City of Grass Valley
Wastewater System Master Plan

August 23, 2016

Prepared for:
City of Grass Valley

Prepared by:
Stantec Consulting Services Inc.
101 Providence Mine Road, Suite 202
Nevada City, California 95959

Stantec

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Sign-off Sheet

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Executive Summary

The City of Grass Valley (City) Wastewater Master Plan (Master Plan) is intended to provide guidance to the City on the management of their existing wastewater treatment plant (WWTP), collection system and associated assets. The Master Plan provides assessments of the existing collection system and treatment plant capacity as well as options for providing additional capacity for potential future development. The Master Plan also describes collection system and WWTP regulatory concerns and potential upgrades to address those concerns. The scope of this master planning effort includes the following major elements:

- Review of existing reports, drawings, land use and zoning maps, and other relevant information.
- Evaluation of existing facilities and operational data, which was used to conduct an assessment of system capacity and condition.
- Projection of future water demands based on historical water use and land use as defined in the City's 2020 General Plan.
- Development of a list of system assets, incorporation of those assets into an electronic database, including updated asset information some of which was not available in a central database, previously, to project repair and replacement costs for the system over time.
- A list of recommended improvement projects, and opinions of probable cost for implementation.

ES-1 Overview

The City currently provides sewer service to a resident population of approximately 12,668 people (Source: CA Dept. of Finance estimate as of January 1, 2014). In addition, the City serves a number of industrial and commercial users whose businesses are located within the City's sewer service area.

The City's sanitary sewer collection system serves an area of approximately 2,630 acres with approximately 61.5 miles of gravity sewer varying in size from 4 inches to 36 inches and nearly 1,400 manholes. Of this system, approximately 59.2 miles of pipe flow by gravity, and between 2 and 3 miles are pressurized pipes fed by pump stations. The system has seven (7) active lift stations that are maintained by City operations personnel.

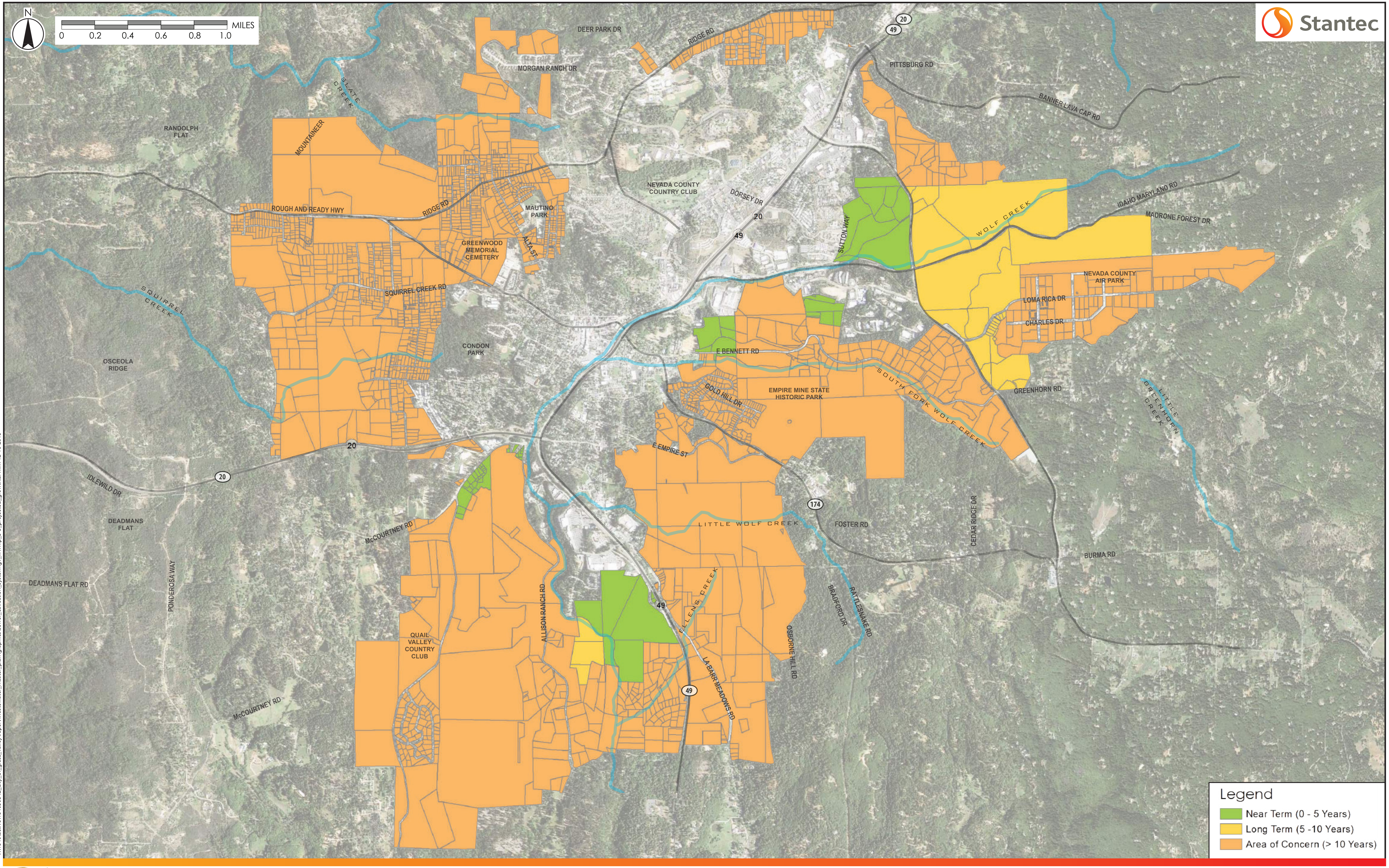
The City's sewer collection system delivers flow to the City's Wastewater Treatment Plant (WWTP). The WWTP includes a nitrification/denitrification activated sludge treatment process followed by advanced treatment facilities to produce a filtered and disinfected equivalent tertiary effluent for discharge to Wolf Creek. The design capacity of the WWTP is 2.78 Mgal/d on an average dry weather flow basis with a peak flow capacity through the activated sludge system of 7.0 Mgal/d. The collection system conveys an average annual flow of approximately 2.2 million gallons per day (Mgal/d) of raw wastewater to the City's WWTP.

ES-2 Collection System

Existing flows are used to identify existing system deficiencies, which are the basis for recommended improvements. Information on existing and future land uses established in the City of Grass Valley 2020 General Plan were used as the basis for developing wastewater flow generation estimates. The future land uses form the basis for estimating additional flows which the system must convey to the WWTP, and are used to determine recommended future improvements. This Master Plan assesses system performance for five projected growth scenarios for the City. The following growth scenarios were selected by the City to allow an analysis of necessary system improvements and their possible timing. The extents of the projected growth horizons are shown in **Figure ES-1**.

- Existing: The current level of development in the City's service area
- Existing plus Infill Development (Existing Build-out): includes Existing development as well as development of all vacant parcels within the City's existing service area.
- Near-term development: Areas which may develop within 5 years
- Long-term Development: Areas which may develop within 10 years
- Area of Concern: Areas that may develop in future years

Dry weather wastewater generation rates and average annual flow for the City's service area were developed using the City's design standards in conjunction with flow monitoring that was conducted in spring of 2014 and influent flow records from the City's WWTP. Existing wet weather peak flows were developed based on wet weather collection system flow monitoring as well as WWTP influent flow records. Future wet weather peak flows were based on estimates of rainfall dependent inflow and infiltration (RDII) for existing areas of the City that most closely resemble future development. The wastewater flow projections used in the collection system and WWTP analysis are shown in **Table ES-1**. It was assumed that the existing development areas will continue to generate peak flows in the future at a similar rate as exhibited at the time of this study.



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City of Grass Valley
Wastewater System Master Plan

Figure ES-1
Projected Growth

Table ES-1 Wastewater Flow Projections

Growth Horizon	ADWF (Mgal/d)	AAF	PMF	PDF	PHF
Existing Conditions	1.3	2.21	5.33	10.01	18.9
Vacant Parcels Within City Limits	1.6	2.62	6.10	11.32	20.5
Near Term	1.9	3.02	6.86	12.62	21.80
Long Term	2.1	3.29	7.37	13.49	23
Areas of Concern	4.0	5.86	12.22	21.76	39.7

Stantec developed a computer model of the City's wastewater collection system for purposes of assessing existing available capacity and the possible need for upgrades to serve future growth scenarios. PCSWMM software, developed by Computational Hydraulics Inc., was selected for use in developing a collection system computer model for this Wastewater Master Plan. The sewer model only includes the primary trunks where detailed elevation data was provided, a process typically referred to as a "skeleton" model of approximately 18 miles of sewer line in the Existing model. A 1:10 year return period storm, with a 24-hour duration following the Huff design storm distribution was selected to assess system capacity under wet weather conditions. The five growth scenarios described above were evaluated using the model to identify system deficiencies.

The City's Level of Service (LOS) criteria was applied to the gravity sections of the sewer collection system to locate areas with limited capacity based on the results of the various scenario simulations. The maximum allowable surcharge in the gravity portion of the sanitary sewer system must remain at least 8 feet from the ground surface (at least 8 feet of freeboard is required) during a design storm scenario. Under this criterion, existing sewers with depths greater than 8 feet have been said to be within LOS criteria if the peak surcharge elevation results in a freeboard of greater than 8 feet. Any sewer identified with depths less than 8 feet are considered deficient should any surcharging result at depths greater than 1 foot above the pipe crown. Thus, the recommended improvements identified in the Master Plan are generally based upon the two criteria below:

a. minimum freeboard 8 feet(depth below rim)

b. surcharging less than 1 foot above pipe crown

Results of the sewer collection system capacity analysis for existing and projected future flows are presented in Chapter 4 of this Master Plan. The sections of pipe which the model predicts to be deficient for each of the growth scenarios are identified in the figures in Chapter 4.

Lift Stations

Lift station design capacity was also assessed with the hydraulic model for all of the growth scenarios using the 1:10 year return period design storm. Inflow hydrographs were compared to the pumping capacity of the lift station to identify any potential capacity constraints. Morgan Ranch Lift Station and Slate Creek Lift Station are currently close to their full capacity and are projected to need additional pumping capacity to support the build-out of the existing City Limits. Growth beyond the City Limits will trigger other pump station improvements dependent upon the location of the future development.

Inflow and Infiltration

As with other foothill communities the City has Inflow and Infiltration (I/I) in its collection system. I/I is water that makes its way into the collection system either through storm related direct inflow (through cracks around manhole rims, cracked pipes, failing or improperly connect service laterals, etc.) or infiltration such as in areas of elevated groundwater which finds its way into the system through failed pipes or joints. I/I is a serious problem that cannot be eliminated completely, but it must be controlled to the extent feasible. Control is achieved by on-going collection system maintenance activities; replacement of pipe segments known to be damaged, or nearing the end of their expected service life; and aggressive enforcement of ordinances developed to minimize private service lateral I/I to the extent feasible.

ES-3 Wastewater Treatment Plant

The Grass Valley WWTP is a nitrification/denitrification activated sludge treatment system with advanced tertiary treatment facilities. The plant is comprised of a headworks (screening and grit removal) with odor control, primary treatment (primary clarifiers), and secondary treatment (aeration basin and secondary clarifier). Secondary effluent is filtered and then disinfected using ultraviolet (UV) light before it is discharged to Wolf Creek. Primary sludge from the primary clarifiers along with waste activated sludge (WAS) from the secondary treatment process is fed to an anaerobic digester for solids stabilization. The WWTP currently produces Class B biosolids which are taken by Synagro and land applied.

Biologically, the WWTP can handle the average flow of 2.78 Mgal/d (the design ADWF) with a peak flow through secondary treatment of up to 10.0 Mgal/d. In the late 1990's upsizing of the main trunk sewer was undertaken to alleviate sewer overflows from the collection system. As a result of these interceptor upgrades, which removed capacity restrictions causing the overflows, the wastewater is conveyed to the treatment plant much faster than before causing the peak influent flow at the plant to increase. The influent flow must be either treated as it comes to the plant or equalized in storage until it can be treated later.

Even though the current ADWF (1.3 Mgal/d) is less than 50% of the design ADWF, the peak flows that the plant currently receives exceed the design peak flows. Currently, the plant design peak hour flow is around 16 Mgal/d and the measured peak hour flows at the plant are 18.9 Mgal/d. These increased peak wet weather conditions at the WWTP are an indication that the collection system is in need of rehabilitation in order to minimize the amount of I/I reaching the WWTP.

The heart of the wastewater treatment plant and the key feature that determines plant capacity is the secondary treatment system, which includes the biological reactor basins and the secondary clarifiers. The WWTP is designed for an average dry weather flow of 2.78 Mgal/d. The secondary treatment was designed for a maximum flow of 7.0 Mgal/d. Filters and disinfection processes are also designed for approximately 7.0 Mgal/d. Any flow in excess of 7.0 Mgal/d is diverted to the 6.1 Mgal equalization storage basin.

Four measures were considered in addressing the hydraulic capacity constraints at the WWTP:

1. Rehabilitate the existing collection system to reduce I/I
2. Provide more equalization storage volume
3. Improve plant hydraulics to push more flow (>7.0 Mgal/d) through the secondary and tertiary treatment systems
4. Provide Side-stream treatment for peak flow

The results of the treatment plant analysis are presented in Chapter 5.

ES-4 Improvement Projects

The extent of the predicted sewer and WWTP upgrades described in this Master Plan are highly dependent on the amount of I/I in the system. The City is currently planning to implement an I/I reduction project in targeted areas of the collection system. The cost of the City's planned initial I/I project is estimated to be approximately $5M. The City is planning to fund this project with a combination of City funds and up to $4M in grant funding from the State Water Board Clean Water State Revolving Fund. This Master Plan recommends the City evaluate the effectiveness of these I/I reduction efforts as they are implemented, re-assess the improvements described here to address existing and anticipated capacity concerns, then adjust those planned upgrades as appropriate (e.g. reducing the magnitude of upgrades recommended).

The opinion of probable cost for the capacity related collection system improvements (pipes and pump stations) recommended as a result of the capacity analysis and system condition assessment conducted with this master planning effort for Existing conditions are summarized in **Table ES-2**.

Table ES-2 Opinion of Probable Cost for Improvements in Existing System

Pipeline Improvements [a]		
Diameter (inches)	**Length (feet)**	**Opinion of Probable Costs**
8 inch	2769	$700,000
10 inch	2420	$716,000
12 inch	615	$209,000
15 inch	304	$114,000
Pipeline Subtotal		$1,739,000
Environmental, Engineering, Construction Management, 30%		$521,700
Contingency, 30%		$521,700
	Subtotal	**$2,780,000**
Lift Station Improvements		
Slate Creek and Morgan Ranch Lift Station Upgrades		$40,000
Environmental, Engineering, Construction Management, 35%		$14,000
Contingency, 40% [b]		$16,000
	Lift Station Subtotal	**$70,000**
	Total	**$2,846,000**

(a) All costs assume a 12 foot depth and replacement of manholes every 250 feet at a cost of $20,000 each. Installation cost of 8-inch to 12-inch pipeline is calculated based on a cost of $18/linear foot/inch diameter. Installation cost of 15-inch pipeline is calculated based on a cost of $246/linear foot.

(b) Lift Station Improvements include additional contingency to allow for unknowns related to electrical systems and control components.

In addition to improvements that the City considers critical to address collection system capacity constraints, segments of the City's collection system are 80 to 100 years old. These segments of the collection system are considered to have reached the end of their useful life. The remaining components of the collection system continue to age and warrant replacement at the appropriate time to avoid significant portions of the system exceeding the useful life of the materials of construction and increasing the risk of failure. As such, the City proposes to continue its program of repair and replacement of collection system assets on a regular basis. Currently the City intends to fund approximately $200,000 of repair and replacement projects per year. Some of these aged collection system components are coincident with the needed improvements listed in **Table ES-2**, but not all. Some will result in repair and replacement projects in addition to those projects listed to alleviate existing capacity constraints.

WWTP improvements that the City is considering are presented below. The timing and extent of implementation of these project components will be highly dependent on the effectiveness of the City's I/I reduction efforts.

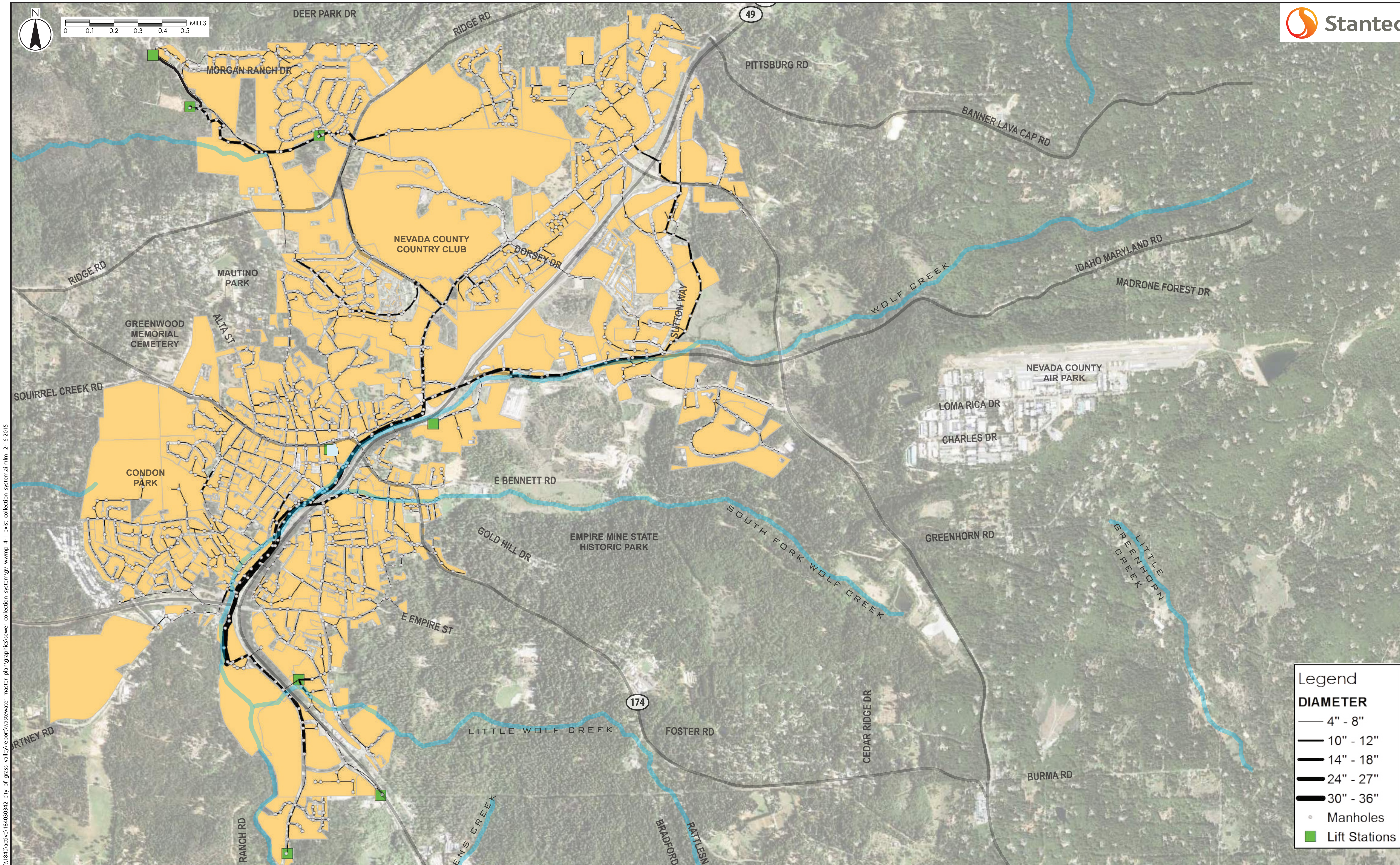
- Automate Diversion gate ahead of the primary clarifiers to send screened wastewater to Equalization Basins
- Add additional Equalization Storage
- Upsize the Equalization pipeline or provide Equalization Pumps to increase flow to Equalization Basins
- Improve Plant Hydraulics through Secondary Treatment Process (>10 Mgal/d, peak flow)
- Upsize filter supply pumps (>10 Mgal/d, peak flow)
- Expand tertiary filter capacity (>10 Mgal/d, peak flow)
- Expand UV system capacity (>10 Mgal/d, peak flow)
- Repair/Refurbish the gravity Belt Thickener

Combined these additional upgrades are estimated to have an approximate total project cost of $6.8 M.

1.0 INTRODUCTION

The City of Grass Valley (City) currently collects and treats wastewater from an area of approximately 2,430 acres, serving a population of approximately 12,668 people, as well as a number of industrial and commercial users. The area to which the City currently provides sewer service is identified in **Figure 1-1.** The City's Wastewater Master Plan (Master Plan) is intended to provide guidance to the City on the management of their existing wastewater treatment plant (WWTP), collection system and associated appurtenances. The Master plan provides assessments of the existing collection system and treatment plant capacity as well as options for providing additional capacity for potential future development. The Master Plan also describes collection system and WWTP regulatory concerns and potential upgrades to address those concerns. The scope of this master planning effort includes the following major elements:

- Review of existing reports, drawings, land use and zoning maps, and other relevant information
- Evaluation of existing facilities and operational data, which was used to conduct an assessment of system capacity and condition.
- Projection of future water demands based on historical water use and land use as defined in the City's 2020 General Plan.
- Development of a list of system assets, incorporation of those assets into an electronic database, including updated asset information some of which was not available in a central database, previously, to project repair and replacement costs for the system over time.
- A list of recommended improvement projects, and opinions of probable cost for implementation.



City of Grass Valley
Wastewater System Master Plan

Figure 1-1
Sewer Service Area

2.0 REGULATORY REQUIREMENTS AND COMPLIANCE

This section describes the regulatory setting under which the City's wastewater utility operates, and areas where improvement to the wastewater utility are believed to be necessary to maintain compliance with regulatory requirements. The regulatory authority under which the WWTP operates is the California Regional Water Quality Control Board, Central Valley Region (Regional Water Board). The Regional Water Board adopts orders at public hearings based on law, regulations, policies, evidence, and testimony. These orders prescribe the conditions under which the wastewater utility must be operated to protect public health and the environment. The City has one order covering the wastewater collection system (i.e., the buried sewer pipes conveying sewage from the property lines of homes and businesses to the wastewater treatment plant) and a separate order covering the wastewater treatment plant and its discharge of treated wastewater (termed "effluent") to Wolf Creek. The sewer pipe from the home or business to its connection to the City's sewer system is private property that is operated and maintained by the property owner under various City requirements.

2.1 WASTEWATER COLLECTION SYSTEM

The City's wastewater collection system collects and conveys wastewater from a service area of approximately 4.1 square miles. The collection system consists of 61.5 miles of pipe ranging in diameter from 4 to 36 inches. Of this, approximately 59.2 miles of pipe flow by gravity, and between 2 and 3 miles are pressurized pipes fed by pump stations. Access to gravity sewer pipes to allow maintenance is provided via manholes. The City's collection system includes approximately 1,395 manholes. Most manholes are located in streets.

2.1.1 Wastewater Collection System Regulatory Requirements

The City's collection system is permitted to operate under State Water Board adopted Water Quality Order 2006-003-DWQ, "Statewide General Waste Discharge Requirements for Sanitary Sewer Systems" (General Order). The City has developed a Sewer System Management Plan (SSMP) that describes how the City operates and maintains the collection system in compliance with General Order requirements.

2.1.2 Wastewater Collection System Regulatory Concerns

The only known problem with the wastewater collection system from a regulatory perspective is the occurrence of spillage of sewage from the collection system. This has been a problem, as for many foothill agencies that the City has dealt with historically. In recent years the City has made efforts to correct these issues and the occurrence of these events has reduced accordingly. Spills reportedly result from 1) partial blockage of a sewer pipe which reduces the hydraulic capacity of the pipe, and 2) flows in excess of the hydraulic capacity of a pipe even if in perfect condition. Causes of partial blockages include:

- Root intrusion (i.e., roots seeking water and nutrients/fertilizer from the wastewater in the pipe)
- Debris dumped or flushed into a sewer
- Buildup of cooking fats, oils, and grease as it congeals in sewers
- Deterioration and/or breakage of pipe material over time

The primary causes of high sewer flows in excess of the design capacity of any particular pipe in the sewer system are inflow of surface water (stream flow, precipitation, snow melt, etc.) and infiltration of shallow groundwater resulting from stream flow, precipitation, snow melt, etc. Inflow and infiltration (I/I) can occur at several points in the overall collection system including damaged pipes and pipe joints, leaking manholes, private sewer pipes serving homes and businesses, etc.

To control I/I, the City has an on-going maintenance and inspection program (as described in the SSMP) for its portion of the sewer system. The City has ordinances regulating what can be discharged lawfully to the sewer system. The City also has ordinances regulating the private sewer service laterals connected to the City's sewer system. Regulation of the private sewer service laterals includes banning connection of roof and yard drains to the sewer service lateral, and similar provisions developed to minimize the entry of I/I into the private service lateral, and therefore into the City's sewer system.

I/I is a severe and on-going problem in foothill communities (compared to most valley communities) for several reasons:

- Increased precipitation at higher elevations, i.e., greater potential for I/I
- Greater chance for snow and subsequent snow melt, which generally causes more I/I than rainfall
- Shallow top soil underlain with bedrock, results in shallower sewers that are more easily damaged 1) by overlying activities (traffic, construction, etc.), and/or 2) by settling causing the pipe to come in contact with (and be potentially damaged by) the irregular surface of the underlying bedrock.
- The shallow, sloped bedrock/soil interfaces underlying portions of many foothill communities often intersect sewer trenches such that the sewer pipe trenches and granular backfill material act as drainage pathways for shallow, perched groundwater resulting from precipitation. Consequently, many foothill sewer pipes, joints, and manholes are inundated in perched, shallow groundwater of precipitation origins.

I/I is a serious problem that cannot be eliminated, but it must be controlled to the extent feasible. Control is achieved by on-going collection system maintenance activities; replacement of pipe segments known to be damaged, or nearing the end of their expected service life; and aggressive enforcement of ordinances developed to minimize private service lateral I/I to the extent feasible.

Because I/I is so difficult to eliminate, collection system measures to reduce hydraulic capacity constraints often result in more I/I driven wastewater flow reaching the WWTP. Thus, correcting capacity problems in the collection system often exacerbate hydraulic flow rate and volume problems at the WWTP. Consequently, hydraulic capacity correction efforts must be planned in concert with WWTP improvements needed such that fixing capacity problems in the collection system does not create WWTP problems.

2.2 WASTEWATER TREATMENT PLANT (WWTP)

2.2.1 WWTP Regulatory Requirements

Conditions regulating the lawful operation and maintenance of the wastewater treatment plant (WWTP), including the discharge of effluent to Wolf Creek, are specified in Regional Water Board Order No. R5-2009-0067 (2009 Order), which is scheduled for revision and renewal in February 2016 (2016 Order). The proposed revisions have been circulated for City and public comment, and reduce the number of constituents with effluent limitations from 15 down to 8 based on both WWTP improvements and changes to Regional Water Board policies. The WWTP is not expected to have problems complying with the proposed 2016 Order effluent limitations. The current 2009 Order and proposed 2016 Order also contain several other requirements related to WWTP operations. These other requirements include effluent testing requirements, general operational and performance requirements for the WWTP, a prohibition on the WWTP from creating nuisance conditions, and control of how solids removed by the WWTP are disposed of, etc.

2.2.2 WWTP Regulatory Concerns

Major regulatory concerns with the current WWTP appear to be focused on two areas: high I/I-based influent wastewater flows exceeding the hydraulic capacities of components of the WWTP, and spills from the solids handling aspects of the overall WWTP process.

High I/I flows causing wastewater spills from the WWTP have been a problem on occasion for the City. As noted previously, City efforts to address capacity limitations in the collection system typically result in an increased I/I hydraulic load on the WWTP. Though the City will be taking steps to increase the hydraulic capacity of the WWTP in concert with collection system improvements, as well as undertaking operational changes which the City believes will reduce the risk of overflow, the concern with overflows at the WWTP remain, primarily because the WWTP site is in a canyon setting with adjacent flatter land already developed. Identifying alternative solutions to the interconnected problems of high I/I, collection system capacity deficiencies, WWTP hydraulic capacity, and site limitations is a key element of this Master Plan.

2.3 FUTURE REGULATORY TRENDS

At this time, the wastewater regulatory environment appears to be relatively stable. From the City's current perspective, the key regulatory concerns for the future are preventing releases from the collection system, preventing I/I-caused spills at the WWTP, and preventing spills at the WWTP caused by plugged pipes, valves, and outlets in the solids handling portion of the WWTP.

Other regulatory trends warranting comment include contaminants of emerging concern (CECs), episodic three-tier bioassay failures, and effluent salinity. Each of these issues is discussed briefly in the following sections.

2.3.1 Contaminants of Emerging Concern (CECs)

CECs include a wide range of household products, personal care products, and pharmaceutical residuals, that are not removed fully by conventional wastewater treatment processes, that react biochemically in various adverse ways, and are an integral part of modern day-to-day living (i.e., complete source control is not realistic). If CECs become regulated, the most cost effective treatment method appears to be ozonation which oxidizes most refractory organics into more benign organic compounds. Ozone also acts as a disinfectant, i.e., it could supplement, or replace the City's current UV effluent disinfection process. If the organic residuals left over from ozonation are of concern, then a biologically active carbon (BAC) filter could be installed after to metabolize these residual organics. The City's WWTP process is suitable for addition of Ozone-BAC if/when needed, with the biggest constraints being the limited vacant area at the WWTP site and the capital cost of the upgrades.

2.3.2 Three-Tier Bioassay Failures

There has been a growing tendency for treated effluent to fail the Selenastrum Capricornutum reproduction phase of the standard 3-tier bioassay test, which the City is required to conduct quarterly (4 times per year). The tendency to fail the reproduction phase of this test may have some correlation with use of UV as the effluent disinfectant. However the actual cause(s) of the problem is unknown and debated. Hypotheses that are not mutually exclusive include:

- UV does not oxidize CECs, whereas chlorine does to some extent and ozone does to a much greater extent.
- UV may create disinfection byproducts that this phase of testing reacts to adversely.
- Ceriodaphnia species hardiness may be decreasing over time as a result of commercial breeding of the species for test usage.

- New CECs are showing up in effluents every year.
- Coagulant type and/or dosage may be contributing factors.

The problem is not universal, and the cause of the problem is unknown.

2.3.3 Effluent Salinity

Salinity added to potable water as a consequence of its use is of concern to the Regional Water Board. The main sources of concern are salinity additions by residential self-regenerating water softeners, industrial processes, and commercial activities. The City's potable water supply is low in salinity and hardness; thus, self-regenerating water softeners are not a problem in Grass Valley, nor are industrial/commercial discharges to the City's sewer system at this time. The salinity threat from the City's effluent discharge is sufficiently low that the proposed 2016 Order does not include the electrical conductivity (an indicator of salinity) effluent limitation contained in the City's 2009 Order. However, as a matter of policy, the proposed 2016 Order requires the City to continue to provide annual reports to the Regional Water Board discussing City efforts to minimize effluent salinity. With the City converting its effluent disinfection system from chlorination/de-chlorination (a salt adding process) to UV (no salt added), the City has completed all major feasible effluent salinity reduction measures at the WWTP.

3.0 LAND USE AND WASTEWATER GENERATION

Land uses and their corresponding wastewater generation rates within the City's service area are developed in this section. Existing flows are used to identify existing system deficiencies, which are the basis for recommending improvements. The future land uses form the basis for estimating additional flows into the system and are used to determine recommended future improvements to the existing system. The computer model of the City's collection system, developed by Stantec, uses these existing and future flows to assess performance. Results from the computer model analysis are presented in Chapter 4. In addition, an analysis of the City's WWTP is presented in Chapter 5.

3.1 PURPOSE AND SCOPE

This chapter provides a summary of land uses, population projections, and wastewater generation rates within the City of Grass Valley (City) wastewater service area. These projections were used to assess the conveyance capacity of the collection system to accommodate existing and future flows (Chapter 4) and to assess the capacity of the Grass Valley wastewater treatment plant (Chapter 5). The land uses described in this Wastewater Master Plan are limited to the City's wastewater service boundary, and considered according to the City's projected General Plan growth.

3.2 LAND USE

3.2.1 Existing Land Uses

Existing land uses within the City are established by the City of Grass Valley 2020 General Plan. The parcel data used for this master plan analysis was obtained from the City of Grass Valley. The existing land uses within the current City wastewater service area boundary are summarized in **Table 3-1** and shown in **Figure 3-1**. The estimates of developed acreages listed in **Table 3-1** are based on the parcel data provided by the City. Property indicated as vacant was considered undeveloped.

Table 3-1 Existing Land Use (a)

Land Use	Est. Developed Acreage Within City Service Limits	Est. Vacant Acreage Developable Within City Service Limits	Total Developable Acreage Within City Service Limits
Urban Estate Density	76.5	0.8	77.3
Urban Low Density	670.5	204.6	875.1
Urban Medium Density	63.2	62.3	125.5
Urban High Density	142.0	16.3	158.3
Commercial	347.4	21.5	368.9
Business Park	138.8	88.4	227.2
Institutional Non-Governmental	76.2	0	76.2
Manufacturing - Industrial	101.9	20.2	122.1
Office and Professional	74.9	16.6	91.5
Open Space	6.2	0	6.2
Parks & Recreation	142.7	0.1	142.8
Public	87.9	17.8	105.7
Schools	231.1	0	231.1
Utilities	18.6	0.1	18.7
Total	**2177.9**	**448.7**	**2626.6**

(a) Source: City of Grass Valley GIS data.

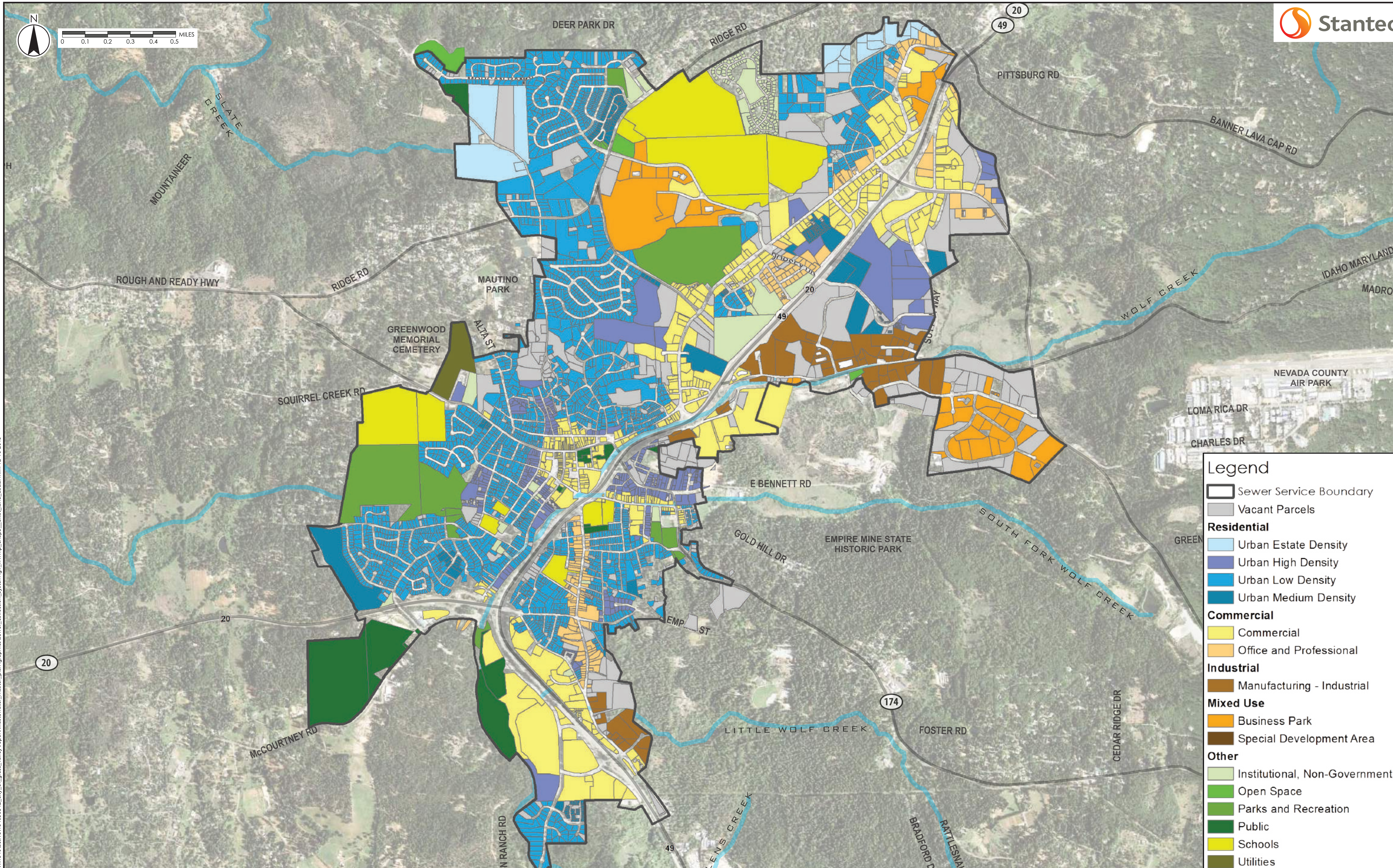


Figure 3-1
Existing Land Use Designations

3.2.2 Future Land Use

Future growth within the City's service area boundary is estimated using the land use designations within the 2020 General Plan. Future increases in wastewater generation will result as the vacant lands within the City limits develop, and as the City grows out to the 2020 General Plan extents. Certain areas within the 2020 General Plan have been identified as "Special Development Areas", and have overriding sources of land use allocation (Loma Rica, North Star, Kenny Ranch, and Berriman Ranch). **Table 3-2** provides a summary of the growth areas and the source for the associated land use designations. The extents of these areas are shown in **Figure 3-2**.

Table 3-2 Growth Area Document Sources

Growth Area	Source Planning Document
Kenny Ranch	"Table 2-1: Land Use and Housing Unit allocations per Annexation Agreements" within the 2020 General Plan Draft EIR
North Star	"Table 2-1: Land Use and Housing Unit allocations per Annexation Agreements" within the 2020 General Plan Draft EIR
Loma Rica Ranch	Loma Rica Ranch Specific Plan (March 2011, Loma Rica Ranch, LLC.)
Berriman Ranch and Adjacent Property	Southern Sphere of Influence Planning and Annexation Project Draft EIR (PMC, October 2013)
All Remaining Area	2020 General Plan Draft EIR

In addition to the land area designated for growth, the City has also provided information regarding the projected growth horizon. The future growth areas are broken into three planning categories:

- Near-term development: Areas which may develop within 5 years
- Long-term Development: Areas which may develop within 10 years
- Area of Concern: Areas that may develop in future years

These time frames were selected by the City to allow an analysis of necessary system improvements and their possible timing. The extents of the projected growth horizons are shown in **Figure 3-3**. The Area of Concern shown in **Figure 3-3** includes areas specifically identified with the "Area of Concern" designation in the 2020 General Plan, as well as areas of the City's sphere of influence not anticipated to develop in the Near-term or Long-term horizons defined above.

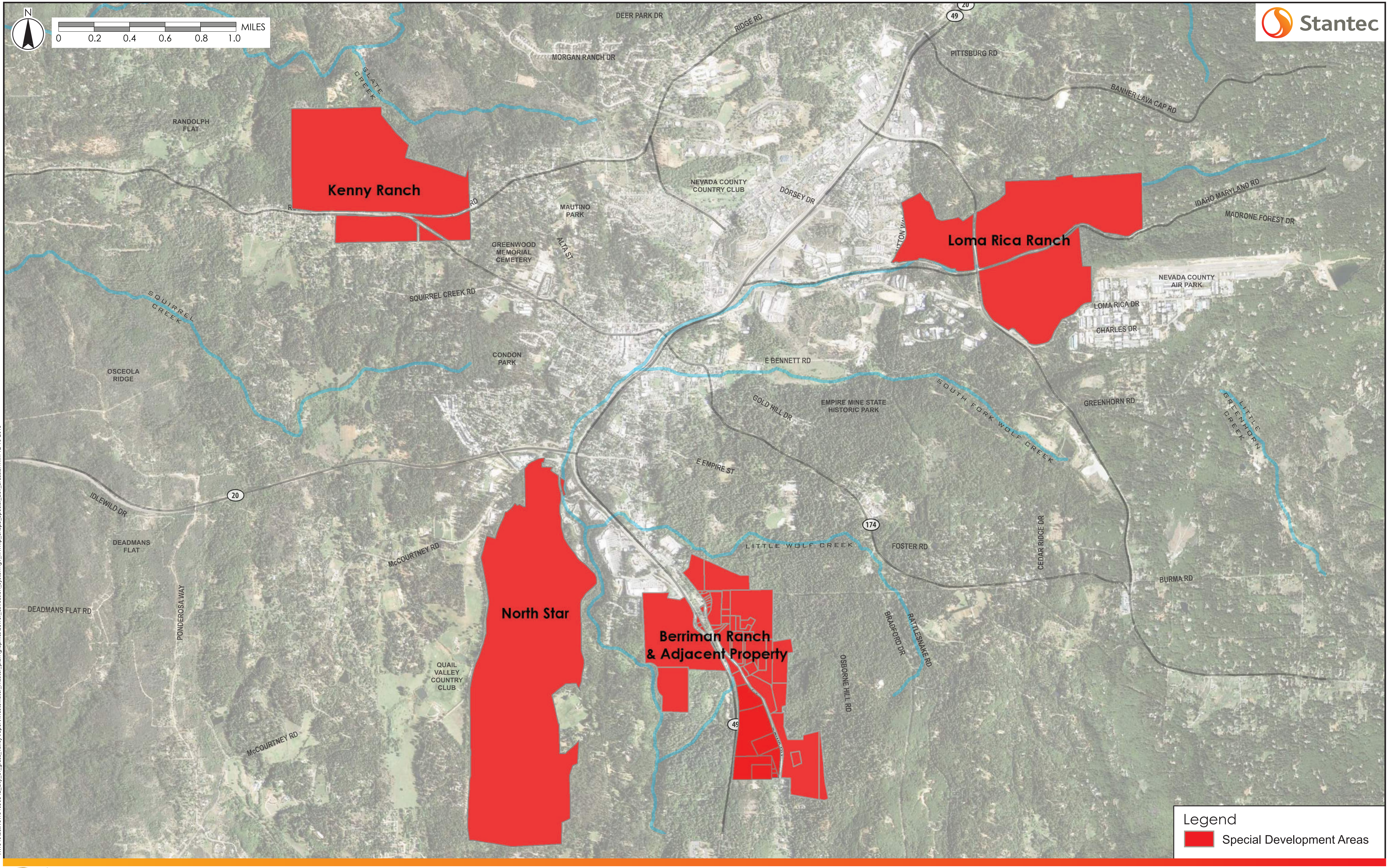


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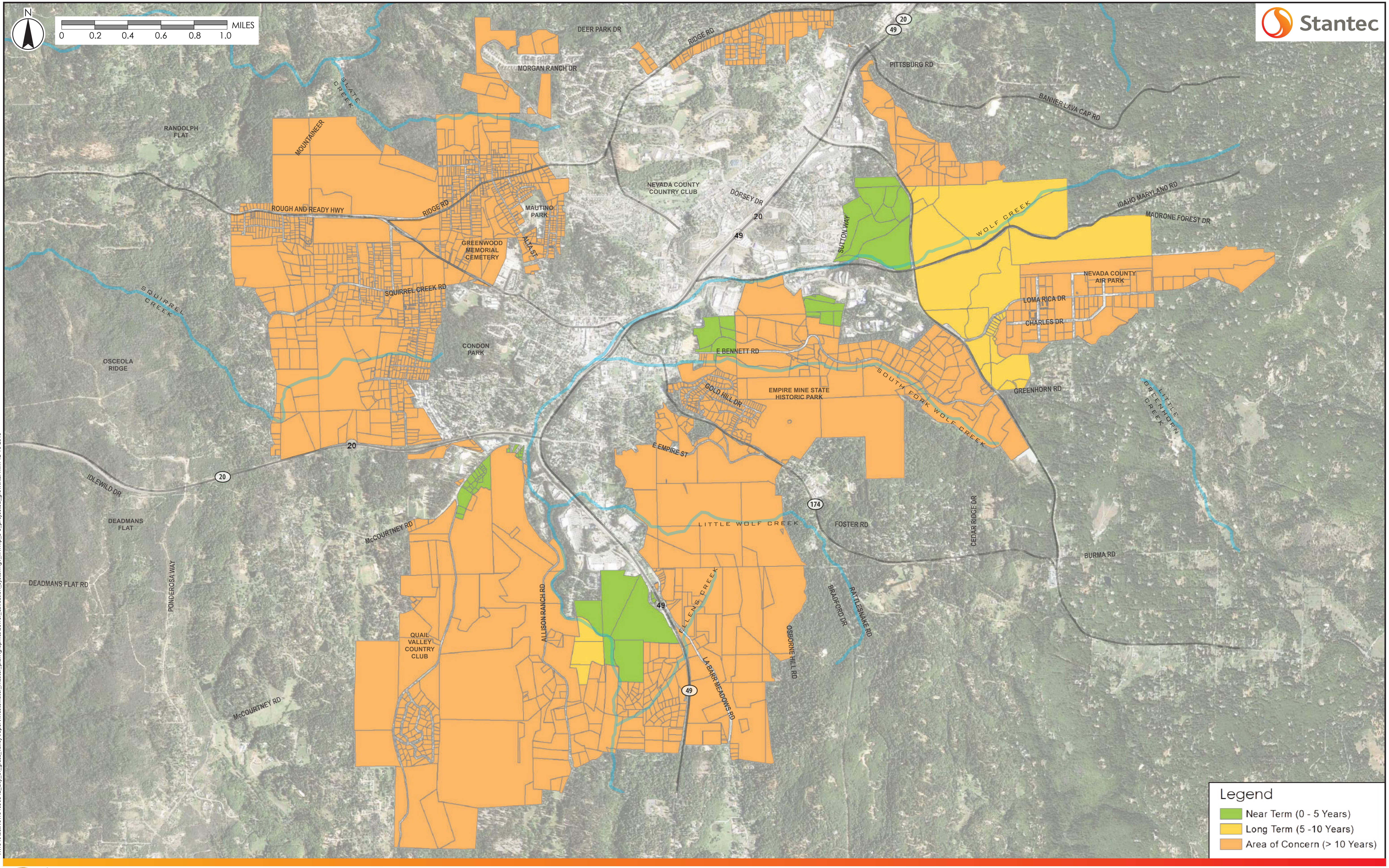
Figure 3-2
Special Development Areas



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City of Grass Valley
Wastewater System Master Plan

Figure 3-3
Projected Growth

Table 3-3 provides a summary of all the growth areas considered in this Master Plan, identified by each growth horizon. Land Use designations of all future growth areas presented in **Table 3-3** are shown in **Figure 3-4**.

Table 3-3 Build-out Land Uses

Land Use Designation	Near Term Area (acres)	Long Term Area (acres)	Area of Concern (acres)	Total Projected Area
Urban Estate Density	21.6	32.4	1947.3	2001.3
Urban Low Density	1.4	0.1	654	655.5
Urban Medium Density	73.4	13.9	97.5	184.8
Urban High Density	1.2	0	10.9	12.1
Commercial	0.02	0	9.3	9.3
Business Park	26.9	0	57	83.9
Institutional Non-Governmental	0	0	8.4	8.4
Manufacturing - Industrial	0	165.1	265.8	430.9
Office and Professional	16.9	0	19.5	36.4
Parks & Recreation	0	0	768	768.0
Public	0.6	0	309	309.6
Schools	0	0	0	0.0
Utilities	0	0	11	11.0
Loma Rica Ranch	52.6 [a]	98.0 [a]	0	150.6
Kenny Ranch	0	0	356	356
North Star	0	0	760	760
Berriman Ranch & Adjacent Property	115	0	283	398
Total	**309.6**	**309.5**	**5556.7**	**6175.8**

(a) Areas listed for Loma Rica represent the areas scheduled for development as per the Loma Rica Ranch Specific Plan:

- The Creeks Neighborhood
- The Farm Neighborhood
- The Lake Neighborhood
- The Trailhead Neighborhood

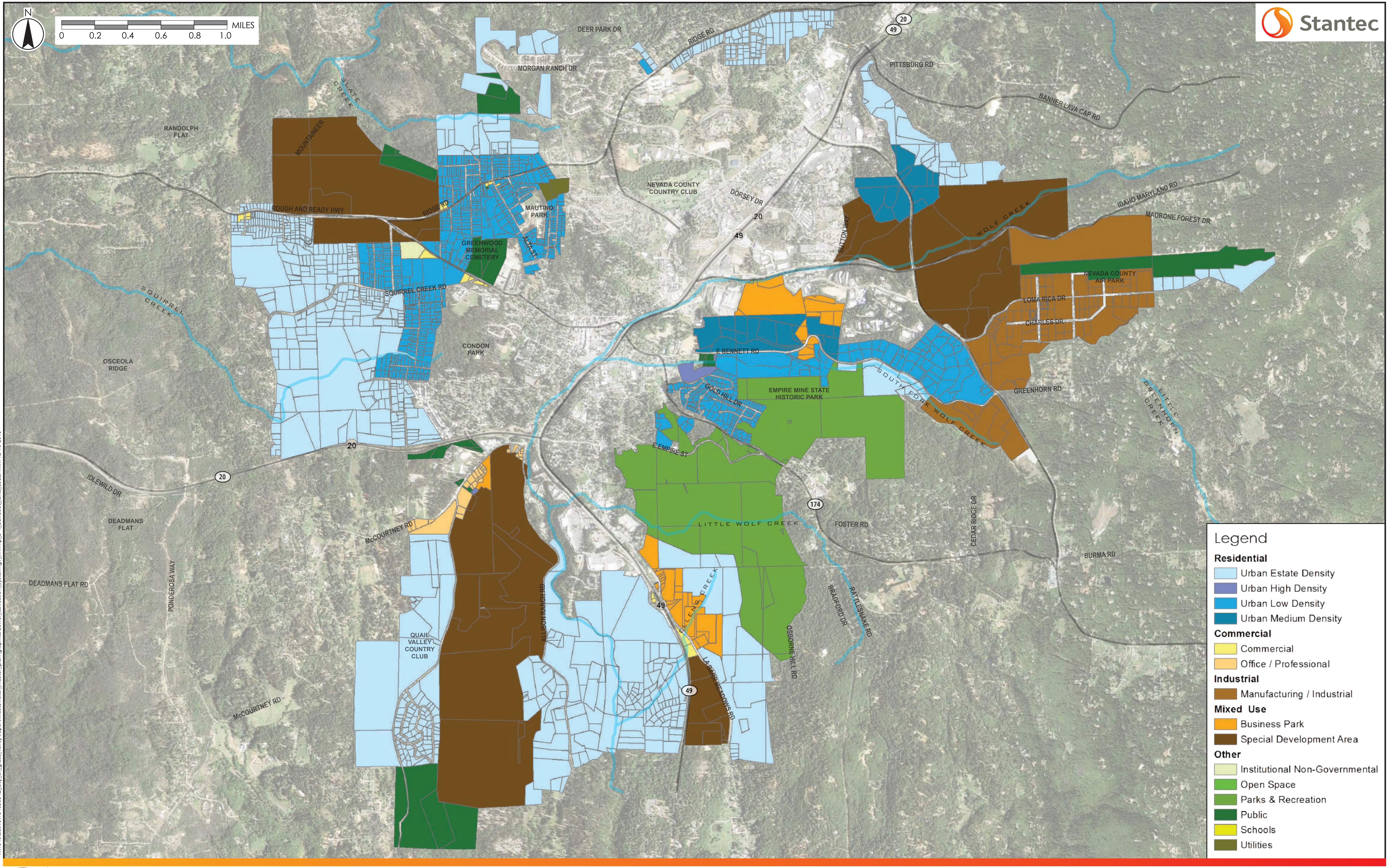


Figure 3-4
Build Out Land Use Designations

3.3 POPULATION PROJECTIONS

Population density projections are established in the 2020 General Plan. During the course of this assessment, it was assumed that the existing developed area would not be redeveloped or densified. An existing population of 12,668 people (Source: CA Dept. of Finance estimate as of January 1, 2014) was used for the basis of this assessment. **Table 3-4** provides a summary of the population densities listed within the 2020 General Plan.

Table 3-4 Residential Land Use Dwelling Unit and Population Densities

	Density Per 2020 General Plan (DU/acre)	Density Used For Assessment (DU/acre)	Persons per Household
Urban Estate Density	1	1	2.4 (a)
Urban Low Density	1.001 - 4	2	2.4 (a)
Urban Medium Density	4.001 - 8	6	2.175 (c)
Urban High Density	8.001 - 20	14	1.95 (b)

(a) Single Family housing unit rate of 2.40 persons per household ("Table 1-1 Facts and Figures, Grass Valley Planning Area")

(b) Multi-Family housing unit rate of 1.95 persons per household ("Table 1-1 Facts and Figures, Grass Valley Planning Area")

(c) Assumes 50% Single Family housing units and 50% Multi-Family housing units

The total growth within the system is defined by the contributions of the development of vacant parcels within the City Limits, the Special Development Areas and all other catchments defined by the City of Grass Valley 2020 General Plan. **Table 3-5** provides a summary of additional population that would be served should all of the aforementioned developments be completed.

Table 3-5 Population Projections

	Area (acres)	Density (DU/acre)	Person per Household	Additional Population	Total Population
Existing					12,668
Vacant Parcels Within City Limits					
Urban Estate Density	0.8	1	2.4	2	
Urban Low Density	204.6	2	2.4	982	
Urban Medium Density	62.3	6	2.175	813	
Urban High Density	16.3	14	1.95	445	
Total	**284.0**			**2,242**	**14,910**
Near Term					
Urban Estate Density	21.6	1	2.4	52	
Urban Low Density	1.4	2	2.4	7	
Urban Medium Density	73.4	6	2.175	958	
Urban High Density	1.2	14	1.95	32	
Loma Rica Ranch	42.2 [e]			686 [a]	
Berriman Ranch & Adjacent Property	115			454	
Total	**254.7**			**2,189**	**17,099**
Long Term					
Urban Estate Density	32.4	1	2.4	78	
Urban Low Density	0.1	2	2.4	1.0	
Urban Medium Density	13.9	6	2.175	182.0	
Urban High Density	0.0	14	1.95	0	
Loma Rica Ranch	65.4 [e]			936 [b]	
Total	**111.4**			**1,197**	**18,296**
Area of Concern					
Urban Estate Density	1,947.3	1	2.4	4,674	
Urban Low Density	654.1	2	2.4	3,140	
Urban Medium Density	97.5	6	2.175	1,273	
Urban High Density	10.9	14	1.95	298	
Kenny Ranch	150 [e]			240 [c]	
North Star	312 [e]			872 [d]	
Berriman Ranch & Adjacent Property	283			424	
Total	**3,452.8**			**10,921**	**29,217**

(a) 225 Single-family and 75 Multi-Family dwelling units (Loma Rica Ranch Specific Plan, May 2011)
(b) 346 Single-family and 54 Multi-Family dwelling units (Loma Rica Ranch Specific Plan, May 2011)
(c) 50 Single-family dwelling units (50% of Annexation Agreement, 2020 General Plan)
(d) 181.5 Single-family dwelling units (50% of Annexation Agreement, 2020 General Plan)
(e) Areas listed for Loma Rica represent the areas scheduled for residential development as per the Loma Rica Ranch Specific Plan and for Kenny Ranch and North Star denote the areas scheduled for residential development as per the 2020 General Plan.

3.4 WASTEWATER GENERATION

3.4.1 Wastewater Flow Characterization

Sewage generation is separated into two distinct types of flow: dry weather flow (DWF) and wet weather flow (WWF).

Dry Weather Flow (DWF)

DWF is the flow that occurs during periods of no precipitation. Dry weather flows are normally composed of base sanitary flow and groundwater infiltration. Base flows have a diurnal pattern which changes throughout the day as a result of variation in human activity (e.g. flows typically drop at night when residents are asleep).

Base Sanitary Flow

Base sanitary flow is the component of wastewater generated directly by residential, commercial, and industrial users throughout a community. It is also referred to as base flow.

The majority of base flow is generated by residential and commercial users (e.g. restaurants, grocery stores, shops, etc.). Some base flow is also generated by light industrial users (e.g. warehouses), and public facilities (such as parks and schools).

Groundwater Infiltration (GWI)

Groundwater infiltration (GWI) is groundwater that enters the collection system through cracks in sewer pipes, leaky joints, damaged sewer lateral connections, and poorly sealed manholes. GWI tends to vary seasonally depending on groundwater depth in relation to the depth of the sewer pipes. Typically, GWI is more significant during the wet season when the groundwater elevations can rise due to rainfall. GWI can however have an impact during drier periods depending on site specific soil and groundwater conditions which can be highly variable in foothill settings, for instance where soil/rock interfaces and topography are contributing factors.

Wet Weather Flow (WWF)

Wet weather flows (WWF) are the result of precipitation, specifically rainfall, affecting a system in two ways: inflow and infiltration.

- **Inflow** (a rapid response to rainfall) is flow created from rainfall directly entering the sanitary system through leaky manholes and improper storm drainage connections. This can include flow that enters leaky sewer lateral connections, new sewer lateral connections which have not yet been made and are not properly capped, exposed pipes which are installed near drainage courses where joints become separated and water directly enters the sewer, roof drains improperly connected to sewer laterals or collectors, etc.

- **Infiltration** (a slower and more extended response to rainfall) is flow created from rainfall entering the system through cracked manholes and pipes. Typically infiltration is considered to be water that enters the sewer after first passing (or percolating) through the surrounding soil and/or trench bedding.

The total sanitary sewer response to a rain event is called *rainfall dependent inflow and infiltration* (RDII). RDII differs from GWI as it is directly related to rainfall events.

3.4.2 Dry Weather Flow Generation

Section 8, Sanitary Sewer (SS), of the City of Grass Valley Design Standards (REV. 02/10) was used in conjunction with flow monitoring conducted by V&A Consulting Engineers (V&A) as the basis for flow generation parameters. Similar to the population projections, it is assumed that the existing development areas will continue to generate flows in the future at the same rate as it did at the time of this study. **Table 3-6** provides a summary of the City's design standards for wastewater generation rates.

Table 3-6 City of Grass Valley Unit Wastewater Generation Rates Design Standards

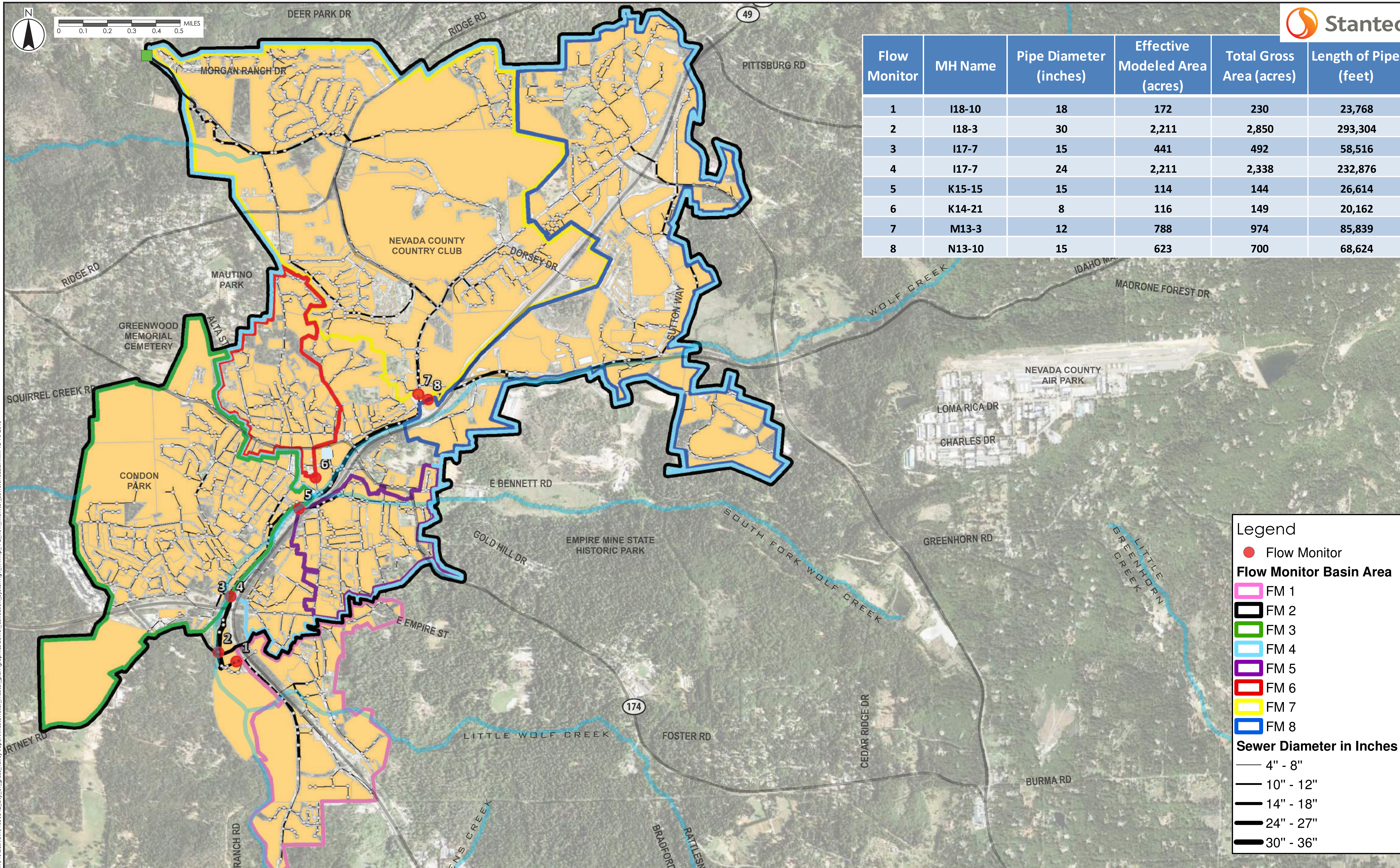
Land Use Designation	City of Grass Valley Design Standards [(a)]
Single Family Residential	191 gpd/DU
Multi-Family Residential	135 gpd/DU
Non-Residential	850 gpd/DU

(a) Source: City of Grass Valley Design Standards, Section 8, February 2010.

Flow monitoring within the City's sewer system was conducted by V&A starting February 6th, 2014. This flow monitoring consisted of 8 meters located throughout the system. A detailed review of the flow monitoring in the context of the wastewater sewer system is presented in Chapter 4.

During the flow monitoring, there were a total of 24 days that shall be considered dry weather flow (DWF). **Figure 3-5** illustrates the extent of the City's existing sewer collection system service area and identifies the flow monitor locations and their corresponding drainage basins.

It should be noted that although the 24 days were designated as DWF, the GWI during this time frame was still significantly higher than would normally be expected to occur during drier months (e.g. August and September). To account for the higher GWI during the flow monitoring period, it was approximated that 80 percent of the minimum recorded (instantaneous) flow was attributed to the GWI and not assessed for base sanitary flow generation. Given that there is no accurate way to determine the actual contribution of GWI, as it is dependent upon a multitude



Flow Monitor	MH Name	Pipe Diameter (inches)	Effective Modeled Area (acres)	Total Gross Area (acres)	Length of Pipe (feet)
1	I18-10	18	172	230	23,768
2	I18-3	30	2,211	2,850	293,304
3	I17-7	15	441	492	58,516
4	I17-7	24	2,211	2,338	232,876
5	K15-15	15	114	144	26,614
6	K14-21	8	116	149	20,162
7	M13-3	12	788	974	85,839
8	N13-10	15	623	700	68,624

Figure 3-5
Flow Monitor Locations and Sewersheds February 2014

For this analysis, the per capita contribution to the DWF was assessed based on the land use zones described in the City's 2020 General Plan. Several assumptions were used to assign per capita contributions to DWF, based upon the designated land use.

1) Open Space and Public lands were assumed to have zero wastewater generation.

2) C-2, NC and NC-Flex were grouped together to represent Commercial land use. Commercial generation was compared at 850 gpd/acre and 1500 gpd/acre.

3) R-1 was assumed to be Urban Low Density at 2 dwelling units per acre with 2.4 people per single family dwelling unit (**Table 3-4**).

4) NG-2 and NG-3 was assumed to be Medium Density Residential at 6 dwelling units per acre with an average of 2.175 people per dwelling unit (**Table 3-4**).

Applying the 2.4 person per household estimate presented in the City's 2020 General Plan, the per capita estimates of wastewater contribution were between 252 gpd/dwelling unit (DU) and 198 gpd/DU.

In addition to the above analysis of the FM basins, the average water demand factor for a service connection (such as a single family residence) recommended in the Water Master Plan (March 2016) Land Use and Water Demand chapter was evaluated. This demand factor is currently 300 gpd/service connection (sc). An industry "rule-of-thumb" comparing wastewater generation to water demand is to assume 70 to 80 percent of average water demand represents expected wastewater generation for residential users. This would suggest an appropriate residential wastewater unit generation factor of somewhere between 175 gpd/DU and 200 gpd/DU.

Using the above generation rates as a comparison to the City's design standards, it was decided to use the wastewater generation rates provided in **Table 3-7**.

Table 3-7 Wastewater Generation Rates Used for DWF Projections

Land Use Designation	Generation Rate (gpd/acre)
Business Park	850
Commercial	850
Institutional Non-Governmental	850
Manufacturing / Industrial	850
Office / Professional	850
Open Space	0
Parks & Recreation	150
Public	150 (400 gpd @ Airport) (a)
Schools	-
Urban Estate Density	200 (b)
Urban Low Density	400 (c)
Urban Medium Density	990 (e)
Urban High Density	1820 (d)
Utilities	200

(a) Total projected generation from the Airport lands is expected to be 400 gpd.
(b) 200 gpd/du @ 1 du/acre (**Table 3-4**)
(c) 200 gpd/du @ 2 du/acre (**Table 3-4**)
(d) 130 gpd/du @ 14 du/acre (**Table 3-4**)
(e) 50% 130 gpd/du and 50% 200 gpd/du @ 6 du/acre (**Table 3-4)**

The generation rates provided in **Table 3-7** were applied to the land uses provided in **Table 3-3**. **Table 3-8** provides a summary of the projected DWF for the City of Grass Valley.

Table 3-8 Projected Dry Weather Flow

	Area (acres)	Additional DWF (mgal/d)	Total Projected DWF (mgal/d)
Existing Service Area			1.3 [g]
Vacant Parcels Within City Limits			
Residential [a]	284.0	0.17	
Non-Residential [b]	164.7	0.13	
Total	**448.7**	**0.30**	**1.6**
Near Term Growth Areas			
Residential[a]	44.5	0.08	
Non-Residential [b]	97.5	0.04	
Loma Rica Ranch [c]	52.6	0.06	
Berriman Ranch & Adjacent Property	115	0.08	
Total	**309.6**	**0.26**	**1.9**
Long Term Growth Areas			
Residential [a]	46.4	0.02	
Non-Residential [b]	165.1	0.09	
Loma Rica Ranch [d]	98	0.10	
Total	**309.5**	**0.21**	**2.1**
Remaining Growth Areas/Areas of Concern			
Residential [a]	2709.8	0.8	
Non-Residential [b]	1355.3	0.5	
Kenny Ranch [e]	356	0.12	
North Star [f]	760	0.3	
Berriman Ranch & Adjacent Property	283	0.17	
Total	**5464.1**	**1.89**	**4.0**

(a) Combined summary of all residential land uses
(b) Combined summary of all non-residential land uses
(c) Represents the development of the Creeks Neighborhood (Loma Rica Ranch Specific Plan, 2011)
(d) Represents the development of the Farm, Lake, and Trailhead Neighborhoods (Loma Rica Ranch Specific Plan, 2011)
(e) 50% of the land area represented in the Kenny Ranch annexation agreement (2020 General Plan)
(f) 50% of the land area represented in the North Star annexation agreement (2020 General Plan)
(g) ADWF measured at the WWTP

3.4.3 Wet Weather Flow Generation

The guidance provided by the United States Environmental Protection Agency (EPA) within the document "I/I Analysis and Project Certification" (Ecology Publication No. 97-03) was used as the basis for estimating the RDII contribution to wastewater flows for future developments. It was assumed new developments would not bring "excessive I/I". It was also assumed that the contribution from the existing developments will not change. This assumption will provide system capacity analysis results which will allow the City to consider worst case conditions when judging where to invest limited funds in their wastewater system.

This document provides the standard for determining what constitutes "excessive I/I" and has been used as an industry standard. **Table 3-9** provides a summary of the EPA standards.

Table 3-9 EPA Standard for Non-Excessive RDII Contribution

EPA 24-hour Wet Weather Flow Allowance	275 gpd/person
Average Dry Weather Flow	83.3 gpd/person (a)
Rain Dependent Inflow and Infiltration Allowance (Population)	191.7 gpd/person (b)
Rain Dependent Inflow and Infiltration Allowance (Land Area)	840 gpd/acre (c)

(a) 200 gpd/DU for single family residential with a density of 2.4 people/DU.

(b) RDII Allowance is the 24-hour additional flow allowed to enter the system as a result of significant rainfall, and does not include DWF contribution.

(c) 16,550 people will occupy 3,792 acres of residential land, at an average density of 4.4 people/acre

Based upon the EPA Standard of 840 gpd/acre RDII allowance, the additional land area of 6,644 acres (Full Build-out projection less existing developments) will allow an RDII contribution of up to 5.6 mgd average over a 24-hour period. It should be noted that due to the temporal variance of rainfall, the peak RDII will be greater than the 24-hour average.

Unfortunately, this creates some difficulty is determining the peak flows which need to be conveyed to the WWTP and the peak flows that the WWTP will need to have capacity to treat. For the purposes of assessing the wastewater sewer collection system (discussed in Chapter 4) a 24-hour, 10-year return period Huff design storm was selected to generate the RDII. The City of Grass Valley has provided Intensity – Duration – Frequency (IDF) curves for the rain gauge station "Grass Valley 2 NNE" based upon the years of record 1951 through 1998. These curves estimate that a 24-hour average intensity for a 10-year return period storm is approximately 0.20 inches/hour. **Figure 3-6** shows the 24-hour Huff distribution (forth quartile, 0-10 square miles) based upon that value.

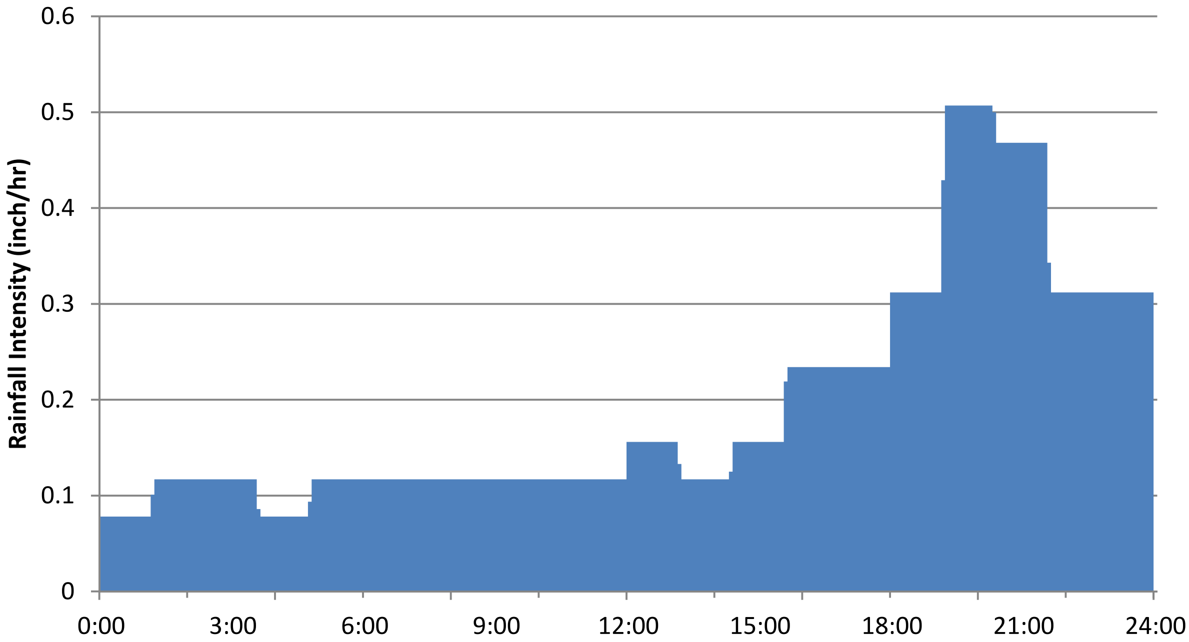


Figure 3-6 Rainfall Hyetograph for 1:10 Year, 24-hour Huff Design Rainfall

The peak rainfall intensity for the 10-year, 24-hour Huff design storm is 0.51 inch/hour. This is roughly 2.55 times the average rainfall intensity. As RDII typically becomes more intense as the ground becomes more saturated and depression storage fills up, it was assumed that the majority of the RDII is resultant from the later hours in the storm. To account for this variability, the peak instantaneous RDII allowance was assumed to be three times the 24-hour average, allowing for approximately 2,520 gpd/acre. The sewershed (defined by the flow monitoring conducted by V&A, discussed in Chapter 4) which most resembles future development in both land use and sewer pipe composition is sewershed 7 (see **Figure 3-5**). This sewershed currently produces a peak RDII of approximately 2,103 gpd/acre, less than the maximum allowable by the EPA. This confirms that a value of 2,520 gpd/acre (or less) is both obtainable and realistic for new developments. **Table 3-10** summarizes the results of estimating future flow within the City's system. These values are also used for the facility capacity evaluations summarized in Chapters 4 and 5.

Table 3-10 Projected Wet Weather Flow

	Acres (acres)	Additional RDII 24-hour Average (mgd)	Total RDII 24-hour Average (mgd)	Peak RDII Instantaneous (mgd)	Average DWF (mgd)	Peak DWF [(a)] (mgd)	Peak WWF Instantaneous [(b)] (mgd)
Existing	2177.9	-	-	16.8	1.3	2.1	18.9 [(c)]
Vacant Parcels Within City Limits	448.7	0.4	0.4	17.9	1.6	2.6	20.5
Near Term	309.6	0.3	0.6	18.7	1.9	3.0	21.8
Long Term	309.5	0.3	0.9	19.5	2.1	3.5	23.0
Area of Concern	5556.7	4.7	5.6	33.5	4.0	6.2	39.7

(a) DWF for Grass Valley has a diurnal peaking factor of approximately 1.6 (discussed in Chapter 4).

(b) Peak WWF (Instantaneous) is typically computed by assuming concurrent peak RDII with peak DWF. This provides a worst-case scenario for conveyance and storage capacity.

(c) 18.9 mgd is estimated to be the true flow measured at the WWTP on March 16, 2012 ("Technical Memorandum No 2 – Correction of Influent Flow Readings when the Parshall Flume is Submerged", Stantec, Oct. 2013)

4.0 COLLECTION SYSTEM

4.1 PURPOSE AND SCOPE

This section presents an overview of the City's wastewater collection system, including existing capacity and possible future capacity which the system must provide to service growth anticipated by the Grass Valley 2020 General Plan. The results of a system capacity assessment are presented here, which takes into consideration existing system conditions as well as potential future wastewater generation demands on the system. Results of flow monitoring within the collection system is also summarized in this chapter.

A summary of system condition is also presented, based on information available to the City at the time this Master Plan was developed. Collection system assets were cataloged and pertinent summaries of system information are presented herein.

Hydraulic modeling performed as part of the system capacity assessment is discussed, including methodologies and assumptions used. The results of the modeling and system assessment have been used to identify needed system improvements. Potential pipe deficiencies are summarized in this chapter and defined in **Appendix F**.

4.2 EXISTING WASTEWATER COLLECTION SYSTEM

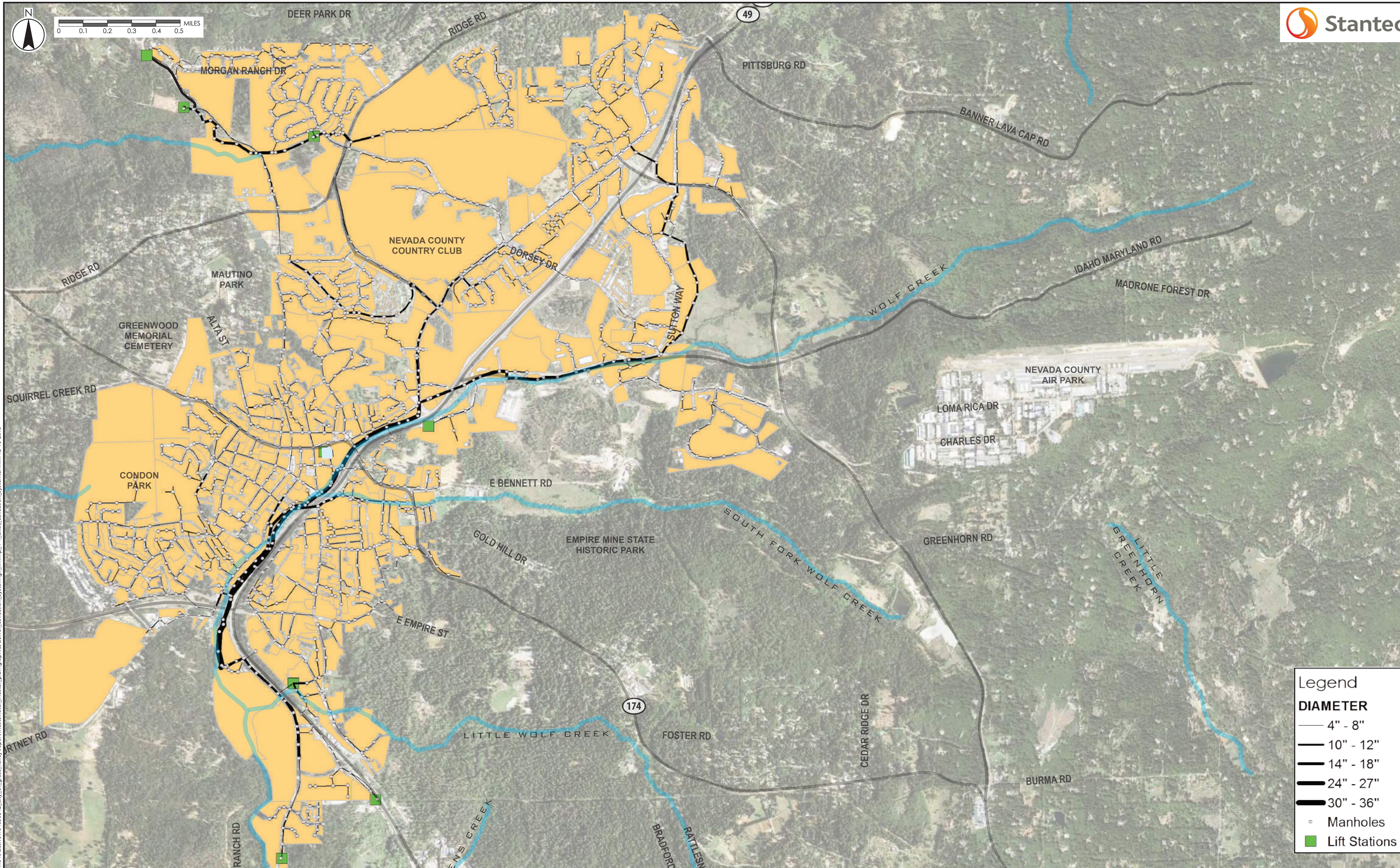
The City of Grass Valley's existing wastewater collection system consists of approximately 61 miles of sewer line and 7 separate pump (lift) stations. Stantec performed an assessment of the system in 2013/2014 using information provided by the City and data collected as part of the development of this Master Plan.

4.2.1 Description of Existing Wastewater Collection System

The City's existing wastewater collection system includes several miles of local collector sewers which drain to a backbone "trunk" system, an electronic model of which Stantec developed as a tool for use in assessing system capacity.

As discussed in Chapter 3 the City's collection system serves an area of approximately 2,430 acres and a resident population of approximately 12,668. In addition, the City of Grass Valley serves a number of industrial and commercial users whose businesses are located within the City's sewer service area.

Figure 4-1 depicts the City's existing collection system. **Table 4-1** provides a summary of the input parameters for the modeled conduits.



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City of Grass Valley
Wastewater System Master Plan

Figure 4-1
Existing Wastewater Collection System

Table 4-1 Sewer Input Summary Conduits in the Model

Diameter (inches)	Gravity Sewers Conduit Total Length (feet)	Pressurized Sewer Conduit Total Length (feet)
4	-	1,409
6	29,602	10,620
8	142,39	-
10	8,411	-
12	8,132	-
14	1,394	-
15	6,089	-
16	395	-
18	5,285	-
21	246	-
24	5,261	-
27	731	-
30	1,033	-
36	51	-
Total	**80,868**	**12,028**

4.2.2 GIS Database

The City's available collection system information was gathered by Stantec and to the extent feasible input into a Geographic Information System (GIS) database. This database serves multiple purposes in the development of this Master Plan. The City intends to build on this GIS database going forward, improving accuracy and completeness as time progresses.

The GIS data includes land uses provided by the City for their existing service area (primarily within the City Limits) as well as land uses contained in the Spheres of Influence (SOI) identified in the Grass Valley 2020 General Plan. The land use and existing collection system data form the basis for estimates of wastewater flows presented in Chapter 3 of this Master Plan, as well as the framework within which the electronic model of the system was developed.

4.2.3 Existing Wastewater Flows

The flows used in assessing collection system performance were developed in Chapter 3 of this Master Plan. In addition to those flow estimates and projections, flow monitoring was performed in order to provide data from specific locations within the collection system to allow calibration of the computer model of the collection system developed with this Master Plan.

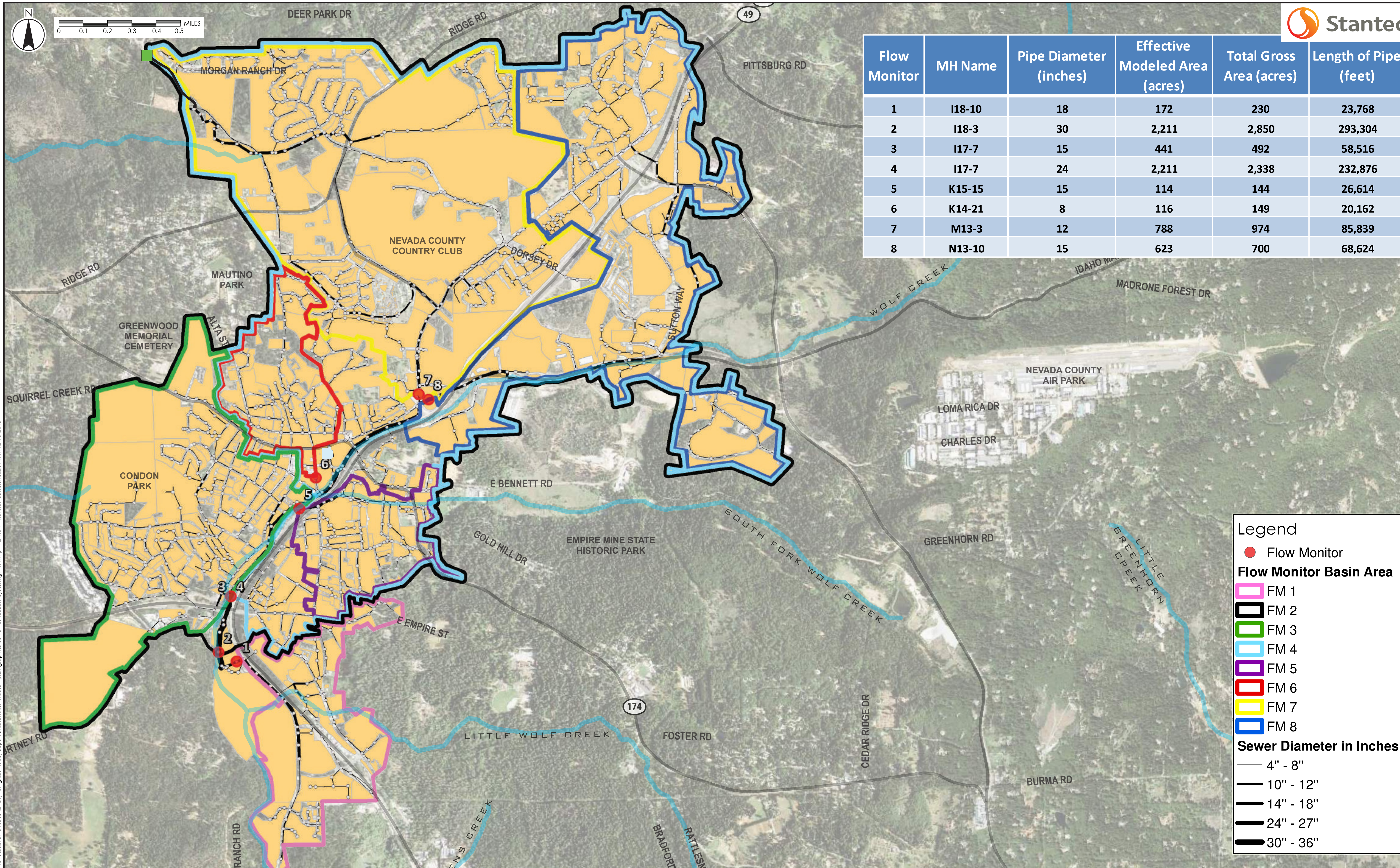
4.2.3.1 Flow Monitoring

The City initiated a flow monitoring program in the spring of 2014 to obtain data for use in developing the system computer model, as well as to investigate sources of rainfall dependent inflow and infiltration (RDII), and groundwater infiltration (GWI) into the City's collection system. Eight flow monitors were deployed from February 6, 2014 through April 8, 2014 establishing eight subsheds within the existing collection system. **Figure 4-2** illustrates the location of the eight flow monitors and their respective subsheds. **Table 4-2** summarizes the characteristics of the 8 sewer sheds established with this flow monitoring effort.

The flow monitoring data was used to analyze potential sources of I/I. The results of this investigation are included in **Appendix A** *(City of Grass Valley Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, May 2014, V&A)*. In addition to using this data in support of an I/I investigation, it was used to calibrate the computer model of the system developed with this Master Plan.

Table 4-2 Sewer Subshed Characteristics

Flow Monitor	MH Name	Pipe Diameter (inches)	Effective Modeled Area (acre)	Total Gross Area (acres)	Length of Pipe (feet)
1	I18-10	18	172	230	23,768
2	I18-3	30	2,211	2,850	293,304
3	I17-7	15	441	492	58,516
4	I17-7	24	2,211	2,338	232,876
5	K15-15	15	114	144	26,614
6	K14-21	8	116	149	20,162
7	M13-3	12	788	974	85,839
8	M13-10	15	623	700	68,624



Flow Monitor	MH Name	Pipe Diameter (inches)	Effective Modeled Area (acres)	Total Gross Area (acres)	Length of Pipe (feet)
1	I18-10	18	172	230	23,768
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4	I17-7	24	2,211	2,338	232,876
5	K15-15	15	114	144	26,614
6	K14-21	8	116	149	20,162
7	M13-3	12	788	974	85,839
8	N13-10	15	623	700	68,624

Figure 4-2
Flow Monitor Locations and Sewersheds February 2014

4.2.3.2 Micromonitoring, I/I, and GWI Investigations

As a result of the findings of the February through March 2014 flow monitoring additional Micromonitoring was implemented in basins 3 and 5 to attempt to isolate sources/locations of I/I. Findings of this additional micromonitoring effort are summarized in a report entitled *Results from Grass Valley Micromonitoring Program – Phase 1* (June 2014, Stantec) included in **Appendix B**.

Following the micromonitoring effort in basins 3 and 5 dry weather flow monitoring was undertaken, along with additional investigatory efforts, in Basins 5, 6 and 8 to attempt to isolate potential sources of GWI. This additional GWI investigation was undertaken from June 11 to July 8, 2014. This effort is summarized in a report entitled *Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study*, (October 2014, Stantec). This report is included in **Appendix B**.

4.2.4 Asset Cataloging

Available information for the Wastewater Treatment Plant (WWTP) and collection system assets was gathered from multiple sources including City maps, GIS database files provided by Global Water (a vendor contracted to handle utility billing for the City), City CAD drawings, and previous Master Plan details. Asset tags were verified and, where missing, were assigned. This information was then used to build individual "asset registries" for the WWTP and collection system. Once completed, the asset registries were organized by asset "class." Asset classes within the wastewater collection system include: lift stations and appurtenances, manholes, wastewater pipelines, and other system elements.

4.2.4.1 Condition Assessments

Replacement cost for linear assets were estimated based on pipe composition, diameter, and industry cost/foot replacement estimates. Further, where available from existing data sources, manufacturers, and vendors, an approximate purchase or replacement cost was assigned to each equipment asset along with the year of approximate installation or in-service placement. Individual "weighting" was assigned to each asset in the following categories:

- Asset Risk: probability of failure, 0 = lowest risk to 25 = highest risk
- Asset Impact: failure impact to population, environment or finances, 1 = no impact to 5 = major interruption and impact
- Asset Probability: probability of failure over time based on EPA longevity estimates or industry standards, 1 = low to 5 = high
- Asset Condition: where available, condition of an asset was estimated, 1 = excellent (80-100% remaining life) to 5 = poor condition (0 to 20% remaining life).

- Reliability: reliability over time, typically based on completed work orders and/or repairs, was not included in the available data.

The completed asset registry with available data in the categories and classes noted in previous paragraphs was uploaded to a NEXGEN Asset Management System for in-depth and predictive analysis.

4.2.4.2 NEXGEN Software

The NEXGEN Asset Management System provides comprehensive analysis of all types of asset data based on and including factors noted in previous paragraphs. All available data compiled in the asset registry, along with estimated or actual installation dates and data-specific asset information was uploaded to a NEXGEN Asset Management System database for comprehensive analysis. The analyses available included the following factors:

- Average life span analysis; expected useful life
- Priority analysis; which assets should be addressed first, i.e. refurbished and/or replaced (R&R)
- Refurbish and/or replacement predictions (timing)
- Estimated budget predictions (cost)

Based on analysis using the NEXGEN Asset Management System engine, expected and actual predictions arrived at from other sources were tested for accuracy and used to provide the City with an overall average for Master Planning activities.

Asset cost data available was considerable; however, there are holes that should be accounted for when considering budgeting for capital replacements or refurbishments. The percentage of cost data available, either real or estimated, was derived from the Asset Registries as follows:

<u>Approximate % of assets with cost data available:</u>

- WWTP assets: 100% of assets accounted for
- Lift stations: 100% of assets accounted for
- Manholes: 100% of assets accounted for
- Sewer pipeline: 97% of assets accounted for

4.3 HYDRAULIC MODEL

Stantec developed a computer model of the City's wastewater collection system for purposes of assessing existing available capacity and the possible need for upgrades to serve future growth scenarios. These future growth scenarios address serving Build-out of the existing City service area, the 2020 General Plan Spheres of Influence, Special Development Areas and Areas of Concern also identified in the General Plan. This section describes the development and calibration of the system model.

4.3.1 Modeling Software

PCSWMM software, developed by Computational Hydraulics Inc., was selected for use in developing a collection system computer model for this Wastewater Master Plan. This software package has been developed using the EPA SWMM 5.0 engine as its basis. This software was selected for its ability to meet the following objectives:

- To determine the existing hydraulic capacity of the City of Grass Valley wastewater collection system and its components.
- To identify system limitations such as bottlenecks and infrastructure incapable of accommodating future growth.

Some of the advantages that PCSWMM holds over other similar hydrodynamic modeling packages are:

- Proven ability to efficiently and accurately model municipal wastewater collection systems for both dry weather flow and wet weather flow regimes.
- Extensive model input tools, visualization, and analysis features.
- GIS-integration and CAD format support.
- Developer's history of consistent and reliable technical and customer support.
- Overall inexpensive investment required by the City of Grass Valley to purchase and maintain this software, if they so choose

4.3.2 Model Construction

The GIS database files containing the physical collection system information (pipe lengths, diameters, inverts, manhole depths, etc.) were imported into the modeling software. The data import resulted in an initial model build containing the necessary information for pipes and junctions.

Once imported into the model, a number of issues were found in the GIS source data:

- Invert and pipe slope, and size inconsistency. In some cases, the GIS data indicated pipes with negative or highly inconsistent slopes. Many of these errors were addressed through a field survey, conducted by Andregg Geomatics. Stantec identified 71 manholes throughout the system where elevations were uncertain and needed to be confirmed. The elevations were gathered from measuring the elevation of the manhole rims, followed by manual measurement of the sewer depth from rim. These elevations were applied to the upstream and downstream sewer inverts. A summary of field survey activities is located in **Appendix C**.

- Connectivity errors. These errors were most common, and were generally a result of slight incompatibilities between CAD and GIS. These errors were resolved by conducting downstream and upstream flow tracing, and manually snapping the links together at any disconnects.

- Incomplete data. Although rare, some assumptions were required to complete the model database for connectivity, pipe sizes, and elevations. In most circumstances, these issues were resolved by obtaining field survey or reviewing as-built data.

The model is comprised of a network of data elements called *nodes* and *links*. The *nodes* and *links* represent the components of a typical wastewater collection system.

- A *node* is a point in the network having an X and Y coordinate. Nodes can represent manholes, wet wells, chambers, or outfalls.

- *Links* convey flow between nodes. They are connected at one end to a start node and the other end to an end node. Links can represent gravity sewers, force mains or pumps.

4.3.2.1 Pipes and Manholes

The City of Grass Valley's existing wastewater collection system consists of approximately 61 miles of sewer line. They are generally defined as all sewers tributary to the City of Grass Valley WWTP, and range in size from 4-inch to 36-inch diameter. The sewer model only includes the primary trunks where detailed elevation data was provided, a process typically referred to as a "skeleton" model of approximately 18 miles of sewer line in the existing model.

4.3.2.2 Pump Stations

The City of Grass Valley currently maintains and operates seven active pump stations throughout the wastewater network, to provide service to low-lying areas within the city that would not otherwise be serviced. The pump stations, which were all included within the model, are listed below:

- Carriage House Sewer Lift Station (953 Freeman Lane)
- Joyce Drive Sewer Lift Station (approximately 174 Joyce Drive)
- Morgan Ranch Sewer Lift Station (495 Morgan Ranch Drive)
- Morgan Ranch West Sewer Lift station (783 Morgan Ranch Drive)
- Railroad Avenue Sewer Lift Station (302 Railroad Avenue)
- Slate Creek Sewer Lift Station (11550 Slate Creek Road)
- Taylorville Sewer lift station (approximately 928 Taylorville Road)

Each lift station is represented in the model by a characteristic storage node, a pump and a force main with simple controls to manage pump starts and stops. **Table 4-3** summarizes the details of each pump station represented in the model.

Table 4-3 Existing Pump Station Characteristics

Pump Station	No. Pumps	Pump Make	Pump Model	Firm Capacity	Total Capacity
Carriage House LS	2	PACO	QDF-415-15	0.23 Mgal/d	0.46 Mgal/d
Joyce Drive LS	2	Flygt	NP 3153.091	0.90 Mgal/d	
Morgan Ranch LS	2	Flygt	NP 3153.452	0.50 Mgal/d	0.66 Mgal/d
Morgan Ranch West LS	2	Flygt	CP 3102.090	0.24 Mgal/d	
Railroad Ave LS	2	Flygt	NP 3085	0.24 Mgal/d	
Slate Creek LS	2 (3 in future)	Godwin Dri-Prime	HL80	0.43 Mgal/d	0.58 Mgal/d
Taylorville LS	2	PACO	495 QDN	0.16 Mgal/d	0.18 Mgal/d

4.3.2.3 Subcatchments

Unlike other hydrodynamic modeling programs, PCSWMM (SWMM5) does not use subcatchments to generate wastewater flows or rain-dependent inflow and infiltration (RDII). All wastewater generation parameters are assigned to sewer nodes based upon the wastewater generation analysis presented in Chapter 3 of this report.

4.3.2.4 Design Storm

Design storms are usually simulated in the hydraulic model to assess the capacity of the sewer system being studied under wet weather conditions. This is typically done with the goal of assessing potential risk of surcharging the system, which may result in sanitary sewer overflows (SSOs). A 1:10 year return period storm, with a 24-hour duration following the Huff design storm distribution was selected to assess system capacity under wet weather conditions. For reference, the storm is shown again in **Figure 4-3**.

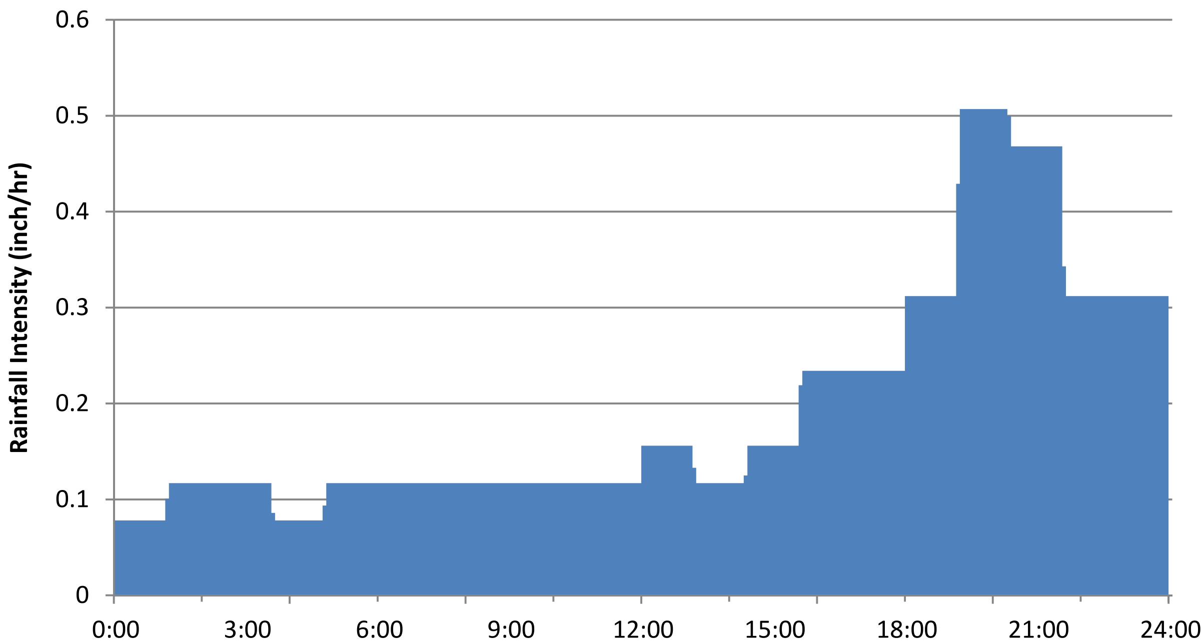


Figure 4-3 Rainfall Hyetograph for 1:10 Year, 24-hour Huff Design Rainfall

4.3.3 Model Calibration

The calibration process is required to verify the accuracy of the model at predicting the system performance under varying flow conditions. The model was calibrated using actual dry weather and wet weather conditions (utilizing both the flow monitoring and precipitation data collected during the 2014 flow monitoring and I/I investigations). The calibrated model was then used to assess system performance under design storm conditions.

4.3.3.1 Dry Weather Flow Calibration

To calibrate dry weather flow (DWF) in the system, flow data was analyzed from the eight different flow monitors located within the pipe network. The catchments were assigned to the contributing flow monitor that detected the wastewater flow produced within the individual sewershed catchments. All catchments within each region were assumed to have similar loading characteristics.

The model was calibrated against the flow monitoring data gathered by V&A from February 6th, 2014 to April 9th, 2014. During this period, there were several rainfall events that resulted in WWF responses. It should also be noted that not all flow monitors were installed for the full range of the monitoring period. However, the data collected during periods between wet weather events was sufficient to allow model DWF calibration.

The comparison and results of the DWF calibration procedure are presented in Figures D-1 to Figures D-4 in **Appendix D** and summarized in **Table 4-4**.

Table 4-4 DWF Calibration Results

	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Average Dry Weather Flow [Mgal/d]								
Modeled	0.14	1.31	0.18	1.13	0.17	0.17	0.37	0.33
Measured	0.14	1.37	0.20	1.19	0.16	0.17	0.37	0.33
% Error	5%	-4%	-6%	-5%	3%	0%	1%	2%
Peak Dry Weather Flow [Mgal/d]								
Modeled	0.44	2.01	0.31	1.73	0.26	0.26	0.74	0.57
Measured	0.68	2.13	0.33	1.89	0.40	0.27	0.73	0.55
% Error	-36%	-6%	-4%	-8%	-35%	-5%	2%	4%

Based upon Best Practices for modeling, the DWF model results are generally within 10% tolerances and are considered to be sufficiently accurate, with the exception of:

- FM#1: The flow data gathered reported peak DWF through FM#1 on weekends almost double the peak DWF on weekdays. It is recommended that this be investigated further.
- FM#5: The flow data gathered reported some anomalously high flows on Mondays that last for approximately 15 minutes. These flows were occurring consistently week to week, indicating that the flows were real, and not some error introduced by the flow monitor. The sources of this flow remain unknown at this time, and were ignored during calibration as it was considered to be a transient event.

4.3.3.2 Wet Weather Flow Calibration

The calibrated DWF model was used as the basis for expanding the model to include wet weather flow (WWF). The two rainfall events Feb. 8th – Feb. 11th, 2014 and Feb. 28 – March 1st, 2014 were used for the calibration process. Additional verification of the model for periods of Nov. 28 – Dec. 4, 2012 has been examined but not shown in this report since data quality are not good as the other two rainfall events.

The WWF model was calibrated using the "Unit Hydrograph" method, with a set of three triangular unit hydrographs (UH) to represent the fast-response, medium-response, and slow-response of the rain dependent inflow and infiltration (RDII). Each UH is represented by three parameters (R, T, and K) which are used to calculate the intensity, duration, and rate of recession of the hydrograph. The R parameter represents the fraction of rainfall volume that enters the sewer system, T represents the time from the onset of rainfall to the peak of the UH (in hours), and K represents the ratio of time to recession of the UH to the time to peak. Newer versions of the SWMM engine have adopted three additional parameters (D_{max}, D_{rec}, and D_o) to more accurately model the antecedent moisture conditions in the soil. D_{max} represents the maximum storage depth (inches). D_{rec} represents the "recovery rate", or how quickly the storage dries out (inches per day). D_o (a parameter which is specific to each scenario) represents the starting moisture condition at the time of the simulation (inches). The three parameters combined result in the model having a delayed response to the start of a rain event.

The calibrated RTK parameters are presented in **Table 4-5** and **Table 4-6**.

Table 4-5 RDII Calibration Parameters - RTK

Flow Monitor	Fast Response			Medium Response			Slow Response		
	R	T	K	R	T	K	R	T	K
FM#1	0.004	1	2	0.006	3	5	0.015	10	5
FM#2	0.013	1	2	0.007	2	5	0.02	10	5
FM#3	0.012	1	0.8	0.012	2.5	8	0.016	10	5
FM#4	0.013	1	2	0.01	2	5	0.02	10	5
FM#5	0.035	1	1.8	0.05	2.5	6	0.03	10	5
FM#6	0.025	1	1	0.03	2.5	8	0.02	10	5
FM#7	0.007	1	1	0.0035	2.5	8	0.011	10	5
FM#8	0.013	1	1	0.005	2	7	0.018	10	5

Table 4-6 RDII Calibration Parameters – D_{max} and D_{rec} (a)

Flow Monitor	Fast Response		Medium Response		Slow Response	
	D_{max}	D_{rec}	D_{max}	D_{rec}	D_{max}	D_{rec}
FM#1	0.004	1	0.006	3	0.015	10
FM#2	0.013	1	0.007	2	0.02	10
FM#3	0.012	1	0.012	2.5	0.016	10
FM#4	0.013	1	0.01	2	0.02	10
FM#5	0.035	1	0.05	2.5	0.03	10
FM#6	0.025	1	0.03	2.5	0.02	10

Flow Monitor	Fast Response		Medium Response		Slow Response	
	D_{max}	D_{rec}	D_{max}	D_{rec}	D_{max}	D_{rec}
FM#7	0.007	1	0.0035	2.5	0.011	10
FM#8	0.013	1	0.005	2	0.018	10

(a) D_o was not listed as it varied per rainfall event. This is an instantaneous value.

The WWF model results for the rain events were plotted against the flow monitoring data. Figure D-5 to Figure D-13 in **Appendix D** show the comparisons of the "Measured" and "Modeled" WWF. Results are summarized in **Table 4-7**.

Table 4-7 WWF Calibration Results

	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Event 1 Peak Wet Weather Flow [Mgal/d]								
Modeled	0.85	9.53	2.1	7.48	1.65	1.13	2.11	2.21
Measured	0.68	9.14	2.31	6.98	1.74	1.06	2.04	1.82
% Error	24.3%	4.4%	-9.0%	7.2%	-5.0%	6.8%	3.6%	21.8%
Event 1 Total Volume [million gallons]								
Modeled	1.64	21.6	4.46	17.13	3.41	2.43	5.01	4.85
Measured	1.36	22.19	4.46	18.92	3.33	2.46	5.53	4.85
% Error	20.7%	-2.7%	0.2%	-9.5%	2.4%	-0.9%	-9.4%	0.0%
Event 2 Peak Wet Weather Flow [Mgal/d]								
Modeled	0.62	6.72	1.31	5.44	1.16	0.79	1.53	1.59
Measured	0.57	7.12	1.16	5.67	1.1	0.86	1.66	1.6
% Error	7.6%	-5.6%	12.5%	-4.1%	5.4%	-7.7%	-8.0%	-0.6%
Event 2 Total Volume [million gallons]								
Modeled	1.39	17.02	2.75	14.27	2.47	2.12	4.45	4.15
Measured	1.12	16.1	2.73	15.73	2.21	2.05	4.62	4.01
% Error	24.0%	5.7%	0.5%	-9.2%	11.8%	3.7%	-3.5%	3.4%

The calibrated WWF model results are generally within 10% of the measured flows, with the exception of the following:

- FM#1: The model is over-predicting the peak flow from this sewershed. Due to the proximity of the flow monitor to upstream pump stations, it is probable that the model is over-predicting the effects of inertia on the flow, and is not introducing enough attenuation to the peaks. At this time, the model should be considered a very conservative representation of this sewershed.

- FM#8: Although the total RDII volume for the first rainfall event was calibrated exactly, the model is not predicting the response of the storm with perfect accuracy. It should be noted that for the second rainfall event, the model is both qualitatively and quantitatively accurate. For this first rainfall event, it is possible that due to the spatial variability of actual storms, this sewershed may have been inundated with higher intensity flows. As this sewershed is upstream and flows through FM#4 and FM#2, the calibration of this sewershed had to be balanced with the calibration of those sewersheds.

4.3.3.3 City of Grass Valley WWTP Influent Flow Verification

In order to verify the calibration of the PCSWMM hydrodynamic model, the flow generated during the recorded rainfall events were compared against the flow data gathered by the City's WWTP influent flow meter. Data at the WWTP has been gathered in approximately 1.5 hour intervals.

Table 4-8 provides a summary of the verification results at the WWTP for the two rainfall events.

Table 4-8 WWF Calibration Results

	Event 1 Peak WWF (Mgal/d)	Event 1 Total Volume (million gallons)	Event 2 Peak WWF (Mgal/d)	Event 2 Total Volume (million gallons)
Modeled	9.91	25.73	7.40	12.55
Measured	10.22	24.50	7.05	11.23
% Error	3.1%	-4.8%	-4.7%	-10.5%

The calibrated WWF model results are generally within 10% of the measured flows, and are considered to be sufficient for capacity evaluation purposes.

4.4 CAPACITY EVALUATION RESULTS

4.4.1 Purpose

The purpose of this section is to provide a summary of the results of the level of service (LOS) performance analysis of the City of Grass Valley wastewater collection system when the 1:10-year, 24-hour design storm is applied to the system.

4.4.2 Recommended Capacity Evaluation Criteria

The 1:10 year, 24-hour design rainfall event was applied to the PCSWMM model to evaluate the LOS performance in meeting the following primary criteria:

- Wastewater flow metrics
- Allowable surcharge
- Lift station capacity

4.4.2.1 Wastewater Flow Metrics

The flows within the wastewater system are assessed based upon three wastewater flow metrics, and results are presented in plan-view figures:

- Peak flow within each within each sewer under design storm conditions. These results are a good indication of relative flow distribution throughout the study area.
- Hydraulic loading ratio within each sewer under design storm conditions. Hydraulic loading ratios are commonly used as a metric to evaluate the performance of a collection system. The hydraulic loading ratio (HLR) is mathematically defined as the peak modeled flow divided by the full pipe capacity, and is denoted "Max/Full Flow" in the results tab of the PCSWMM Sewer model.
- Residual capacity within each sewer when subjected to the peak flows of the design storm conditions. This result is a calculation of the Manning's full pipe capacity minus the peak flow, and presented in plan-view. This performance indicator is useful in illustrating the relative remaining capacity throughout the study area and for use in evaluating future servicing strategy.

4.4.2.2 Allowable Surcharge Criteria

The maximum allowable surcharge (HGL) in the gravity portion of the sanitary sewer system must remain at least 8 feet from the ground surface (at least 8 feet of freeboard is required) during a design storm scenario. Under this criterion, existing sewers with depths greater than 8 feet have been said to be within LOS criteria if the peak surcharge elevation results in a freeboard of greater than 8 feet. Any sewer identified with depths less than 8 feet are considered deficient should any surcharging result. Thus, the recommended improvements identified in Section 4.5 are generally based upon the two criteria below:

c. minimum freeboard 8 feet(depth below rim);

d. surcharging less than 1 foot above pipe crown.

If either of the above two criteria fails, the conduit is proposed to be upgraded.

4.4.2.3 Lift Station Capacity Criteria

This result compares the inflow hydrograph to the pumping capacity of the lift station to identify potential capacity constraints that would not be identified by the surcharging criteria due to the typical depth of lift station wet well structures.

4.4.3 Modeled Scenarios

This study assessed system performance for five projected growth scenarios for the City of Grass Valley wastewater collection system. These growth scenarios are:

- **Existing Development**. This scenario assesses the impact of the design storm on the existing system.
- **Existing plus Infill Development (Existing Build-out)**. During the assessment, it was determined that there were properties within the existing City service area boundary that were unoccupied/undeveloped. This scenario assesses the impact to the system should the design storm occur once all of these vacant parcels have been developed.
- **Existing Build-out plus Near-Term Growth Horizon**. This scenario includes lands identified within the Near-Term Sphere of influence in the City's 2020 General Plan and described in Chapter 3. The Near-Term growth scenario (~5-year) includes a portion of the "Loma Rica Special Development Area" (lands west of Brunswick Road, north of Idaho Maryland Rd and east of Sutton Way), and a portion of the Berriman Ranch & Adjacent Property Area.
- **Existing Build-out plus Near-Term plus Long-Term growth**. This scenario expands the service boundary to include anything within the Near-Term (5-year) and Long-Term (10-year) growth horizons as identified in Chapter 3. This includes the balance of the Loma Rica special development area.
- **Full Build-out Growth Horizon**. This includes all additional lands identified by the 2020 General Plan including Special Development Area of North Star and Kenny Ranch and the balance of the Berriman Ranch & Adjacent Property Area, as well as all Areas of Concern identified in the 2020 General Plan.

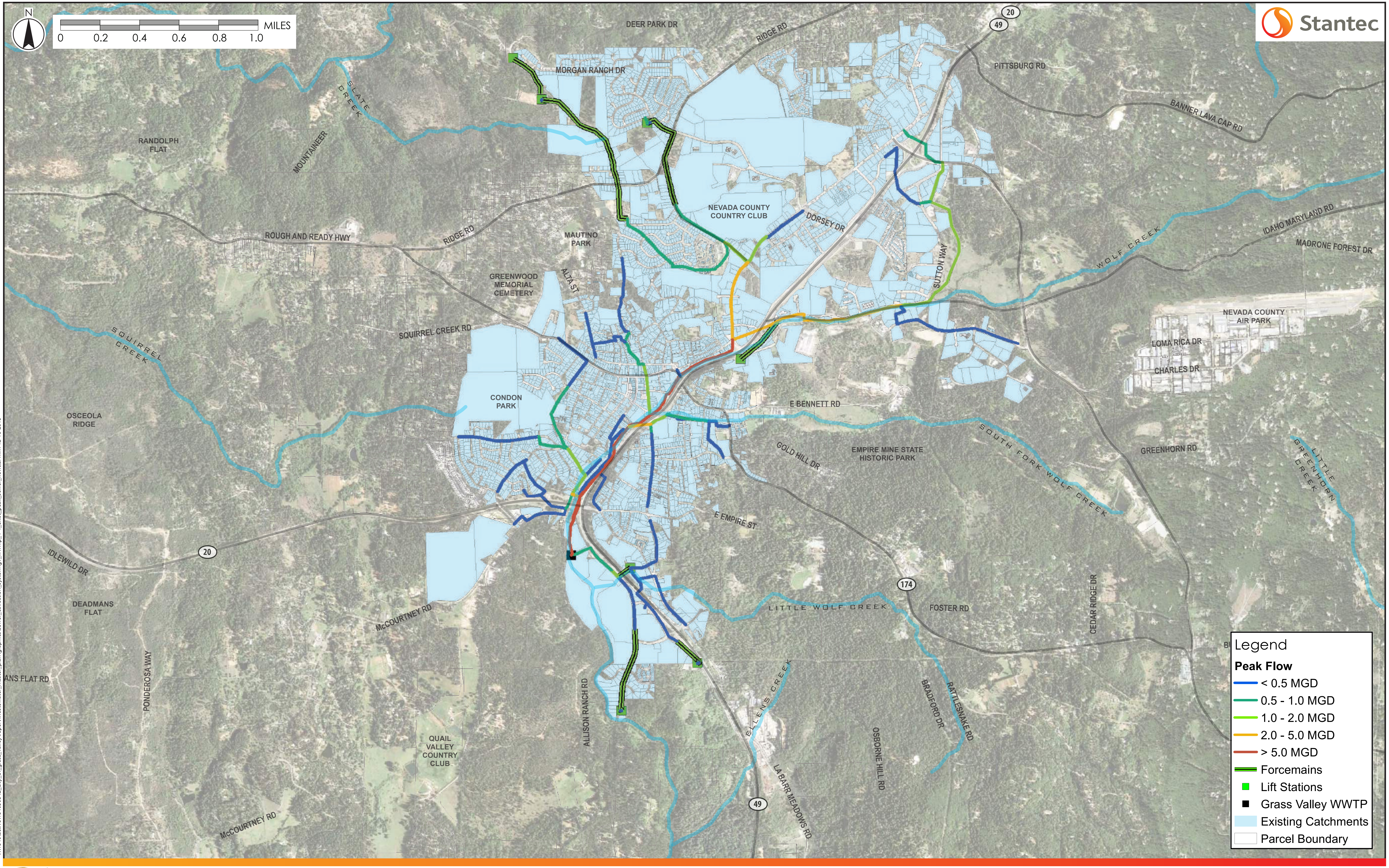
4.4.4 Model Results – Existing Level of Development

The peak modeled sewer flows for the 1:10 year, 24-hour Huff design event under Existing conditions are shown in **Figure 4-4**. As this figure shows, the majority of flow within the study area is conveyed along one trunk sewer (parallel to Highway 49) that is fed by five main trunk laterals. An additional trunk sewer from the southeast (also parallel to Highway 49) serves the southernmost portion of the service area.

Table 4-9 shows the summary of flows for each of the flow monitor locations (primary sewer shed nodes) modeled for the 1:10 year Huff rainfall event.

Table 4-9 Flow Characteristics of the Existing System under Existing Conditions

	WWTP	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Catchment Area (acre)	2,178	170	2,008	432	1,575	108	108	782	456
Average DWF (Mgal/d)	1.5	0.1	1.3	0.2	1.1	0.2	0.2	0.4	0.4
Peak DWF (Mgal/d)	2.4	0.4	2.0	0.3	1.7	0.2	0.3	0.7	0.6
Peak WWF 10yr Huff (Mgal/d)	13.4	0.9	12.7	2.4	10.3	2.4	1.4	2.8	2.9
Peak Flow (RDI only) (Mgal/d)	11.0	0.5	10.6	2.1	8.6	2.1	1.1	2.1	2.3
Peak RDII rate (gpd/acre)	5,066	2,847	5,300	4,802	5,430	19,471	10,372	2,630	5,108



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City of Grass Valley
Wastewater System Master Plan

Figure 4-4
Existing Peak Sewer Flows

4.4.4.1 Collection System Capacity to Accommodate Existing Flows

Figure 4-5 shows the hydraulic loading ratio of the existing system for the 1:10 year Huff design rainfall. **Figure 4-6** shows the residual capacity in the existing system under the design rainfall conditions. **Figure 4-7** shows the minimum freeboard expressed as depth below manhole rim in the existing system under the design rainfall conditions.

Under Existing conditions, the 1:10 year Huff design storm is predicted to generate a peak flow of approximately 13.4 Mgal/d at the WWTP. This storm event is predicted to cause surcharging in several reaches throughout the network. To help identify the extent of surcharging within the existing network, hydraulic grade line (HGL) profiles have been included within **Appendix E**. These HGL profiles show the peak surcharge elevation along the identified reach. Note that the profiles also include the results for other growth scenarios, to be discussed in the following sections. **Figure 4-8** shows the plan view for eight identified HGL profiles discussed.

The following provides a summary of the existing system surcharging and corresponding HGL profiles, presented in **Appendix E**:

- **HGL Profile 1** (**Figure E-1**): Minor surcharging occurs in two separate manholes (S10-4, S12-2) as a result of insufficient capacity in the 8-inch sewers downstream, respectively. Manhole S10-4 is situated 476 feet east of Sutton Way and Manhole S12-2 near the intersection of Sutton Way and Idaho Maryland Rd. Manhole S10-4 is predicted to result in a freeboard of less than 8-feet, and therefore does not meet the recommended LOS criteria.

- **HGL Profile 2** (**Figure E-2**): Surcharging occurs, resulting in capacity exceedance at manhole J13-10 and five additional manholes in the vicinity fail the recommended LOS. Manhole J13-10 is situated on North Church Crourt near North Church Street. In general, the sewer reach downstream of manhole J13-11 ending at manhole K15-7 is predicted to be near or exceeding capacity.

- **HGL Profile 3** (**Figure E-3**): Minor surcharging occurs in one manhole (M15-8) on Colfax Avenue and Henderson Street. This surcharging is a result of insufficient capacity in the downstream 8-inch sewer and is predicted to cause a minimum freeboard of less than 8-feet and therefore, does not meet recommended LOS criteria.

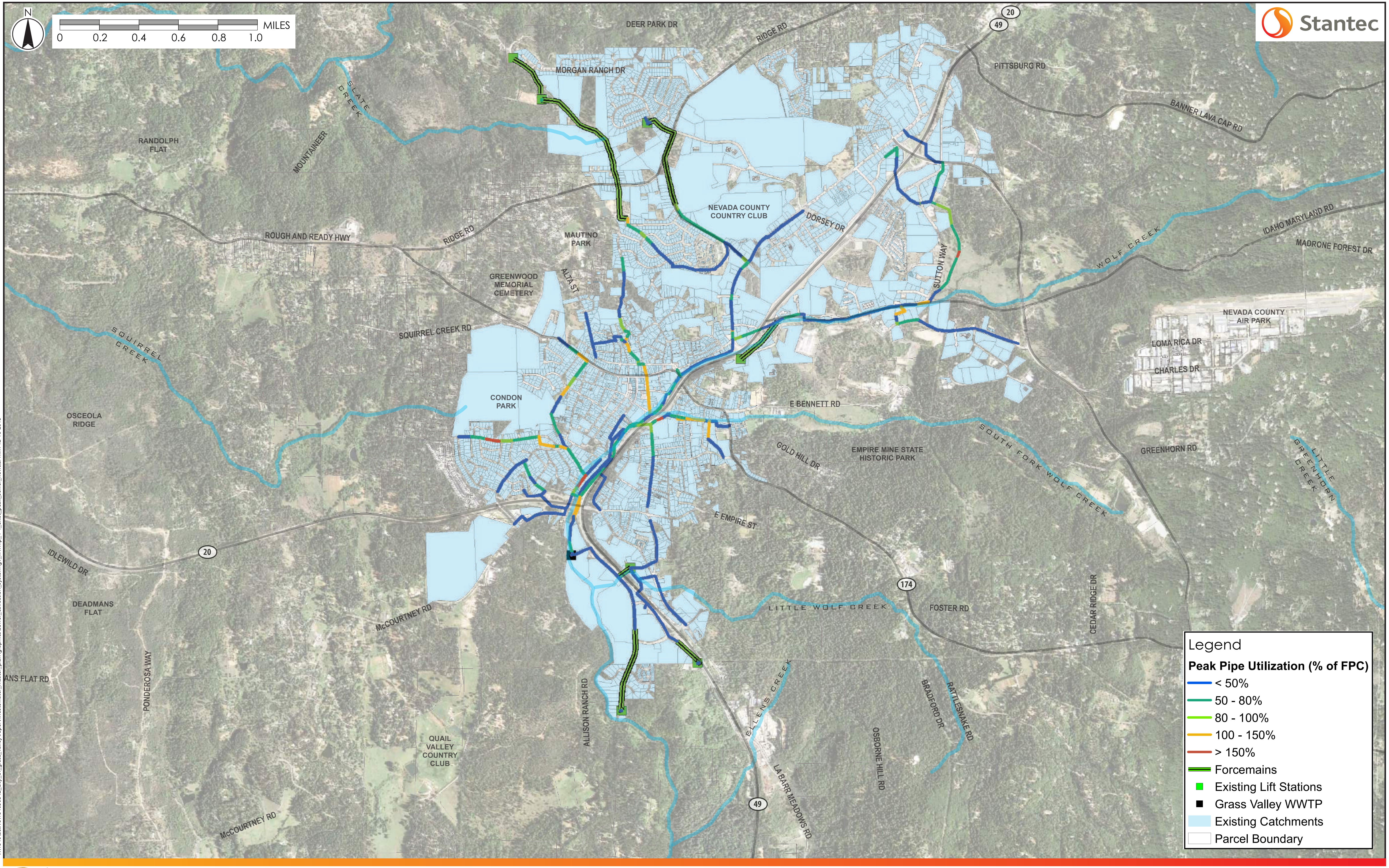
- **HGL Profile 4** (**Figure E-4**): Severe surcharging occurs in one manhole (I16-22) and minor surcharging in an additional three manholes (I13-9, I14-15, I17-12) along this reach. Manhole I16-22 is situated 197 feet northeast of the intersection of Mill Street and Rhode Island Street. The severe surcharging is predicted to result along Mill Street, near the intersection of Rhode Island Street. The three additional manholes are all expected to fail recommended LOS criteria.

- **HGL Profile 5** (**Figure E-5**): Very minor surcharging occurs in one manhole I17-7 (66 feet south of French Ave) and is a result of insufficient capacity in the twin 18-inch sewers crossing underneath Highway 20. There is predicted to be a minimum freeboard of greater than 20 feet for Existing, and therefore meets the recommended LOS criteria. It should be noted that this information is based upon a degree of upstream throttling due to capacity constraints, and this surcharging will worsen as those capacity constraints are eliminated.

- **HGL Profile 6** (**Figure E-6**): Minor surcharging occurs in several sewers upstream of the Idaho-Maryland trunk. This surcharging is a result of insufficient capacity in the 6-inch sewer and results in two manholes that do not meet the recommended LOS criteria (R12-11, R12-12). The two manholes are situated approximately 197 feet south of Idaho Maryland Rd and 279 feet north of Whispering Pines Ln.

- **HGL Profile 7** (**Figure E-7**): Very minor surcharging occurs in one manhole (M12-15) on East Main Street and Harris Street. This manhole is shallow and does not meet the minimum 8-feet cover. The predicted surcharging is less than 0.25 feet but does not meet recommended LOS criteria.

- **HGL Profile 8** (**Figure E-8**): Severe surcharging occurs in five manholes (G15-4, G15-5, H15-4, H16-4, I16-3) along Butler Street. The predicted surcharging is a result of insufficient capacity of four 6 inch sewer conduits (1055, 114, 1039 and 115). These five manholes are all expected to fail recommended LOS criteria.



V:\1840\active\184030342_city_of_grass_valley\report\wastewater_master_plan\graphics\sewer_collection_system\gv_wwmp_4-4_exist_peak_sewer_flows.ai.mlm 12-16-2015

Figure 4-5
Existing Peak Flow Capacity Utilization

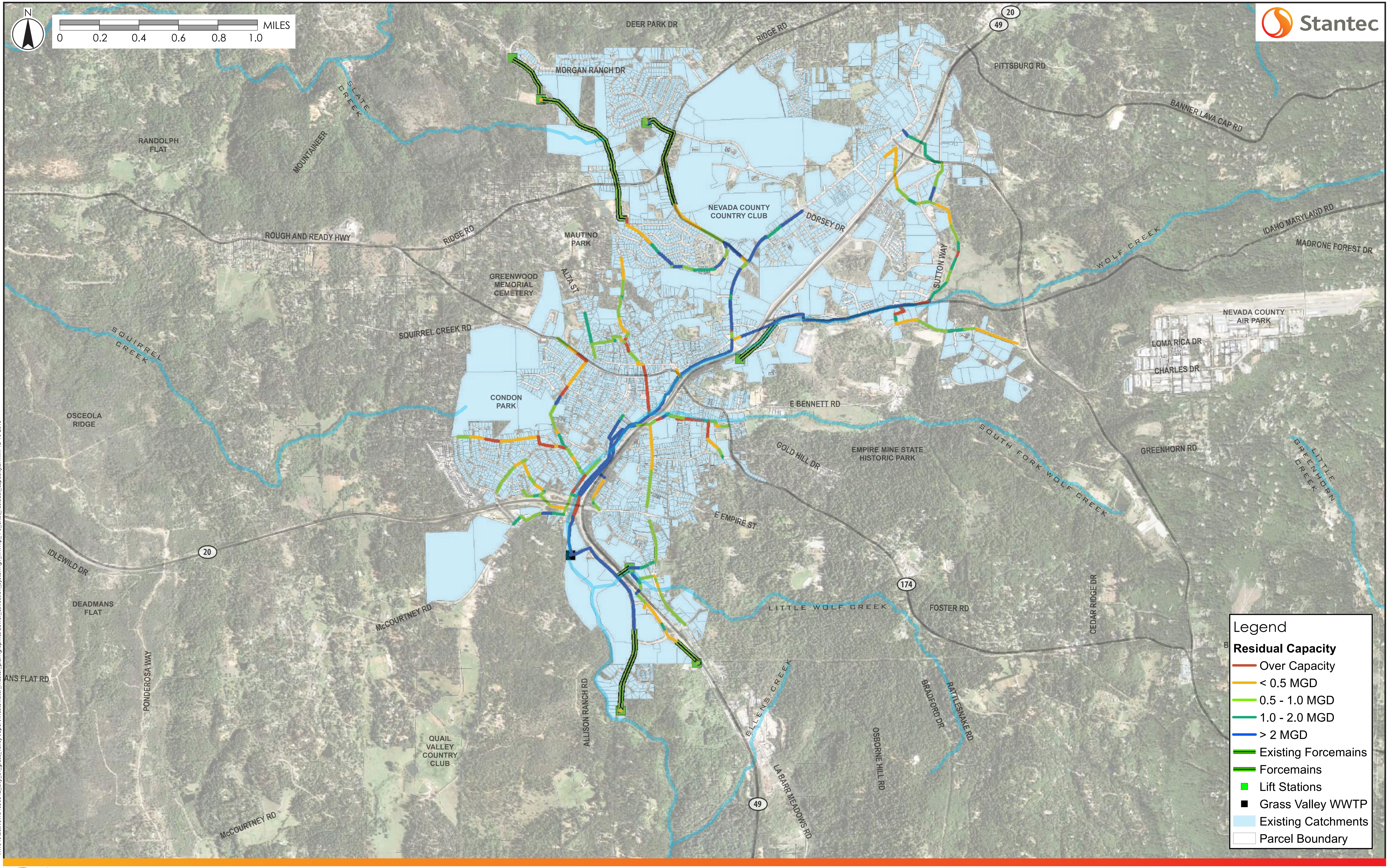


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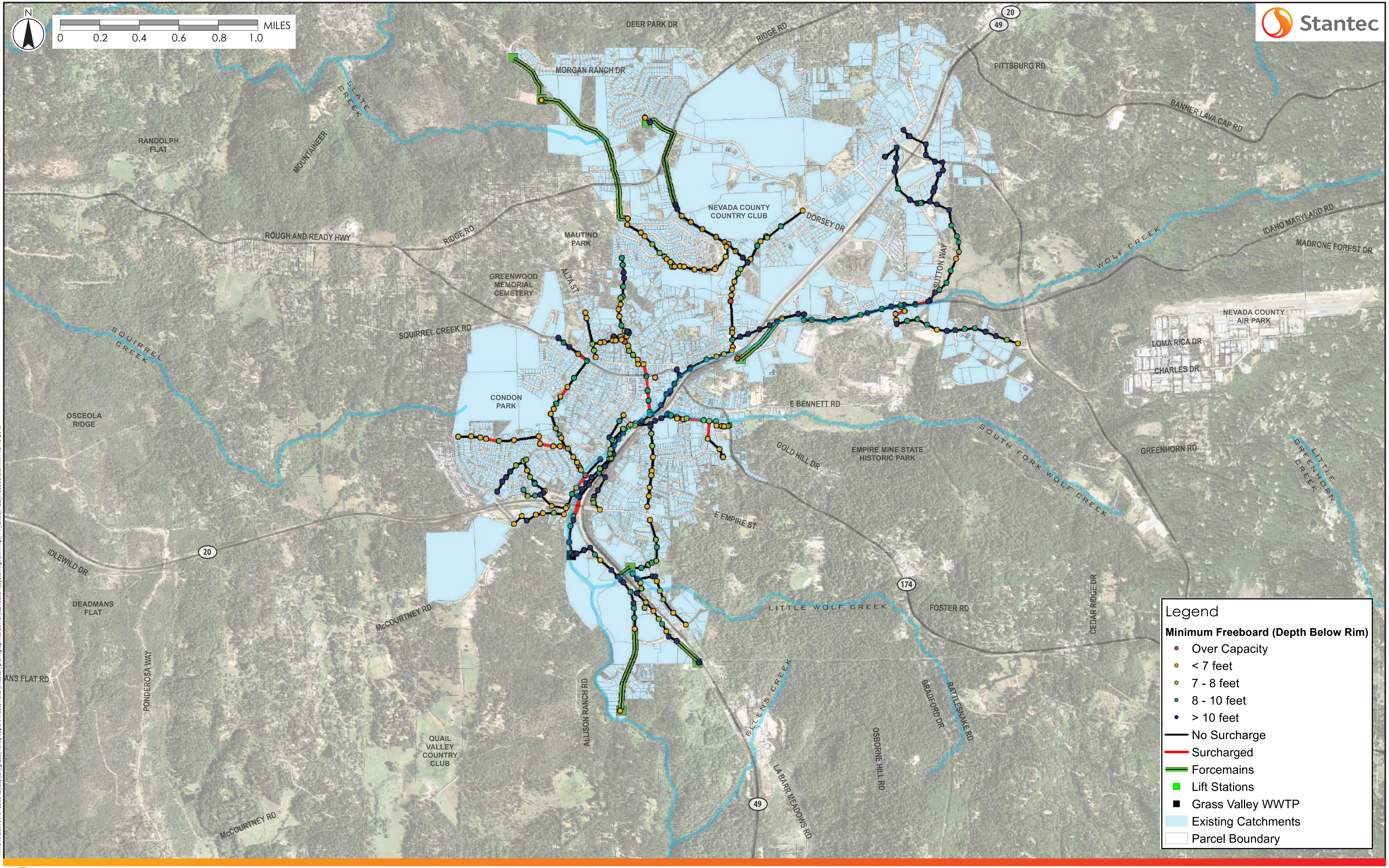
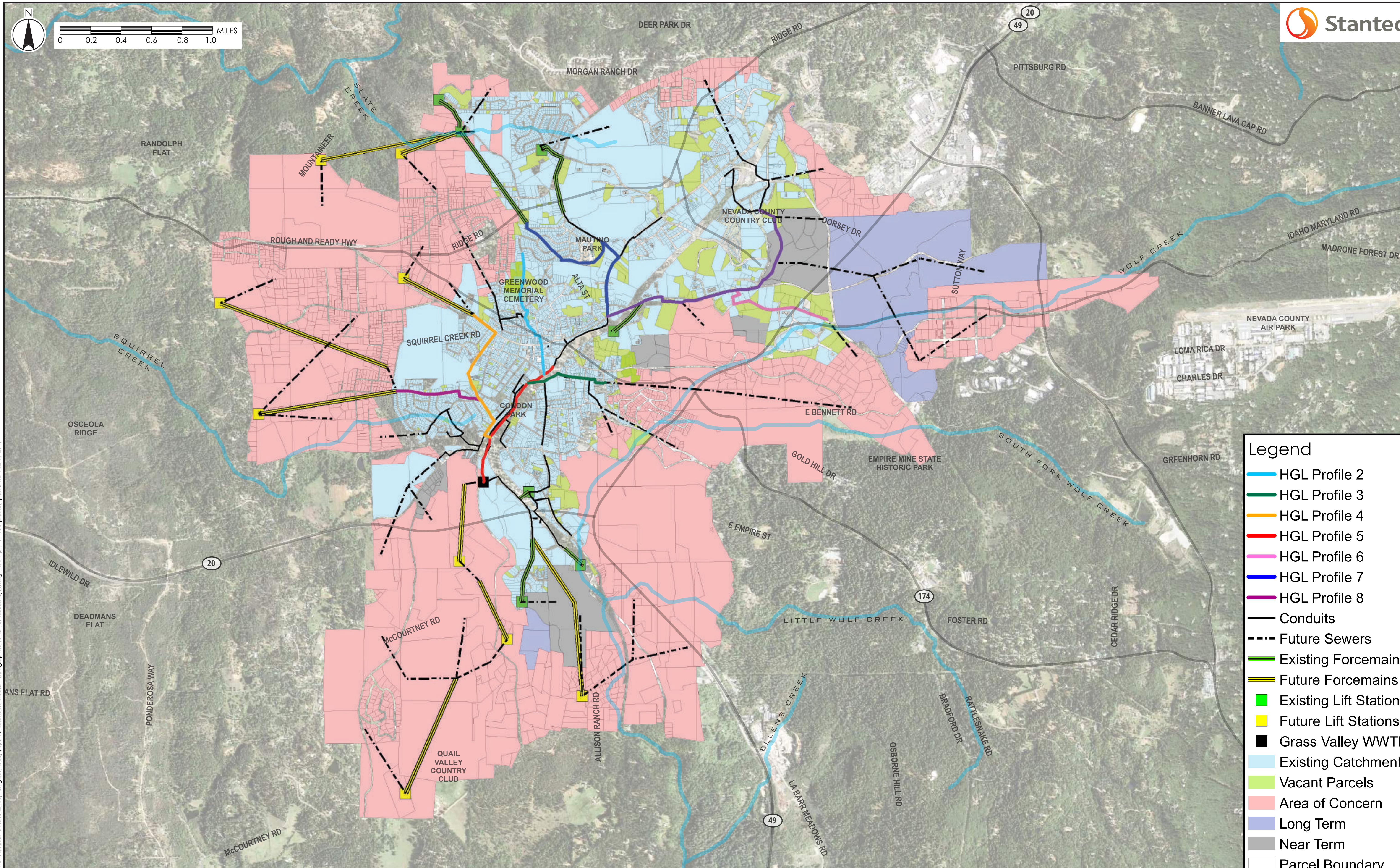
Figure 4-6
Existing Residual Capacity



V:\1840\active\184030342_city_of_grass_valley\report\wastewater_master_plan\graphics\sewer_collection_system\gv_wwmp_4-7_exist_HGL_freeboard.ai\mlm 12-16-2015

Figure 4-7
Existing HGL Freeboard



V:\1840\active\184030342_city_of_grass_valley\report\wastewater_master_plan\graphics\sewer_collection_system\gv_wwmp_4-8_HGL_profiles_plan.ai mlm 12-16-2015

Figure 4-8
HGL Profiles Plan View for Existing & Buildout

4.4.4.2 Lift Station and Forcemain Capacity to Accommodate Existing Flows

In general, lift station design should provide firm capacity for the peak wastewater design flow. Firm capacity is defined as the pumping capacity of the station with the largest unit out of service in the case where multiple pumps are installed.

4.4.4.2.1 Carriage House Sewer Lift Station

Figure 4-9 shows the inflow and pump discharge at the Carriage House lift station. The peak flow into the wet well for the 1:10 year Huff design event is 0.09 Mgal/d. This facility has adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event.

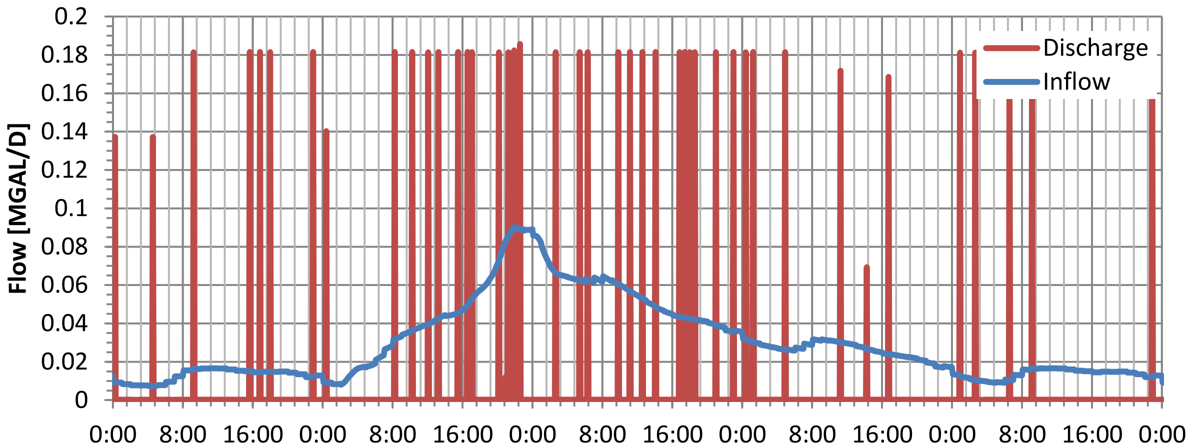


Figure 4-9 Carriage House Lift Station Flow Balance 1:10 Year Huff Design Rainfall Event

4.4.4.2.2 Joyce Drive Sewer Lift Station

Figure 4-10 shows the inflow and pump discharge at the Joyce Drive lift station. The existing peak flow into the wet well for the 1:10 year Huff design event is 0.36 Mgal/d. This facility has adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event.

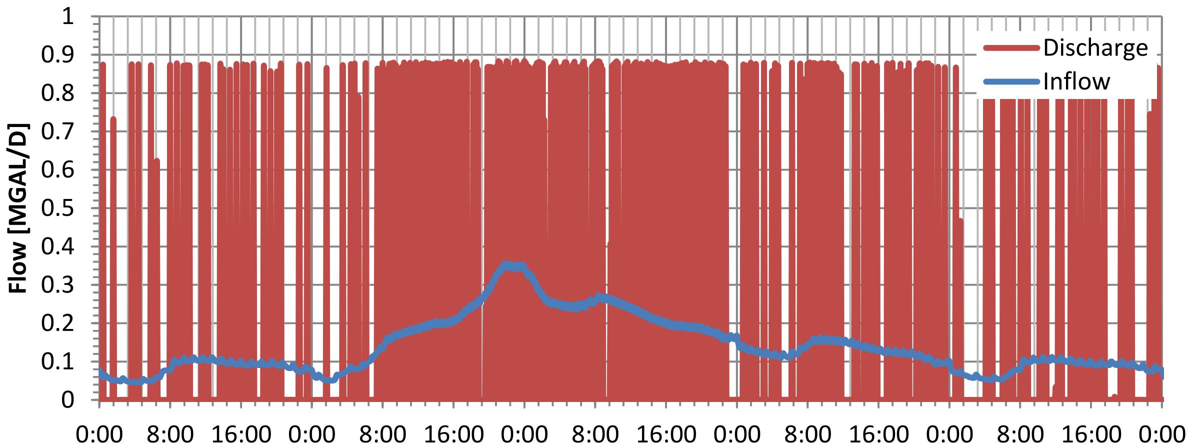


Figure 4-10 Joyce Drive Lift Station Flow Balance, 1:10 Year Huff Design Rainfall Event

4.4.4.2.3 Morgan Ranch Sewer Lift Station

Figure 4-11 shows the inflow and pump discharge at the Morgan Ranch lift station. The peak flow into the wet well for the 1:10 year Huff design event is 0.606 Mgal/d. This facility will rely upon the backup pump during peak flows as it does not have adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event. Upgrades to this lift station should be considered in the City's Improvement Plan.

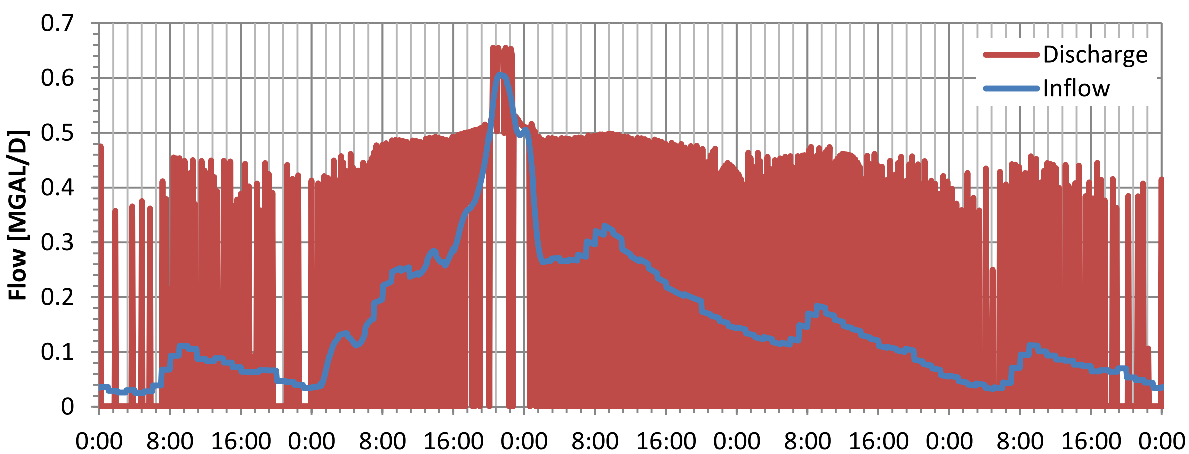


Figure 4-11 Morgan Ranch Lift Station Flow Balance, 1:10 Year Huff Design Rainfall Event

4.4.4.2.4 Morgan Ranch West Sewer Lift Station

Although the Morgan Ranch West Lift Station is included in the hydraulic model it only serves a small number of users and the City does not anticipate that it will serve additional users during future development. For this reason a more limited analysis of this facility was conducted for this Master Plan and no future upgrades are recommended for this lift station.

4.4.4.2.5 Railroad Avenue Sewer Lift Station

Figure 4-12 shows the inflow and pump discharge at the Railroad Avenue lift station. The peak flow into the wet well for the 1:10 year Huff design event is 0.02 Mgal/d. This facility has adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event.

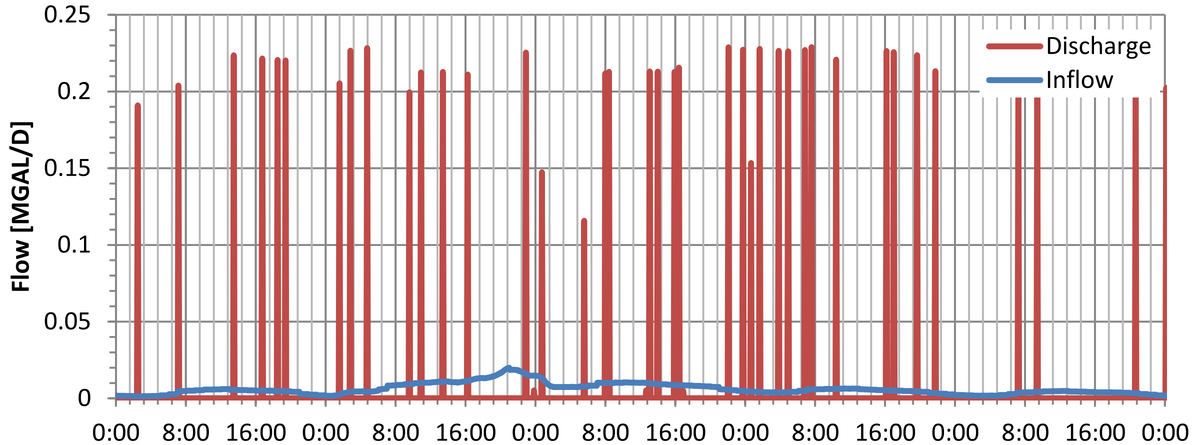


Figure 4-12 Railroad Avenue Lift Station Flow Balance, 1:10 Year Huff Design Rainfall Event

4.4.4.2.6 Slate Creek Sewer Lift Station

Figure 4-13 shows the inflow and pump discharge at the Slate Creek lift station. The peak flow into the wet well for the 1:10 year Huff design event is 0.49 Mgal/d. This facility will rely upon the backup pump during peak flows as it does not have adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event. Upgrades to this lift station should be considered in the City's Improvement Plan.

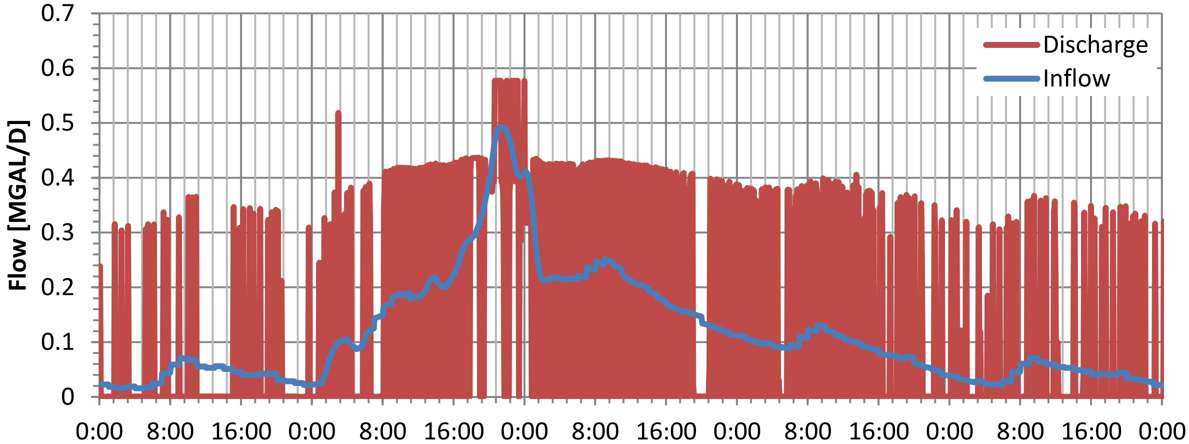


Figure 4-13 Slate Creek Lift Station Flow Balance, 1:10 Year Huff Design Rainfall Event

4.4.4.2.7 Taylorville Sewer Lift Station

Figure 4-14 shows the inflow and pump discharge at the Taylorville lift station. The peak flow into the wet well for the 1:10 year Huff design event is 0.013 Mgal/d. This facility has adequate firm capacity to accommodate the existing inflows modeled for the 1:10 year Huff design event.

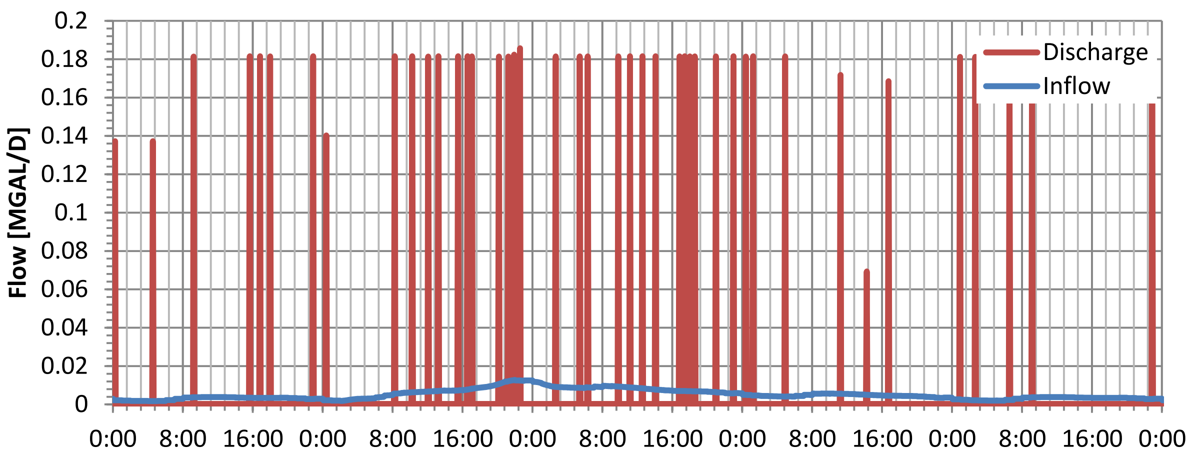


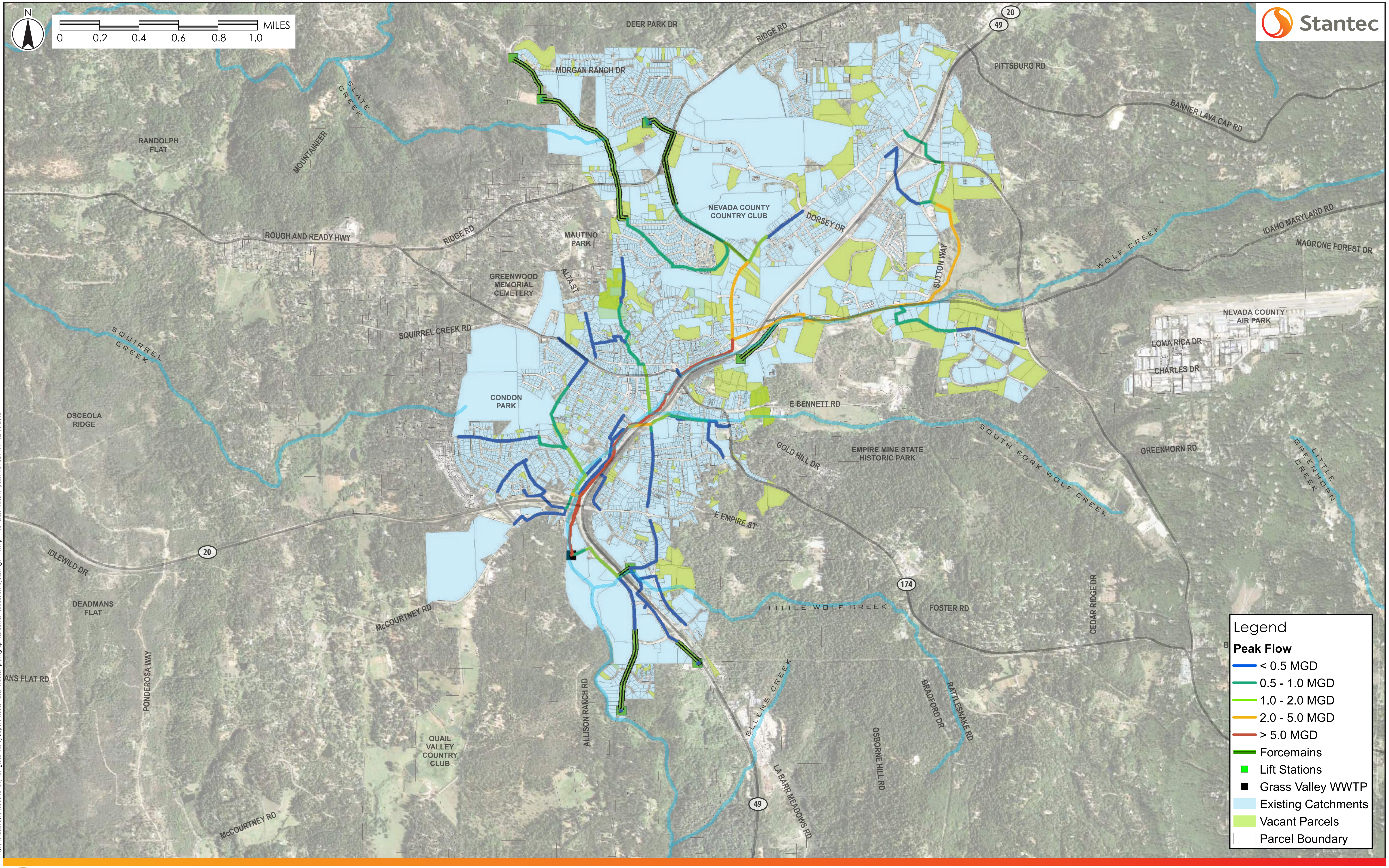
Figure 4-14 Taylorville Lift Station Flow Balance, 1:10 Year Huff Design Rainfall Event

4.4.5 Model Results – Existing Service plus Infill Level of Development (Existing Service Area Build-out)

The peak modeled sewer flows for the 1:10 year, 24-hour Huff design event with the Existing Service Area Built-out are shown in **Figure 4-15**. **Table 4-10** shows the summary of flows for each of the primary sewer shed nodes modeled for the 1:10 year Huff rainfall event.

Table 4-10 Flow Characteristics of the Existing System under Existing + Infill Conditions

	WWTP	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Catchment Area (acre)	2,627	196	2,431	449	1,982	160	144	855	681
Average DWF (Mgal/dMgal/d)	1.8	0.2	1.6	0.2	1.4	0.2	0.2	0.4	0.5
Peak DWF (Mgal/d)	2.9	0.4	2.5	0.3	2.2	0.3	0.3	0.8	0.8
Peak WWF 10yr Huff (Mgal/d)	14.9	1.0	14.03	2.4	11.7	2.6	1.4	3.0	3.7
Peak Flow (RDI only) (Mgal/d)	12.0	0.6	11.5	2.1	9.5	2.3	1.10	2.2	2.9
Peak RDII rate (gpd/acre)	4,583	2,868	4,735	4,606	4,772	14,091	7,610	2,566	4,293



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City of Grass Valley
Wastewater System Master Plan

Figure 4-15
Existing + Vacant Peak Sewer Flows

4.4.5.1 Collection System Capacity to Accommodate Existing plus Infill Flows

Figure 4-16 shows the hydraulic loading ratio of the existing system for the 1:10 year Huff design rainfall under Existing Service Area Build-out conditions. **Figure 4-17** shows the residual capacity in the existing sewer system for the 1:10year Huff design rainfall under Existing Service Area Build-out conditions. **Figure 4-18** shows the minimum freeboard in the existing sewer system for the 1:10year Huff design rainfall under Existing Service Area Build-out conditions.

Under Existing plus infill conditions, the 1:10year Huff design storm is predicted to generate a peak flow of 14.94 Mgal/d at the WWTP. This storm event is predicted to cause surcharging in several reaches throughout the network. To help identify the extent of surcharging within the existing network, hydraulic grade line (HGL) profiles have been included within **Appendix E**, which show the peak surcharge elevation along the identified reach. Note that the profiles also include the results for other growth scenarios, to be discussed in the preceding and following sections. A plan view of the eight identified HGL profiles is presented in **Figure 4-8**.

The following provides a summary of the surcharging and corresponding HGL profiles presented in **Appendix E** that relate to the Existing Service Area Build-out scenario:

- **HGL Profile 1** (**Figure E-1**): For this scenario the capacity concerns at manholes S10-4 and S12-2 are expected to result in increased surcharging at those locations. Manhole S10-4 is situated 476 feet east of Sutton Way and Manhole S12-2 near the intersection of Sutton Way and Idaho Maryland Rd. The most upstream manhole, S10-4, was previously identified as failing LOS criteria. This surcharging is not predicted to affect any other manholes for this scenario. The surcharging at the 2nd manhole, S12-2, has worsened and is now affecting one additional manhole upstream (S12-1). Manhole S12-1 is located on Idaho Maryland Road and Railroad Avenue. The minimum freeboard for these two manholes exceeds 8 feet and therefore meets the LOS criteria.

- **HGL Profile 2** (**Figure E-2**): The severe surcharging identified in the Existing scenario is predicted to worsen with the addition of infill development, affecting the reach from manhole J12-5 to manhole K15-7. This reach is situated along Carol Drive to North Church Street, to North Auburn Street and ending at the intersection of CA-20 West and South Auburn Street. The majority of manholes in this reach either fails or nearly fails the LOS criteria.

- **HGL Profile 3** (**Figure E-3**): In the Existing system scenario, it was identified that manhole M15-8 on Colfax Avenue and Henderson Street may experience some surcharging. With the addition of infill development, it is predicted that the surcharging in this manhole will increase to the point that the freeboard will drop below 5-feet, less than the 8-foot minimum freeboard of the LOS criteria.

- **HGL Profile 4** (**Figure E-4**): There is no additional surcharging predicted to occur in this reach. However, the surcharging predicted with the Existing system scenario continues to exist/worsen with Build-out of the service area.

- **HGL Profile 5** (**Figure E-5**): Very minor surcharging is predicted in one manhole I17-7 (66 feet south of French Ave) resulting from insufficient capacity in the twin 18-inch sewers crossing underneath Highway 20. The predicted freeboard has fallen to 19-feet, but is still much greater than the recommended 8-feet, meeting the LOS criteria.

- **HGL Profile 6** (**Figure E-6**): The surcharging in this reach has significantly increased, and is now affecting manholes R12-17 through to R12-10. There is predicted to be an SSO in manhole R12-12.

- **HGL Profile 7** (**Figure E-7**): Very minor surcharging is predicted in one manhole (M12-15) on East Main Street and Harris Street. This manhole is shallow and does not meet the minimum 8-foot cover criteria. The predicted surcharging is less than 0.25 feet but does not meet the recommended LOS criteria.

- **HGL Profile 8** (**Figure E-8**): There is no additional surcharging predicted to occur in this reach.

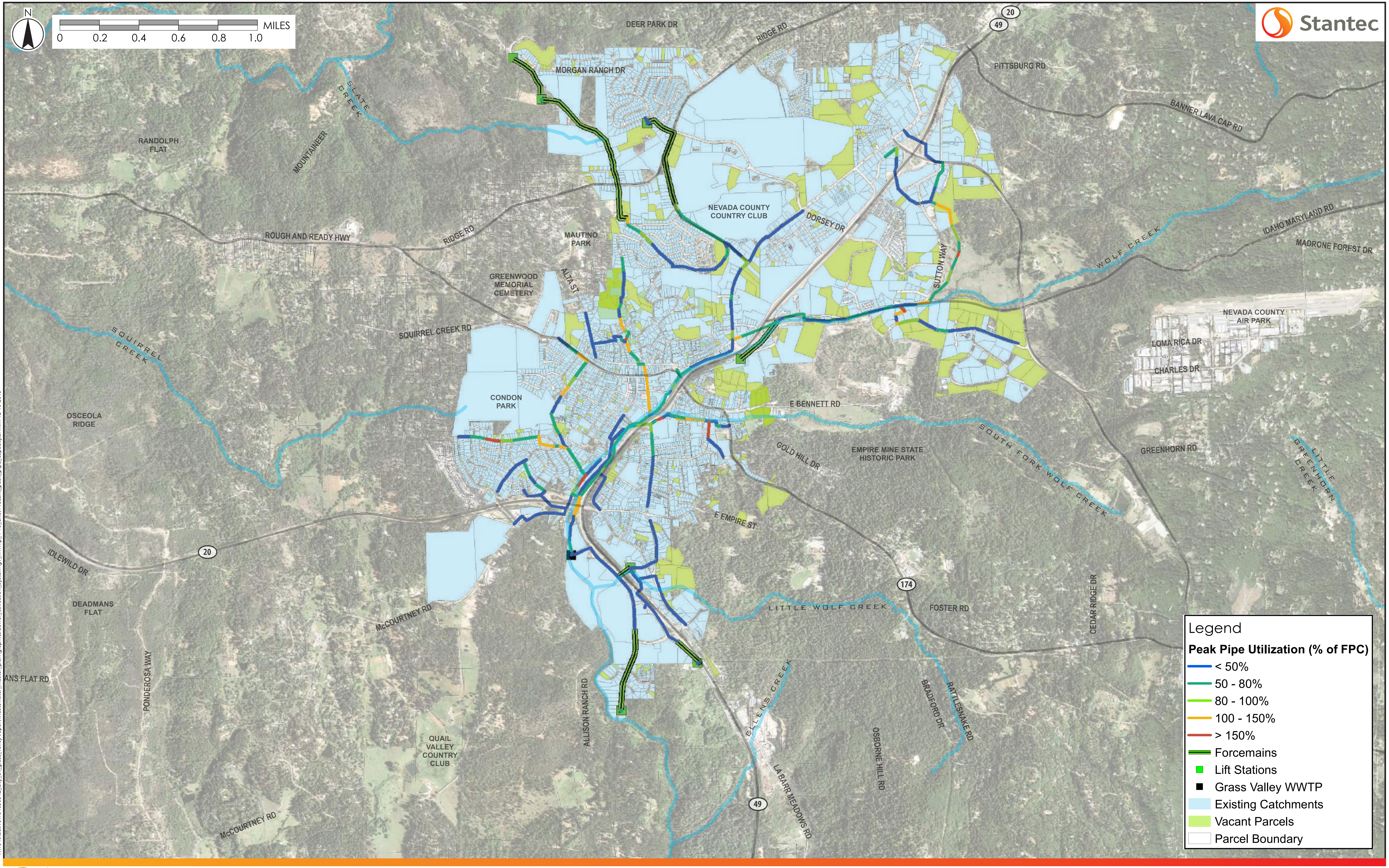

Figure 4-16
Existing + Vacant Peak Flow Capacity Utilization

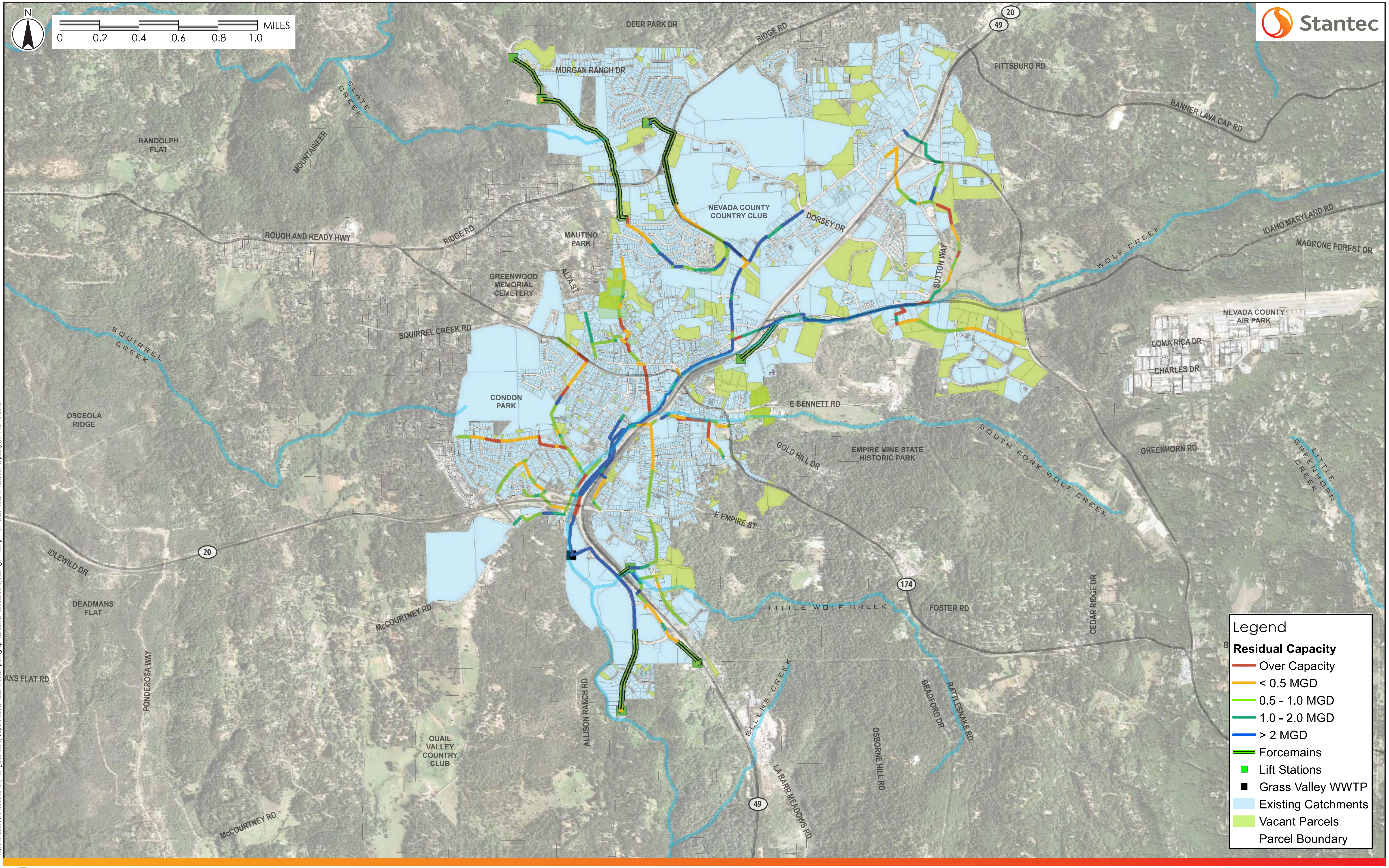

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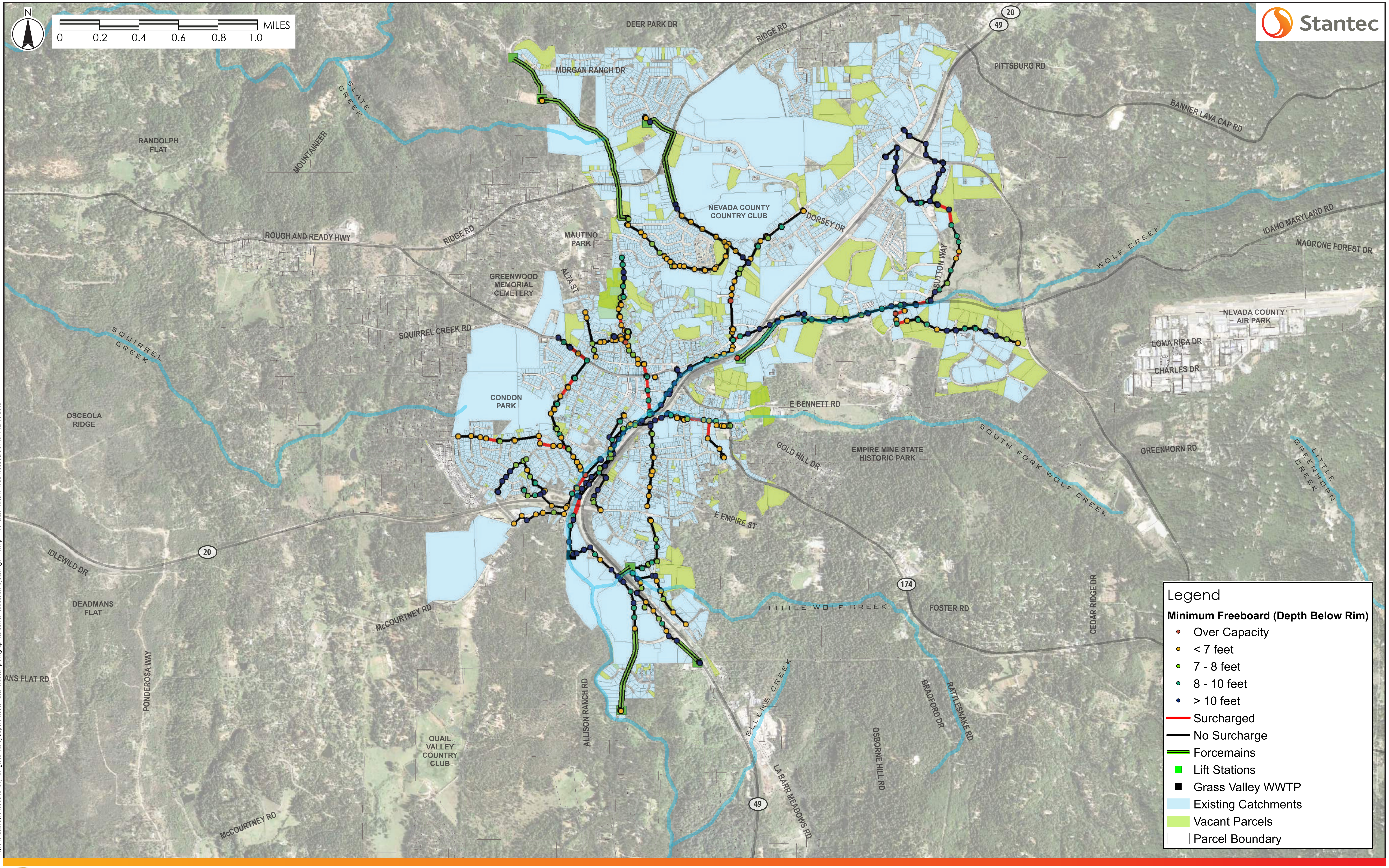
Figure 4-17
Existing + Vacant Residual Capacity



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City of Grass Valley
Wastewater System Master Plan

Figure 4-18
Existing + Vacant HGL Freeboard

4.4.6 Model Results – Existing Service Area Build-out plus Near-Term Level of Development

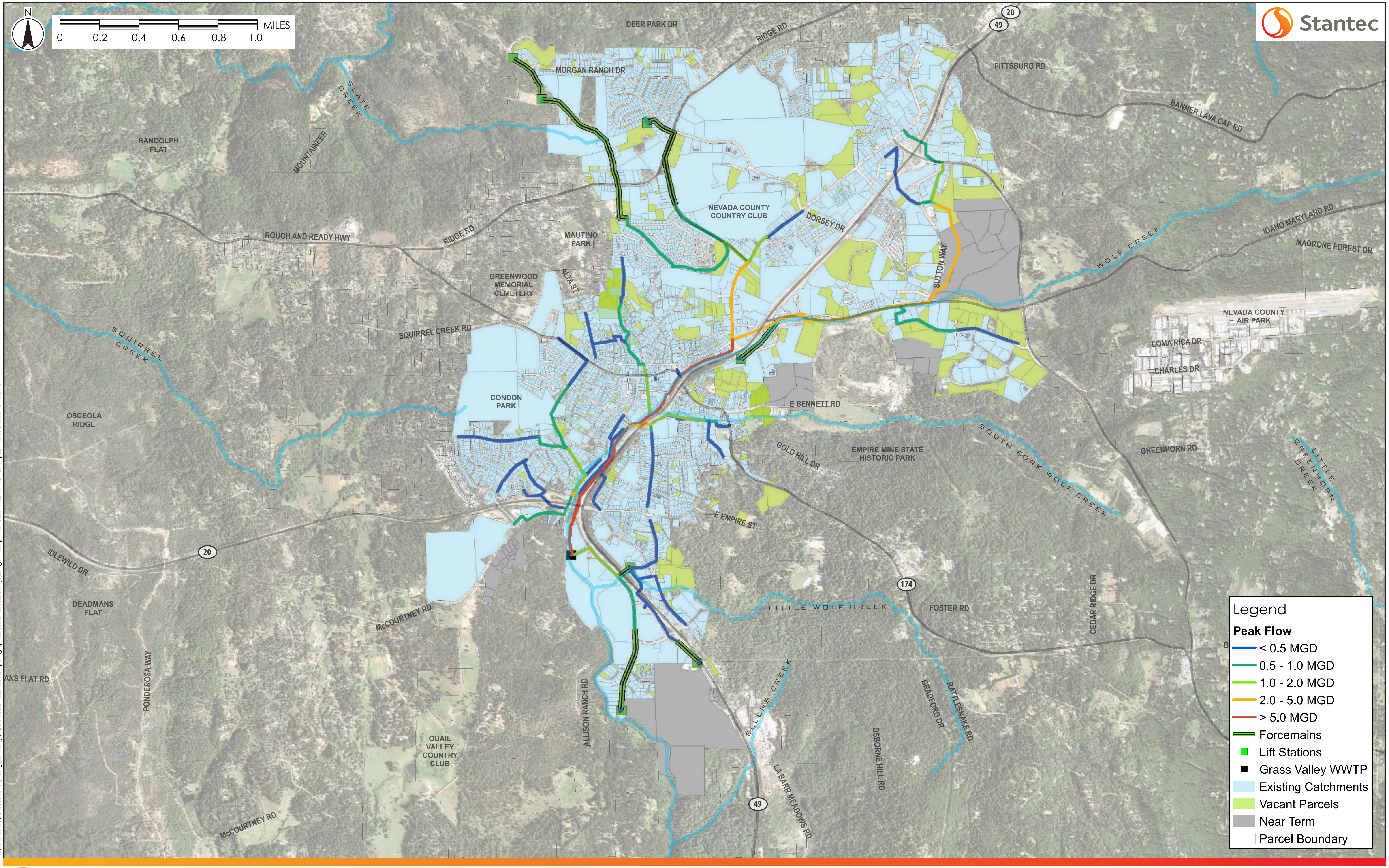
The peak modeled sewer flows for the 1:10 year, 24-hour Huff design event predicted under Existing Service Area Build-out conditions, plus Near-Term growth (as described in Chapter 3) are shown in **Figure 4-19**. This model scenario and corresponding results assume no upgrades to the existing collection system. **Table 4-11** shows the summary of flows for each of the primary sewer shed nodes modeled using the 1:10 year Huff rainfall event.

