CITY OF BEVERLY HILLS
SANITARY SEWER COLLECTION SYSTEM MASTER PLAN

City of Beverly Hills

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Executive Summary

The City of Beverly Hills sewer system is an aging system. Approximately 50% of the system is 80 years old and is considered to have reached the end of its useful life. Approximately 27% of the pipe material in the system is concrete pipe. Concrete pipe is no longer recommended for installation in gravity sewer systems due to corrosion of the concrete from the hydrogen sulfide gases. Of the approximately 77,000 lineal feet of sewer pipeline inspected as a part of this project (approximately 15% of the total system), approximately 39% of the inspected pipelines are recommended for replacement or rehabilitation. A temporary flow monitoring program was conducted as part of this project from January 16, 2009 to February 12, 2009, a period of 28 days. A storm event occurred from February 5 to February 6, 2009 where the City received approximately 0.8 inches of rain each day. Smaller storm events also occurred during the monitoring period. The rainfall amount for each day during the monitoring period is shown in Table 6-1. The flow monitoring indicated levels of inflow and infiltration that are higher than desired, which is expected in an aging system. From a capacity stand point, the City's system appears to be adequately sized. The hydraulic model analysis indicated that there are areas in the southern, flatter portion of the system that are susceptible to surcharge and flooding during a significant wet-weather event. Three projects have been recommended to alleviate these capacity concerns.

The City of Beverly Hills owns, operates and maintains a gravity sewer system to serve all of the residents and businesses within the City limits. In late 2008, the City of Beverly Hills awarded a contract to RBF Consulting to prepare an updated Sewer Master Plan (Beverly Hills SMP). The City's sewer collection system was last evaluated as part of a master plan in 1996. This master plan has been prepared to summarize the condition of the City's existing sewer system and evaluate the system's capacity to accommodate various existing and future flow conditions. The master plan will also be used to meet the City's requirements to comply with the State of California's General Waste Discharge Requirements (WDR) for Sanitary Sewer Systems. This master plan will serve as a new planning document for the City that will describe the needed improvements for the collection system over the next 10 years.

The major elements completed as a part of the master plan project are:

- An update of City's GIS of the sewer system
- The scanning of all sewer system record drawings and linking to the GIS
- GPS survey of every manhole in the system for location and rim elevation

- Flow monitoring to analyze inflow and infiltration within the City
- Recommendations for City design criteria
- Evaluating the condition of representative portions of the City of Beverly Hills sewer system through closed circuit television (CCTV) pipeline inspections and manhole inspections
- Extended period modeling analysis to evaluate hydraulic capacity in various design conditions
- Determining necessary capital improvement projects
- Providing a financial plan for proposed improvements and maintenance

The City of Beverly Hills sewer system is a gravity flow system consisting primarily of vitrified clay pipe constructed from the 1920s to the present. The local sewer collection pipelines are predominantly 8-inch in diameter, but diameters range from 6-inch to 36-inch and total over 100 miles. Approximately 34,000 residents are served by the City's sewer system. Wastewater generated in the City of Beverly Hills is conveyed by the City owned gravity sewer (mains) pipelines into the sewer system owned by the City of Los Angeles and finally into the Hyperion Treatment Plant in Playa del Rey.

Flow monitoring was conducted for a period of 28-days to evaluate the normal flow conditions within the system and an extended flow data analysis was performed to assess the amount of rainfall entering the sanitary sewer collection system through defects. The analysis found the majority of the water entering the sewer system is through inflow directly into the sewer system. Included in the Capital Improvement Program (CIP), is an Inflow and Infiltration Study to develop a plan to reduce defect flows into the sewer system.

A system analysis was conducted with the use of the updated and calibrated hydraulic model. The purpose of the analysis was to simulate varying scenarios of different flow conditions to identify system deficiencies. As a result of the hydraulic model analysis, three projects were identified to alleviate the hydraulic capacity problems.

Table ES-1: Recommended Projects from Capacity Analysis

Priority	Street	Existing Diameter	Proposed Diameter	Total Length (LF)
1	Peck Drive	8"-21"	12"-27"	5,891
2	Gregory Way	24"-30"	30"-39"	4,286
3	Oakhurst Drive	10"-15"	15"-21"	2,915

The proposed diameters were sized to accommodate ultimate wet weather wastewater flows. The construction cost estimate for the three recommended pipeline capacity projects totals $8.2M (including engineering fees and contingencies). The limits of each recommended project is shown in Exhibit 6-2.

The pipeline and manhole condition assessment consisted of evaluations of a total of 77,106 LF of sewer pipe and 328 manholes (approximately 15% of the total system). The condition assessment consisted of a CCTV inspection and analysis of the sewer mains and their associated manholes. The condition assessment found that 39% of the pipelines were in need of some form of rehabilitation, such as relining or spot repairs. The estimated construction cost to replace or rehabilitate the pipelines and manholes inspected as a part of this project is $4.1M (including engineering fees and contingencies). The 15% of the system inspected was used to project construction $24.4M will be required to upgrade the remaining portions of the system.

A 10-year Capital Improvement Program (CIP) has been prepared to address the aging sewer collection system. The CIP has been prepared as a separate report, however it has been summarized within the master plan. The recommended projects have been prioritized to address the pipeline and manhole condition, pipe capacity, and inflow and infiltration concerns. The total recommended budget for the 10-year CIP is $39M.

Section 1—Introduction

1.1 Purpose and Scope

The City of Beverly Hills (City) is located in Los Angeles County, California, west of the City of West Hollywood and surrounded on all other sides by the City of Los Angeles. A regional vicinity map of the City is shown in Exhibit 1-1. The City of Beverly Hills' is 5.7 square miles in area and home to approximately 34,000 persons. It is estimated that the daytime population of the City can reach up to 200,000 people due to business, shopping and tourism. The City is a full-service agency, providing fire protection, police, building inspections, street maintenance, water treatment and distribution, refuse collection and sanitary sewer services to its residents.

The City provides sanitary sewer service for all residents and businesses located within the City limits. The sanitary sewer system consists of approximately 100 miles of gravity pipelines ranging in diameter from 6-inch to 36-inch, and includes 2,178 manholes. The City's collection system discharges wastewater directly into sewer lines owned by the City of Los Angeles, which are eventually conveyed to the Hyperion Treatment Plant in Playa del Rey.

The City of Beverly Hills Sewer Master Plan (Beverly Hills SMP) has been prepared to evaluate the City's sewer system for existing and future conditions. The City's sewer collection system was last evaluated as part of a master plan in 1997. The City has subsequently performed other special studies and flow monitoring on parts of the system. This master plan project will create a new planning document for the City that will describe the needed improvements for the collection system over the next 10 years. Lastly, a detailed financial plan will address the capital improvements for the collection system.

The State Water Resources Control Board (State Water Board) adopted Water Quality Order 2006-0003, on May 2, 2006, requiring all public agencies to comply with the Statewide General Waste Discharge Requirements (WDR) for Sanitary Sewer Systems. All public agencies must have applied for coverage by November 2, 2006, by completing the notice of intent (NOI) and legally responsible official (LRO) forms that the State Water Board distributed. The City of Beverly Hills has completed the NOI and is within the regulatory time frames. The intent of the WDR is to provide consistent statewide requirements for

managing and regulating sanitary sewer systems throughout California. There are three major components to the WDR, including:

- Sanitary Sewer Overflow (SSO) Prohibitions;
- Sanitary Sewer Management Plan (SSMP) Elements; and
- SSO reporting.

While there are many other relevant components and findings within the WDR, the major components identified above represent most of the State Water Board's regulatory expectations for the implementation of the WDR. The Sewer Master Plan will address many of the requirements of the WDR, including a system evaluation and capacity assurance plan. As part of the master plan, an update to the City's GIS will provide a useful application for engineering and operations staff to access the data stored in the GIS databases.

The major components of the master plan are:

- An update of City's GIS of the sewer system
- The scanning of all sewer system record drawings and linking to the GIS
- GPS survey of every manhole in the system for location and rim elevation
- Flow monitoring to analyze inflow and infiltration within the City
- Recommendations for City design criteria
- Evaluating the condition of representative portions of the City of Beverly Hills sewer system through pipeline inspections and manhole inspections
- Extended period modeling analysis to evaluate hydraulic capacity in various design conditions
- Determining necessary capital improvement projects
- Providing a financial plan for proposed improvements and maintenance

1.2 Study Area

As shown in Exhibit 1-1, the City of Beverly Hills is located in Los Angeles County, west of the City of West Hollywood and surrounded on all other sides by the City of Los Angeles. The study area for the sewer master plan is the City's

sewer collection system, which is located entirely within the City boundaries. The surrounding sewer service providers include the City of Los Angeles and the City of West Hollywood. The service area is approximately 5.7 square miles and serves a population of approximately 34,000 residents. The primary land use within the City is single family residential. Exhibit 1-2 shows the sewer master plan study area, along with the surrounding City and County boundaries.

1.3 Capital Improvements

The Capital Improvement Program (CIP) prepared as a part of this project identifies improvement projects based on a condition assessment (CCTV inspection) of a portion of the system and a capacity evaluation (hydraulic model analysis). The CIP identifies projects for implementation over the next 10 years (through 2020).

Projects were generally prioritized in the following order:

- Condition Assessments – existing facilities that have been determined to be in poor condition or failing found during the CCTV inspections, along with further CCTV inspections
- Capacity Evaluation – existing pipelines that demonstrated capacity deficiencies during dry-weather or wet-weather events as determined by the hydraulic modeling results
- Further Evaluation and Investigation - system studies and further inspections that are recommended

A summary of the capital improvement program is included in Section 9. The detailed Capital Improvement Program has been prepared as a separate report.

Section 2—Background

2.1 History of the City

The City of Beverly Hills has a long and rich history, starting long before the area was discovered by the movie industry in the early 1900's. From the beginning, the City of Beverly Hills was blessed with an abundance of water. The land was first occupied by Native American inhabitants, the Tongva or Gabrielinos. They thrived off the water that would collect from the surrounding Franklin, Benedict, and Coldwater Canyons. The convergence of this water was known as El Rodeo de las Aguas, or Gathering of the Waters. Prior to European influence, the Tongva lived off the land, characterized by wild game and abundant vegetation. The first European explorers arrived in 1769. Between 1769 and 1900, the land was used for farming, sheep and cattle ranching and oil exploration.

The first modern developments within the City occurred in 1907, when the City's first streets were constructed: Rodeo, Canon, Crescent, Carmelita, Elevado and Lomitas. The famous Beverly Hills Hotel was constructed in 1912 and became the center of the community, serving as the theatre, church and meeting hall. Shortly thereafter in 1914, the City was incorporated.

Drawn to the elegant lifestyle created by the Beverly Hills Hotel, several famous movie stars moved to the area and built mansions. The City prospered during the Roaring 20s. After a staving off annexation into the City of Los Angeles in 1923, the City continued to grow. By the 1950s few vacant lots remained in the City.

The City of Beverly Hills is now known as one of the most glamorous cities in the world. Destination hotels, restaurants, and shopping districts continue to attract visitors from around the world.

2.2 Land Use

The City last prepared and adopted a General Plan in on May 17, 1977. Over the years numerous amendments to the General Plan have occurred. The latest amendment occurred on January 12, 2010. The latest amended included three new elements: Historic Preservation, Economic Sustainability and Public Services. The Land Use Element in the Amended General Plan remains unchanged.

The City of Beverly Hills consists of diverse residential neighborhoods, retail and office districts, and civic buildings. The area north of Santa Monica Boulevard

consists of low and medium density single family residential neighborhoods. The area along Wilshire Boulevard is primarily commercial and office districts. South of Wilshire Boulevard consists of commercial, medium to high density residential neighborhoods, and mixed-use elements.

The City is essentially built-out. The lack of vacant land, high land values, restrictions on properties, and traffic combine to create a significant constraint on any new major developments within the City. According to the General Plan, any new development should coincide with the existing land use pattern.

The Land Use Plan from the General Plan has been provided in Appendix A. The map shows the associated land use for each area of the City. The Land Use Summary Table from the May 17, 1977 General Plan is also included in Appendix A, and summarized below in Table 2-1.

Table 2-1: Land Use Summary (reference City General Plan)

Land Use Description	Acres
Single-Family Residential	2,053.0
Multi-Family Residential	229.6
Commercial	267.4
Industrial	3.9
Educational	53.4
Public & Quasi Public	33.5
Parks	73.5
Religious / Schools	6.5
Vacant Land	38.2
TOTAL	**2,759.0**

2.3 General Topography and Climate

The general topography of the City is fairly level in the southern portions, while steadily increasing in elevation to the north. From the northeast corner of the City to the southeast, elevations range from 1350 feet to 130 feet above sea level.

The climate for Beverly Hills is dry and subtropical. The City is characterized by low humidity with a large amount of sunshine. The annual temperature range is between 57 and 75 degrees Fahrenheit, with an average of 66 degrees. The average annual rainfall for the area is 14.8 inches.

2.4 Sewer System Overview

The City of Beverly Hills sewer system is a gravity flow system consisting primarily of vitrified clay pipe constructed from the 1920s to the present. The local sewer collection pipelines are primarily 8-inch in diameter, but diameters range from 6-inch to 36-inch and total over 100 miles. Approximately 33,784 residents are served by the City's sewer system. The general direction of flow in the City of Beverly Hills' sewer system is from north to south. No sewer lift stations are required to convey flow.

Wastewater generated in the City of Beverly Hills is conveyed through service laterals to the City owned gravity sewer (mains) pipelines, then into the sewer system owned by the City of Los Angeles and finally into the Hyperion Treatment Plant in Playa del Rey. This master plan focuses on the gravity sewer system that is owned and operated by the City of Beverly Hills.

2.4.1 Backbone Sewer

The backbone of the City's sewer system consists of pipelines larger than 8-inch diameter. The backbone sewer pipelines (10-inch to 36-inch) collect flow from the 8-inch and smaller pipelines and convey wastewater into the City of Los Angeles' and Los Angeles County Sanitation District's (LACSD) collection system. The backbone sewer pipelines generally flow from north to south, and west to east. The backbone sewer lines are shown on Exhibit 2-1. Pipelines that are 15 inches or greater have also been shown on Exhibit 2-2.

2.4.2 Connections to Other Sewer Systems

The City's collection system is surrounded by pipelines owned primarily by the City of Los Angeles. The City of Beverly Hills also sits adjacent to the City of West Hollywood. The City of West Hollywood owns their collection system, but general maintenance is performed by the LACSD. A 39-inch LACSD trunk line, which receives flow from the City, is located in Hamel Drive north of Gregory Way, and then continues south in Shenandoah Street as it leaves the City. A diversion structure is located in Gregory Way between Arnaz Drive and Hamel Drive. The purpose of

the diversion structure is to send a portion of the flow to the 42-inch City of Los Angeles connection at La Cienega Boulevard and a portion of the flow to the 39-inch trunk line owned by LACSD that is located at Gregory Way and Shenandoah Street.

There are several connections to other sewer systems owned either by the City of Los Angeles, the City of West Hollywood, or LACSD. Those locations are shown on Exhibit 2-1. The larger flow outfall points are described below:

- 42-inch connection at La Cienega Boulevard, City of Los Angeles (District No. 3)
- 27-inch connection at Whitworth Drive, City of Los Angeles (District No. 1)
- 39-inch connection at Gregory Way and Hamel Drive, LACSD (District No. 3)

2.5 Previous Reports

In 1996, the City of Beverly Hills hired Willdan and Associates to prepare a Sewer Master Plan. The report, titled "Sanitary Sewer Master Plan for the City of Beverly Hills," analyzed the dry-weather flow conditions only, due to climatic circumstances during the flow monitoring period.

A second report was submitted in April of 1996, titled "Wet Weather Master Plan," which is sometimes referred to as "Phase 1 Inflow and Infiltration Report." The 1996 Master Plans reported on the current state of the system, and did not analyze future flow conditions. Capital improvements were recommended based on a hydraulic model simulation.

The 1996 Master Plan references a flow monitoring study that was conducted in 1995 titled "Wet Season Inflow and Infiltration Analysis." Flow monitors were installed at 35 locations for a period of 60 days. Five significant storm events occurred during the monitoring period. The study identified the areas within the City that were significantly affected by inflow and infiltration.

2.6 Trenchless Technology

This master plan identifies capital improvements to rehabilitate or upsize existing sewer pipe. Various trenchless technologies provide the City with an opportunity to reduce capital cost while maintaining and improving the level of sewer service.

In general, trenchless pipe construction can be utilized for new construction, upsizing existing facilities, and replacing or repairing existing facilities. Due to the congestion of underground utilities, aging infrastructure, environmental restrictions, public safety concerns, and political pressures, trenchless construction and rehabilitation methods are being considered for many projects as an alternative to open trench construction.

There are a number of trenchless and semi-trenchless methods in practice today for pipeline construction and rehabilitation. The technologies currently in use which are most pertinent to this master plan for structural and semi-structural pipe construction, rehabilitation and/or replacement include:

- Cured-in-place Pipe (CIPP) – The CIPP lining process involves inserting and inverting a resin-saturated felt tube into the existing pipe (like a very large inside-out sock). The tube is inflated to conform to the host pipe and hot air or water is circulated throughout the tube to cure (harden) the resin. When the curing process is complete, a new pipe is created that no longer has the joints, cracks, and holes that allowed infiltration and roots to enter the pipe causing operational problems such as blockages and overflows. The finished product has a 50-year design life. This method can also be used to make spot repairs, or sectional point repairs.

- Folded Pipe (fold and form, deformed/reformed) – A pre-fabricated HDPE (high density polyethylene) pipe, that has been collapsed in upon itself so that it is U-shaped, is pulled through an upstream manhole by a cable from the downstream manhole. Heat and pressure are applied to the U-shaped pipe causing it to revert to its original round profile.

- Sewer Lateral/Main Connection Re-Establishment – After a sewer line is rehabilitated with a trenchless liner the laterals are restored with robotically controlled cutting devices. These robots cut out a round coupon from the new liner at the lateral connections to allow flow into the pipe.

- Sewer Lateral/Main Connection (SLC) – This uses a specially shaped liner, which looks like a top hat (without the top of the hat) or inverted 'T.' The stem of the top hat forms the section of liner that is placed into the lateral pipe while the top hat crossbar sits against the inner wall of the main. Resin impregnation of the liner material allows it to be positioned using a specially designed remote control robotic installer and held in position for curing of the resin. Once cured, the robot and retention system are removed leaving the 'T' – shaped liner forming a seal across the lateral/main joint. The top hat stem can be installed as deep into the lateral as necessary.

Selection of trenchless technology may be the result of exploring the answers to such questions as:

- Are there environmental impacts or permit restrictions?
- Are there societal impacts (i.e. disruption to businesses or traffic routes)?
- Will the repair impact existing customers? Can the pipes remain in service during the repair?
- What are the site limitations (is space available for the necessary equipment and materials)?
- What is the cost of the repair compared to a traditional open-trench application?

Selection of which trenchless technology would be best for use should focus on the following questions:

- What is the problem that needs to be fixed?
- What is the existing pipe age, material, depth, size, condition, slope?
- Does the pipe require a structural repair or non-structural repair?
- What are the soil conditions?
- Can the trenchless method accommodate the ultimate hydraulic conditions?

- Will the repair result in the performance desired? What is the expected life of the repair?

The use of trenchless technology must also consider the interaction with the other components of a functional sewer system, such as:

- Laterals – Are there laterals that need to be reconnected, repaired or replaced?
- Manholes – Do the manholes need repair? Does the trenchless rehabilitation method require entry into the manholes?

Section 3—Inflow and Infiltration Analysis

This section discusses the findings of an extended flow data analysis performed to assess the amount of rainfall entering the sanitary sewer collection system through defects. For the purposes of this report, “defect flow” is defined as flow that enters the sewer collection system through inflow and infiltration (I/I). This analysis generally characterizes the change in flow between dry weather and specific wet weather events as an indicator of the system physical condition.

3.1 Flow Monitoring

Temporary flow monitoring was conducted at five City manholes, from January 16, 2009 to February 12, 2009, a period of 28 days. The basis for selecting the flow monitoring locations was to:

- Capture flow from a large tributary area
- Capture flow from different land uses, including residential and commercial
- Select locations that would aid in the calibration of the hydraulic model

During the 28-day monitoring period, a storm event was captured that was used in determining the inflow and infiltration impact to the system. The five temporary monitoring locations are shown on Exhibit 3-1, and described below:

- CHA01 – Charleville Boulevard and South Canon Drive (18” VCP)
- CRE – Crescent Drive, between Wilshire Boulevard and Charleville Boulevard (15” VCP)
- ECR – In alley at Pavilions Supermarket, east of El Camino Drive and north of West Olympic Boulevard (15” VCP)
- OAK – North Oakhurst Drive, between Wilshire Boulevard and Clifton Way (15” VCP)
- WIL – Wilshire Boulevard, between North Maple Drive and South Maple Drive (21” VCP)

Each of the temporary flow monitor locations received flow from a mix of both residential and commercial land uses. The results of the flow monitoring show repeating flow patterns throughout each day and throughout each week.

The influence of inflow and infiltration into the sewer system was observed from the results of the flow monitors. A storm event occurred from February 5 to February 6, 2009 where the City received approximately 0.8 inches of rain each day. Smaller storm events also occurred during the monitoring period. The rainfall amount for each day during the monitoring period is shown in Table 6-1. A noticeable increase in the flows in each sewer line occurred during the storm events. Detailed results of the five temporary flow monitoring locations have been included in the flow monitoring report in Appendix B.

There are also eight permanent flow monitoring locations located throughout the City. Four of the flow monitors record flow as it leaves the City sewer system. The other four flow monitors record incoming flow to the City's sewer system and are owned by the City of Los Angeles. Exhibit 3-1 identifies the five temporary and eight permanent flow monitoring locations, as well as their corresponding basins.

3.2 Analysis Techniques

This section describes the analysis techniques used to analyze the flow data. As indicated, the basis of the flow analysis is the determination of an Average Dry Weather Flow (ADWF) at each flow monitoring location. Once defined, the ADWF is used as the basis to quantify what occurs in the system during a wet weather event. This section discusses the components of flow, the design storm, and rain induced inflow and infiltration (RFI/I) projections.

3.2.1 Average Dry Weather Flow (ADWF)

Average Dry Weather Flow (ADWF) is the average flow that occurs on a day that is not influenced by rainfall. Selection of the dry days to develop the ADWF is important since this synthetic day becomes the basis of wet weather calculations. Wet weather responses are the deviations from the ADWF. ADWF is determined by selecting days on which several conditions are met. These conditions include:

- No rainfall occurred on that day
- No rainfall occurred on the preceding days

- Flow volumes were within a specified range (not less than 85% of average or more than 115% of average) (minimal deviation from other dry day)

The day of the week is also considered, since significant changes may occur in the flow patterns from weekday to weekend. Dry days for both weekdays and weekends are defined and the volume of flow and shape of the hydrograph for each flow monitor location was determined.

The ADWF is composed of two components: wastewater production and base inflow and infiltration. The relationship is shown in the following formula:

$$\textit{Average Dry Weather Flow} = \textit{Wastewater Production} + \textit{Base Inflow and Infiltration}$$

or

$$ADWF = WWP + BI/I$$

3.2.1.1 Wastewater Production (WWP)

Wastewater production (WWP) is the actual wastewater flow that is generated as a result of water use. The volume of the wastewater production is a function of the population and land use of the area. It is also strongly related to the actual water consumption in the area.

To generate WWP based on water records alone is inherently difficult due to the amount of detailed information required. As a result, other methods for determining WWP and its counterpart base inflow and infiltration have been developed. The methods used are consistent with reports published by the Environmental Protection Agency (EPA) and the Water Environment Research Foundation (WERF). The relationship between the average flow volume and the nighttime minimum flow rates are fundamental to the calculation of WWP and RFI/I (Rainfall Induced Inflow and Infiltration).

One method frequently used for the determination of WWP is as follows:

Wastewater Production = (Average Dry Weather Flow -Average Dry Weather Minimum) / Wastewater Production Factor

or

WWP = (ADWF-ADMF) / WWP Factor

Where the WWP factor varies from 1.0 to 0.75 and averages to 0.80 for predominately residential basins (EPA Publication "Computer Tools for Sanitary Sewer System Capacity Analysis and Planning" by Vallabhaneni, et al 2007). This factor is dependent on the land use in the area and may require adjustment to reflect nighttime activities if flow measurements indicate higher rates.

3.2.1.2 Base Inflow and Infiltration (BI/I)

Base inflow and infiltration (BI/I) is the flow that exists in the system at all times that is not a direct result of water usage or rainfall. This flow is considered to be constant and enters from such sources as cracked pipe below dry weather groundwater levels. Groundwater levels during dry weather are at their lowest. During wet weather, groundwater levels rise. If pipes experience inflow and infiltration based on the dry weather groundwater levels, this is considered BI/I. Inflow and infiltration caused by increased groundwater levels during wet weather events is considered rainfall induced inflow and infiltration, or RFI/I, which is discussed in the following section. The BI/I also may occur from irrigation drainage or faulty plumbing. The BI/I can be reduced through inflow and infiltration programs, or pipeline rehabilitation programs, however, a certain amount of BI/I is present in all sewer systems.

The amount of BI/I in a metered basin is derived from the previous formulas rearranged:

Base Inflow and Infiltration = Avg. Dry Weather Flow -Wastewater Production

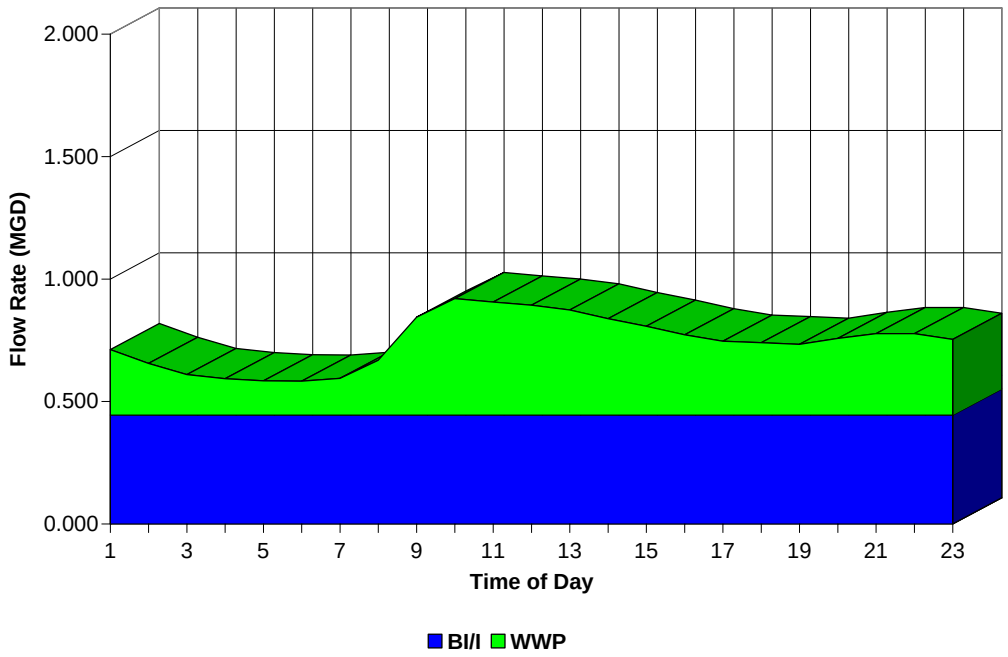
or

BI/I = ADWF –WWP

In this analysis the WWP and the Base BI/I are not separated due to the difficulty of isolating and removing the BI/I

component. Figure 3-1 shows a representation of the components of the Average Dry Weather Flow.

Figure 3-1: Average Dry Weather Components



Once the location's ADWF has been calculated, an assessment of the effects of rainfall on the system is performed. The following section discusses the analysis of the wet weather system response.

3.2.2 Wet Weather Flow Components

Wet weather flow analysis is performed to determine how particular portions of the collection system respond to rainfall events. Understanding these responses allows subsequent efforts to be focused primarily in areas with the greatest wet weather flow responses, which indicate the greatest density of defects in the system.

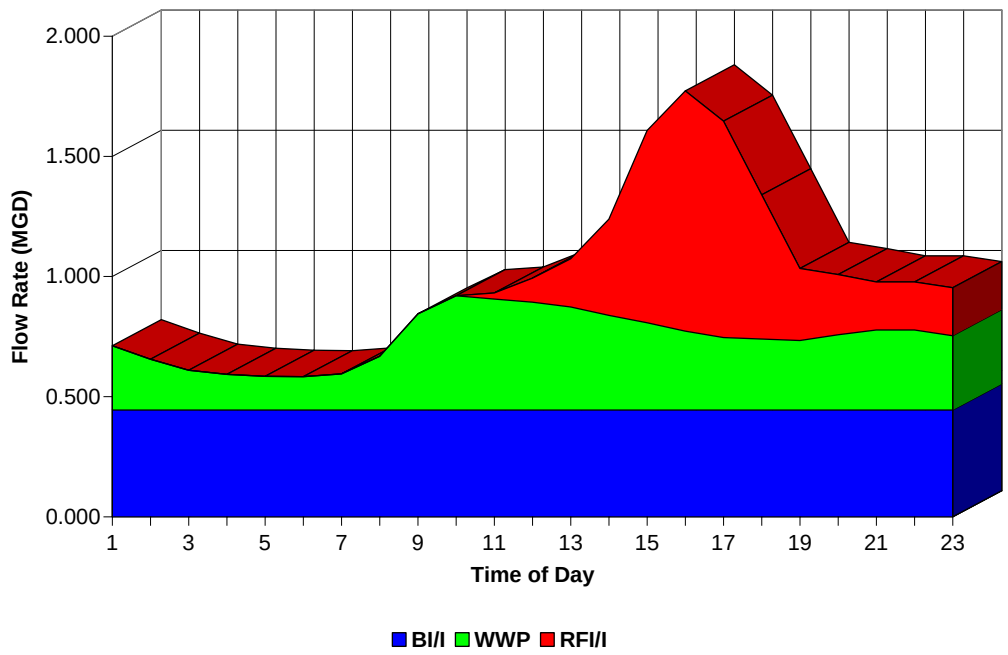
3.2.2.1 Rainfall Induced Inflow and Infiltration (RFI/I)

Rainfall Induced Inflow and Infiltration (RFI/I) is the additional flow (over and above ADWF) that occurs as a direct result of rainfall. It is composed of:

- The inflow component from defects directly connected to the surface
- The infiltration component from defects activated by saturated soils or elevated ground water tables

Of these two components, the inflow response most often dominates the shape of the RFI/I hydrograph. In considering the hydraulic capacity of the system, the inflow rate has the greatest effect on peak flows. Figure 3-2 shows a hydrographic response of the system to a storm event and its components.

Figure 3-2: Wet Weather Flow Components



Rainfall induced inflow and infiltration has both a shape, as described by hourly variations, and an associated volume. The assignment of the RFI/I volumes and shapes are extremely important in the hydraulic assessment of the system. Both the volume and the shape generally dominate the composite hydrograph and represent the “extreme event” used to assess the hydraulic capacity of the system.

3.2.2.2 Dominant Defect Flow Type

An important aspect of the wet weather response is the determination of what types of defects dominate the sub-basin. A basin that responds quickly to a rainfall event and recovers quickly from that event is dominated by inflow defects. A basin that shows slow increases over the ADWF curve and slow return to the ADWF is dominated by rainfall induced infiltration. Of course the most desirable situation is to have no response to the rain event. Figure 3-3 shows an example of a typical inflow dominated sub-basin. Figure 3-4 shows an example of a typical infiltration dominated sub-basin.

Figure 3-3: Inflow Dominated Response

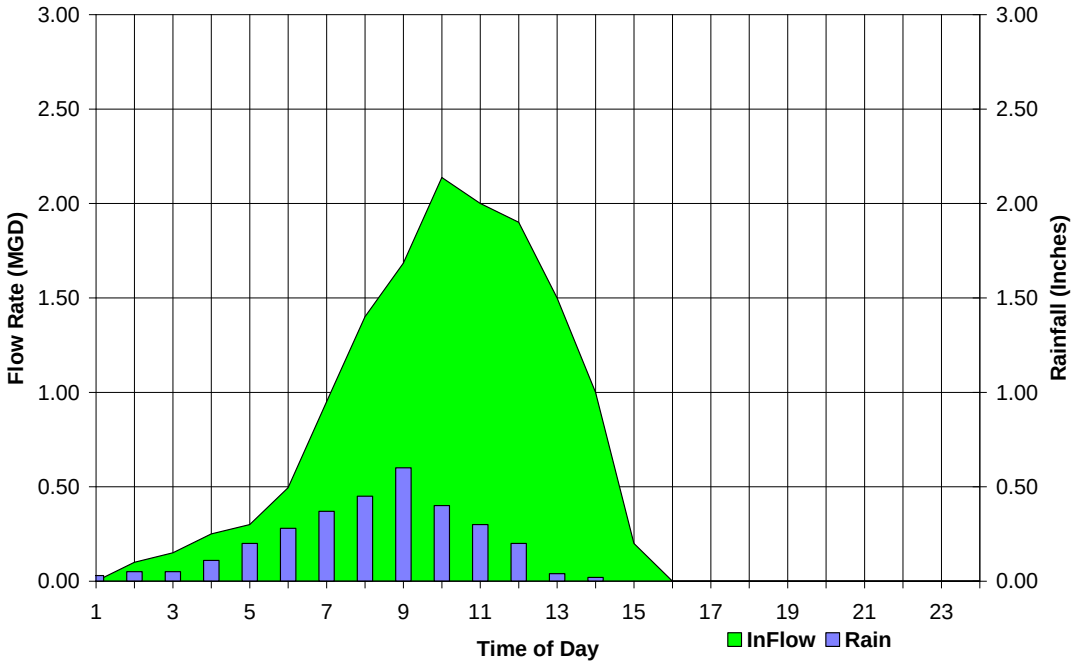
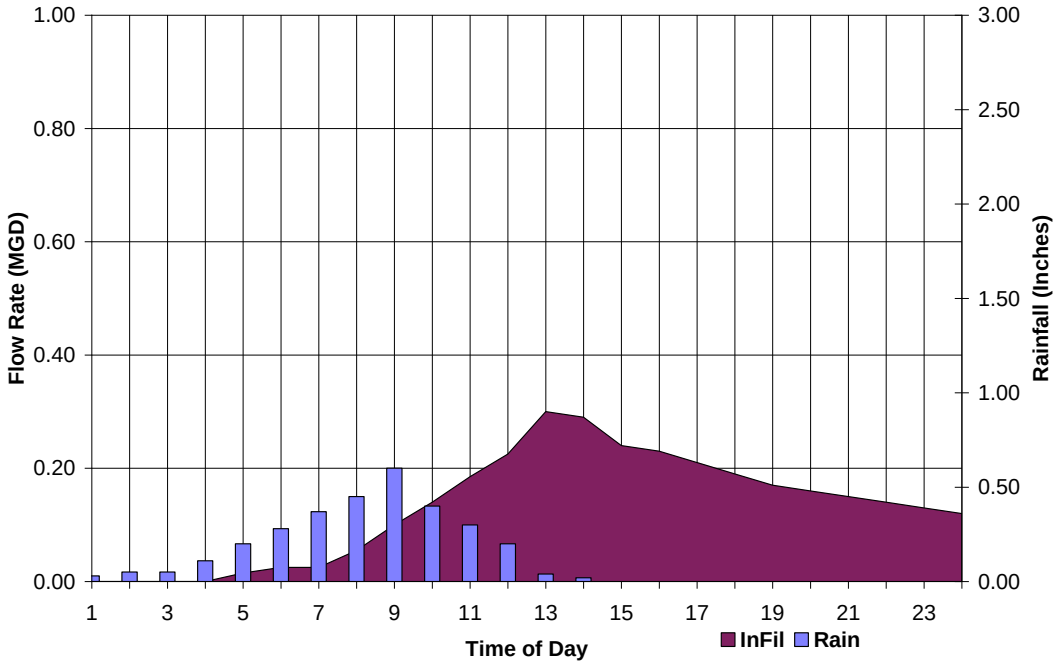


Figure 3-4: Infiltration Dominated Response

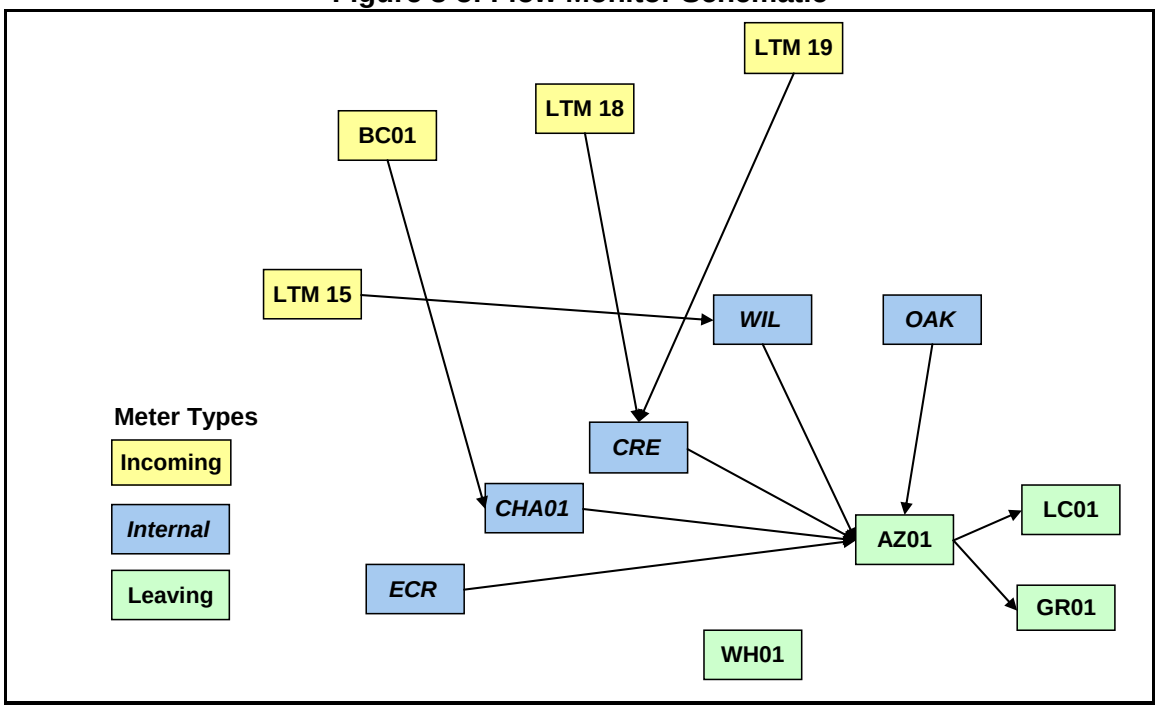


3.2.2.3 Total and Net Flows

No discussion of the various flow components would be complete without an explanation of "net" or incremental flows. Flow metering data yields flow information about the total flows accumulated at the meter location. The flows measured at that point are the sum of all upstream flows. This is called the total or "gross" flow.

