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# HYDROLOGIC CHARACTERIZATION OF A RAIN GARDEN MITIGATING STORMWATER RUNOFF FROM A COMMERCIAL AREA

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HYDROLOGIC CHARACTERIZATION OF A RAIN GARDEN MITIGATING  
STORMWATER RUNOFF FROM A COMMERCIAL AREA

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THESIS

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A thesis submitted in partial fulfillment of the  
requirements for the degree of Master of Science in Biosystems and Agricultural Engineering  
in the College of Engineering at the University of Kentucky

By

John Thomas McMaine

Lexington, Kentucky

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Biosystems and Agricultural Engineering

Lexington, Kentucky

2013

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## ABSTRACT OF THESIS

### HYDROLOGIC CHARACTERIZATION OF A RAIN GARDEN MITIGATING STORMWATER RUNOFF FROM A COMMERCIAL AREA

Impervious surfaces such as roads, sidewalks, and roofs increase the volume of runoff generated in a watershed. Traditional stormwater management techniques emphasize conveyance of runoff away from impervious surfaces in order to reduce flooding. Rain gardens are becoming popular as a different means to manage stormwater in such a way that runoff is captured and infiltrated onsite rather than conveyed offsite. A stormwater management system consisting of a rainwater harvest system, rain garden, and infiltration chamber was built at the Coca-Cola Refreshments USA, Inc. distribution center in Lexington, Kentucky during the fall of 2011. Precipitation, inflow, and water level were measured from May, 2012 to April, 2013 to evaluate the hydrologic performance of the rain garden. The rain garden had a high infiltrative capability and was able to capture and infiltrate 100% of the runoff generated during the study period. The results of the study were used to formulate recommendations for rain garden design and construction in central Kentucky.

**KEYWORDS:** Rain garden/ bioretention, hydrology, low impact development, stormwater, infiltration

John Thomas McMaine

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October 16, 2013

Date



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STORMWATER RUNOFF FROM A COMMERCIAL AREA

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For my family.

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## CHAPTER 1: INTRODUCTION

### 1.1 INTRODUCTION

Urbanization leads to the conversion of pervious surfaces (e.g. forests, pastures and grasslands) into impervious ones (e.g. parking lots, roads and buildings). This conversion results in a decrease in the amount of precipitation that infiltrates into the soil thus resulting in increased runoff volumes and peak flows and decreased times of concentration. As watersheds are urbanized, their hydrology transforms from an infiltration and groundwater driven system to a surface runoff driven one (Paul and Meyer 2001). Streams that are groundwater driven experience less variable flow patterns, while streams in urbanized, runoff-driven watersheds experience highly variable flow patterns. Urbanized streams experience greater peak flows and flow volumes during precipitation events and lower base flows between precipitation events (Simmons and Reynolds 1982). Booth (2000) found that streams in a watershed in King County, Washington exhibited signs of destabilization when the amount of impervious area exceeded 10% in a previously undeveloped watershed. Schiff and Benoit (2007) found that streams in New Haven, Connecticut experienced a decline in water quality, macroinvertebrate assemblages (e.g. EPT taxa and other pollutant-intolerant organisms), and physical habitat with small amounts of impervious area in their watersheds. EPT taxa richness indicates the number of mayfly, stonefly, and caddisfly taxa present in a water sample. Signs of stream habitat quality degradation were present when total impervious areas exceeded 5%, and habitat quality was consistently degraded when total impervious areas exceeded 10%.

Traditional stormwater management techniques emphasize flood reduction through conveyance of stormwater runoff away from impervious surfaces using structures such as gutters, curbs, and curb inlets. These structures rapidly route stormwater runoff into the storm sewer system which in turn conveys these waters to receiving water bodies such as streams and lakes. These rapid conveyance controls exacerbate issues related to increased peak flows, such as flooding, decreased water quality, streambank erosion, and aquatic habitat destruction, since they reduce a watershed's time to peak. Traditional stormwater management systems also use structures to store and slowly release stormwater, such as retention and detention basins, in conjunction with conveyance infrastructure. Note that

both retention and detention basins are structures that collect stormwater runoff and release it at a controlled rate over a period of time, the difference being that retention basins maintain a permanent pool while detention basins do not.

These basins are designed to mimic predevelopment peak flows, but generally offer little in the way of mechanisms to reduce runoff volumes, such as through infiltration or evapotranspiration (Emerson, 2003). While detention and retention basins can decrease peak flows on a small-scale such as for streams immediately down-gradient from development sites, they have little to no effect on peak flow reductions on a watershed scale even if spread throughout the watershed (McCuen, 1974; Emerson et al. 2005; Goff and Gentry, 2006). The reason for this is related in part to the design storms used and the basins cumulative hydrologic effect. Constructed to manage flooding, stormwater basins are primarily designed for larger, lower frequency design storms, and as such, the peaks from smaller, more frequent storms receive little if any attenuation (Emerson, 2003; WEF et al., 2012). Thus, for these smaller storms, which comprise a greater percentage of the annual runoff, peak flows and runoff volumes are not necessarily dampened. The ineffectiveness of these basins is cumulative and is particularly impacting on streams at the lower end of the watershed. As demonstrated by McCuen (1974), runoff from the upper reaches of a watershed can combine with delayed releases of stormwater from the lower parts of the basin to produce higher peak flows for extended periods of time. Longer durations of elevated flows mean that streambanks are subjected to higher shear stresses more frequently and for a longer period of time (WEF et al., 2012) resulting in greater levels of streambank erosion (Moglen and McCuen, 1988). Furthermore, groundwater recharge is low with conventional stormwater structures as they infiltrate only small amounts of runoff. This decreased level of groundwater recharge means that streams also experience diminished baseflow levels unless inflows come from another source such as irrigation runoff or industrial discharges.

Recognizing that conventional stormwater management practices can negatively impact stream systems, efforts are underway to better emulate multiple pre-development hydrologic conditions (e.g. volume, peaks, and durations) rather than focusing only on peak flows (USEPA, 2008). These efforts are embodied in the philosophy of low impact development (LID). LID emphasizes management of stormwater at the individual lot or site level rather than the quick conveyance of stormwater to offsite or down-gradient locations.

By largely enhancing infiltration and evapotranspiration and also decreasing impervious area amounts and connectedness, LID seeks to decrease the amount of stormwater leaving a site (Prince Georges County, 1999; USEPA, 2008). In essence, LID seeks to decrease a watershed's curve number and increase its time of concentration. LID also seeks to improve water quality onsite through mechanisms such as filtration, sedimentation, sorption, and the promotion of biological processes (WEF et al., 2012). Some examples of LID structural controls include: permeable pavement, rainwater harvest systems, stormwater wetlands, and rain gardens or bioretention (WEF et al., 2012; Agouridis and McMaine, 2013).

Of the LID structural controls, rain gardens are one of the most popular especially with homeowners as evidenced by programs such as Montgomery County, Maryland's RainScapes and Kansas City, Missouri's 10,000 Rain Gardens Project. A rain garden is a shallow depression with amended soils and landscaping that is used to control runoff, often from a 25 mm storm event, mainly by increasing infiltration with some evapotranspiration also occurring. This best management practice (BMP) is fairly easy to construct, at the residential level, and is aesthetically appealing with its landscaped appearance making it popular with gardeners. Rain gardens are also becoming more popular at commercial sites such as Coca Cola (Lexington, Kentucky and Wichita, Kansas), schools, and local governments (Lexington-Fayette Urban County Government or LFUCG). It is with these larger rain gardens that designs become more complex and challenging.

While research into rain garden hydrologic performance has been conducted, to date, none has been performed in Kentucky. Areas of research have included: field performance of two rain gardens in Connecticut (Dietz and Clausen 2005); mitigating peak flows using bioretention in North Carolina (Hunt et al. 2008); mitigating the hydrologic effects of impervious area in North Carolina and Maryland (Li et al, 2009); and the effects of media depth on hydrologic performance in North Carolina (Brown and Hunt, 2010). First proposed in College Park, Maryland (Coffman et al, 1993), research into this BMP has predominantly occurred in the mid-Atlantic states of Maryland, North Carolina, and Pennsylvania (Heasom et al, 2006; Hunt et al 2006; Hunt et al,2008; Brown et al, 2009; Davis et al 2011). While these studies helped in understanding rain garden hydrologic performance, the lack of regionally relevant rain garden research for Kentucky is of concern, as the design and installation of these BMPs, in residential and commercial settings, is being promoted by the three of the largest urban stormwater entities in central Kentucky: LFUCG

in Lexington, Metropolitan Sewer District in Louisville, and Sanitary District 1 in Northern Kentucky. In central Kentucky, rain garden promotion and adoption has increased through programs such as the LFUCG's Stormwater Quality Projects Incentive Grants Program and the University of Kentucky's Cooperative Extension rain garden workshops. However, recommended design parameters, such as in LFUCG's stormwater manual (LFUCG, 2009), may not accurately reflect rain garden performance in central Kentucky meaning rain gardens at commercial facilities may be over-designed and hence larger and more expensive than needed or under-designed and not capturing and treating the desired amount of stormwater. By monitoring a commercial rain garden constructed by a licensed and experienced professional, data collected on the actual field performance allows for the comparison of measured parameters to recommended design parameters in LFUCG's Stormwater Manual (LFUCG, 2009). Such comparisons can lead to design improvements.

## **1.2 OBJECTIVES**

The goal of this study was to evaluate the effectiveness of a rain garden at mitigating stormwater runoff from a commercial development. Specific objectives were to:

1. Determine stormwater inflow volumes for individual precipitation events,
2. Measure the vertical flux of infiltrated stormwater,
3. Calculate the potential evapotranspiration,
4. Compare actual field performance to theoretical performance using the design parameters specified in LFUCG's Stormwater Manual (LFUCG, 2009), and
5. Develop recommendations for improving the design process.

## **1.3 ORGANIZATION OF THESIS**

Chapter 1 introduces the thesis. Background information on the hydrologic issues associated with urbanization is provided along with information on how LID and more specifically rain gardens address these issues. This chapter also outlines the objectives of the project. Chapter 2 is a review of available literature related to the hydrologic performance of rain gardens as well as design parameters and guidance given in current stormwater manuals, both local (LFUCG) and from other states (Iowa, Maryland, and North Carolina). Chapter 3 explains the procedures used to meet the objectives stated in Chapter 1 of the thesis,

specifically data acquisition and analysis methods. Chapter 4 presents the results of the data analysis and an interpretation of these results. Chapter 5 provides conclusions drawn from the project and gives recommendations for future rain garden designs. Chapter 6 discusses future work. The appendixes supply graphics and pertinent procedures not presented in the main text.

## CHAPTER 2: LITERATURE REVIEW

### 2.1 RAIN GARDEN COMPONENTS

A rain garden is a landscaped depression with amended soils that is used to capture stormwater typically from small, frequent rainfall events, such as the 25 mm (1 inch) rainfall depth (Clar et al, 2004). Existing soils are amended, often with sand, an organic material such as leaf compost, and sometimes wood chips, in order to increase infiltration rates (Roy-Poirier et al, 2010). Native shrubs, flowers, grasses and/or trees are used to landscape the rain garden to increase evapotranspiration and aesthetic appeal (Table 2.1). Native plants are used since they are the better adapted to local climate conditions and require less maintenance (MDE, 2009). Since rain gardens experience periods of inundation and drought, careful plant selection is required (Hunt and White, 2001; Andruczyk et al, 2009; Agouridis and McMaine, 2013).

When designing a rain garden, seven parameters require careful consideration (Hunt and White, 2001). These parameters include:

- Pretreatment structure (e.g. forebay or grass filter strip),
- Above-ground storage (e.g. surface area and outlet structure invert elevation),
- Filter bed surface area,
- Depth of amended filter bed,
- Composition of filter bed,
- Native plant types and densities, and
- Underdrain structure.

A forebay, which is a small depression located immediately upgradient from the main treatment area, is often used as a pretreatment structure although grass filter strips are also used. The purpose of the pretreatment structure is to prevent clogging of the rain garden filter bed by settling out sediments and other larger particles such as road and roof debris (PGCDER, 2007; Davis et al, 2009). Stormwater enters the forebay either as overland flow (grass filter strip used as pretreatment) or through a pipe or gutter (forebay used as pretreatment). After exiting the forebay, stormwater flows into the filter bed where it is



Table 2.1. Native plants commonly used in rain gardens.<sup>1</sup>

Common Name	Scientific Name
Eastern Redbud	<i>Cercis canadensis</i>
Dogwood	<i>Cornus amomum/racemosam/sericea</i>
Blackeyed Susan	<i>Rudbeckia hirta</i>
Switch Grass	<i>Panicum virgatum</i>
Jewelweed	<i>Impatiens capensis</i>
Iris	<i>Iris lousiana/pseudacorus/versicolor/virginica</i>
Big Bluestem	<i>Andropogon gerardii</i>

Source: Andruczyk et al. (2009) and LFUCG (2009)

temporarily stored. The amount of storage available in the rain garden is dependent on the shape of the rain garden, as reflected by its surface area, and the invert elevation of the outlet structure. Water ponded or stored in the rain garden should infiltrate within 24 to 48 hours (WEF et al, 2012; MDE, 2009), the rate of which is controlled largely by the depth and composition of the filter bed. In some instances, an underdrain, which is typically a 10.2 to 15.2 cm perforated PVC or corrugated plastic pipe beneath the amended soil and running the length of the filter bed, beneath the amended soil layer is used. An underdrain is typically recommended if the existing soil is unable to infiltrate water quickly enough to meet the ponding time guideline. If an underdrain is used, water leaves the rain garden more quickly since it is able to exit the underdrain faster than it is able to infiltrate into the surrounding soil. As previously mentioned, native plants are added to increase evapotranspiration rates, improve aesthetics, and in some instances, improve water quality (Pilon-Smits and Freeman, 2006; Read et al., 2008).

## 2.2 RAIN GARDEN DESIGN METHODOLOGY

Rain garden design mainly consists of answering three questions: 1) How much stormwater will enter the rain garden?, 2) How much stormwater will the rain garden store?, and 3) How quickly will stormwater leave the rain garden? The amount of stormwater entering the rain garden is determined in large part by drainage area, land use, soils, and rainfall depth (Dietz and Clausen, 2005). Once stormwater enters the rain garden, it is

stored in one of three zones: above-ground storage, subsurface storage within the root zone, and subsurface storage below the root zone (Davis et al., 2011). Storage is determined by height of the outlet structure invert, filter bed surface area, and depth and composition of the filter bed media. Stormwater exits the rain garden via the outlet structure (for large events), via underdrain (if present), as exfiltration through the filter media and into the existing soil, and/or as evapotranspiration. Thus, flux out of the rain garden is largely determined by the infiltration rate and storage provided by the filter bed, type and compaction of existing soil beneath the rain garden, presence of structures (underdrain and outlet), and type and density of native plants. In the case of evapotranspiration, climate and season also play important roles in rain garden performance. Evapotranspiration rates are higher in areas with greater amounts of solar radiation, such as the southwestern part of the United States where annual evapotranspiration is nearly equal to annual precipitation (Hanson, 1991). With regards to season, plants are predominantly dormant during the winter months when temperatures are low, and as such, evapotranspiration rates are at their lowest (Allen et al, 1998).

### **2.2.1 Design Volume**

Rain gardens are designed to treat a specific volume, usually the Water Quality Volume (WQV). The WQV generally represents 80-90% of the average annual runoff for a site (WEF et al., 2012). Since most of this runoff is generated by relatively small storm events, rain gardens are designed to capture runoff from these small events, which are often about 25 mm in depth though this value can vary by region (Hunt, 2009). This rainfall depth is applied to the drainage area contributing to the rain garden to determine the volume of stormwater requiring treatment by the rain garden.

The amount of stormwater produced by a watershed is dependent to a large degree on the percentage of imperviousness in the watershed. In a naturally vegetated watershed, approximately 10% of rainfall becomes runoff with the other 90% becoming deep infiltration, shallow infiltration, and evapotranspiration. As imperviousness increases to 75 – 100% of a watershed (as is the case in many urbanized watersheds), approximately 55% of rainfall becomes runoff (Arnold and Gibbons, 1996). Increased imperviousness alters the hydrology of a watershed by increasing the amount of runoff generated (Paul and Meyer, 2001).

As watersheds are developed, meaning the percentage of imperviousness increases, the WQV also increases. Thus, an important first step in designing a rain garden is to determine the WQV, a process which can differ across state and local entities. One method is to assign a design depth to a percentage of imperviousness. LFUCG (2009), for instance, specifies the design depth for 100% impervious area as 40 mm. Another method is to use the Simple Method (Schueler, 1987) as seen in equation 2.1.

$$WQV = P \cdot R_v \cdot A \quad (\text{eqn. 2.1})$$

The variable P indicates rainfall depth,  $R_v$  is the volumetric runoff coefficient, and A is drainage area. Both IDNR (2009) and MDE (2009), for instance, use the Simple Method to determine the WQV.

Rainfall depth is specified, as in the case of IDNR (2009) and MDE (2009), or it is calculated such as with a cumulative probability distribution (WEF et al., 2012). The  $R_v$  is an adjustment factor that takes into account the amount of impervious surface in a watershed. The variable  $R_v$  is determined through regression equations such as equations 2.2 or 2.3, which were developed by Urbonas et al. (1990) and Driscoll (1983), respectively.

$$R_v = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \quad (\text{eqn. 2.2})$$

$$R_v = 0.05 + 0.9i \quad (\text{eqn. 2.3})$$

The variable  $i$  represents the fraction of imperviousness in the watershed. Equation 2.3 produces higher values of  $R_v$ , and hence greater WQVs, than equation 2.2.

A third technique is to use the SCS curve number (CN) method (NRCS, 2004). Infiltration is largely determined by land use and characteristics of the underlying soils. Curve numbers range from 30 for a meadow protected from grazing with a hydrologic soil group (HSG) A to 98 for impervious areas such as paved parking lots, roofs, driveways. If a watershed has multiple land uses and/or soil types, a CN is calculated for each land use/soil combination individually (individual flow depths or volumes are summed to arrive at a total depth or volume). The CN is then used to determine the maximum storage or soil water retention parameter, S using equation 2.4. The initial abstraction,  $I_a$ , is the amount captured

by the surface before runoff begins, and is calculated from S, which is generally assumed to be 0.2. The variables P, which is the depth of the design storm,  $I_a$ , and S are then used to calculate runoff depth, Q using equation 2.5.

$$S = \frac{25,400}{CN} - 254 \quad (\text{eqn. 2.4})$$

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad (\text{eqn. 2.5})$$

To calculate the volume of runoff generated by the design storm, Q is multiplied by drainage area.

### 2.2.2 Above-ground Storage

The required above-ground storage volume varies between states and local municipalities. Table 2.2 contains a summary of rain garden or bioretention design specifications for select stormwater manuals. The North Carolina Department of Environment and Natural Resources (NCDENR) (2009) specifies that the surface area of a rain garden should be equal to the WQV divided by the desired ponding depth. However, LFUCG (2009), IDNR (2009), and MDE (2009) require the designer to use Darcy's Law to determine the above-ground storage requirements. Darcy's Law is a relationship describing flow through a porous media (Schwartz and Zhang, 2003), as shown in equation 2.6.

Table 2.2. Stormwater manual rain garden design specifications.

Stormwater Manual	Above-ground Storage <sup>1</sup>	Filter Bed Composition	Coefficient of Permeability	Maximum Ponding Depth	Underdrain	Filter Bed Depth	Dewatering Time
North Carolina	Containment of 100% of WQV above-ground	85-88% sand 8-12% silt and clay 3-5% organics	2.5-15 cm h <sup>-1</sup> required 2.5-5 cm h <sup>-1</sup> recommended	30 cm max 23 cm recommended	If existing soils permeability <50 mm h <sup>-1</sup>	For grassed planting 0.6 m For shrubs/trees 0.9 m Maximum 1.2 m	12 h from surface 48 h to 0.6 m below surface
Maryland	Containment of 75% of WQV above-ground	--	15 cm d <sup>-1</sup>	30 cm maximum	--	0.4 – 1.2m	48 hours
LFUCG <sup>2</sup>	--	10-25% clay 30-55% silt 35-60% sand	15 cm d <sup>-1</sup>	15 cm	Provide an underdrain	1.2 m	72 h
Iowa	--	50-60% sand 20-25% topsoil (sandy clay loam) 20-25% leaf compost	15 cm d <sup>-1</sup> if using existing soil profile 2.5 cm h <sup>-1</sup> for sandy loam amended soil	15 – 23 cm + freeboard	If existing soils have poor infiltrative capacity	1.4 –1.7 m total 0.5–0.8 m planting bed 0.5 m minimum	48 h

<sup>1</sup>WQV indicates water quality volume

<sup>2</sup>LFUCG indicates Lexington Fayette Urban County Government (Kentucky)

$$A_f = \frac{WQV \cdot d_f}{[k \cdot (h_f + d_f) \cdot t_f]} \quad (\text{eqn. 2.6})$$

where:

- $A_f$  = surface area of the filter bed ( $\text{m}^2$ )
- WQV = water quality volume (to be infiltrated) ( $\text{m}^3$ )
- $d_f$  = filter bed depth (m)
- $k$  = coefficient of permeability of filter bed ( $\text{m d}^{-1}$ )
- $h_f$  = average height of ponded water above filter bed (m)
- $t_f$  = time required for WQV to infiltrate (d)

Note that  $h_f$  is equal to half the maximum height,  $h_{\text{max}}$ , of ponded water above the filter bed. NCDENR (2009) limits  $h_{\text{max}}$  to 31 cm but gives 23 cm as the preferred value. MDE (2009) also specifies  $h_{\text{max}}$  as 31 cm but is more conservative in that they state that the rain garden must be able to contain 75% of the WQV before infiltration occurs. LFUCG specifies  $h_{\text{max}}$  as 15 cm which is the lowest of the group.

Many entities limit maximum ponding depths are limited because of health concerns (e.g. mosquito breeding) and concerns regarding vegetation establishment. High ponding levels can mean that plants experience excessive levels on inundation, soils become compacted, and that undesirable plant species and vectors can proliferate if the rain garden does not dewater within a short period of time. However, anecdotal evidence suggests that deeper ponding depths such as 0.9 m can have healthy vegetation establishment, but with potentially more unwelcome weed species, and thus allow greater storage volume (Hunt et al., 2012).

### 2.2.3 Amended Filter Bed Depth and Composition

The required filter bed depth is most often given as a range. Deeper media allows for greater subsurface storage volume and consequently the ability to infiltrate a greater volume of runoff. NCDENR (2009) recommends a minimum filter bed depth of 0.6 m and a maximum of 1.2 m. The lower limit is reflective of the fact that most pollutant removal procedures occur within the top 0.6 m of the filter bed (Davis et al, 2003), while the upper limit is based on increased cost of excavating and amending the soil (NCDENR, 2009;

Brown and Hunt, 2010). Filter bed depth is influenced by the type of plants desired in the rain garden. If using shallow rooted plants, then a shallow filter bed depth is acceptable (0.6 m), however, if using trees and shrubs, then a deeper filter bed depth is necessary (0.9 m) (NCDENR, 2009; Hsieh and Davis, 2005). Shallow rooted plants are recommended for shallow filter bed depths since LFUCG (2009) recommends a 1.2 m filter bed depth; greater depths are allowed. MDE (2009) provides a range of 0.8 m to 1.2 m while IDNR (2009) recommends greater depths of 1.4 to 1.7 m with a minimum depth of 0.5 m.

The composition of the filter bed media affects infiltration rates, and hence the performance of the rain garden. Stormwater manuals provide recommendations for desired soil physical properties (e.g. sand, silt, clay and organic matter content; permeability) and chemical properties (e.g. pH and nutrient contents). NCDENR (2009) recommends a permeability of 0.6 m to 1.2 m d<sup>-1</sup> but allows permeability as high as 3.7 m d<sup>-1</sup>. A lower permeability is recommended in order to provide greater filtration of pollutants. LFUCG (2009) and MDE (2009) recommend the use of a permeability of 0.15 m d<sup>-1</sup> in calculations. IDNR (2009) allows a permeability of 0.6 m d<sup>-1</sup> assuming the soil is a silt loam. The permeability values provided in these stormwater manuals are on the order of fine sand (0.01 to 10 m d<sup>-1</sup>) to sand and gravel (0.3 to 10 m d<sup>-1</sup>) and may encompass some silt (0.0001 to 1 m d<sup>-1</sup>) (Dunne and Leopold, 1978). It is important to remember that compaction decreases permeability, so in urban areas where compaction is likely to have occurred as part of the development process, care should be taken in assigning a permeability based on soil texture alone (Pitt et al., 1999). Laboratory and/or field measurements for soil permeability are recommended.

Unless existing soils meet the physical or chemical recommendations, the addition of an amendment is required. Stormwater manuals generally specify the amendment mix on a volume basis. MDE (2009) specifies the use of an amendment mixture that includes a loamy sand or sandy loam soil with 10% and 5% organic and clay contents, respectively. To achieve this amendment mixture, one of two mix ratios are recommended: 1) 60-65% loamy sand and 35-40% compost or 2) 30% sandy loam, 30% coarse sand, and 40% compost. LFUCG (2009) specifies a similar amendment mixture with 35-60% sand, 30-55% silt, 10-25% clay, and 1.5-4% organic content by volume. Both MDE (2009) and LFUCG (2009) also specify the use of a 0.6 m sand filter layer at the bottom of the filter bed media. This sand filter is shown in both stormwater manuals with rain gardens in which underdrains are

incorporated; however, it is not specified if the sand layer can be omitted if an underdrain is not used. IDNR (2009) specifies a similar amendment mixture (50%-60% sand, 20%-25% topsoil or sandy clay loam, and 20%-25% compost) with the main difference being the requirement of leaf-based compost. NCDENR (2009) specifies a much higher sand content in the amendment mixture than these other stormwater manuals. This mixture, by volume, is 85-88% sand, 8-12% fines (silt and clay), and 3-5% organic matter.

Many stormwater manuals recommend compost as the organic matter amendment because it improves the water holding capacity, porosity, bulk density and structure of the existing soils. Pitt et al. (1999) found that soils amended with compost decreased surface runoff by 5 to 10 times when compared with unamended soils. However, it is important to note that compost can increase levels of nitrogen and phosphorus in the effluent, which is the reason some stormwater manuals specify the use of leaf-based compost.

#### **2.2.4 Underdrains**

As rain gardens are infiltration-based stormwater controls, existing soils (as well as depth to a bedrock layer) play a significant role in determining if a rain garden will have the capacity to infiltrate the WQV. If existing soils are poorly drained and no underdrain is installed, then water that infiltrates through the rain garden filter bed will begin to “back-up” once it reaches the less permeable boundary created by the existing soil. This layer will extend the ponding period of the rain garden which can in turn affect the vegetation. In such situations where a low permeable layer is encountered, an underdrain is needed to improve subsurface drainage. NCDENR (2009) recommends the installation of an underdrain system only in cases where existing soils have an infiltration rate of less than 1.2 m d<sup>-1</sup>. LFUCG (2009) recommends an underdrain for all rain gardens. MDE (2009) does not require an underdrain if the infiltration rate of existing soils is greater than 0.3 m d<sup>-1</sup> yet recommends the use of an underdrain for all rain gardens. The underdrain typically outlets to the existing storm sewer or to a drainage ditch.

The amount and rate of water transported through the underdrain is controlled by its design. Underdrains that are horizontal and without risers move a greater amount of infiltrated water more quickly to the storm sewer system (Dietz and Clausen, 2005). However, even with horizontal underdrains, peak flows are reduced. Hunt et al. (2008) measured a mean peak flow reduction of 99% (outlet flow compared to inlet flow) for a rain



garden with a horizontal underdrain. Underdrains with risers (a 90° fitting that raises the outlet some height above the underdrain) create temporary zones of saturation, and as such, release less of the infiltrated waters to the storm sewer system (Li et al., 2009; Brown and Hunt, 2011). These saturated zones experience anaerobic conditions and thus have the potential to improve nitrogen removal from runoff via denitrification (Passeport et al., 2009). Stormwater in these underdrains slowly exfiltrates into the surrounding subsoil.

## **2.3 ATTRIBUTES OF HYDROLOGICALLY HIGHLY EFFECTIVE RAIN GARDENS**

Although each design component (e.g. pretreatment structure, above-ground storage, filter bed surface area, filter bed depth and composition, native plant types and densities, and underdrain structure) listed in Section 2.1 is important with regards to the effectiveness of an individual rain garden, highly effective rain gardens share some common characteristics. Table 2.3 contains a summary of the hydrologic performance of various rain gardens.

Selbig and Balster (2010) examined the effectiveness of four types of rain gardens in Madison, Wisconsin. Each rain garden had a different dominant vegetation type and soil type for infiltrating roof runoff. The treatments consisted of turf grass and sandy soils, prairie grass and sandy soils, turf grass and clayey soils, and prairie grass and clayey soils. Soils were excavated with a skid loader, and 10-15 cm of compost was placed on the bottom of the rain garden. The sandy soils rain gardens were in parallel and received runoff from the same roof. Likewise, the clayey soils rain gardens, which were at a different location, were in parallel and received runoff from the same roof. Each rain garden was sized such that its surface area was equivalent to 20% of the contributing drainage area (5:1 contributing to receiving scale). The design storm depth or WQV was not specified. The design pond depth was 15 cm. After four years of monitoring, the prairie grass and sandy soils rain garden as well as the turf grass and clayey soils rain garden were removed from the study so that all of the runoff from each respective roof flowed only to either the turf grass and sandy soils rain garden or the prairie grass and clayey soils rain garden. This modification resulted in a doubling of the contributing drainage to receiving area such that the ratios were 10:1 and

Table 2.3. Rain garden characteristics and hydrologic performance.

Location	C:R <sup>1</sup>	Underdrain	d <sub>f</sub> (m) <sup>2</sup>	Soil Type	h <sub>f</sub> (cm) <sup>3</sup>	Hydrologic Performance	Source
Haddam, Connecticut	12:1	Yes	0.6	Loamy sand	15	99% of inflow exited as subsurface flow	Dietz and Clausen (2005)
Greensboro, NC	20:1	Yes	1.2	Organic sand soil mixture	52	78% volume reduction volume; significantly more outflow during winter months	Hunt et al. (2006)
Villanova, PA	10:1	No	1.2	Sandy loam	52	Mean permeability 13 cm d <sup>-1</sup> at 12°C	Emerson and Traver (2008)
Charlotte, NC	16:1	Yes	1.2	Loamy sand		Peak flow reduction of 96% (rainfall depth < 40 mm)	Hunt et al. (2008)
Trondheim, Norway	20:1	Yes	0.5-0.6	Topsoil/sand mixture	15	Peak flow reduction of 42% entire study period; 27% during winter months	Muthanna et al. (2008)
Asheville, NC	6:1	Yes; 135 cm below surface	--	--	--	No outflow from 88% of rainfall events	Jones and Hunt (2009)
Brevard West, NC	9:1	Yes; 43 cm below surface	--	--	--	No outflow from 73% of rainfall events	Jones and Hunt (2009)
Lenoir, NC	22:1	Yes; 95 cm below surface	--	--	--	No outflow from 21% of rainfall events	Jones and Hunt (2009)
Brevard East, NC	14:1	Yes; 48 cm below surface	--	--	--	No outflow from 24% of rainfall events	Jones and Hunt (2009)
Alamance County, North Carolina	34:1	Yes; upturned elbow 30 cm below ground surface	0.6	80% expanded slate fines, 15% sand, 5% OM <sup>5</sup>	23	18% peak reductions	Passeport et al. (2009)

Table 2.3 cont'd.

Location	C:R <sup>1</sup>	Underdrain	d <sub>f</sub> (m) <sup>2</sup>	Soil Type	h <sub>f</sub> (cm) <sup>3</sup>	Hydrologic Performance	Source
Alamance County, North Carolina	34:1	Yes; upturned elbow 30 cm below ground surface	0.9	80% expanded slate fines, 15% sand, 5% OM <sup>5</sup>	23	14% peak reductions	Passeport et al. (2009)
Louisburg, North Carolina	22:1	Yes	0.5 – 0.6	Sandy loam	15	Median peak reduction of 96% and 90% (lower performance with an impermeable liner)	Li et al. (2009)
Silver Springs, Maryland	50:1	Yes	1.2	Sandy clay loam	30	Median peak reduction of 98%	Li et al. (2009)
College Park, Maryland	17:1	Yes	0.5 – 0.8	Sandy loam	10 - 34	Median peak reduction of 86%	Li et al. (2009)
Nashville, NC	23:1	Yes	0.6	Loamy sand	13 <sup>4</sup>	No outflow from 41% of events	Brown and Hunt (2011)
Nashville, NC	21:1	Yes	0.9	Loamy sand	15 <sup>4</sup>	No outflow from 45% of events	Brown and Hunt (2011)

<sup>1</sup>Contributing to receiving drainage area ratio

<sup>2</sup>Filter bed depth

<sup>3</sup> Average height of ponded water above filter bed

<sup>4</sup>Surface storage lacking due to construction and design errors

8:1 for the turf grass and sandy soil and prairie grass and clayey soil rain gardens, respectively. Note that underdrains were used.

During the study period, the lowest infiltration rate observed was  $7.1 \text{ mm h}^{-1}$  for the turf grass and clayey soil rain garden, and the highest infiltration rate was  $106.7 \text{ mm h}^{-1}$  for the prairie grass and sandy soil rain garden. Evaporation ranged from 12 to 25% with higher values recorded for the turf vegetated rain gardens. Even though precipitation depths were 35% higher than normal during the study period, the rain gardens were able to capture and retain 96% of all precipitation events. Even if no infiltration had occurred, these rain gardens were able to store about 90% of the precipitation as above-ground storage. Thus, in spite of the different infiltration rates from the soils, the rain gardens were highly effective in treating stormwater from the roofs. This effectiveness was attributed to available above-ground storage, alleviated compaction, and root penetration.

Li et al (2009) examined the hydrologic performance of six rain gardens located in College Park and Silver Spring, Maryland and Greensboro and Louisburg, North Carolina. The contributing to receiving drainage area ratio ranged from 50:1 to 17:1 (rain garden surface area of 2 to 6% of the contributing drainage area) with an average of 22:1 or 4.5%. Most of the contributing watersheds consisted of parking lots. The design storm used for each rain garden was not specified. Soils in the rain gardens were classified as sandy loam, sandy clay loam, or loamy sand. Filter bed depths ranged from 0.5 m to 1.2 m, and vegetation was predominately trees and shrubs. Li et al. (2009) found that the rain gardens reduced peak flows and runoff volumes in all cases, but the greatest reductions occurred with rain gardens having deeper filter beds ( $>0.9 \text{ m}$ ). Important to note was that all of the rain gardens had underdrains and as such no impermeable or low permeable layers were present to cause water to “back-up” or pond to greater depths and potentially overflow. The authors concluded that filter bed depth and absence of an impermeable layer (e.g. liner, clay layer, or bedrock) were the two most important design parameters in determining rain garden performance. Evapotranspiration also played an important role accounting for 19% of the stormwater. The rain gardens in this study were between one and three years old when monitored, so it is expected that evapotranspiration rates will increase over time as the trees mature (Angel et al., 2006; Taylor et al., 2009). Lastly, the authors concluded that for the larger, more infrequent events, the ratio of contributing to receiving drainage area played

a more significant role in the amount of stormwater that was captured and retained. Rain gardens with smaller ratios (i.e. greater percentages) tended to perform better.

Dietz and Clausen (2005) studied two rain gardens in Haddam, Connecticut that received stormwater from a roof. These rain gardens were sized such that the contributing to receiving drainage area ratio was about 12:1. The design storm was a 25 mm event. As the existing soil was loamy sand and had a sufficient infiltration rate ( $0.9 \text{ m d}^{-1}$ ), no amendments were added. The filter bed media was 0.6 m in depth and consisted of excavated and replaced native soil, hence, compaction was alleviated. Vegetation consisted of shrubs. Evapotranspiration was not estimated but was assumed to be the difference between inflow and outflow. A liner and underdrain were installed in each rain garden to allow more effluent sampling to assess water quality parameters such as nitrogen and metals. The authors found that nearly 99% of the inflow exited the rain gardens as subsurface flow via the underdrains; the remainder was overflow. This performance was achieved even though precipitation levels were 24% higher than normal.

Jones and Hunt (2009) examined the ability of four rain gardens to reduce stormwater temperatures. While hydrologic performance was not the primary focus of the study, the researchers noted that rain gardens with smaller contributing area to receiving area ratios retained a greater percentage of the inflow volume. With regards to filter bed depths, Brown and Hunt (2011) found that a 0.9 m filter bed depth reduced 24-hr runoff volumes to the LID goal (ratio of volume out to volume in for a 24-hr period of less than 0.33) for 44% of the storm events in the study as compared to 21% for a 0.6 m filter bed depth. Note that the rain gardens in Brown and Hunt (2011) were undersized.

In summary, rain gardens that are highly effective hydrologically seem to possess these characteristics:

- Low contributing to receiving drainage area ratio (i.e. small percentage of the watershed contributes stormwater to the rain garden) and hence shallower ponding depths,
- Deeper filter beds,
- Reduced soil compaction, and
- Absence of impermeable or lower permeable layer.

Evapotranspiration was also an important component, with estimates of its contribution to stormwater runoff treatment between 12 and 25% depending on the vegetation type (e.g. turf, prairie grass, or trees and shrubs). Thus while rain gardens are designed to dewater within a short time period, such as less than 48 hours, vegetation can exert a sizeable influence on stormwater fate.

## **2.4 ATTRIBUTES OF HYDROLOGICALLY MINIMALLY EFFECTIVE RAIN GARDENS**

Passeport et al. (2009) examined pollutant load reductions in two rain gardens in Alamance County, North Carolina. The rain gardens had a contributing to receiving drainage area ratio of 34:1. The rain gardens treated runoff from a parking lot and lawn. The existing soils were amended with an engineered filter mix of 80% expanded slate fines (Carolina stalite), 15% sand, and 5% organic matter. The filter bed was 0.6 m in depth in one cell and 0.9 m in the other. Vegetation consisted of Bermuda sod. The design storm was not specified. In an effort to enhance pollutant removal, an underdrain equipped with a riser was installed in each rain garden. The outlet elevation of the riser was 30 cm below the grass surface, and as such, the filter media remained fairly saturated. Because of this configuration, flow volumes were not significantly reduced; however, for most rain events the outflow volume was lower than the inflow volume. Peak flow reduction was low averaging 14 and 18% for the two rain gardens. Though nitrogen and phosphorus loads were significantly reduced, the reduction came at the expense of reduced stormwater infiltration.

Davis (2008) evaluated two rain gardens located in College Park, Maryland. The rain gardens had a contributing to receiving drainage area ratio of 45:1 and received runoff from asphalt parking lot. Existing soils were excavated and replaced with a mixture of sand (50% by volume), topsoil (30%), and compost (20%). The filter media depth was 0.9 m in one rain garden and 1.2 m in the other. In the latter rain garden (1.2 m filter bed depth), an anoxic sump was created using a mixture of 17 g of newspaper per kg of sand. This sump was located at the bottom of the rain garden, the purpose of which was to promote denitrification. The design storm was not specified, and the vegetation consisted of shrubs and herbaceous plants. For monitoring purposes, a liner was installed at the bottom of each

rain garden along with an underdrain. The underdrain was located below the filter bed and was wrapped in a nonwoven geotextile filter fabric to prevent clogging.

The rain gardens were effective in decreasing peak flows, time to peak, and volumes for only one-third to one-half of the study precipitation events. Complete capture of stormwater, meaning no outflow was observed, occurred for only 18% of the rainfall events. Even with the underdrains, overflows were estimated to have occurred during 15% of the rainfall events. The authors noted that for several rainfall events, effluent flow durations lasted several days. It is quite possible that the filter fabric around the underdrain reduced its effectiveness at dewatering the rain gardens. Hence, the moisture content of the rain garden filter media may have remained high thus decreasing storage volume available for subsequent rain events.

In summary, rain gardens that are less hydrologically effective seem to have more saturated filter bed media. While saturated filter bed media have been shown to enhance water quality through nutrient transformations (Passeport et al., 2009), this treatment comes at the expense of reduced hydrologic performance. Dietz and Clausen (2005) attributed the poor water quality treatment of the rain gardens in their study to rapid dewatering via an underdrain. The authors recommended the omission of underdrains in rain garden designs if pollutant load reductions were a priority.

## **2.5 OTHER FACTORS AFFECTING RAIN GARDEN HYDROLOGIC PERFORMANCE**

### **2.5.1 Evapotranspiration**

Evapotranspiration is a significant part of predevelopment hydrology, with estimates that it accounts for 40% of precipitation, and as such should be considered in rain garden design and performance assessment (Arnold and Gibbons, 1996). Of the previously examined studies, evapotranspiration rates were quite varied. Dietz and Clausen (2005) estimated that only 0.4% of the influent runoff left the rain garden as evapotranspiration, based on a water balance approach. Li et al. (2009) also used a water balance approach but estimated that evapotranspiration accounted for an average of 19% of the stormwater treated in the case of two of the six studied rain gardens. Evapotranspiration was not determined for the other four rain gardens in the study.

### **2.5.2 Seasonality**

Hydrologic effectiveness can also vary seasonally and over time. Emerson and Traver (2008) determined that the drawdown time of a rain garden depends on season and temperature. They determined that this variation was due to the physical characteristics of water as dynamic viscosity increases over 160% from 0°C to 38°C. The researchers concluded that this caused ponding time to vary between 80 and 120 hours over the course of a year. The authors recommended that designs account for lower infiltration rates during winter months by increasing the above-ground storage. Hunt et al. (2006) also found that seasonality affected the hydrologic performance of a rain garden in North Carolina. The mean outflow to runoff ratio was significantly higher during the winter months as compared to the spring, summer and fall months due in part to the lower evapotranspiration rates and higher water table levels.

### **2.5.3 Sediment**

One common recommendation among stormwater guidance documents was to minimize soil disturbances and/or implement a sound sediment and erosion control plan in the contributing drainage area (Hunt and White, 2001; Tschantz et al., 2003; Jaber et al., 2012). If construction occurs in the contributing drainage area and no sediment control is implemented, the rain garden will experience clogging before it is even in operation and its lifespan will be shortened. Consideration should also be given to including a forebay and/or grass filter into the rain garden design. Sediment, if present in high quantities in the contributing drainage area, can lead to clogging and failure of the rain garden. As such, its entry into the rain garden should be prevented. Clogging, both during construction and post-construction, has been identified as a primary cause of failure in rain gardens (Brown and Hunt, 2011; Wardynski and Hunt, 2012). Wardynski and Hunt (2012) noted that a rain garden, which received sediment from the construction of an adjacent parking lot, had a decreased hydraulic conductivity due to the formation of a fine layer of particles on its surface. Hydraulic conductivity is most often decreased near the rain garden inlet or in the lowest areas of the rain garden, where sedimentation occurs (Asleson et al., 2009; Jenkins et al., 2010). Jenkins et al. (2009), however, noted that even though the hydraulic conductivity near the inlet was decreased, overall hydrologic performance was not significantly impacted after nine years, suggesting that if sediment loading is not significant (e.g. in an unstable



watershed or downstream from construction with no sediment control), then the rain garden will continue to maintain positive hydrologic performance.

## **CHAPTER 3: MATERIALS AND METHODS**

### **3.1 PROJECT SITE**

The project site is located at the Coca-Cola Refreshments USA, Inc. (38.07681° N, 84.53942W) in Lexington, Kentucky (Figure 3.1). The project site is located in the Inner Bluegrass region, which is characterized by gently rolling hills, phosphate-rich soils, and limestone geology (McDowell, 1986; KGS, 2007). Because of the limestone or karst geology, the Inner Bluegrass region has many “sink holes, sinking streams, springs and caves” (KGS, 2007). The climate is humid and temperate with an average annual rainfall of 118 cm. Temperature 30-year normal values range from monthly highs of 32°C (January) to 86°C (July) with an annual average of 65°C (NCDC, 2002).

### **3.2 STORMWATER MANAGEMENT DESIGN**

In 2011, Coca-Cola Refreshments USA, Inc. was awarded \$189,090 through the Lexington Fayette Urban County Government (LFUCG) Stormwater Quality Projects Incentive Grant Program to construct a 880 m<sup>2</sup> rain garden with a 370 m<sup>2</sup> filter bed area and install a 47,430 L infiltration chamber and a rainwater harvest system with a 37,850 L tank to manage and treat rooftop and parking lot runoff (LFUCG, 2011) (Figure 3.2). The project was designed and constructed by Ridgewater, LLC (Eric Dawalt, president) and EcoGro (Jim Hanssen, president). As-built surveys indicate that 6,530 m<sup>2</sup> of impervious area (parking lot and rooftop) was treated by the stormwater management system. This value does not include the 410 m<sup>2</sup> that bypassed the inlet pipes.