Table 4-11 Flow Characteristics of the Existing System under Existing Service Area Build-out + Near-Term Conditions

	WWTP	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Catchment Area (acre)	2,964	326	2,628	470	2,158	188	142	855	830
Average DWF (Mgal/d)	2.0	0.2	1.7	0.2	1.5	0.2	0.2	0.4	0.6
Peak DWF (Mgal/d)	3.4	0.6	2.8	0.3	2.5	0.4	0.3	0.8	1.0
Peak WWF 10yr Huff (Mgal/d)	16.5	1.5	14.9	2.5	12.5	2.7	1.4	3.0	4.3
Peak Flow (RDI only) (Mgal/d)	14.7	1.4	13.3	2.2	11.1	2.4	1.2	2.8	3.8
Peak RDII rate (gpd/acre)	4,959	4,320	5,076	4,784	5,155	13,023	8,337	3,223	4,608

4.4.6.1 Collection System Capacity to Accommodate Existing Service Area Build-out + Near-Term Flows

Figure 4-20 shows the hydraulic loading ratio of the existing system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near-Term growth. **Figure 4-21** shows the residual capacity in the existing sewer for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near-Term growth. **Figure 4-22** shows the minimum freeboard in the existing sewer for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near-Term growth.



City of Grass Valley
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Figure 4-19
Existing + Vacant + Near Term Peak Sewer Flows

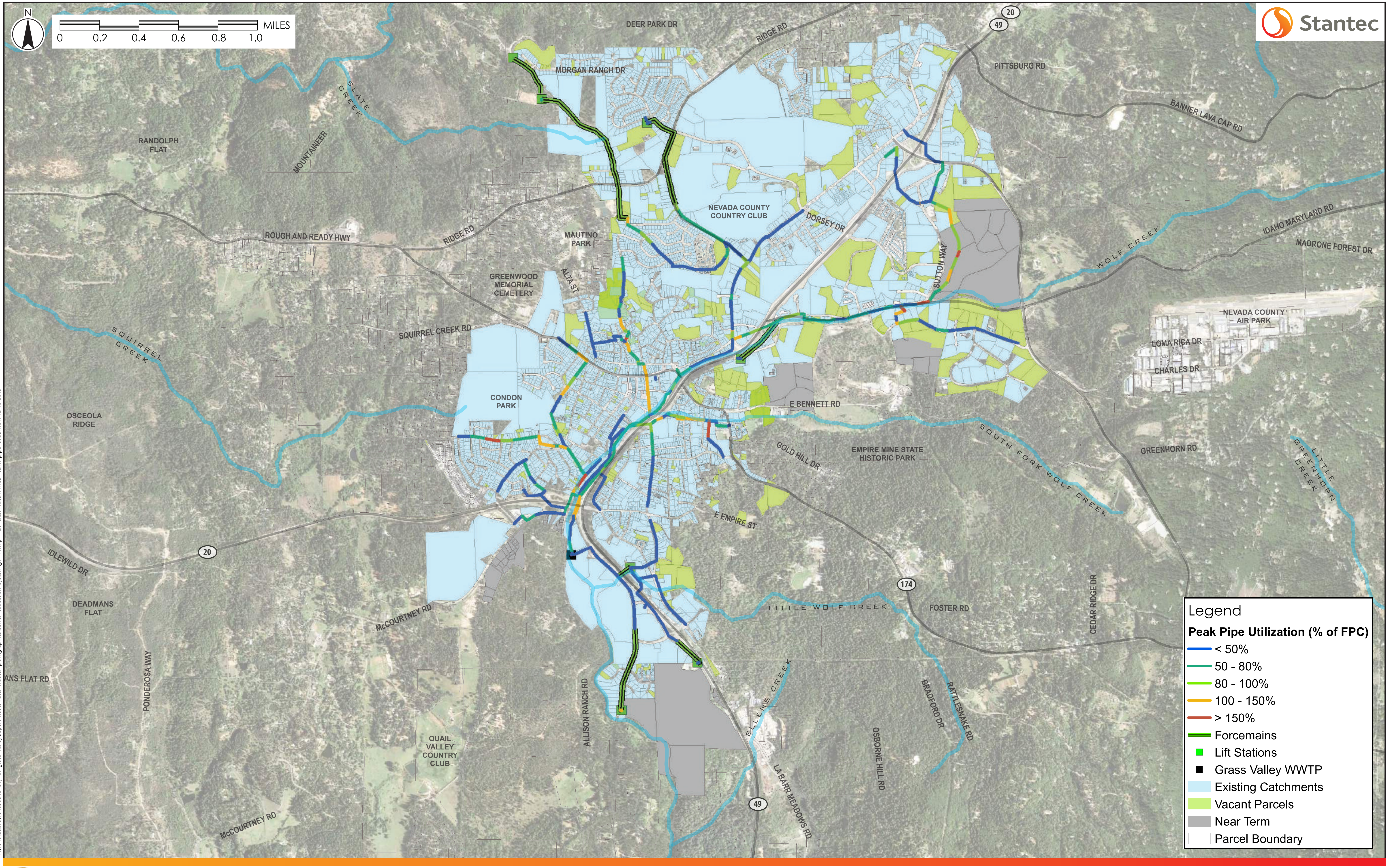

Figure 4-20
Existing + Vacant + Near Term Peak Flow Capacity Utilization

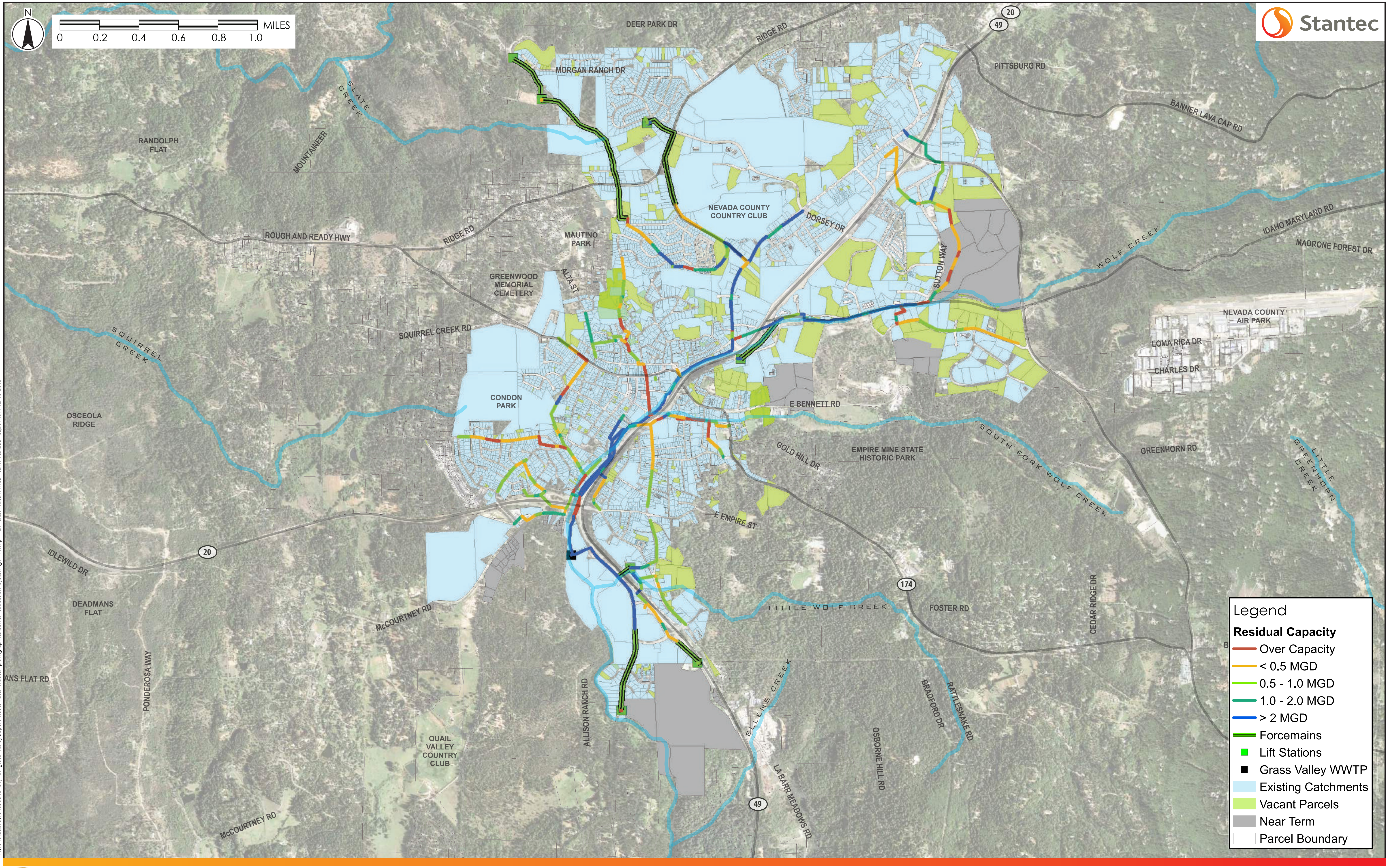

Figure 4-21
Existing + Vacant + Near Term Residual Capacity

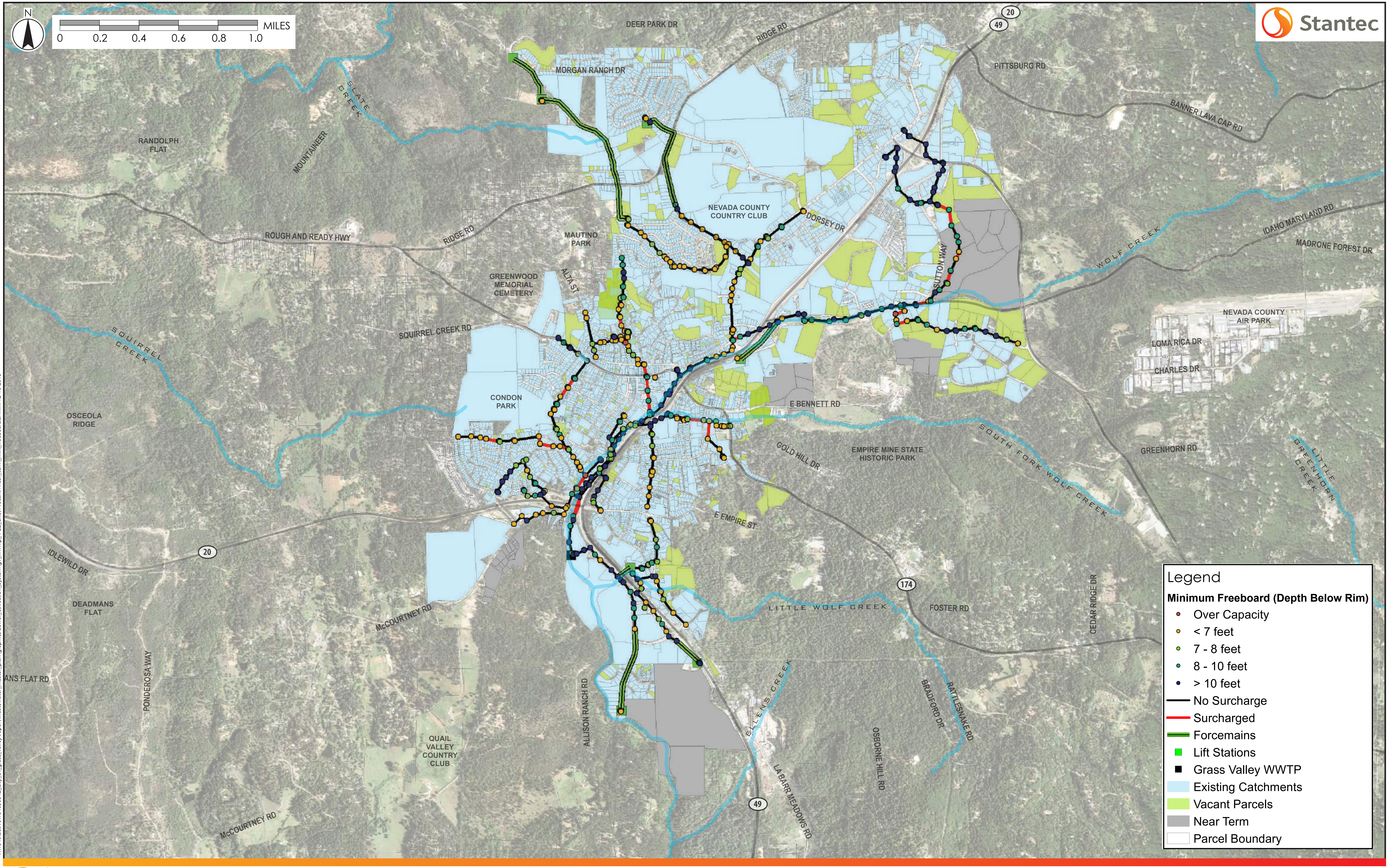

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Figure 4-22
Existing + Vacant + Near Term Minimum Freeboard

Under Existing Service Area Build-out plus Near-Term growth conditions, the 1:10 year Huff design storm is predicted to generate a peak flow of 16.5 Mgal/d at the WWTP. This storm event is predicted to cause surcharging in several reaches throughout the network. As with previously described scenarios, to help identify the extent of surcharging within the existing network, HGL profiles have been included within **Appendix E**, which show the peak surcharge elevation along the identified reach. Note that the profiles also include the results for other growth scenarios. A plan view of eight identified HGL profiles is shown in **Figure 4-8**.

The following provides a summary of the existing system surcharging and corresponding HGL profiles presented in **Appendix E**, under Existing Service Area Build-out plus Near-Term growth conditions:

- **HGL Profile 1**(**Figure E-1**): This scenario is predicting that surcharging will now occur upstream (S9-6, S9-5) but will not result in LOS failures at those locations. The surcharging in manhole S12-1 and S12-2 is predicted to result in a freeboard of approximately 6.5 feet, failing the LOS criteria. Manhole S12-1 is situated on Idaho Maryland Road and Railroad Avenue and Manhole S12-2 is near the intersection of Sutton Way and Idaho Maryland Rd.
- **HGL Profile 2** (**Figure E-2**): There is predicted to be no additional surcharging in this reach. However, the SSOs and surcharging predicted previously under Existing conditions remains.
- **HGL Profile 3** (**Figure E-3**): In the Existing Service Area plus Build-out scenario, it was identified that manhole M15-8 on Colfax Avenue and Henderson Street may experience surcharging. Due to the additional flow from the growth area south of Idaho-Maryland Road, it is predicted that the surcharging in the manhole M15-8 will further increase to the point that the freeboard drops below 4-feet, failing the LOS criteria.
- **HGL Profile 4** (**Figure E-4**): There is no additional surcharging predicted to occur in this reach. However, the surcharging predicted with the Existing Service Area plus Build-out scenario continues to exist/worsen with the addition of Near-Term growth areas.
- **HGL Profile 5** (**Figure E-5**): Minor surcharging in one manhole I17-7 (66 feet South of French Avenue) is a result of insufficient capacity in the twin 18-inch sewers crossing underneath Highway 20. The predicted freeboard has fallen to 18-feet, but is still much greater than the recommended 8-feet, and meets the LOS criteria. It should be noted that this information is based upon a degree of upstream throttling due to capacity constraints, and this surcharging will worsen as those capacity constraints are eliminated.
- **HGL Profile 6** (**Figure E-6**): The surcharging in this reach has significantly increased, and is now affecting the manholes R12-16 through R12-11, except manhole R12-13. There is predicted to be an SSO in manhole R12-12. These manholes do not meet the recommended LOS criteria.

- **HGL Profile 7** (**Figure E-7**): There is predicted to be no additional surcharging in this reach. It should be noted that sewer conduit 787 has reverse slope which will cause minor surcharging at manhole L11-8 at all scenarios discussed in the report.

- **HGL Profile 8** (**Figure E-8**): There is no additional surcharging predicted to occur in this reach.

4.4.7 Model Results – Existing Service Area Build-out plus Both Near and Long-Term Level of Development

The peak modeled sewer flows for the 1:10 year, 24-hour Huff design event predicted under Existing Service Area Build-out conditions, plus both Near and Long-Term growth (as described previously) are shown in **Figure 4-23**. This scenario and corresponding results assume no upgrades to the existing collection system.

Table 4-12 shows the summary of flows for each of the primary sewer shed nodes modeled for the 1:10 year Huff rainfall event.

Table 4-12 Flow Characteristics of the Existing System under Existing Service Area Build-out plus Both Near and Long-Term Conditions

	WWTP	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Catchment Area (acre)	3,543	326	3,217	512	2,705	188	142	855	1,377
Average DWF (Mgal/d)	2.2	0.2	2.0	0.2	1.8	0.2	0.2	0.4	0.9
Peak DWF (Mgal/d)	3.8	0.6	3.3	0.3	3.0	0.4	0.3	0.8	1.5
Peak WWF 10yr Huff (Mgal/d)	17.0	1.5	15.5	1.9	12.9	2.7	1.4	3.0	4.8
Peak Flow (RDI only) (Mgal/d)	15.1	1.4	13.7	1.7	11.4	2.4	1.2	2.8	4.2
Peak RDII rate (gpd/acre)	4,265	4,263	4,269	3,379	4,204	13,023	8,337	3,223	3,027

4.4.7.1 Collection System Capacity to Accommodate Existing Service Area Build-out + both Near and Long-Term Flows

Figure 4-24 shows the hydraulic loading ratio of the existing system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near and Long-Term growth. **Figure 4-25** shows the residual capacity in the existing system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near and Long-Term growth. **Figure 4-26** shows the minimum freeboard in the existing sewer system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus Near-Term and Long-Term growth areas.

Under Existing Service Area Build-out plus Near-Term and Long-Term growth conditions, the 1:10 year Huff design storm is predicted to generate a peak flow of 17.0 Mgal/d at the WWTP. This storm event is predicted to cause surcharging in several reaches throughout the network. HGL profiles have been included within **Appendix E**, which show the peak surcharge elevation along the identified reach for this scenario. Note that the profiles also include the results for other growth scenarios. A plan view of eight identified HGL profiles is shown in **Figure 4-8**.

The following provides a summary of the existing system surcharging and corresponding HGL profiles presented in **Appendix E**, for this scenario:

- **HGL Profile 1** (**Figure E-1**): Severe surcharging is predicted at two manholes, S10-4 and S11-5 (upstream of manhole S12-1 and S12-2 identified previously). This surcharging occurs as a result of the additional development in the Loma Rica Special Development Area. Note that although the downstream portion of the profile indicates no surcharging, the elimination of the upstream capacity restrictions would increase the flow downstream, resulting in similar surcharge concerns.

- **HGL Profile 2** (**Figure E-2**): No additional flow is predicted through this reach; however, the SSOs and surcharging predicted previously remain.

- **HGL Profile 3** (**Figure E-3**): No additional flow is predicted through this reach; however, the surcharging predicted previously remains.

- **HGL Profile 4** (**Figure E-4**): No additional surcharging is predicted to occur in this reach; however, the surcharging predicted previously remains.

- **HGL Profile 5** (**Figure E-5**): No additional surcharging is predicted in this reach; however, the surcharging predicted previously remains.

- **HGL Profile 6** (**Figure E-6**): No additional flow is predicted through this reach. There is still predicted to be an SSO in manhole R12-12.

- **HGL Profile 7** (**Figure E-7**): No additional surcharging is predicted in this reach.

- **HGL Profile 8** (**Figure E-8**): There is no additional surcharging predicted to occur in this reach.

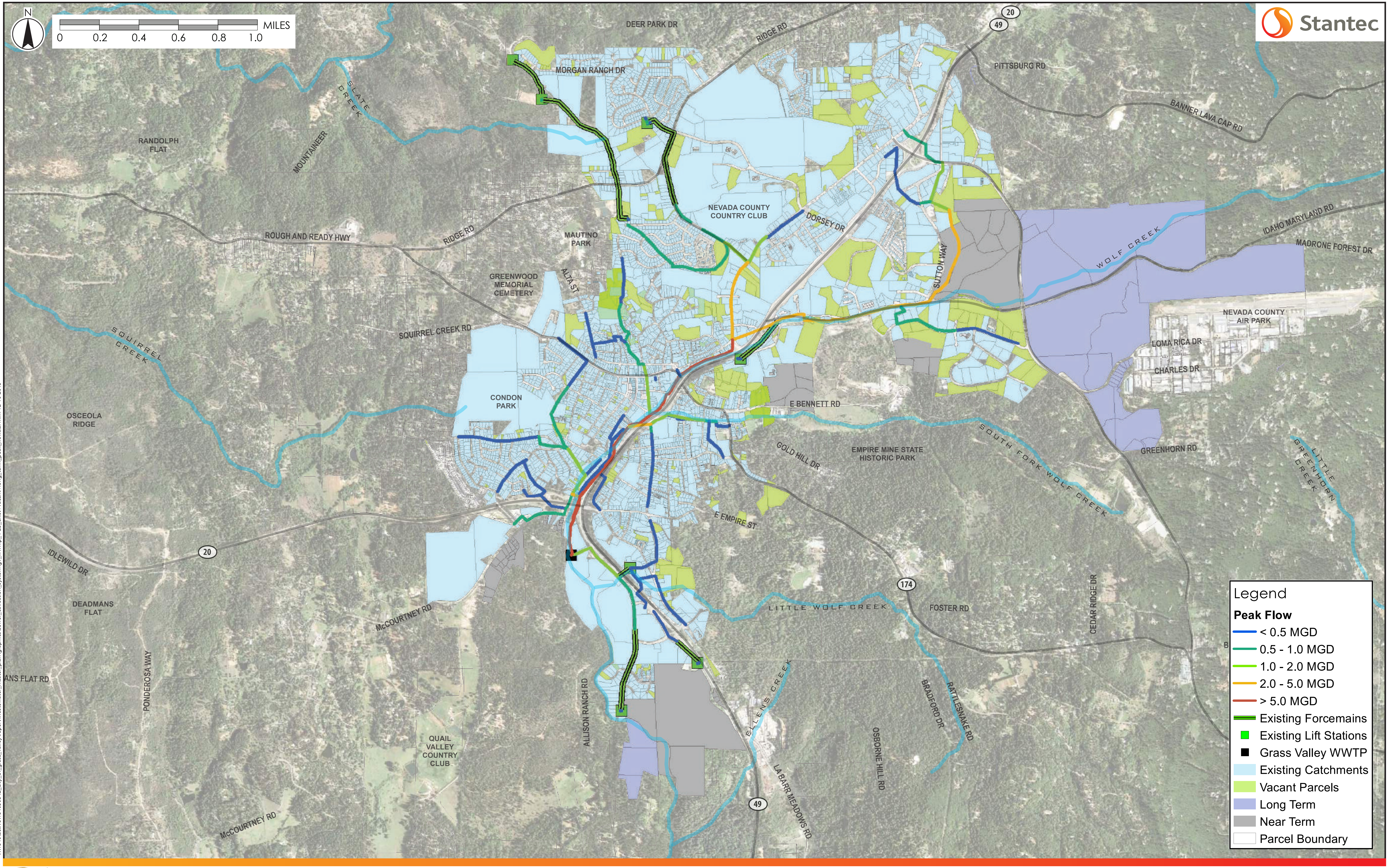

Figure 4-23
Existing + Vacant + Long Term Peak Sewer Flows

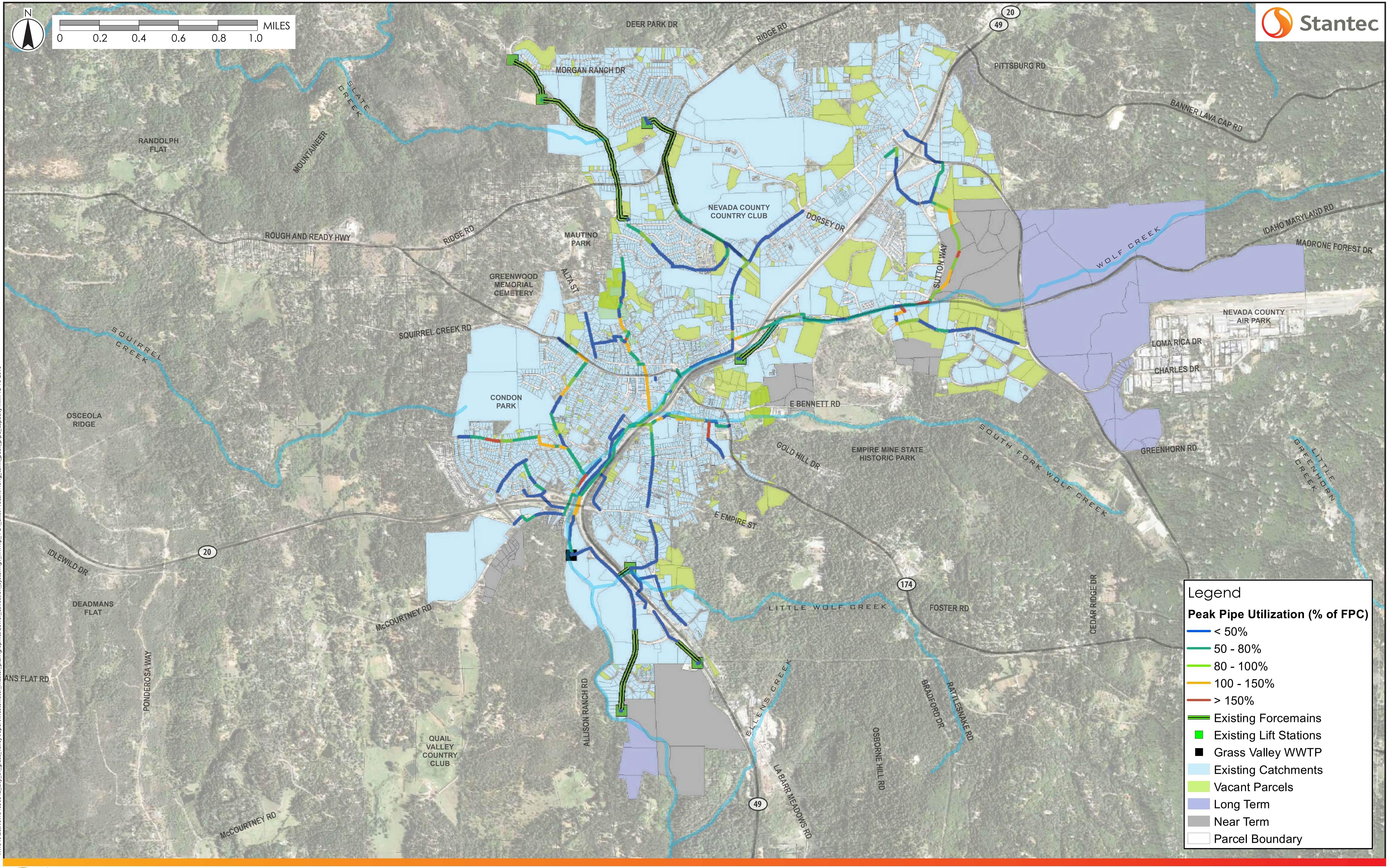

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Figure 4-24
Existing + Vacant + Long Term Peak Flow Capacity Utilization

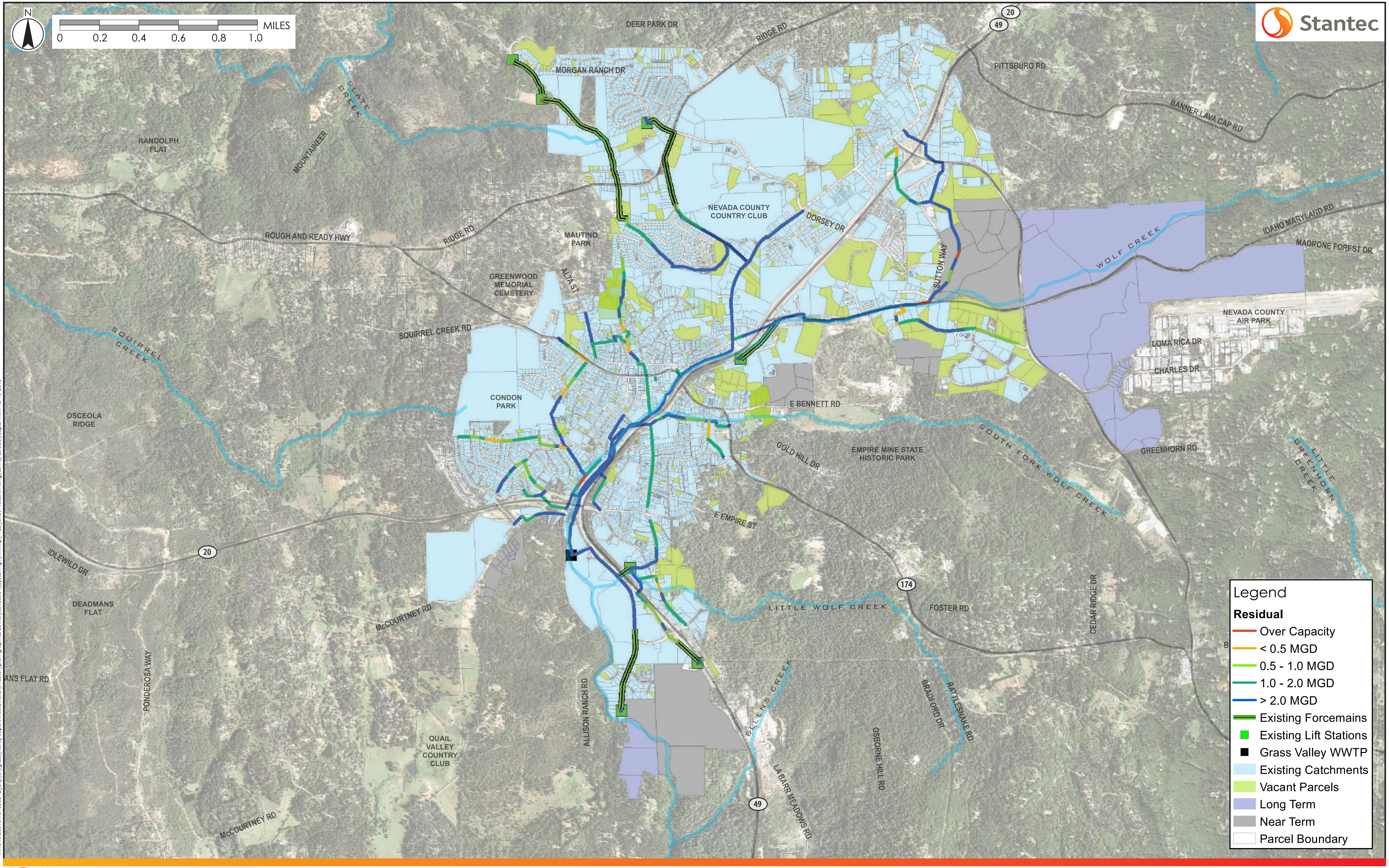

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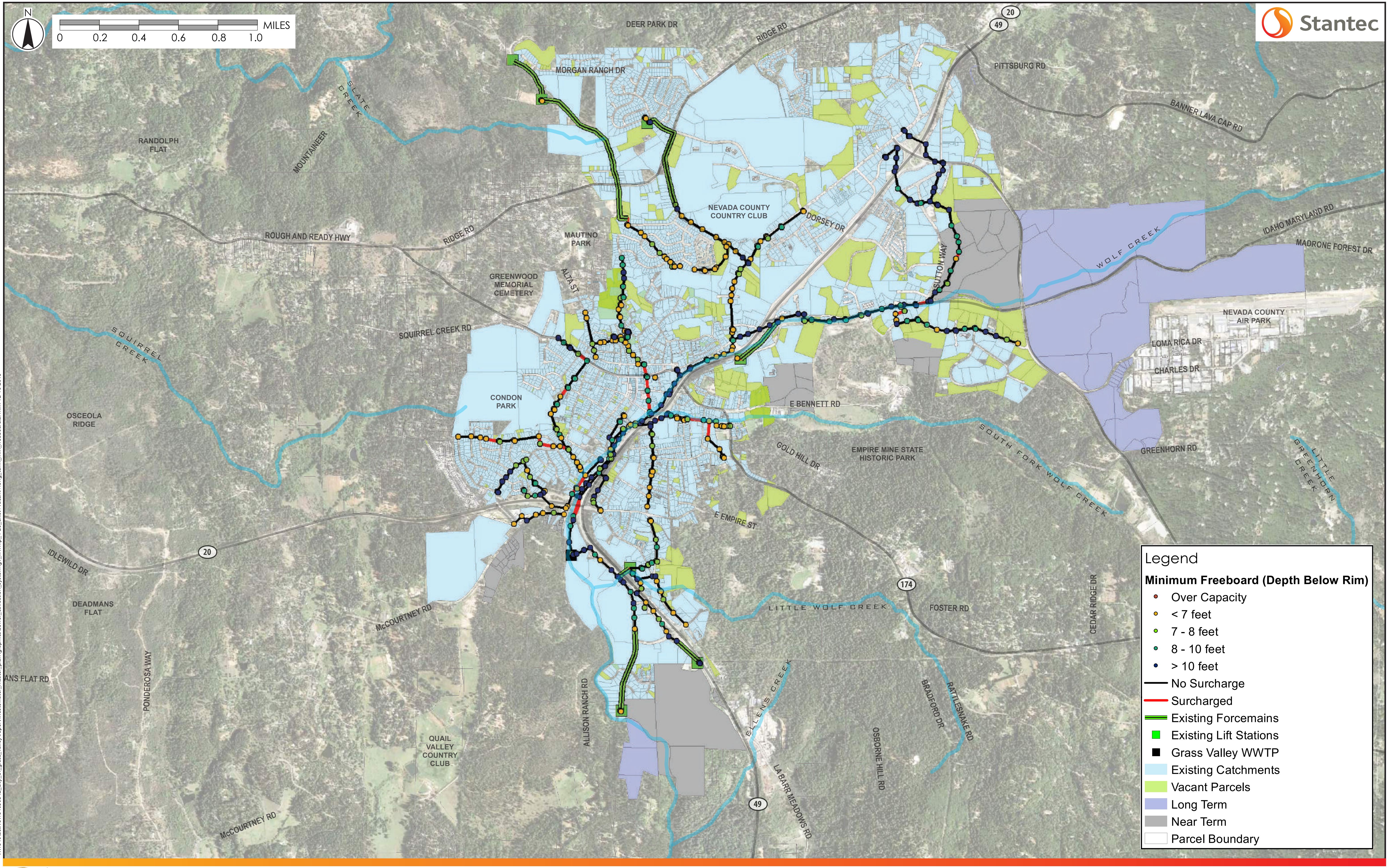
Figure 4-25
Existing + Vacant + Long Term Residual Capacity



V:\1840\active\184030342_city_of_grass_valley\report\wastewater_master_plan\graphics\sewer_collection_system\gv_wwmp_4-26_exist+vacant+long_term_min_freeboard.ai mlm 12-16-2015

Figure 4-26
Existing + Vacant + Long Term Minimum Freeboard

4.4.8 Model Results – Full Build-out of the Grass Valley Sewer System

The peak modeled sewer flows for the 1:10 year, 24-hour Huff design event predicted under full Future Service Area Build-out conditions (as described in Chapter 3) are shown in **Figure 4-27**. **Table 4-13** shows the summary of flows for each of the flow monitor locations modeled for the 1:10 year Huff rainfall event.

Table 4-13 Flow Characteristics of the Existing System under Full Build-out Conditions

	WWTP	FM#1	FM#2	FM#3	FM#4	FM#5	FM#6	FM#7	FM#8
Catchment Area (acre)	8,552	799	6,414	1,537	4,877	965	244	1,516	2,010
Average DWF (Mgal/d)	3.9	0.4	3.1	0.5	2.6	0.5	0.2	0.6	1.2
Peak DWF (Mgal/d)	7.5	1.4	5.3	0.8	4.4	0.9	0.3	1.1	2.1
Peak WWF 10yr Huff (Mgal/d)	25.6	3.7	16.9	3.1	13.7	2.9	1.4	3.6	4.9
Peak Flow (RDI only) (Mgal/d)	22.4	3.3	14.3	2.7	11.7	2.4	1.2	3.2	4.0
Peak RDII rate (gpd/acre)	2,620	4,183	2,224	1,763	2,389	2,497	4,738	2,090	1,985

4.4.8.1 Collection System Capacity to Accommodate Full Build-out of Service Area

Figure 4-28 shows the hydraulic loading ratio of the existing system for the 1:10 year Huff design rainfall under a development scenario that includes build-out of the City's entire potential future service area (as defined previously). **Figure 4-29** shows the residual capacity in the existing sewer system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus the City's entire potential future service area as described in Chapter 3. **Figure 4-30** shows the minimum freeboard in the existing sewer system for the 1:10 year Huff design rainfall under Existing Service Area Build-out plus the City's entire potential future service area.

Under Future Service Area Build-out conditions, the 1:10 year Huff design storm is predicted to generate a peak flow of 25.6 Mgal/d at the WWTP. Note that the elimination of the upstream capacity restrictions identified in the existing collection system would increase the flow downstream. With the recommended improvements in Section 4.5, the modeled peak flow could be increased up to 39.4 Mgal/d assuming nothing is done to reduce I/I due to storm flows. This storm event is predicted to cause surcharging in several reaches throughout the network. HGL profiles have been included within **Appendix E**, which show the peak surcharge elevation along the identified reach. A plan view of the eight identified HGL profiles discussed below is shown in **Figure 4-8**.

The following provides a summary of the existing system surcharging and corresponding HGL profiles, under Future Service Area Build-out conditions:

- **HGL Profile 1** (**Figure E-1**): This reach starts at manhole S9-4 on Plaza Drive to the south paralleling Sutton Way and further toward southwest along Idaho Maryland Road. Severe surcharging is shown in the upstream of the profile. Minor surcharging in Manhole N13-10 is predicted to occur downstream of the reach but will not result in LOS failures. It is important to note that this capacity constraint is throttling the flow, resulting in less flow downstream. Should the constraint be eliminated, the downstream portion of this profile will not have enough capacity to convey the full flow.

- **HGL Profile 2** (**Figure E-2**): There is predicted to be no additional flow through this reach. However, the surcharging predicted previously remains.

- **HGL Profile 3** (**Figure E-3**): The additional flow from the growth area south of Idaho-Maryland Road is predicted to cause major capacity concerns along Colfax Avenue through this trunk, resulting in multiple deficiencies along this reach.

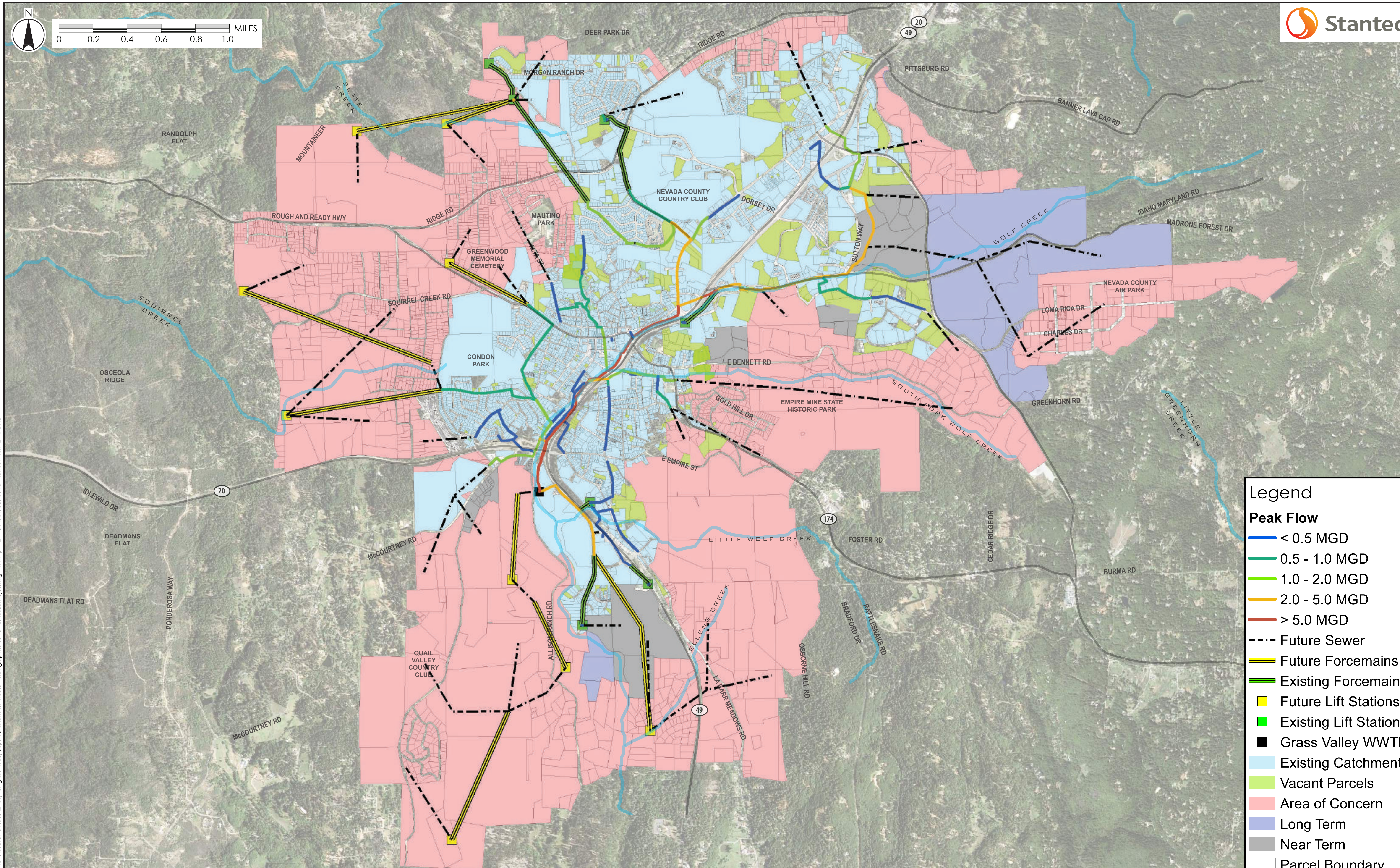
- **HGL Profile 4** (**Figure E-4**): The existing sewer trunk in this profile is not sized to accommodate the Full Build-out from future developments, resulting in many deficiencies throughout the reach.

- **HGL Profile 5** (**Figure E-5**): Surcharging in one manhole I17-7 (66 feet south of French Avenue) is a result of insufficient capacity in the twin 18-inch sewers crossing underneath Highway 20. The predicted freeboard has further fallen to 16-feet, but is still much greater than the recommended 8-feet. It should be noted that this information is based upon a degree of upstream throttling due to capacity constraints, and this surcharging will worsen as those capacity constraints are eliminated. The upgrades will be required then.

- **HGL Profile 6** (**Figure E-6**): The surcharging upstream of the manhole R12-12 has further increased with surcharging occurring in manholes R12-17 and R12-12. This surcharging is a result of insufficient capacity in the 6-inch sewer and results in five manholes that do not meet the recommended LOS criteria (R12-17, R12-16, R12-14, R12-12 and R12-11).

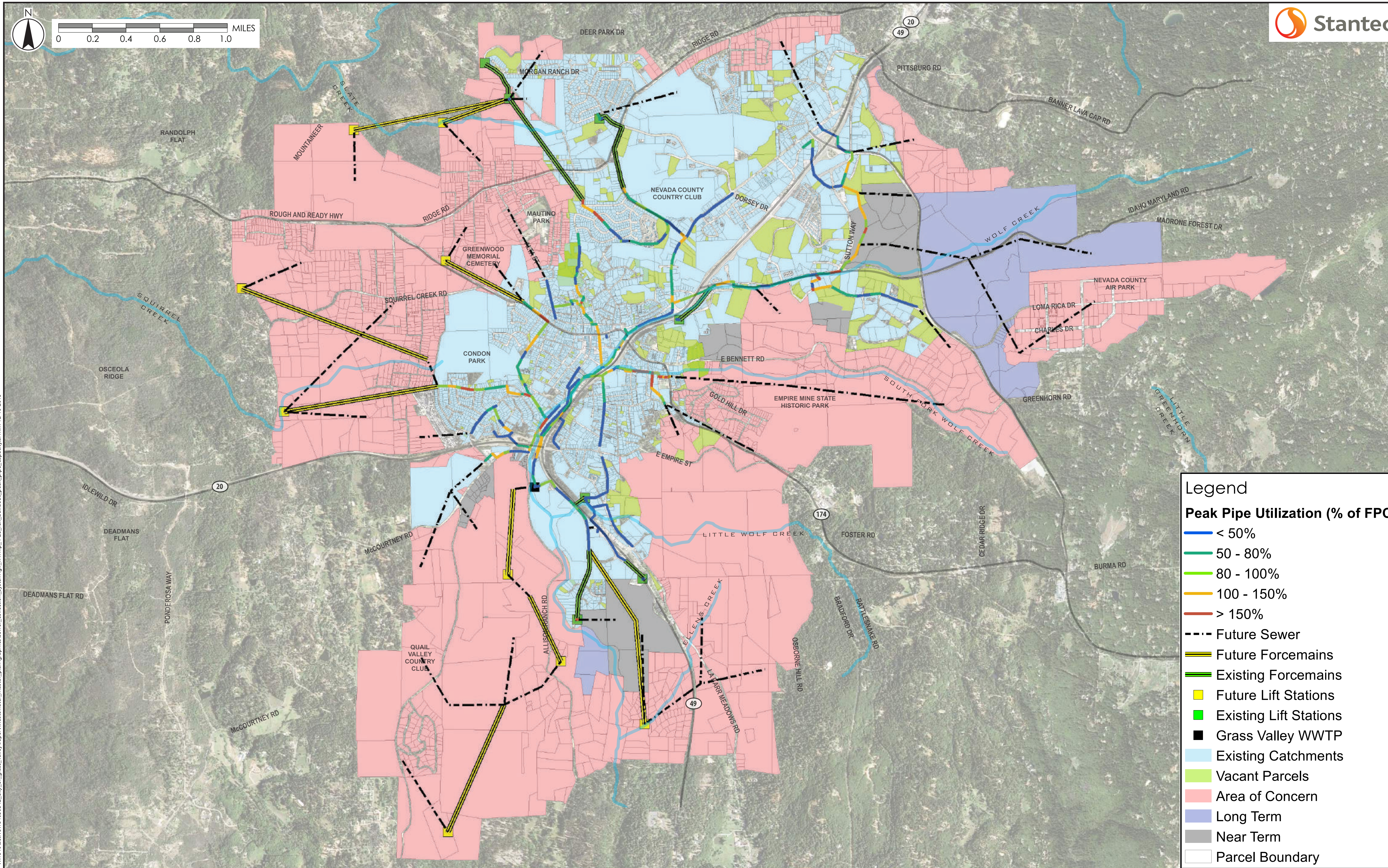
- **HGL Profile 7** (**Figure E-7**): Two manholes (K9-1 and K10-1) are now predicted to have deficiencies along this profile. These two manholes are situated approximately at the intersection of Lidster Avenue and Cypress Hill Drive. Between the two, there is predicted to be a combined max flood rate of nearly 2 Mgal/d. Additional surcharging at two manholes (N11-1 and N11-4) is a result of insufficient capacity of sewer conduit 68 and sewer conduit 71.

- **HGL Profile 8** (**Figure E-8**): the additional flow from the growth area North of CA-20 and Southeast of Squirrel Creek Rd predicted to cause major capacity concerns, resulting in several deficiencies along the reach. It should be noted that in order to accommodate flow from future areas with proper slope, the invert elevation of manhole F15-4 at the Full Build-out is assumed to be 9 feet less than the existing invert elevation. Upon future development, the slope should further be investigated.



City of Grass Valley
Wastewater System Master Plan

Figure 4-27
Full Build-out Peak Sewer Flows



City of Grass Valley
Wastewater System Master Plan

Figure 4-28
Full Build-out Peak Flow Capacity Utilization

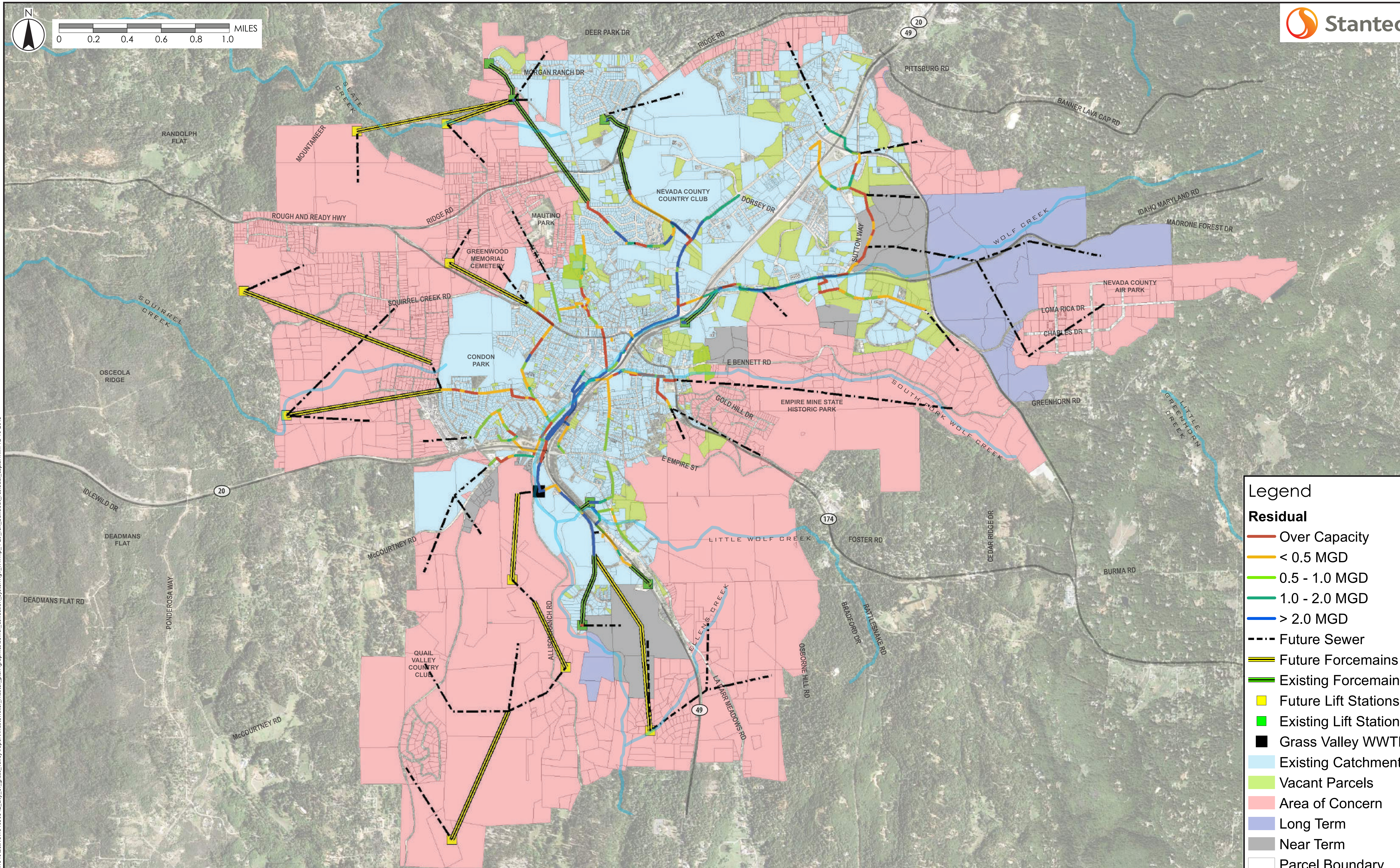


Figure 4-29
Full Build-out Residual Capacity

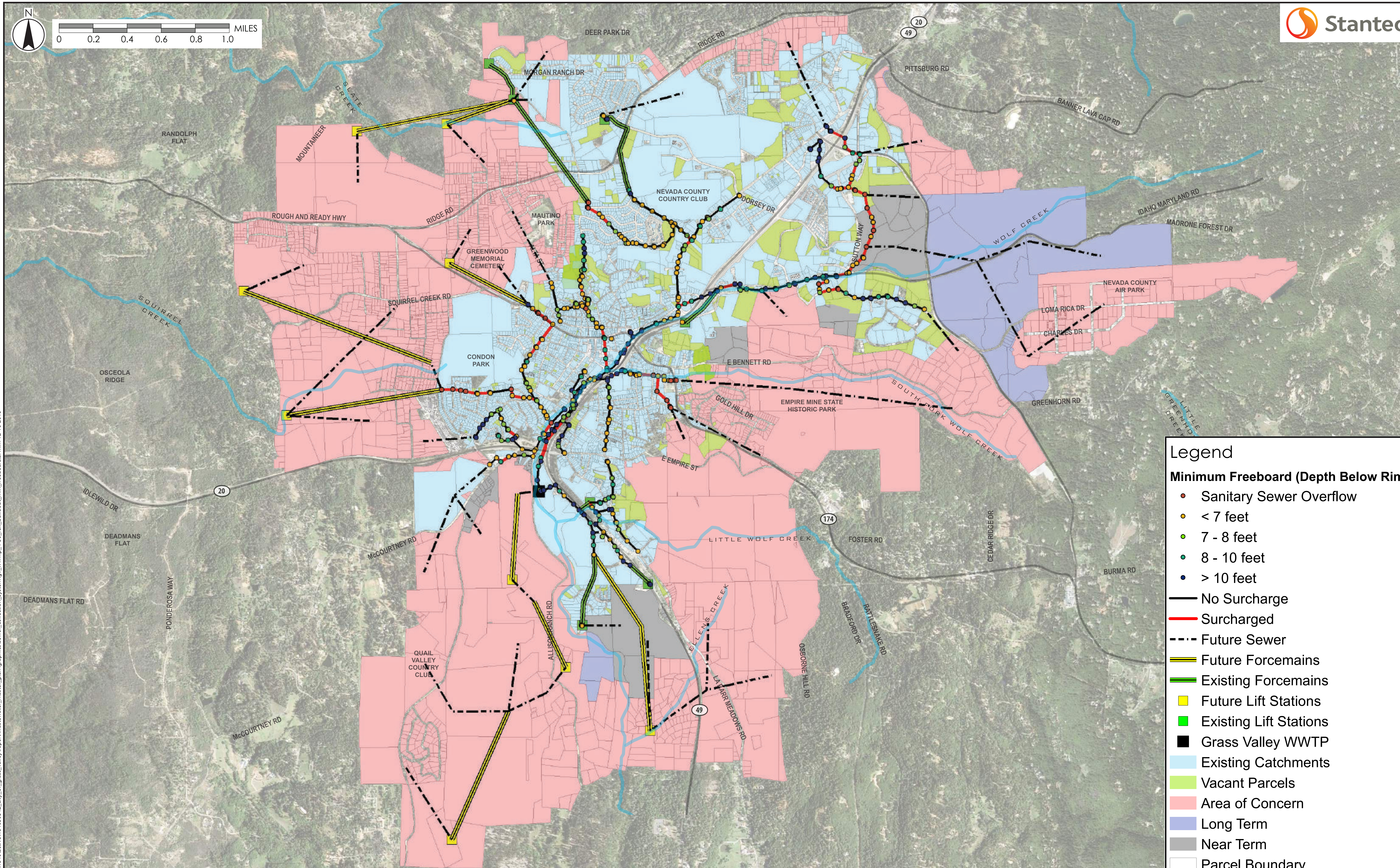


Figure 4-30
Full Build-out Minimum Freeboard

4.4.9 Lift Station and Forcemain Capacity to Accommodate Future Flows

This section discusses apparent capacity limitations of the existing City lift stations and correlates existing capacity to modeled future capacity demands on each lift station. Through analysis of the model results and understanding of the lift stations' firm capacities, flow based trigger points for requiring upgrades can be identified.

4.4.9.1 Carriage House Sewer Lift Station

It is projected that the Carriage House sewer lift station will provide wastewater pumping service for up to 110 acres of land at Full Build-out conditions. **Table 4-14** below provides a summary of the firm pumping capacity and full pump capacity of the lift station, as well as the projected flows for each growth scenario.

Table 4-14 Carriage House Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.23
Full Pumping Capacity (Mgal/d)	0.46
Peak WWF Influent – Existing Scenario (Mgal/d)	0.09
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.10
Peak WWF Influent – Existing Built-out + Near-Term (Mgal/d)	0.20
Peak WWF Influent – Existing Built-out + Near-Term + Long-Term (Mgal/d)	0.20
Peak WWF Influent – Full Service Area Build-out (Mgal/d)	0.32

As currently planned, the Carriage House sewer lift station should have sufficient peak pumping capacity to accommodate the Full Build-out of the service area. However, upgrades for additional firm pumping capacity should be considered to serve Full Build-out.

4.4.9.2 Joyce Drive Sewer Lift Station

It is projected that the Joyce Drive sewer lift station will provide wastewater pumping service for up to 119 acres of land at Build-out conditions. **Table 4-15** below provides a summary of the firm pumping capacity and full pump capacity of the lift station, as well as the projected flows for each growth scenario. Note that in this Master Plan flow from the area served by this lift station is previously planned to be partially diverted (downstream of the Taylorville Pump Station) due to capacity concerns at this lift station. The current results of the model indicate that the lift station should have sufficient firm pumping capacity at the Existing Build-out plus Near and Long-Term scenario and adequate full pumping capacity at Future Build-out. Thus, the recommended diversion might not be required. Should the City wish to pursue this diversion upgrade, the modeling result for the Future Build-out scenario reflects this proposed flow diversion with a new gravity sewer to divert flow at MH K20-3 to the gravity sewer in Freeman Lane & Taylorville Road at MH K20-9.

Table 4-15 Joyce Drive Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.90
Full Pumping Capacity (Mgal/d)	1.50
Peak WWF Influent – Existing Scenario (Mgal/d)	0.36
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.45
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.46
Peak WWF Influent – Existing Build-out + Near-Term + Long-Term (Mgal/d)	0.46
Peak WWF Influent – Full Service Area Build-out (Mgal/d)	1.09

4.4.9.3 Morgan Ranch Sewer Lift Station

It is projected that the Morgan Ranch sewer lift station will provide wastewater pumping service for up to 214 acres of land at Full Build-out conditions. **Table 4-16** below provides a summary of the firm pumping capacity and full pump capacity of the lift station, as well as the projected flows for each growth scenario.

Table 4-16 Morgan Ranch Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.50
Full Pumping Capacity (Mgal/d)	0.66
Peak WWF Influent – Existing Scenario (Mgal/d)	0.61
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.67
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.67
Peak WWF Influent – Existing Build-out +Near-Term + Long-Term (Mgal/d)	0.67
Peak WWF Influent – Full Service Area Build-out (Mgal/d)	0.75

Based upon the projected flows, the Morgan Ranch sewer lift station is currently near full pumping capacity. Therefore it is projected that the trigger point for supplemental pumping capacity will be at the time the growth anticipated by the Existing Build-out scenario is in place.

4.4.9.4 Railroad Avenue Sewer Lift Station

It is projected that the Railroad Avenue sewer lift station will provide wastewater pumping service for up to 3 acres of land at Build-out conditions. **Table 4-17** below provides a summary of the firm pumping capacity and full pumping capacity of the lift station, as well as the projected flows for each growth scenario.

Table 4-17 Railroad Avenue Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.21
Full Pumping Capacity (Mgal/d)	0.23
Peak WWF Influent – Existing Scenario (Mgal/d)	0.02
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.02
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.02
Peak WWF Influent – Existing Build-out +Near-Term + Long-Term (Mgal/d)	0.02
Peak WWF Influent – Full Service Area Build-out (Mgal/d)	0.02

Based upon the projected flows, the Railroad Avenue sewer lift station has sufficient firm capacity for the Full Build-out scenario.

4.4.9.5 Slate Creek Sewer Lift Station

It is projected that the Slate Creek sewer lift station will provide wastewater pumping service for up to 801 acres of land at Full Build-out conditions. **Table 4-18** below provides a summary of the firm pumping capacity and full pump capacity of the lift station, as well as the projected flows for each growth scenario.

Table 4-18 Slate Creek Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.43
Full Pumping Capacity (Mgal/d)	0.58
Peak WWF Influent – Existing Scenario (Mgal/d)	0.49
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.55
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.55
Peak WWF Influent – Existing Build-out + Near-Term + Long-Term (Mgal/d)	0.55
Peak WWF Influent – Full Service Area Build-out (Mgal/d)	3.05

The Slate Creek sewer lift station is currently projected to exceed the firm pumping capacity. At the time growth anticipated by the Existing Build-out scenario occurs, the lift station will be close to its full pumping capacity. Should the City of Grass Valley choose to continue with the philosophy of making Slate Creek the regional pump station to serve Future Build-out (including Kenny Ranch) in this portion of the collection system, the entire pump station and forcemain will need to be replaced or supplemented with a station of much larger capacity.

4.4.9.6 Taylorville Sewer Lift Station

It is projected that the Taylorville Creek sewer lift station will provide wastewater pumping service for up to 5 acres of land at Full Build-out conditions, further discussed in Section 4.2.10. **Table 4-19** below provides a summary of the firm pumping capacity and full pumping capacity of the lift station, as well as the projected flows for each growth scenario.

Table 4-19 Taylorville Lift Station Capacity to Accommodate Growth Scenarios

Firm Pumping Capacity (Mgal/d)	0.16
Full Pumping Capacity (Mgal/d)	0.18
Peak WWF Influent – Existing Scenario (Mgal/d)	0.01
Peak WWF Influent – Existing + Vacant (Build-out) (Mgal/d)	0.02
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.02
Peak WWF Influent – Existing Build-out + Long-Term (Mgal/d)	0.02
Peak WWF – Full Service Area Build-out (Mgal/d)	0.02

The Taylorville sewer lift station is predicted to have sufficient firm capacity to accommodate all future flows.

4.4.10 New Regional Pump Station and Possible Flow Diversions

The scenarios applied to the wastewater collection system model assume specific future improvements which include a new regional lift station within the proposed Berriman Ranch development Area and a flow diversion from the gravity line downstream of the Taylorville lift station at MH K20-3 to the gravity sewer in Freeman Lane & Taylorville Rd at MH K20-9. This new gravity diversion would be 8 inch in diameter. As discussed in Section 4.4.9, the Joyce Drive lift station is predicted to have sufficient full pumping capacity to accommodate the Full Build-out of the service area. The proposed flow diversion might not be needed; however, should the City wish to pursue this upgrade, the modeling results at the Full Build-out scenario reflect this proposed flow diversion.

The regional lift station is assumed to be located in the proximity of Brookside Way. In the analysis of the Existing Service Area Build-out plus Near-Term growth scenario, the maximum projected wet weather flow modeled with the 1:10 year Huff design rainfall from the future development area (79.9 acre) in Berriman Ranch & Adjacent Property could be up to 0.319 Mgal/d, which will exceed the full pumping capacity (0.18 Mgal/d) of the Taylorville sewer lift station, discussed in Section 4.4.9. Therefore, the trigger for construction of this proposed regional lift station would be at the time Near-Term development, as described here in, is to take place. **Table 4-20** provides a summary of the projected DWF and WWF at the Regional lift station to accommodate the growth scenarios. These assumed pump station improvements are important to note as the model results and subsequent recommendations made herein are highly dependent upon their being in place.

Table 4-20 Projected DWF and WWF at Proposed Regional Lift Station

Average DWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.06
Average DWF Influent – Existing Build-out + Long-Term (Mgal/d)	0.06
Average DWF Influent – Full Service Area Build-out (Mgal/d)	0.22
Peak DWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.12
Peak DWF Influent – Existing Build-out + Long-Term (Mgal/d)	0.12
Peak DWF Influent – Full Service Area Build-out (Mgal/d)	0.40
Peak WWF Influent – Existing Build-out + Near-Term (Mgal/d)	0.35
Peak WWF Influent – Existing Build-out + Long-Term (Mgal/d)	0.35
Peak WWF – Full Service Area Build-out (Mgal/d)	2.09

4.5 RECOMMENDATIONS

4.5.1 Purpose

The purpose of this section is to provide recommendations for capital improvements to the City of Grass Valley wastewater collection system, to provide sufficient capacity to accommodate peak wet weather flows that were predicted during a 10-year, 24-hour design rainfall event.

4.5.2 Improvements Identified

All the recommended pipe improvements are based upon the LOS criteria discussed in Section 4.4.2. As discussed in Section 4.4.4, the predicted locations of system surcharging are identified through eight HGL profiles– the peak surcharge elevations along identified impacted reaches are shown in **Figure E-1** to **Figure E-8** of **Appendix E**. A plan view of these profiles is shown in **Figure 4-8**. The HGL profiles with proposed pipe sizes (upgraded condition) are shown in **Figure E-9** to **Figure E-48** in **Appendix E** in comparison to the former HGL profiles which assume upgrades are not in place. A summary of the recommended pipe improvements are illustrated as follows. **Appendix F** provides tabulated recommended pipe improvements for each scenario in detail.

4.5.2.1 HGL Profile 1 (Figure E-9, E-17, E-25, E-33, E-41)

- Existing Level of development: Approximately 208 feet of 10 inch sewer is recommended to be upgraded to 15 inch sewer line upstream of Idaho Maryland Road (**Figure E-9**).
- Existing Service Area Build-out: There is no additional upgrade required, beyond that described for the Existing level of development (**Figure E-17**).

- Existing Build-out plus Near-Term: In addition, approximately 782 feet of 10 inch sewer is proposed to be upgraded to 12 inch sewer line and 220 feet of 10 inch sewer would need to be upgraded to 18 inches (**Figure E-25**).

- Existing Build-out plus both Near and Long-Term: Approximately 1,517 feet of 12 inch sewer, 1,972 feet of 15 inch sewer and 3,801 feet of 18 inch sewer would be required to replace the existing sewers to meet the LOS criteria (**Figure E-33**).

- Full Build-out: Approximately 1,718 feet of 15 inch sewer, 1,619 feet of 18 inch sewer, 1,664 feet of 21 inch sewer, and 3,933 feet of 24 inch sewer would be required (to replace the existing sewers) to accommodate future flow (**Figure E-31**).

4.5.2.2 HGL Profile 2 (Figure E-10, E-18, E-26, E-34 and E-42)

- Existing level of development: Approximately 151 feet of 6 inch sewer would need to be upgraded to 8 inches (**Figure E-10**).

- Existing Service Area Build-out: Additional 422 feet of 6 inch sewer would need to be upgraded to 8 inches (**Figure E-18**).

- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described for the Existing Service area Build-out scenario (**Figure E-26**).

- Existing Build-out plus both Near and Long-Term: There is no additional upgrade required beyond that required for the Existing Service Area Build-out scenario (**Figure E-34**).

- Full Build-out: Approximately 573 feet of 8 inch, 605 feet of 10 inch and 1,649 feet of 12 inch sewers would be required (to replace the existing sewers) to accommodate future flows (**Figure E-42**).

4.5.2.3 HGL Profile 3 (Figure E-11, E-19, E-27, E-35 and E-43)

- Existing level of development: Approximately 462 feet of 8 inch sewer would need to be upgraded to 10 inches.

- Existing Service Area Build-out: There is no additional upgrade required beyond that described for the Existing level of development.

- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described for the Existing level of development.

- Existing Build-out plus both Near and Long-Term: Additionally, 161 feet of 15 inch sewer would need to be upgraded to 18 inches.

- Full Build-out: Approximately 804 feet of 15 inch, 982 feet of 18 inch and 909 feet of 21 inch sewers would be required (to replace the existing sewers) to accommodate future flows.

4.5.2.4 HGL Profile 4 (Figure E-12, E-20, E-28, E-36 and E-44)

- Existing level of development: Approximately 300 feet of 6 inch sewer would need to be upgraded to 8 inches, 310 feet of 8 inch would need to be upgraded to 10 inch, and 615 feet of 8 inch would need to be upgraded to 12 inch sewers.

- Existing Service Area Build-out: Among the above 615 feet of 12 inches sewers needed in the Existing level of development, 410 feet are proposed to be upgraded further to 15 inches.

- Existing Build-out plus Near-Term: An additional 7 feet of 6 inch sewer is would need to be upgraded to 8 inches.

- Existing Build-out plus both Near and Long-Term: There is no additional upgrade required beyond that described for the Existing Build-out plus Near-Term growth scenario.

- Full Build-out: Approximately 1,169 feet of 8 inch sewer, 1,228 feet of 10 inch sewer, 849 feet of 12 inch and 782 feet of 21 inch sewer would be required (to replace the existing pipes) to accommodate future flows.

4.5.2.5 HGL Profile 5 (Figure E-13, E-21, E-29, E-37 and E-45)

- Existing level of development: It should be noted that there is no sewer conduit proposed to be upgraded along this reach. The HGL disparity between pre-improvement and post-improvement shown in the HGL profile is due to the elimination of the upstream capacity restrictions. Even though the surcharging depth at Manhole I17-7 exceeds 1foot above the pipe crown, the minimum freeboard of the manhole is greater than 20 feet. As the cost for upgrading one of the 18 inches twin sewers would be high, it is proposed to not upgrade the twin sewer currently.

- Existing Service Area Build-out: Similar to the existing system, it is not critical to upgrade one of the 18 inch twin sewers at the time.

- Existing Build-out plus Near-Term: Approximately 897 feet of 18 inch sewer would need to be upgraded to 24 inches.

- Existing Build-out plus both Near and Long-Term: There is no additional upgrade required beyond that described for the Existing Build-out plus Near-Term growth scenario.

- Full Build-out: Approximately 4,065 feet of sewer would be required to be upgraded to 30 inches and 310 feet would need to be upgraded to 36 inch sewers to accommodate future flows. It should be noted that one of the twin sewer is proposed to be upgraded to 30 inches or another new conduit with a 27 inch in diameter would be required to be paralleled to accommodate future flows. Further investigation for upgrades of the existing twin sewer should be examined due to the high construction costs at this location.

4.5.2.6 HGL Profile 6 (Figure E-14, E-22, E-30, E-38 and E-46)

- Existing level of development: Approximately 255 feet of 6 inch sewer is should to be upgraded to 8 inches.
- Existing Service Area Build-out: Approximately 354 feet of 6 inch sewer would need to be upgraded to 8 inches; 152 feet of 8 inch sewer would need to be upgraded to 10 inches.
- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described for the Existing Service Area Build-out scenario.
- Existing Build-out plus both Near and Long-Term: Additional 293 feet of sewers are would need to be upgraded to 8 inches.
- Full Build-out: Approximately 576 feet of sewers are would need to be upgraded to 8 inches and 506 feet would need to be upgraded to 10 inches.

4.5.2.7 HGL Profile 7 (Figure E-15, E-23, E-31, E-39 and E-47)

- Existing level of development: Approximately 96 feet of sewer need to be upgraded to 15 inches.
- Existing Service Area Build-out: There is no additional upgrade required, beyond that described for Existing development.
- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described for Existing development.
- Existing Build-out plus both Near and Long-Term: Approximately 169 feet of sewers would need to be upgraded to 10 inches.
- Full Build-out: Approximately 904 feet of sewers would need to be upgraded to 15 inches, 705 feet sewers would need to be upgraded to 18 inches, 2,421 feet of sewers would need to be upgraded to 21 inches, and 169 feet sewers would need to be upgraded to 30 inches.

4.5.2.8 HGL Profile 8 (Figure E-16, E-24, E-32, E-40 and E-48)

- Existing level of development: Approximately 1,135 feet sewer proposed to be upgraded to 8 inches.
- Existing Service Area Build-out: There is no additional upgrade required beyond that described for the Existing level of development.
- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described for the Existing level of development.
- Existing Build-out plus both Near and Long-Term: There is no additional upgrade required beyond that described for the Existing level of development.
- Full Build-out: A total of approximately 2,796 feet of sewers would need to be upgraded to 15 inches and 351 feet of sewers would need to be upgraded to 18 inches.

4.5.2.9 Sewer required to be upgraded beyond the extent of the above eight HGL profiles

- Existing level of development: Approximately 651 feet of sewer needs to be upgraded to 8 inch sewer.
- Existing Service Area Build-out: There is no additional upgrade required beyond that described as the Existing level of development.
- Existing Build-out plus Near-Term: There is no additional upgrade required beyond that described as the Existing level of development.
- Existing Build-out plus both Near and Long-Term: There is no additional upgrade required beyond that described as the Existing level of development.
- Full Build-out: A total of approximate 1,018 feet of sewers would need to be upgraded to 8 inches, 1,279 feet of sewers would need to be upgraded to 10 inches, 2,192 feet of sewers would need to be upgraded to 12 inches, 1216 feet of sewers would need to be upgraded to 15 inches, 400 feet of sewers would need to be upgraded to 21 inches, 905 feet of sewer would need to be upgraded to 27 inches and 397 feet of sewers would need to be upgraded to 30 inches.

4.5.3 Site Visits

The sewer system has experienced sagging, grease, root intrusion, offset joints, build-up of deposits, and inadequate grade issues. A comprehensive sewer repair and replacement program is considered a priority in the City's Improvement Plan, presented in Chapter 6 of this document, to reduce extraneous inflow and infiltration in the existing collection system. The extent of the model predicted sewer upgrades described in this section is highly dependent on the amount of I/I in the system. The City is implementing a sewer rehab project and is planning to incorporate an ongoing I/I reduction program in to their budget.

This Master Plan recommends the City evaluate the effectiveness of these I/I reduction efforts as they are implemented, re-assess the collection system improvements described here to address existing and anticipated capacity concerns, then adjust those planned upgrades as appropriate (e.g. reducing the magnitude of upgrades recommended).

4.5.3.1 City Database

Due to the inconsistency of invert elevation, pipe slope and pipe sizes identified in the initial sewer system model, a field survey was conducted as discussed in Section 4.3.2. A summary of the survey results are attached in **Appendix C**. It is recommended that the City consider updating the database of collection system asset data (rim/invert elevations) to enhance the accuracy of future modeling efforts.

4.5.3.2 Sewer Lift Station Issues

As discussed in Section 4.4.4 and Section 4.4.9, there are seven lift stations in the existing sewer system. Carriage House lift station is predicted to have sufficient full pumping capacity to accommodate the Full Build-out of the service area. But the additional pumping capacity to ensure sufficient firm capacity should be considered prior to of Full Build-out development occurring. The lift station should have sufficient capacity to serve Near and Long-Term growth with planned upgrades.

The Joyce Drive lift station should have sufficient full pumping capacity to accommodate flow from Full Build-out of the service area. The previously proposed flow diversion downstream of Taylorville Road and upstream of Joyce Drive may not be required. The capacity issue identified is recommended to be further considered.

Railroad Avenue lift station and Taylorville lift station are both predicted to have sufficient firm capacity to accommodate planned future flows.

The Slate Creek lift station is currently identified as deficient in terms of firm pumping capacity. The need for this lift station to be upgraded has been considered in the City's Improvement Plan. Further upgrades to serve Full Build-out (including the Kenny Ranch area) may be required in the future.

Morgan Ranch lift station is also predicted to be close to the full pumping capacity with the current Existing level of development. Therefore, upgrades for Morgan Ranch have been considered in the City's Improvement Plan.

As discussed in Section 4.2.4, the Carriage House lift station and Taylorville lift station had not yet been upgraded during 2014 site visits. Based upon the results of those site visits, an upgrade of the electrical conduit serving the wetwell and pumps at the Taylorville lift station is recommended in order to meet current NFPA code. It is also recommended to install a backup generator at the Carriage House lift station. It is understood that the City is currently (March 2016) designing upgrades to the Carriage House lift station, which may be complete prior to the wet season of 2016/2017.

5.0 WASTEWATER TREATMENT PLANT

5.1 PURPOSE AND SCOPE

The City owns and operates a wastewater treatment plant (WWTP, plant) that serves the area identified in **Figure 4-1**. The plant includes a nitrification/denitrification activated sludge treatment system followed by advanced treatment facilities to produce a filtered and disinfected effluent for discharge to Wolf Creek. The original design capacity was 2.78 Mgal/d as an average dry weather flow with a peak flow capacity through the activated sludge system of 7.0 Mgal/d.

The design of a wastewater treatment plant is based on wastewater flow rates (hydraulic capacity) and on the amounts of pollutants contained in the wastewater (biological capacity). The most common pollutants are oxygen demanding substances, which are measured as biochemical oxygen demand (BOD); solid particles, which are measured as total suspended solids (TSS); and ammonia and organic nitrogen, which together are measured as total Kjeldahl nitrogen (TKN). However, many other pollutants are also important. The amounts of the various pollutants are best expressed as mass loadings or simply loads. A pollutant load is calculated as the influent flow rate multiplied by the concentration of the pollutant in question.

The purpose of this Chapter is to evaluate each major component of the wastewater treatment system to determine existing capacity, existing deficiencies, and evaluate improvements needed at different incremental growth horizons. In the development of recommended improvements, alternative methods of accomplishing desired goals are evaluated with the objective of developing the most cost-effective system to serve the needs of the City. In many cases, alternatives and considerations in one area of the plant have major implications in other areas. Therefore, an integrated analysis is provided.

5.2 ANALYSIS OF HISTORICAL FLOWS AND LOADS

5.2.1 Historical Flows

Wastewater flow data recorded at the Grass Valley WWTP for the period of January 2009 through April 2014 were obtained and analyzed. The average dry weather flow, 30-day rolling average (monthly average) flow, and 365-day rolling average (annual average) flows were calculated for the available data and plotted in **Figure 5-1**.

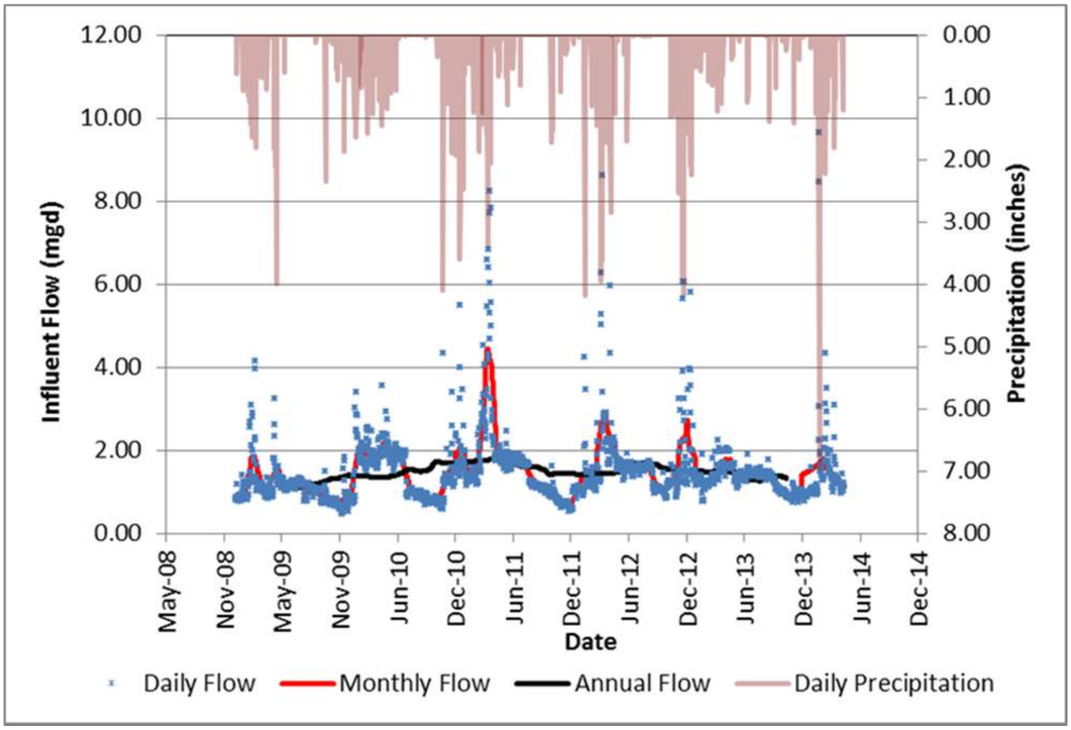


Figure 5-1 Influent Flow and Precipitation

Average Dry Weather Flow (ADWF)

The average dry weather flow is calculated here as the average of the lowest two months of flows for the year. From the data analysis, it appears that the influent flows in the months of September and October are the lowest flows of the year, as shown in **Figure 5-2**. The ADWF ranges between 1.0 Mgal/d and 1.3 Mgal/d as shown in **Figure 5-3** and **Table 5-1**.

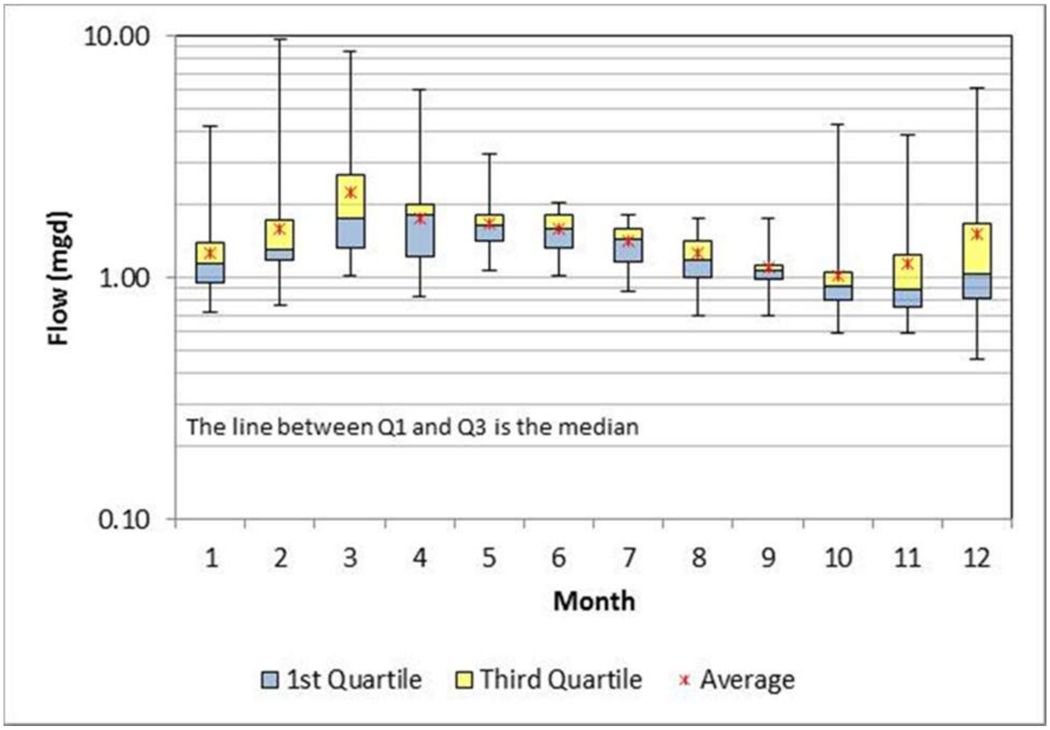


Figure 5-2 Variations in Monthly Influent Flow

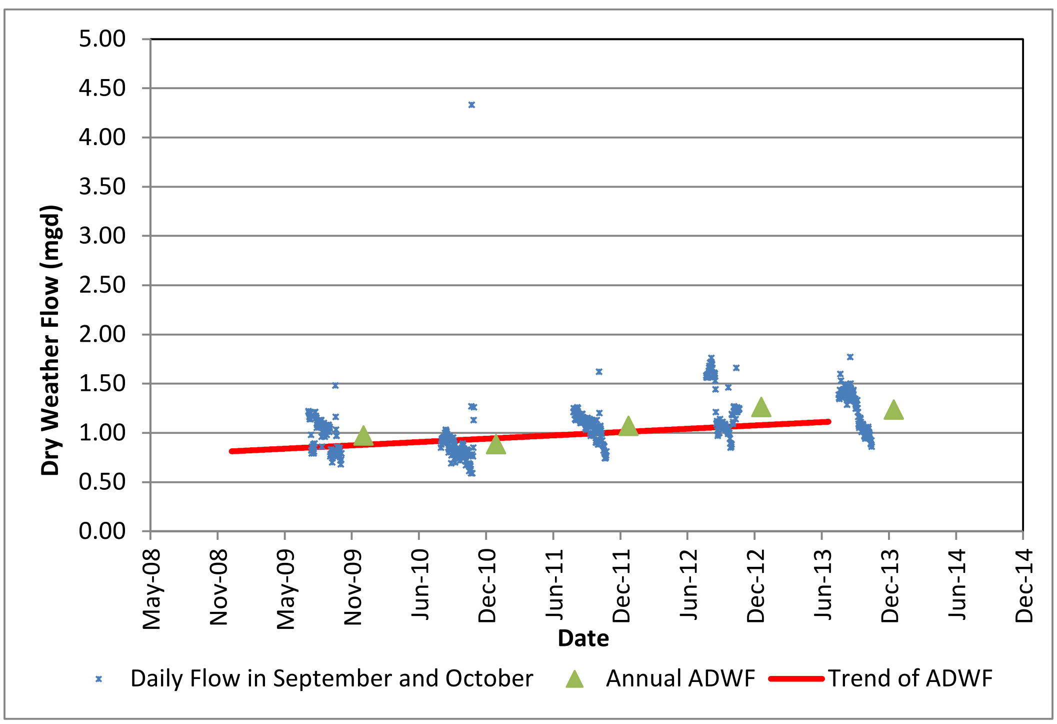


Figure 5-3 Influent Average Day Weather Flow (ADWF)

Table 5-1 Annual Flows and Peaking Factors

	2009	2010	2011	2012	2013	Current Condition (2014 basis) to be used for Flow Projections
Precipitation (inches)	46.8	72.4	45.9	44.4	34.8	
ADWF (Mgal/d)	0.97	0.88	1.07	1.26	1.23	1.3
AAF (Mgal/d)	1.10	1.53	1.62	1.68	1.30	
PMF (Mgal/d)	1.84	2.18	4.39	2.95	1.78	
PDF (Mgal/d)	4.14	5.50	8.24	8.61	2.60	
AAF/ADWF (ratio)	1.1	1.7	1.5	1.3	1.1	1.7
PMF/ADWF (ratio)	1.90	2.47	4.10	2.35	1.44	4.1
PDF/ADWF (ratio)	4.27	6.25	7.71	6.84	2.27	7.7

Annual Average Flow (AAF)

The annual average flow is calculated as the average flow from July 1st through June 30th of the following year. The reason for selecting these dates is to capture the wet season. The magnitude of the AAF depends on the amount of precipitation. The City's AAF ranges between 1.1 Mgal/d to 1.68 Mgal/d (see **Table 5-1**).

Peak Month Flow (PMF)

For the purposes of this analysis, it is desirable to develop a peak month flow criterion that would occur with some regularity, say about once per year on average. From **Figure 5-1**, the maximum recorded peak month flow value (4.4 Mgal/d, recorded in March 2011) was a result of moderate rainfall. The monthly rainfall during that month was 19.6 inches. This monthly rainfall has a return frequency between 2 and 5 years as shown in **Table 5-2**.

Table 5-2 Rainfall Return Frequency at Grass Valley

Station		Station No		County	Lat.	Long.	Elev.					
Grass Valley		A60 3571 00		Nevada	39.226	-121.059	2693					
Return Period for Rainfall For Indicated Number Of Consecutive Days												
	1	2	3	4	5	6	8	10	15	20	30	60	W-YR
RP 2	3.73	5.29	6.53	7.42	8.23	8.94	10.27	11.25	13.52	15.02	18.18	27.05	51.92
RP 5	5.01	7.34	9.13	10.38	11.47	12.35	14.11	15.41	18.33	20.51	24.69	36.47	66.41
RP 10	5.85	8.71	10.87	12.33	13.60	14.51	16.50	17.95	21.17	23.82	28.60	42.02	74.64
RP 25	6.86	10.43	13.05	14.75	16.23	17.13	19.37	20.96	24.46	27.68	33.18	48.45	83.92
RP 50	7.60	11.68	14.64	16.50	18.15	19.01	21.41	23.08	26.72	30.37	36.37	52.89	90.19
RP 100	8.31	12.92	16.21	18.22	20.02	20.82	23.37	25.11	28.86	32.93	39.40	57.07	96.03
RP 200	9.01	14.14	17.76	19.91	21.87	22.59	25.27	27.06	30.91	35.39	42.31	61.07	101.54
RP 500	9.92	15.75	19.80	22.12	24.28	24.88	27.72	29.57	33.49	38.52	46.02	66.13	108.42
RP 1000	10.60	16.95	21.33	23.78	26.09	26.59	29.53	31.41	35.38	40.82	48.74	69.83	113.39
RP 10000	12.83	20.95	26.41	29.23	32.05	32.13	35.40	37.35	41.36	48.14	57.41	81.52	128.84

Peak Day Flow (PDF)

As shown in **Figure 5-1**, peak day average flows in excess of 8.0 Mgal/d have been measured on several occasions in the period between 2009 and 2014. Some peak day flows exceeded 9.0 Mgal/d.

Peak Hour Flow (PHF)

The peak hour flow, also referred to as the instantaneous wet weather flow (WWF) is computed by assuming concurrent peak diurnal dry weather flow with the peak of RDII of a 10 year, 24 hour storm. The WWF at different ADWF conditions is established and presented in Chapter 3.

Summary of Flow Peaking Factors

Flow peaking factors were calculated by dividing the daily, monthly, and annual flows by the appropriate historical ADWF. These peaking factors are summarized in **Table 5-1**.

5.2.2 Historical Loads

BOD Load

Plant influent Biochemical oxygen demand (BOD_5 or simply BOD) concentrations from January 2009 through April 2014 were collected and analyzed. Samples were flow-proportional composites and were taken three times a week. Samples were reportedly taken from a well-mixed location, upstream from any in-plant recycle streams. The influent BOD load was calculated from the BOD concentrations and the daily influent flows. The monthly and annual loadings were calculated as the 30-day rolling average and 365-day rolling average, respectively.

BOD Load Peaking Factors

Daily, monthly, and annual BOD loadings are plotted in **Figure 5-4**. As shown in **Figure 5-4**, there are several recorded high daily BOD loadings, which are believed to be unreliable outliers and should not be considered when developing future load projections. For example, BOD loadings on March 8, 2011 were about 9,500 lb/d, which is about 4.5 times the average annual load at that time. Such high loadings are believed to be unrepresentative and were not evidenced by any plant performance problems. To eliminate such outlier data from further analysis, the highest 5% of the BOD data were disregarded. The 95th percentile of the peaking factors for peak month load (peak month load/average annual load) and peak day load (peak day load/average annual load) were 1.4 and 1.9, respectively (See **Figures 5-5** to **5-7**).

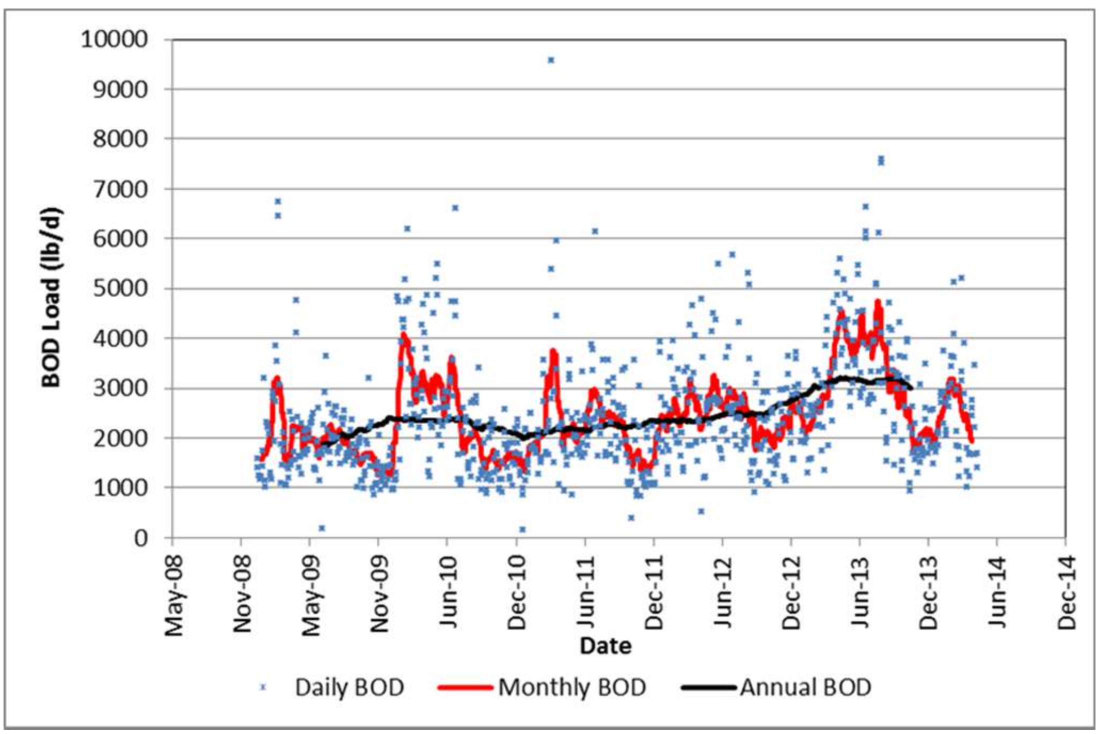


Figure 5-4 Influent BOD Load

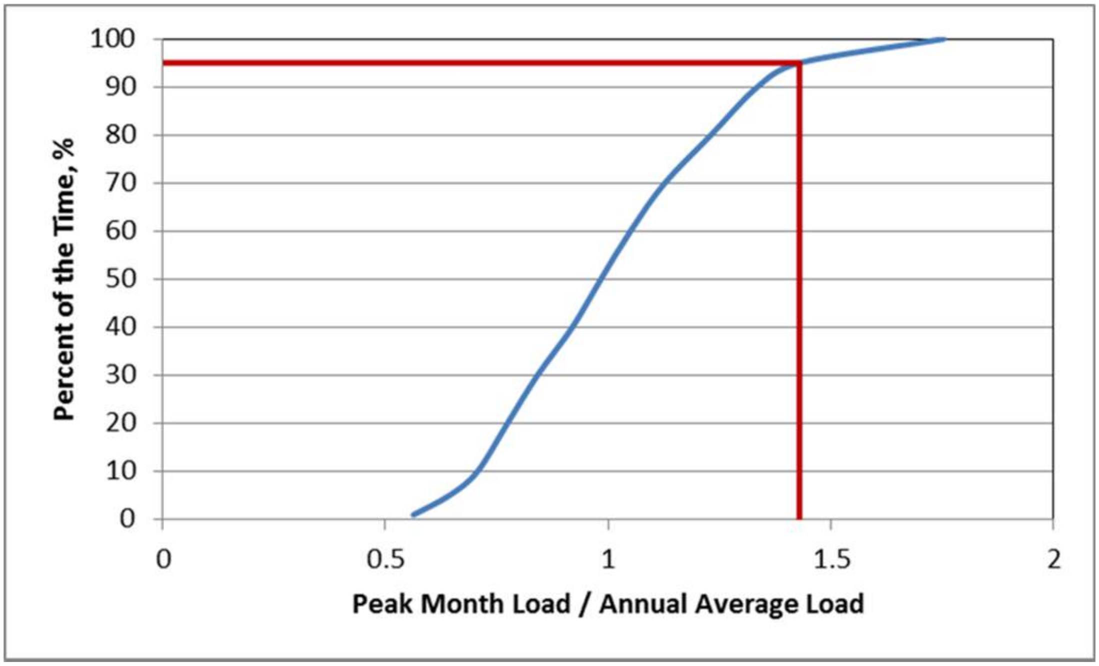


Figure 5-5 Cumulative Frequency Distribution for BOD Peak Month Load Peaking Factor

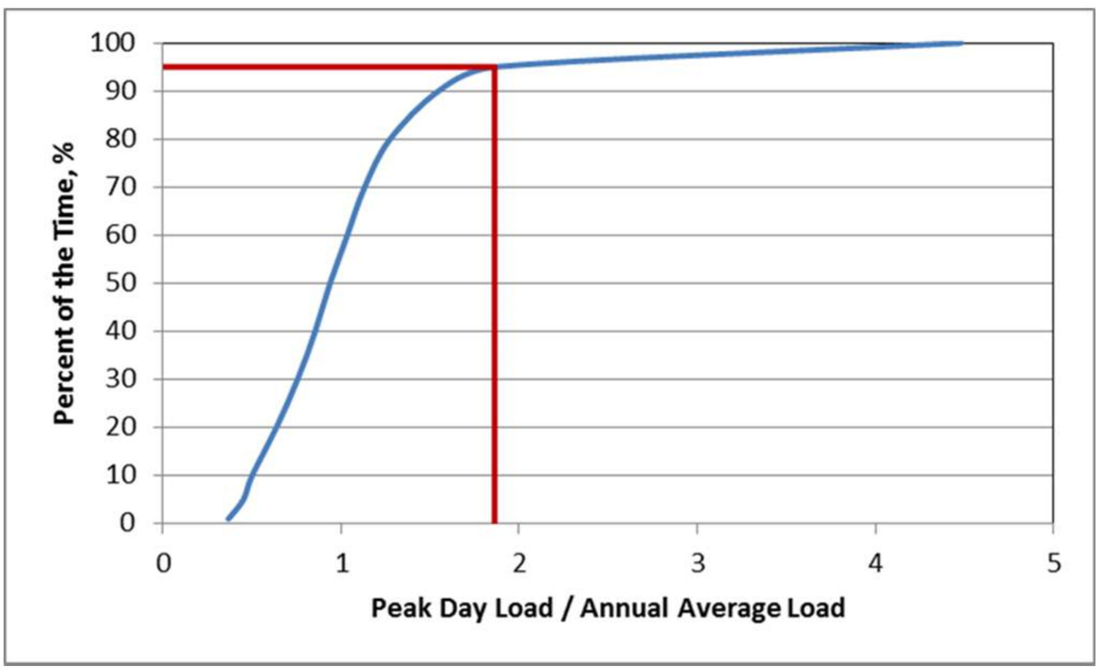


Figure 5-6 Cumulative Frequency Distribution for BOD Peak Day Load Peaking Factor

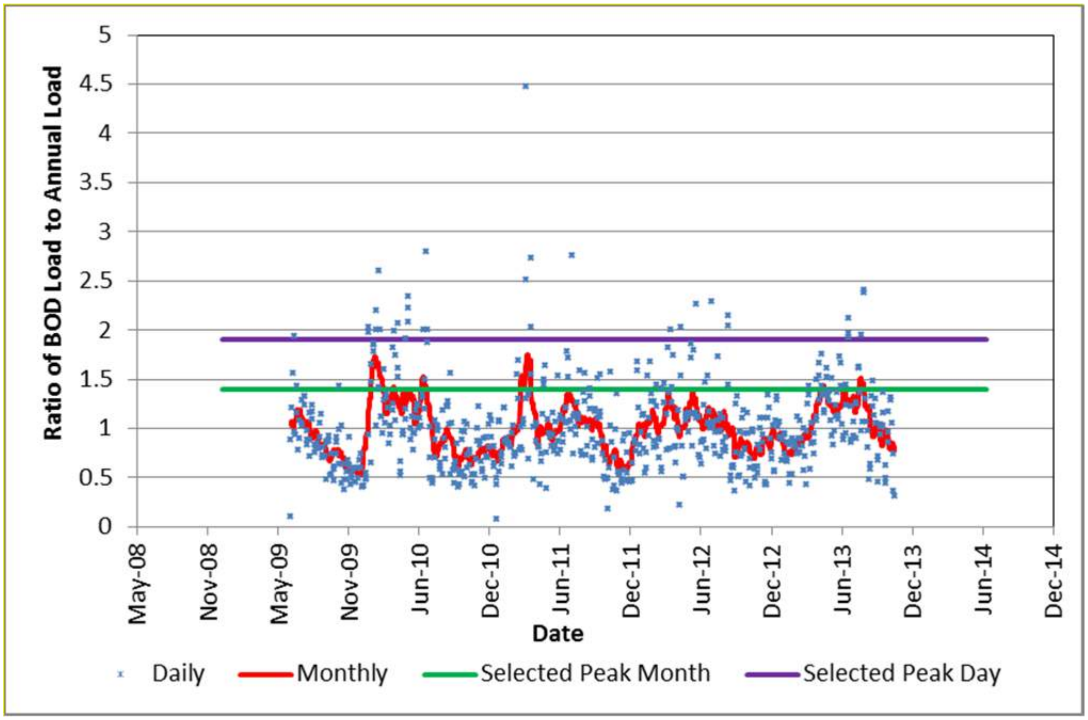


Figure 5-7 BOD Peaking Factors

TSS Load

Influent monthly average and annual average total suspended solids (TSS) loads were developed in the same manner as the BOD. As with BOD, some of the TSS data are outliers (see **Figure 5-8**).

A typical ratio of TSS to BOD is about 1.0-1.2. Although the TSS/BOD ratio was variable, the average TSS/BOD was about 1.1 after excluding the outliers. TSS/BOD ratios of more than 3 or less than 0.33 were considered outliers as shown in **Figure 5-9**. It is assumed that the TSS peaking factors will be similar to the BOD peaking factors.

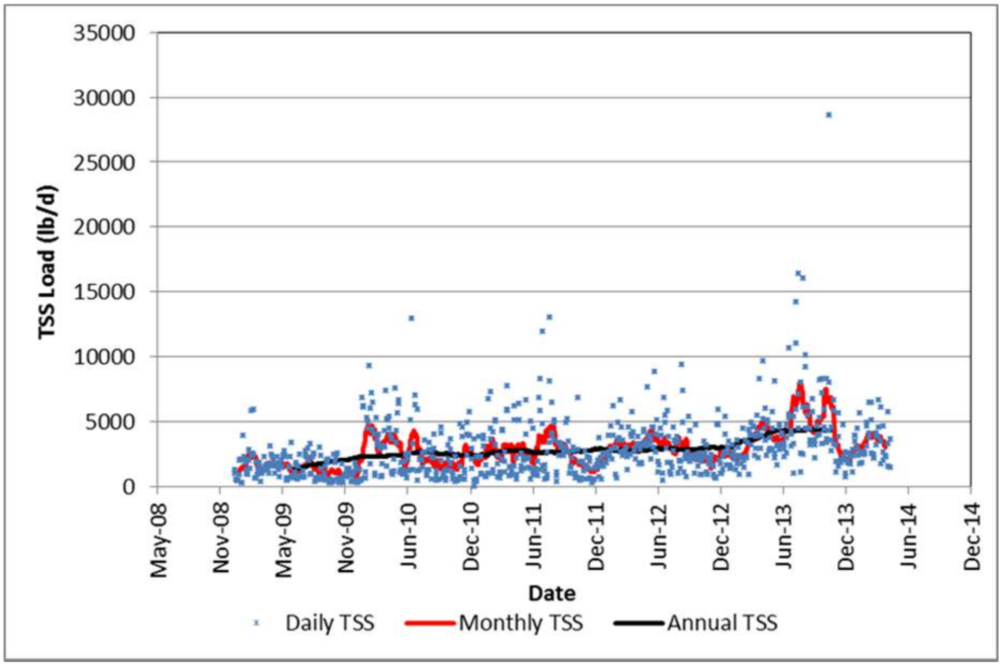


Figure 5-8 Influent TSS Load

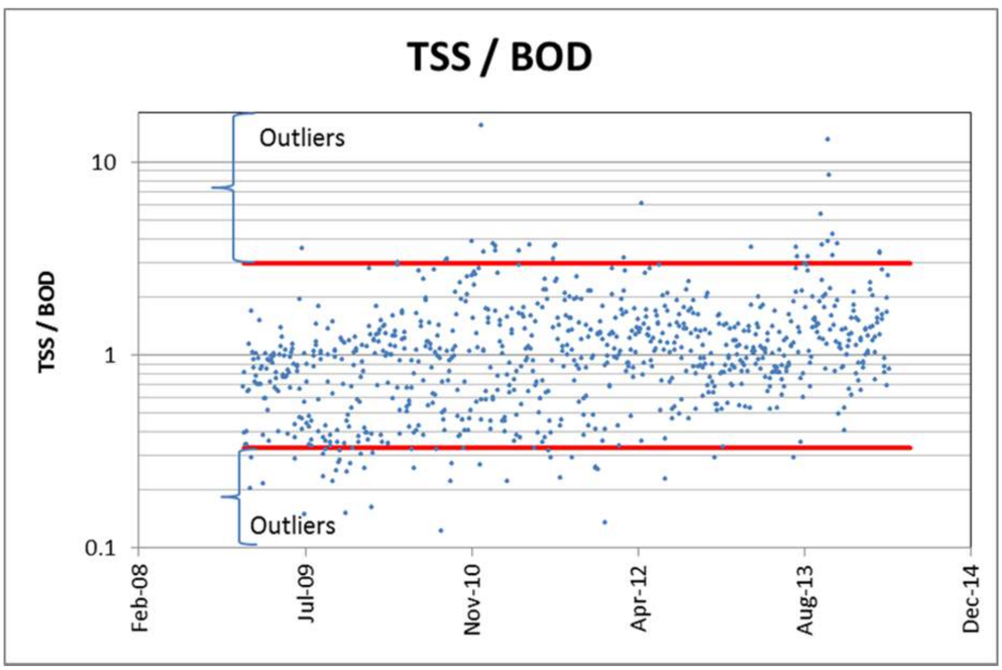


Figure 5-9 TSS/BOD Ratio

TKN Load

There is no historical influent TKN or ammonia data available. However, a typical municipal wastewater will have a ratio of TKN/BOD of 18-20%. For the purpose of projecting nitrogen load to the plant, TKN was assumed to be 20% of the BOD.

5.3 PROJECTIONS OF FUTURE FLOWS AND LOADS

Population projections were used to estimate future influent wastewater flows and loads as discussed in Chapter 3. Projected flows and loads will be the basis for evaluating wastewater treatment capacity for the Grass Valley WWTP.

5.3.1 Approach to Flow Projections

Typically, average dry weather flows are used as the basis for flow projections. ADWF was projected in Chapter 3 for five conditions as shown in **Table 5-3**.

Table 5-3 Average Dry Weather Flow Projections

Case	ADWF (Mgal/d)
Existing Conditions	1.3
Vacant Parcels Within City Limits	1.6
Near Term	1.9
Long Term	2.1
Area of Concern	4.0

All wastewater flow normally includes domestic flow and infiltration and inflow (I/I) flow fractions. The City of Grass Valley has been pursuing the reduction of I/I in its sewage collection system for many years.

The current annual average flow (based on 2009-2014 data set) including I/I is 140 percent of the domestic wastewater flow, which is significant but not atypical of Sierra foothill communities. The increase in peak flows due to I/I resulting from new connections must be projected. In this regard, it is noted that much of the backbone sewage collection system that will serve the new connections is already in place and contributing I/I. The only additional I/I will come from sewer main extensions and from new service laterals, which will be built to modern standards and it is hoped will contribute less I/I than corresponding older, existing facilities. Therefore, the amount of I/I added per incremental unit of ADWF increase should be significantly lower than the amount of I/I per unit of ADWF for existing users. Accordingly, it is assumed that the rate of increase in the I/I component of peak flows (the increment of flow above the ADWF, or excess flow) will be 50 percent of the rate of increase in ADWF. For example, if the ADWF was to increase by

10 percent, excess flows due to I/I would be projected to increase by 5 percent. Obviously, there is substantial judgment involved in projections of this nature and the City will have to assess the results of planned collection system rehabilitation efforts in terms of actual I/I reductions as they occur over the years. Actual results could necessitate adjustments to the projections and the timing of future plant expansions to accommodate additional increases in domestic flows.

5.3.2 Load Projections

As shown in **Figure 5-4** and summarized in **Table 5-4**, the annual average BOD load has been gradually increasing from about 1,880 lb/d in 2009 to about 3,100 lb/d in 2013. This annual increase in BOD load is substantial over such a short period of time. As a reality check, the average BOD expected from Grass Valley was calculated based on Grass Valley's population of 12,680 persons and the typical BOD generation of 0.22 lb/capita/day when disposal grinders are utilized in a community (Metcalf and Eddy, 4th edition). The resulting BOD load is approximately 2,800 lb/d, which is slightly higher than the 2012 load and lower than the 2013 load. Since there is a sudden and unexplained load increase from 2012 to 2013, it is reasonable to select an average value for the BOD load. A BOD load of 3,000 lb/d was selected to be used for future load projections. TSS and TKN will be projected by multiplying the BOD load by 1.1 and 0.2, respectively.

Table 5-4 Annual Average BOD Load

	2009	2010	2011	2012	2013	current condition (2014 basis) to be used for BOD projections
BOD Load (lb/d)	1883	2374	2211	2477	3132	3000

5.3.3 Summary of Flows and Loads Projections

Projected wastewater flows and characteristics to be used for future conditions are presented in **Table 5-5**.

Table 5-5 Current and Projected Flows and Loads

Parameter	Unit	Current Conditions			Vacant Parcels Within City Limits			Near Term			Long Term			Area of Concern		
		Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total
Flow																
ADWF	Mgal/d	1.30	0.00	1.30	1.6	0.00	1.60	1.9	0.00	1.90	2.1	0.00	2.10	4.0	0.00	4.00
AAF	Mgal/d	1.30	0.91	2.21	1.6	1.02	2.62	1.9	1.12	3.02	2.1	1.19	3.29	4.0	1.86	5.86
PMF	Mgal/d	1.30	4.03	5.33	1.6	4.50	6.10	1.9	4.96	6.86	2.1	5.27	7.37	4.0	8.22	12.22
PDF	Mgal/d	1.30	8.71	10.01	1.6	9.72	11.32	1.9	10.72	12.62	2.1	11.39	13.49	4.0	17.76	21.76
PHF	Mgal/d	1.30	17.60	18.90	1.6	18.90	20.50	1.9	19.90	21.80	2.1	20.90	23.00	4.0	35.70	39.70
BOD Loads																
AAL	lb/day			3,000			3,690			4,380			4,850			9,230
PML	lb/day			4,200			5,170			6,140			6,780			12,920
PDL	lb/day			5,700			7,020			8,330			9,210			17,540
TSS Loads																
AAL	lb/day			3,300			4,060			4,820			5,330			10,150
PML	lb/day			4,620			5,690			6,750			7,460			14,220
PDL	lb/day			5,700			7,020			8,330			9,210			17,540
TKN Loads																
AAL	lb/day			600			740			880			970			1,850
PML	lb/day			840			1,030			1,230			1,360			2,580
PDL	lb/day			1,140			1,400			1,670			1,840			3,510
Average Dry Weather Constituent Concentrations																
BOD	mg/L			277			277			276			277			277
TSS	mg/L			304			304			304			304			304

Parameter	Unit	Current Conditions			Vacant Parcels Within City Limits			Near Term			Long Term			Area of Concern		
		Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total	Domestic Flow	I/I	Total
TKN	mg/L			55			55			56			55			55
Annual Average Constituent Concentrations																
BOD	mg/L			163			169			174			177			189
TSS	mg/L			179			186			191			194			208
TKN	mg/L			33			34			35			35			38
Flow Peaking Factors																
AAF/ADWF	-			1.70			1.63			1.59			1.57			1.46
PMF/ADWF	-			4.10			3.81			3.61			3.51			3.05
PDF/ADWF	-			7.70			7.07			6.64			6.42			5.44
PHF/ADWF	-			14.54			12.81			11.47			10.95			9.93
Load Peaking Factors																
PML/AAL				1.40			1.40			1.40			1.40			1.40
PDL/AAL				1.90			1.90			1.90			1.90			1.90

(a) Assumptions

1. Fractional I/I increase will be 50% of the fractional ADWF increase for future connections (the difference between a peak flow and ADWF is used as an indicator of I/I).
2. TSS loads are 1.1 times BOD loads.
3. TKN loads are 0.2 times BOD loads.
4. All loads increase in direct proportion to ADWF.

5.4 EXISTING WWTP EVALUATION

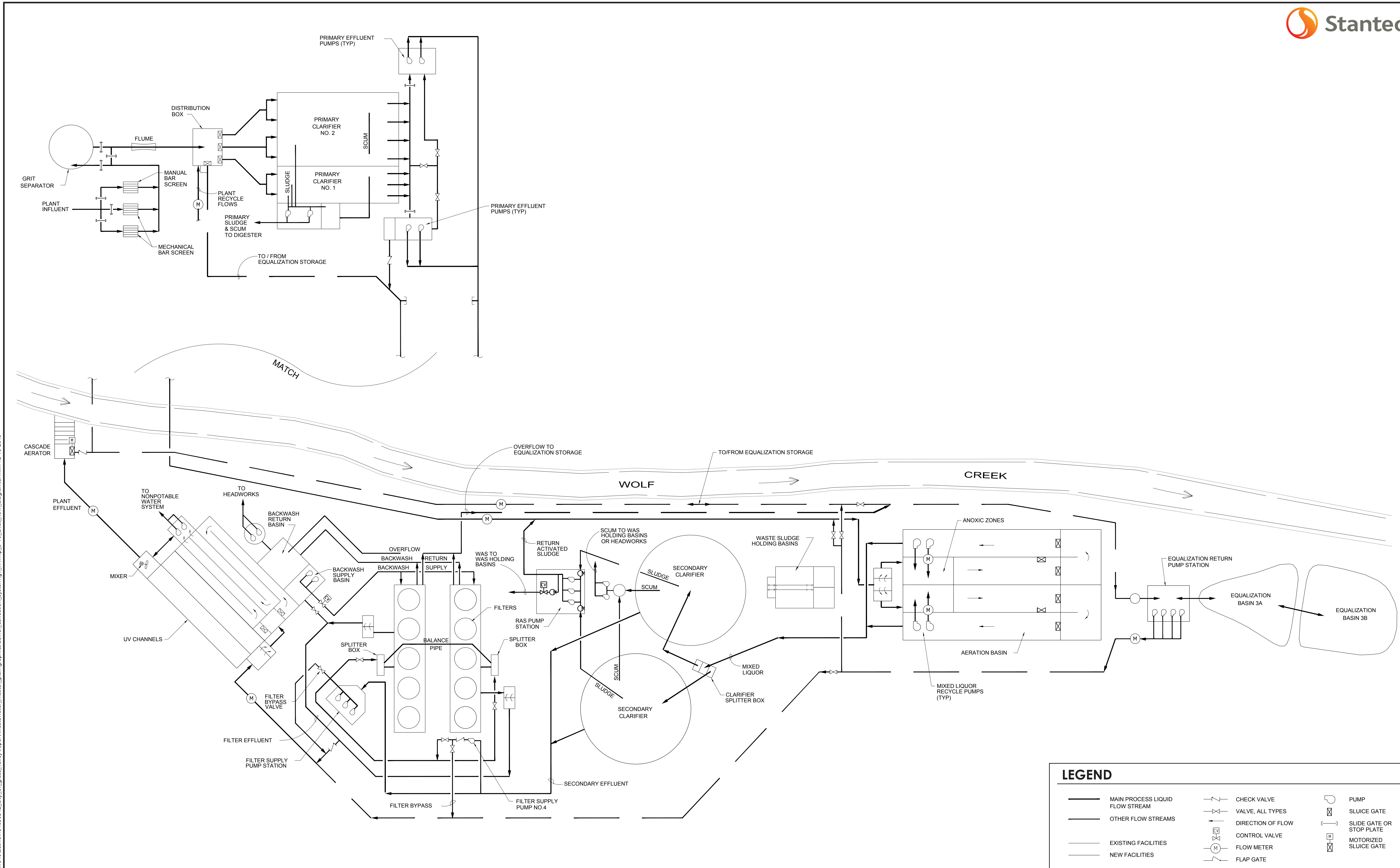
5.4.1 Wastewater Facility Overview

The Grass Valley WWTP is a biological nutrient removal activated sludge system that uses plug flow reactors. The plant is comprised of a headworks (screening and grit removal) with odor control, primary treatment (primary clarifiers), and Secondary Treatment (aeration basin and secondary clarifiers). Secondary effluent is filtered and disinfected using ultraviolet (UV) disinfection before it is discharged to Wolf Creek. Primary sludge with waste activated sludge (WAS) is fed to one anaerobic digester for solids stabilization. **Figure 5-10** show a process flow diagram for the existing plant. The WWTP design criteria are summarized in **Table 5-6**.

Table 5-6 Existing WWTP Design Criteria

Design Flows and Loads	
Influent Flow, Mgal/d	
Average Dry Weather Flow (ADWF)	2.78
Peak Month	4.78
Peak Day	12.0
Peak Hour	16.0
Equalized Peak	7.0
Average Dry Weather Constituent Concentration, Mgal/L	
BOD5	315
SS	315
TKN	56
Alkalinity	142
Average Dry Weather Constituent Loads, lb/d	
BOD5	7300
SS	7300
TKN	1300
Alkalinity	3300
Peak Month Constituent Loads, lb/d	
BOD5	9800
SS	10200
TKN	1700
Alkalinity	4300

Effluent Criteria	
Monthly Mean or Median, Except As Noted	
BOD5, Mgal/L	10
SS Mgal/L	10
Ammonia-N, Mgal/L	2.0
Turbidity, NTU	2
Chlorine Residual (Hourly Avg) Mgal/L	0.02
Total Coliform, MPN/100ml	2.2


Stantec
PRIMARY EFFLUENT PUMPS (TYP)
DISTRIBUTION BOX
FLUME
GRIT SEPARATOR
MANUAL BAR SCREEN
PLANT INFLUENT
MECHANICAL BAR SCREEN
PLANT RECYCLE FLOWS
PRIMARY CLARIFIER NO. 2
PRIMARY CLARIFIER NO. 1
SCUM
SLUDGE
PRIMARY SLUDGE & SCUM TO DIGESTER
PRIMARY EFFLUENT PUMPS (TYP)
TO / FROM EQUALIZATION STORAGE
MATCH
CASCADE AERATOR
PLANT EFFLUENT
TO NONPOTABLE WATER SYSTEM
TO HEADWORKS
BACKWASH RETURN BASIN
MIXER
BACKWASH SUPPLY BASIN
UV CHANNELS
OVERFLOW
BACKWASH
RETURN
BACKWASH
SUPPLY
WAS TO WAS HOLDING BASINS
FILTERS
BALANCE PIPE
SPLITTER BOX
SPLITTER BOX
FILTER BYPASS VALVE
FILTER EFFLUENT
FILTER SUPPLY PUMP STATION
FILTER BYPASS
FILTER SUPPLY PUMP NO.4
OVERFLOW TO EQUALIZATION STORAGE
WOLF
CREEK
TO/FROM EQUALIZATION STORAGE
RETURN ACTIVATED SLUDGE
RAS PUMP STATION
SCUM TO WAS HOLDING BASINS OR HEADWORKS
SECONDARY CLARIFIER
SLUDGE
SCUM
SECONDARY CLARIFIER
SCUM
SLUDGE
MIXED LIQUOR
CLARIFIER SPLITTER BOX
SECONDARY EFFLUENT
WASTE SLUDGE HOLDING BASINS
ANOXIC ZONES
AERATION BASIN
MIXED LIQUOR RECYCLE PUMPS (TYP)
EQUALIZATION RETURN PUMP STATION
EQUALIZATION BASIN 3A
EQUALIZATION BASIN 3B
LEGEND
MAIN PROCESS LIQUID FLOW STREAM
OTHER FLOW STREAMS
EXISTING FACILITIES
NEW FACILITIES
CHECK VALVE
VALVE, ALL TYPES
DIRECTION OF FLOW
CONTROL VALVE
FLOW METER
FLAP GATE
PUMP
SLUICE GATE
SLIDE GATE OR STOP PLATE
MOTORIZED SLUICE GATE

5.4.2 Headworks

Screens

Wastewater from the City collection system flows by gravity to the plant headworks where debris, floatables, and grit are removed from the wastewater. There are three screens channels at the headworks: two channels are equipped with ¼" mechanically cleaned bar screens and one channel is equipped with a manual bar screen.

Grit Removal

After being screened, wastewater flows into the vortex grit chamber where heavy inorganic material such as rocks, sand, and shells are removed. The grit collected at the bottom of the chamber is pumped out and discharged into the grit classifier where it is washed to separate inorganics from biodegradable compounds. Inorganic material is discharge into a disposal bin and wash-water and organics are returned back to the process for further treatment. The grit removal system consists of one grit chamber with an estimated peak hydraulic capacity of 12 Mgal/d. No redundant units are provided, however, the grit chamber can be bypassed temporarily in case of equipment failure or during a peak flow event.

Parshall Flume

Following grit removal, the screened and de-gritted wastewater flow rate is measured by a 36-inch Parshall flume. The capacity of the Parshall Flume is 32.6 Mgal/d. The hydraulics of the Parshall flume are limited based on downstream water levels at high flows. A recent analysis (October 2013, Stantec) indicates that upstream and downstream conditions limit the capacity of the flume to 17 Mgal/d with unencumbered downstream freeboard in the primary clarifiers. Further, the October 2013 memo identifies correction factors that should be applied to flume readings with various levels of downstream submergence.

Odor Control

The headworks building is equipped with a carbon scrubber odor control system to reduce emission of hydrogen sulfide gas and other odorous compounds to the atmosphere from the building headworks screen channels.

5.4.3 Flow Equalization

The wastewater plant currently has two (2) equalization basins with a total capacity of 6.1 Mgal. The basins are lined and are located at the south side of the WWTP. The basins are hydraulically off-line and are designed to accept raw influent or primary effluent flows in excess of 7.0 Mgal/d. When the peak flow subsides, the stored wastewater is returned to the plant to be treated. In the event that the actual peak flow exceeds the design flow conditions or sustained longer, the equalization basins may become full. When influent flow conditions are such that equalized influent or primary effluent cannot be returned for treatment at a rate sufficient to prevent the basins over tipping and spilling, the basin contents would be pumped directly to the UV basin to prevent discharge of un-disinfected wastewater to Wolf Creek. The flow pumped from equalization storage would be blended with the filtered secondary effluent for disinfection prior to discharge. Historically, (from 2000 to 2014), peak flows have been diverted to the equalization basin about 5 times per year. The maximum reported volume that has been stored was 6.1 Mgal (the basins were full) on March 27, 2012.

5.4.4 Primary Treatment

Screened and de-gritted raw wastewater from the headworks flows by gravity into the primary clarifier distribution box where it is combined with plant recycle flows which include filter backwash, belt thickener filtrate, belt filter press filtrate, and miscellaneous process drains including building drains. From the primary distribution box, a portion of the flow can be diverted to the equalization basins as needed to shave off the peak flows. The flow from the equalization basin can also be returned to the primary distribution box using the same pipe.

Primary treatment is accomplished using two rectangular clarifiers. The primary clarifiers are designed to remove readily settleable suspended solids, thus reducing the BOD load on the secondary process.

Primary effluent flows by gravity into one of the two primary effluent pump chambers where it is pumped to the aeration basins splitter box. Primary sludge collected with chain and flight mechanisms and primary scum are pumped using progressive cavity pumps to the anaerobic digester.

5.4.5 Secondary Treatment

The last major upgrade to the Grass Valley WWTP was in 1999 when a new secondary treatment activated sludge system was constructed. The secondary process is a Modified Ludzack Ettinger (MLE) activated sludge process for nitrogen removal. Secondary treatment facilities include two plug flow reactor basins, two secondary clarifiers, and a return activated sludge (RAS) pump station. The reactor basins are split to pre-anoxic and aerobic zones.

Primary effluent and RAS (Return Activated Sludge) are combined and enter the anoxic zone. The mixed liquor passes through the anoxic zone and enters the aerobic zones of the reactor basin. Oxygen is supplied to the aerobic zone through fine bubble diffusers and aeration blowers. The mixed liquor flows by gravity from the aeration basins to the secondary clarifiers where solids are settled out. The settled sludge is returned back to the anoxic zone as RAS. A small portion of this settled sludge is wasted daily to control the solids inventory to maintain a desirable level suitable for treatment.

Biological Reactor basins

There are two plug flow biological reactor basins. Each reactor basin is subdivided into 7 smaller compartments (zones). Zones 1 through 3 are pre-anoxic and anoxic zones while zones 5 through 7 are aerobic zones. Zone 4 is a swing zone, i.e., can serve as either aerobic zone or an anoxic zone. A mixture of primary effluent and RAS normally enter zone 1 and exit zone 7. Air is provided for each aerobic zone by mechanical blower and fine bubble diffusers. Submersible mixers are provided for each anoxic zone for mixing. At the end of the last aerobic zone (zone 7), some of the mixed liquor is returned to first anoxic zone (zone 1) to facilitate the denitrification process; the rest continue to the secondary clarifiers.

Secondary Clarifiers

There are currently two circular clarifiers with suction header mechanisms. Each clarifier includes modern design features such as energy dissipating inlet, flocculation wells, scum baffles and a scum collection system.

RAS Pump Station

The RAS pump station includes three centrifugal screw pumps: two duty and one stand by. Each duty pump is connected to the suction header of the corresponding secondary clarifier. The RAS pumps withdraw the settled sludge from the bottom of the secondary clarifiers and discharge it to the head of the reactor basins (zone 1, the first anoxic zone). The RAS pump station structure is located northwest of the secondary clarifiers.

5.4.6 Tertiary Filtration Facilities

Effluent from the secondary clarifier (secondary effluent) is pumped to the tertiary filters where secondary effluent suspended solids are removed. The tertiary filtration system consists of ten filter cells. Each cell is 11 feet in diameter and 16.5 feet high filled with 48-inches of 1.5 mm anthracite coal. In each filter cell, the wastewater flows downward through the filter medium. Solids particles are trapped on and within the filter medium, while the clean liquid stream passes through the filter bed. Polymer is added to the filter influent as a filtration aid, as needed.

As solids accumulate on and in the filter medium, the head loss will increase and the liquid level in the filter cell will rise. At a certain maximum level or after a set operation time, the filter is backwashed to remove the accumulated solids. Backwash is accomplished by the upward flow of air and water through the medium. The combined action of the air and water act to churn the medium and flush out the accumulated solids, which are then recycled with the backwash water to the plant headworks. After the combined air and water backwash, the air is turned off, the water flow is continued to purge air bubbles from the underdrain compartment and the filter medium.

The filtration system consists of filter feed pumps, polymer feed facilities, a backwash water supply, a backwash air supply system, and backwash water return basin, in addition to the filter cells themselves.

Filter Feed Pumps

The filter feed pump station was upgraded in1992. The upgrade included modifications to the filter feed pump station structure and the replacement of impellers on two of the filter feed pumps. In 1999, a fourth filter feed pump was added. The maximum (reliable) capacity of the filter feed pump system with three pumps in operation and pumping to the filters is estimated at approximately 7.0 Mgal/d.

Polymer Feed Facilities

The polymer feed facilities consist of redundant storage and metering pump facilities that can flow pace operator selected coagulants to enhance the filtration and coagulant process.

Tertiary Filters

The original tertiary filters were replaced with newer units in 1992 and three new filter units were added as part of the 1999 project for total of 10 filter units. According to the WWTP O&M manual, using the maximum design loading rate of 6.0 gpm/sf, the existing filters could treat up to 7.04 Mgal/d with one unit in backwash, and up to 7.82 Mgal/d with all units in operation.

The existing filter cells are showing signs of corrosion on inside and outside surfaces and should be recoated. Additionally, even though the filter media has been added as needed it has not been replaced since the filters were originally installed (1999 for the most recent filter addition). It is recommended that filter media be replaced once every ten years.

Filter Backwashing

Filter backwashing is accomplished by the upward flow of air and water through the medium. A mixture of water, air, and filter media deposits is sent to the backwash water return basin from where it is drained into the drain pump station. The filter backwash system consists of a backwash water supply basin, backwash supply pumps, backwash air supply blowers, backwash water return basin, and backwash return pump station.

5.4.7 Effluent Disinfection

Prior to 2008, Grass Valley WWTP final effluent disinfection was accomplished using gaseous chlorine. The system consisted of three chlorine contact basins and chlorine storage and feed facilities. In 2008, the plant made a transition to UV disinfection. A portion of the existing chlorine contact basins was converted to accommodate the open channel UV disinfection system. Two of the existing three channels were equipped with Trojan 3000 Plus UV disinfection systems. Each channel has three banks of UV lights with one bank serving as standby. Each bank contains seven modules with eight lamps each for a total of 336 lamps. The system is designed for a theoretical 70% transmittance and an operating transmittance of 75% or higher.

The existing UV system has sufficient capacity to disinfect the current equalized peak flow of 7.5 Mgal/d. The existing UV system and UV channels are in good condition and operate well and no improvement is needed at this time.

5.4.8 Solids Handling Facilities

Under normal operations, WAS is pumped from the secondary clarifiers to the gravity belt thickener where WAS is thickened to an approximate concentration of 50,000 mg/L (5% solids). Due to the relatively high man-hour demands, the current solids handling operation at the WWTP does not incorporate the gravity belt thickener. Instead, WAS is pumped to the primary clarifiers where it is mixed with the influent. In the primary clarifiers WAS is co-settled with primary sludge.

A mixture of WAS or (thickened WAS) and primary sludge is fed from the primary clarifiers to the anaerobic digester where solids are stabilized under anaerobic conditions. As new solids are fed to the digester, the same volume of digested solids is removed from the digester and sent to the sludge storage lagoon. Periodically the sludge lagoon is decanted with decant being sent directly to the head of the plant or to the filtrate holding tank. Sludge from the sludge lagoon is dewatered using a belt filter press. Filtrate from the belt filter press is pumped to the filtrate holding tank where it is stored and gradually returned back to the head of the plant. The WWTP currently produces Class B biosolids which are taken by Synagro and land applied.

5.5 PEAK FLOW MANAGEMENT MEASURES

5.5.1 Relationship between Secondary Treatment Hydraulic Capacity and Flow Equalization

The heart of the wastewater treatment plant and the key feature that determines plant capacity is the secondary treatment system, which includes the biological reactor basins and the secondary clarifiers. Once the secondary treatment capacity is determined, improvements to other unit processes will be discussed. The original plant is designed for an average dry weather flow of 2.78 Mgal/d with peak daily flow of 12 Mgal/d. However, the secondary treatment was designed for an equalized flow of 7.0 Mgal/d. Filters and disinfection processes are also designed for approximately 7.0 Mgal/d. Any flow in excess of the equalized 7.0 Mgal/d is diverted to the 6.1 Mgal equalization storage basin. After the plant upgraded in 1999/2000, upsizing of the main trunk sewer was undertaken as described previously to alleviate sewer overflows from the collection system. As a result of these interceptor upgrades, the wastewater is conveyed to the treatment plant much faster than before and the peak flow to the plant has increased significantly. Currently, the ratio of the peak day flow to the ADWF is 7.7, which is much higher than the original design ratio of 4.3 (12/2.78).

Biologically, the WWTP can still handle an average flow of 2.78 Mgal/d (the design ADWF) with a peak flow through secondary treatment of up to 7.0 Mgal/d. However, from a hydraulic perspective, as indicated above, the wet weather flow is more severe than the original design anticipated. The influent flow must be either treated immediately through the plant or equalized in storage until it can be treated later. The more flow that can be processed through the plant, the less equalization volume required. **Figure 5-11** shows the relationship between the equalization volume available and the maximum flow that can be treated through the plant. Each curve in the family of curves represents an ADWF with an associated wet-weather flow (5-day storm that has a return period of 10 years). The horizontal line represents the current equalization volume (6.1 Mgal) and the vertical line represents the current maximum flow that can be passed through the plant (7.0 Mgal/d). The capacity of the plant (in ADWF) is determined by looking up the closest curve to the point of intersection between the horizontal line and the vertical line. The graph shows that currently, the plant ADWF hydraulic capacity is about 1.4 Mgal, which is very close to the current flow (1.3 Mgal/d).

Assuming that the plant's biological capacity is not limiting, the hydraulic capacity of the plant (in ADWF) can be increased by either providing more equalization volume (which corresponds to moving the horizontal line up in **Figure 5-11**) or allowing more equalized flow to pass through the plant (which corresponds to moving the vertical line to the right in **Figure 5-11**), or both.

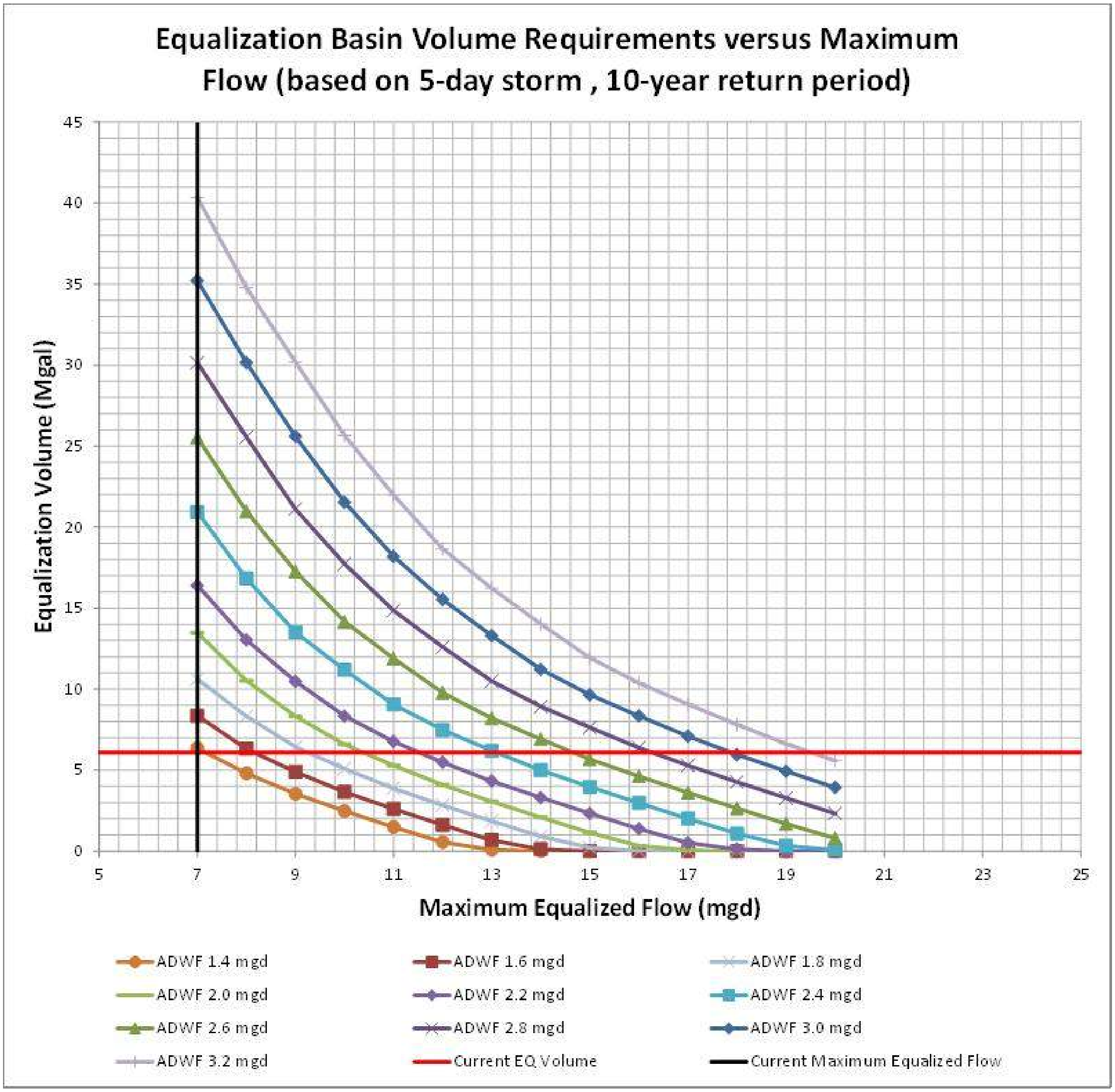


Figure 5-11 Relationship Between Equalization Volume and Equalized Flow

5.5.2 Site Limitations and Alternatives Development to Restore Secondary Treatment Capacity

The peak wet weather flows experienced at the WWTP indicate that the collection system is in need of rehabilitation to minimize inflow/infiltration (I/I). An on-going collection system rehabilitation program must be initiated to effectively control the I/I. If the I/I is not adequately controlled and the peak flow to the plant cannot be handled, then more equalization volume is needed. The wastewater treatment plant site is very tight with limited space to add more equalization storage or other process improvements. Ideally, equalization basins should be located near the WWTP. However, equalization storage could possibly be added within the collection system in strategic location(s). In wet-weather flow events, wastewater can be diverted to the equalization basin(s). When the flow subsides, the stored volume can be returned to the collection system and conveyed to the WWTP for treatment.

Pushing more flow through the current secondary treatment system could be done by upsizing the pipes from the aeration basins to the secondary clarifiers splitter box and upsizing the pipes from the secondary clarifiers to the tertiary filter pump station. However, this approach is very difficult due to the fact that the piping corridors are very congested and that the plant has to remain in service during construction. Additionally, the filter system and disinfection capacity will have to be augmented.

Another alternative is to utilize a physical/chemical side-stream treatment system to treat short-term peak flow conditions that result from wet-weather events. The system will come online when the influent flow is in excess of the secondary treatment capacity and the available equalization volume is full.

The side-stream treatment system could include ultrafiltration for removal of suspended solids (TSS), granular activated carbon (GAC) for removal of soluble organic material (BOD), and zeolite ammonia removal (ZAR) for the removal of nitrogen in the ammonium form.

In summary, the four measures to be considered to address hydraulic capacity constraints in the WWTP are:

1. Rehabilitate the existing collection system to reduce I/I
2. Provide more equalization storage volume
3. Improve plant hydraulics to push more flow (>7.0 Mgal/d) through secondary treatment and tertiary system)
4. Provide Side-stream treatment for peak flow

5.5.3 Rehabilitate the Existing Collection System to Minimize I/I

EPA considers a wet weather flow of more than 275 gallons per capita per day (gpcd) excessive for a sewer collection system. The wet weather flow per capita for Grass Valley is approximately 780 gpcd (10 Mgal/d / 12,700 people), which is higher than the referenced EPA threshold. Rehabilitation of collection systems can result in reduction in inflows. The City is pursuing a program of I/I reduction including limited replacement of sewer service laterals and rehabilitation of significant portions of the existing collection system piping and manholes. The City is also pursuing grant funds to subsidize these planned improvements. Further studies must be conducted to quantify excessive inflow and evaluate alternative corrective measures to further reduce it, as well as to assess the effectiveness of completed projects in actually reducing I/I in the system. Based on the results of these studies the next, most cost-effective sewer rehabilitation project will be identified and implemented, and the process repeated when the next project is completed. These iterative evaluations must constantly be reassessing the expected cost effectiveness of further I/I rehabilitation projects compared to the cost of capacity improvements at the WWTP itself. The treatment plant must be able to handle the inflow that cannot be cost effectively removed as I/I, but the cost to treat this I/I must be weighed when deciding the best course of action.

5.5.4 Provide More Equalization Volume

The wastewater treatment plant site is tight and has limited opportunities to add more equalization storage. Possible locations to add more storage volume would be the shooting range west of the secondary clarifiers, a nearby parking area (Northstar Powerhouse Museum), and the animal shelter. At the gun range location up to 1.5 Mgal of tank storage can be constructed. Another option would be to raise the levees of the existing equalization basins to add about 3.0 Mgal of storage. If larger equalization volume is required, then, off-site equalization would be necessary. Multiple steel bolted tanks within the collection system might be beyond these two options, provided each tank will be equipped with feed pump and odor control system. In peak flow events, sewage will be pumped or diverted by gravity from key manholes to the equalization tank. When the peak flow subsides, the stored sewage will be gradually returned to the collection system. Other options would be to construct a concrete underground tank under the parking lot of the Northstar Powerhouse Museum near the WWTP. An off-site equalization alternative is likely to be cost prohibitive compared to I/I reduction and WWTP Improvements.

5.5.5 Improve Plant Hydraulics to Push More Flow through Secondary Treatment

Currently, the hydraulic drop between the clarifier splitter box and the filter feed pump station is very low, limiting the maximum flow through the secondary process. The following elevations are retrieved from the 1999 upgrade project construction documents:

- Max water surface elevation in filter feed pump station is 27.33
- Clarifier effluent weir (top) 27.78
- Clarifier splitter box (invert) 29.5
- Aeration basin outlet weir (top) 33.0

Based on these elevations and the existing pipe diameters, the maximum flow through the secondary process is 7.0 Mgal/d. This maximum flow can be increased to 10 Mgal/d with the following upgrades:

- Upsize common secondary effluent pipe from wye to filter feed PS – from 30-inch to 42-inch (225 feet long).
- Upsize secondary effluent pipe from secondary clarifier #1 to wye – from 24-inch to 30-inch (140 feet long).
- Modify /Construct new secondary clarifier splitter box: Raise weir 1 feet Weir @ EL 30.50 and change the 4 feet straight weir to 8-10 feet folded weir.
- Upsize common mixed liquor pipe from aeration basin to clarifier splitter box from 30-inch to 36-inch (210 feet long).

5.5.6 Provide Storm Flow Treatment for Peak Flow

One possible side stream storm flow treatment solution is stormBLOX™. The stormBLOX™ Technology (by Ovivo) is a complete, membrane-based physical-chemical treatment process designed to treat storm flows. Excess flows greater than the design maximum capacity of the WWTP would be sent to the stormBLOX™ system, which can come on-line instantaneously. Alum would be added to raw, screened influent and fed directly to the UF (Ultra Filtration) membranes where total suspended solids, viruses, and bacteria (of a certain size and larger) are removed. The UF system is a vacuum driven process utilizing 0.03 mm PVDF membranes that operate at high flux and low energy (no air scour). The filtered effluent would pass through activated carbon for BOD removal and Zeolite for ammonia removal. The sizes of the carbon and zeolite media depend on the frequency and magnitude of the peak flow events. The frequency/magnitude of peak flows vary significantly from year to year. Based on historical data, it is assumed that the system would be designed to treat 10 peak events per year, each event being 6.0 Mgal/d sustained for 2 days. A proposal was solicited from Ovivo on a stormBLOX™ system with these design criteria and is included in **Appendix G**.

5.6 ALTERNATIVE ANALYSIS FOR WWTP UPGRADE

5.6.1 Headworks

Capacity Assessment

The existing headworks has two mechanical screens with a total combined capacity of 24.0 Mgal/d and one manual bar screen. The existing headworks is limited to a maximum peak flow of 24.0 Mgal/d assuming that both units are operated. In case one unit is out of service the excess flow would have to be sent through the third channel equipped with a manual bar screen. The existing grit removal system is a 12-feet vortex system, possibly rated for peak flow of 12 Mgal/day with no redundant unit. The headworks capacity assessment for future flow conditions as presented in Chapter 3 is summarized in **Table 5-7**.

Table 5-7 Headworks Capacity Assessment

Headworks Unit Processes	Approximate Existing Capacity Mgal/d	Peak Hour Flow, Mgal/d or Various Growth Scenarios			
		Vacant Parcels within City Limit (ADWF 1.6 Mgal/d)	Near Term (ADWF 1.9 Mgal/d)	Long Term (ADWF 2.1 Mgal/d)	Area of Concern (ADWF 4.0 Mgal/d)
Headworks Screens	24.0	20.5	21.8	23.0	39.7
Grit Removal System	12				
Parshall Flume	32.6				

*Parentheses denote capacity deficit

**The Parshall flume is currently limited with instrument calibration.

The existing headworks screens have sufficient capacity to process projected peak flows up to the projection of long term growth projection (ADWF of 2.1 Mgal/d). To upgrade the plant to ADWF of 4.0 Mgal/d, as indicated by the flow estimate presented for Area of Concern in **Table 5-7**, additional screening capacity of 15.7 Mgal/d will be required.

Even though the grit removal capacity is less than the expected peak hour flow, higher flow will reduce the efficiency of the grit removal system which can be tolerated during a storm event. Alternatively, the grit removal can be temporarily bypassed during a storm event.

The influent Parshall flume at the plant has a 36-inch wide throat. Under normal operation, without submergence, it is capable of measuring flows from 0.4 to 32.6 Mgal/d. The Parshall flume capacity is adequate for the long term growth projection (ADWF of 2.1 Mgal/d). However, the level sensor is calibrated to measure flows only from 0 to 10 Mgal/d, which is the typical flow range for the plant. Calibration to a higher range is required. Further, downstream hydraulics can significantly limit the range and accuracy of the flume. A new meter system is recommended.

5.6.2 Primary Clarification

Capacity Assessment

The primary clarification system consists of two basins; each 90 feet long and 12 feet deep. Basin No.1 is 18 feet wide whereas basin No.2 is 36 feet wide. The total combined surface area of both primary clarifiers is 4,860 square feet with total combined volume of 58,320 cubic feet. Based on a typical recommended overflow rate of 3000 gpd/sf, the capacity of the existing clarifiers is about 14.6 Mgal/day. The existing primary clarification capacity versus what is required in future conditions is summarized in **Table 5-8**.

Table 5-8 Primary Clarification Capacity Assessment

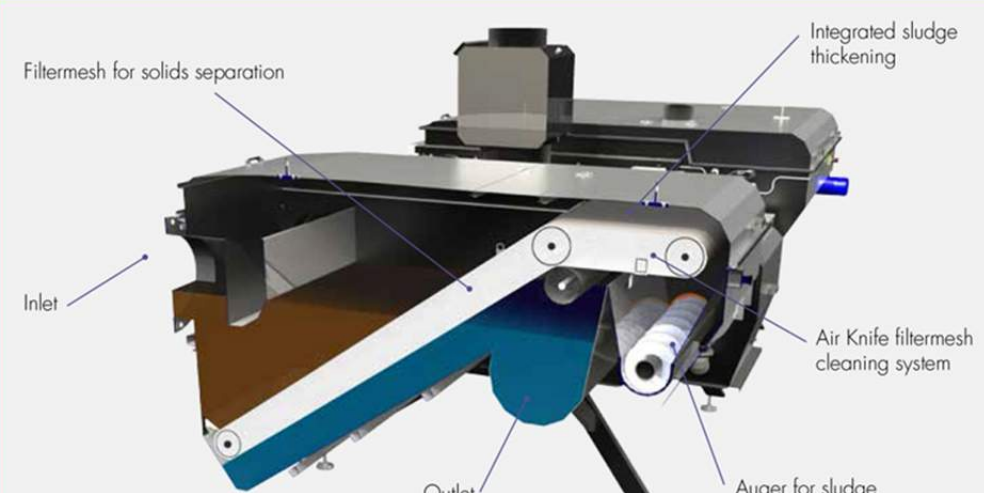
Existing Capacity Mgal/d	Peak Hour Flow, Mgal/d				
	Existing Capacity Need (Mgal/d)	Vacant Parcels within City Limit (ADWF 1.6 Mgal/d)	Near Term (ADWF 1.9 Mgal/d)	Long Term (ADWF 2.1 Mgal/d)	Area of Concern (ADWF 4.0 Mgal/d)
14.6	18.9	20.5	21.8	23.0	39.7

Different options available to meet the minimum design at future growth horizons include:

- **Add additional primary clarifiers.** Ideally new clarifiers would be constructed parallel to the existing clarifiers, however due to site limitations this option is not likely to be feasible without major site improvements. Alternatively, compact design of primary clarification can be employed. Salsnes filters (one of the major manufacturers of this technology) offer units that are comparable to primary clarifiers at 10 percent of the footprint. The largest unit rated for treatment of up to 3.65 Mgal/d with a footprint of 10-ft by 10-ft could be placed in close proximity to the primary clarifiers; however this option would require extensive piping modifications and would likely take valuable space.

  Both primary clarifiers and the Salsnes filter use separation for the treatment technology. While there are many differences, the fundamental difference is that separated primary solids are sent to the anaerobic digester for stabilization and disposal; the filter wastes are compacted and discarded to a dumpster for disposal.

Salsnes Filter



Cross Section of Salsnes Filter

- **Divert excess flow to the equalization basin ahead of primary clarifiers.** Ideally, it is preferred if the excess flow stored in the equalization basin first passes through primary clarification. However, it is reasonable to divert excess flow ahead of the primary clarifiers and store screened raw wastewater. In this case, an automation of the diversion gate ahead of the primary clarifiers would be required. This gate would automatically open when the level in the primary clarifier reaches an unacceptable level and then modulate to keep the desired level in the primary clarifier.

Primary Sludge Pumps

There are two primary sludge pumps, one duty and one standby. Each pump has a total capacity of 125 gpm at 92 feet of TDH. Based on estimated preliminary sludge and scum quantities and assuming that primary sludge is pumped for 5 minutes every one hour, the existing pumps are sufficient for the current primary clarifiers.

5.6.3 Primary Effluent Pump

The primary clarifier effluent pumps were part of the original construction and there have been no major improvements to the primary effluent pump station. In 2000 when the activated sludge secondary process was added, the primary effluent piping was modified to allow flow to be sent to the new secondary treatment facilities; however no improvements to the primary effluent pumps were made at that time. There are currently four primary effluent pumps. Each pump is capable of pumping 1,700 gpm at 20 feet TDH (Total Dynamic Head) providing a reliable pumping capacity of 7.34 Mgal/d with one unit out of service. The amount of flow pumped to the secondary treatment process is directly related to the hydraulic capacity of the secondary treatment process. Currently, the primary effluent pumps are appropriately sized for the capacity of the existing secondary treatment system. If the capacity of the secondary treatment system is increased, the primary effluent pumps will need to be upsized accordingly.

5.6.4 Equalization Diversion Pipe/Pumping

The original design capacity of the equalization diversion pipeline from the end of the primary clarifier to the equalization storage basin is 9.69 Mgal/d.

The required capacity of this pipeline should be the difference between the peak instantaneous flow (peak hour flow) and the maximum flow that can be handled by the secondary treatment process. This subject is discussed further in the secondary treatment discussion (Section 5.6.5) and the overall alternative analysis (Section 5.6.9).

Diversion of flows higher than about 6.0 Mgal/d to the existing equalization basin will require upsizing the existing diversion pipe or adding a pump station to pump extra flow through the existing pipe. Upsizing the equalization pipeline will be very difficult and expensive due to the very tight working space. The evaluation of pumping versus gravity to the existing equalization basin will need to be evaluated before final design.

Pumping to any new equalization basin(s) will require pumping either to or from the equalization basins/tanks. The cost of pumping to/from any new equalization basin is included with the equalization basins cost presented in the secondary treatment process discussion (Section 5.6.5) below.

5.6.5 Secondary Treatment

As previously indicated, the WWTP is designed for an average dry weather flow of 2.78 Mgal/d with peak daily flow of 12 Mgal/d. However, the secondary treatment was designed for an equalized flow of 7.0 Mgal/d. Even though the current ADWF (1.3 Mgal/d) is less than 50% of the design ADWF, the peak flows that the plant currently receives exceed the design peak flows. To restore the secondary treatment capacity lost to high I/I flows, different options were investigated. These options include:

1. Rehabilitate the existing collection system to reduce I/I
2. Provide more equalization volume
3. Improve plant hydraulics to push more flow through the secondary treatment process
4. Provide Side-stream treatment for peak flow

Each of these options is investigated as the sole measure to maximize the secondary treatment capacity in order to serve future growth. In reality, it may not be possible or may not be cost effective to adopt only one measure to increase the secondary treatment capacity. It may be more cost effective to implement more than one measure and to varying degrees. For example, **Table 5-9**, indicates that either 40% reduction in I/I or 4.0 Mgal of peak flow treatment capacity (such as stormBLOX™) will be required to serve the expected flow and load projected through the Long-Term growth scenario presented in Chapter 3. 40% reduction in I/I may be difficult to achieve cost effectively. Depending on the success of wastewater collection rehabilitation, a smaller or larger peak flow treatment option may be required. Different permutations of the above four measures were investigated and listed in **Appendix H**.

Table 5-9 Secondary Treatment Capacity Requirements of Different Phases

	Alternative 1 Adding equalization	Alternative 2 Reducing I/I	Alternative 3 Adding Peak Flow Treatment	Alternative 4 Improve Plant Hydraulics
Vacant Parcels within City Limit (ADWF 1.6 Mgal/d)	Add Extra 2.4 Mgal of Equalization Storage	Reduce I/I by 12.5 %	Add 1.5 Mgal/d of Storm Treatment System [a]	Improve plant hydraulics to push 10 Mgal/d through secondary treatment [b]
Near Term (ADWF 1.9 Mgal/d)	Add Extra 6.0 Mgal of Equalization Storage	Reduce I/I by 27 %	Add 3.0 Mgal/d of Storm Treatment System [a]	Improve plant hydraulics to push 10 Mgal/d through secondary treatment [b]

	Alternative 1 Adding equalization	Alternative 2 Reducing I/I	Alternative 3 Adding Peak Flow Treatment	Alternative 4 Improve Plant Hydraulics
Long Term (ADWF 2.1 Mgal/d)	Add Extra 8.9 Mgal of Equalization Storage	Reduce I/I by 40%	Add 4.0 Mgal/d of Storm Treatment System [(a)]	Improving plant hydraulics alone may not be adequate. A reasonable solution would be to improve plant hydraulics to push 10 Mgal/d through secondary treatment [(b)] and add 1.4 Mgal of Equalization storage.
Area of Concern (4.0 Mgal/d)	To increase the biological capacity of the secondary treatment process, adding a third clarifier and increasing the reactor volume by 30% (extension to the south), will be adequate to increase the ADWF capacity to 4.0 Mgal/d. However, the peak flow through the secondary must be limited to 11.0 Mgal/d. In reality, this phase will largely depend on the effectiveness of I/I reduction efforts and WWTP improvements undertaken in previous phases.			

(a) Analysis based on stormBLOX™ by Ovivo

(b) Upsize pipes to and from secondary clarifiers and the secondary clarifier splitter box to increase the secondary capacity to 10 Mgal/d, plus augment filtration and disinfection capacity.

5.6.6 Filtration

The available area for new construction at the existing wastewater treatment plant site is limiting. To reduce the area requirement, tertiary filtration alternatives with compact footprints were evaluated as alternatives to the existing deep media filters. Two tertiary filtration alternatives were evaluated to identify the most cost effective filtration option while considering space limitations of the existing site. These alternatives include Schreiber Fuzzy Filters and Cloth Media Disc Filters. With any filtration option, it is assumed that the additional peak flows would be processed using the new filters and the existing gravity filters would continue to be used in the future. When the existing filters reach the end of their useful life, they will be replaced.

Fuzzy Filters

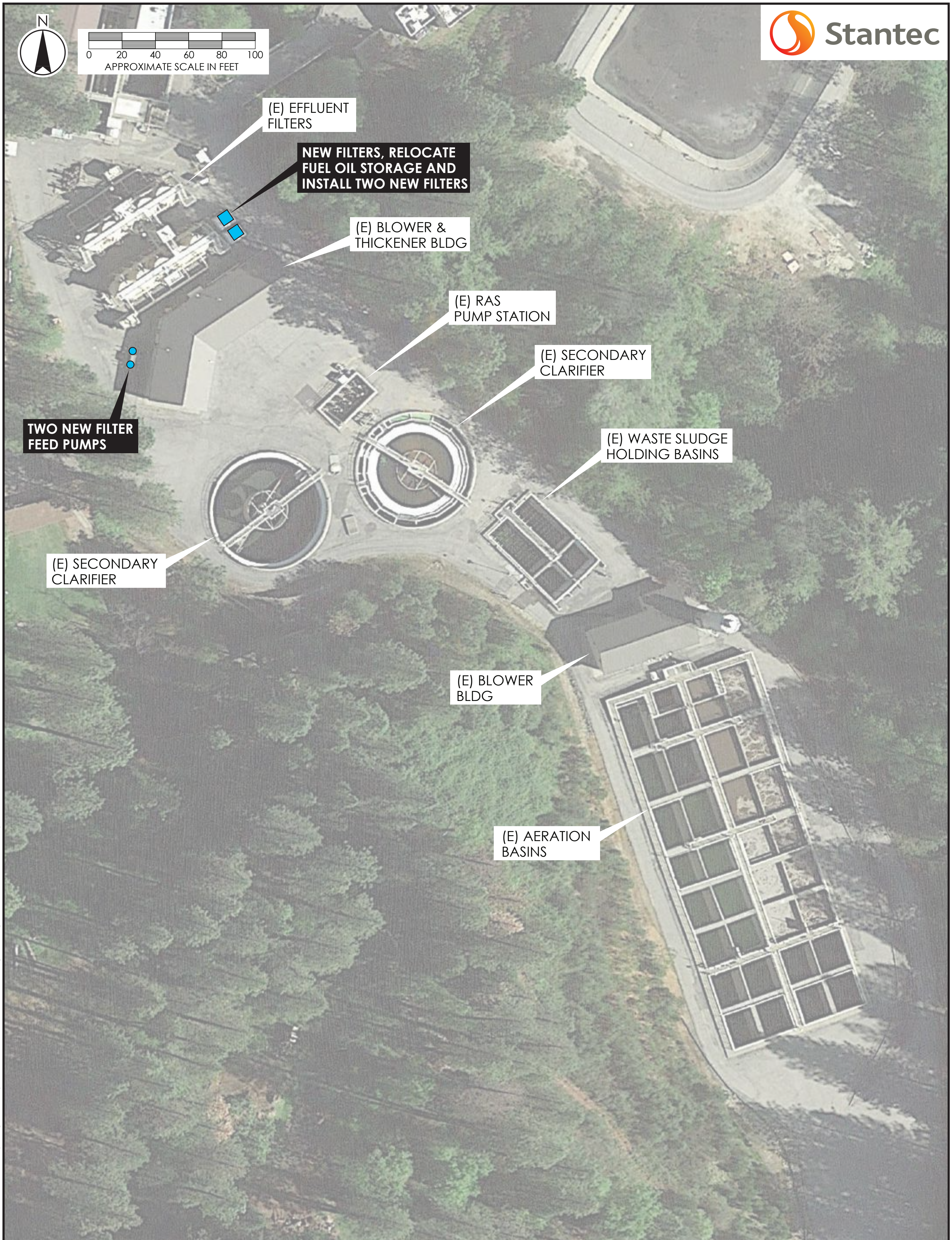
The Fuzzy Filter media bed is composed of compressible, porous, synthetic balls that are 30 mm (1.25 inches) in diameter and have a much higher porosity than conventional filter media. Unlike conventional media, the porous spheres are configured so that flow can occur through the spheres which increase flow through area. Another innovative characteristic of the Fuzzy Filter is that the filter media can be compressed using a movable top retainer plate. By varying media compression, the porosity and the effective pore size can be adjusted to accommodate different influent characteristics.

Because of its unique properties, the Fuzzy Filter has been shown to perform successfully at filter loading rates that are much higher than those of conventional filtration. Conventional filters are normally designed for loading rates from 3 to 8 gallons per minute per square foot (gpm/sq-ft). The Fuzzy Filter has been shown to perform effectively at loading rates between 10 and 40 gpm/sq-ft.

Cloth Media Disc Filters

Cloth media disc filters consist of filter fabric with a nominal pore size of 10 mm supported by open frame structures arranged in disks. During normal operation, the disks are submerged completely in the water. Water flows by gravity from the outside of the disks through the filter cloth into the center of the disks to a central collection header. As solids accumulate on the media, a mat forms on the surface, headloss increases, and the liquid level in the tank increases. When the water reaches a certain level (or after a set time), the backwash cycle is initiated.

A layout showing a potential location for additional filters is shown in **Figure 5-12**. With the existing filtration capacity, adding an additional 4.0 Mgal/d would provide adequate capacity for all phases considered.



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City of Grass Valley
Wastewater System Master Plan

Figure 5-12
Potential Location of the Additional Tertiary Filters

5.6.7 Disinfection

The existing UV system has two channels that are sufficient to disinfect flow of 7.5 Mgal/d. The UV channels are constructed within the old chlorine contact basin. Adding one more channel and UV lamps, will increase the disinfection capacity to about 10.5 Mgal/d. If less UV capacity is needed, fewer lamps will be required but the cost for constructing the channel will be essentially the same.

5.6.8 Solids Handling

Solids production flows and quantities for each of the future design conditions were estimated by performing solids mass balance calculations. It should be noted that solids loading estimates were calculated assuming no co-settling. Namely, as described earlier in this chapter, the primary clarifiers have limited capacity and any additional increase in flows will require the current practice of co-settling to be halted. Solids production calculations were developed assuming that the existing gravity belt thickener is operational and will be used to thicken WAS before it is fed to the digester.

Capacity requirements for WAS and filtrate equalization storage were estimated based on a minimum of three days of storage. Assuming that both the gravity belt thickener and the belt filter press will be operated only during week days, WAS storage basins and filtrate equalization need to have sufficient volume for weekend storage. For three day storage (Friday afternoon to Monday morning) the existing WAS storage basin has sufficient capacity for Long Term design flows and loads. At future conditions that correspond to 4.0 Mal/d ADWF, the WAS basin volume is sufficient to provide only 2 days of storage. To meet these design requirements either additional WAS storage volume would need to be provided, or the gravity thickener would have to be operated on one weekend day, or sludge wasting would have to be stopped once the WAS tank is filled. In that case WAS would have to be temporarily stored in secondary treatment process for the duration of up to one day. The filtrate equalization tank has sufficient capacity for design peak month flows and loads for all future conditions. If the filtrate equalization tank is repurposed for any other reason, such as influent equalization, the City must account for this needed storage volume.

Anaerobic digester capacity was evaluated based on typical volatile solids loading rates and typical SRT values. For a typical VSS loading of 0.12 lb VSS/cft/day, the existing digester has sufficient capacity to handle peak month loads that correspond to ADWF of 2.1 Mgal/d or Long Term design conditions. Similarly, at the same design condition, digester SRT at peak month flows is 22 days which is within the range of recommended operating values for mesophilic, single stage digesters. For peak month flows and loads that correspond to 4.0 Mgal/d ADWF (Area of Concern) the estimated volatile solids loading to the digester is 0.19 lb VSS/cft/day which exceeds the maximum recommended volatile solids loading. Similarly, digester SRT for the same design condition is 11 days which is not sufficient to achieve solids treatment that will produce Class B biosolids.

