

Appendix A

Tookany/Tacony-Frankford Creek SWMM Validation

A.1 Introduction

This appendix describes the development and validation of the Tookany/Tacony-Frankford Creek Tributary H&H Model used to provide the hydraulic and bacteria loadings to the United States Environmental Protection Agency (US EPA) Water Quality Analysis Simulation Program (WASP) model.

The US EPA Storm Water Management Model (SWMM) version 5 was used to develop the Combined Sewer System (CSS) Model and Watershed Model that comprise the Tributary Hydraulic and Hydrologic (H&H) Model. SWMM hydrology, represented by subcatchments, simulated both the quantity and quality of runoff in a drainage basin and the routing of flows and contaminants to sewers or receiving waters. SWMM hydrology can accept precipitation (rainfall or snowfall) hyetographs and perform a step by step accounting of snowmelt, infiltration losses in pervious areas, surface detention, overland flow, and water quality constituents leading to the calculation of one or more hydrographs and/or pollutographs at a certain geographic point such as a sewer inlet. SWMM hydraulics, represented by nodes and links, simulate dynamic hydraulic flow routing and pollutant routing through open channel and closed conduit systems (US EPA, 2010).

The Tookany/Tacony-Frankford Creek Tributary H&H Model was developed to simulate the stormwater runoff and water quality loading from combined sewer overflows (CSOs) and minor tributaries to the receiving waters. The model was developed primarily utilizing information obtained through previous modeling efforts and Geographic Information Systems (GIS), and was driven using continuous radar rainfall time series. Event mean concentrations (EMCs) were used to predict the stormwater quality components of the model. Flow validations were performed using monitoring records from United States Geological Survey (USGS) gaging stations. The model validation period of the Tributary H&H Model was dictated by available USGS streamflow and radar rainfall data.

The following sections of this report further describe the models used in development and validation of the Tookany/Tacony-Frankford Creek Tributary H&H Model.

A.2 Discussion of Legacy Models and Reports

Several previously published models and reports were used to develop the Tookany/Tacony-Frankford Creek Tributary H&H Model. This section discusses the role of legacy publications in model development.

A.2.1 Long Term Control Plan Update Combined Sewer System (CSS) Model

The Northeast District combined sewer system of Philadelphia was originally modeled as part of the Long Term Control Plan (Philadelphia Water Department, 1997). Additional refinement of the CSS Model occurred as part of the Long Term Control Plan Update. Combined sewer system

model development and calibration methodology are discussed within Supplemental Documentation Volume 4 (Philadelphia Water Department, 2011).

The CSS Model domain included:

- The combined service area within the City borders, which drains to the Philadelphia Water Department (Water Department) Water Pollution Control Plants.
- The sanitary portion of the separate sewered area, within and outside the City, which drains to the Water Department Water Pollution Control Plants. A simplified version of the sanitary collection system is modeled inside the City, and indirectly modeled outside the City.
- The combined sewer overflow and interceptor relief outfall pipes within the City, which discharge into the receiving waters.

A.2.2 HEC-2 Model

An open channel HEC-2 model of Tookany/Tacony-Frankford Creek was developed as part of a Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS). The Tookany/Tacony-Frankford Creek HEC-2 Model was originally developed in 1974 (Federal Emergency Management Agency, 2001). Channel geometry data within the Tookany/Tacony-Frankford Creek Watershed Model was supplemented with historic FEMA HEC-2 model data.

A.2.3 Fluvial Geomorphology (FGM) Study

Fluvial geomorphology data was collected as part of the Tookany/Tacony-Frankford Creek Watershed Comprehensive Characterization Report (Philadelphia Water Department, 2005). Channel geometry and bed roughness data derived in the FGM study were used in development of the Tookany/Tacony-Frankford Creek Watershed Model.

A.2.4 Act 167 Stormwater Management Plan

A coupled CSS and Watershed Model of Tookany/Tacony-Frankford Creek was developed as part of the Tookany/Tacony-Frankford Creek Act 167 Stormwater Management Plan (Philadelphia Water Department, 2008). The Northeast District CSS Model was integrated into a model of the Tookany/Tacony-Frankford Creek watershed. Outfall pipes from the regulator structures in the CSS Model were connected to open channel nodes of the Watershed Model so wet weather overflows could be routed to the Tookany/Tacony-Frankford Creek. The result was a model that included the collection system pipe network and all upstream inputs. The model developed for the Act 167 Plan served as the starting point for Tributary H&H Model development.

The Act167 Model domain included:

- The combined service area within the City borders, which drains to the Water Department Water Pollution Control Plants.
- The sanitary portion of the separate sewered area, within and outside the City, which drains to the Water Department Water Pollution Control Plants. A simplified version

of the sanitary collection system is modeled inside the City, and indirectly modeled outside the City.

- The combined sewer overflow and interceptor relief outfall pipes within the City, which discharge into receiving waters.
- Open channel representations of the receiving waters and major tributaries within the watershed.
- The stormwater and direct runoff areas within and outside of the City borders. Stormwater collection system conduits are not explicitly modeled.

A.3 Model Development

The coupled CSS and Watershed Model of Tookany/Tacony-Frankford Creek, developed as part of the Tookany/Tacony-Frankford Creek Act 167 Plan report, served as the starting point for Tributary H&H Model development. The Act 167 Model was simplified by removing bridges, short culverts, and short cross sections. This simplification also improved model stability. The simplified model is referred to as the Tributary H&H Model and was validated against two USGS stream gages. This model provides the hydraulic and bacteria loadings to the WASP model.

The Tookany/Tacony-Frankford Creek Tributary H&H Model includes the entire stream drainage area and extends beyond the Water Department service area into Montgomery County. The model representation of both the channel and the watershed areas beyond the Water Department service area was intended to capture the water quality effect of outfall T-01 which discharges to Rock Creek, a small tributary stream upstream of the City boundary. Figure 1-5 shows the model extents, subwatershed areas, and locations of Water Department and municipal collection system CSOs within the watershed area. In addition to the main channel of Tookany/Tacony-Frankford Creek, several smaller streams were represented in the model to control and improve the timing and shape of the simulated hydrograph at the validation locations.

Within the City, both combined sewer and sanitary sewered areas are included in the Tributary H&H Model. Watershed areas outside the City and outside the CSS are also accounted for in the model. The non-combined sewer areas are mostly within the communities neighboring the City of Philadelphia to the north and west. These areas contribute runoff and associated pollutant loads to the receiving waters either through stormwater collection systems, direct runoff, or through minor tributary waterways.

A.3.1 Hydraulic Model Development

SWMM uses a link-node description of sewer and open channel systems facilitating the physical prototype and the mathematical solution of the gradually-varied unsteady flow (St. Venant) equations which form the mathematical basis of the model. The links transmit the flow from node to node. Properties associated with the links include roughness, length, cross-sectional area, hydraulic radius, and surface width. The primary dependent variable for the links is discharge. Variables associated with nodes include volume, head, and surface area. The

primary dependent variable for nodes is head, which is assumed to be changing in time, but constant throughout any one node.

Open Chanel Cross Sections

The hydraulic network consisted of open channel representations of the Tookany/Tacony-Frankford Creek and major tributaries within the watershed. It was developed from two separate data sets:

- Cross sectional data from HEC-2 models used in the Philadelphia County FIS
- Cross sectional data obtained through the FGM Study

The Water Department surveyed cross sections were used as the main channel for the Tookany/Tacony-Frankford Creek Act 167 Model; the Act 167 Model was the starting point for development of the open channel portion of the Tributary H&H Model. For the Act 167 Model, the cross sections were extended by using GIS data to draw lines perpendicular to 2 ft contour lines. The extended cross sections were then plotted in MS Excel and corrected if any obvious elevation discontinuities existed between the two data sets. Because contour lines are taken to a datum, the surveyed cross section was shifted up or down to be a smooth extension of the contours. Cross sections were extended until they reached about 40 ft higher than the stream bed elevation and were assumed to be representative until the next upstream survey point. Cross sections were also drawn for the top decks of bridges and culverts using the 2 ft contours to accurately model flood conditions in the Act 167 Plan. However, these bridges and short culverts were removed to improve model stability in the Tributary H&H model.

The Water Department stream survey ended roughly at Bristol Street. Between this point and the Delaware River, data from the Philadelphia County FIS HEC-2 Model of the Tookany/Tacony-Frankford Creek was used to supplement the stream survey. Where the two data sets overlapped, cross sections were compared. The two models agreed fairly well so it was assumed HEC-2 cross sections, while less accurate, were a reasonable approximation. All Water Department surveyed cross sections along the main stem of the Tookany/Tacony-Frankford Creek were sorted by river mile and connected.

Minor tributaries were assembled individually and tied into the main stem. Because the Water Department surveys did not exist at the exact location where tributaries tie into the mainstem, the elevation of these points were interpolated from the Water Department cross sections on either side.

The resultant tributary channel network consisted of over 200 channel cross sections representing more than 22 miles of stream. Since conduit length is the primary constraint when stabilizing a model, some conduits were replaced with lengthened equivalent conduits before being put into the model. However, this was done only when necessary. The Act 167 Model contains documentation of extended reaches. In the Tookany/Tacony-Frankford Creek Tributary H&H Model, bridge crossings, most culverts, and short channels were consolidated and simplified to longer open channel conduits. The hydraulic configurations of the stream are based upon the best available information, but should not be considered a truly accurate depiction of actual stream conditions beyond the objectives of the water quality modeling tasks.

All elevations within the model are based on inverts calculated from the topographic contours and surveyed cross sections. Ground elevations are set as the maximum elevation found in the station elevation pairs defining any natural cross section. They are sufficiently high to prevent the hydraulic grade line within the channel from exceeding the user defined channel elevation.

The Water Department pebble counts along Tookany/Tacony-Frankford Creek were used to estimate the value for Manning's roughness within the channel. Channel roughness values for the Tookany/Tacony-Frankford Creek were assigned at all Water Department channel survey points. These roughness values are assumed to be representative until the next upstream survey point.

Floodplain roughness values for the transects were estimated from ortho-photography, field photographs at all Water Department survey points, and a flyover video of Frankford Creek. These values are assumed to be representative until the next upstream survey point.

A.3.2 Hydrologic Model Development

Subcatchment Delineation

Watersheds were delineated for the Tookany/Tacony-Frankford Creek using GIS surface contour layers obtained from Pennsylvania Spatial Data Access (PASDA). Sheds were delineated to survey rebars and were defined by 2 ft topographic contour lines. Subcatchment watersheds were delineated to selected points along the main channel at critical stream junctions or at pre-defined intervals. Since runoff from combined areas reaches the receiving water as a CSO input, sheds were defined for any watershed area not defined as a CSO subcatchment in the CSS Model. In this way runoff from areas defined as sanitary service areas, storm sheds, and non-contributing areas (to the collection system) was included in the Watershed Model. The defined subwatersheds serve as the modeling unit in the Tributary H&H Model.

Overland Slope

In SWMM, runoff is calculated by approximating a non-linear reservoir which forms above the surface once the demands of infiltration, evaporation, and storage have been satisfied. The overland flow is generated using Manning's equation:

$$Q = \frac{1.486}{n} * W * (d - d_s)^{5/3} * S^{1/2}$$

Where:

Q = surface runoff (cfs),

W = width of watershed (ft),

S = average slope of watershed (ft/ft)

d = depth in the non-linear reservoir (ft),

n = Manning roughness coefficient, and

d_s = depression storage depth in the non-linear reservoir (ft).

Overland slope is a quantifiable physical parameter and is not adjusted during model validation. An average overland flow slope technique was used to define subcatchment slopes for the Tookany/Tacony-Frankford Creek Tributary H&H Model.

The average slope values of the Tookany/Tacony-Frankford sheds were found by using Spatial Analyst, a toolset in ArcGIS that analyzes and models cell based data. The average slope was calculated by using a 1 arc-second DEM that was bounded by a polygon feature class representation of the model subcatchments. Within Spatial Analyst, the "Slope" function calculates the maximum rate of change in value from that cell to its neighbors. For this analysis, the percent slope was calculated and redrawn as a new raster file. The "Zonal Statistics" function was then used to average the slope values within the shed boundaries.

Width

The width parameter impacts the time of concentration and the hydrograph shape in the hydrologic portion of SWMM. Subcatchments are represented as rectangular areas defined by the subcatchment width parameter. By definition, the width of a watershed is equal to the area of the watershed divided by the length of the overland flow path. The width parameter is one of the main validation parameters used to adjust hydrograph shape, and to some degree hydrograph volume. Watersheds assigned a large width, and therefore a small overland flow length, will have a short time of concentration. Reducing width increases the flow length and will thus increase the time of concentration. Increasing the overland flow length also reduces the runoff volume as the flow is exposed to evaporation and infiltration over pervious areas for a longer period of time. Stormwater collection systems and minor undefined waterways are examples of how the overland flow length can be shortened, and the width increased. Overland flow length is a more intuitive parameter, and therefore is used as the basis for adjusting and discussing the SWMM width parameter. Since the width parameter is adjusted during validation, it is only necessary to obtain an approximate initial value.

The initial estimates for watershed width were obtained by calculating two times the square root of the watershed area. This approach provides a width based on an idealized square watershed along the middle of the watershed. Since actual watersheds are not rectangular with properties of symmetry and uniformity, it should be expected that these values will be widely adjusted during the validation process.

Gross Impervious Cover

An estimate of gross impervious area for model subcatchments was based on clipping the Water Department impervious surface coverage with the Tookany/Tacony-Frankford Creek subcatchment shed coverage and calculating the percent impervious cover for each subcatchment. Impervious surface information was obtained from the 2004 Sanborn planimetric layer maintained by the Office of Watersheds. Impervious surface classifications in the layer were grouped into three broad categories (buildings, parking, streets/sidewalks). For each subcatchment, the area of impervious surface was summed to generate a total impervious area.

Land use data was used to estimate impervious cover for portions of the watershed areas outside of the Water Department service area and within neighboring Montgomery County. The Delaware Valley Regional Planning Commission completed a digital land use file based on aerial photography flown in March through May of 1995 that was used in this analysis. Land use was interpreted in seventeen categories from 1330 Photo Atlas Sheets at the 1 inch = 400 ft. scale.

Table A-1 lists the percent impervious value assigned to each of the land uses for areas outside of the City with the exception of single family attached residential, single family detached residential, and multi-family residential. For these residential land uses outside of the City, population densities were developed using 2000 census and block area data. Two equations, Stankowski (1974) and Manning *et al.* (1987) that use population density as the independent variable to define percent impervious were selected for this modeling application. The equations are expressed as:

$$\text{Stankowski, } I = 0.117D^{0.792 - 0.039 \log D}$$

$$\text{Manning, } I = 104.95 - 81.27(0.974)^{PD}$$

where: I = Percent impervious
 PD = Population density per acre
 D = Population density per square mile

Percent impervious estimates for each census block were calculated with both equations. For population densities less than 35 persons per acre (ppa), the Stankowski and Manning equations were averaged. However, only the Manning estimate was used when the population density was greater than 35 ppa. This distinction was made because the Stankowski equation is less accurate for high density urban areas. Each land use classification was assigned a percent impervious cover based upon regional averages and/or population density.

Table A-1: Percent Impervious by Land Use Classification

Land Use Classification	Percent Impervious
Agricultural	5
Commercial	95
Communications & Utilities	50
Community Services	80
Manufacturing	65
Military	80
Mining	20
Multi-Family Residential	Dependent on population
Recreational	20
Single Family Attached Residential	Dependent on population
Single Family Detached Residential	Dependent on population
Transportation	90
Vacant	40
Water	0
Wooded	3

The estimate of percent impervious cover of total shed area was entered into the model as percent impervious for each subcatchment and was kept constant during the tributary validation process. The influence of impervious cover on total shed stormwater runoff was controlled by

the percent routed parameter, which was considered to be the impervious runoff validation parameter.

Percent Impervious Cover Routed

SWMM simulates surface runoff from drainage areas using three “planes” of overland flow. One plane represents all impervious surfaces directly connected to the hydraulic system and included initial abstraction or surface detention storage (puddles, cracks, etc. which do not permit immediate runoff). A second plane represents all pervious areas and impervious areas not directly connected to the hydraulic system. The third plane is defined as the fraction of the directly connected area that provides no detention storage and thus produces runoff immediately. Furthermore, a portion of the pervious runoff can be routed to the impervious surface (i.e. Pervious Routing), or a portion of the impervious runoff can be routed to the pervious surface (i.e. Impervious Routing). The runoff from the drainage area is the sum of the flow from the three planes, after considering internal routing.

Impervious surfaces, by definition, do not have an infiltration component. However, impervious surfaces are not always connected to a collection or conveyance system, and may instead route to a pervious surface where the runoff generated has the opportunity to infiltrate. The fraction of the total area that is impervious and drains directly to the collection or conveyance system is defined as the directly connected impervious area (DCIA). In SWMM, the “% Impervious” hydrologic parameter represents the gross impervious area as a percentage of the total shed area. The PERVIOUS method of subcatchment routing is used to approximate hydrograph flow paths from the subcatchments. When the PERVIOUS method is used, a value for the percent of total runoff from the impervious area that is routed over the remaining pervious area of the shed is defined. The remaining impervious area not routed is termed the DCIA, which drains directly to the collection and conveyance system. The PERVIOUS routing method allows for complex hydrographs to be reproduced as DCIA is manifested as a near immediate system response to precipitation, while the pervious area runoff may have a slower response.

Soils

The rate of infiltration is a function of soil properties in the drainage area, ground slopes, and ground cover. The Green-Ampt method of calculating infiltration was employed for the Tookany/Tacony-Frankford Creek Tributary H&H Model. The Green-Ampt equation for infiltration has physically based parameters that can be estimated based on soil characteristics. The soil parameters used in this method are: average capillary suction, saturated hydraulic conductivity, and the initial moisture deficit. In SWMM, pervious area runoff is generated when rainfall volume and intensity exceed the soil infiltration capacity and evaporation demands. Infiltration is primarily controlled by setting the soil saturated hydraulic conductivity, which is the rate at which water will infiltrate the soil after the soil has reached a point of saturation. By lowering the saturated hydraulic conductivity, the pervious area begins to generate runoff. Once the saturated hydraulic conductivity threshold for pervious runoff is found, additional increment adjustments downward tend to produce very significant responses. Therefore, saturated hydraulic conductivity was a primary validation parameter.

Soil information for the Tookany/Tacony-Frankford Creek watershed was obtained from the

U.S. Department of Agriculture’s Natural Resources Conservation Service, which is responsible for collecting, storing, maintaining, and distributing soil survey information for lands in the United States. Infiltration parameters were assigned to each subcatchment based on soil texture classification. Table A-2 lists the parameter ranges for several different soil classifications. Soil classifications can vary widely within each subcatchment, and a weighted value was calculated for the three parameters. It should be noted that 70% of the Tookany/Tacony-Frankford Creek watershed is classified as either “Urban Land” or “Made Land” and infiltration values can vary greatly for these classifications and are site-specific. This limitation of soils data, and the modification of native soils and various degrees of compaction due to construction within Philadelphia and the surrounding municipalities implies that infiltration parameters are very significant in the validation process. Flow monitoring at trunk sewers throughout the service area was critical in determining infiltration parameters for the CSO sheds in the CSS Models. This flow monitoring data was used in validation to estimate infiltration parameters such as hydraulic conductivity for monitored CSO sheds. These established infiltration parameters were used for CSO subcatchments with origins in the CSS Model. The hydraulic conductivity for the subcatchments in the Tributary H&H Model outside of the CSO area was determined through model validation at the USGS gages.

Table A-2: Soil Infiltration Parameter Estimates for Several Soil Texture Classifications

USDA Soil Texture Classification	Saturated Hydraulic Conductivity (in/hr)	Initial Moisture Deficit for Soil (Vol. Air / Vol. of Voids, expressed as a fraction)	Avg. Capillary Suction (in)
		Moist Soil Climates (Eastern US)	
Sand	9.27	0.346	1.95
Loamy Sand	2.35	0.312	2.41
Sandy Loam	0.86	0.246	4.33
Loam	0.52	0.193	3.50
Silt Loam	0.27	0.171	6.57
Sandy Clay Loam	0.12	0.143	8.60
Clay Loam	0.08	0.146	8.22
Silty Clay Loam	0.08	0.105	10.75
Sandy Clay	0.05	0.091	9.41
Silty Clay	0.04	0.092	11.50
Clay	0.02	0.079	12.45

Depression Storage

Depression storage is the rainfall abstraction volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. In SWMM, every subcatchment is divided into three subareas: pervious area with depression storage, impervious area with zero depression storage, and impervious area with depression storage. Water stored as depression storage on pervious areas is subject to infiltration and evaporation. Water stored in depression storage on impervious areas is depleted only by evaporation, therefore replenishment of the retention storage typically takes longer when compared to pervious areas. Both impervious and pervious

depression storage were adjusted as validation parameters. For all subcatchments, impervious depression storage was initially set as 0.063 inches and pervious depression storage was set at 0.25 inches, (Viessman and Lewis, 2002). By default, the model assumes 25% of the impervious area has zero depression storage. This default value was not altered in the setup of the Tookany/Tacony-Frankford Creek Tributary H&H Model.

Subcatchment Manning's Roughness

Manning's roughness values must be estimated for both pervious and impervious overland flow. For impervious areas, these values can be estimated fairly easily. For pervious areas the estimation is somewhat more difficult as there can be considerable variability in ground cover along the flow path. Table A-3 lists Manning's n estimates for several types of surfaces. For the Tookany/Tacony-Frankford Creek Tributary H&H Model, all subcatchments were assigned a Manning's n value of 0.013 for impervious areas and 0.1 for pervious areas.

Table A-3: Roughness Coefficients (Manning's n) for Sheet Flow

Surface Description		n
Smooth surfaces	Concrete, asphalt, gravel, or bare soil	0.011
Fallow	No residue	0.05
Cultivated soils	Residue cover < or = 20%	0.06
	Residue cover > 20%	0.17
Grass	Short grass prairie	0.15
	Dense grasses	0.24
	Bermudagrass	0.41
	Range (natural)	0.13
Woods	Light underbrush	0.40
	Dense underbrush	0.80

Stormwater Event Mean Concentrations

The development of the EMC dataset used for the Tributary H&H Models is detailed in Section 3.7.1. The vast majority of data used developing this EMC dataset were obtained from urban runoff monitoring programs, which represents the urban land use throughout the Water Department service area, but does not necessarily represent the suburban and rural land use throughout the upstream portions of the watershed drainage area. However, the rainfall-runoff response is overwhelmingly driven by impervious surfaces, which would represent the urbanized portions of the watershed. Therefore, applying these EMCs universally to all subcatchments (both city and suburban) in the Tributary H&H Models was considered a reasonable and conservative approach for modeling the runoff pollutants. The water quality function was used in SWMM to determine bacteria loadings at the outfalls for the WASP water quality models.

A.3.3 Model Input

Precipitation

Radar rainfall was obtained from Vieux & Associates, Inc. and processed to be used for the hydrologic validation period (years 2010 and 2012) and the water quality validation period (years 2003 and 2004). The radar grid was calibrated to the existing Water Department rain gage network, consisting of 24 gages within the City limits. Precipitation for each model subcatchment was calculated by area weighting 1 km² radar grids intersecting the individual subcatchment boundaries. The radar rainfall data was necessary to provide coverage for sheds outside the coverage of the Water Department gage network. This improved the simulation of events with complex hydrographs and quick runoff responses. In a rainfall-runoff model, much of the uncertainty in the model results from the uncertainty inherent in the precipitation data. Therefore, using a detailed precipitation input that accounts for the temporal and spatial distribution in rainfall intensities and volumes, increases the accuracy and precision of the model results. Compared to using the Water Department rain gage network directly, radar rainfall has the potential to better represent the spatial distribution of rainfall between gages within the City, and for locations outside the rain gage network.

Evaporation

Evaporation data was loaded into the model in the form of average monthly evaporation rates. Evaporation data usually can be obtained from the National Weather Service or from other pan measurements, however, long-term daily evaporation data was limited in the Philadelphia area. Neither the Philadelphia Airport nor the Wilmington Airport recorded evaporation data. One site located in New Castle County, Delaware recorded daily evaporation data from 1956 through 1994. Average daily evaporation (inches per day) from this site was used to estimate typical monthly evaporation rates, which were used by the model.

Baseflow

Baseflow is required for each tributary in order to perform the in-stream hydrological validation of the watershed models. An estimate of baseflow was calculated using USGS gages 01467086 and 01467807. Baseflow is mostly comprised of groundwater discharge to the stream, while runoff is a result of overland discharge. Baseflow separation involved disaggregating monitored flow time series into wet and dry components based upon expected hydrological response times. In order to extract a baseflow from monitored streamflow data, the data set was first merged with rainfall data. All data during wet weather and 72 hours afterwards was removed to isolate dry weather flows. An average annual baseflow was estimated and used as a dry weather input to the model. The dry weather flow was weighted by contributing watershed area. Baseflow values were controlled within SWMM as multipliers on a constant dry weather inflow at the inflow nodes.

Boundary Condition

Tide elevation data was needed where the Frankford Creek meets the Delaware River to provide an accurate head boundary. Since the Torresdale Avenue dam was the most downstream extent of the water quality modeling effort, the mean tide value from NOAA tide data was used as the ultimate boundary condition at the Delaware River. However, the model should be updated with

a tidal time series if it is to be used to simulate the tidal portion of the Tookany/Tacony-Frankford Creek.

A.4 Validation

A.4.1 Validation Approach

Validation of the Tributary H&H Model was performed using flow data collected at USGS monitoring locations along the Tookany/Tacony-Frankford Creek. Flow monitoring data was available for the 2010-2012 validation period. Initial estimates of selected variables were adjusted within a specified range until a satisfactory correlation between simulated and measured streamflow, over a range of events, was obtained. Stormwater runoff and the associated pollutant loadings from the watershed areas, including the stormwater contributions from separate sanitary sewered areas, were input to the Tributary H&H Model at nodes along the tributary networks. Flow from CSOs upstream of the USGS gaging stations is captured in the streamflow data, so was therefore a part of the validation dataset. However, flow from CSOs was considered fixed by the overflow validation process in the CSS Model, and therefore CSO shed parameters were not adjusted during validation of the Tributary H&H Model.

As discussed in Section A.3.2, runoff in SWMM is generated when the precipitation exceeds the demands of infiltration, evaporation, and storage. The rate of runoff is determined by the subcatchment area, width, slope, depth of the non-linear reservoir, and the roughness coefficient. Subcatchment area, slope, and total impervious area are quantifiable parameters and, therefore, were not adjusted during the validation process. Infiltration parameters, width, and percent routed were the primary parameters adjusted during validation. In addition, depression storage and overland roughness coefficients were adjusted during the validation process. Depression storage and overland roughness parameters were set independently for the pervious and impervious portions of the subcatchments.

The first phase of validation utilized the hydrologic parameters that control event hydrograph volume, namely:

- Percent Routed / DCIA
- Saturated Hydraulic Conductivity
- Initial Soil Moisture Deficit
- Soil Capillary Suction Head

The second phase of validation utilized the hydrologic parameters that control event hydrograph timing and peak, namely:

- Subcatchment Width
- Impervious and Pervious Depression Storage

A.4.2 Model Validation Data

Streamflow

The model was validated with streamflow at USGS gaging sites along the Tookany/Tacony-Frankford Creek. It was important for hydrologic validation that both the Tributary H&H Model and the flow monitoring record included the contributing wet weather flows from the City CSOs, since this flow accounts for a sizable portion of the streamflow in this urban stream.

Instantaneous USGS flow monitoring data in 15 minute intervals was available over recent time periods for two gages along the Tookany/Tacony-Frankford Creek.

Presently there are two gages along the Tookany/Tacony-Frankford Creek, 01467086 and 01467087. Gage 01467086, located near the Adams Avenue bridge, is the more upstream gage and near where the stream passes through the City border. Gage 01467087 at the Castor Avenue bridge is the more downstream gage. The Castor Avenue gage is approximately 2.8 miles upstream of the mouth of the stream at the Delaware River and above the influence of tide. Current USGS gage data from 2010 through 2012 was used in favor of historical daily records for hydrologic validation. Current data better represents existing wet weather response of the watershed. The data is recorded in 15 minute increments, which provides a finer resolution for hydrographs.

USGS gage 01467086 has a well defined rating curve with wet weather flows up to 2000 cfs. Flows from this USGS gage were used to calibrate the model primarily outside the City. USGS gage 01467087 has a poorly defined rating curve. The peak field measured flow rate is about 2000 cfs. Field notes for this point report the reading as “poor”. The next highest recorded point is 158 cfs. Since the downstream gage 01467087 at Castor Avenue is the only active gage during the water quality validation period (2003-2004), and a water quality sampling station exists at the gage site, it was not excluded in the current year validation. The upper gage 01467086 was used to validate the modeled watershed areas outside the City, and the lower gage was used to validate the watershed areas below the upstream gage. Section 2.2.2 contains additional discussion of USGS gage station rating curve development and published flow data error ranges.

Event Definition

Wet weather events were defined for each monitored dataset using the flow and precipitation monitoring records. A total of 146 events were defined for both gages within the hydrologic validation period of January 2010 through December 2012. The monitored and predicted hydrographs were split into discrete wet weather events over time, so comparisons could be made on an event by event basis. Events were defined not by continuous rainfall, but continuous wet weather response. Additionally, since snowmelt was not simulated, snowfall and all potential snow-melt events were removed from the validation data set.

During the validation process, each of these events was evaluated repeatedly to compare the observed and simulated event statistics for event volume and peak flow. The observed and simulated hydrographs were compared for their general response shape and timing. During the validation process, the model hydrologic parameters were adjusted to provide a better fit between the simulated and monitored flows. Care was taken to exclude the influence of events

which had unusually large or small responses to the rainfall hyetograph so that the overall model validation would not be skewed by a select number of events. The ratio of monitored flow to precipitation volume over the contributing watershed area was also used as a reason to exclude events from the validation process. If there seemed to be a mismatch between rainfall and runoff response or hydrograph shape and timing, the event was deemed an outlier.

A.4.3 Validation Results

Flow monitoring data from USGS stations 01467087 Frankford Creek at Castor Avenue and 01467086 Tacony Creek above Adams Avenue were used to perform the hydrologic validation of the Tookany/Tacony-Frankford Creek Tributary H&H Model. The upstream gage, 01467086 at Adams Avenue, is close to the City border and was used to validate the more upstream sheds that mostly lie outside the City and the Water Department service area. The downstream gage, 01467087 at Castor Avenue, was used to perform the hydrologic validation for the watershed area below the Adams Avenue gage. Flow data at this gage was used with caution since there is limited field measured flow data for large wet weather flows. USGS assigned a site rating condition of “poor” to this station. The rating curve includes more field measured flow rates of small and medium sized events and should provide a good representation of actual conditions in this range. Model flow time series were compared to observed flow for the events defined at both gages within the hydrologic validation period of January 2010 through December 2012.

Since the percent DCIA value was fixed as the total impervious area of the watershed subcatchments, it was not adjusted during validation. The model parameter adjusted to represent the effect of directly connected impervious areas was the “percent routed” parameter, since the pervious method of subcatchment routing was employed to represent the routing of runoff from impervious areas onto adjacent pervious areas. Through validation the percent routed parameter was determined to be 30% and was universally applied to all watershed subcatchments. This takes into account both the upstream and downstream gages.

A width of 10 times the square root of the shed area was assigned to the Tookany/Tacony-Frankford Creek watersheds. This value, which is larger than the initial area based estimates, was increased to reduce peak flows and to extend the simulated hydrograph.

The saturated hydraulic conductivity was adjusted to 0.30 in/hr for all sheds. This value was increased from the initial estimate based on the GIS soil information. The saturated hydraulic conductivity was first determined for the headwater sheds upstream of the Adams Avenue gage. Similar infiltration parameters were then applied to all sheds downstream of the gage and a reasonable fit was observed for the events at the Castor Avenue gage.

The volumetric validation plot for Tookany/Tacony-Frankford Creek at the Adams Avenue gage is presented in Figure A-1. Volume is predicted fairly well by the model although somewhat over-simulated overall. Due to the limitations of the rating curve for the Adams gage, monitored events larger than 2000 cfs were excluded from the validation. This constraint also limits the validation to events that are on the same scale as the water quality validation events. Special attention was given to events in the range of 100 to 1500 cfs to best represent the hydrology for

storms on the water quality event scale. A balance was found between small and medium sized events and the best fit for the volume regressions. The validation plot for Event 47, March 10, 2011 is provided in Figure A-2 as an example of a medium sized event where the monitored and modeled hydrographs match very well.

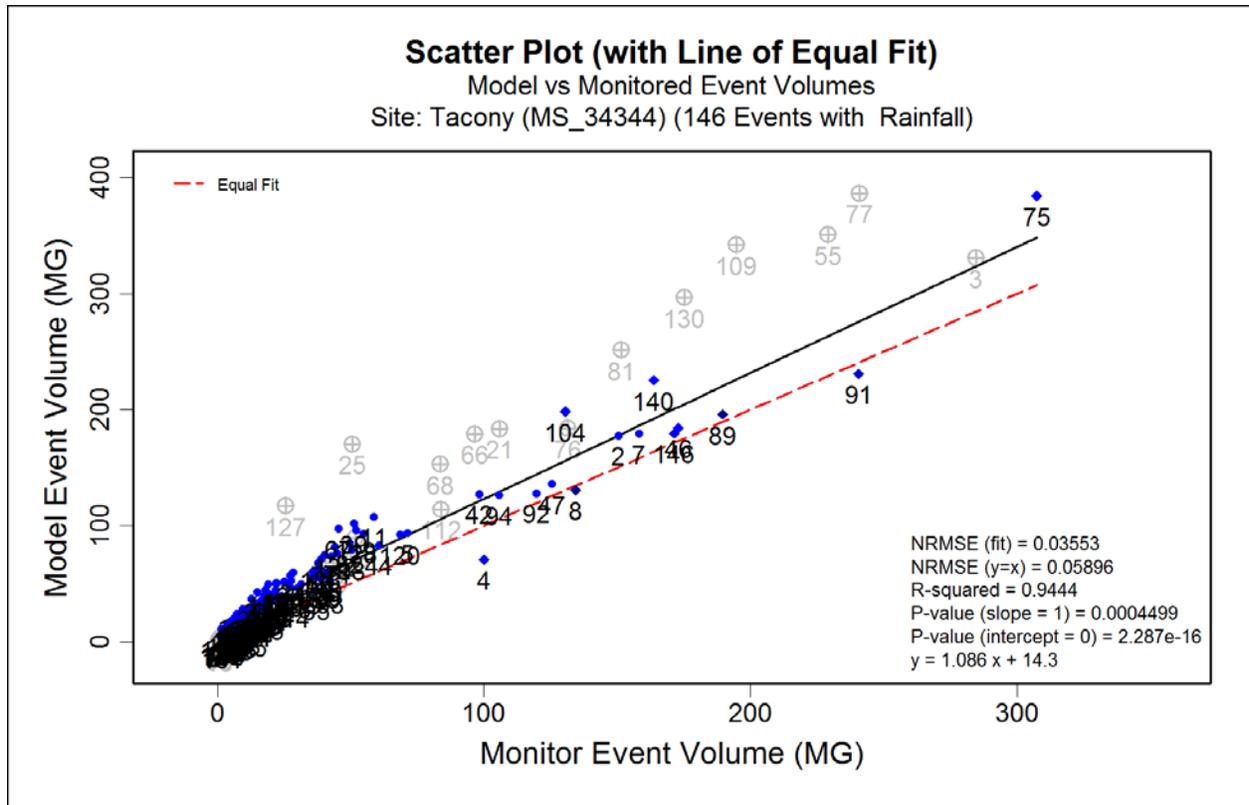


Figure A-1: Tookany/Tacony-Frankford Creek Volume Validation at Gage 01467086 (Adams Avenue)

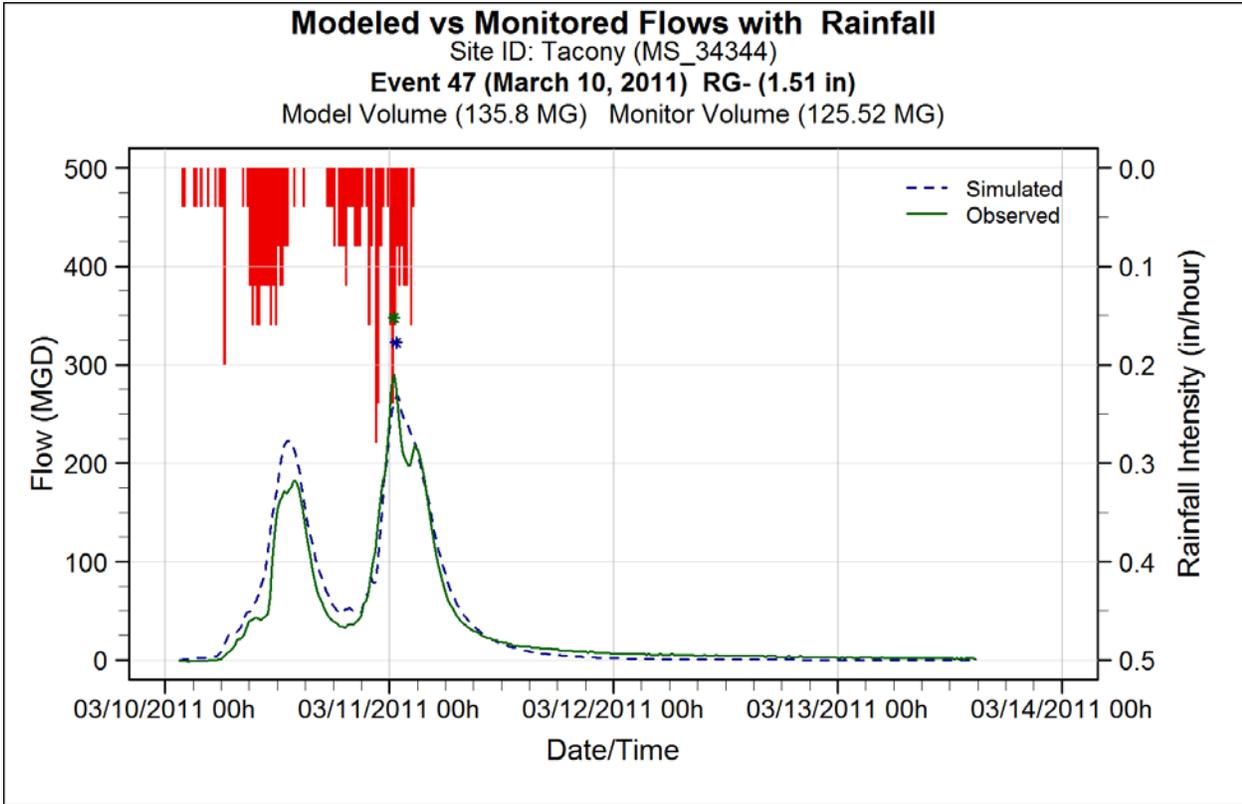


Figure A-2: Tookany/Tacony-Frankford Creek Example Validation Plot at Gage 01467086 (Adams Avenue)

The volumetric validation plot for Tookany/Tacony-Frankford Creek at the Castor Avenue gage is presented in Figure A-3. Overall, the model does a good job of predicting volume. The regression is driven by one extreme event, however moderate and small-sized events seem to be simulated fairly well. These results suggest that the limitations of the rating curve for the Castor Avenue gage may not have a large impact on the overall validation for moderate and small events. The relatively good volume results at the Castor Avenue gage also suggest that the validation at the Adams Avenue gage was adequate. The validation plot for Event 140, October 28, 2012 is provided in Figure A-4 as an example of a medium to large-sized event where the monitored and modeled hydrographs match very well.

The validation results show that the model adequately represents the monitored data, and that overall it provides a good representation of the flows in Tookany/Tacony-Frankford Creek.

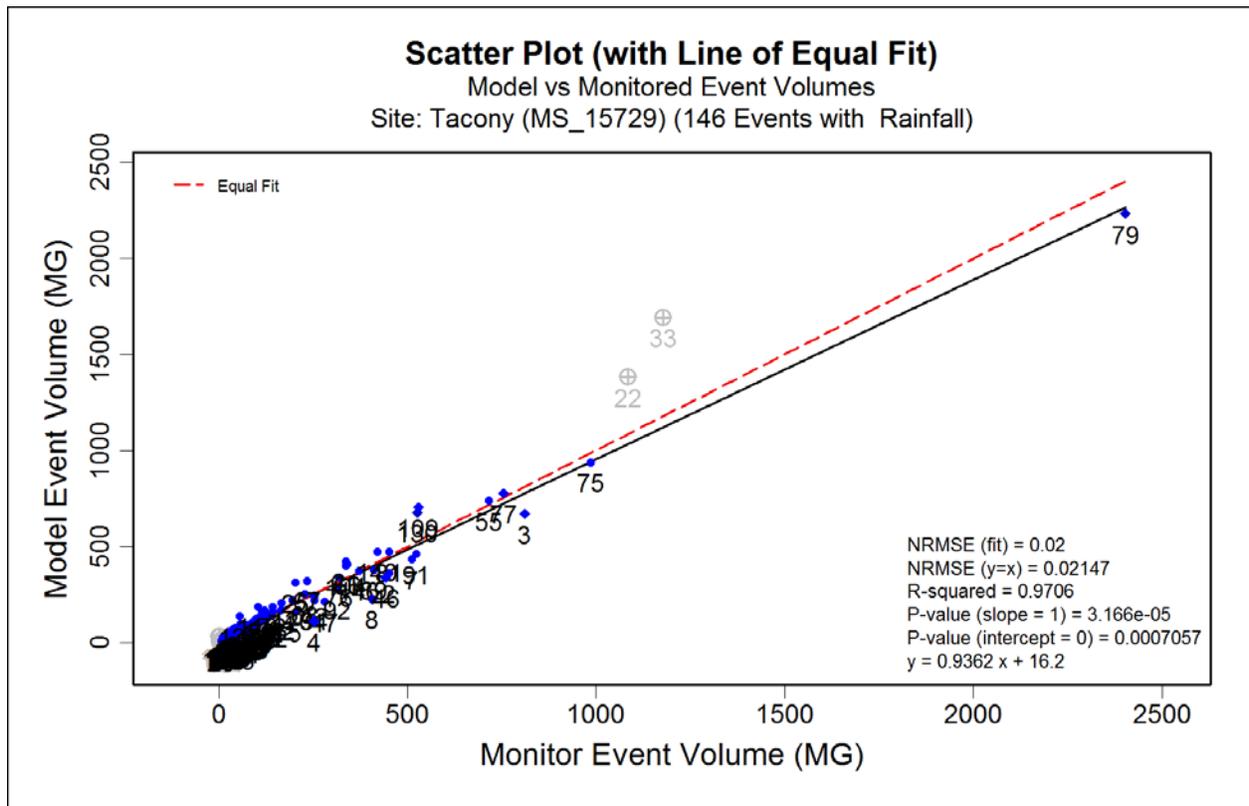


Figure A-3: Tookany/Tacony-Frankford Creek Volume Validation at Gage 01467087 (Castor Avenue)

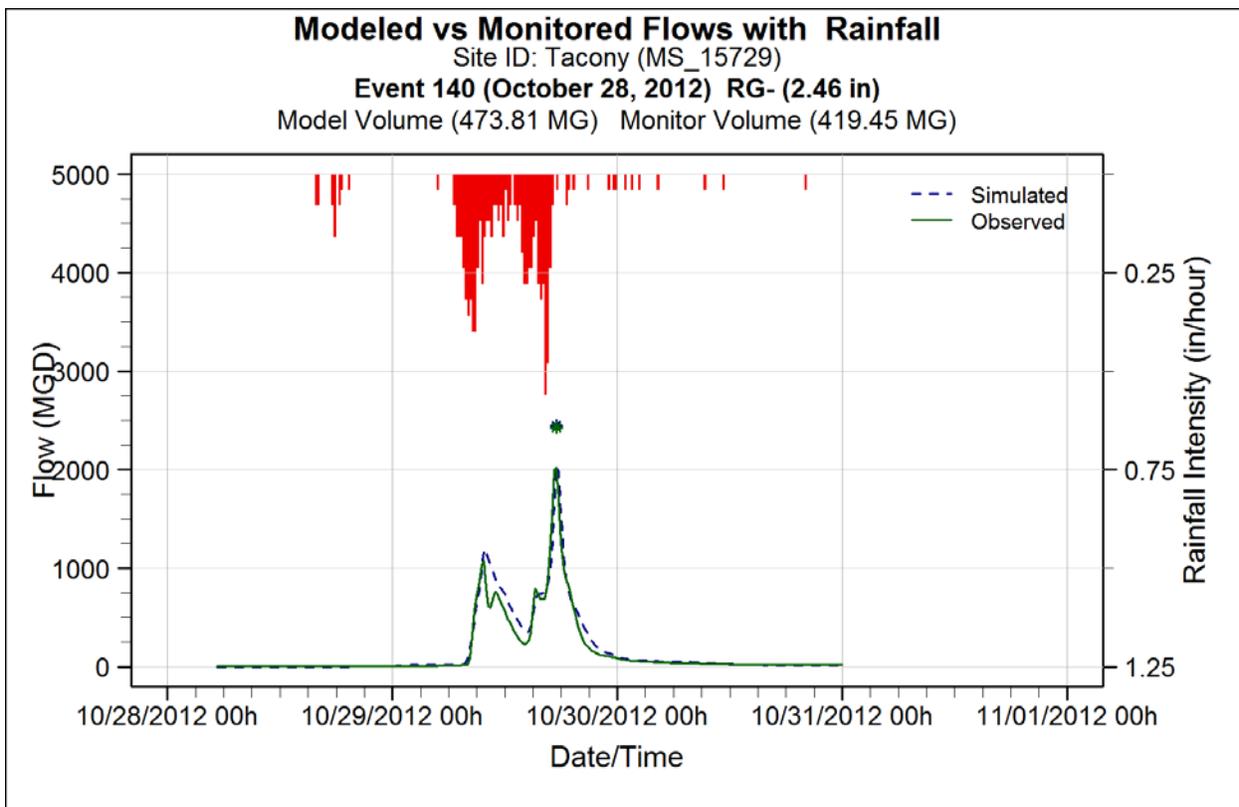


Figure A-4: Tookany/Tacony-Frankford Creek Example Validation Plot at Gage 01467087 (Castor Avenue)

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Appendix B

Cobbs Creek SWMM Validation

B.1 Introduction

This appendix describes the development and validation of the Cobbs Creek Tributary H&H Model used to provide the hydraulic and bacteria loadings to the United States Environmental Protection Agency (US EPA) Water Quality Analysis Simulation Program (WASP) model.

The US EPA Storm Water Management Model (SWMM) version 5 was used to develop the Combined Sewer System (CSS) Model and Watershed Model that together comprise the Tributary Hydraulic and Hydrologic (H&H) Model. SWMM5 hydrology, represented by subcatchments, simulated both the quantity and quality of runoff in a drainage basin and the routing of flows and contaminants to sewers or receiving waters. SWMM5 hydrology can accept precipitation (rainfall or snowfall) hyetographs and perform a step by step accounting of snowmelt, infiltration losses in pervious areas, surface detention, overland flow, and water quality constituents leading to the calculation of one or more hydrographs and/or pollutographs at a certain geographic point such as a sewer inlet. SWMM5 hydraulics, represented by nodes and links, simulates dynamic hydraulic flow routing and pollutant routing through open channel and closed conduit systems (US EPA, 2010).

The Cobbs Creek Tributary H&H Model was developed to simulate the stormwater runoff and water quality loading from combined sewer overflows (CSOs) and minor tributaries to the receiving waters. The model was developed primarily utilizing information obtained through previous modeling efforts and Geographic Information Systems (GIS) and was driven using continuous radar rainfall time series. Event mean concentrations (EMCs) were used to predict the stormwater quality components of the model. Flow validations were performed using monitoring records from United States Geological Survey (USGS) gaging stations. The model validation period of the Tributary H&H Model was dictated by available USGS stream flow and radar-rainfall data.

The following sections of this report further describe the models used in development and validation of the Cobbs Creek Tributary H&H Model.

B.2 Discussion of Legacy Models and Reports

Several previously published models and reports were used to develop the Cobbs Creek Tributary H&H Model. This section discusses the role of legacy publications in model development.

B.2.1 Long Term Control Plan Update Combined Sewer System (CSS) Model

The Southwest District combined sewer system of Philadelphia was originally modeled as part of the Long Term Control Plan (Philadelphia Water Department, 1997). Additional refinement of the CSS Model occurred as part of the Long Term Control Plan Update. CSS Model development and calibration methodology are discussed within Supplemental Documentation Volume 4 (Philadelphia Water Department, 2011).

The CSS Model domain included:

- The combined service area within the City borders, which drains to the Philadelphia Water Department (Water Department) Water Pollution Control Plants.
- The sanitary portion of the separate sewer area, within and outside the City, which drains to the Water Department Water Pollution Control Plants. A simplified version of the sanitary collection system is modeled inside the City, and indirectly modeled outside the City.
- The CSO and interceptor relief outfall pipes within the City, which discharge into the receiving waters.

B.2.2 HEC-2 Model

An open channel HEC-2 model of Cobbs Creek was developed as part of a Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS). The Cobbs Creek HEC-2 Model was originally developed in 1974 (Federal Emergency Management Agency, 2001). Channel geometry data within the Cobbs Creek Watershed Model was supplemented with historic FEMA HEC-2 Model data.

B.2.3 Fluvial Geomorphology (FGM) Study

Fluvial geomorphology data was collected as part of the Darby-Cobbs Creek Watershed Comprehensive Characterization Report (Philadelphia Water Department, 2004). Channel geometry and bed roughness data derived in the FGM study were used in development of the Cobbs Creek Watershed Model.

B.2.4 Act 167 Stormwater Management Plan

A coupled CSS and Watershed Model of Cobbs Creek was developed as part of the Darby-Cobbs Creeks Act 167 Stormwater Management Plan (Philadelphia Water Department, 2008). The Southwest District CSS Model was integrated into a model of the Darby-Cobbs Creek watershed. Outfall pipes from the regulator structures in the CSS Model were connected to open channel nodes of the Watershed Model so wet weather overflows could be routed to the Cobbs Creek. The result was a model that included the collection system pipe network and all upstream inputs. The model developed for the Act 167 Plan served as the starting point for Tributary H&H Model development.

The Act 167 Model domain included:

- The combined service area within the City borders, which drains to the Water Department Water Pollution Control Plants.
- The sanitary portion of the separate sewer area, within and outside the City, which drains to the Water Department Water Pollution Control Plants. A simplified version of the sanitary collection system is modeled inside the City, and indirectly modeled outside the City.
- The combined sewer overflow and interceptor relief outfall pipes within the City, which discharge into receiving waters.

- Open channel representations of the receiving waters and major tributaries within the watershed.
- The stormwater and direct runoff areas within and outside of the City borders. Stormwater collection system conduits are not explicitly modeled.

B.3 Model Development

The coupled CSS and Watershed Model of Cobbs Creek developed as part of the Cobbs Creek Act 167 Plan report served as the starting point for Tributary H&H Model development. The Act 167 Model was simplified by removing bridges, short culverts, and short cross sections. This simplification also improved model stability. The simplified model is referred to as the Tributary H&H Model and was validated against two USGS stream gages. This model provides the hydraulic and bacteria loadings to the WASP model.

The Cobbs Creek Tributary H&H Model includes the entire stream drainage area and extends beyond the Water Department service area and into Delaware and Montgomery Counties. The model representation of both the channel and the watershed areas beyond the Water Department service area was intended to capture the water quality effect of both East and West Indian Creeks. Figure 1-6 shows the model extents, subwatershed areas, and locations of Water Department and municipal collection system CSOs within the watershed area. In addition to the main channel of Cobbs Creek, the stream Naylor's Run was represented in the model to control and improve the timing and shape of the simulated hydrograph at the validation locations.

Within the City, both combined sewer and sanitary sewered areas are included in the Tributary H&H Model. Watershed areas outside the City and outside the CSS are also accounted for in the model. The non-combined sewer areas are mostly within the communities neighboring the City of Philadelphia to the north and west. These areas contribute runoff and associated pollutant loads to the receiving waters either through stormwater collection systems, direct runoff, or through minor tributary waterways.

B.3.1 Hydraulic Model Development

SWMM5 uses a link-node description of sewer and open channel systems facilitating the physical prototype and the mathematical solution of the gradually-varied unsteady flow (St. Venant) equations which form the mathematical basis of the model. The links transmit the flow from node to node. Properties associated with the links include roughness, length, cross-sectional area, hydraulic radius, and surface width. The primary dependent variable for the links is discharge. Variables associated with nodes include volume, head, and surface area. The primary dependent variable for nodes is head, which is assumed to be changing in time, but constant throughout any one node.

Open Chanel Cross Sections

The hydraulic network consisted of open channel representations of the Cobbs Creek and major tributaries within the watershed. It was developed from two separate data sets:

- Cross sectional data from HEC-2 models used in the Philadelphia County FIS
- Cross sectional data obtained through the FGM Study

The Water Department surveyed cross sections were used as the main channel for the Cobbs Creek Act 167 Model; the Act 167 Model was the starting point for development of the open channel portion of the Tributary H&H Model. At each of the surveyed locations, detailed profiles of the stream cross section were taken and two rebar were installed to mark each. The cross sections were extended out beyond the 100-year flood plain using GIS data. For cross sections that lie within the City of Philadelphia, a 1/3rd arc-second digital elevation model (DEM) was used. For cross sections that fell outside of the City boundaries, a 1 arc-second DEM was used. Cross sections were also drawn for the top decks of bridges and culverts to model flood conditions in the Act 167 Plan. However, these bridges and short culverts were removed to improve model stability in the Tributary H&H Model.

Cross sections from the HEC-2 Model of the Cobbs Creek used in the Philadelphia County FIS was used to supplement the Water Department survey. The HEC-2 Model was created to evaluate potential flooding in Cobbs Creek and included detailed cross sectional data of potential flow obstructions including dams and roadway/railway bridges. The cross sectional data was converted to City datum and formatted for use by SWMM5. Where the two data sets overlapped, cross sections were compared. The two models agreed fairly well so it was assumed HEC-2 cross sections, while less accurate, were a reasonable approximation. All Water Department surveyed cross sections along the main stem of the Cobbs Creek were sorted by river mile and connected.

The resultant tributary channel network consisted of over 100 channel cross sections representing more than 16 miles of stream. Since conduit length is the primary constraint when stabilizing a model, some conduits were replaced with lengthened equivalent conduits before being put into the model. However, this was done only when necessary. The Act 167 Model contains documentation of extended reaches. In the Cobbs Creek Tributary H&H Model, bridge crossings, most culverts, and short channels were consolidated and simplified to longer open channel conduits. The hydraulic configurations of the stream are based upon the best available information, but should not be considered a truly accurate depiction of actual stream conditions beyond the objectives of the water quality modeling tasks.

All elevations within the model are based on inverts calculated from the topographic contours and surveyed cross sections. Ground elevations are set as the maximum elevation found in the station elevation pairs defining any natural cross section. They are sufficiently high to prevent the hydraulic grade line within the channel from exceeding the user defined channel elevation.