#### **3.2.1 Contributing Drainage Areas**

Three distinct drainage areas contribute stormwater to the rain garden. One drainage area is a 2,680 m<sup>2</sup> section of the warehouse. The roof of this building is flat and is comprised of rubber sheeting. No rocks are present on the roof, as is sometimes the case with roofs on large commercial buildings. Over time, exposure to the elements is causing the roofing material to break-down;

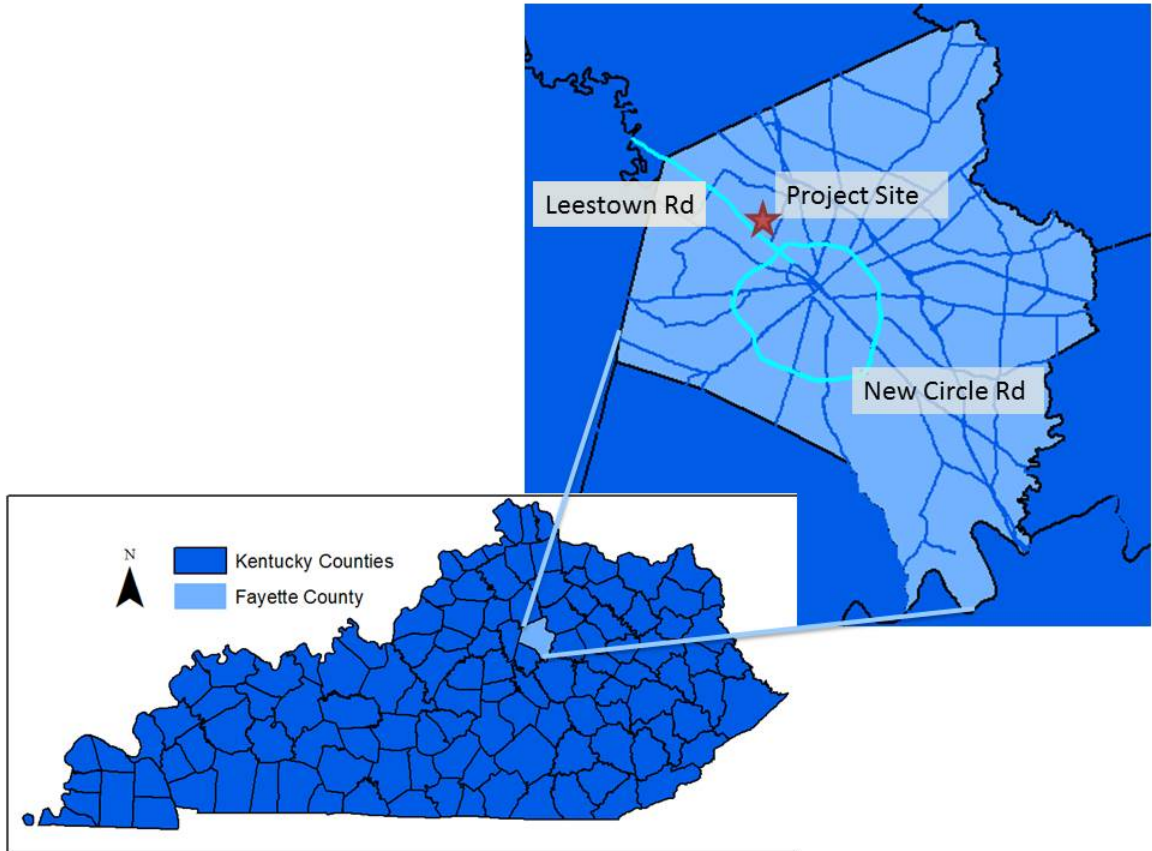


Figure 3.1. Project location map. The red star identifies the project location within Fayette County, Kentucky.

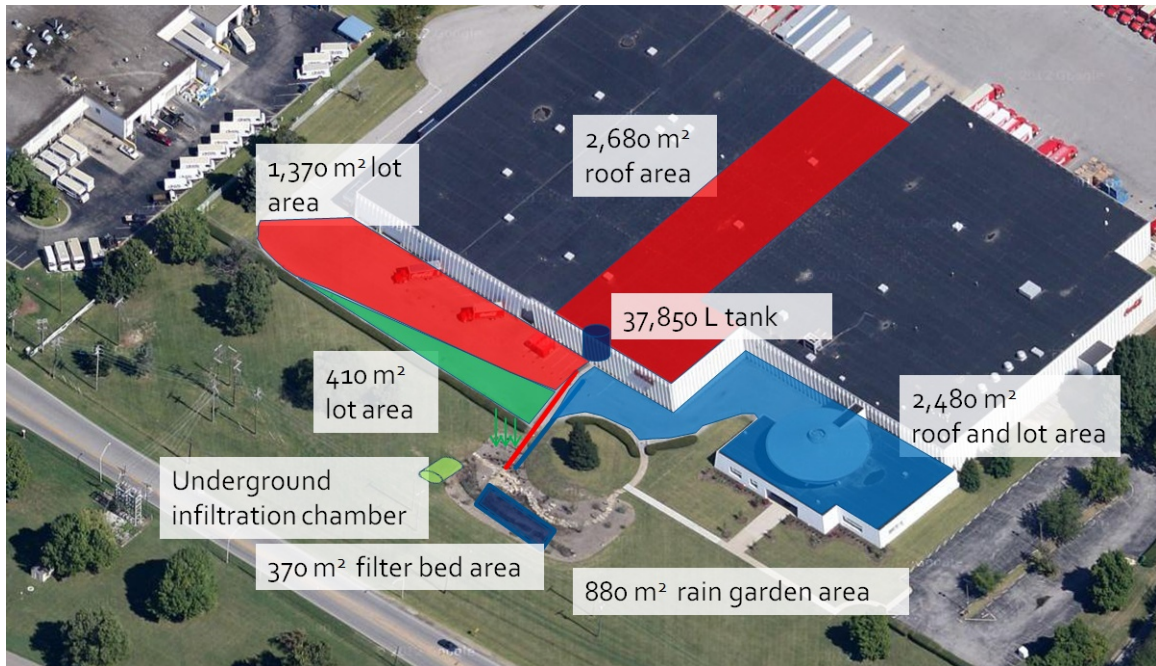


Figure 3.2. Stormwater management system at Coca-Cola Refreshments, USA in Lexington, Kentucky. Runoff from areas in red drain to the rain garden via the red pipe. Areas in blue drain to the rain garden via the blue pipe. The green area bypasses the pipes and drains to the rain garden as overland flow.

pieces are conveyed to the roof's drainage system during storm events. In addition to the roofing material debris, geese frequently roost on the roof where their feathers and goose droppings are also transported to the storm sewer system (Figure 3.3). Runoff from the warehouse roof first flows into the rainwater harvesting system before the excess amounts enter the rain garden.

### **3.2.2 Rainwater Harvest System**

The rainwater harvest system consists of three filters of decreasing grain size, a control panel, and a 37,850 L storage tank (Figure 3.4). The filters consist of a coarse filter (350  $\mu\text{m}$ ), a floating filter (100  $\mu\text{m}$ ) inside the storage tank, and a self-cleaning filter (5  $\mu\text{m}$ ) on the same skid as the control panel. Runoff from the warehouse roof (influent) enters the rainwater harvest system via the coarse filter. After exiting the coarse filter, stormwater flows to the right towards the first flush pipe. The pipe to the left of the coarse filter is a complete bypass of the system and remains closed; it is only for use if the rainwater harvesting system is in need of maintenance. The first flush (approximately 20 L) of a runoff producing rain event bypasses the storage tank and flows directly into the rain garden via the first flush pipe and the bypass pipe. Once the first flush event has bypassed the rainwater harvesting system, a float valve redirects runoff into the storage tank. Following the first flush, the tank is filled to about 30,280 L (not maximum capacity). Once the tank is full, all of the remaining runoff flows into the rain garden via a 30 cm HDPE smooth wall pipe which serves as the overflow. Following the storm event, stormwater in the first flush pipe is slowly released to the rain garden via the bypass pipe. Figure 3.5 shows an example of the stormwater once it has passed through the rainwater harvesting system for either storage in the tank or discharge to the rain garden. As seen in this figure, the water is quite clear and with no visible suspended particles.

Coca-Cola Refreshments USA, Inc. presently uses the harvested rainwater to wash delivery trucks and warehouse floors, flush one toilet, refill fork lift batteries, and irrigate the rain garden during times of drought and vegetation establishment. The control panel does not record water levels in the tank; however, it does show the instantaneous water level in the tank. Hence, water usage rates are not known. If the water level in the storage tank falls below 66 cm (approximately 4,820 L) before a subsequent rainfall event, municipal water is pumped into the tank to prevent the pump from



Figure 3.3. Goose feathers and droppings at the roof drain of the Coca-Cola Refreshments USA, Inc. warehouse rooftop.



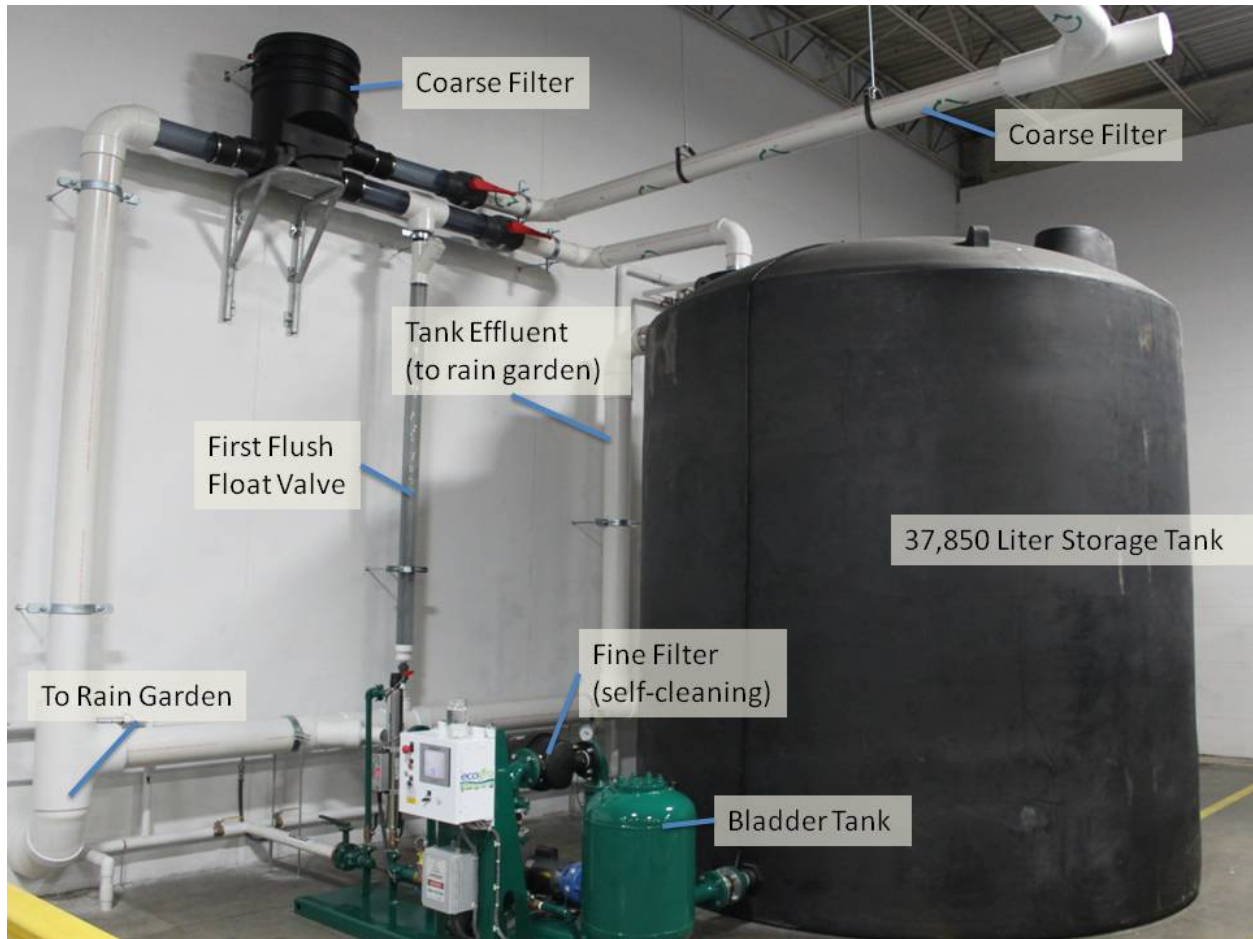


Figure 3.4. The rainwater harvesting system consists of three filters, a storage tank, and a control panel.



Figure 3.5. The mason jar contains water from the rainwater harvesting system.



running. Conversations with warehouse personnel at Coca-Cola indicate municipal water was seldom used during the project period.

### 3.2.3 Rain Garden

The rain garden was designed and constructed per guidance provided in the LFUCG Stormwater Manual (2009). An impervious area of 100% was used with the corresponding rainfall depth of 40.6 mm which yielded a WQV of 265 m<sup>3</sup> and a design ponding depth of 0.23 m using a coefficient of permeability of 0.73 m d<sup>-1</sup>. The coefficient of permeability used by the designer was much greater than the value of 0.15 m d<sup>-1</sup> specified in the LFUCG Stormwater Manual (2009). The designer also used a factor of safety of 1.25 when determining the rain garden surface area. A 370 m<sup>2</sup> filter bed was created by excavating the soil to a depth of 2.4 m. The bottom 1.2 m of soil was loosened with a backhoe and replaced while the top 1.2 m was amended. The amendment consisted of sand (25%), leaf compost (25%), and existing soil (50%). The design time to filter the WQV through the filter bed media was 1 day (3 days would be required if a coefficient of permeability of 0.15 m d<sup>-1</sup> was used).

It was expected that the act of loosening the bottom 1.2 m of soil in the filter bed would increase infiltration rates and storage and hence the long-term hydrologic effectiveness of the rain garden. Prior research on reclaimed mined lands has shown that loosening the soil to alleviate compaction has a significant effect on hydrology and vegetation (Angel et al., 2006; Taylor et al., 2009). Once constructed, the surface of the rain garden was covered with 5 cm of hardwood mulch to prevent erosion during the winter months and then planted with birch, redwood, flowering dogwood, red-twig dogwood, silky dogwood, serviceberry, itea, oakleaf hydrangea, little blue stem (native ornamental grass), tulips, and dogwood trees the following spring.

Stormwater enters the rain garden primarily through a 25 cm vitrified clay pipe and a 30 cm HDPE smooth wall pipe. Only the stormwater from the 30 cm pipe travels through the rainwater harvesting system. Stormwater from the 25 cm pipe does not. The rain garden also receives runoff from a small area (6% of the total impervious area) via overland flow from a portion of a parking lot. Before entering the rain garden, stormwater in the pipes enters two sump areas measuring about 30 cm x 30 cm x 30 cm each. These sumps are lower than the rain garden surface elevation with the sump for the 25 cm pipe about 17 cm below the ground surface and that for the 30 cm pipe about 5

cm below the ground surface. The sumps serve a dual purpose of preventing larger material such as sands and gravels from entering the rain garden as well dissipating energy to prevent the rain garden soils from eroding.

### **3.2.4 Infiltration Chamber**

When ponded waters in the rain garden reach a depth of 0.46 m above the ground surface, stormwater overflows into a 47,430 L infiltration chamber (Triton, Brighton, MI) via a stand pipe. The infiltration chamber is equipped with a pervious bottom to allow water to infiltrate into the surrounding soil. In lieu of gravel about 119,400 kg of recycled glass (about sand size) was placed around the infiltration chamber.

## **3.3 RAIN GARDEN HYDROLOGIC DATA ACQUISITION**

### **3.3.1 Topographic Survey**

Topographic surveys were conducted in November of 2011 using a Sokkia 530R Total Station equipped with a Carlson Explorer II handheld datalogger. Benchmarks established by the contractor were used while performing the topographic survey.

### **3.3.2 Soil Physical and Chemical Properties**

Samples of the existing soils and the amended soils were collected from the rain garden during the construction phase. Samples were collected in 190 L barrels. Four subsamples of each soil type (existing and amended) were sent to the University of Kentucky's Regulatory Services (UKRS) for analysis for pH, phosphorus (P), potassium (K), calcium (Ca), magnesium (Mg), zinc (Zn), organic matter content (OM), texture, plant available water, field capacity, and wilting point per standard methods. Soils were oven dried at 38°C and ground (2 mm sieve). Soil pH was determined using a 1:1 soil:water mixture per standard methods (Soil and Plant Analysis Council, 2000a). Available P, K, Mg, Zn was determined via Mehlich III extraction (Soil and Plant Analysis Council, 2000b; 2000c). Soil organic matter content was calculated from % C (Nelson and Sommers, 1982). Texture (percent sand, silt and clay) was determined via the micropipette method (Miller and Miller, 1987; Burt et al., 1993). Water holding capacity was determined via the pressure plate method (Topp et al., 1993).

### **3.3.3 Rainfall**

Rainfall data were collected with a Rain Collector II tipping bucket rain gage (Davis Instruments, Hayward, CA) equipped with a HOBO Pendant Event Data Logger (Onset Computer Corporation, Bourne, MA) (Figure 3.6). Unfortunately, large amount of the data were lost in part due to equipment failure. Because of this, rainfall data from United States Geological Survey (USGS) rain gages at Town Branch at Yarnallton Pike (gage number 03289200) and Cane Run Creek at Citation Boulevard (gage number 03288180) were used instead. These gages are located 5.1 km and 6.0 km from the project site, respectively, and 7.7 km from each other. Rainfall data were recorded at five minute intervals at these USGS gages. Three hour gaps were used to separate rainfall events. Rainfall events with a depth less than 4 mm were not used in the analysis since these events were not thought to be runoff producing (McCuen, 2005).

Because it is known that rainfall depths vary across a watershed, particularly in the summertime when convective storms occur, radar images of precipitation depths from the National Oceanic and Atmospheric Administration (NOAA) (available at: <http://nmq.ou.edu>) for storm events were examined. Additionally, runoff volumes (product of depth and area) were compared to inflow volumes (Section 3.4.1.1) and curve numbers (CN) were computed (Section 3.4.1.2) and compared to the CN of 98 for impervious areas.

### **3.3.4 Inflow**

Stormwater entered the rain garden through two pipes: 25 cm vitrified clay pipe and a 30 cm HDPE smooth wall pipe. Water level and average velocity were recorded every 10 minutes using two 4250 flow meters (one per pipe) (Teledyne-ISCO, Lincoln, NE) (Figures 3.6-3.7). Water level was measured using an integral pressure transducer and average velocity was measured using

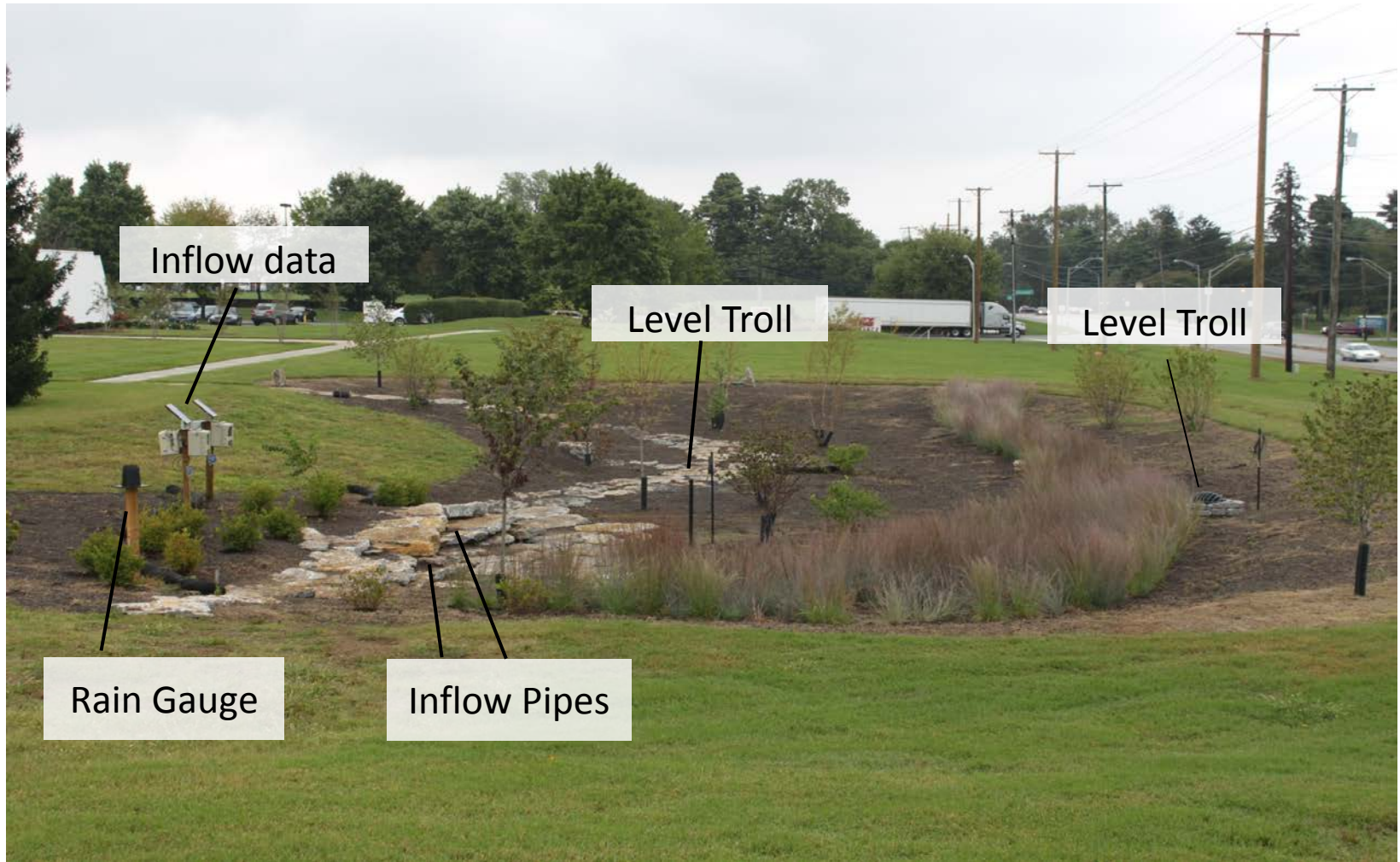


Figure 3.6. Layout of the instrumentation for measuring hydrologic performance of the rain garden.



Figure 3.7. Flow meters were used to determine stormwater inflow rates through the two inlet pipes.

Doppler technology. The pipe diameters were input into the flow meters, and discharge was computed using the area velocity method. Sporadic checks were conducted on the data by comparing the discharge computed by the flow meters with discharge estimates using Manning's equation (pipe slope of 1.91% for 30 cm pipe and 0.96% for the 25 cm pipe;  $n$  of 0.015).

Flow data were recorded from December 1, 2011 to May 8, 2013. For the period of July 18, 2012 to August 11, 2012, equipment failure meant no flow data were recorded for the 30 cm HDPE smooth wall pipe. During April and May of 2013, a groundhog (*marmota monax*) placed large amount of sediment in the 30 cm HDPE pipe; removal of the sediment did not dissuade the groundhog (Figure 3.8). To account for the missing flow data during these periods, an event-based flow volume relationship was developed between the 25 cm and 30 cm pipe.

### **3.3.5 Water Level**

The above-ground and subsurface water levels in the rain garden were monitored using a 1.07 m slotted well, which was installed at the deepest point in the rain garden, and a Level Troll 500 (34.5 kPa) pressure transducer (In-Situ Inc., Fort Collins, CO) which was located inside the well. The well was slotted from 0.30 to 1.07 m below the ground surface; the slots did not extend the full length of the well. A cap was placed on the bottom of the well to prevent the entry of soil into the well from the bottom. No holes were drilled into the cap, and as such, a small amount of water remained in the well at all times, regardless of the water table depth. Bentonite was placed around the well at the ground surface interface to prevent piping of water from the surface along the well wall. The pressure transducer recorded water level data at 10-minute intervals.

A 2.4 m deep well was installed in March 2013. The well was installed near the 1.07 m deep well. The deeper well was of similar configuration as the shallower well. The well was slotted from 0.3 m below the ground surface to 2.4 m below the ground surface. A Level Troll 500 (34.5 kPa) was installed at the bottom of monitoring well. While augering the soil for the well, soil moisture and texture was noted at varying depths. This was done to confirm the depth of the boundary between amended soil and existing soil and also to determine if water was perched at the boundary between the amended and existing soil layers.

To determine the amount of stormwater entering the infiltration chamber from the rain garden via the surface overflow, a second Level Troll 500 (34.5 kPa) was installed at the bottom of



an overflow sump. The elevation of the inlet pipe to the infiltration chamber in relation to the bottom of the overflow sump was known, as were the characteristics of the inlet pipe to the infiltration chamber. Hence, if the water elevation in the sump reached the threshold level, then inflow into the infiltration chamber could be computed. However, the water level in the overflow sump never reached this level during the study period.



Figure 3.8. Sediment placed in the 30 cm HDPE pipe by a groundhog.

### **3.3.6 Infiltration Rates and Coefficient of Permeability**

Both field and laboratory measurements were performed to determine infiltration rates and the coefficient of permeability for the rain garden at the project site. Information on field determined infiltration rates and coefficients of permeability are presented in Section 3.4.3. The coefficient of permeability (i.e. amount of water flowing through a defined area) is used by LFUCG (2009) and other stormwater manuals to size bioretention facilities and/or rain gardens.

#### 3.3.6.1 Laboratory Determined Infiltration Rates

Constant head permeameter tests were performed on five soil cores collected from inside and three from immediately outside the rain garden. These soil cores represent the amended and existing soil types, respectively. The top 5-8 cm of mulch (amended soils) and the top 5-8 cm of grass and soil (existing soils) were removed before the cores were collected. These soil cores were collected in June of 2013 after the monitoring component of the project had ended.

The constant head permeameter tests were conducted following the method described in Bowles (1970). To prepare the soil cores, which were located inside the double-ring permeameters, the following steps were completed. The permeameters were connected to a plastic tube that contained a glass tube inside. The top of the plastic tube was sealed using a rubber stopper. The glass tube went through the rubber stopper and extended above the plastic tube. The bottom of the glass tube was kept at a constant height to provide a constant head to the permeameter. Before the tests were conducted, the plastic tubes were filled with water and sealed. The clamp at the bottom of the tube (the permeameter influent clamp) was opened to allow water to begin flowing to the permeameter. The clamp was left open until water began to exit the bottom of the permeameter. At this point, the permeameter influent clamp was closed, the stopper removed, the plastic tube filled with deionized water, and the stopper reinserted. Then, the permeameter effluent tubes were clamped to prevent flow from exiting the permeameter. The permeameter influent clamp was left open for approximately 60 hours to completely saturate the soil core inside the permeameter.

To conduct the tests, the influent clamps were closed and the effluent clamps were opened. Graduated cylinders were placed beneath both the central effluent tube and the sidewall effluent tube. The influent clamp was then opened to allow water to flow through the permeameter. After



50 mL of water had flowed into the graduated cylinder, the influent clamp was closed. The time from when the influent clamp was opened to when it was closed was recorded.

Note that the double-ring permeameters collected effluent from the center of the core as well as effluent along the sidewalls of the core. If the volume of effluent through the outer tube was greater than 50% of the total effluent flow, then the sample was deemed unusable due to a majority of the flow being directed along the sidewalls and not through the soil column.

### **3.3.7 Soil Porosity**

The permeameter soil cores were also used to estimate porosity. The cores were placed in the permeameters and the bottom of the permeameter was left open. The cores were allowed to drain for 48 hours. At this point, the effluent tube was sealed so water could not enter the effluent tube from the soil core. A graduated cylinder was filled to 50 ml with deionized water. Water from the cylinder was gradually added to the soil core until it ponded on the surface. The soil core was covered, so no water could evaporate and left for 6-12 hours to allow water to further percolate into the core. This was repeated until water remained on the surface of the core after 12 hours. The volume of water that had been poured into the soil core was measured. This was divided by the total volume of the soil core to determine porosity. Porosity was used to determine subsurface storage available in the amended soil layer.

### **3.3.8 Soil Moisture**

An EnviroSCAN soil moisture sensor (Sentek Sensor Technologies, Stepney, SA, Australia) equipped with a Campbell Scientific datalogger (Logan, UT) measured soil moisture for a 3-month period, from March 2013 through April 2013 (Figure 3.9). The sensor was installed near the lowest part of the rain garden approximately 3 m from the well. Soil moisture data were collected at 1-minute intervals at depths of 10, 30, 50, 70, 90, 110, and 130 cm as measured from the ground surface.

### **3.3.9 Evapotranspiration**

Evapotranspiration was not directly measured. Instead, 24-hr rates were computed using a spreadsheet model developed by the University of Kentucky Agriculture Weather Center along with weather data from the National Climatic Data Center. This spreadsheet was based on Raes (2009).

The following weather data were required for the model: daily maximum temperature ( $^{\circ}\text{C}$ ), daily minimum temperature ( $^{\circ}\text{C}$ ), 24-hr average wind speed ( $\text{km h}^{-1}$ ), 24-hr average dew point temperature ( $^{\circ}\text{C}$ ), 24-hr average amount of cloud cover (%), latitude (decimal degrees), and Julian day. Reference evapotranspiration was computed using equation 3.1.



Figure 3.9. An EnviroScan soil moisture sensor with a Campbell Scientific datalogger (shown) were used to collect soil moisture data at seven vertical intervals.

$$ET_0 = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T + 273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)} \quad (\text{eqn. 3.1})$$

where

- $ET_0$  = reference evapotranspiration ( $\text{mm d}^{-1}$ )
- $R_n$  = net radiation at crop surface ( $\text{MJ m}^{-2} \text{d}^{-1}$ )
- $G$  = soil heat flux density ( $\text{MJ m}^{-2} \text{d}^{-1}$ )
- $T$  = mean daily air temperature at 2 m height ( $^{\circ}\text{C}$ )
- $u_2$  = wind speed at 2 m height ( $\text{m s}^{-1}$ )
- $e_s$  = saturation vapor pressure (kPa)
- $e_a$  = actual vapor pressure (kPa)
- $e_s - e_a$  = saturation vapor pressure deficit (kPa)
- $\Delta$  = slope vapor pressure curve ( $\text{kPa } ^{\circ}\text{C}^{-1}$ )
- $\gamma$  = psychrometric constant ( $\text{kPa } ^{\circ}\text{C}^{-1}$ )

The variable  $R_n$  is the difference between short-wave radiation and long-wave radiation. Short wave radiation is calculated using solar radiation data, or recommended constants if no solar radiation data are available, and percent cloud cover. When the time-scale for estimating evapotranspiration is greater than 24 hours, soil heat flux density,  $G$ , becomes zero. Mean daily air temperature is assumed to be the average between the daily maximum and minimum temperatures. Saturation vapor pressure is calculated from the daily maximum and minimum temperatures while actual vapor pressure is calculated from the dew point temperature. The slope of the vapor pressure curve is also calculated using average daily temperature. Finally, the psychrometric constant is calculated using atmospheric pressure, which for the spreadsheet model is assumed to a standard pressure of 101.23 kPa. This differs from Raes (2009) who determined atmospheric pressure for a site rather than assuming a constant value.

Once reference evapotranspiration was obtained, it was multiplied by a crop coefficient, to determine the actual evapotranspiration rate. An area weighted monthly crop coefficient was determined from Allen et al (1998). Crop coefficients were determined for the initial part of the growing season, the growing season, and the end of the growing season. Crop coefficients for

ornamental plants have not been determined, so crop coefficients for cereal grasses, almond trees, and berry bushes were used for little bluestem, medium and small trees, and shrubs respectively. An overall area weighted crop coefficient was determined by multiplying each crop coefficient by the area that it occupied and dividing the summation by the total area.

### **3.3.10 Photo-documentation**

To help verify the water level data in the well, time lapse photographs were taken in June of 2013 using Timelapse Plant Cams (Wingscapes, Alabaster, AL). Refer to Appendix A.

## **3.4 RAIN GARDEN HYDROLOGIC DATA ANALYSIS**

### **3.4.1 Rainfall**

Rainfall depth, duration, average intensity, and 30-min peak intensity were computed for each storm event with a rainfall depth greater than 4 mm. Note that runoff was generated for storm events less than 4 mm, however these smaller events were excluded from the analysis. Since rainfall data were not collected at the project site, it was important to determine which nearby but offsite rain gage(s) to use to assess the hydrologic performance of the rain garden. Two methods, in addition to examining radar patterns as described in Section 3.3.3 were used.

#### 3.4.1.1 Runoff Volume Comparison

Using the volumes of inflow and the contributing drainage area, precipitation depths were computed for each storm event. The resulting precipitation depths were compared to the rainfall depths recorded at the two USGS rain gages. If the difference between the computed rainfall depth and the rainfall depth recorded at a USGS gage for a storm event was greater than 10 mm, then data from that gage were not used for the storm event. If both gages had differences greater than 10 mm, then an average between the two gages was used. In such instances, averages for duration and intensities were also computed.

#### 3.4.1.2 Curve Number

The CN method was used to verify the rain gage selection. Since the contributing drainage area consisted of impervious surfaces (e.g. roof tops and parking lots), a CN of 98 was expected (McCuen, 2005). For each storm event, a CN was calculated using equations 3.2 and 3.3.

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad (\text{eqn. 3.2})$$

$$S = \frac{25,400}{CN} - 254 \quad (\text{eqn. 3.3})$$

The variable Q is runoff depth (mm), P is precipitation depth (mm),  $I_a$  is initial abstraction (mm), and S is maximum soil retention (mm). An initial abstraction of  $0.2 \cdot S$  was assumed. If the CN for a storm event was lower than 90 or equal to 100, then that storm event was removed from the analysis.

### 3.4.2 Inflow

Descriptive statistics (maximum, minimum, mean, median, and standard deviations) were computed for the inflow data for the hydrologic parameter total volume and peak flow. Peak flow data from the 30 cm pipe were not included in the analysis for instances when the flow data were estimated using the 25 cm flow data.

### 3.4.3 Infiltration

Descriptive statistics (maximum, minimum, mean, median, and standard deviations) were computed for the field determined infiltration rates and coefficients of permeability.

#### 3.4.3.1 Field Determined Infiltration Rates

Water level data in the well (Section 3.3.5) were used to determine infiltration rates for each storm event. Infiltration rates were computed using the rate of falling head as described in Selbig and Balster (2008) with the exception that these infiltration rates were determined for subsurface water levels as no ponded or above ground water was recorded during the monitoring period. Once inflow ceased, infiltration rates were computed from this point until the point at which there was less than a 1.5 cm drop in a 10 minute period (less than  $9 \text{ cm h}^{-1}$ ). This end point was chosen as it seemed to consistently represent the inflection point between quick infiltration and steady state conditions. For example, a starting time of 4:26 with a depth of 0.71 m and an ending time of 5:26

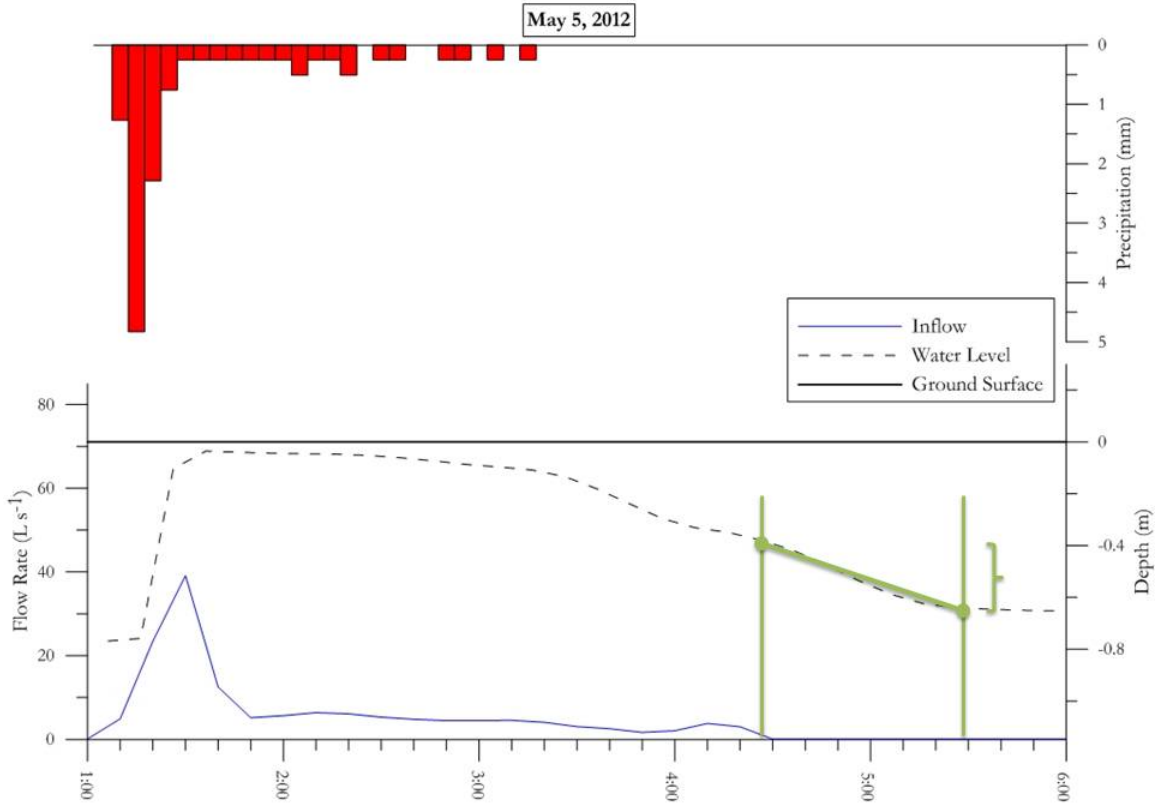


Figure 3.10. Infiltration rates were determined by noting the point of inflow cessation to the point of inflection for the water level.

with a depth of 0.449 m were used for the storm event occurring April 5, 2012 (Figure 3.10). The starting point is the water level point closest to the time inflow ceased. The ending point represents the occurrence of a 1.5 cm depth difference in a 10 minute period (e.g. 0.449 m at 5:26 to 0.439 m at 5:36). Multiple infiltration rates were computed for a single storm event (with gaps in the rainfall) in cases where the minimum infiltration rate ( $9 \text{ cm h}^{-1}$ ) was reached more than once.

In some instances, infiltration rates were not computed for the corresponding storm event. Such instances occurred when the rate of inflow was so low that infiltration rates remained fairly steady (i.e. no rapid increase and then decrease in water level). Also, if both flow meters were not functioning properly, infiltration rates were not computed as it was not known when inflow ceased.

#### 3.4.3.2 Field Determined Coefficient of Permeability

Using field determined infiltration rates and Darcy's Law, it was anticipated that a range of coefficient of permeabilities could be determined for the rain garden (Schwartz and Zhang, 2003) as shown in equation 3.4, provided water ponded.

$$Q = kA \frac{\Delta h}{\Delta l} \quad (\text{eqn. 3.4})$$

The variable  $Q$  represents the vertical flux of the stormwater through the filter bed ( $\text{m}^3 \text{ hr}^{-1}$ );  $k$  is the coefficient of permeability of the filter bed ( $\text{m hr}^{-1}$ ),  $A$  is the filter bed area ( $\text{m}^2$ ),  $\Delta h$  is the change in head of ponded water (m), and  $\Delta l$  is the media depth (m). Because ponding in the rain garden was not recorded during the monitoring period, cross-sectional area could not be determined, and hence neither could the coefficient of permeability. As such, an assumption was made that the maximum coefficient of permeability for a storm event was controlled by the inflow as the rain garden could not infiltrate water faster than it entered. These maximums are likely higher than actual values. If the soil was as permeable as the theoretical maximum coefficient of permeability indicated, there would be little to no increase in water level.

### 3.4.4 Above-ground Storage

A stage-volume curve was developed to determine the above-ground storage (refer to Section 3.4.4.1). This curve was used to determine the maximum above-ground storage which is just before water begins to overflow into the infiltration tank.

#### 3.4.4.1 Stage-Volume Development

An as-built survey was performed in November of 2011 following construction and planting. This survey was used to construct a topographic map of the rain garden for use in developing a stage-volume relationship. To develop this relationship, a flat surface was created in ArcMap version 10.0. The volume of the rain garden was determined for 3 cm increments in elevation with the deepest point of the rain garden representing the datum. Volume (y-axis) was plotted against stage (x-axis) in Microsoft Excel, and the data were fitted with a trendline. Equation 3.5 relates water level height above the ground surface of the rain garden ( $s$ , cm) to above-ground storage volume ( $v$ , L).

$$v = 40s^2 + 616s + 3322 \quad (\text{eqn. 3.5})$$

This curve was used to model hydrologic performance under LFUCG design recommendations. When the water level in the rain garden was approximately 46 cm above the ground surface, water would then begin flowing into the outlet structure and to the infiltration chamber. Refer to Appendix B for the topographic survey and stage-volume graph.

#### 3.4.4.2 Water Level Verification

During the latter part of the monitoring period, a rain event occurred (March 24, 2013) in which water was observed to pond at a depth of 2.5 cm; however, the pressure transducer in the well indicated that the water level was 14 cm below ground surface (Figure 3.11). Pondered water was also photographed by Eric Dawalt (the designer and builder) on March 18, 2013 but the depth of pondered water could not be determined from the photograph (Figure 3.12). At the time of the photograph, the pressure transducer recorded a pondered depth of 4 cm. To ensure that there was no error in the pressure transducer, a calibration of the level loggers was performed (Appendix C). The test of the pressure transducer revealed that it was recording water levels accurately.





Figure 3.11. Ponded water was observed in the rain garden on March 24, 2013.



Figure 3.12. Ponded water was observed in the rain garden on March 18, 2013.

Further verification of the water level in the rain garden during times of ponding was obtained by installing a Plant Cam in the rain garden. The Plant Cam took photos of the rain garden, at the location where the pressure transducer was installed, at 5-minute intervals. A survey rod was affixed to a post near the well in order to include a scale in the photograph. Unfortunately, the photographs produced by the Plant Cam were of insufficient quality to read the water level on the survey rod. But, the photographs did show how the pressure transducer responded to ponding. The Plant Cam captured 2 hours and 50 minutes of inflow for a storm event that had ponded water (June 26, 2013), the last 25 minutes of ponded water for another storm event (June 27, 2013), and a third storm event with ponded water (June 27, 2013) (Appendix A). For the storm on June 26, 2013, ponding appears to peak at a time of 18:38 and at a depth of 36 cm. At this same time, the ponding depth recorded by the pressure transducer was 19 cm. For the June 27, 2013 event, ponding appeared to peak at 14:50 and at a depth of 9 cm. The pressure transducer indicates the peak water level occurred at 15:03 with no ponding; the water level was 19 cm below the ground surface. For the third event, which occurred on June 27, 2013, the Plant Cam showed that the inflow was completely infiltrated into the soil in one hour. This infiltration rate is equal to the change in water level read by the pressure transducer over this same period. Based on the photographs, it appears that when ponding peaks, the pressure transducer will read a water level about 28 cm lower. Computed infiltration rates mirror the rates seen in the photographs.

The highest recorded ponding depth during the monitoring period was 15 cm. Even accounting for an offset of 28 cm, the maximum water level in the rain garden would not have reached the outlet structure to the infiltration chamber. Therefore, 100% of the inflow into the rain garden was infiltrated or evapotranspired. The infiltration chamber did not infiltrate stormwater during the monitoring period.

#### **3.4.5 Soil Moisture**

Graphs of soil moisture at the various intervals were developed and visually compared to the water level data in the well. These data were used to evaluate changes in moisture levels over time.

#### **3.4.6 Hydrologic Parameter Comparison**

A spreadsheet model was developed to determine the number of times the water level in the rain garden would have reached the outlet if LFUCG (2009) design parameters were used. This

value was then compared to the number of times the water level in the rain garden actually reached the outlet. Instantaneous flow data from the two inlet pipes were input at each 10-minute time step. These flows were multiplied by the 10-minute time step to determine the influent volume over each time step. The volume of water stored above-ground was used with the stage-volume curve that was developed for the rain garden to determine the ponding depth. Infiltration was approximated using Darcy's Law. A value of  $0.3 \text{ m d}^{-1}$  was used for hydraulic conductivity, per LFUCG guidelines; filter bed area was  $370 \text{ m}^2$ ; and hydraulic gradient was approximated by the change in water depth (above the rain garden surface) divided by the filter bed media depth of 1.2 m. To determine the volume of ponded water at each time step, inflow for the current time step ( $t_i$ ) was added to the above-ground water volume from the previous time step ( $t_{i-1}$ ) and infiltration volume was subtracted from the previous time step. This process was repeated for all inflow data available for the study.

Note that this spreadsheet model does not account for effluent through the outlet structure. This spreadsheet model also does not determine the level of the water table within the rain garden. The primary purpose of the spreadsheet model was to determine the number of storm events that likely would have produced outflow via the outlet structure.

## CHAPTER 4: RESULTS AND DISCUSSION

### 4.1 SOIL PHYSICAL AND CHEMICAL PROPERTIES

Soil physical and chemical properties for both the existing soil (ES) and amended soil (AS) are presented in Tables 4.1 and 4.2. The existing soil was classified as clay based on the USDA soil textural triangle; the amended soil was classified as a sandy loam. The lower pH and higher clay content of the ES indicated that the ES was likely fill material rather than native topsoil. Soil texture is a driving factor in the infiltration capability of the rain garden (WEF, 2012). The texture of the amended soil is a determinant in how well stormwater will infiltrate through the filter bed while the texture of the underlying or existing soil factors into how well water will exfiltrate from the rain garden and into the surrounding soil. Dunne and Leopold (1978) noted that the minimum saturated hydraulic conductivity expected for a sandy loam is  $2.6 \text{ cm h}^{-1}$  while it is less than  $0.1 \text{ cm h}^{-1}$  for clay. The AS had higher sand and organic matter contents as compared to the ES. Higher sand contents, and hence lower clay contents, are desirable in rain gardens as such sandy soil textures promote infiltration. The sand and clay contents of the AS are within the ranges recommended by LFUCG (2009) of 35-60% sand and 10-25% clay; however, the silt content is slightly lower than the recommended range of 30-55%. The higher OM content in the AS soil is promising as OM increases available moisture storage capacity, especially in soils with a clay content between 13 and 20% (Jamison and Kroth, 1958).