Options to expand the anaerobic digestion system to meet design flows and loads at future 4.0 Mgal/d ADWF (Area of Concern) include:

- Double the digester capacity. A new digester of the same size would be constructed next to the existing digester. The two digesters would be of the same size and would be operated in parallel. The Digester heating equipment including hot water boiler, hot water pumps, heat exchanges and recirculation pumps would be replaced with new equipment that can serve both digesters simultaneously.

- Convert system into two-phased digestion process. With this approach a new smaller digester would be added ahead of the existing digester. The new structure would serve as an acid phase digester with a design SRT of only 1 to 3 days. The existing digester would serve as a second stage digester (gas stage). Because the first stage, low SRT digester is favored by acid-forming bacteria, the second stage digester SRT can be reduced below 15 days which is typically needed for conventional single stage digesters. This alternative would also require modifications to the digester heating equipment to accommodate addition of a new first stage digester. The advantage of this option is that it has lower capital cost and requires less space. The disadvantage is that it does not provide process redundancy which would be the case with two parallel single stage digesters. Additionally, operation of a two-phased digestion system is more complex and provides increased control challenges, and acid-phase biogas has to be handled separately from the gas-stage digester.

Gravity belt thickener and belt filter press capacity sizing is a function of operating hours per week. Increases in flows and loads can be handled simply by increasing the number of operating hours. For design peak month flows and loads that correspond to 4.0 Mgal/d ADWF (Area of Concern) the gravity belt thickener and belt filter press will have to be operated 60 and 38 hours per week, respectively. Considering the likelihood of such design conditions, weekday operation of 12 hour/day for the gravity belt thickener and 8 hour/day for the belt filter press may be acceptable. However, with such operating loads on the equipment, redundant facilities are recommended for reliability.

Costs for future repair and maintenance, as well as labor and energy costs for extended operation should be considered as the City's service area grows. Technologies often improve and the conclusions regarding how the City may address servicing the Area of Concern described in Chapter 3, will be the subject of future City planning. Firm, detailed plans for these servicing needs are premature at this time.

5.6.9 Overall Alternative Analysis

In the previous sub-sections, every major component of the Grass Valley wastewater treatment and disposal system has been investigated to determine improvements needed to handle the design flows and loads established for different growth horizons in Chapter 3. In many cases, alternative analyses were completed to analyze several options for a particular unit process. However, a selection of the apparent best alternative for a particular part of the plant could not be made based solely on analysis of that process, because of interdependencies with other plant components and with possible improvements in influent flow conditions as a result of current and planned I/I reduction efforts. An overall alternative analysis is presented to assist in selection of the apparent best combination of components, considering all the interdependencies involved. The capacity of each unit process was executed in a methodical manner as illustrated in **Figure 5-13**. **Table 5-10** through **Table 5-12** present different options to upgrade the plant up to the Long Term conditions (2.1 Mgal/d). The upgrade to serve the Area of Concern (ADWF of 4.0 Mgal/d) will largely depend on the selected alternative(s) for the expansions which will occur prior.

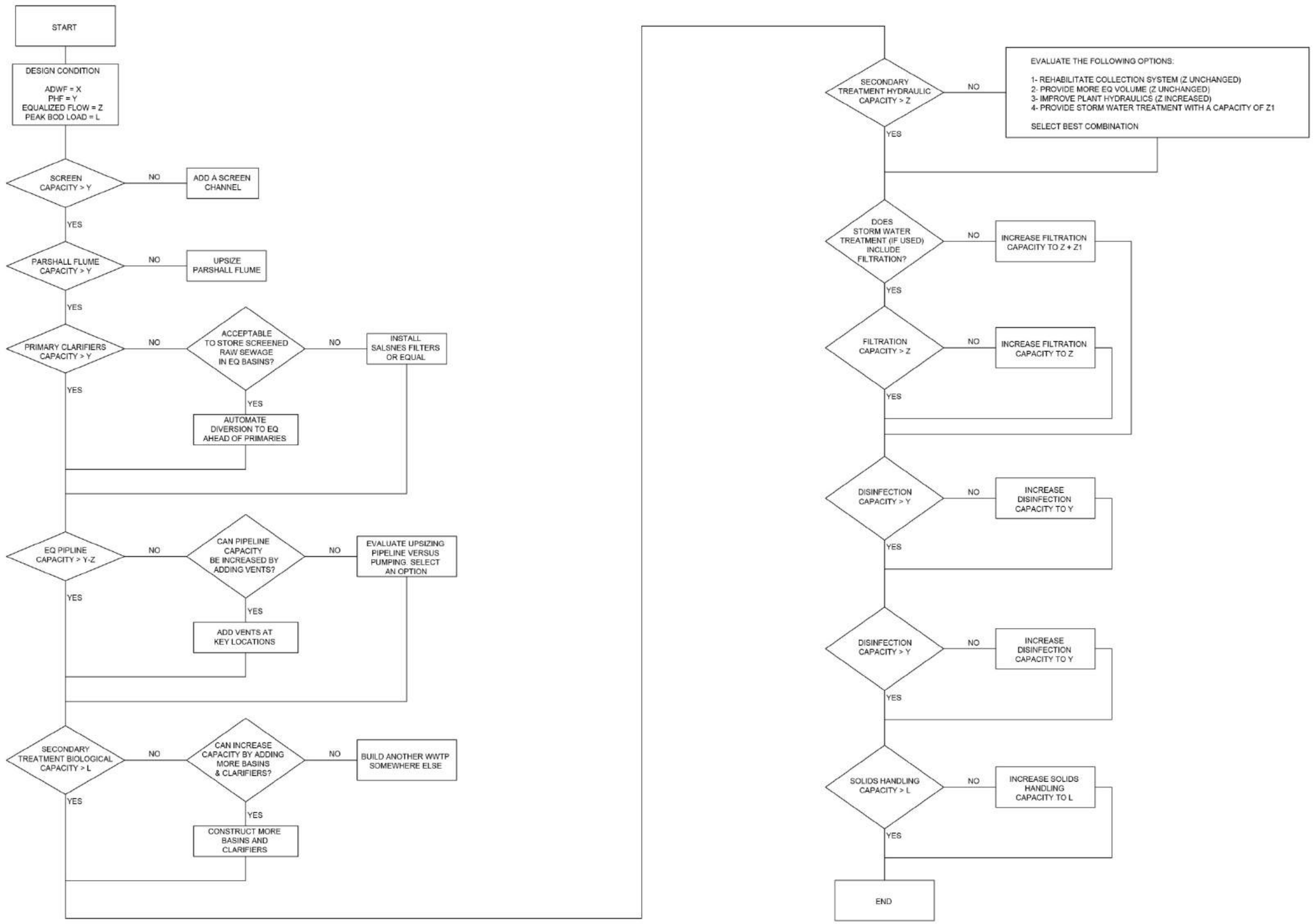

Figure 5-13 Illustration of Plant Expansion Rationale

Table 5-10 Alternative Analysis for Expansion to Serve Vacant Parcels within the City (ADWF of 1.6 Mgal/d)

	Alternative 1	Alternative 2	Alternative 3	Alternative 4
	Add Equalization	**Reduce I/I**	**Add Storm Water Treatment System**	**Improve Plant Hydraulics Through Secondary Treatment**
Secondary Treatment	Add Extra 2.4 Mgal of Equalization Storage	Reduce I/I by 12.5 %	Add 1.5 Mgal/d of Storm Treatment System[1]	Improve plant hydraulics to push 10 Mgal/d to secondary treatment[2]
Headworks	No Improvements Required			
Primary Clarifier	Automate diversion gate ahead of the primary clarifiers to send screened wastewater to Equalization			
Primary Effluent Pump	No Improvement Required	No Improvement Required	Additional Separate Pumps will need to be added for the Storm Treatment System	Primary Effluent Pumps will need to be upsized to pass 10 Mgal/d
Equalization Pipeline	Upsize Equalization Pipeline or Provide Pumps to deliver 13.5 Mgal/d to the EQ Basin	Upsize Equalization Pipeline or Pump to pass 11.3 Mgal/d to the existing EQ basin	Upsize Equalization Pipeline or Pump to pass 12.5 Mgal/d to the existing EQ basin	Upsize Equalization Pipeline or Pump to pass 10.5 Mgal/d to the existing EQ basin
Equalization Basin	Add 2.4 Mgal of Extra Equalization Storage	No Improvement Required	No Improvement Required	No Improvement Required
Filter Supply Pump Station	No Improvement Required	No Improvement Required	No Improvement Required	Filter Supply Pumps will need to be upsized to pass 10 Mgal/d
Tertiary Filtration	No Improvement Required	No Improvement Required	No Improvement Required	Filters will need to be upsized to pass 10 Mgal/d
Disinfection	No Improvement Required	No Improvement Required	Disinfection will need to be upsized to pass 8.5 mgal/d	Disinfection will need to be upsized to pass 10 mgal/d
Solids handling	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener

Table 5-11 Alternative Analysis for Near Term Expansion (ADWF of 1.9 Mgal/d)

	Alternative 1	Alternative 2	Alternative 3	Alternative 4
	Add Equalization	**Reduce I/I**	**Add Storm Water Treatment System**	**Improve Plant Hydraulics Through Secondary Treatment**
Secondary Treatment	Add Extra 6.0 Mgal of Equalization Storage	Reduce I/I by 27 %	Add 3.0 Mgal/d of Storm Treatment System(1)	Improve plant hydraulics to push 10 Mgal/d to secondary treatment(2)
Headworks	No Improvements Required			
Primary Clarifier	Automate diversion gate ahead of the primary clarifiers to send screened wastewater to Equalization			
Primary Effluent Pump	No Improvement Required	No Improvement Required	Additional Separate Pumps will need to be added for the Storm Treatment System	Primary Effluent Pumps will need to be upsized to pass 10 Mgal/d
Equalization Pipeline	Upsize Equalization Pipeline or Provide Pumps to deliver 14.8 Mgal/d to the EQ Basin	Upsize Equalization Pipeline or Pump to pass 10.4 Mgal/d to the existing EQ basin	Upsize Equalization Pipeline or Pump to pass 11.8 Mgal/d to the existing EQ basin	Upsize Equalization Pipeline or Pump to pass 11.8 Mgal/d to the existing EQ basin
Equalization Basin	Add 6.0 Mgal of Extra Equalization Storage	No Improvement Required	No Improvement Required	No Improvement Required
Filter Supply Pump Station	No Improvement Required	No Improvement Required	No Improvement Required	Filter Supply Pumps will need to be upsized to pass 10 Mgal/d
Tertiary Filtration	No Improvement Required	No Improvement Required	No Improvement Required	Filters will need to be upsized to pass 10 Mgal/d
Disinfection	No Improvement Required	No Improvement Required	Disinfection will need to be upsized to pass 8.5 mgal/d	Disinfection will need to be upsized to pass 10 mgal/d
Solids handling	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener

Table 5-12 Alternative Analysis for Long Term Expansion (ADWF of 2.1 Mgal/d)

	Alternative 1	Alternative 2	Alternative 3	Alternative 4
	Add Equalization	Reduce I/I	Add Storm Water Treatment System	Improve Plant Hydraulics Through Secondary Treatment
Secondary Treatment	Add Extra 8.9 Mgal of Equalization Storage	Reduce I/I by 40%	Add 4.0 Mgal/d of Storm Treatment System(1)	Improve plant hydraulics to push 10 Mgal/d to secondary treatment and add 1.4 Mgal of equalization storage.
Headworks	No Improvements Required			
Primary Clarifier	Automate diversion gate ahead of the primary clarifiers to send screened wastewater to Equalization			
Primary Effluent Pump	No Improvement Required	No Improvement Required	Additional Separate Pumps will need to be added for the Storm Treatment System	Primary Effluent Pumps will need to be upsized to pass 10 Mgal/d
Equalization Pipeline	Upsize Equalization Pipeline or Provide Pumps to deliver 16 Mgal/d to the EQ Basin	No Improvement Required	Upsize Equalization Pipeline or Pump to pass 12 Mgal/d to the existing EQ basin	Upsize Equalization Pipeline or Pump to pass 13 Mgal/d to the existing EQ basin
Equalization Basin	Add 8.9 Mgal of Extra Equalization Storage	No Improvement Required	No Improvement Required	Add 1.4 Mgal of Extra Equalization Storage
Filter Supply Pump Station	No Improvement Required	No Improvement Required	No Improvement Required	Filter Supply Pumps will need to be upsized to pass 10 Mgal/d
Tertiary Filtration	No Improvement Required	No Improvement Required	No Improvement Required	Filters will need to be upsized to pass 10 Mgal/d
Disinfection	No Improvement Required	No Improvement Required	Disinfection will need to be upsized to pass 8.5 mgal/d	Disinfection will need to be upsized to pass 10 mgal/d
Solids handling	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener	Repair / Refurbish Gravity Belt Thickener

6.0 IMPROVEMENT PLAN

In the preceding chapters, current and future regulatory issues as well as collection system and WWTP assessments were discussed. In this chapter, an improvement plan is suggested based on the information presented in chapters 4 and 5 that provides the City with an approach to address the current system limitations over the next few years.

6.1 PURPOSE AND SCOPE

The purpose of this chapter is to summarize an improvement plan that addresses the most critical deficiencies of the existing wastewater collection and treatment facilities identified in Chapters 4 and 5. The improvement plan includes improvement alternatives for existing users and development which may occur on vacant parcels within the existing City Limits. It does not include details of improvements required to serve future development outside of the current City Limits. The City intends to utilize the alternatives identified in Chapters 4 and 5 as the starting point for determining necessary system improvements. This improvement plan includes planning level costs for the proposed projects.

6.2 I/I REDUCTION MEASURES

As discussed throughout this document, the primary limitation of the City's collection system and Wastewater Treatment Plant (WWTP) is the hydraulic capacity. This is largely due to the inflow and infiltration (I/I) occurring within the collection system. In chapter 5, the discussion of alternatives to address WWTP capacity constraints is presented. Reduction of I/I in the collection system is one of the four alternative approaches considered. The I/I reduction will alleviate the capacity constraints for both the collection system and the WWTP. As a result, the City is currently planning to implement an I/I reduction project in targeted areas of the collection system. As part of the City's initial I/I reduction efforts, the City is planning to repair and rehabilitate approximately 11 miles of sewer and approximately 200 manholes. An increase in the capacity of the City's system is not anticipated to result from the proposed project; however, peak flow sewer capacity may be restored as I/I is reduced.

The cost of the City's planned initial I/I project is estimated to be approximately $5M. The City is planning to fund this project with a combination of City funds and approximately $4M in grant funding from the Clean Water State Revolving Fund. Although a recent investigation has been completed to isolate the most significant sources of I/I in the system, additional study/investigation is warranted in order to further isolate problem areas and maximize the benefit of available funds.

6.3 COLLECTION SYSTEM IMPROVEMENTS

The collection system capacity analysis presented in Chapter 4 describes the collection system upgrades required to meet the level of service (LOS) criteria that the City has set. Although these system deficiencies have been identified for the existing conditions, it should be recognized that reduction of I/I resulting from the project described above may reduce some of these capacity limitations and the related improvement costs.

The cost of implementing the proposed upgrades to eliminate existing collection system capacity constraints is anticipated to be in the range of $2.8 M and is summarized in **Table 6-1**.

Table 6-1 Opinion of Probable Cost for Improvements in Existing System

Pipeline Improvements (a)		
Diameter (inches)	**Length (feet)**	**Opinion of Probable Costs**
8 inch	2769	$700,000
10 inch	2420	$716,000
12 inch	615	$209,000
15 inch	304	$114,000
Pipeline Subtotal		$1,739,000
Environmental, Engineering, Construction Management, 30%		$521,700
Contingency, 30%		$521,700
	Subtotal	**$2,780,000**
Lift Station Improvements		
Slate Creek and Morgan Ranch Lift Station Upgrades		$40,000
Environmental, Engineering, Construction Management, 35%		$14,000
Contingency, 40% (b)		$16,000
	Lift Station Subtotal	**$70,000**
	Total	**$2,846,000**

(a) All costs assume a 12 foot depth and replacement of manholes every 250 feet at a cost of $20,000 each. Installation cost of 8-inch to 12-inch pipeline is calculated based on a cost of $18/linear foot/inch diameter. Installation cost of 15-inch pipeline is calculated based on a cost of $246/linear foot.

(b) Lift Station Improvements include additional contingency to allow for unknowns related to electrical systems and control components.

In addition to improvements that the City considers critical to address collection system capacity constraints, segments of the City's collection system are 80 to 100 years old. These segments of the collection system are considered to have reached the end of their useful life. In addition, the remaining components of the collection system continue to age and warrant replacement at the appropriate time to avoid significant portions of the system exceeding the useful life of the materials of construction. As such, the City proposes to continue its program of repair and replacement of collection system assets on a regular basis. Currently the City intends to fund approximately $200,000 of repair and replacement projects per year. Some of these aged collection system components are coincident with the needed improvements listed in **Table 6-1**, but not all. Some are unique resulting in repair and replacement projects in addition to those projects listed to alleviate capacity constraints.

As the City is designated a disadvantaged community under the State Water Board's guidelines for project financing, they can expect to qualify for some grant funds as they become available. As a result, rather than budgeting $200,000 per year for repair and replacement projects, the City expects it may pool repair and replacement projects into groupings with cost estimates greater than $200,000 to maximize the value the City can realize from available grant funds. This means the City does not expect to strictly adhere to a minimum annual repair and replacement program; rather the average of expenditures on such projects is expected to equal approximately $200,000 per year.

6.4 WWTP IMPROVEMENTS

Capacity limitations of the WWTP and proposed alternatives to improve the WWTP at different growth horizons are identified in Chapter 5. It is anticipated that the City will implement an integrated approach including I/I reduction and a combination of the proposed Chapter 5 alternatives in order to meet their treatment capacity requirements. This is expected to involve different aspects of the alternatives considered in chapter 5, drawing primarily from alternatives 1, 2 and 4. The City is not anticipating installation of high rate treatment units at this time to address existing peak flow treatment capacity needs, although they remain open to such solutions to address capacity needs for future growth occurring outside the City Limits.

Plant improvements that the City is considering are presented below and discussed in detail in Chapter 5. The timing and extent of implementation of these project components will be highly dependent on the effectiveness of the City's I/I reduction efforts.

- Automate Diversion gate ahead of the primary clarifiers to send screen wastewater to Equalization Basins
- Add additional Equalization Storage
- Upsize the Equalization pipeline or provide Equalization Pumps to increase flow to Equalization Basins
- Improve Plant Hydraulics through Secondary Treatment Process (10 Mgal/d, peak flow)
- Upsize filter supply pumps (10 Mgal/d, peak flow)
- Expand tertiary filter capacity (10 Mgal/d, peak flow)

- Expand UV system capacity (10 Mgal/d, peak flow)
- Repair/Refurbish the gravity Belt Thickener

Combined these additional upgrades are estimated to have an approximate total project cost of $6.8 M.

6.4.1 O&M and Facility Condition Considerations

The City's headworks screening facilities have sufficient hydraulic capacity to serve the existing users and expected in-fill growth within the City Limits, as described in Chapter 5. However, the screening equipment, solids removal and handling, and odor control facilities located at the headworks are aging. In addition, there are concerns with the level of operational effort (and cost) required to maintain the screens in operational condition. As such, depending on the availability of grant funds, the City may elect to replace the existing headworks in the near future.

The City's WWTP is equipped with one solids digester. There is no redundant unit in place to receive waste sludge when the existing digester must be taken off line for cleaning and repair. In the past this has created significant operational hardships for the City. Similar to the headworks facilities, the City may elect to opportunistically pursue grant funding for construction of a redundant anaerobic digester at the WWTP. It is possible that this facility may be sized such that it is not a fully sized replacement unit for the existing digester, but sized for a reasonable expected maintenance project time frame to control capital cost.

APPENDICES

Appendix A SANITARY SEWER FLOW MONITORING AND INFLOW/INFILTRATION STUDY, MAY 2014

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

City of Grass Valley

May 2014

V&A

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

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Prepared by

V&A

May 2014

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ABBREVIATIONS, TERMS AND DEFINITIONS USED IN THIS REPORT

Table i.
Abbreviations

Abbreviation	Term
ADWF	average dry weather flow
CCTV	closed-circuit television
CIP	capital improvement plan
CO	carbon monoxide
d/D	depth/diameter ratio
FM	flow monitor
gpd	gallons per day
gpm	gallons per minute
GWI	groundwater infiltration
H_2S	hydrogen sulfide
I/I	inflow and infiltration
IDM	inch-diameter-mile (miles of pipeline multiplied by the diameter of the pipeline in inches)
IDW	inverse distance weighting
LEL	lower explosive limit
mgd	million gallons per day
NOAA	National Oceanic and Atmospheric Administration
Q	flow rate
RDI	rainfall-dependent infiltration
RRI	rainfall-responsive infiltration
RG	rain gauge
SSO	sanitary sewer overflow
WEF	Water Environment Federation
WRCC	Western Regional Climate Center

Table ii.
Terms and Definitions

Term	Definition
Average dry weather flow (ADWF)	Average flow rate or pattern from days without noticeable inflow or infiltration response. ADWF usage patterns for weekdays and weekends differ and must be computed separately. ADWF can be expressed as a numeric average or as a curve showing the variation in flow over a day. ADWF includes the influence of normal groundwater infiltration (not related to a rain event).
Basin	Sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. Also refers to the ground surface area near and enclosed by pipelines. A basin may refer to the entire collection system upstream from a flow meter or exclude separately monitored basins upstream.
Depth/diameter (*d/D*) ratio	Depth of water in a pipe as a fraction of the pipe's diameter. A measure of fullness of the pipe used in capacity analysis.
Design storm	A theoretical storm event of a given duration and intensity that aligns with historical frequency records of rainfall events. For example, a 10-year, 24-hour design storm is a storm event wherein the volume of rain that falls in a 24-hour period would historically occur once every 10 years. Design storm events are used to predict I/I response and are useful for modeling how a collection system will react to a given set of storm event scenarios.
Infiltration and inflow	Infiltration and inflow (I/I) rates are calculated by subtracting the ADWF flow curve from the instantaneous flow measurements taken during and after a storm event. Flow in excess of the baseline consists of inflow, rainfall-responsive infiltration, and rainfall-dependent infiltration. **Total I/I** is the total sum in gallons of additional flow attributable to a storm event.
Infiltration, *groundwater*	Groundwater infiltration (**GWI**) is groundwater that enters the collection system through pipe defects. GWI depends on the depth of the groundwater table above the pipelines as well as the percentage of the system that is submerged. The variation of groundwater levels and subsequent groundwater infiltration rates is seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
Infiltration, *rainfall-dependent*	Rainfall-dependent infiltration (**RDI**) is similar to groundwater infiltration but occurs as a result of storm water. The storm water percolates into the soil, submerges more of the pipe system, and enters through pipe defects. RDI is the slowest component of storm-related infiltration and inflow, beginning gradually and often lasting 24 hours or longer. The response time depends on the soil permeability and saturation levels.
Infiltration, *rainfall-responsive*	Rainfall-responsive infiltration (**RRI**) is storm water that enters the collection system through pipe defects, but normally in sewers constructed close to the ground surface such as private laterals. RRI is independent of the groundwater table and reaches defective sewers via the pipe trench in which the sewer is constructed, particularly if the pipe is placed in impermeable soil and bedded and backfilled with a granular material. In this case, the pipe trench serves as a conduit similar to a French drain, conveying storm drainage to defective joints and other openings in the system.
Inflow	**Inflow** is defined as water discharged into the sewer system, including private sewer laterals, from **direct** connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins. Inflow creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to

Term	Definition
	carry these peak instantaneous flows. Overflows are often attributable to high inflow rates.
Normalization	To run an "apples-to-apples" comparison amongst different basins, calculated metrics must be **normalized**. Individual basins will have different runoff areas, pipe lengths and sanitary flows. There are three common methods of normalization. Depending on the information available, one or all methods can be applied to a given project: ❖ Pipe Length: The metric is divided by the length of pipe in the upstream basin expressed in units of inch-diameter-mile (IDM). ❖ Basin Area: The metric is divided by the estimated drainage area of the basin in acres. ❖ ADWF: The metric is divided by the average dry weather sanitary flow (ADWF).
Normalization, *inflow*	The peak I/I flow rate is used to quantify inflow. Although the instantaneous flow monitoring data will typically show an inflow peak, the inflow response is measured from the I/I flow rate (in excess of baseline flow). This removes the effect of sanitary flow variations and measures only the I/I response: ❖ Pipe Length: The peak I/I flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). ❖ Basin Area: The peak I/I flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. ❖ ADWF: The peak I/I flow rate is divided by the average dry weather flow (ADWF). This is a ratio and is expressed without units.
Normalization, *GWI*	The estimated GWI rates are compared to acceptable GWI rates, as defined by the Water Environment Federation, and are used to identify basins with high GWI: ❖ Pipe Length: The GWI flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). ❖ Basin Area: The GWI flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. ❖ ADWF: The GWI flow rate is divided by the average dry weather flow (ADWF). This is a ratio and is expressed without units.
Normalization, *RDI*	The estimated RDI rates at a period 24 hours or more after the conclusion of a storm event are used to identify basins with high RDI: ❖ Pipe Length: The RDI flow rate is divided by the length of pipe (IDM) in the upstream basin. The result is expressed in gallons per day (gpd) per IDM (gpd/IDM). ❖ Basin Area: The RDI flow rate is divided by the geographic area of the upstream basin. The result is expressed in gpd per acre. ❖ ADWF: The RDI flow rate is divided by the average dry weather flow

Term	Definition
	(ADWF). This is a ratio and is expressed without units.
Normalization, *total I/I*	The estimated totalized I/I in gallons attributable to a particular storm event is used to identify basins with high total I/I. Because this is a totalized value rather than a rate and can be attributable solely to an individual storm event, the volume of the storm event is also taken into consideration. This allows for a comparison not only between basins but also between storm events: ❖ Pipe Length: Total gallons of I/I is divided by the length of pipe (IDM) in the upstream basin and the rainfall total (inches) of the storm event. The result is expressed in gallons per IDM per inch-rain. ❖ Basin Area (R-Value): Total gallons of I/I is divided by total gallons of rainfall water that fell within the acreage of the basin area. This is a ratio and is expressed as a percentage. R-Value is described as "the percentage of rainfall that enters the collection system." Systems with R-Values less than 5%[1] are often considered to be performing well. ❖ ADWF: Total gallons of I/I is divided by the ADWF and the rainfall total of the storm event. The result is expressed in million gallons per MGD of ADWF per inch of rain.
Peaking factor	Ratio of peak measured flow to average dry weather flow. This ratio expresses the degree of fluctuation in flow rate over the monitoring period and is used in capacity analysis.
Surcharge	When the flow level is higher than the crown of the pipe, then the pipeline is said to be in a **surcharged** condition. The pipeline is surcharged when the *d/D* ratio is greater than 1.0.
Synthetic hydrograph	A set of algorithms has been developed to approximate the actual I/I hydrograph. The synthetic hydrograph is developed strictly using rainfall data and response parameters representing response time, recession coefficient and soil saturation.
Weekend/weekday ratio	The ratio of weekend ADWFs to weekday ADWFs. In residential areas, this ratio is typically slightly higher than 1.0. In business districts, depending on the type of service, this ratio can be significantly less than 1.0.

[1] Keefe, P.N. "Test Basins for I/I Reduction and SSO Elimination." 1998 WEF Wet Weather Specialty Conference, Cleveland.

EXECUTIVE SUMMARY

Scope and Purpose

V&A has completed sanitary sewer flow monitoring and rainfall monitoring with inflow and infiltration (I/I) analysis within the City of Grass Valley (City). Flow and rainfall monitoring was performed over a period of approximately two months from February 6, 2014 to April 8, 2014 at eight open-channel flow monitoring sites and one rain gauge location. The purpose of this study was to measure sanitary sewer flows at the flow monitoring sites, estimate available sewer capacity and conduct analyses pertaining to infiltration and inflow (I/I) occurring in the basins upstream from the flow monitoring sites.

Site Flow Monitoring and Capacity Results

Peak measured flows and the flow level (depth) at peak flow times are important factors to consider in order to understand the capacity of the flow monitoring system. Table 1 summarizes the peak recorded flows, levels, *d*/*D* ratios, and peaking factors per site during the flow monitoring period. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the point site locations. Hydraulic conditions in other areas of the collection system will differ.

Table 1. Capacity Analysis Summary

Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	*d*/*D* Ratio	Level Surcharged above Crown (in)
Site 1	0.10	0.68	6.70	18	6.73	0.37	-
Site 2	1.24	9.14	7.38	30	13.27	0.44	-
Site 3	0.19	2.31	12.38	15	24.68	1.65	9.7
Site 4	1.04	6.98	6.70	24.875	19.72	0.79	-
Site 5	0.15	1.74	11.97	15	15.16	1.01	1.0
Site 6	0.15	1.06	6.95	7.25	10.41	1.44	3.5
Site 7	0.35	2.04	5.78	12	5.88	0.49	-
Site 8	0.33	1.82	5.54	15	5.86	0.39	-

The following capacity analysis results are noted:

- **Peaking Factor:** All of the sites had peaking factors higher than the common threshold value.
- ***d*/*D* Ratio:** Sites 3, 4, 5, and 6 had *d*/*D* ratios that exceeded common threshold values. Sites 3, 5, and 6 reached a surcharged condition during the study.

Figure 1 and Figure 2 show bar graphs summarizing the site by site peaking factors and d/D ratios, respectively. Figure 3 shows a schematic diagram of the peak measured flows with peak flow levels.

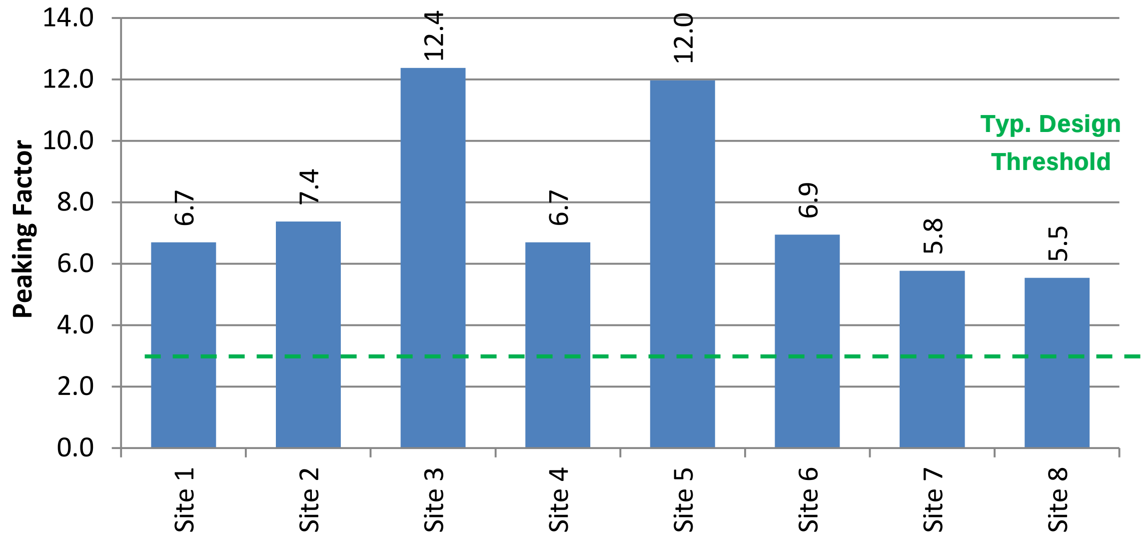


Figure 1. Capacity Summary Bar Graphs: Peaking Factors

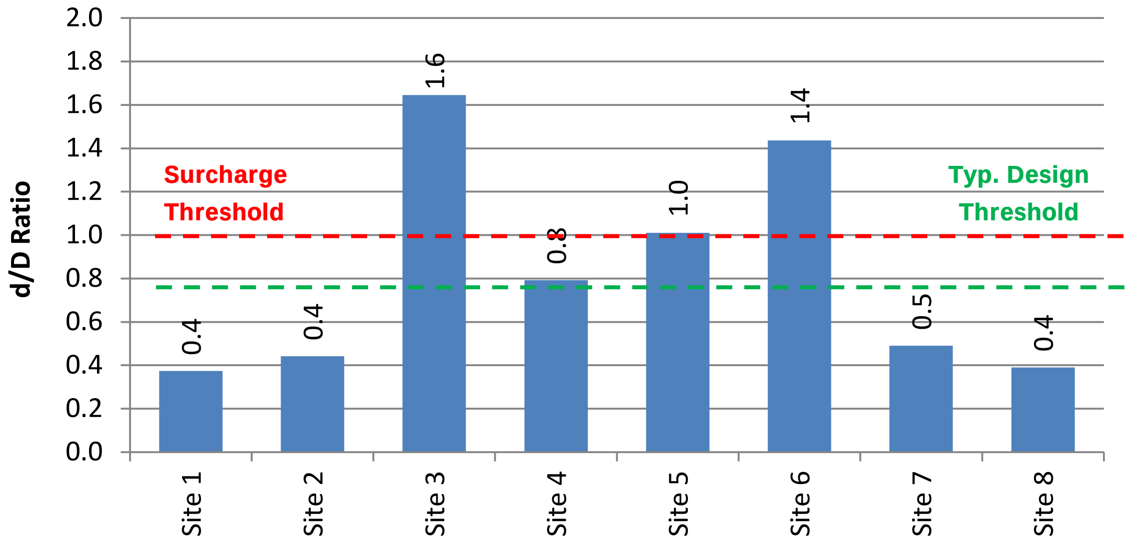


Figure 2. Capacity Summary Bar Graphs: d/D Ratios

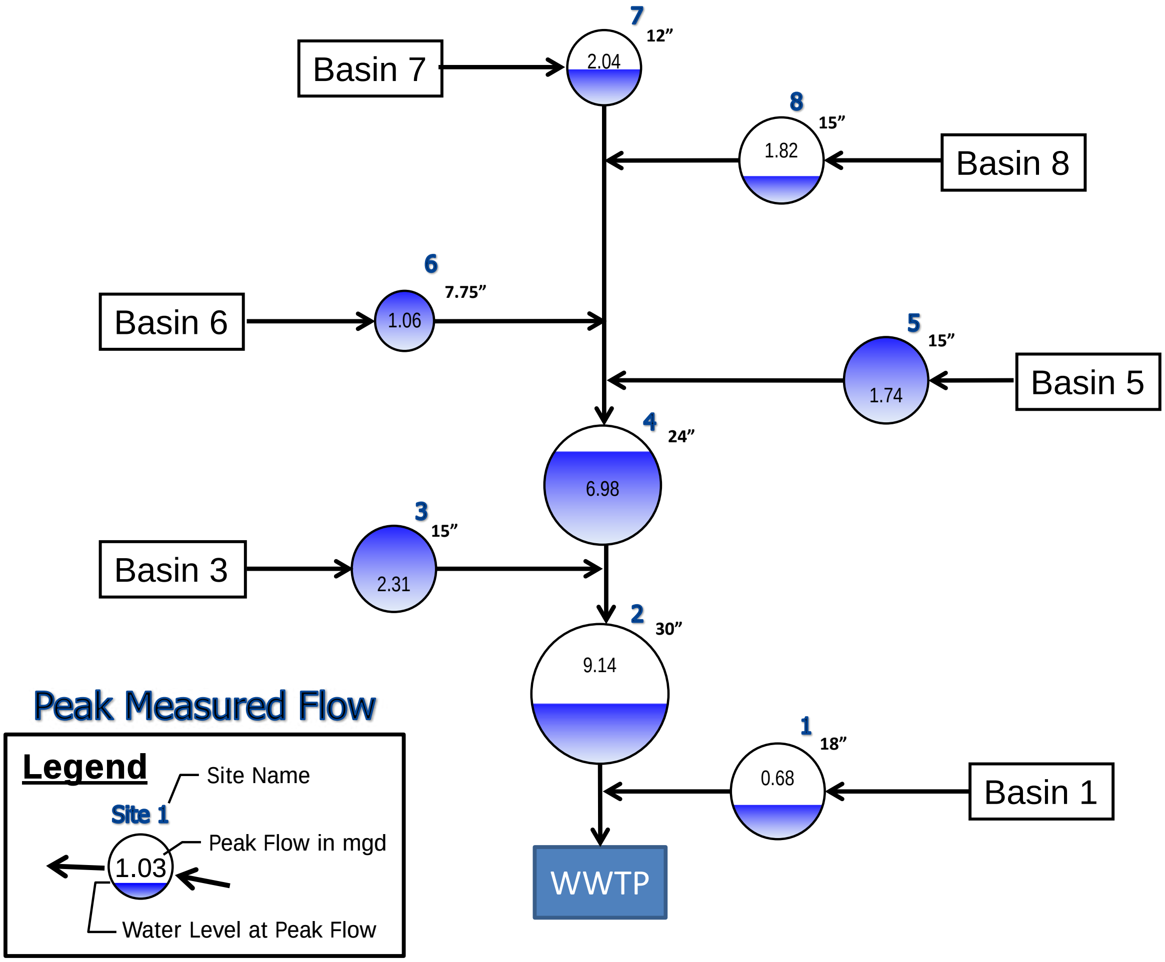


Figure 3. Peak Measured Flow Schematic

Basin Inflow and Infiltration Analysis Results

Table 2 summarizes the flow monitoring and I/I results for the flow monitoring basins that were isolated during this study. Infiltration and inflow rankings are shown such that 1 represents the highest infiltration or inflow contribution and 6 represents the least. The final I/I values and I/I analysis data were taken from the February 5 – 14, 2014 rainfall event. Refer to the *I/I Methods* section for more information on inflow analysis methods.

Table 2. I/I Analysis Summary

Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Total I/I (million gallons)	Inflow Ranking	RDI Ranking	Evidence of High GWI?	Combined I/I Ranking
Basin 1	0.102	0.648	0.761	3	6	No	6
Basin 3	0.187	2.190	3.372	1	1	No	1
Basin 5	0.146	1.659	2.478	2	2	Yes	2
Basin 6	0.152	0.961	1.597	4	4	Yes	3
Basin 7	0.353	1.836	3.477	5	3	Yes	4
Basin 8	0.328	1.577	2.948	6	5	Yes	5

Recommendations

V&A advises that future I/I reduction plans consider the following recommendations:

1. **Determine I/I Reduction Program:** The City should examine its I/I reduction needs to determine a future I/I reduction program.
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow was occurring in Basins 3 and 5.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems.
        i. The highest normalized rainfall-dependent infiltration was occurring in Basins 3 and 5.
        ii. The highest groundwater infiltration was occurring in Basins 5, 6, 7, and 8.
2. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Nighttime reconnaissance work to (1) investigate and determine direct point sources of inflow and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
3. **I/I Reduction Cost-Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow and infiltration and systematically rehabilitating or replacing the faulty pipelines or (2) continued treatment of the additional rainfall-dependent I/I flow.

    Downstream Pipe Capacity Analysis: High levels of inflow resulted in peak flow problems at Sites 3, 5, and 6 where surcharged conditions occurred. If bigger storm events occur, this issue can become more severe and can result in a sanitary sewer overflow (SSO). Pipeline capacity issues within the local collection system should be analyzed to minimize the potential for SSOs

INTRODUCTION

Scope and Purpose

V&A has completed sanitary sewer flow monitoring and rainfall monitoring with inflow and infiltration (I/I) analysis within the City. Flow and rainfall monitoring was performed over a period of approximately two months from February 6, 2014 to April 8, 2014 at eight open-channel flow monitoring sites and one rain gauge location. The purpose of this study was to measure sanitary sewer flows at the flow monitoring sites, estimate available sewer capacity and conduct analyses pertaining to infiltration and inflow (I/I) occurring in the basins upstream from the flow monitoring sites.

Flow Monitoring Sites and Rain Gauges

Flow monitoring sites are the locations where the flow monitors were placed. Flow monitoring site data may include the flows of one or many drainage basins. To isolate a flow monitoring basin, an addition or subtraction of flows may be required[2]. Capacity and flow rate information is presented on a site-by-site basis. Rain data was obtained from one rain gauge installed by V&A. Additional rain data was obtained from the weather stations owned and maintained by the weather enthusiasts near the monitoring sites. V&A performed quality control on the third-party rainfall data. However, V&A does not have any quality assurance on the collection of the raw rainfall data. The flow monitoring and rain gauge locations are listed in Table 3 and shown in Figure 4.

Table 3. List of Flow Monitoring and Rain Gauge Locations

Site	Pipe Diameter (in)	Location
Site 1	18	Treatment Plant Building 3 Latitude: 39.206425°; Longitude: -121.067991° Rim Elevation: 2,381 feet above sea level
Site 2	30	Allison Ranch Road Latitude: 39.206951°; Longitude: -121.069597° Rim Elevation: 2,347 feet above sea level
Site 3	15	Southbound Golden Chain Highway Rood Expressway off-ramp Latitude: 39.210316°; Longitude: -121.068587° Rim Elevation: 2,370 feet above sea level
Site 4	24	Southbound Golden Chain Highway Rood Expressway off-ramp Latitude: 39.210316°; Longitude: -121.068587° Rim Elevation: 2,370 feet above sea level
Site 5	15	Southbound Golden Chain Highway Auburn Street on-ramp Latitude: 39.215503°; Longitude: -121.063023° Rim Elevation: 2,405 feet above sea level
Site 6	7.25	South Auburn Street North of Neal Street Latitude: 39.217328°; Longitude: -121.061864° Rim Elevation: 2,400 feet above sea level

[2] There is error inherent in flow monitoring. Adding and subtracting flows increases error on an additive basis. For example, if Site A has error ±10% and Site B has error ±10%, then the resulting flow when subtracting Site A from Site B would be ±20%.

Site	Pipe Diameter (in)	Location
Site 7	12	East Main Street and Harris Street Latitude: 39.222365°; Longitude: -121.053918° Rim Elevation: 2,434 feet above sea level
Site 8	15	126 Idaho Maryland Road Latitude: 39.221984°; Longitude: -121.053146° Rim Elevation: 2,436 feet above sea level
Rain Gauges		
RG 1	V&A rain gauge installed at Grass Valley Wastewater Treatment Plant Latitude: 39.205°; Longitude: -121.068°	
RG 2	Third-party rain gauge (GRASS24) Latitude: 39.233°; Longitude: -121.079°	
RG 3	Third-party rain gauge (GRASS25) Latitude: 33.213°; Longitude: -121.058°	

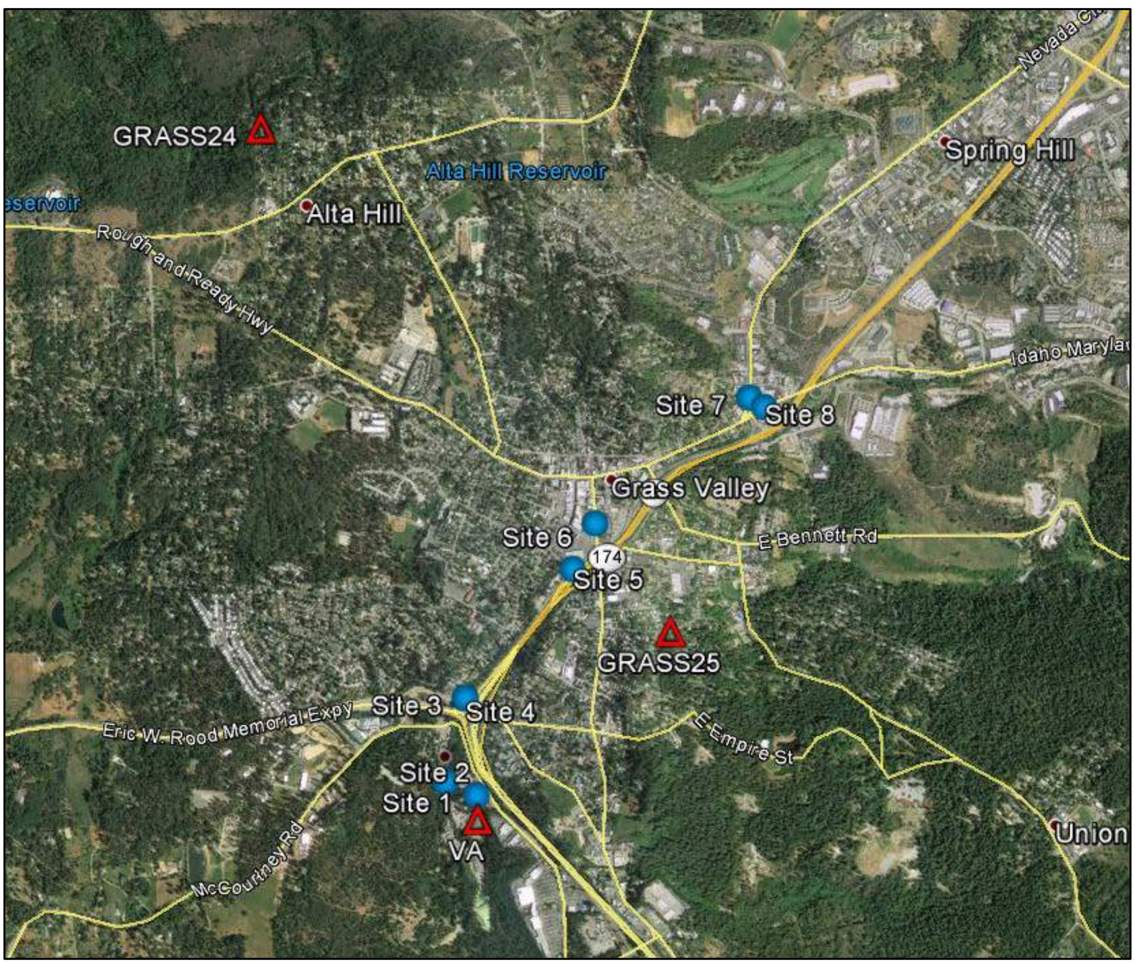


Figure 4. Map of Flow Monitoring Sites and Rain Gauges

Flow Monitoring Basins

Flow monitoring basins are localized areas of a sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. The basin refers to the ground surface area near and enclosed by pipelines. After a basin of interest is established, the flow generated within the basin is the difference between the flows measured in the inlet and outlet of the basin. If there is no inlet flow, the flow from the basin is exclusively measured by the outlet flow meter. This is the case for Sites/Basins 1, 3, 5, 6, 7, and 8 (Figure 5). Site 4 measured the total flow from Basins 5, 6, 7, and 8. Site 2 measured the total flow from Basins 3 and 4. Sites 2 and 4 can be used as a quality control for the measurement of the upstream sites. I/I analysis in this report will be conducted on a basin-by-basin basis for Basins 1, 3, 5, 6, 7, and 8.

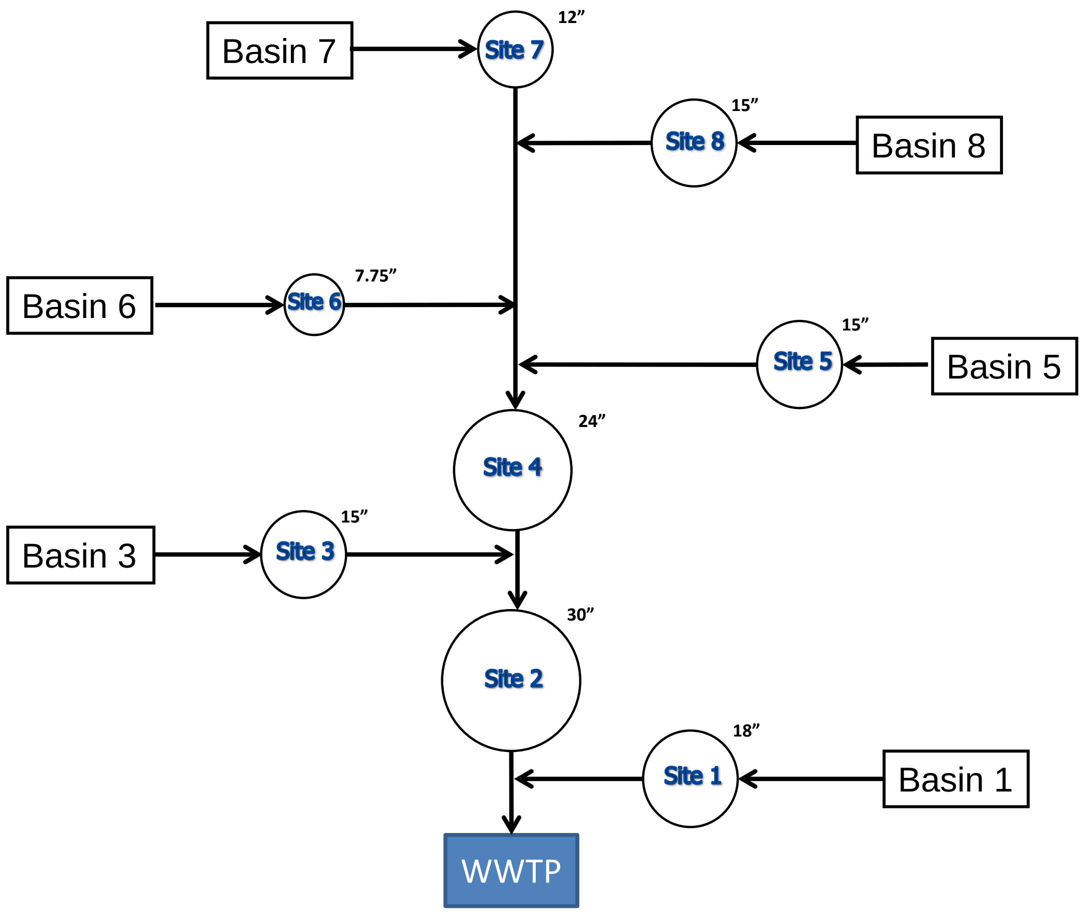


Figure 5. Flow Monitoring Locations Schematic

METHODS AND PROCEDURES

Confined Space Entry

After the flow monitoring sites were determined, a confined space entry was followed in order to install the flow meter into the manhole. A confined space (Photo 1) is defined as any space that is large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit and is not designed for continuous employee occupancy. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5% to 23.0%), and the absence of hydrogen sulfide (H_2S) gas, carbon monoxide (CO) gas, and lower explosive limit (LEL) levels. A typical confined space entry crew has members with OSHA-defined responsibilities of Entrant, Attendant and Supervisor. The Entrant is the individual performing the work. He or she is equipped with the necessary personal protective equipment needed to perform the job safely, including a personal four-gas monitor (Photo 2). If it is not possible to maintain line-of-sight with the Entrant, then more Entrants are required until line-of-sight can be maintained. The Attendant is responsible for maintaining contact with the Entrants to monitor the atmosphere using another four-gas monitor and maintaining records of all Entrants, if there are more than one. The Supervisor is responsible for developing the safe work plan for the job at hand prior to entering.

Photo 1. Confined Space Entry

Photo 2. Typical Personal Four-Gas Monitor

Flow Meter Installation

V&A installed one Teledyne Isco 2150 meter in each monitoring site listed in Table 3. Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during installation of the flow meters and again when they were removed and compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. Figure 6 shows a typical installation for a flow meter with a submerged sensor.

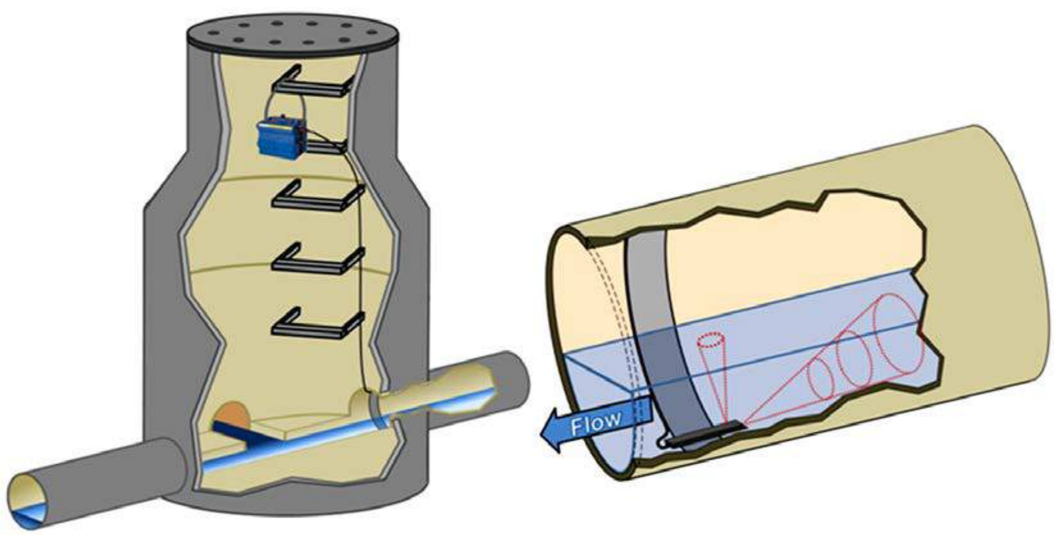


Figure 6. Typical Installation for Flow Meter with Submerged Sensor

Change of Hydraulic Condition

V&A altered the hydraulics of Site 6 to make it more monitor-able. A weir was installed in the downstream from the flow meter. The weir can increase the flow levels and decrease the flow velocities. Therefore, the hydraulic information (level and velocity) shown in this report is different than actual and level data should not be used to calibrate model level estimates; however the flow volumes/flow rates should be accurate.

Flow Calculation

Data retrieved from the flow meter was placed into a spreadsheet program for analysis. Data analysis includes data comparison to field calibration measurements, as well as necessary geometric adjustments as required for sediment (sediment reduces the pipe's wetted cross-sectional area available to carry flow). Area-velocity flow metering uses the continuity equation,

$$Q = V \cdot A$$

where Q is the volume flow rate, V is the average velocity as determined by the ultrasonic sensor, and A is the cross-sectional area of flow as determined from the depth of flow. For circular pipe,

$$A = \left[\frac{D^2}{4}\cos^{-1}\left(1-\frac{2d}{D}\right)\right] - \left[\left(\frac{D}{2}-d\right)\left(\frac{D}{2}\right)\sin\left(\cos^{-1}\left(1-\frac{2d}{D}\right)\right)\right]$$

where D is the pipe diameter and d is the depth of flow.

Average Dry Weather Flow Calculation

Weekday and weekend flow patterns differ and must be separated when determining average dry weather flows. Days least affected by rainfall were used to estimate weekend and weekday average flows. The overall average dry weather flow (ADWF) is calculated per the following equation:

$$ADWF = \left(ADWF_{Mon-Fri} \times \frac{5}{7}\right) + \left(ADWF_{Sat-Sun} \times \frac{2}{7}\right)$$

Figure 7 illustrates the varying flow patterns within a work week.

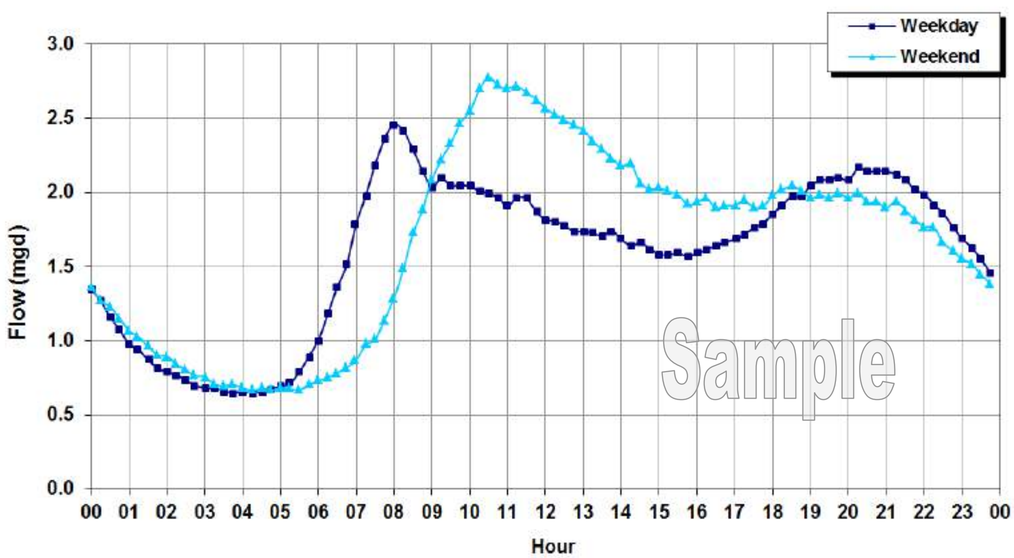


Figure 7. Sample ADWF Diurnal Flow Patterns

Background Knowledge of Inflow / Infiltration

Inflow and infiltration (I/I) consists of storm water and groundwater that enter the sewer system through pipe defects and improper storm drainage connections and is defined as follows:

Definition and Typical Sources

- **Inflow:** Storm water inflow is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins.
- **Infiltration:** Infiltration is defined as water entering the sanitary sewer system through defects in pipes, pipe joints, and manhole walls, which may include cracks, offset joints, root intrusion points, and broken pipes.

Figure 8 illustrates the possible sources and components of I/I.

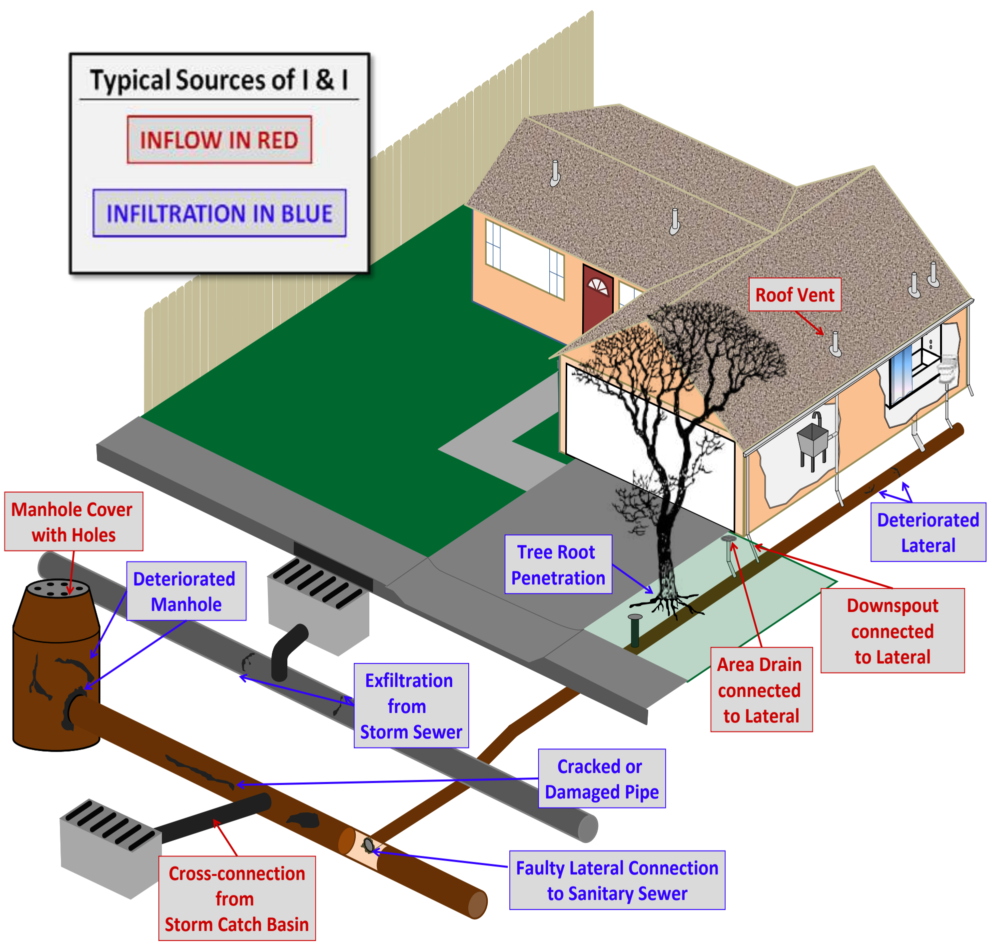


Figure 8. Typical Sources of Infiltration and Inflow

Infiltration Components

Infiltration can be further subdivided into components as follows:

- **Groundwater Infiltration:** Groundwater infiltration depends on the depth of the groundwater table above the pipelines as well as the percentage of the system submerged. The variation of groundwater levels and subsequent groundwater infiltration rates is seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
- **Rainfall-Dependent Infiltration:** This component occurs as a result of storm water and enters the sewer system through pipe defects, as with groundwater infiltration. The storm water first percolates directly into the soil and then migrates to an infiltration point. Typically, the time of concentration for rainfall-related infiltration may be 24 hours or longer, but this depends on the soil permeability and saturation levels.
- **Rainfall-Responsive Infiltration** is storm water which enters the collection system indirectly through pipe defects, but normally in sewers constructed close to the ground surface such as private laterals. Rainfall-responsive infiltration is independent of the groundwater table and reaches defective sewers via the pipe trench in which the sewer is constructed, particularly if the pipe is placed in impermeable soil and bedded and backfilled with a granular material. In this case, the pipe trench serves as a conduit similar to a French drain, conveying storm drainage to defective joints and other openings in the system. This type of infiltration can have a quick response and graphically can look very similar to inflow.

Impact and Cost of Source Detection and Removal

- **Inflow:**
  - **Impact:** This component of I/I creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow is tied closely to the intensity of the storm event, the short-term peak instantaneous flows may result in surcharging and overflows within a collection system. Severe inflow may result in sewage dilution, resulting in upsetting the biological treatment (secondary treatment) at the treatment facility.
  - **Cost of Source Identification and Removal:** Inflow locations are usually less difficult to find and less expensive to correct. These sources include direct and indirect cross-connections with storm drainage systems, roof downspouts, and various types of surface drains. Generally, the costs to identify and remove sources of inflow are low compared to potential benefits to public health and safety or the costs of building new facilities to convey and treat the resulting peak flows.

- **Infiltration:**
  - **Impact:** Infiltration typically creates long-term annual volumetric problems. The major impact is the cost of pumping and treating the additional volume of water, and of paying for treatment (for municipalities that are billed strictly on flow volume).
  - **Cost of Source Detection and Removal:** Infiltration sources are usually harder to find and more expensive to correct than inflow sources. Infiltration sources include defects in deteriorated sewer pipes or manholes that may be widespread throughout a sanitary sewer system.

Graphical Identification of I/I

Inflow is usually recognized graphically by large-magnitude, short-duration spikes immediately following a rain event. Infiltration is often recognized graphically by a gradual increase in flow after a wet-weather event. The increased flow typically sustains for a period after rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels. Realtime flows were plotted against ADWF to analyze the I/I response to rainfall events. Figure 9 illustrates a sample of how this analysis is conducted and some of the measurements that are used to distinguish infiltration and inflow. Similar graphs were generated for the individual flow monitoring sites and can be found in *Appendix A*.

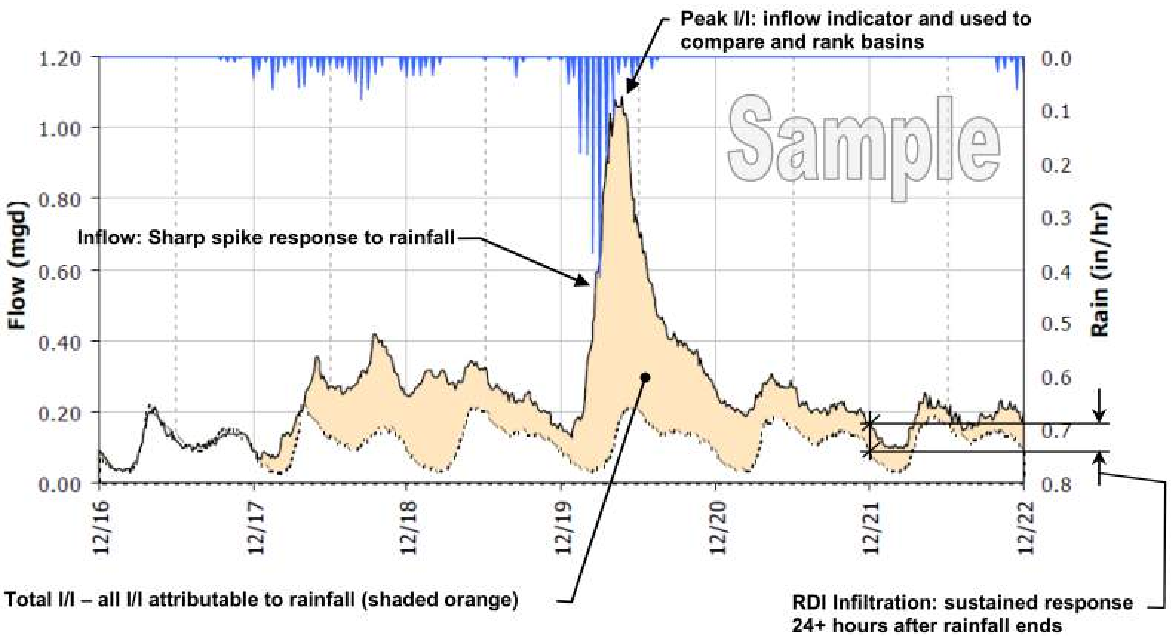


Figure 9. Sample Infiltration and Inflow Isolation Graph

Figure 10 shows sample graphs indicating the typical graphical response patterns for inflow and infiltration in a more detailed version.

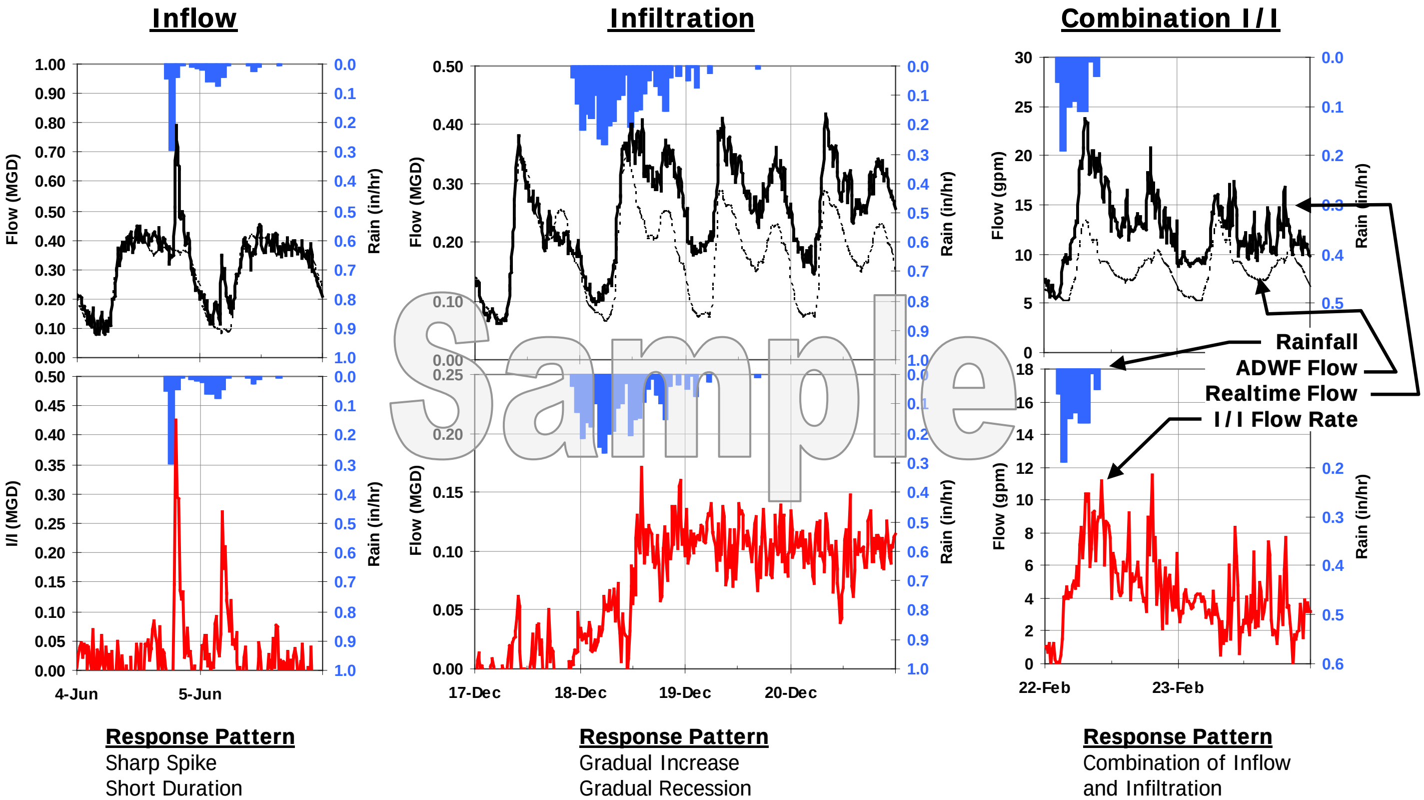


Figure 10. Inflow and Infiltration: Graphical Response Patterns

Analysis Methods

After differentiating I/I flows from ADWF flows, various calculations can be made to: (1) determine which I/I component (inflow or infiltration) is more prevalent at a particular site, and (2) to compare the relative magnitude of the I/I components between drainage basins and between storm events. Some analysis methods are shown as follows:

Inflow Indicators

Inflow is characterized by sharp, direct spikes occurring during a rainfall event. Peak I/I rates are used for inflow analysis[3]. After determining the peak I/I flow rate for a given site, and for a given storm event, there are three ways to *normalize* the peak I/I rates for an "apples-to-apples" comparison amongst the different drainage basins:

- ***Peak I/I Flow Rate per IDM:*** Peak measured I/I rate divided by length of pipe within the drainage basin, expressed in units of inch-diameter-mile (IDM) (miles of pipeline multiplied by the diameter of the pipeline in inches). Final units are gallons per day (gpd) per IDM.

[3] I/I flow rate is the realtime flow less the estimated average dry weather flow rate. It is an estimate of flows attributable to rainfall. By using peak measured flow rates (inclusive of ADWF), the I/I flow rate would be skewed higher or lower depending on whether the storm event I/I response occurs during low flow or high flow hours.

- ***Peak I/I Flow Rate per Acre:*** Peak measured I/I rate divided by the geographic area of the upstream basin in acres. Units are gpd per acre.
- ***Peak I/I Flow Rate to ADWF Ratio:*** Peak measured I/I rate divided by average dry weather flow (ADWF). This is a ratio and is expressed without units.

Infiltration Indicators

- **Rainfall-Dependent Infiltration:** Infiltration occurring after the conclusion of a storm event is classified as rainfall-dependent infiltration. Analysis is conducted by looking at the infiltration rates at set periods after the conclusion of a storm event. Depending on the system and the time required for flows to return to ADWF levels, different set periods may be examined to determine the basins with the greatest or most sustained rainfall-dependent infiltration rates.

- **Dry Weather Groundwater Infiltration:** GWI analysis is conducted by looking at minimum dry weather flow to average dry weather flow ratios and comparing them to established standards to quantify the rate of excess groundwater infiltration. As with inflow, GWI infiltration rates can be normalized by means of pipe length (IDM), basin area (acres), and dry weather flow rates (ADWF). These methods are discussed in further detail in the *Groundwater Analysis* section later in this report.

Combined I/I Indicators

The total inflow and infiltration is measured in gallons per site and per storm event. Because it is based on total I/I volume, it is an indicator of combined inflow and infiltration and is used to identify the overall volumetric influence of I/I within the monitoring basin. As with inflow, pipe length, basin area, and dry weather flow are used to normalize combined I/I for basin comparison:

- ***Combined I/I Flow Rate per IDM:*** Total infiltration (gallons) divided by length of pipe (IDM) and divided by storm event rainfall (inches of rain). Final units are gallons per day (gpd) per IDM per inch-rain.
- ***R-Value:*** Total infiltration (gallons) divided by the total rainfall that fell within the acreage of a particular basin (gallons of rainfall). This is expressed as a percentage and is explained as "the percent of rain that falls that enters the sanitary sewer collection system." Systems with R-values less than 5%[4] are often considered to be performing well.
- ***Combined I/I Flow Rate per ADWF:*** Total infiltration (gallons) divided by the ADWF (gpd) and divided by storm event rainfall (inches of rain). Final units are million gallons per MGD of ADWF per inch-rain.

[4] Keefe, P.N. "Test Basins for I/I Reduction and SSO Elimination." 1998 WEF Wet Weather Specialty Conference, Cleveland.

RESULTS AND ANALYSIS

Rainfall Event Analysis

In order to perform I/I analysis, rainfall data should be collected in order to distinguish the wet weather days from the dry weather days. Rainfall intensity, duration, and frequency are also required to conduct the synthetic I/I analysis. Rain data collected from three sites was analyzed for the duration of the study to capture rainfall across the limits of the City boundary, illustrated earlier in Figure 4.

Rain Gauge Data

There were three main rainfall events that occurred over the course of the flow monitoring period. Figure 11 graphically displays the rainfall activity recorded at RG 1 over the flow monitoring period for illustration purpose.

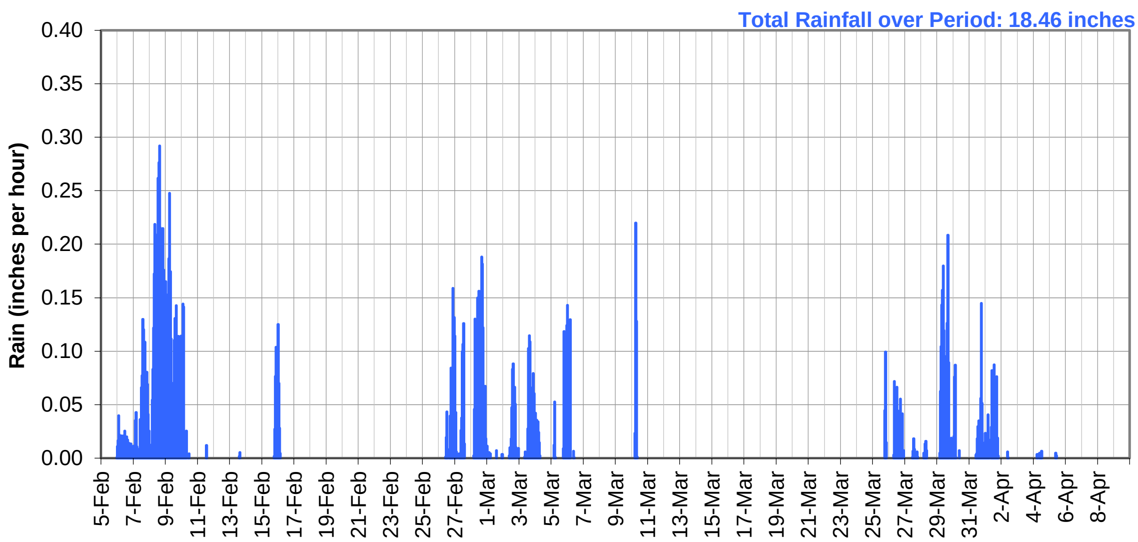


Figure 11. Rainfall Activity at the RG 1

Figure 12 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[5] in the City during this project duration. The total historical rainfall is 17.43 inches. The rainfall recorded at each rain gauge location is summarized in Table 4.

[5] Historical data taken from the WRCC (Station 046377 in Oceanside): http://www.wrcc.dri.edu/summary/climsmnca.html

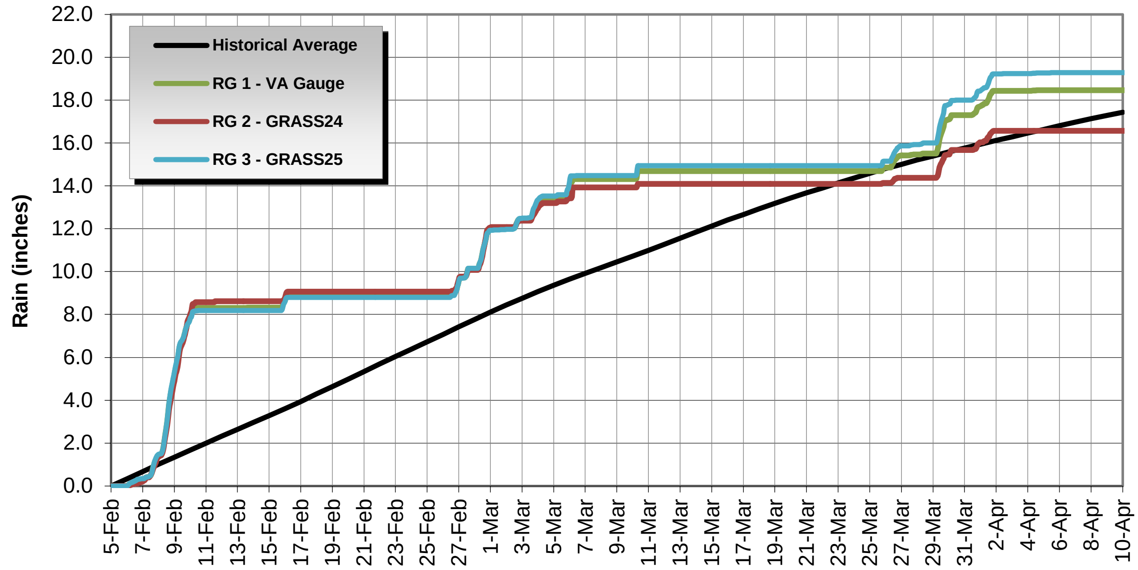


Figure 12. Accumulated Precipitation Monitored from Different Locations

Table 4. Rainfall Events Used for I/I Analysis

Rain Gauge	Event 1 Feb 5- 14, 2014 (in)	Event 2 Feb 26- Mar 7, 2014 (in)	Event 3 Mar 25- Apr 4, 2014 (in)	*Total over Monitoring Period (in)*	*Total Rainfall / Historical Rainfall (%)*
RG 1	8.32	5.44	3.77	18.46	106
RG 2	8.62	4.86	2.47	16.57	95
RG 3	8.19	5.68	4.32	19.27	111

Rainfall Event Classification

It is important to classify the relative size of a major storm event that occurs over the course of a flow monitoring period[6]. Storm events are classified by intensity and duration. Based on historical data, frequency contour maps for storm events of given intensity and duration have been developed by the National Oceanic and Atmospheric Administration (NOAA) for all areas within the continental United States. For example, the NOAA Rainfall Frequency Atlas[7] classifies a 10-year, 24-hour storm event in Grass Valley as approximately 7.5 inches (Figure 13). This means that in any given year, at this specific location, there is a 10% chance that 7.5 inches of rain will fall in any 24-hour period.

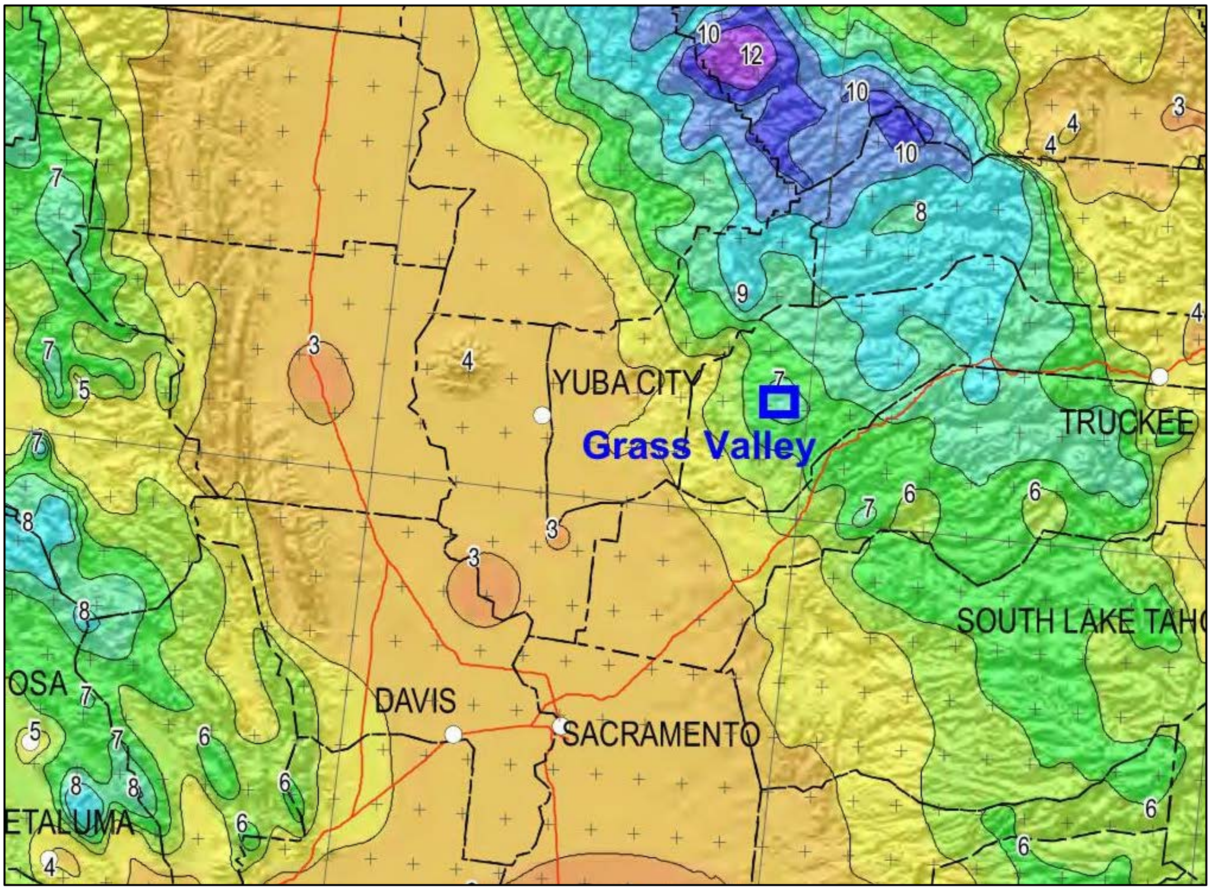


Figure 13. NOAA Isopluvials of 10-Year, 24-Hour Precipitation in inches

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 1 day to 10 days are known for rain events ranging from 1-year to 100-year intensities. These are plotted to develop a rain event frequency map specific to each rainfall monitoring site. Superimposing the peak measured densities for all the rainfall events on the rain event frequency plot determines the classification of the storm event, shown in Figure 14 through Figure 16 for all the rain gauges.

[6] Sanitary sewers are often designed to withstand I/I contribution to sanitary flows for specific-sized "design" storm events.

[7] NOAA Atlas 14, Volume 6, Version 2 California ftp://hdsc.nws.noaa.gov/pub/hdsc/data/sw/ca10y24h.pdf

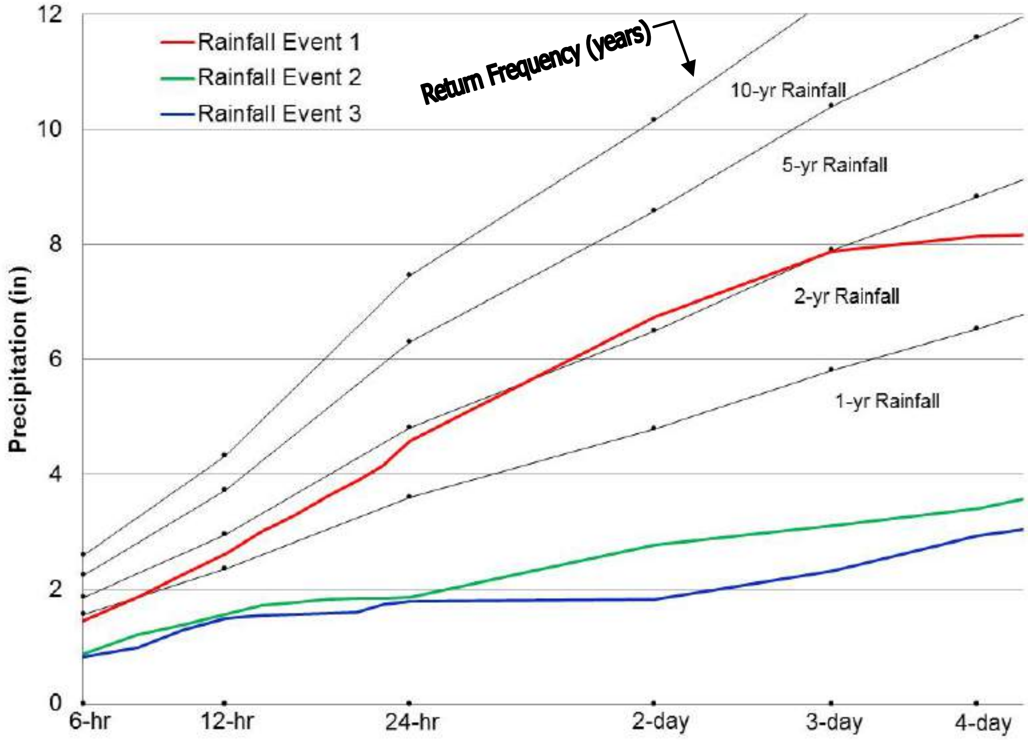


Figure 14. Storm Event Classification at RG 1

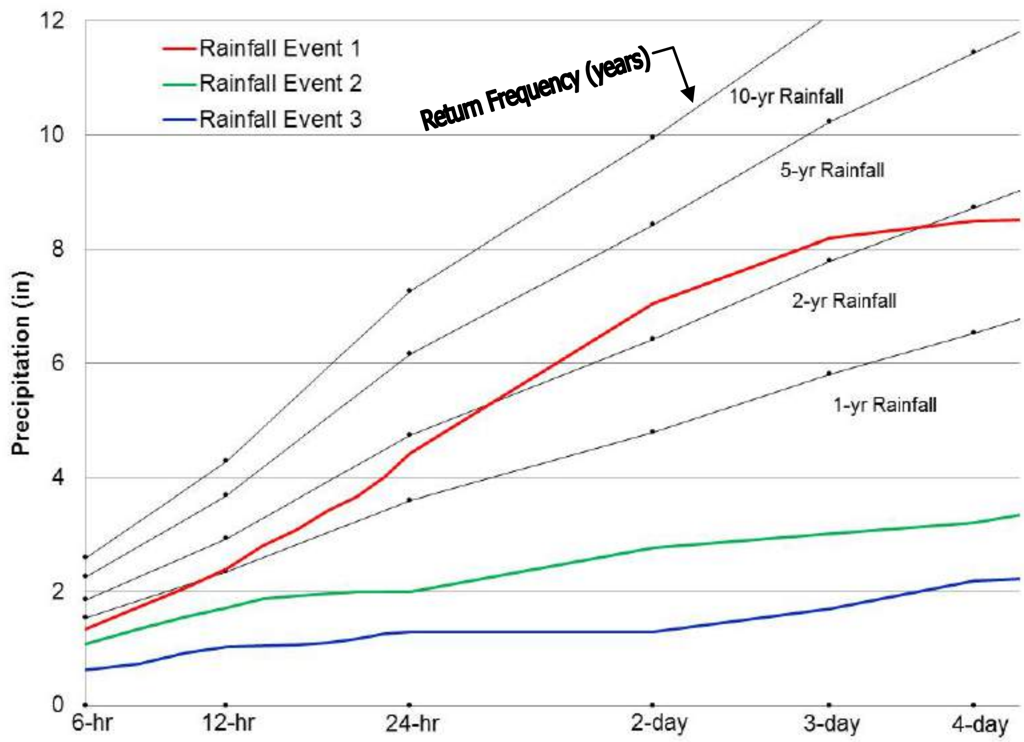


Figure 15. Storm Event Classification at RG 2

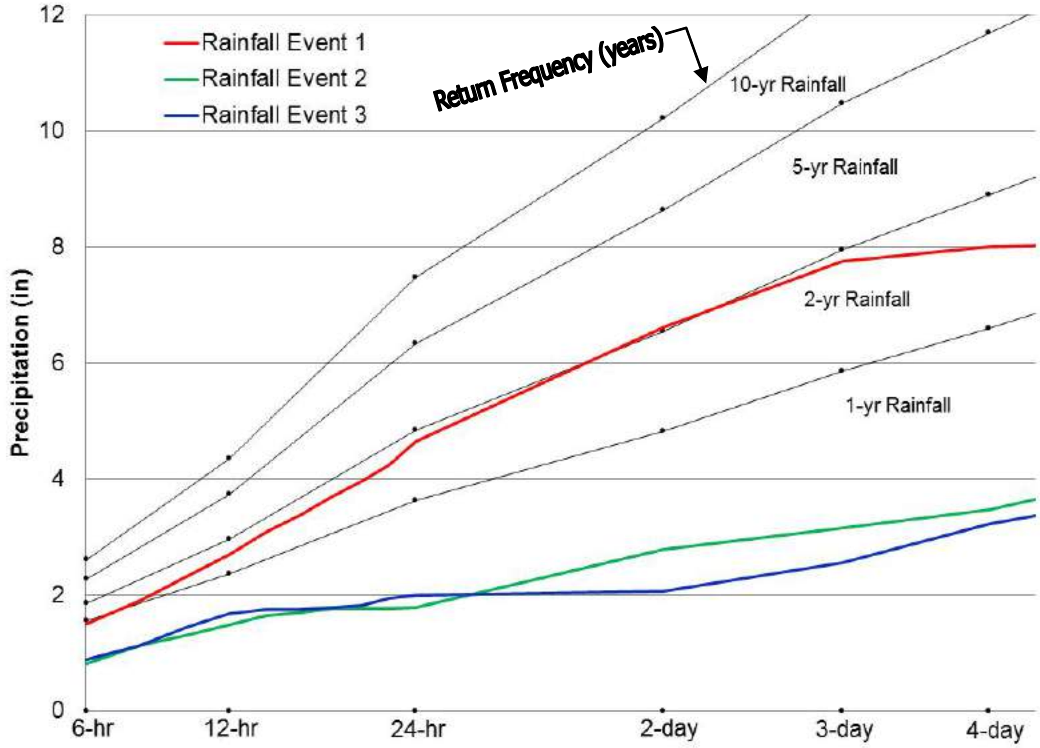


Figure 16. Storm Event Classification at RG 3

Table 5 summarizes the classification of the rainfall events that occurred during the flow monitoring period.

Table 5. Classification of Rainfall Events

Rain Gauge	Event 1 Feb 5- 14, 2014 (in)	Event 2 Feb 26- Mar 7, 2014 (in)	Event 3 Mar 25- Apr 4, 2014 (in)
RG 1	< 2-year, 24-hour > 2-year, 2-day 2 year, 3-day	< 1-year	< 1-year
RG 2	< 2-year, 24-hour > 2-year, 2-day > 2 year, 3-day	< 1-year	< 1-year
RG 3	< 2-year, 24-hour 2-year, 2-day < 2 year, 3-day	< 1-year	< 1-year

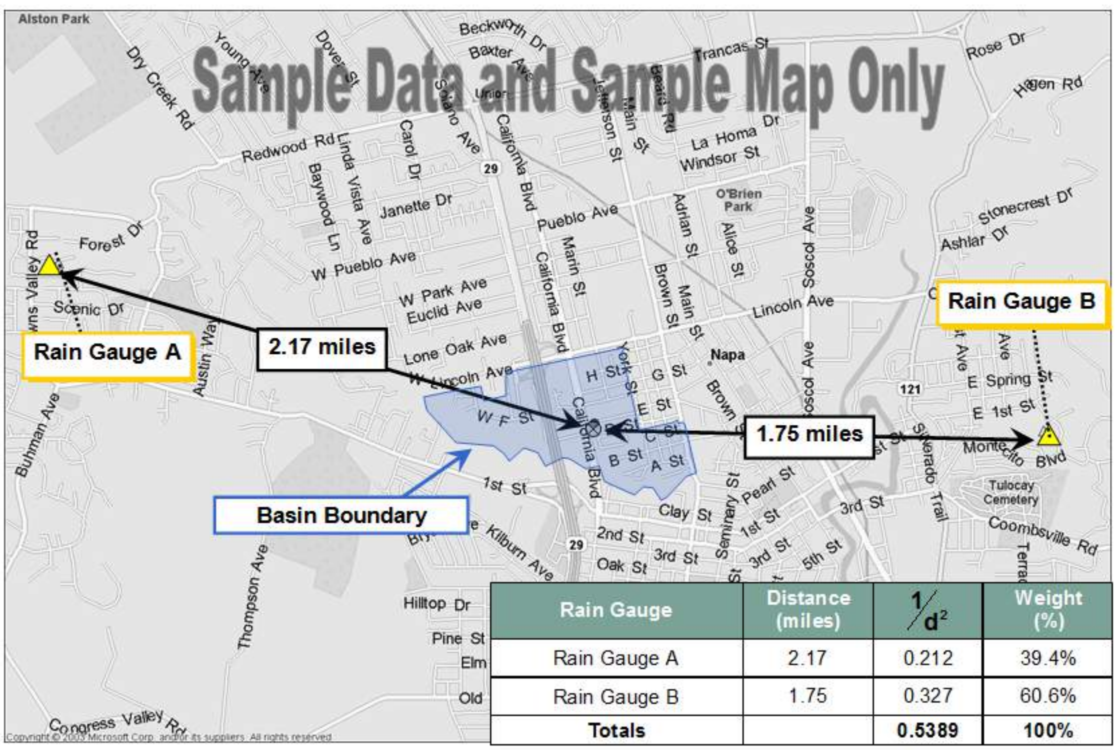
Rainfall: Rain Gauge Triangulation

The rainfall affecting the sanitary sewer collection system basins must be calculated based on the proximity to the rain gauge locations. The mean precipitation for the sanitary sewer collection system was calculated by taking data from seven local rain gauges and using the Inverse Distance Weighting (IDW) method. The IDW is an interpolation method that assumes the influence of each rain gauge location diminishes with distance. The center of a sanitary sewer collection system was identified and a weighted average was taken of the precipitation data from nearby rain gauge locations. The IDW function is as follows:

$$weight(d) = \frac{1/d^p}{\sum 1/d^p}, \quad \text{where:} \quad d = \text{distance} \quad p = \text{power } (p > 0)$$

The value of *p* is user defined. The most common choice for hydrological studies of watershed areas is *p* = 2. Figure 17 illustrate the IDW method with sample data.

It can be seen from the rainfall analysis that all three rain gauges monitored the similar rainfall intensity, duration, and frequency for all the rainfall events during the entire flow monitoring period. Also the flow monitoring sites are located close to each other. Therefore, only one set of rainfall data is applied to all the flow monitoring sites. The rainfall data is composed of rain data monitored from the three rain gauges (RG 1: 40%, RG 2: 20%, and RG 3: 40%).



Rain Gauge	Distance (miles)	$1/d^2$	Weight (%)
Rain Gauge A	2.17	0.212	39.4%
Rain Gauge B	1.75	0.327	60.6%
Totals		**0.5389**	**100%**

Figure 17. Rainfall Inverse Distance Weighting Method

Flow Monitoring: Average Dry Weather Flows

Table 6 lists the average dry weather flow (ADWF) recorded during this study for the flow monitoring sites. Figure 18 shows a schematic diagram of the average dry weather flows and flow levels. The high flow level at Site 6 can be due to the change of hydraulic condition by installing a weir downstream from flow meter. Detailed graphs of the flow monitoring data on a site-by-site basis are included in *Appendix A*.

Table 6. Dry Weather Flow Summary

Monitoring Site	Weekday ADWF (mgd)	Weekend ADWF (mgd)	Overall ADWF (mgd)	Weekend/ Weekday Ratio
Site 1	0.101	0.105	0.102	1.036
Site 2	1.262	1.179	1.238	0.935
Site 3	0.187	0.187	0.187	1.001
Site 4	1.052	1.017	1.042	0.967
Site 5	0.148	0.141	0.146	0.953
Site 6	0.154	0.149	0.152	0.966
Site 7	0.360	0.333	0.353	0.923
Site 8	0.343	0.290	0.328	0.846

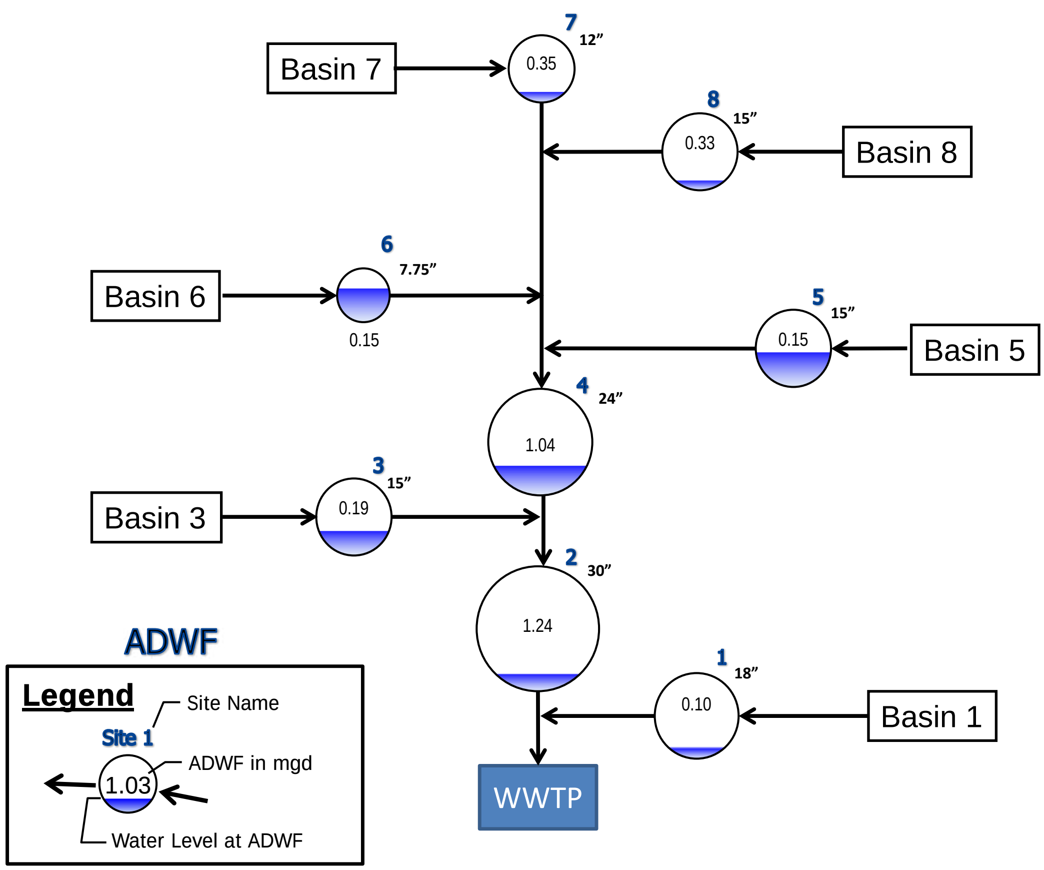


Figure 18. Average Dry Weather Flow Schematic

Flow Monitoring: Peak Measured Flows and Pipeline Capacity Analysis

Peak measured flows and the corresponding flow levels (depths) are important to understand the capacity limitations of a collection system. The peak flows and flow levels reported are from the peak measurements as taken across the entirety of the flow monitoring period. Peak flows and levels may not correspond to a rainfall event, but instead may be caused due to blockages, grease or roots that cause a backflow condition.

The following capacity analysis terms are defined as follows:

- **Peaking Factor:** Peaking factor is defined as the peak measured flow divided by the average dry weather flow (ADWF). A peaking factor threshold value of 3.0 is commonly used for sanitary sewer design.
- ***d/D* Ratio:** The *d/D* ratio is the peak measured depth of flow (*d*) divided by the pipe diameter (*D*). A *d/D* ratio of 0.75 is a common maximum threshold value used for pipe design. The *d/D* ratio for each site was computed based on the maximum depth of flow for the flow monitoring study.

Table 7 summarizes the peak recorded flows, levels, *d/D* ratios, and peaking factors per site during the flow monitoring period. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the point site locations. Hydraulic conditions in other areas of the collection system will differ.

Table 7. Capacity Analysis Summary

Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	*d/D* Ratio	Level Surcharged above Crown (in)
Site 1	0.10	0.68	6.70	18	6.73	0.37	-
Site 2	1.24	9.14	7.38	30	13.27	0.44	-
Site 3	0.19	2.31	12.38	15	24.68	1.65	9.7
Site 4	1.04	6.98	6.70	24.875	19.72	0.79	-
Site 5	0.15	1.74	11.97	15	15.16	1.01	1.0
Site 6	0.15	1.06	6.95	7.25	10.41	1.44	3.5
Site 7	0.35	2.04	5.78	12	5.88	0.49	-
Site 8	0.33	1.82	5.54	15	5.86	0.39	-

The following capacity analysis results are noted:

- **Peaking Factor:** All of the sites had peaking factors higher than the common threshold value.
- ***d/D* Ratio:** Sites 3, 4, 5, and 6 had *d/D* ratios that exceeded common threshold values. Sites 3, 5, and 6 reached a surcharged condition during the study. The surcharged condition at Site 6 can be due to the change of hydraulic condition by installing a weir downstream from flow meter.

Figure 19 and Figure 20 show bar graphs summarizing the site by site peaking factors and d/D ratios, respectively. Figure 21 shows a schematic diagram of the peak measured flows with peak flow levels.

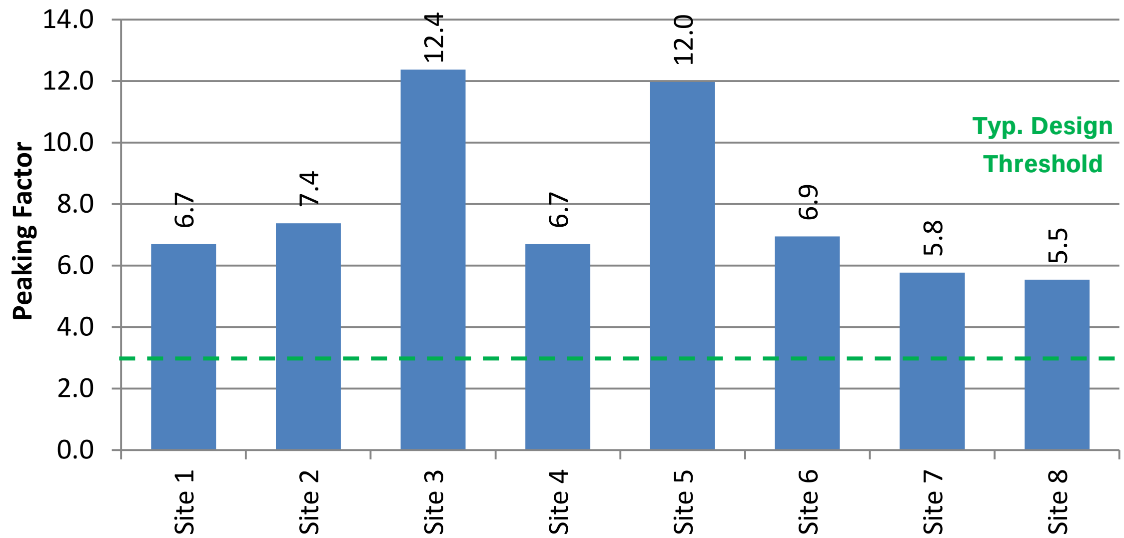


Figure 19. Capacity Summary Bar Graphs: Peaking Factors

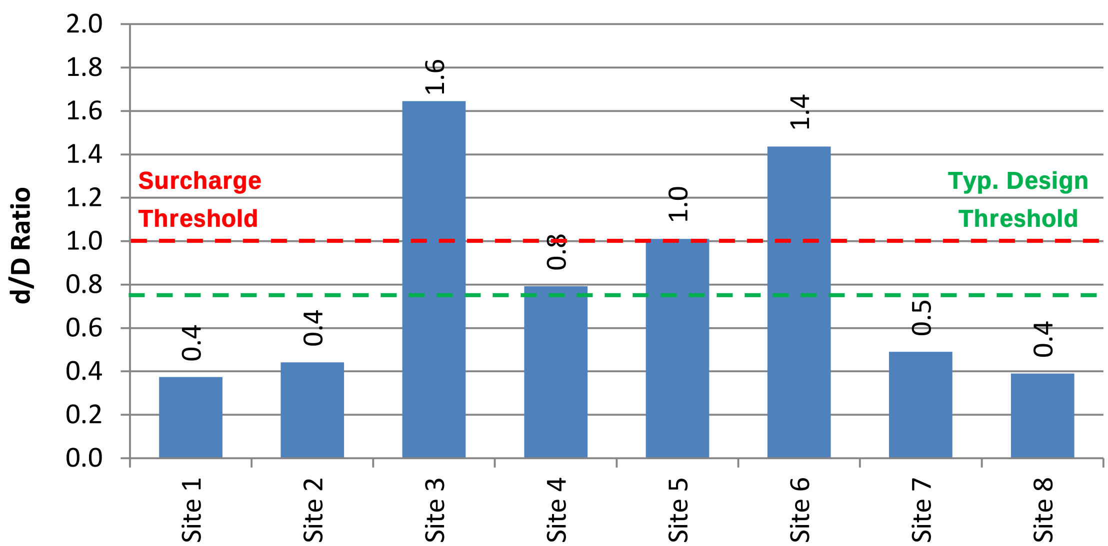


Figure 20. Capacity Summary Bar Graphs: d/D Ratios

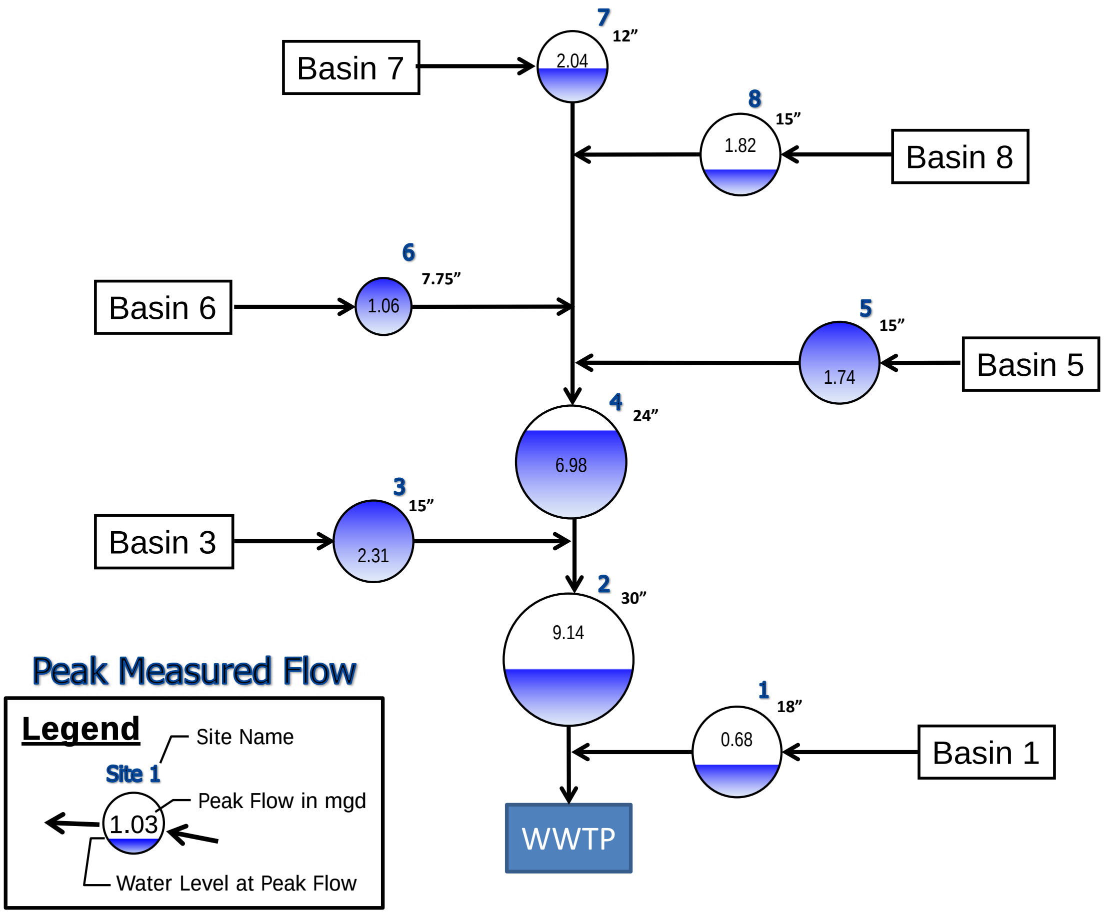


Figure 21. Peak Measured Flow Schematic

Inflow and Infiltration: Results

Storm Event 1 elicited the greatest I/I response of all monitored storm events. The following analyses for inflow and infiltration are based on Storm Event 1 data. Refer to Appendix A for more detailed information on Storm Events 2 and 3.

Inflow Results Summary

Table 8 summarizes the peak measured I/I flows and inflow analysis results for Rainfall Event 1 (February 6 to February 14, 2014). Figure 22 shows bar graph summary of the inflow per ADWF.

Table 8. Basins Inflow Analysis Summary

Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Peak I/I per ADWF	Inflow Ranking
Basin 1	0.102	0.648	6.3	3
Basin 3	0.187	2.190	11.7	1
Basin 5	0.146	1.659	11.4	2
Basin 6	0.152	0.961	6.3	4
Basin 7	0.353	1.836	5.2	5
Basin 8	0.328	1.577	4.8	6

Ranking of 1 represents most inflow after normalization.

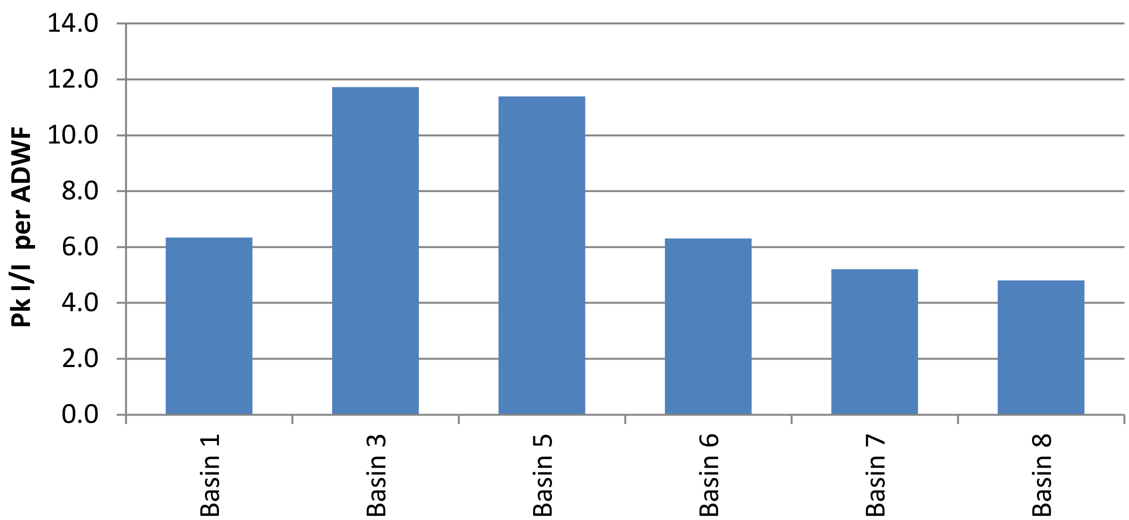


Figure 22. Bar Graphs: Inflow Analysis Summary – Peak I/I to ADWF

Infiltration Results Summary

Table 9 summarizes the RDI analysis for Rainfall Event 1. The analysis was performed for the 24-hour period from February 11, 12 pm to February 12, 12pm, approximately 24 hours after the conclusion of the rain event (refer to the *I/I Methods* section for more information on inflow analysis methods and ranking procedures). Figure 23 shows bar graph summaries of the RDI per ADWF.

Table 9. Basins RDI Analysis Summary

Basin	ADWF (mgd)	RDI Rate (mgd)	RDI / ADWF	RDI Ranking
Basin 1	0.102	0.023	23%	6
Basin 3	0.187	0.191	102%	1
Basin 5	0.146	0.098	66%	2
Basin 6	0.152	0.055	35%	4
Basin 7	0.353	0.210	58%	3
Basin 8	0.328	0.104	30%	5

Ranking of 1 represents most RDI after normalization.

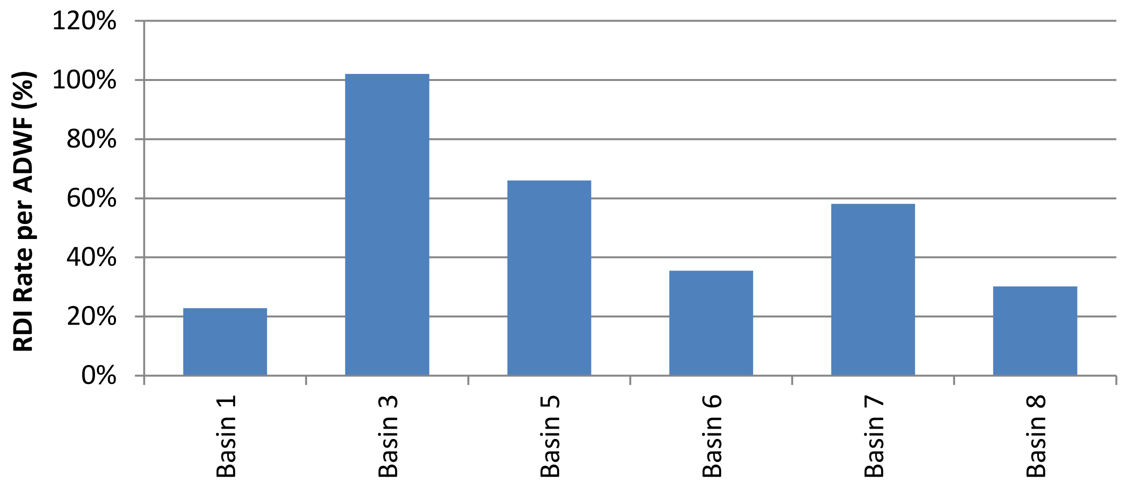


Figure 23. Bar Graphs: RDI Analysis Summary – RDI Rate to ADWF

Groundwater Infiltration Results Summary

Dry weather (ADWF) flow can be expected to have a predictable diurnal flow pattern. While each site is unique, experience has shown that, given a reasonable volume of flow and typical loading conditions, the daily flows fall into a predictable range when compared to the daily average flow. If a site has a large percentage of groundwater infiltration occurring during the periods of dry weather flow measurement, the amplitudes of the peak and low flows will be dampened[8]. Figure 24 shows a sample of two flow monitoring sites, both with nearly the same average daily flow, but with considerably different peak and low flows. In this *sample* case, Site B1 may have a considerable volume of groundwater infiltration.

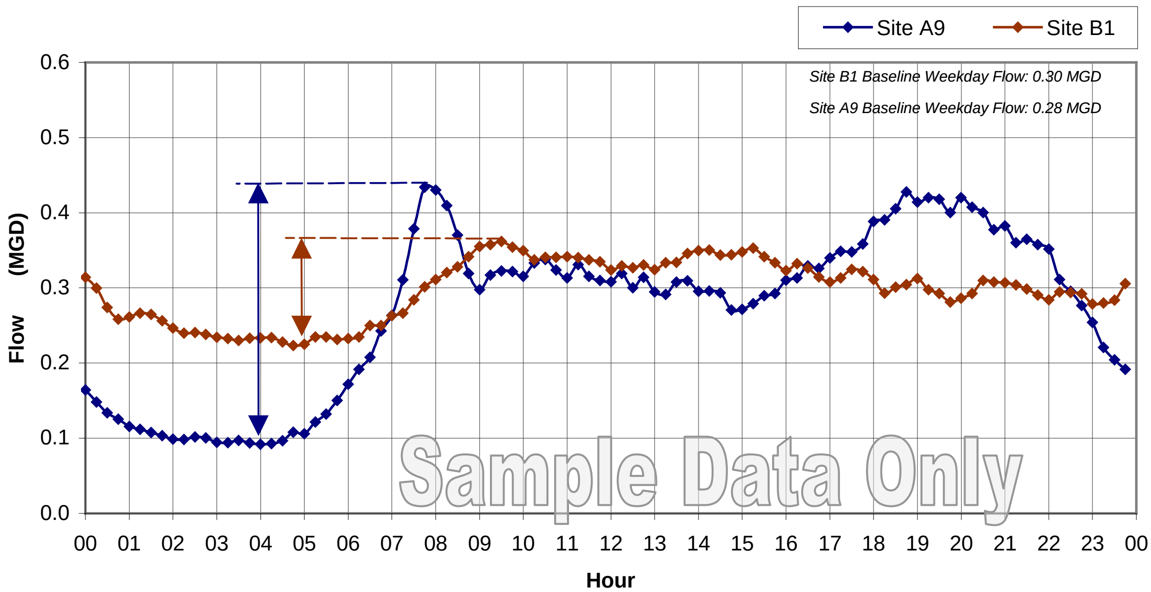


Figure 24. Groundwater Infiltration Sample Figure

It can be useful to compare the low-to-ADWF flow ratios for the flow metering sites. A site with abnormal ratios, and with no other reason to suspect abnormal flow patterns (such as proximity to pump station, treatment facilities, etc.), has a possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system. Figure 25 plots the low-to-ADWF flow ratios against the ADWF flows for the sites monitored during this study. The dotted line shows "typical" low-to-ADWF ratios per the Water Environment Federation (WEF)[9].

[8] Theoretically imagining an extreme case, if there were 0.2 mgd of ADWF flow and 2.0 mgd of groundwater infiltration, the peaks and lows would be barely recognizable; the ADWF flow would be nearly a straight line.

[9] WEF Manual of Practice No. 9, "Design and Construction of Sanitary and Storm Sewers."

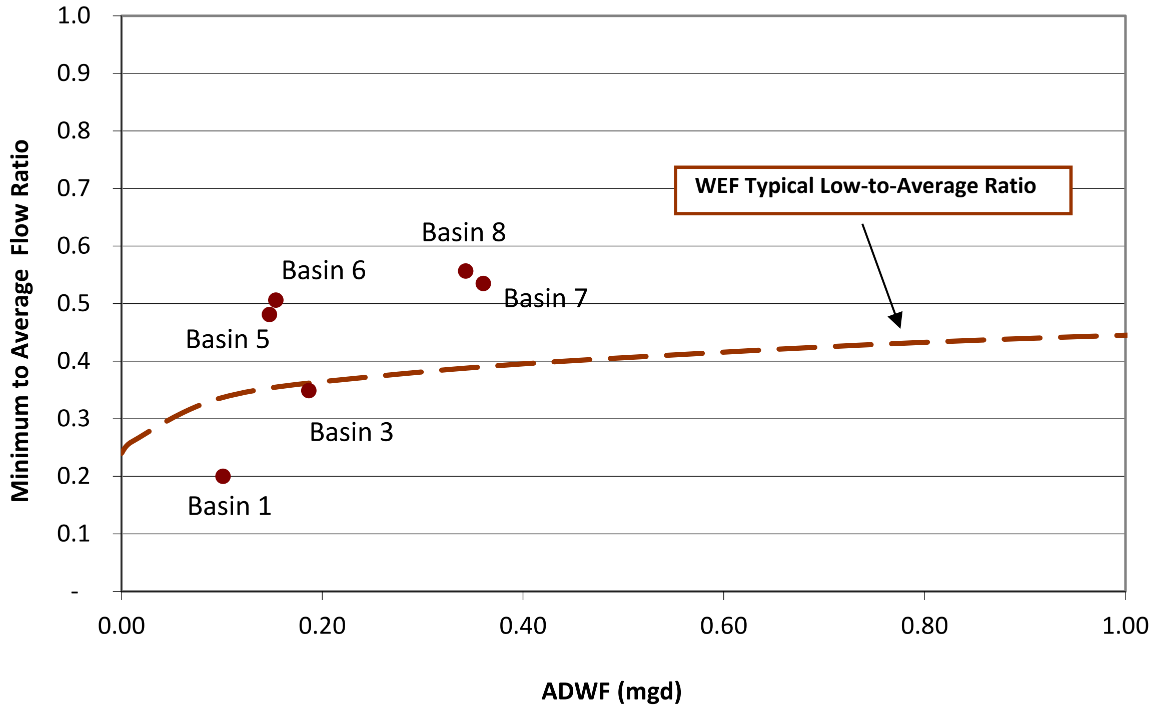


Figure 25. Minimum Flow Ratios vs. ADWF[10]

The following GWI results are noted:

- Basins 5, 6, 7, and 8 had GWI rates that were **above** the WEF typical Low-to-Average Ratio, indicating the possibility of excessive groundwater infiltration.

Combined I/I Results Summary

Combined I/I analysis considers the totalized volume of both inflow and rainfall-dependent infiltration over the course of a storm event.

Table 10 summarizes the combined I/I flow results for the Rainfall Event 1. Combined I/I flows were normalized by the ADWF and rainfall. Figure 26 show bar graph summaries of the combined I/I analysis.

[10] Due to attenuation, it should be expected that sites with larger flow volumes should not have quite the peak-to-average and low-to-average flow ratios as sites with lesser flow volumes, which is why the WEF typical trend lines slope closer to 1.0 as the ADWF increases, as shown in the figure.

Table 10. Basins Combined I/I Analysis Summary

Basin	ADWF (mgd)	Total I/I (million gallons)	Total I/I per ADWF per inch of Rain (day/in)	Combined I/I Ranking
Basin 1	0.102	0.761	0.886	6
Basin 3	0.187	3.372	2.150	1
Basin 5	0.146	2.478	2.026	2
Basin 6	0.152	1.597	1.248	3
Basin 7	0.353	3.477	1.175	4
Basin 8	0.328	2.948	1.071	5

Ranking of 1 represents most combined I/I after normalization.

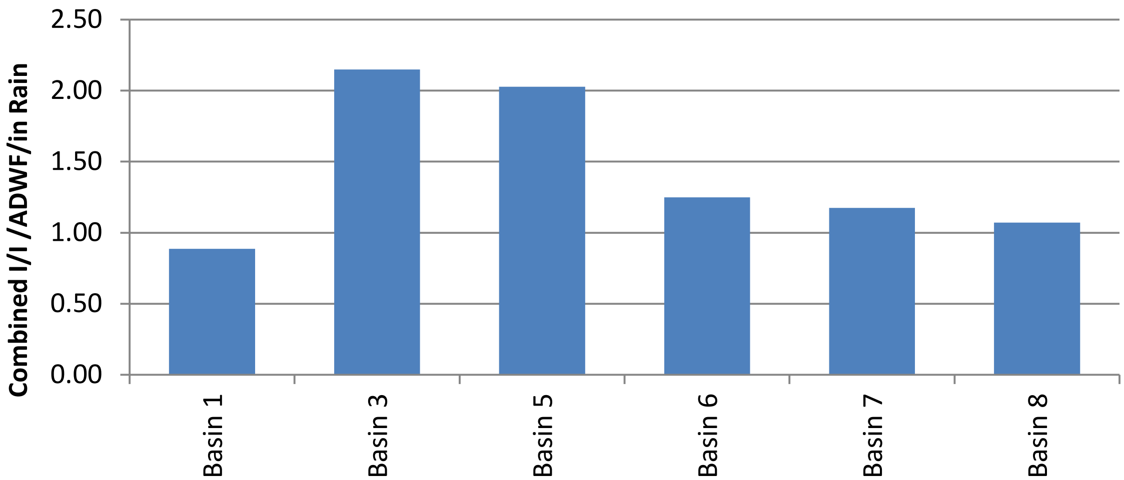


Figure 26. Bar Graphs: Combined I/I Analysis Summary – Total I/I to ADWF

<u>Synthetic Hydrographs</u>

In order to model design storms, synthetic hydrographs were developed to approximate the actual RDI hydrograph shape in terms of the time to the peak and the recession coefficient. The actual RDI hydrograph was best matched with a synthetic hydrograph by separating the synthetic hydrograph into seven volume components (R1 through R7). The seven components represent different response times to the rainfall event and, therefore, different infiltration or inflow paths into the sewer system. R1 is characterized by a short response time and is assumed to consist of mainly inflow. R7 represents slower response and longer recession times and consists of mostly infiltration. Levels of soil saturation are also considered. Using synthetic hydrograph analysis, appropriate time and recession parameters were estimated by a trial-and-error procedure until a good match was obtained. For example, the hydrograph and its component hydrographs for Rainfall Event 1 for Site 3 are shown in Figure 27.

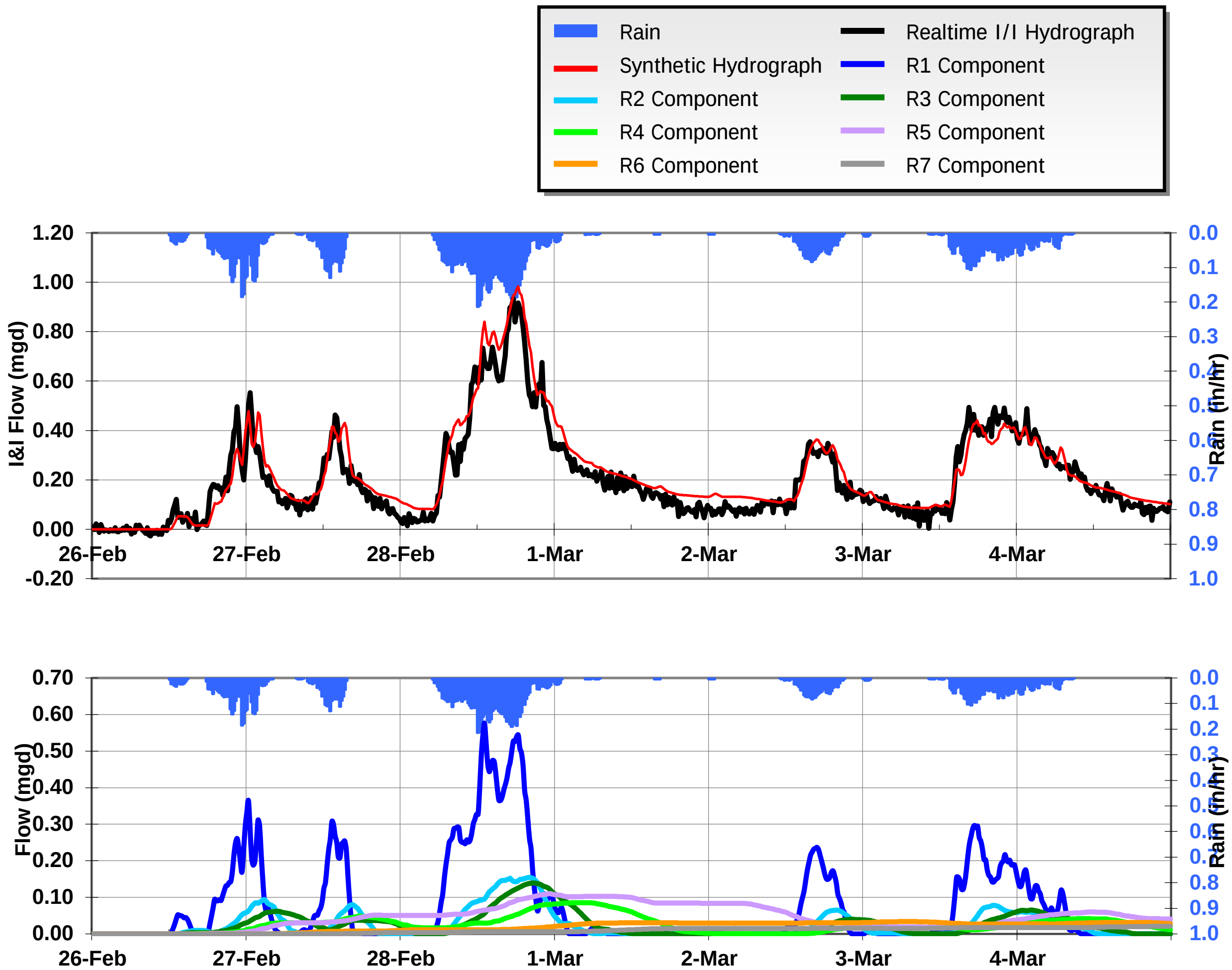


Figure 27. Synthetic Hydrograph for Site 3

Design Storm Development

With the I/I response modeled by a synthetic hydrograph, design storms can be applied. This serves two functions: (a) predicted flows are based on the same storm event and are therefore normalized to each other, making for easier and better comparisons, and (b) the resulting I/I flows can be predicted for a design storm event. This helps to calibrate modeling efforts that will determine if the collection system has adequate capacity to handle very large storm events.

V&A used a 10-year, 24-hour design storm for this analysis. Storm events were taken from the NOAA Precipitation-Frequency Atlas of the Western United States. Figure 28 demonstrates the design storm magnitude and profile for RG 1 for illustration purposes.

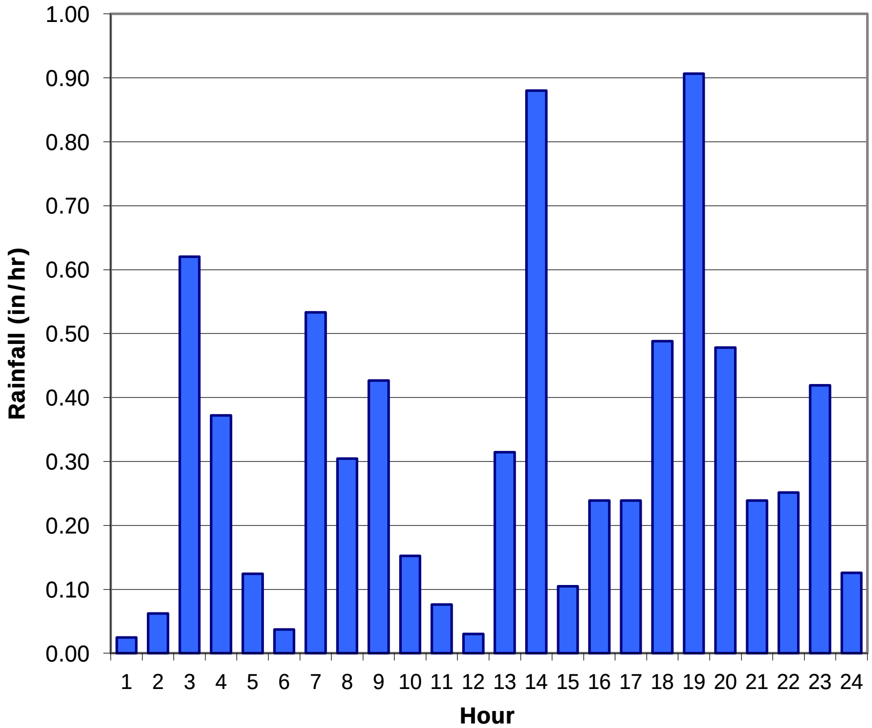


Figure 28. 10-Year, 24-Hour Design Storm Values and Profile for Rain Site GRASS24

Design Storm Response Summary

The 10-year, 24-hour storm event was applied to the synthetic I/I hydrograph components developed for each flow monitoring site. This method produces the best estimated response to the design storm events. These results assume full ground saturation, and the peak I/I flows from the design storm coincide with peak dry weather flows to get a “worst-case” scenario of peak wet weather flows. Table 11 summarizes the final results for each design storm on a **site-by-site** basis.

Table 11. Design Storm I/I Analysis Summary

Site	Peak Dry Weather Flow (mgd)	Peak I/I Rate (mgd)	Peak Flow (mgd)	Total I/I (million gallons)
Site 1	0.21	1.14	1.35	0.79
Site 2	1.69	17.51	19.20	12.92
Site 3	0.28	4.17	4.45	2.78
Site 4	1.45	16.03	17.48	13.00
Site 5	0.22	3.62	3.84	2.35
Site 6	0.21	2.76	2.97	1.59
Site 7	0.51	4.89	5.41	3.26
Site 8	0.46	3.71	4.17	2.75

RECOMMENDATIONS

V&A advises that future I/I reduction plans consider the following recommendations:

4. **Determine I/I Reduction Program:** The City should examine its I/I reduction needs to determine a future I/I reduction program.
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow was occurring in Basins 3 and 5.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems.
        i. The highest normalized rainfall-dependent infiltration was occurring in Basins 3 and 5.
        ii. The highest groundwater infiltration was occurring in Basins 5, 6, 7, and 8.
5. **I/I Investigation Methods:** Potential I/I investigation methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Nighttime reconnaissance work to (1) investigate and determine direct point sources of inflow and (2) determine the areas and pipe reaches responsible for high levels of infiltration contribution.
6. **I/I Reduction Cost-Effectiveness Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow and infiltration and systematically rehabilitating or replacing the faulty pipelines or (2) continued treatment of the additional rainfall-dependent I/I flow.
7. **Downstream Pipe Capacity Analysis:** High levels of inflow resulted in peak flow problems at Sites 3, 5, and 6 where surcharged conditions occurred. If bigger storm events occur, this issue can become more severe and can result in a sanitary sewer overflow (SSO). Pipeline capacity issues within the local collection system should be analyzed to minimize the potential for SSOs.

APPENDIX A

FLOW MONITORING SITES: DATA, GRAPHS, INFORMATION

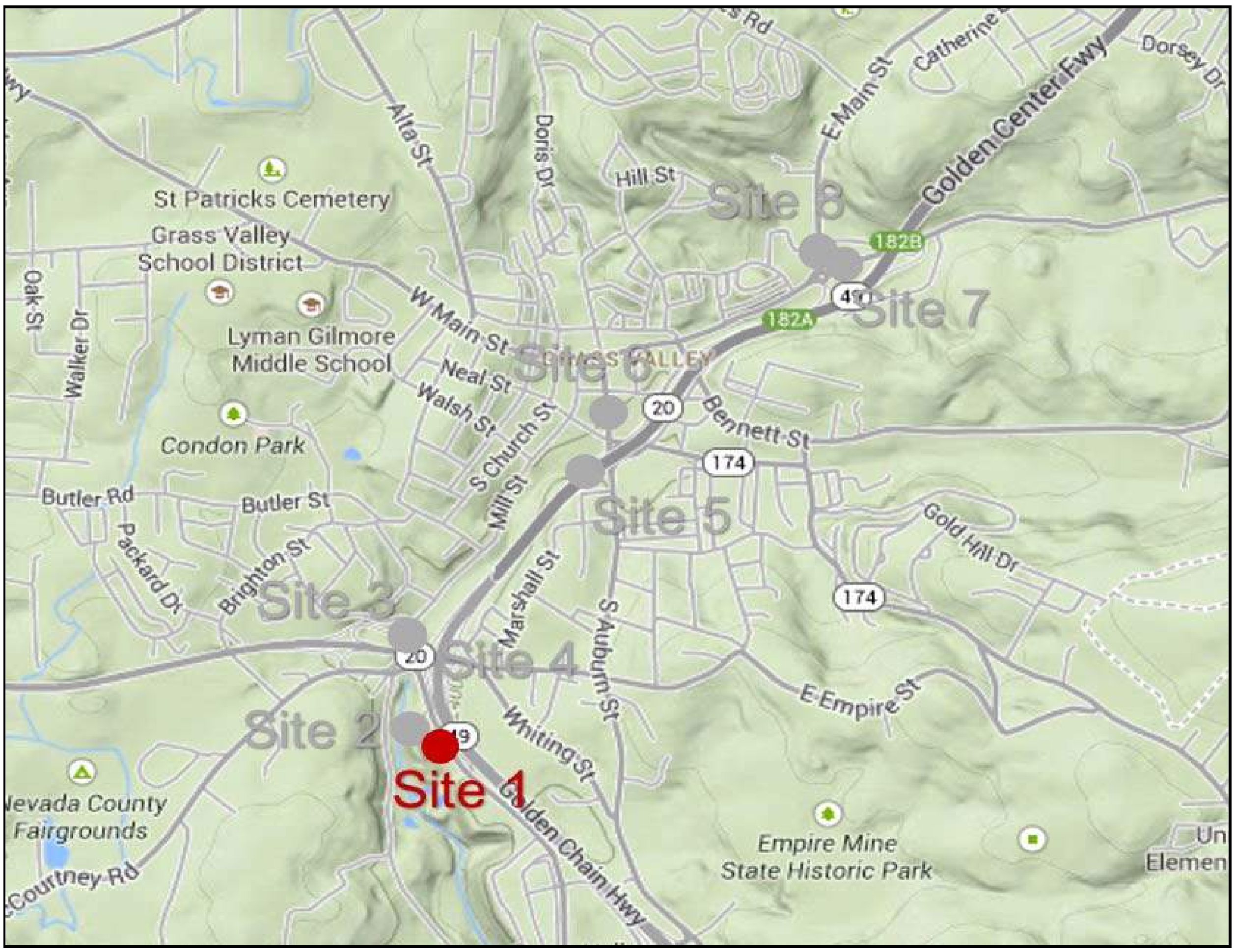
City of Grass Valley

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 1

Location: Treatment Plant Building 3

Data Summary Report



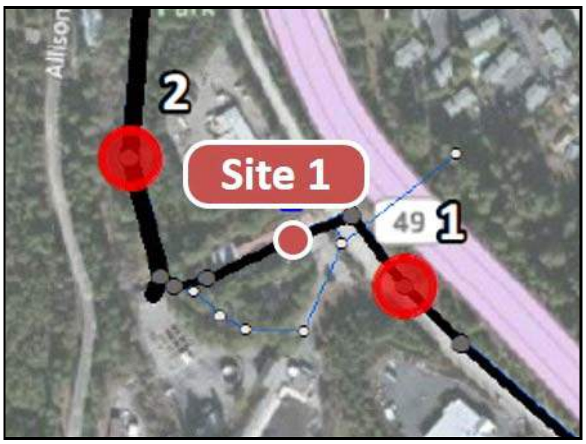
Vicinity Map: Site 1

SITE 1

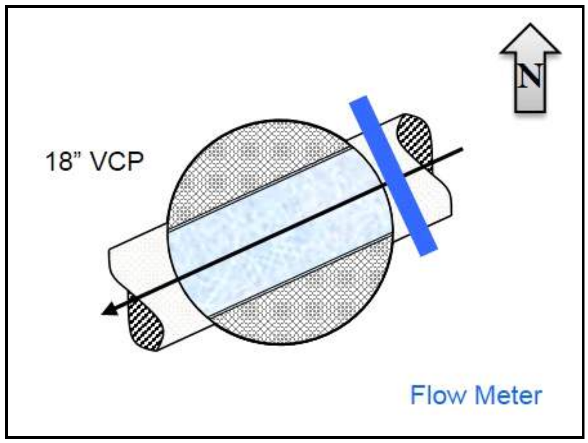
Site Information

Location:	Treatment Plant Building 3
Coordinates:	121.0680° W, 39.2064° N
Rim Elevation:	2381 feet
Pipe Diameter:	18 inches
Baseline Flow:	0.102 mgd
Peak Measured Flow:	0.685 mgd

Satellite Map

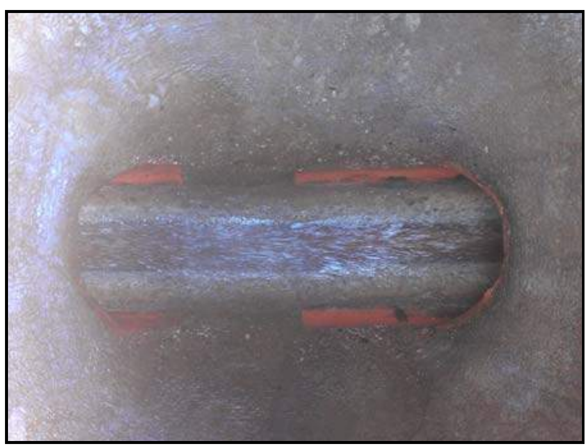


Sewer Map



Flow Sketch

Street View



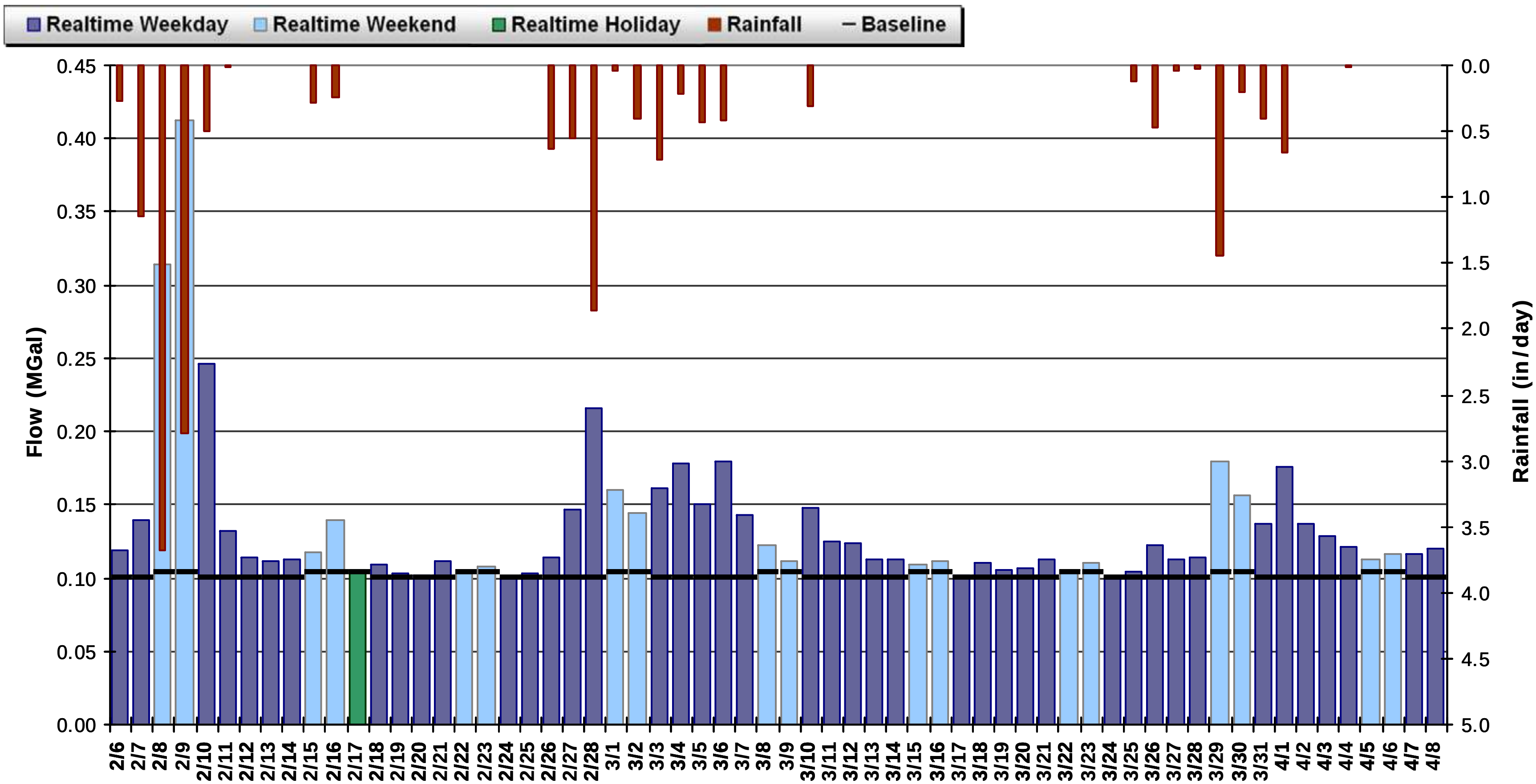
Plan View

SITE 1

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.135 MGal **Peak Daily Flow: 0.412 MGal** **Min Daily Flow: 0.099 MGal**

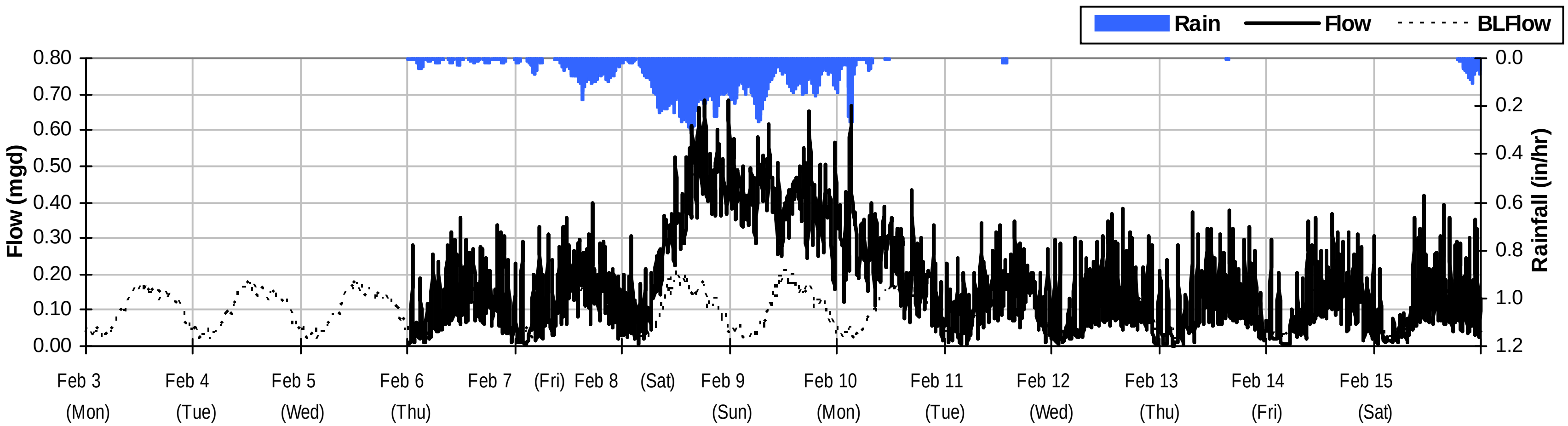
Total Period Rainfall: 17.91 inches

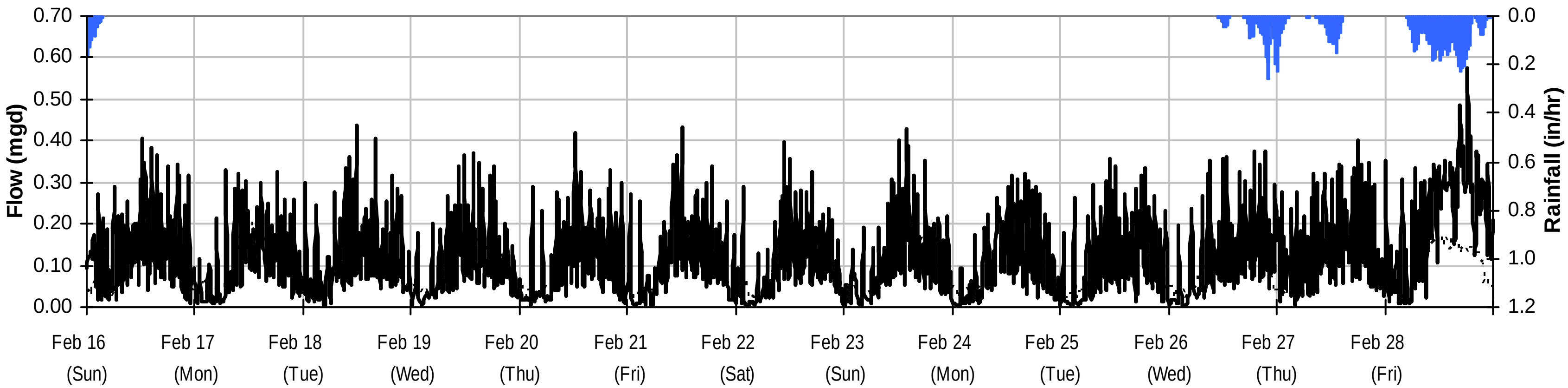


SITE 1

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.147 mgd** **Peak Flow: 0.685 mgd** **Min Flow: 0.002 mgd**

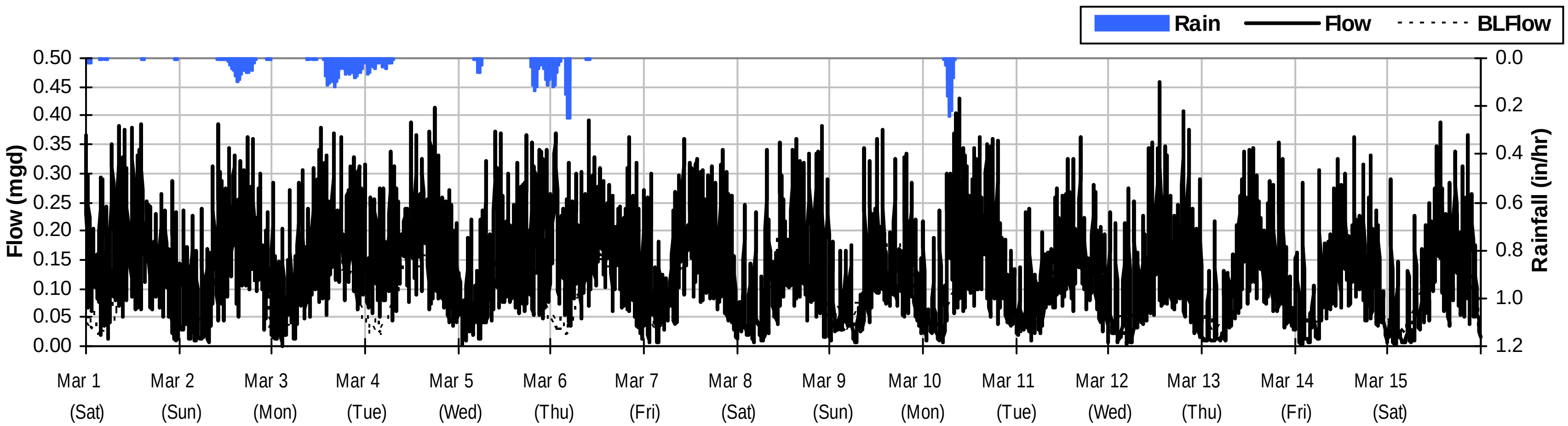


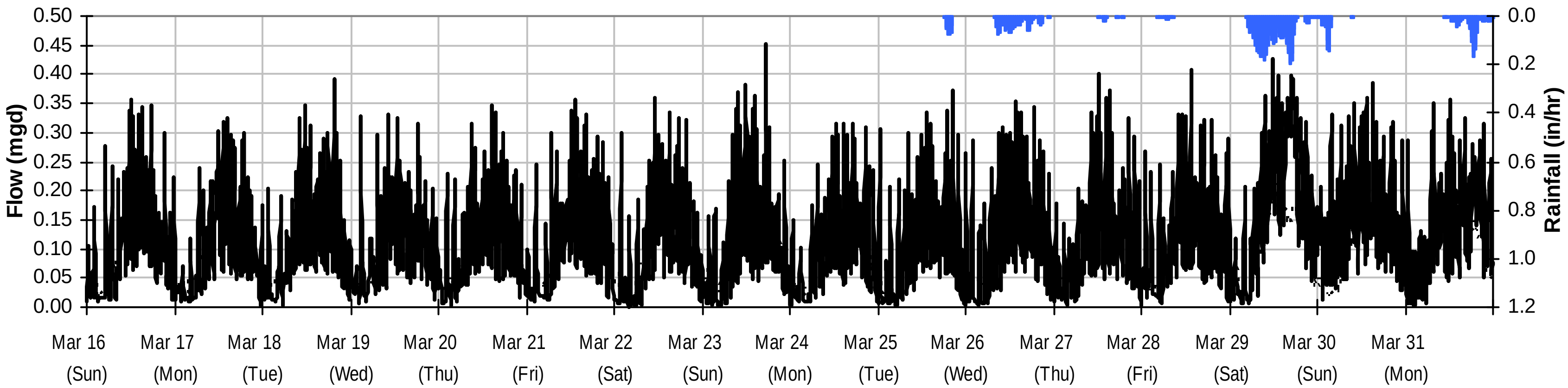


SITE 1

Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 5.24 inches **Avg Flow: 0.128 mgd** **Peak Flow: 0.458 mgd** **Min Flow: 0.002 mgd**

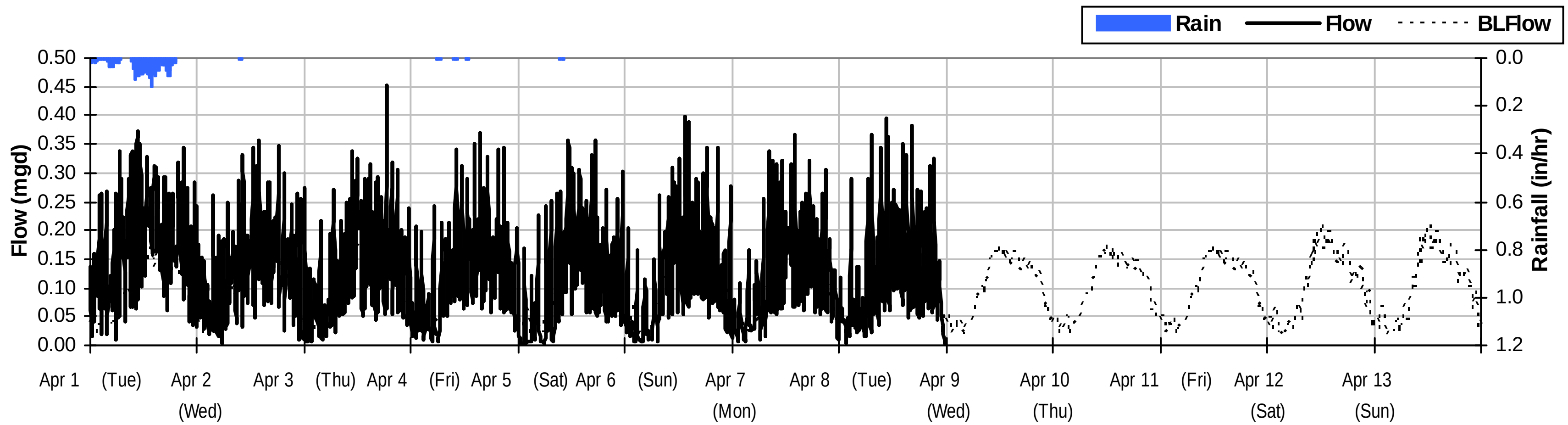




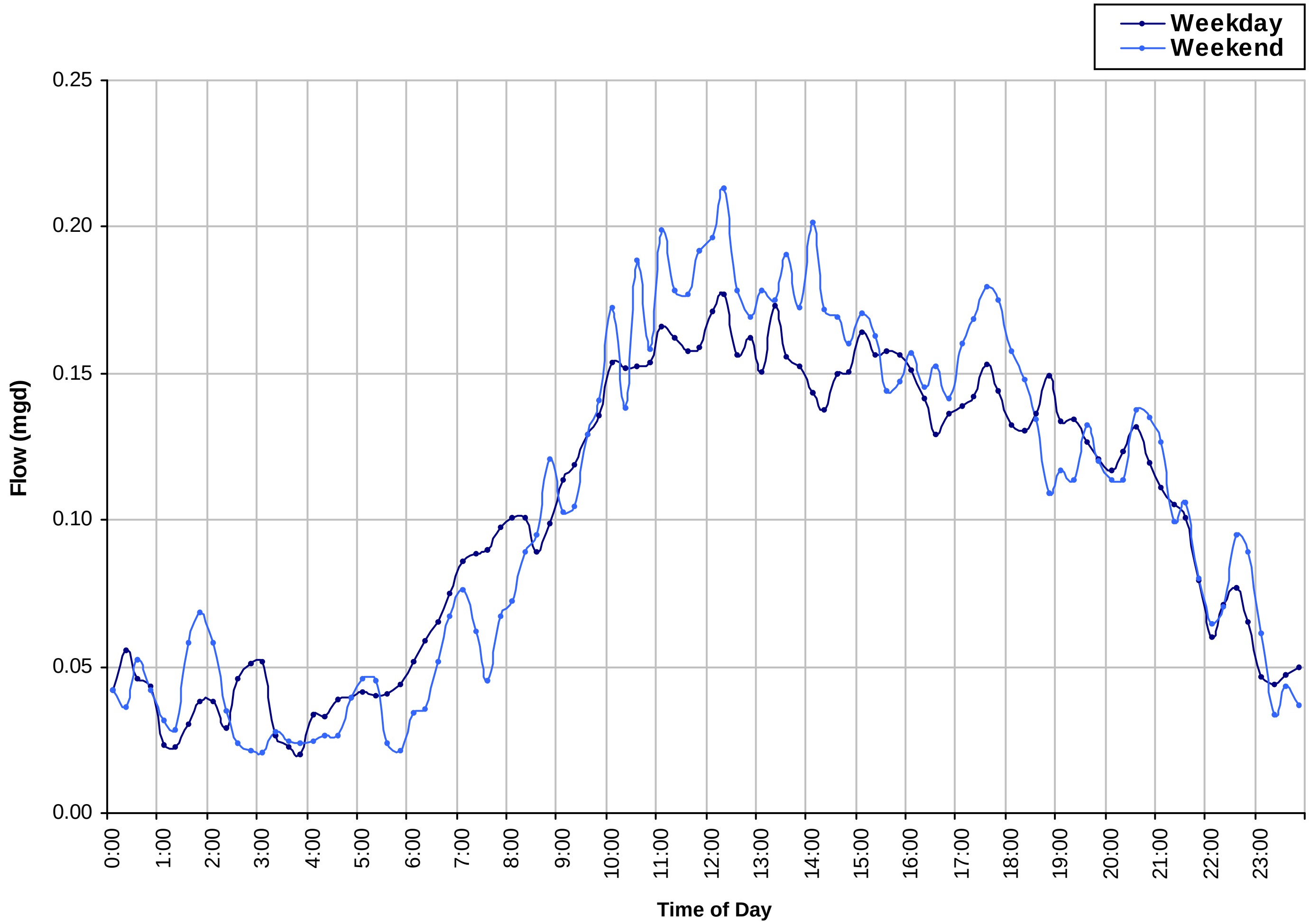
SITE 1

Monthly Flow Summary: April, 2014

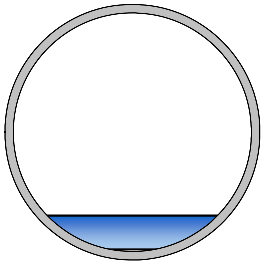
Total Monthly Rainfall: 0.69 inches **Avg Flow: 0.128 mgd** **Peak Flow: 0.452 mgd** **Min Flow: 0.004 mgd**



SITE 1
Baseline Flow Hydrographs

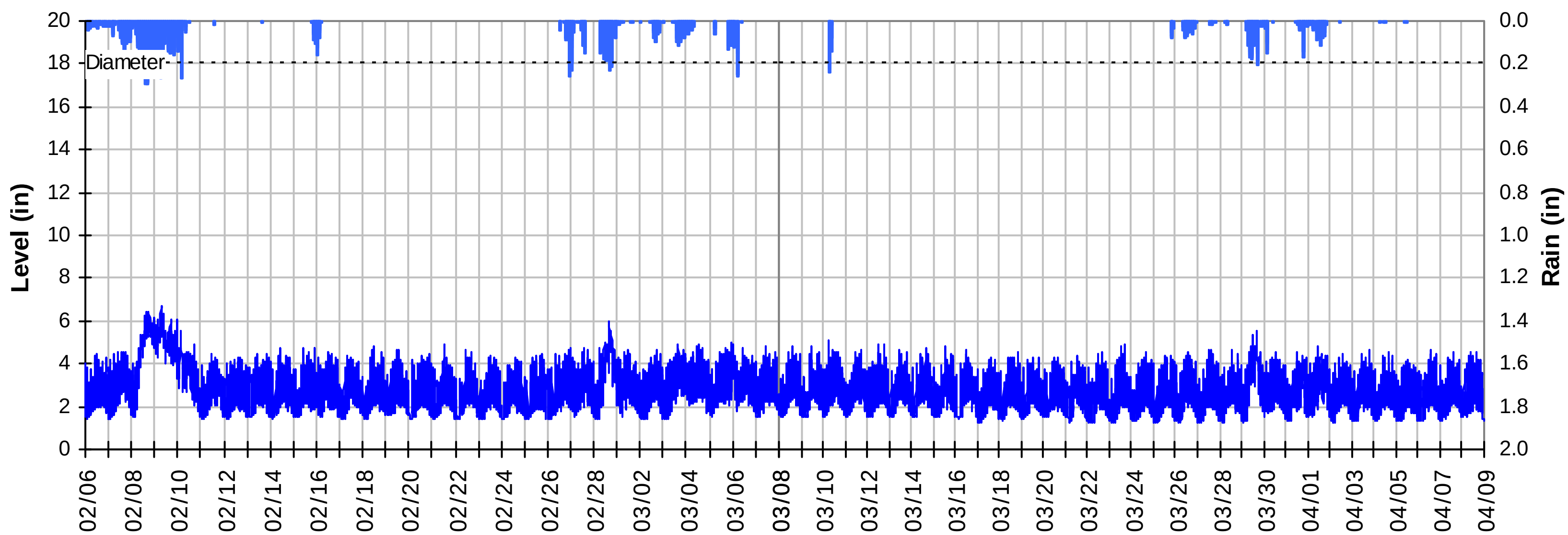


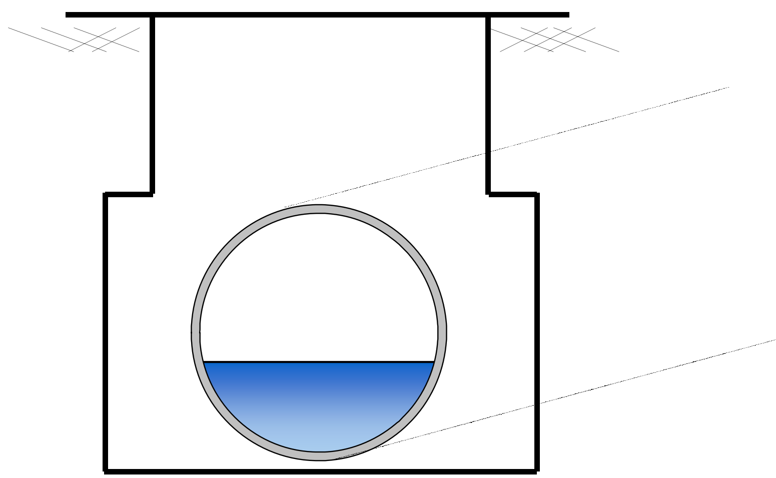
Baseline Flow:
0.102 *mgd*



SITE 1
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

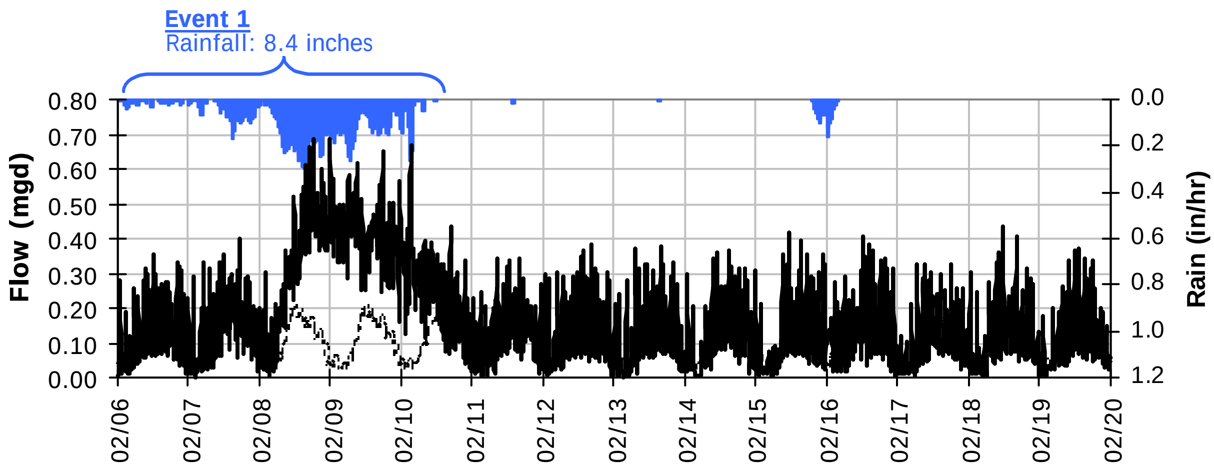




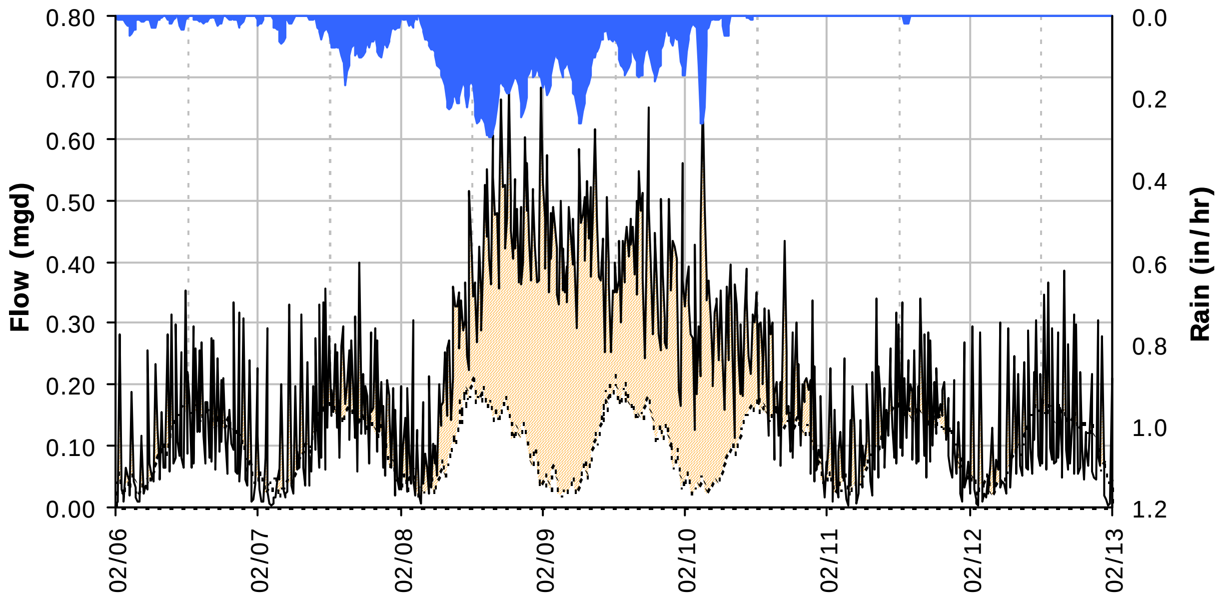
Pipe Diameter:	18	*inches*
Peak Measured Level:	6.73	*inches*
Peak d/D Ratio:	0.37	

SITE 1
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

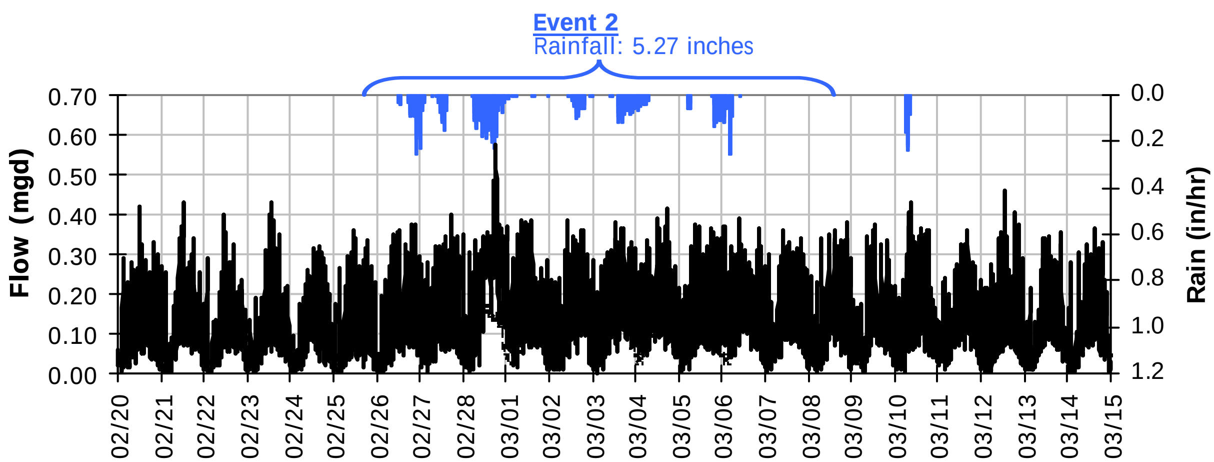


Storm Event I/I Analysis (Rain = 8.40 inches)