While this information is useful, more detailed characterization of the system is found by subtracting the immediately upstream flows from the meter under study. This process yields "net" flows that describe the flow arising between the upstream and downstream locations. Using this detailed information the system can be characterized and basins compared. The following figure shows the flow schematic used for calculations of base flows in this study. The system downstream of AZ01 is diverted to locations GR01 and LC01 which requires special consideration in the hydraulic model and in the flow data analysis.

Figure 3-5: Flow Monitor Schematic



The colors in Figure 3-5 describe the role of the meter location in the flow measurement program. Note that the meter locations are a mixture of permanent and temporary installations. The incoming and leaving locations are permanent installations while the internal locations, shown in blue are temporary, and were installed from January to February 2009. The locations of each flow monitor are shown in Exhibit 3-1.

3.2.3 Analysis Products

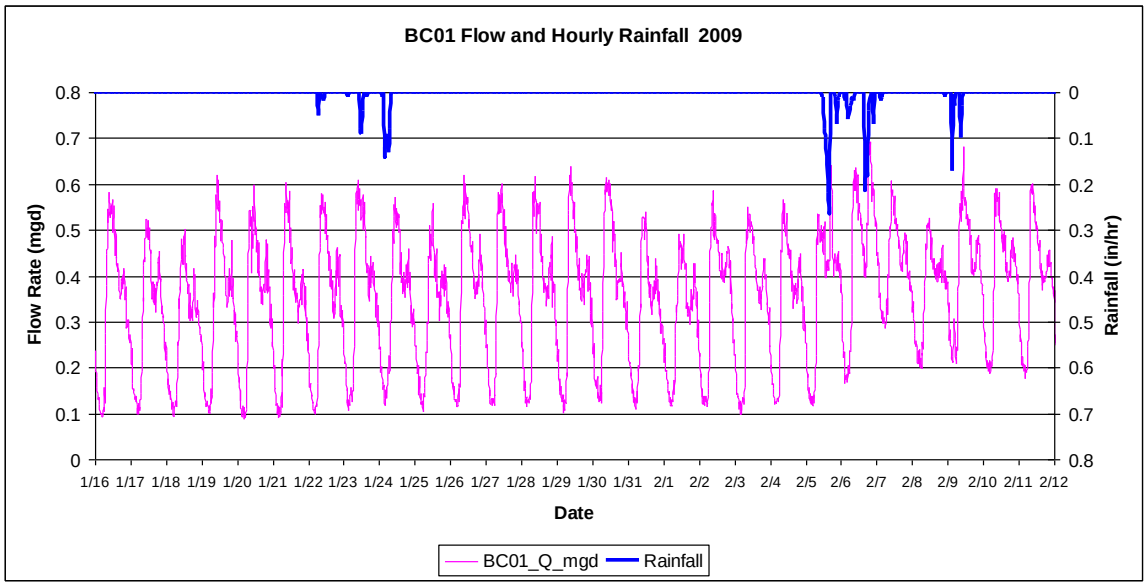
Analysis products are graphs which illustrate the basis of calculations for each flow monitor location. There are four primary types of graphs that will be shown and discussed: monitoring period, ADWF, storm event, and defect regression graphs. The data used for the calculations is provided in table format. Calculations include the determination of net and gross defect flows, normalization and prioritization.

3.2.3.1 Monitoring Period Graph

The data shown in the monitoring period graph, Figure 3-6, provides 15 minute average flow data and hourly accumulated rainfall, for the entire monitoring period (January 16 to

February 12, 2009) for the BC01 location. The whole monitoring period flow data, combined with rainfall data allows interpretation of the overall effects of rainfall on the system.

Figure 3-6: Monitoring Period Graph (BC01)



3.2.3.2 Average Dry Weather Flow (ADWF)

The average dry day graphs indicate the hourly variations of flow that occur at the monitoring location. This is used as the basis of the flow loading for a collection system hydraulic model. There may be dramatic differences in the weekend and weekday patterns as the dominant land use changes.

The average dry day graph, shown in Figure 3-7, show the nighttime minimum flows are practically identical for weekday and weekends, while the total volume and the peak hour rate differ substantially. This is an important consideration when reviewing the rainfall events that may occur on any combination of weekdays and weekends.

Figure 3-7: Hourly Average Dry Weather Flows (LC01)

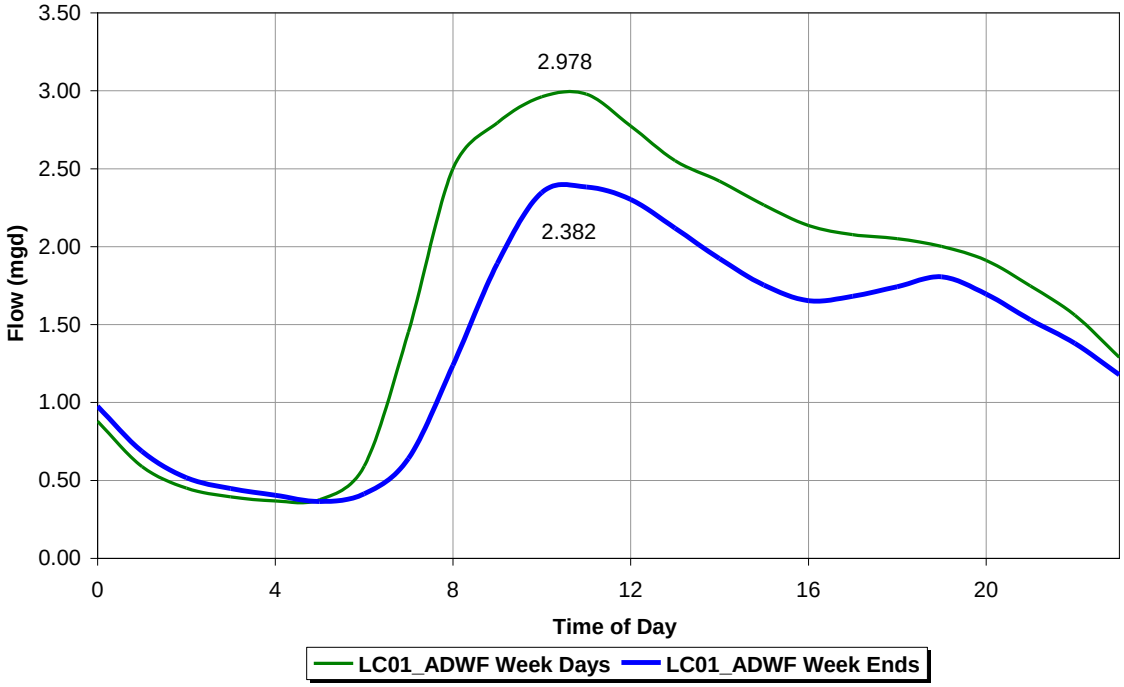


Figure 3-7 shows the actual flows that were observed. Hourly Flow Factors (HFFs) are also used in the hydraulic model which is the Hourly Average Flow / Daily Average Flow. This provides the basis for diurnal flow patterns which are used in the model and are also used to compare basin flow patterns where flows may be widely different due to a number of factors, including contributing land use and system defects within the basin. The HFFs for the monitoring locations are shown in the results section (Section 3.3).

3.2.3.3 Storm Event Graph

Figure 3-8: Storm Event Graph (LC01)

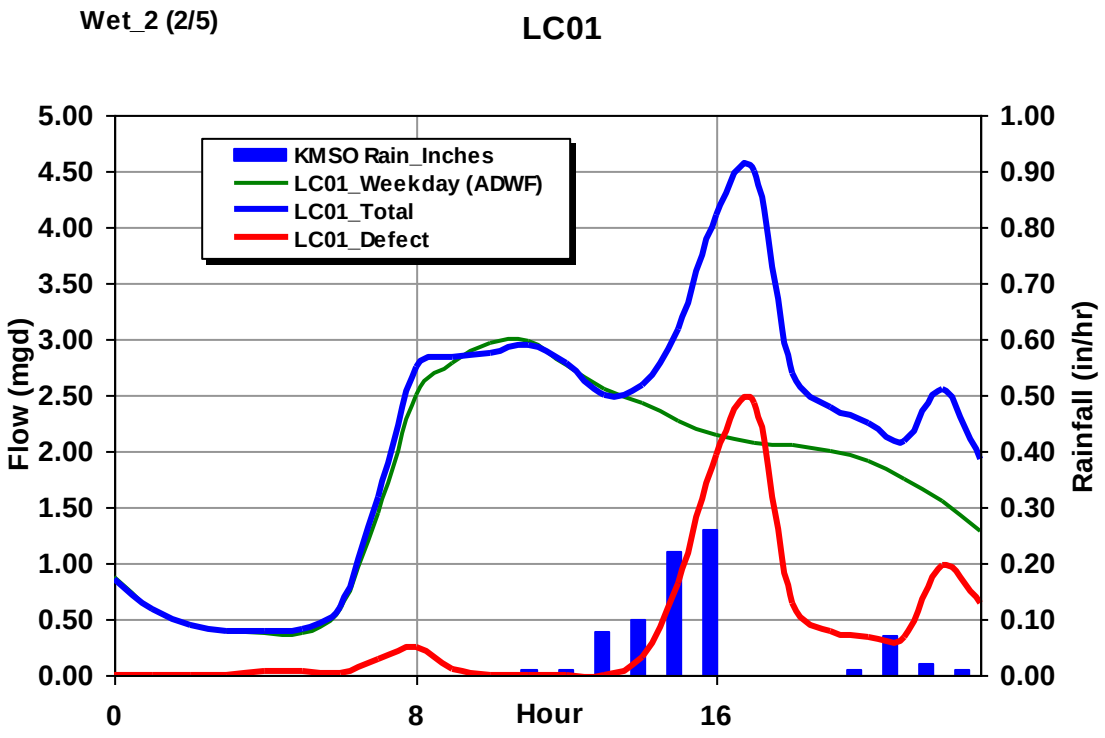


Figure 3-8 shows the storm event graph. The graph shows in greater detail the nature of the flows during the viewed event. Note that in this example the blue line indicates the total flow for the rainfall event that occurred on February 5, 2009. The blue bars indicate the rainfall and the red line indicates the defect flows. The rapid rise in flows during the rainfall event indicates that this is an inflow dominated location. The green line represents the average dry weather weekday flow for that individual flow monitor during the monitoring period. It represents an average of approximately 20 days (only weekdays during the 28-day monitoring period). Theoretically, the blue line should always be at the same level or above the green line. However, because the blue line is representing a one-day total, and the green line is representing a 20-day average, it is reasonable for this to happen. When the green line exceeds the blue line, it means the 20-day average dry

weather flow at that hour happened to exceed the actual observed flow on that particular day.

3.2.3.4 Normalization

The flows measured at the flow monitor locations in both dry and wet weather vary widely depending on the upstream length and the contributing land use areas. To allow direct comparison and the calculation of unit flow load factors, normalization is performed. In average dry weather normalization, the net flow is divided by the net basin length or count of manholes to create a flow loading factor expressed in mgd/foot or mgd/manhole.

Defect flow normalization may be performed using the length of pipe or the inch-diameter-mile (IDM) method. Defect flows are generally related to the surface area of the pipe. An IDM is defined by the total length of pipe, expressed in miles, multiplied by the diameter of the pipe in inches.

In addition to normalization using system lengths, manhole counts or IDMs, rainfall normalization is also performed. This results in a normalized or unit flow value that is expressed as gpd per foot of pipe per inch of rainfall. The normalization process allows basins of different sizes to be compared directly and is the basis of basin prioritization. For this analysis, normalized unit flows were determined by dividing the defect flow by inch-diameter-mile and inches of rainfall.

3.2.3.5 Design Storm Regression Analysis

One of the primary considerations for the hydraulic capacity analysis is the behavior of the system during a design event. The design event is a rainfall event with a specified return frequency and duration. The design event for this analysis is a 10-Year, 6-Hour storm. To determine the defect flow rate associated with the design event, regression analyses are performed for rainfall and defect flow volumes and rates for each of the locations. The results are a set of equations that predict the increased rate or volume of I/I for each of the locations and the r-squared value. The r-squared value is an indicator of the reliability of the prediction or “goodness of fit”

of the equation. This analysis provides a table and two graphs for each of the locations.

Table 3-1: Defect Flow Statistical Analysis (LC01)

LC01 Q vs i				
Q vs i	Regression Method			
	Vol to Vol		Peak to Peak	
	Rainfall	Defect	Rainfall	Defect
Event	(inches)	(mg)	(iph)	(mgd)
1	0.70	0.267	0.14	0.914
2	0.79	0.379	0.26	2.425
3	0.85	1.213	0.21	2.386
4	0.36	0.272	0.17	1.035
Equation	y = 0.61Ln(x) + 0.80		y = 2.85Ln(x) + 6.43	
R^2	0.2831		0.8458	

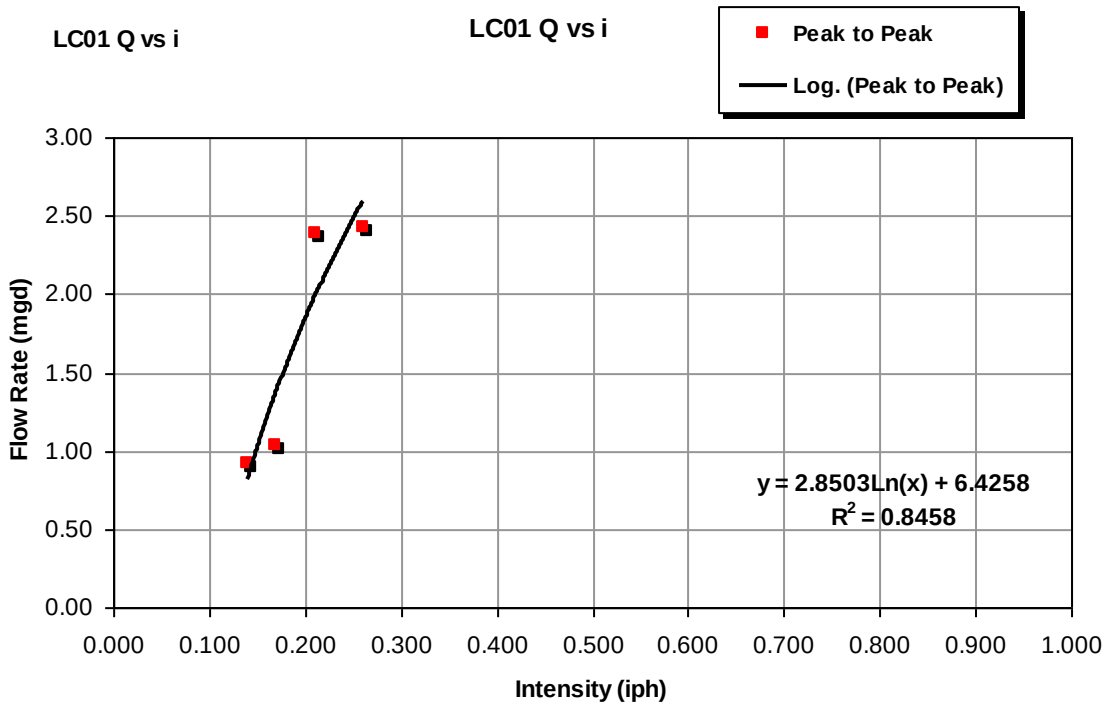
Table 3-1 provides the independent values (rainfall) and the dependent (defect flow) for the regression analysis which is done using the curve fitting feature of Excel charts. The method used to obtain defect flow was to generate an average dry weather flow for each flow monitor for weekday, weekend, and combined weekday/weekend. During each of the storm events, the total flow is recorded and compared to the average dry weather flow for that particular type of day (weekday, weekend, or combined). The defect flow for that particular storm event was determined by subtracting the average dry weather flow from the total observed flow. This was done for the four different storm events that occurred, and a graph was plotted based on the intensity of rainfall and the amount of defect flow. Using the four storm events, a mathematical relationship was determined from the four storm events where rainfall was the independent variable and defect flow was the dependent variable. The mathematical relationship can be used to predict the amount of defect flow a given rainfall intensity would create.

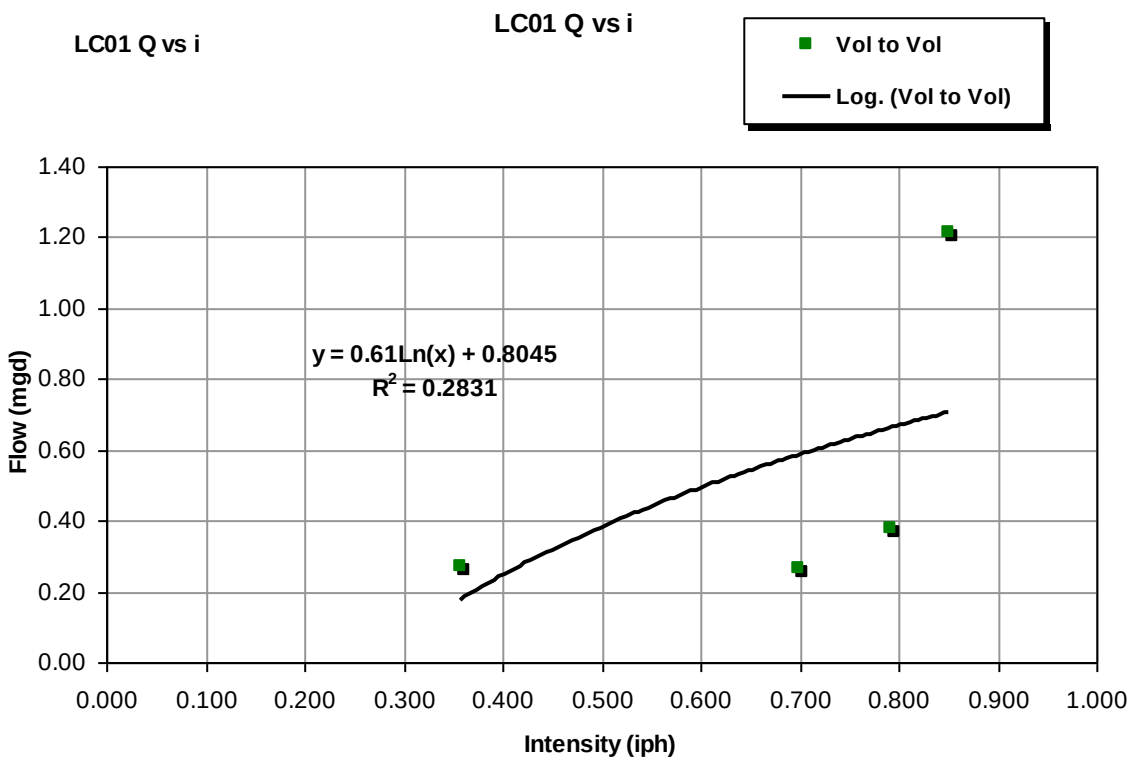
Figure 3-9 shows the regressions performed using the data in Table 3-1. It should be noted that Table 3-1 shows data from specific storm events recorded during the flow monitoring

period. The flow monitoring period recorded four storm events. The regression analysis used the four storm events to determine a conservative estimate for defect flow for this report. Any future analyses for precisely predicting defect flow would require monitoring several additional storm events of varying intensity and duration.

Note that the regression for rainfall volume to defect volume is very low (0.28) which indicates that the calculated formula is not a good predictor of defect flow volume. Fortunately the peak to peak (or rate to rate) regression has a high r-squared value (0.85) which indicates that the formula is a good predictor of defect flow rate from rainfall intensity.

Figure 3-9: Defect Flow Statistical Analysis Graphs (LC01)

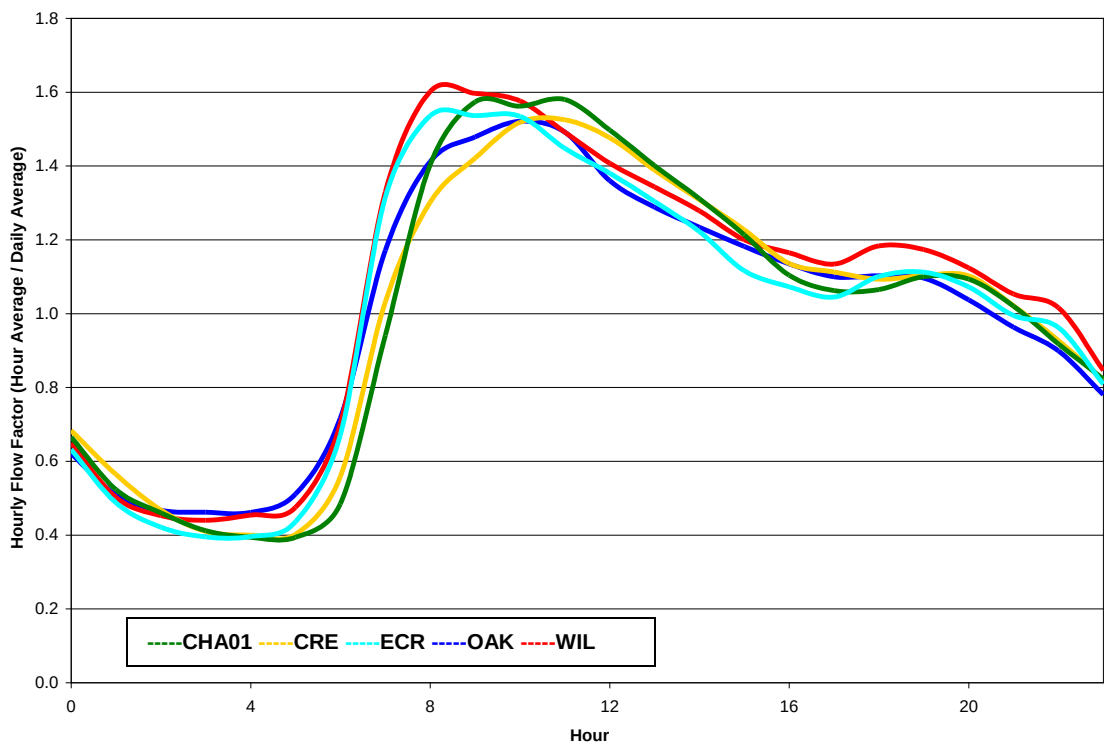




3.3 Results

Using the analysis techniques described in the previous section, the following results were obtained. Average Dry Weather Flows were analyzed for ten of the flow monitoring locations and characteristic Hourly Flow Factors were calculated. These are used as the basis of assessing wet weather impacts and are also used in the hydraulic model to generate individual diurnal flow patterns. The use of Hourly Flow Factors for comparison allows basins with differing flow rates to be viewed in the same graph. Figure 3-10 shows the hourly flow factors for the five temporary flow monitoring locations, and shows that peaking factors for each basin follow a similar pattern.

Figure 3-10: Hourly Flow Factors



3.4 Wet Weather Events

Four wet weather events were identified and used as the basis for this analysis. Defect flows (inflow and infiltration) were determined by subtracting the appropriate ADWF flows from the total wet weather event flow. This additional volume and rate is directly attributable to system defects. Normalization of these results was performed to directly compare the wet weather flows of basins of varying size and rainfall amounts. The normalized results are used to rank the

basins for further evaluation in the order of greatest to least defect flow. The dominant defect type, inflow or infiltration, was determined by comparing the relative amounts of flow increase during the most intense rainfall. A regression analysis was also performed to determine the reaction of the system to a 10-year design storm. The result of the wet weather analysis for each location is provided in Appendix C.

3.5 Findings

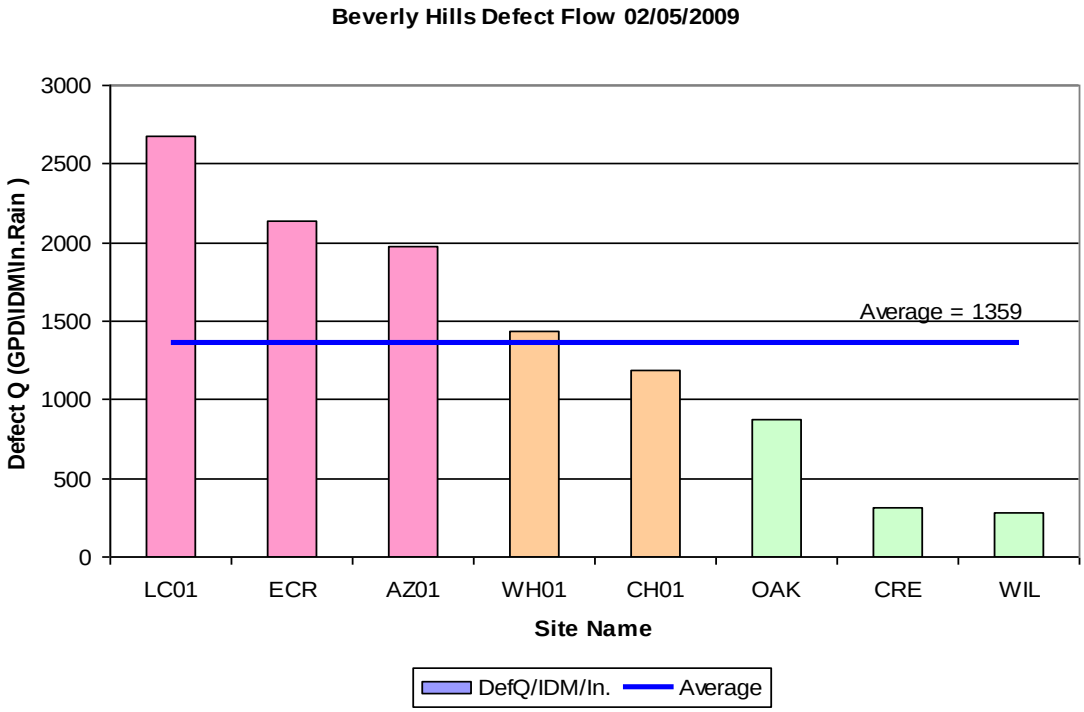
The graphs showing the dry weather and wet weather flows for each flow monitoring location are included in Appendix C.

The following observations are based on the analysis of the flow monitoring data:

- Sufficient rainfall occurred during the flow monitoring period to create wet weather flow increases
- The observed wet weather flow increases indicate the sewer system has defects that allows storm water to enter the sewer system
- The analysis of the flows shows the majority of the defects are inflow or directly connected defects
- Basins LC01, ECR, AZ01 and WH01 contributed over 2/3 of the defect flow

The unit defect flows for each basin are shown in Figure 3-11 and provide a visual comparison of the defect flows in gallons per day per inch-diameter mile per inch of rain.

Figure 3-11: Unit Defect Flows



3.6 Recommendations

Based on the findings presented above, it is recommended that basins LC01, ECR, AZ01 and WH01 be further investigated.

Basins LC01, ECR and AZ01 should be immediately inspected to determine if cross connections to the storm or roof drains exist. These three basins experienced above average defect flows, and rapid increases in flow with small low intensity rainfall events. Of the top ranked basins based on defect flow, basin WH01 may be prioritized last of the four basins for inspection. The remaining basin investigations may be deferred as their observed leakage rates are less than the overall average.

The basins should be inspected using smoke testing and dye tracing to determine direct connections into the system. Smoke testing in this area will be a challenge but is the least expensive and most positive way to determine interconnections. Prior to smoke testing, construction plans should be reviewed to determine if traps have been installed that would preclude a successful program. This recommendation has been included in the Capital Improvement Program.

Section 4—Sewer Design Criteria

The use of design criteria and standards are to ensure that sewer facilities meet a certain requirement in order to maintain a properly functioning sewer system. Design criteria for a sewer system is the basis for determining if a pipeline is deficient and requires improvement.

4.1 Existing City Design Criteria

Design criteria was established in the 1997 Master Plan as the basis for the system analysis and recommendation for capital improvement projects. The design criteria included flow coefficients, collection system design criteria, and inflow and infiltration assumptions.

4.1.1 Wastewater Flow Coefficients

Wastewater flow coefficients are developed to determine the quantity of wastewater flow generated by a specific land use type. Flow coefficients can be based on the land use area, dwelling count, population or building square footage. Flow coefficients are used to help estimate existing flows and predict future flows. They are used to allocate system flow inputs at manholes/nodes in a hydraulic model. The 1997 Master Plan derived individual flow coefficients for the major land uses throughout the City based on published data and other local agency standards. The flow coefficients utilized in the 1997 Master Plan are listed in Table 4-1. The flow coefficients are provided in ranges depending on the density.

Table 4-1: 1997 Wastewater Flow Coefficients

Land Use Type	Flow Coefficient (gpd/ac)
Single Family Res.	700 - 900
Multi Family Res.	5,700 - 6,500
Commercial	10,000 – 13,600
Municipal/Industrial	7,000 - 8,725
Education/Religious	500 – 1,000

4.1.2 Collection System Design Criteria

The 1997 Master Plan established "depth to diameter" (d/D) criteria for both dry weather and wet weather conditions. The d/D ratio represents the depth of flow in relation to the overall diameter of the pipe. The following d/D criteria from the 1997 Master Plan were used as the basis for determining hydraulic deficiencies.

Table 4-2: 1997 Master Plan Collection System Design Criteria

Criteria	Dry Weather	Wet Weather
≤ 15"	d/D = 0.50	d/D = 0.90
> 15"	d/D = 0.75	d/D = 0.90

4.1.3 Inflow and Infiltration

There are two components to inflow and infiltration used in the 1997 Master Plan. Base inflow and infiltration (BII) refers to the additional flow in the sewer system that is not a direct result of water usage. It typically is a result of pipe cracks and faulty plumbing. BII is present in the sewer system at all times. Rainfall induced inflow and infiltration (RFII) is the additional flow in the sewer system that is a direct result of rainfall, and is only accounted for in the wet weather analysis.

4.2 Recommended City Design Criteria

The design criteria established in this report was used as the basis for the system analysis, as discussed in Section 6. Flow coefficients were developed based on water billing data and refined with flow monitoring data. Flow loading based on the anticipated defect flow was also refined with the flow monitoring data. Collection system criteria was used to determine deficient pipelines and as the basis for the capital improvement program. Inflow and infiltration was determined based on flow monitoring data during storm events.

4.2.1 Wastewater Flow Coefficients

As a part of this analysis, the previously established wastewater flow coefficients were evaluated. The City provided water billing data for the previous three complete fiscal years (07/08, 08/09, 09/10). The water billing data was evaluated to determine annual water use and water usage per land use. On average, the metered records indicate City customers use 8.7 million gallons of water per day. Based on the system-wide wastewater generation determined from flow monitoring data, it was

calculated that approximately 73% of the billed water is returned to the sewer system. The calculations did not include water specifically used for irrigation purposes and it excluded wastewater generated from the treatment plant. The return to sewer ratios for each major land use type were calculated and identified in Table 4-3.

Table 4-3: Return to Sewer Ratios

Land Use Type	Return to Sewer Ratio (%)
Single Family Res.	60
Multi Family Res.	90
Commercial	95
Municipal/Industrial	95
Education/Religious	90
Total	73

The return to sewer ratios were multiplied by the billed water data for each land use type to calculate a wastewater flow coefficient. The calculated wastewater coefficients are identified in Table 4-4.

Table 4-4: Wastewater Flow Coefficients

Land Use Type	Flow Coefficient (gpd/ac)
Single Family Res.	1,000 -1,500
Multi Family Res.	6,500 -8,000
Commercial	5,000 – 6,600
Municipal/Industrial	10,000 – 13,500
Education/Religious	2,500 – 3,000

Wastewater flow loading for the hydraulic model was based on anticipated defect and wastewater flow. The defect flow analysis used the data from the temporary and permanent flow monitors. The model was

loaded with wastewater flows to replicate the results of the temporary flow monitors.

4.2.2 Collection System Design Criteria

Table 4-5 identifies the design criteria used in this master plan and as the basis for the Capital Improvement Program (CIP), based on future (2030) wet weather flow.

Table 4-5: Design Criteria for Existing Pipelines

Pipe Size	d/D Ratio		
(inches)	0.50 to 0.75	0.75 to 0.90	≥ 0.90
< 18”	Watch	Schedule	Replace
≥ 18”	OK	Watch	Replace

Notes:
1. “Watch” indicates that special attention needs to be paid to increased flows that are tributary to this pipe. A proposed development may create a situation where the performance criteria is exceeded.
2. “Schedule” indicates that a replacement project needs to be considered but can be scheduled at some point in the future.
3. “Replace” indicates that an immediate project funding and design should begin.

Table 4-6 identifies the minimum design criteria to be used for all new pipelines.

Table 4-6: Design Criteria for New Pipelines

Pipe Dia. (in)	Min. Slope	Max. d/D	Min. Velocity (ft/s)
8	0.0040	d/D = 0.50	2.0
10	0.0028	d/D = 0.50	2.0
12	0.0022	d/D = 0.50	2.0
≥15	0.0015	d/D = 0.75	2.0

The maximum allowable slope shall be the slope which generates a maximum flow velocity of 8.0 feet per second at the peak flow rate.

4.2.3 Inflow and Infiltration

The effect of inflow and infiltration on the City sewer system was determined based on evaluation of previous reports and studies, as well as the flow monitoring that took place from January 16, 2009 to February 12, 2009 (28 days). An extensive inflow and infiltration analysis was described in Section 3. For future analyses, average dry weather flows shall be increased by a factor of 1.5 to predict defect flows, or flows

arising from inflow and infiltration. This factor is consistent with the design storm event discussed in Section 6.5.3.

4.2.4 Mannings Roughness Coefficient (“n”)

For future analyses, a value of 0.013 shall be used for Manning’s Roughness Coefficient (“n”). This is the industry accepted value for vitrified clay pipe and concrete pipe, which represent essentially all of the sewer system.

Section 5—Existing System

5.1 Description of Existing System

The City of Beverly Hills sewer collection system consists of an entirely gravity flow system. The City's sewer system consists of primarily pipelines and manholes.

5.1.1 Pipelines

The City of Beverly Hills' sanitary sewer system consists of 6-inch through 36-inch gravity pipelines constructed within the City limits. The majority of the system is comprised of vitrified clay pipe, with some of the system being composed of concrete pipe as well. The collection system is divided into 21 sewer districts. District number 1 is located in the southern portion of the City and District number 21 is located in the northern most area of the City. The district boundaries and locations are shown on Exhibit 5-1. Table 5-1 shows a breakdown of the City's sewer lines by pipe diameter. Table 5-2 shows a breakdown of the City's sewer lines by pipe material.

Table 5-1: Existing Pipe Diameters

Pipe Diameter (in)	Length (ft)	Percentage
6	39,277	7.40%
8	348,636	65.68%
10	61,471	11.58%
12	23,828	4.49%
15	29,799	5.61%
18	12,964	2.44%
21	3,522	0.66%
24	7,214	1.36%
27	317	0.06%
30	2,525	0.48%
32	225	0.04%
33	635	0.12%
36	408	0.08%
	530,821	**100.0%**

It should be noted that the City GIS identifies approximately 0.85% of the system to have an "unknown" material. As the City completes the remaining pipeline inspections for their system (discussed further in Section 7), the unknown pipe parameters will be determined and the GIS should be updated accordingly.

Table 5-2: Existing Pipe Materials

Material	Length (ft)	Percentage
Unknown	4,507	0.85%
Concrete	141,749	26.70%
Vitrified Clay	384,269	72.39%
Cast Iron	35	0.01%
Ductile Iron	261	0.05%
	530,821	**100.0%**

5.1.2 Manholes

There are 2,236 manholes in the City's sewer collection system. Most of the manholes are 48 inches in diameter. The predominant construction material is precast concrete, but several manholes were constructed with brick and mortar. The manholes are primarily located within public streets and alleys. In the northern parts of the City (north of Sunset Boulevard) there are over 100 manholes located on private property. The manhole identification numbers are based on the sewer district they are located in, as shown on Exhibit 5-1.

5.1.3 Lampholes

The City's sewer collection system also consists of approximately 100 lampholes. Lampholes were constructed in the 1920s and 1930s as a way of allowing for visual inspection and cleaning from the surface, without manned entry. Lampholes were typically 12-24 inches in diameter. They appear to be very similar to today's sewer cleanout. The City has gradually replaced the lampholes with manholes as system improvements have occurred.

5.2 System Age

The majority of the City of Beverly Hills' sewer system (48.1%) was constructed in the 1930s. Table 5-3 and Figure 5-1 depicts the decade in which the sewer collection system was constructed. The useful life of vitrified clay pipe depends on many factors but often is considered to be roughly 60 years. Conditions such as earth loads, root intrusion and grease build-up will adversely affect both pipe capacity and useful life. In addition, the technology used to fabricate the pipe and type of joint gaskets has significantly improved since the 1960s.

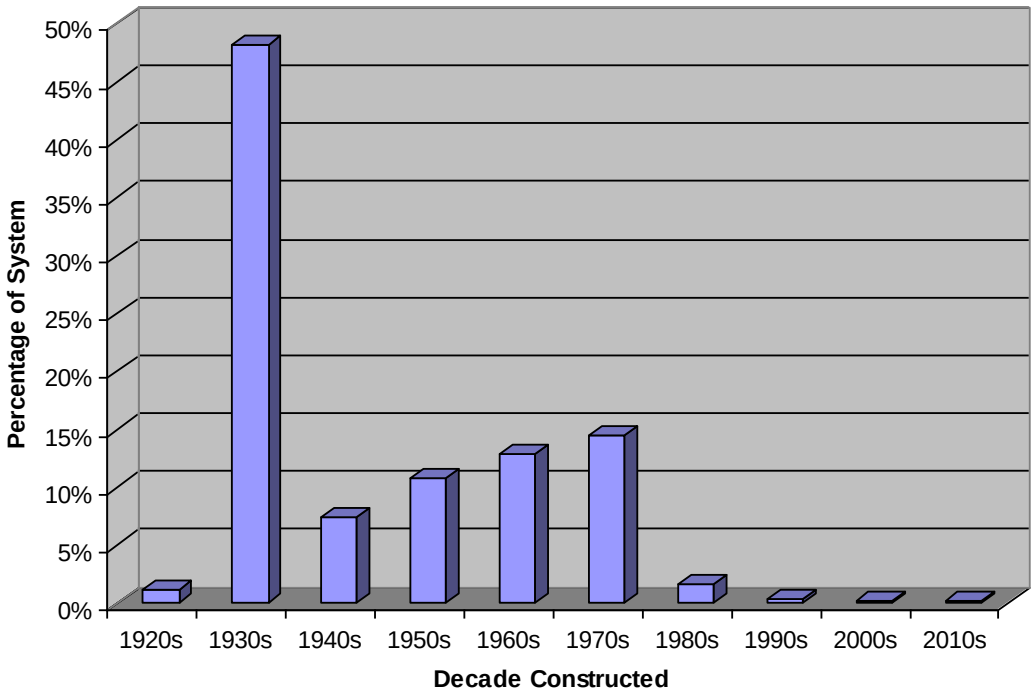
Closed circuit television inspection (CCTV) was conducted on a representative sample of approximately 15% of the entire sewer system. The purpose of the

CCTV was to assess the condition of the pipelines and determine what measures must be taken for rehabilitation or replacement. Of the pipes that were inspected during the CCTV, approximately 39% were recommended for some type of rehabilitation method, such as relining or spot repair. These recommendations have been included in the Capital Improvement Program (CIP).

