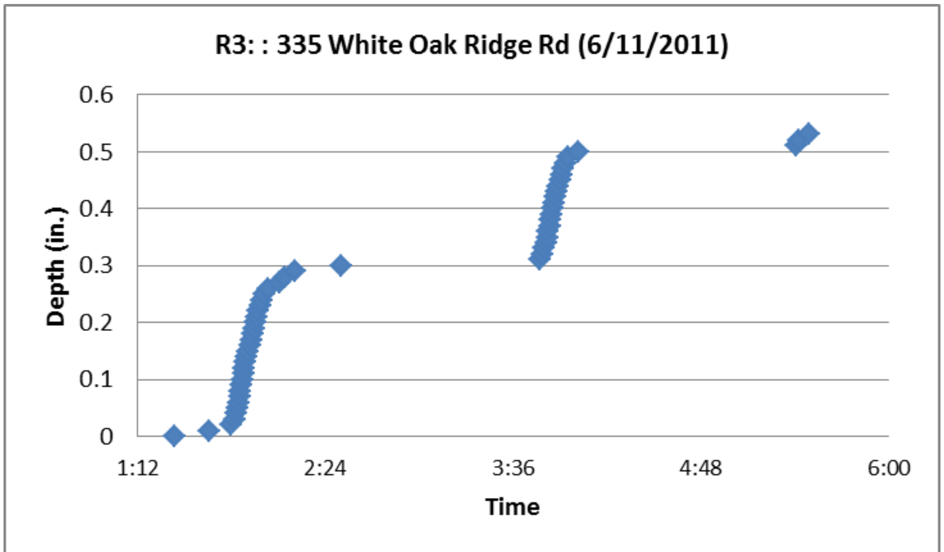

R3: : 335 White Oak Ridge Rd (2/28/2011)
Depth (in.)
0.6
0.5
0.4
0.3
0.2
0.1
0
2:24
3:36
4:48
6:00
7:12
8:24
9:36
10:48
12:00
Time

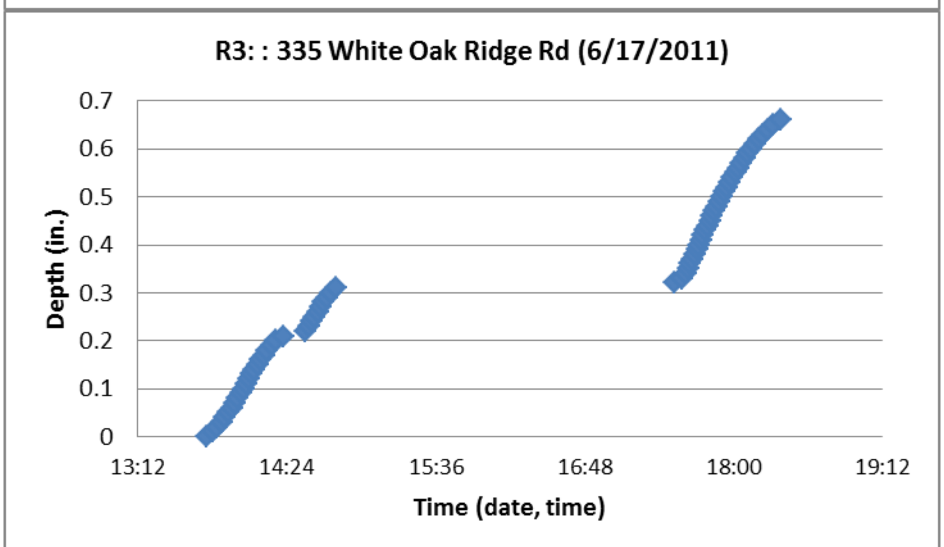

R3: : 335 White Oak Ridge Rd (3/6/2011 - 3/7/2011)
Depth (in.)
3
2.5
2
1.5
1
0.5
0
4:48
7:12
9:36
12:00
14:24
16:48
19:12
21:36
0:00
2:24
4:48
Time

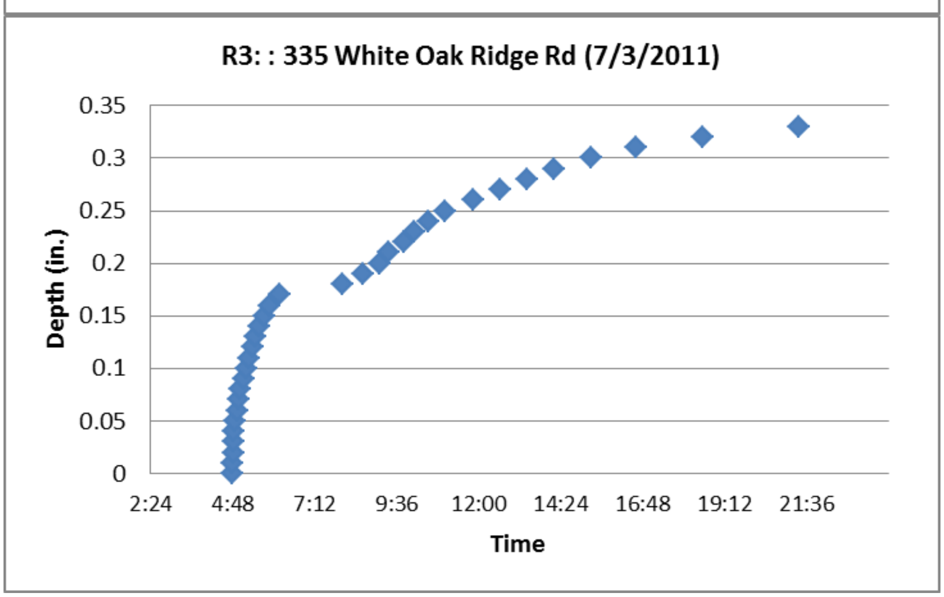

R3: : 335 White Oak Ridge Rd (3/10/2011 - 3/11/2011)
Depth (in.)
3.5
3
2.5
2
1.5
1
0.5
0
0:00
4:48
9:36
14:24
19:12
0:00
4:48
9:36
Time


R3: : 335 White Oak Ridge Rd (5/23/2011)
Depth (in.)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
22:12
22:19
22:26
22:33
22:40
22:48
22:55
23:02
23:09
23:16
Time


R3: : 335 White Oak Ridge Rd (5/30/2011)
Depth (in.)
0.3
0.25
0.2
0.15
0.1
0.05
0
6:00
6:07
6:14
6:21
6:28
6:36
6:43
Time

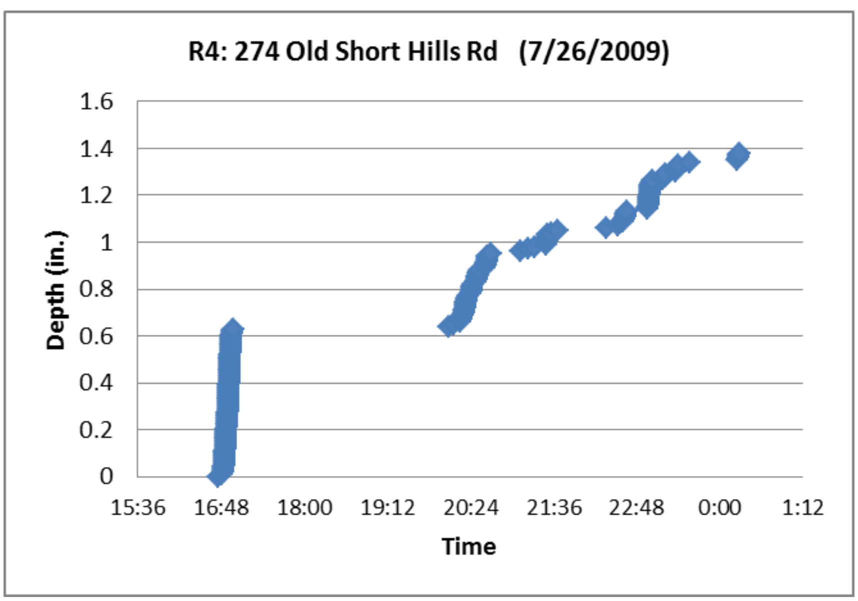

R3: : 335 White Oak Ridge Rd (6/11/2011)
Depth (in.)
0.6
0.5
0.4
0.3
0.2
0.1
0
1:12
2:24
3:36
4:48
6:00
Time

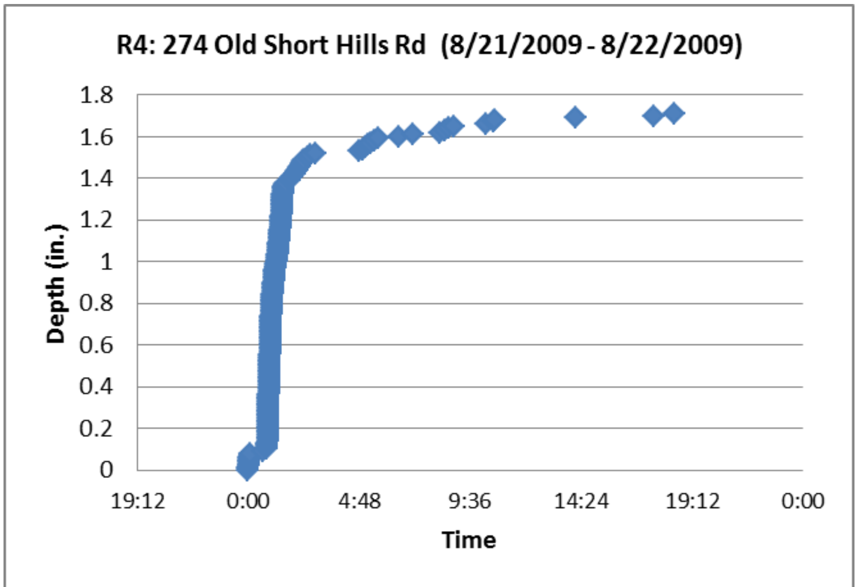

R3: : 335 White Oak Ridge Rd (6/17/2011)
Depth (in.)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
13:12
14:24
15:36
16:48
18:00
19:12
Time (date, time)


R3: : 335 White Oak Ridge Rd (7/3/2011)
Depth (in.)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
2:24
4:48
7:12
9:36
12:00
14:24
16:48
19:12
21:36
Time

R4: Old tennis court at Greenwood Gardens on Old Short Hills Rd – Calibrated and launched at 16:00 on 5/6/09 by HDB

Start time		End time		Duration (hr)	Depth (in.)	Average intensity (in./hr)
7/26/2009	16:46	7/27/2009	0:17	7:31	1.38	0.18
8/21/2009	23:57	8/22/2009	18:25	18:28	1.71	0.09
8/29/2009	5:45	8/29/2009	12:27	6:42	0.52	0.08
8/22/2010	11:20	8/22/2010	19:19	7:59	1.43	0.18
9/30/2010	4:20	9/30/2010	9:42	5:22	0.92	0.17
10/1/2010	2:17	10/1/2010	16:48	14:31	1.73	0.12
10/11/2010	18:33	10/12/2010	5:52	11:19	0.17	0.02
11/27/2010	7:40	11/27/2010	12:36	4:56	0.34	0.07
12/1/2010	2:15	12/2/2010	1:05	22:50	0.67	0.02
12/12/2010	2:13	12/12/2010	11:38	9:25	0.33	0.04
12/12/2010	17:25	12/13/2010	3:18	9:53	0.23	0.02
2/24/2011	21:58	2/25/2011	13:46	15:48	0.59	0.04
2/28/2011	4:12	2/28/2011	11:32	7:20	0.22	0.03
3/6/2011	9:00	3/7/2011	3:22	18:22	1.15	0.06
3/10/2011	5:30	3/11/2011	4:33	23:03	0.98	0.04
3/16/2011	4:29	3/16/2011	8:48	4:19	0.23	0.05

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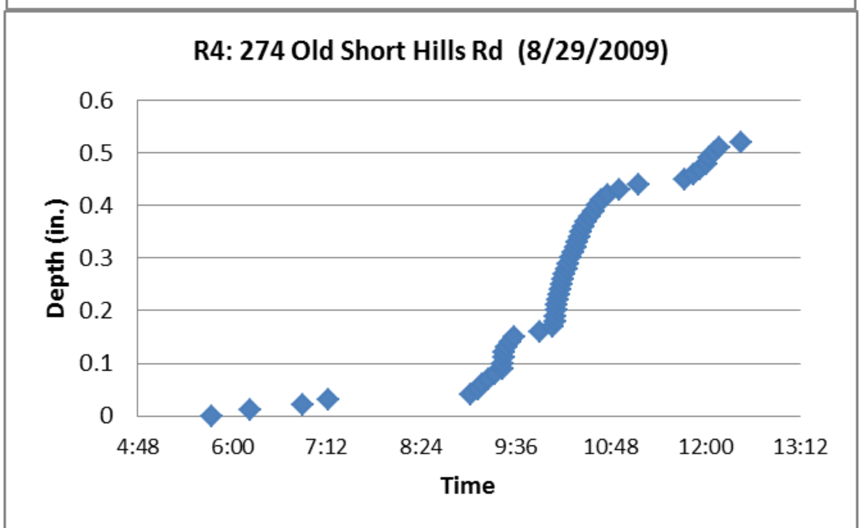
Depth (in.)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
19:12
0:00
4:48
9:36
14:24
19:12
0:00
Time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Depth (in.)
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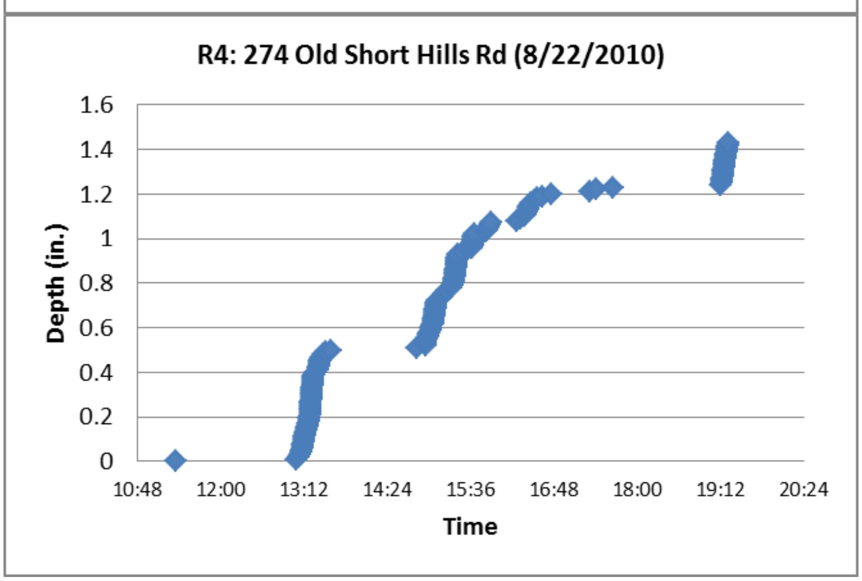
0.4
0.3
0.2
0.1
0
4:48
6:00
7:12
8:24
9:36
10:48
12:00
13:12
Time


R4: 274 Old Short Hills Rd (8/22/2010)
Depth (in.)
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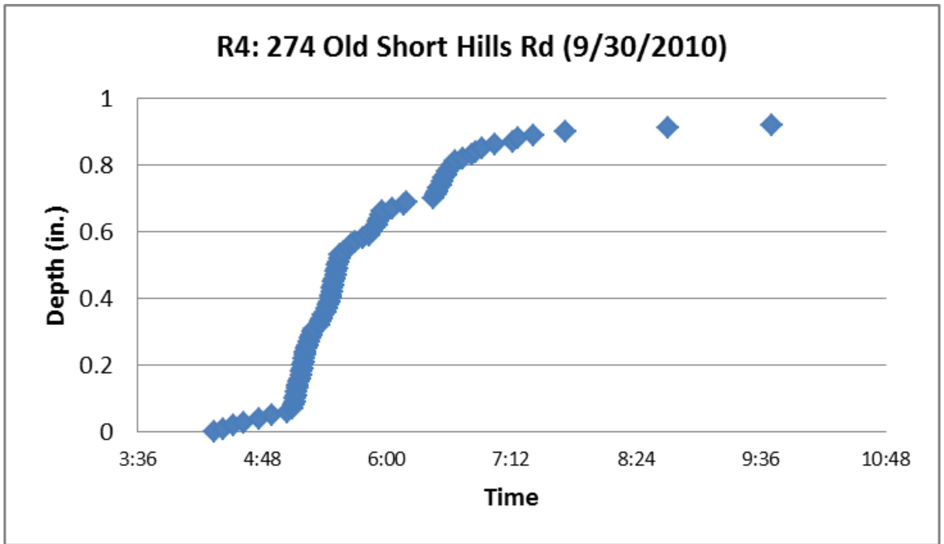
1.4
1.2
1
0.8
0.6
0.4
0.2
0
10:48
12:00
13:12
14:24
15:36
16:48
18:00
19:12
20:24
Time


R4: 274 Old Short Hills Rd (9/30/2010)
Depth (in.)
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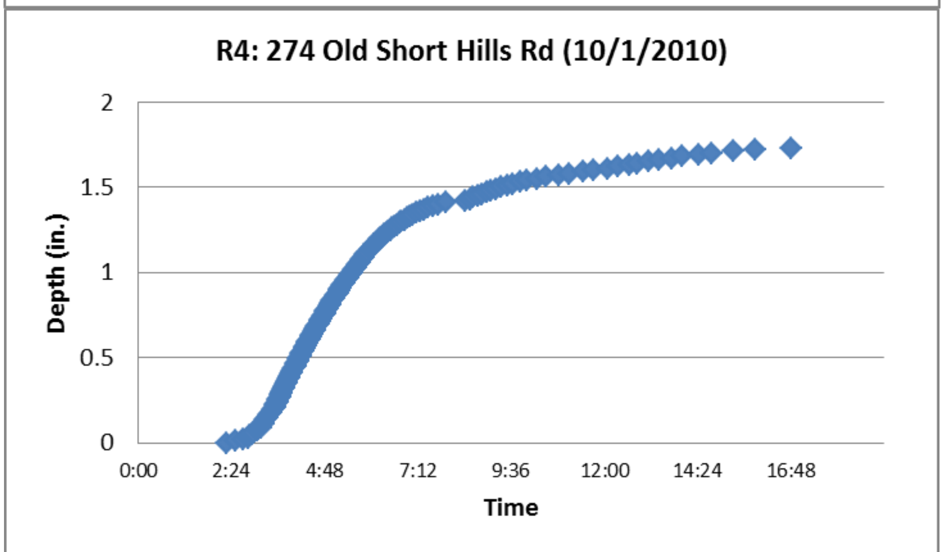
0.8
0.6
0.4
0.2
0
3:36
4:48
6:00
7:12
8:24
9:36
10:48
Time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Depth (in.)
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1.5
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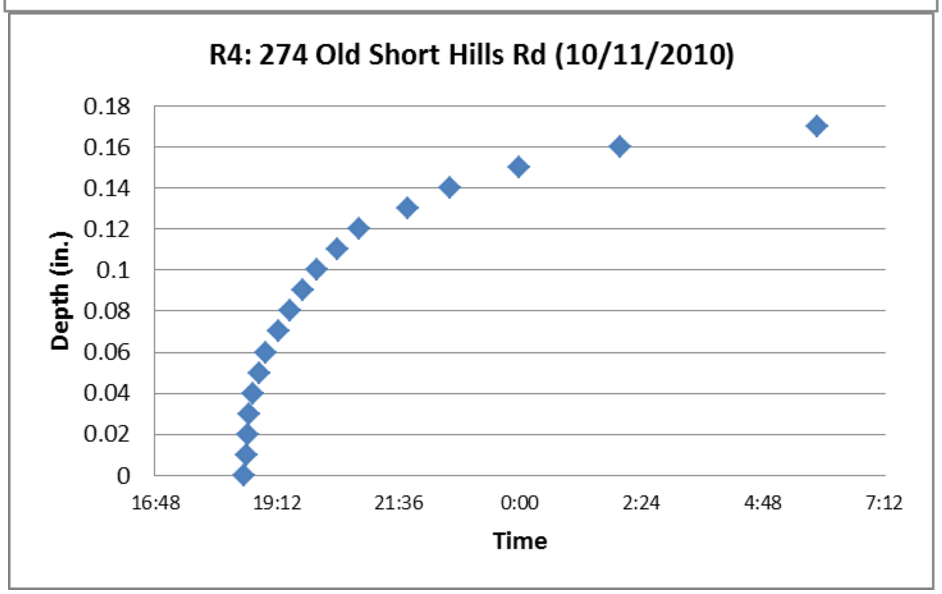
0.5
0
0:00
2:24
4:48
7:12
9:36
12:00
14:24
16:48
Time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Depth (in.)
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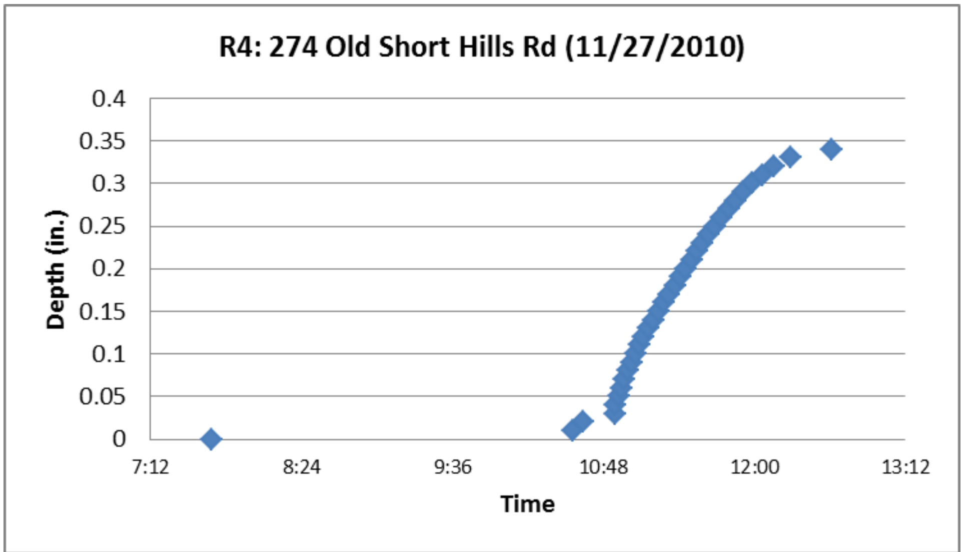
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
16:48
19:12
21:36
0:00
2:24
4:48
7:12
Time


R4: 274 Old Short Hills Rd (11/27/2010)
Depth (in.)
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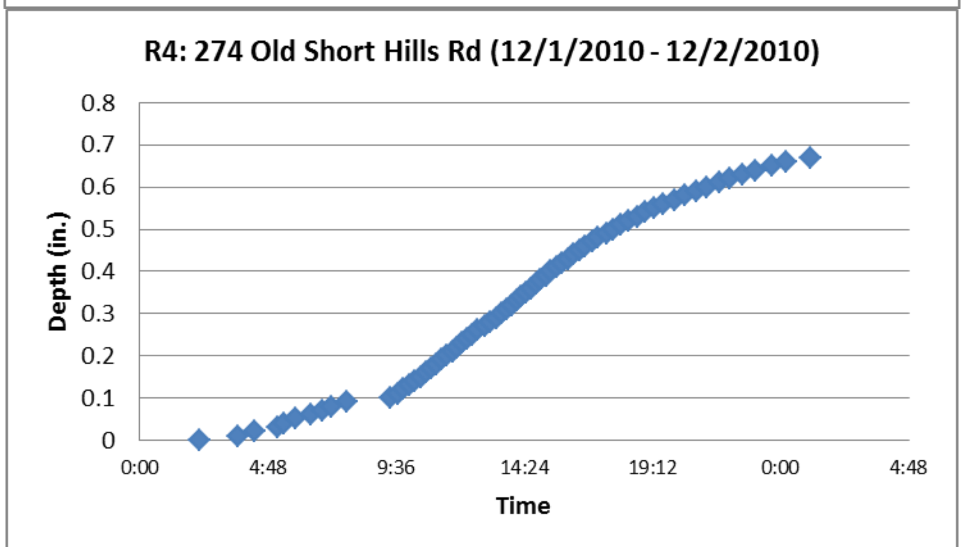
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
7:12
8:24
9:36
10:48
12:00
13:12
Time


R4: 274 Old Short Hills Rd (12/1/2010 - 12/2/2010)
Depth (in.)
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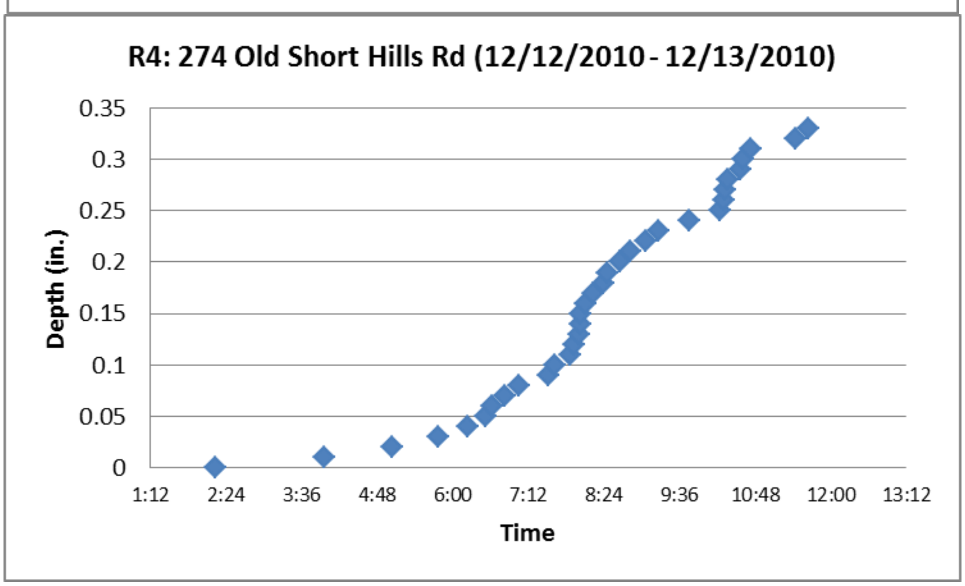
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0:00
4:48
9:36
14:24
19:12
0:00
4:48
Time


R4: 274 Old Short Hills Rd (12/12/2010 - 12/13/2010)
Depth (in.)
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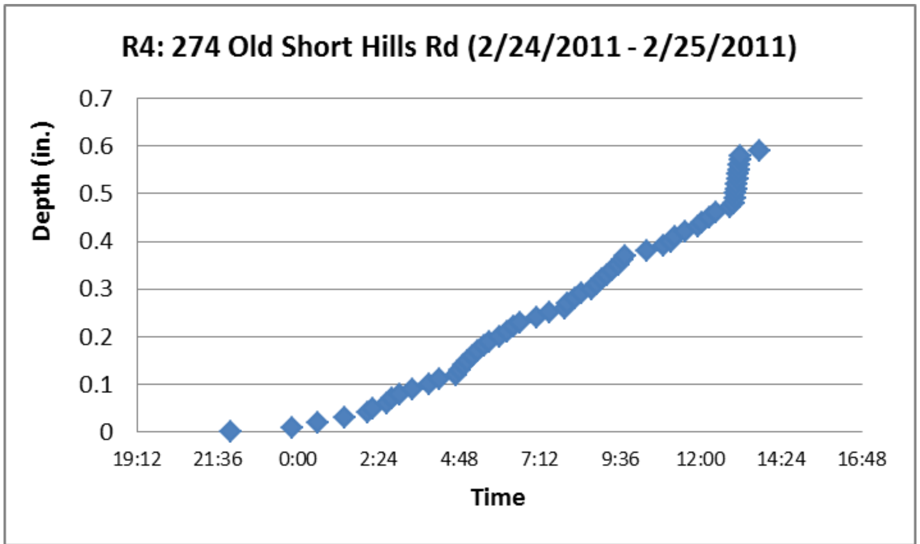
0.3
0.25
0.2
0.15
0.1
0.05
0
1:12
2:24
3:36
4:48
6:00
7:12
8:24
9:36
10:48
12:00
13:12
Time


R4: 274 Old Short Hills Rd (2/24/2011 - 2/25/2011)
Depth (in.)
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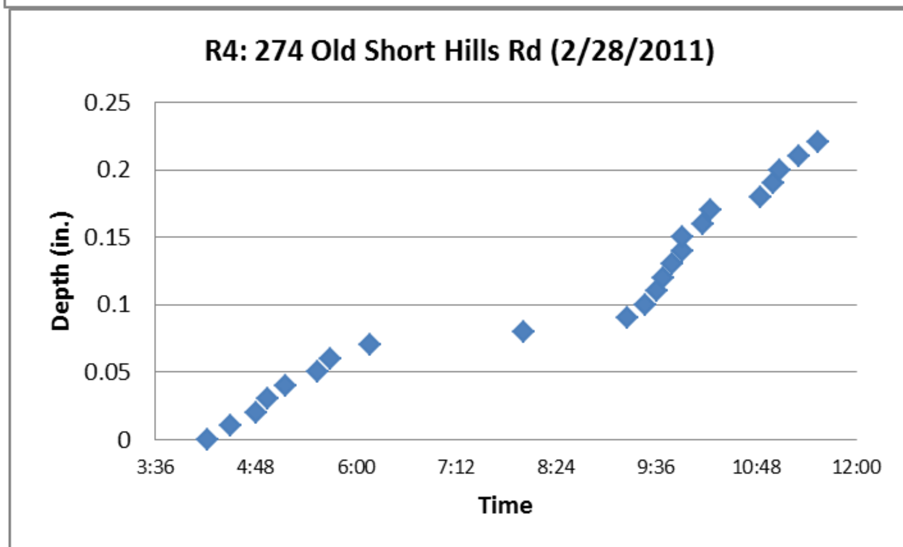
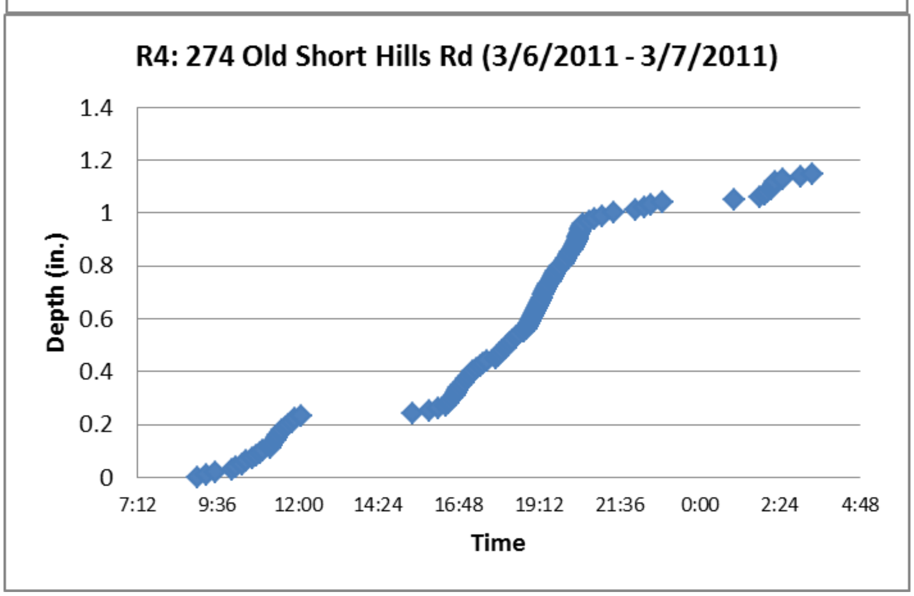
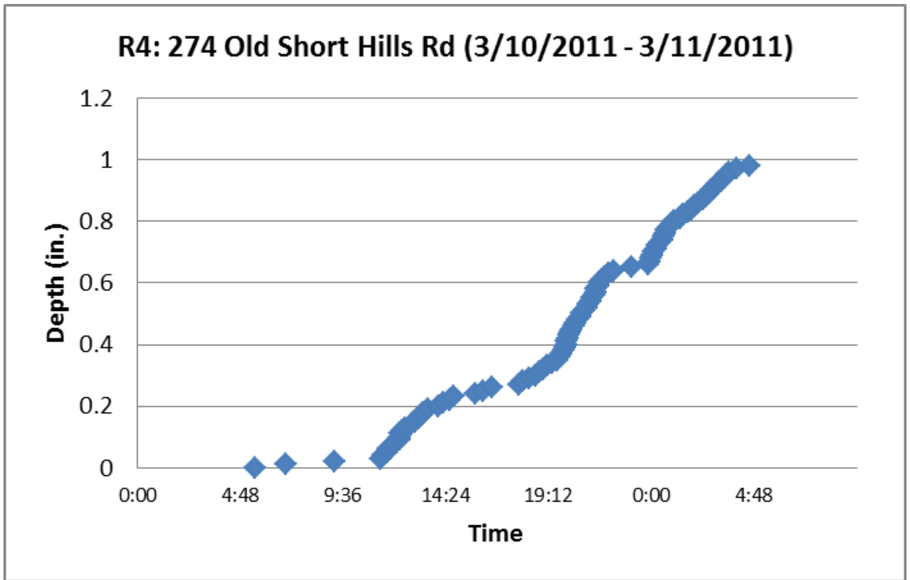
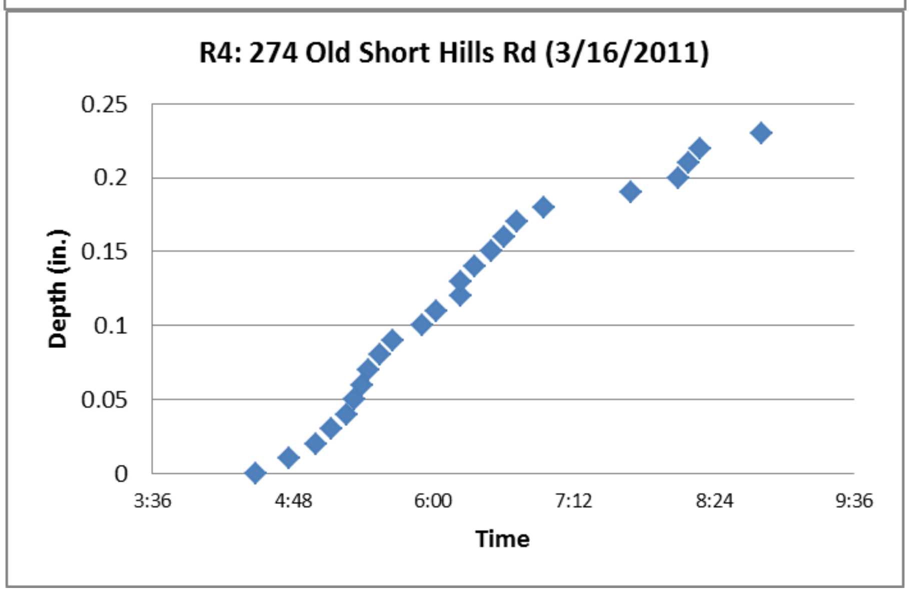
0.6
0.5
0.4
0.3
0.2
0.1
0
19:12
21:36
0:00
2:24
4:48
7:12
9:36
12:00
14:24
16:48
Time


R4: 274 Old Short Hills Rd (2/28/2011)
Depth (in.)
0.25
0.2
0.15
0.1
0.05
0
3:36
4:48
6:00
7:12
8:24
9:36
10:48
12:00
Time


R4: 274 Old Short Hills Rd (3/6/2011 - 3/7/2011)
Depth (in.)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
7:12
9:36
12:00
14:24
16:48
19:12
21:36
0:00
2:24
4:48
Time





Infiltration Analysis

Tables 9 to 30 present summary of Horton parameters and rain characteristics as well as statistical analysis for each dry well for infiltration study test and different rain events. Table 6 is a site summary by event showing the test conditions, the Horton parameter values, rain depth, and maximum and minimum dry well water levels during the event. Also noted is the likely presence of high water table conditions at the end of the monitoring event.

11 Woodfield Dr

Table C-1. Summary of infiltration hydrant water test (11 Woodfield Dr)

Date	Horton's parameters			Study Test				Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-13-2009	13.945	1.2	0.012	10/13/2009 10:07	10/13/2009 10:30	0:23	156.52	43.68	0.72

Table C-2. Summary of Horton parameters (f_o, f_c, and k) for rain events (11 Woodfield Dr)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
10-24-2009	2.987	0.95	0.005	10/23/2009 5:40	10/24/2009 20:30	25:33	2.20	0.09	28.11	0.57
12-09-2009	4.117	0.72	0.006	12/9/2009 0:03	12/9/2009 11:38	11:35	2.01	0.17	39.12	0.03

Table C-3. Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (11 Woodfield Dr)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	COV	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
10/28/2009	0.83	0.13	0.16	10/27/2009 5:40	10/28/2009 14:04	32:24	1.6	0.05	11.1	0.45
12/13/2009	0.44	0.25	0.56	12/13/2009 10:39	12/13/2009 18:38	7:59	0.99	0.12	9.02	0.2

Table C-4. Statistical Analysis for Horton parameters (fo, fc, and k) (11 Woodfield Dr)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	3	5	3	5	5
Minimum	2.99	0.44	0.01	9.02	0.03
Maximum	13.95	1.20	0.01	43.68	0.72
Average	7.02	0.83	0.01	26.21	0.39
Std Dev	6.03	0.28	0.00	15.81	0.28
COV	0.86	0.34	0.49	0.60	0.71

15 Marion Ave

Table C-5. Summary of infiltration rain events (15 Marion Ave)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
6-16-2010	9.95	0.5	0.02	6/16/2010 23:45	6/17/2010 0:41	0:56	0.69	0.74	56.74	0.36

Table C-6. Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (15 Marion Ave)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
6-22-2010	0.20	0.16	0.8	6/22/2010 18:33	6/22/2010 18:54	0:21	0.37	1.06	6.51	0.25
7-14-2010	0.30	0.18	0.6	7/14/2010 8:21	7/14/2010 10:04	1:02	1.22	1.18	23.02	0.35
8-1-2010	0.34	0.26	0.76	8/1/2010 8:21	8/1/2010 9:54	1:33	1.34	0.86	26.85	0.25

Table C-7. Statistical Analysis for Horton parameters (fo, fc, and k) (15 Marion Ave)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	1	4	1	4	4
Minimum	9.95	0.20	0.02	6.51	0.25
Maximum	9.95	0.50	0.02	56.74	0.36
Average	9.95	0.34	0.02	28.28	0.30
Std Dev	n/a	0.12	n/a	20.93	0.06
COV	n/a	0.37	n/a	0.74	0.20

258 Main St

Table C-8. Summary of Horton parameters (f_o, f_c, and k) for different events (258 Main St)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
06-17-2010	34.653	5.308	0.06	6/16/2010 23:45	6/17/2010 0:41	0:56	0.69	0.74	22.32	0.11
07-14-2010	75.142	6.808	0.07	7/14/2010 8:21	7/14/2010 10:04	1:02	1.22	1.18	53.62	0.67
08-01-2010	74.916	4.662	0.045	8/1/2010 8:21	8/1/2010 9:54	1:33	1.34	0.86	54.77	0.53

Table C-9. Statistical Analysis for Horton parameters (fo, fc, and k) (258 Main St)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	3	3	3	3	3
Minimum	34.65	4.66	0.05	22.32	0.11
Maximum	75.14	6.81	0.07	54.77	0.67
Average	61.57	5.59	0.06	43.57	0.44
Std	23.31	1.10	0.01	18.41	0.29
COV	0.38	0.20	0.22	0.42	0.67

2 Undercliff Rd

Table C-10. Summary of infiltration hydrant water test (2 Undercliff Rd)

Date	Horton's parameters			Study Test				Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-2-2009	3.881	0.566	0.013	10/2/2009 9:07	10/2/2009 9:26	0:19	131.58	54.21	0.23

Table C-11. Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (2 Undercliff Rd)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
7-29-2009	2.368	0.007	0.003	7/29/2009 10:00	7/29/2009 19:13	9:13	1.33	0.14	9.16	5.01 (high watertable)
8-2-2009	0.17	0.093	0.55	8/2/2009 6:29	8/2/2009 12:55	6:26	1.31	0.2	16.54	0.39

Table C-12 Statistical Analysis for Horton parameters (fo, fc, and k) (2 Undercliff Rd)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	1	3	1	3	3
Minimum	3.88	0.17	0.01	9.16	0.23
Maximum	3.88	2.37	0.01	54.21	5.01
Average	3.88	1.03	0.01	26.64	1.88
Std	n/a	1.17	n/a	24.16	2.71
COV	n/a	1.13	n/a	0.91	1.45

383 Wyoming Ave

Table C-13 Summary of infiltration hydrant water test (383 Wyoming Ave)

Date	Horton's parameters			Study Test				Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-2-2009	5.631	1.171	0.0045	10/2/2009 10:14	10/2/2009 10:43	0:29	100.00	40.65	0.53

Table C-14 Summary of Horton parameters (f_o, f_c, and k) for rain events (383 Wyoming Ave)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
7-26-2009	3.188	0.659	0.005	7/26/2009 16:46	7/26/2009 23:22	6:36	1.37	0.21	22.73	0.22
7-29-2009	10.253	1.139	0.0035	7/29/2009 10:00	7/29/2009 19:13	9:13	1.33	0.14	75.85	7.34 (high watertable)
8-02-2009	5.45	0.928	0.003	8/2/2009 6:29	8/2/2009 12:55	6:26	1.31	0.2	77.87	0.43
8-22-2009	3.623	1.186	0.03	8/21/2009 23:54	8/22/2009 10:43	10:49	1.9	0.18	35.82	0.37

Table C-15 Statistical Analysis for Horton parameters (f_o, f_c, and k) (383 Wyoming Ave)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	5	5	5	5	5
Minimum	3.19	0.66	0.00	22.73	0.22
Maximum	10.25	1.19	0.03	77.87	7.34
Average	5.63	1.02	0.01	50.58	1.78
Std	2.80	0.23	0.01	24.88	3.11
COV	0.50	0.22	1.27	0.49	1.75

260 Hartshorn Dr

Table C-17 Summary of Horton parameters (f_o, f_c, and k) for different events (260 Hartshorn Dr)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-10-2010	8.774	0.4	0.009						53.76	15.35 (high watertable)
08-22-2010	8.4097	0.6	0.011	8/22/2010 12:41	8/22/2010 20:42	8:01	1.51	0.19	55.71	28.81 (high watertable)
08-25-2010	1.0131	0.23	0.02	8/25/2010 2:00	8/25/2010 10:21	8:21	0.43	0.05	46.52	0.77
09-16-2010	2.411	0.3	0.005	9/16/2010 15:55	9/16/2010 22:14	6:19	0.61	0.10	40.20	8.49 (high watertable)
09-30-2010	8.158	0.65	0.03	9/30/2010 4:14	9/30/2010 10:01	5:47	1.83	0.32	56.81	38.64 (high watertable)
10-01-2010	5.862	0.7	0.02	10/1/2010 1:41	10/1/2010 13:05	11:24	2.53	0.22	63.97	7.41 (high watertable)
02-25-2011	1.897	0.4	0.02	2/25/2011 0:25	2/25/2011 18:44	18:19	1.36	0.06	54.45	36.27 (high watertable)
03-07-2011	1.586	0.4	0.002	3/6/2011 7:55	3/7/2011 3:29	19:34	2.78	0.14	54.47	31.64 (high watertable)
06-17-2011	9.6229	0.6	0.05	6/17/2011 13:45	6/17/2011 18:22	4:37	2.78	0.62	56.00	18.24 (high watertable)
07-08-2011	9.284	0.45	0.035	7/8/2011 16:02	7/8/2011 20:46	4:44	0.73	0.15	55.19	1.14
08-01-2011	1.434	0.25	0.015	8/1/2011 0:25	8/1/2011 0:42	0:17	0.46	1.62	31.74	24.38 (high watertable)
08-04-2011	3.045	0.6	0.008	8/3/2011 16:28	8/4/2011 4:02	11:34	0.65	0.06	49.56	5.40 (high watertable)

Table C-18 Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (260 Hartshorn Dr)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-16-2010	0.21	0.13	0.6	8/16/2010 16:24	8/16/2010 20:53	4:29	0.26	0.06	29.91	7.87 (high watertable)
09-13-2010	0.23	018	081	9/13/2010 17:00	9/13/2010 17:58	0:58	0.51	0.53	28.47	14.62 (high watertable)
09-27-2010	0.21	0.25	1.19	9/27/2010 7:40	9/28/2010 12:36	28:56	0.69	0.02	29.58	20.48 (high watertable)
05-23-2011	0.23	0.21	0.93	5/23/2011 22:19	5/23/2011 23:17	0:58	0.68	0.70	41.68	15.31 (high watertable)
05-30-2011	0.19	0.11	0.6	5/30/2011 6:07	5/30/2011 6:41	0:34	0.27	0.48	24.07	0.94
06-11-2011	0.22	0.15	0.68	6/11/2011 1:26	6/11/2011 5:29	4:03	0.56	0.14	19.16	11.72 (high watertable)
07-03-2011	0.18	0.11	0.62	7/3/2011 4:46	7/3/2011 21:23	16:37	0.33	0.02	19.74	14.67 (high watertable)

Table C-19 Statistical Analysis for Horton parameters (f_o, f_c, and k) (260 Hartshorn Dr)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	12	19	12	19	19
Minimum	1.01	0.18	0.00	19.16	0.77
Maximum	9.62	0.70	0.05	63.97	38.64
Average	5.12	0.37	0.02	42.68	15.90
Std	3.52	0.18	0.01	14.35	11.64
COV	0.69	0.48	0.74	0.34	0.73

87/89 Tennyson Dr

Table C-20 Summary of Horton parameters (f_o, f_c, and k) for rain events (87/89 Tennyson Dr)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
09-30-2010	1.717	0.196	0.006	9/30/2010 4:14	9/30/2010 10:01	5:47	1.83	0.32	89.08	82.98 (high watertable)
10-01-2010	1.721	0.251	0.008	10/1/2010 1:41	10/1/2010 13:05	11:24	2.53	0.22	93.08	35.37 (high watertable)
03-06-2011	3.281	0.45	0.015	3/6/2011 7:55	3/7/2011 3:29	19:34	2.78	0.14	93.85	82.135 (high watertable)
03-11-2011	2.899	0.28	0.015	3/10/2011 2:47	3/11/2011 8:05	29:18	2.90	0.10	93.37	46.85 (high watertable)
06-17-2011	10.99	0.28	0.12	6/17/2011 13:45	6/17/2011 18:22	4:37	2.78	0.62	91.17	64.71 (high watertable)

Table C-21 Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (87/89 Tennyson Dr)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-10-2010	0.18	0.12	0.64						67.23	45.66 (high watertable)
08-23-2010	0.199	0.14	0.72	8/22/2010 12:41	8/22/2010 20:42	8:01	1.51	0.19	80.90	74.83 (high watertable)
08-25-2010	0.18	0.12	0.67	8/25/2010 2:00	8/25/2010 10:21	8:21	0.43	0.05	83.47	34.33 (high watertable)
09-14-2010	0.16	0.10	0.64	9/13/2010 17:00	9/13/2010 17:58	0:58	0.51	0.53	50.06	45.91 (high watertable)
09-28-2010	0.35	0.33	0.94	9/27/2010 7:40	9/28/2010 12:36	28:56	0.69	0.02	51.81	48.77 (high watertable)
11-05-2010	0.26	0.19	0.73	11/4/2010 3:26	11/5/2010 7:35	28:09	1.16	0.04	58.29	26.45 (high watertable)
12-01-2010	0.23	0.18	0.79	12/1/2010 1:05	12/1/2010 15:07	14:02	1.88	0.13	71.94	44.4 (high watertable)
12-13-2010	0.26	0.21	0.81	12/12/2010 0:57	12/13/2010 6:51	29:54	1.87	0.06	83.88	26.63 (high watertable)
02-28-2011	0.27	0.21	0.78						89.79	74.40 (high)
05-23-2011	0.22	0.17	0.75	5/23/2011 22:19	5/23/2011 23:17	0:58	0.68	0.70	83.67	69.66 (high watertable)
05-30-2011	0.15	0.10	0.68	5/30/2011 6:07	5/30/2011 6:41	0:34	0.27	0.48	74.62	58.65 (high watertable)
06-11-2011	0.18	0.13	0.73	6/11/2011 1:26	6/11/2011 5:29	4:03	0.56	0.14	69.38	63.55 (high watertable)
07-08-2011	0.22	0.14	0.65	7/8/2011 16:02	7/8/2011 20:46	4:44	0.73	0.15	81.71	46.41 (high watertable)
08-01-2011	0.18	0.13	0.7	8/1/2011 0:25	8/1/2011 0:42	0:17	0.46	1.62	61.96	56.53 (high watertable)
08-04-2011	0.18	0.08	0.48	8/3/2011	8/4/2011	11:34	0.65	0.06	73.23	72.4 (high

				16:28	4:02					watertable)

Table C-22 Statistical Analysis for Horton parameters (f_o, f_c, and k) (87/89 Tennyson Dr)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	5	20	5	20	20
Minimum	1.72	0.20	0.01	50.06	26.45
Maximum	10.99	0.45	0.12	93.85	82.98
Average	4.12	0.29	0.03	77.12	5.03
Std	3.90	0.10	0.05	13.82	17.60
COV	0.95	0.33	1.49	0.18	0.32

1 Sinclair Terrace

Table C-23 Summary of infiltration hydrant water test (1 Sinclair Terrace)

Date	Horton's parameters			Study Test				Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
07-15-2009	3.306	0.700	0.0015	7/15/2009 10:40	7/15/2009 11:30	0:50	66.00	51.02	0

142 Fairfield Dr

Table C-24 Summary of Horton parameters (f_o, f_c, and k) for rain events (142 Fairfield Dr)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-10-2010	3.061	0.051	0.01						35.55	0.64
10-01-2010	3.010	0.61	0.002	10/1/2010 2:17	10/1/2010 16:48	14:31	1.73	0.12	73.75	0.28
10-07-2010	1.543	0.548	0.01						25.95	0.49

Table C-25 Summary of Horton parameters (f_c; f_o and k are n/a) for different rains having "constant" infiltration rates (142 Fairfield Dr)

Date	f_c infiltration rate (in./hr)			Rain Characteristics					Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-22-2010	0.33	0.63	1.87	8/22/2010 11:20	8/22/2010 19:19	7:59	1.43	0.18	28.82	0.47
12-01-2010	0.33	0.18	0.56	12/1/2010 2:15	12/2/2010 1:05	22:50	0.67	0.02	24.59	0.56
02-26-2011	0.32	0.51	1.59	2/24/2011 21:58	2/25/2011 13:46	15:48	0.59	0.04	33.8	12.06 (high watertable)
03-07-2011	0.72	0.37	0.52	3/6/2011 9:00	3/7/2011 3:22	18:22	1.15	0.06	73.69	23.29 (high watertable)

Table C-26 Statistical Analysis for Horton parameters (f_o, f_c, and k) (142 Fairfield Dr)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	3	7	3	7	7
Minimum	1.54	0.05	0.00	24.59	0.28
Maximum	3.06	0.72	0.01	73.75	23.29
Average	2.54	0.42	0.01	42.31	5.40
Std	0.86	0.23	0.00	21.81	8.99
COV	0.34	0.54	0.63	0.52	1.67

8 South Beechcroft Rd

Table C-27 Summary of infiltration hydrant water test (8 Beechcroft Rd)

Date	Horton's parameters			Study Test				Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-02-2009	16.12	0.08	0.017	10/2/2009 12:07	10/2/2009 12:15	0:08	112.50	16.76	0.32

Table C-28 Summary of Horton parameters (f_o, f_c, and k) for rain events (8 Beechcroft Rd)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
7/26/2009	45.29	2.02	0.026	7/26/2009 16:46	7/27/2009 0:17	7:31	1.38	0.18	41.29	1.94
8/22/2009	45.95	0.3	0.011	8/21/2009 23:57	8/22/2009 18:28	18:28	1.71	0.09	47.04	0.10
8/29/2009	19.78	0.24	0.009	8/29/2009 5:45	8/29/2009 12:27	6:42	0.52	0.08	32.25	0.13

Table C-29 Statistical Analysis for Horton parameters (f_o, f_c, and k) (8 Beechcroft Rd)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	4	4	4	4	4
Minimum	16.12	0.08	0.01	16.76	0.10
Maximum	45.95	2.02	0.03	47.04	1.94
Average	31.79	0.66	0.02	34.34	0.62
Std	16.05	0.91	0.01	13.20	0.88
COV	0.50	1.38	0.48	0.38	1.42

7 Fox Hill Ln

Table C-30 Summary of Horton parameters (f_o, f_c, and k) for rain events (7 Fox Hill Ln)

Date	Horton's Parameters			Rain Characteristics					Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Depth (in.)	Average intensity (in./hr)	Max.	Min.
08-10-2010	3.667	0.19	0.013						50.13	18.32 (high watertable)
08-22-2010	2.800	0.57	0.014	8/22/2010 11:20	8/22/2010 19:19	7:59	1.43	0.18	58.29	1.34
09-30-2010	2.200	0.39	0.014	9/30/2010 4:20	9/30/2010 9:42	5:22	0.92	0.17	58.85	46.7 (high watertable)
10-01-2010	3.506	0.46	0.014	10/1/2010 2:17	10/1/2010 16:48	14:31	1.73	0.12	63.98	10.07 (high watertable)
11-05-2010	1.701	0.34	0.015						42.51	5.62 (high watertable)
12-01-2010	3.891	0.49	0.020	12/1/2010 2:15	12/2/2010 1:05	22:50	0.67	0.02	59.5	5.65 (high watertable)
12-13-2010	2.189	0.368	0.017	12/12/2010 17:25	12/13/2010 3:18	9:53	0.23	0.02	56.53	0.19
02-25-2011	3.116	0.45	0.020	2/24/2011 21:58	2/25/2011 13:46	15:48	0.59	0.04	57.44	41.48 (high watertable)
02-28-2011	1.941	0.423	0.019	2/28/2011 4:12	2/28/2011 11:32	7:20	0.22	0.03	56.39	23.97 (high watertable)
03-06-2011	2.748	0.40	0.021	3/6/2011 9:00	3/7/2011 3:22	18:22	1.15	0.06	56.73	42.19 (high watertable)
03-11-2011	1.924	0.276	0.018	3/10/2011 5:30	3/11/2011 4:33	23:03	0.98	0.04	58.05	25.86 (high watertable)

Table C-31 Statistical Analysis for Horton parameters (f_o, f_c, and k) (7 Fox Hill Ln)

Statistical Analysis	Horton's Parameters			Water Depth in Dry well (in.)	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Max.	Min.
Number of Events	11	11	11	11	11
Minimum	1.70	0.19	0.01	42.51	0.19
Maximum	3.89	0.57	0.02	63.98	46.70
Average	2.70	0.40	0.02	56.22	20.13
Std	0.77	0.10	0.00	5.59	17.24
COV	0.28	0.26	0.17	0.10	0.86

9 Fox Hill Ln

Table C-32 Summary of infiltration hydrant water test, "constant" rate (f_c; f_o and k are n/a) (9 Fox Hill Ln)

Date	f_c infiltration rate (in./hr)			Study Test				Water Depth in Dry well (in.)	
	Average	Std Deviation	Cov.	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-02-2009	0.12	0.16	1.32	10/2/2009 12:44	10/2/2009 13:15	0:31	83.87	21.06	9.023* (high watertable)

* on 10/12/2009

11 Fox Hill Ln

Table C-33 Summary of infiltration hydrant water test (11 Fox Hill Ln)

Date	Horton's parameters			Study Test				Horton's parameters	
	f_o (in./hr)	f_c (in./hr)	k (1/min)	Start Time	End Time	Duration (hr:min)	Fill Rate (gal/min)	Max.	Min.
10-02-2009	1.09	0.25	0.012	10/2/2009 13:16	10/2/2009 14:00	0:44	77.27	31.73	0.12*

* on 10/12/2009

Table 6a. 11 Woodfield Dr. (D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
hydrant	10/13/2009	13.945	1.2	0.012	n/a	43.68	0.72	OK
Horton	10/24/2009	2.987	0.95	0.005	2.2	28.11	0.57	OK
constant	10/28/2009	n/a	0.83	n/a	1.6	11.1	0.45	OK
Horton	12/9/2009	4.117	0.72	0.006	2.01	39.12	0.03	OK
constant	12/13/2009	n/a	0.44	n/a	0.99	9.02	0.2	OK
	number	3	5	3	4	5	5	
	Minimum	2.99	0.44	0.01	0.99	9.02	0.03	
	Maximum	13.95	1.20	0.01	2.20	43.68	0.72	
	Average	7.02	0.83	0.01	1.70	26.21	0.39	
	Std Dev	6.03	0.28	0.00	0.54	15.81	0.28	
	COV	0.86	0.34	0.49	0.31	0.60	0.71	

Table 6b. 15 Marion (D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	6/16/2010	9.95	0.5	0.02	0.69	56.74	0.36	OK
constant	6/22/2010	n/a	0.2	n/a	0.37	6.51	0.25	OK
constant	7/14/2010	n/a	0.3	n/a	1.22	23.02	0.35	OK
constant	8/1/2010	n/a	0.34	n/a	1.34	26.85	0.25	OK
	number	1	4	1	4	4	4	
	Minimum	9.95	0.20	0.02	0.37	6.51	0.25	
	Maximum	9.95	0.50	0.02	1.34	56.74	0.36	
	Average	9.95	0.34	0.02	0.91	28.28	0.30	
	Std Dev	n/a	0.12	n/a	0.45	20.93	0.06	
	COV	n/a	0.37	n/a	0.50	0.74	0.20	

Table 6c. 258 Main St. (A and D surface HSG soil conditions, and A subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	6/17/2010	34.653	5.308	0.06	0.69	22.32	0.11	OK
Horton	7/14/2010	75.142	6.808	0.07	1.22	53.62	0.67	OK
Horton	8/1/2010	74.916	4.662	0.045	1.34	54.77	0.53	OK
	number	3	3	3	3	3	3	
	Minimum	34.65	4.66	0.05	0.69	22.32	0.11	
	Maximum	75.14	6.81	0.07	1.34	54.77	0.67	
	Average	61.57	5.59	0.06	1.08	43.57	0.44	
	Std Dev	23.31	1.10	0.01	0.35	18.41	0.29	
	COV	0.38	0.20	0.22	0.32	0.42	0.67	