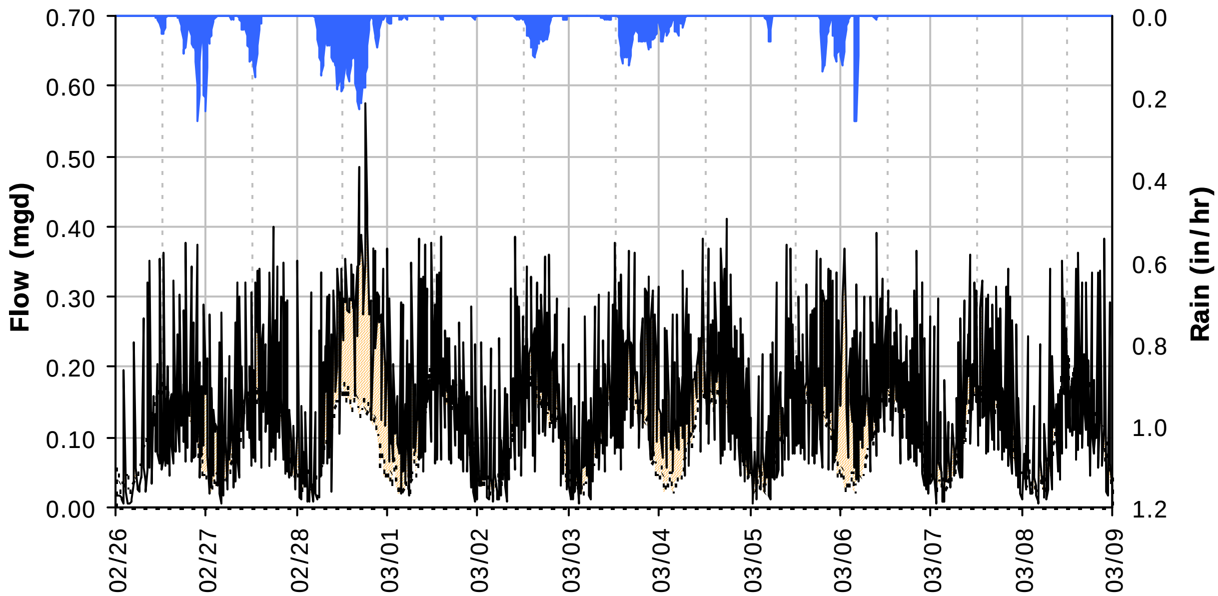
Capacity		Inflow / Infiltration	
Peak Flow:	0.68 *mgd*	**Peak I/I Rate:**	0.65 *mgd*
PF:	6.70	**Total I/I:**	761,000 *gallons*

SITE 1
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

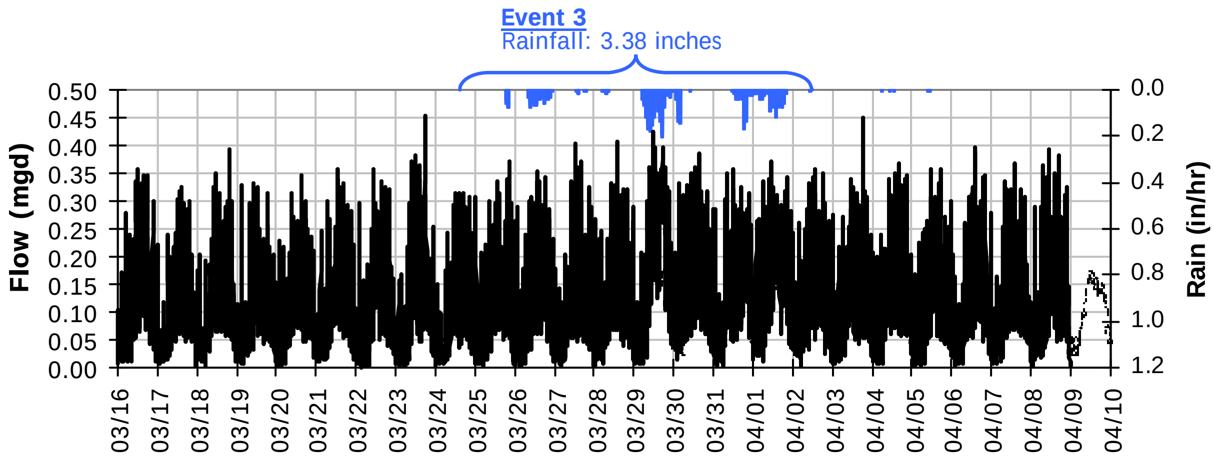


Storm Event I/I Analysis (Rain = 5.27 inches)

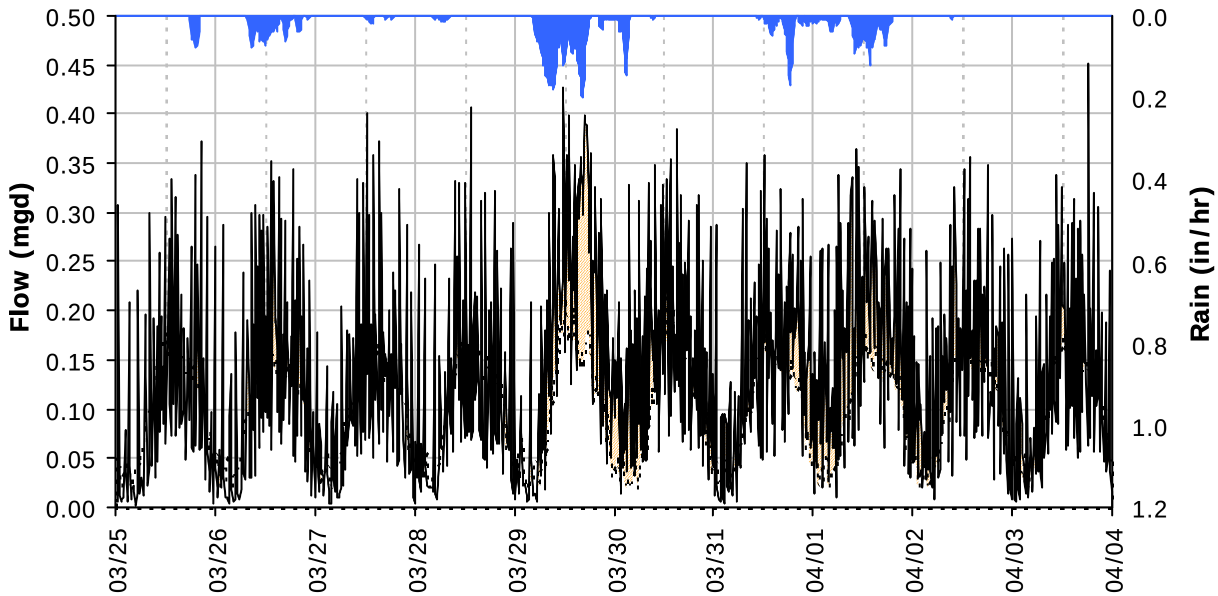
Capacity		Inflow / Infiltration	
Peak Flow:	0.57 *mgd*	**Peak I/I Rate:**	0.44 *mgd*
PF:	5.61	**Total I/I:**	591,000 *gallons*

SITE 1
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



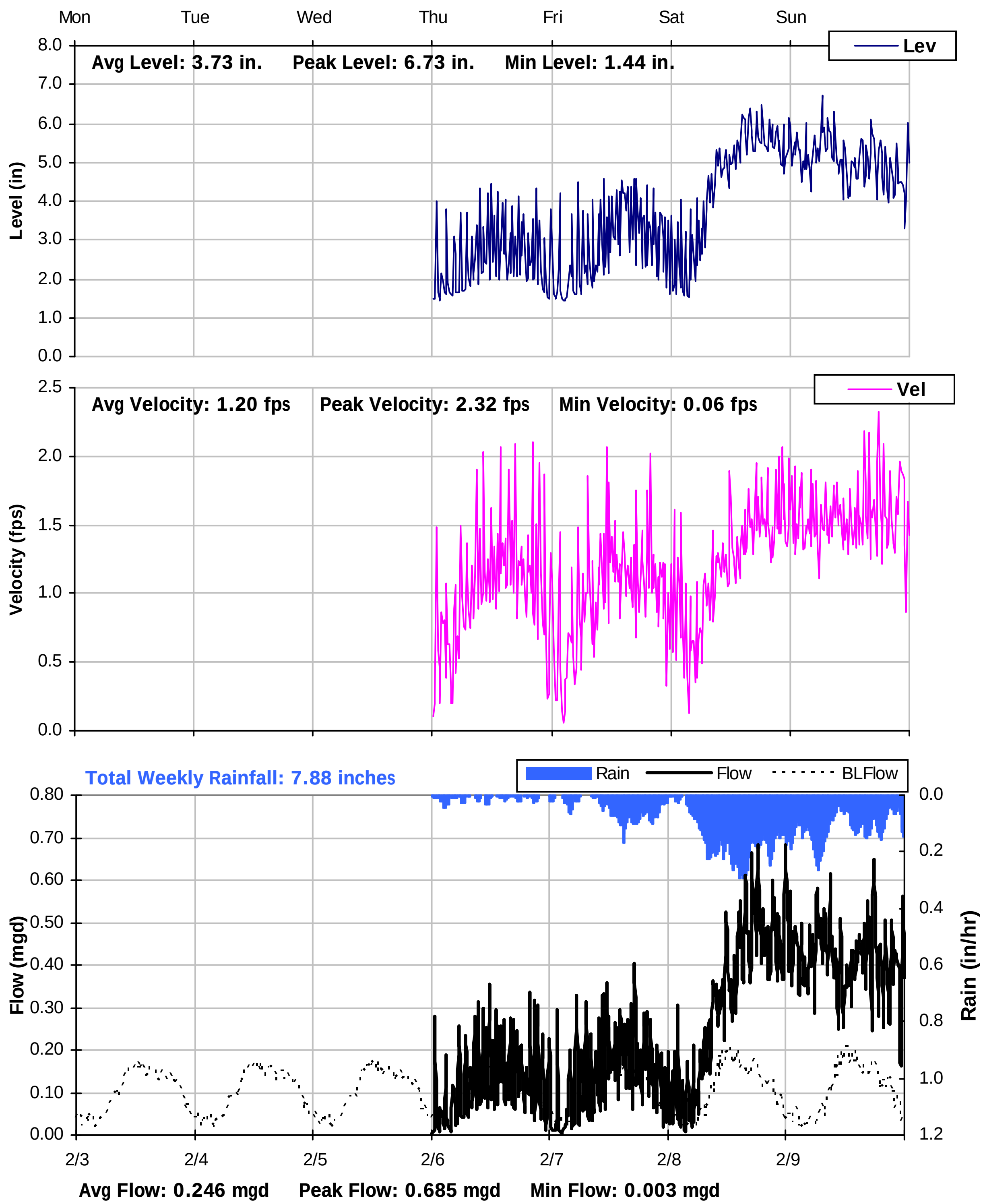
Event 3 Detail Graph



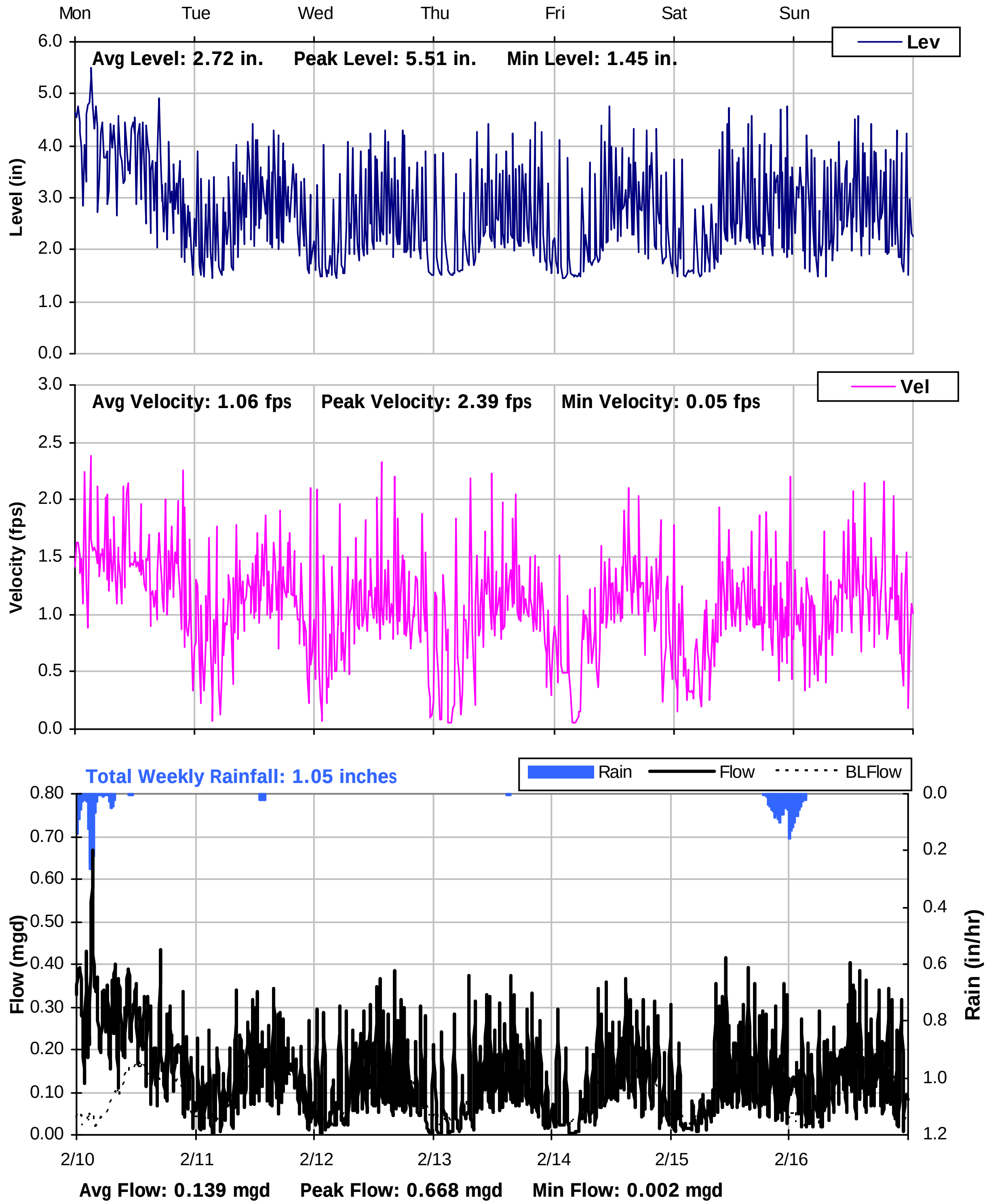
Storm Event I/I Analysis (Rain = 3.38 inches)

Capacity		**Inflow / Infiltration**	
Peak Flow:	0.45 *mgd*	**Peak I/I Rate:**	0.32 *mgd*
PF:	4.42	**Total I/I:**	350,000 *gallons*

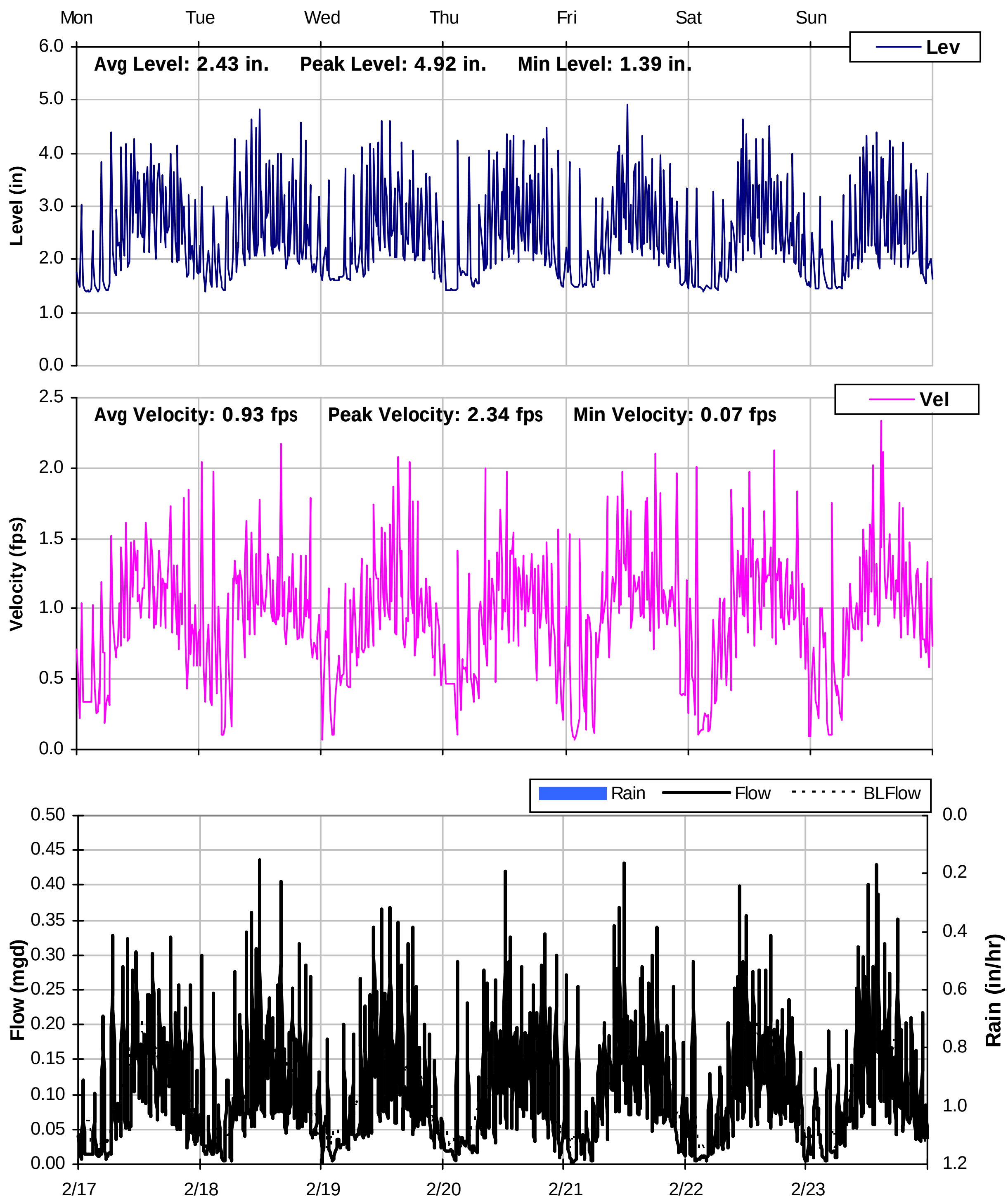
SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014



SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014

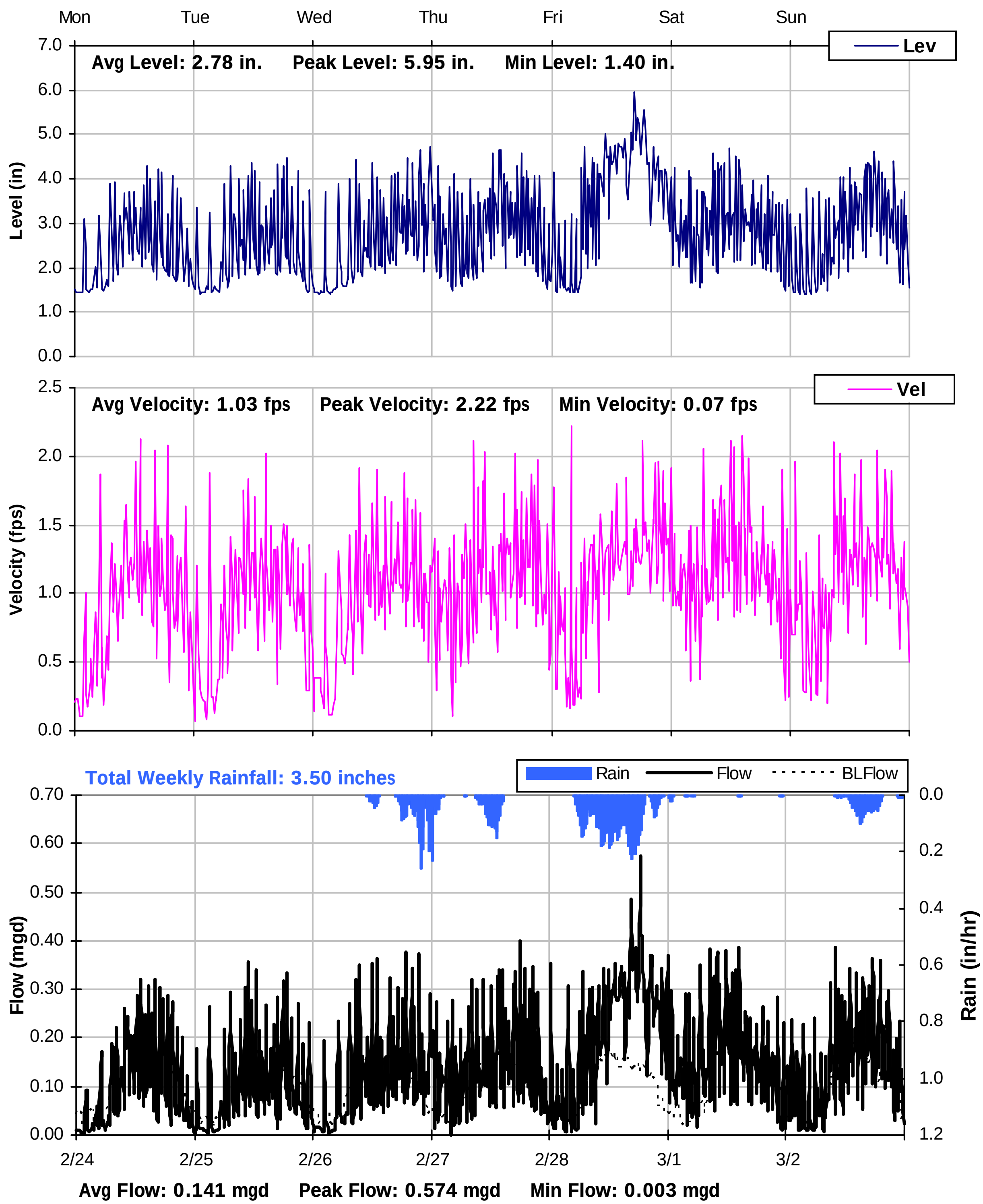


SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014

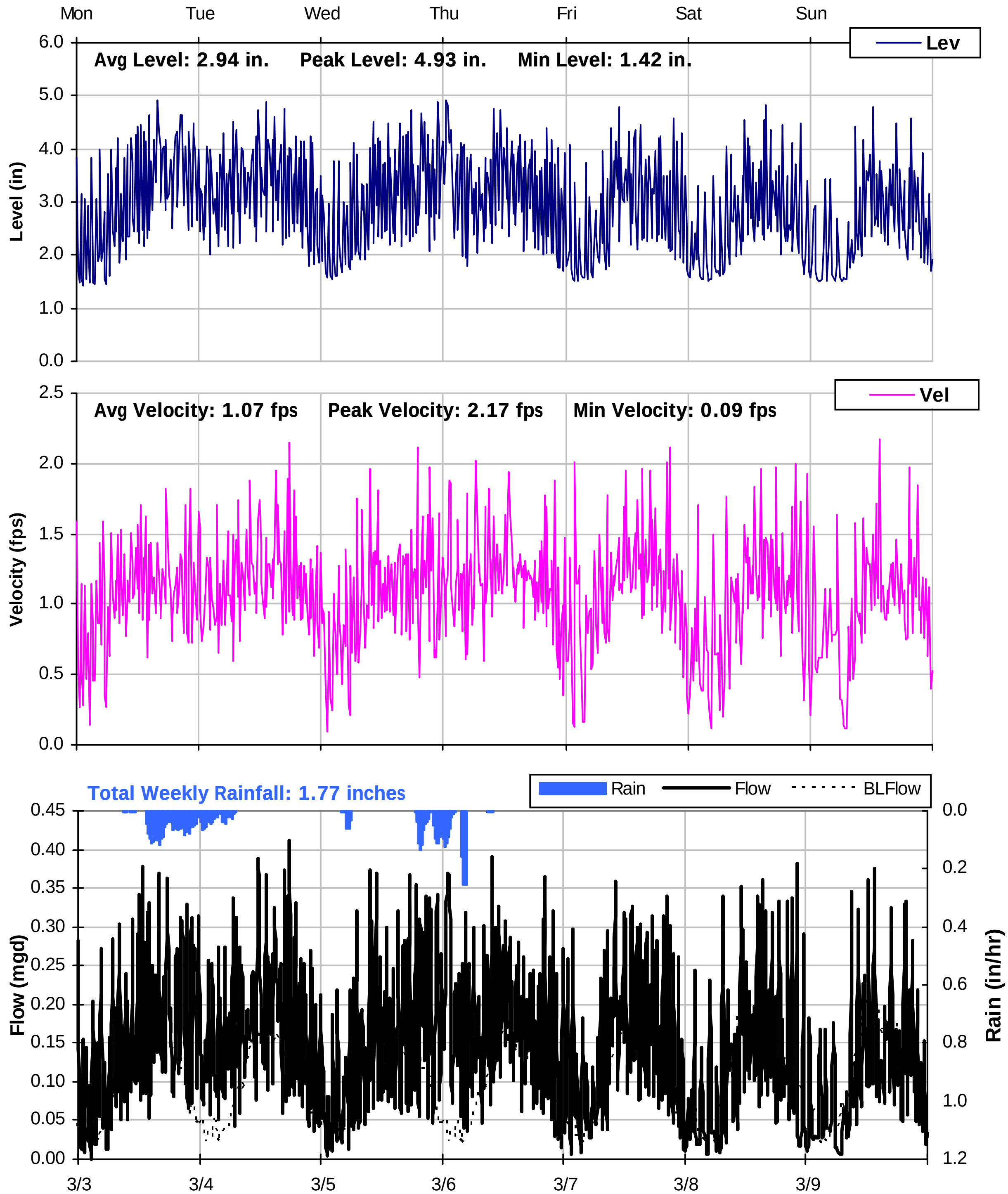


Avg Flow: 0.106 mgd   Peak Flow: 0.435 mgd   Min Flow: 0.003 mgd

SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



SITE 1
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014

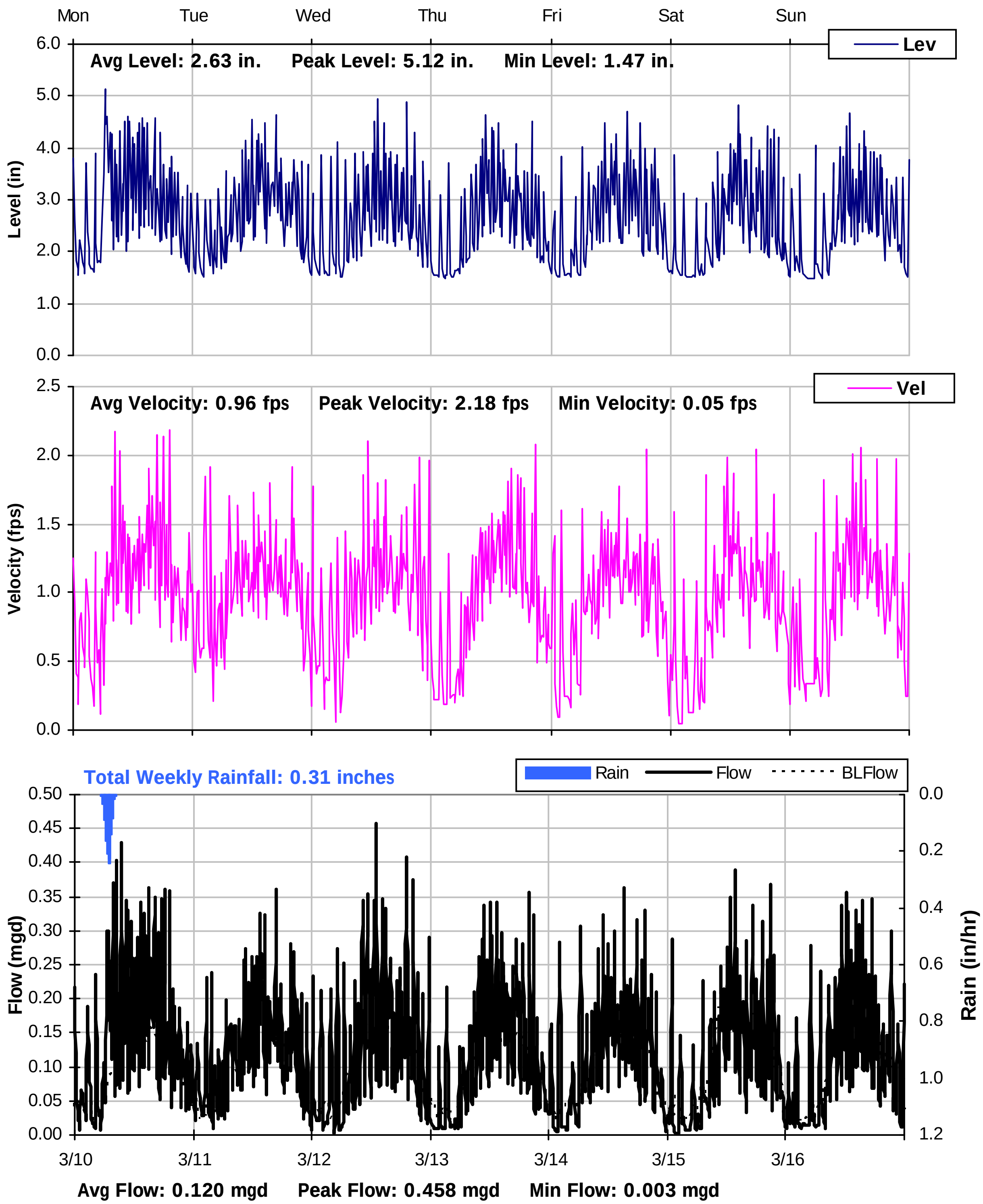


Avg Flow: 0.149 mgd **Peak Flow: 0.410 mgd** **Min Flow: 0.004 mgd**

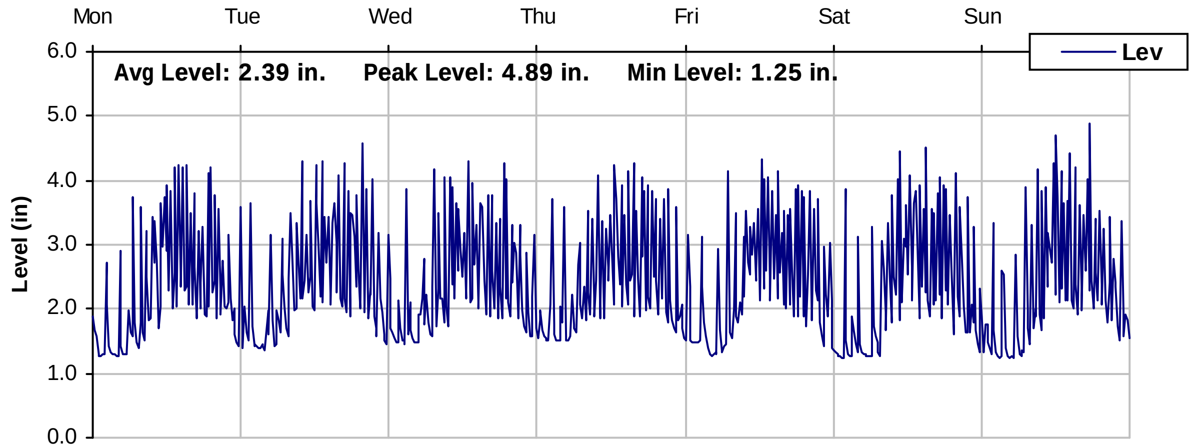
SITE 1

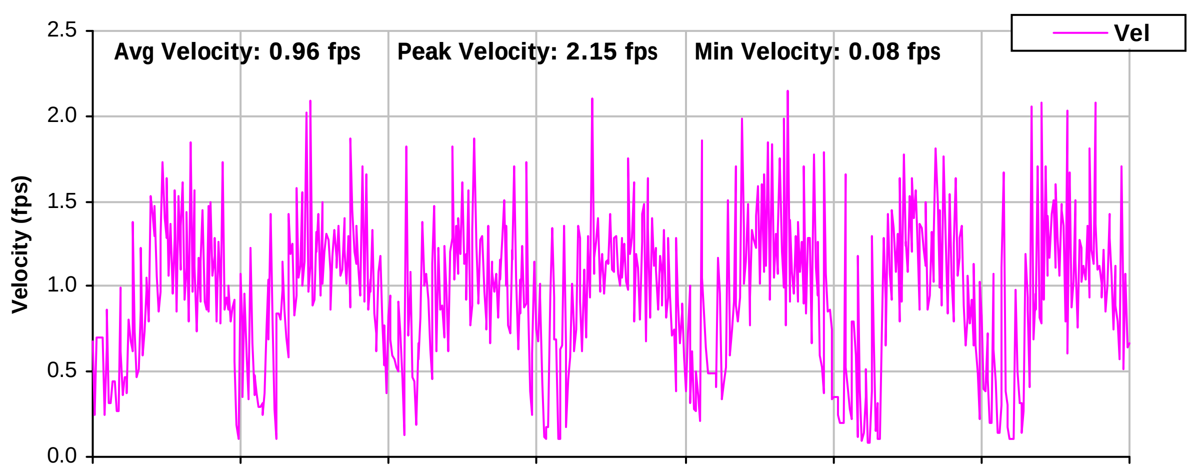
Weekly Level, Velocity and Flow Hydrographs

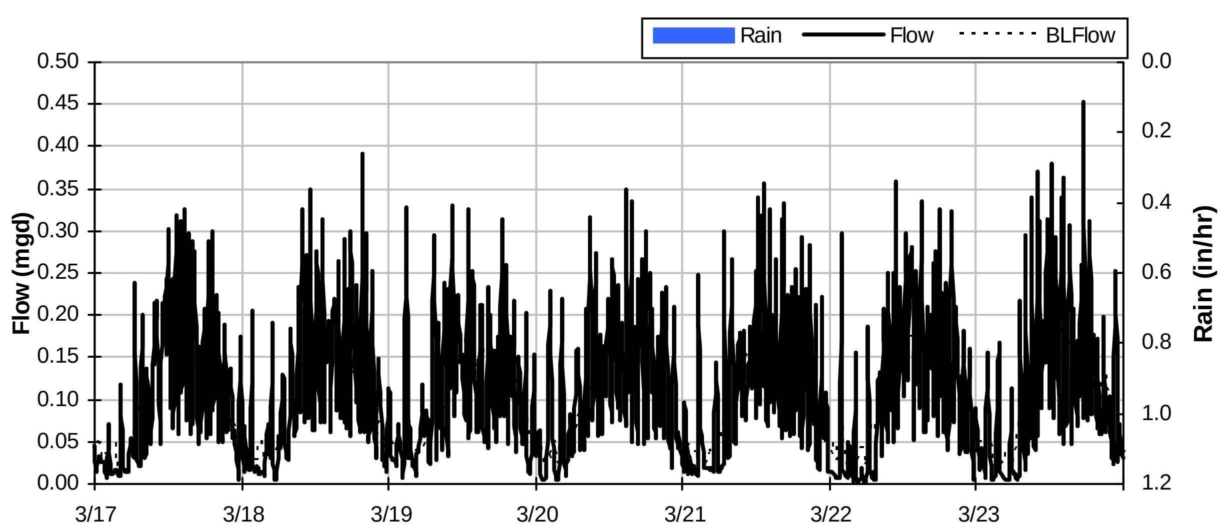
3/10/2014 to 3/17/2014



SITE 1
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014

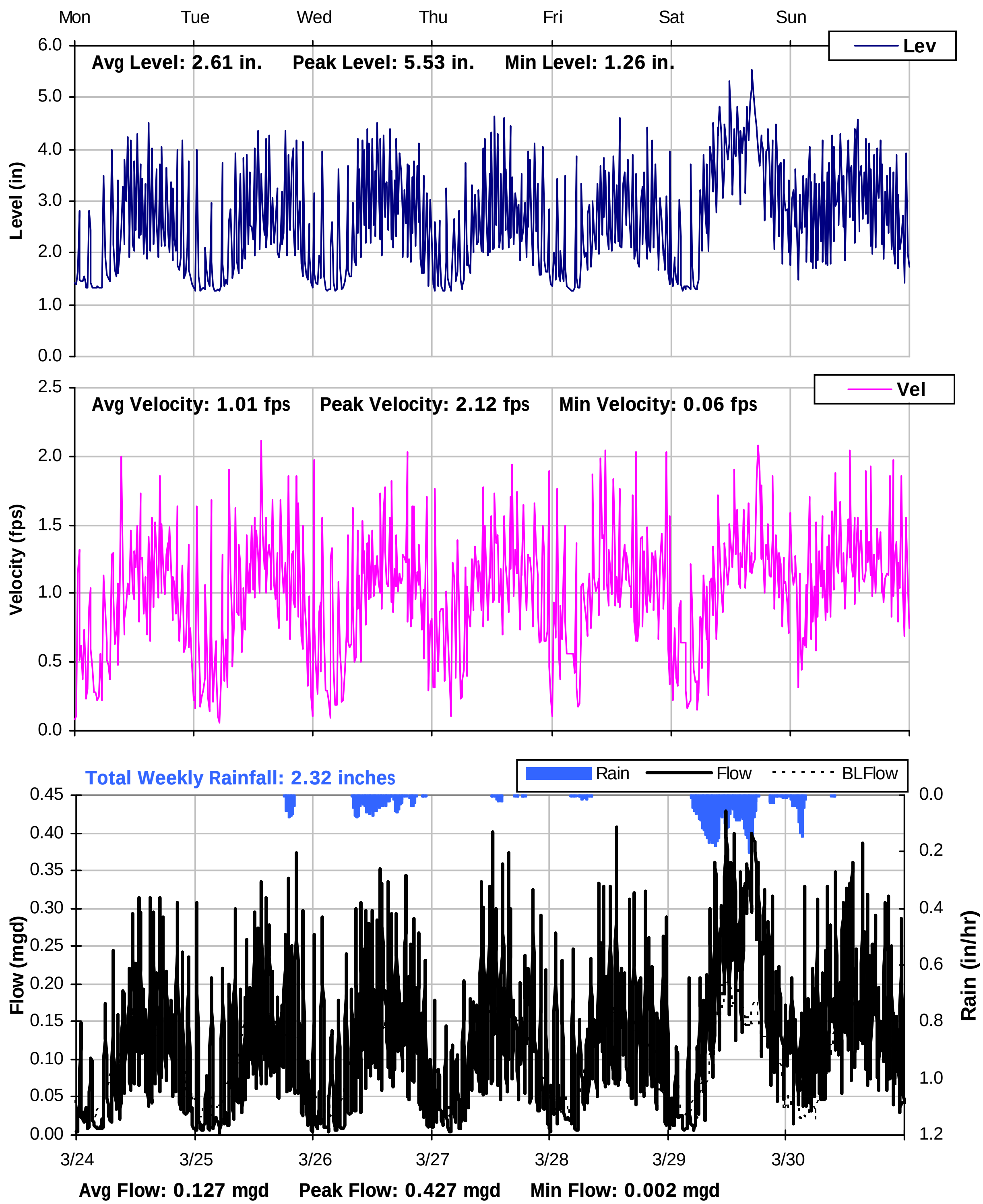




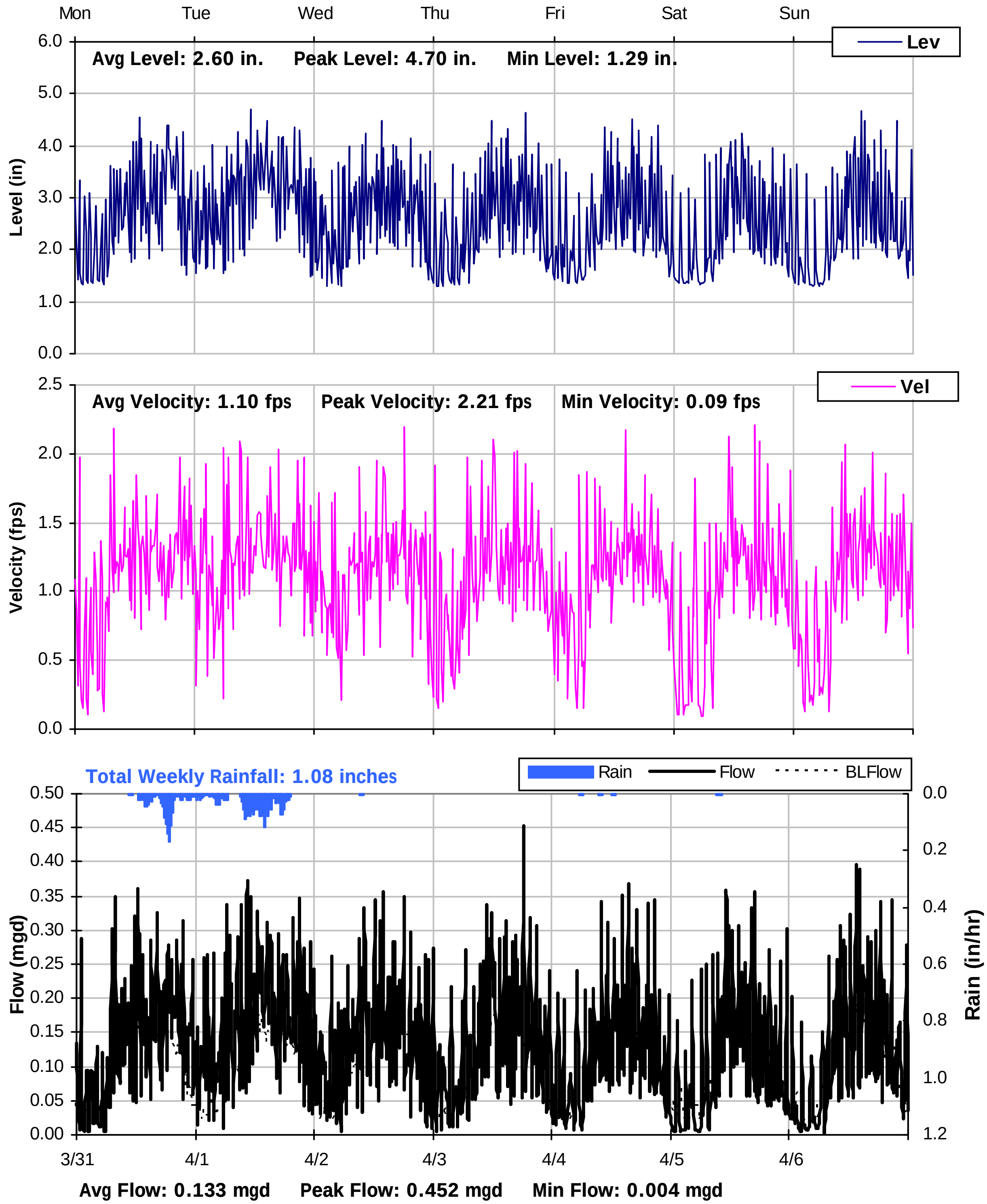


Avg Flow: 0.108 mgd **Peak Flow: 0.454 mgd** **Min Flow: 0.003 mgd**

SITE 1
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014

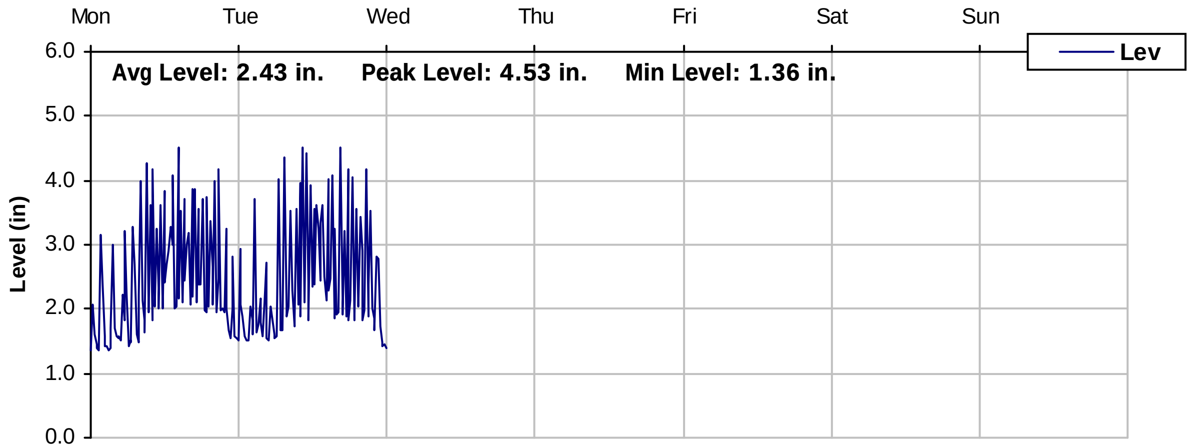


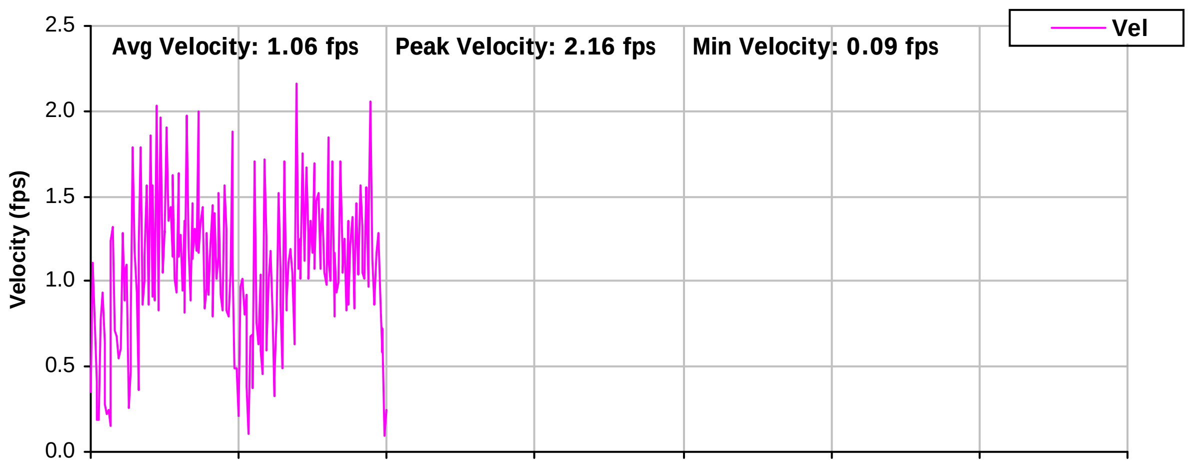
SITE 1
Weekly Level, Velocity and Flow Hydrographs
3/31/2014 to 4/7/2014

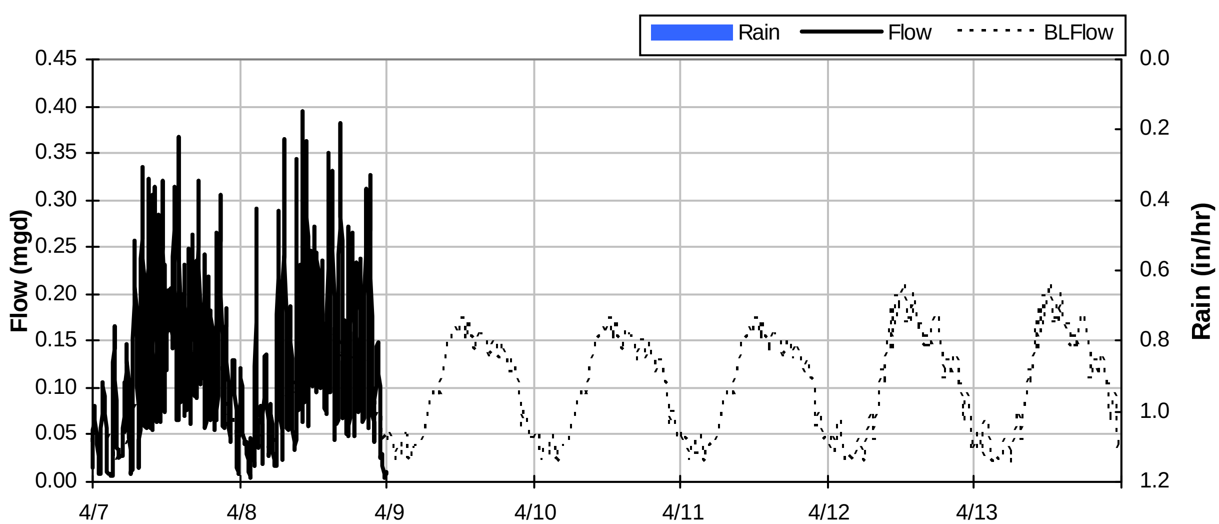


Avg Flow: 0.133 mgd Peak Flow: 0.452 mgd Min Flow: 0.004 mgd

SITE 1
Weekly Level, Velocity and Flow Hydrographs
4/7/2014 to 4/14/2014







Avg Flow: 0.118 mgd **Peak Flow: 0.394 mgd** **Min Flow: 0.004 mgd**

City of Grass Valley

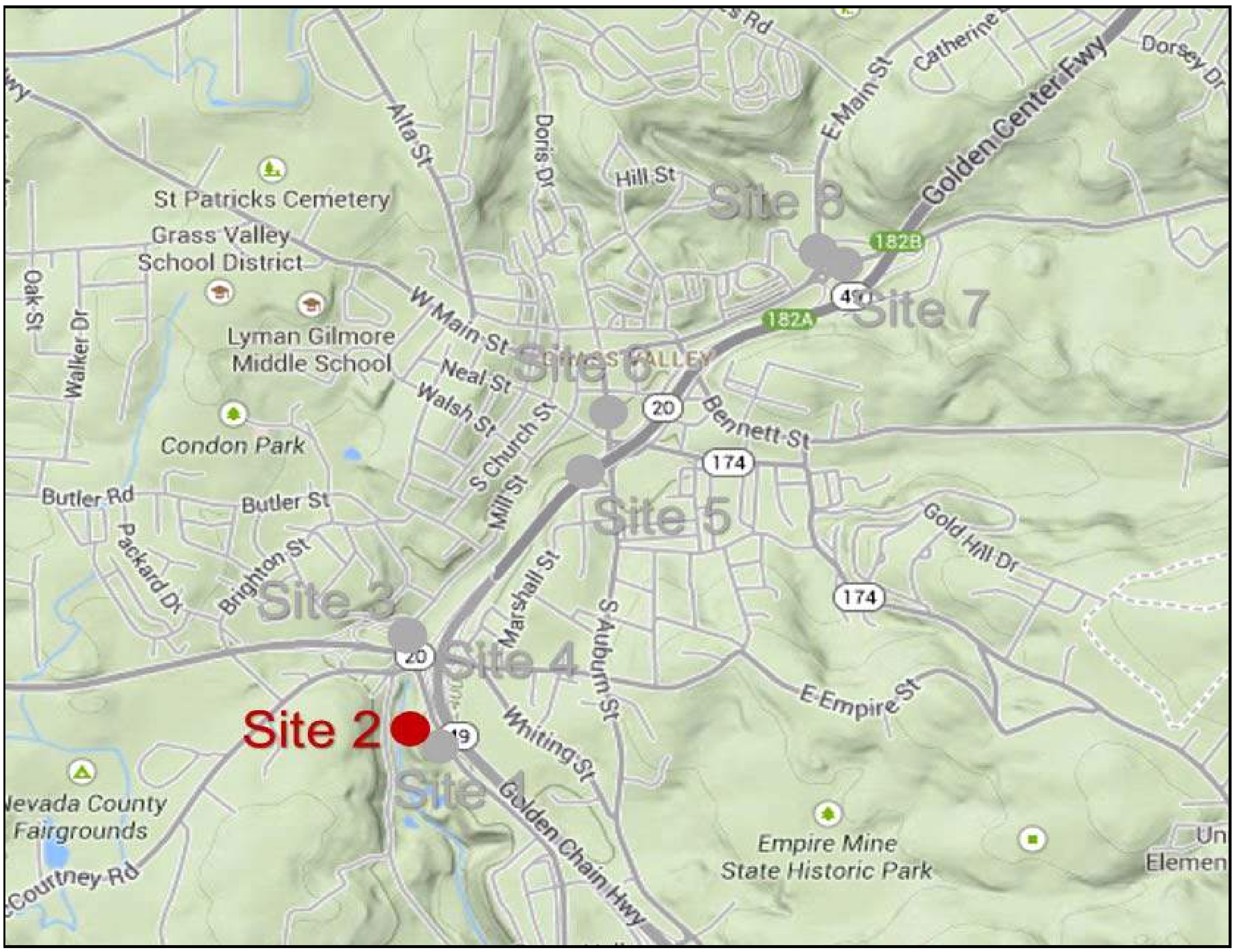
Sanitary Sewer Flow Monitoring

Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 2

Location: Allison Ranch Road

Data Summary Report



Vicinity Map: Site 2

SITE 2

Site Information

Location: Allison Ranch Road

Coordinates: 121.0696° W, 39.2070° N

Rim Elevation: 2347 feet

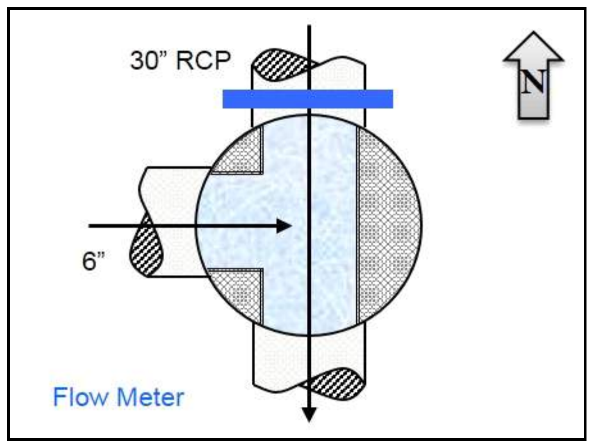
Pipe Diameter: 30 inches

Baseline Flow: 1.238 mgd

Peak Measured Flow: 9.135 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

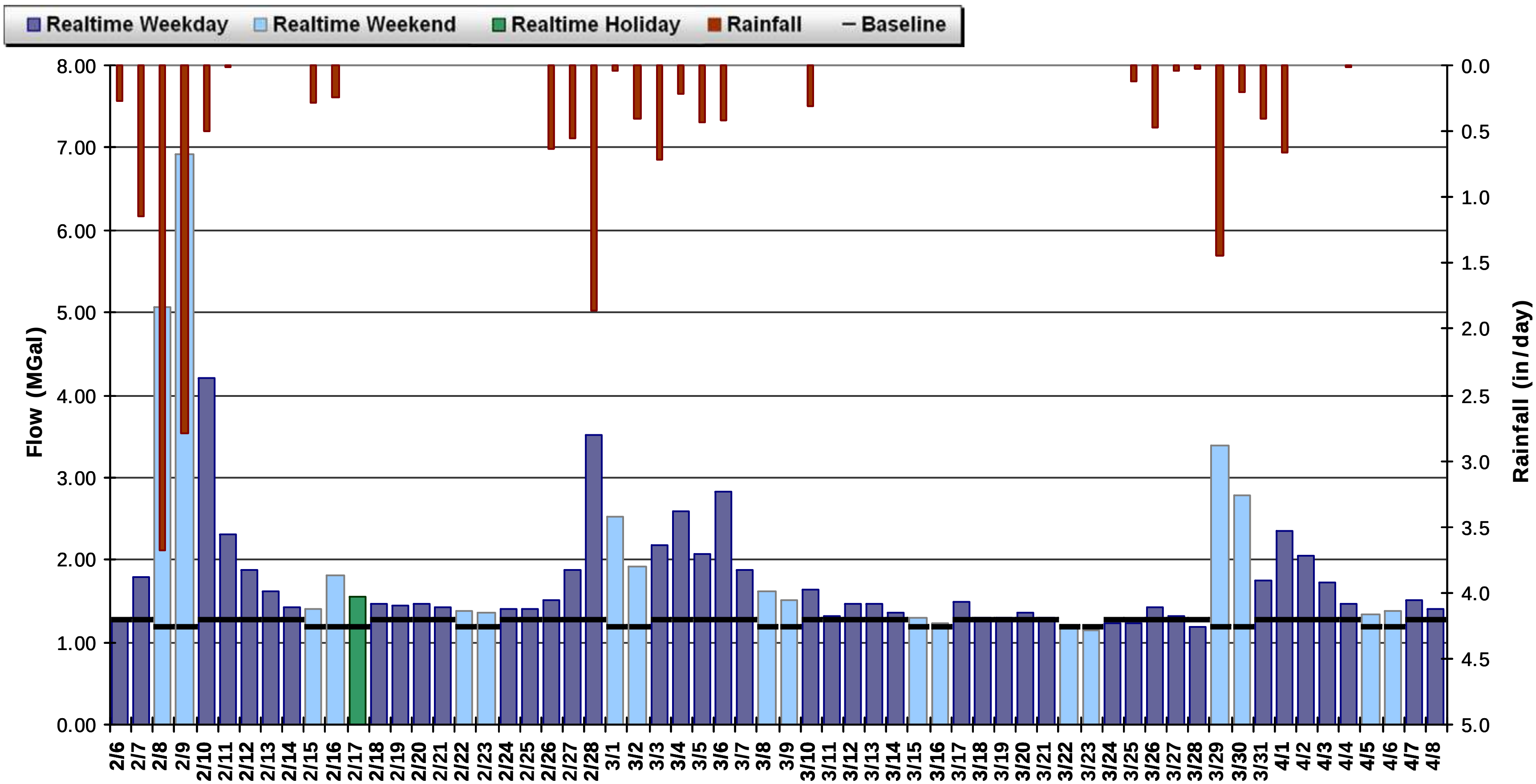
Plan View

SITE 2

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 1.855 MGal **Peak Daily Flow: 6.917 MGal** **Min Daily Flow: 1.148 MGal**

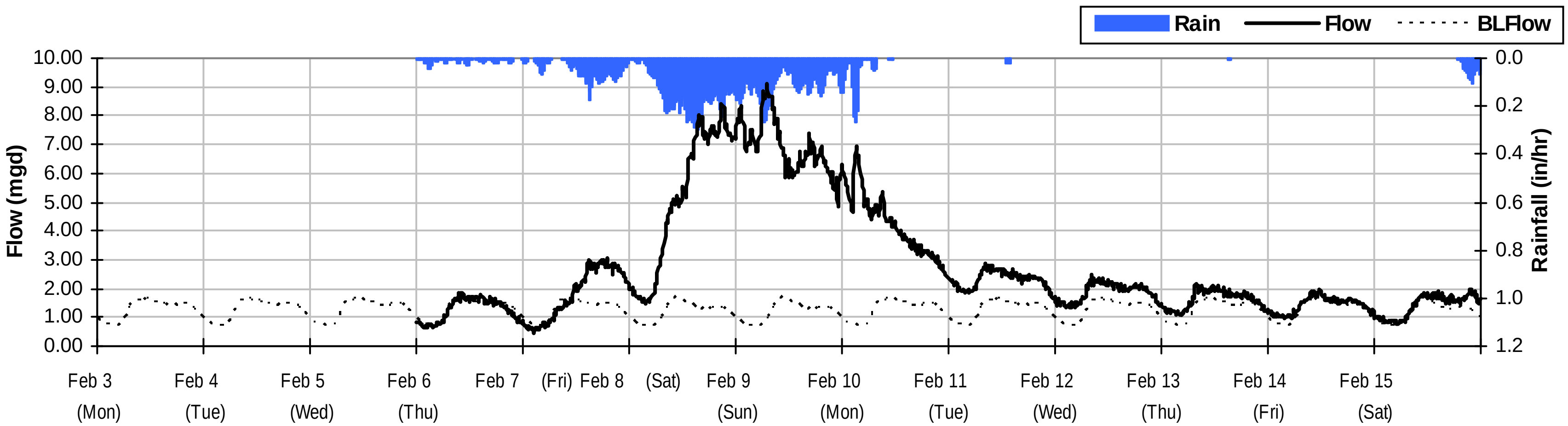
Total Period Rainfall: 17.91 inches

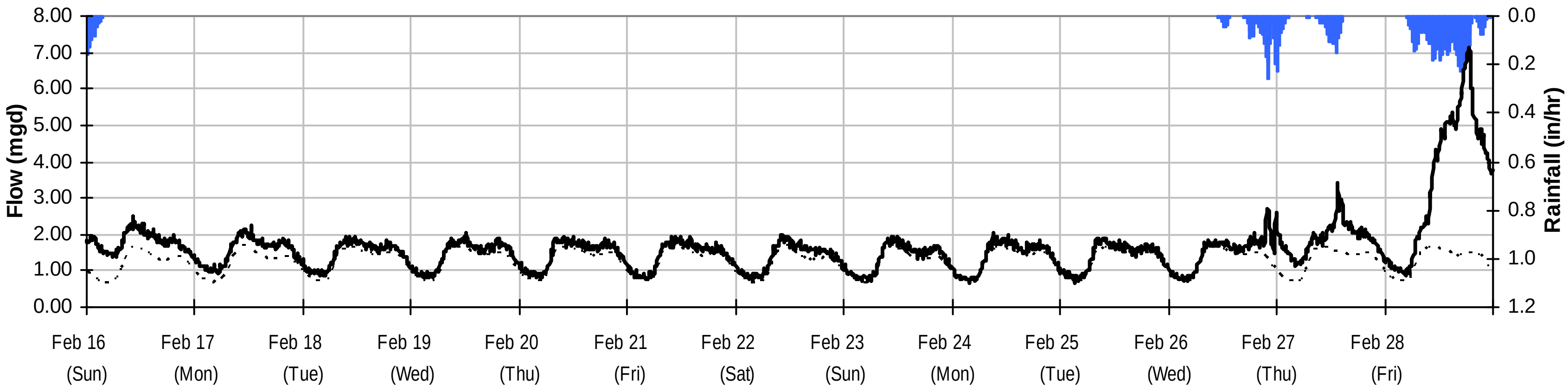


SITE 2

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 2.153 mgd** **Peak Flow: 9.135 mgd** **Min Flow: 0.453 mgd**

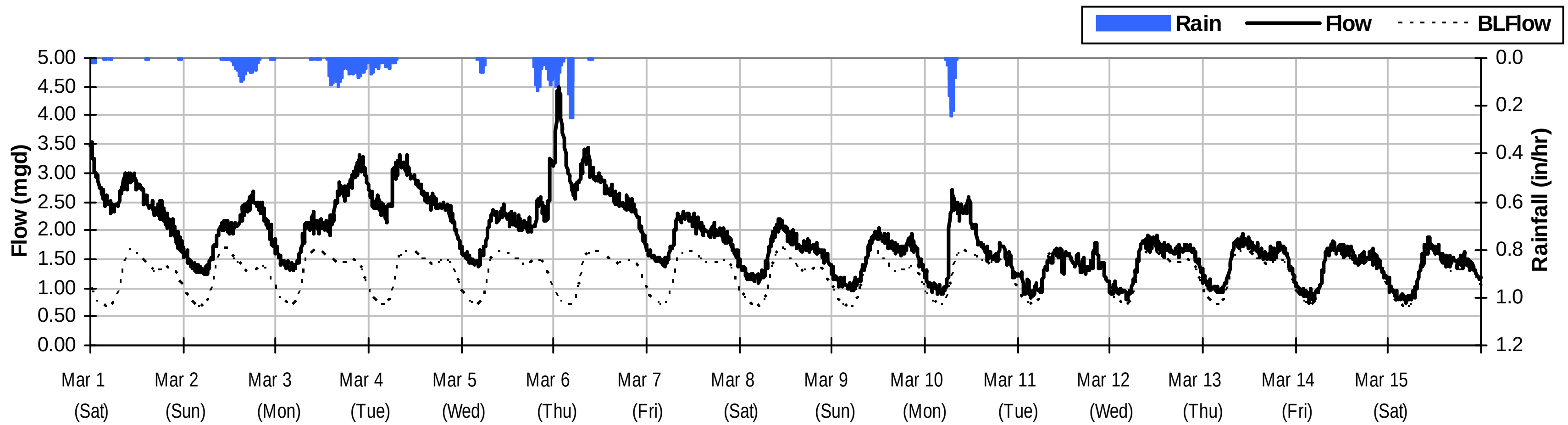


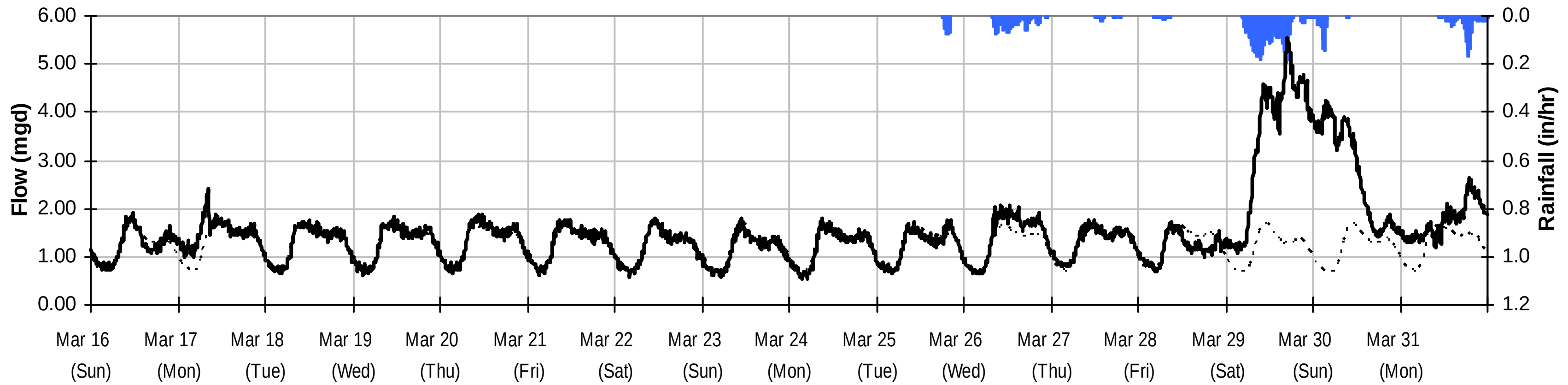


SITE 2

Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 5.24 inches **Avg Flow: 1.686 mgd** **Peak Flow: 5.559 mgd** **Min Flow: 0.543 mgd**

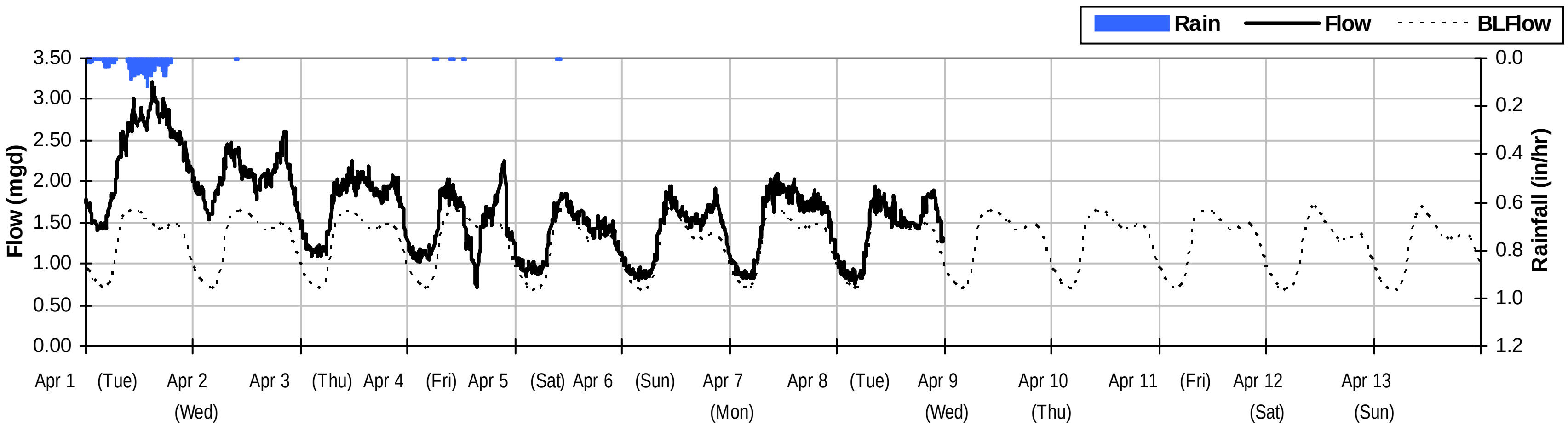




SITE 2

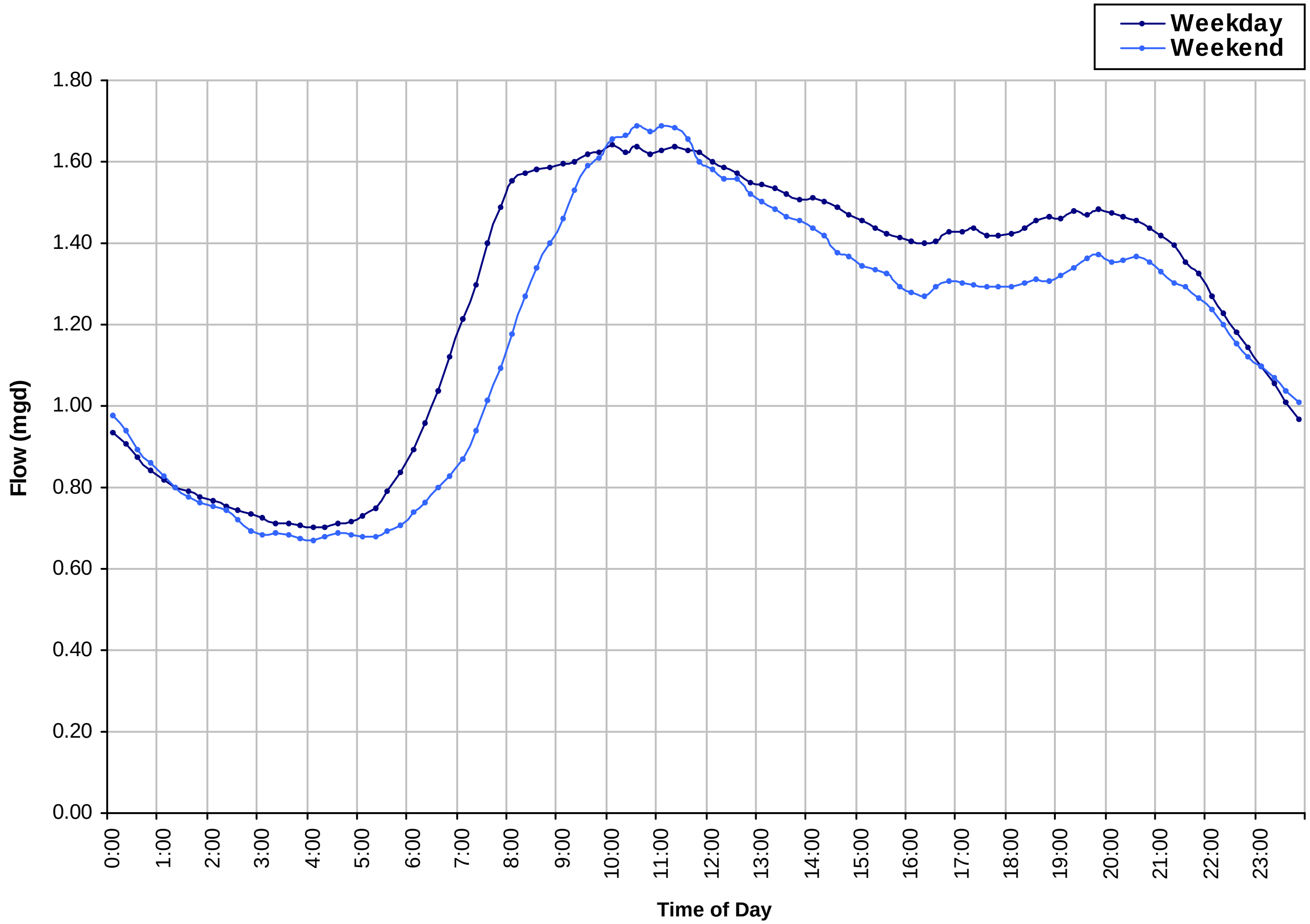
Monthly Flow Summary: April, 2014

Total Monthly Rainfall: 0.69 inches **Avg Flow: 1.651 mgd** **Peak Flow: 3.221 mgd** **Min Flow: 0.709 mgd**

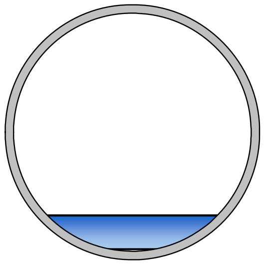


SITE 2

Baseline Flow Hydrographs

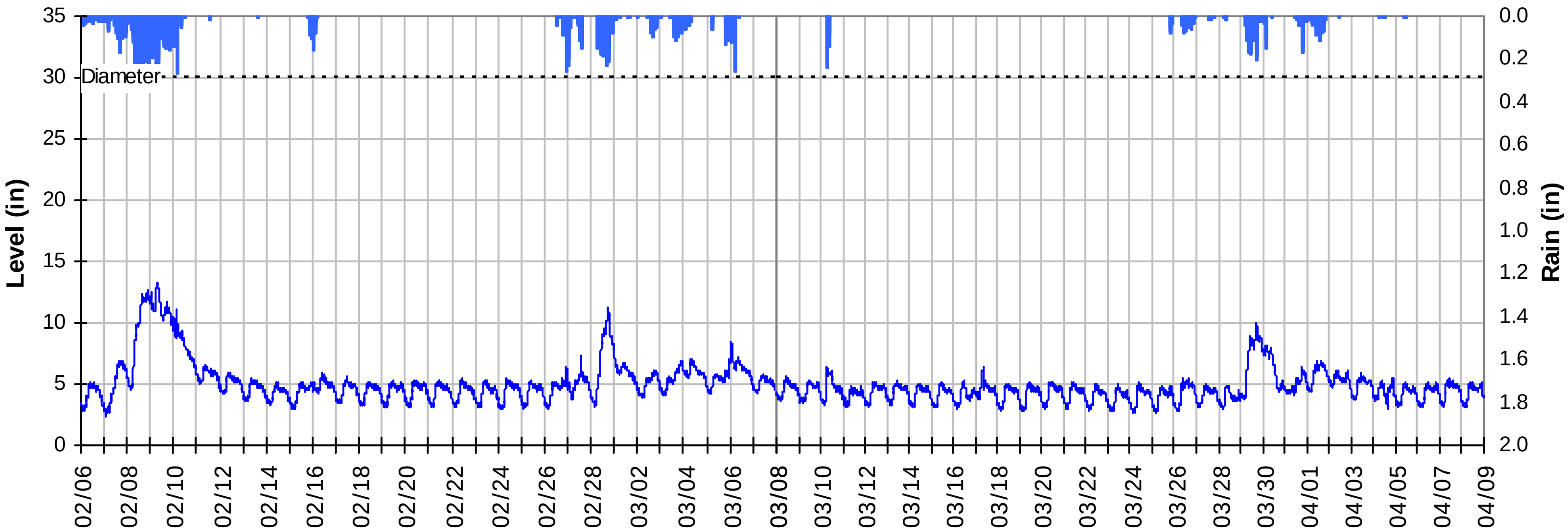


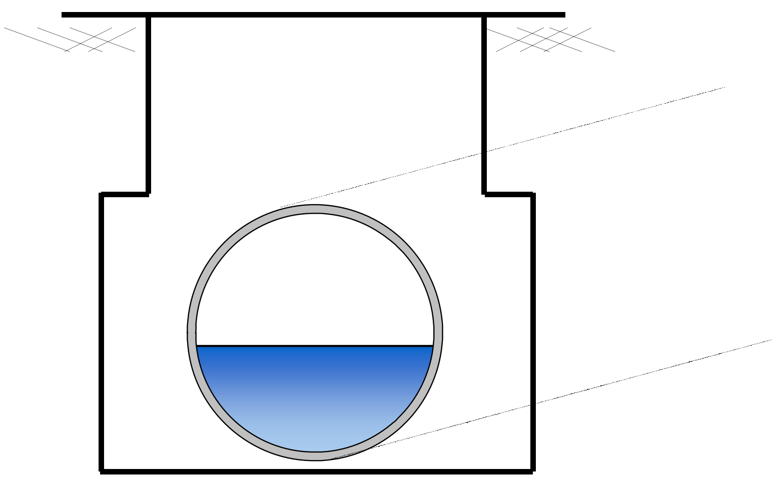
Baseline Flow:
1.238 *mgd*



SITE 2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

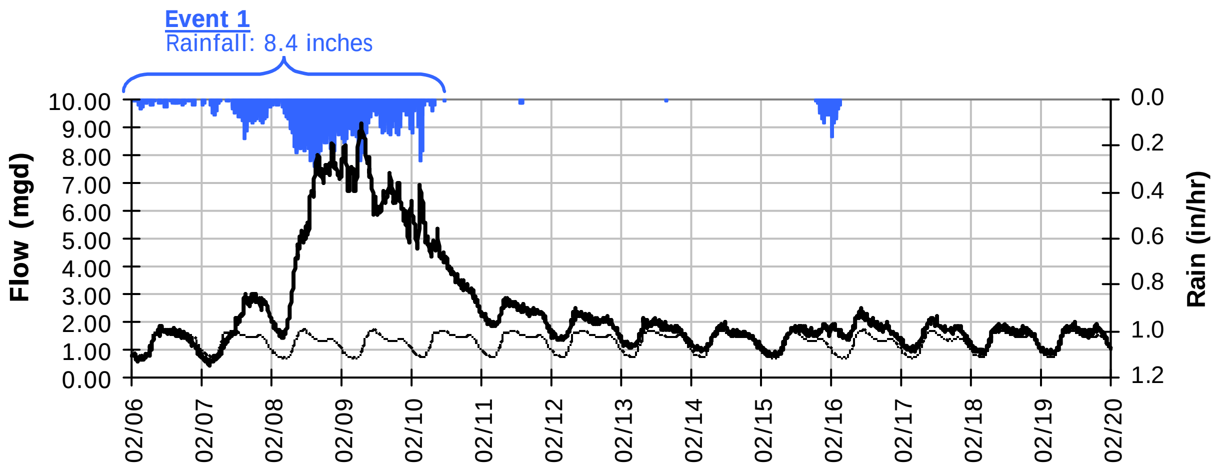




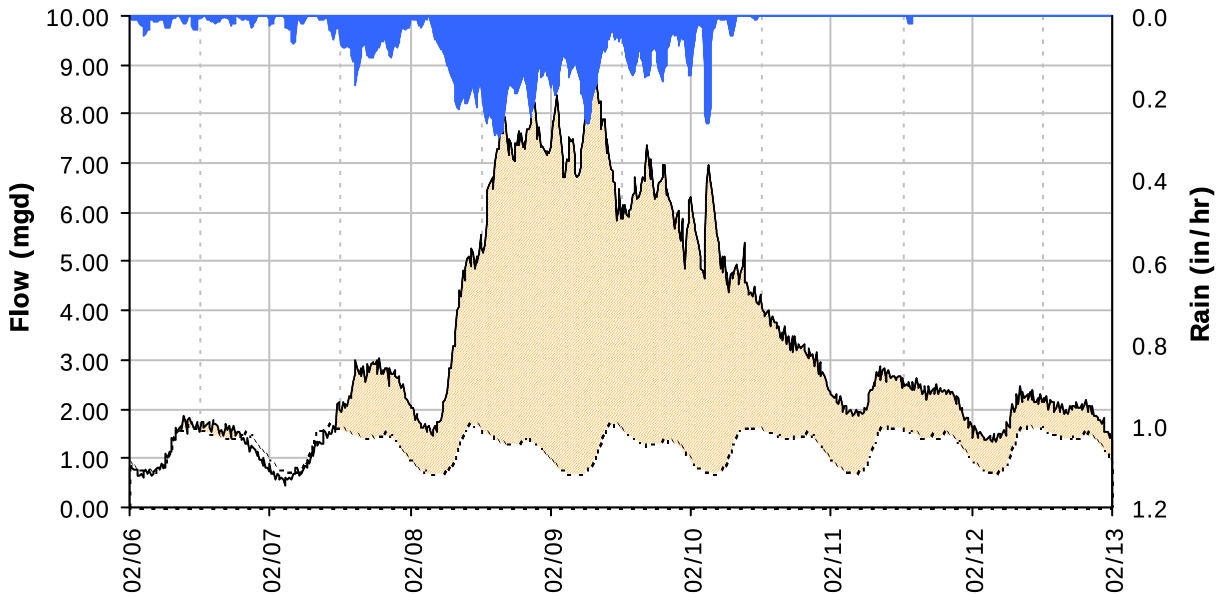
Pipe Diameter:	30	*inches*
Peak Measured Level:	13.3	*inches*
Peak d/D Ratio:	0.44	

SITE 2
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

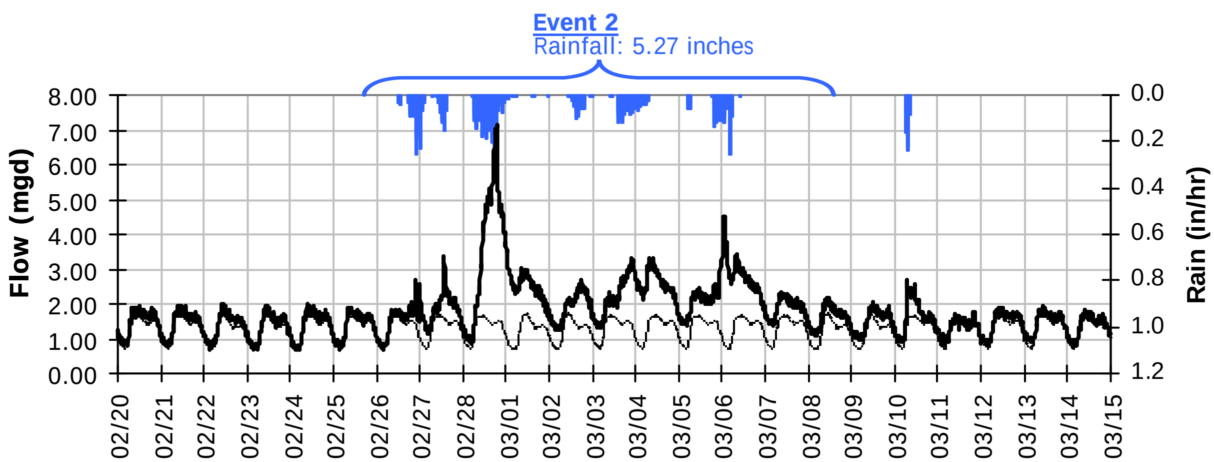


Storm Event I/I Analysis (Rain = 8.40 inches)

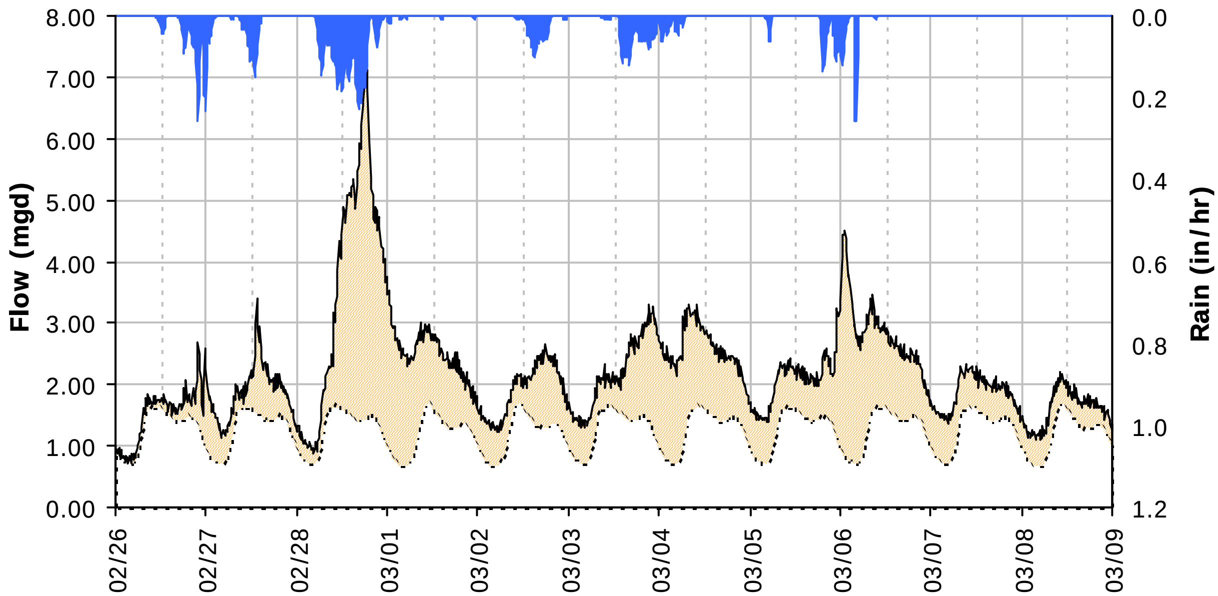
Capacity		**Inflow / Infiltration**	
Peak Flow:	9.14 *mgd*	**Peak I/I Rate:**	8.27 *mgd*
PF:	7.38	**Total I/I:**	14,792,000 *gallons*

SITE 2
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

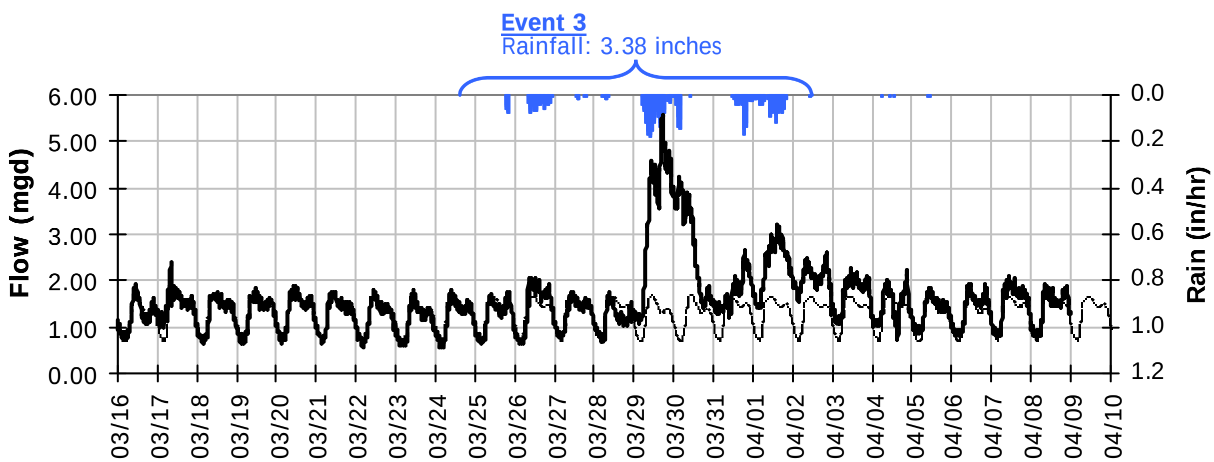


Storm Event I/I Analysis (Rain = 5.27 inches)

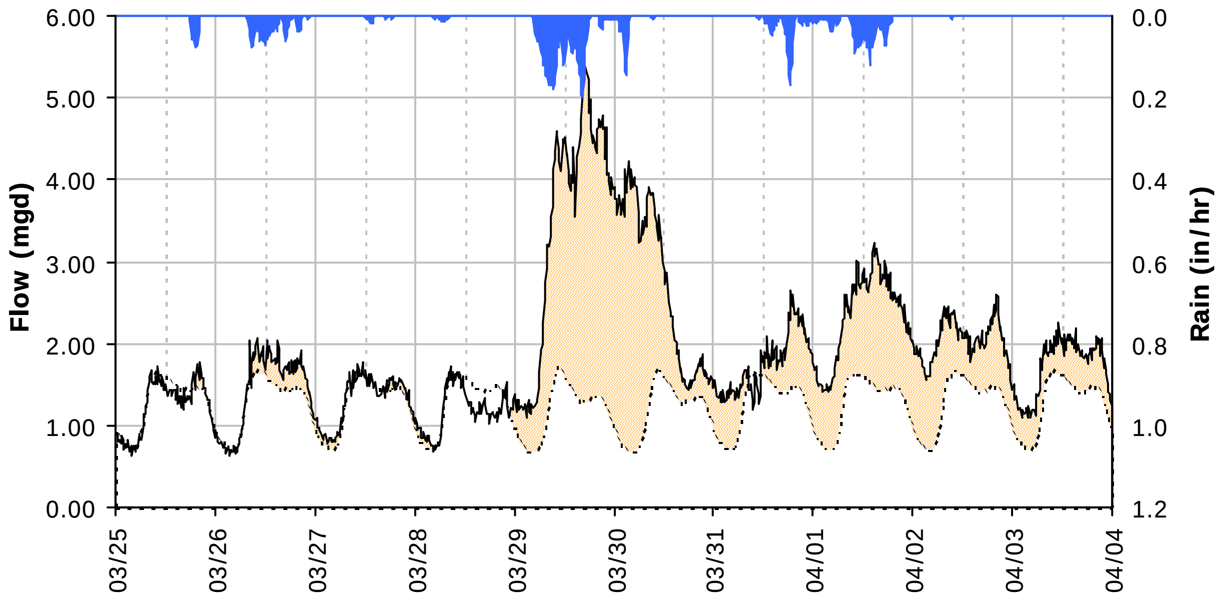
Capacity		**Inflow / Infiltration**	
Peak Flow:	7.12 *mgd*	**Peak I/I Rate:**	5.67 *mgd*
PF:	5.75	**Total I/I:**	10,857,000 *gallons*

SITE 2
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



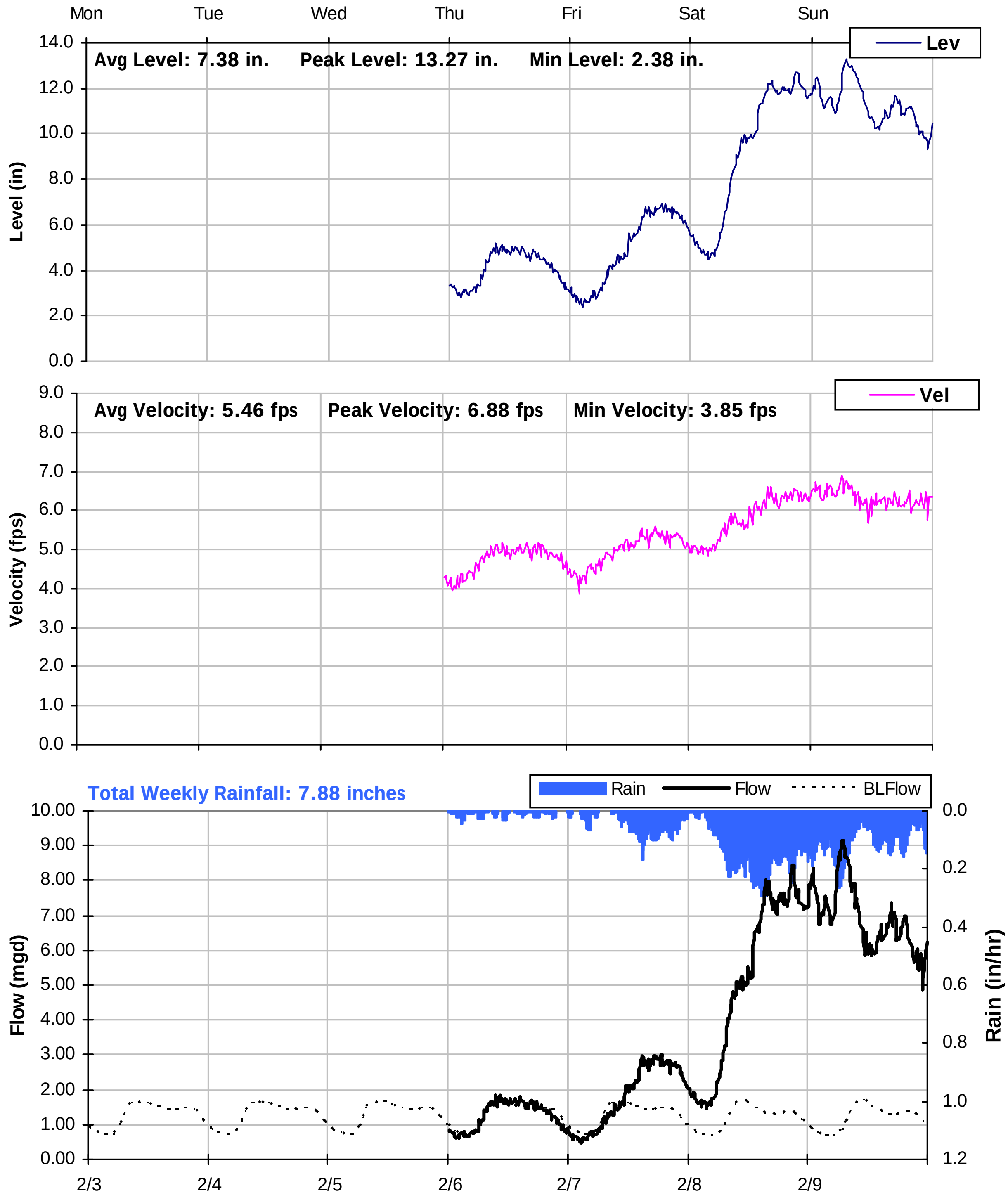
Event 3 Detail Graph



Storm Event I/I Analysis (Rain = 3.38 inches)

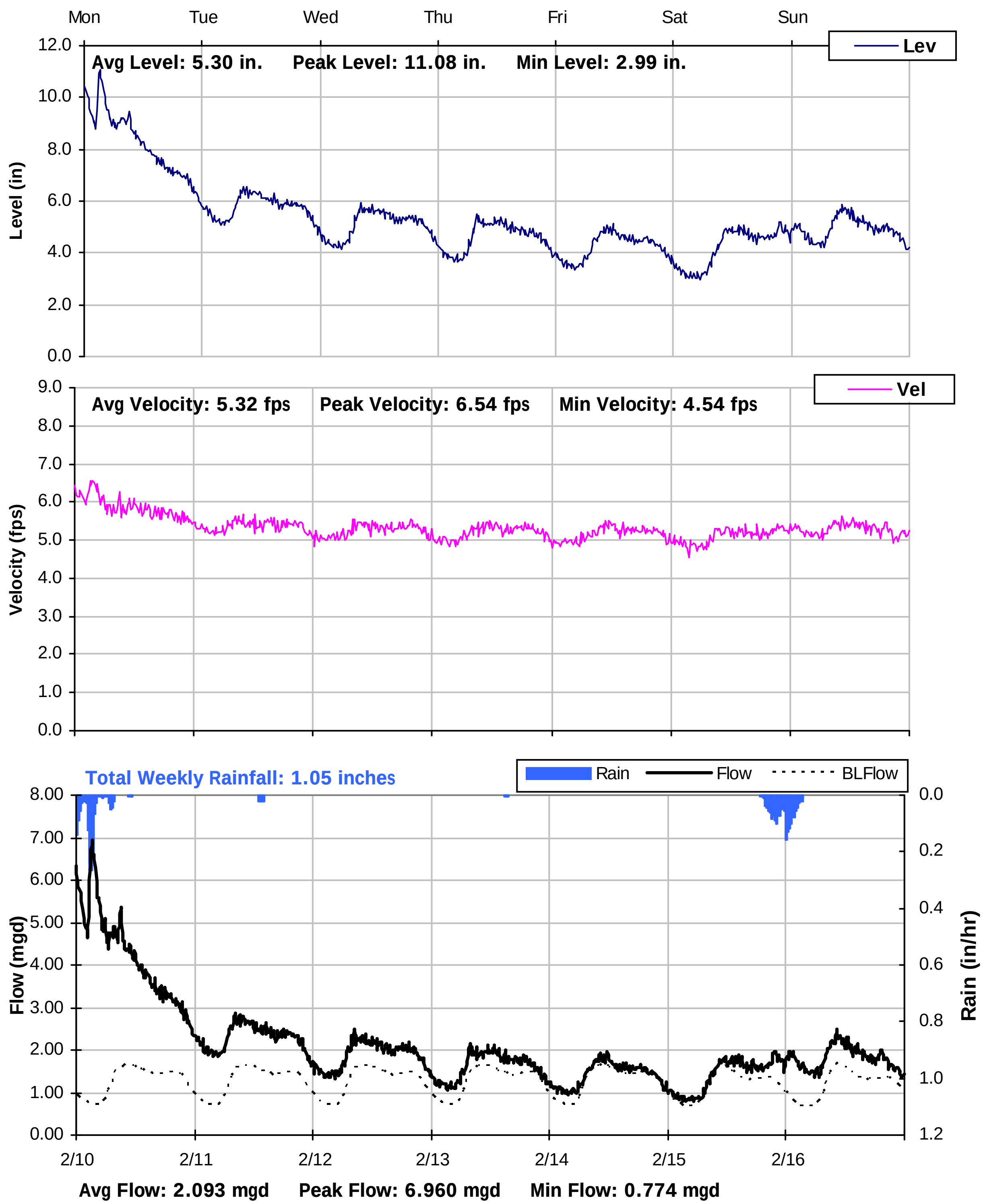
Capacity		Inflow / Infiltration	
Peak Flow:	5.56 *mgd*	**Peak I/I Rate:**	4.25 *mgd*
PF:	4.49	**Total I/I:**	6,733,000 *gallons*

SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014

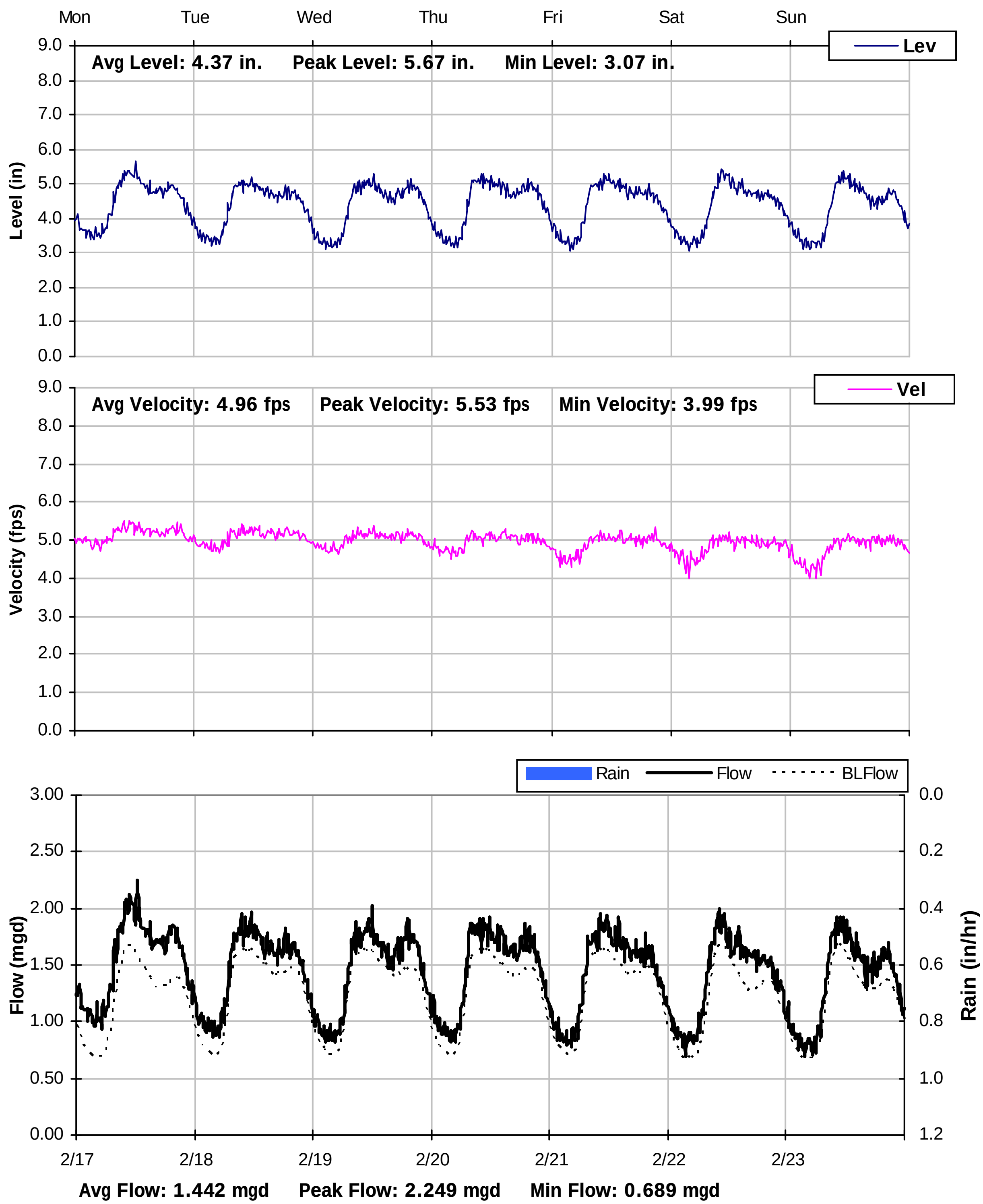


Avg Flow: 3.766 mgd **Peak Flow: 9.135 mgd** **Min Flow: 0.453 mgd**

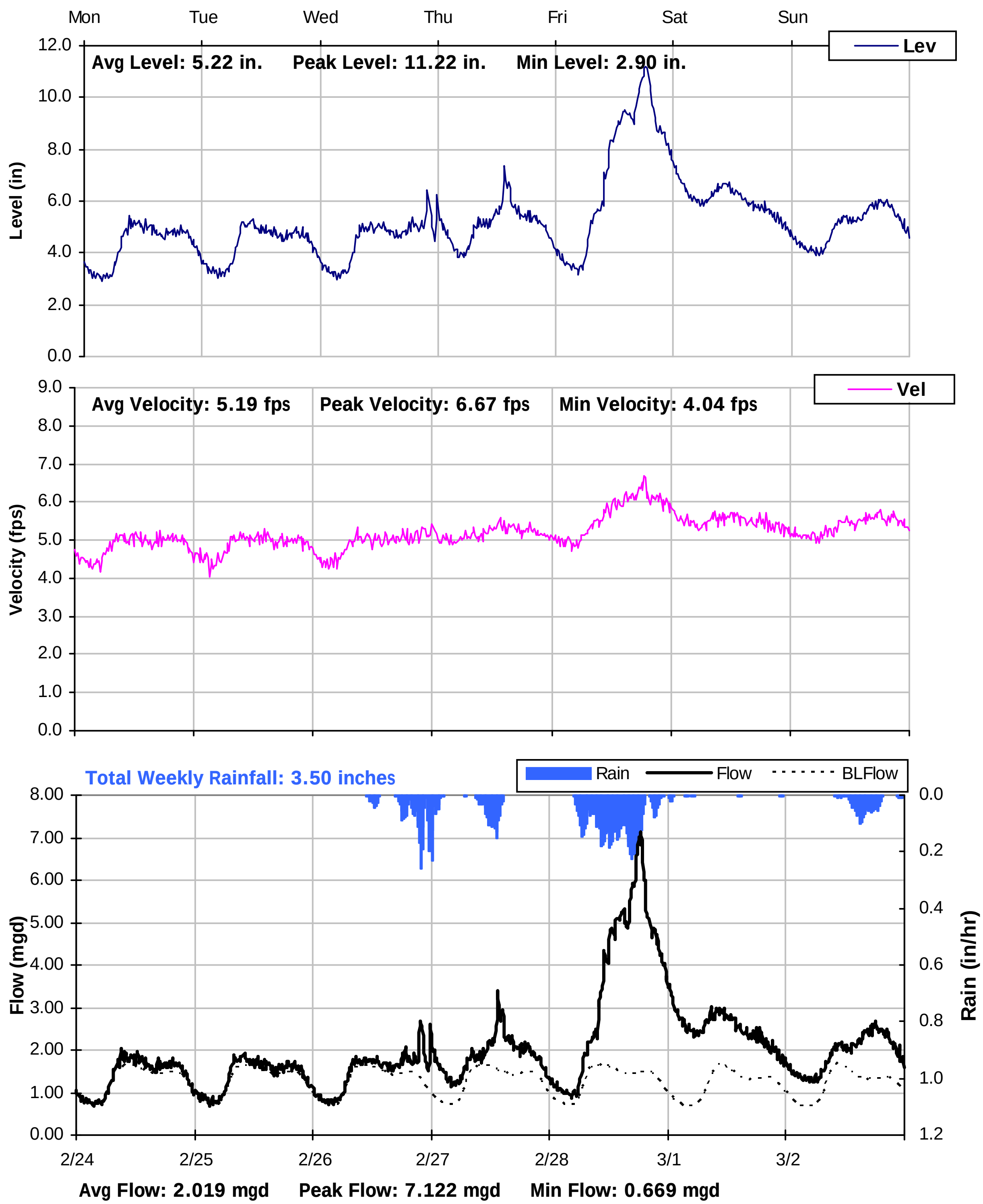
SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014



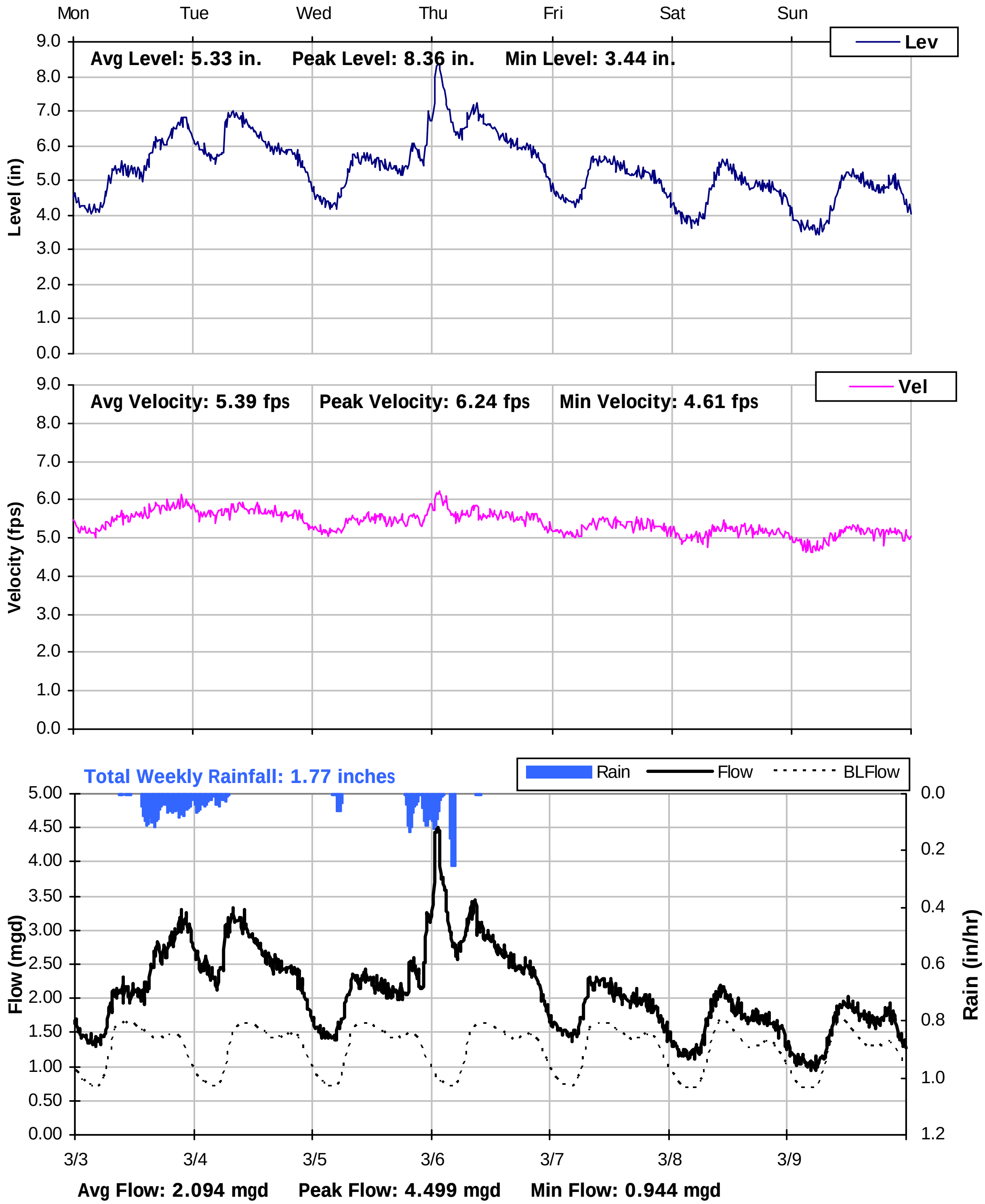
SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014



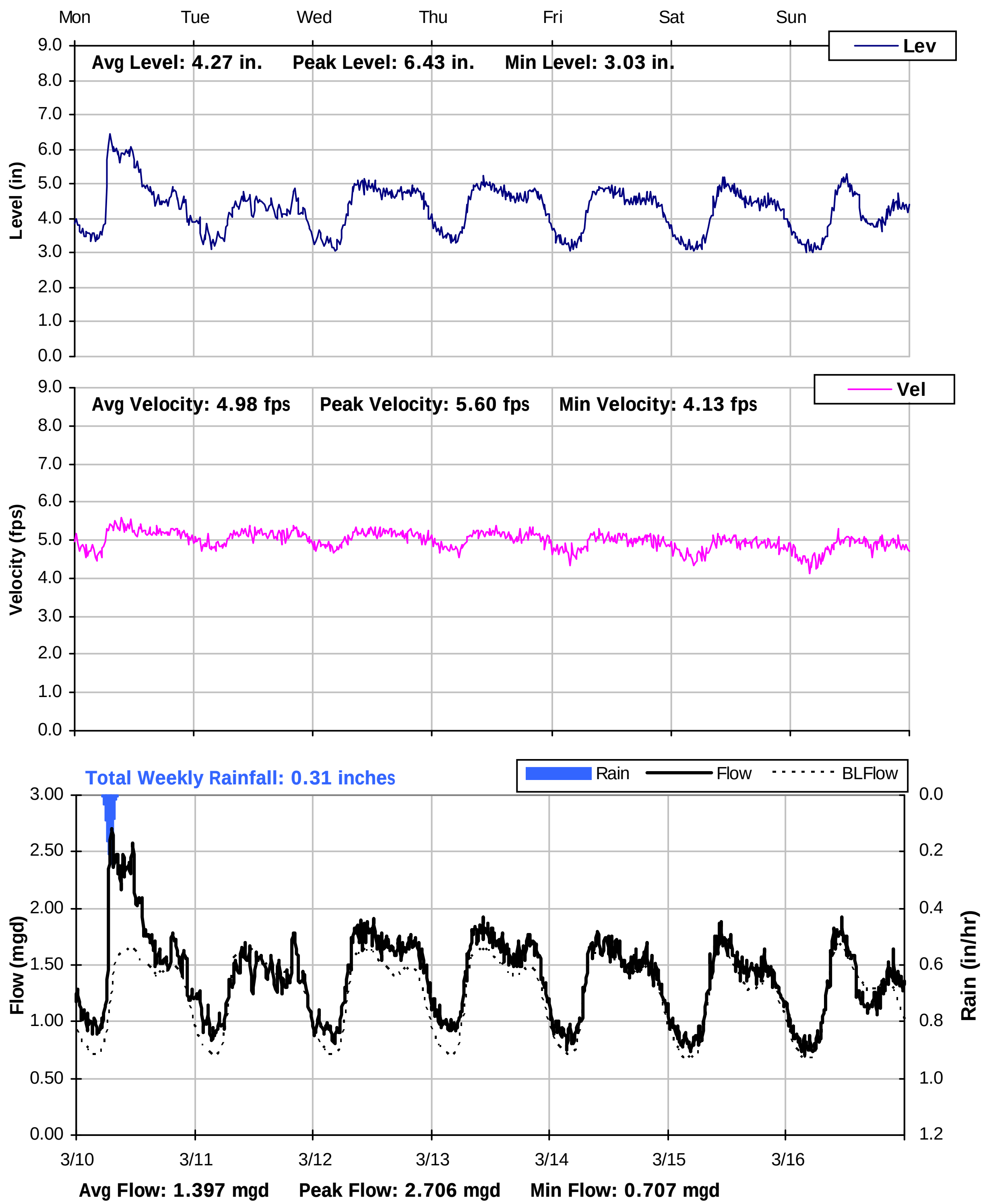
SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



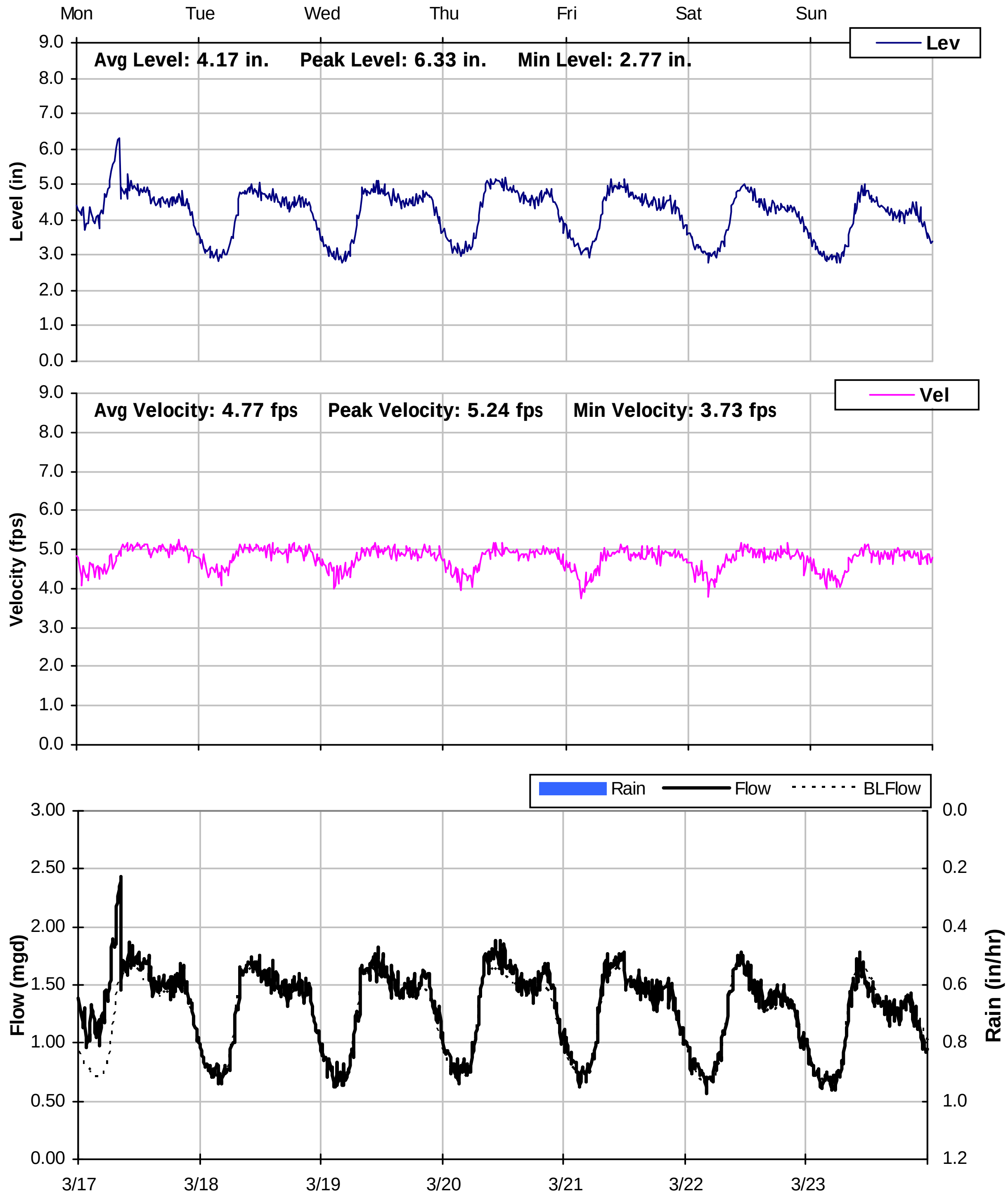
SITE 2
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



SITE 2
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014

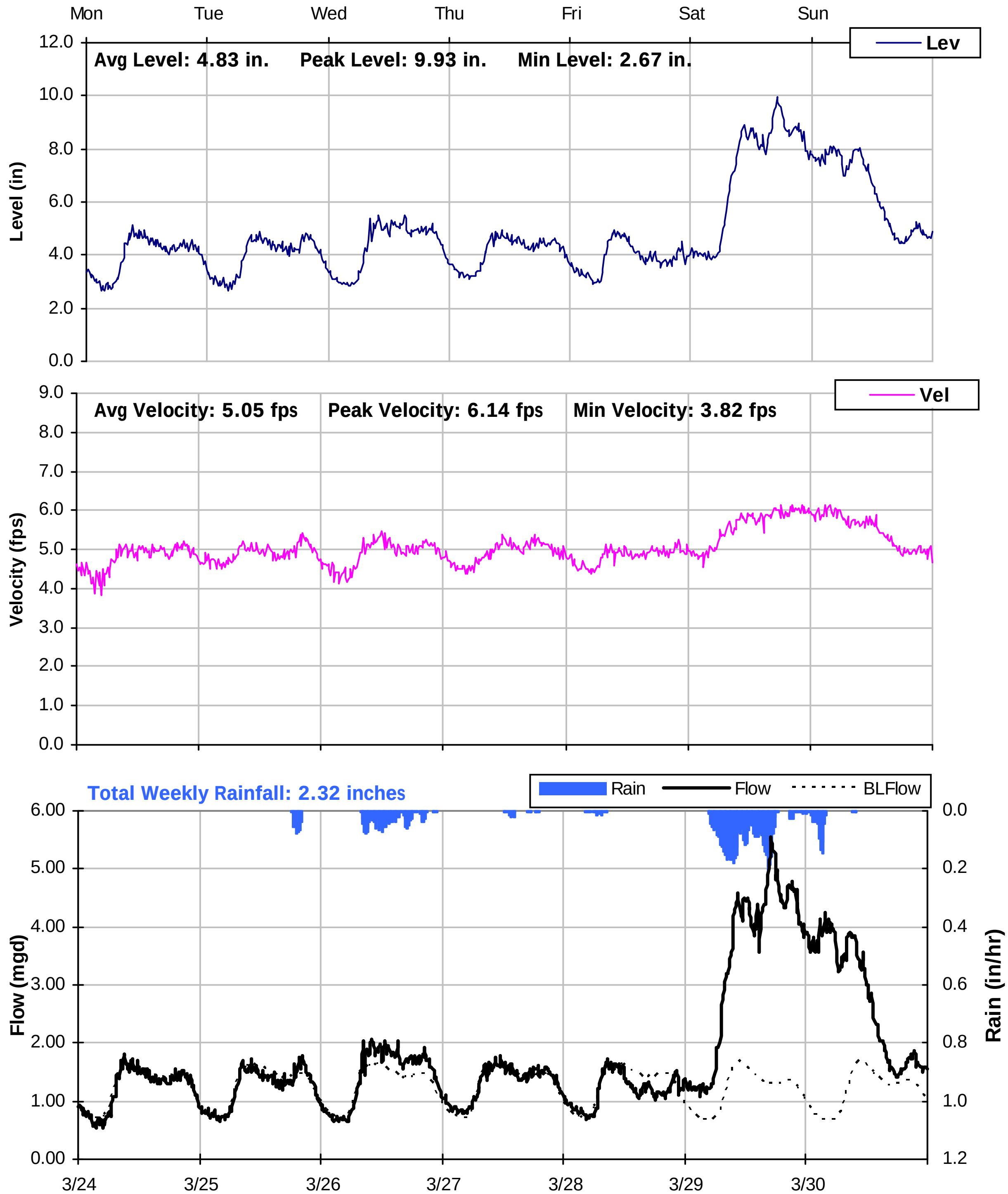


SITE 2
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



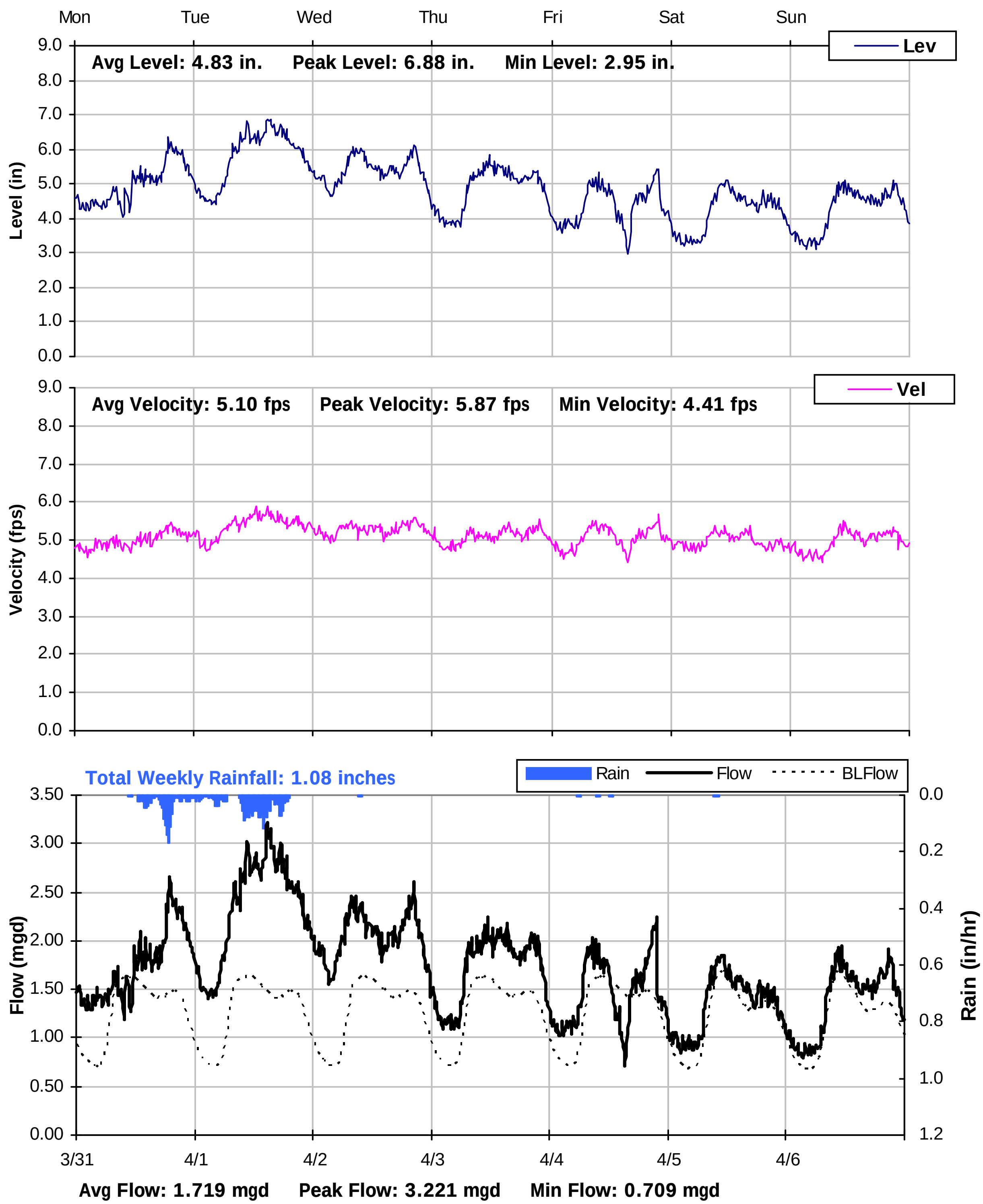
Avg Flow: 1.299 mgd **Peak Flow: 2.415 mgd** **Min Flow: 0.562 mgd**

SITE 2
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014

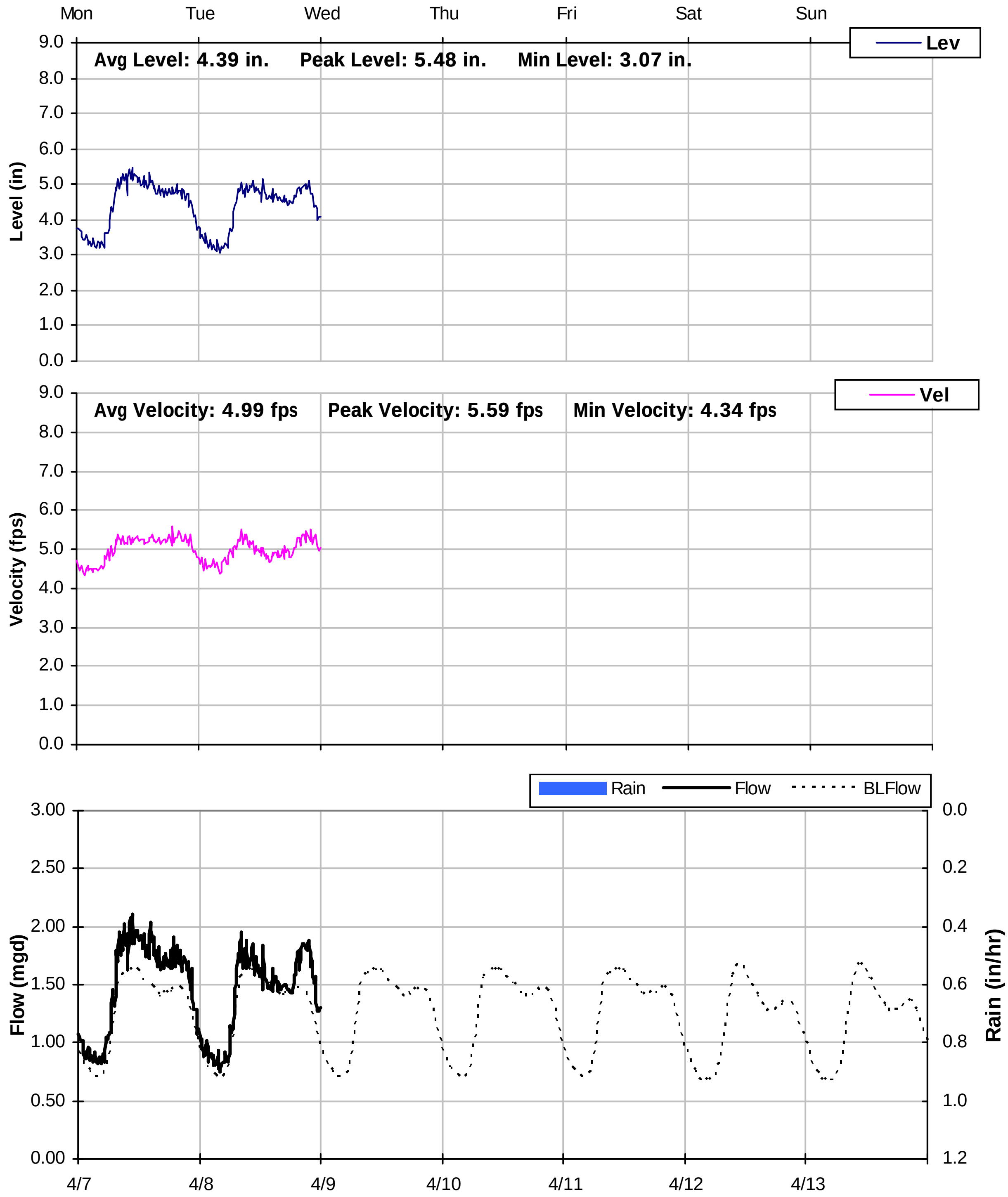


Avg Flow: 1.794 mgd **Peak Flow: 5.559 mgd** **Min Flow: 0.543 mgd**

SITE 2
Weekly Level, Velocity and Flow Hydrographs
3/31/2014 to 4/7/2014



SITE 2
Weekly Level, Velocity and Flow Hydrographs
4/7/2014 to 4/14/2014



Avg Flow: 1.460 mgd **Peak Flow: 2.105 mgd** **Min Flow: 0.749 mgd**

City of Grass Valley

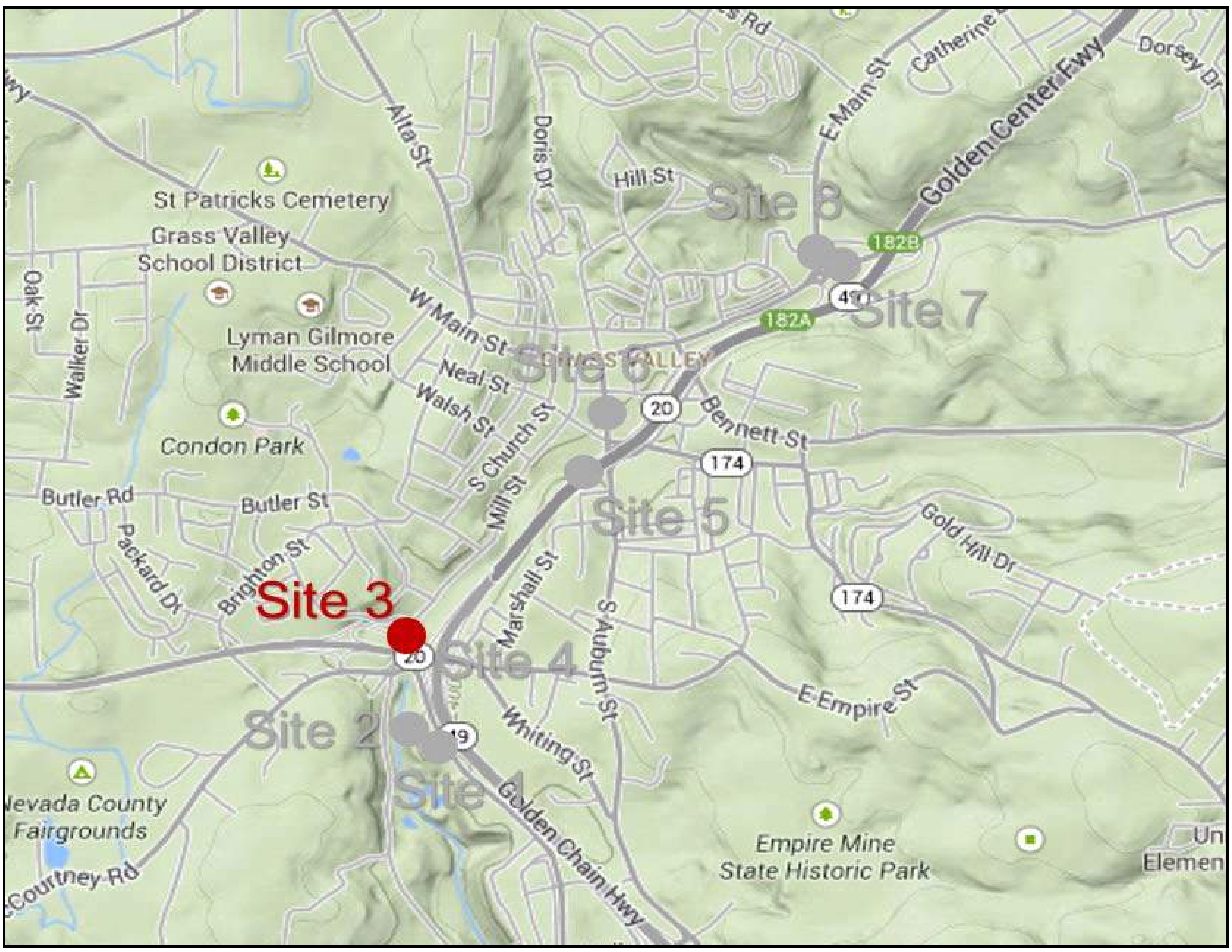
Sanitary Sewer Flow Monitoring

Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 3

Location: Southbound Golden Chain Highway Rood Expressway off-ramp

Data Summary Report



Vicinity Map: Site 3

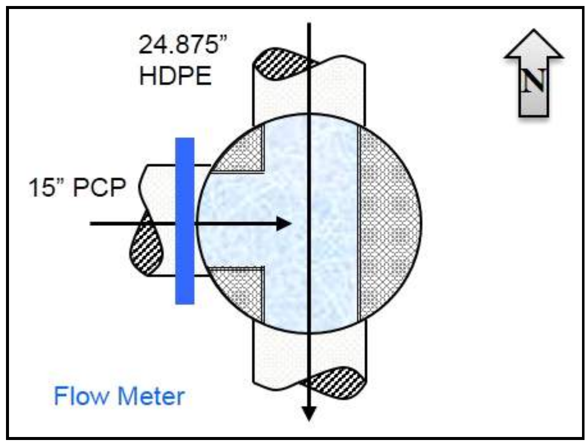
SITE 3

Site Information

Location:	Southbound Golden Chain Highway Rood Expressway off-ramp
Coordinates:	121.0686° W, 39.2103° N
Rim Elevation:	2370 feet
Pipe Diameter:	15 inches
Baseline Flow:	0.187 mgd
Peak Measured Flow:	2.312 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

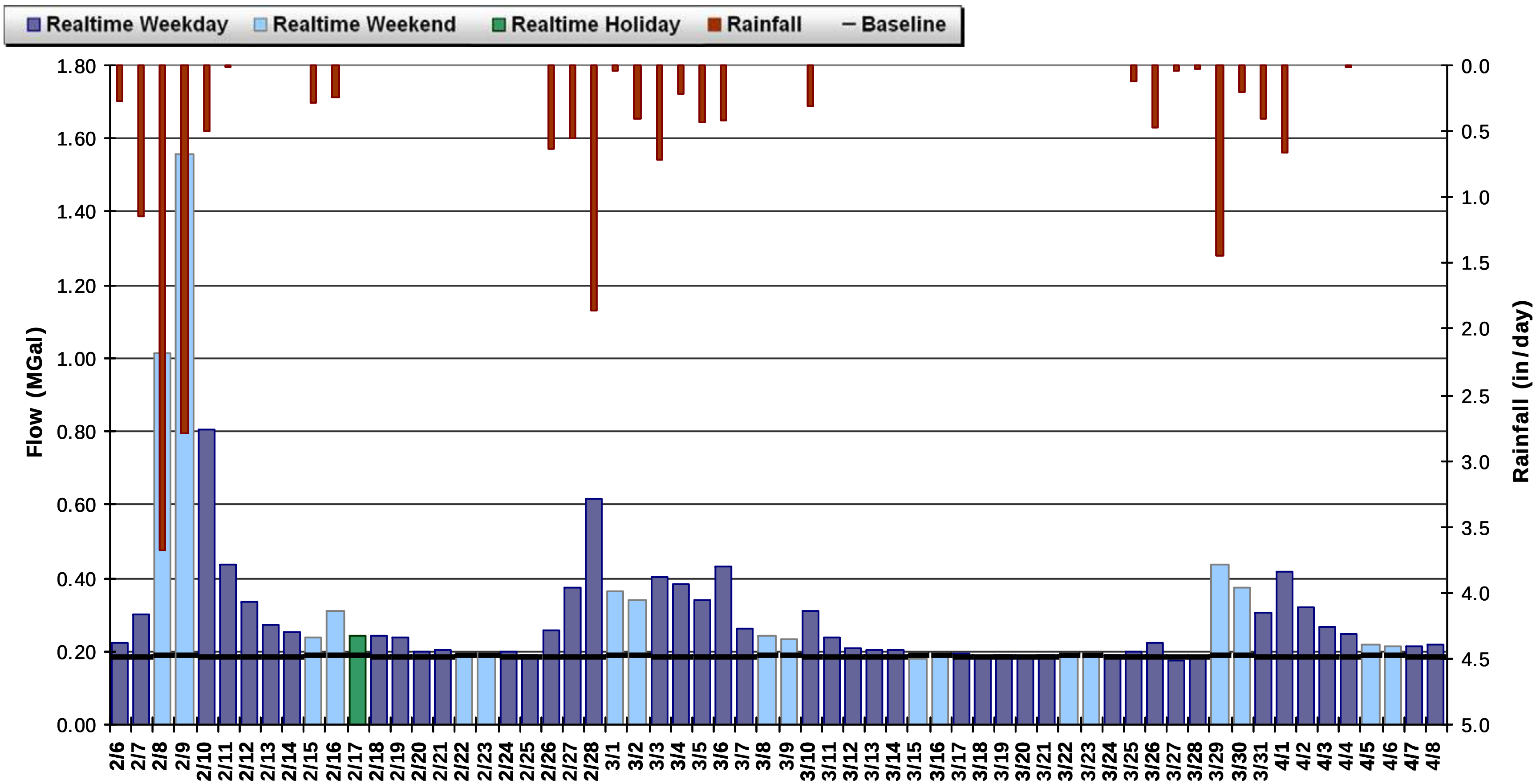
Plan View

SITE 3

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.305 MGal **Peak Daily Flow: 1.559 MGal** **Min Daily Flow: 0.175 MGal**

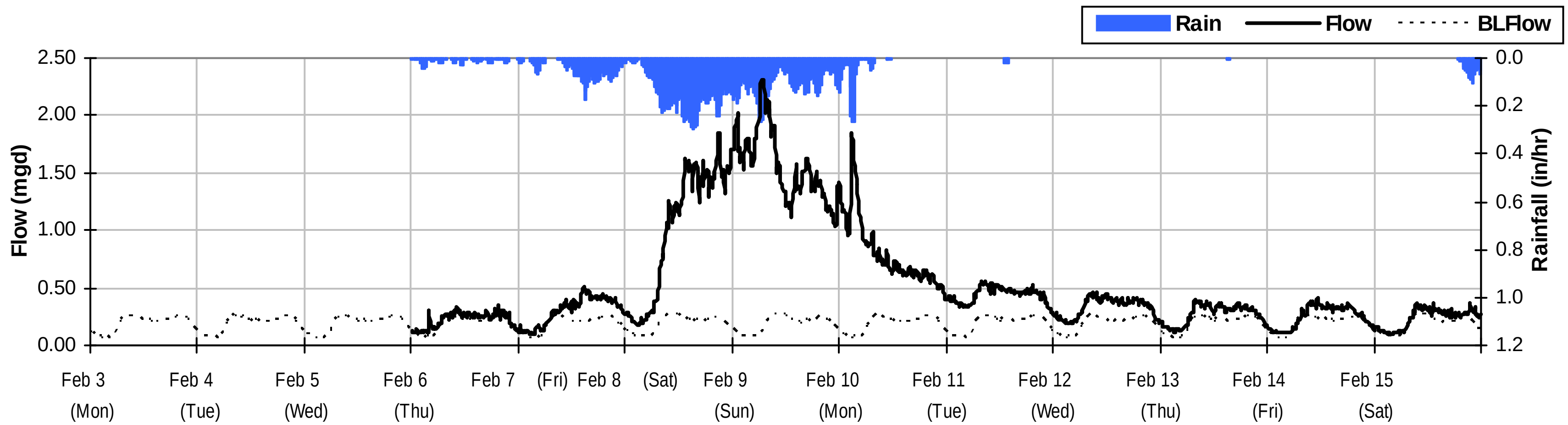
Total Period Rainfall: 17.91 inches

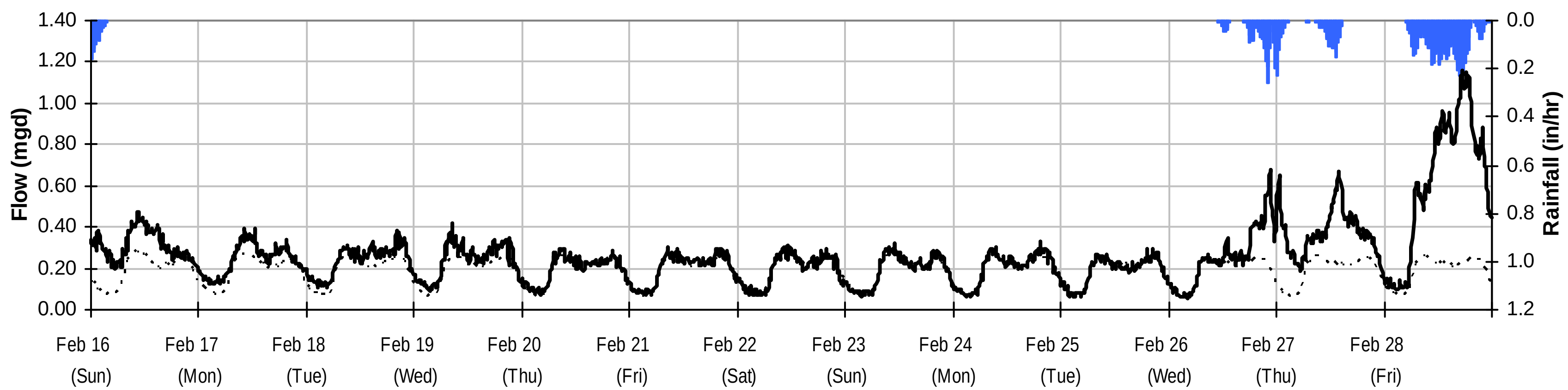


SITE 3

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.386 mgd** **Peak Flow: 2.312 mgd** **Min Flow: 0.058 mgd**

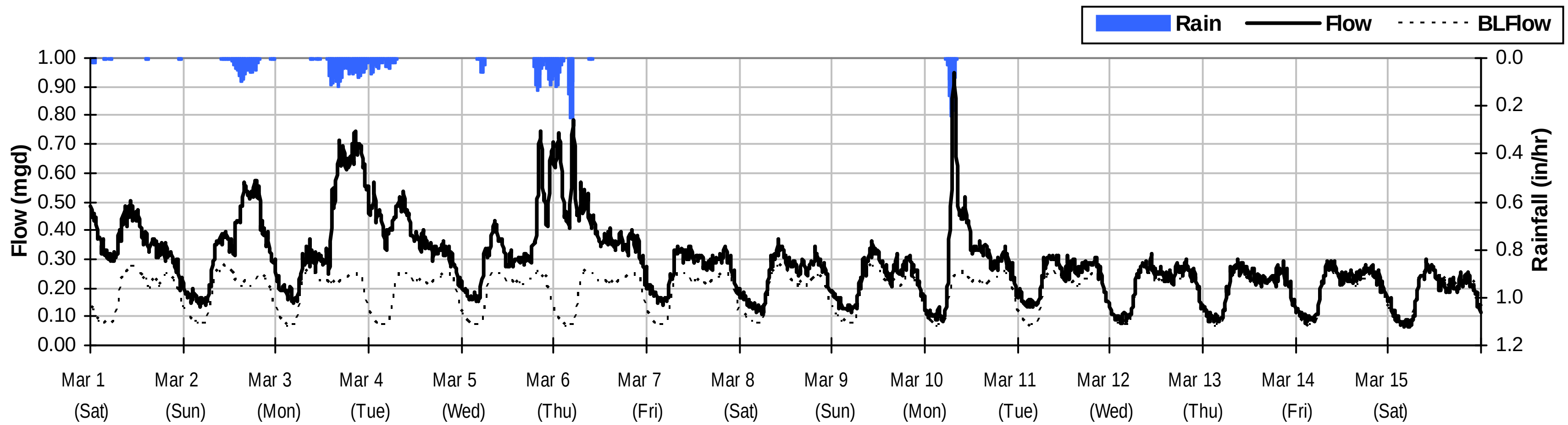


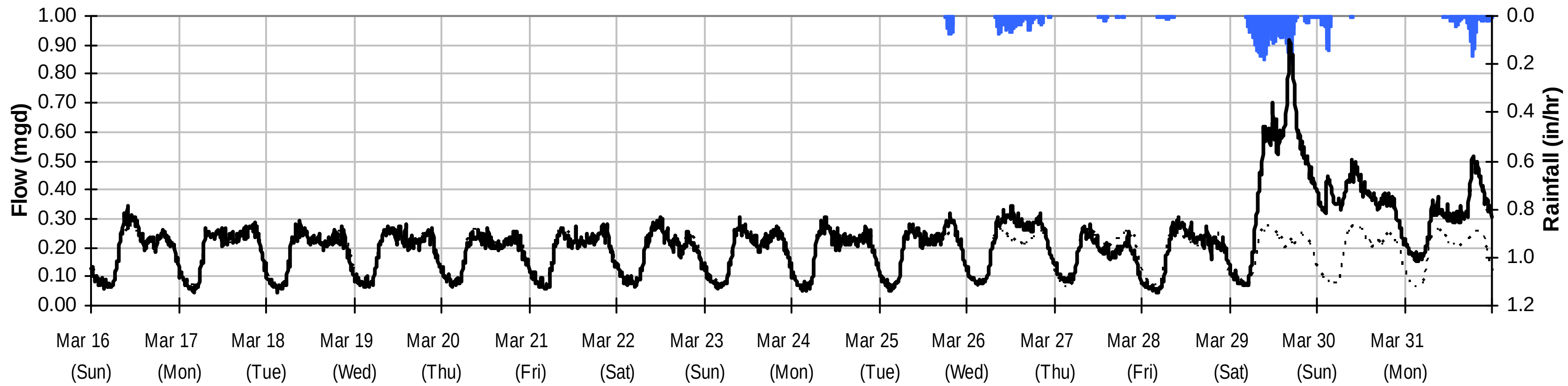


SITE 3

Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 5.24 inches **Avg Flow: 0.256 mgd** **Peak Flow: 0.948 mgd** **Min Flow: 0.044 mgd**

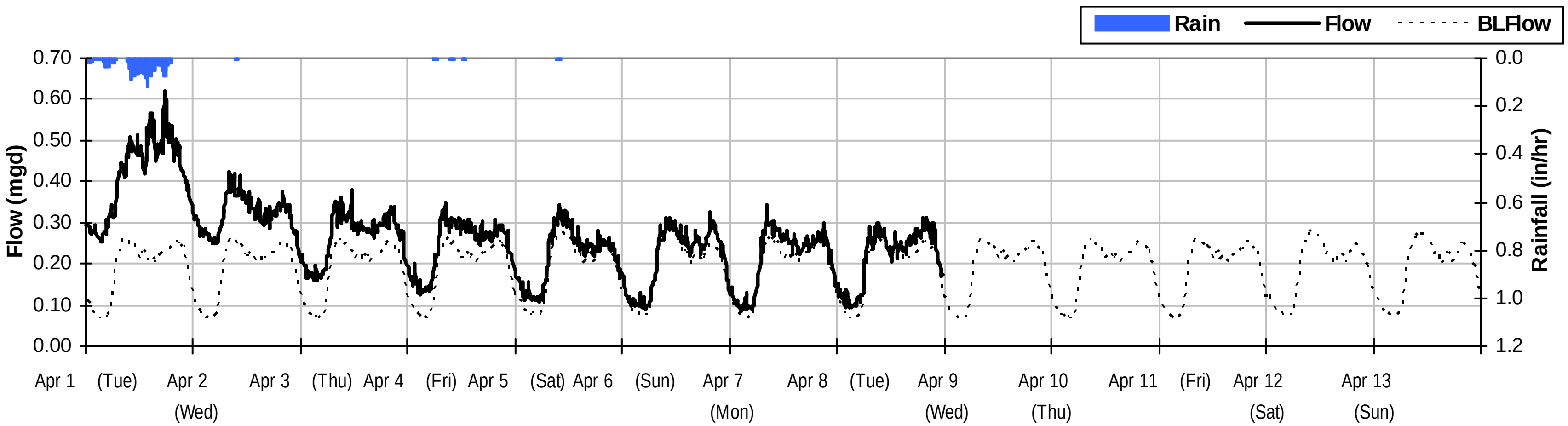




SITE 3

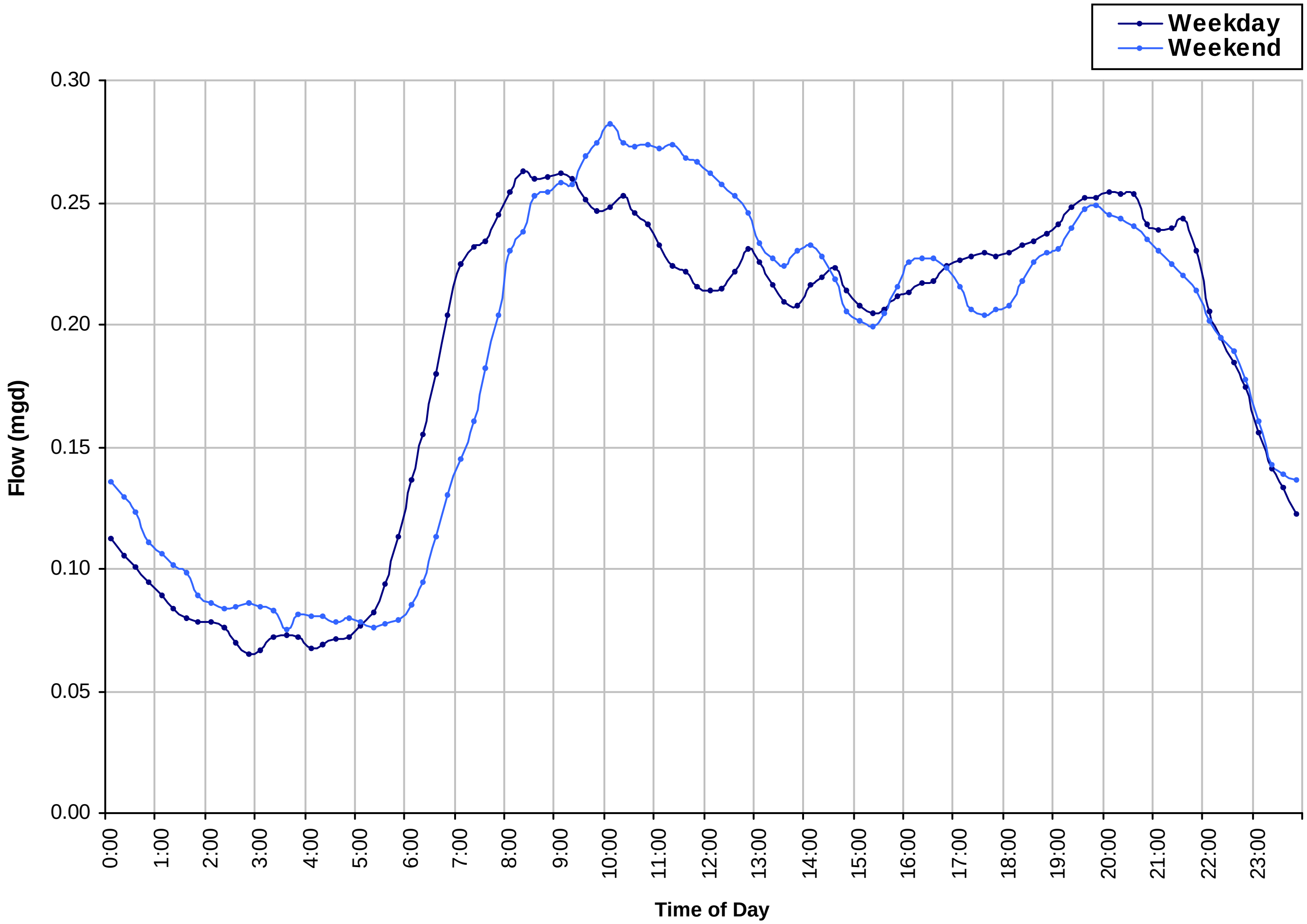
Monthly Flow Summary: April, 2014

Total Monthly Rainfall: 0.69 inches **Avg Flow: 0.264 mgd** **Peak Flow: 0.620 mgd** **Min Flow: 0.086 mgd**

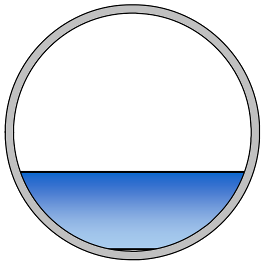


SITE 3

Baseline Flow Hydrographs

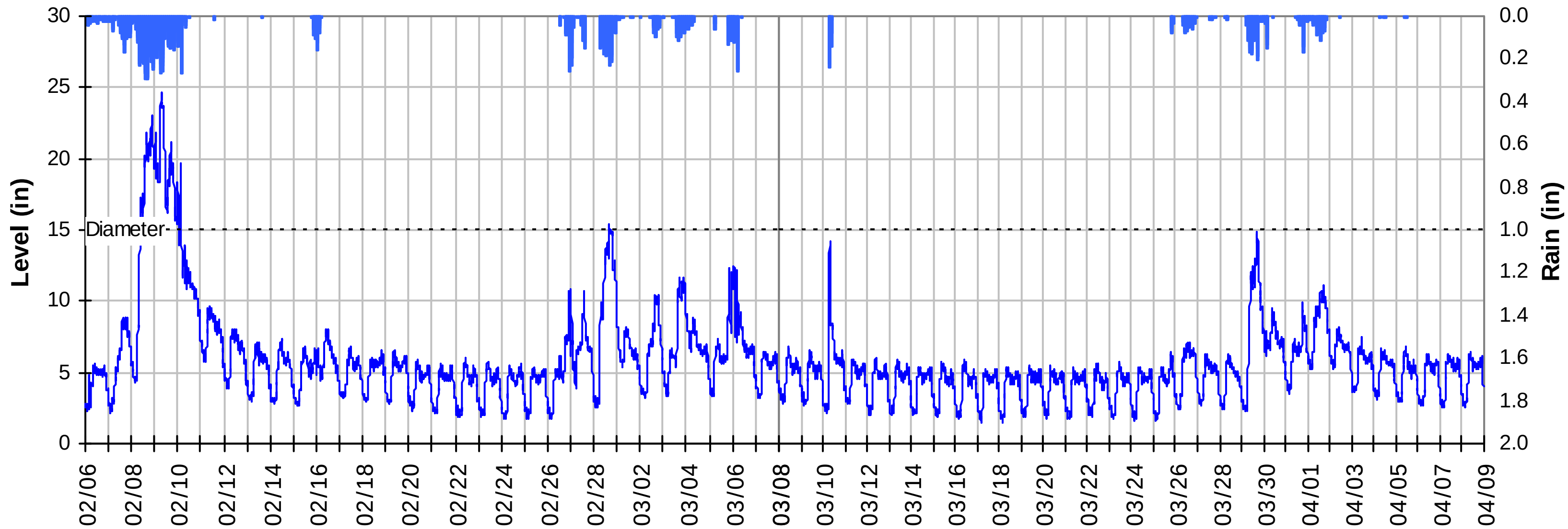


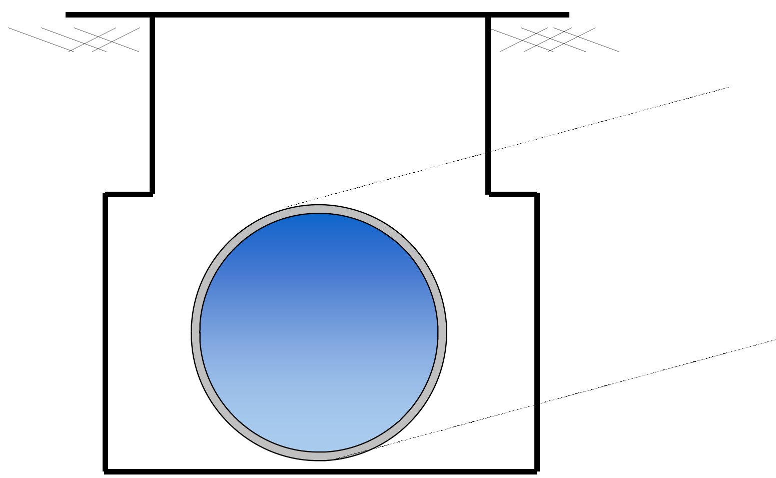
Baseline Flow:
0.187 *mgd*



SITE 3
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 15 *inches*

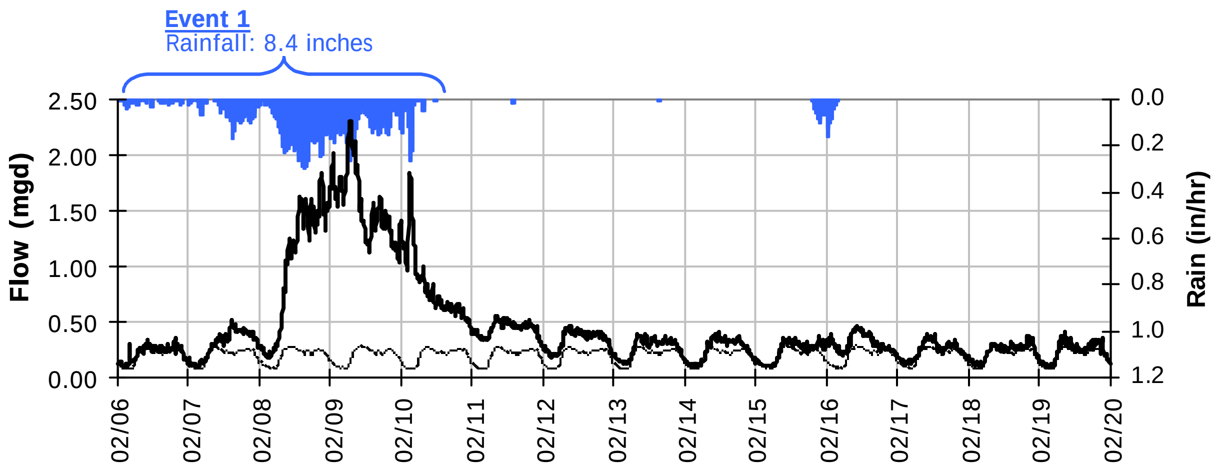
Peak Measured Level: 24.7 *inches*

Peak d / D Ratio: 1.65

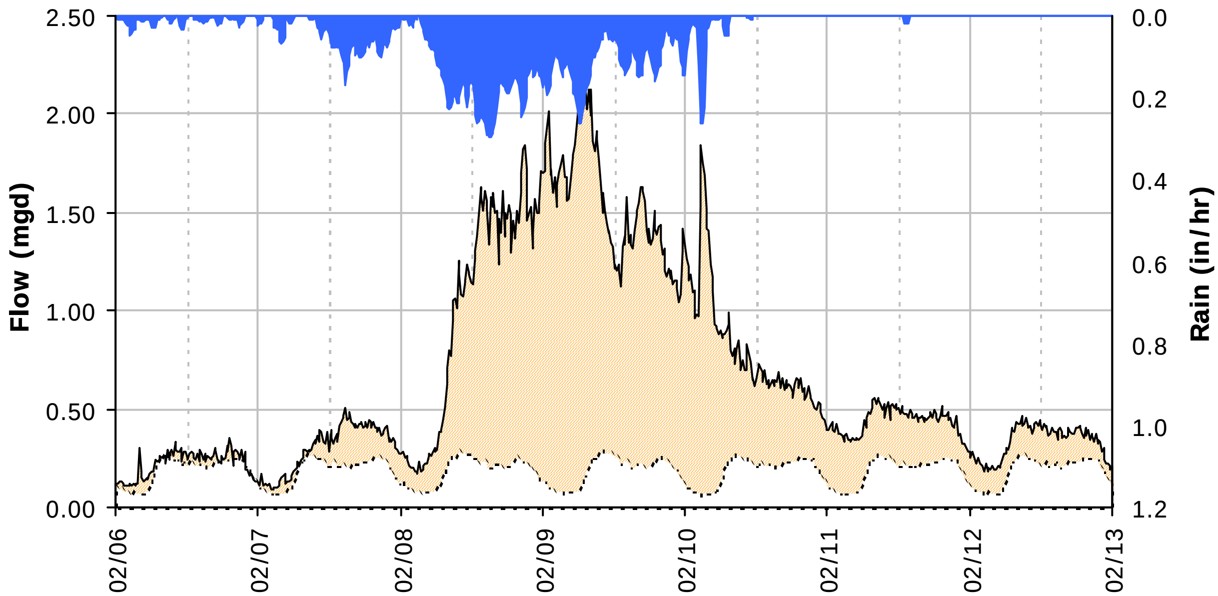
Surcharged 9.7 inches over crown

SITE 3
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

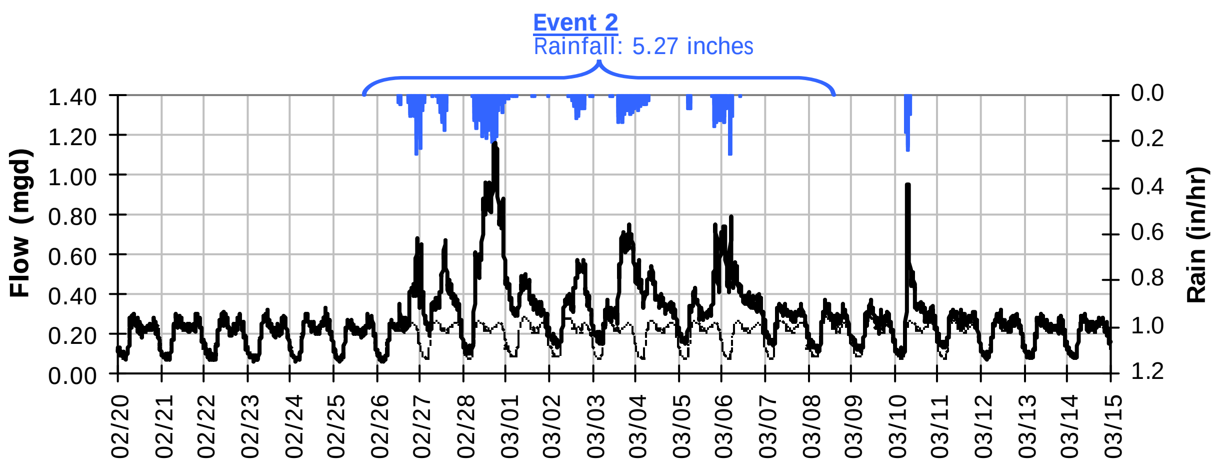


Storm Event I/I Analysis (Rain = 8.40 inches)

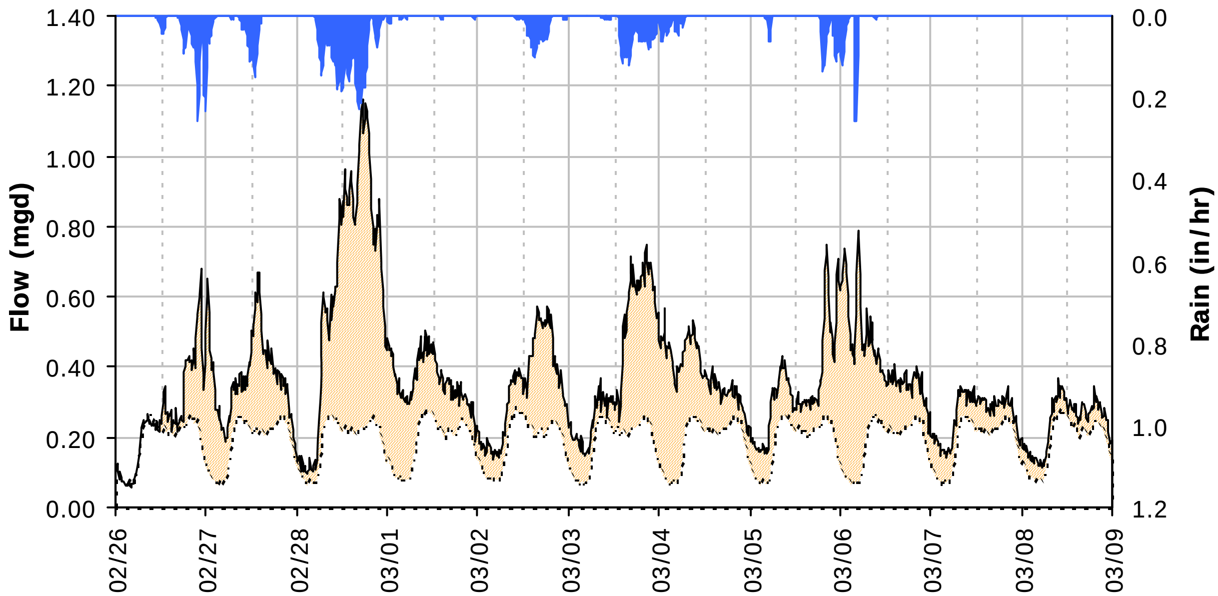
Capacity		Inflow / Infiltration	
Peak Flow:	2.31 *mgd*	**Peak I/I Rate:**	2.19 *mgd*
PF:	12.38	**Total I/I:**	3,372,000 *gallons*

SITE 3
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

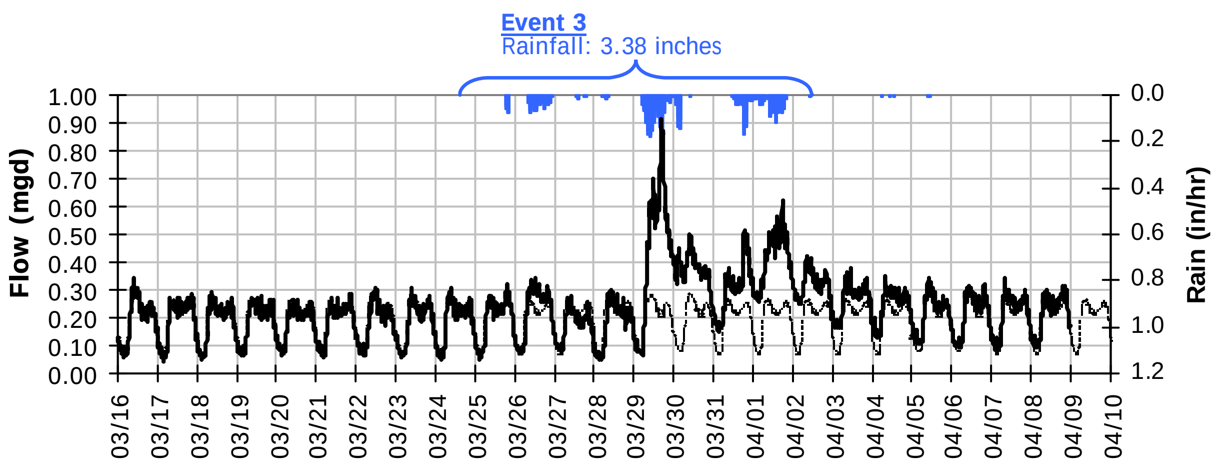


Storm Event I/I Analysis (Rain = 5.27 inches)

