Winter 2020

Infiltration Analysis of Calibrated Stormwater Models in SWMM

Anna Laura Alegria Silveira
University of New Hampshire, Durham

INFILTRATION ANALYSIS OF CALIBRATED STORMWATER MODELS IN SWMM

BY

ANNA LAURA ALEGRIA SILVEIRA

B.S. Pontifical Catholic University of Rio de Janeiro, 2017

THESIS

Submitted to the University of New Hampshire

in Partial Fulfillment of

the Requirements for the Degree of

Master of Science

in

Civil Engineering

December 2020

This thesis has been examined and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering by:

Thesis Director, Dr. Thomas P. Ballestero

Associate Professor

Civil and Environmental Engineering

University of New Hampshire

Dr. James Houle

Director

UNH Stormwater Center

University of New Hampshire

Dr. James P. Malley, Jr.

Professor

Civil and Environmental Engineering

University of New Hampshire

On December 1st, 2020

Approval signatures are on file with the University of New Hampshire Graduate School.

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ABSTRACT

INFILTRATION ANALYSIS OF CALIBRATED STORMWATER MODELS IN SWMM

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This research project analyzes the hydrology of two Green Stormwater Infrastructure (GSI) systems located at the University of New Hampshire (UNH) campus in Durham, NH, and compares field data to modeling results of a calibrated Storm Water Management Model (SWMM) model of each system. The studied systems were a Philadelphia Tree Trench and an Infiltration Trench, located in Parking Lot A and Parking Lot E, respectively. The Stormwater Center at UNH monitored the system wells, precipitation, and collected data since system constructions, to analyze the infiltration behavior.

The fundamental reason for this research is that the SWMM model only computes infiltration out the bottom of GSI systems whereas field data indicate that significant additional water infiltrates horizontally out the system walls. The objective of this research is to understand how well the model results match the observed system performance. The methods used in this evaluation were the visual comparison of observed water volume versus model water volume; the Mean Square Error (RMSE), and the Nash-Sutcliffe equation (NSE).

The model was originally planned to be calibrated by changing only infiltration parameters in the system, according to the Green-Ampt method of infiltration. A sensitivity analysis showed

that the hydraulic conductivity was the most relevant parameter in the seepage loss calculation in SWMM. However, changing model infiltration area to include sidewalls in both systems significantly improved the results. This was found to be necessary due to SWMM not considering horizontal infiltration for the seepage loss calculations.

The hydraulic conductivity values of the calibrated model were below the expected values for the soil types present in the field, even with the correction of the infiltration area. This calibration concluded that SWMM predicts infiltration rates 33% of the rates expected for the soil types on average, but very similar infiltration rates when compared to the ones measured on the field for these systems. SWMM predicted modeled infiltrated volumes 14% of observed volumes when using storage units to model infiltration systems. Final NSE and RMSE values were improved in the calibration, but not as expected for goodness-of-fit.

Two methods were tested in the attempt to obtain modeled infiltrated volumes matching the ones observed in the field. The first one was to model the system as a LID control option. It was concluded to be ineffective when modeling the systems in this study, as this method underpredicted infiltrated volumes for some storms events (around 59%), and overpredicted for others (around 149%). This may be due to the proportion of runoff volume entering the system in the model not matching the one observed in reality when using LID control options to model infiltration systems.

The last method was to calibrate the model with the addition of a fictitious underdrain to help improve infiltration in the systems. This was concluded to be the best option, as the modeled infiltrated volumes matched almost 100% the ones observed for both systems. This method presented a significant improvement in final NSE and RMSE values when compared to the original calibration process. The water flowing through the fictitious underdrain would simulate the water

flowing through the sidewalls of the system in reality. Therefore, the modeled volume of water flowing out of the system through the fictitious underdrain would simulate the observed infiltrated volume of water flowing through the sidewalls of the system in reality. However, this is not a feasible method to implement, as it is not practical to estimate the diameter of the fictitious underdrain during the design phase of new systems.

The conclusion of this study is that the calibration was only possible due to the availability of observed data. When comparing modeled results to observed data, it was noticed that it is important to consider parameters other than infiltration rate when modeling GSI systems in SWMM. This means that SWMM models of GSI systems are incapable of adequately representing lateral infiltration, when considering only the available infiltration parameters in SWMM.

CHAPTER 1: INTRODUCTION

1.1 Literature Review on Stormwater

When rainfall intensity exceeds the soils infiltration capacity, water runs off. Water may first pond on the soil surface then runoff. Runoff occurs when the excess ponded water on the soil surface flows as guided by topography. This flow may be derived from various sources such as stormwater or snow/ice meltwater. The runoff flows generally perpendicular to topographic contours and the area of land that drains all water to a common outlet is termed a watershed. The watershed may also be referred to as a drainage basin or a subcatchment, as described by the Storm Water Management Model (SWMM).

More impervious land surfaces are found in urbanized watersheds, for example roofs, sidewalks, roads, driveways, alleys, porches and parking lots. The increase of watershed imperviousness profoundly increases surface runoff, potentially resulting in erosion and flooding unless addressed. Associated with the flow are the pollutants found in urban runoff, for example suspended or dissolved solids, which then contaminate receiving waters such as streams, ponds, wetlands, lakes, estuaries and oceans. These pollutants may adversely affect aquatic ecosystems and creates health problems to users/consumers of theses receiving waters.

Stormwater management is necessary in order to: move runoff away from infrastructure, protect infrastructure, reduce runoff, minimize erosion, and improve water quality. In urban areas, runoff usually runs into sewers and drains, and then to receiving waters. This sometimes causes "downstream flooding, stream bank erosion, increased turbidity from erosion, habitat destruction,

combined storm and sanitary sewer system overflows, infrastructure damage, contaminated streams, rivers and coastal water" (US EPA, 2019).

Historically, conventional stormwater management systems were employed to collect and redirect the runoff to a sewer system or to a pond or swale, in order to control peak flows. Conventional infrastructure typically did not address water quality other than possibly sedimentation of large sediment.

Recently, Low Impact Development (LID), that includes Green Stormwater Infrastructure (GSI), was developed to better manage runoff in order to reduce runoff peaks, reduce runoff volume, reduce stormwater pollutants and, in many GSI systems, provide ecological niches in urban settings. "This type of infrastructure combines pipes and hard surfaces with softer assets like vegetation and green space" (WEF, 2015).

GSI refers to practices that use natural processes of infiltration to protect water quality and support natural habitat while it maximizes the time water spends in storage. It considers stormwater as a resource rather than a waste. Implementing GSI is a cost-effective approach to manage the impacts of stormwater runoff while creating healthier urban environments. "At the city or county scale, green infrastructure is a patchwork of natural areas that provides habitat, flood protection, cleaner air, and cleaner water. At the neighborhood or site scale, stormwater management systems that mimic nature soak up and store water" (US EPA, 2019).

Some common GSI practices include: bioretention systems, permeable pavements, green roofs, rain gardens, tree filters, infiltration systems, and subsurface gravel wetlands. Each practice has its own characteristics, having different assets (or components) needing to perform at an acceptable level of service.

Design of GSI systems vary with state regulations. For the State of New Hampshire, the following design elements are needed: Water Quality Volume (WQV), Water Quality Flow (WQF), Groundwater Recharge Volume (GRV), Effective Impervious Cover (EIC), Undisturbed Cover (UDC), Channel Protection (CP) and Peak Control (NHDES, 2008). The equations and variables for each parameter are presented on Table 1.

For this study, the first system to be analyzed is a Philadelphia Tree Trench. This system incorporates a Tree Box Filter that collects runoff from the parking lot and then drains the excess flow to a stone-filled trench. The trench is unlined and allows infiltration out of the bottom and sidewalls. A perforated pipe at the bottom of the trench collects excess water and delivers it to the storm sewer.
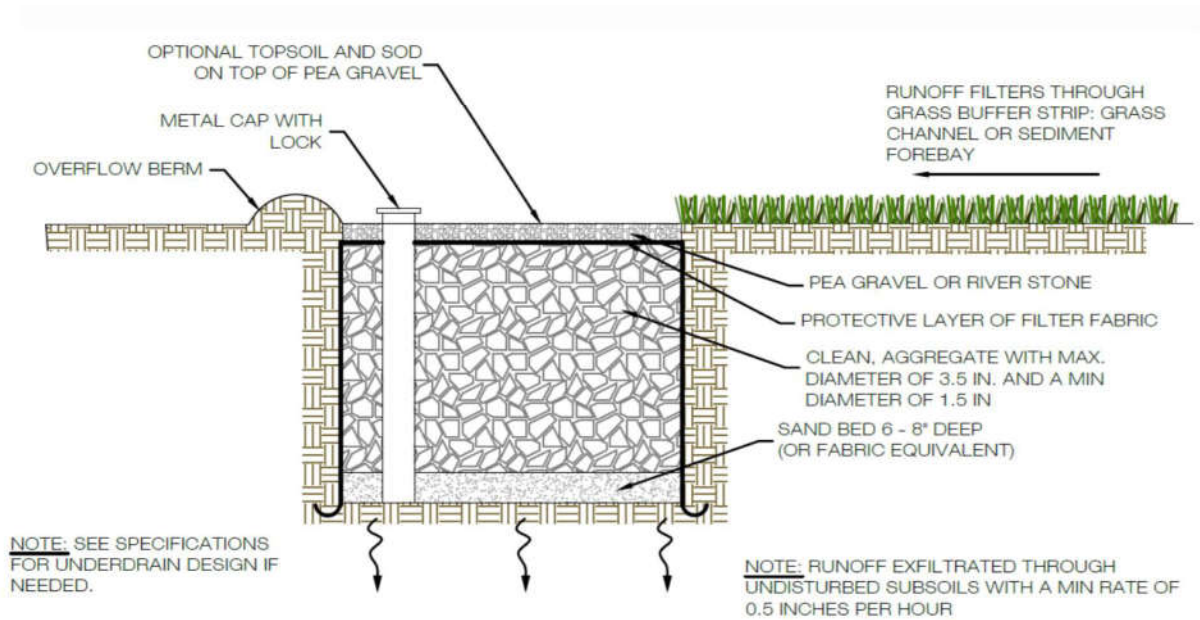
Figure 1 – Example of an infiltration trench system (Virginia DCR, 2011)

The second studied system is an Infiltration Trench. Figure 1 shows an example of an infiltration trench system. Infiltration trenches "are structures designed to temporarily store runoff,

allowing all or a portion of the water to infiltrate into the ground" (NHDES, 2008). They are designed to retain and infiltrate the Water Quality Volume and drain between storm events.

Design Criteria	Description

Water Quality Volume (WQV)	WQV = (P)(Rv)(A)
	    P = 1" of rainfall
	    Rv = unitless runoff coefficient = Rv = 0.05 + 0.9(I)
	    I = percent impervious cover draining to the structure converted to decimal form
	    A = total site area draining to the structure

Water Quality Flow (WQF)	WQF = (q_u)(WQV)
	    WQV = water quality volume calculated in accordance with Design Criteria above
	    q_u = unit peak discharge from TR-55 exhibits 4-II and 4-III
	
	Variables needed for exhibits 4-II and 4-III:
	    Ia = the initial abstraction = 0.2S
	    S = potential maximum retention in inches = (1000/CN) – 10
	    CN = water quality depth curve number
	        = 1000/ (10+5P+10Q–10[Q^2 + 1.25(Q)(P)]$^{0.5}$)
	    P = 1" of rainfall
	    Q = the water quality depth in inches = WQV/A
	    A = total area draining to the design structure

Groundwater Recharge Volume (GRV)	GRV = $(A_i)(R_d)$
	    A_i = the total area of effective impervious surfaces that will exist on the site after development
	    R_d = the groundwater recharge depth based on the USDA/NRCS hydrologic soil group, as follows:
	
	Hydrologic Group Rd (inches)
	A 0.40
	B 0.25
	C 0.10
	D 0.00

EIC & UDC	%EIC = area of effective impervious cover/total drainage area within a project area X 100 %UDC = area of undisturbed cover/total drainage area within a project area X 100
Channel Protection (CP)	If the 2 yr, 24-hr post-development storm volume *does not increase* due to development then: control the 2-year, 24-hour post-development peak flow rate to the 2-yr, 24-hr pre-development level. If the 2yr, 24-hr post development storm volume *does increase* due to development then: Control the 2-yr, 24-hr post-development peak flow rate to ½ of the 2-year, 24-hr pre-development level or to the 1-yr, 24-hr pre-development level.
Peak Control	Post-development peak discharge rates can not exceed pre-development peak discharge rates for the 10 & 50-yr, 24-hr storm events.

¹ Appendix A provides rainfall data for New Hampshire, for use with these design criteria.

Table 1 - Summary of Design Criteria for Stormwater management systems (NHDES, 2008)

Treatment occurs due to the settlement of solids and pollutants, as well as biogeochemical processes that occur in the system media as well as the soil below. It is important to consider the preservation of infiltration functions in order to maintain the level of service of the system (NHDES, 2008). Design criteria for an infiltration trench in the state of New Hampshire is specified in Table 2.

Design Parameter	Criteria
Pretreatment	Required (see Section 4-4)
BMP Volume	≥ the larger of WQV or GRV, depending on purpose of BMP excluding sediment forebay capacity, if present, and exclude infiltration occurring during the design event
Slope of Basin Floor	0% (flat)
Design Infiltration Rate	See Section 2-4 for a discussion on selecting a design infiltration rate
Drain Time	< 72 hours for complete drainage of the water quality volume
Depth to Bedrock and to Seasonal High Water Table Elevation	≥ 3 feet from bottom of BMP, except: ≥ 4 feet if within groundwater or water supply intake protection area ≥ 1 foot if runoff has been treated prior to entering the BMP
Design Overflow Discharge Capacity (Subsurface Capacity)	10-year, 24 hour storm
Infiltration Media Material (if used for subsurface basin)	Clean, washed, uniform (well-sorted) aggregate Diameter 1.5 to 3 inches Porosity = 40%
Observation Well (subsurface basin)	Well or accessible manhole structure required

Table 2 - Design Criteria for an infiltration basin system (NHDES, 2008)

1.2 Hypothesis and Objectives

The hypothesis of this study is that it is possible to calibrate a model of a stormwater system in the Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) software by changing infiltration parameters in order to match the volume of water generated in

the SWMM model to the volume of water observed in the real systems, based on a certain goodness-of-fit criteria.

For this project, the objectives are to analyze how changing infiltration parameters affect the volume of water in the SWMM modeled systems; to calibrate the SWMM model to match the observed volume of water in the stormwater system; and to identify possible challenges of modeling infiltration in SWMM. This allows an assessment on how well SWMM models infiltration and then how it is possible to modify the modeling to better estimate the infiltration on the systems. This is useful to evaluate how to use SWMM in the design phase, when all that is available are soil characteristics of the site.

1.3 Infiltration Systems

Regardless of having an above surface or a subsurface GSI approach, "infiltration systems have three primary components: storage, treatment and infiltration" (Contech ES, 2019).

Storage can be defined as the system retention volume of runoff. In some subsurface systems, the storage component can be an open vault or be comprised of the porosity in gravel or other porous media. System selection and treatment are designed based on the pollutants found in the site and site constraints.

The infiltration process can be defined as "the flow of water from aboveground into the subsurface environment" (Ferre and Warrick, 2005). However, when it comes to GSI systems, infiltration happens not only vertically (through the bottom of the system) but laterally (out of the system walls). In this case, water flows from the system to the surrounding soil. Studying infiltration phenomena is relevant to several topics such as contaminant transport, ecosystem viability, irrigation, and groundwater recharge. Infiltration ranges in complexity from "steady-

state, saturated flow in a homogeneous, isotropic medium to transient, unsaturated flow through an anisotropic, heterogeneous medium" (Ferre and Warrick, 2005).

Several factors can affect infiltration performance in a system. The most relevant ones are land use, soil type, and level of soil saturation. For example, "sandy soils tend to have higher infiltration rates than finer silt and clay soils […] and saturated soils typically have lower infiltration rates than relatively dry soils" (Virginia DCR, 2010). Water infiltrates at a higher rate at the start of the infiltration process if the soil is dry, and it slows down as the water content in the soil increases. Figure 2 shows an example of how infiltration rates vary overtime for three different soil types.

Infiltration can also be affected by the degree of saturation since the last rainfall event, evapotranspiration, and drainage rates. "Evapotranspiration and drainage between storm events vary with soil type, vegetative cover, position on the landscape, aspect, geology, land use, climate, and time and weather since the last rainfall" (Bonta, 2005).
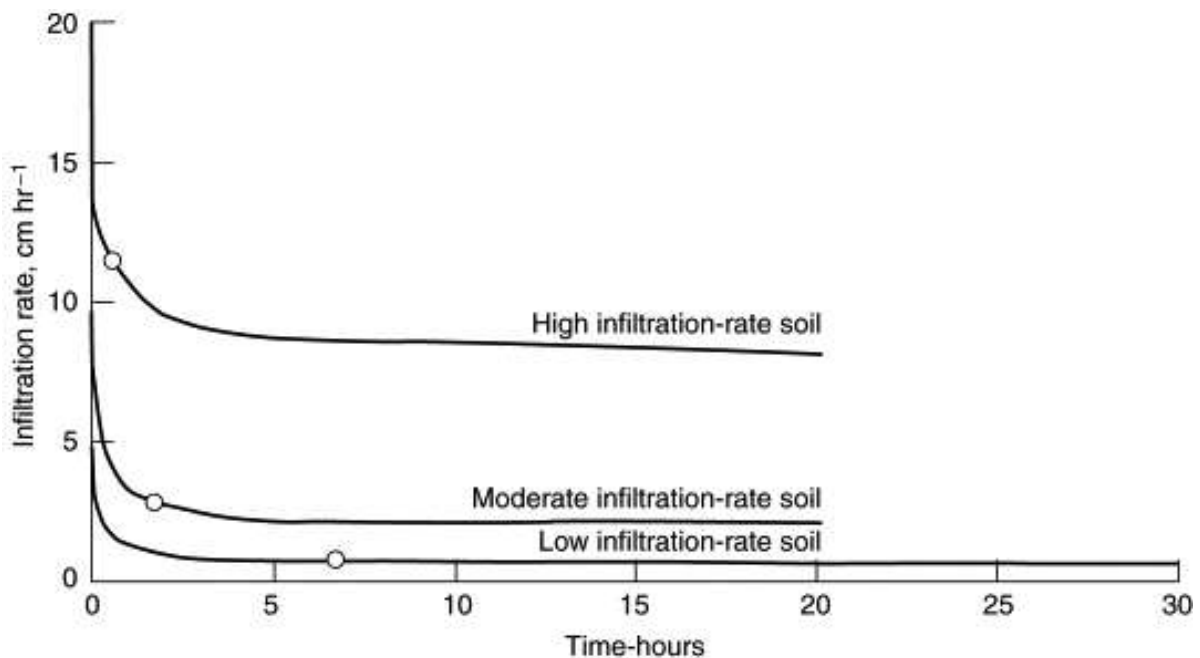
Figure 2 - Measured infiltration rates over time for three different soils (Nimmo, 2009)

When designed appropriately, infiltration systems have high retention capabilities. Soil properties need to be considered during initial site layout in order to select soils with the optimum infiltration rates. Usually, areas with soils belonging to Hydrologic Soil Groups A or B are desired, since they show the highest infiltration capacities (Virginia DCR, 2010).

Both studied systems rely on infiltration to manage stormwater runoff. These practices capture the Water Quality Volume through different medias to treat stormwater runoff and remove pollutants. "Filtration BMPs have shown to be very effective at removing a wide range of pollutants from stormwater runoff, particularly when organic soil filter media have been used" (NHDES, 2008).

Modeling infiltration systems is an important tool used during the design phase of stormwater systems, as they help engineers simulate the site conditions and precipitation patterns. This enables the creation of several different scenarios the system would face and engineers can use them to predict the behavior of infiltration in these systems, for each analyzed storm event.

SWMM models are widely used with this purpose. Usually, infiltration systems can be modeled as "storage units" or as "LID control options". Modeling in SWMM is discussed further in Chapter 4.

CHAPTER 2: SITE AND SYSTEM DESCRIPTION

2.1 Regional Setting

The town of Durham is located in southeastern New Hampshire, in Strafford County. The warm season starts in June and ends in September, while the cold season starts in December and ends in March. The average temperature in the warm season is 72 °F and in the cold season it's 42 °F. "Over the course of the year, the temperature typically varies from 18°F to 81°F" (Weather Spark, 2019). The wetter season starts in March and ends in December, while the drier season starts in December and ends in March. The most common form of precipitation varies throughout the year, being rain in the warm season and snow in the cold season.

Rain falls throughout the year with the most rain centered around late October with an average total accumulation of 4.1 inches. The least rain falls around late January with an average total accumulation of 1.6 inches. The most snowfall is centered around late January with an average total liquid-equivalent accumulation of 1.4 inches. "Durham experiences significant seasonal variation in the perceived humidity" (Weather Spark, 2019).

The University of New Hampshire Weather Station is located on the roof of Morse Hall, in the UNH campus in Durham, NH. "In July 2016 all of the weather sensor hardware was replaced with newer models for the wind speed, direction, temperature and relative humidity, and the photosynthetically active radiation (PAR) device. Only the rain gauge was kept from the original hardware" (UNH Weather, 2016). All precipitation data (inches) used in SWMM modeling was retrieved on the UNH Weather Station website.

When it comes to topography, Durham has only modest variations in elevation and an average elevation above sea level of 65 feet, and this continues within 10 miles of the town. "The area within 2 miles of Durham is covered by trees (79%) and artificial surfaces (18%), within 10 miles by trees (68%) and artificial surfaces (20%), and within 50 miles by trees (51%) and water (33%)" (Weather Spark, 2019).

The two GSI systems under study for this research are both located in Durham, NH on the campus of the University of New Hampshire (UNH). The sites are both parking lots: Lot A and Lot E.

2.2 Lot A characteristics

Commuter parking lot A is located on Gables Way, across the train tracks from the Durham train station. There are six stormwater management systems operating around this asphalt parking lot. These systems were built between 2014 and 2016 as retrofit projects. Figure 3 shows the location of Parking Lot A on the UNH campus. Figure 4 shows the location of the stormwater systems and the drainage area of their respective watersheds. Of the six systems at parking Lot A, the Philadelphia Tree Trench is one of the systems studied in this research. The system was designed according to the Philadelphia Water Department Stormwater Manual specifications for a Philadelphia Tree Trench.

The watershed (Wb) used for the design of the tree trench system has an area of 25,472 square feet or 0.58 acres (UNH Stormwater Center, 2014), a slope of 2.3% (calculated on AutoCAD from the site survey and 1 ft contours) and an overland flow path of 400 ft. The watershed has a percent imperviousness of 95%. Web Soil Survey was used to delineate soils at

this site. The parking lot itself was constructed almost three decades earlier than the systems. There was almost three feet of fill placed above the native soil at the Philadelphia tree trench system. The native soil here is a Scantic silt loam (ScA) soil with a hydrologic soil group of C/D, as seen on Figure 5. This soil has a natural drainage class of poorly drained, a Medium runoff class and a very low capacity of the most limiting layer to transmit water: 0.06 to 0.2 in/hr (Web Soil Survey, 2019). During construction, the infiltration rate of the native soil at the bottom of the Philadelphia tree trench was measured at 0.03 in/hr (Ballestero et. al, 2016).

The Time of Concentration of the watershed was calculated based on the following method developed by Simas, found on the National Engineering Handbook (NRCS, 2010):

$$T_c = 0.0085 * W^{0.5937} * S^{-0.1505} * S_{nat}^{0.3131} \qquad \textit{(Eq. 2.1)}$$

$$S_{nat} = {1000}/{CN} - 10 \qquad \textit{(Eq. 2.2)}$$

Where:

T_c = time of concentration (hours)

S_{nat} = storage coefficient (in)

W = watershed width (ft)

S = watershed slope (ft/ft)

CN = curve number for the watershed

Following SWMM guidelines for the calculation of the watershed width:

$$W = {A_w}/{O_p}$$
(Eq. 2.3)

Where:

A_w = Watershed Area (ft^2)

O_p = Overland flow path (ft)

Substituting the parameter values of Wb in Eq. 2.3:

W_{wb} = 25472 ft^2 / 400 ft = 63.68 ft

The chosen Curve Number for Wb was 97, based on the weighted CN of 98 for Impervious surfaces and the CN of 84 for an Open Space in Fair Condition for a Soil Type D (USDA, 1986). For Wb, the impervious surface covers 95% of the watershed, while the pervious open space covers the other 5%.

$$CN = (P * CN_{perv} + I * CN_{imperv})$$
(Eq. 2.4)

Where:

P = pervious cover in watershed (%)

CN_{perv} = curve number of pervious cover

I = impervious cover in watershed (%)

CN_{imperv} = curve number of impervious cover

Substituting the values of Wb in Eq. 2.4:

$CN_{Wb} = (0.95*98 + 0.05*84) = 97.3 = 97$
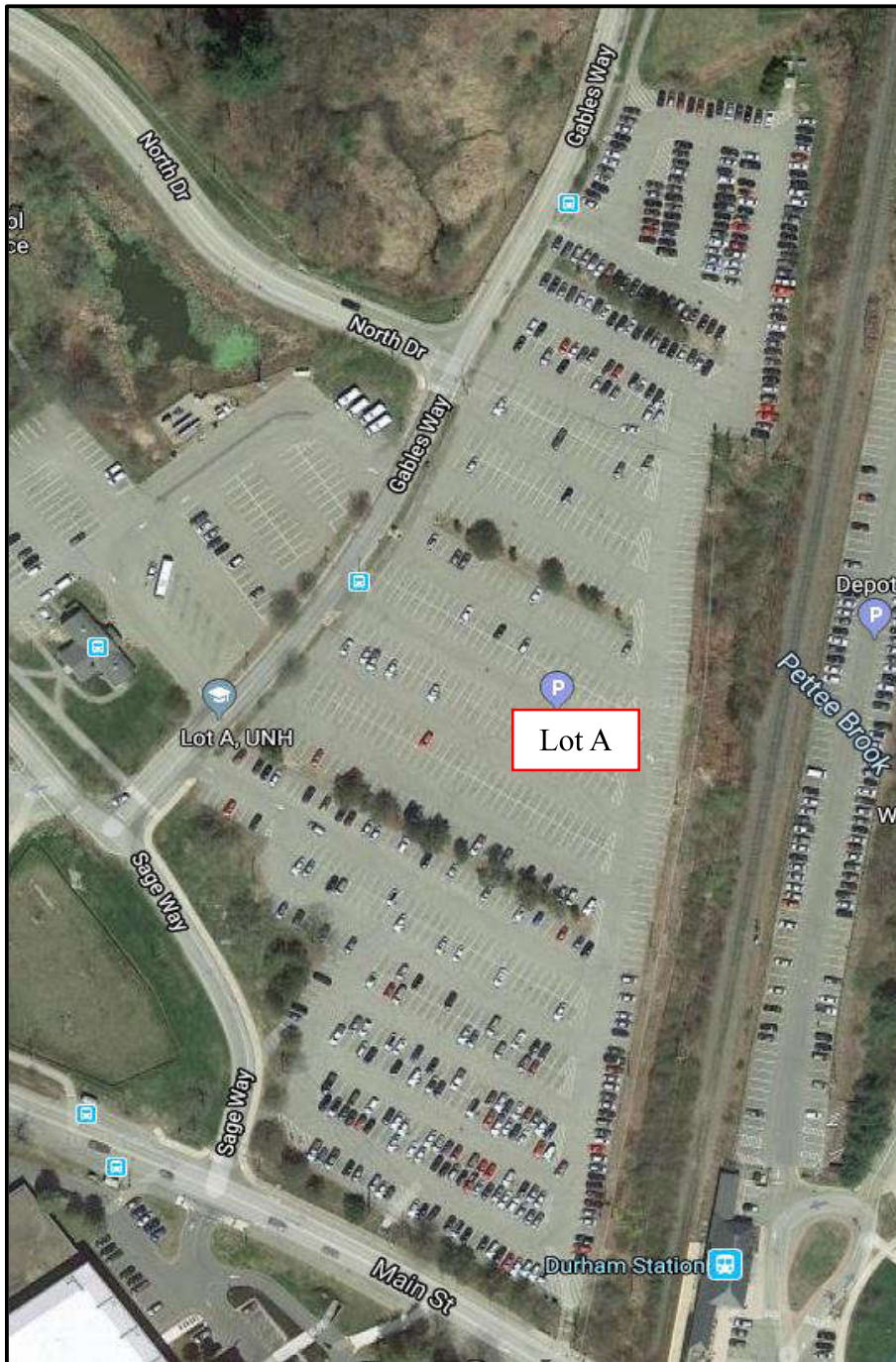
Figure 3 - Parking Lot A location in Durham, NH (Google Maps, 2020)

Substituting CN $_{Wb}$ in Eq. 2.2:

$$S_{nat} = 1000/97 - 10 = 0.309 \text{ in}$$

Finally, substituting the calculated values in Eq. 2.1:

$$T_c = 0.0085 \ (63.68 \text{ ft})^{0.5937} \ (0.023)^{-0.1505} \ (0.309 \text{ in})^{0.3131} = 0.12 \text{ hours}$$

The Time of Concentration for watershed Wb is 0.12 hours or 7.2 minutes. The design plans used in the construction of the system are in Appendix section A.2.1.
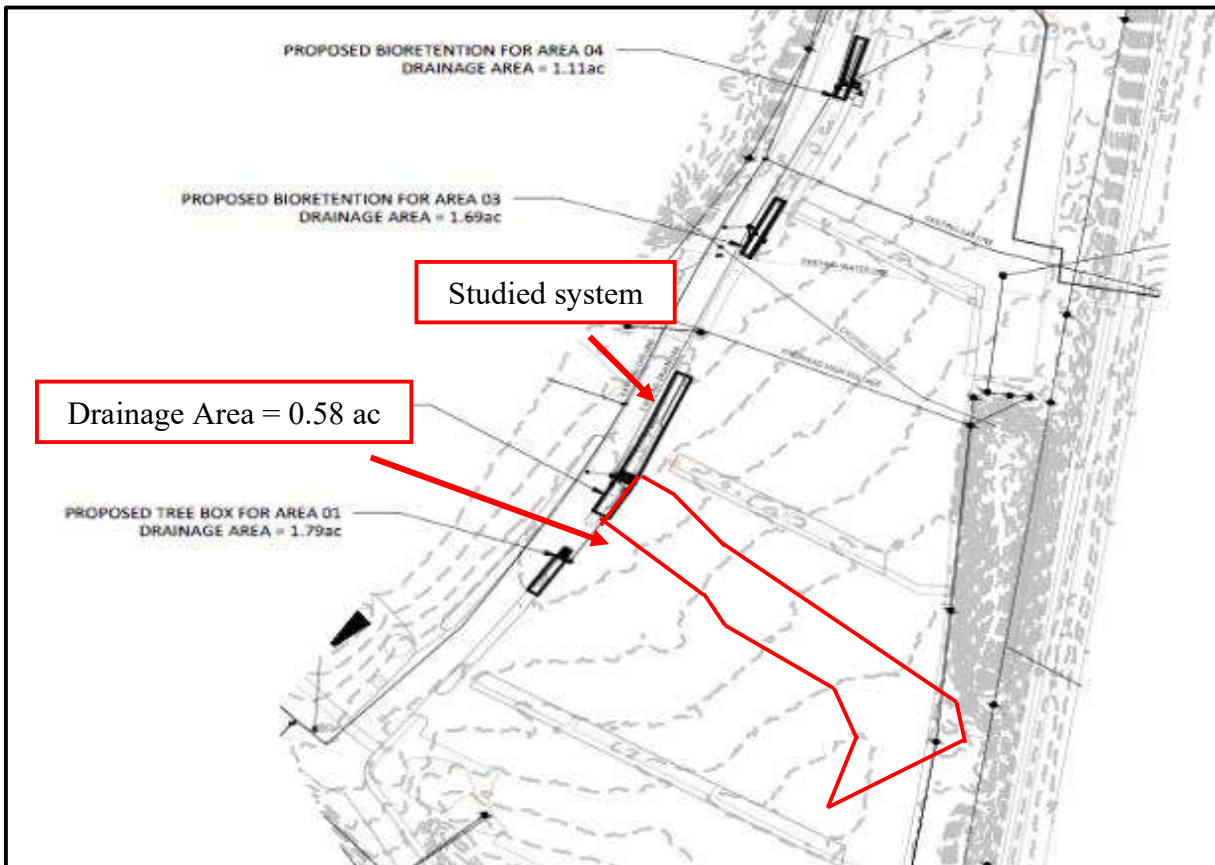
Figure 4 - Systems in parking lot A and their drainage areas (UNH Stormwater Center, 2014)
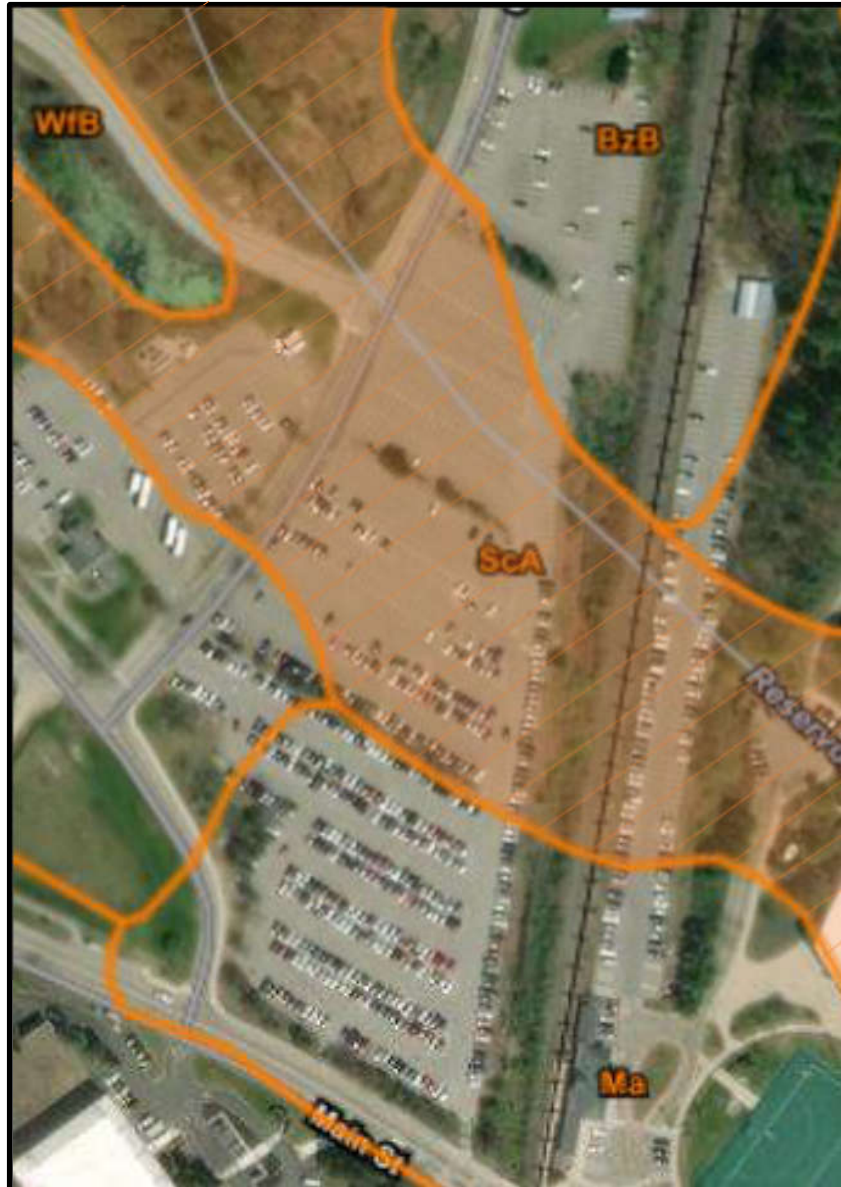
Figure 5 – Soil types in Parking Lot A (Web Soil Survey, 2019)

2.3 Lot E characteristics

Parking lot E is located on Evergreen Drive, across the street from Christensen Hall on the

UNH campus. Figure 6 shows the location of Parking Lot E on campus. About half of Lot E drains

to a subsurface gravel filter intended to also infiltrate runoff from the parking lot. The subsurface

gravel filter was built in 2016 as a retrofit project developed by the UNH Stormwater Center. Figure 7 shows the location of the system and its drainage area.

Figure 6 - Parking Lot E location in Durham, NH (Google Maps, 2020)

The watershed (Wc) used for the design of the subsurface gravel filter has an area of 23,086 square feet or 0.53 acres (UNH Stormwater Center, 2014), a slope of 2% based on design plans (UNH Stormwater Center, 2016) and an overland flow path of 210 ft. The watershed has a percent imperviousness of 100%, since it fully covers an asphalt parking lot. Web Soil Survey was used to delineate soils at this site. The soil at the site is a Hollis-Charlton very rocky fine sandy loam (HdC) with a hydrologic soil group D, as seen on Figure 8. This soil has a very high runoff potential

and a high capacity of the most limiting layer to transmit water: 0.06 in/hr (Web Soil Survey, 2019).

The watershed Time of Concentration was calculated using the same method as for Wb (NRCS, 2010). Substituting the parameter values of Wc in Eq. 2.3:
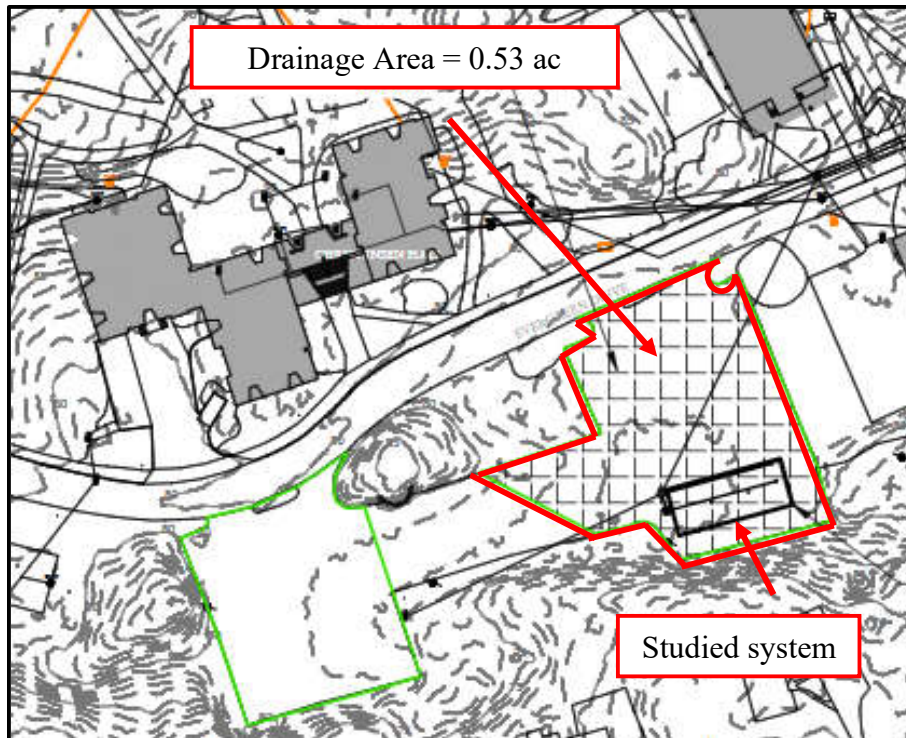
Figure 7 – Lot E watershed and system location (UNH Stormwater Center, 2016)

$$W_{Wc} = 23086 \text{ ft}^2 / 210 \text{ ft} = 109.9 \text{ ft}$$

The chosen Curve Number for Wc was 98, since the watershed consists completely of impervious surfaces (UNH Stormwater Center, 2016). Substituting CN_{Wc} in Eq. 2.2:

$$S_{nat} = 1000/98 - 10 = 0.204 \text{ in}$$

Finally, substituting all calculated values in Eq. 2.1:

$$T_c = 0.0085 \, (109.9 \text{ ft})^{0.5937} \, (0.02)^{-0.1505} \, (0.204 \text{ in})^{0.3131} = 0.15 \text{ hours}$$

The Time of Concentration for watershed Wc is 0.15 hours or 9 minutes. The design plans used in the construction of the infiltration basin are in Appendix section A.2.2.

Figure 8 – HdC soil type in Parking Lot E (Web Soil Survey, 2019)

CHAPTER 3: METHODOLOGY

3.1 Retrieving Data

Water level and barometric pressure were measured in two different spots in the Parking Lot A system (described as MW1 and MW2 in the design plans), and in three different spots in Parking Lot E (described as Inlet, Well 1 and Well 2 in the design plans). The data is measured continuously with a HOBO water level logger in all basins. For the Parking Lot A system, the water level in the trench was measured in 15-minutes intervals. For the Parking Lot E system, the water level in the inlet and in the trench was measured in 1-minute intervals.

HOBOs are pressure transducer sensors that measure temperature and absolute pressure. The absolute pressure at the sensor is measured from the elevation of its diaphragm to the water surface elevation. Each sensor has its own physical elevation in a system. Since the HOBO measures absolute pressure, first the raw HOBO data is adjusted by removing barometric pressure. Next, the elevation of the bottom of the system is used to adjust the HOBO water level to the 1988 North American Vertical Datum (NAVD). The result is the water elevation at each monitored location. This elevation may then be converted to a volume when multiplied by the area of the system times the porosity (usually 40% for stone).

Using precipitation data from the UNH Weather Station website, it is possible to model storms in SWMM. Along with watershed characteristics for both areas and design data from the original system designs, it is possible to simulate watershed runoff generated by these storms and model system performance (infiltration and water depth). This procedure is described in Chapter 4. The procedure used to model the systems in SWMM is described in Chapter 4. Some of the

SWMM outputs include the depth of water and volume of water in each system during and after the modeled storm.

The comparison of the simulated system water volume versus field measured, for each system, is described in Chapter 5.

3.1.1 Retrieving data in Lot A

Each monitoring location in the Lot A system is depicted in Figure 9. One transducer is located at Monitoring Well 1 (MW1) and the other is located at Monitoring Well 2 (MW2). Pond 1 represents the first inlet, where runoff is collected. Physically, this is a curb inlet catch basin. Pond 2 represents the second inlet, connecting Pond 1 to the system. Physically, this is also a catch basin. Pond 1 and Pond 2 are separated by a weir. Cross sections of the system are presented in section 3.1.3.
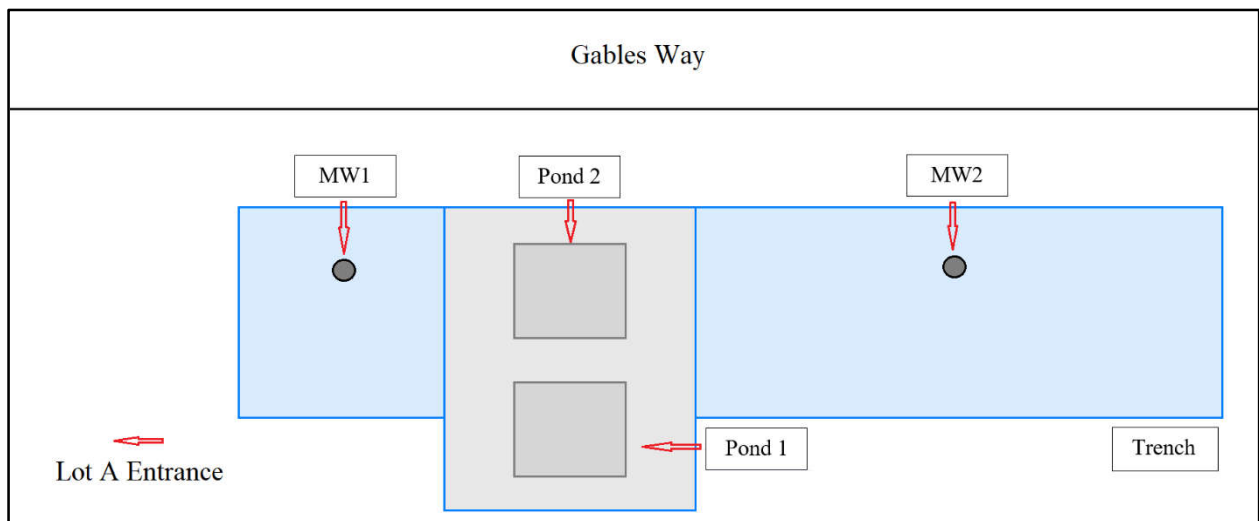
Figure 9 - Plan view of monitoring well locations in Parking Lot A

Table 3 shows well elevation. "Total depth" of a well is the distance from the well casing rim to the bottom of the well. "Distance from bottom" is the distance of the sensor in the well from the bottom of the system well. "Elevation of sensor" is the sensor elevation in respect to the 1988 NAVD.

Field ID	Total Depth of Well (ft)	Distance from Bottom (ft)	Elevation of Sensor (ft)
MW 1	8.88	1.3	89.44
MW 2	11.75	1.23	86.38

Table 3 – Elevation data for MW 1 and MW 2 in the Lot A system

In order to synthetize and utilize the HOBO data, it is necessary to import all data using the manufacturer's software called HOBOware. This software plots the absolute pressure and the temperature for each pressure transducer for the monitoring period. One important thing to consider is collecting the Barometric Pressure as well with a separate pressure transducer, in order to convert the absolute pressure monitored by the transducers in the wells to a depth of water over the sensor. Figure 10 shows the plotted barometric pressure collected near Pond 2. Because of the proximity of the two sites, this barometric data was also used for the Parking Lot E monitoring well pressure transducers.

After importing all the data into the software, it is necessary to use the Barometric Compensation Assistant in order to obtain the depth of water over each sensor at every monitoring location. The procedure performed in order to get the depth of water over the sensor and plot it in HOBOware is described in section A.5 in Appendix.

PTT - BARO
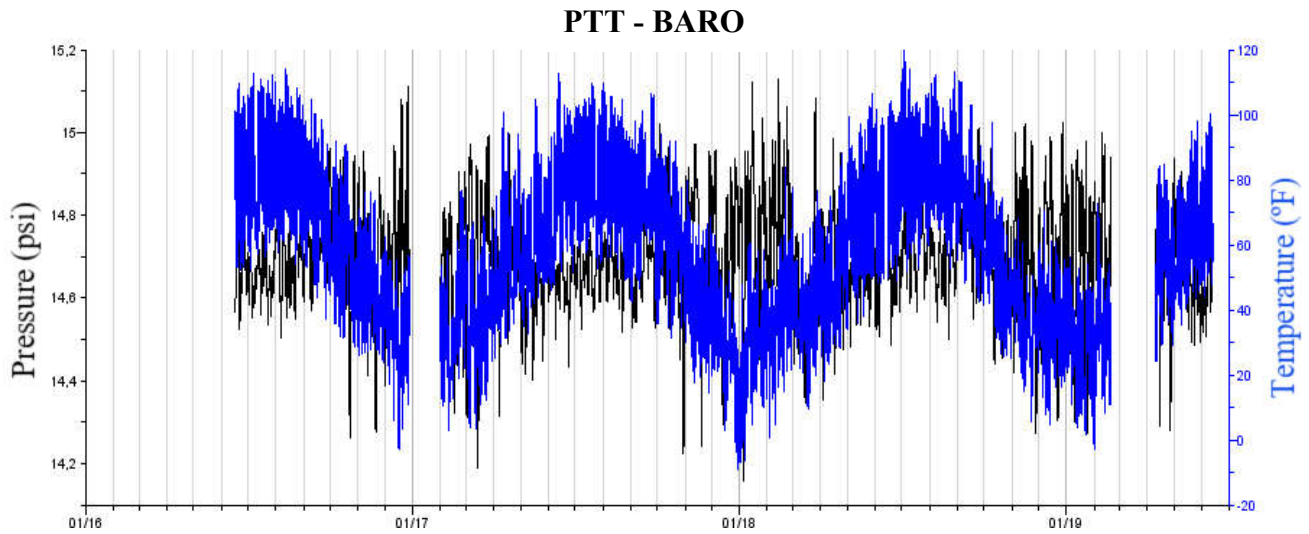
Figure 10 - Barometric pressure and air temperature data collected at Lot A

When the procedure is completed, a new file is created with the plot of absolute pressure, temperature, and water depth versus time. The output plots of the HOBOware analyses are located in Appendix section A.5.

For the system in Lot A, the elevation data given for each pressure transducer was related to each observation well. The well bottoms were not at the same elevation as the bottom of the trench. Therefore, it was necessary to correct the water level in the well to the water level in the trench in order to calculate water volume in the system. This procedure is described in section 3.1.3.

Since there were no sensors measuring the water level data in the pretreatment vaults (Pond 1 and Pond 2) for this system, lag time for the water to flow from them through the connecting pipe to the trench needed to be estimated. Model calibration will take into consideration the changing water levels in Pond 1 and Pond 2 versus the trench to calibrate this lag time. In addition, the model will be ultimately calibrating the seepage loss at the bottom of the system to match observed seepage rates. This process is described in Chapter 5.

3.1.2 Retrieving data in Lot E

Each monitoring location in the system is seen in Figure 11. One transducer is located at the Inlet where runoff is collected, and two others are located at Monitoring Well 1 and at Monitoring Well 2. Cross sections of the system are presented in section 3.1.3.
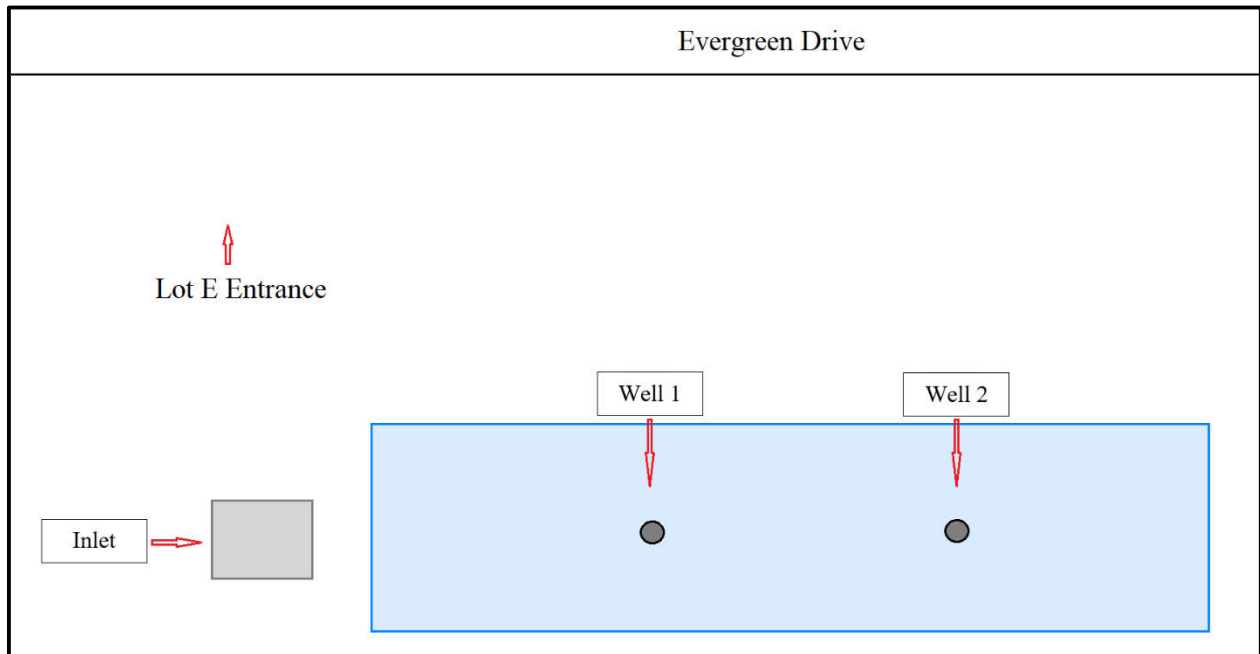
Figure 11 – Plan view of monitoring well locations in Parking Lot E

Table 4 shows the elevation data for the inlet and the subsurface gravel filter wells. "Total depth" of a well is the distance from the well casing rim to the bottom of the well. "Distance from bottom" is the distance of the sensor in the well from the well bottom. "Elevation of sensor" is the sensor elevation in respect to the 1988 NAVD.

Field ID	Total Depth of Well (ft)	Distance from Bottom (ft)	Elevation of Sensor (ft)
Inlet	5.6	0.991	71.84
Well 1	5	2.152	73.22
Well 2	5	1.706	72.77

Table 4 – Elevation data for Inlet, Well 1 and Well 2 in the Lot E system

The procedure to generate the water level and volume data from the sensor to HOBOware and convert it with the barometric compensation assistant was followed as mentioned in section 3.1.1. The output plots from the HOBOware analysis are located in Appendix section A.5.

For the Lot E system, the elevation data was related to the observation well, which is not at the same elevation as the bottom of the trench and it is needed to calculate system water volume. Therefore, it was necessary to convert the depth of water in the well to the depth of water in the trench. This procedure will be described in section 3.1.3.

Since there is a pressure transducer in the Inlet, it was possible to determine the initial water level in the inlet. In this case, the system model calibration was performed based on the observed infiltration (seepage loss) only. This process is described in Chapter 5.