Table 5-3: Pipe Construction Timeline

Decade Const.	Length (ft)	Percentage
Unknown	15,119	2.85%
1920s	6,611	1.25%
1930s	255,403	48.11%
1940s	39,223	7.39%
1950s	56,959	10.73%
1960s	68,081	12.83%
1970s	76,999	14.51%
1980s	8,617	1.62%
1990s	1,602	0.30%
2000s	1,099	0.21%
2010s	1,107	0.21%
Total	**530,821**	**100.0%**

Figure 5-1: Pipe Construction Timeline



Section 6—System Analysis

The system analysis was conducted with the use of the updated and calibrated hydraulic model of the City of Beverly Hills sewer system. The goal of this analysis was to simulate varying scenarios of different flow conditions to identify system deficiencies. The development of the hydraulic model and results of the analysis are discussed in the following sections.

6.1 Hydraulic Model Development

A hydraulic model is a computer simulation of a sewer system. The purpose of the hydraulic model is to simulate a sewer system and sewer flows in order to identify deficiencies as well as impacts of future developments, population growth, land use changes, or many other variables.

The hydraulic model was created by importing the physical system from the updated Geographic Information System (GIS) and the flows from a mixture of temporary and permanent flow monitors (discussed further in Section 3). The integration of these two data sources creates a model that is used as the basis of a capacity evaluation. The software used for the hydraulic model is Hydra by Pizer Incorporated.

6.1.1 System Configuration

As a part of the overall effort to better understand the complexities of the collection system, substantial field investigations were performed. These investigations produced updated field maps which were then used to prepare a digital map of the collection system.

Additionally, information related to the size and condition of the pipes was also gathered and entered into databases. These two sources of information were used to define the overall system layout which was subsequently used as the basis of the collection system model. The system representation was then combined with information collected in the flow monitoring portion of the program to provide a calibrated model which represents the current state of the collection system. These products should also be quite useful in improving maintenance operations for the City.

6.1.2 GIS to Model

As a portion of the Sewer System Management Plan (SSMP) compliance and Sewer Master Plan Update, an updated GIS was prepared. This data collection was then converted into the modeling files for use in the Hydra hydraulic model. The Hydra model is a publicly available collection system model that is suitable for capacity planning and project development.

The Hydra model is a link-node model. Links represent gravity and pressure pipes while nodes represent manholes, lift stations, diversions and reservoirs. Each element's hydraulic characteristics are mathematically modeled and solved in a way that closely represents the physical world.

6.1.3 Assumptions

In all endeavors of this magnitude, certain assumptions must be made when data is unavailable or of questionable origin. In these areas a conservative approach has been taken to assure that the model represents the system under a "worst-case" scenario. It is important to note that as the system data is updated and verified, this can be incorporated into the system model and its overall impacts assessed.

A common assumption or estimate of this type is in the roughness of the pipe. The actual ability to measure this roughness may be possible in university laboratories but is practically impossible in the field. The roughness coefficient used in the hydraulic model tends to generate a capacity number that is slightly less than would be measured in the laboratory setting. This results in a conservative estimate of the flow. The roughness coefficient used in the model was 0.013, which is the industry accepted value for vitrified clay and concrete pipe.

Another example of this type of assumption would be in the determination of pipe slope. Pipe slope is a very important parameter in the determination of the hydraulic capacity. To acquire this information, as built records or field measurements must be utilized. A reasonable assumption (in the absence of field data or as built records) would be that the pipe was laid at minimum slope. The capacity that would be calculated would be the minimum that would be accepted by the local

agency. In common field practice the slope is almost always greater than this number and the corresponding capacity greater as well.

Similarly, in the few locations where pipe diameters were not identified in the GIS, the diameter was estimated as the smallest of the nearest upstream pipes. This is noted in the modeling database to readily identify areas that may need to be field verified.

All assumptions are subject to a certain element of risks. In this program all assumptions that were made were within industry accepted ranges.

6.1.4 Diversions

Diversions in the collection system are defined where flow enters two or more pipes from a single manhole. Diversions are most often constructed to divert flow from one part of the system to another. Eleven diversions were identified in the collection system model. Most occur in the same measurement basin so the diversion flow was estimated to be equally applied to each downstream leg. Two special cases of diversions were considered where flow is diverted into different flow measurement basins. In this case the flows would have a direct impact on the calibration process. The diverted flow ratios required for the model, were calculated in the calibration process to determine how much flow was diverted in each direction. The ratios were varied until the model was calibrated with the flow monitoring data.

6.2 Rainfall Analysis

National Oceanic and Atmospheric Association (NOAA) rainfall data for the area was obtained from the Federal Aviation Administration's (FAA) KSMO weather station at the Santa Monica Airport. This station is located approximately 5 miles west, southwest from the City of Beverly Hills. While It is likely that the rainfall records slightly less total volumes than would have been experienced in the upper elevations of Beverly Hills, a good correlation between rainfall and wet weather responses was observed.

Rainfall data was also obtained from Personal Weather Stations (PWS) for Benedict Canyon and the CBS building. The comparisons of rainfall daily totals show that there is an expected variation in rainfall by elevation and distance from the coast.

6.2.1 Storm Event Analysis

Table 6-1 describes the date, day, rainfall total inches and weather classification (WXClass) for the temporary flow monitoring period.

The weather classification field is used to determine how the flow and rainfall data for that day is used. DRY indicates that it is used for calculation of the average dry day, WET_X indicates that it is used in the Wet Weather Event Analysis where the X indicates the event. Four events were considered. UNC indicates unclassified and that the data is not used in the analysis since it may be influenced by rainfall or inadequate data.

The total rainfall during the rainfall period was just over 2.7 inches.

Table 6-1: Rainfall Data from January 16, 2009 to February 12, 2009

Weather Classifications for Beverly Hills				
Date	**Rain_Inches**	**WxClass**	**Day**	**DayClass**
1/16/2009	0.00	Dry	Fri	WeekDay
1/17/2009	0.00	Dry	Sat	WeekEnd
1/18/2009	0.00	Dry	Sun	WeekEnd
1/19/2009	0.00	Dry	Mon	WeekDay
1/20/2009	0.00	Dry	Tue	WeekDay
1/21/2009	0.00	Dry	Wed	WeekDay
1/22/2009	0.07	Unc	Thu	WeekDay
1/23/2009	0.22	Wet_1	Fri	WeekDay
1/24/2009	0.46	Wet_1	Sat	WeekEnd
1/25/2009	0.00	Unc	Sun	WeekEnd
1/26/2009	0.00	Unc	Mon	WeekDay
1/27/2009	0.00	Dry	Tue	WeekDay
1/28/2009	0.00	Dry	Wed	WeekDay
1/29/2009	0.00	Dry	Thu	WeekDay
1/30/2009	0.00	Dry	Fri	WeekDay
1/31/2009	0.00	Dry	Sat	WeekEnd
2/1/2009	0.00	Dry	Sun	WeekEnd
2/2/2009	0.00	Dry	Mon	WeekDay
2/3/2009	0.00	Dry	Tue	WeekDay
2/4/2009	0.00	Dry	Wed	WeekDay
2/5/2009	0.79	Wet_2	Thu	WeekDay
2/6/2009	0.81	Wet_3	Fri	WeekDay
2/7/2009	0.03	Wet_3	Sat	WeekEnd
2/8/2009	0.01	Wet_4	Sun	WeekEnd
2/9/2009	0.34	Wet_4	Mon	WeekDay
2/10/2009	0.00	Unc	Tue	WeekDay
2/11/2009	0.00	Unc	Wed	WeekDay
2/12/2009	0.00	Dry	Thu	WeekDay
Total	2.73			

Table 6-2 shows data for each of the four rainfall events.

Table 6-2: Rainfall Event Characteristics

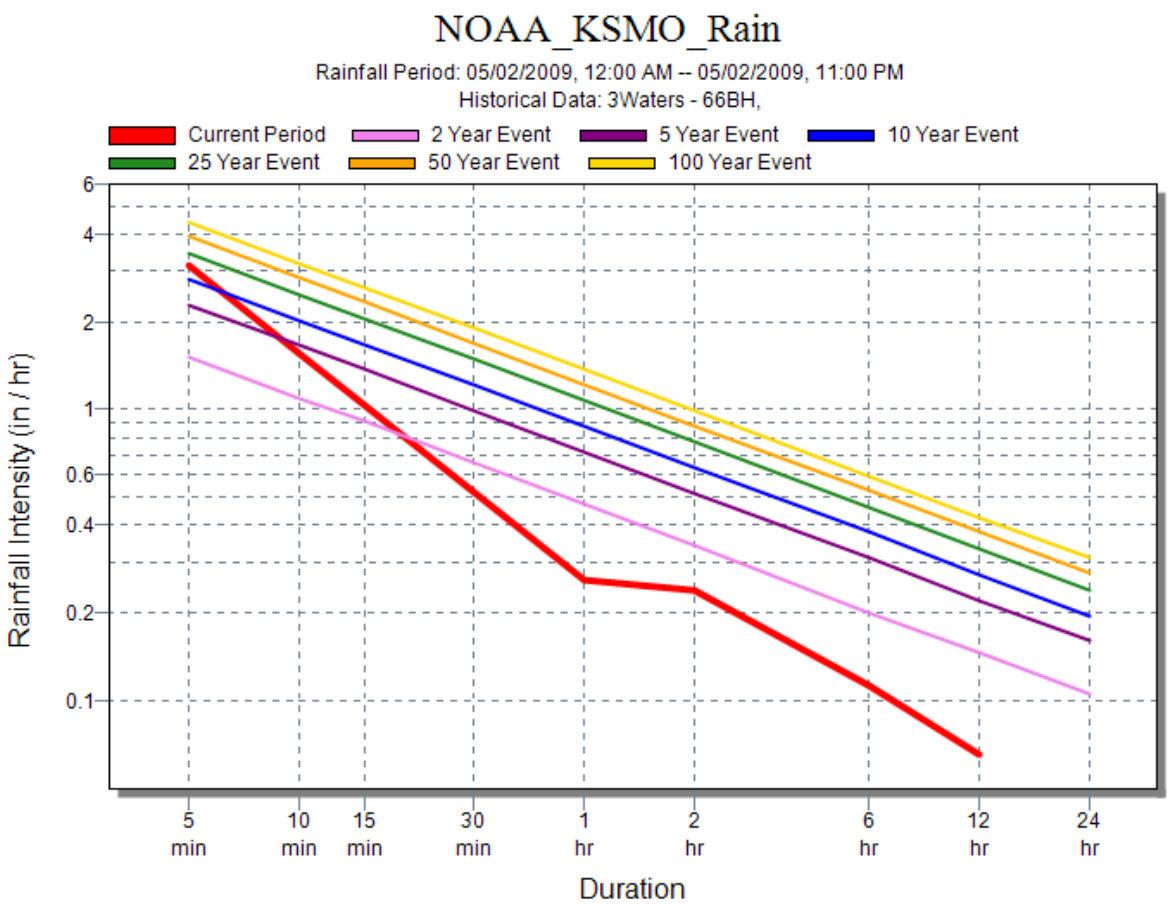
Event Name and Date	Rain Vol. (Inches)	Rain Rate (in/hr)
Wet_1 (1/23-1/24)	0.70	0.14
Wet_2 (2/5)	0.79	0.26
Wet_3 (2/6-2/7)	0.85	0.21
Wet_4 (2/8-2/9)	0.36	0.17

6.2.2 Design Storm Development

Rainfall event Wet_2 was selected as the basis of the wet weather calibration and as the event that the design storm is scaled from. This event was preceded by significant dry weather and had a significant intensity and volume to create widespread flow increases throughout the system. Figure 6-1 shows the relationship of the storm event to the Intensity, Duration and Frequency Curves (IDF) from the Los Angeles County Hydrology Manual for the Beverly Hills area. The storm event plotted in red indicates that the peak intensity that occurred during the selected event exceeded the 10 year event for a short period of time. This is important in that the rainfall rate increases the inflow rate from defects and this event reflects a significant impact on the collection system.

The capacity of the system to handle this event is discussed later in the capacity analysis.

Figure 6-1: Rainfall Event with Intensity, Duration, and Frequency Curves



Total Rainfall During Period	0.78 in
Hours in Period	23.0 hrs

	5 min	10 min	15 min	30 min	1 hr	2 hrs	6 hrs	12 hrs	24 hrs
Current Period	3.12	1.56	1.04	0.52	0.26	0.24	0.11	0.07	0.00
2 Year Event	1.52	1.10	0.91	0.66	0.47	0.34	0.20	0.15	0.11
5 Year Event	2.30	1.66	1.37	0.99	0.72	0.52	0.31	0.22	0.16
10 Year Event	2.81	2.03	1.68	1.21	0.87	0.63	0.38	0.27	0.20
25 Year Event	3.46	2.50	2.06	1.49	1.08	0.78	0.46	0.33	0.24
50 Year Event	3.94	2.84	2.35	1.70	1.23	0.88	0.53	0.38	0.28
100 Year Event	4.42	3.19	2.64	1.90	1.37	0.99	0.59	0.43	0.31

6.3 Wastewater Flow Allocation

Flow for the model is introduced with a flow file that describes the location of flow injection, the volume, and the pattern. The process of integrating the flow data requires consideration of flow allocation techniques. Common flow allocation techniques include manhole, length, inch-diameter-mile (IDM) and lateral counts.

In the hydraulic model, two techniques were used based on the dry and wet weather conditions. This allows the base flow to be separated from the defect flow for incorporation of the design storm and potential rehabilitation strategies. In the dry weather model, flow for a sub-basin was determined and allocated to the sub-basin manholes. This technique evenly distributes flow among the manholes in a basin. This technique is used when there is limited lateral information.

Defect flows are best represented by allocations based on defect data. In the absence of detailed defect inventories, defect flow is most often allocated by the parameter Inch Diameter Mile (IDM). This parameter is used as a surrogate for pipe surface area and is widely accepted in the industry. Note that this technique provides more defect loading in larger diameter pipes which are traditionally found in lower areas that may be subject to higher ground water factors.

Once the flows are allocated through the creation of flow files, the flow data is integrated through running the hydraulic model. During the run process, flow files are defined and used to create specific flow scenarios that are required for the analysis. Prior to analysis, the model must be calibrated by comparing the model predicted flow and the measured flow at the flow monitoring locations.

6.4 Model Calibration

Model calibration is a process that is used to adjust the physical system or the flow representations to closely match the measured results. Examples of adjusting the physical system include varying the diversion amounts between separate sub-basins or changing the roughness coefficients. Examples of adjusting the flow representation include changing the base flow volume or the diurnal flow pattern. The latter is frequently required since the flow is typically measured at a downstream location while the upstream flow injections would have higher and more pronounced peaks. The calibration process ends when the target calibration range is achieved or no further benefit comes from the adjustments.

Table 6-3 indicates the level of calibration that was achieved at each flow monitoring location. The locations were a mix of temporary and permanent installations with varying technologies. Site AZ01 could not be calibrated to within the target range of 25%. This range is selected because of the inherent inaccuracies of open channel flow measurements. It is likely that there is an undocumented diversion to the City of L.A. system upstream of this location. The

model may be modified to reflect any additional information as it is located and verified.

Table 6-3: Model Calibration Results

Flow Monitor	ADWF Calibration		Wet Wx Calibration	
	Volume (% Difference)	Peak (% Difference)	Volume (% Difference)	Peak (% Difference)
BC01	-0.2%	-1.3%	0.4%	-10.1%
LTM_15	0.0%	0.0%	0.0%	0.0%
LTM_18	0.0%	0.0%	0.0%	0.0%
LTM_19	0.0%	0.0%	0.0%	0.0%
ECR	0.5%	-2.5%	6.5%	4.0%
OAK	1.4%	-7.2%	3.0%	-11.6%
CH01	-0.2%	-8.4%	12.2%	6.2%
WIL	0.4%	-11.3%	7.9%	2.4%
CRE	-0.5%	-10.7%	2.4%	-15.6%
WH01	0.4%	-9.5%	0.6%	-17.4%
LC01	-0.1%	-3.3%	11.9%	20.8%
AZ01	0.1%	-4.3%	19.7%	30.8%
Overall	**0.2%**	**-4.9%**	**5.4%**	**0.8%**

The calibration results are well within the target accuracy range. The calibration results validate the hydraulic model to be used to determine hydraulic capacity. The high peak results at location AZ01 will represent a conservative approach to the capacity determination. Location GR01 is immediately downstream of a diversion at AZ01 so there is essentially no basin for GR01.

For each of the locations, a graph comparing the measured and modeled flow for the ADWF and Wet Weather Event are presented in Appendix D.

6.5 Model Scenarios

Hydraulic model scenarios are used to analyze different flow conditions and storm conditions. Scenarios were used in the hydraulic model to analyze the sewer system in 2009 and 2030 based on future projections. Design storms of different intensities were also modeled using scenarios.

6.5.1 Existing Dry Weather

In this scenario, flow from the 2009 flow measurement period was used to derive the ADWF for each of the twelve metered basins. Base flow volumes and patterns were developed for the weekday and weekend days. For locations LTM15, LTM18 and LTM19, the default flow pattern

was used along with the quarterly reported volume. The default flow pattern was determined from the temporary flow monitors and is shown in Figure 3-10. For all other locations, the flow patterns were determined from average hourly flows. This scenario represents the calibrated Existing Dry Weather conditions.

6.5.2 Existing Wet Weather

In this scenario, flow from the Existing ADWF is increased with the defect flow from the February 5, 2009 storm event. The defect flows are allocated based on the IDMs for each basin and segment, as discussed in Section 3. The model represents the wet weather conditions that occurred on February 5, 2009. This event had widespread responses to the rainfall event and is used as the basis of all other wet weather modeling. This scenario represents the calibrated wet weather conditions.

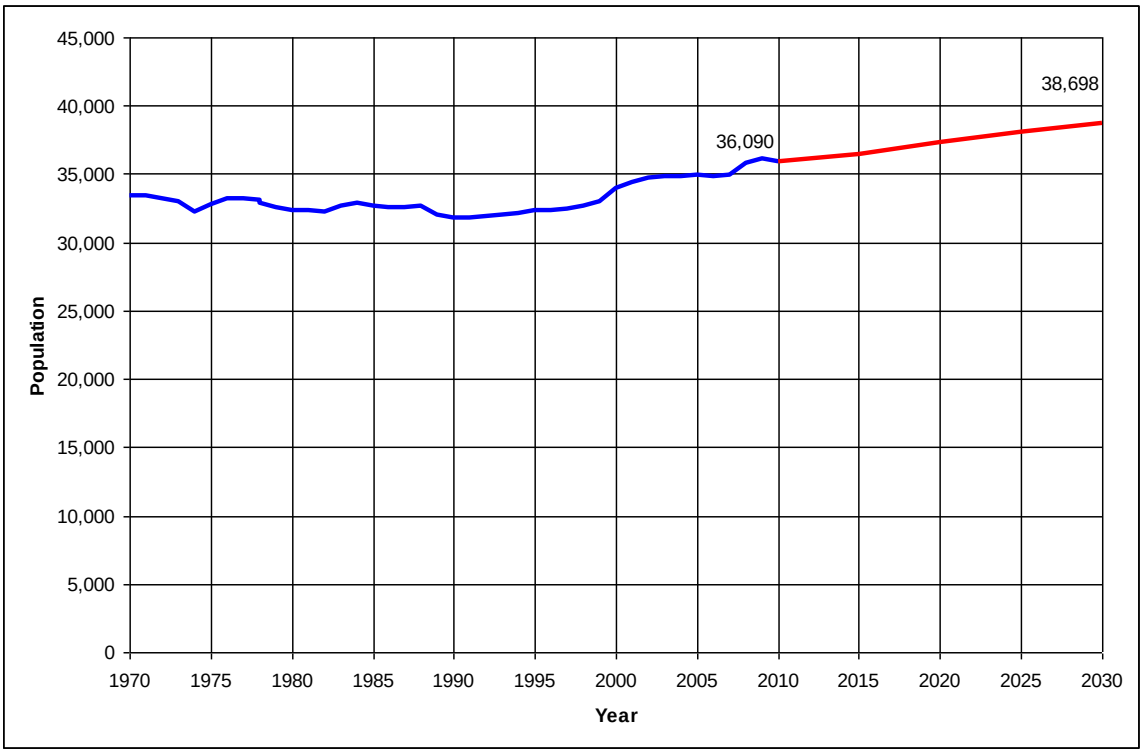
6.5.3 Design Storm Event

This scenario represents the predicted defect flows for a 10 year, 6-hour storm. This storm has a peak intensity of 0.38 inches per hour. During the February 5, 2009 event, the peak hourly intensity was 0.26 inches per hour. The design event has a peak intensity that is approximately 1.5 times the event that occurred on February 5, 2009. The existing flows and projected future flows were both modeled under this condition. The 10-Year, 6-Hour storm represents an intense storm event that has a 10% chance of occurrence. In the design storm event, the defect flows were increased by a factor of 1.5 and added to the baseline flows for the existing and future system.

6.5.4 Future Population and Projected Flows

To determine the additional population that may need to be served by the collection system, the California Department of Finance was consulted for the historical and current population. The Southern California Association of Governments (SCAG) database was used for future projections. The results of this analysis are shown in Figure 6-2. The City is in a built out condition, with very little annual increase in population. The total population predicted for the year 2030 represents just over a 7.2% increase over the 2009 population. Figure 6-2 represents 40 years of historical population and 20 years of projected population. Section 6.6.6 discusses the flow increase and results associated with the future flow scenarios.

Figure 6-2: Historical and Projected Population



The scenarios also consider redevelopment projects within the City. The proposed projects that were considered in this analysis are included in Appendix A. The proposed projects were not included in the existing model scenarios, but were included in the future scenarios. The flow projections for the redevelopment projects are discussed in Section 6.6.6.

There are areas adjacent to the City of Beverly Hills that generate wastewater flow that is conveyed through the City system. These areas were investigated to determine if future growth could occur that would impact wastewater flows in the City system. The areas outside the City that are tributary to the City's sewer system are essentially built out. The community immediately north of the City is the Bel Air – Beverly Crest community, which is within the City of Los Angeles. This area is built out. The only adopted specific plan for this area is the Mulholland Scenic Parkway Specific Plan which preserves Mulholland Drive as a scenic parkway and does not have any effect on sewer flows. The community immediately north of this area is the Sherman Oaks – Studio City

community. The only adopted specific plan for this area is the Ventura – Cahuenga Corridor Specific Plan which allows improvements to the commercial corridor to redevelop some of the lots and promote circulation. For all areas outside the City, the hydraulic model flows were assumed to increase at the same rate as flows within the City. The flow increases will account for moderate population increase and minor redevelopment within the areas upstream of the City. It should be noted that any significant new development or redevelopment that affects sewer flows upstream of the City's system will require a specific plan that addresses the existing sewer infrastructure from the specific plan area to the treatment plant.

6.6 Capacity Analysis

6.6.1 Performance Criteria

To evaluate the capacity of the system, the hydraulic model was used to determine the segments of pipe that exceeded the performance capacity. Built out cities generally use the depth to diameter ratio of 0.90 (90% of full depth) to flag capacity problems. This was used throughout the capacity analysis to be consistent with previous reports. New pipe design would not use this criterion.

A pipe was considered to have inadequate capacity if the depth was equal to or more than 90% of the pipe diameter. The following table indicates the classifications considered in the analysis of pipe capacity.

Table 6-4: Pipe Performance Criteria

Pipe Dia.	d/D Ratio		
(in)	0.50 to 0.75	0.75 to 0.90	≥ 0.90
< 18"	Watch	Schedule	Replace
≥ 18"	OK	Watch	Replace

Notes:
1. "Watch" indicates that special attention needs to be paid to increased flows that are tributary to this pipe. A proposed development may create a situation where the performance criteria is exceeded.
2. "Schedule" indicates that a replacement project needs to be considered but can be scheduled at some point in the future.
3. "Replace" indicates that an immediate project funding and design should begin.

To determine the capacity based replacement projects, the analysis first identified capacity deficient pipe segments in each scenario. Then logical sequences of collocated pipes were grouped to create candidate projects.

The focus of the project development is constructability. Single pipe segments in a reach of capacity deficient pipe segments are often replaced even though there is not a capacity restriction identified.

6.6.2 Pipe Capacity Deficiencies

To determine the hydraulic capacity of the system under a variety of conditions, flow scenarios were prepared as described in Section 6.5. Table 6-5 indicates the number of pipes that were found in each category for each scenario. The results of these analyses become the basis of identification of recommended projects.

Table 6-5: Pipe Capacity Deficiencies by Scenario

Scenario	d/D Ratio		
	0.50 to 0.75	0.75 to 0.90	>0.90
Existing (2009) ADWF	35	0	7
Existing (2009) Wet Weather Flow	58	17	34
Existing (2009) Flow with Design Storm	128	10	59
Future (2015) Flow with Design Storm	198	12	59
Future (2025) Flow with Design Storm	206	13	61
Future (2030) Flow with Design Storm	206	15	61
Future (2030) Flow with Design Storm with Defect Flow reduced by 50%	113	15	53

At the 2030 population plus the design storm event, there are 61 pipe segments (MH to MH) that exceed the depth to diameter ratio of 0.90. The last scenario reduces the defect flow by 50%, which reduces the predicted number of pipes exceeding the depth to diameter ratio to 53 pipes. All project recommendations will be based on the pipe size required to meet the 2030 design population plus design storm.

6.6.3 Existing Average Dry Weather Flow

As the system exists in 2009, there are 7 pipe segments (MH to MH) that are predicted to exceed a depth to diameter ratio of 0.90. All but one of these pipes is located in the southwest corner of the City, primarily along South Peck Drive, south of Olympic Boulevard. The predicted hydraulic grade line is beneath the ground surface so overflows are not predicted for these conditions. These pipes will become the first priority for replacement in the CIP.

In total there are 42 pipes that are predicted to flow more than 50% full. This indicates that 98% of the system operates at less than half full during the existing dry weather conditions. The system has excellent hydraulic capacity for these conditions. Exhibit 6-1 shows the pipes that were found to have a d/D ratio greater than 0.90 under the existing dry weather scenario.

6.6.4 Existing Wet Weather Flow

During the wet weather event that occurred on February 5, 2009 the calibrated model indicates that there would have been 34 pipes in the system that would have exceeded a depth to diameter ratio of 0.90. This represents approximately 7% of the system. Exhibit 6-1 shows the pipes that were found to have a d/D ratio greater than 0.90 under the existing wet weather scenario.

6.6.5 Design Storm Flow

To determine the capability of the existing system to convey the design storm event, an analysis was performed using the existing system with defect flows increased by a factor of 1.5. This resulted in the number of pipes exceeding a d/D ratio of 90% to be 59. During these conditions, overflows were predicted by the model. The predicted overflow area starts at Bedford Drive and extends east all the way down the alley north of Olympic Boulevard to the City of L.A. connection at Gregory Way, location GR01. Exhibit 6-1 shows the pipes that were found to have a d/D ratio greater than 0.90 under the design storm scenario.

6.6.6 Future Flows based on Census projections

This ultimate future flow scenario represents the sanitary flows increased by 7.2% from the existing flows, which accounts for the population projections in 2030. The system is also loaded with the design storm, a 10 year, 6 hour storm event. This scenario represents the upper bounds of the capacity testing. During this scenario, the model predicted flooding along Bedford Drive to the Gregory Way Trunk and is also predicted to occur along Oakhurst Drive. Note that in this condition the number of hydraulically challenged pipes increased from 59 in the existing design storm scenario, to 61. This indicates that 2.8% of the system is predicted to exceed a d/D of 90%. Exhibit 6-1 shows these pipelines.

Although the scenario for 2030 represents the worst case scenario, scenarios for 2015 and 2025 were also run using similar projections for population and flow. The predicted population and flow increases for all future scenarios are shown in Table 6-6.

Table 6-6: Population and Flow Projection for Future Scenarios

Year	Population	Increase
2009	36,090	0.0%
2015	36,835	2.1%
2025	38,077	5.5%
2030	38,698	7.2%

The proposed redevelopment projects were included in all future scenarios (2015, 2025, and 2030). The redevelopment projects and the proposed flow projections are shown in Table 6-7. The flow projections are based on flow factors from the Los Angeles County Sanitation District. The specific flow factors used for the redevelopment projects are also included in Appendix A.

Table 6-7: Redevelopment Project Flow Projections

Project Name/Location	Description	Area	Unit	Daily Flow (gpd)
New Century Plan 1	Shopping Center	360,000	SF	54,000
	Office	106,000	SF	21,200
	Condos	262	DU	40,872
9900 Wilshire	Condos	235	DU	36,660
	Retail	11,656	SF	1,748
	Restaurant	4,200	SF	4,200
Hilton Project [1]	Hotel	N/A	N/A	N/A
SunCal Project	Condos	177	DU	27,612
	Retail	100	SF	150
	Club	20,000	SF	2,500
Montage Hotel	Hotel	201	ROOMS	25,125
	Condos	20	DU	3,120
	Banquet	8,630	SF	3,021
	Spa	19,710	SF	11,826
	Retail	5,265	SF	790
Wallis Annenberg Center for the Performing Arts	Perf. Hall	2,100	SF	263
	Classroom	1,750	SF	263
	Lounge	730	SF	91
	Admin Office	3,900	SF	780
	Theater	7,000	SF	875
Beverly Hills Gateway Project	Commercial	45,000	SF	6,750
	Office	45,000	SF	9,000
9750 Wilshire Blvd	Office	101,000	SF	20,200
			Total	*271,046*

[1] The Hilton Project is a revitalization of an existing hotel. This will not alter sewer generation from the existing condition.

6.7 Recommendations

To alleviate the hydraulic capacity problems identified through the modeling, three primary projects were identified, as shown in Table 6-9. These projects would remove the majority of capacity restrictions, although there are still a few individual pipe segments in the system that are restricted. The remaining pipelines that are restricted are shown in Table 6-8, with the recommended upsized diameters. These should be watched and replaced when the opportunity arises through routine maintenance or street rehabilitation.

Table 6-8: Restricted Pipelines Not Included in Capacity Replacements

Pipe Segment ID	Length (LF)	Slope	Existing Dia (in)	Recommended Dia (in)
SSP-15-01122	285.94	0.0096	8	10
SSP-15-01111	116.49	0.0118	8	10
SSP-15-01108	20.10	0.0194	10	12
SSP-07-01071	228.73	0.0044	15	18
SSP-12-01890	185.91	0.0107	6	8
SSP-12-01904	198.01	0.0428	6	8

Three projects were identified to address the majority of the predicted capacity restrictions. The three pipelines were found to be deficient under several of the model scenarios. The recommended diameters have been sized to accommodate the 2030 flows with the design storm impacting the system. This represents the worst case flow scenario for the pipelines. The hydraulic model was run with the recommended diameters to verify that the pipe capacity restrictions had been alleviated. The project parameters are shown in Table 6-9, and graphically in Exhibit 6-2. These projects, and their estimated costs, have been included in the CIP.

Table 6-9: Recommended Projects

Priority	Street	Existing Diameter	Proposed Diameter	Total Length (LF)
1	Peck Drive	8"-21"	12"-27"	5,891
2	Gregory Way	24"-30"	30"-39"	4,286
3	Oakhurst Drive	10"-15"	15"-21"	2,915

Section 7—Pipeline and Manhole Condition Assessment

7.1 Background

The pipeline and manhole condition assessment consisted of evaluations of a total of 77,106 LF of sewer pipe and 328 manholes, which is approximately 15% of the sewer system. The condition assessment consisted of a closed-circuit television (CCTV) inspection and analysis of the selected sewer mains and their associated manholes. The selected sewer mains were used as a statistical sample for the entire sewer system. The inspections were completed in three phases. The first phase of inspections, Area 1, was conducted between December 1, 2008 and January 5, 2009. The second phase of inspections, Area 2, was conducted between December 16, 2009 and February 11, 2009. The third phase of inspections, Area 3, was conducted between January 6, 2009 and February 4, 2009. All of the pipelines and manholes inspected during the condition assessment are shown on Exhibit 7-1. Recommendations from the condition assessment are incorporated into the CIP. Separate reports for each of the three areas of the CCTV condition assessment have been prepared and submitted to the City. Each report includes detailed logs of each pipe segment and manhole inspected. A summary of the results is included herein.

Although 15% of the system was inspected in 2009, 1% of the system was also inspected in 2007 by Psomas Engineering. The pipelines inspected in 2007 by Psomas Engineering have also been shown in Exhibit 7-1.

7.2 Pipeline Replacement and Rehabilitation Methods

Recommendations as a result of the condition assessment include open-trench replacement and trenchless technology methods. Trenchless technologies provide techniques for the installation or renewal of underground utilities with a minimum disturbance of the surface. Utilizing trenchless technologies usually represents a significant savings when compared with open trench methods.

There are a number of trenchless and semi-trenchless methods in practice today for pipeline rehabilitation. The technologies currently in use which were considered for structural and semi-structural pipe rehabilitation and/or replacement include:

- Cured-in-place Pipe (CIPP) (Greenbook 500-1.4) – The CIPP lining process involves inserting and inverting a resin-saturated felt tube into the

existing pipe (like a very large inside-out sock). The tube is inflated to conform to the host pipe and hot air or water is circulated throughout the tube to cure (harden) the resin. When the curing process is complete, a new pipe is created that no longer has the joints, cracks, and holes that allowed infiltration and roots to enter the pipe causing operational problems such as blockages and overflows. The finished product has a 50-year design life. This method can also be used to make spot repairs, or sectional spot repairs.

- Folded Pipe (fold and form, deformed/reformed) (Greenbook 500-1.7) – A pre-fabricated HDPE (high density polyethylene) pipe, that has been collapsed in upon itself so that it is U-shaped, is pulled through an upstream manhole by a cable from the downstream manhole. Heat and pressure are applied to the U-shaped pipe causing it to revert to its original round profile.

- Machine Spiral Wound PVC Pipe Liner (Greenbook 500-1.13) – Panels of PVC are spirally wound into the existing pipe. This can be performed on pipe diameters from 6-inch to 30-inch. The advantage to this method is no bypassing of sewer is needed during lower flow conditions.

- Sewer Lateral/Main Connection Re-Establishment (Greenbook 500-1.4.7) – After a sewer line is rehabilitated with a trenchless liner the laterals are restored with robotically controlled cutting devices. These robots cut out a round coupon from the new liner at the lateral connections to allow flow into the pipe.

- Sewer Lateral/Main Connection (SLC) – This uses a specially shaped liner, which looks like a top hat (without the top of the hat) or inverted 'T.' The stem of the top hat forms the section of liner that is placed into the lateral pipe while the top hat crossbar sits against the inner wall of the main. Resin impregnation of the liner material allows it to be positioned using a specially designed remote control robotic installer and held in position for curing of the resin. Once cured, the robot and retention system are removed leaving the 'T' – shaped liner forming a seal across the lateral/main joint. The top hat stem can be installed as deep into the lateral as necessary.

CCTV inspection videos often show a pipe segment to be in overall good condition, with the exception of one location where there is a large fracture or

other failure in the pipe. For locations such as this, it is often recommended to perform a spot repair on the failure location. Methods for performing spot repairs include open trench replacement as well as trenchless technologies. The trenchless technologies most commonly in use today for performing spot repairs include:

- Cured-in-place Pipe (CIPP) (Greenbook 500-1.2) – Spot repair CIPP involves the same steps as a CIPP of a full pipe segment. Spot repair CIPP can range from 3 feet to 30 feet.

- Trenchless Sleeve – This method involves inserting a prefabricated stainless steel sleeve into the host pipe. The sleeve is located over the failure and expanded outward to provide a structurally sound and sealed pipe section.

See Section 9 for the recommended methodology for the City of Beverly Hills sewer pipeline rehabilitation.

7.3 Pipeline Condition Assessment

The purpose of the pipeline inspections was to assess the condition of a statistical sample of the pipelines, determine the required remedial action if necessary (repair or replacement), and to assign a priority level to each recommendation. The following is an extrapolation from the inspection data and analysis for the purpose of determining the budgetary needs for repairs to the system as a whole.

7.3.1 Statistical Analysis

The sample population consisted of approximately 15% of the total system with a distribution based on nominal pipe diameter, pipe material, and year of construction. The sample was chosen to closely represent the system as a whole. Tables 7-1, 7-2, and 7-3 compare the sample to the system as a whole based on the selection criteria.

Table 7-1: Sample Population by Diameter

Diameter (in)	System Footage	Percent of System	Sample Footage	Percent of Sample
Unknown	0	0.00%	143	0.19%
6	39,277	7.40%	5,683	7.37%
8	348,636	65.68%	48,402	62.77%
10	61,471	11.58%	8,563	11.11%
12	23,828	4.49%	3,751	4.86%
15	29,799	5.61%	5,664	7.35%
18	12,964	2.44%	1,292	1.68%
21	3,522	0.66%	695	0.90%
24	7,214	1.36%	1,269	1.65%
27	317	0.06%	328	0.43%
30	2,525	0.48%	308	0.40%
32	225	0.04%	42	0.05%
33	635	0.12%	0	0.00%
36	408	0.08%	966	1.25%

Table 7-2: Sample Population by Pipe Material

Material	System Footage	Percent of System	Sample Footage	Percent of Sample
Unknown	4,507	0.85%	143	0.19%
Concrete	141,749	26.70%	16,896	21.91%
Vitrified Clay	384,269	72.39%	60,025	77.85%
Cast Iron	35	0.01%	0	0.00%
Ductile Iron	261	0.05%	42	0.05%

Table 7-3: Sample Population by Year Constructed

Decade	System Footage	Percent of System	Sample Footage	Percent of Sample
Unknown	15,119	2.85%	1,903	2.47%
1920s	6,611	1.25%	1,237	1.60%
1930s	255,403	48.11%	44,969	58.32%
1940s	39,223	7.39%	2,251	2.92%
1950s	56,959	10.73%	3,414	4.43%
1960s	68,081	12.83%	9,967	12.93%
1970s	76,999	14.51%	11,912	15.45%
1980s	8,617	1.62%	1,453	1.88%
1990s	1,602	0.30%	0	0.00%
2000s	1,099	0.21%	0	0.00%
2010s	1,107	0.21%	0	0.00%

7.4 Manhole Replacement and Rehabilitation Methods

If manholes are in severely poor condition due to structural failure, complete replacement is necessary. If the structure of the manhole is in moderate condition and rehabilitation is an option, there are two rehabilitation methods to be considered:

- PVC Liner (Greenbook 500-2.2) – PVC lining provides a monolithic cured resin lining for a manhole for sealing infiltration and root penetration. Prior to installation, the structure must be thoroughly cleaned and grouted. On site, a prefabricated liner is fully saturated with an epoxy resin that will bond to wet or dry surfaces. The liner is lowered into the structure and inflated and cured with a removable bladder filled with hot steam and high pressures. The bladder is removed and pipe openings are restored.

- Polyurethane Liner (Greenbook 500-2.4) – Polyurethane lining provides protection for the structure against caustic gases that arise from the sewer that attack concrete and mortar. The structure is cleaned and pressure blasted to achieve a clean structure surface. Some polyurethane products are applied directly to the cleaned concrete, while some products require an epoxy base coat prior to the polyurethane application.