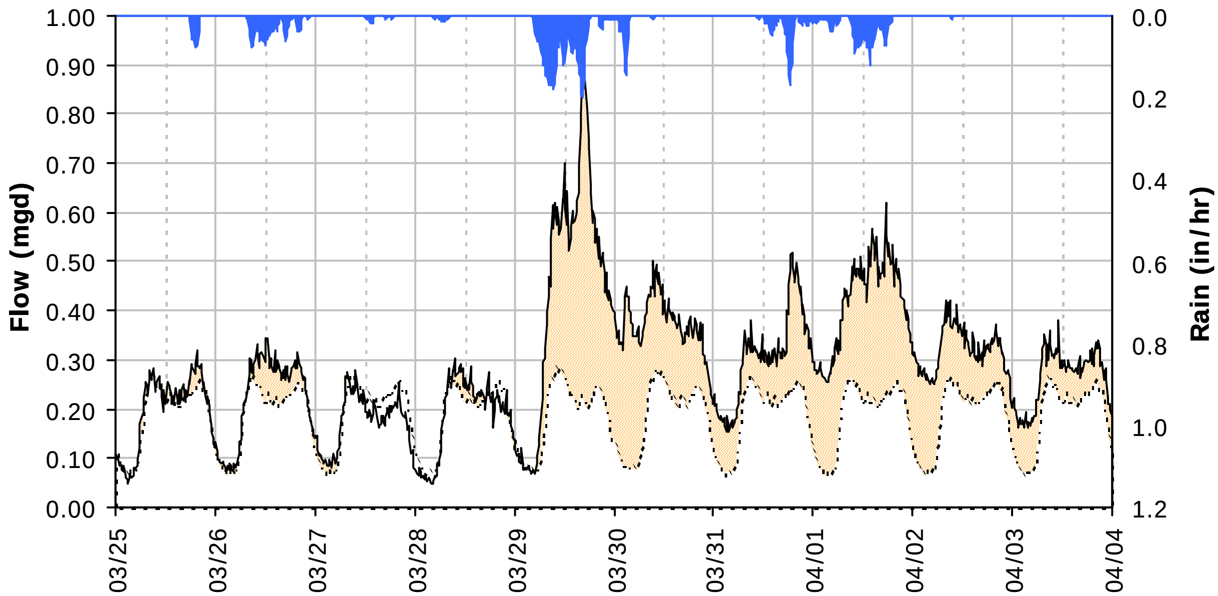
Capacity		**Inflow / Infiltration**	
Peak Flow:	1.16 *mgd*	**Peak I/I Rate:**	0.93 *mgd*
PF:	6.23	**Total I/I:**	1,958,000 *gallons*

SITE 3
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



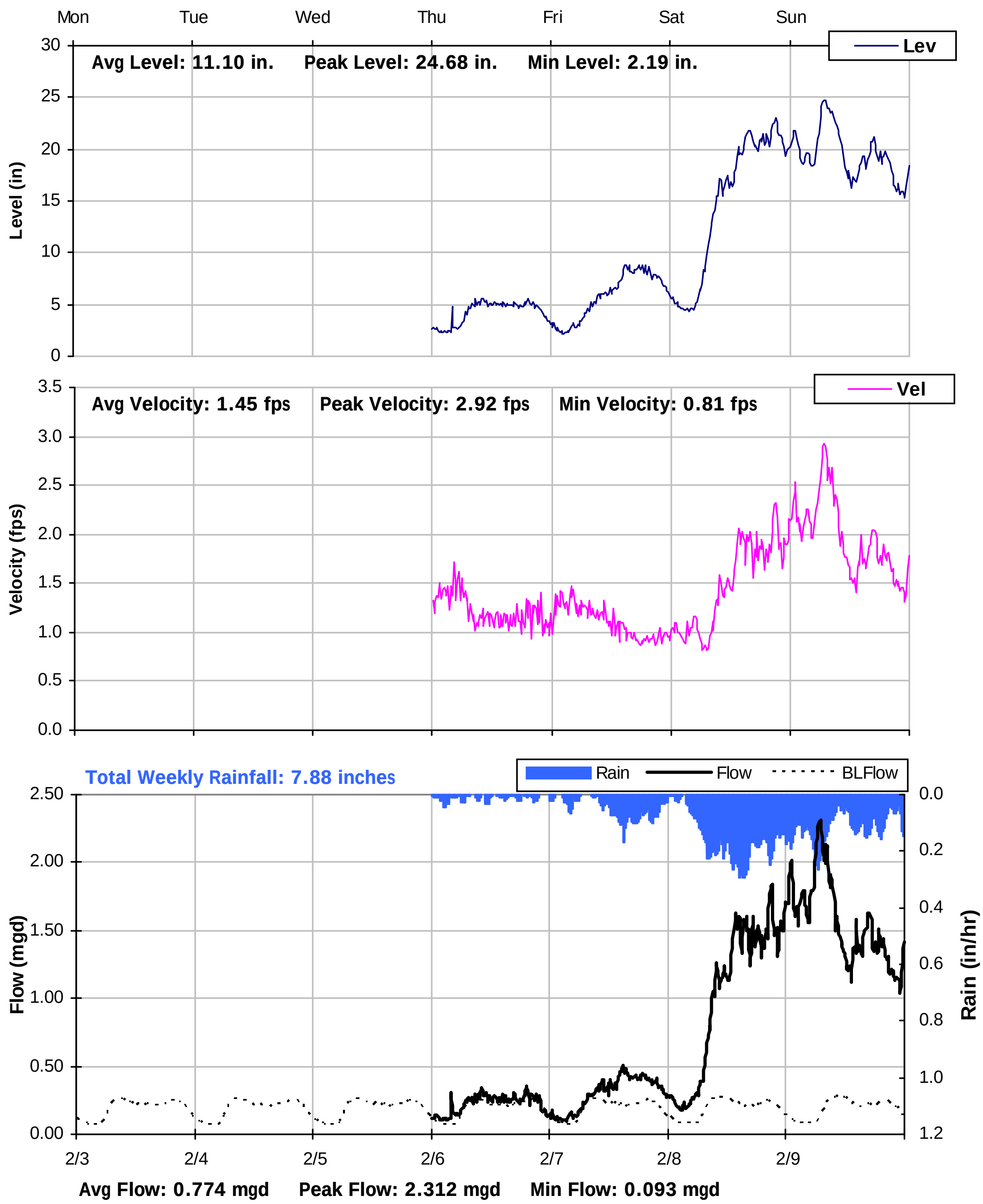
Event 3 Detail Graph



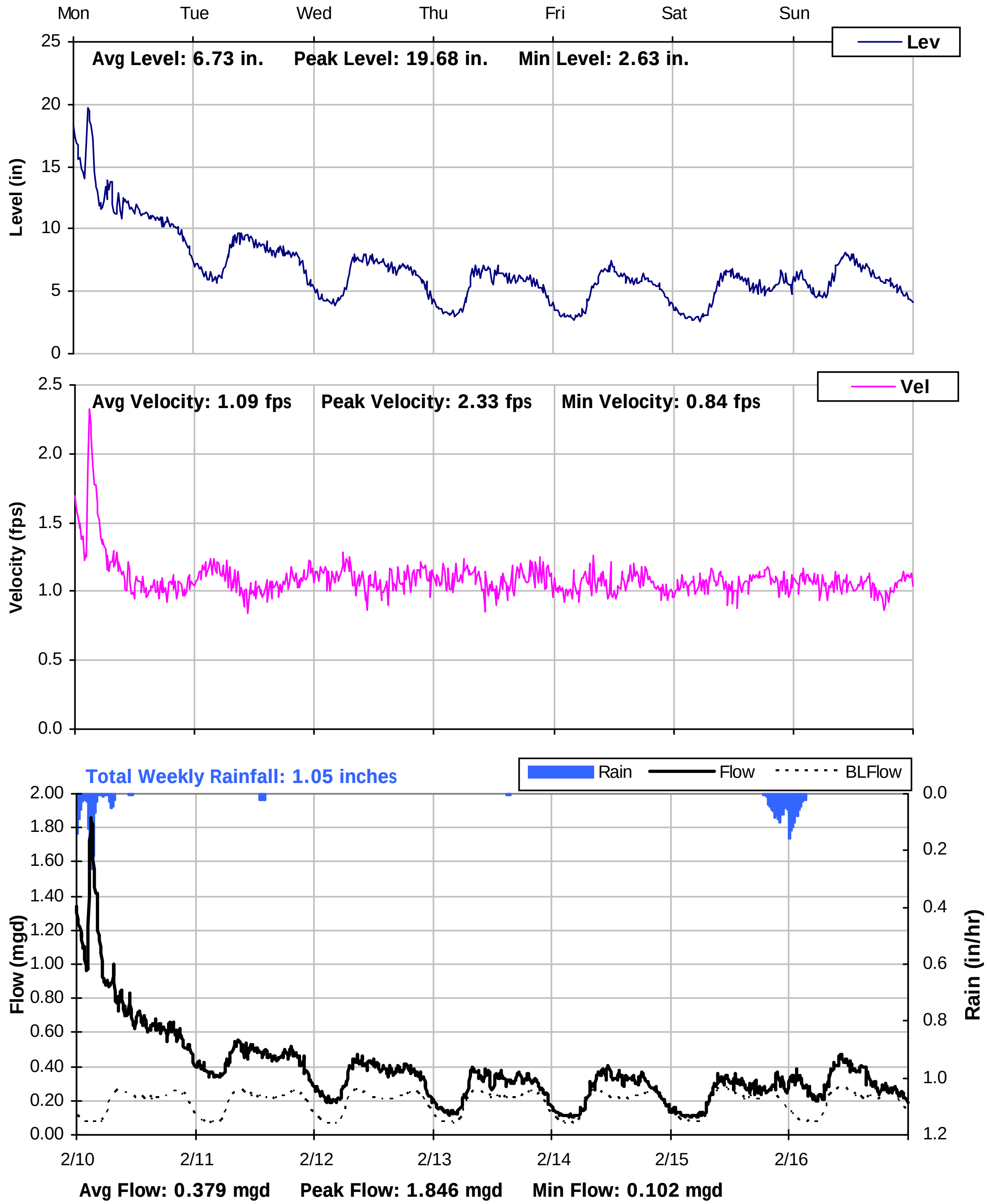
Storm Event I/I Analysis (Rain = 3.38 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.92 *mgd*	**Peak I/I Rate:**	0.69 *mgd*
PF:	4.90	**Total I/I:**	1,031,000 *gallons*

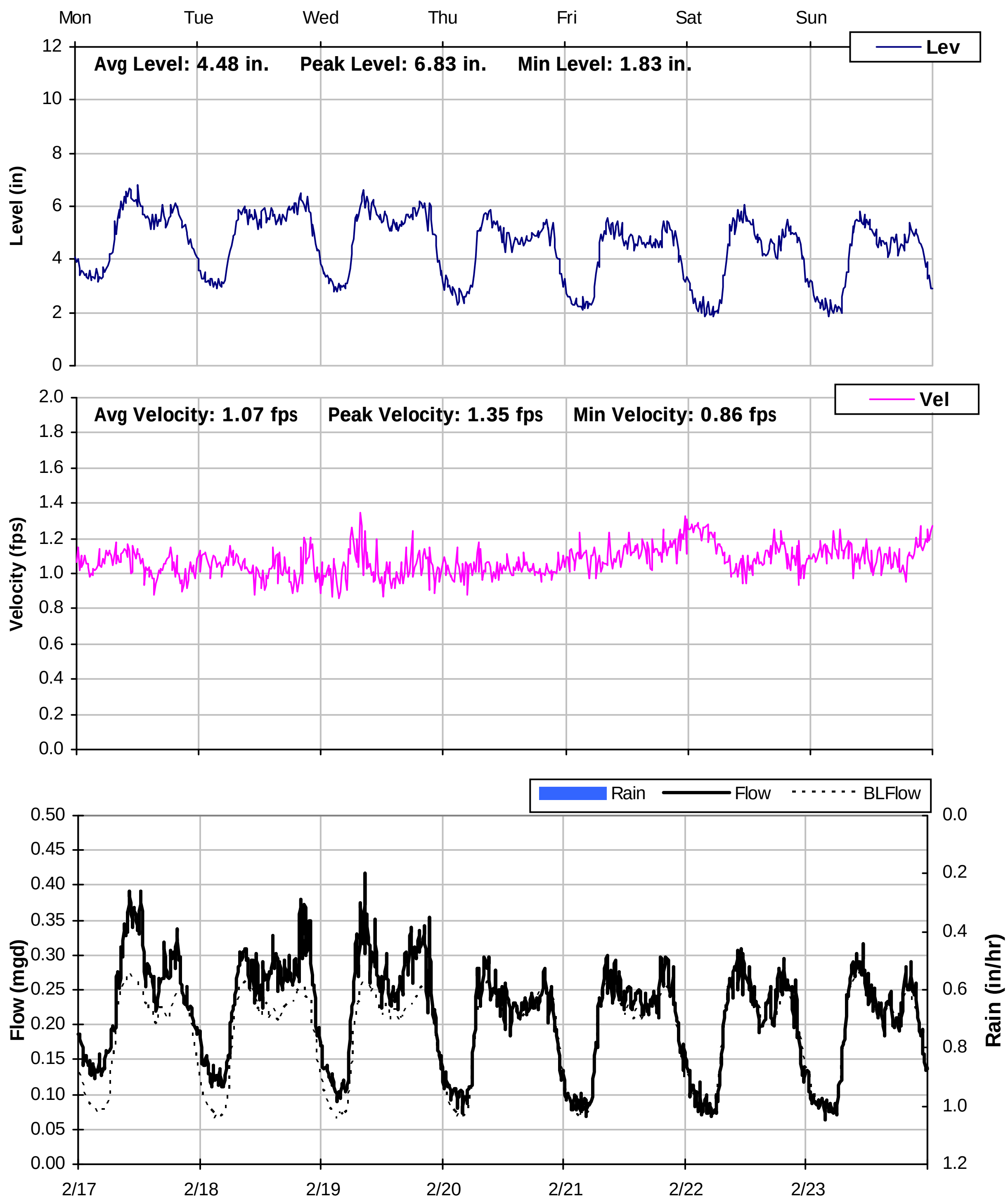
SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014



SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014

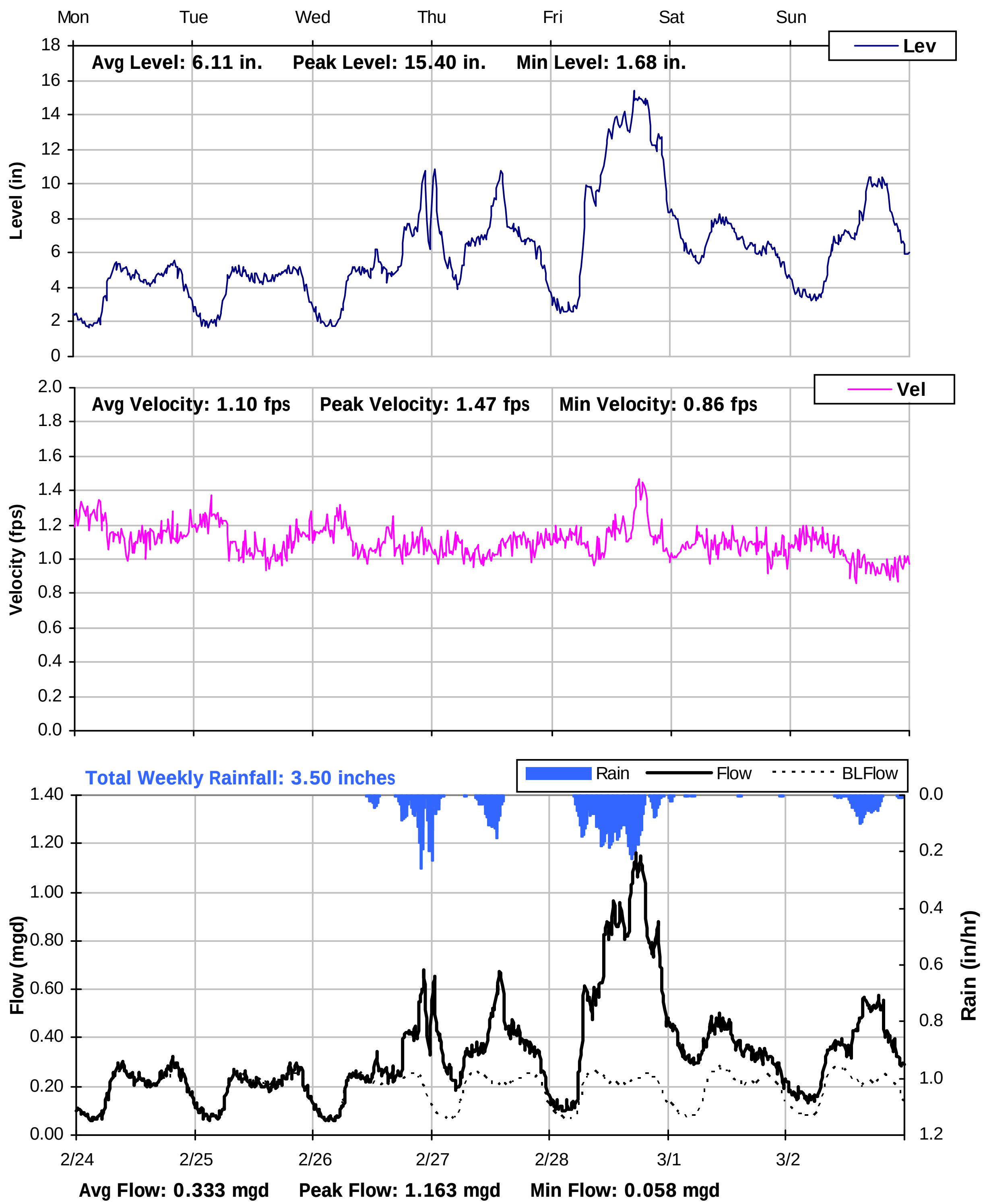


SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014

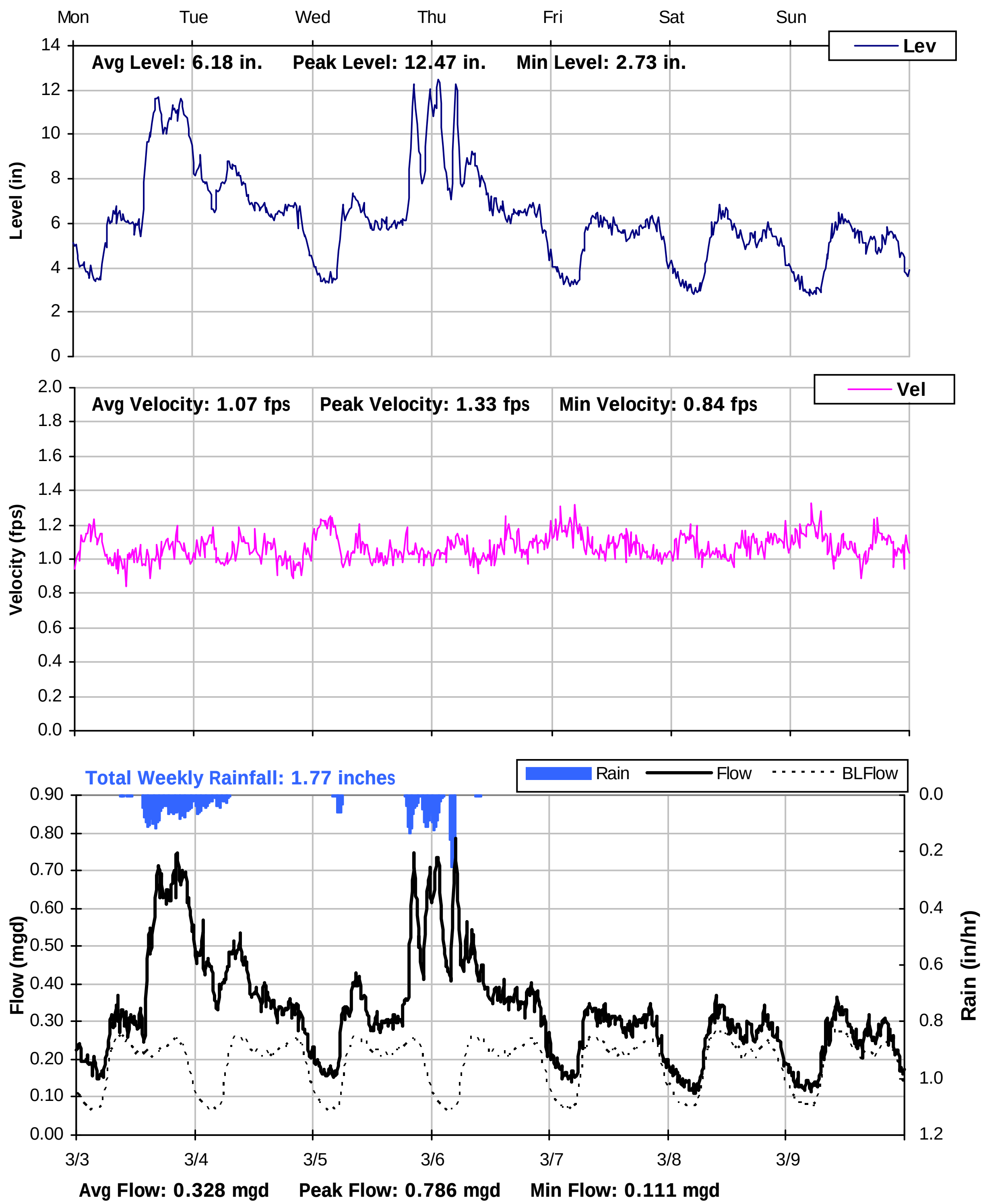


Avg Flow: 0.215 mgd **Peak Flow: 0.416 mgd** **Min Flow: 0.064 mgd**

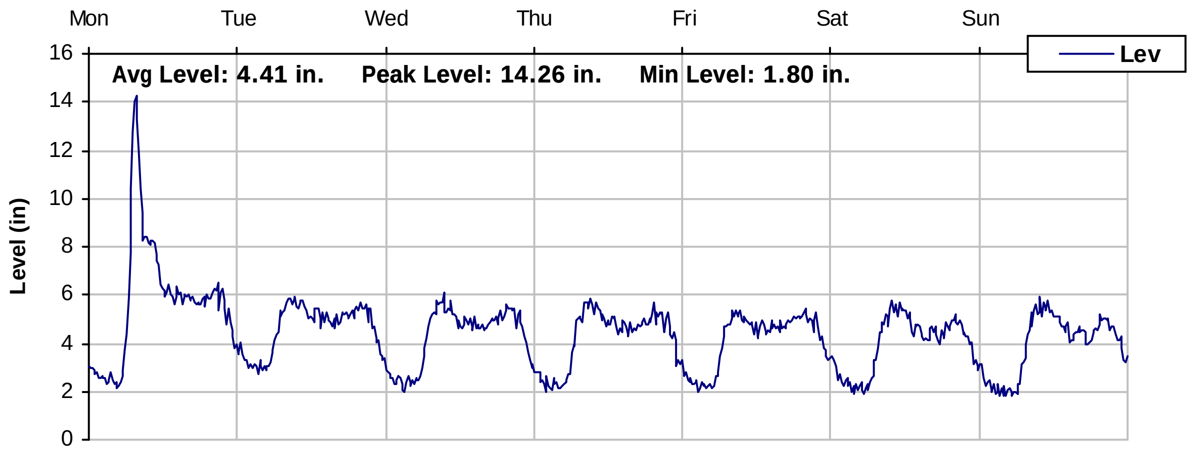
SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014

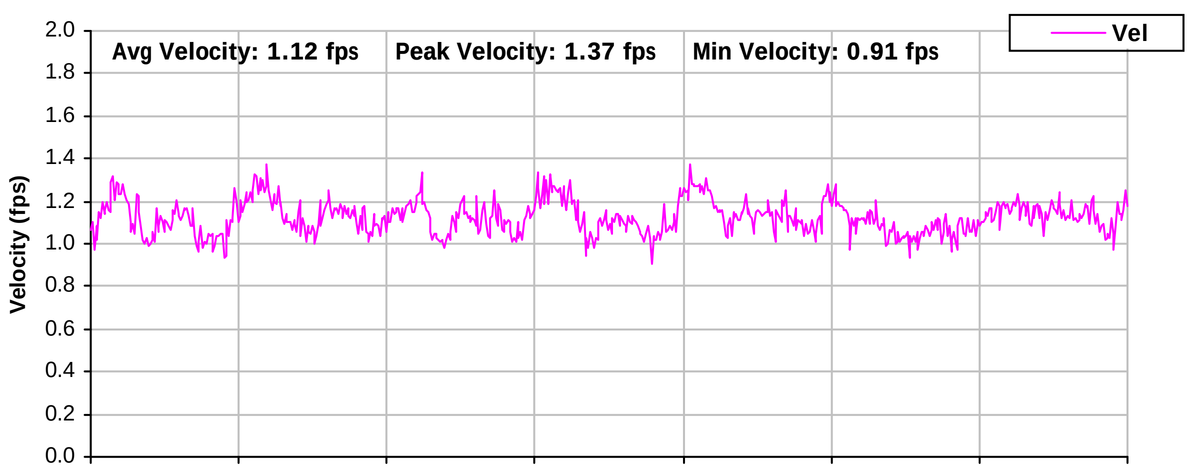


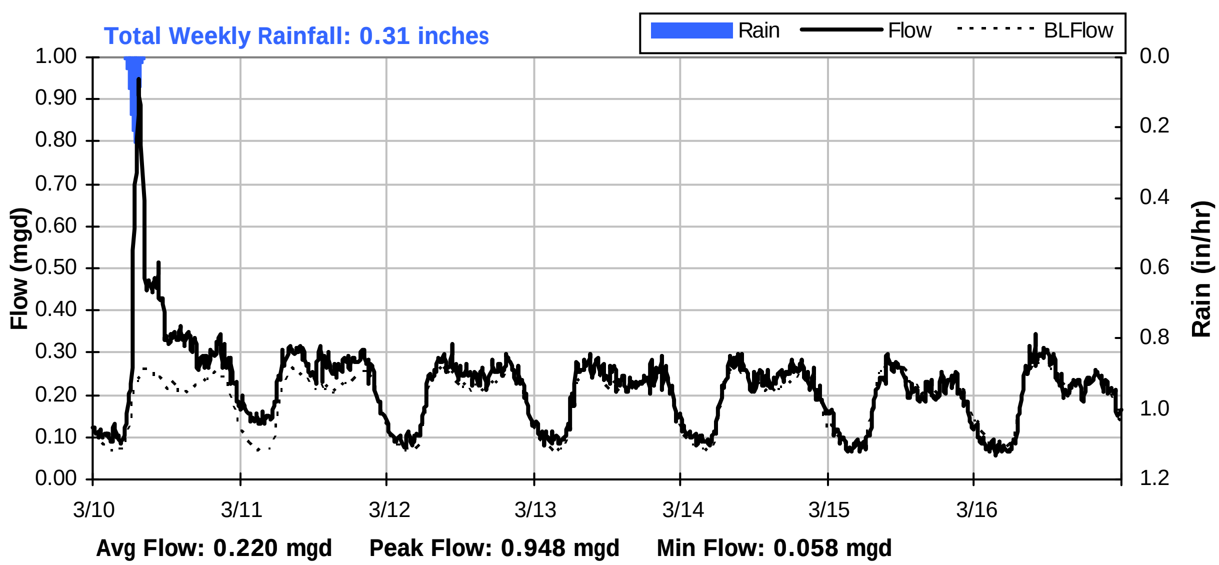
SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



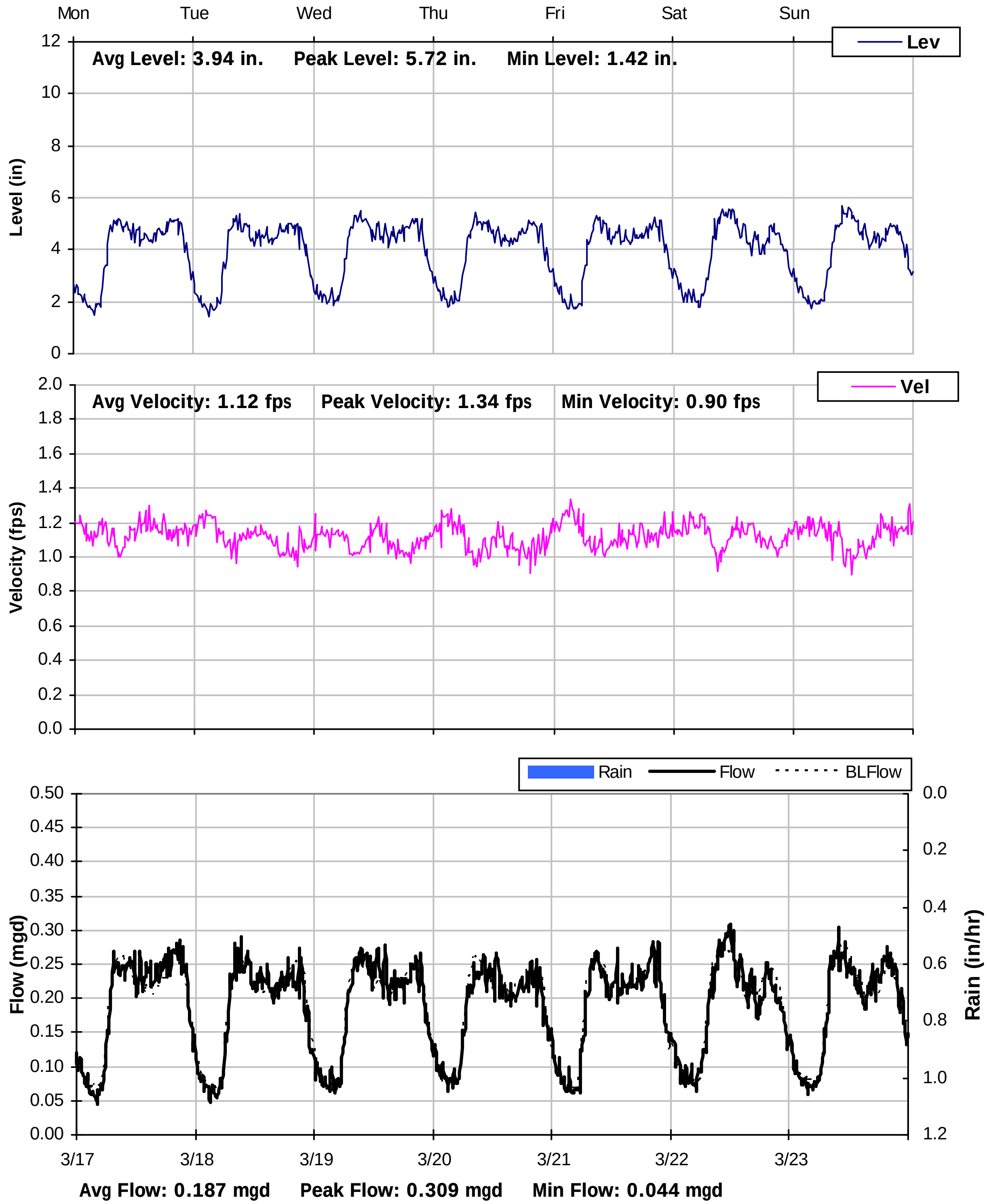
SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014



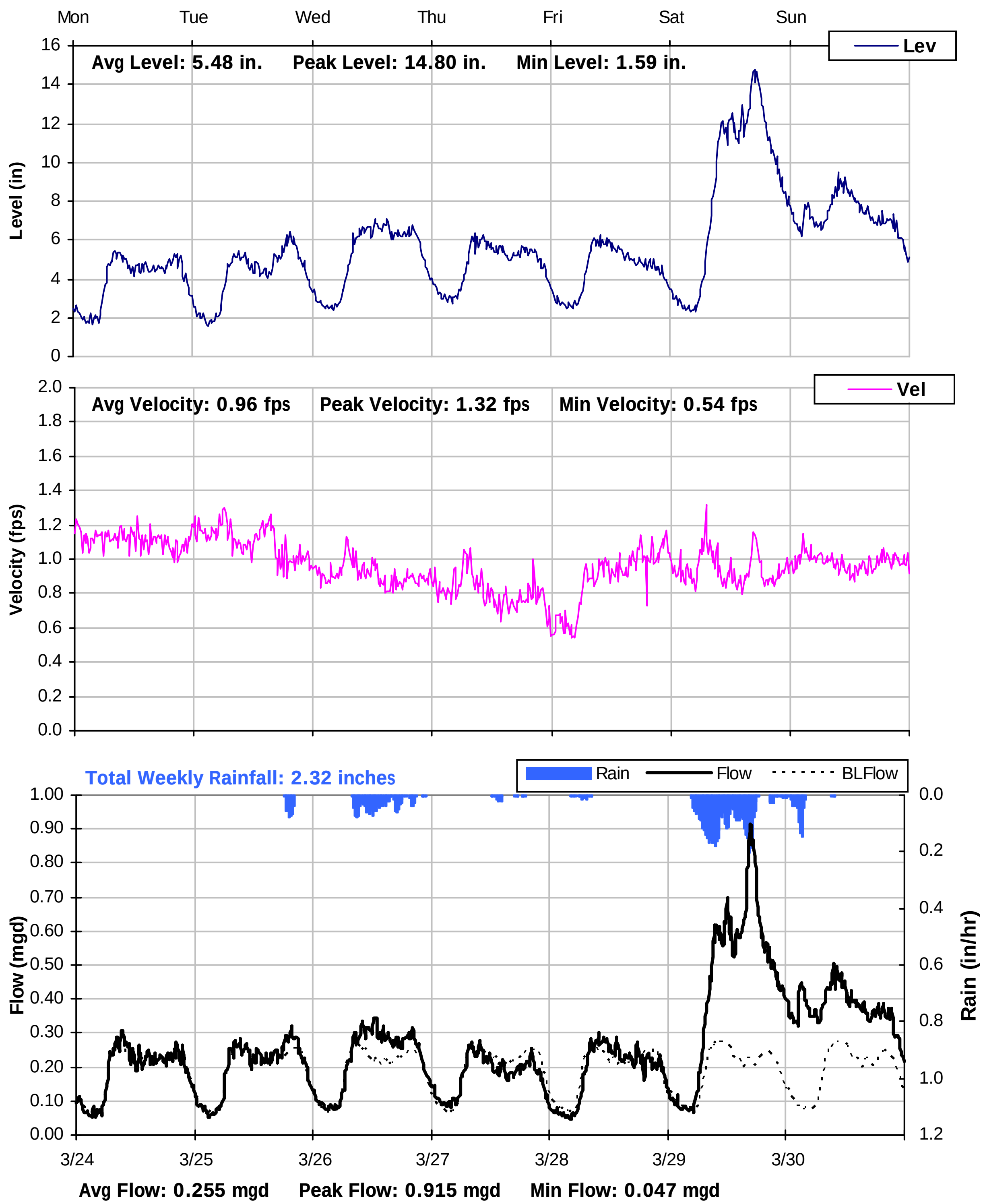




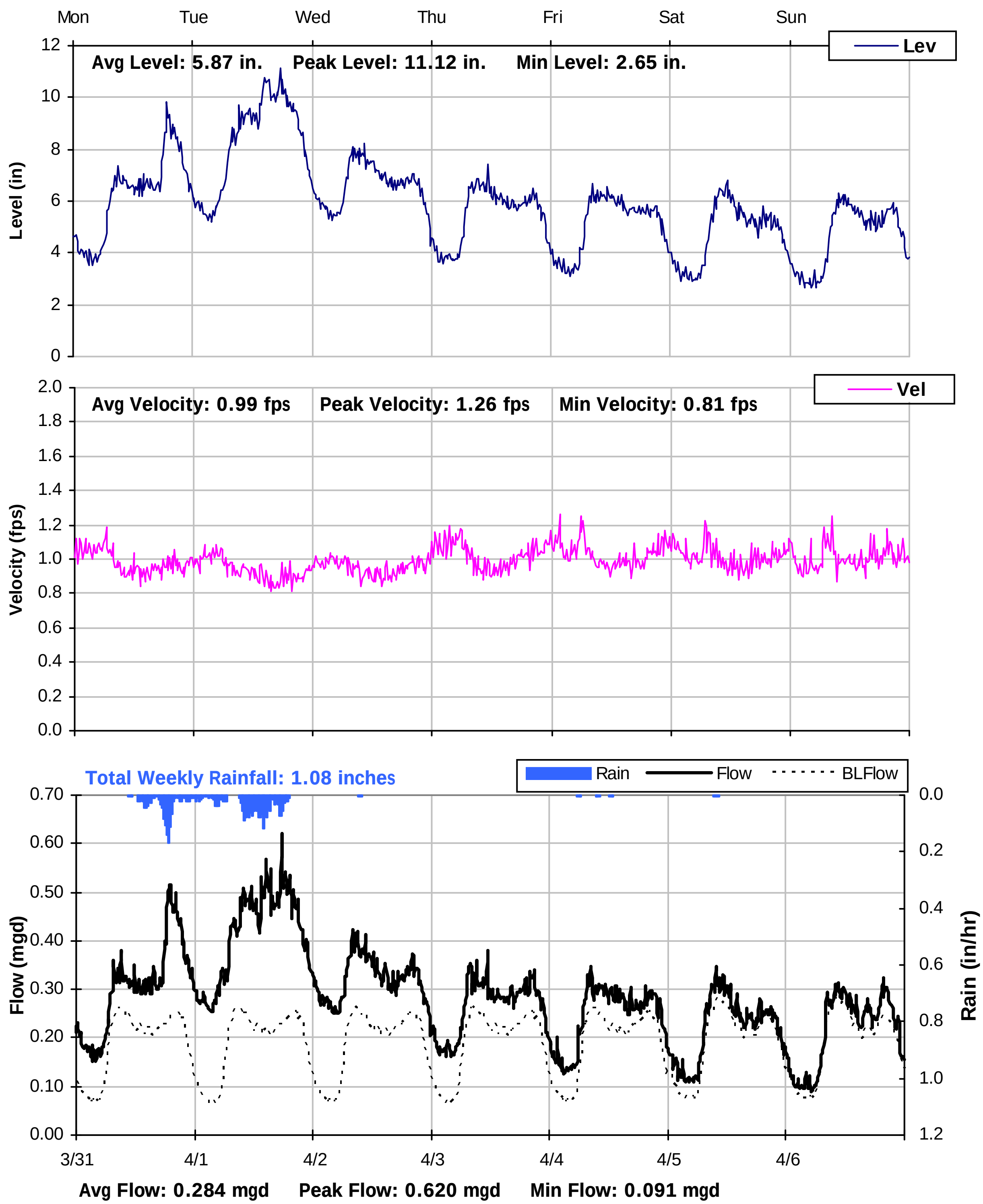
SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



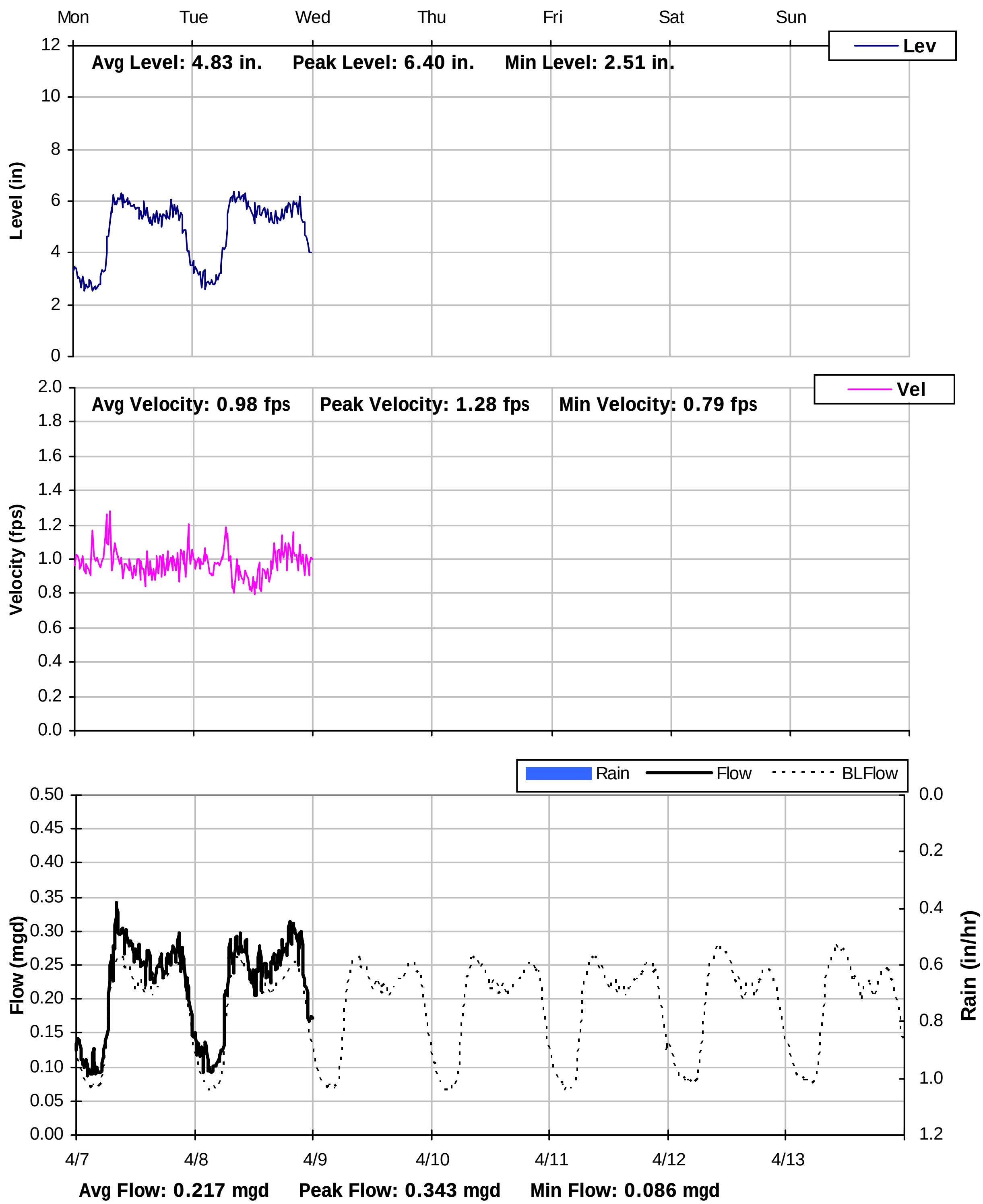
SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014



SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/31/2014 to 4/7/2014



SITE 3
Weekly Level, Velocity and Flow Hydrographs
4/7/2014 to 4/14/2014



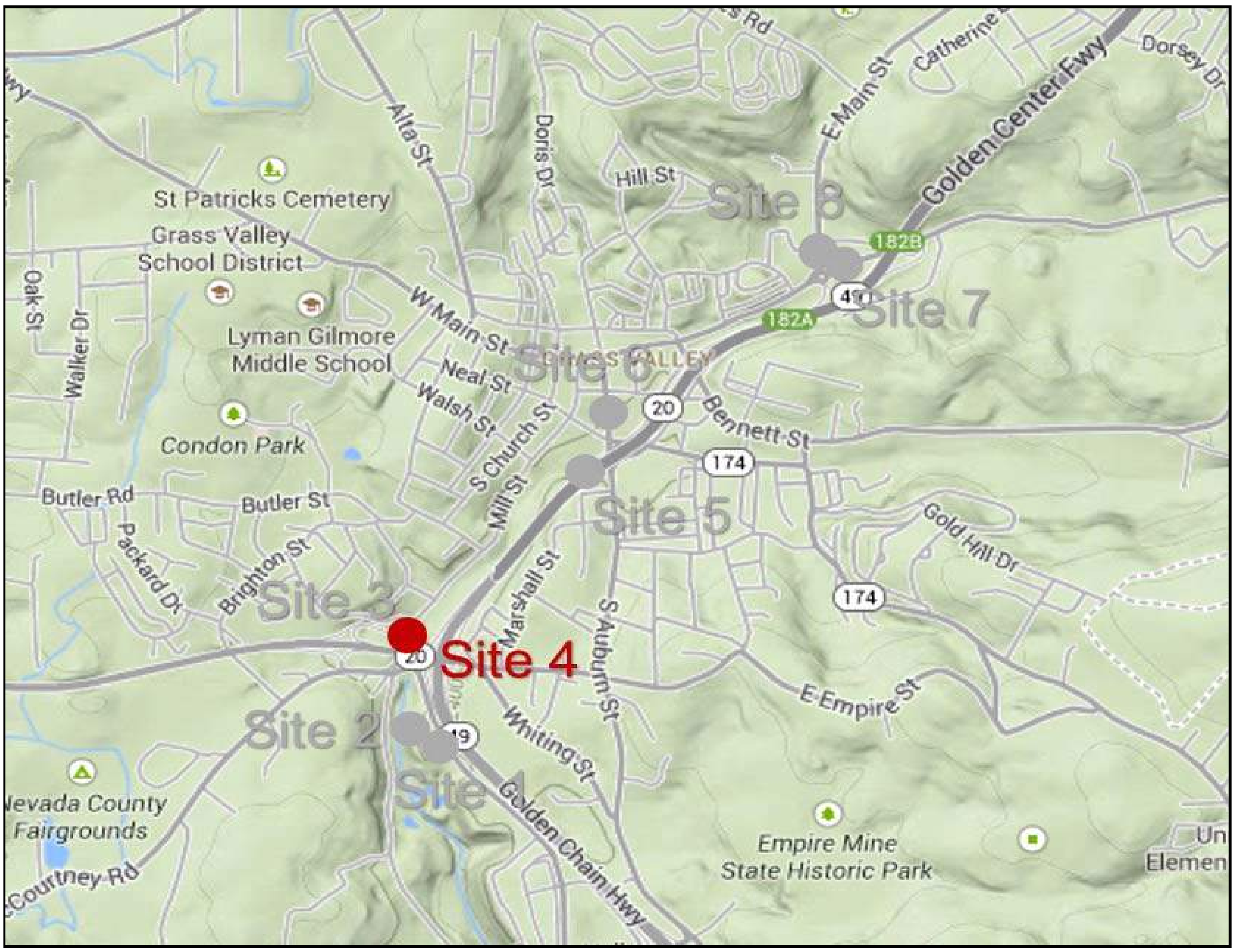
City of Grass Valley

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 4

Location: Southbound Golden Chain Highway Rood Expressway off-ramp

Data Summary Report



Vicinity Map: Site 4

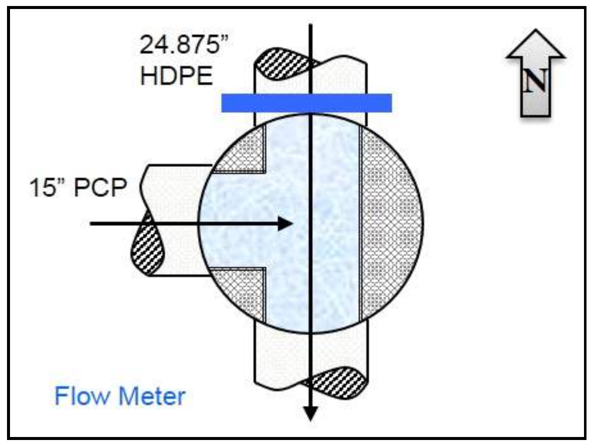
SITE 4

Site Information

Location:	Southbound Golden Chain Highway Rood Expressway off-ramp
Coordinates:	121.0686° W, 39.2103° N
Rim Elevation:	2370 feet
Pipe Diameter:	24.875 inches
Baseline Flow:	1.042 mgd
Peak Measured Flow:	6.984 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

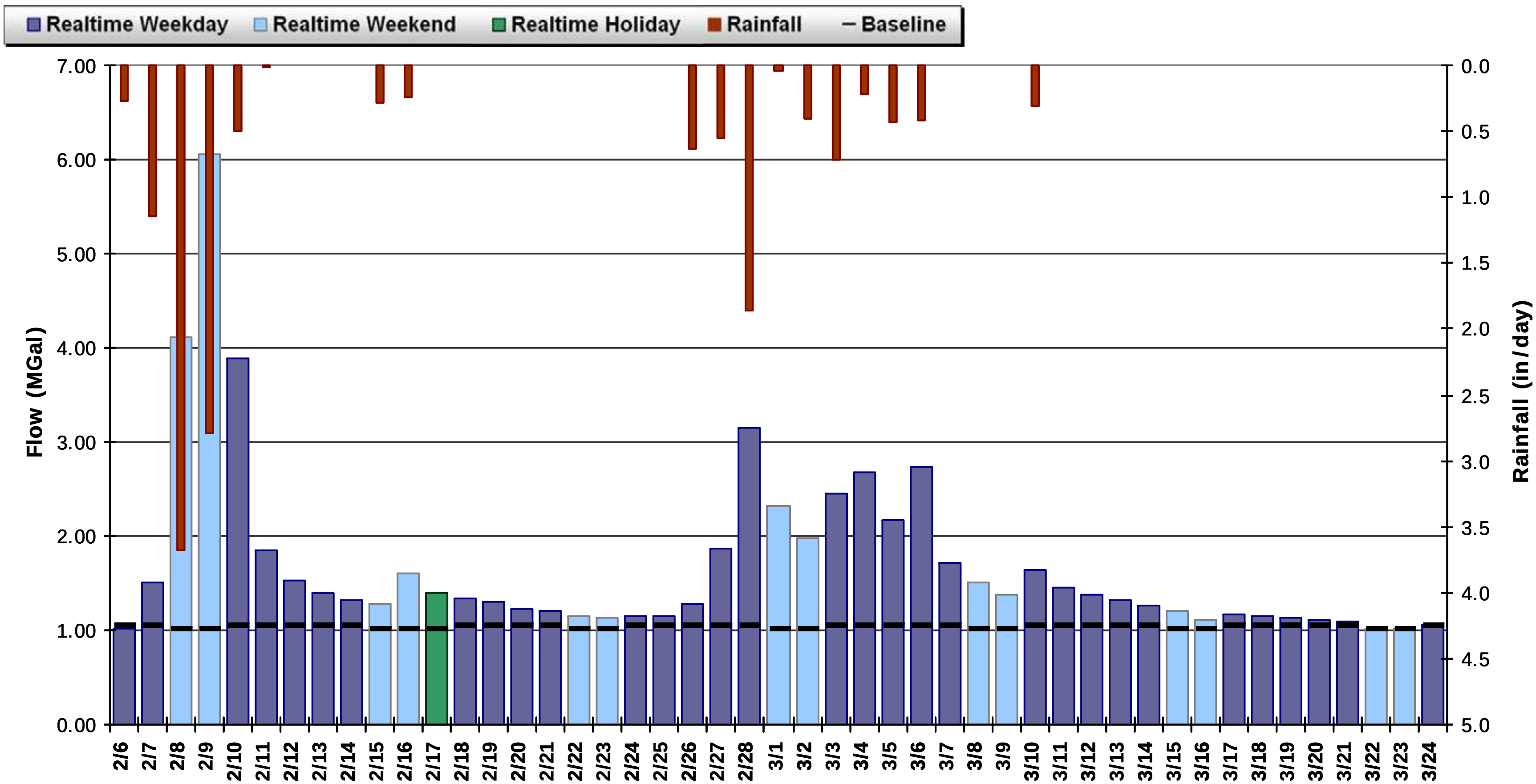
Plan View

SITE 4

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 1.702 MGal **Peak Daily Flow: 6.053 MGal** **Min Daily Flow: 1.013 MGal**

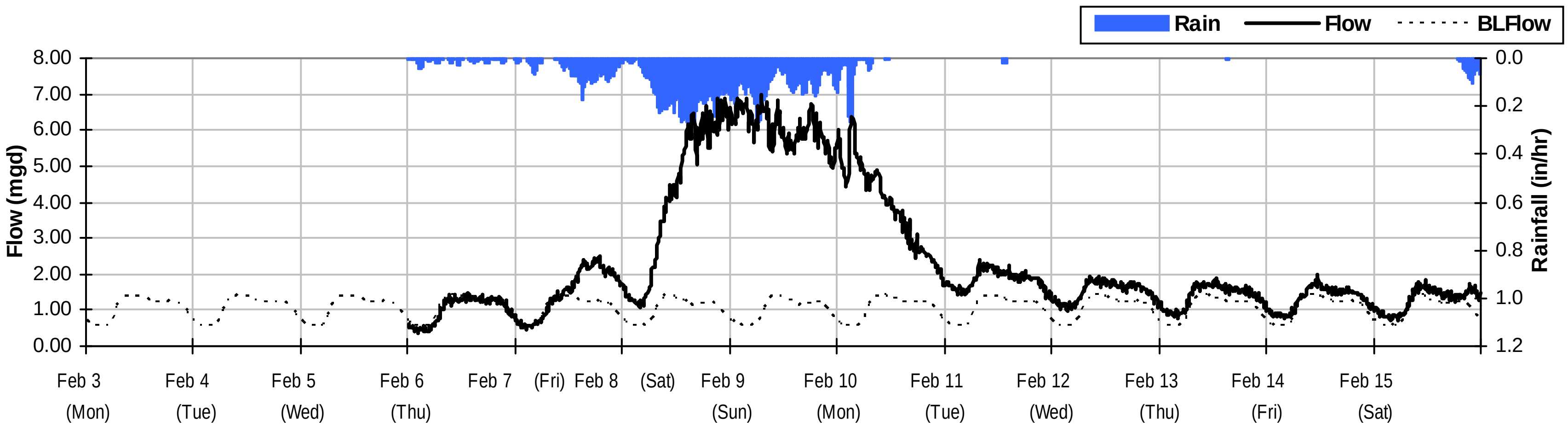
Total Period Rainfall: 14.51 inches

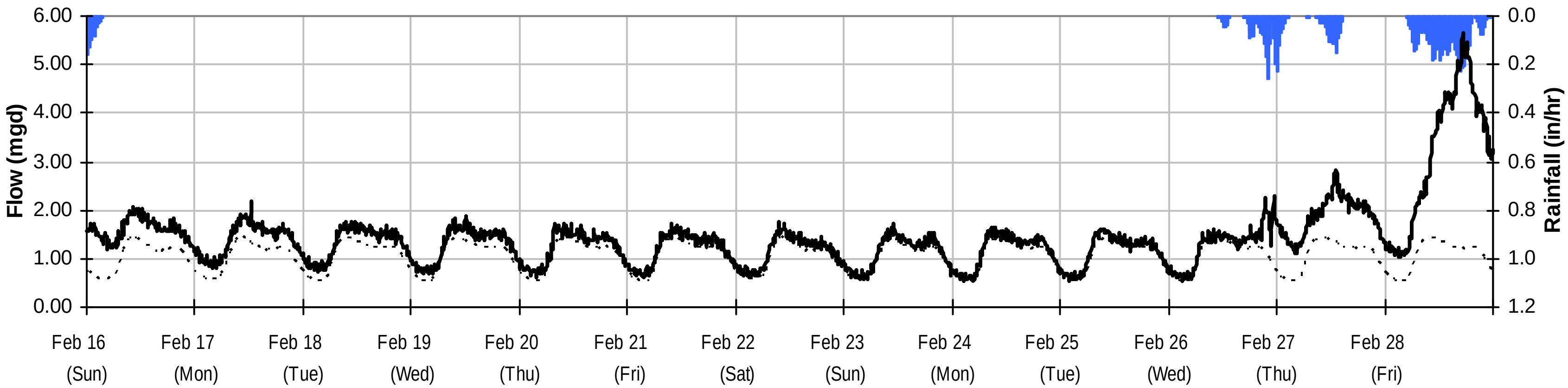


SITE 4

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 1.865 mgd** **Peak Flow: 6.984 mgd** **Min Flow: 0.377 mgd**

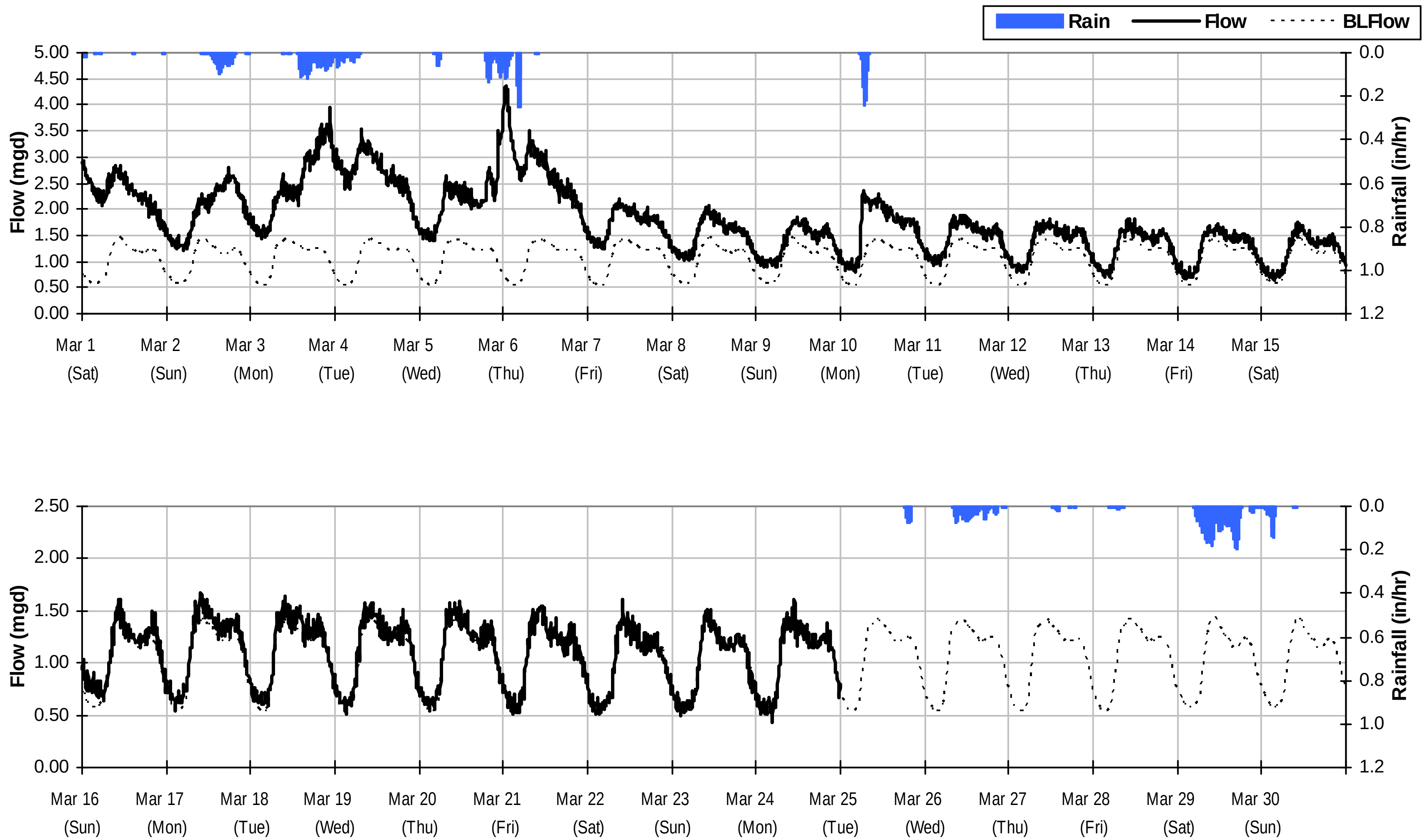




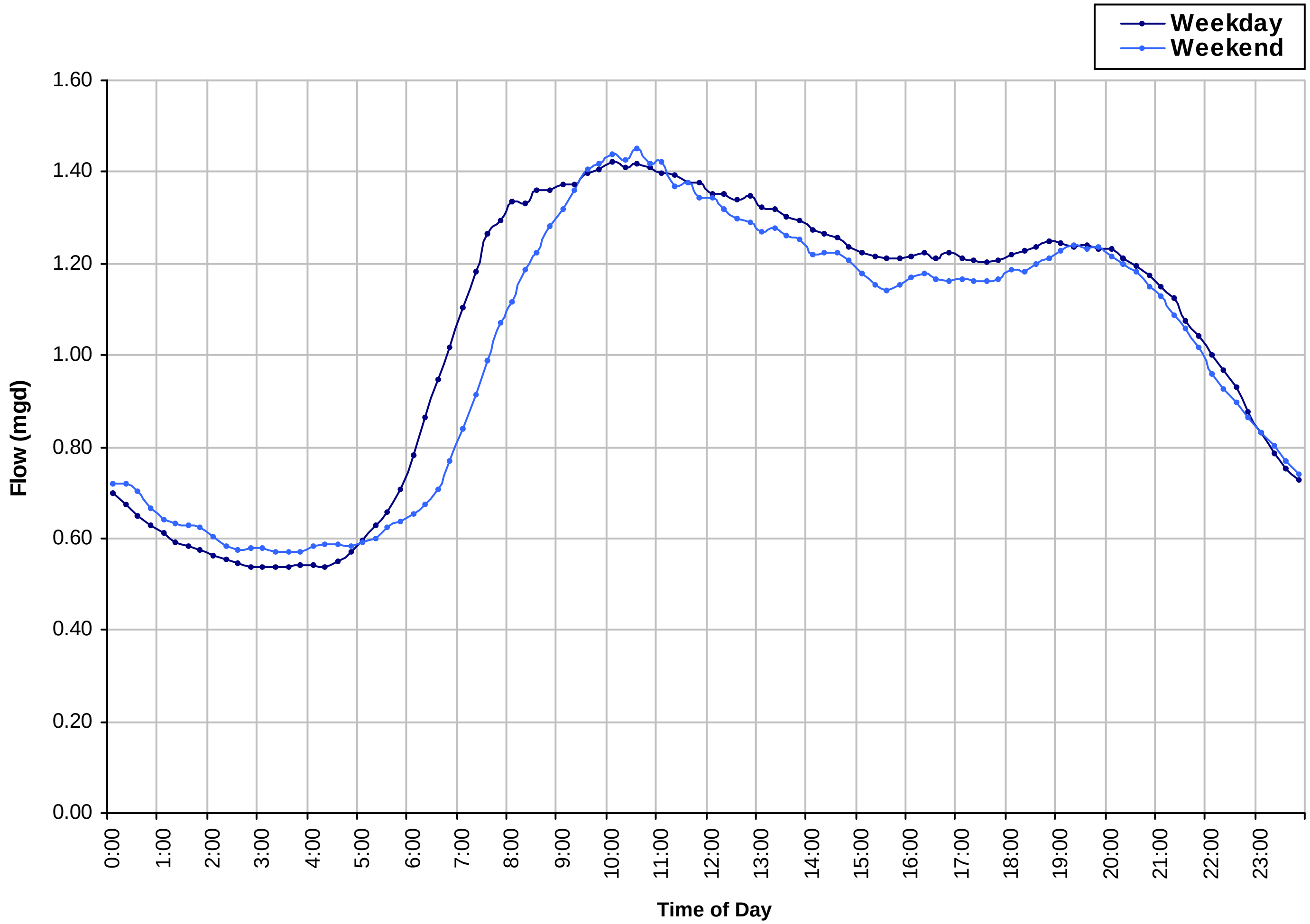
SITE 4

Monthly Flow Summary: March, 2014

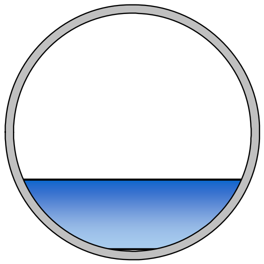
Total Monthly Rainfall: 4.85 inches **Avg Flow: 1.545 mgd** **Peak Flow: 4.359 mgd** **Min Flow: 0.432 mgd**



SITE 4
Baseline Flow Hydrographs

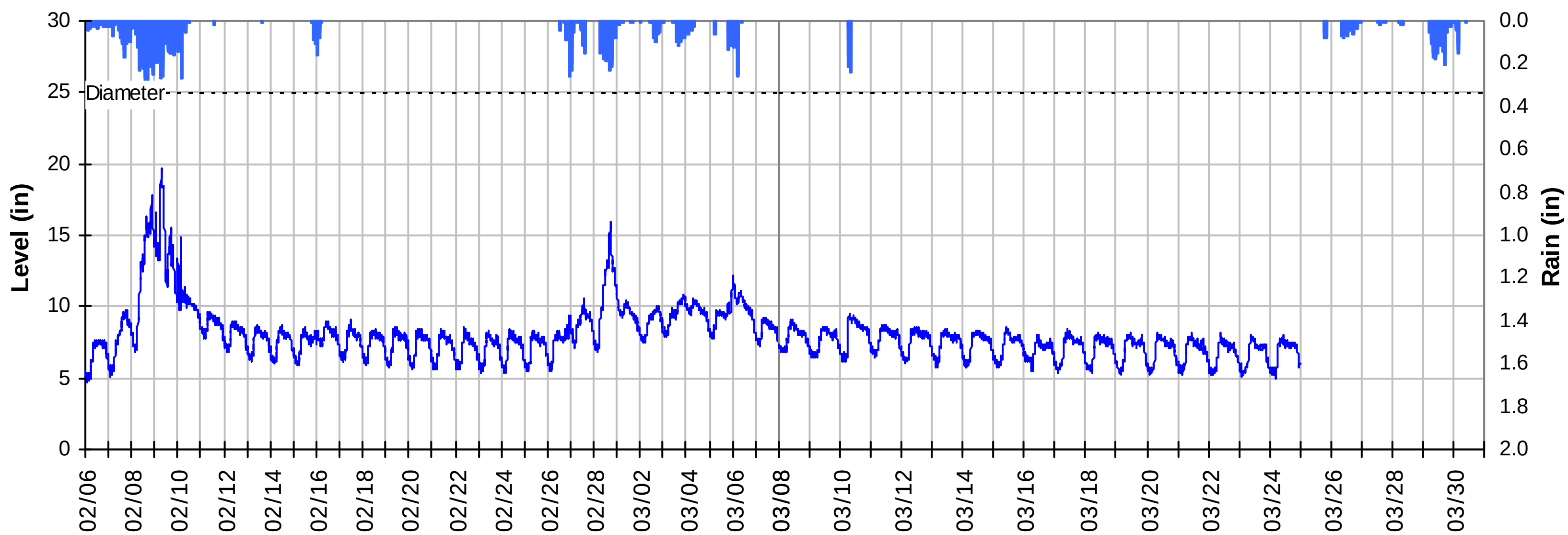


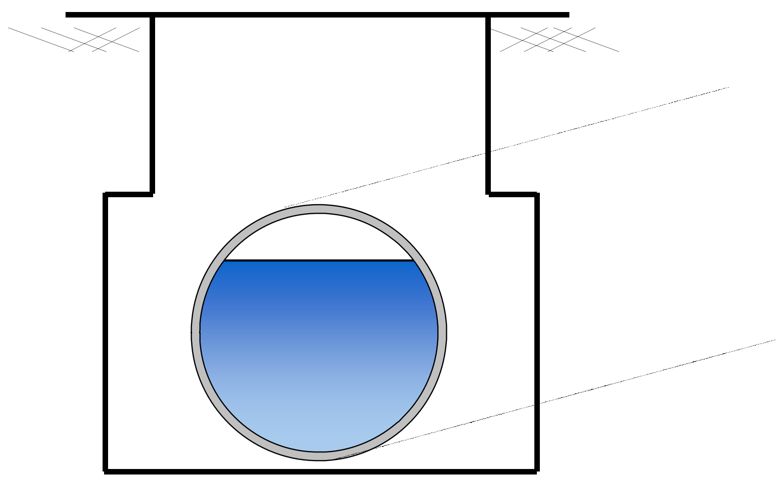
Baseline Flow:
1.042 *mgd*



SITE 4
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

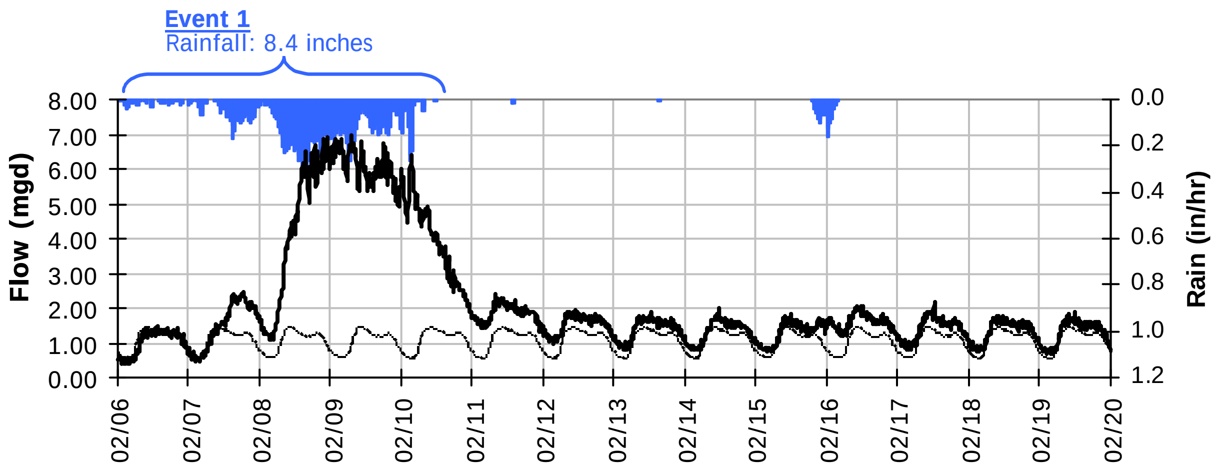




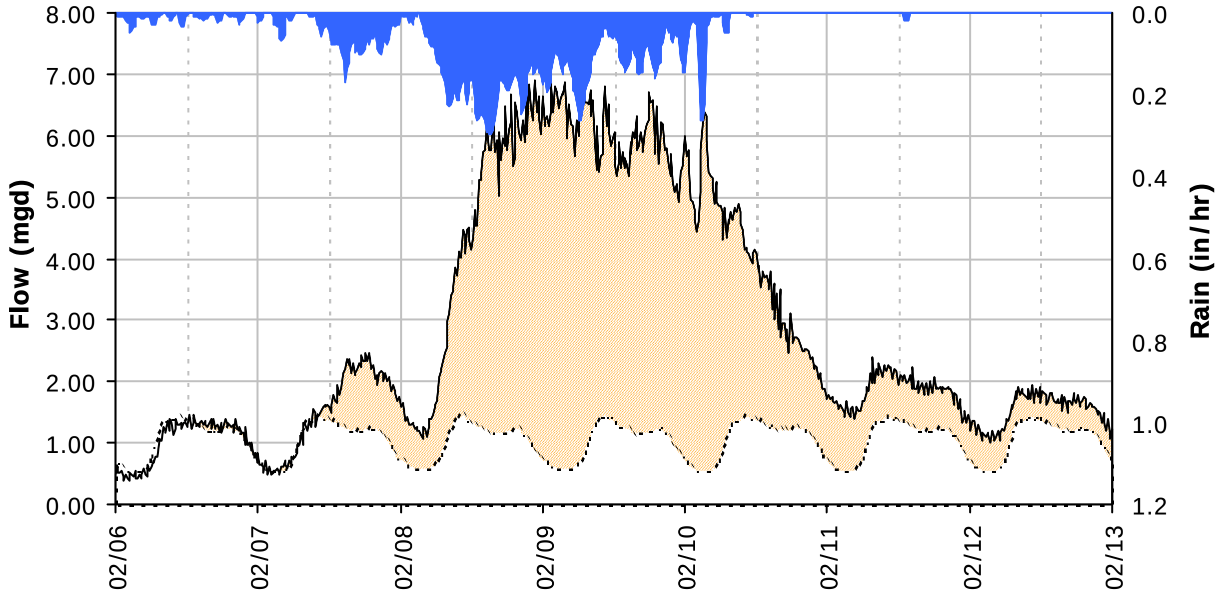
Pipe Diameter:	24.9 *inches*
Peak Measured Level:	19.7 *inches*
Peak d/D Ratio:	0.79

SITE 4
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

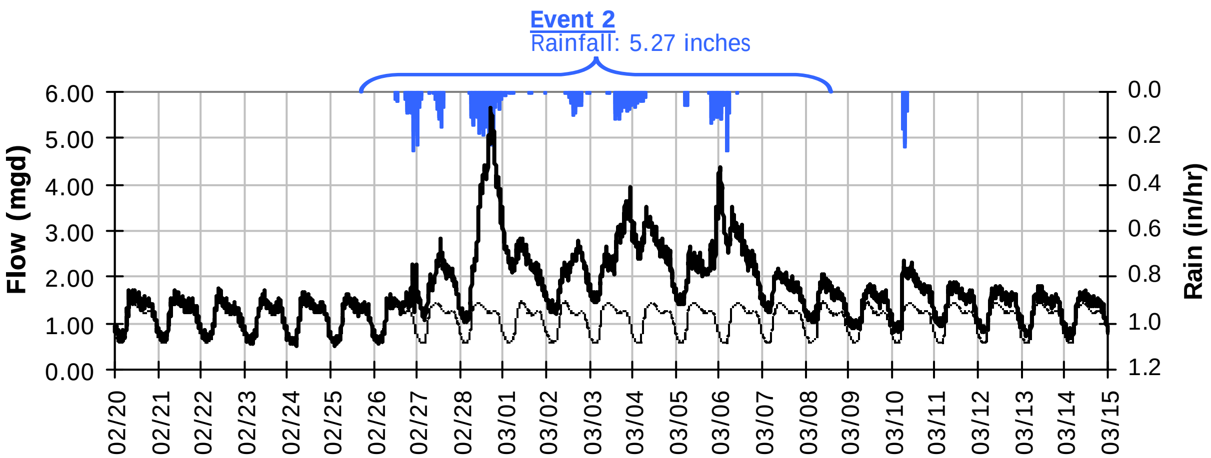


Storm Event I/I Analysis (Rain = 8.40 inches)

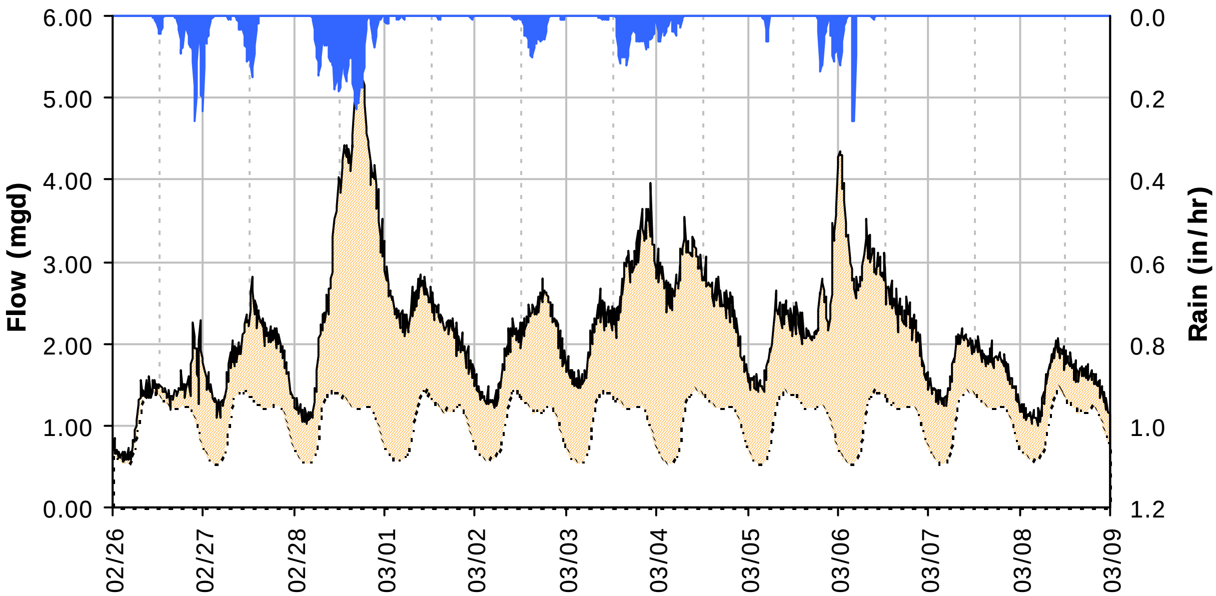
Capacity		Inflow / Infiltration	
Peak Flow:	6.98 *mgd*	**Peak I/I Rate:**	6.30 *mgd*
PF:	6.70	**Total I/I:**	12,653,000 *gallons*

SITE 4
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



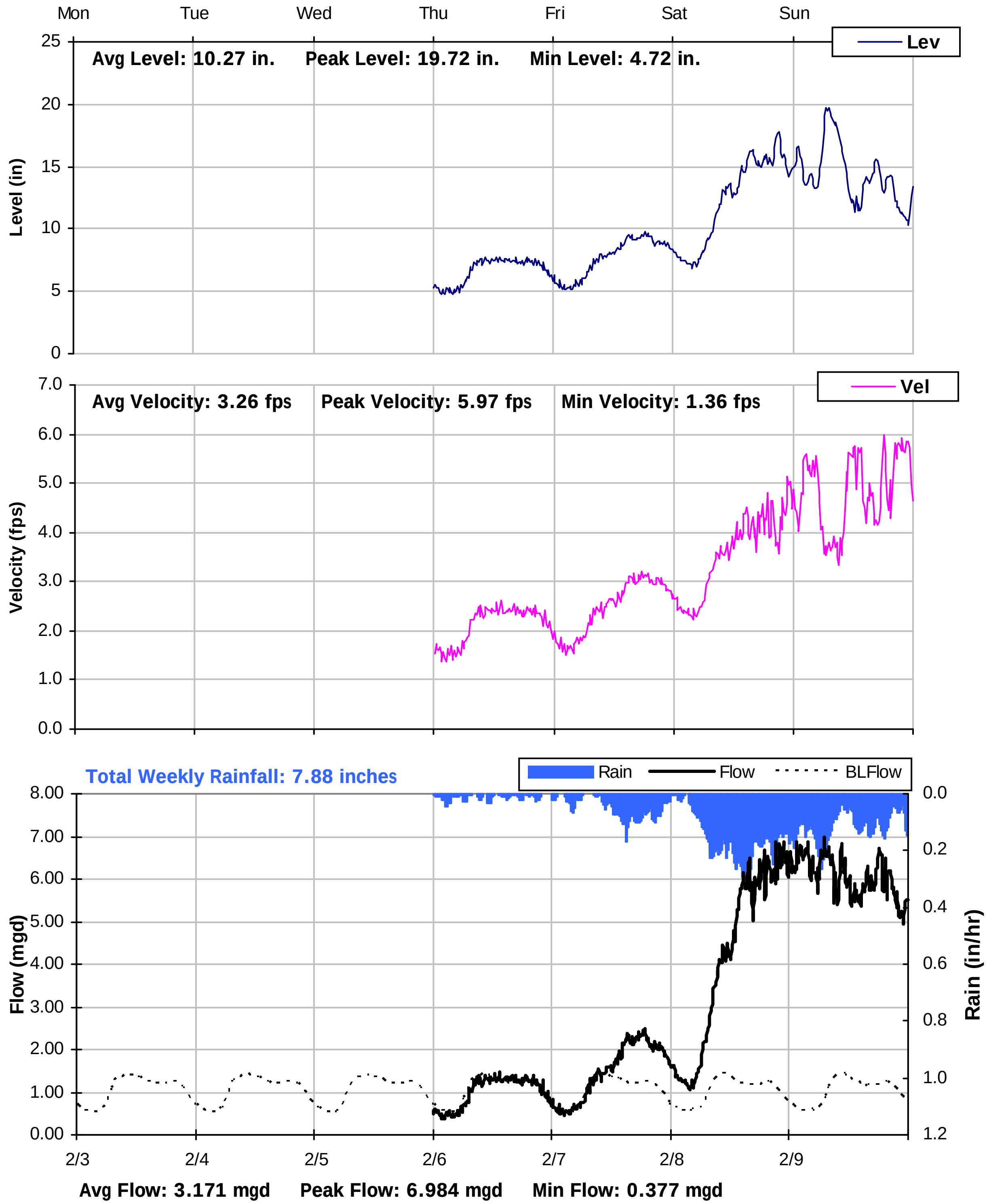
Event 2 Detail Graph



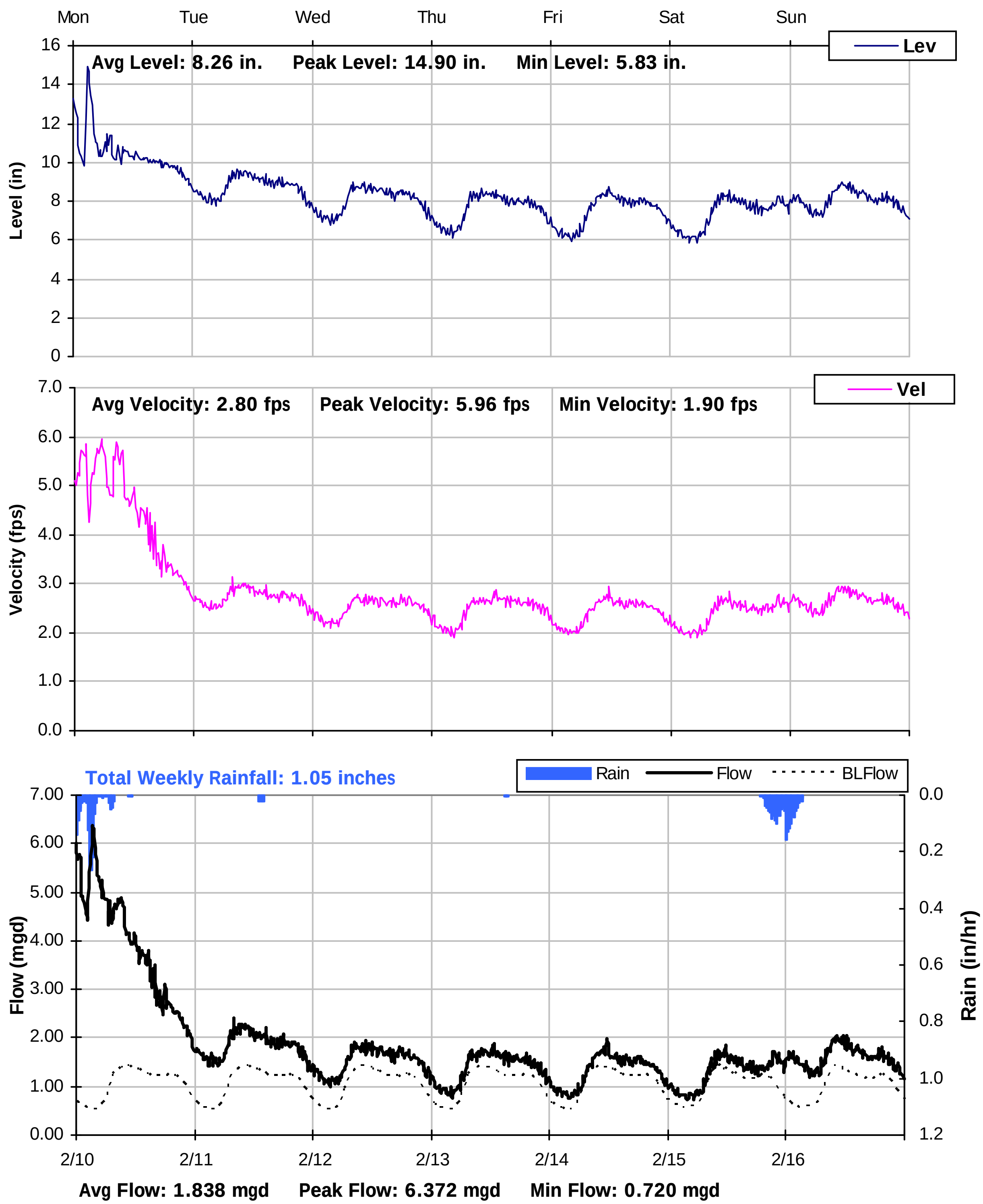
Storm Event I/I Analysis (Rain = 5.27 inches)

Capacity		**Inflow / Infiltration**	
Peak Flow:	5.67 *mgd*	**Peak I/I Rate:**	4.46 *mgd*
PF:	5.44	**Total I/I:**	12,414,000 *gallons*

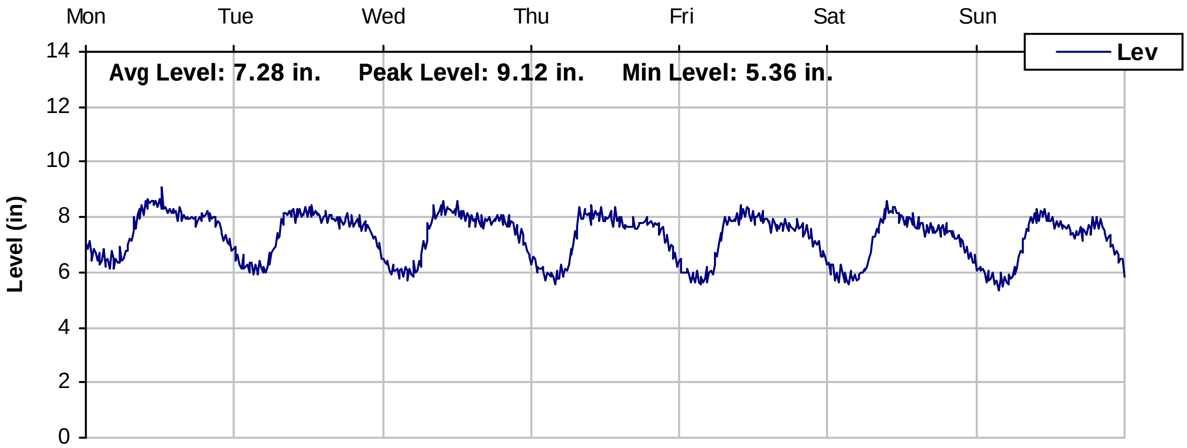
SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014

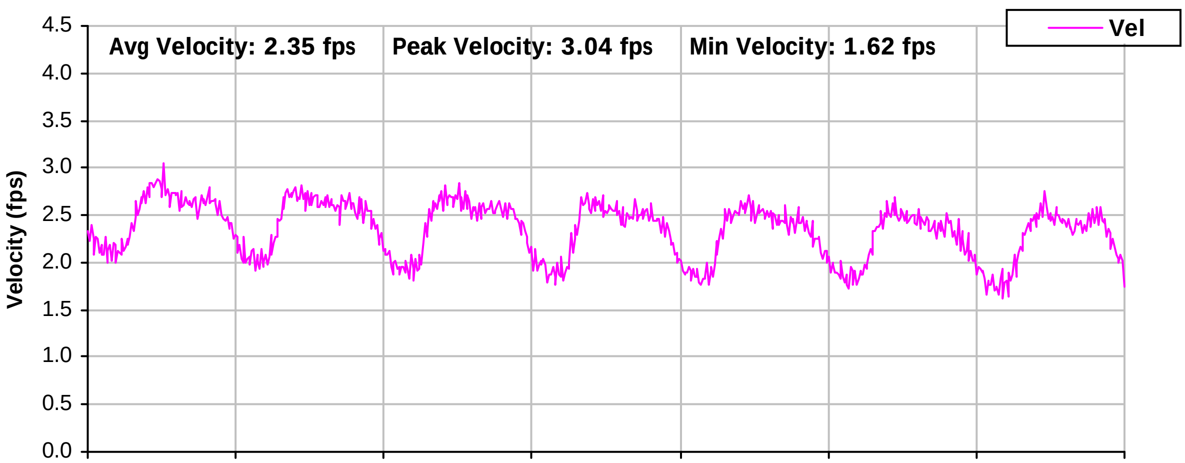


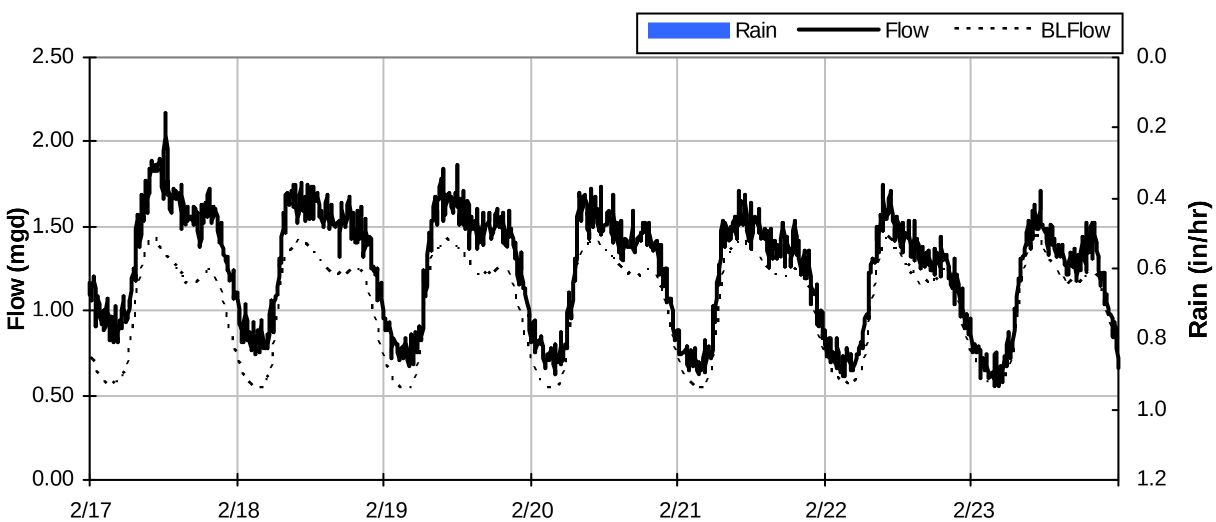
SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014



SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014

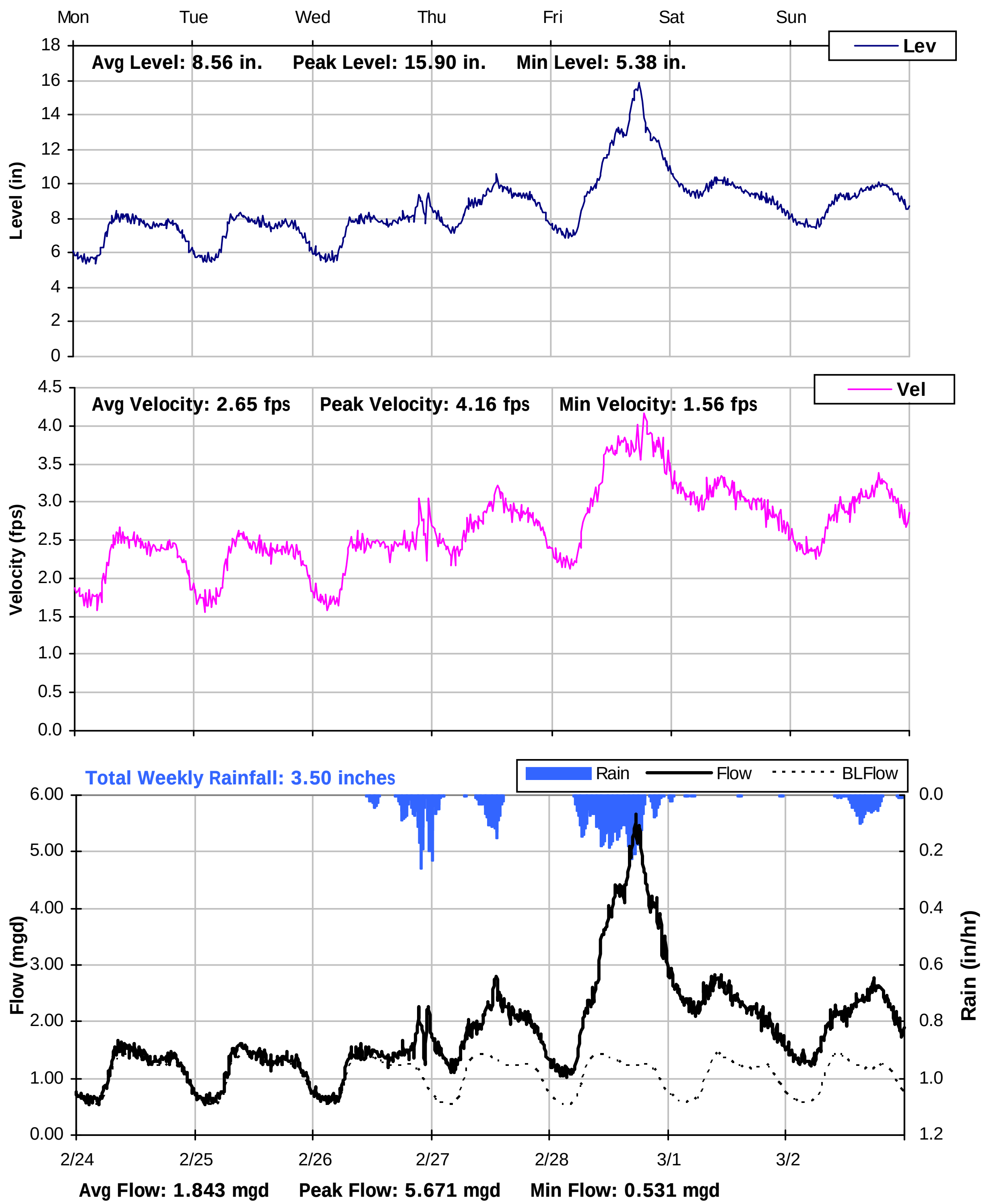




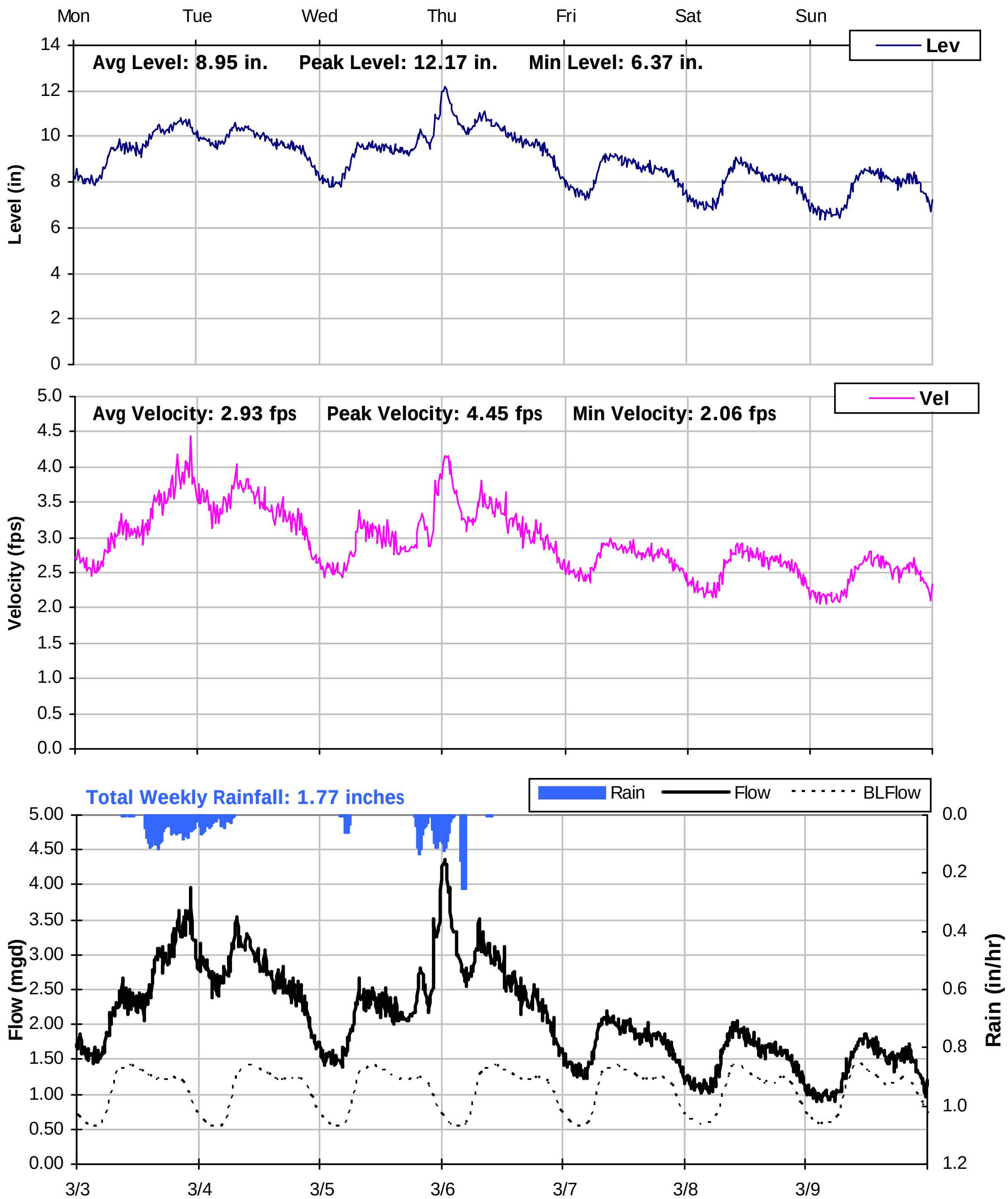


Avg Flow: 1.250 mgd **Peak Flow: 2.173 mgd** **Min Flow: 0.557 mgd**

SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014

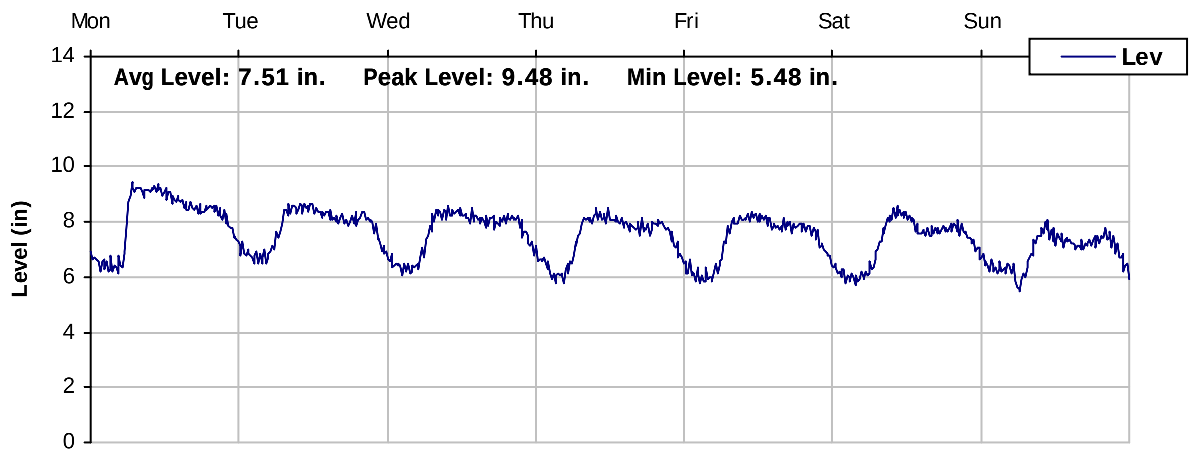


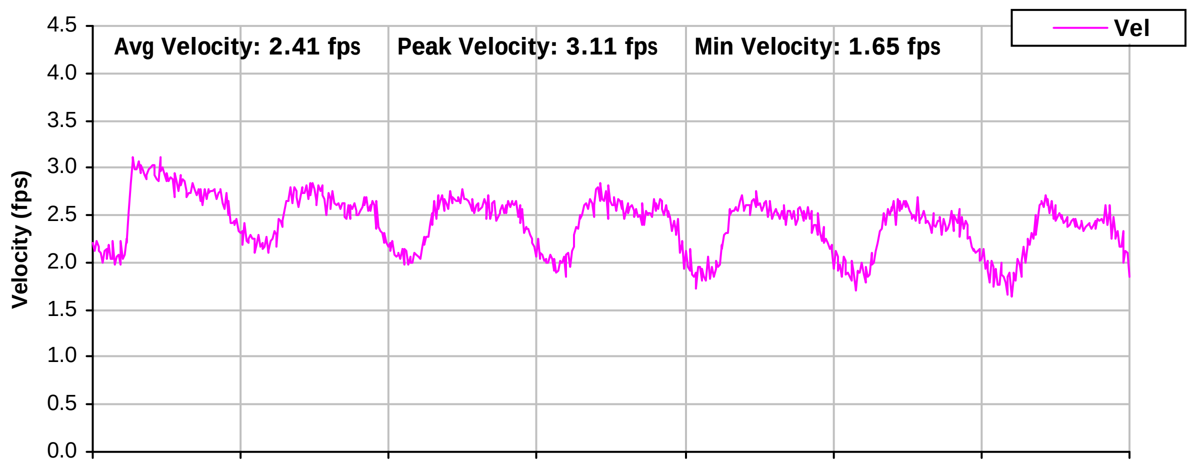
SITE 4
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014

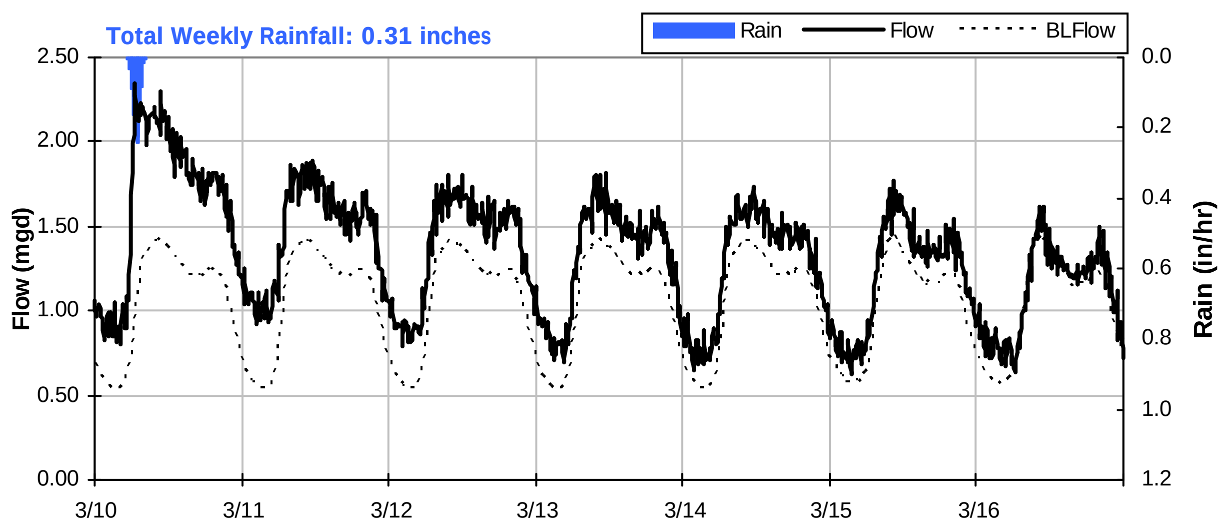


Avg Flow: 2.094 mgd **Peak Flow: 4.359 mgd** **Min Flow: 0.888 mgd**

SITE 4
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014

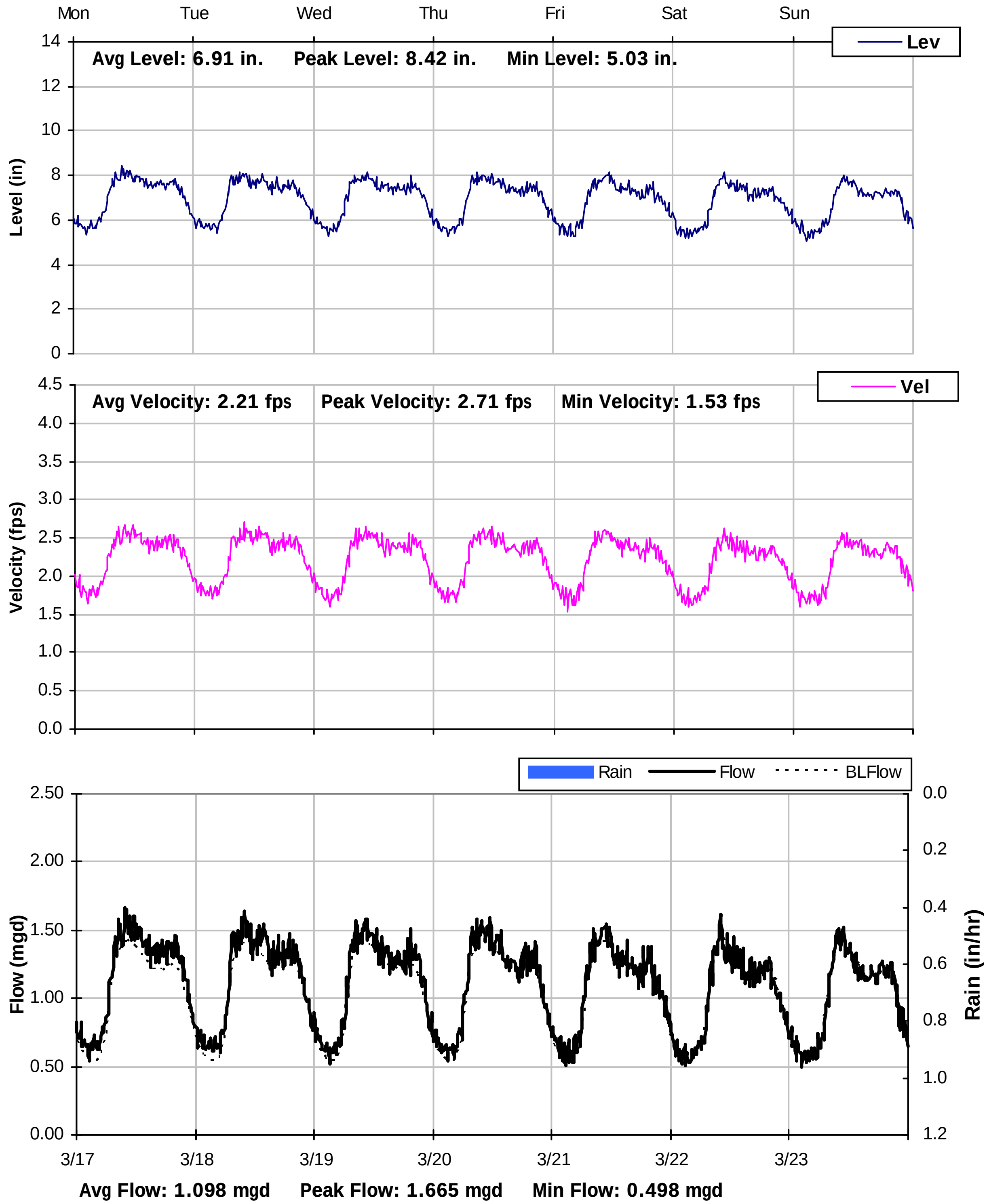




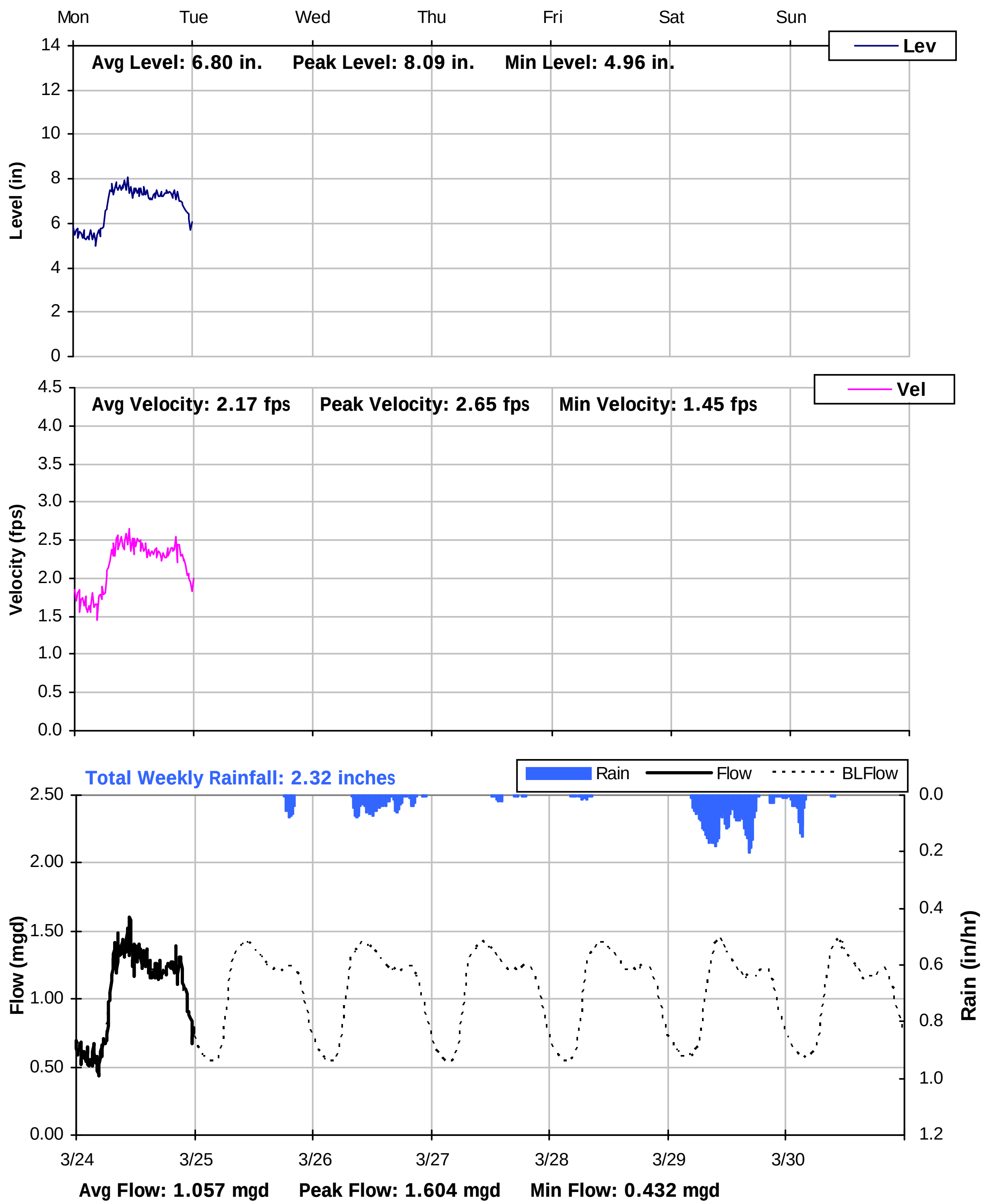


Avg Flow: 1.340 mgd **Peak Flow: 2.346 mgd** **Min Flow: 0.627 mgd**

SITE 4
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



SITE 4
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014



City of Grass Valley

Sanitary Sewer Flow Monitoring

Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 5

Location: Southbound Golden Chain Highway Auburn Street on-ramp

Data Summary Report

Vicinity Map: Site 5

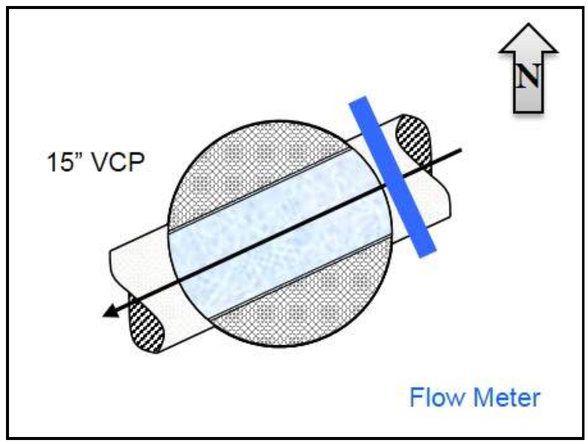
SITE 5

Site Information

Location:	Southbound Golden Chain Highway Auburn Street on-ramp
Coordinates:	121.0630° W, 39.2155° N
Rim Elevation:	2405 feet
Pipe Diameter:	15 inches
Baseline Flow:	0.146 mgd
Peak Measured Flow:	1.743 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

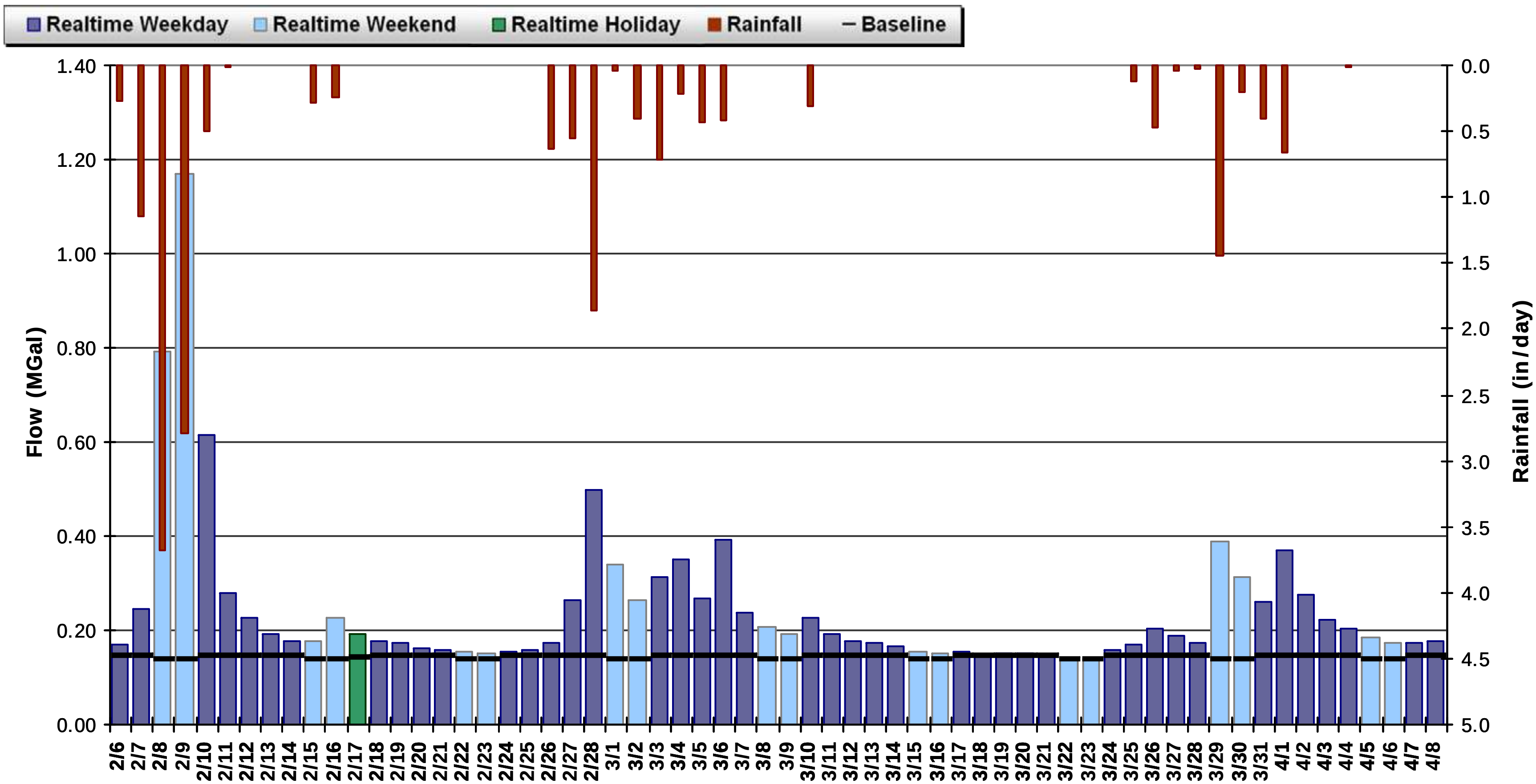
Plan View

SITE 5

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.245 MGal **Peak Daily Flow: 1.170 MGal** **Min Daily Flow: 0.141 MGal**

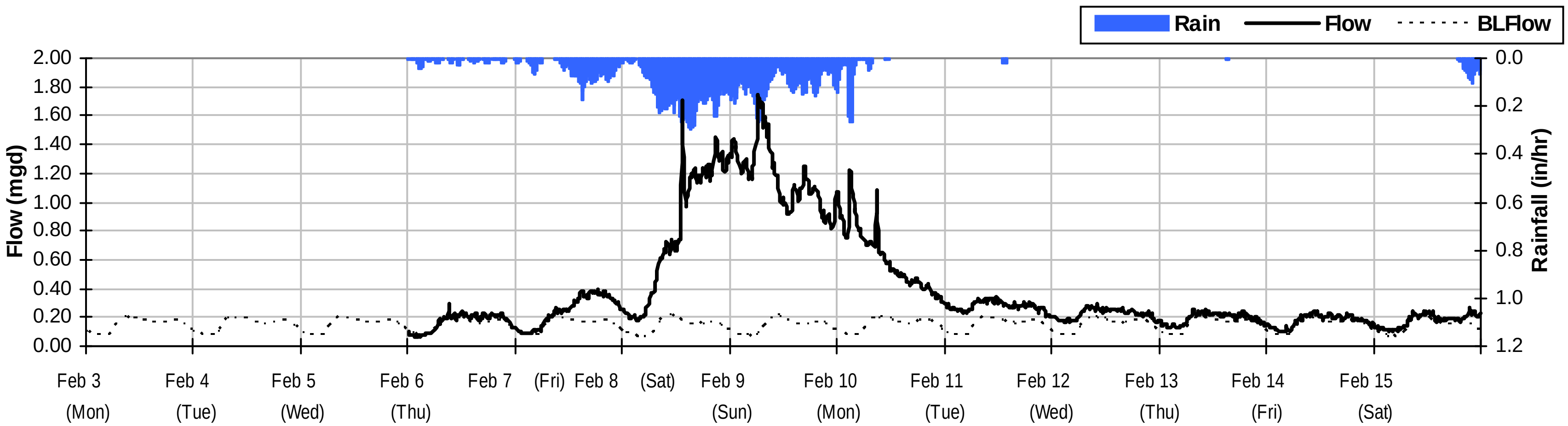
Total Period Rainfall: 17.91 inches

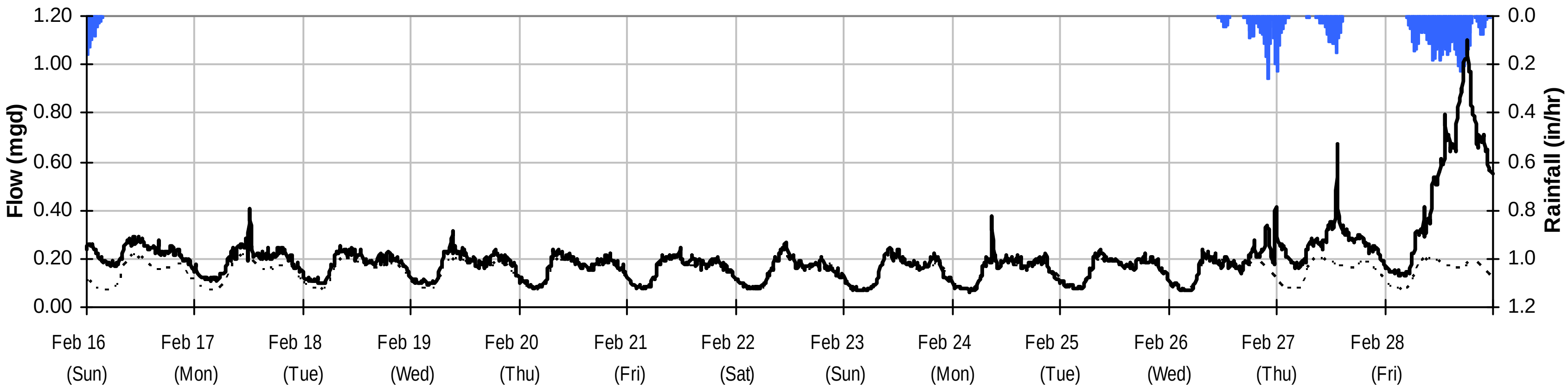


SITE 5

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.291 mgd** **Peak Flow: 1.743 mgd** **Min Flow: 0.061 mgd**

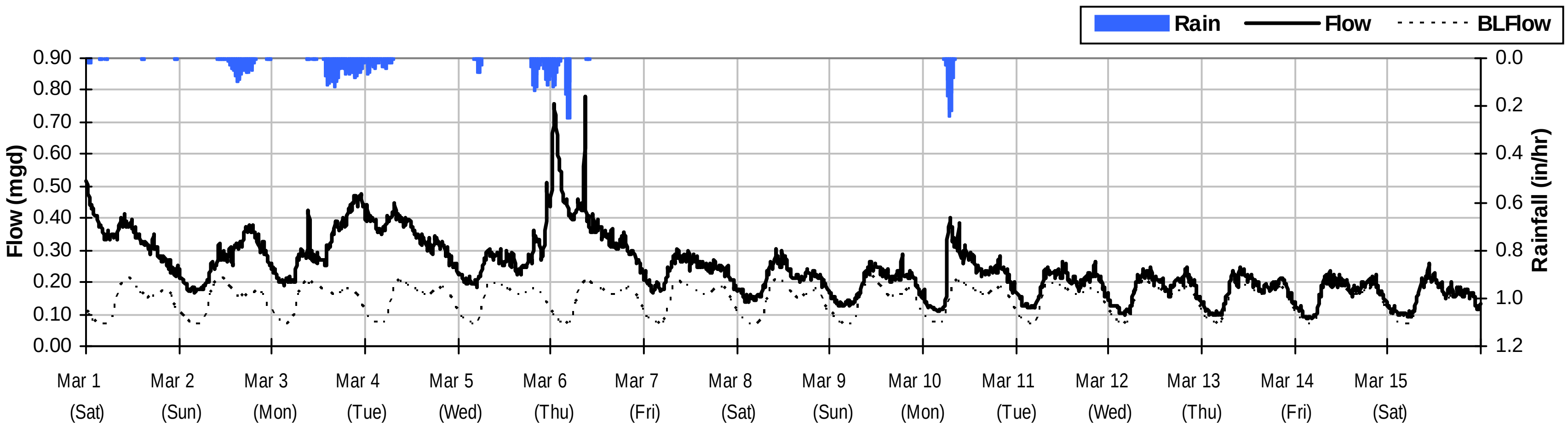


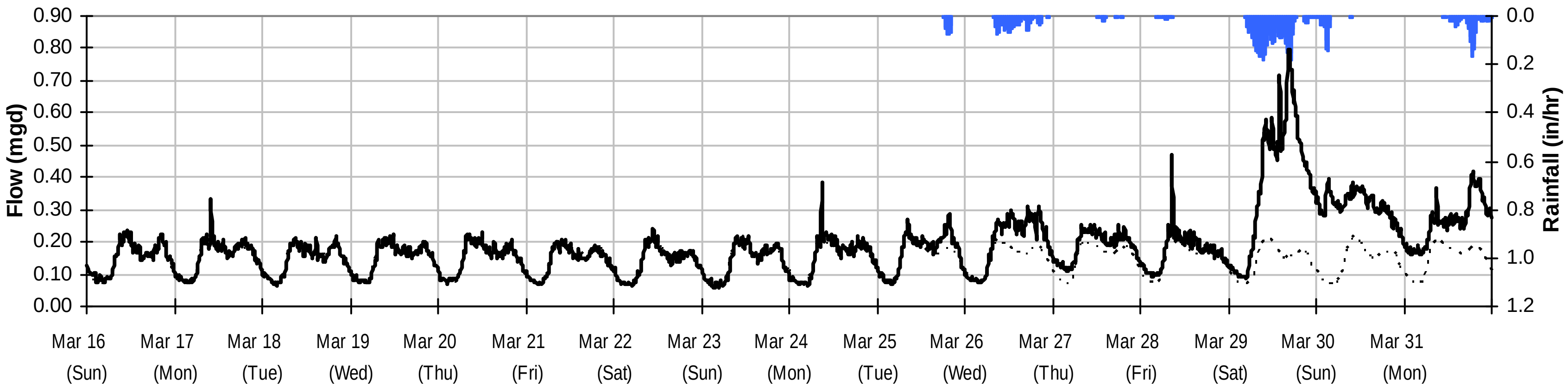


SITE 5

Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 5.24 inches **Avg Flow: 0.216 mgd** **Peak Flow: 0.796 mgd** **Min Flow: 0.056 mgd**

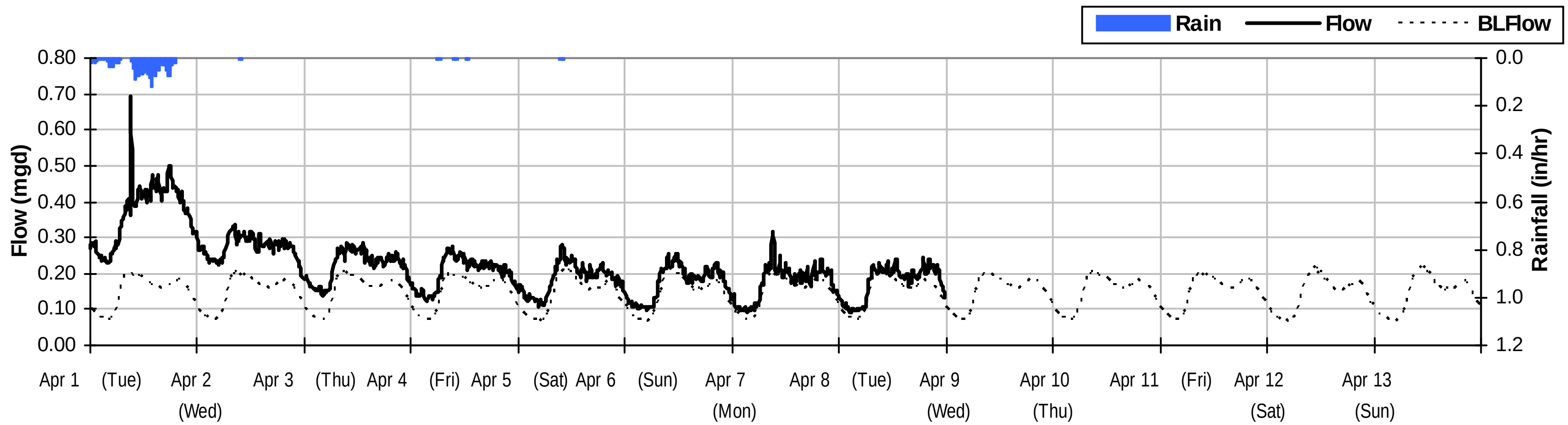




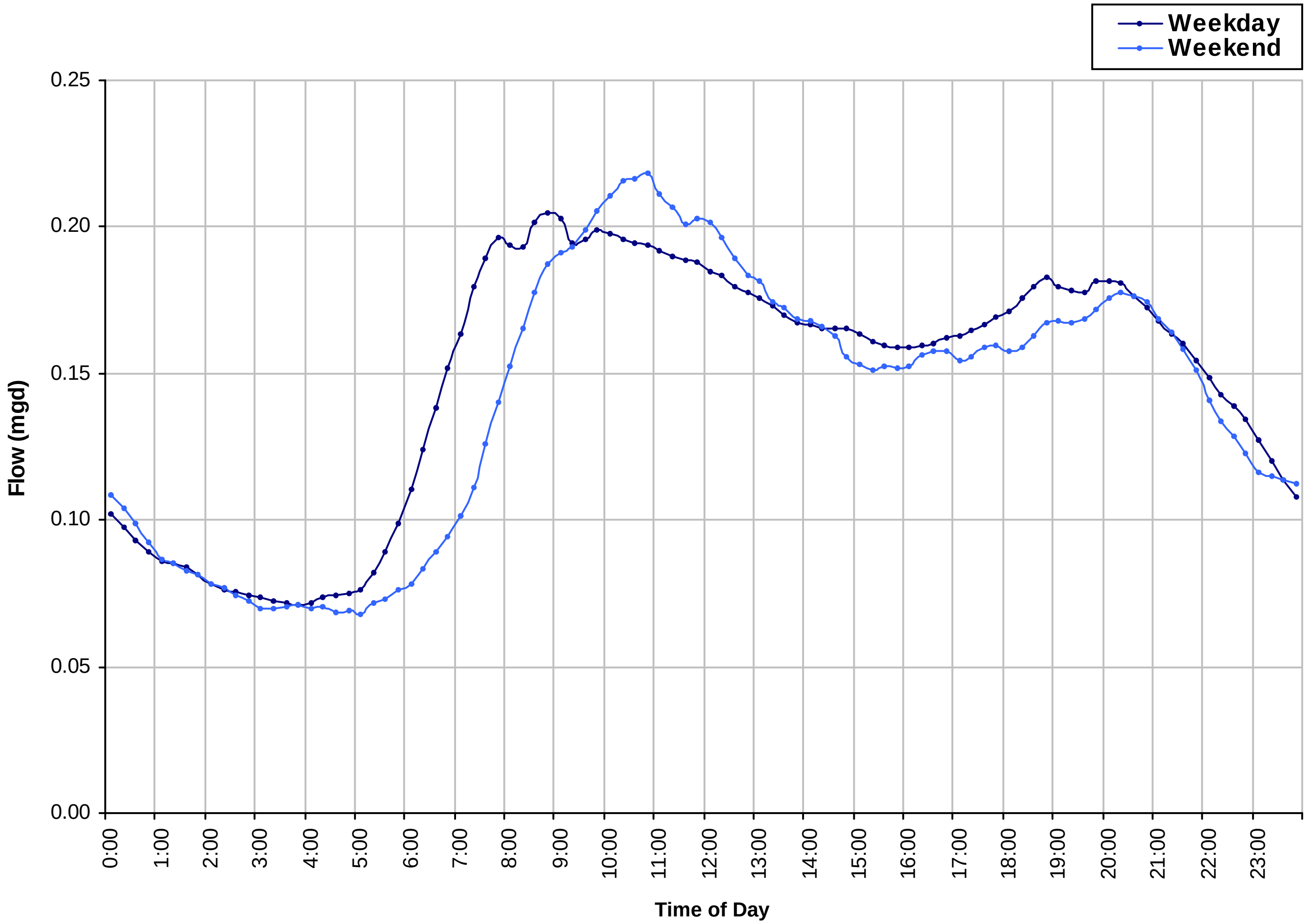
SITE 5

Monthly Flow Summary: April, 2014

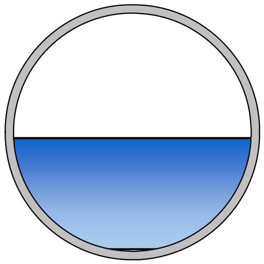
Total Monthly Rainfall: 0.69 inches **Avg Flow: 0.222 mgd** **Peak Flow: 0.695 mgd** **Min Flow: 0.090 mgd**



SITE 5
Baseline Flow Hydrographs

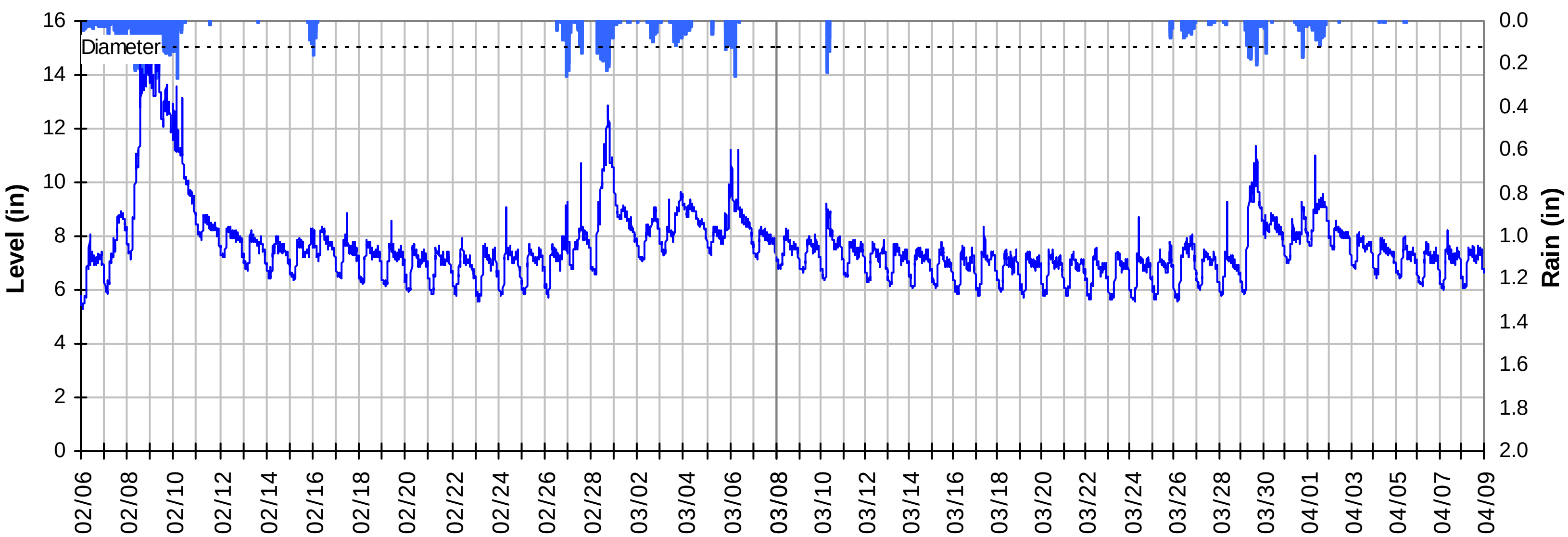


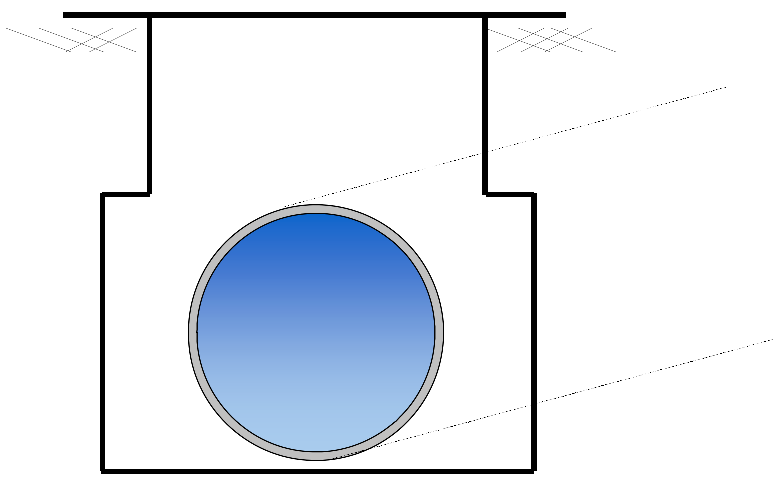
Baseline Flow:
0.146 *mgd*



SITE 5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 15 *inches*

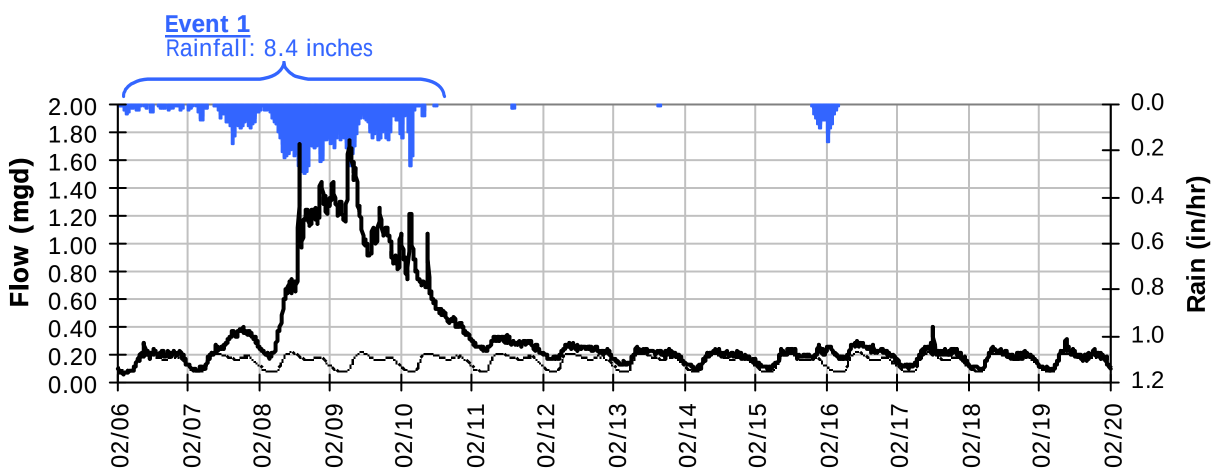
Peak Measured Level: 15.2 *inches*

Peak d/D Ratio: 1.01

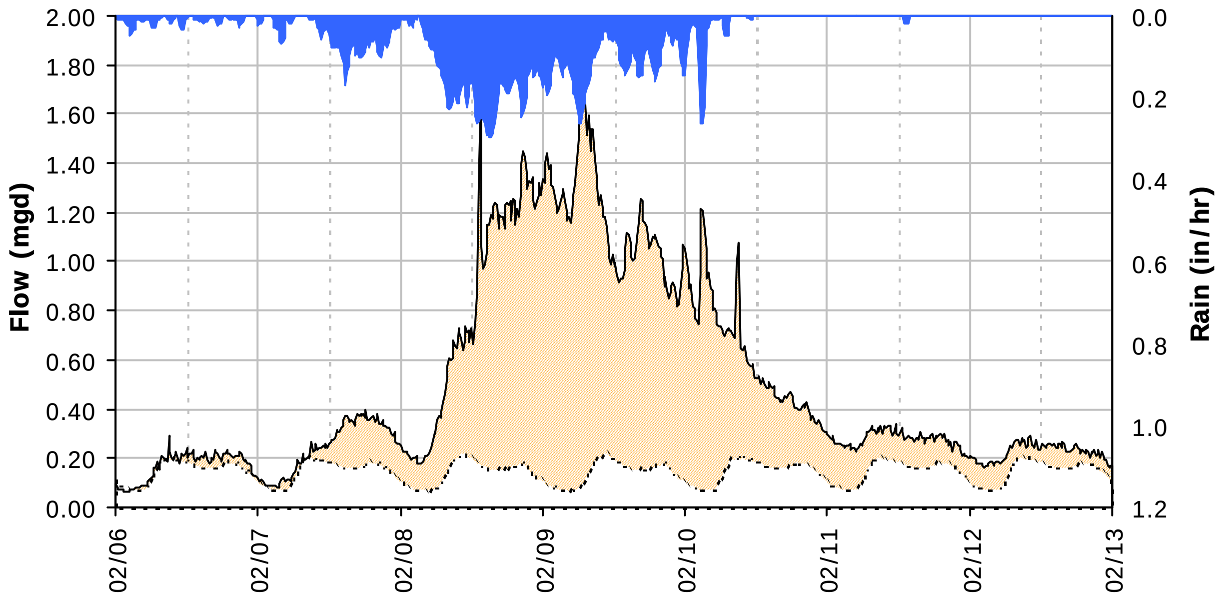
Surcharged 0.2 inches over crown

SITE 5
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

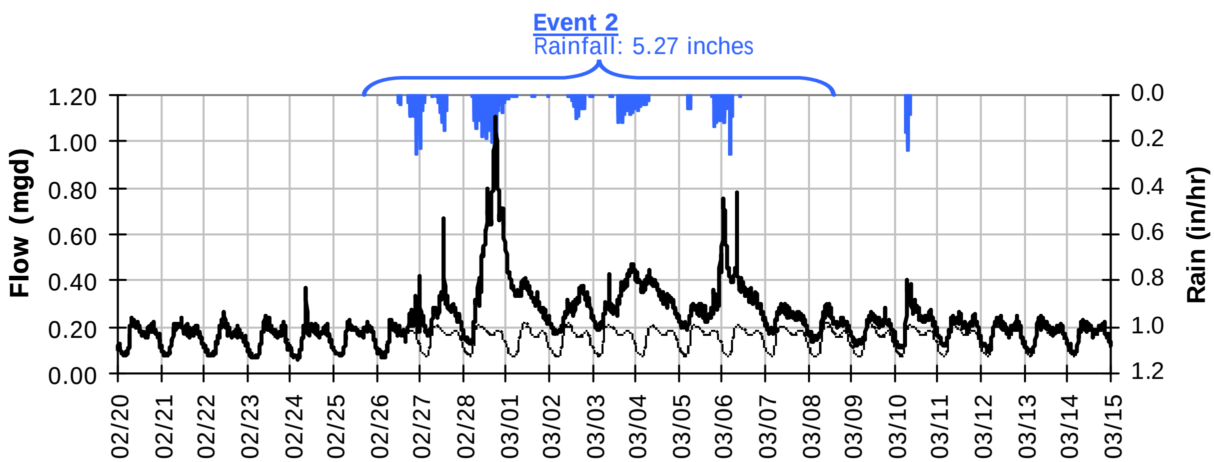


Storm Event I/I Analysis (Rain = 8.40 inches)

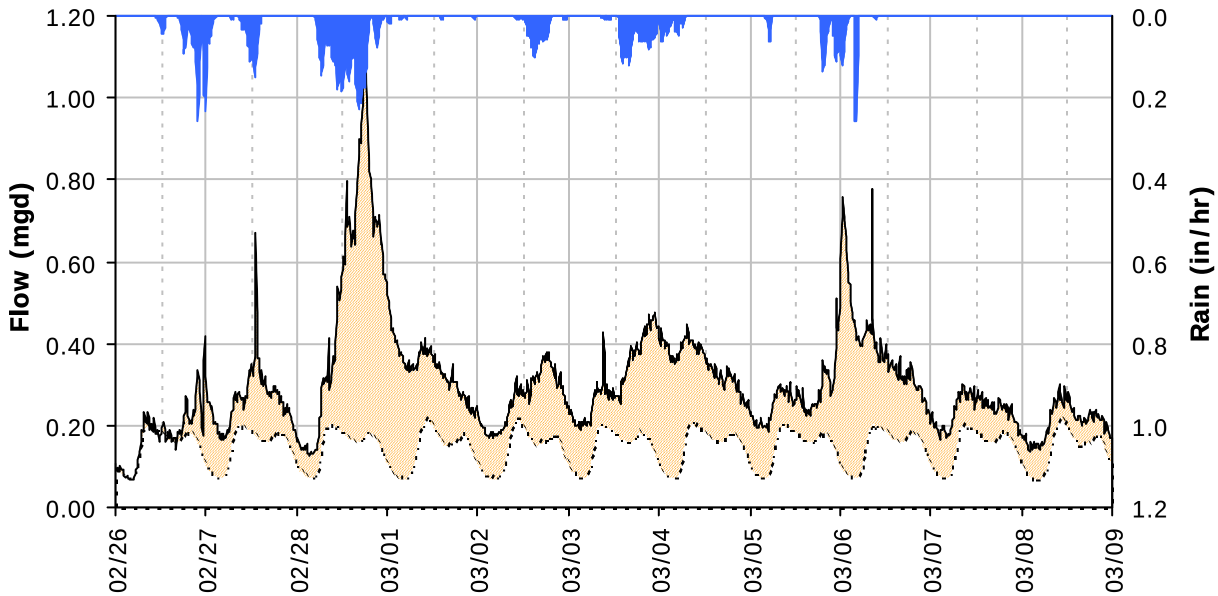
Capacity		**Inflow / Infiltration**	
Peak Flow:	1.74 *mgd*	**Peak I/I Rate:**	1.66 *mgd*
PF:	11.97	**Total I/I:**	2,478,000 *gallons*

SITE 5
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 2 Detail Graph

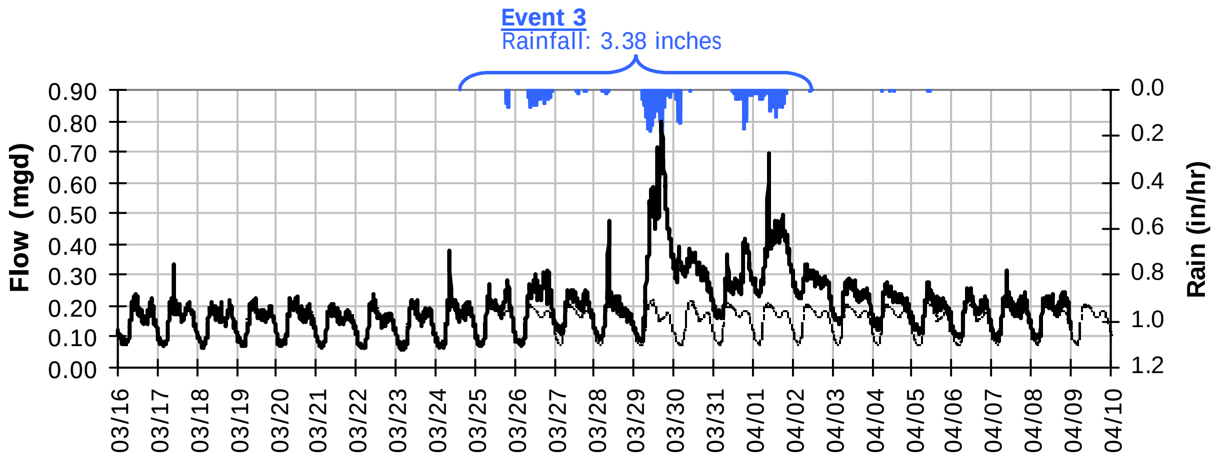


Storm Event I/I Analysis (Rain = 5.27 inches)

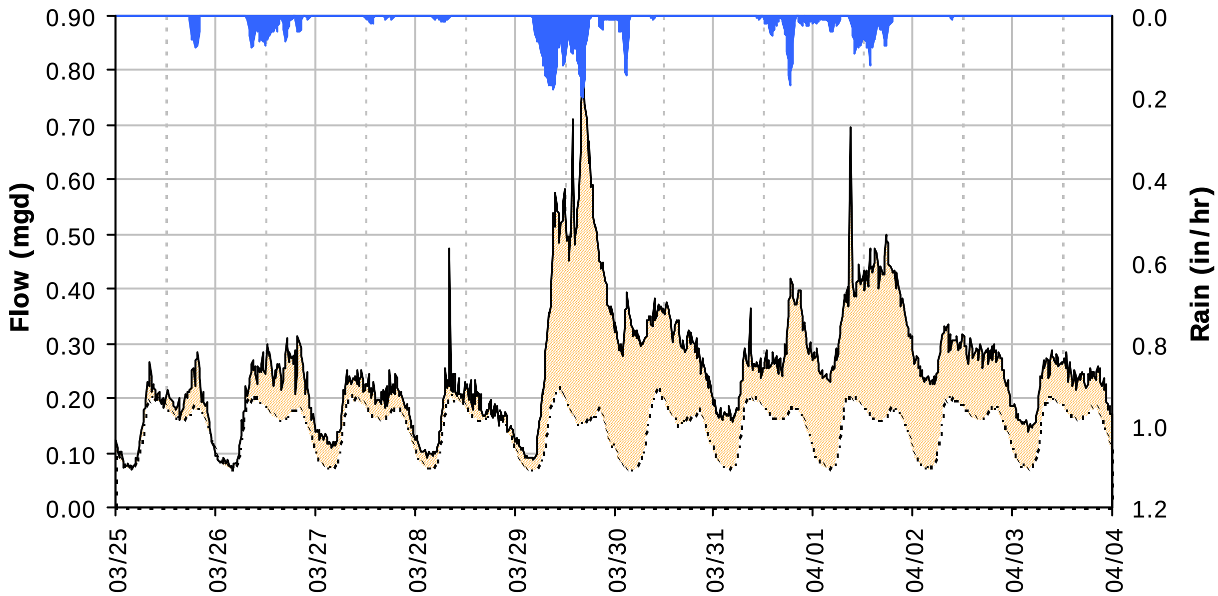
Capacity		**Inflow / Infiltration**	
Peak Flow:	1.10 *mgd*	**Peak I/I Rate:**	0.93 *mgd*
PF:	7.57	**Total I/I:**	1,709,000 *gallons*

SITE 5
I/I Summary: Event 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



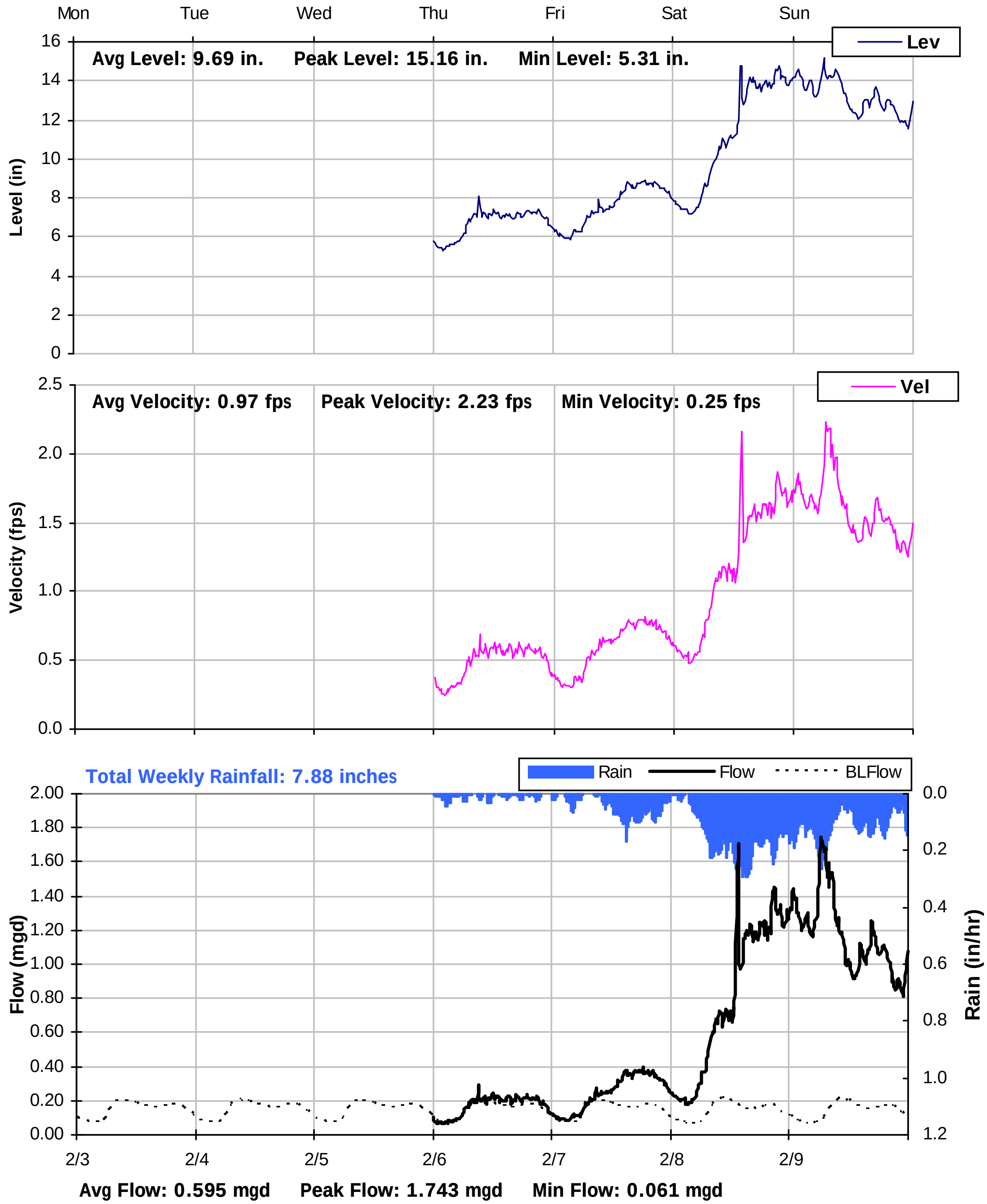
Event 3 Detail Graph



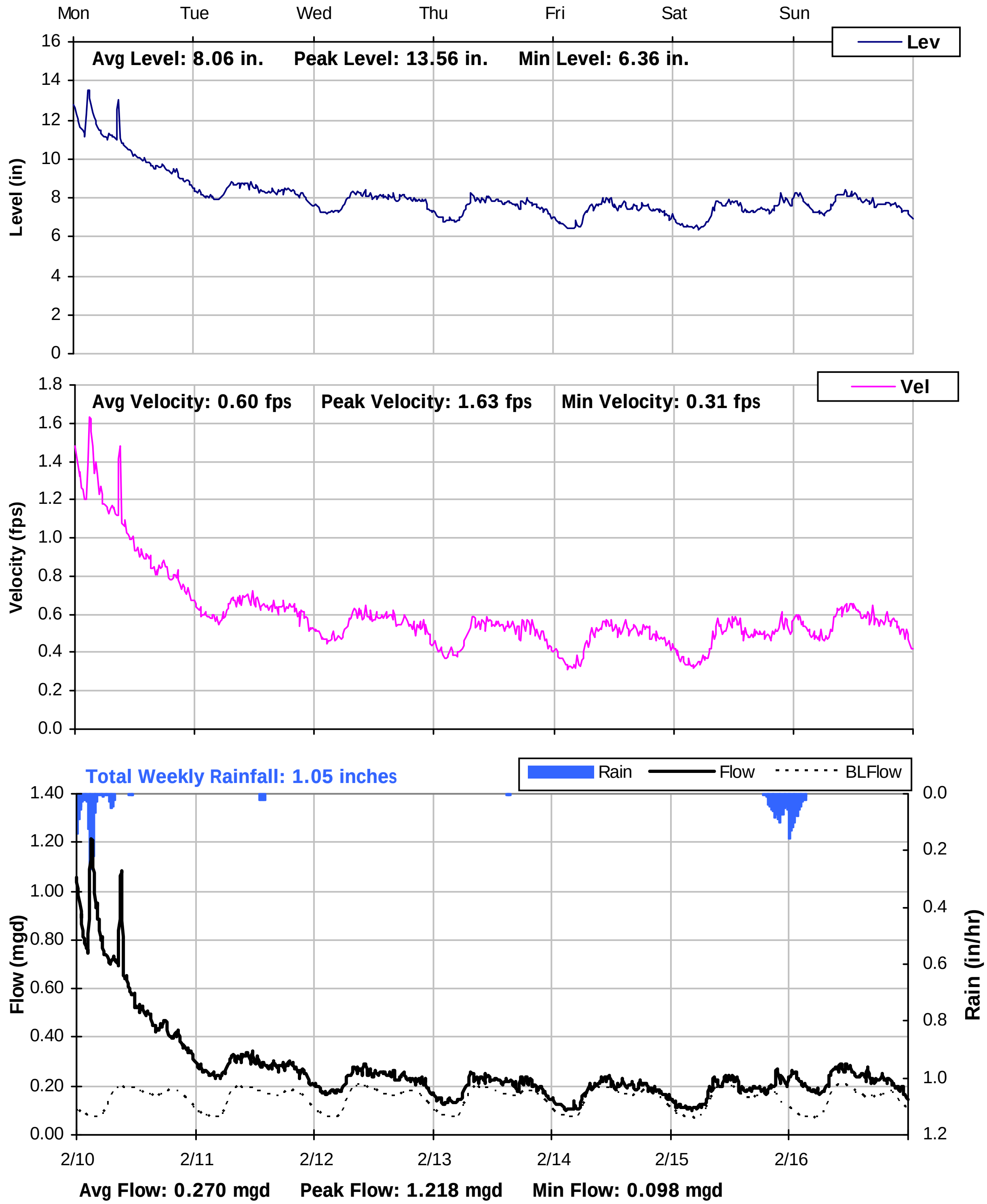
Storm Event I/I Analysis (Rain = 3.38 inches)

Capacity		**Inflow / Infiltration**	
Peak Flow:	0.80 *mgd*	**Peak I/I Rate:**	0.64 *mgd*
PF:	5.47	**Total I/I:**	1,102,000 *gallons*

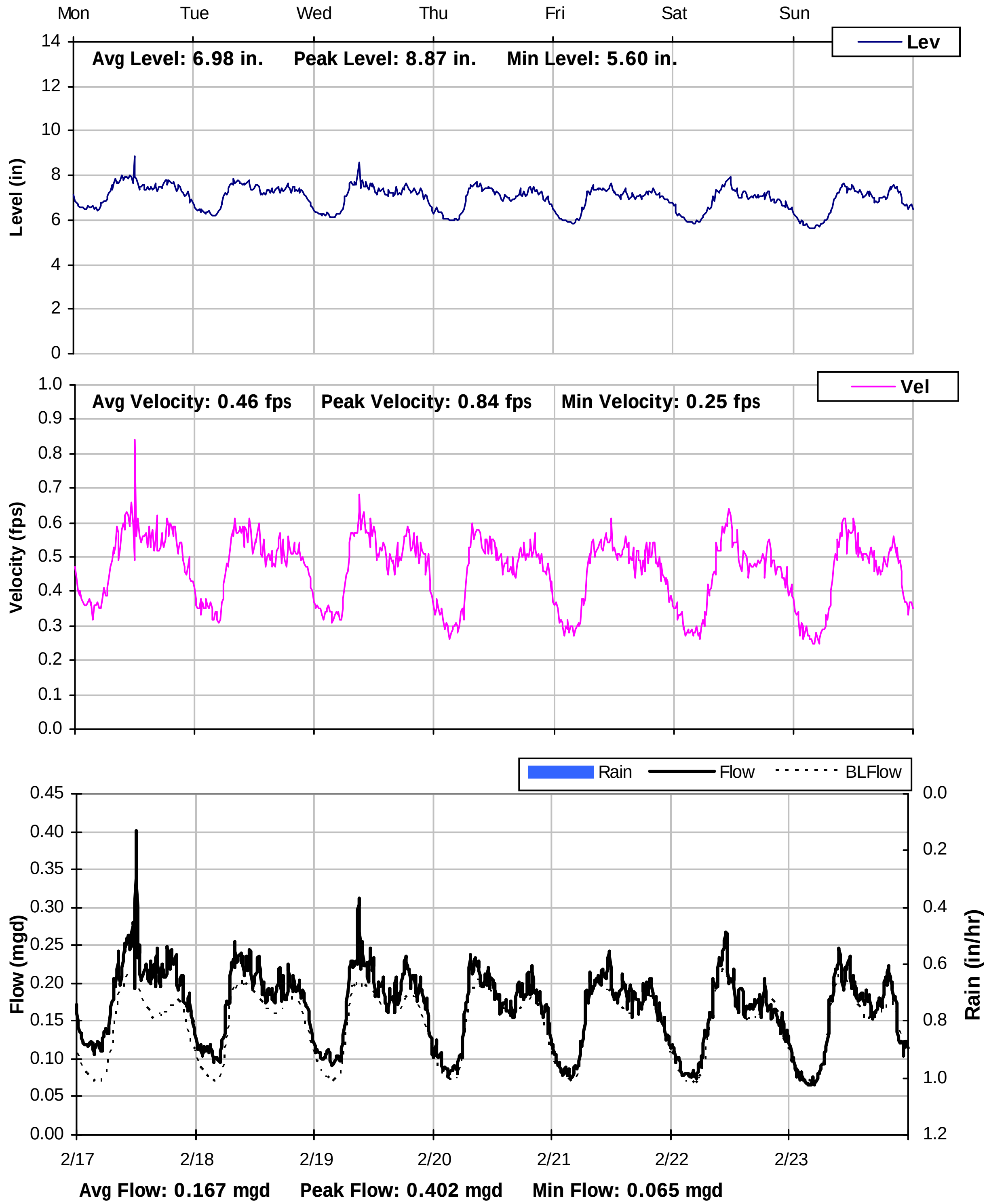
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014



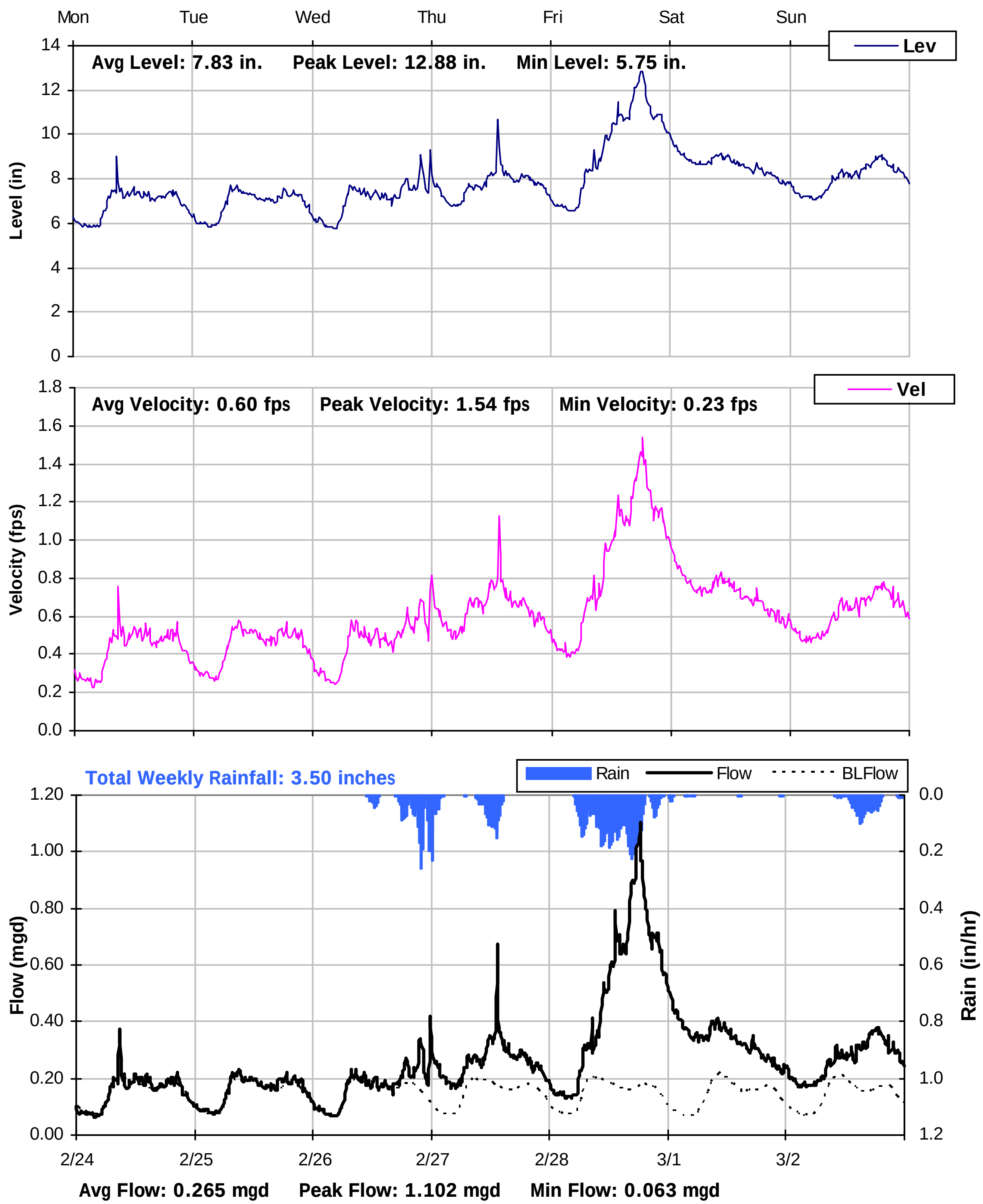
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014



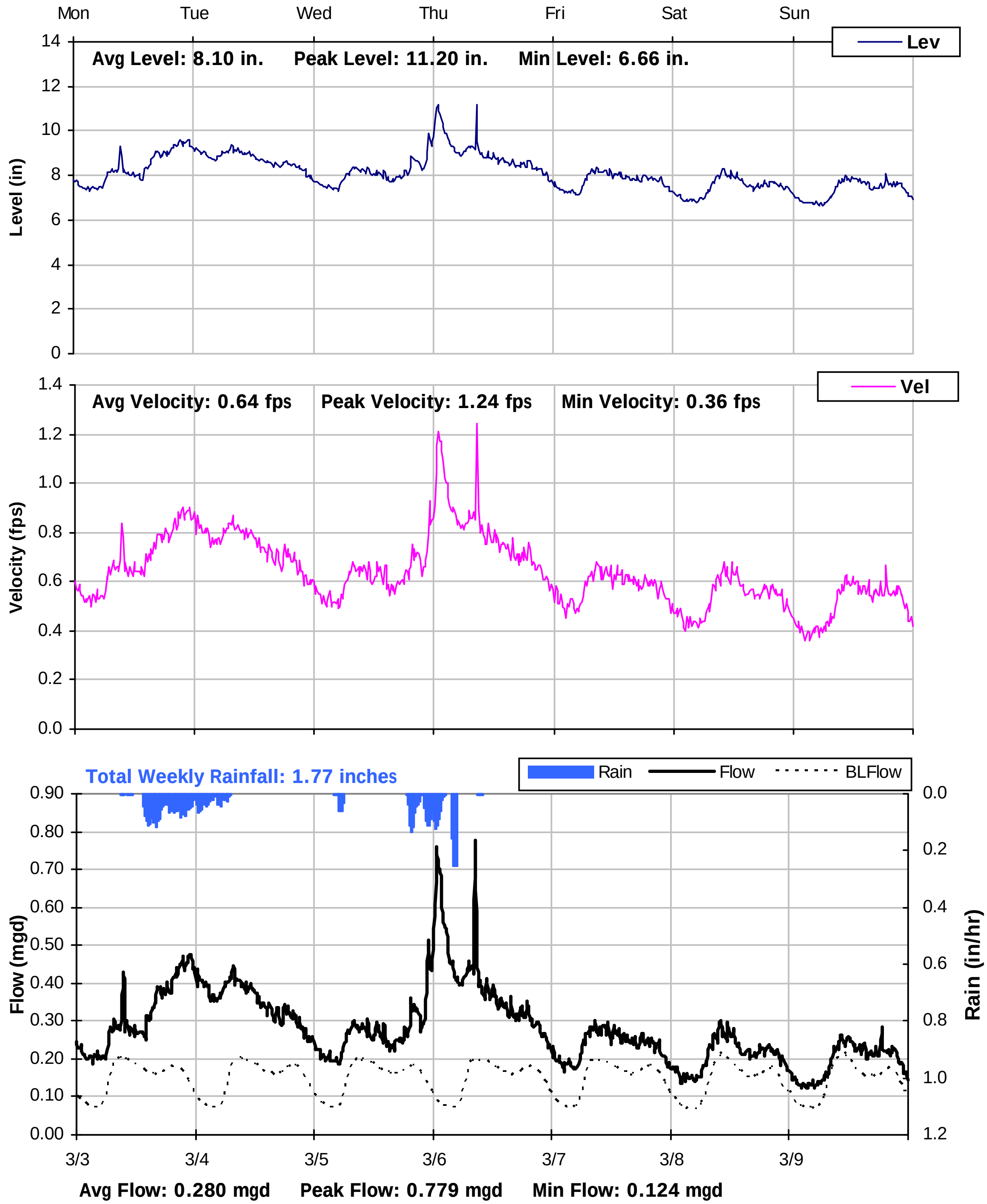
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014



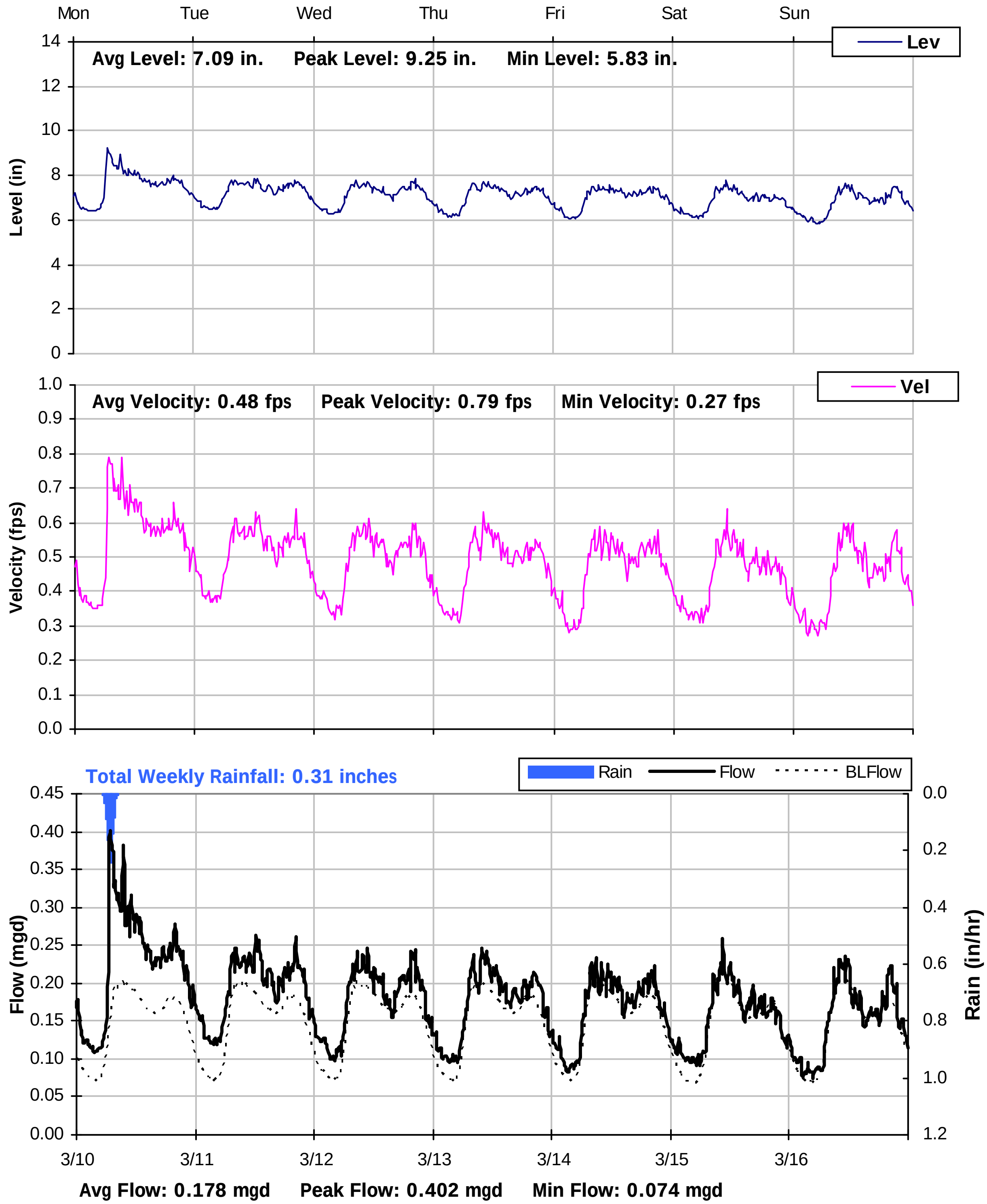
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