The Water Department pebble counts along Cobbs Creek were used to estimate the value for Manning's roughness within the channel. Channel roughness values for the Cobbs Creek were

assigned at all Water Department channel survey points. These roughness values are assumed to be representative until the next upstream survey point.

Floodplain roughness values for the transects were estimated from ortho-photography, field photographs at all Water Department survey points. These values are assumed to be representative until the next upstream survey point.

B.3.2 Hydrologic Model Development

Subcatchment Delineation

Subcatchment areas outside of the City were delineated by analyzing 1/3rd arc-second DEM to identify drainage divides. Further subdelineations at the subcatchment level were implied from topography, since limited to no collection system information was available. While, in general, subcatchment areas can be defined by surface topography, this is not always the case as subsurface drainage systems (sewers) can cross surface topographic divides. Subcatchment watersheds were delineated to selected points along the main channel at critical stream junctions or at pre-defined intervals. Since runoff from combined areas reaches the receiving water as a CSO input, sheds were defined for any watershed area not defined as a CSO subcatchment in the CSS Model. In this way runoff from areas defined as sanitary service areas, storm sheds, and non-contributing areas (to the collection system) was included in the Watershed Model. The defined sub-watersheds serve as the modeling unit in the Tributary H&H Model.

Overland Slope

In SWMM, runoff is calculated by approximating a non-linear reservoir which forms above the surface once the demands of infiltration, evaporation, and storage have been satisfied. The overland flow is generated using Manning's equation:

$$Q = \frac{1.486}{n} * W * (d - d_s)^{5/3} * S^{1/2}$$

Where:

Q = surface runoff (cfs),

W = width of watershed (ft),

S = average slope of watershed (ft/ft)

d = depth in the non-linear reservoir (ft),

n = Manning roughness coefficient, and

d_s = depression storage depth in the non-linear reservoir (ft).

Overland slope is a quantifiable physical parameter and is not adjusted during model validation. An average overland flow slope technique was used to define subcatchment slopes for the Cobbs Creek Tributary H&H Model.

The average slope values of the Cobbs sheds were found by using Spatial Analyst, a toolset in ArcGIS that analyzes and models raster based data. The average slope was calculated by using a

1/3rd arc-second DEM that was bounded by a polygon feature class representation of the model subcatchments. Within Spatial Analyst, the "Slope" function calculates the maximum rate of change in value from that cell to its neighbors. For this analysis, the percent slope was calculated and redrawn as a new raster file. The "Zonal Statistics" function was then used to average the slope values within the shed boundaries.

Width

The width parameter impacts the time of concentration and the hydrograph shape in the hydrologic portion of SWMM. Subcatchments are represented as rectangular areas defined by the subcatchment width parameter. By definition, the width of a watershed is equal to the area of the watershed divided by the length of the overland flow path. The width parameter is one of the main validation parameters used to adjust hydrograph shape, and to some degree hydrograph volume. Watersheds assigned a large width, and therefore a small overland flow length, will have a short time of concentration. Reducing width increases the flow length and will thus increase the time of concentration. Increasing the overland flow length also reduces the runoff volume as the flow is exposed to evaporation and infiltration over pervious areas for a longer period of time. Stormwater collection systems and minor undefined waterways are examples of how the overland flow length can be shortened, and the width increased. Overland flow length is a more intuitive parameter, and therefore is used as the basis for adjusting and discussing the SWMM width parameter. Since the width parameter is adjusted during validation, it is only necessary to obtain an approximate initial value.

The initial estimates for watershed width were obtained by calculating two times the square root of the watershed area. This approach provides a width based on an idealized square watershed along the middle of the watershed. Since actual watersheds are not rectangular with properties of symmetry and uniformity, it should be expected that these values will be widely adjusted during the validation process.

Gross Impervious Cover

Gross percent impervious area was estimated from the 2006 National Land Cover Database impervious land coverage. An estimate of total impervious area for model subcatchments was based on clipping the impervious surface coverage with the Cobbs Creek subcatchment shed coverage and calculating the percent impervious cover for each subcatchment.

The estimate of percent impervious cover of total shed area was entered into the model as percent impervious for each subcatchment and was kept constant during the tributary validation process. Gross impervious area was not adjusted during the validation process. The influence of impervious cover on total shed stormwater runoff was controlled by the percent routed parameter, which was considered to be the impervious runoff validation parameter and simulated the effect of directly connected impervious area (DCIA).

Percent Impervious Cover Routed

SWMM5 simulates surface runoff from drainage areas using three "planes" of overland flow. One plane represents all impervious surfaces directly connected to the hydraulic system and included initial abstraction or surface detention storage (puddles, cracks, etc. which do not

permit immediate runoff). A second plane represents all pervious areas and impervious areas not directly connected to the hydraulic system. The third plane is defined as the fraction of the directly connected area that provides no detention storage and thus produces runoff immediately. Furthermore, a portion of the pervious runoff can be routed to the impervious surface (i.e. Pervious Routing), or a portion of the impervious runoff can be routed to the pervious surface (i.e. Impervious Routing). The runoff from the drainage area is the sum of the flow from the three planes, after considering internal routing.

Impervious surfaces, by definition, do not have an infiltration component. However, impervious surfaces are not always connected to a collection or conveyance system, and may instead route to a pervious surface where the runoff generated has the opportunity to infiltrate. The fraction of the total area that is impervious and drains directly to the collection or conveyance system is defined as DCIA. In SWMM, the “% Impervious” hydrologic parameter represents the gross impervious area as a percentage of the total shed area. The PERVIOUS method of subcatchment routing is used to approximate hydrograph flow paths from the subcatchments. When the PERVIOUS method is used, a value for the percent of total runoff from the impervious area that is routed over the remaining pervious area of the shed is defined. The remaining impervious area not routed is termed the DCIA, which drains directly to the collection and conveyance system. The PERVIOUS routing method allows for complex hydrographs to be reproduced as DCIA is manifested as a near immediate system response to precipitation, while the pervious area runoff may have a slower response.

Soils

The rate of infiltration is a function of soil properties in the drainage area, ground slopes, and ground cover. The Green-Ampt method of calculating infiltration was employed for the Cobbs Creek Tributary H&H Model. The Green-Ampt equation for infiltration has physically based parameters that can be estimated based on soil characteristics. The soil parameters used in this method are: average capillary suction, saturated hydraulic conductivity, and the initial moisture deficit. In SWMM, pervious area runoff is generated when rainfall volume and intensity exceed the soil infiltration capacity and evaporation demands. Infiltration is primarily controlled by setting the soil saturated hydraulic conductivity, which is that rate at which water will infiltrate the soil after the soil has reached a point of saturation. By lowering the saturated hydraulic conductivity, the pervious area begins to generate runoff. Once the saturated hydraulic conductivity threshold for pervious runoff is found, additional increment adjustments downward tend to produce very significant responses. Therefore, saturated hydraulic conductivity was a primary validation parameter.

Soil information for the Cobbs watershed was obtained from the United States Department of Agriculture’s Natural Resources Conservation Service, which is responsible for collecting, storing, maintaining, and distributing soil survey information for lands in the United States. Infiltration parameters were assigned to each subcatchment based on soil texture classification. Table B-1 lists the parameter ranges for several different soil classifications. Soil classifications can vary widely within each subcatchment, and a weighted value was calculated for the three parameters. It should be noted that 80% of the Cobbs Creek watershed is classified as either

“Urban Land” or “Made Land” and infiltration values can vary greatly for these classifications and are site-specific. This limitation of soils data, and the modification of native soils and various degrees of compaction due to construction within Philadelphia and the surrounding municipalities implies that infiltration parameters are very significant in the validation process.

Flow monitoring at trunk sewers throughout the service area was critical in determining infiltration parameters for the CSO sheds in the CSS Models. This flow monitoring data was used in validation to estimate infiltration parameters such as hydraulic conductivity for monitored CSO sheds. These established infiltration parameters were used for CSO subcatchments with origins in the CSS model. The hydraulic conductivity for the subcatchments in the Tributary H&H Model outside of the CSO area was determined through model validation at the USGS gages.

Table B-1: Soil Infiltration Parameter Estimates for Several Soil Texture Classifications

USDA Soil Texture Classification	Saturated Hydraulic Conductivity (in/hr)	Initial Moisture Deficit for Soil (Vol. Air / Vol. of Voids, expressed as a fraction)	Avg. Capillary Suction (in)
		Moist Soil Climates (Eastern US)	
Sand	9.27	0.346	1.95
Loamy Sand	2.35	0.312	2.41
Sandy Loam	0.86	0.246	4.33
Loam	0.52	0.193	3.50
Silt Loam	0.27	0.171	6.57
Sandy Clay Loam	0.12	0.143	8.60
Clay Loam	0.08	0.146	8.22
Silty Clay Loam	0.08	0.105	10.75
Sandy Clay	0.05	0.091	9.41
Silty Clay	0.04	0.092	11.50
Clay	0.02	0.079	12.45

Depression Storage

Depression storage is the rainfall abstraction volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. In SWMM5, every subcatchment is divided into three subareas: pervious area with depression storage, impervious area with zero depression storage, and impervious area with depression storage. Water stored as depression storage on pervious areas is subject to infiltration and evaporation. Water stored in depression storage on impervious areas is depleted only by evaporation, therefore replenishment of the retention storage typically takes longer when compared to pervious areas. Both impervious and pervious depression storage were adjusted as validation parameters. For all subcatchments, impervious depression storage was initially set as 0.063 inches and pervious depression storage was set at 0.25 inches, (Viessman and Lewis, 2002). By default, the model assumes 25% of the impervious area has zero depression storage. This default value was not altered in the setup of the Cobbs Creek Tributary H&H Model.

Subcatchment Manning's Roughness

Manning's roughness values must be estimated for both pervious and impervious overland flow. For impervious areas, these values can be estimated fairly easily. For pervious areas the estimation is somewhat more difficult as there can be considerable variability in ground cover along the flow path. Table B-2 lists Manning's n estimates for several types of surfaces. For the Cobbs Creek Tributary H&H Model, all subcatchments were assigned a Manning's n value of 0.013 for impervious areas and 0.02 for pervious areas.

Table B-2: Roughness Coefficients (Manning's n) for Sheet Flow

Surface Description		n
Smooth surfaces	Concrete, asphalt, gravel, or bare soil	0.011
Fallow	No residue	0.05
Cultivated soils	Residue cover < or = 20%	0.06
	Residue cover > 20%	0.17
Grass	Short grass prairie	0.15
	Dense grasses	0.24
	Bermudagrass	0.41
	Range (natural)	0.13
Woods	Light underbrush	0.40
	Dense underbrush	0.80

Source: US Department of Agriculture, June 1986

Stormwater Event Mean Concentrations

The development of the EMC dataset used for the Tributary H&H Models is detailed in Section 3.7.1. The vast majority of data used developing this EMC dataset were obtained from urban runoff monitoring programs, which represents the urban land use throughout the Water Department service area, but does not necessarily represent the suburban and rural land use throughout the upstream portions of the watershed drainage area. However, the rainfall-runoff response is overwhelmingly driven by impervious surfaces, which would represent the urbanized portions of the watershed. Therefore, applying these EMCs universally to all subcatchments (both city and suburban) in the Tributary H&H Models was considered a reasonable and conservative approach for modeling the runoff pollutants. The water quality function was used in SWMM to determine bacteria loadings at the outfalls for the WASP water quality models.

B.3.3 Model Input

Precipitation

Radar rainfall was obtained from Vieux & Associates, Inc. and processed to be used for the hydrologic validation period (years 2011 and 2012) and the water quality validation period (year 2003). The radar grid was calibrated to the existing Water Department rain gage network, consisting of 24 gages within the city limits. Precipitation for each model subcatchment was calculated by area weighting 1 km² radar grids intersecting the individual subcatchment boundaries. The radar-rainfall data was necessary to provide coverage for sheds outside the coverage of the Water Department gage network. This improved the simulation of events with complex hydrographs and quick runoff responses. In a rainfall-runoff model, much of the uncertainty in the model results from the uncertainty inherent in the precipitation data. Therefore, using a detailed precipitation input that accounts for the temporal and spatial distribution in rainfall intensities and volumes, increases the accuracy and precision of the model results. Compared to using the Water Department rain gage network directly, radar rainfall has the potential to better represent the spatial distribution of rainfall between gages within the City, and for locations outside the rain gage network.

At the time of writing this report, the above mentioned radar rainfall data was not readily available for the year 2000 bacteria validation event for Cobbs Creek. For this event, instead of using radar rainfall to drive the model, the Water Department rain gage network was used directly.

Evaporation

Evaporation data was loaded into the model in the form of average monthly evaporation rates. Evaporation data usually can be obtained from the National Weather Service or from other pan measurements, however, long-term daily evaporation data was limited in the Philadelphia area. Neither the Philadelphia Airport nor the Wilmington Airport recorded evaporation data. One site located in New Castle County, Delaware recorded daily evaporation data from 1956 through 1994. Average daily evaporation (inches per day) from this site was used to estimate typical monthly evaporation rates, which were used by the model.

Baseflow

Baseflow is required for each tributary in order to perform the in-stream hydrological validation of the watershed models. An estimate of baseflow was calculated using USGS gage 01475548. Baseflow is mostly comprised of groundwater discharge to the stream, while runoff is a result of overland discharge. Baseflow separation involved disaggregating monitored flow time series into wet and dry components based upon expected hydrological response times. In order to extract a baseflow from monitored streamflow data, the data set was first merged with rainfall data. All data during wet weather and 72 hours afterwards was removed to isolate dry weather flows. An average annual baseflow was estimated and used as a dry weather input to the model. The dry weather flow was weighted by contributing watershed area. Baseflow values were controlled within SWMM5 as multipliers on a constant dry weather inflow at the inflow nodes.

B.4 Validation

B.4.1 Validation Approach

Validation of the Tributary H&H Model was performed using flow data collected at USGS monitoring locations along the Cobbs Creek. Flow monitoring data was available for the 2011-2012 validation period. Initial estimates of selected variables were adjusted within a specified range until a satisfactory correlation between simulated and measured stream flow, over a range of events, was obtained. Stormwater runoff and the associated pollutant loadings from the watershed areas, including the stormwater contributions from separate sanitary sewered areas, were input to the Tributary H&H Model at nodes along the tributary networks. Flow from CSOs upstream of the USGS gaging stations is captured in the stream flow data, so is therefore a part of the validation dataset. However, flow from CSOs was considered fixed by the overflow validation process in the CSS Model, and therefore CSO shed parameters were not adjusted during validation of the Tributary H&H Model.

As discussed in Section B.3.2, runoff in SWMM is generated when the precipitation exceeds the demands of infiltration, evaporation, and storage. The rate of runoff is determined by the subcatchment area, width, slope, depth of the non-linear reservoir, and the roughness coefficient. Subcatchment area, slope, and total impervious area are quantifiable parameters and, therefore, were not adjusted during the validation process. Infiltration parameters, width, and percent routed were the primary parameters adjusted during validation. In addition, depression storage and overland roughness coefficients were adjusted during the validation process. Depression storage and overland roughness parameters were set independently for the pervious and impervious portions of the subcatchments.

The first phase of validation utilized the hydrologic parameters that control event hydrograph volume, namely:

- Percent Routed / DCIA
- Saturated Hydraulic Conductivity
- Initial Soil Moisture Deficit
- Soil Capillary Suction Head

The second phase of validation utilized the hydrologic parameters that control event hydrograph timing and peak, namely:

- Subcatchment Width
- Impervious and Pervious Depression Storage

B.4.2 Model Validation Data

Streamflow

The model was validated with streamflow at USGS gaging sites along the Cobbs Creek. It was important for hydrologic validation that both the Tributary H&H Model and the flow monitoring record included the contributing wet weather flows from the City CSOs, since this flow accounts

for a sizable portion of the stream flow in this urban stream. Instantaneous USGS flow monitoring data in 15 minute intervals was available over recent time periods for two gages along the Cobbs Creek.

Presently there are two gages along the Cobbs Creek, 01475530 and 01475548. Gage 01475530, located at the Route 1 bridge, is the more upstream gage and near where the stream passes through the City border. Gage 01475548 at the Mt. Moriah Cemetery is the more downstream gage. The Mt. Moriah gage is approximately two-thirds the river mile distance downstream of the Route. 1 gage to the mouth of the Cobbs Creek Watershed (confluence with Darby Creek) and above the influence of tide. Current USGS gage data from 2011 through 2012 was used in favor of historical daily records for hydrologic validation. Current data better represents existing wet weather response of the watershed. The data is recorded in 15 minute increments, which provides a finer resolution for hydrographs. Flows from the Route 1 gage were used to calibrate the model primarily outside the City, while the lower gage at Mt. Moriah was used to validate the watershed areas below the upstream gage. Section 2.2.2 contains additional discussion of USGS gage station rating curve development and published flow data error ranges.

Event Definition

Wet weather events were defined for each monitored dataset using the flow and precipitation monitoring records. A total of 110 wet-weather events at USGS Gage 01475530 (Rt. 1), and 109 wet weather events at USGS Gage 01475548 (Mt. Moriah) were defined within the hydrologic validation period of January 2011 through December 2012. The monitored and predicted hydrographs were split into discrete wet weather events over time, so comparisons could be made on an event by event basis. Events were defined not by continuous rainfall, but continuous wet weather response. Additionally, since snowmelt was not simulated, snowfall and all potential snowmelt events were removed from the validation data set. This determination was based on precipitation and temperature data obtained from the Philadelphia International Airport.

The defined events were mostly similar in duration, except for small differences in event timing, and other differences due to the spatial distribution of rainfall. There was a higher percentage of pervious cover contributing to the upper gage at Route 1, so the events responded slower, with more prolonged wet weather tails. Consequently, a few of the events at Route 1 were merged, based on timing and extended wet weather tails, as compared to the Mt. Moriah events.

During the validation process, each of these events was evaluated repeatedly to compare the observed and simulated event statistics for event volume and peak flow. The observed and simulated hydrographs were compared for their general response shape and timing. During the validation process, the model hydrologic parameters were adjusted to provide a better fit between the simulated and monitored flows. Care was taken not to include the influence of events which had unusually large or small responses to the rainfall hyetograph so that the overall model validation would not be skewed by a select number of events. The ratio of monitored flow to precipitation volume over the contributing watershed area was also used as a reason to exclude events from the validation process. If there seemed to be a mismatch between rainfall and runoff response or hydrograph shape and timing, the event was deemed an outlier.

B.4.3 Validation Results

Flow monitoring data from USGS stations USGS Gage 01475530 Cobbs Creek at Route 1 and 01475548 Cobbs Creek at Mt. Moriah were used to perform the hydrologic validation of the Cobbs Creek Tributary H&H Model. The upstream gage, Gage 01475530 at Route 1 is close to the City border and was used to validate the more upstream sheds that mostly lie outside the City and the Water Department service area. The downstream gage, 01475548 at Mt. Moriah was used to perform the hydrologic validation for the watershed area below the Route 1 gage. Model flow time series were compared to observed flow for the events defined at both gages within the hydrologic validation period of January 2011 through December 2012.

Since the percent DCIA value was fixed as the total impervious area of the watershed subcatchments, it was not adjusted during validation. The model parameter adjusted to represent the effect of directly connected impervious areas was the “percent routed” parameter, since the pervious method of subcatchment routing was employed to represent the routing of runoff from impervious areas onto adjacent pervious areas. Through validation the percent routed parameter was determined to be 30% and was universally applied to all watershed subcatchments. This takes into account both the upstream and downstream gages.

Width was increased by 50% of the initial value for sheds above the Route 1 gage to adjust peak flows at the Route 1 gage. This value, which is larger than the initial area-based estimates, was increased to reduce peak flows and to extend the simulated hydrograph. The initial width estimate of 2 times the square root of the shed area was assigned to the Cobbs Creek watersheds below the Route 1 gage.

The saturated hydraulic conductivity was adjusted to an average value of 0.22 in/hr across the watershed subcatchments. This value was increased from the initial estimate based on the GIS soil information. The saturated hydraulic conductivity was determined by examining validation results at both gages.

The volumetric validation plot for Cobbs Creek at the Route 1 gage is presented in Figure B-1. Volume is predicted fairly well at this gage by the model as indicated by achieved a best fit slope of 0.9515 and an R-Square value of 0.9137 when modeled flow is plotted against monitored flow.

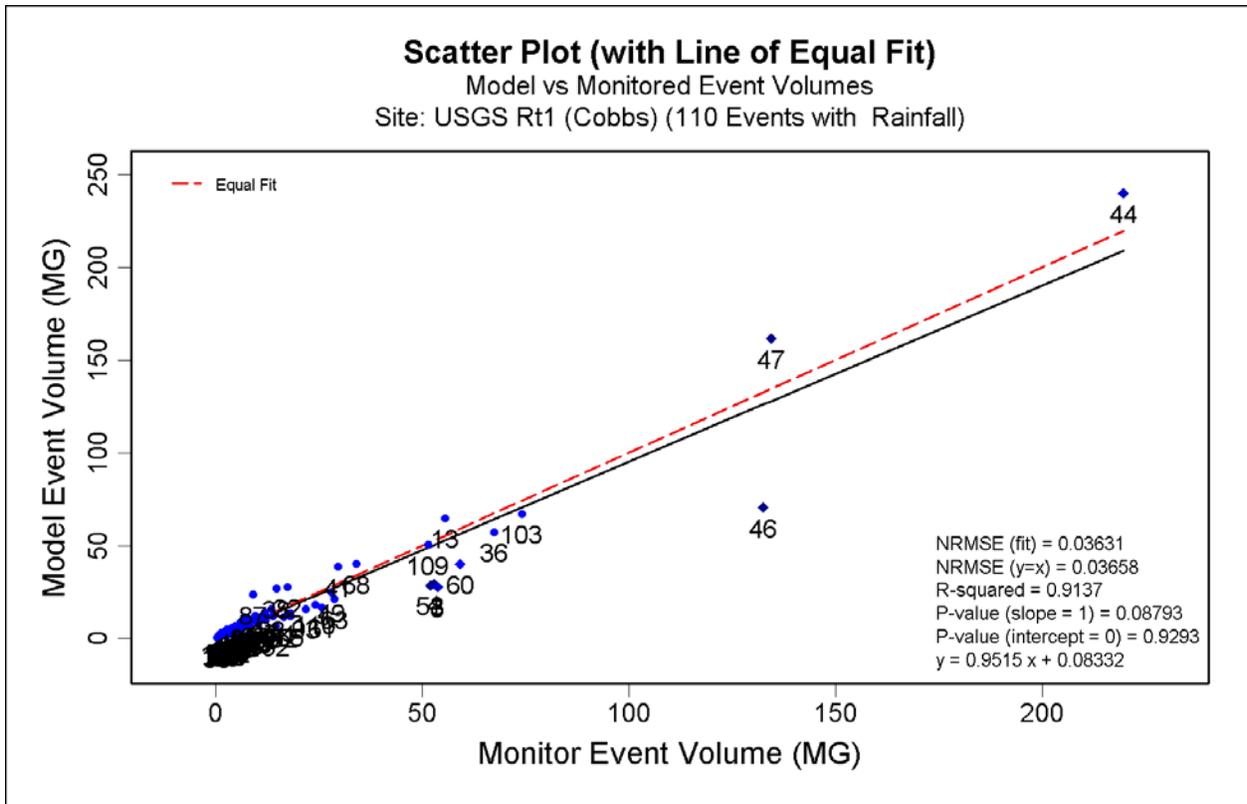


Figure B-1: Cobbs Creek Volume Validation at Gage 01475530 (Route 1)

The volumetric validation plot for Cobbs Creek at the Mt. Moriah gage is presented in Figure B-2. Overall, the model predicts volume well. The regression is driven by a few extreme events, however moderate and small sized events are also simulated well. The plot of modeled flow against monitored flow achieved a best fit slope of 0.9928 and an R-Square value 0.9001. The validation plot for Event 100, October 28, 2012 is provided in Figure B-3 as an example of a medium to large-sized event where the monitored and modeled hydrographs match very well.

The validation results show that the model adequately represents the monitored data, and that overall it provides a good representation of the flows in Cobbs Creek.

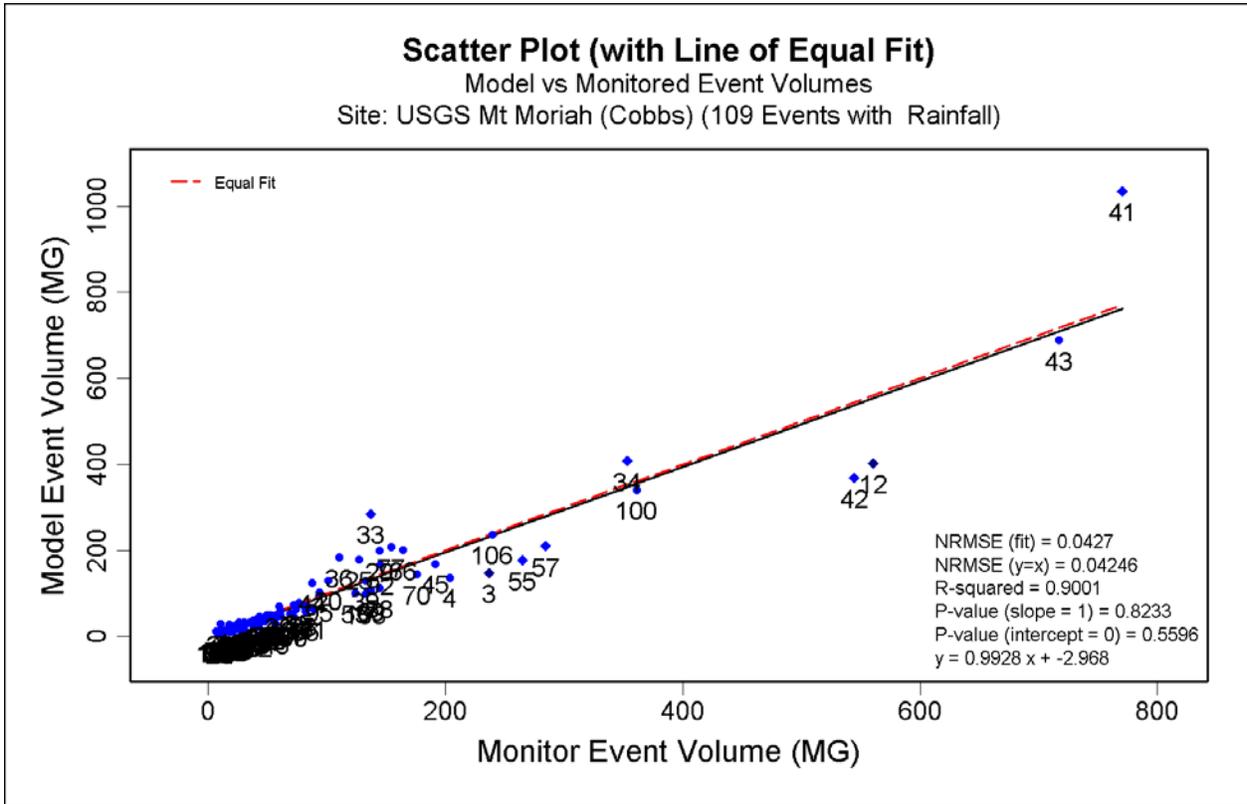


Figure B-2: Cobbs Creek Volume Validation at Gage 01475548 (Mt. Moriah)

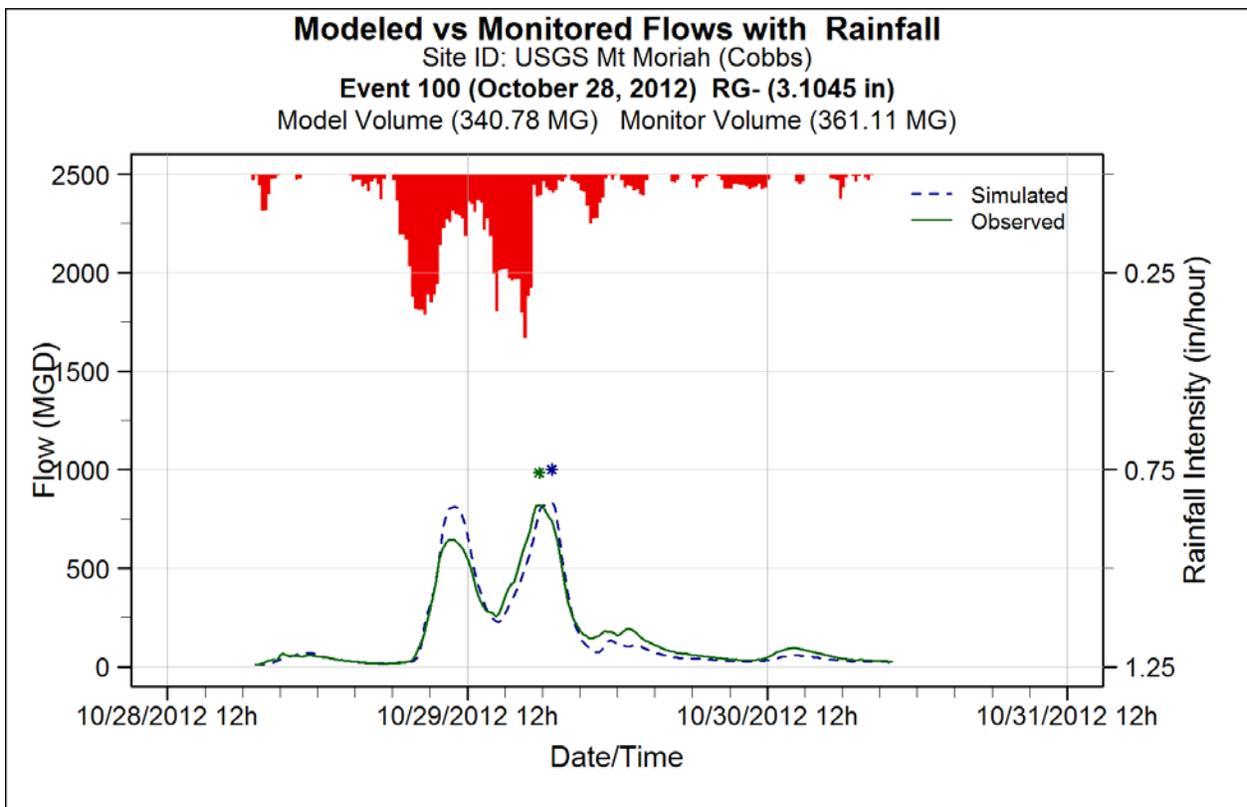


Figure B-3: Cobbs Creek Example Validation Plot at 01475548 (Mt. Moriah)

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Appendix C

Tookany/Tacony-Frankford Creek Bacteria Model Validation Simulations

Appendix C: Tacony Creek Bacteria Model Validation Simulations:

This report compares observed bacteria data and WASP predicted outputs for the following eight storms: May 06, 2003, May 07, 2003, May 16, 2003, July 10, 2003, Sept 23, 2003, Oct 14, 2003, July 07, 2004, and Aug 30, 2004.

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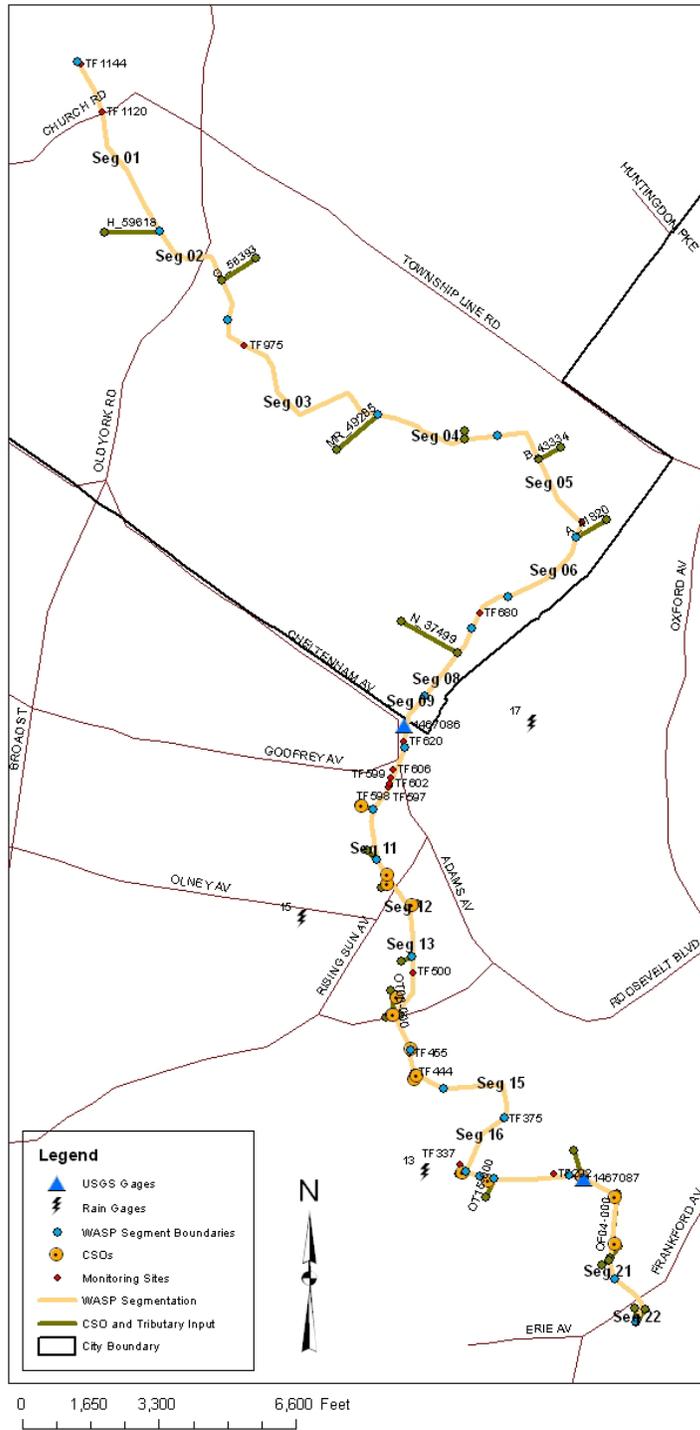
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Chapter 1. General Information

Table 1-1. TTF Monitoring Locations

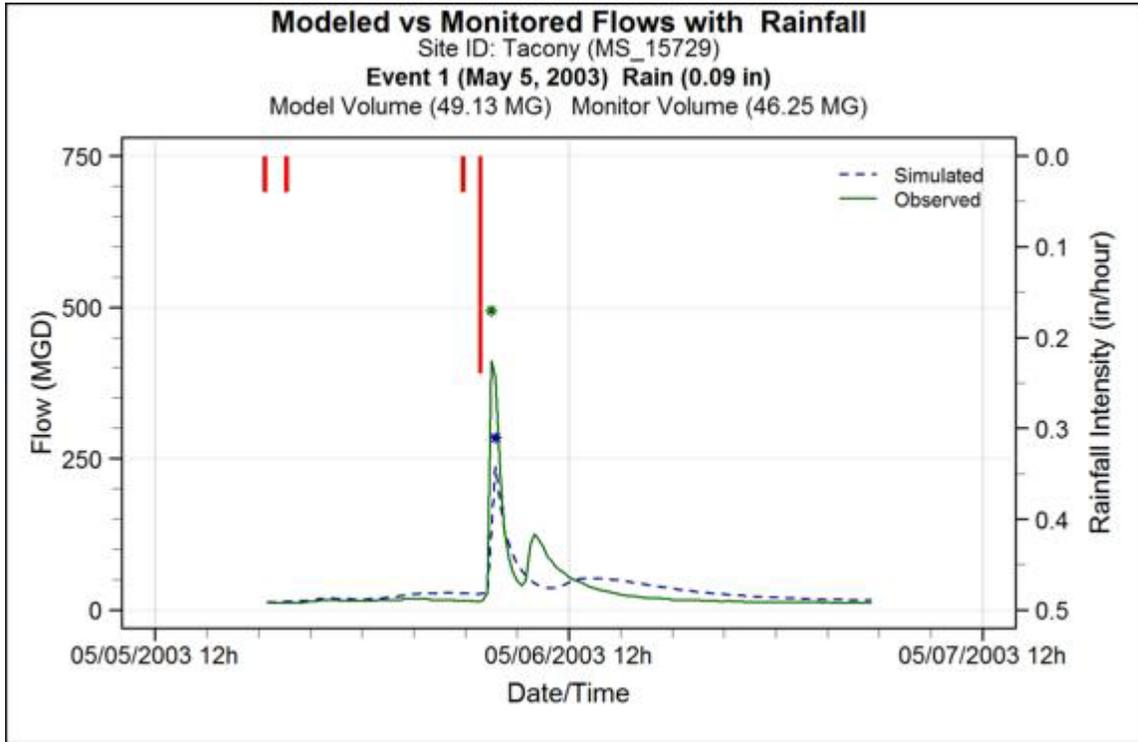
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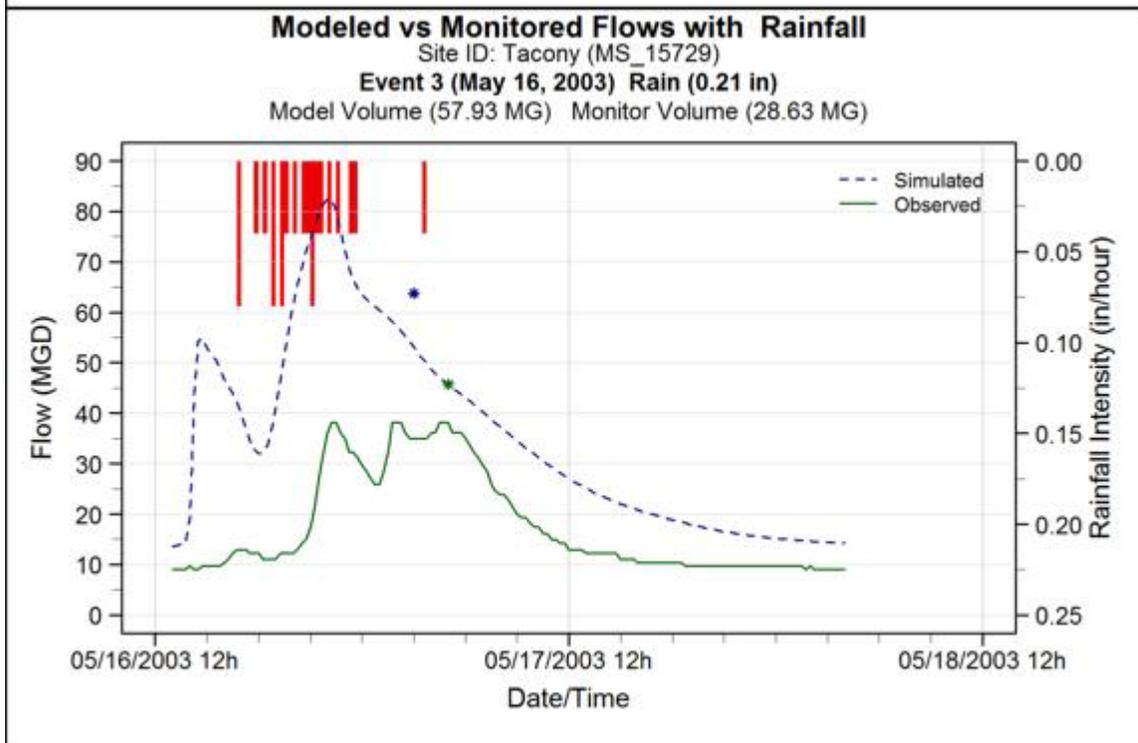
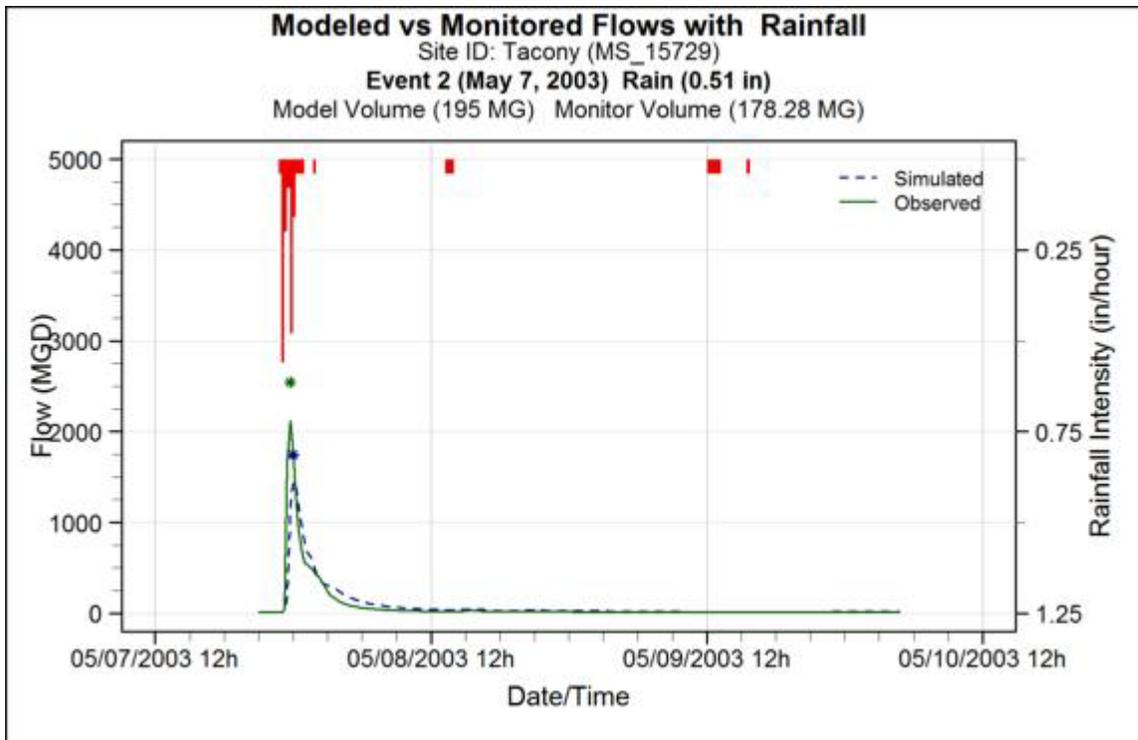
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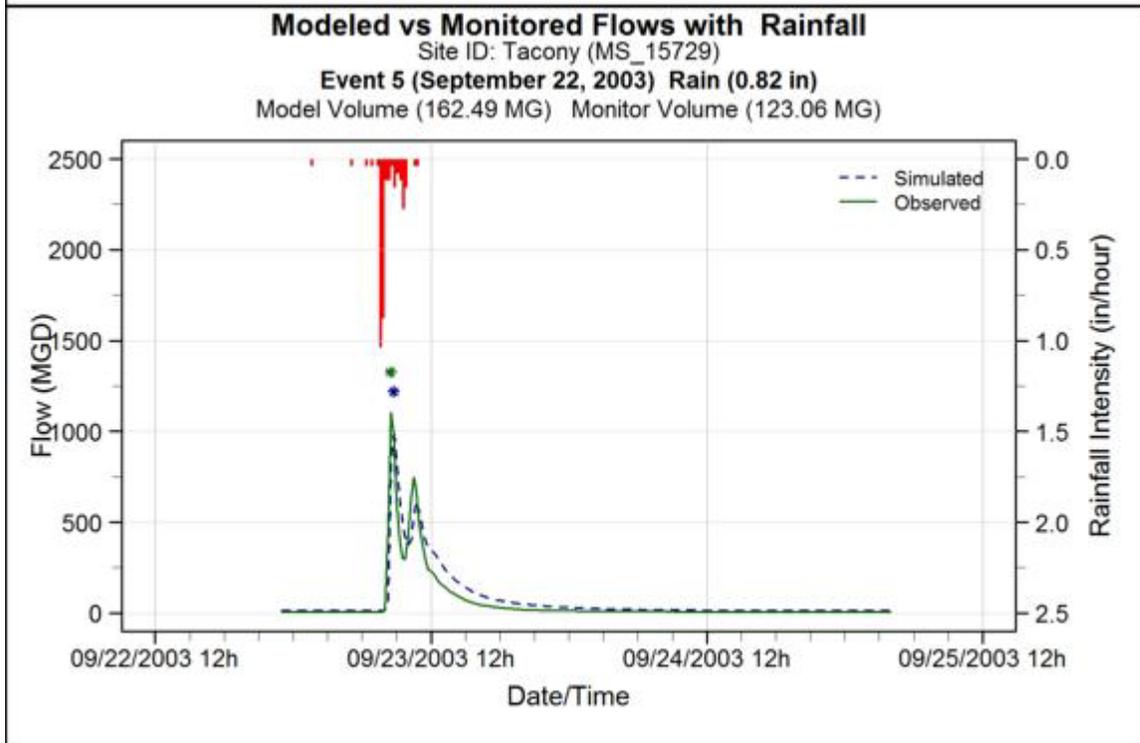
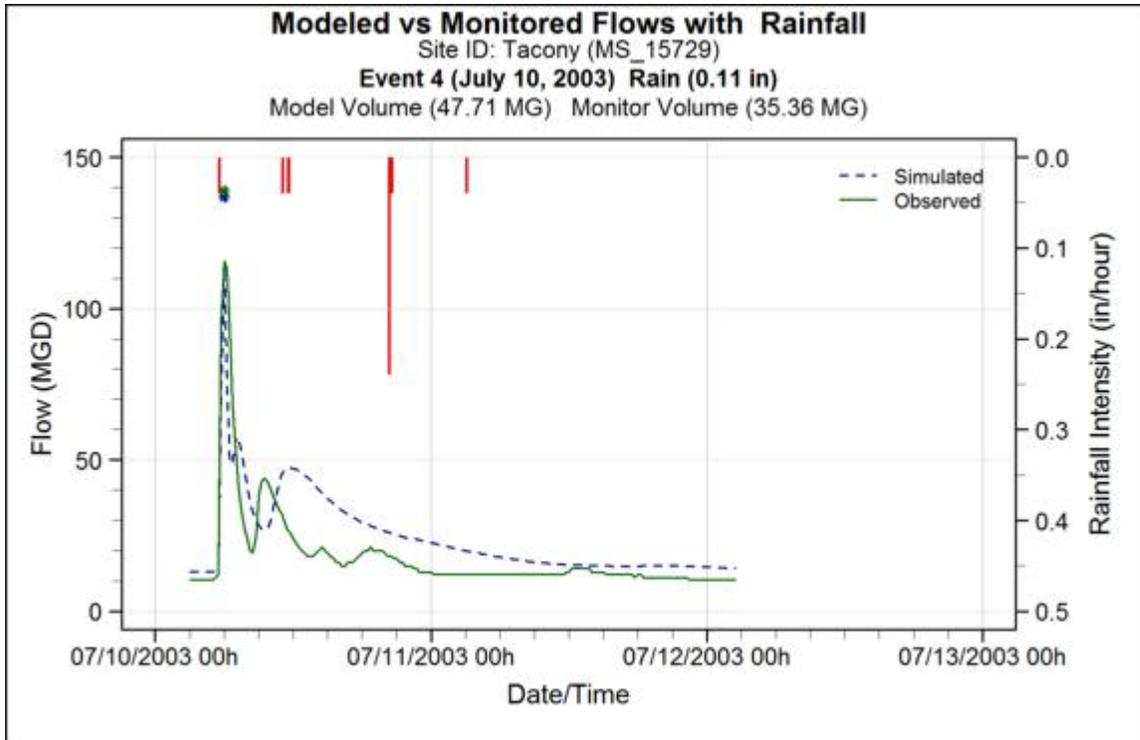


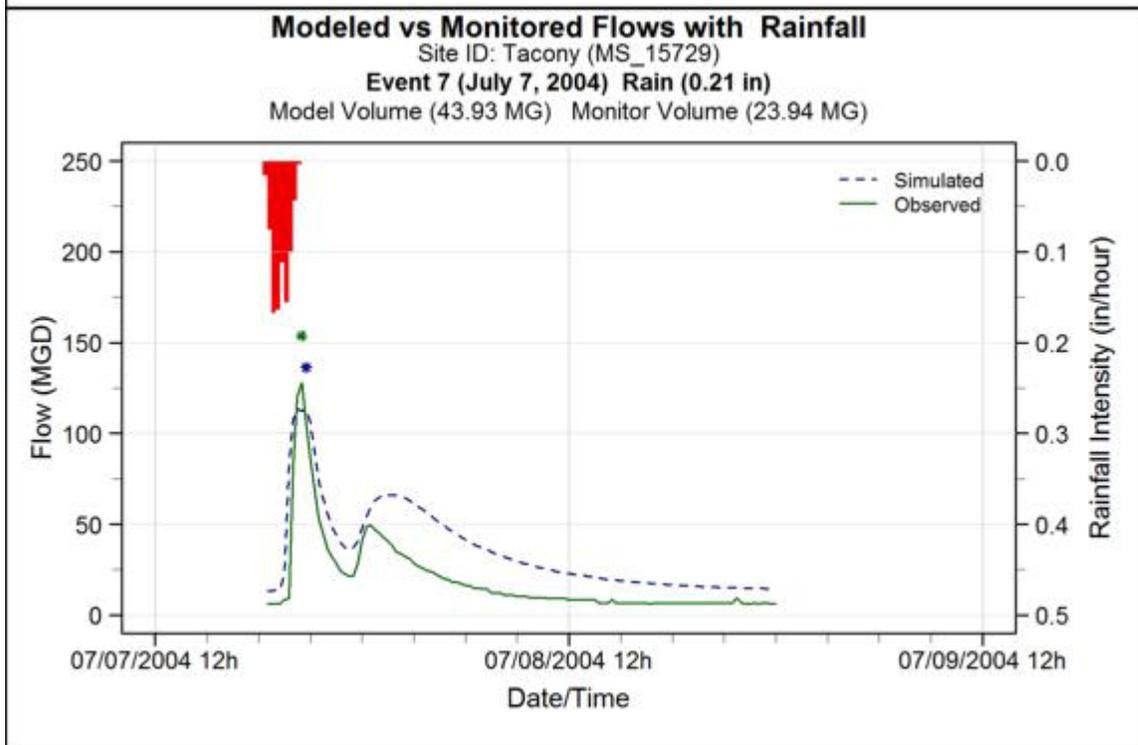
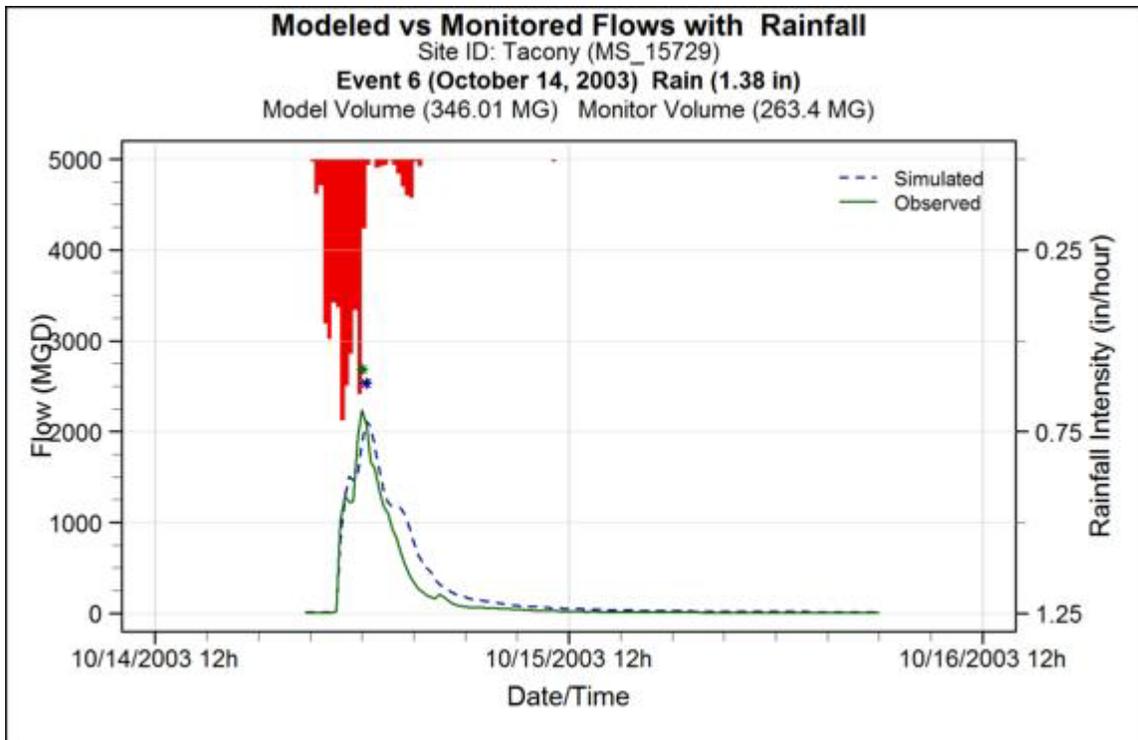
Chapter 2. SWMM Model Validations