Table 6d. 2 Undercliff Rd (C surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
constant	7/29/2009	n/a	2.368	n/a	1.33	9.16	5.01	high
constant	8/2/2009	n/a	0.17	n/a	1.31	16.54	0.39	OK
hydrant	10/2/2009	3.881	0.566	0.013	n/a	54.21	0.23	OK
	number	1	3	1	2	3	3	
	Minimum	3.88	0.17	0.01	1.31	9.16	0.23	
	Maximum	3.88	2.37	0.01	1.33	54.21	5.01	
	Average	3.88	1.03	0.01	1.32	26.64	1.88	
	Std Dev	n/a	1.17	n/a	0.01	24.16	2.71	
	COV	n/a	1.13	n/a	0.01	0.91	1.45	

Table 6e. 383 Wyoming Ave (C surface HSG soil conditions, and A subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	7/26/2009	3.188	0.659	0.005	1.37	22.73	0.22	OK
Horton	7/29/2009	10.253	1.139	0.0035	1.33	75.85	7.34	high
Horton	8/2/2009	5.45	0.928	0.003	1.31	77.87	0.43	OK
Horton	8/22/2009	3.623	1.186	0.03	1.9	35.82	0.37	OK
hydrant	10/2/2009	5.631	1.171	0.0045	n/a	40.65	0.53	OK
	number	5	5	5	4	5	5	
	Minimum	3.19	0.66	0.00	1.31	22.73	0.22	
	Maximum	10.25	1.19	0.03	1.90	77.87	7.34	
	Average	5.63	1.02	0.01	1.48	50.58	1.78	
	Std Dev	2.80	0.23	0.01	0.28	24.88	3.11	
	COV	0.50	0.22	1.27	0.19	0.49	1.75	

Table 6f. 260 Hartshorn Dr (D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	8/10/2010	8.774	0.4	0.009	n/a	53.76	15.35	high
constant	8/16/2010	n/a	0.21	n/a	0.26	29.91	7.87	high
Horton	8/22/2010	8.4097	0.6	0.011	1.51	55.71	28.81	high
Horton	8/25/2010	1.0131	0.23	0.02	0.43	46.52	0.77	OK
constant	9/13/2010	n/a	0.23	n/a	0.51	28.47	14.62	high
Horton	9/16/2010	2.411	0.3	0.005	0.61	40.2	8.49	high
constant	9/27/2010	n/a	0.21	n/a	0.69	29.58	20.48	high
Horton	9/30/2010	8.158	0.65	0.03	1.83	56.81	38.64	high
Horton	10/1/2010	5.862	0.7	0.02	2.53	63.97	7.41	high
Horton	2/25/2011	1.897	0.4	0.02	1.36	54.45	36.27	high
Horton	3/7/2011	1.586	0.4	0.002	2.78	54.47	31.64	high
constant	5/23/2011	n/a	0.23	n/a	0.68	41.68	15.31	high
constant	5/30/2011	n/a	0.19	n/a	0.27	24.07	0.94	OK
constant	6/11/2011	n/a	0.22	n/a	0.56	19.16	11.72	high
Horton	6/17/2011	9.6229	0.6	0.05	2.78	56	18.24	high
constant	7/3/2011	n/a	0.18	n/a	0.33	19.74	14.67	high
Horton	7/8/2011	9.284	0.45	0.035	0.73	55.19	1.14	OK
Horton	8/1/2011	1.434	0.25	0.015	0.46	31.74	24.38	high
Horton	8/4/2011	3.045	0.6	0.008	0.65	49.56	5.4	high

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
	number	12	19	12	18	19	19	
	Minimum	1.01	0.18	0.00	0.26	19.16	0.77	
	Maximum	9.62	0.70	0.05	2.78	63.97	38.64	
	Average	5.12	0.37	0.02	1.05	42.68	15.90	
	Std Dev	3.52	0.18	0.01	0.87	14.35	11.64	
	COV	0.69	0.48	0.74	0.82	0.34	0.73	

Table 6g. 87/89 Tennyson Dr (D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
constant	8/10/2010	n/a	0.18	n/a	n/a	67.23	45.66	high
constant	8/23/2010	n/a	0.199	n/a	1.51	80.9	74.83	high
constant	8/25/2010	n/a	0.18	n/a	0.43	83.47	34.33	high
constant	9/14/2010	n/a	0.16	n/a	0.51	50.06	45.91	high
constant	9/28/2010	n/a	0.35	n/a	0.69	51.81	48.77	high
Horton	9/30/2010	1.717	0.196	0.006	1.83	89.08	82.98	high
Horton	10/1/2010	1.721	0.251	0.008	2.53	93.08	35.37	high
constant	11/5/2010	n/a	0.26	n/a	1.16	58.29	26.45	high
constant	12/1/2010	n/a	0.23	n/a	1.88	71.94	44.4	high
constant	12/13/2010	n/a	0.26	n/a	1.87	83.88	26.63	high
constant	2/28/2011	n/a	0.27	n/a	n/a	89.79	74.4	high
Horton	3/6/2011	3.281	0.45	0.015	2.78	93.85	82.135	high
Horton	3/11/2011	2.899	0.28	0.015	2.9	93.37	46.85	high
constant	5/23/2011	n/a	0.22	n/a	0.68	83.67	69.66	high
constant	5/30/2011	n/a	0.15	n/a	0.27	74.62	58.65	high
constant	6/11/2011	n/a	0.18	n/a	0.56	69.38	63.55	high
Horton	6/17/2011	10.99	0.28	0.12	2.78	91.17	64.71	high
constant	7/8/2011	n/a	0.22	n/a	0.73	81.71	46.41	high
constant	8/1/2011	n/a	0.18	n/a	0.46	61.96	56.53	high
constant	8/4/2011	n/a	0.18	n/a	0.65	73.23	72.4	high
	number	5	20	5	18	20	20	
	Minimum	1.72	0.20	0.01	0.27	50.06	26.45	
	Maximum	10.99	0.45	0.12	2.90	93.85	82.98	
	Average	4.12	0.29	0.03	1.35	77.12	55.03	
	Std Dev	3.90	0.10	0.05	0.93	13.82	17.60	

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
	COV	0.95	0.33	1.49	0.69	0.18	0.32	

Table 6h. 1 Sinclair Terrace (D surface HSG soil conditions, and A subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
hydrant	7/15/2009	3.306	0.7	0.0015		51.02	0	OK

Table 6i. 142 Fairfield Dr (D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	8/10/2010	3.061	0.051	0.01	n/a	35.55	0.64	OK
constant	8/22/2010	n/a	0.33	n/a	n/a	28.82	0.47	OK
Horton	10/1/2010	3.01	0.61	0.002	1.73	73.75	0.28	OK
Horton	10/7/2010	1.543	0.548	0.01	n/a	25.95	0.49	OK
constant	12/1/2010	n/a	0.33	n/a	n/a	24.59	0.56	OK
constant	2/26/2011	n/a	0.32	n/a	n/a	33.8	12.06	high
constant	3/7/2011	n/a	0.72	n/a	n/a	73.69	23.29	high
	number	3	7	3	1	7	7	
	Minimum	1.54	0.05	0.00	1.73	24.59	0.28	
	Maximum	3.06	0.72	0.01	1.73	73.75	23.29	
	Average	2.54	0.42	0.01	1.73	42.31	5.40	
	Std Dev	0.86	0.23	0.00	n/a	21.81	8.99	
	COV	0.34	0.54	0.63	n/a	0.52	1.67	

Table 6j. 8 So. Beechcroft Rd (2 years old, D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	7/26/2009	45.29	2.02	0.026	1.38	41.29	1.94	OK
Horton	8/22/2009	45.95	0.3	0.011	1.71	47.04	0.1	OK
Horton	8/29/2009	19.78	0.24	0.009	0.52	32.25	0.13	OK
hydrant	10/2/2009	16.12	0.08	0.017	n/a	16.76	0.32	OK
	number	4	4	4	3	4	4	
	Minimum	16.12	0.08	0.01	0.52	16.76	0.10	
	Maximum	45.95	2.02	0.03	1.71	47.04	1.94	
	Average	31.79	0.66	0.02	1.20	34.34	0.62	
	Std Dev	16.05	0.91	0.01	0.61	13.20	0.88	
	COV	0.50	1.38	0.48	0.51	0.38	1.42	

Table 6k. 7 Fox Hill Lane (2.3 years old, D surface HSG soil conditions, and A and B subsurface soil conditions)

test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
Horton	8/10/2010	3.667	0.19	0.013	n/a	50.13	18.32	high
Horton	8/22/2010	2.8	0.57	0.014	1.43	58.29	1.34	OK
Horton	9/30/2010	2.2	0.39	0.014	0.92	58.85	46.7	high
Horton	10/1/2010	3.506	0.46	0.014	1.73	63.98	10.07	high
Horton	11/5/2010	1.701	0.34	0.015	n/a	42.51	5.62	high
Horton	12/1/2010	3.891	0.49	0.02	0.67	59.5	5.65	high
Horton	12/13/2010	2.189	0.368	0.017	0.23	56.53	0.19	OK
Horton	2/25/2011	3.116	0.45	0.02	0.59	57.44	41.48	high
Horton	2/28/2011	1.941	0.423	0.019	0.22	56.39	23.97	high
Horton	3/6/2011	2.748	0.4	0.021	1.15	56.73	42.19	high
Horton	3/11/2011	1.924	0.276	0.018	0.98	58.05	25.86	high
	number	11	11	11	9	11	11	
	Minimum	1.70	0.19	0.01	0.22	42.51	0.19	
	Maximum	3.89	0.57	0.02	1.73	63.98	46.70	
	Average	2.70	0.40	0.02	0.88	56.22	20.13	
	Std Dev	0.77	0.10	0.00	0.51	5.59	17.24	
	COV	0.28	0.26	0.17	0.58	0.10	0.86	

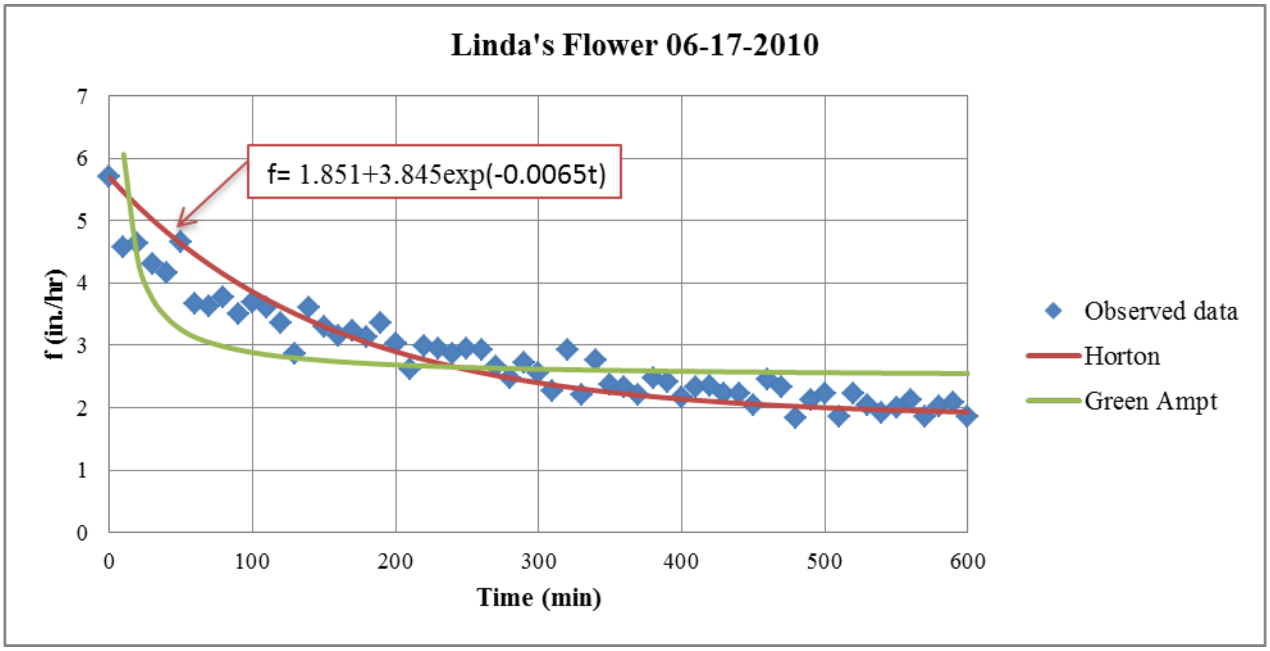
Table 6l. 9 Fox Hill Lane (D surface HSG soil conditions, and A and B subsurface soil conditions)

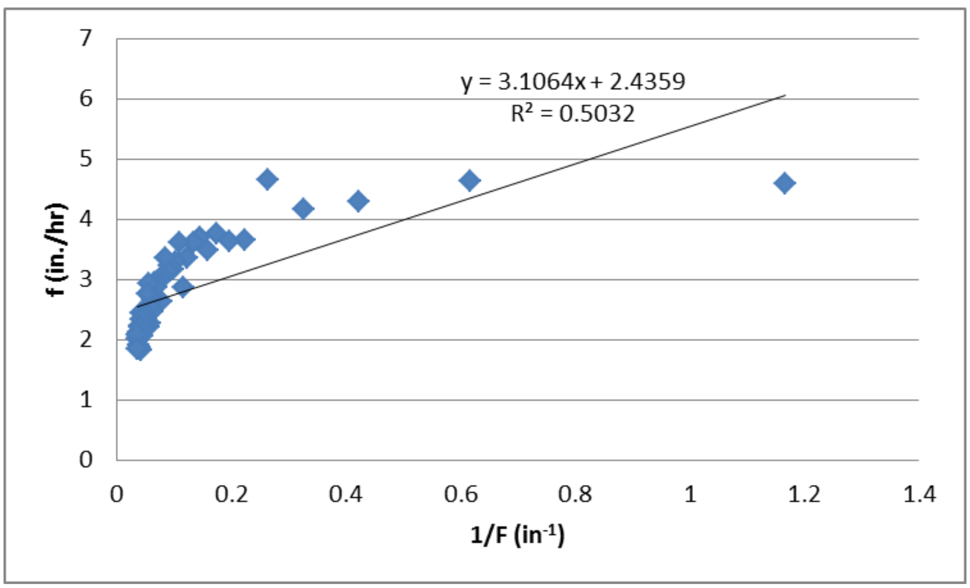
test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
hydrant	10/2/2009	n/a	0.12	n/a	n/a	21.06	9.023	high

Table 6m. 11 Fox Hill Lane (D surface HSG soil conditions, and A and B subsurface soil conditions)

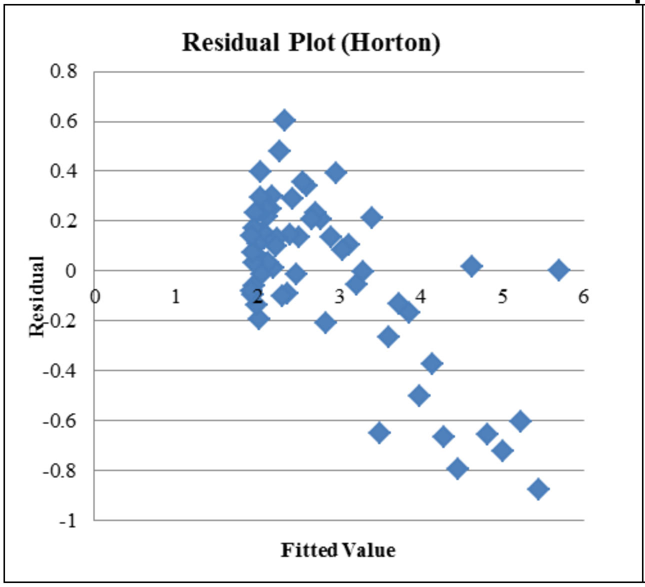
test conditions	date	f_o (in./hr)	f_c (in./hr)	k (1/min)	Rain Depth (in.)	Max. depth (in.)	Min. depth (in.)	water table conditions
hydrant	10/2/2009	1.09	0.25	0.012	n/a	31.73	0.12	OK

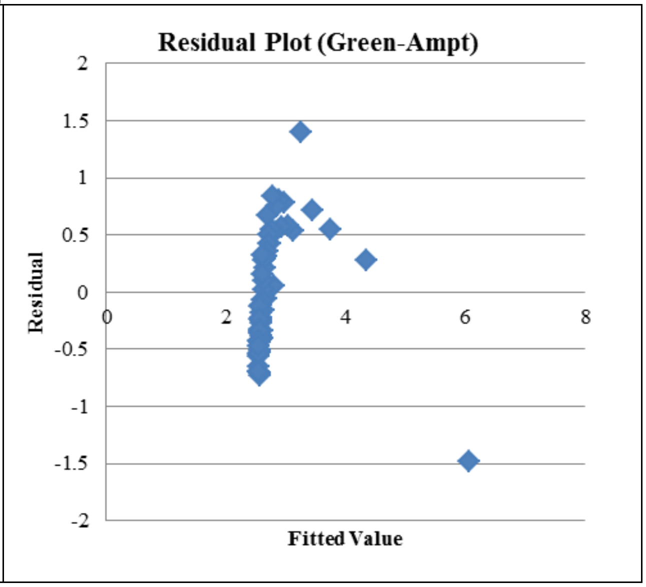
Linda's Flower 06-17-2010





Residual Plots for Horton and Green-Ampt fitted values



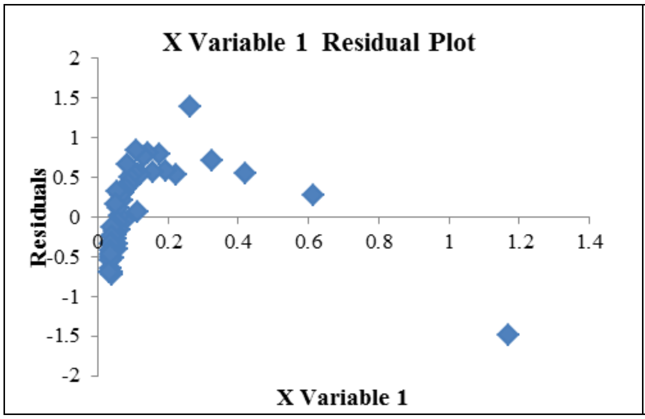


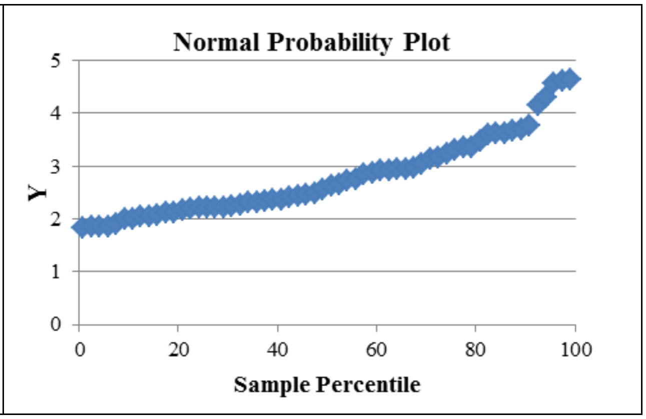
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

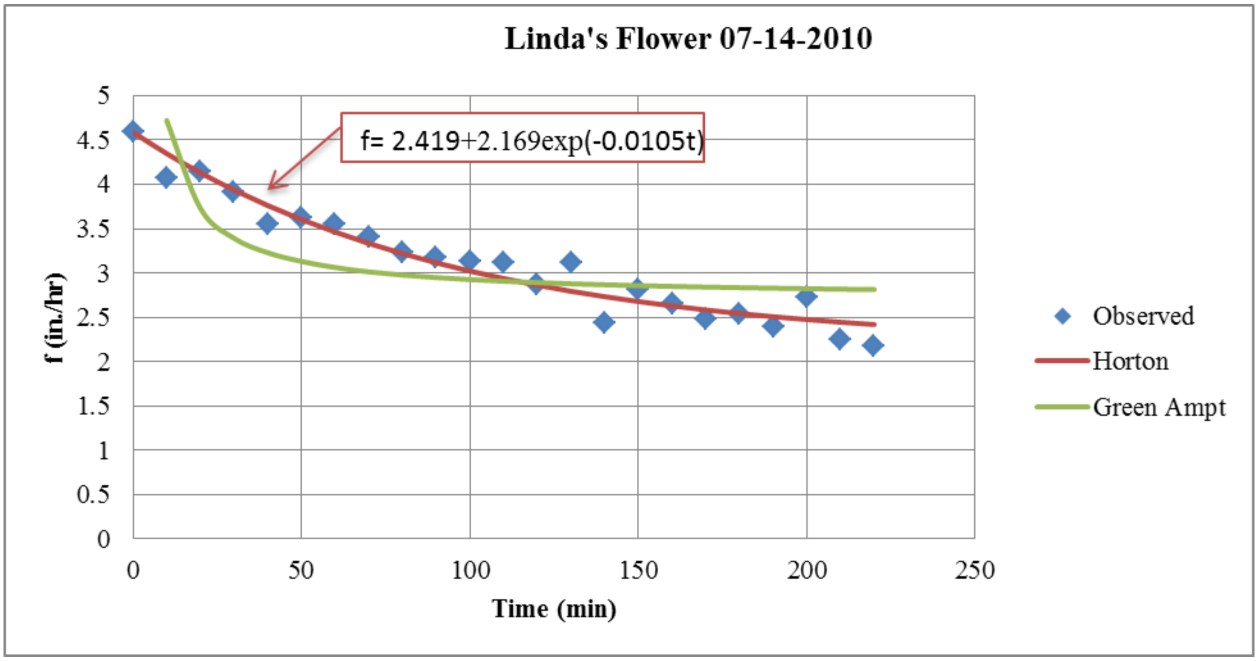
	df	SS	MS	F	Significance F
Regression	1	16.62278	16.62278	58.74998	2.24E-10
Residual	58	16.41058	0.282941		
Total	59	33.03335			

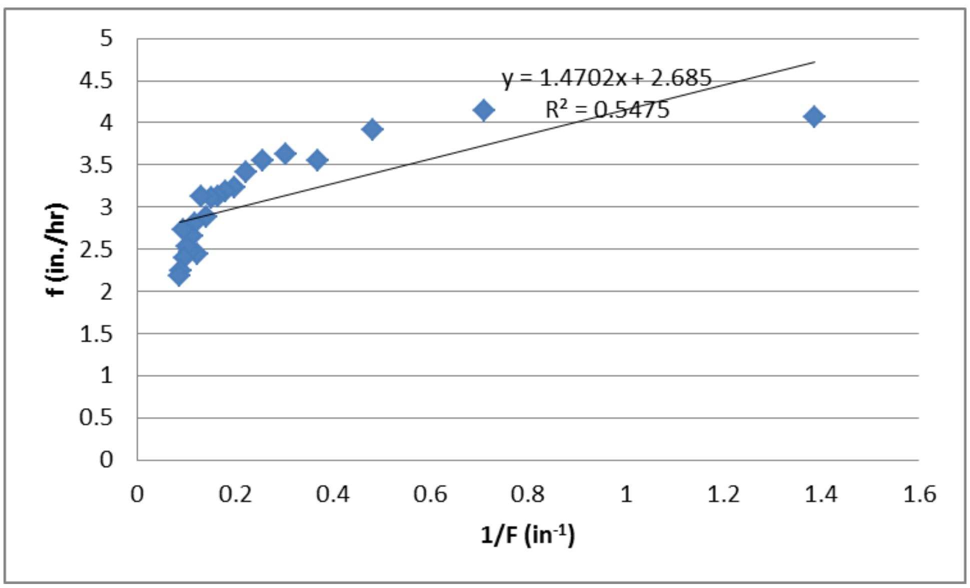
	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	2.435893	0.082297	29.5987	1.07E-36	2.271157	2.600629
X Variable 1	3.106365	0.405274	7.664854	2.24E-10	2.295121	3.917609



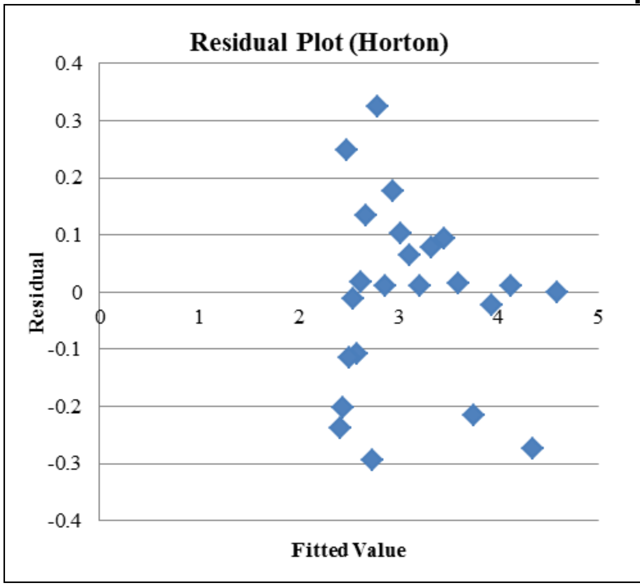


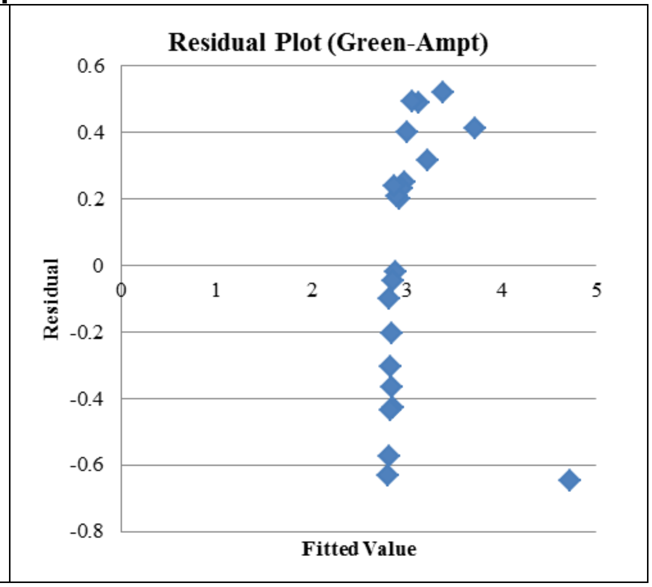
Linda's Flower 07-14-2010





Residual Plots for Horton and Green-Ampt fitted values



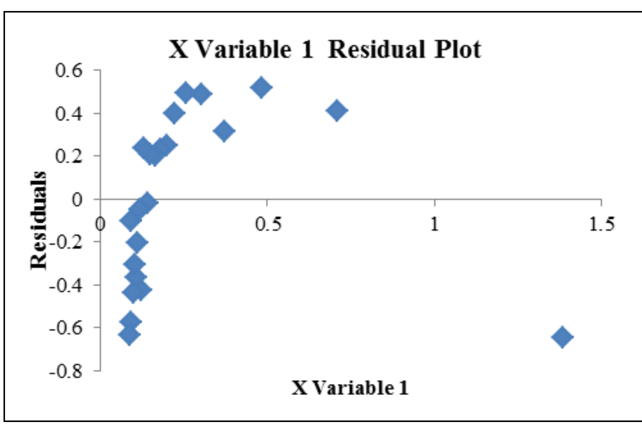


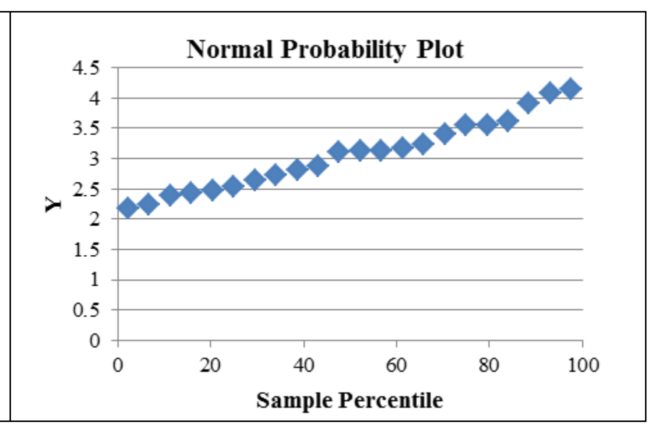
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

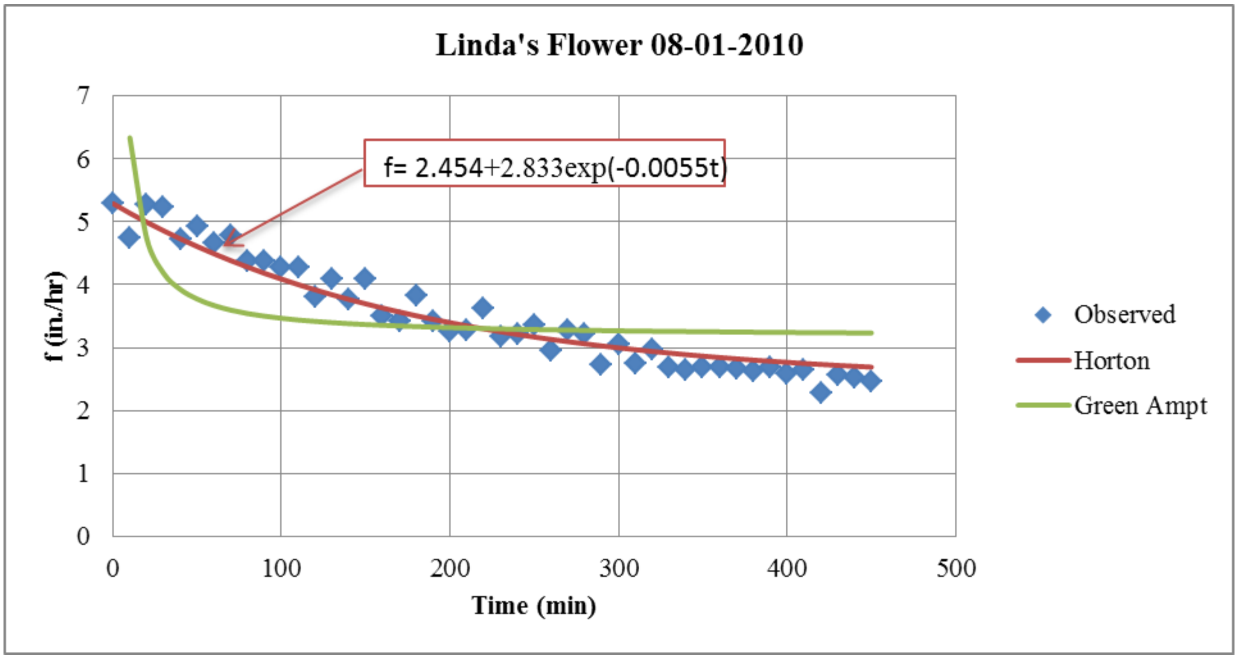
	df	SS	MS	F	Significance F
Regression	1	3.929311	3.929311	24.20193	8.27E-05
Residual	20	3.247105	0.162355		
Total	21	7.176416			

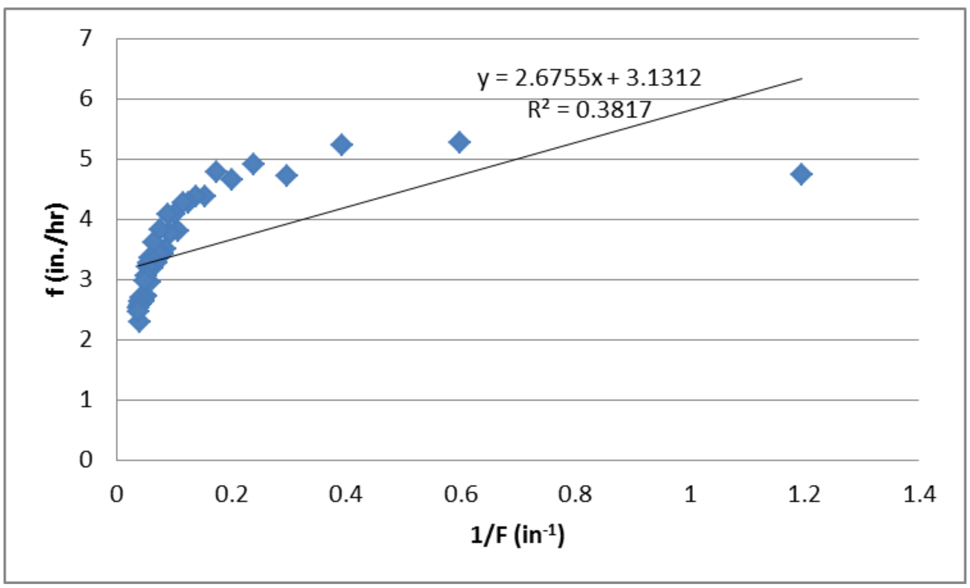
	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	2.684982	0.115014	23.34489	5.53E-16	2.445068	2.924897
X Variable 1	1.47021	0.298851	4.919545	8.27E-05	0.846818	2.093602



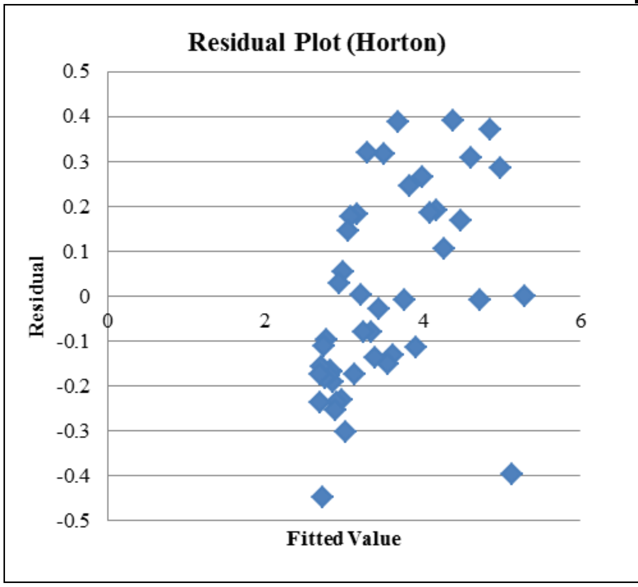


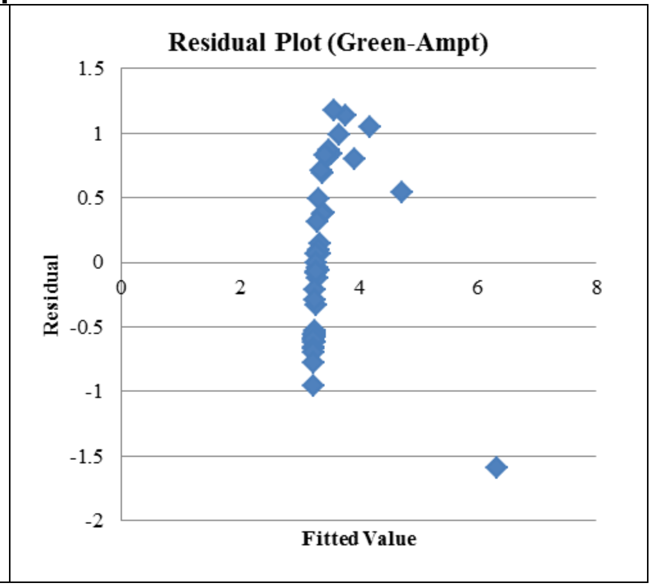
Linda's Flower 08-01-2010





Residual Plots for Horton and Green-Ampt fitted values



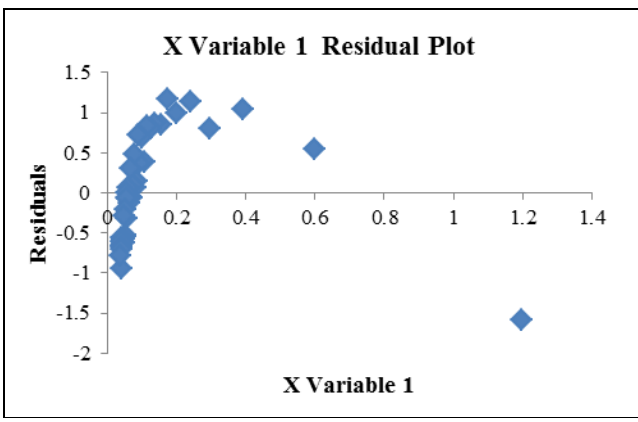


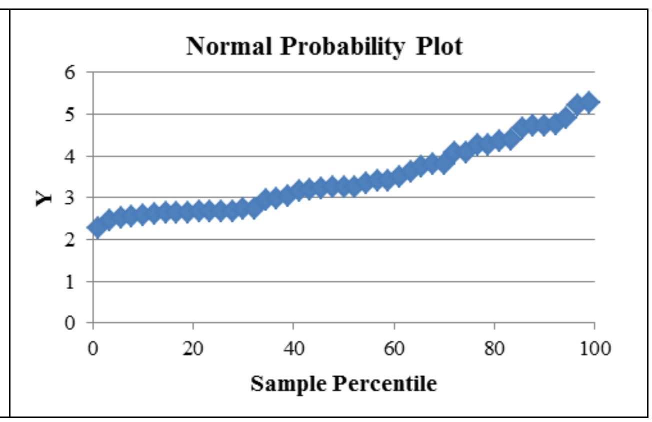
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

	df	SS	MS	F	Significance F
Regression	1	11.85864	11.85864	26.54858	6.13E-06
Residual	43	19.20711	0.446677		
Total	44	31.06574			

	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	3.13121	0.118969	26.31959	3.81E-28	2.891286	3.371133
X Variable 1	2.675481	0.519256	5.152531	6.13E-06	1.628302	3.72266





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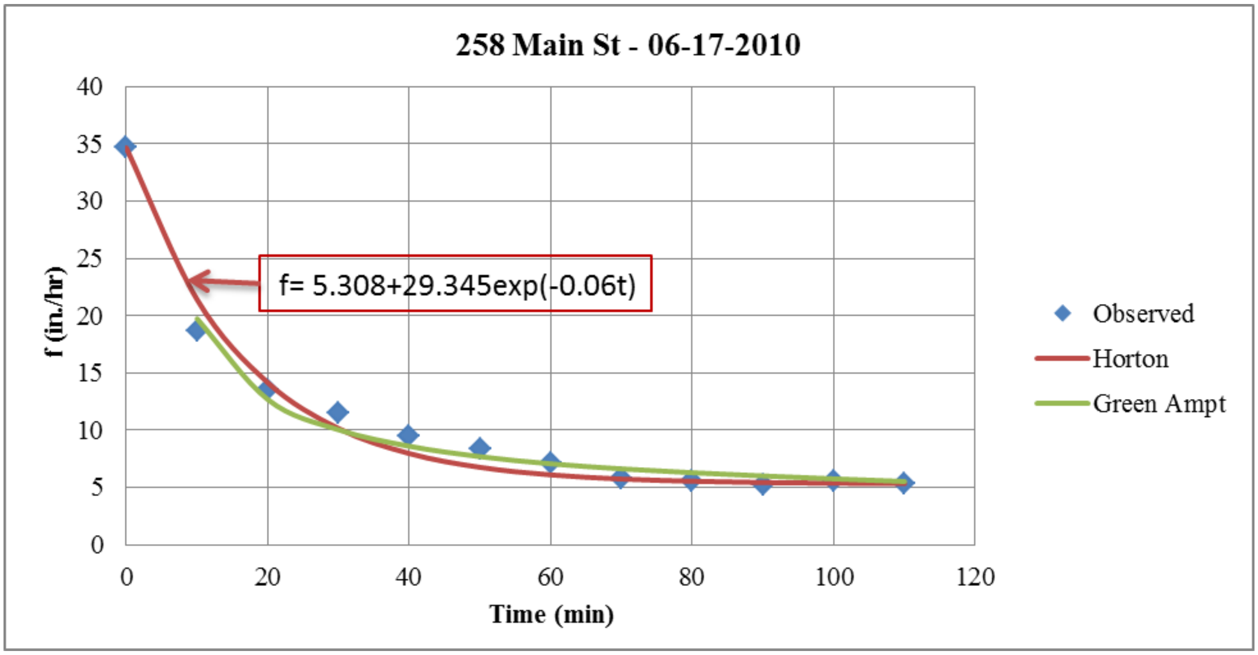
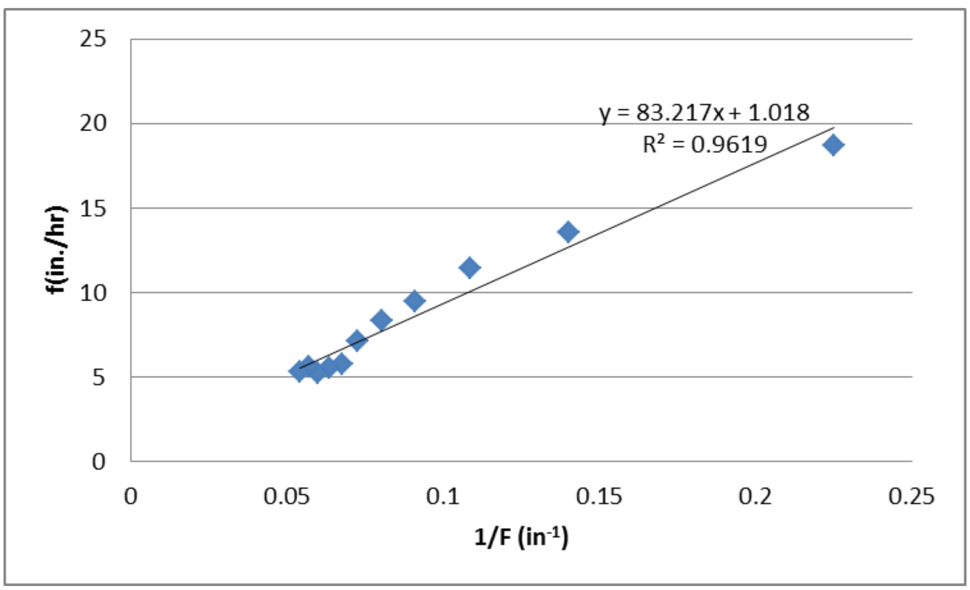
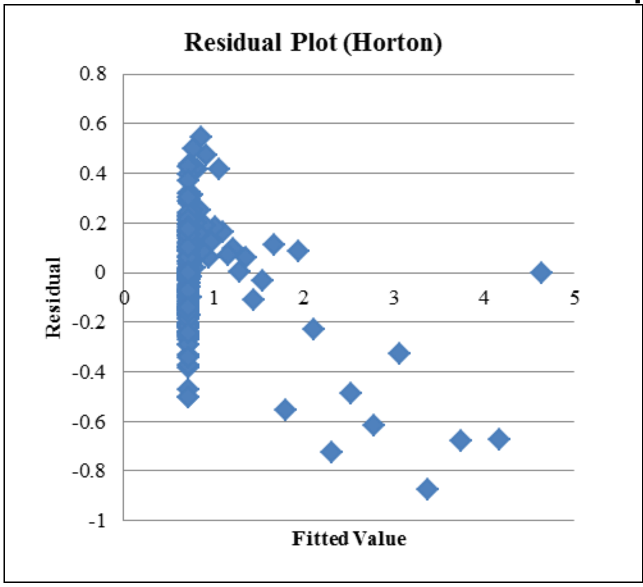
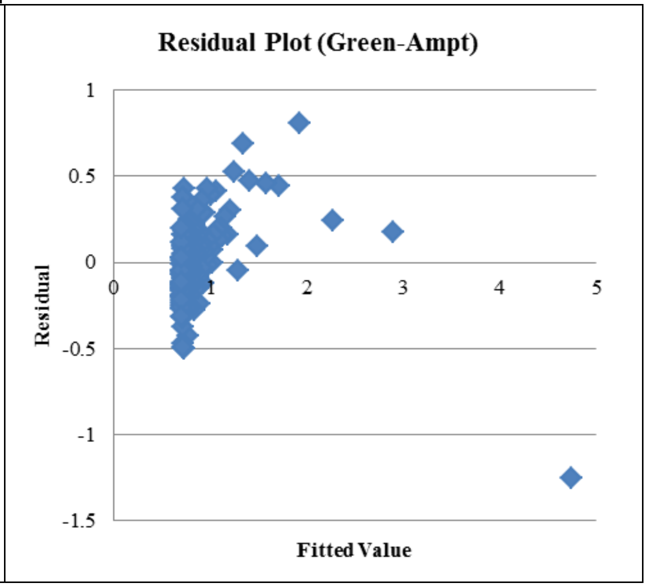




Residual Plots for Horton and Green-Ampt fitted values



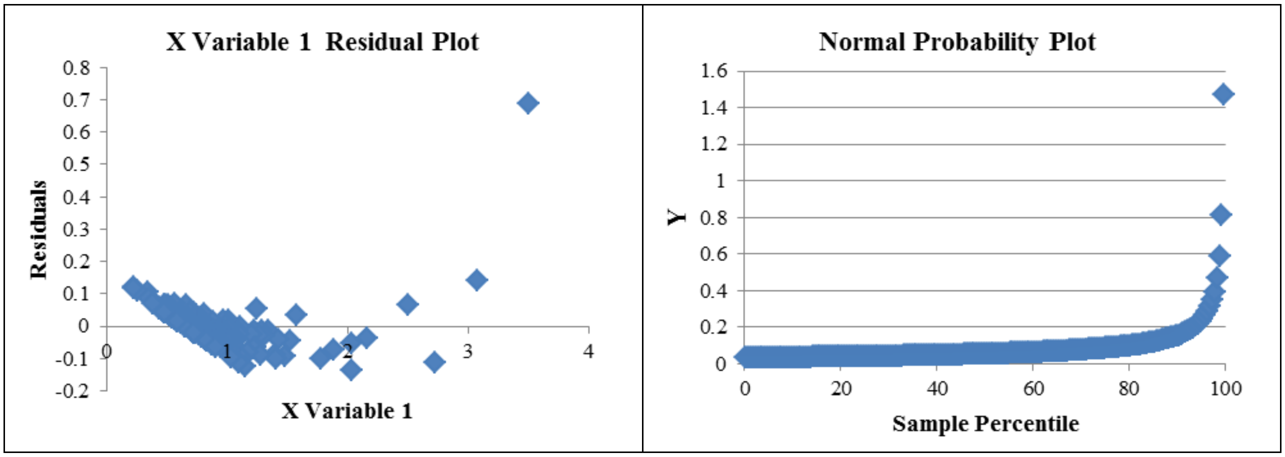


Regression Analysis for f vs. 1/F (Green-Ampt)

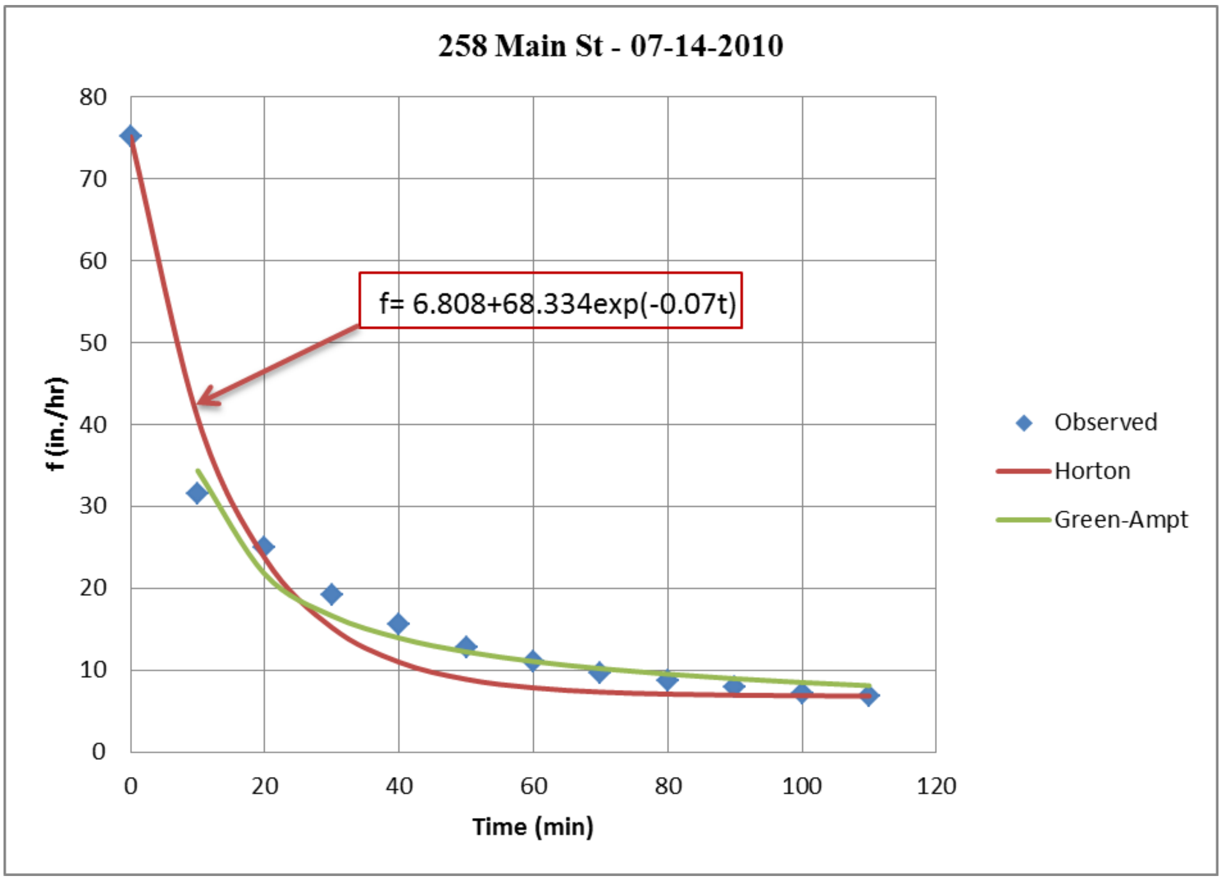
ANOVA

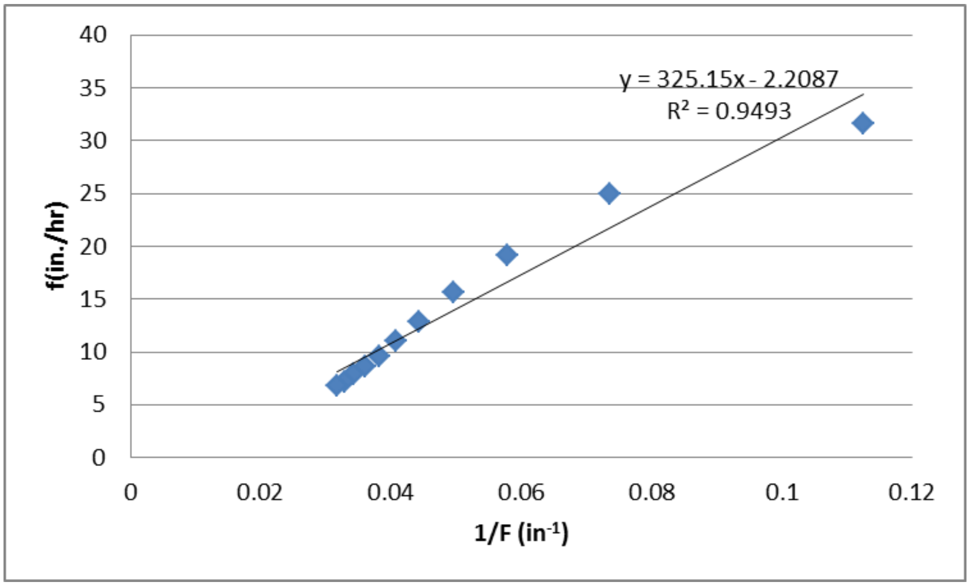
	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2.60167	2.60167	561.9697	9.75E-60
Residual	198	0.916652	0.00463		
Total	199	3.518323			

	Coefficients	*Standard Error*	*t Stat*	*p-value*	*Lower 95%*	*Upper 95%*
Intercept	-0.13475	0.01066	- 12.6412	3.05E-27	-0.15578	-0.11373
X Variable 1	0.262705	0.011082	23.7059	9.75E-60	0.240851	0.284558

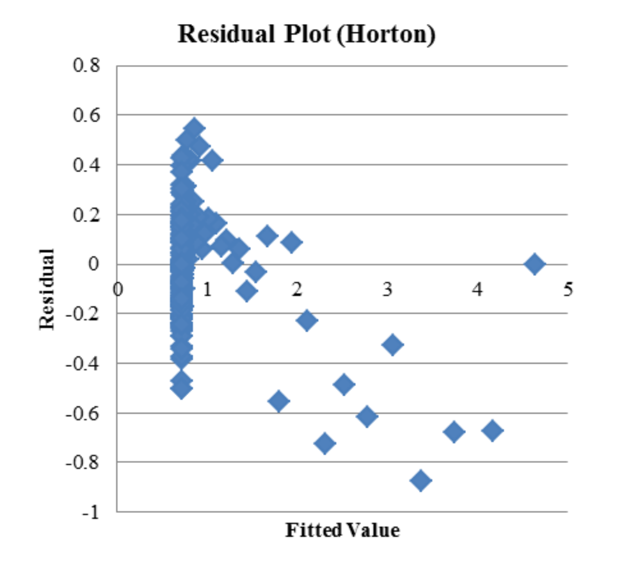


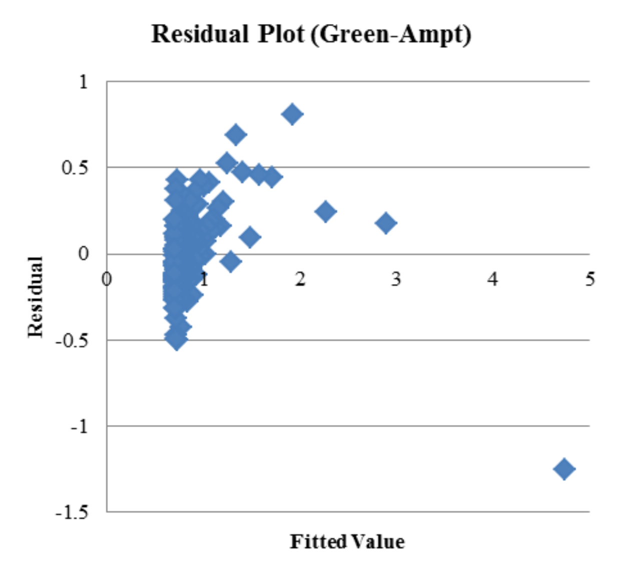
258 Main St - 07-14-2010




y = 325.15x - 2.2087
R² = 0.9493
f(in./hr)
1/F (in-1)
40
35
30
25
20
15
10
5
0
0
0.02
0.04
0.06
0.08
0.1
0.12

Residual Plots for Horton and Green-Ampt fitted values



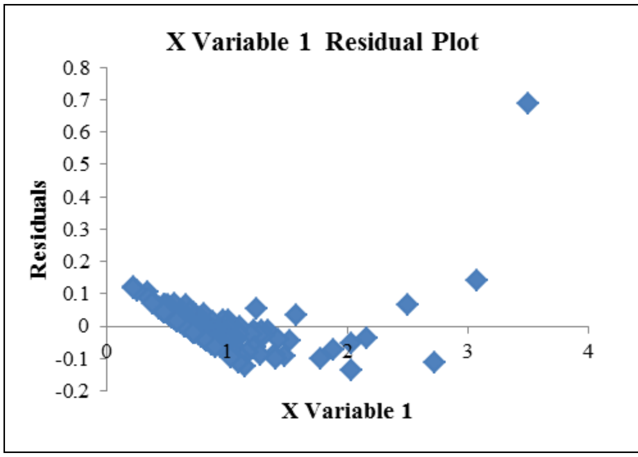


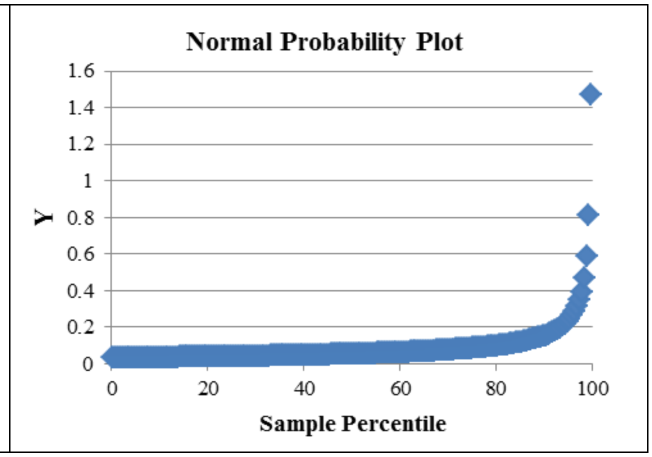
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

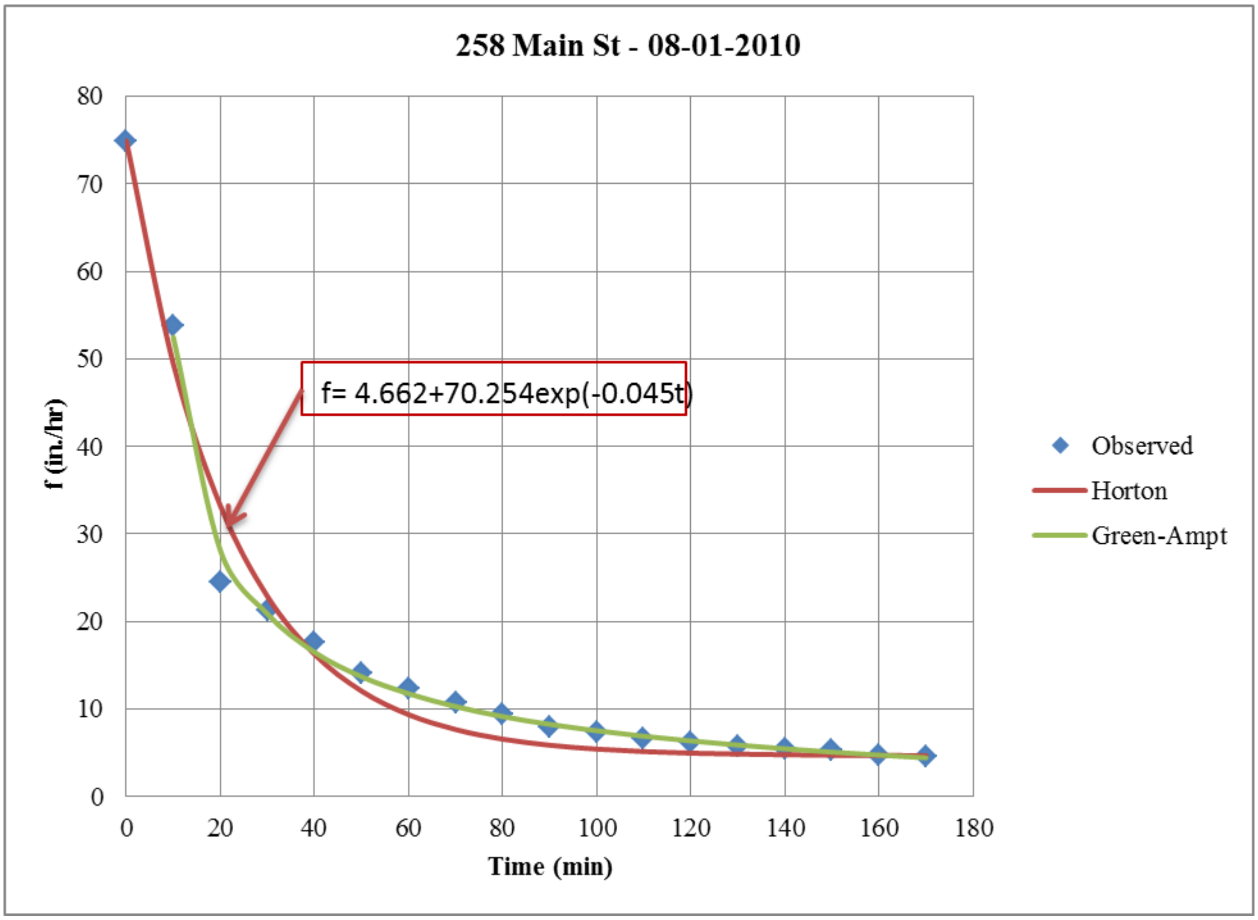
	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2.60167	2.60167	561.9697	9.75E-60
Residual	198	0.916652	0.00463		
Total	199	3.518323			