3.1.3 Preparing the data

As mentioned previously in sections 3.1.1 and 3.1.2, it is necessary to correct the monitoring well depth of water data to reflect the depth of water in each system since the monitoring well bottoms do not have the same elevation as the bottom of the systems. Figures 12 and 13 diagram the well and trench elevations for Parking Lot A and Parking Lot E, respectively. All elevations are based on the NAVD 1988.
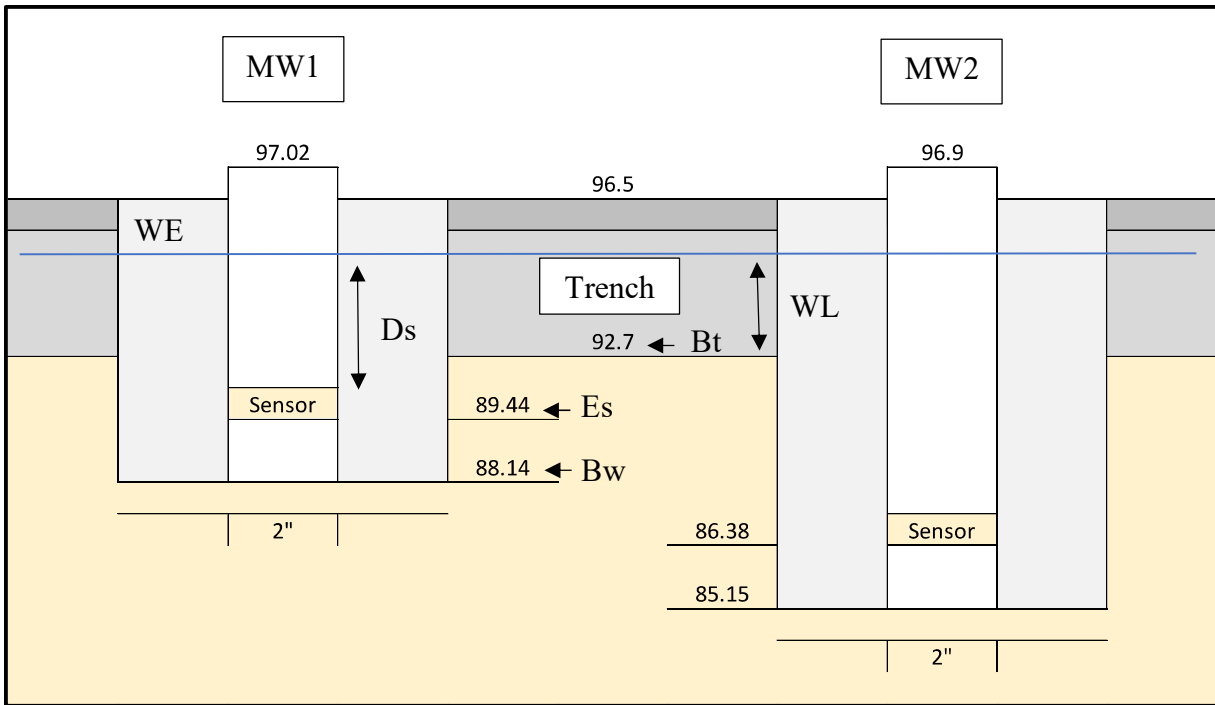
Figure 12 – Profile diagram of well and trench elevations (ft) in MW1 and MW2 at Lot A

Each well has a pressure transducer at a different elevation, and this elevation is above or below the bottom of the system. For this reason, the following procedure was performed to correct the elevation of the water level in the system in relation to the ones measured in the wells:

$$WE = D_s + B_w + E_s$$ *(Eq. 3.1)*

Where:

WE = water elevation in well (ft)

D_s *= depth of water measured above sensor in well (ft)*

E_s *= elevation of sensor in well (ft)*

B_w *= elevation of bottom of the well (ft)*
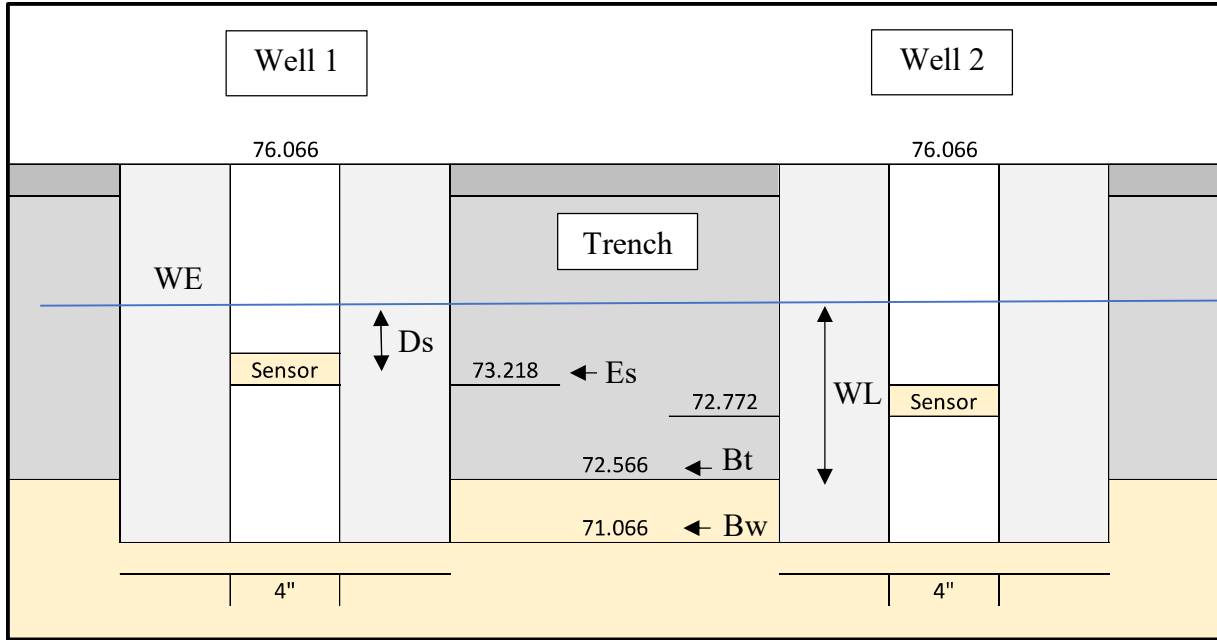
Figure 13 – Profile diagram of well and trench elevations (ft) in Well 1 and Well 2 at Lot E

$$WL = WE - B_t \qquad\qquad (Eq.\ 3.2)$$

Where:

WL = depth of water in trench (ft)

B_t = *elevation of bottom of the trench (ft)*

Each trench was designed with a stone porosity of 40%. This means that the volume of water in the trench is only 40% of the total volume of the trench. Therefore, the system volume computed from water levels corrected to the system bottom is multiplied by 0.4 to then obtain the water volume in the system.

$$V_t = WL * A_t * 0.4 \qquad\qquad (Eq.\ 3.3)$$

Where:

V_t = *volume of water in trench (ft³)*

A_t = *area of trench (ft²)*

It was established that the analysis would be performed based on an average system water volume, since there were two monitoring wells in the subsurface gravel filter. Each well had one pressure transducer and the water level readings were different for each.

$$V_e = (V_{t1} + V_{t2}) / 2 \hspace{4cm} (Eq.\ 3.4)$$

Where:

V_e = *average observed volume of water in trench (ft³)*

V_{t1} = *observed volume of water measured using well 1 data (ft³)*

V_{t2} = *observed volume of water measured using well 2 data (ft³)*

3.2 Data Analysis

"Hydrological modeling can be defined as the characterization of real hydrologic features and system by the use of small-scale physical models, mathematical analogues, and computer simulations" (Allaby and Allaby, 1999). The following sections will describe how the SWMM model was calibrated and evaluated in this project.

3.2.1 Model Calibration

The calibration of a model demonstrates that the model is capable of reproducing values of depth and volume of water in the stormwater systems as observed in the field, after a process of optimization of certain parameters. "Generally, the goodness-of-fit between simulated and measured variables is not satisfactory based on the initial values of hydrologic and hydraulic parameters used in the model" (Yu, 2015). It is possible to improve the goodness-of-fit between model and reality with an adjustment of certain parameters in the model. An example of this process is shown in Figure 14. However, in this study, the model results will not intervene on the real monitored system, as it is already built. The results are only used for evaluation of model performance.

The process starts by comparing the output of collected values (in this case, volume of water) from the real systems and the output of modeled values from SWMM. This initial model is made using initial estimates of parameters. The first comparison is made by an error analysis, which is described in section 3.2.2. If the error is acceptable by the standards of the evaluation, the calibration is considered complete, and the model is satisfactory. If the error is unacceptable, there is another adjustment in these model parameters to start the process again.

The process is trial and error, and there is the option of using a sensitivity analysis in order to identify which parameters are most relevant. The sensitivity analysis is made by changing one parameter at a time, while other model parameters are held constant. If the change in this parameter causes a significant effect in the final result, this parameter is considered important in the calibration. "The range of adjustment to values of hydrologic and hydraulic parameters must be constrained by plausible site-specific data" (Yu, 2015).
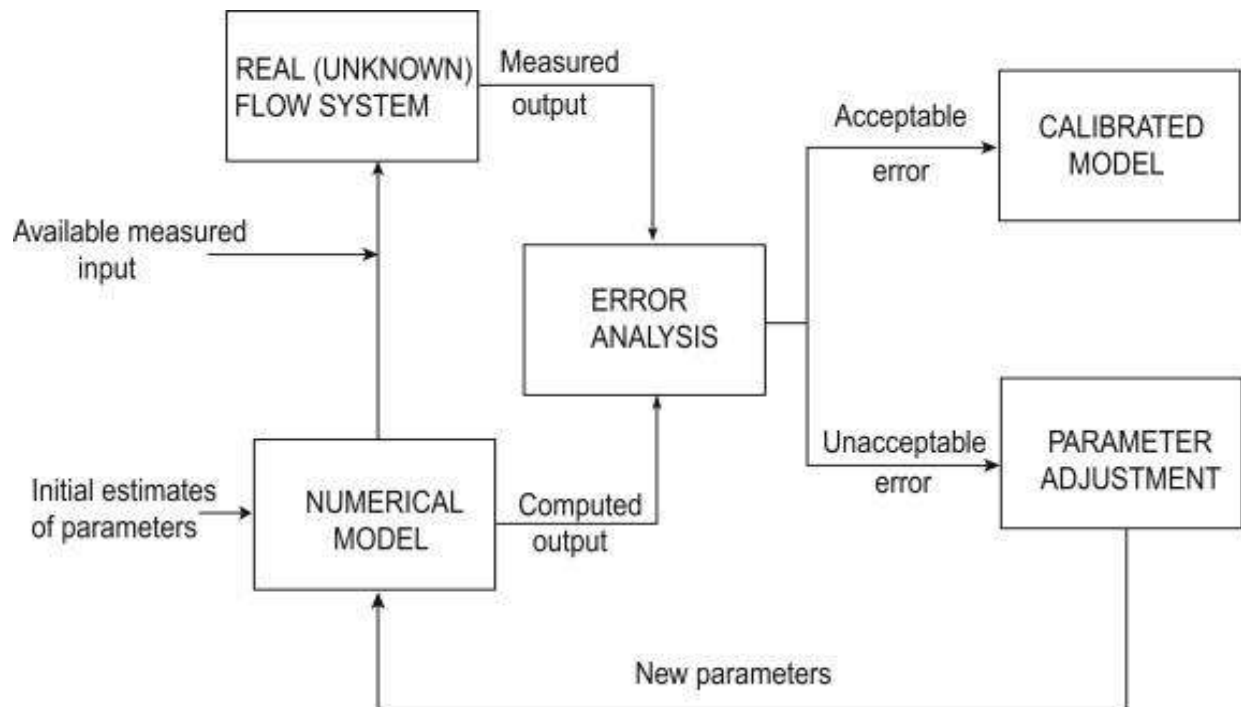
Figure 14 – Flow diagram of the calibration processes (Freeze and Cherry, 1979)

3.2.2 Model Evaluation

The evaluation of the model based on a goodness-of-fit criteria implies that the data generated by the model is compared to the observed data using a fitting statistic or a discrepancy measure (Mishra and Datta-Gupta, 2018). The process of evaluating the model is a part of the process of calibrating the model.

Goodness-of-fit evaluation can be applied through a visual comparison of the plotted observed data versus the model results, as well as proper statistical methods. The visual comparison usually includes a plot of simulated and measured variables. The statistical methods consist of measures to quantify error between the data that is being compared. This can be made by several measures of discrepancy. In this project, the statistical goodness-of-fit measures

employed to calculate the discrepancy between observed data and the model are the Root Mean Square Error (RMSE) and the Nash Sutcliffe Efficiency equation (NSE).

The Root Mean Square Error measures the square root of the average squared difference between the observed data and their corresponding results in the model. RMSE calculation is presented in Equation 3.5. The desirable value for the RMSE is close to zero, meaning there is a small discrepancy between observed data and expected data.

$$\text{RMSE} = \sqrt{\frac{1}{n} \sum_{i=1}^{n}(O_i - E_i)^2} \qquad\qquad (Eq.\ 3.5)$$

Where:

n = *number of observations in the dataset*

O_i = *observed volume of water for data point i (ft³)*

E_i = *expected volume of water for data point i (ft³)*

The Nash-Sutcliffe equation (NSE) is another method to measure the predictive power of a model. It can range from $-\infty$ to 1. NSE close to 1 indicates that the model matches almost perfectly the observed data in the field. The NSE equation can be found on the Handbook of Hydrology (Maidment, 1993).

$$NSE = 1 - \sum_{i}^{n}(E_i - O_i)^2 \ / \ \sum_{i}^{n}(O_i - \bar{O})^2 > 0 \qquad\qquad (Eq.\ 3.6)$$

Where:

n = *number of data points*

E_i = *expected volume of water for data point i (ft³)*

O_i = *observed volume of water for data point i (ft³)*

$\bar{O}$ = *mean value of observed volume of water dataset*

These two measurements, along with the visual comparison of observed data and the model results, were used to verify the goodness-of-fit of the model in this project. Calibration will stop when the RMSE results are as low as possible, when the NSE results are as high as possible and when the compared curves are matching as close as possible in the visual comparison.

CHAPTER 4: MODELING IN SWMM

SWMM is a commonly used, free desktop program, developed to support stormwater management. It was developed in 1971 and it is currently on version 5.1. It may be used for "planning, analysis, and design related to stormwater runoff, combined and sanitary sewers, and other drainage systems" (EPA, 2020). SWMM has the ability to evaluate stormwater control strategies and recently has been a tool for modeling GSI stormwater control solutions. It "contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures" (EPA, 2020).

SWMM uses a "dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas" (SWMM Manual, 2015). SWMM calculates runoff volume based on the characteristics of a subcatchment that receives precipitation. This runoff is then routed to a system modeled as a storage unit and it is infiltrated to the surrounding soil. Also, SWMM may be used to estimate pollutant loads associated with runoff, but this analysis is not included in this study.

Since it was created, SWMM has been used in several sewer and stormwater studies. Typical applications of SWMM include the "design and sizing of drainage system components for flood control, sizing of detention facilities and their appurtenances for flood control and water quality protection, (…) evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings" (SWMM Manual, 2015).

It is possible to model the quantity and quality of runoff, the flow rate and depth in each pipe, "during a simulation period comprised of multiple time steps" (SWMM Manual, 2015).

4.1 Modeling Storms

The first topic to discuss about modeling in SWMM is how to model the storms in the software. The storm events were selected based on data collection monitoring time period for both parking lots. The periods of data collection were different for each lot. The process of selecting the storm events is detailed in sections 4.1.1 and 4.1.2. The storm events were separated into three categories of intensity based on the total precipitation: a Small storm is from 0 in to 1 in, a Medium storm is from 1 in to 2 in and a Large storm is greater than 2 in of precipitation. All precipitation data from these storms was retrieved in the UNH Weather Station website http://www.weather.unh.edu. The procedure of modeling the storms in SWMM is described in Appendix section A.1.1.

4.1.1 Lot A Storms

The depth of water of the west side of the Philadelphia tree trench system was collected using monitoring well (**MW1**) during the period of July 2016 to October 2017, and from July 2018 to May 2019. The depth of water for the east side of the Philadelphia tree trench was collected using monitoring well (**MW2**) during the period of June 2016 to June 2018, and from November 2018 to May 2019.

The summary of all selected storms for the Parking Lot A analysis is presented in Table 5. Figure 15 shows the time series rainfall depth for the September 19th, 2016 storm event plotted by SWMM.

Storm	Date	Total Precipitation (in)	Start	End	Category
1	9/19/2016	1.31	3:00 AM	8:00 AM	Medium
2	10/21/2016	3.42	7:00 PM	12:00 AM	Large
3	10/30/2017	1.82	12:00 AM	7:00 AM	Medium
4	11/27/2018	1.22	12:00 AM	9:00 AM	Medium
5	01/01/2019	0.52	12:00 AM	5:00 AM	Small
6	4/22/2019	0.57	4:00 PM	11:00 PM	Small

Table 5 - Summary of selected modeled storms for Parking Lot A calibration
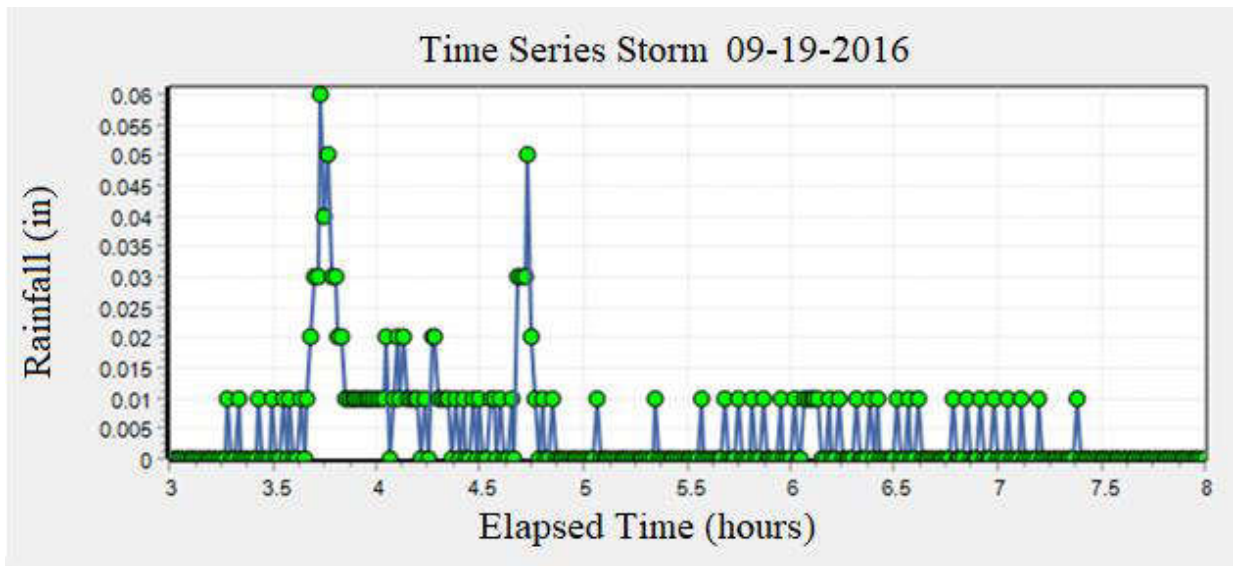
Figure 15 - Time Series of the September 19th, 2016 storm plotted in SWMM

4.1.2 Lot E Storms

The depth of water for the subsurface graver filter inlet was collected using a monitoring well (**Inlet**) during the period of September 2016 to December 2017, from February 2018 to June 2018, and from September 2018 to November 2018. The depth of water for the subsurface gravel filter was collected using two monitoring wells (**Well 1** and **Well 2**). The depth of water was collected in Well 1 during the period of September 2016 to June 2018, and from September 2018 to November 2018. Finally, the depth of water was collected in Well 2 during the period of September 2016 to December 2017, from February 2018 to June 2018, and from September 2018 to November 2018.

The summary of all selected storms for Parking Lot E analysis may be seen in Table 6.

Storm	Date	Total Precipitation (in)	Start	End	Category
1	9/19/2016	1.31	3:00 AM	8:00 AM	Medium
2	10/21/2016	3.42	7:00 PM	12:00 AM	Large
3	04/12/2017	0.18	2:00 PM	5:00 PM	Small
4	5/22/2017	0.41	6:00 AM	11:00 PM	Small
5	10/30/2017	1.82	12:00 AM	7:00 AM	Medium
6	4/16/2018	2.31	10:00 AM	10:00 PM	Large

Table 6 - Summary of selected modeled storms for Parking Lot E calibration

4.2 System in Parking Lot A

4.2.1 Modeling the subcatchment

As mentioned on Chapter 2, the watershed area draining into the Philadelphia tree trench system is 25,472 square feet (0.58 acres), a slope of 2.3%, a Time of Concentration of 7.2 minutes and a Curve Number of 97.

The watershed was modeled in SWMM as subcatchment *Wb*, and its parameters were added in the SWMM subcatchment editing menu. The procedure to model the Subcatchments in SWMM is described in Appendix section A.1.2.

According to the New Hampshire Stormwater Manual, a stormwater system has to fully drain within 72 hours (NHDES, 2008). However, the studied system was designed to fully drain in 24 hours.

Some challenges were faced in this part of the modeling since some of the parameters such as the %Zero and the Percent Routed were estimated. According to the SWMM manual, the former parameter is described as the impervious area in the watershed with no depression storage, "which is the maximum surface storage provided by ponding, surface wetting, and interception" (SWMM Manual, 2015). The latter is described as the percent of runoff routed between subareas (pervious and impervious) in the watershed. The value estimated for the Impervious area with zero storage was 0% and for the Percent routed it was 100%.

Parameter values and sources for the SWMM model are described in Table 7.

Parameter	Value	Source
Width	63.68 ft	Calculated in Chapter 2
Impervious n	0.016	Rough Asphalt (Chow, 1959)
Pervious n	0.05	Scattered brush, heavy weeds (Chow, 1959)
Impervious Storage	0.1 in	SWMM suggested value
Pervious Storage	0.25 in	SWMM suggested value
Subarea Routing	Outlet	Runoff from both areas flows to outlet
Percent Routed	100%	Estimated based on watershed
Infiltration Data	Curve Number Method	Calculated on Chapter 2

Table 7 - Summary of initial input parameters used to create watershed Wb

After modeling the watershed, it was possible to run the model in order to generate a runoff hydrograph for each storm. Figure 16 shows the hyetograph of the storm of September 19th, 2016 and Figure 17 shows the time series plot of the Runoff in subcatchment Wb for the duration of this storm event. The procedure to run the model and plot the runoff hydrograph in SWMM is described in Appendix section A.1.3.

Hyetographs for all other storms are located in Appendix section A.3. Runoff for all other storms was modeled and plotted in similar fashion. The figures are located in Appendix section A.4.1.
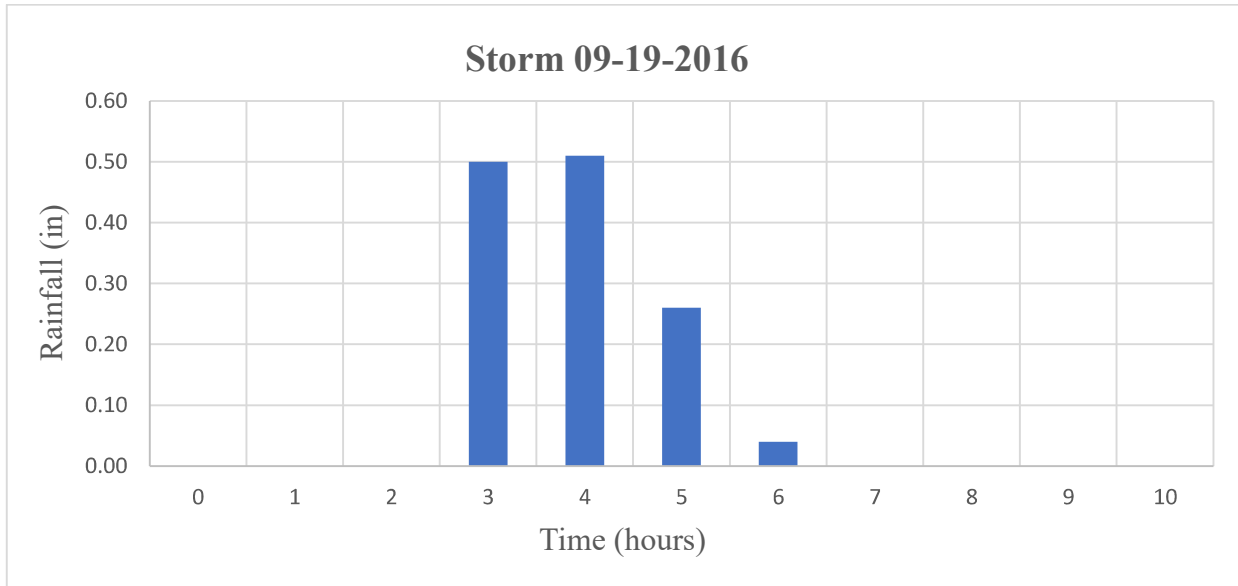
Figure 16 - Hyetograph of Storm 09-19-2016
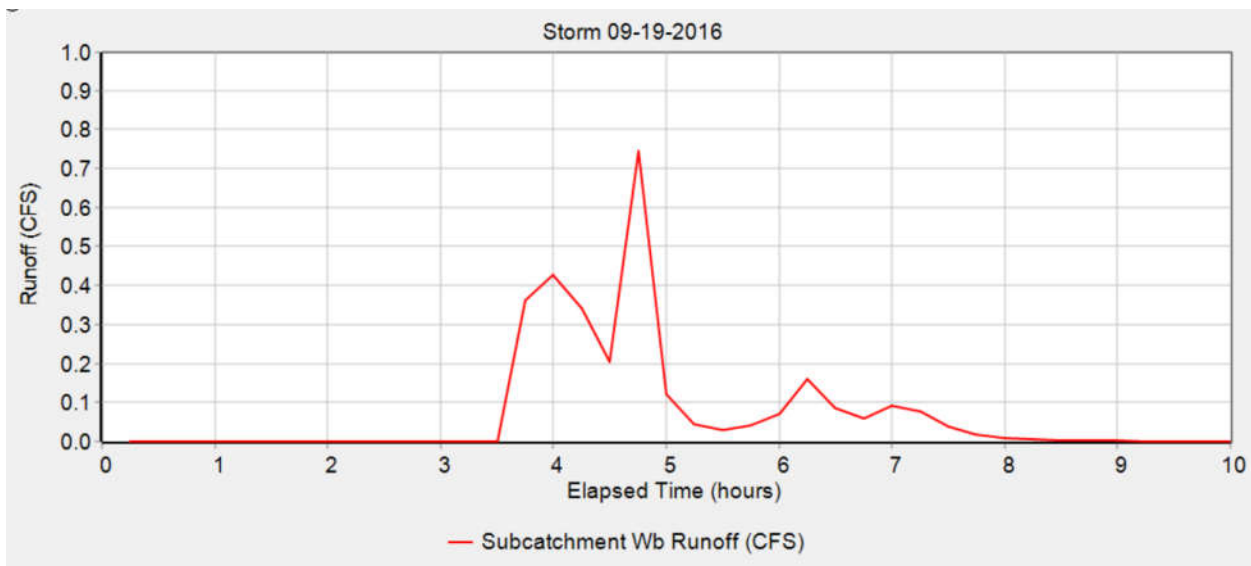
Figure 17 - Sub catchment Wb Runoff in cfs of Storm 09-19-2016

4.2.2 Modeling the Lot A Philadelphia tree trench system

There are two distinct ways to model this system. The first way is as a storage unit which does not consider the soil and stone layers. There must be a preliminary process to simulate the

porosity in the system. This process is described in section 3.1.3. Infiltration may be simulated as a seepage loss at the bottom of the storage unit (SWMM Manual, 2015).

The second way to model the system is as a LID Control system. This option does not show the storage results in a time series manner. The results provided by SWMM are initial and final storage of the LID system, and this does not correspond to the needs of this analysis which compares continuous data. Thus, this method cannot be used to compare the data collected in the field. However, the modeling of the systems as a LID Control option will be included in Chapter 6 for the comparison of total infiltrated volumes.

The conceptual SWMM model of the Philadelphia tree trench system and its watershed may be seen in Figure 18. This figure serves as guidance to better understand the procedure to model each structure in the system.

As seen in Appendix section A.2.1, the design plans show that the Philadelphia tree trench is divided into two components. The first component is the inlet catch basin and was modeled as *Pond1*, and this unit receives all runoff from the watershed (subcatchment Wb). The second component is the stone storage layer modeled as *Pond2*. Pond2 receives the flow from Pond1. The dimensions for these components are based on the design plans provided by the UNH Stormwater Center and described in Table 8.

Parameter	Pond 1 Value	Pond 2 Value
Width	7 ft	7 ft
Length	3 ft	7.5 ft
Area	21 ft²	52.5 ft²
Depth	4.67 ft	4.67 ft

Table 8 - Summary of parameters used to define storage units Pond1 and Pond2 in Lot A
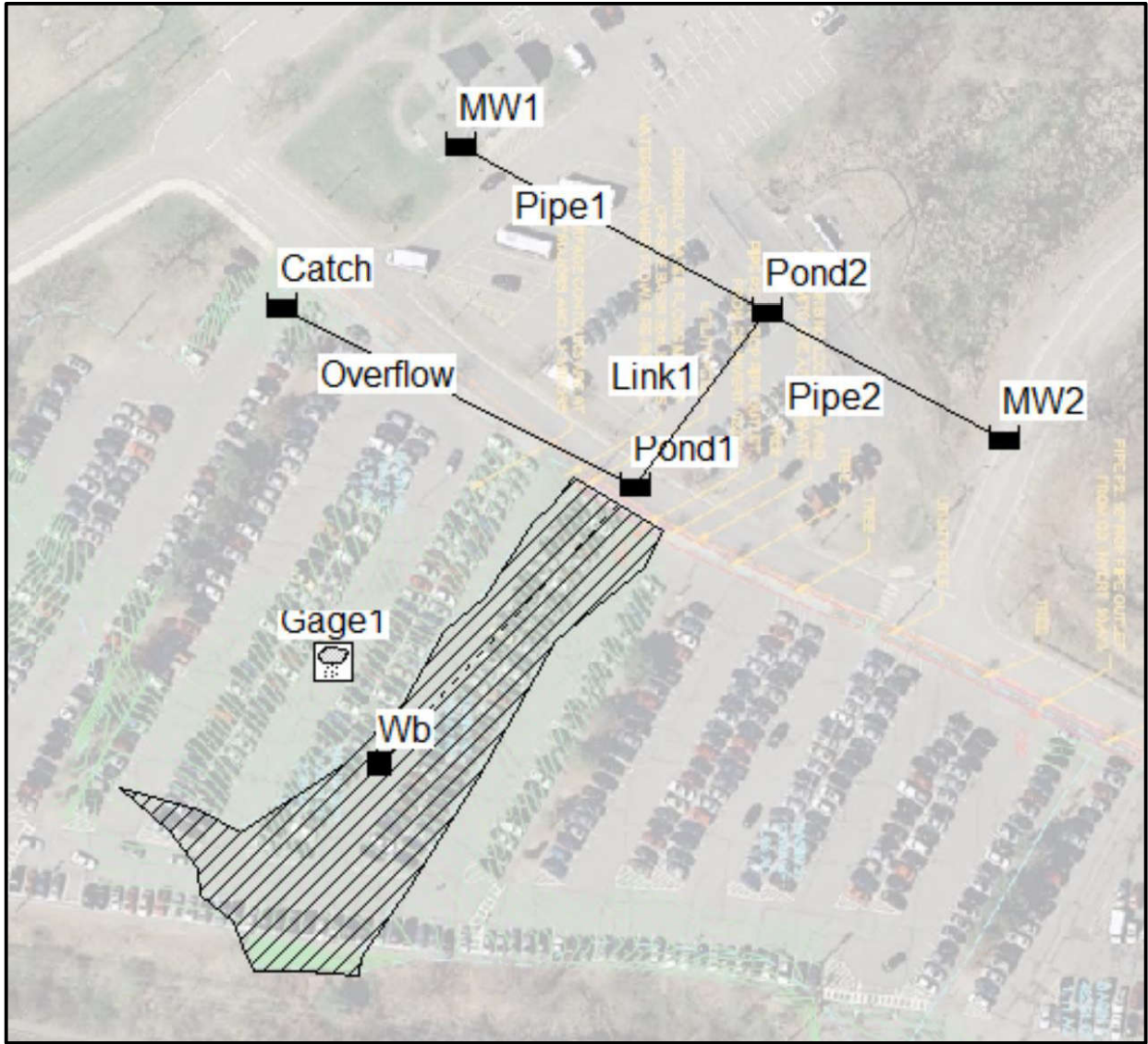
Figure 18 – Final Lot A system model

The procedure to model the Storage Units in SWMM is described in Appendix section A.1.4.

Pond1 and Pond2 are separated by a concrete weir. In the model, the weir was called **_Link1_** and was modeled as a closed rectangular conduit link. The procedure to model the Links in SWMM is described in Appendix section A.1.5.

SWMM models conduit links using Manning's equation. This equation is represented as Equation 4.1. If the conduit link's shape is defined as a Force Main, it uses either Hazen-William's equation or Darcy-Weisbach's equation. This is not the case for this study.

$$Q = V*A = (1.49/n)*A*R^{2/3}*\sqrt{S} \qquad\qquad (Eq.\ 4.1)$$

Where:

Q = flow (ft3/s)

V = velocity (ft/s)

A = flow area (sf)

n = Manning's roughness coefficient

R = hydraulic radius (ft)

S = slope (ft/ft)

Modeling the weir in SWMM uses the physical geometric characteristics also found on the design plans (Appendix section A.2.1). These characteristics are described in Table 9.

Parameter	Value	Source
Shape	Closed Rectangular	Stormwater Center design plans[1]
Maximum Depth	0.083 ft	[1]
Length	0.5 ft	[1]
Roughness Coefficient	0.012	Concrete (ACPA, 2012)
Inlet Offset	4.587 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	4.587 ft	Elevation of pipe in outlet storage unit [1]

Table 9 - Summary of parameters used to model the weir Link1 in Lot A

In order to plot the depth of water in a storage unit in SWMM, the elapsed time should be enough to simulate a maximum depth of the storage for each storm. For this study, the elapsed time was selected for each storm separately varying from 1 to 2 days, while a time step of 1 minute was chosen for all storms. The elapsed time was chosen depending on how long it takes for the system to reach a stable maximum depth of storage. For storms starting early in the day, an elapsed time of 1 day was enough. For storms starting later in the day, an elapsed time of 2 days was selected. The procedure to set the elapsed time in SWMM is described in Appendix section A.1.6.

The depth of water of the storage units (Pond1 and Pond2) for the storm of September 19th, 2016 is show in Figure 20. The procedure to plot the depth of storage in SWMM is described in Appendix section A.1.7. Figure 20 shows that for this storm, the water level in Pond1 reaches Link1 elevation and overflows to Pond2 within a few time steps. After that, water flows to the trench when it reaches Pipe1 and Pipe2 elevation. This preliminary model did not consider infiltration, essentially assuming that the units are made of concrete. Therefore, the water depth stays almost constant after the cessation of runoff.

After modeling the inlet storage units, it is necessary to model the actual geometry of the Lot A Philadelphia tree trench. The trench was split into two storage units: *MW1* representing the west side of the trench (monitored by MW 1) and *MW2* representing the east side of the trench (monitored by MW 2). Each unit receives half of the total runoff volume generated by the Subcatchment. They were modeled following the same procedure described in Appendix section A.1.4.

Figure 19 - Hyetograph of Storm 09-19-2016
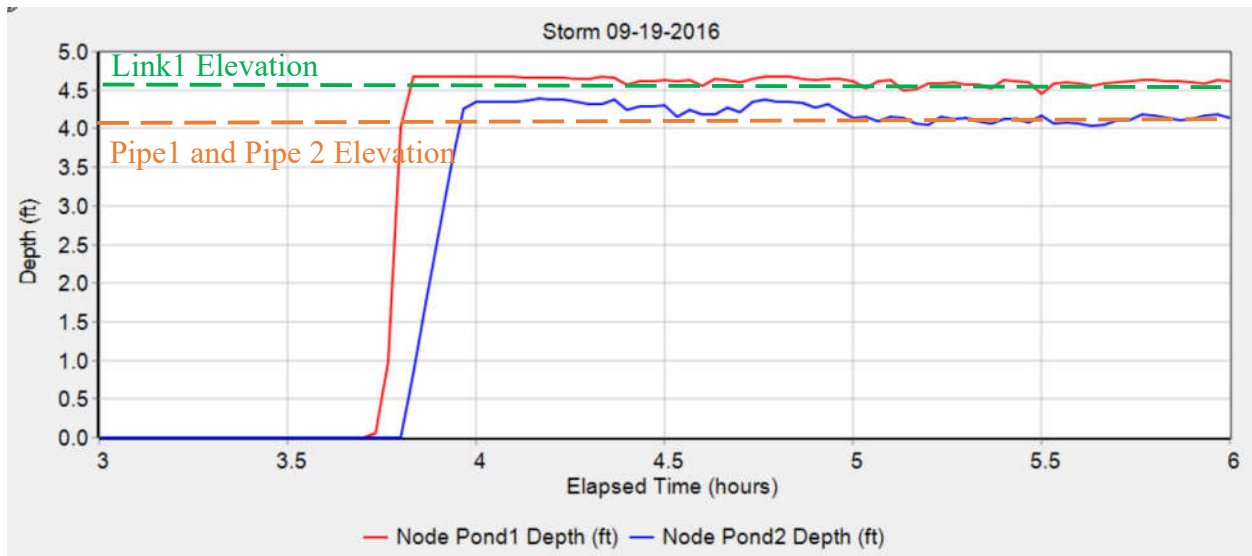
Figure 20 – Modeled depth of water in Pond1 and Pond2 for Storm 09-19-2016, assuming no

infiltration

The system dimensions were based on the design plans provided by the UNH Stormwater

Center and are described in Table 10.

Parameter	Trench MW 1 Value	Trench MW 2 Value
Area	612 ft²	1700 ft²
Depth	3.8 ft	3.8 ft

Table 10 – System dimensions for storage units MW1 and MW2

A conduit link ***Pipe1*** was created to connect Pond2 to MW1 and a conduit link ***Pipe2*** was created to connect Pond2 to MW2. The same procedure to create and edit a conduit link was followed, as described in Appendix section A.1.5. Pipe1 simulates the perforated pipe connecting Pond2 to the west side of the infiltration trench (MW1). Pipe2 simulates the perforated pipe connecting Pond2 to the east side of the infiltration trench (MW2). The dimensions for Pipe1 and Pipe2 were based on the design plans provided by the UNH Stormwater Center and are described in Table 11.

Parameter	Value	Source
Shape	Circular	Stormwater Center design plans[1]
Maximum Depth	0.67 ft	Pipe diameter [1]
Length	0.5 ft	[1]
Roughness Coefficient	0.02	PVC (ACPA, 2012)
Inlet Offset	4 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	0.95 ft	Elevation of pipe in outlet storage unit [1]

Table 11 - Summary of parameters used to create Pipe1 and Pipe2 in Lot A

SWMM uses the Manning equation to model links. It is not possible to model links as perforated pipes in SWMM. There are two possible ways to try to simulate the performance of perforated pipes in SWMM. The first one is to reduce the diameter of the pipe, as it simulates the velocity the water fills the trench. The orifices make the trench fill in a much slower rate, so does a smaller pipe diameter in SWMM. The second way is to model the link connecting to several

junctions before it connects to the trench. This is very labor intensive, and it is not effective if the exact number of orifices in the pipe is not known beforehand.

An initial length of 0.5 ft was chosen to simulate the thickness of the walls of Pond2. When modeling the perforated pipe in SWMM with the lengths seen in the design plans, there would be a significant lag time before water started filling the trench. This could be due to the fact that in real life there are orifices that make water flow to the trench faster than it would if the water needed to go through 30 ft of pipe before reaching the trench. The maximum depth represents the diameter of the pipe and a value of 0.67 ft was chosen based on the design plans found in Appendix section A.2.1.

The Philadelphia tree trench system contains a catch basin. This basin was modeled as a storage unit *Catch* and it receives overflow water from Pond1, simulating the inlet overflow to the parking lot if full. This unit was modeled based on the design plans in Appendix Section A.2.1. Table 12 shows Catch storage unit dimensions.

Catch Basin	
Parameter	**Value**
Area	52.5 ft²
Depth	4 ft

Table 12 - Summary of Catch basins dimensions in Lot A

Parameter	Value	Source
Shape	Open Rectangular	Simulating overflow to parking lot
Maximum Depth	1 ft	Simulating overflow to parking lot
Length	0.01 ft	Simulating overflow to parking lot
Roughness Coefficient	0.016	Rough Asphalt (Chow, 1959)
Inlet Offset	4.67 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	4 ft	Elevation of pipe in outlet storage unit [1]

Table 13 - Summary of parameters used to create Overflow in Lot A

Pond1 is connected to Catch by link, modeled as ***Overflow***. Table 13 shows Overflow link dimensions.

There is also an underdrain connecting both MW1 and MW2 storage units to the catch basin. However, this structure was not used in this research since the underdrain is currently capped. This means that no flow can leave the system this way.

As done previously, this preliminary model assumed no infiltration for MW1 and MW2. Figure 22 shows the depth of storage of MW1 and MW2 before, during and after the storm of September 19th, 2016 for the preliminary run.

Infiltration will be included in the analysis in the calibration processes of the Philadelphia tree trench system as described in Chapter 5.
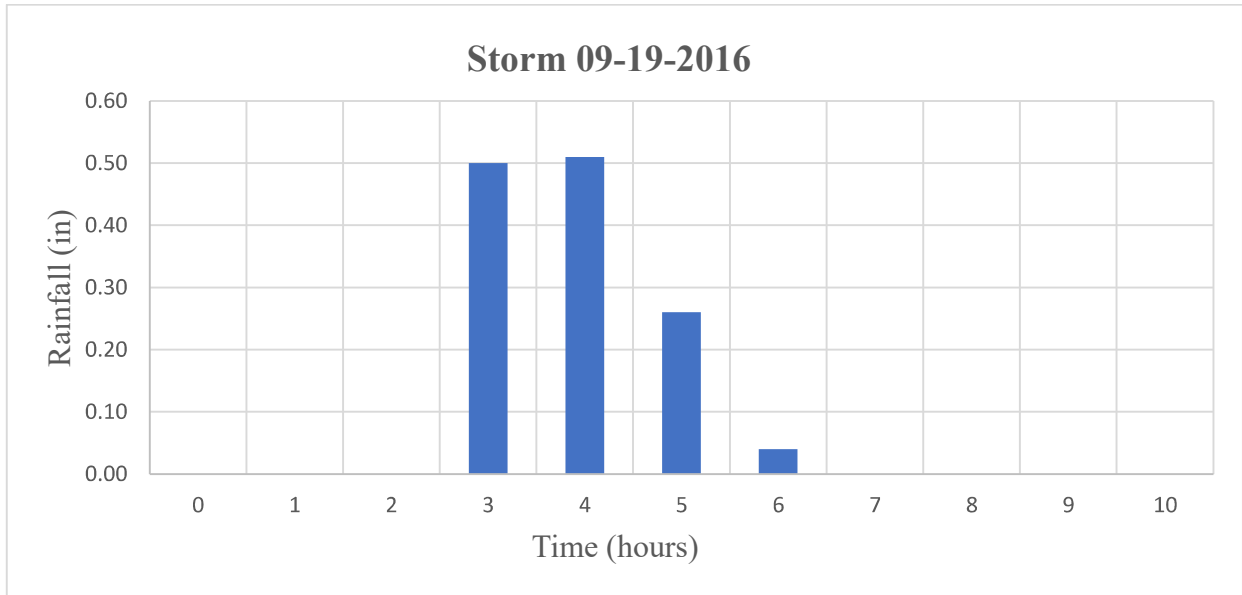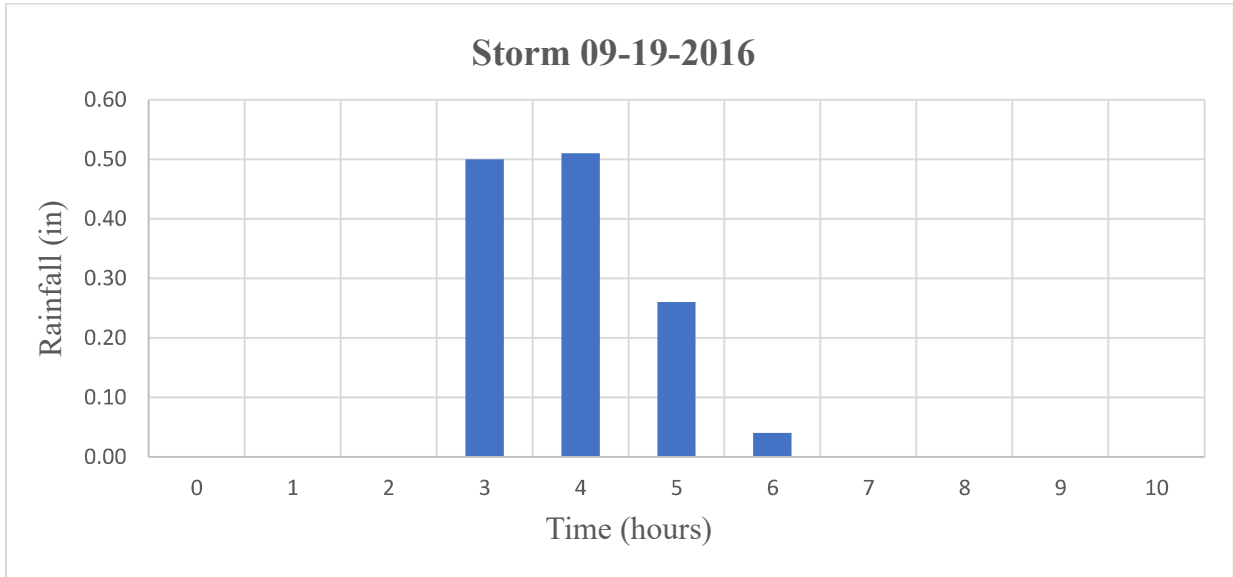

Figure 21 - Hyetograph of Storm 09-19-2016
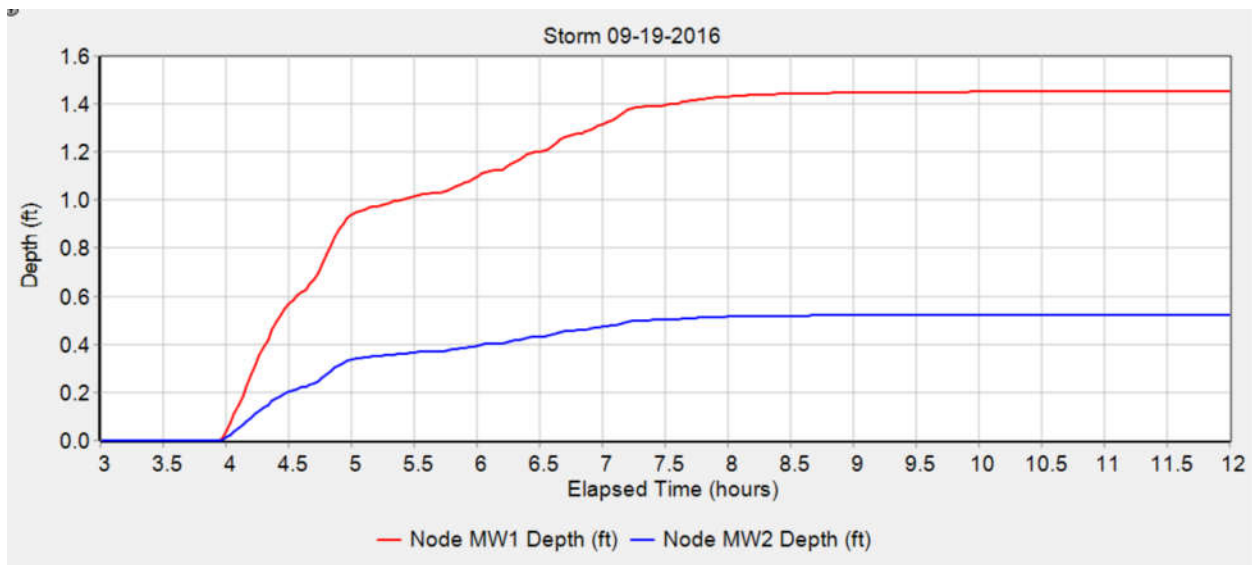
Figure 22 – Modeled depth of water in units MW1 and MW2 for Storm 09-19-2016, assuming no infiltration

4.3 System in Parking Lot E

4.3.1 Modeling the subcatchment

As mentioned in Chapter 2, the watershed in Lot E has 23,086 square feet (0.53 acres) of area, a slope of 2%, a time of concentration of 9 minutes and a Curve Number of 98.

The watershed was modeled in SWMM as subcatchment *Wc*, and its parameters were added in the SWMM menu for sub catchment editing. The same procedure from Appendix section A.1.2 was followed in SWMM in order to create and edit a subcatchment in the file.

As previously mentioned for Lot A, some challenges were faced in this part of the modeling since some of the parameters such as the %Zero and the Percent Routed were estimated. According to the SWMM manual, the former parameter is described as the impervious area in the watershed with no depression storage, "which is the maximum surface storage provided by ponding, surface wetting, and interception" (SWMM Manual, 2015). The latter is described as the percent of runoff routed between subareas (pervious and impervious) in the watershed. The value estimated for the Impervious area with zero storage was 0% and for the Percent routed it was 100%. System dimensions and modeling methods are presented in Table 14.

Parameter	Value	Source
Width	109.9 ft	Calculated on Chapter 2
Impervious n	0.016	Rough Asphalt (Chow, 1959)
Impervious Storage	0.1 in	SWMM suggested value
Subarea Routing	Outlet	Runoff from both areas flows to outlet
Percent Routed	100%	Estimated based on watershed
Infiltration Data	Curve Number Method	Calculated on Chapter 2

Table 14 - Summary of initial input parameters used to model Lot E watershed Wc
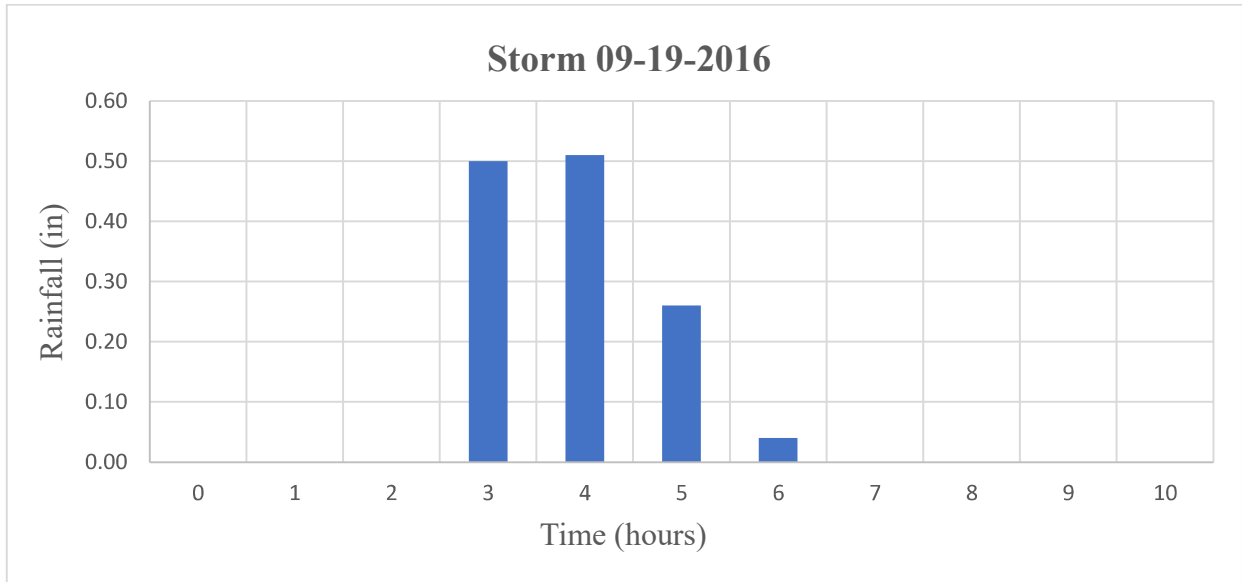
Figure 23 - Hyetograph of Storm 09-19-2016
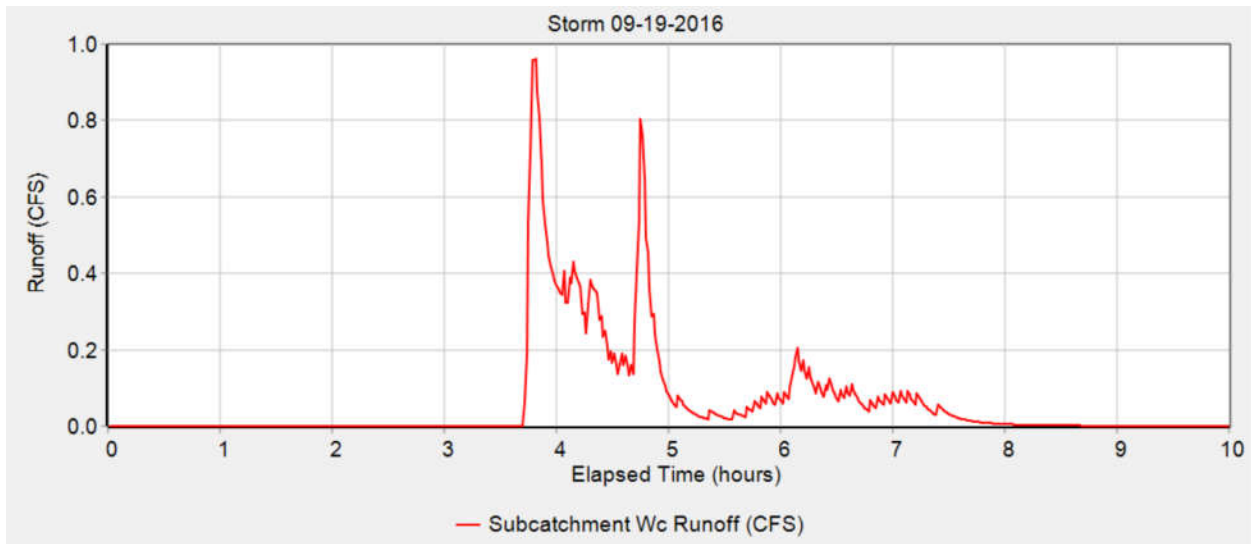
Figure 24 - Subcatchment Wc Runoff for Storm 09-19-2016

The chosen drying time was 1 day, using the same criteria as described in section 4.2.1.