One item worth noting is the standard deviations associated with the sand and silt textural classes. The standard deviations are larger for the AS soils as compared to the ES soil. This difference is likely due to the method of mixing the amendment. The contractor mixed the amendment components (sand, compost and existing soil) using an excavator at the project site. In this case, the mixing appears well-done; however, this may not always be the case when mixing soil amendments onsite. For example, Hunt et al. (2006) found small zones of saturation caused by thin clay lenses in a rain garden in North Carolina. Such zones, while potentially beneficial to water quality (e.g. anoxic conditions for denitrification) would reduce infiltration rates though the extent of which is not known.

Table 4.1. Amended (AS) and existing (ES) soil physical properties.

Sample Type	Organic Matter (%)	Soil Texture	Sand (%)	Silt (%)	Clay (%)	Plant Available Water (%)	Field Capacity Water (%)	Wilting Point Water (%)
ES-1	0.6	clay	11	36	54	21	51	29
ES-2	0.5	clay	10	37	53	20	49	29
ES-3	0.8	clay	12	35	53	22	50	28
ES-4	0.5	clay	10	37	53	22	52	30
Mean±Stdev	0.6±0.2	clay	11±0.7	36±0.8	53±0.3	21±1.1	50±1.0	29±0.8
AS-1	6.1	sandy loam	57	27	17	14	27	13
AS-2	5.7	sandy loam	55	28	17	14	26	12
AS-3	6.3	sandy loam	59	24	17	12	23	11
AS-4	6.1	sandy loam	64	21	15	12	23	11
Mean±Stdev	6.1±0.2	sandy loam	58±3.9	25±3.1	16±0.8	13±1.2	25±2.1	12±0.9

Table 4.2. Amended (AS) and existing (ES) soil chemical properties.

Sample Type	pH	Total N (%)	P (kg ha <sup>-1</sup> )	K (kg ha <sup>-1</sup> )	Ca (kg ha <sup>-1</sup> )	Mg (kg ha <sup>-1</sup> )	Zn (kg ha <sup>-1</sup> )
ES-1	4.9	0.1	529	186	3,661	508	1
ES-2	4.7	0.1	520	203	3,526	524	1
ES-3	5.1	0.1	358	232	5,306	613	4
ES-4	4.7	0.1	506	197	3,621	508	1
Mean±Stdev	4.8±0.2	0.1±0.0	478±72	205±17	4,028±761	538±45	2±1
AS-1	7.5	0.2	205	256	7,801	450	6
AS-2	7.4	0.1	170	252	7,927	415	5
AS-3	7.6	0.2	214	229	8,192	462	7
AS-4	7.6	0.2	192	254	7,840	421	
Mean±Stdev	7.5±0.1	0.2±0.0	195±17	248±11	7,940±157	437±20	6±1



Although water quality analysis was not a part of the project, soil chemical properties were assessed to see how the AS compared to the ES. Following amendment, the soil changed from slightly acidic to neutral/slightly basic. Much of the underlying bedrock in this area is limestone ( $\text{CaCO}_3$ ). It is likely that the sand added to the amended soil was derived from limestone. This is apparent by the increased presence of Ca within the AS which was about twice that of the ES. The addition of a limestone derivative increased pH. WEF (2012) states that a pH less than 6 can result in the release of metals sorbed to oxides in the soil. Hence, the increase in pH is likely beneficial from this stand point, particularly as runoff to the rain garden is partially from parking lots. Another important item to note is that by amending the soil, the resulting soil P concentrations were reduced by about 60%. Such a reduction is likely beneficial especially considering Kentucky's large P contributions to waterways such as the Gulf of Mexico (Alexander et al., 2008).

#### **4.2 RAINFALL CHARACTERISTICS**

Data from a total of 156 storm events were collected during the one-year project monitoring period (May 2012 to May 2013). A total of 77 storm events were eliminated because they had a total rainfall depth less than 4 mm. Of these 79 remaining storm events, 23 were eliminated based upon the expected depth, as computed based upon inflow data and drainage area, or yielded a CN outside the project accepted limit ( $\text{CN} > 90$ ;  $\text{CN} \neq 100$ ). Table 4.3 contains the descriptive statistics of the rainfall characteristics depth, duration, average intensity, 30-minute intensity, time since prior rainfall event, and depth of the prior rainfall event for the 56 storm events which were used to evaluate the rain garden. Appendix D contains the values for each of the 56 storm events. Rainfall depths ranged from 4 to 56 mm with an average value of 17 mm. As seen in Figure 4.1, rainfall depths tended to be greatest during winter and early spring months. The majority of the storm events had rainfall depths between 10 and 20 mm (Figure 4.2). For the monitored period, monthly rainfall depths were largely below the 30-year normal values (NCDC, 2002) for 2012 with the exceptions of July, September and December (Figure 4.3). In 2013, monthly rainfall depths were generally greater than the 30-year normal values. The largest rainfall event, normalized over a 24-hr period was 64 mm which is a 1-yr event (Table 4.4) (KDNREP, 1979).



Rainfall durations and intensities displayed large variations throughout the monitoring period. Rainfall durations ranged from 0.1 to 19.5 hours with a mean value of 7.2 hours. Storm events were longest during the winter months when frontal storms were more prevalent (Figure 4.4).

Table 4.3. Storm event characteristics for the monitored period.

	Duration	Depth (mm)	Average Intensity (mm h <sup>-1</sup> )	30 Minute Peak Intensity (mm h <sup>-1</sup> )
Mean±Std. Dev.	7.2±4.9	16.9±10.8	4.7±8.4	11.7±7.2
Median	5.8	15.0	2.8	10.2
Maximum	19.5	56.0	60.2	31.2
Minimum	0.1	4.0	0.4	1.3

Table 4.4. Rainfall depths for 24-hr recurrence intervals for Fayette County, Kentucky.

Frequency (yr)	24-hr Rainfall Depth
1	66
2	79
5	97
10	109
25	130

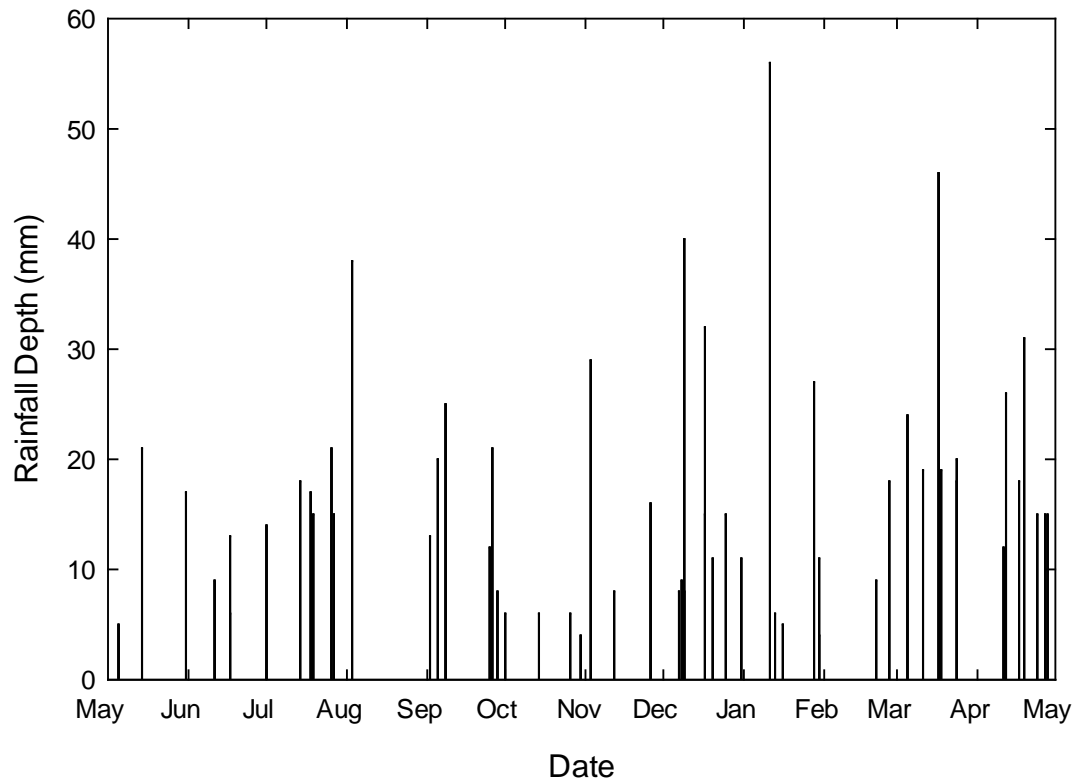


Figure 4.1. Rainfall depths for the project monitoring period.

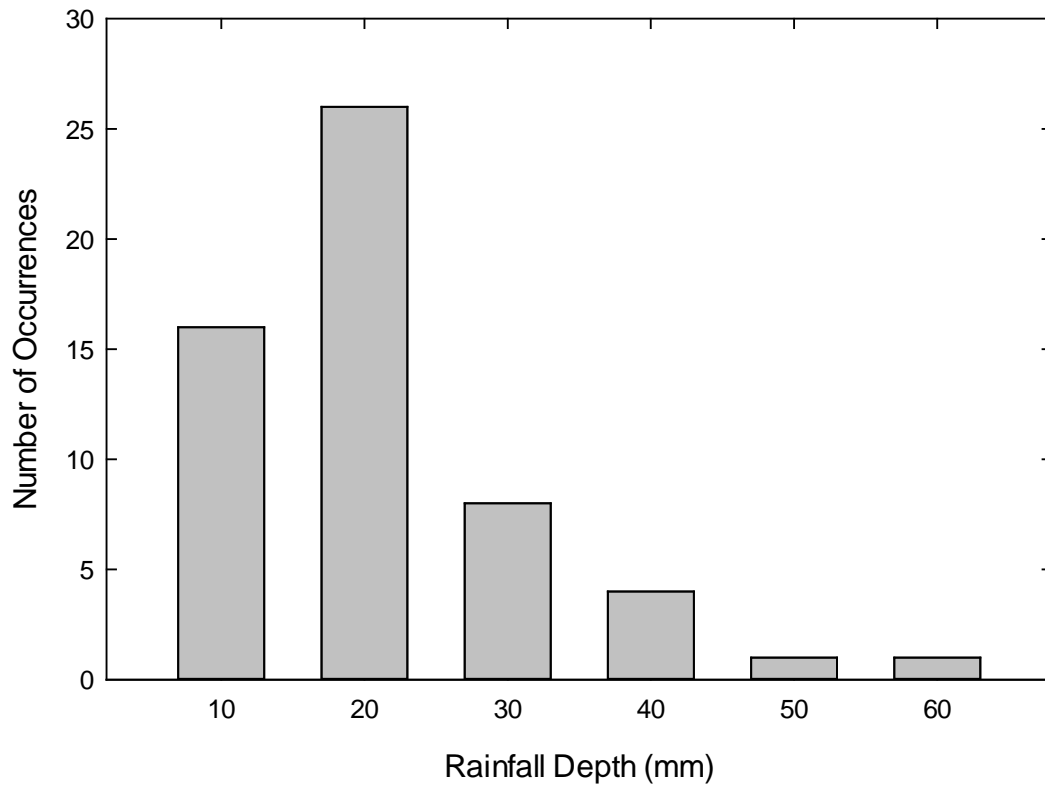


Figure 4.2. Rainfall event frequency for the monitored period.

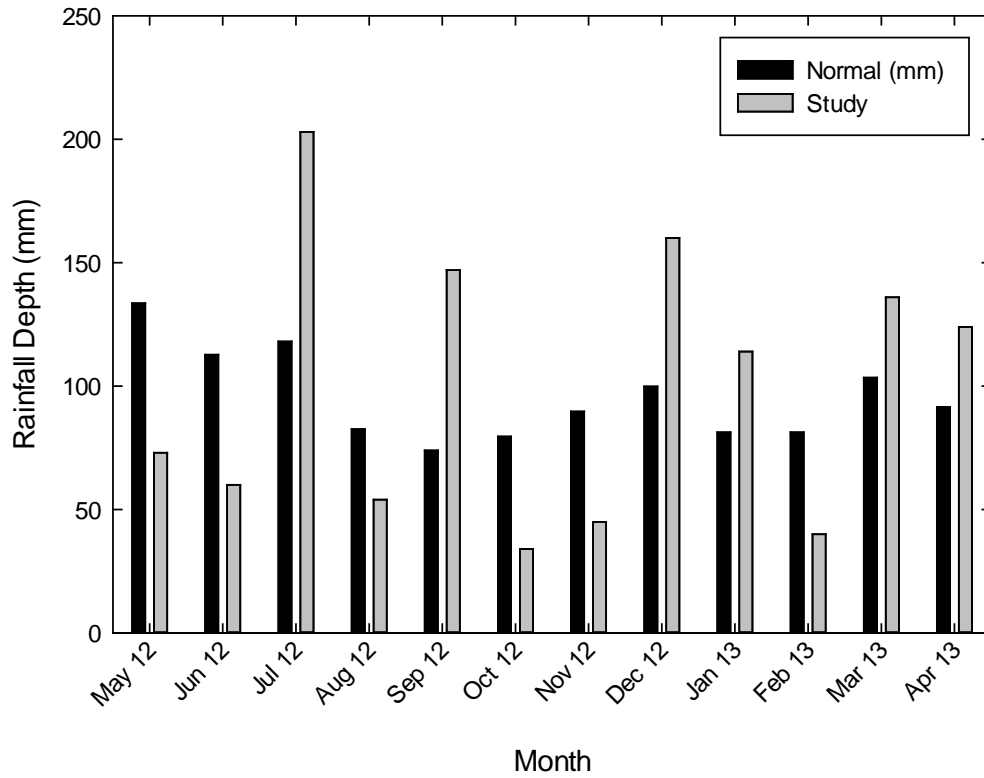


Figure 4.3. Monthly rainfall depths for the project monitoring period as compared to 30-year normal values.

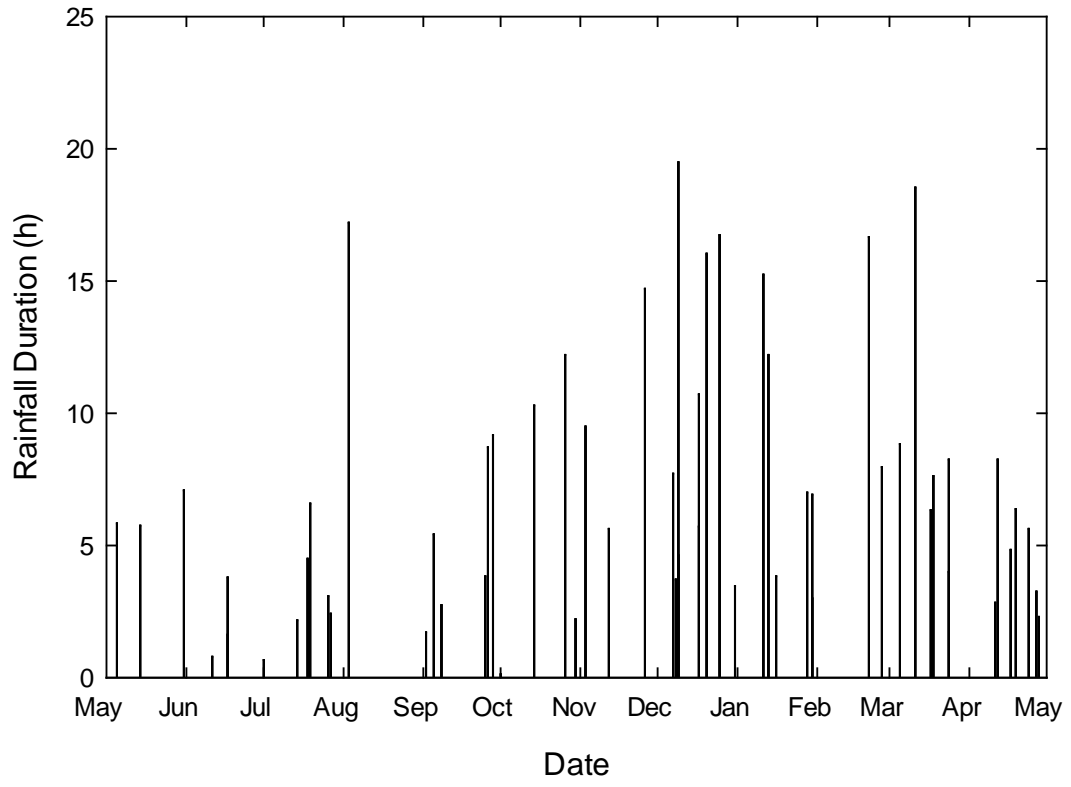


Figure 4.4. Rainfall durations for the project monitoring period.

Average intensities were highest during the summer months and lowest during the winter months (Figure 4.5). These results were expected as convective storms are typical during the summer months while frontal storms are common in the winter months (Warner et al., 2010).

### **4.3 INFLOW**

Table 4.5 contains a summary of the inflow volumes and peak inflows for the monitored period. Refer to Appendix E for hydrographs and Appendix F for hydrograph parameters for each storm event. Note that while inflow volumes were computed for each storm event, peak flows were not. Peak flows were only computed if flow data were collected for both the 25 cm and 30 cm inlet pipes. As seen in Table 4.5, the rain garden captured large amounts of runoff during the study period. It is important to note the rain garden was never overtopped during the study period. Hence, all inflow was either infiltrated, evapotranspired or stored. The range of peak inflows is similar to those reported by Hunt et al. (2008). And since the rain garden was not equipped with an underdrain system meaning inflow did not flow out of the rain garden to a storm sewer system, peak flow reductions were 100% for all of the monitored storms.

One unknown parameter was the volume of inflow captured and stored by the rainwater harvesting system during the study period. The maximum amount of stormwater the rainwater harvesting system can store is 25,460 L as the storage tank maintains a minimum volume of 4,820 L and begins to overflow at 30,280 L. If the tank was emptied to its maximum point before each of these storms, it would have been possible to store (and use) about 1,077,305 L of rainwater (23% of the inflow). However, it is unlikely that the rainwater harvesting system substantially reduced inflow volumes to the rain garden, at least for the analyzed storm events (>4 mm, CN>90). The average time between storm events used in this study was 2.9 days making it unlikely Coca Cola used much of the harvested rainwater. Also, the average CN for these storm events was 97 which indicate minimal amounts of rainfall were stored in the tank.

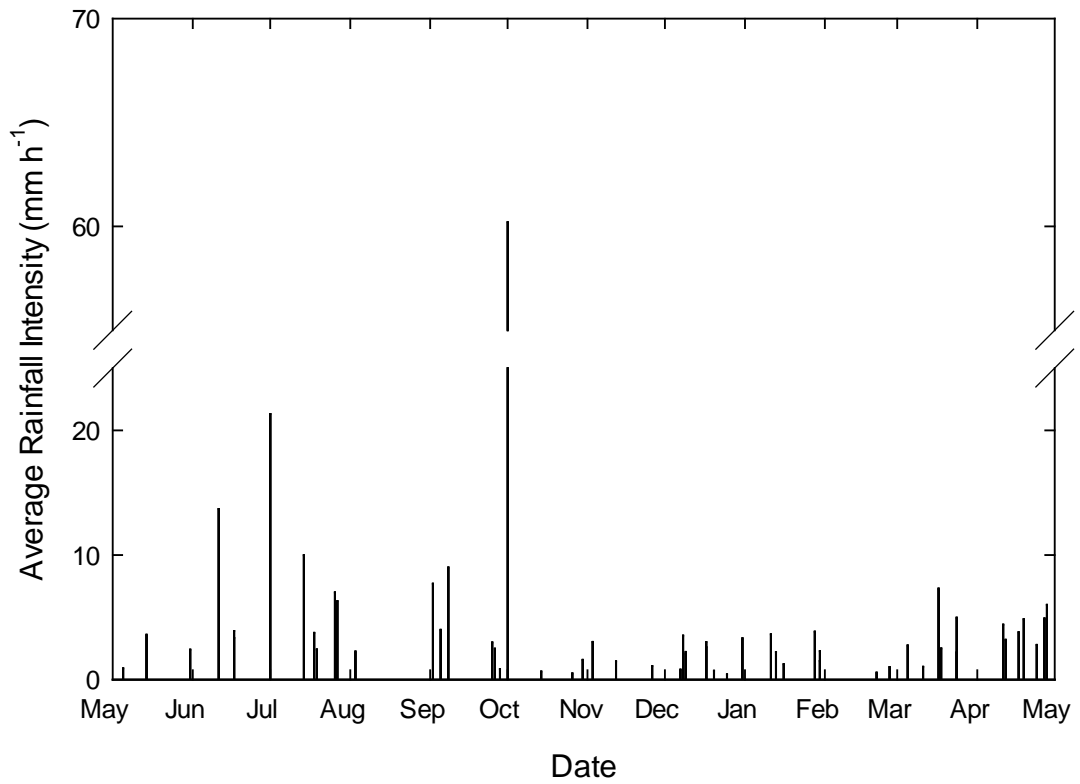


Figure 4.5. Average rainfall intensities for the project period.

Table 4.5. Descriptive statistics for inflow volumes and peak flows for the monitored storm events.

Descriptive Statistic	Inflow Volume (L)	Peak Inflow (L s <sup>-1</sup> )
Mean±Stdev	83,640±64,160	19±15
Median	72,270	17
Maximum	328,760	57
Minimum	3,500	1

From March 3, 2012 to May 8, 2013, the flow meters recorded an inflow volume of 8,020,000 L entering the rain garden. This total includes storm unanalyzed events, those smaller than the threshold for analysis established for this project (>4 mm), those with CN outside of the threshold for analysis (CN>90, CN≠100), and estimates of inflow through the 30 cm pipe during periods of malfunction due to groundhog activity. This total, however, does not include the small section of parking lot whose flow bypassed the rain garden inlet pipes (Figure 3.2). This small area accounts for 6% of the total impervious area contributing to the rain garden. In addition, this does not account for the volume of water that enters the rain garden via direct rainfall. The area of the rain garden is 880 m<sup>2</sup>, or 14% the size of the contributing drainage area. The volume of rainfall that fell directly on the rain garden can be determined from the total volume recorded by the flow meters. Since the rain garden area is 14%, it is estimated that 1,122,800 L of rainfall fell directly on the rain garden over the study period. It is known that less than 100% of the rainfall becomes runoff whereas all of the rain that falls directly on the rain garden is captured and infiltrated, thus this approximation is likely lower than the actual volume of water that fell directly on the rain garden. The total amount of stormwater treated by the rain garden over the study period was approximately 9,142,800 L.

The size distribution of inflow volumes mirrored that of rainfall depths, as seen in Figure 4.6, which is expected. The majority of the inflow was generated from smaller rainfall events (<20 mm). The average rainfall depth associated with storm event inflows of 150,000 L or less (70% of the inflows) was 14 mm.



## 4.4 STORAGE

### 4.4.1 Surface Storage

As determined from equation 3.5, the above-ground storage volume available before water began to flow into the infiltration chamber was about 112,650 L, which accounts for 42% of the WQV, (265,000 L) per LFUCG (2009) guidelines. Assuming no infiltration, the above-ground storage volume would have accommodated 80% of the storm events without overflow. The top of the rain garden is approximately 0.3 m above the elevation of the outlet. The above ground storage available before the rain garden overtops is 281,180 L as determined from equation 3.5. This would have accommodated all storm events without overflow except for the event occurring January 13.

### 4.4.2 Sub-surface Storage

Pore space within the amended soil allowed for sub-surface storage of runoff after it had infiltrated into the ground. The amended soil was made up of approximately 29% void space and 71% solids. There was approximately 131,390 L of subsurface storage available. This is greater than the volume of available above ground storage.

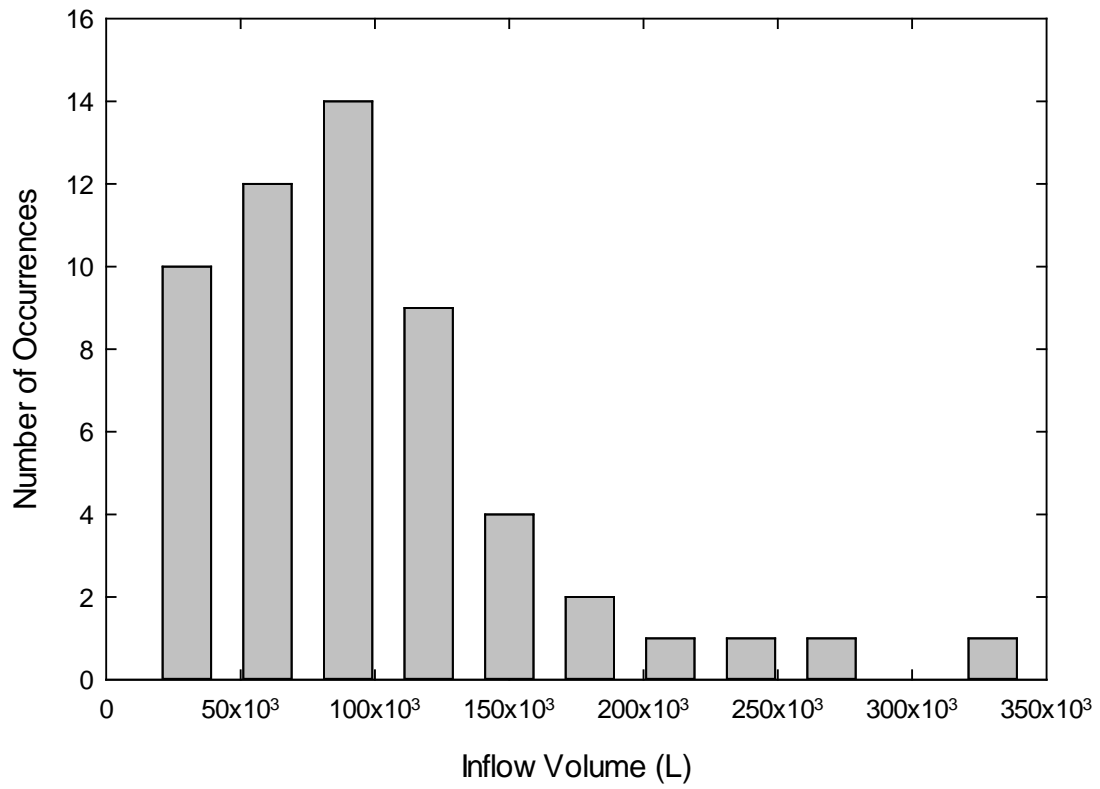


Figure 4.6. Distribution of inflow volumes, as measured by the flow meters.

## 4.5 INFILTRATION

### 4.5.1 Field Determined Infiltration Rates

Infiltration rates ranged between 8.0 and 46.4 cm h<sup>-1</sup> with an average value of 23.8 cm h<sup>-1</sup> for the study period (Table 4.6). These values were much higher than those reported by Selbig and Balster (2010) for prairie and turf grass rain gardens in Wisconsin.

Interestingly, while infiltration is the main unit process of a rain garden (WEF, 2012), infiltration rates were seldom reported so further comparisons were not possible.

As seen in in Figure 4.7, infiltration rates showed a significant but slight decrease over time ( $p=0.002$ ). Infiltration rates decreased around 0.3 mm h<sup>-1</sup> d<sup>-1</sup>. However, this decrease was due in part to temperature. Figure 4.8 shows significant changes in infiltration rates with changes in daily maximum air temperature. As air temperatures increased, infiltration rates increased as well at a rate of about 5 mm h<sup>-1</sup> °C. While soil and stormwater temperatures were not monitored, this trend with air temperature shows that infiltration rates are affected by fluid properties. Emerson and Traver (2008) examined hydraulic conductivity values in a bioretention traffic island at Villanova University. Over a four year period, the researchers found no decrease in performance of the bioretention traffic island, but they did note seasonal changes related to temperature. Hydraulic conductivity decreased at an average rate of 0.09 mm d<sup>-1</sup> °C due to changes in water density and dynamic viscosity.

Table 4.6 Descriptive characteristics for field determined infiltration rates.

Descriptive Statistic	Infiltration Rate (cm h <sup>-1</sup> )
Mean±Stdev	23.8±7.8
Median	23.3
Maximum	46.4
Minimum	8.0

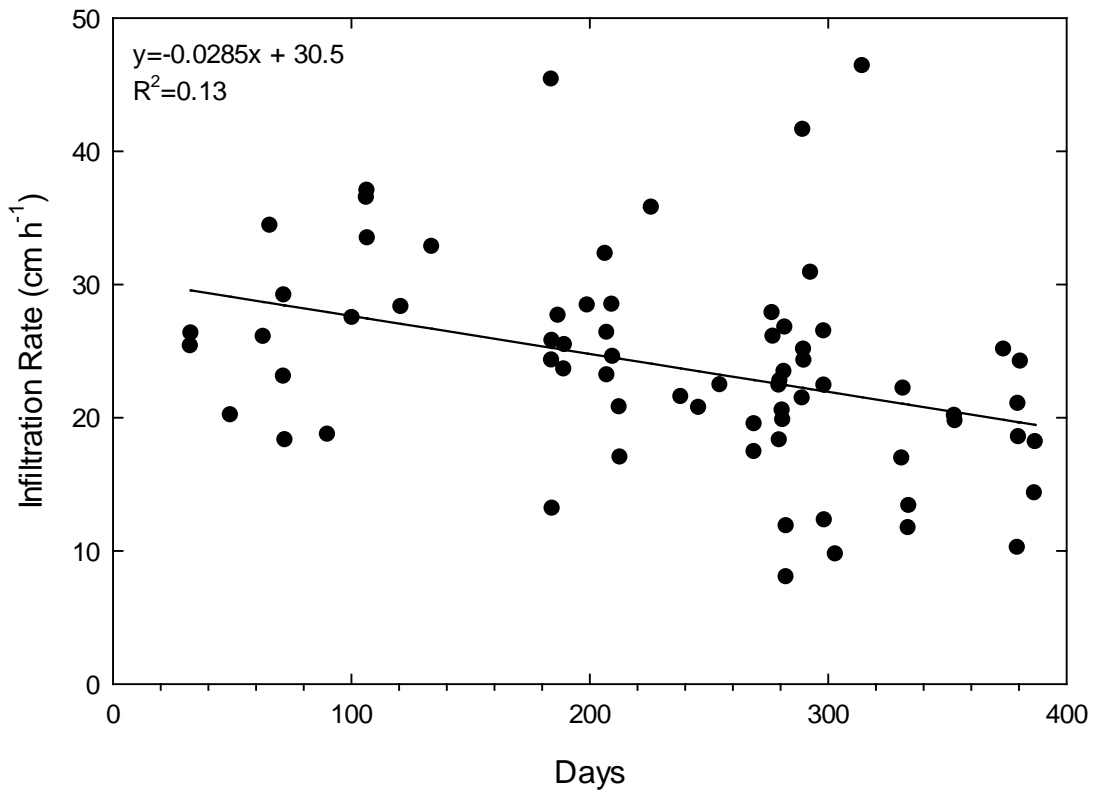


Figure 4.7. Temporal changes in rain garden infiltration rates.

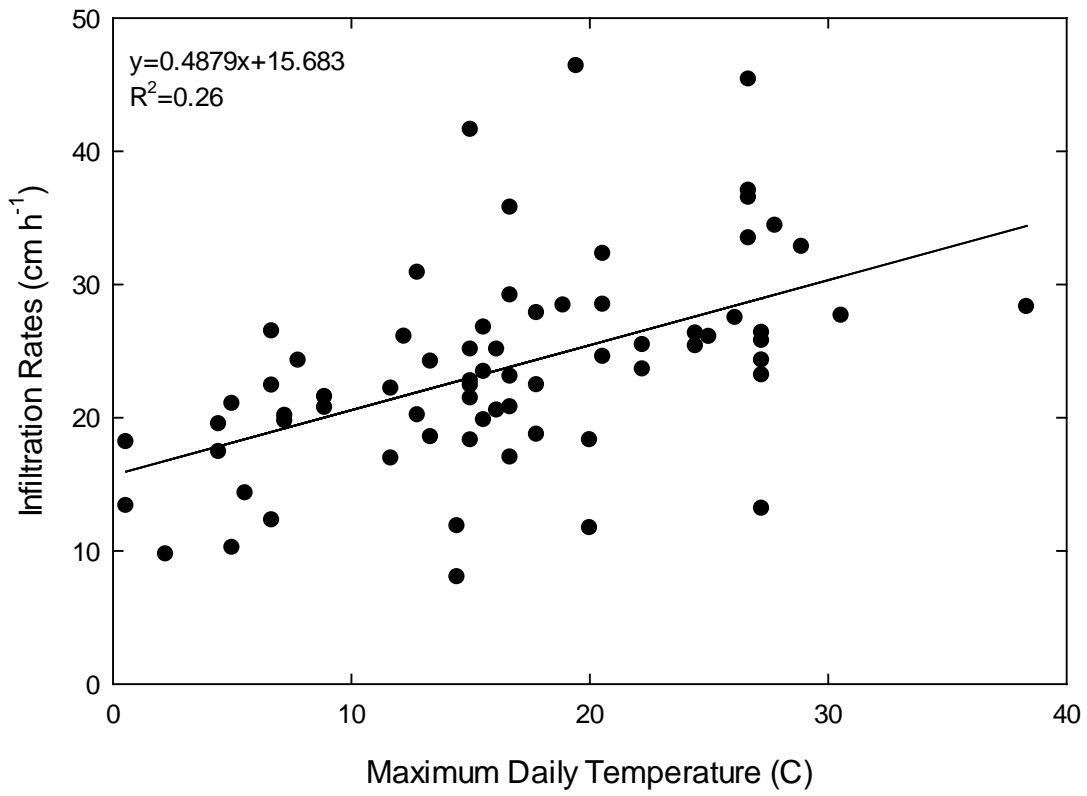


Figure 4.8. Effect of temperature on infiltration rates.

#### 4.5.2 Laboratory Determined Coefficient of Permeability

As seen in Table 4.7, a large amount of variability was present in the data. The mean coefficient of permeability for the ES was  $6.2 \text{ cm h}^{-1}$ ; for the AS it was  $17.7 \text{ cm h}^{-1}$ . Based on the higher coefficient of permeability value obtained for ES1, it is likely that this sample was taken too close to the rain garden edge, and as such may have been within the range of soil disturbance and amendment. AS4 has a much lower coefficient of permeability than the other AS samples. The reason for this difference is not known, but may have been taken from a pocket of existing soil within the rain garden.

As seen in Table 2.1, recommended values for coefficients of permeability range from  $0.625 \text{ cm h}^{-1}$  to  $15 \text{ cm h}^{-1}$  (Table 2.2). LFUCG (2009) recommends a value of  $0.625 \text{ cm h}^{-1}$  which is an order of magnitude lower than the average laboratory measured value for ES. While it is anticipated that field conditions are less favorable than laboratory conditions, these results indicate that the coefficient of permeability of AS is much greater than the value recommended for designing rain gardens in Lexington, KY.

Table 4.7. Results from the Constant Head Permeameter Tests.

Sample	Time Elapsed (s)	Inner Q (mL)	Outer Q (mL)	Total Q (mL)	k ( $\text{cm h}^{-1}$ )
ES1	35.2	41	20	61	10.4
ES3	188.5	56	3	59	1.9
Mean $\pm$ Stdev					6.5 $\pm$ 6.0
AS1	22.8	60	6	66	17.3
AS2	17.6	35	1	36	12.2
AS3	17.2	46	14	60	20.8
AS4	49.6	22	8	30	3.6
AS5	8.9	50	2	52	34.8
Mean $\pm$ Stdev					17.7 $\pm$ 11.5

## 4.6 SOIL MOISTURE

Results from the soil moisture sensor show that at a 10 cm depth, the soil experiences rapid fluctuations in moisture in response to rainfall, as expected. When water inflows into the rain garden, the 10 cm depth becomes saturated (Note that the soil moisture sensors only measure up to 50% water content because the program assumes this is the maximum amount of water the soil matrix can hold). Following storm events, soil moisture returns to an equilibrium of 30-35% after the stormwater has infiltrated (Figures 4.9-4.11). This equilibrium level, which is higher than that of the 30 and 50 cm depths, is reflective of the mulch application immediately above this layer. Mulumba and Lal (2008) noted that soil moisture retention at a 10 cm depth (at low suctions), available water capacity, and total porosity increased with wheat-straw mulch applications. The mulch layer is an important component of the rain garden for reasons beyond erosion control. It increases the amount of “easily available” water for the plants (Brady and Weil, 2008). This is particularly important for the plants in the rain garden as the amended soils have less plant available water as compared to the existing soils (13% versus 21%, respectively).

The 30 and 50 cm depths experienced large and rapid changes in soil moisture. The soil moisture at the 30 and 50 cm depths was typically about 25% in between storm events. When it rained, the soil moisture at these levels quickly rose to saturation (or almost saturation). And as expected with the high infiltration rates measured in the well, the soil moisture at the 30 and 50 cm depths quickly returned to equilibrium as the water was drained to greater depths. Soil moisture at the 70 and 90 cm depths had a higher equilibrium than the 30 and 50 cm depths, but like those two depths, it increased quickly in response to inflow. However, it took a little longer for the soil moisture at the 70 and 90 cm depths to return to equilibrium. The slower return to equilibrium was likely related to the closeness of these two sensors to the boundary between the amended and existing soil; it was estimated that this boundary occurred at about 120 cm. It was not known why the soil moisture at 90 cm experienced changes of greater magnitude than the 70 cm sensor or vice versa why the soil moisture at 70 cm had a higher equilibrium. The difference could be due to variations in the amendment mixture (e.g. lack of uniform mixing and thus the presence of more clay at 70 cm).

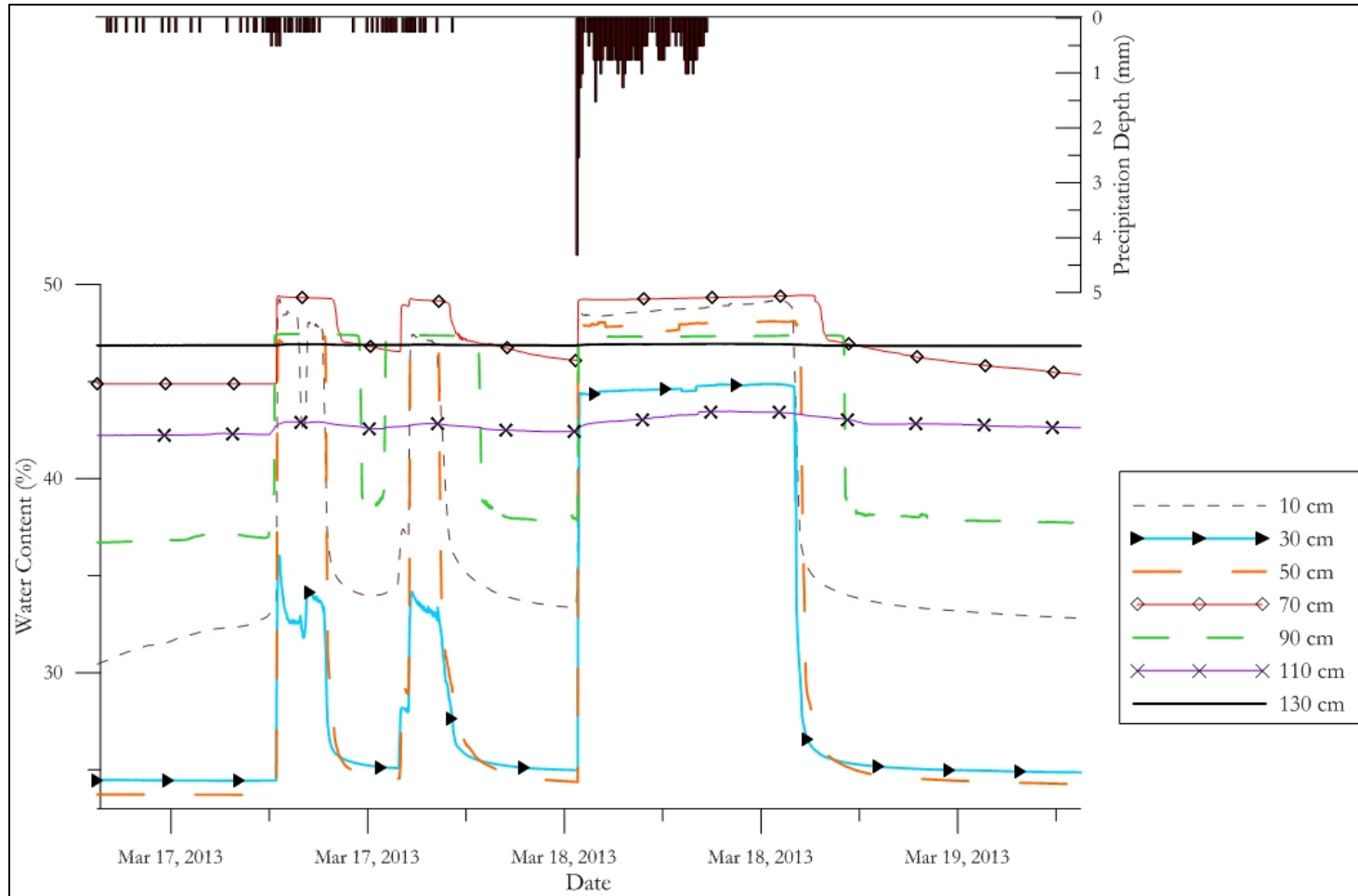


Figure 4.9. Soil moisture data March 17, 2013.



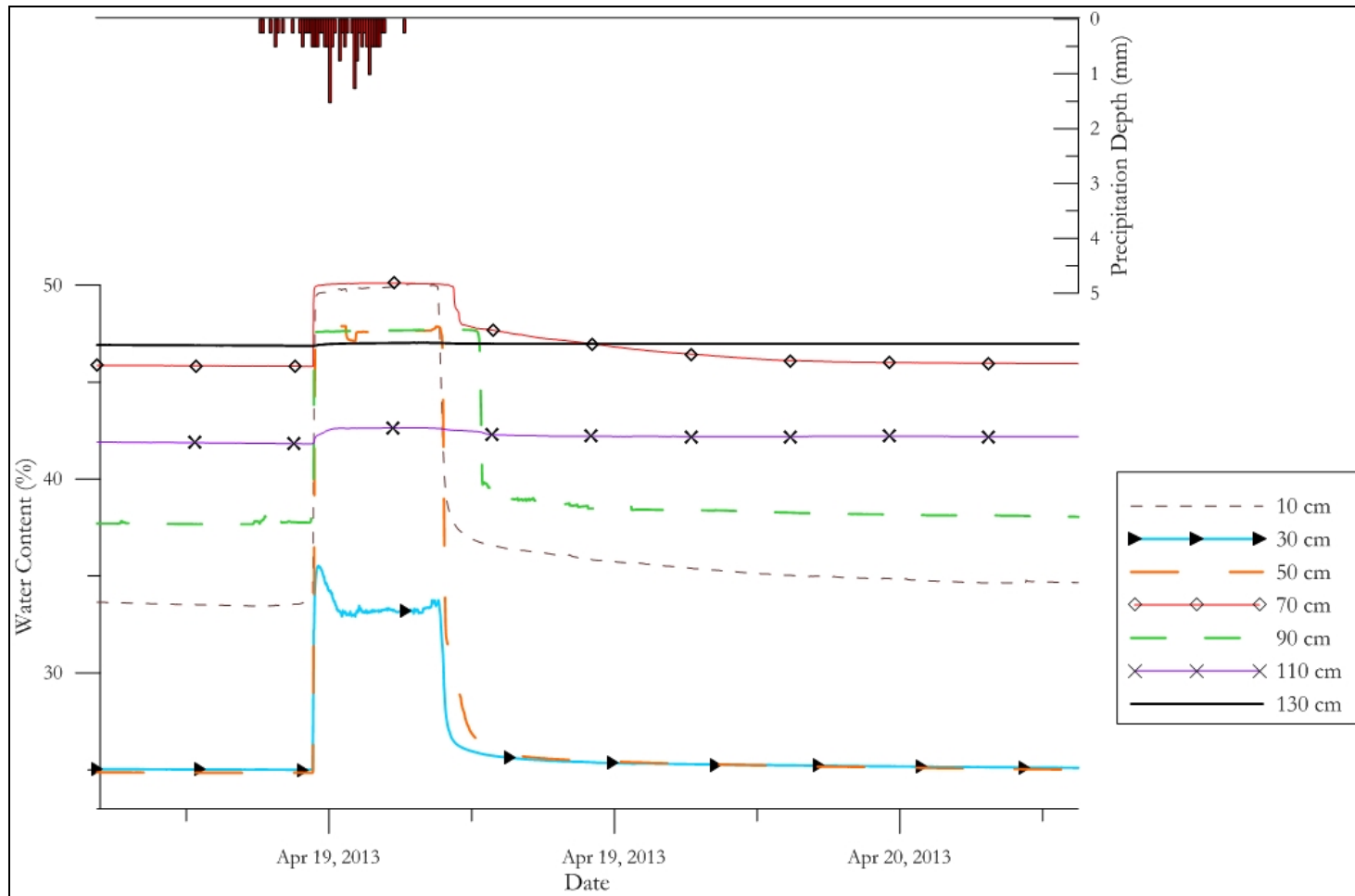


Figure 4.10. Soil moisture readings April 19, 2013.