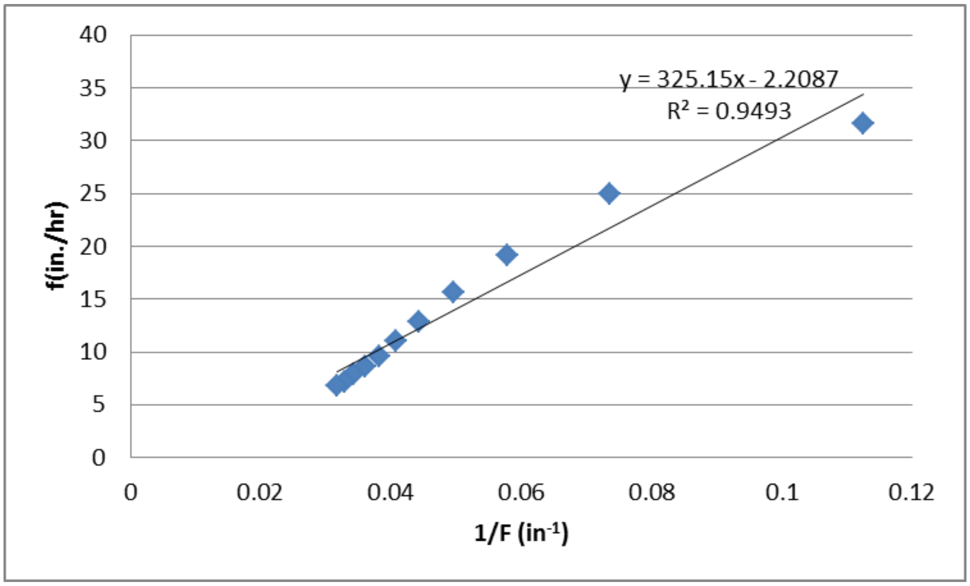
	Coefficients	*Standard Error*	*t Stat*	*p-value*	*Lower 95%*	*Upper 95%*
Intercept	-0.13475	0.01066	-12.6412	3.05E-27	-0.15578	-0.11373
X Variable 1	0.262705	0.011082	23.7059	9.75E-60	0.240851	0.284558





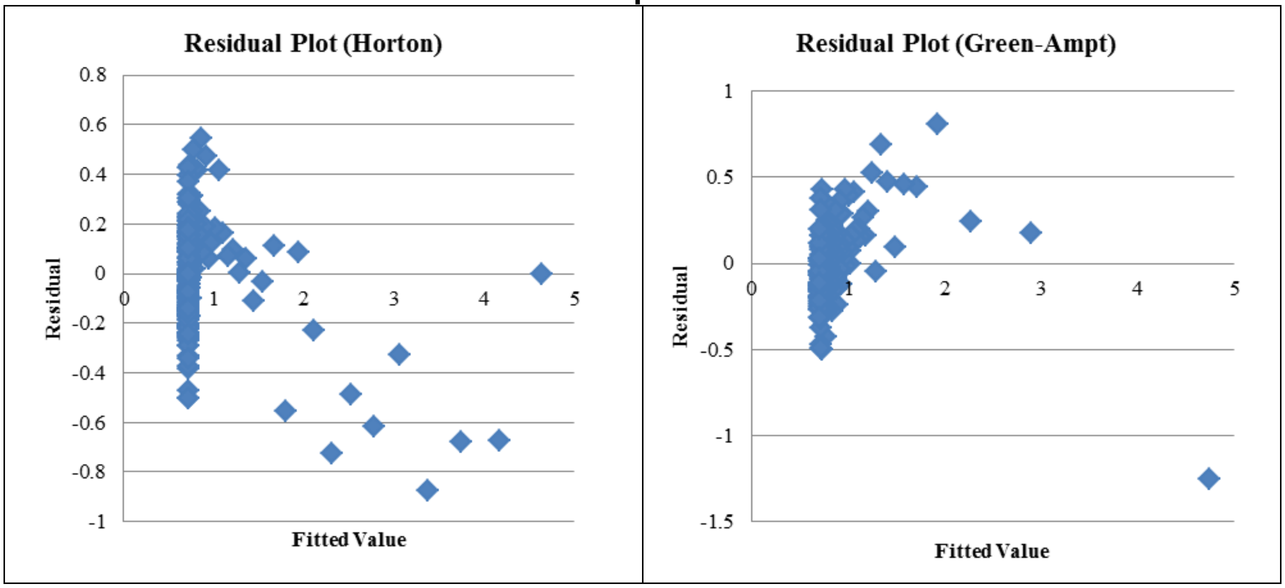
258 Main St - 08-01-2010





y = 325.15x - 2.2087
R² = 0.9493
f(in./hr)
1/F (in⁻¹)
0
5
10
15
20
25
30
35
40
0
0.02
0.04
0.06
0.08
0.1
0.12

Residual Plots for Horton and Green-Ampt fitted values

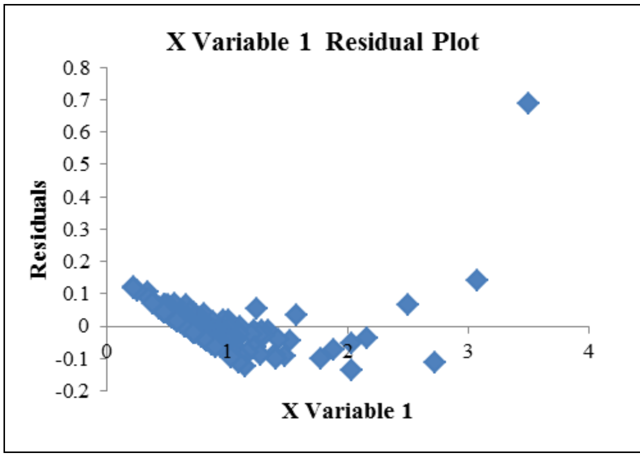


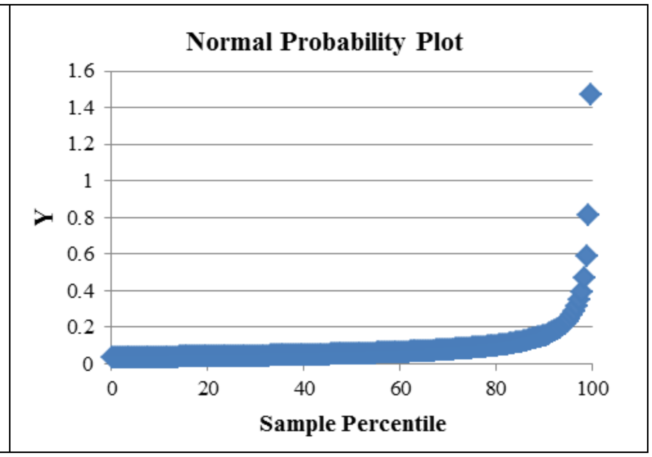
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

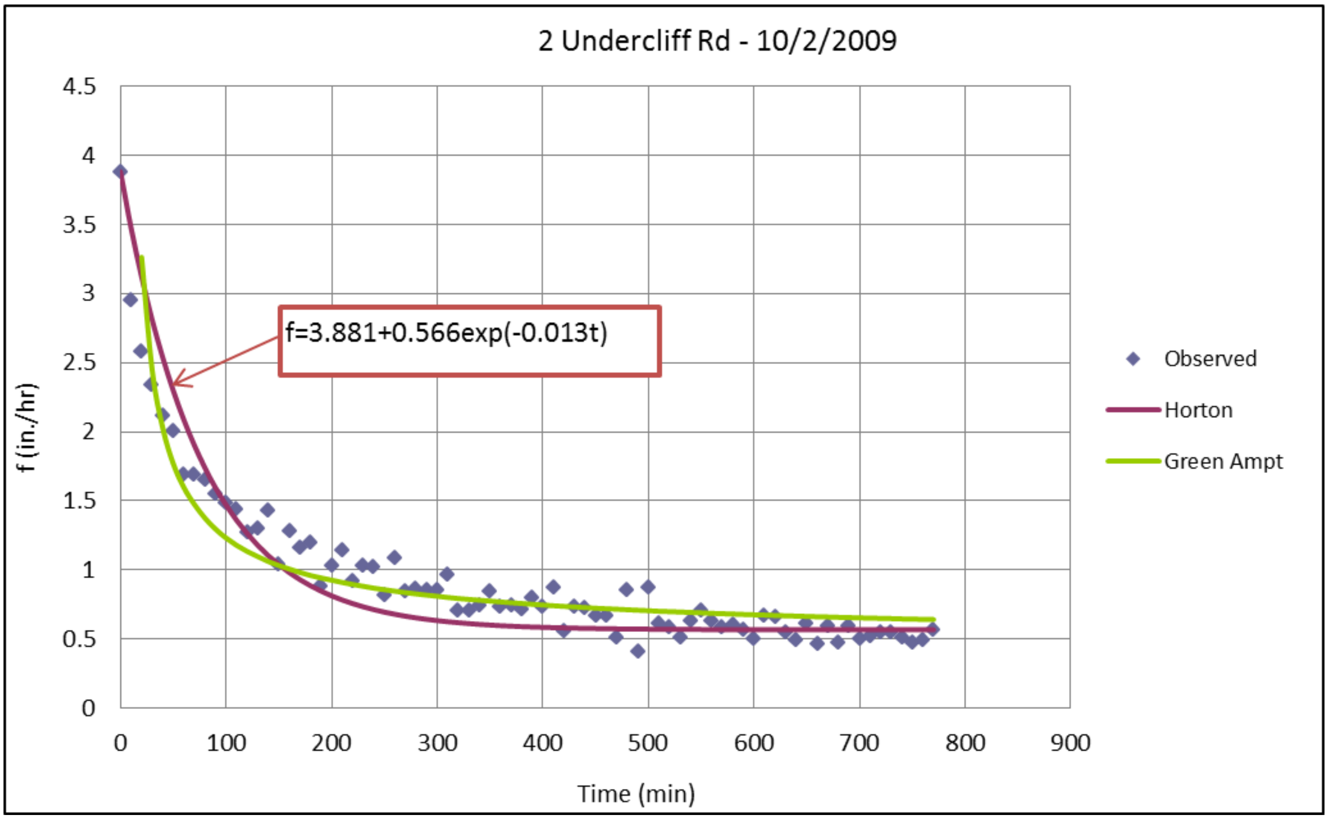
	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2.60167	2.60167	561.9697	9.75E-60
Residual	198	0.916652	0.00463		
Total	199	3.518323			

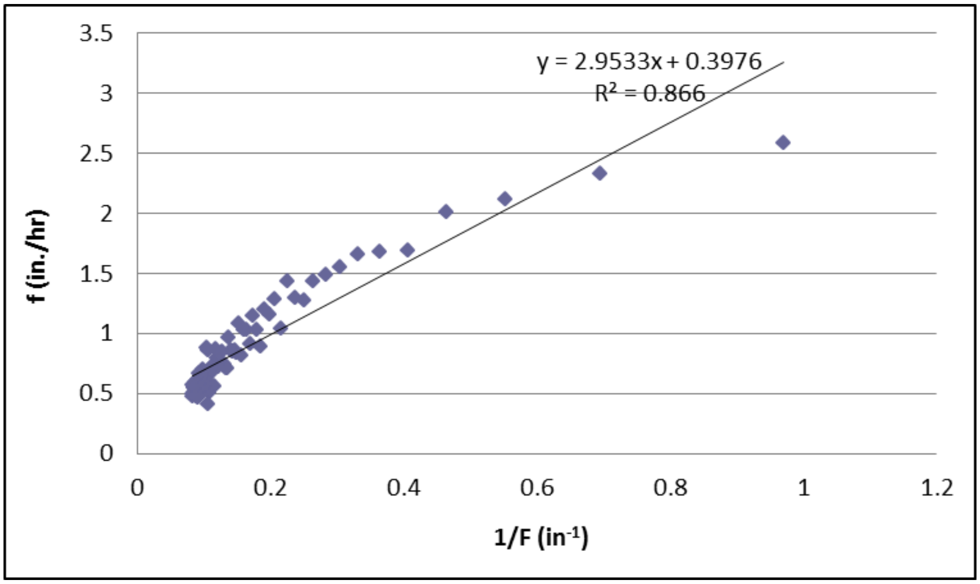
	Coefficients	*Standard Error*	*t Stat*	*p-value*	*Lower 95%*	*Upper 95%*
Intercept	-0.13475	0.01066	- 12.6412	3.05E-27	-0.15578	-0.11373
X Variable 1	0.262705	0.011082	23.7059	9.75E-60	0.240851	0.284558



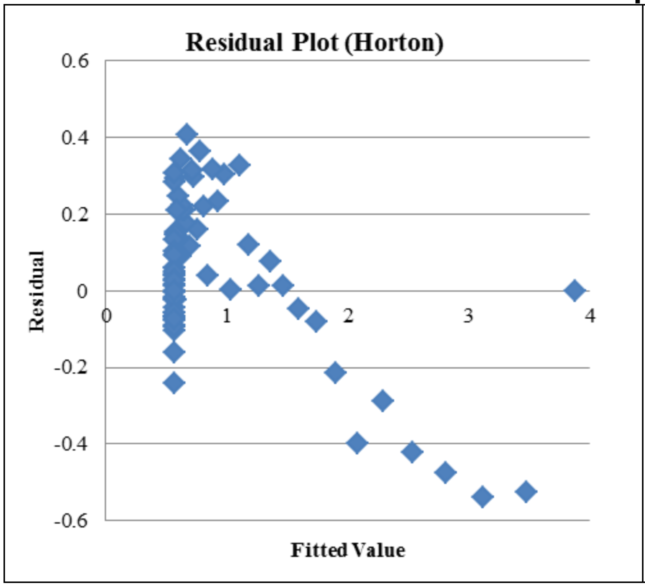


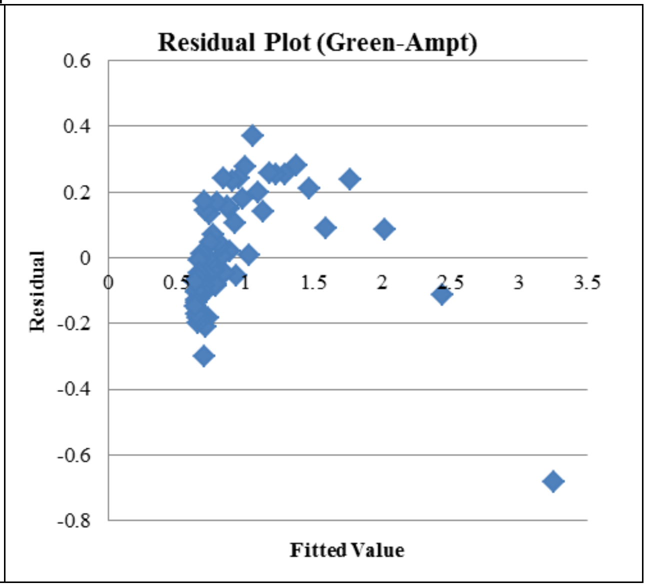
2 Undercliff Rd 10-2-2009




y = 2.9533x + 0.3976
R² = 0.866
f (in./hr)
1/F (in-1)
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
1.2

Residual Plots for Horton and Green-Ampt fitted values



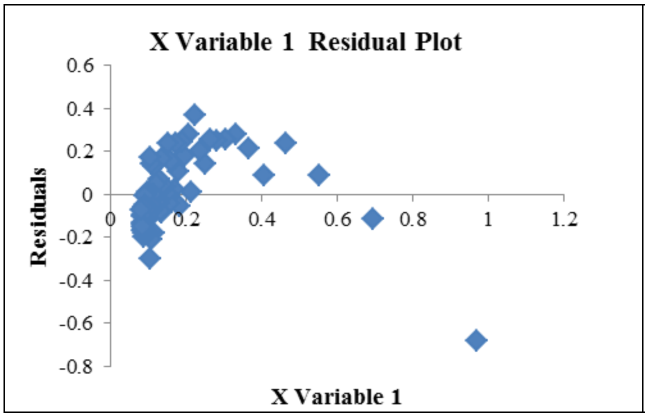


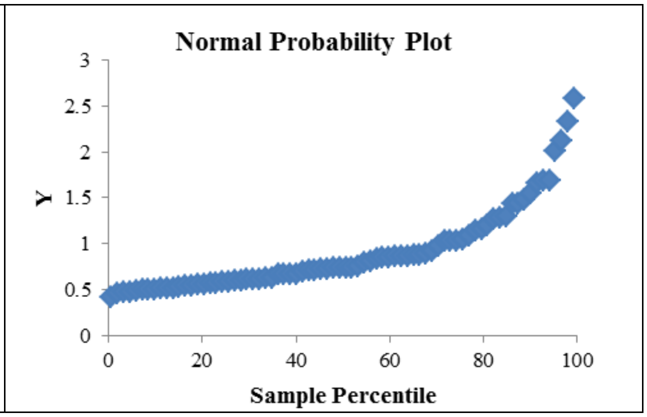
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

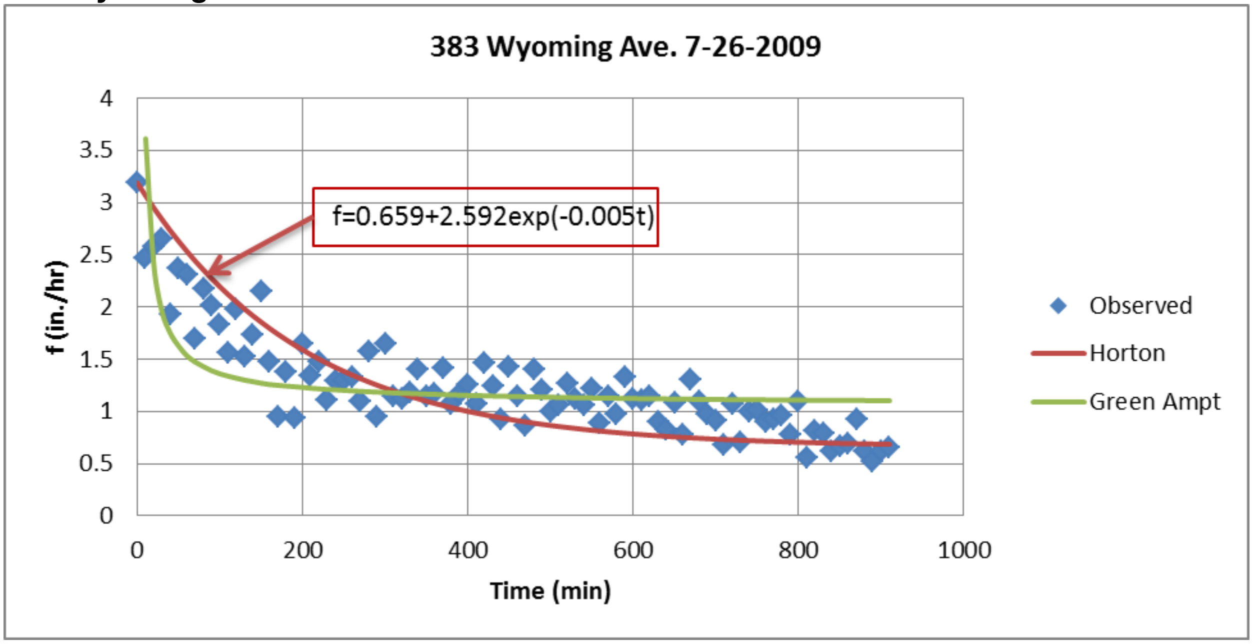
	df	SS	MS	F	Significance F
Regression	1	13.53029	13.53029	478.1812	5.02E-34
Residual	74	2.093854	0.028295		
Total	75	15.62415			

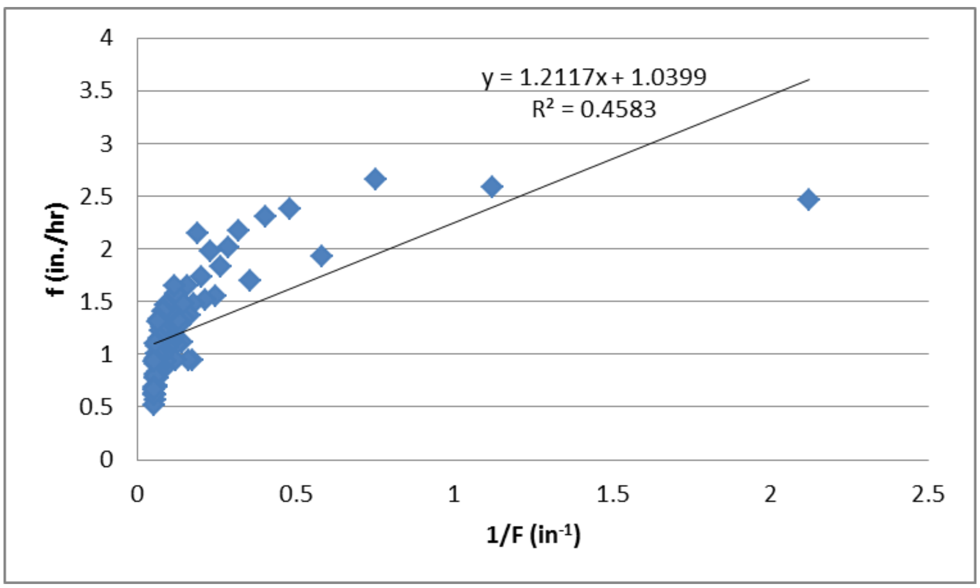
	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	0.397557	0.029966	13.2671	3E-21	0.337849	0.457265
X Variable 1	2.953296	0.135055	21.86735	5.02E-34	2.684193	3.222399



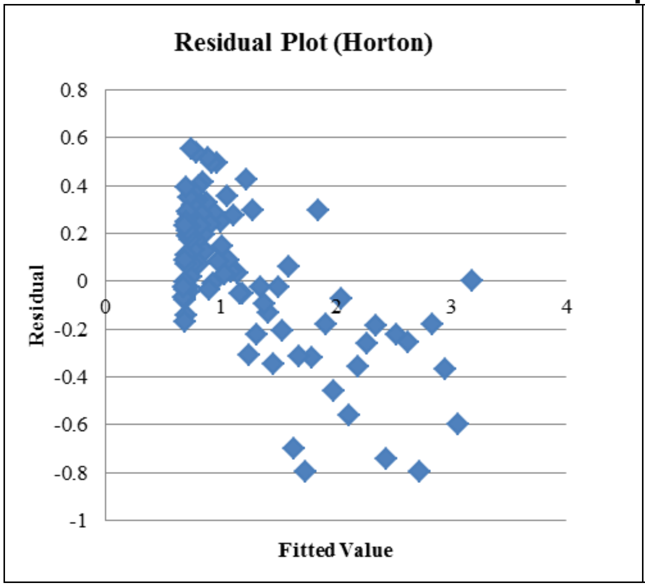


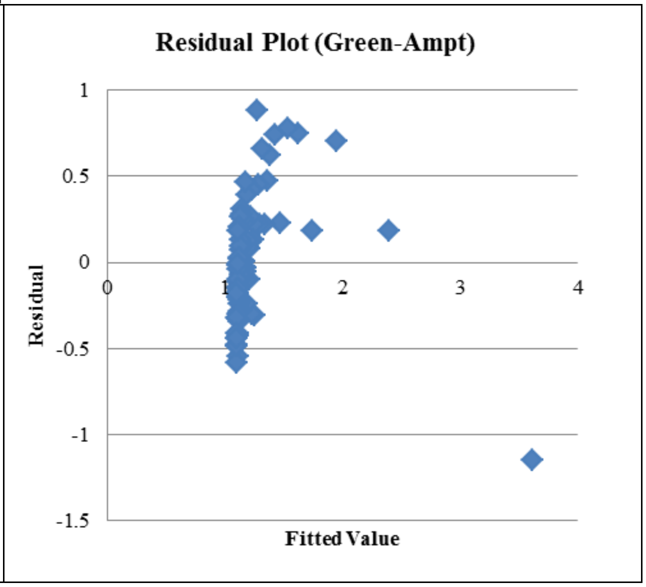
383 Wyoming Ave. 7-26-2009





Residual Plots for Horton and Green-Ampt fitted values



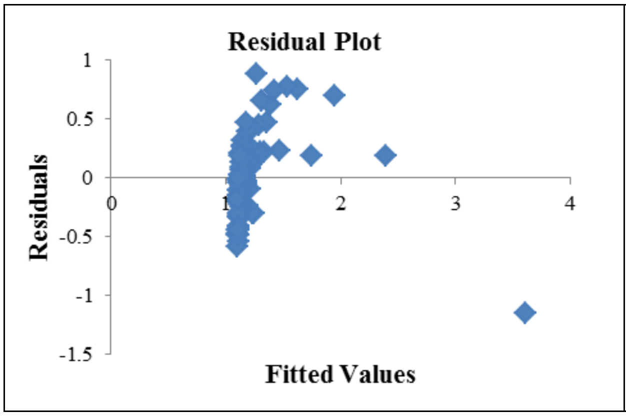


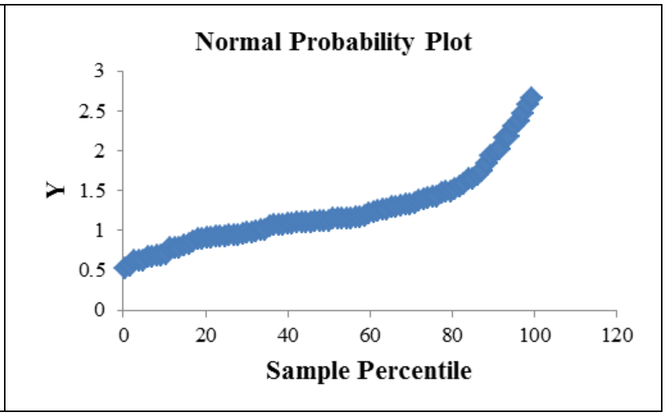
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

	df	SS	MS	F	Significance F
Regression	1	8.909624	8.909624	75.30019	1.75E-13
Residual	89	10.5306	0.118321		
Total	90	19.44023			

	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.039948	0.042319	24.57386	1.97E-41	0.95586	1.124035
X Variable 1	1.211721	0.139638	8.677568	1.75E-13	0.934263	1.48918





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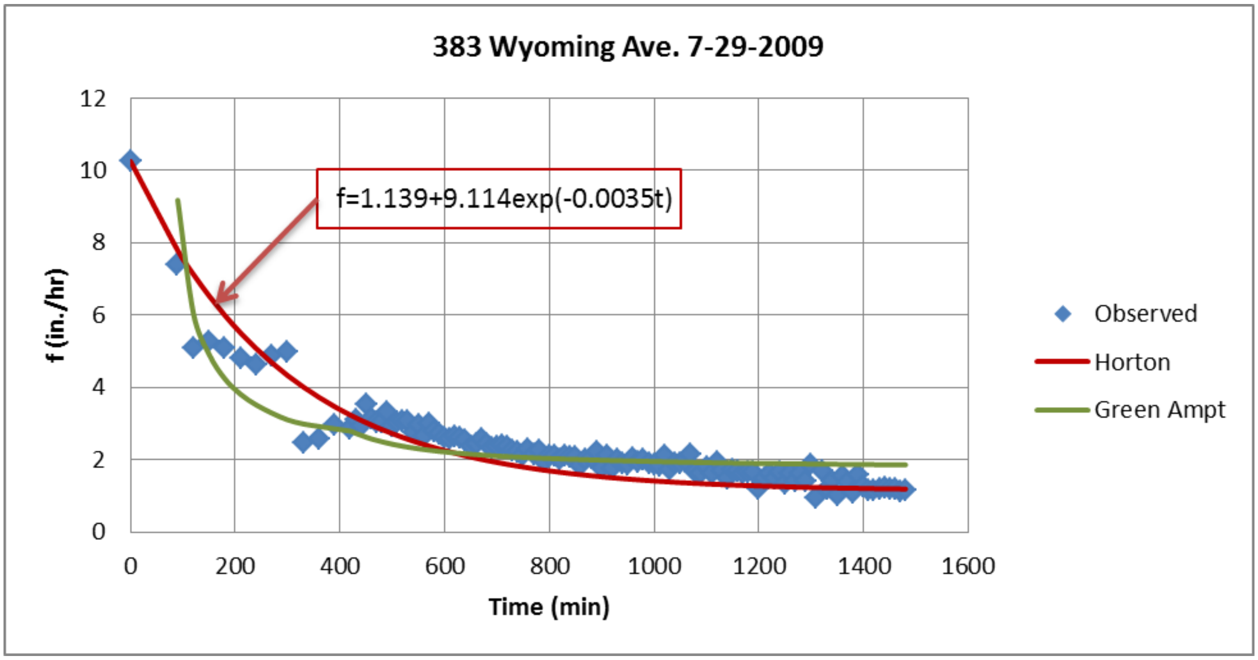
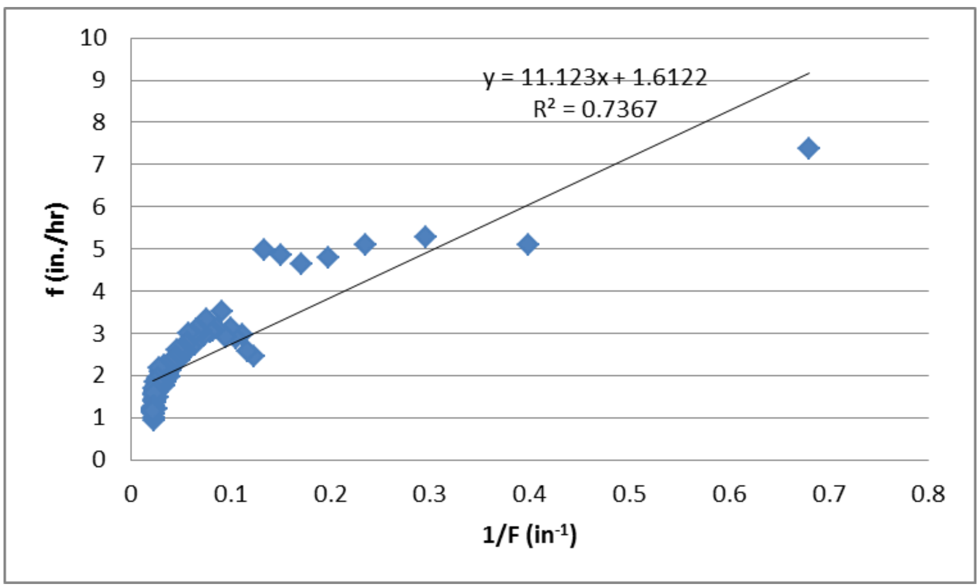
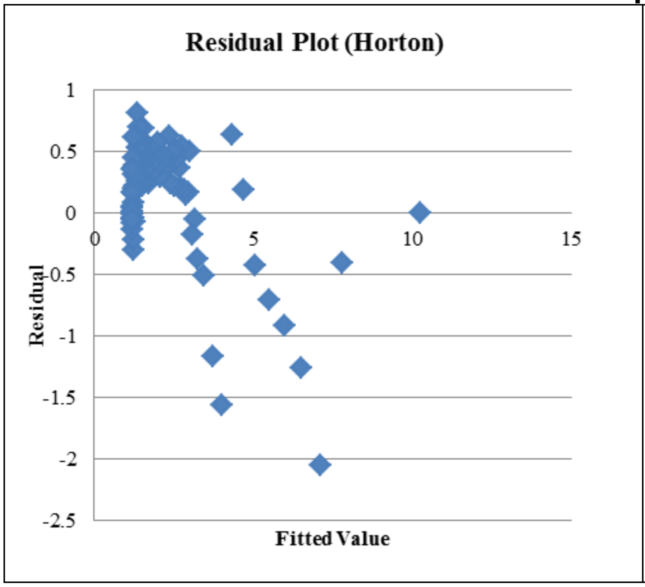
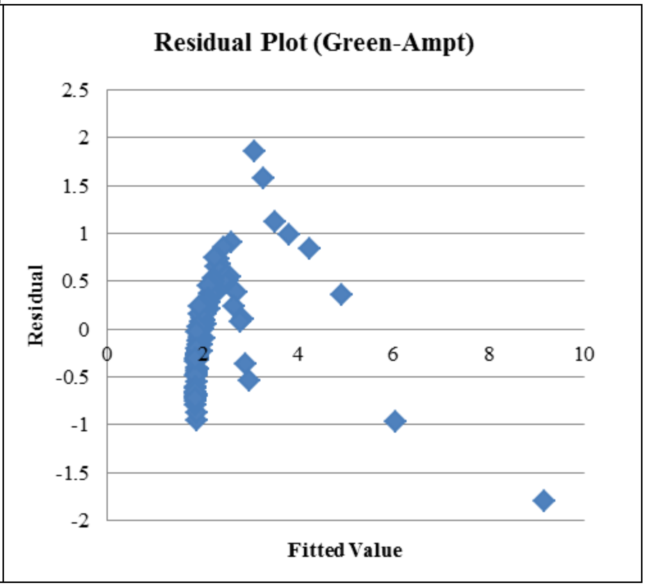




Residual Plots for Horton and Green-Ampt fitted values



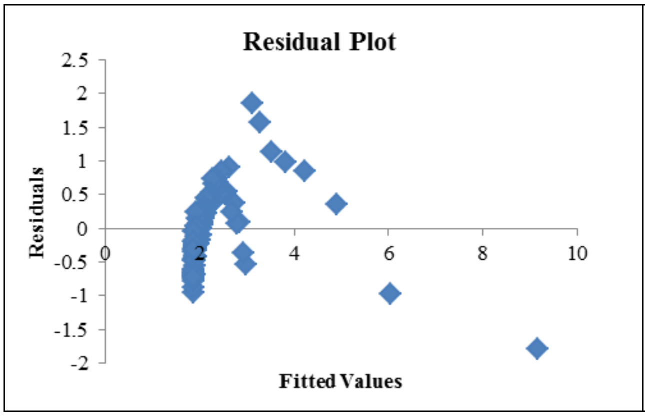


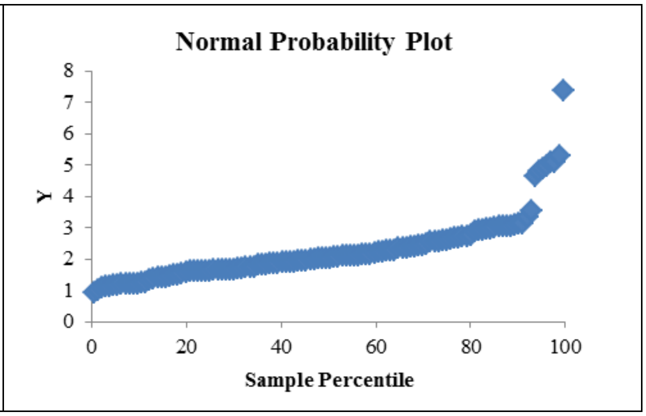
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

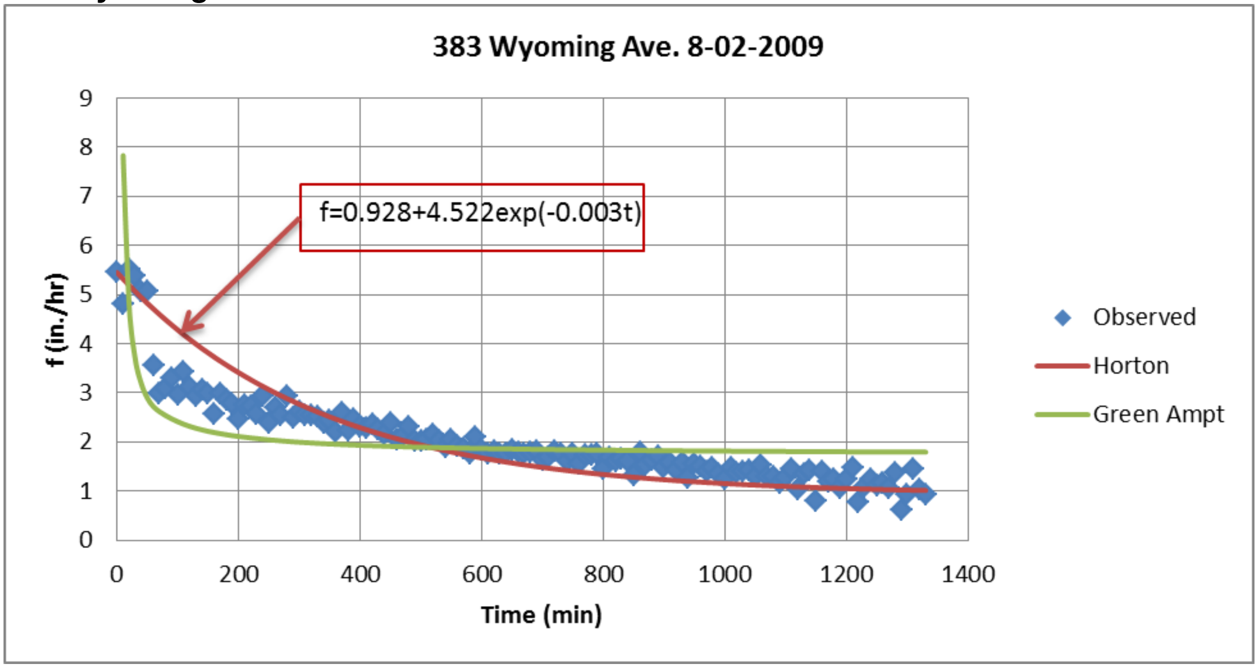
	df	SS	MS	F	Significance F
Regression	1	89.86926	89.86926	324.6359	2.07E-35
Residual	116	32.11239	0.276831		
Total	117	121.9817			

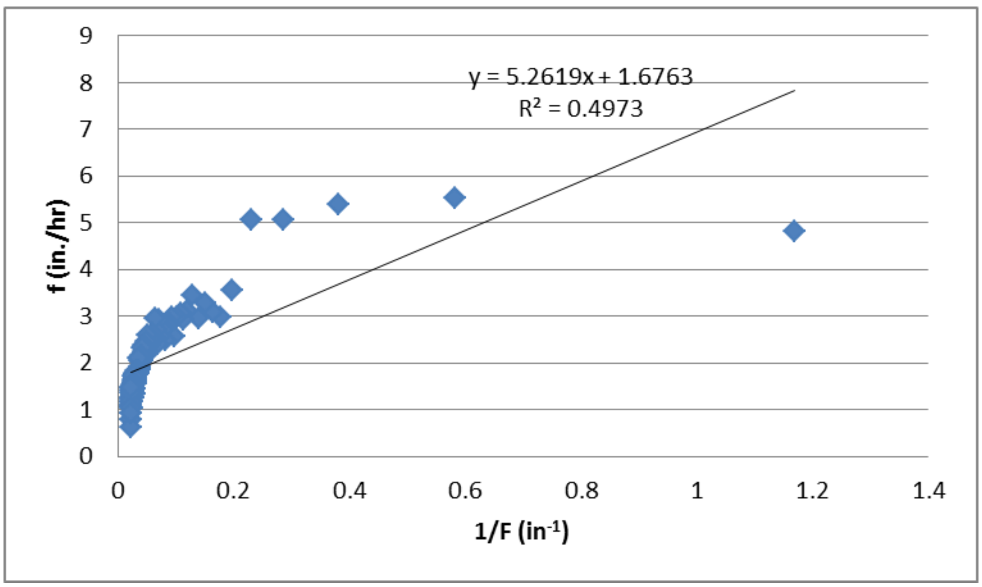
	Coefficients	Standard Error	t Stat	p-value	Lower 95%
Intercept	1.612247	0.060141	26.80775	1.56E-51	1.49313
X Variable 1	11.12316	0.617348	18.01765	2.07E-35	9.900421



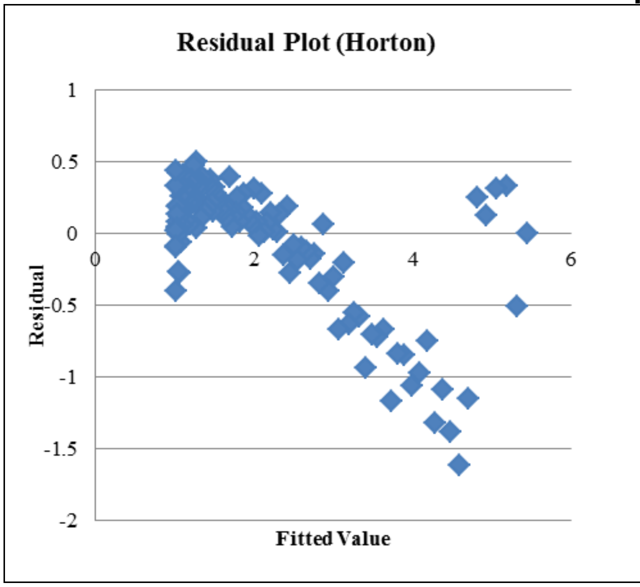


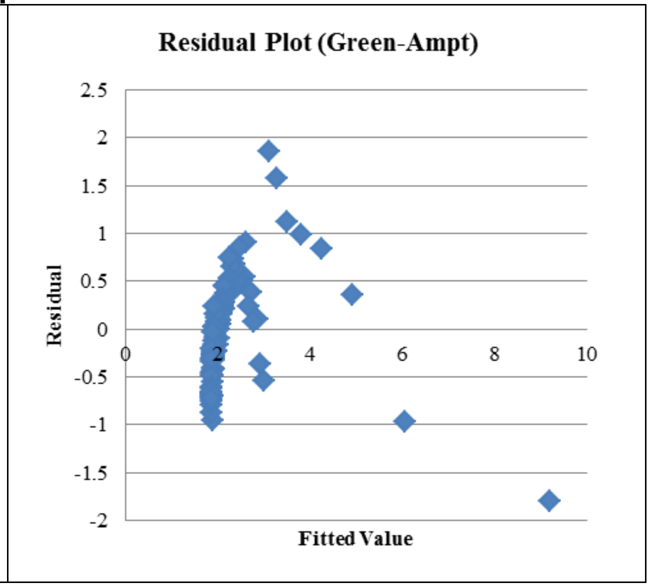
383 Wyoming Ave. 8-02-2009





Residual Plots for Horton and Green-Ampt fitted values



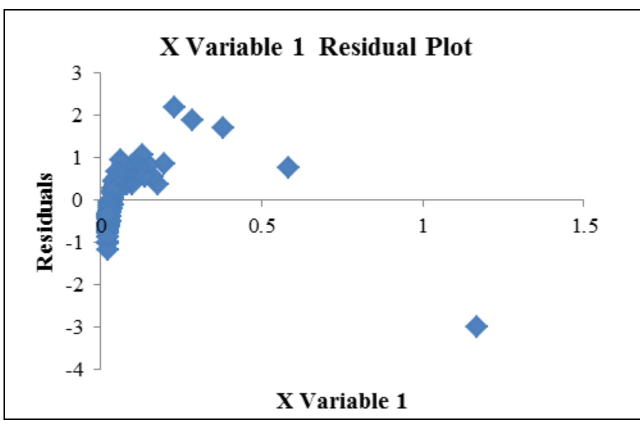


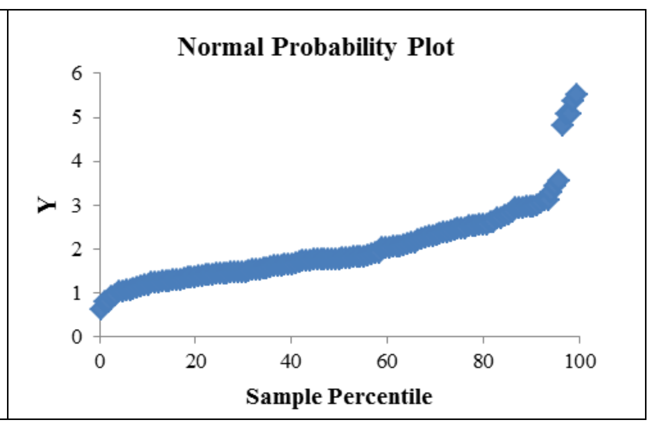
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

	df	SS	MS	F	Significance F
Regression	1	51.25879	51.25879	129.5995	2.67E-21
Residual	131	51.81269	0.395517		
Total	132	103.0715			

	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.676322	0.062127	26.98234	2.4E-55	1.553421	1.799223
X Variable 1	5.26193	0.462214	11.38418	2.67E-21	4.34756	6.1763





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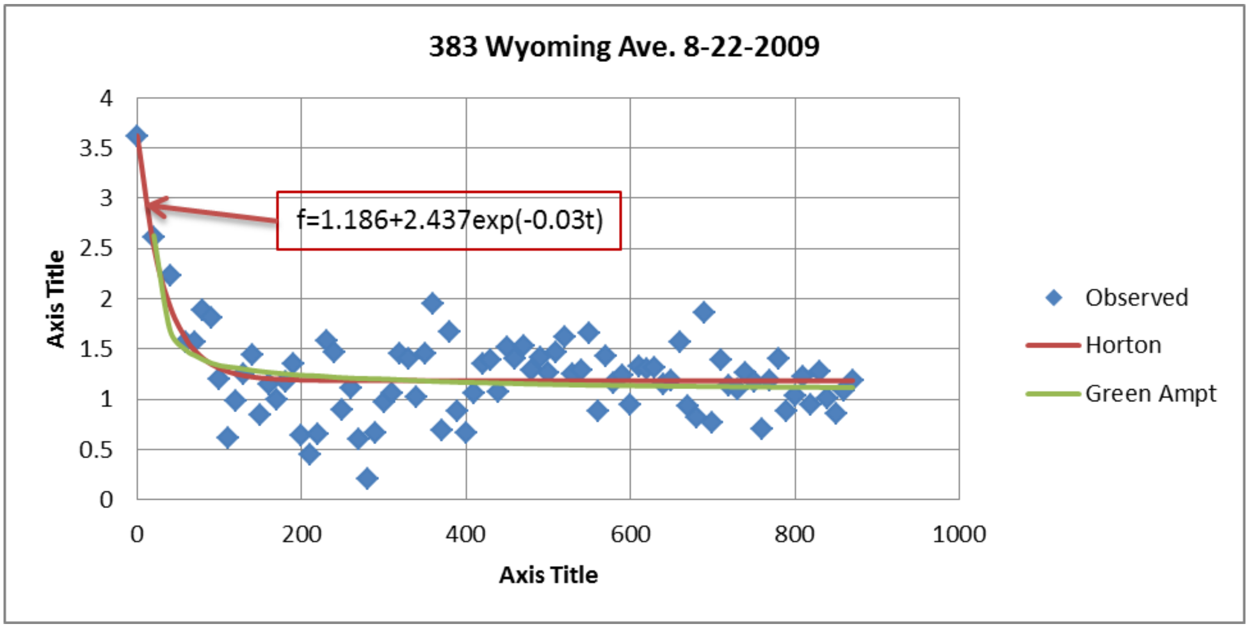
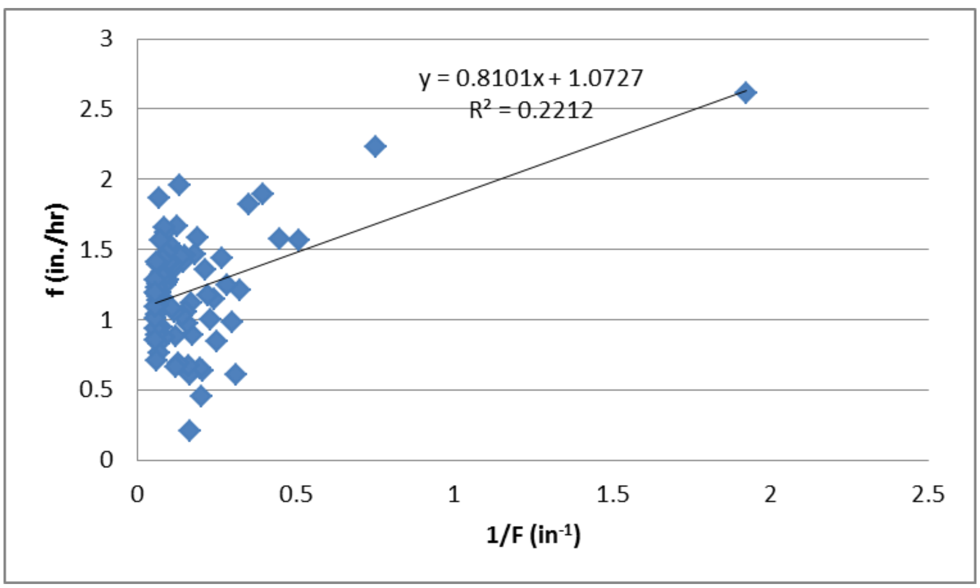
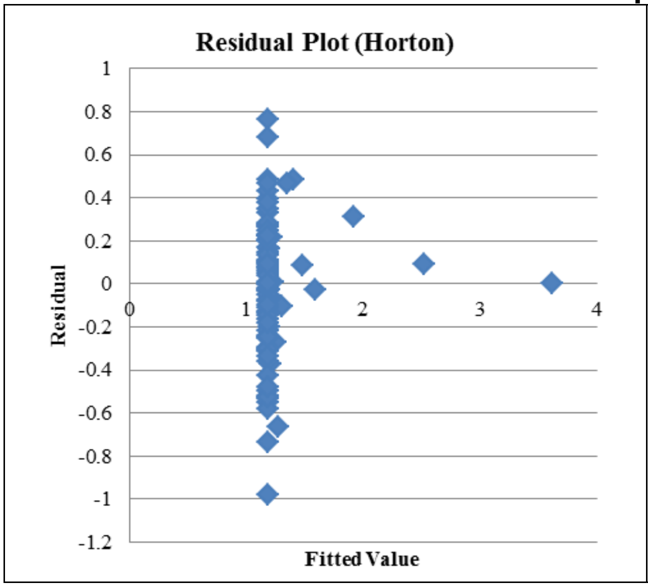
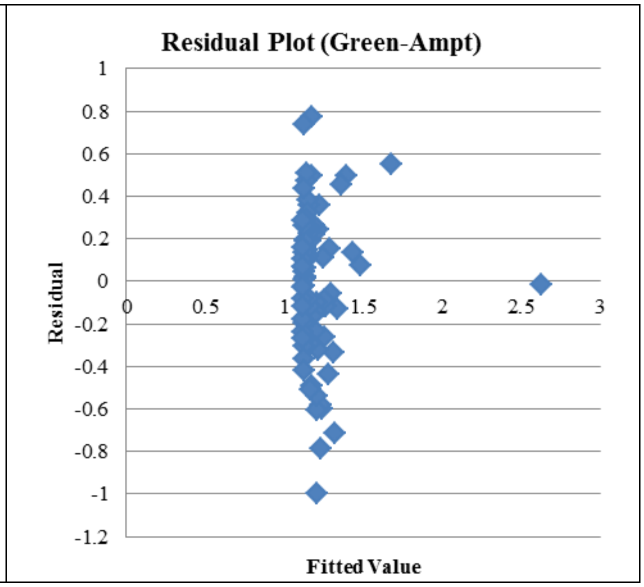




Residual Plots for Horton and Green-Ampt fitted values



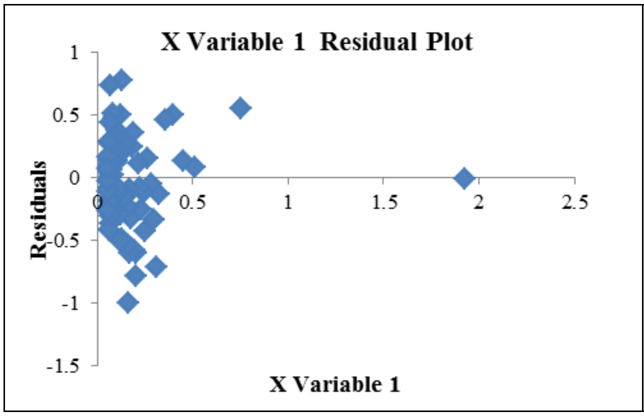


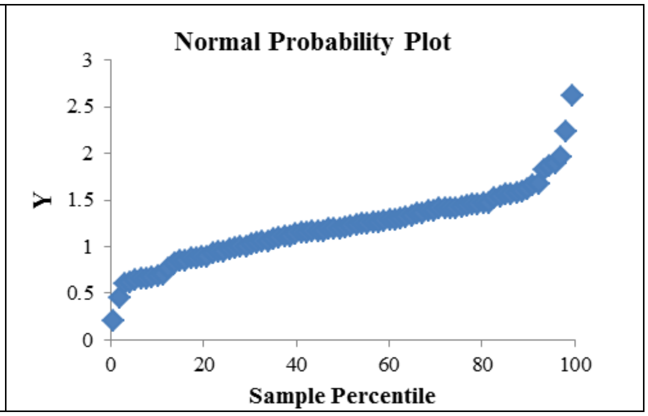
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

	df	SS	MS	F	Significance F
Regression	1	2.771267	2.771267	23.2845	6.37E-06
Residual	82	9.759448	0.119018		
Total	83	12.53072			

	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.072661	0.046968	22.83825	2.7E-37	0.979227	1.166095
X Variable 1	0.810101	0.167883	4.825402	6.37E-06	0.476129	1.144073





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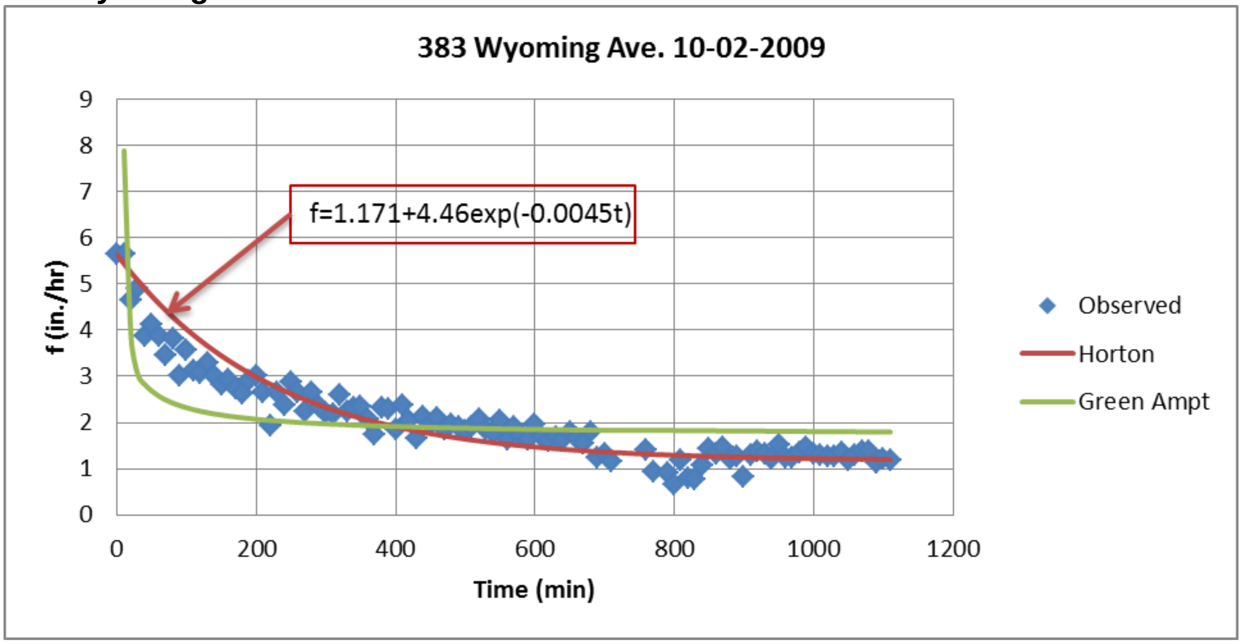
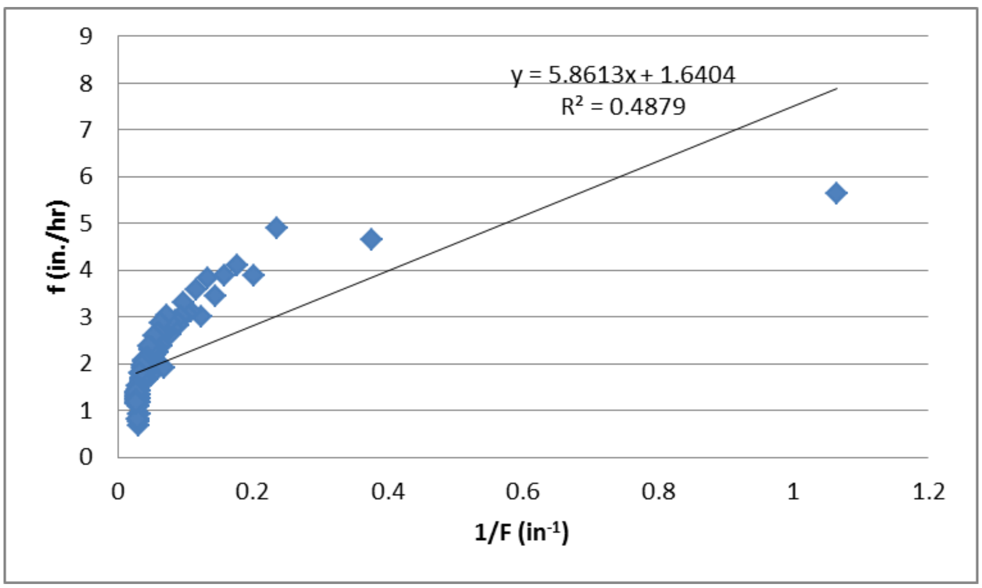
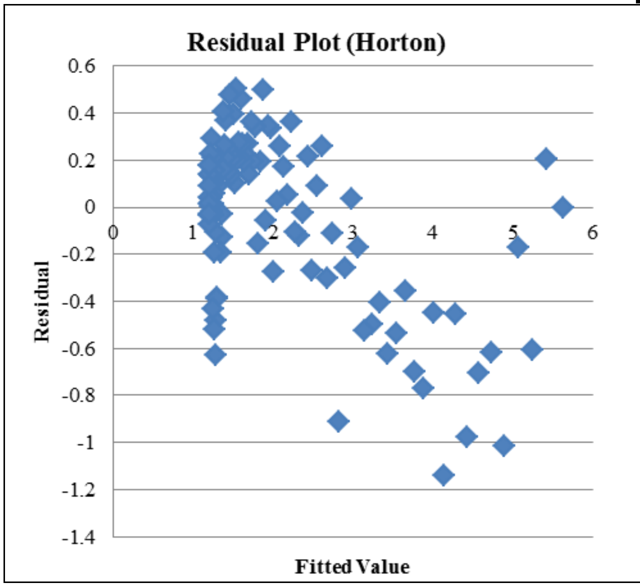
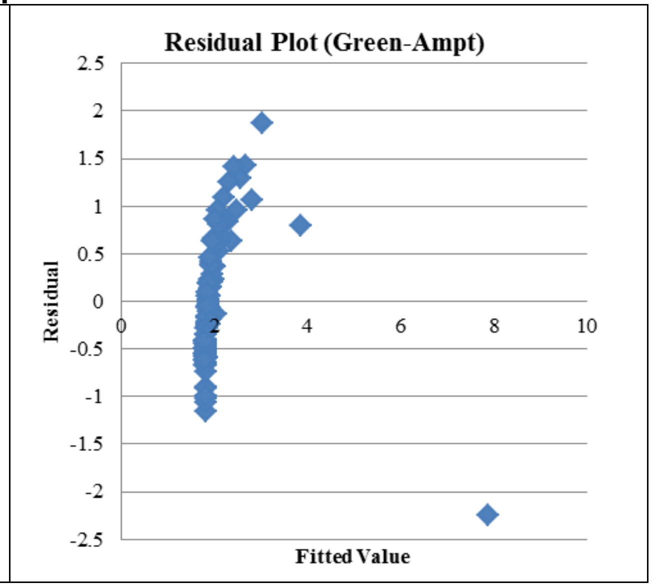