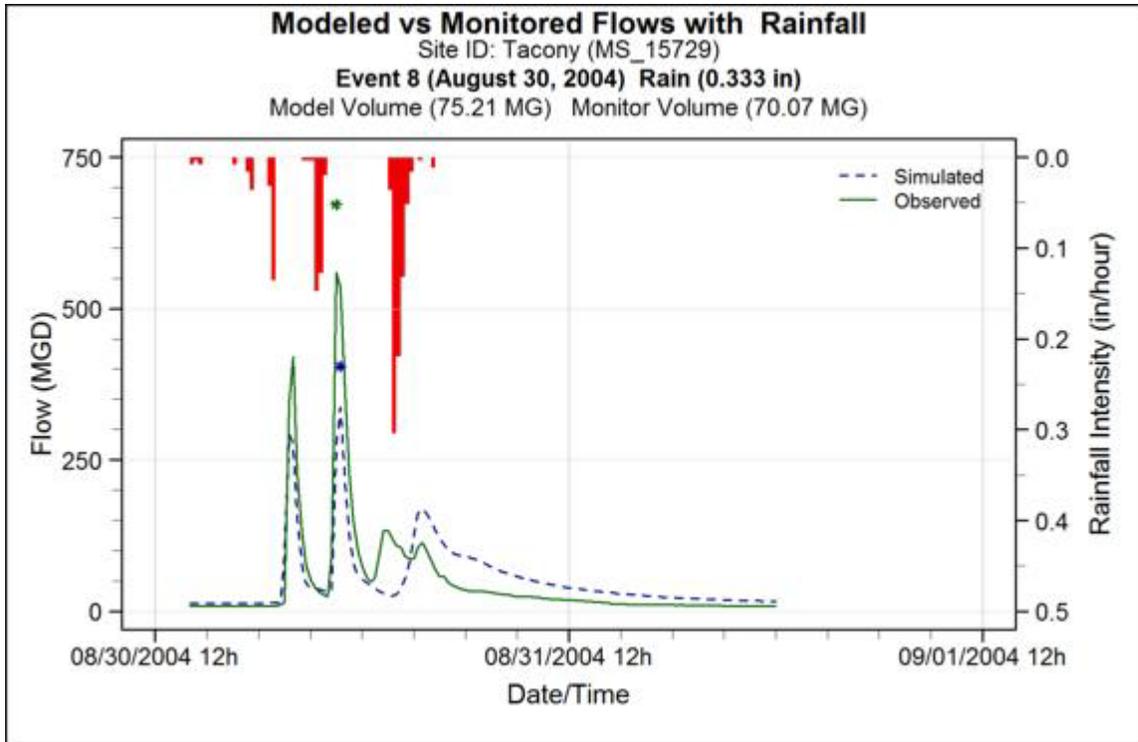
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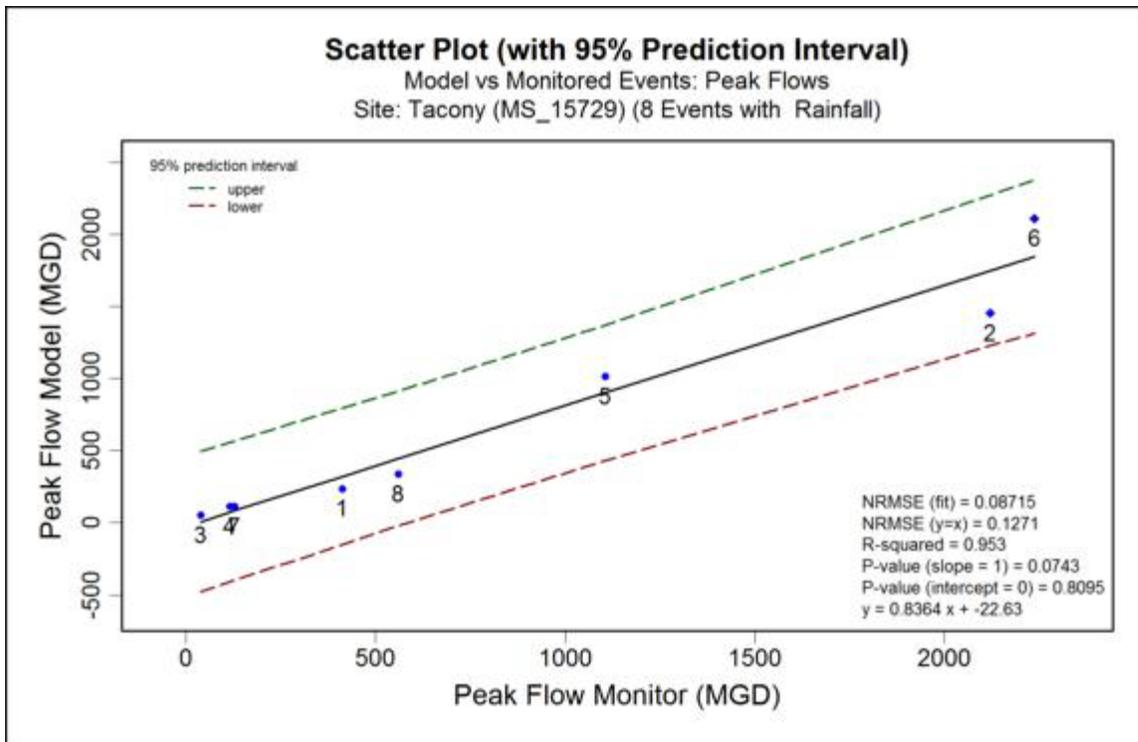


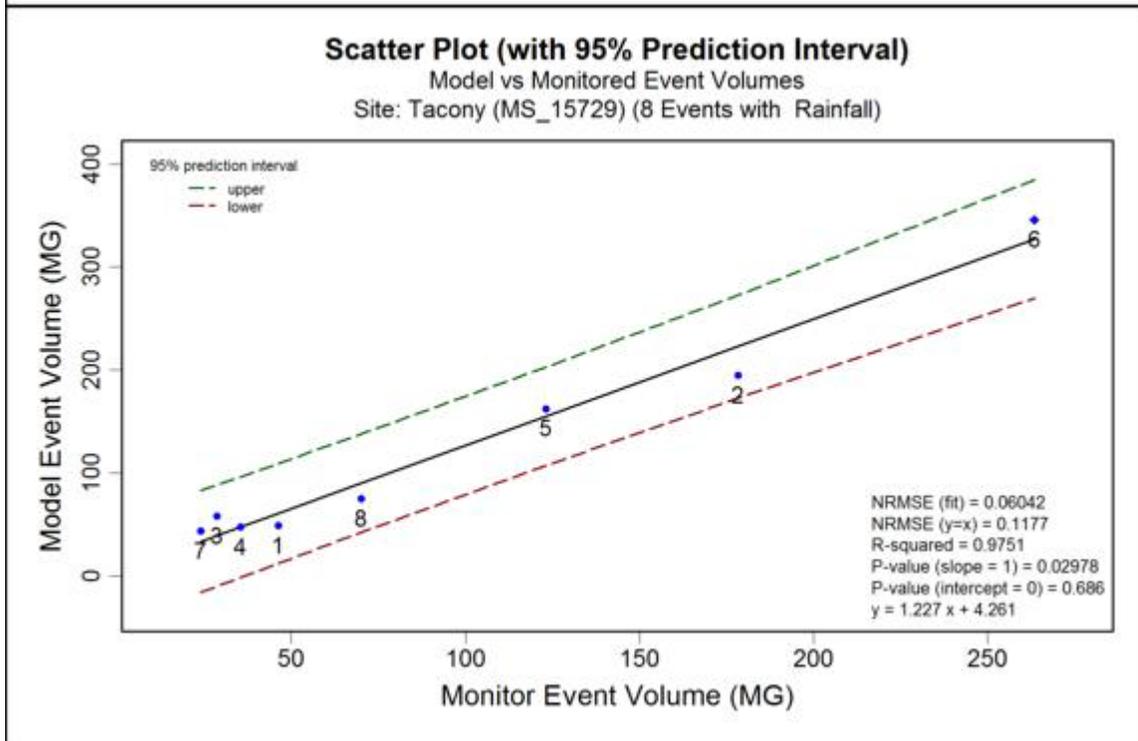
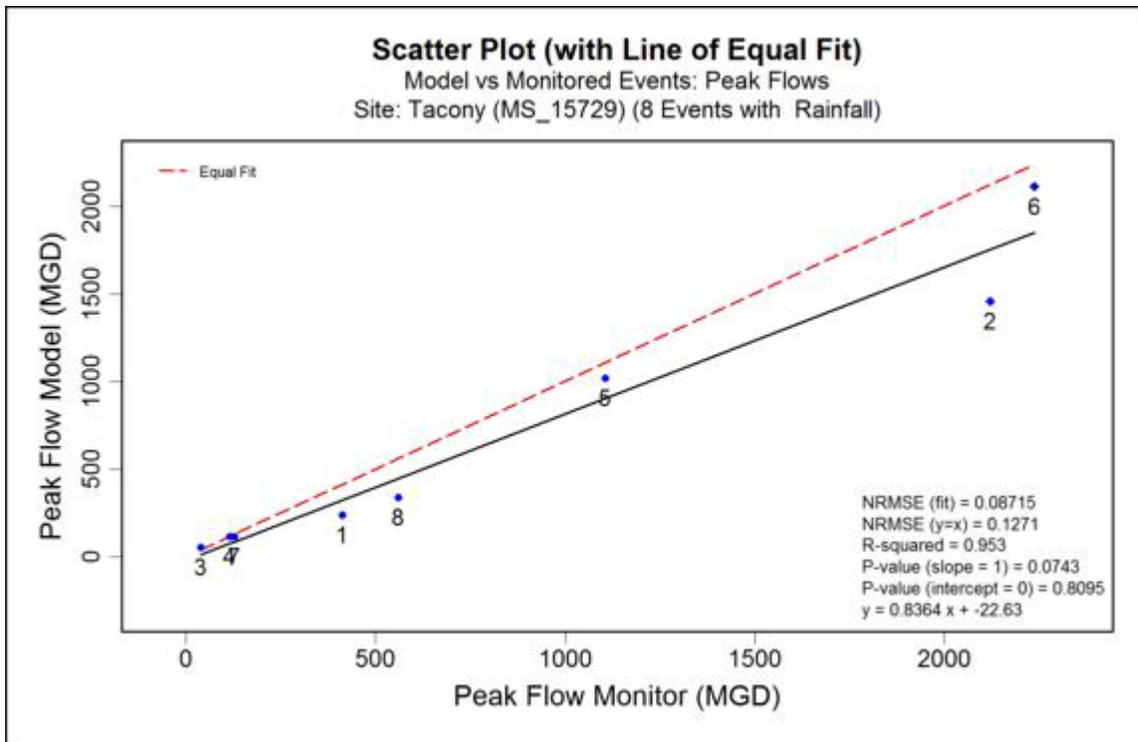


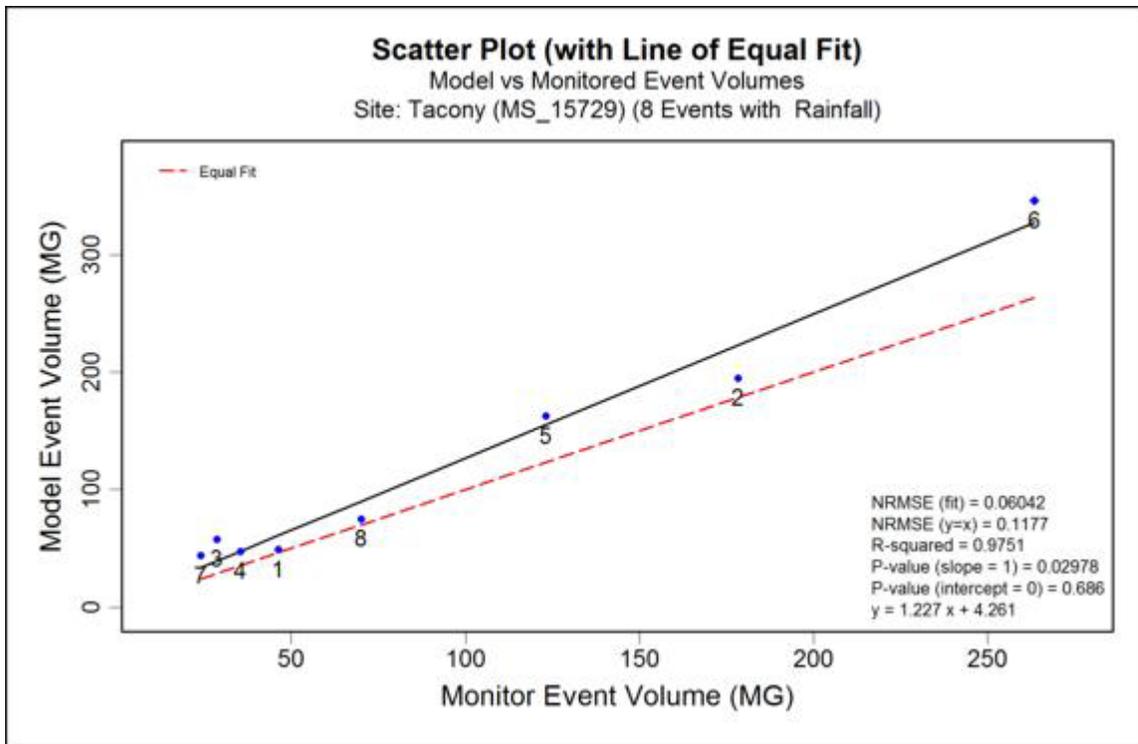




2.2. Validation Scatter Plots



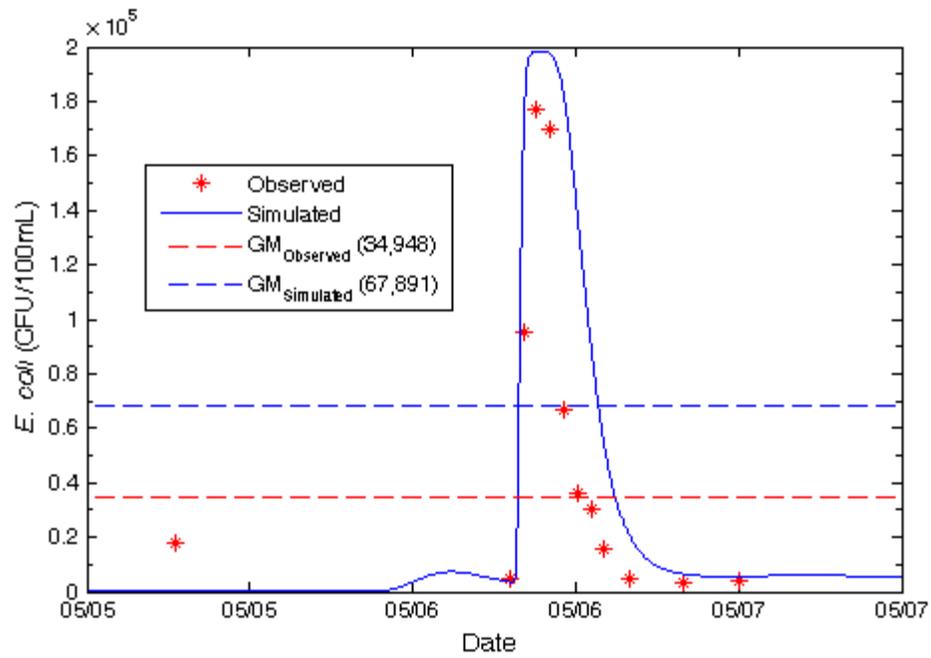




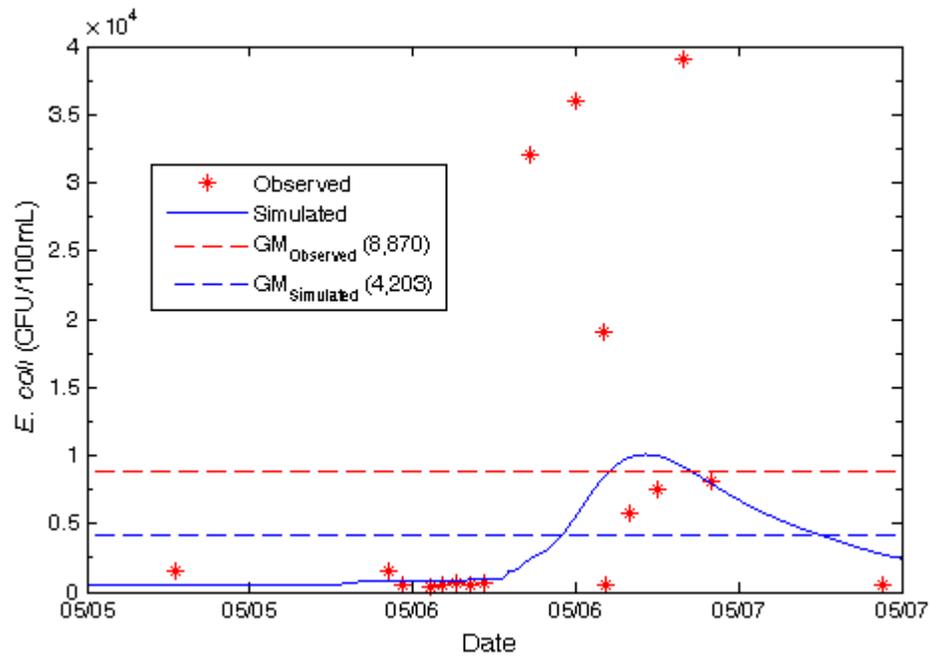
Chapter 3. *E. coli*

3.1. *E. coli* Time Series

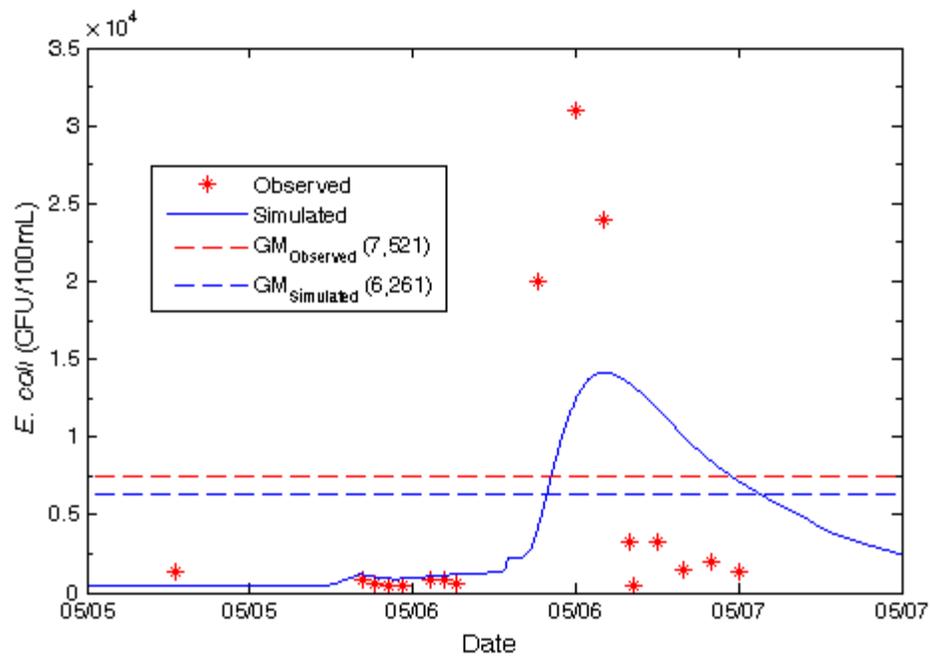
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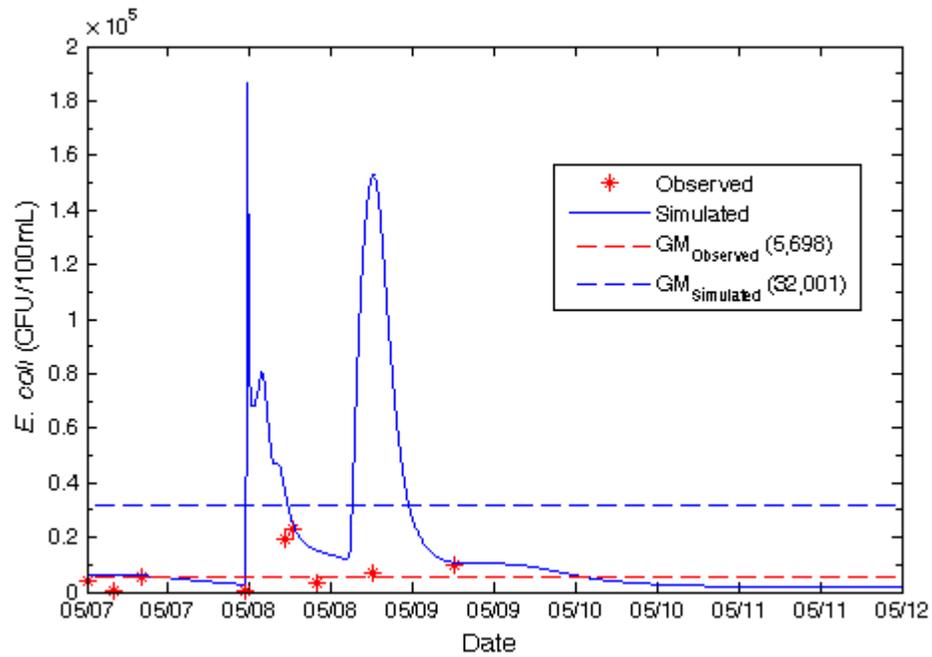
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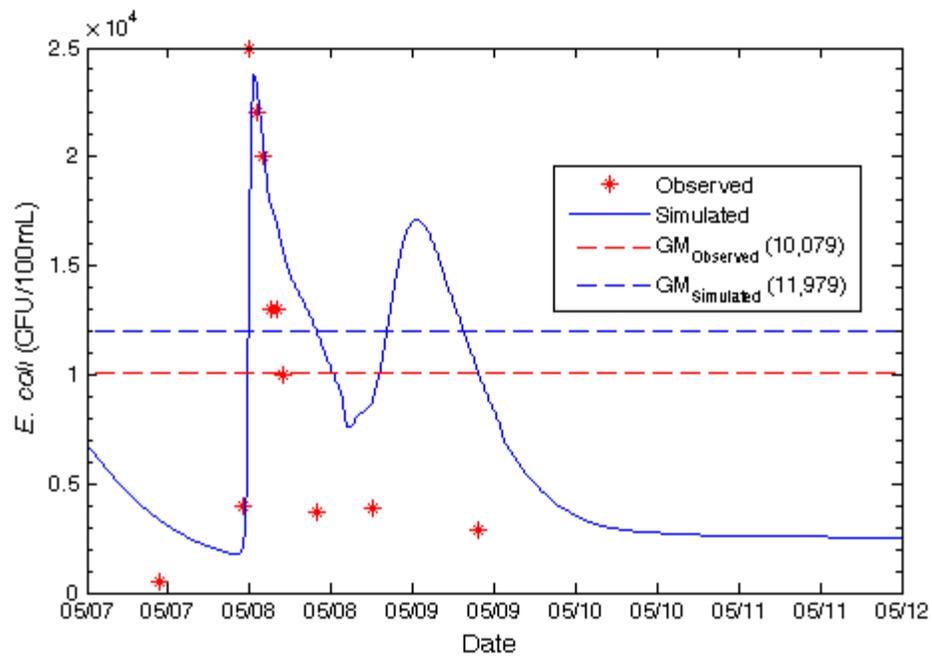
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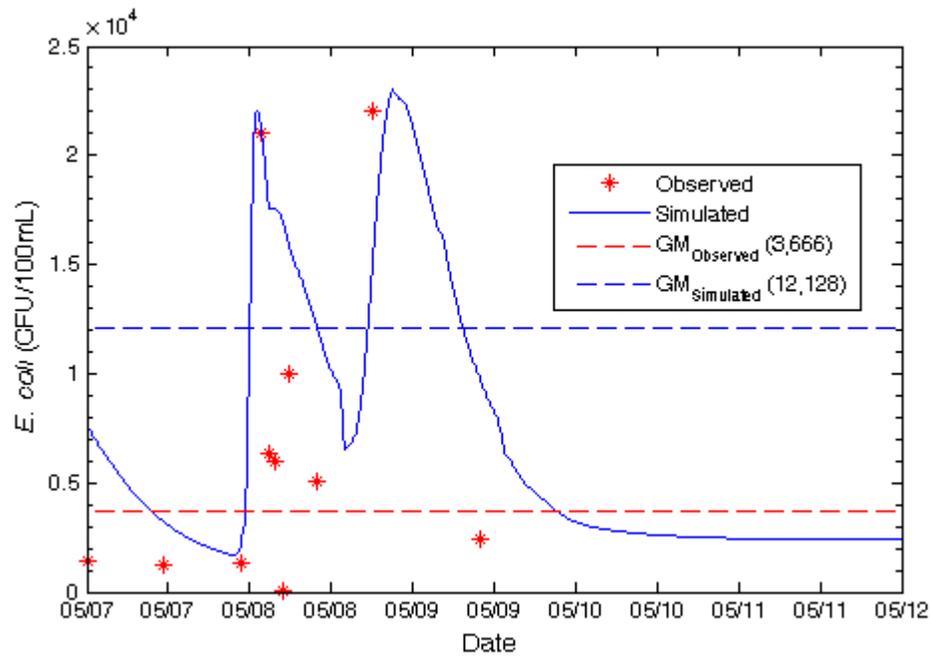
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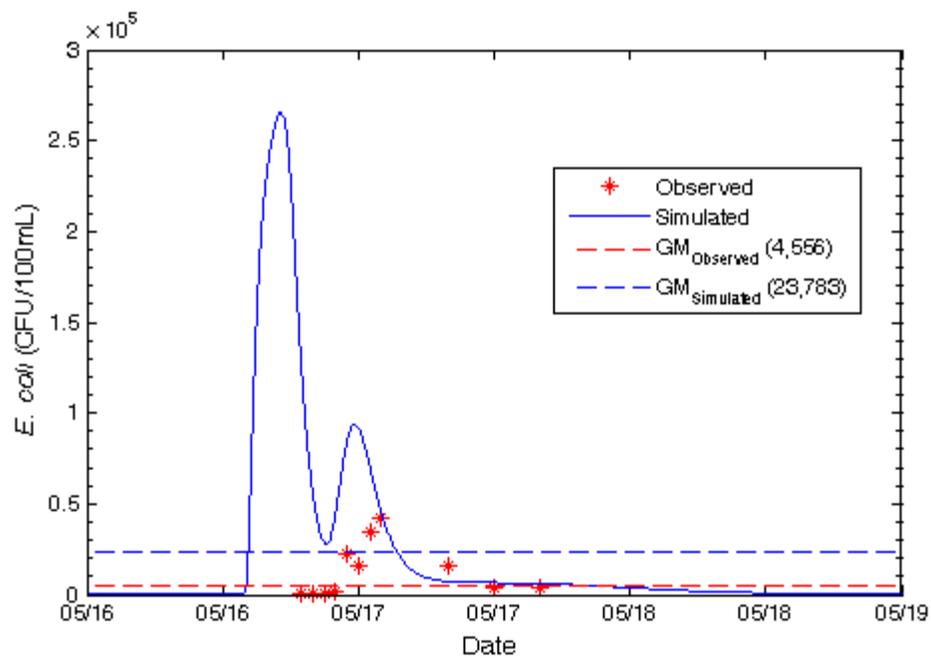
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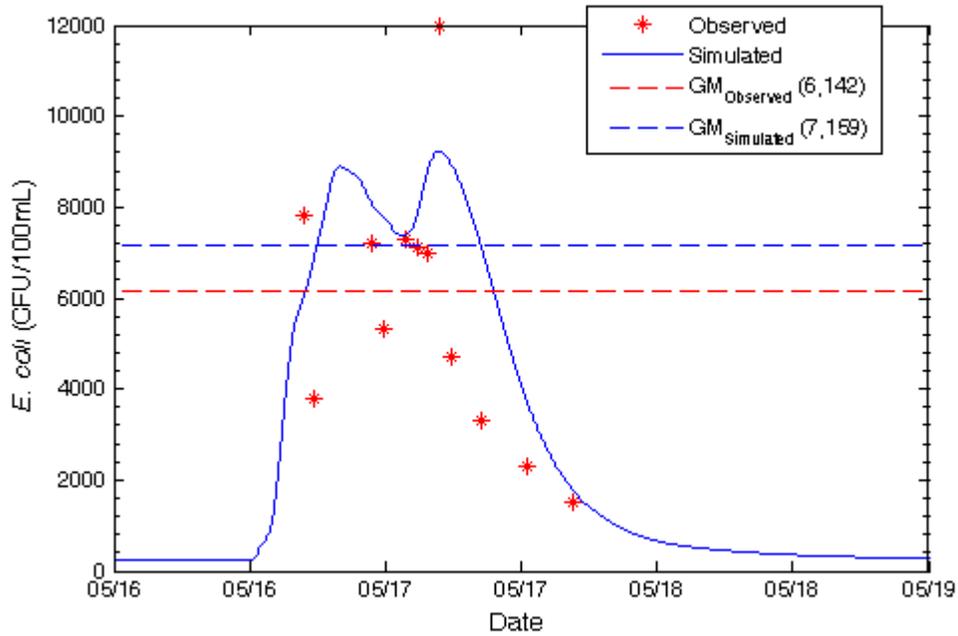
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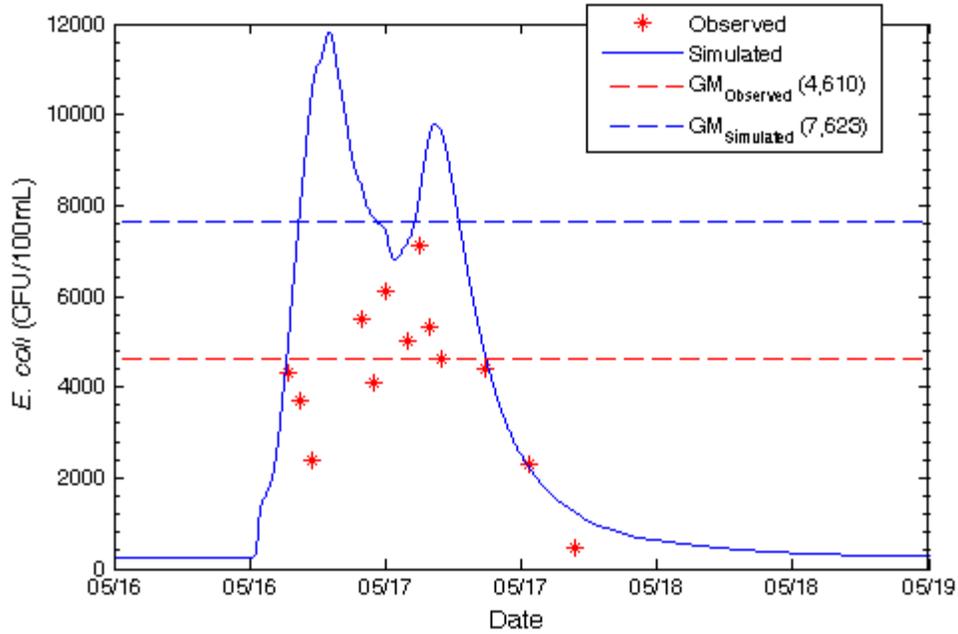
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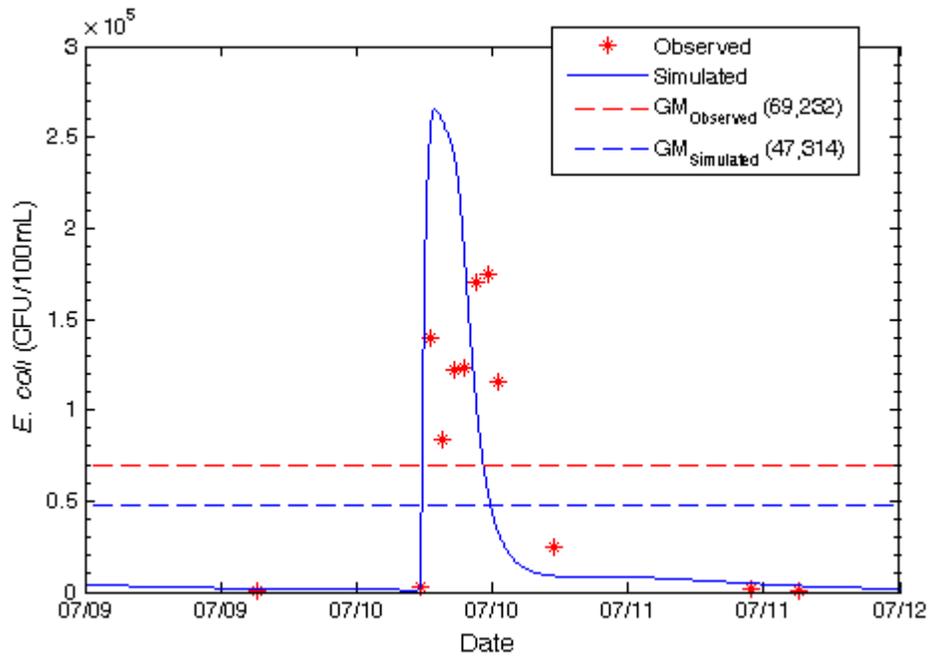
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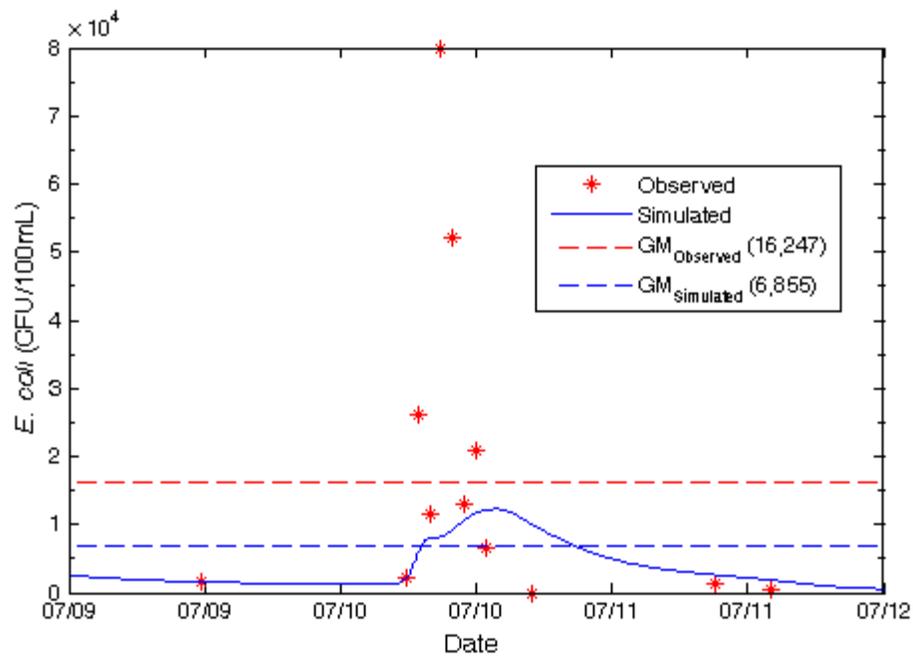
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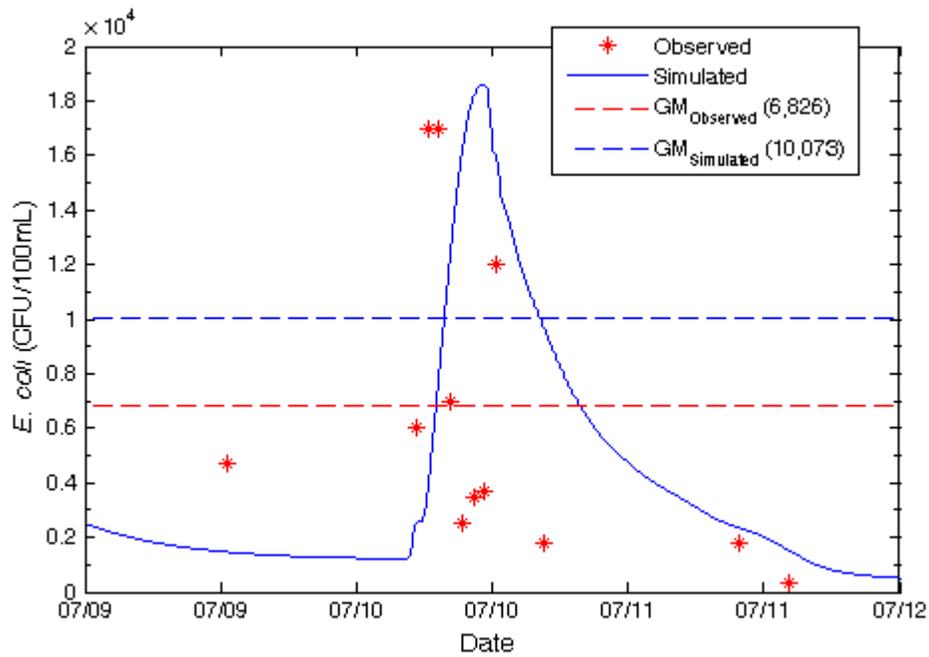
3.1.10. July 10, 2003 Storm - TF280



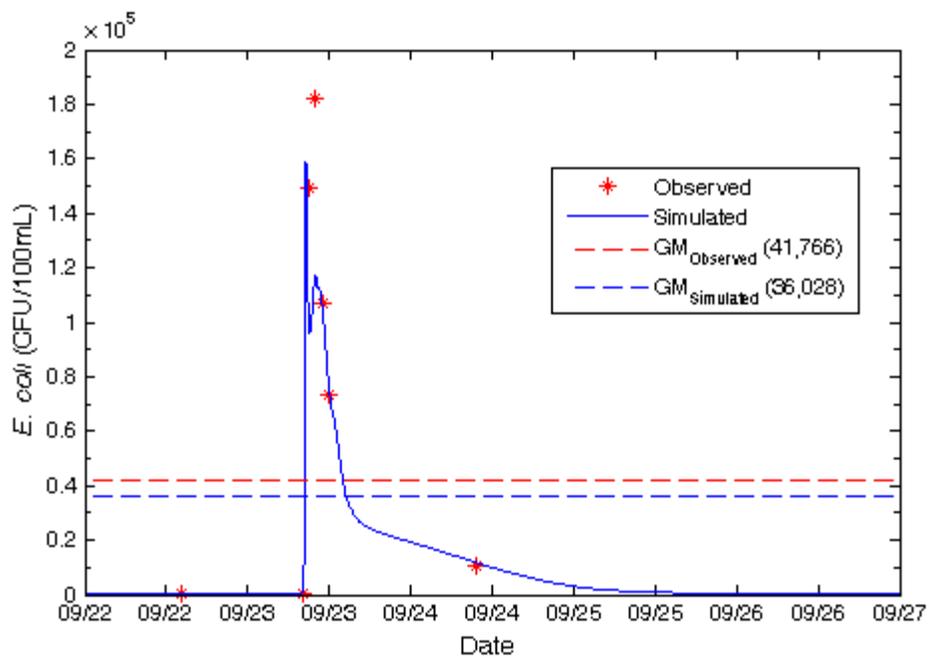
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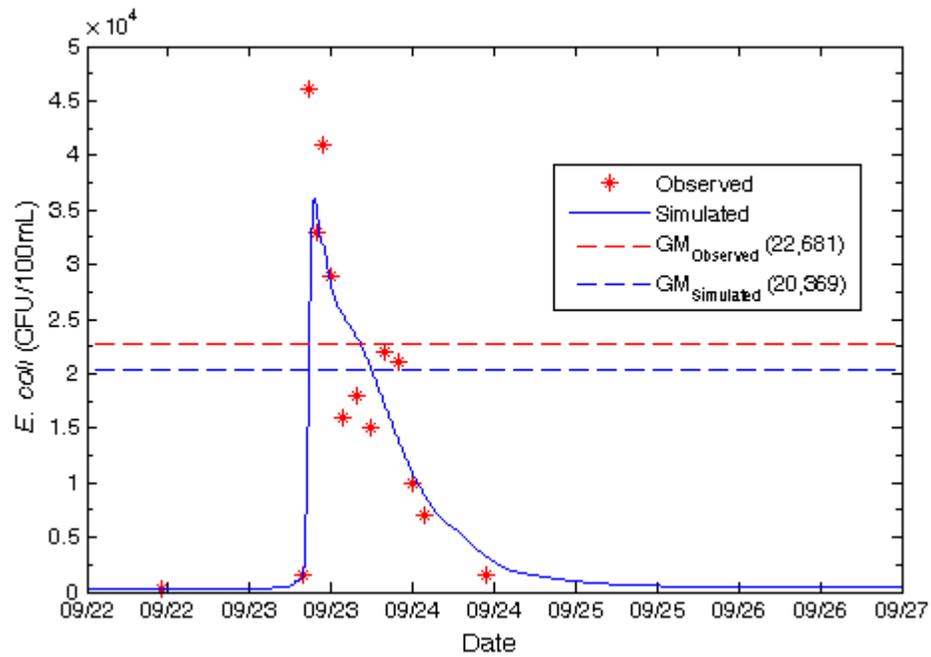
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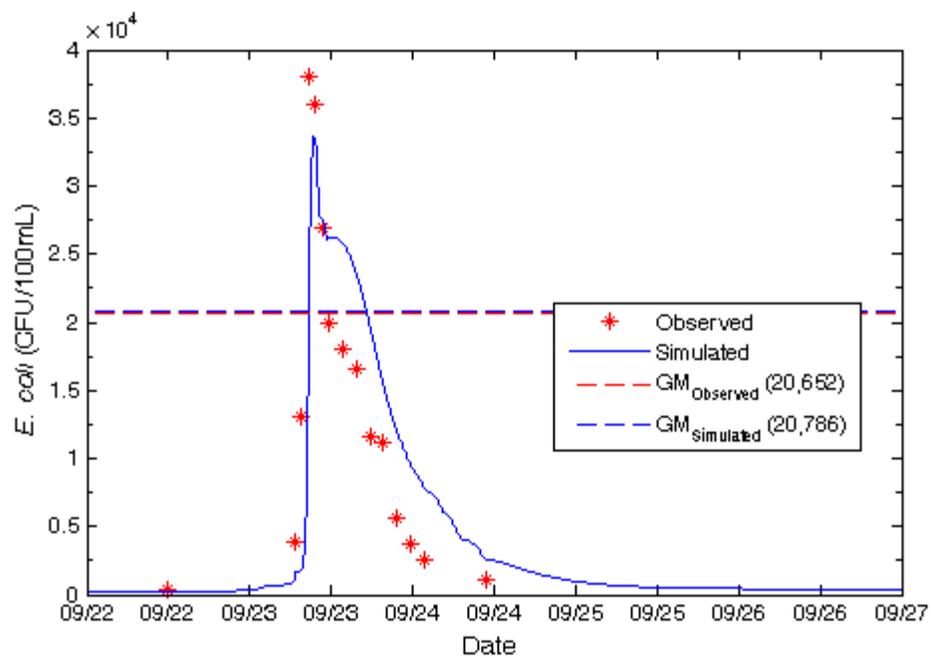
3.1.13. Sept 23, 2003 Storm - TF280