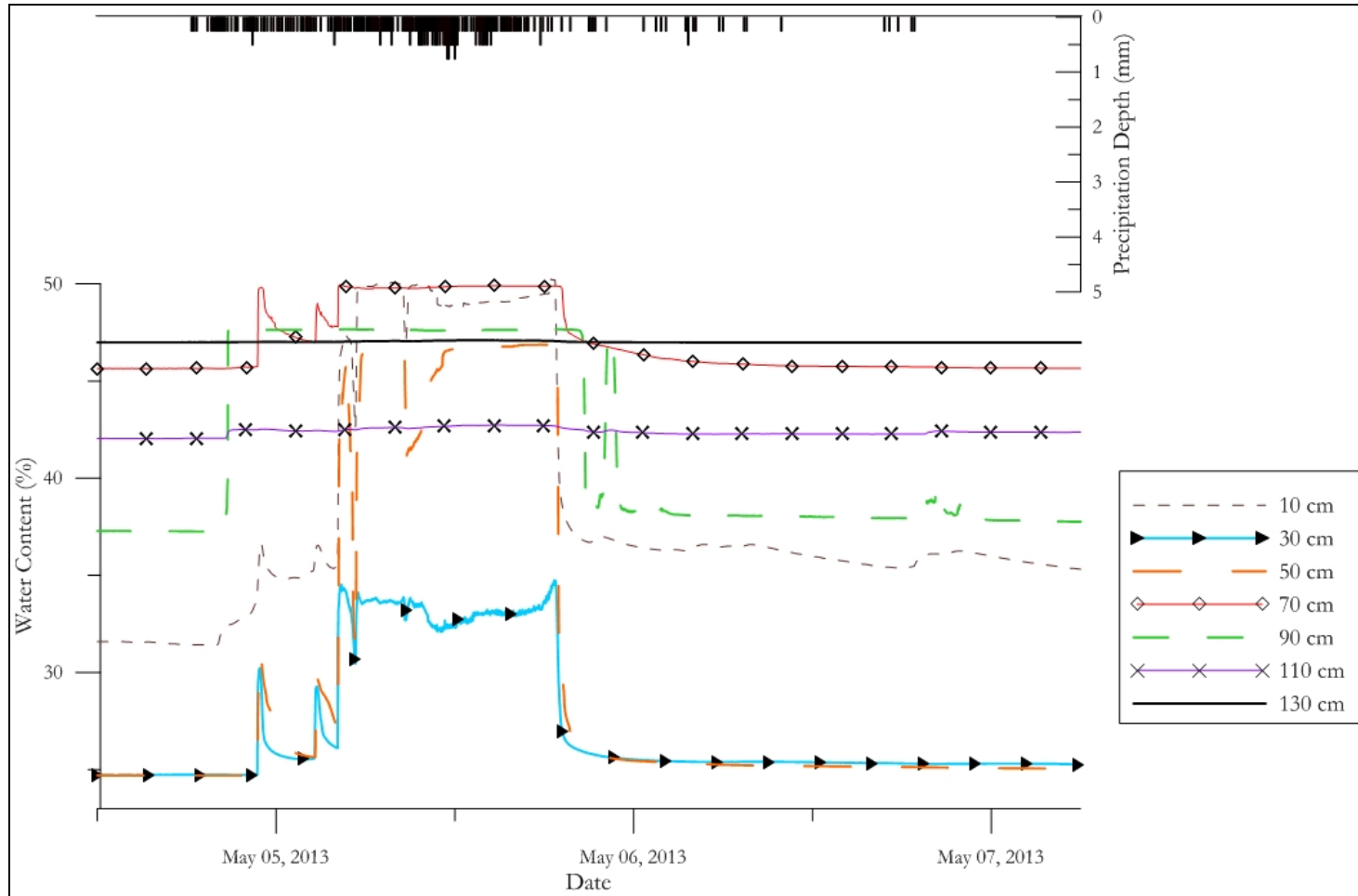


Figure 4.11. Soil moisture data May 5, 2013.

Soil moistures at the 110 and 130 cm depths experienced the least amount of change in response to storm events. These depths represent soil moisture immediately above and below the amended/existing soil boundary. Soil moisture at the 110 cm depth remained at about 42% during storm events while it was about 47% at the 130 cm depth. The existing soil has a measured field capacity of 50 % (Table 4.1), hence soil at the 130 cm depth was nearing saturation.

The results from the sensors at 110 and 130 cm indicated that there might have existed a constant state of saturation at the boundary between the amended and existing soil layers. However, the results from the level logger installed in the deep well (2.4 m) indicated that this was not the case. The level logger showed that water level maintained an equilibrium of 2.1 m below the ground surface between storm events. Also, while augering the deeper well, saturated soil was not encountered until 2.25 m below the ground surface. So, while soil moisture remained high for the lowest two sensors, the results of the 2.4 m deep well indicated that the soil at the boundary between the amended and existing soil did not remain saturated.

#### **4.7 EVAPOTRANSPIRATION**

For this project, it was difficult to quantify evapotranspiration rates by a water balance approach. This was because after stormwater entered the soil matrix, it was unknown how much left the rain garden via exfiltration versus evapotranspiration. Thus, evapotranspiration rates were estimated based in part on prior rain garden and bioinfiltration research (Wadzuk et al., 2011). As seen in Table 4.8 and Figure 4.12, evapotranspiration rates varied seasonally, which was expected. Evapotranspiration rates were highest during the summer months and lowest during the winter months. Crop coefficients also varied depending on active plant growth. The bare ground crop coefficient was determined to be 0.5 throughout the year, the medium and small trees had crop coefficients of 0.45, 0.9, and 0.65, shrubs had crop coefficients of 0.3, 1.05, and 0.5, little bluestem had crop coefficients of 0.3, 1.15, and 0.4. Little bluestem occupied an area of 85 m<sup>2</sup>, medium and small trees occupied an area of 15 m<sup>2</sup>, shrubs occupied an area of 15 m<sup>2</sup>, and bare ground (mulched) occupied an area of 765 m<sup>2</sup>. The overall area weighted crop coefficients were 0.48, 0.58, and 0.49 for initial growth stage, growing stage and end of growth stage respectively. The initial

Table 4.8. Monthly average of daily evapotranspiration rates ( $\text{mm d}^{-1}$ ).

Month	Mean $\pm$ Stdev	Maximum	Minimum
-----2012-----			
May	4.3 $\pm$ 0.8	6.6	2.8
June	7.0 $\pm$ 2.1	13.4	2.3
July	7.4 $\pm$ 1.7	11.5	4.9
August	5.0 $\pm$ 0.8	6.5	3.7
September	1.4 $\pm$ 0.4	2.1	0.3
October	1.1 $\pm$ 0.5	2.1	0.2
November	1.4 $\pm$ 0.8	3.8	0.4
December	0.8 $\pm$ 0.6	2.1	0.0
-----2013-----			
January	0.6 $\pm$ 0.4	1.5	0.1
February	0.8 $\pm$ 0.5	2.3	0.2
March	0.6 $\pm$ 0.4	1.6	0.0
April	1.3 $\pm$ 0.5	2.2	0.4
May	3.6 $\pm$ 1.6	6.9	0.9

growth stage crop coefficient was used for March and April, the growth stage crop coefficient was used for May – August, and the end of growth stage crop coefficient was used for September – February.

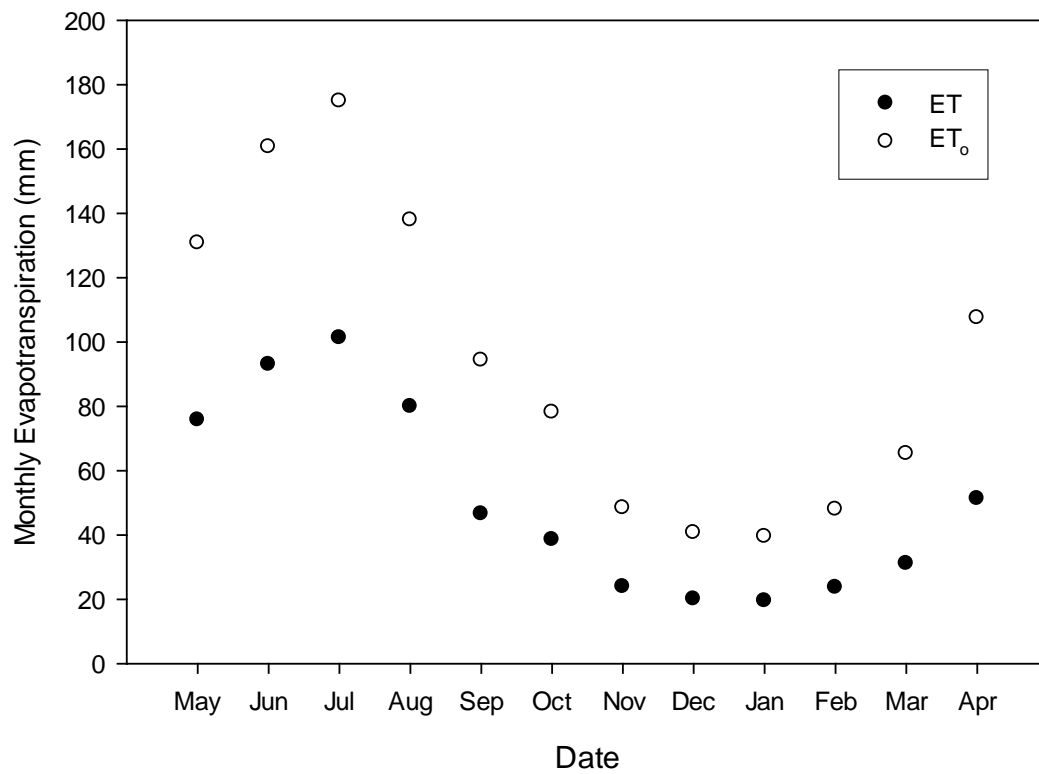


Figure 4.12. Reference ( $ET_0$ ) and predicted (ET) evapotranspiration depths during the study period.

Overall, evapotranspiration rates accounted for a small percentage of the total inflow (Table 4.9). The summer months of June, July and August accounted for a relatively large percentage of the inflow. These values were similar to the percentage computed by Li et al. (2009) who noted that 19% of the inflow was evapotranspired by a rain garden in North Carolina. Even during the winter months, the low amounts of inflow accounted for by evapotranspiration were higher than the 0.4% computed by Dietz and Clausen (2005). While evapotranspiration is not a significant component of the water balance at during a storm event, it can impact storage capacity in the soil between rainfall events (WEF, 2012). After the rain garden soils have drained to field capacity, plants will continue to utilize stored water through evapotranspiration.

Table 4.9. Evapotranspiration (ET) as a percentage of inflow.

Month	Total Inflow Depth (mm)	Total Monthly Predicted ET (mm)	% of Inflow <sup>1</sup>
-----2012-----			
May	1,588	76	5
June	868	93	11
July	1,796	101	6
August	513	80	16
September	2,767	46	2
October	379	38	10
November	736	24	3
December	2,981	20	1
-----2013-----			
January	2,265	19	1
February	604	24	4
March	2,649	31	1
April	1,888	51	3

<sup>1</sup>Total inflow was used to compute these percentages and not just inflow associated with the analyzed storm events.

## 4.8 HYDROLOGIC PARAMETER COMPARISON

### 4.8.1 Overtopping Event

Following the removal of the monitoring equipment from the rain garden, a large rain event occurred on June 26, 2013 that overtopped the outlet structure. This storm event was the only one known to have produced a water depth high enough to exceed the inlet elevation of the outlet structure. The total depth was 42 mm and the duration was one hour. During the most intense period of this storm event, 32 mm of rain fell in a 5-minute period (intensity of 384 mm h<sup>-1</sup>). This event was equated to a 24-hour event using the NRCS Type II rainfall distribution (SCS, 1983). When distributed over a 24 hour period, the storm event (42 mm, 1 hour) was equivalent to a 141 mm depth which equates to a 25-yr 24-hr storm for Fayette County (DNREP, 1979).

### 4.8.2 LFUCG Based Guidelines

#### 4.8.2.1 Scenario 1

Table 4.10 contains the input parameters for the spreadsheet model for Scenario 1. Using the inflow parameters recorded during the study period along with the hydrologic parameters recommended by LFUCG (2009) and assuming the size of the rain remained the same (as built), the water level in the rain garden would have been high enough to overtop the outlet structure 8 times (Figure 4.13). Note that the invert of the outlet structure is approximately 46 cm above the lowest point of the rain garden (i.e. maximum ponding depth of 46 cm).

#### 4.8.2.2 Scenario 2

Table 4.10 contains the input parameters for the spreadsheet model for Scenario 2. If the rain garden was constructed with the same footprint but with the LFUCG maximum ponding depth of 15 cm, then the water level in the rain garden would have overtopped the outlet structure 26 times (Figure 4.13).

#### 4.8.2.3 Coefficient of Permeability

Figure 4.14 shows that the LFUCG (2009) recommended values for the design parameters, most notably coefficient of permeability, under-estimate the ability of the

Table 4.10. Input parameters for modeling Scenarios 1 and 2.

Parameter	Scenario 1	Scenario 2
Filter Bed Area (m <sup>2</sup> )	370	370
Coefficient of Permeability (cm h <sup>-1</sup> )	1.27	1.27
Maximum Ponding Depth (cm)	46	15
Above-ground Storage	Stage-volume curve	Stage-volume curve
Number of Overtopping Events	10	22

amended soil to infiltrate runoff. The coefficient of permeability measured in this study had an average of 21.3 cm h<sup>-1</sup> (excluding AS4 with a value of 3.6 cm h<sup>-1</sup>), which was much greater than the LFUCG recommended value of 0.625 cm h<sup>-1</sup> and the value of 3 cm h<sup>-1</sup> used by the designer.



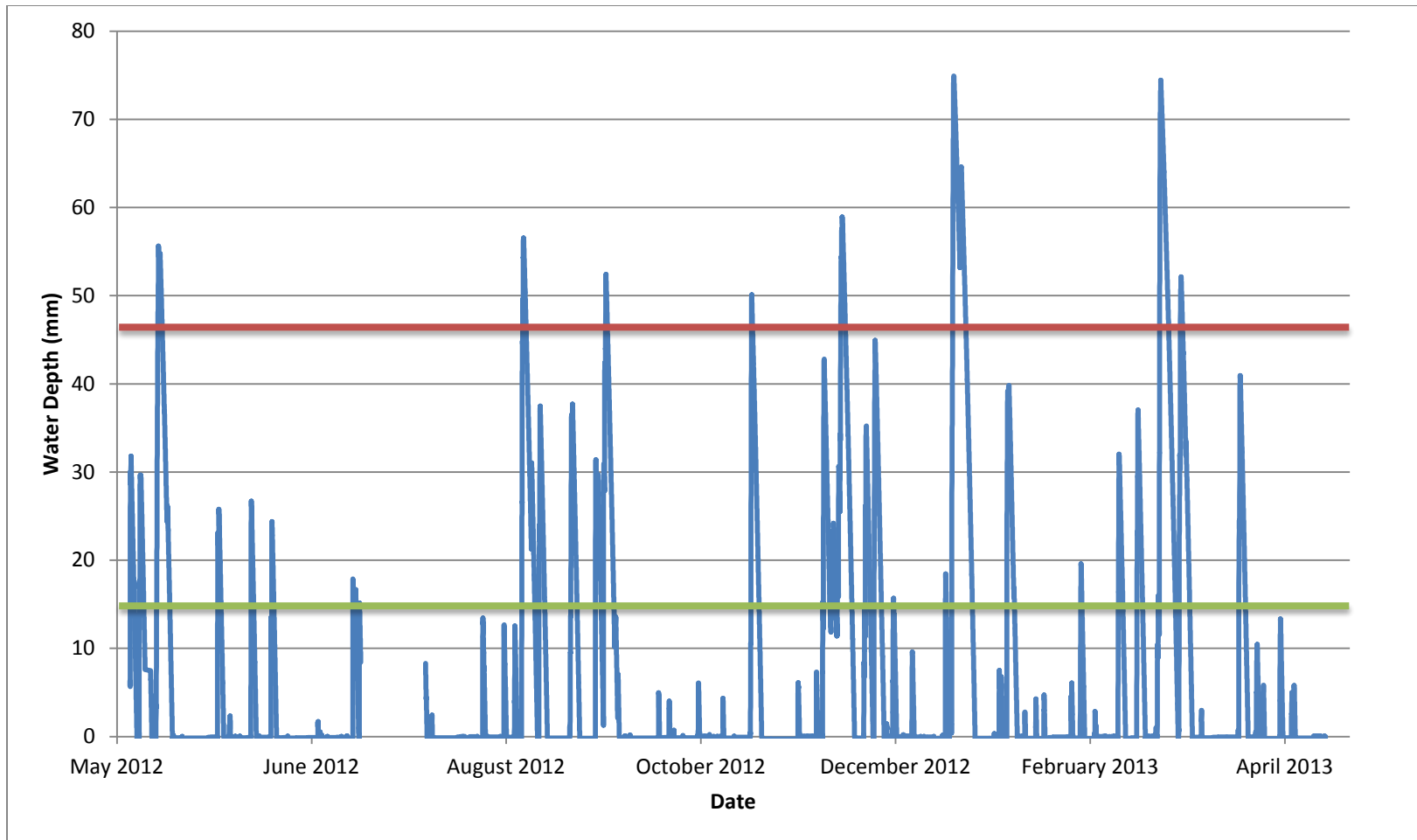


Figure 4.13. Predicted ponded water level in the rain garden (coefficient of permeability= $1.27 \text{ cm h}^{-1}$ ) with the designed maximum ponding depth of 46 cm and the LFUCG (2009) recommended ponding depth of 15 cm.

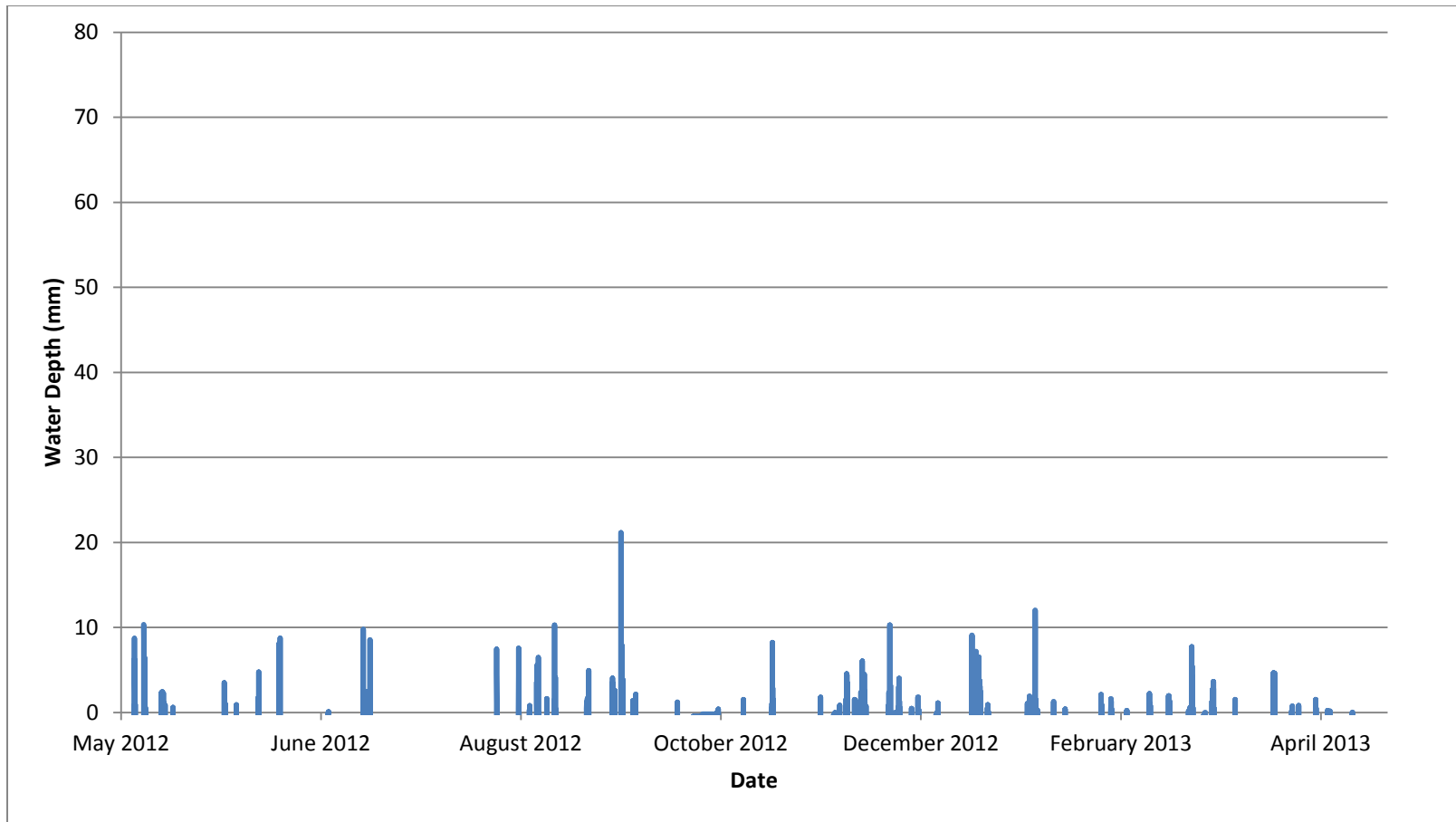


Figure 4.14. Predicted ponded water level in the rain garden based on measured average coefficient of permeability of  $21.3 \text{ cm h}^{-1}$ .

## **CHAPTER 5: CONCLUSIONS**

A stormwater management system consisting of a rain garden and rainwater harvest system was installed at Coca-Cola Refreshments USA, Inc. in Lexington, Kentucky. The primary purpose of this system was to manage stormwater runoff from an impervious drainage area consisting of a portion of the warehouse roof and adjacent parking lot and the entire office roof.

### **5.1 RAIN GARDEN**

The rain garden design was based on the LFUCG Stormwater Manual (2009) with modifications to the coefficient of permeability and maximum ponding depth, both of which were increased. The rain garden was sized to capture and infiltrate runoff volume from the 41 mm of rainfall, which LFUCG defines as the WQV for a 100% impervious area. However, monitoring results demonstrated that the rain garden had a higher hydrologic capacity than designed, as it was able to capture and infiltrate storm events greater than the WQV within hours. The majority of inflow into the rain garden was infiltrated and subsequently stored in the soil profile and/or exfiltrated to the existing soil below. Except for the summer months of June, July and August of 2012, evapotranspiration accounted for less than 4% of the inflow, on average.

#### **5.1.1 High Infiltration Rates**

The reason for the high infiltration rates was due to two design components: amending the soil and alleviating soil compaction. The existing soil was clay, but it was amended using a combination of sand (25%), leaf compost (25%), and existing soil (50%) such that the resultant was a sandy loam. Amending the soil increased its coefficient of permeability. However, it is not known to what extent the act of amending the soil on its own changed the coefficient of permeability. MDE (2009) recommends the addition of leaf compost as this material has a higher coefficient of permeability than sand. With regards to compaction alleviation, a 1.2 m depth of soil beneath the upper amended 1.2 m layer was loosened with an excavator. Although the coefficient of permeability was not determined for the loosened existing soil, evidence suggests that this action also helped increase the coefficient of permeability. Gregory et al. (2006) noted that soil compaction is highly correlated with the coefficient of permeability. Reductions in compaction result in increased values for the

coefficient of permeability. One question that remains is with regards to the hydrologic performance of the rain garden as the loosened soils (amended layer and existing layer) settle.

One potential trade-off is that with high infiltration rates is that water quality treatment of stormwater is less (WEF, 2012). Shorter contact times may mean that the soil was able to sorb lesser amounts of pollutants from the stormwater. The lack of an underdrain in the rain garden means the infiltrated stormwater will not enter the storm sewer system, but with the local karst geology, it could quickly come into contact with the groundwater.

### **5.1.2 Large Capture Volumes**

The maximum ponding height in the rain garden was greater than almost all other rain gardens found in the literature (Table 2.3). Generally, maximum ponding depths are limited to 30 cm or less (Table 2.2) to prevent the damage to vegetation from extensive inundation and mosquito breeding. However, the greater ponding depth at the Coca-Cola rain garden did not raise these concerns as the water infiltrated so quickly that ponding rarely occurred and when it did, it tended to be shallow and short-lived. The greater ponding depth allowed for much more stormwater storage.

Since runoff infiltrates into the soil matrix as it enters the rain garden, available subsurface storage is an important consideration regarding available capture and storage volume. This storage volume would be available in addition to available above ground storage. However, the design method and recommendations do not account for subsurface storage volume available as pore space. This component of storage needs to be considered as part of the overall design.

### **5.1.3 Recommendations**

As designed, the rain garden did not overtop during the study period. However, if the rain garden was designed in accordance with the LFUCG Stormwater Manual (2009) with regards to the coefficient of permeability and maximum ponding depth, the rain garden would have produced outflow for 26 of the monitored storm events. Based upon the results of this study, it is recommended that LFUCG evaluate requiring designers to test the infiltration capabilities of the existing soil at a proposed rain garden site as well as the infiltration capabilities of soils amended with the planned mixture. The composition of the amended soil with regards to % sand and % clay were within the ranges recommended by LFUCG, however infiltration rates far exceeded those specified for amended soils. For this reason, it is recommended that greater emphasis be placed on

amending a soil to a specific coefficient of permeability rather than a certain composition. Infiltration capabilities of existing soils can be tested *in situ* in the form of a percolation test or in a laboratory whereas the proposed amendment mix would require testing in a laboratory prior to final design and implementation.

And while slower infiltration rates generally increase pollutant removal, if inflow water quality is not a concern, then higher infiltration rates enable a rain garden to capture greater volumes of runoff. In the case of this rain garden, performance results show that the high infiltration rates and greater ponding depth meant the rain garden could have a higher contributing area to receiving area ratio and still be able to maintain an acceptable hydrologic performance. The spreadsheet model predicts that the contributing drainage area could be increased by up to five times (about 20:1 to 100:1) before the level in the rain garden would reach the outlet structure and hence flow into the infiltration chamber. Even with a factor of safety of two, the rain garden should be able to capture and infiltrate runoff generated from 2.5 times of additional contributing drainage area. At this point, it is recommended that Coca-Cola evaluate adding an additional section of the warehouse and monitor the rain garden's performance subsequently.

## **5.2 RAINWATER HARVESTING SYSTEM**

One of the main goals of the rainwater harvesting system was to reduce the use of potable water for tasks such as truck washing, toilet flushing, and other such activities that do not require drinking quality water. The results of this study (e.g. average CN of 97) indicate that the rainwater harvesting system is underutilized. While usage of stormwater stored in the tank is not tracked, results of this study suggest that the tank was near or at capacity for a large number of storm events. The average duration between storm events (depth > 4 mm) was 2.9 days, thus it is unlikely that Coca-Cola used all of the stored water in the tank for toilet flushing or truck washing, particularly since truck washing was not a daily event. As such, it is likely that only a small percentage of the stormwater was stored in the tank and used by Coca-Cola. If Coca-Cola had used all of the stormwater stored in the tank in between storms, an improbable scenario given the frequency of storms and currently designated uses of the stored water, then about 20-25% of the stormwater would not have bypassed the tank and entered the rain garden.

The design and construction of the rainwater harvesting system was approximately \$47,000, thus determining more uses for this resource will better enable both Coca-Cola and LFUCG to utilize their investment. For this reason, it is recommended that Coca-Cola conduct an audit of their water usage to identify additional uses for the non-potable stormwater. It is recommended that an emphasis be placed on usages that do not require (or require limited) additional capital expenses. The main challenge with conducting the audit will be the lack of a data logger on the rainwater harvesting system. At present, the control panel for the rainwater harvesting system only notes the water level present in the tank in addition to the total usage over the life of the tank. The control panel does not record specific water usage information such as amounts and times. Once a pattern of usage has been established, such as daily, weekly or monthly, the excess water could be released to the rain garden in between storm events in order to maximize storage volume in the tank. Lastly, it is recommended that such stormwater audits with usage meters be installed on future LFUCG sponsored projects to help maximize utilization of the harvested rainwater and thus effectiveness of rainwater harvesting systems as a stormwater best management practice.

## CHAPTER 6: FUTURE WORK

While this study demonstrated that the Coca-Cola rain garden was very effective at capturing and infiltrating large quantities of stormwater, it did not address water quality. Since stormwater management and low impact development deal with water quantity and quality issues, future work should address performance related to water quality, particular with such high infiltration rates. While a number of prior studies have investigated the ability of rain gardens to treat stormwater contaminants, no studies have taken place in Kentucky. Other water quality related questions center around 1) the ability to use soil amendment such as clays to remove heavy metals while balancing infiltration needs and considering long-term accumulation, and 2) the selection and management of plant species for the removal of certain constituents.

Additional work is also needed in the area of evapotranspiration. Measuring evapotranspiration in the field is challenging. Most studies, including this one, have estimated evapotranspiration from either a water balance or from meteorological data. Evapotranspiration is a major component of predevelopment hydrology, and since the low impact development philosophy attempts to mimic predevelopment hydrology, better understanding of the role of evapotranspiration in rain garden hydrology is necessary to improve rain garden performance.

Finally, the long-term hydrologic performance of this rain garden is not known. Conducting an additional hydrologic analysis on this rain garden in 10 years, for example, would determine if the hydrologic performance of the rain garden has changed over time. The lower 1.2 m of existing soil that had been loosened could settle over the years, thus reducing hydrologic performance. Although the trees and shrubs planted in the rain garden are small in nature, it is unknown what effect their growth and maturity will have on long term evapotranspiration rates and conversely hydrologic performance.

## **APPENDIX A: PLANT CAM PHOTOGRAPHS**



Figure A.1. Beginning of event on June 26, 2013.



(a)



(b)



(c)



(d)

85

83

Figure A.2. Rainfall and inflow of event on June 26, 2013: a) 15, b) 20, c) 25, and d) 30 minutes after start of event.



(a)



(b)



(c)



(d)

Figure A.3. Water begins to decrease in depth from event on June 26, 2013: a) 35, b) 40, c) 45, and d) 50 minutes after start of rainfall.



(a)



(b)



(c)



(d)

Figure A.4. Infiltration and rainfall continue on June 26, 2013: a) 55, b) 60, c) 65, and d) 70 minutes after start of rainfall.





(a)



(b)



(c)



(d)

Figure A.5. Infiltration and rainfall continue on June 26, 2013: a) 75, b) 80, c) 85, and d) 90 minutes after start of rainfall.



(a)



(b)



(c)



(d)

Figure A.6. Infiltration and rainfall continue on June 26, 2013: a) 95, b) 100, c) 105, and d) 110 minutes after the start of rainfall.



(a)



(b)



(c)



(d)

Figure A.7. Water continues to infiltrate on June 26, 2013: a) 120, b) 125, c) 130, d) 145 minutes after the start of rainfall.





(a)



(b)



(c)



(d)

Figure A.8. Water continues to infiltrate on June 26, 2013: a) 150, b) 155, c) 160, and d) 170 minutes after the start of rainfall.



(a)



(b)



(c)



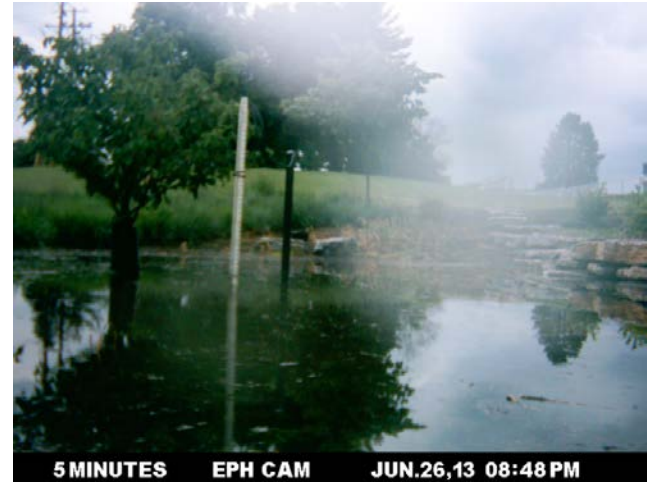
(d)

Figure A.9. Infiltration and rainfall continue on June 26, 2013: a) 175, b) 180, c) 185, and d) 190 minutes after start of rainfall. Plant Cam entered sleep, so not subsequent photos of this event were recorded.





(a)



(b)



(c)



(d)

Figure A.10. The last areas of ponded water from an overnight storm, the morning of June 27, 2013.



(a)



(b)



(c)



(d)

Figure A.11. The last areas of ponded water from a storm that occurred overnight on the morning of June 27, 2013.



(a)



(b)



(c)



Figure A.12. A small event occurred in the afternoon of June 27, 2013. Photo (b) shows water just starting to pond.



(a)



(b)



(c)



(d)

FigureA.13. Ponded water infiltrates after a rain event on June 27, 2013: a) 10, b) 15, c) 20, and d) 25 minutes after the start of rainfall.



(a)



(b)



(c)



(d)



Figure A.14: Much of the surface showing and only micropools holding water. Photos are a) 30, b) 35, c) 40, and d) 45 minutes after the start of rainfall.



(a)



(b)



(c)



(d)

Figure A.15: The rain garden has gone from ponded water to no water ponded in about an hour. Also, the groundhog appears in the photo (d).



(a)



(b)



(c)



(d)

## APPENDIX B: TOPOGRAPHIC SURVEY AND STAGE-VOLUME CURVE



Figure B.1. Arc-GIS surface of rain garden.

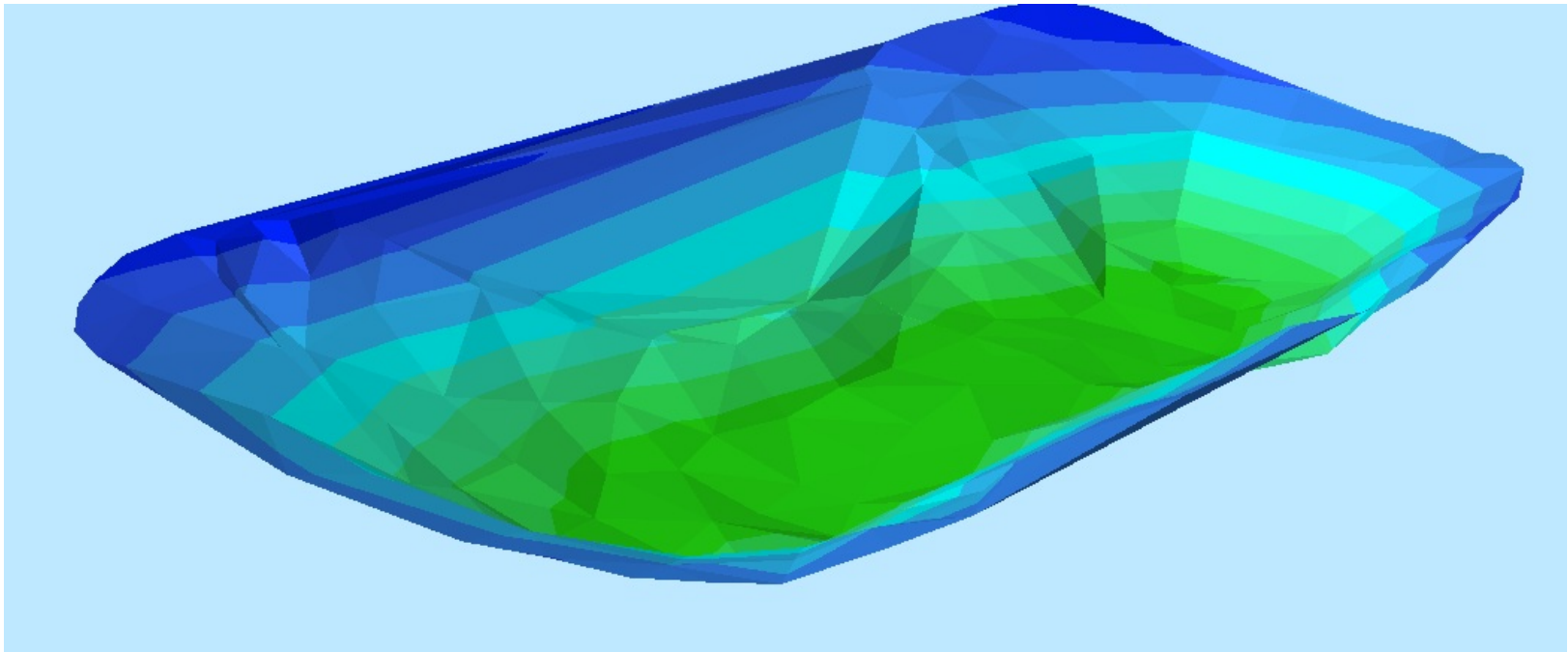
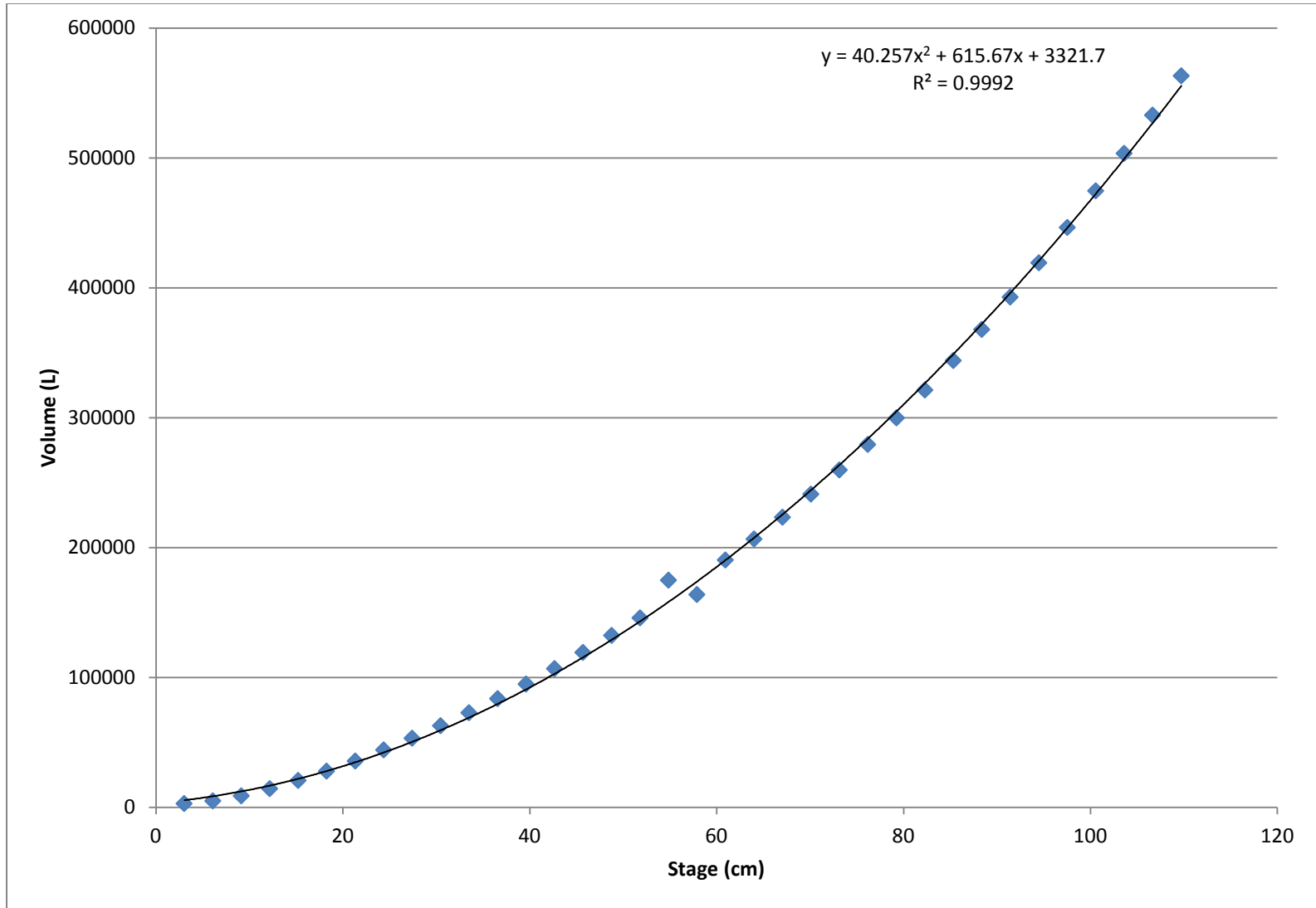


Figure B.2. Stage-volume rating curve for rain garden.



## APPENDIX C: LEVEL TROLL CALIBRATION

Figure C.1. Calibration curve for Level Troll placed in shallow well.

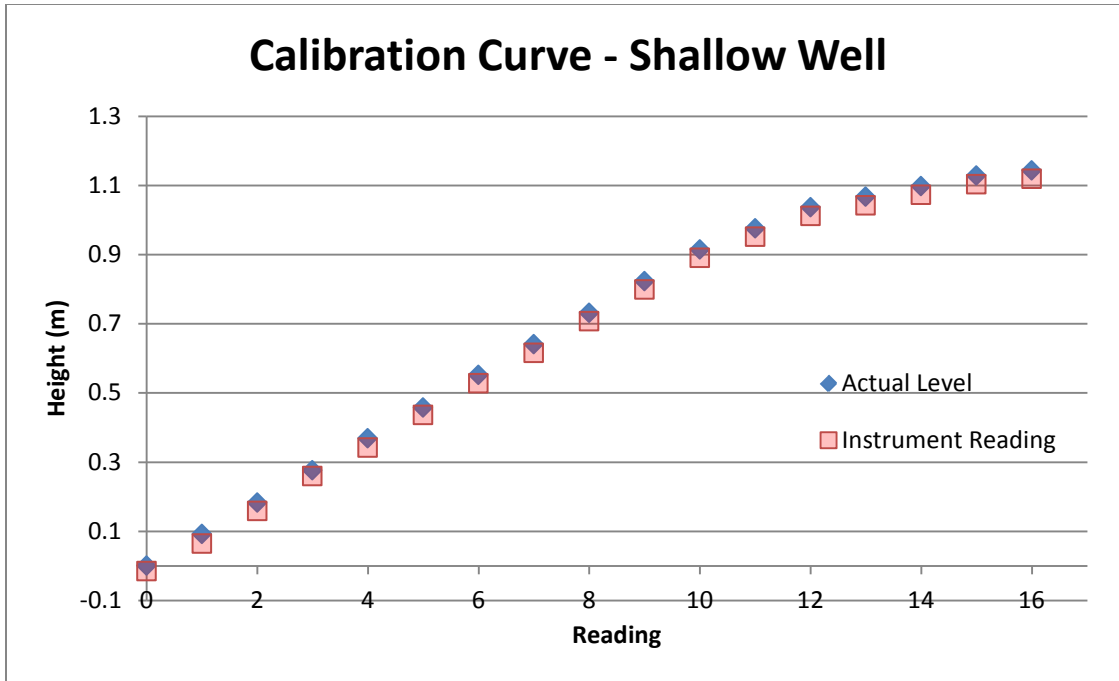
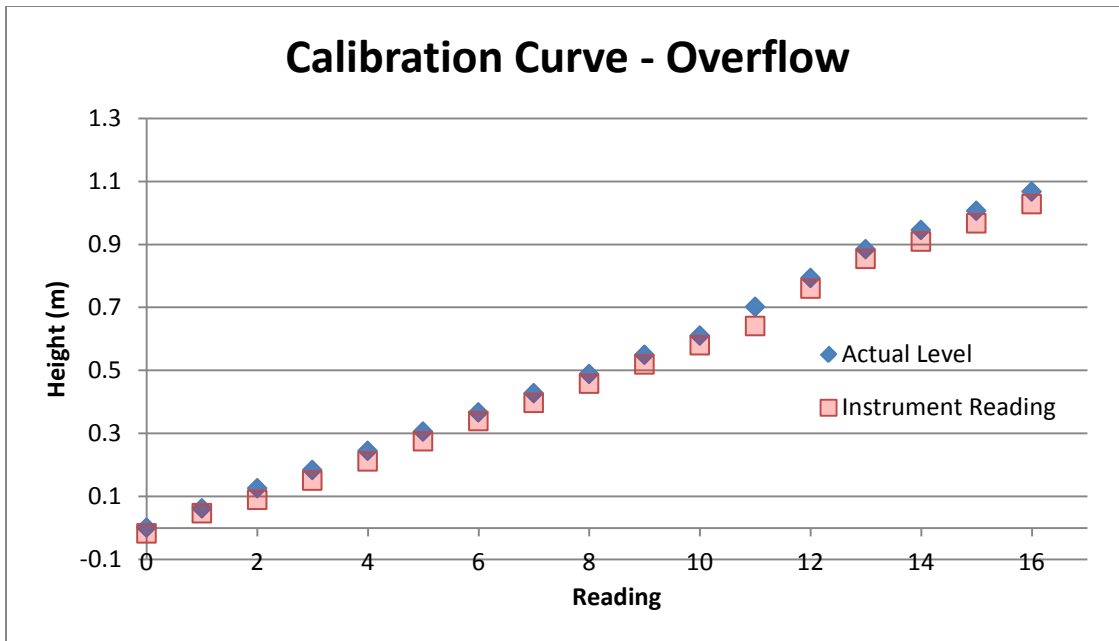


Figure C.2. Calibration curve for Level Troll placed in overflow sump.



## **APPENDIX D: STORM EVENT CHARACTERISTICS**

Table D.1. Storm Event Characteristics.