After modeling the watershed, it was possible to run the model in order to generate a runoff hydrograph for each storm. Figure 24 shows the runoff hydrograph for subcatchment Wc for the storm of September 19th, 2016.

Runoff for the other storms was modeled and plotted in a similar fashion. The figures are located in Appendix section A.4.2.

4.3.2 Modeling the Lot E subsurface gravel filter system

This system was modeled as a storage unit following the same criteria discussed in section 4.2.2. It was also modeled with the LID control options described in Chapter 6.

The conceptual SWMM model of the subsurface gravel filter system and its watershed conceptual area may be seen in Figure 25. This figure serves as guidance to better understand the procedure to model each structure in the system.

As seen in Appendix section A.2.2, the design plans show that the system is divided into two components. The first component is the pretreatment concrete vault, termed in SWMM as *Inlet*, and this unit receives all runoff from watershed Wc. The second component is the subsurface gravel filter system, termed in SWMM as *Trench*, and it receives the outflow from the Inlet. The system dimensions for these two units are presented in Table 15.

Parameter	Inlet Value	Trench Value
Area	4 ft²	2250 ft²
Depth	5.5 ft	3 ft

Table 15 - Summary of parameters used to define storage units Pond1 and Pond2 in Lot E

For this project, Inlet and Trench were modeled following the same procedure of modeling storage units in SWMM, described in Appendix section A.1.4.
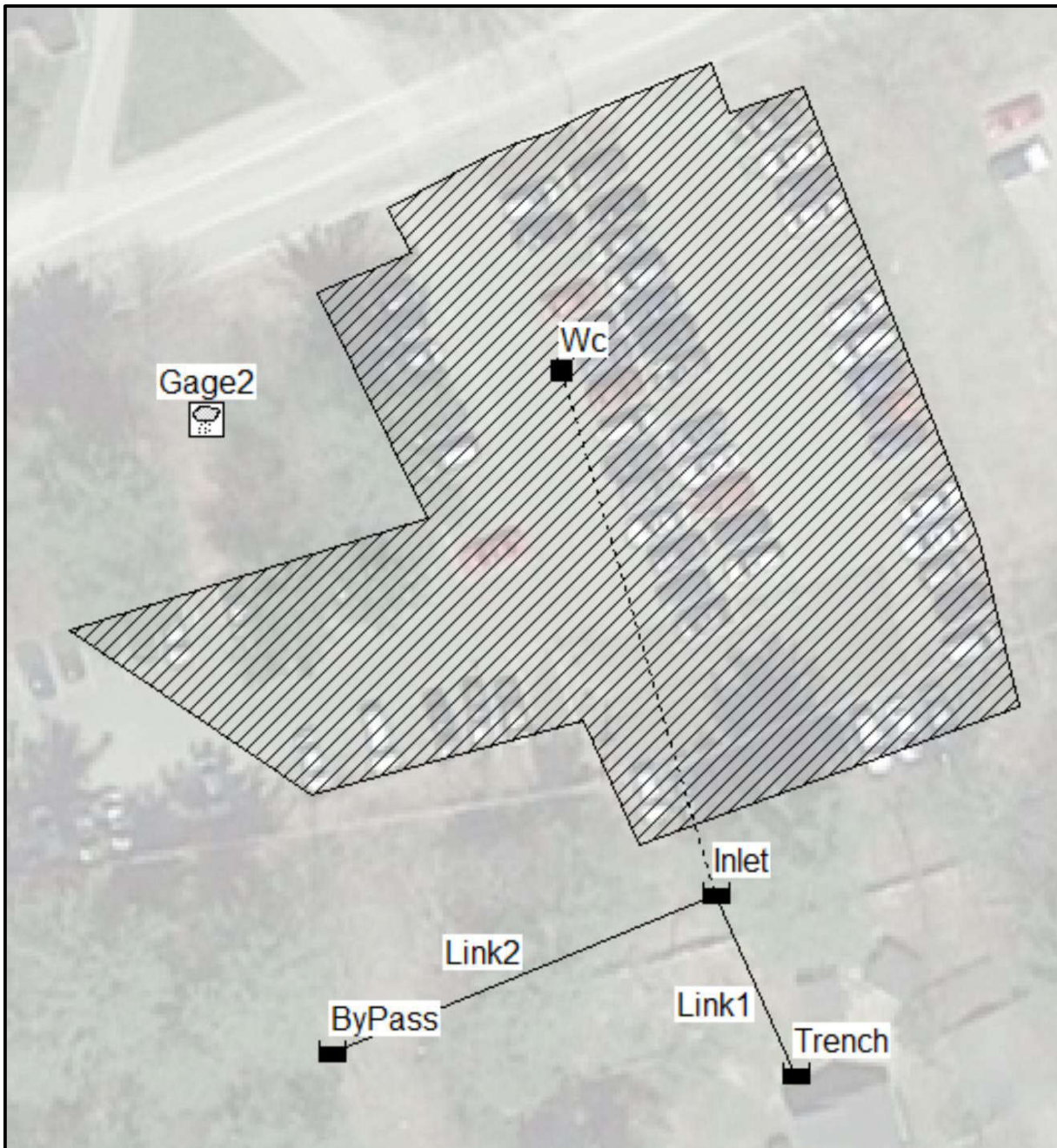
Figure 25 – Final Lot E system model

The units are connected by an HDPE perforated pipe. This connection was called ***Link1***

and was modeled as a closed circular conduit link. The procedure to create and edit a conduit link

was followed as described in Appendix section A.1.5. The dimensions used for the SWMM input

parameters were based on the design plans provided by the UNH Stormwater Center and are found in Table 16.

Parameter	Value	Source
Shape	Circular	Stormwater Center design plans[1]
Length	1 ft	[1]
Maximum Depth	0.67 ft	[1]
Roughness Coefficient	0.012	HDPE (ACPA, 2012)
Inlet Offset	2.5 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	0.5 ft	Elevation of pipe in outlet storage unit [1]

Table 16 - Summary of parameters used to define Link1 in Lot E

For this system, there is an overflow bypass solid pipe connecting the Inlet to the original system catch basin. The original system catch basin directs the water to a storm sewer. The original system catch basin was modeled in SWMM as an overflow storage unit. In the SWMM model, this was called the ***ByPass*** and it was modeled with an area of 25 ft^2 and a maximum depth of 5 ft. The pipe connecting Inlet and ByPass was modeled as ***Link2***. The system dimensions used as SWMM input parameters for Link2 were based on the design plans provided by the UNH Stormwater Center and found in Table 17.

This unit was modeled based on the design plans in Appendix Section A.2.1. Table 18 shows ByPass storage unit dimensions.

There is also an underdrain connecting the Trench to the Bypass catch basin. However, this structure was not modeled in this research since the underdrain is currently capped. This means that no flow can leave the system this way.

For this study, the elapsed time was selected for each storm separately varying from 1 to 2 days, while a time step of 1 minute was chosen for all storms. The elapsed time was chosen

depending on how long it takes for the system to reach a stable maximum depth of storage. For storms starting early in the day, an elapsed time of 1 day was enough. For storms starting later in the day, an elapsed time of 2 days was selected. The procedure used to achieve this setting is described in Appendix section A.1.6. The procedure used to plot the depth of storage was the same as described in Appendix section A.1.7. The depth of water of the storage units for the storm of September 19th, 2016 is shown in Figure 27.

Parameter	Value	Source
Shape	Circular	Stormwater Center design plans[1]
Length	6 ft	[1]
Maximum Depth	0.5 ft	[1]
Roughness Coefficient	0.012	HDPE (ACPA, 2012)
Inlet Offset	4.56 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	3.6 ft	Elevation of pipe in outlet storage unit [1]

Table 17 - Summary of parameters used to define Link2 in Lot E

ByPass	
Parameter	Value
Area	25 ft²
Depth	5 ft

Table 18 - Summary of ByPass catch basins dimensions in Lot E

An arbitrary initial depth of 2 ft in the Inlet was chosen to simulate this result in the preliminary run. This initial depth does not relate to the real conditions in the field and was chosen only for the purpose of demonstration of this procedure. This initial depth was chosen to create an overflow to the subsurface gravel system. If the Inlet was initially empty, the storm event would be insufficient to overflow to the Trench.
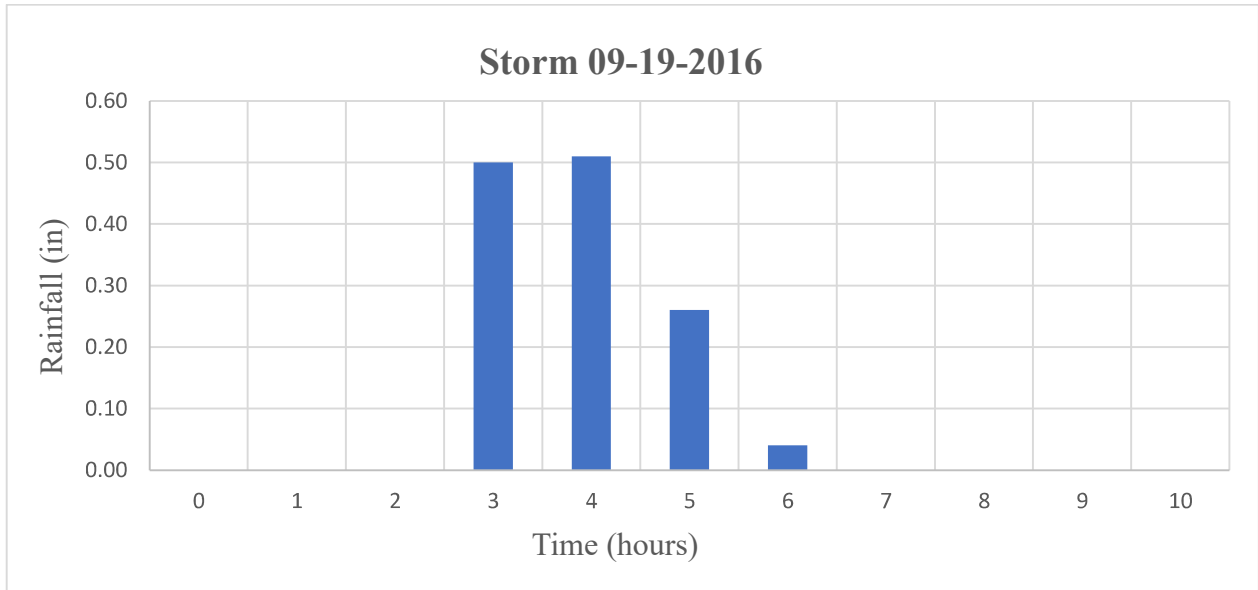
Figure 26 - Hyetograph of Storm 09-19-2016
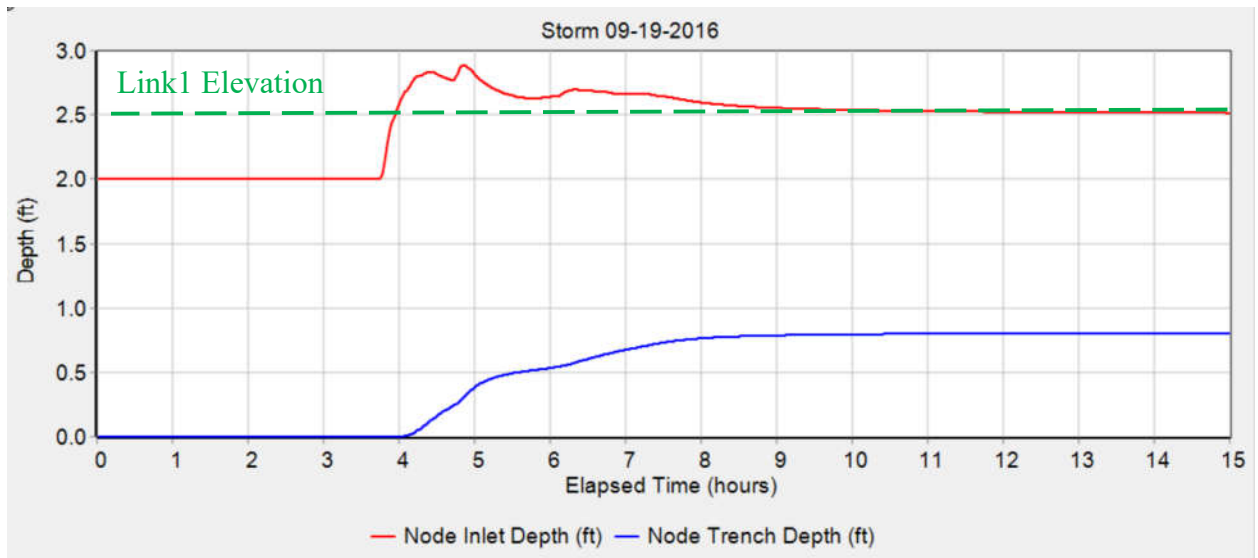
Figure 27 – Modeled depth of storage units Inlet and Trench for Storm 09-19-2016, assuming no

infiltration

Figure 27 shows that for this storm, the water level in Inlet reached Link1 and overflowed

to Trench. After the water level reaches Link2 elevation, it should flow to the ByPass basin. Since

this model is not considering infiltration in the original run, the water level stays constant after the event.

4.4 How SWMM models infiltration

SWMM models infiltration using different methods: Horton, Green-Ampt and Curve Number. For this study, the selected method was the Green-Ampt method. The Horton method considers the basic behavior of infiltration, but the physical interpretation of the results is uncertain. The Green-Ampt method presents an approach that is based on fundamental physics and the results match empirical observations (Green and Ampt, 1911).

The Green-Ampt method was selected to dictate infiltration in this study. The Green-Ampt method considers water being infiltrated by seepage loss. SWMM only considers the bottom area of the system in its seepage loss calculations. This means that water in the model is only being infiltrated to the soil at the bottom of the infiltration trench, although in the real system it infiltrates from the sides of the trench as well. This is an important factor to consider in the simulations as the infiltration rate can be underestimated in the model. Equation 4.2 presents how the Green-Ampt method calculates infiltration in the soil.

$$F_p = |\Psi_f| \, K_s \, (\theta_s - \theta_i) / \, (P - K_s) \qquad\qquad (Eq.\ 4.2)$$

Where:

F_p = *amount of water that infiltrates before water begins to pond (in)*

Ψ_f = *matric pressure at the wetting front (in)*

K_s = saturated hydraulic conductivity (in/hr)

θ_s = saturated moisture content

θ_i = initial moisture content before infiltration began

P = rainfall intensity (in/hr)

For its seepage loss calculations, SWMM considers three major parameters: hydraulic conductivity (in/hr), suction head (in) and initial deficit. As mentioned in the SWMM manual, suction head is the average value of soil capillary suction along the wetting front, and the initial deficit is the difference between soil porosity and initial moisture content. The seepage loss editing menu is depicted in Figure 28.

Property	Value
Suction Head	4.33
Conductivity	0.06
Initial Deficit	0

Storage Seepage Editor

Properties of soil beneath storage unit:

Capillary suction head (inches or mm).

OK   Cancel   Help
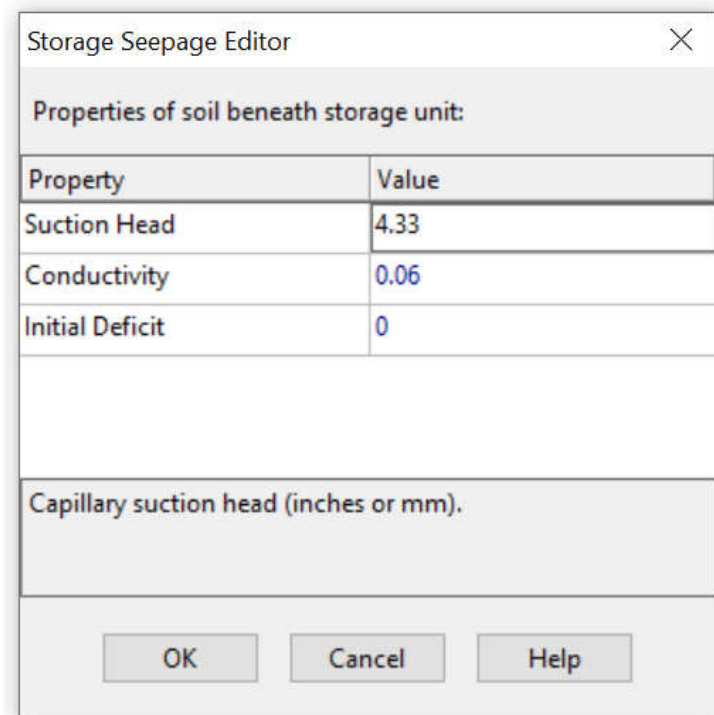
Figure 28 - Editing menu of seepage loss in a storage unit in SWMM

Table 19 shows suggested values for some soil characteristics in SWMM, based on soil type. For example, a type C soil ranges from Sandy Loam to Sandy Clay Loan, and a type D soil ranges from Sandy Clay Loam to Clay.

Table 3 shows suggested values for some soil characteristics in SWMM, based on soil type. For example, a type C soil ranges from Sandy Loam to Sandy Clay Loan, and a type D soil ranges from Sandy Clay Loam to Clay.

Soil Texture Class	K	Ψ	φ	FC	WP
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	0.43	4.33	0.453	0.190	0.085
Loam	0.13	3.50	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.60	0.475	0.378	0.265

K = hydraulic conductivity, in/hr
Ψ = suction head, in
φ = porosity, fraction
FC = field capacity, fraction
WP = wilting point, fraction

Table 19 – Soil characteristics for different soil types (Rawls et. al, 1983)

CHAPTER 5: DATA ANALYSIS AND RESULTS

5.1 Observed Data

5.1.1 Observed Volume in Lot A System

As previously mentioned in Chapter 3, the data collected at the field is absolute pressure. Then, these values are converted to depth of water in HOBOware by removing atmospheric pressure. With these values, it is possible to compute the system volume of water corresponding to this depth. As an example, Figure 29 shows the observed volume in unit MW1 for the September 19th, 2016 storm event after the correction for porosity.
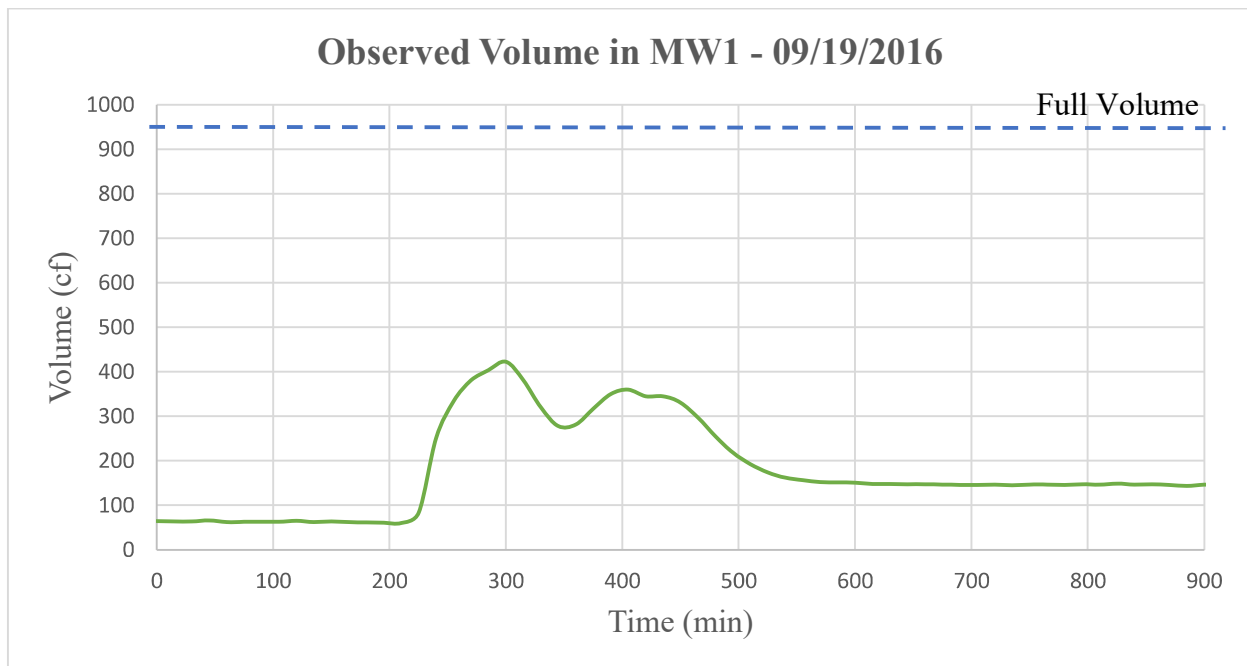
Figure 29 – Observed volume of water in MW1 for the storm event of 09-19-2016

5.1.2 Observed Volume in Lot E System

Following the same procedure described in section 5.1.1, the system water volume is estimated from the HOBO water level data. As an example, Figure 30 shows the estimated volume in the Trench for the September 19th, 2016 storm event after the correction for porosity.
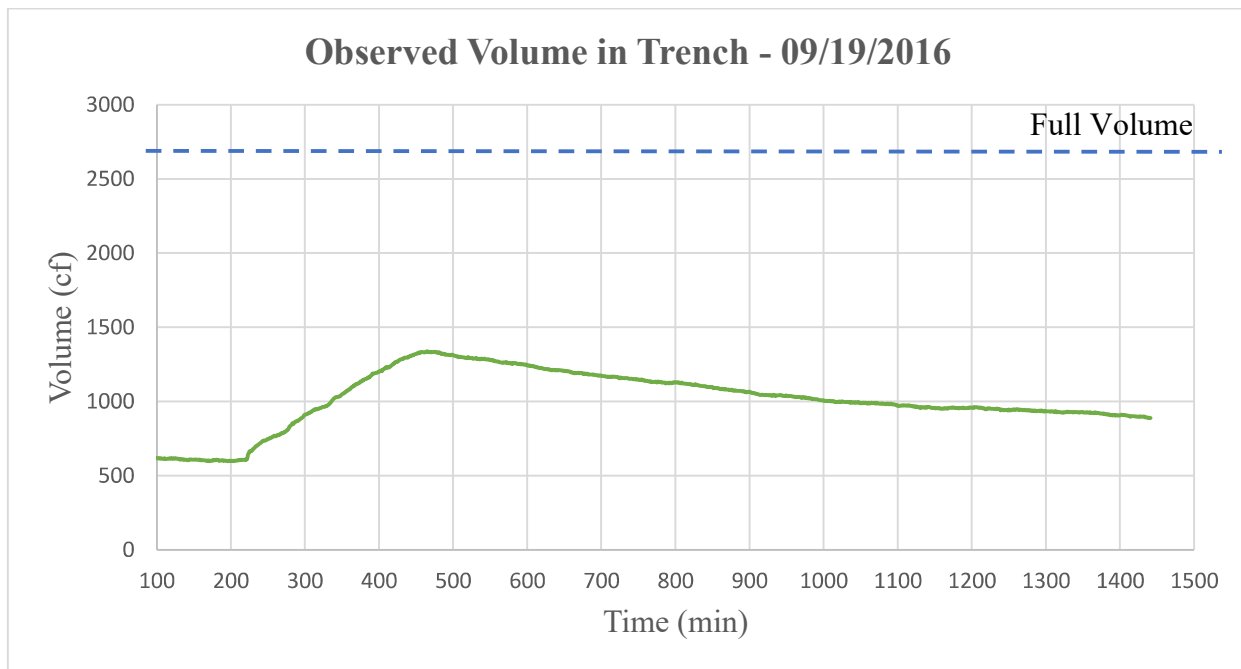
Figure 30 – Observed volume of water in Trench for the storm event of 09-19-2016

5.2 SWMM Model Sensitivity Analysis

A sensitivity analysis was conducted to determine which parameters most affect the modeled system water volume. The procedure for this sensitivity analysis consisted of varying the values of model parameters that relate to the infiltration rate in the system. As previously mentioned in Chapter 4, SWMM models infiltration based on seepage at the bottom of the storage unit. It does not consider horizontal seepage out of system walls. The most important parameters

for the calculation of the Green-Ampt seepage loss in SWMM are suction head (in), hydraulic conductivity (in/hr) and initial deficit.

The first parameter to be tested in this sensitivity analysis was the suction head. For the Lot A system, the range of values for the suction head in a C soil is from 4.33 in to 8.66 in. The result of running this suction head range was that varying the suction head from 4.33 in to 8.66 in did not appreciably affect the volume of water in the system or seepage volume, as seen in Figure 31. Therefore, a symbolic value of 4.33 in was kept for the rest of the analysis and calibration processes.

Modeled Volume of Water in MW1 - 09/19/2016

- - - Suction Head = 8.66 in   ── Suction Head = 4.33 in
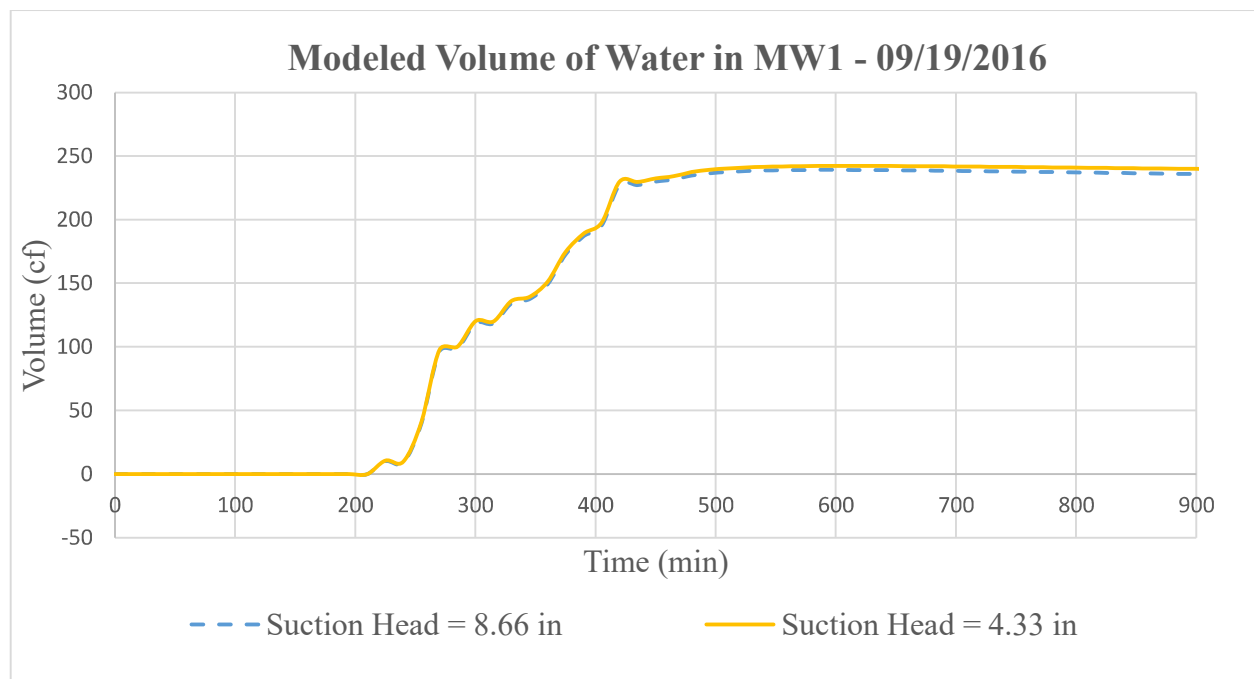
Figure 31 – Modeled volume of water in MW1 during sensitivity analysis modifying suction head for the storm event of 09-19-2016

The next parameter to be tested was the hydraulic conductivity of the soil below each system. For the Lot A system, the range of values for the hydraulic conductivity in a C soil is from

0.06 in/hr to 0.57 in/hr. This hydraulic conductivity range drastically affected the system water volume and seepage, as seen in Figure 32. Therefore, it was concluded that the hydraulic conductivity is a very important parameter to consider when modeling an infiltration system in SWMM.
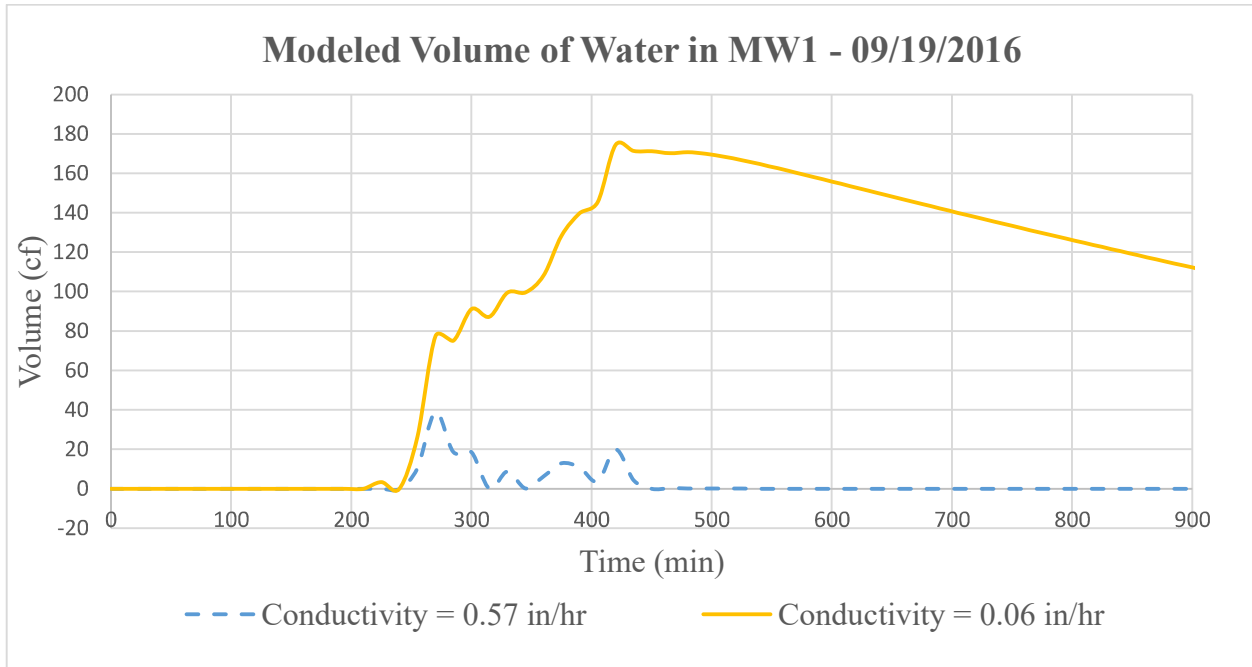
Figure 32 – Modeled volume of water in MW1 during sensitivity analysis modifying hydraulic conductivity for the storm event of 09-19-2016

The last sensitivity analysis parameter to be tested was the initial soil moisture deficit (soil moisture less than saturation). For the Lot A system, the range of values for the initial deficit in a C soil is from 0.19 to 0.244. Varying the initial deficit from 0.19 to 0.244 did not have much effect on the modeled system volume, as seen in Figure 33. Therefore, a symbolic value of 0.19 was kept for the rest of the analysis and calibration processes.
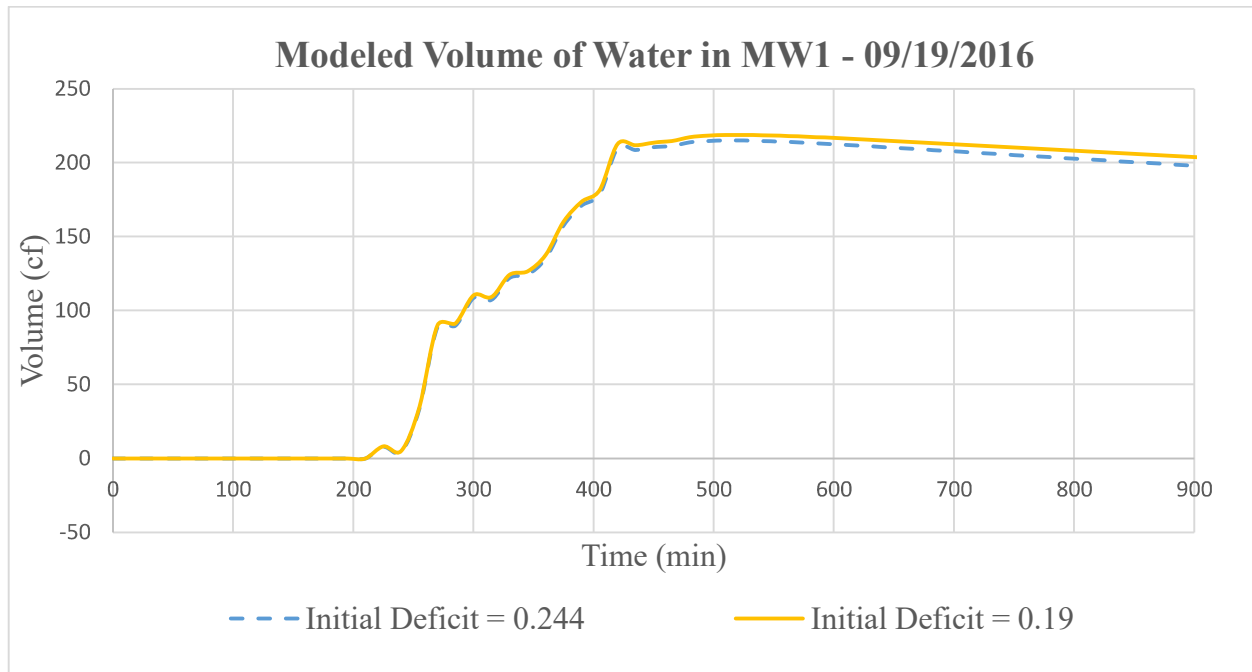
Figure 33 – Modeled volume of water in MW1 during sensitivity analysis modifying initial

deficit for the storm event of 09-19-2016

For the system in Lot E, the same process for sensitivity analysis was performed. The range

of values for the suction head in a D soil is from 8.66 in to 12.6 in. The range of values for the

hydraulic conductivity in a D soil is from 0 in/hr to 0.06 in/hr. Lastly, the range of values for the

initial deficit in a D soil is from 0.244 to 0.378.

The same conclusions were obtained from the sensitivity analysis for the system in Lot E.

Symbolic values of 8.66 in for the suction head and 0.244 for the initial deficit were kept for the

rest of the analyses and calibration processes. The only infiltration parameter that significantly

affects the system water volume is the hydraulic conductivity.

Other parameter to consider in the calibration were the diameter of the pipe connecting

Pond2 to MW1 (for the Lot A model) and the diameter of the pipe connecting the Inlet to the

Trench (for the Lot E model). Although the design plans specify pipe diameter, SWMM does not

model perforated pipes, and the calibration of the diameter of the pipes is a way to simulate the orifices in the perforated pipe as discussed in Chapter 4.

The last parameter considered in the calibration is the seepage area. SWMM models infiltration only at the bottom area of the system, it makes sense that if this area is increased it could better represent the observed infiltration.

As seen in the design plans, the area of the bottom of the system in Lot A is 612 ft^2 (for MW1 only) and in Lot E the area is 2250 ft^2. Table 20 shows the summary of the areas of the bottom and sides of each system.

Location	Area (ft^2)	
	Lot A	Lot E
Bottom	612	2250
Side	402.8	420
Total	1014.8	2670

Table 20 - Summary of bottom and side areas for the systems in Lot A and Lot E

If a larger area (adding bottom and sides) is used as the bottom area of the modeled storage unit, it may better reflect observed infiltration (via change in water volume after runoff ceases). A study on infiltration rate in C soils showed that lateral infiltration rates vary with water depth (Ballestero et. al, 2016). Calibrating the area of the bottom of the system between the original design area and the total area would better reflect this phenomenon. However, the lateral infiltration rates will remain constant in the model, as it is not possible to vary lateral infiltration values in SWMM.

In summary, three major parameters were considered in this calibration: the diameter of the pipe connecting a storage inlet to the system (to reflect perforated/slotted pipes), the hydraulic

conductivity of the soil at the bottom of the system and the seepage area of the bottom of the system. These parameters were found to be the most important ones in this calibration, after the sensitivity analysis was conducted.

5.3 Calibrating the Lot A model

It was perceived during calibration that the modeled water level values for the MW2 well were not high enough to match the observed Philadelphia Tree Trench system elevation. This is probably due to the fact that the east side of the trench (MW2) has a much larger area than the west side (MW1), and the monitoring well MW2 is too far from the inlet. The selected storm events were not able to fill this side of the trench in a timely manner that would make it possible to compare observed and modeled data. Thus, despite the model running with both storage units connected to the inlet, only the data from the west side of the trench (MW1) was included in this analysis. As previously mentioned, each unit receives half of the generated runoff volume.
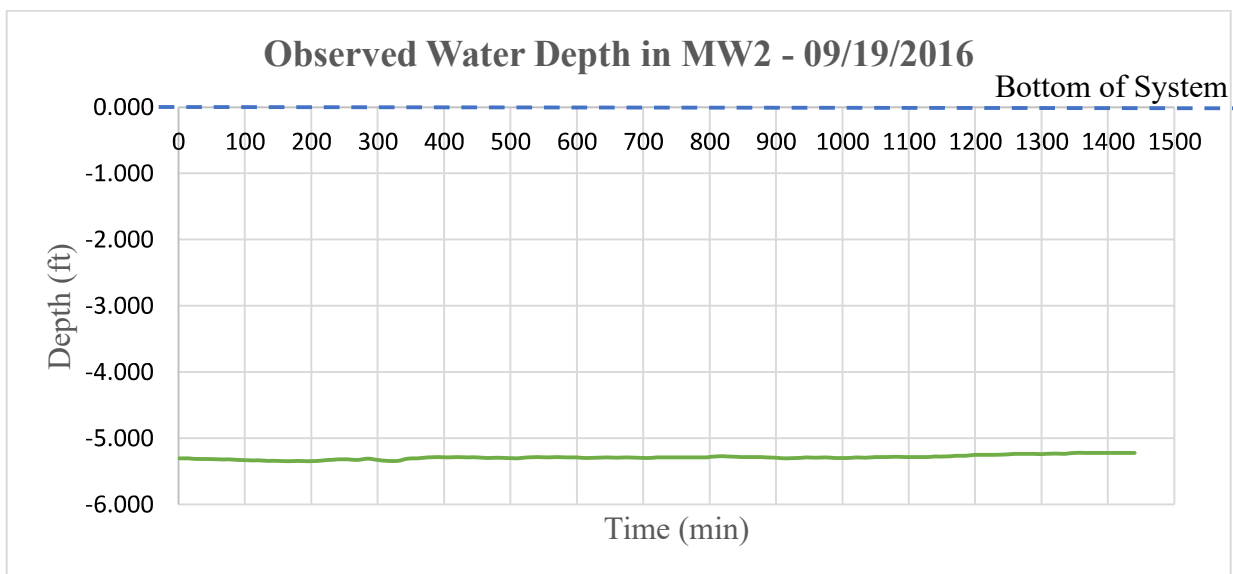
Figure 34 – Observed depth of water in MW2 for the storm event of 09-19-2016

Figure 34 shows the observed depth of water in MW2. It is possible to notice that the observed depth of water is below the elevation at the bottom of the system, represented by 0 ft in the graph.

The summary of the original and final values of the parameters modified in the calibration of the system at Lot A is shown in Table 21 and Table 22, respectively. A hydraulic conductivity of 0 in/hr was chosen to simulate no infiltration in the initial run of the model. Calibration was performed based on the procedure described in section 3.2. Parameter values were changed in each run, and the final result was obtained when the evaluation presented the most satisfactory results.

Values for Initial Run		
Pipe1 Diameter (ft)	**Conductivity (in/hr)**	**Bottom Area (sf)**
0.67	0	612

Table 21 – Initial values of the parameters modified in Lot A analysis

Date	Values after Calibration			
	Rainfall Depth (in)	**Pipe1 Diameter (ft)**	**Conductivity (in/hr)**	**Bottom Area (sf)**
09/19/2016	1.31	0.5	0.02	750
10/21/2016	3.42	0.35	0.02	700
10/30/2017	1.82	0.35	0.03	700
11/27/2018	1.22	0.45	0.015	750
01/01/2019	0.52	0.5	0.01	650
04/22/2019	0.57	0.5	0.01	650

Table 22 – Final values of the parameters modified in Lot A analysis

It is possible to see that the diameter of Pipe1 values have decreased in the calibration when compared to the original values for all storm events. The Hydraulic Conductivity values were found to be close in range for all storm events, but still lower than the expected value found in

literature for the type of soil present in the site. However, they are very similar to the ones measured on the field. Finally, the bottom areas were increased a little in the calibration, as expected.

Some correlations between the rainfall depth and the calibrated parameters in the model can be made for the storm events. The smaller storm events needed less reduction of pipe diameter, while the larger events needed more reduction. The smaller storm events presented a smaller hydraulic conductivity, while the larger events presented higher values, closer to the value measured on the field. The smaller storm events needed less increments of seepage area, while the larger events needed more increments.

5.3.1 Evaluation of the Lot A model

As mentioned in section 3.2, the calibration was made using visual and numerical criterion. The visual criteria may be observed in Figure 35 and Figure 36, where the volume of water in MW1 before calibration and the volume of water in MW1 after calibration are shown respectively for the storm event of September 19th, 2016. It is possible to see that the visual comparison of the observed volume of water in the field and the expected volume of water of the model differ significantly before calibration. After calibration, the volume of water for both observed and expected curves visually match better, but it still not enough to assume the model accurately predicts the volume of water in the system.

The calibration process stopped when it was perceived that if the parameters changed, the modeled volume of water would decay faster than the observed volume of water. This would make it impossible to match the observed volume of water and the obtained NSE and RMSE values would become more unsatisfactory; therefore the final values found in Table 22 for hydraulic

conductivity and bottom area are the optimal values obtained for the model, for each analyzed storm event.

Table 23 shows the original values of the NSE and RMSE calculations for the analysis of the Lot A model. It also shows the final NSE and RMSE values after calibration. It is possible to notice that for all storm events the NSE values increased with the calibration, and the RMSE values were reduced with the calibration. The changes in RMSE and NSE results after the calibration were as expected based on the procedure described in section 3.2.2.
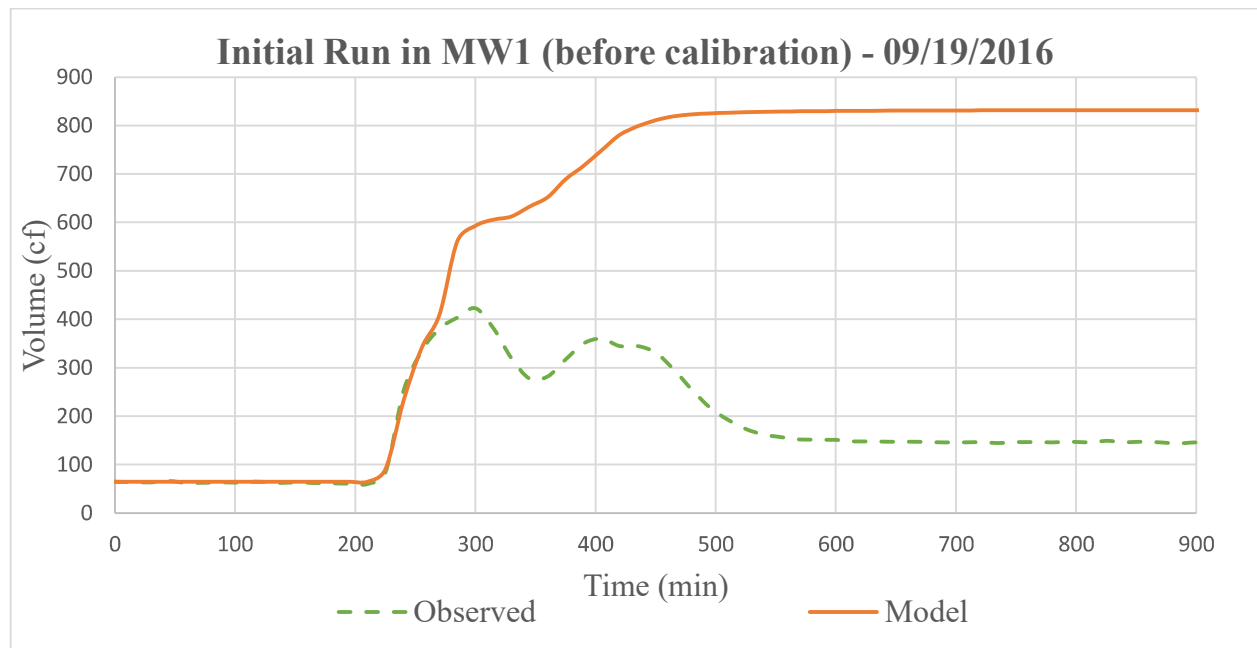

Figure 35 – Volume of water in Lot A MW1 with initial run during and after Storm 09-19-2016

The before and after calibration curves for the other storm events are shown in Appendix Section A.6.
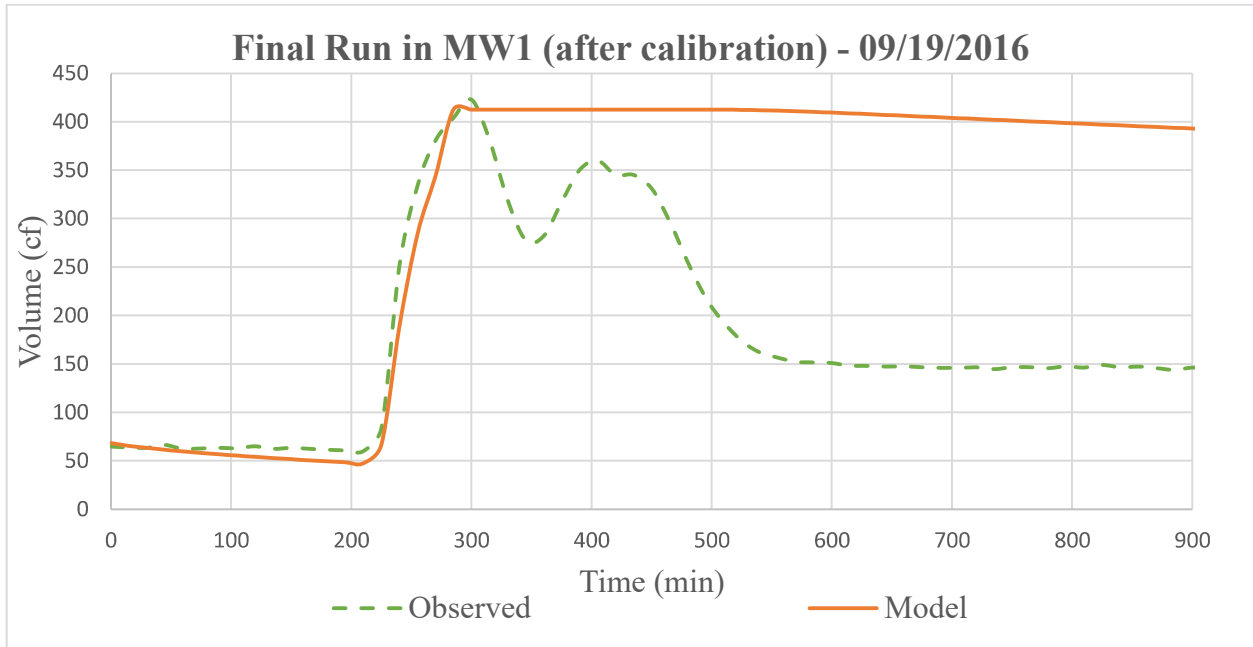
Figure 36 – Volume of water in Lot A MW1 after calibration during and after Storm 09-19-2016

		Initial Run		After Calibration	
Date	Rainfall Depth (in)	NSE	RMSE	NSE	RMSE
09/19/2016	1.31	-21.11	495.019	-4.48	199.338
10/21/2016	3.42	-72.10	955.169	-3.00	223.430
10/30/2017	1.82	-83.04	1226.806	-6.14	357.576
11/27/2018	1.22	-185.80	732.887	-2.64	102.354
01/01/2019	0.52	-10.24	213.118	-5.46	161.530
04/22/2019	0.57	-3.31	79.655	-0.40	45.378

Table 23 – Initial and Final values of NSE and RMSE for the Lot A analysis

Some correlations between the rainfall depth and the NSE and RMSE values in the model can be made for the small storm events. These events presented a smaller reduction in the final values of NSE and RMSE when compared to the values obtained in the initial run. No correlation can be made for the medium and large storm events.

5.3.2 Averaging the final parameters

The final step in the calibration process is to average the values of the parameters after calibration. It helps estimating how the parameters should be modeled in case the field data was not available in the first place. For example, during the process of system design. In this study, the NSE values are used as a weight for the calculation of a weighted mean since the models with the higher NSE values represent the ones with most accuracy to the field data. Equation 5.1 shows how the weighted mean was calculated.

$$W = \left. \sum_i^n V_i * NSE_i \middle/ \sum_i^n NSE_i \right.$$ (Eq. 5.1)

Where:

n = total number of storm events

i = number of the event being analyzed

V = value of the parameter after calibration (ft or in/hr)

NSE = value of the NSE for the model after calibration

For the Lot A model, the weighted average for the calibrated parameter is shown on Table 24.

Weighted Average - Lot A		
Pipe1 Diameter (ft)	**Conductivity (in/hr)**	**Bottom Area (sf)**
0.43	0.02	702.9

Table 24 – Weighted average of the calibrated parameters for the system in Lot A

This shows a reduction of pipe diameter by a factor of 0.64 when compared to design value. It shows an increase in the bottom area by a factor of 1.14 when compared to design value.

5.4 Calibrating the Lot E model

For this analysis, the initial water level in the Inlet was first considered, since there was a sensor monitoring the depth of water in this unit. Figure 37 shows the observed depth of water in the Inlet for the September 19th, 2016 storm event. This depth of water was obtained following the same procedure described in Section 3.1.3. When water exceeds the bypass (overflow) elevation in the inlet, it flows directly to the storm sewer and does not enter the system stone.
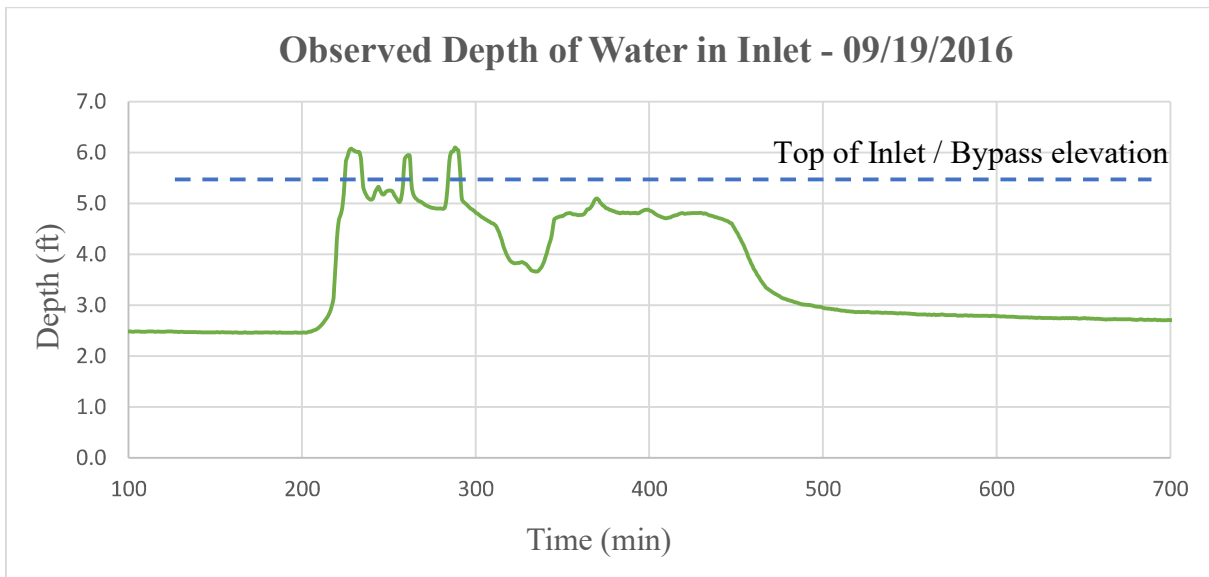

Figure 37 – Observed depth of water in Inlet for the storm event of 09-19-2016

Before the calibration starts, it is necessary to compare observed and modeled depth of water in the inlet to confirm if the model is representing correctly the volume of water entering this storage unit.