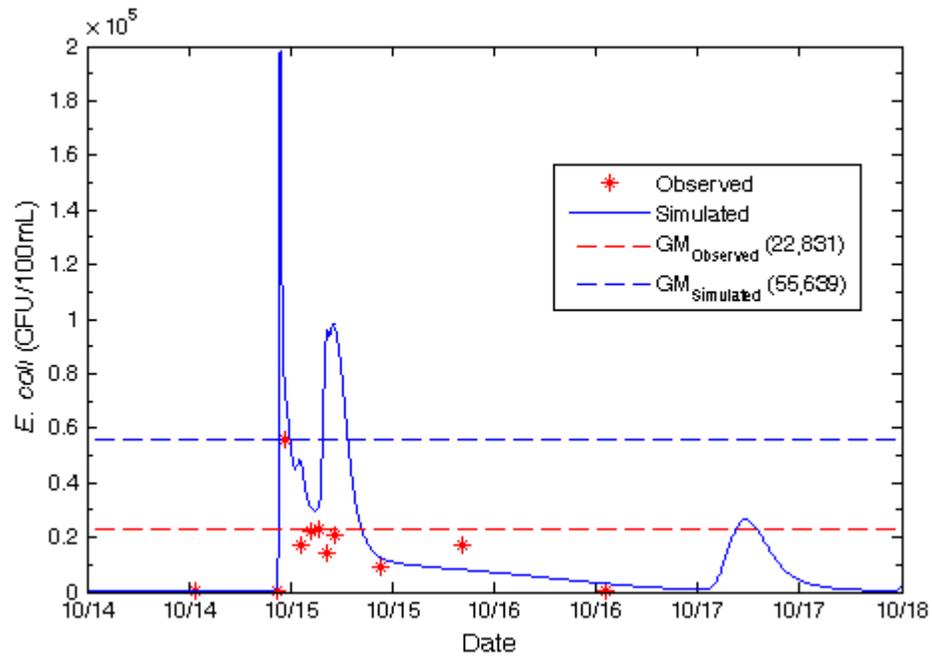
3.1.14. Sept 23, 2003 Storm - TF680



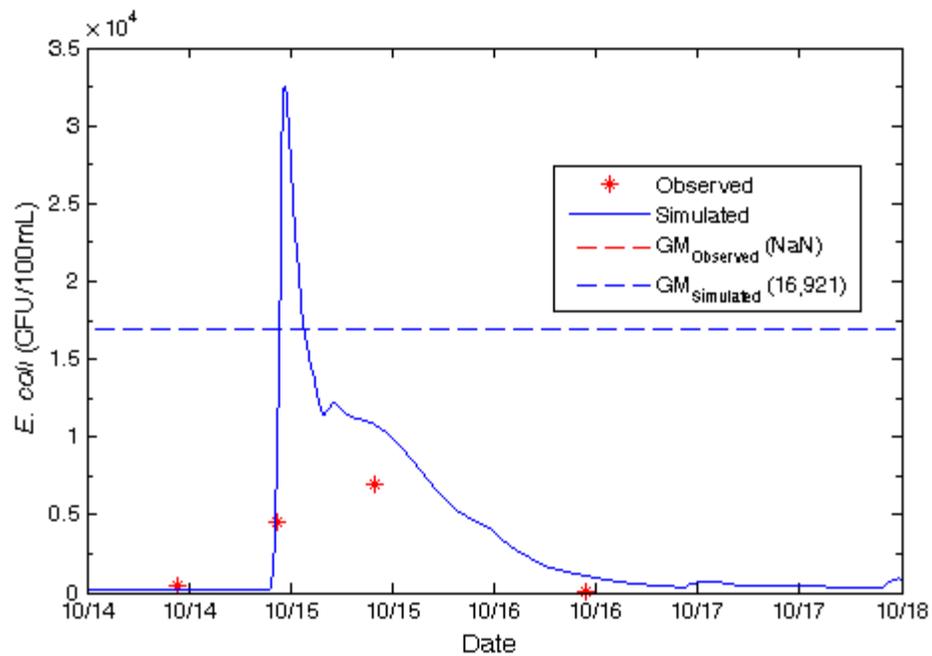
3.1.15. Sept 23, 2003 Storm - TF975



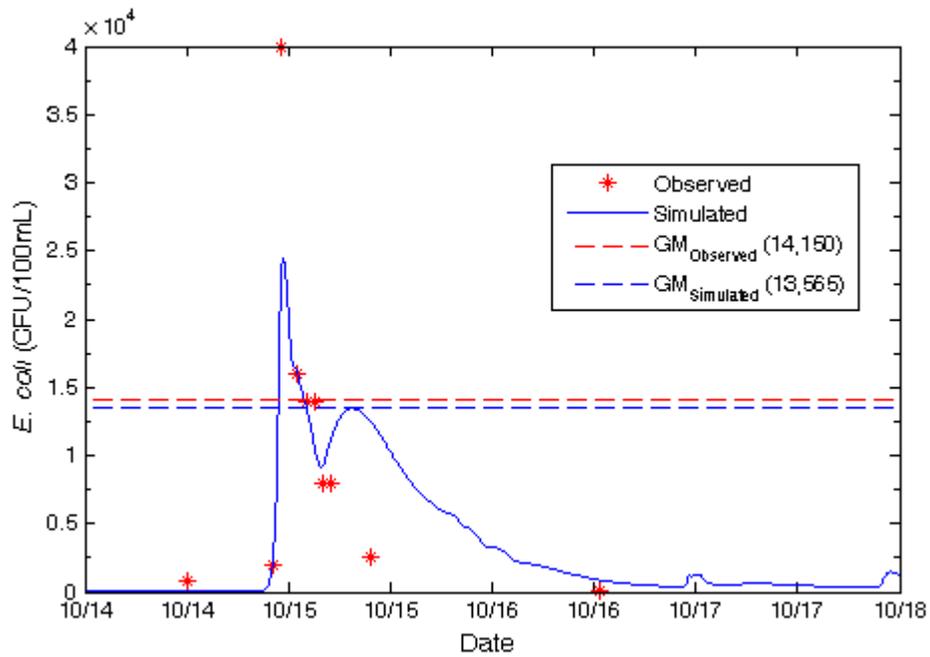
3.1.16. Oct 14, 2003 Storm - TF280



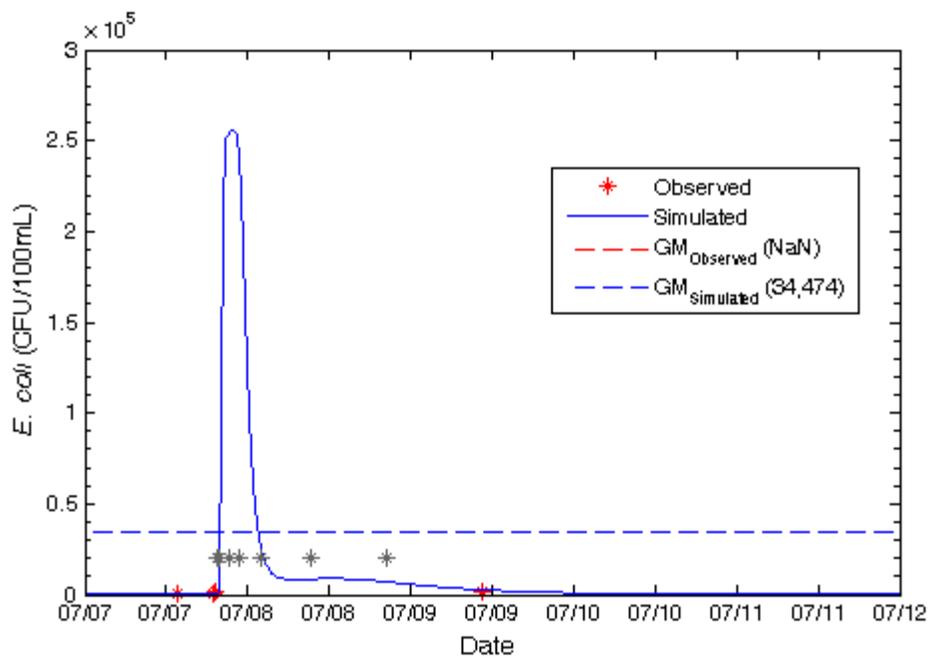
3.1.17. Oct 14, 2003 Storm - TF680



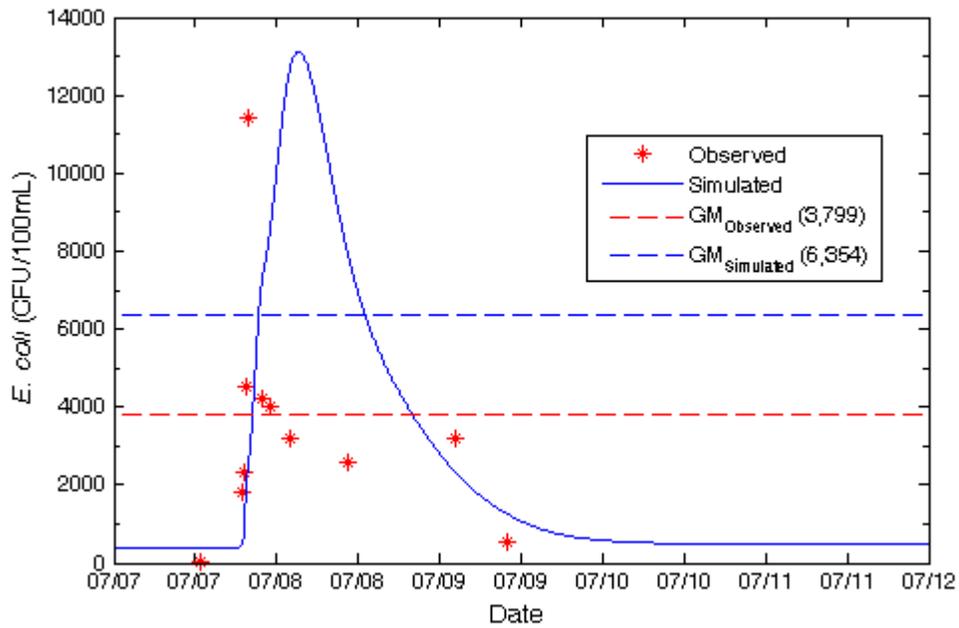
3.1.18. Oct 14, 2003 Storm - TF975



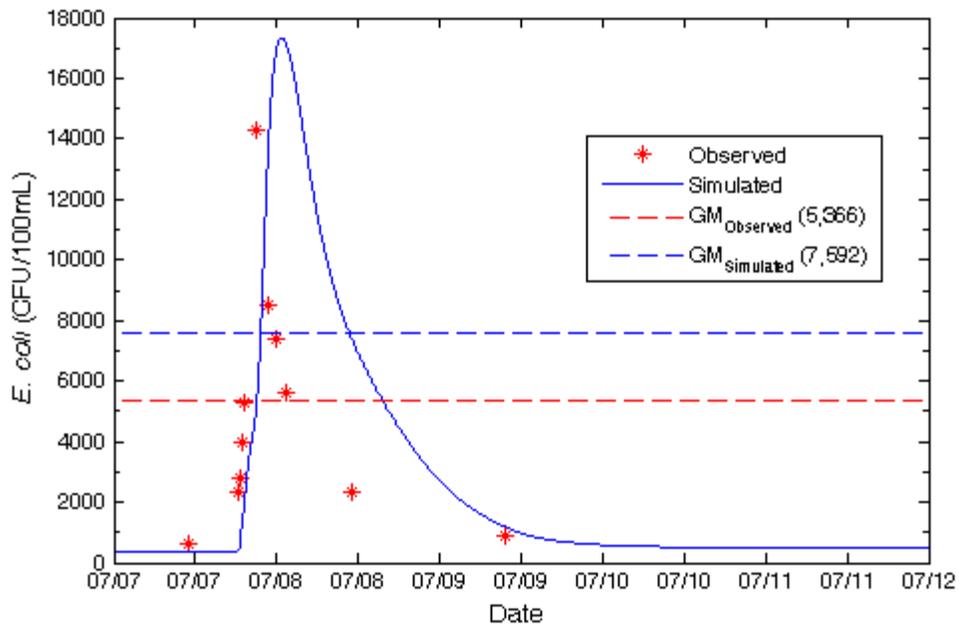
3.1.19. July 07, 2004 Storm - TF280



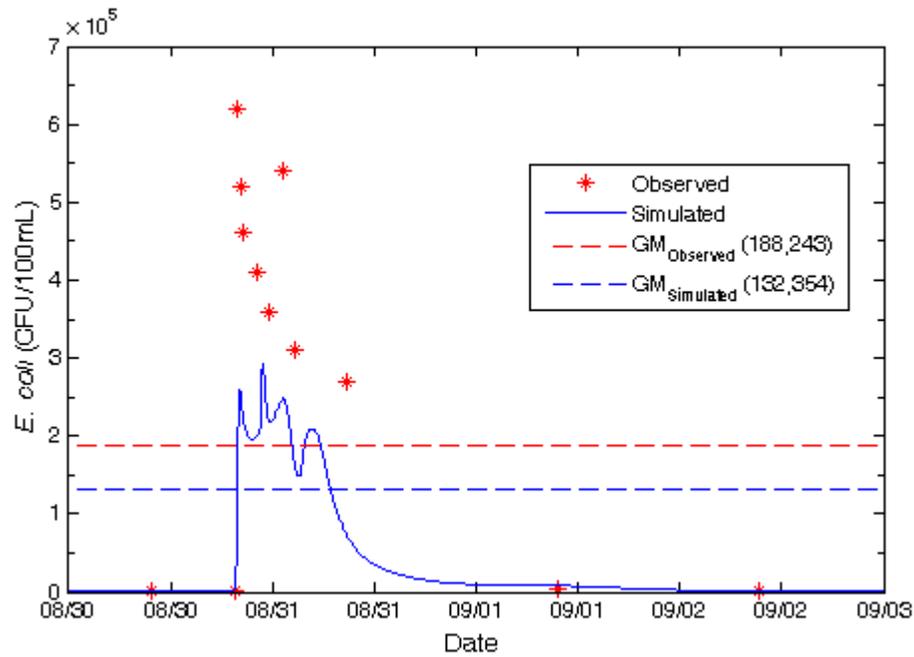
3.1.20. July 07, 2004 Storm - TF680



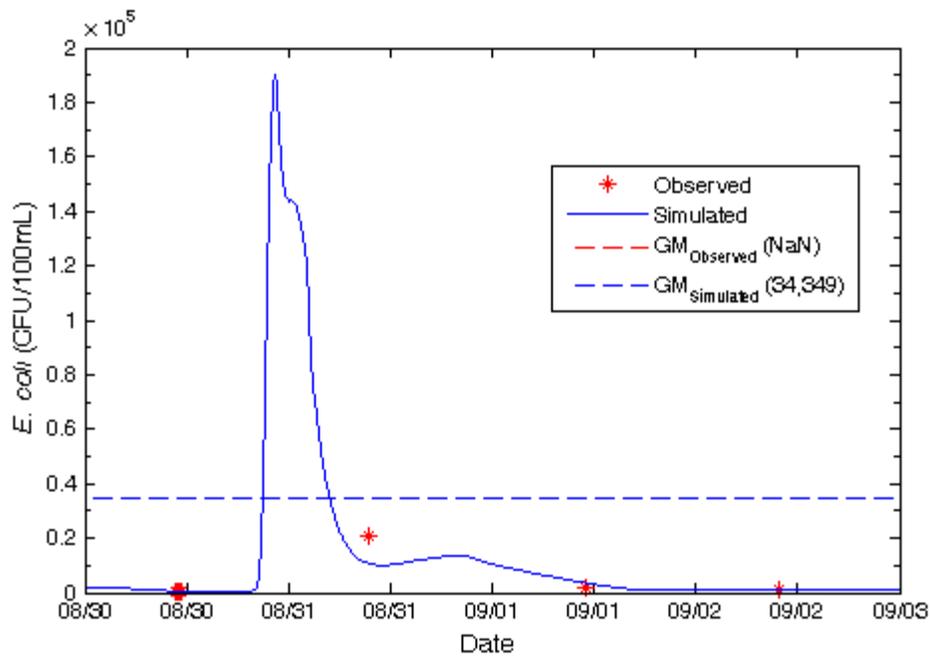
3.1.21. July 07, 2004 Storm - TF975



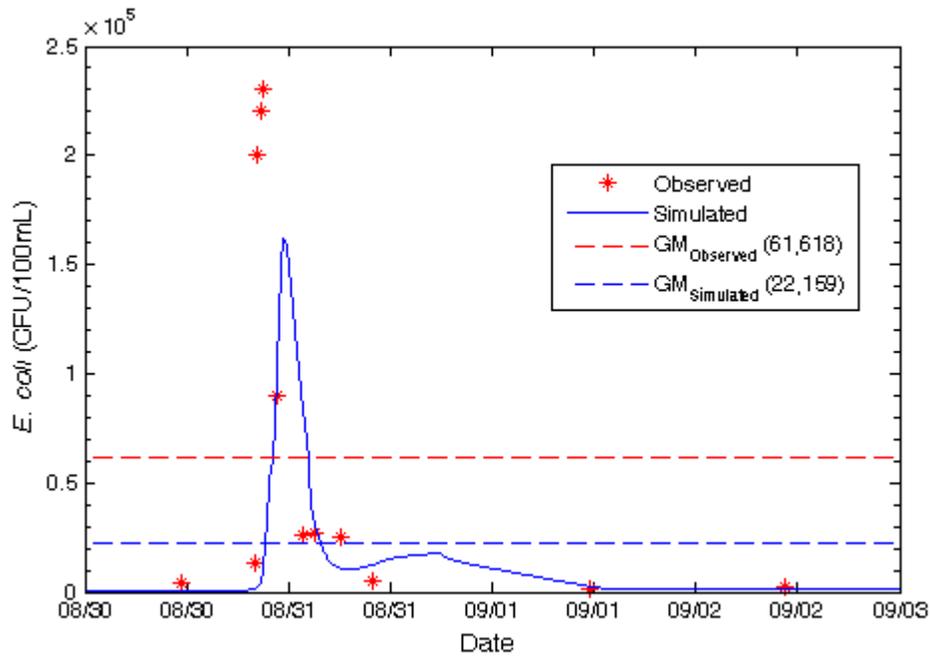
3.1.22. Aug 30, 2004 Storm - TF280



3.1.23. Aug 30, 2004 Storm - TF680

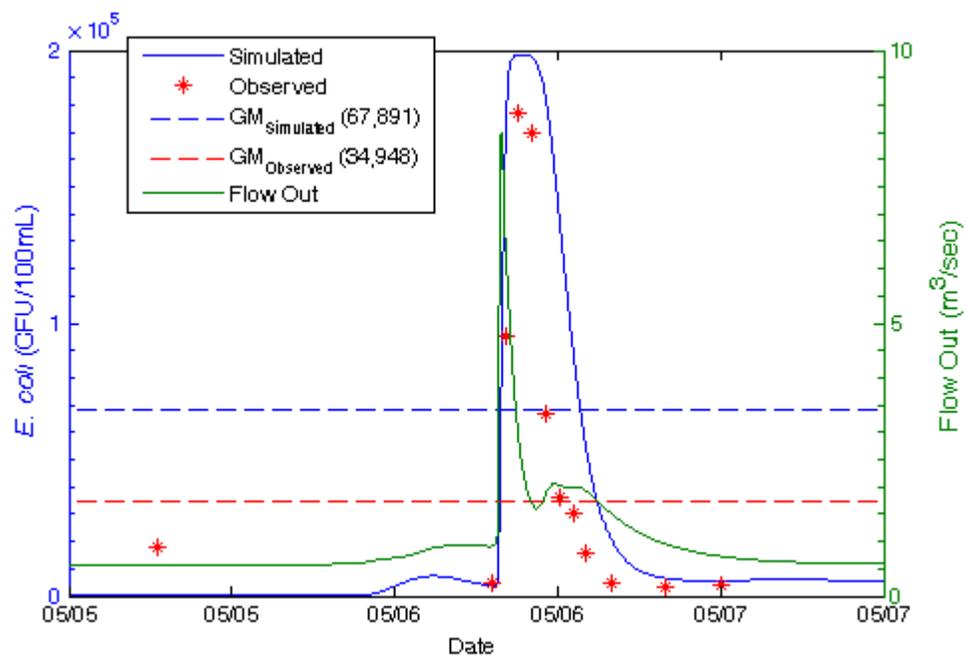


3.1.24. Aug 30, 2004 Storm - TF975

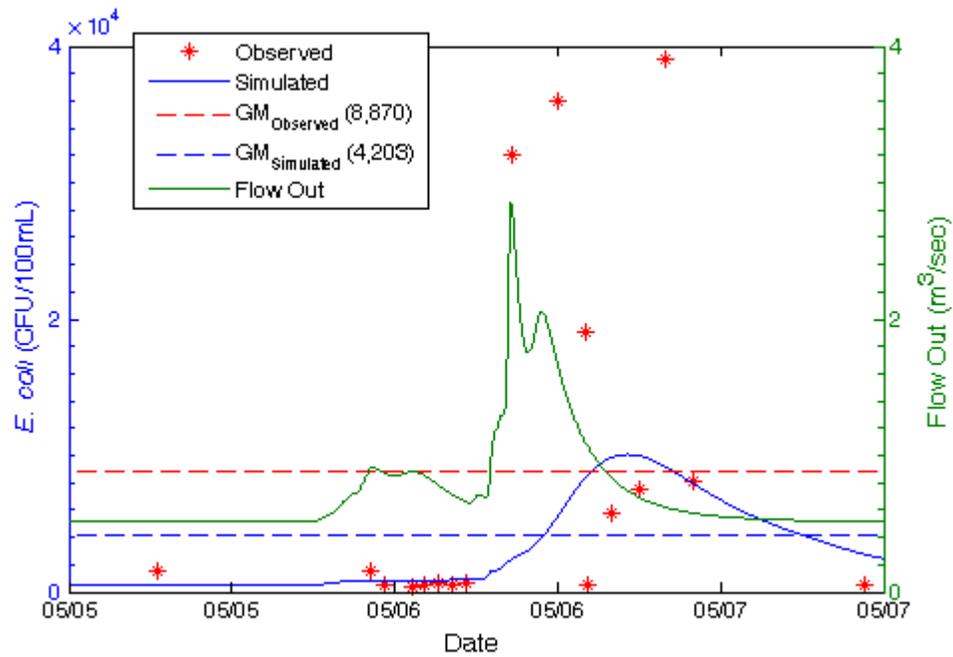


3.2. *E. coli* and Flow Out Time Series

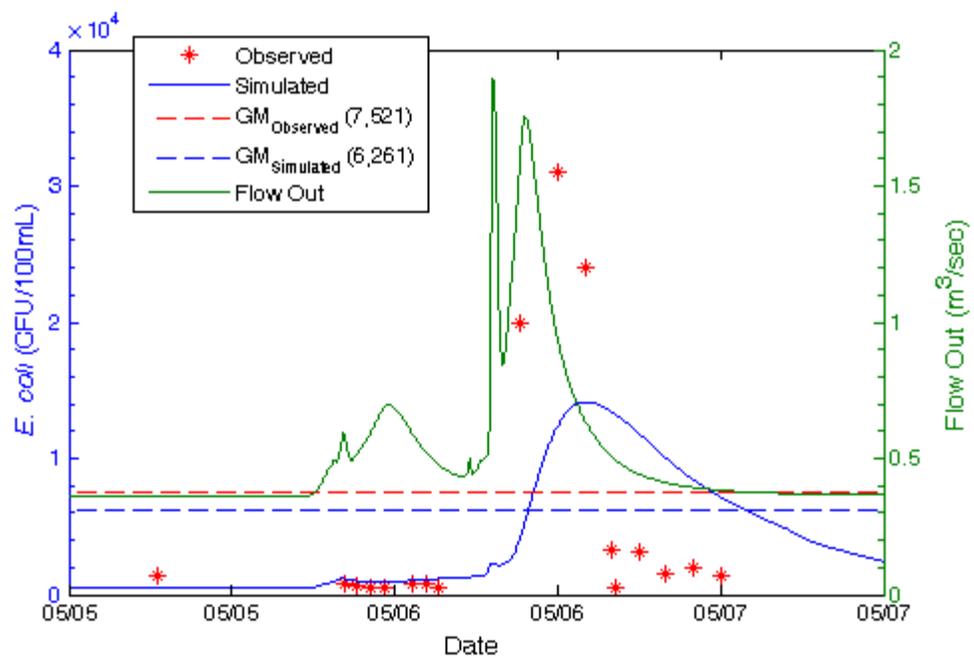
3.2.1. May 06, 2003 Storm - TF280



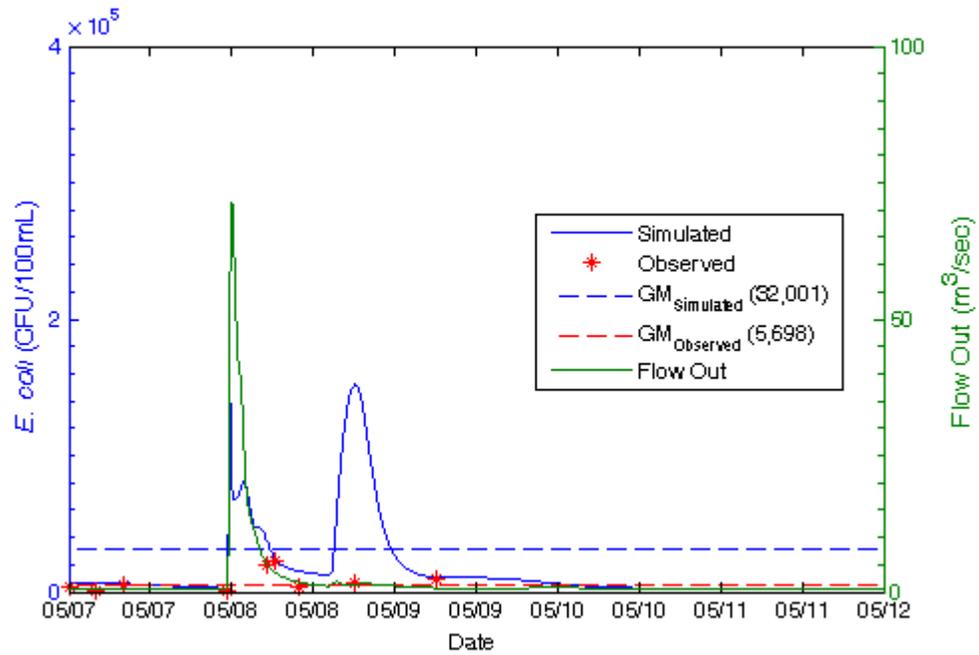
3.2.2. May 06, 2003 Storm - TF680



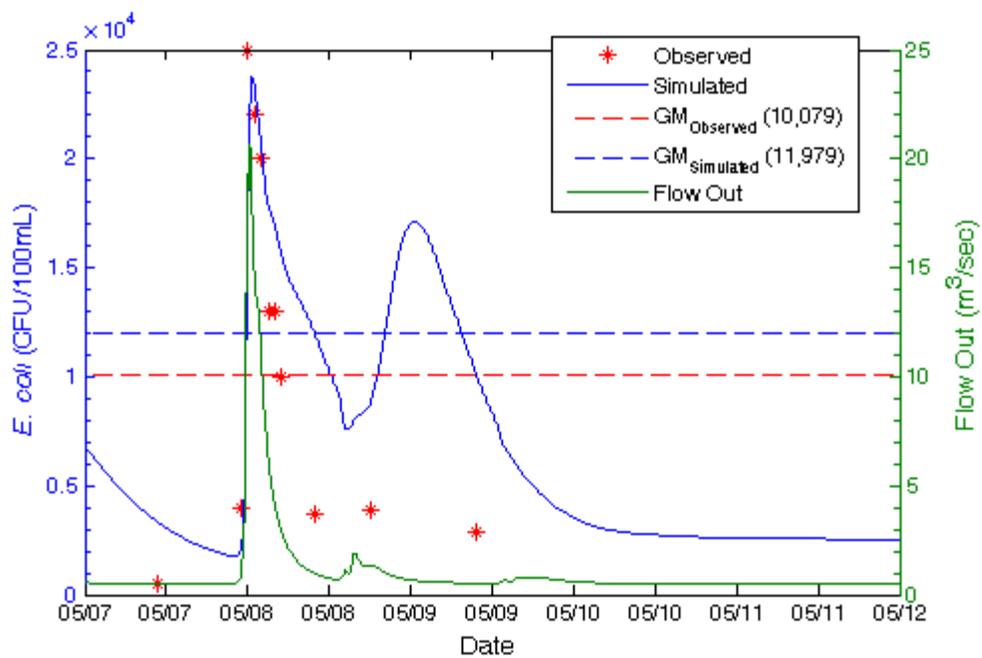
3.2.3. May 06, 2003 Storm - TF975



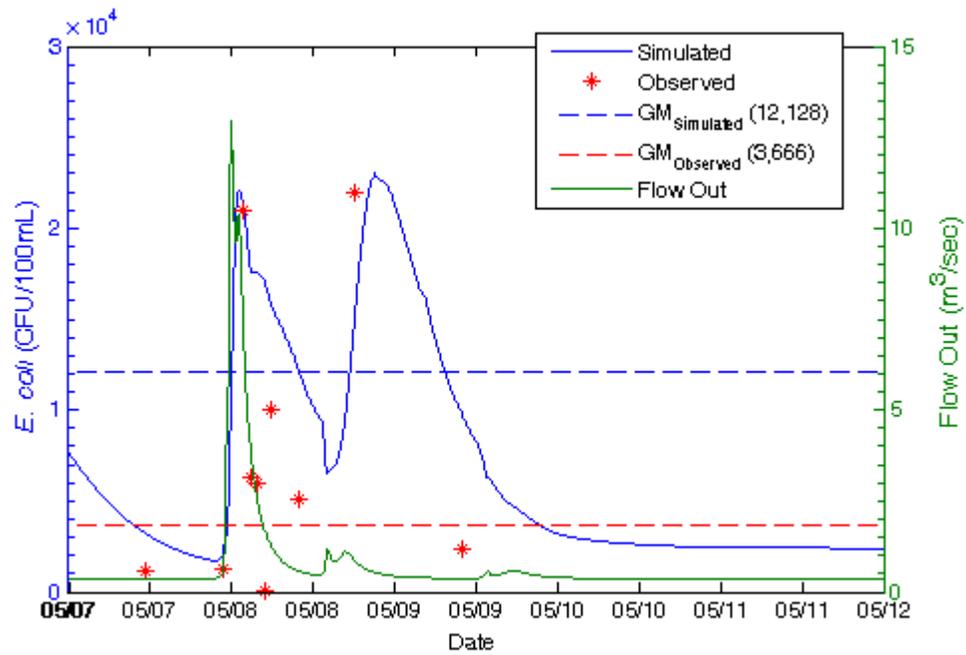
3.2.4. May 07, 2003 Storm - TF280



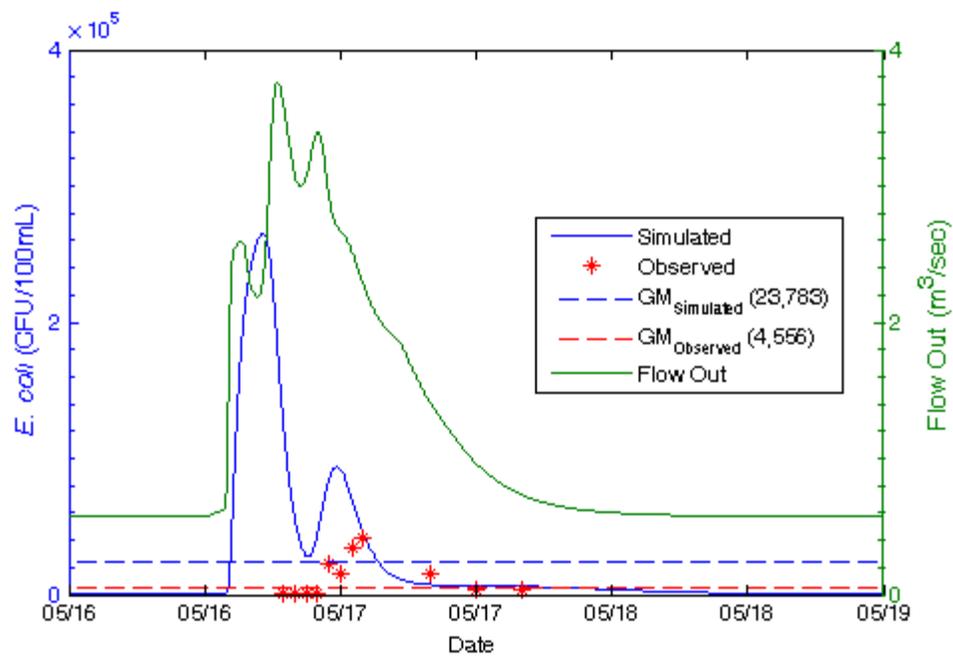
3.2.5. May 07, 2003 Storm - TF680



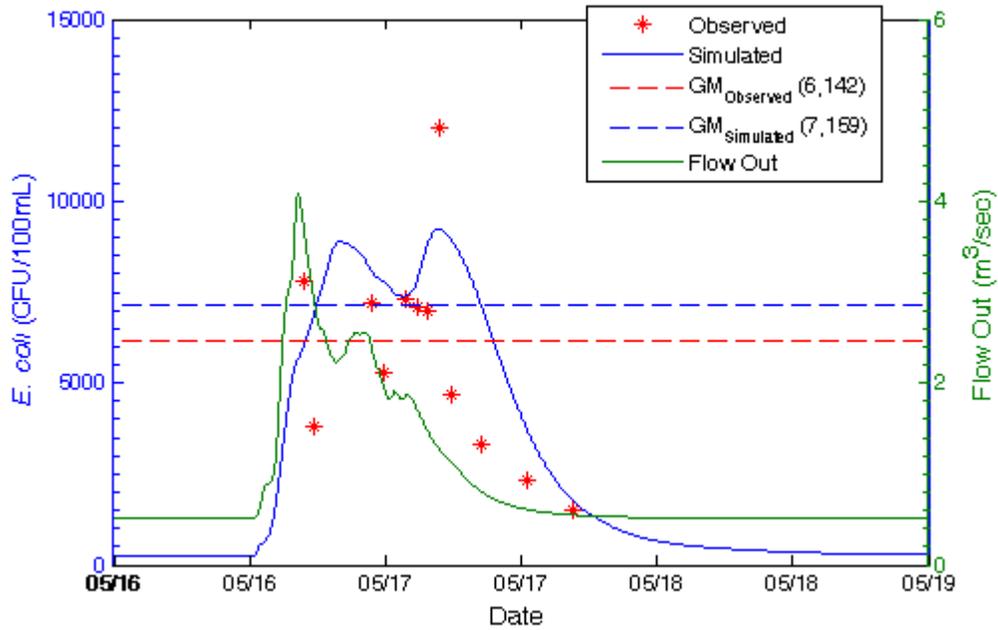
3.2.6. May 07, 2003 Storm - TF975



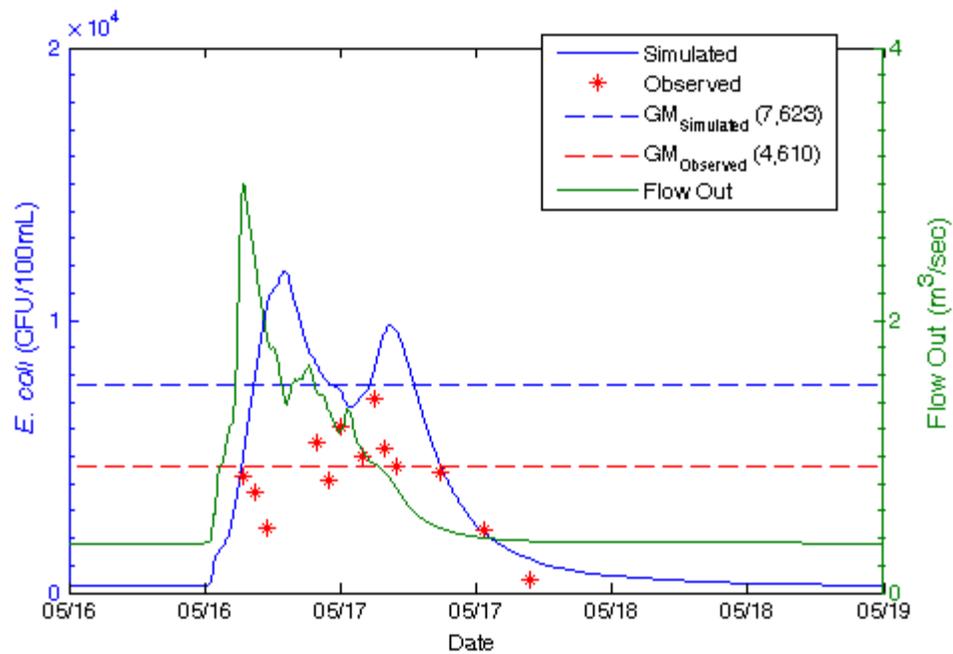
3.2.7. May 16, 2003 Storm - TF280



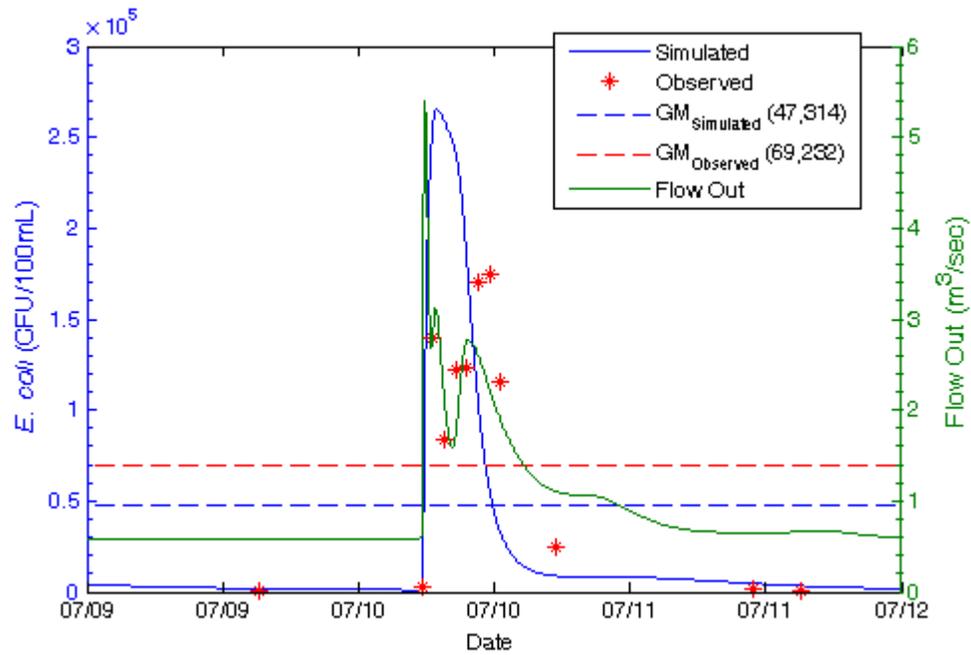
3.2.8. May 16, 2003 Storm - TF680



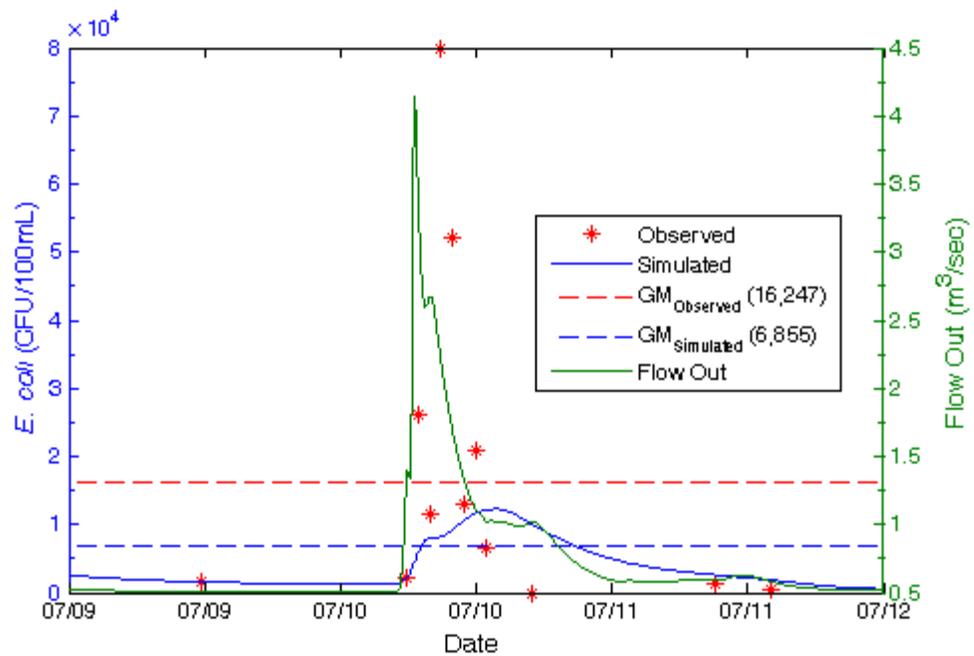
3.2.9. May 16, 2003 Storm - TF975



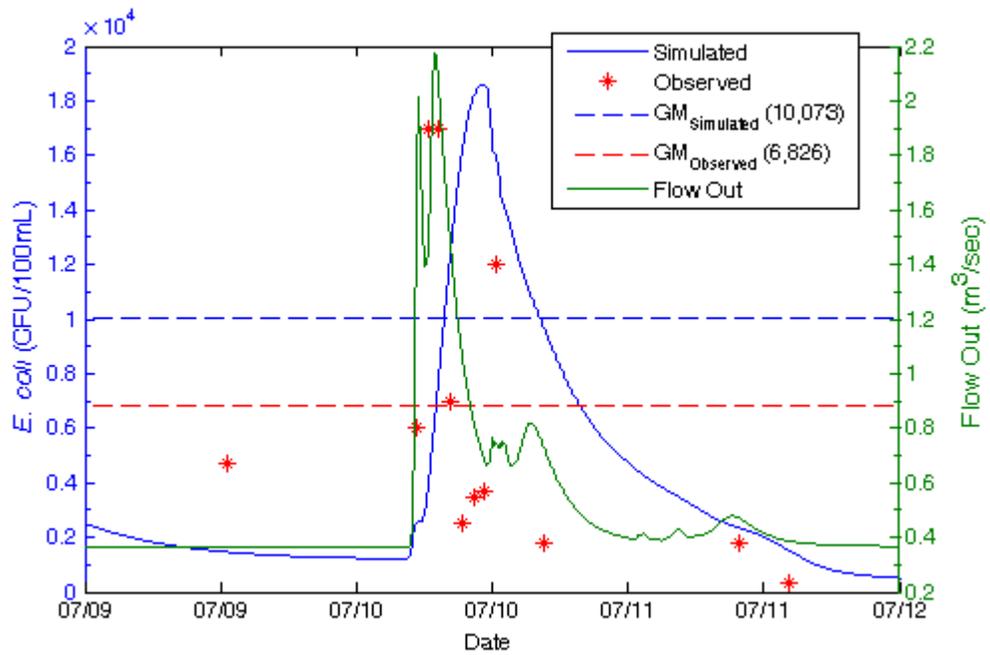
3.2.10. July 10, 2003 Storm - TF280



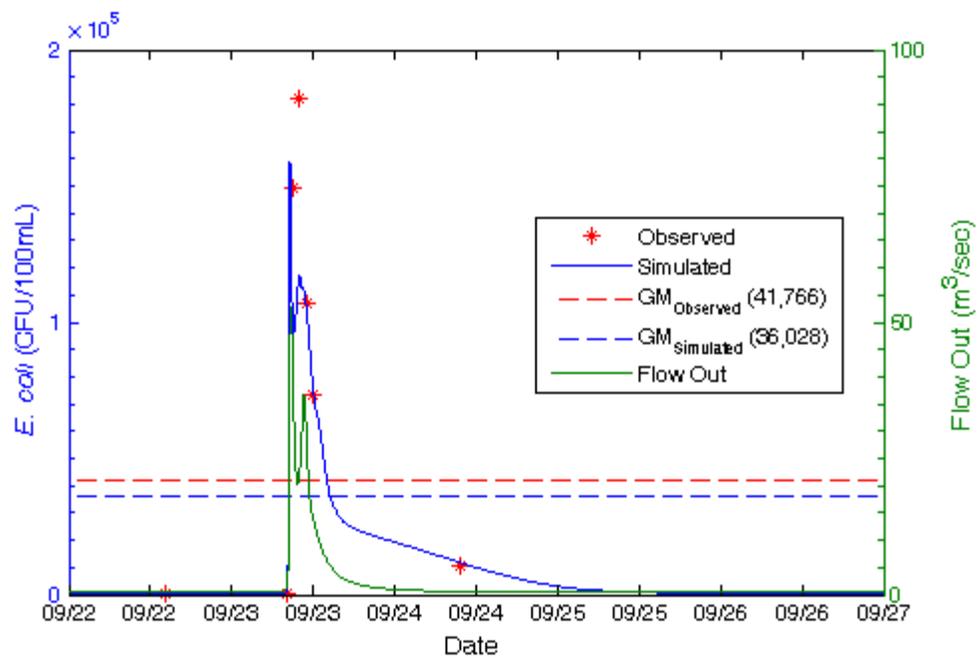
3.2.11. July 10, 2003 Storm - TF680



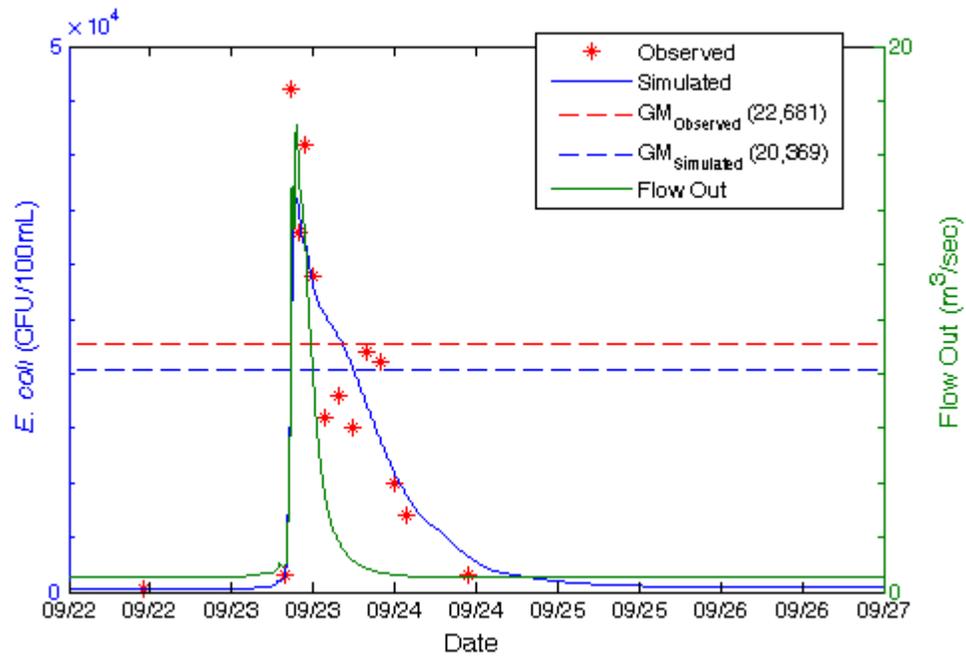
3.2.12. July 10, 2003 Storm - TF975



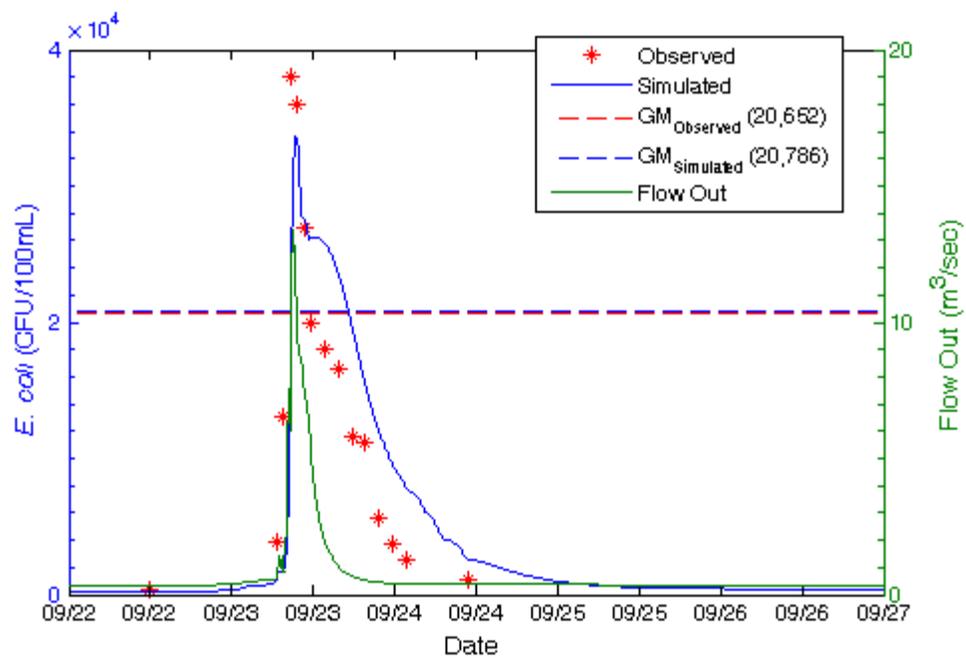
3.2.13. Sept 23, 2003 Storm - TF280



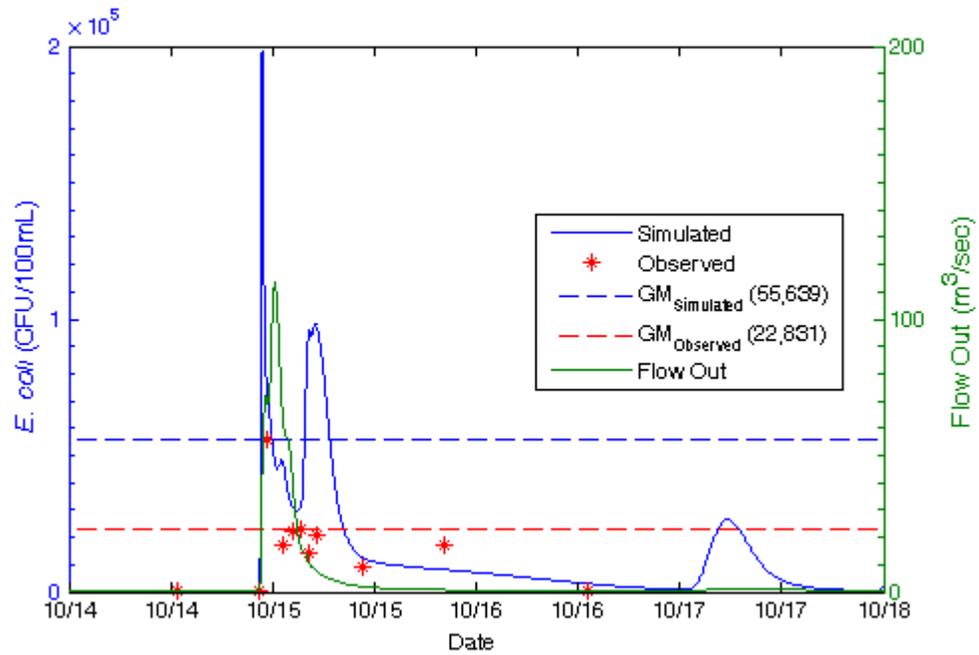
3.2.14. Sept 23, 2003 Storm - TF680



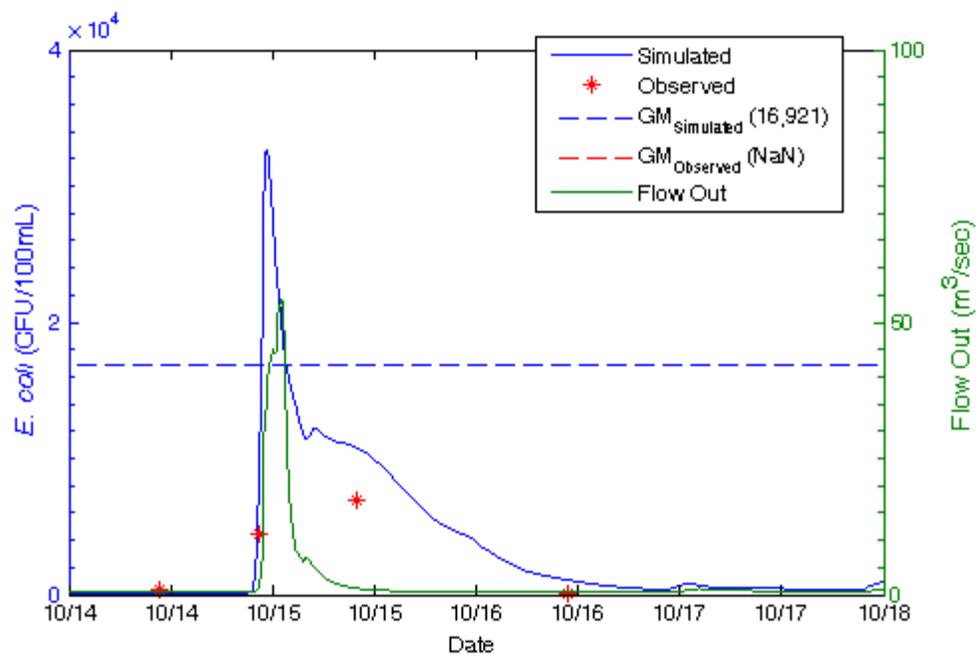
3.2.15. Sept 23, 2003 Storm - TF975



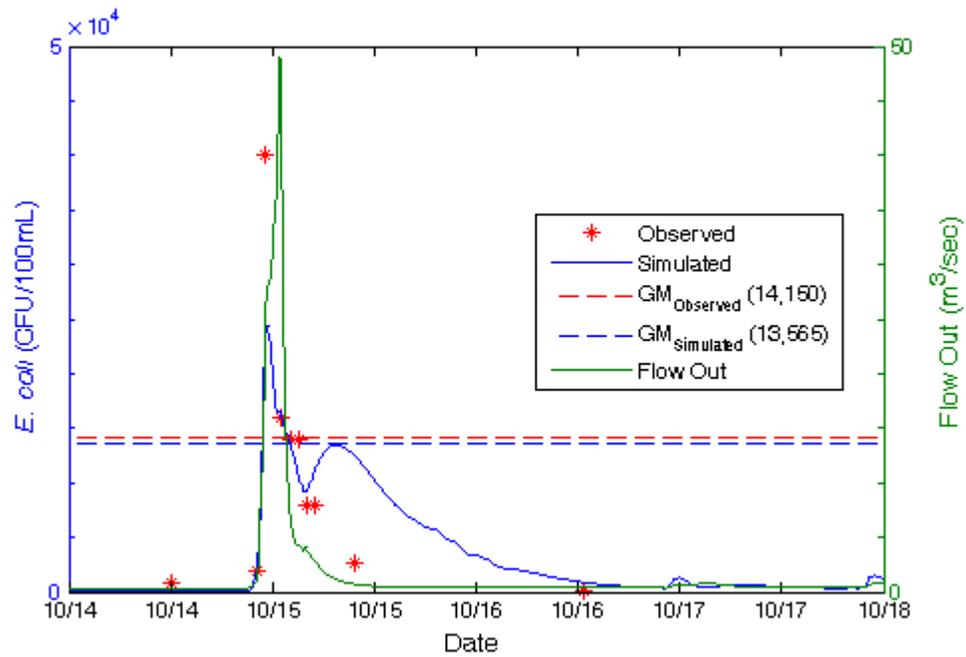
3.2.16. Oct 14, 2003 Storm - TF280



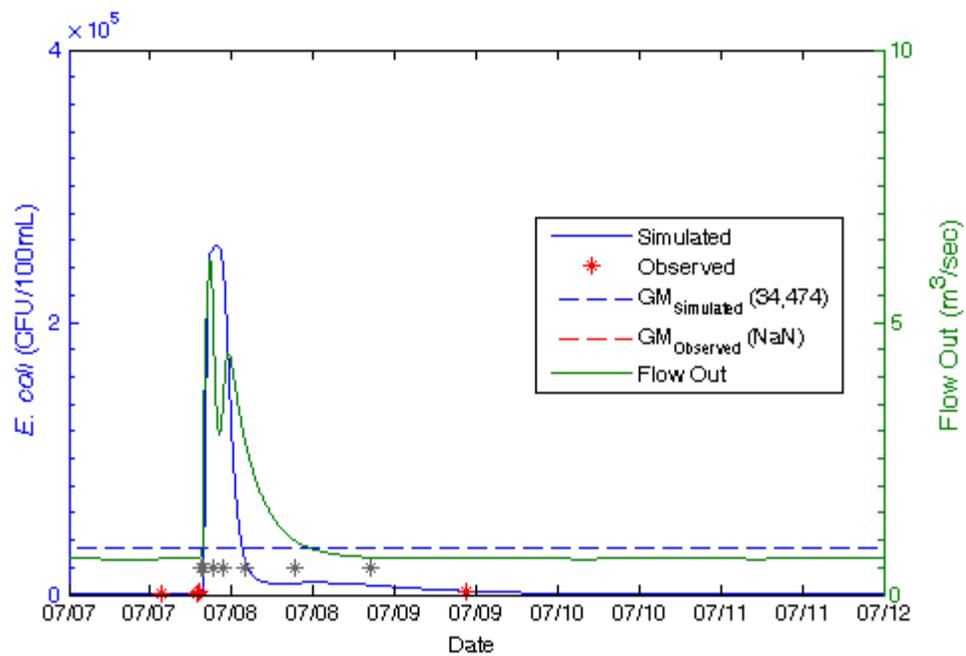
3.2.17. Oct 14, 2003 Storm - TF680



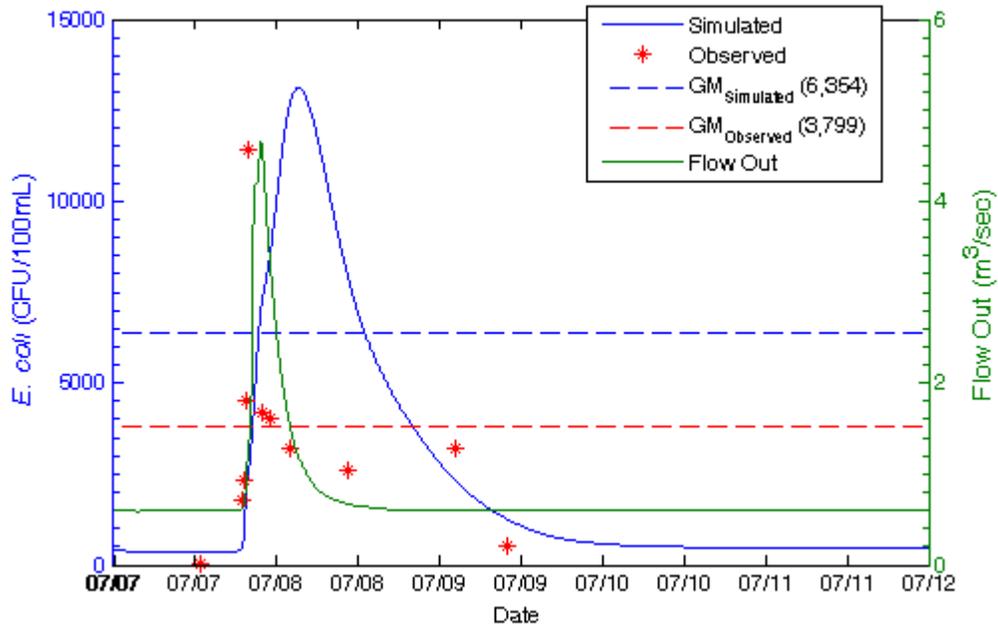
3.2.18. Oct 14, 2003 Storm - TF975



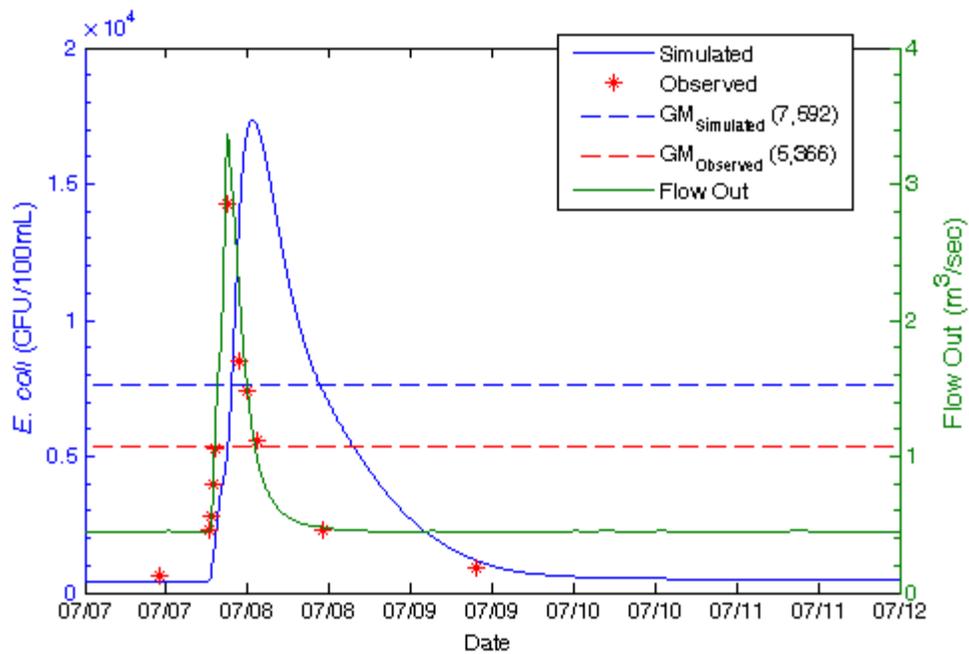
3.2.19. July 07, 2004 Storm - TF280



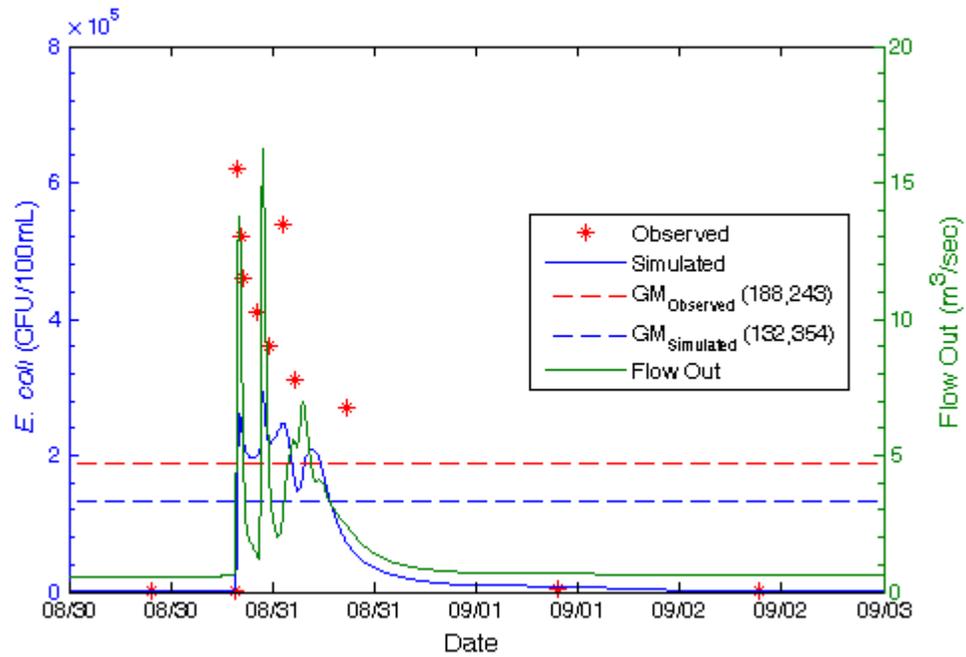
3.2.20. July 07, 2004 Storm - TF680



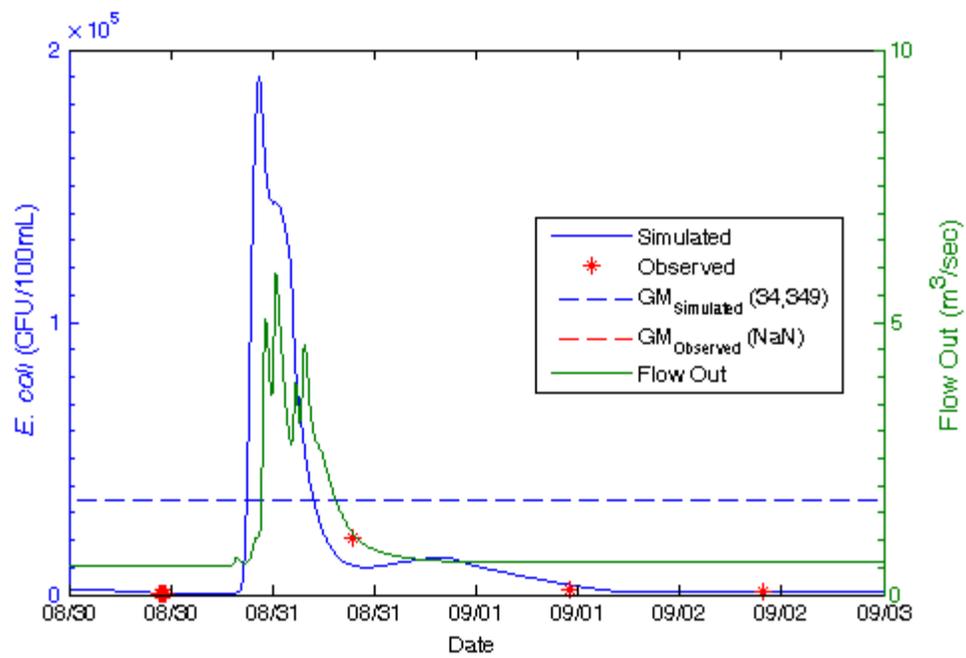
3.2.21. July 07, 2004 Storm - TF975



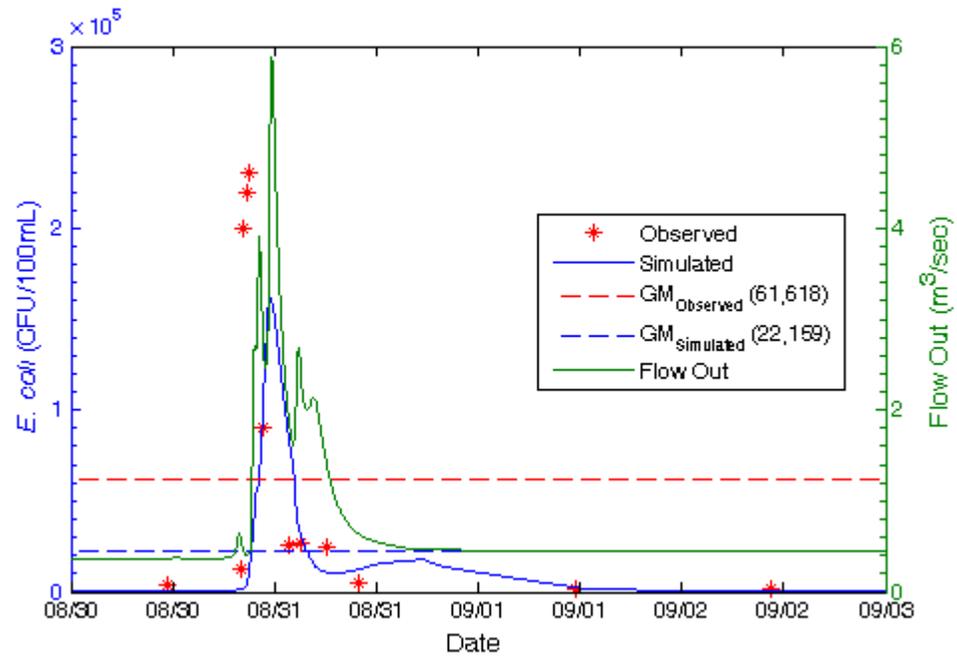
3.2.22. Aug 30, 2004 Storm - TF280



3.2.23. Aug 30, 2004 Storm - TF680

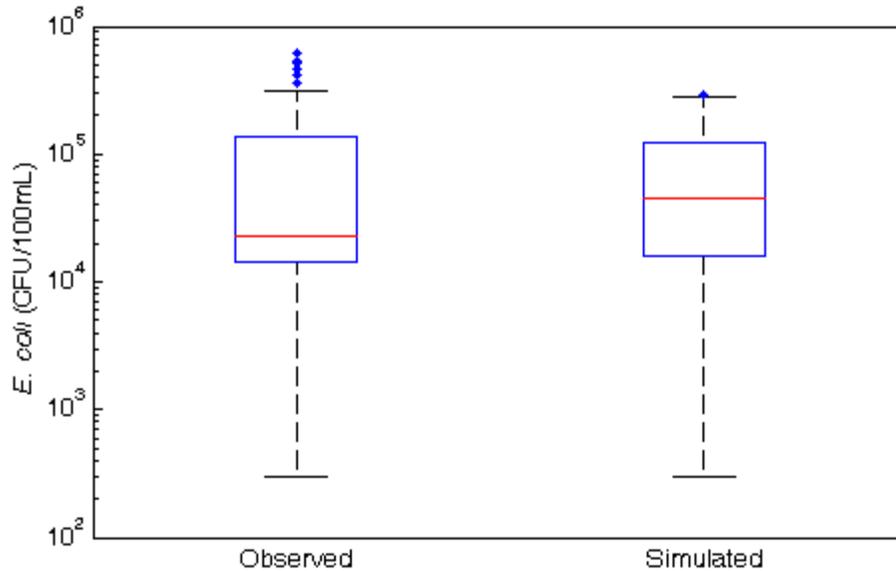


3.2.24. Aug 30, 2004 Storm - TF975

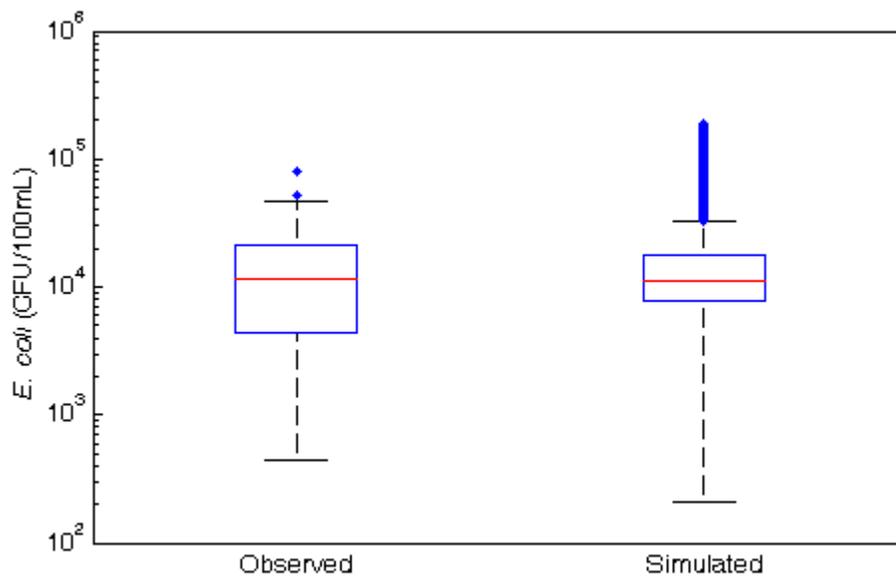


3.3. Predicted and Observed *E. coli* Concentration Box Plots

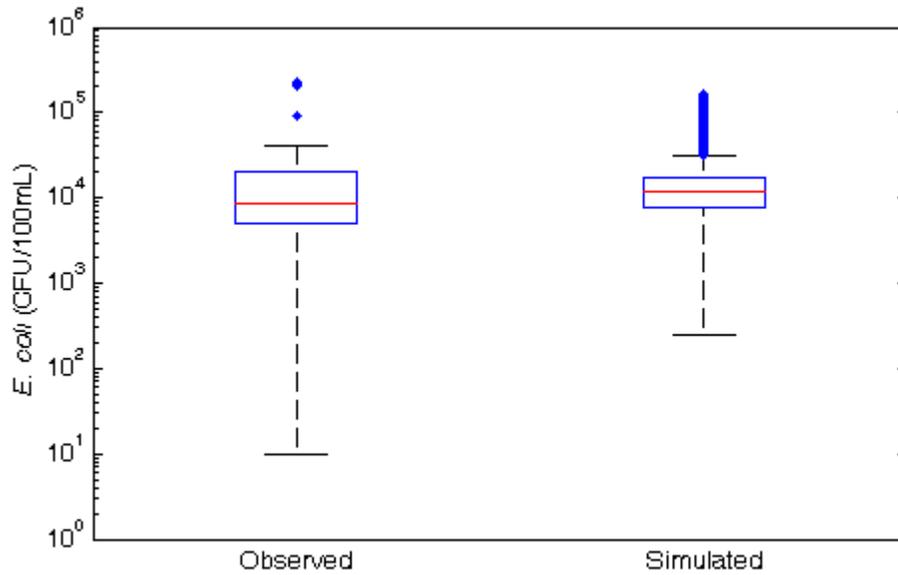
3.3.1. *E. coli* Concentration for 2003-2004 Storms - TF280