Date	Rainfall Depth (mm)	Rainfall Duration (h)	Average Intensity (mm h <sup>-1</sup> )	30-minute Peak Intensity (mm h <sup>-1</sup> )	Time Since Prior Rainfall Event (d)	Depth of Prior Rainfall Event (mm)	24-h Normalized Depth (mm)
May 5, 2012 <sup>1</sup>	14	2.1	7	20	4.1	3	26.1
May 14, 2012 <sup>1</sup>	5	5.8	0.9	3	0.2	0.8	7.1
May 31, 2012 <sup>1</sup>	21	5.8	4	5	17.5	5	30.1
June 11, 2012 <sup>1</sup>	17	7.1	2	10	6.6	0.5	23.1
June 17, 2012a	9	0.5	21	21	5.9	17	25.1
June 17, 2012b	13	3.1	3	11	0.2	10	21.9
July 1, 2012	6	1.6	4	8	14.2	9	12.0
July 14, 2012 <sup>2</sup>	14	0.7	21	28	1.1	2	35.8
July 18, 2012	18	1.3	14	34	3.1	4	37.9
July 19, 2012	17	6.0	4	32	0.8	19	24.1
July 26, 2012	15	4.3	3	11	0.2	1	23.3
July 27, 2012	21	2.8	8	19	0.7	11	36.2
August 3, 2012	15	2.6	8	19	6.6	22	26.5
September 2, 2012	38	18.4	1	12	5.4	1.0	40.6
September 5, 2012	13	1.5	8	22	2.3	25	26.5
September 8, 2012	20	3.1	5	19	2.3	12	33.7
September 25, 2012 <sup>2</sup>	25	3.1	9	15	7	3	42.1
September 26, 2012	12	3.9	4	17	0.6	53	19.0
September 28, 2012	21	8.8	3	12	0.1	1	27.0
October 1, 2012	8	8.2	0.7	4	2.7	24	10.5
October 14, 2012	6	0.1	95	---	8.7	0.8	33.1
October 26, 2012	6	9.4	0.5	3	7.2	1	7.6
October 30, 2012	6	10.8	0.4	1	3.1	4	7.3
November 3, 2012	4	3.4	2	11	0.1	0.8	6.6

Table D.1 cont'd.

November 12, 2012 <sup>2</sup>	29	9.5	3	13	4.8	2	36.6
November 26, 2012	8	5.4	1	4	14.2	21	11.7
December 7, 2012	16	16.1	1	5	0.2	2	17.7
December 8, 2012	8	7.7	0.5	3	0.4	0.8	10.7
December 9, 2012a	9	5.4	1	6	0.2	4	13.1
December 9, 2012b	40	18.1	3	22	0.2	7	43.0
December 17, 2012a	8	1.5	3	6	1.5	1	16.3
December 17, 2012b	15	5.0	3	14	0.2	5	22.3
December 20, 2012	32	10.2	3	12	2.4	12	39.7
December 25, 2012	11	15.8	0.7	4	1.7	3	12.2
December 31, 2012	15	28.3	0.5	3	1.9	3	14.4
January 11, 2013	11	2.8	3	16	0.2	0.5	18.9
January 13, 2013	56	15.4	4	17	0.5	4	62.6
January 16, 2013	6	14.0	4	10	0.3	0.5	6.9
January 28, 2013	5	3.4	1	4	0.5	5	8.2
January 30, 2013a	27	6.9	4	25	1.6	4	37.0
January 30, 2013b	4	3.1	2	6	0.3	26	6.7
February 21, 2013	11	12.8	2	7	2.7	5	12.9
February 26, 2013	9	19.8	0.5	5	3.8	19	9.4
March 5, 2013	18	13.4	1	6	3.3	0.5	20.8
March 11, 2013	24	8.8	2	6	3.8	0.5	30.9
March 17, 2013	19	18.3	1	5	0.2	1	20.3
March 18, 2013	46	6.3	6	12	0.3	19	64.4
March 24, 2013a	19	7.6	2	7	5.7	39	25.4
March 24, 2013b	18	7.4	2	11	0.1	17	24.2
April 11, 2013	20	4.0	5	17	0.2	3	31.5
April 12, 2013	12	2.8	4	12	0.1	19	20.7

Table D.1 cont'd.

April 17, 2013	26	8.3	3	24	0.1	2	34.0
April 19, 2013	18	4.8	4	9	1.6	26	27.0
April 24, 2013	31	6.4	5	19	5.0	18	43.3
April 27, 2013	15	5.4	3	10	3.4	32	21.9
April 28, 2013	15	2.3	6	17	0.5	18	27.4
Mean±Stdev	17±11	7.3±6.0	6±13	12±8	--	--	--
Median	15	5.8	3	11	--	--	--

<sup>1</sup> Indicates only data from the Town Branch at Yarnallton Road gage station (03289200) were used.

<sup>2</sup> Indicates only data from the Cane Run Creek at Citation Boulevard gage station (03288180) were used.

<sup>3</sup>30-minute peak intensity not computed as duration was less than 30 minutes.



**APPENDIX E: STORM EVENT HYDROGRAPHS, HYETOGRAPHS AND  
WATER LEVEL**

Figure E.1. Event occurring May 5, 2012.

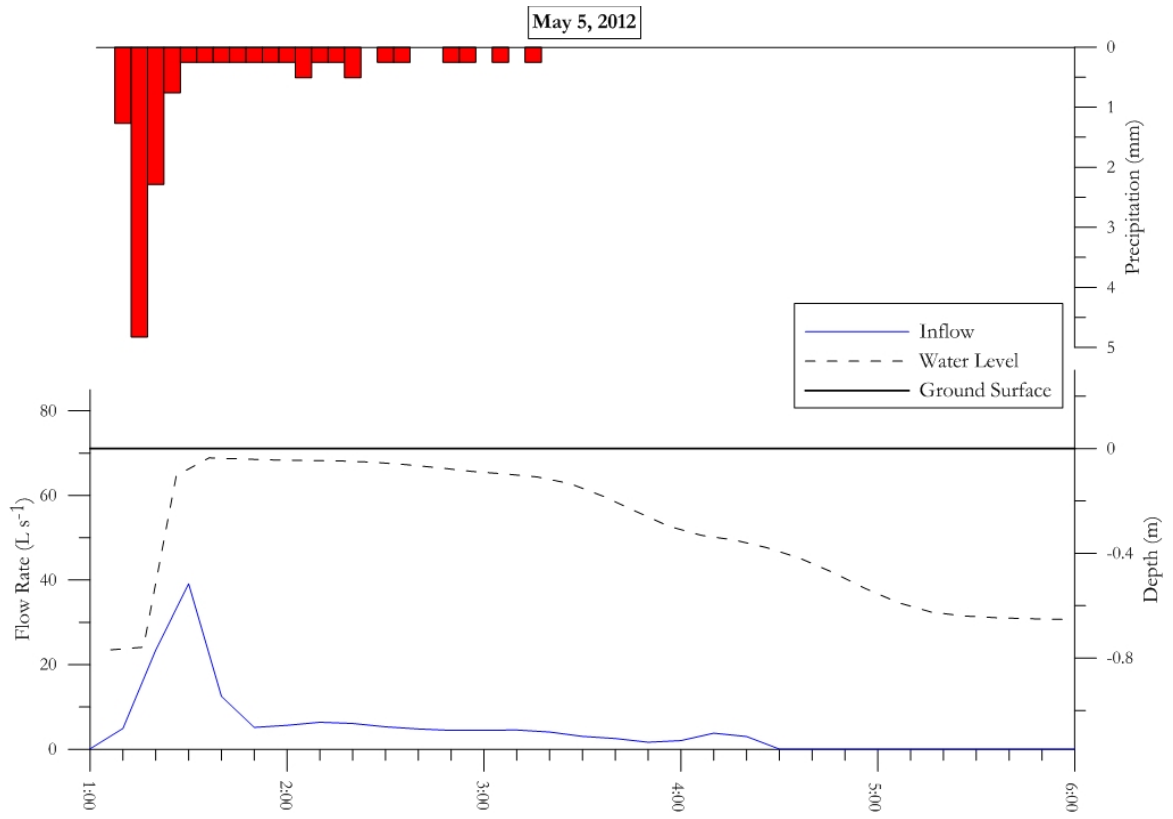


Figure E.2. Event occurring May 14, 2012.

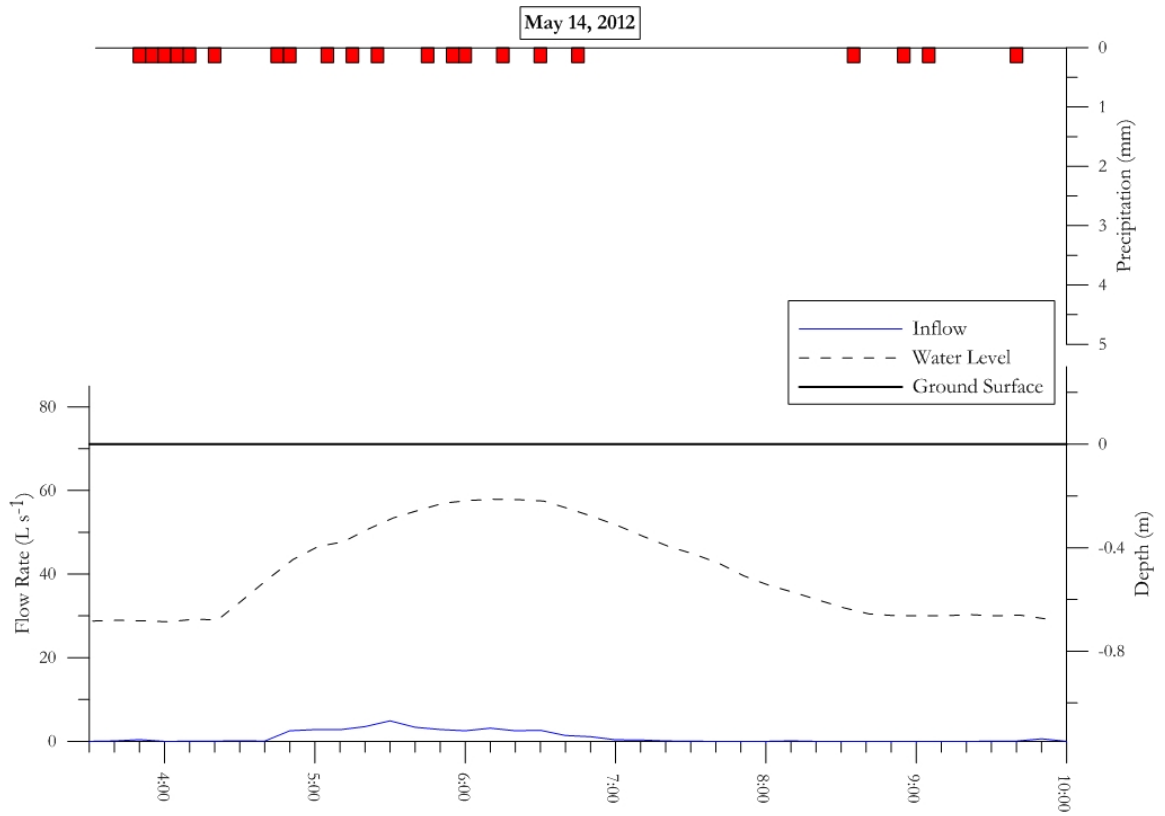


Figure E.3. Event occurring May 31 to June 1, 2012.

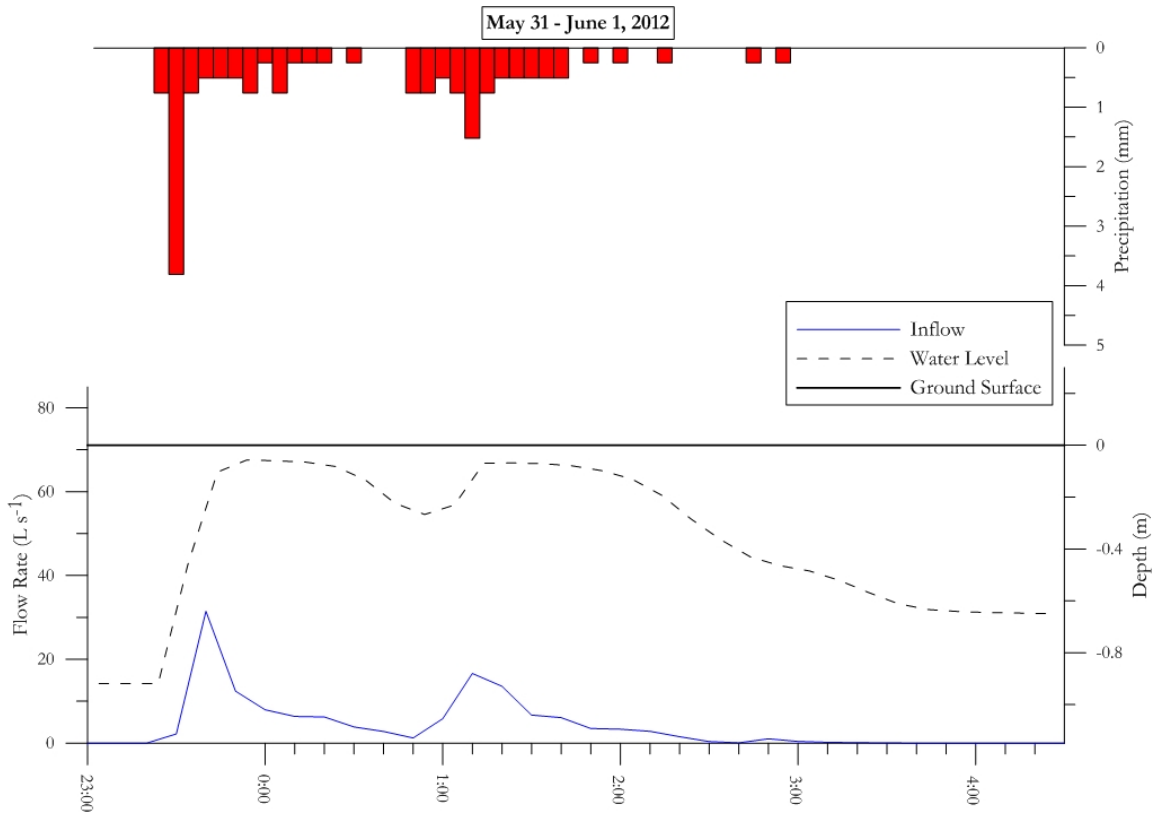


Figure E.4. Event occurring June 11, 2012.

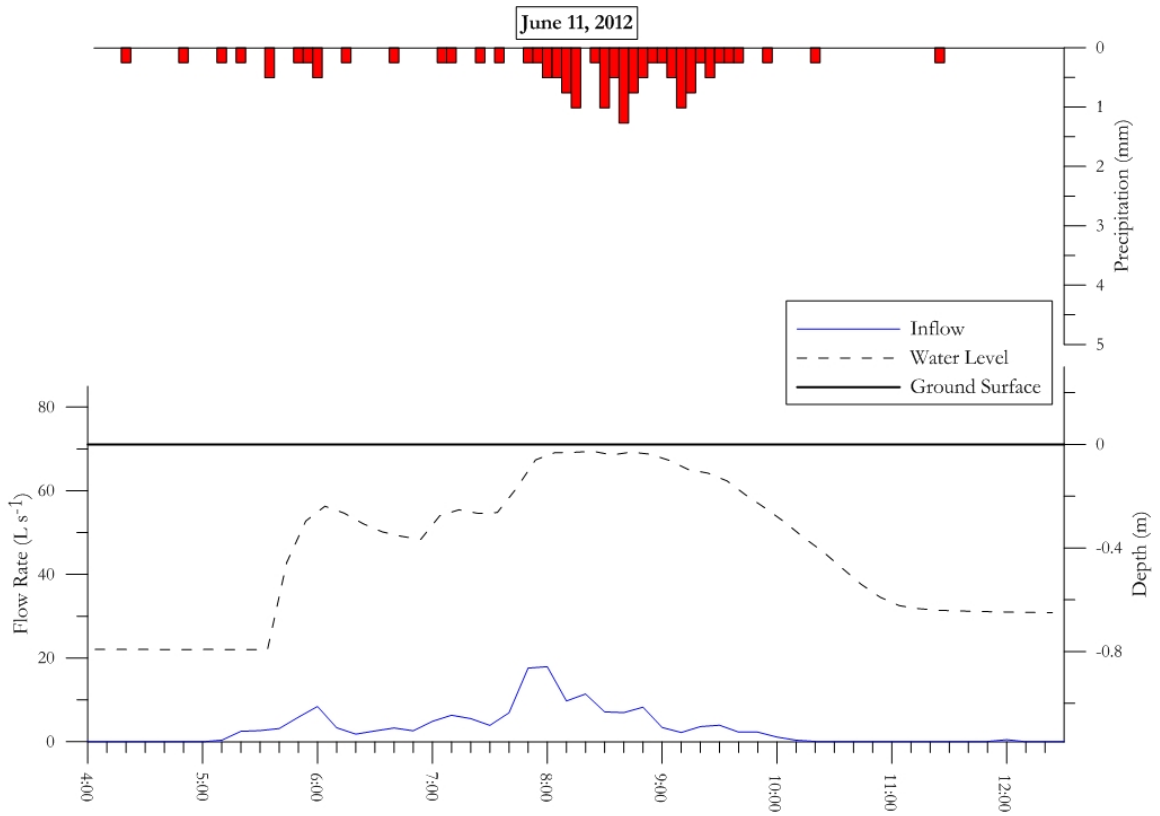


Figure E.5. First event occurring June 17, 2012.

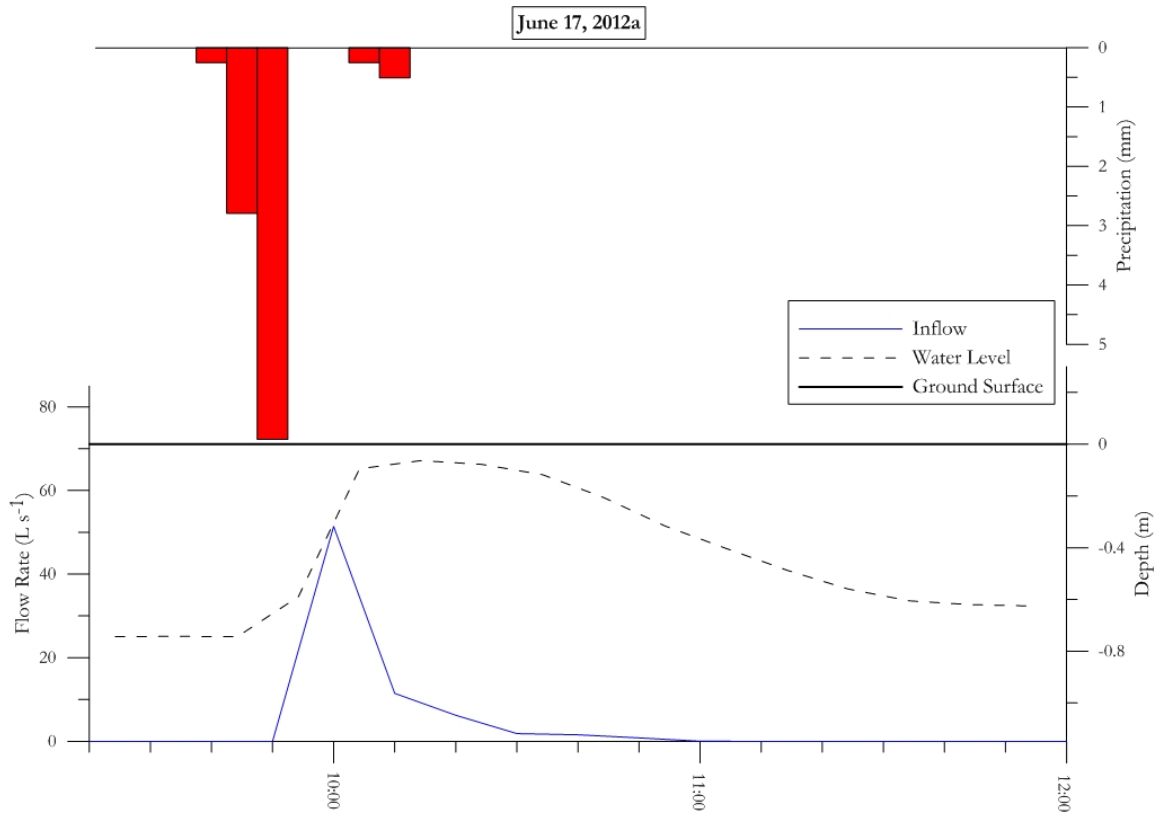


Figure E.6. Second event occurring June 17, 2012.

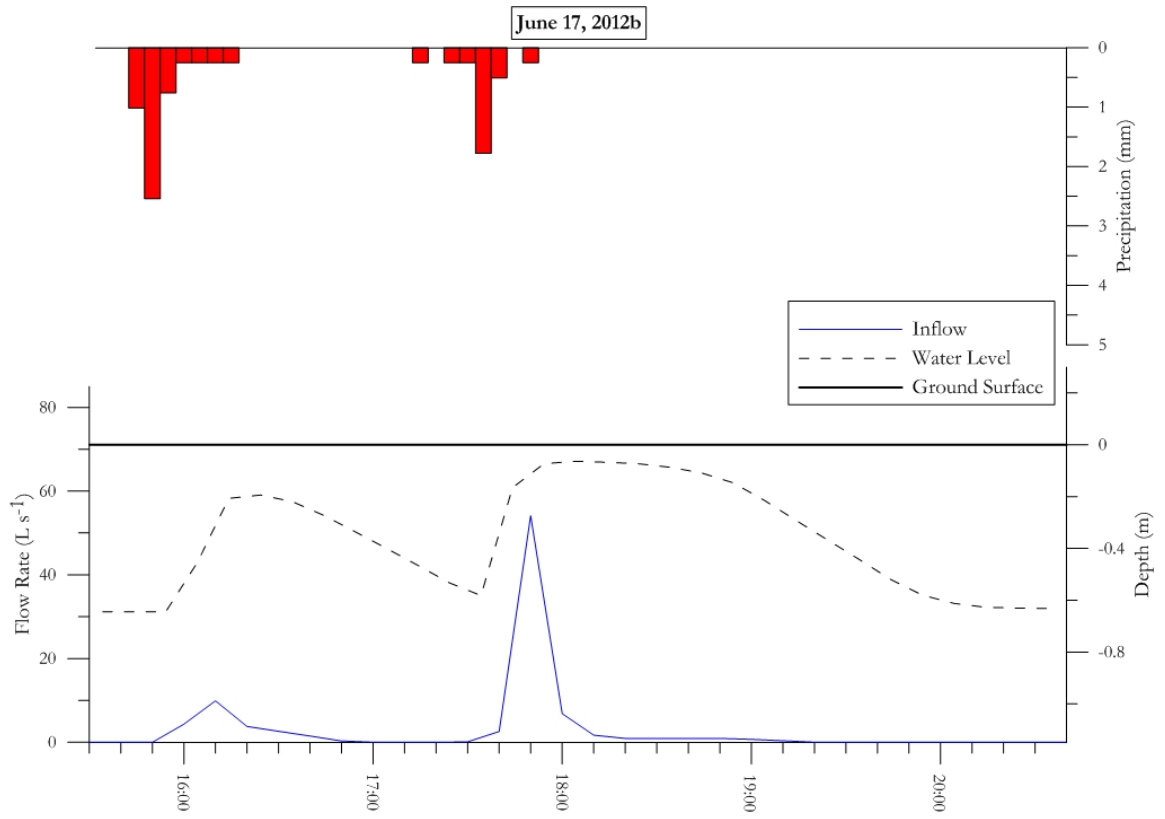


Figure E.7. Event occurring July 1 to July 2, 2012

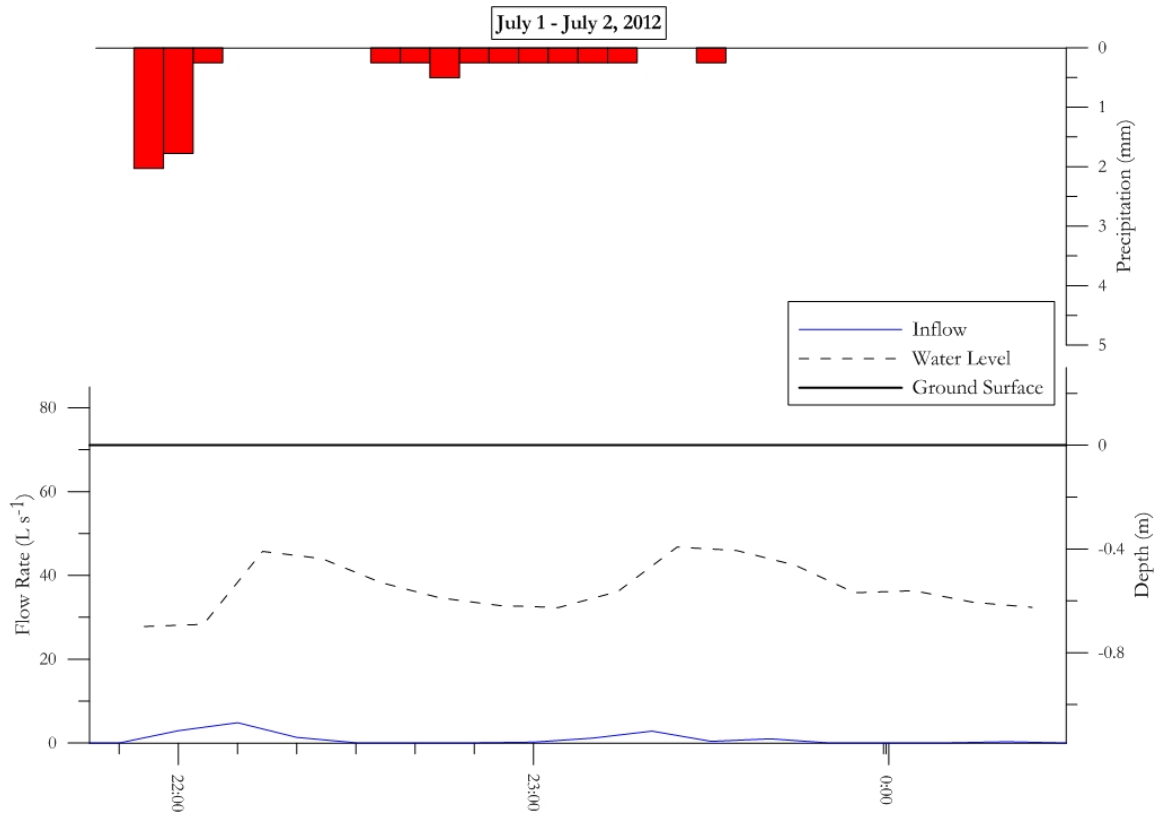




Figure E.8. Event occurring July 14, 2012

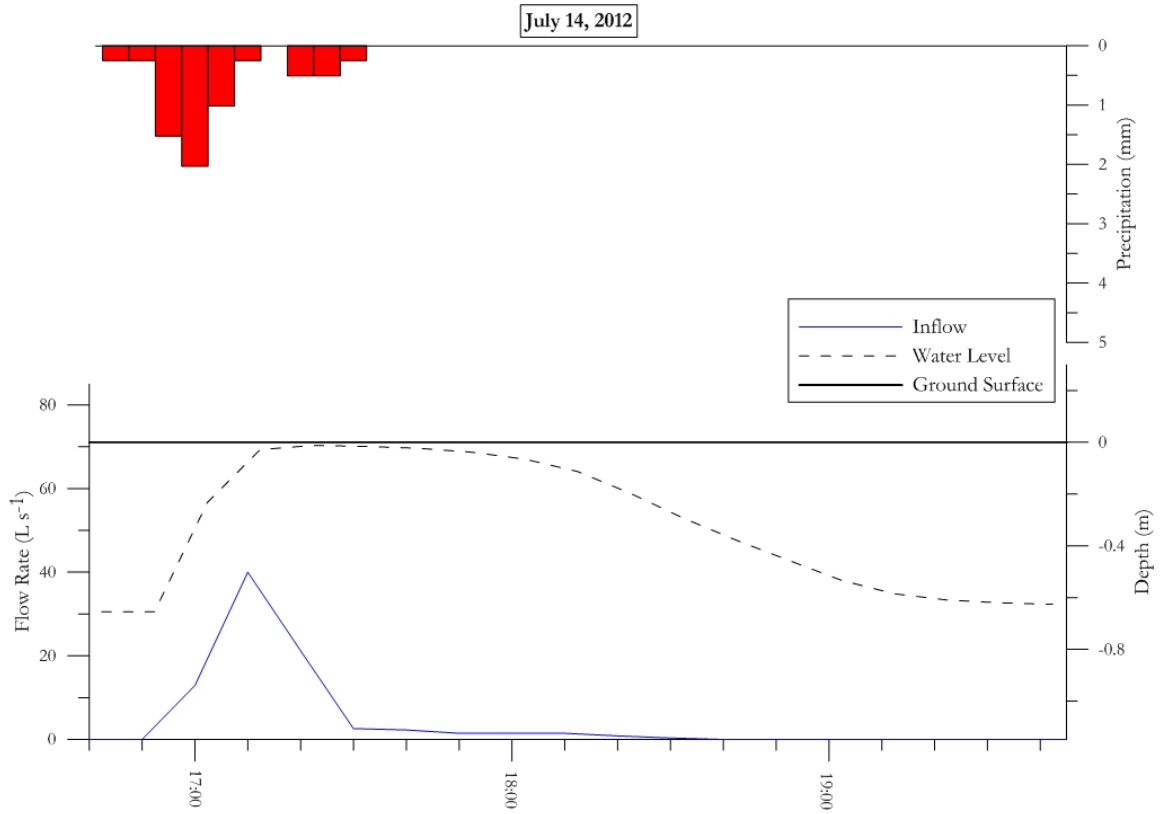


Figure E.9. Event occurring July 18, 2012. \*Note that only flow from the 25 cm pipe was recorded.

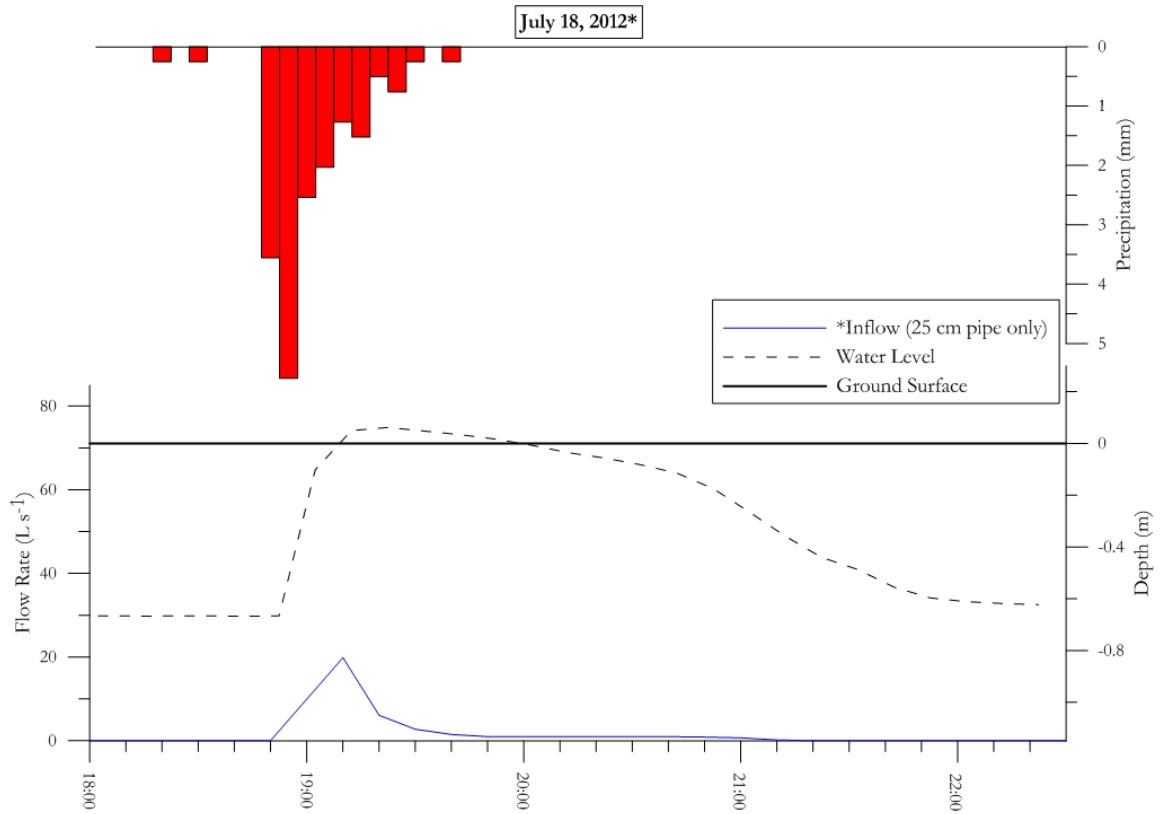


Figure E.10. Event occurring July 19, 2012. \*Note that only flow from the 25 cm pipe was recorded.

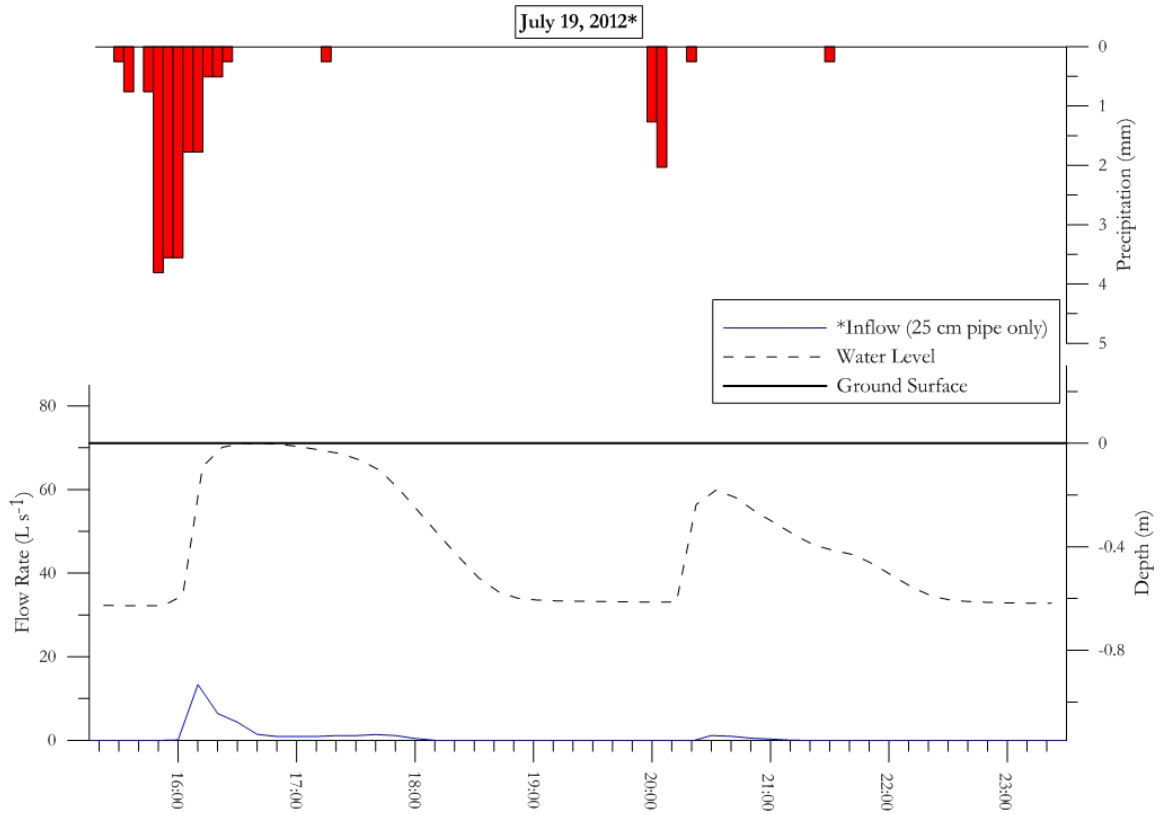


Figure E.11. Event occurring July 26 to July 27, 2012. \*Note that only flow from the 25 cm pipe was recorded.

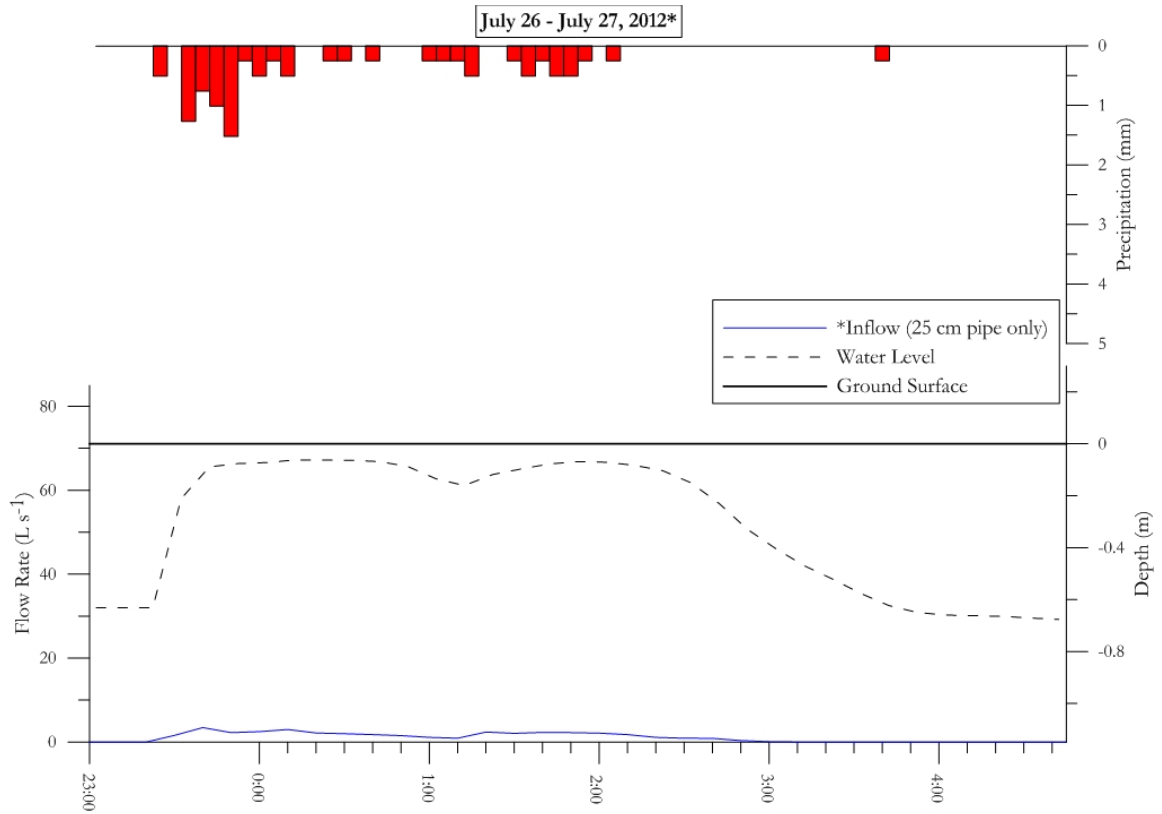


Figure E.12. Event occurring July 27 to July 28, 2012. \*Note that only flow from the 25 cm pipe was recorded.

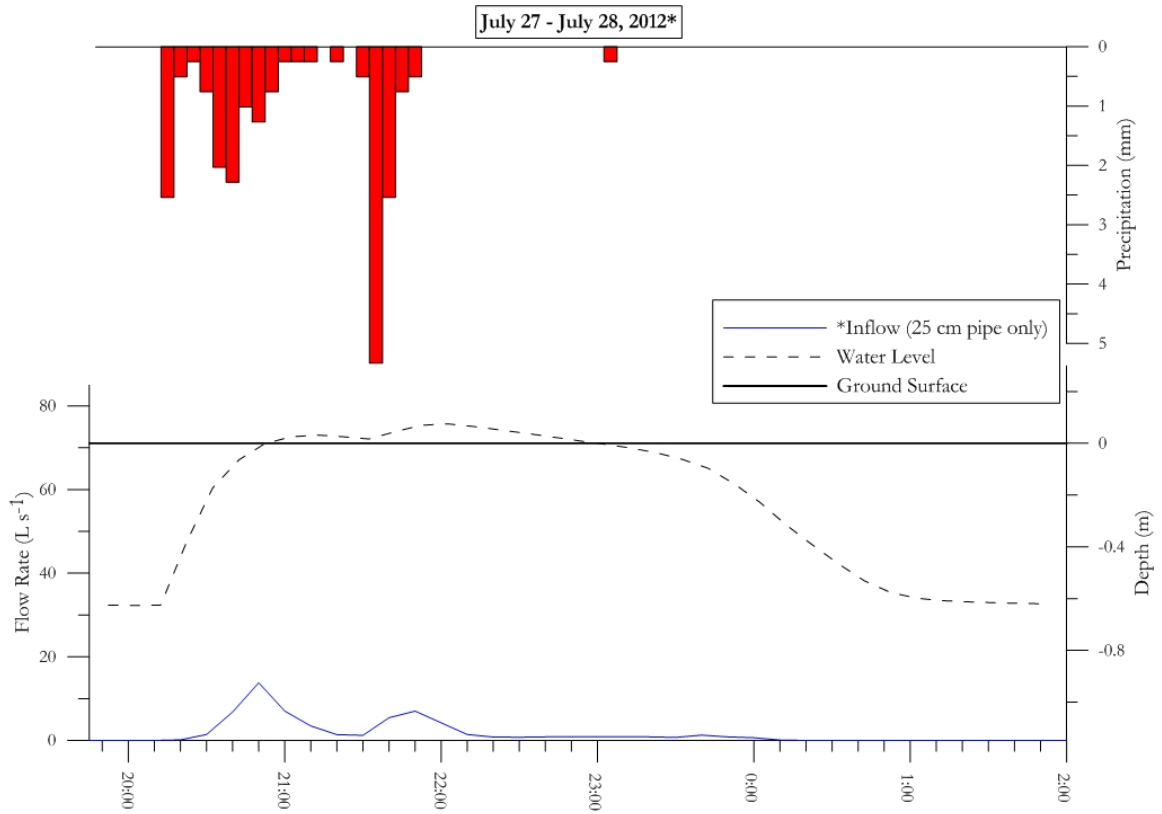
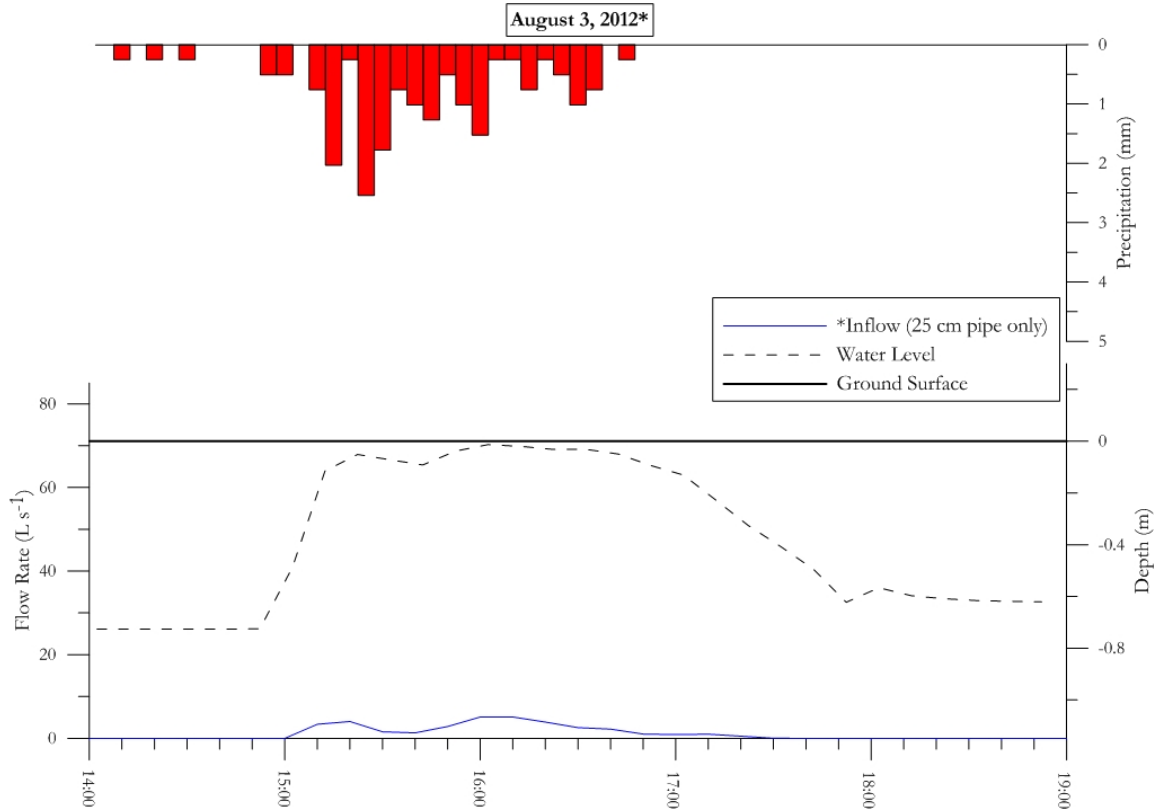


Figure E.13. Event occurring August 3, 2012. \*Note that only flow from the 25 cm pipe was recorded.



Event E.14. Event occurring September 2 – September 3, 2012.