Residual Plots for Horton and Green-Ampt fitted values



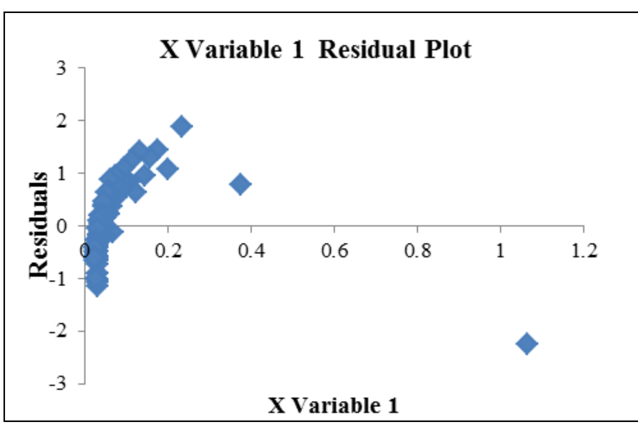


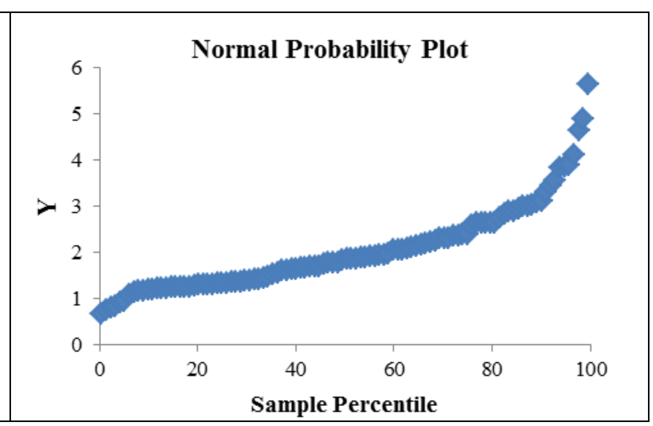
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

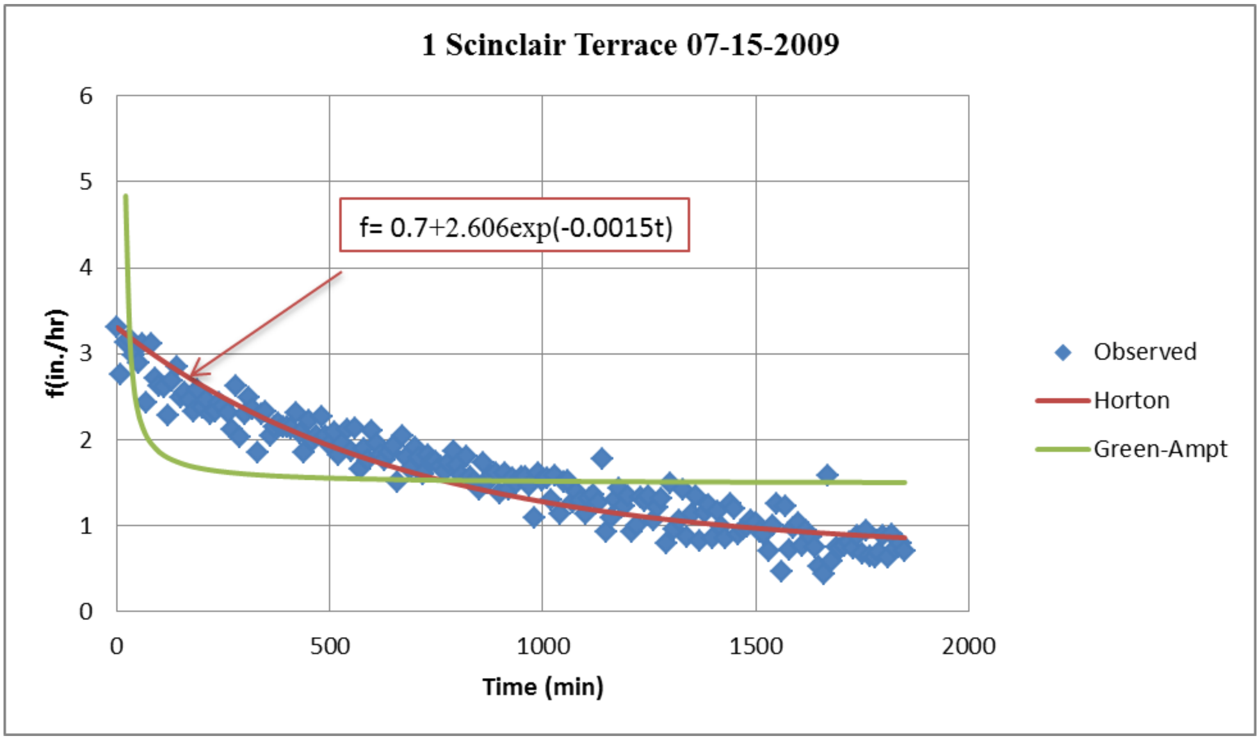
	df	SS	MS	F	Significance F
Regression	1	43.29224	43.29224	99.08553	8.52E-17
Residual	104	45.43946	0.436918		
Total	105	88.7317			

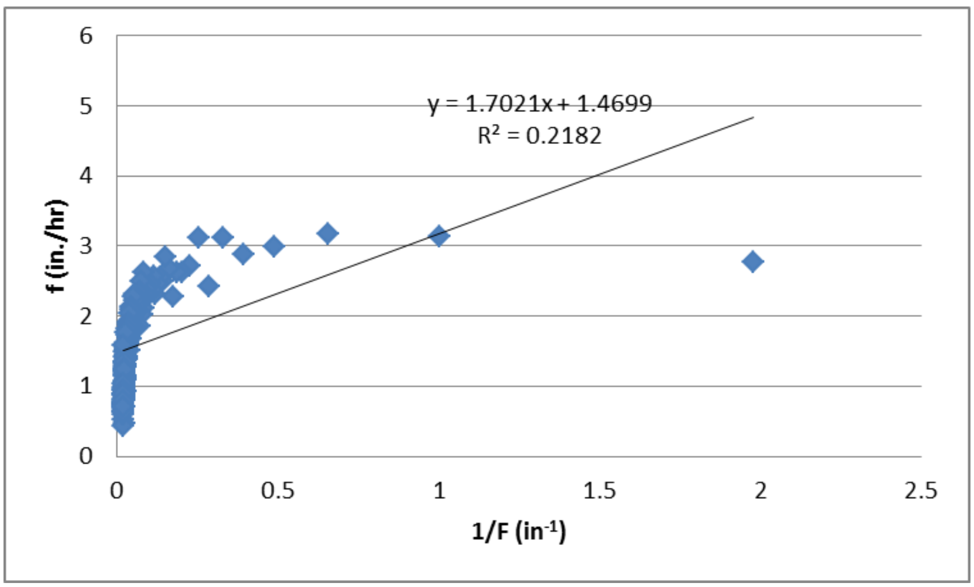
	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.640422	0.074959	21.88437	1.02E-40	1.491776	1.789067
X Variable 1	5.861307	0.588829	9.954172	8.52E-17	4.693636	7.028977



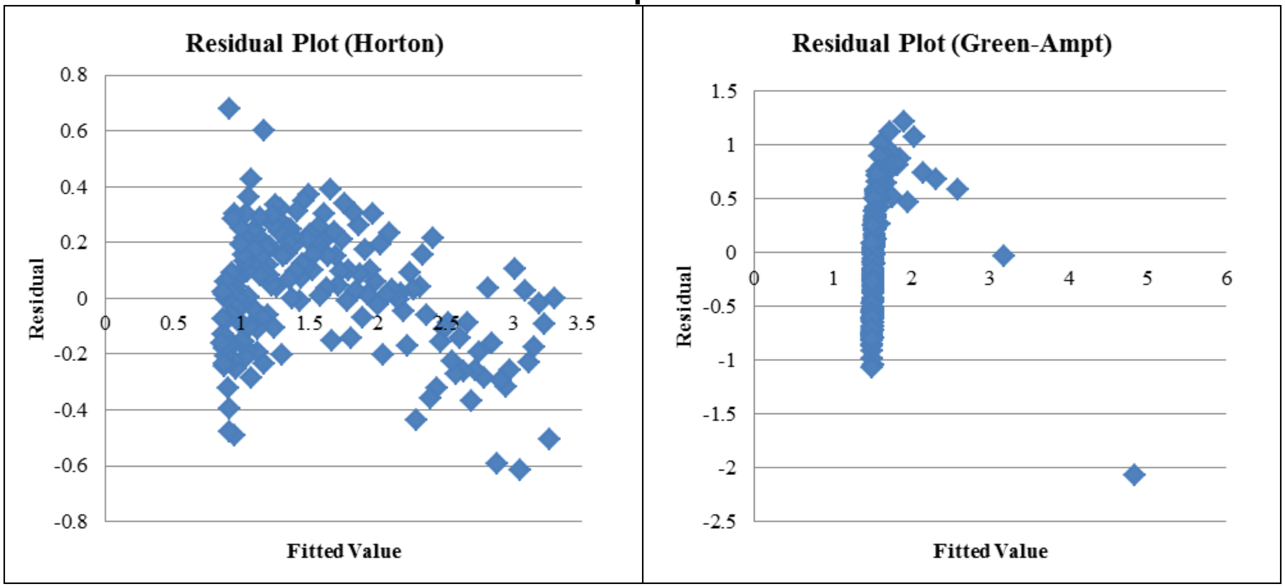


1 Sinclair Terrace 07-15-2009





Residual Plots for Horton and Green-Ampt fitted values

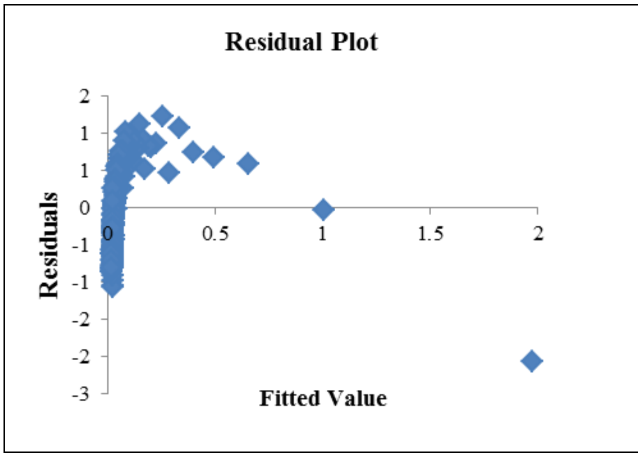


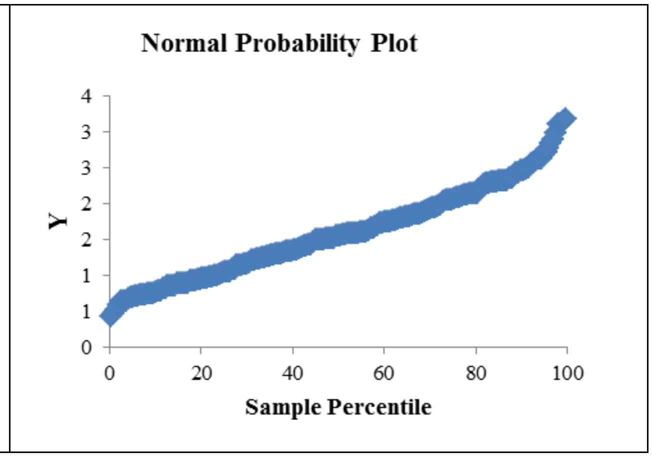
Regression Analysis for f vs. 1/F (Green-Ampt)

ANOVA

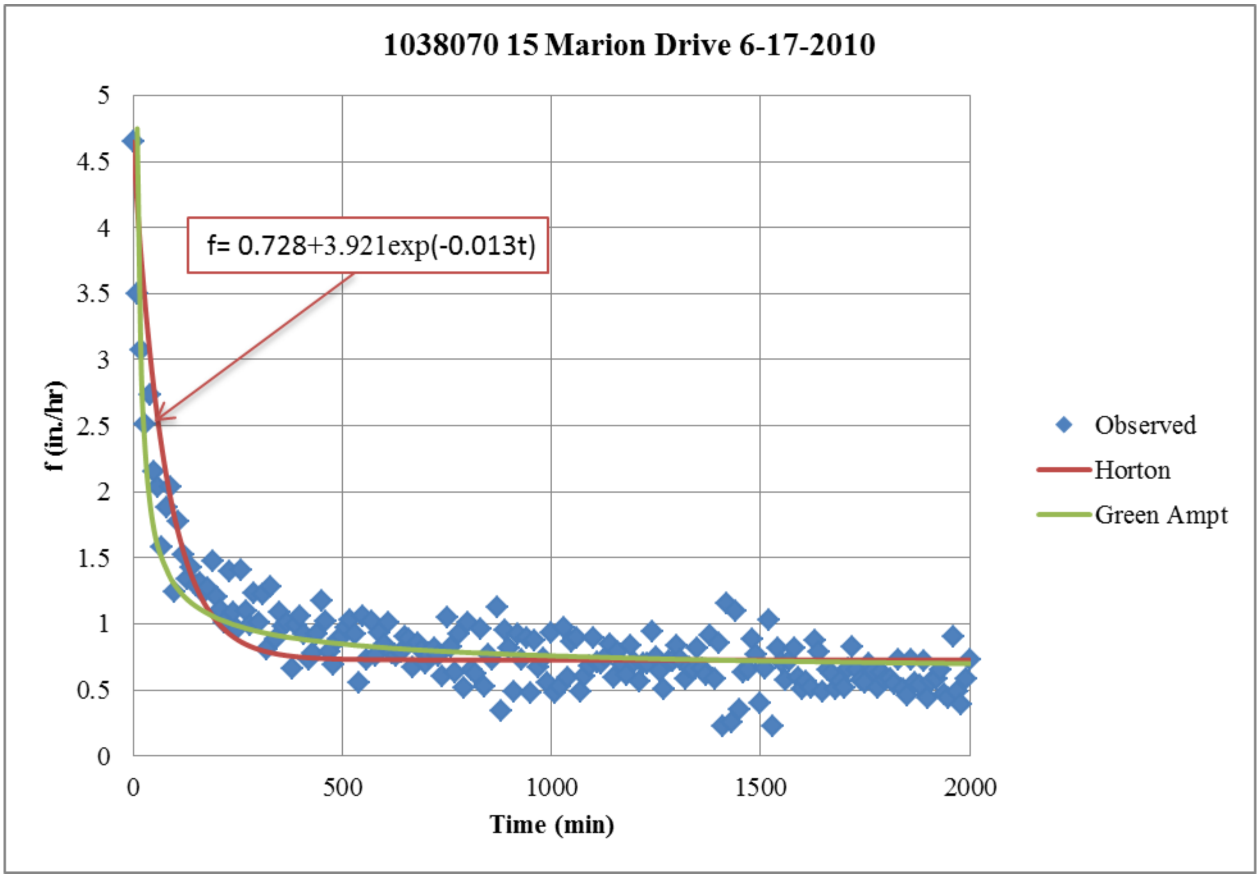
	df	SS	MS	F	Significance F
Regression	1	16.316	16.316	51.085	2.04E-11
Residual	183	58.449	0.319		
Total	184	74.765			

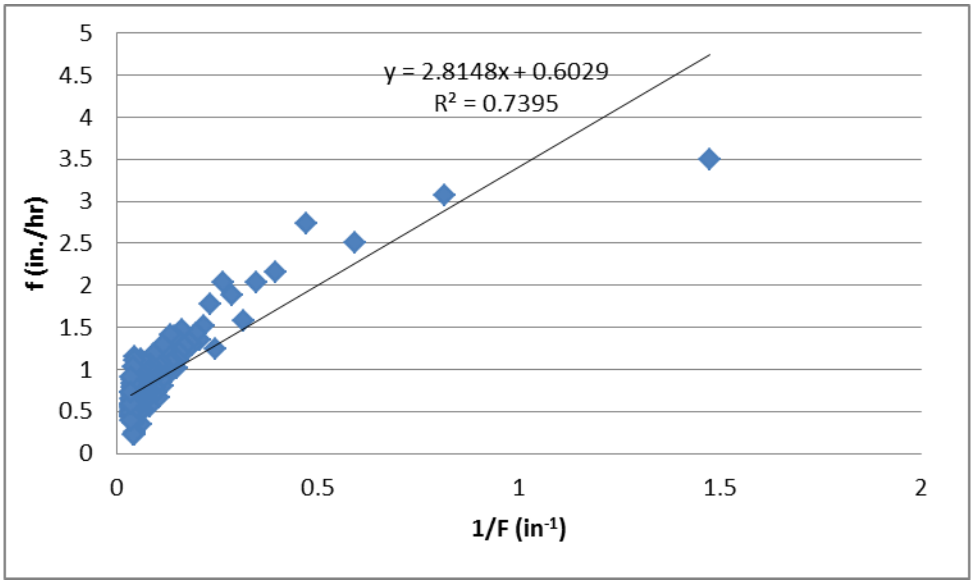
	Coefficients	Standard Error	t Stat	p-value	Lower 95%	Upper 95%
Intercept	1.470	0.045	32.746	1.91E-78	1.381	1.558
X Variable 1	1.702	0.238	7.147	2.04E-11	1.232	2.172



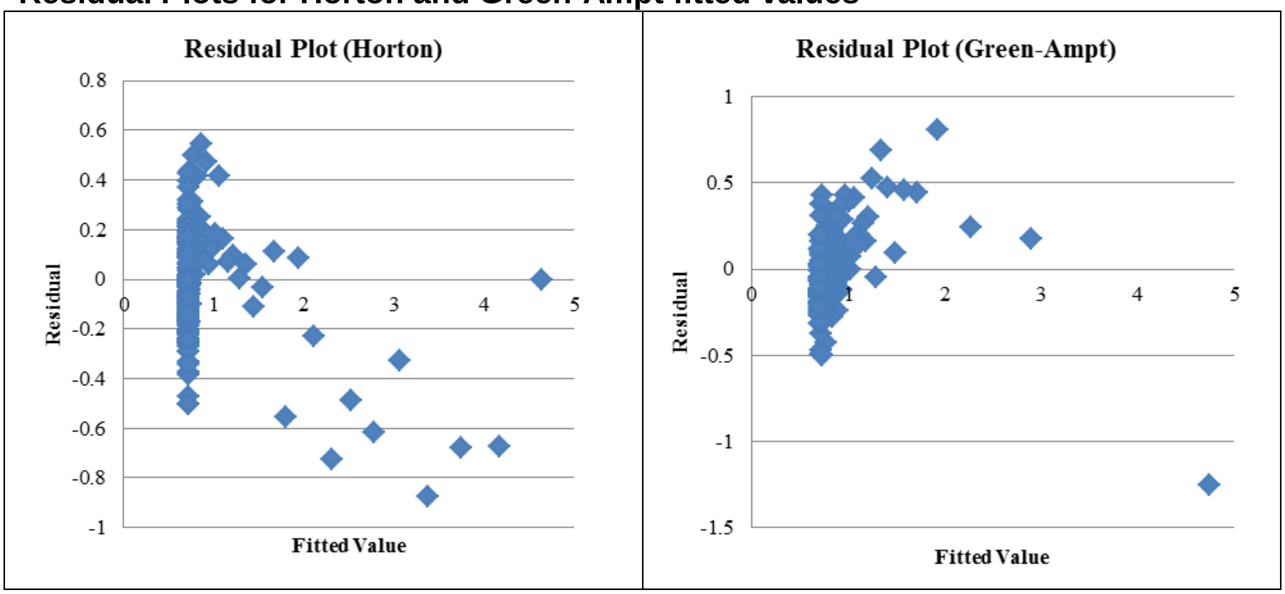


15 Marion Drive 6-17-2010





Residual Plots for Horton and Green-Ampt fitted values



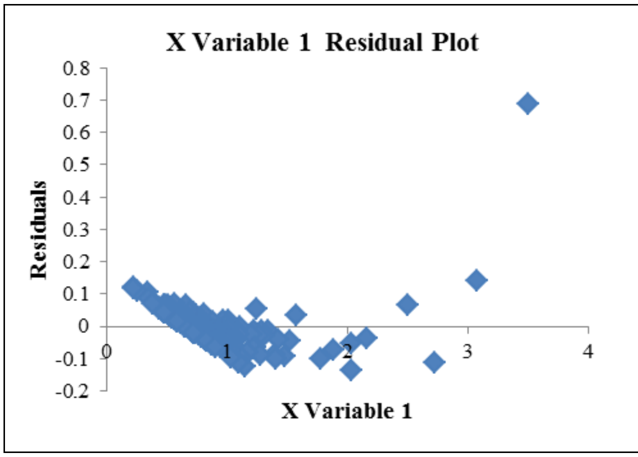
Regression Analysis for f vs. 1/F (Green-Ampt)

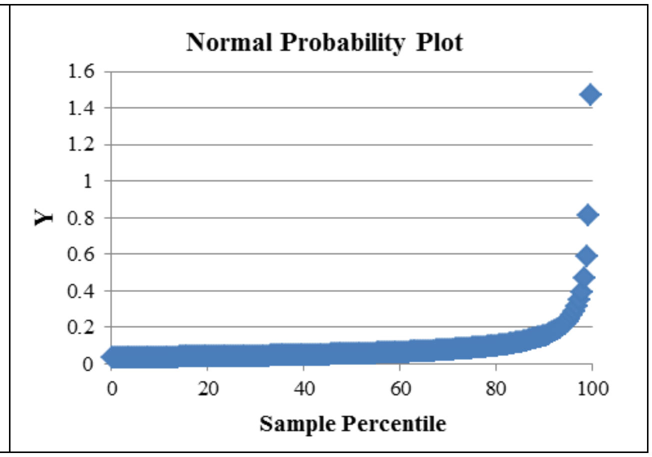
ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	2.60167	2.60167	561.9697	9.75E-60
Residual	198	0.916652	0.00463		
Total	199	3.518323			

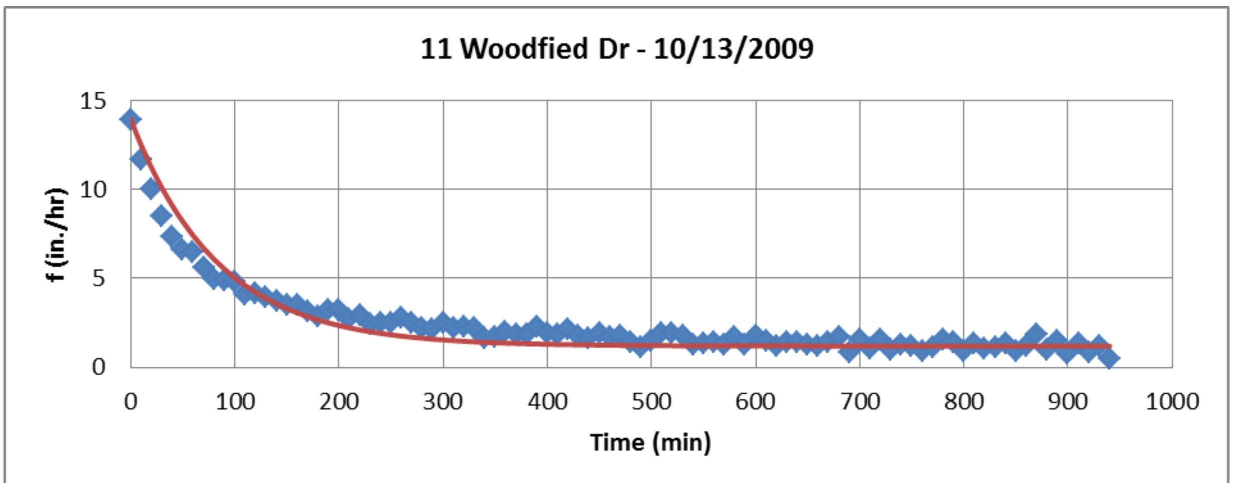
	Coefficients	*Standard Error*	*t Stat*	*p-value*	*Lower 95%*	*Upper 95%*

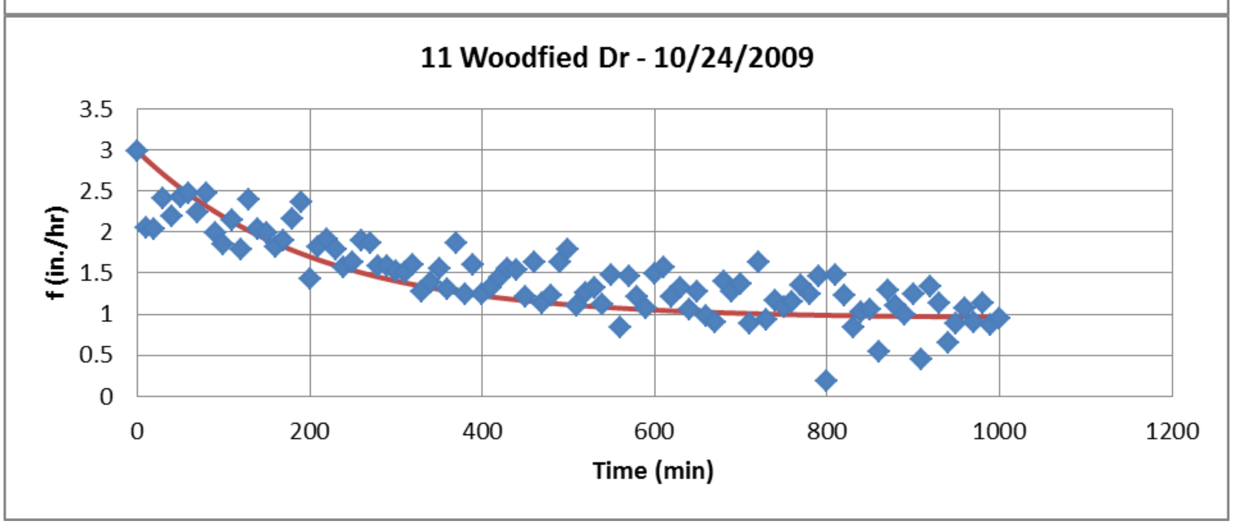
Intercept	-0.13475	0.01066	-12.6412	3.05E-27	-0.15578	-0.11373
X Variable 1	0.262705	0.011082	23.7059	9.75E-60	0.240851	0.284558

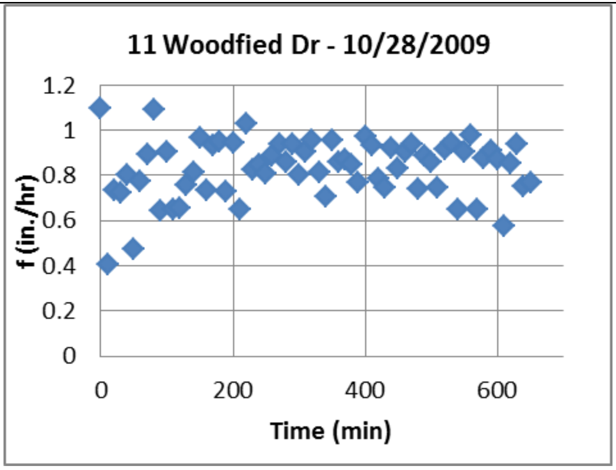


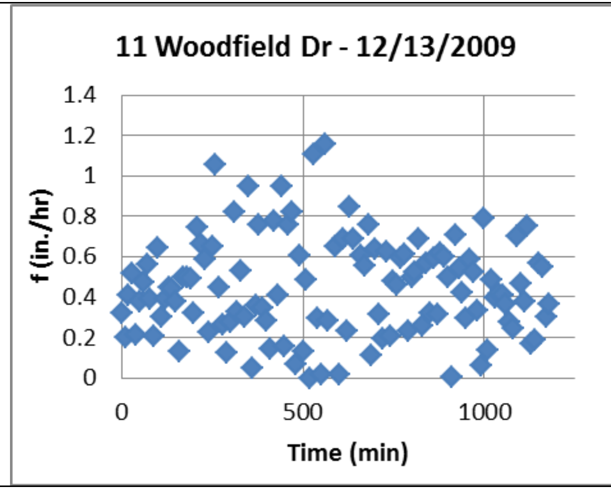


11 Woodfield Drive

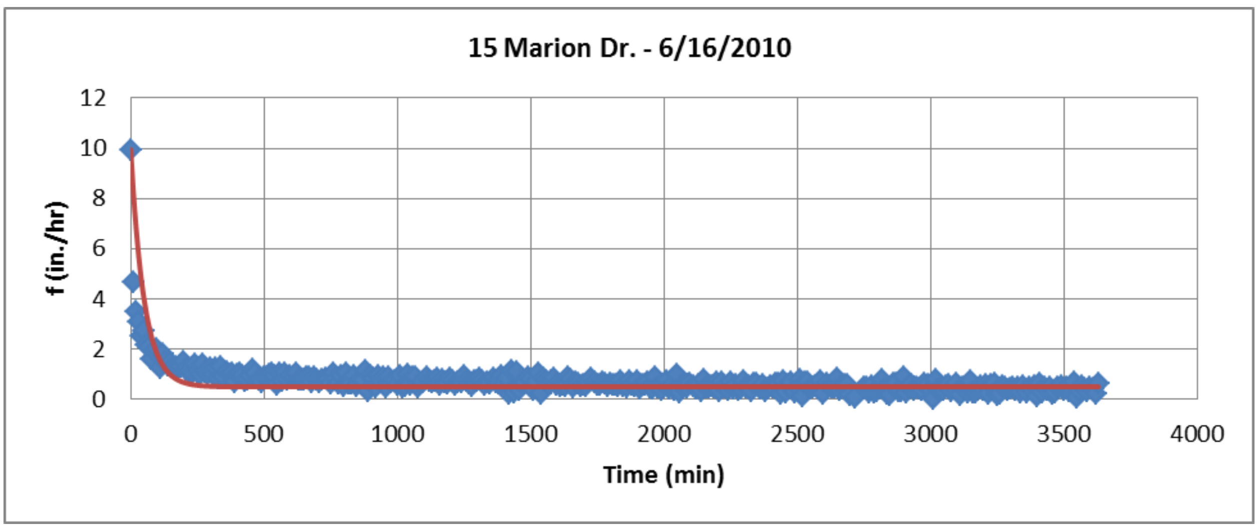


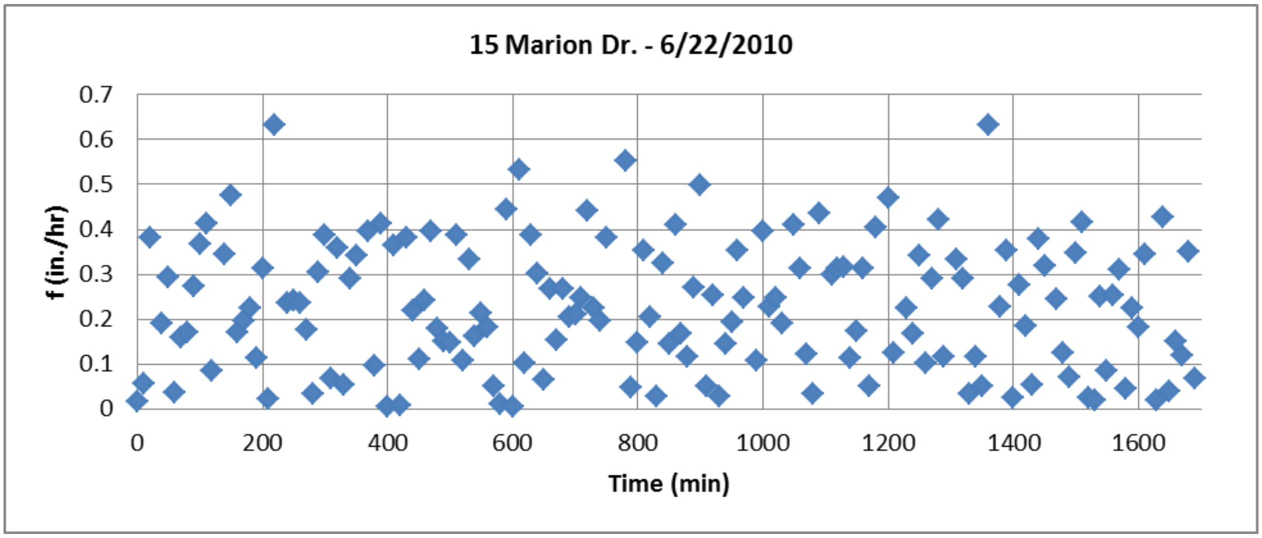






15 Marion Drive





15 Marion Dr. - 6/22/2010
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Time (min)
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0.4
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0.6
0.7
0
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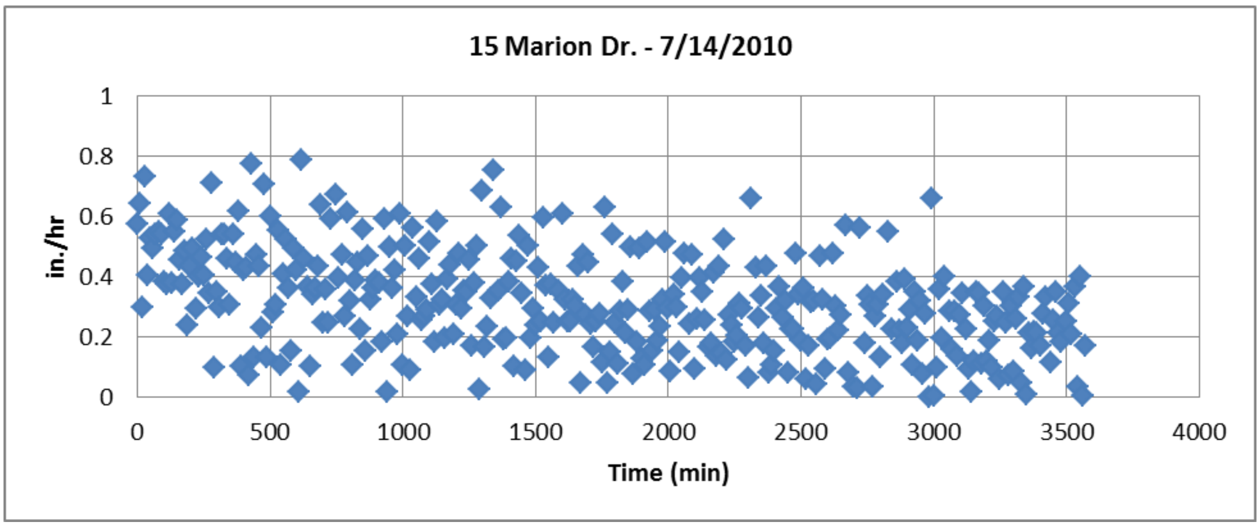
400
600
800
1000
1200
1400
1600



15 Marion Dr. - 7/14/2010
in./hr
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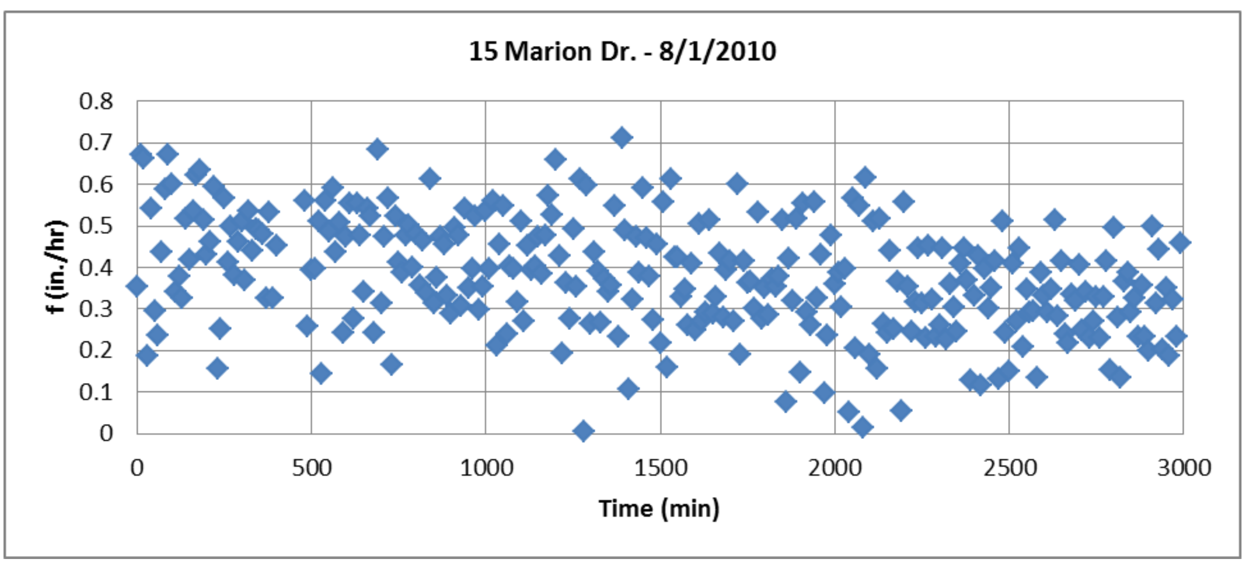
3500
4000



15 Marion Dr. - 8/1/2010
f (in./hr)
Time (min)
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0.2
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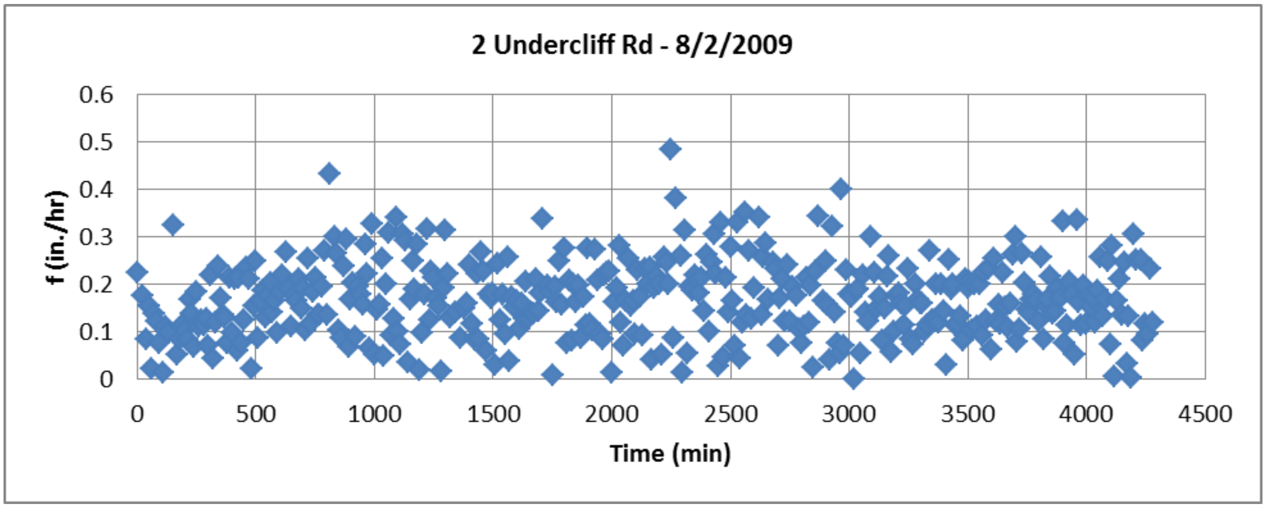
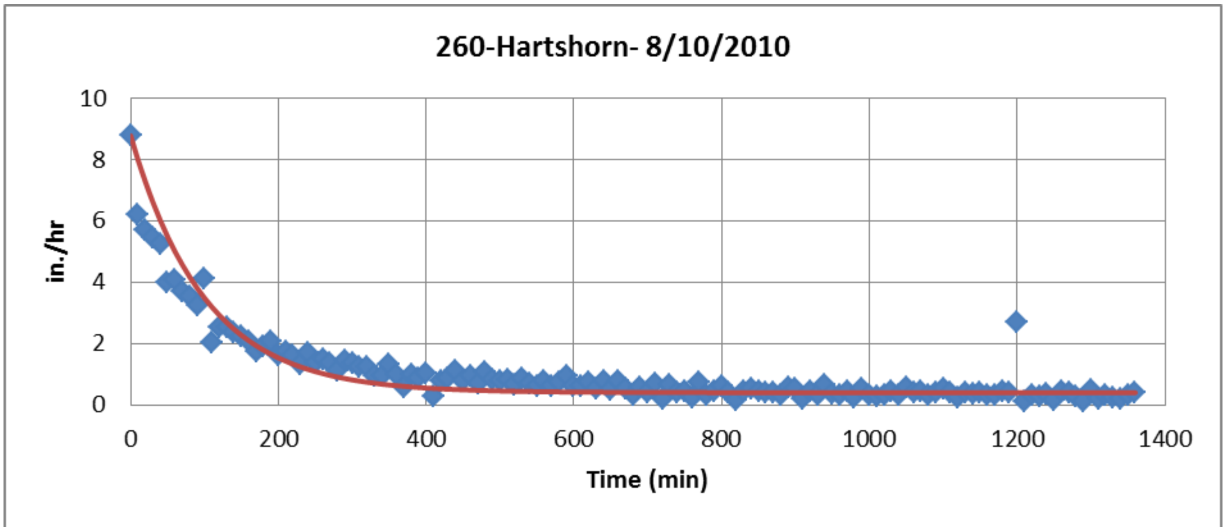
0.7
0.8
0
500
1000
1500
2000
2500
3000

2 Underclif Road



260
Hartshorn


260-Hartshorn- 8/10/2010
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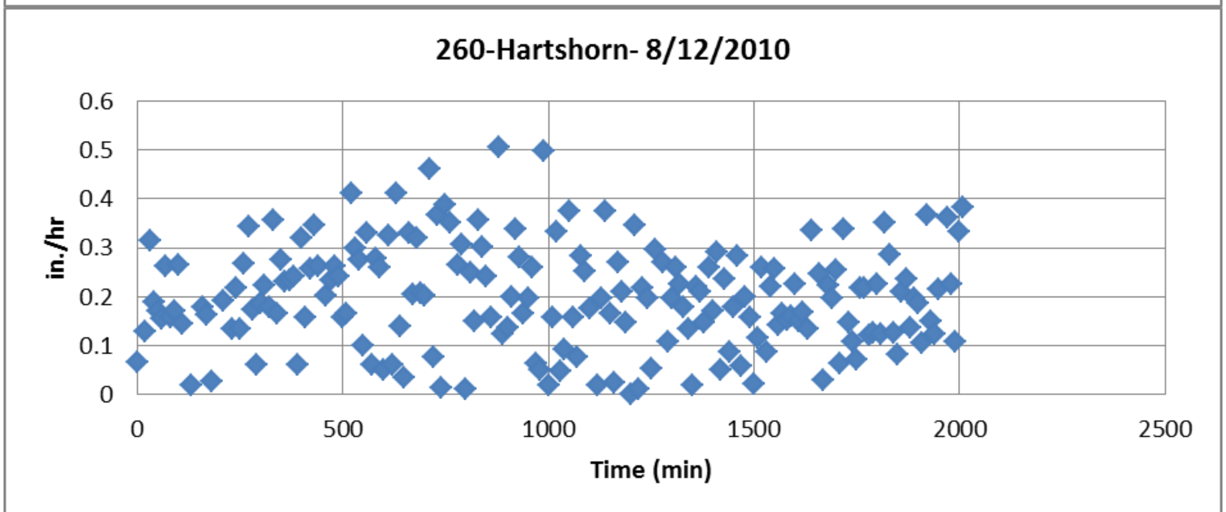
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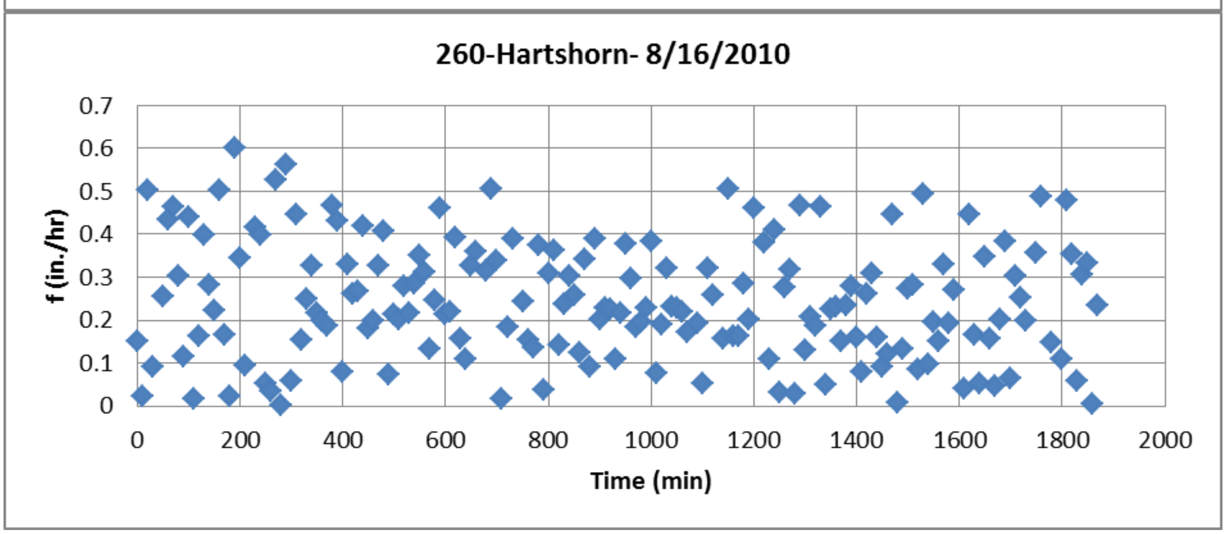
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1800
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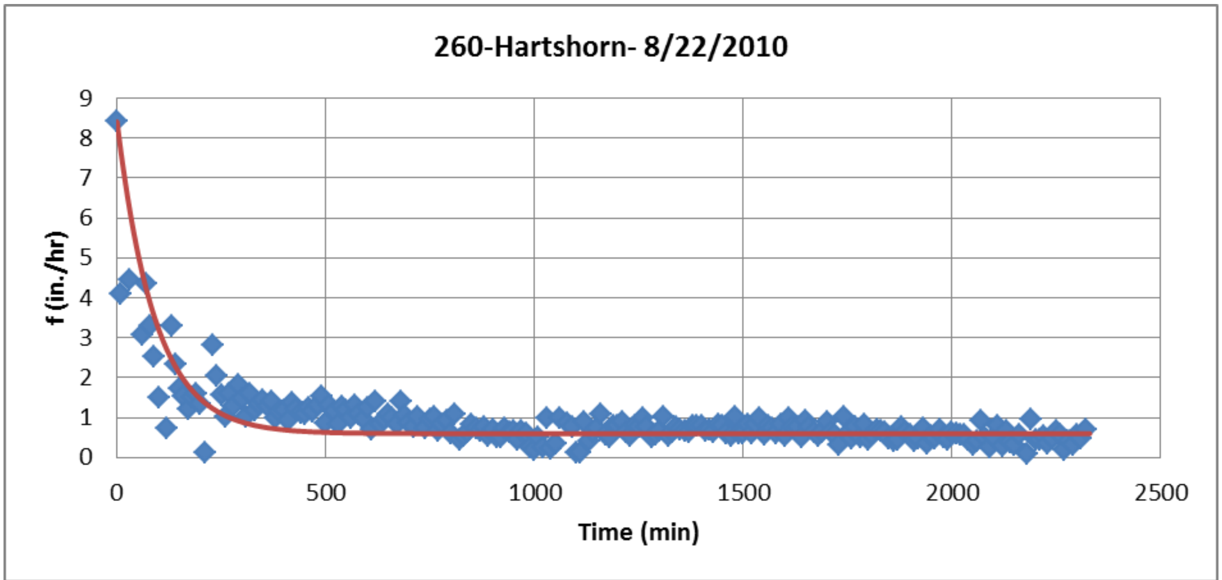
0.5
0.4
0.3
0.2
0.1
0


260-Hartshorn- 8/22/2010
f (in./hr)
Time (min)
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1
2
3
4
5
6
7
8
9
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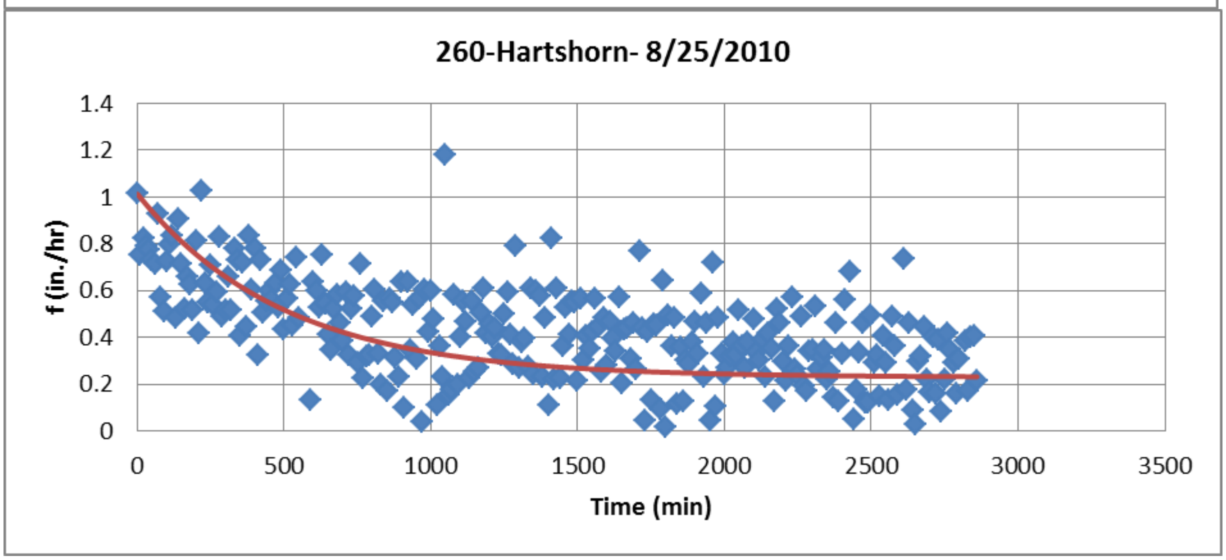
1000
1500
2000
2500


260-Hartshorn- 8/25/2010
f (in./hr)
Time (min)
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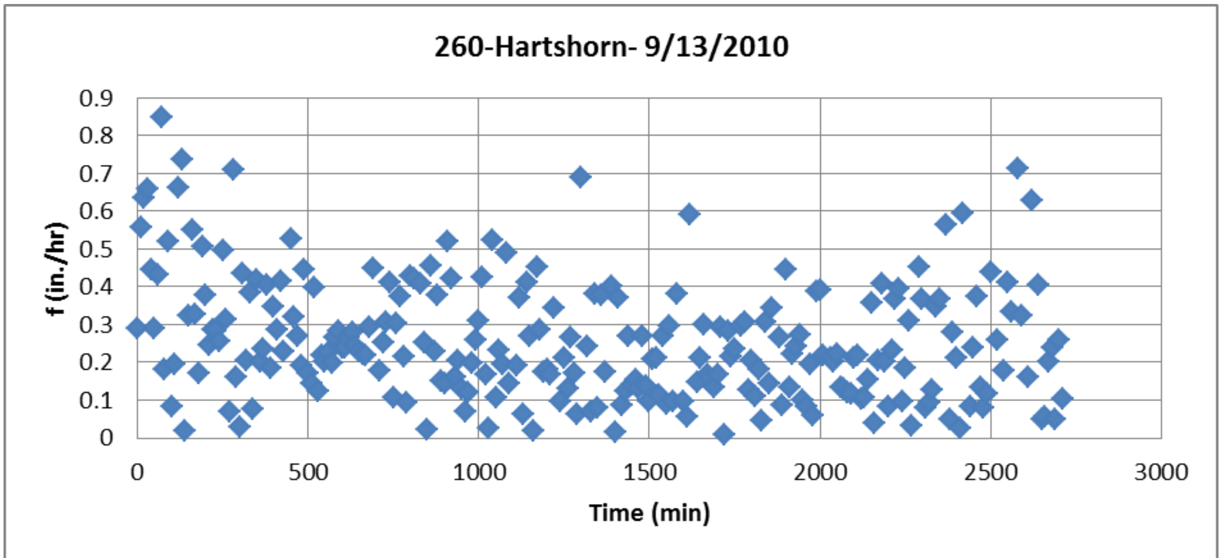
0.2
0.4
0.6
0.8
1
1.2
1.4
0
500
1000
1500
2000
2500
3000
3500


260-Hartshorn- 9/13/2010
f (in./hr)
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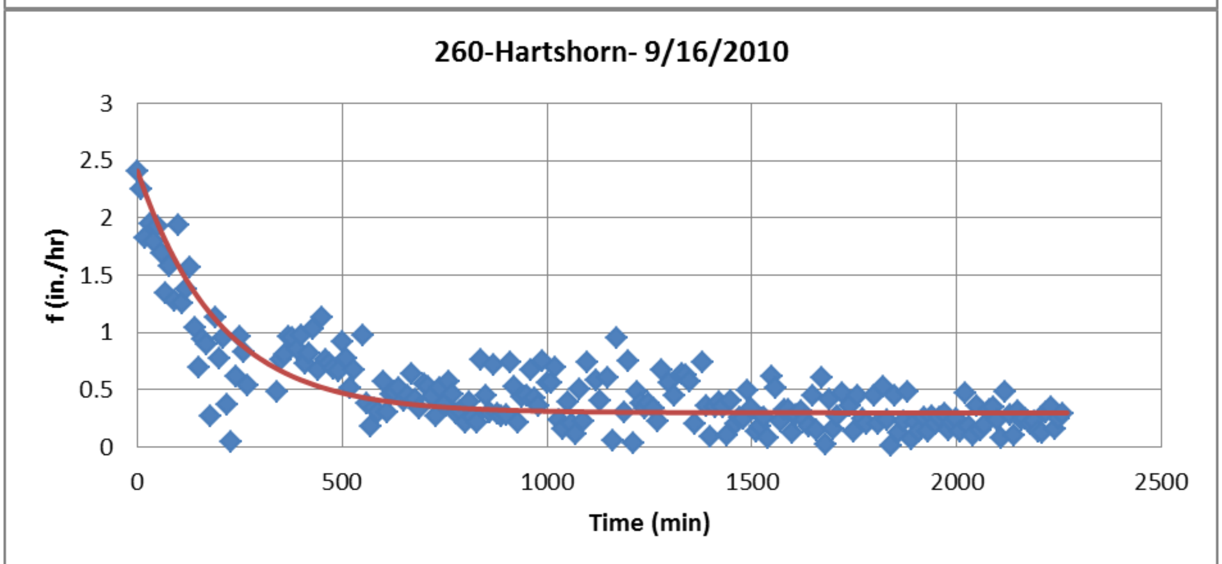
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
0
500
1000
1500
2000
2500
3000
Time (min)


260-Hartshorn- 9/16/2010
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0.5
1
1.5
2
2.5
3
0
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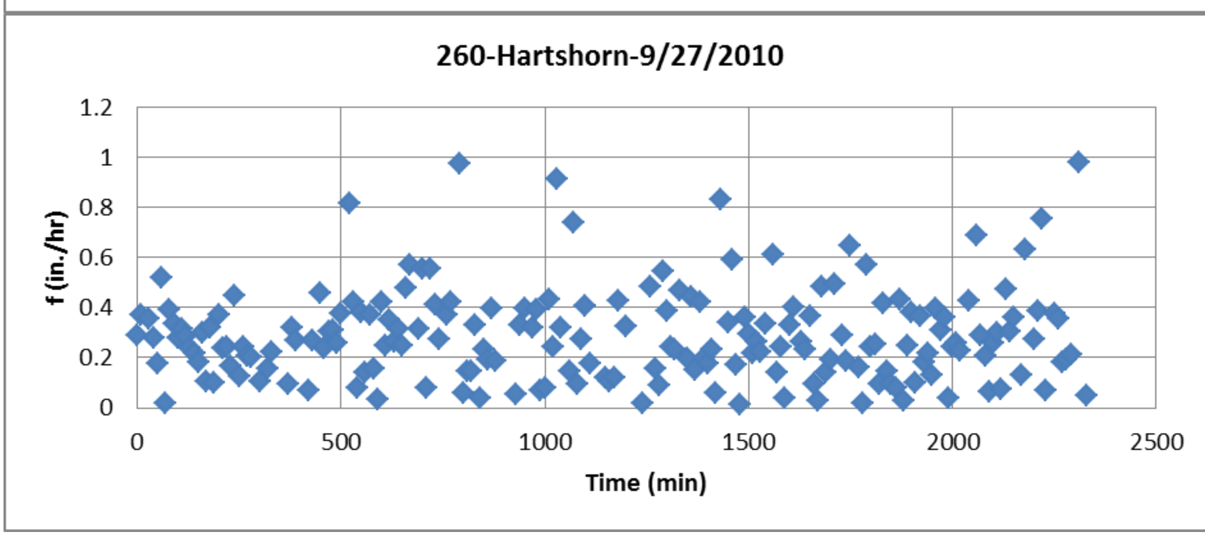
1000
1500
2000
2500
Time (min)