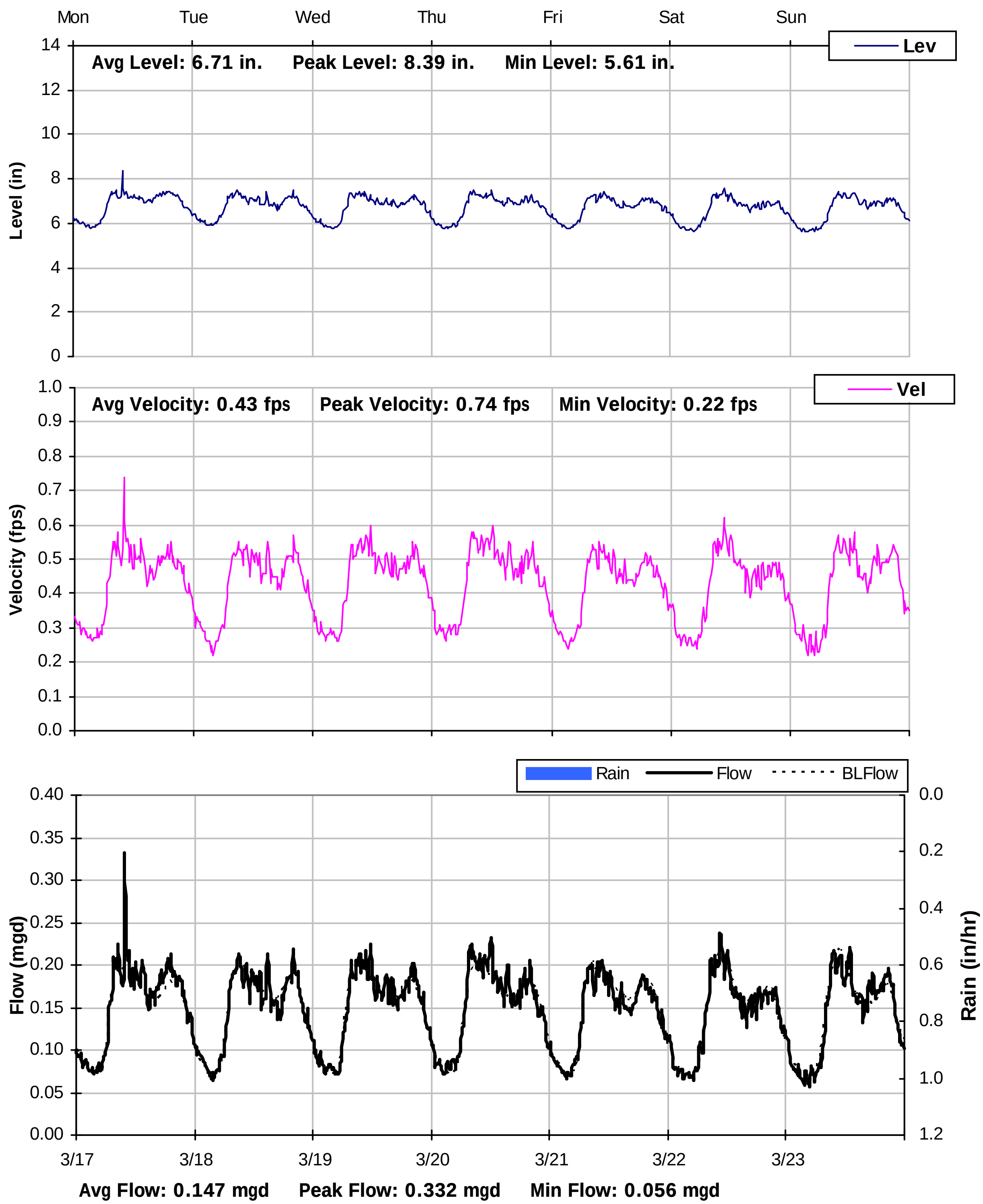
SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



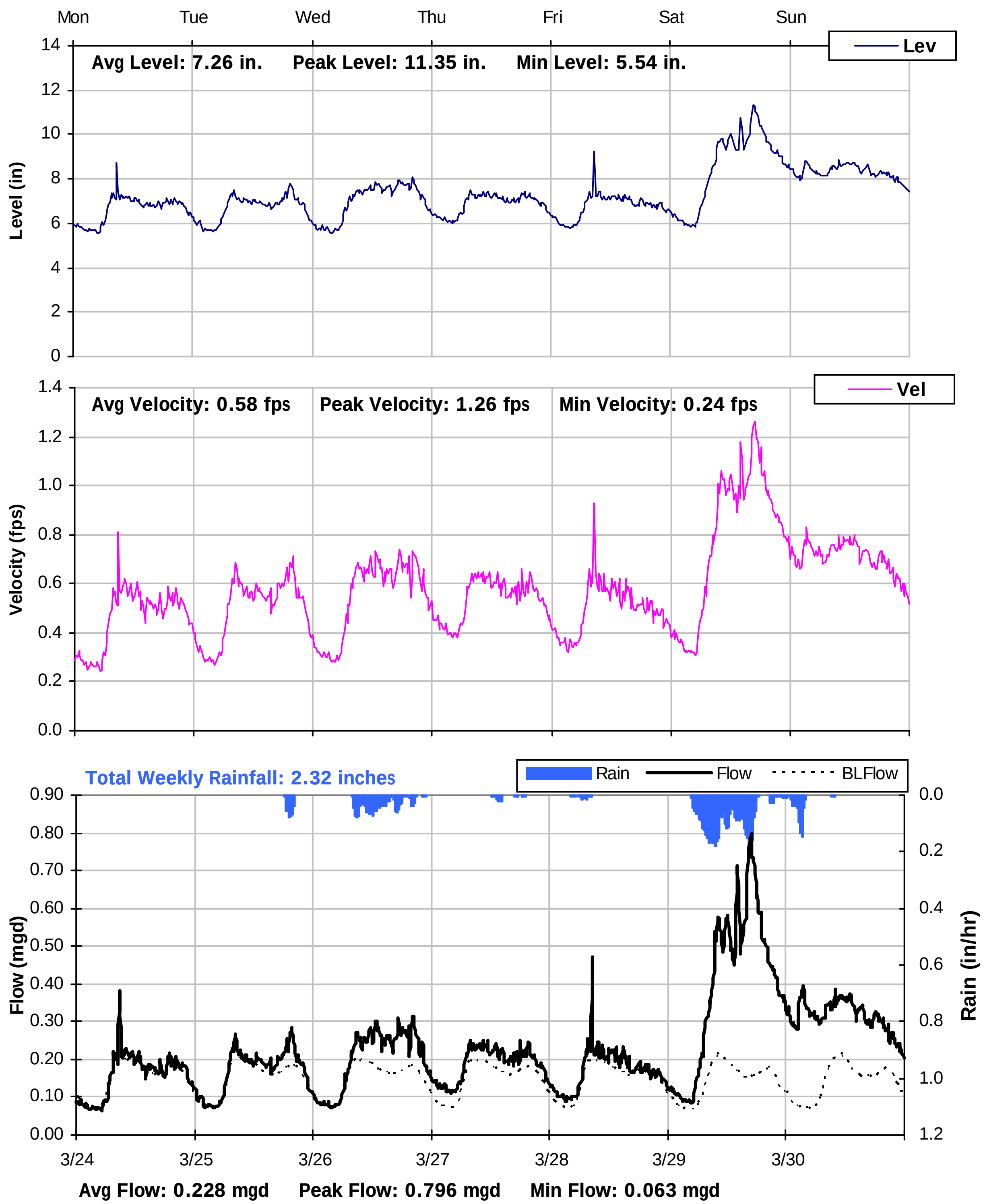
SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014



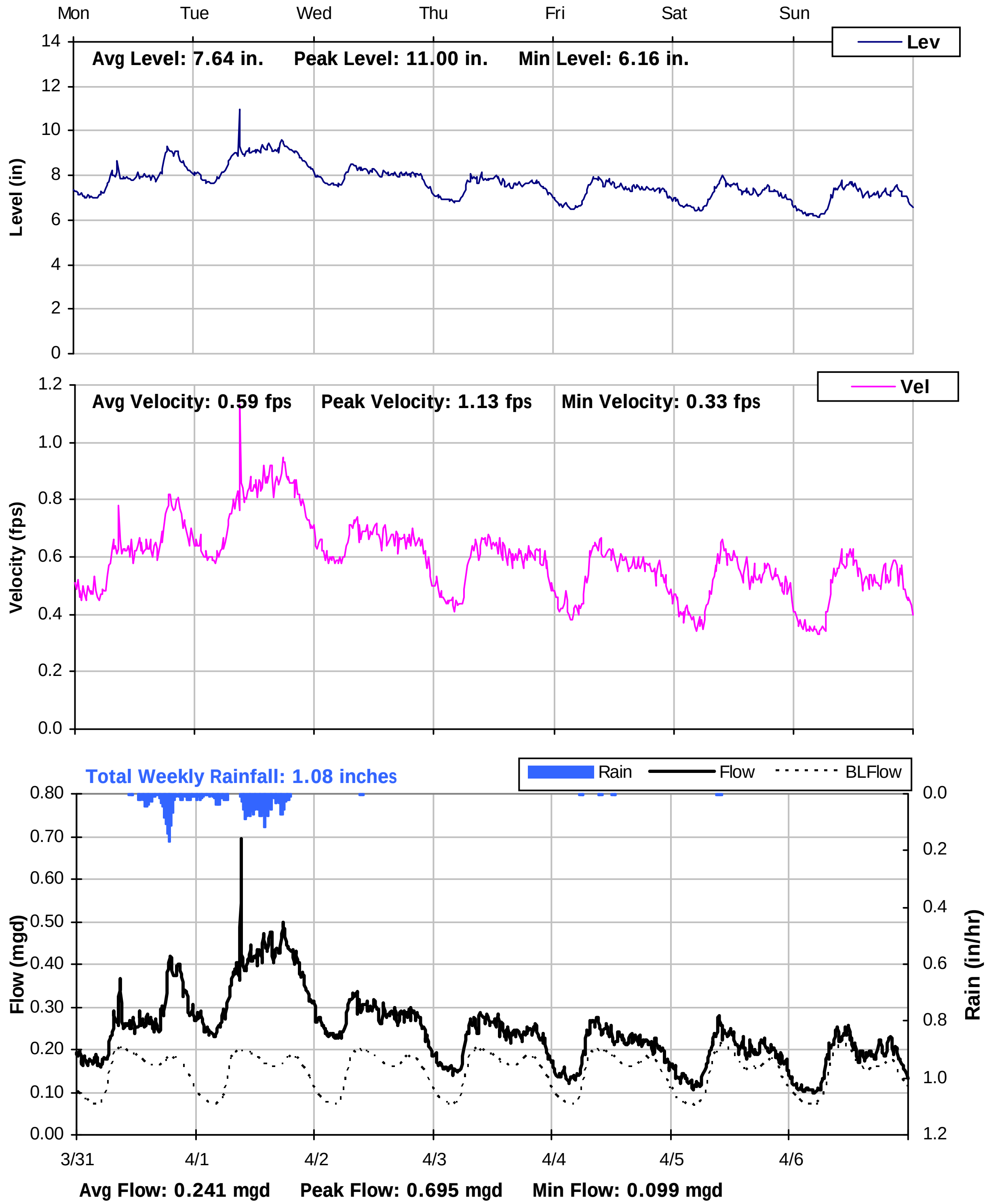
SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



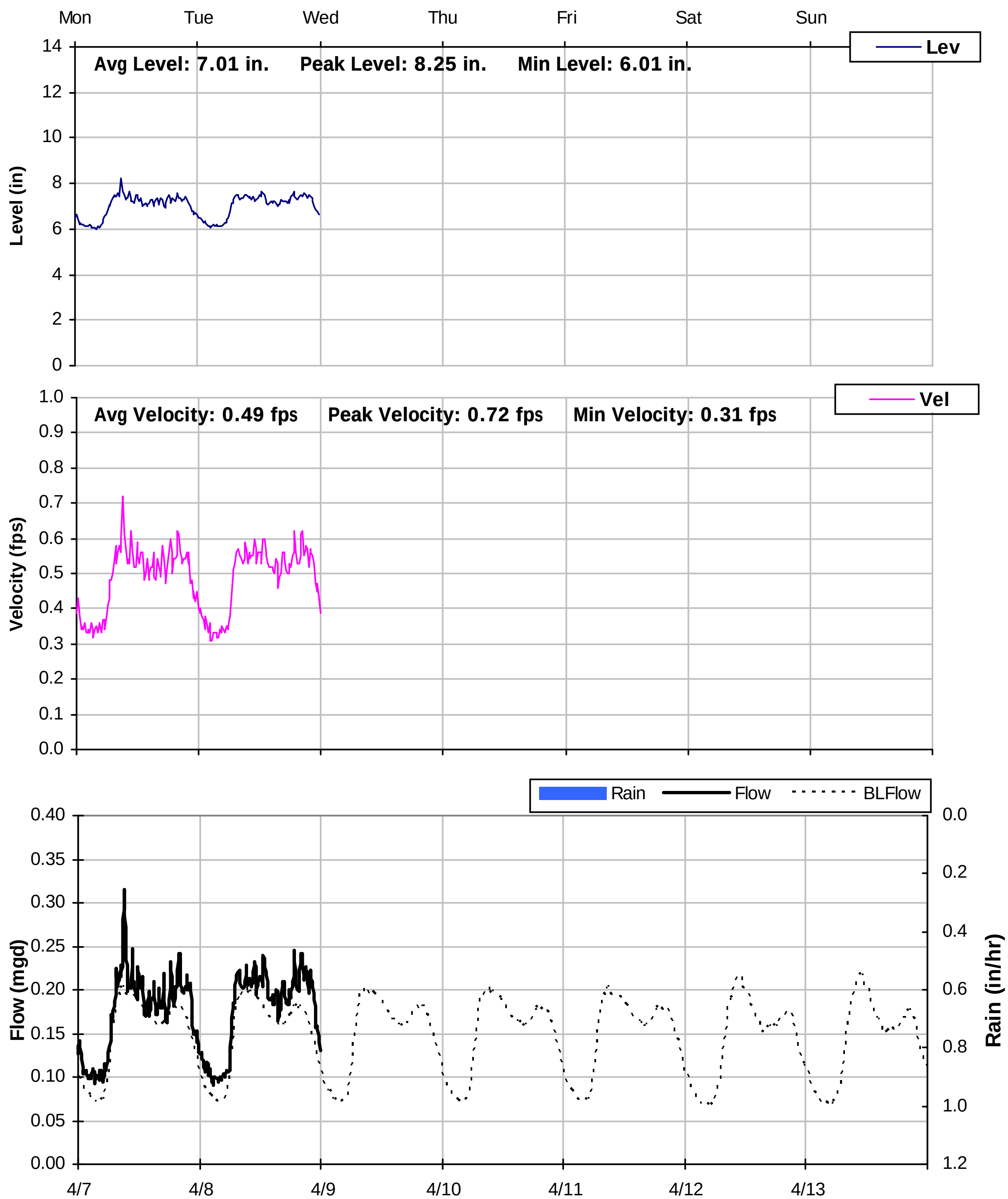
SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014



SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/31/2014 to 4/7/2014



SITE 5
Weekly Level, Velocity and Flow Hydrographs
4/7/2014 to 4/14/2014



Avg Flow: 0.176 mgd Peak Flow: 0.315 mgd Min Flow: 0.090 mgd

City of Grass Valley

Sanitary Sewer Flow Monitoring

Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 6

Location: South Auburn Street North of Neal Street

Data Summary Report

Vicinity Map: Site 6

SITE 6

Site Information

Location: South Auburn Street North of Neal Street

Coordinates: 121.0619° W, 39.2173° N

Rim Elevation: 2400 feet

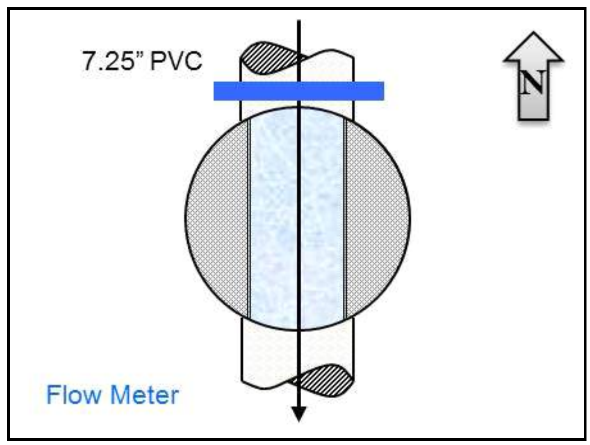
Pipe Diameter: 7.25 inches

Baseline Flow: 0.152 mgd

Peak Measured Flow: 1.059 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

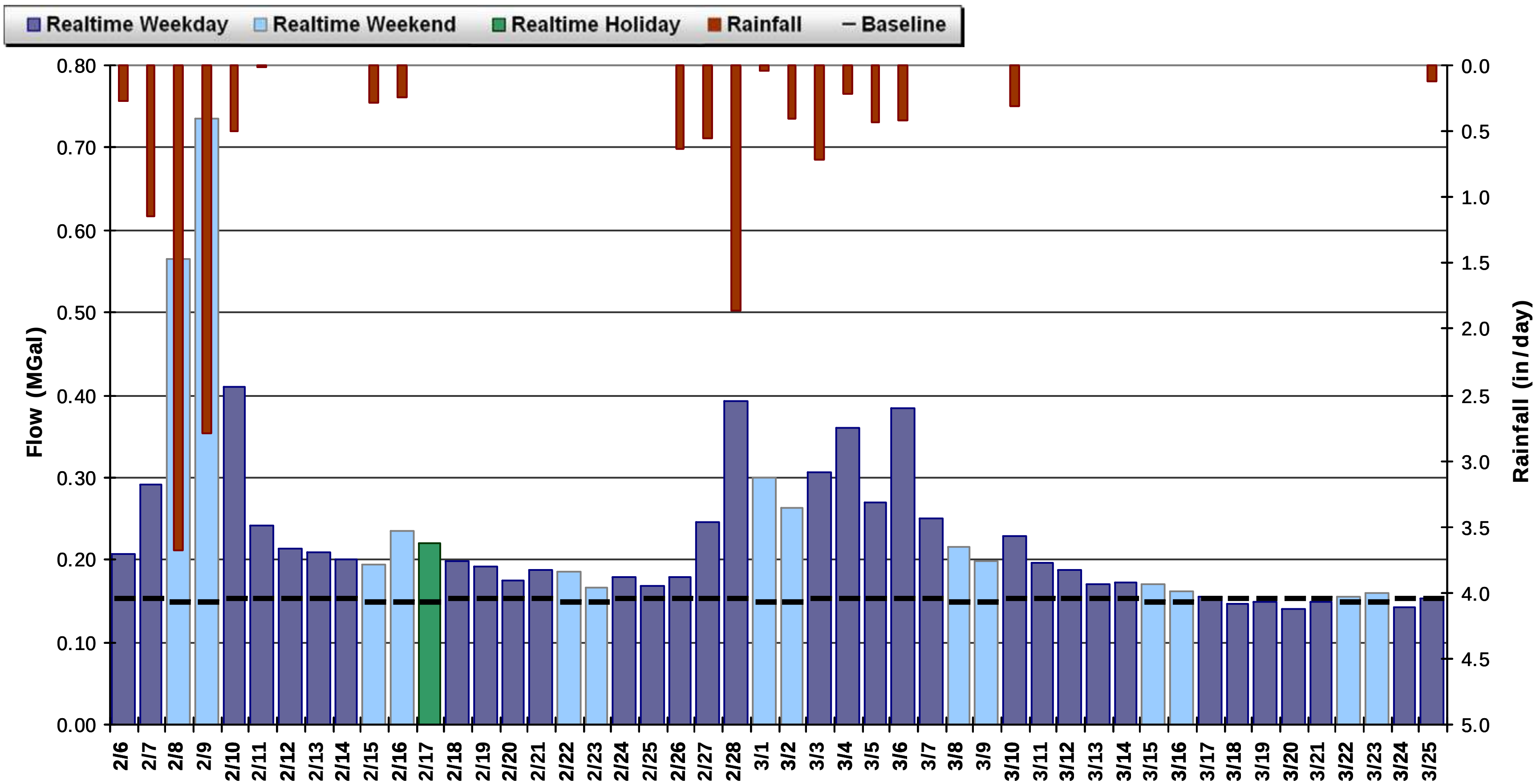
Plan View

SITE 6

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.233 MGal   Peak Daily Flow: 0.735 MGal   Min Daily Flow: 0.141 MGal

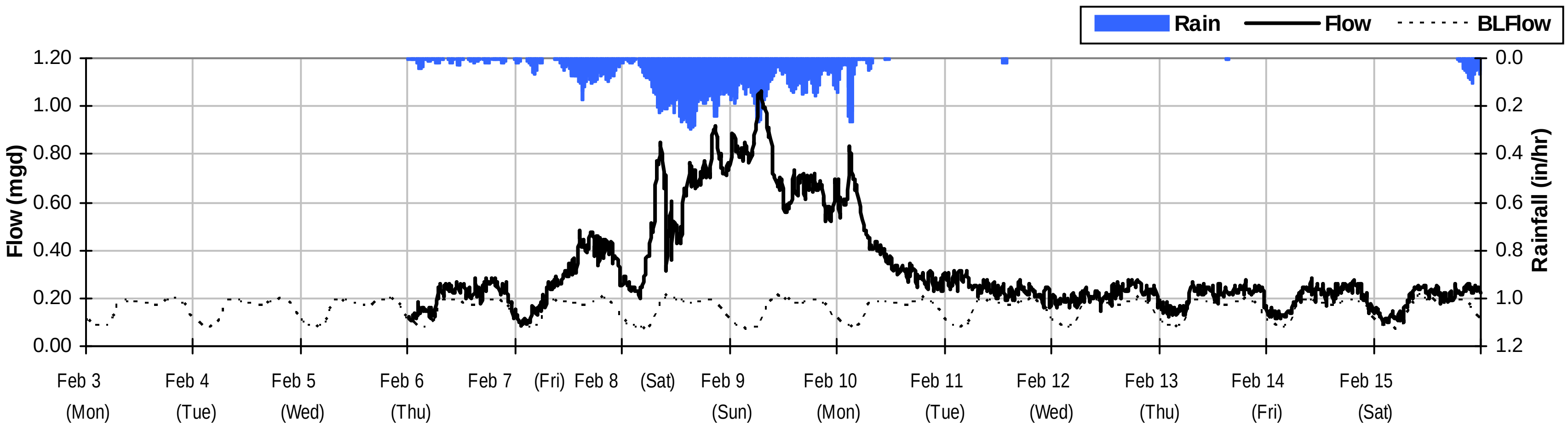
Total Period Rainfall: 14.64 inches

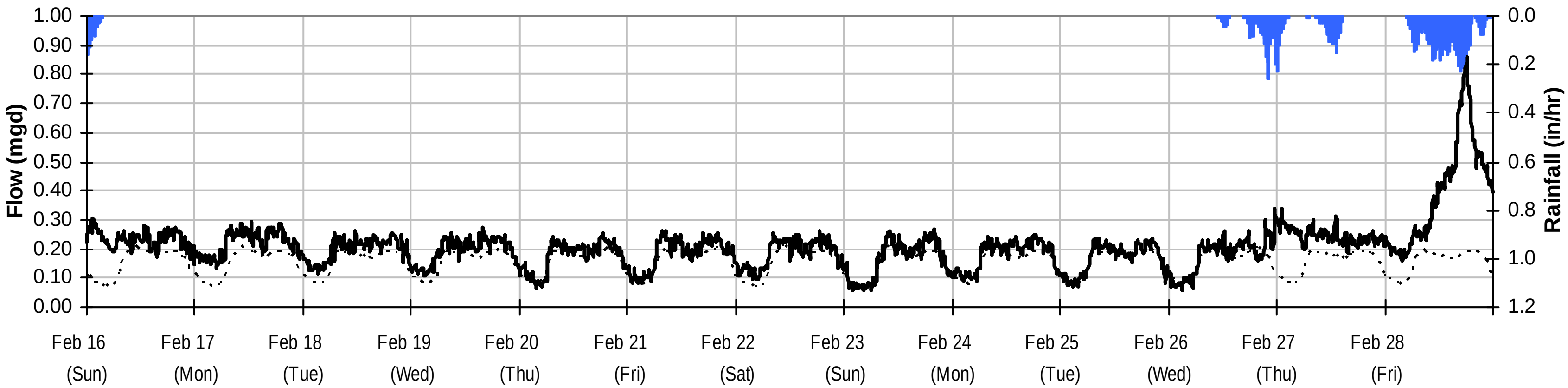


SITE 6

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.260 mgd** **Peak Flow: 1.059 mgd** **Min Flow: 0.058 mgd**

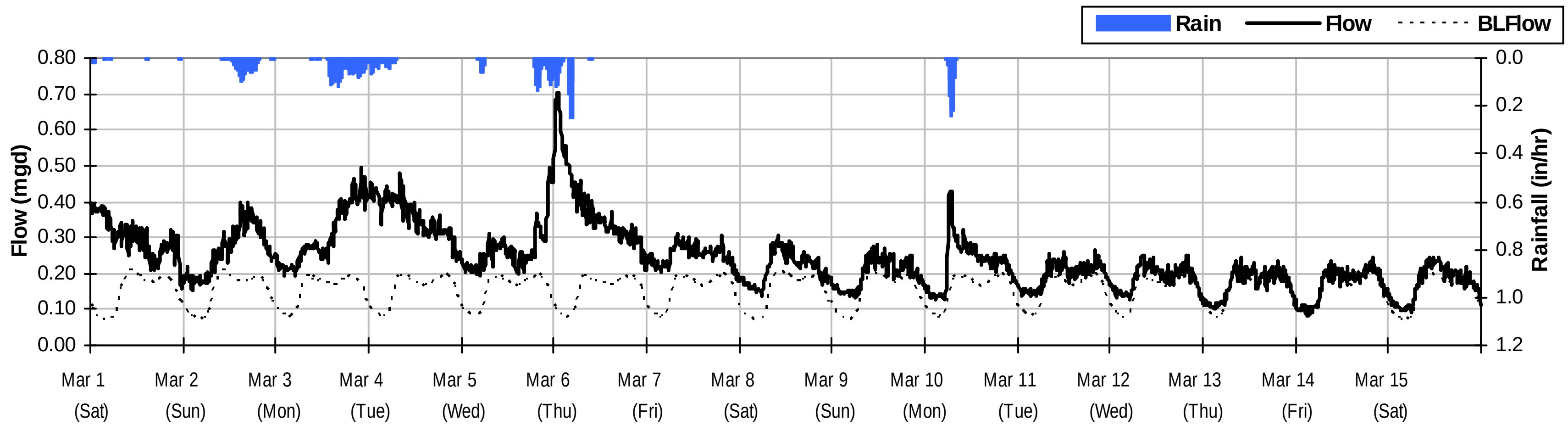


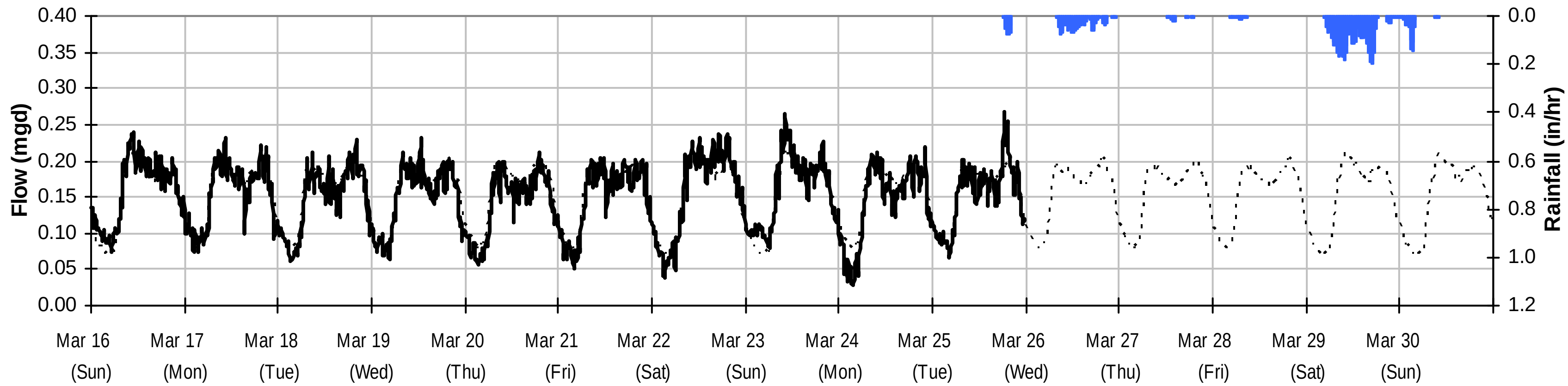


SITE 6

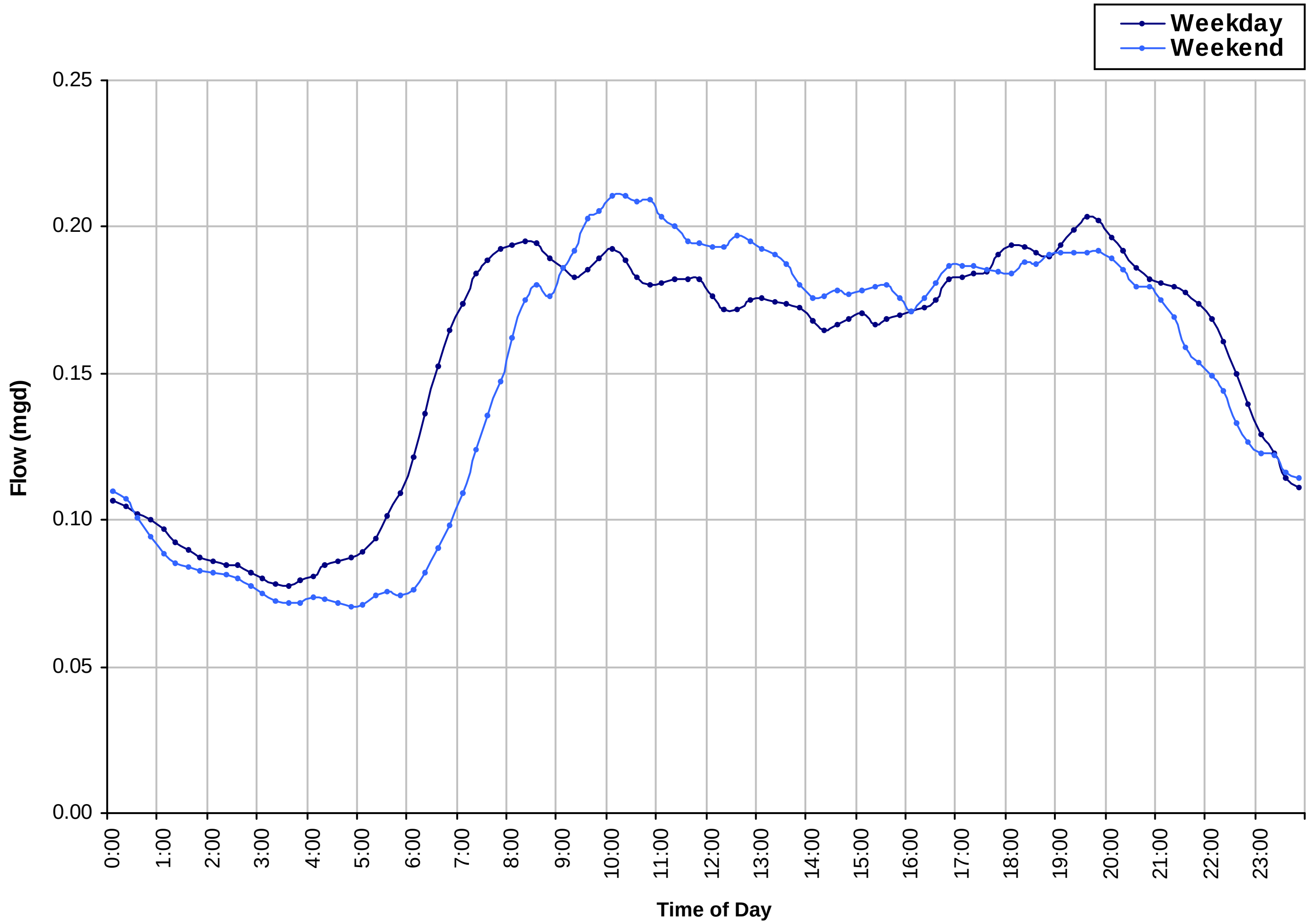
Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 4.85 inches   Avg Flow: 0.207 mgd   Peak Flow: 0.703 mgd   Min Flow: 0.029 mgd

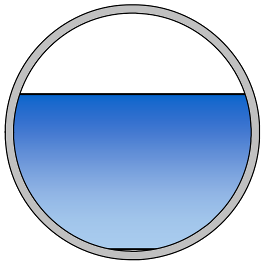




SITE 6
Baseline Flow Hydrographs

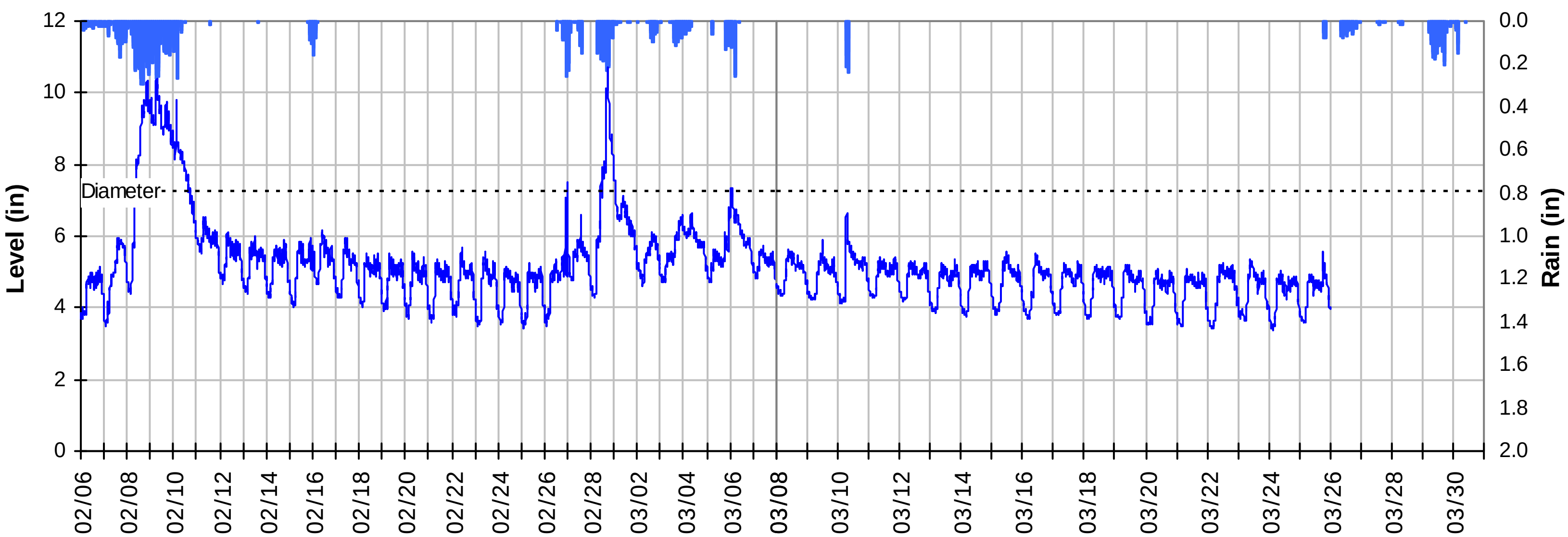


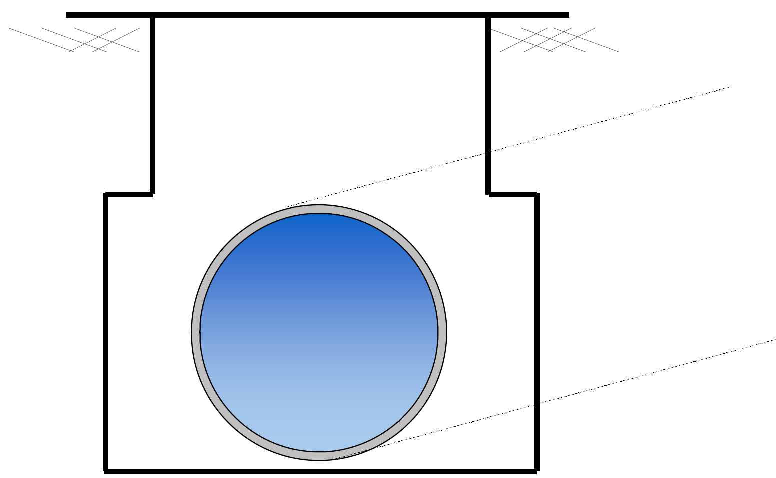
Baseline Flow:
0.152 *mgd*



SITE 6
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 7.25 *inches*

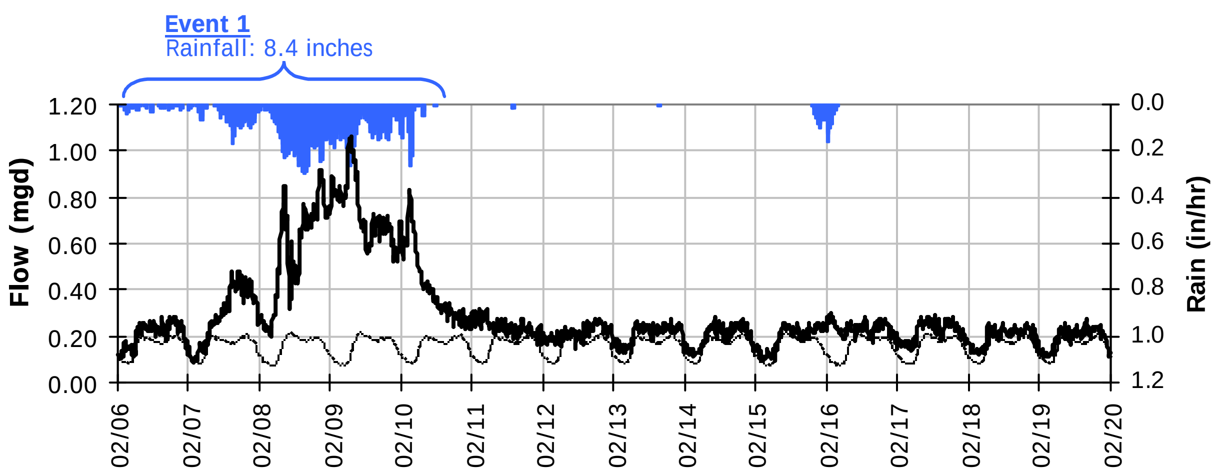
Peak Measured Level: 10.7 *inches*

Peak d/D Ratio: 1.48

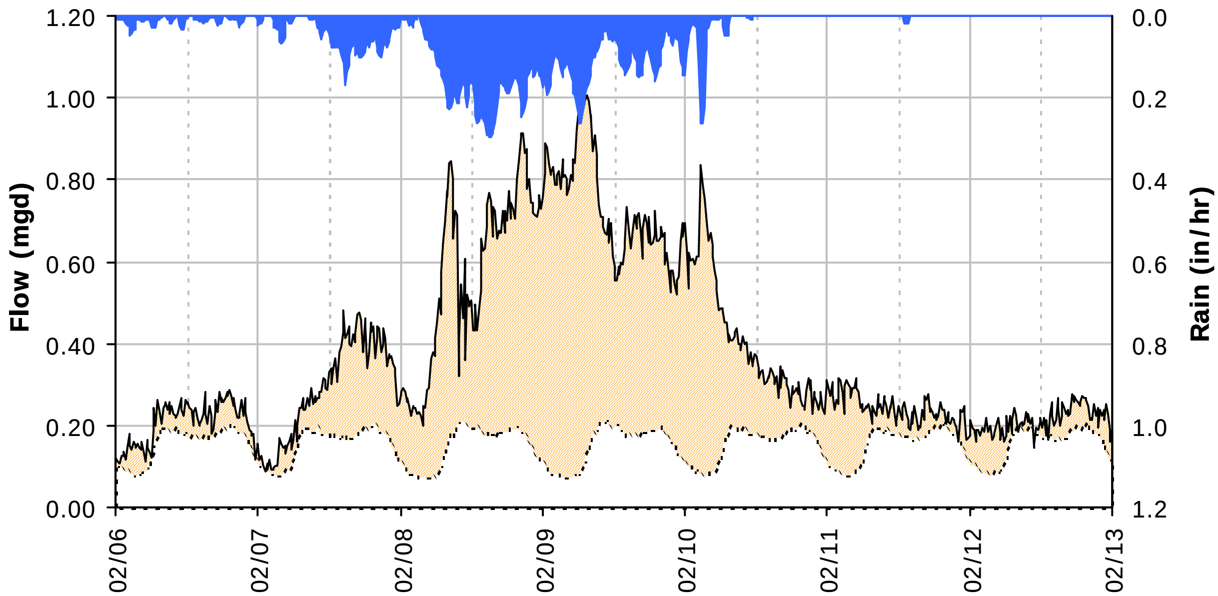
Surcharged 3.5 inches over crown

SITE 6
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

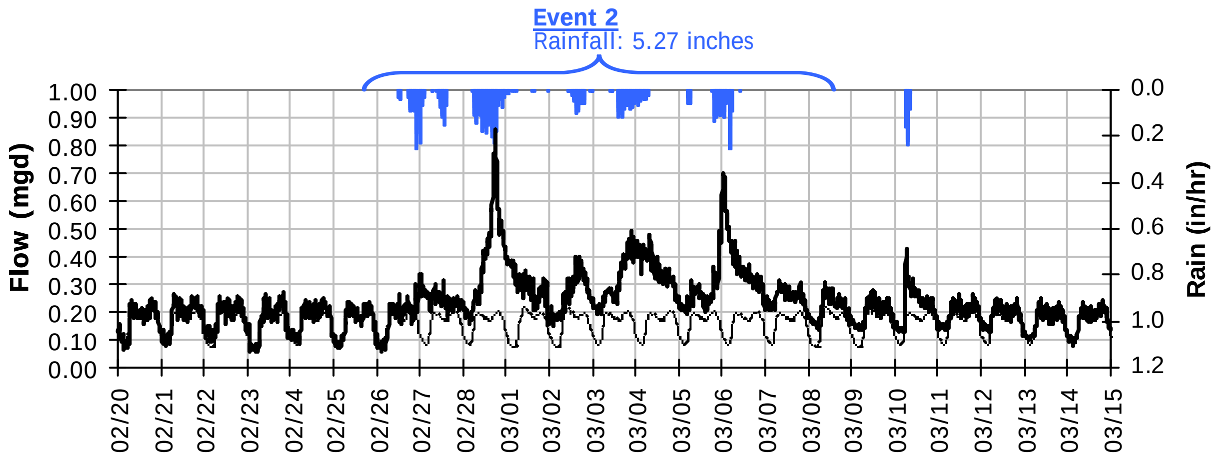


Storm Event I/I Analysis (Rain = 8.40 inches)

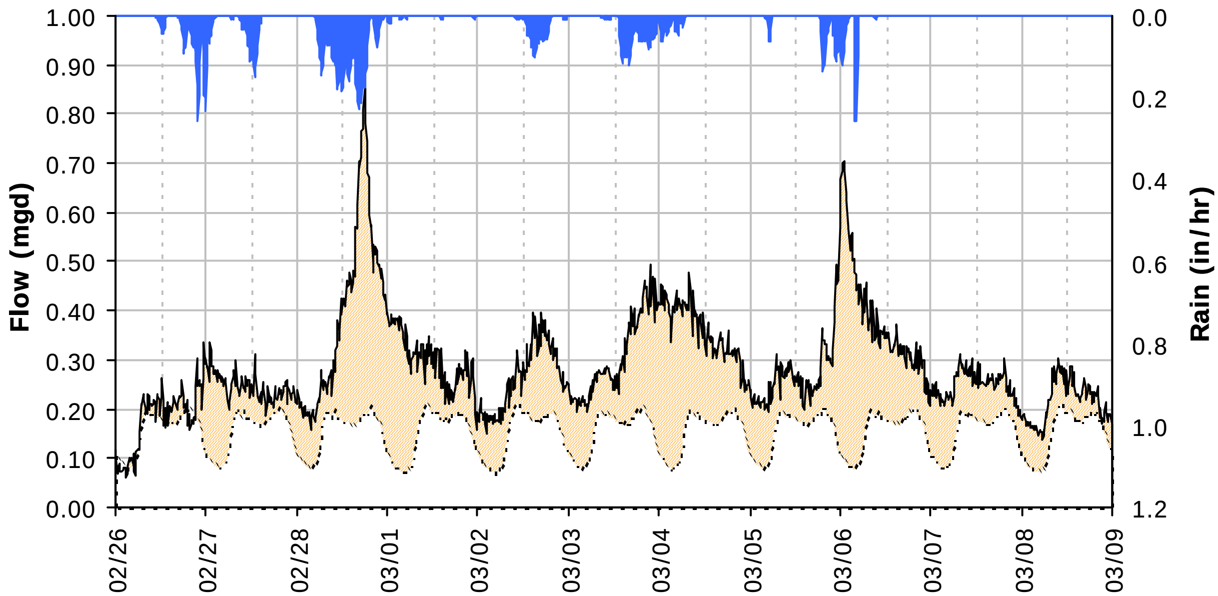
Capacity		**Inflow / Infiltration**	
Peak Flow:	1.06 *mgd*	**Peak I/I Rate:**	0.96 *mgd*
PF:	6.95	**Total I/I:**	1,597,000 *gallons*

SITE 6
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



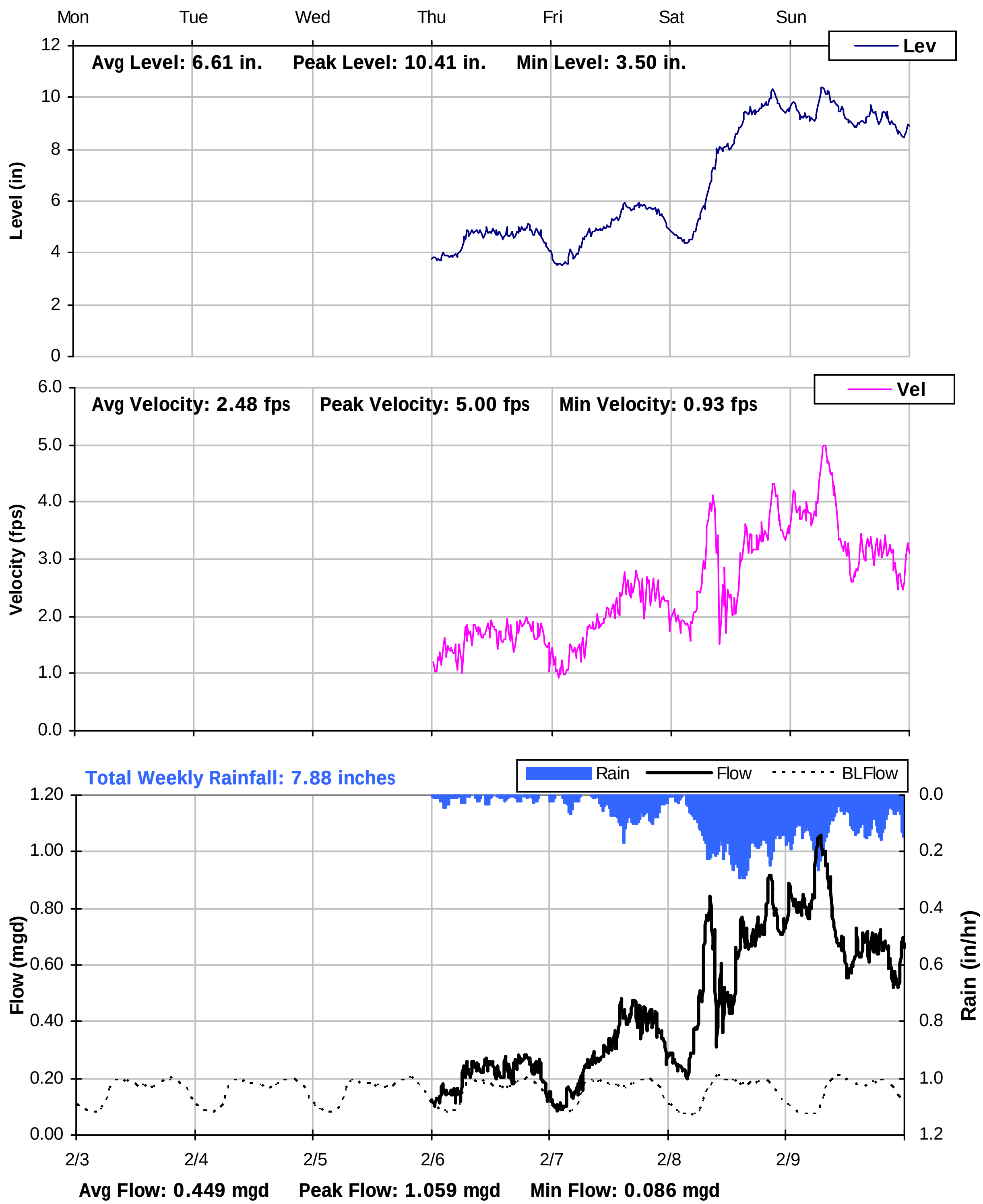
Event 2 Detail Graph



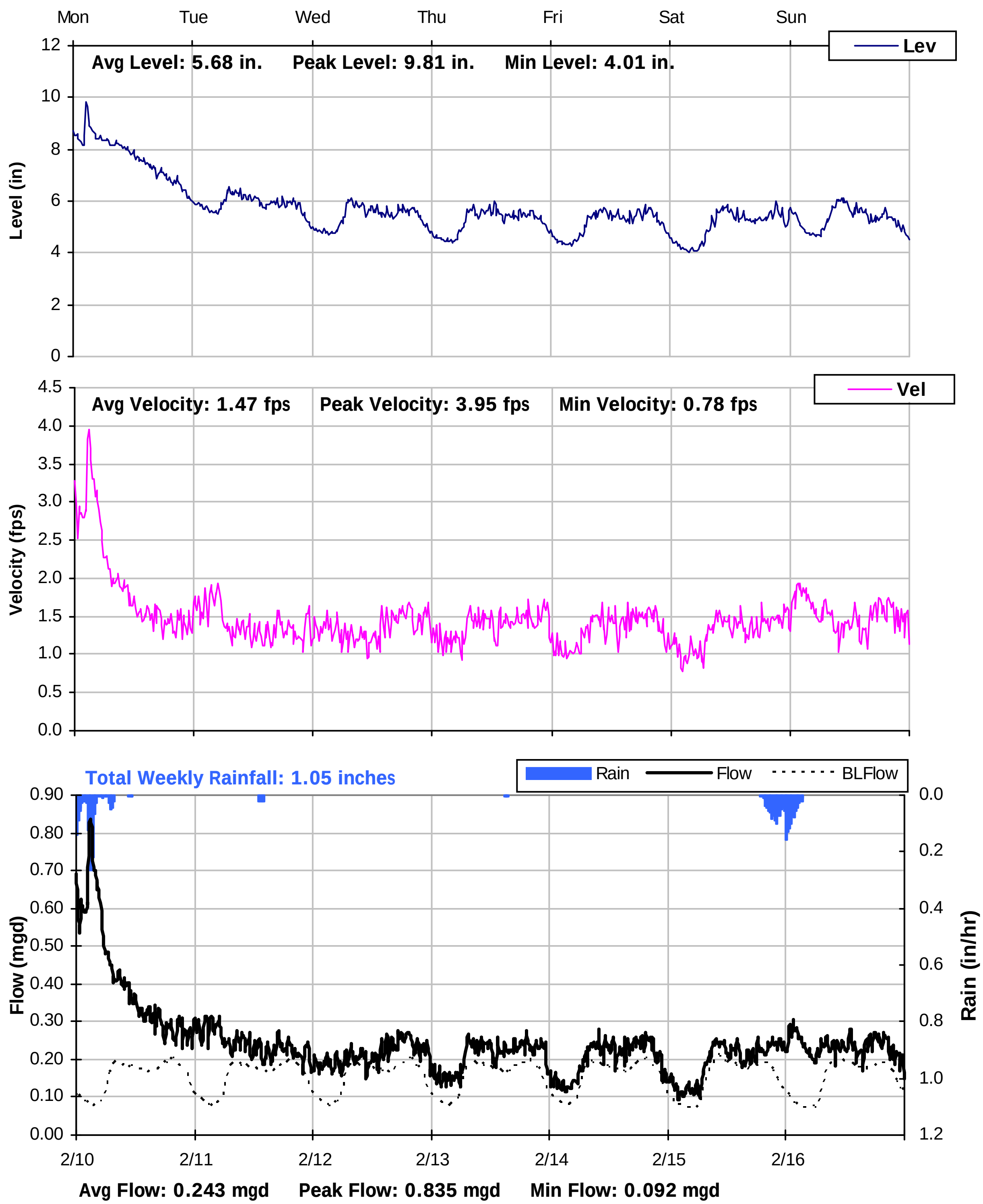
Storm Event I/I Analysis (Rain = 5.27 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.86 *mgd*	**Peak I/I Rate:**	0.67 *mgd*
PF:	5.64	**Total I/I:**	1,487,000 *gallons*

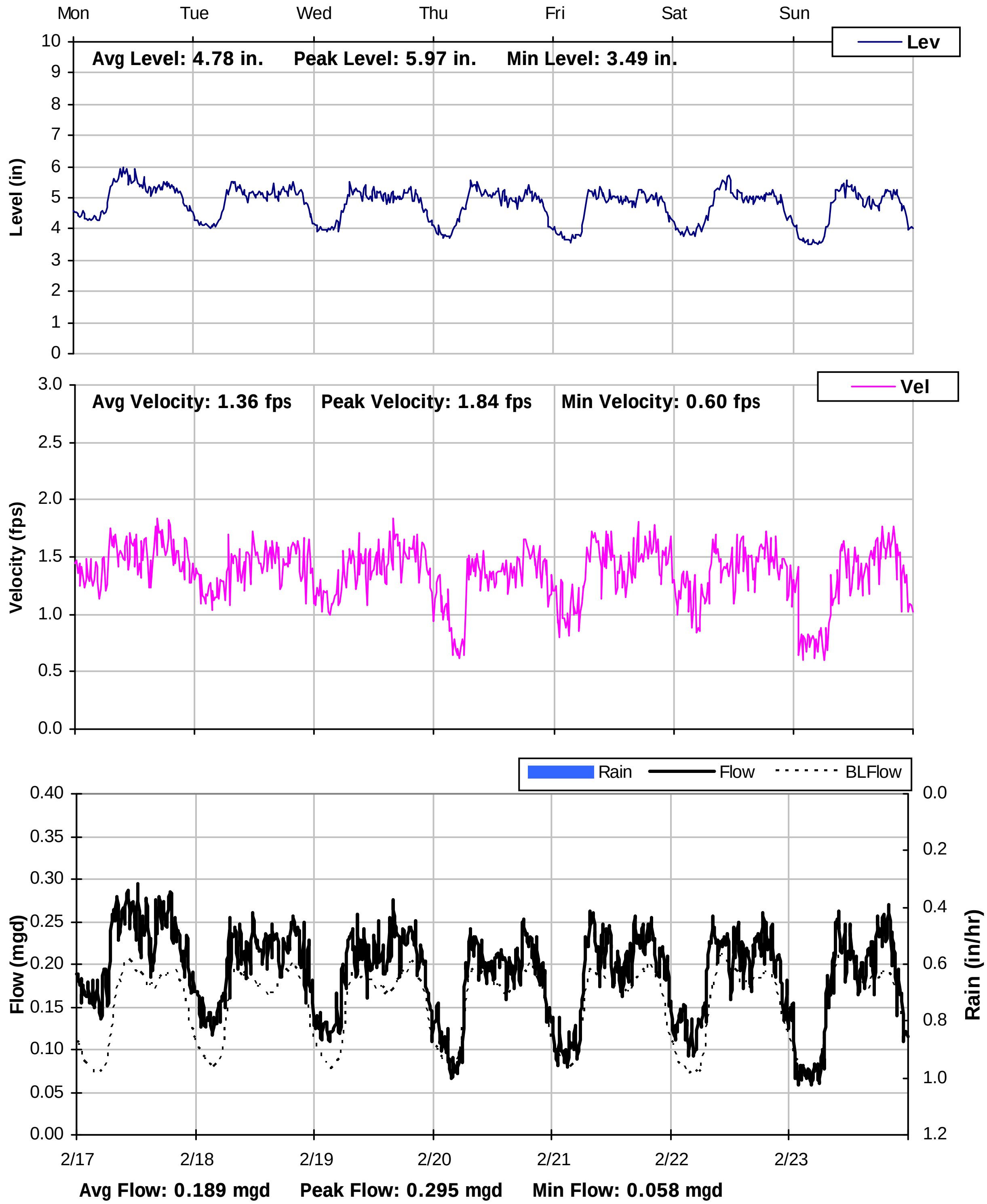
SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014



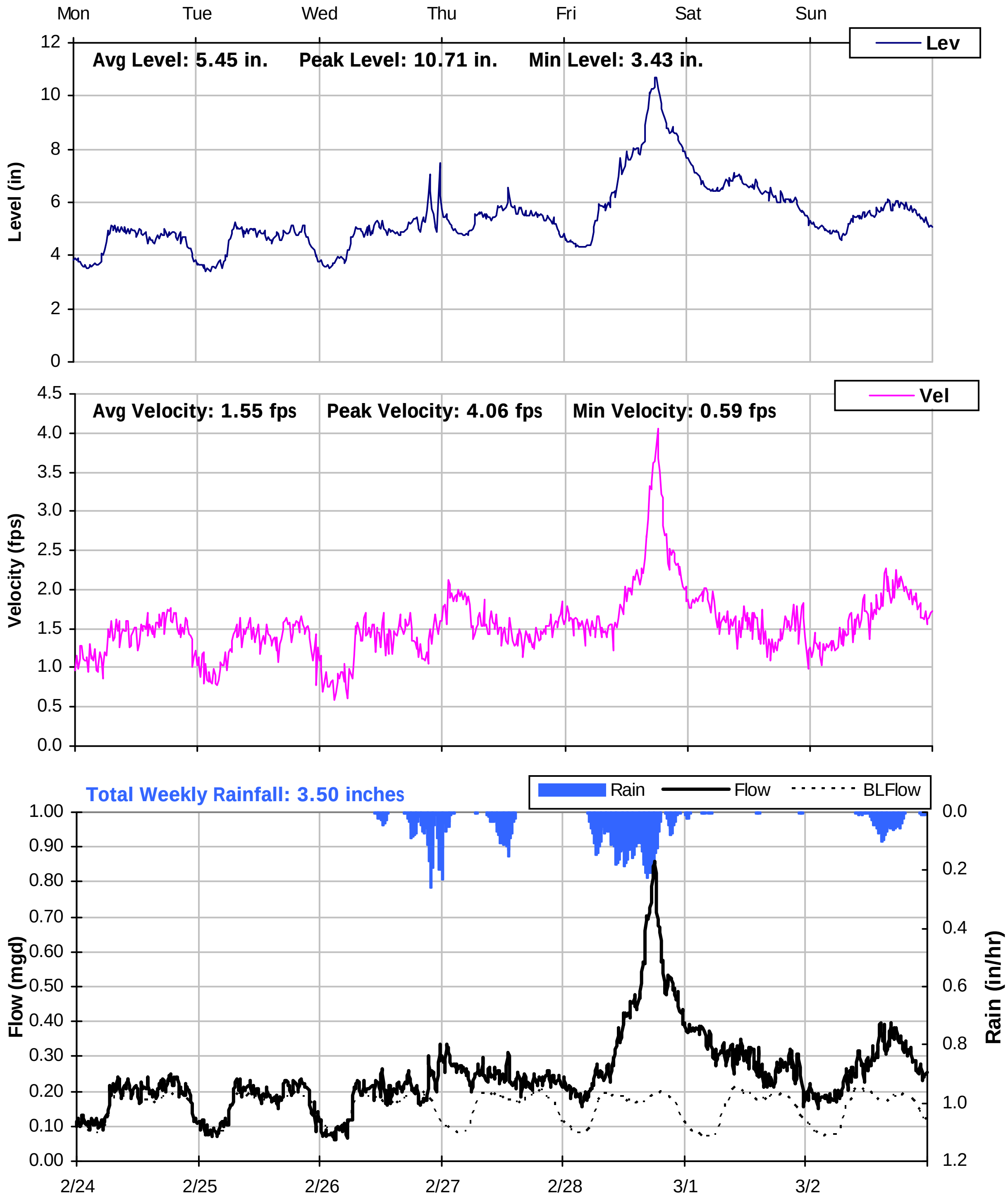
SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014



SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014



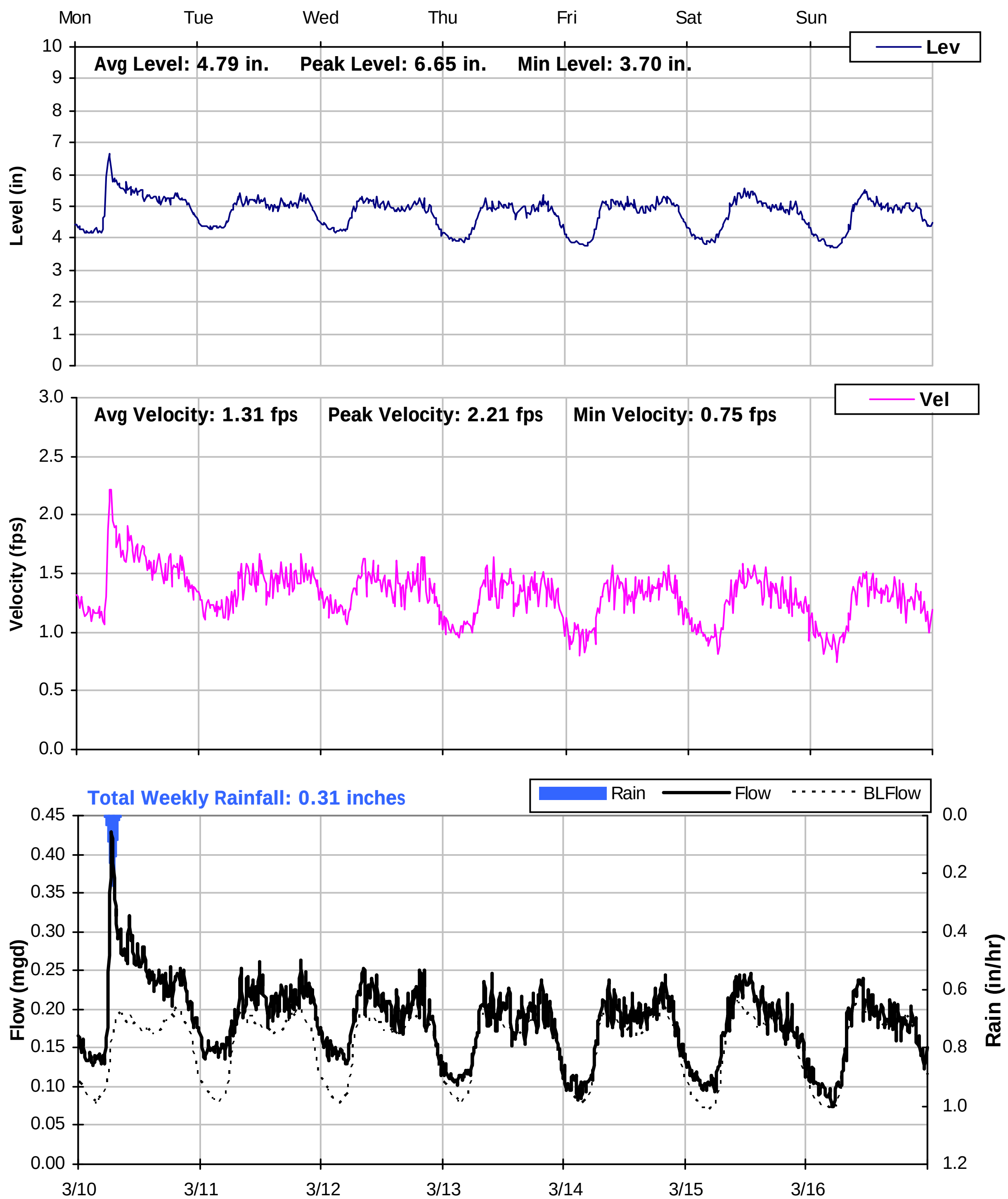
SITE 6
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



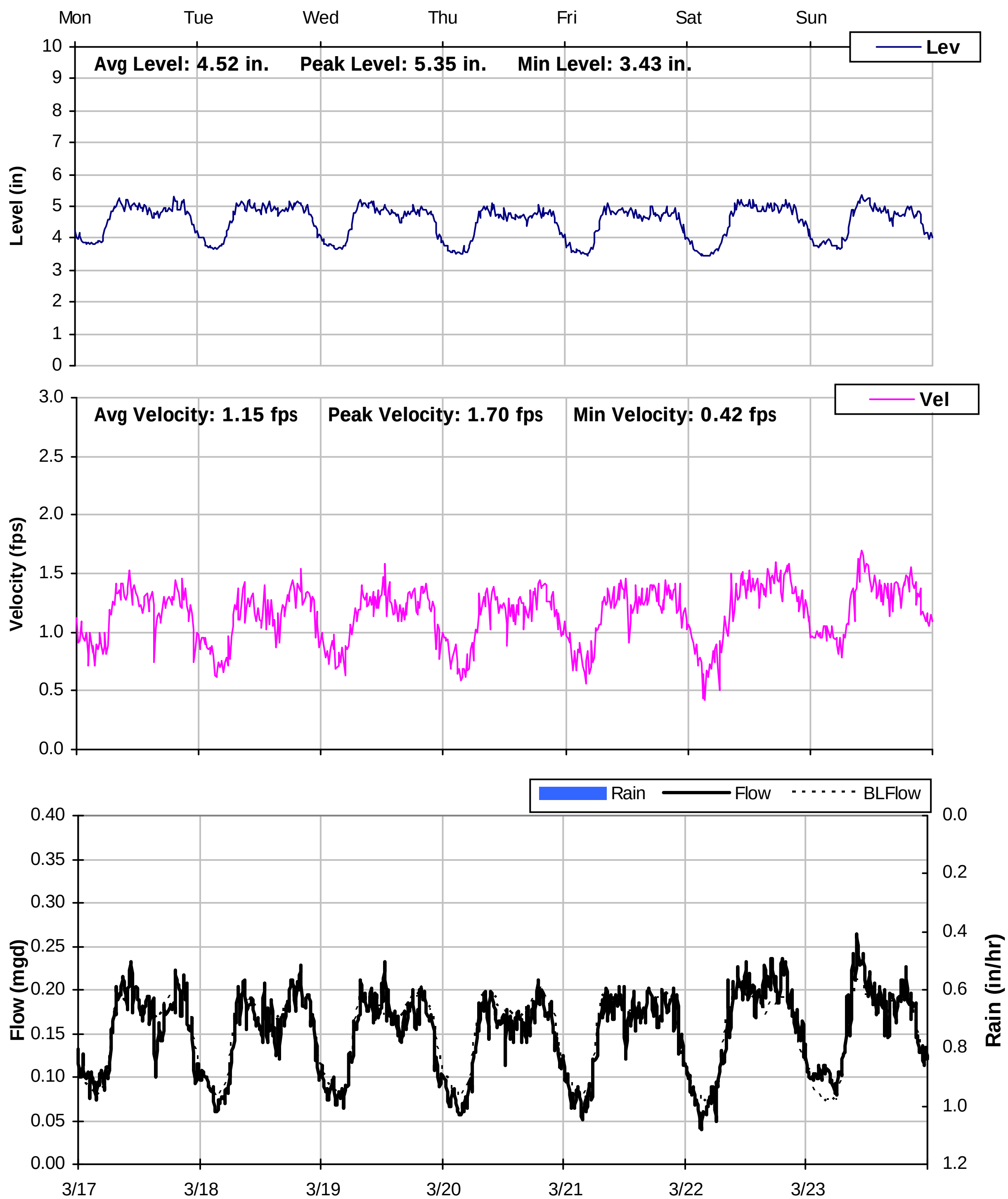
Avg Flow: 0.247 mgd **Peak Flow: 0.859 mgd** **Min Flow: 0.060 mgd**

SITE 6
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014

SITE 6
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014

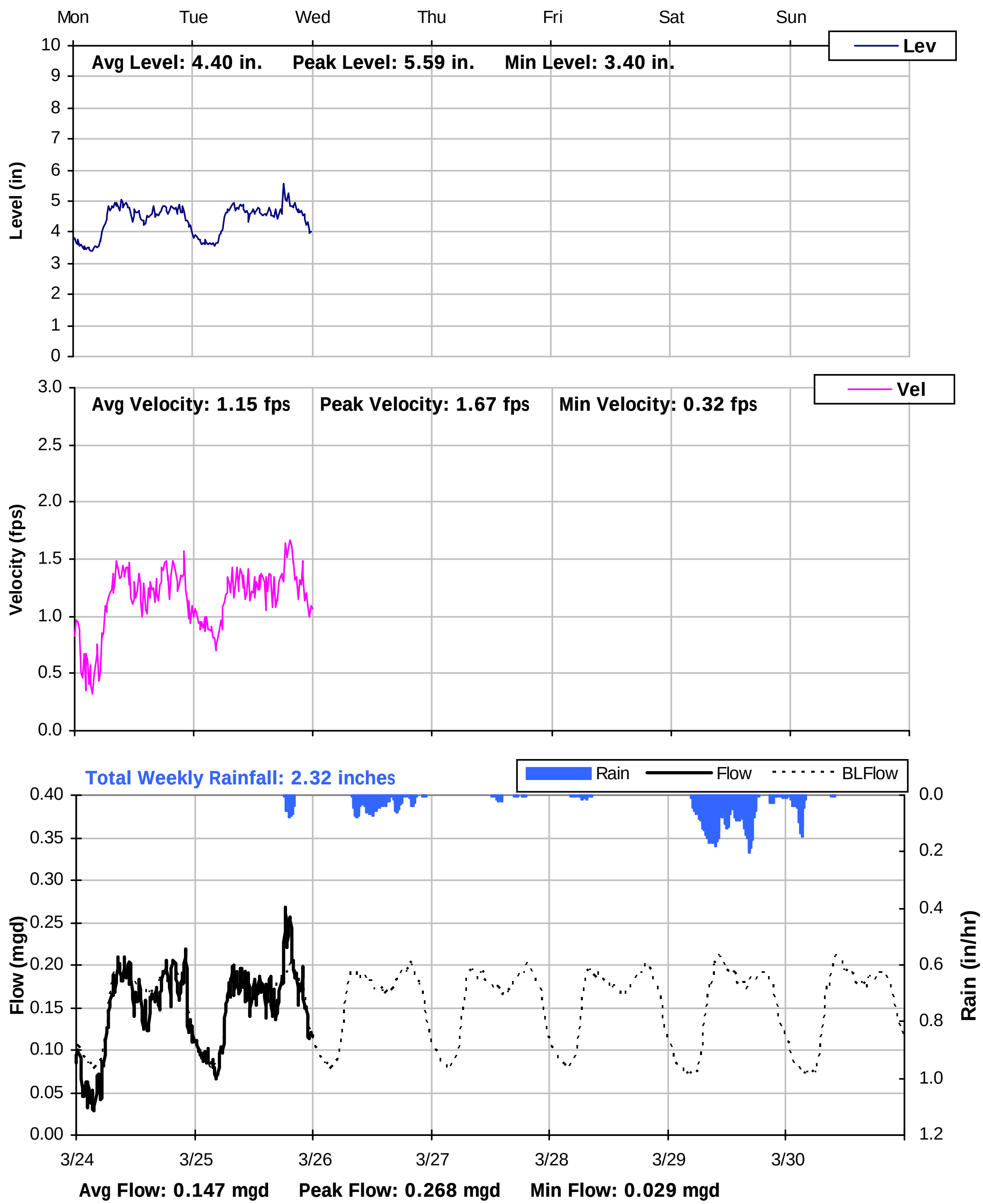


SITE 6
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



Avg Flow: 0.151 mgd **Peak Flow: 0.264 mgd** **Min Flow: 0.039 mgd**

SITE 6
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014



City of Grass Valley

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 7

Location: East Main Street and Harris Street

Data Summary Report

Vicinity Map: Site 7

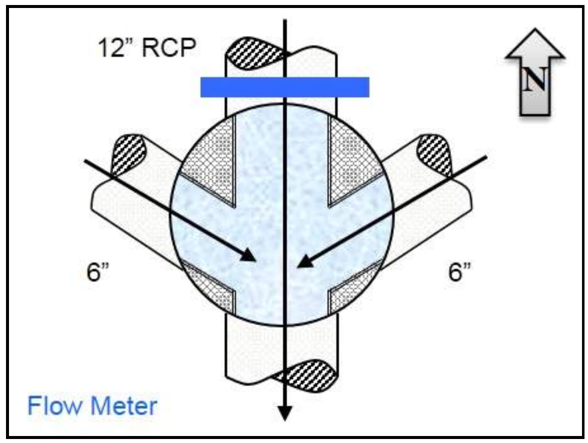
SITE 7

Site Information

Location:	East Main Street and Harris Street
Coordinates:	121.0539° W, 39.2224° N
Rim Elevation:	2434 feet
Pipe Diameter:	12 inches
Baseline Flow:	0.353 mgd
Peak Measured Flow:	2.037 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

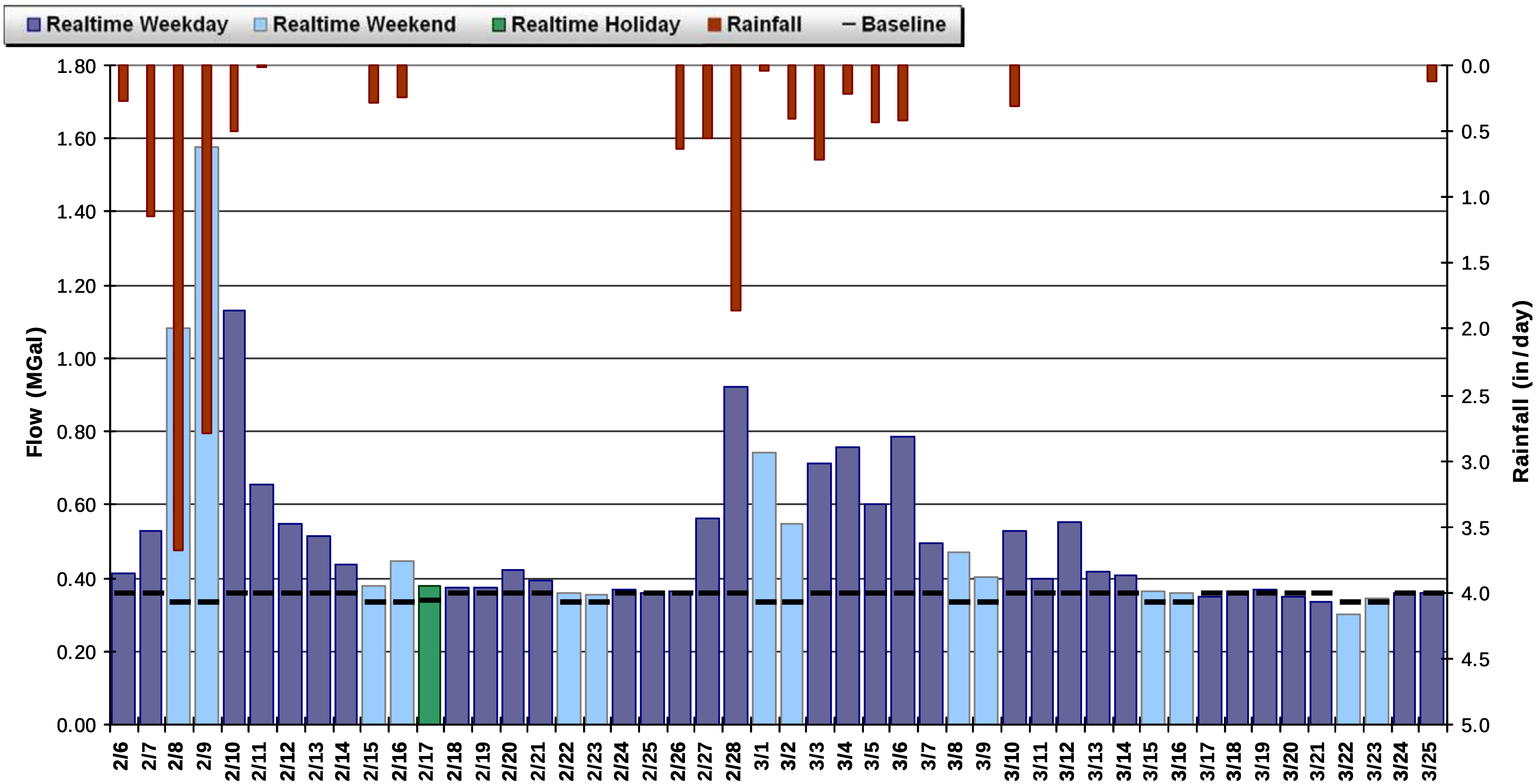
Plan View

SITE 7

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.513 MGal **Peak Daily Flow: 1.579 MGal** **Min Daily Flow: 0.302 MGal**

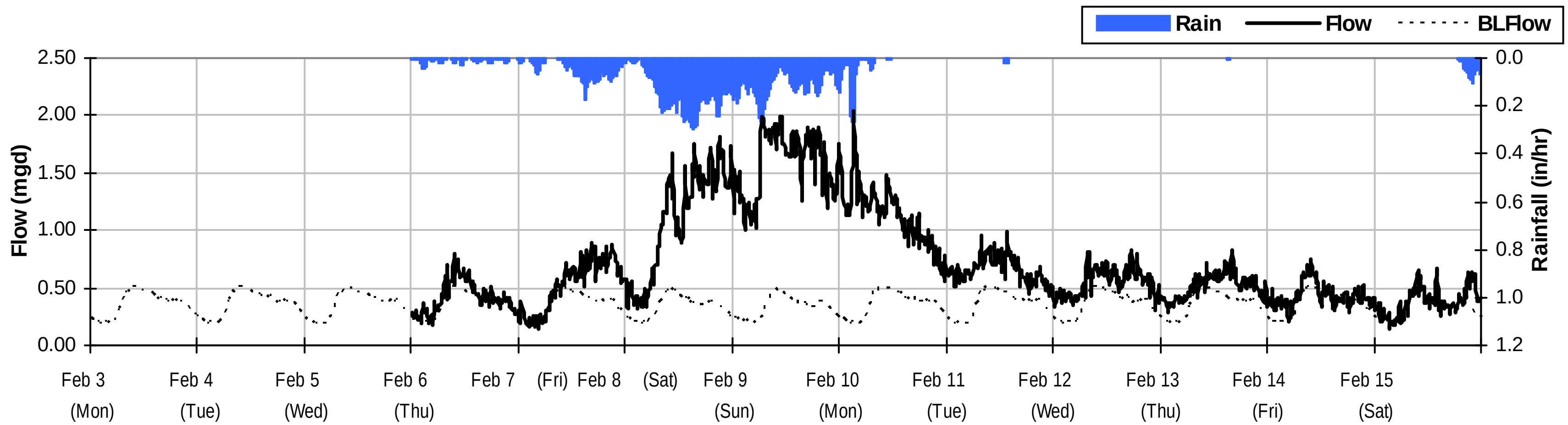
Total Period Rainfall: 14.64 inches

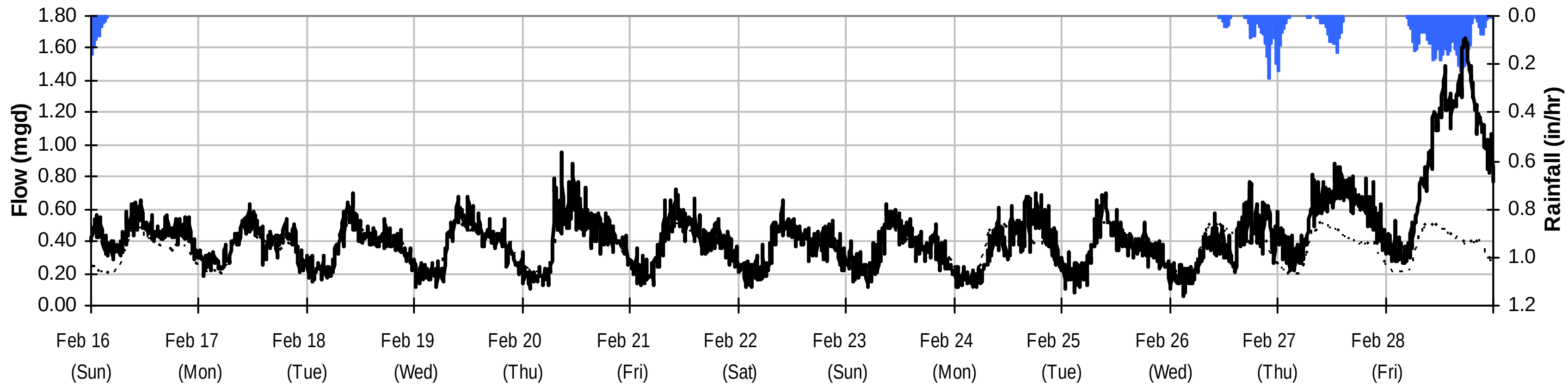


SITE 7

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.563 mgd** **Peak Flow: 2.037 mgd** **Min Flow: 0.060 mgd**

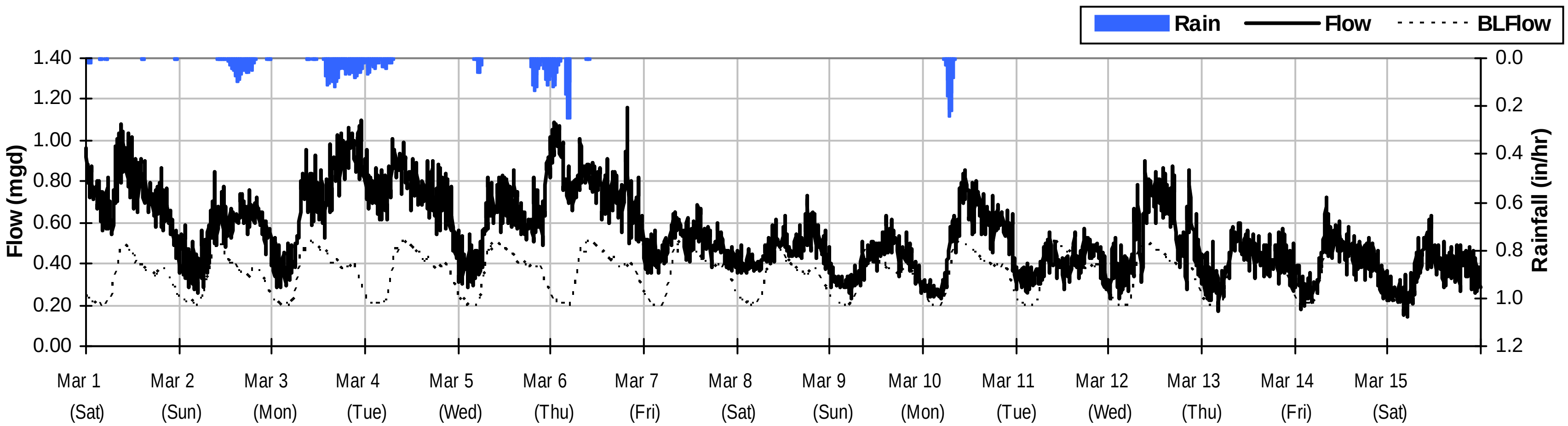


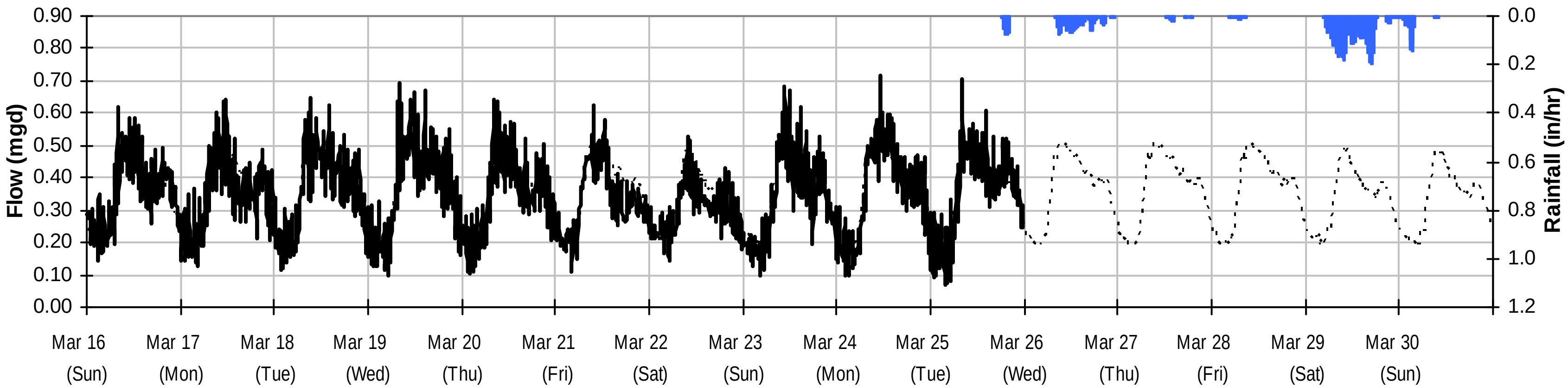


SITE 7

Monthly Flow Summary: March, 2014

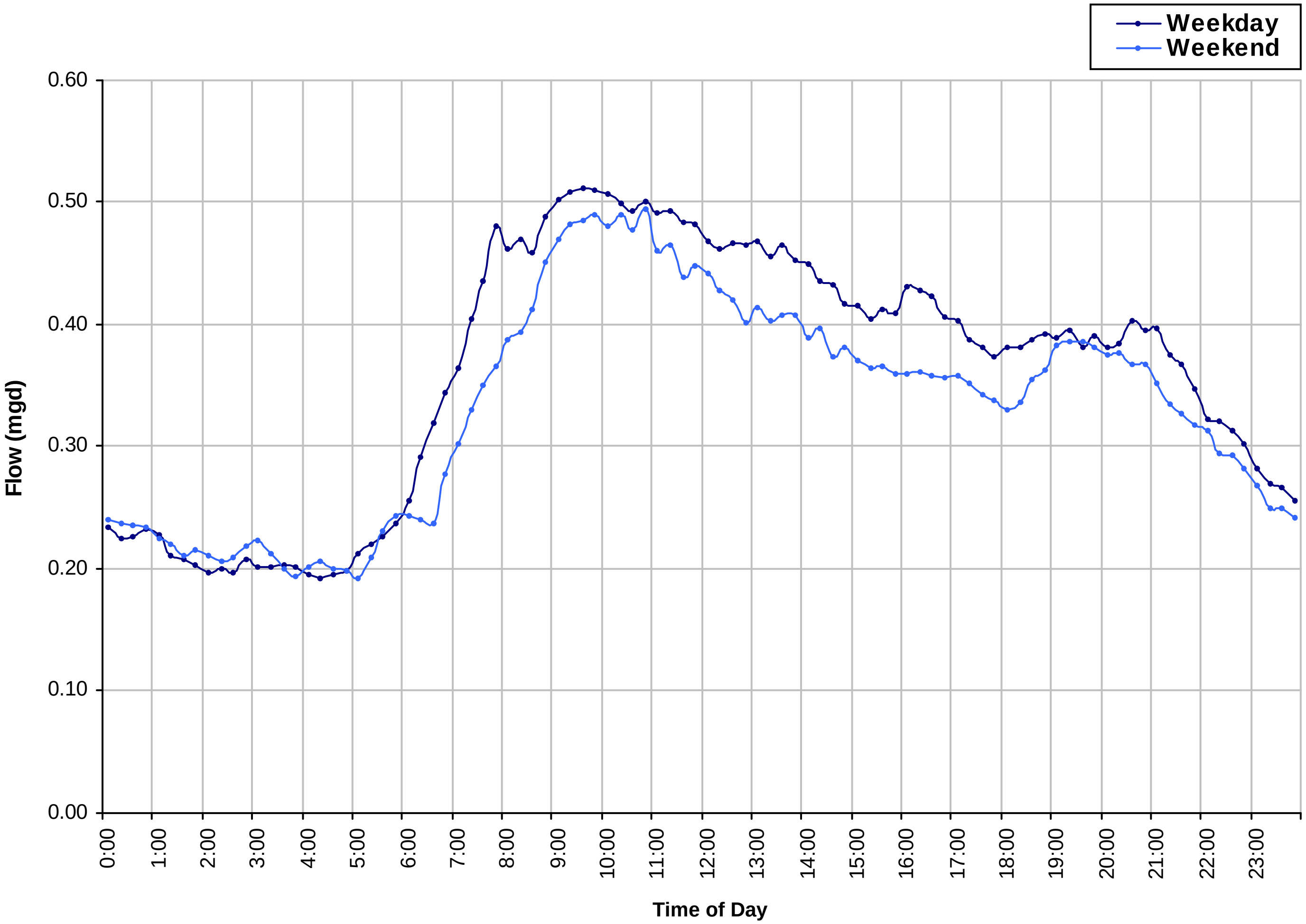
Total Monthly Rainfall: 4.85 inches **Avg Flow: 0.467 mgd** **Peak Flow: 1.153 mgd** **Min Flow: 0.067 mgd**



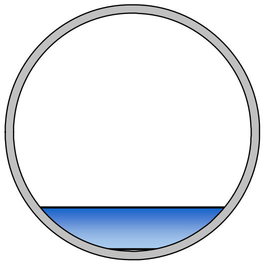


SITE 7

Baseline Flow Hydrographs

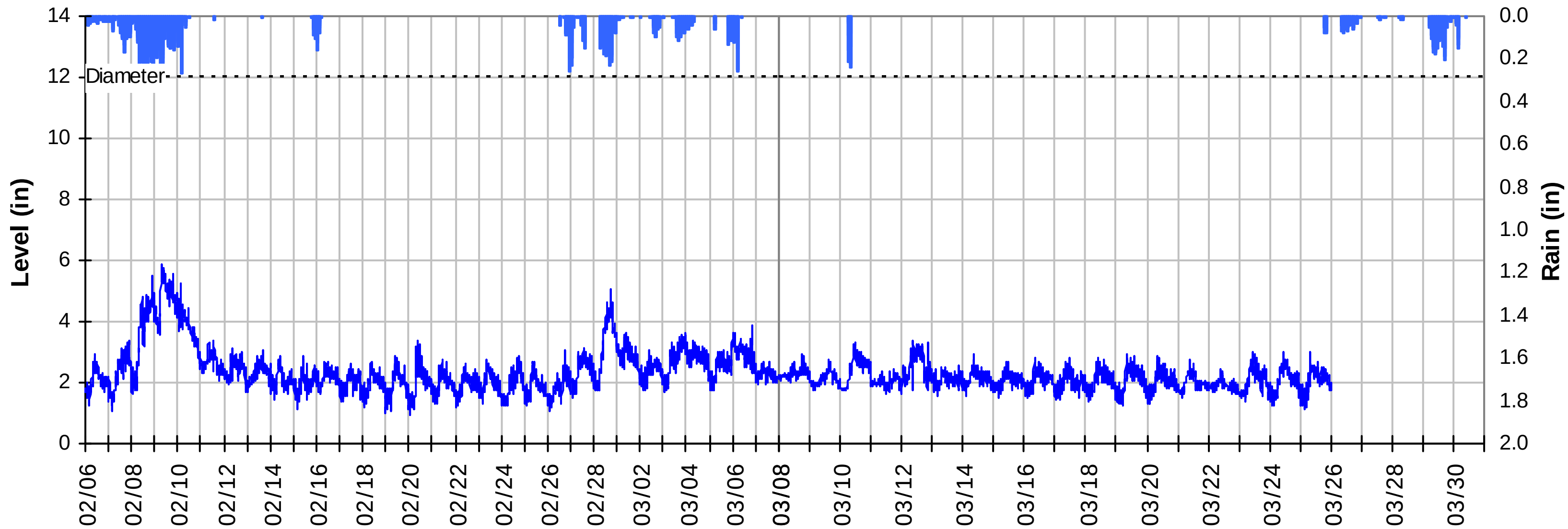


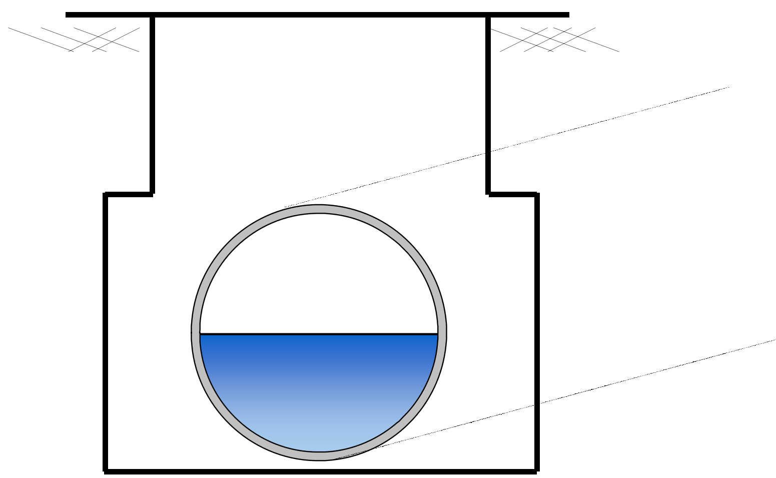
Baseline Flow:
0.353 *mgd*



SITE 7
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





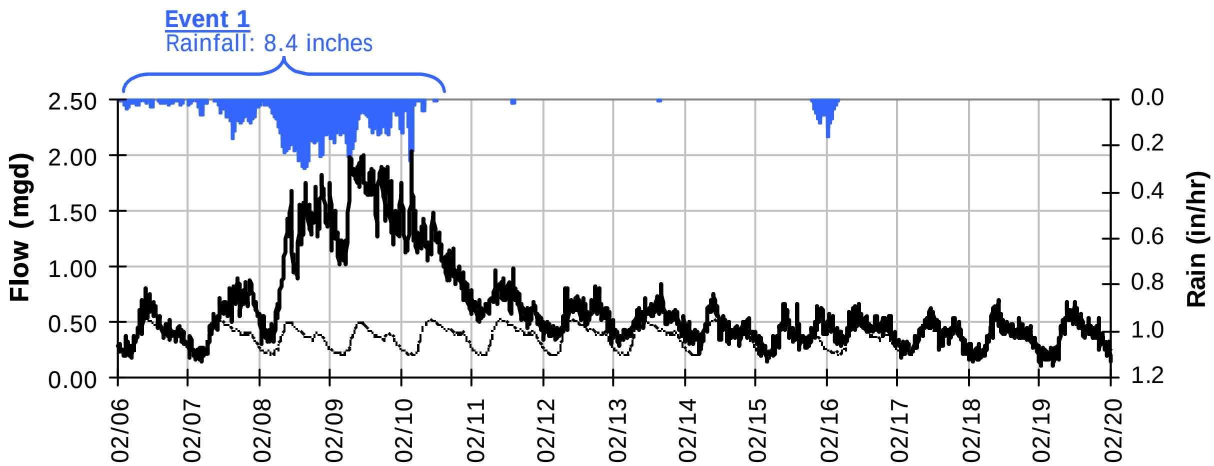
Pipe Diameter: 12 *inches*

Peak Measured Level: 5.88 *inches*

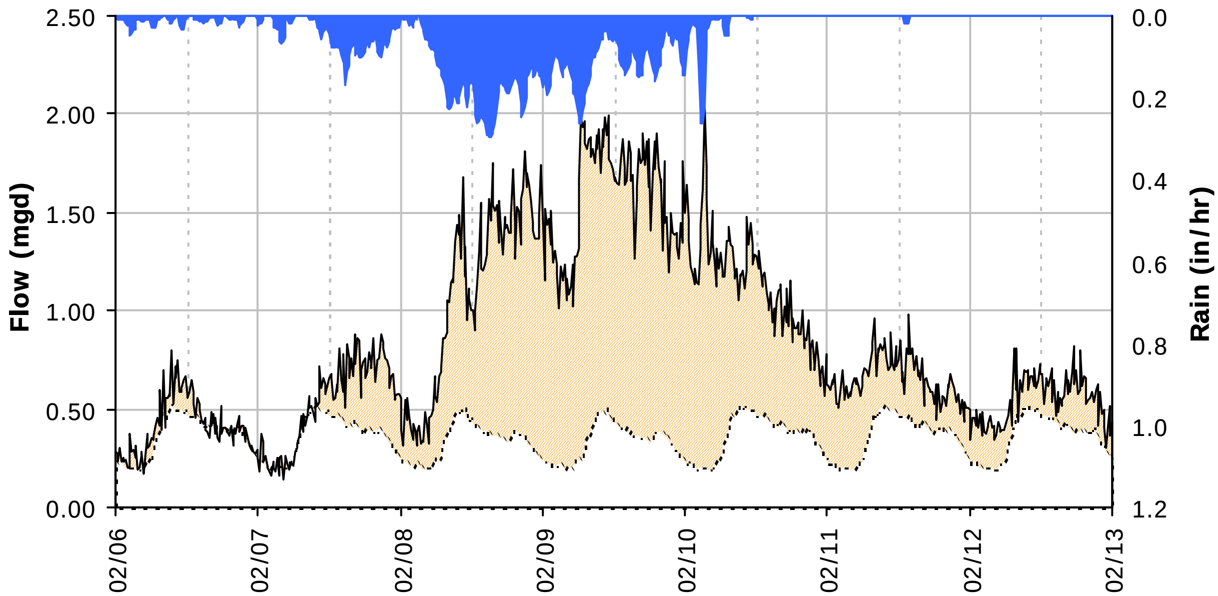
Peak d/D Ratio: 0.49

SITE 7
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

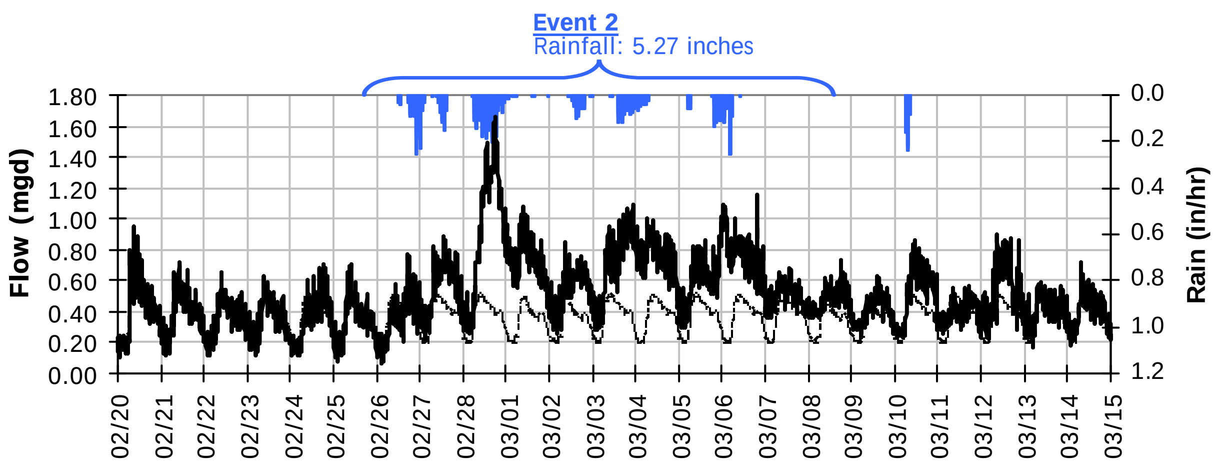


Storm Event I/I Analysis (Rain = 8.40 inches)

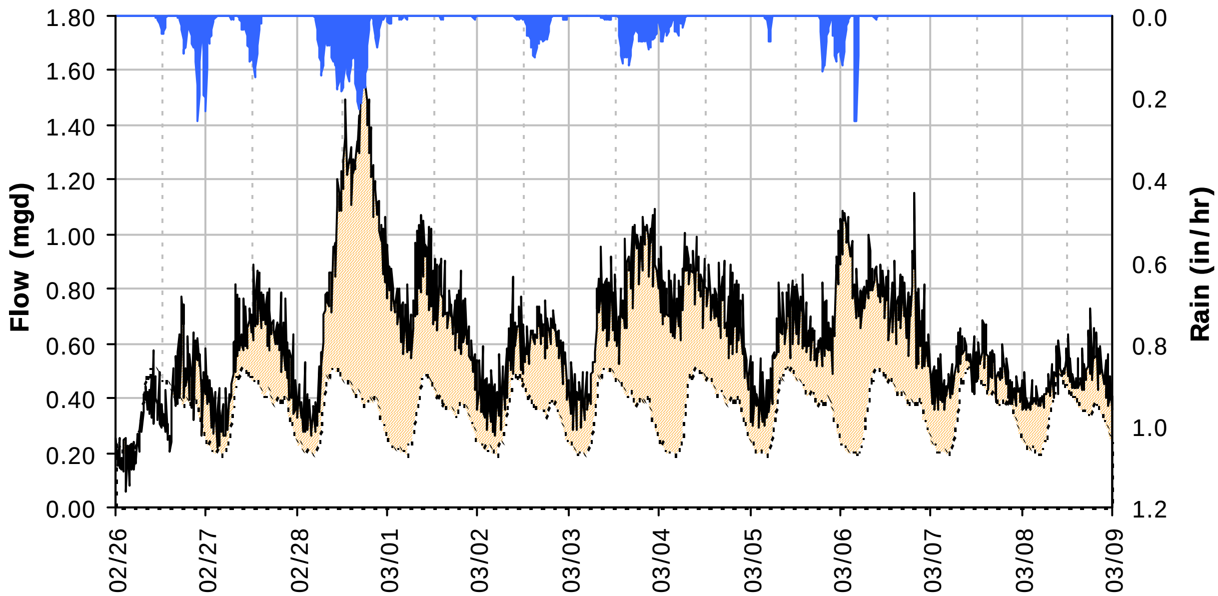
Capacity		**Inflow / Infiltration**	
Peak Flow:	2.04 *mgd*	**Peak I/I Rate:**	1.84 *mgd*
PF:	5.78	**Total I/I:**	3,477,000 *gallons*

SITE 7
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



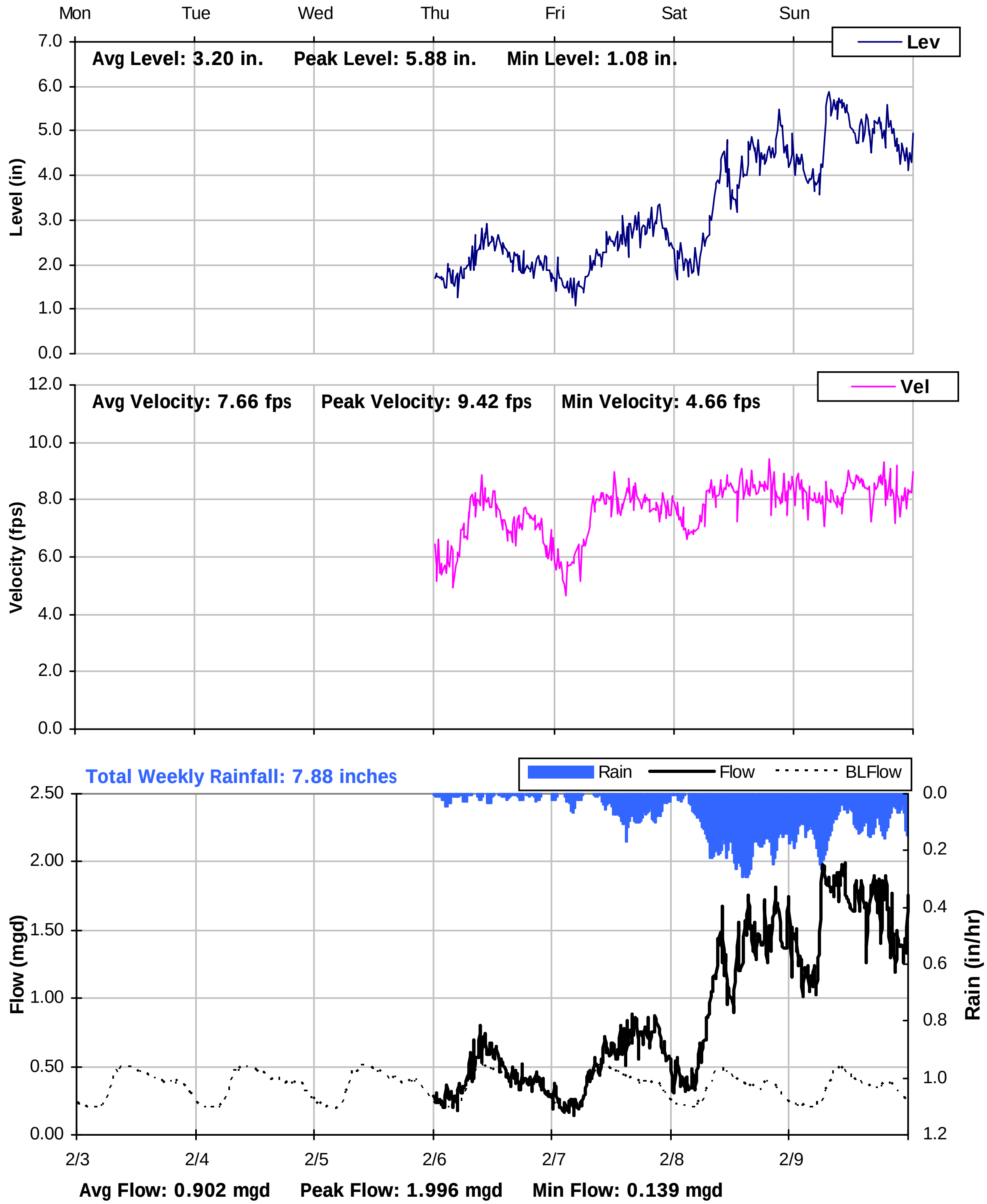
Event 2 Detail Graph



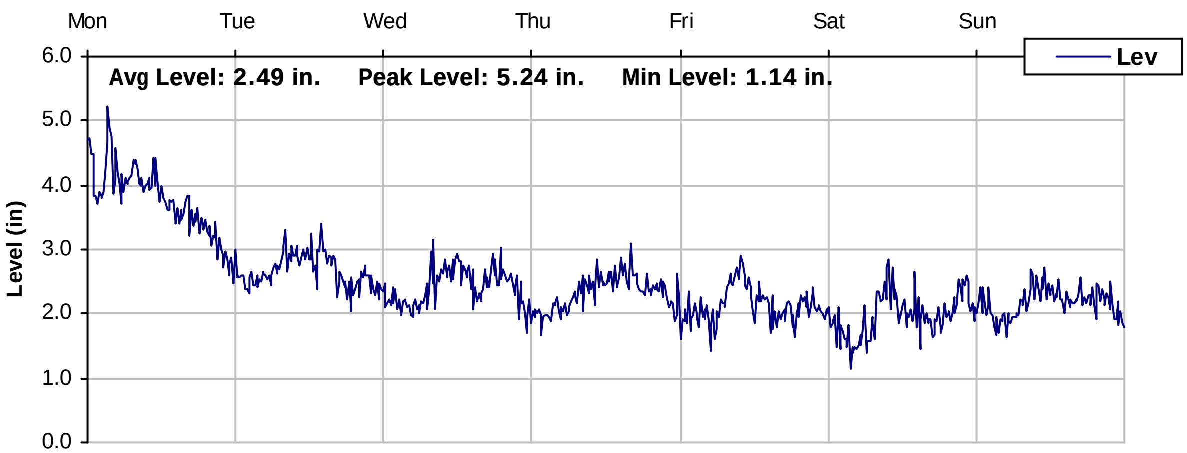
Storm Event I/I Analysis (Rain = 5.27 inches)

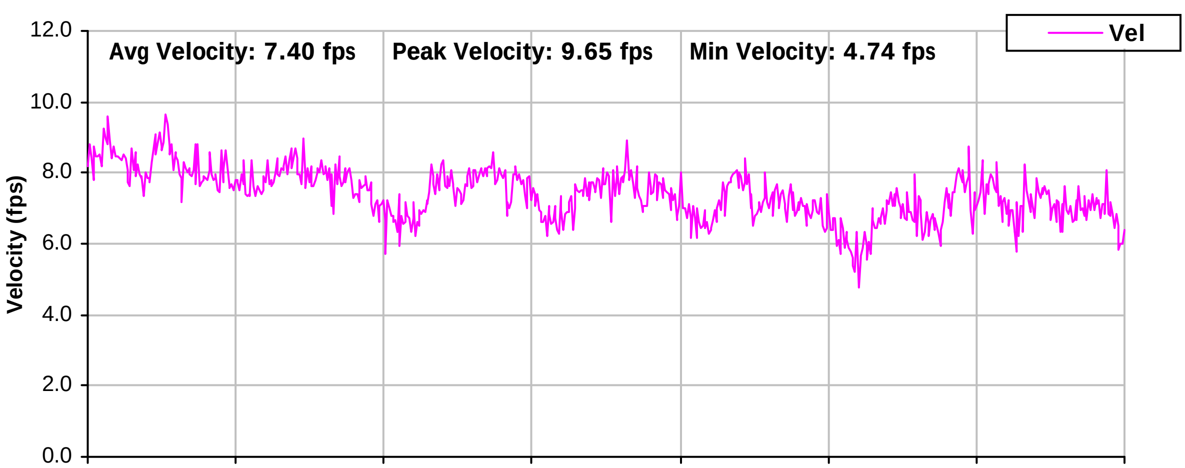
Capacity		Inflow / Infiltration	
Peak Flow:	1.66 *mgd*	**Peak I/I Rate:**	1.29 *mgd*
PF:	4.72	**Total I/I:**	3,092,000 *gallons*

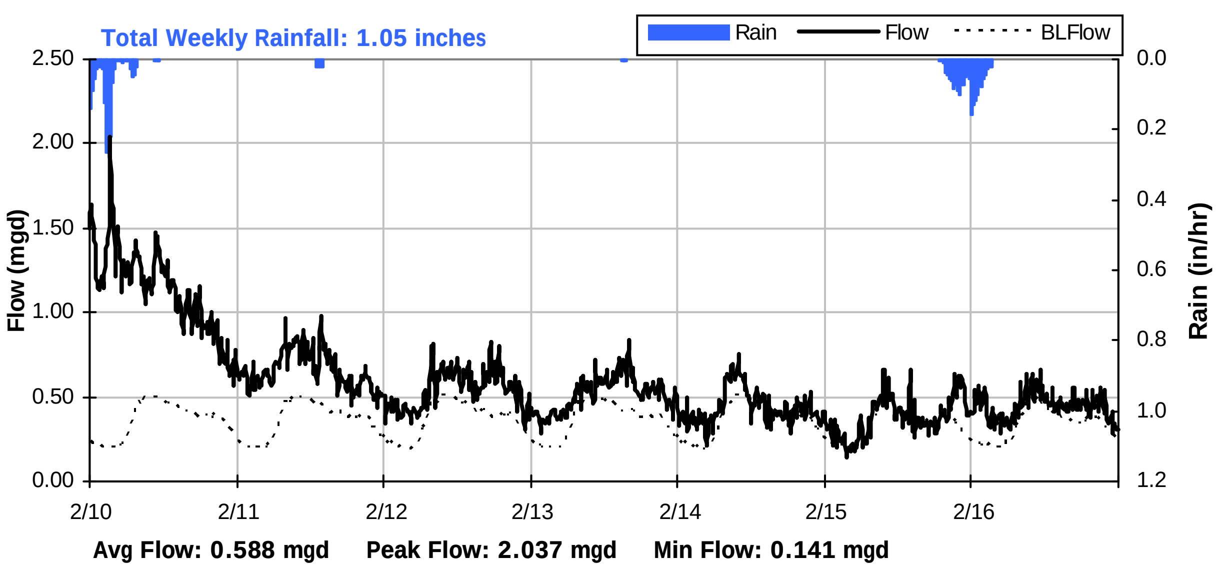
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014



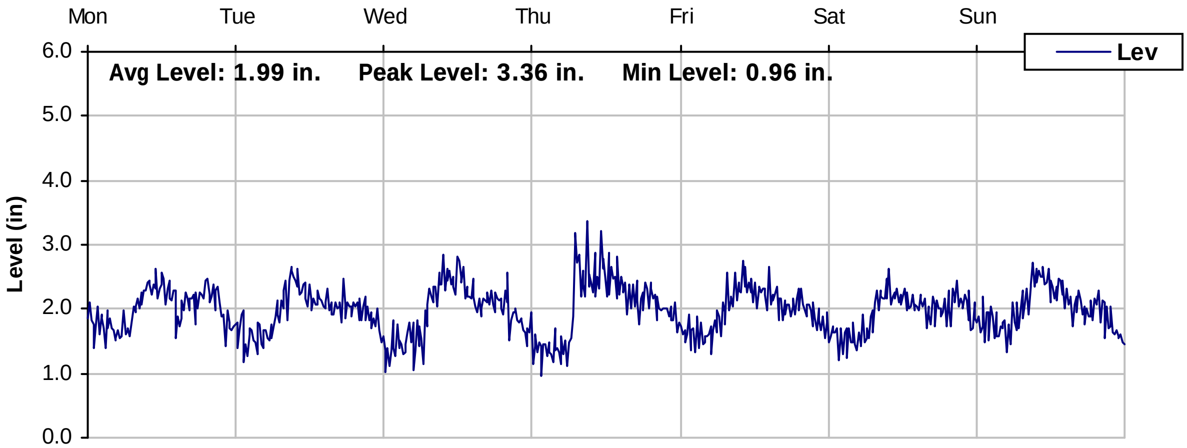
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014

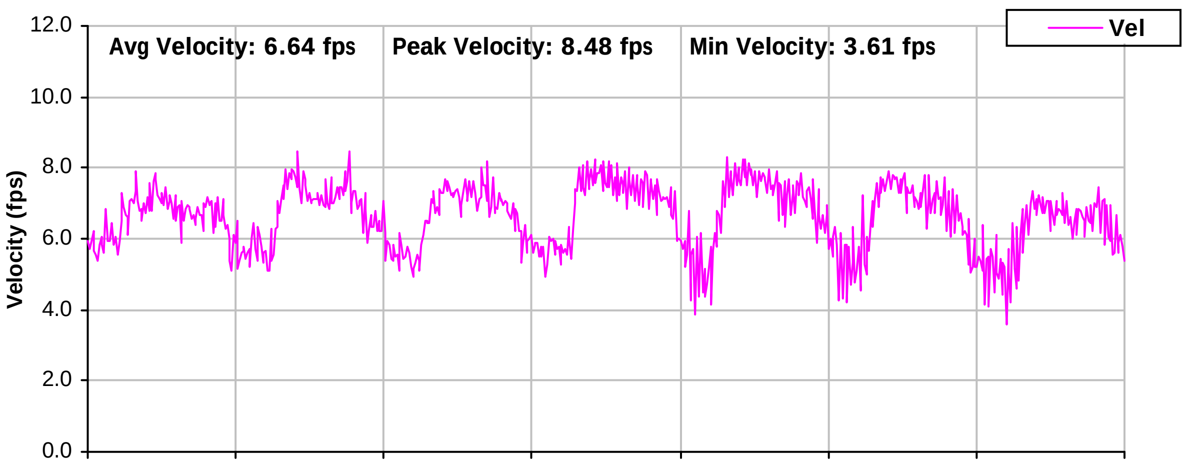


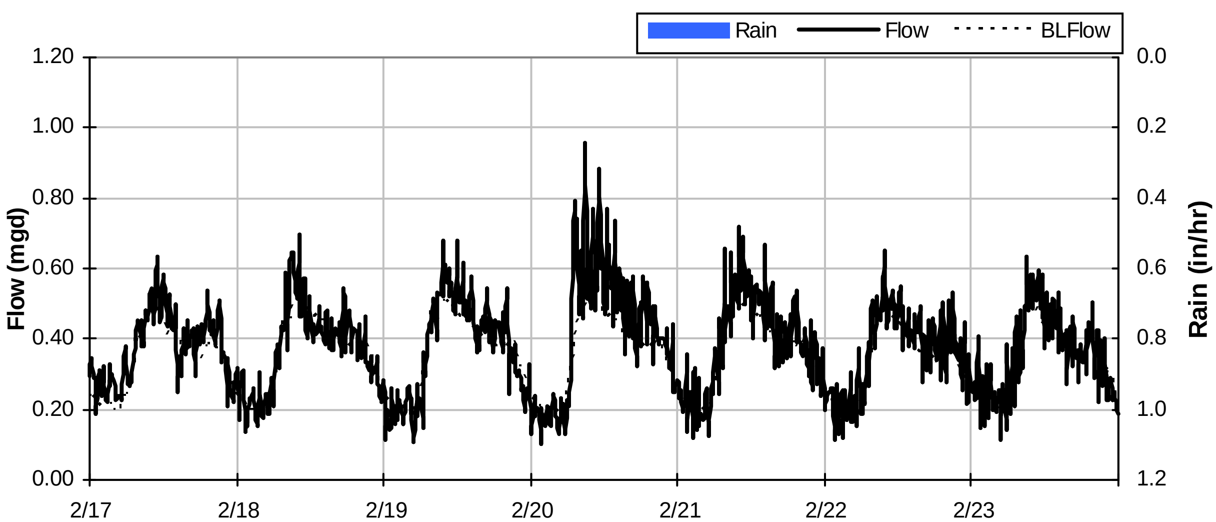




SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014

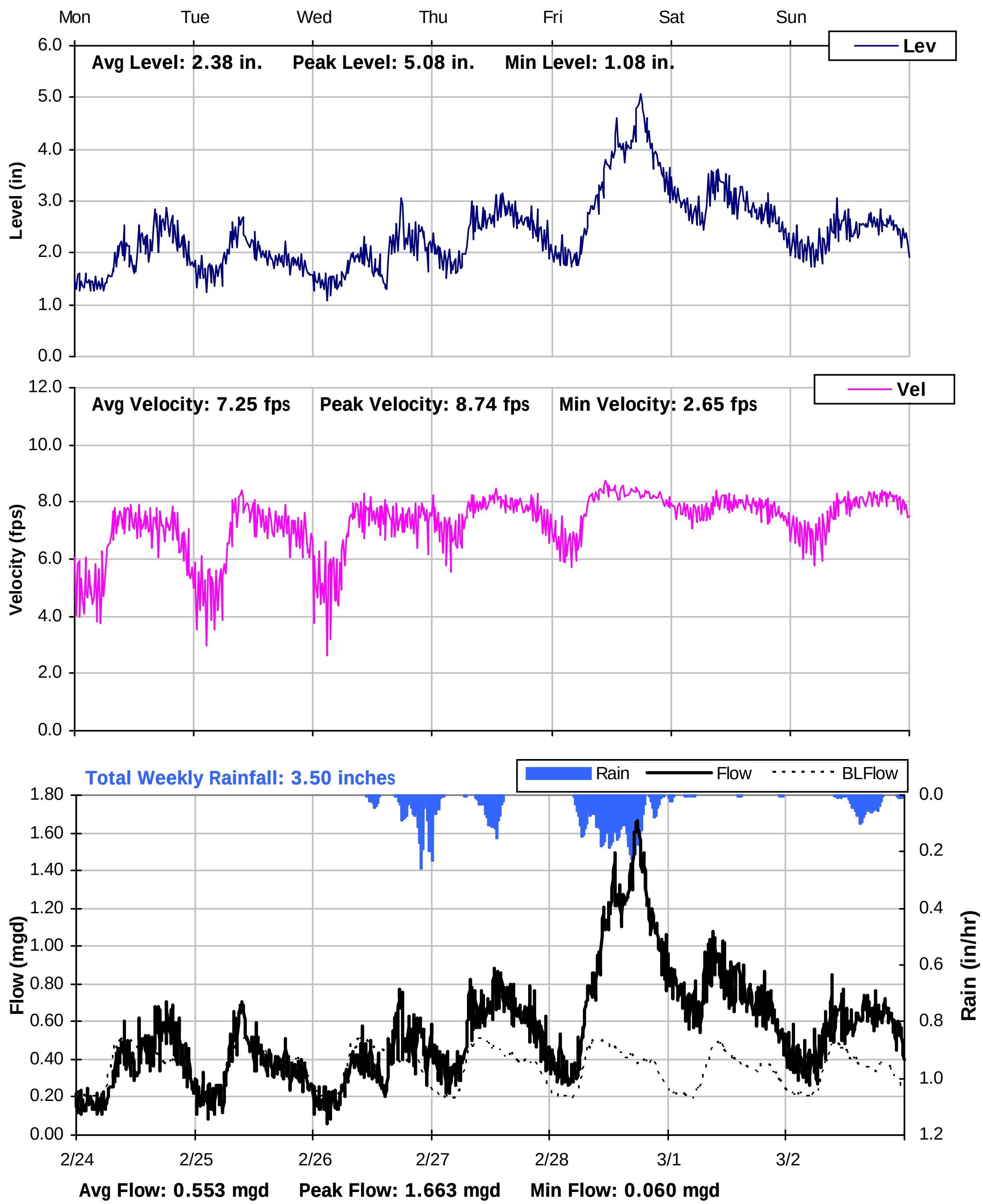




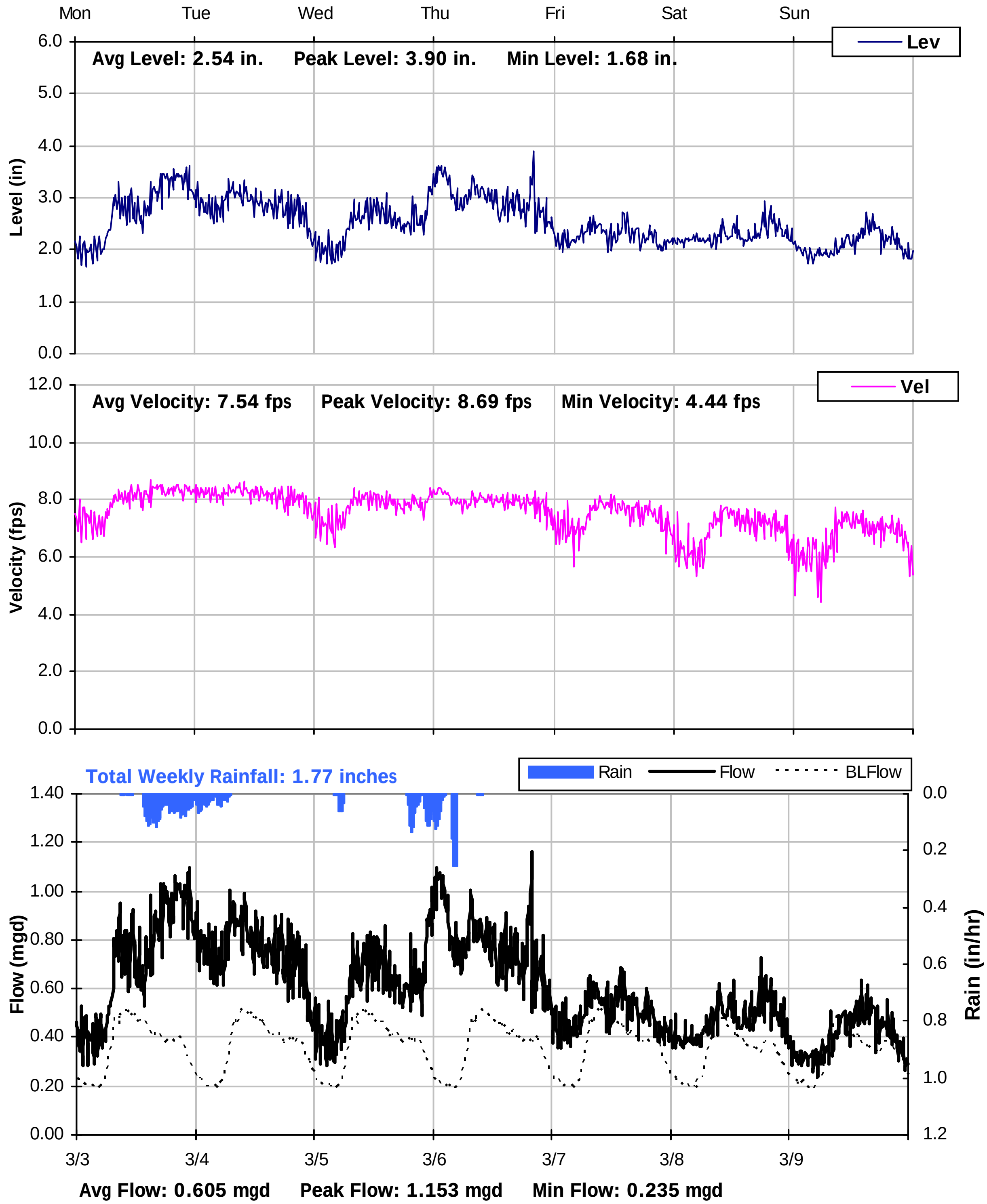


Avg Flow: 0.379 mgd **Peak Flow: 0.954 mgd** **Min Flow: 0.105 mgd**

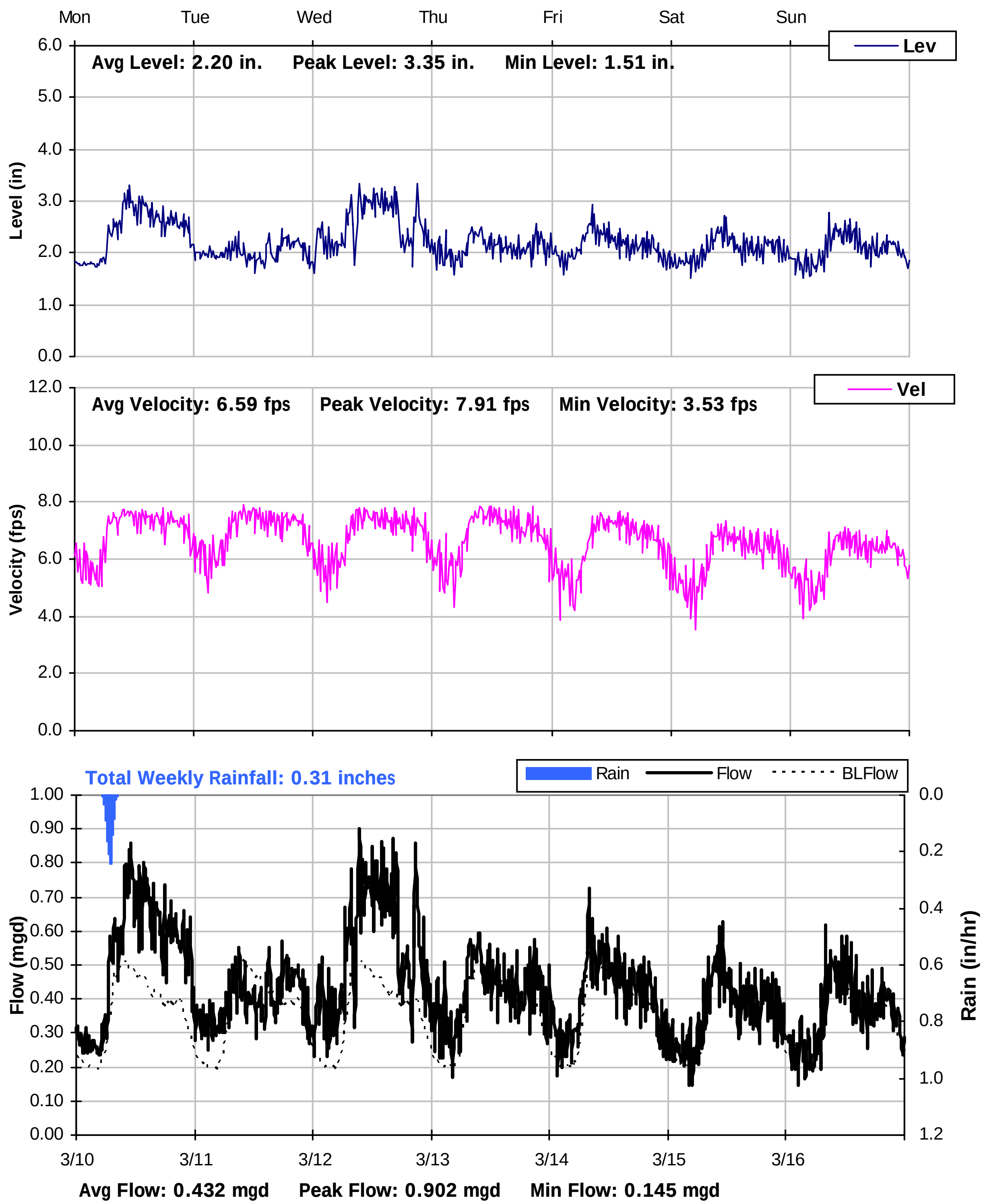
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



SITE 7
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



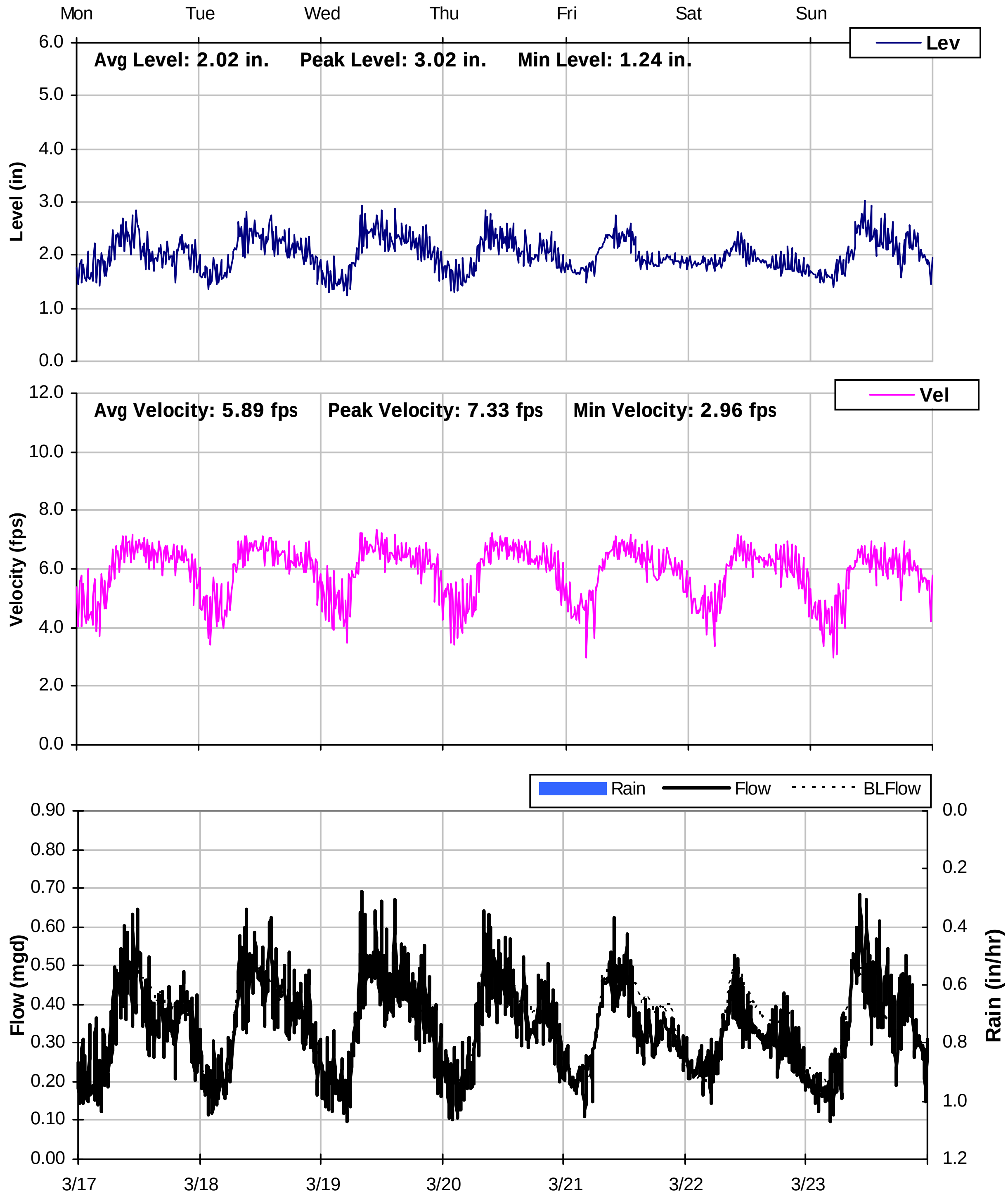
SITE 7
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014



SITE 7

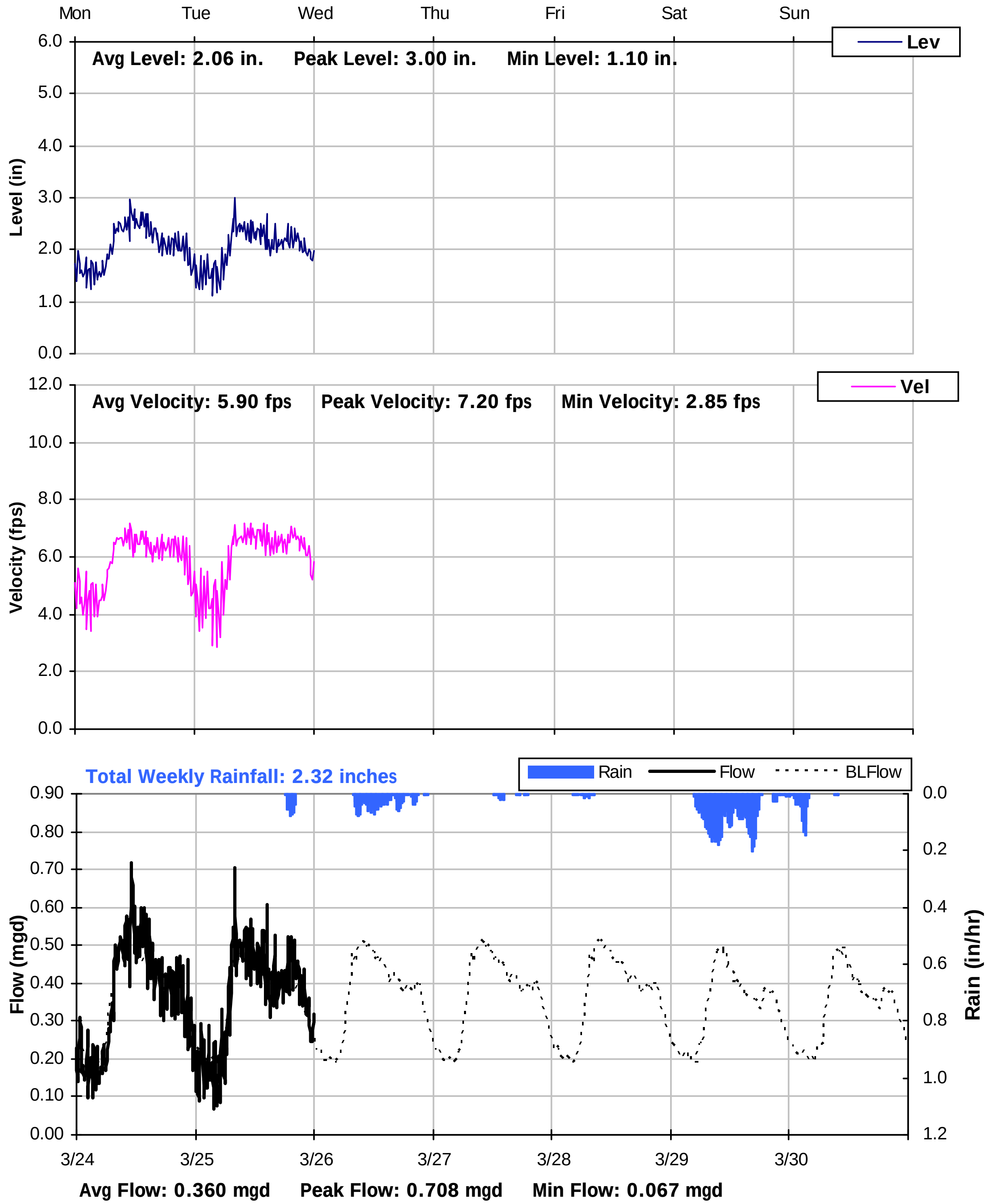
Weekly Level, Velocity and Flow Hydrographs

3/17/2014 to 3/24/2014



Avg Flow: 0.344 mgd **Peak Flow: 0.692 mgd** **Min Flow: 0.096 mgd**

SITE 7
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014



City of Grass Valley

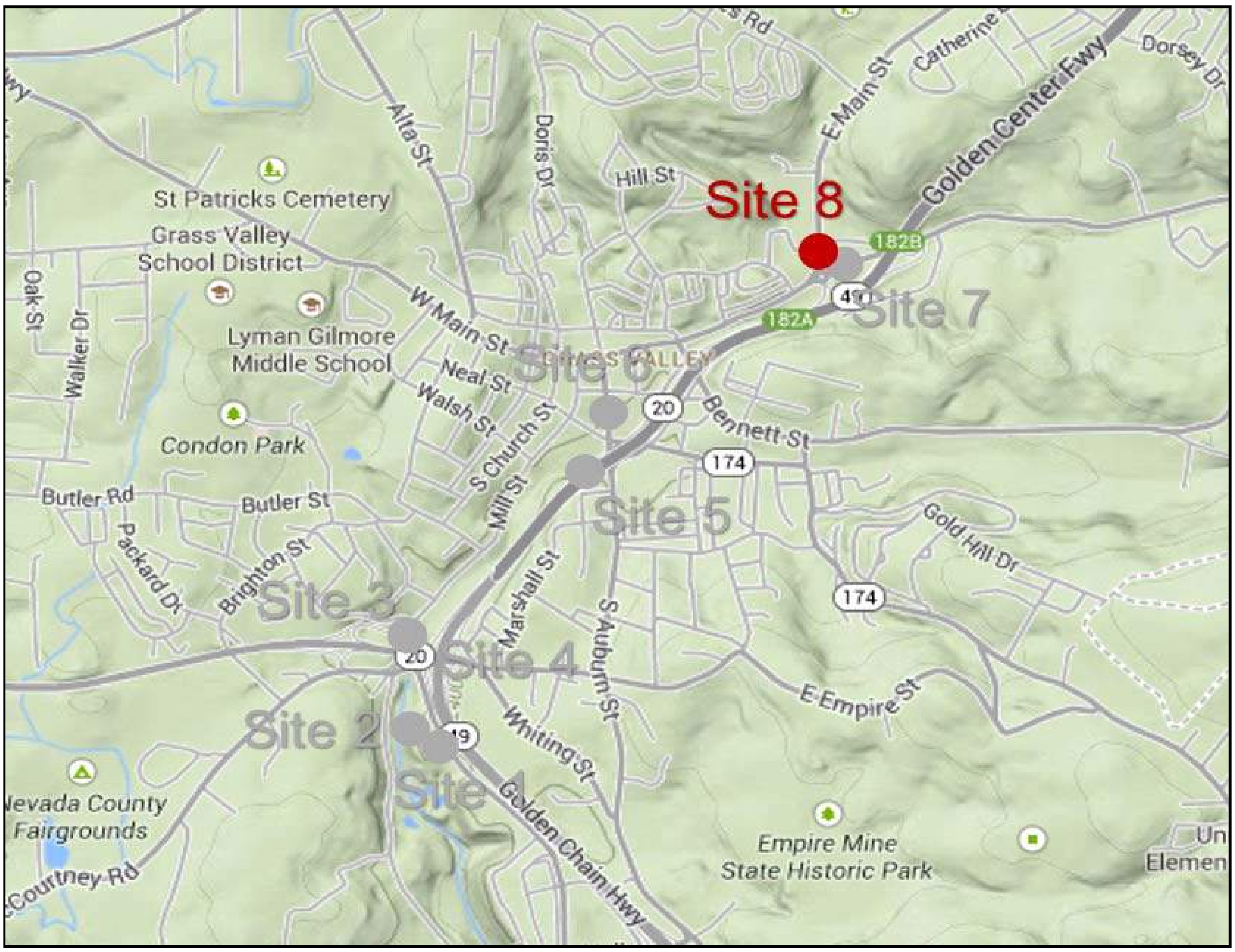
Sanitary Sewer Flow Monitoring

Temporary Monitoring: February 6 to April 8, 2014

Monitoring Site: Site 8

Location: 126 Idaho Maryland Road

Data Summary Report



Vicinity Map: Site 8

SITE 8

Site Information

Location: 126 Idaho Maryland Road

Coordinates: 121.0531° W, 39.2220° N

Rim Elevation: 2436 feet

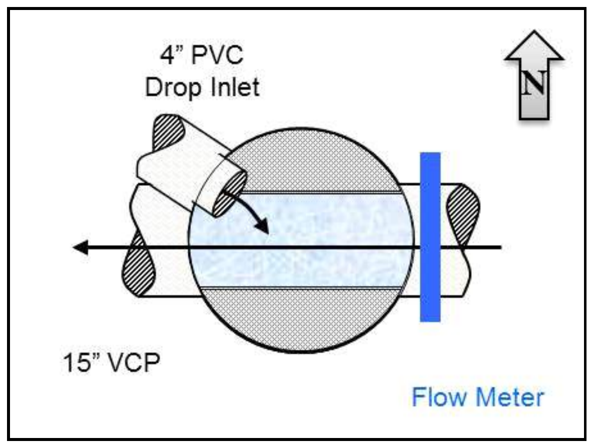
Pipe Diameter: 15 inches

Baseline Flow: 0.328 mgd

Peak Measured Flow: 1.816 mgd

Satellite Map

Sewer Map



Flow Sketch

Street View

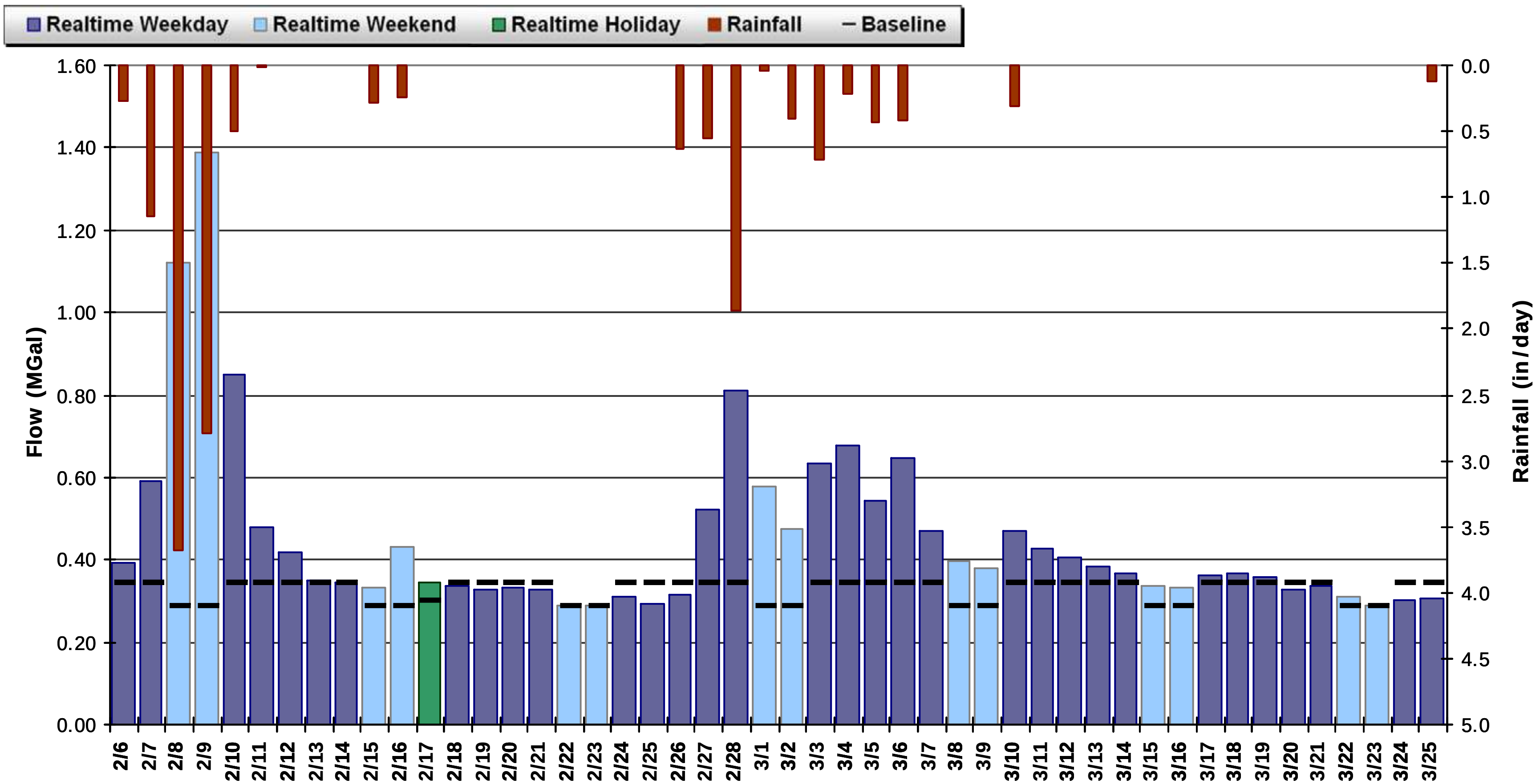
Plan View

SITE 8

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.452 MGal **Peak Daily Flow: 1.388 MGal** **Min Daily Flow: 0.287 MGal**

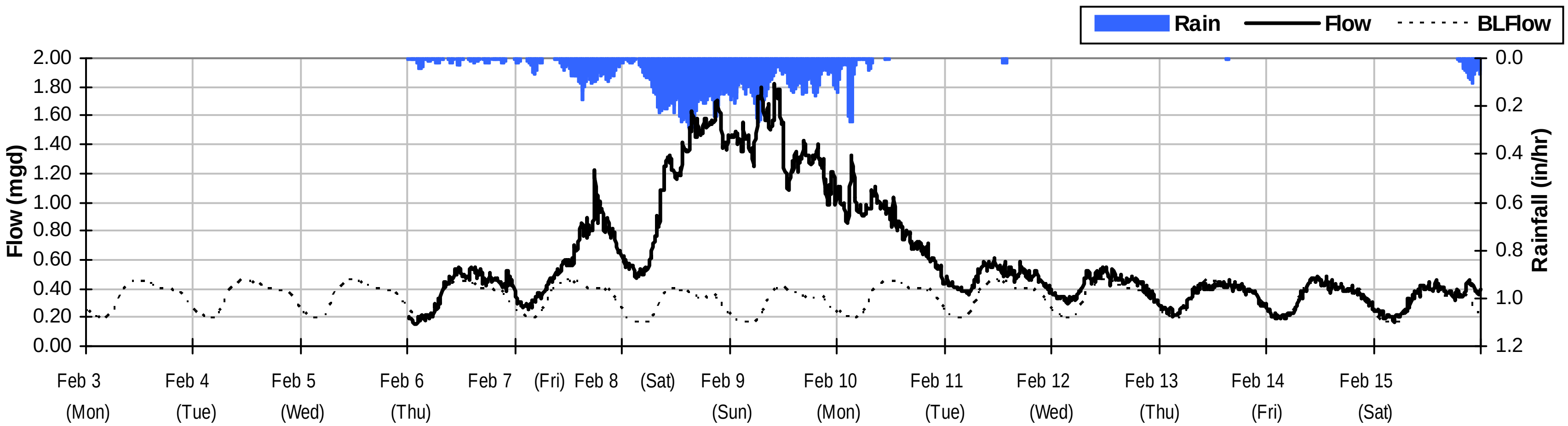
Total Period Rainfall: 14.64 inches

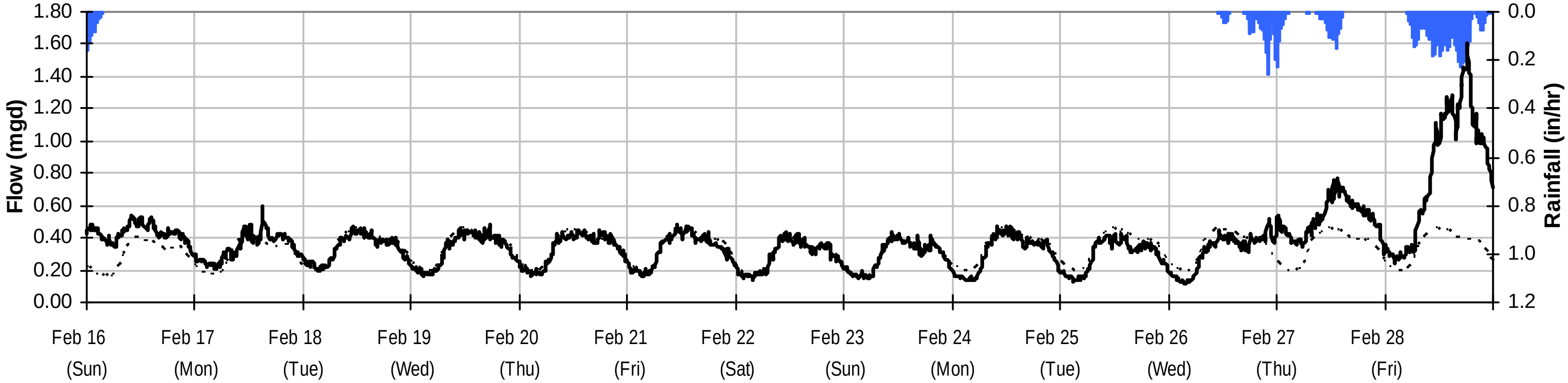


SITE 8

Monthly Flow Summary: February, 2014

Total Monthly Rainfall: 11.98 inches **Avg Flow: 0.487 mgd** **Peak Flow: 1.816 mgd** **Min Flow: 0.116 mgd**

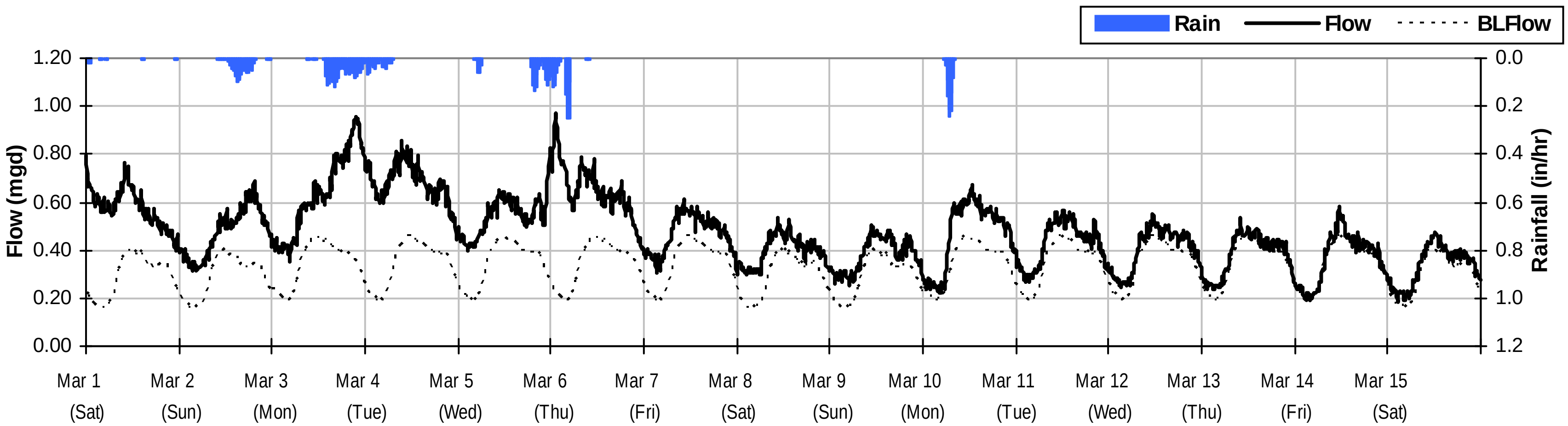


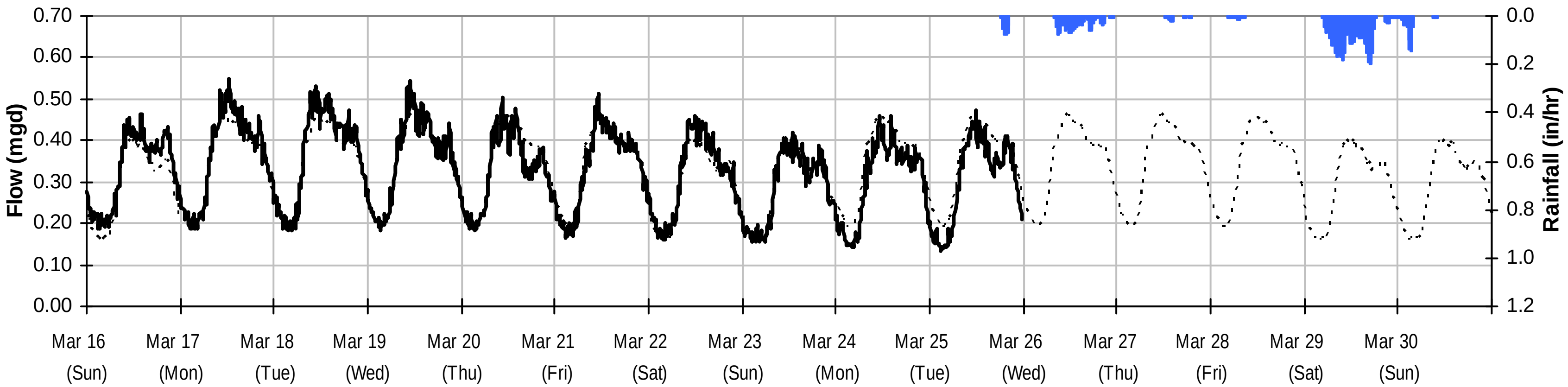


SITE 8

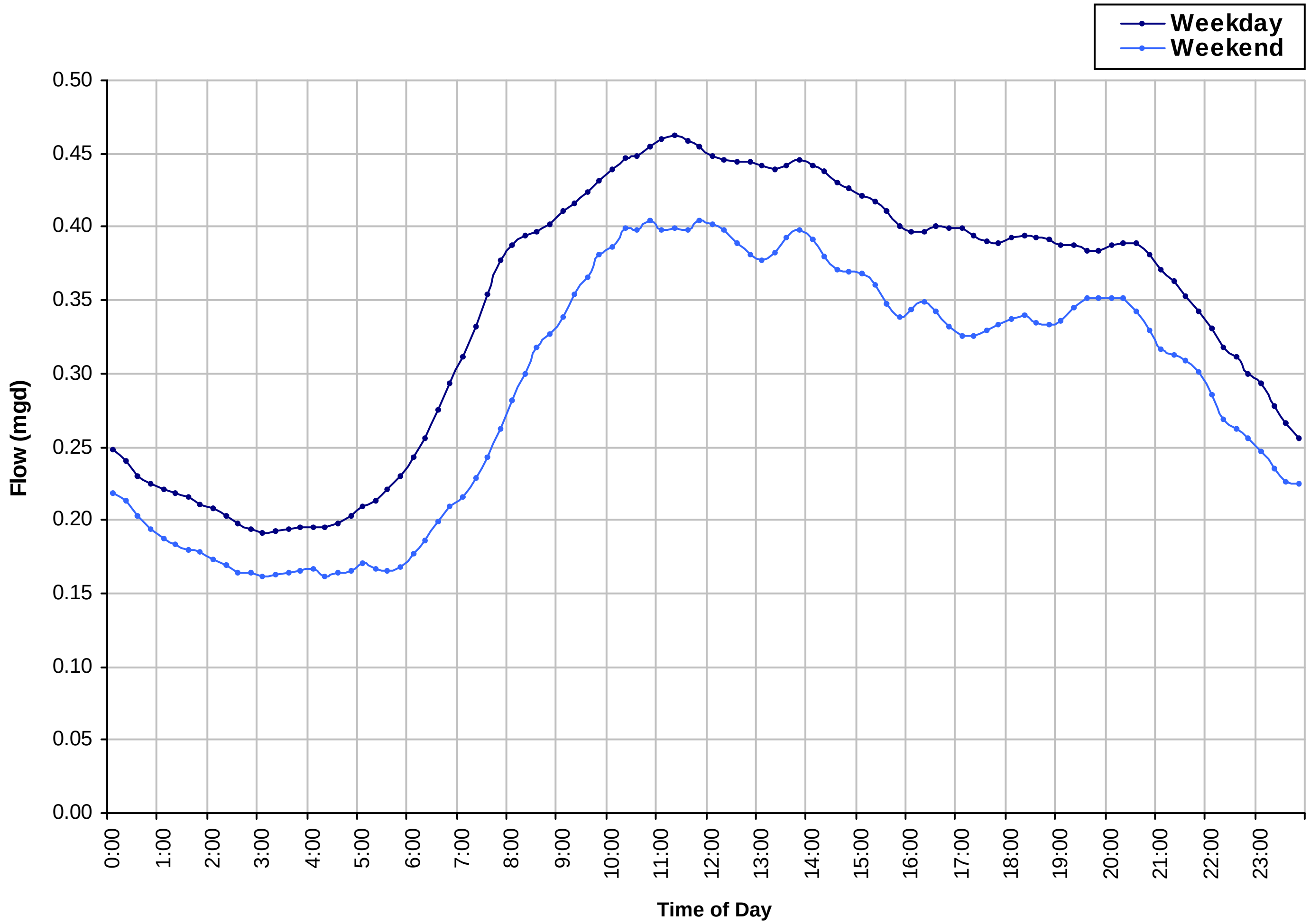
Monthly Flow Summary: March, 2014

Total Monthly Rainfall: 4.85 inches **Avg Flow: 0.419 mgd** **Peak Flow: 0.974 mgd** **Min Flow: 0.134 mgd**

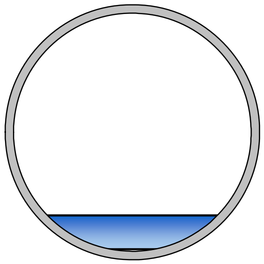




SITE 8
Baseline Flow Hydrographs

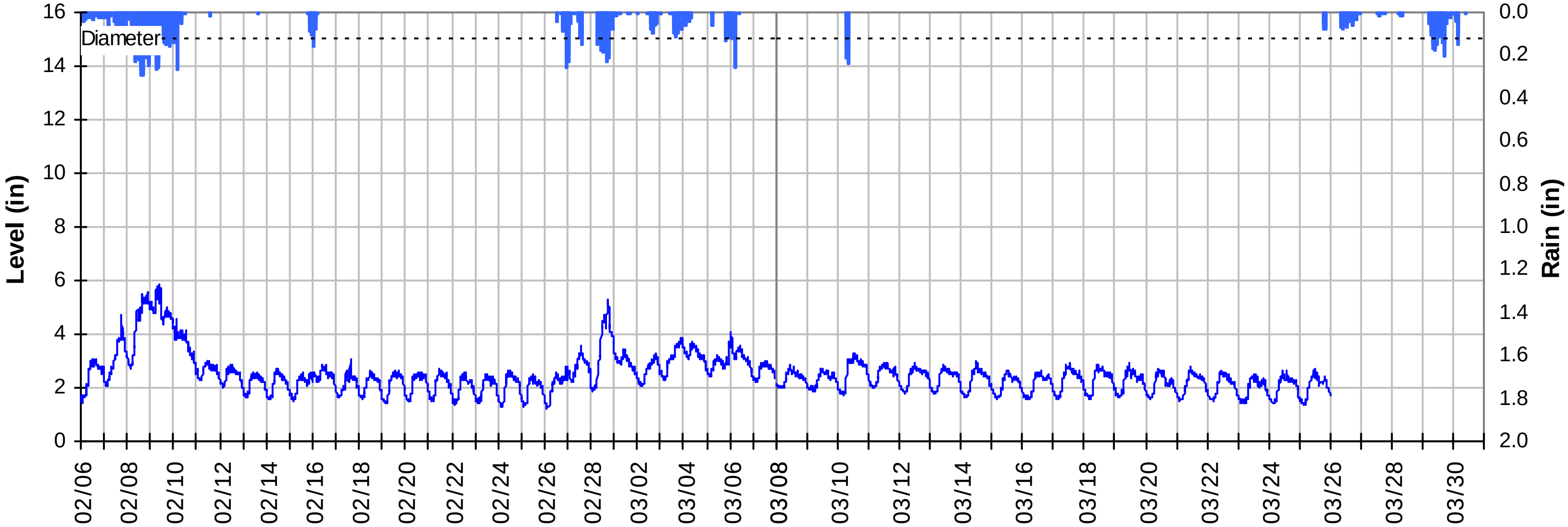


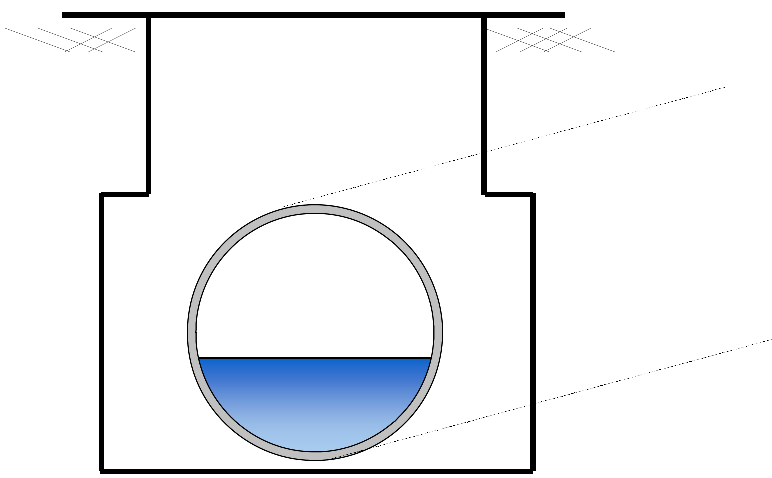
Baseline Flow:
0.328 *mgd*



SITE 8
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

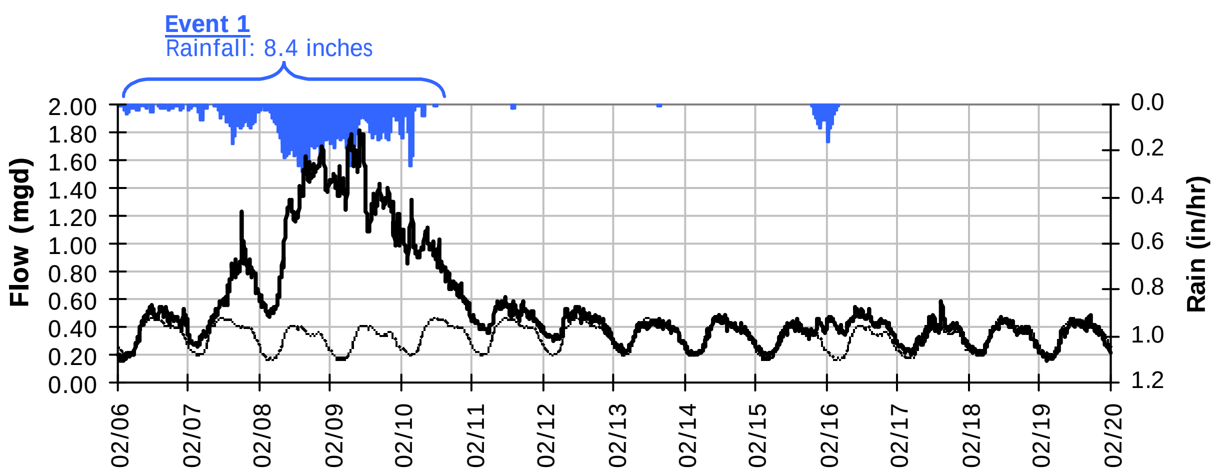




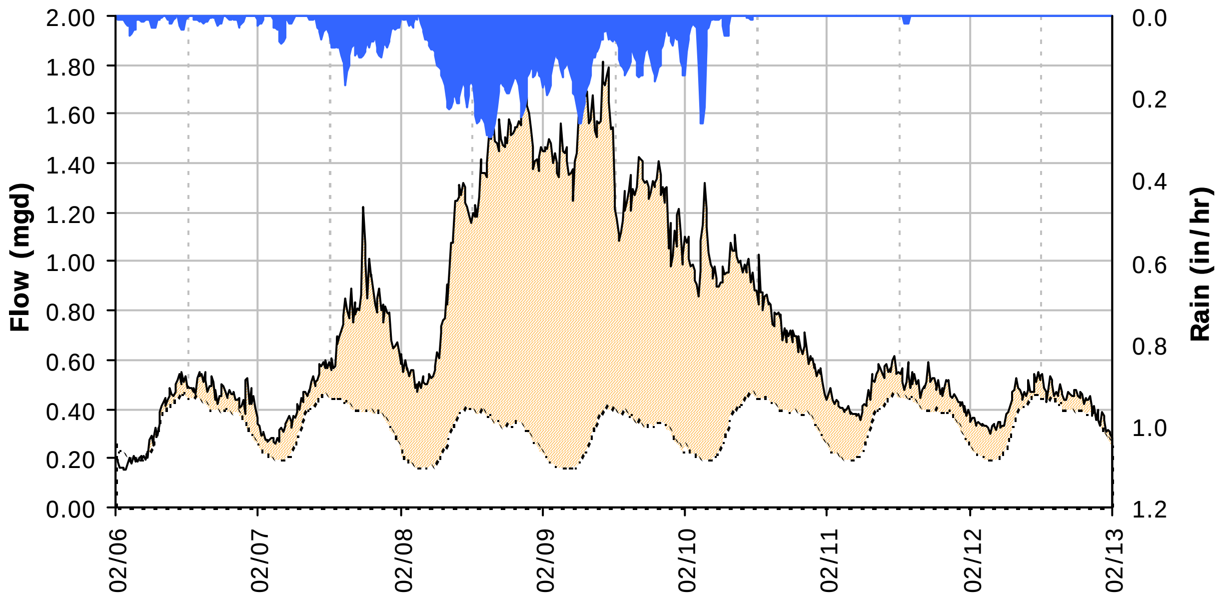
Pipe Diameter:	15	*inches*
Peak Measured Level:	5.86	*inches*
Peak d/D Ratio:	0.39	