After this analysis, it was possible to notice in Figure 38 that the model is satisfactorily predicting the depth of water in the Inlet. Between 200 and 300 minutes it is possible to see that the depth of water is constant. This means that the depth of water reached the top of the inlet and is overflowing to the bypass.

All other storm events were analyzed, and their observed and modeled depth of water plots are found in Appendix Section A.6.7. Table 25 shows the summary of the initial depth of water in the Inlet for each analyzed storm event.

Date	Rainfall Depth (in)	Initial Depth of Storage (ft)
09/19/2016	1.31	2.469
10/21/2016	3.42	2.121
04/12/2017	0.18	2.599
05/22/2017	0.41	2.559
10/30/2017	1.82	2.839
04/16/2018	2.31	3.0

Table 25 – Initial depth of water in the Inlet for each storm event

This analysis is important to verify that there is a portion of runoff volume that is overflowing and not entering storage, and a portion of the runoff is actually flowing to the system. The modeled system is receiving a volume of water similar to the volume received in reality. The average portion of runoff volume entering the system and the average volume of water not entering the system (overflowing to the bypass or staying in the inlet) for each storm event is presented in Table 26. This information was obtained from the summary of results table that SWMM presents for each storm event.

Date	Total Modeled Runoff Volume (cf)	Average Volume entering system (cf)	Average Volume in Inlet (cf)	Average Volume in ByPass (cf)
09/19/2016	2327.9	387	10	1650
10/21/2016	6118.0	342	10	1671
04/12/2017	153.9	122	10	0
05/22/2017	480.9	133	10	0
10/30/2017	3232.2	699	11	2355
04/16/2018	3751.6	888	11	921

Table 26 – Average volume of runoff entering the system and not entering the system at Lot E
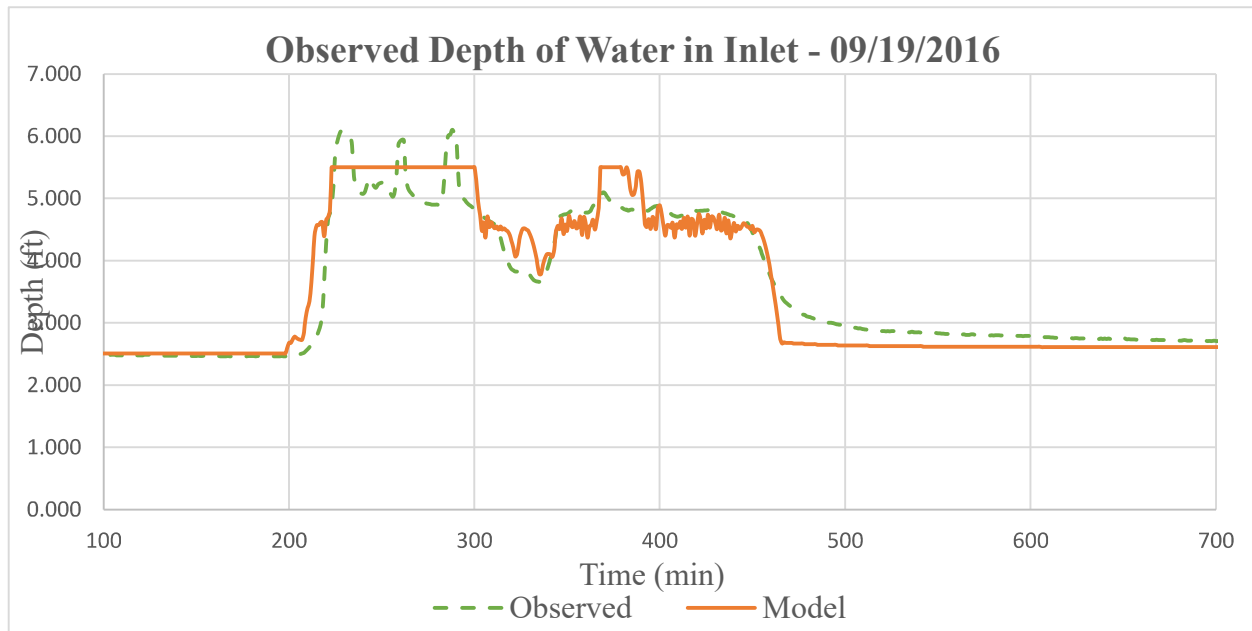

Figure 38 – Observed and Modeled Depth of Water in Inlet for the storm event of 09-19-2016

The summary of the original and final values of the parameters modified in the calibration of the Trench at Lot E is shown in Table 27 and Table 28, respectively. A hydraulic conductivity of 0 in/hr was chosen to simulate no infiltration in the initial run of the model. Calibration was performed based on the procedure described in section 3.2. Parameter values were changed in each run, and the final result was obtained when the evaluation presented the most satisfactory results.

Values for Initial Run		
Pipe1 Diameter (ft)	**Conductivity (in/hr)**	**Bottom Area (sf)**
0.67	0	2250

Table 27 – Initial values of the parameters modified in Lot E analysis

	Values After Calibration			
Date	**Rainfall Depth (in)**	**Pipe1 Diameter (ft)**	**Conductivity (in/hr)**	**Bottom Area (sf)**
09/19/2016	1.31	0.55	0.03	2400
10/21/2016	3.42	0.35	0.03	2400
04/12/2017	0.18	0.25	0.005	2300
05/22/2017	0.41	0.25	0.0055	2300
10/30/2017	1.82	0.375	0.035	2400
04/16/2018	2.31	0.25	0.03	2400

Table 28 – Final values of the parameters modified in Lot E analysis

It is possible to see that the diameter of Pipe1 values have decreased in the calibration when compared to the original values for all storm events. The Hydraulic Conductivity values were found to be very similar for all storm events except for the two smaller events. However, all values are still lower than the expected value found in literature for the type of soil present in the site. Finally, the bottom areas were increased a little in the calibration, as expected.

Some correlations between the rainfall depth and the calibrated parameters in the model can be made for the storm events. The smaller storm events needed more reduction of pipe diameter, the opposite of what happened for the system in Lot A. The smaller storm events presented a smaller hydraulic conductivity, while the larger events presented higher values. The smaller storm events needed less increments of seepage area, while the larger events needed more increments.

5.4.1 Evaluation of the Lot E model

As mentioned in section 3.2.1, the calibration was made using visual and numerical criteria. The visual criteria can be observed in Figure 39 and Figure 40, where the original volume of water in the Trench (before calibration) and the final volume of water in the Trench (after calibration) are shown for the storm event of September 19th, 2016. It is possible to see that the visual comparison of observed volume of water in the field and the expected volume of water of the model differ significantly before the calibration. After calibration, the volume of water for both observed and expected curves match better visually, but it still not enough to assume the model adequately predicts the volume of water in the system.

The calibration process stopped when it was perceived that if the parameters changed, the modeled volume of water would decay faster than the observed volume of water. This would make it impossible to match the observed volume of water and the obtained NSE and RMSE values would become more unsatisfactory; therefore the final values found in Table 28 for hydraulic conductivity and bottom area are the optimal values obtained for the model, for each analyzed storm event.

The before and after calibration curves for the other storm events are shown in Appendix Section A.6.
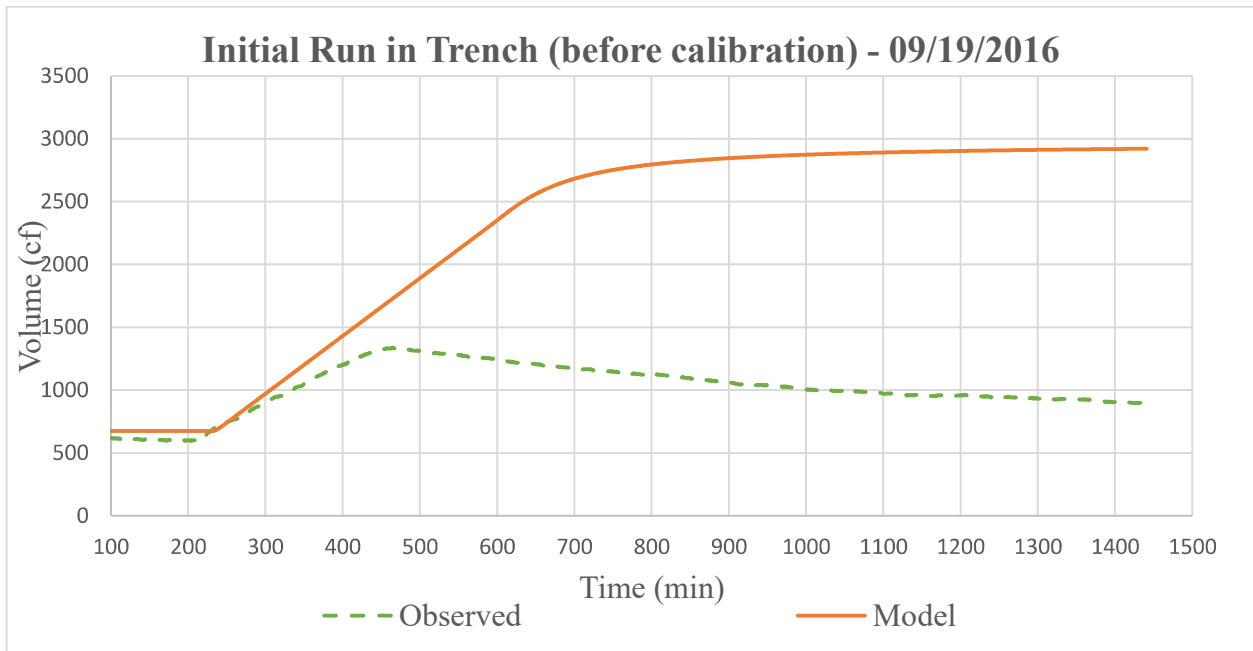
Figure 39 – Volume of water in Lot E Trench with initial run during and after Storm 09-19-2016
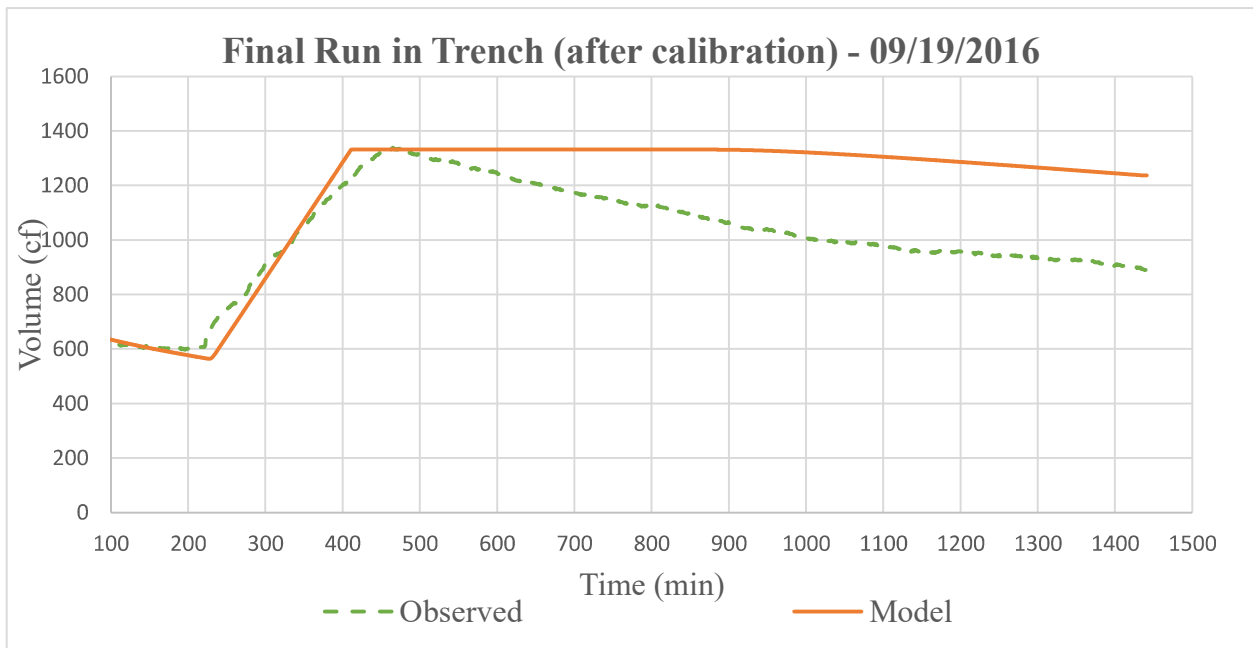

Figure 40 - Volume of water in Lot E Trench after calibration during and after Storm 09-19-2016

Table 29 shows the original values of the NSE and RMSE calculations for the analysis of

the Lot A model. It also shows the final NSE and RMSE values after calibration. It is possible to

notice that for all storm events the NSE values increased with the calibration, and the RMSE values were reduced with calibration. The changes in RMSE and NSE results after the calibration were as expected based on the procedure described in section 3.2.2.

Date	Rainfall Depth (in)	Original Analysis		After Calibration	
		NSE	RMSE	NSE	RMSE
09/19/2016	1.31	-43.88	1405.125	0.69	144.499
10/21/2016	3.42	-66.96	1930.22	-0.77	307.989
04/12/2017	0.18	-9.01	207.210	-0.52	80.799
05/22/2017	0.41	-16.23	212.170	-2.56	94.657
10/30/2017	1.82	-120.64	2256.800	-1.43	319.248
04/16/2018	2.31	-60.93	6270.394	-1.31	484.132

Table 29 – Initial and final values of NSE and RMSE for the Lot E analysis

5.4.2 Averaging the final parameters

The final step in the calibration process is to average the values of the parameters after calibration. It helps estimating how the parameters should be modeled in case the field data was not available in the first place. In this study, the NSE values are used as a weight for the calculation of a weighted mean since the models with the higher NSE values represent the ones with most accuracy to the field data. Equation 4.1 shows how the weighted mean was calculated.

For the Lot E modeled system analysis, the weighted average for the calibrated parameter is shown on Table 30.

Weighted Average - Lot E		
Pipe1 Diameter (ft)	Conductivity (in/hr)	Bottom Area (sf)
0.26	0.02	2347

Table 30 – Weighted average of the calibrated parameters for the system in Lot E

This shows a reduction of pipe diameter by a factor of 0.39 when compared to design value.

It shows an increase in the bottom area by a factor of 1.04 when compared to design value.

CHAPTER 6: DISCUSSION OF SWMM MODELING OF GSI SYSTEMS

After calibrating the model for both systems and analyzing the results, it is possible to notice some difficulties in modeling infiltration systems in SWMM.

SWMM does not have a tool for modeling perforated pipes. This type of pipe had to be modeled as conduit links with a diameter as presented in the design plans for the original run. The reduced diameter of the pipe connecting the inlet to the storage unit in both systems shows that the model does not account for the volume of water entering the system as it happens in reality if modeled with the same dimensions found in the design plans. Reducing the diameter is a way to deceive the model into moving water between units as it would with the use of a perforated pipe.

It is important to notice that in order to model the infiltration systems as storage units it was necessary to simulate the volume of water inside the pores of the system, as discussed in section 3.1.3.

After the correction of storage volume, calibration of pipe diameter and hydraulic conductivity, the final infiltrated volumes of water obtained in the model were much lower than the ones observed in the field and still not satisfactory based on the evaluation criteria.

In this chapter, two methods are presented in an attempt to increase infiltrated volumes in the model: modeling the system as a LID control option or adding a fictitious underdrain to improve infiltration rate in the system. However, not necessarily they are recommended to be used in the design of infiltration systems in SWMM.

6.1 Modeling as a LID system

The LID control option provides more detail when modeling the stone and soil layers of a GSI system, although it does not improve modeling of system infiltration. It considers the layers of the infiltration system and their properties, as well as infiltration and porosity of the materials. In this case, the corrections in the water volume for media porosity is not necessary.

With the SWMM LID controls, the method to generate the water volume level would be via the analysis of the change in storage over time. However, this time series analysis was not possible since SWMM does not provide a time series report for storage volume when using LID controls. The only SWMM provided data for the LID controls are the initial and final storage volume in the system, and these values are not useful in a comparison of water level using 1- and 15-minutes time steps. However, it is possible to assess infiltrated volumes over a complete storm.

The LID models of the systems in Lot A and Lot E will be described below, as a guidance for future projects. Both systems can be modeled as Infiltration Trenches. The procedure performed to create a new LID control may be found in Appendix section A.1.8. The criteria used to decide the values for the infiltration trench parameters in the Lot A model are described in Table 31.

The summary of the initial and final storage volume, as well as total infiltrated volume for each storm event modeled as LID control for the system in Lot A is presented in Table 32. These values were retrieved from the SWMM report summary of the LID control results. Observed infiltrated volumes are presented in section 6.3.

These results are not useful for the analysis of the system in Lot A since it is known that the runoff is not 100% routed to MW1 only. In this case, the total infiltrated volume will be multiplied by a factor of 0.24, calculated using Equation 6.1.

Factor = Area of MW1 / Total area of Trench   *(Eq. 6.1)*

Substituting the area of MW1 and the area of the Philadelphia tree trench in the equation:

Factor = 612 sf / (17 ft * 150 ft) = 0.24

Parameter	Value	Source
Surface		
Berm Height	0	Stormwater Center design plans[1]
Vegetation Volume	0	[1]
Manning's n	0.03	Short grass normal (Chow, 1959)
Surface Slope	0.5%	[1]
Storage		
Thickness	46 in	[1]
Void Ratio	0.5	SWMM suggested values
Seepage Rate	0.02 in/hr	Value obtained in original calibration
Clogging factor	0	SWMM suggested values
Drain		
Flow Coefficient	0	Simulating no flow
Open/Closed Level	0	SWMM suggested values
Other		
Area	612 sf	[1]
Width	17 ft	[1]

Table 31 - Summary of LID Control parameters used to model the system in Parking Lot A

The criteria used to decide the values of the infiltration trench parameters in the Lot E model are described in Table 33. The summary of the initial and final storage volume, as well as total infiltrated volume for each storm event modeled as LID control for the system in Lot E is presented in Table 34. Calculation of observed infiltrated volumes is presented in section 6.3.

Storm	Rainfall Depth (in)	Initial Storage (cf)	Final Storage (cf)	Infiltrated Volume (cf)	*Observed Volume (cf)*
09/19/2016	1.31	0	222.768	45.39	*159.03*
10/21/2016	3.42	0	304.572	29.07	*175.42*
10/30/2017	1.82	0	297.432	48.45	*210.51*
11/27/2018	1.22	0	199.308	47.94	*198.12*
01/01/2019	0.52	0	62.22	47.94	*125.46*
04/22/2019	0.57	0	75.888	28.56	*251.65*

Table 32 - Summary of storage and infiltrated volume for the system in Parking Lot A

Parameter	Value	Source
Surface		
Berm Height	0	Stormwater Center design plans[1]
Vegetation Volume	0	[1]
Manning's n	0.016	Asphalt (Chow, 1959)
Surface Slope	2%	[1]
Storage		
Thickness	24 in	[1]
Void Ratio	0.5	SWMM suggested values
Seepage Rate	0.03 in/hr	Value obtained in original calibration
Clogging factor	0	SWMM suggested values
Drain		
Flow Coefficient	0	Simulating no flow
Open/Closed Level	0	SWMM suggested values
Other		
Area	2250 sf	[1]
Width	30 ft	[1]

Table 33 - Summary of LID Control parameters used to model the system in Parking Lot E

These results can be used to compare observed and modeled infiltrated volumes. However, for both systems, it is necessary to observe that the runoff originally flows to an inlet and then it flows to the infiltration trench. In both cases, the inlet overflows to a catch basin, meaning that not all runoff is routed to the trench during the storm event, as it is simulated in the LID control option.

Storm	Rainfall Depth (in)	Initial Storage (cf)	Final Storage (cf)	Infiltrated Volume (cf)	*Observed Volume (cf)*
09/19/2016	1.31	0	459.75	384.375	*1104.79*
10/21/2016	3.42	0	497.25	296.25	*1497.07*
04/12/2017	0.18	0	0	153.75	*972.65*
05/22/2017	0.41	0	67.5	313.125	*425.27*
10/30/2017	1.82	0	457.5	401.25	*234.89*
04/16/2018	2.31	0	492.75	335.625	*781.74*

Table 34 - Summary of storage and infiltrated volume for the system in Parking Lot E

6.2 Adding an Underdrain

Since SWMM is underpredicting infiltration, one more way to calibrate the systems is using a fictitious underdrain to help the water flow out of the systems. This is physically unrealistic, but it helps drain the system without relying on a high soil hydraulic conductivity value, either for vertical or horizontal seepage. This method was applied only to obtain a good match of observed and modeled values, but it is not necessarily recommended in the design process for new GSI systems.

As previously mentioned, SWMM does not account for lateral infiltration. The volume of water leaving the system through this fictitious underdrain would simulate the volume of water leaving the system through the walls of the system in reality. As mentioned in section 4.2.2,

SWMM models conduit links using Manning's equation (Equation 4.1). Manning's equation calculates the flow going through the fictitious underdrain. Infiltration rate in inches per second can be transformed into a flow when multiplied by the flow area. In this case, the flow area would be the sidewall area of the systems. After the calibration of the underdrain diameter, the flow of water going through the fictitious underdrain would be adequately representing the flow of water going through the sidewalls of the system. Therefore, the volume of water flowing through the fictitious underdrain would simulate the volume of water infiltrating through the sidewalls of the system, making it possible to match both observed and modeled water volumes in the system.

The fictitious drainage pipe connecting MW1 and MW2 to Catch was modeled as ***Pud1***, also based on the dimensions found in the design plans in Appendix Section A.2.1. Table 35 shows Pud1 link dimensions.

The fictitious drainage pipe connecting the Trench to the ByPass catch basin was modeled as ***Pud2***, also based on the dimensions found in the design plans in Appendix Section A.2.2. Table 36 shows Pud2 link dimensions.

Parameter	Value	Source
Shape	Circular	Stormwater Center design plans[1]
Maximum Depth	0.67 ft	[1]
Length	1 ft	[1]
Roughness Coefficient	0.02	PVC (ACPA, 2012)
Inlet Offset	0.24 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	0.3 ft	Elevation of pipe in outlet storage unit [1]

Table 35 - Summary of parameters used to define Pud1 in Lot A

Calibration of the Lot A model followed the procedure described in Chapter 5 but modifying only the diameter of the drainage pipe and the hydraulic conductivity. Area was not

considered in this calibration since the fictitious underdrain would be simulating lateral infiltration. Therefore, it is not necessary to vary bottom area. New NSE and RMSE values and comparison curves were obtained. Table 37 shows the new parameters and results found in this new calibration of the model at Lot A. Figure 41 shows the new curves obtained in this calibration.

The before and after calibration curves for the other storm events are shown in Appendix Section A.6.3.

Parameter	Value	Source
Shape	Circular	Stormwater Center design plans[1]
Maximum Depth	0.67 ft	[1]
Length	1 ft	[1]
Roughness Coefficient	0.012	HDPE (ACPA, 2012)
Inlet Offset	0.5 ft	Elevation of pipe in inlet storage unit [1]
Outlet Offset	2.5 ft	Elevation of pipe in outlet storage unit [1]

Table 36 - Summary of parameters used to define Pud2 in Lot E

Date	Rainfall Depth (in)	Calibration of Lot A model – Underdrain			
		Hydraulic Conductivity (in/hr)	Diameter of Pud1 (ft)	NSE	RMSE
09/19/2016	1.31	0.025	0.05	0.93	22.989
10/21/2016	3.42	0.025	0.06	0.93	29.003
10/30/2017	1.82	0.025	0.065	0.72	70.779
11/27/2018	1.22	0.02	0.05	0.74	27.369
01/01/2019	0.52	0.01	0.0425	0.70	34.561
04/22/2019	0.57	0.01	0.03	0.65	22.690

Table 37 – Final values for the new Lot A analysis with an underdrain

In this analysis, the NSE values have increased significantly from the previous values obtained after the first calibration. The RMSE values presented a drastic reduction from the

previous values obtained after the first calibration. The visual comparison shows that the modeled volume of water matches almost perfectly the observed volume of water. Overall, with the addition of the underdrain it was possible to obtain results very close to the ones expected in Section 3.2.2.
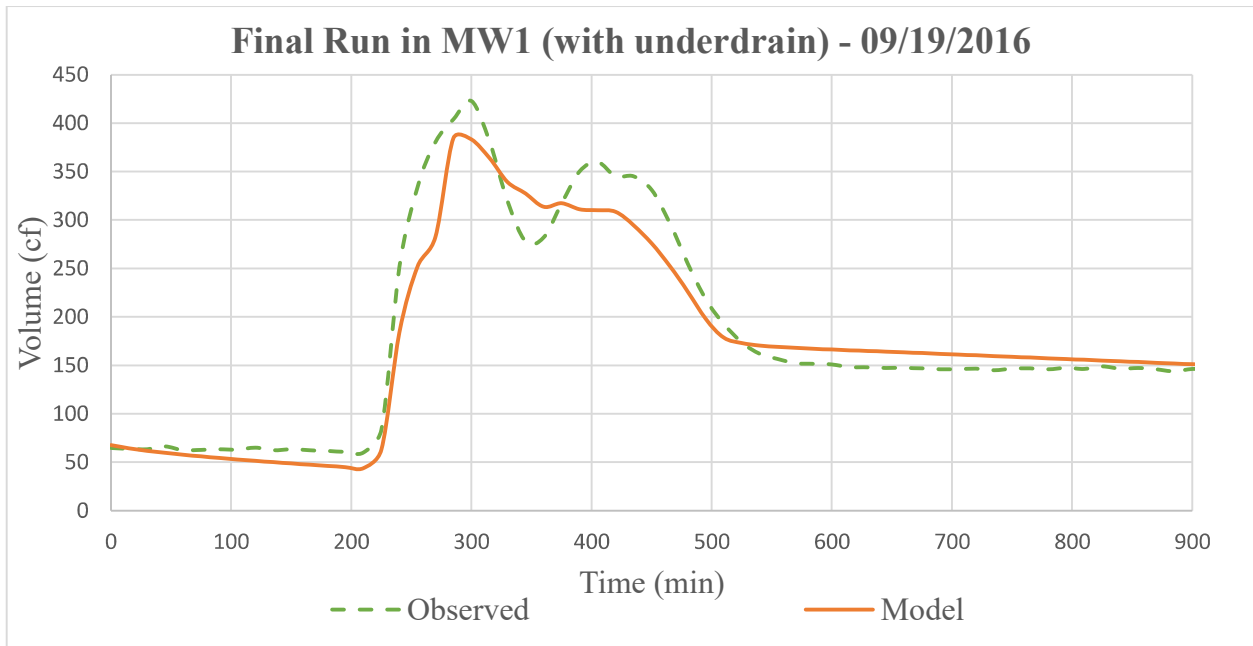
Figure 41 - Volume of Water in MW1 with an underdrain during and after Storm 09-19-2016

Weighted averages of the new parameters can be calculated using Equation 5.1. These results are presented in Table 38. The weighted value of hydraulic conductivity stayed the same when compared to the one obtained in the calibration.

New weighted Average - Lot A	
Pud1 Diameter	Conductivity
0.05 ft	0.02 in/hr

Table 38 – Weighted average of the calibrated parameters for the system in Lot A

Calibration of the Lot E model followed the same procedure. Again, area was not considered as the fictitious underdrain would be simulating lateral infiltration. New NSE and RMSE values and comparison curves were obtained. Table 39 shows the new parameters and results found in this new calibration of the model at Lot E. Figure 42 shows the new curves obtained in this calibration. The before and after calibration curves for the other storm events are shown in Appendix Section A.6.6.

| Date | Rainfall Depth (in) | Calibration of Lot E model – Underdrain | | | |
		Hydraulic Conductivity (in/hr)	Diameter of Pud2 (ft)	NSE	RMSE
09/19/2016	1.31	0.03	0.05	0.90	81.20
10/21/2016	3.42	0.035	0.045	0.76	112.771
04/12/2017	0.18	0.005	0.045	0.20	58.55
05/22/2017	0.41	0.006	0.05	0.11	47.275
10/30/2017	1.82	0.04	0.065	0.52	142.372
04/16/2018	2.31	0.035	0.075	0.54	216.714

Table 39 – Final values for the new Lot E analysis with an underdrain

In this analysis, the NSE values have increased significantly from the previous values obtained after the first calibration. The RMSE values presented a drastic reduction from the previous values obtained after the first calibration. The visual comparison shows that the modeled volume of water matches almost perfectly the observed volume of water. Overall, with the addition of the underdrain was possible to obtain results very close to the ones expected in Section 3.2.2.

Weighted averages of the new parameters can be calculated using Equation 5.1. These results are presented in Table 40. The weighted value of hydraulic conductivity increased when compared to the one obtained in the calibration.
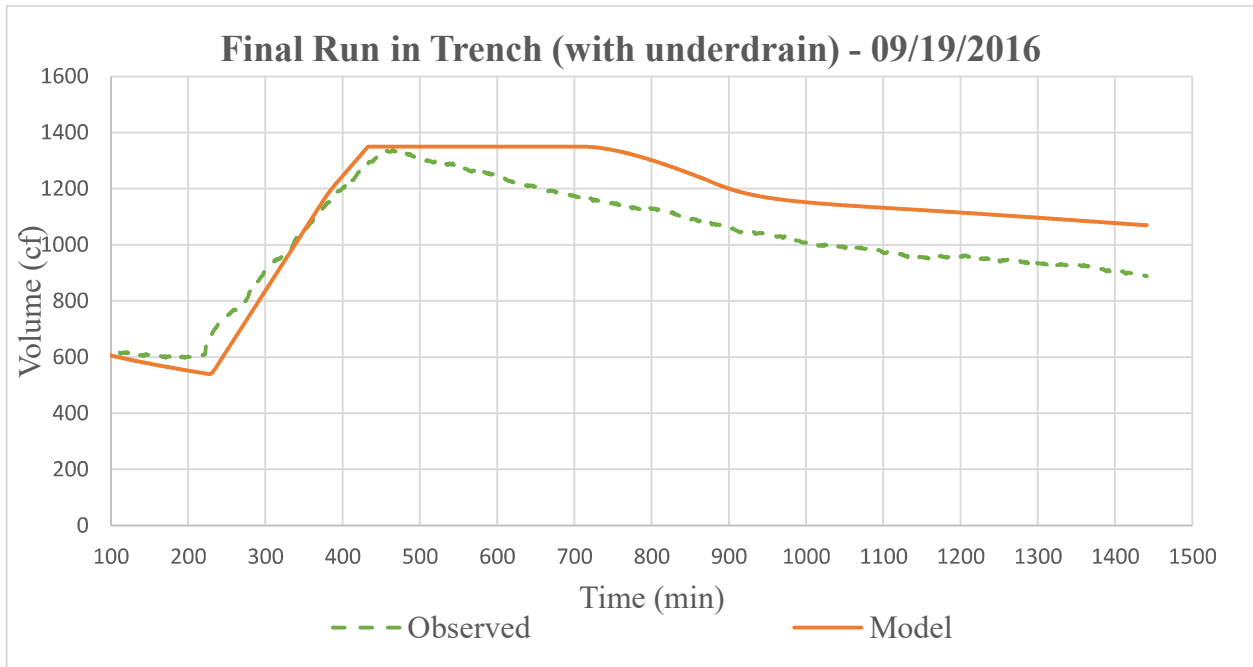
Figure 42 - Volume of Water in Trench with an underdrain during and after Storm 09-19-2016

New weighted Average - Lot E	
Pud2 Diameter	**Conductivity**
0.055 ft	0.03 in/hr

Table 40 – Weighted average of the calibrated parameters for the system in Lot E

6.3 Comparing infiltrated volumes

After all analyses are completed, it is possible to compare the infiltrated volume of water for the different analyses. This assesses whether the models adequately reflect the observed infiltrated volume. Observed infiltration rates were calculated using equations 6.2, 6.3 and 6.4 found in the study conducted by Ballestero et. al in 2016. This study concluded that the higher the water depth (driving head), the higher the infiltration rate. The conclusion being that the fill along

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the sidewalls possessed much higher infiltration capacity than the bottom of the system. These equations were used in the analysis of both systems.

Observed and modeled infiltration volumes were calculated using Equation 6.5 (Ballestero et. al, 2016). Then, the cumulative observed volume was calculated with the sum of the infiltrated volume for each time step.

if WD > 0.75, IR = 1.2648*WD - 0.8935   (Eq. 6.2)

if 0.75 > WD > 0.025, IR = 0.1637*WD + 0.0168   (Eq. 6.3)

if WD < 0.025, IR = 0.0684   (Eq. 6.4)

Where:

WD = Observed Depth of water in the system before correction for porosity (ft)

IR = Observed Infiltration rate (in/hr)

$$V_i = {}^{HC}/_{12} * A_b * {}^{Ts}/_{60}$$ (Eq. 6.5)

Where:

V_i = *Infiltrated Volume for a time step i (cf)*

HC = Hydraulic Conductivity (in/hr), for observed HC = IR

A_b = *Bottom area (sf)*

Ts = Time step (min), 15 min for Lot A and 1 min for Lot E

Table 41 shows the summary of the infiltrated volumes obtained in the analysis of the Lot A system. Table 42 shows the summary of the infiltrated volumes obtained in the analysis of the Lot E system.

As a measure of model performance, the ratio of the total modeled infiltrated volume and the total observed infiltrated volume was calculated.

Table 43 shows the comparison of observed vs modeled infiltrated volume for each method of calibration of the system in Lot A. Table 44 shows the comparison of observed vs modeled infiltrated volume for each method of calibration of the system in Lot E.

Date	Rainfall Depth (in)	Infiltrated Volume – Lot A			
		Cumulative Observed (cf)	Calibration (cf)	Underdrain Analysis (cf)	Using LID (cf)
9/19/2016	1.31	159.03	30.31	157.86	113.475
10/21/2016	3.42	175.42	28.29	181.95	72.675
10/30/2017	1.82	210.51	42.44	236.69	121.125
11/27/2018	1.22	198.12	22.73	213.03	119.85
1/1/2019	0.52	125.46	8.80	124.22	119.85
4/22/2019	0.57	251.65	26.14	260.82	71.4

Table 41 – Infiltrated volume of water for different analyses of the system in Lot A

Date	Rainfall Depth (in)	Infiltrated Volume – Lot E			
		Cumulative Observed (cf)	Calibration (cf)	Underdrain Analysis (cf)	Using LID (cf)
09/19/2016	1.31	1104.79	144.10	1026.57	960.94
10/21/2016	3.42	1497.07	135.09	651.93	740.63
04/12/2017	0.18	972.65	23.02	961.20	384.38
05/22/2017	0.41	425.27	25.32	456.32	782.81
10/30/2017	1.82	234.89	93.33	278.86	1003.13
04/16/2018	2.31	781.74	118.20	865.39	839.06

Table 42 – Infiltrated volume of water for different analyses of the system in Lot E

For Lot A, on average the calibrated SWMM model predicted 14% the observed infiltrated volume, on average. When the underdrain was added, the calibrated SWMM model predicted 104% of the observed infiltrated volume, on average. Finally, when using the LID control options, the SWMM model predicted 59% of the observed infiltrated volume, on average.

Date	Rainfall Depth (in)	Percent Predicted − Lot A		
		Observed vs Calibrated (%)	Observed vs Underdrain (%)	Observed vs LID (%)
9/19/2016	1.31	19.1	99.3	71.4
10/21/2016	3.42	16.1	103.7	41.4
10/30/2017	1.82	20.2	112.4	57.5
11/27/2018	1.22	11.5	107.5	60.5
1/1/2019	0.52	7.0	99.0	95.5
4/22/2019	0.57	10.4	103.6	28.4

Table 43 – Calculation of percent predicted of observed vs modeled volume of water in Lot A

Date	Rainfall Depth (in)	Percent Predicted − Lot E		
		Observed vs Calibrated (%)	Observed vs Underdrain (%)	Observed vs LID (%)
09/19/2016	1.31	13.0	92.9	87.0
10/21/2016	3.42	9.0	43.5	49.5
04/12/2017	0.18	2.4	98.8	39.5
05/22/2017	0.41	6.0	107.3	184.1
10/30/2017	1.82	39.7	118.7	427.1
04/16/2018	2.31	15.1	110.7	107.3

Table 44 – Calculation of percent predicted of observed vs modeled volume of water in Lot E

Similarly for Lot E, on average the calibrated SWMM model predicted 14% of the observed infiltrated volume. When the underdrain was added, the calibrated SWMM model predicted a volume of water 95% of the observed infiltrated volume, on average. Finally, when

using the LID control options, the SWMM model predicted 149% of the observed infiltrated volume, on average. It is noticed that for the LID modeling option, the last three storm events presented an overprediction of infiltrated volumes, while the first ones presented an underprediction. This may be due to the fact that this LID option does not consider the inlet overflow and bypass of runoff water. This option considers the full volume of runoff flowing to the system, and this does not happen in reality.

In summary, it is concluded that the original calibration process resulted with underpredicted infiltrated volumes, the calibration process performed with the underdrain predicted almost 100% of the observed infiltrated volumes and the LID model underpredicted the infiltrated volumes for the system in Lot A and overpredicted the infiltrated volumes for the system in Lot E.

Figure 43 shows the comparison of infiltrated volumes for the Lot A system using the different methods described previously. Figure 44 shows the comparison of infiltrated volumes for the Lot E system using the different methods described previously.
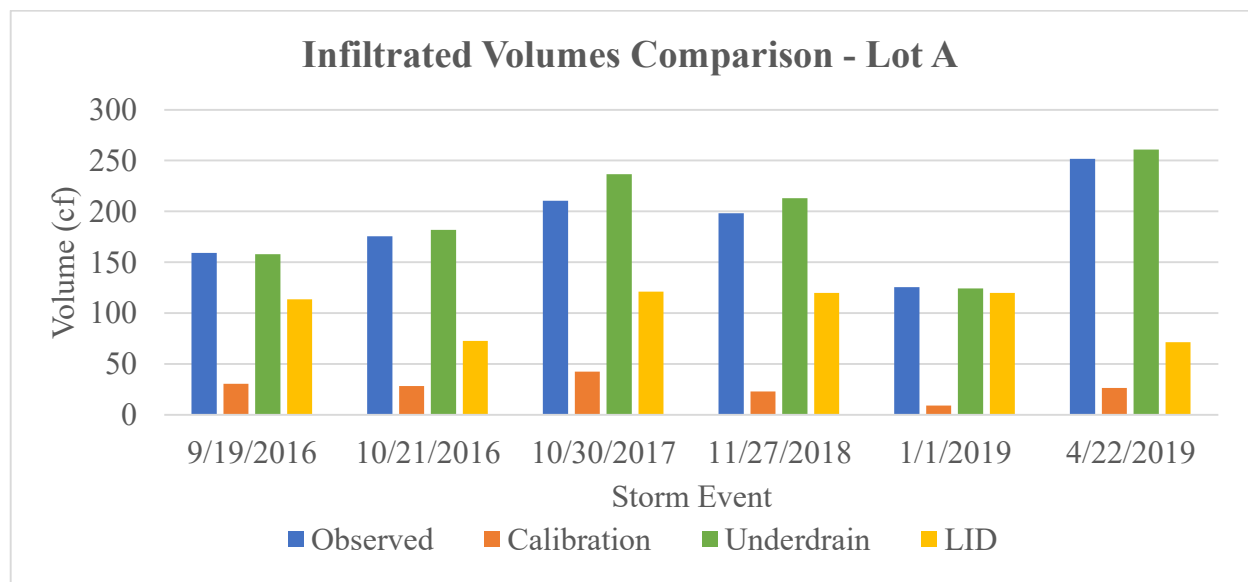
Figure 43 – Comparison of infiltrated volumes for each analyzed storm event in Lot A
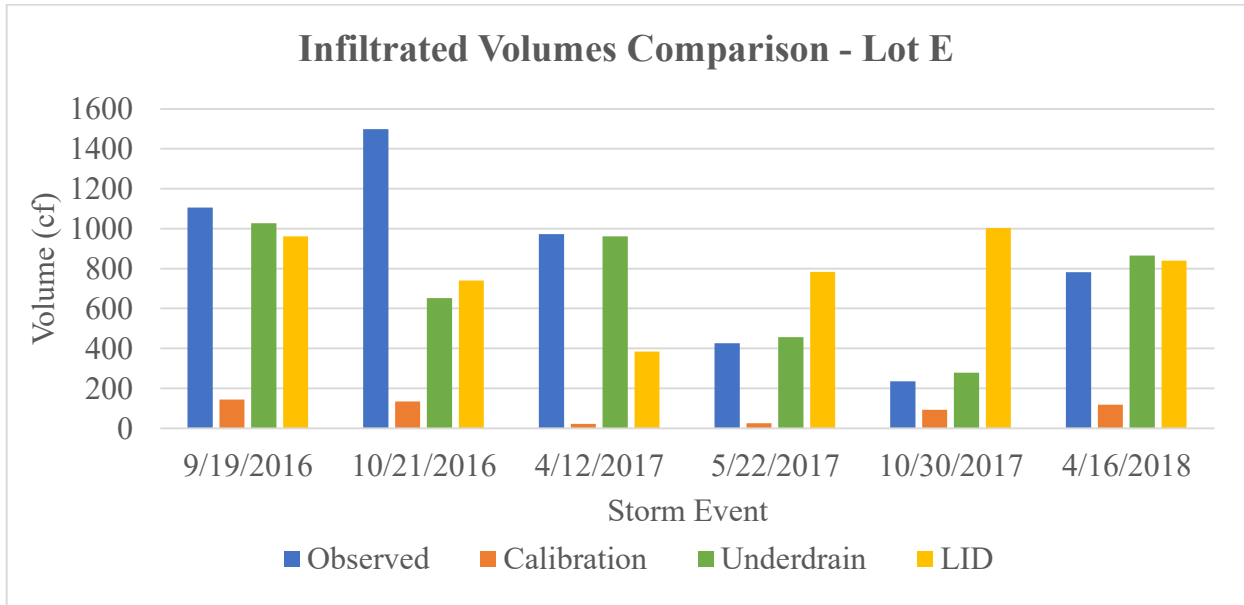
Figure 44 – Comparison of infiltrated volumes for each analyzed storm event in Lot E

Overall, the best modeling option was found to be the calibrated modeled with the addition of the fictitious underdrain, when it comes to comparison of observed and modeled infiltrated volumes. The infiltrated volumes of water obtained with this method were the most similar to the ones observed on the field. As mentioned in section 6.2, this fictitious underdrain would compensate for the lack of lateral infiltration in SWMM. The flow going through the underdrain, calculated using Manning's equation in SWMM, would represent the flow infiltrating through the sidewalls of the system based on the infiltrated rate of the surrounding soil and the flow area of the sidewalls.

However, this is not a recommended method to use since it is not feasible to implement during design phase. It is not possible nor realistic to estimate the diameter of the fictitious underdrain, or even its existence, during design phase.

CHAPTER 7: CONCLUSION

This research aimed to study the infiltration processes of two Best Management Practice (BMP) stormwater systems by modeling them in SWMM and comparing the volume of water data obtained in the models to the volume of water data observed in the field. These systems are a Philadelphia Tree Trench and an Infiltration Trench, located in Parking Lot A and Parking Lot E, respectively. The parking lots are located at the University of New Hampshire (UNH) campus in Durham, NH.

Both systems were built as retrofit projects by the UNH Stormwater Center, in 2014 and 2015. The project aimed to implement BMPs in these two parking lots. The systems were built in compliance to the criteria described in the Philadelphia Water Department (Lot A) and in the New Hampshire Stormwater Manual (Lot E).

The six storm events were modeled for each parking lot, based on the available data collected on the field. Every storm event had its precipitation (in) and runoff (cfs) plotted for each system. Both systems had their observed volume of water (cf) plotted. The final volume of water data generated by the SWMM model (expected value) should match the collected (observed value) data based on a visual comparison of plots, and on a numerical evaluation using the Nash Sutcliffe Efficiency (NSE) and the Mean Square Error (RMSE).

After a sensitivity analysis was performed, it was found that the hydraulic conductivity (in/hr) was the most sensitive parameter in the infiltration analysis, while the suction head (in) and the initial deficit were not very sensitive to modeled water volumes.

In the first calibration process, the systems were calibrated by varying the seepage loss parameters at the bottom of the storage unit and seepage area. This approach was used to calibrate the model to match the observed water volumes. After this calibration process, for both systems it was noticed that the final RMSE results were reduced and the NSE results increased when compared to the ones obtained in the original run of the model. Overall, the final results of the calibrated models were not satisfactory based on visual and statistical comparison. This concludes that the calibrated model does a better job at matching the observed water volumes than the initial model, but the model is still underpredicting the infiltrated volumes if compared to the observed infiltrated volumes. This is most likely due to the model's inability to model sidewall infiltration, as it would happen in reality for the studied systems.

The calibrated result for soil hydraulic conductivity in the Lot A system was 0.02 in/hr, which is lower than expected based on the soil type of the soil surrounding the system (at least 0.06 in/hr), but it is similar to the one measured on the field (0.03 in/hr). The final calibrated result for hydraulic conductivity in the system at Lot E was 0.02 in/hr, which is also lower than expected for the soil type in the lot (at least 0.06 in/hr), but it is within the range for this soil type. At first, it may seem obvious that increasing the infiltration rate in the model would increase the modeled infiltrated volumes. However, as mentioned previously, in the calibration process it was noticed that if the hydraulic conductivity was increased further than the final calibrated value, the volume of water in the system would decay significantly faster than the one observed in the field.

This concludes that SWMM underpredicts infiltration rates in the model if compared to the ones expected for each soil type, when calibrating the model by varying hydraulic conductivity and increasing seepage area. However, final calibrated infiltration rate is very similar to the one measured in the field and it is within the range of what is expected of soil types C and D. The

calibrated SWMM model predicted a total infiltrated volume 14% of the observed infiltrated volume on average, for both systems. This would explain the slow decay of the modeled water volume curve, as seen on the visual comparison of observed and modeled water levels for both systems. Without lateral infiltration, the water builds up in the system, instead of flowing out of the system through the sidewalls as it would happen in reality.

Two other methods of modeling in SWMM were performed in order to avoid underestimating the infiltration rate and infiltrated water volumes in the model. The first method is to model the systems as a LID control option. Both systems were modeled as infiltration trenches and the final infiltrated volumes were obtained for each storm event. With a hydraulic conductivity of 0.02 in/hr, the LID model for the system in Lot A predicted an infiltrated volume of water 59% of the observed volume of water, on average. With a hydraulic conductivity of 0.03 in/hr, the LID model for the system in Lot E predicted an infiltrated volume of water 149% of the observed volume of water, on average.

This concludes that this is not a reliable method to model infiltration trenches. This may be due to the fact that it does not consider real proportions of runoff volumes flowing to the system, as it is not possible to model the bypass (overflow) flow in the inlet. This method was not presented in a time series manner; therefore, it was not possible to calculate NSE and RMSE values, or to plot the curves for visual comparison. This method also does not consider lateral infiltration, but since the results were very different for each storm event, no correlation can be made based on the final infiltrated volumes and the prediction of the infiltration rates.

The last method of modeling in SWMM was the attempt to improve the calibrated infiltrated volumes by adding a fictitious underdrain in the systems. The final result for soil hydraulic conductivity in the Lot A system was all 0.02 in/hr, the same as the one obtained in the

original calibration. An underdrain diameter of 0.05 ft was obtained in the calibration. The total modeled infiltrated volume improved significantly, predicting an infiltrated volume of water 104% of the observed volume of water, on average. The final result of conductivity in the system at Lot E was 0.03 in/hr, which is higher than the one obtained in the original calibration. An underdrain diameter of 0.055 ft was obtained in the calibration. The total modeled infiltrated volume improved significantly, predicting an infiltrated volume of water 95% of the observed volume of water, on average.

After this last calibration process, for both systems it was noticed that the final RMSE results were drastically reduced and the NSE results increased significantly when compared to the ones obtained in the original run of the model, and also when compared to the ones obtained in the first calibration process. This may be explained by comparing the flow going through the fictitious underdrain (and out of the system) and the water infiltrating through the sidewalls of the system. The flow in the fictitious underdrain is calculated in SWMM using the Manning's equation. Lateral flow can be obtained multiplying the infiltration rate obtained for each system, by the sidewall area of each system. By adding this fictitious underdrain, the volume of water flowing out of the system through the underdrain would be simulating the volume of water flowing out of the systems through the sidewalls.

Overall, the results of the calibrated models using the underdrain were satisfactory based on visual comparison. This concludes that adding the underdrain helped the model obtain a very similar (to almost 100%) total infiltrated volume of water to the one observed in the field, making this a reliable method of calibration. However, this method of modeling GSI systems in SWMM is not recommended, as it is not feasible nor practical to guess the diameter of the underdrain during the design phase of a new system without having the observed volume data for comparison.

This study showed that the calibration processes for all applied methods were only possible due to the availability of observed data. It is not common to have this information during design phase. Common parameters to have during design phase are soil characteristics, such as infiltration rate, and site constraints. Using only these available parameters, the modeled infiltrated volumes were not satisfactory if compared to the ones observed in the real system. During the process of calibration, it was noticed that it is important to consider parameters other than soil characteristics when modeling GSI systems in SWMM, such as seepage area and underdrain pipe diameter. This proves that the hypothesis that it is possible to calibrate a model of a GSI system in SWMM by varying infiltration parameters is not correct.

The objectives of this study were reached, as it was possible to assess how the varying infiltration parameters would affect the volume of water in the model; different methods of modeling GSI systems were implemented and their results were analyzed; and, finally, some difficulties of modeling GSI systems in SWMM were noticed and discussed.

Although good results were obtained for some modeling methods, it is still important to consider that SWMM underestimates infiltration and that other parameters or modeling devices were needed to be included in the calibration process in order to obtain results that match observations. There is not a practical way to estimate these parameters during design phase, even if the factors and correlations obtained in this study were implemented.

Recommendations for future studies would be trying to perform this analysis on a system surrounded by different soil types (A or B) to verify if SWMM is a good tool to model infiltration in these types of soils; to include more storm events in the analysis in order to obtain a better correlation of total precipitation and infiltration parameters; to model pipes as orifices instead of conduit links to verify if this way it is necessary to reduce pipe diameter to simulate perforated

pipes; and to study more about how the underdrain flow reflects the lateral infiltration in the systems, to make possible to design reliable models of infiltration systems using only the parameters available in SWMM.

A recommendation for EPA would be to include lateral infiltration modeling in SWMM in order to reflect how GSI systems behave in reality and improve infiltrated volumes in the model. This would facilitate the analysis and design of infiltration systems that perform with lateral infiltration as well. Another recommendation would be to include an option to model inlets connecting to the subcatchment where the LID control option is in. This way, it is possible to obtain more realistic proportions of runoff volumes flowing to the subcatchment; therefore, more realistic infiltrated volumes would be obtained using this method of modeling infiltration systems.

Finally, some recommendations to modelers in the design phase that want to use SWMM to model infiltration systems can be made. The first one is to be careful when obtaining infiltrated volumes using the methods described in this study. Some methods underpredict infiltrated volumes and some overpredict infiltrated volumes, depending on the intensity of the storm event and the presence of bypass overflow units. Another recommendation would be to use the hydraulic conductivity value obtained in the field, as the calibrated hydraulic conductivity was very similar to the one measured on the field. However, when using hydraulic conductivities estimated for soil types as found in the literature, it is necessary to consider that SWMM under predicts infiltration rates around 33%, at least for soil types C and D. The final recommendation when using SWMM would be to model systems that consider vertical infiltration only, as SWMM does not account lateral (horizontal) infiltration in its analysis. This would allow SWMM to generate a more reliable modeled infiltrated volume of water.

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APPENDICES

A.1 Modeling procedures in SWMM

A.1.1 Modeling Storms

The first storm to be inserted and modeled in SWMM was on September 19, 2016. This storm was selected to the analysis in the systems for both parking lots, therefore it is going to be used to illustrate the process of modeling storms in SWMM.

The precipitation started at 3 am and stopped at 8 am, for a total of 5 hours. The total precipitation was 1.31 in; therefore it was considered a Medium storm. At every minute, a precipitation depth in inches was recorded and inserted in SWMM to create a precipitation Time Series. The following procedure was performed in SWMM in order to create and plot a new Time Series in the file:

- **Step 1:** Left Menu Project

- **Step 2:** Time Series

- **Step 3:** Green Plus Button

- **Step 4:** Insert Time (H:M)

- **Step 5:** Insert Value (in)

- **Step 6:** View

Figure 45 shows the time series editor for storms in SWMM.

Figure 45 - Creation of Time Series in SWMM for the September 19th, 2016 storm

The same procedure was followed to add and plot the remaining selected storms in SWMM as precipitation Time Series. The time series plot for all other storms is on the Appendix section A.3.

In order to simulate the storms, SWMM needs a set up rain gage that is connected to the subcatchment of interest. On this project, **Gage1** was created to link the Storms with the subcatchments. The following procedure was performed in SWMM in order to create and edit a Rain Gage in the file:

- **Step 1:** Left Menu Project

- **Step 2:** Hydrology

- **Step 3:** Rain Gages

- **Step 4:** Green Plus Button

- **Step 5:** Edit Rain Gage parameters

- **Step 6:** Connect to Time Series

The chosen time interval was 1 minute, since this was the interval used in the creation of each storm and the interval of the collected precipitation data.