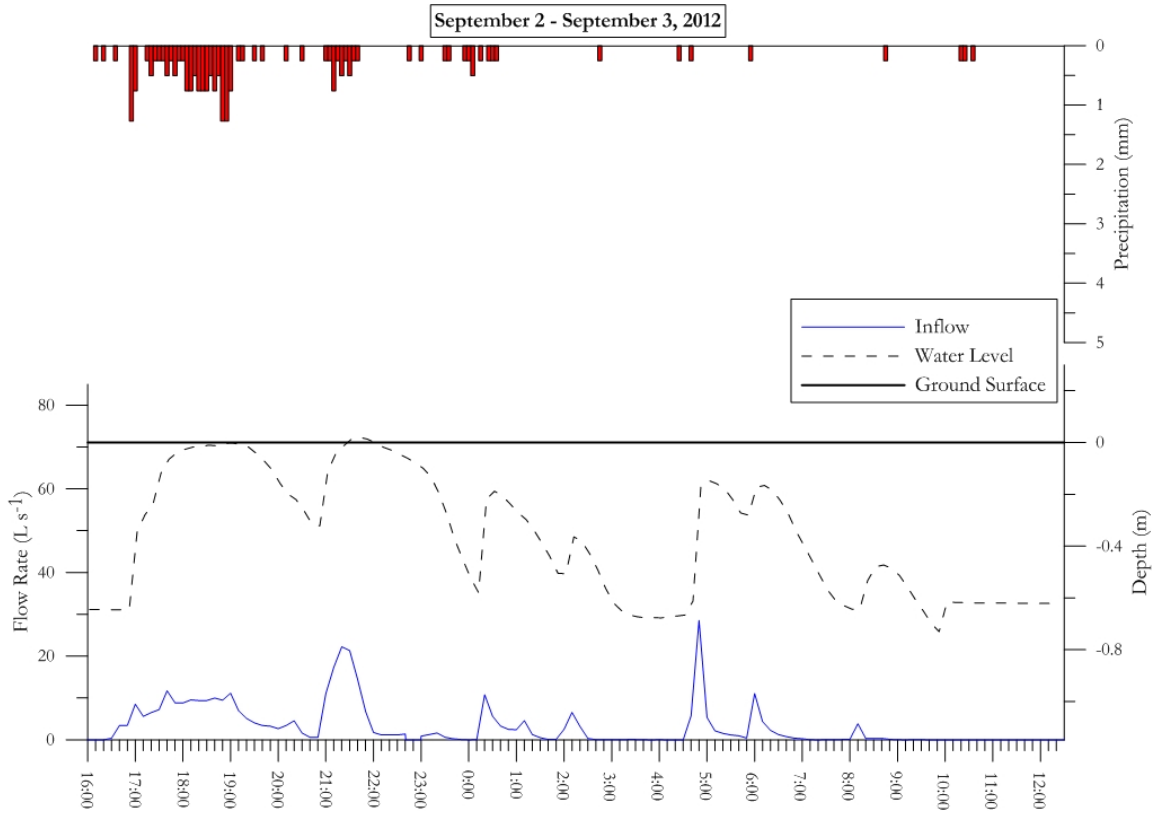


Figure E.15. Event occurring September 5, 2012.

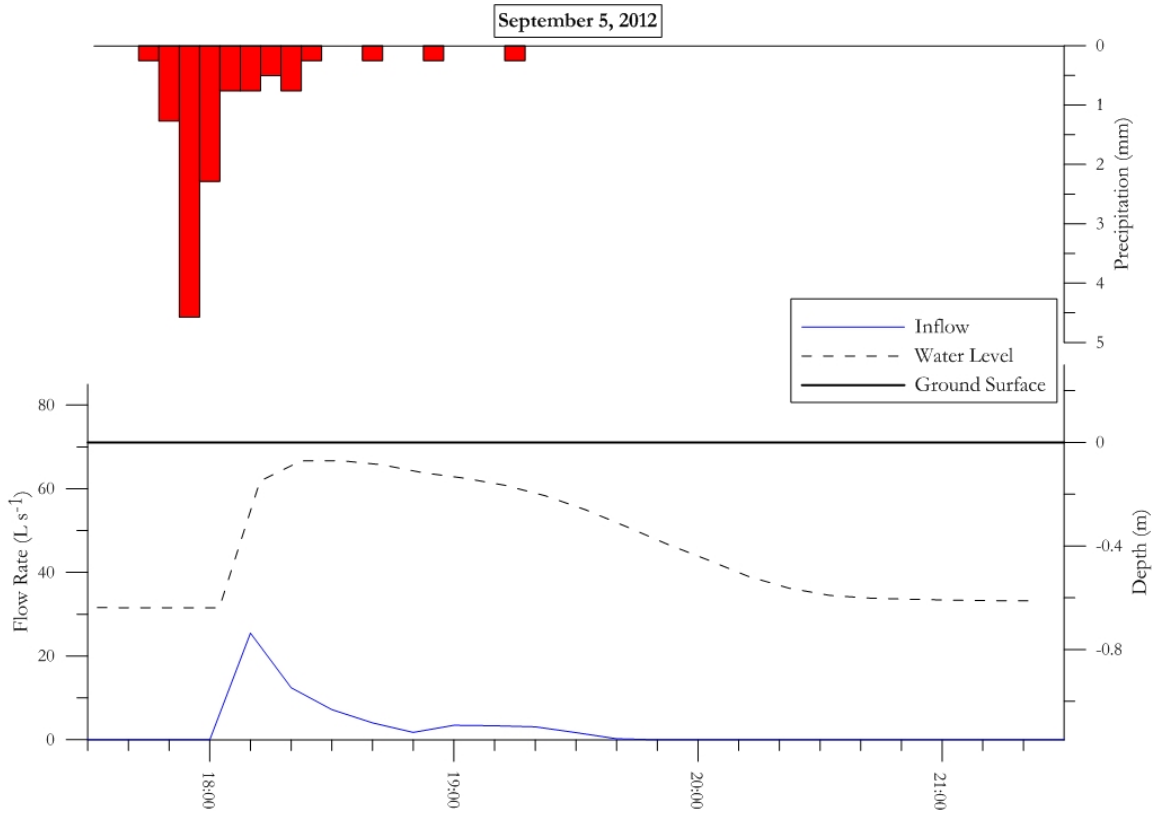




Figure E.16. Event occurring September 8, 2012.

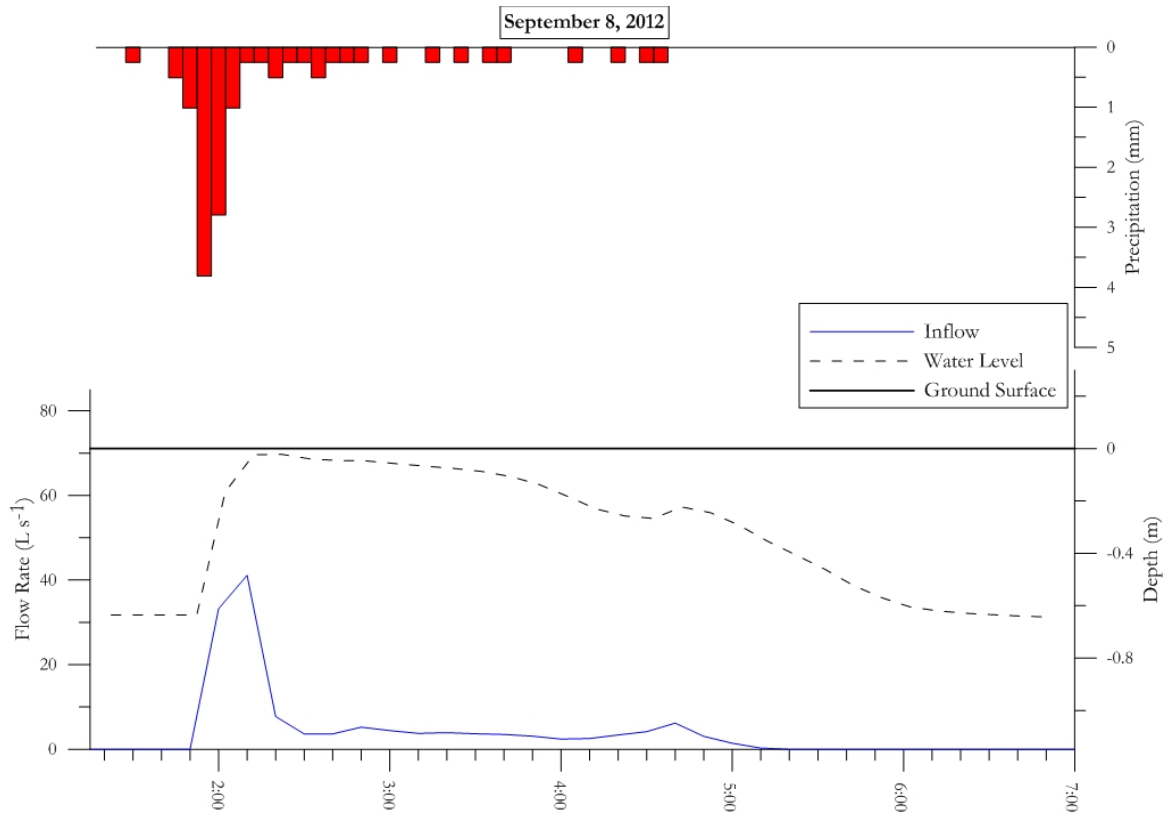


Figure E.17. Event occurring September 25, 2012.

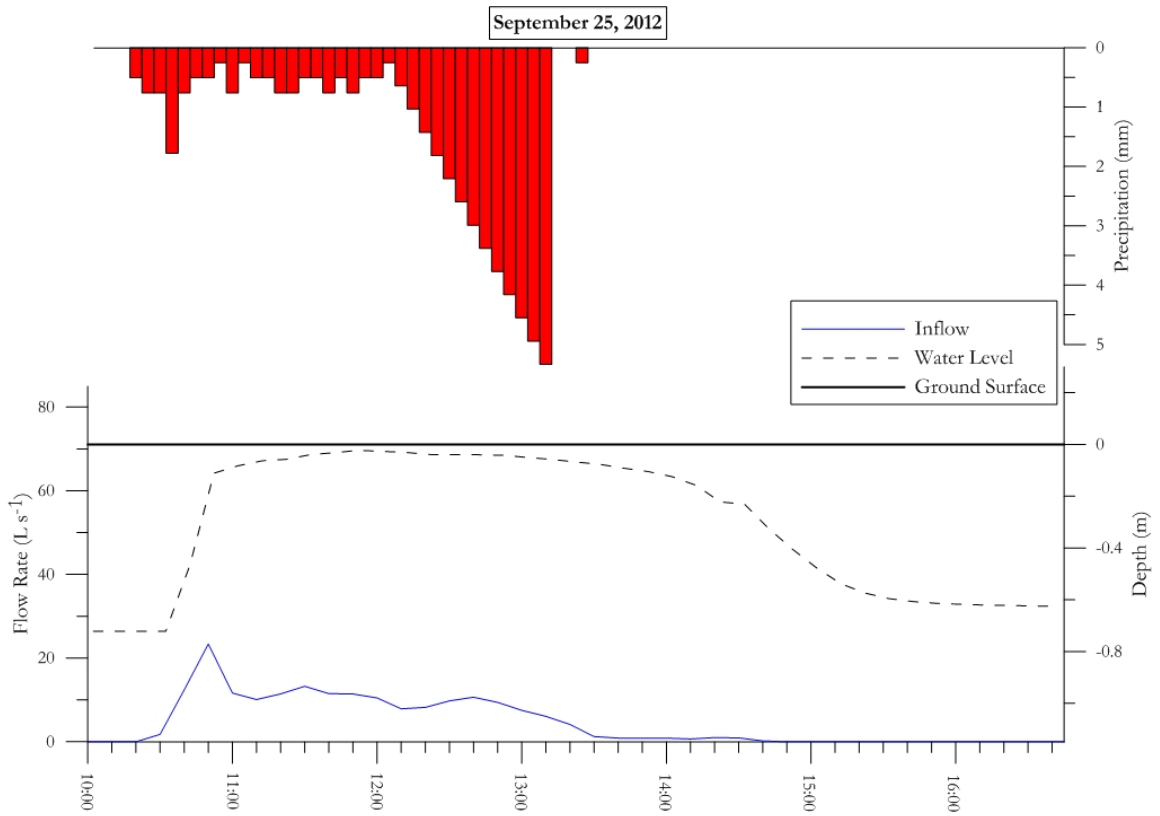


Figure E.18. Event occurring September 26, 2012.

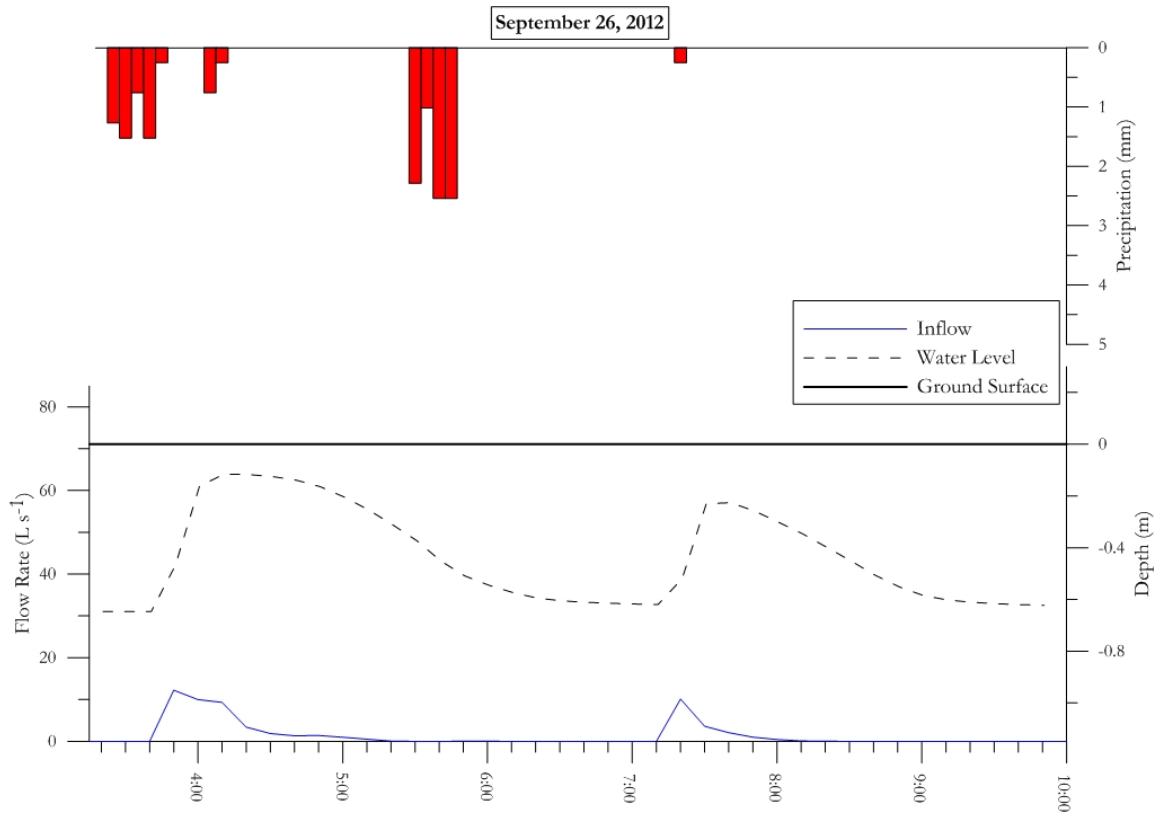


Figure E.19. Event occurring September 28, 2012.

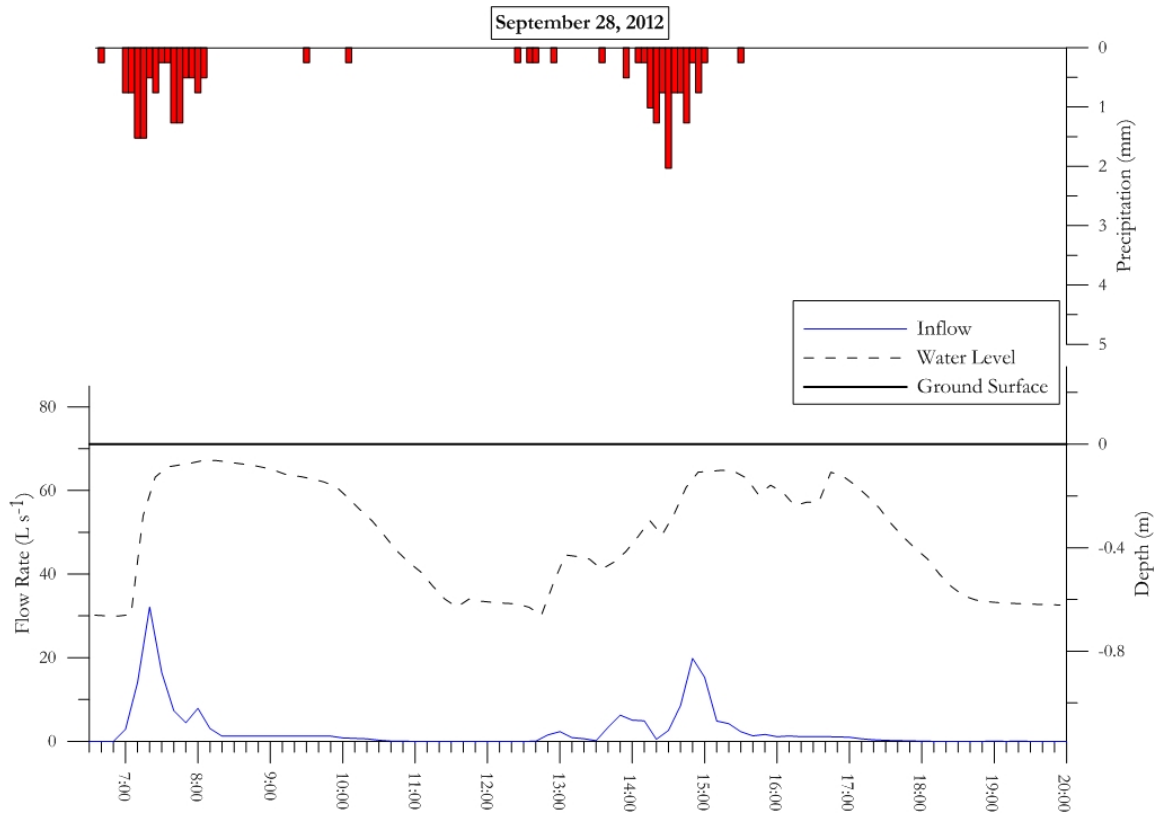


Figure E.20. Event occurring October 1, 2012.

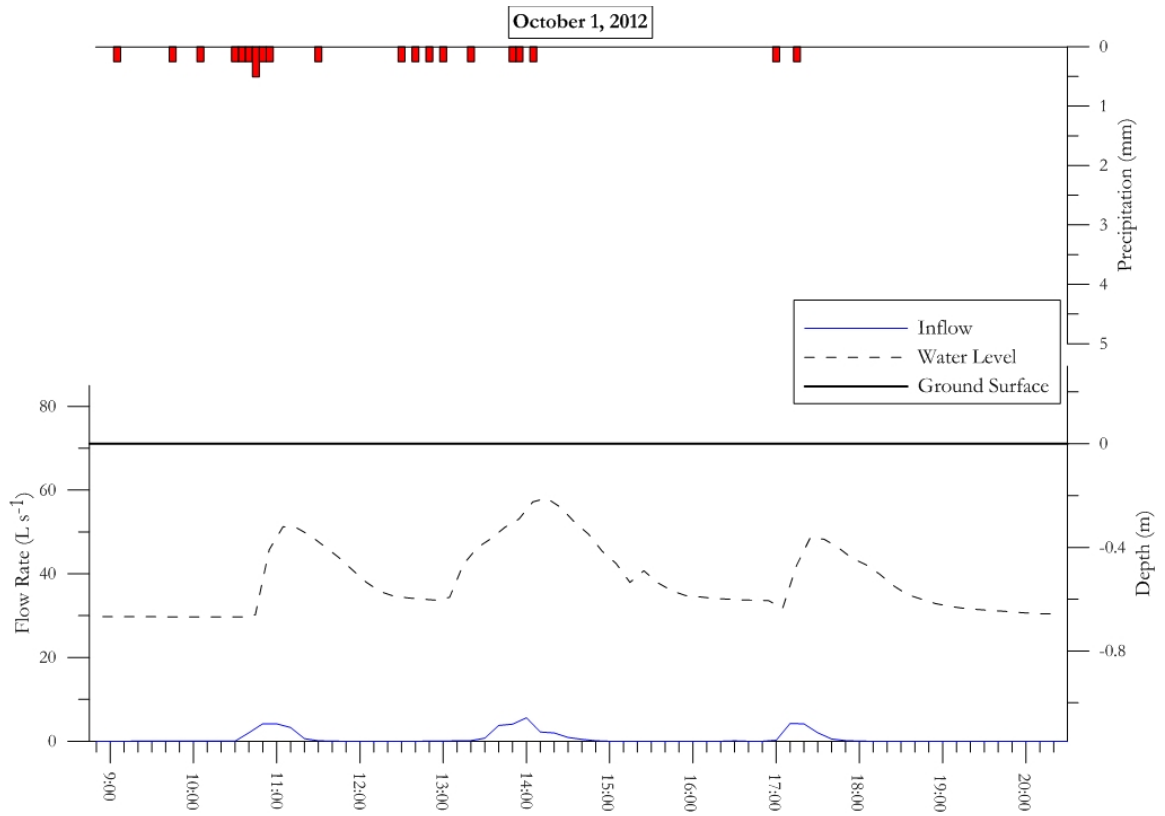


Figure E.21. Event occurring October 14, 2012.

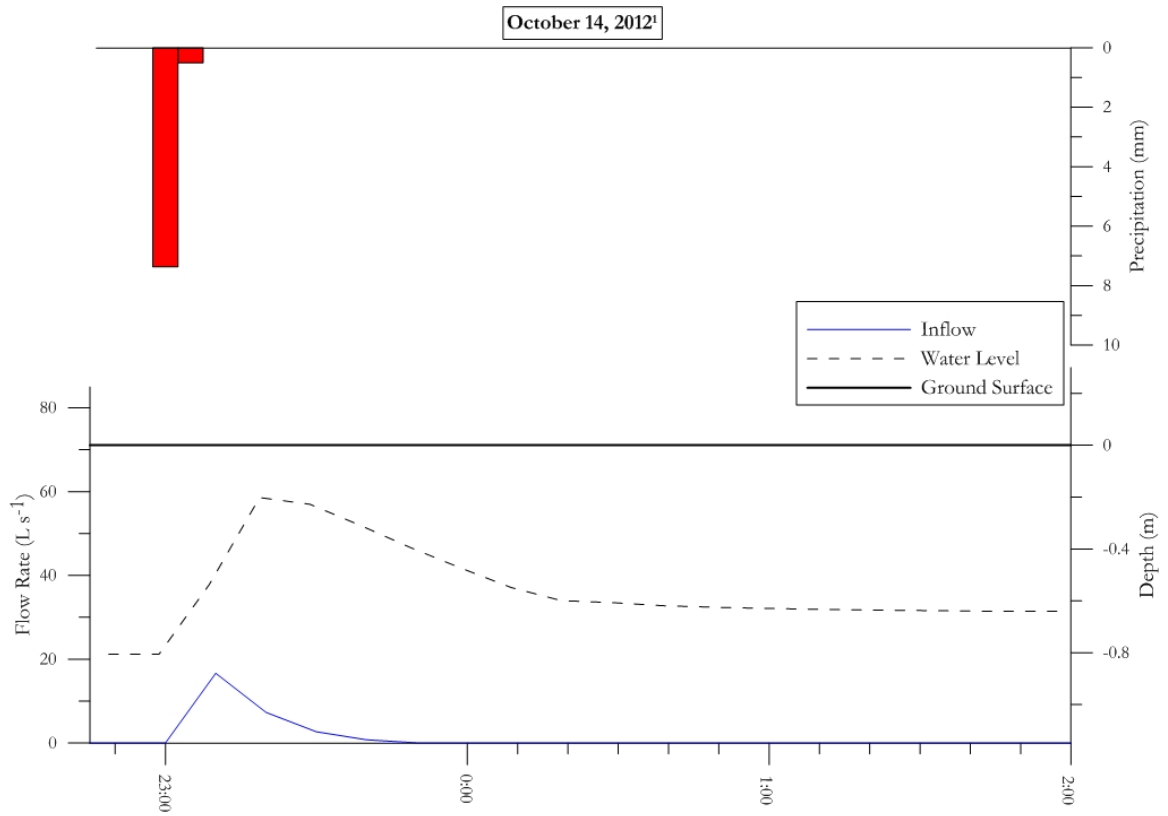


Figure E.22. Event occurring October 26 – October 27, 2012.

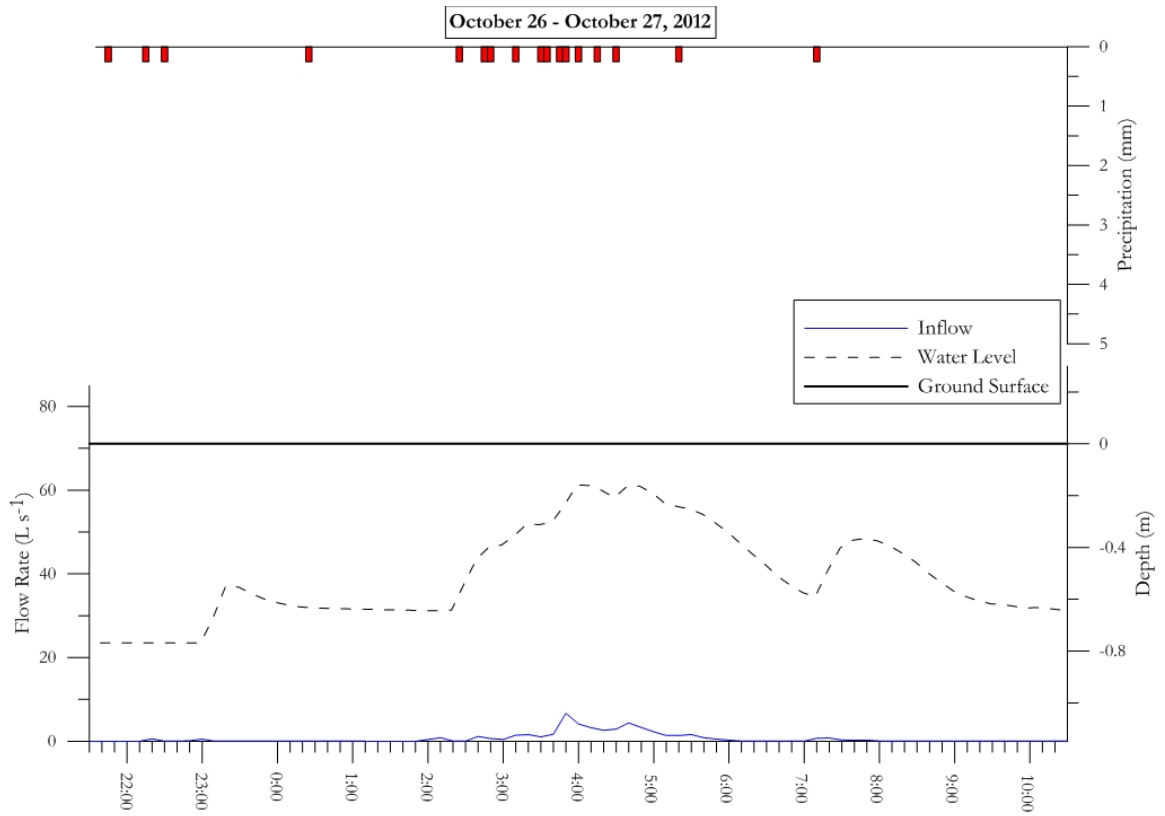


Figure E.23. Event occurring October 30, 2012.

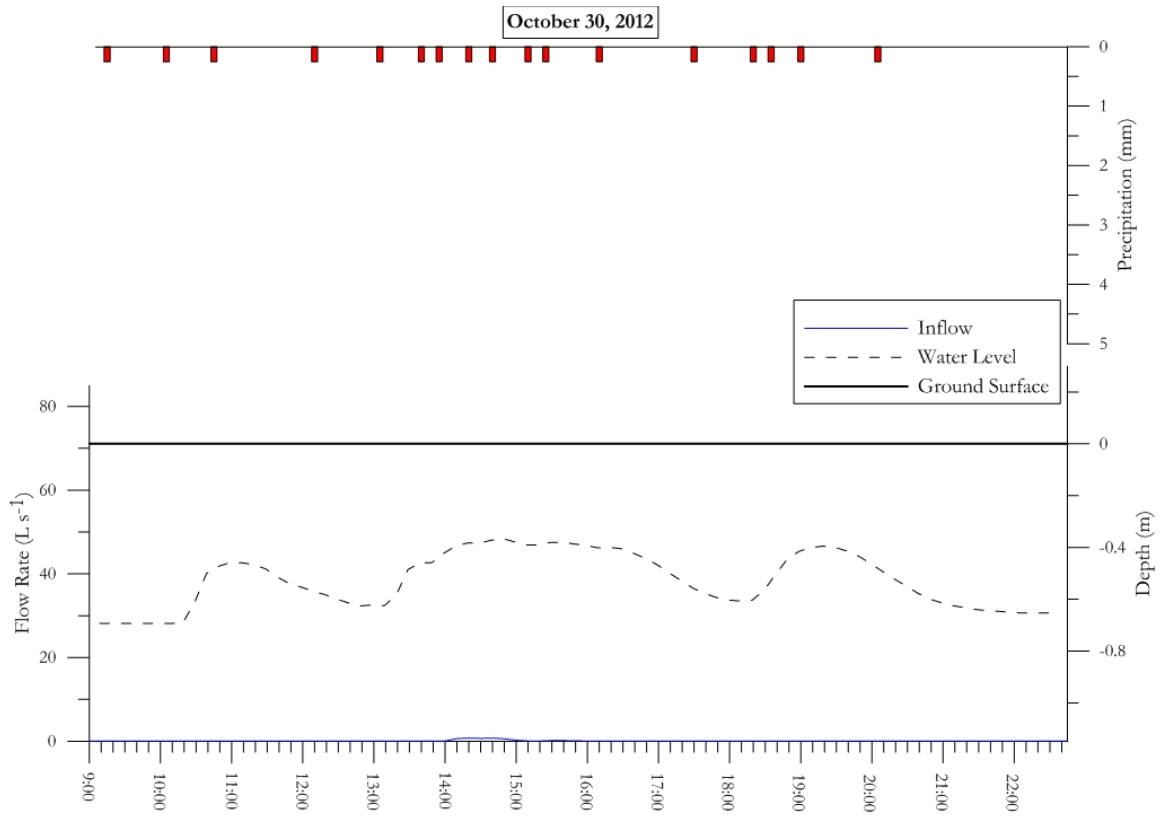




Figure E.24. Event occurring November 3, 2012.

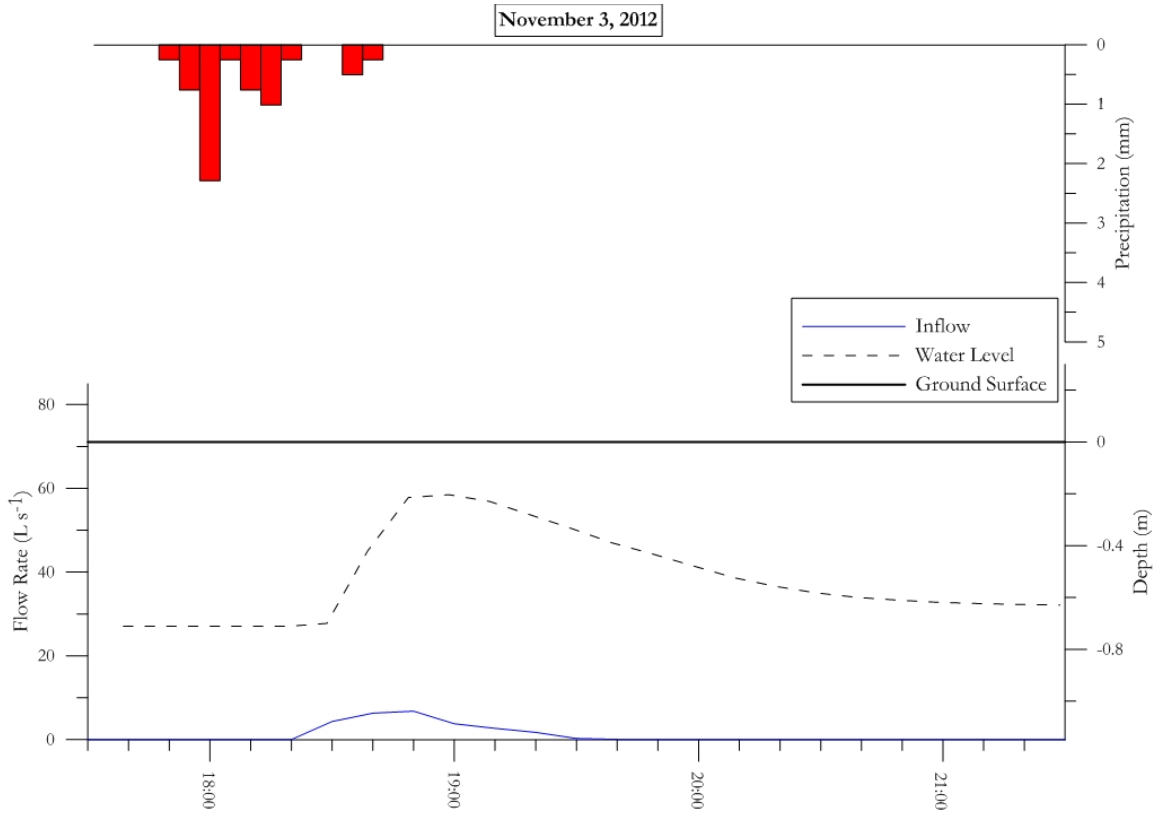


Figure E.25. Event occurring November 12, 2012.

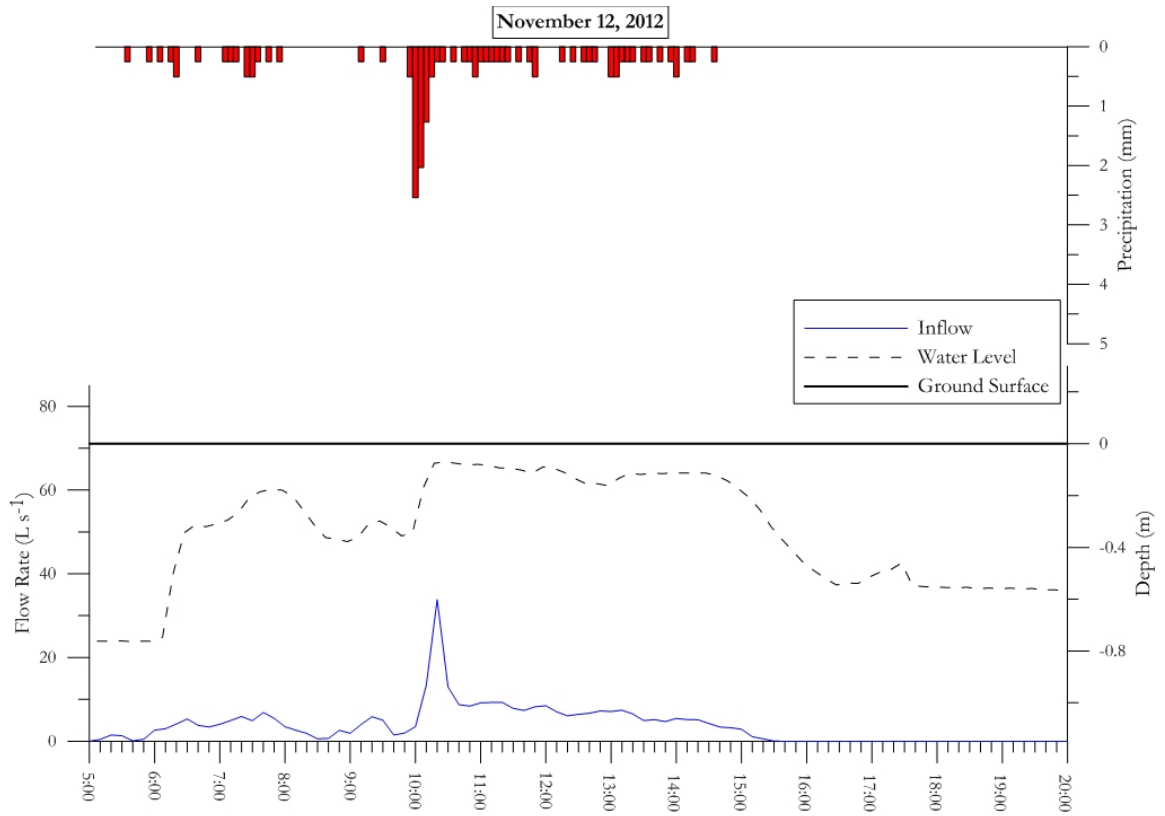


Figure E.26. Event occurring November 26 – November 27, 2012.

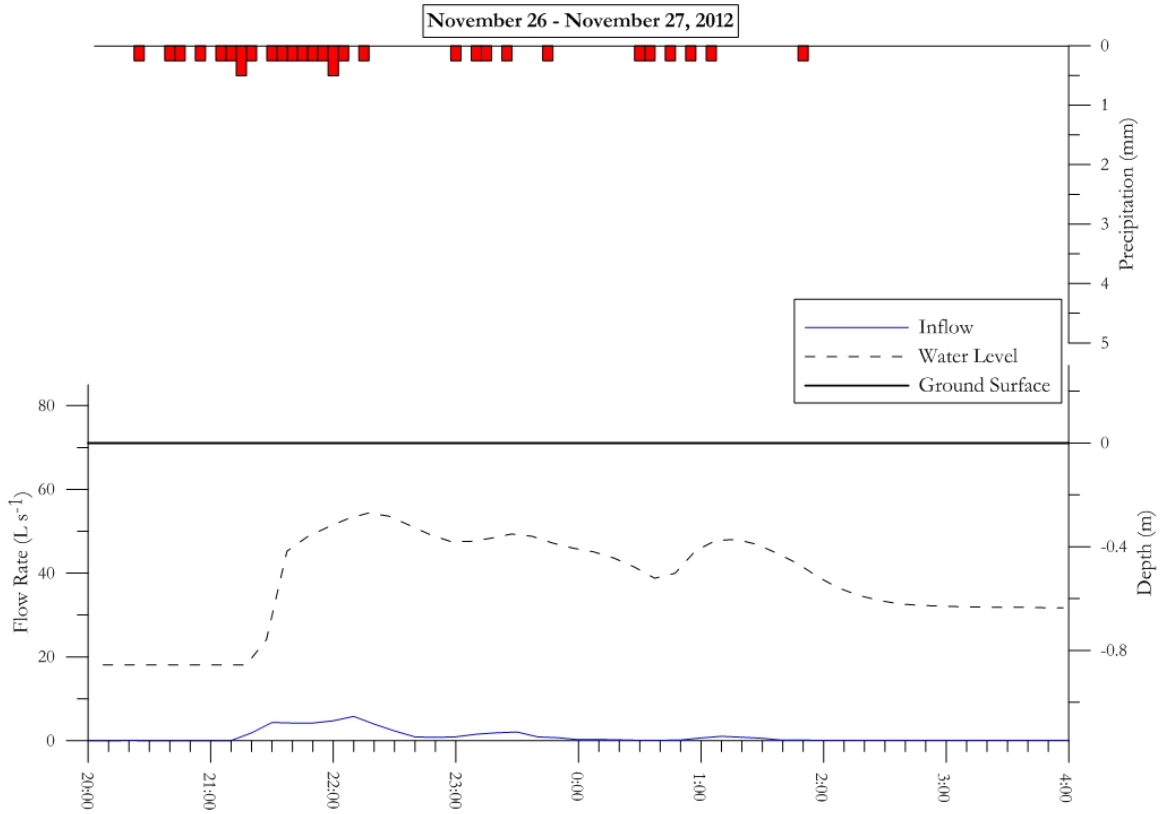


Figure E.27. Event occurring December 7 to December 8, 2012.

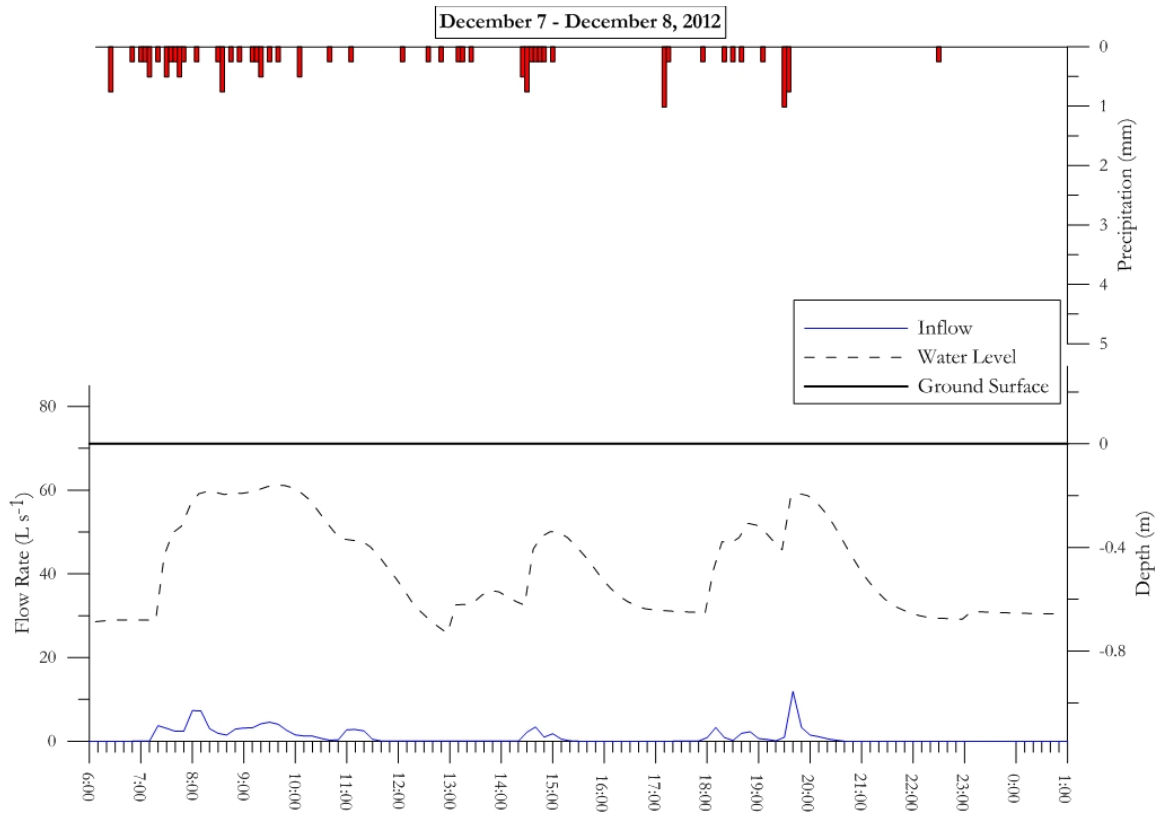


Figure E.28. Event occurring December 8 to December 9, 2012.