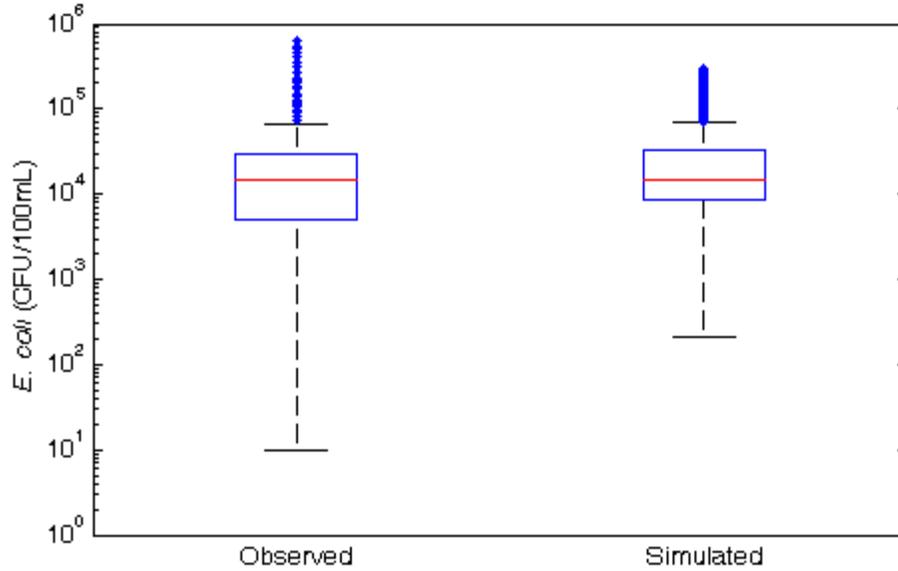
3.3.2. *E. coli* Concentration for 2003-2004 Storms - TF680



3.3.3. *E. coli* Concentration for 2003-2004 Storms - TF975



3.3.4. *E. coli* Concentration for 2003-2004 Storms - All Sites



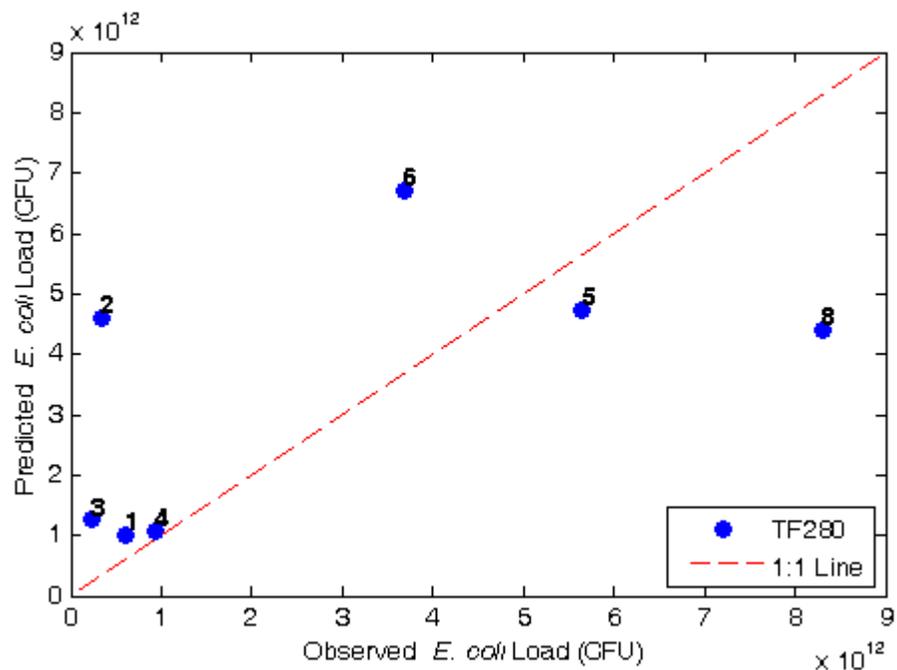
3.4. Predicted vs. Observed *E. coli* Load Scatter Plots

Table 3-1. Total Rainfall and Peak Flow for Eight Storms

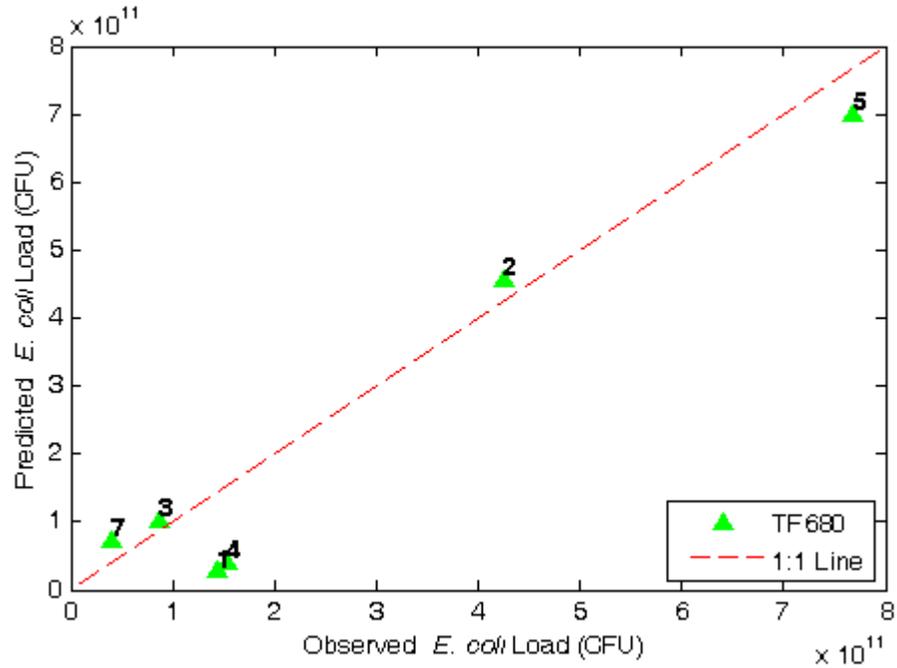
Event Number	Storm Date	Event Rainfall (in)	Peak Flow* (cfs)
1	May 06, 2003	0.1600	637
2	May 07, 2003	0.7130	3280
3	May 16, 2003	0.3100	59
4	July 10, 2003	0.1900	179
5	Sept 23, 2003	0.7090	1710
6	Oct 14, 2003	1.2780	3460
7	July 07, 2004	0.2000	198
8	Aug 30, 2004	0.4250	866

*Peak flow observed at Castor Ave. (Gage 1467087)

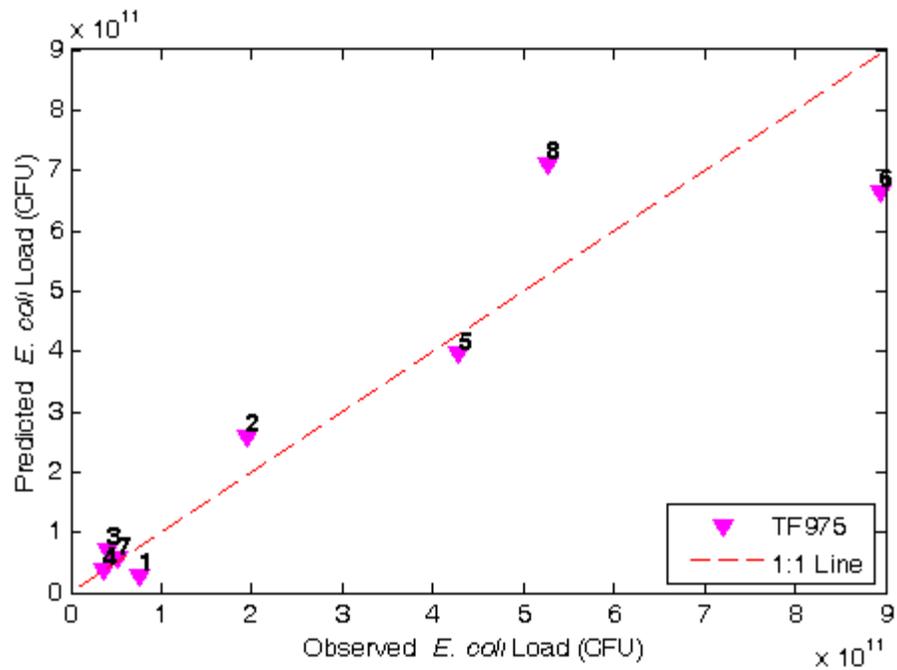
3.4.1. *E. coli* Load for 2003-2004 Storms - TF280

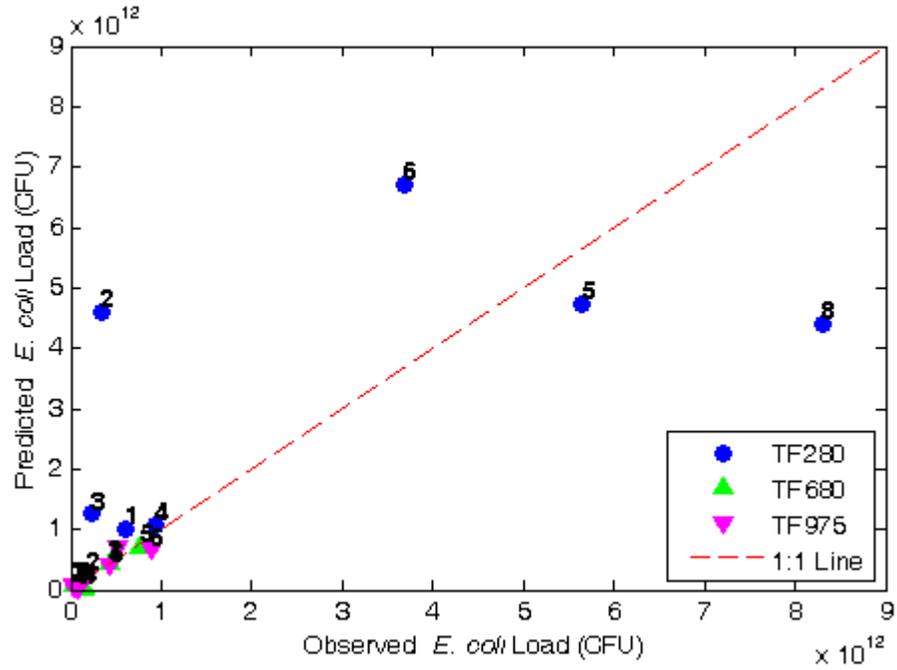


3.4.2. *E. coli* Load for 2003-2004 Storms - TF680



3.4.3. *E. coli* Load for 2003-2004 Storms - TF975



3.4.4. *E. coli* Load for 2003-2004 Storms - All Sites

3.5. Statistical Analysis - Observed Data

Table 3-2. *E. coli* - Observed Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10 th Percentiles	90 th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
05/06/2003	TF280	34948	36000	67220	4820	174200
05/06/2003	TF680	8870	19000	15642	440	36000
05/06/2003	TF975	7521	20000	13291	490	31000
05/07/2003	TF280	5698	6700	9863	570	23000
05/07/2003	TF680	10079	13000	8160	3780	23800
05/07/2003	TF975	3666	6300	8351	1028	21800
05/16/2003	TF280	4556	4000	14703	522	37200
05/16/2003	TF680	6142	7050	2491	3550	9900
05/16/2003	TF975	4610	4600	1251	3180	6500
07/10/2003	TF280	69232	122000	59618	11440	173000
07/10/2003	TF680	16247	17000	26567	3490	71600
07/10/2003	TF975	6826	6500	5967	2800	17000
09/23/2003	TF280	41766	107000	70279	600	182000
09/23/2003	TF680	22681	21500	11818	12500	43500
09/23/2003	TF975	20652	19000	10102	12020	37400
10/14/2003	TF280	22831	21500	15320	14300	52700
10/14/2003	TF680	NaN	NaN	NaN	NaN	NaN
10/14/2003	TF975	14150	14000	11911	8000	37600
07/07/2004	TF280	NaN	NaN	NaN	NaN	NaN
07/07/2004	TF680	3799	4000	3209	1900	10020
07/07/2004	TF975	5366	5450	3874	2450	12560
08/30/2004	TF280	188243	410000	184093	108180	588000
08/30/2004	TF680	NaN	NaN	NaN	NaN	NaN
08/30/2004	TF975	61618	58500	96569	16600	227000

3.6. Statistical Analysis - Simulated Data

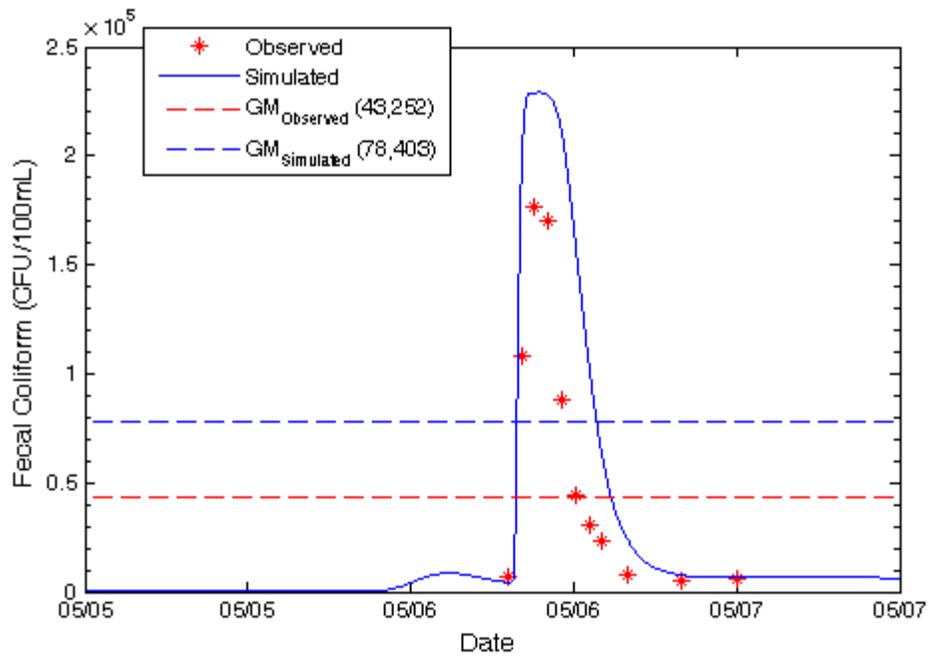
Table 3-3. *E. coli* - Simulated Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10 th Percentiles	90 th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
05/06/2003	TF280	67891	100519	73654	15362	198202
05/06/2003	TF680	4203	4831	3164	1494	9783
05/06/2003	TF975	6261	9996	5164	1378	14061
05/07/2003	TF280	32001	28297	40279	12867	112286
05/07/2003	TF680	11979	12405	4806	7870	20386
05/07/2003	TF975	12128	12944	4560	6932	19122
05/16/2003	TF280	23783	18413	72931	6487	185570
05/16/2003	TF680	7159	7881	1709	5300	8948
05/16/2003	TF975	7623	7932	2167	5026	10980
07/10/2003	TF280	47314	43964	98394	9255	255578
07/10/2003	TF680	6855	8230	3260	1996	11750
07/10/2003	TF975	10073	14411	5812	2591	18461
09/23/2003	TF280	36028	27226	35084	20325	108739
09/23/2003	TF680	20369	22145	7193	12433	31942
09/23/2003	TF975	20786	25552	7211	13165	29904
10/14/2003	TF280	55639	48225	34044	30589	97280
10/14/2003	TF680	16921	15183	6914	11709	30323
10/14/2003	TF975	13565	13079	4593	9425	22234
07/07/2004	TF280	34474	23669	97089	8404	253108
07/07/2004	TF680	6354	10083	4368	945	13028
07/07/2004	TF975	7592	14288	6221	1476	17214
08/30/2004	TF280	132354	196391	69406	63757	243473
08/30/2004	TF680	34349	47437	61570	3447	155697
08/30/2004	TF975	22159	22997	50871	1725	138842