Rain Gage Gage1	×
Property	**Value**
Name	Gage1
X-Coordinate	5952.298
Y-Coordinate	6478.046
Description	
Tag	
Rain Format	VOLUME
Time Interval	0:01
Snow Catch Factor	1.0
Data Source	TIMESERIES
TIME SERIES:	
- Series Name	Storm09-19-16
DATA FILE:	
- File Name	*
- Station ID	*
- Rain Units	IN
User-assigned name of rain gage	
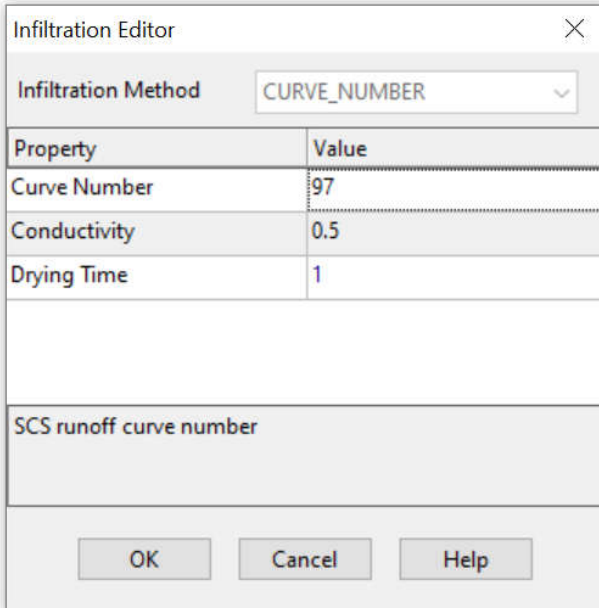
Figure 46 - Editing menu of Rain Gage Gage1

A.1.2 Modeling Subcatchments

The following procedure was performed in SWMM in order to create and edit a subcatchment in the file:

- **Step 1:** Left Menu Project

- **Step 2:** Hydrology

- **Step 3:** Sub catchments

- **Step 4:** Green Plus Button

- **Step 5:** Delineate watershed

- **Step 6:** Connect to Rain gage

- **Step 7:** Connect to Outlet

- **Step 8:** Edit parameters

Figure 47 shows the Curve Number menu, where the chosen drying time was 1 day.

Figure 47 - Editing menu of Infiltration data for the Curve Number method

Figure 48 shows the editing menu where the parameters were added for the watershed.

Property	Value
Subcatchment Wb	
Name	Wb
X-Coordinate	4246.904
Y-Coordinate	4331.520
Description	
Tag	
Rain Gage	Gage1
Outlet	Pond1
Area	0.58
Width	63.68
% Slope	2.3
% Imperv	95
N-Imperv	0.016
N-Perv	0.05
Dstore-Imperv	0.1
Dstore-Perv	0.25
%Zero-Imperv	0
Subarea Routing	OUTLET
Percent Routed	100
Infiltration Data	CURVE_NUMBER
Groundwater	NO
Snow Pack	
LID Controls	0
Land Uses	0
Initial Buildup	NONE
Curb Length	0

User-assigned name of subcatchment

Figure 48 - Editing menu of Sub catchment Wb

A.1.3 Plotting Runoff Hydrographs

The following procedure was performed in SWMM in order to run the model:

- **Step 1:** Top Menu
- **Step 2:** Project
- **Step 3:** Run Simulation

The following procedure was performed in SWMM in order to plot the Runoff generated by the September 19th, 2016 event storm in the subcatchment Wb:

- **Step 1:** Top Menu
- **Step 2:** Report
- **Step 3:** Graph
- **Step 4:** Time Series
- **Step 5:** Add
- **Step 6:** Sub catchment
- **Step 7:** Runoff
- **Step 8:** Accept

A.1.4 Modeling Storage Units

Before modeling a storage unit, it is necessary to create a storage curve to define the geometry of the system. For this project, Pond1 and Pond2 were modeled as rectangular prisms. The following procedure was performed in order to create and edit the storage curve in SWMM:

- **Step 1:** Left Menu Project
- **Step 2:** Curves
- **Step 3:** Storage Curves
- **Step 4:** Green Plus Button
- **Step 5:** Insert geometry details

Figure 49 shows the editing menu where the parameters were added for the storage unit curve.

All elevations of the model were selected referencing the design plans in Appendix section A.2.1. The following procedure was performed in order to create and edit the storage system in SWMM:

- **Step 1:** Left Menu Project
- **Step 2:** Hydraulics
- **Step 3:** Nodes
- **Step 4:** Storage Units
- **Step 5:** Edit Storage Unit parameters
- **Step 6:** Connect to a Storage Curve
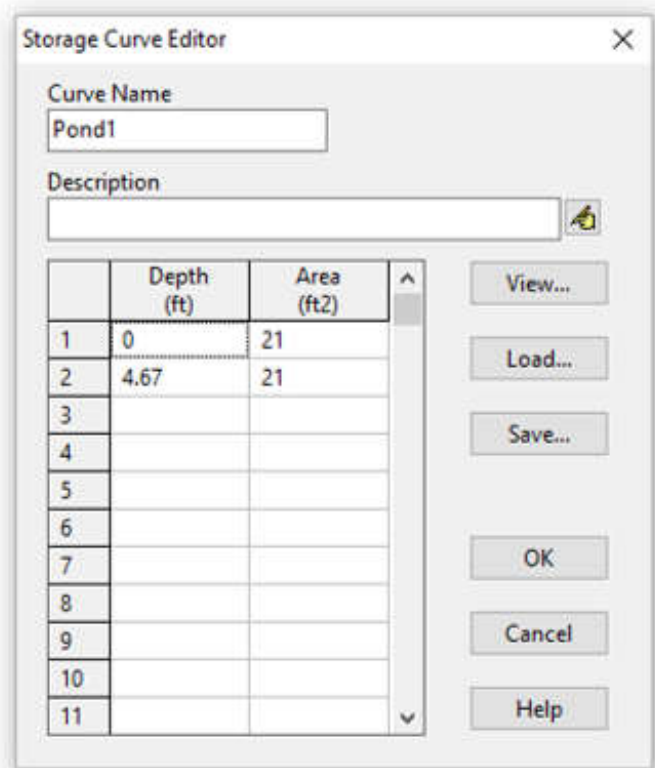
Figure 49 - Editing menu of a Storage Curve in SWMM

Figure 50 shows the editing menu where the parameters were added for the storage unit.

Surcharge depth is described in SWMM as the "depth in excess of maximum depth before

flooding occurs" (EPA, 2009). It is an optional property of a node element in SWMM. It will not

be used in this model since it is not simulating a pressurized condition in the system.

Figure 50 - Editing menu of a Storage Unit in SWMM

A.1.5 Modeling Links

The following procedure was performed in order to create and edit the conduit in SWMM:

- **Step 1:** Left Menu Project

- **Step 2:** Hydraulics

- **Step 3:** Links

- **Step 4:** Conduits

- **Step 5:** Connect storage unit to outfall

- **Step 6:** Edit parameters of conduit

Figure 51 shows the editing menu where the parameters were added for the conduit link.

Conduit Link1	x
Property	**Value**
Name	Link1
Inlet Node	Pond1
Outlet Node	Pond2
Description	
Tag	
Shape	RECT_CLOSED
Max. Depth	0.083
Length	0.5
Roughness	0.012
Inlet Offset	4.587
Outlet Offset	4.587
Initial Flow	0
Maximum Flow	0
Entry Loss Coeff.	0
Exit Loss Coeff.	0
Avg. Loss Coeff.	0
Seepage Loss Rate	0
Flap Gate	NO
Culvert Code	
User-assigned name of Conduit	

Figure 51 - Editing menu of the conduit link Link1 in SWMM

A.1.6 Choosing Analysis settings

The following procedure was performed to achieve the elapsed time setting:

- **Step 1:** Left Menu Project
- **Step 2:** Options
- **Step 3:** Dates > Start Analysis on > End Analysis on
- **Step 4:** Time Steps > Reporting Step

A.1.7 Plotting Depth of Storage

The following procedure was performed in SWMM in order to plot the depth of both storage units before, during and after the storm events:

- **Step 1:** Top Menu
- **Step 2:** Report
- **Step 3:** Graph
- **Step 4:** Time Series
- **Step 5:** Add
- **Step 6:** Node > Select storage units
- **Step 7:** Depth
- **Step 8:** Accept.

A.1.8 Modeling LID Control Systems

- **Step 1:** Left Menu Project

- **Step 2:** Hydrology

- **Step 3:** LID Controls

- **Step 4:** Green Plus Button

- **Step 5:** Edit Parameters

Figures 52 to 54 show the editing menu for every step of the creation of a infiltration trench in the LID Control option menu.
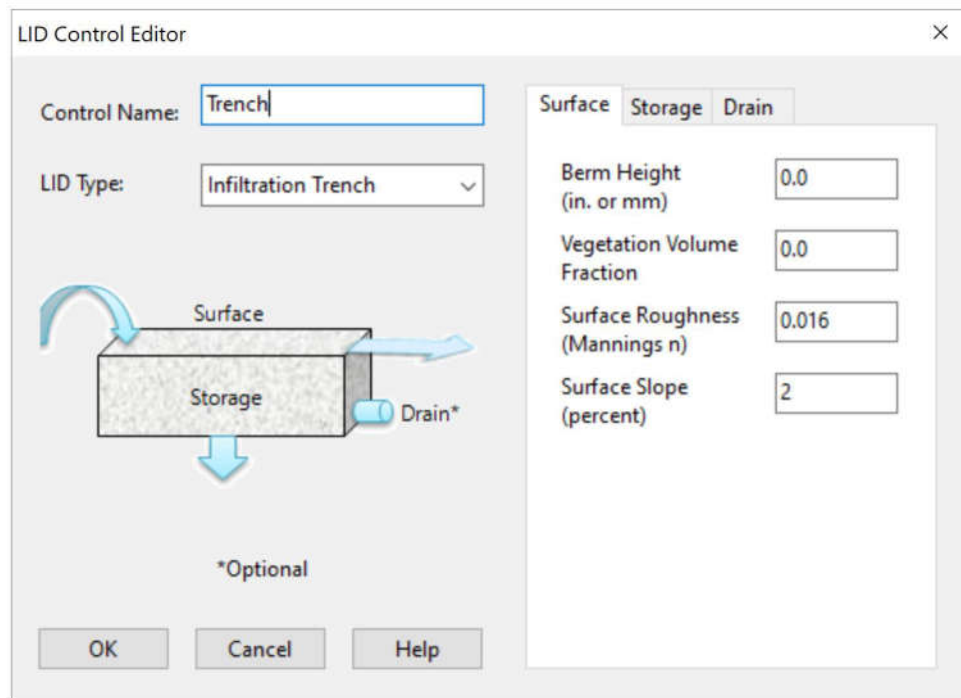
Figure 52 – Infiltration trench editing menu for the surface tab
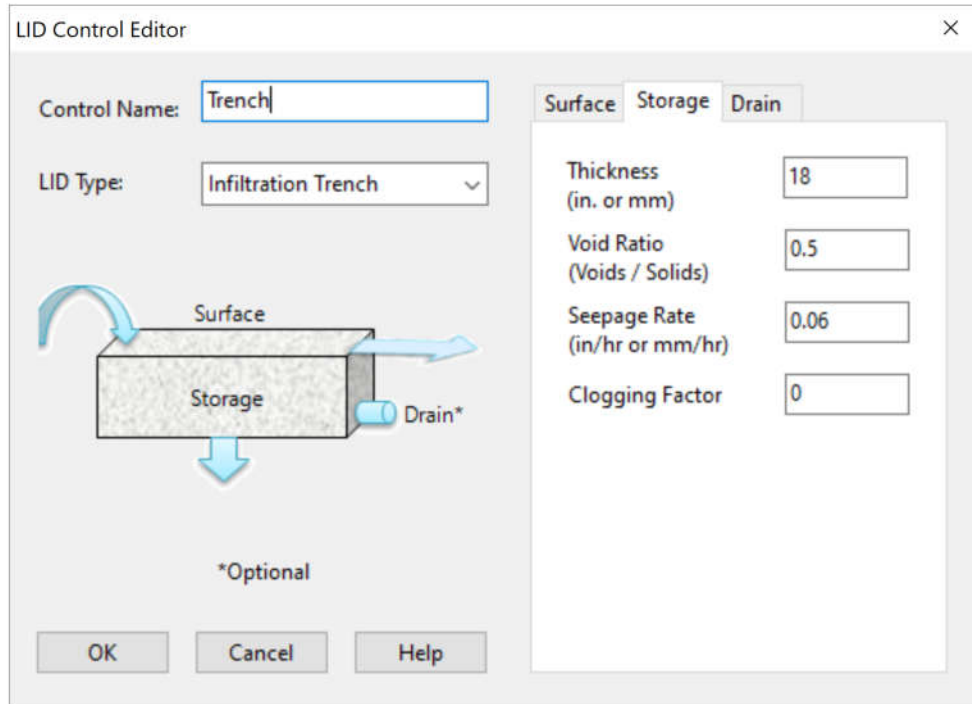
Figure 53 – Infiltration trench editing menu for the storage tab
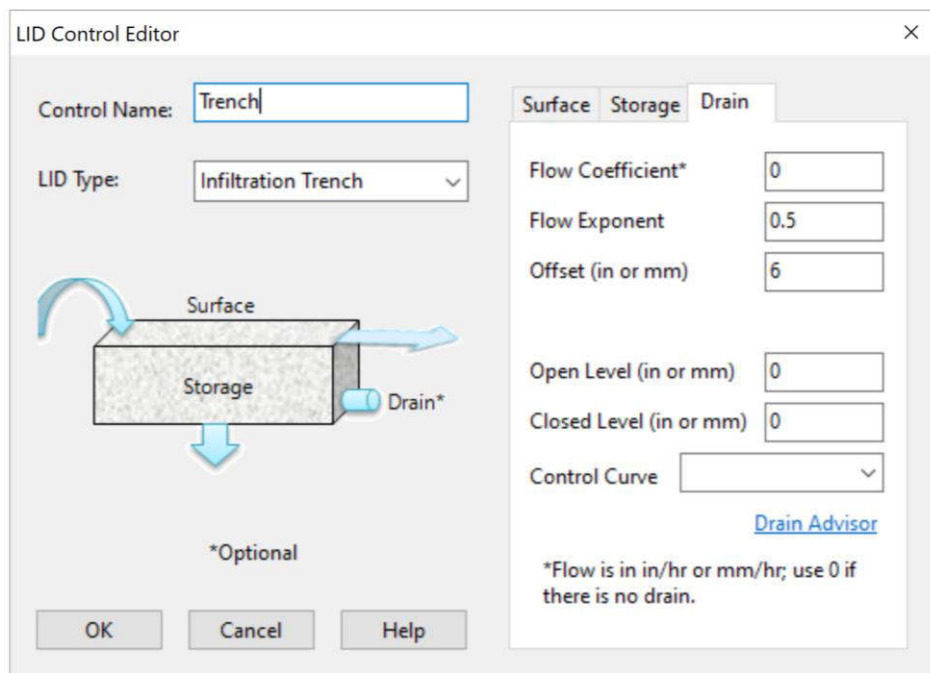
Figure 54 – Infiltration trench editing menu for the drain tab

A.2 Design Plans

A.2.1 Parking Lot A plans
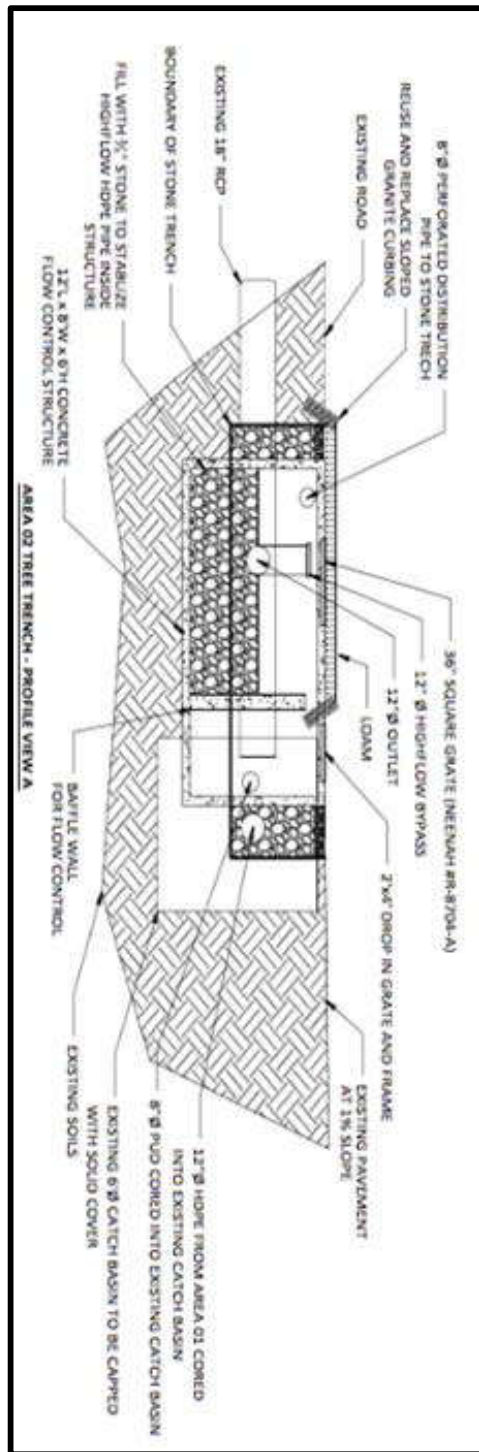
Figure 55 - Tree Trench system Profile View A (UNH Stormwater Center, 2014)
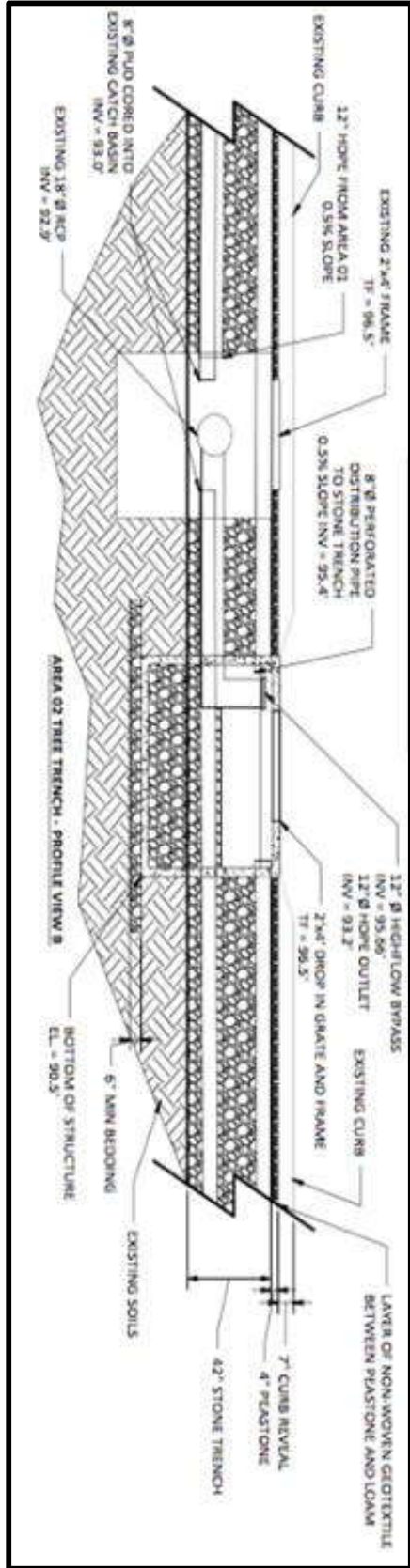
Figure 56 - Tree Trench system Profile View B (UNH Stormwater Center, 2014)
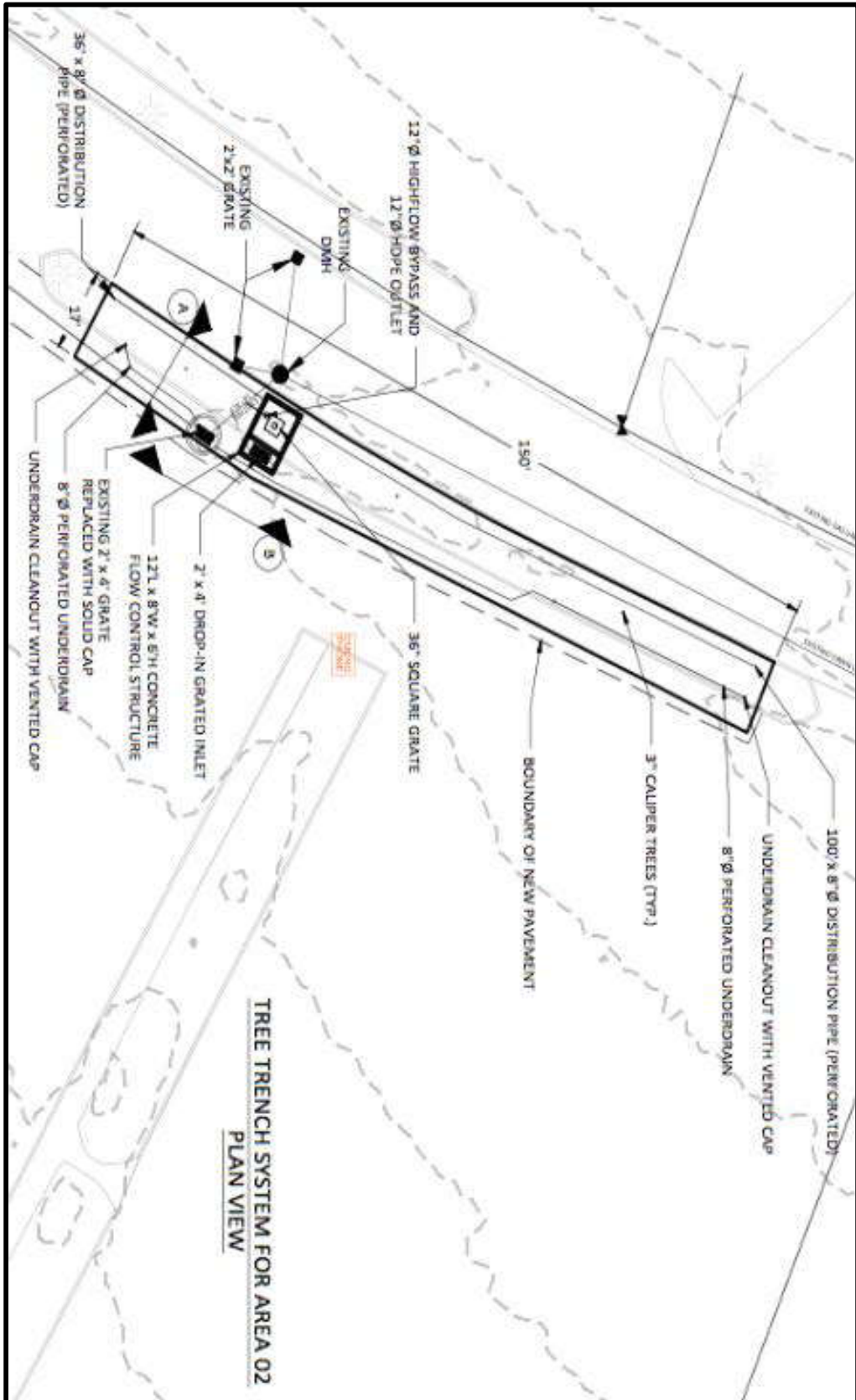
Figure 57 - Tree Trench system Plan View (UNH Stormwater Center, 2014)
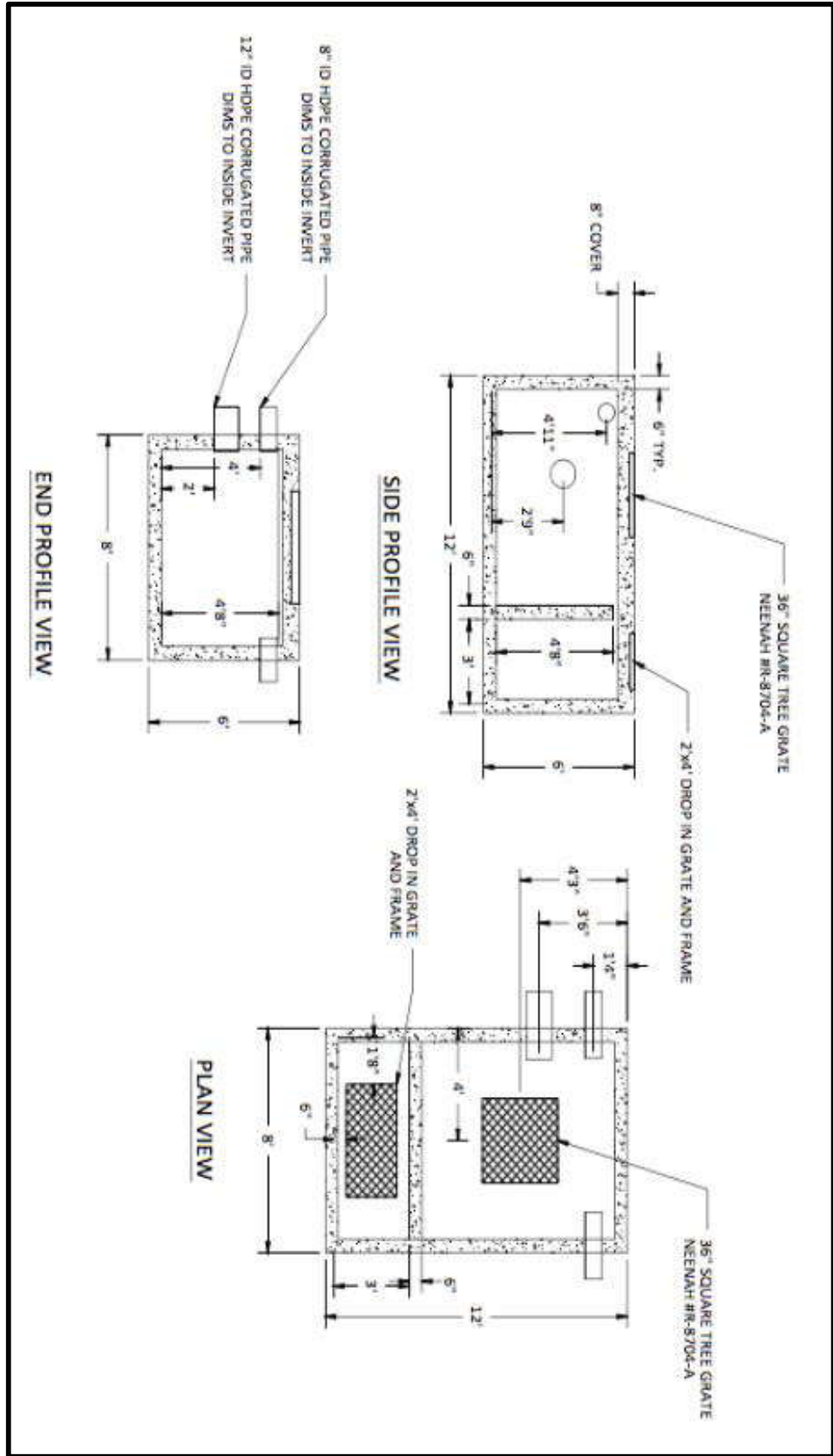
Figure 58 - Tree Trench system detail (UNH Stormwater Center, 2014)
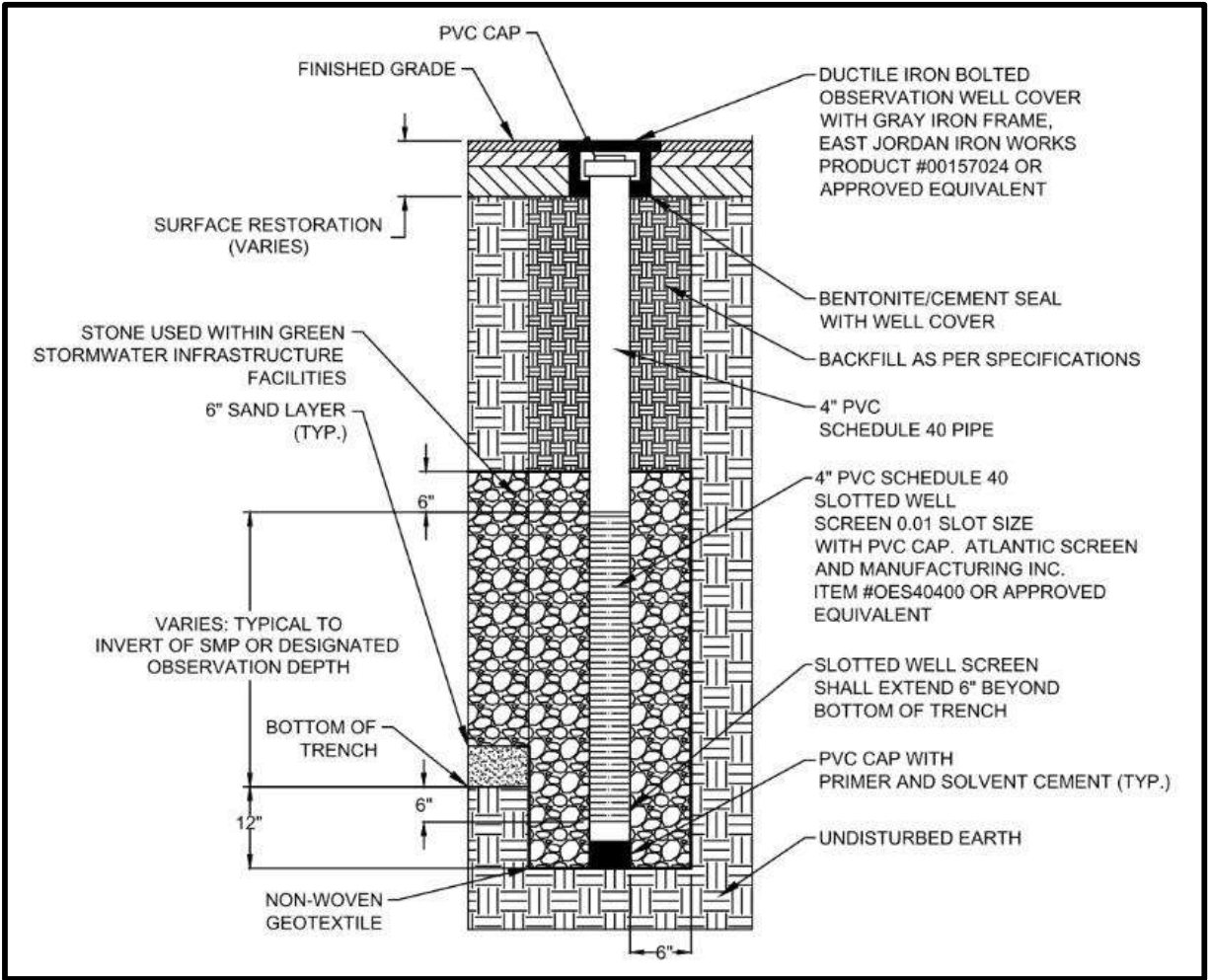
Figure 59 - Detail of observation well on trench

A.2.2 Parking Lot E plans
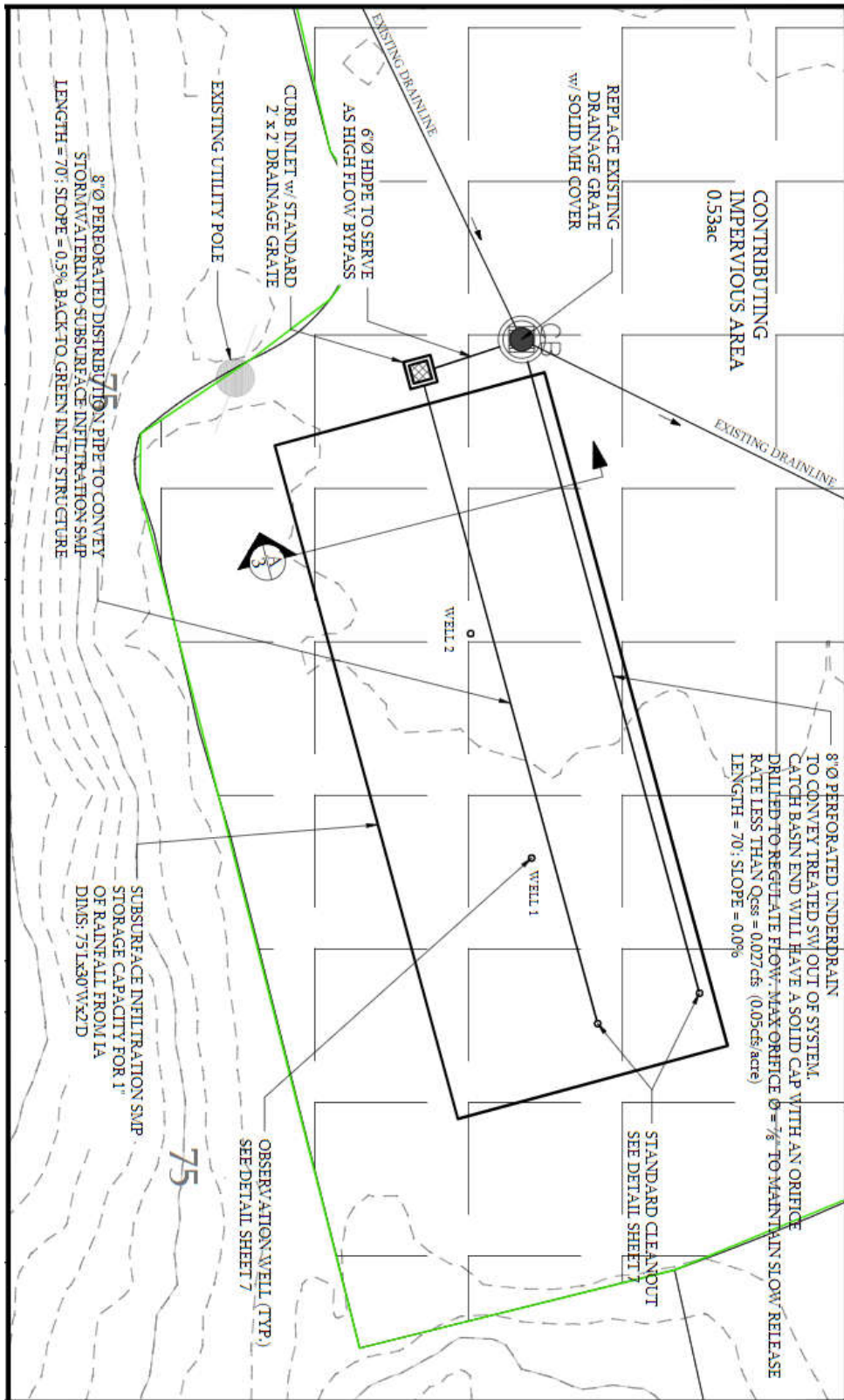
Figure 60 - Basin Plan View (UNH Stormwater Center, 2016)
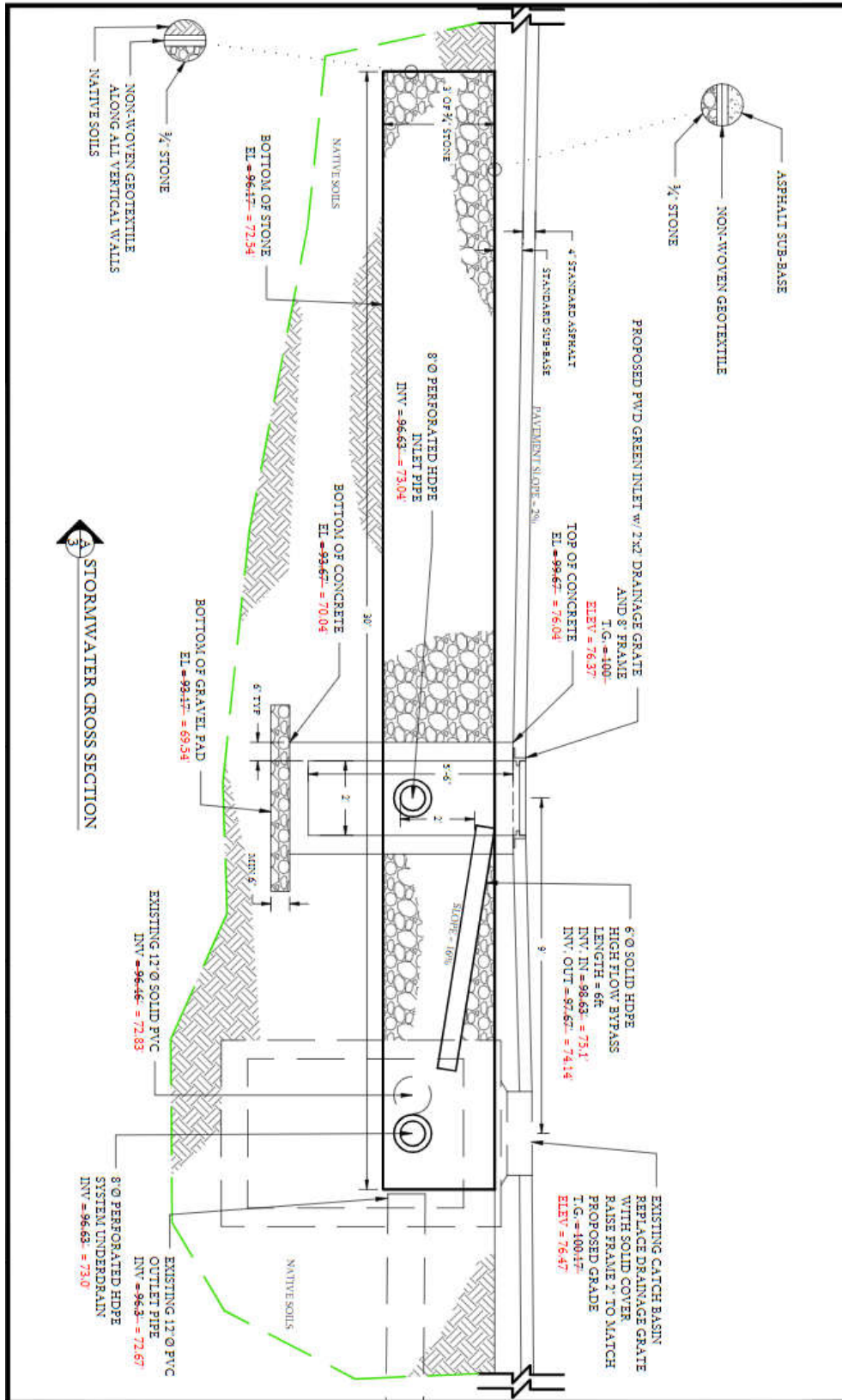
Figure 61 - Basin Cross Section View (UNH Stormwater Center, 2016)