7.5 Manhole Condition Assessment

The purpose of the manhole inspections was to assess the overall condition of the manholes, determine the required remedial action if necessary (repair or replacement), and to assign a priority level to each recommendation. The

manhole inspection consisted of 328 total manholes being inspected. Of those facilities, an appropriate recommendation was made for each manhole based on the inspection including, but not limited to: manhole rehabilitation, or replacement. The assigned priority levels range from “red flag”, which requires immediate attention, to “urgent”, to “low priority”.

7.5.1 Statistical Analysis

The sample population consisted of approximately 15% of the total system. The sample was chosen to closely represent the system as a whole. The manholes inspected were connected to the pipelines inspected, which is shown in Exhibit 7-1. Table 7-4 compares the sample to the system as a whole based on the repair or rehabilitation method.

Table 7-4: Sample Manhole Population By Repair Method

Repair Type	System Manholes	Percent of System	Sample Manholes	Percent of Sample
No Action	1,448	66.46%	218	66.46%
Cleaning	0	0.00%	0	0.00%
Rehabilitation	711	32.62%	107	32.62%
Replacement	20	0.91%	3	0.91%
TOTAL	**2,178**		**328**	

The high priority, or “urgent”, projects that resulted from the manhole condition assessment have been incorporated into the CIP. Refer to the Area 1, 2 and 3 CCTV and Manhole Inspection Reports for detailed results. Refer to Section 9 for specific project recommendations.

7.6 Recommendations

The CCTV inspection consisted of 77,106 LF of pipe and 328 manholes. Of those facilities, an appropriate recommendation was made for each pipe segment based on the CCTV inspection including, but not limited to: replacement, rehabilitation with trenchless technologies, or spot repair. The assigned priority levels range from “red flag”, which requires immediate attention, to “urgent”, to “low priority”. From the results of the CCTV inspection, approximately 39% of the pipelines inspected were recommended for some type of rehabilitation, such as CIPP or spot repairs. The inspections also showed a general correlation between pipe material or age, and the condition of the pipe. For example, most pipes that required repair or rehabilitation were constructed in the 1930’s. Furthermore, several of the concrete pipes were found to require rehabilitation using CIPP.

The high priority, or "urgent", projects that resulted from the condition assessment have been incorporated into the CIP. The low priority projects were included in the "Reinspect in Five Years" category. See Section 9 and the Sewer Assessment Report for detailed project information.

As discussed previously, the facilities inspected during the CCTV inspection represent a statistical sample of the City's sewer system. One of the purposes of the inspection was to use the statistical sample to assess the City's system as a whole. Table 7-5 shows the relation to the facilities that were recently inspected, and a rehabilitation estimate for the rest of the sewer system.

Table 7-5: Estimated Costs for Entire System Rehabilitation

Repair Type	Sample Footage	Sample Repair Costs	Expected System Footage	Expected System Costs
Reinspect in Five Years	43,844	$0	301,837	$0
Complete CIPP Lining	19,430	$1,834,382	133,763	$12,628,491
Sectional CIPP Lining	188	$4,900	1,295	$33,733
Complete Open Trench Replacement	1,564	$477,800	10,769	$3,289,333
Further Inspection Required	434	$0	2,988	$0
Cleaning	2,398	$0	16,509	$0
Spot Repair	8,965	370,714	61,719	$2,552,117
Does Not Exist - No Action	282	$0	1,941	$0
Subtotal	*77,106*	*$2,687,796*	*530,821*	*$18,504,000*
Construction Contingencies		*$537,559*		*$3,700,800*
Engineering and Administration		*$403,169*		*$2,775,600*
Total		**$3,629,000**		**$25,000,000**

Repair Type	Sample Manholes	Sample Repair Costs	Expected System Manholes	Expected System Costs
No Action	218	$0	1,448	$0
Cleaning	0	$0	0	$0
Rehabilitation	107	$353,550	711	$2,347,658
Replacement	3	$32,000	20	$212,488
Subtotal	*328*	*$385,550*	*2,178*	*$2,560,000*
Construction Contingencies		*$77,110*		*$512,000*
Engineering and Administration		*$57,833*		*$384,000*
Total		**$520,000**		**$3,500,000**
TOTAL		**$4,149,000**		**$28,500,000**

Table 7-5 provides an estimate of the construction costs required for rehabilitation of the remaining 85% of the system based on the 15% sample inspected in 2009. It is estimated that $28.5M will be required to upgrade the entire system.

The 1% of the sewer system inspected by Psomas Engineering in 2007 was taken into consideration for the CCTV Inspection Program in the CIP. The CCTV Inspection Program included in the CIP will include inspecting the remaining 84% of the system in two phases. Details for the CCTV Inspection Program with exhibits are included in the CIP. The results from the CCTV Inspection Program in the CIP will verify / adjust the rehabilitation requirements and expected construction costs for the portions of the system that have not been inspected.

Section 8—Conclusions and Recommendations

8.1 Conclusions and Recommendations

The primary conclusions reached due to the analysis and evaluations performed as a part of this project include:

- The City's gravity sewer collection system is an aging and deteriorating system. Approximately 50% of the pipeline system is 80 years old and has reached the end of its useful life.
- Approximately 27% of the City's pipelines are concrete pipelines. Concrete will degrade quickly due to hydrogen sulfide gases.
- Of the 77,000 lineal feet of sewer pipe (15% of the system) that was inspected with video for this project, approximately 39% was determined to require rehabilitation or replacement.
- The temporary flow monitoring conducted as a part of this project indicated that above-average inflow and infiltration is occurring within the system. The flow monitoring was not performed specifically as an inflow and infiltration analysis, however rainfall events were captured and it was concluded from the data that the primary source of defect flow is inflow. Basins LC01, ECR, AZ01 and WH01 are the most impacted by inflow and infiltration.
- The current capacity of the City's collection system is adequate to accommodate existing dry-weather flows. However, due to above-average inflow, there is potential for pipelines within the system to surcharge during a wet-weather event.
- The City is essentially built-out. The population growth projection to build out is an increase of approximately 7%. The population growth does not have significant impact on the capacity of the existing system, however a storm event does.
- The model analysis determined three primary projects that would eliminate most of the capacity restrictions in the system. The three projects are located on Peck Drive, Gregory Way, and Oakhurst Drive.

The recommendations to mitigate these conclusions are as follows:

- Rehabilitate or replace the high-priority pipelines and manholes as identified in Areas 1, 2 and 3 of the CCTV investigation.
- Initiate a CCTV program to complete the investigation of the remaining 84% of the system (1% was inspected in 2007 and 15% was inspected in 2009) over the next two years.
- Begin to budget for the rehabilitation and replacement of the remaining portions of the system. Special attention should be paid to pipelines 80 years in age and older and the concrete pipelines.
- Conduct a specific Inflow and Infiltration Study to identify the source of the defect inflow and take actions to reduce the amount of inflow into the system. Basins LC01, ECR, AZ01 and WH01 (in order of importance) are the highest priority.
- Implement the three pipeline projects to reduce the capacity restrictions in the system and decrease the potential for a sewer spill.

Section 9—Capital Improvement Program

9.1 Introduction

Projects that have been included in the Capital Improvement Program (CIP) were primarily based on the assessment of the conditions of pipelines and manholes from Closed Circuit Television (CCTV) inspection and results from hydraulic modeling. The CCTV inspection identifies particular deficiencies associated with the inspected pipelines and manholes and recommends the appropriate improvement method for each situation. The CCTV recommendations may include the following: pipeline spot repair, trenchless rehabilitation, open trench replacement, manhole rehabilitation, or manhole replacement. The hydraulic modeling effort resulted in the identification and/or confirmation of undersized pipelines and the recommended diameters for upsizing critical pipeline segments. Projects for system studies and further CCTV inspections as well as budgeting for future expected improvements has also been included. The CIP identifies projects for implementation over the next 10 years (through 2020).

9.2 Priority Criteria and Project Cost

Projects were generally prioritized in the following order:

- Projects resulting from CCTV inspections (condition assessments)
- CCTV Inspection Program
- Projects resulting from hydraulic modeling results (pipe capacity deficiencies)
- System studies (Inflow and Infiltration)
- Repair and Rehabilitation Program

Each of the projects within the first five years of the CIP is considered a critical project, however it is not reasonable for the City to coordinate, fund, and complete a combined $14M of sewer projects in the first year or two of the CIP. Therefore the projects were prioritized based on certain factors. The pipeline deficiencies occur during a design storm event, but not during average dry weather flows. There was a high amount of inflow and infiltration observed in the sewer system, which contributes to the pipelines being deficient in capacity. It is recommended that the City conduct an inflow and infiltration study and complete

the CCTV investigation of the entire system prior to implementing the capacity improvement projects. The reason for prioritizing these projects is to identify the locations of inflow and infiltration and mitigate the problems, in the hopes that the capacity upgrades will no longer be required.

The most urgent projects were those resulting from the CCTV inspections already conducted. These recommendations are based on the severity of actual field observed conditions. The CCTV Inspection Program was assigned the second highest priority so the City could complete the inspection of the entire sewer system and assess the level of funding necessary to complete the repair and rehabilitation of the system.

Projects resulting from hydraulic modeling results have been assigned the third highest priority since they are based on results from a computer simulation of the system. Although the hydraulic model is considered an accurate representation of the City's system, the model results should be field verified by a CCTV inspection prior to performing the recommended improvements. As part of the CCTV Inspection Program, the pipe segments inspected first will be those determined to be deficient by the hydraulic model.

Special consideration has been given to CIP Project 07-000, Infiltration and Inflow Study, since determining the presence of cross connections and sources of infiltration and inflow were considered a high priority.

The Repair and Rehabilitation Program would be implemented towards the latter half of the CIP. The Repair and Rehabilitation Program will be the resulting projects from the CCTV Inspection Program, which will represent a considerable cost. The purpose of completing the CCTV Inspection Program early in the CIP is to allow the City to take the necessary measures to provide funding for these projects. For the purposes of this CIP, the costs used for the Repair and Rehabilitation Program were based on the statistical analysis of the 2009 CCTV inspection, discussed further in Section 7 of the Master Plan.

The projects have been distributed over a 10-year period. The estimated construction cost for each improvement project was based on the construction cost parameters that are shown in the CIP. Construction cost parameters are based on recent construction bid results of similar projects as well as engineering experience. Construction contingencies as well as engineering and administration fees have been included in the total project cost.

9.3 Capital Improvement Program Summary

The 10-year CIP is shown in Table 9-1. The CIP includes the project identification, project title, implementation year, and estimated construction cost. Detailed project sheets with exhibits have been included in a separate report. The total cost of the 10-year CIP is $39M, which includes the repair and rehabilitation of the entire system over the 10-year period.

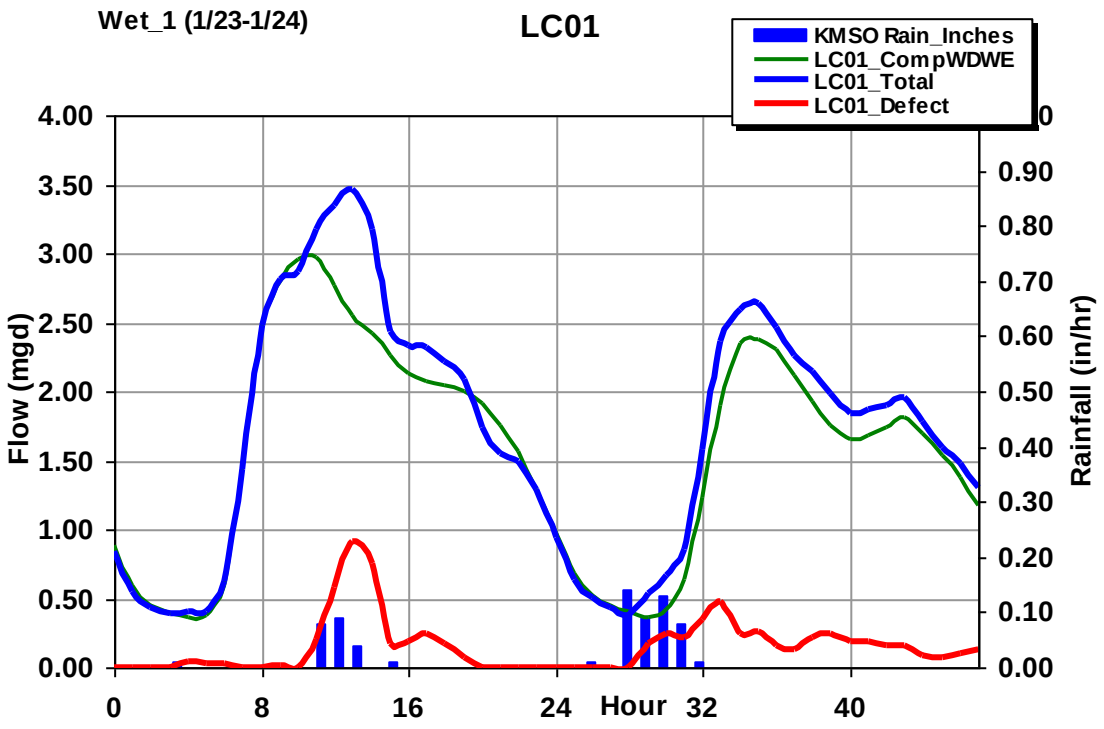
Table 9-1: 10-year Capital Improvement Program
[INSERT]