260-Hartshorn-9/27/2010
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0.4
0.6
0.8
1
1.2
0
500
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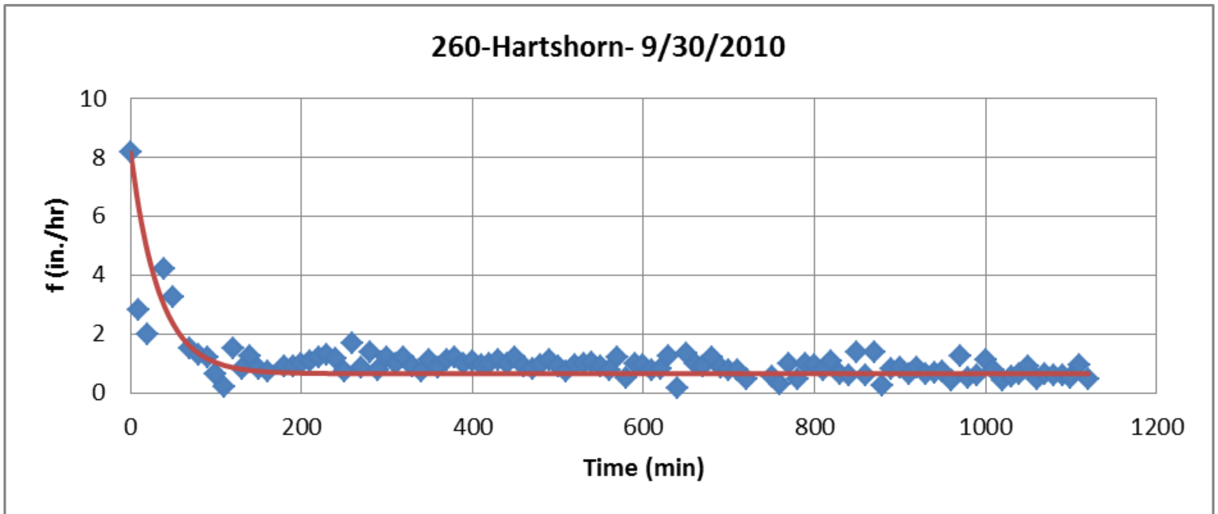
1500
2000
2500
Time (min)


260-Hartshorn- 9/30/2010
f (in./hr)
Time (min)
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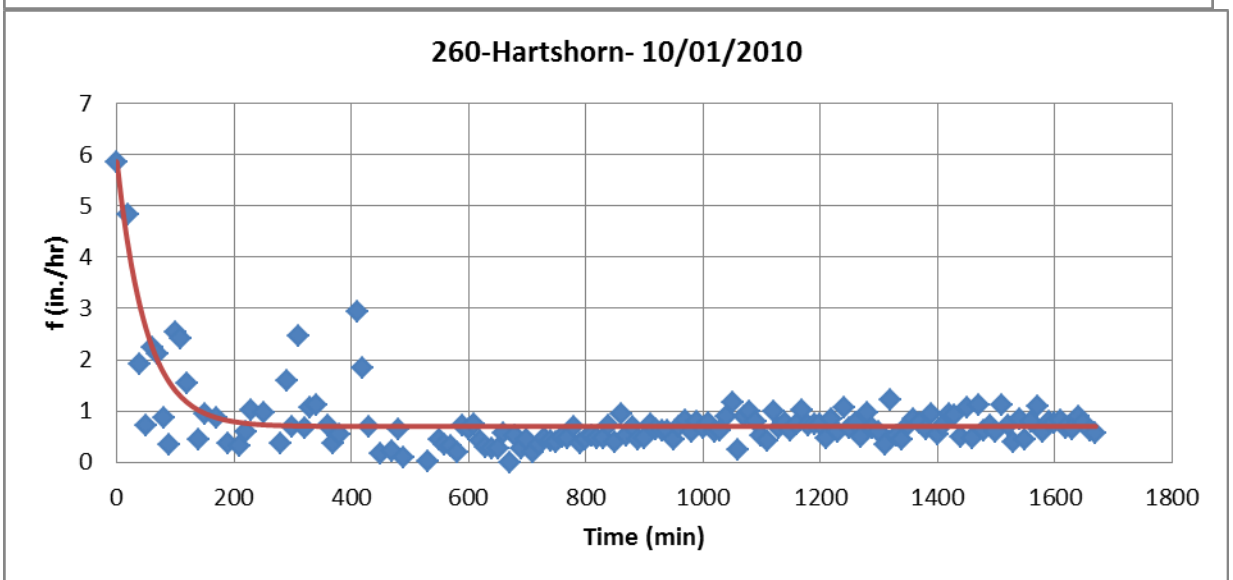
2
4
6
8
10
200
400
600
800
1000
1200


260-Hartshorn- 10/01/2010
f (in./hr)
Time (min)
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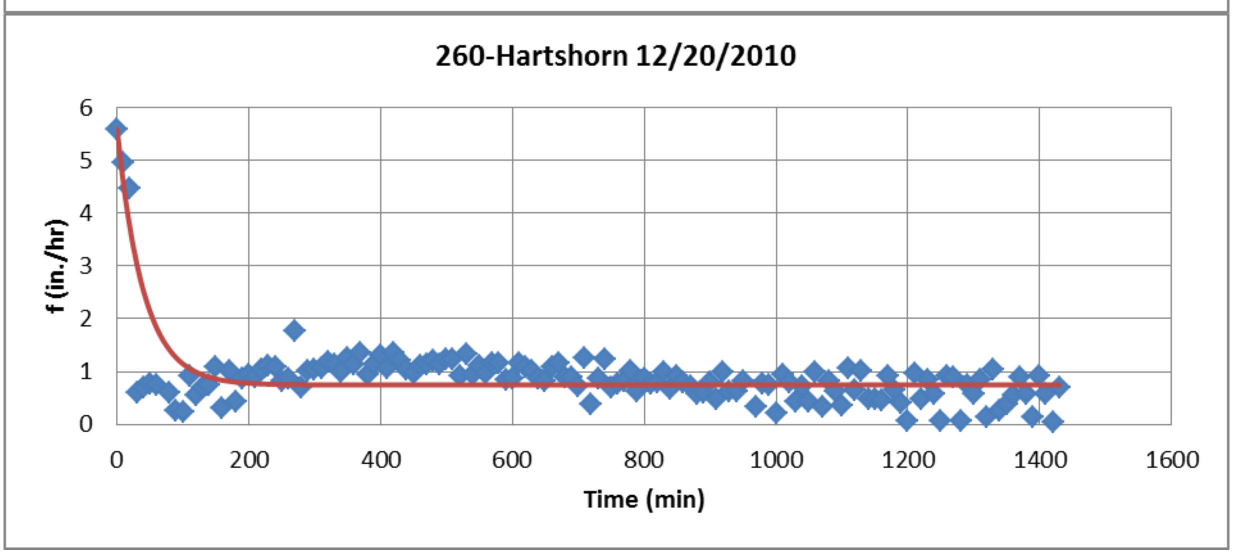
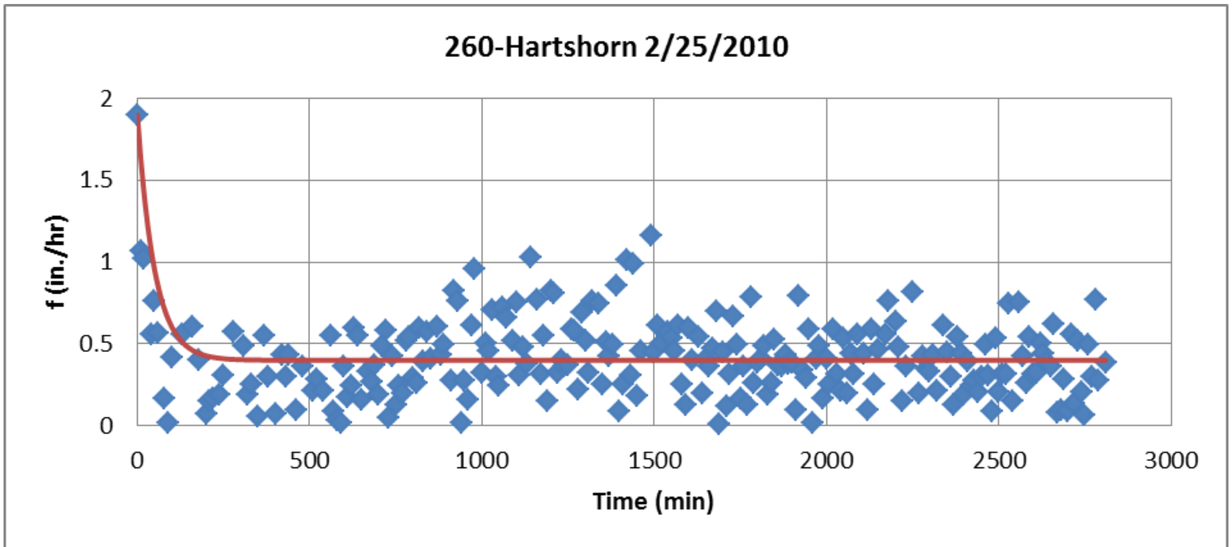
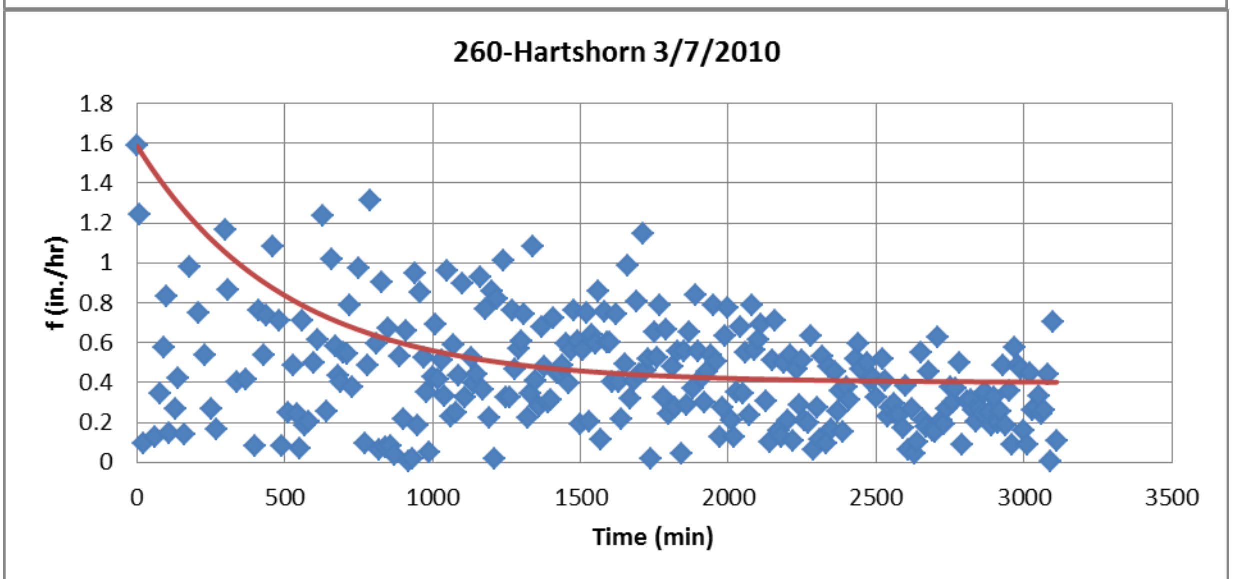
1
2
3
4
5
6
7
200
400
600
800
1000
1200
1400
1600
1800


260-Hartshorn 12/20/2010
f (in./hr)
Time (min)
0
1
2
3
4
5
6
200
400
600
800
1000
1200
1400
1600


260-Hartshorn 2/25/2010
f (in./hr)
2
1.5
1
0.5
0
0
500
1000
1500
2000
2500
3000
Time (min)


260-Hartshorn 3/7/2010
f (in./hr)
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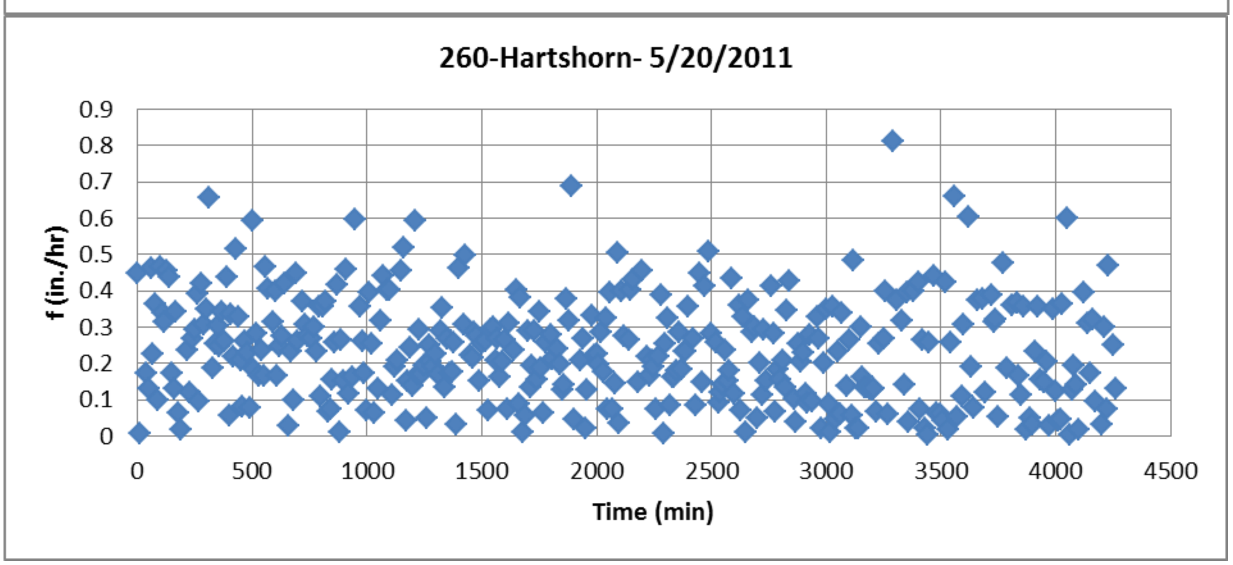
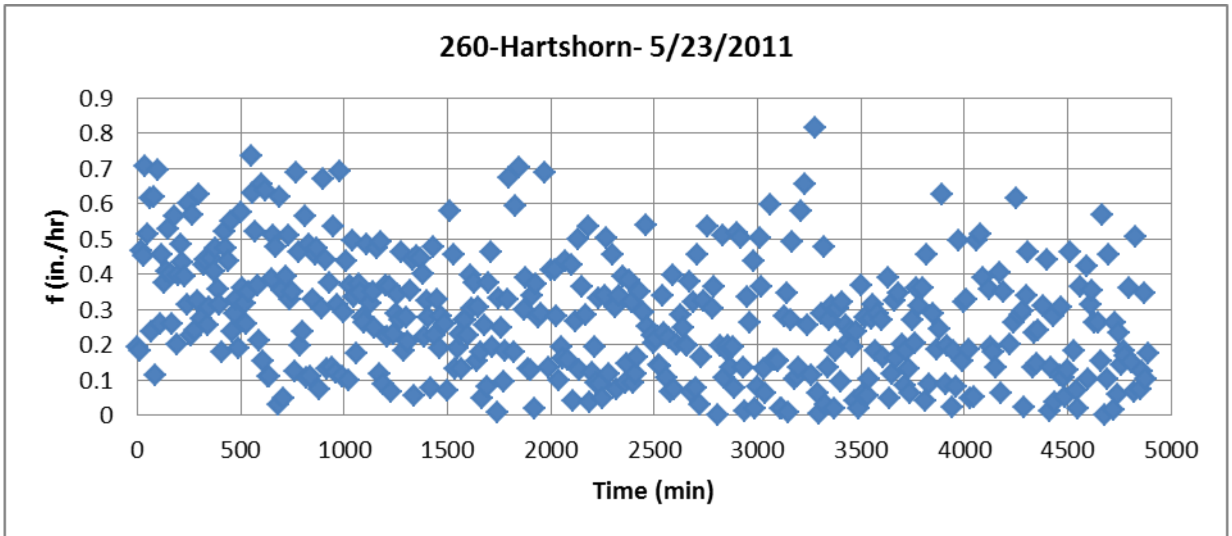
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
500
1000
1500
2000
2500
3000
3500
Time (min)


260-Hartshorn- 5/20/2011
f (in./hr)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500
Time (min)


260-Hartshorn- 5/23/2011
f (in./hr)
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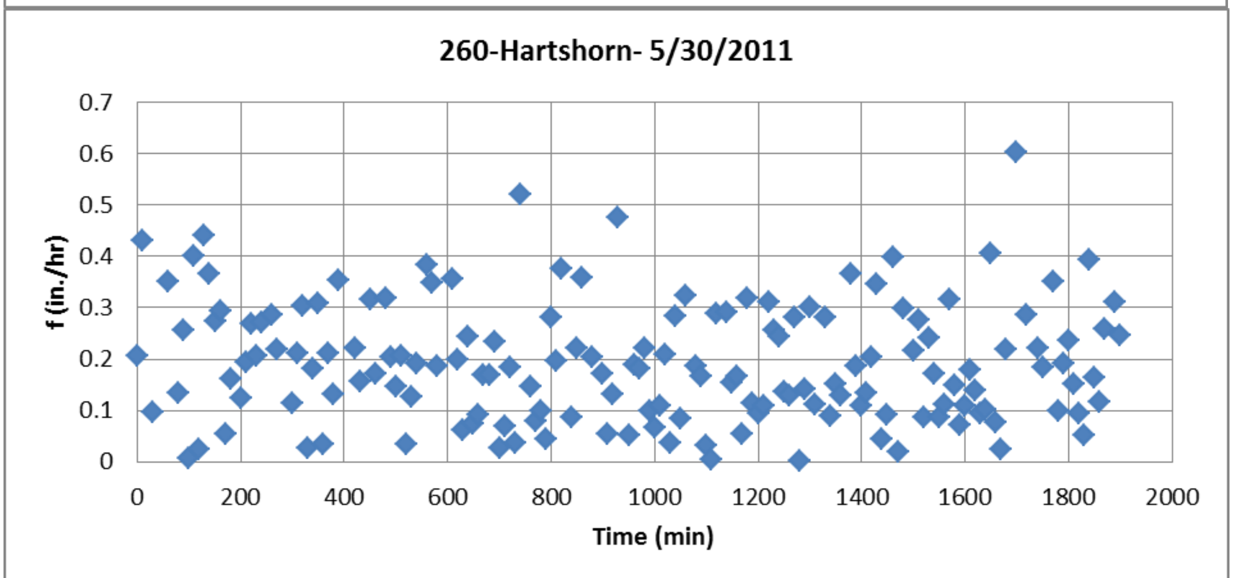
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Time (min)


260-Hartshorn- 5/30/2011
f (in./hr)
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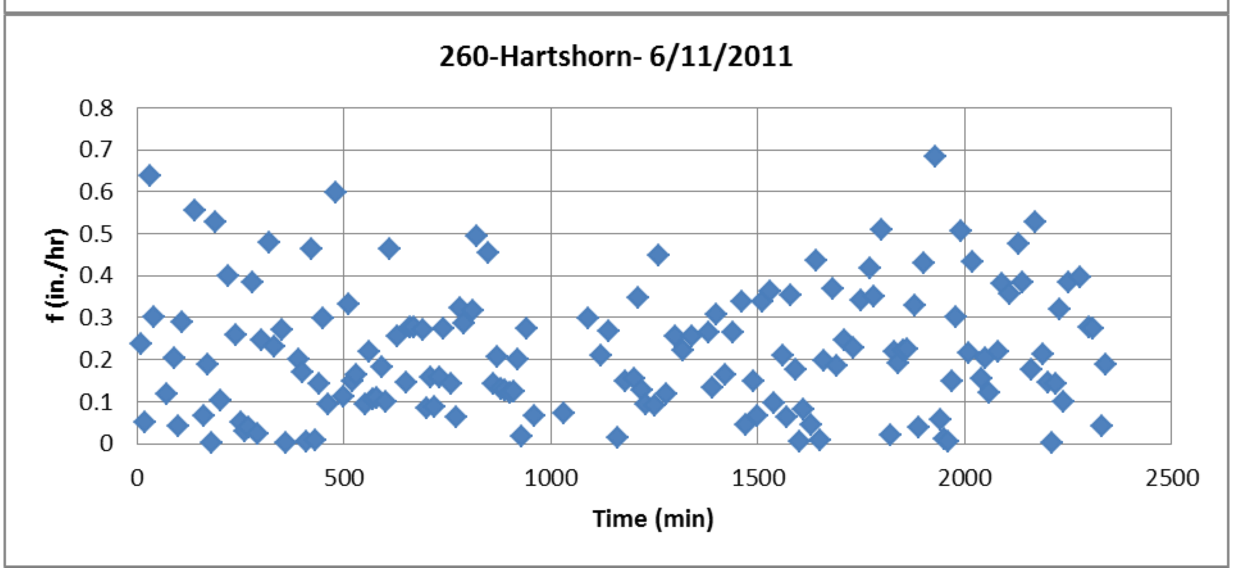
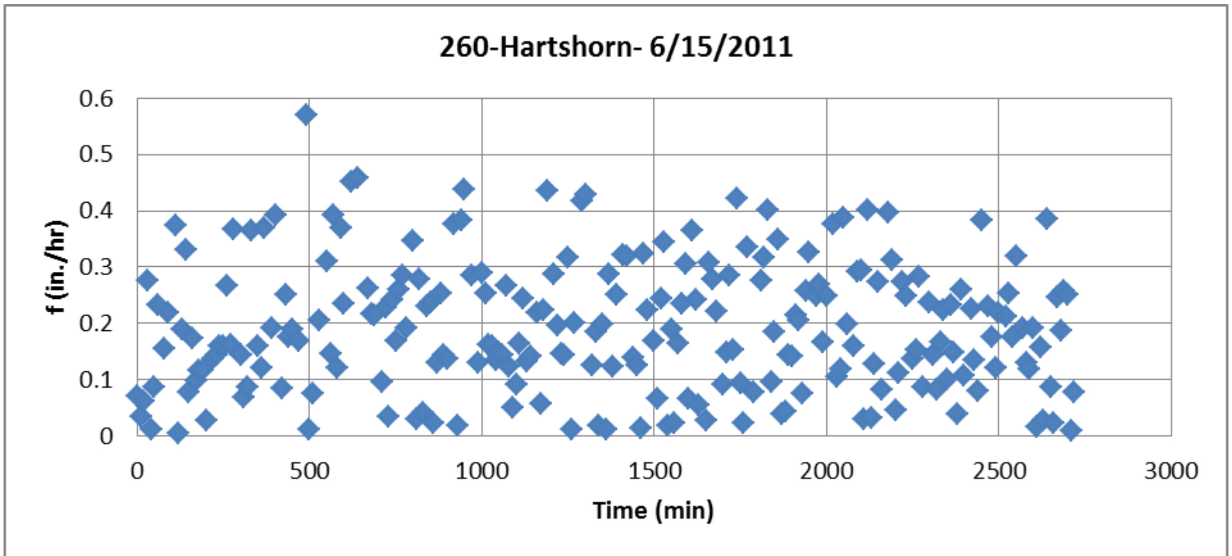
0.6
0.5
0.4
0.3
0.2
0.1
0
0
200
400
600
800
1000
1200
1400
1600
1800
2000
Time (min)


260-Hartshorn- 6/11/2011
f (in./hr)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
Time (min)


260-Hartshorn- 6/15/2011
f (in./hr)
Time (min)
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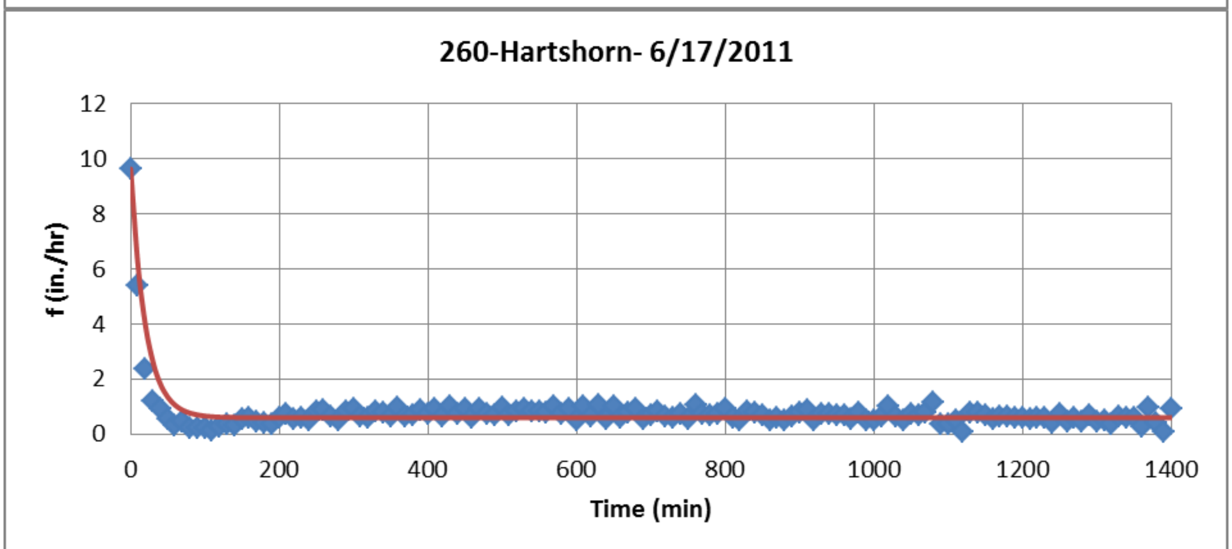
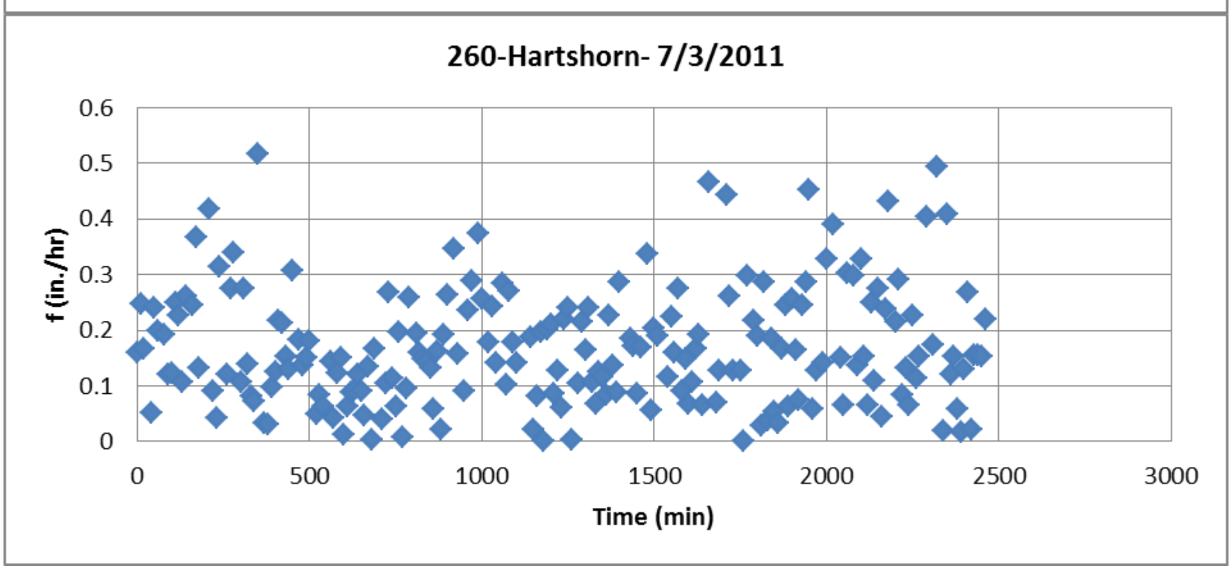
0.1
0.2
0.3
0.4
0.5
0.6
0
500
1000
1500
2000
2500
3000


260-Hartshorn- 6/17/2011
f (in./hr)
Time (min)
0
2
4
6
8
10
12
0
200
400
600
800
1000
1200
1400


260-Hartshorn- 7/3/2011
f (in./hr)
Time (min)
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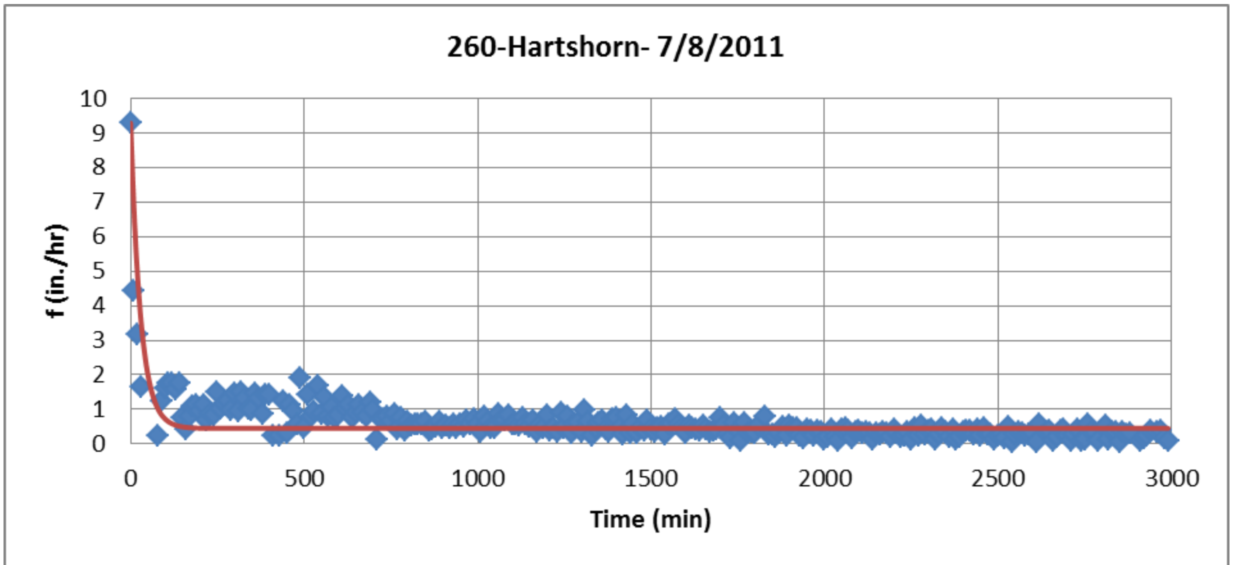
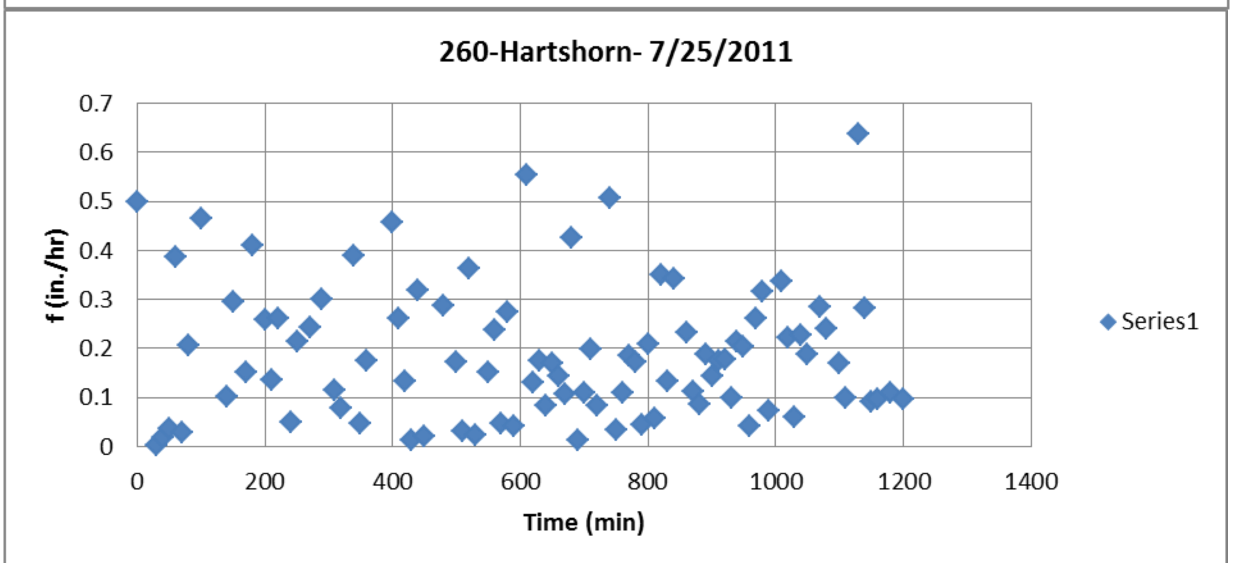
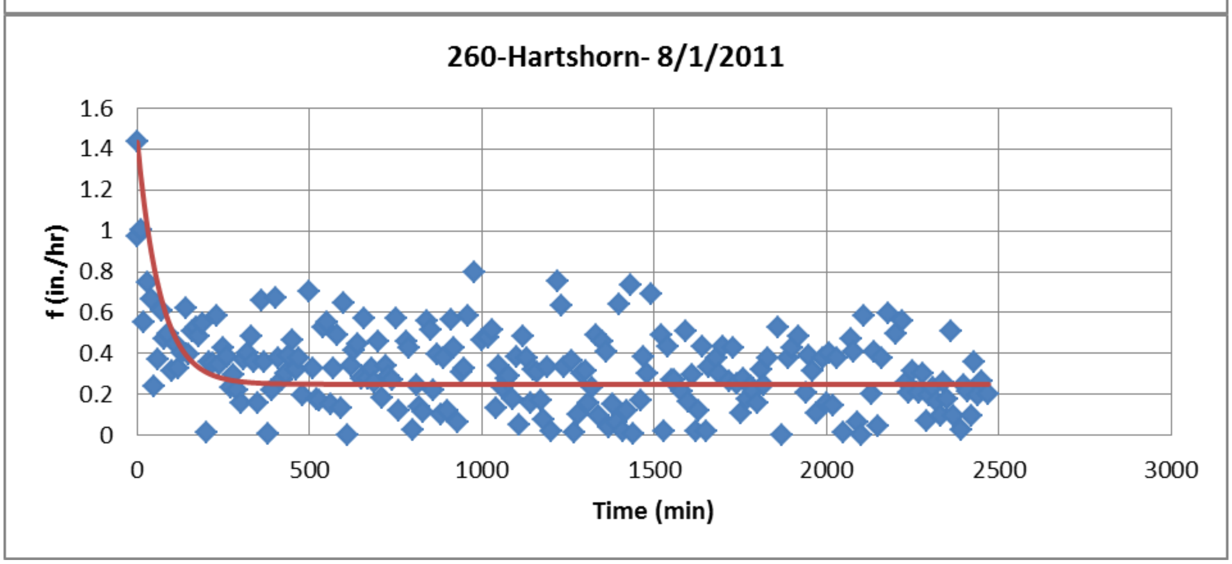
0.2
0.3
0.4
0.5
0.6
0
500
1000
1500
2000
2500
3000


260-Hartshorn- 7/8/2011
f (in./hr)
Time (min)
10
9
8
7
6
5
4
3
2
1
0
0
500
1000
1500
2000
2500
3000


260-Hartshorn- 7/25/2011
f (in./hr)
Time (min)
Series1
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
200
400
600
800
1000
1200
1400


260-Hartshorn- 8/1/2011
f (in./hr)
Time (min)
1.6
1.4
1.2
1
0.8
0.6
0.4
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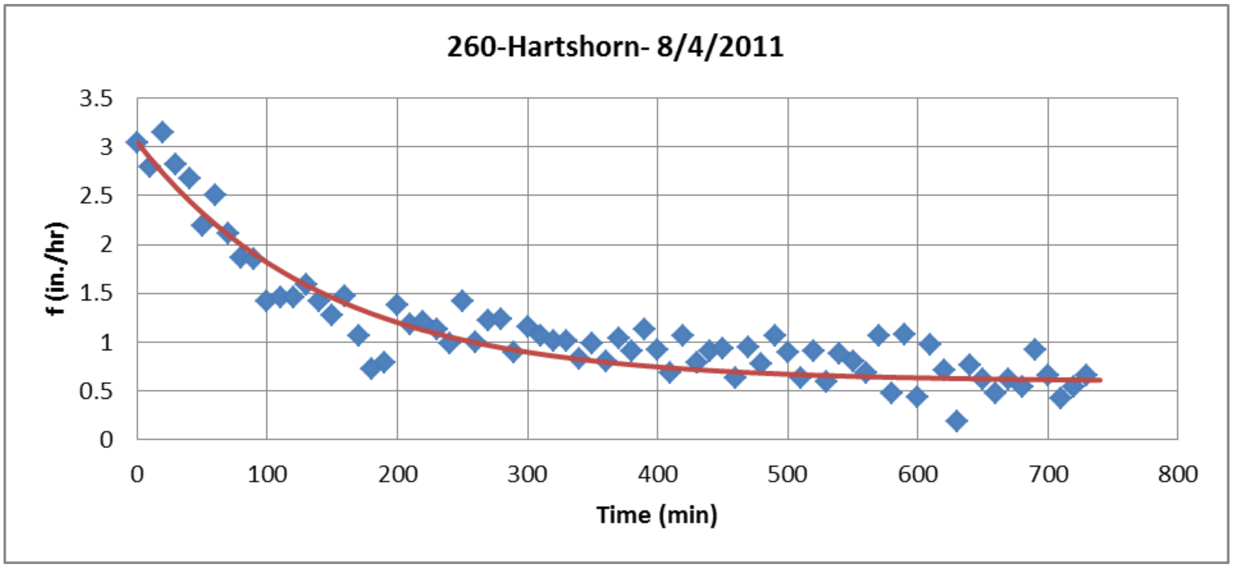
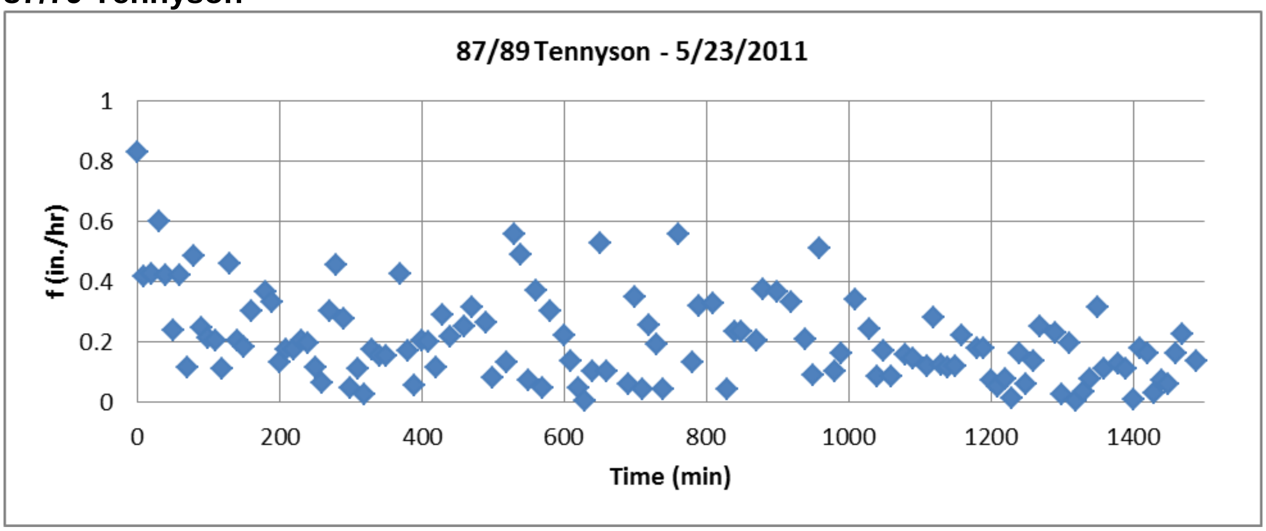
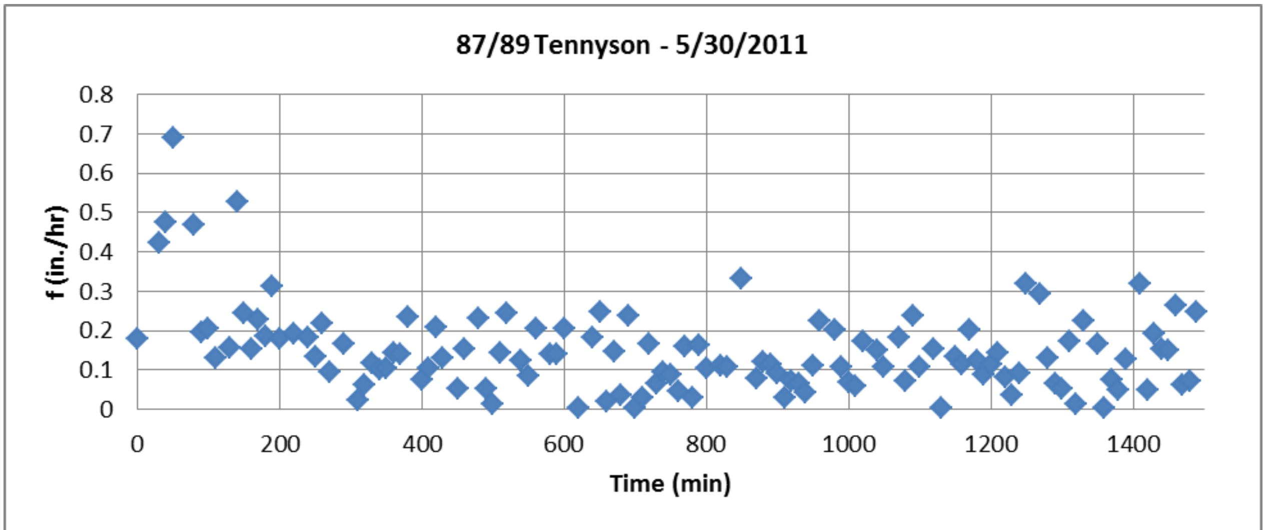
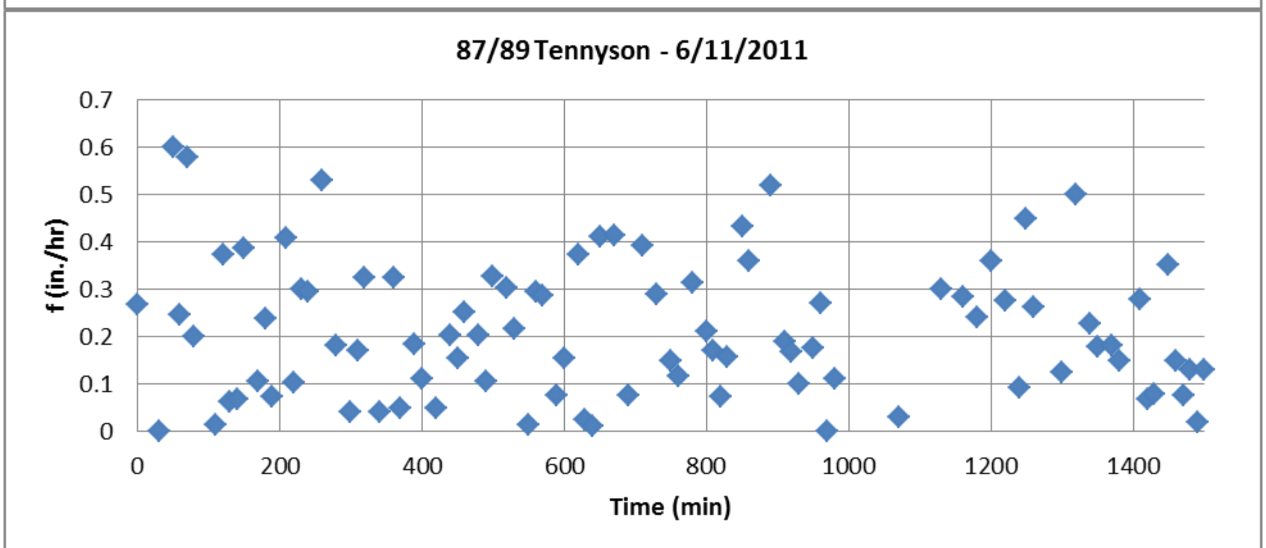
1000
1500
2000
2500
3000

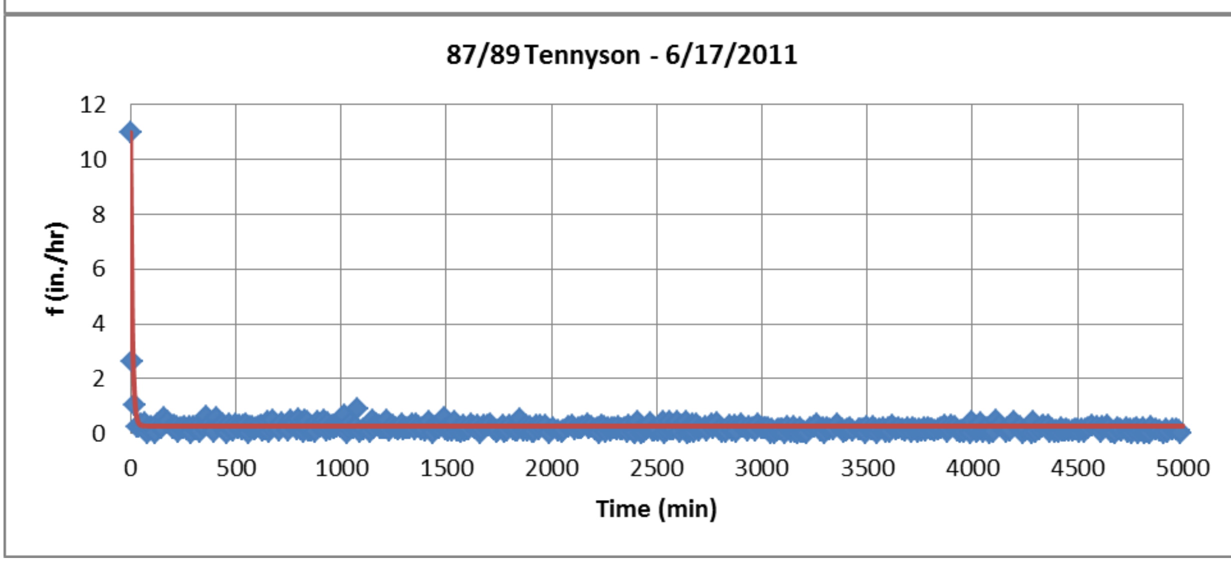


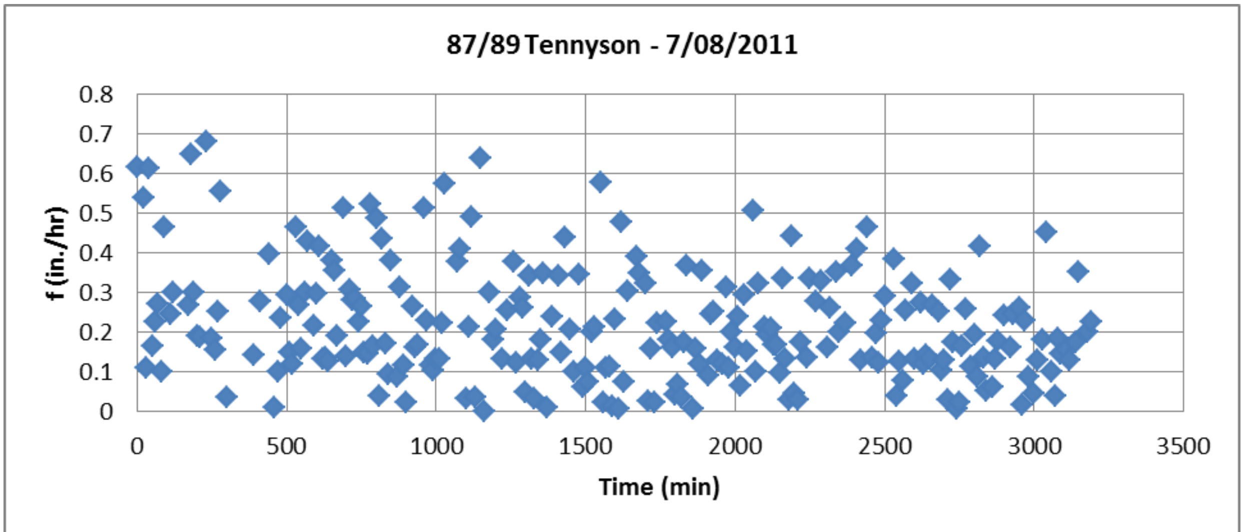
87/79 Tennyson




87/89 Tennyson - 5/30/2011
f (in./hr)
Time (min)


87/89 Tennyson - 6/11/2011
f (in./hr)
Time (min)


87/89 Tennyson - 6/17/2011
f (in./hr)
Time (min)


87/89 Tennyson - 7/08/2011
f (in./hr)
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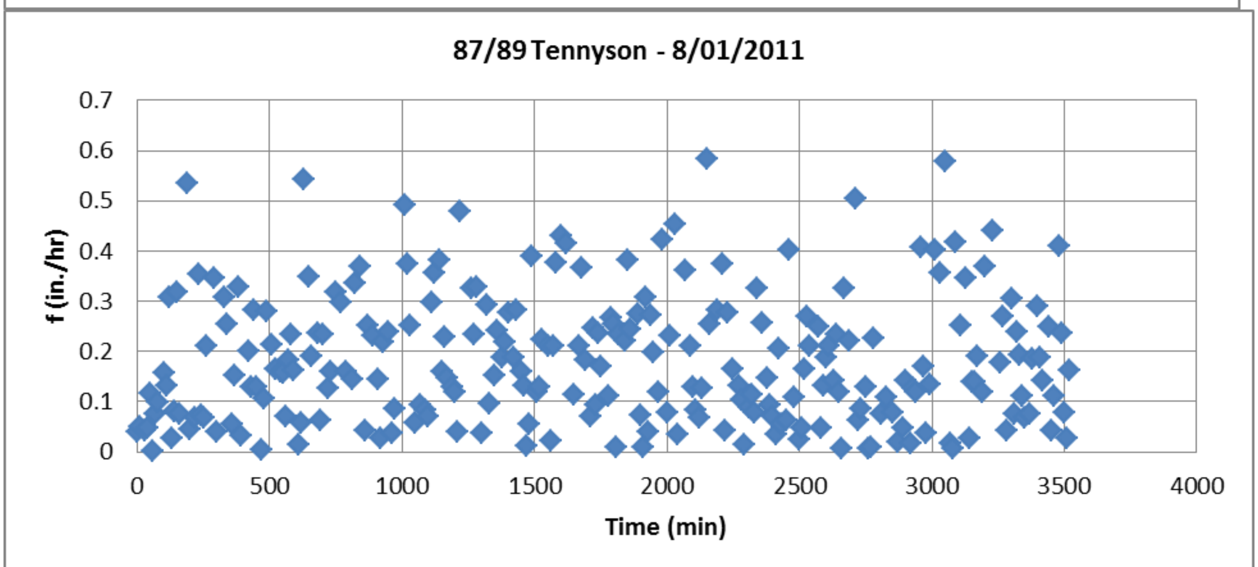
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
3000
3500
Time (min)


87/89 Tennyson - 8/01/2011
f (in./hr)
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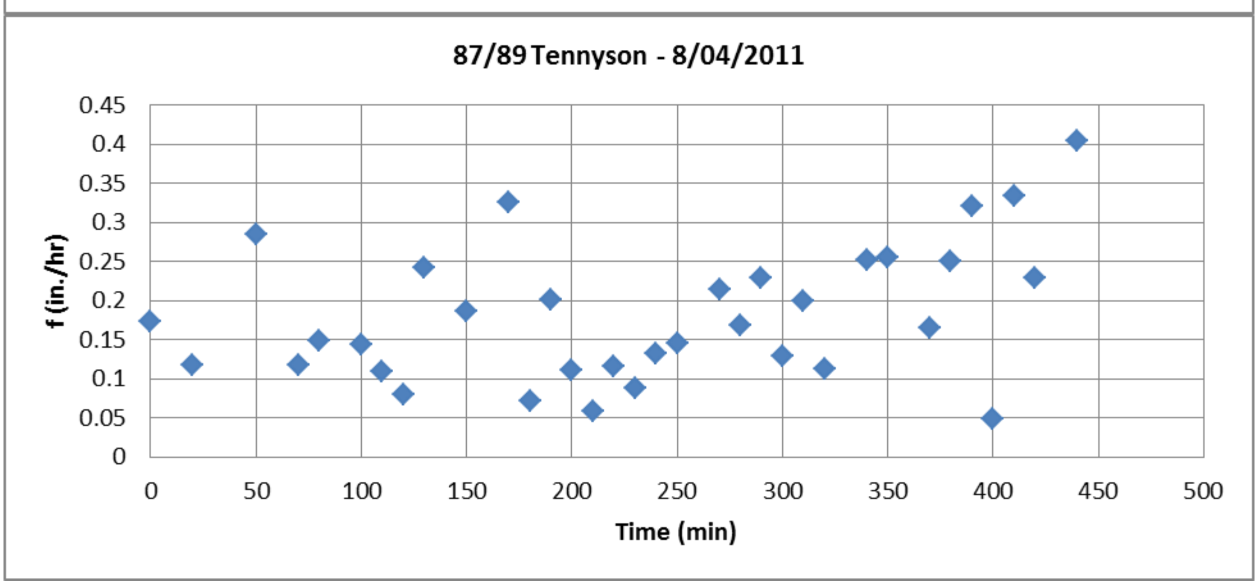
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
3000
3500
4000
Time (min)


87/89 Tennyson - 8/04/2011
f (in./hr)
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0.4
0.35
0.3
0.25
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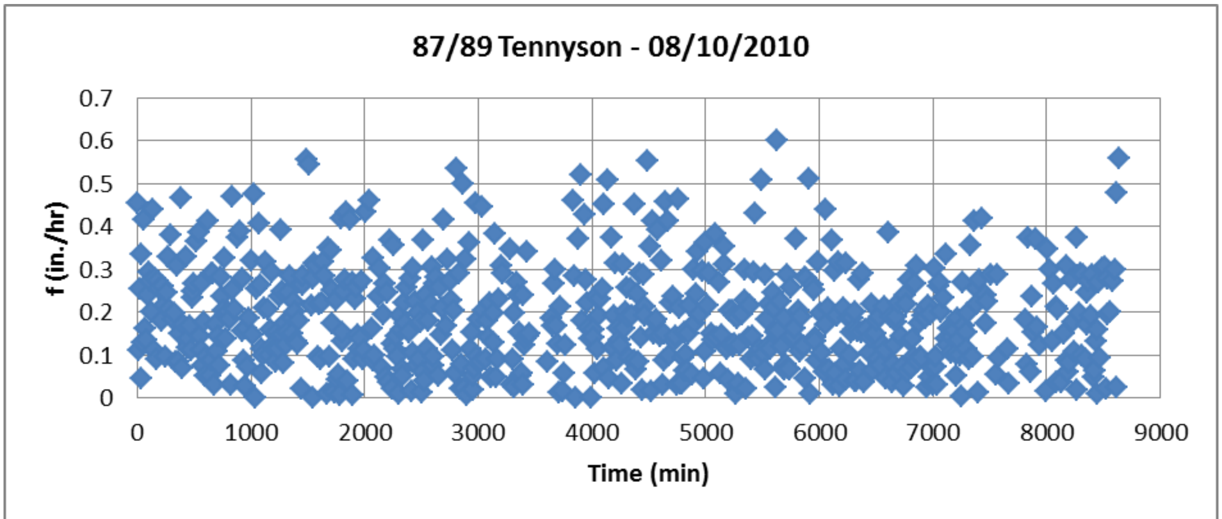
0.1
0.05
0
0
50
100
150
200
250
300
350
400
450
500
Time (min)


87/89 Tennyson - 08/10/2010
f (in./hr)
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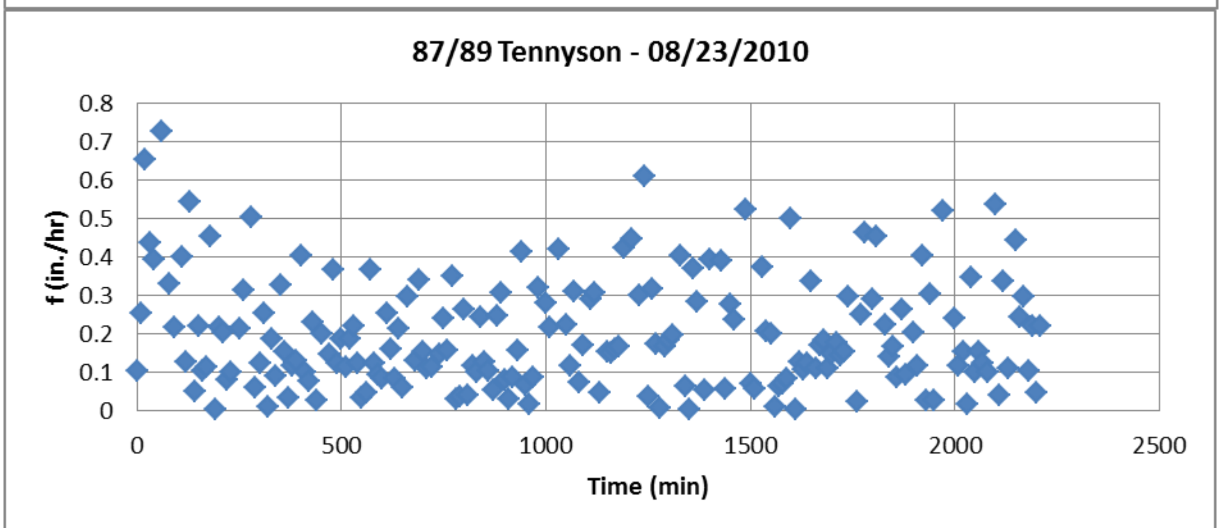
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Time (min)