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A.3 Storms Time Series and Hyetographs
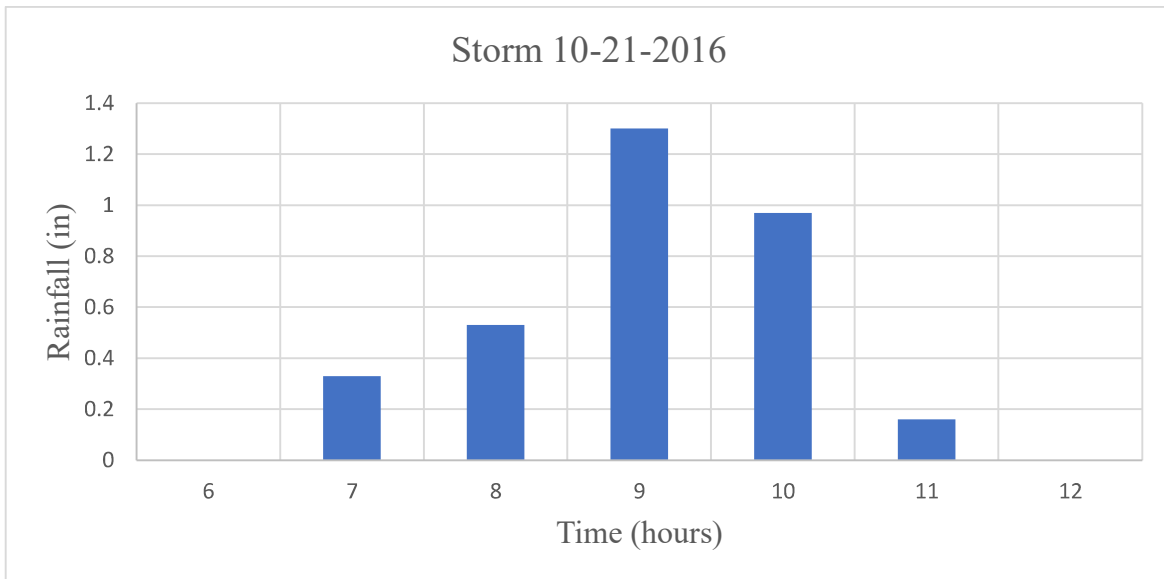
Figure 62 – Hyetograph of the Storm 10-21-2016
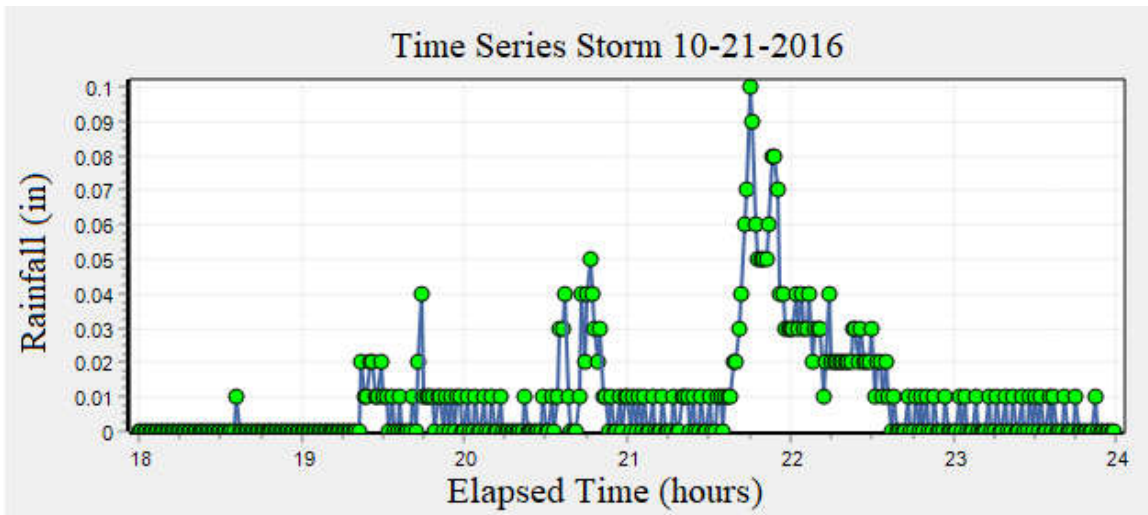
Figure 63 - Time Series for the Storm 10-21-2016
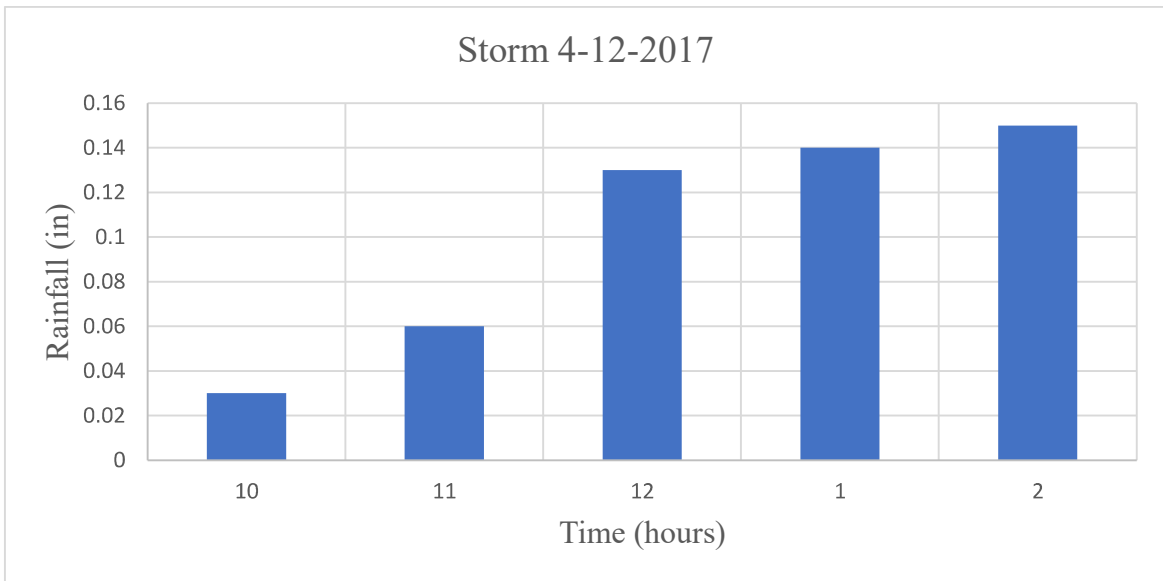
Figure 64 – Hyetograph of the Storm 04-12-2017
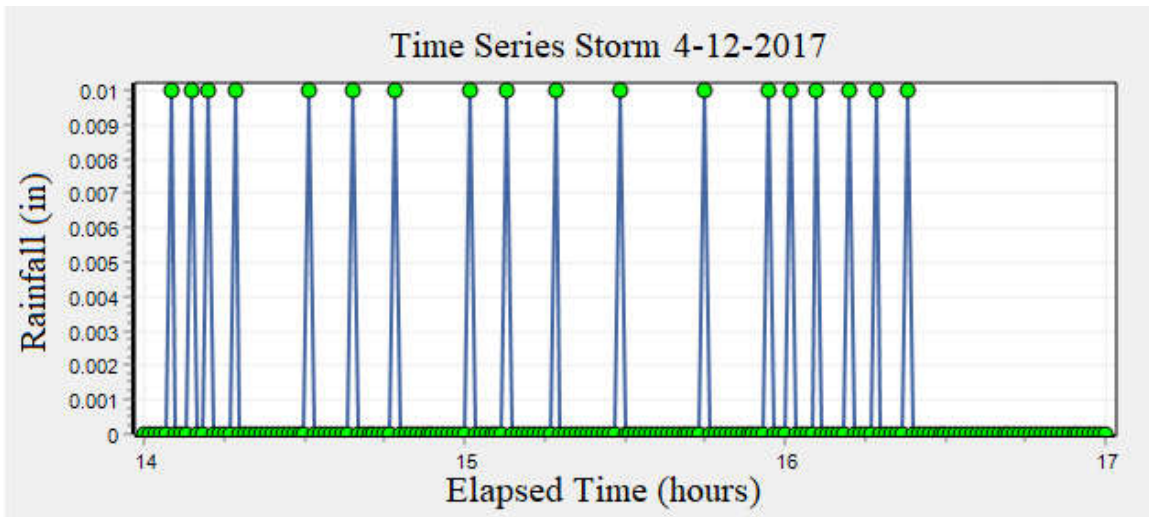
Figure 65 - Time Series for the Storm 04-12-2017
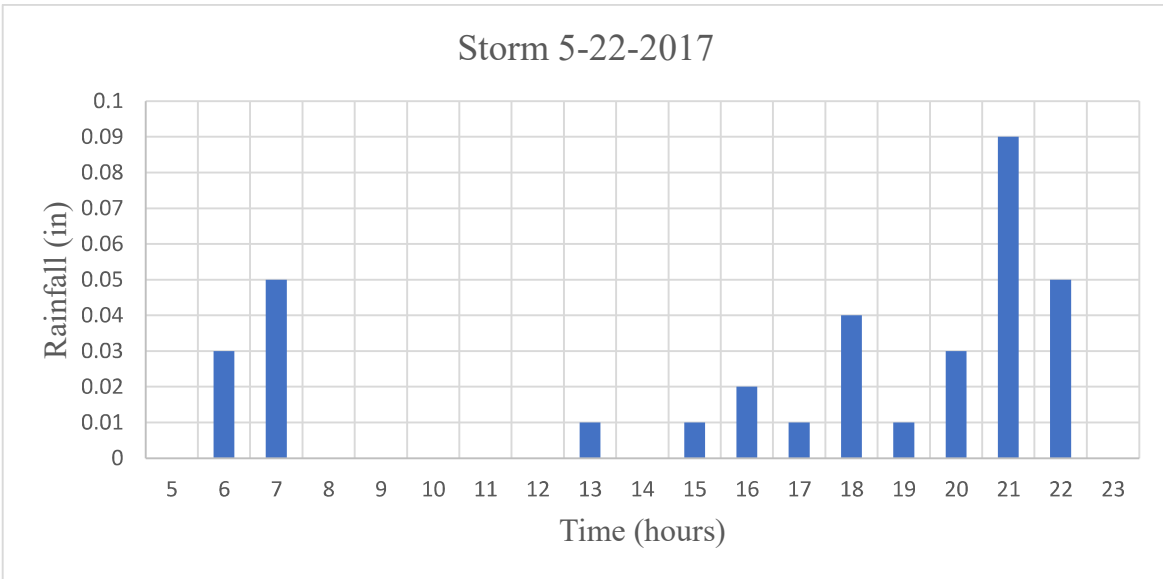
Figure 66 – Hyetograph of the Storm 05-22-2017
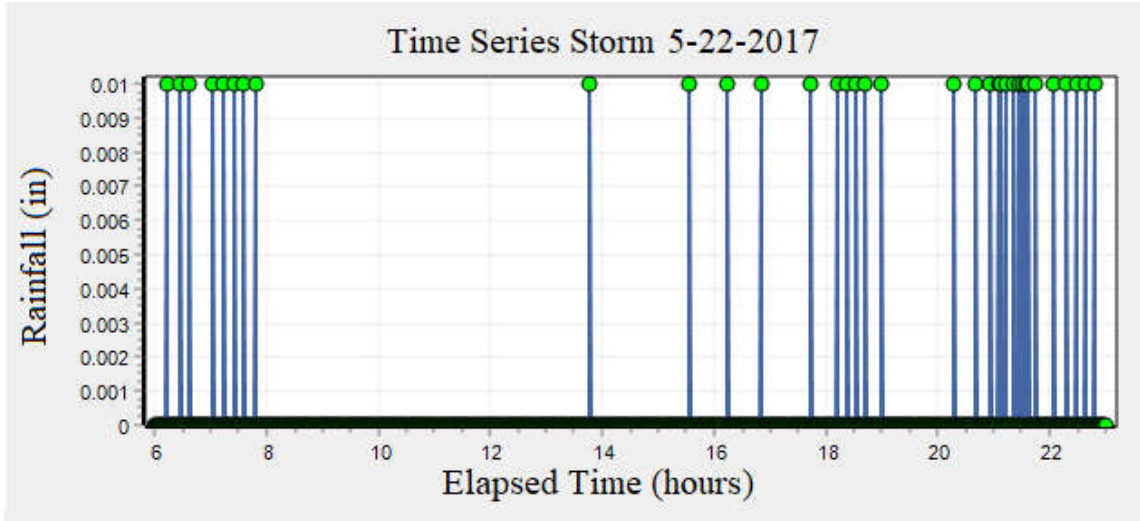
Figure 67 - Time Series for the Storm 05-22-2017
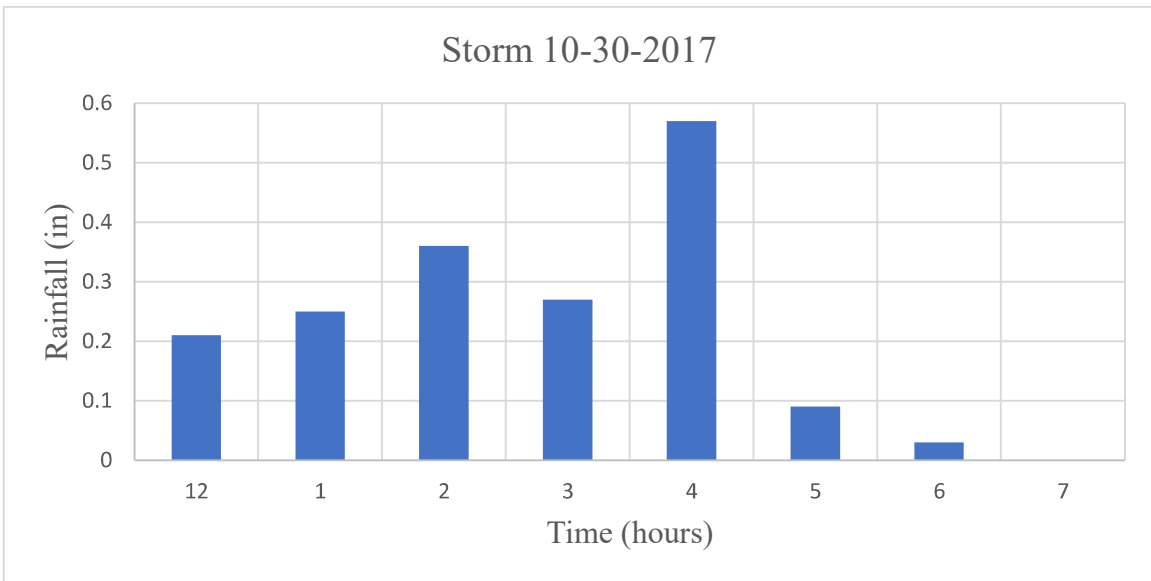
Figure 68 – Hyetograph of the Storm 10-30-2017
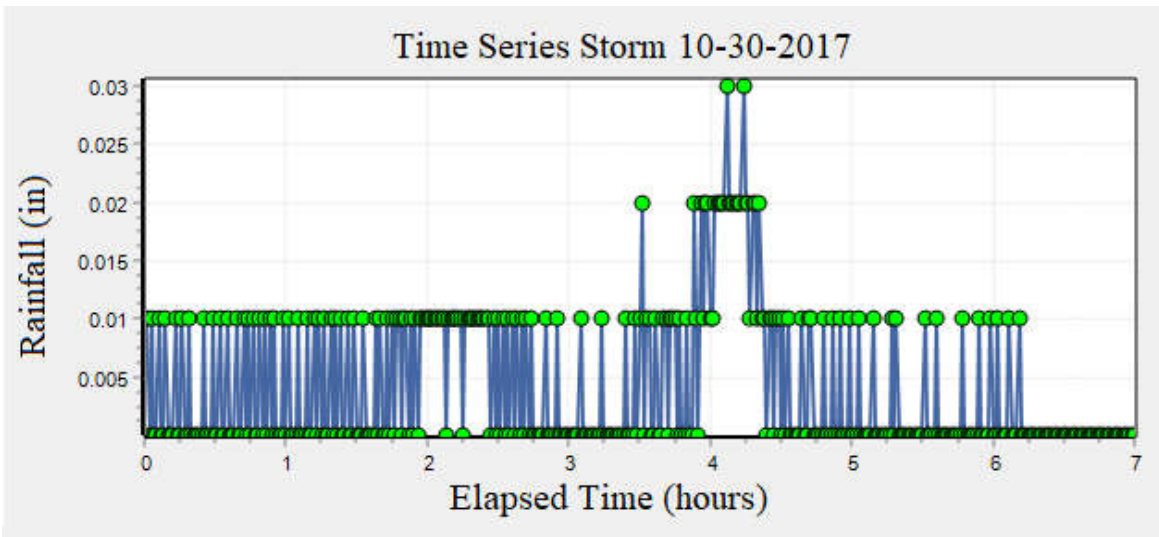
Figure 69 - Time Series for the Storm 10-30-2017
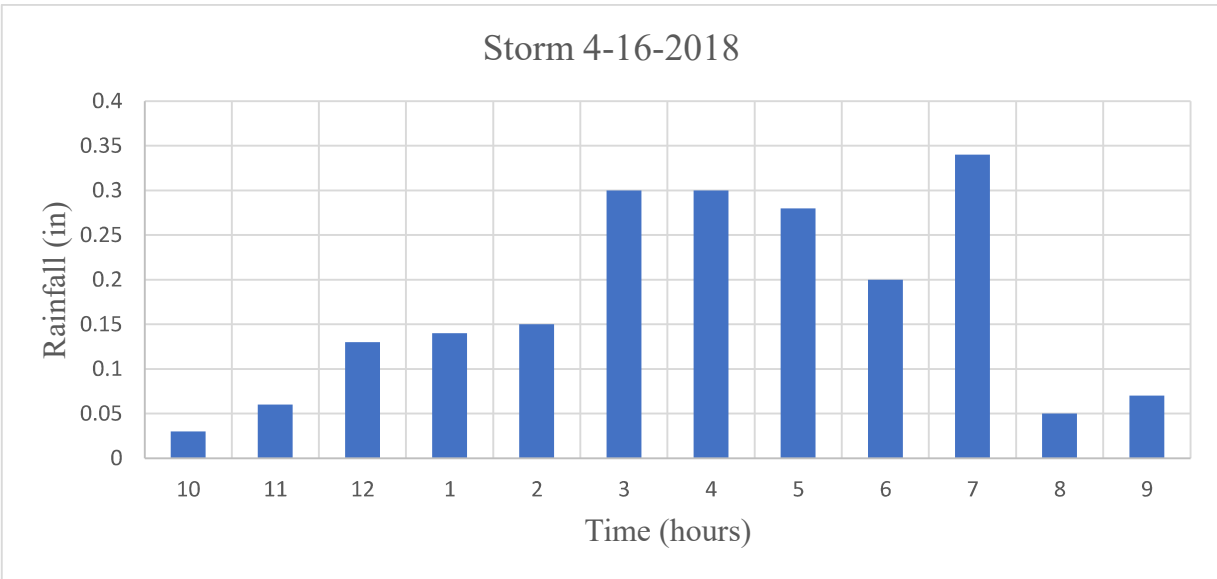
Figure 70 – Hyetograph of the Storm 04-16-2018
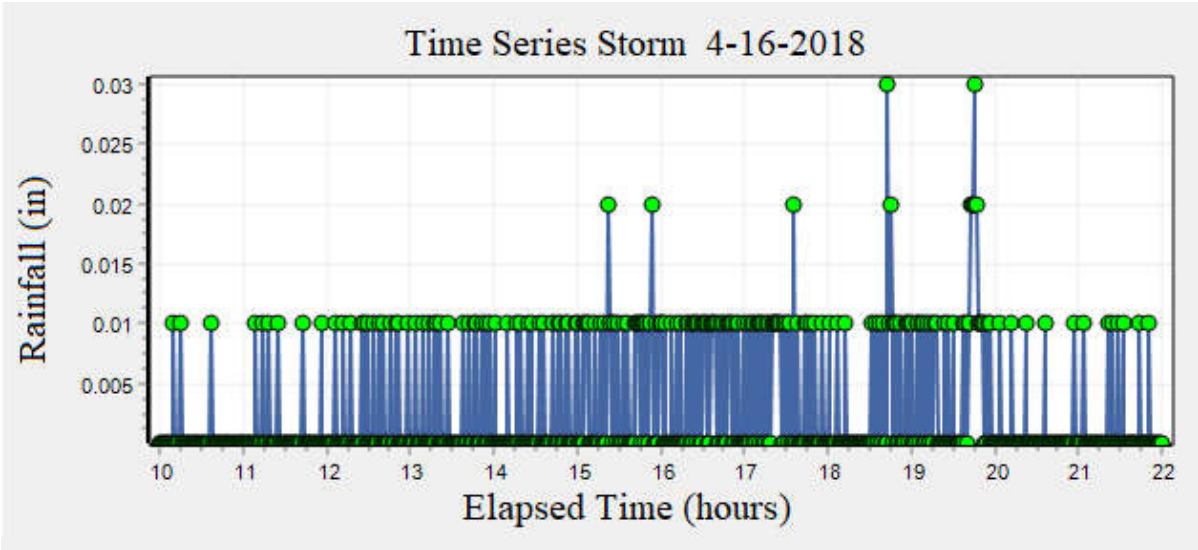
Figure 71 - Time Series for the Storm 04-16-2018
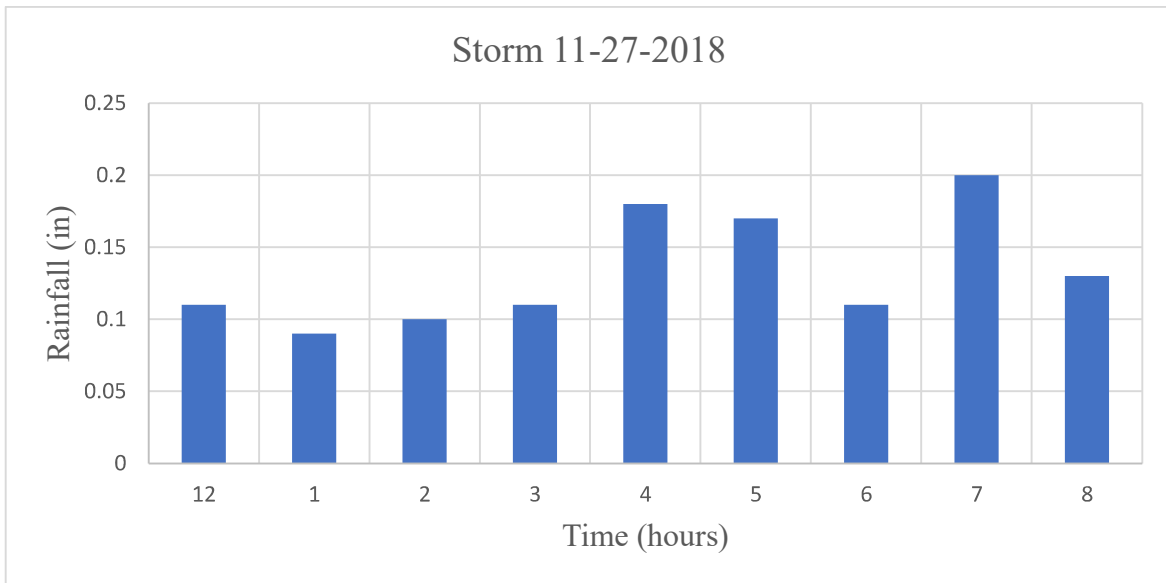
Figure 72 – Hyetograph of the Storm 11-27-2018
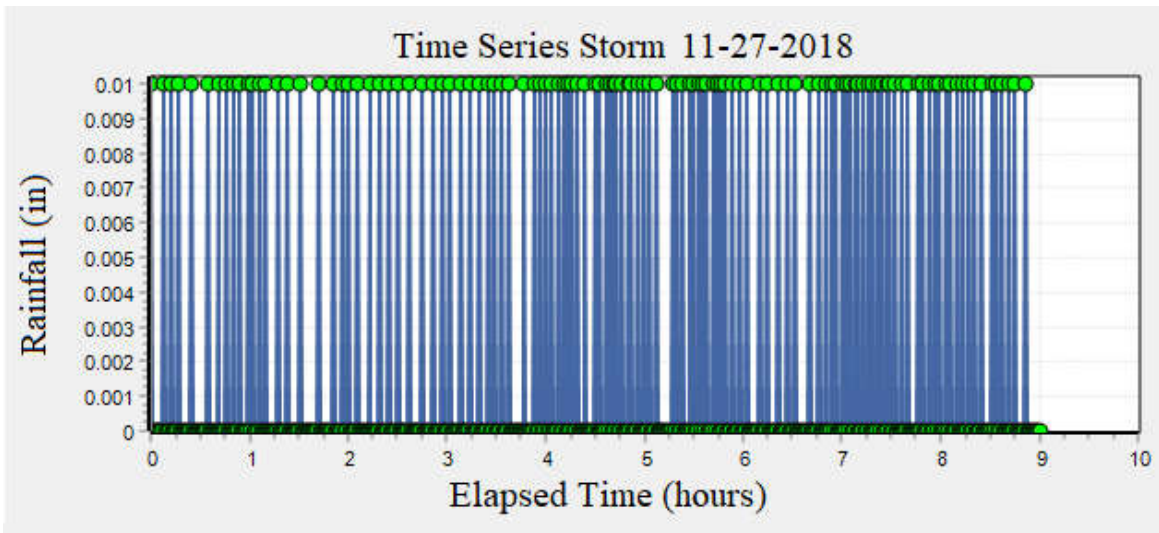
Figure 73 - Time Series for the Storm 11-27-2018
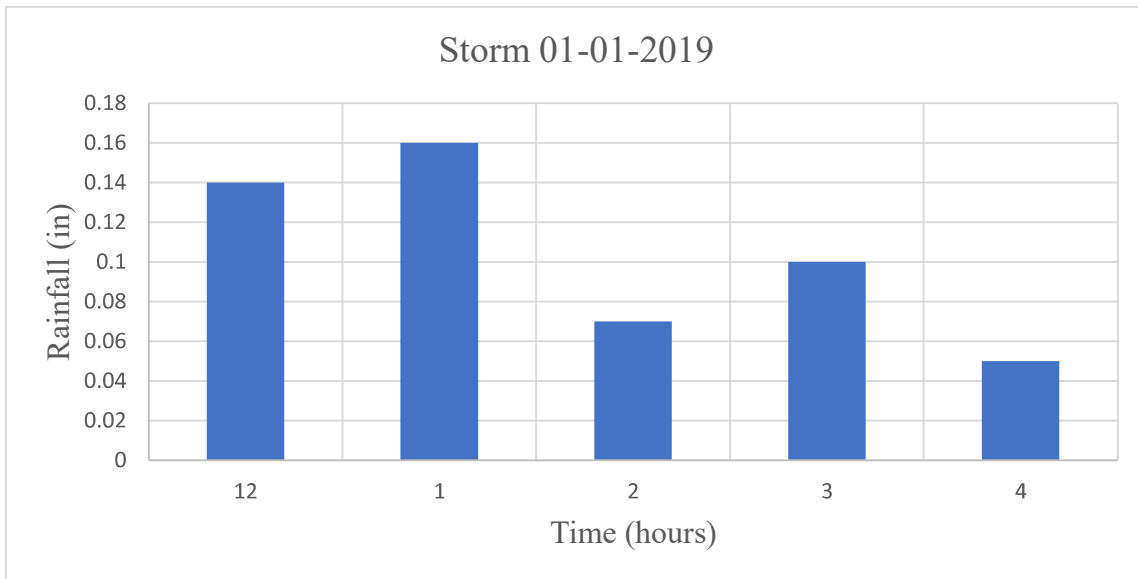
Figure 74 – Hyetograph of the Storm 01-01-2019
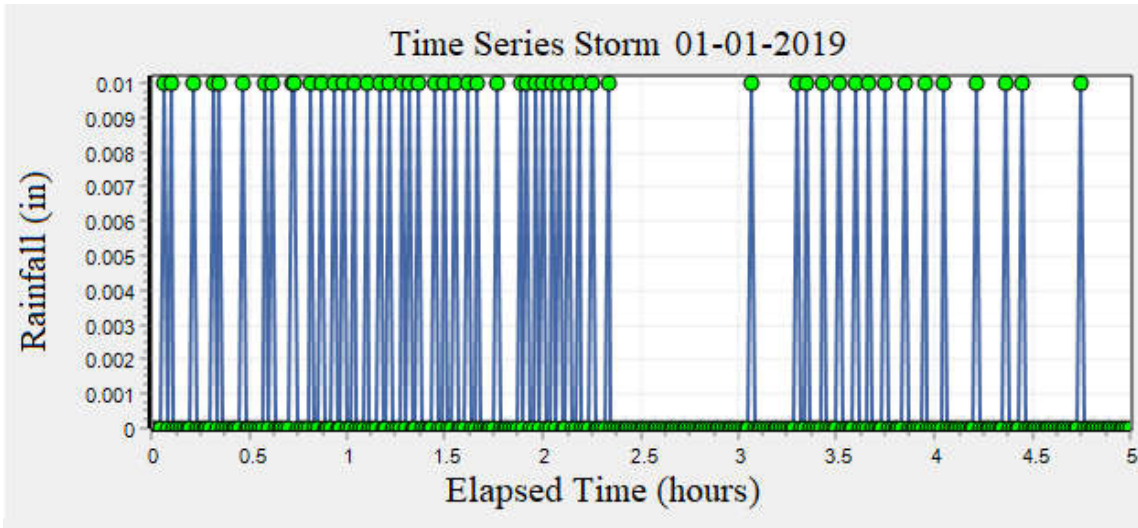
Figure 75 - Time Series for the Storm 01-01-2019
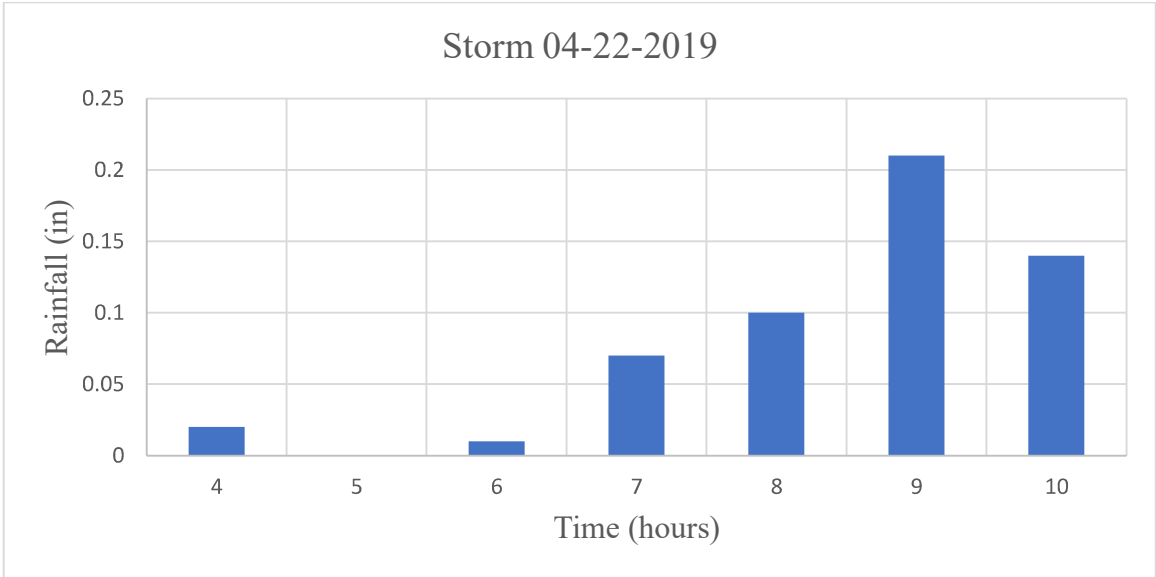
Figure 76 – Hyetograph of the Storm 04-22-2019
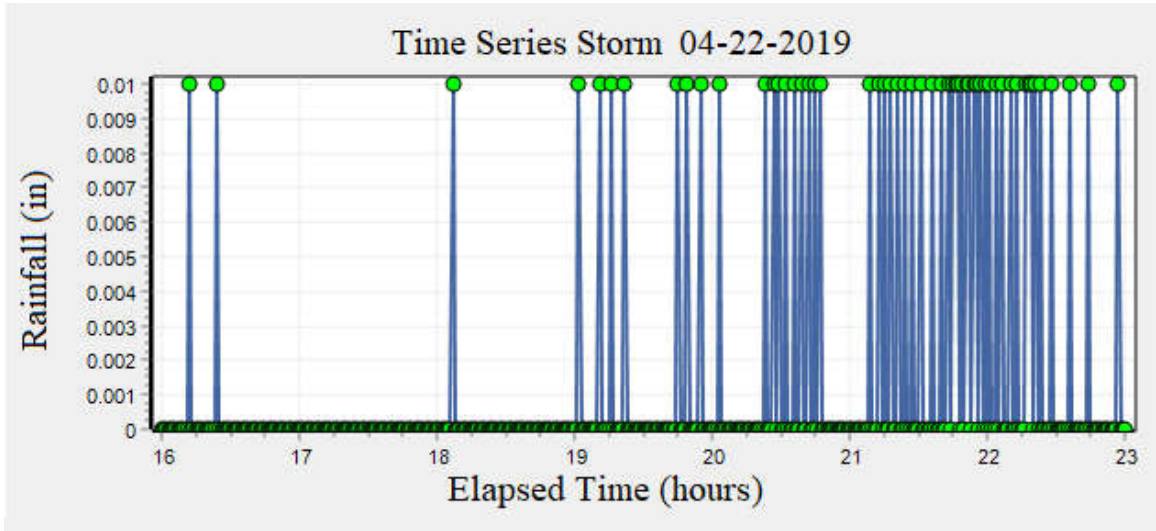
Figure 77 - Time Series for the Storm 04-22-2019

A.4 Runoff Plots

A.4.1 Parking Lot A runoff plots
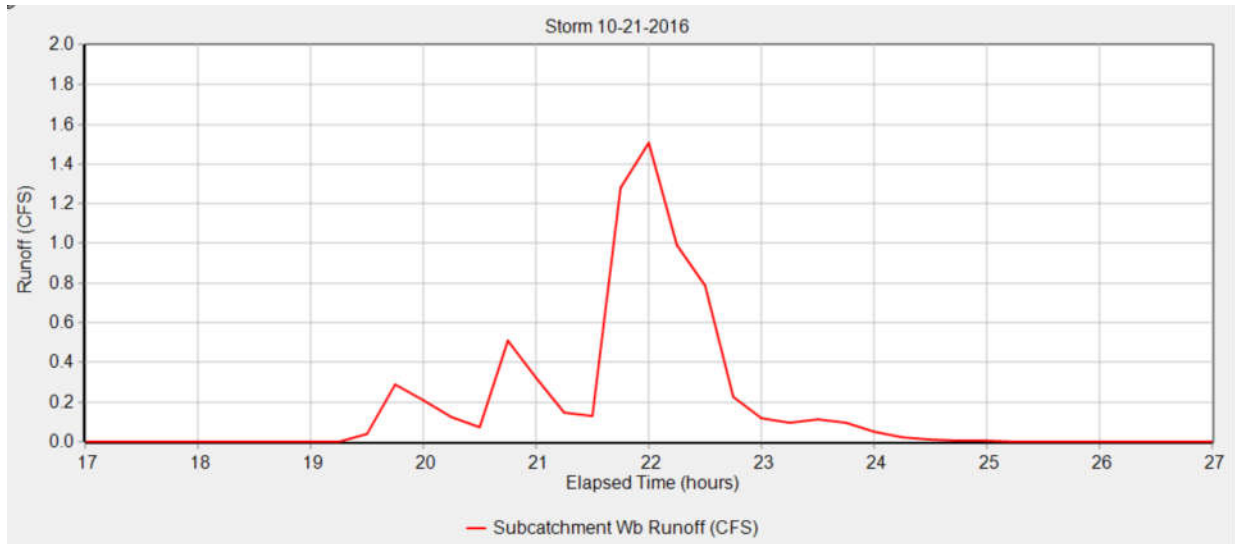
Figure 78 - Sub catchment Wb Runoff in cfs of Storm 10-21-2016
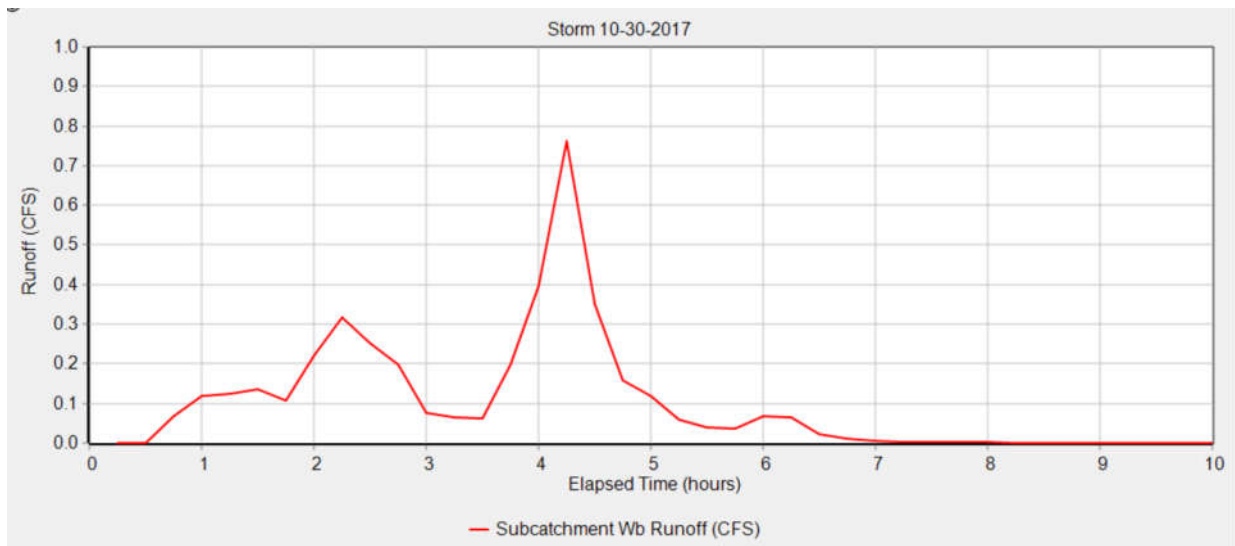
Figure 79 - Sub catchment Wb Runoff in cfs of Storm 10-30-2017
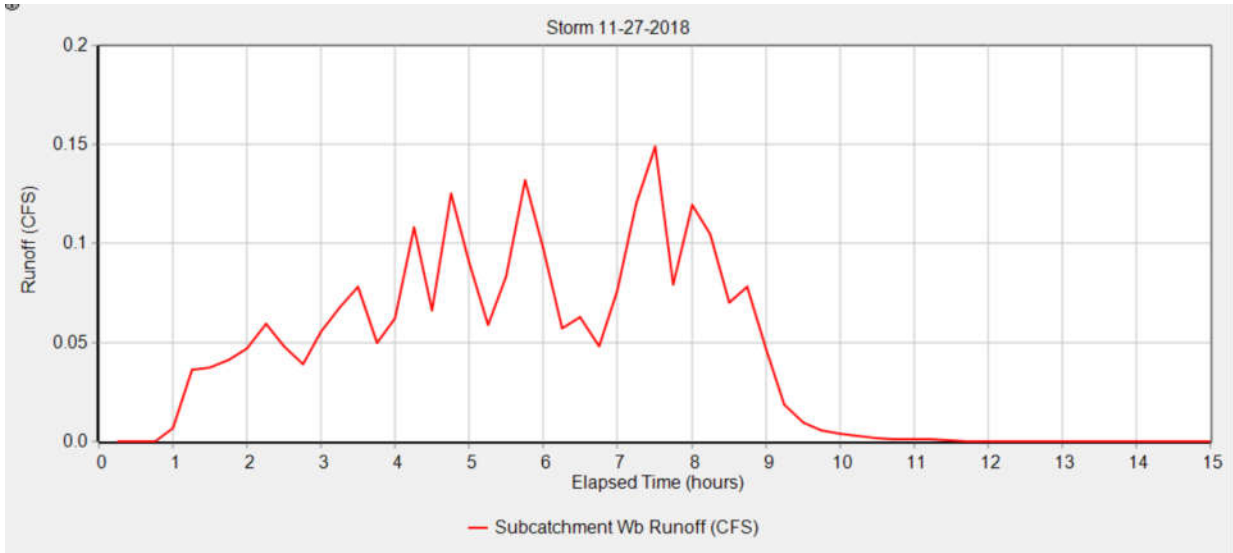
Figure 80 - Sub catchment Wb Runoff in cfs of Storm 11-27-2018
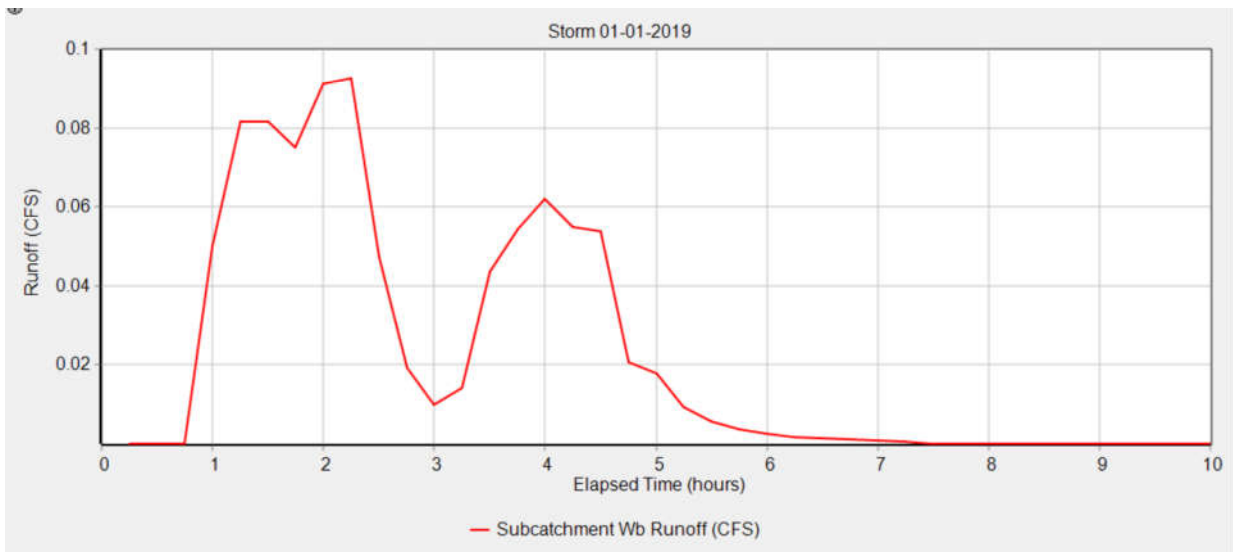
Figure 81 - Sub catchment Wb Runoff in cfs of Storm 01-01-2019
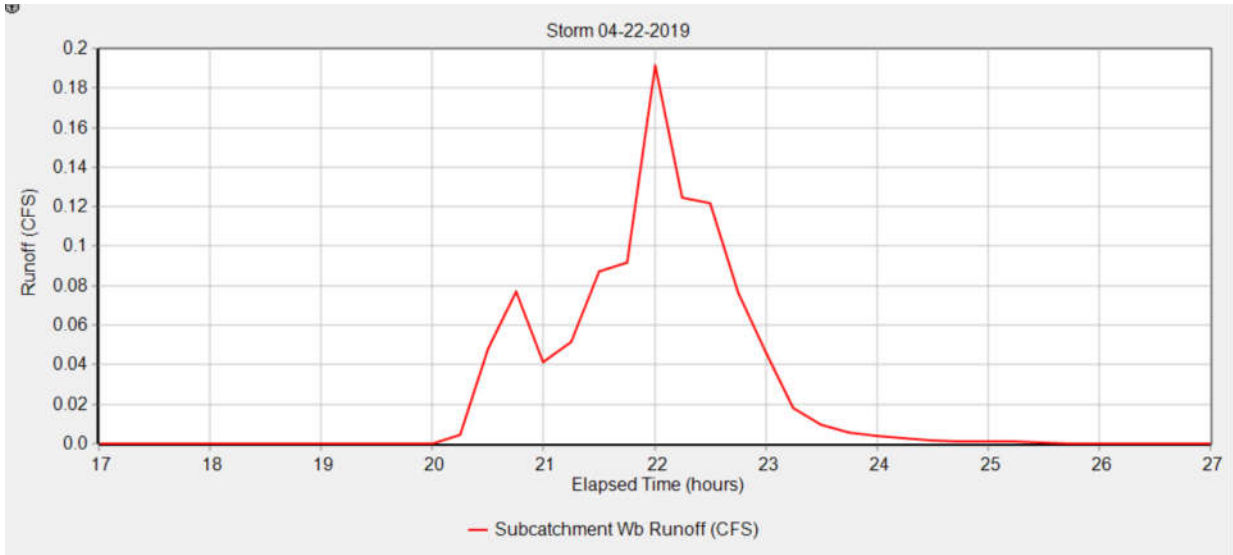
Figure 82 - Sub catchment Wb Runoff in cfs of Storm 04-22-2019

A.4.2 Parking Lot E runoff plots
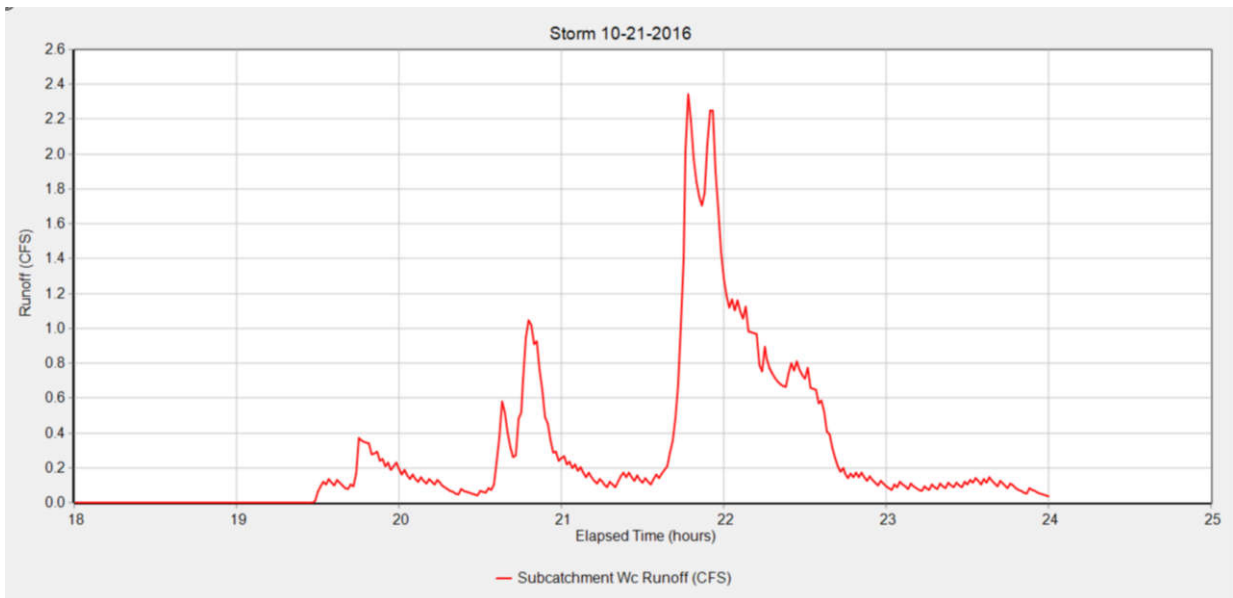
Figure 83 - Subcatchment Wc Runoff for Storm 10-21-2016
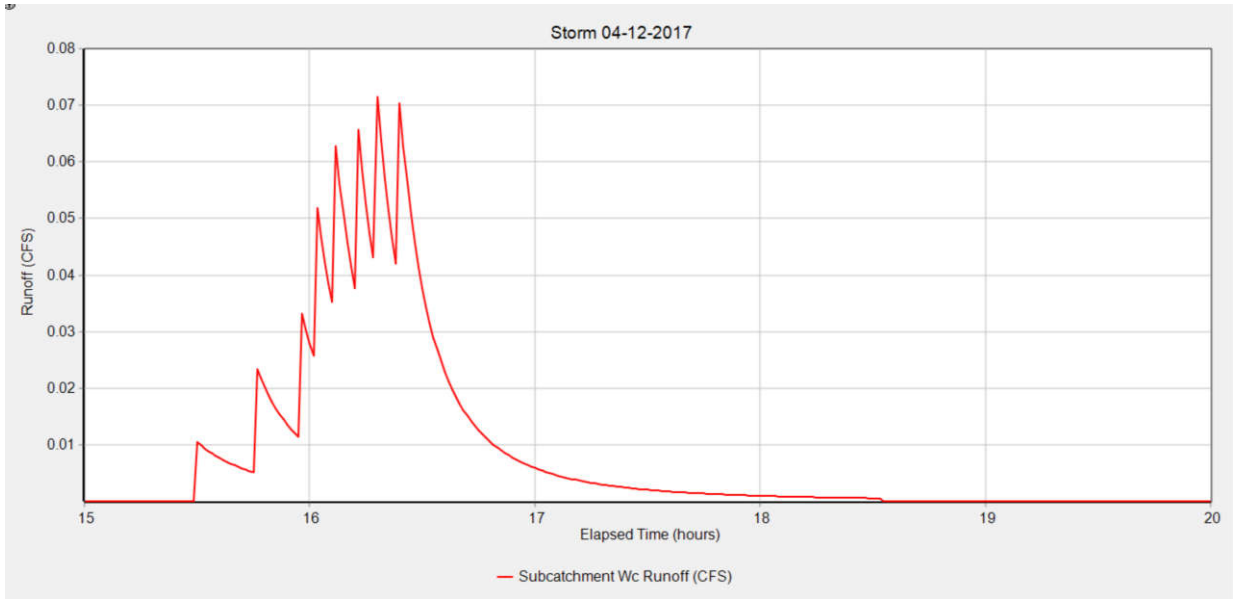
Figure 84 - Subcatchment Wc Runoff for Storm 04-12-2017
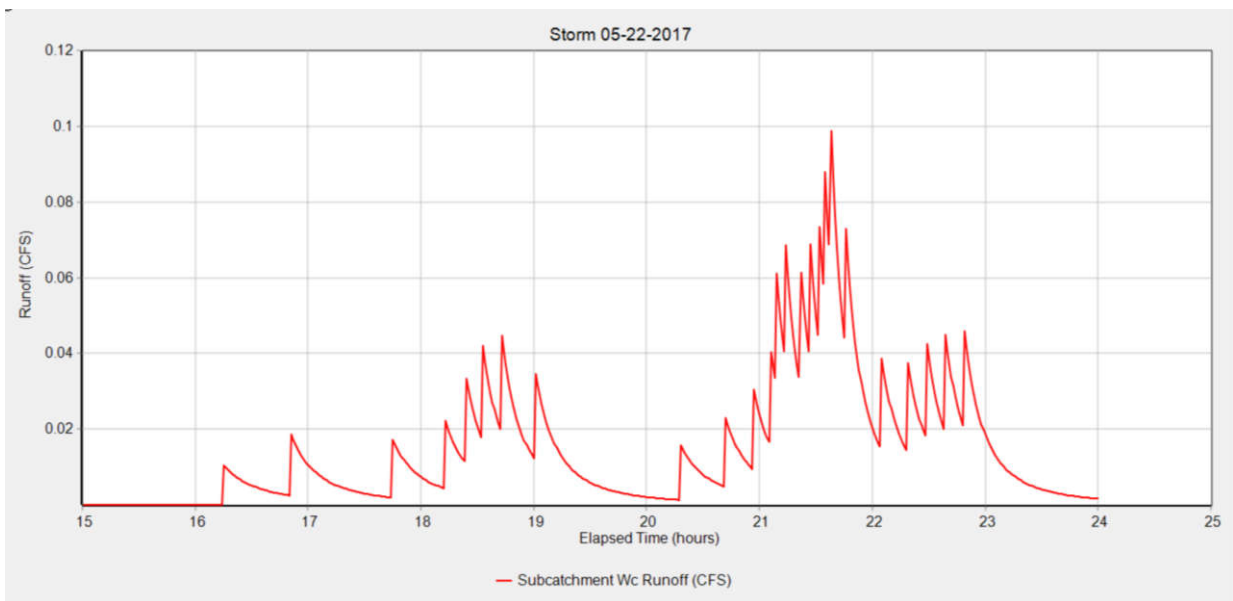
Figure 85 - Subcatchment Wc Runoff for Storm 05-22-2017

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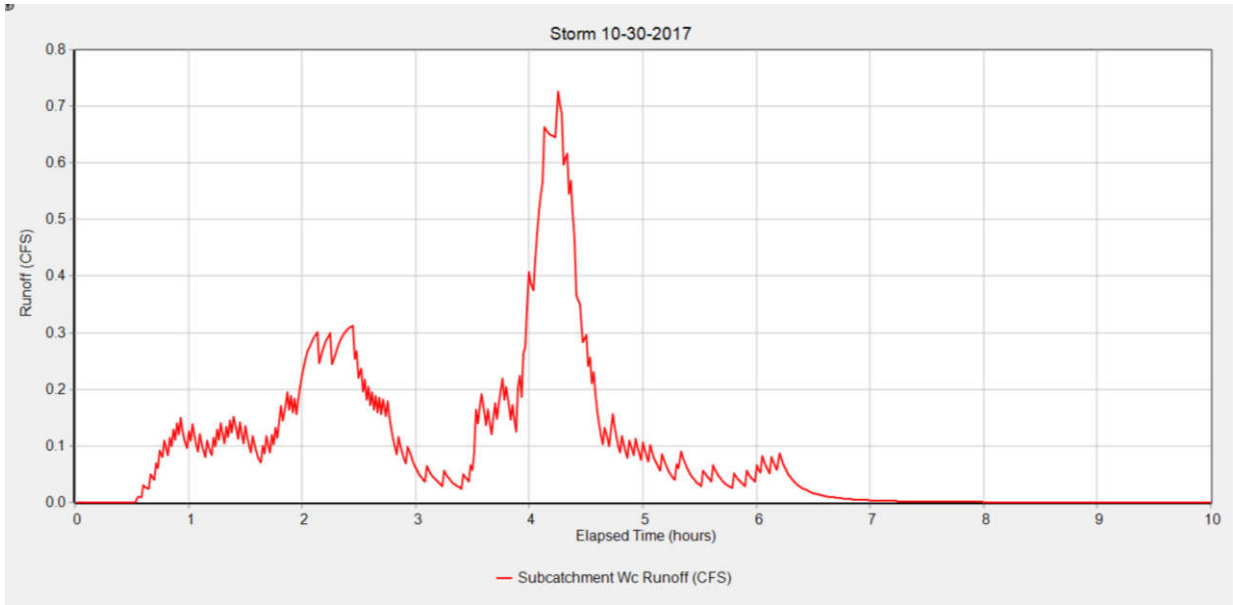
Figure 86 - Subcatchment Wc Runoff for Storm 10-30-2017
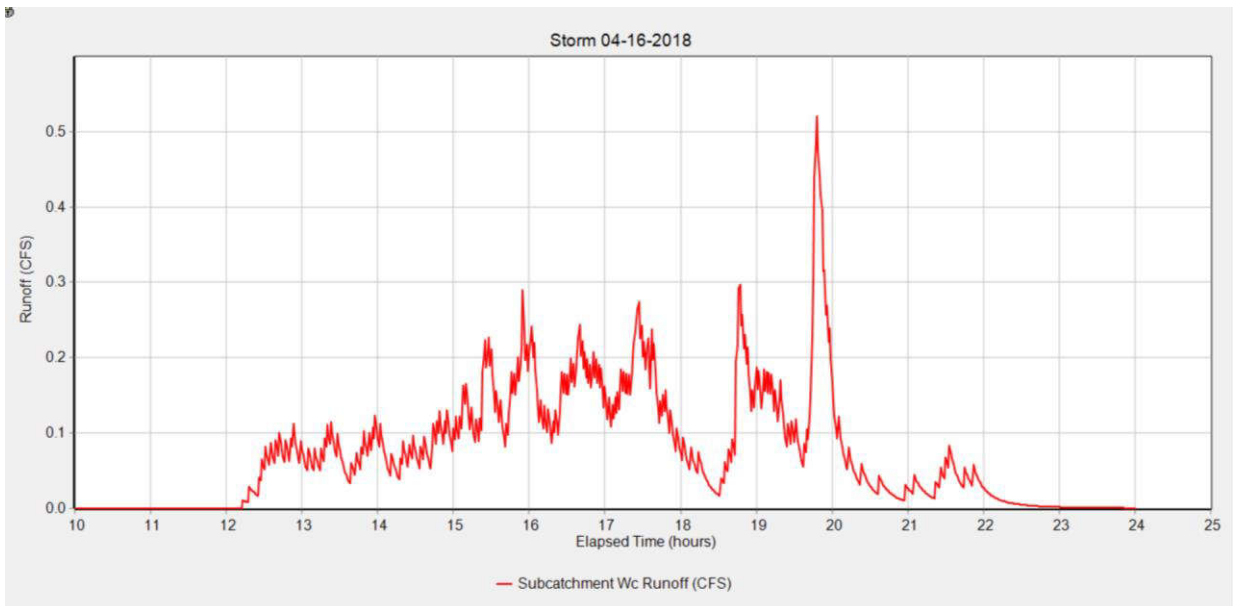
Figure 87 - Subcatchment Wc Runoff for Storm 04-16-2018

A.5 Barometric Compensation

The following procedure was performed in order to get the depth of water over the sensor and plot it in HOBOware:

- **Step 1:** Top menu Edit

- **Step 2:** Data Assistants

- **Step 3:** Barometric Compensation Assistant

- **Step 4:** Use Barometric Datafile

- **Step 5:** Select Barometric file in .txt

- **Step 6:** Create new series

- **Step 7:** Select text file information

- **Step 8:** Select Option (2) to consider all values in the barometric file
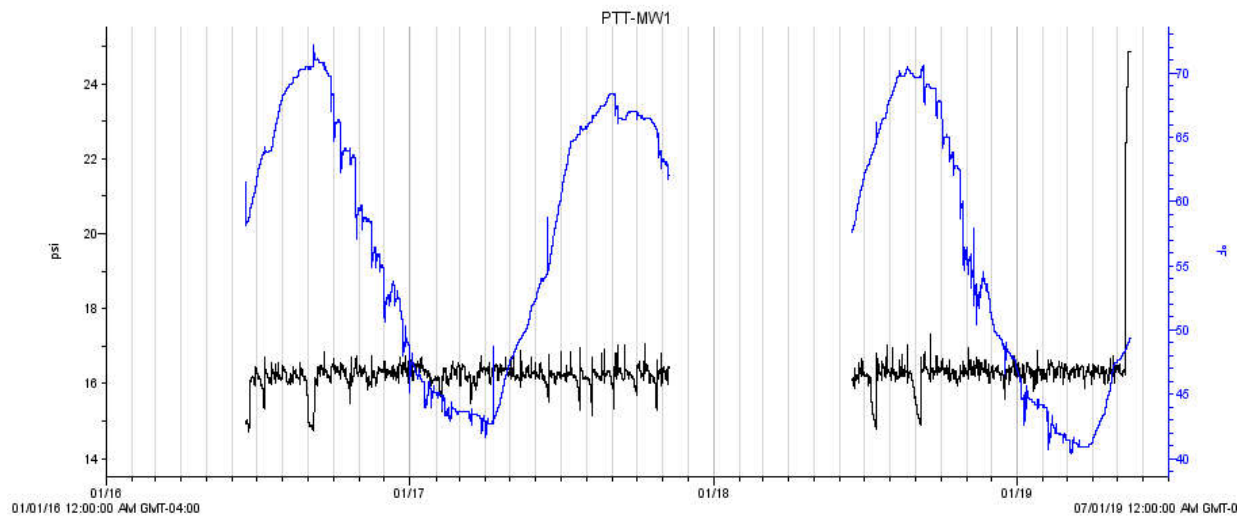
Figure 88 – MW 1 data without barometric compensation
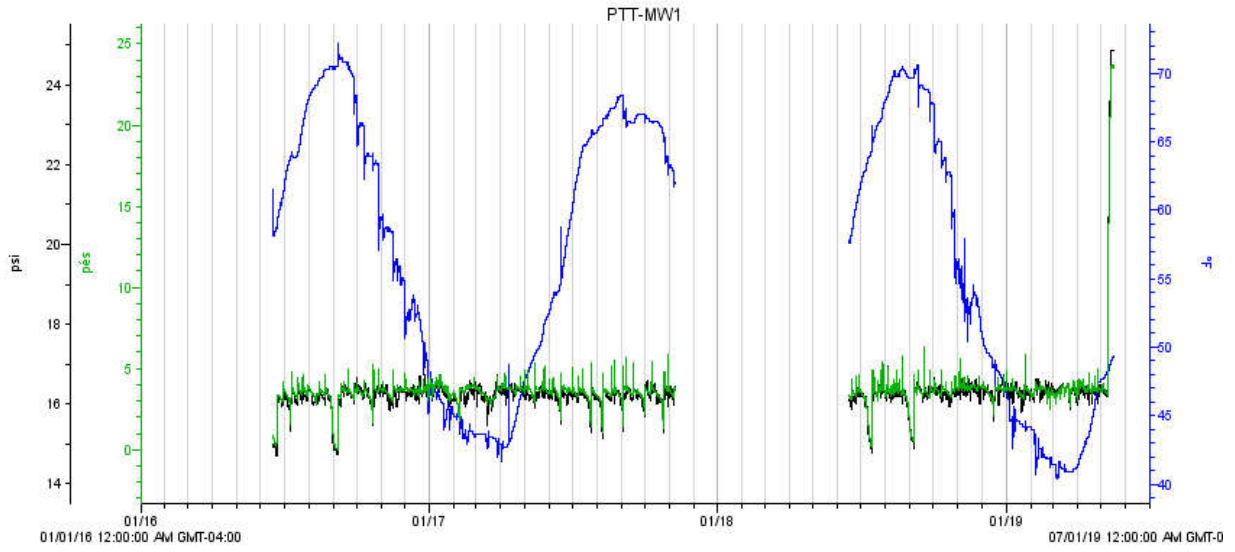
Figure 89 – MW 1 data with barometric compensation
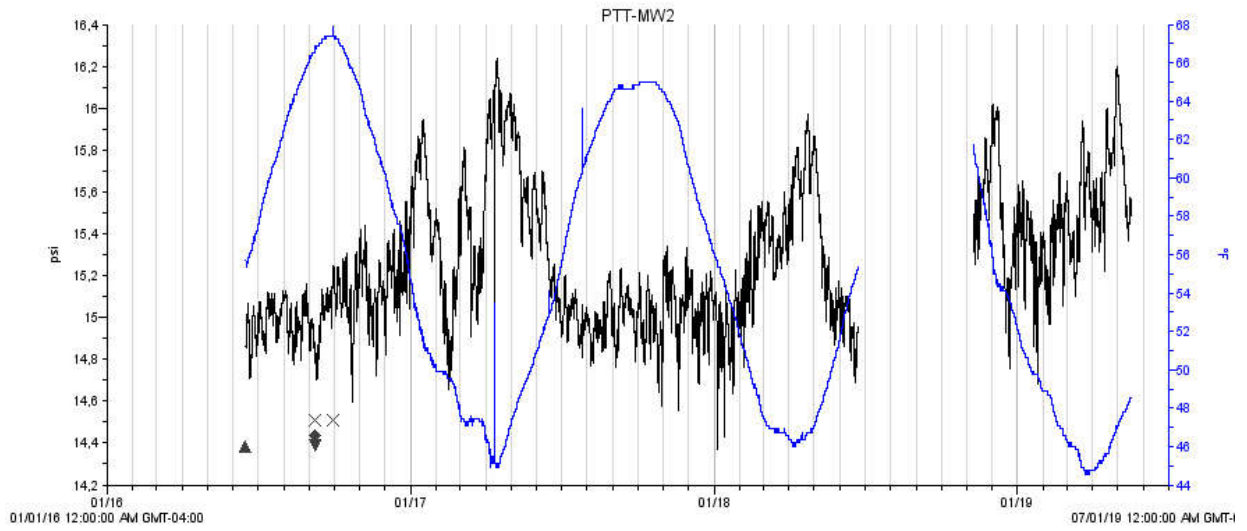
Figure 90 – MW 2 data without barometric compensation
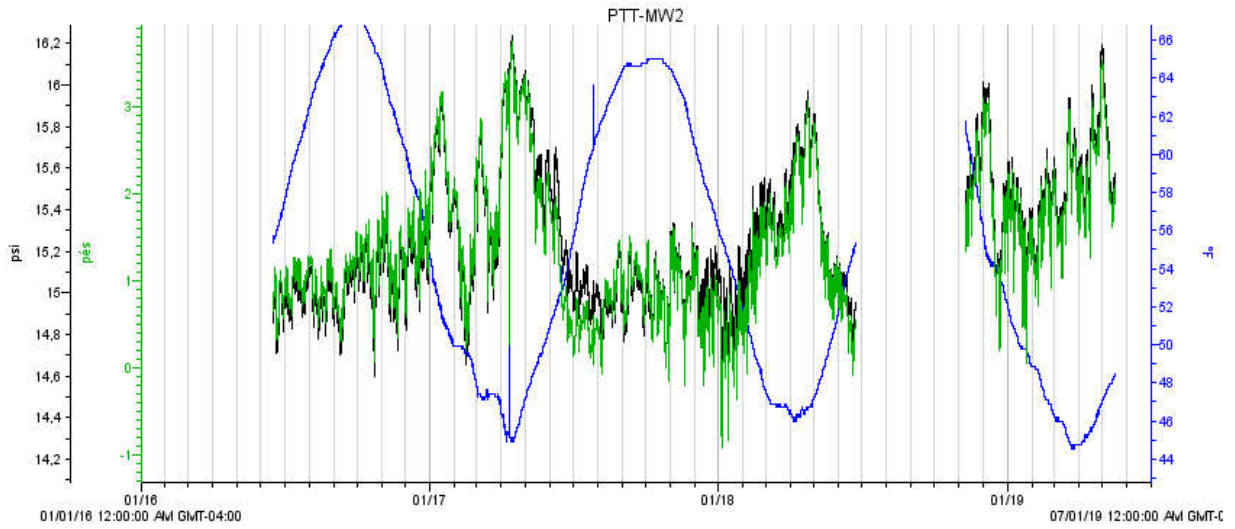
Figure 91 – MW 2 data with barometric compensation
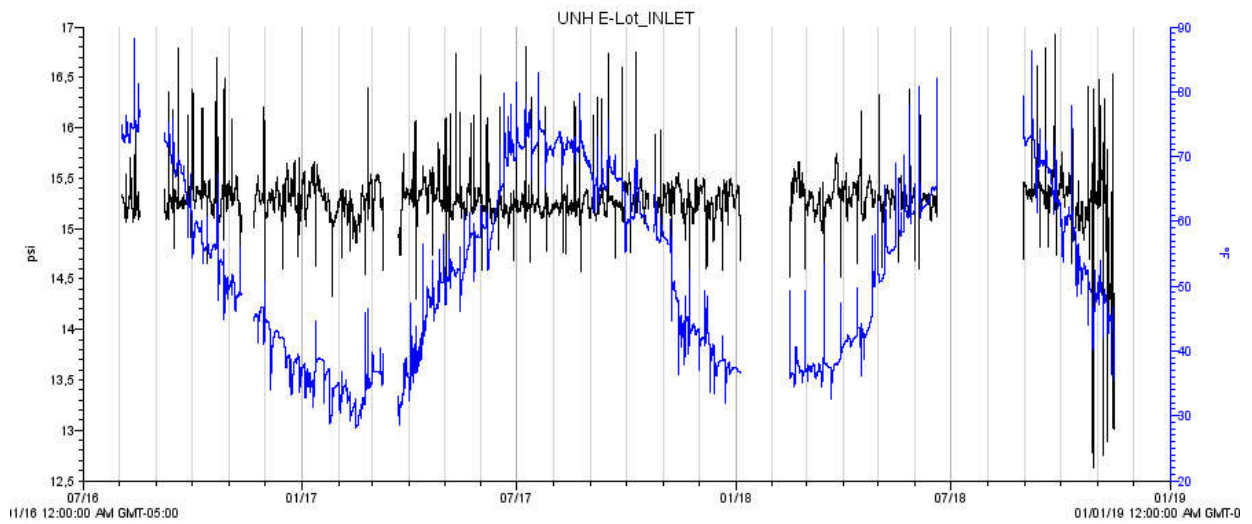
Figure 92 – Inlet data without barometric compensation