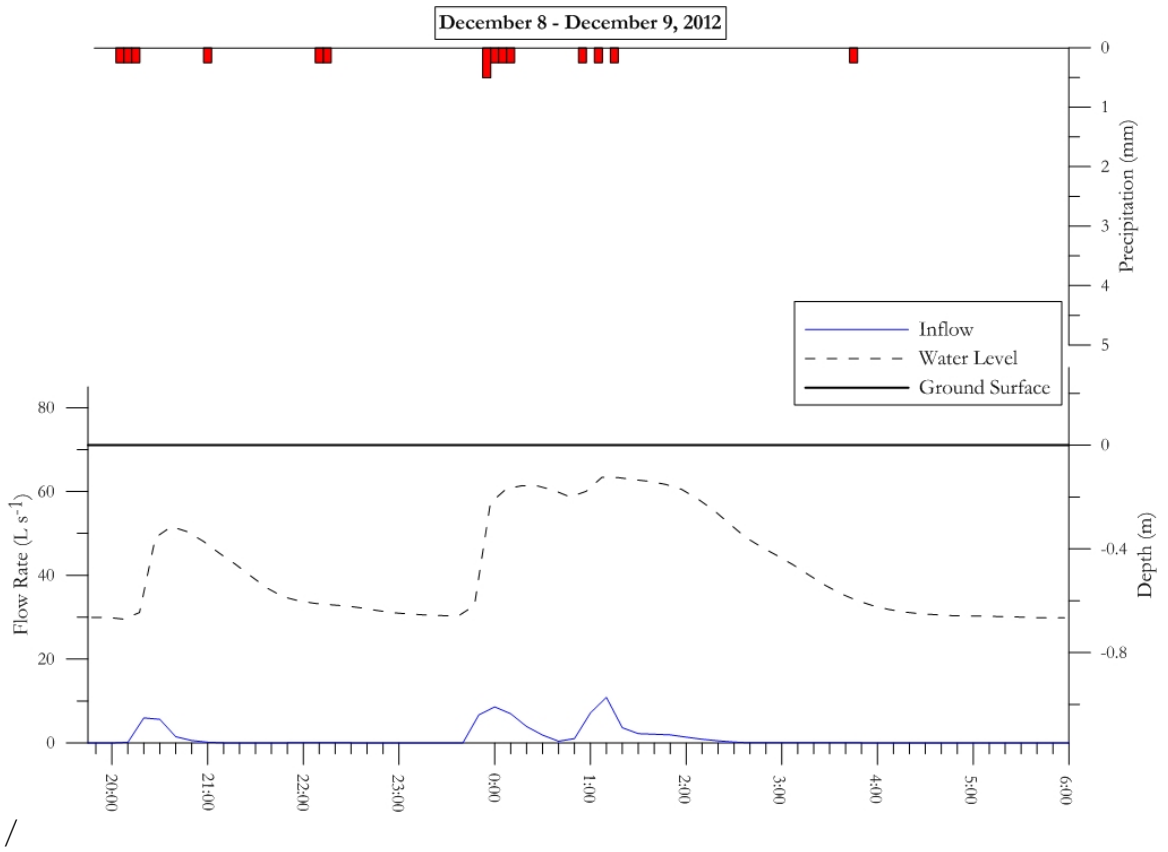


Figure E.29. Event occurring December 9 to December 10, 2012

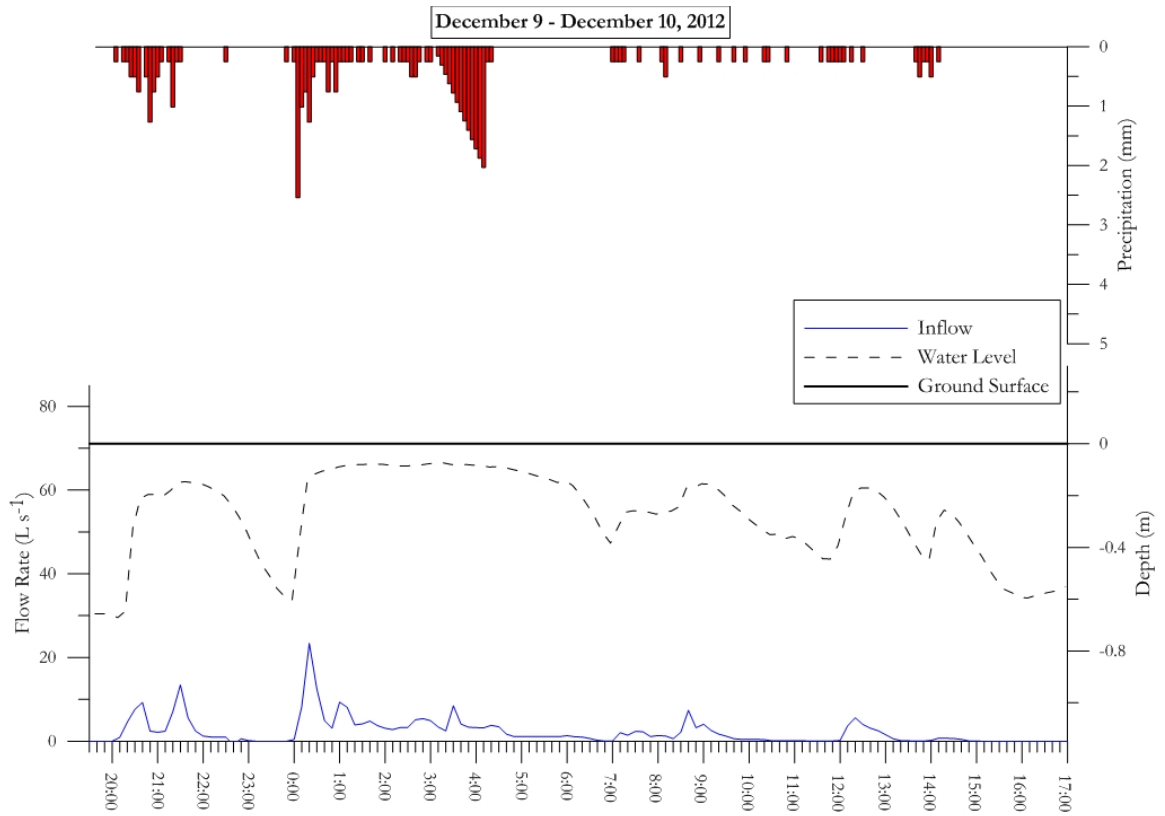


Figure E.30. Event occurring December 9, 2012

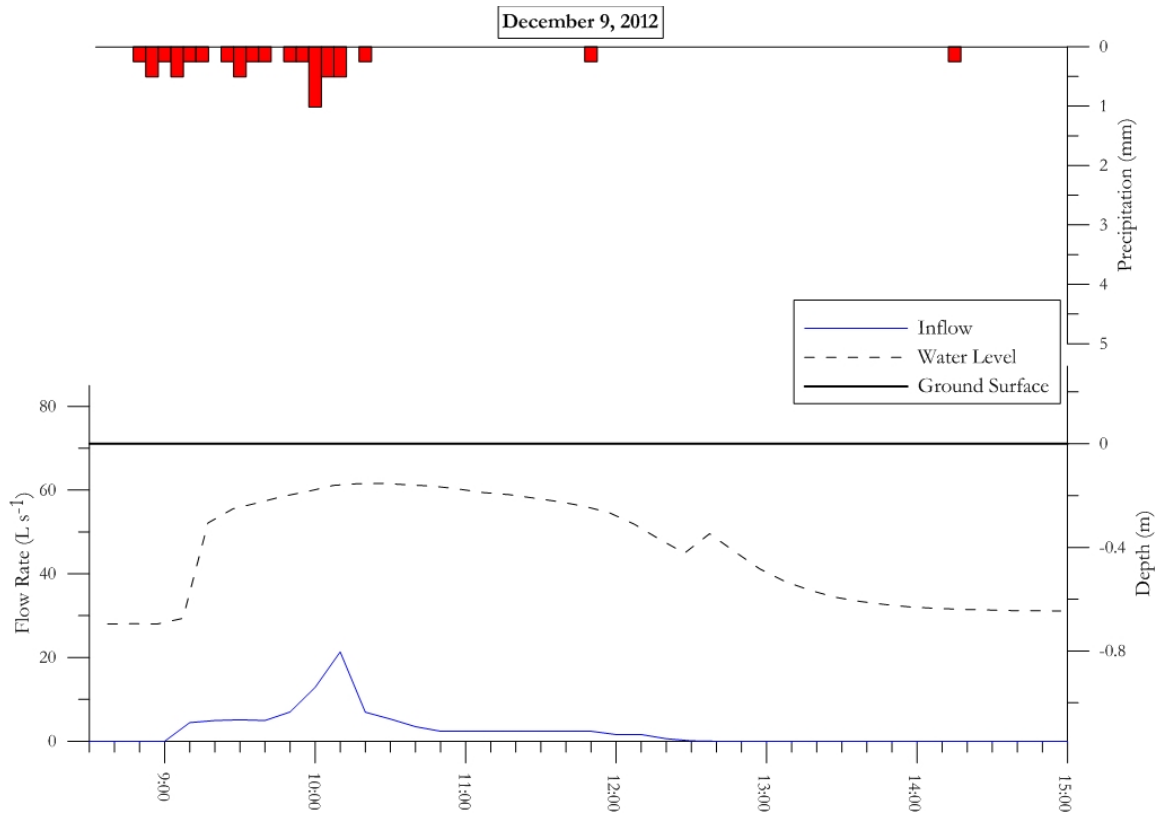


Figure E.31. Event occurring December 17 – December 18, 2012

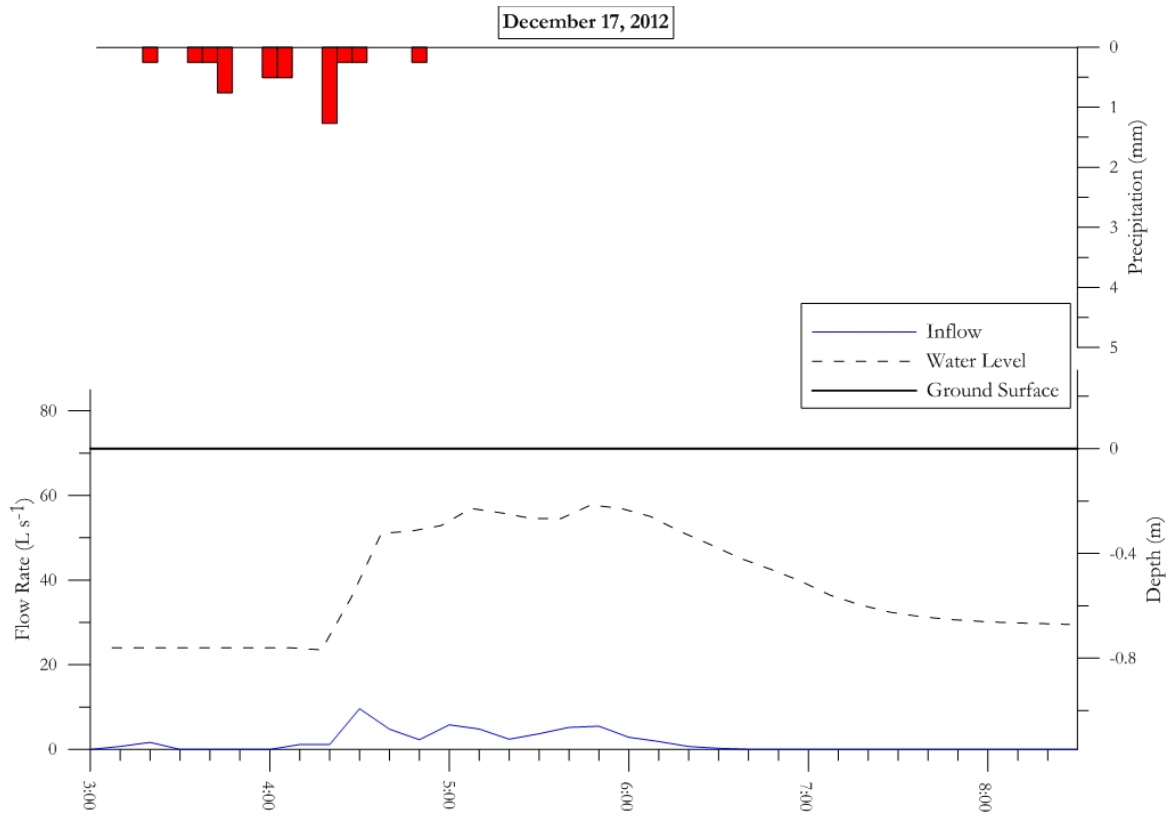




Figure E.32. Event occurring December 17, 2012

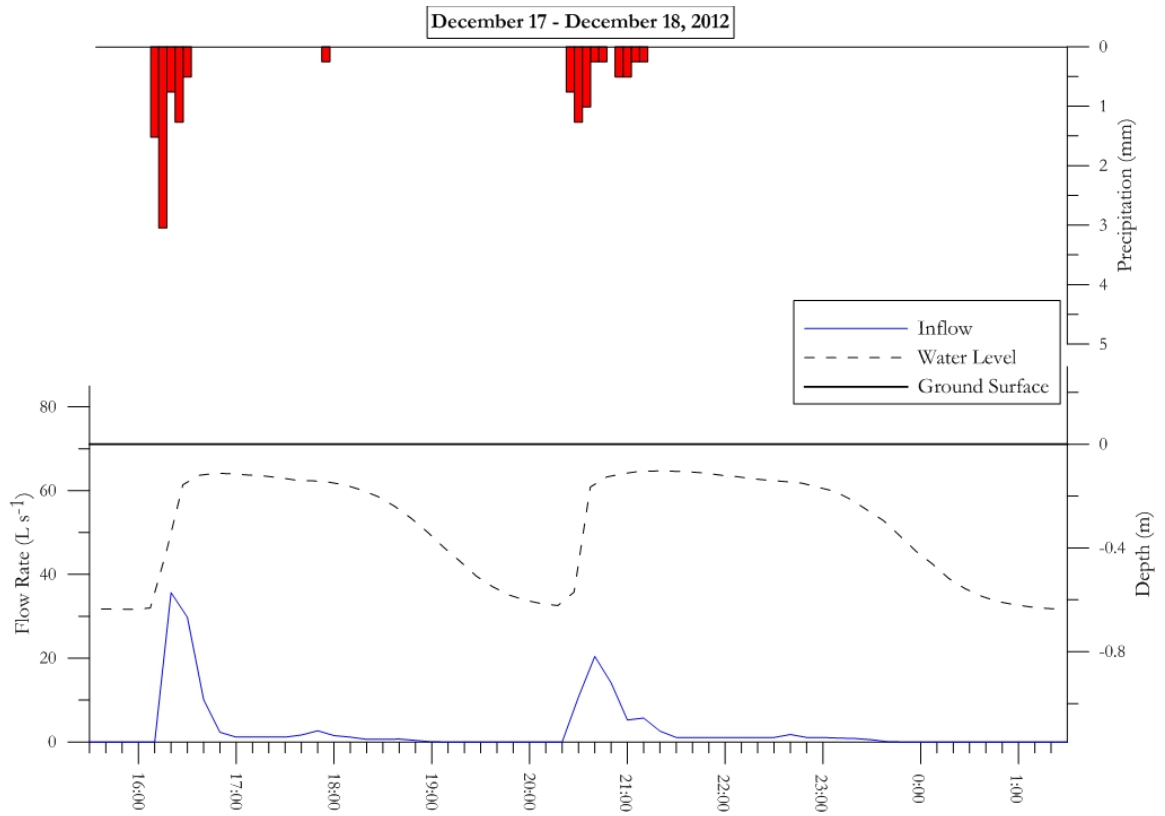


Figure E.33. Event occurring December 20, 2012.

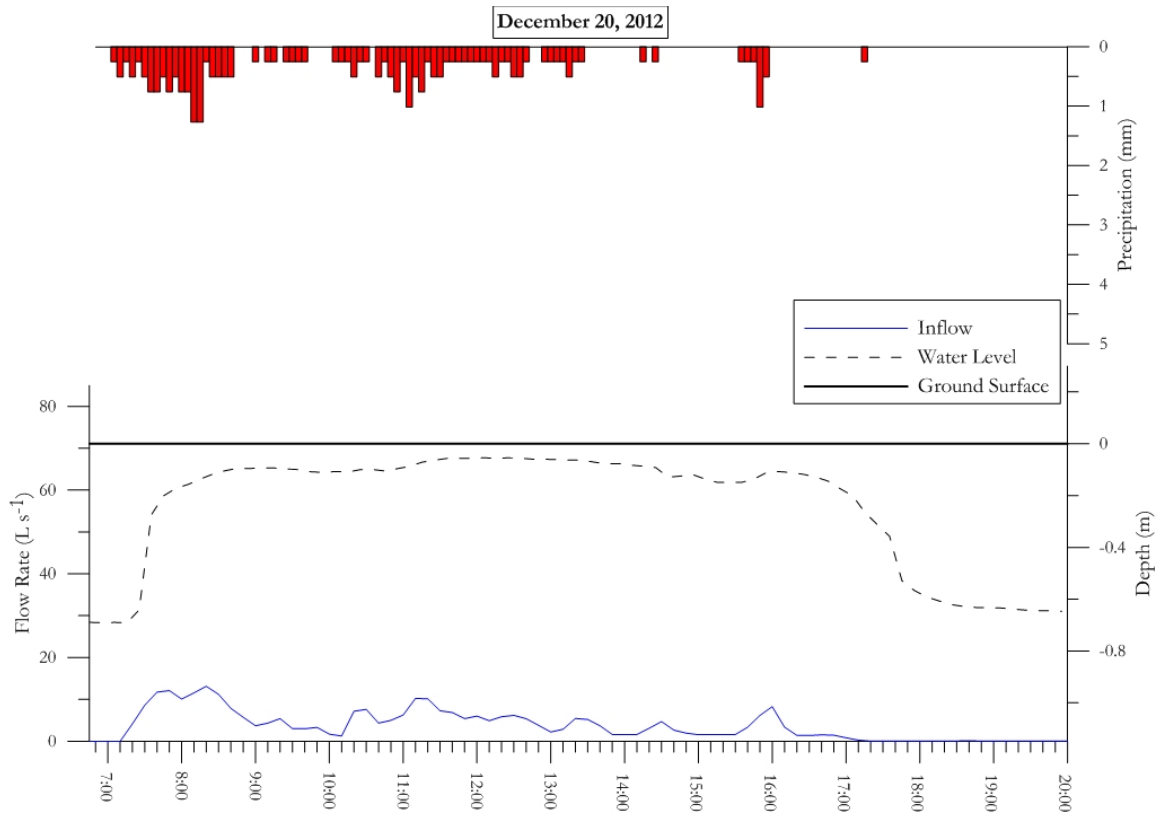


Figure E.34. Event occurring December 25 to December 26, 2012.

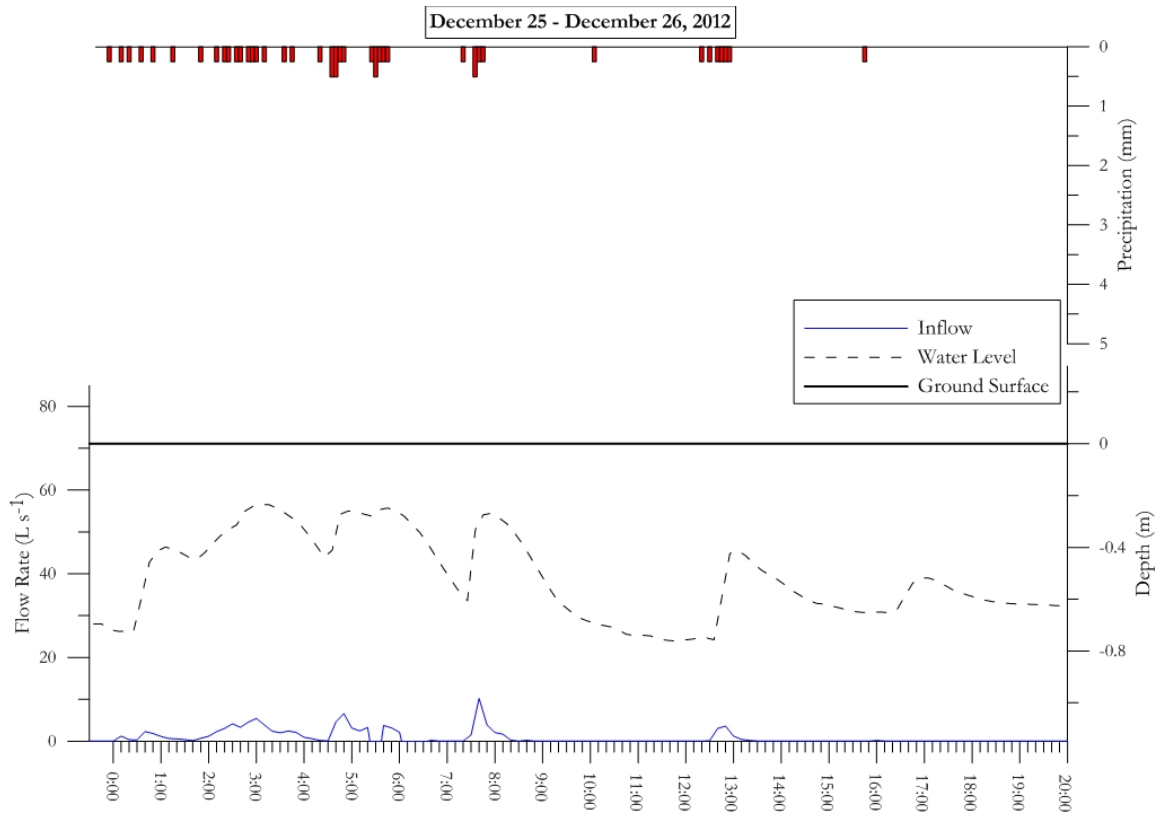


Figure E.35. Event occurring December 31, 2012 to January 1, 2013.

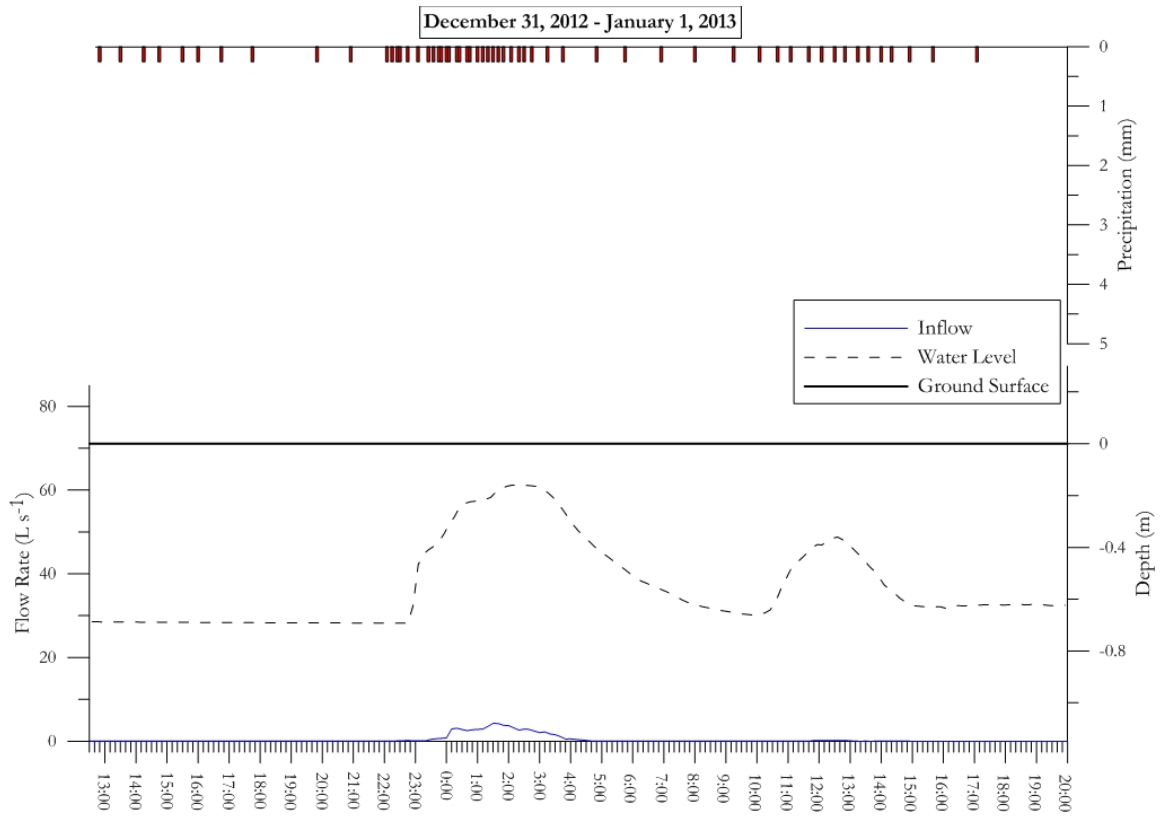


Figure E.36. Event occurring January 11, 2013.

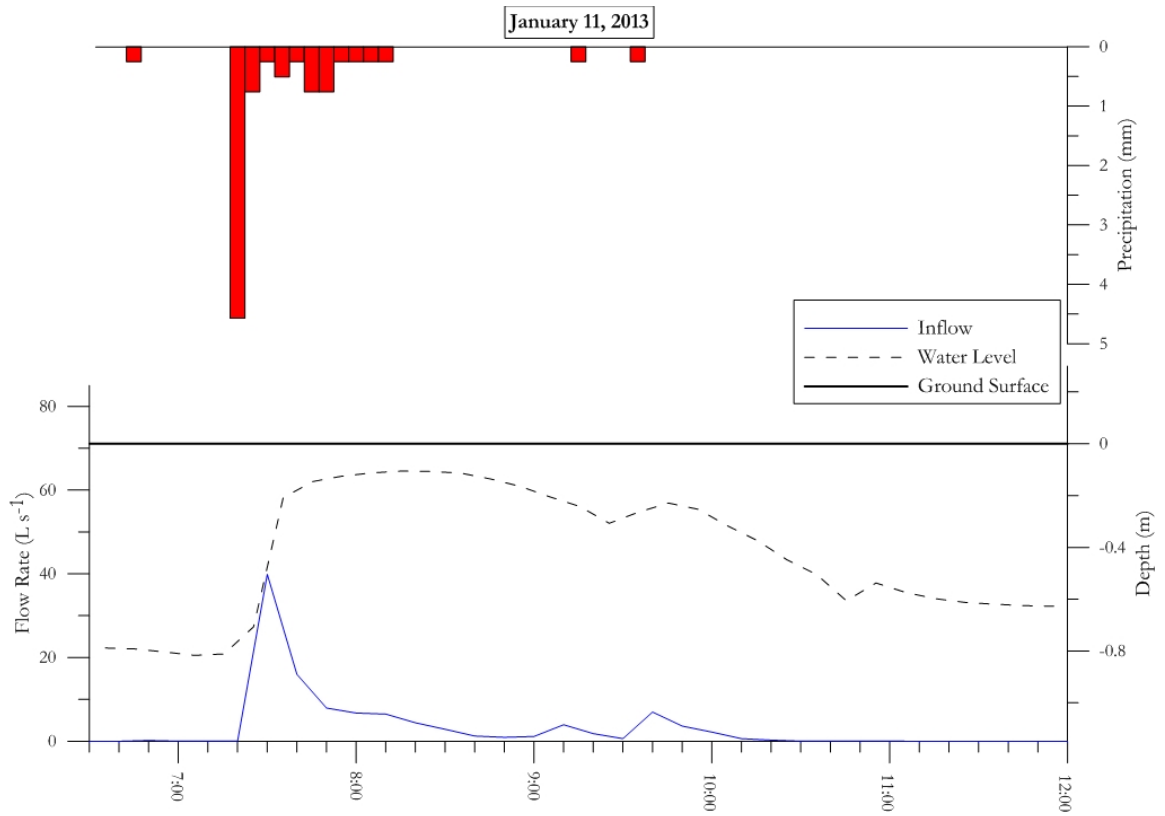


Figure E.37. Event occurring January 13 to January 14, 2013.

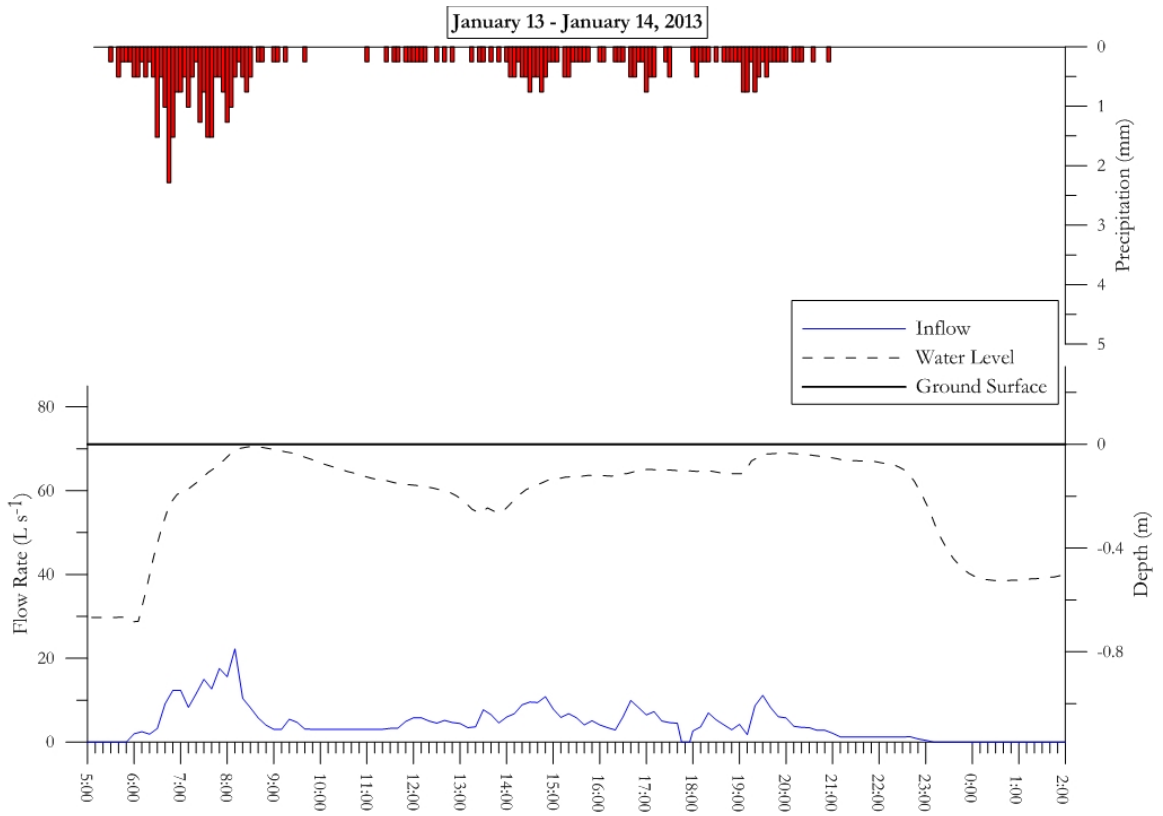


Figure E.38. Event occurring on January 16, 2013.

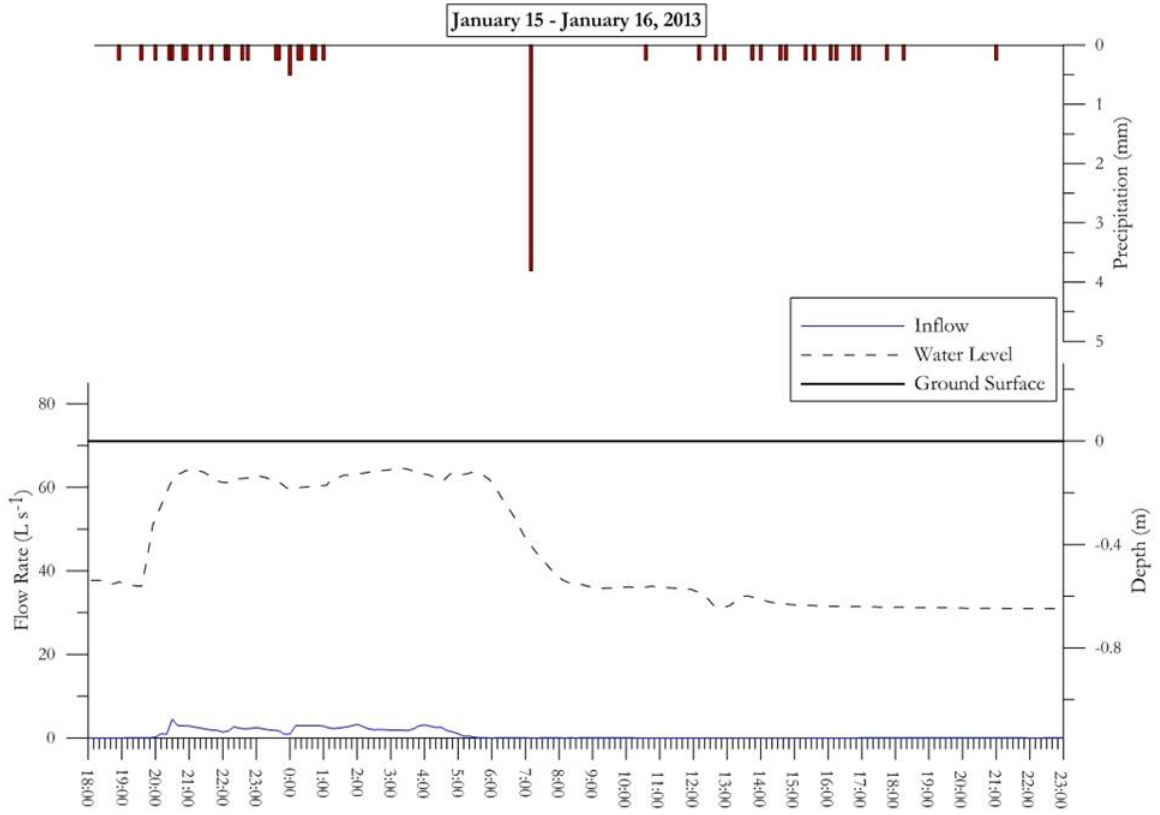


Figure E.39. Event occurring January 28, 2013.

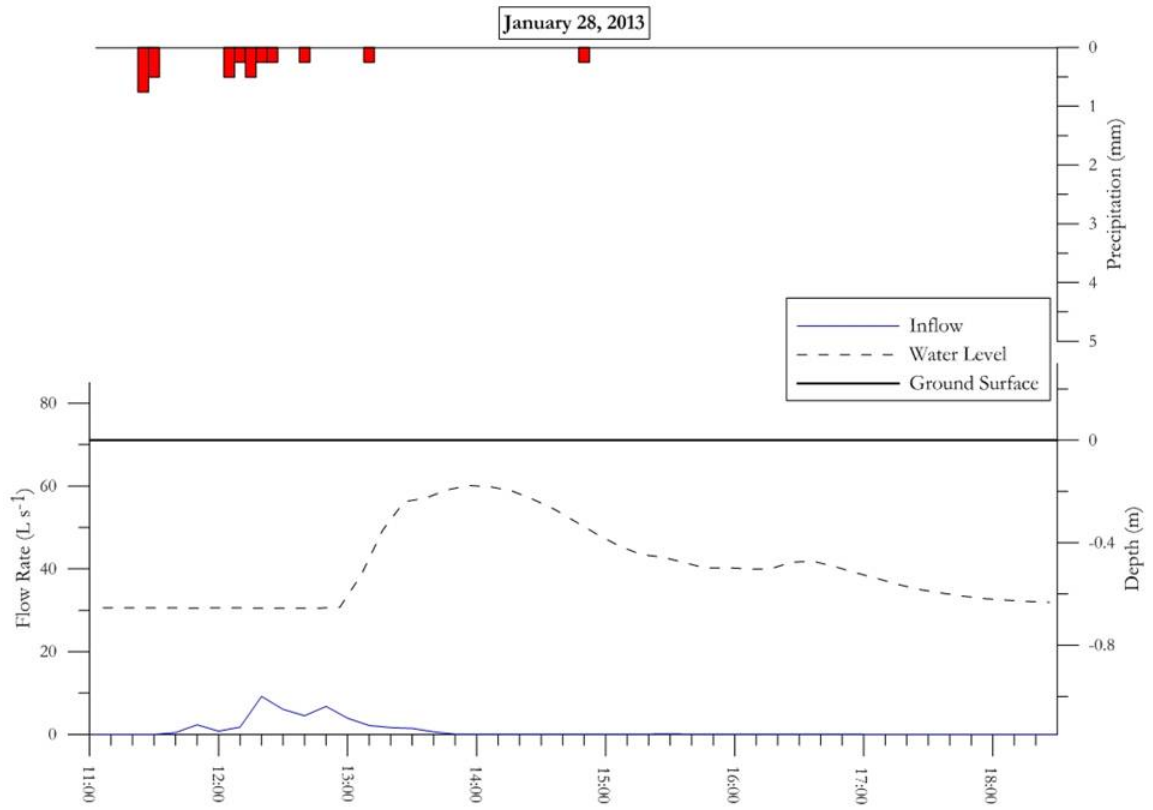




Figure E.40. Event occurring January 30, 2013.

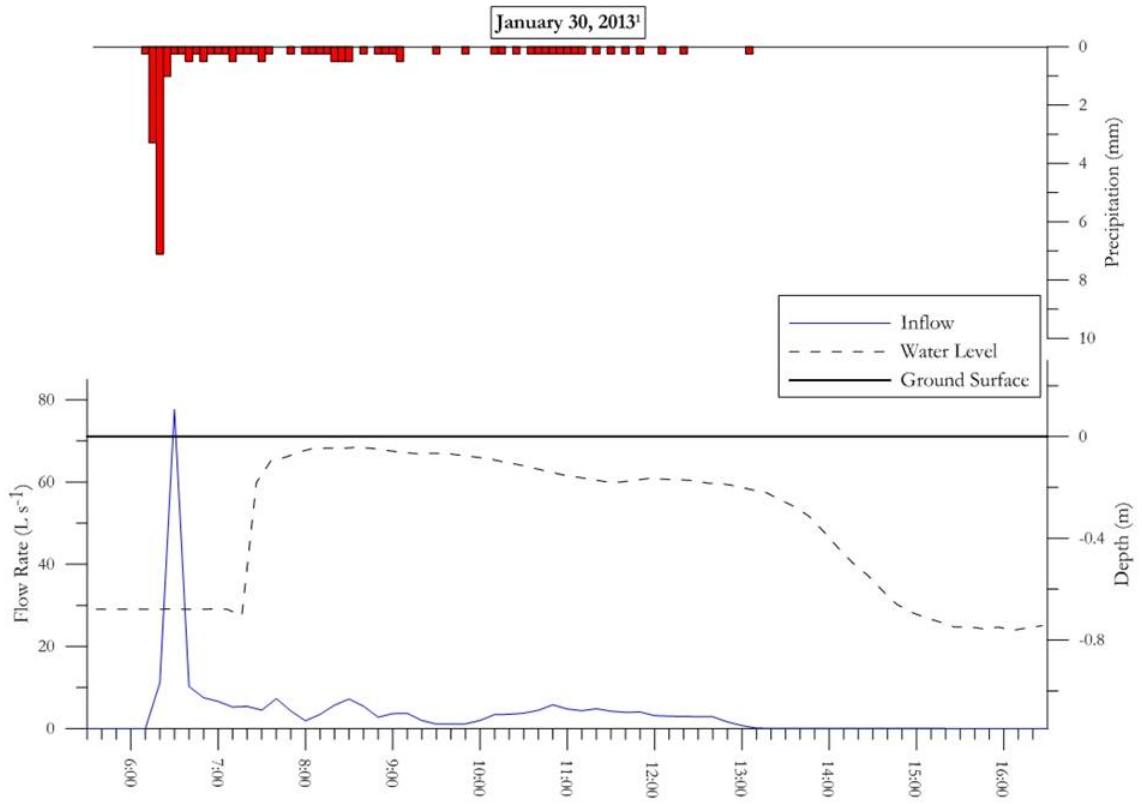


Figure E.41. Event occurring January 30 – January 31, 2013.

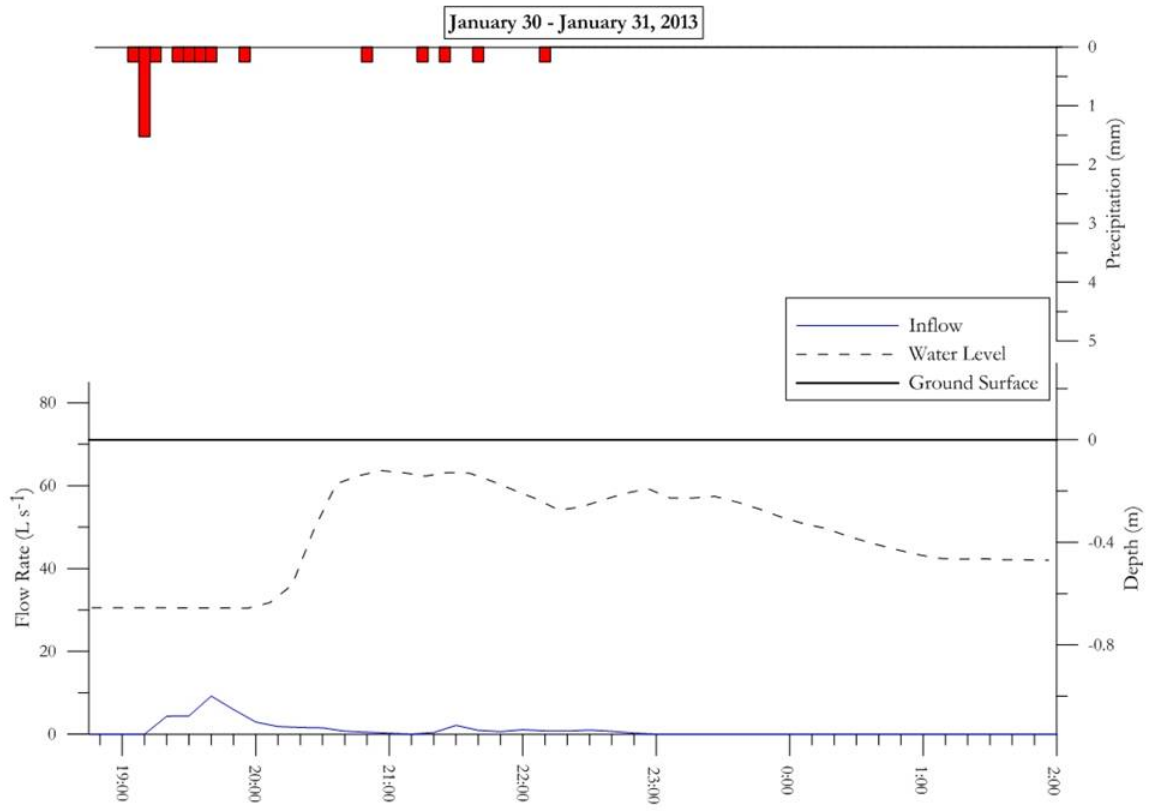


Figure E.42. Event occurring February 21 to February 22, 2013.

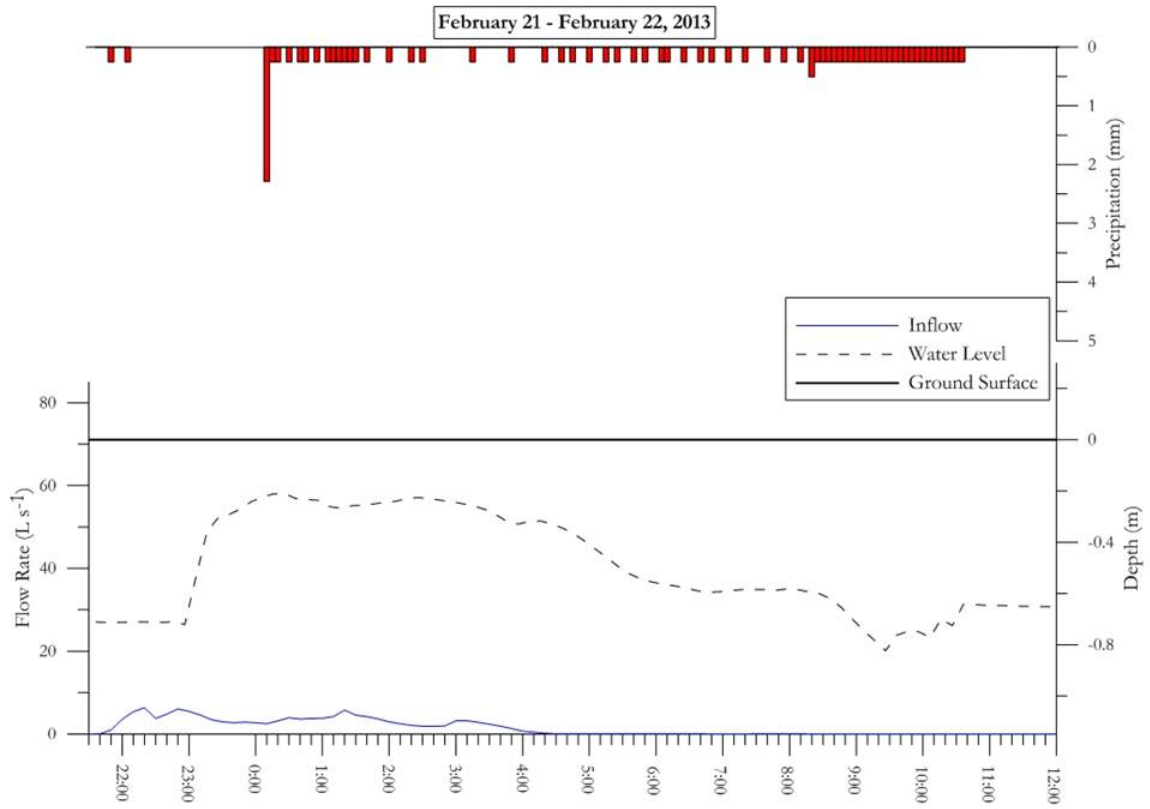


Figure E.43. Event occurring February 26 to February 27, 2013.