Appendix A: General Plan Land Use Map and Redevelopment Projects

Appendix B: Temporary Flow Monitoring Report

Appendix C: Wet Weather Analysis for Each Flow Monitor

Appendix D: Hydraulic Model Calibration Data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LC01_CompWDWE
LC01_Total
LC01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
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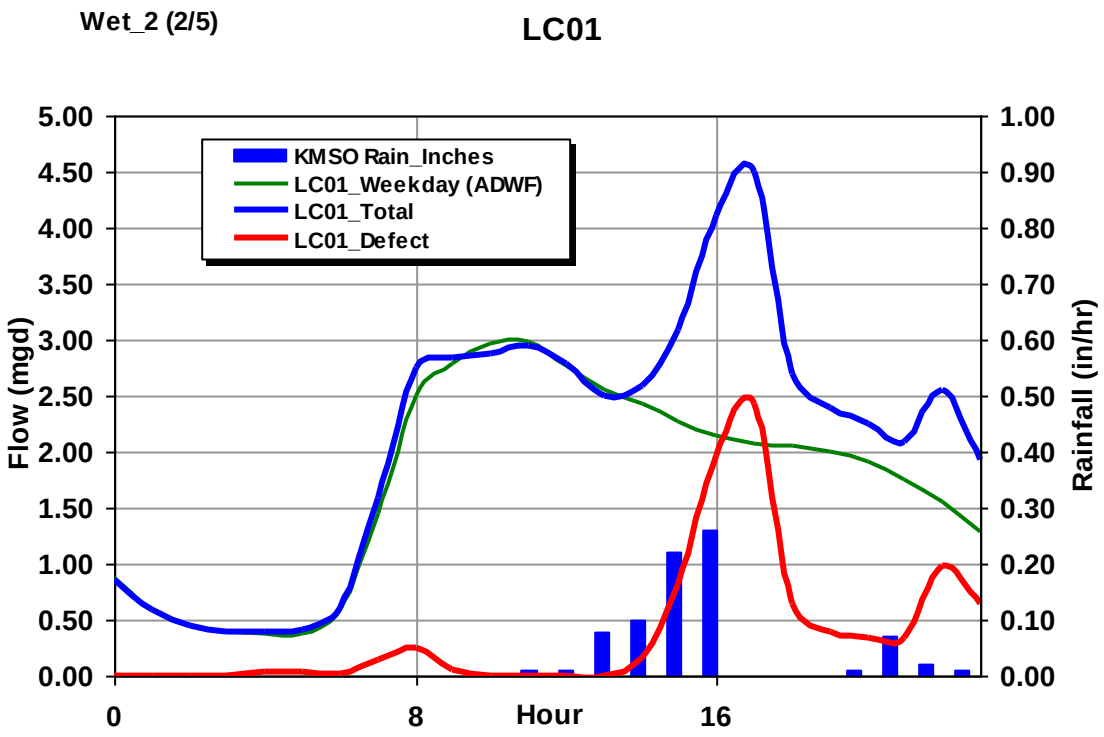
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LC01_Weekday (ADWF)
LC01_Total
LC01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
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4.50
4.00
3.50
3.00
2.50
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1.50
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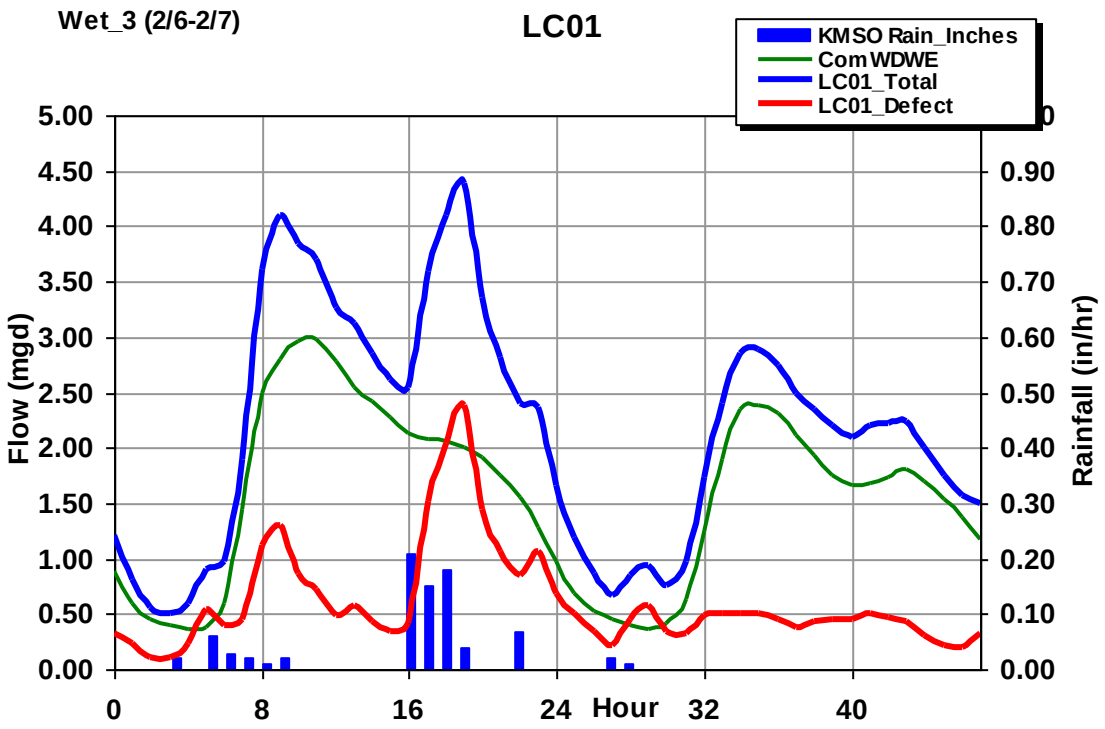
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LC01
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ComWDWE
LC01_Total
LC01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
5.00
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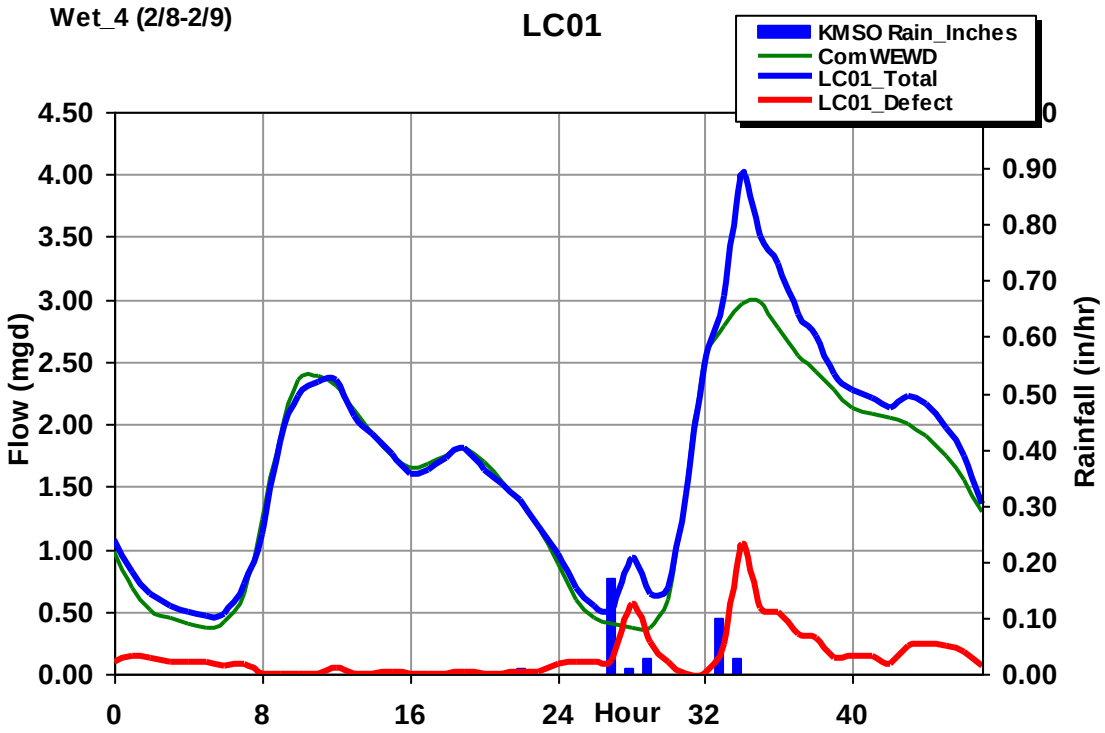
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ComWEWD
LC01_Total
LC01_Defect
Flow (mgd)
Rainfall (in/hr)
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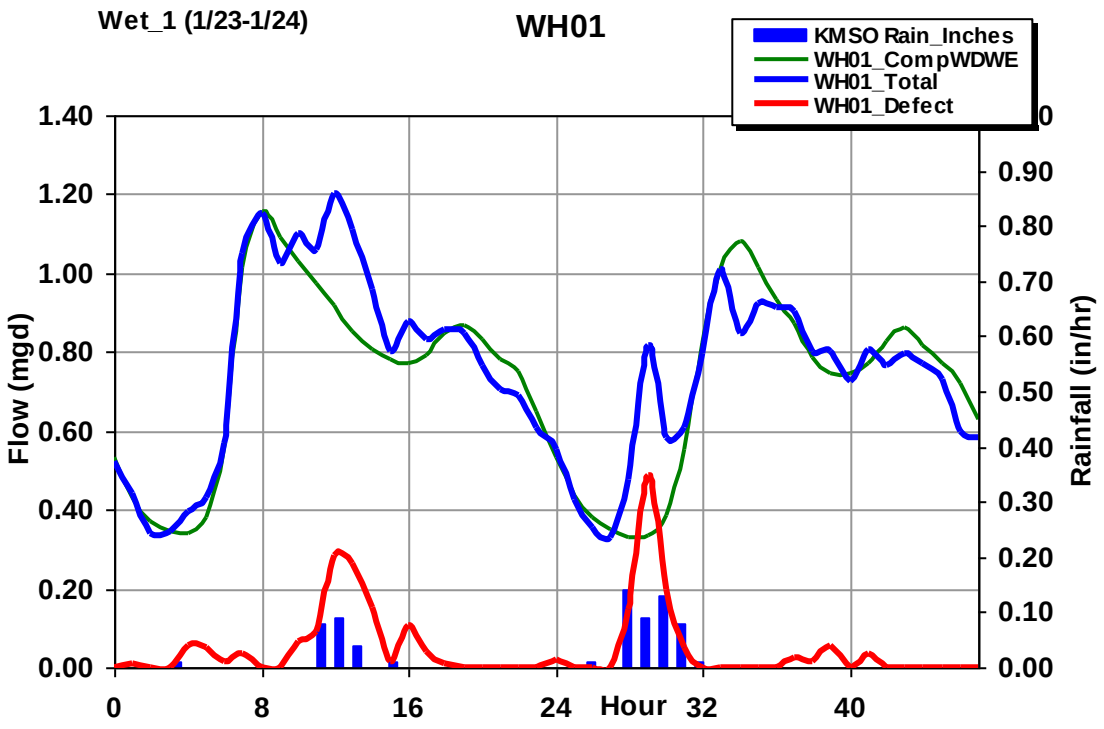
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WH01_Defect
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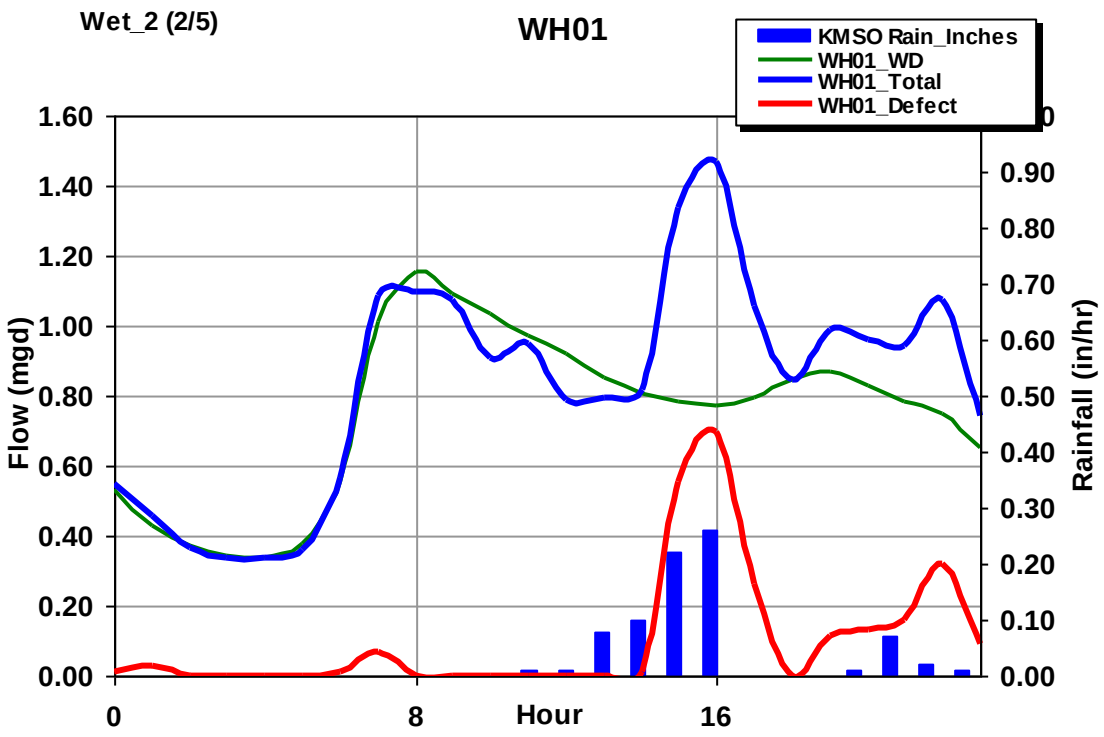
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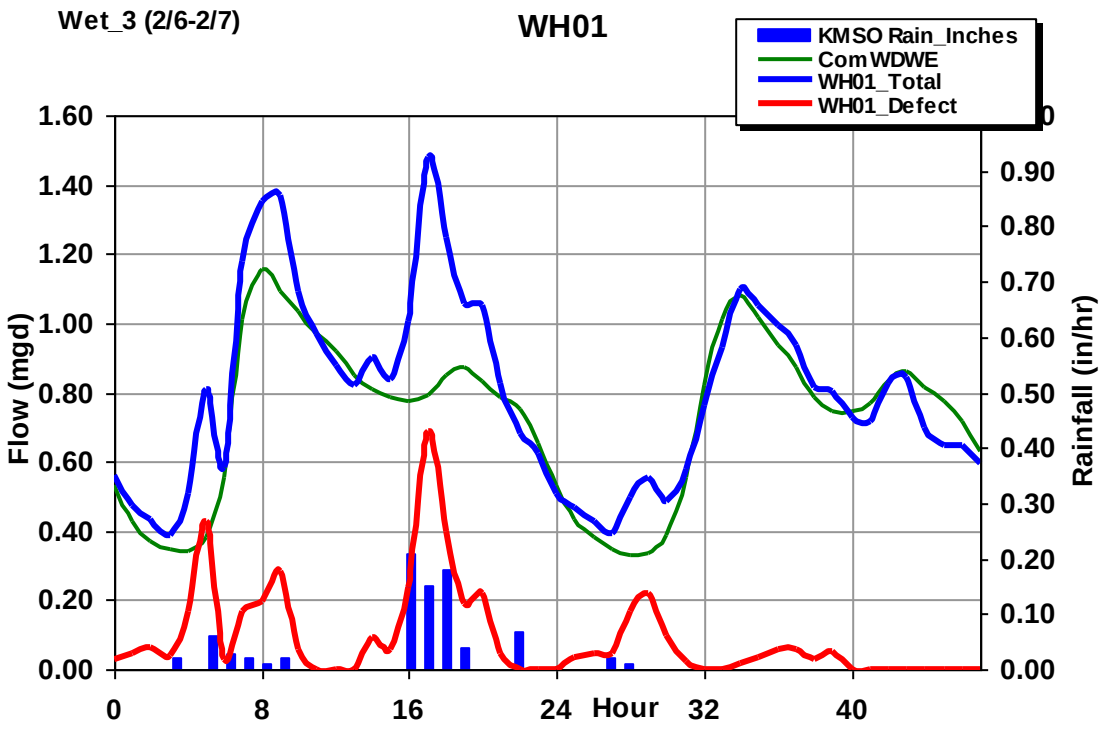
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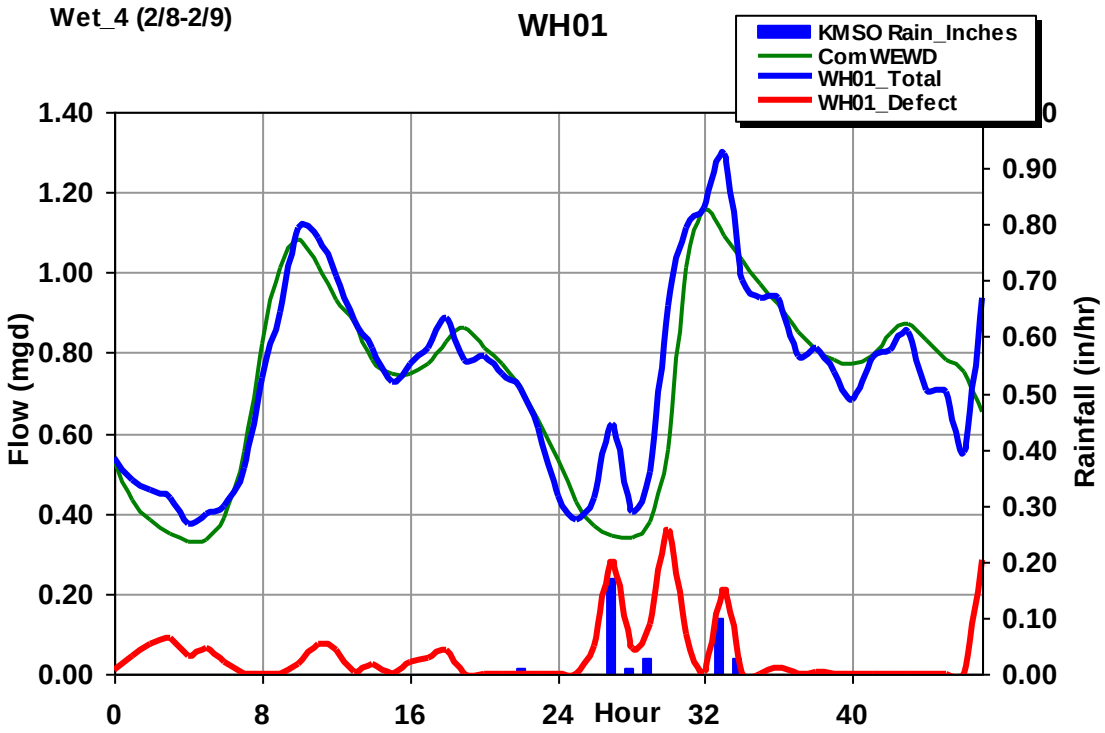
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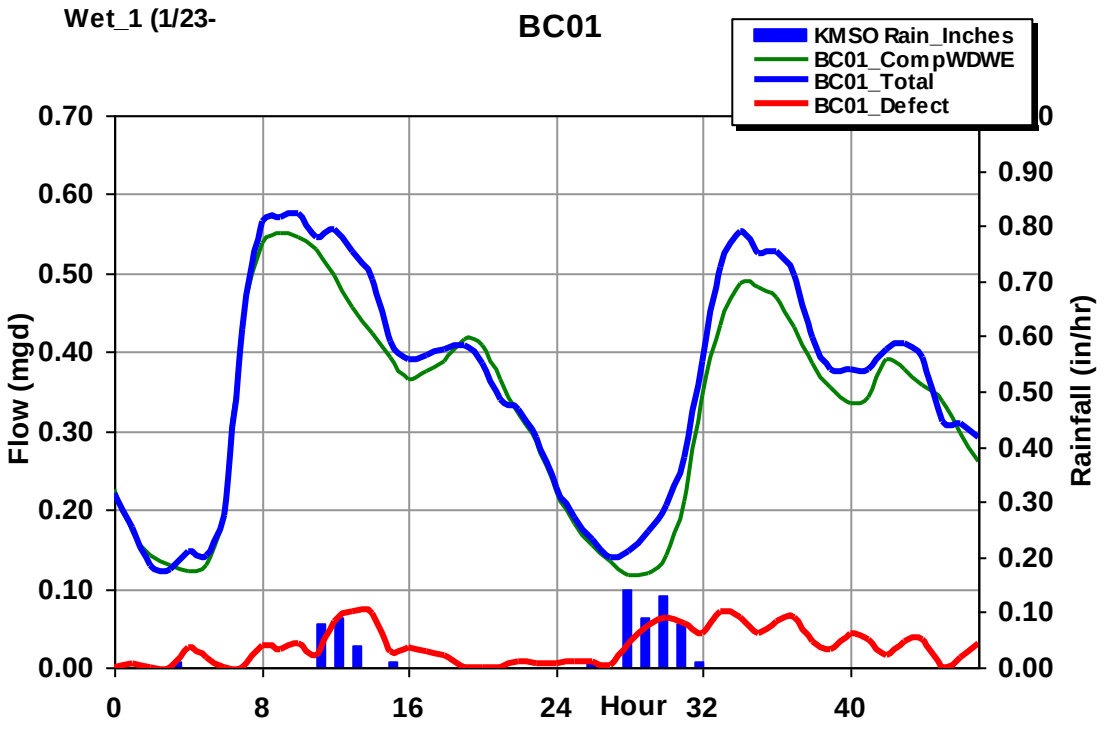
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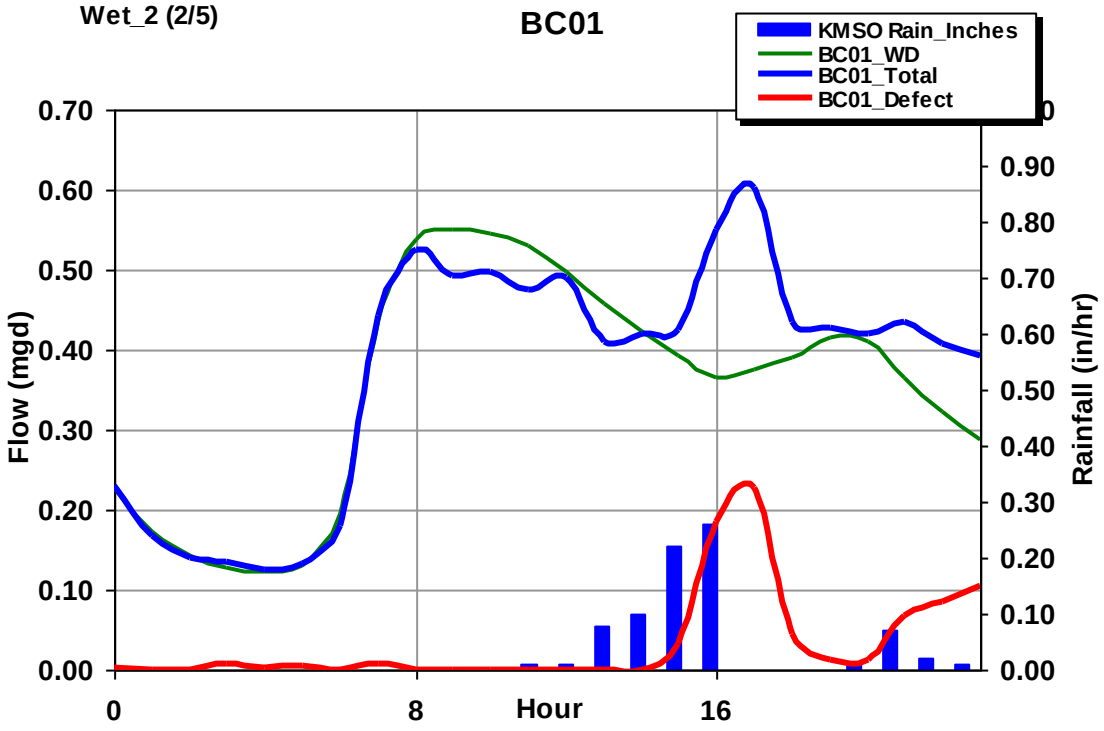
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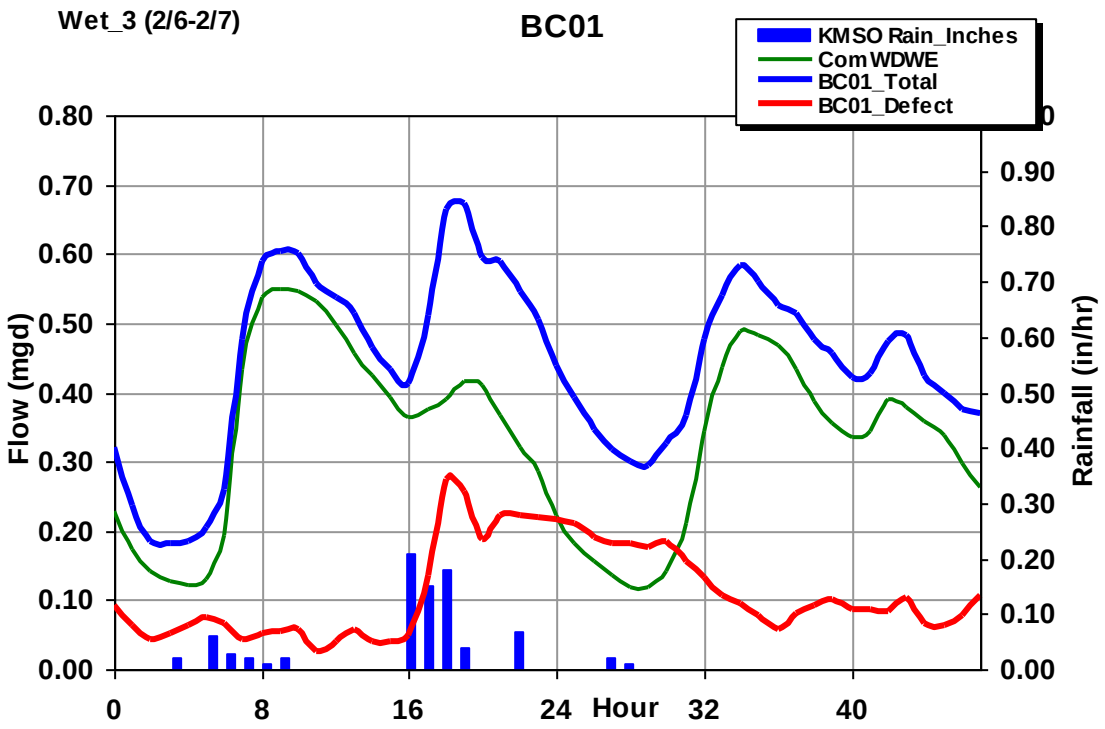
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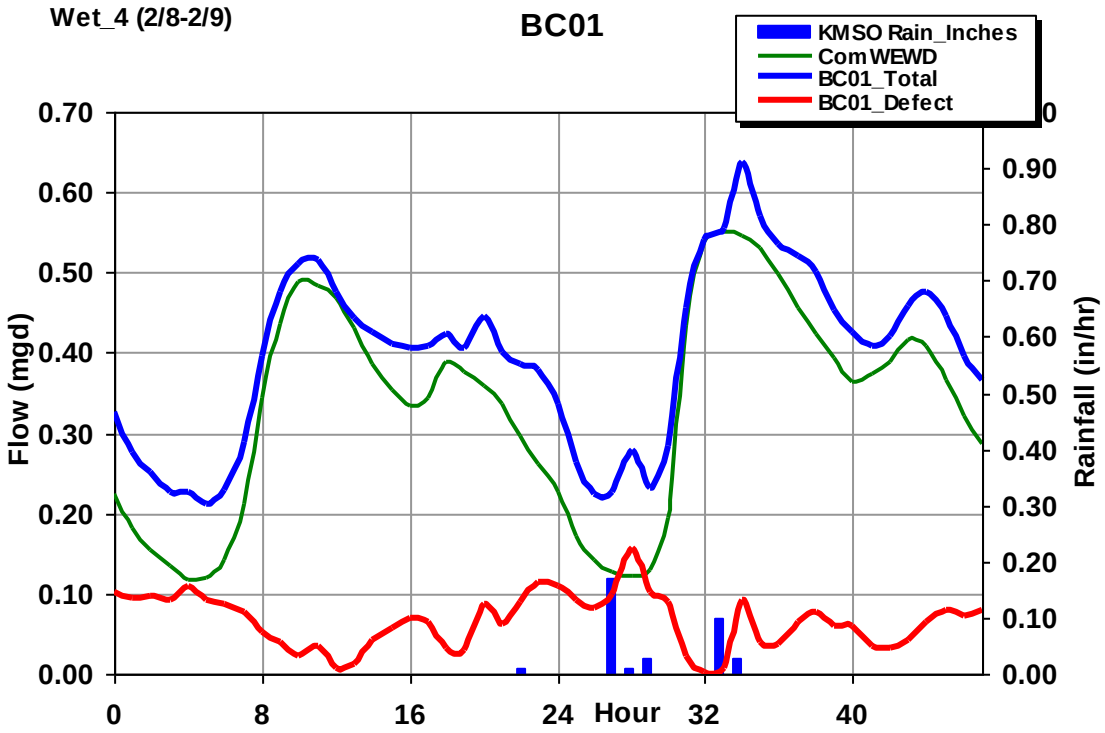
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BC01_Defect
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Rainfall (in/hr)
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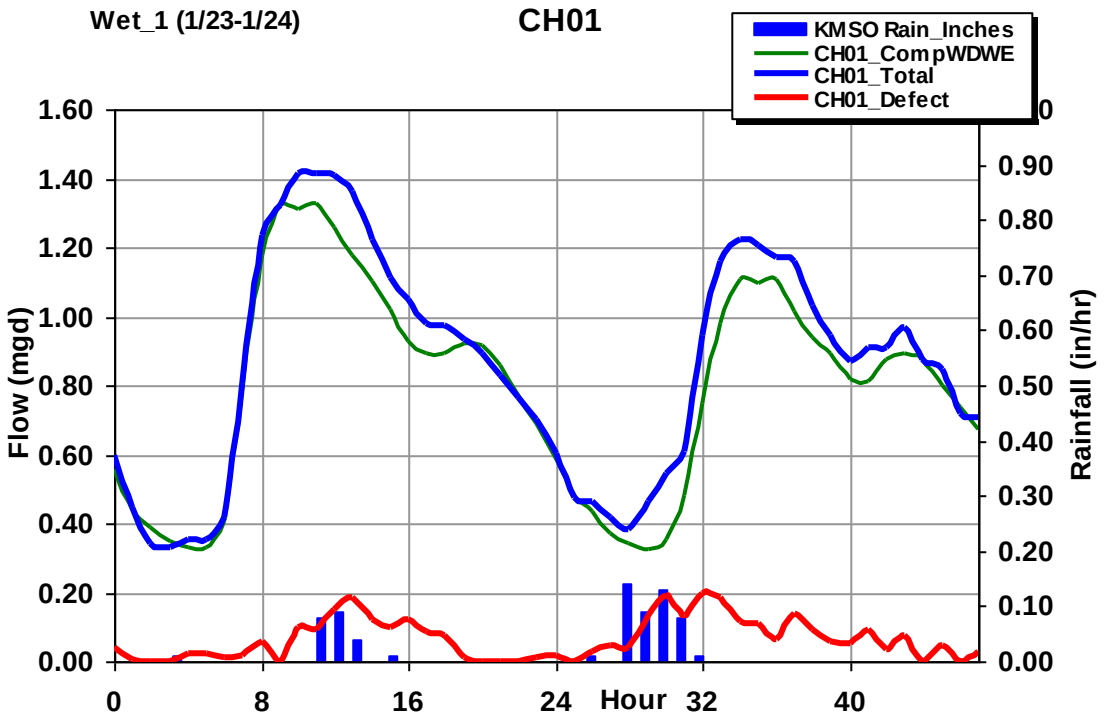
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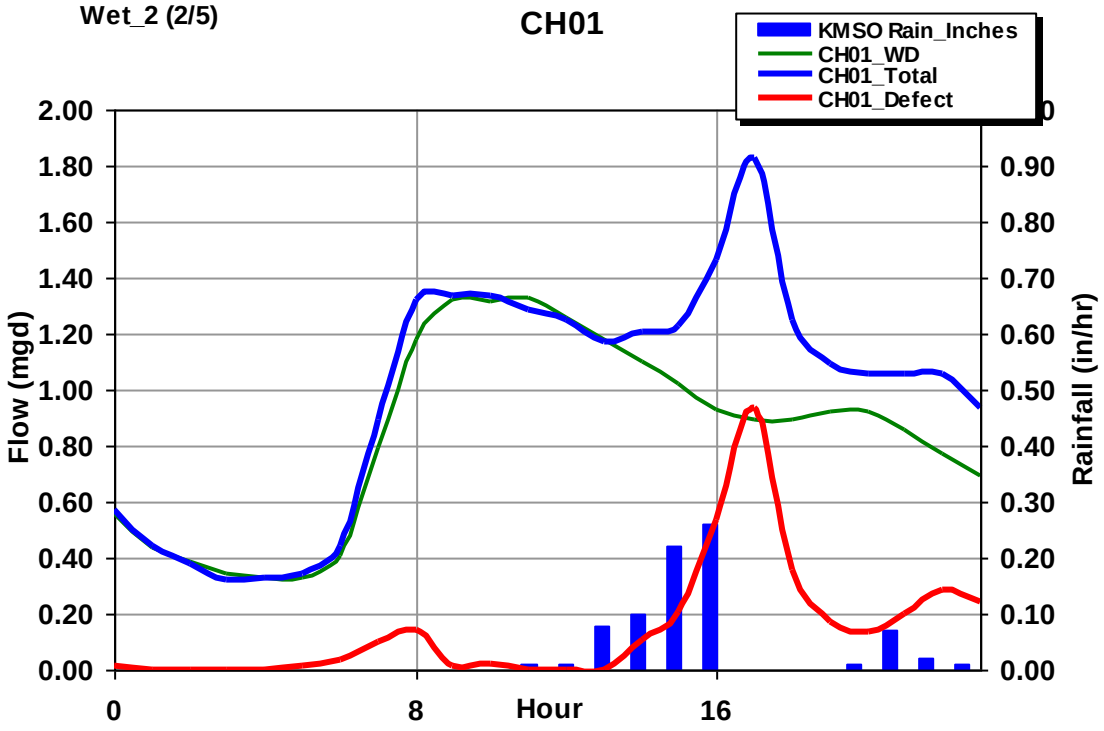
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Rainfall (in/hr)
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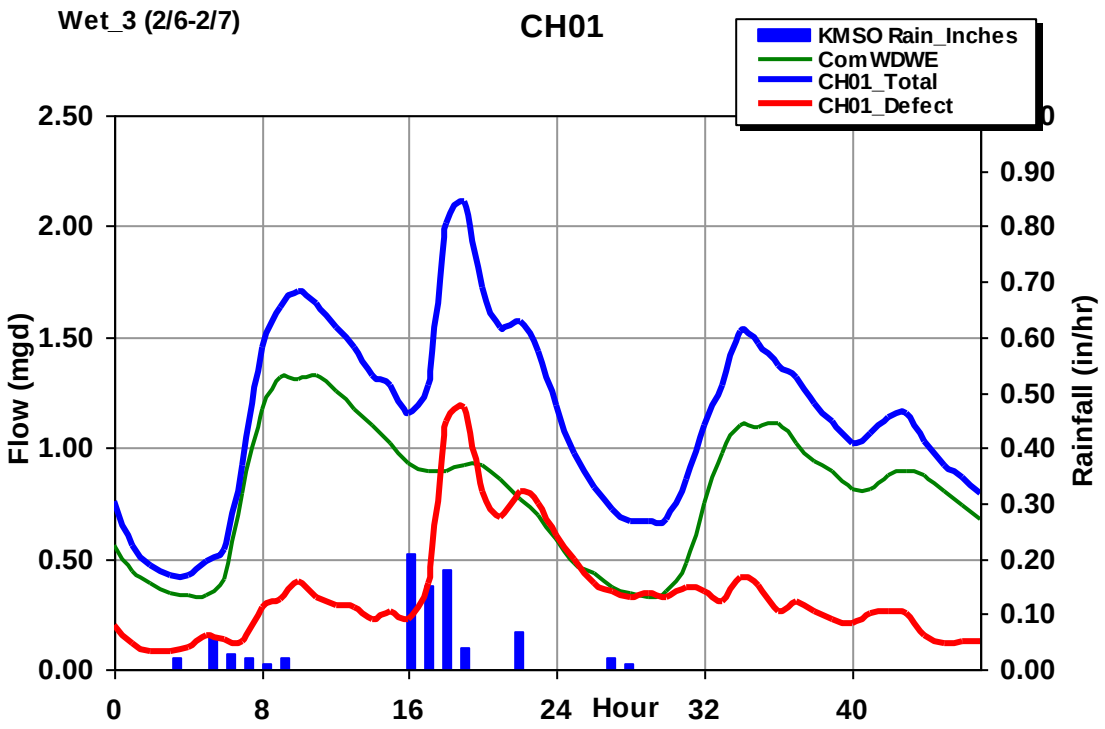
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KMSO Rain_Inches
ComWDWE
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Flow (mgd)
Rainfall (in/hr)
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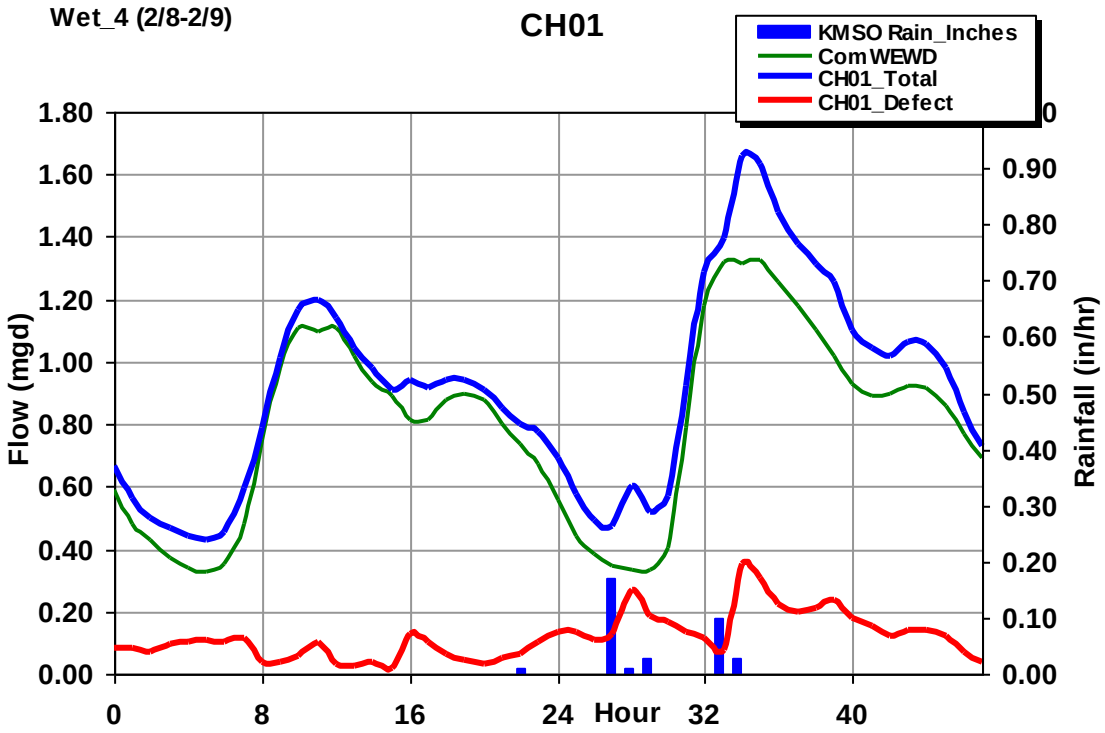
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CH01
KMSO Rain_Inches
ComWEWD
CH01_Total
CH01_Defect
Flow (mgd)
Rainfall (in/hr)
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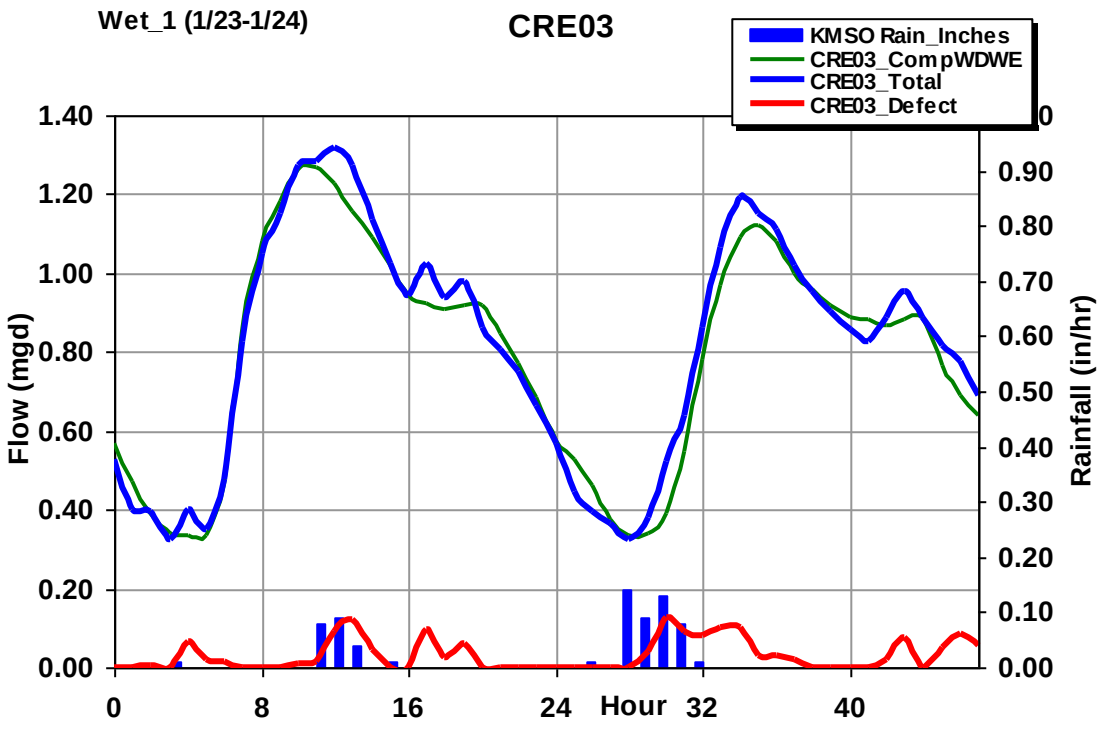
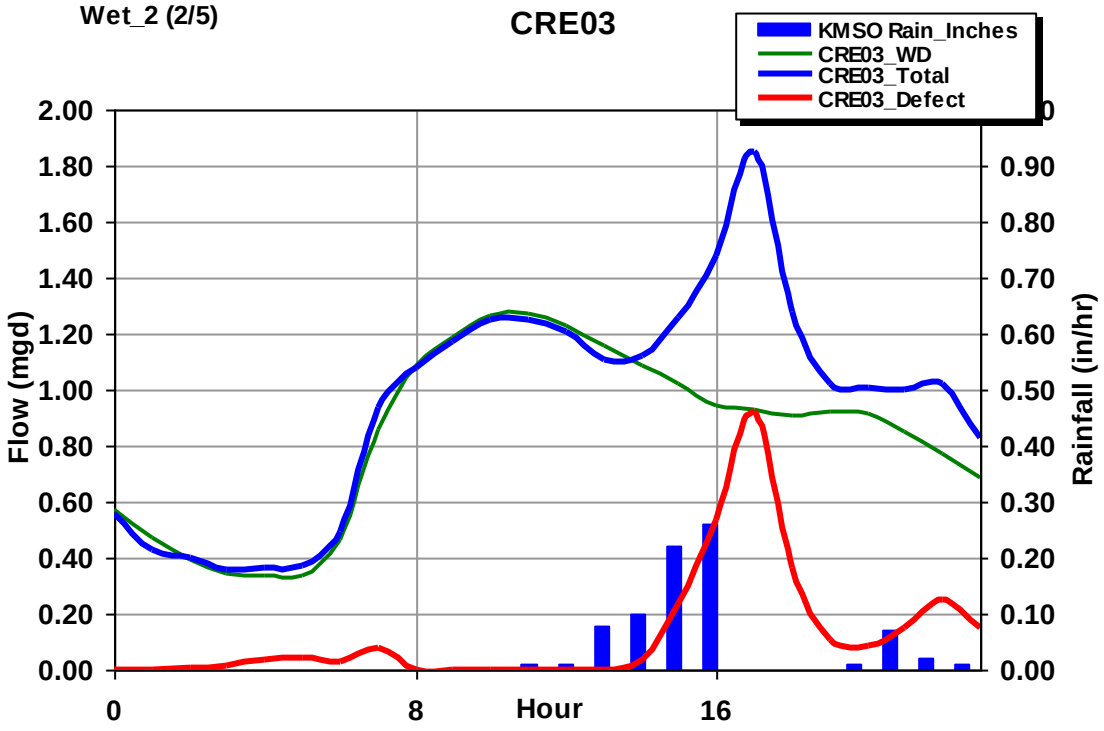
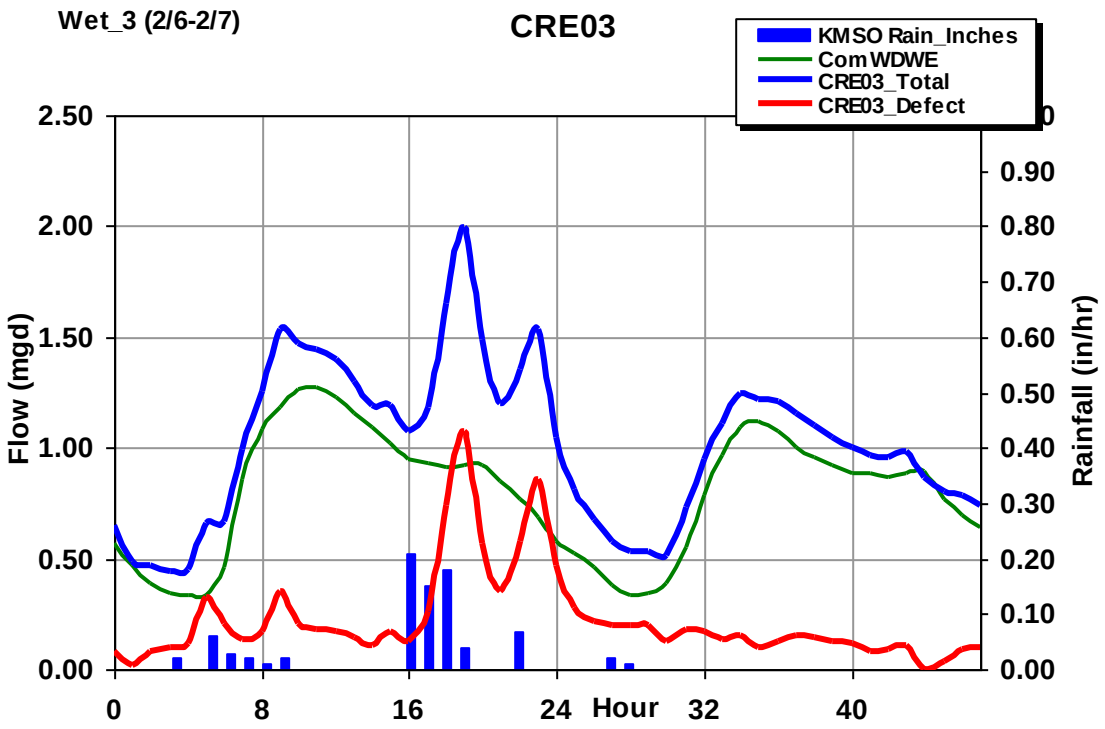
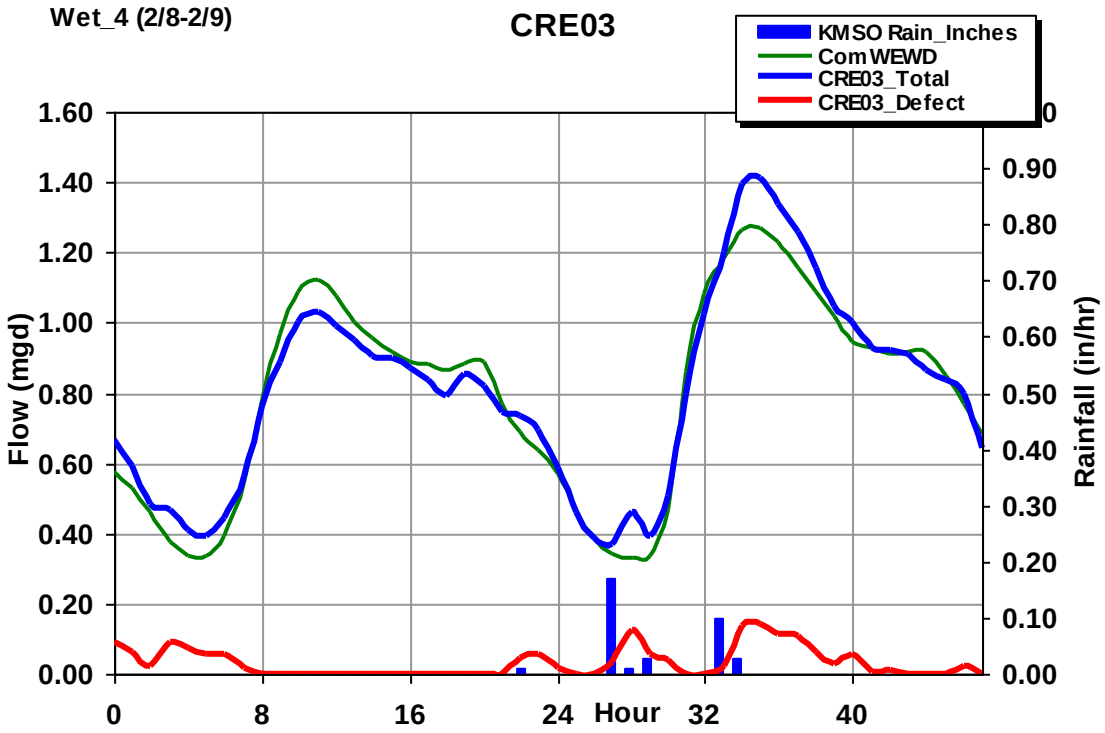
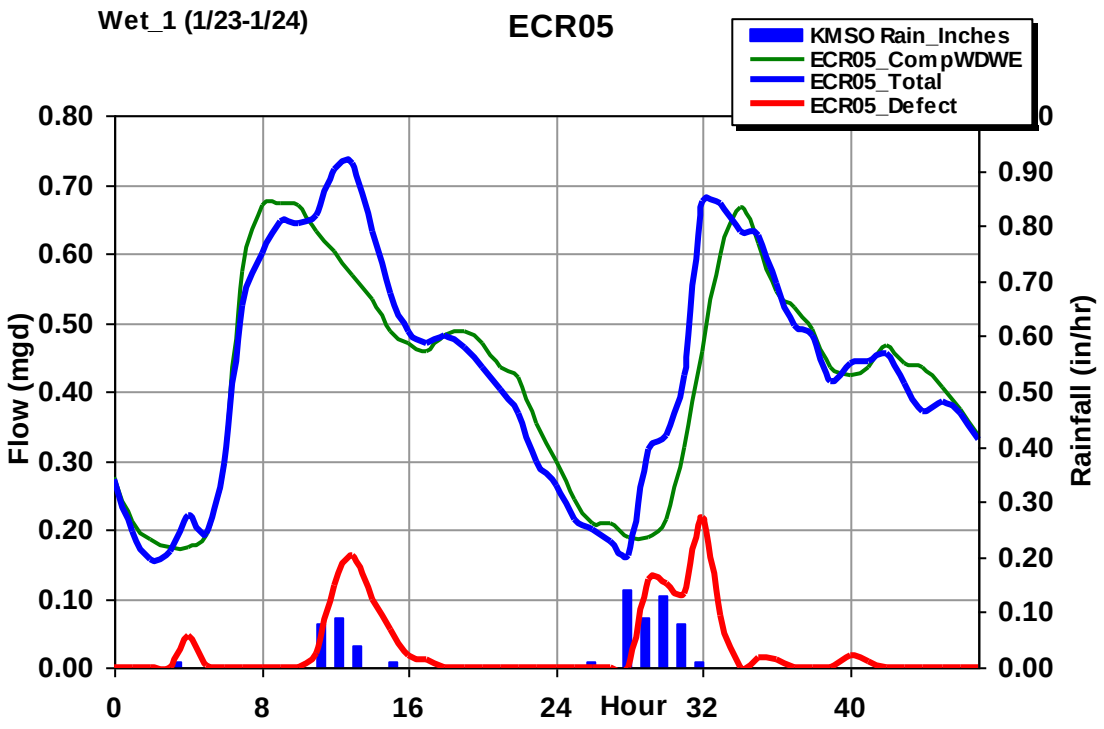
32
40

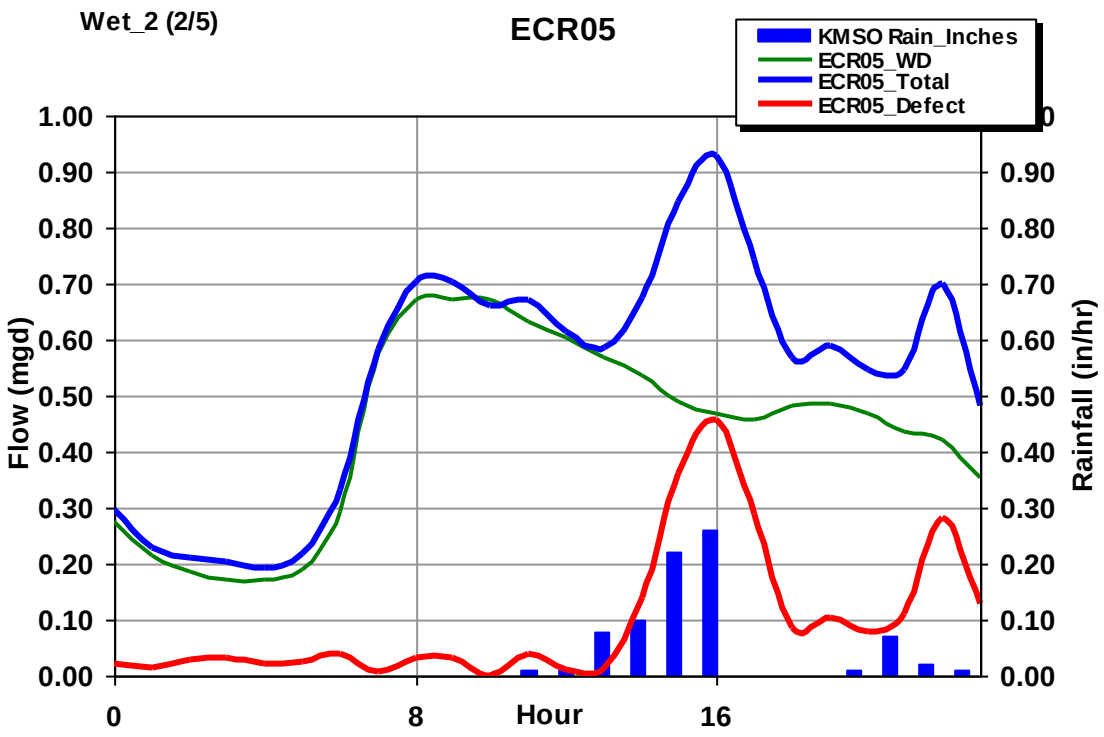

Wet_1 (1/23-1/24)
CRE03
KMSO Rain_Inches
CRE03_CompWDWE
CRE03_Total
CRE03_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

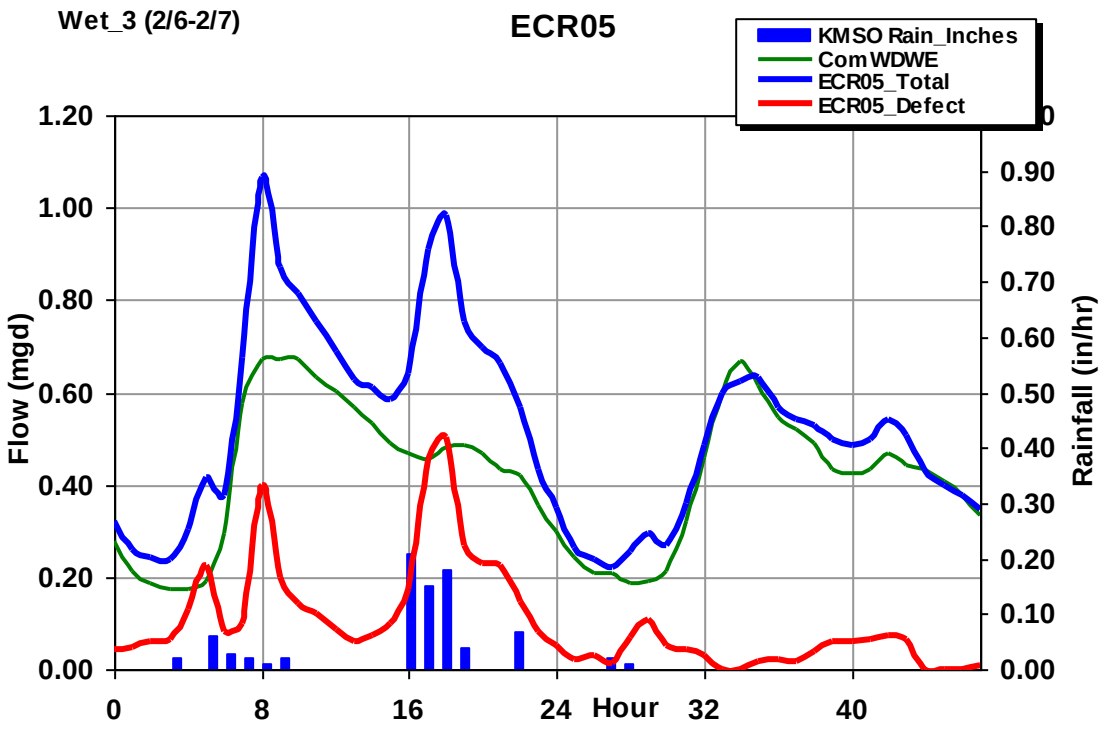

Wet_2 (2/5)
CRE03
KMSO Rain_Inches
CRE03_WD
CRE03_Total
CRE03_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16

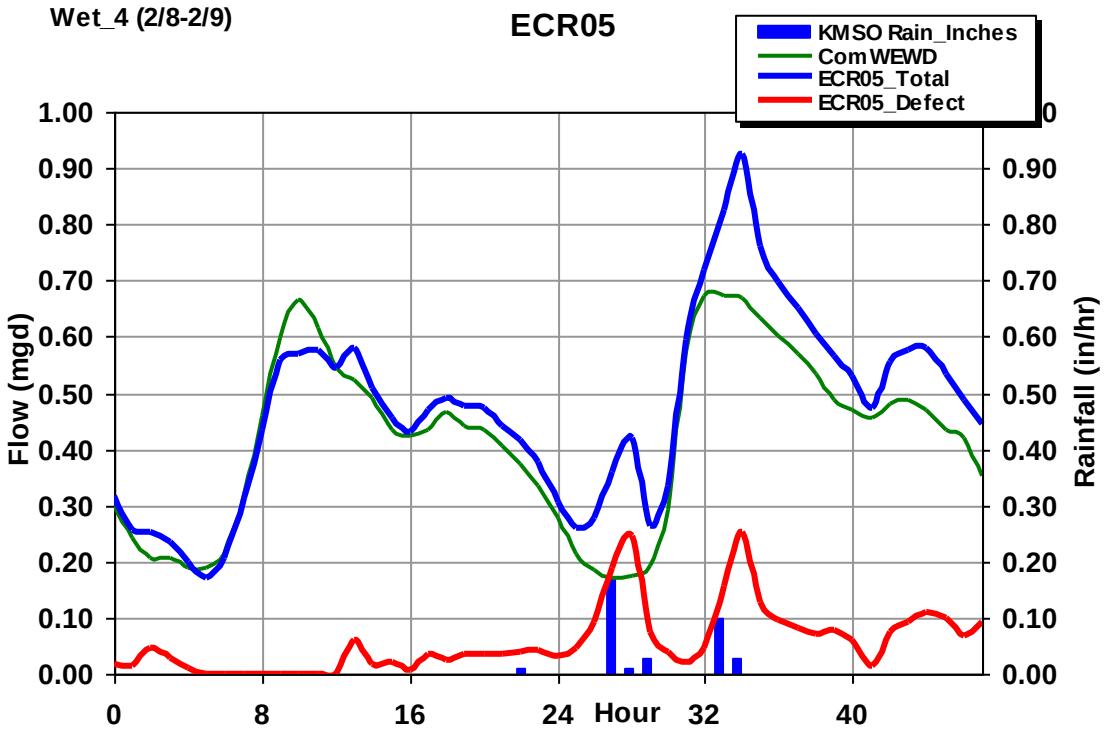

Wet_3 (2/6-2/7)
CRE03
KMSO Rain_Inches
ComWDWE
CRE03_Total
CRE03_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
2.50
2.00
1.50
1.00
0.50
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

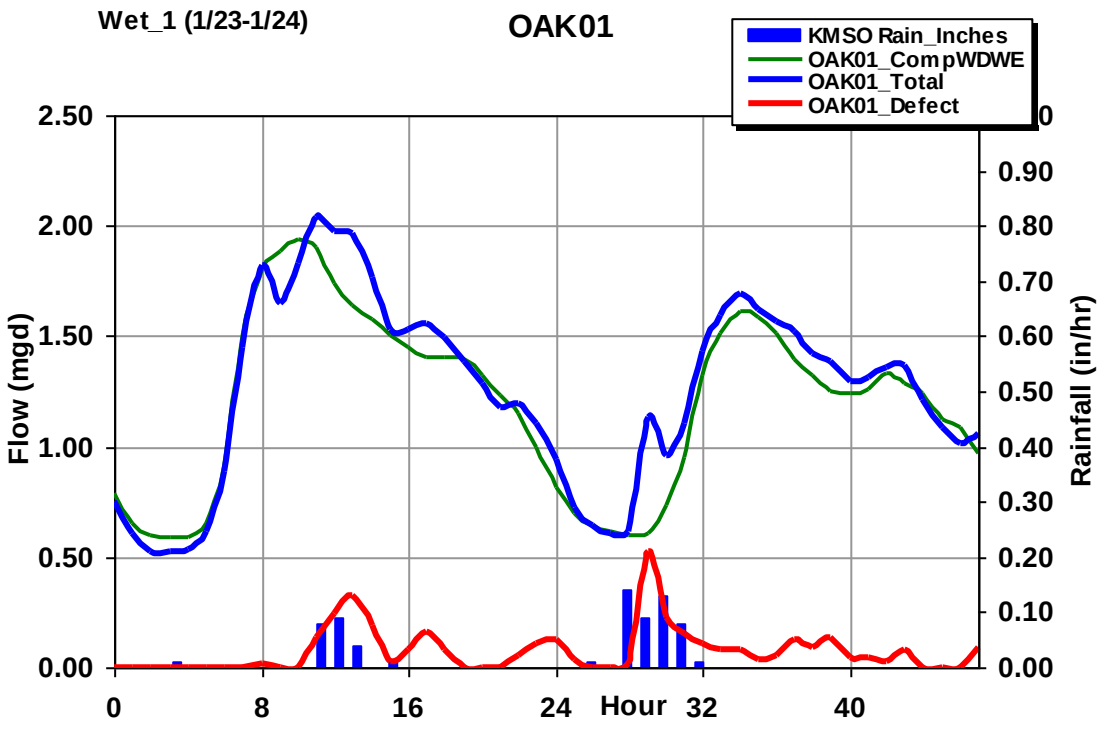

Wet_4 (2/8-2/9)
CRE03
KMSO Rain_Inches
ComWEWD
CRE03_Total
CRE03_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