SITE 8
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph

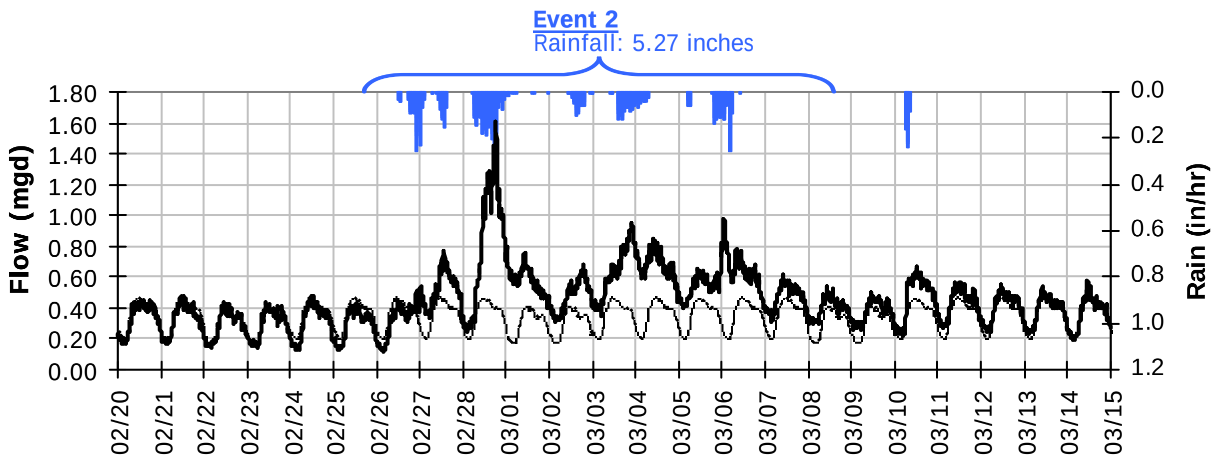


Storm Event I/I Analysis (Rain = 8.40 inches)

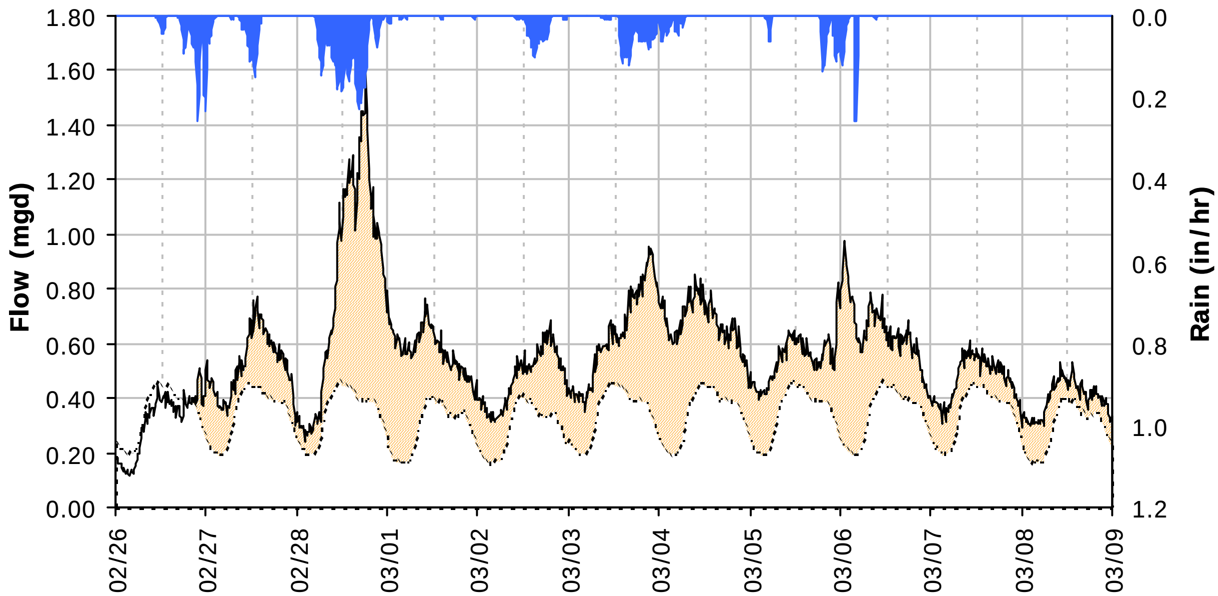
Capacity		Inflow / Infiltration	
Peak Flow:	1.82 *mgd*	**Peak I/I Rate:**	1.58 *mgd*
PF:	5.54	**Total I/I:**	2,948,000 *gallons*

SITE 8
I/I Summary: Event 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



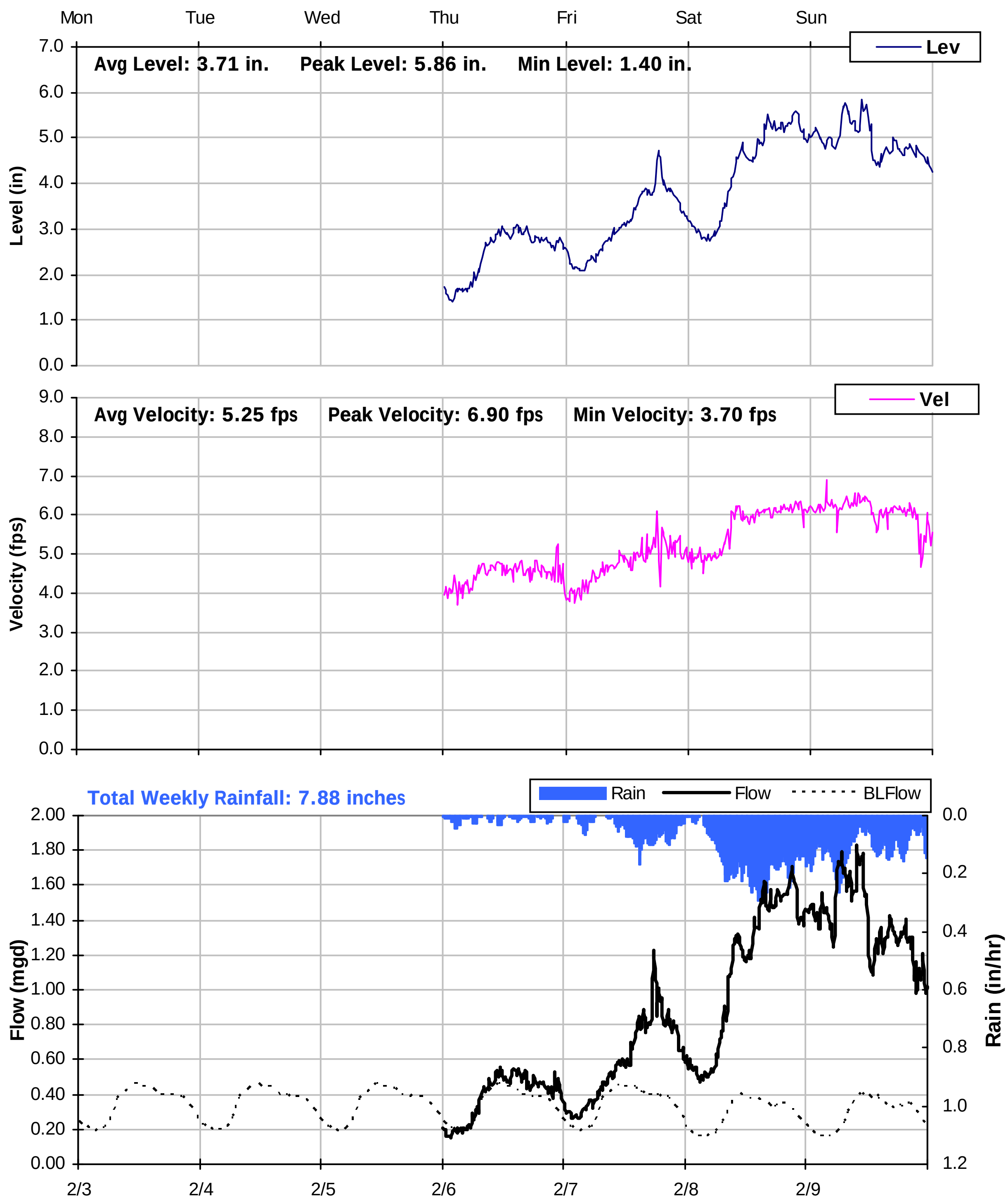
Event 2 Detail Graph



Storm Event I/I Analysis (Rain = 5.27 inches)

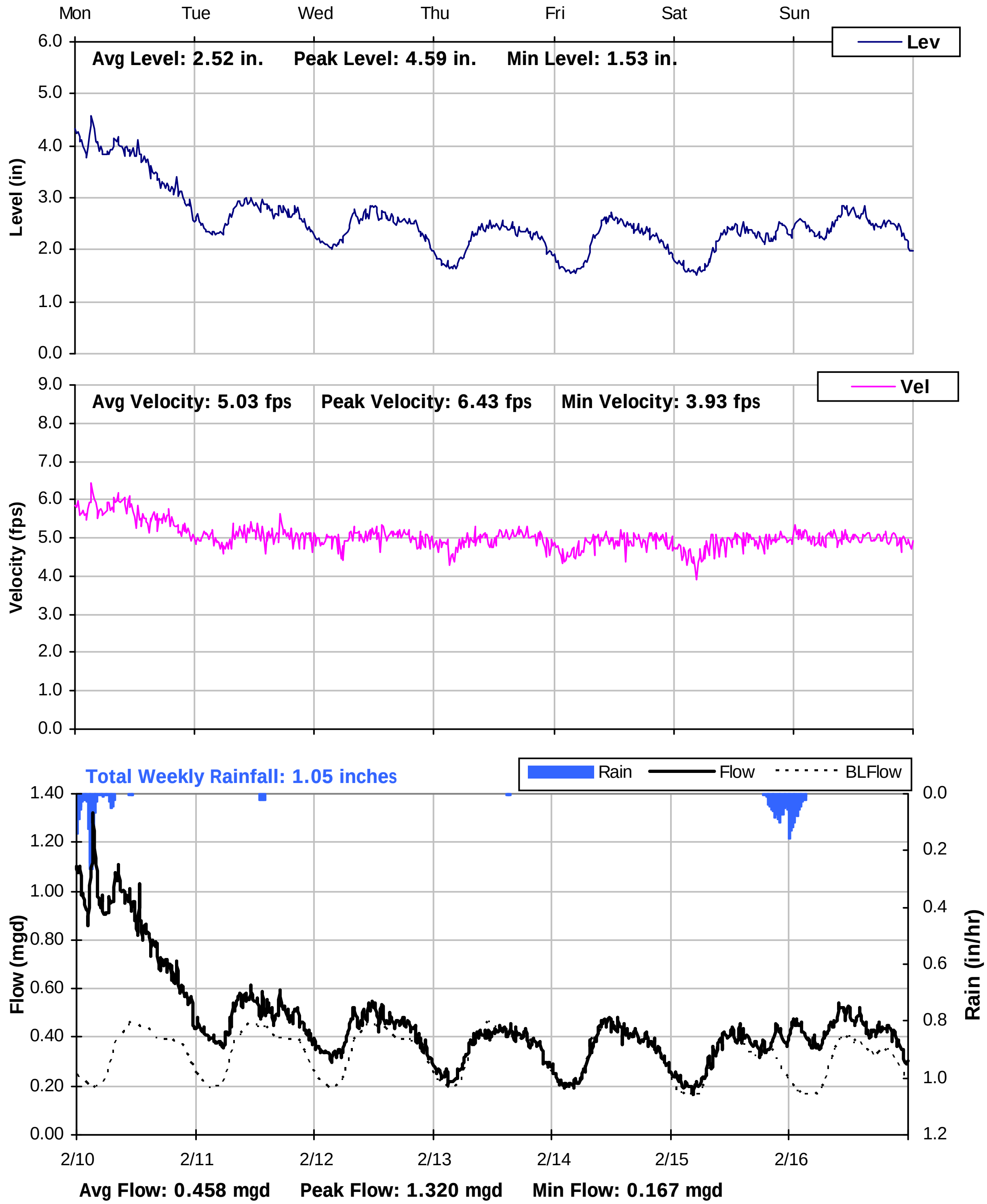
Capacity		**Inflow / Infiltration**	
Peak Flow:	1.60 *mgd*	**Peak I/I Rate:**	1.21 *mgd*
PF:	4.89	**Total I/I:**	2,462,000 *gallons*

SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/3/2014 to 2/10/2014

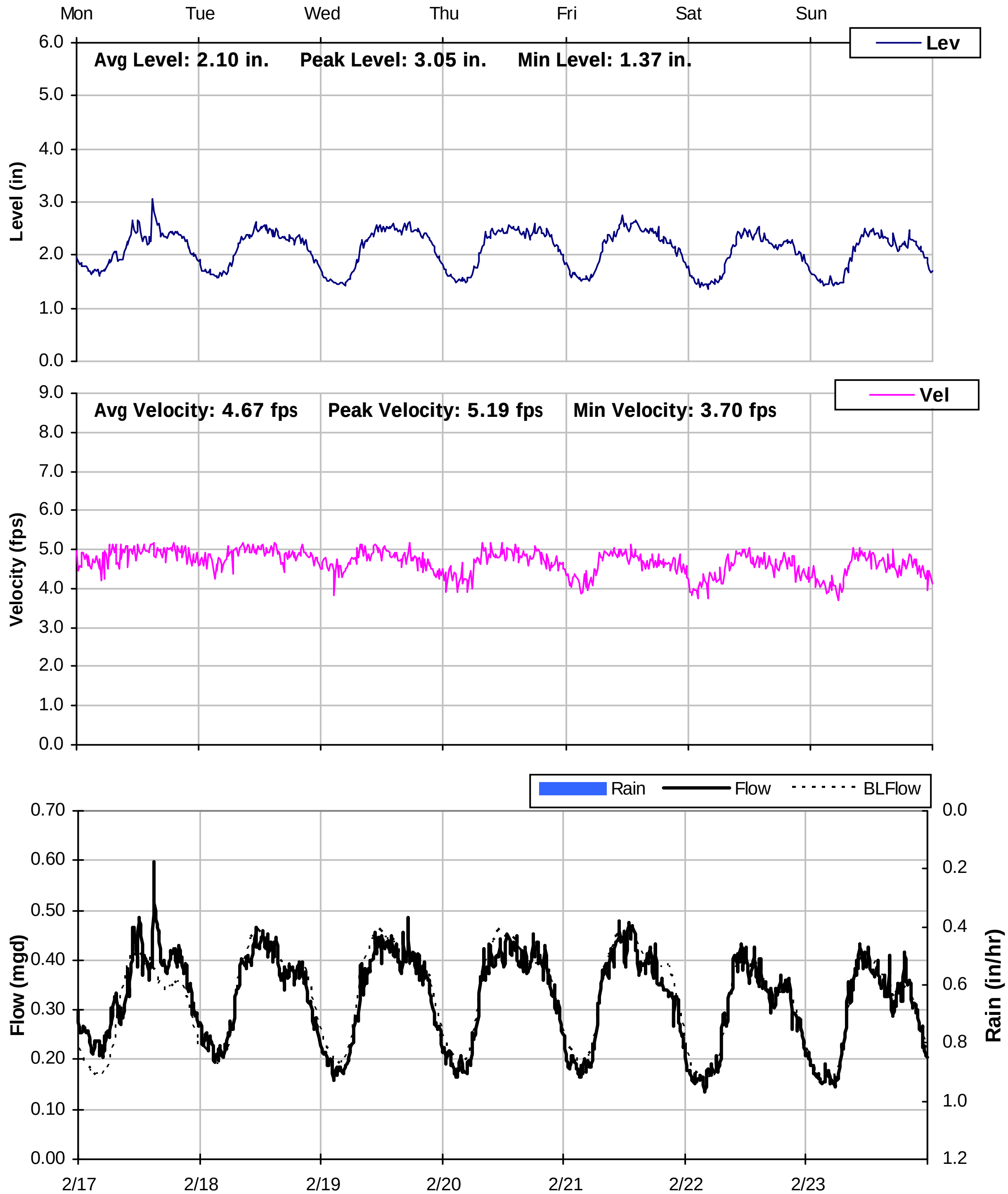


Avg Flow: 0.874 mgd **Peak Flow: 1.816 mgd** **Min Flow: 0.151 mgd**

SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/10/2014 to 2/17/2014

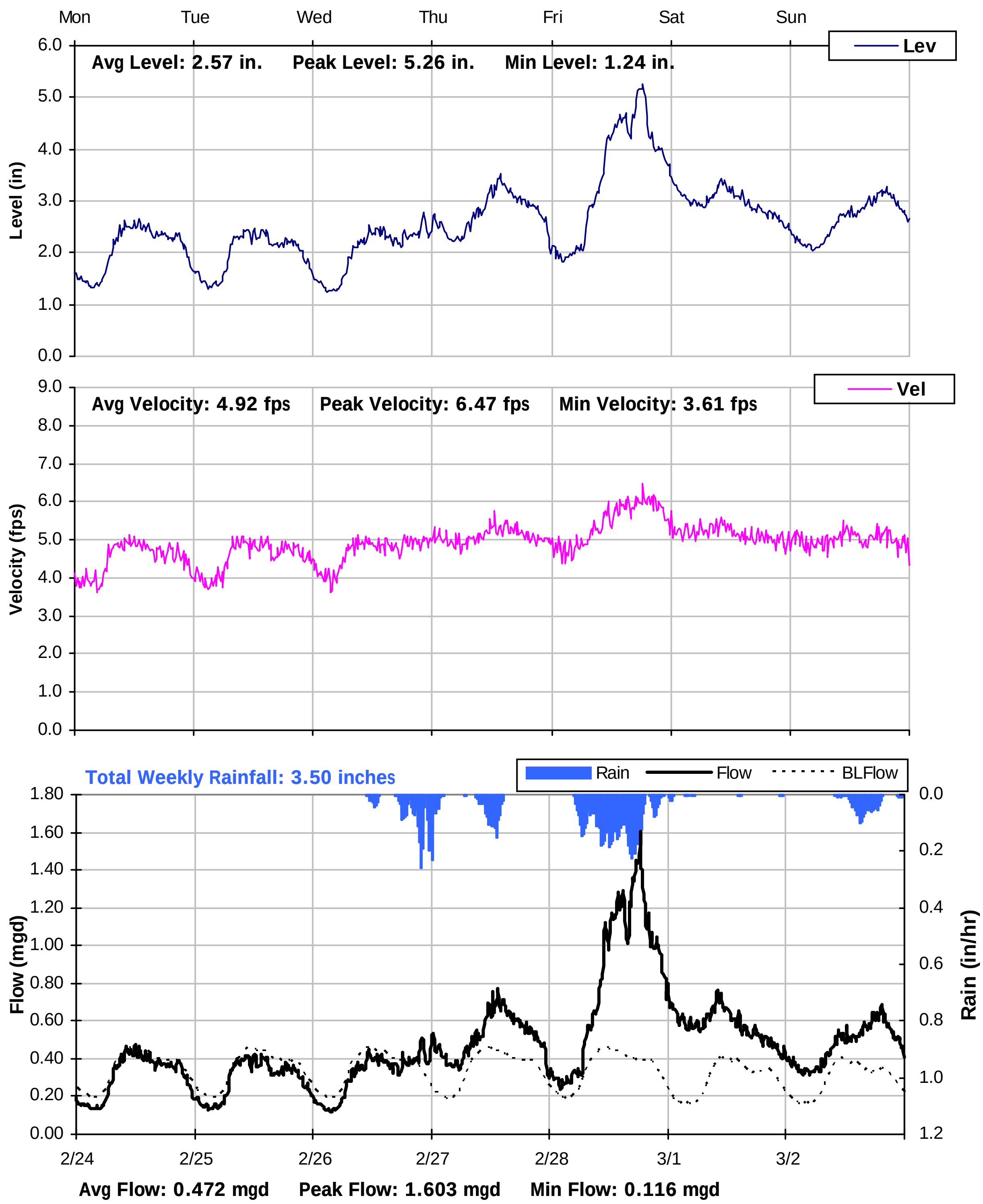


SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/17/2014 to 2/24/2014

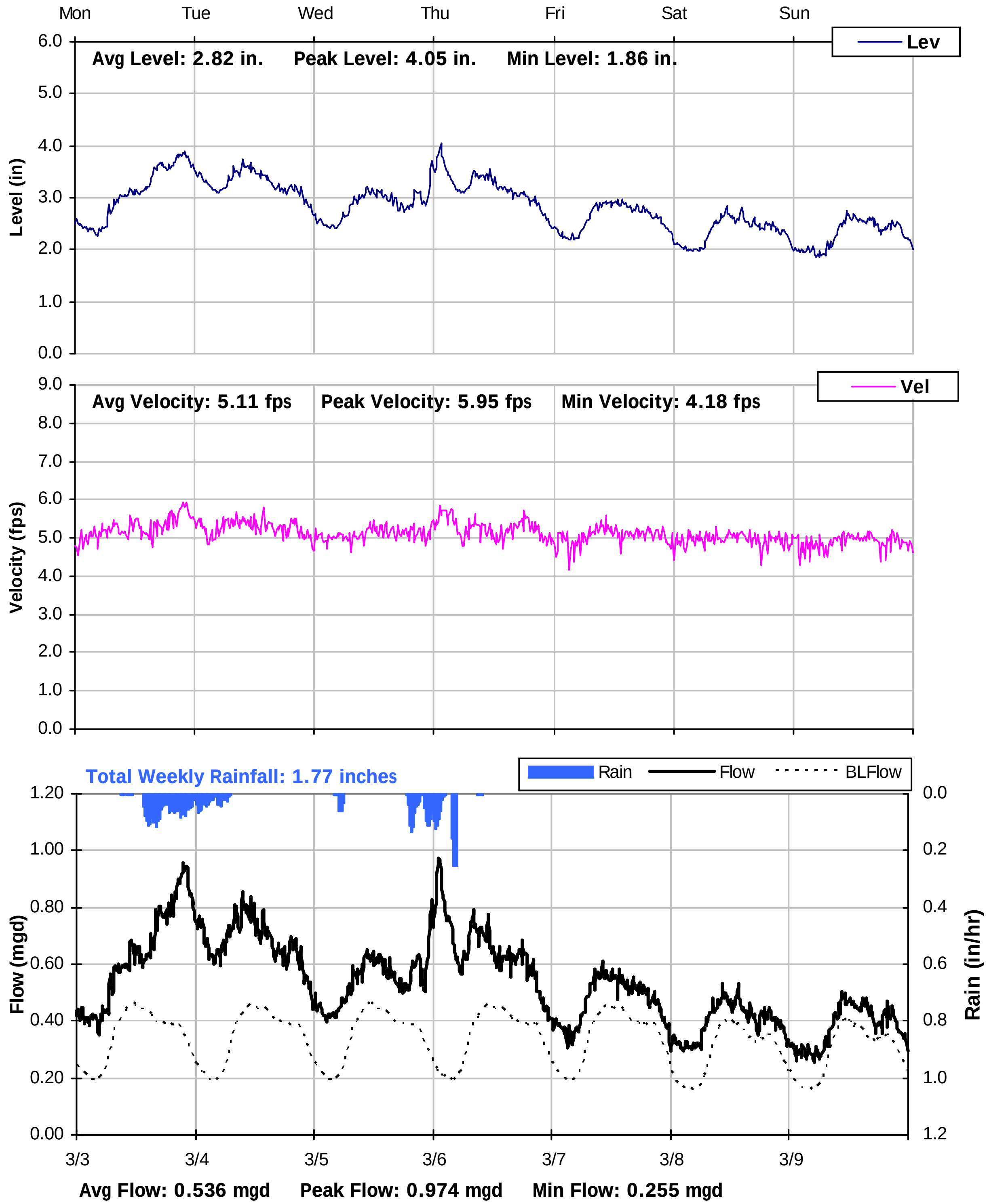


Avg Flow: 0.321 mgd **Peak Flow: 0.592 mgd** **Min Flow: 0.136 mgd**

SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



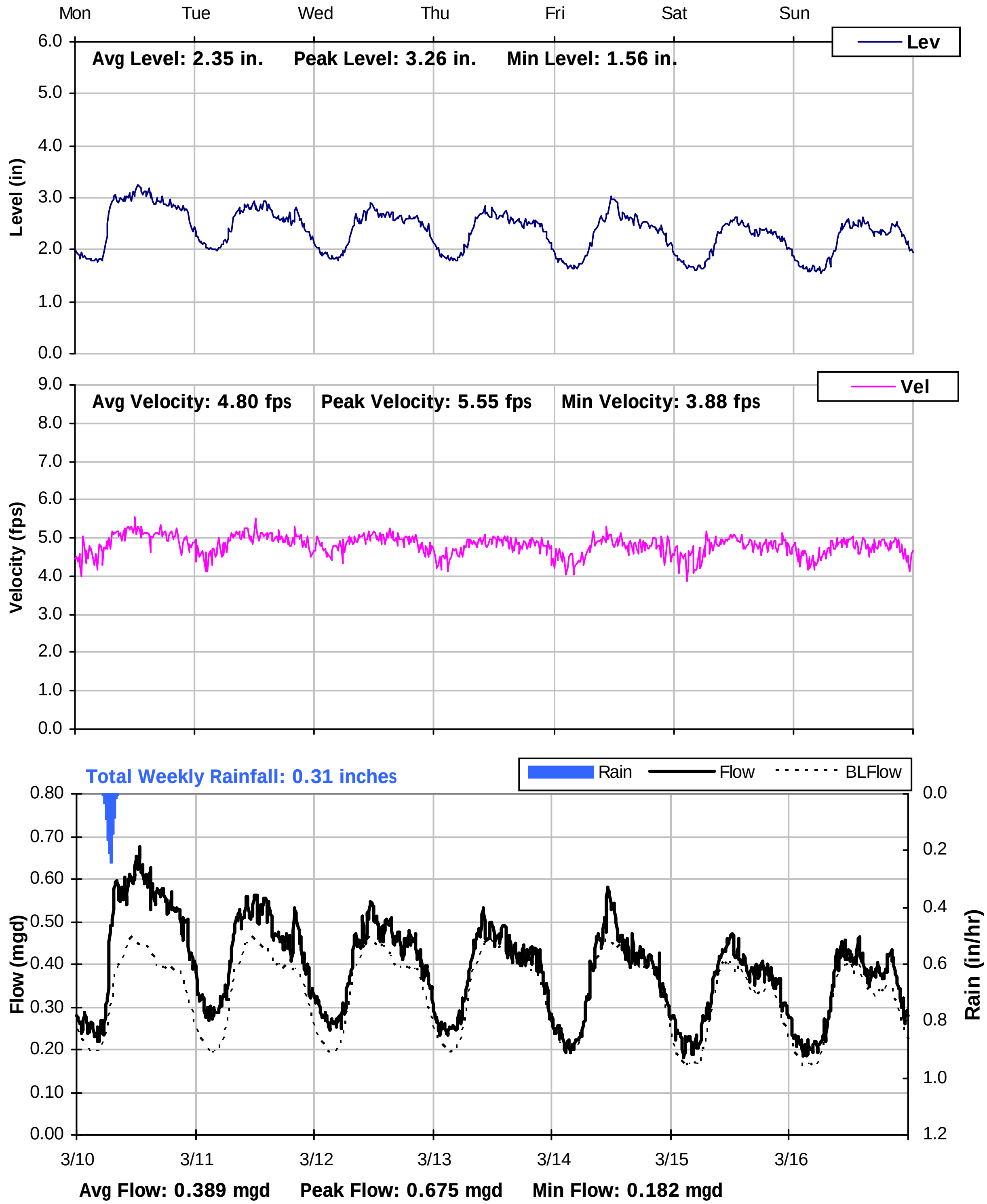
SITE 8
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



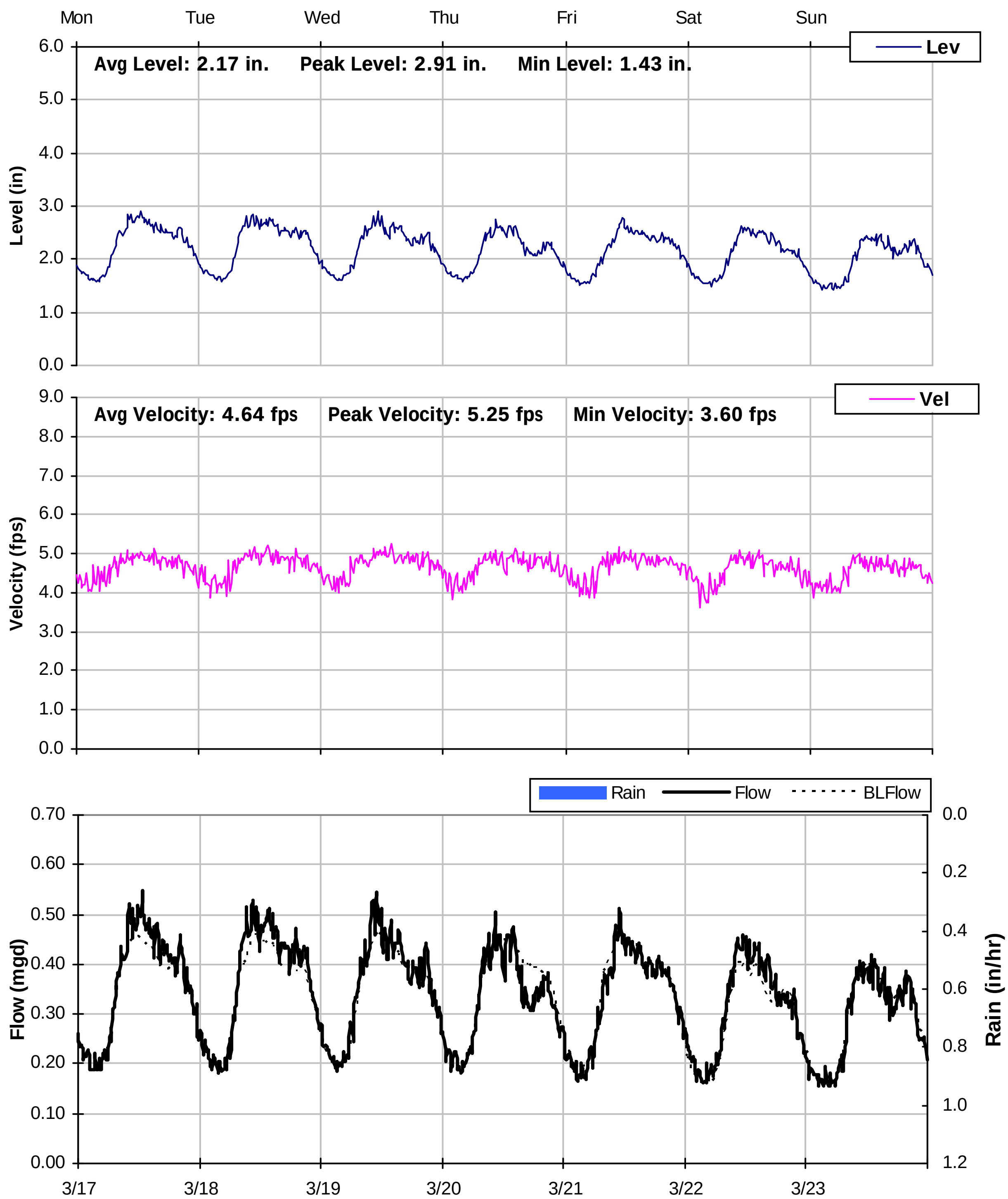
SITE 8

Weekly Level, Velocity and Flow Hydrographs

3/10/2014 to 3/17/2014

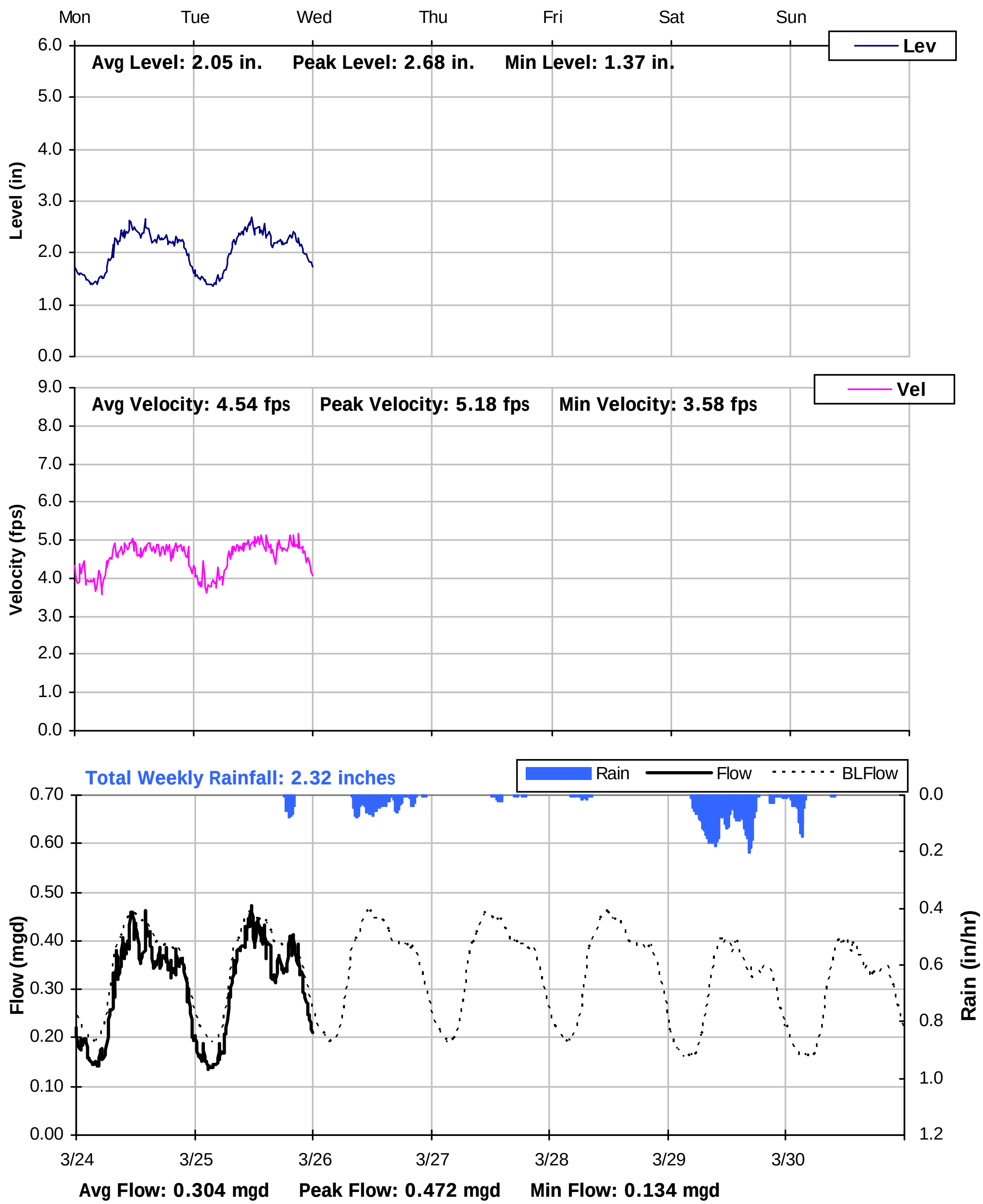


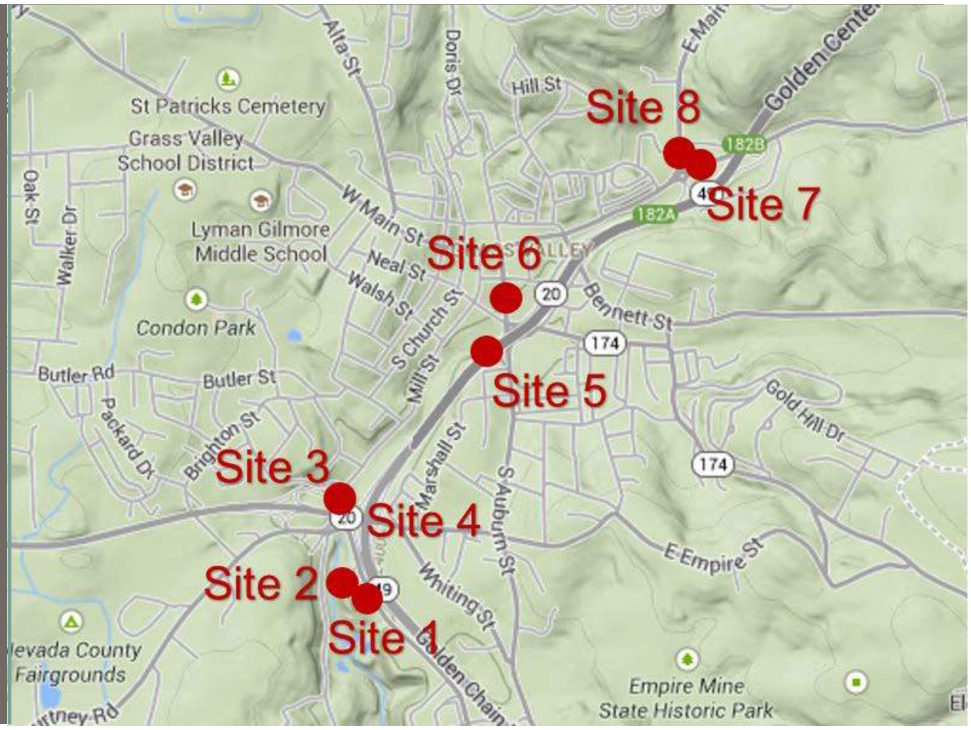
SITE 8
Weekly Level, Velocity and Flow Hydrographs
3/17/2014 to 3/24/2014



Avg Flow: 0.336 mgd Peak Flow: 0.547 mgd Min Flow: 0.155 mgd

SITE 8
Weekly Level, Velocity and Flow Hydrographs
3/24/2014 to 3/31/2014





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858.576.0226 **Tel**
858.576.0004 **Fax**

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8220 Jones Road, Suite 500
Houston, TX 77065
713.568.9067 **Tel**
713.568.9068 **Fax**

Las Vegas
3430 East Russell Road, Suite 316
Las Vegas, NV 89120
702.522.7967 **Tel**
702.553.4694 **Fax**

vaengineering.com

Appendix B MICROMONITORING PROGRAM

B.1 PHASE 1 (JUNE 2014, STANTEC)

B.2 PHASE 2 GWI STUDY (OCTOBER 2014, STANTEC)

Stantec Consulting Services Inc.
101 Providence Mine Road, Suite 202, Nevada City CA 95959

June 10, 2014

Attention: Tim Kiser, P.E., Public Works Director/City Engineer
City of Grass Valley – Engineering Division
125 East Main Street
Grass Valley, CA 95945

Dear Mr. Kiser,

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

SUMMARY OF FINDINGS

Inflow and infiltration (I/I) consists of stormwater and groundwater entering the sewer system through pipe defects and improper storm drainage connections. Typically, inflow is stormwater that enters the system through direct connections, such as roof downspouts, sump pumps in basements, driveway drains, and cross connections with storm drains; and infiltration is typically groundwater or groundwater influenced by surface water that enters the sewer pipes and manholes through joint separations, connection failures, missing pipe sections, breaks and other such openings. It is important to distinguish whether inflow or infiltration or if a combination of both are entering the City's collection system and at what locations, to allow the City to best assess possible corrective measures.

Flow monitoring is one tool used in conducting investigations to isolate the sources of I/I. This report summarizes the results of flow monitoring conducted in spring 2014 by Stantec Consulting Services, Inc. (Stantec) and V&A Consulting Engineers (V&A). This flow monitoring is being conducted as part of an overall wastewater system assessment Stantec is undertaking on behalf of the City, which will culminate in the development of a wastewater system master plan and capital improvement program.

In the City of Grass Valley, CA rainfall events cause a considerable increase in flow due to I/I in the sanitary sewer collection system. In the spring of 2014 a micromonitoring program was conducted in sewer collection system Basins 3 and 5. Data was collected for two weeks and two major storm events were analyzed. This effort was initiated based on flow monitoring data acquired in February and March of 2014 indicating wet weather peak flow was highest in these two basins. The methodology and results of this earlier monitoring are summarized in a report titled *Sanitary Sewer Flow Monitoring and I/I Report* (May 2014, V&A).

The results of the micromonitoring effort indicate that sub-basins S-3F, S-5C and, S-5D had the largest amount of I/I. Figure 1 shows the locations of the sub-basins and the color coding of the I/I results. Attachment A includes tables of the I/I calculations.

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

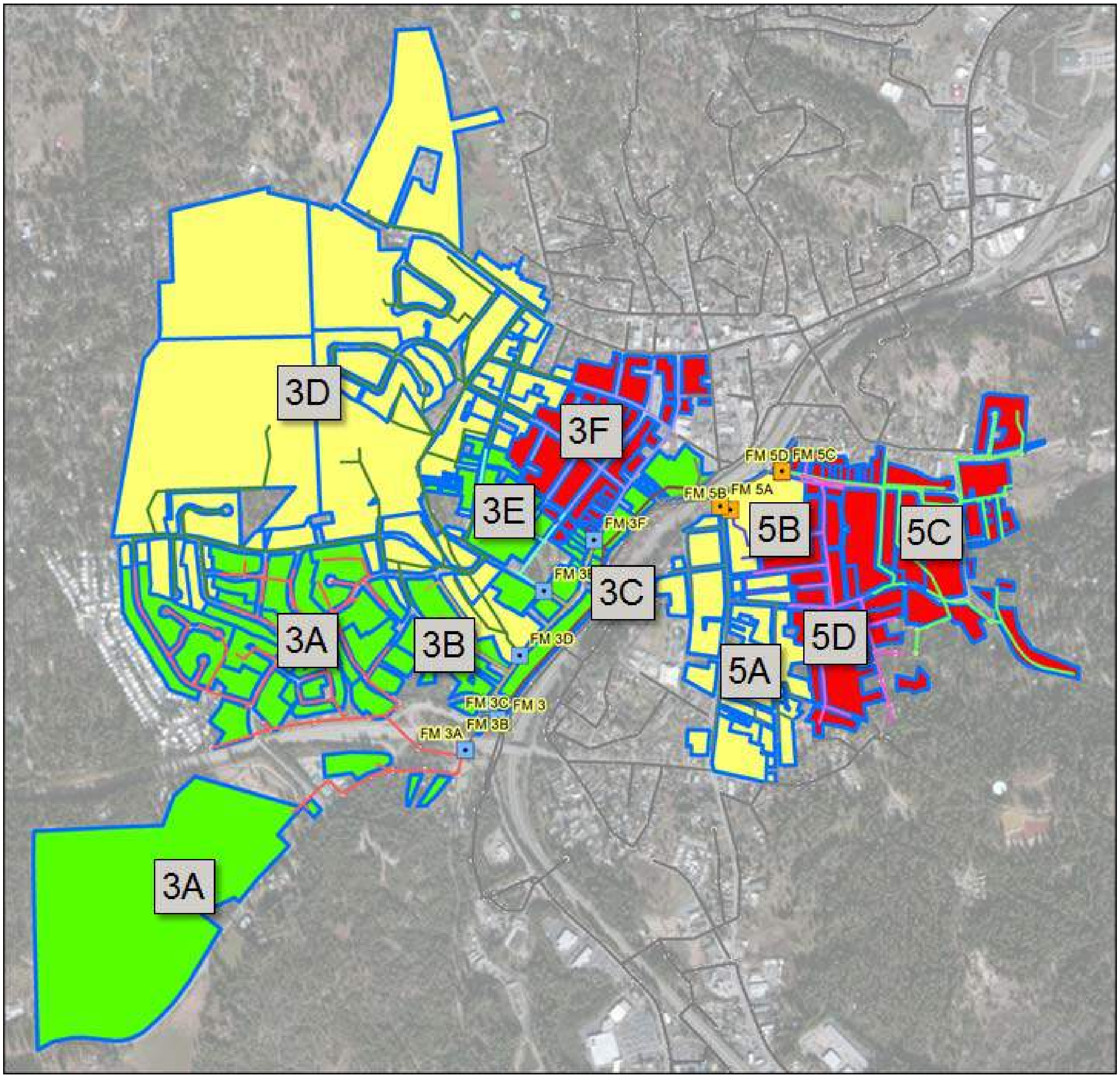


Figure 1: Summary of sub-basins warranting additional investigation. Red is high I/I. Yellow is medium I/I. Green is small I/I.

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

MICROMONITOR LOCATIONS

Six micromonitors were installed in Basin 3 and four micromonitors were installed in Basin 5. Both basins had a regional flow meter at the downstream end of the basin. A rain gauge was installed at the City wastewater treatment plant site by V&A and data was obtained from two weather stations owned and maintained by weather enthusiasts near the monitoring sites. **Figure 2a and 2b** show a schematic of the flow monitoring network for basins 3 and 5 respectively.

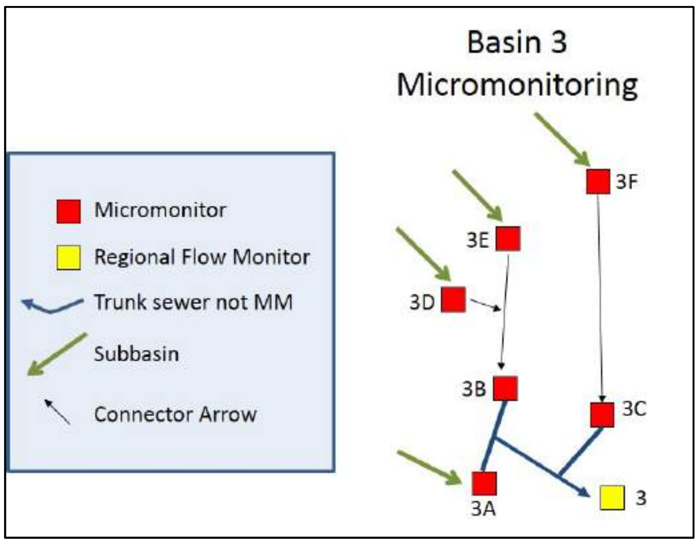


Figure 2a: Monitoring schematic for Basin 3

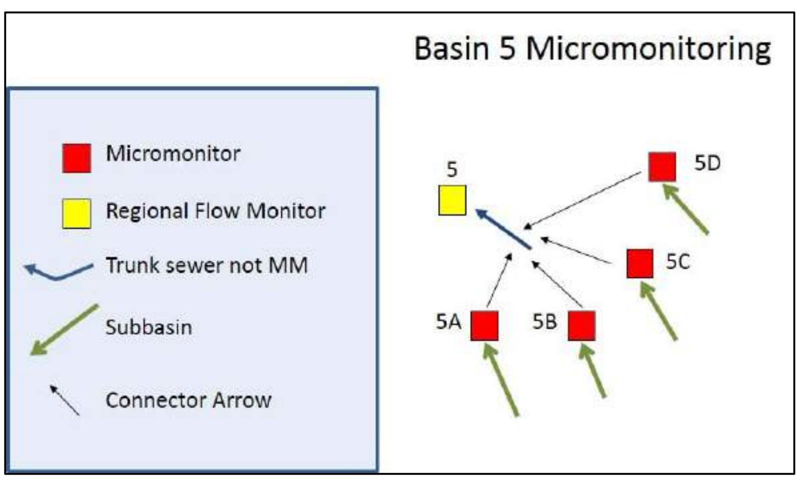


Figure 2b: Monitoring schematic for Basin 5

Table 1 provides the manhole number (using the City's numbering system) and location details for each monitor installed.

Table 1: Monitor Locations

Monitor ID/Manhole ID	Manhole	Pipe Size (in)	Area Monitored	Basin
FM3A/I17-8	I17-8	10	S-3A	Basin 3
MM3B/I17-16	I17-16	8	S-3B	Basin 3
FM3C/I17-5	I17-5	13.5	S-3C	Basin 3
MM3D/I16-22	I16-22	8	S-3D	Basin 3
MM3E/J16-3	J16-3	6	S-3E	Basin 3
MM3F/J15-14	J15-14	8	S-3F	Basin 3
FM03/I17-7	I17-7	15	S-3	Basin 3 – Regional Flow monitor
MM5A/K15-19	K15-19	6	S-5A	Basin 5
MM5B/K15-20	K15-20	6	S-5B	Basin 5
MM5C/L15-20	L15-20	8	S-5C	Basin 5
MM5D/K15-4	K15-4	6	S-5D	Basin 5
FM05/K15-5	K15-15	15	S-5	Basin 5 – Regional Flow monitor

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

Figure 3 shows the sub-basin delineations for the project area.

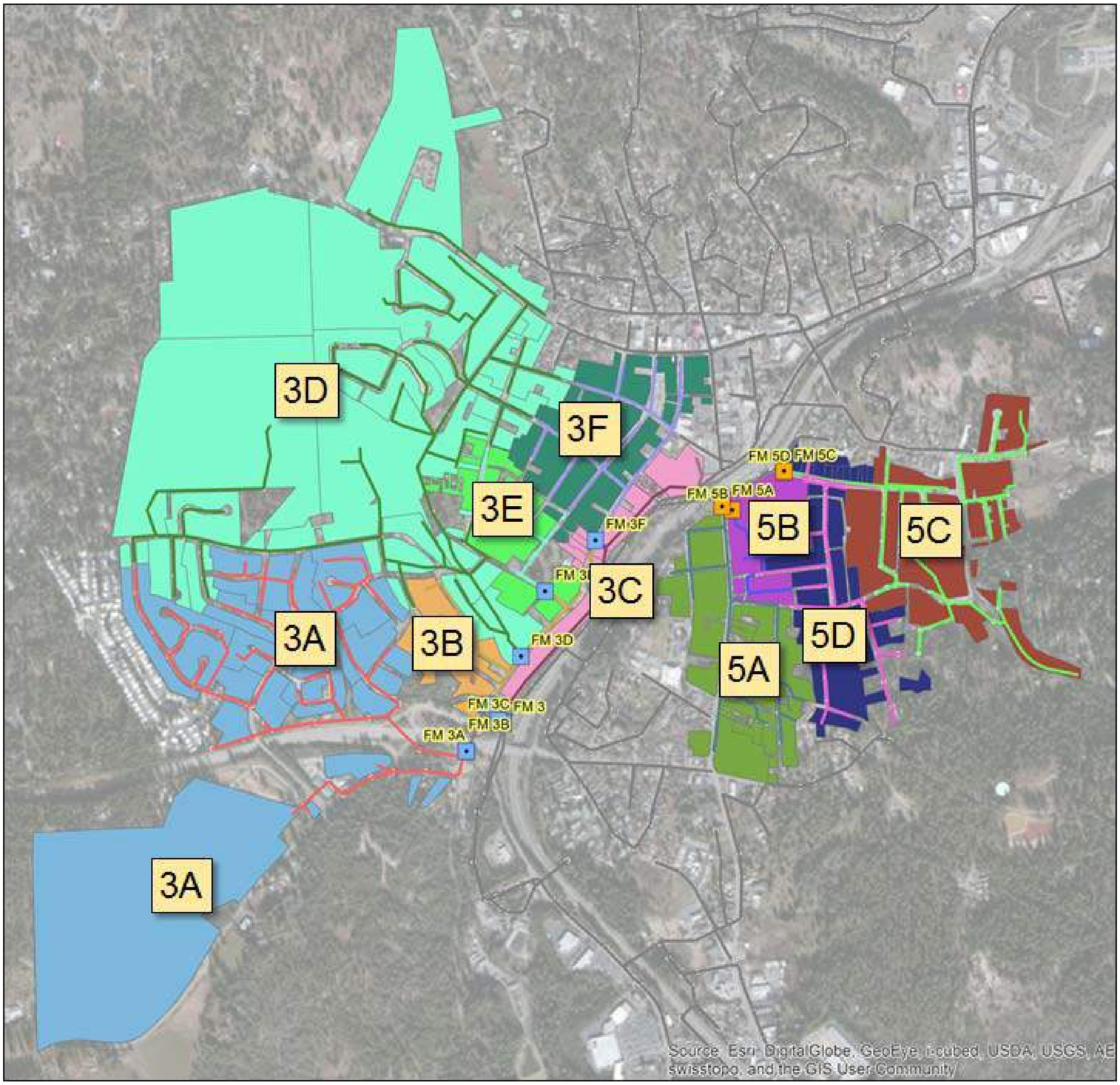


Figure 3: Sub-basins for Basins 3 and 5

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

DATES OF MONITOR INSTALLATION

Table 2 below provides the dates of installation and removal for the micromonitors.

Table 2: Monitor Installation Dates

Monitor ID/Manhole ID	Upstream Micromonitors	Date Installed	Date Removed	Basin
FM3A/I17-8		3/25/2014	4/9/2014	3
MM3B/I17-16	3D, 3E	3/25/2014	4/9/2014	3
FM3C/I17-5	3F	3/25/2014	4/9/2014	3
MM3D/I16-22		3/26/2014	4/9/2014	3
MM3E/J16-3		3/25/2014	4/9/2014	3
MM3F/J15-14		3/26/2014	4/9/2014	3
MM5A/K15-19		3/25/2014	4/9/2014	5
MM5B/K15-20		3/26/2014	4/9/2014	5
MM5C/L15-20		3/25/2014	4/9/2014	5
MM5D/K15-4		3/26/2014	4/9/2014	5

DATA COLLECTED

The graphs of the data from major storms analyzed are included in Attachment B. The complete data is available in the SFM software provided under separate cover.

SUMMARY OF I/I ANALYSIS

Table 3 provides a summary of the I/I analysis results, which are shown graphically in **Figure 1**. A more complete table of the analysis is included in Attachment A. Two storms were used for the I/I analysis: 1.95 inches on March 29 2014 and 1.21 inches on March 31 2014. In Basin 3, sub-basin S-3F had the highest rate of I/I and sub-basin 3D also showed a moderate I/I response. In Basin 5, sub-basins S-5C and S-5D together contributed almost all of the I/I; sub-basins 5A and 5B showed moderate I/I.

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

Table 3: Summary of Micromonitoring Results by Sub-basin

Subbasin	Percentage of I/I	Effective Width of I/I (ft)	Comments
S-3A	3%	1	Very small I/I
S-3B	0%	0	Most of the I/I at this meter comes from 3D; 3E has very little or no contribution.
S-3C	0%	0	Possible loss of flow due to an overflow between 3C and 3F, making the flow at 3C smaller than 3F.
S-3D	30%	9	Some moderate I/I in this sub-basin.
S-3E	1%	2	Very small I/I
S-3F	66%	68	Significant I/I. Almost all of the I/I in Basin 3 comes from this area.
S-3	0%	0	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.
S-5A	9%	3	Some I/I
S-5B	5%	6	Some I/I
S-5C	42%	10	Significant I/I
S-5D	43%	15	Significant I/I
S-5	0%	0	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.

Table 3 above provides two metrics of I/I. The Percent of I/I is the percentage of the total I/I measured originating within the individual sub-basin. The Effective Width of I/I is found by dividing the Total Volume of I/I during a storm by the Depth of Rainfall and the Length of Pipe in the sub-basin. A comparison of the data from two sites is presented in **Figure 5**.

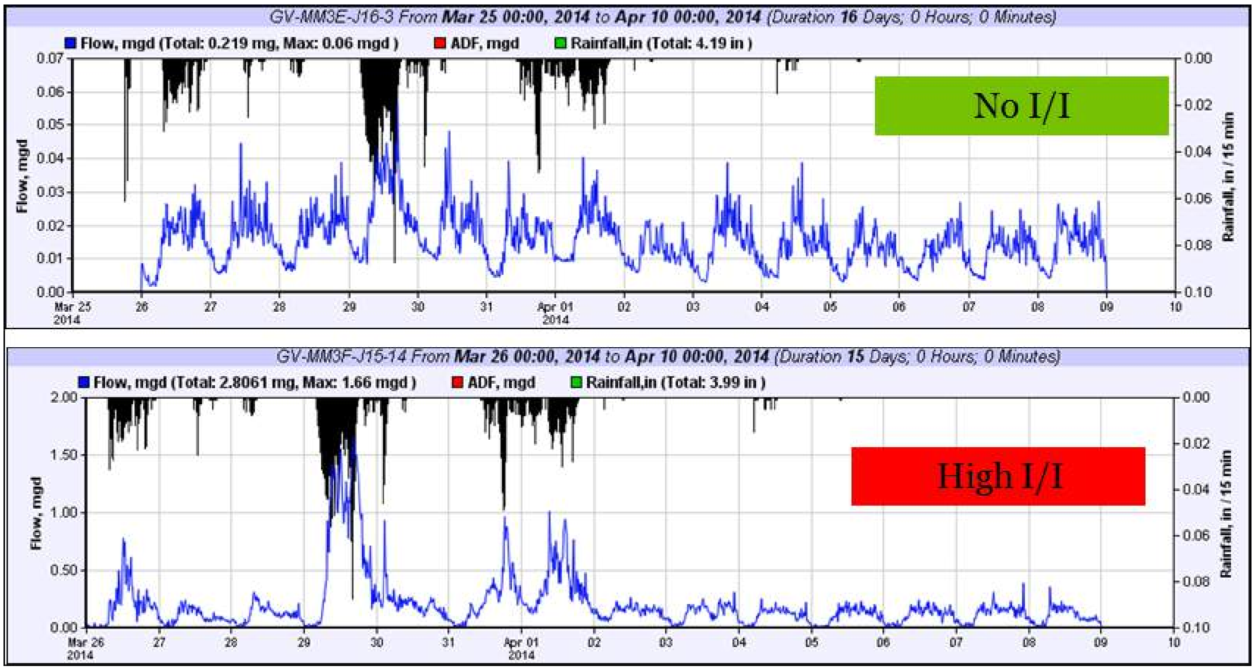


Figure 5: Comparison of I/I response at two basins in a small storm

RECOMMENDATIONS

It has been determined by the City that additional flow monitoring during late spring/early summer 2014 in Basins 5, 6 and 8 is to be undertaken. This is largely due to City staff concerns with possible infiltration or other water entering the system not entirely of wastewater origins. The details of this monitoring and findings will be summarized in a separate report.

In addition to the dry weather evaluation in Basins 5, 6 and 8, it is recommended that additional wet weather micromonitoring take place in Basin(s) 3 and 5. Specifically sub-basins 3F, 5C and 5D should be prioritized based on the initial results presented here.

Reference: Results from Grass Valley Micromonitoring Program – Phase 1

Regards,

STANTEC CONSULTING SERVICES INC.

Dave Price
Senior Engineer, Water
Phone: (530) 470-0515
dave.w.price@stantec.com

ATTACHMENT A – I/I CALCULATION TABLES FOR BASINS 3 & 5

Basin 3

Subbasin	Percentage of I/I	Effective Width of I/I (ft)	Comments
S-3A	3%	1	No I/I
S-3B	0%	0	Most of the I/I comes from 3D; 3E has very little or no contribution.
S-3C	0%	0	Possible loss of flow due to an overflow between 3C and 3F, making the flow at 3C smaller than 3F.
S-3D	30%	9	Some moderate I/I in this sub-basin.
S-3E	1%	2	No I/I
S-3F	66%	68	Significant I/I. Almost all of the I/I in Basin 3 comes from this area.
S-3	0%	0	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.

Grass Valley Basin 3 Subbasin	Upstream Meters	Total Upstream Pipe Length (ft)	Sub-Basin Pipe Length (ft)
S-3A		16,414.0	16,414.0
S-3B	3D,3E	31,886.0	4,076.0
S-3C	3F	10,093.0	2,896.0
S-3D		25,556.0	25,556.0
S-3E		2,254.0	2,254.0
S-3F		7,197.0	7,197.0
S-3	3A,3B,3C	59,380.0	987.0

Grass Valley Basin 3	Total Upstream Storm Volume (Qstorm from meter 24 hr max) (Gallons)		Sub-basin Storm Volume (Gallons)		Total Upstream Effective Area (acres)	
	3/29/14	3/31/14	3/29/14	3/31/14	3/29/14	3/31/14
	1.95	1.21	1.95	1.21	1.95	1.21
Subbasin	Gallons	Gallons	Gallons	Gallons	Acres	Acres
S-3A	36,650	12,950	36,650	12,950	0.69	0.39
S-3B	194,800	67,420	93,610	-217,960	3.68	2.05
S-3C	279,100	116,200	-384,200	-212,200	5.27	3.54
S-3D	89,370	285,100	89,370	285,100	1.69	8.68
S-3E	11,820	280	11,820	280	0.22	0.01
S-3F	663,300	328,400	663,300	328,400	12.53	10.00
S-3	346,900	99,250	-163,650	-97,320	6.55	3.02

Grass Valley Basin 3 Subbasin	Average Total Upstream Effective Area (Acres)	Average Sub-Basin Effective Area (Acres)	Sub-Basin Percentage of I/I	Effective Width of I/I (ft)	Percentage I/I per Percentage of Pipe	Color Coding of Rehab Recommendation	Rehab Recommendation	Comments
S-3A	0.54	0.54	3%	1	0.1	Green	**No action**	No I/I
S-3B	2.87	0.00	0%	0	0.0	Green	**No action**	Most of the I/I comes from 3D; 3E has very little or no contribution.
S-3C	4.40	0.00	0%	0	0.0	Green	**No action**	Possible loss of flow due to an overflow between 3C and 3F, making the flow at 3C smaller than 3F.
S-3D	5.18	5.18	30%	9	0.7	Yellow	**Maybe**	Some moderate I/I in this sub-basin.
S-3E	0.12	0.12	1%	2	0.2	Green	**No action**	No I/I
S-3F	11.26	11.26	66%	68	5.4	Red	**Investigate**	Significant I/I. Almost all of the I/I in Basin 3 comes from this area.
S-3	4.79	0.00	0%	0	0.0	Green	**No action**	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.

Basin 5

Subbasin	Percentage of I/I	Effective Width of I/I (ft)	Comments
S-5A	9%	3	Some I/I
S-5B	5%	6	Some I/I
S-5C	42%	10	Significant I/I
S-5D	43%	15	Significant I/I
S-5	0%	0	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.

Grass Valley Basin 5 Subbasin	Upstream Meters	Total Upstream Pipe Length (ft)	Sub-Basin Pipe Length (ft)
S-5A		6,501.0	6,501.0
S-5B		1,970.0	1,970.0
S-5C		10,685.0	10,685.0
S-5D		7,039.0	7,039.0
S-5	5A, 5B,5C,5D	28,119.0	1,924.0

Grass Valley Basin 5	Total Upstream Storm Volume (Qstorm from meter 24 hr max) (Gallons)		Sub-basin Storm Volume (Gallons)		Total Upstream Effective Area (acres)	
	3/29/14	3/31/14	3/29/14	3/31/14	3/29/14	3/31/14
	1.95	1.21	1.95	1.21	1.95	1.21
Subbasin	Gallons	Gallons	Gallons	Gallons	Acres	Acres
S-5A	21,050	20,430	21,050	20,430	0.40	0.62
S-5B	20,190	6,103	20,190	6,103	0.38	0.19
S-5C	148,200	61,720	148,200	61,720	2.80	1.88
S-5D	165,400	54,300	165,400	54,300	3.12	1.65
S-5	288,900	109,500	-65,940	-33,053	5.46	3.33

Grass Valley Basin 5 Subbasin	Average Total Upstream Effective Area (Acres)	Average Sub-Basin Effective Area (Acres)	Sub-Basin Percentage of I/I	Effective Width of I/I (ft)	Percentage I/I per Percentage of Pipe	Largest I/I Volume per length of pipe (gal/ft)	Color Coding of Rehab Recommendation	Rehab Recommendation	Comments
S-5A	0.51	0.51	9%	3	0.4	3	Yellow	**Maybe**	Some I/I
S-5B	0.28	0.28	5%	6	0.7	10	Yellow	Maybe	Some I/I
S-5C	2.34	2.34	42%	10	1.1	14	Red	**Investigate**	Significant I/I
S-5D	2.39	2.39	43%	15	1.7	23	Red	**Investigate**	Significant I/I
S-5	4.39	0.00	0%	0	0.0	-17	Green	No action	Upstream flows exceed flow at this meter due to possible overflow or result of meter subtraction.

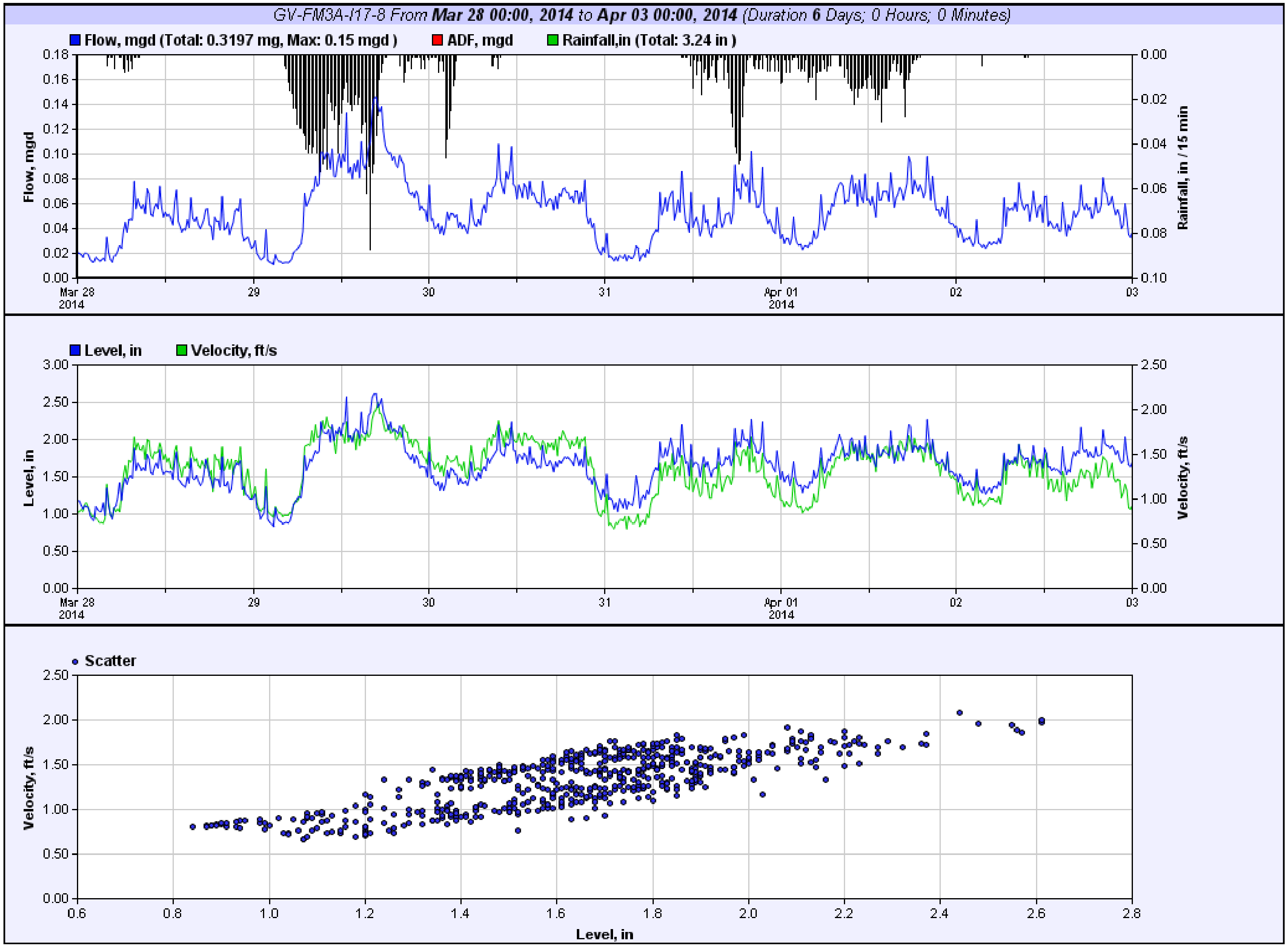
Reference: Results from Grass Valley Micromonitoring Program – Phase 1

ATTACHMENT B – MICROMONITOR GRAPHS DURING KEY STORM EVENTS FOR BASINS 3 & 5

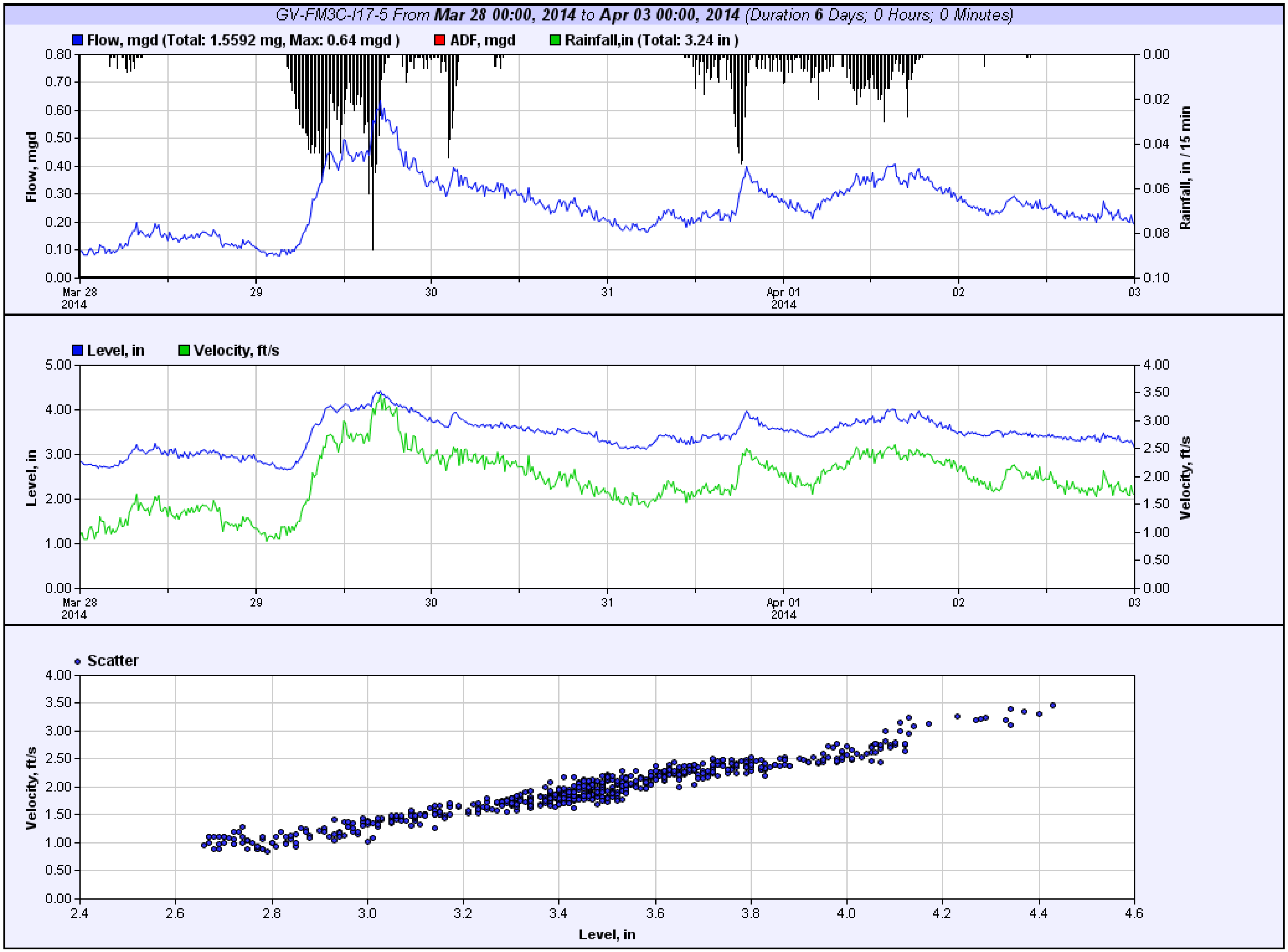
Basin 3

Micromonitor Graphs During Key Storms

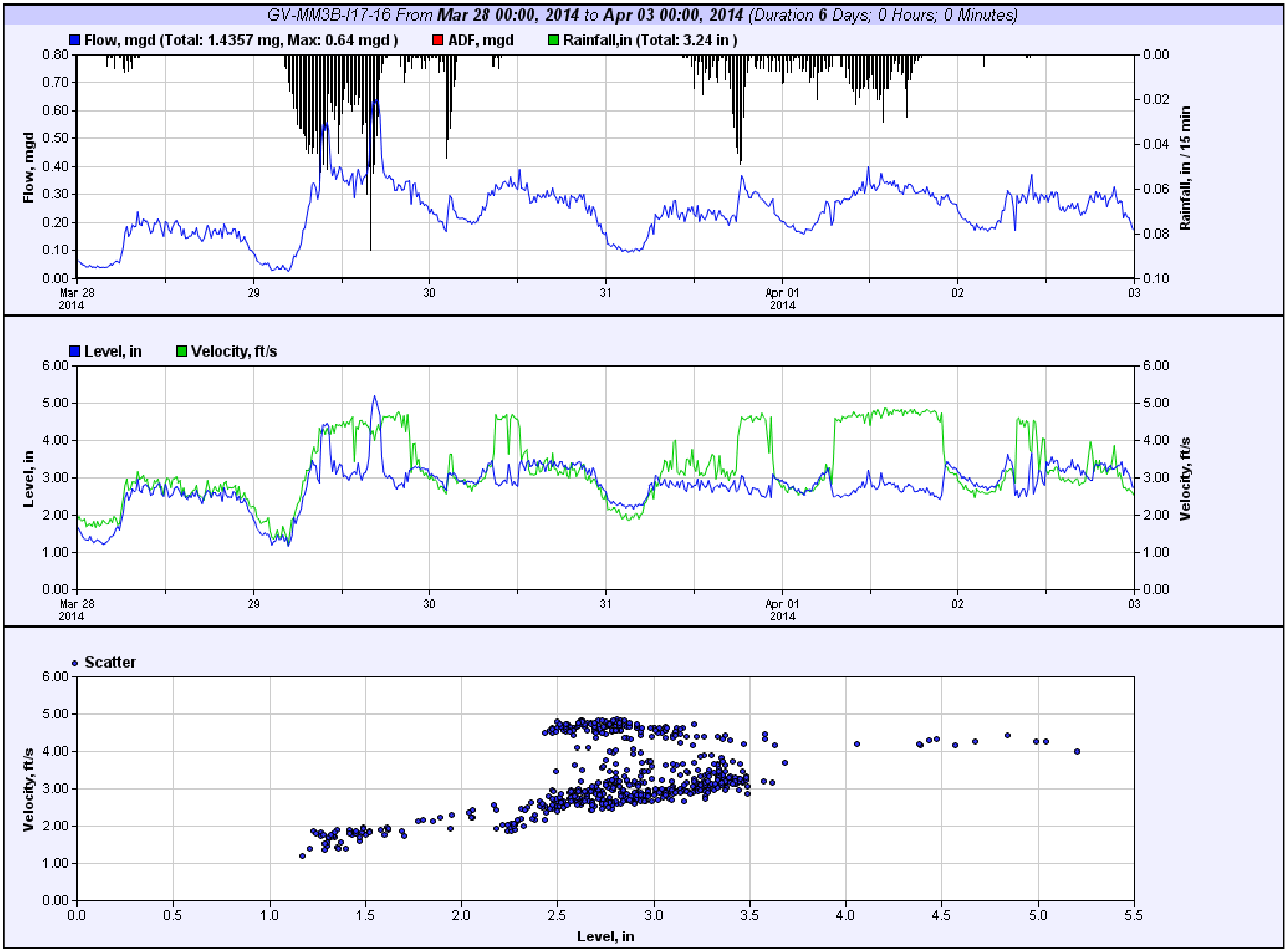
GV-FM3A-I17-8
March 29 and 31 Storms



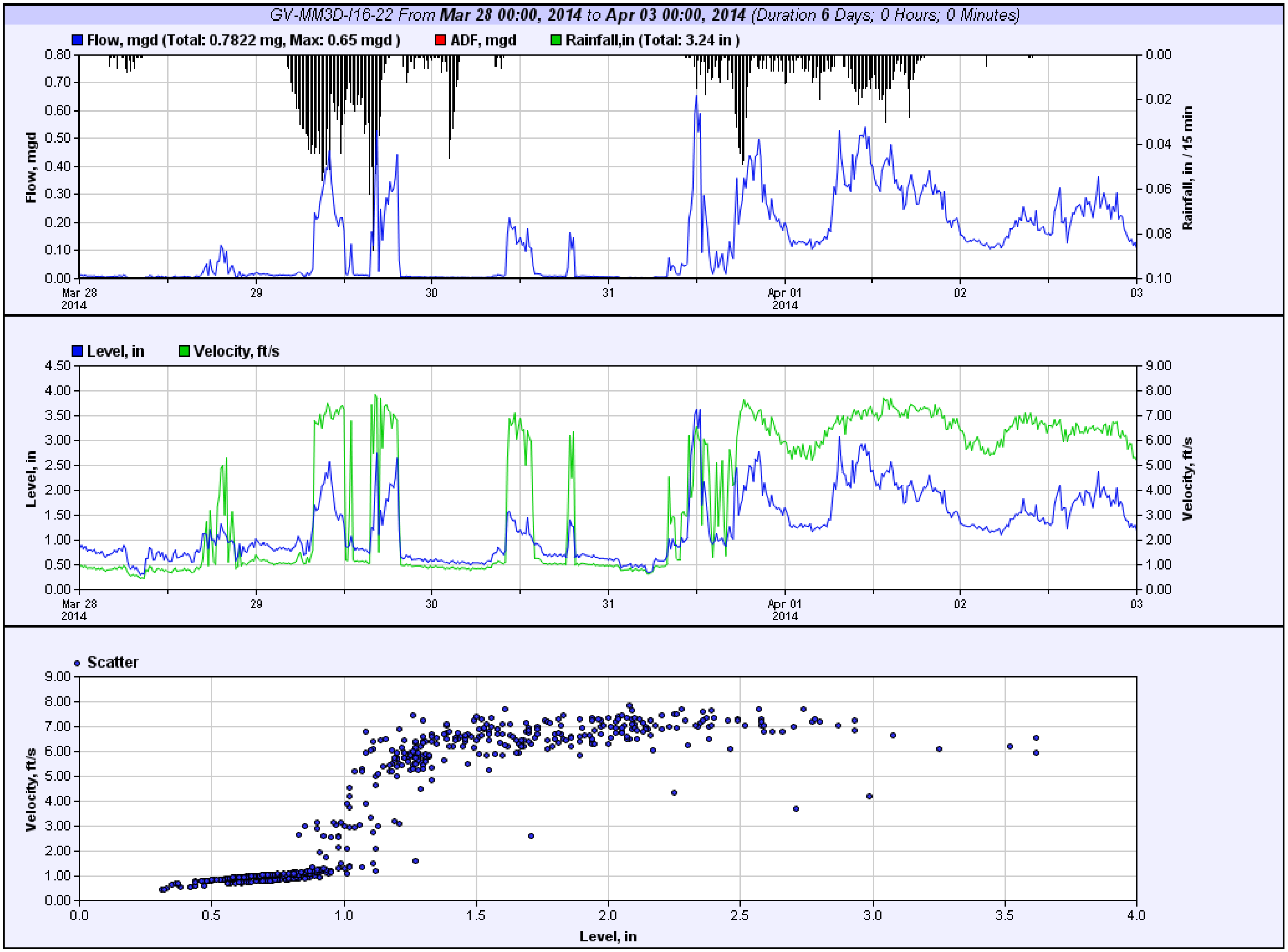
GV-FM3C-I17-5
March 29 and 31 Storms



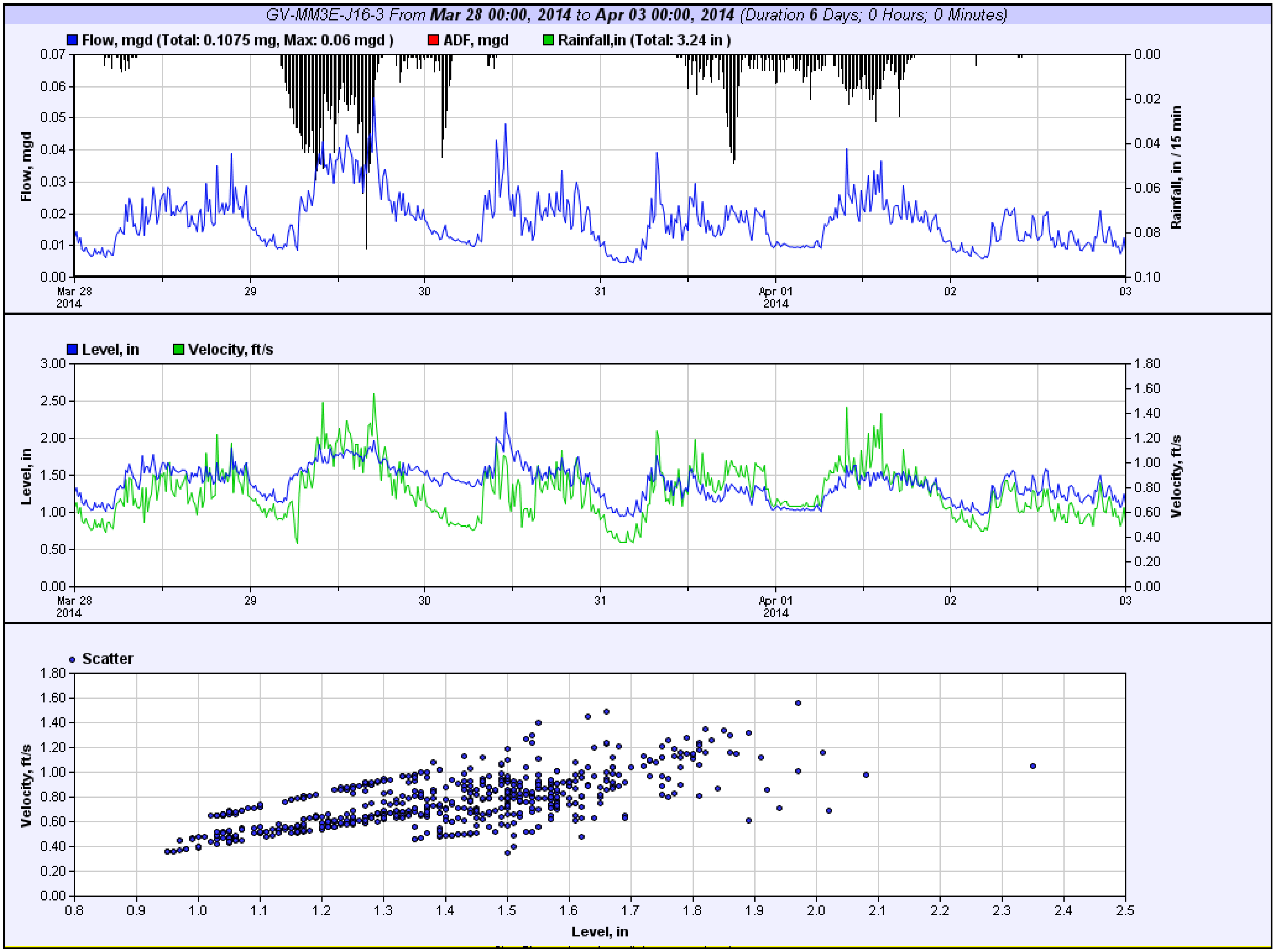
GV-MM3B-I17-16
March 29 and 31 Storms



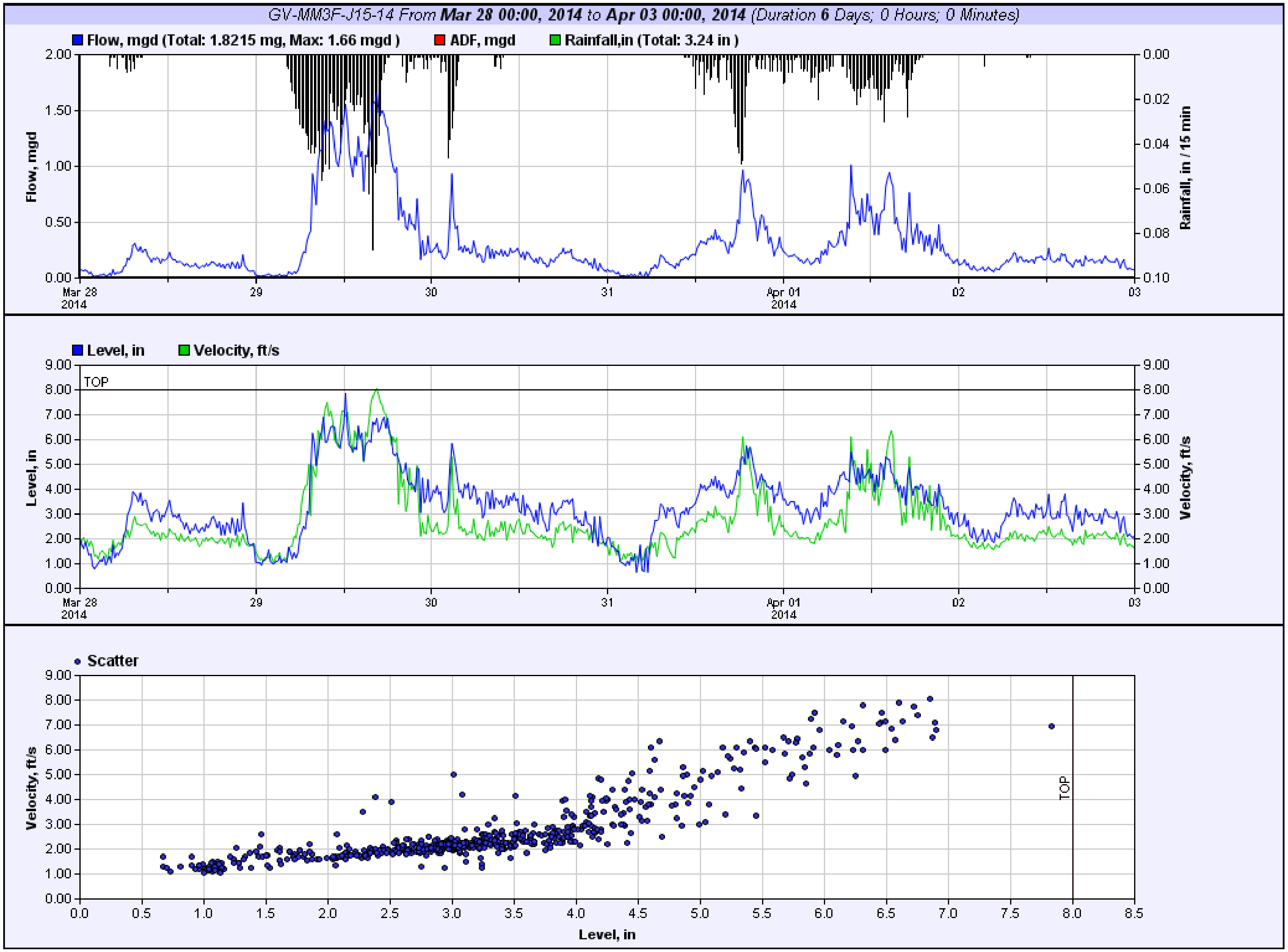
GV-MM3D-I16-22
March 29 and 31 Storms



GV-MM3E-J16-3
March 29 and 31 Storms



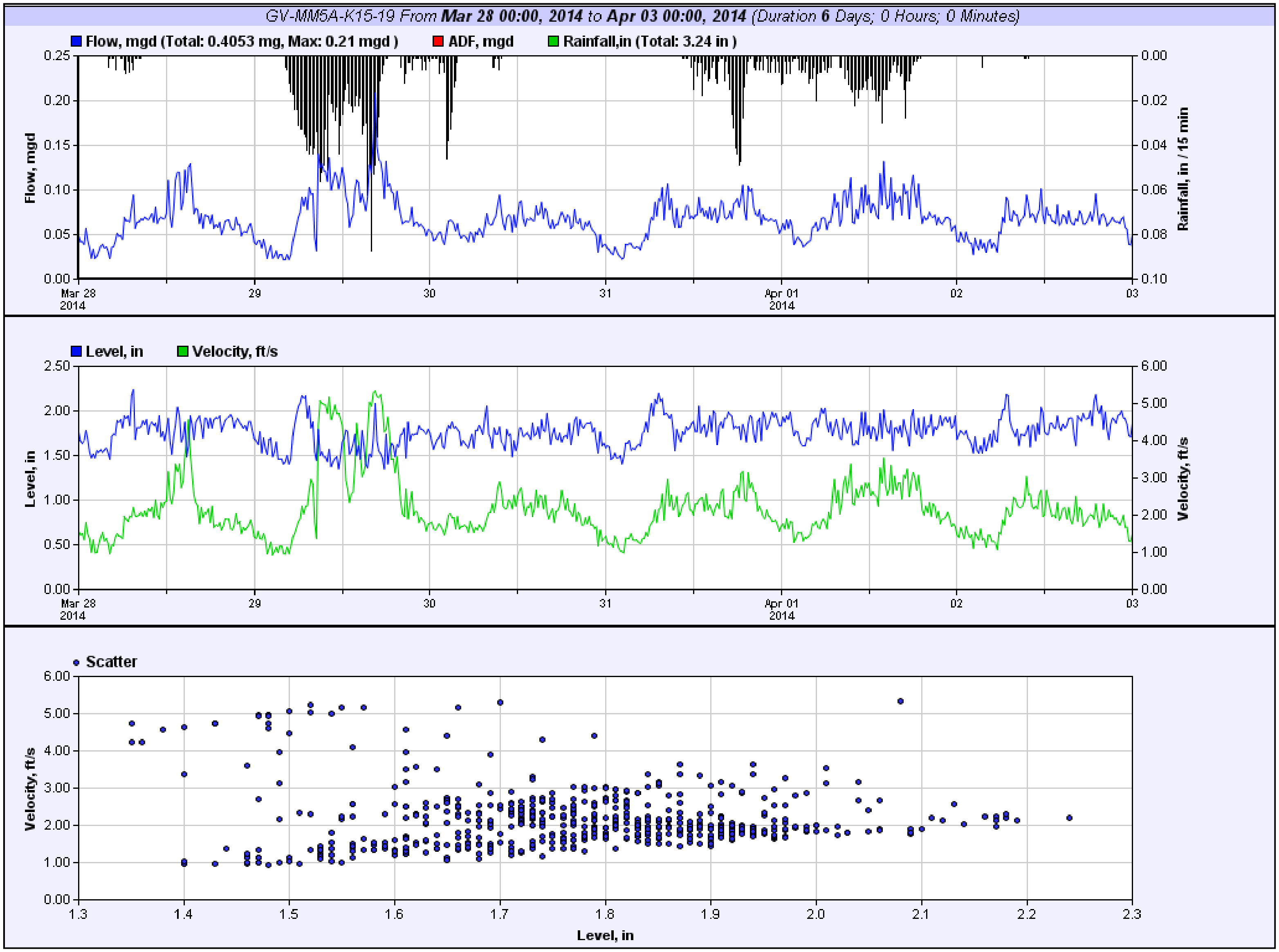
GV-MM3F-J15-14
March 29 and 31 Storms



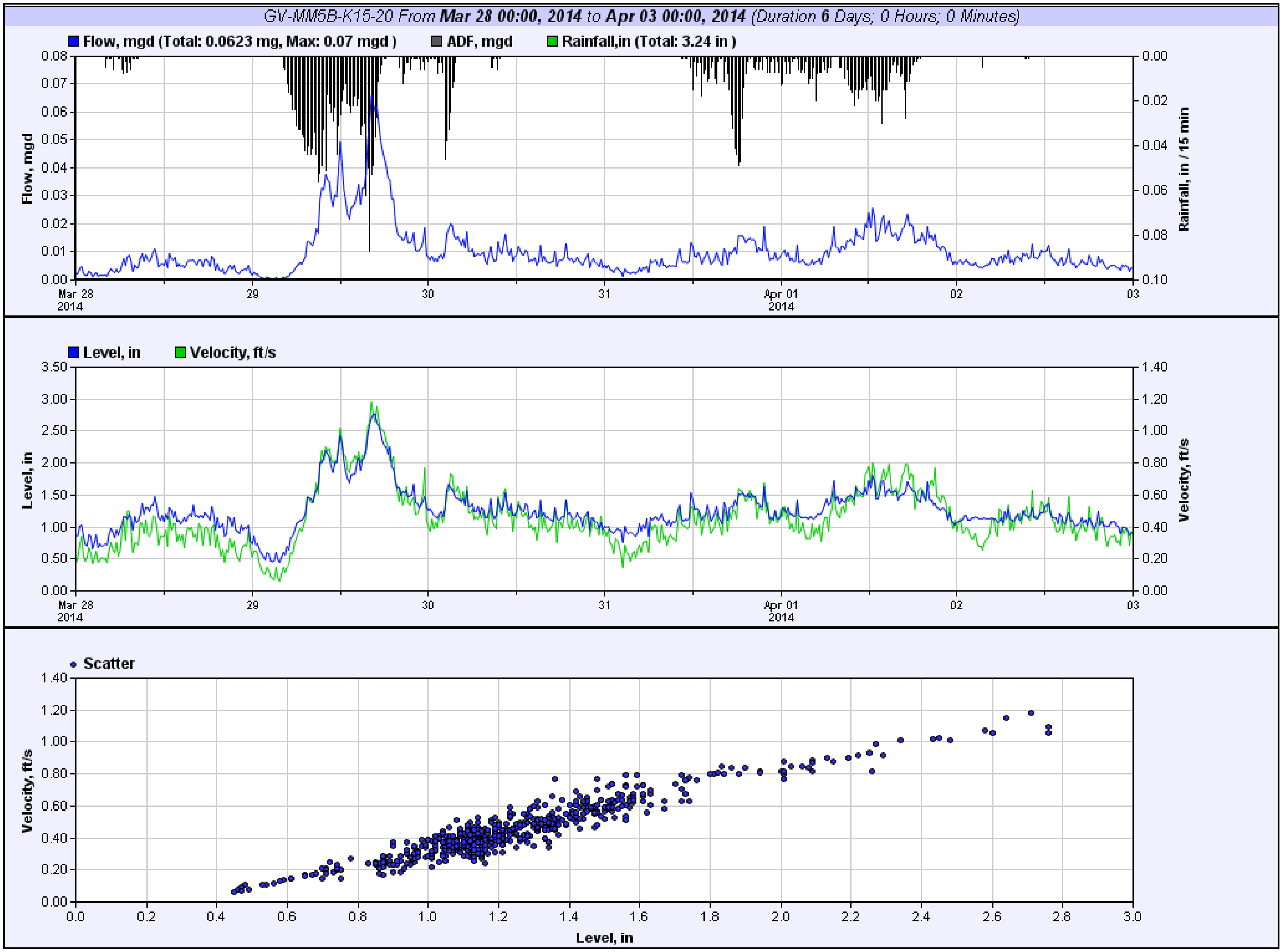
Basin 5

Micromonitor Graphs During Key Storms

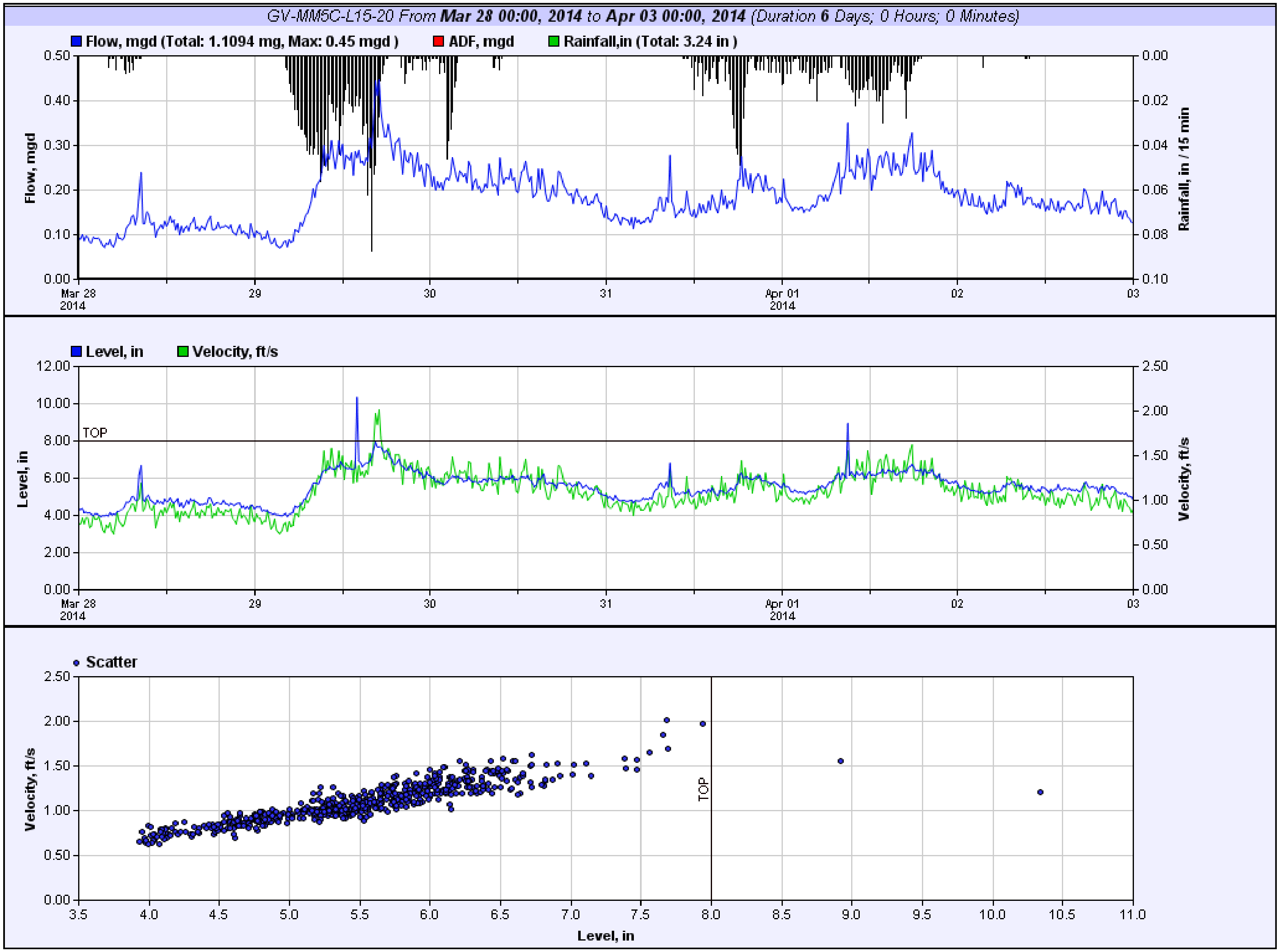
GV-MM5A-K15-19
March 29 and 31 Storms



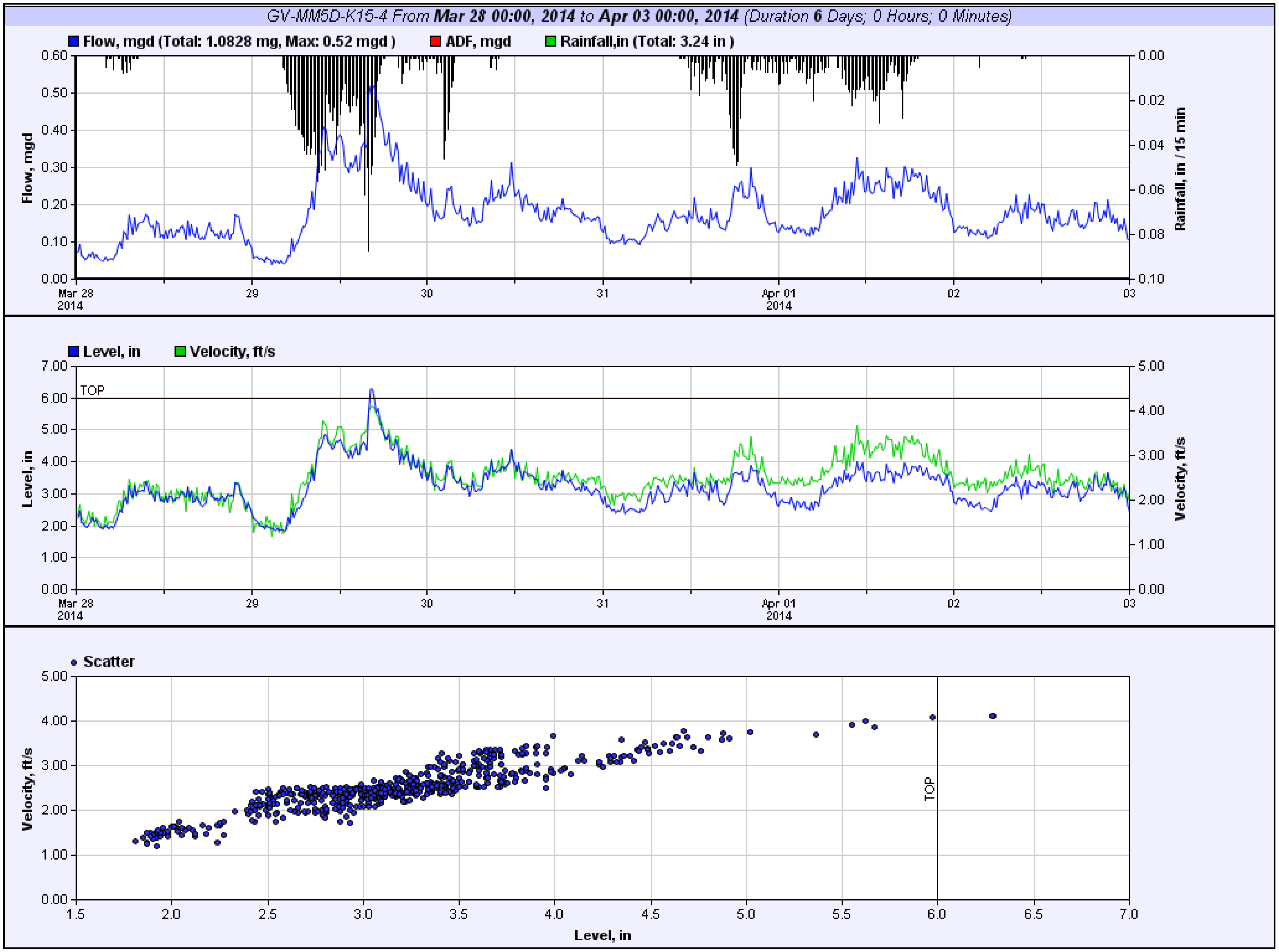
GV-MM5B-K15-20
March 29 and 31 Storms



GV-MM5C-L15-20
March 29 and 31 Storms



GV-MM5D-K15-4
March 29 and 31 Storms



Stantec Consulting Services Inc.
101 Providence Mine Road, Suite 202, Nevada City CA 95959

October 17, 2014

Attention: Tim Kiser, P.E., Public Works Director/City Engineer
City of Grass Valley – Engineering Division
125 East Main Street
Grass Valley, CA 95945
United States of America

Dear Mr. Kiser,

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

SUMMARY OF FINDINGS

Stantec Consulting Services, Inc. (Stantec) has conducted flow monitoring on behalf of the City of Grass Valley (City) as part of an overall wastewater system assessment, which will culminate in the development of a wastewater system master plan and capital improvement program. Stantec previously submitted the results of Phase 1 of the Grass Valley Micromonitoring Program (Phase 1) to the City on June 10, 2014. The Phase 1 study focused on monitoring inflow and infiltration (I/I) of stormwater and ground water entering the City's sewer system during the spring of 2014. Based in part on the Phase 1 analysis, the City determined that additional dry weather flow monitoring was necessary in Basins 5, 6, and 8, in order to investigate ground water infiltration (GWI) issues in those basins. This report summarizes the results of the Grass Valley Micromonitoring Program Phase 2 GWI study (Phase 2) conducted during summer of 2014 between June 11 and July 8, by Stantec and V&A Consulting Engineers (V&A).

V&A performed the field services to collect the flow data. Level and velocity data was recorded using ISCO 2150 Area-Velocity flow meters. During the Phase 2 study a total of 17 flow meters were installed in basins 5, 6, and 8. More details about Phase 2 field activities are presented in Attachment A – V&A Technical Memorandum 2.

Analysis of the flow monitoring data in conjunction with nighttime field observations by V&A provide insight into GWI patterns in the study area. Several locations have been identified for further investigation and/or remedial action. Exhibits 1 – 3 (Attachment B) illustrate the results of the Phase 2 GWI study in basins 5, 6 and 8. The sub-basins that appear to warrant priority for further investigation are identified as S-5D-1, S-5D2, S-5C-3, S-6-2, S-6-4, S-8-2, and S-8-4. Sub-basins S-5C-1 S-5D-4, S-6-3, S-8-2, and S-8-3 are recommended as second tier priority sites for further investigation.

FLOW MONITOR INSTALLATIONS

Eight flow monitors were installed in Basin 5, four flow monitors were installed in Basin 6 and five flow monitors were installed in Basin 8. Exhibit 4 (Attachment B) indicates the locations of the flow monitors and the associated sub-basins, and Figure 1 shows a schematic of the flow monitoring network for Basins 5, 6 and 8. Location details such as the manhole number (using the City's numbering system) and the sub basin being monitored are provided in Table 1. The prefix MM indicates that a micromonitor was used in that location and the prefix FM indicates that a traditional flow monitor was used in that location. Respective pipe sizes are also provided in Table 1. Dates of installation and removal are provided in Table 2.

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

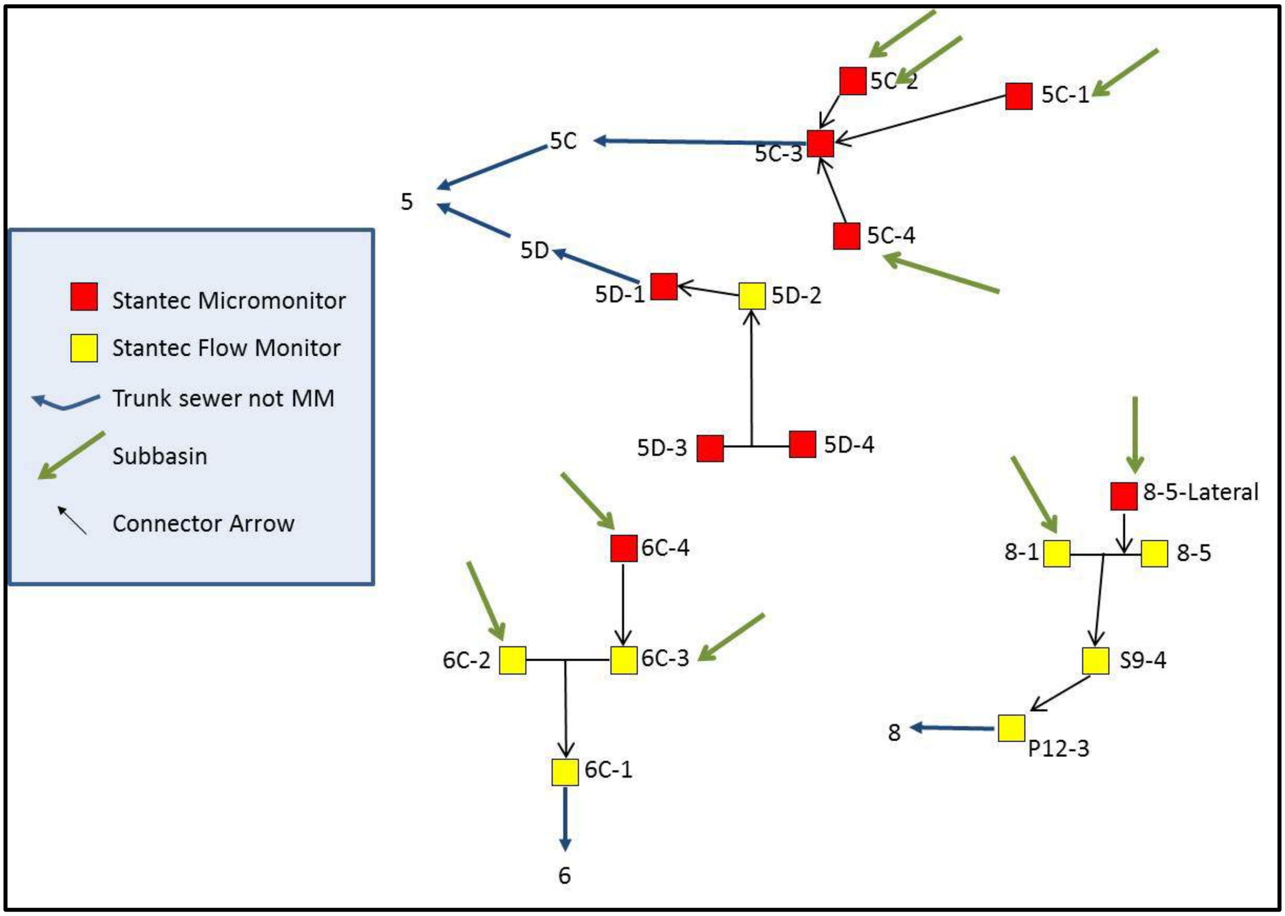


Figure 1: Monitoring schematic for Basins 5, 6, and 8

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

Table 1: Monitor Locations

Monitor	Manhole	Pipe Size (in)	Sub-basin Monitored	Basin
MM5C-1	M15-5	6	S-5C-1	Basin 5C
MM5C-2	M15-7	6	S-5C-2	Basin 5C
MM5C-3	L15-8-1	8	S-5C-3	Basin 5C
MM5C-4	M15-28	6	S-5C-4	Basin 5C
MM5D-1	L15-9	6	S-5D-1	Basin 5D
FM5D-2	L15-10	6	S-5D-2	Basin 5D
MM5D-3	L16-12	8	S-5D-3	Basin 5D
MM5D-4	L16-12	8	S-5D-4	Basin 5D
FM6-1	K13-17	8	S-6-1	Basin 6
FM6-2	J13-5	6	S-6-2	Basin 6
FM6-3	K13-2	6	S-6-3	Basin 6
MM6-4	J11-5	6	S-6-4	Basin 6
FM8-1	S8-1	12	S-8-1	Basin 8
FM8-5	S8-5	6	S-8-3	Basin 8
FMP12-3	P12-3	18	S-8-4	Basin 8
FMS9-4	S9-4	12	S-8-2	Basin 8
MM8-5-LATERAL[1]	LATERAL	6	S-8-2 LATERAL	Basin 8

1. Note the 6 inch lateral coming into manhole S8-5 actually collects flow from a portion of sub-basin S-8-2

Table 2: Monitor Installation Dates

Monitor	Upstream Monitors	Date Installed	Date Removed	Basin
MM5C-1		6/25/2014	7/8/2014	Basin 5C
MM5C-2		6/25/2014	7/8/2014	Basin 5C
MM5C-3	MM5C-1, MM5C-2, MM5C-4	6/25/2014	7/8/2014	Basin 5C
MM5C-4		6/25/2014	7/8/2014	Basin 5C
MM5D-1	FM5D-2	6/11/2014	7/8/2014	Basin 5D
FM5D-2	MM5D-3, MM5D-4	6/11/2014	7/8/2014	Basin 5D
MM5D-3		6/11/2014	6/23/2014	Basin 5D
MM5D-4		6/11/2014	6/23/2014	Basin 5D
FM6-1	FM6-2, FM6-3	6/12/2014	6/24/2014	Basin 6
FM6-2		6/12/2014	6/24/2014	Basin 6
FM6-3	MM6-4	6/12/2014	6/24/2014	Basin 6
MM6-4		6/12/2014	6/24/2014	Basin 6
FM8-1		6/25/2014	7/8/2014	Basin 8
FM8-5		6/25/2014	7/8/2014	Basin 8
FMP12-3	FMS9-4	6/25/2014	7/8/2014	Basin 8
FMS9-4	FM8-1, FM8-5, MM8-5-Lateral	6/25/2014	7/8/2014	Basin 8
MM8-5-LATERAL		6/25/2014	7/8/2014	Basin 8

DATA COLLECTED

The graphs of the collected data are included in Attachment C. The level sensor at Site S-6-2 in manhole J13-5 failed, so no monitoring data is reported for that location. The complete set of data is available in the SFM software provided under separate cover.

SUMMARY OF GROUND WATER INFILTRATION ANALYSIS

The flow monitoring data collected for the Phase 2 study was used to estimate GWI rates and average daily sewer flow (ADSF) rates for each individual sub-basin. The ADSF and GWI are related as shown below:

$$ADSF + GWI = ADF$$

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

ADSF and GWI were calculated using the Stevens/Schutzbach Equation[1], which is an empirical equation designed to make the best estimate of ADSF and GWI:

$$\mathrm{GWI} = \frac{0.4 \times \mathrm{MDF}}{1 - 0.6\left(\frac{\mathrm{MDF}}{\mathrm{ADF}}\right)^{\mathrm{ADF}^{0.7}}}$$

Records from the flow analysis done using the SFM software are presented in Attachment D.

In order to compare infiltration flow rates between sub-basins of different sizes, an effective infiltration rate (flow rate per linear foot of contributing sewer) is used. A summary of the dry weather base flow rates per foot of sewer in each sub basin is presented in Table 3. Each sub-basin was ranked with low, medium, or high rehab priority based on the effective GWI rate and physical observations from V&A. The priority rating is a relative rating comparing the GWI to other sub-basins within their larger basin, but does not reflect comparisons between major basins. A more complete record of the analysis is included in Attachment E. During the analysis of sub-basin S-6-3 a negative value was calculated for ADSF (Attachment E). The monitoring data indicates that there is very little flow introduced between the monitor FM6-3 and MM6-4 (Figure 2), likely because of the small basin size. In order to isolate the flow from the individual basins for comparison, flows from upstream basins are subtracted from flows at downstream monitors. The large difference in basin size and the short duration of data can introduce some error in the calculation of ADSF. For the purposes of this study the values have been deemed accurate enough to do a relative comparison.

1 Mitchell, P., Stevens, P., Nazaroff, A., "A Comparison of Methods and a Simple Empirical Solution to Quantifying Base Infiltration in Sewers," Water Practice, Volume 1, Number 6, December 2007, Water Environment Federation

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

Table 3: Summary of flow data by Sub-basin

	Sub-basin	ADSF (gpd/lf)	GWI (gpd/lf)	ADF (gpd/lf)	Field Findings	Rehab Priority
Basin 5	S-5C-1	3.8	1.8	5.6	Area upstream of 5C-1 was observed to have severe infiltration.	Medium
	S-5C-2	3.5	0.4	4.0	Majority of infiltration from the western inlet.	Low
	S-5C-3	5.0	4.7	9.7	N/A	High
	S-5C-4	4.5	0.3	4.8	More infiltration than 5C-2 from field observation.	Low
	S-5D-1	49.5	29.4	78.9	Infiltration possibly from the creek under Colfax Avenue.	High
	S-5D-2	0.2	2.6	2.8	Negligible amount of infiltration along Clark Road.	Medium
	S-5D-3	0.9	0.1	1.0	Infiltration evenly distributed along Lucas Lane	Low
	S-5D-4	6.8	0.5	7.3	Majority of infiltration occurs between Neville Way and Fiddick Lane	Medium
Basin 6	S-6-1	0.1	0.0	0.1	Minimal infiltration	Low
	S-6-2[1]				Large amount of infiltration, possibly from a private lodging area upstream.	High
	S-6-3	0.0	1.5	1.5	Clear infiltration observed.	Medium
	S-6-4	6.3	2.8	9.1	Estimated that more than 60% of infiltration is from St. Johns Drive.	Medium
Basin 8	S-8-1	1.5	0.6	2.1	N/A	Low
	S-8-2	4.3	1.6	5.9	N/A	Medium
	S-8-3	2.5	0.8	3.3		Low
	S-8-2 LATERAL[2]	-	-	-	Clear water from lateral serving shopping center. Lateral maybe damaged.	Low
	S-8-4	2.7	1.3	3.9	N/A	Medium

1. Level sensor failed at 6-2, so no good quantitative is data available, however based on field observation further evaluation is warranted.
2. Lateral coming into MHS8-5. Length of lateral is unknown.
ADF = Average Daily Baseflow

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

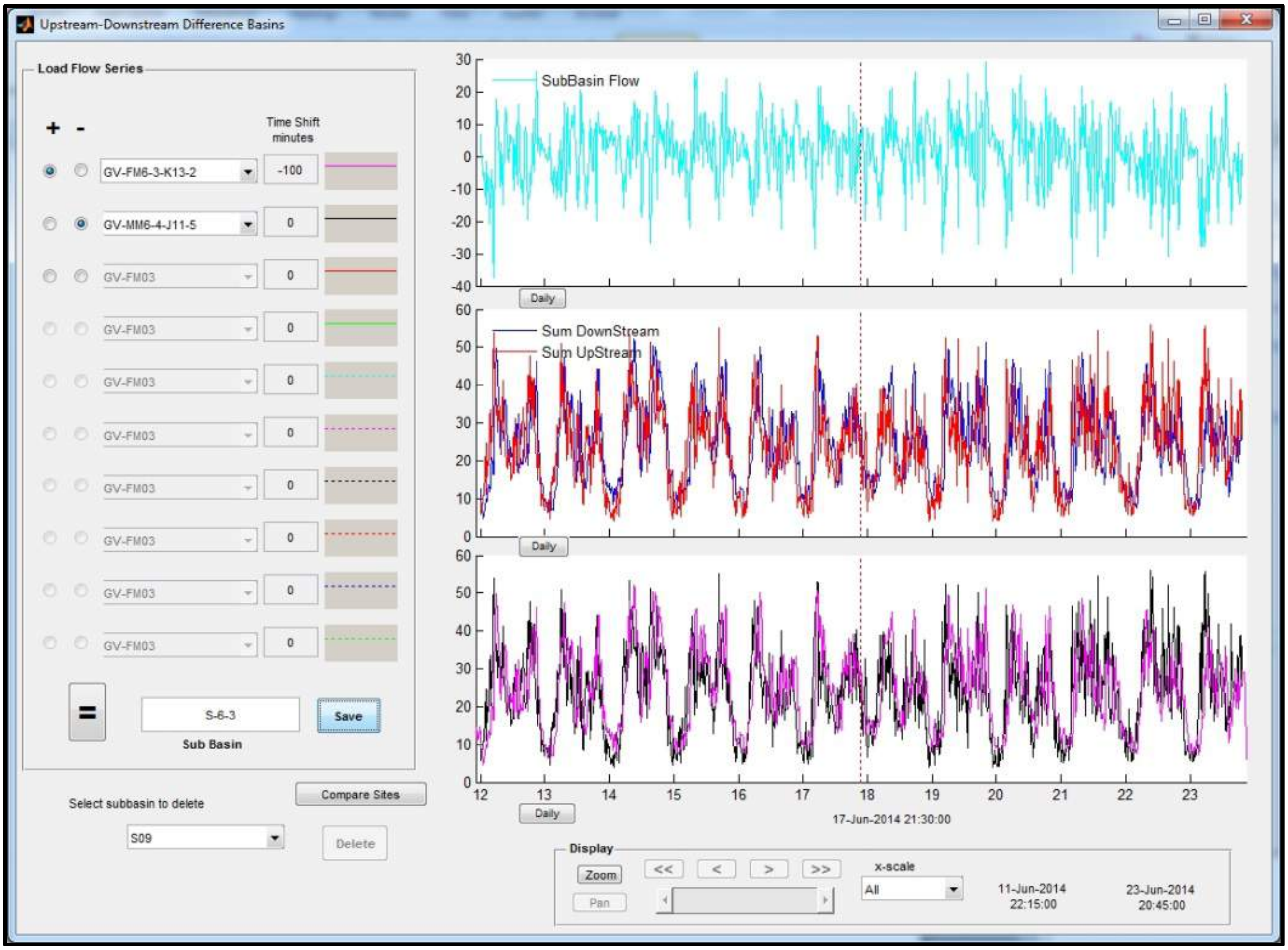


Figure 2: Comparison of monitoring data from FM6-3 and MM6-4

CONCLUSIONS

Based on the results of the GWI analysis there are several locations in Basins 5, 6, and 8 that should be prioritized for further evaluation and possible rehabilitation. In Basin 5, sub-basins S-5D-1, S-5D2, S-5C-1, and S-5C-3 are indicated by medium to high rates of GWI per linear foot of pipe. It seems probable that the high flows measured at MM5D-1 and FM5D-2 are coming from the creek under Colfax Ave. It should also be noted that the estimated GWI from sub-basin S-5D-4, although not significant (993.9 gpd), is primarily occurring between Neville Way and Fiddick Lane (V&A report, page 2).

Most of the sub-basins in Basin 6 are identified as having medium rehabilitation priority. It should be noted that the level sensor at Site FM6-2 in MH J13-5 failed, so no monitoring data is reported for that location, but field observations show high amounts of infiltration in this location with the majority possibly coming from the residential area upstream of manhole J13-5. It is possible that the laterals are damaged, and it is recommended all pipes upstream of this location be investigated. For the basin with the highest GWI in basin 6, S-6-4, it is estimated that 60% of the infiltration is coming from St. Johns Ave.

Reference: Results from Grass Valley Micromonitoring Program – Phase 2 GWI Study

Additional flow monitoring is recommended in Basin 8, specifically in sub-basins S-8-4 and S-8-1, in order to further identify the location of groundwater infiltration into these sub-basins. The effective GWI rate per length of pipe is relatively low in basin 8, but it should be noted that it has the largest pipe network of the three basins in the study and consequently contributes a significant amount of GWI into the system. Although V&A observed an unusual amount of clear water in the lateral serving the shopping center in sub-basin S-8-2, the analysis of flow data from monitor MM8-5-LATERAL did not indicate a significant amount of GWI. The data table for Basin 8 in Attachment D shows the estimated ADSF is high (4,172 gpd) given the small area being served, but may be accounted for given the proximity to a laundromat.

Stantec will prepare a separate memorandum summarizing further recommendations along with those from the Phase 1 report. This will include comprehensive recommendations on next steps for the City, a work plan for further flow monitoring for fiscal year 2014/15, and a suggested outline for long term flow monitoring and I/I reduction program. This information will be finalized for inclusion in the City of Grass Valley Wastewater Master Plan.

Regards,

STANTEC CONSULTING SERVICES INC.

Dave Price, P.E
Senior Engineer, Water
Tel: 530-470-0515
dave.w.price@stantec.com

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Attachment:

A - V&A Technical Memorandum – Sanitary Sewer Flow Monitoring and Infiltration Study in Drainage Basins 5, 6, and 8 in Grass Valley, California
B - Exhibits 1 - 4
C – Flow Monitoring Graphs
D – GWI Analysis records
E – Baseflow Calculations

ATTACHMENT A

V&A Technical Memorandum – Sanitary Sewer Flow Monitoring and Infiltration Study in Drainage Basins 5, 6, and 8 in Grass Valley, California

Sanitary Sewer Flow Monitoring and Infiltration Study in Drainage Basins 5, 6, and 8 in Grass Valley, California

Prepared for: Dave Price, P.E., Stantec

Prepared by: Yanming Zhang, Ph.D., P.E., V&A

Reviewed by: Kevin Krajewski, P.E., V&A

Ray Yep, P.E., V&A

Date: August 5, 2014

1.0 INTRODUCTION

V&A was retained by Stantec to perform sanitary sewer flow monitoring and inflow/infiltration study within the City of Grass Valley (City). The Phase 1 study was conducted from February 6, 2014 to April 8, 2014 at eight open-channel flow monitoring sites; the flow monitoring and infiltration and inflow (I/I) report was submitted in May 2014.

The Phase 2 study focused on ground water infiltration issues in Basins 5, 6 and 8. During the Phase 2 study (June 11 to July 7, 2014), V&A monitored the sanitary sewer flow at 17 open-channel flow monitoring sites within the three basins. For the Phase 2 study, V&A also conducted night-time I/I reconnaissance: several of the manholes within Basins 5, 6 and 8 were opened during low-flow hours (from 11:00 P.M. to 4:00 A.M.) to evaluate volumes of "clear-water" flow and other evidence of infiltration. These series of evaluations for infiltration are qualitative in nature, but intend to locate the sub-basins where large quantities of infiltration exists and eliminate the sub-basins that do not have a groundwater infiltration problem. This technical memorandum documents the field findings and observations.

2.0 METHODS AND PROCEDURES

A total of 17 flow meters were installed in Basins 5, 6, and 8. The methods used for the Phase 2 study flow monitoring were identical to the Phase 1 study (refer to *12-0314 CofGrassValley FM and II Rpt, May 2014*, for the methods and procedures of flow monitoring). V&A was not responsible for quantitative flow analysis for the Phase 2 study and the flow monitoring results are not presented as a part of this technical memorandum; the raw flow monitoring data were given to Stantec directly.

For infiltration reconnaissance, V&A engineers chose 11 P.M. to 4 A.M. for work hours as the sanitary flow is minimal and ground-water infiltration (clear-water flow) can more easily be distinguished. The goal was to track the infiltration in each basin and locate the sources of infiltration into reasonably small sub-basins so the City may consider initiating cost-effective capital improvement projects for infiltration mitigation.

3.0 FIELD FINDINGS

3.1 Basin 5

A total of eight flow meters were installed in this basin. The infiltration issues throughout the basin are illustrated in Figure 1 and stated below.

- Sites 5D-1 and 5D-2 were used to check the potential infiltration from the creek under Colfax Avenue. The amount of infiltration is the difference of the flows measured from the two monitoring sites.
- There was a negligible amount of infiltration coming between Site 5D-2 and Site 5D-3/4.
- Sites 5D-3 and 5D-4 were installed in the same manhole to quantify the infiltration from the west inlet and the east inlet.
    - The infiltration in Lucas Lane was evenly distributed.
    - The infiltration from Pine Street was mostly from Neville Way and Fiddick Lane. Though the infiltration is notable in this area, the amount of infiltration was relatively low compared to other locations in Grass Valley.
- The area upstream of Site 5C-1 had the most severe infiltration observed within the basin. It is recommended that the infiltration issues for the pipe segments between 5C-1 and 5C-3 be evaluated by quantitative flow analysis from the data collected.
- For Site 5C-2, the majority of infiltration was from the western inlet.
- For Site 5C-4, the infiltration was more severe than Site 5C-2 from field observation.

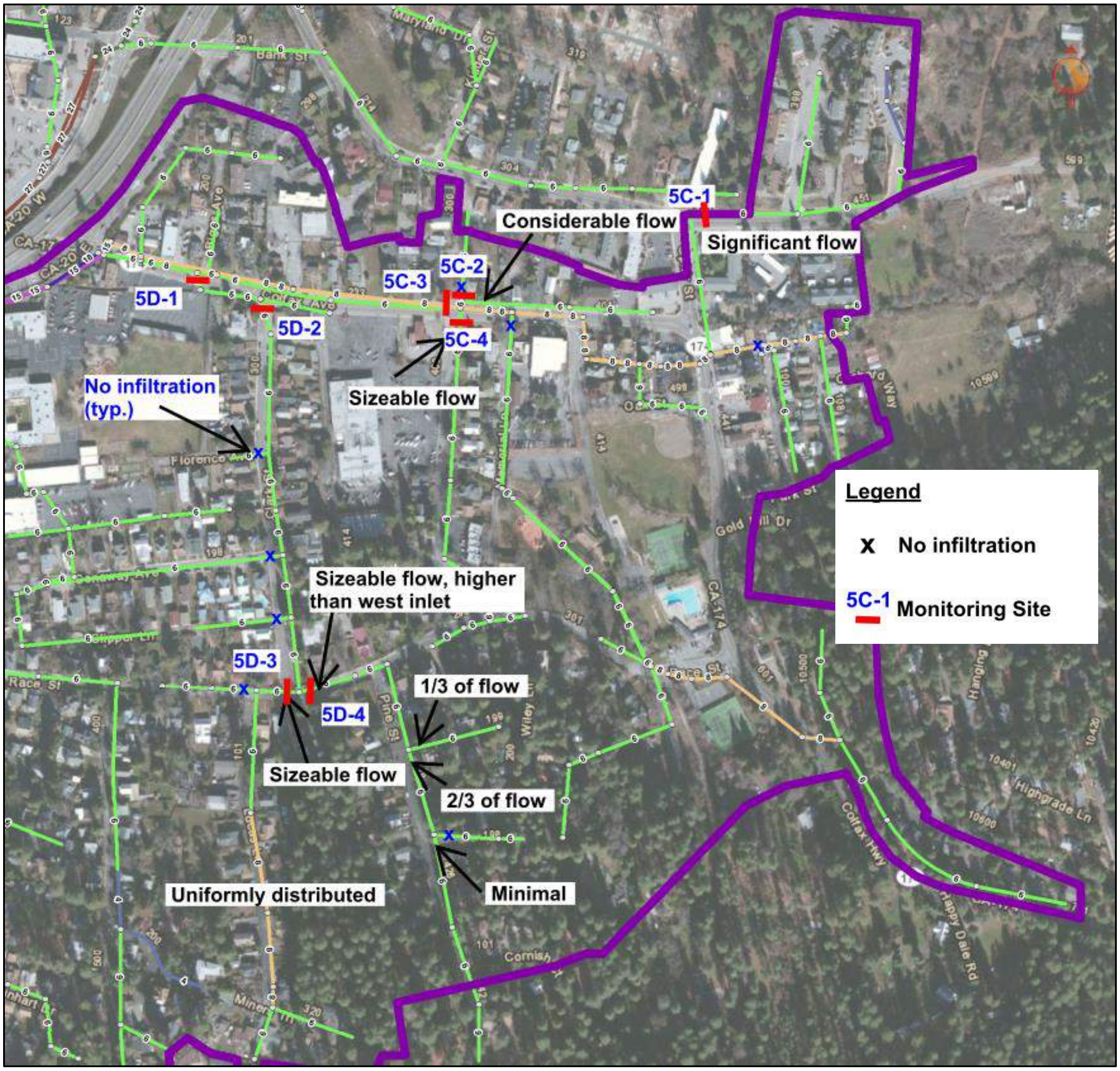


Figure 1. Infiltration Issues in Basin 5

The photos of each monitored site are shown below.

Photo 3-1. Manhole for Meter 5C-1

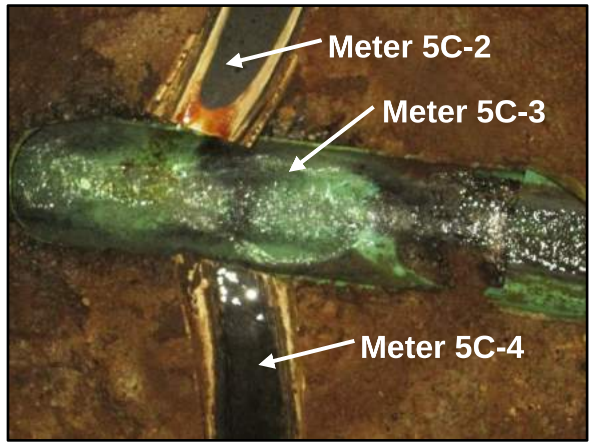


Photo 3-2. Manhole for Meter 5C-2/3/4

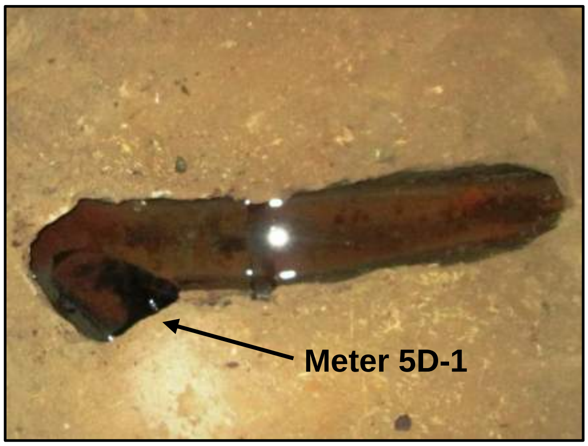


Photo 3-3. Manhole for Meter 5D-1

Photo 3-4. Manhole for Meter 5D-2

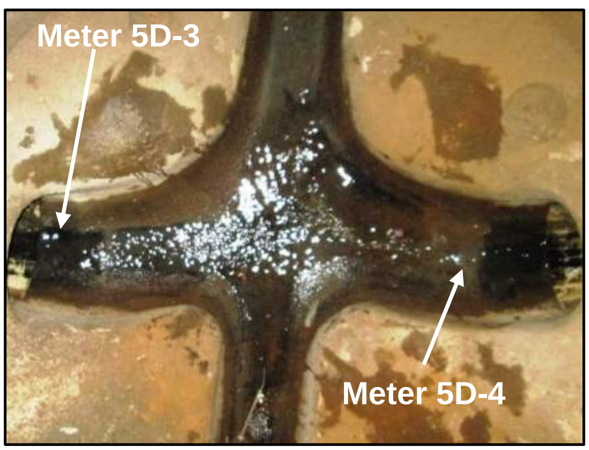


Photo 3-5. Manhole for Meter 5D-3/4

3.2 Basin 6

There are four flow meters installed in this basin. The infiltration issues throughout the basin are illustrated in Figure 2 and stated below.

- The amount of infiltration for each site should be determined from quantitative flow analysis from data collected.
- For Site J11-5, it was estimated that more than 60% of infiltration was from St Johns Drive.
- A large amount of infiltration was found at Site J13-5, the majority of the infiltration was from a private lodging area upstream. It is recommended that the City further investigate the pipelines upstream from Site J13-5.

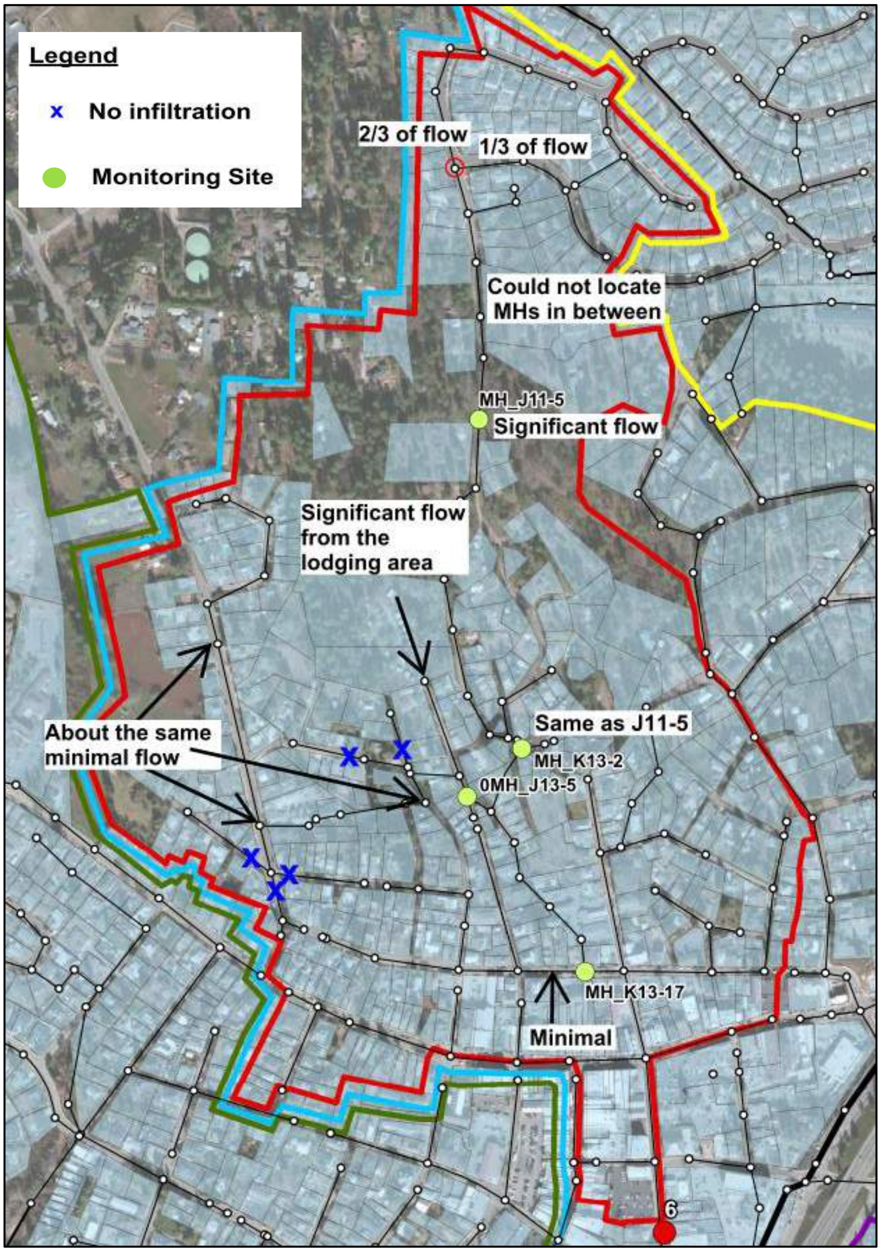


Figure 2. Infiltration Issues in Basin 6

The photos of each monitored site are shown below.

Photo 3-6. Manhole for Meter J11-5

Photo 3-7. Manhole for Meter K13-2

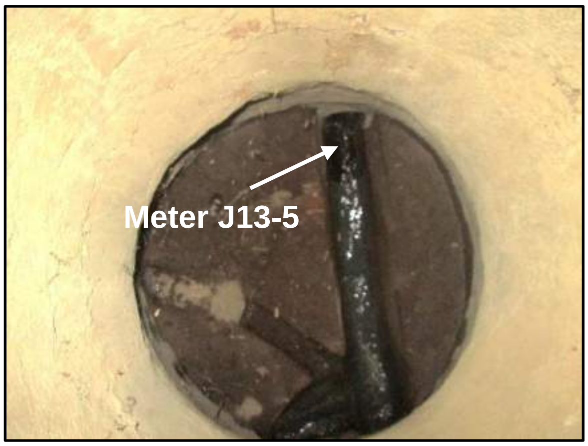


Photo 3-8. Manhole for Meter J13-5

Photo 3-9. Manhole for Meter K13-17

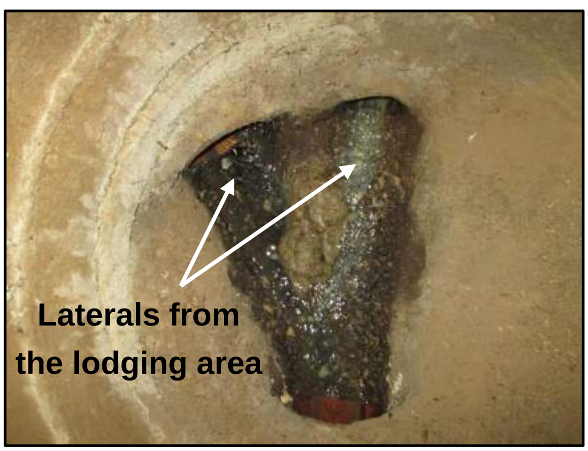


Photo 3-10. Manhole in Front of the Lodging Area

3.3 Basin 8

There are five flow meters installed in this basin. The infiltration issue through the basin is illustrated in Figure 2 and stated below.

- The amount of infiltration should be quantified for each site from flow analysis.
- V&A opened the manholes at Location A and Location B. Both of them had notable infiltration. V&A considers infiltration as a city-wide issue and did not proceed with more field investigation.
- V&A recommends a further infiltration study by installing more flow meters in the upstream area of Basin 8 and locate the source of infiltration.
- Site 8-5: The volume of clear water flowing from the 6-inch lateral that serves only a shopping center appeared to be considerably higher than would be expected. The lateral may be damaged,

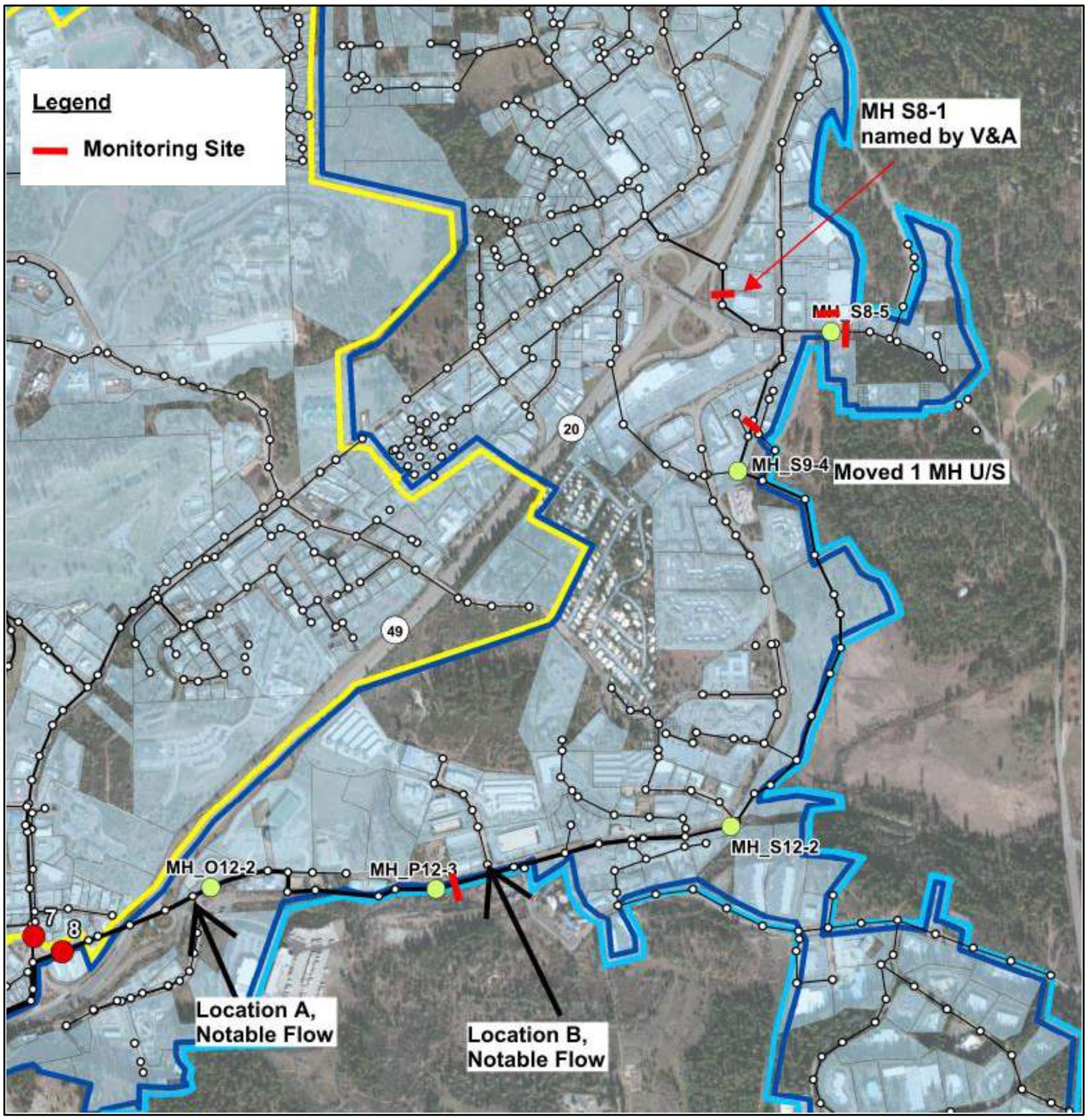


Figure 3. Infiltration Issues in Basin 8

The photos of each monitored site are shown below.

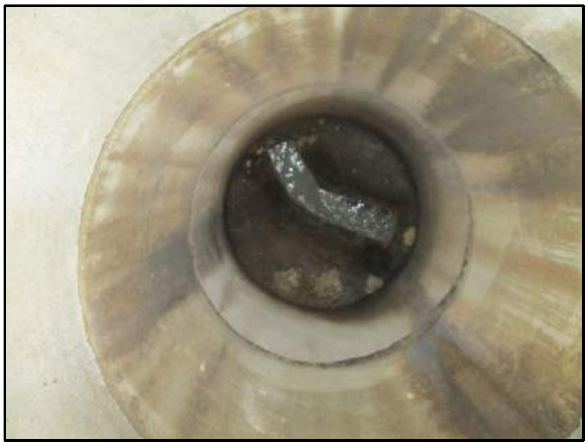


Photo 3-11. Manhole for Meter S8-1

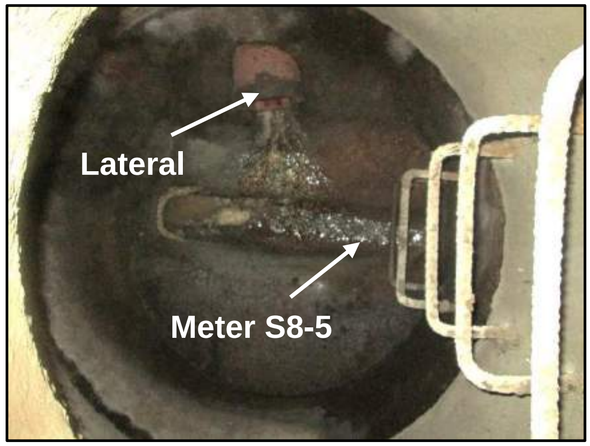


Photo 3-12. Manhole for Meter S8-5

Photo 3-13. Manhole for Meter S9-4

Photo 3-14. Manhole for Meter P12-3

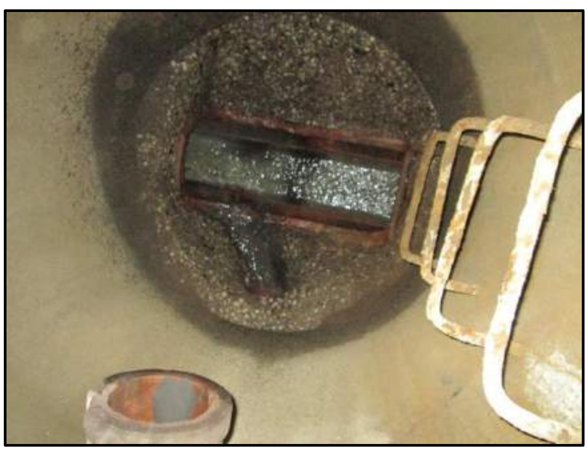


Photo 3-15. Manhole for Location A

Photo 3-16. Manhole for Location B

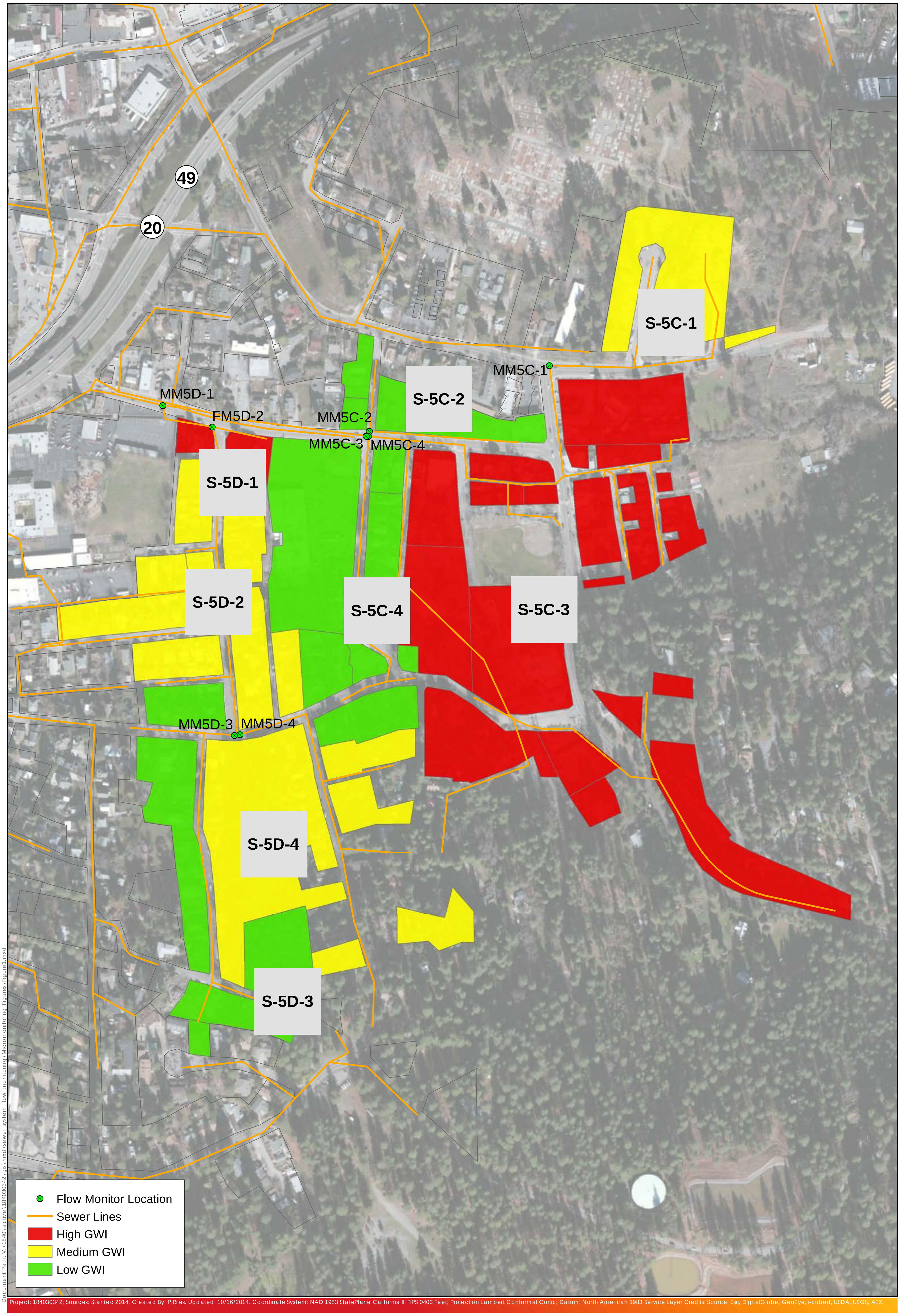
4.0 RECOMMENDATIONS

The amount of infiltration should be quantified for each site from flow analysis. V&A advises that future study consider the following recommendations:

1. **Significant Groundwater Infiltration:** One area of City focus should be the private lodging area in the upstream of Site J13-5 in Basin 6. It is possible that the laterals upstream from Site J13-5 may be damaged.
2. **Considerable Groundwater Infiltration:** The City should consider I/I mitigation measures for the following segments of pipeline as they are considered to have considerable infiltration issues for short segments.
    a. Upstream of Site 5C-1
    b. Upstream of Site 5C-2
    c. Upstream of Site 5C-4
    d. The segment between Site 5D-1 and Site 5D-2, depending on the quantitative flow analysis results.
    e. Lateral at Site 8-5
3. **Areas Requiring Additional Study:** Additional flow monitoring should be performed for Basin 8 in order to isolate the area for night-time infiltration reconnaissance.