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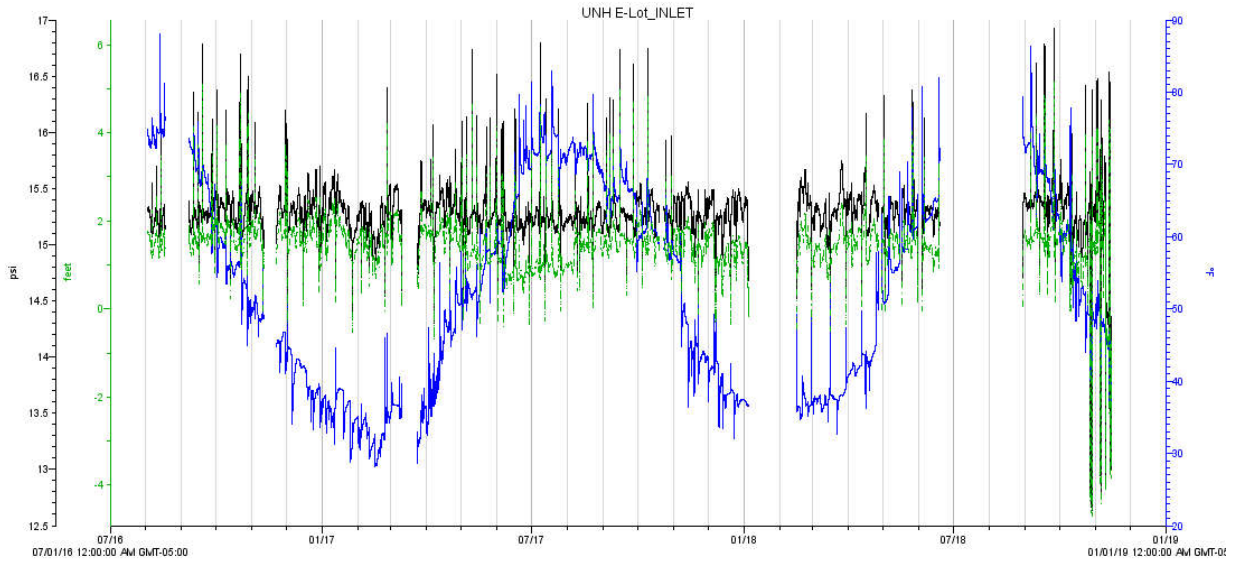
Figure 93 – Inlet data with barometric compensation
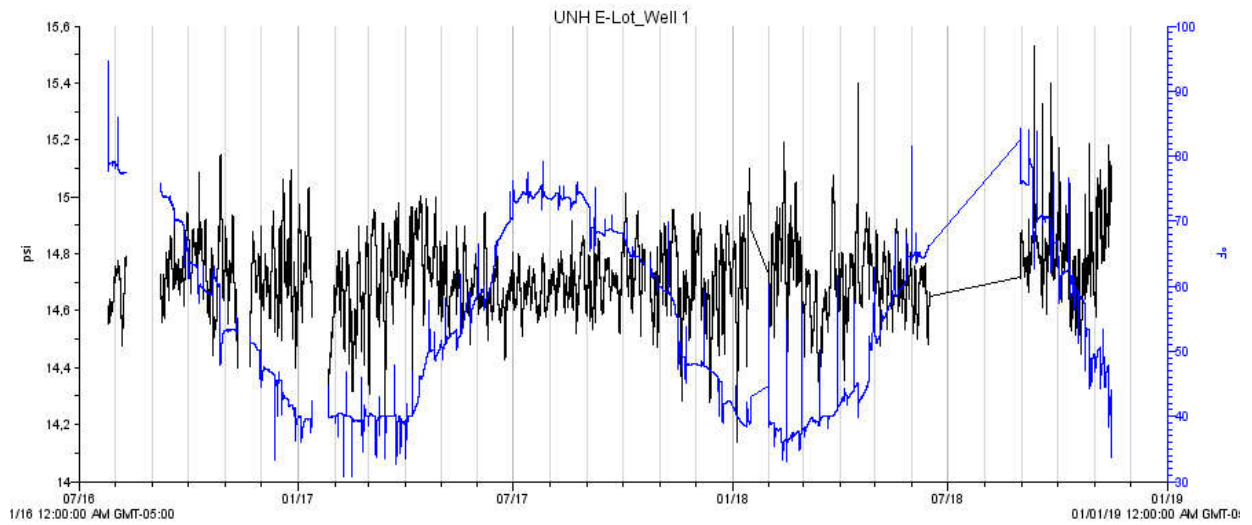
Figure 94 – Well 1 data without barometric compensation
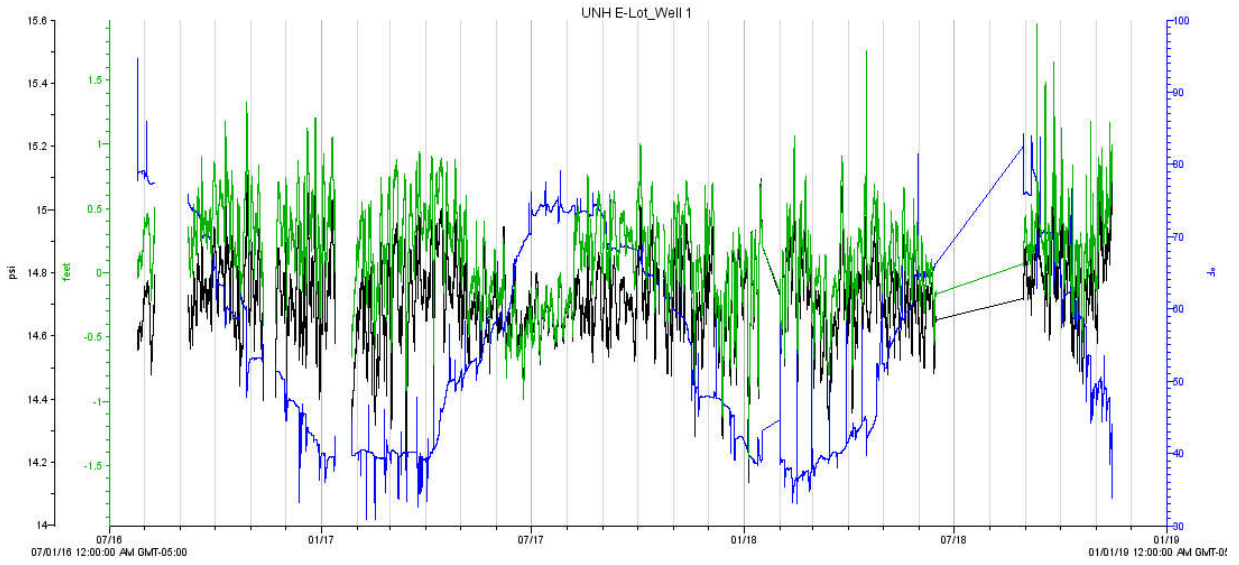
Figure 95 – Well 1 data with barometric compensation
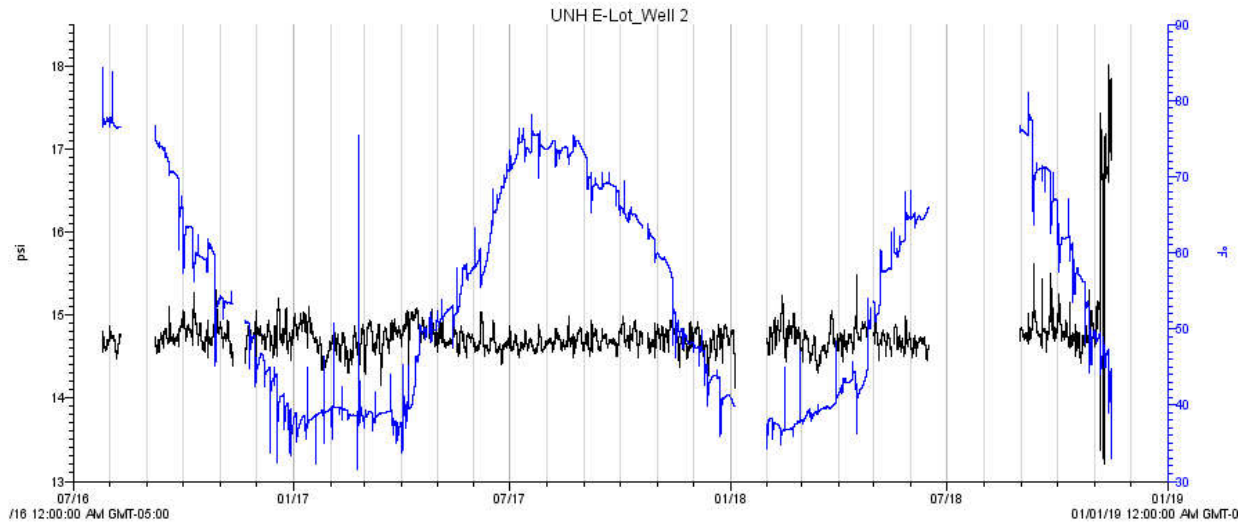
Figure 96 – Well 2 data without barometric compensation

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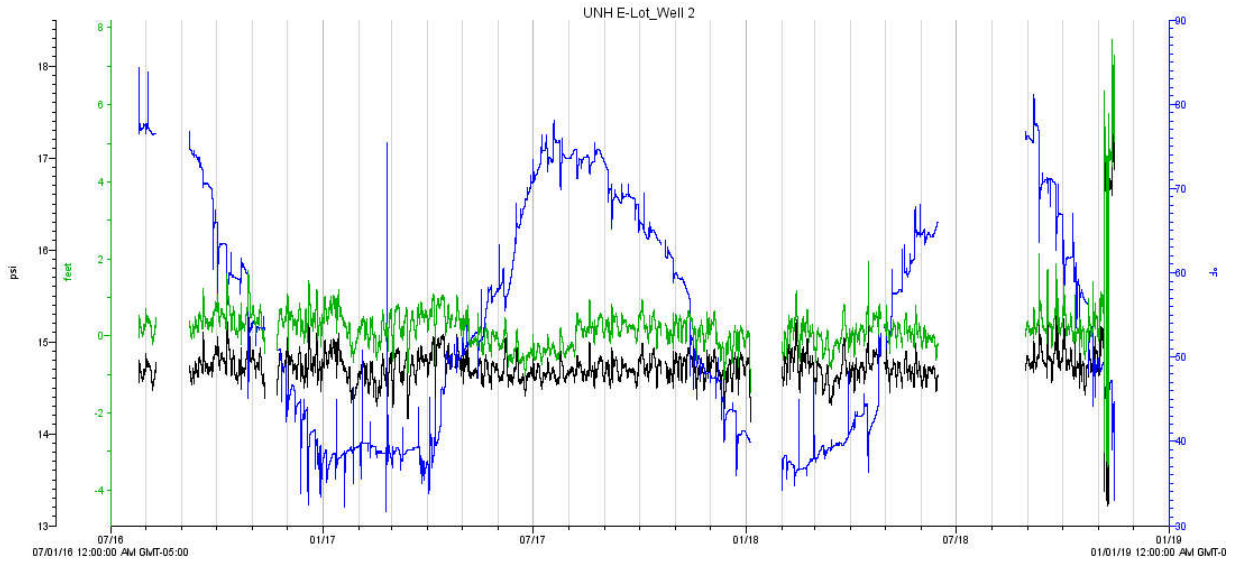
Figure 97 – Well 2 data with barometric compensation

A.6 Volume of Water in Storage

A.6.1 Volume of water in MW1 with initial run
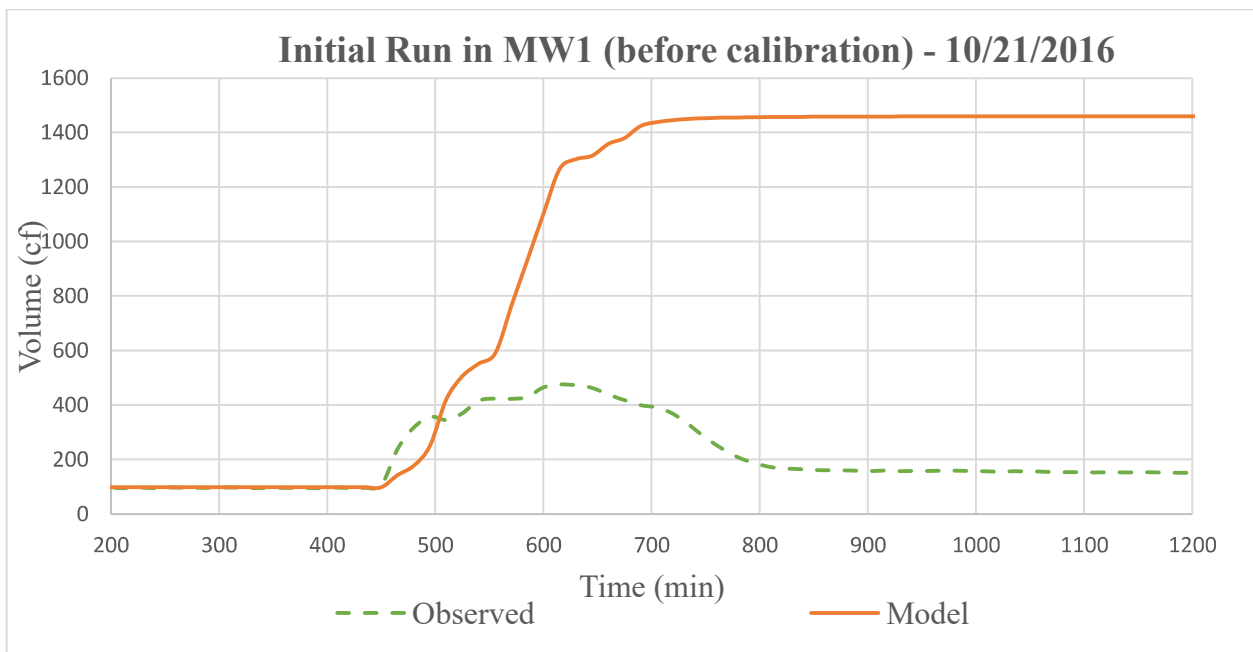
Figure 98 – Volume of Water in MW1 with initial run during and after Storm 10-21-2016
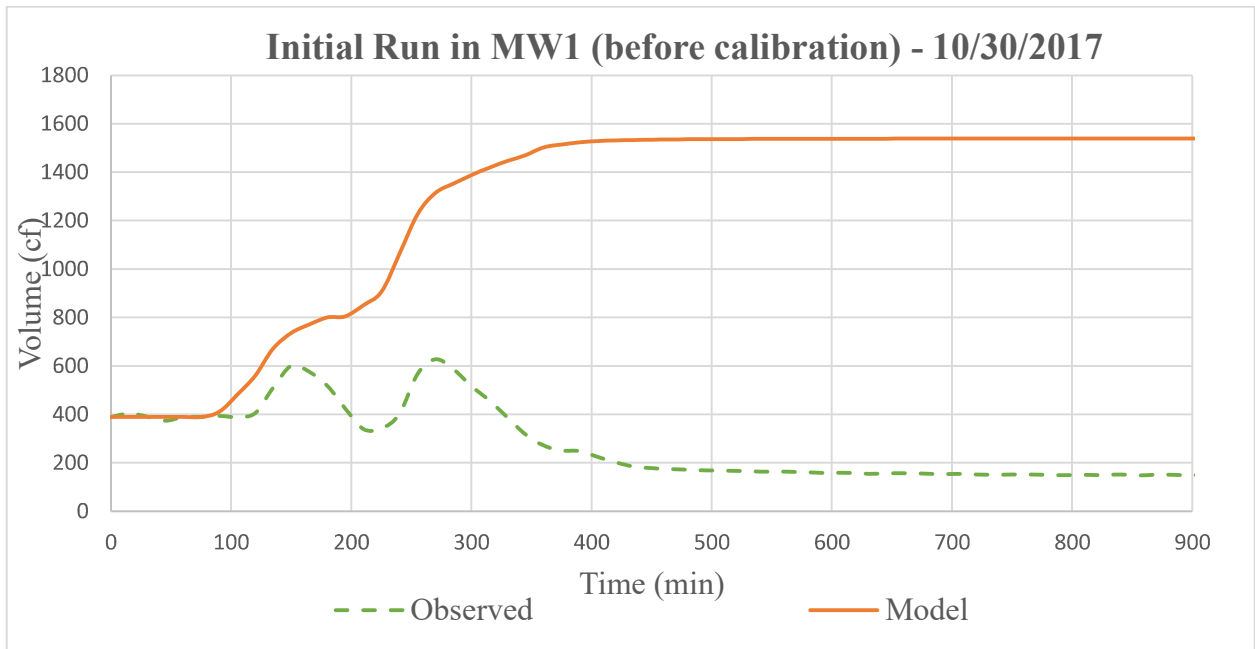
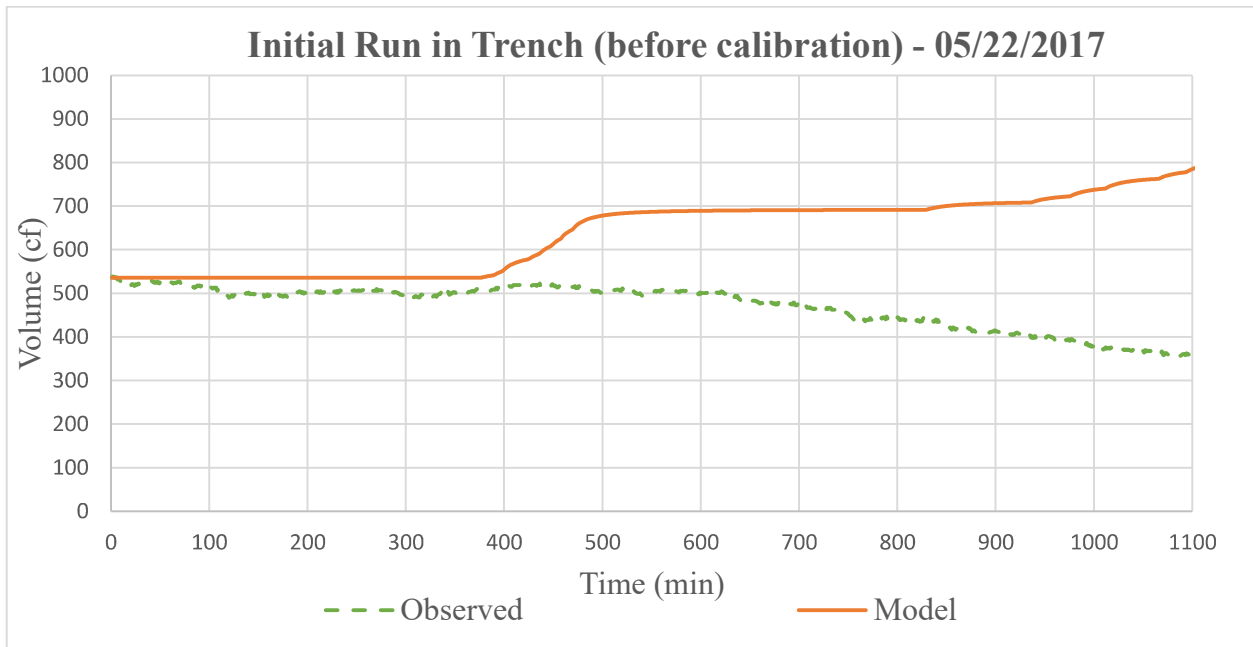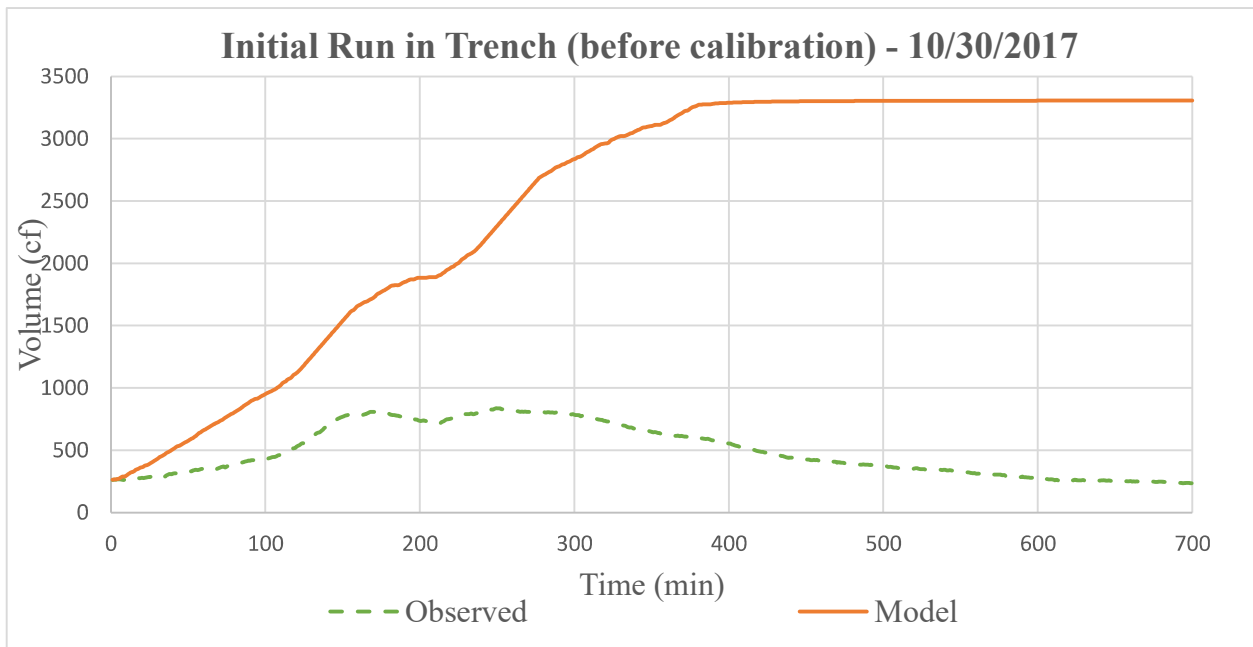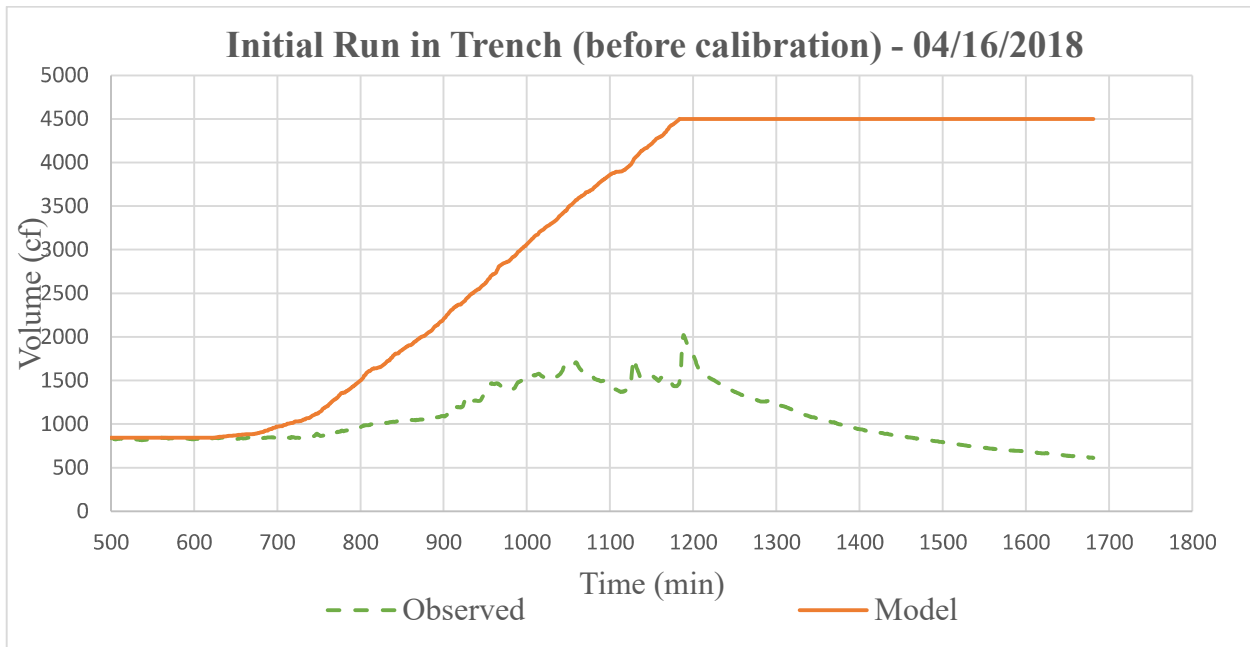Figure 99 - Volume of Water in MW1 with initial run during and after Storm 10-30-2017
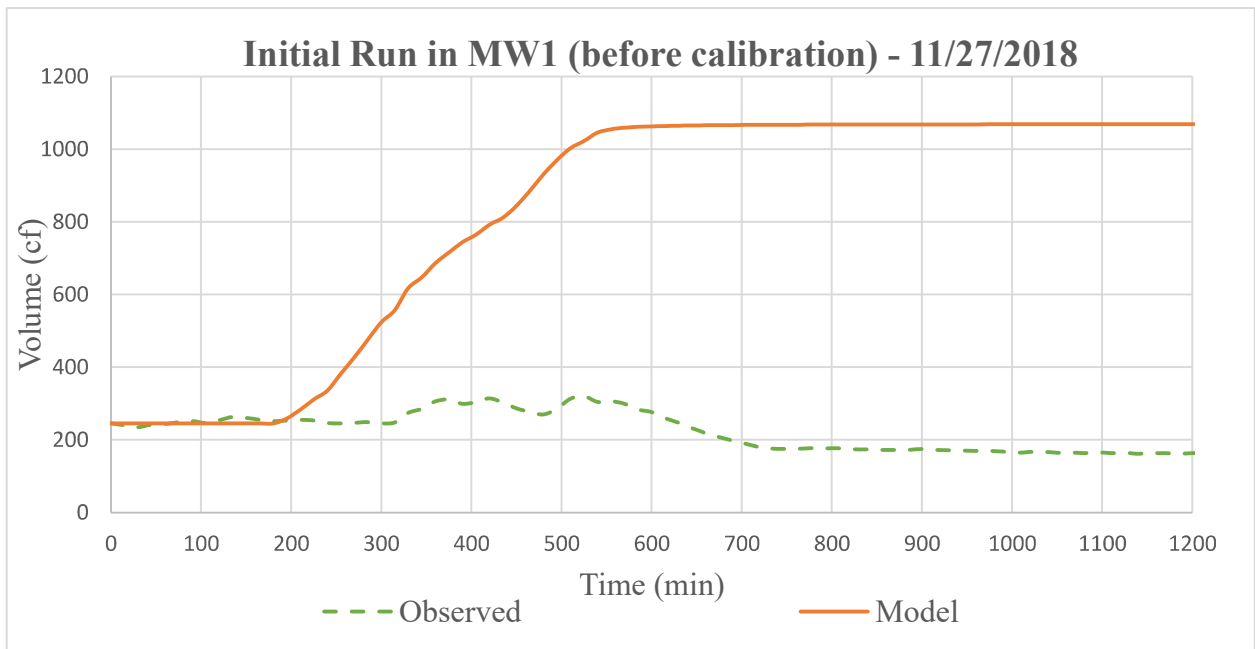
Figure 100 - Volume of Water in MW1 with initial run during and after Storm 11-27-2018
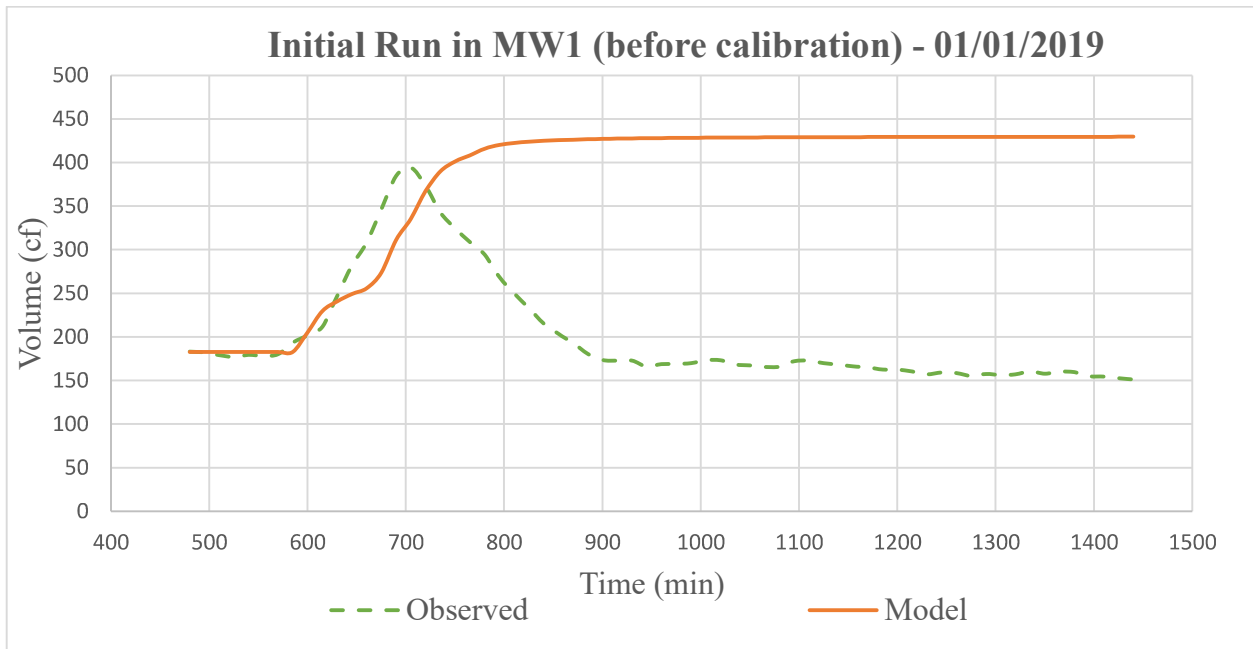
Figure 101 - Volume of Water in MW1 with initial run during and after Storm 01-01-2019
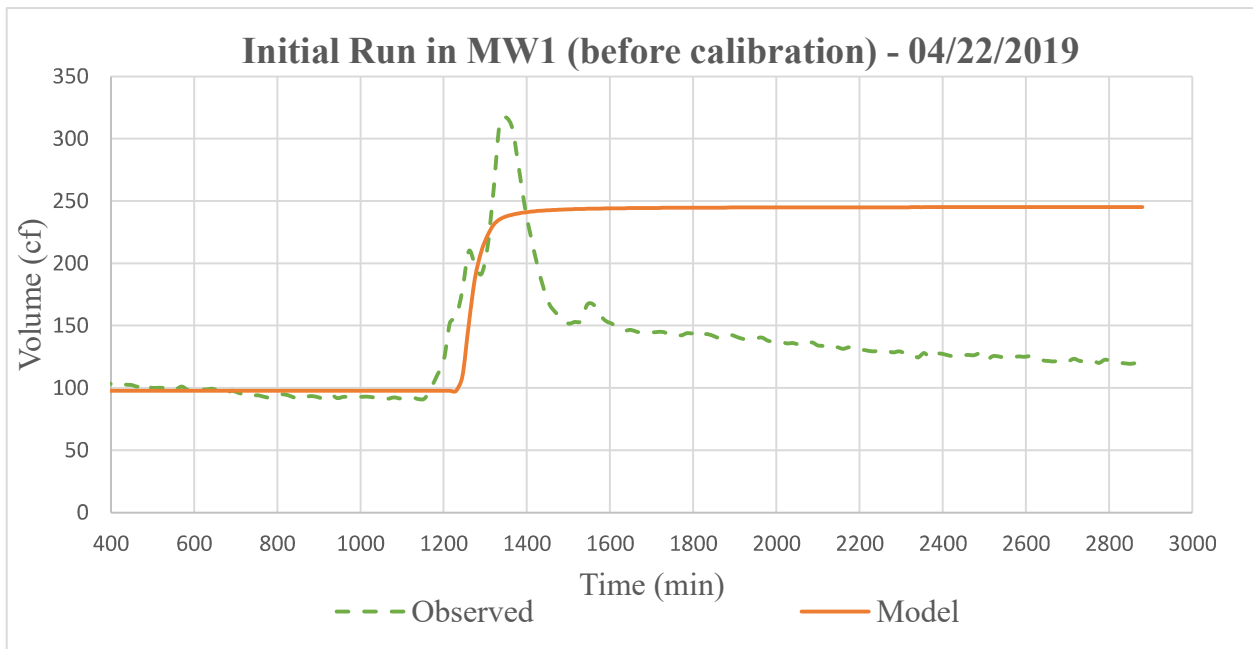
Figure 102 - Volume of Water in MW1 with initial run during and after Storm 04-22-2019

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A.6.2 Volume of water in MW1 after calibration
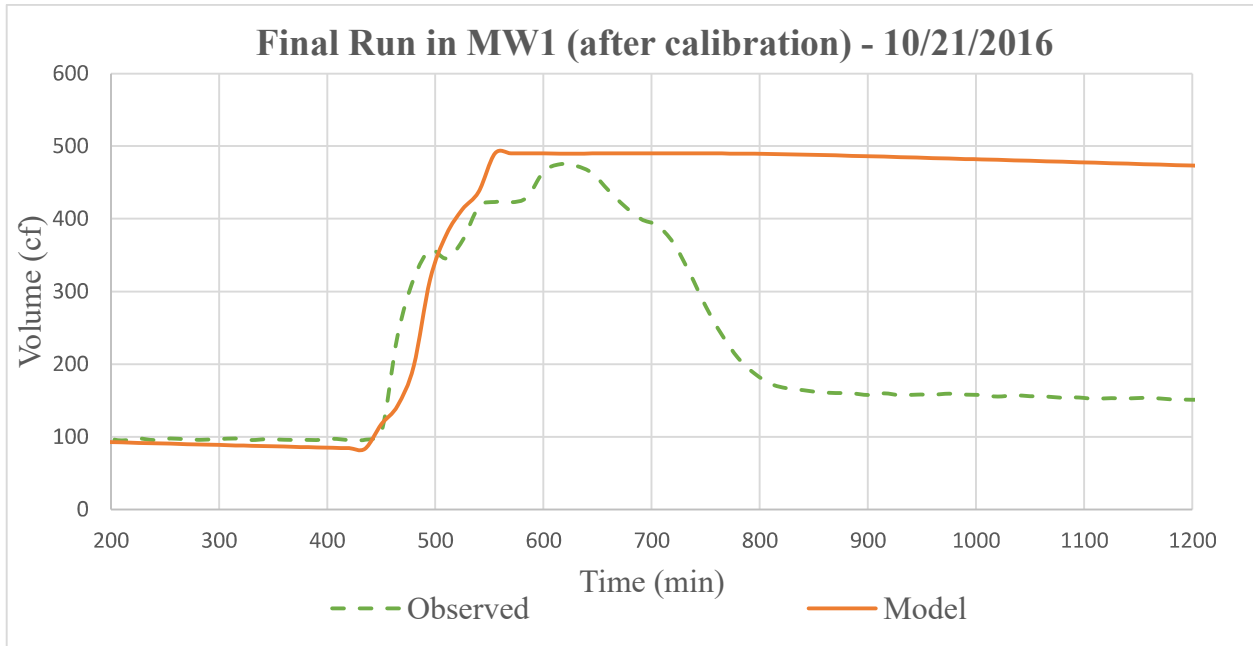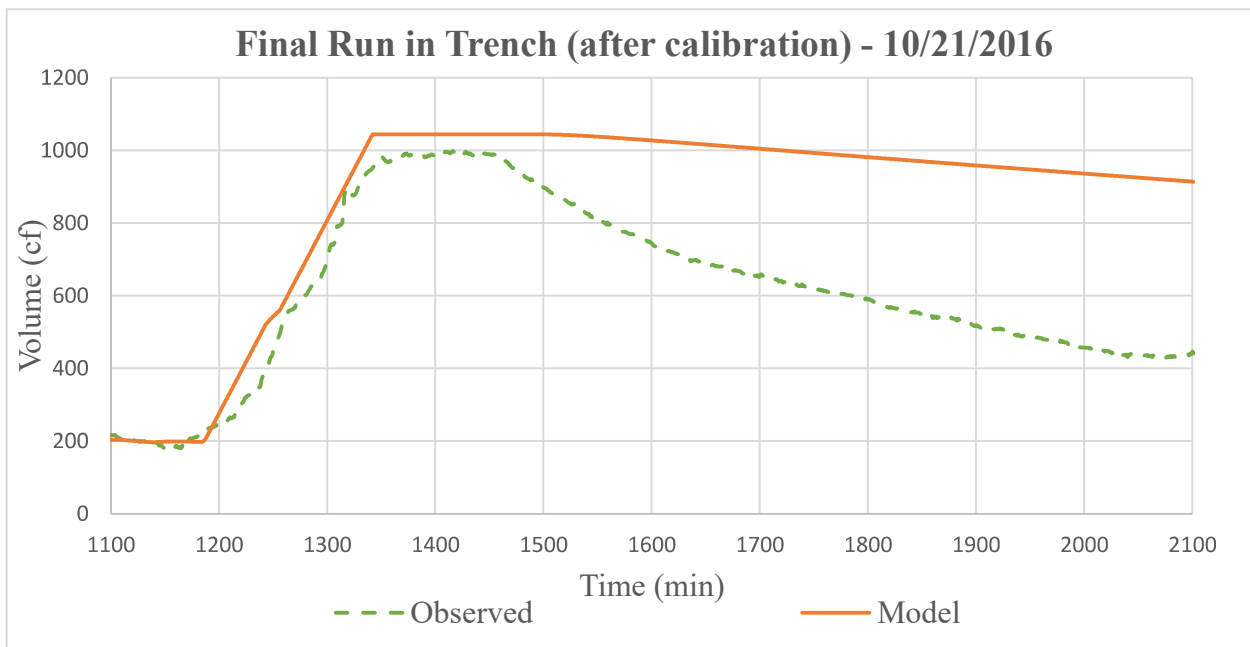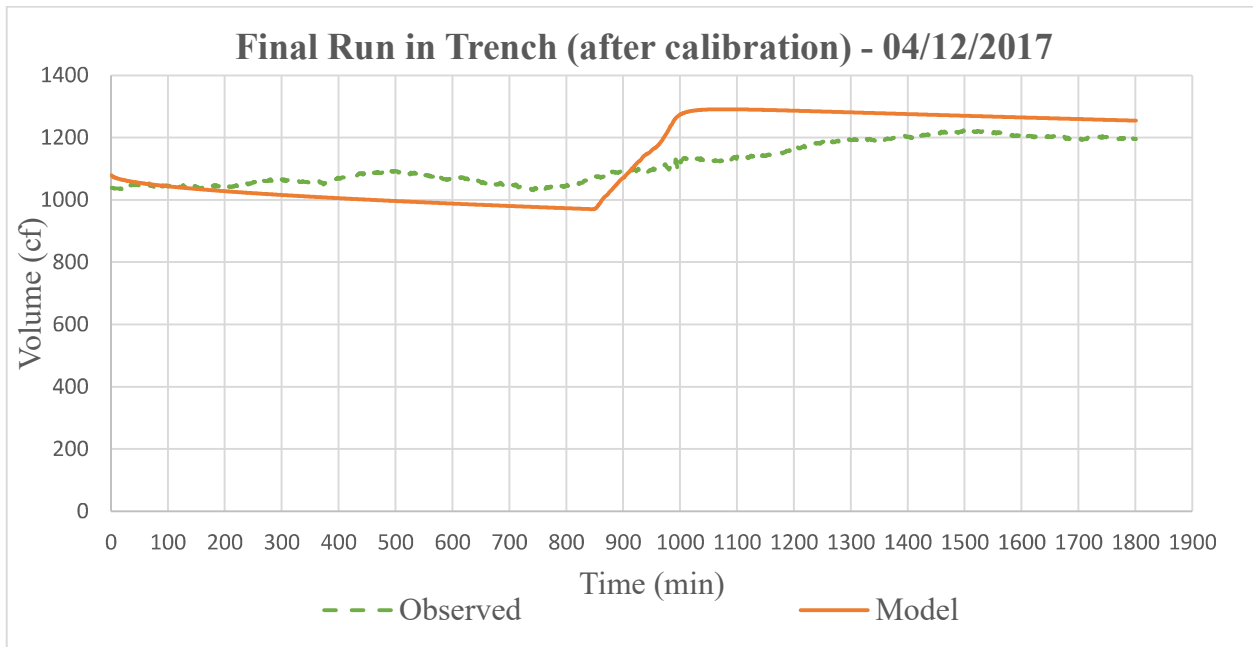

Figure 103 - Volume of Water in MW1 after calibration during and after Storm 10-21-2016
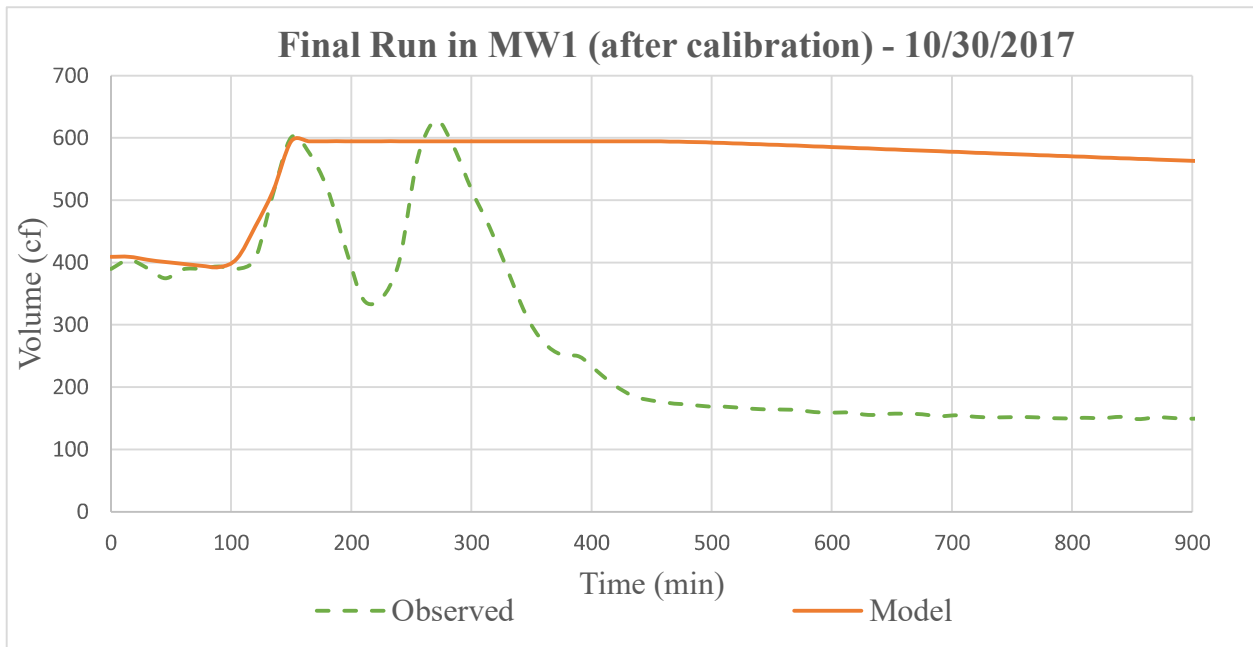
Figure 104 - Volume of Water in MW1 after calibration during and after Storm 10-30-2017
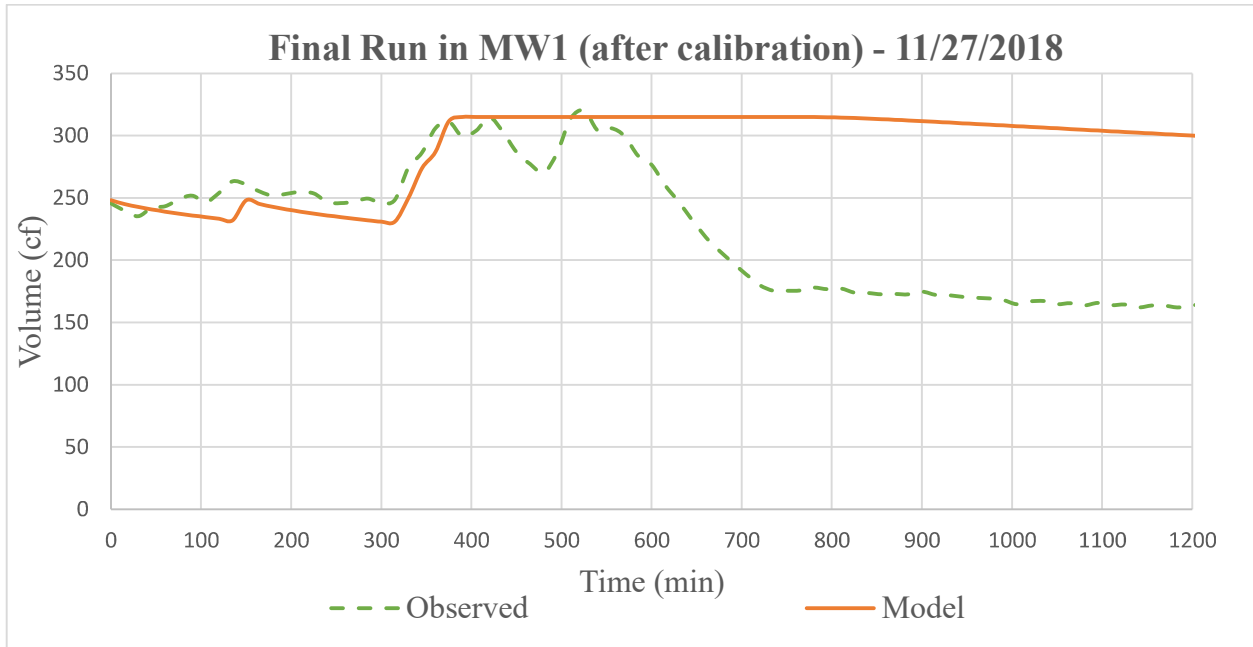
Figure 105 - Volume of Water in MW1 after calibration during and after Storm 11-27-2018
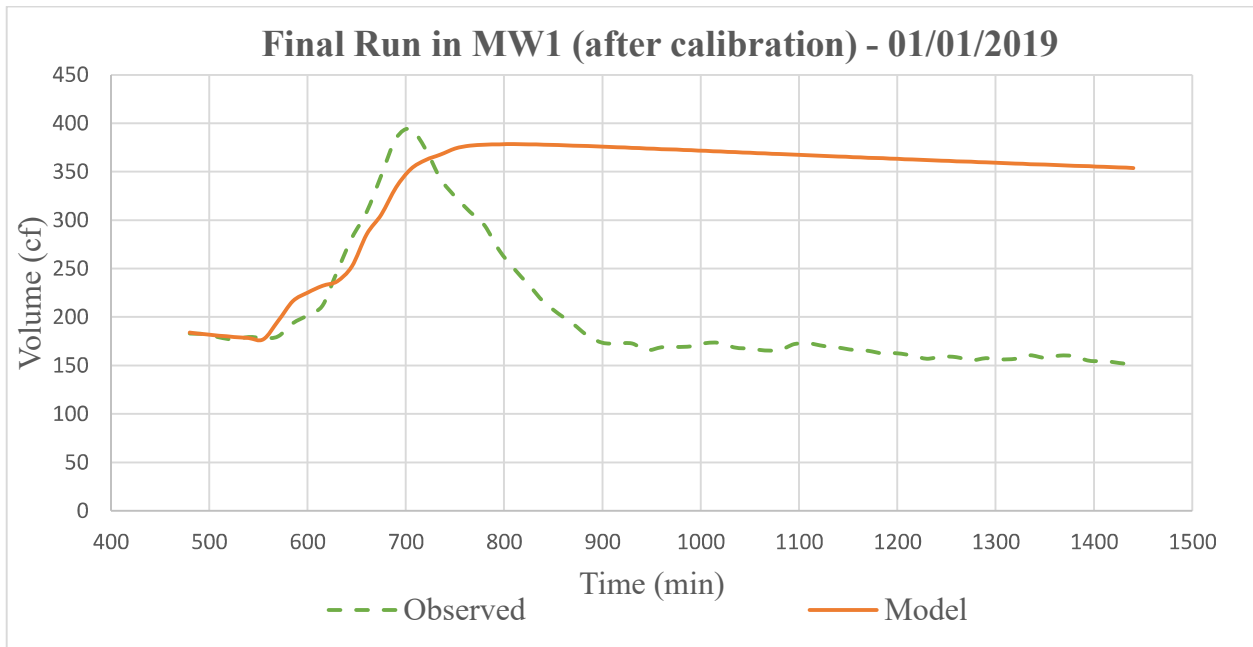
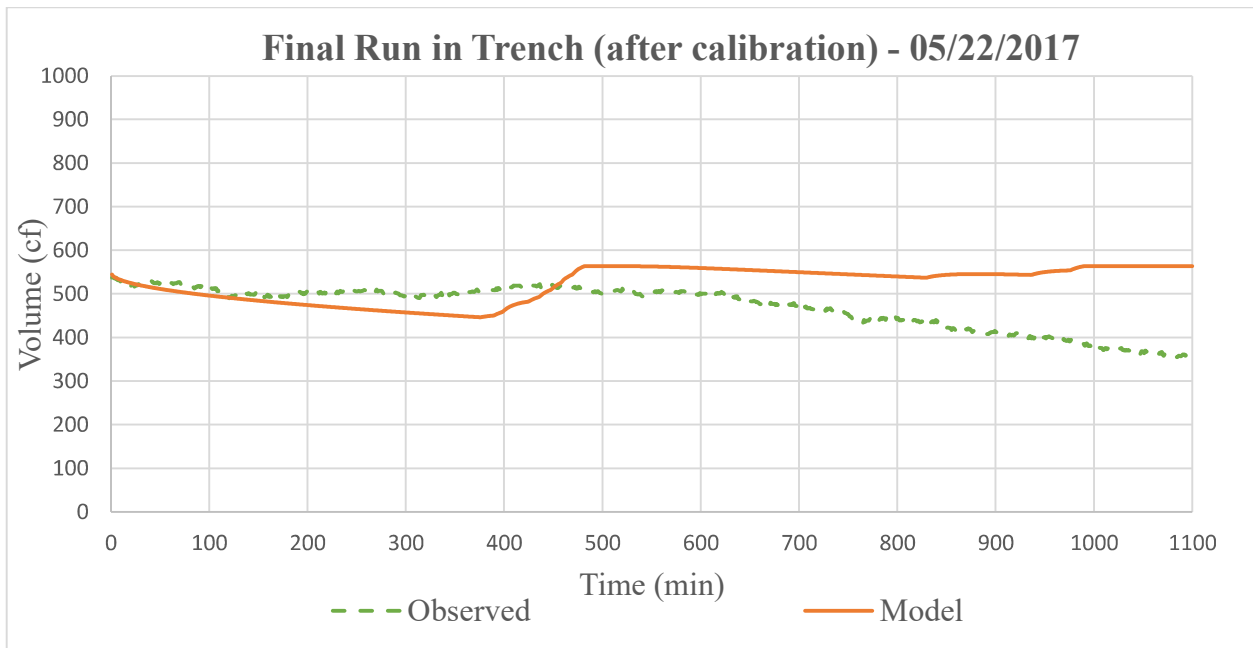Figure 106 - Volume of Water in MW1 after calibration during and after Storm 01-01-2019