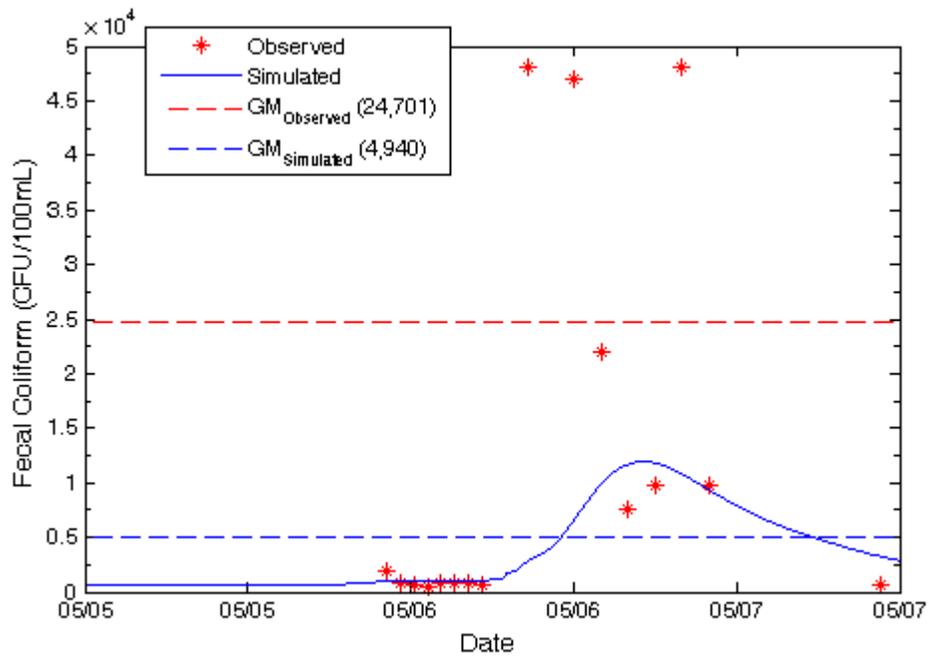
Chapter 4. Fecal Coliform

4.1. Fecal Coliform Time Series

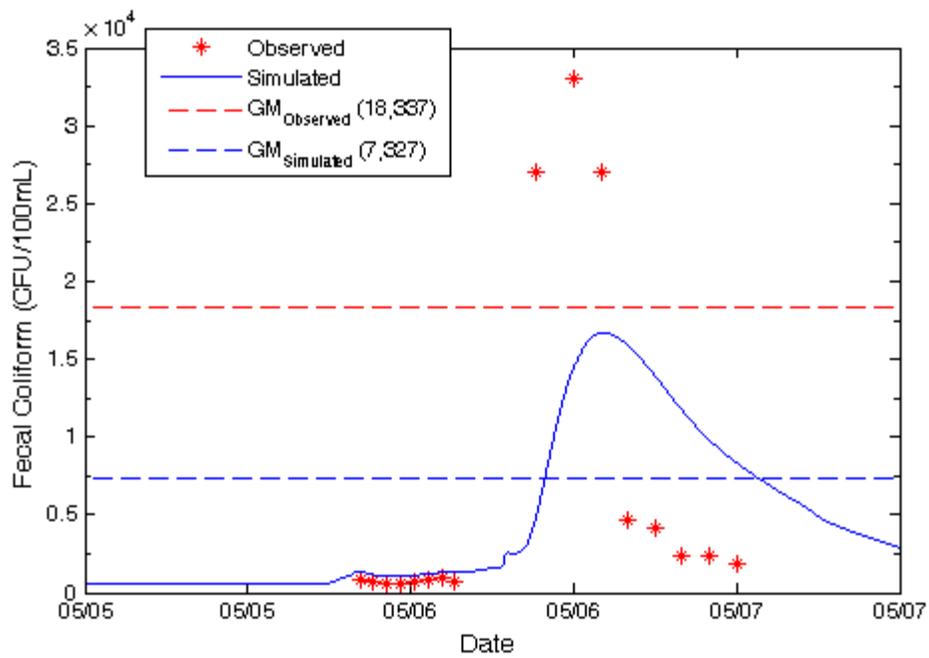
4.1.1. May 06, 2003 Storm - TF280



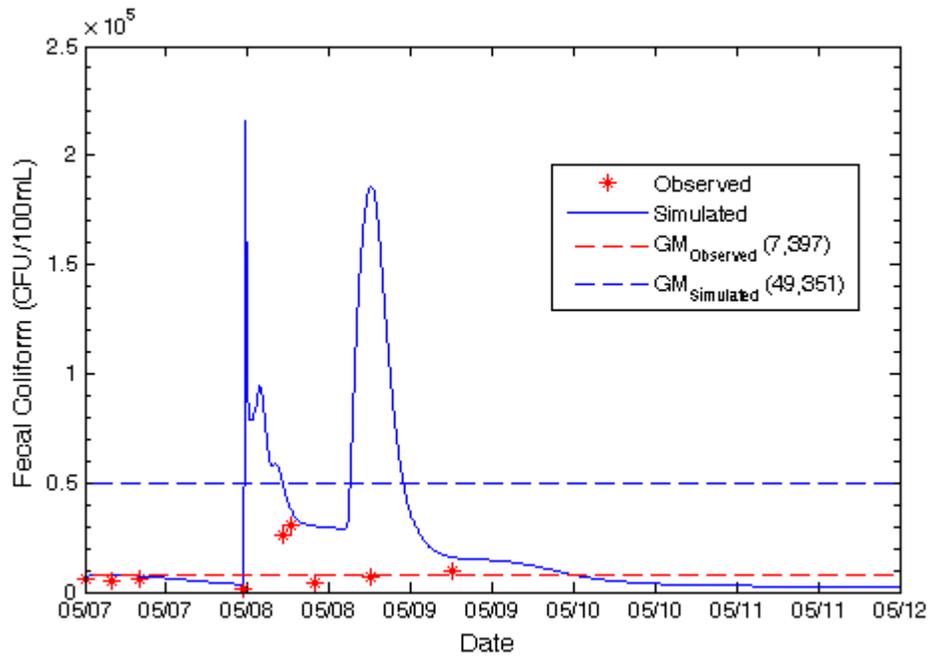
4.1.2. May 06, 2003 Storm - TF680



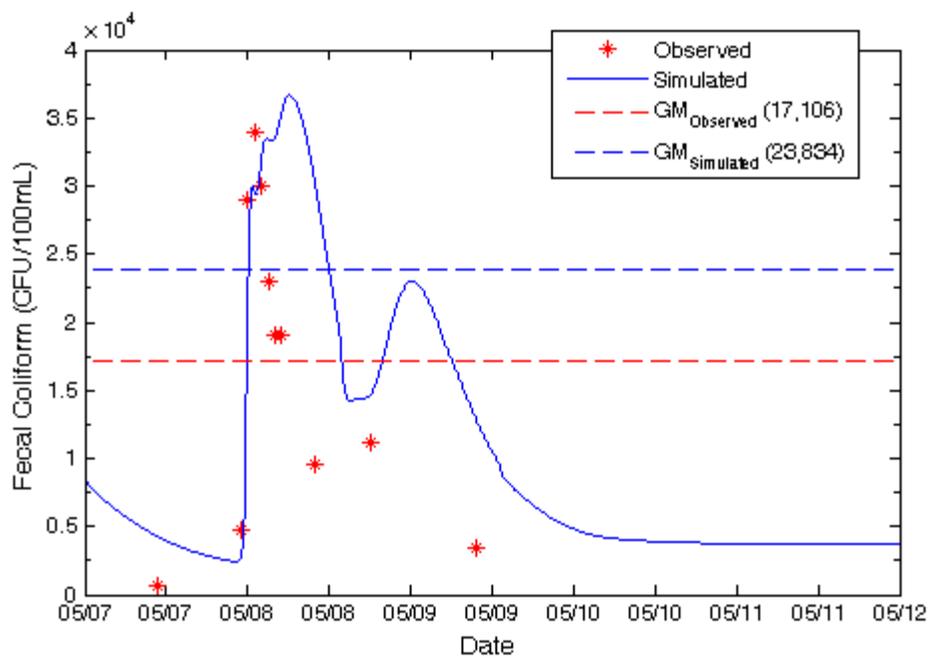
4.1.3. May 06, 2003 Storm - TF975



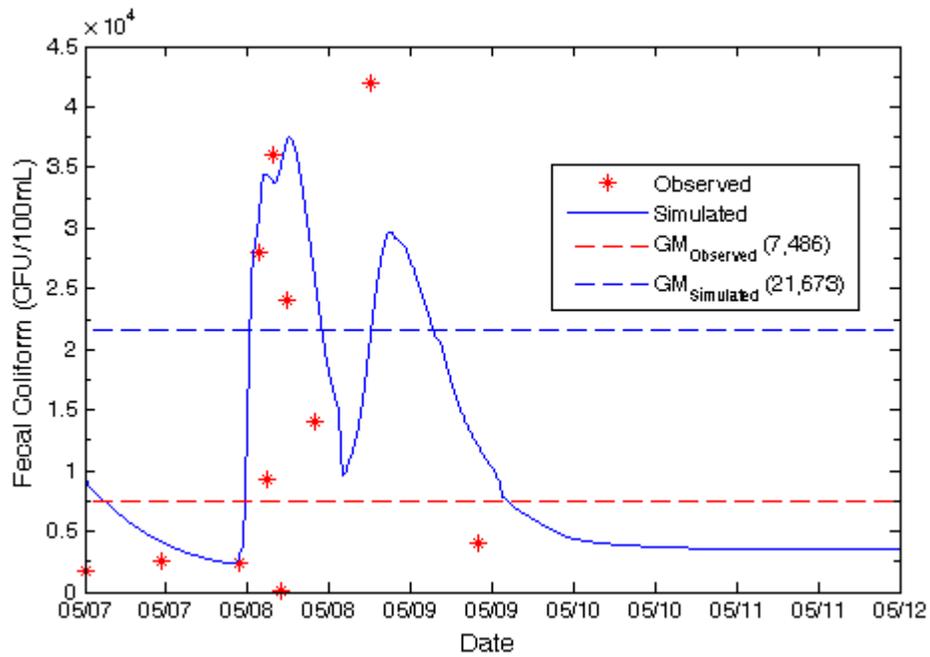
4.1.4. May 07, 2003 Storm - TF280



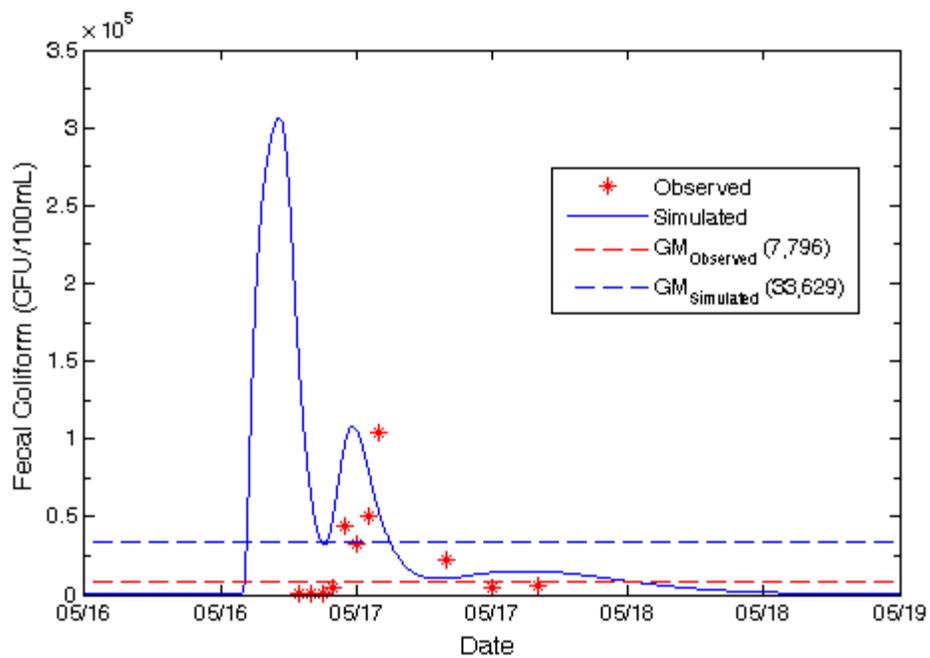
4.1.5. May 07, 2003 Storm - TF680



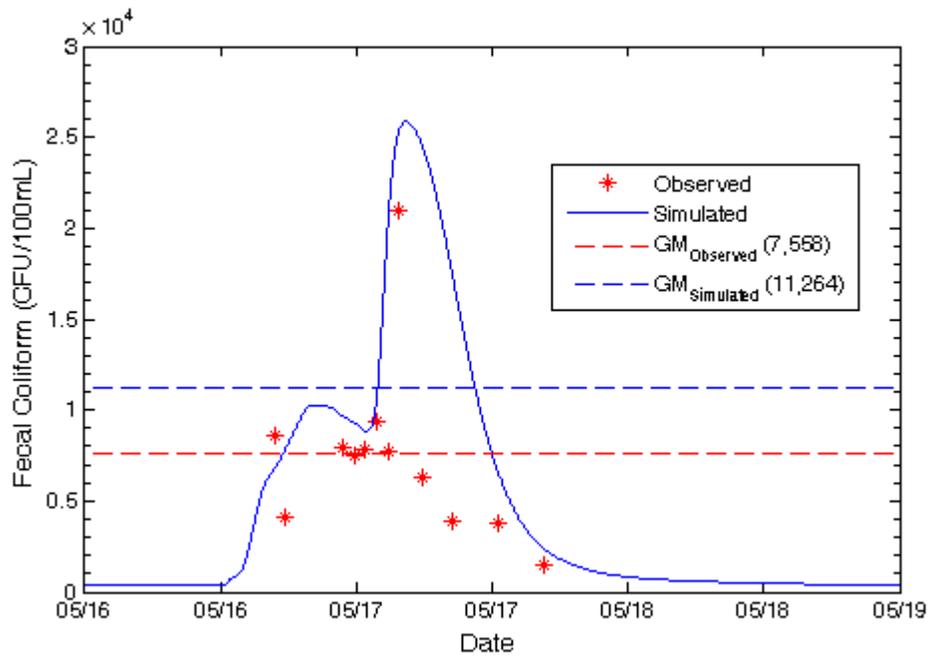
4.1.6. May 07, 2003 Storm - TF975



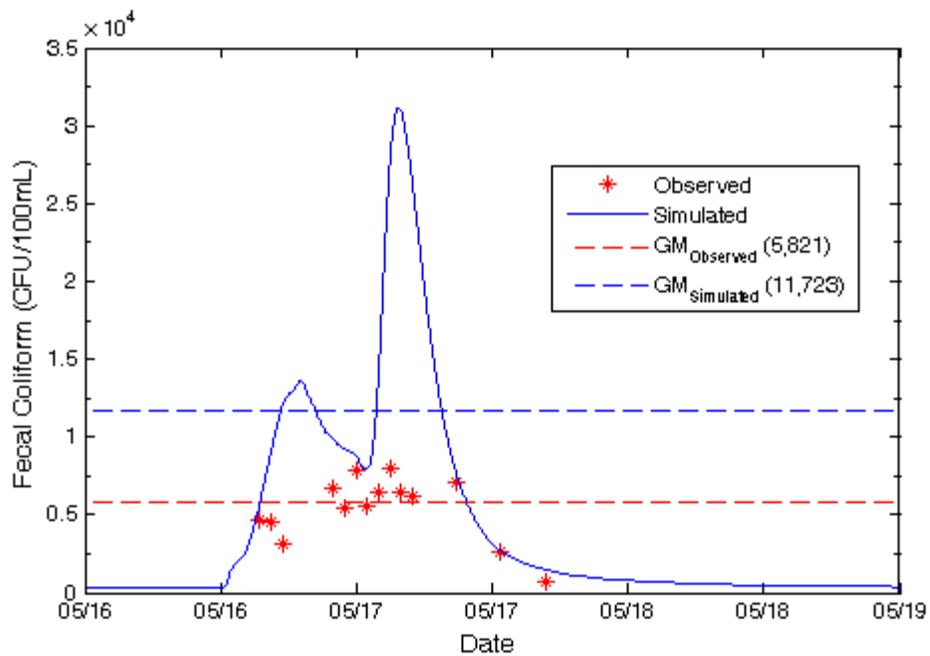
4.1.7. May 16, 2003 Storm - TF280



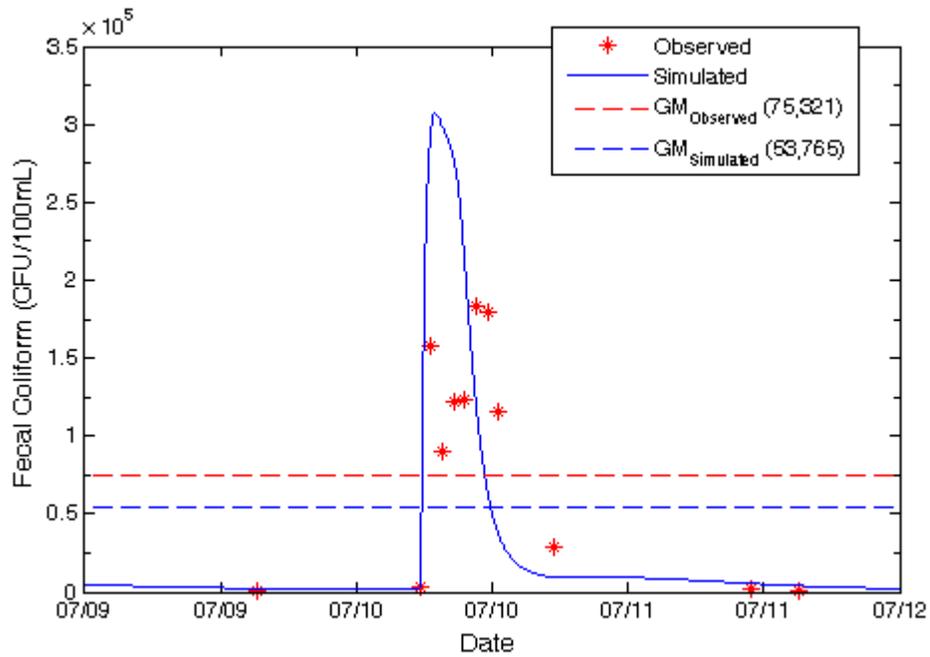
4.1.8. May 16, 2003 Storm - TF680



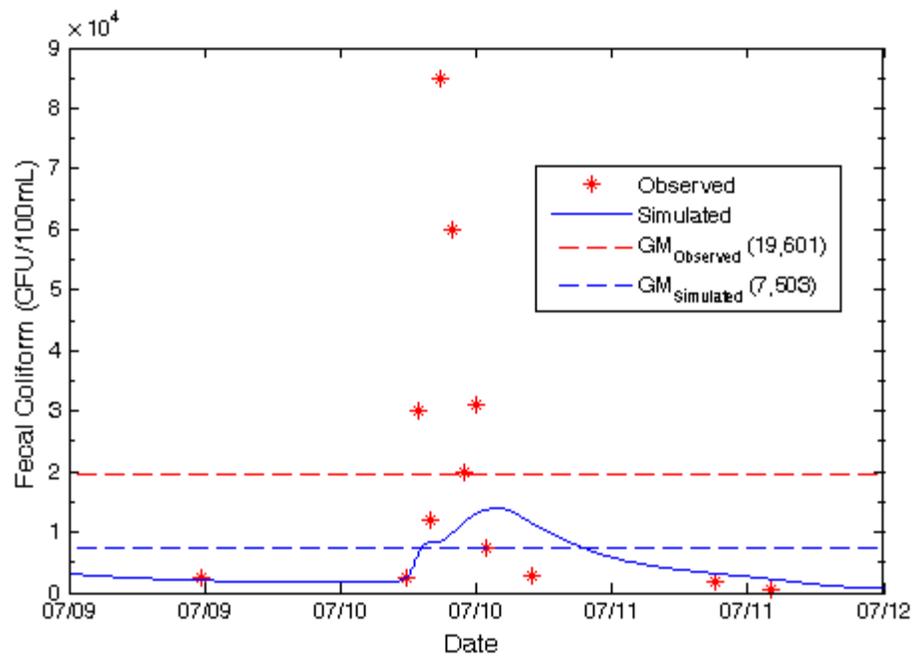
4.1.9. May 16, 2003 Storm - TF975



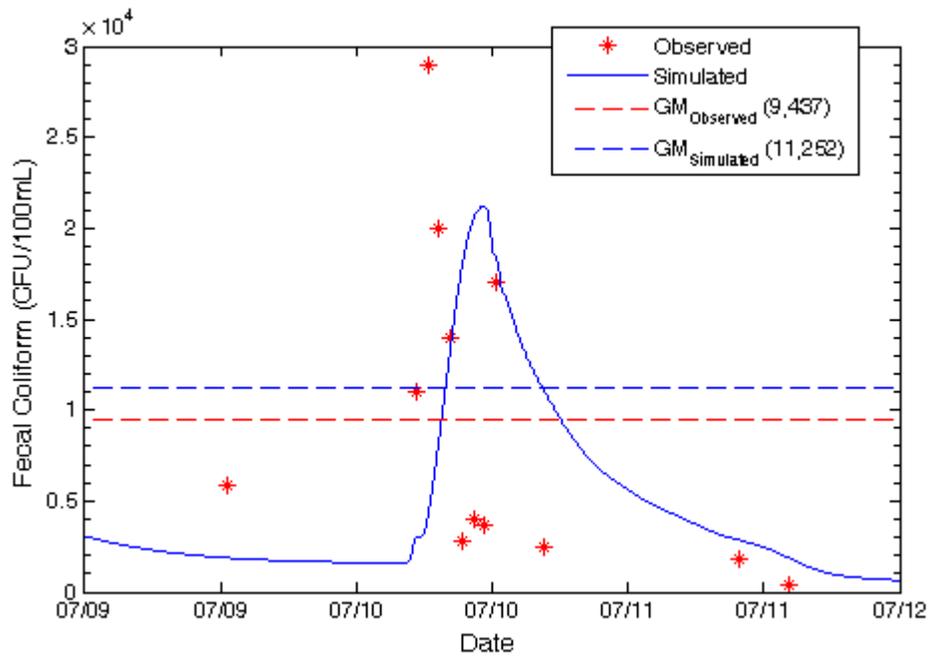
4.1.10. July 10, 2003 Storm - TF280



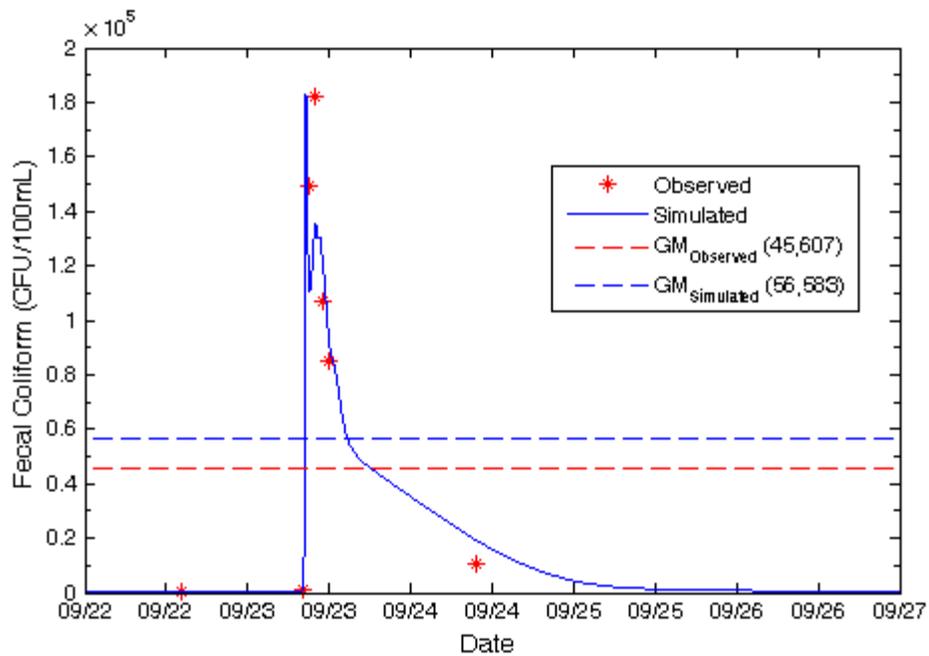
4.1.11. July 10, 2003 Storm - TF680



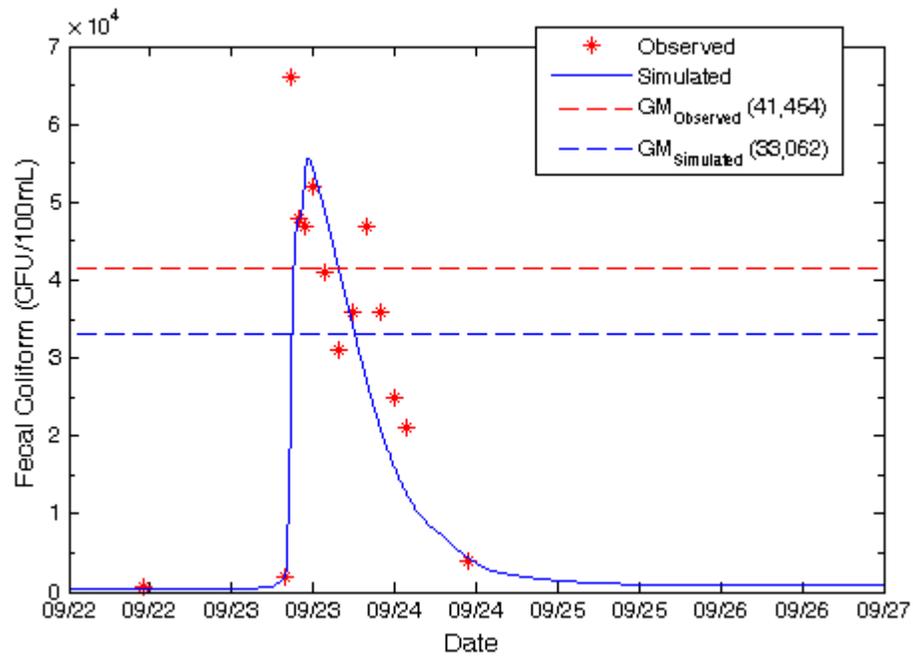
4.1.12. July 10, 2003 Storm - TF975



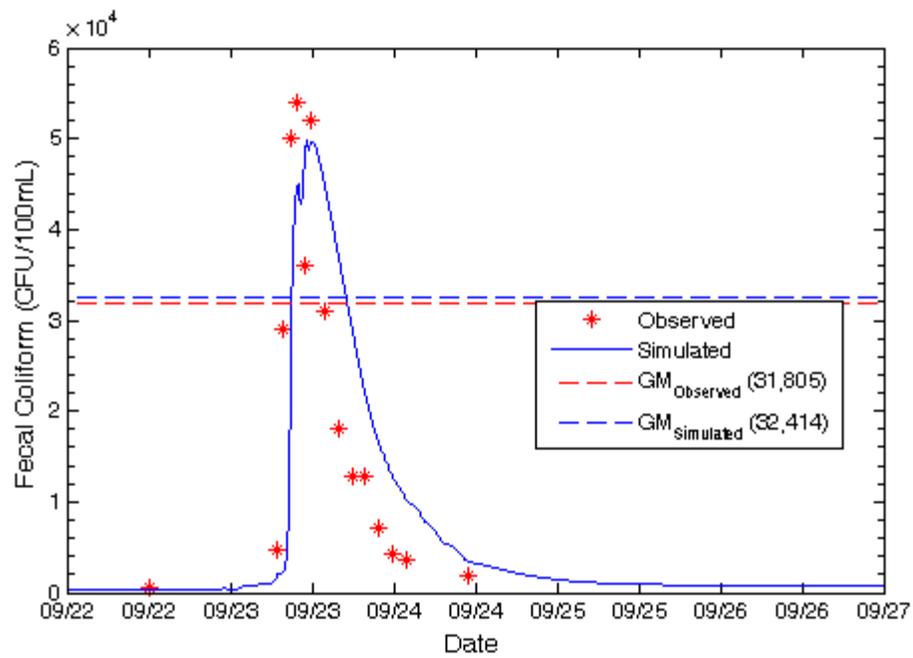
4.1.13. Sept 23, 2003 Storm - TF280



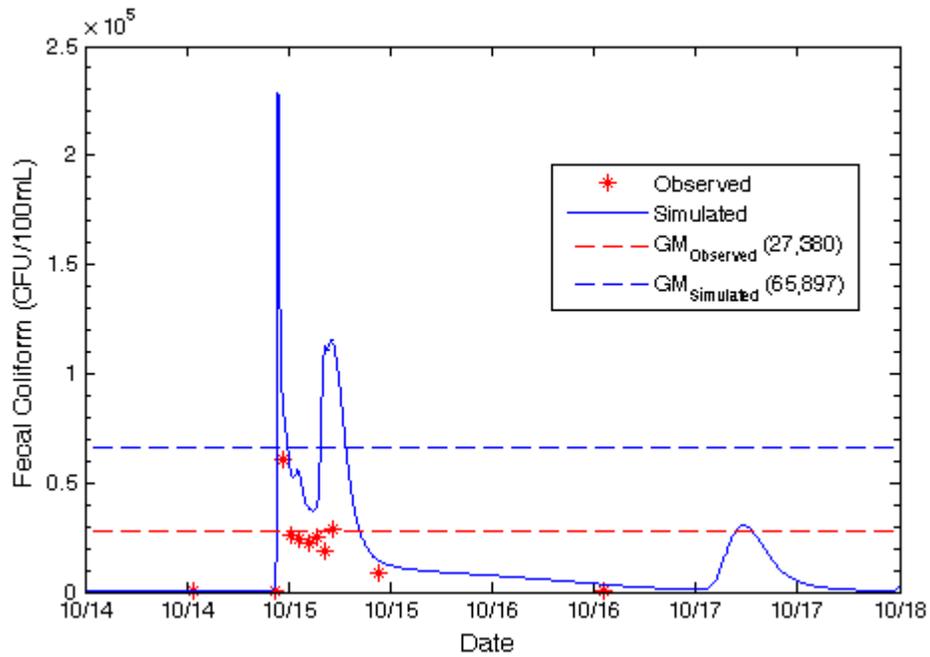
4.1.14. Sept 23, 2003 Storm - TF680



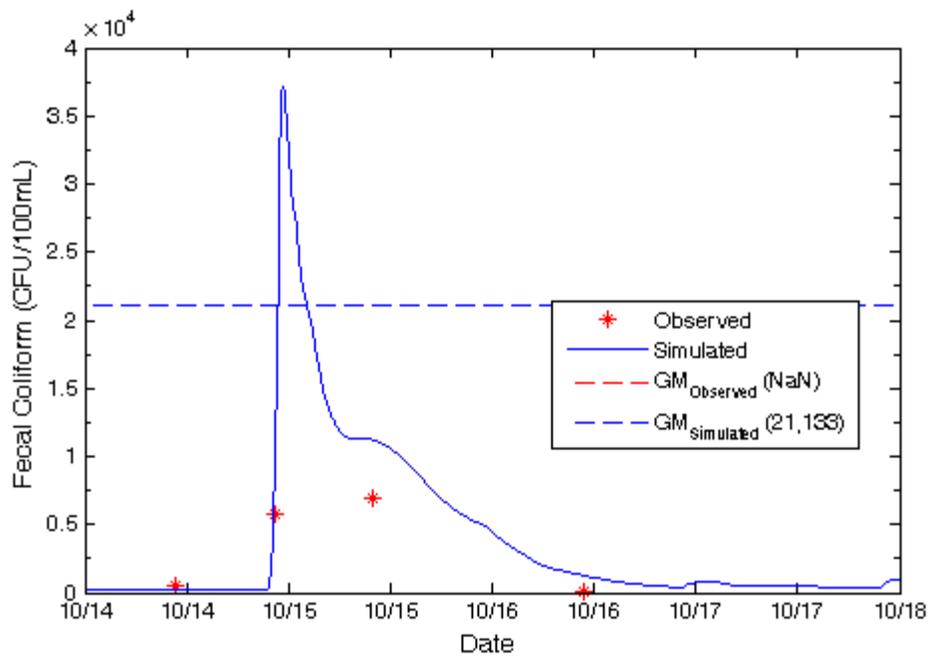
4.1.15. Sept 23, 2003 Storm - TF975



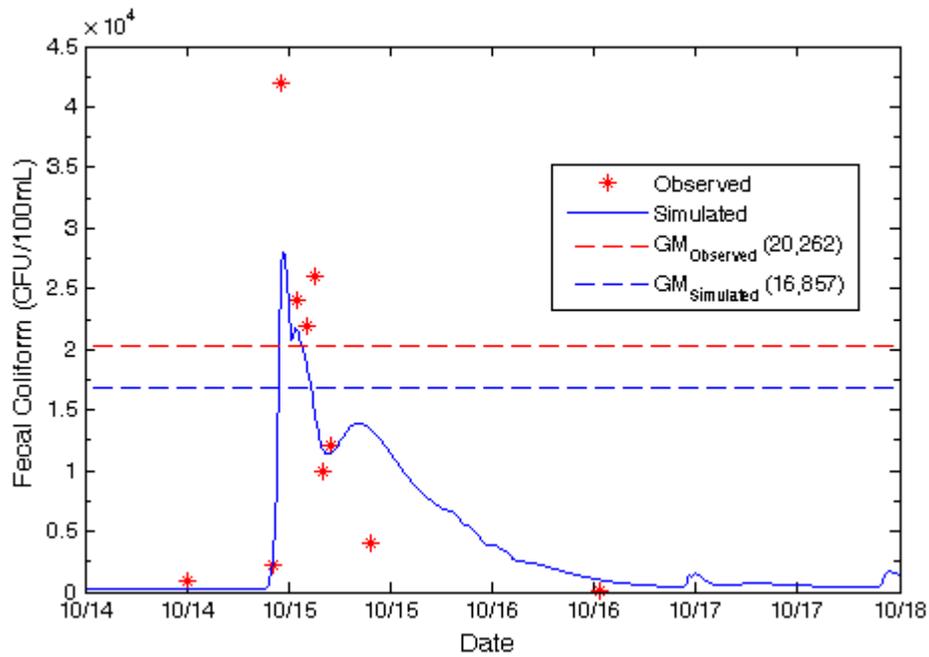
4.1.16. Oct 14, 2003 Storm - TF280



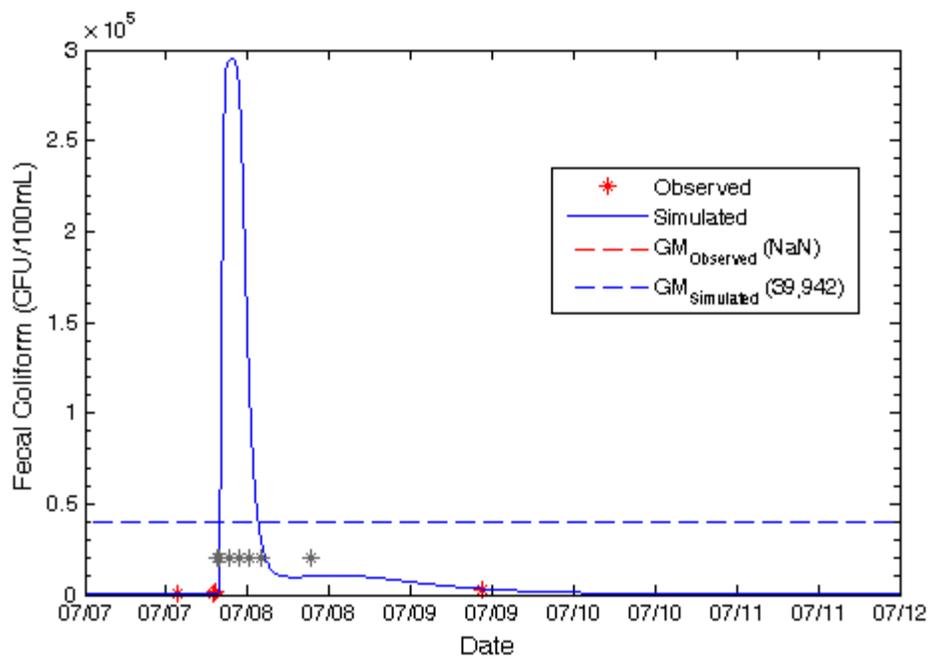
4.1.17. Oct 14, 2003 Storm - TF680



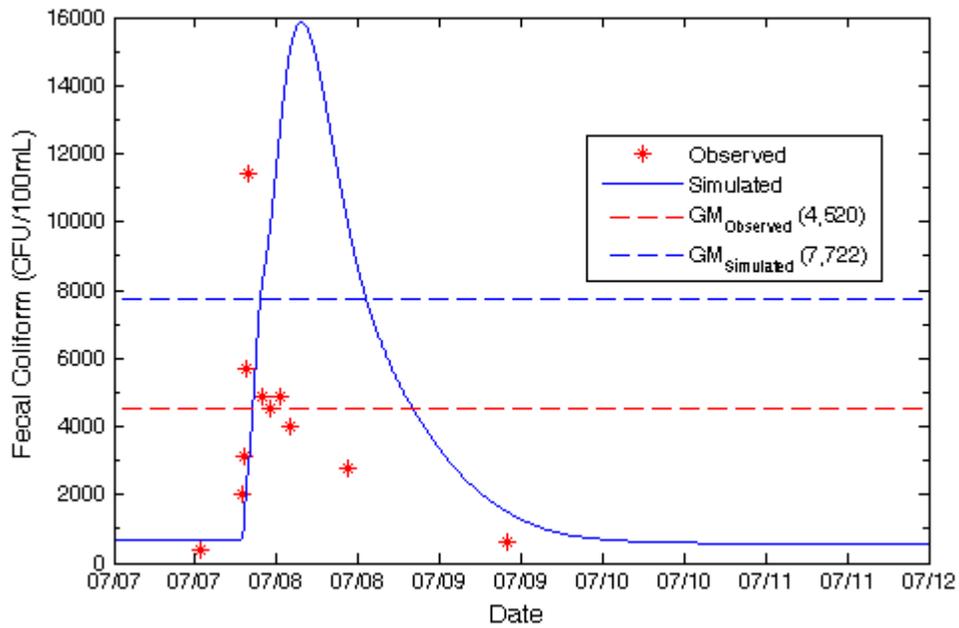
4.1.18. Oct 14, 2003 Storm - TF975



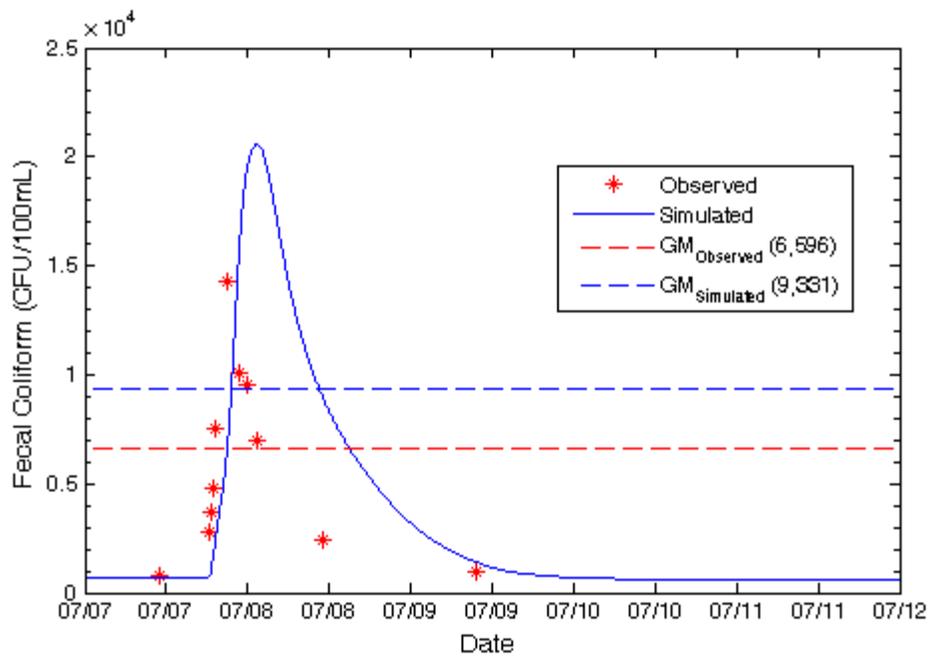
4.1.19. July 07, 2004 Storm - TF280



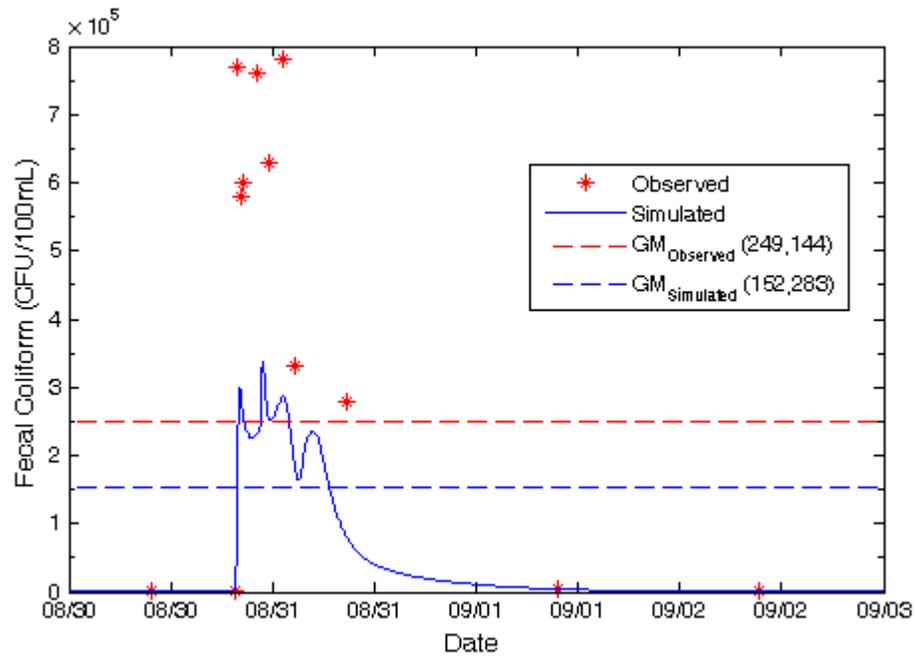
4.1.20. July 07, 2004 Storm - TF680



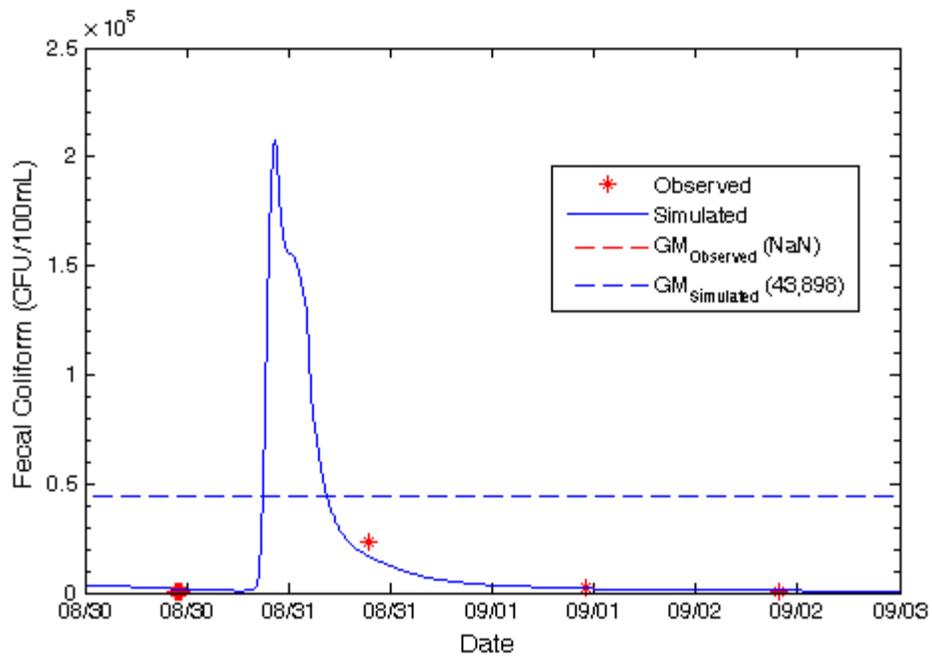
4.1.21. July 07, 2004 Storm - TF975



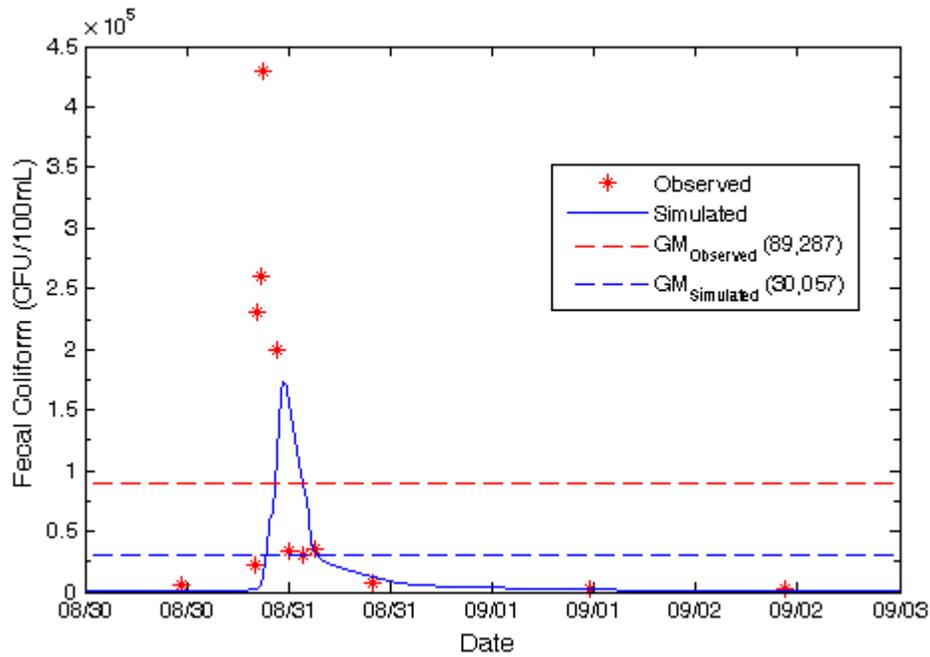
4.1.22. Aug 30, 2004 Storm - TF280



4.1.23. Aug 30, 2004 Storm - TF680

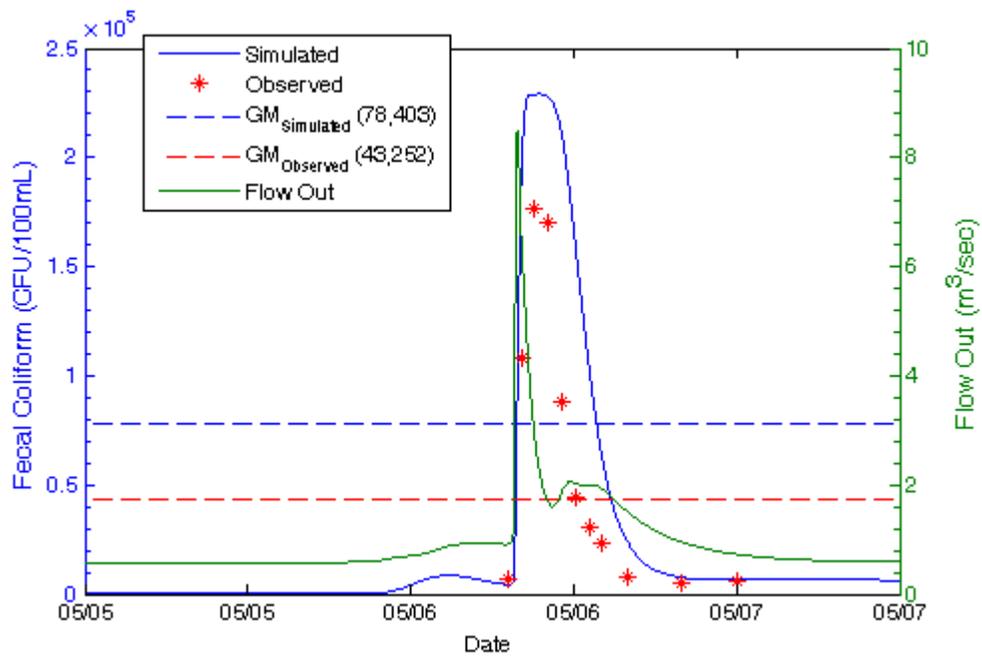


4.1.24. Aug 30, 2004 Storm - TF975

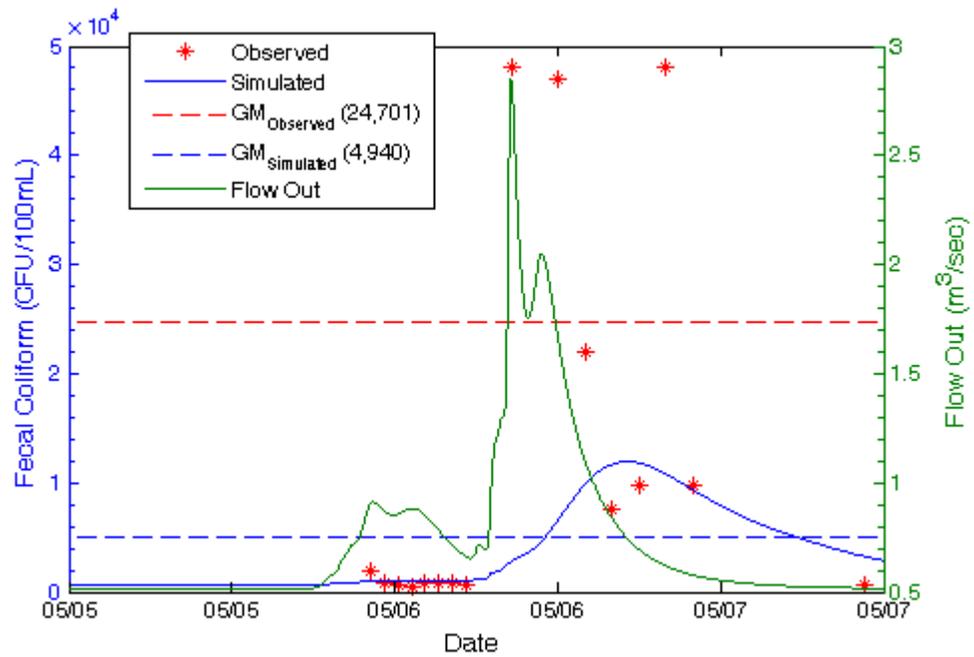


4.2. Fecal Coliform and Flow Out Time Series

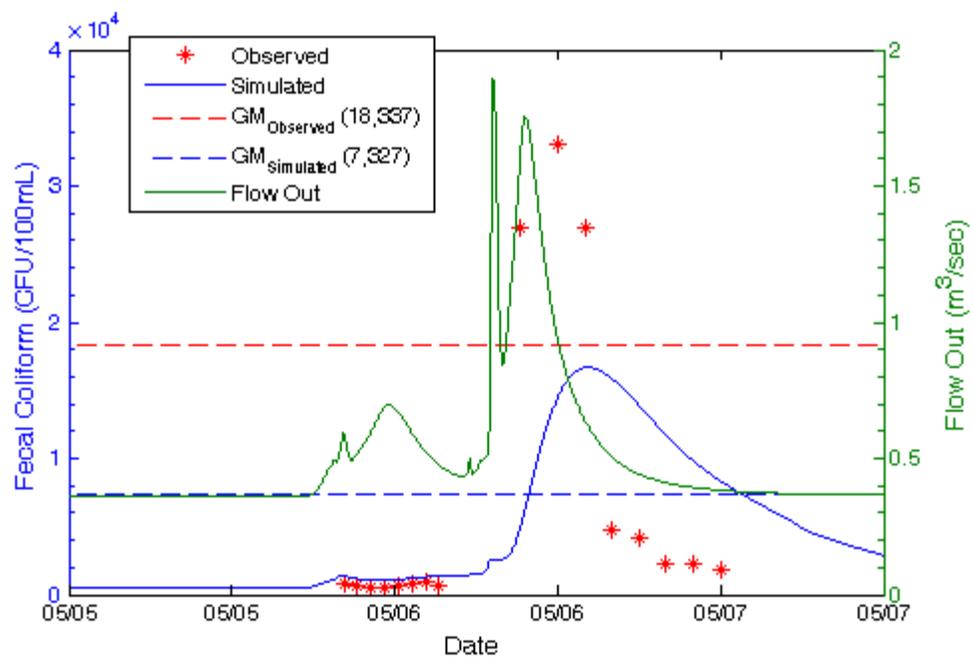
4.2.1. May 06, 2003 Storm - TF280



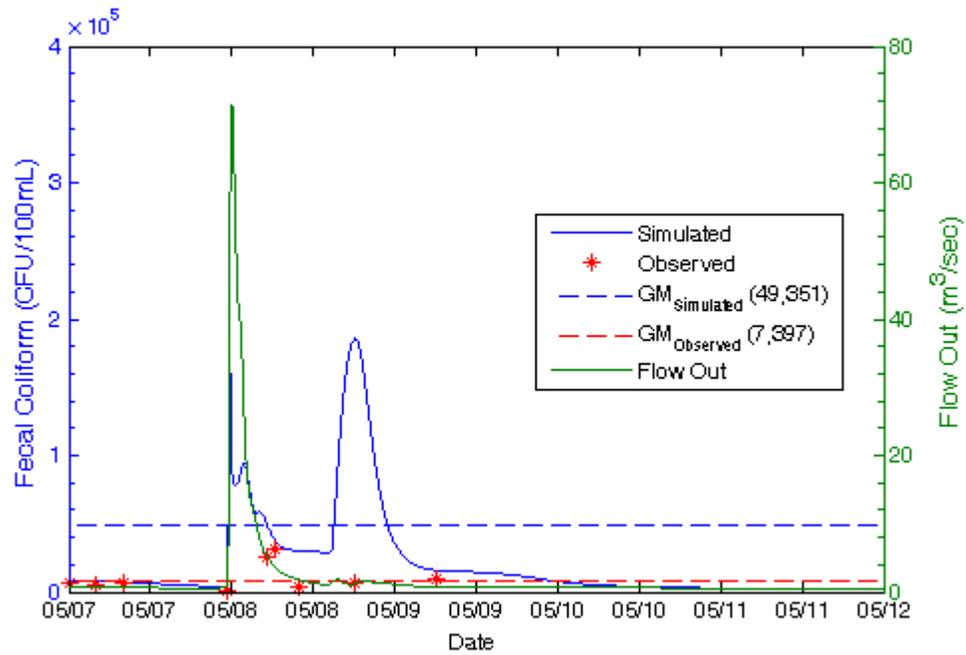
4.2.2. May 06, 2003 Storm - TF680



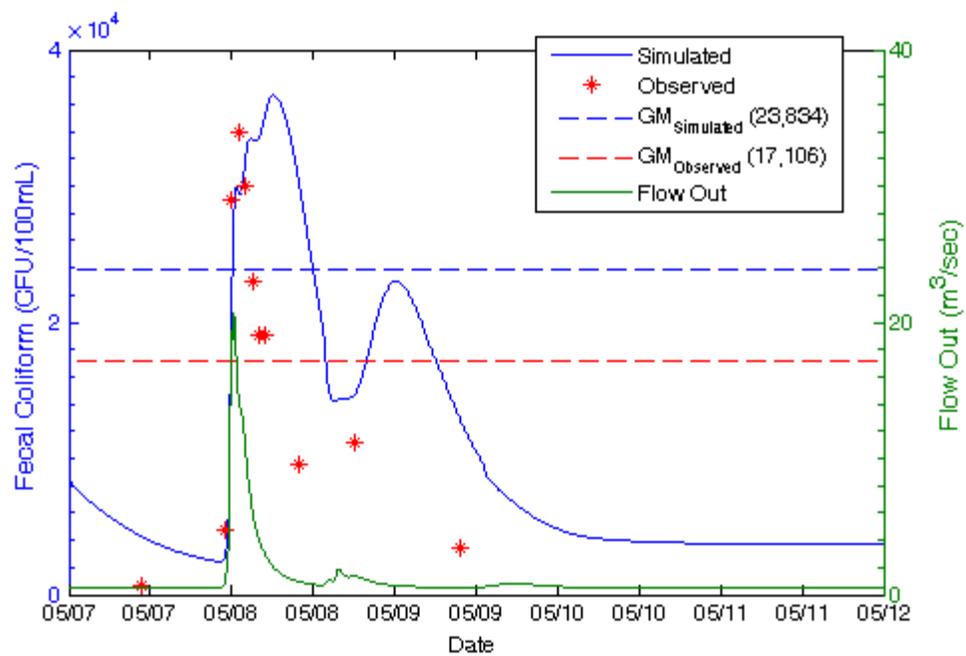
4.2.3. May 06, 2003 Storm - TF975



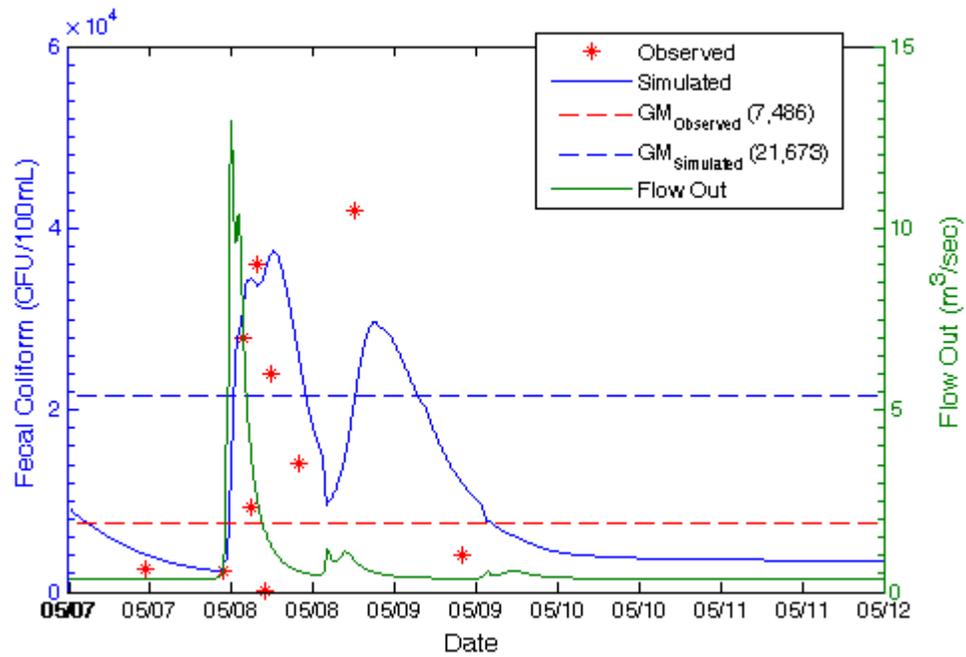
4.2.4. May 07, 2003 Storm - TF280



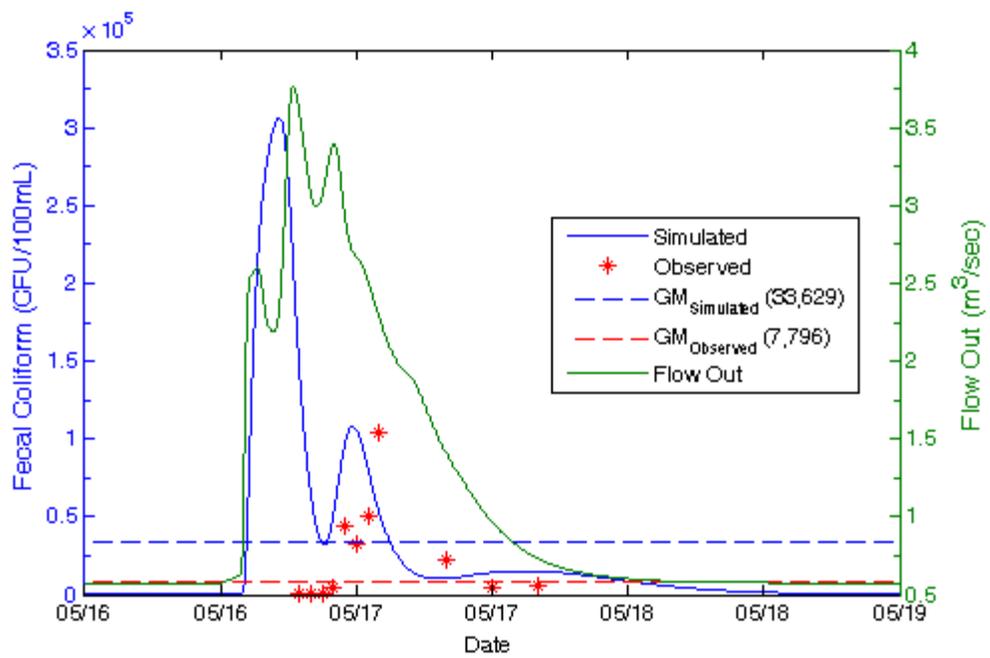
4.2.5. May 07, 2003 Storm - TF680



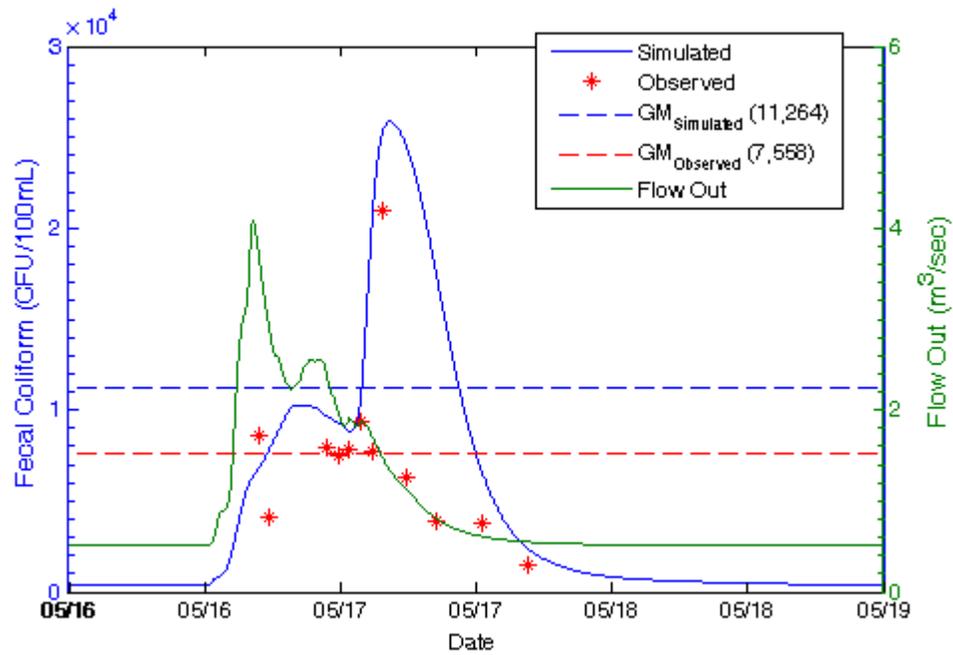
4.2.6. May 07, 2003 Storm - TF975



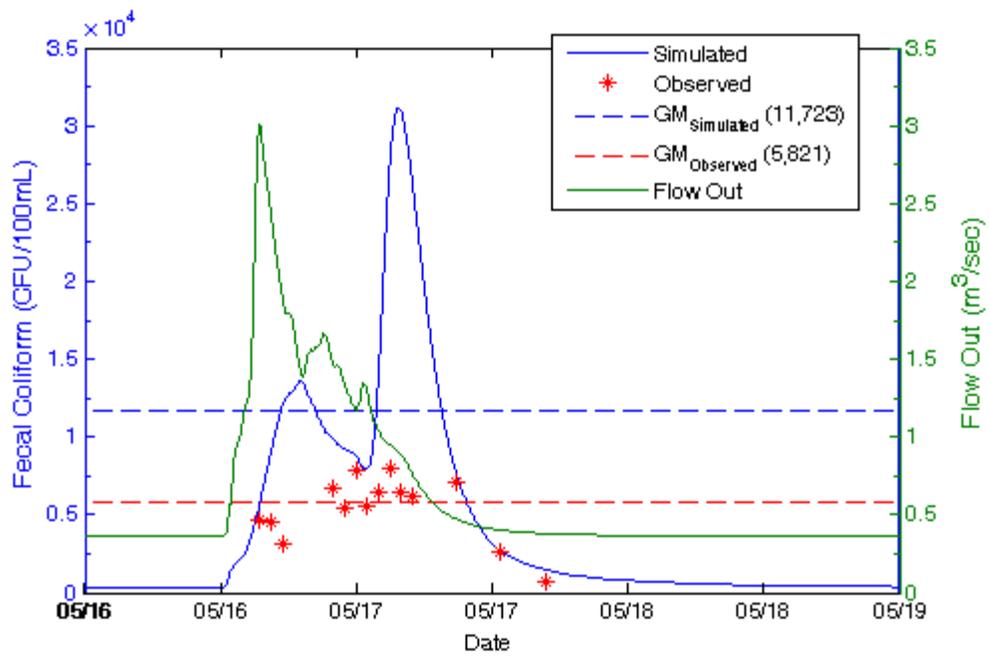
4.2.7. May 16, 2003 Storm - TF280



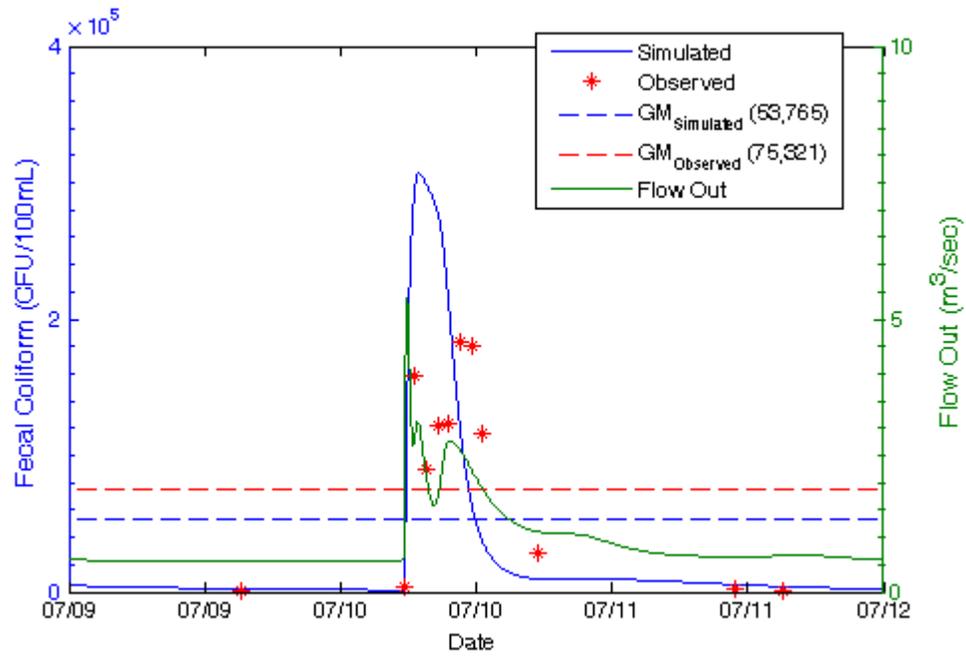
4.2.8. May 16, 2003 Storm - TF680



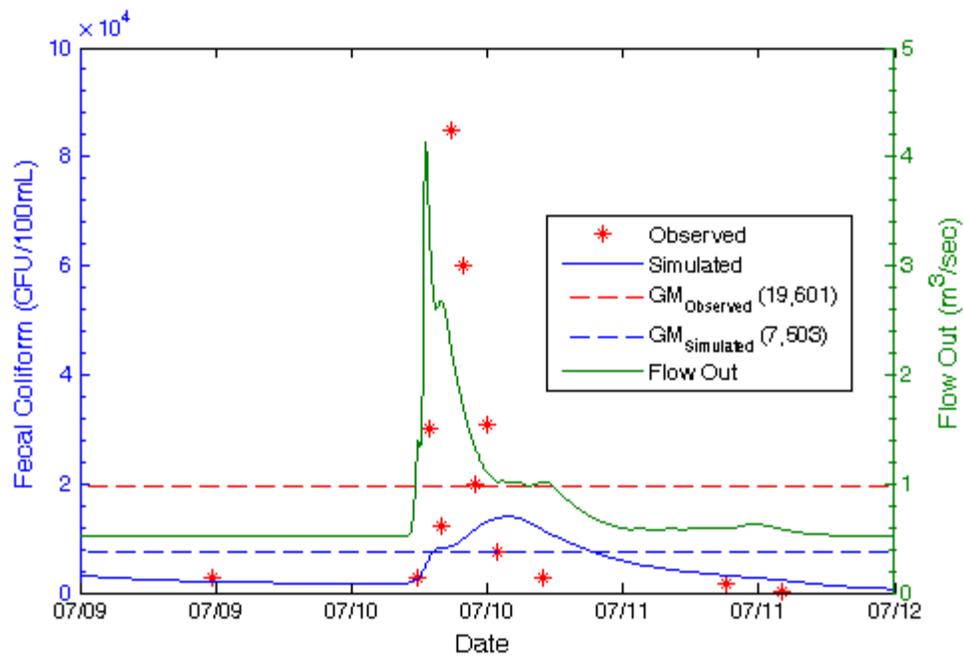
4.2.9. May 16, 2003 Storm - TF975



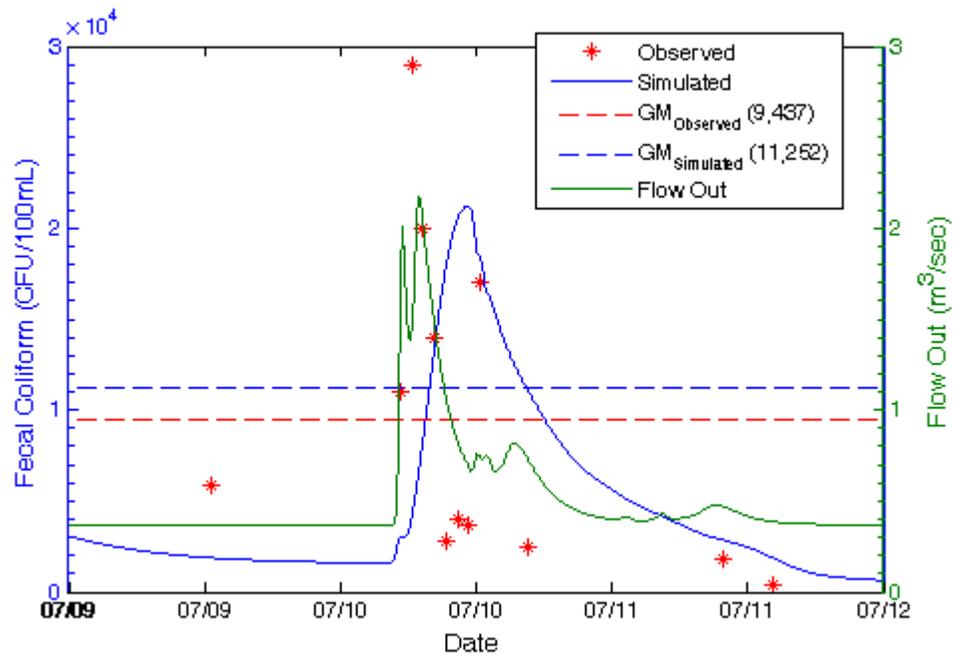
4.2.10. July 10, 2003 Storm - TF280



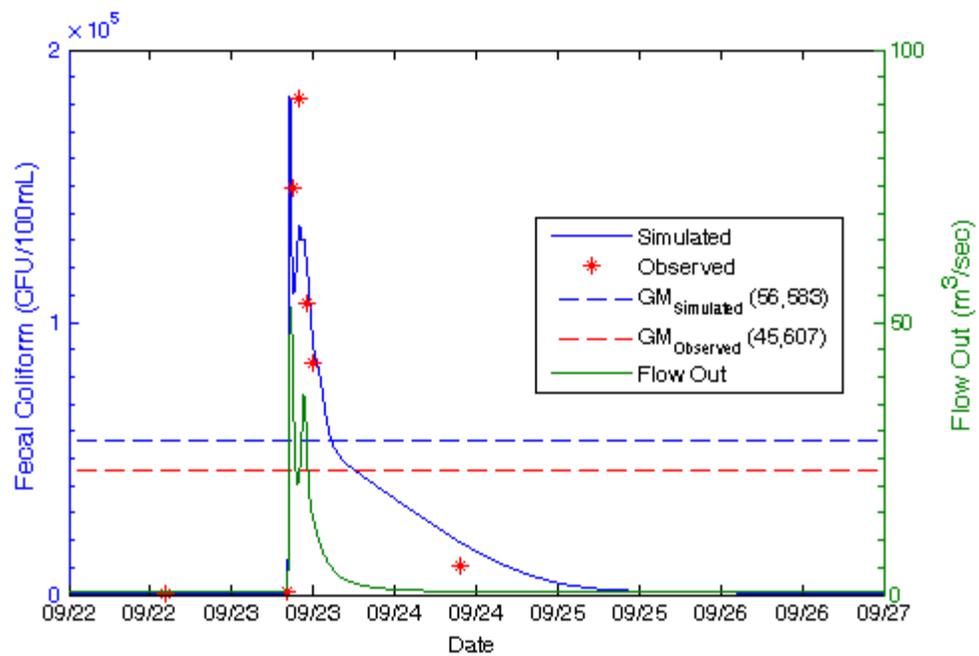
4.2.11. July 10, 2003 Storm - TF680



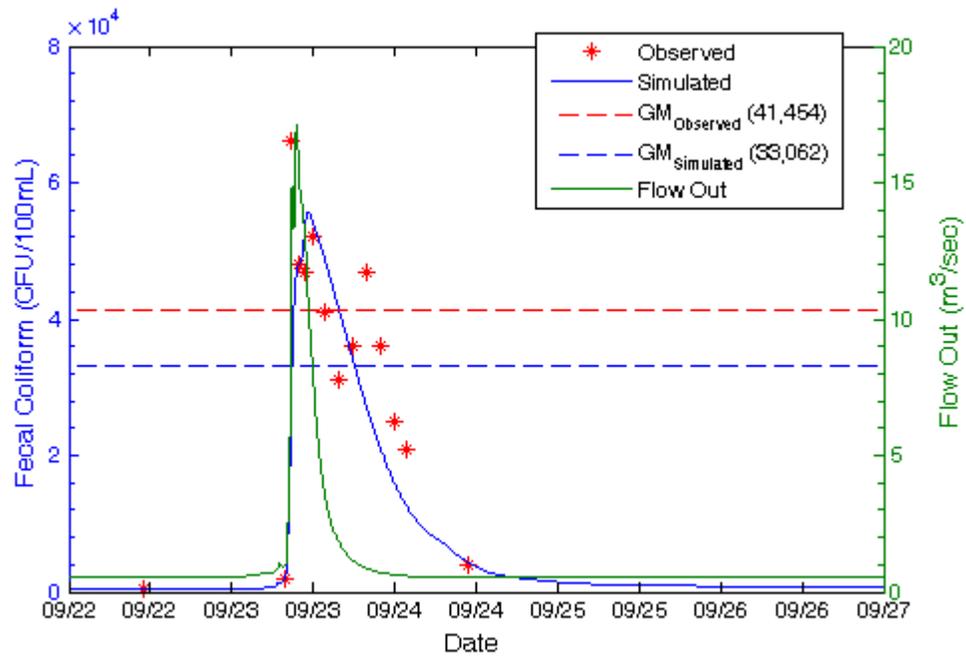
4.2.12. July 10, 2003 Storm - TF975



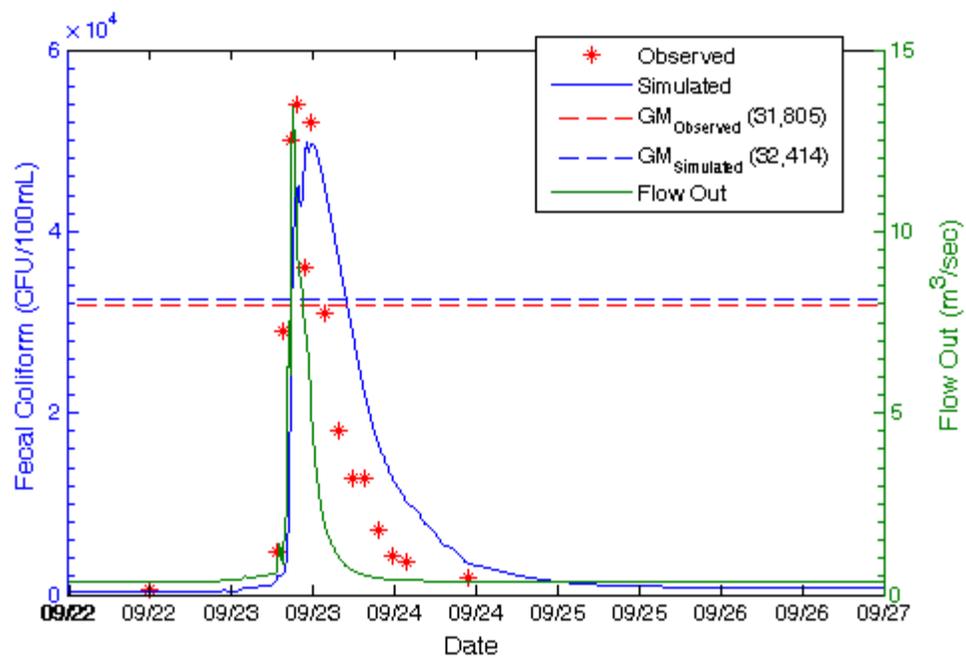
4.2.13. Sept 23, 2003 Storm - TF280



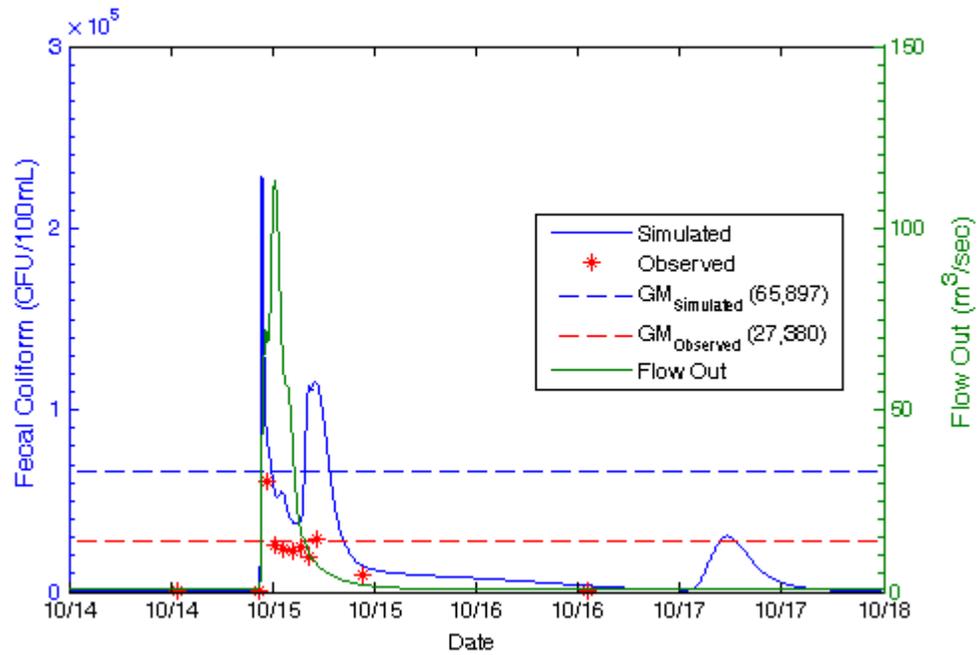
4.2.14. Sept 23, 2003 Storm - TF680



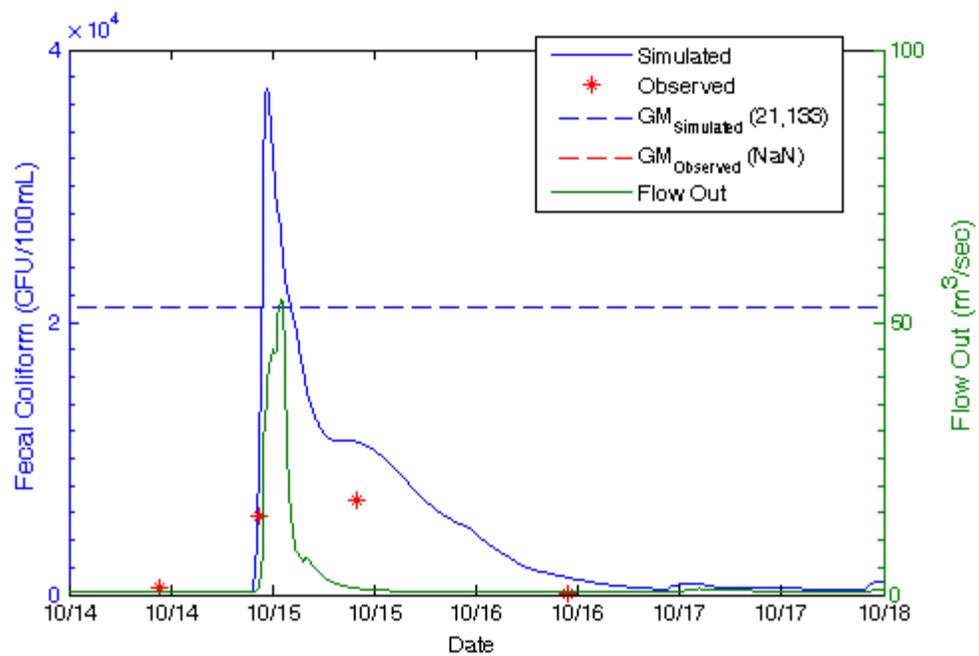
4.2.15. Sept 23, 2003 Storm - TF975



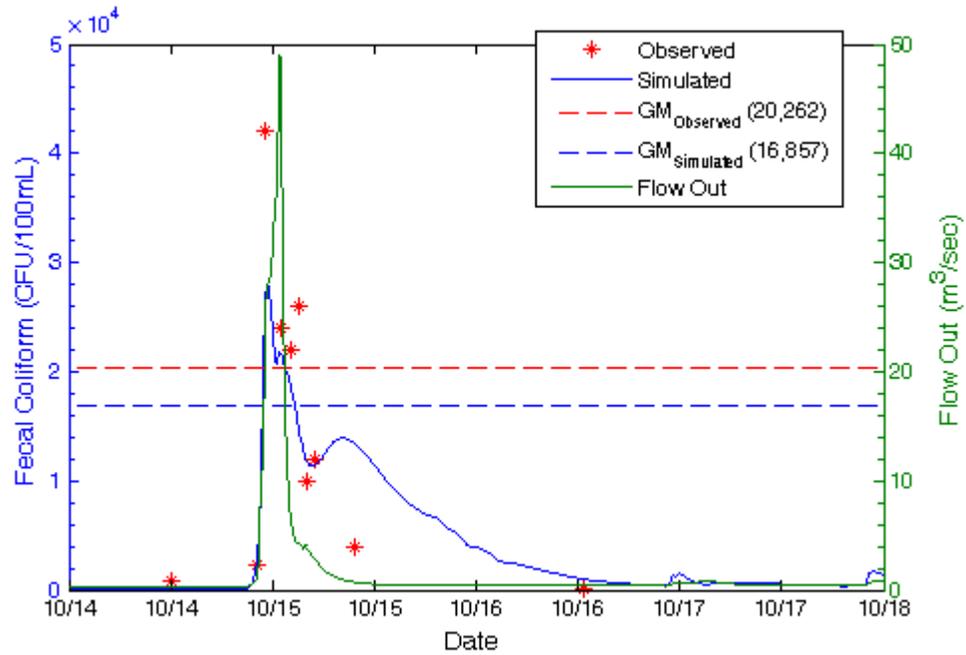
4.2.16. Oct 14, 2003 Storm - TF280



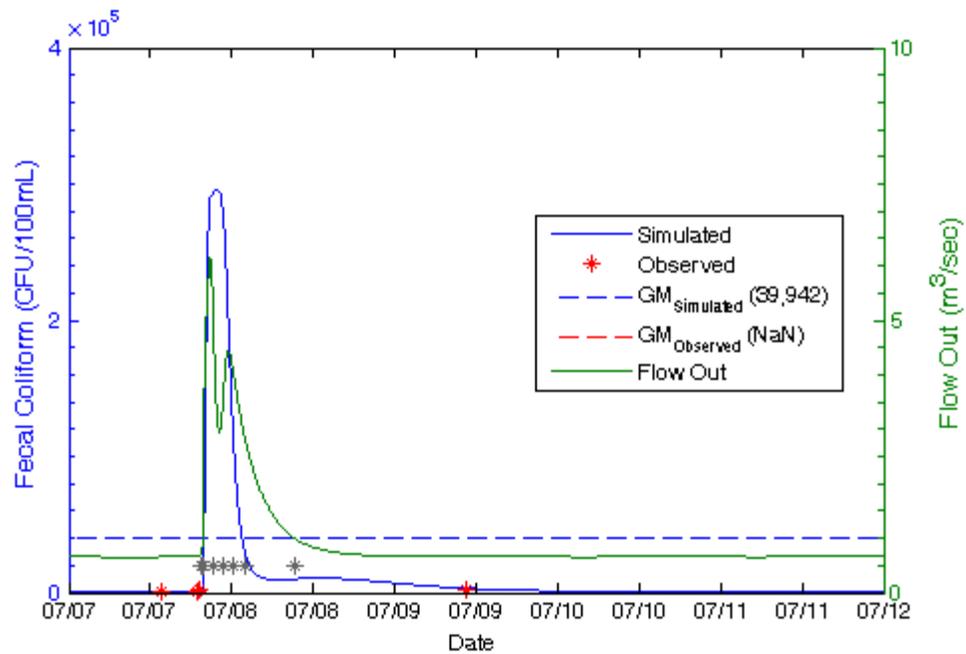
4.2.17. Oct 14, 2003 Storm - TF680



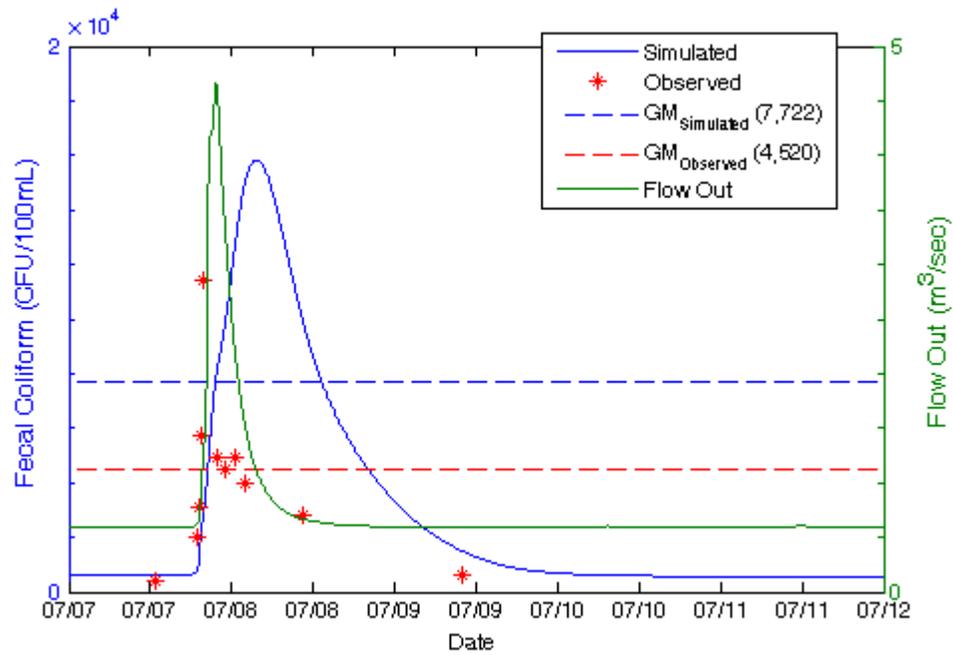
4.2.18. Oct 14, 2003 Storm - TF975



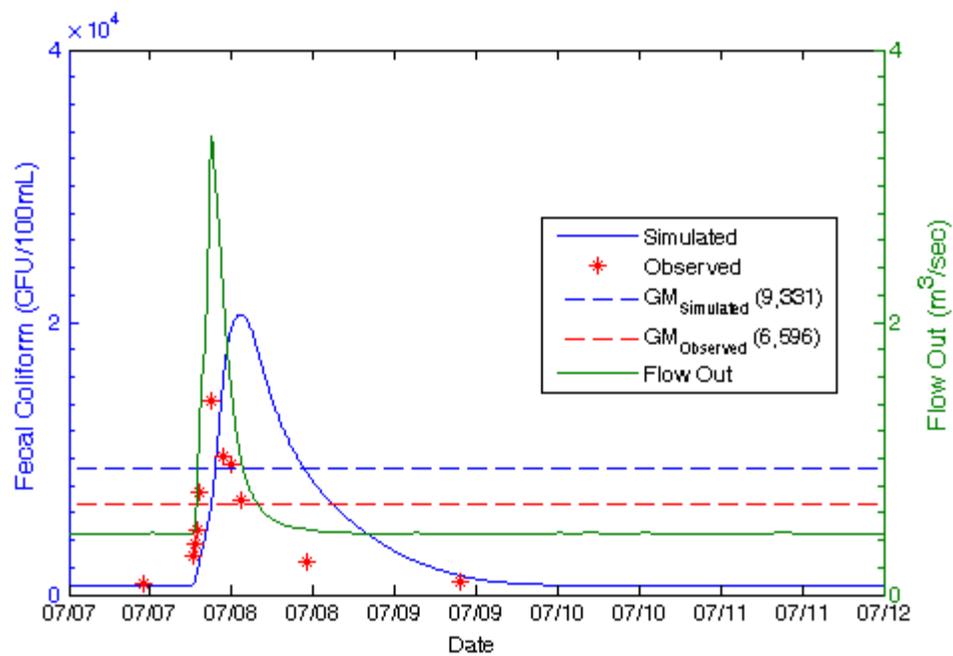
4.2.19. July 07, 2004 Storm - TF280



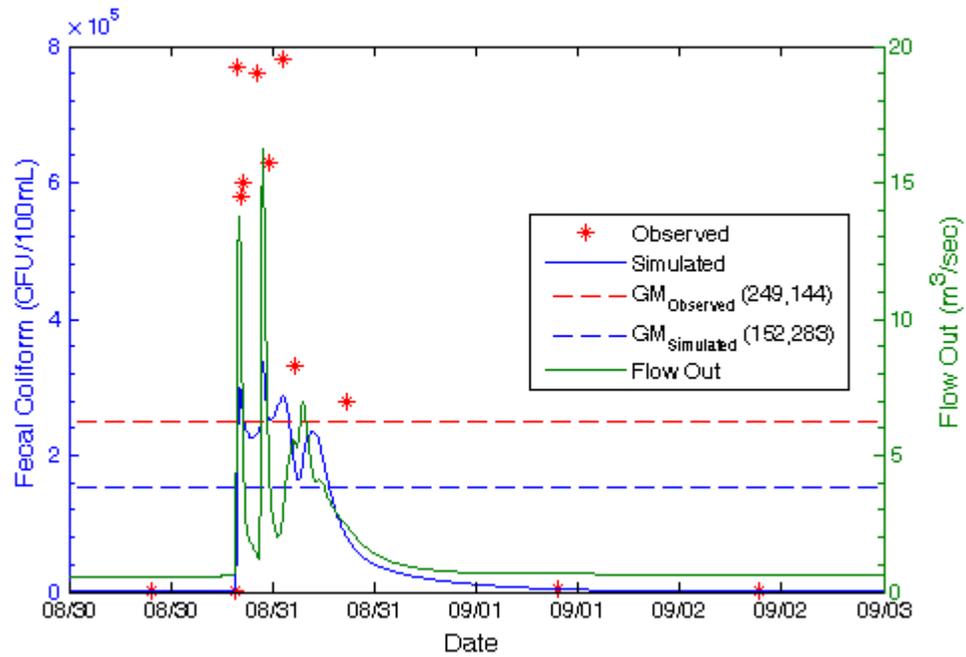
4.2.20. July 07, 2004 Storm - TF680



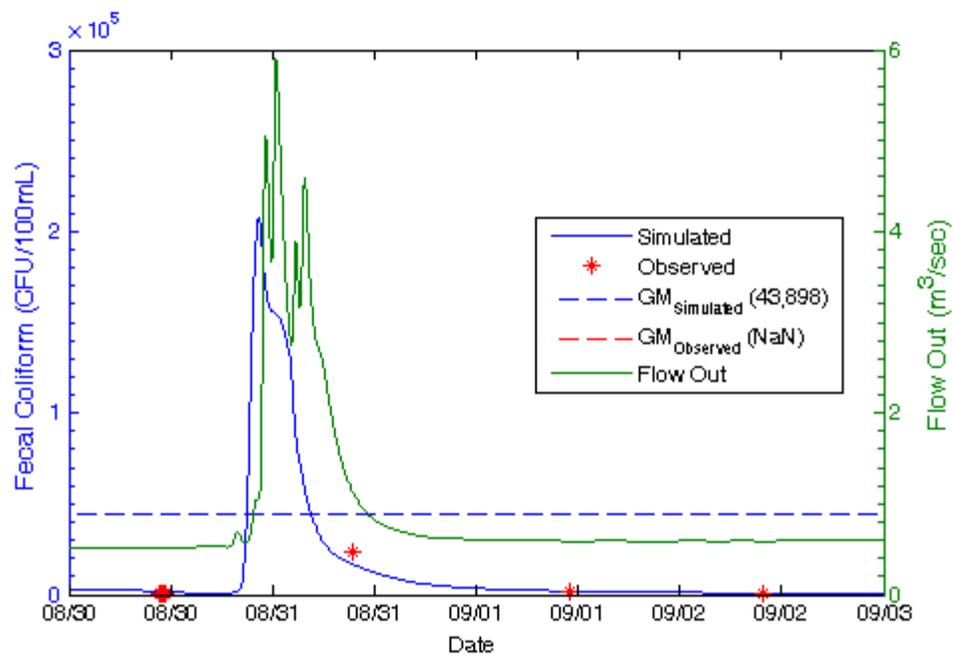
4.2.21. July 07, 2004 Storm - TF975



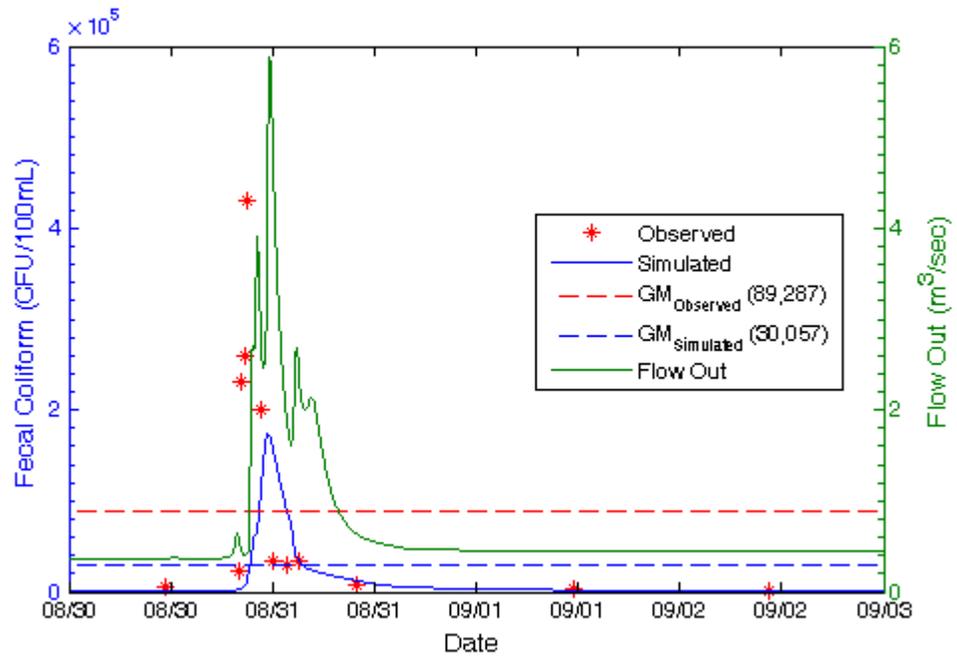
4.2.22. Aug 30, 2004 Storm - TF280



4.2.23. Aug 30, 2004 Storm - TF680

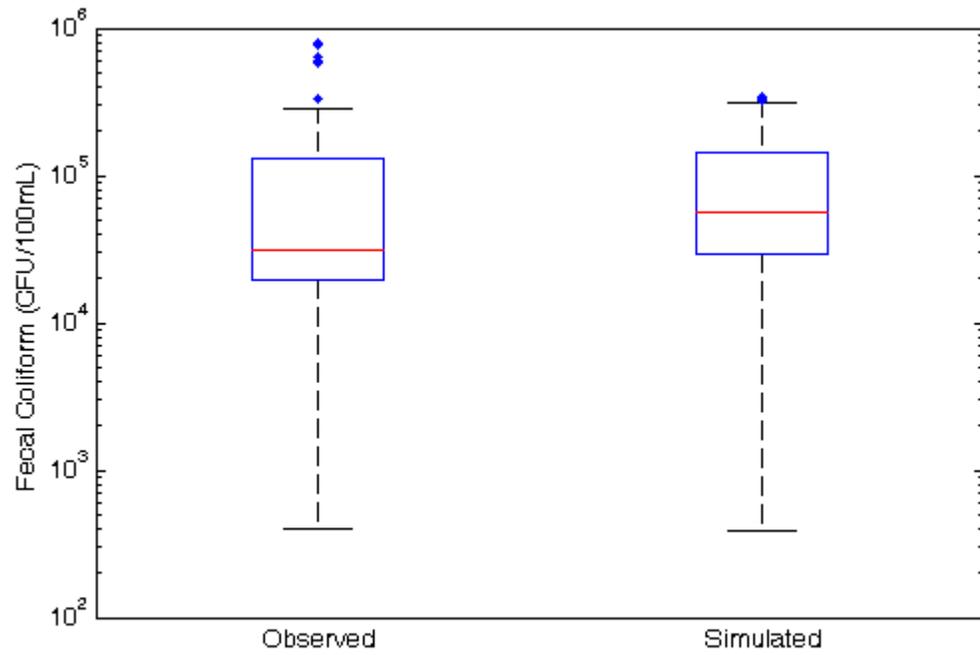


4.2.24. Aug 30, 2004 Storm - TF975

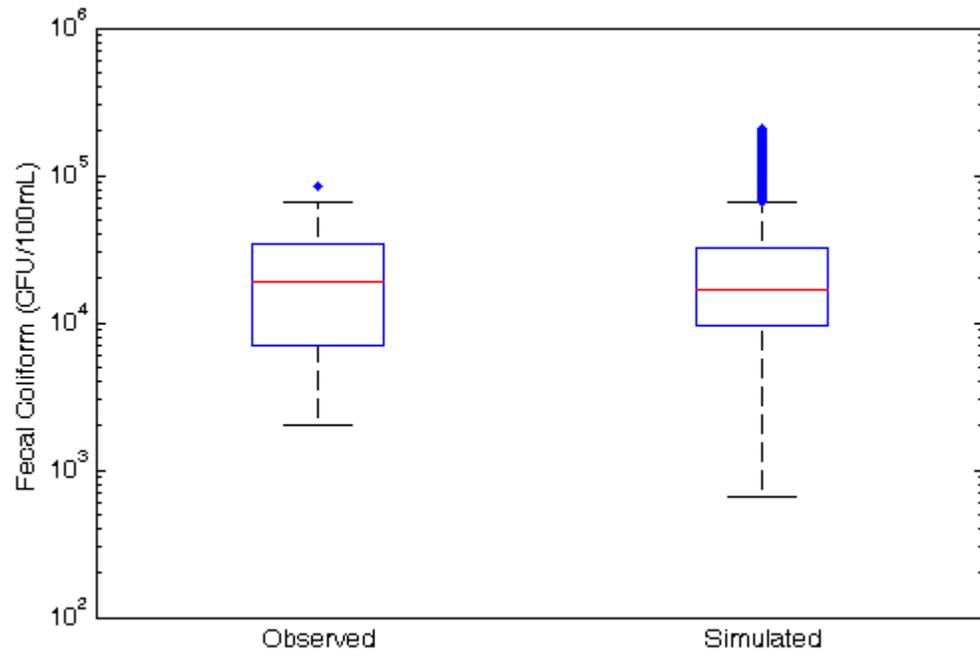


4.3. Predicted and Observed Fecal Coliform Concentration Box Plots

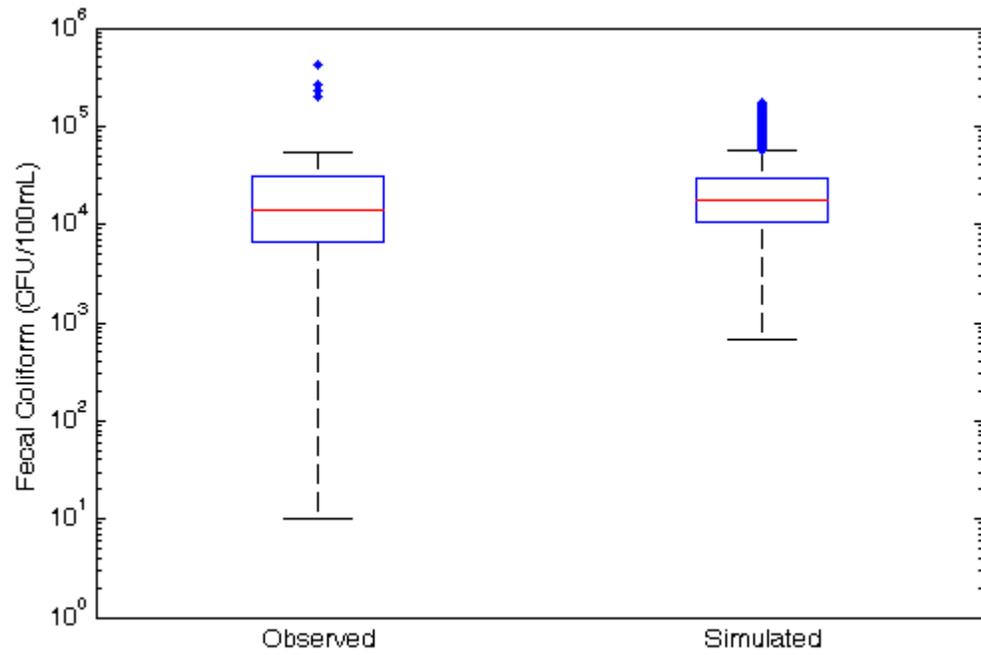
4.3.1. Fecal Coliform Concentration for 2003-2004 Storms - TF280



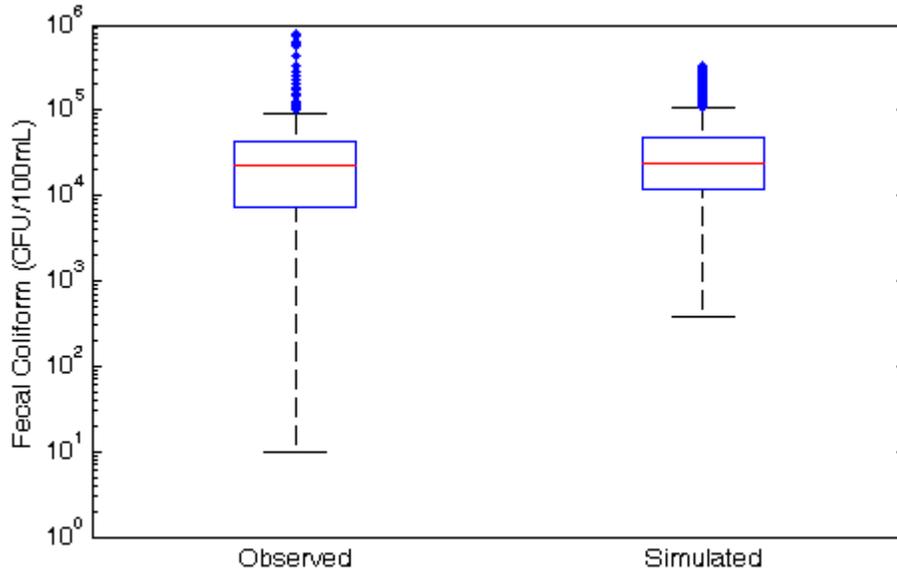
4.3.2. Fecal Coliform Concentration for 2003-2004 Storms - TF680



4.3.3. Fecal Coliform Concentration for 2003-2004 Storms - TF975



4.3.4. Fecal Coliform Concentration for 2003-2004 Storms - All Sites



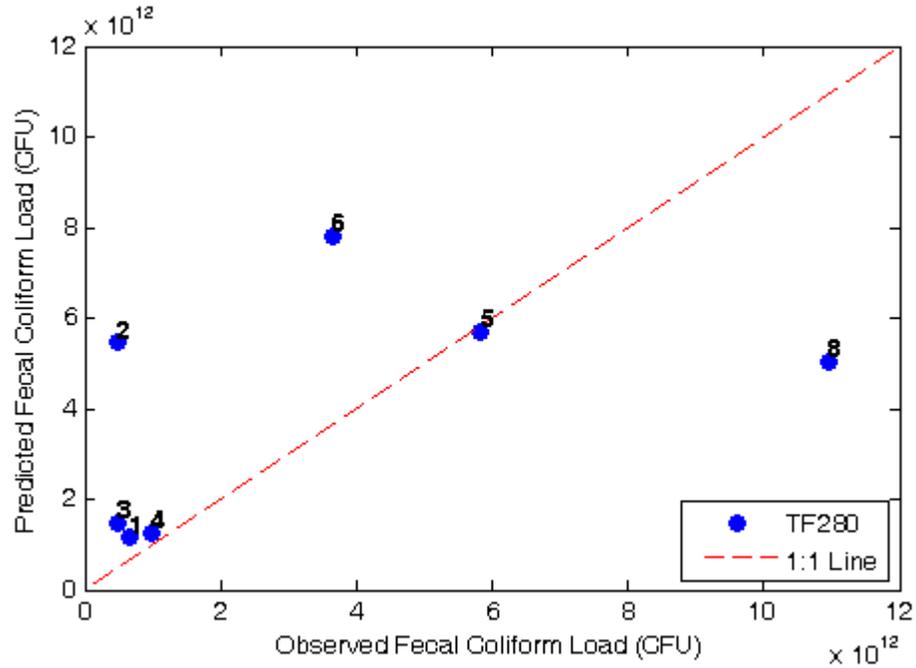
4.4. Predicted vs. Observed Fecal Coliform Load Scatter Plots

Table 4-1. Total Rainfall and Peak Flow for Eight Storms

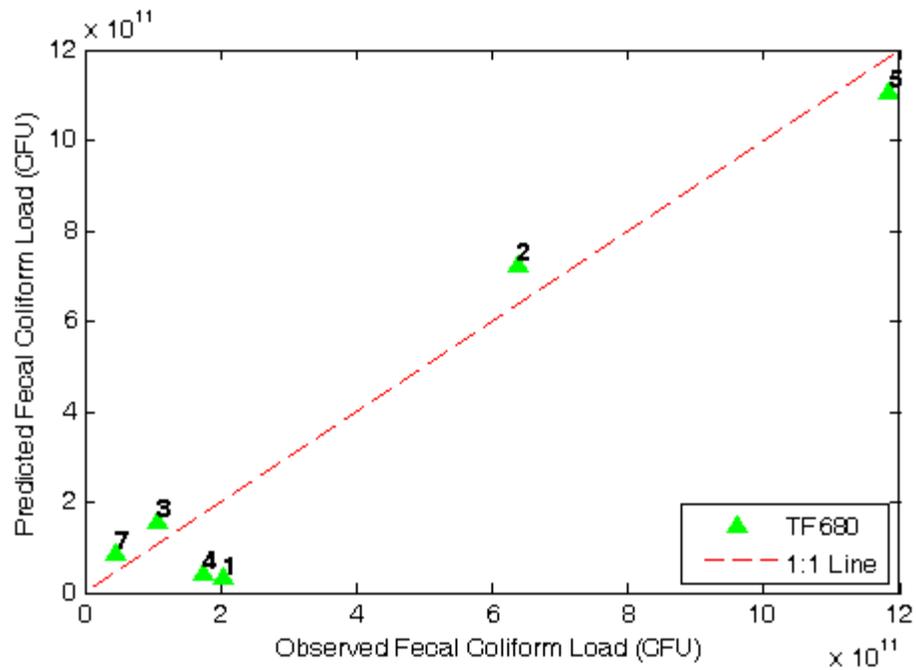
Event Number	Storm Date	Event Rainfall (in)	Peak Flow* (cfs)
1	May 06, 2003	0.1600	637
2	May 07, 2003	0.7130	3280
3	May 16, 2003	0.3100	59
4	July 10, 2003	0.1900	179
5	Sept 23, 2003	0.7090	1710