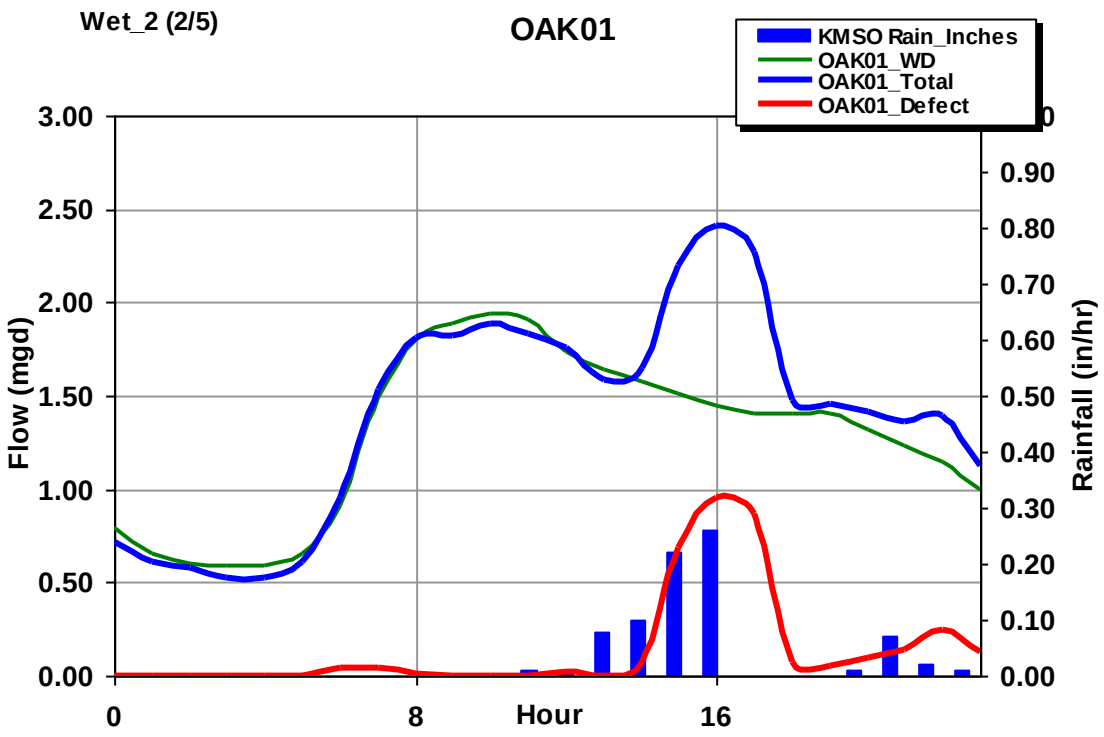

Wet_1 (1/23-1/24)
ECR05
KMSO Rain_Inches
ECR05_CompWDWE
ECR05_Total
ECR05_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

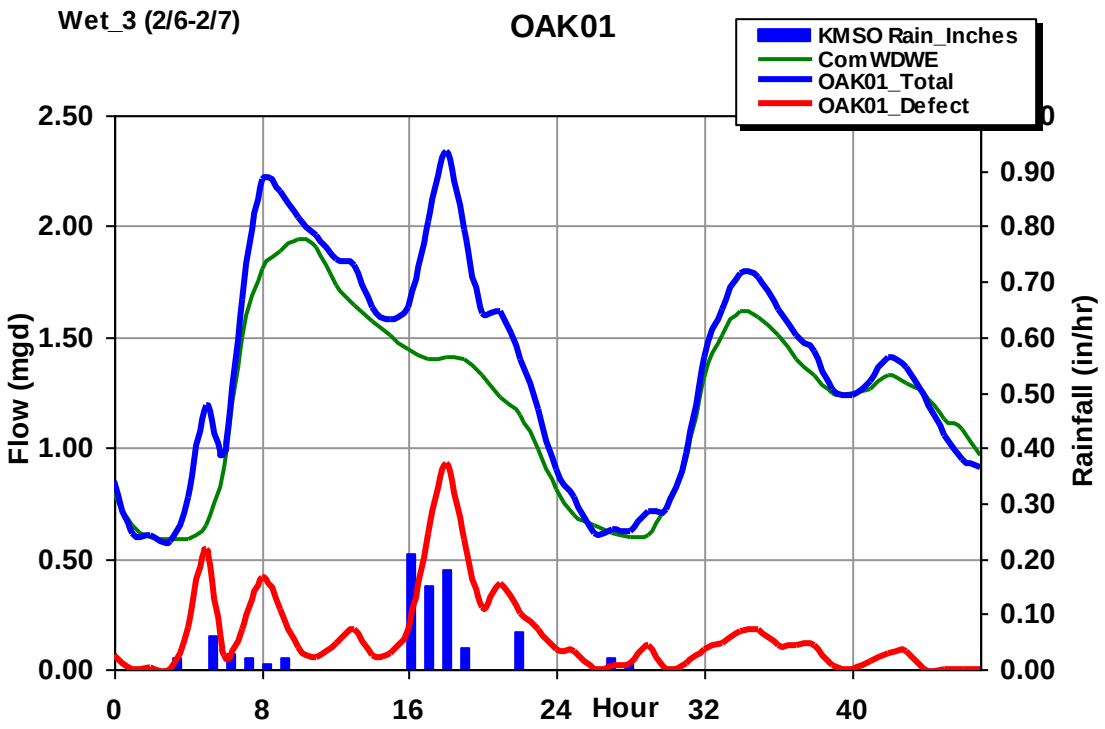

Wet_2 (2/5)
ECR05
KMSO Rain_Inches
ECR05_WD
ECR05_Total
ECR05_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16

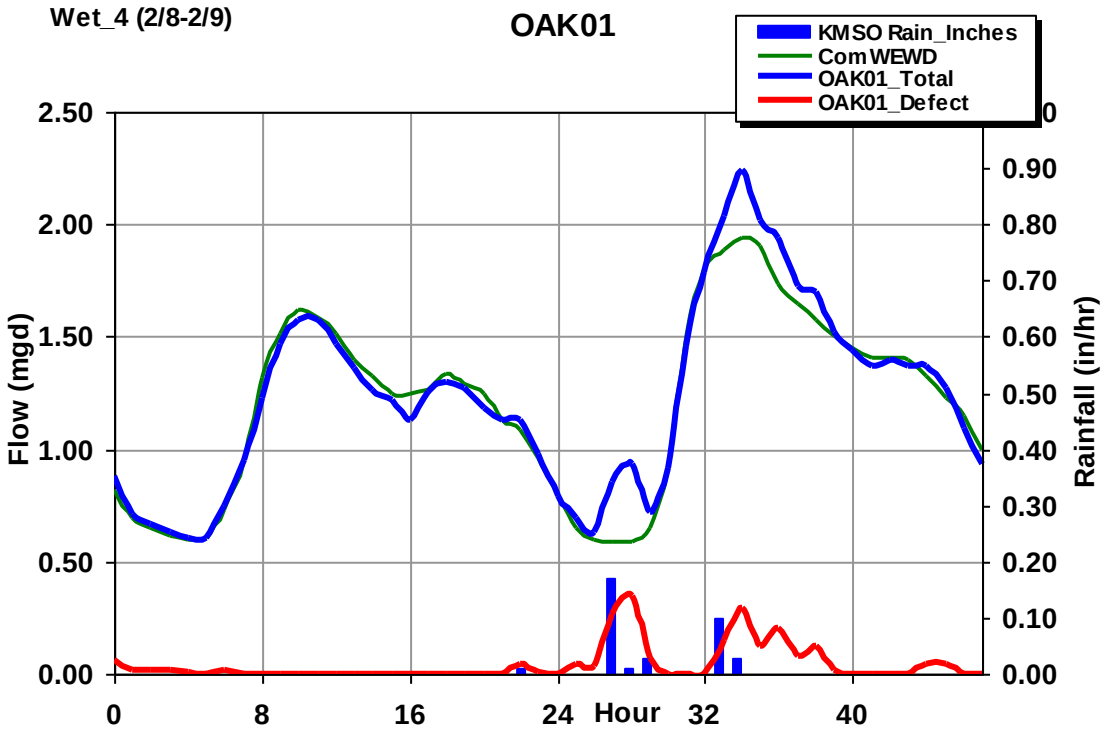

Wet_3 (2/6-2/7)
ECR05
KMSO Rain_Inches
ComWDWE
ECR05_Total
ECR05_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

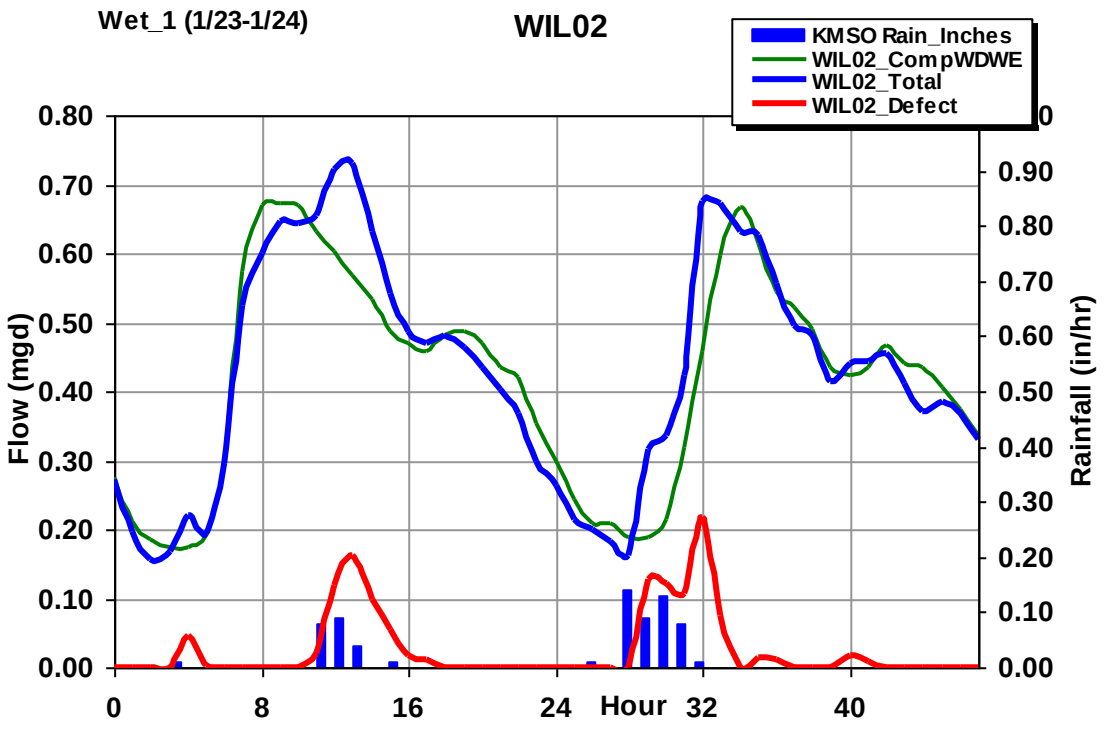

Wet_4 (2/8-2/9)
ECR05
KMSO Rain_Inches
ComWEWD
ECR05_Total
ECR05_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

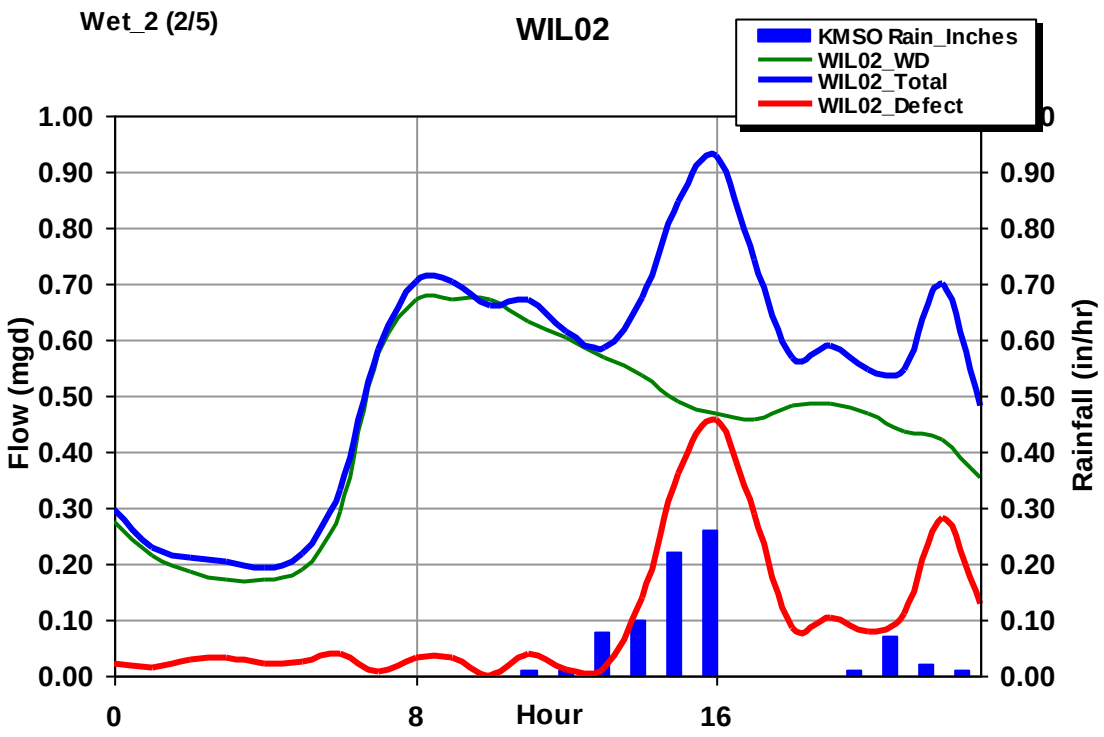

Wet_1 (1/23-1/24)
OAK01
KMSO Rain_Inches
OAK01_CompWDWE
OAK01_Total
OAK01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
2.50
2.00
1.50
1.00
0.50
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

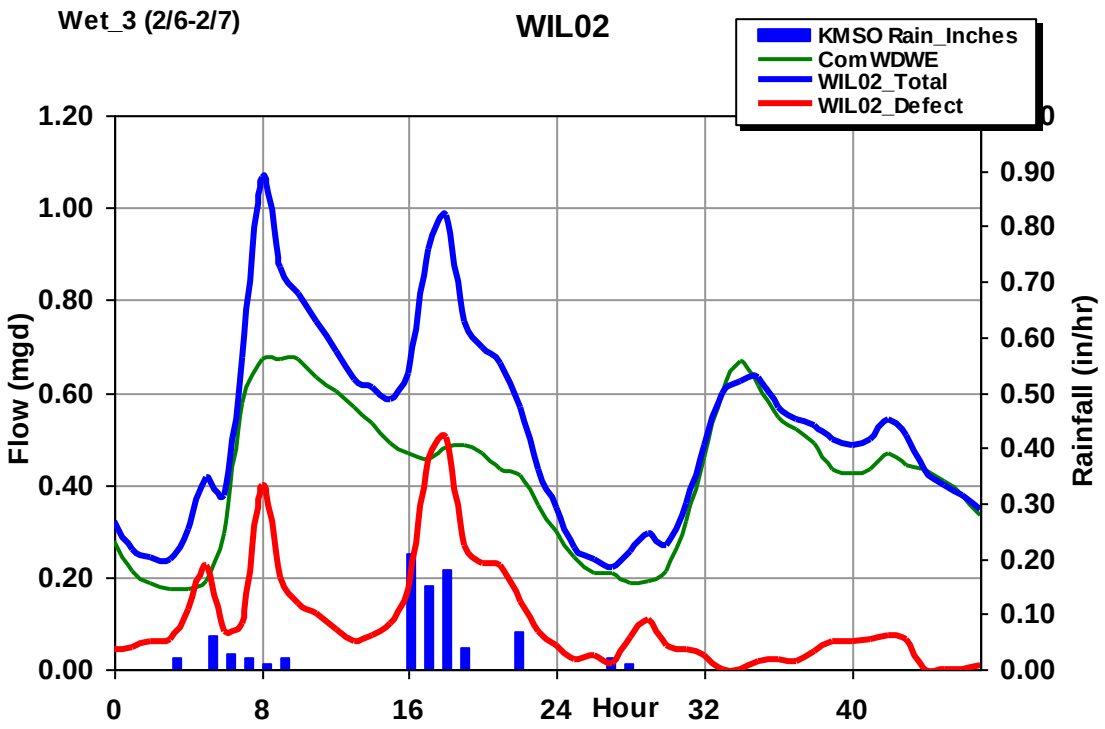

Wet_2 (2/5)
OAK01
KMSO Rain_Inches
OAK01_WD
OAK01_Total
OAK01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16

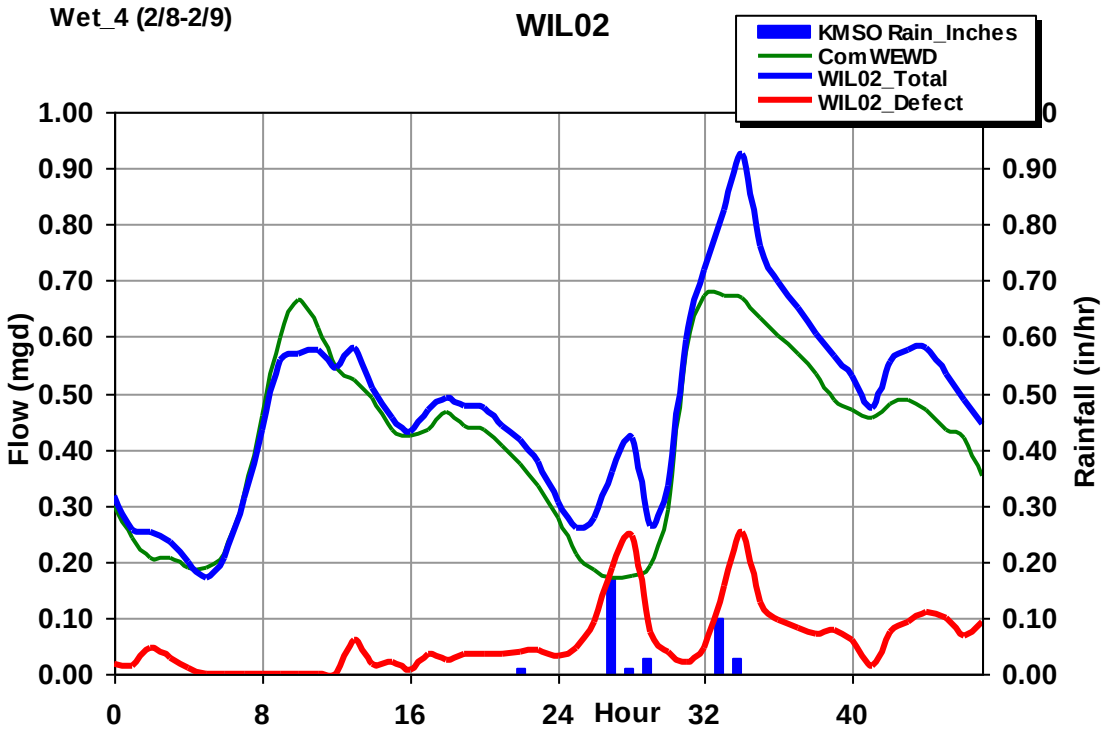

Wet_3 (2/6-2/7)
OAK01
KMSO Rain_Inches
ComWDWE
OAK01_Total
OAK01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
2.50
2.00
1.50
1.00
0.50
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

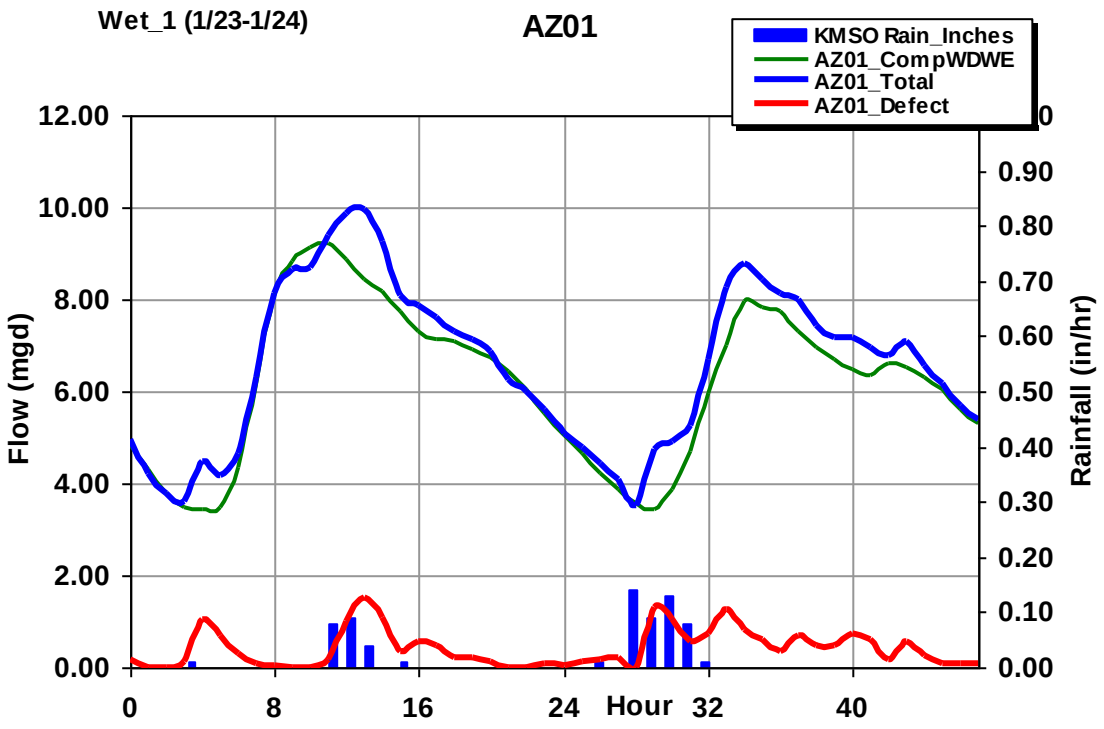

Wet_4 (2/8-2/9)
OAK01
KMSO Rain_Inches
ComWEWD
OAK01_Total
OAK01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
2.50
2.00
1.50
1.00
0.50
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

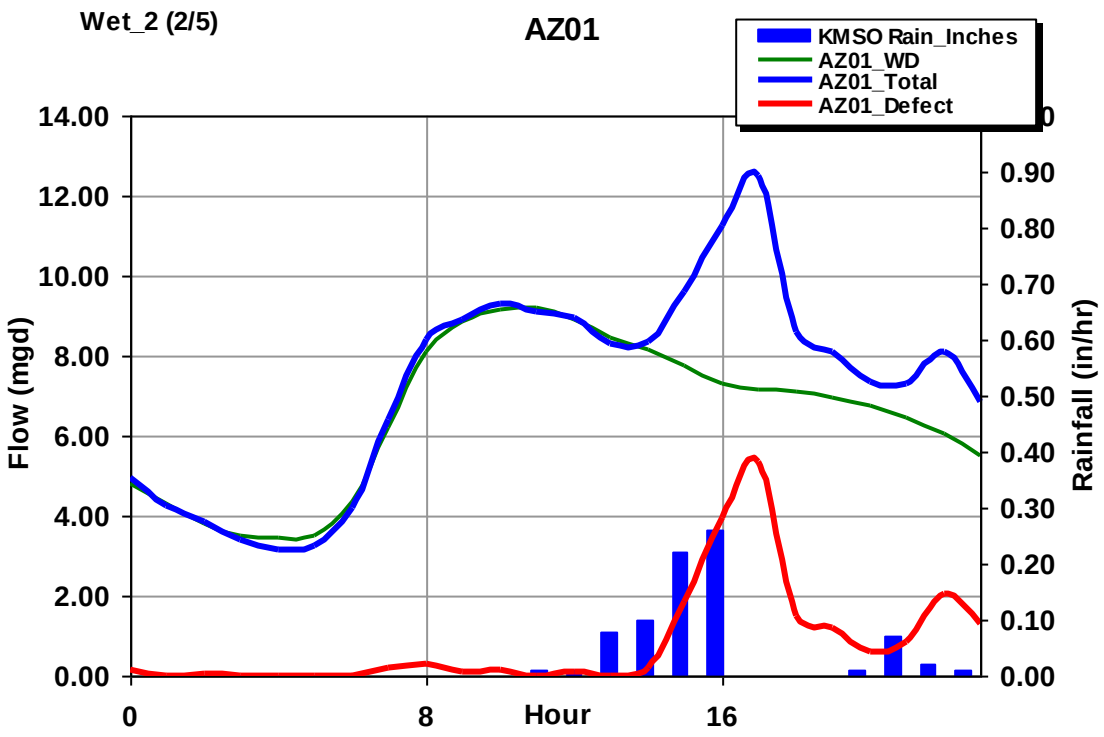

Wet_1 (1/23-1/24)
WIL02
KMSO Rain_Inches
WIL02_CompWDWE
WIL02_Total
WIL02_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
0
8
16
24
32
40

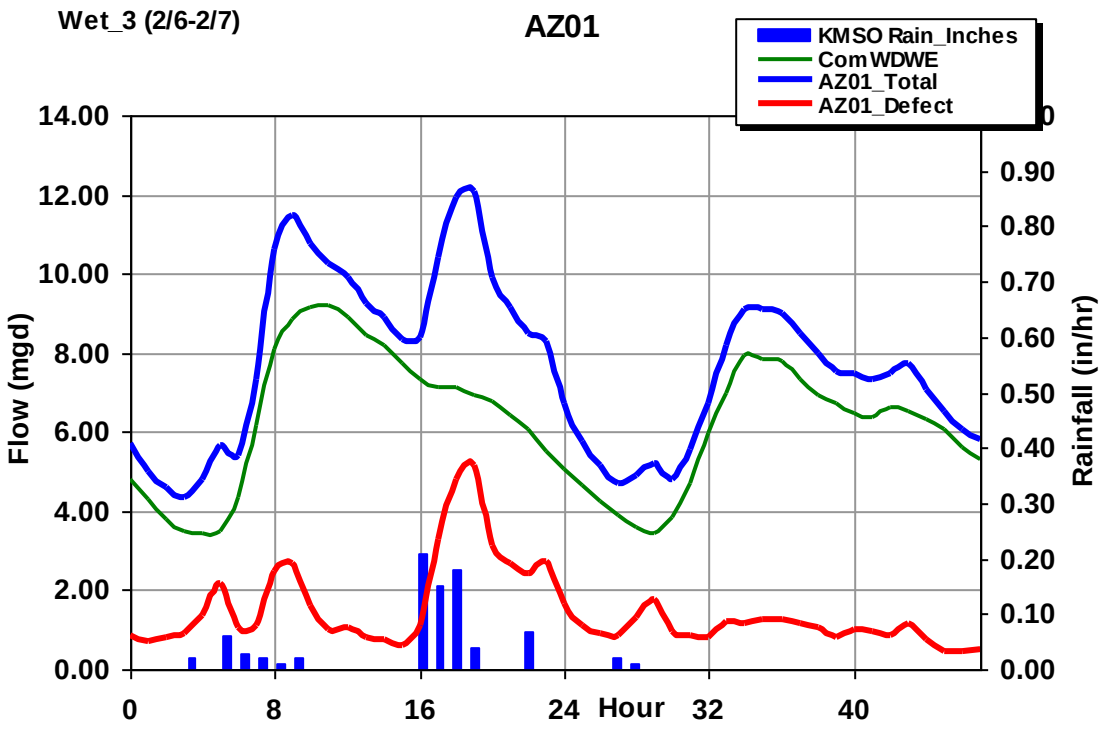

Wet_2 (2/5)
WIL02
KMSO Rain_Inches
WIL02_WD
WIL02_Total
WIL02_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
0
8
16

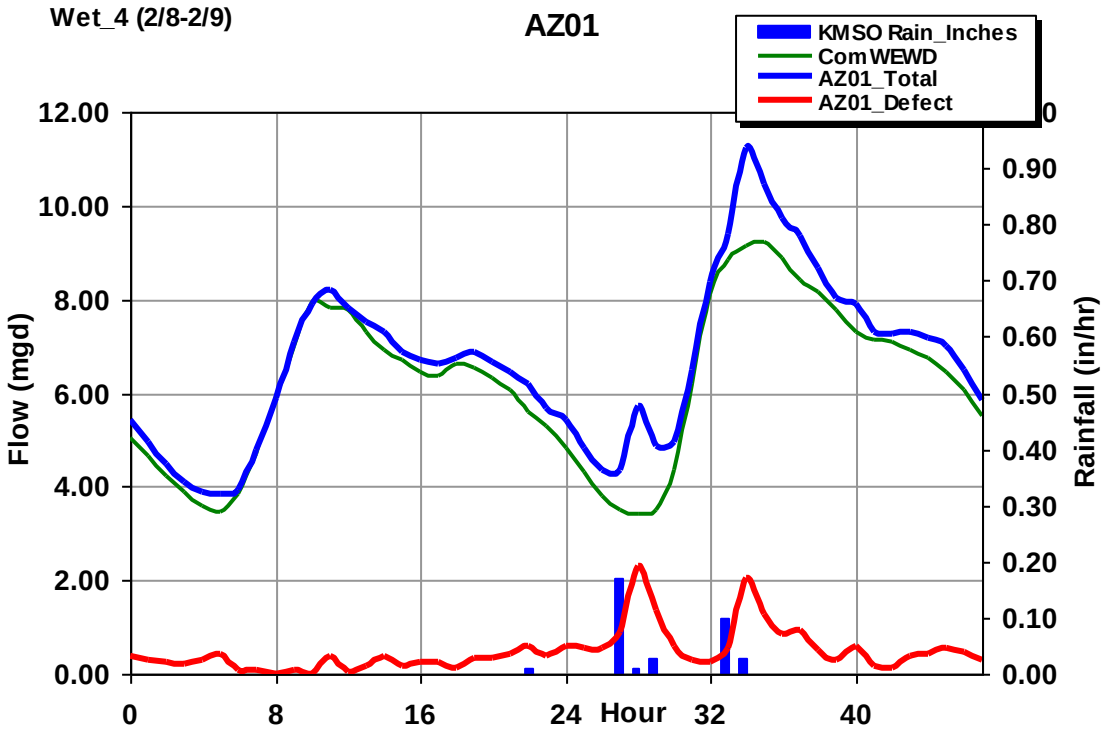

Wet_3 (2/6-2/7)
WIL02
KMSO Rain_Inches
ComWDWE
WIL02_Total
WIL02_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

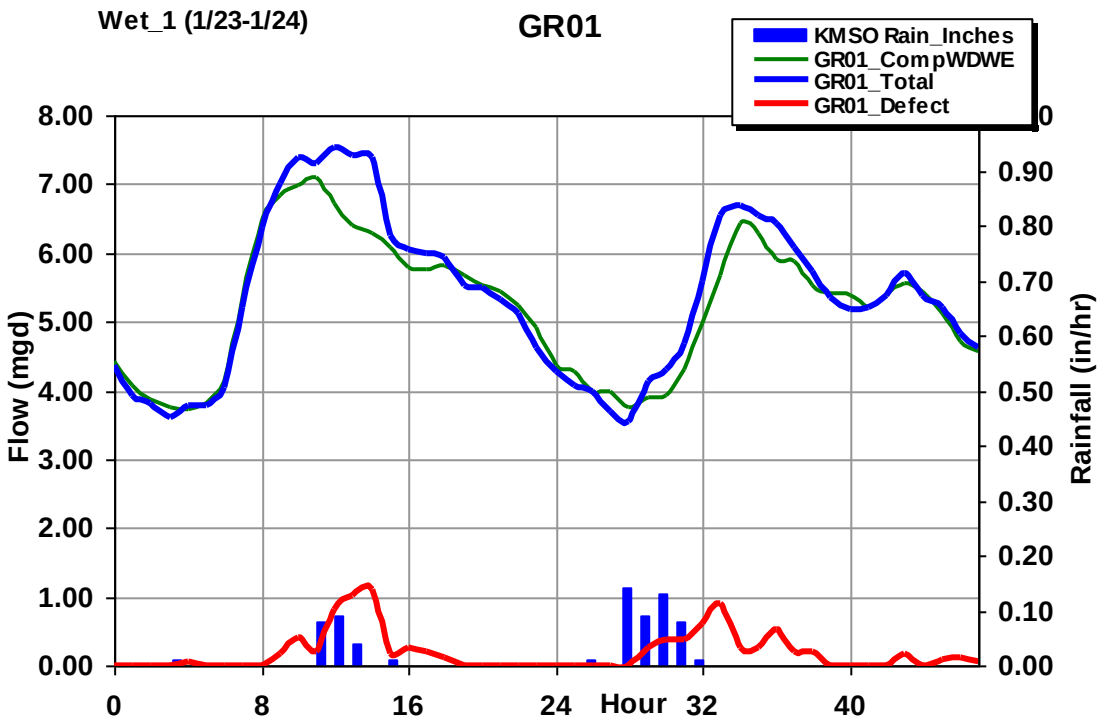

Wet_4 (2/8-2/9)
WIL02
KMSO Rain_Inches
ComWEWD
WIL02_Total
WIL02_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

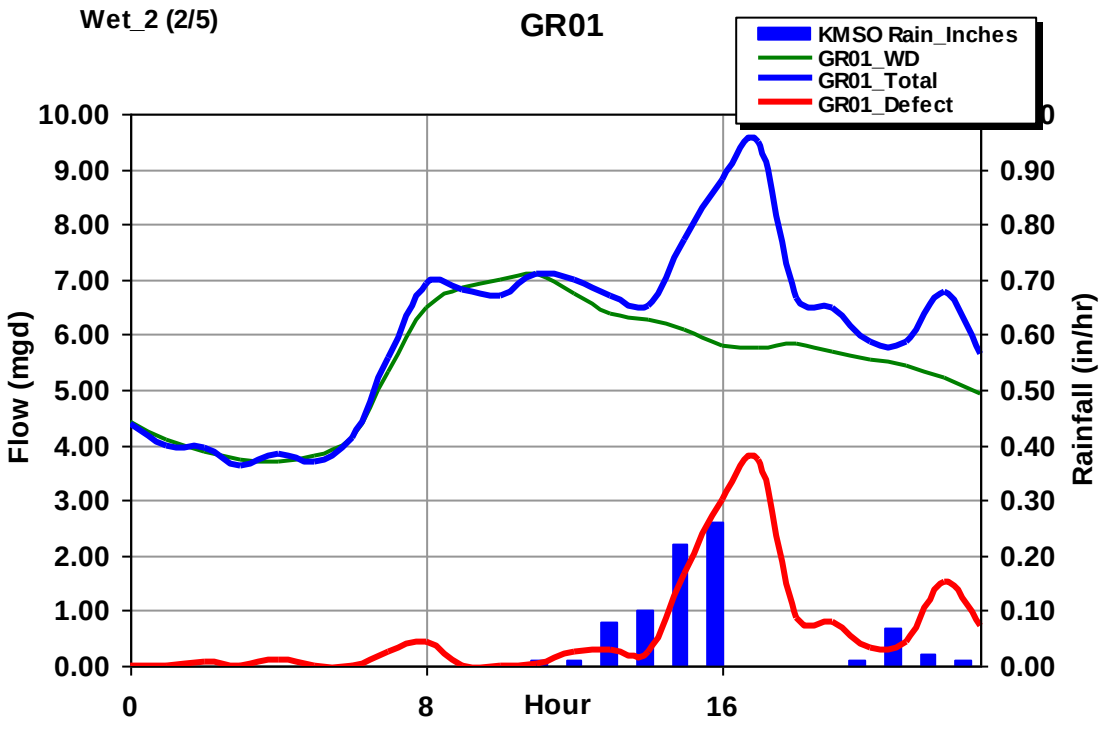

Wet_1 (1/23-1/24)
AZ01
KMSO Rain_Inches
AZ01_CompWDWE
AZ01_Total
AZ01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

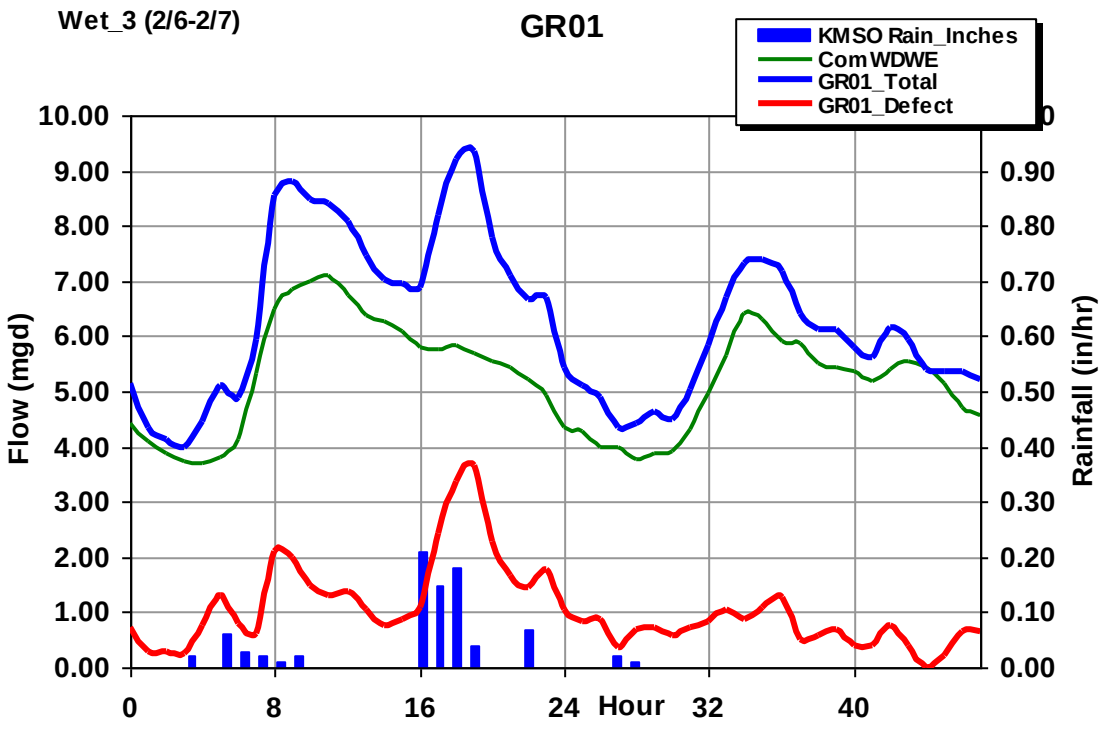

Wet_2 (2/5)
AZ01
KMSO Rain_Inches
AZ01_WD
AZ01_Total
AZ01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16