87/89 Tennyson - 08/23/2010
f (in./hr)
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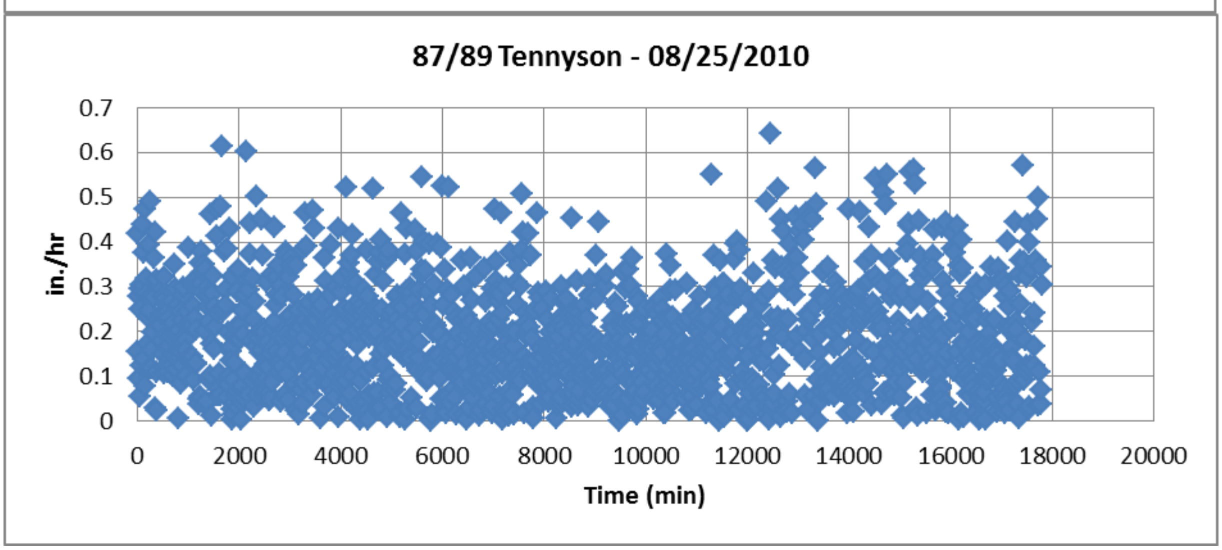
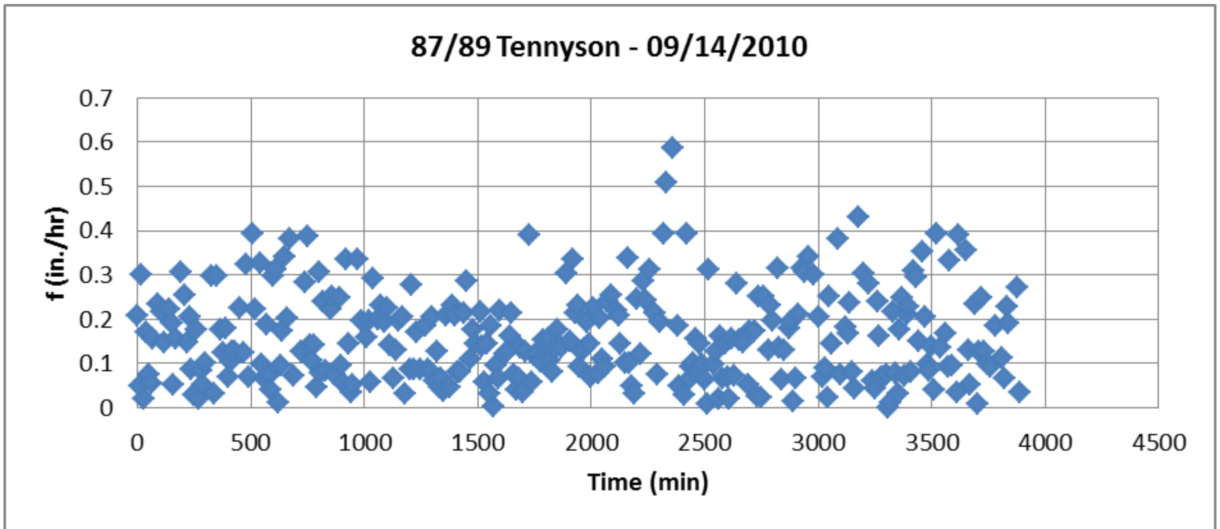
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
Time (min)


87/89 Tennyson - 08/25/2010
in./hr
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Time (min)


87/89 Tennyson - 09/14/2010
f (in./hr)
Time (min)
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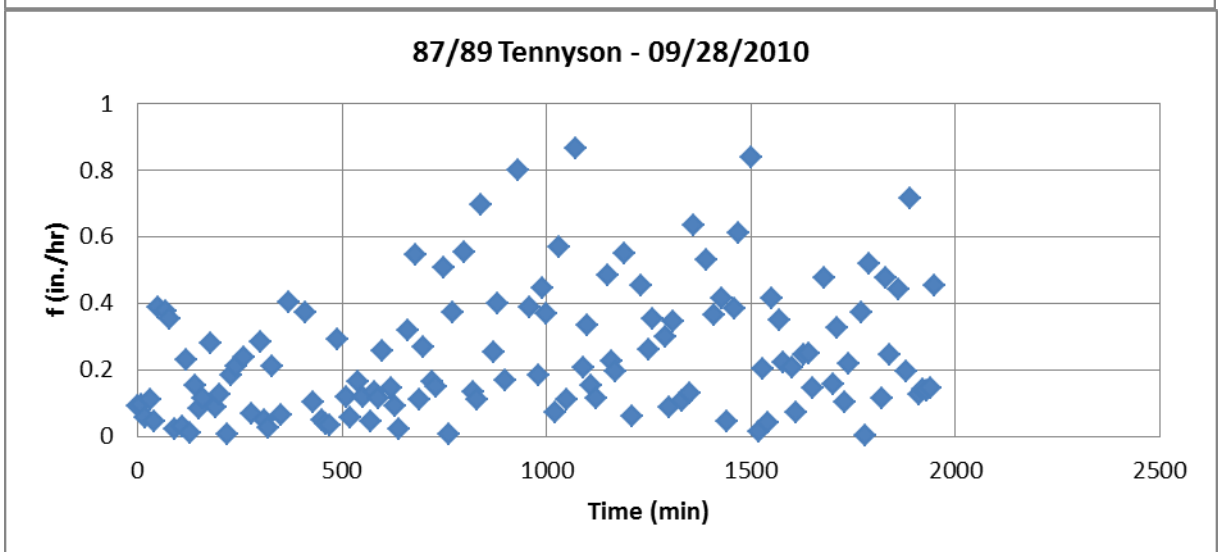
0.2
0.3
0.4
0.5
0.6
0.7
0
500
1000
1500
2000
2500
3000
3500
4000
4500


87/89 Tennyson - 09/28/2010
f (in./hr)
Time (min)
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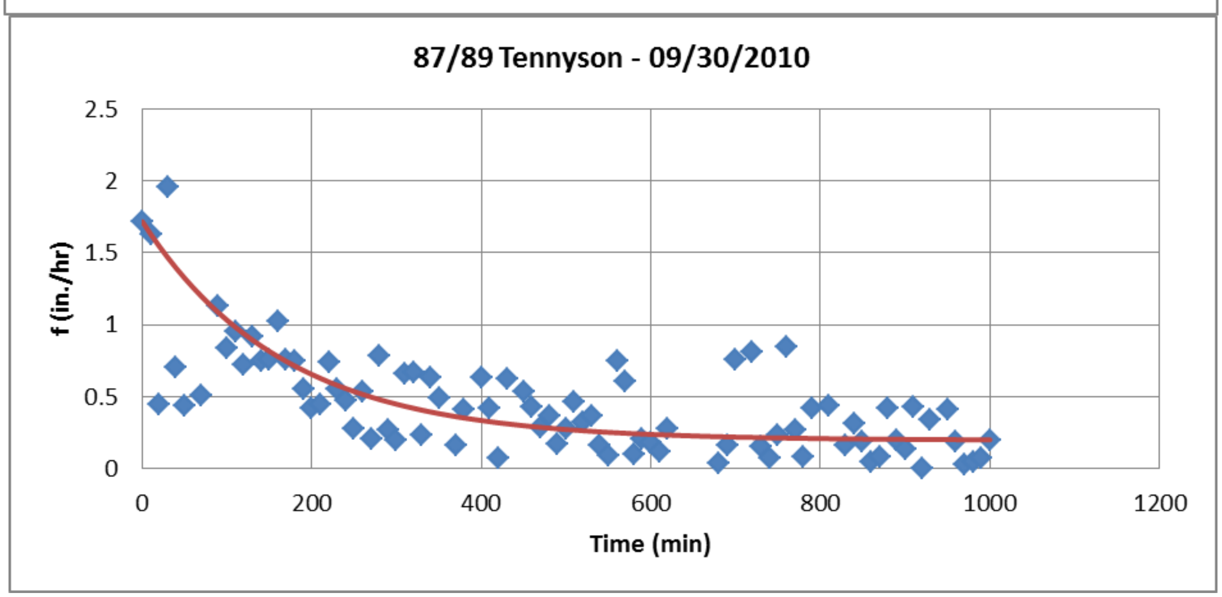
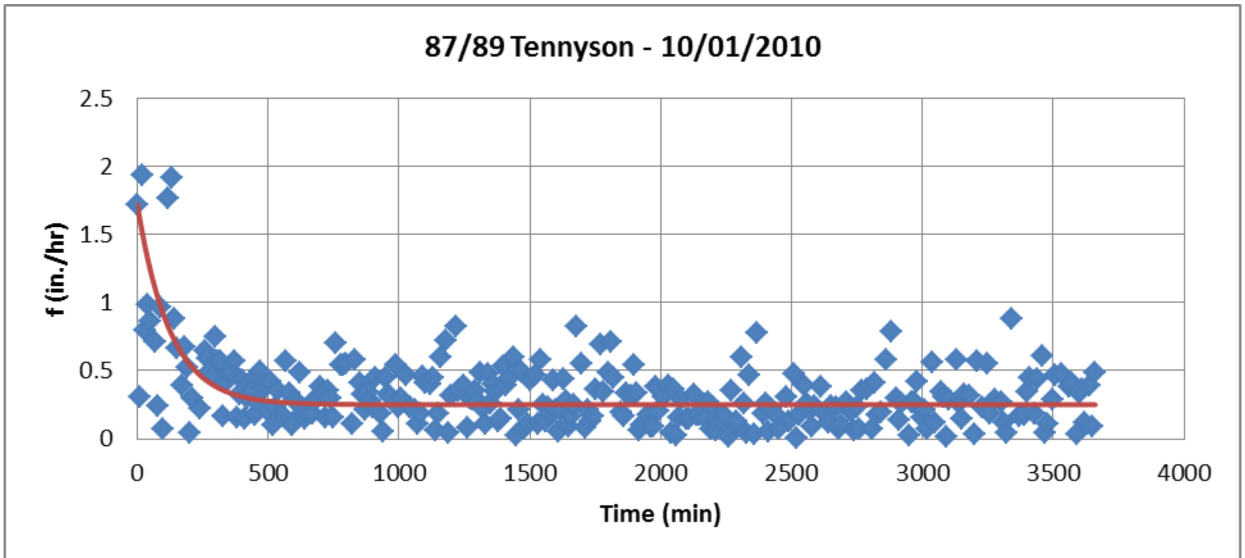
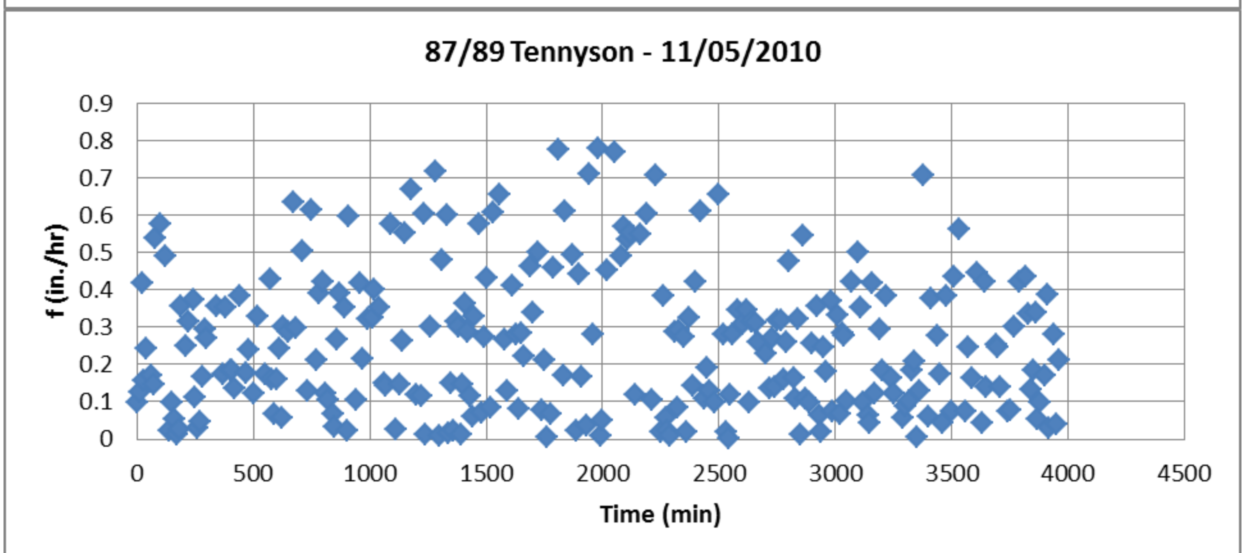
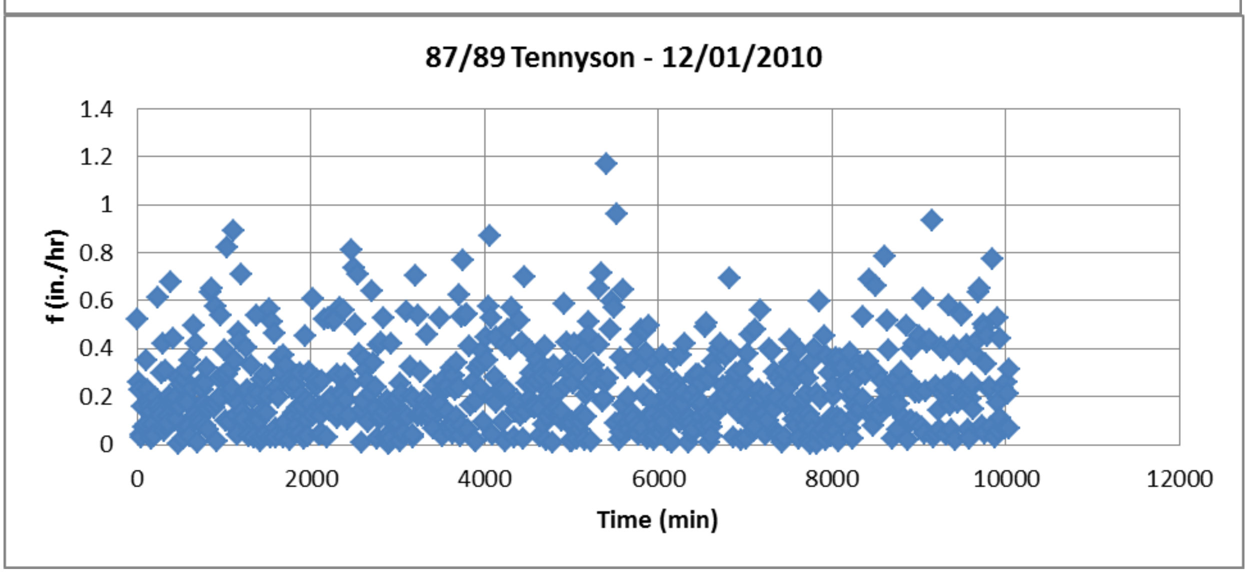
0.4
0.6
0.8
1
0
500
1000
1500
2000
2500


87/89 Tennyson - 09/30/2010
f (in./hr)
Time (min)
0
0.5
1
1.5
2
2.5
0
200
400
600
800
1000
1200


87/89 Tennyson - 10/01/2010
f (in./hr)
Time (min)
2.5
2
1.5
1
0.5
0
0
500
1000
1500
2000
2500
3000
3500
4000


87/89 Tennyson - 11/05/2010
f (in./hr)
Time (min)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500


87/89 Tennyson - 12/01/2010
f (in./hr)
Time (min)
1.4
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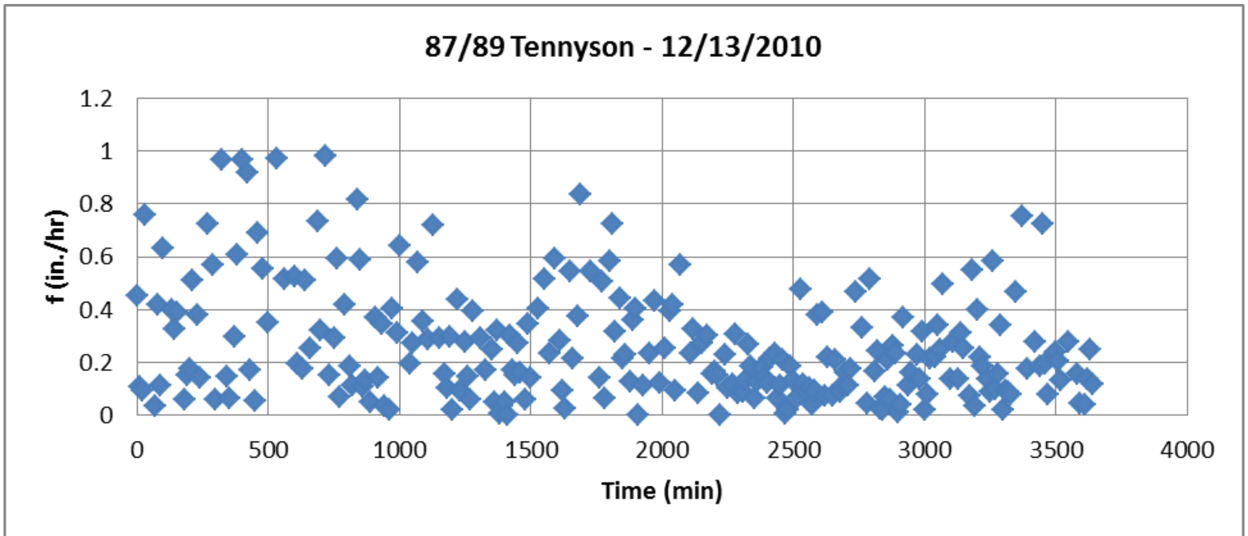
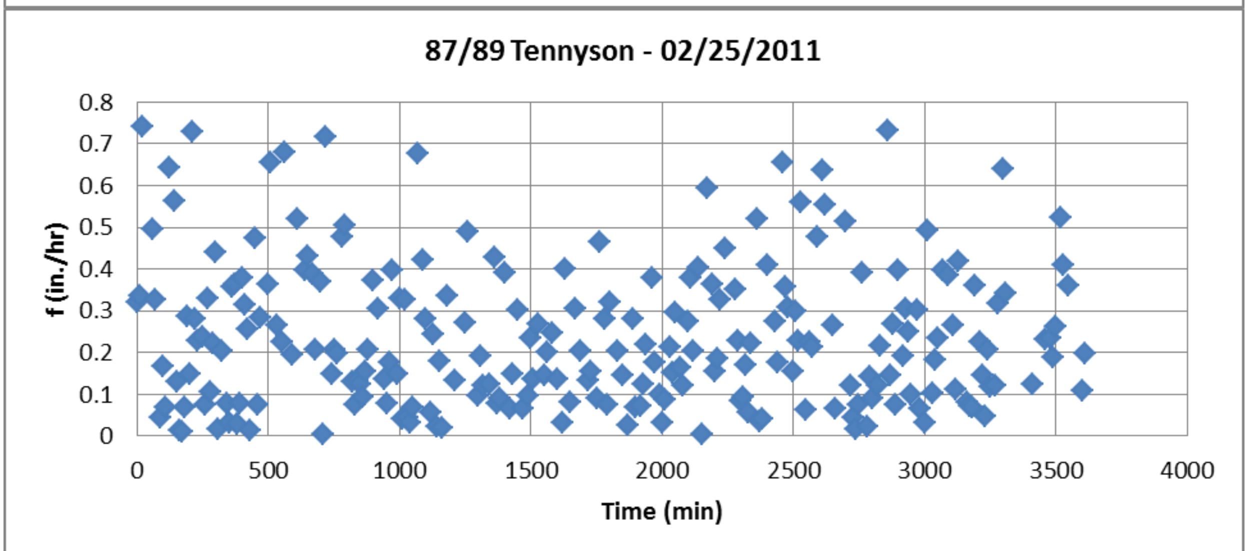
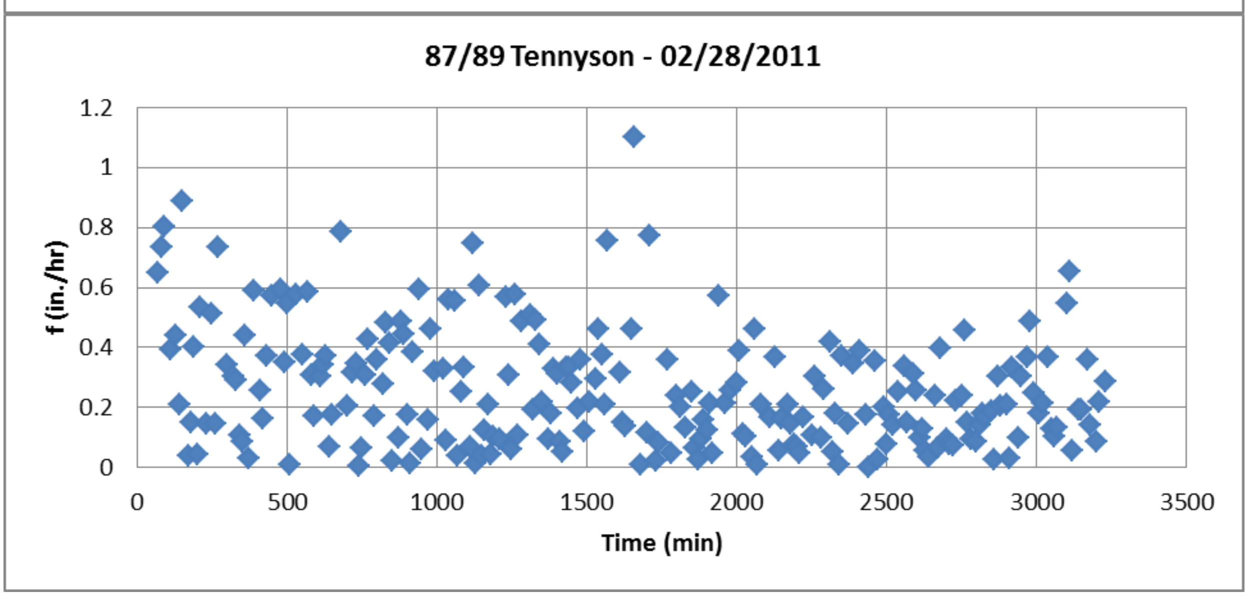
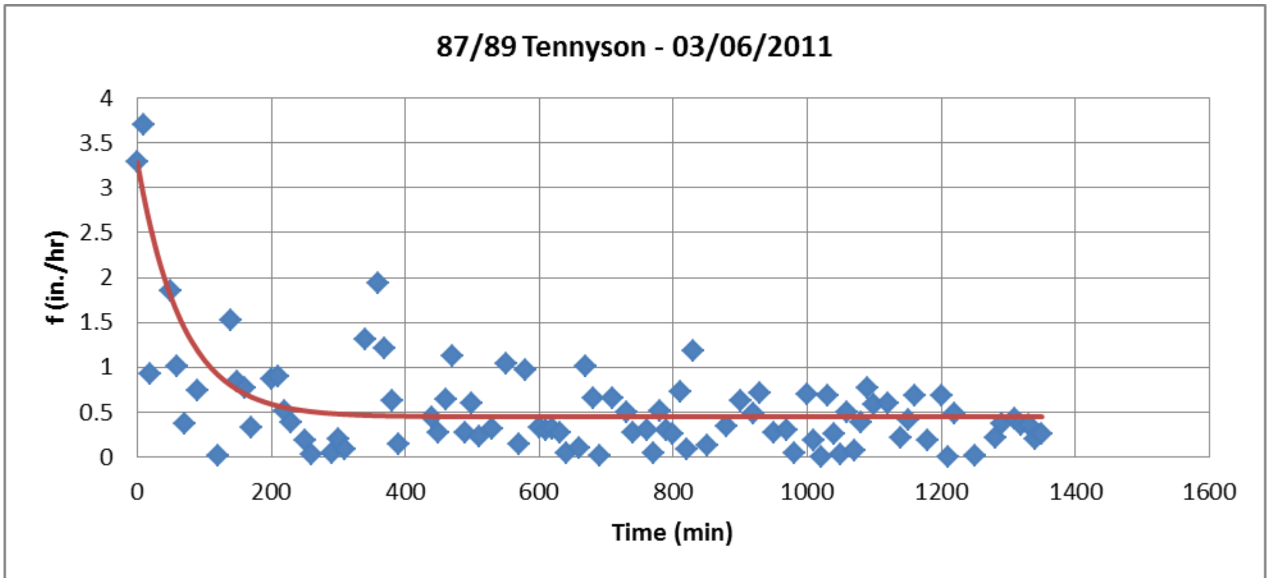
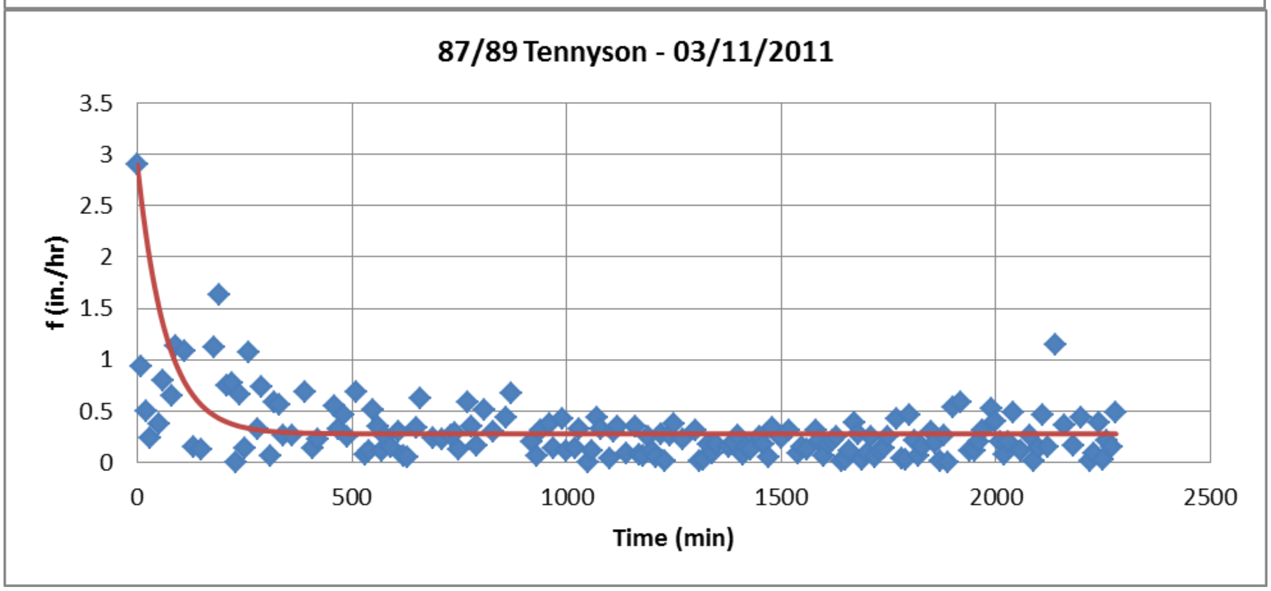
1
0.8
0.6
0.4
0.2
0
0
2000
4000
6000
8000
10000
12000


87/89 Tennyson - 12/13/2010
f (in./hr)
0
0.2
0.4
0.6
0.8
1
1.2
0
500
1000
1500
2000
2500
3000
3500
4000
Time (min)

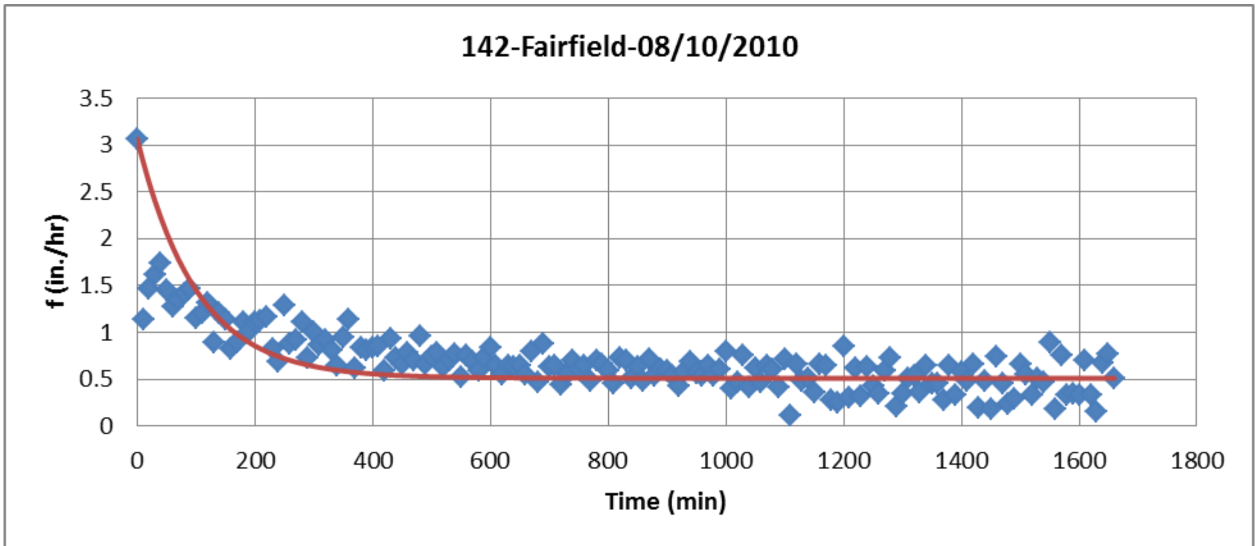

87/89 Tennyson - 02/25/2011
f (in./hr)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
500
1000
1500
2000
2500
3000
3500
4000
Time (min)

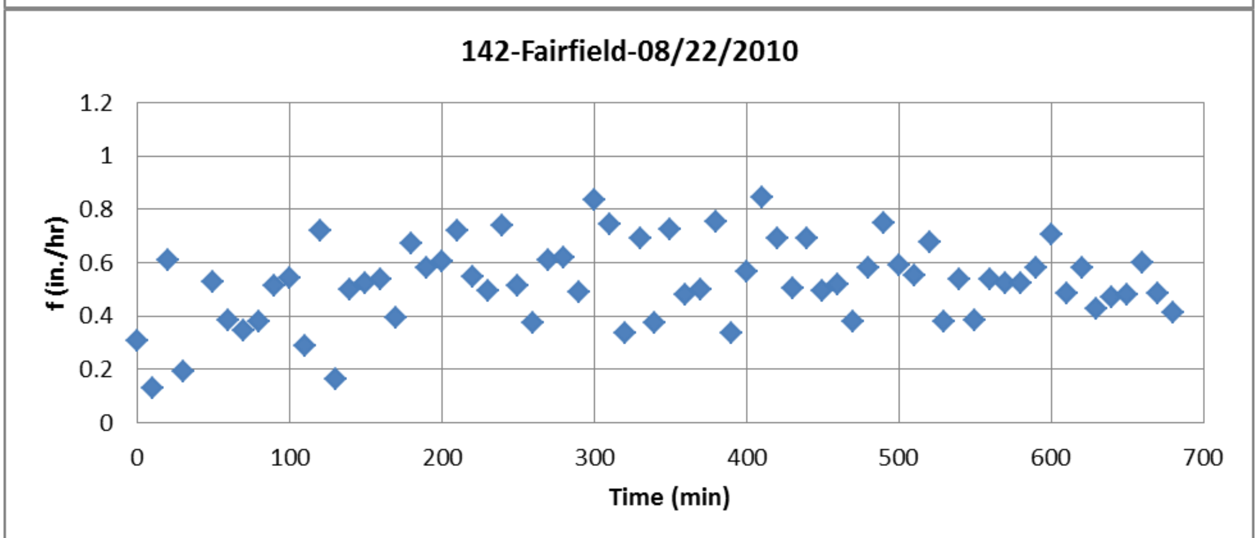

87/89 Tennyson - 02/28/2011
f (in./hr)
0
0.2
0.4
0.6
0.8
1
1.2
0
500
1000
1500
2000
2500
3000
3500
Time (min)

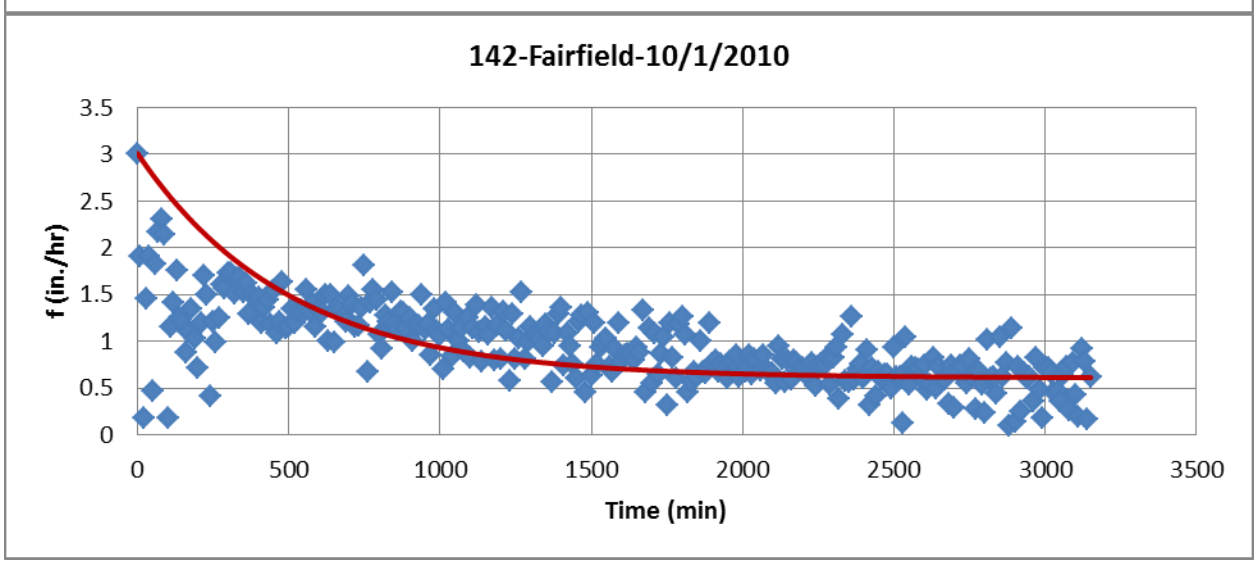


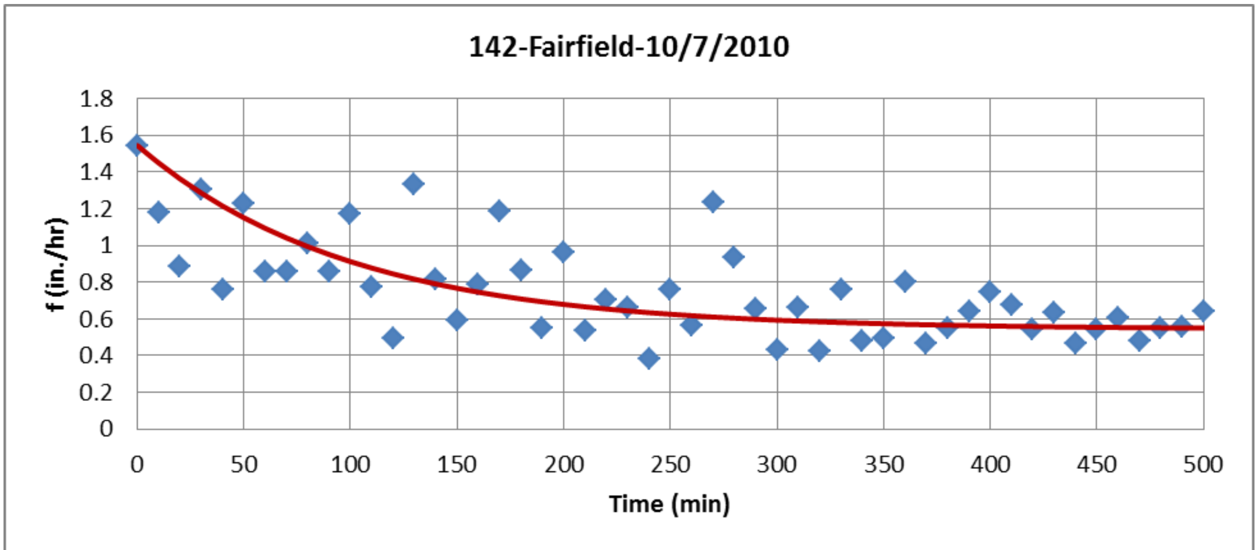


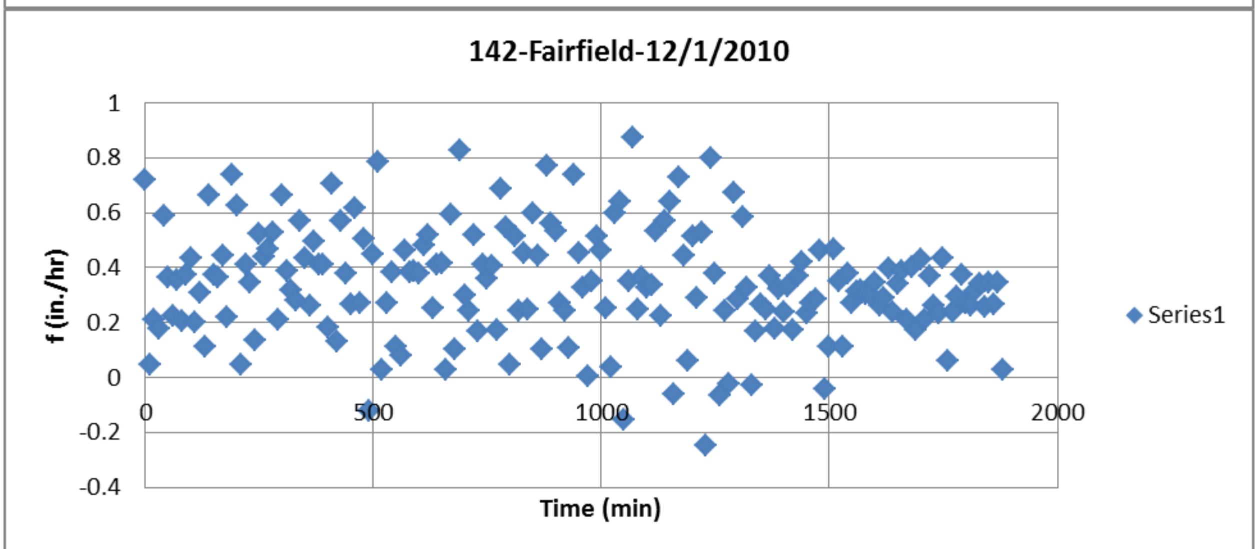
142
Fairfield

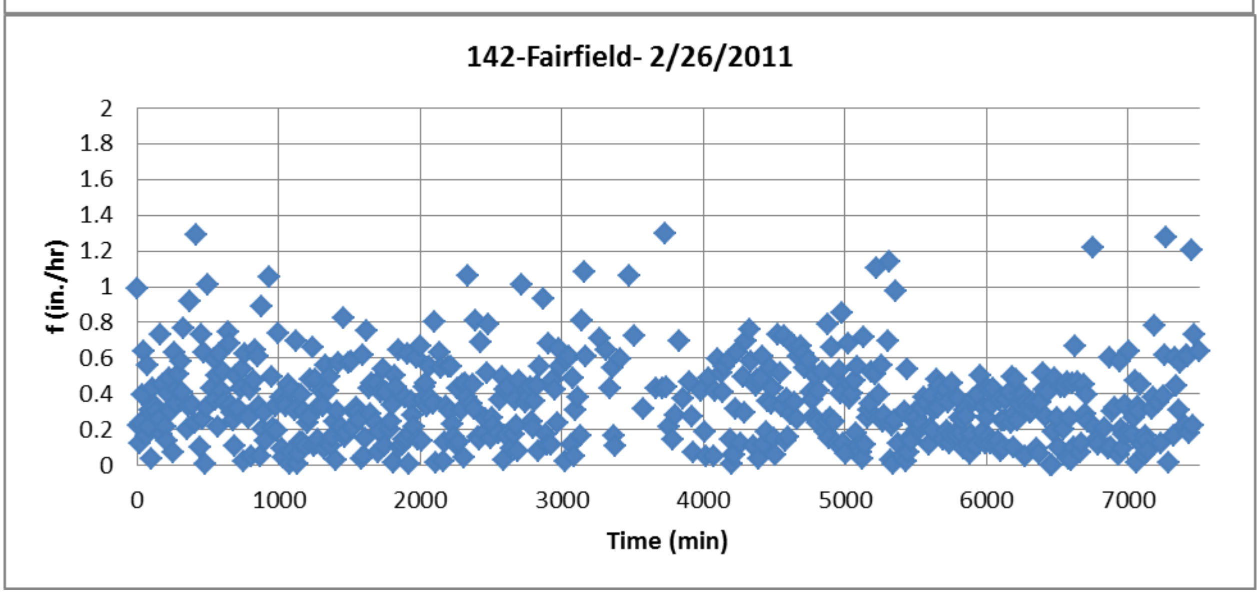

142-Fairfield-08/10/2010
f (in./hr)
3.5
3
2.5
2
1.5
1
0.5
0
0
200
400
600
800
1000
1200
1400
1600
1800
Time (min)

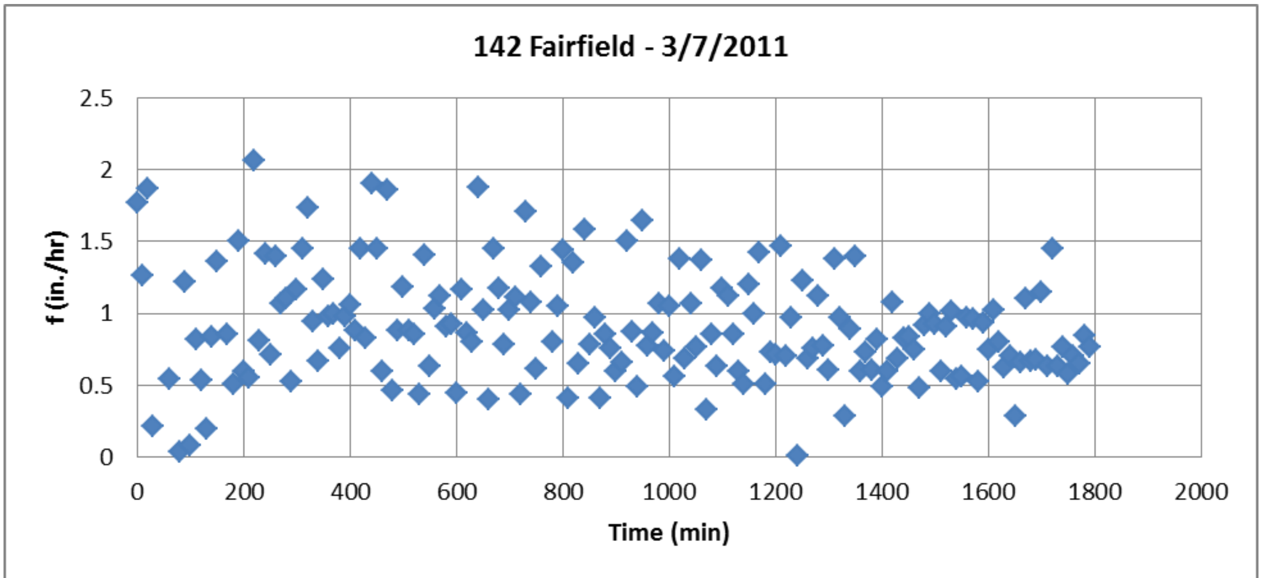

142-Fairfield-08/22/2010
f (in./hr)
1.2
1
0.8
0.6
0.4
0.2
0
0
100
200
300
400
500
600
700
Time (min)

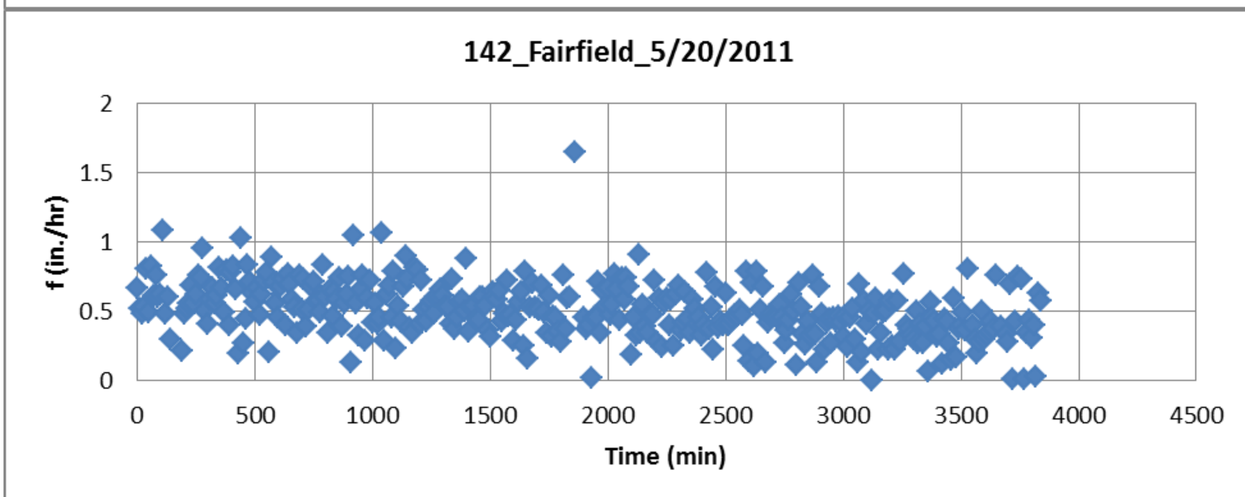

142-Fairfield-10/1/2010
f (in./hr)
3.5
3
2.5
2
1.5
1
0.5
0
0
500
1000
1500
2000
2500
3000
3500
Time (min)

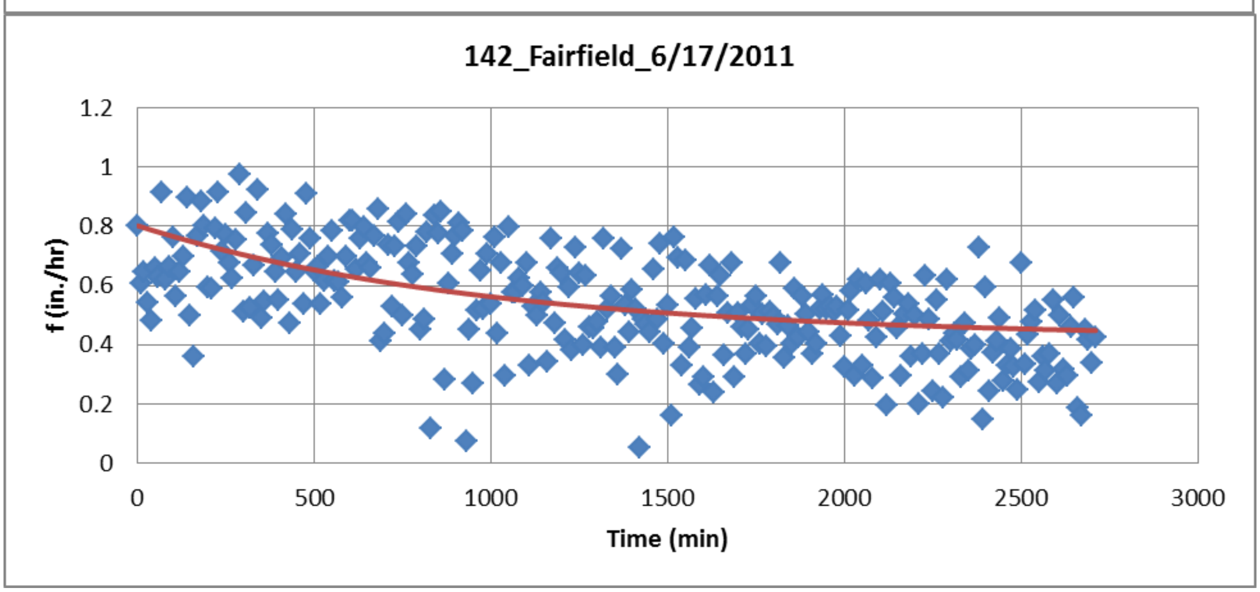

142-Fairfield-10/7/2010
f (in./hr)
Time (min)


142-Fairfield-12/1/2010
f (in./hr)
Time (min)
Series1

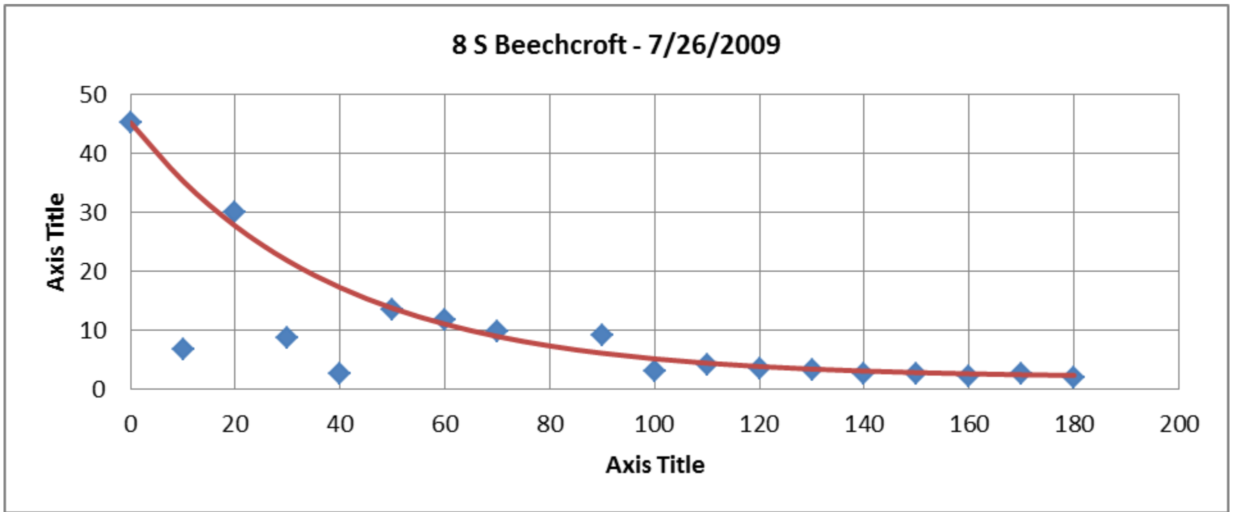

142-Fairfield- 2/26/2011
f (in./hr)
Time (min)

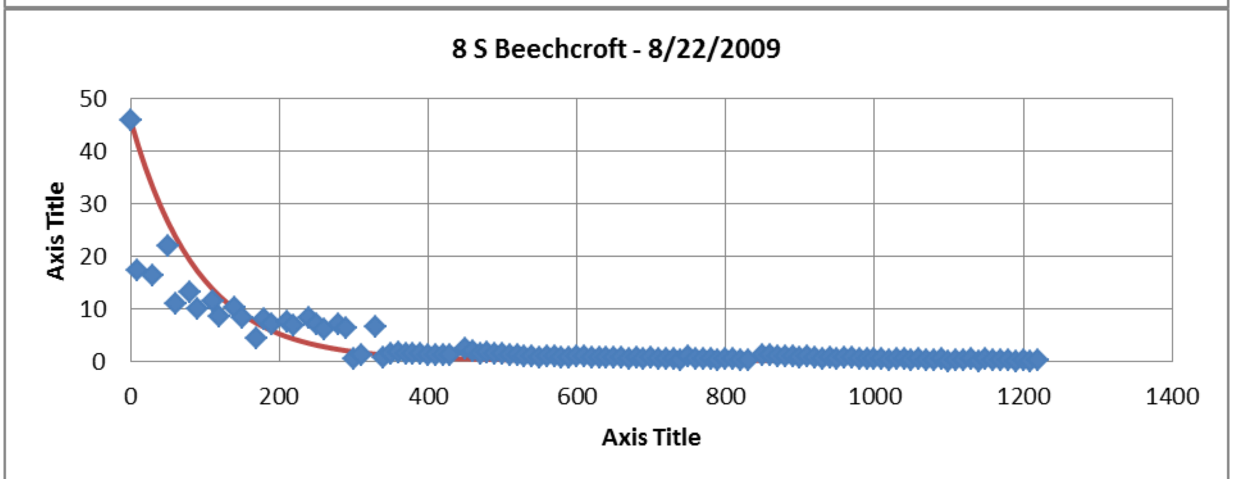

142 Fairfield - 3/7/2011
f (in./hr)
Time (min)

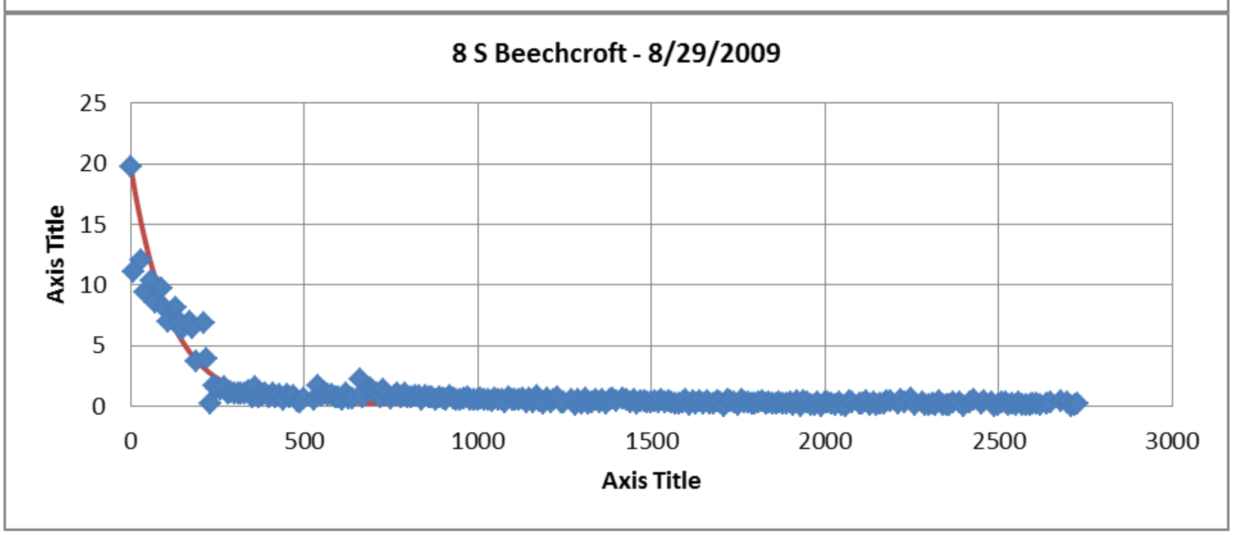

142_Fairfield_5/20/2011
f (in./hr)
Time (min)

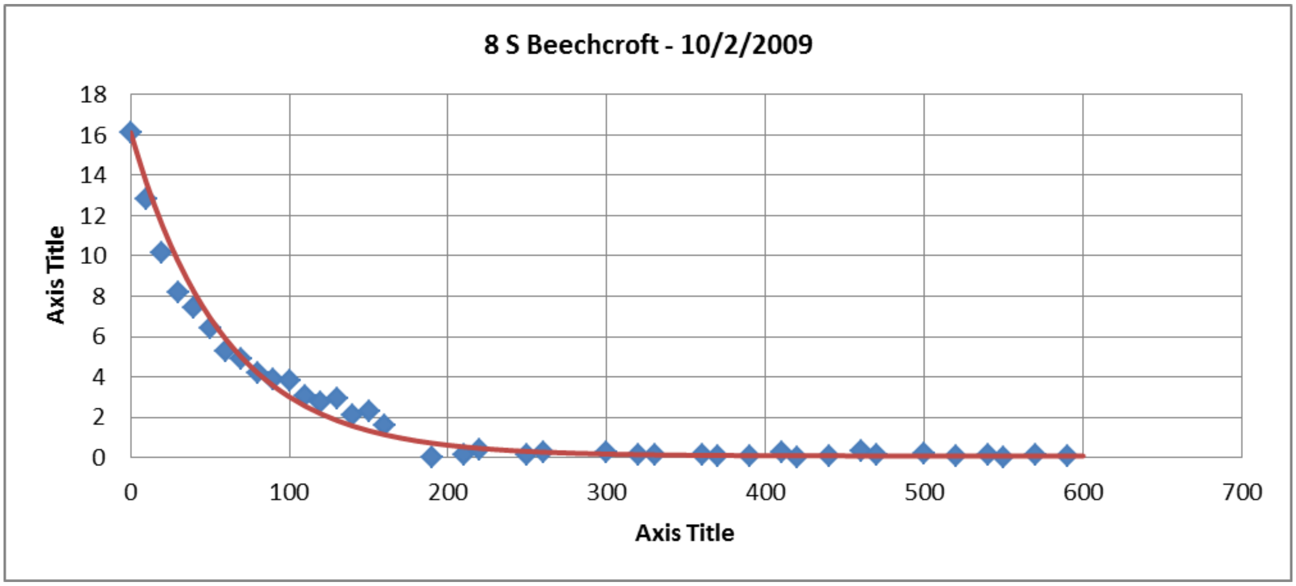

142_Fairfield_6/17/2011
f (in./hr)
Time (min)