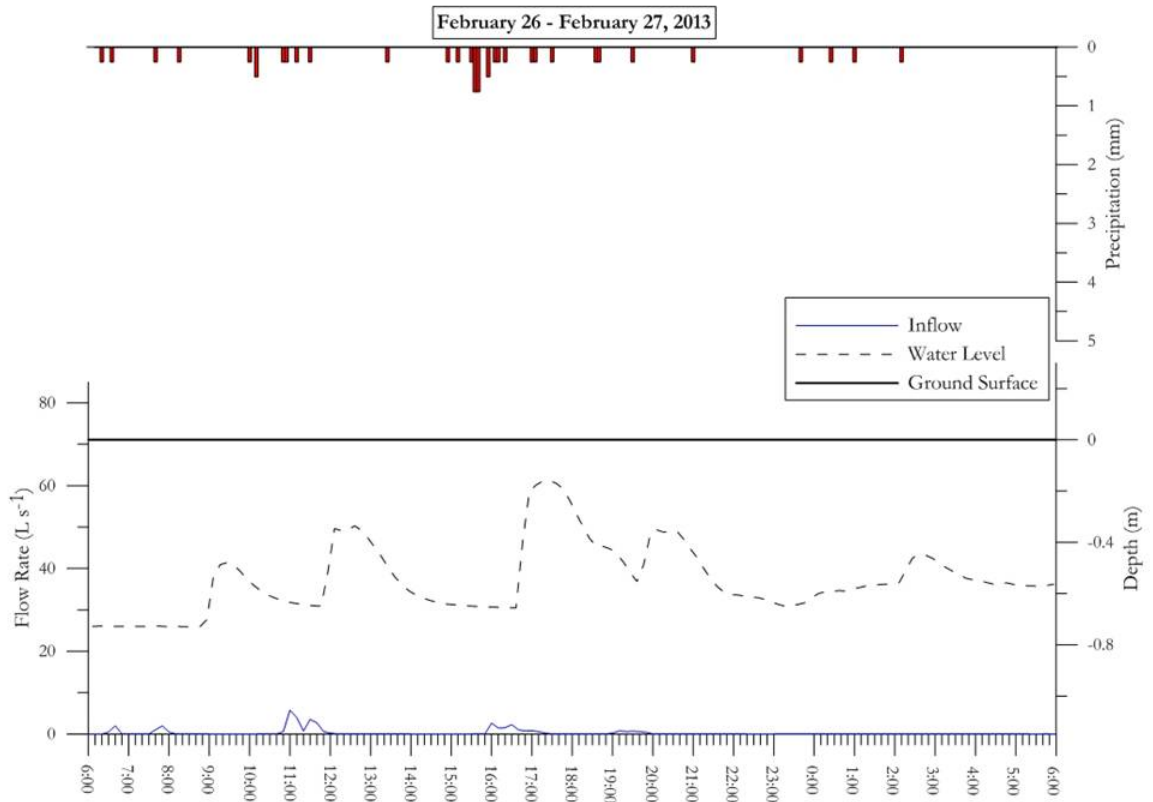


Figure E.44. Event occurring March 5 to March 6, 2013

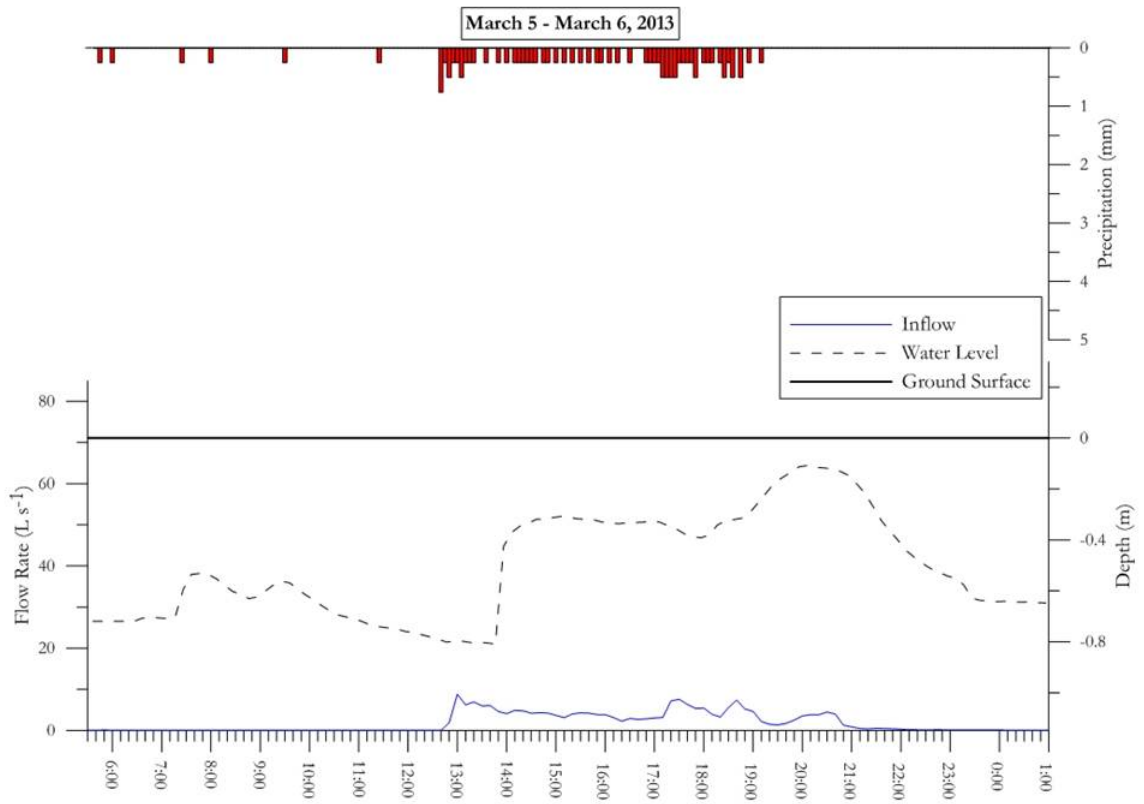


Figure E.45. Event occurring March 11, 2013

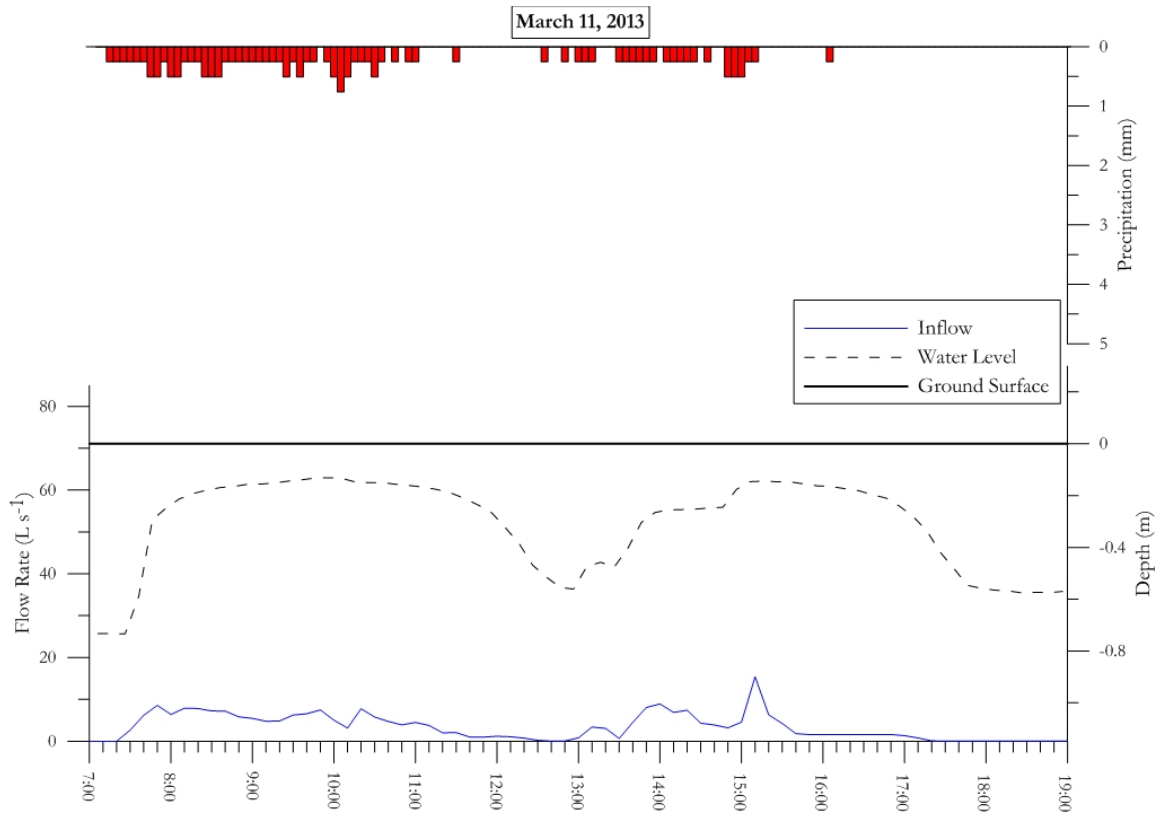


Figure E.46. Event occurring March 17 to March 18, 2013.

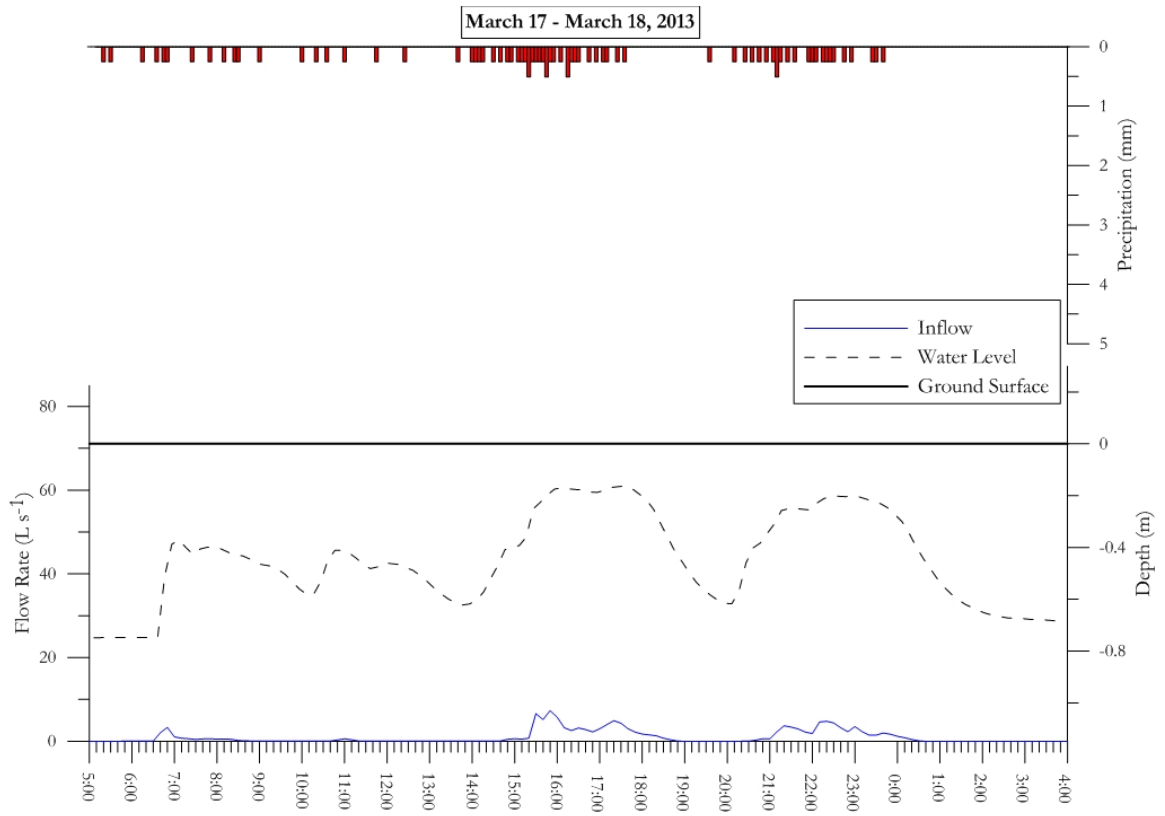


Figure E.47. Event occurring March 18, 2013.

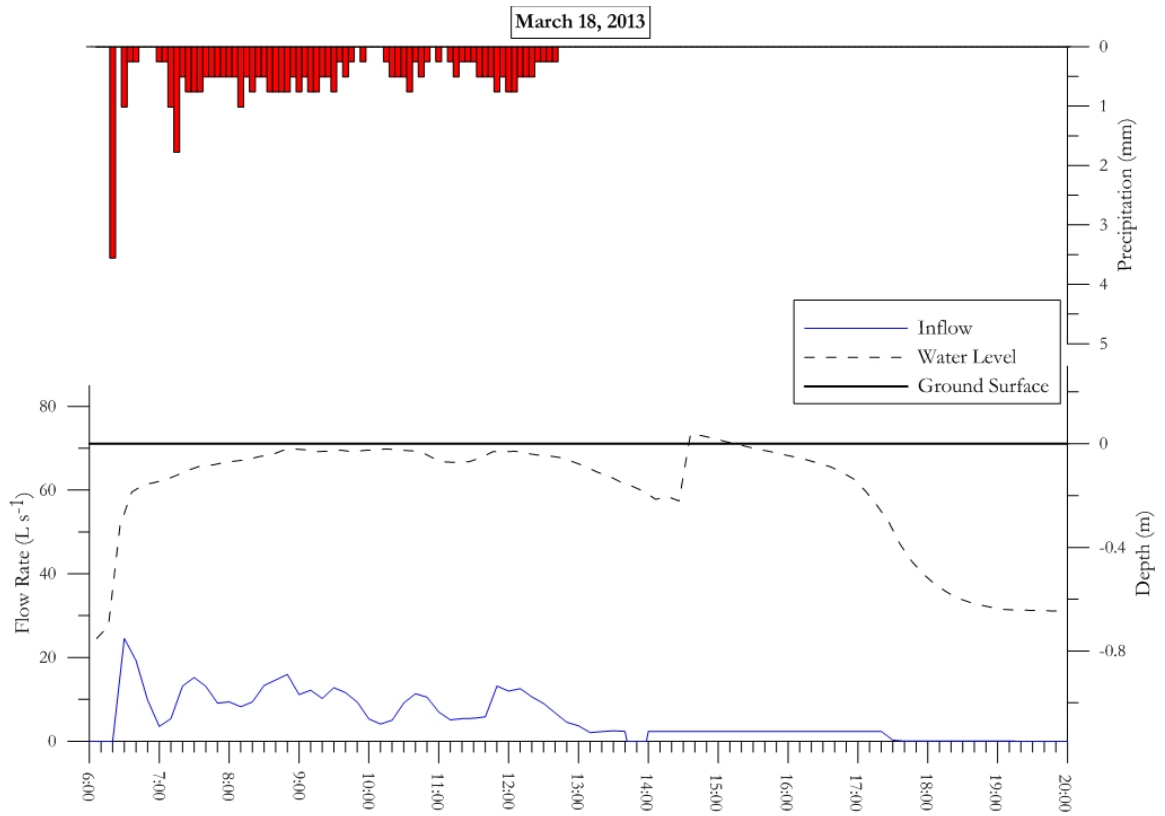




Figure E.48. Event occurring March 24, 2013.

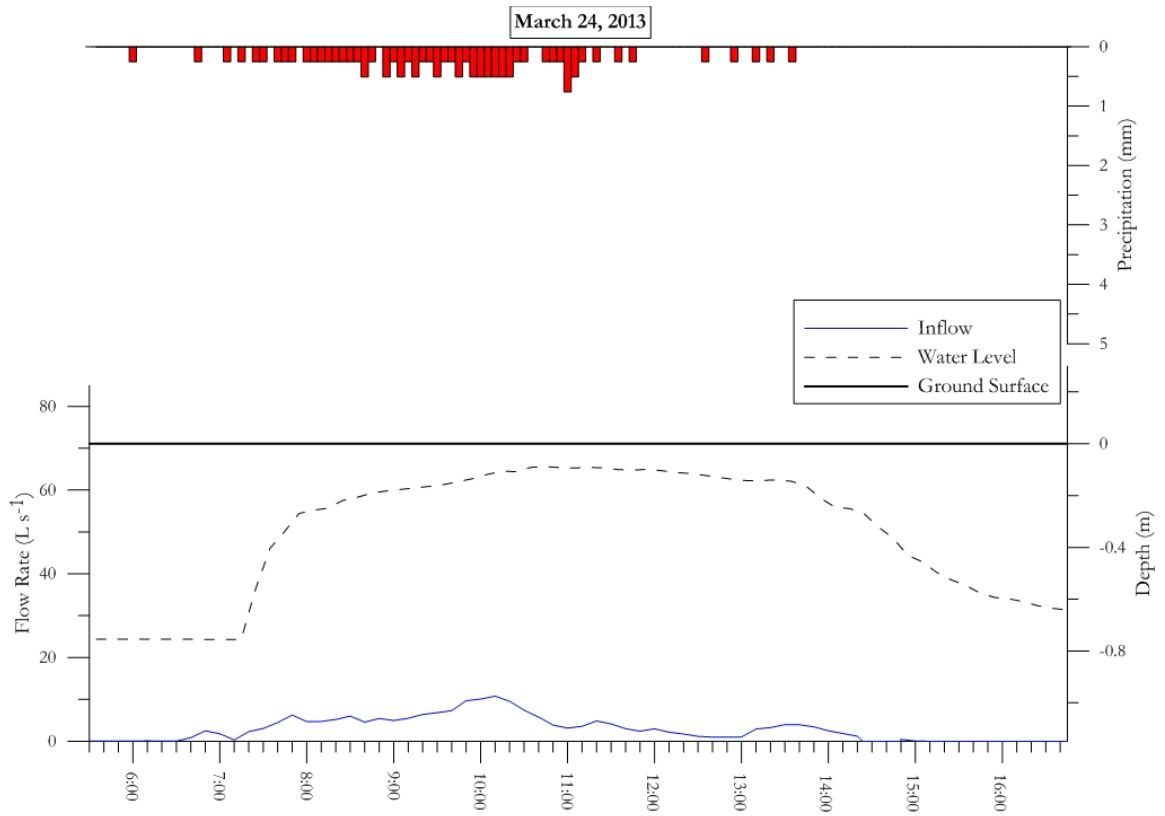


Figure E.49. Event occurring March 24 to March 25, 2013.

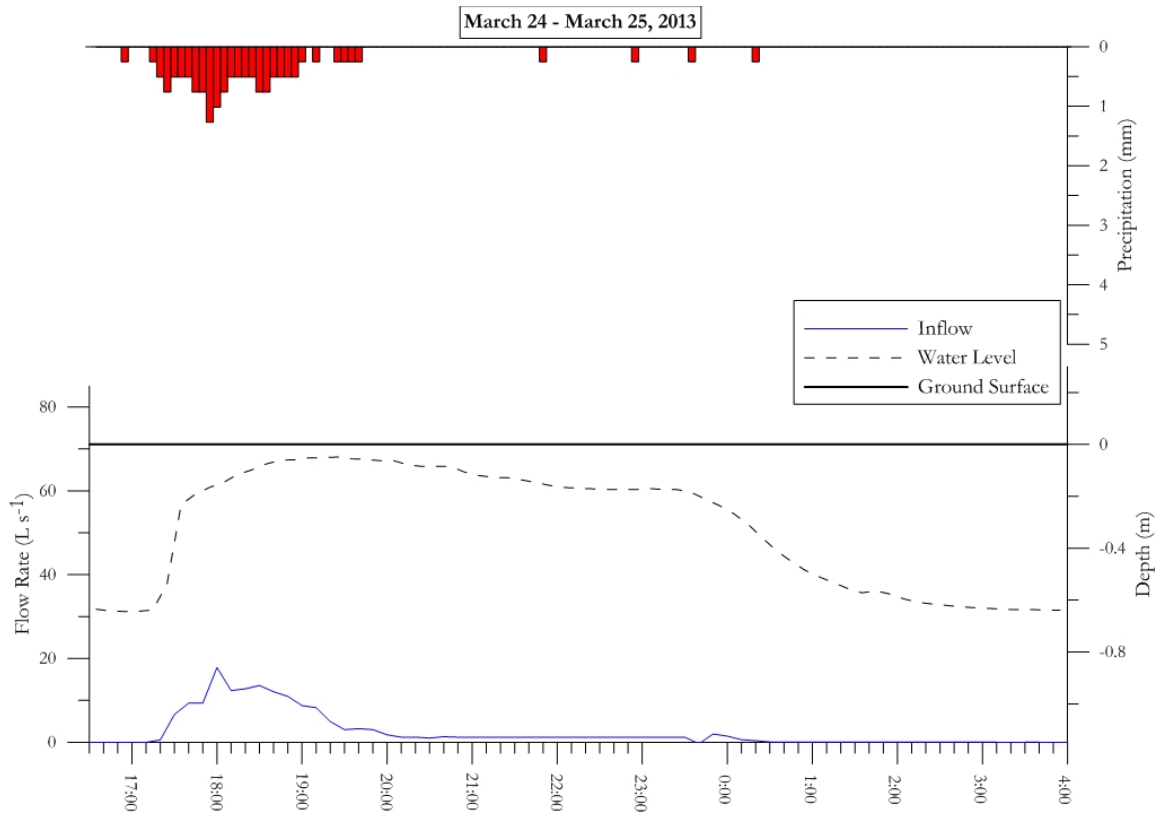


Figure E.50. Event occurring April 11 to April 12, 2013.

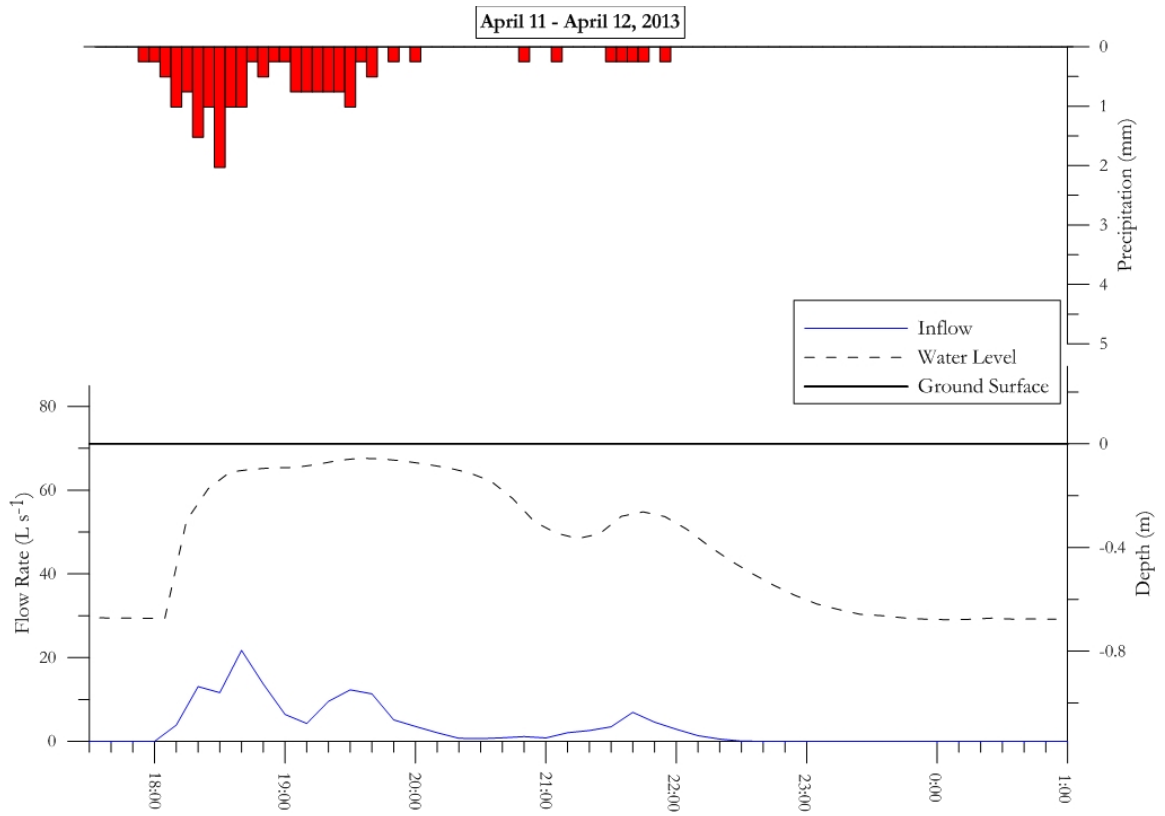


Figure E.51. Event occurring April 12, 2013.

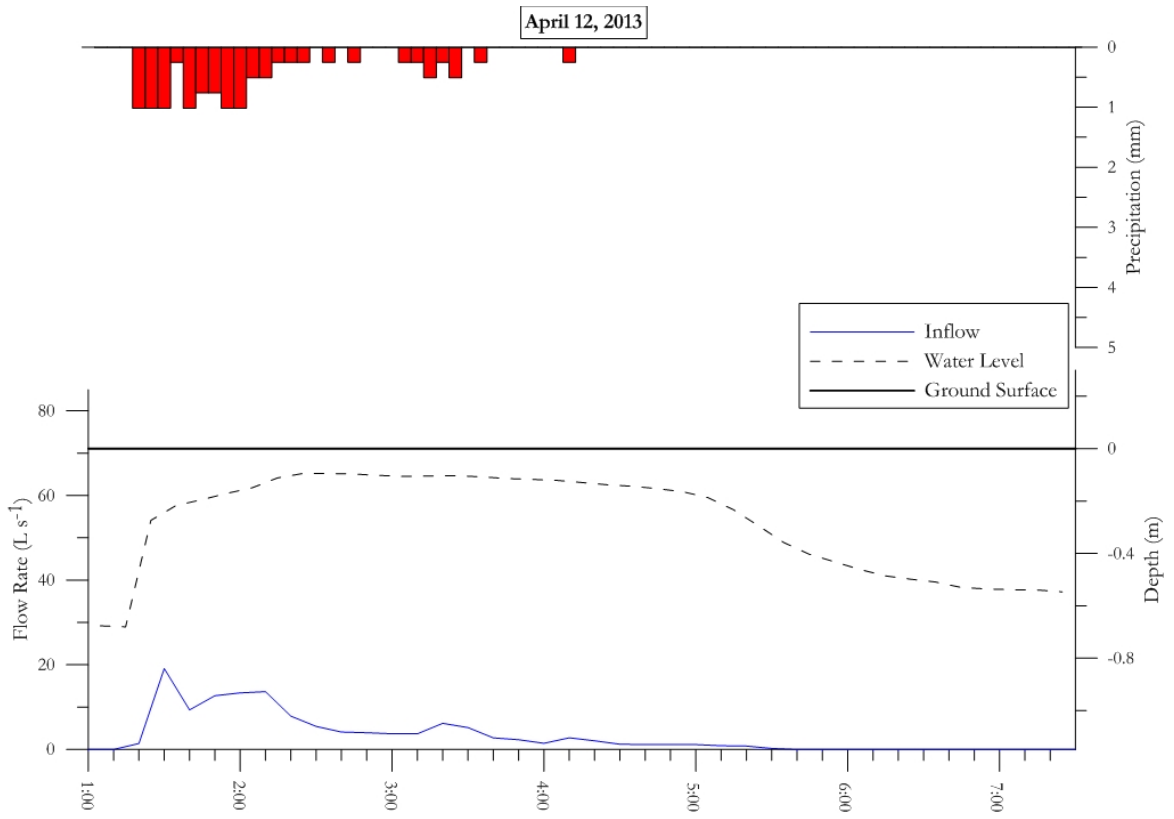


Figure E.52. Event occurring April 17, 2013. \*Note that only flow from the 25 cm pipe was recorded.

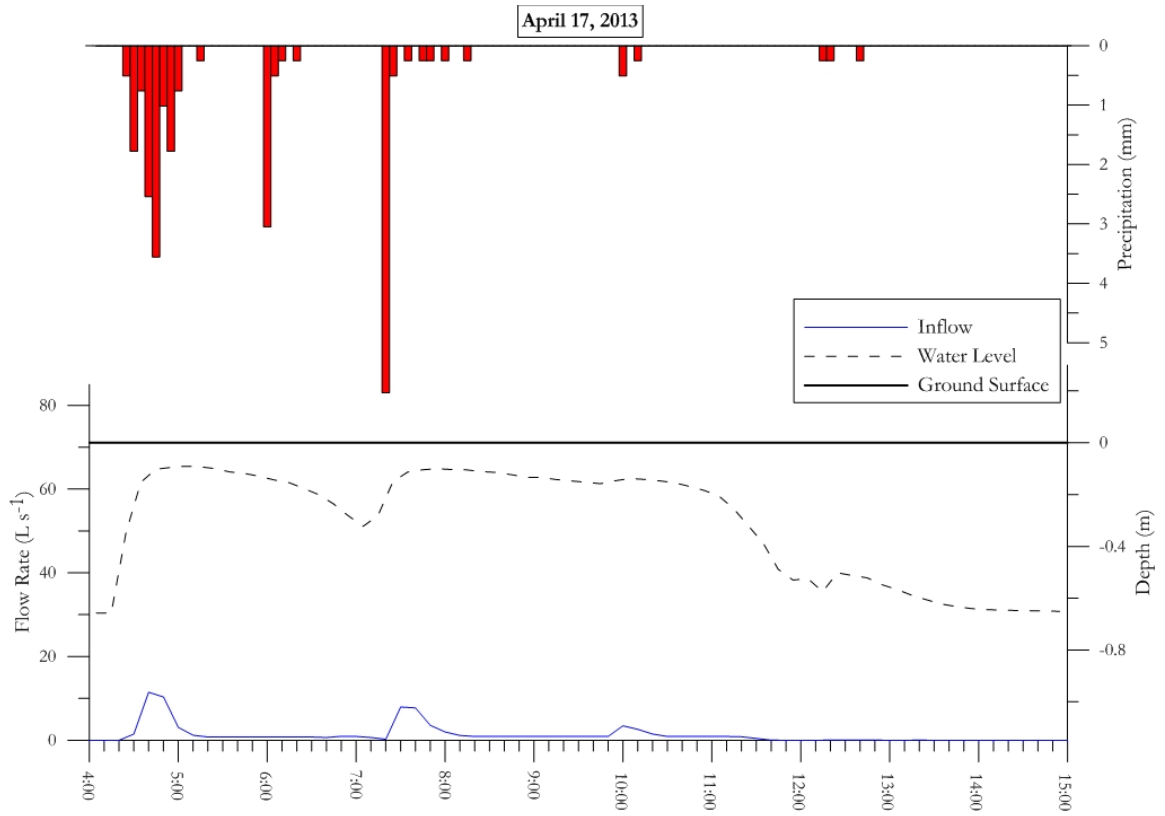


Figure E.53. Event occurring April 19, 2013. \*Note that only flow from the 25 cm pipe was recorded.

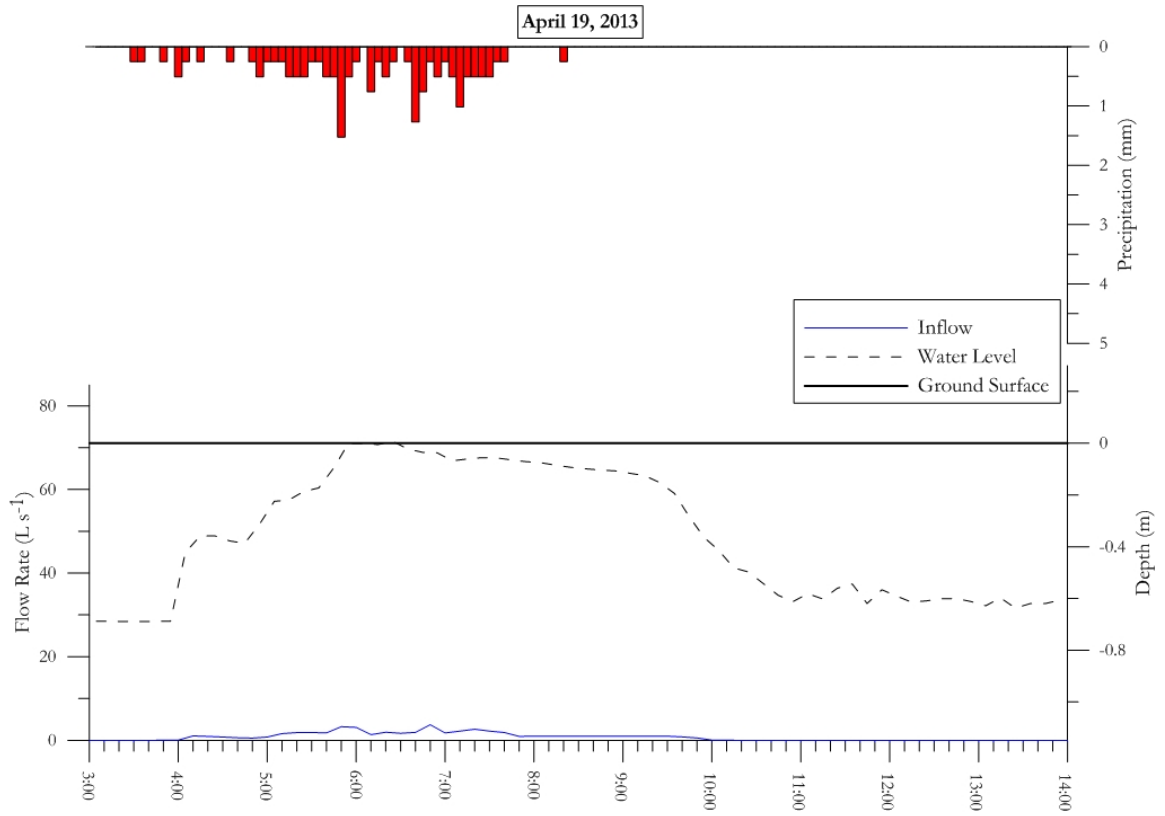


Figure E.54. Event occurring April 24, 2013. \*Note that only flow from the 25 cm pipe was recorded.

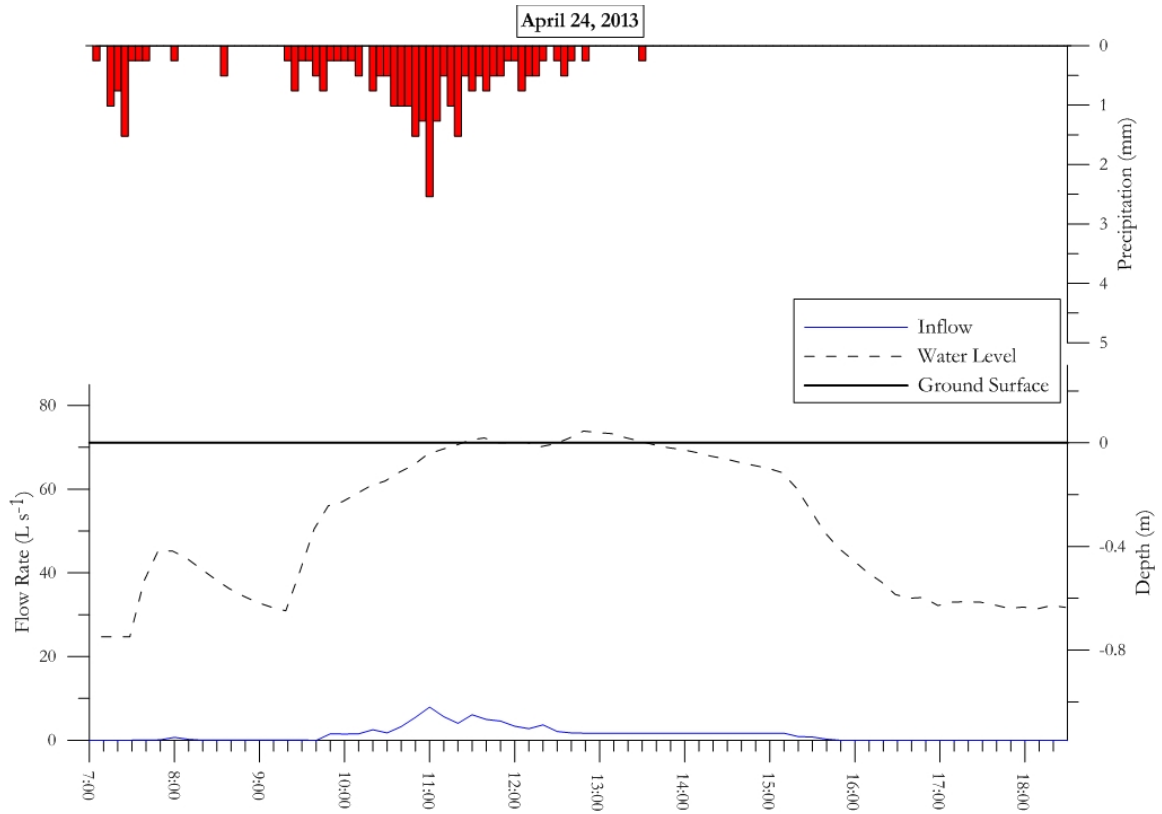


Figure E.55. Event occurring April 27 – April 28, 2013. \*Note that only flow from the 25 cm pipe was recorded.

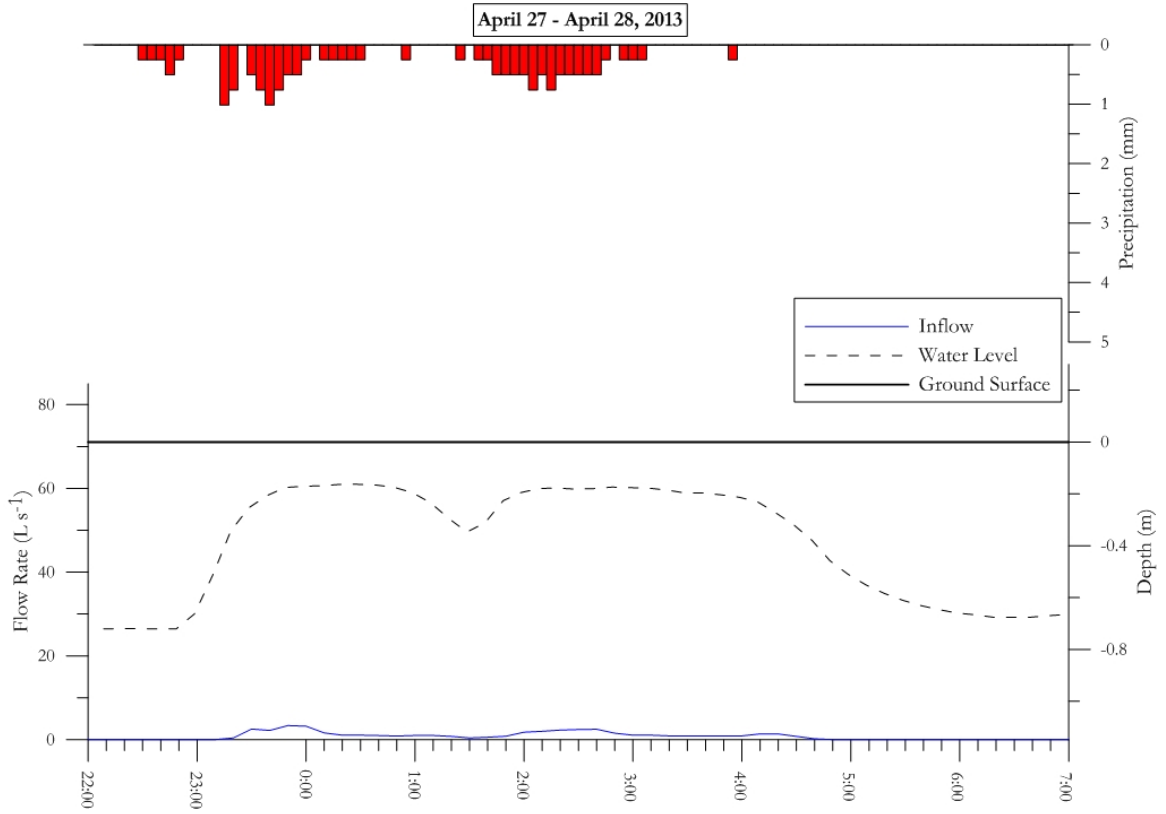
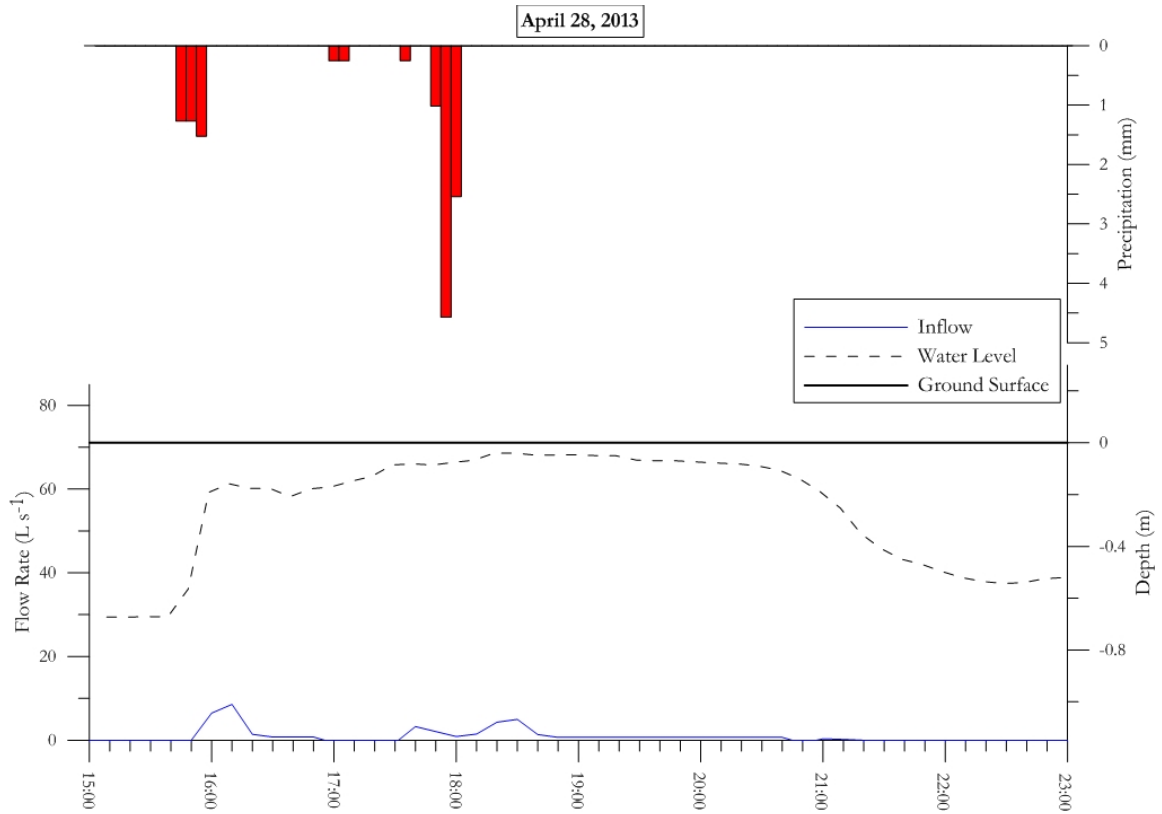




Figure E.56. Event occurring April 28, 2013. \*Note that only flow from the 25 cm pipe was recorded.



## APPENDIX F: INFLOW CHARACTERISTICS

Table F.1. Inflow characteristics for all storm events.

Event Date	Rainfall Depth (mm)	25 cm Inflow Volume (L)	30 cm Inflow Volume (L)	Total Inflow Volume (L)	Peak Flow Rate (L s <sup>-1</sup> )
May 5, 2012	14	28,682	62,327	91,009	13
May 14, 2012	5	6,185	17,077	23,262	5
May 31, 2012	21	26,813	49,430	76,243	31
June 11, 2012	17	25,726	72,505	98,231	18
June 17, 2012	9	12,387	31,656	44,043	51
June 17, 2012	13	16,754	38,912	55,666	54
July 1, 2012	6	357	9,533	9,889	5
July 14, 2012	14	18,657	32,149	50,806	--
July 18, 2012	18	28,699	57,170 <sup>1</sup>	85,870	--
July 19, 2012	17	22,039	46,109 <sup>1</sup>	68,148	--
July 26, 2012	15	22,973	47,661 <sup>1</sup>	70,634	--
July 27, 2012	21	37,688	72,097 <sup>1</sup>	109,785	--
August 3, 2012	15	19,762	42,328 <sup>1</sup>	62,090	--
September 2,	38	61,375	155,307	216,682	26
September 5,	13	9,787	27,884	37,671	39
September 8,	20	22,090	62,293	84,382	17
September 25,	25	35,853	76,634	112,487	35
September 26,	12	9,516	25,539	35,054	32
September 28,	21	43,584	75,801	119,386	6
October 1, 2012	8	8,819	18,929	27,748	17
October 14, 2012	6	3,432	13,033	16,465	7
October 26, 2012	6	10,178	19,507	29,685	1
November 3,	4	5,709	10,977	16,686	34

Table F.1 cont'd.

Event Date	Rainfall Depth	25 cm Inflow	30 cm Inflow	Total Inflow	Peak Flow Rate
November 12, 2012	29	78,452	120,269	198,721	6
November 26, 2012	8	13,594	14,834	28,428	14
December 7, 2012	16	21,019	45,946	66,965	11
December 8, 2012	8	12,659	32,115	44,774	21
December 9, 2012	9	27,221	30,925	58,147	23
December 9, 2012	40	59,557	113,184	172,741	10
December 17, 2012	8	7,154	25,607	32,761	8
December 17, 2012	15	34,681	64,179	98,859	36
December 20, 2012	32	66,099	111,790	177,889	10
December 25, 2012	11	9,702	45,624	55,326	--
December 31, 2012	15	16,244	25,403	41,647	40
January 11, 2013	11	18,623	46,201	64,824	22
January 13, 2013	56	157,363	171,398	328,761	4
January 16, 2013	6	30,840	43,415	74,255	2
January 28, 2013	5	6,389	170	6,559	9
January 30, 2013	27	41,342	106,268	147,610	77
January 30, 2013	4	8,207	16,992	25,199	6
February 21, 2013	11	24,944	52,284	77,229	15
February 26, 2013	9	11,741	13,016	24,757	7
March 5, 2013	18	32,693	75,563	108,256	25
March 11, 2013	24	48,512	98,791	147,304	8
March 17, 2013	19	30,263	48,359	78,622	57
March 18, 2013	46	119,386	147,796	267,182	22
March 24, 2013	19	30,246	81,205	111,451	19
March 24, 2013	18	42,973	58,249	101,221	57

Table F.1 cont'd.

Event Date	Rainfall Depth	25 cm Inflow	30 cm Inflow	Total Inflow	Peak Flow Rate
April 11, 2013	20	26,915	60,390	87,305	22
April 12, 2013	12	28,886	47,544	76,430	19
April 17, 2013	26	49,107	91,059 <sup>1</sup>	140,166	--
April 19, 2013	18	26,083	52,825 <sup>1</sup>	78,908	--
April 24, 2013	31	44,706	83,751 <sup>1</sup>	128,457	--
April 27, 2013	15	13,305	31,606 <sup>1</sup>	44,911	--
April 28, 2013	15	13,492	31,916 <sup>1</sup>	45,408	--

<sup>1</sup> Inflow volume from 30 cm pipe was estimated from relationship between 25 cm and 30 cm inflow volumes. Peak flows were not estimated if data were missing.

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