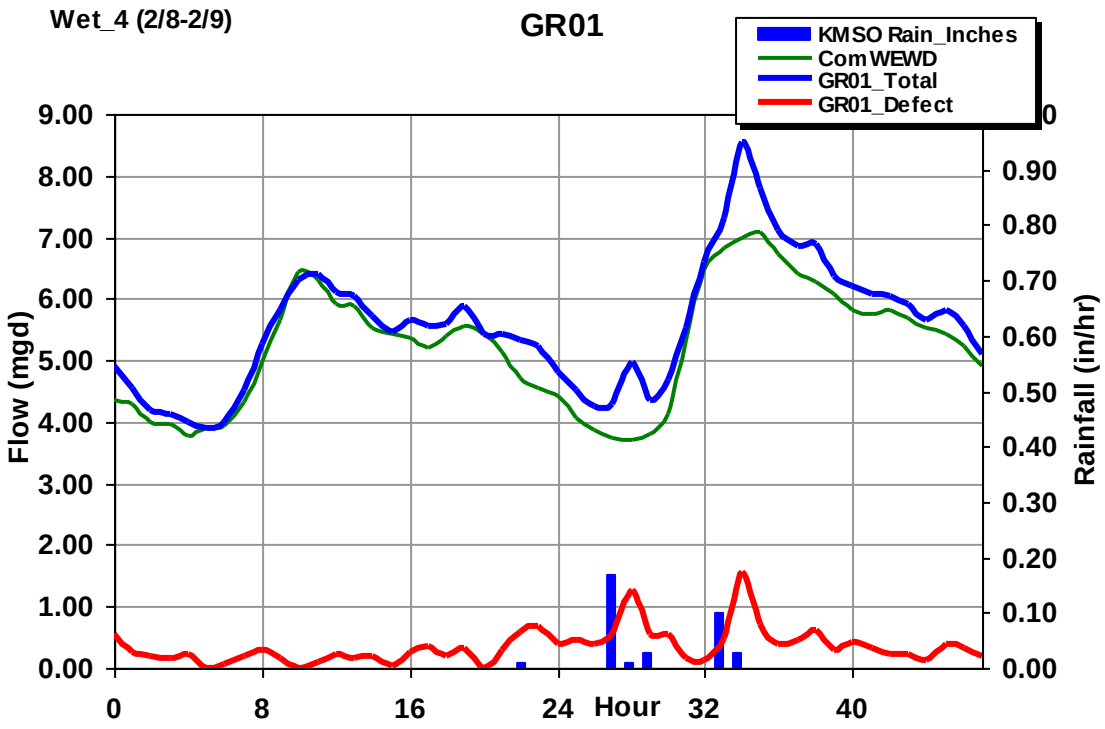

Wet_3 (2/6-2/7)
AZ01
KMSO Rain_Inches
ComWDWE
AZ01_Total
AZ01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40


Wet_4 (2/8-2/9)
AZ01
KMSO Rain_Inches
ComWEWD
AZ01_Total
AZ01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

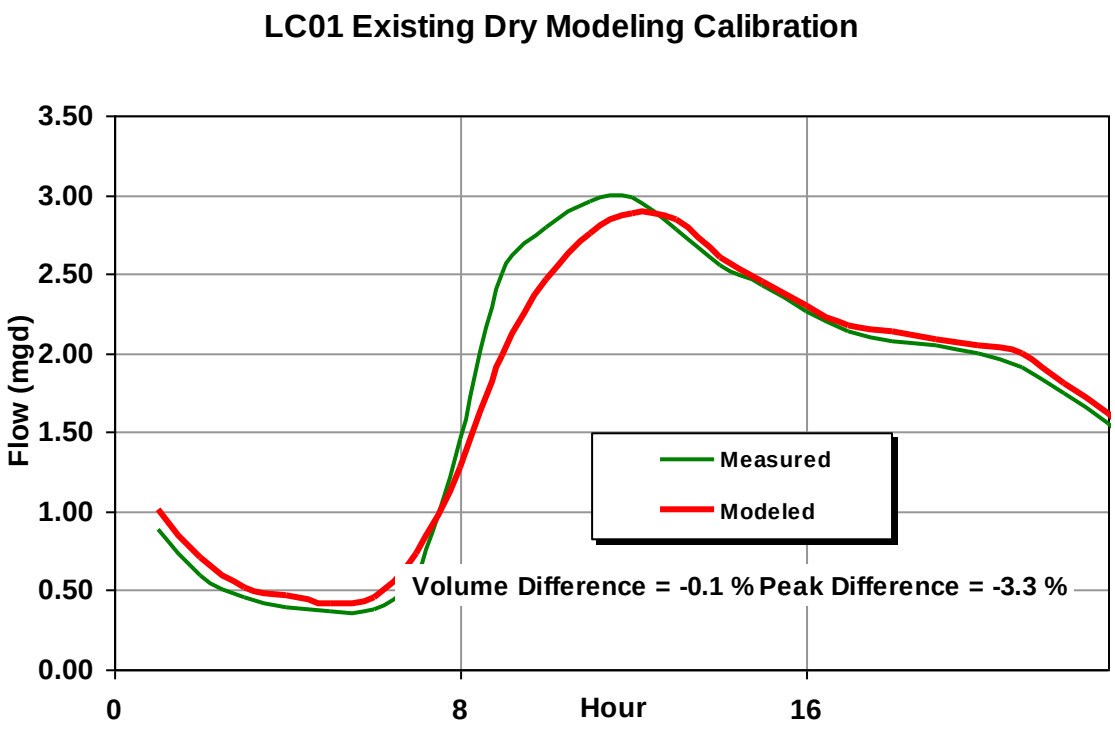

Wet_1 (1/23-1/24)
GR01
KMSO Rain_Inches
GR01_CompWDWE
GR01_Total
GR01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40


Wet_2 (2/5)
GR01
KMSO Rain_Inches
GR01_WD
GR01_Total
GR01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16

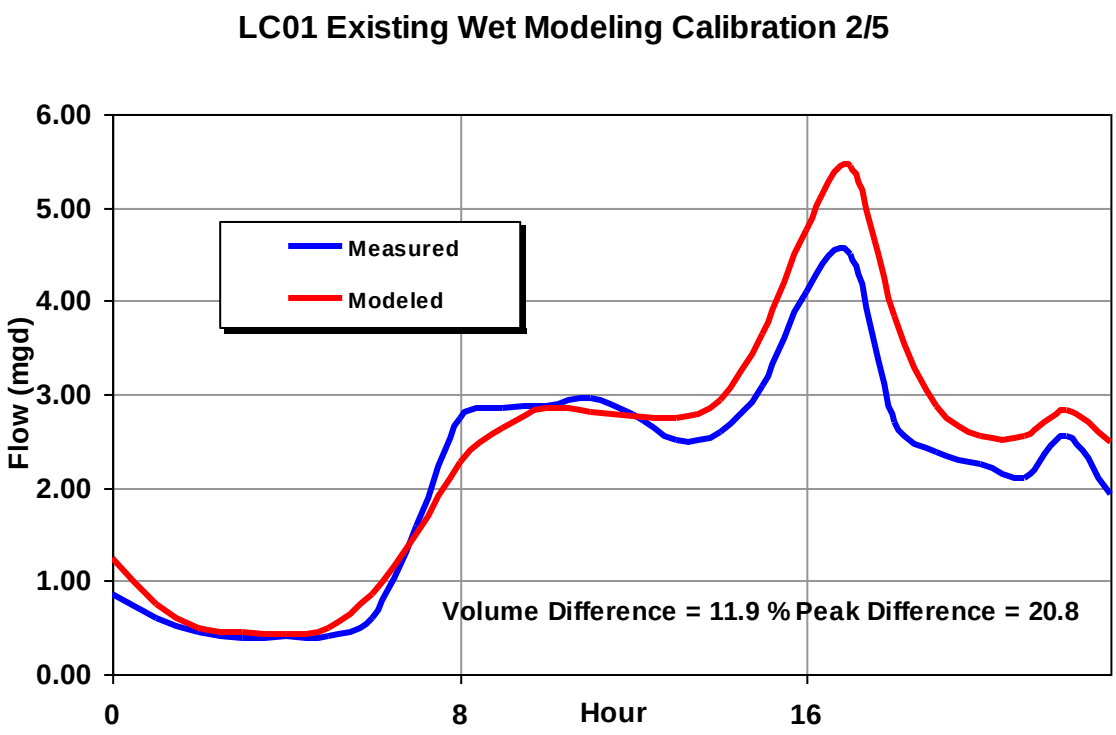

Wet_3 (2/6-2/7)
GR01
KMSO Rain_Inches
ComWDWE
GR01_Total
GR01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40


Wet_4 (2/8-2/9)
GR01
KMSO Rain_Inches
ComWEWD
GR01_Total
GR01_Defect
Flow (mgd)
Rainfall (in/hr)
Hour
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
8
16
24
32
40

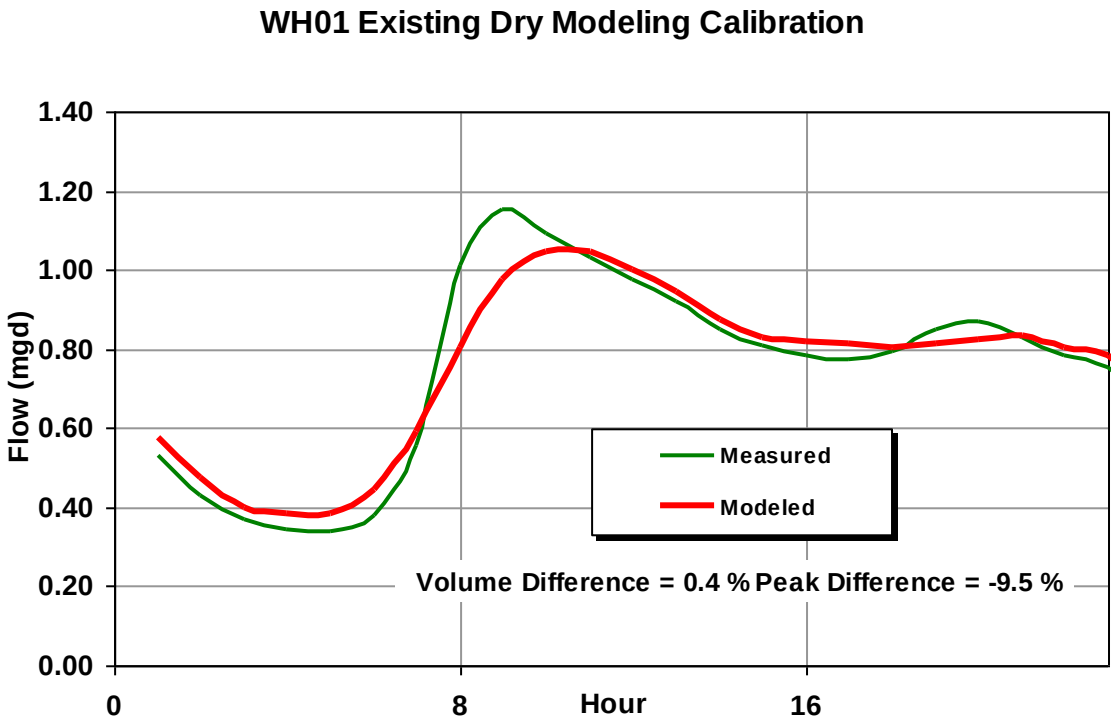
Modeling Results LC01 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.880	1.016	1.573
1	0.588	0.711	1.100
2	0.450	0.523	0.810
3	0.395	0.464	0.719
4	0.369	0.415	0.642
5	0.376	0.454	0.703
6	0.591	0.730	1.130
7	1.453	1.282	1.984
8	2.500	1.982	3.067
9	2.793	2.464	3.815
10	2.962	2.760	4.272
11	2.978	2.878	4.455
12	2.774	2.839	4.394
13	2.551	2.602	4.028
14	2.421	2.454	3.799
15	2.267	2.297	3.555
16	2.136	2.179	3.372
17	2.077	2.129	3.296
18	2.050	2.080	3.220
19	2.001	2.041	3.159
20	1.911	1.991	3.083
21	1.748	1.804	2.793
22	1.560	1.617	2.503
23	1.289	1.371	2.122
Volume	1.713	1.712	-0.1%
Peak	2.978	2.878	-3.3%



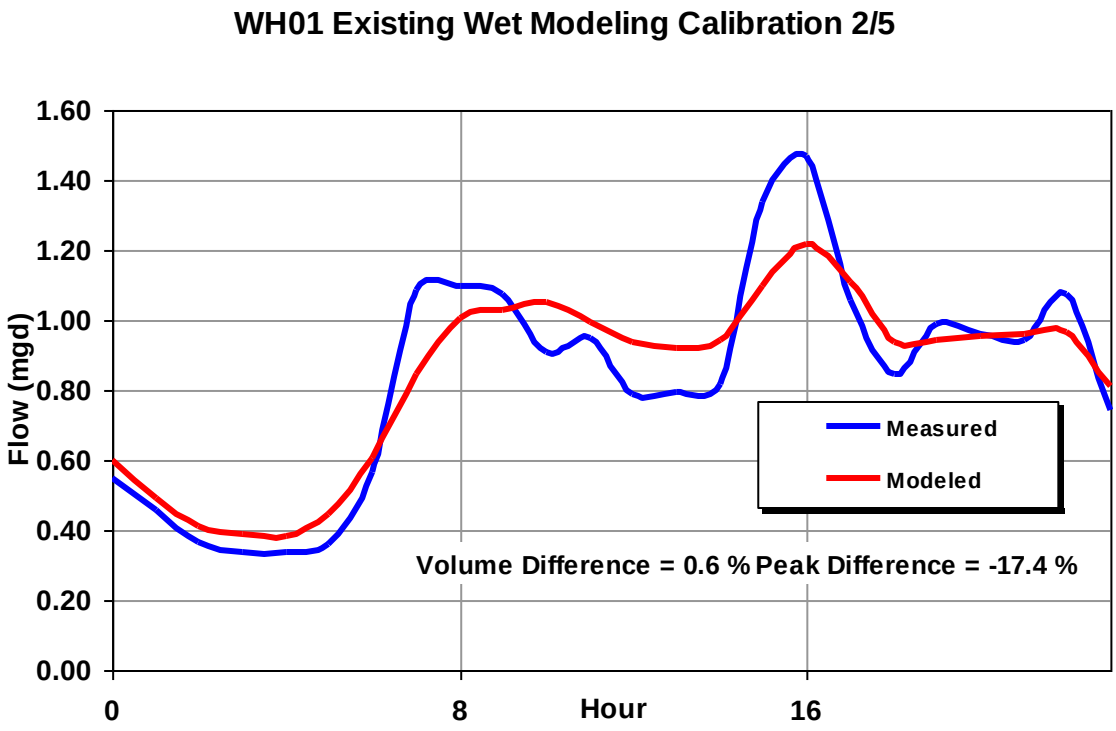
Modeling Results LC01 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.860	1.248	1.932
1	0.594	0.746	1.155
2	0.454	0.485	0.751
3	0.390	0.445	0.689
4	0.401	0.425	0.658
5	0.417	0.485	0.751
6	0.609	0.847	1.310
7	1.595	1.509	2.335
8	2.746	2.271	3.515
9	2.842	2.632	4.074
10	2.879	2.853	4.416
11	2.948	2.813	4.354
12	2.780	2.772	4.292
13	2.508	2.752	4.261
14	2.584	2.933	4.540
15	3.086	3.635	5.627
16	4.103	4.759	7.366
17	4.503	5.441	8.422
18	2.699	3.836	5.938
19	2.388	2.873	4.447
20	2.245	2.552	3.950
21	2.094	2.552	3.950
22	2.548	2.833	4.385
23	1.936	2.492	3.857
Volume	2.092	2.341	11.9%
Peak	4.503	5.441	20.8%



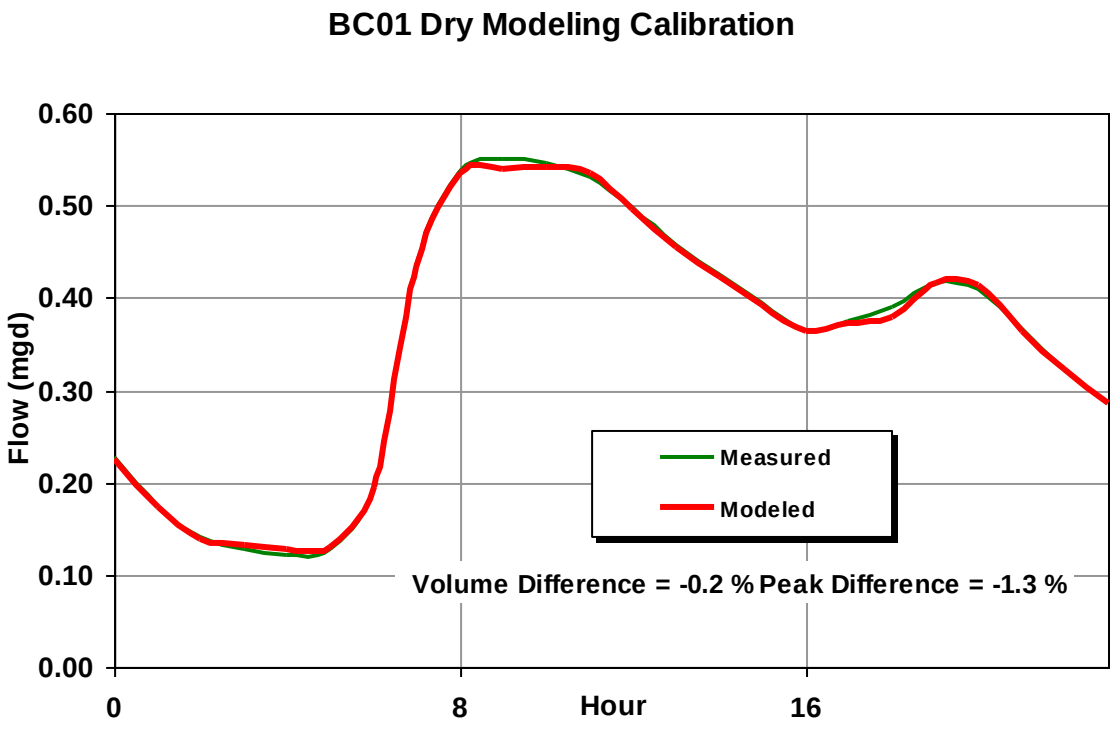
Modeling Results WH01 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.532	0.575	0.890
1	0.430	0.477	0.739
2	0.370	0.400	0.620
3	0.345	0.382	0.591
4	0.340	0.382	0.591
5	0.379	0.445	0.689
6	0.556	0.591	0.915
7	1.013	0.806	1.248
8	1.154	0.973	1.506
9	1.089	1.045	1.617
10	1.032	1.045	1.617
11	0.973	0.999	1.547
12	0.920	0.944	1.461
13	0.851	0.875	1.354
14	0.807	0.827	1.280
15	0.783	0.819	1.268
16	0.771	0.814	1.260
17	0.792	0.806	1.248
18	0.847	0.814	1.260
19	0.869	0.824	1.276
20	0.830	0.835	1.293
21	0.783	0.803	1.243
22	0.751	0.785	1.215
23	0.651	0.681	1.055
Volume	0.745	0.748	0.4%
Peak	1.154	1.045	-9.5%



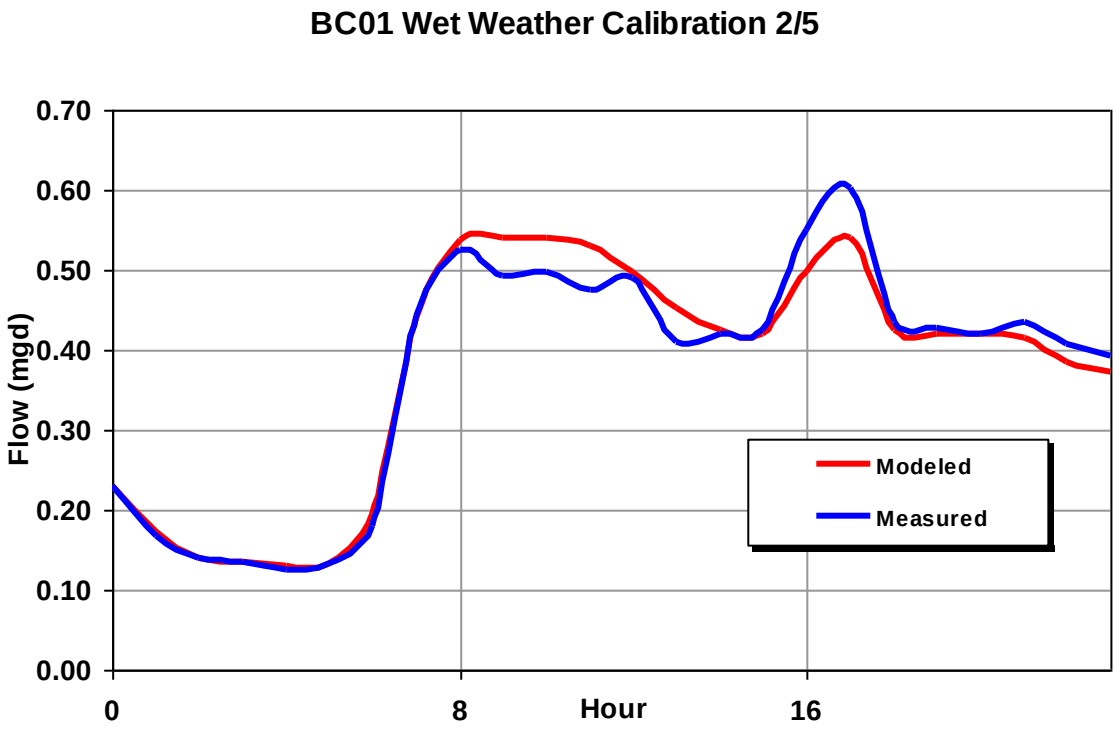
Modeling Results WH01 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.546	0.599	0.928
1	0.457	0.490	0.758
2	0.367	0.410	0.634
3	0.336	0.386	0.598
4	0.339	0.383	0.593
5	0.363	0.446	0.691
6	0.566	0.606	0.938
7	1.083	0.845	1.309
8	1.099	1.005	1.556
9	1.074	1.028	1.592
10	0.909	1.052	1.628
11	0.949	0.992	1.535
12	0.791	0.935	1.448
13	0.794	0.919	1.422
14	0.820	0.942	1.458
15	1.339	1.098	1.700
16	1.471	1.215	1.880
17	1.057	1.108	1.716
18	0.844	0.939	1.453
19	0.986	0.945	1.463
20	0.961	0.952	1.474
21	0.943	0.959	1.484
22	1.073	0.969	1.499
23	0.742	0.812	1.257
Volume	0.829	0.835	0.6%
Peak	1.471	1.215	-17.4%



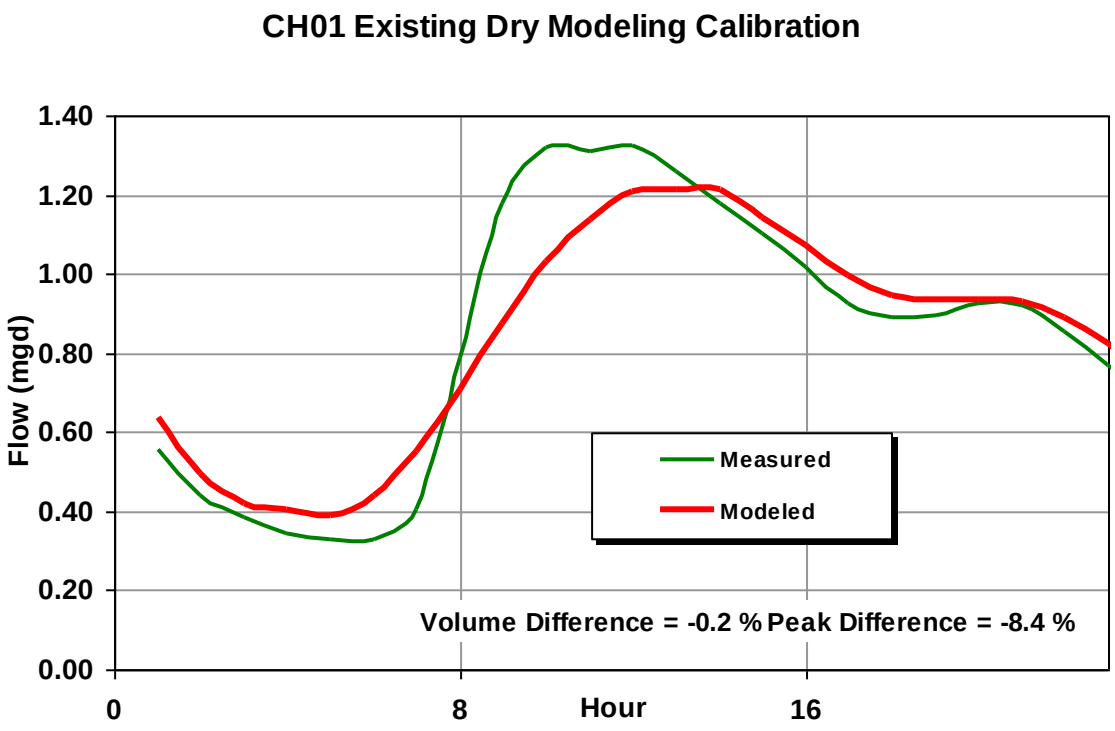
Modeling Results BC01 Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.227	0.226	0.350
1	0.173	0.173	0.267
2	0.142	0.140	0.216
3	0.128	0.133	0.206
4	0.122	0.128	0.198
5	0.129	0.130	0.201
6	0.195	0.196	0.303
7	0.435	0.433	0.670
8	0.538	0.536	0.829
9	0.549	0.539	0.834
10	0.545	0.542	0.840
11	0.531	0.536	0.829
12	0.498	0.496	0.768
13	0.457	0.455	0.704
14	0.424	0.423	0.655
15	0.394	0.392	0.606
16	0.364	0.363	0.563
17	0.375	0.372	0.575
18	0.389	0.380	0.588
19	0.415	0.416	0.645
20	0.409	0.413	0.639
21	0.366	0.363	0.563
22	0.322	0.322	0.498
23	0.287	0.285	0.442
Volume	0.350	0.350	-0.2%
Peak	0.549	0.542	-1.3%



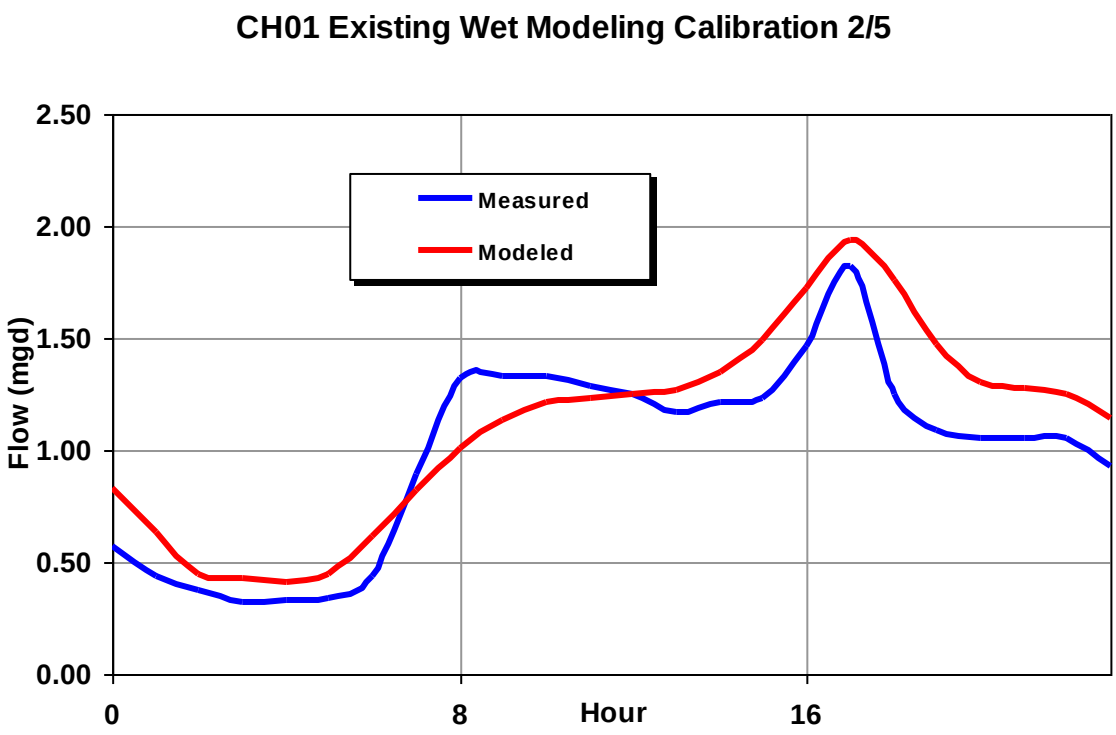
Modeling Results BC01 Wet Weather Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.231	0.230	0.356
1	0.169	0.172	0.266
2	0.141	0.139	0.215
3	0.135	0.134	0.208
4	0.125	0.129	0.200
5	0.134	0.133	0.205
6	0.180	0.195	0.302
7	0.442	0.440	0.682
8	0.526	0.538	0.832
9	0.493	0.539	0.835
10	0.497	0.541	0.837
11	0.474	0.529	0.819
12	0.491	0.496	0.768
13	0.410	0.454	0.702
14	0.420	0.426	0.659
15	0.426	0.421	0.651
16	0.550	0.498	0.771
17	0.602	0.541	0.837
18	0.435	0.424	0.656
19	0.427	0.421	0.651
20	0.421	0.419	0.649
21	0.435	0.416	0.644
22	0.408	0.384	0.595
23	0.393	0.373	0.577
Volume	0.373	0.375	0.4%
Peak	0.602	0.541	-10.1%



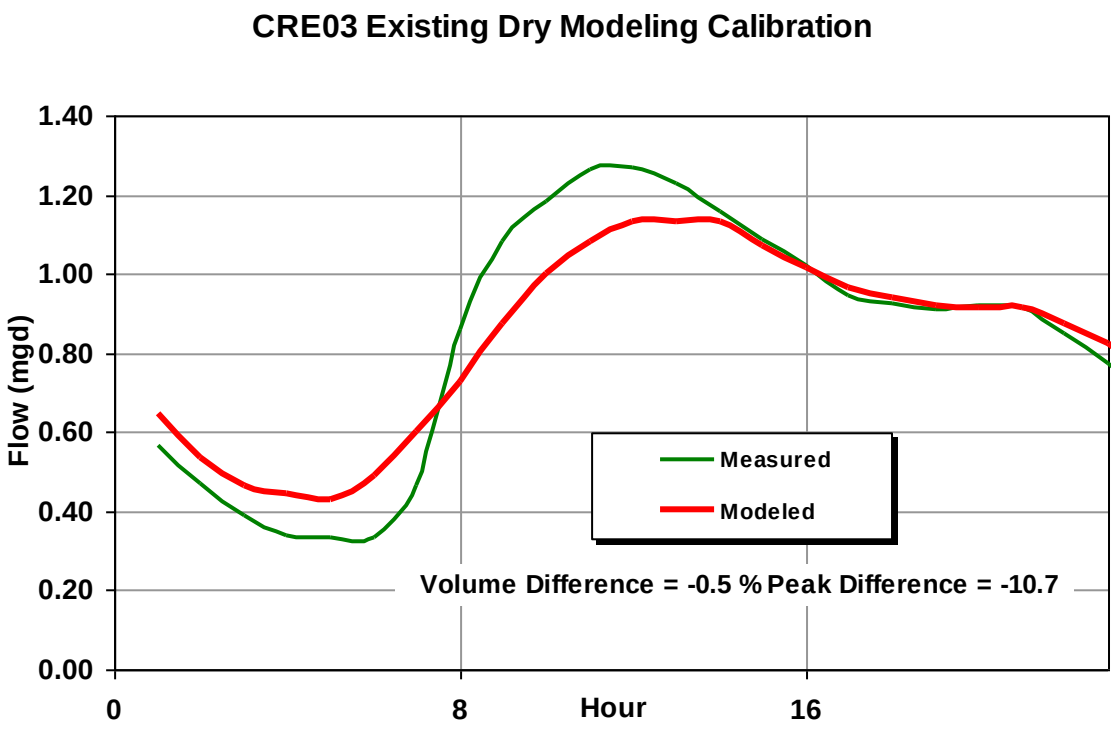
Modeling Results CH01 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.558	0.636	0.984
1	0.439	0.493	0.764
2	0.386	0.421	0.651
3	0.346	0.404	0.625
4	0.331	0.387	0.600
5	0.330	0.440	0.682
6	0.407	0.550	0.851
7	0.790	0.705	1.092
8	1.178	0.874	1.353
9	1.321	1.029	1.594
10	1.311	1.135	1.758
11	1.326	1.208	1.870
12	1.257	1.211	1.875
13	1.176	1.215	1.880
14	1.101	1.142	1.768
15	1.018	1.069	1.655
16	0.927	0.996	1.542
17	0.892	0.943	1.460
18	0.894	0.937	1.450
19	0.924	0.933	1.445
20	0.918	0.930	1.440
21	0.857	0.887	1.373
22	0.770	0.824	1.276
23	0.691	0.728	1.127
Volume	0.839	0.838	-0.2%
Peak	1.326	1.215	-8.4%



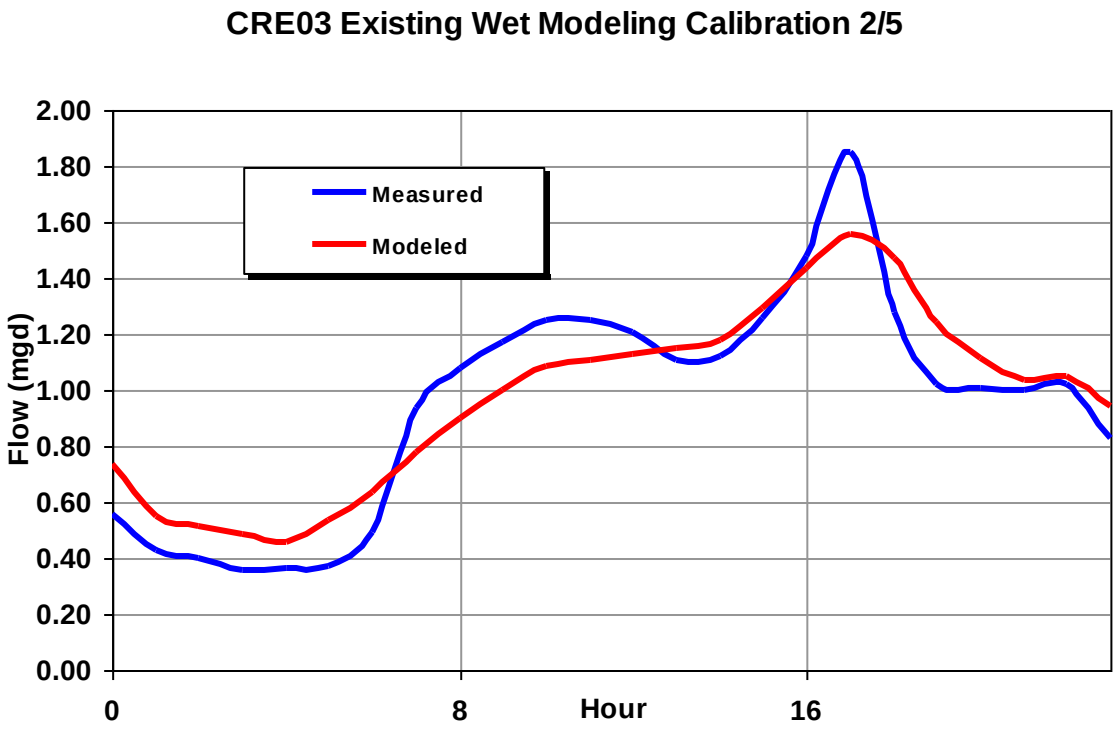
Modeling Results CH01 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.571	0.830	1.284
1	0.441	0.634	0.981
2	0.377	0.450	0.696
3	0.323	0.431	0.668
4	0.327	0.407	0.630
5	0.344	0.444	0.687
6	0.441	0.615	0.952
7	0.892	0.823	1.275
8	1.321	1.013	1.569
9	1.332	1.136	1.758
10	1.334	1.215	1.881
11	1.283	1.234	1.910
12	1.249	1.252	1.938
13	1.169	1.264	1.957
14	1.210	1.350	2.090
15	1.229	1.491	2.308
16	1.461	1.724	2.668
17	1.826	1.938	3.000
18	1.251	1.760	2.725
19	1.092	1.473	2.280
20	1.055	1.307	2.024
21	1.055	1.277	1.976
22	1.057	1.246	1.929
23	0.932	1.142	1.768
Volume	0.982	1.102	12.2%
Peak	1.826	1.938	6.2%



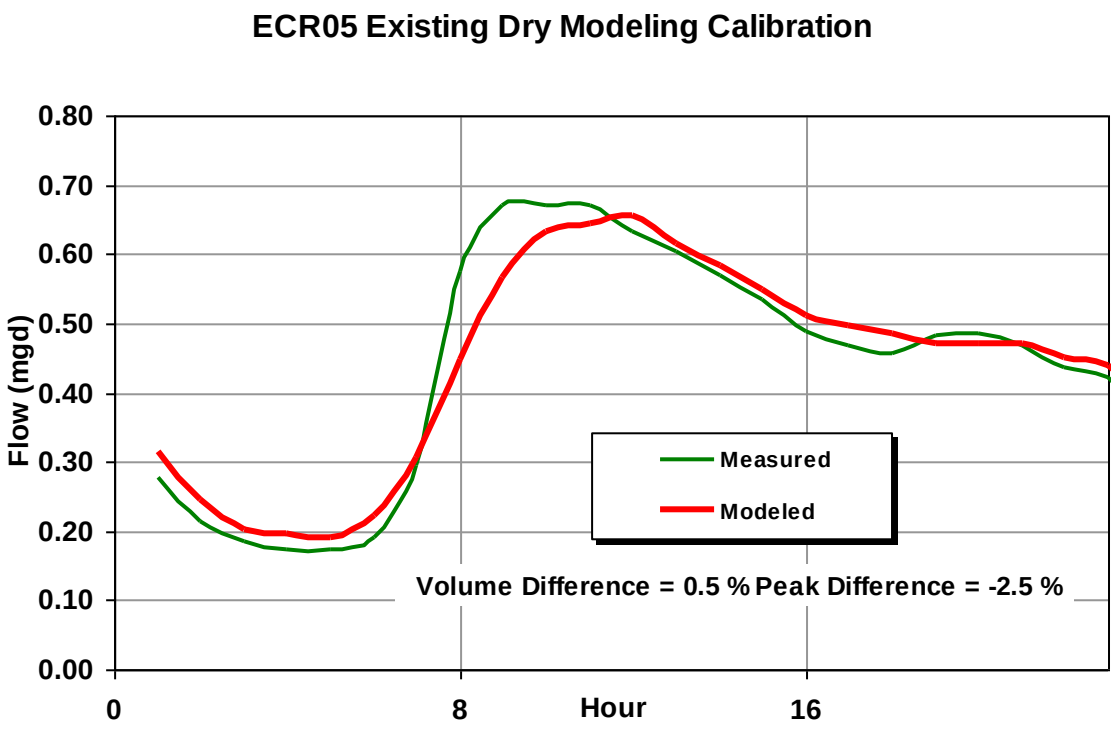
Modeling Results CRE03 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.568	0.645	0.998
1	0.470	0.535	0.828
2	0.389	0.464	0.718
3	0.341	0.447	0.692
4	0.333	0.427	0.662
5	0.334	0.492	0.762
6	0.464	0.599	0.928
7	0.858	0.726	1.125
8	1.083	0.876	1.356
9	1.182	1.003	1.553
10	1.263	1.079	1.671
11	1.269	1.133	1.754
12	1.228	1.133	1.754
13	1.155	1.130	1.749
14	1.089	1.074	1.662
15	1.021	1.017	1.575
16	0.945	0.964	1.492
17	0.925	0.941	1.457
18	0.909	0.918	1.422
19	0.918	0.916	1.417
20	0.917	0.913	1.413
21	0.849	0.873	1.352
22	0.771	0.822	1.273
23	0.683	0.741	1.146
Volume	0.832	0.828	-0.5%
Peak	1.269	1.133	-10.7%