8 South Beechcroft

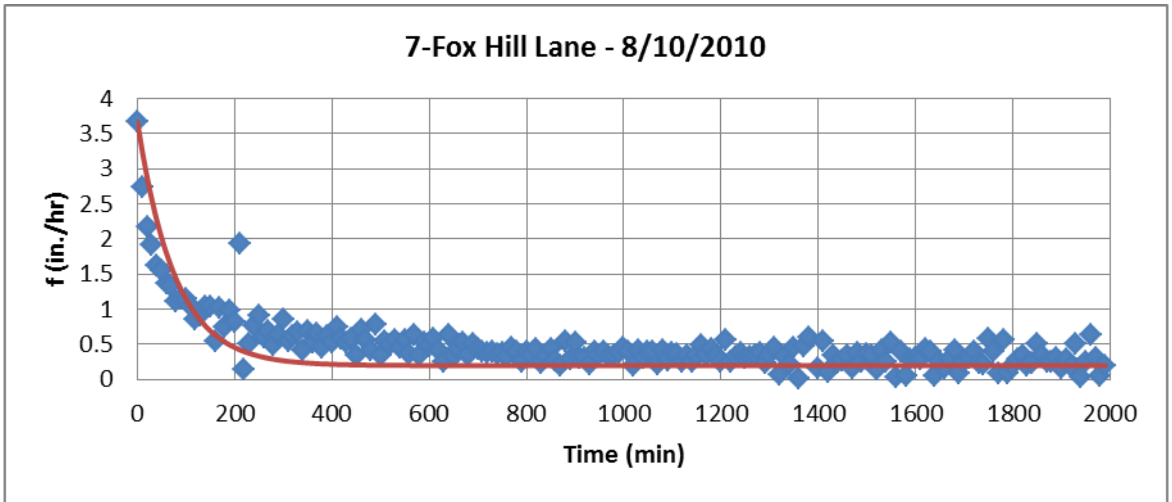


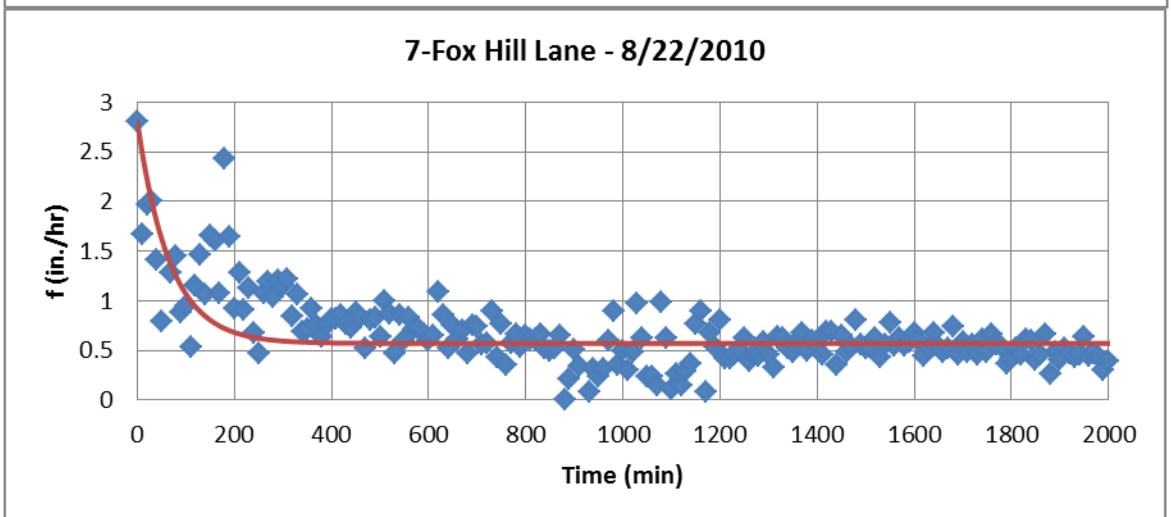


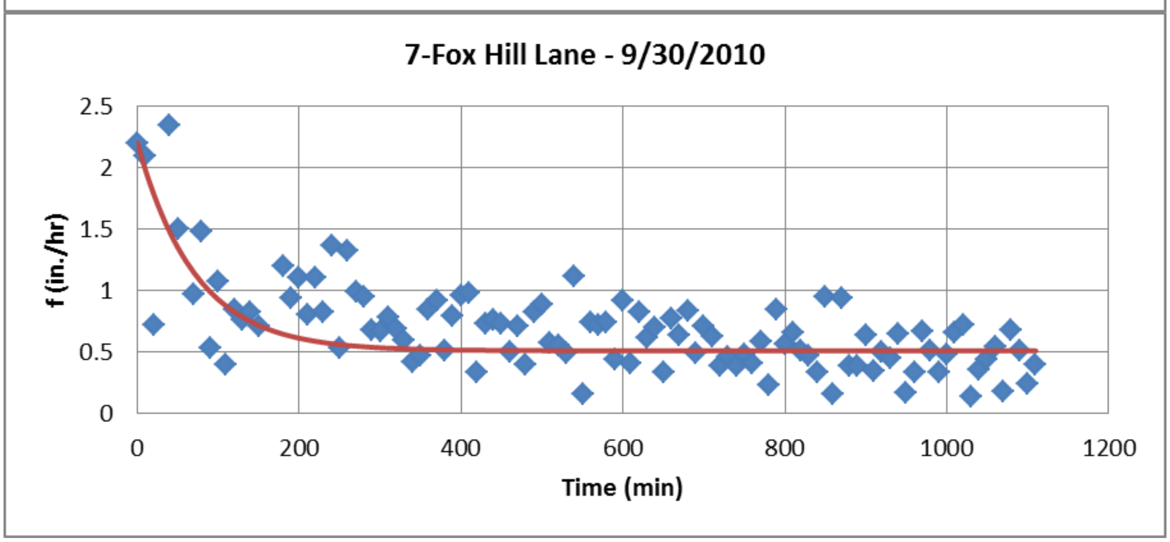


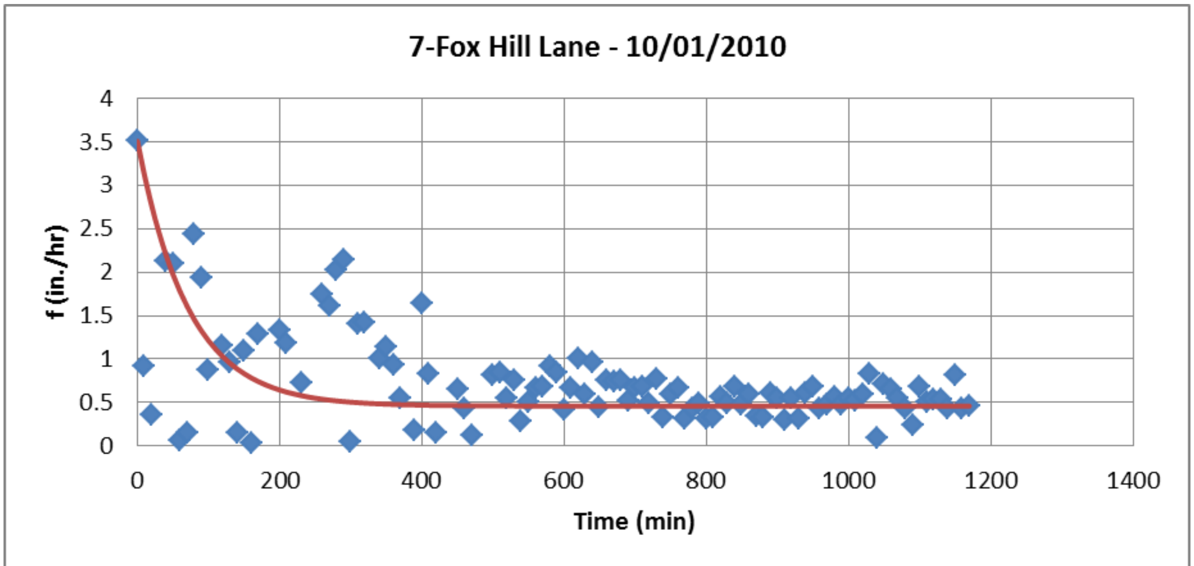

8 S Beechcroft - 10/2/2009
Axis Title
18
16
14
12
10
8
6
4
2
0
0
100
200
300
400
500
600
700
Axis Title

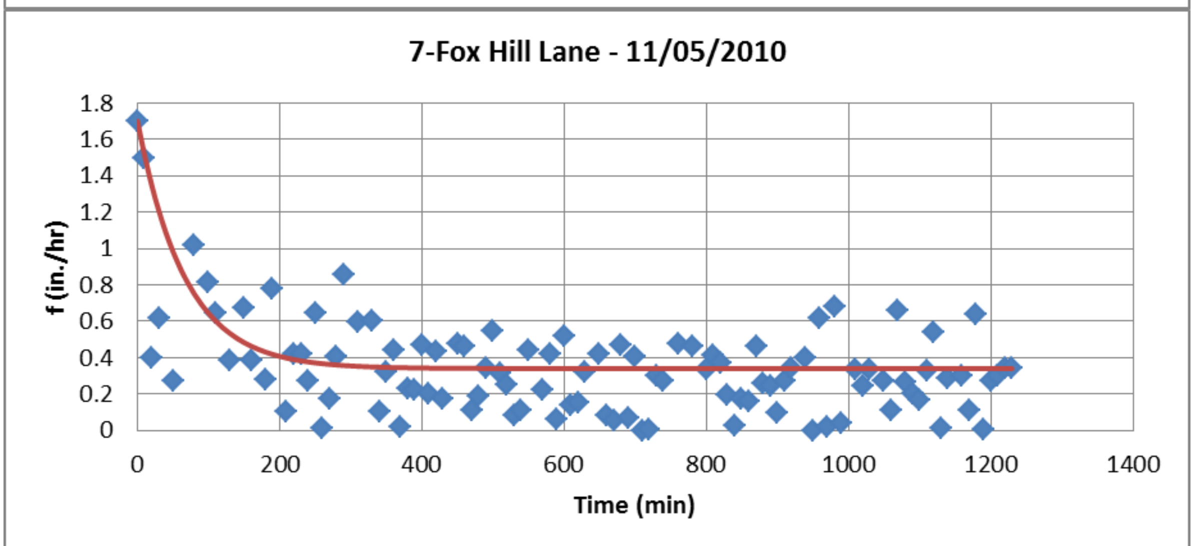
7 Fox Hill Lane 8-10-2010

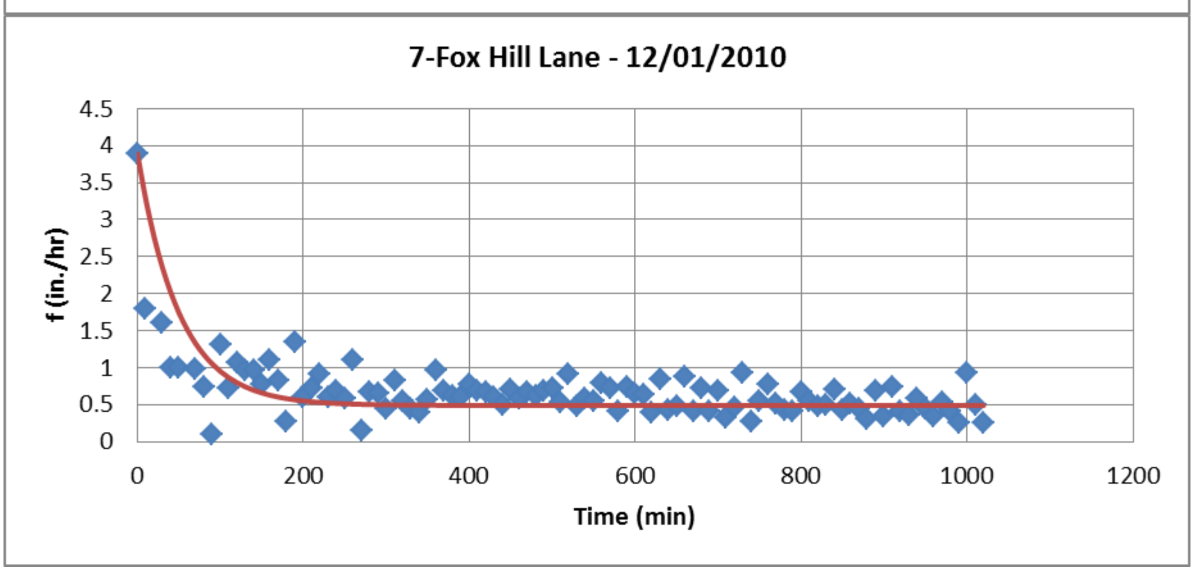


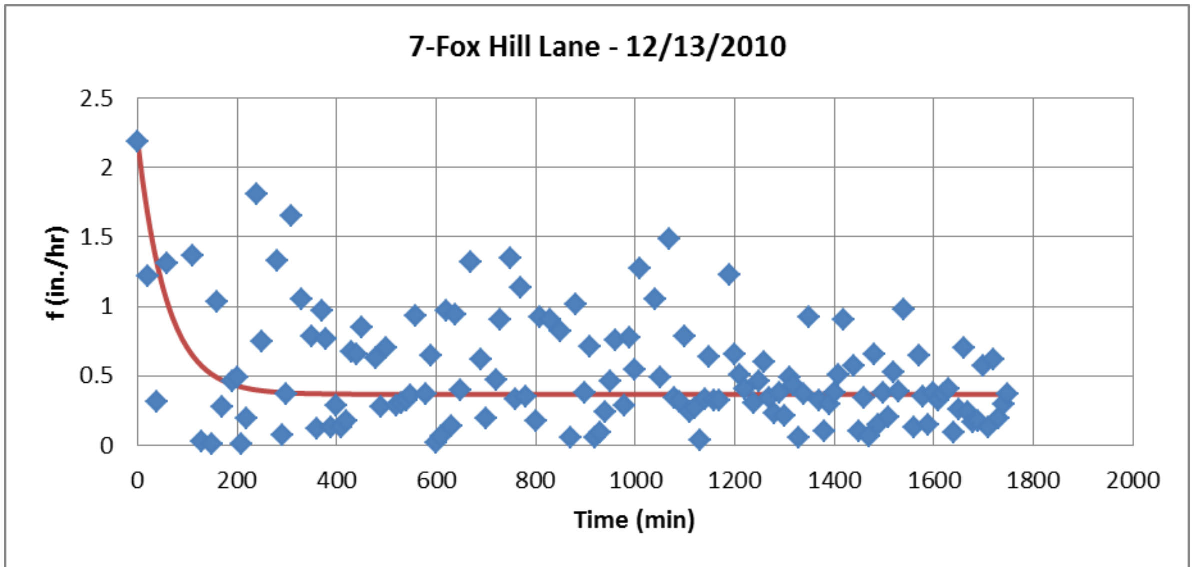


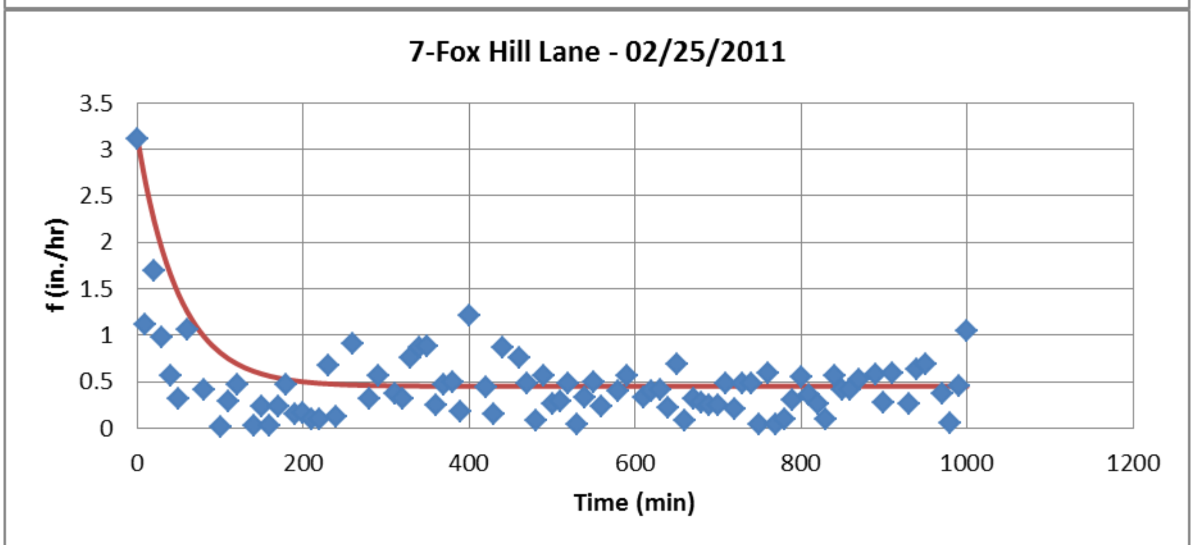


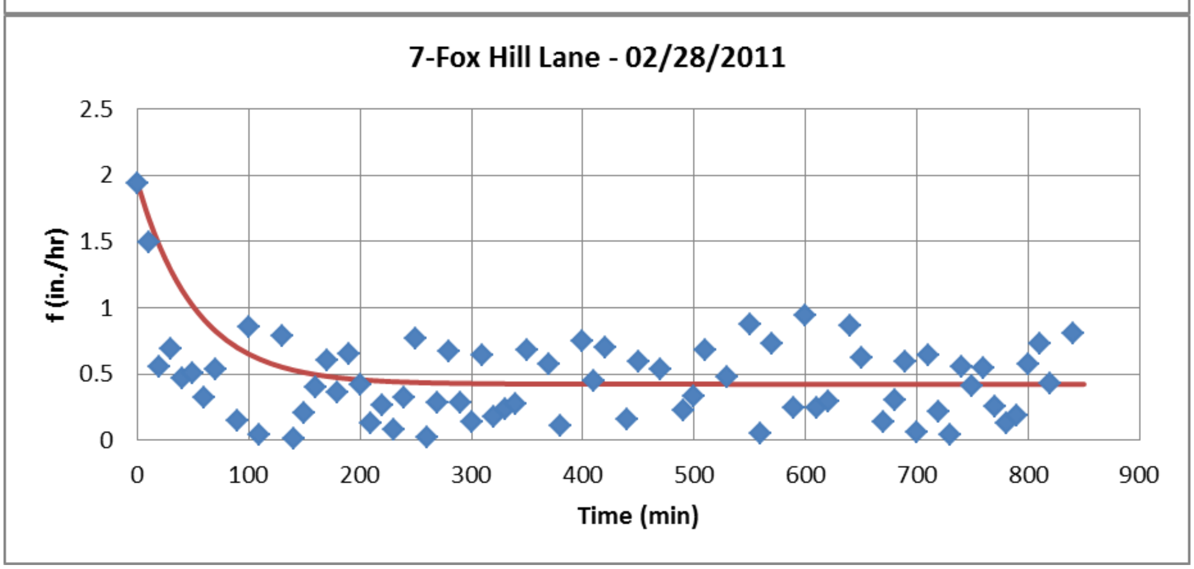

7-Fox Hill Lane - 10/01/2010
f (in./hr)
Time (min)
4
3.5
3
2.5
2
1.5
1
0.5
0
0
200
400
600
800
1000
1200
1400

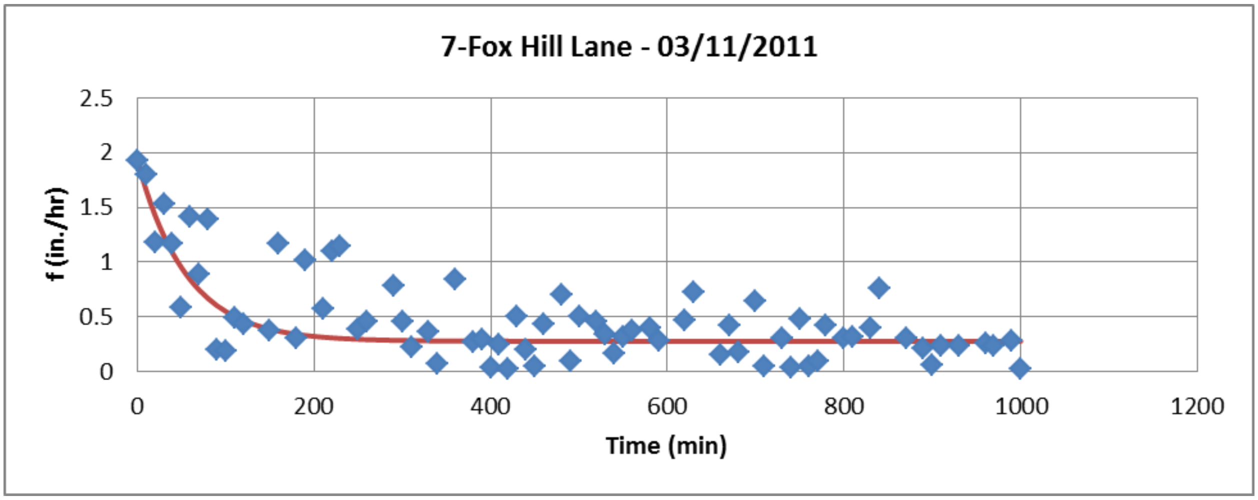

7-Fox Hill Lane - 11/05/2010
f (in./hr)
Time (min)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
200
400
600
800
1000
1200
1400


7-Fox Hill Lane - 12/01/2010
f (in./hr)
Time (min)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
200
400
600
800
1000
1200

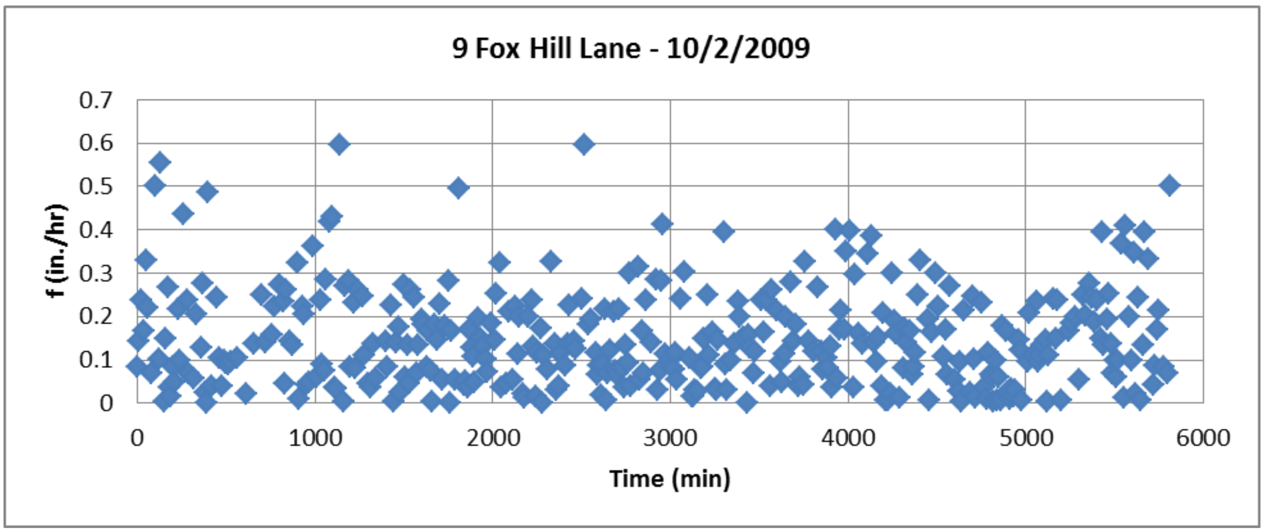

7-Fox Hill Lane - 12/13/2010
f (in./hr)
Time (min)


7-Fox Hill Lane - 02/25/2011
f (in./hr)
Time (min)

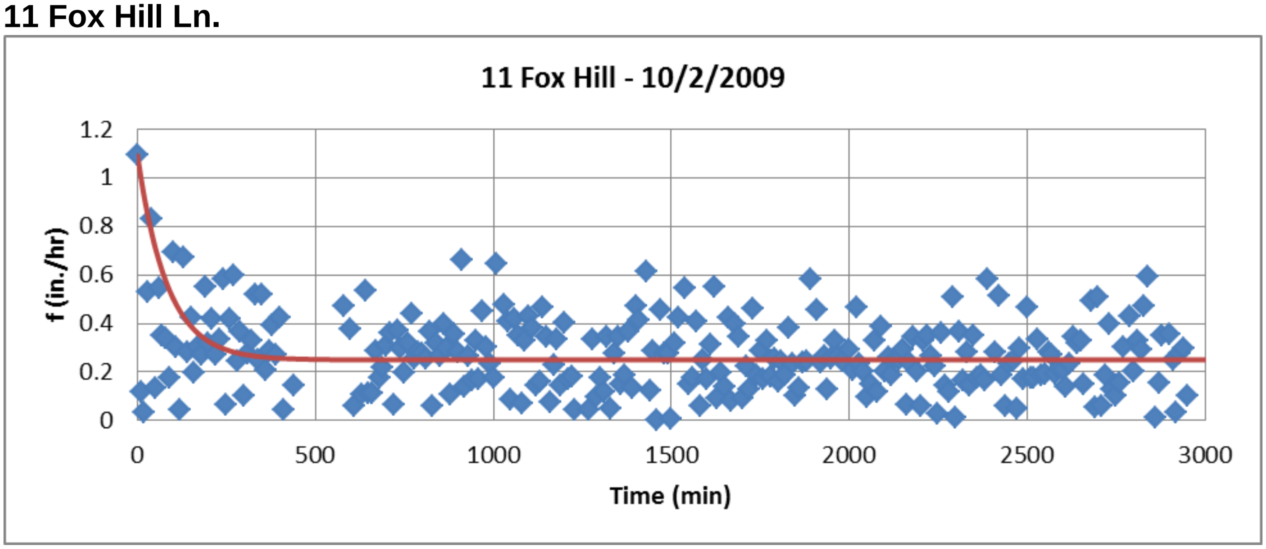

7-Fox Hill Lane - 02/28/2011
f (in./hr)
Time (min)



9 Fox Hill Ln.



11 Fox Hill Ln.



Appendix D. Dry Well Water Quality Analyses

Probability Plots

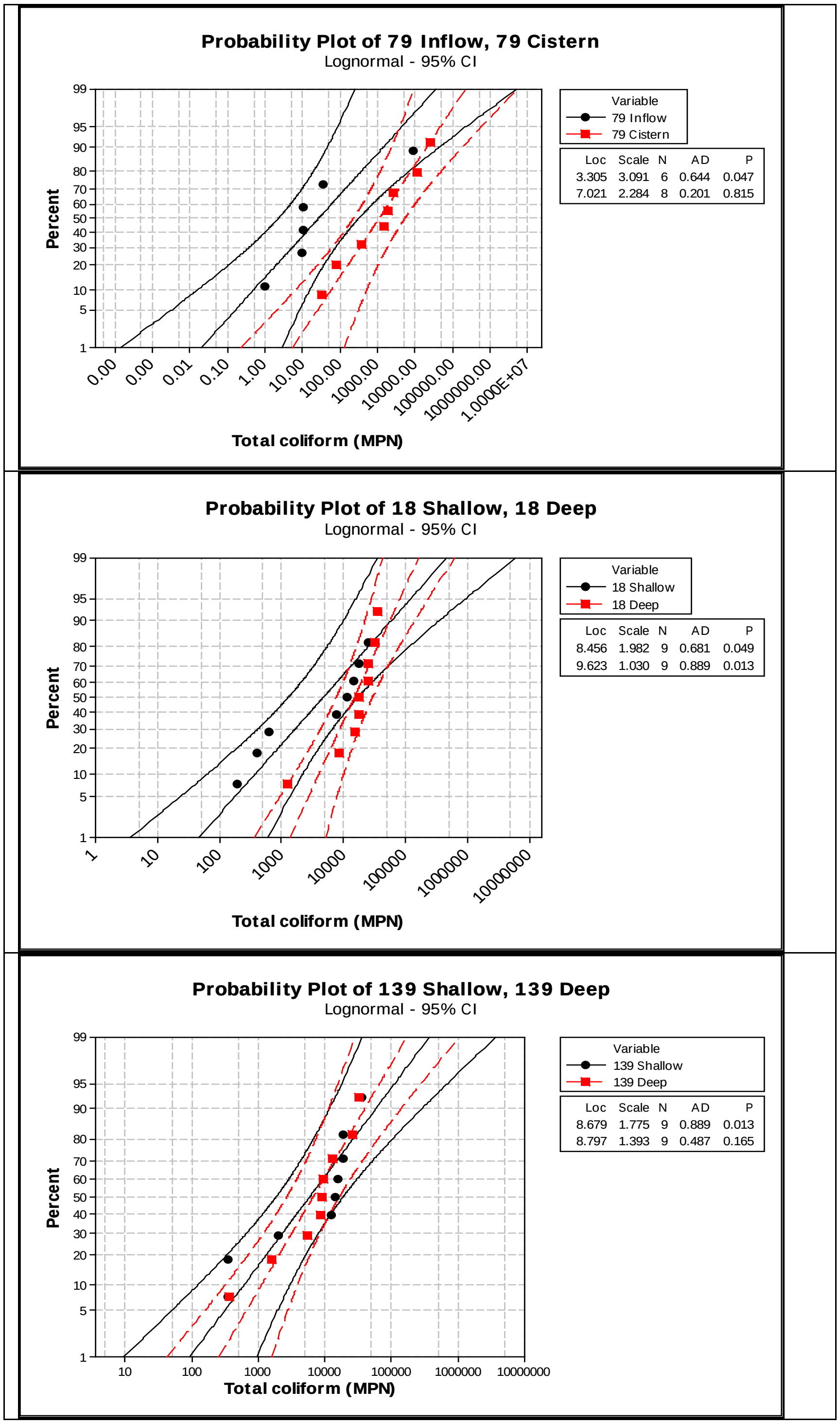
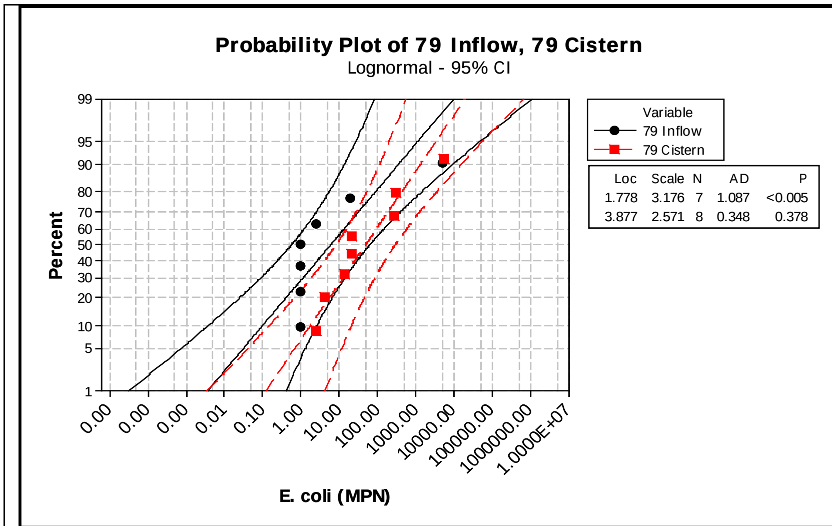
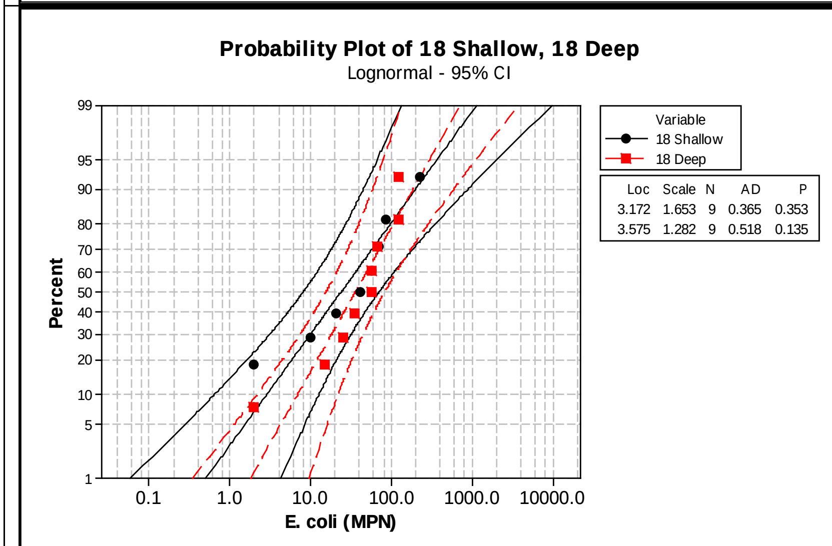


Figure D-1 Probability plots for total coliform in different locations





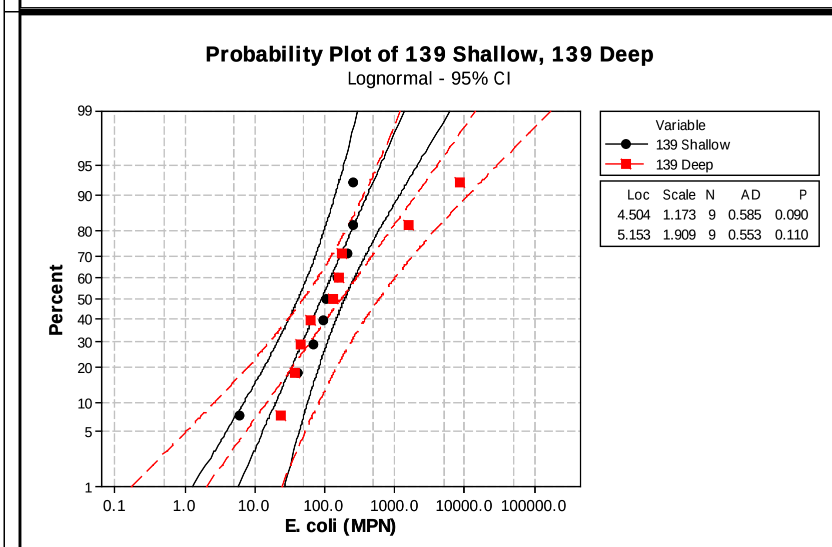


Figure D-2 Probability plots for *E.coli* in different locations

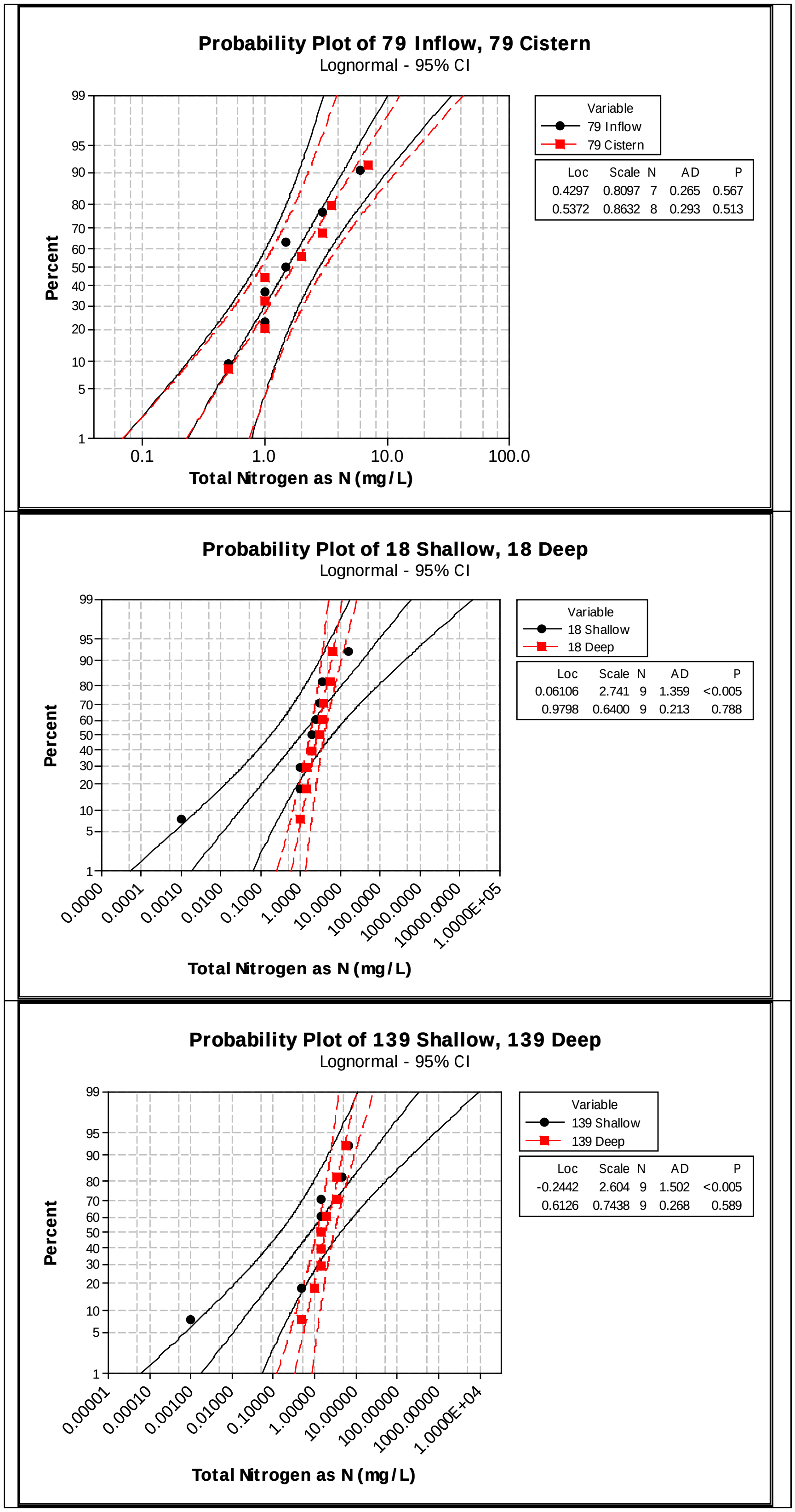


Figure D-3 Probability plots for total nitrogen in different location

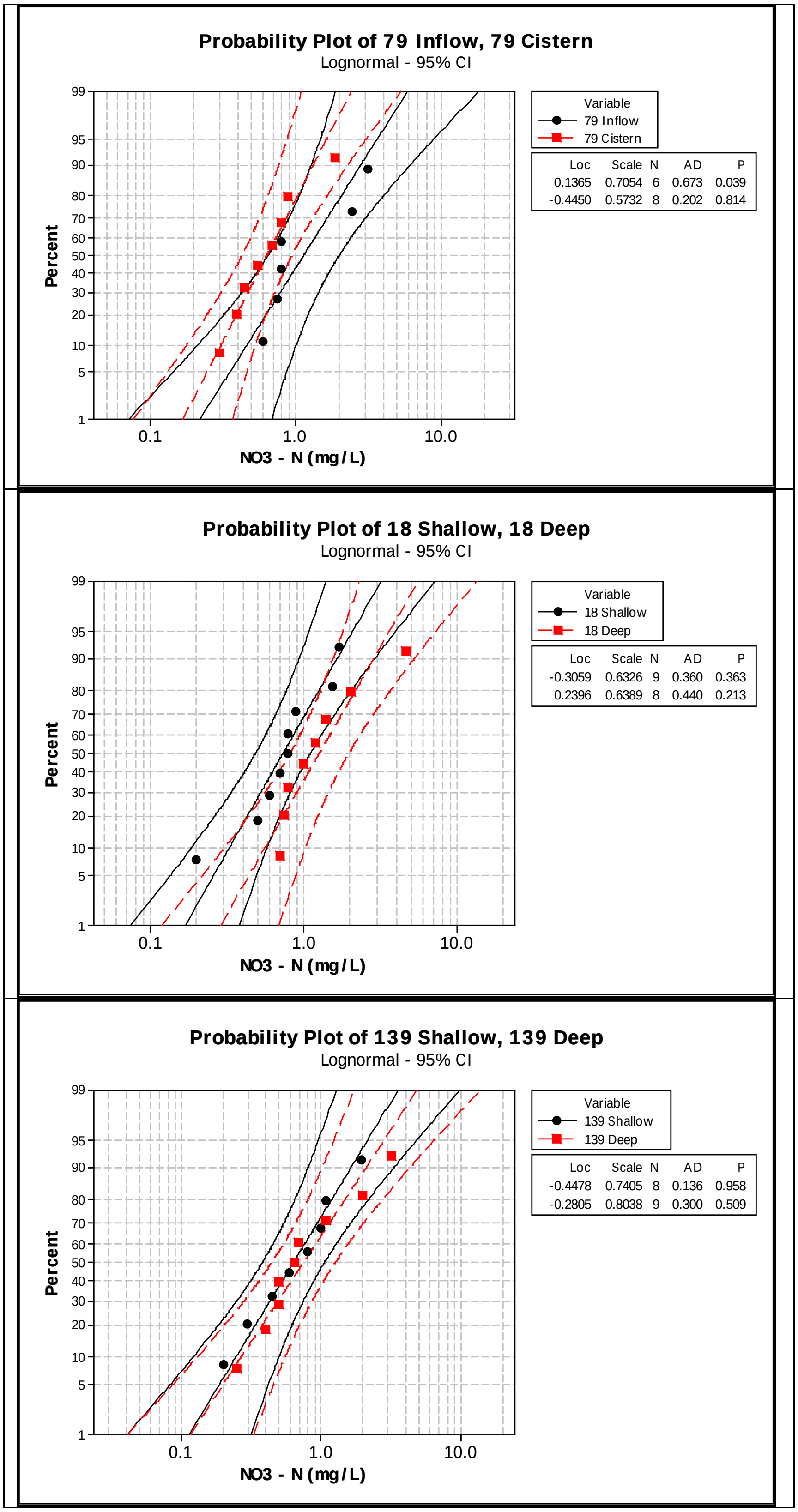


Figure D-4 Probability plots for NO_3 in different location

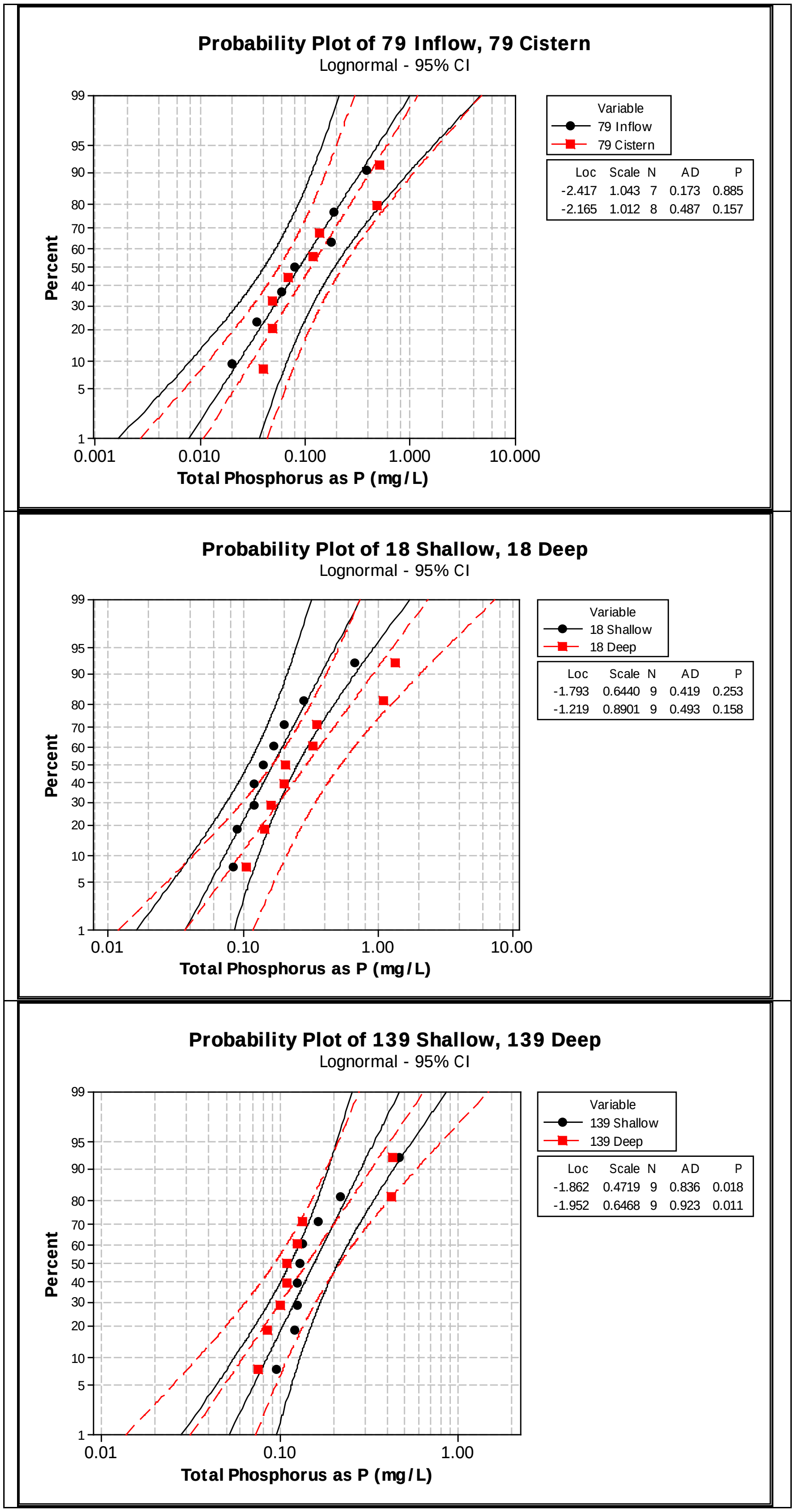


Figure D-5 Probability plots for total phosphorus in different location

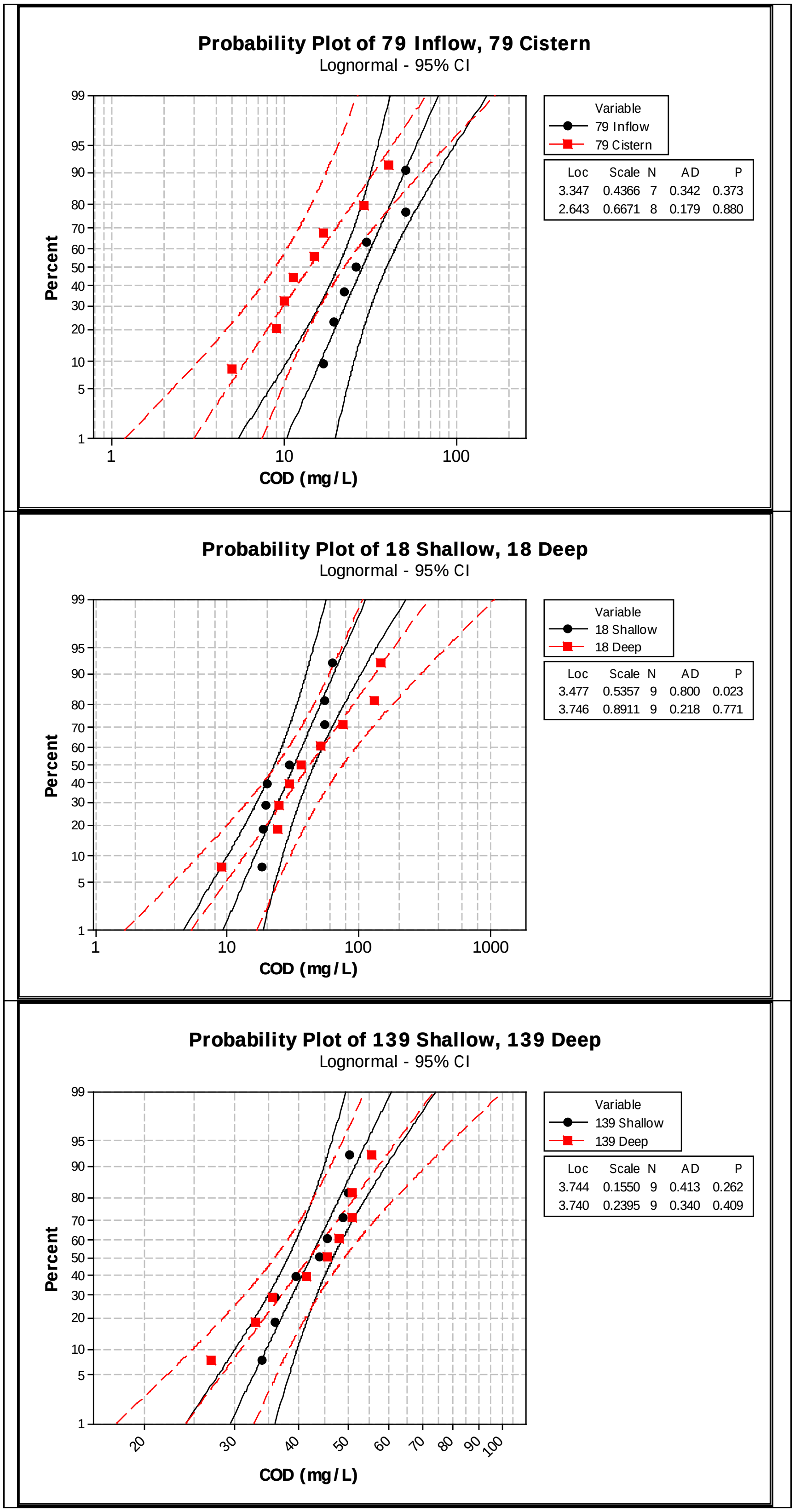


Figure D-6 Probability plots for COD in different location

Mann-Whitney Test Results

Mann-Whitney Test Results for Total Coliforms

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern
N Median
79 Inflow 6 11.0
79 Cistern 8 1753.4
Point estimate for ETA1-ETA2 is -1472.3
95.5 Percent CI for ETA1-ETA2 is (-11660.6,-23.3)
W = 28.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0332
The test is significant at 0.0330 (adjusted for ties)
Since the p-value is less than the chosen a level of 0.05, you reject H0. Therefore,there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep
N Median
135 Shallow 10 6892
135 Deep 10 13423
Point estimate for ETA1-ETA2 is -2965
95.5 Percent CI for ETA1-ETA2 is (-16238,7259)
W = 93.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.4057
The test is significant at 0.4048 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep
N Median
18 Shallow 9 11672
18 Deep 9 18539
Point estimate for ETA1-ETA2 is -7848
95.8 Percent CI for ETA1-ETA2 is (-18340,6242)
W = 69.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.1577
The test is significant at 0.1564 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep
N Median
139 Shallow 9 14207
139 Deep 9 8911
Point estimate for ETA1-ETA2 is 2977

95.8 Percent CI for ETA1-ETA2 is (-9022,11988)
W = 90.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.7239
The test is significant at 0.7237 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test Results for *E. coli*

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern
N Median
79 Inflow 7 1.0
79 Cistern 8 21.8
Point estimate for ETA1-ETA2 is -19.3
95.7 Percent CI for ETA1-ETA2 is (-285.5,-0.0)
W = 38.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0491

The test is significant at 0.0469 (adjusted for ties)
Since the p-value is less than the chosen a level of 0.05, you reject H0. Therefore,there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep
N Median
135 Shallow 10 347.2
135 Deep 10 313.9
Point estimate for ETA1-ETA2 is 79.5
95.5 Percent CI for ETA1-ETA2 is (-261.5,2589.6)
W = 112.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.5967

The test is significant at 0.5966 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep
N Median
18 Shallow 9 40.7
18 Deep 9 57.0
Point estimate for ETA1-ETA2 is -11.2
95.8 Percent CI for ETA1-ETA2 is (-55.0,41.8)
W = 80.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.6911

The test is significant at 0.6903 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep
N Median
139 Shallow 9 105.9
139 Deep 9 136.0
Point estimate for ETA1-ETA2 is -3.5
95.8 Percent CI for ETA1-ETA2 is (-1340.7,91.7)
W = 85.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 1.0000

The test is significant at 1.0000 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test Results for Total Nitrogen as N

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern
N Median
79 Inflow 7 1.500
79 Cistern 8 1.500
Point estimate for ETA1-ETA2 is 0.000
95.7 Percent CI for ETA1-ETA2 is (-1.999,2.001)
W = 54.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.8622

The test is significant at 0.8593 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep
N Median
135 Shallow 10 1.500
135 Deep 10 1.500
Point estimate for ETA1-ETA2 is 0.000
95.5 Percent CI for ETA1-ETA2 is (-1.000,0.500)
W = 96.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.5205
The test is significant at 0.5033 (adjusted for ties)

Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep
N Median
18 Shallow 9 2.000
18 Deep 9 3.000
Point estimate for ETA1-ETA2 is -0.500
95.8 Percent CI for ETA1-ETA2 is (-2.999,1.001)
W = 76.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.4268
The test is significant at 0.4241 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep
N Median
139 Shallow 9 1.500
139 Deep 9 1.500
Point estimate for ETA1-ETA2 is 0.000
95.8 Percent CI for ETA1-ETA2 is (-1.999,0.999)
W = 80.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.6588
The test is significant at 0.6437 (adjusted for ties)Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test Results for NO_3 plus NO_2 as N

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern
N Median
79 Inflow 6 0.800
79 Cistern 8 0.625
Point estimate for ETA1-ETA2 is 0.325
95.5 Percent CI for ETA1-ETA2 is (-0.100,2.100)
W = 57.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.1376
The test is significant at 0.1359 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep
N Median

135 Shallow 10 0.8000
135 Deep 10 0.5500
Point estimate for ETA1-ETA2 is 0.2000
95.5 Percent CI for ETA1-ETA2 is (-0.1998,0.5999)
W = 121.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.2413
The test is significant at 0.2394 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep
N Median
18 Shallow 9 0.800
18 Deep 8 1.100
Point estimate for ETA1-ETA2 is -0.375
95.1 Percent CI for ETA1-ETA2 is (-1.200,0.100)
W = 65.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.1489
The test is significant at 0.1477 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep
N Median
139 Shallow 8 0.700
139 Deep 9 0.650
Point estimate for ETA1-ETA2 is -0.050
95.1 Percent CI for ETA1-ETA2 is (-0.901,0.500)
W = 68.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.7728
The test is significant at 0.7726 (adjusted for ties)Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test Results for Total Phosphorus as P

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern
N Median
79 Inflow 7 0.0800
79 Cistern 8 0.0950
Point estimate for ETA1-ETA2 is -0.0150
95.7 Percent CI for ETA1-ETA2 is (-0.3100,0.1300)

W = 53.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.7723
The test is significant at 0.7721 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep
N Median
135 Shallow 10 0.0975
135 Deep   10 0.0825
Point estimate for ETA1-ETA2 is 0.0025
95.5 Percent CI for ETA1-ETA2 is (-0.0400,0.1000)
W = 106.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.9397

The test is significant at 0.9396 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep
N Median
18 Shallow 9 0.1400
18 Deep   9 0.2050
Point estimate for ETA1-ETA2 is -0.0750
95.8 Percent CI for ETA1-ETA2 is (-0.4299,0.0250)
W = 66.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.1023

The test is significant at 0.1020 (adjusted for ties)
Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep
N Median
139 Shallow 9 0.1300
139 Deep   9 0.1100
Point estimate for ETA1-ETA2 is 0.0200
95.8 Percent CI for ETA1-ETA2 is (-0.0300,0.0599)
W = 98.5

Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.2697

The test is significant at 0.2682 (adjusted for ties)Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test Results for COD

Mann-Whitney Test and CI: 79 Inflow, 79 Cistern

N Median
79 Inflow   7   26.00
79 Cistern 8   13.15
Point estimate for ETA1-ETA2 is 12.25
95.7 Percent CI for ETA1-ETA2 is (1.01,33.99)
W = 74.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.0372

The test is significant at 0.0371 (adjusted for ties)

Since the p-value is less than the chosen a level of 0.05, you reject H0. Therefore,there is a difference between the population medians.

Mann-Whitney Test and CI: 135 Shallow, 135 Deep

N Median
135 Shallow 10   38.75
135 Deep   10   28.00
Point estimate for ETA1-ETA2 is 8.75
95.5 Percent CI for ETA1-ETA2 is (-3.50,19.01)

W = 125.0
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.1405

Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 18 Shallow, 18 Deep

N Median
18 Shallow 9   29.50
18 Deep   9   36.50
Point estimate for ETA1-ETA2 is -7.00

95.8 Percent CI for ETA1-ETA2 is (-68.52,20.51)

W = 75.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.4015

The test is significant at 0.4011 (adjusted for ties)

Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Mann-Whitney Test and CI: 139 Shallow, 139 Deep

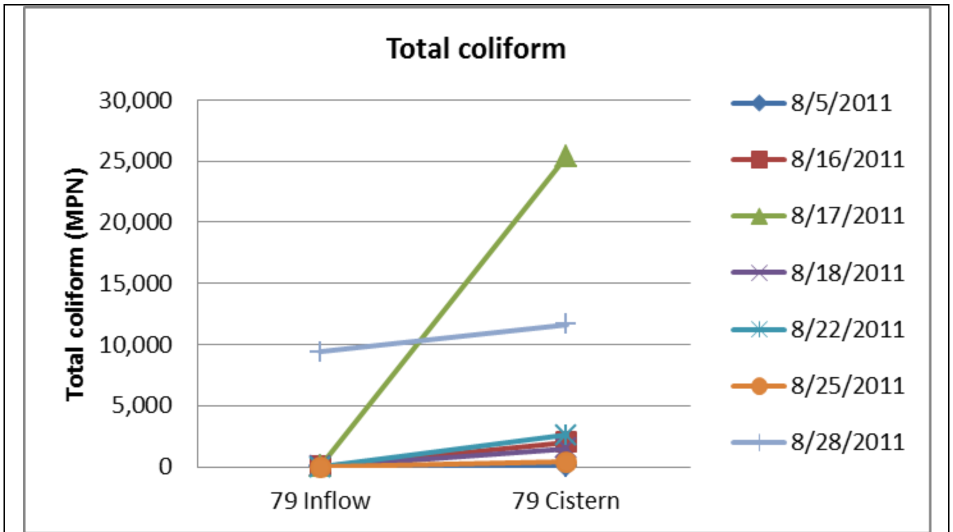
N Median
139 Shallow 9   44.00
139 Deep   9   45.50
Point estimate for ETA1-ETA2 is -1.00
95.8 Percent CI for ETA1-ETA2 is (-10.00,9.00)

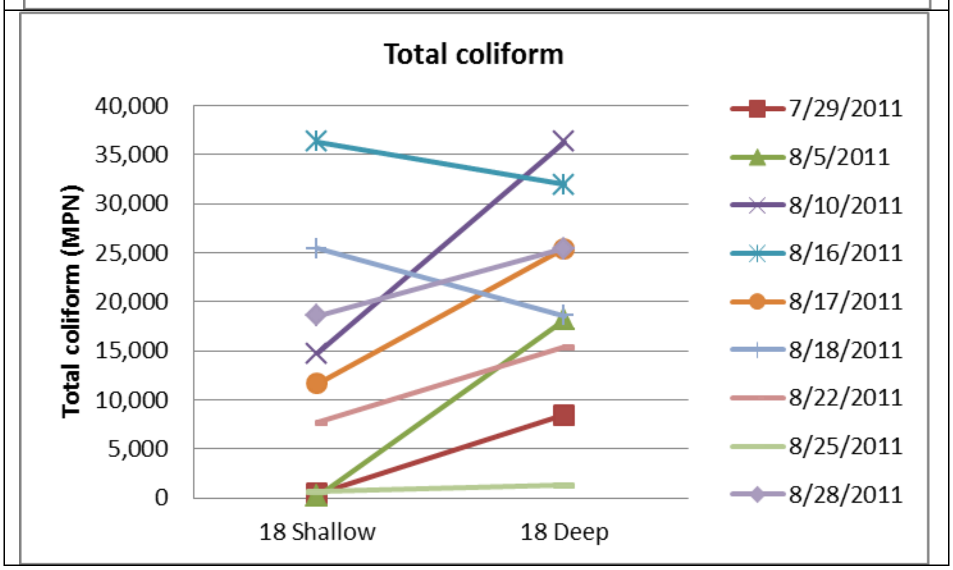
W = 82.5
Test of ETA1 = ETA2 vs ETA1 not = ETA2 is significant at 0.8253

The test is significant at 0.8250 (adjusted for ties)

Since the p-value is not less than the chosen a level of 0.05, you conclude that there is insufficient evidence to reject H0. Therefore, the data does not support the hypothesis that there is a difference between the population medians.