6	Oct 14, 2003	1.2780	3460
7	July 07, 2004	0.2000	198
8	Aug 30, 2004	0.4250	866

*Peak flow observed at Castor Ave. (Gage 1467087)

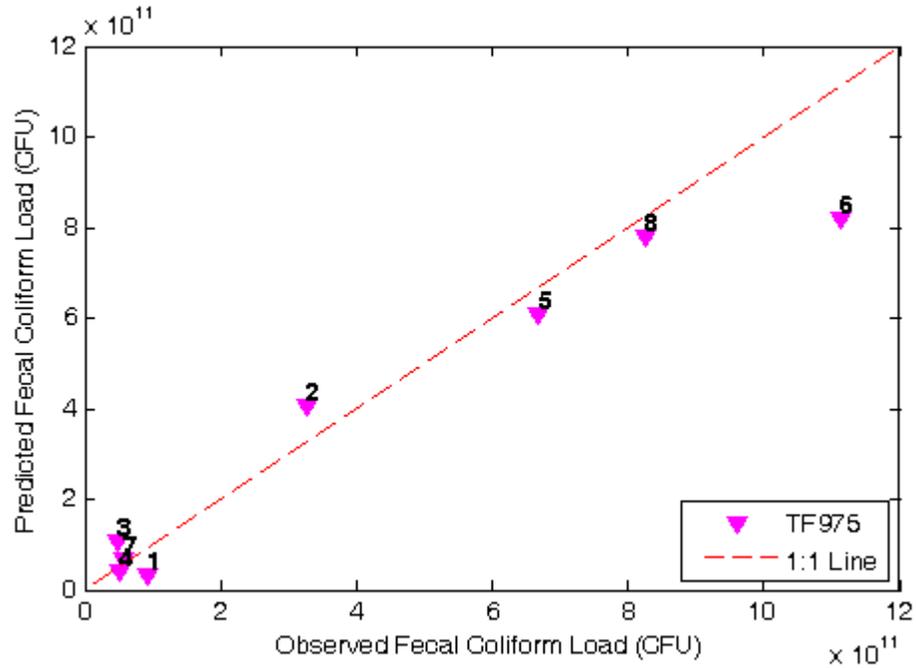
4.4.1. Fecal Coliform Load for 2003-2004 Storms - TF280



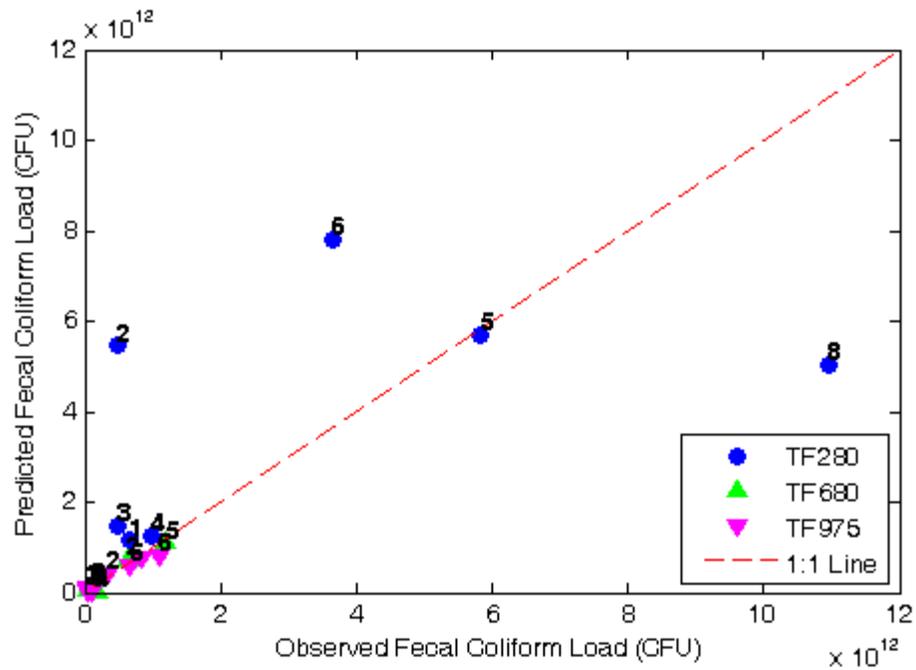
4.4.2. Fecal Coliform Load for 2003-2004 Storms - TF680



4.4.3. Fecal Coliform Load for 2003-2004 Storms - TF975



4.4.4. Fecal Coliform Load for 2003-2004 Storms - All Sites



4.5. Statistical Analysis - Observed Data

Table 4-2. Fecal Coliform - Observed Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10 th Percentiles	90 th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
05/06/2003	TF280	43252	44000	66435	7600	174200
05/06/2003	TF680	24701	34500	19817	7500	48000
05/06/2003	TF975	18337	27000	12475	4700	33000
05/07/2003	TF280	7397	6700	13721	1000	31000
05/07/2003	TF680	17106	19000	10042	6680	32400
05/07/2003	TF975	7486	24000	14976	1848	40800
05/16/2003	TF280	7796	5300	32157	668	72200
05/16/2003	TF680	7558	7750	4771	4000	15200
05/16/2003	TF975	5821	6350	1428	4080	7860
07/10/2003	TF280	75321	122000	62455	13580	181800
07/10/2003	TF680	19601	25000	28296	4010	77500
07/10/2003	TF975	9437	12500	9233	3070	26300
09/23/2003	TF280	45607	107000	69149	800	182000
09/23/2003	TF680	41454	44000	11723	28000	59000
09/23/2003	TF975	31805	33500	15624	14360	53400
10/14/2003	TF280	27380	25000	14270	19600	54600
10/14/2003	TF680	NaN	NaN	NaN	NaN	NaN
10/14/2003	TF975	20262	23000	11501	10200	40400
07/07/2004	TF280	NaN	NaN	NaN	NaN	NaN
07/07/2004	TF680	4520	4700	2811	2330	9690
07/07/2004	TF975	6596	7250	3800	3070	13040
08/30/2004	TF280	249144	600000	267676	112240	776000
08/30/2004	TF680	NaN	NaN	NaN	NaN	NaN
08/30/2004	TF975	89287	117500	149597	24400	379000

4.6. Statistical Analysis - Simulated Data

Table 4-3. Fecal Coliform - Simulated Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10 th Percentiles	90 th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
05/06/2003	TF280	78403	116003	84977	17775	228711
05/06/2003	TF680	4940	5666	3758	1737	11603
05/06/2003	TF975	7327	11631	6087	1608	16576
05/07/2003	TF280	49351	41640	44888	29145	139947
05/07/2003	TF680	23834	29388	9002	14331	35980
05/07/2003	TF975	21673	25296	9878	10798	36118
05/16/2003	TF280	33629	21393	82823	11037	214138
05/16/2003	TF680	11264	10119	7200	5888	24993
05/16/2003	TF975	11723	11193	7308	7176	27154
07/10/2003	TF280	53765	50478	113762	10124	294976
07/10/2003	TF680	7503	8595	3572	2386	13131
07/10/2003	TF975	11252	16482	6669	3025	20996
09/23/2003	TF280	56583	49467	34984	37599	126159
09/23/2003	TF680	33062	37230	13201	18317	53093
09/23/2003	TF975	32414	41868	12676	16069	49215
10/14/2003	TF280	65897	55834	39179	38003	114819
10/14/2003	TF680	21133	20832	7601	13406	34816
10/14/2003	TF975	16857	18001	5363	11408	25679
07/07/2004	TF280	39942	27258	112057	9746	292119
07/07/2004	TF680	7722	11709	5289	1237	15747
07/07/2004	TF975	9331	16915	7381	1849	20433
08/30/2004	TF280	152283	225837	80786	70333	280896
08/30/2004	TF680	43898	52822	65175	5715	169617
08/30/2004	TF975	30057	28075	52481	2778	148069

Appendix D

Cobbs Creek Bacteria Model Validation Simulations

Appendix D: Cobbs Creek Bacteria Model Validation Simulations:

This report compares observed bacteria data and WASP predicted outputs for the following four storms: Jul 26, 2000, Jul 23, 2003, Jul 24, 2003 and Sept 13, 2003.

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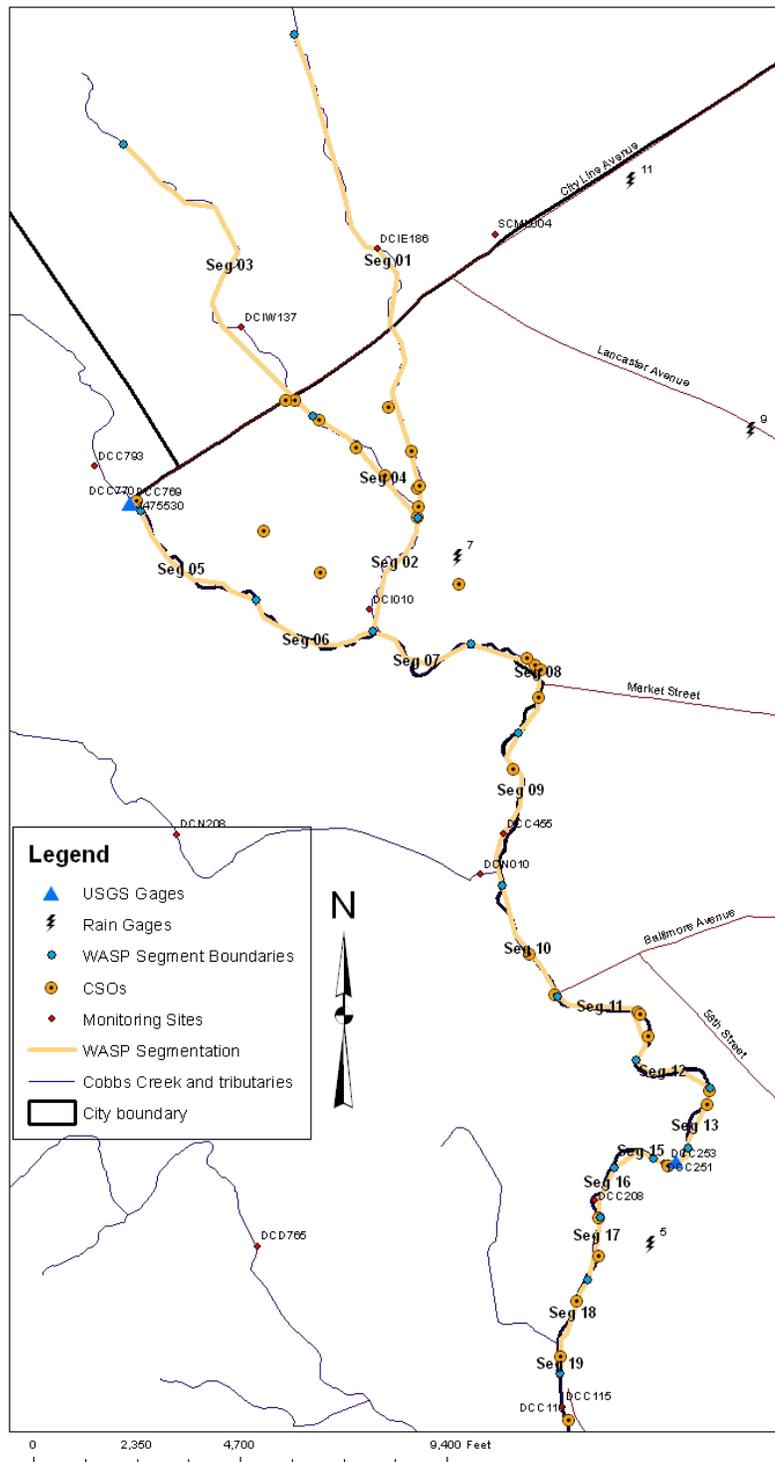
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Chapter 1. General Information

Table 1-1. Cobbs Monitoring Locations

Site ID	Loc ID	WASP Segment
DCC110	COBB175	Segment 19
DCC208	COBB350	Segment 16
DCC455	COBB550	Segment 09

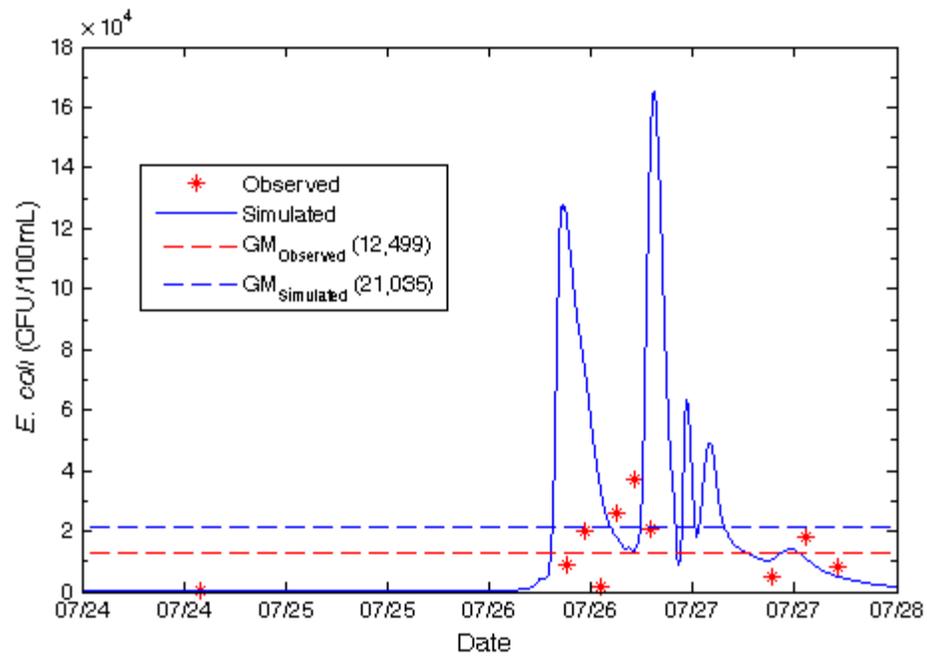
1.1. Segmentation Map



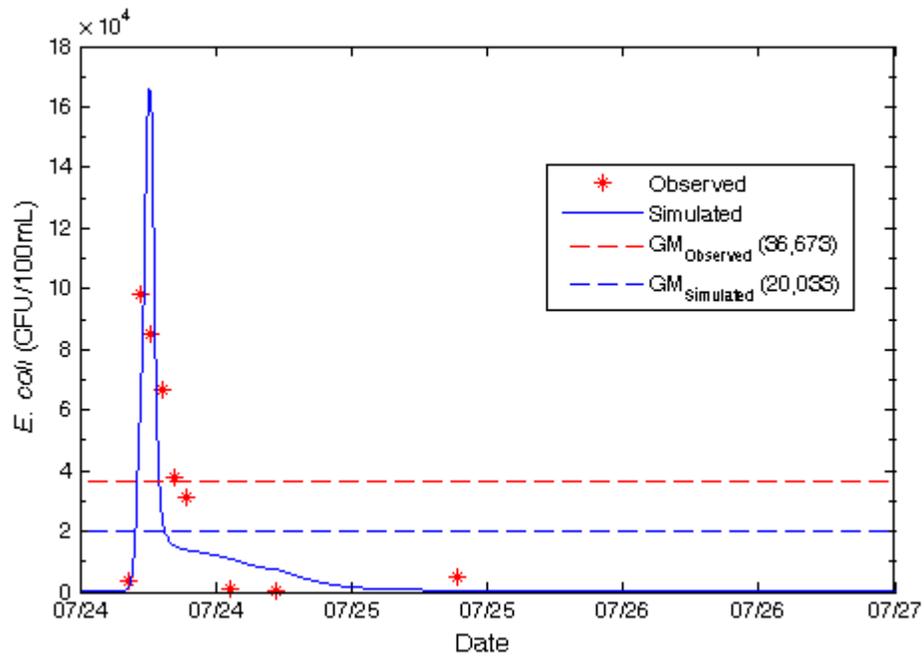
Chapter 2. *E. coli*

2.1. *E. coli* Time Series

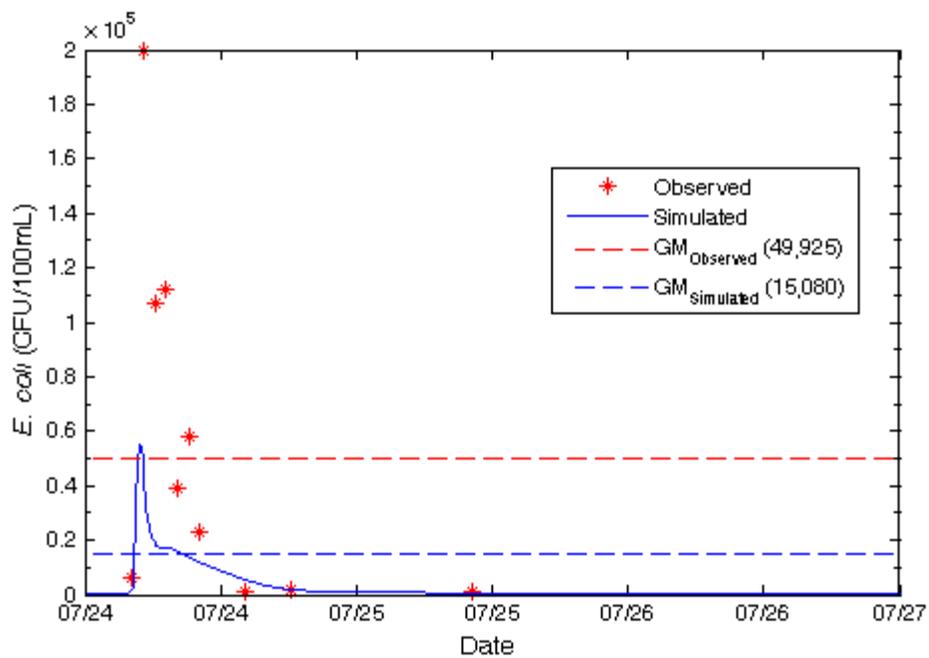
2.1.1. July 26, 2000 Storm - DCC110



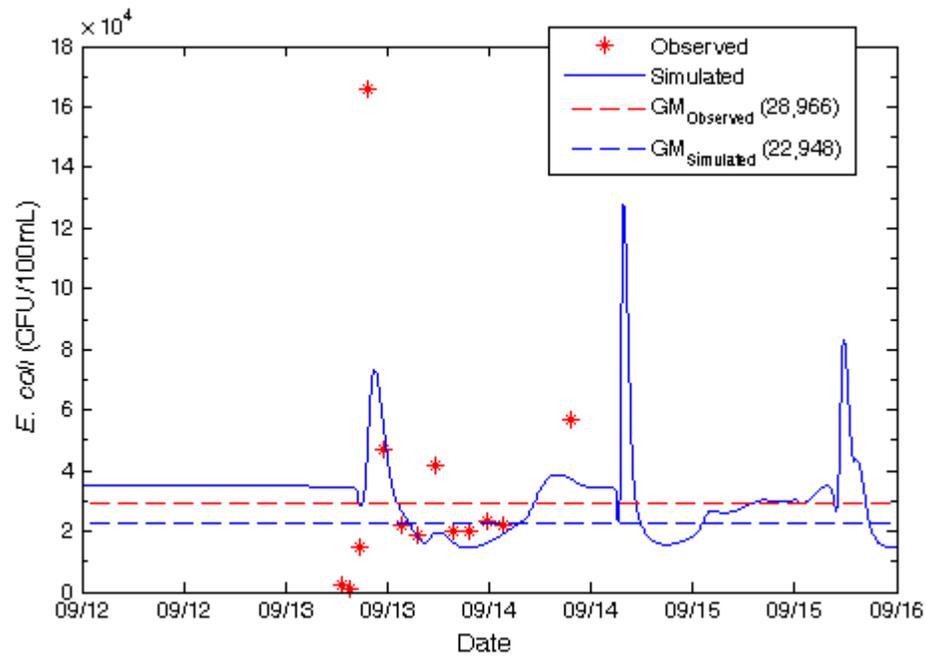
2.1.4. July 24, 2003 Storm - DCC208



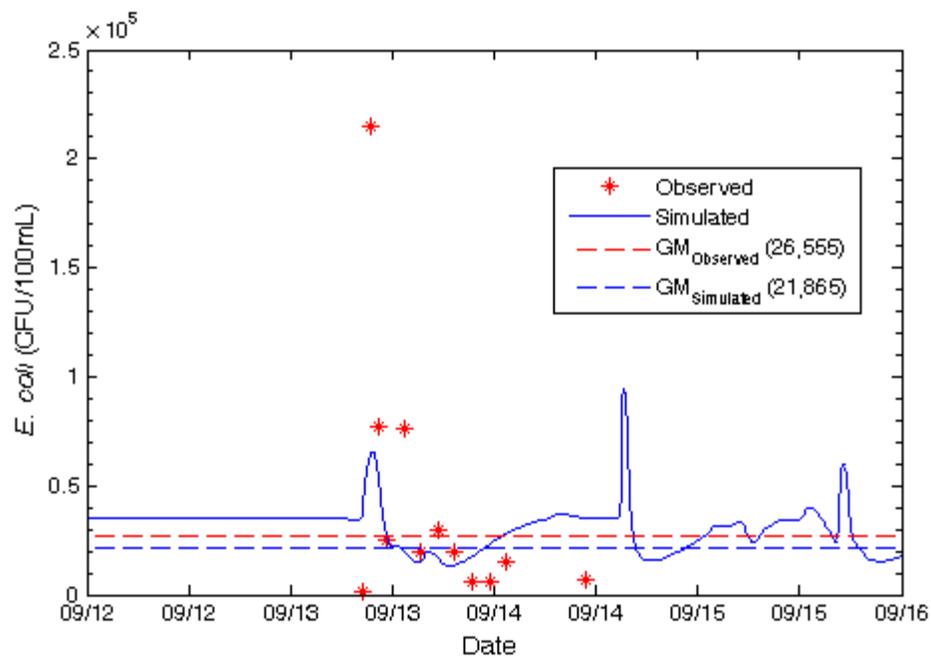
2.1.5. July 24, 2003 Storm - DCC455



2.1.6. Sept 13, 2003 Storm - DCC208

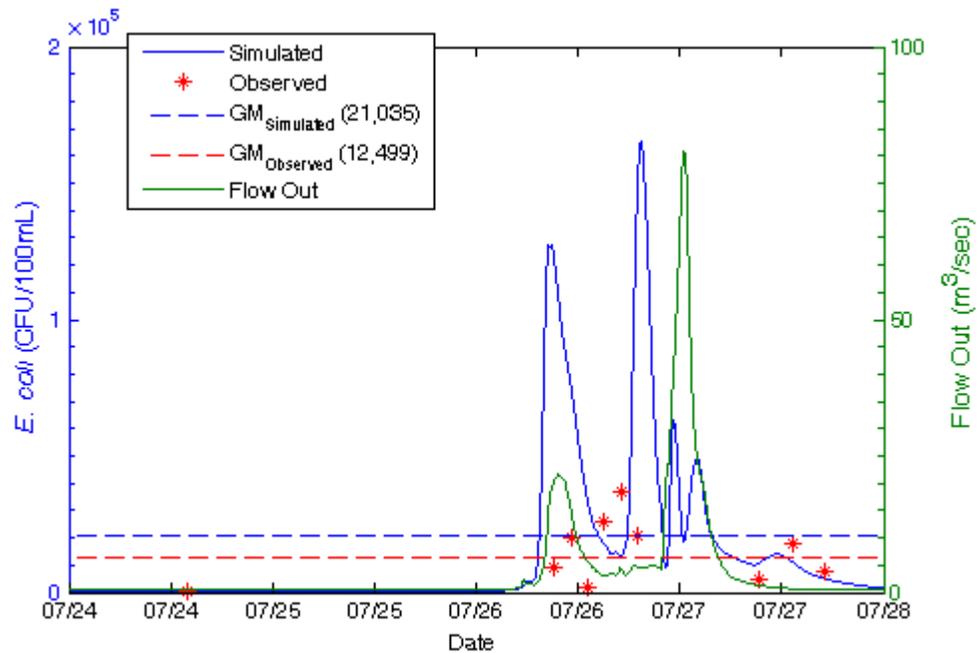


2.1.7. Sept 13, 2003 Storm - DCC455

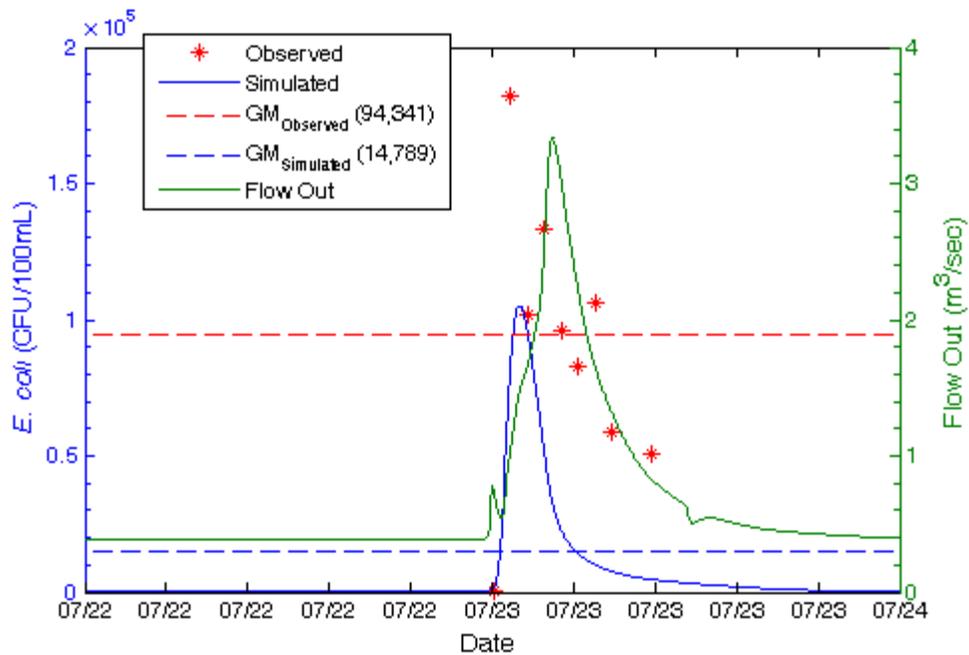


2.2. *E. coli* and Flow Out Time Series

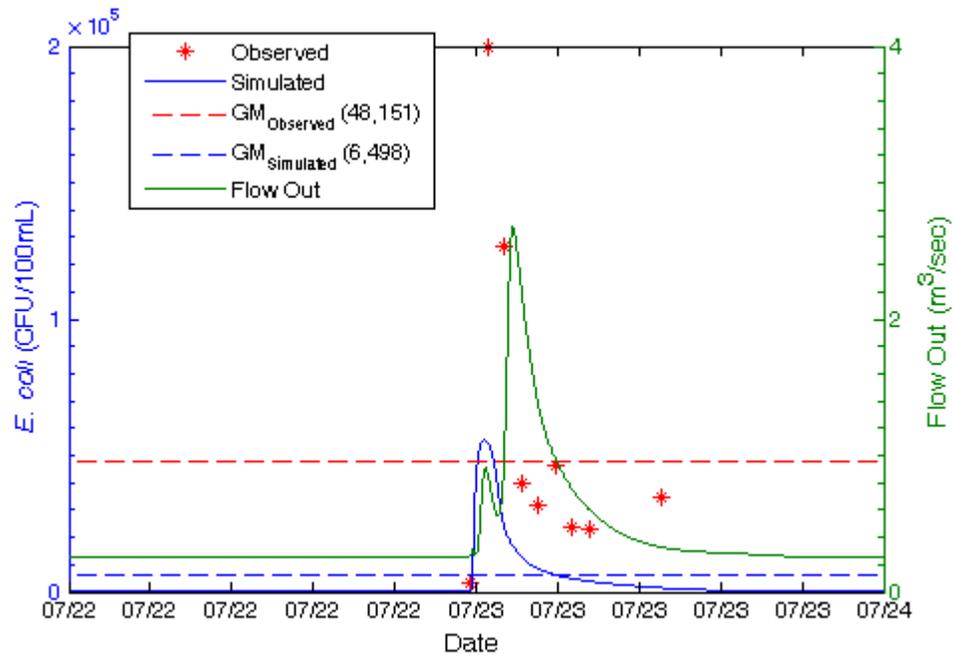
2.2.1. July 26, 2000 Storm - DCC110



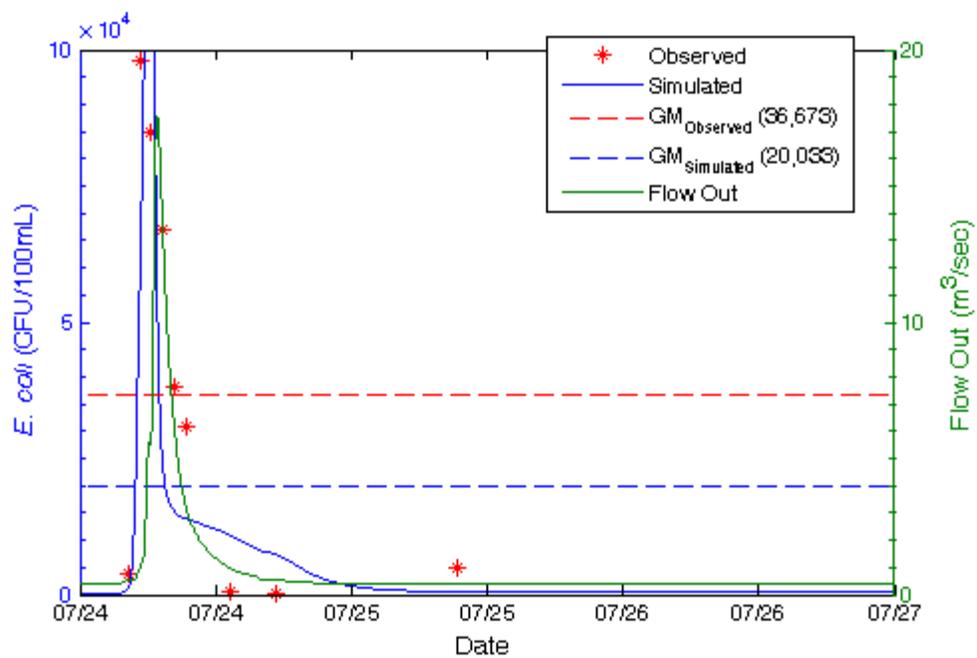
2.2.2. July 23, 2003 Storm - DCC208



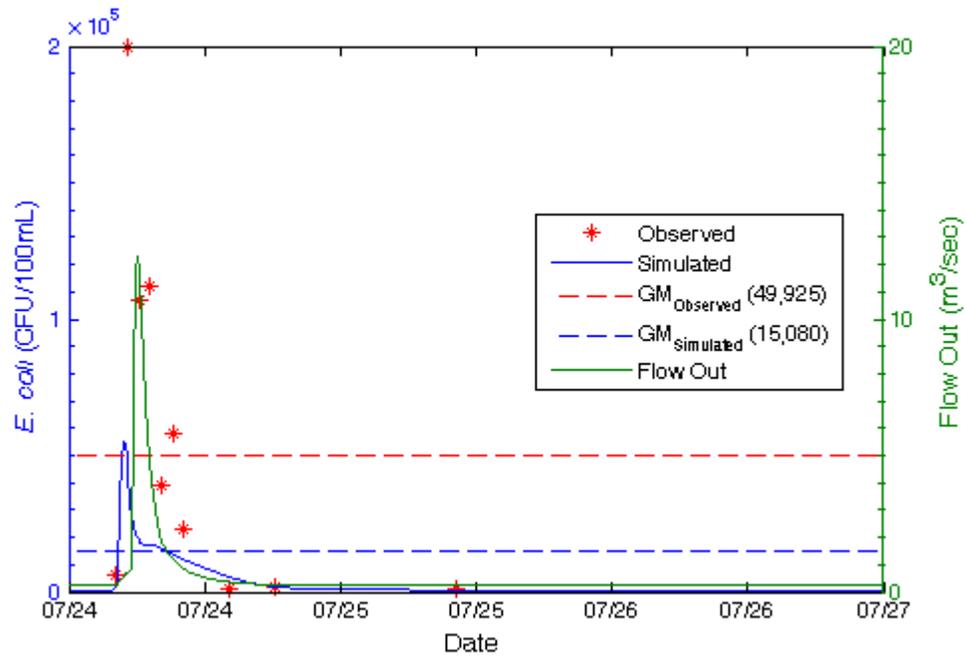
2.2.3. July 23, 2003 Storm - DCC455



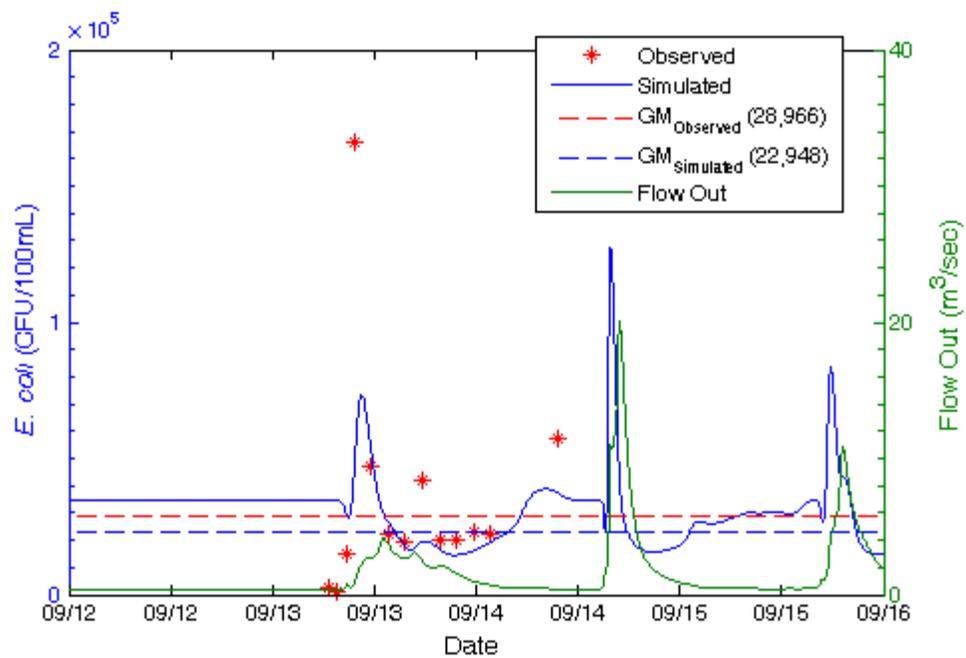
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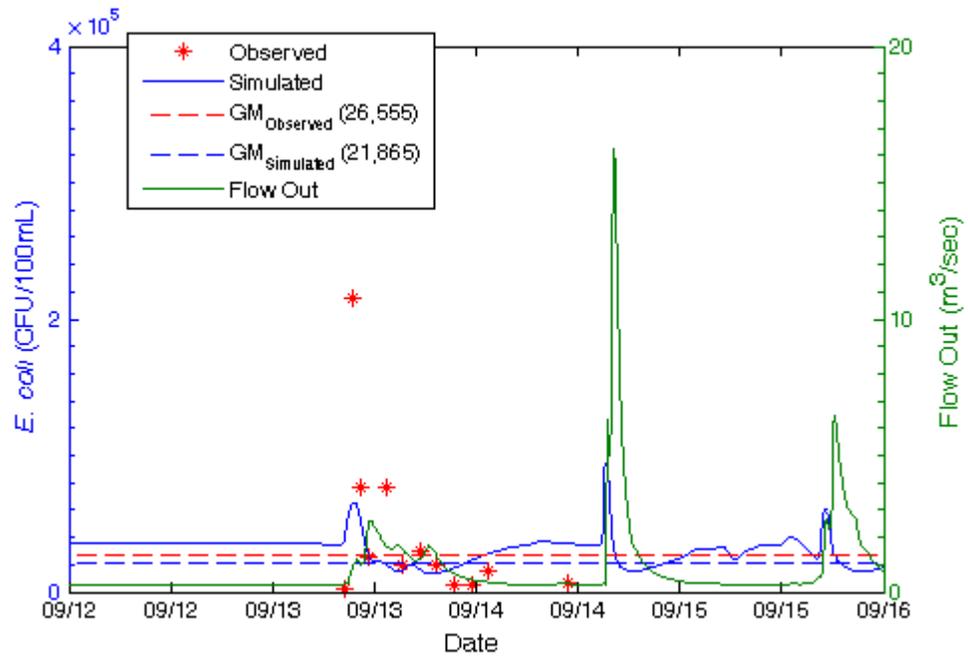
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2.2.6. Sept 13, 2003 Storm - DCC208