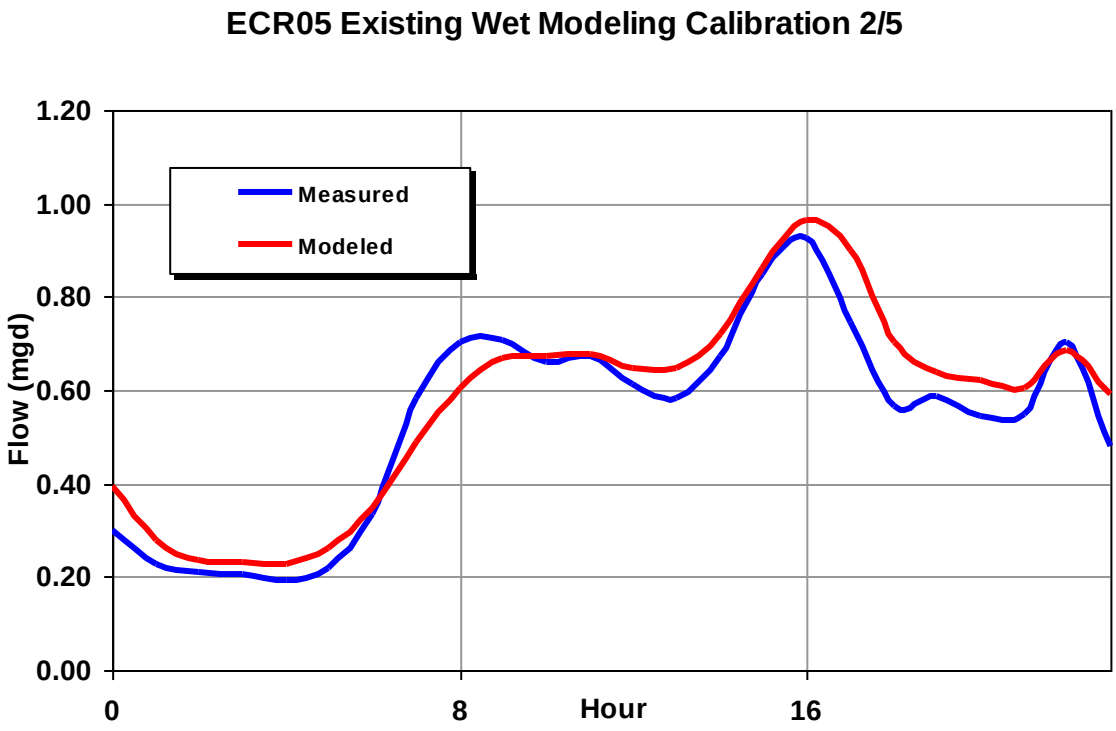
Modeling Results CRE03 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.555	0.737	1.141
1	0.431	0.547	0.846
2	0.397	0.516	0.798
3	0.355	0.485	0.751
4	0.366	0.454	0.703
5	0.375	0.534	0.826
6	0.496	0.635	0.984
7	0.938	0.781	1.210
8	1.079	0.901	1.395
9	1.172	0.998	1.545
10	1.252	1.082	1.676
11	1.247	1.105	1.710
12	1.211	1.127	1.744
13	1.107	1.153	1.785
14	1.123	1.175	1.820
15	1.255	1.295	2.005
16	1.478	1.437	2.224
17	1.849	1.561	2.416
18	1.275	1.472	2.279
19	1.023	1.246	1.929
20	1.006	1.113	1.724
21	1.001	1.038	1.607
22	1.020	1.051	1.628
23	0.832	0.941	1.456
Volume	0.952	0.974	2.4%
Peak	1.849	1.561	-15.6%



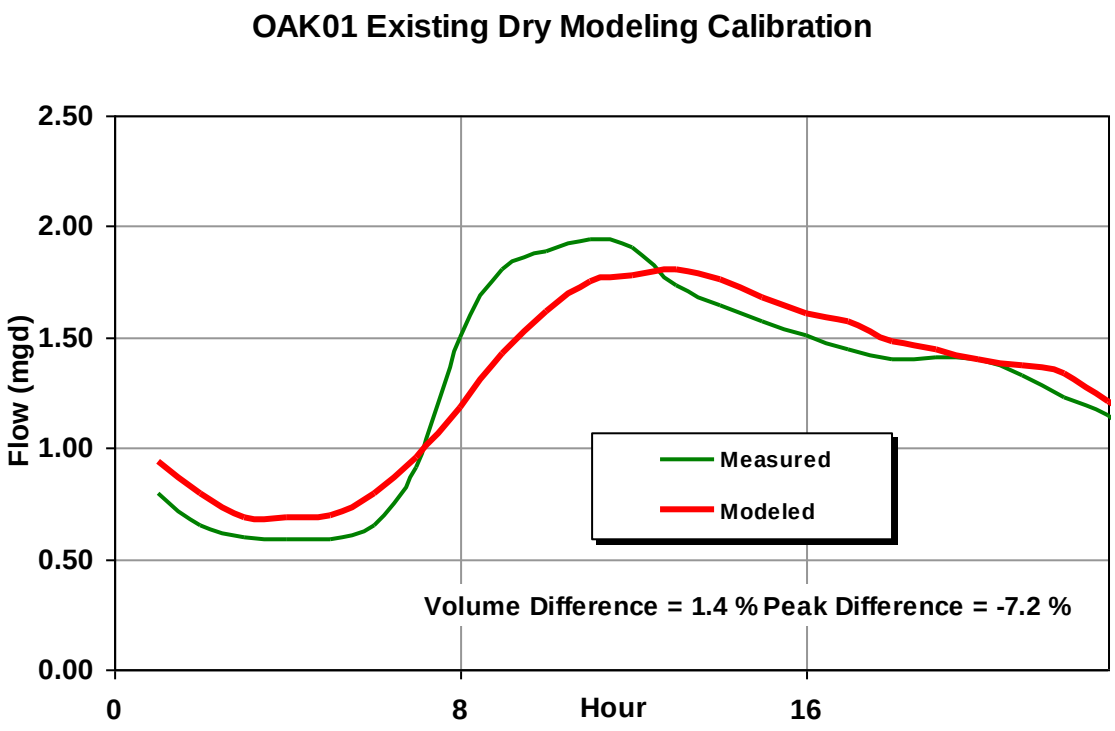
Modeling Results ECR05 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.276	0.314	0.486
1	0.213	0.245	0.380
2	0.184	0.201	0.311
3	0.172	0.197	0.305
4	0.173	0.191	0.296
5	0.191	0.221	0.342
6	0.293	0.306	0.474
7	0.575	0.447	0.692
8	0.671	0.566	0.876
9	0.671	0.632	0.979
10	0.671	0.644	0.996
11	0.633	0.655	1.014
12	0.603	0.616	0.953
13	0.570	0.582	0.902
14	0.534	0.549	0.850
15	0.488	0.510	0.790
16	0.469	0.497	0.770
17	0.457	0.484	0.750
18	0.481	0.471	0.730
19	0.486	0.471	0.730
20	0.468	0.469	0.727
21	0.435	0.451	0.698
22	0.420	0.440	0.681
23	0.353	0.380	0.589
Volume	0.437	0.439	0.5%
Peak	0.671	0.655	-2.5%



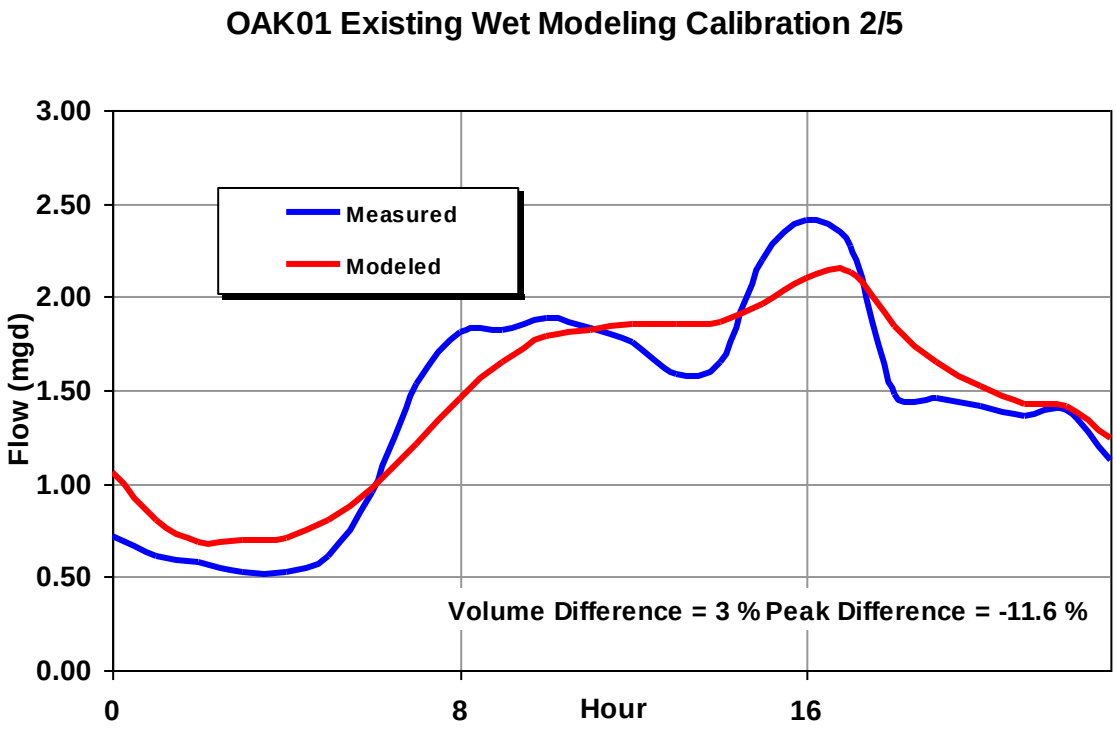
Modeling Results ECR05 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.298	0.396	0.613
1	0.229	0.278	0.431
2	0.211	0.234	0.362
3	0.205	0.231	0.358
4	0.194	0.228	0.353
5	0.217	0.260	0.403
6	0.334	0.346	0.535
7	0.581	0.487	0.754
8	0.703	0.605	0.936
9	0.705	0.667	1.032
10	0.662	0.673	1.041
11	0.671	0.675	1.046
12	0.614	0.646	1.000
13	0.585	0.649	1.005
14	0.675	0.720	1.114
15	0.848	0.861	1.333
16	0.927	0.964	1.492
17	0.744	0.899	1.392
18	0.568	0.702	1.087
19	0.588	0.640	0.991
20	0.546	0.623	0.964
21	0.547	0.605	0.936
22	0.701	0.687	1.064
23	0.482	0.593	0.918
Volume	0.535	0.570	6.5%
Peak	0.927	0.964	4.0%



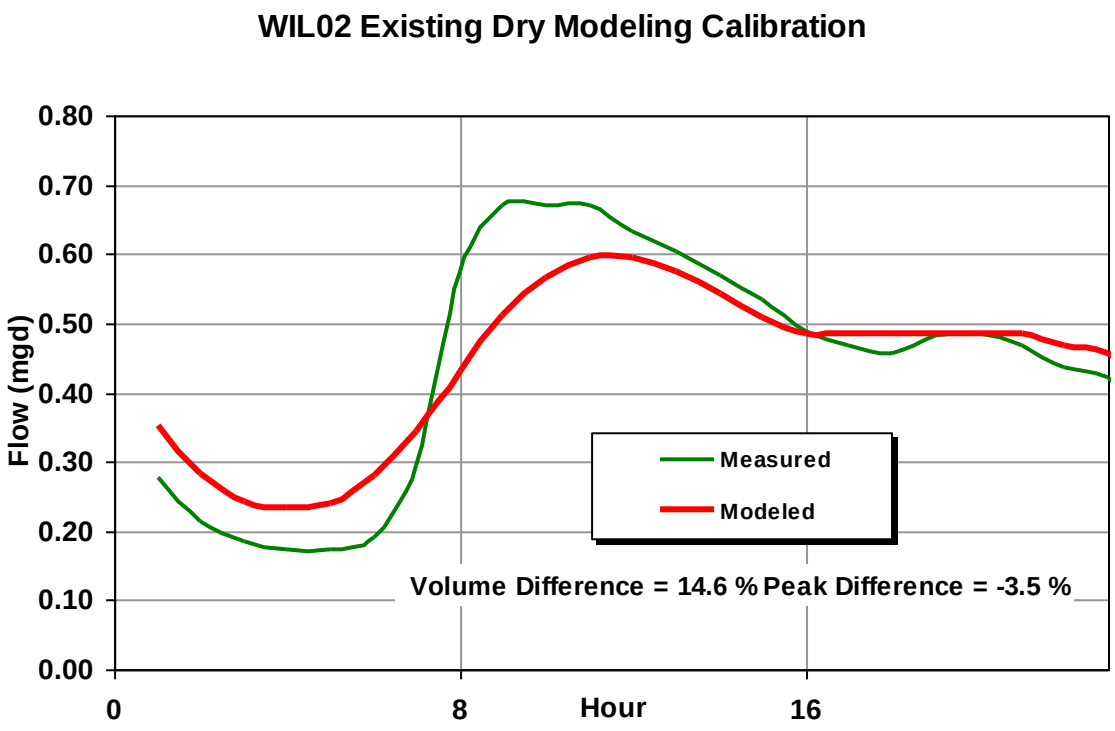
Modeling Results OAK01 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.792	0.935	1.447
1	0.649	0.792	1.225
2	0.595	0.685	1.060
3	0.589	0.689	1.067
4	0.588	0.693	1.073
5	0.652	0.792	1.225
6	0.914	0.957	1.481
7	1.494	1.185	1.834
8	1.801	1.426	2.207
9	1.886	1.618	2.504
10	1.940	1.747	2.705
11	1.903	1.774	2.746
12	1.735	1.801	2.787
13	1.643	1.761	2.725
14	1.574	1.680	2.601
15	1.507	1.604	2.483
16	1.447	1.573	2.435
17	1.403	1.479	2.290
18	1.406	1.444	2.235
19	1.398	1.403	2.172
20	1.323	1.368	2.117
21	1.228	1.332	2.062
22	1.147	1.211	1.875
23	0.996	1.100	1.702
Volume	1.276	1.294	1.4%
Peak	1.940	1.801	-7.2%



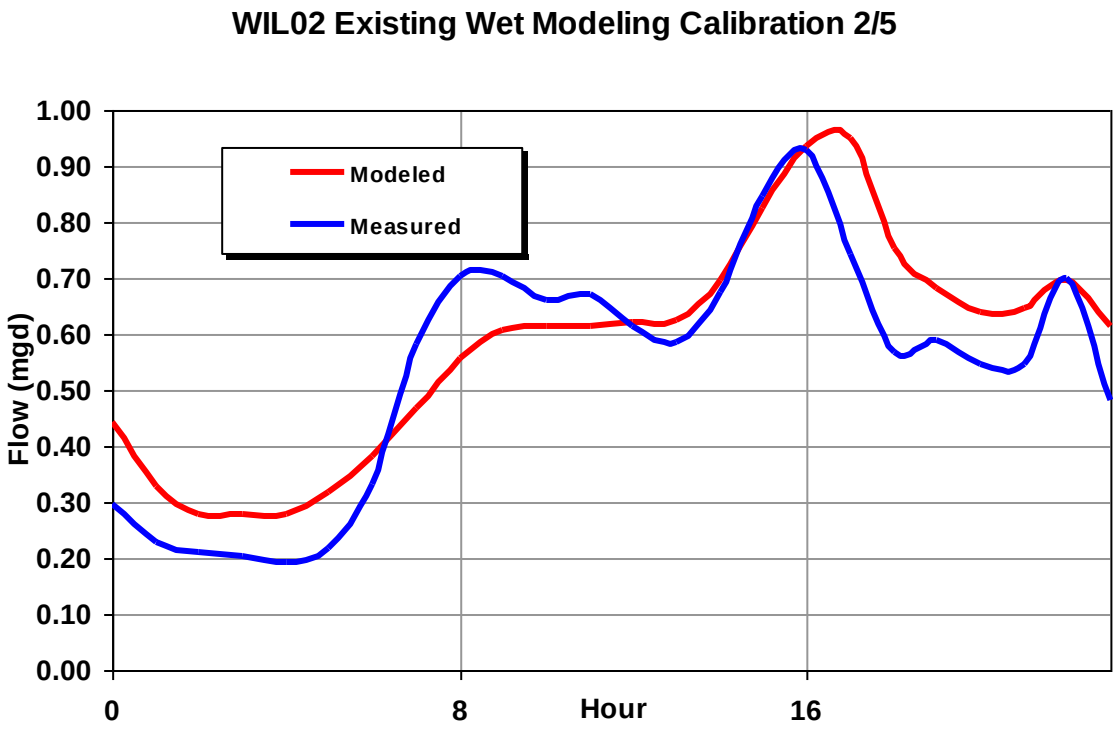
Modeling Results OAK01 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.723	1.060	1.642
1	0.616	0.801	1.240
2	0.581	0.686	1.061
3	0.528	0.691	1.070
4	0.529	0.703	1.088
5	0.611	0.801	1.240
6	0.955	0.980	1.517
7	1.536	1.210	1.874
8	1.815	1.458	2.258
9	1.821	1.655	2.561
10	1.888	1.787	2.767
11	1.830	1.822	2.820
12	1.755	1.856	2.874
13	1.588	1.856	2.874
14	1.648	1.862	2.883
15	2.197	1.966	3.043
16	2.406	2.104	3.258
17	2.268	2.127	3.293
18	1.480	1.839	2.847
19	1.456	1.655	2.561
20	1.417	1.522	2.356
21	1.363	1.424	2.204
22	1.396	1.412	2.186
23	1.127	1.245	1.927
Volume	1.397	1.438	3.0%
Peak	2.406	2.127	-11.6%



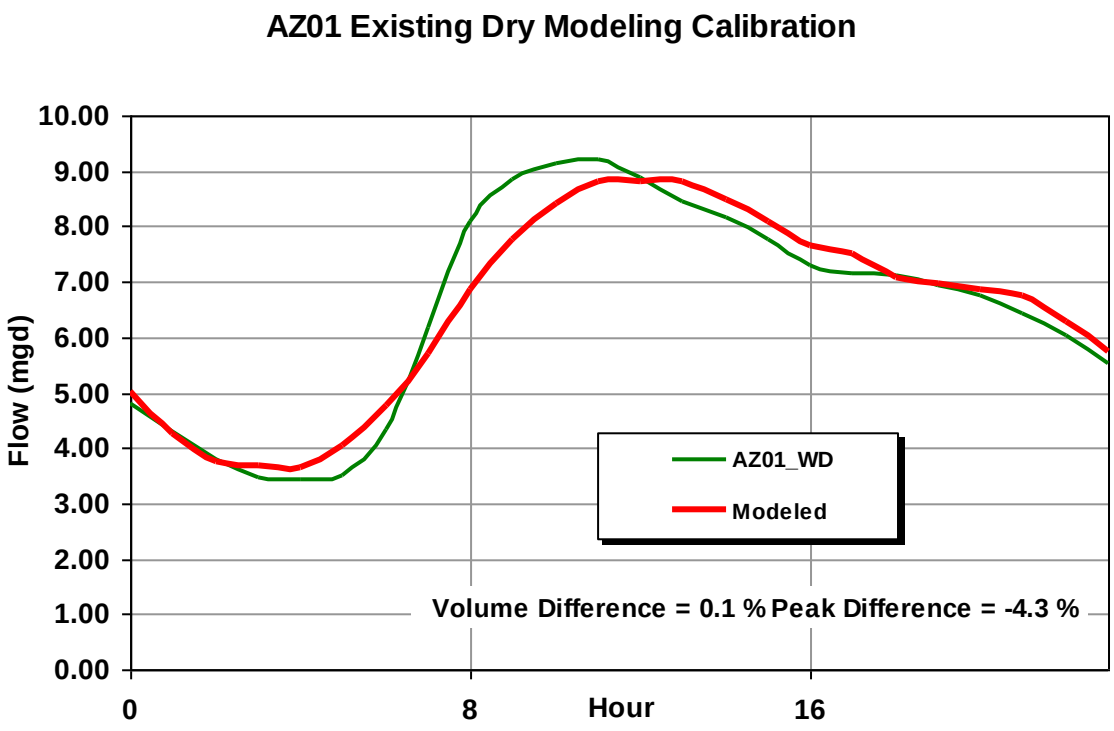
Modeling Results WIL02 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	0.276	0.351	0.543
1	0.213	0.283	0.438
2	0.184	0.243	0.376
3	0.172	0.234	0.362
4	0.173	0.240	0.371
5	0.191	0.279	0.432
6	0.293	0.344	0.532
7	0.575	0.429	0.664
8	0.671	0.510	0.789
9	0.671	0.566	0.877
10	0.671	0.594	0.919
11	0.633	0.595	0.921
12	0.603	0.574	0.888
13	0.570	0.542	0.839
14	0.534	0.509	0.787
15	0.488	0.484	0.749
16	0.469	0.484	0.749
17	0.457	0.484	0.749
18	0.481	0.484	0.749
19	0.486	0.484	0.749
20	0.468	0.485	0.751
21	0.435	0.467	0.722
22	0.420	0.457	0.707
23	0.353	0.406	0.628
Volume	0.437	0.439	0.4%
Peak	0.671	0.595	-11.3%



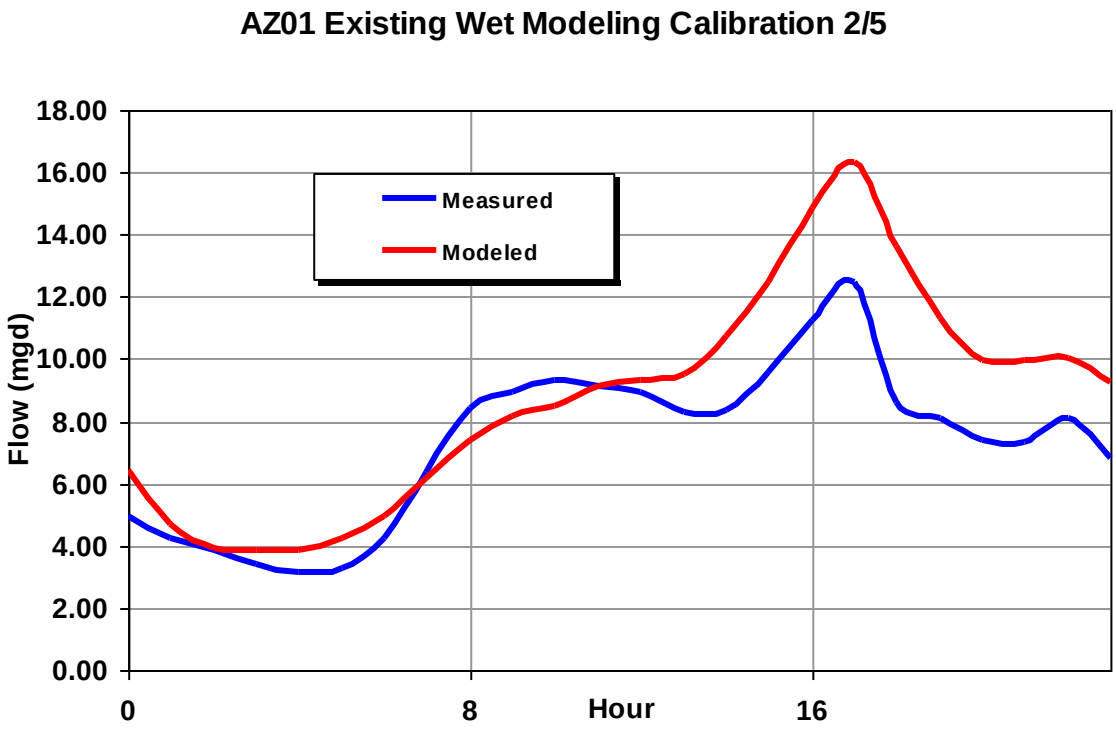
Modeling Results WIL02 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	0.298	0.441	0.683
1	0.229	0.328	0.508
2	0.211	0.280	0.433
3	0.205	0.280	0.433
4	0.194	0.277	0.429
5	0.217	0.317	0.491
6	0.334	0.382	0.591
7	0.581	0.468	0.724
8	0.703	0.557	0.862
9	0.705	0.608	0.941
10	0.662	0.613	0.949
11	0.671	0.616	0.953
12	0.614	0.621	0.962
13	0.585	0.624	0.966
14	0.675	0.697	1.078
15	0.848	0.826	1.278
16	0.927	0.936	1.449
17	0.744	0.949	1.470
18	0.568	0.753	1.166
19	0.588	0.680	1.053
20	0.546	0.640	0.991
21	0.547	0.645	0.999
22	0.701	0.697	1.078
23	0.482	0.613	0.949
Volume	0.535	0.577	7.9%
Peak	0.927	0.949	2.4%

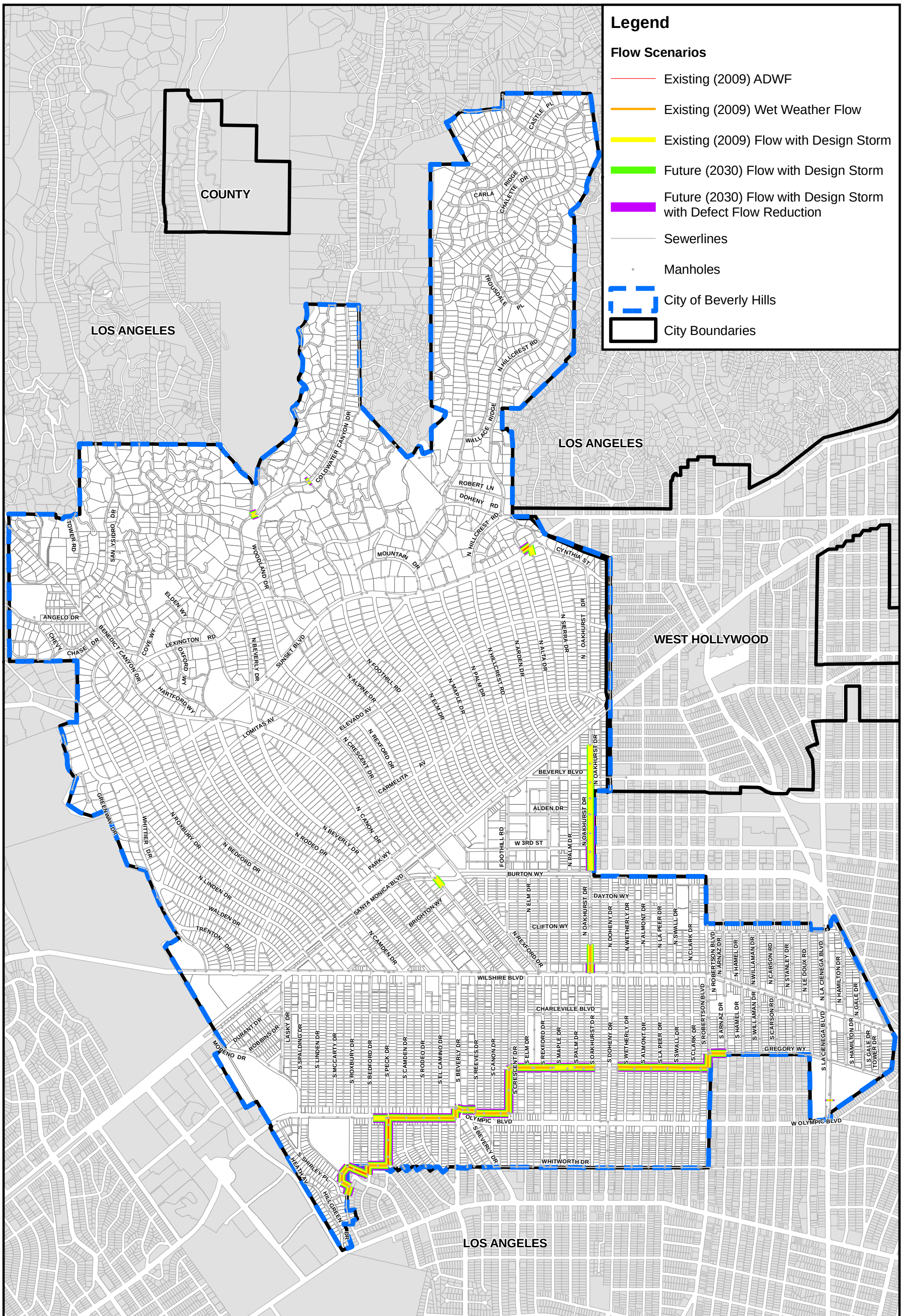


Modeling Results AZ01 Existing Dry Modeling Calibration			
Hour	Measured	Modeled	Mod_cfs
0	4.813	5.016	7.764
1	4.299	4.253	6.584
2	3.787	3.737	5.785
3	3.481	3.695	5.719
4	3.435	3.653	5.655
5	3.491	4.041	6.255
6	4.344	4.765	7.376
7	6.187	5.721	8.856
8	8.103	6.842	10.592
9	8.833	7.772	12.031
10	9.147	8.411	13.020
11	9.212	8.801	13.623
12	8.880	8.819	13.652
13	8.434	8.816	13.646
14	8.168	8.487	13.137
15	7.753	8.073	12.496
16	7.308	7.659	11.856
17	7.142	7.494	11.601
18	7.107	7.084	10.965
19	6.932	6.980	10.805
20	6.748	6.877	10.645
21	6.431	6.750	10.449
22	6.047	6.276	9.714
23	5.524	5.739	8.883
Volume	6.484	6.490	0.1%
Peak	9.212	8.819	-4.3%



Modeling Results AZ01 Existing Wet Modeling Calibration 2/5			
Hour	Measured	Modeled	Mod_cfs
0	4.945	6.461	10.001
1	4.261	4.676	7.239
2	3.830	3.935	6.091
3	3.413	3.883	6.010
4	3.155	3.830	5.929
5	3.259	4.224	6.539
6	4.215	4.971	7.695
7	6.393	6.170	9.551
8	8.395	7.418	11.483
9	8.920	8.158	12.629
10	9.303	8.455	13.089
11	9.111	9.147	14.159
12	8.956	9.342	14.461
13	8.281	9.537	14.763
14	8.371	10.725	16.602
15	9.597	12.490	19.334
16	11.240	14.861	23.004
17	12.483	16.331	25.281
18	8.624	13.627	21.095
19	8.116	11.335	17.546
20	7.364	9.938	15.384
21	7.300	9.990	15.464
22	8.097	10.015	15.503
23	6.829	9.259	14.333
Volume	7.269	8.699	19.7%
Peak	12.483	16.331	30.8%





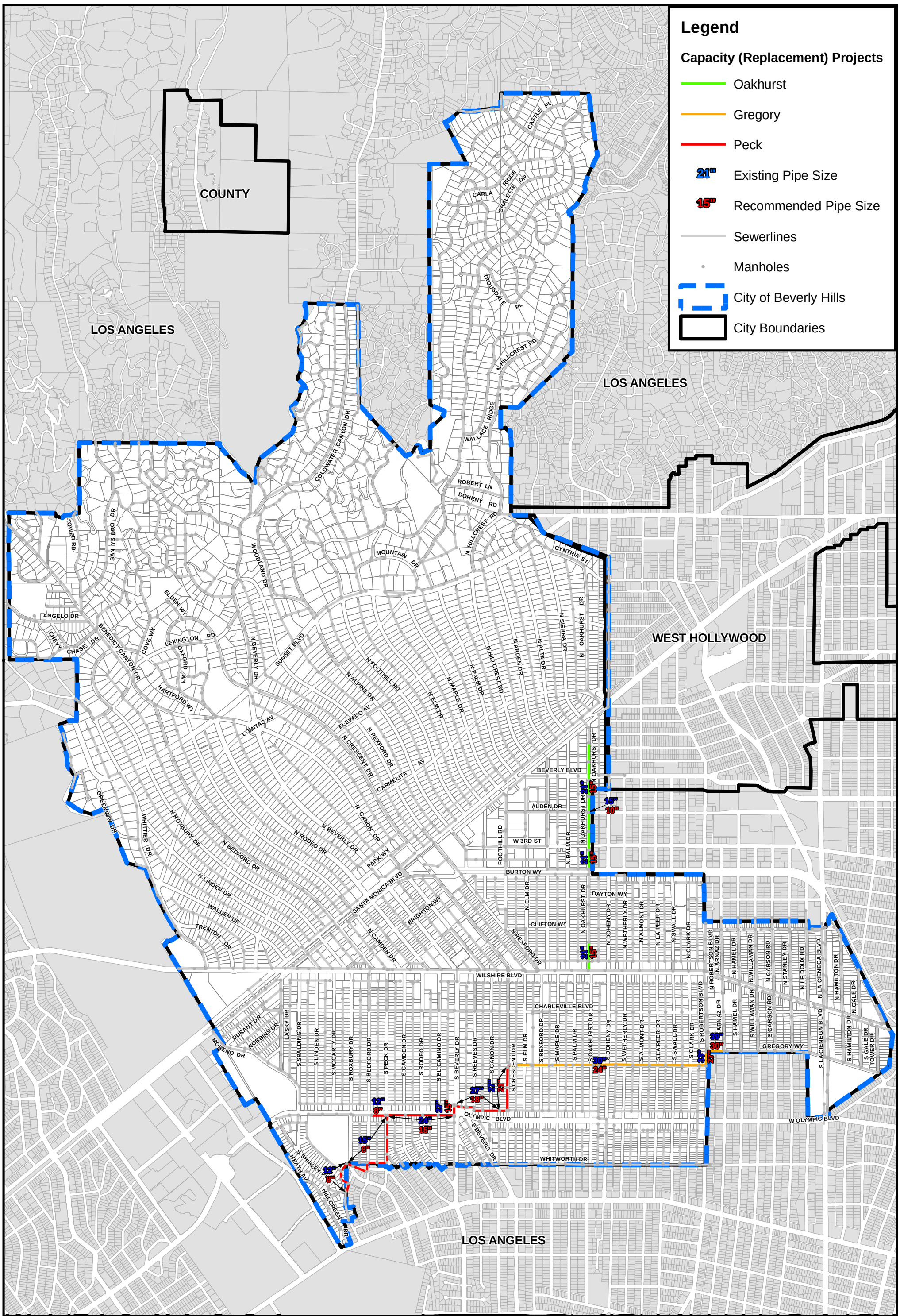
City of Beverly Hills

Deficient Pipeline Summary

Exhibit 6-1

Source:
City of Beverly Hills, 2009

M:\Mdata\10106431\GIS\Ex6-1_DeficientPipelines_11X17P.mxd 9/02/10 - DK



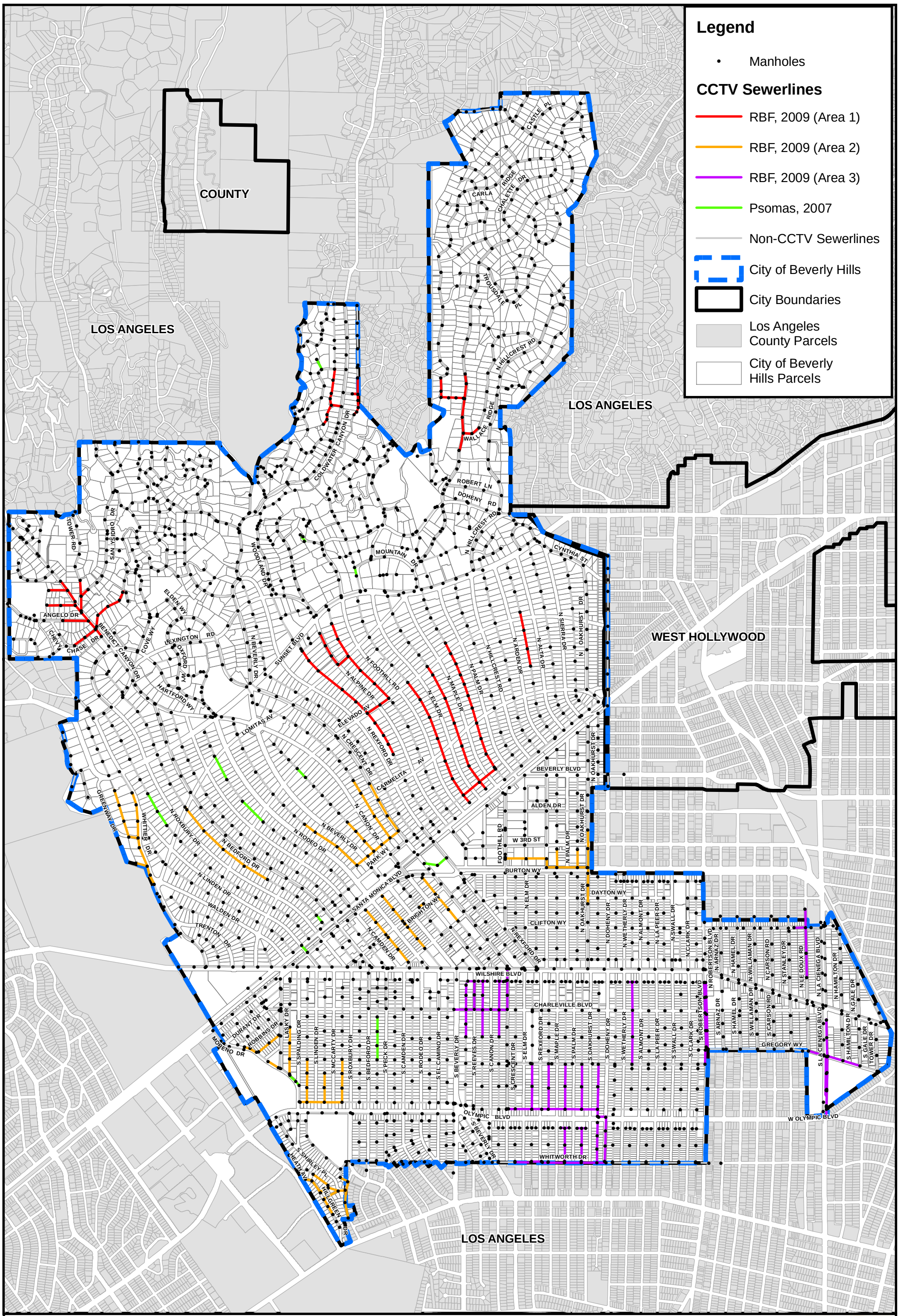
City of Beverly Hills

Capacity Analysis Recommendations

Exhibit 6-2

Source:
City of Beverly Hills, 2009

M:\Mdata\10106431\GIS\Ex6-2_CapacityAnalysisRecommendations_11X17P.mxd 9/02/10-- DK



City of Beverly Hills

Pipeline and Manhole Inspections

Exhibit 7-1

Source:
City of Beverly Hills, 2009

M:\Mdata\10106431\GIS\Ex7-1_Pipeline&ManholeInspections_11X17P.mxd 9/3/10 -- DK