Paired Line Plots





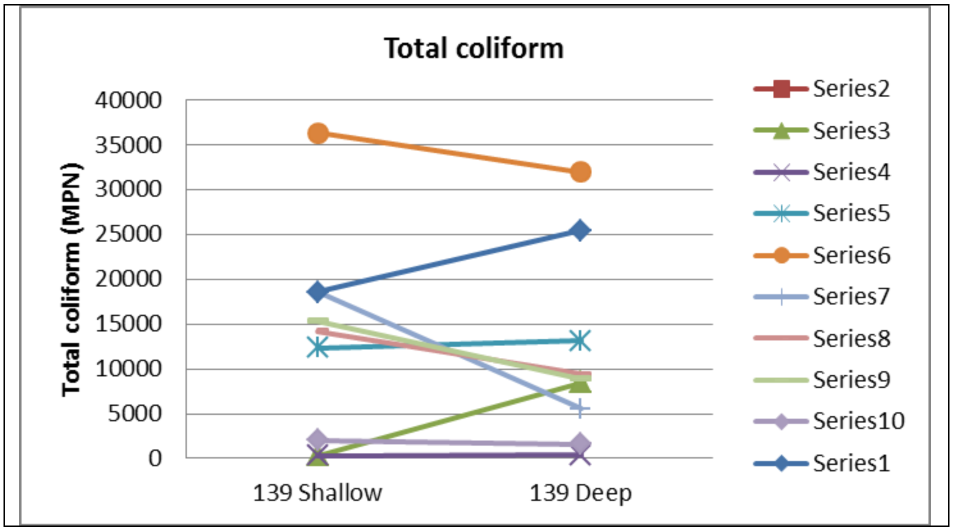
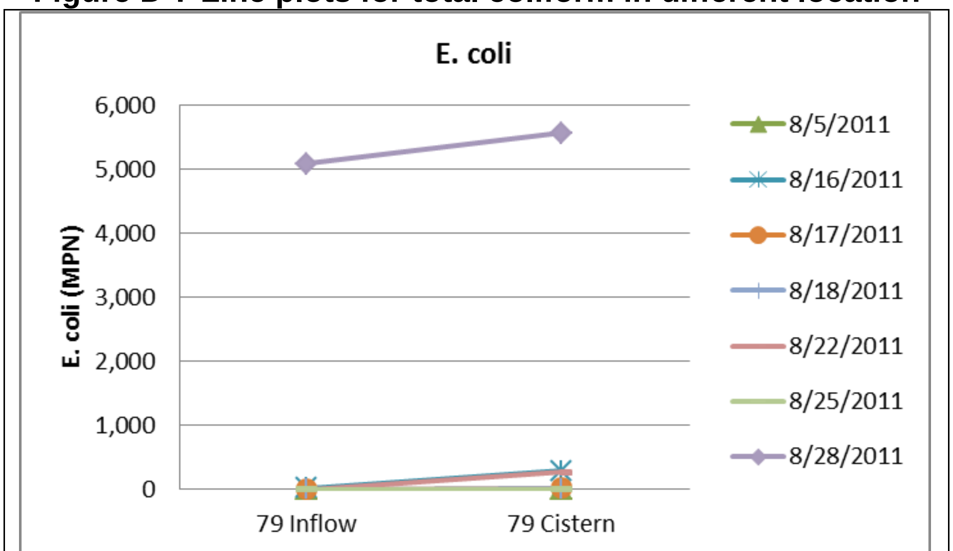
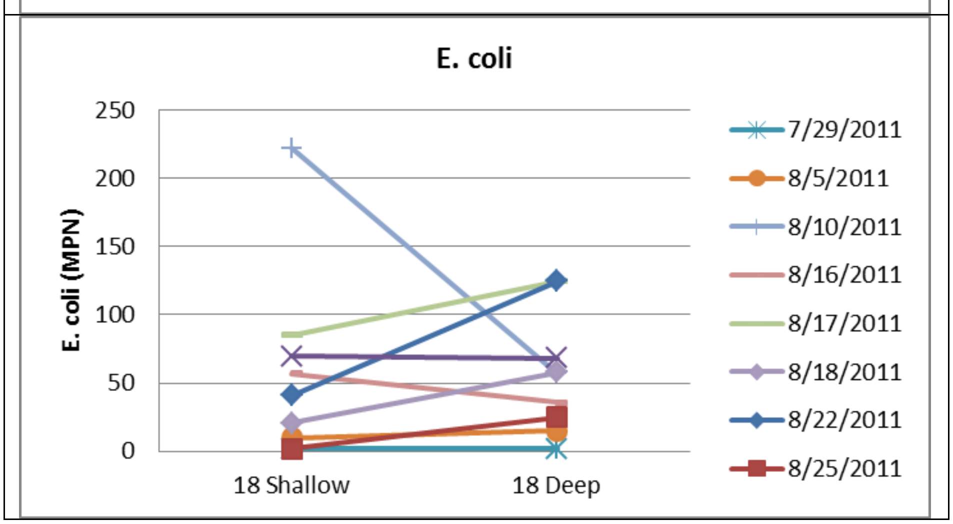


Figure D-7 Line plots for total coliform in different location





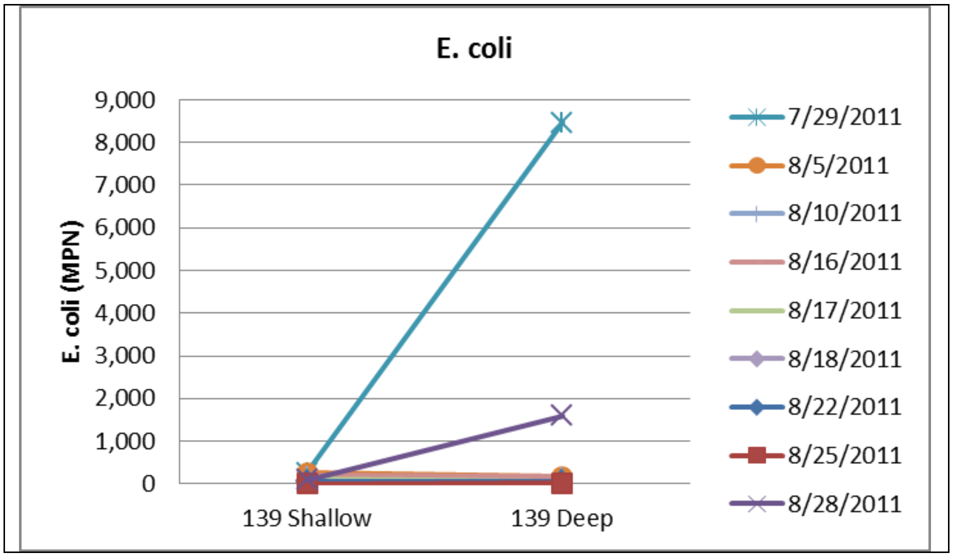
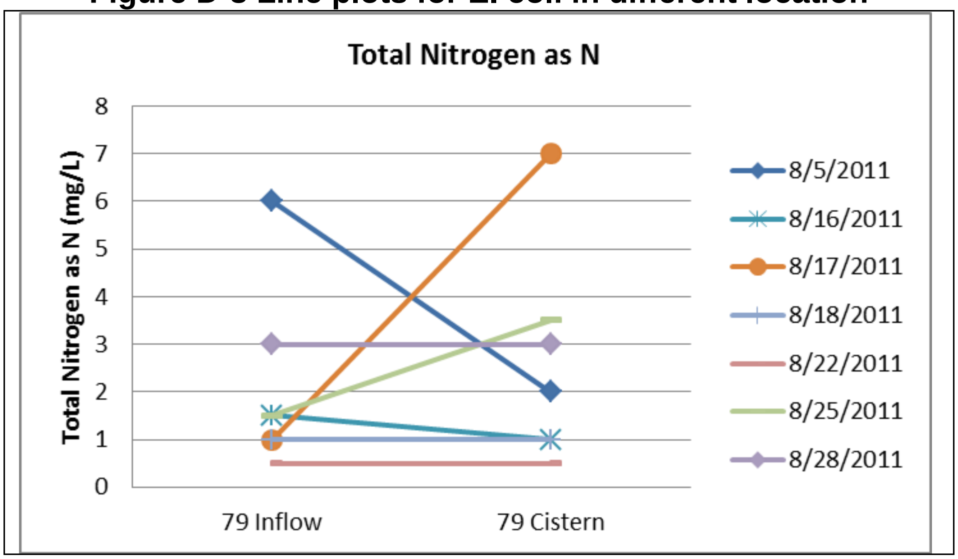


Figure D-8 Line plots for E. coli in different location



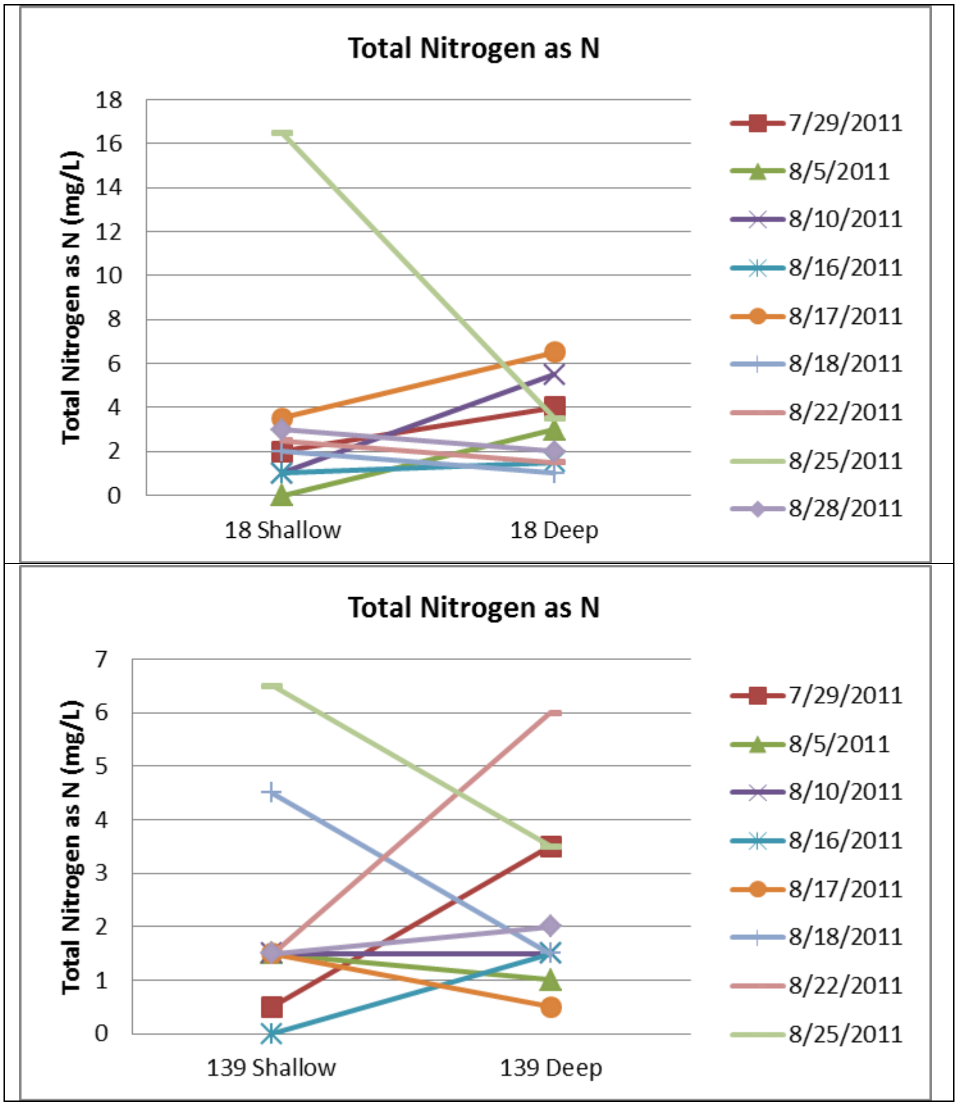


Figure D-9 Line plots for total nitrogen in different location

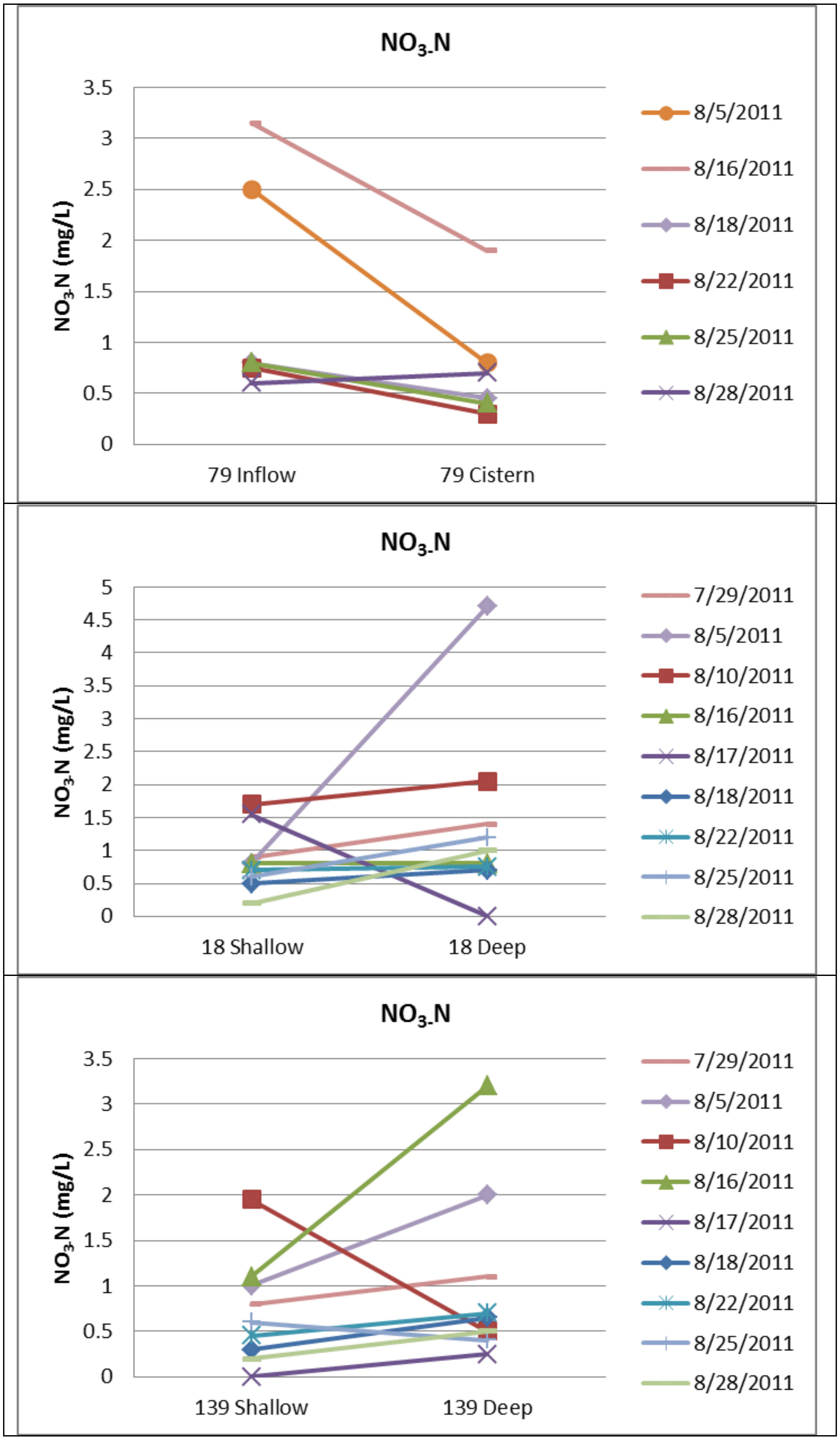


Figure D-10 Line plots for NO_3 in different location

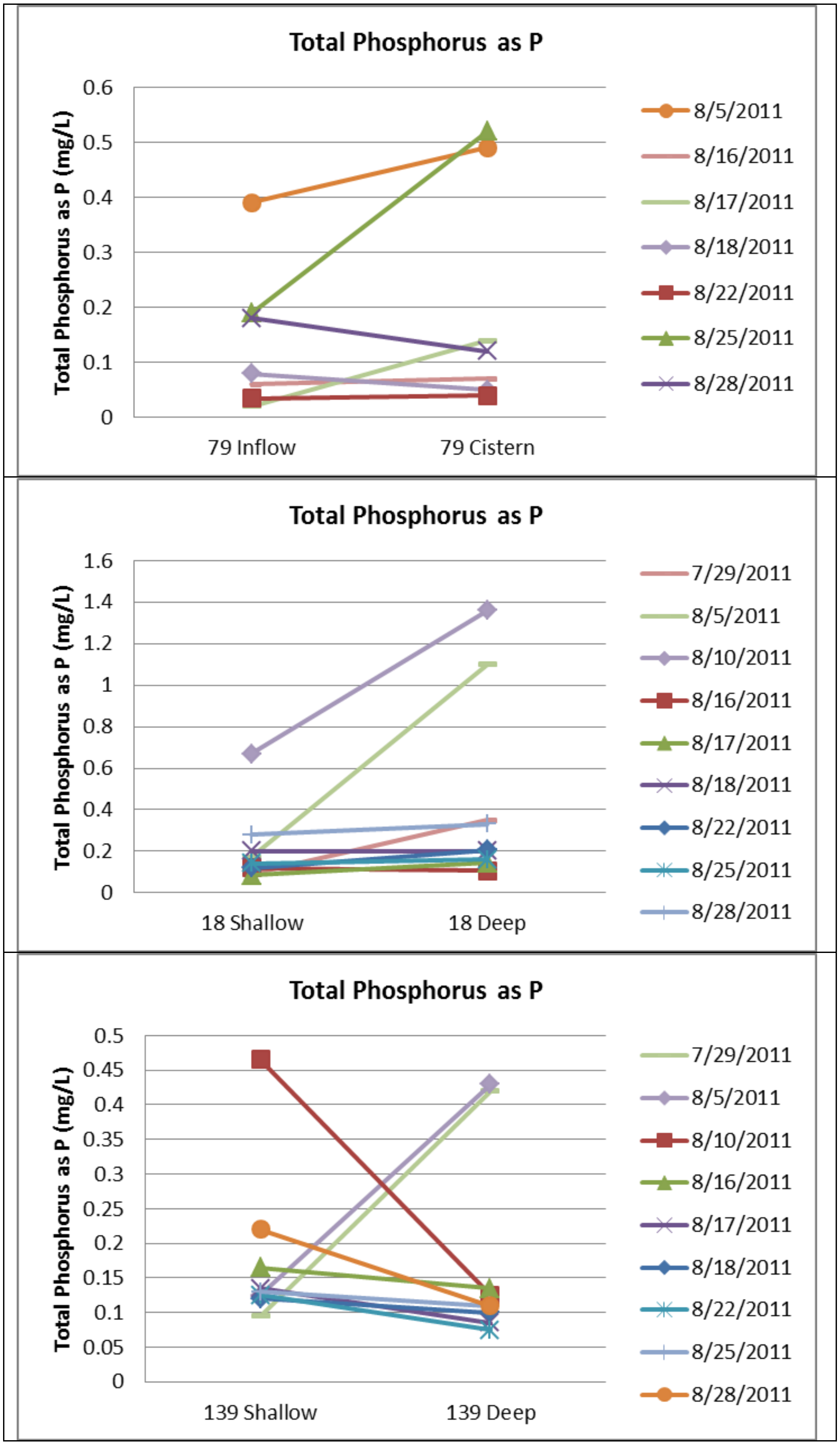


Figure D-11 Line plots for total phosphorus in different location

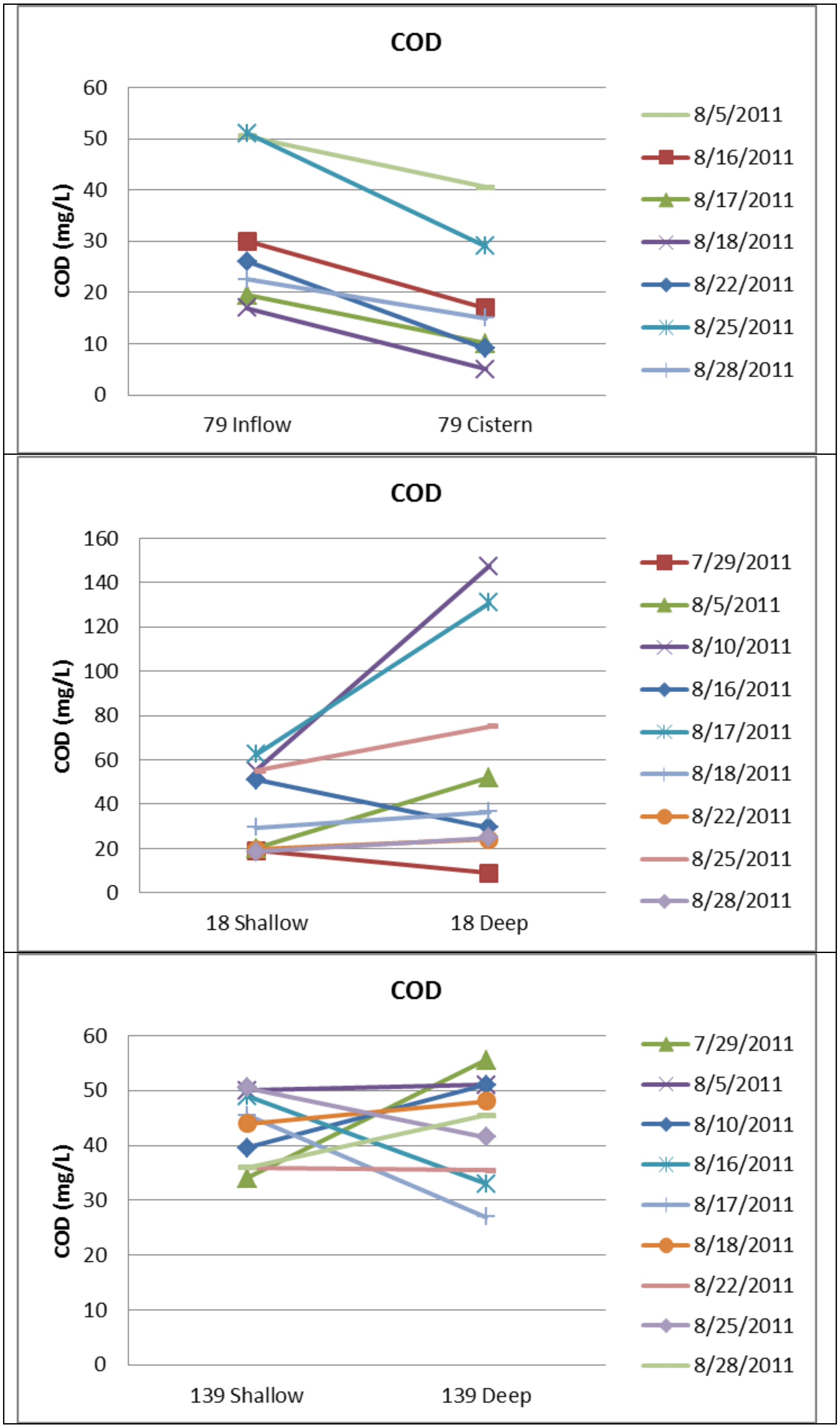
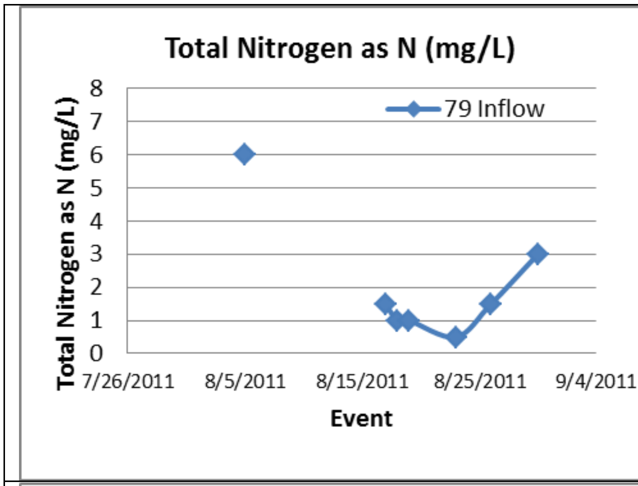
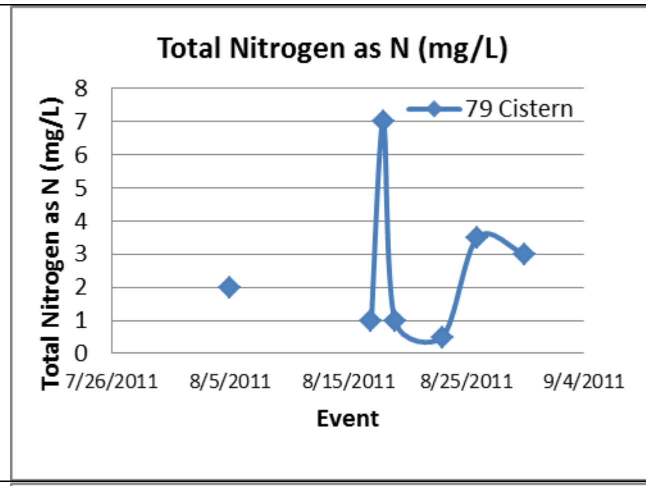
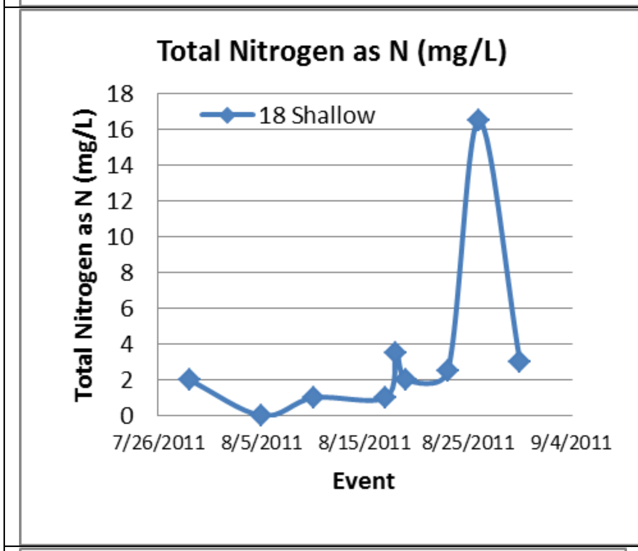


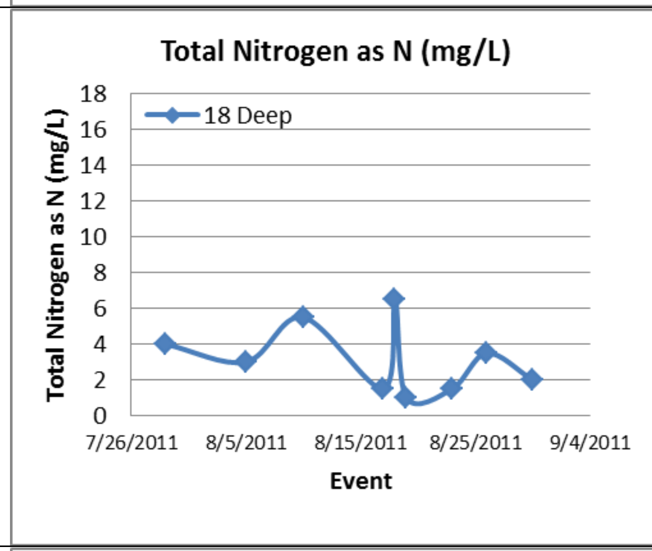
Figure D-12 Line plots for COD in different location

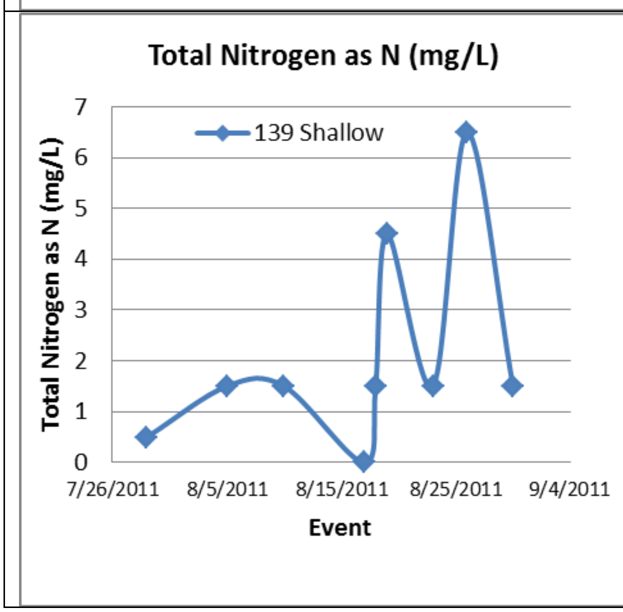
Time Series Plots

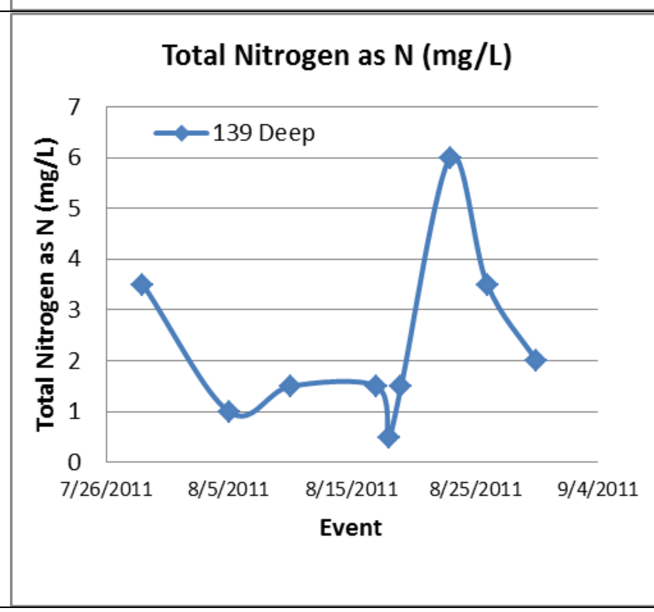


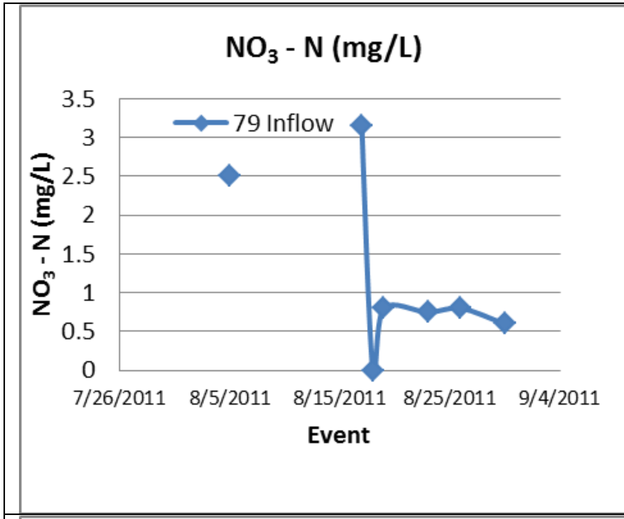


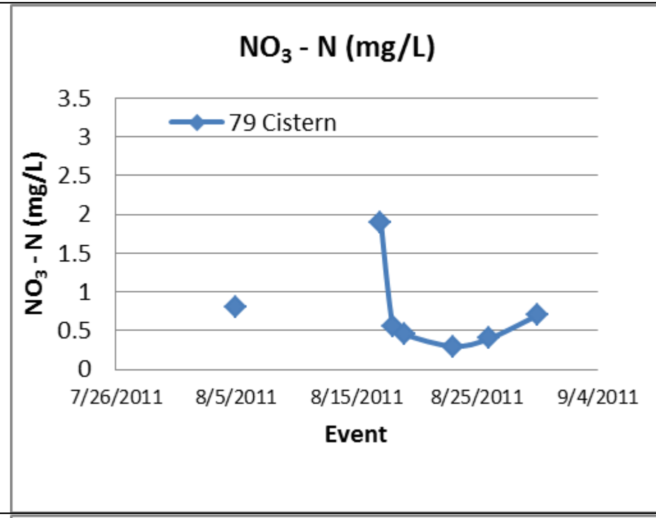


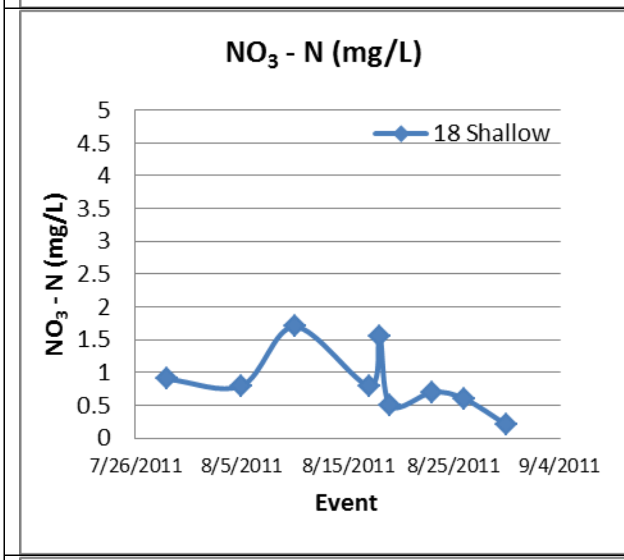


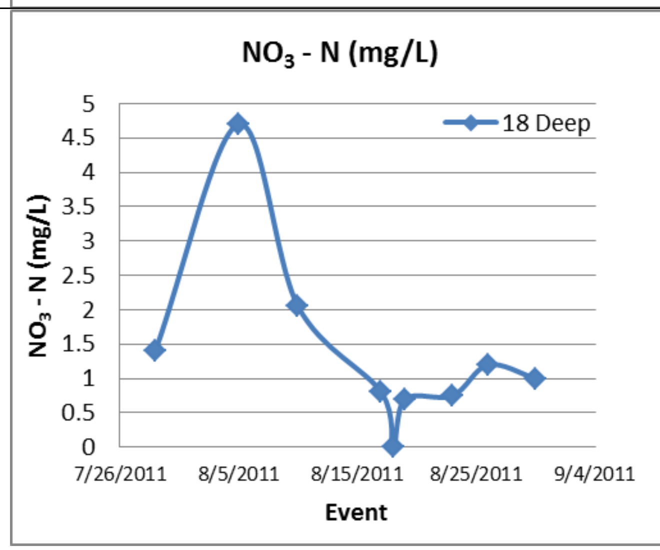


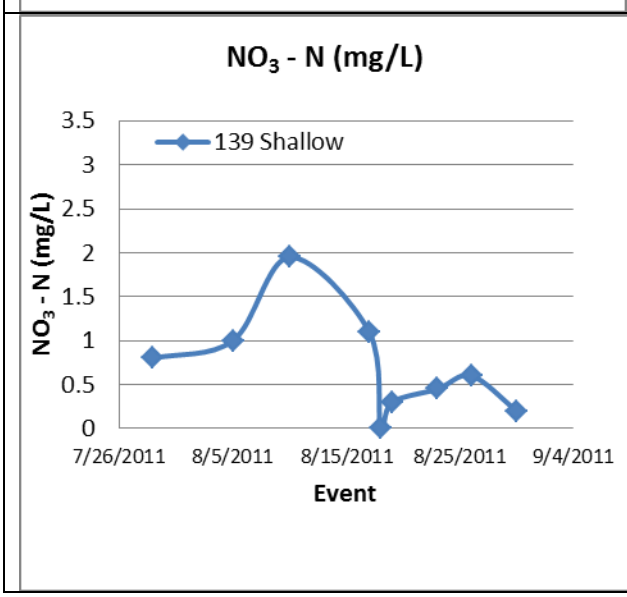


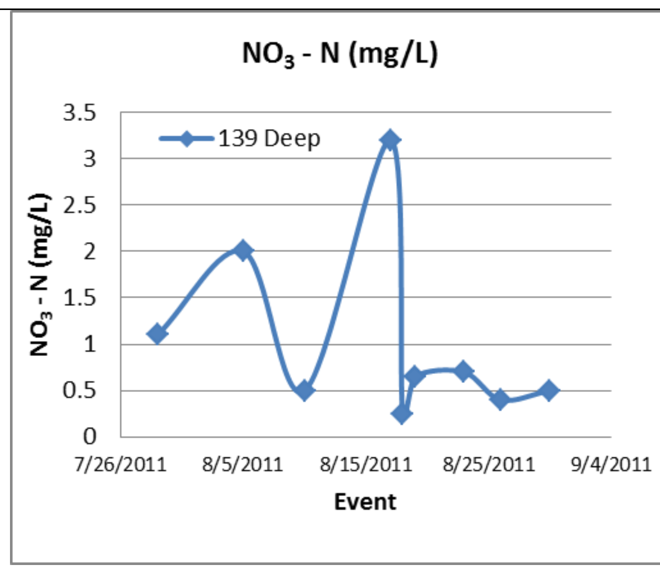

NO3 - N (mg/L)
79 Inflow
NO3 - N (mg/L)
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

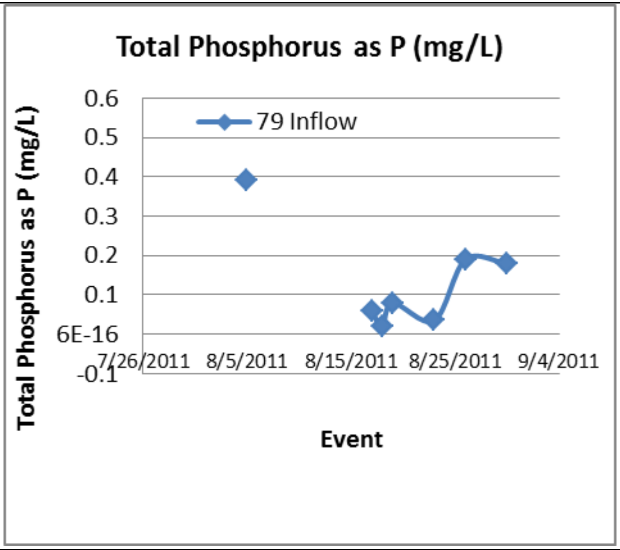

NO3 - N (mg/L)
79 Cistern
NO3 - N (mg/L)
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

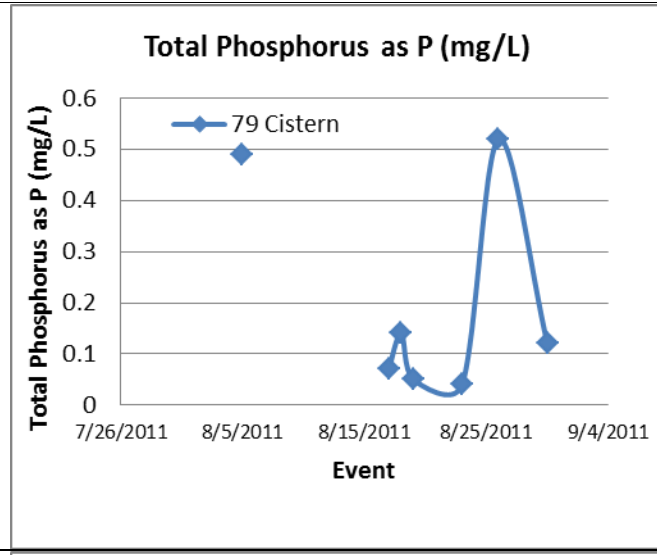

NO3 - N (mg/L)
18 Shallow
NO3 - N (mg/L)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

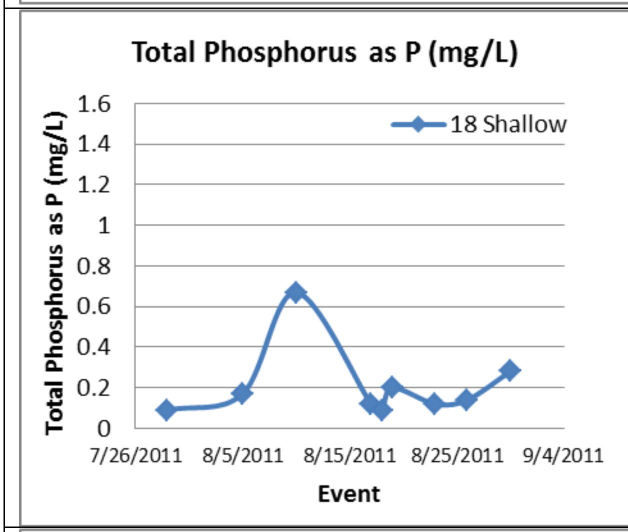

NO3 - N (mg/L)
18 Deep
NO3 - N (mg/L)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

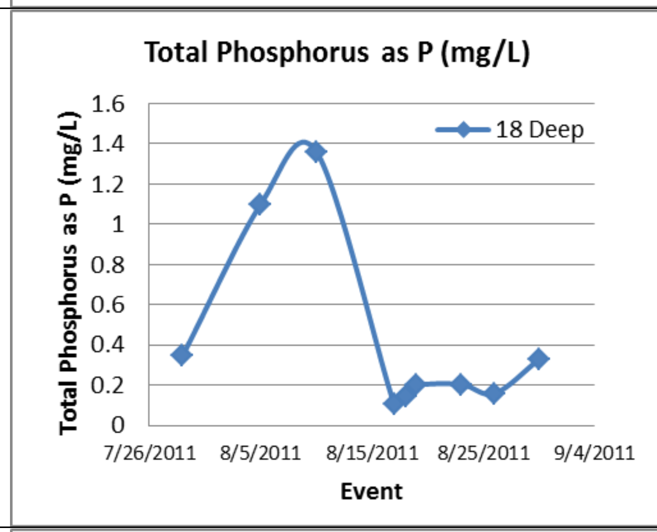

NO3 - N (mg/L)
139 Shallow
NO3 - N (mg/L)
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

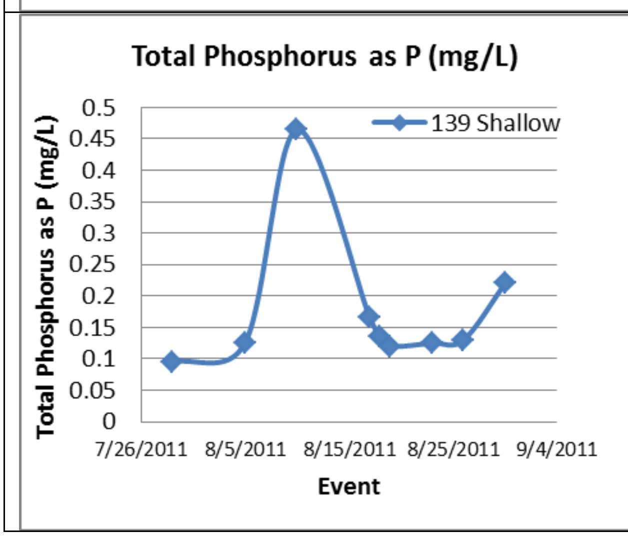

NO3 - N (mg/L)
139 Deep
NO3 - N (mg/L)
3.5
3
2.5
2
1.5
1
0.5
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

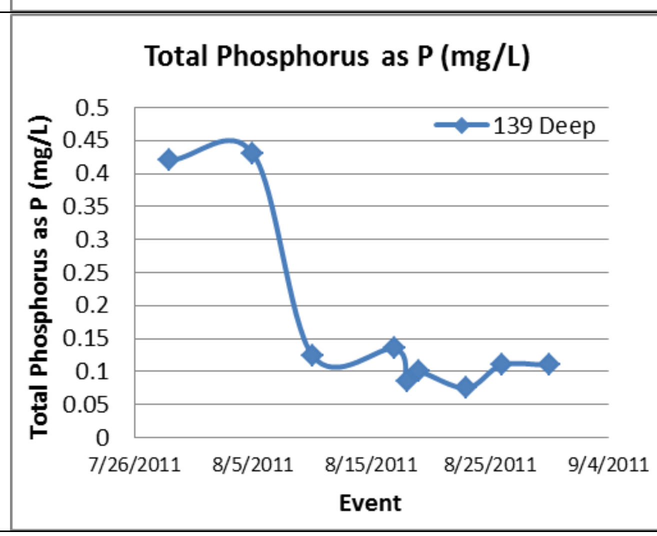

Total Phosphorus as P (mg/L)
79 Inflow
Total Phosphorus as P (mg/L)
0.6
0.5
0.4
0.3
0.2
0.1
6E-16
-0.1
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

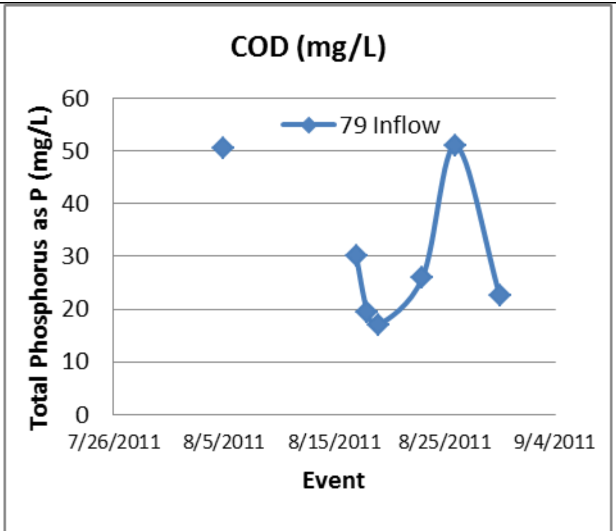

Total Phosphorus as P (mg/L)
79 Cistern
Total Phosphorus as P (mg/L)
0.6
0.5
0.4
0.3
0.2
0.1
0
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

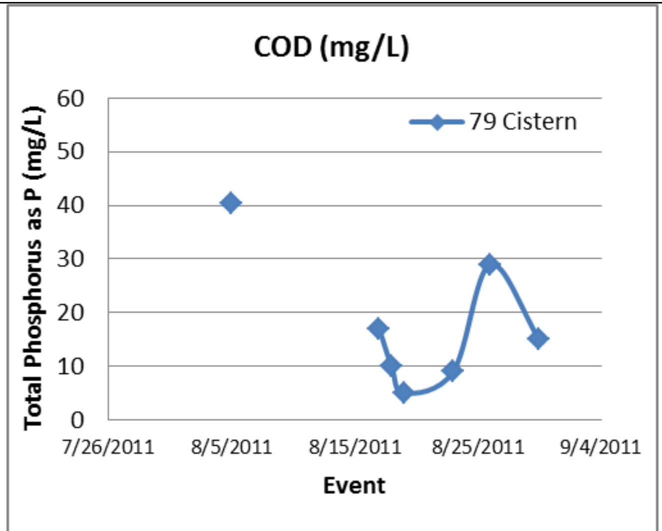

Total Phosphorus as P (mg/L)
18 Shallow
Total Phosphorus as P (mg/L)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

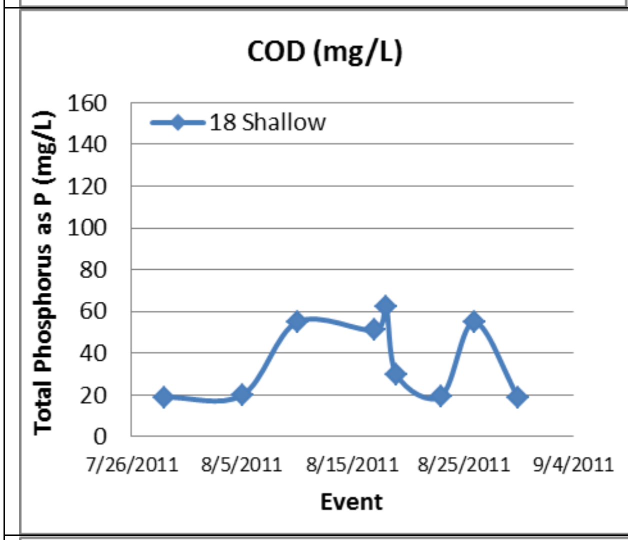

Total Phosphorus as P (mg/L)
18 Deep
Total Phosphorus as P (mg/L)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

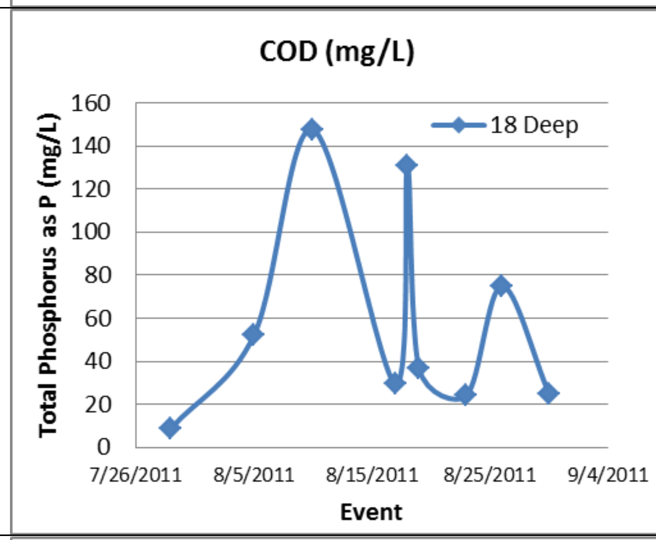

Total Phosphorus as P (mg/L)
139 Shallow
Total Phosphorus as P (mg/L)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

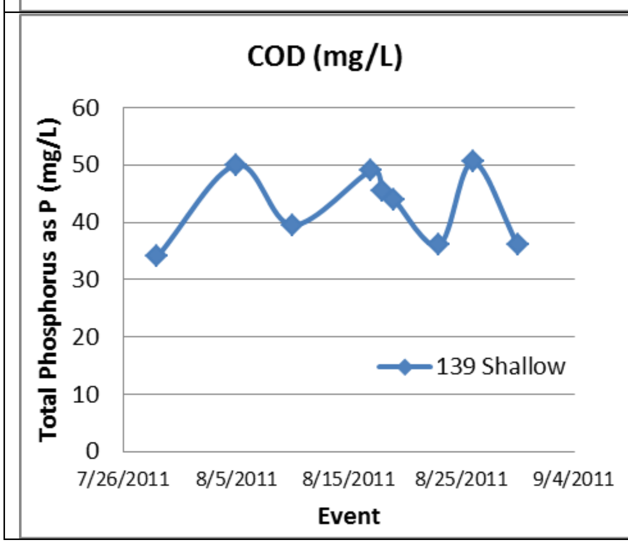

Total Phosphorus as P (mg/L)
139 Deep
Total Phosphorus as P (mg/L)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
7/26/2011 8/5/2011 8/15/2011 8/25/2011 9/4/2011
Event

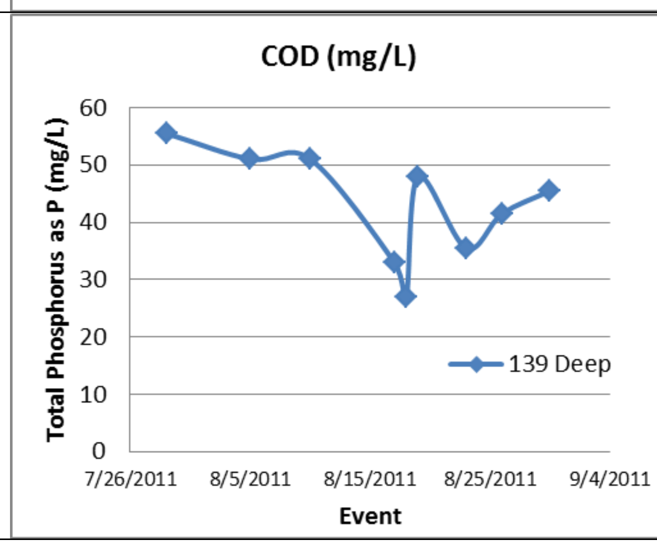

COD (mg/L)
79 Inflow
Total Phosphorus as P (mg/L)
60
50
40
30
20
10
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event


COD (mg/L)
79 Cistern
Total Phosphorus as P (mg/L)
60
50
40
30
20
10
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event


COD (mg/L)
18 Shallow
Total Phosphorus as P (mg/L)
160
140
120
100
80
60
40
20
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event


COD (mg/L)
18 Deep
Total Phosphorus as P (mg/L)
160
140
120
100
80
60
40
20
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event


COD (mg/L)
139 Shallow
Total Phosphorus as P (mg/L)
60
50
40
30
20
10
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event


COD (mg/L)
139 Deep
Total Phosphorus as P (mg/L)
60
50
40
30
20
10
0
7/26/2011
8/5/2011
8/15/2011
8/25/2011
9/4/2011
Event

Appendix E. Urban Evapotranspiration (ET) Values for Irrigation Calculations

Knowledge of site ET conditions on a monthly basis is needed to determine the irrigation deficit that can be met by using harvested stormwater. ET monitoring primarily occurs in agricultural and wild land environments. With increasing water conservation interests in urban areas, there have been new interests in applying the available ET data to urban environmental conditions. ET data is needed when investigating sanitary wastewater and stormwater reuse options applied to supplemental irrigation, and for more accurate modeling of rain garden and green roof controls for stormwater management. Climate-based methods are the most common method used to monitor ET. Evapotranspiration potential, ET_o, is normalized for a standard condition that reflects agricultural conditions (usually for irrigated alfalfa). The ET_o value therefore needs to be adjusted according to the soils, plants, and growing season conditions for the site of interest. Most of these adjustment factors were developed for agricultural situations and their use in highly disturbed urban environments has not been well-documented. Pitt, *et al.* 2011 examined and mapped available ET_o values most suitable for major urban areas.

Evapotranspiration Data

Applying agricultural and wildland ET information to urban areas can be useful, but necessarily accurate. During a recent comparison of rain gardens in clay and sandy soils by the United States Geological Survey (USGS), ET was used extensively to compare bioretention performance for turf grasses and natural prairie vegetation (Selbig 2010). The ET was calculated using data collected from an onsite weather station and compared to measured ET values by mass balances. The calculated ET values did not compare accurately to the measured values, but both methods indicated similar trends. The following discussion presents ET data for the Millburn, NJ area and a common calculation method using local metrological data.

ASCE Standardized Reference Equation

The ASCE Standardized Equation (ASCE 2005) is the most recent in a series of standards that have been adopted for reference ET calculations. Both the ASCE and Food and Agriculture Organization (FAO-56) have approved versions of the equation with only minor differences (standard crop height being the major difference). ASCE reference ET can be calculated for only two crop heights, short (grasses) and tall (alfalfa). The data available in this report was calculated for a short reference crop. The result, ET_o or ET_r, is the reference ET for a well-watered crop. It is calculated in millimeters per day$\left(\frac{mm}{day}\right)$, and was then converted to inches per day$\left(\frac{in.}{day}\right)$. The general form for the equation is shown below in Equation D-1.

Equation D-1. ASCE Standardized Reference Equation

$$ET_{ref} = \frac{0.408\Delta(R_n - G) + \gamma\frac{(C_n)}{T + 273}(e_s - e_a)u_2}{\Delta + \gamma(1 + c_d u_2)}$$

Rainmaster

Rainmaster by Irritrol is the most complete and easiest to use resource for estimated average monthly ET rates for urban areas in the U.S. The method only requires a nearby zip code to generate ET values for a site. This is a commercial site used as a resource for their irrigation equipment business at: http://www.rainmaster.com/historicET.asp. Monthly ET values for zip code 07041 (Millburn, NJ) shown on the proprietary Rainmaster ET web site are very similar to the ASCE standardized reference ETo values, while the Kimberly Penman ETr values are about 30% larger. The differences are associated with the adjustment factors that are included in the different forms of the equations.

Evapotranspiration

ET is defined as the rate at which readily available water is removed from the soil and plant surfaces expressed as the rate of latent heat transfer per unit area λET_{ref} or expressed as a depth of water evaporated and transpired from a reference crop (Jensen, *et al.* 1990). Unless soil moisture is kept near field capacity, there will be times when ET estimates outweigh actual ET removed from the soil. Calculated ET values for the short reference crop for well watered cool season grasses is further modified for specific plants. A plants actual ET is calculated from the product of these original equations by multiplying ET_o by coefficients for each plant type providing a daily estimate for the crop under well watered conditions. There are lists of approved coefficients (such as WUCOLS III) for both grass reference and alfalfa values; however these values are not interchangeable. Table D-1 shows some crop coefficient factors (used to modify the reference ET_o values), along with the root depths. Generally, deeper rooted plants can remove water from deeper soil layers.

Table D-1. Crop Coefficient Factors and Root Depths (Pitt, *et al.* 2008)

Plant	Crop Coefficient Factor (Kc)	Root Depth (ft)
Cool Season Grass (turfgrass)	0.80	1
Common Trees	0.70	3
Annuals	0.65	1
Common Shrubs	0.50	2
Warm Season Grass	0.55	1
Prairie Plants (deep rooted)	0.50	6

Grasses are resilient plants and often recover in difficult drought conditions. However, grasses have limitations such as shallow root depths that reduce their effectiveness in stormwater reuse. Therefore, some users may believe that some plants and shrubs may be modeled better using an alfalfa reference ET. Alfalfa has a much deeper root system than turf grass. Hence some plants and shrubs with deeper root systems could have the ability to remove water held deeper in the soil than grass, increasing the storage potential for a site as well as reducing losses from runoff.