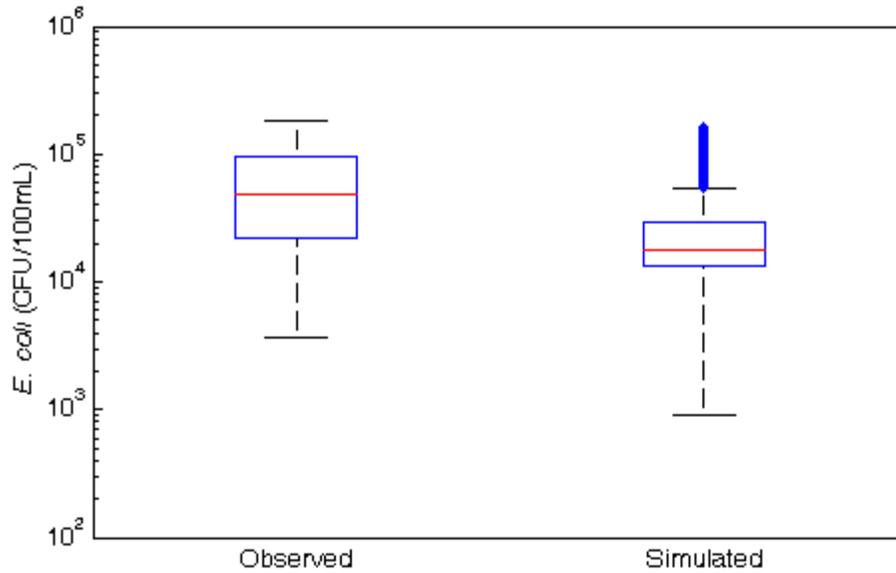


2.2.7. Sept 13, 2003 Storm - DCC455

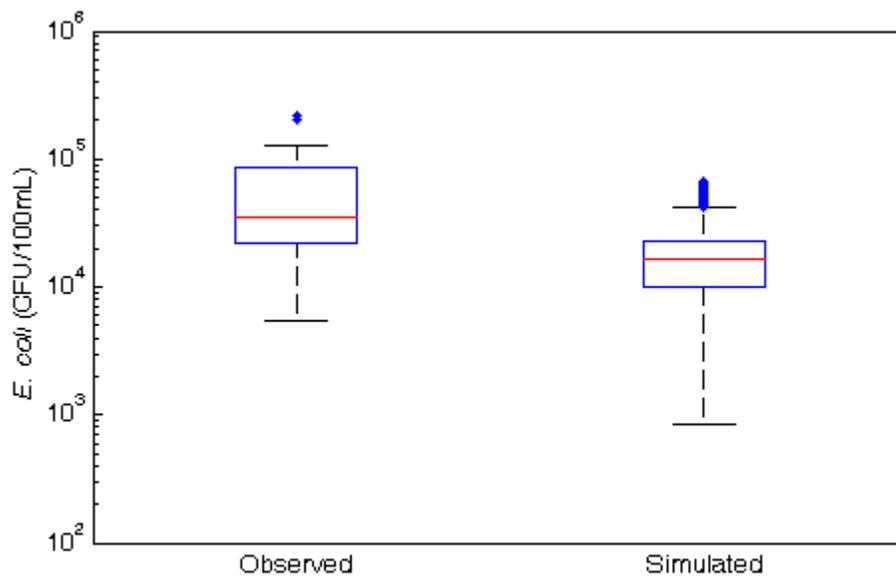


2.3. Predicted and Observed *E. coli* Concentration Box Plots

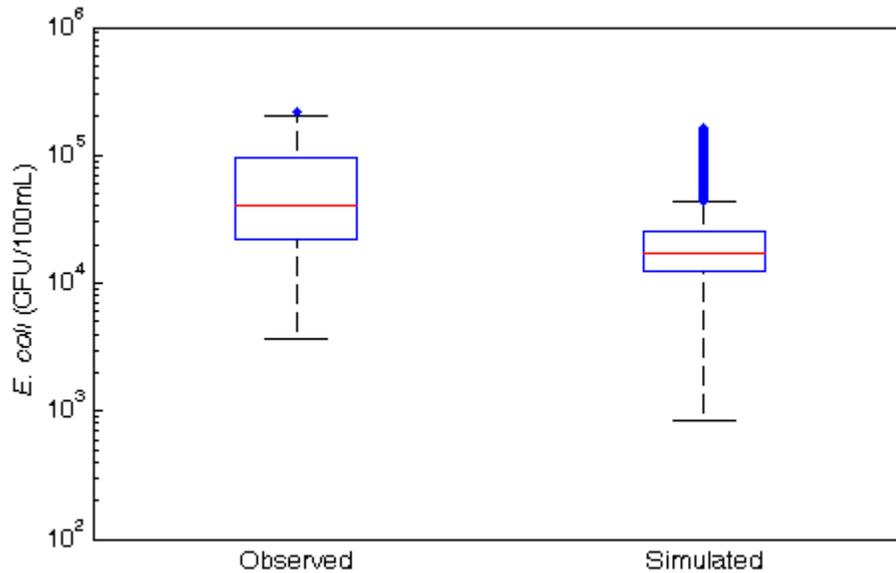
2.3.1. *E. coli* Concentration for 2003-2004 Storms - DCC208



2.3.2. *E. coli* Concentration for 2003-2004 Storms - DCC455



2.3.3. *E. coli* Concentration for 2003-2004 Storms - All Sites



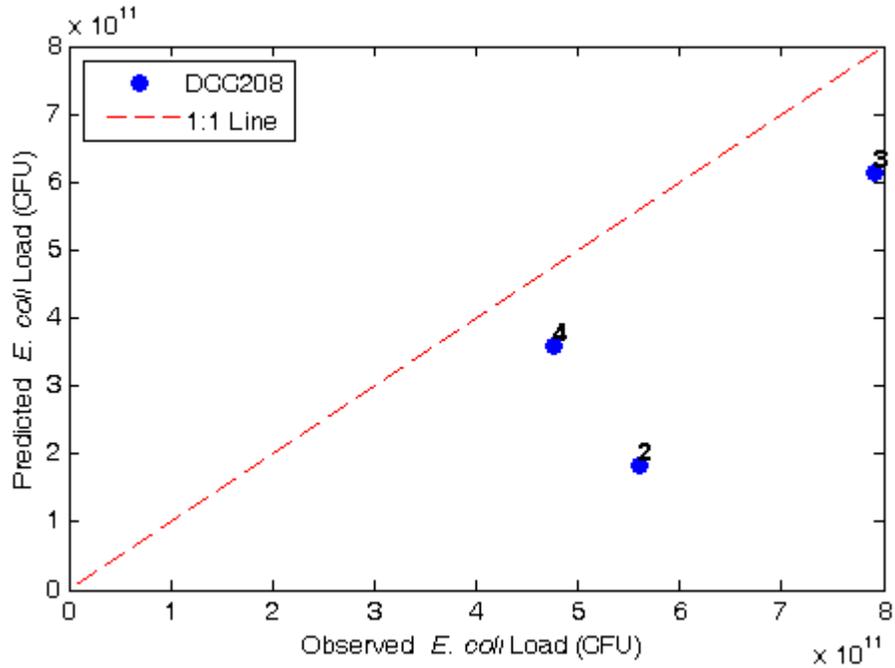
2.4. Predicted vs. Observed *E. coli* Load Scatter Plots

Table 2-1. Total Rainfall and Peak Flow for Four Storms

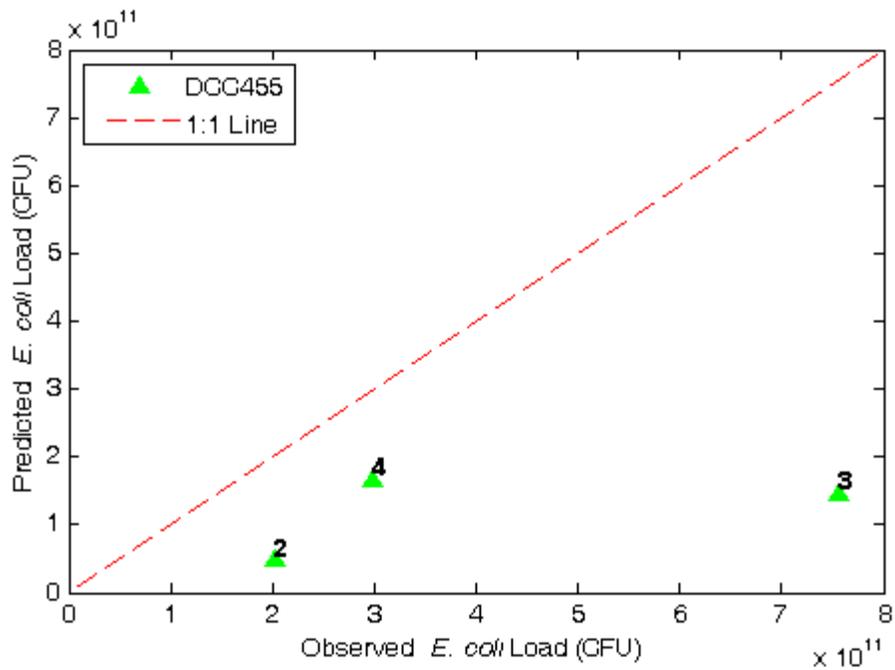
Event Number	Storm Date	Event Rainfall (in)	Peak Flow* (cfs)
1	July 26, 2000	2.6820	2600
2	July 23, 2003	0.2800	720
3	July 24, 2003	0.4610	100
4	Sept 13, 2003	0.5500	140

*Peak flow predicted at Mt. Moriah gage 1475548

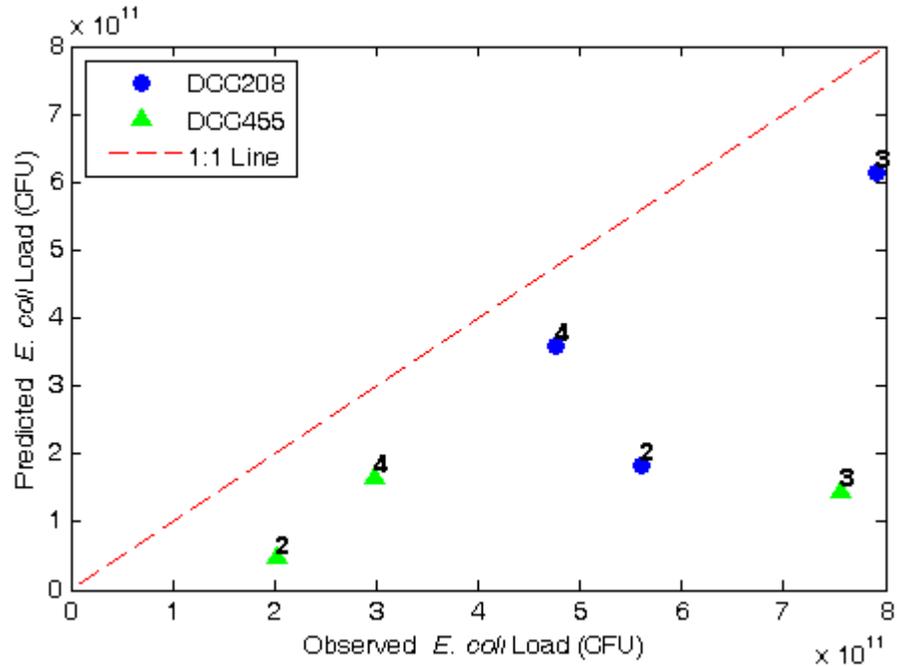
2.4.1. *E. coli* Load for 2003-2004 Storms - DCC208



2.4.2. *E. coli* Load for 2003-2004 Storms - DCC455



2.4.3. *E. coli* Load for 2003-2004 Storms - All Sites



2.5. Statistical Analysis

Table 2-2. *E. coli* - Observed Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10 th Percentiles	90 th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
07/26/2000	DCC110	12499	19000	11613	2760	33700
07/23/2003	DCC208	94341	99000	41785	53400	167300
07/23/2003	DCC455	48151	37500	63789	23300	178100
07/24/2003	DCC208	36673	52500	35723	6430	96700
07/24/2003	DCC455	49925	58000	67060	9560	182400
09/13/2003	DCC208	28966	22000	45620	17000	106500
09/13/2003	DCC455	26555	22500	63788	5550	146000

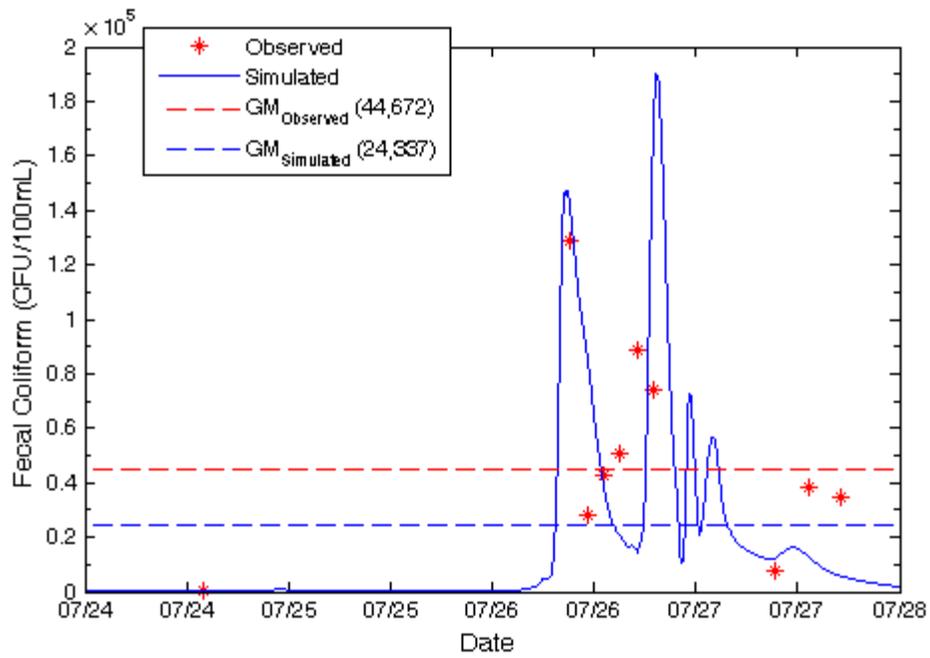
Table 2-3. *E. coli* - Predicted Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10th Percentiles	90th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
07/26/2000	DCC110	21035	18619	39649	4252	106171
07/23/2003	DCC208	14789	10720	32700	4317	91481
07/23/2003	DCC455	6498	5098	15503	1924	44506
07/24/2003	DCC208	20033	14194	42237	11568	105851
07/24/2003	DCC455	15080	15128	11455	9045	33575
09/13/2003	DCC208	22948	19080	15588	14906	53940
09/13/2003	DCC455	21865	19626	13367	14140	50085

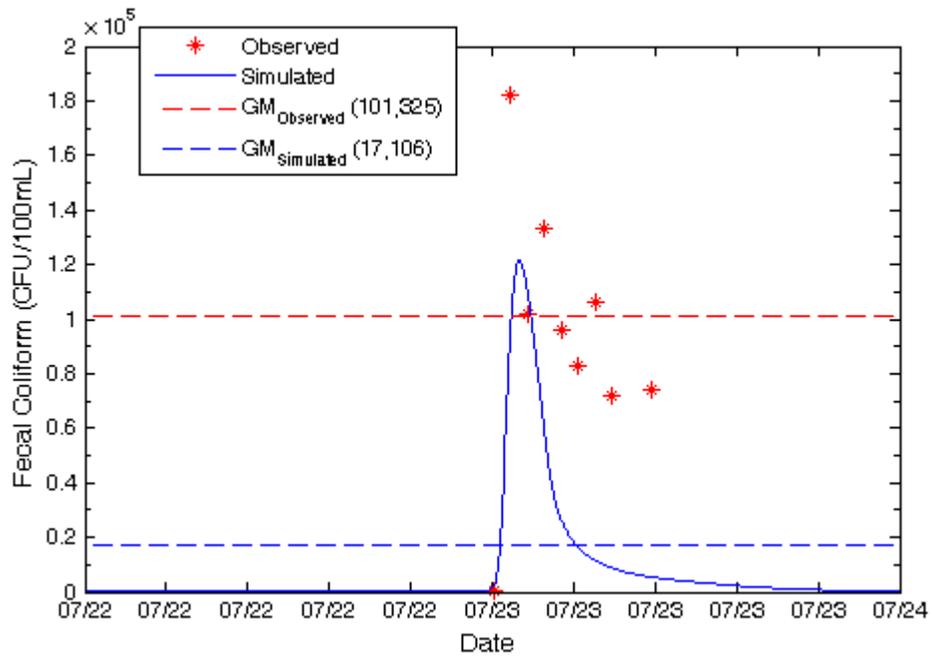
Chapter 3. Fecal Coliform

3.1. Fecal Coliform Time Series

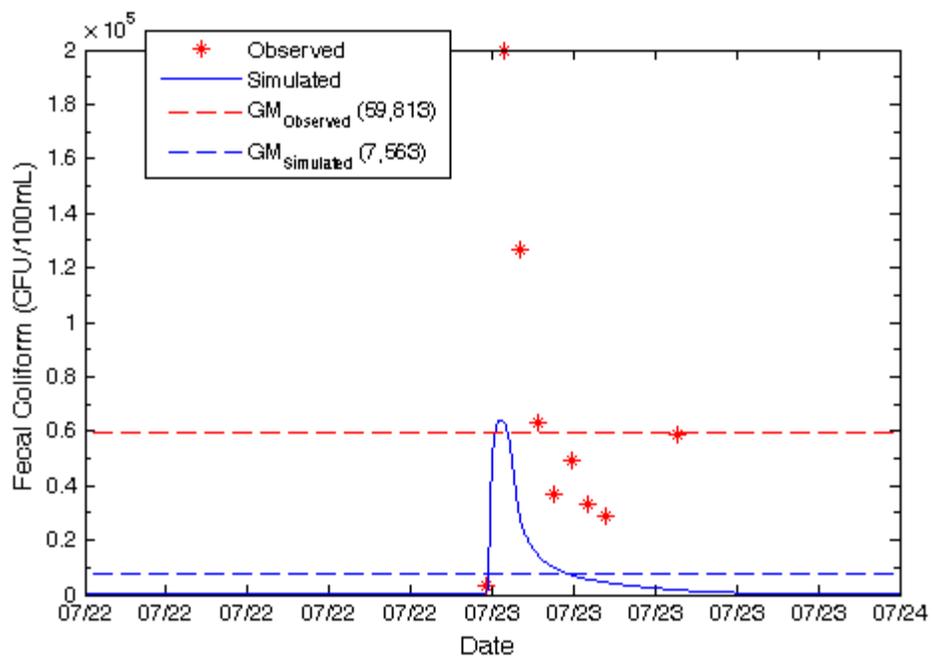
3.1.1. July 26, 2000 Storm - DCC110



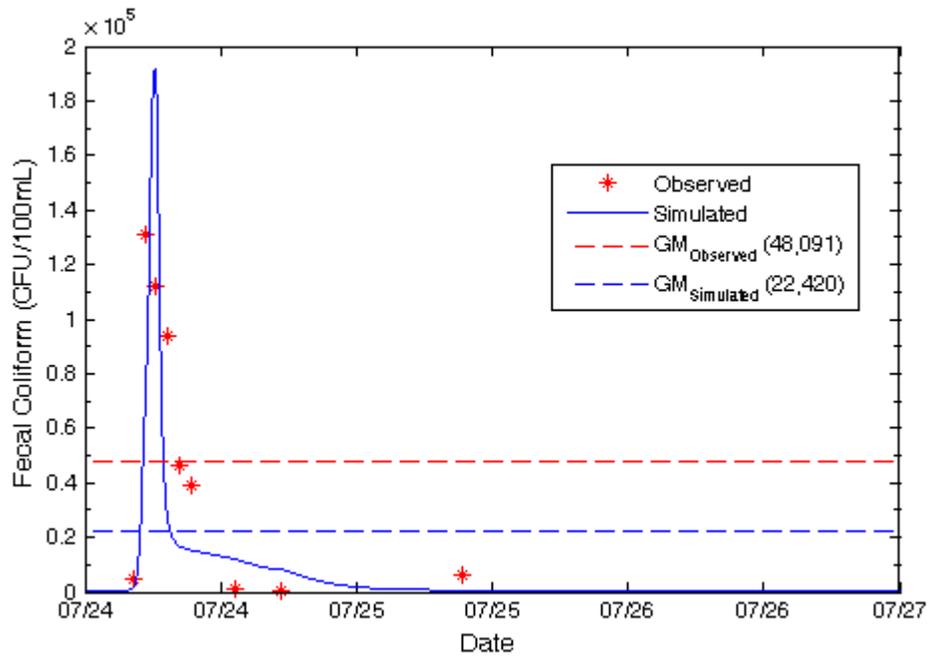
3.1.2. July 23, 2003 Storm - DCC208



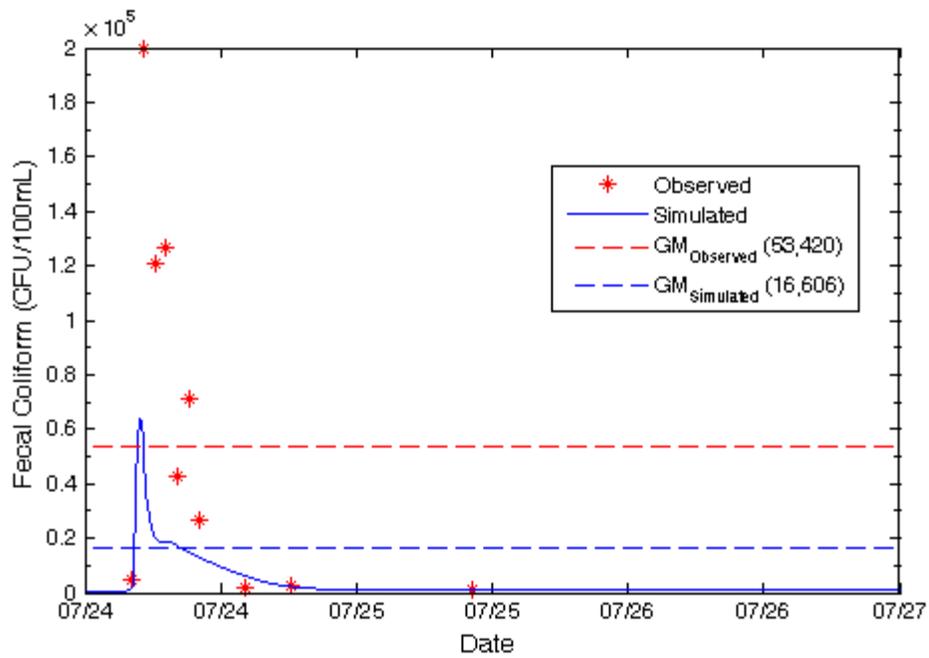
3.1.3. July 23, 2003 Storm - DCC455



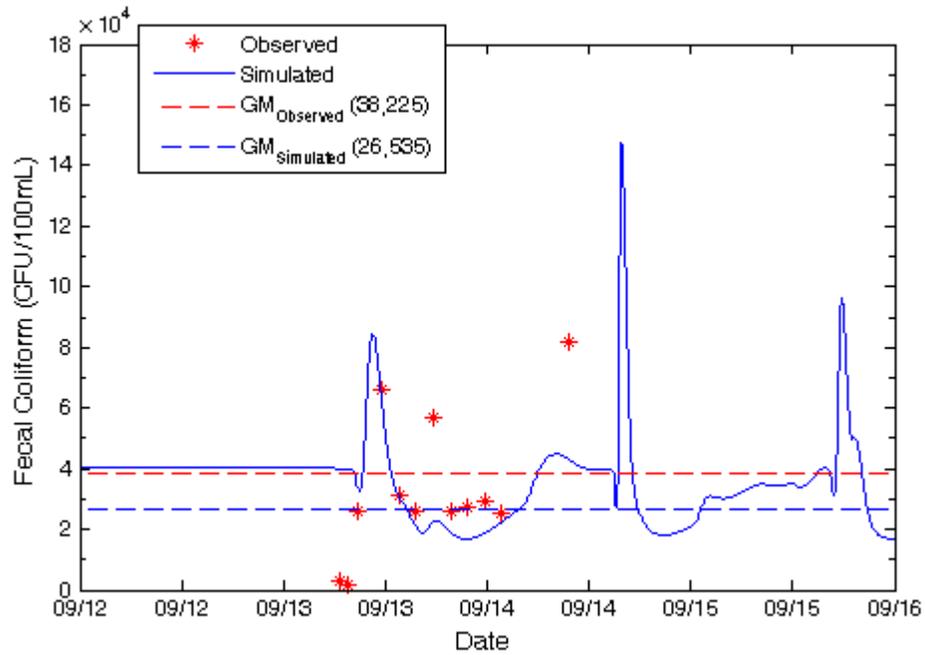
3.1.4. July 24, 2003 Storm - DCC208



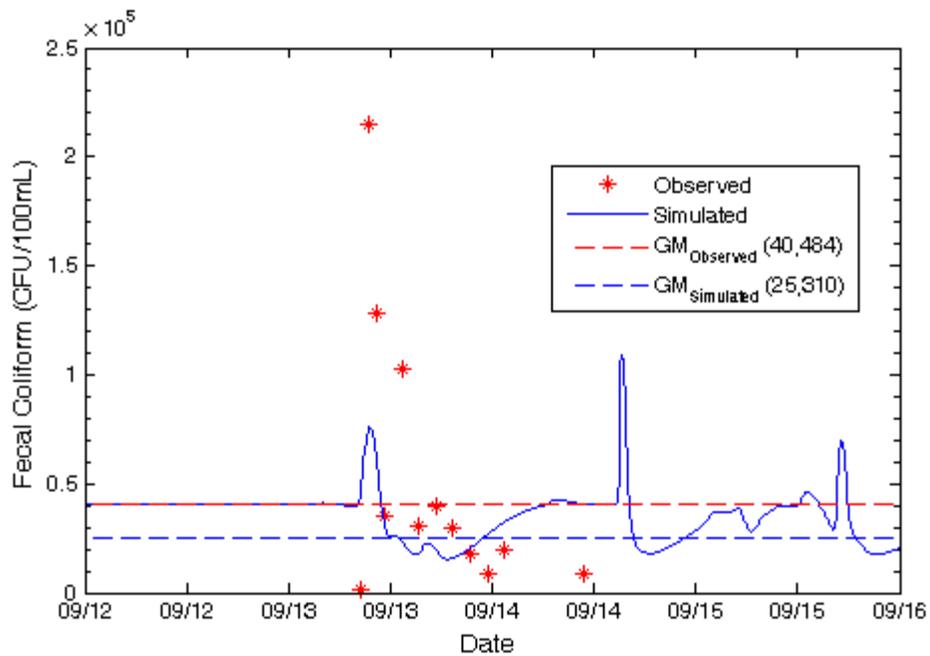
3.1.5. July 24, 2003 Storm - DCC455



3.1.6. Sept 13, 2003 Storm - DCC208

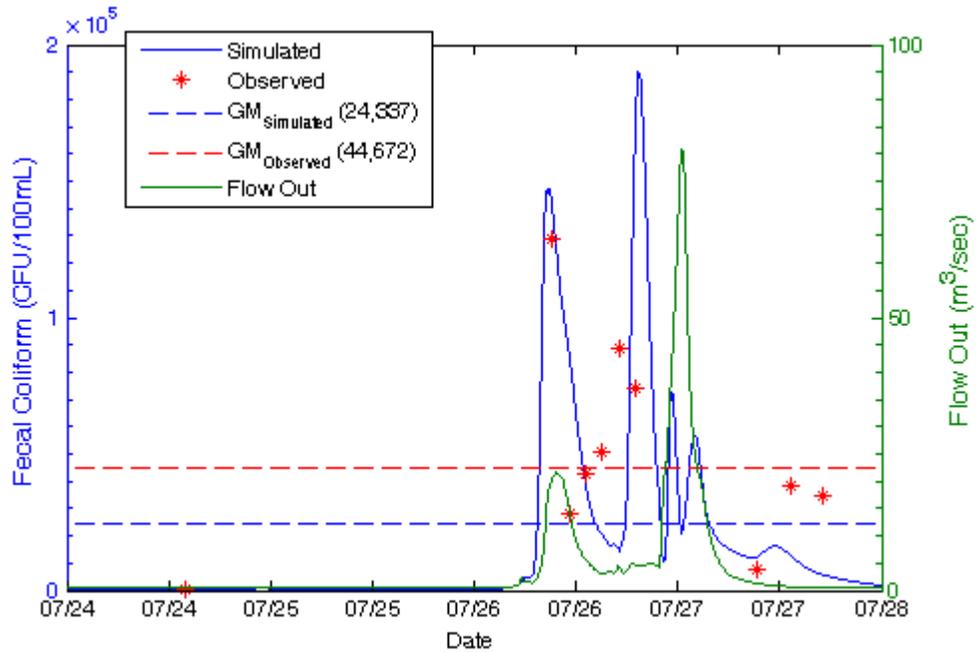


3.1.7. Sept 13, 2003 Storm - DCC455

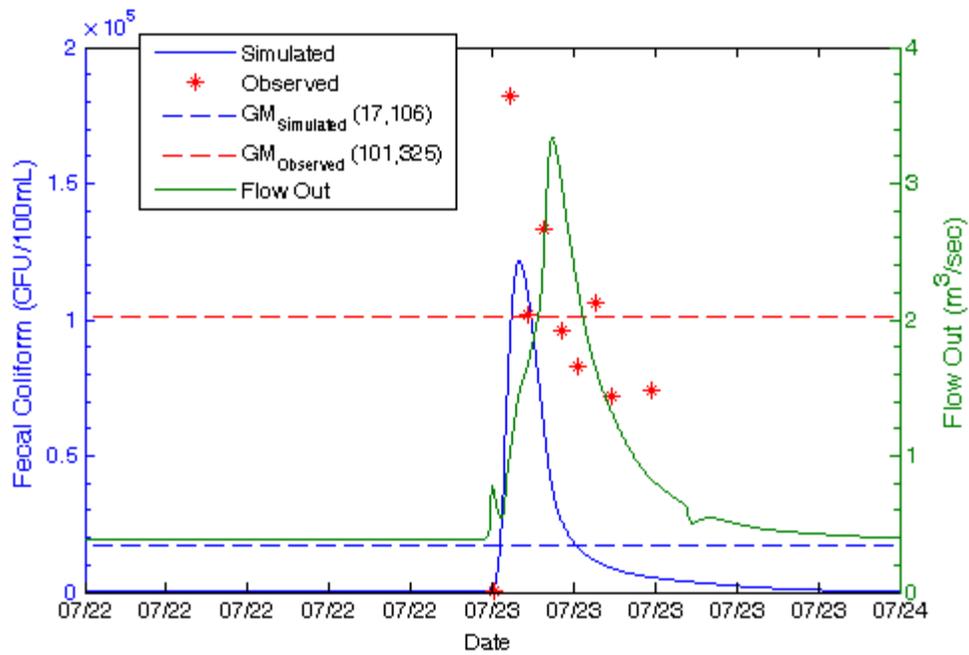


3.2. Fecal Coliform and Flow Out Time Series

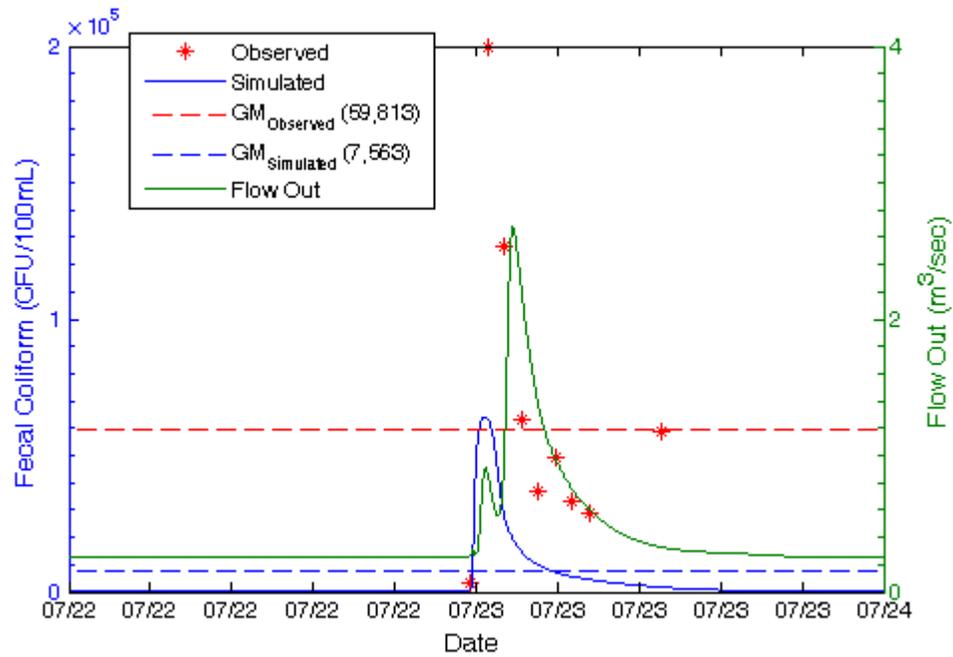
3.2.1. July 26, 2000 Storm - DCC110



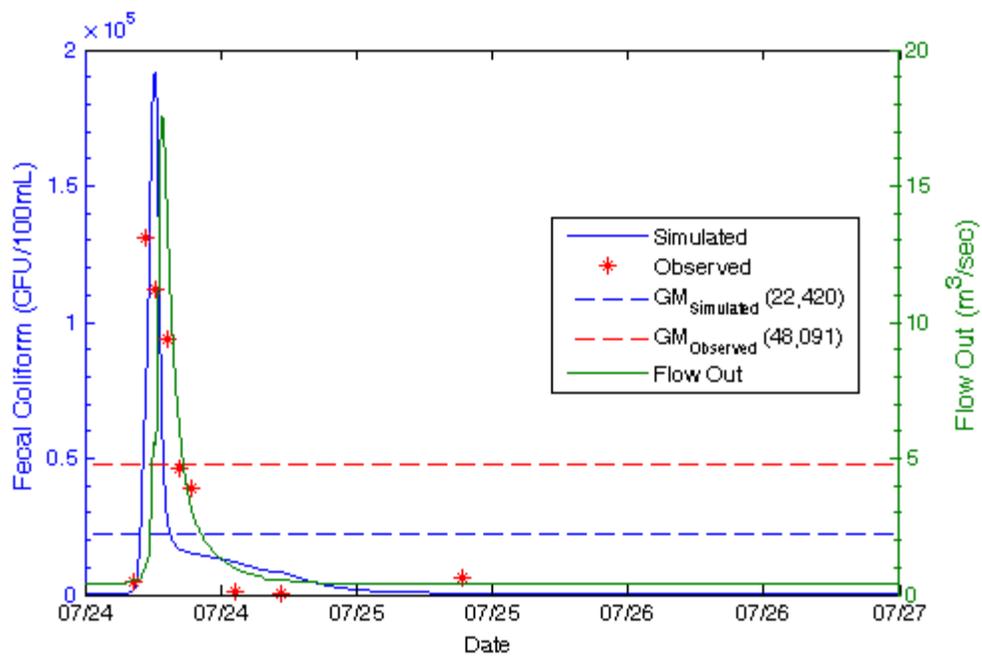
3.2.2. July 23, 2003 Storm - DCC208



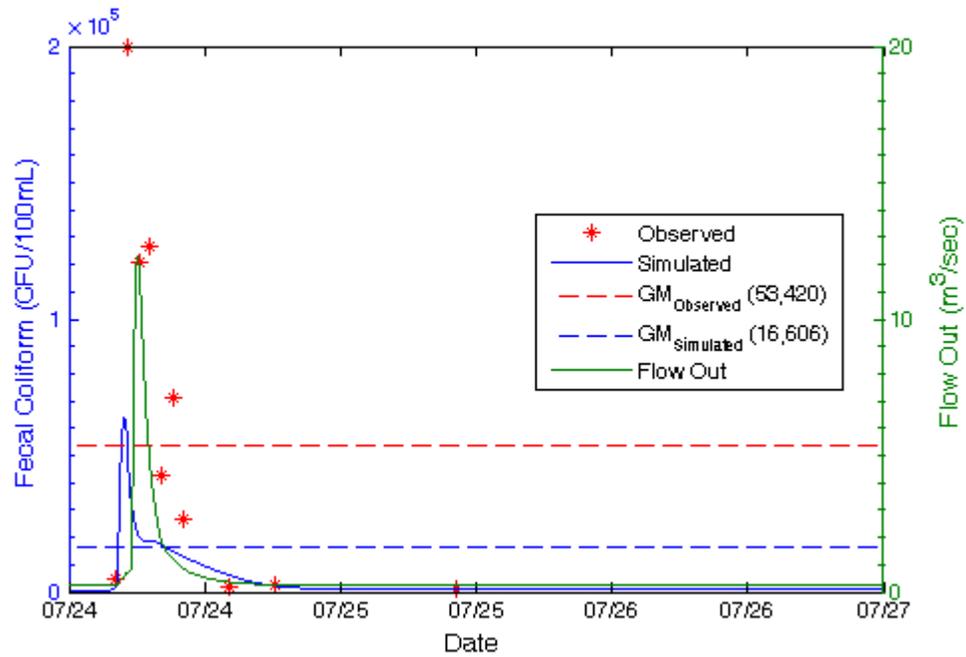
3.2.3. July 23, 2003 Storm - DCC455



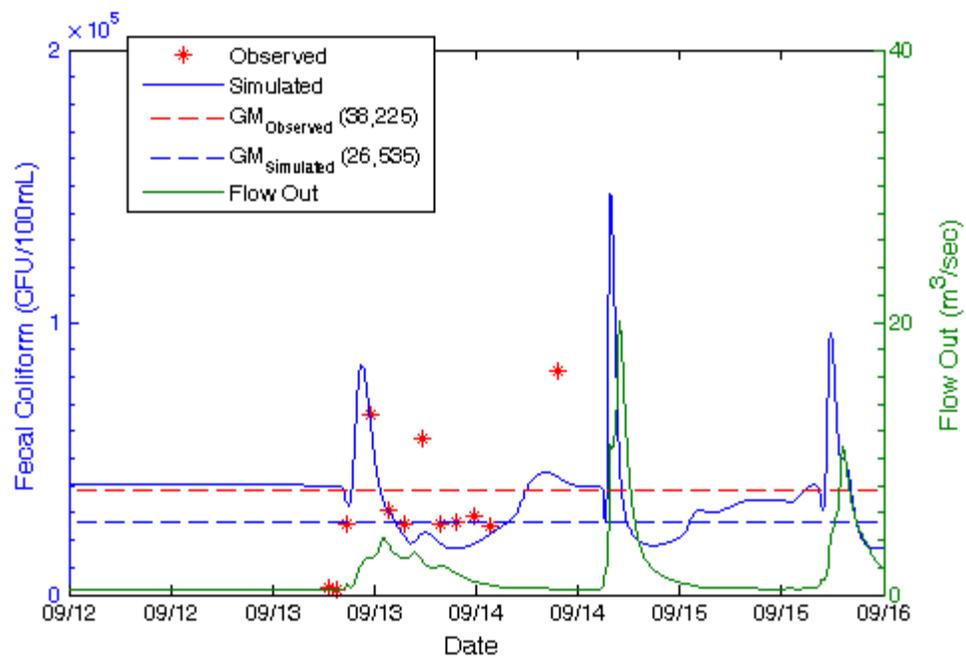
3.2.4. July 24, 2003 Storm - DCC208



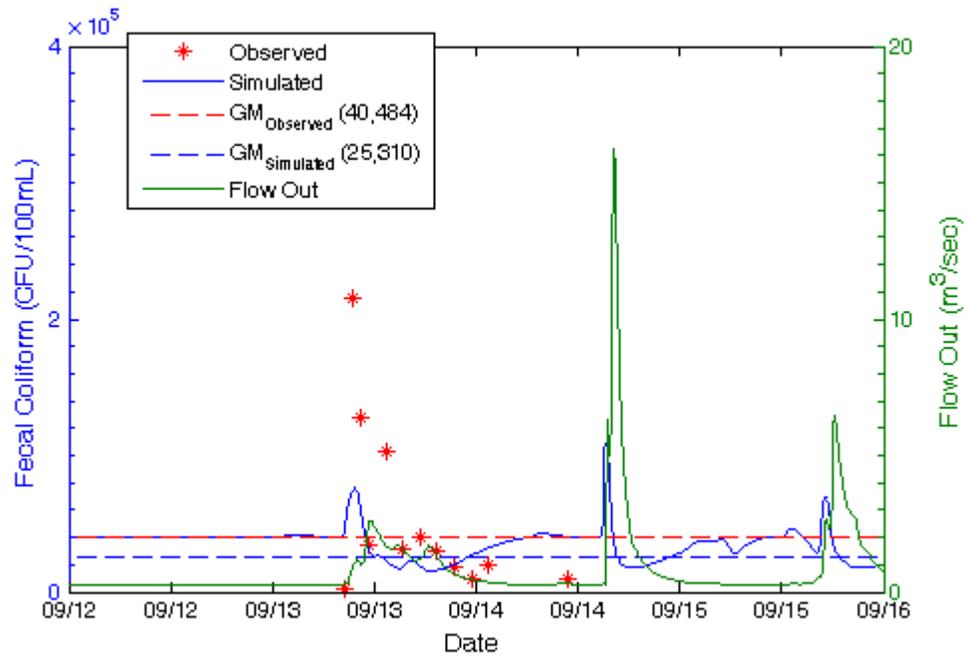
3.2.5. July 24, 2003 Storm - DCC455



3.2.6. Sept 13, 2003 Storm - DCC208

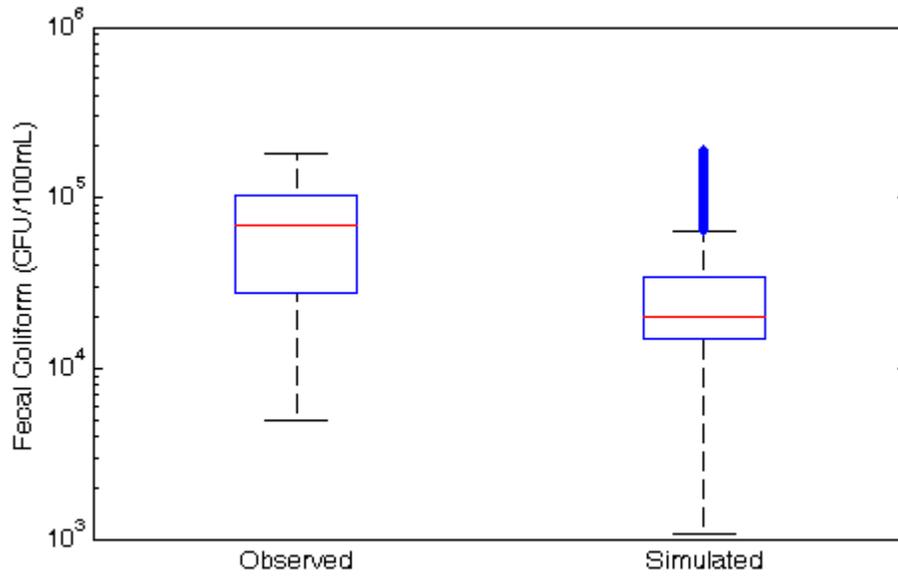


3.2.7. Sept 13, 2003 Storm - DCC455

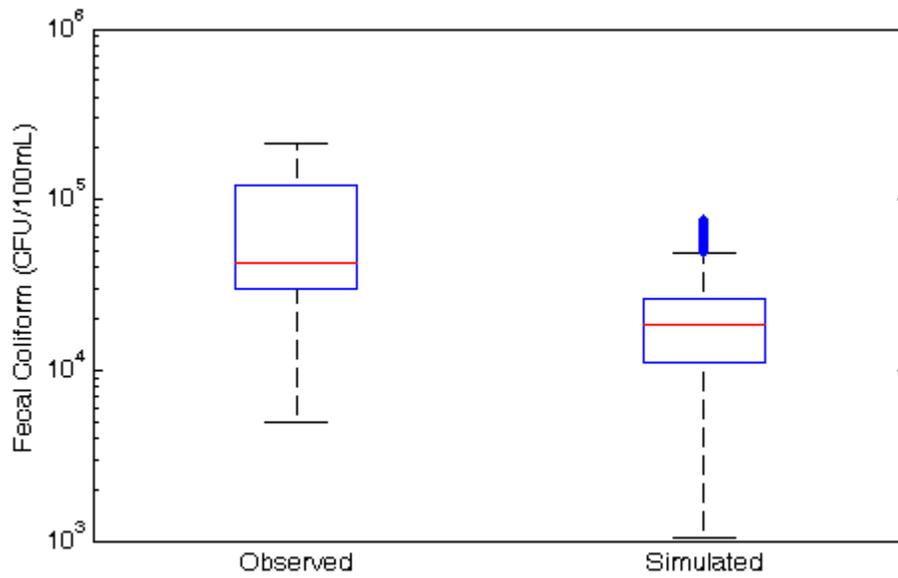


3.3. Predicted and Observed Fecal Coliform Concentration Box Plots

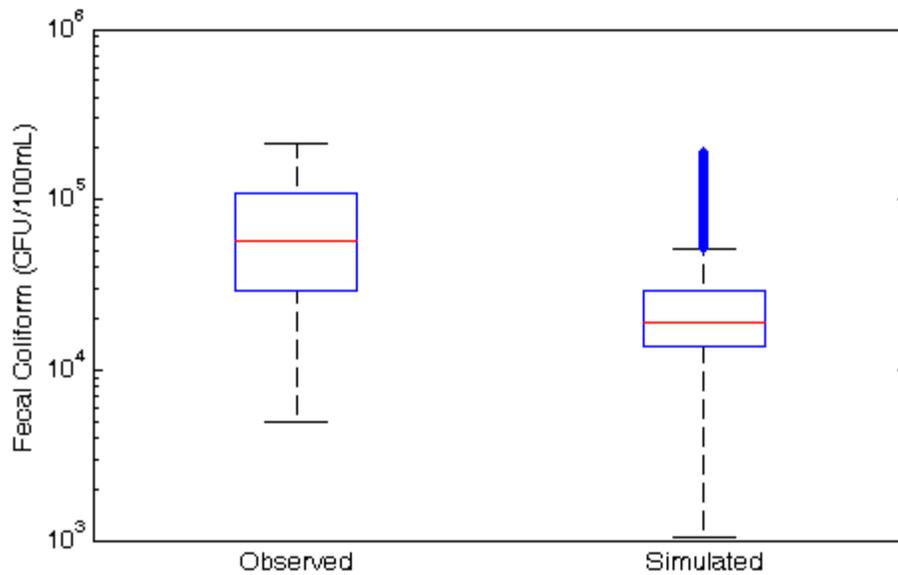
3.3.1. Fecal Coliform Concentration for 2003-2004 Storms - DCC208



3.3.2. Fecal Coliform Concentration for 2003-2004 Storms - DCC455



3.3.3. Fecal Coliform Concentration for 2003-2004 Storms - All Sites



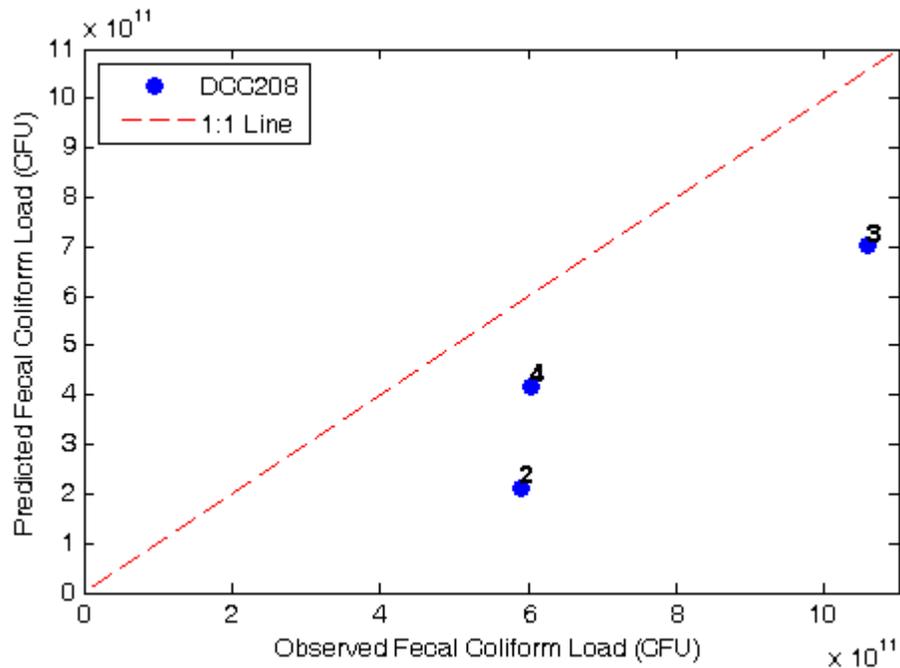
3.4. Predicted vs. Observed Fecal Coliform Load Scatter Plots

Table 3-1. Total Rainfall and Peak Flow for Four Storms

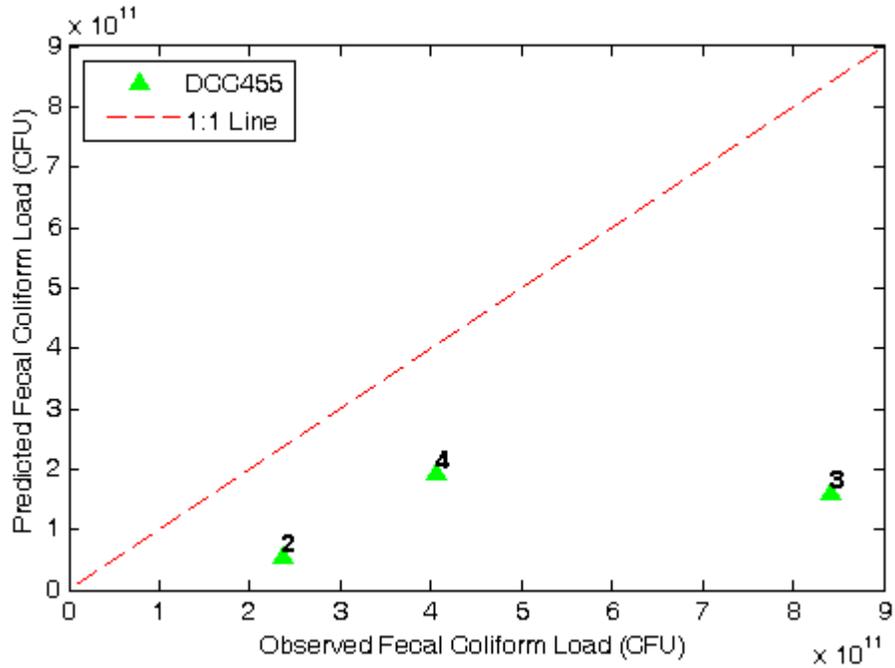
Event Number	Storm Date	Event Rainfall (in)	Peak Flow* (cfs)
1	July 26, 2000	2.6820	2600
2	July 23, 2003	0.2800	720
3	July 24, 2003	0.4610	100
4	Sept 13, 2003	0.5500	140

*Peak flow predicted at Mt. Moriah gage 1475548

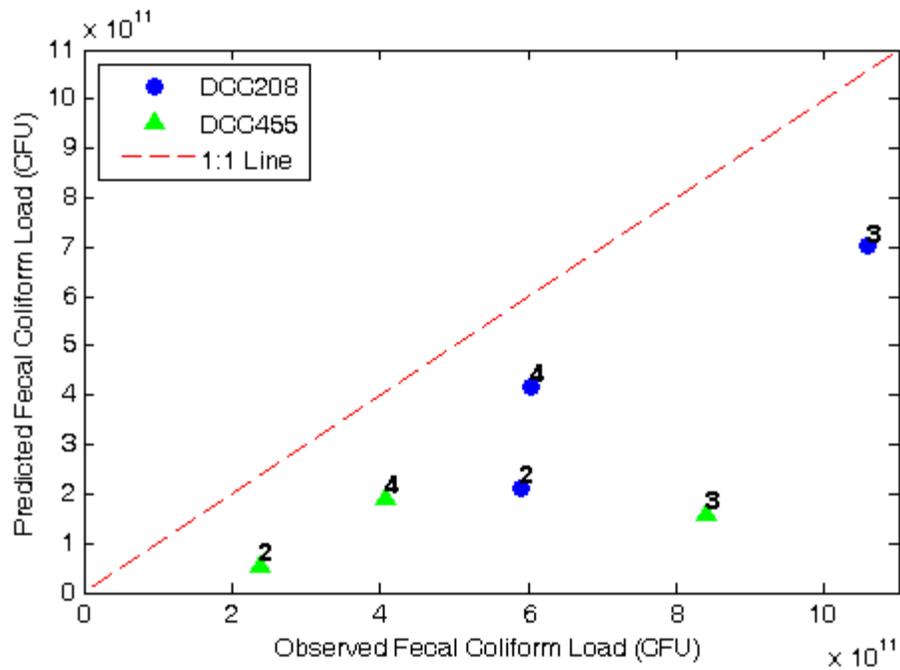
3.4.1. Fecal Coliform Load for 2003-2004 Storms - DCC208



3.4.2. Fecal Coliform Load for 2003-2004 Storms - DCC455



3.4.3. Fecal Coliform Load for 2003-2004 Storms - All Sites



3.5. Statistical Analysis

Table 3-2. Fecal Coliform - Observed Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10th Percentiles	90th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
07/26/2000	DCC110	44672	47000	38415	14000	117000
07/23/2003	DCC208	101325	99000	36508	72600	167300
07/23/2003	DCC455	59813	54000	59435	30200	178100
07/24/2003	DCC208	48091	70000	48610	8400	129100
07/24/2003	DCC455	53420	71000	68343	9320	185400
09/13/2003	DCC208	38225	28000	43971	25500	116000
09/13/2003	DCC455	40484	33000	65959	13450	171500

Table 3-3. Fecal Coliform - Predicted Data Statistical Analysis

Storm Date	Site	Geometric Mean	Median	Standard Deviation	10th Percentiles	90th Percentiles
		CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL	CFU/100mL
07/26/2000	DCC110	24337	21499	45748	4928	122522
07/23/2003	DCC208	17106	12390	37728	5011	105577
07/23/2003	DCC455	7563	5915	17881	2270	51375
07/24/2003	DCC208	22420	15617	48937	12689	122015
07/24/2003	DCC455	16606	16392	13406	9869	38541
09/13/2003	DCC208	26535	22070	17977	17256	62270
09/13/2003	DCC455	25310	22725	15412	16394	57839