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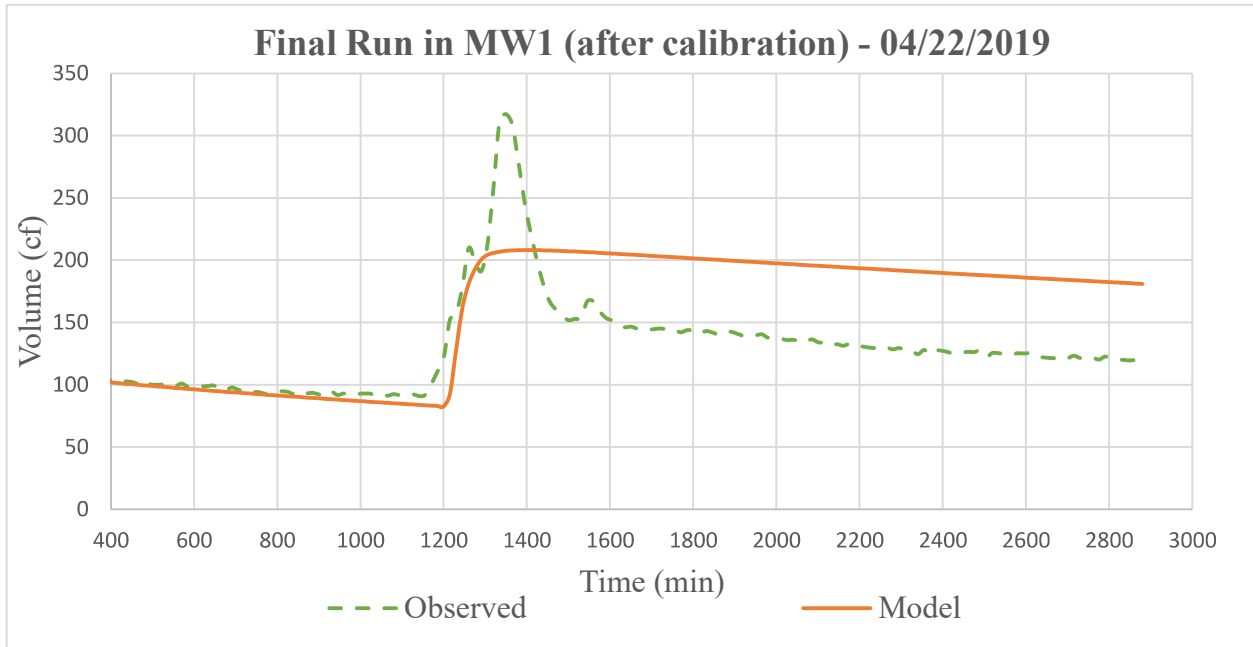
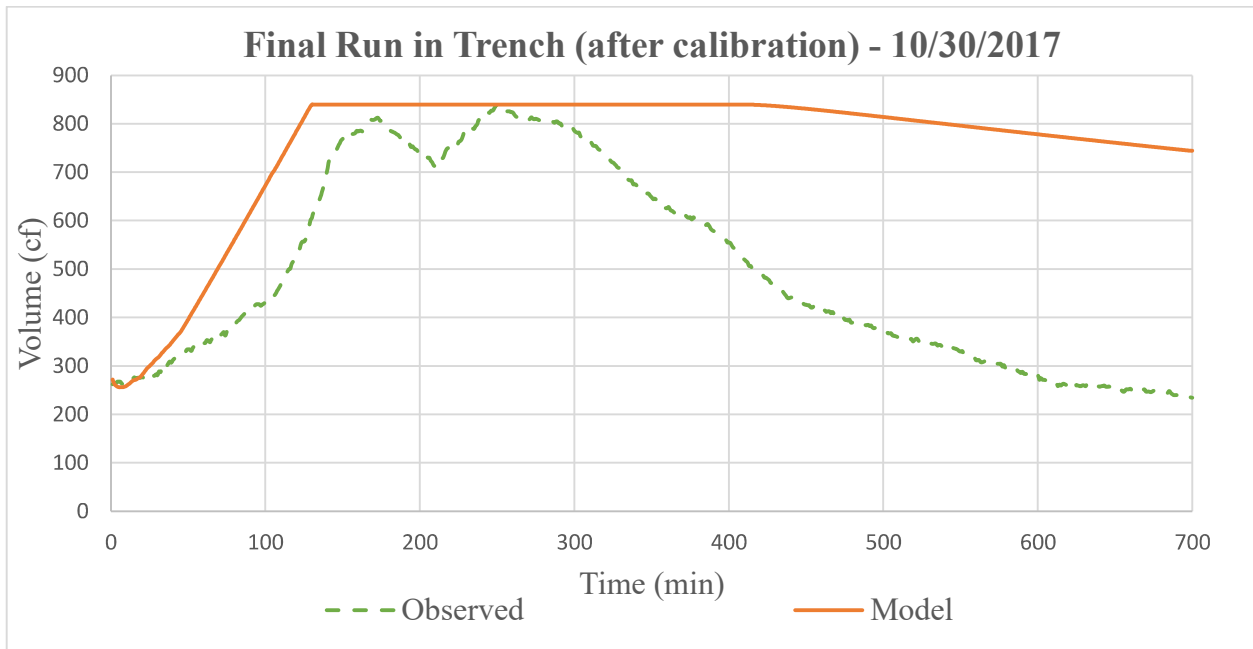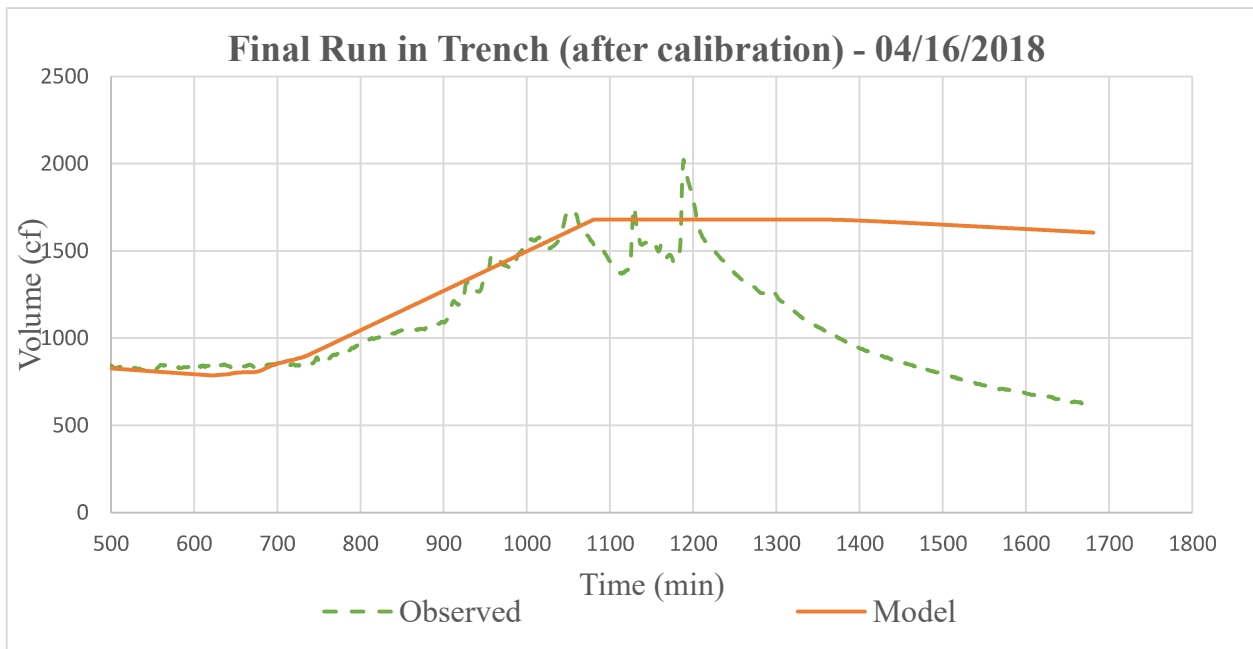Figure 107 - Volume of Water in MW1 after calibration during and after Storm 04-22-2019

A.6.3 Volume of water in MW1 with fictitious underdrain
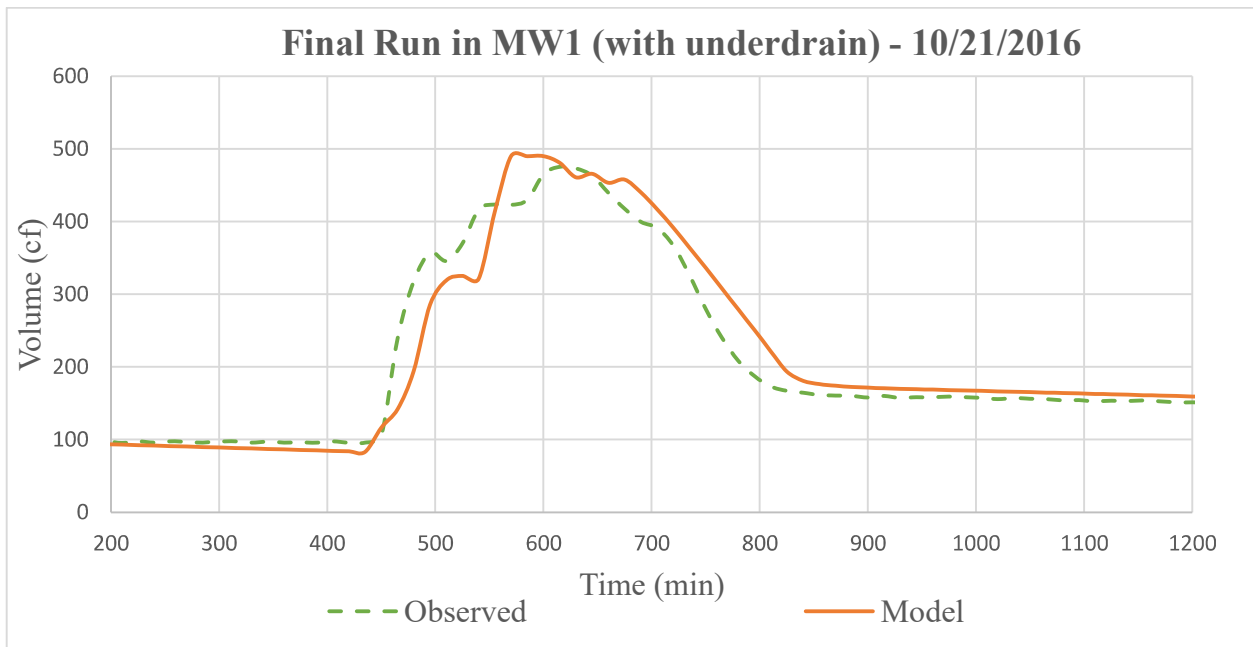
Figure 108 - Volume of Water in MW1 with fictitious underdrain during and after Storm 10-21-

2016

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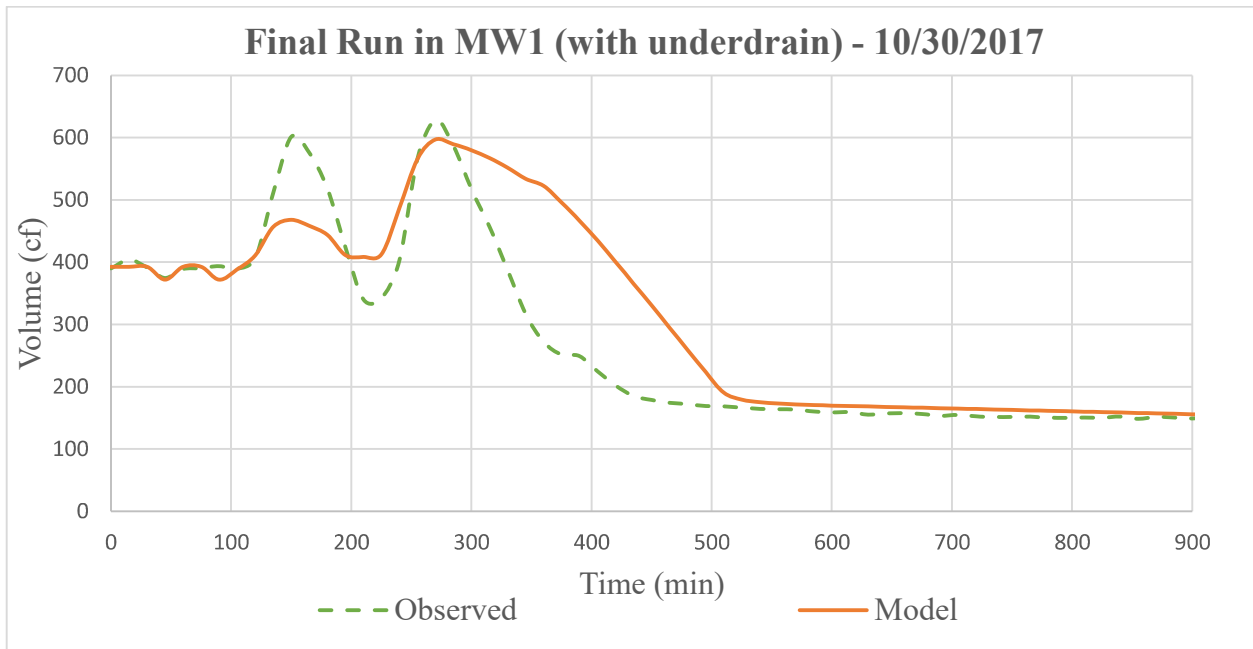
Figure 109 - Volume of Water in MW1 with fictitious underdrain during and after Storm 10-30-

2017
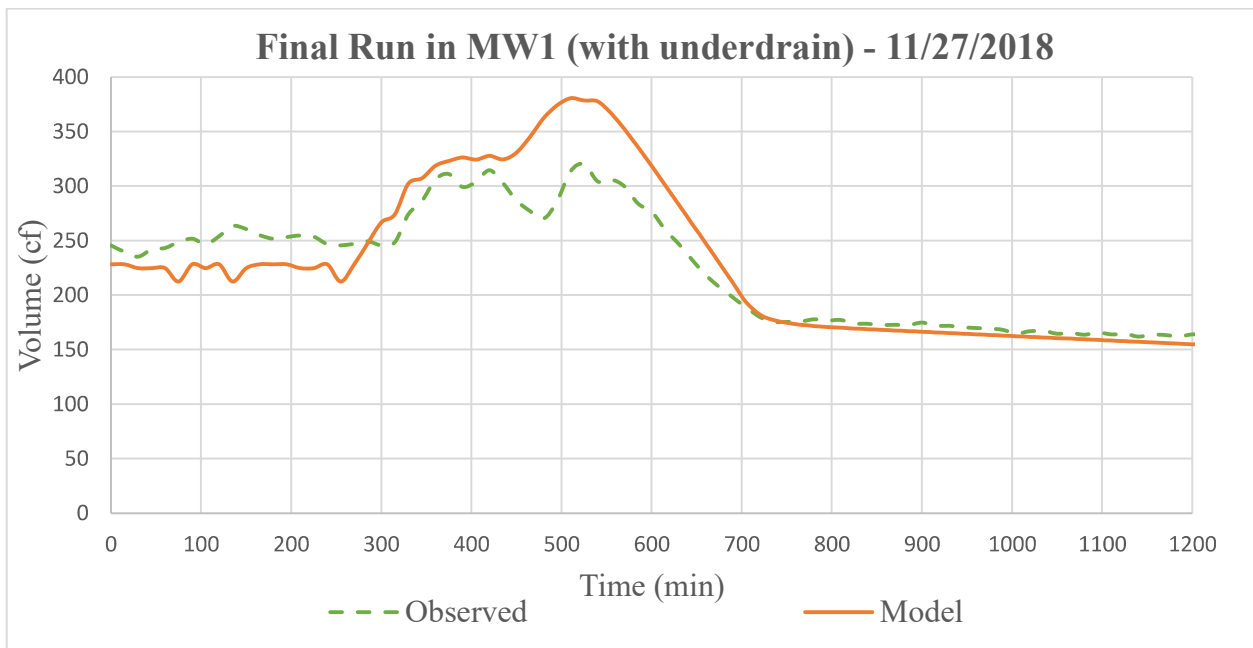
Figure 110 - Volume of Water in MW1 with fictitious underdrain during and after Storm 11-27-

2018
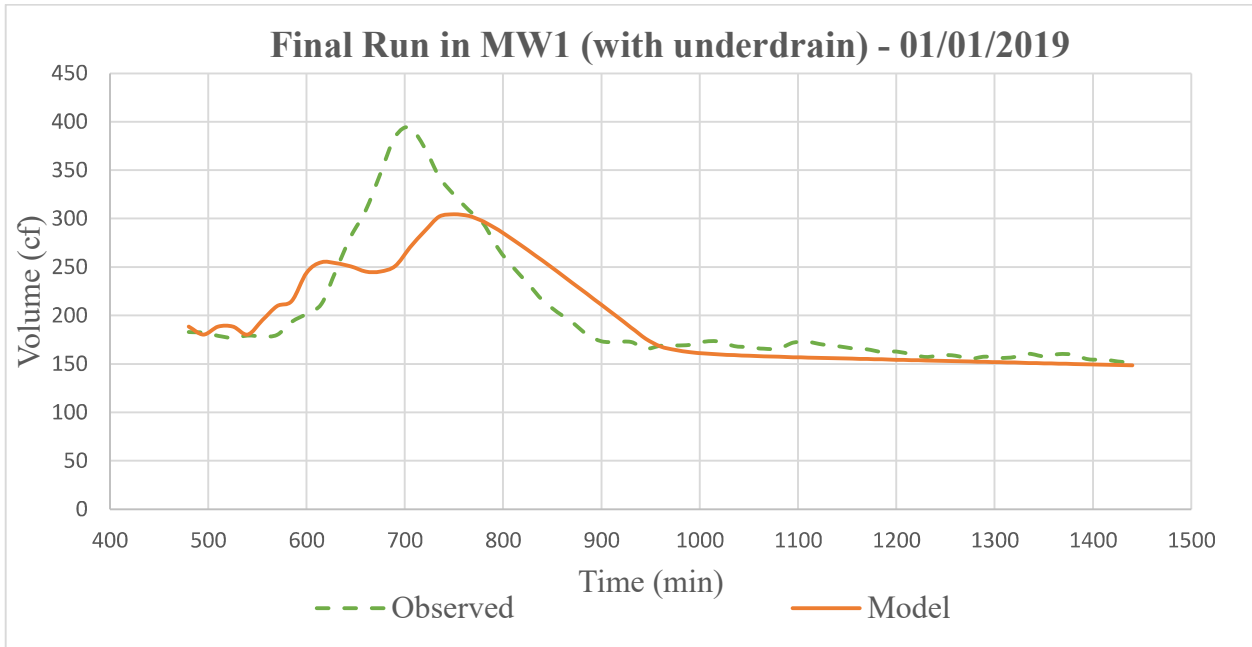
Figure 111 - Volume of Water in MW1 with fictitious underdrain during and after Storm 01-01-
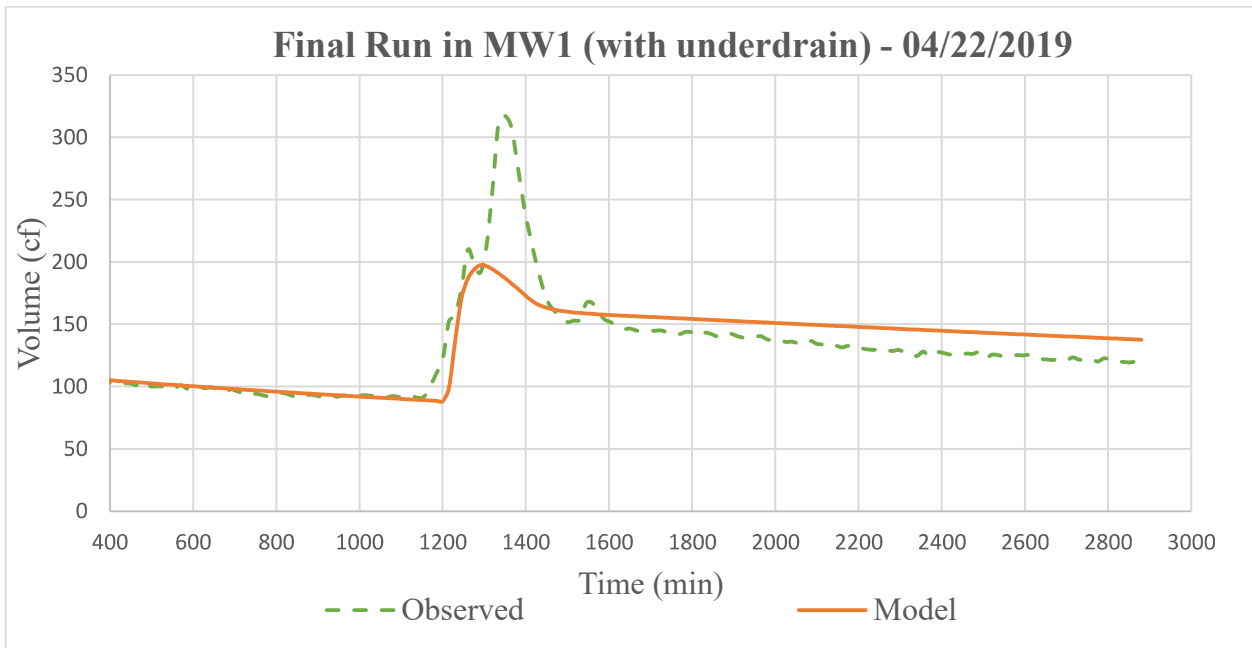
2019

Figure 112 - Volume of Water in MW1 with fictitious underdrain during and after Storm 04-22-

2019

A.6.4 Volume of water in Trench with initial run
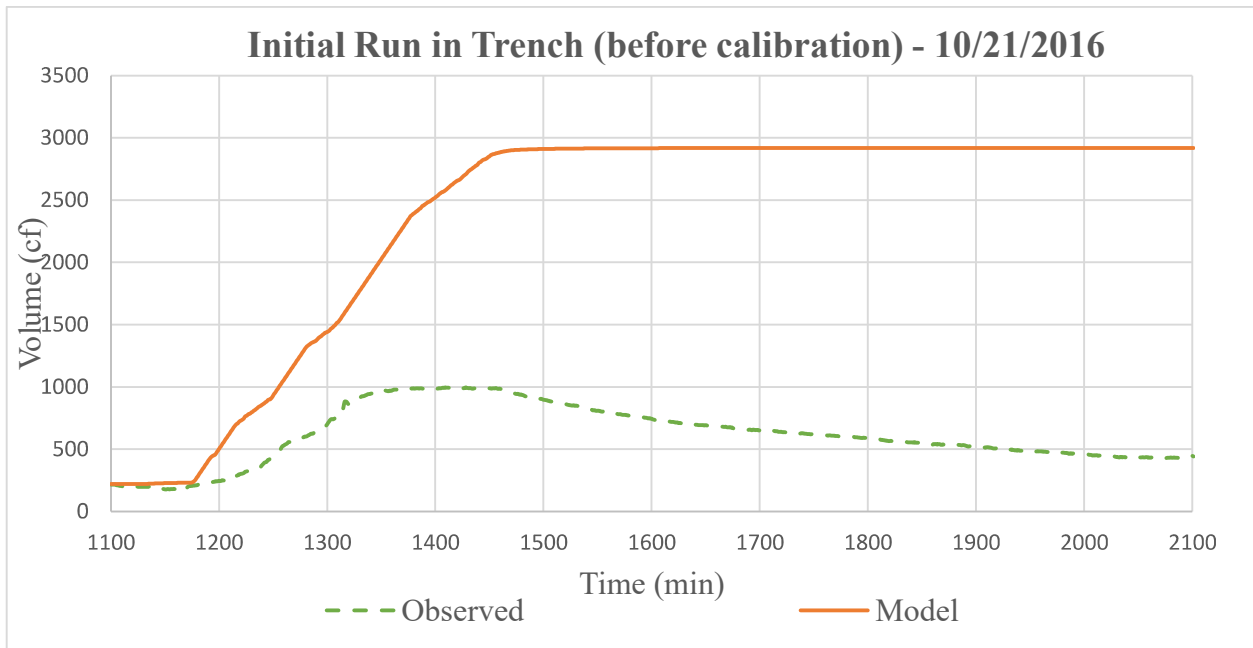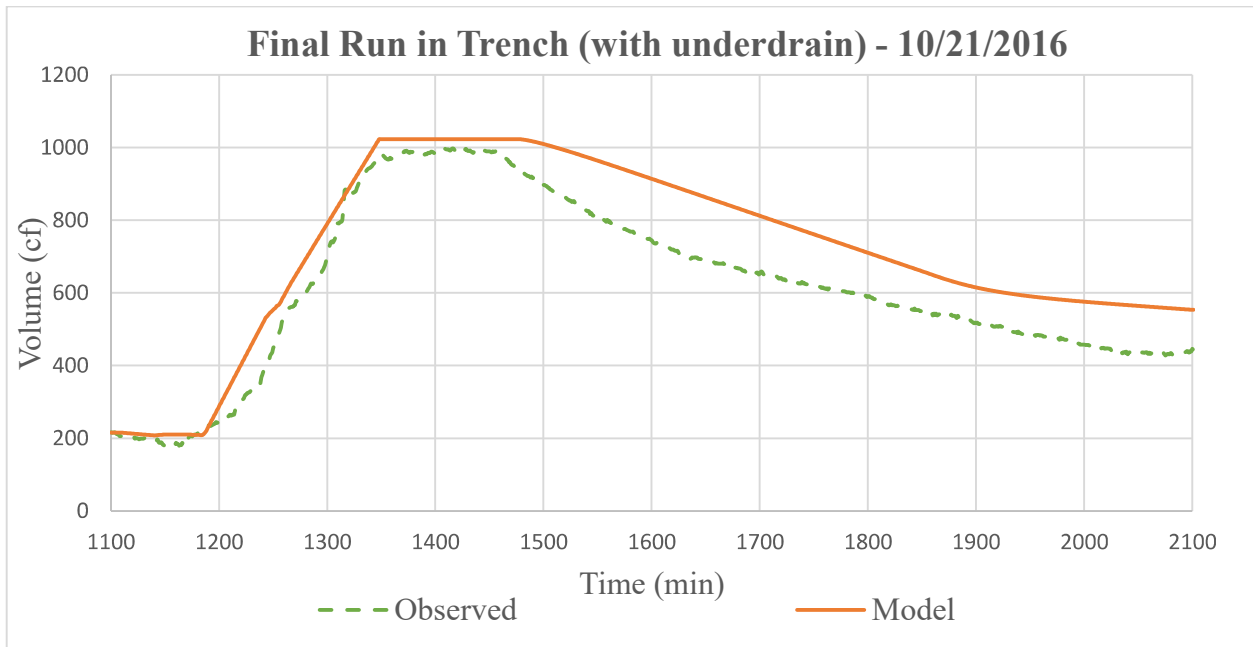

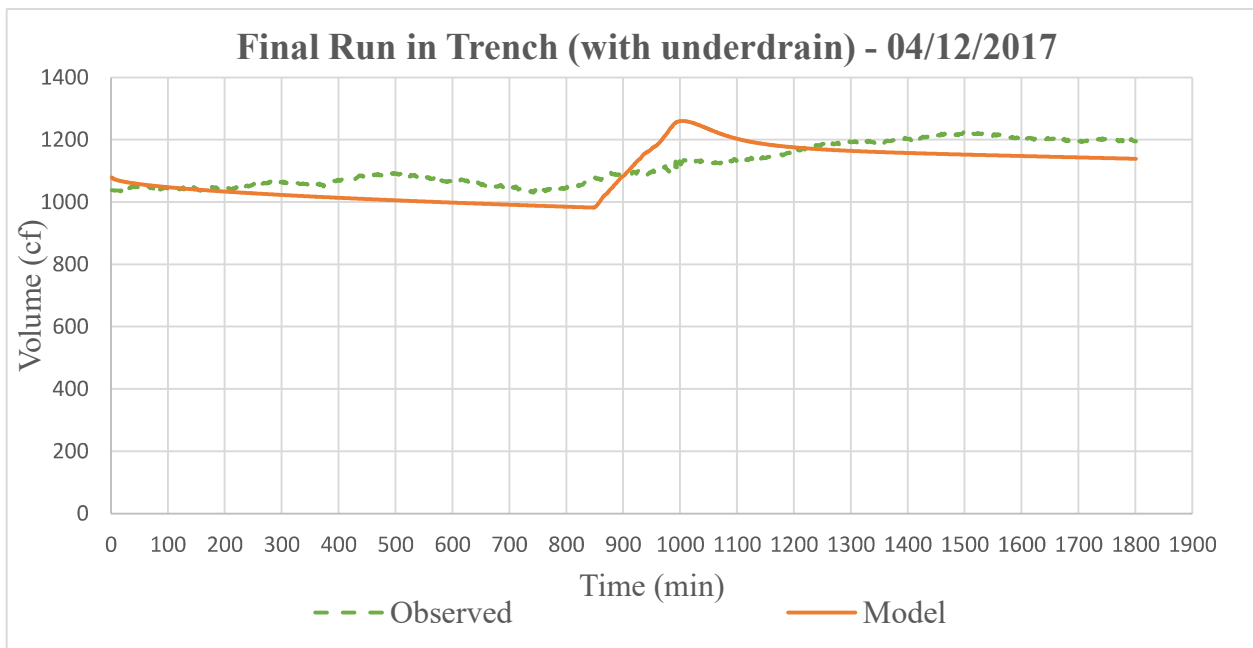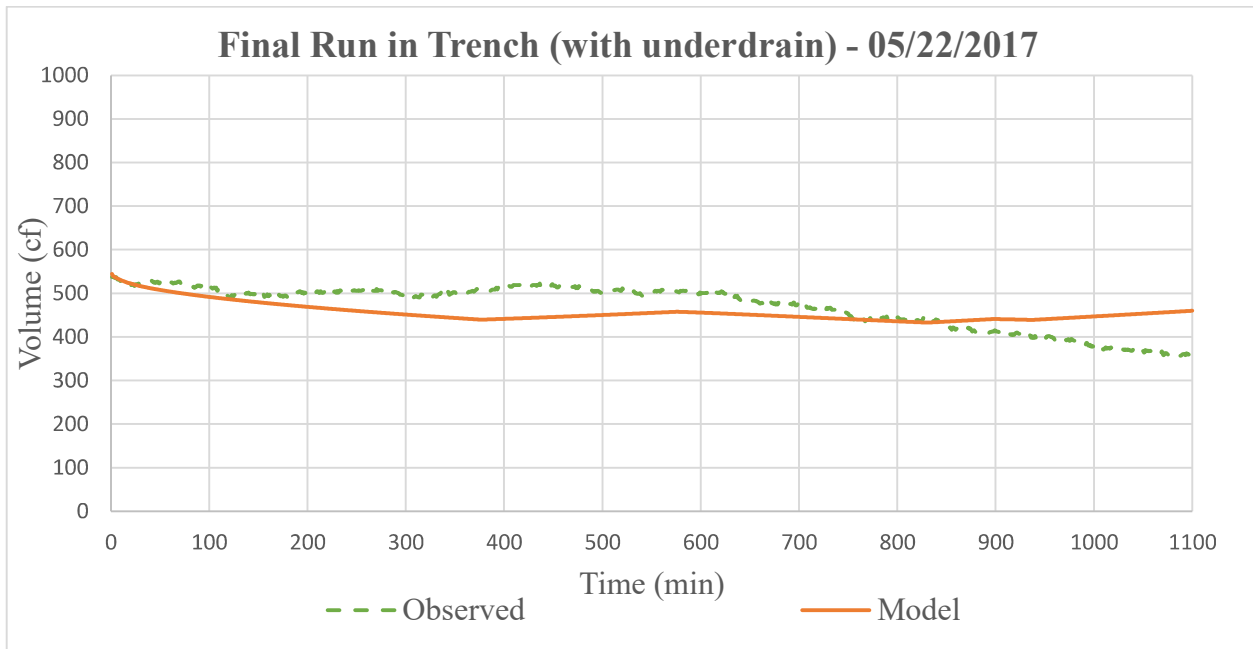Figure 113 - Volume of Water in Trench with initial run during and after Storm 10-21-2016
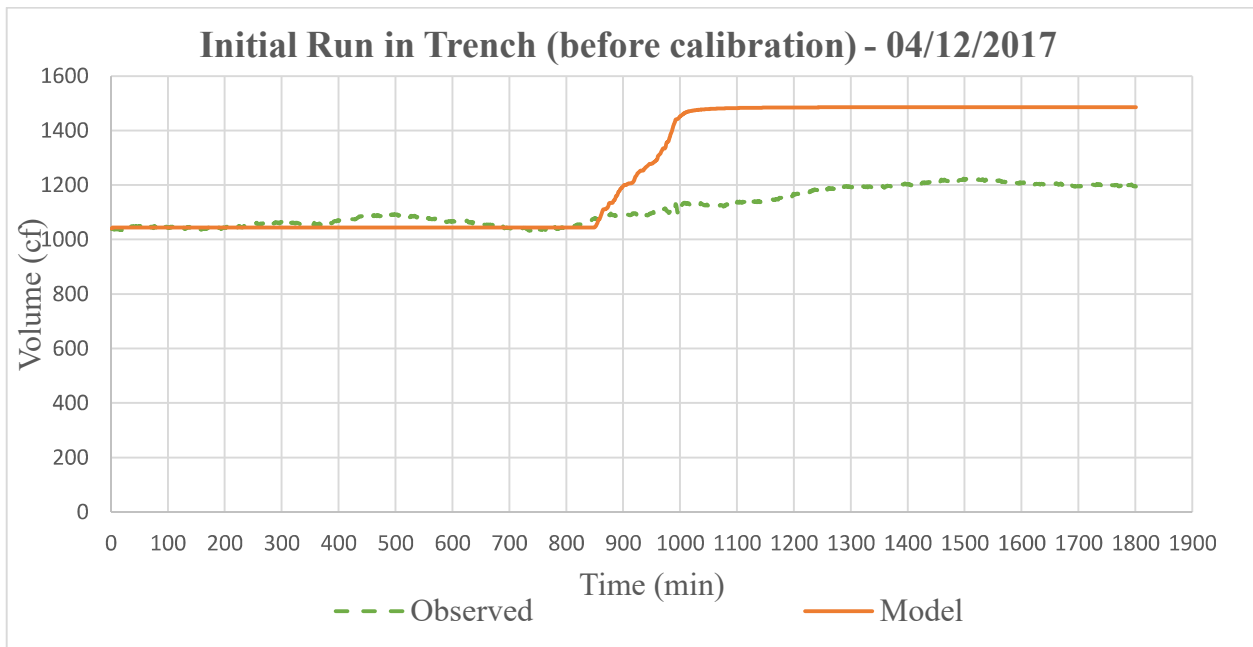
Figure 114 - Volume of Water in Trench with initial run during and after Storm 04-12-2017

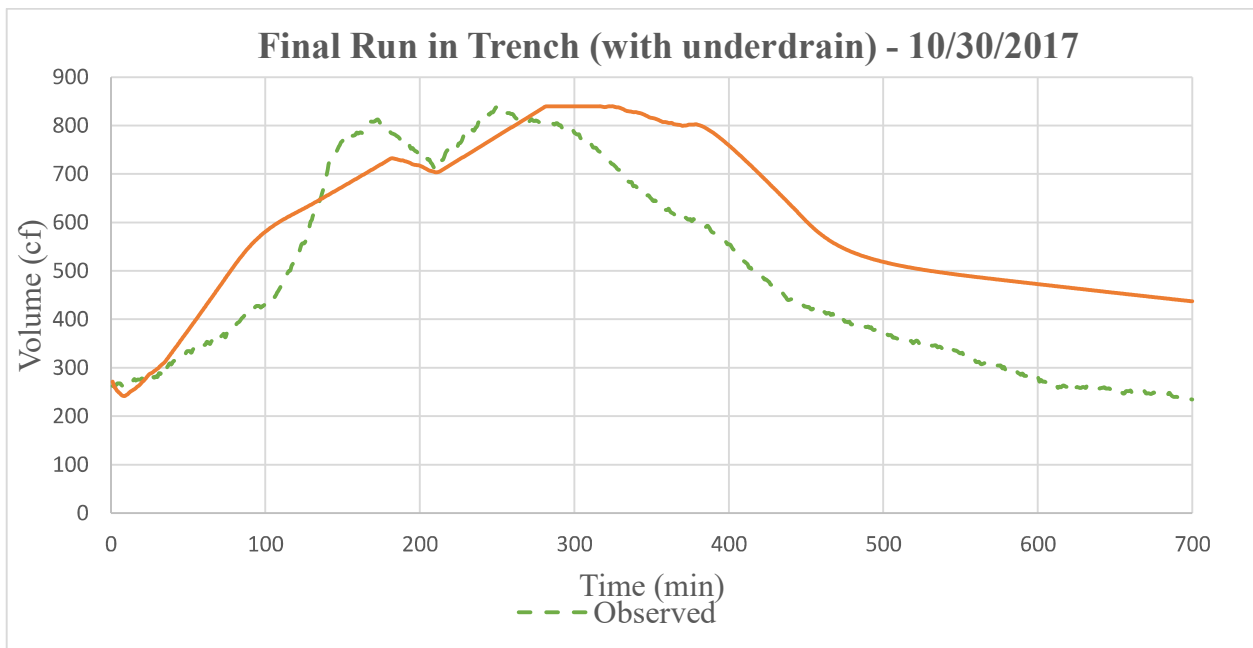Figure 115 - Volume of Water in Trench with initial run during and after Storm 05-22-2017

Figure 116 - Volume of Water in Trench with initial run during and after Storm 10-30-2017

Figure 117 - Volume of Water in Trench with initial run during and after Storm 04-16-2016

A.6.5 Volume of water in Trench after calibration

Figure 118 - Volume of Water in Trench after calibration during and after Storm 10-21-2016

Figure 119 - Volume of Water in Trench after calibration during and after Storm 04-12-2017

Figure 120 - Volume of Water in Trench after calibration during and after Storm 05-22-2017

Figure 121 - Volume of Water in Trench after calibration during and after Storm 10-30-2017

Figure 122 - Volume of Water in Trench after calibration during and after Storm 04-16-2018

A.6.6 Volume of water in Trench with fictitious underdrain

Figure 123 - Volume of Water in Trench with fictitious underdrain during and after Storm 10-21-

2016

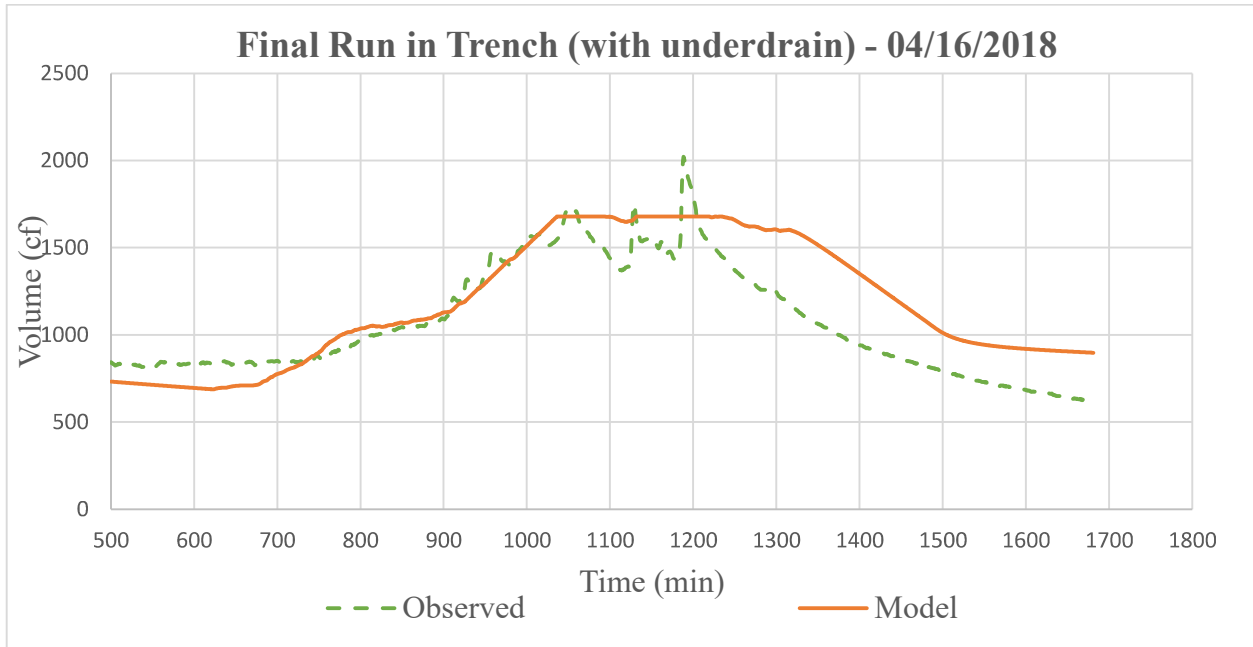Figure 124 - Volume of Water in Trench with fictitious underdrain during and after Storm 04-12-
2017

Figure 125 - Volume of Water in Trench with fictitious underdrain during and after Storm 05-22-2017

Figure 126 - Volume of Water in Trench with fictitious underdrain during and after Storm 10-30-2017

Figure 127 - Volume of Water in Trench with fictitious underdrain during and after Storm 04-16-2018

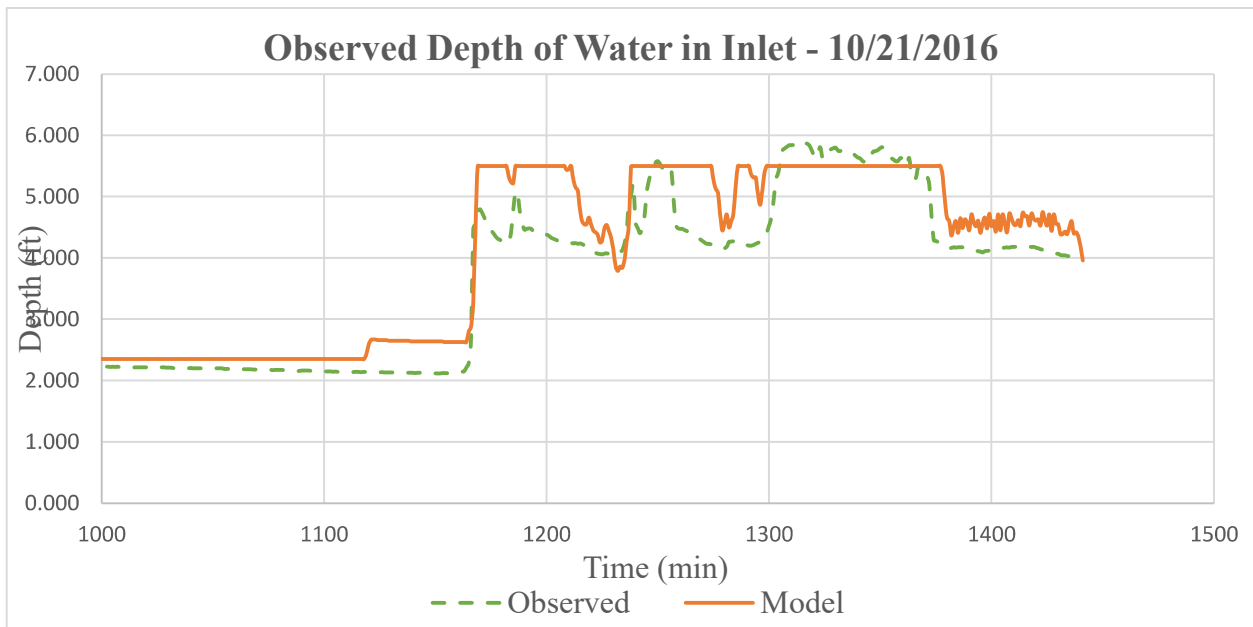**A.6.7 Observed and Modeled depth of water in Inlet**

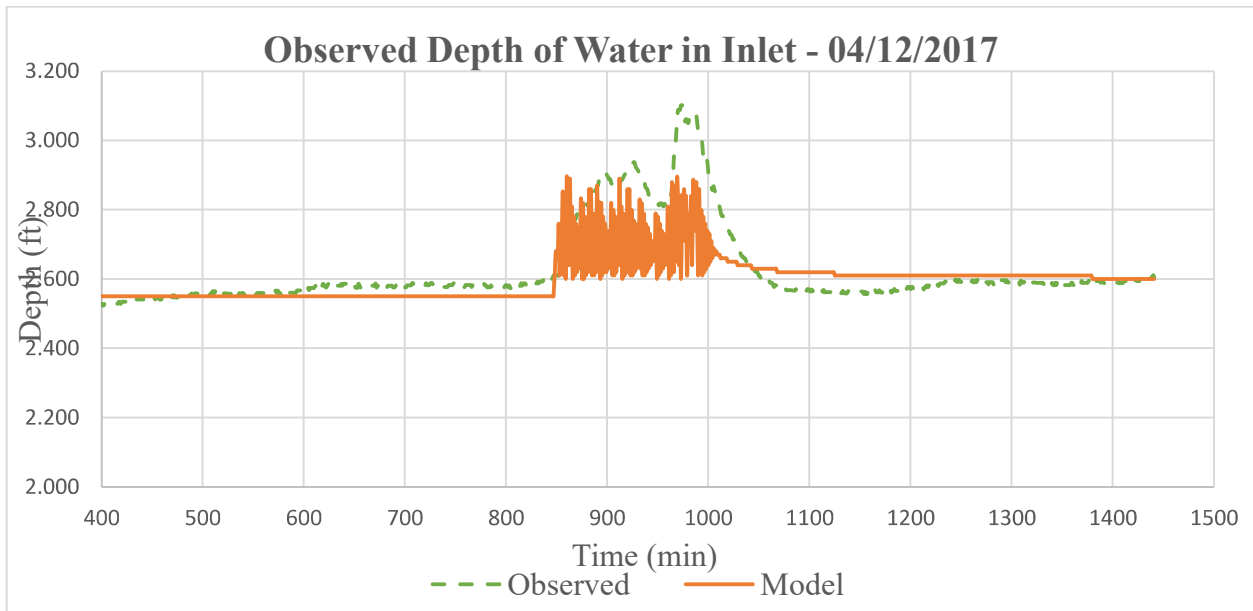Figure 128 – Observed and Modeled Depth of Water in Inlet during and after Storm 10-21-2016

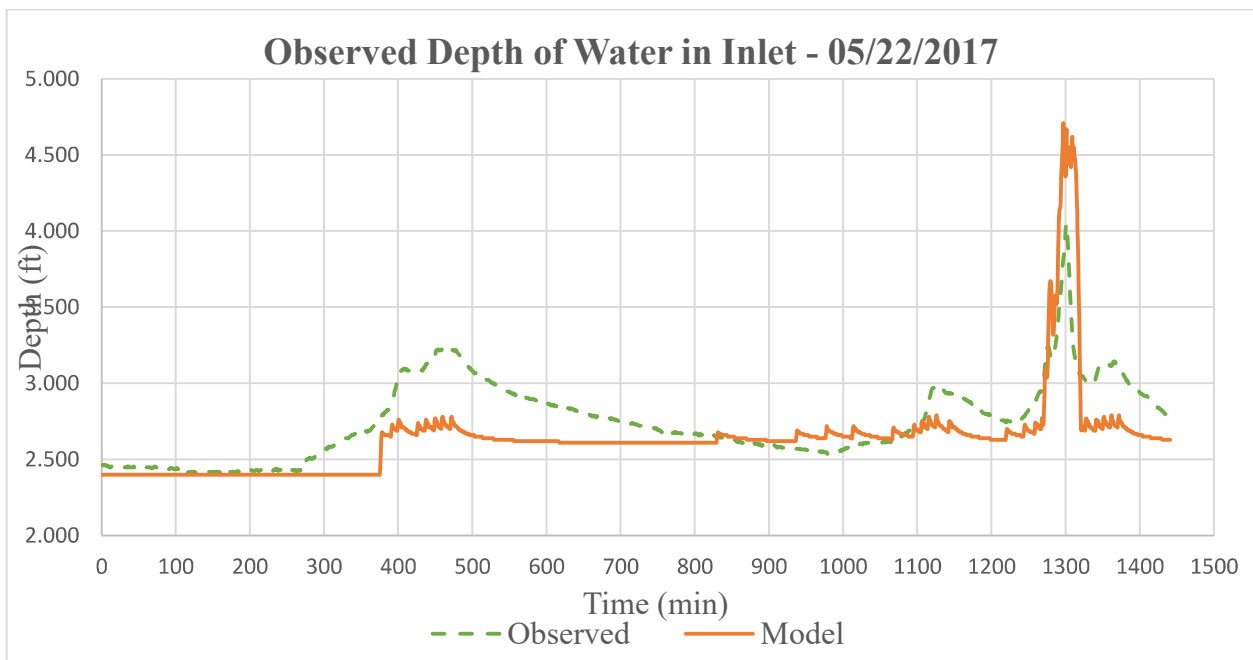Figure 129 - Observed and Modeled Depth of Water in Inlet during and after Storm 04-12-2017

Figure 130 - Observed and Modeled Depth of Water in Inlet during and after Storm 05-22-2017
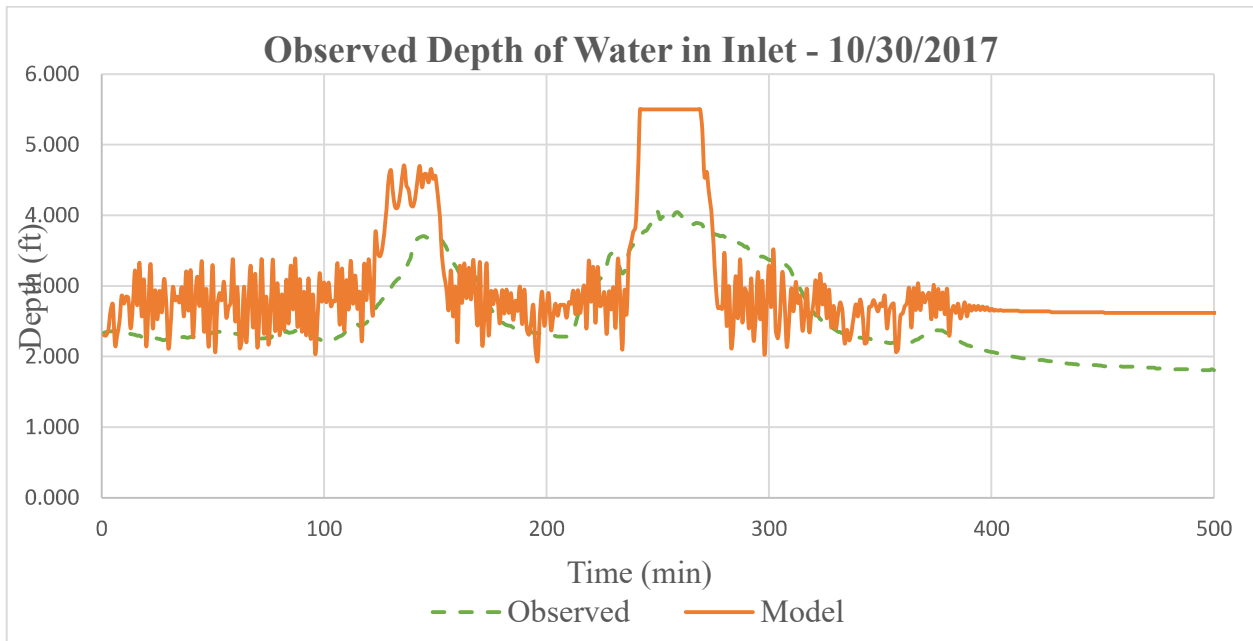
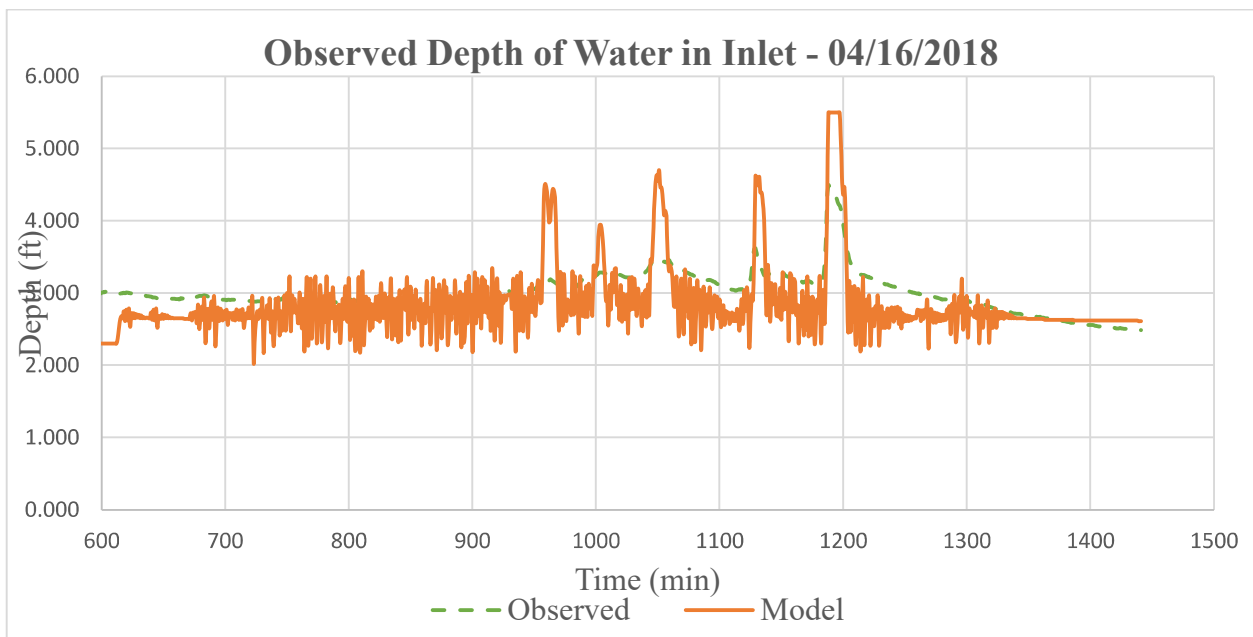Figure 131 - Observed and Modeled Depth of Water in Inlet during and after Storm 10-30-2017

Figure 132 - Observed and Modeled Depth of Water in Inlet during and after Storm 04-16-2018

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