ATTACHMENT B

Exhibits 1 - 4



Project: 184030342; Sources: Stantec 2014. Created By: P.Riles. Updated: 10/16/2014. Coordinate System: NAD 1983 StatePlane California III FIPS 0403 Feet; Projection:Lambert Conformal Conic; Datum: North American 1983 Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX,

Stantec

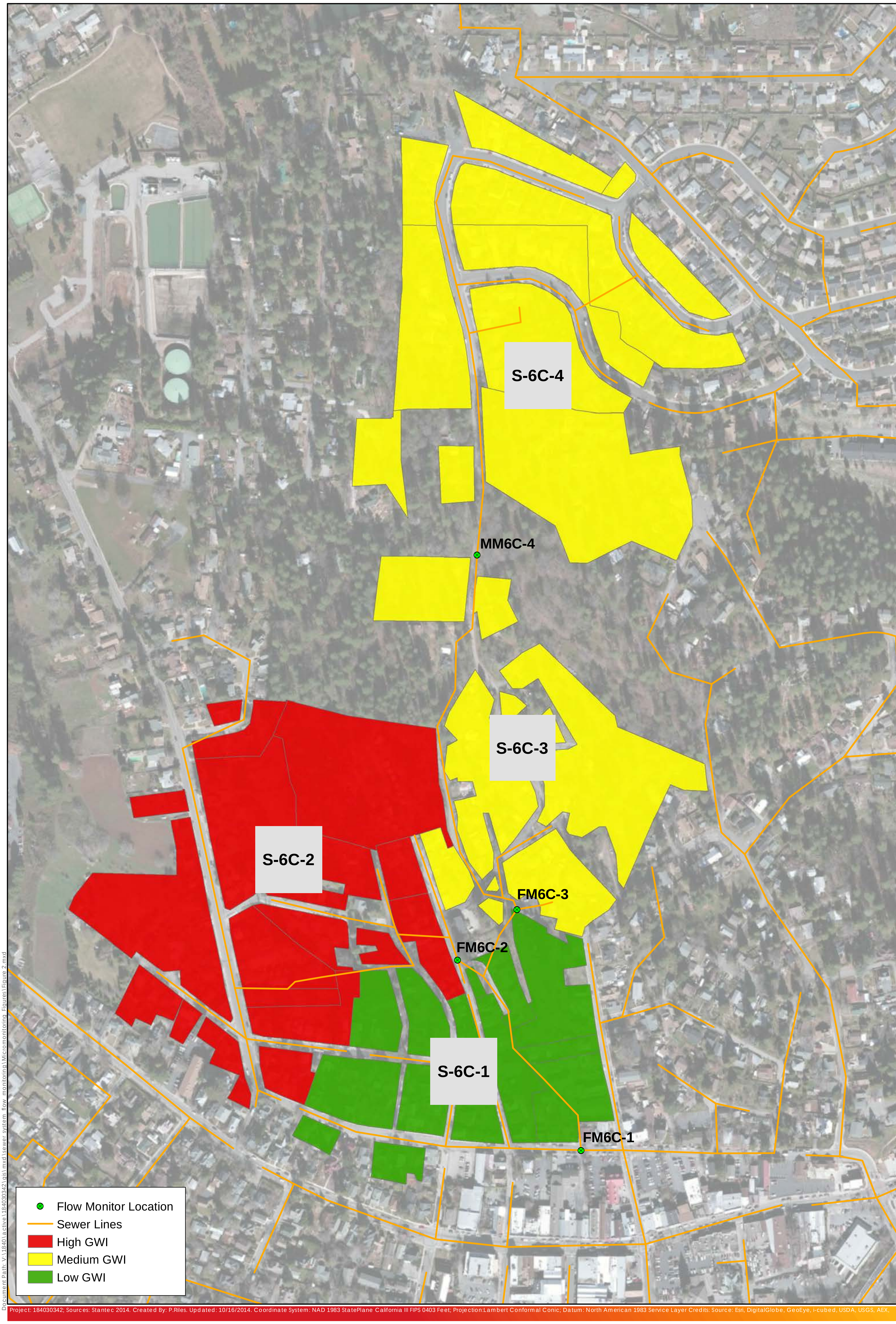
0 300 Feet

1 inch = 300 feet

Exhibit 1

Basin 5 GWI Monitoring Results

City of Grass Valley Micromonitoring Program - Phase2



Document Path: V:\1840\active\184030342\gis\mxd\sewer system flow monitoring\Micromonitoring Figures\Figure 2.mxd

Project: 184030342; Sources: Stantec 2014. Created By: P.Riles. Updated: 10/16/2014. Coordinate System: NAD 1983 StatePlane California III FIPS 0403 Feet; Projection:Lambert Conformal Conic; Datum: North American 1983 Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX,

Stantec

N

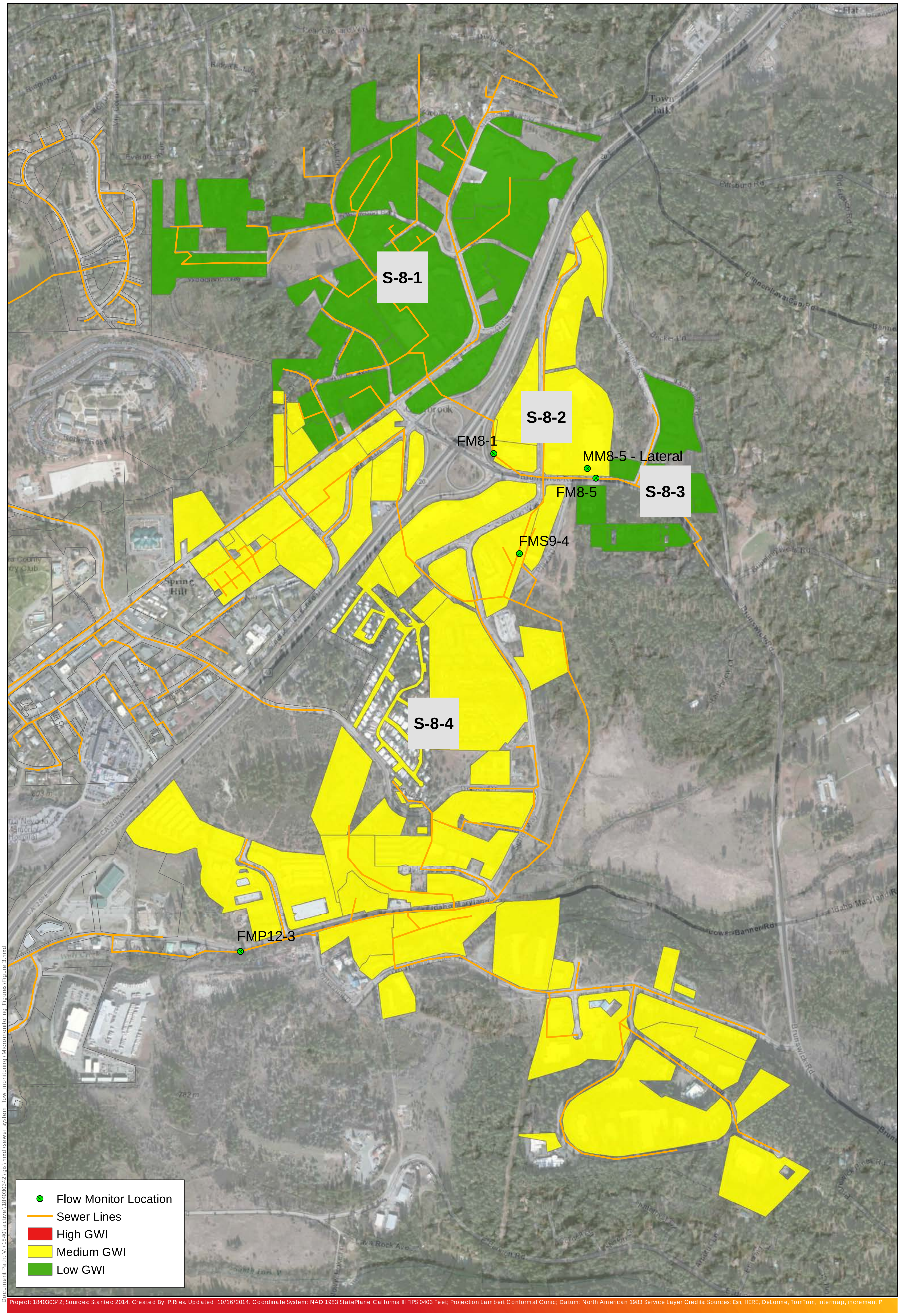
0 300 Feet

1 inch = 300 feet

Exhibit 2

Basin 6 GWI Monitoring Results

City of Grass Valley Micromonitoring Program - Phase 2



Project: 184030342; Sources: Stantec 2014. Created By: P.Riles. Updated: 10/16/2014. Coordinate System: NAD 1983 StatePlane California III FIPS 0403 Feet; Projection:Lambert Conformal Conic; Datum: North American 1983 Service Layer Credits: Sources: Esri, HERE, DeLorme, TomTom, Intermap, increment P

Stantec

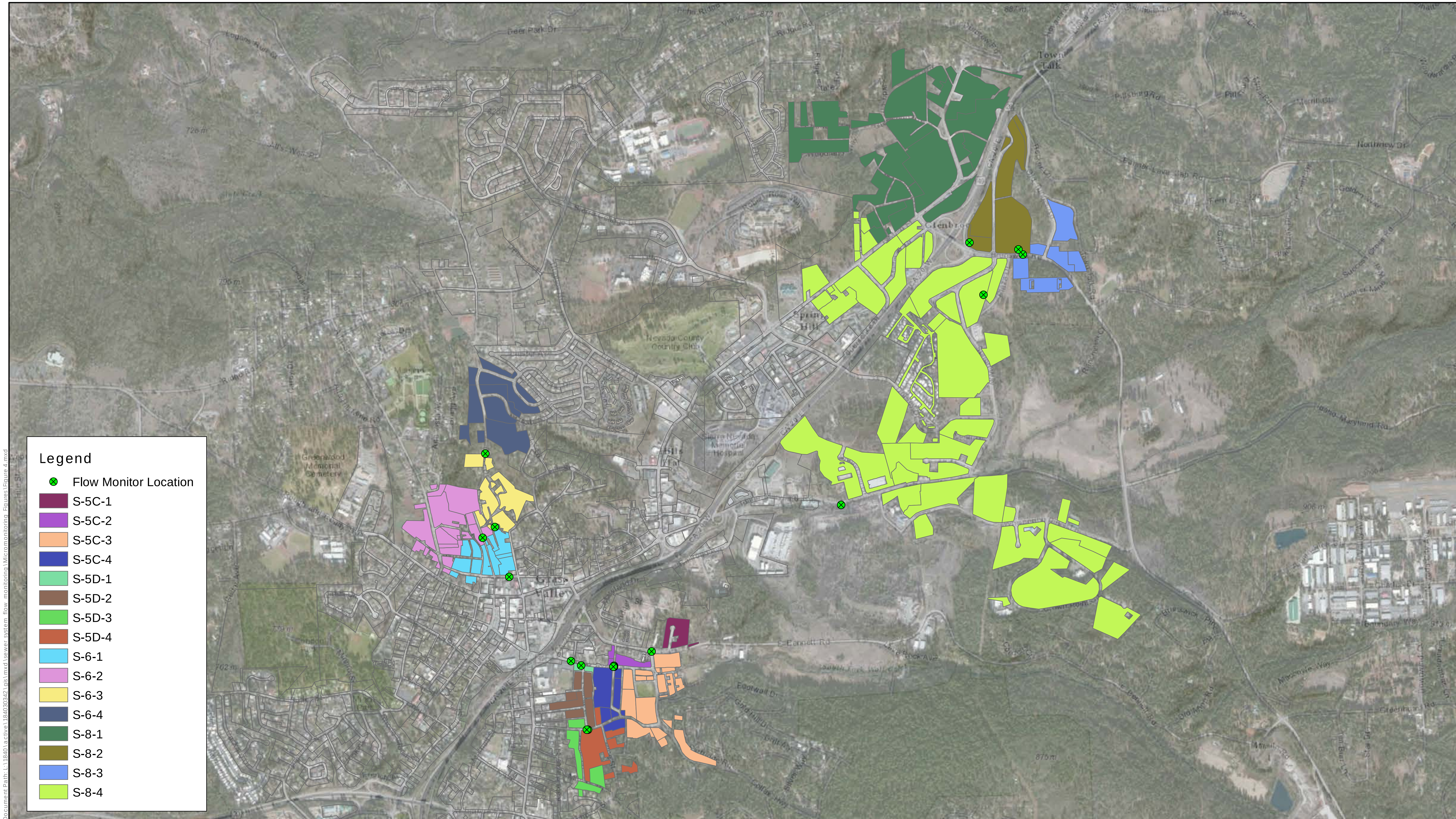
0 750 Feet

1 inch = 750 feet

Exhibit 3

Basin 8 GWI Monitoring Results

City of Grass Valley Micromonitoring Program - Phase 2



Project: 184030342; Sources: Stantec 2014. Created By: P.Riles. Updated: 10/16/2014. Coordinate System: NAD 1983 StatePlane California II FIPS 0402 Feet; Projection:Lambert Conformal Conic; Datum: North American 1983 Service Layer Credits: Sources: Esri, HERE, DeLorme, TomTom, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap

Exhibit 4

Stantec

0 0.5 Miles

1 inch = 1,500 feet

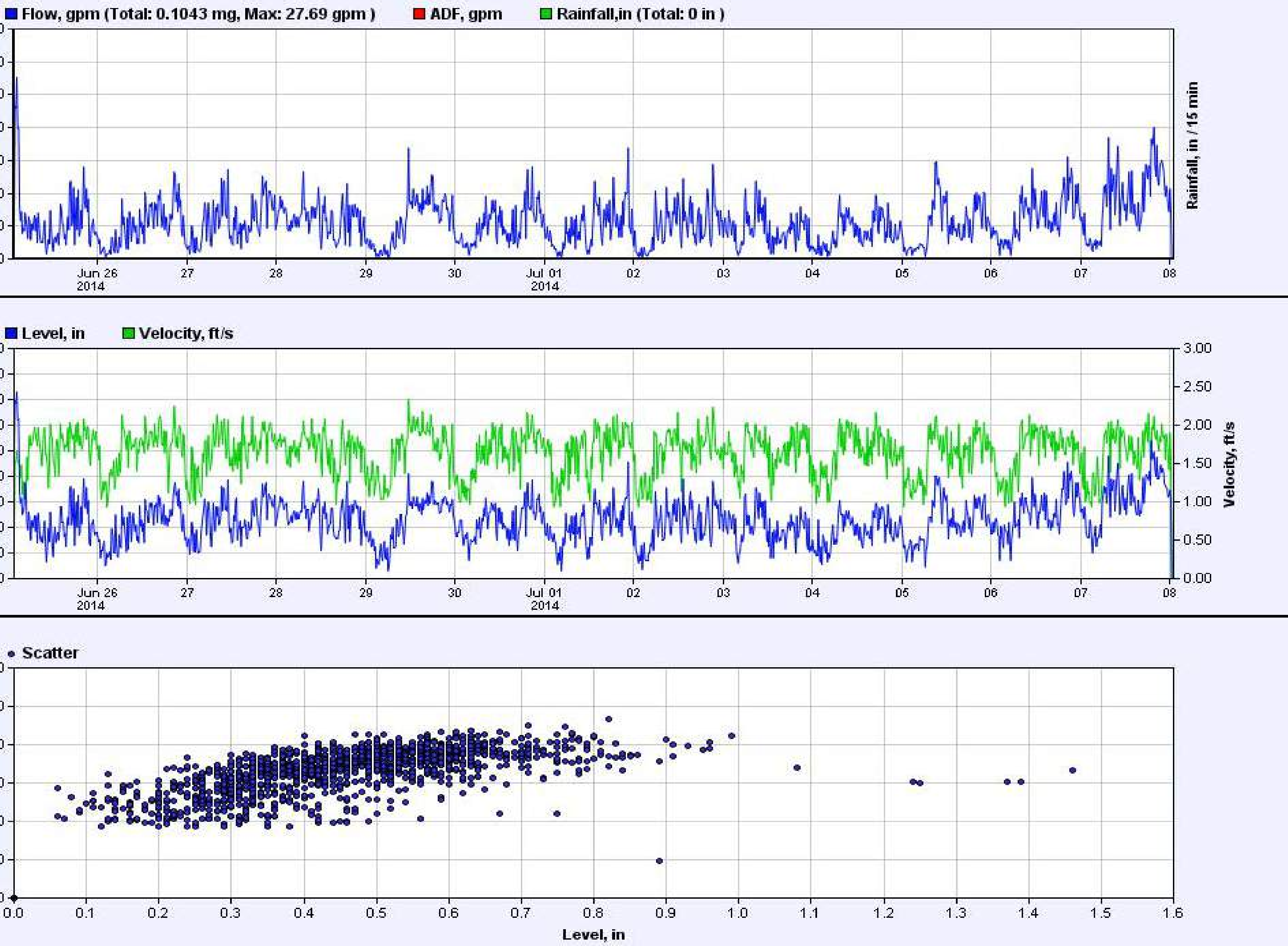
Basin Locations

City of Grass Valley Micromonitoring Program - Phase 2

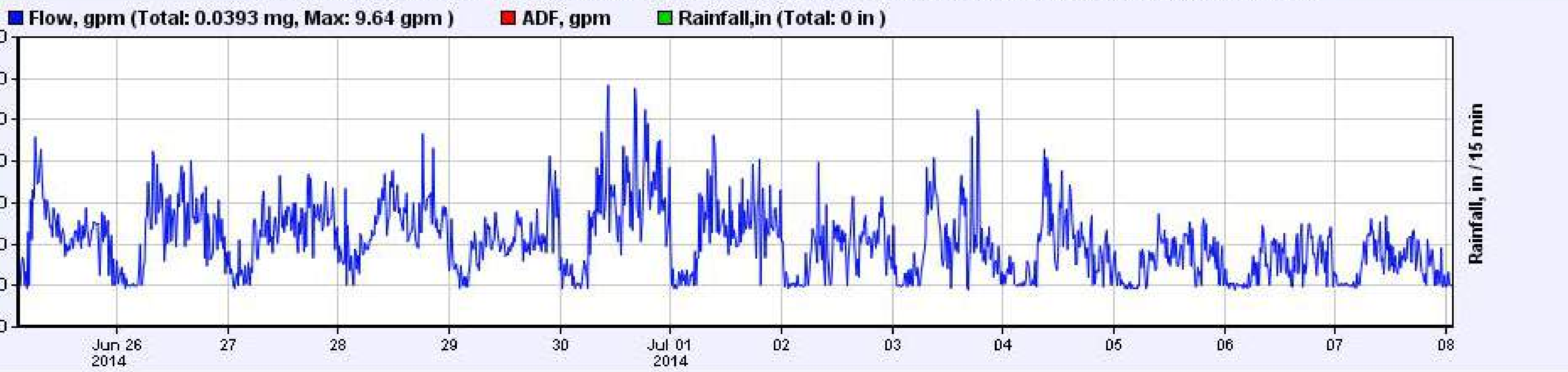
ATTACHMENT C

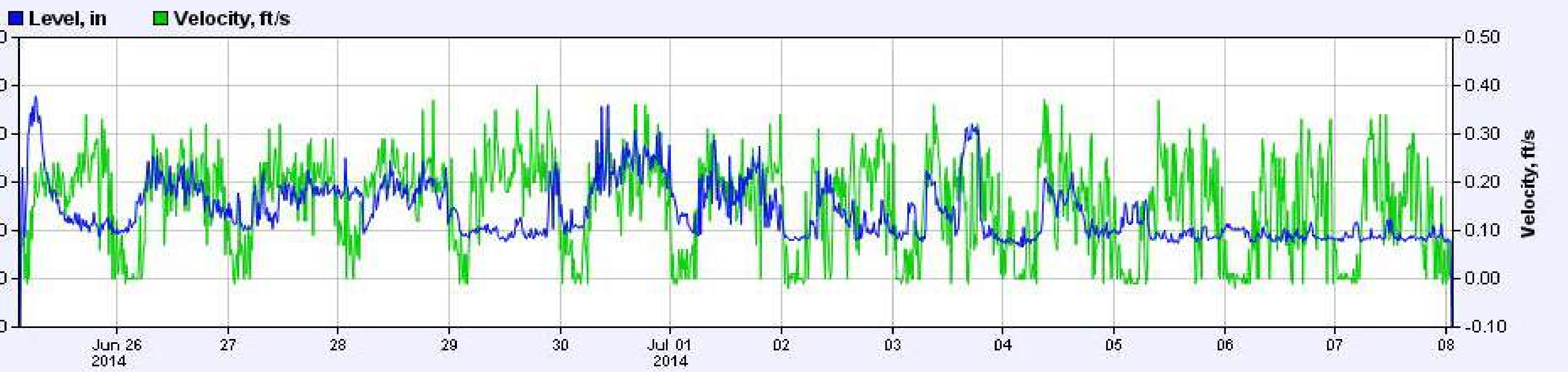
Flow Monitoring Graphs

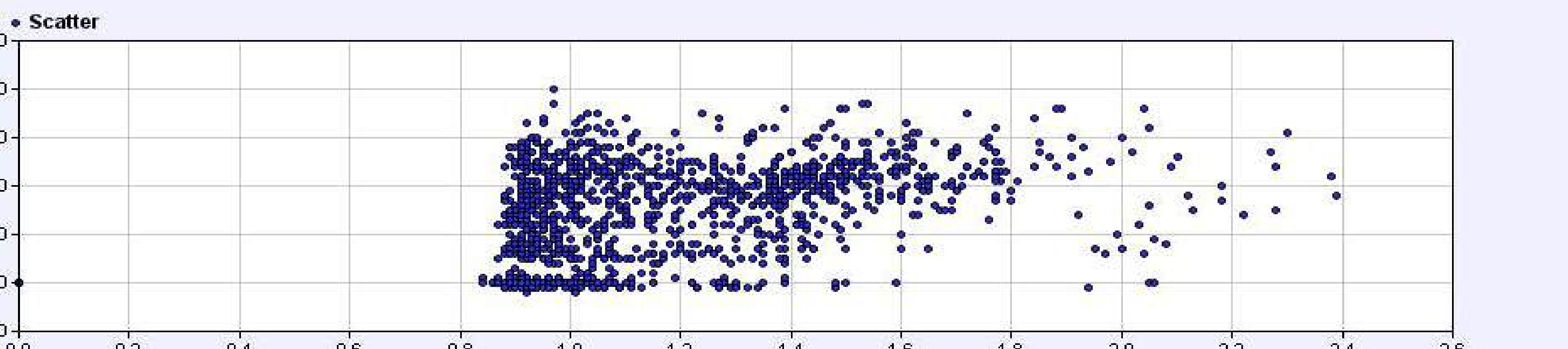
GV-MM5C-1-M15-5 From **Jun 25 01:30, 2014** to **Jul 08 01:00, 2014** (Duration **13** Days; 0 Hours; 30 Minutes)



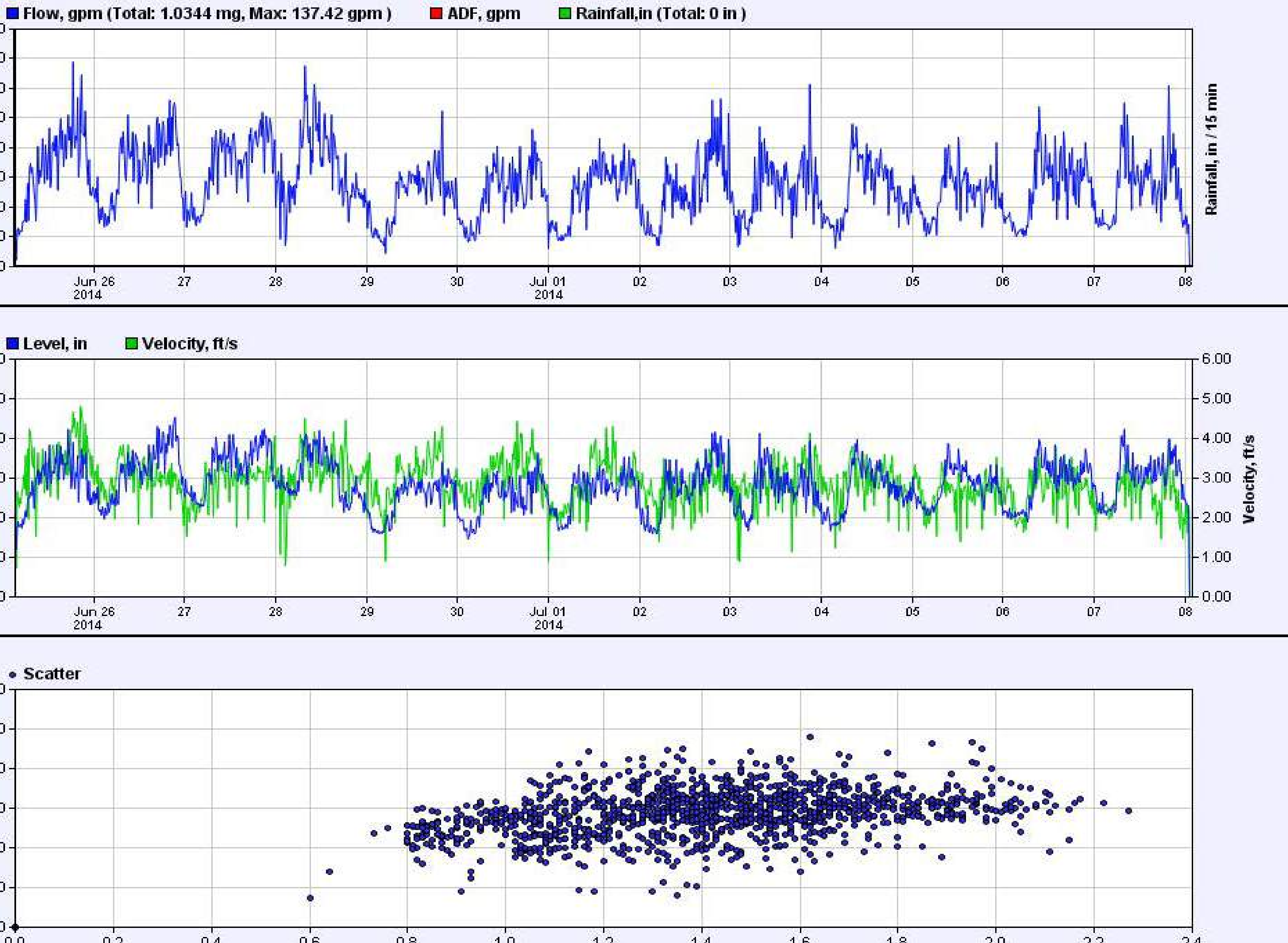
GV-MM5C-2-M15-7 From **Jun 25 02:45, 2014** to **Jul 08 01:30, 2014** (Duration **12** Days; 23 Hours; 45 Minutes)



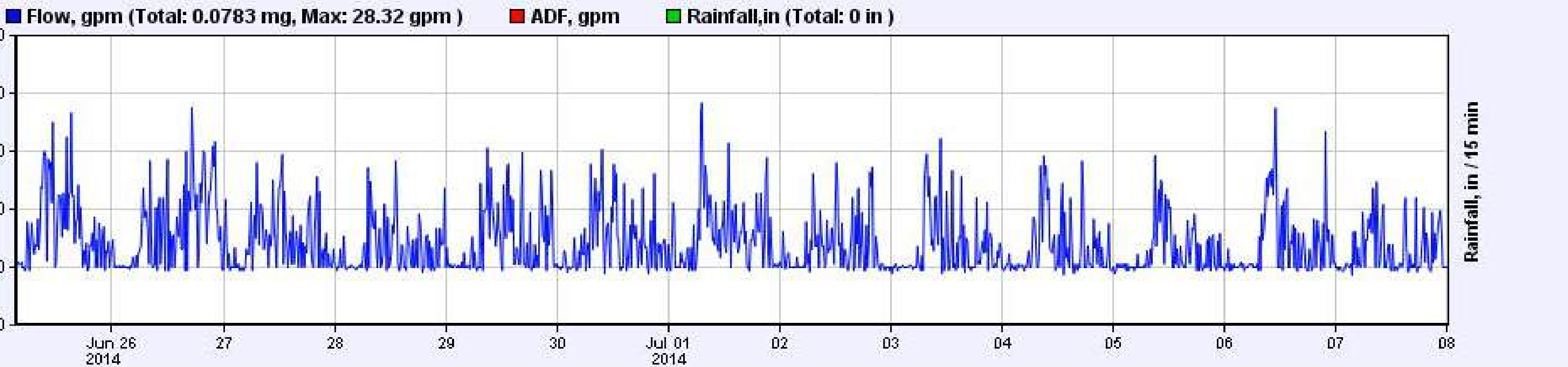


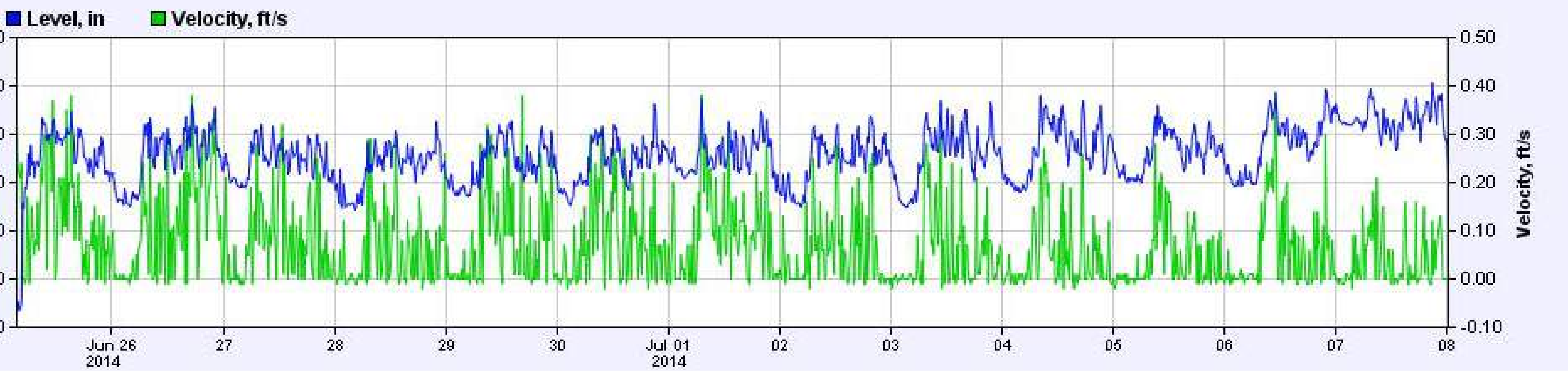


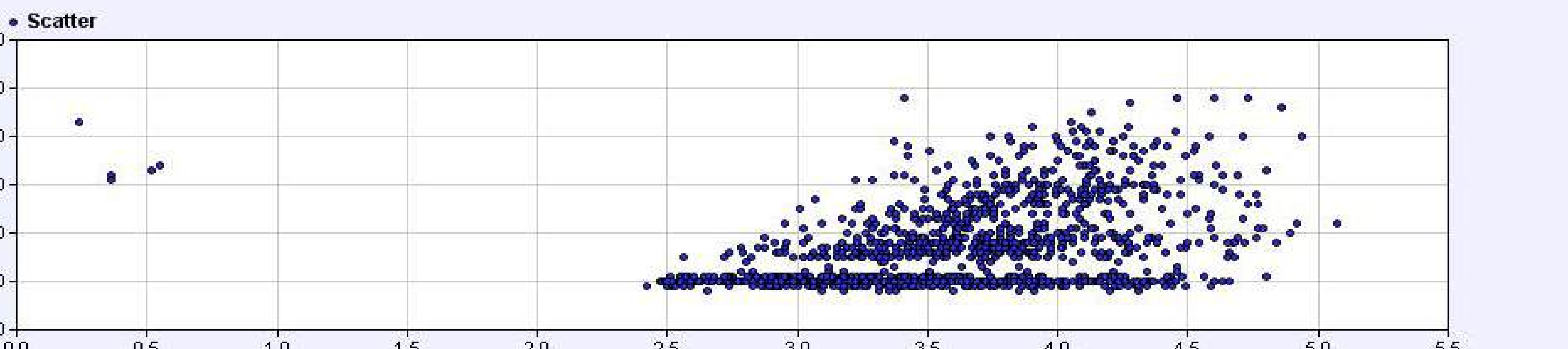
GV-MM5C-3-L15-8-1 From **Jun 25 03:15, 2014** to **Jul 08 02:00, 2014** (Duration **12** Days; 23 Hours; 45 Minutes)



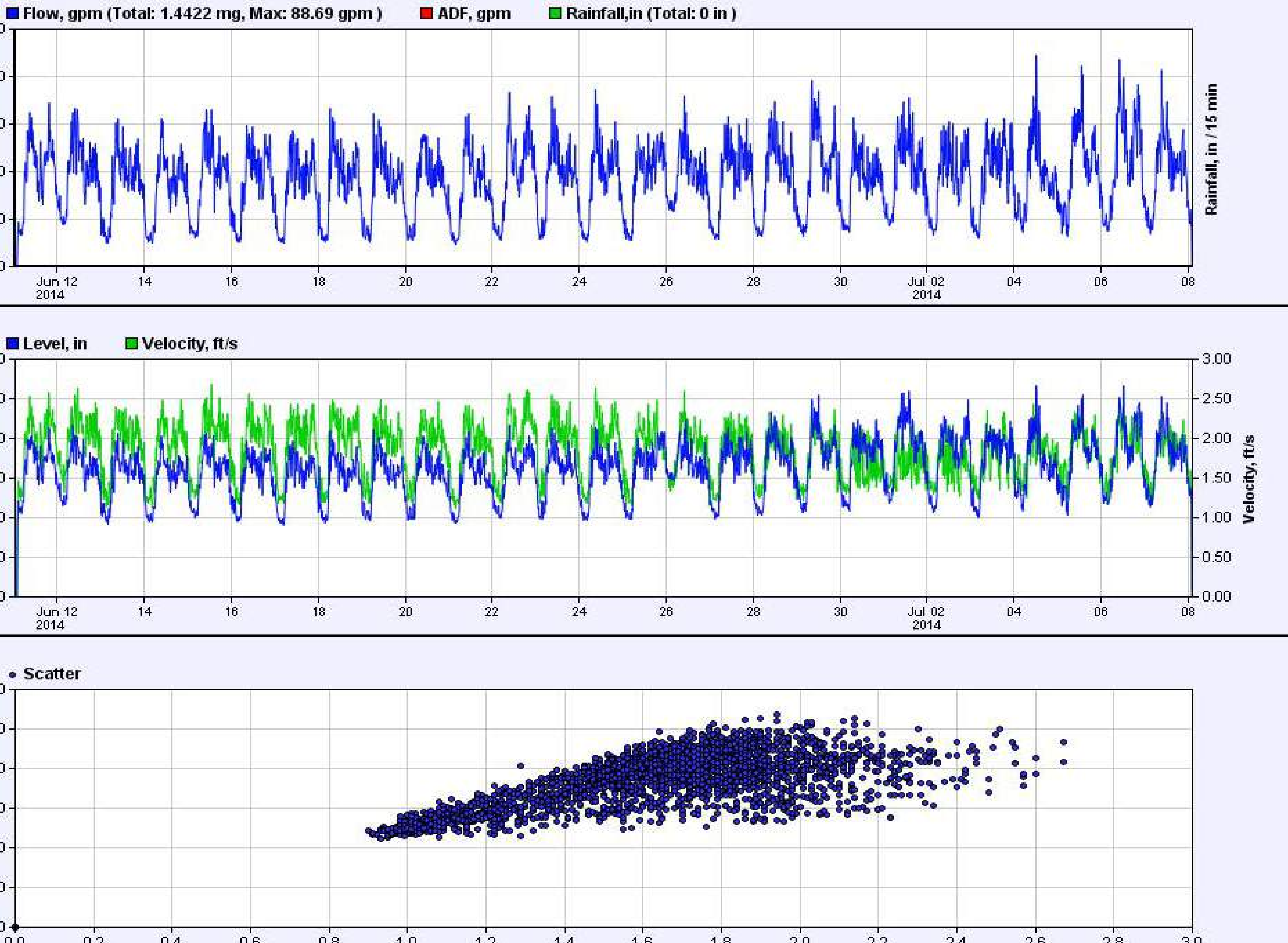
GV-MM5C-4-M15-28 From **Jun 25 03:30, 2014** to **Jul 08 00:15, 2014** (Duration **12** Days; 21 Hours; 45 Minutes)

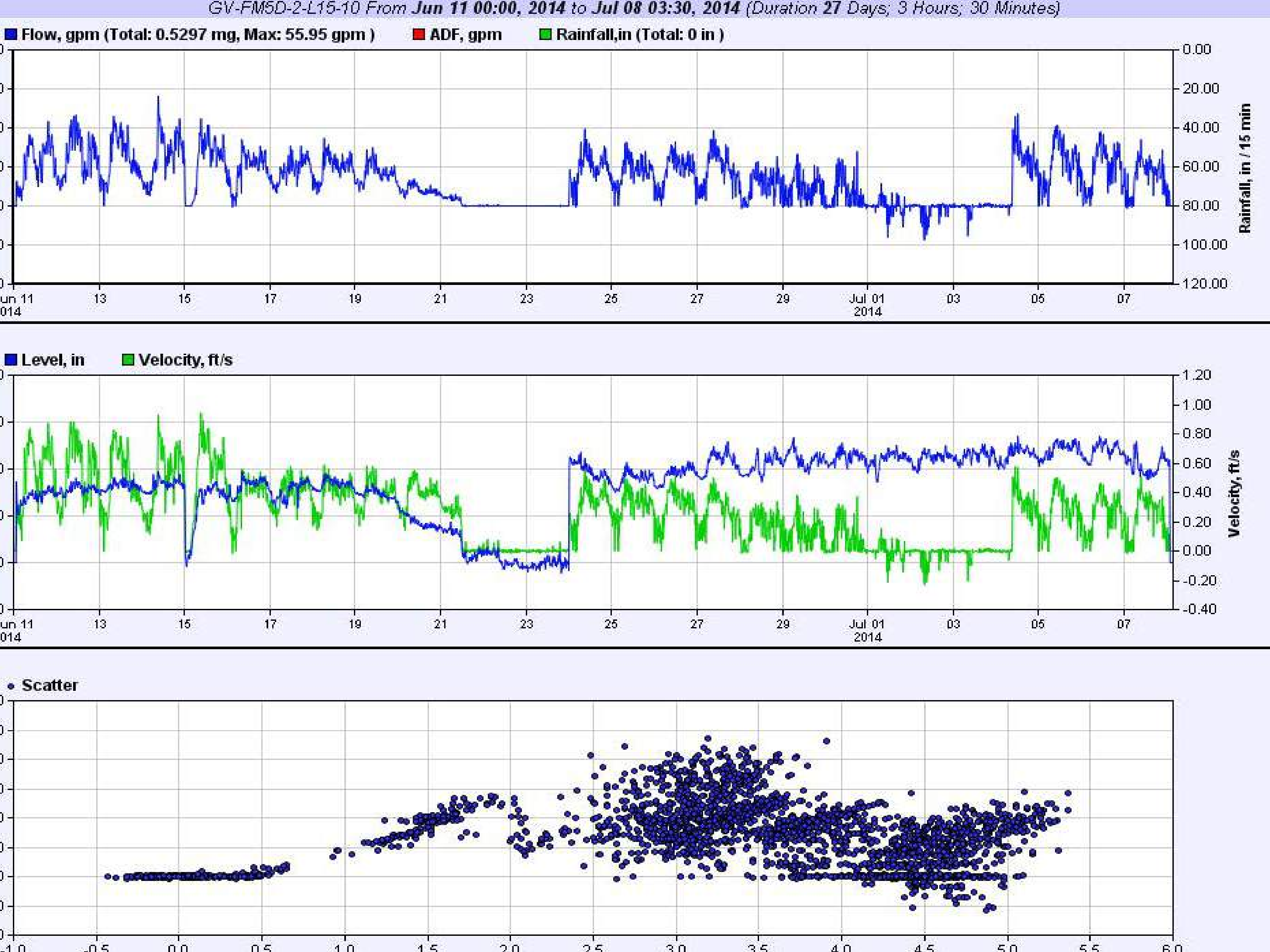




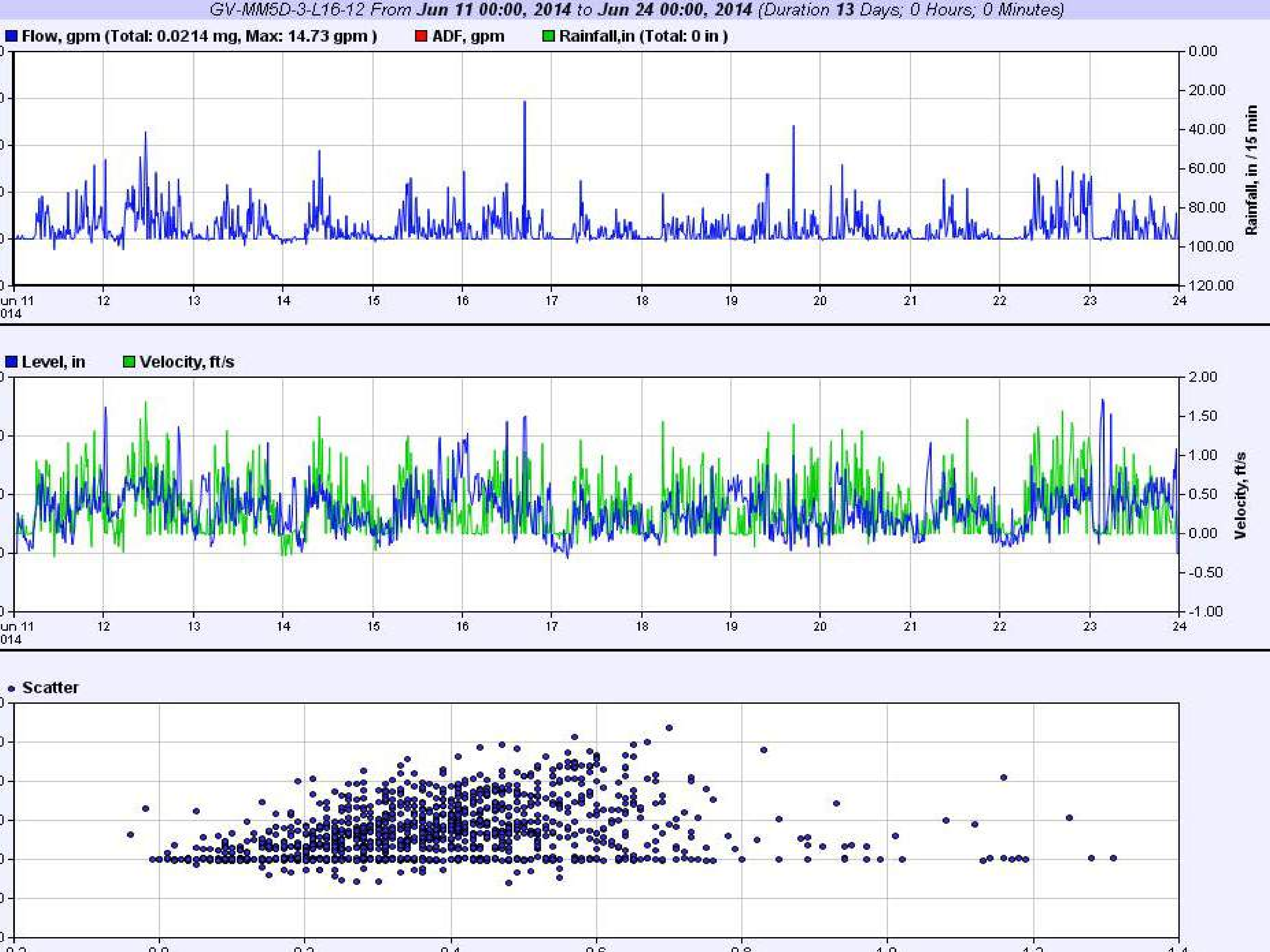


GV-MM5D-1-L15-9 From **Jun 11 00:45, 2014** to **Jul 08 02:30, 2014** (Duration **27** Days; 2 Hours; 45 Minutes)

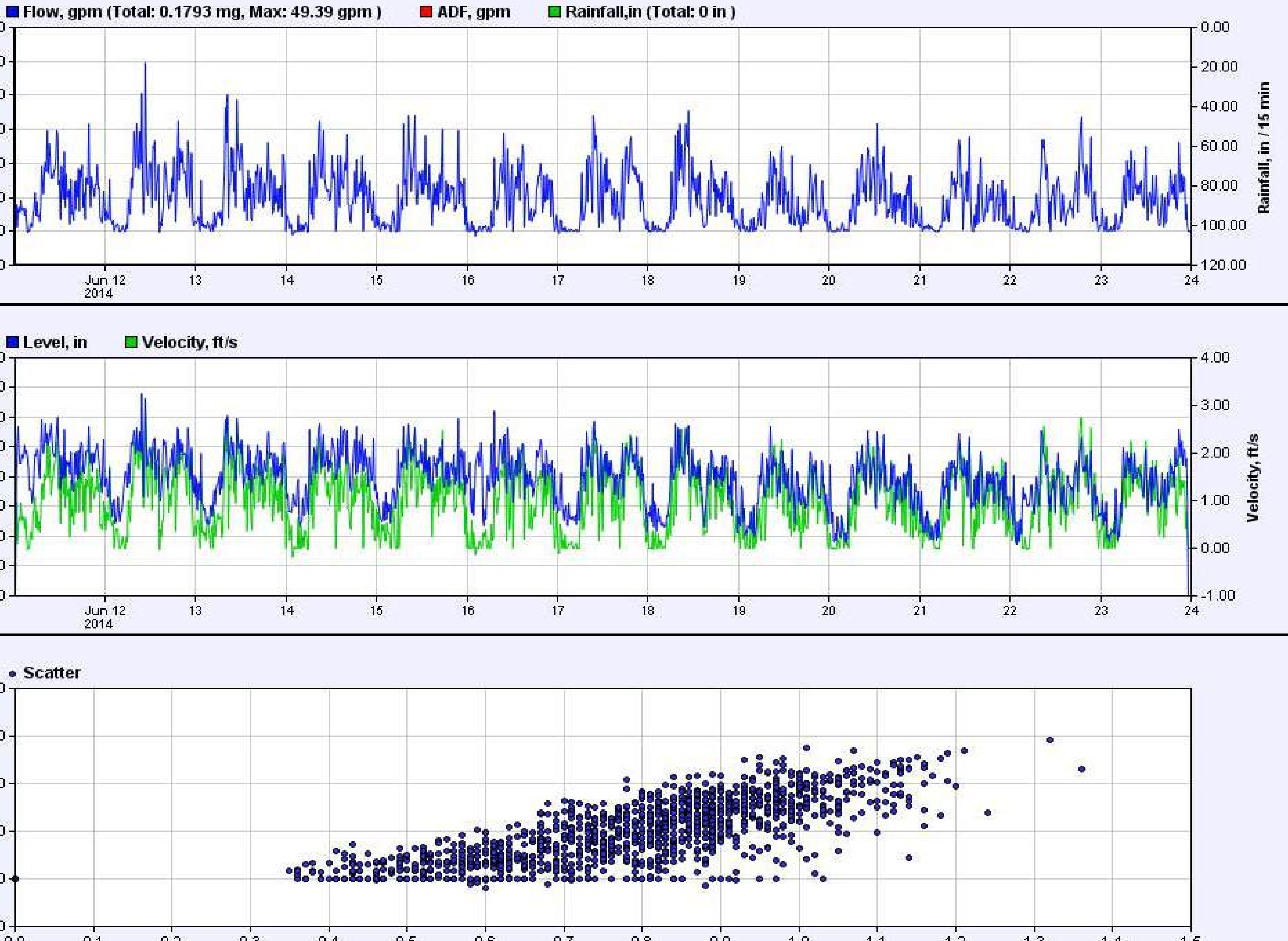


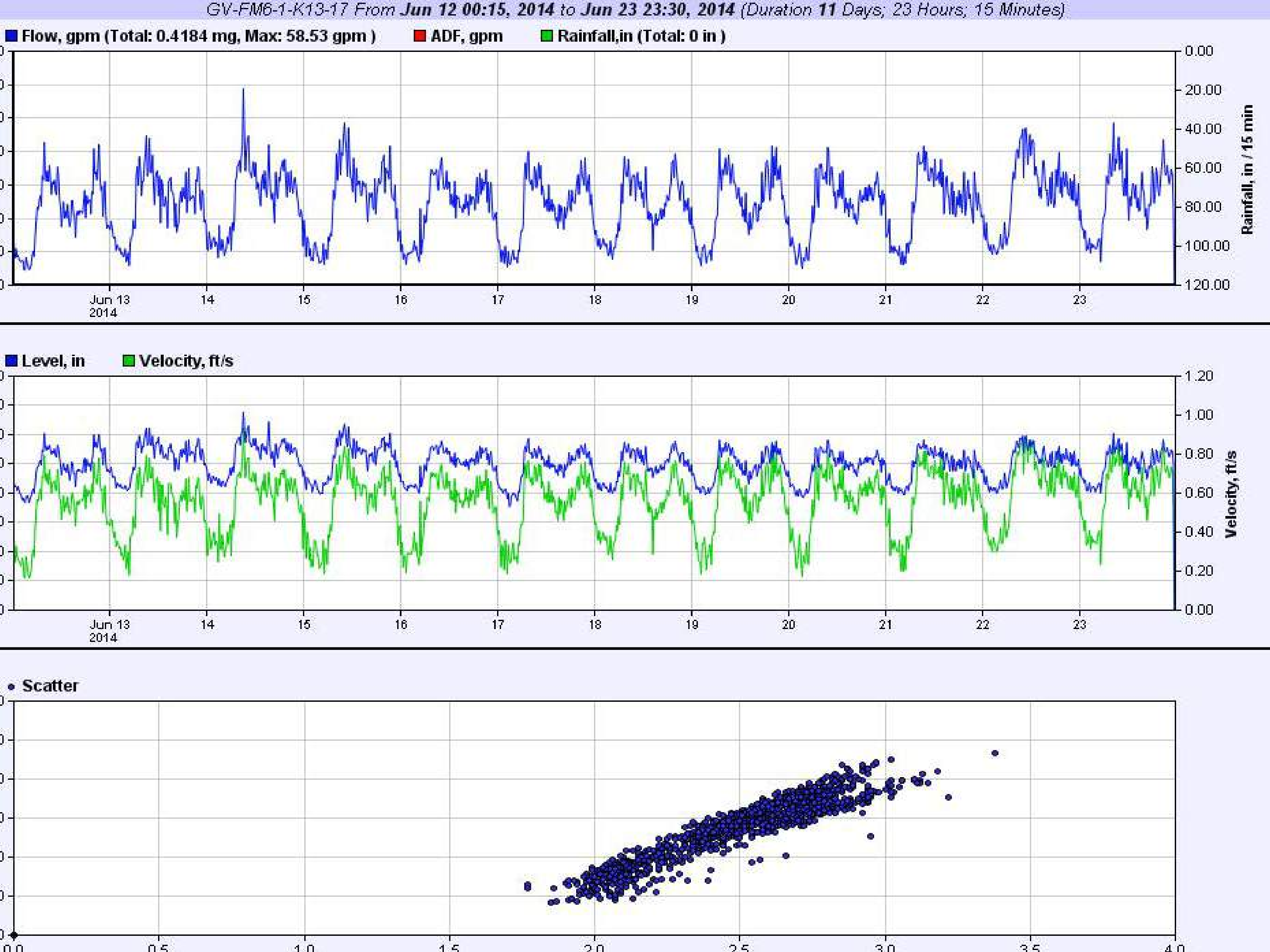


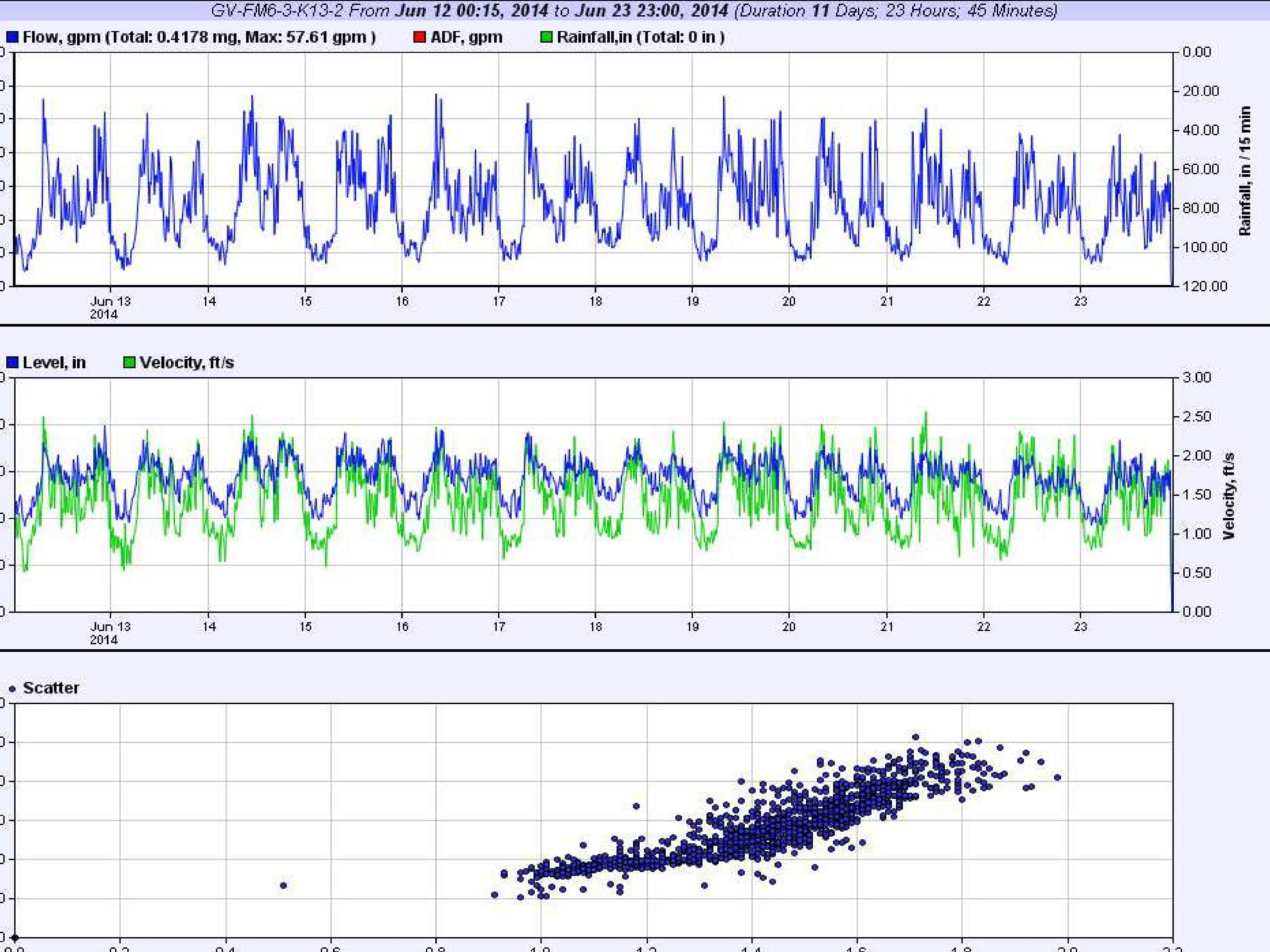
GV-FM5D-2-L15-10 From Jun 11 00:00, 2014 to Jul 08 03:30, 2014 (Duration 27 Days; 3 Hours; 30 Minutes)
Flow, gpm (Total: 0.5297 mg, Max: 55.95 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
Level, in
Velocity, ft/s
Velocity, ft/s
Scatter


GV-MM5D-3-L16-12 From Jun 11 00:00, 2014 to Jun 24 00:00, 2014 (Duration 13 Days; 0 Hours; 0 Minutes)
Flow, gpm (Total: 0.0214 mg, Max: 14.73 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 11 2014
12
13
14
15
16
17
18
19
20
21
22
23
24
Level, in
Velocity, ft/s
Velocity, ft/s
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
Jun 11 2014
12
13
14
15
16
17
18
19
20
21
22
23
24
Scatter

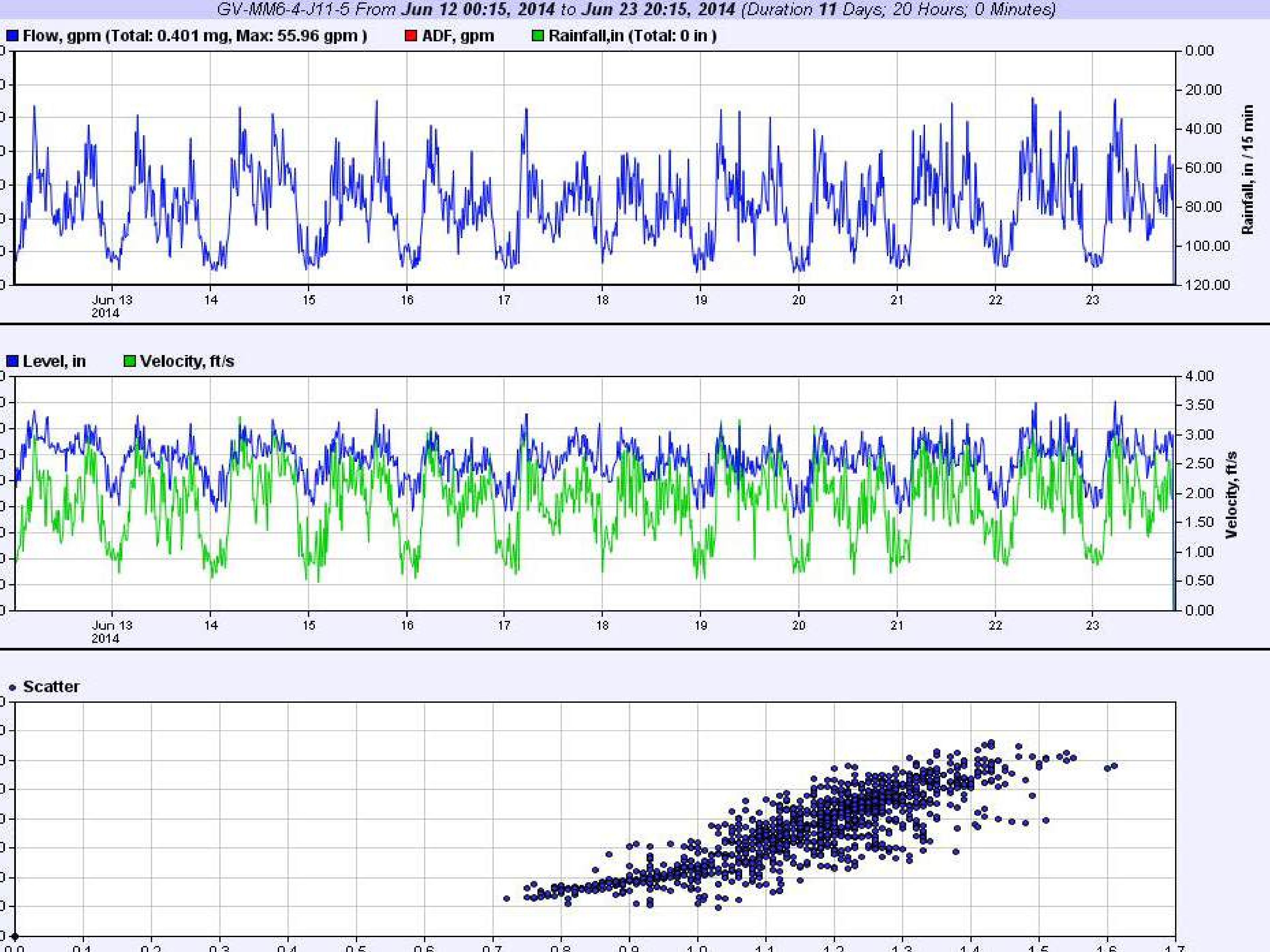
GV-MM5D-4-L16-12 From **Jun 11 00:15, 2014** to **Jun 24 00:00, 2014** (Duration **13** Days; 0 Hours; 45 Minutes)

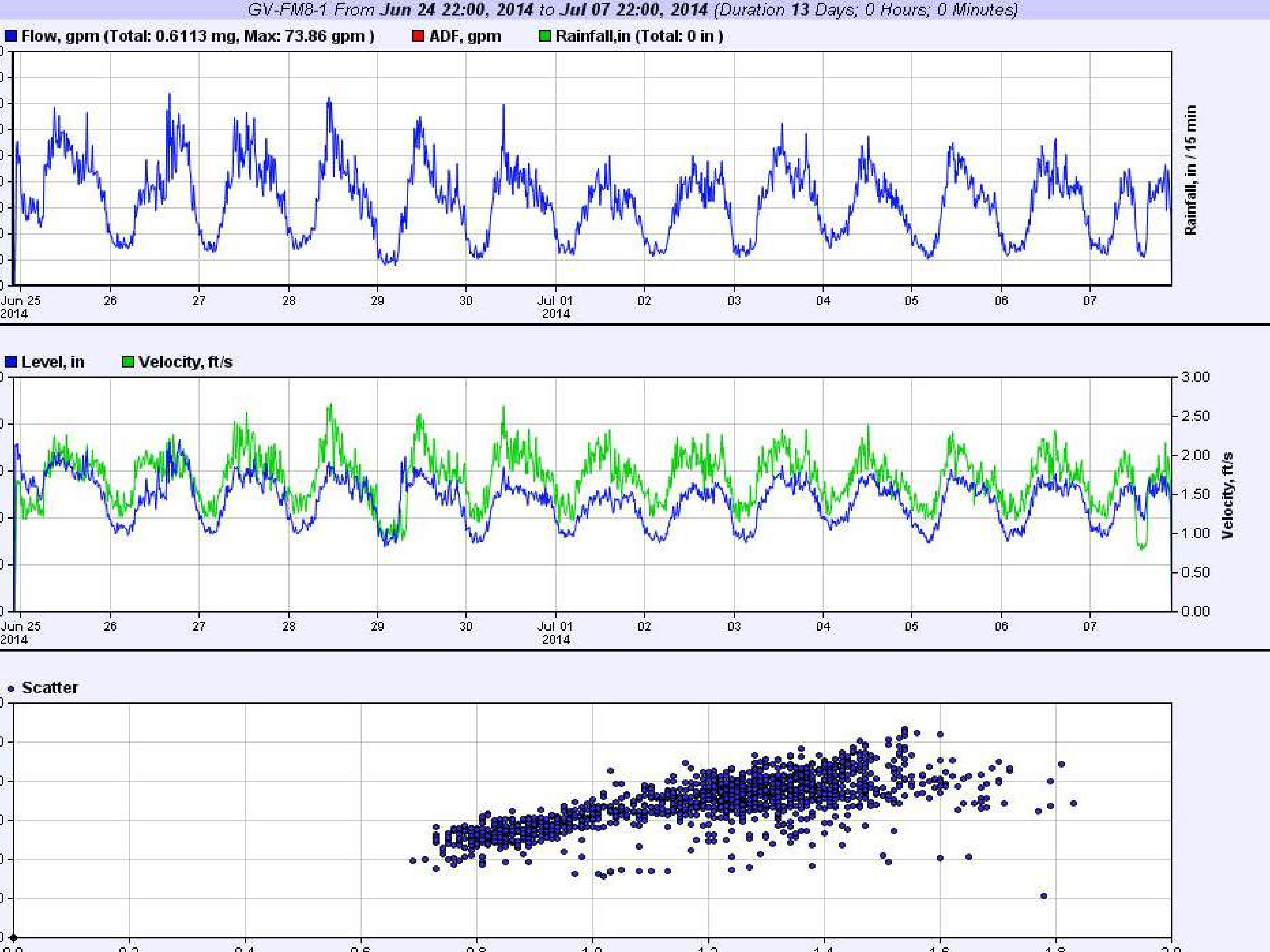



GV-FM6-1-K13-17 From Jun 12 00:15, 2014 to Jun 23 23:30, 2014 (Duration 11 Days, 23 Hours, 15 Minutes)
Flow, gpm (Total: 0.4184 mg, Max: 58.53 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 13
2014
14
15
16
17
18
19
20
21
22
23
Level, in
Velocity, ft/s
1.20
1.00
0.80
0.60
0.40
0.20
0.00
Scatter
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

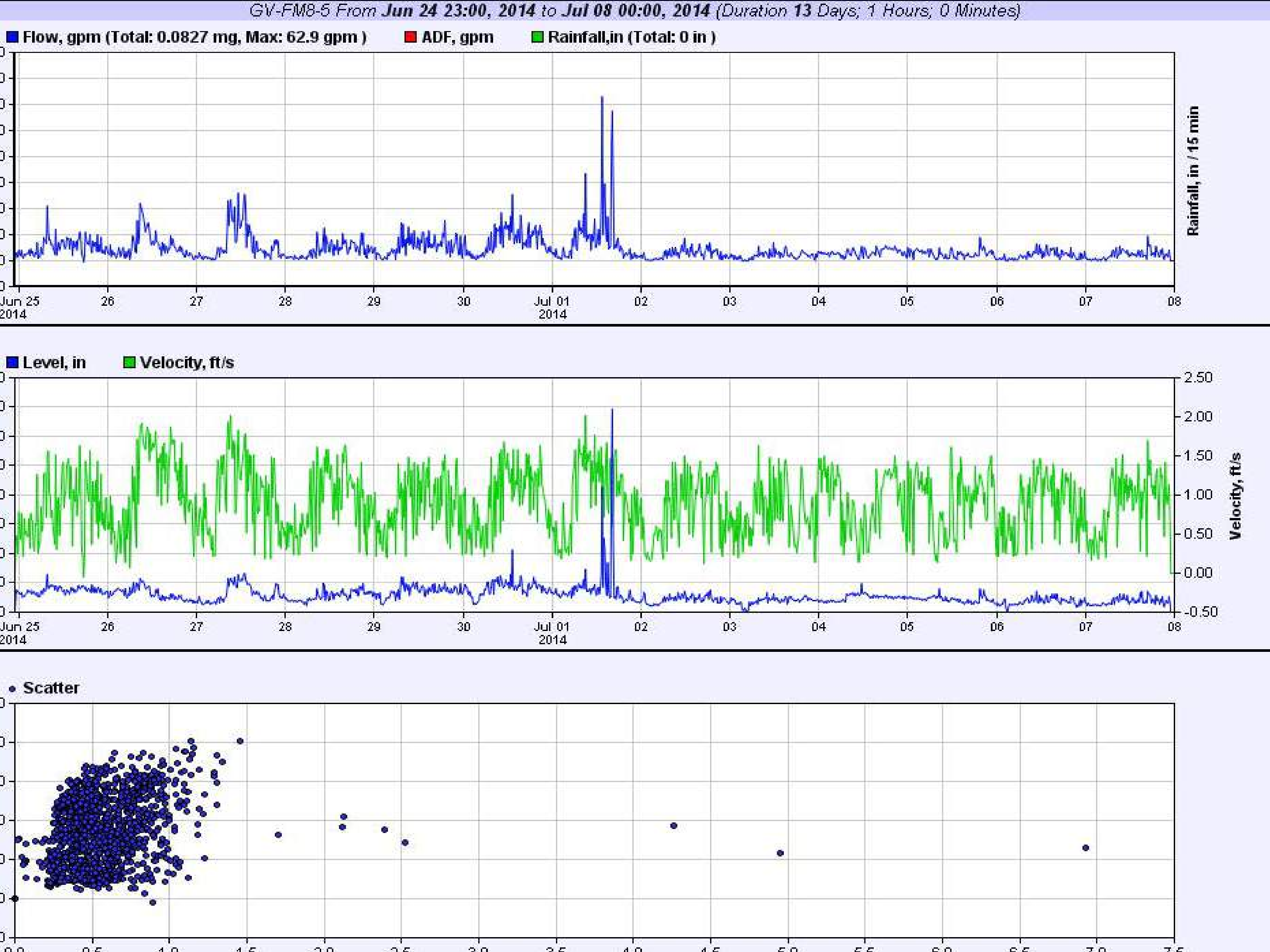


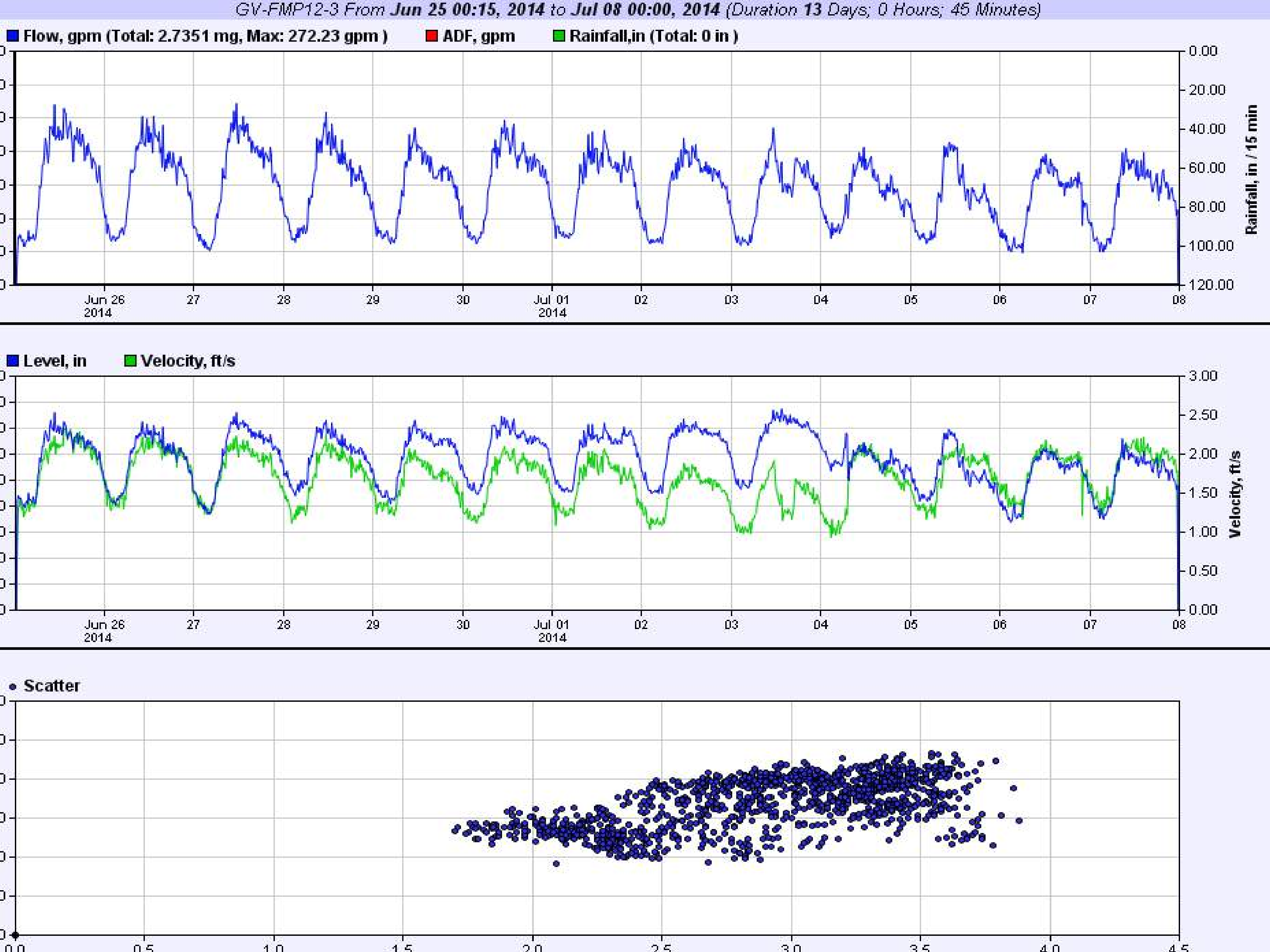
GV-FM6-3-K13-2 From Jun 12 00:15, 2014 to Jun 23 23:00, 2014 (Duration 11 Days; 23 Hours; 45 Minutes)
Flow, gpm (Total: 0.4178 mg, Max: 57.61 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 13
2014
14
15
16
17
18
19
20
21
22
23
Level, in
Velocity, ft/s
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Scatter


GV-MM6-4-J11-5 From Jun 12 00:15, 2014 to Jun 23 20:15, 2014 (Duration 11 Days, 20 Hours, 0 Minutes)
Flow, gpm (Total: 0.401 mg, Max: 55.96 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 13
2014
14
15
16
17
18
19
20
21
22
23
Level, in
Velocity, ft/s
Velocity, ft/s
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Jun 13
2014
14
15
16
17
18
19
20
21
22
23
Scatter
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7

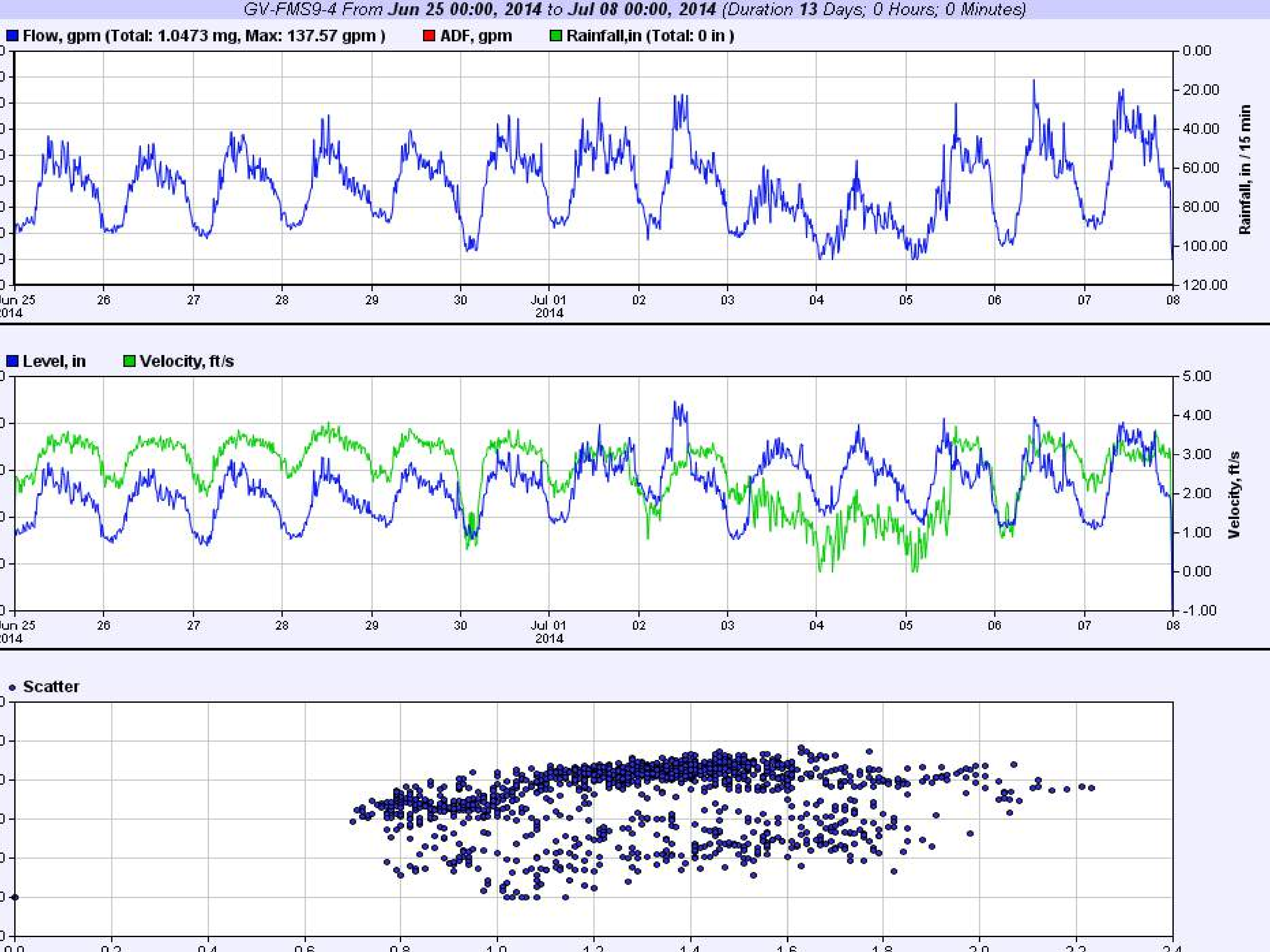


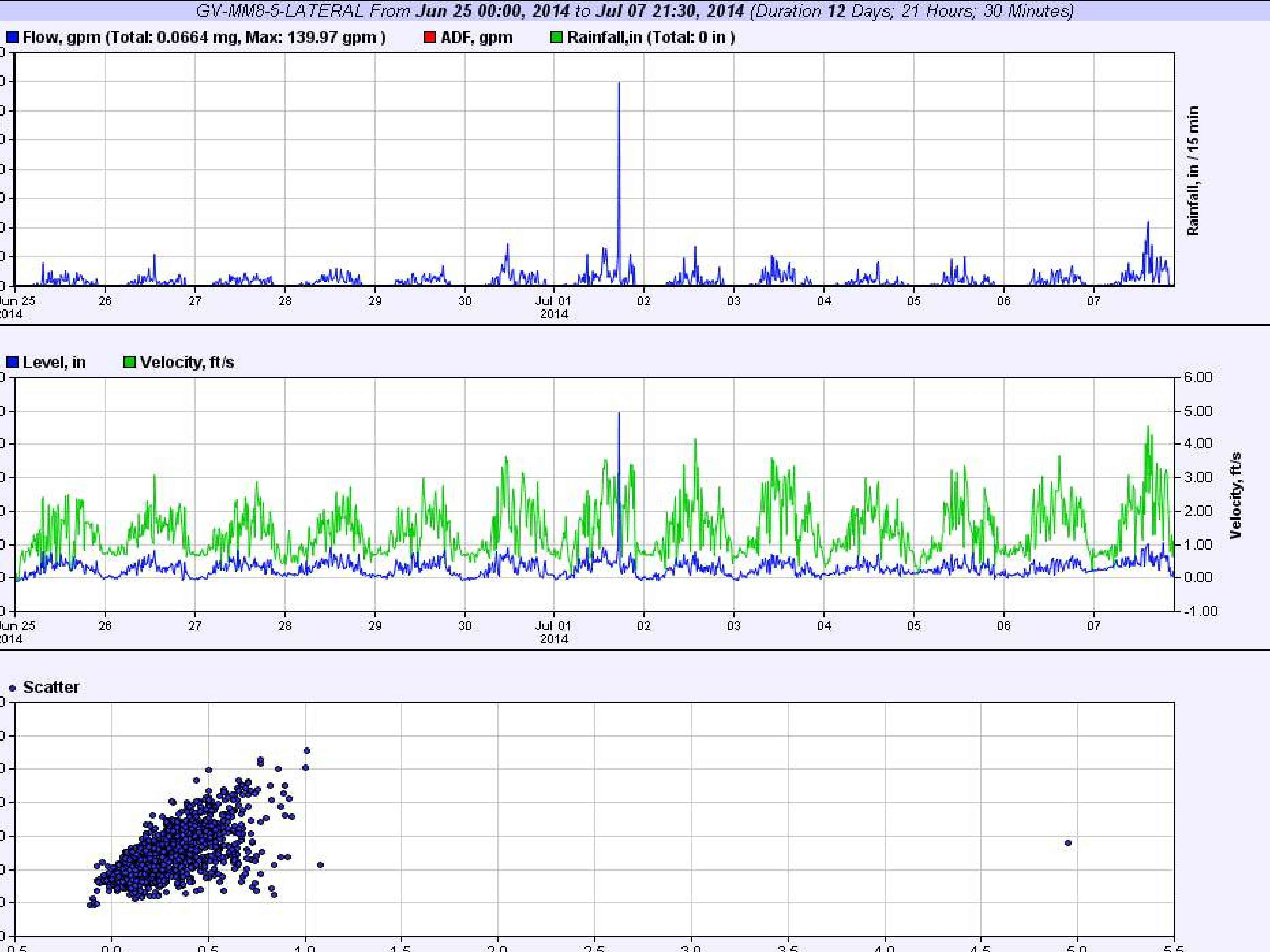
GV-FM8-1 From Jun 24 22:00, 2014 to Jul 07 22:00, 2014 (Duration 13 Days; 0 Hours; 0 Minutes)
Flow, gpm (Total: 0.6113 mg, Max: 73.86 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
Jun 25 2014
26
27
28
29
30
Jul 01 2014
02
03
04
05
06
07
Level, in
Velocity, ft/s
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Velocity, ft/s
Scatter


GV-FM8-5 From Jun 24 23:00, 2014 to Jul 08 00:00, 2014 (Duration 13 Days; 1 Hours; 0 Minutes)
Flow, gpm (Total: 0.0827 mg, Max: 62.9 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
Jun 25 2014
26
27
28
29
30
Jul 01 2014
02
03
04
05
06
07
08
Level, in
Velocity, ft/s
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
Velocity, ft/s
Scatter



GV-FMP12-3 From Jun 25 00:15, 2014 to Jul 08 00:00, 2014 (Duration 13 Days; 0 Hours; 45 Minutes)
Flow, gpm (Total: 2.7351 mg, Max: 272.23 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 26
2014
27
28
29
30
Jul 01
2014
02
03
04
05
06
07
08
Level, in
Velocity, ft/s
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Scatter
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5

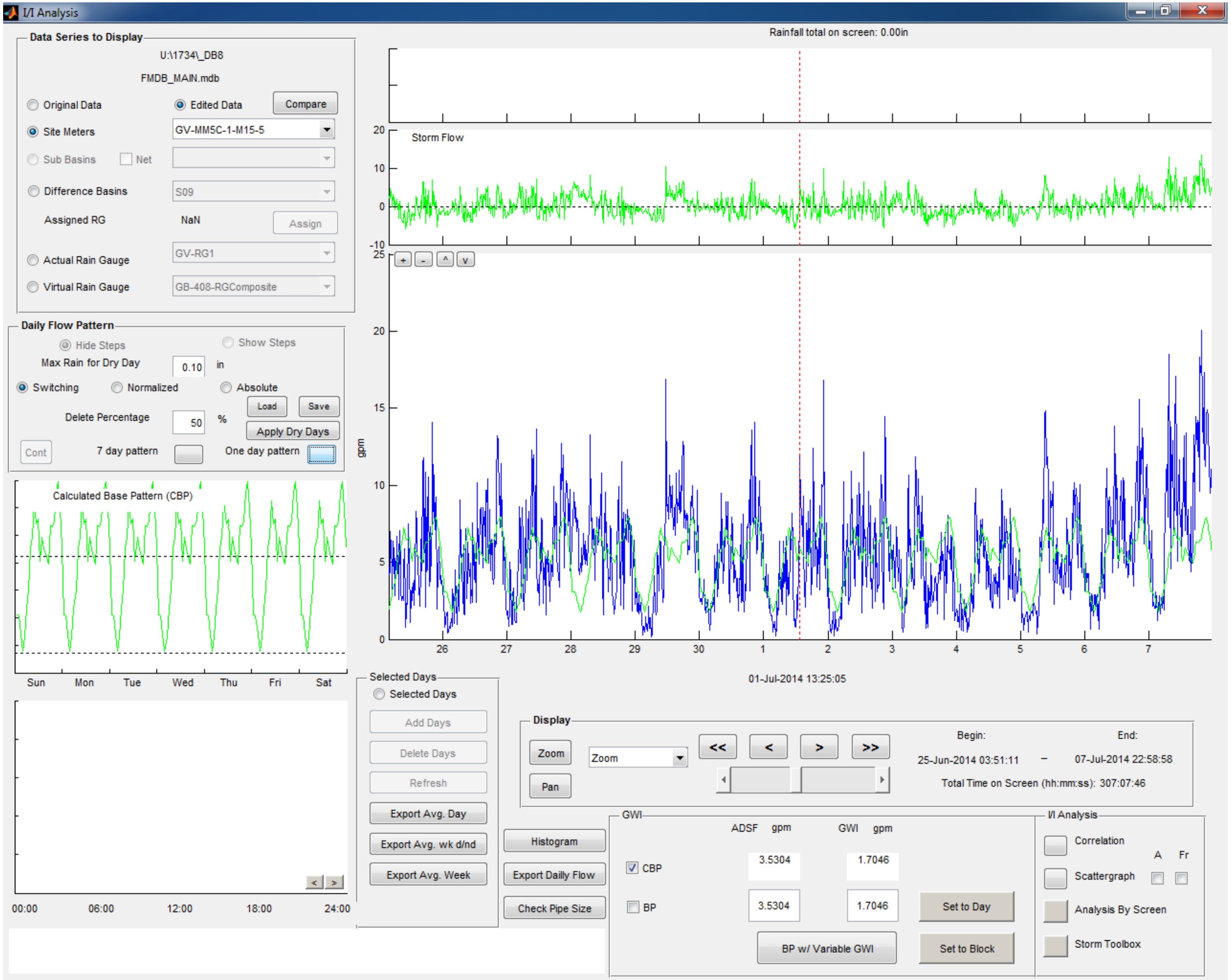

GV-FMS9-4 From Jun 25 00:00, 2014 to Jul 08 00:00, 2014 (Duration 13 Days; 0 Hours; 0 Minutes)
Flow, gpm (Total: 1.0473 mg, Max: 137.57 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Jun 25 2014
26
27
28
29
30
Jul 01 2014
02
03
04
05
06
07
08
Level, in
Velocity, ft/s
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
Scatter
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4


GV-MM8-5-LATERAL From Jun 25 00:00, 2014 to Jul 07 21:30, 2014 (Duration 12 Days; 21 Hours; 30 Minutes)
Flow, gpm (Total: 0.0664 mg, Max: 139.97 gpm)
ADF, gpm
Rainfall,in (Total: 0 in)
Rainfall, in / 15 min
Jun 25 2014
26
27
28
29
30
Jul 01 2014
02
03
04
05
06
07
Level, in
Velocity, ft/s
Velocity, ft/s
6.00
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
Jun 25 2014
26
27
28
29
30
Jul 01 2014
02
03
04
05
06
07
Scatter
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5

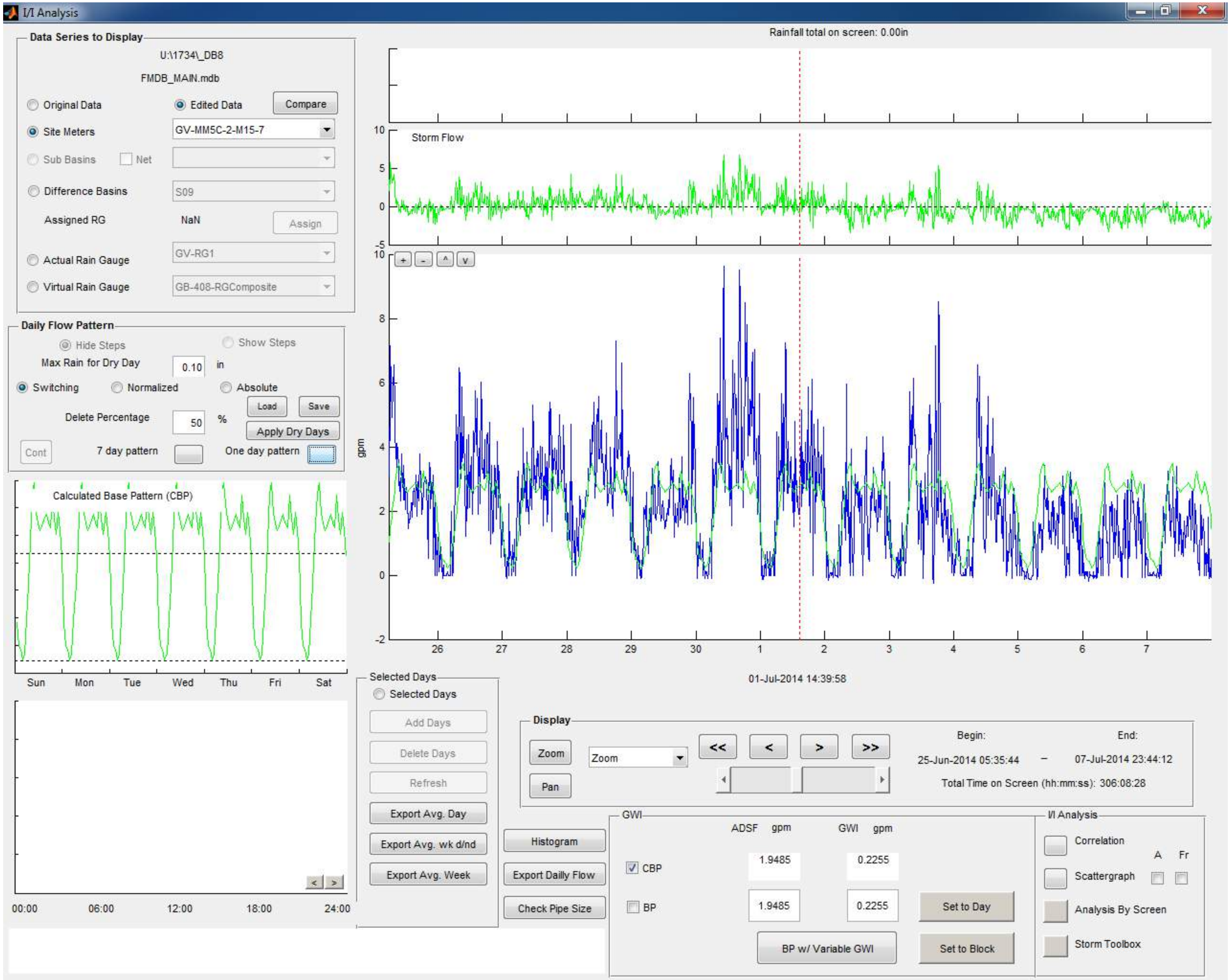
ATTACHMENT D

GWI Analysis Records

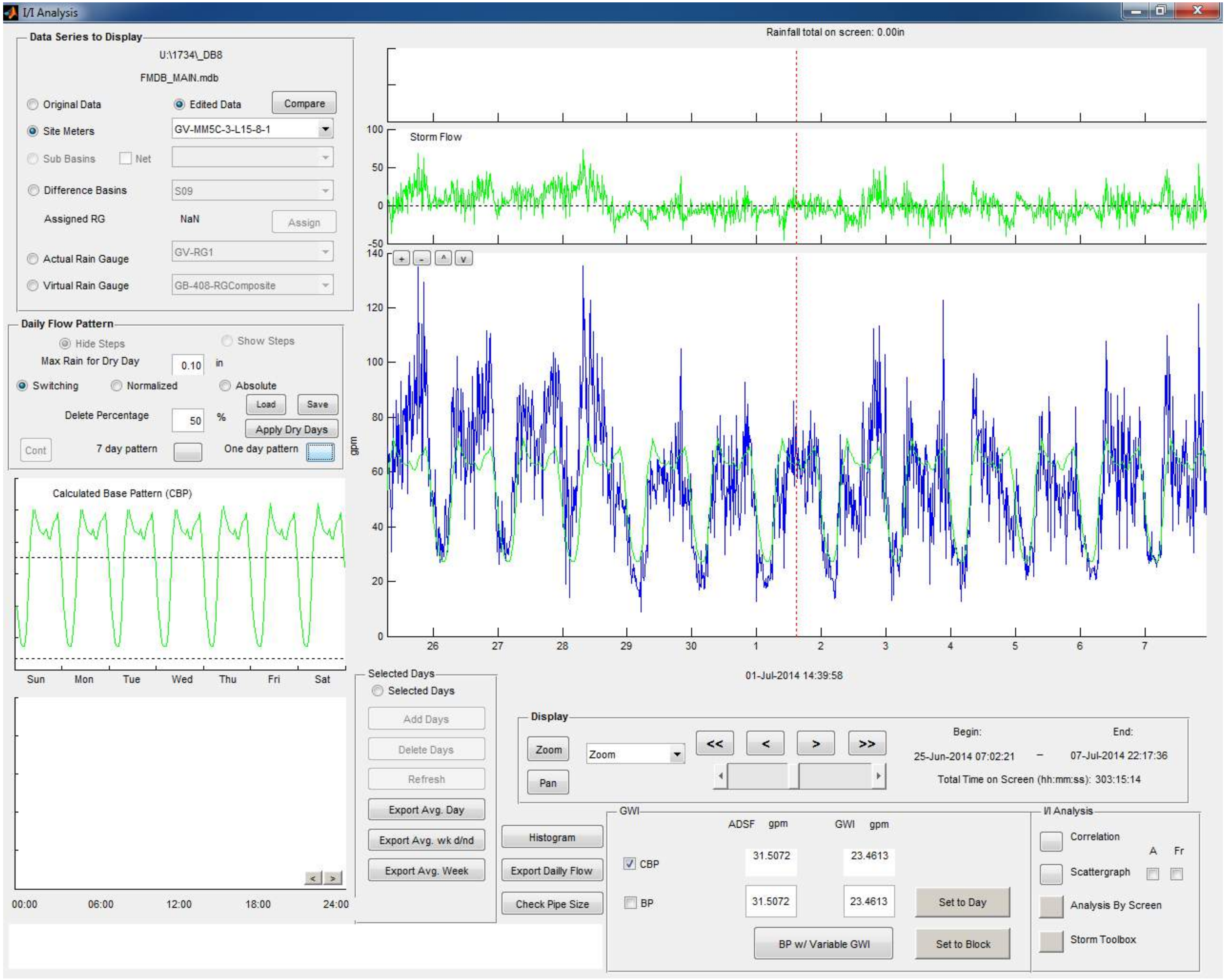
Basin 5 Phase 2
Dry Weather Flows



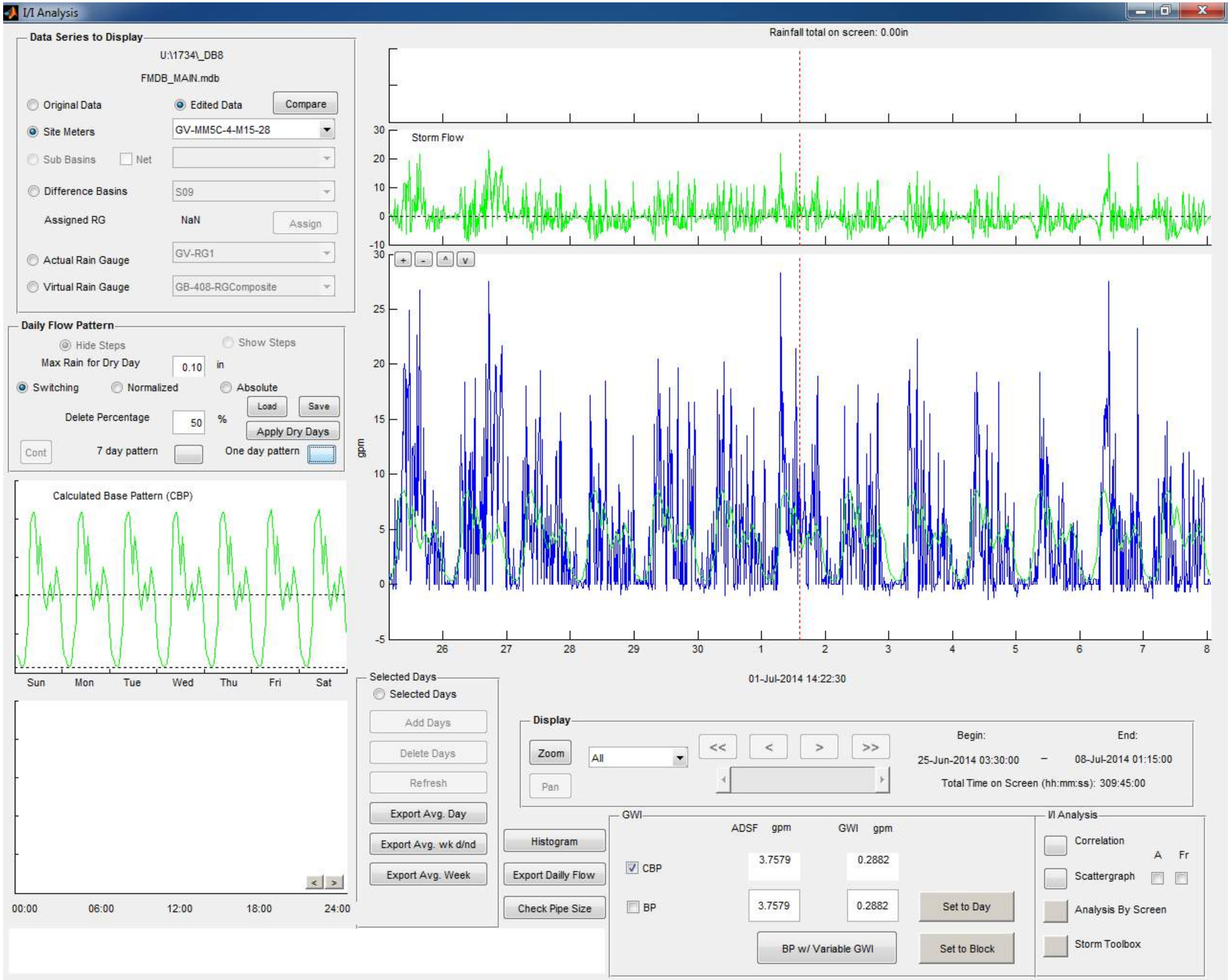
GV-MM5C-1-M15-5



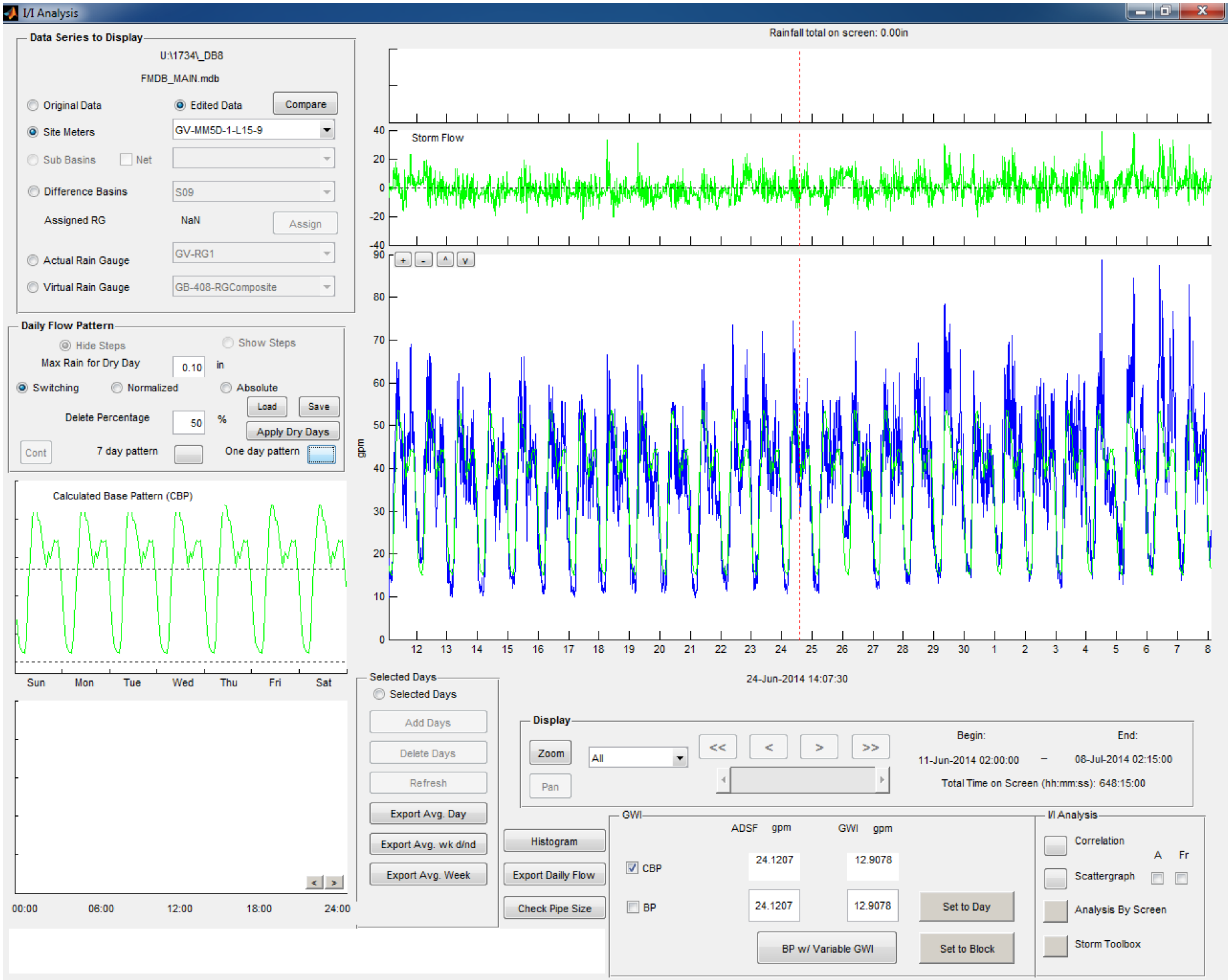
GV-MM5C-2-M15-7



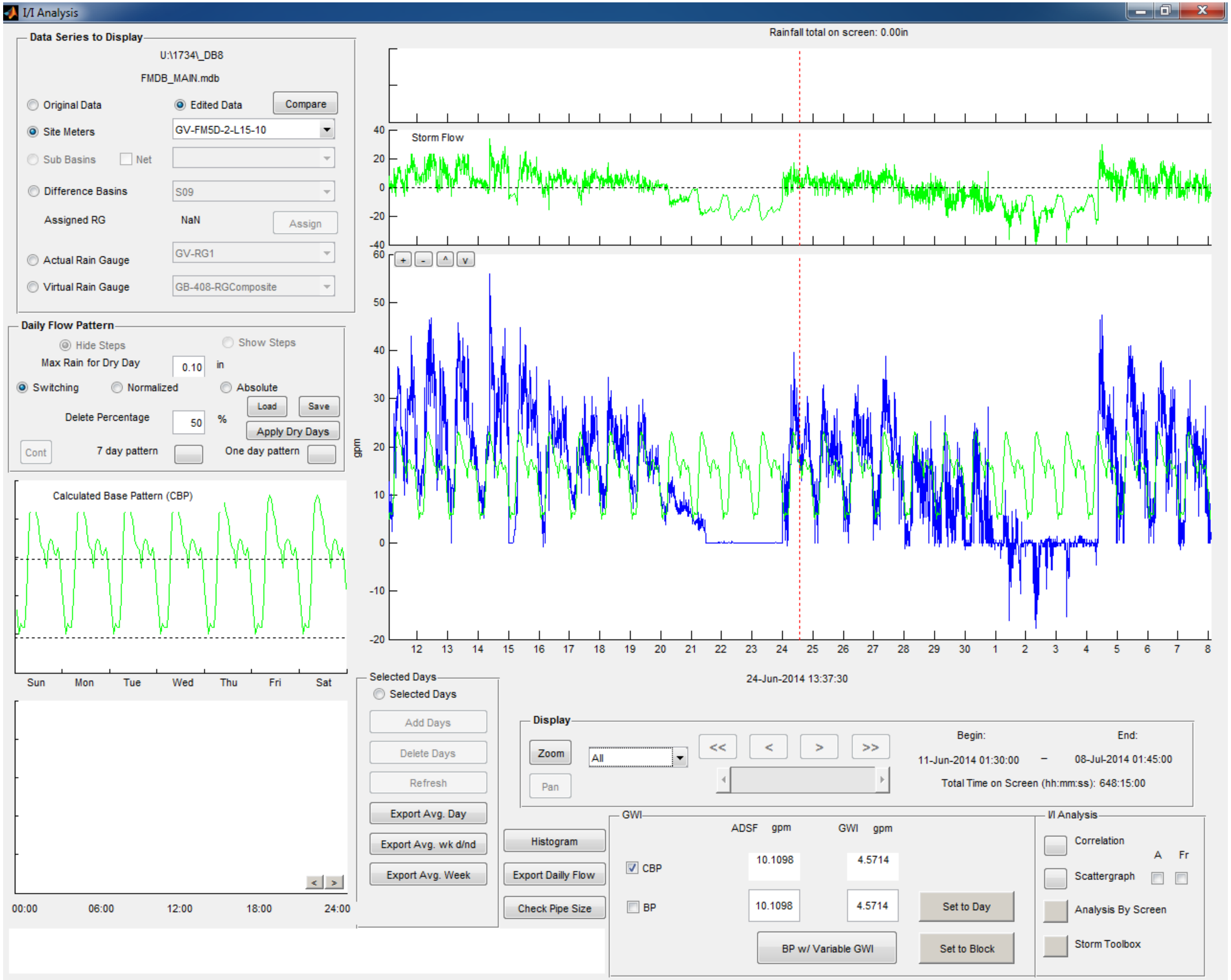
GV-MM5C-3-L15-8-1



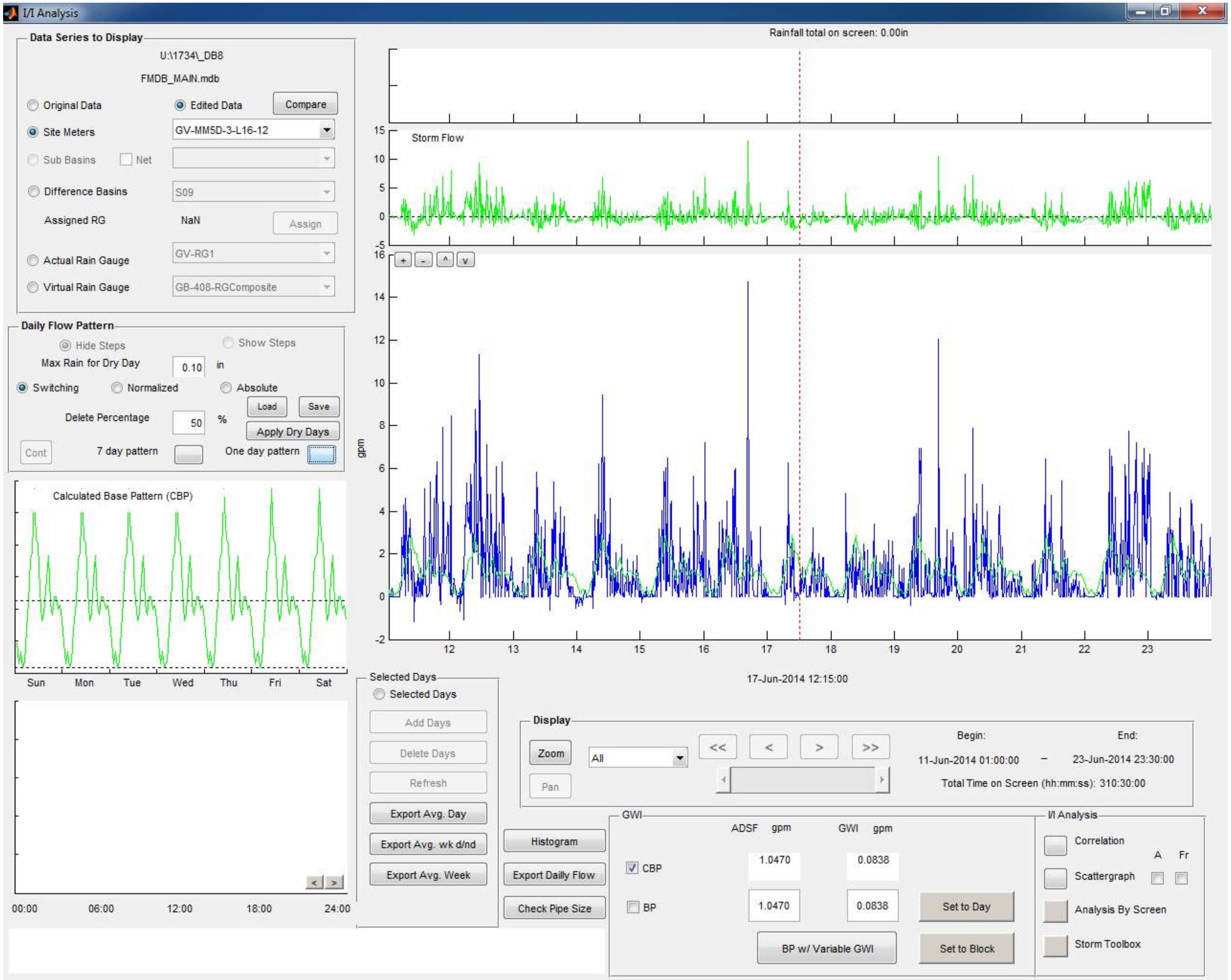
GV-MM5C-4-M15-28



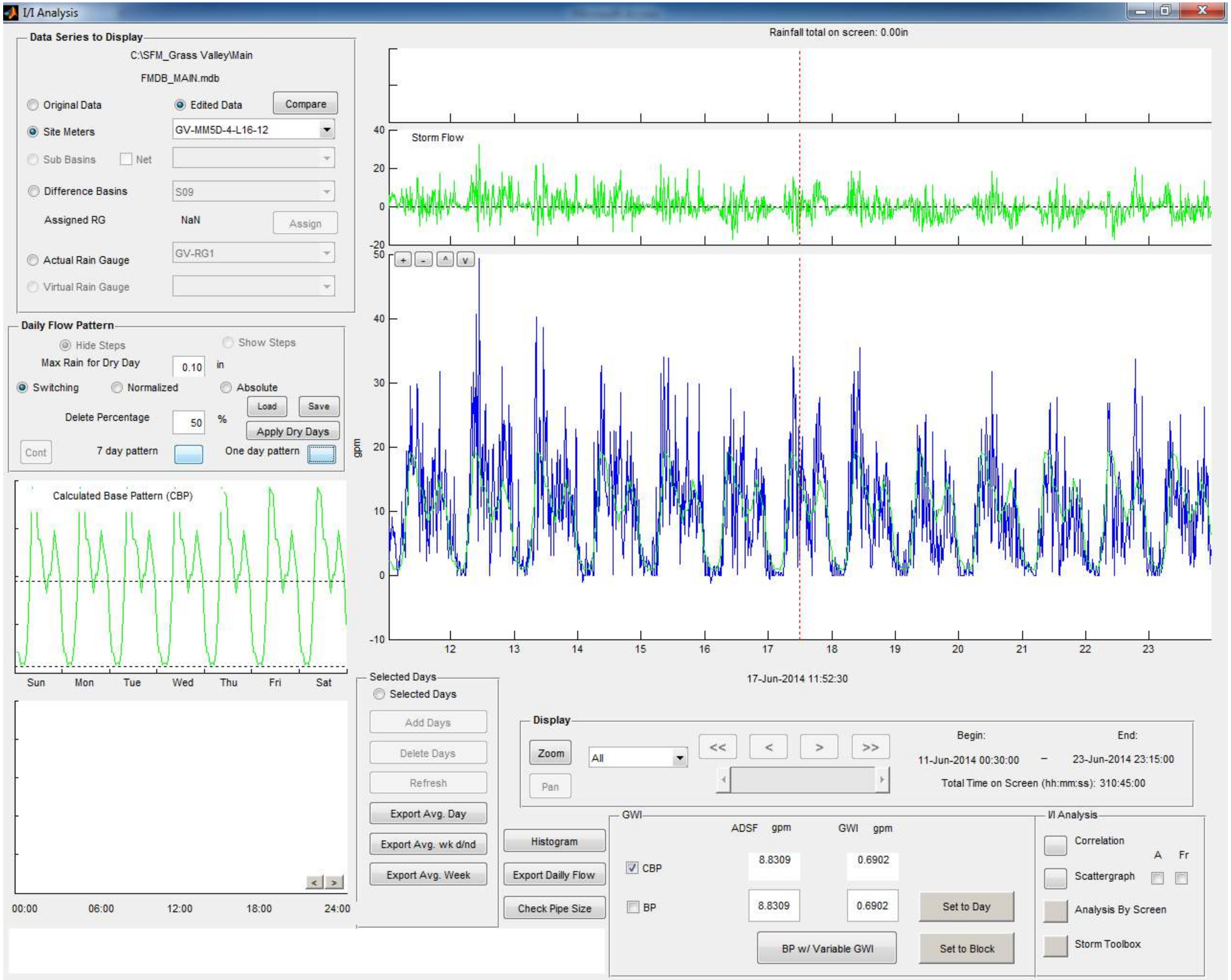
GV-MM5D-1-L15-9



GV-FM5D-2-L15-10

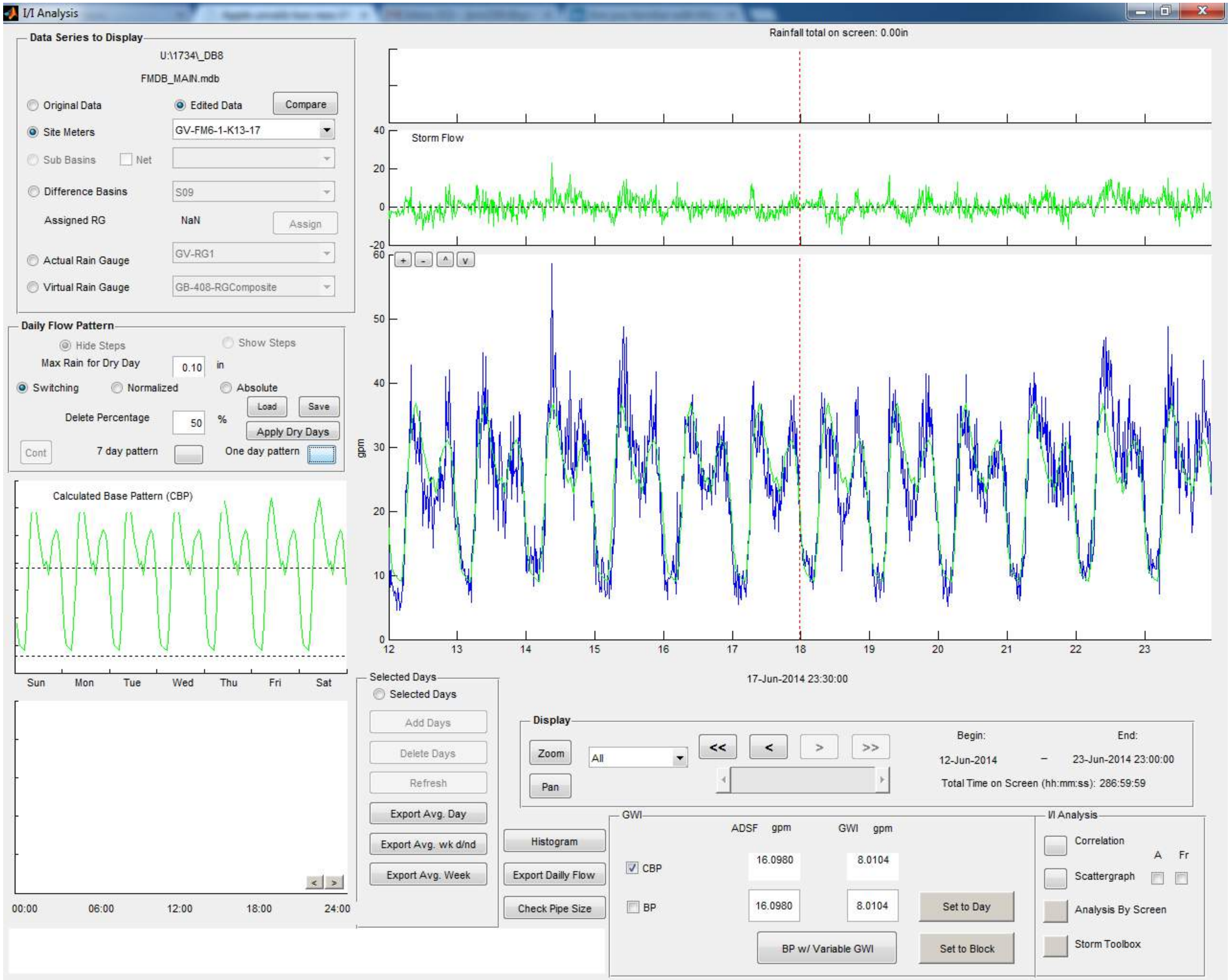


GV-MM5D-3-L16-12



GV-MM5D-4-L16-12

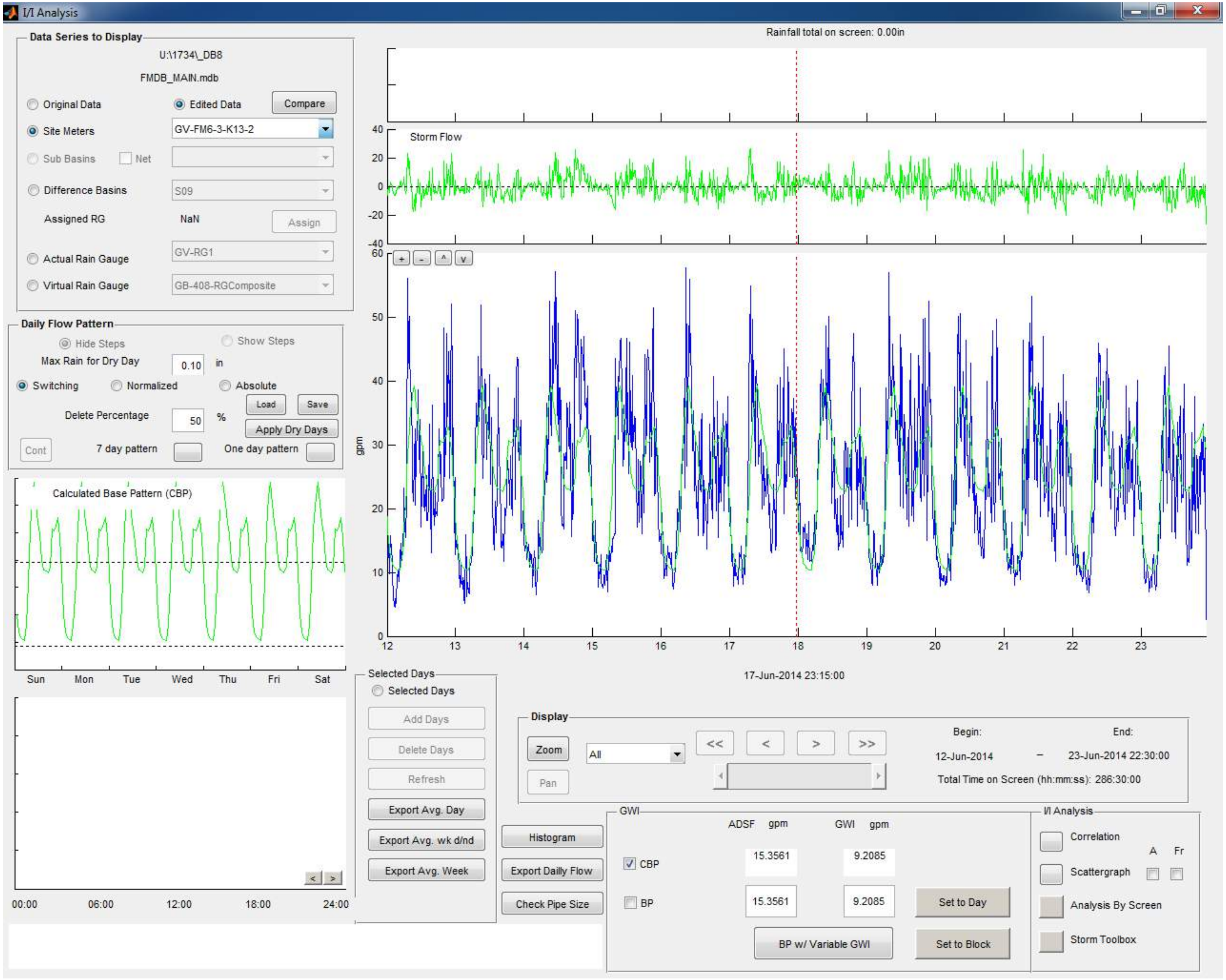
Basin 6
Dry Weather Flows



GV-FM6-1-K13-17

No good data available. Level sensor failed during the program.

GV-FM6-2-J13-5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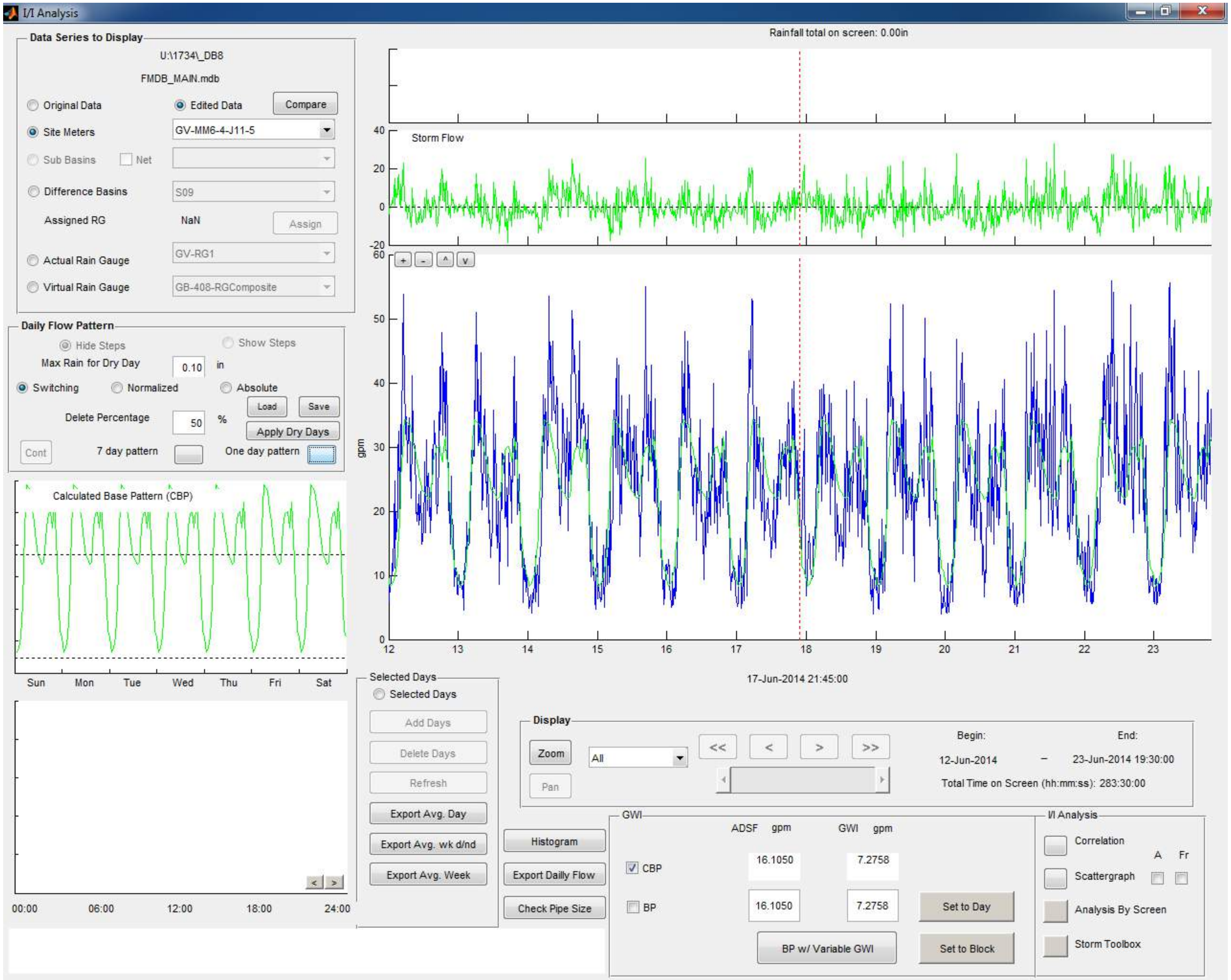
Edited Data
Compare
Site Meters
GV-FM6-3-K13-2
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun
Mon
Tue
Wed
Thu
Fri
Sat
00:00
06:00
12:00
18:00
24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
17-Jun-2014 23:15:00
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
All
Pan
<<
<
>
>>
Begin:
End:
12-Jun-2014
23-Jun-2014 22:30:00
Total Time on Screen (hh:mm:ss): 286:30:00
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
15.3561
9.2085
BP
15.3561
9.2085
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A
Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-FM6-3-K13-2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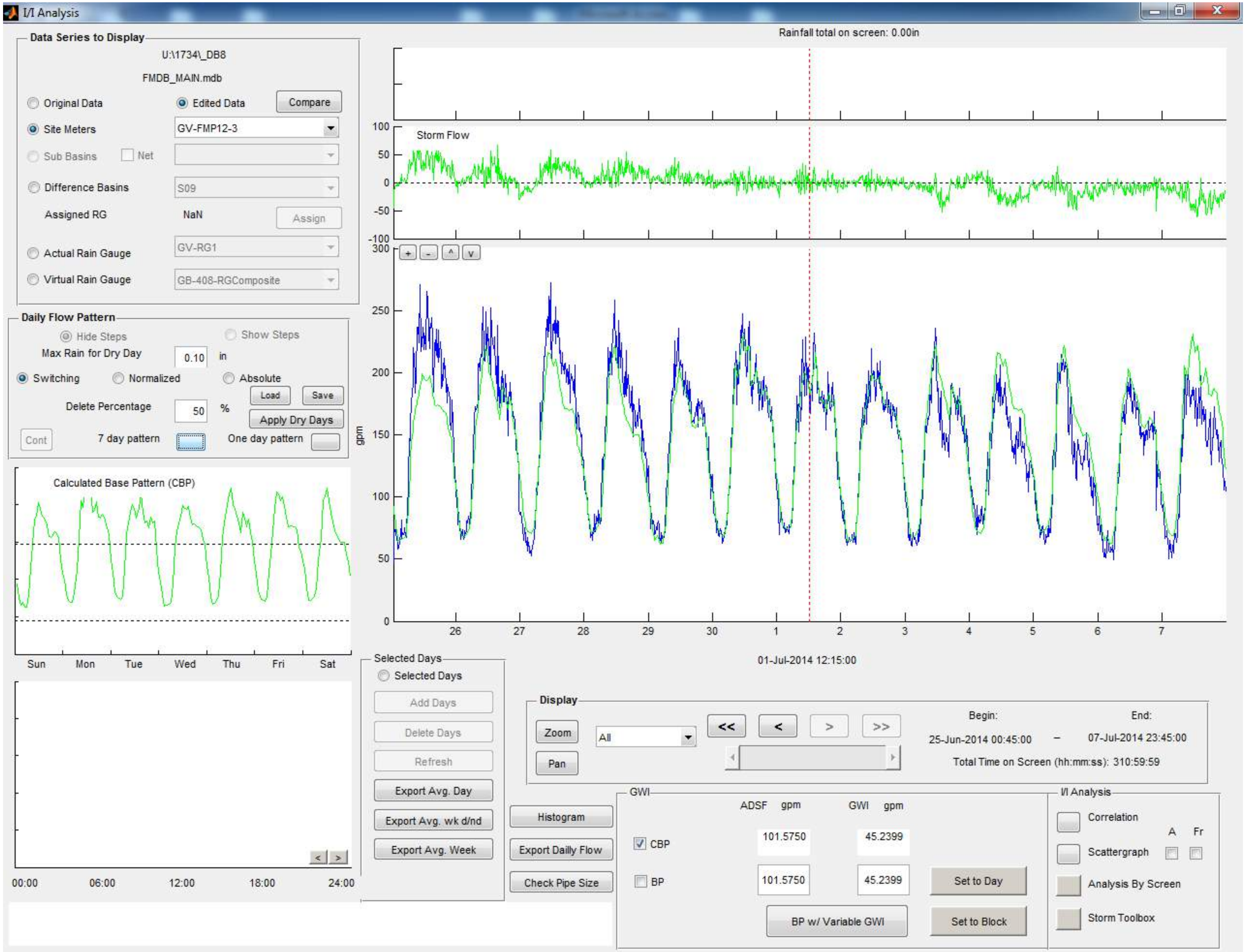
GV-MM6-4-J11-5
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun
Mon
Tue
Wed
Thu
Fri
Sat
00:00
06:00
12:00
18:00
24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
17-Jun-2014 21:45:00
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
All
Pan
Begin:
End:
12-Jun-2014
23-Jun-2014 19:30:00
Total Time on Screen (hh:mm:ss): 283:30:00
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
16.1050
7.2758
BP
16.1050
7.2758
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A
Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-MM6-4-J11-5

Basin 8
Dry Weather Flows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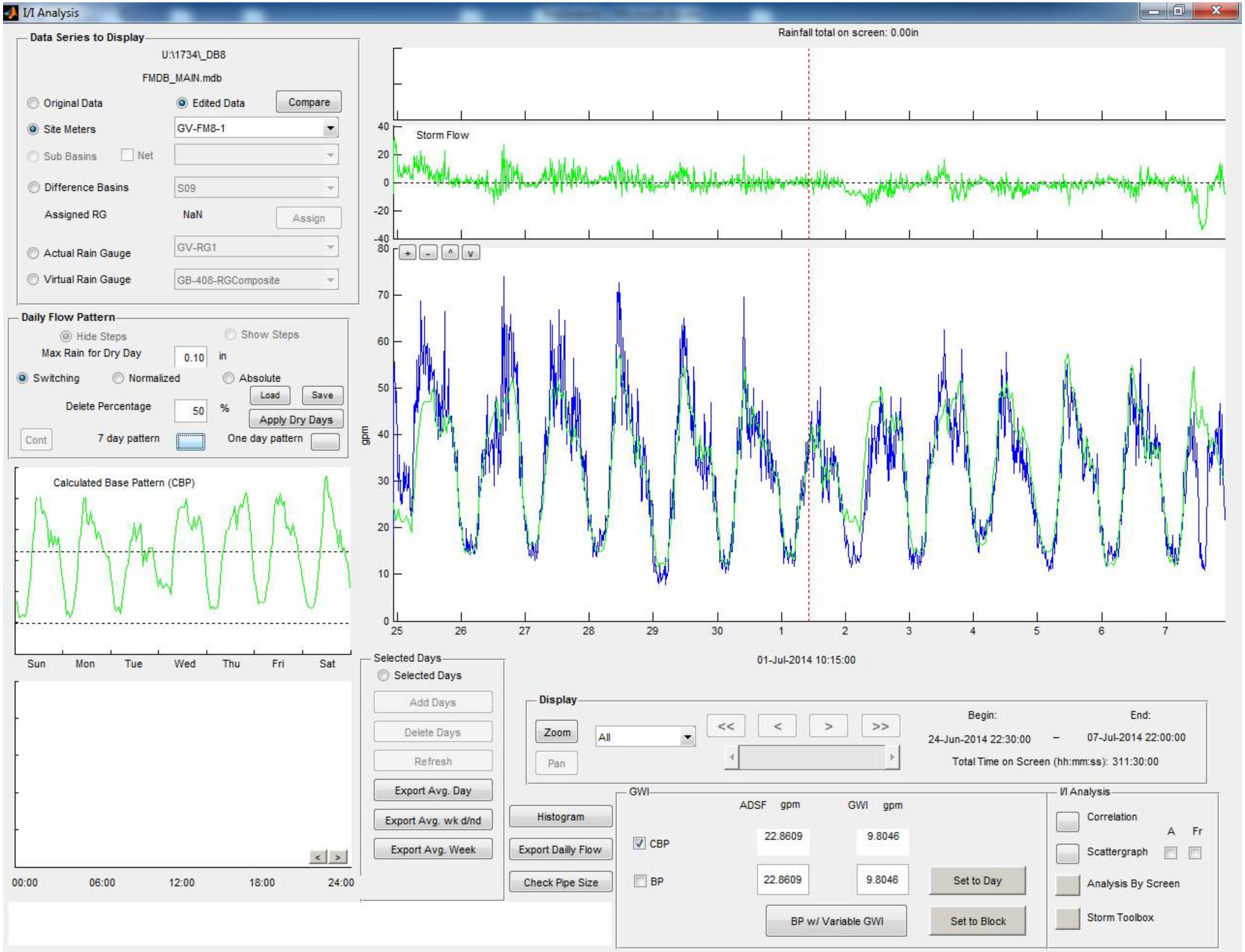
Original Data
Edited Data
Compare
Site Meters
GV-FMP12-3
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun
Mon
Tue
Wed
Thu
Fri
Sat
00:00
06:00
12:00
18:00
24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
01-Jul-2014 12:15:00
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
Pan
All
Begin:
End:
25-Jun-2014 00:45:00
07-Jul-2014 23:45:00
Total Time on Screen (hh:mm:ss): 310:59:59
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
101.5750
45.2399
BP
101.5750
45.2399
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A
Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-FMP12-3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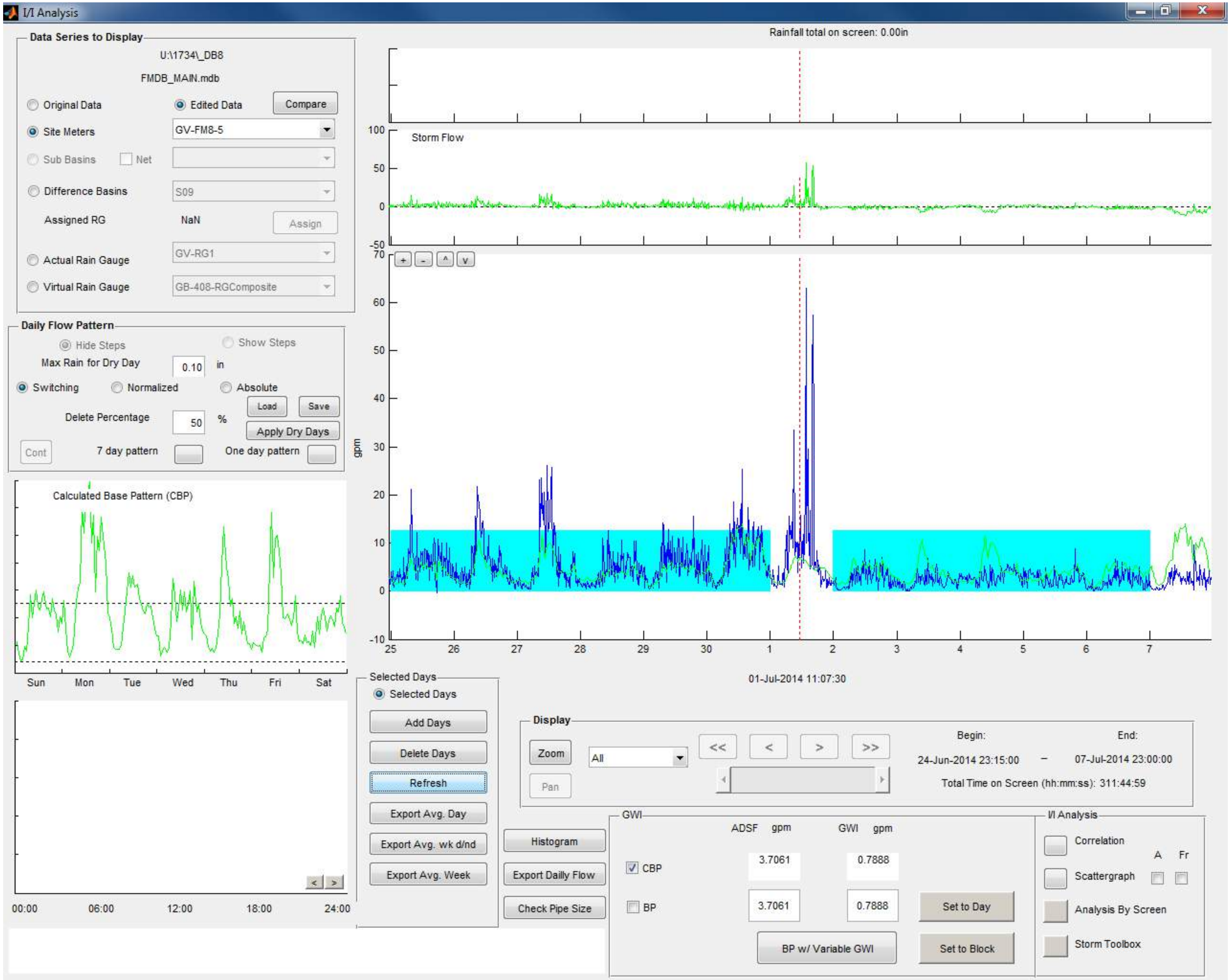
FMDB_MAIN.mdb
Original Data
Edited Data
Compare
Site Meters
GV-FM8-1
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun
Mon
Tue
Wed
Thu
Fri
Sat
00:00
06:00
12:00
18:00
24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
01-Jul-2014 10:15:00
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
Pan
All
Begin:
End:
24-Jun-2014 22:30:00
07-Jul-2014 22:00:00
Total Time on Screen (hh:mm:ss): 311:30:00
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
22.8609
9.8046
BP
22.8609
9.8046
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A
Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-FM8-1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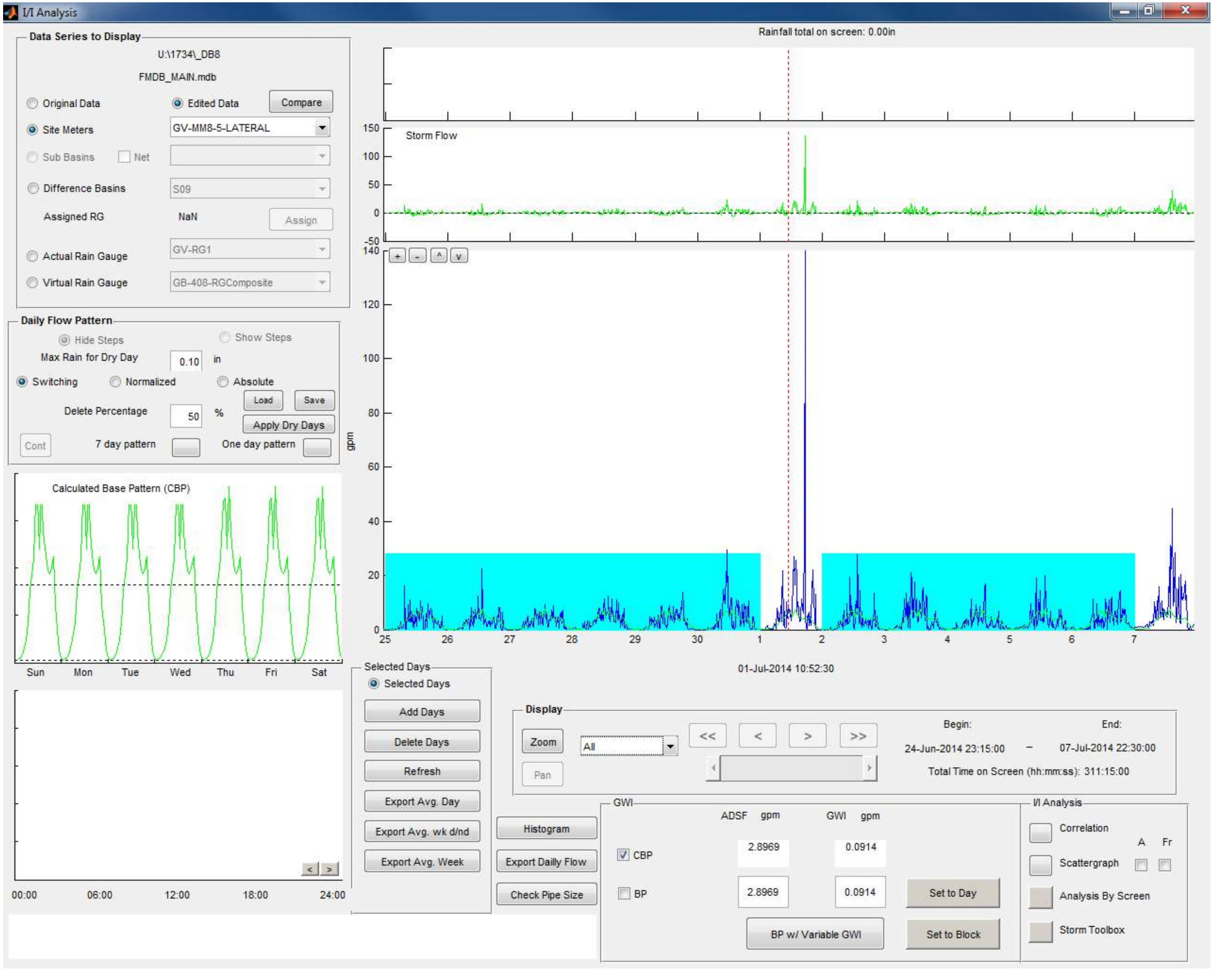
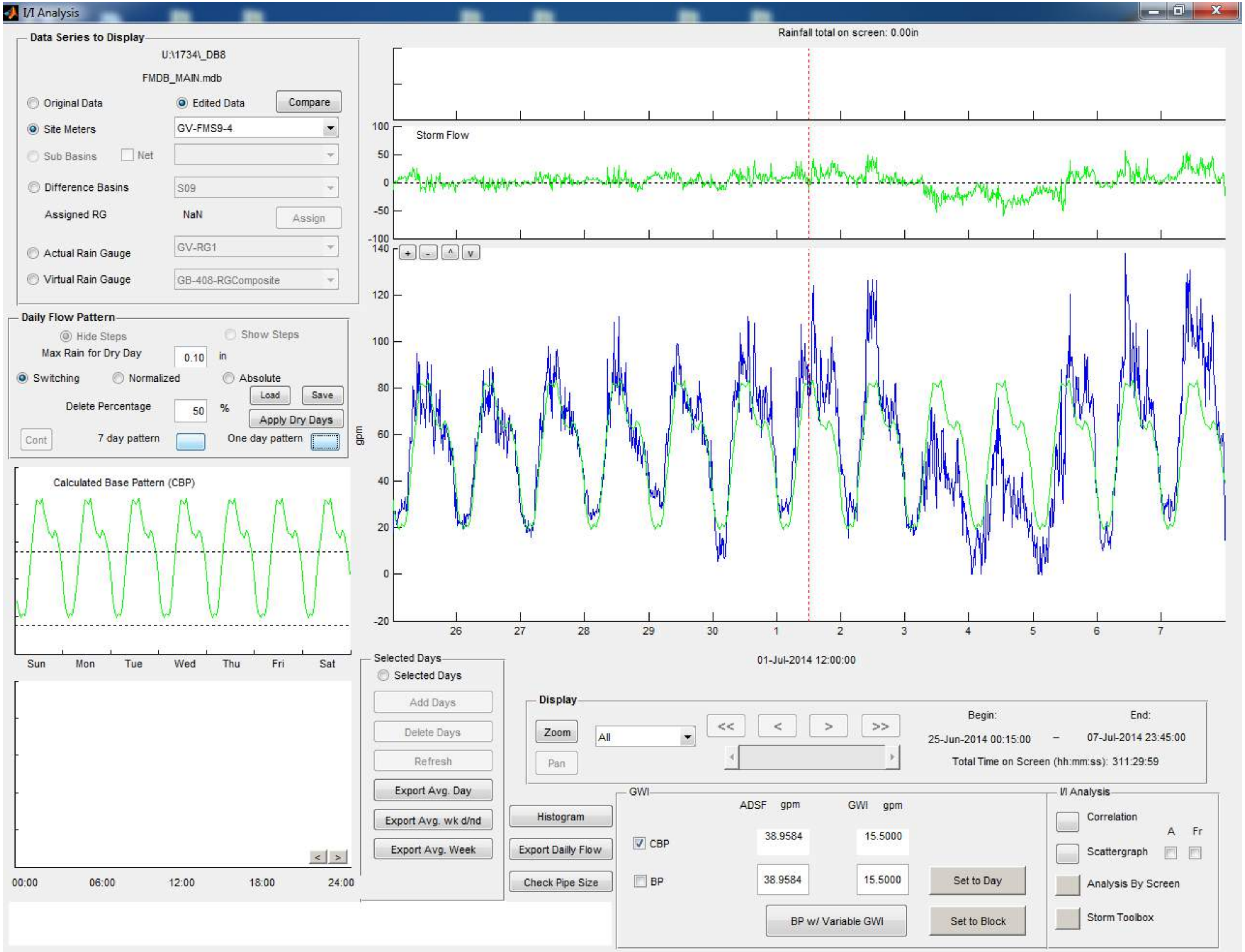
FMDB_MAIN.mdb
Original Data
Edited Data
Compare
Site Meters
GV-FM8-5
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun
Mon
Tue
Wed
Thu
Fri
Sat
00:00
06:00
12:00
18:00
24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
01-Jul-2014 11:07:30
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
All
Pan
Begin:
End:
24-Jun-2014 23:15:00
07-Jul-2014 23:00:00
Total Time on Screen (hh:mm:ss): 311:44:59
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
3.7061
0.7888
BP
3.7061
0.7888
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A
Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-FM8-5



GV-FM8-5-LATERAL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Edited Data
Compare
Site Meters
GV-FMS9-4
Sub Basins
Net
Difference Basins
S09
Assigned RG
NaN
Assign
Actual Rain Gauge
GV-RG1
Virtual Rain Gauge
GB-408-RGComposite
Daily Flow Pattern
Hide Steps
Show Steps
Max Rain for Dry Day
0.10
in
Switching
Normalized
Absolute
Load
Save
Delete Percentage
50
%
Apply Dry Days
Cont
7 day pattern
One day pattern
Calculated Base Pattern (CBP)
Sun Mon Tue Wed Thu Fri Sat
00:00 06:00 12:00 18:00 24:00
Rainfall total on screen: 0.00in
Storm Flow
gpm
01-Jul-2014 12:00:00
Selected Days
Selected Days
Add Days
Delete Days
Refresh
Export Avg. Day
Export Avg. wk d/nd
Export Avg. Week
Display
Zoom
Pan
All
Begin:
25-Jun-2014 00:15:00
End:
07-Jul-2014 23:45:00
Total Time on Screen (hh:mm:ss): 311:29:59
Histogram
Export Daily Flow
Check Pipe Size
GWI
ADSF gpm
GWI gpm
CBP
38.9584
15.5000
BP
38.9584
15.5000
Set to Day
BP w/ Variable GWI
Set to Block
I/I Analysis
Correlation
A Fr
Scattergraph
Analysis By Screen
Storm Toolbox

GV-FM9-4

ATTACHMENT E

Baseflow Calculations

Grass Valley Basin 5 - Phase 2 Sub-Basin	Upstream Meters	Total Upstream Pipe Length (ft)	Sub-Basin Pipe Length (ft)	Sub-basin Average Daily Sewer Flow ADSF (gpd)	Sub- basin Ground Water Infiltration GWI (gpd)	Sub-basin Average Daily Baseflow ADF (gpd)	ADSF (gpd/lf)	GWI (gpd/lf)	ADF (gpd/lf)
S-5C-1		1,339.0	1,339.0	5,083.8	2,454.6	7,538.4	3.8	1.8	5.6
S-5C-2		791.0	791.0	2,805.8	324.7	3,130.6	3.5	0.4	4.0
S-5C-3	5C-1, 5C-2,5C-4	9,795.0	6,458.0	32,069.4	30,589.9	62,659.3	5.0	4.7	9.7
S-5C-4		1,207.0	1,207.0	5,411.4	415.0	5,826.4	4.5	0.3	4.8
S-5D-1	5D-2	6,029.0	408.0	20,175.7	12,004.4	32,180.1	49.5	29.4	78.9
S-5D-2	5D-3,5D-4	5,621.0	2,109.0	333.9	5,468.3	5,802.2	0.2	2.6	2.8
S-5D-3		1,635.0	1,635.0	1,507.7	120.7	1,628.4	0.9	0.1	1.0
S-5D-4		1,877.0	1,877.0	12,716.5	993.9	13,710.4	6.8	0.5	7.3

Grass Valley Basin 6 - Phase 2 Subbasin	Upstream Meters	Total Upstream Pipe Length (ft)	Sub-Basin Pipe Length (ft)	Average Daily Sewer Flow ADSF (gpd)	Ground Water Infiltration GWI (gpd)	Average Daily Baseflow ADF (gpd)	ADSF (gpd/lf)	GWI (gpd/lf)	ADF (gpd/lf)
S-6-1	6C-2,6C-3	13,809.0	8,246.0	1,068.3	0.0	1,068.3	0.1	0.0	0.1
S-6-2		5,005.0	5,005.0						
S-6-3	6C-4	5,563.0	1,875.0	-1,078.4	2,783.1	1,704.7	-0.6	1.5	0.9
S-6-4		3,688.0	3,688.0	23,191.2	10,477.2	33,668.4	6.3	2.8	9.1

Grass Valley Basin 8 - Phase 2 Subbasin	Upstream Meters	Total Upstream Pipe Length (ft)	Sub-Basin Pipe Length (ft)	Average Daily Sewer Flow ADSF (gpd)	Ground Water Infiltration GWI (gpd)	Average Daily Baseflow ADF (gpd)	ADSF (gpd/lf)	GWI (gpd/lf)	ADF (gpd/lf)
S-8-4	9-4	62,467.0	33,914.0	90,167.9	42,825.5	132,993.4	2.7	1.3	3.9
S-8-1		22,107.0	22,107.0	32,919.7	14,118.6	47,038.3	1.5	0.6	2.1
S-8-3		2,525.0	2,525.0	5,336.8	1,135.9	6,472.7	2.1	0.4	2.6
S-8-2 LATERAL		-	-	4,171.5	131.6	4,303.2	-	-	-
S-8-2	8-1,8-5,8-5Lateral	28,553.0	3,921.0	13,672.1	6,933.9	20,606.0	3.5	1.8	5.3

Appendix C FIELD SURVEY SUMMARY

Sewer ID	Sewer Description	From Manhole	To Manhole	GIS Elevations		Survey Elevations		Comments
				From INV	To INV	Survey_Up	Survey_Dn	
23	Id Mary-Whisp P XC	R12-10	R12-9	2517.0	2508.7	-	2509.7	
25	Idaho Maryland Rd	R12-5	R12-9	2509.3	2508.7	-	2508.7	
26	Idaho Maryland Rd	R12-9	R12-4	2508.7	2498.0	2508.6	-	
29	Idaho Maryland Rd	Q12-2	Q12-1	2484.7	2483.7	-	2483.9	
30	Idaho Maryland Rd	Q12-1	P12-3	2483.7	2481.0	2483.7	-	
37	East Main St	O9-3	O10-18	2636.0	2595.0	2635.8	-	
41	Idaho Maryland Rd	O12-3	O12-2	2450.2	2445.6	-	2446.0	Inconsistent Diameters
43	Idaho Maryland Rd	O12-2	O12-1	2445.6	2442.3	2445.9	-	Inconsistent Diameters
48	E Main@Idaho Marylan	N13-10	M13-7	2423.0	2421.5	-	2421.8	Inconsistent Diameters
49	Slate Creek Inlet Pipe	H6-2	9012	2455.0	2450.0	2450.9	-	
50	Cypress Hill Dr	K10-1	K10-4	2664.0	2663.0	2664.3	-	
57	Cypress-Hughes XC	M11-12	M11-1	2606.0	2593.0	-	2592.4	
58	Cypress-Hughes XC	M11-1	M10-6	2593.0	2591.5	2592.4	-	
68	Hughes Rd	N11-1	N11-4	2527.3	2526.0	-	2526.5	Inconsistent Diameters, Missing Sewer
70	East Main St	N11-2	N11-4	2541.0	2526.0	-	2526.4	Inconsistent Diameters, Missing Sewer
71	East Main St	N11-4	N11-5	2526.0	2523.0	2526.4	-	Inconsistent Diameters, Missing Sewer
75	East Main St	M13-3	M13-7	2428.0	2420.3	-	2424.0	Inconsistent Diameters
76	Wolf Creek Intercept	M13-7	M13-15	2420.3	2419.2	2421.4	-	Inconsistent Diameters
83	Wolf Creek Intercept	L14-7	L14-8	2393.1	2391.6	-	2392.3	

Appendix C Field Survey Summary
August 23, 2016

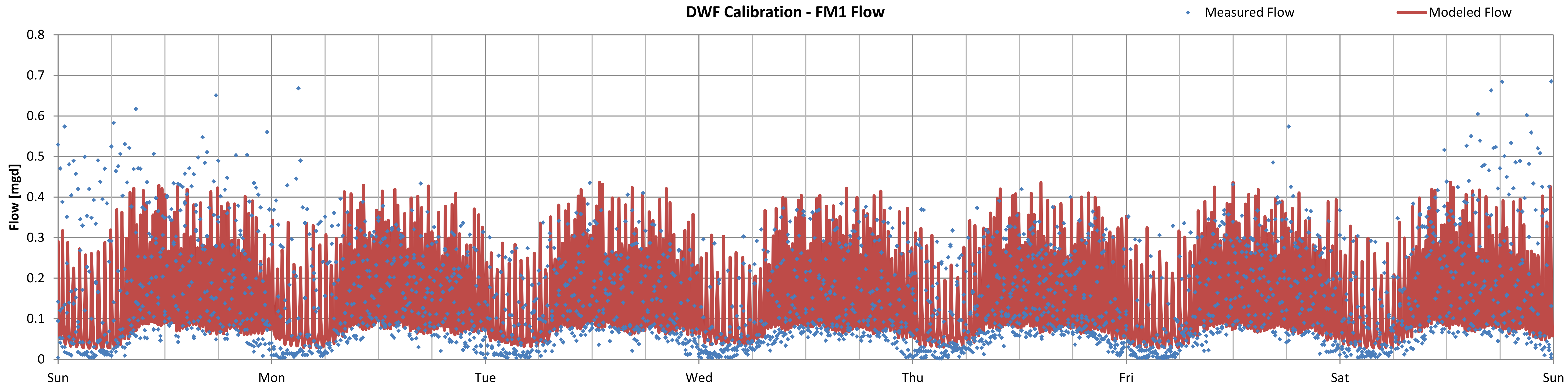
Sewer ID	Sewer Description	From Manhole	To Manhole	GIS Elevations		Survey Elevations		Comments
				From INV	To INV	Survey_ Up	Survey_ Dn	
86	Alta St	I12-2	J13-6	2570.6	2494.3	-	2494.1	
108	Condon Park XC	I14-9	I14-12	2480.0	2465.0	-	2465.1	
109	XC Condon Park	I15-3	H15-2	2465.0	2464.0	2463.3	2461.5	
110	Condon Park XC	H15-2	I15-10	2464.0	2462.7	2461.2	2450.3	
111	Condon-Brighton XC	I15-13	I15-20	2440.3	2430.0	2447.1	2440.3	
112	Condon-Brighton XC	I15-20	I16-7	2430.0	2423.3	2440.2	2424.0	
115	N of Brighton	I16-3	I16-7	2426.5	2423.3	2426.6	2423.5	
118		K19-10	K19-8	2398.1	2379.0	-	2379.3	
121	Taylorville Rd	K20-4	K20-3	2411.8	2408.7	-	2408.8	
122	Taylorville Rd Ext	K20-3	K20-1	2408.7	2397.0	2408.7	2396.4	
123	Freeway Xing	K20-1	K19-15	2397.0	2384.0	2396.3	2385.1	
124		K19-15	K19-13	2384.0	2382.1	2384.8	2382.2	
125		K19-13	K19-8	2382.1	2379.0	2382.3	2379.2	
126		K19-8	K19-7	2379.0	2374.7	2379.2	2374.9	
129	Joyce Dr	K19-3	K19-2	2410.5	2392.0	-	2390.2	
134	Freeman Ln	K20-10	K20-8	2415.0	2407.0	2429.9	2414.5	
135	Freeman Ln	K20-8	K20-9	2407.0	2402.1	2414.4	2407.2	
136	Freeman Ln	K19-17	J19-5	2402.1	2401.2	2402.5	-	
139	Freeman Ln	J19-3	I19-2	2399.2	2388.4	-	2388.0	
140	Freeman Ln	I19-2	J18-7	2388.4	2375.2	2387.9	-	
152	Colfax @ Memorial	M15-27	M15-8	2415.0	2407.0	-	2407.4	
161	Brighton-Mill XC	I16-20	I16-21	2389.7	2375.0	2389.7	2381.3	
162	Brighton-Mill XC	I16-21	I16-27	2375.0	2373.0	2380.9	2375.4	
163	Brighton-Mill XC	I16-27	I16-22	2373.0	2362.2	2375.2	-	
166	Fairgrounds	H17-9	H18-2	2430.0	2426.0	2421.1	-	
169	McCourtney Rd	I17-10	I17-11	2386.0	2366.0	-	2363.9	
170	Mill St	I17-9	I17-8	2364.0	2362.0	2360.8	2360.0	

Sewer ID	Sewer Description	From Manhole	To Manhole	GIS Elevations		Survey Elevations		Comments
				From INV	To INV	Survey_ Up	Survey_ Dn	
171	Mill St under Hwy 20	I17-8	I17-4	2362.0	2355.8	2359.9	-	
174	WCI Frontage Rd	L15-18	K15-11	2382.5	2379.7	-	2380.2	
175	WCI Frontage Rd	K15-11	K15-14	2379.6	2379.3	2380.1	2380.1	
176	WCI Frontage Rd	K15-14	K15-16	2379.1	2378.8	2379.9	2379.5	
179	Wolf Creek Intercept	J16-11	J16-15	2365.6	2362.0	-	2362.3	
785	New Pipe	L11-6	L11-7	2616.0	2615.0	-	2610.4	
787	New Pipe	L11-7	L11-8	2615.0	2611.5	2610.4	-	
809	New Pipe	S12-4	S12-3	2624.8	2595.8	2624.8	-	
811	New Pipe	S13-1	S12-4	2642.5	2624.8	-	2625.0	
813	New Pipe	U13-2	U13-1	2704.3	2702.8	2704.3	-	
825	New Pipe	K15-3	K15-7	2388.4	2384.3	-	2386.4	
827	New Pipe	K15-7	K15-13	2384.3	2382.3	2384.8	-	
841	Under Freeway	K15-16	K15-15	2378.7	2377.8	2379.4	-	
853	New Pipe	L15-20	L15-17	2391.0	2386.0	2392.6	-	
855	New Pipe	L15-7	L15-20	2403.0	2392.3	-	2392.6	
867	New Pipe	M15-8	L15-7	2407.0	2403.0	2406.7	-	
881	New Pipe	M16-6	M16-5	2438.0	2437.0	2430.9	-	
893	New Pipe	J16-15	J16-16	2362.0	2358.0	2362.2	-	
901	New Pipe	L14-8	L14-10	2391.6	2386.8	2392.2	2387.2	
903	New Pipe	L14-10	L14-11	2386.8	2386.1	2387.2	2386.9	
905	New Pipe	L14-11	K14-2-SOUTH	2386.1	2385.3	2386.8	-	
911	New Pipe	K15-6	K15-7	2384.4	2384.3	-	2384.9	
921	New Pipe	I12-1	I12-2	2585.1	2570.6	2585.0	-	Inconsistent Diameters
931	New Pipe	J13-10	K13-24	2437.9	2434.1	2437.6	-	
933	New Pipe	J13-5	J13-10	2443.9	2437.9	-	2437.9	
965	New Pipe	J13-8	J13-3	2456.0	2454.9	2465.3	-	
967	New Pipe	J13-7	J13-8	2475.6	2456.0	2475.3	2465.5	
969	New Pipe	J13-29	J13-7	2487.0	2475.6	2483.8	2475.5	

Sewer ID	Sewer Description	From Manhole	To Manhole	GIS Elevations		Survey Elevations		Comments
				From INV	To INV	Survey_ Up	Survey_ Dn	
977	New Pipe	J13-14	J13-6	2500.7	2494.3	-	2494.1	
991	New Pipe	I16-7	I16-9	2423.3	2389.7	2423.4	2408.2	
993	New Pipe	I16-9	I16-26	2404.0	2400.0	2408.1	2398.2	
995	New Pipe	I16-26	I16-20	2400.0	2389.7	2397.2	2389.8	
1001	New Pipe	I15-10	I15-13	2446.9	2440.3	2450.2	2447.3	
1003	New Pipe	I14-12	I15-3	2465.0	2464.0	2465.1	2463.3	
1005	New Pipe	G18-1	H17-9	2434.0	2430.0	-	2421.4	
1009	New Pipe	I17-11	I17-9	2366.0	2364.0	2363.8	2361.2	
1011	New Pipe	H17-8	H17-15	2404.0	2378.0	2400.7	-	
1013	New Pipe	H17-15	I17-9	2378.0	2361.0	-	2361.8	
1019	New Pipe	H17-5	H17-8	2415.0	2410.0	-	2401.0	
1023	New Pipe	H17-2	H17-6	2430.8	2427.3	-	2427.5	
1025	New Pipe	H17-6	H17-7	2427.3	2420.0	2427.5	2408.7	
1027	New Pipe	H17-7	H17-8	2408.5	2404.0	2408.5	2401.2	
1037	New Pipe	I16-1	I16-3	2441.0	2426.5	-	2426.7	
1045	New Pipe	F15-4	F15-6	2509.0	2508.0	-	2493.7	
1049	New Pipe	F15-6	G15-4	2508.0	2489.0	2493.6	-	
1091	Joyce Drive Inlet	K19-7	9006	2374.7	2374.4	2374.7	-	
1095	New Pipe	K19-2	9006	2390.3	2374.4	2390.1	-	
1123	New Pipe	L21-1	K20-6	2435.7	2415.0	2435.3	-	
1125	New Pipe	L21-2	L21-1	2436.6	2435.7	-	2435.5	
1133	New Pipe	K20-9	K19-17	2407.0	2402.1	2407.2	2402.6	
1135	New Pipe	J23-1	9010	2265.2	2264.8	2264.8	-	
1139	New Pipe	K9-1	K10-1	2665.0	2664.0	-	2664.5	
1153	New Pipe	S10-3	S10-4	2568.4	2564.1	-	2561.9	Inconsistent Diameters
1155	New Pipe	S10-4	S11-3	2564.1	2559.0	2561.8	2561.1	Inconsistent Diameters
1157	New Pipe	S11-3	S11-5	2559.0	2552.1	2560.9	-	Inconsistent Diameters
1159	New Pipe	S11-5	S11-9	2552.1	2544.8	-	2545.1	Inconsistent Diameters

Sewer ID	Sewer Description	From Manhole	To Manhole	GIS Elevations		Survey Elevations		Comments
				From INV	To INV	Survey_ Up	Survey_ Dn	
1161	New Pipe	S11-9	S11-10	2544.8	2537.0	2544.8	-	Inconsistent Diameters
1205	New Pipe	R9-3	R9-6	2600.6	2600.2	-	2599.9	
1207	New Pipe	R9-6	R9-5	2600.2	2598.2	2599.9	2599.8	
1209	New Pipe	R9-5	S9-4	2598.2	2593.6	2599.6	-	
1221	Freeman Lane	K20-11	K20-10	2447.6	2429.8	2447.8	2429.9	
1223	Carriage House FM	CH_CHAMBER	K20-11	2268.0	2447.6	-	2447.9	
1225	New Pipe	J13-11	J13-10	2440.0	2437.9	-	2438.1	
1231	New Pipe	J13-30	J13-29	2492.0	2487.0	2486.8	2483.9	
1239	New Pipe	J13-6	J13-30	2494.3	2492.0	2493.8	2486.9	
1243		K15-19	K15-16	2386.4	2383.3	-	2383.8	
1265	New Pipe	K7-4	9018	2630.0	2613.0	2630.1	-	
1267	Taylorville LS Inlet	L21-10	9008	2373.0	2370.0	2361.1	-	

Appendix D DWF CALIBRATION HYDROGRAPHS AND WWF CALIBRATION HYDROGRAPHS



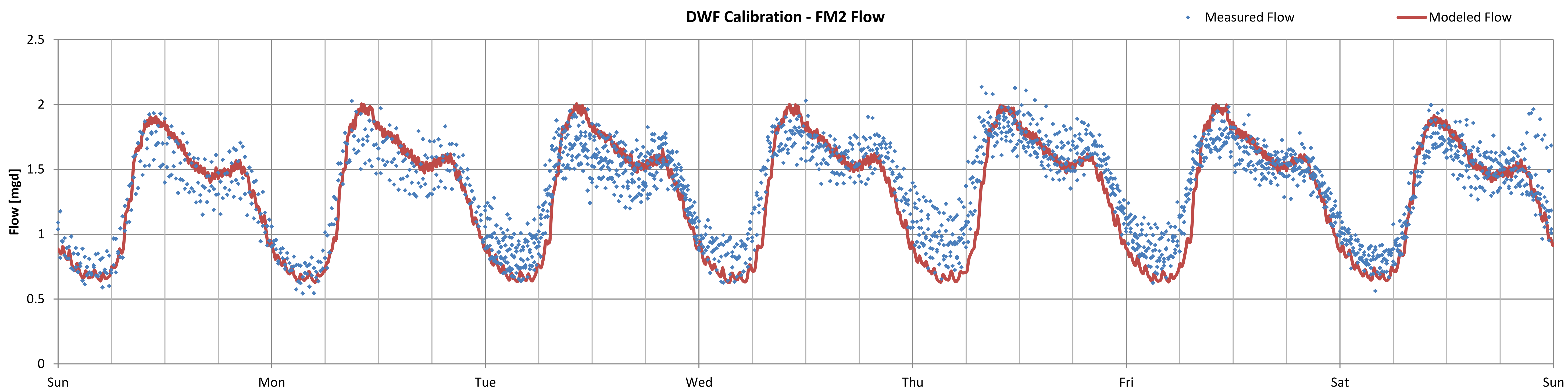


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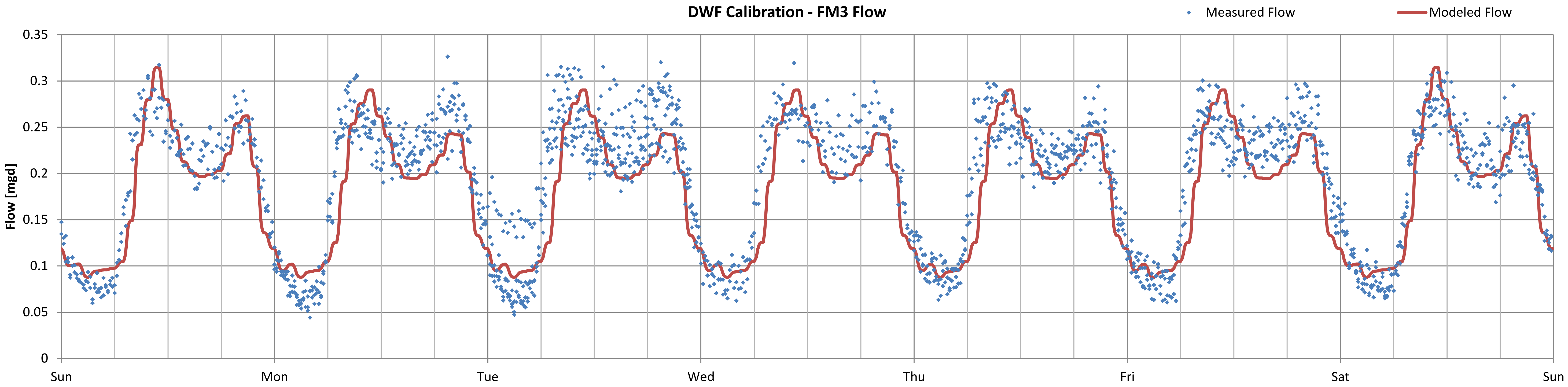
Figure No.
D-1

Title
DWF Calibration Hydrographs
Flow Monitors 1 & 2



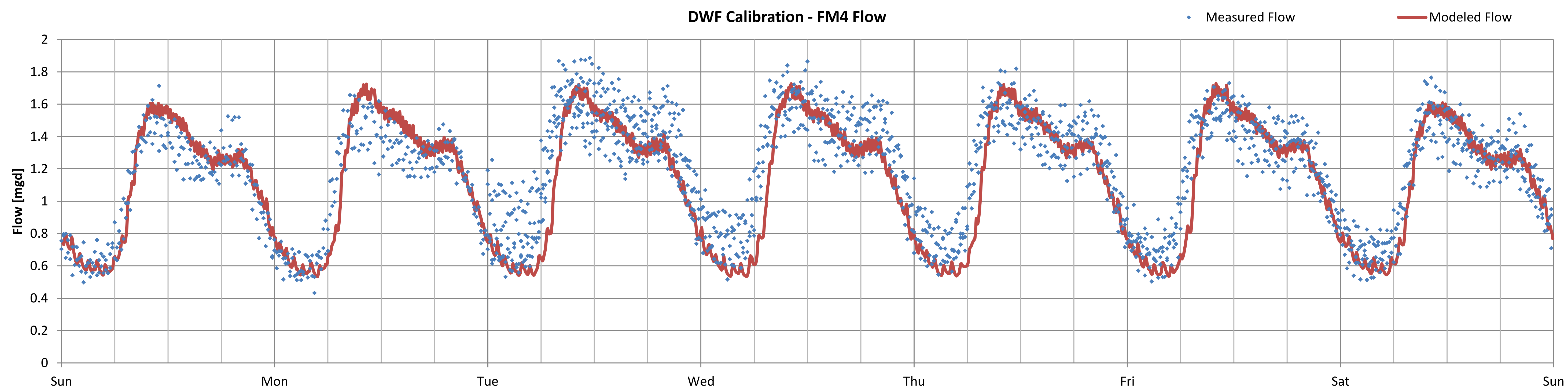


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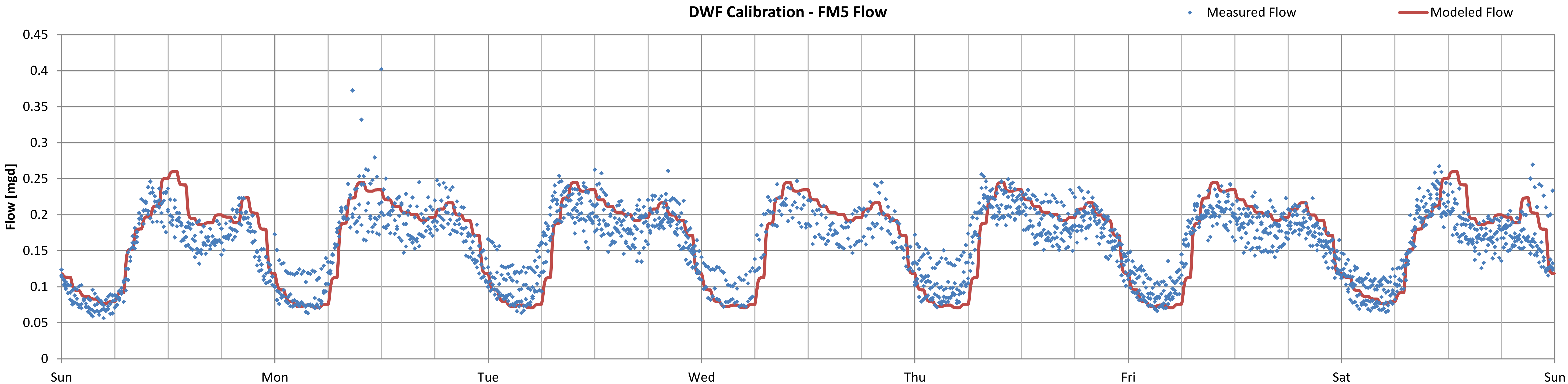
Figure No.
D-2

Title
DWF Calibration Hydrographs
Flow Monitors 3 & 4



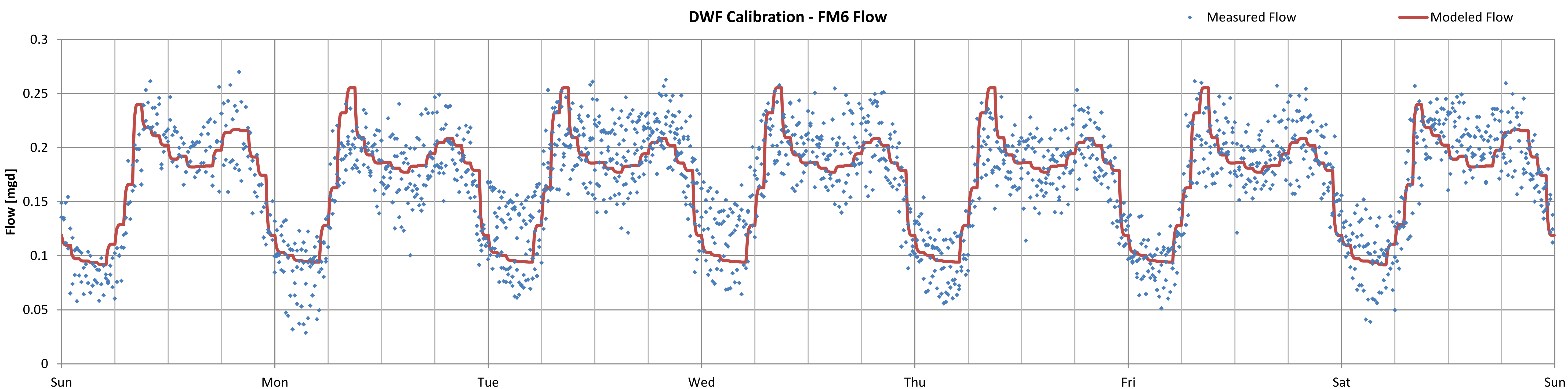


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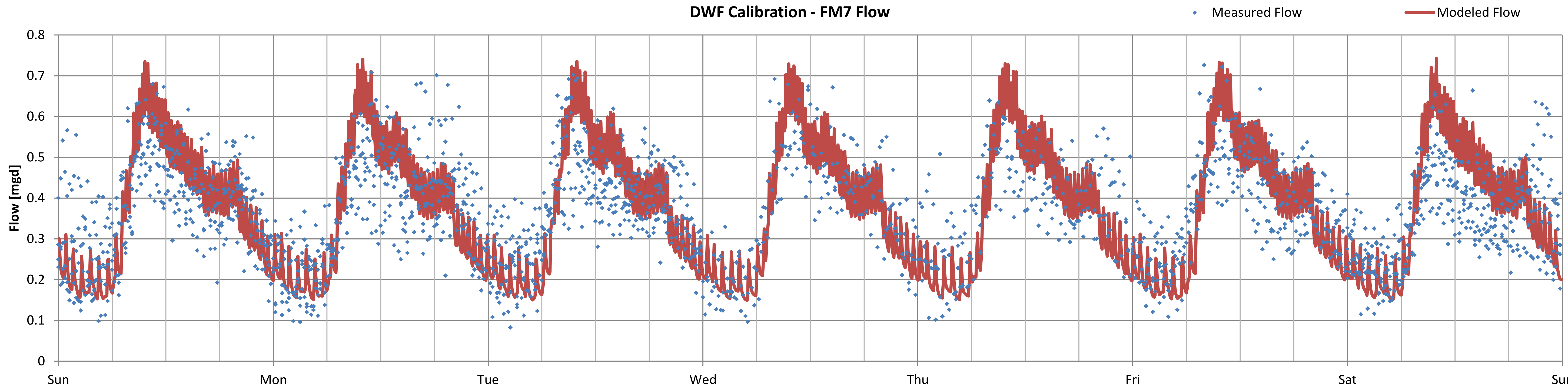
Figure No.
D-3

Title
DWF Calibration Hydrographs
Flow Monitors 5 & 6



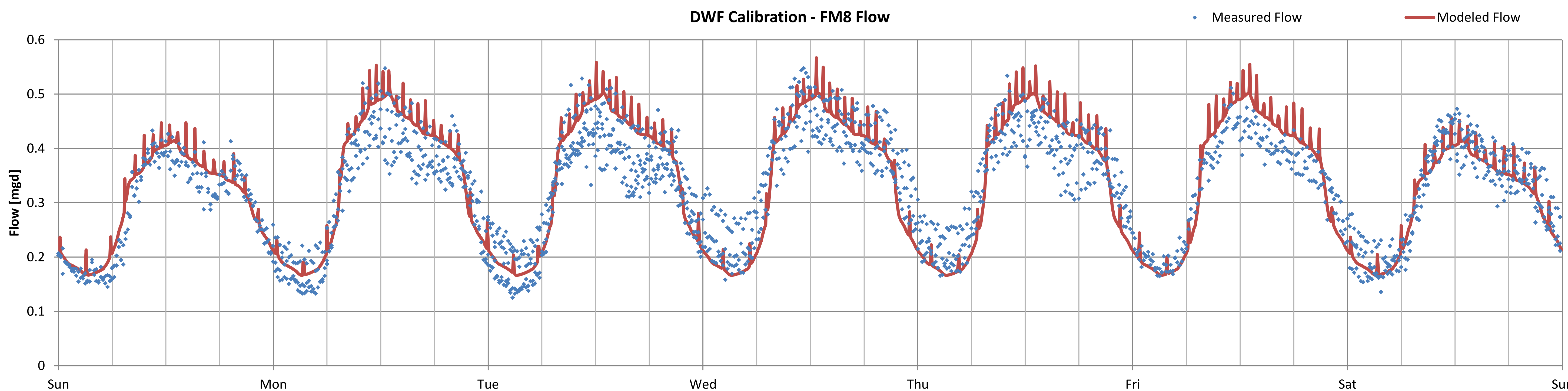


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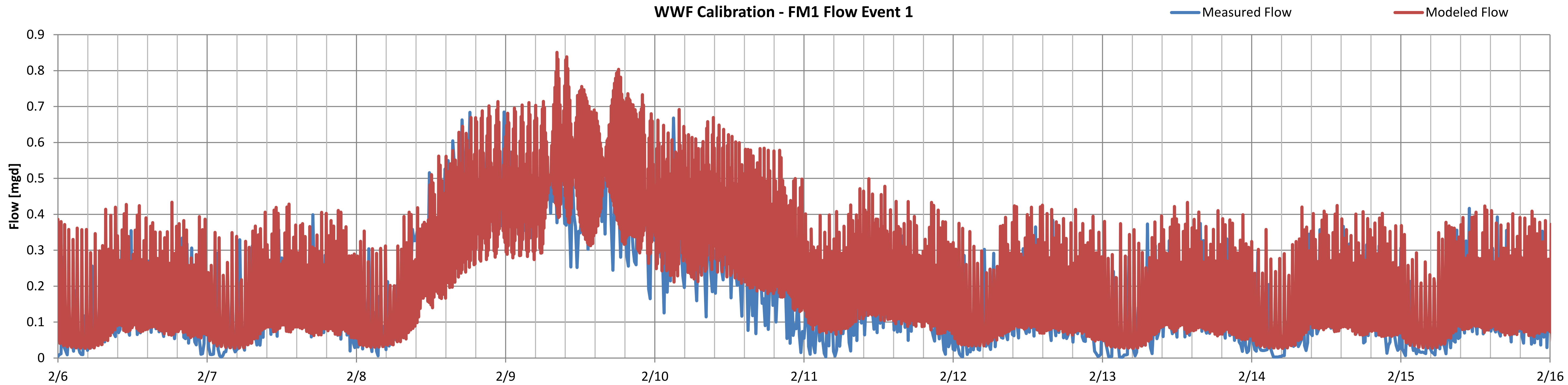
Figure No.
D-4

Title
DWF Calibration Hydrographs
Flow Monitors 7 & 8



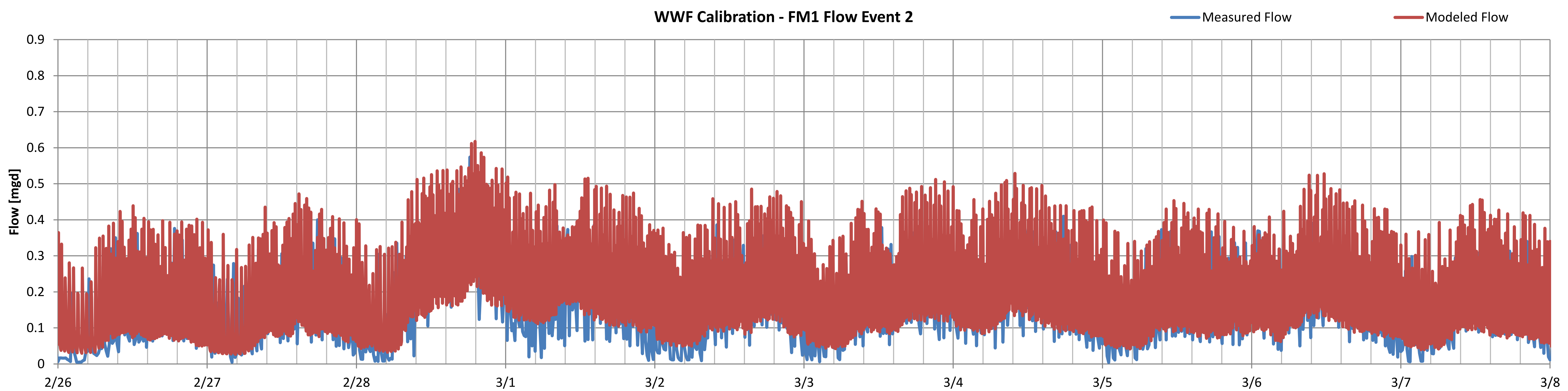


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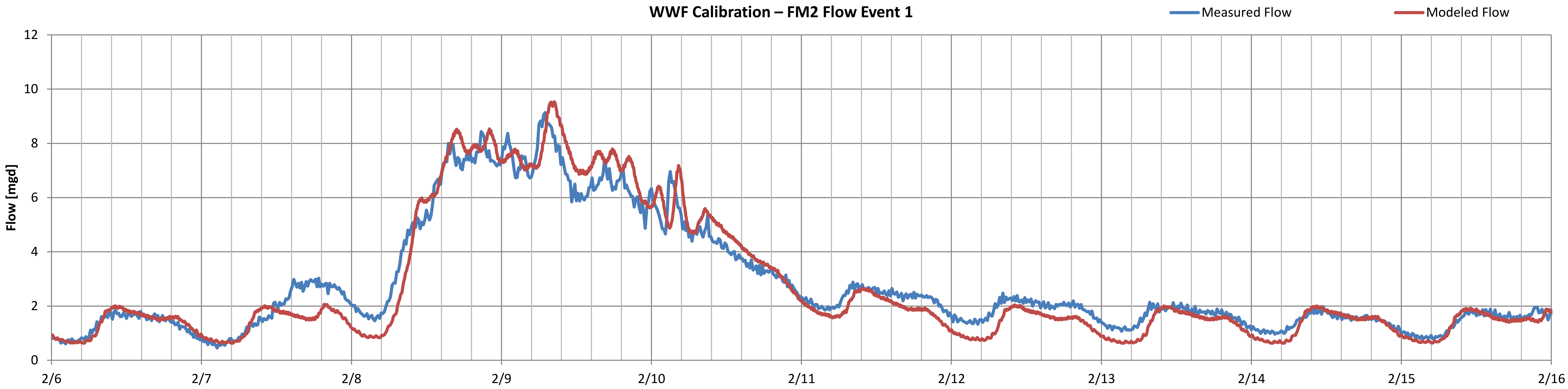
Figure No.
D-5

Title
WWF Calibration Hydrographs
Flow Monitor 1



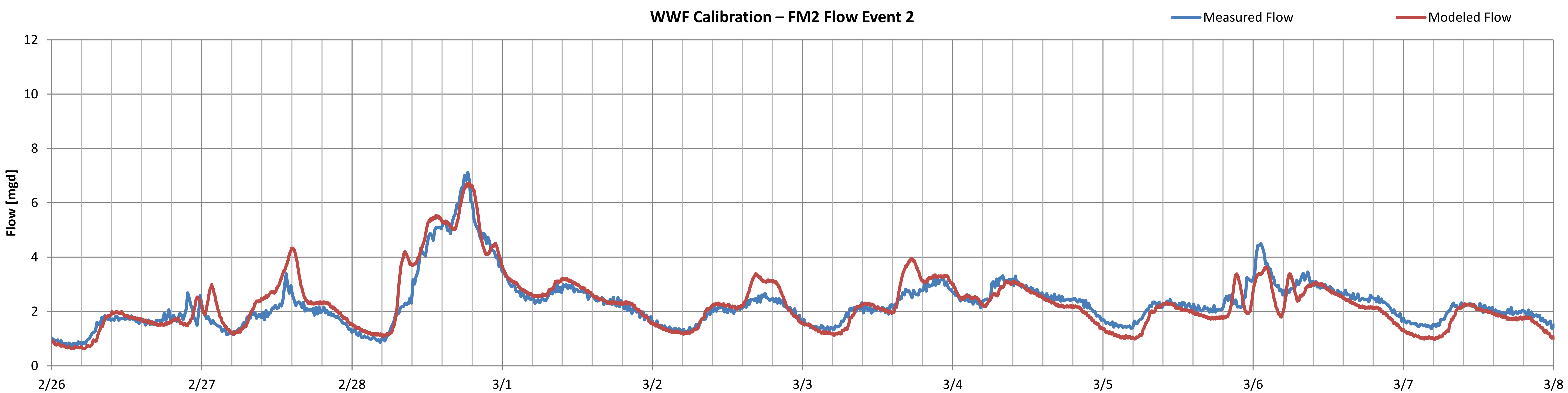


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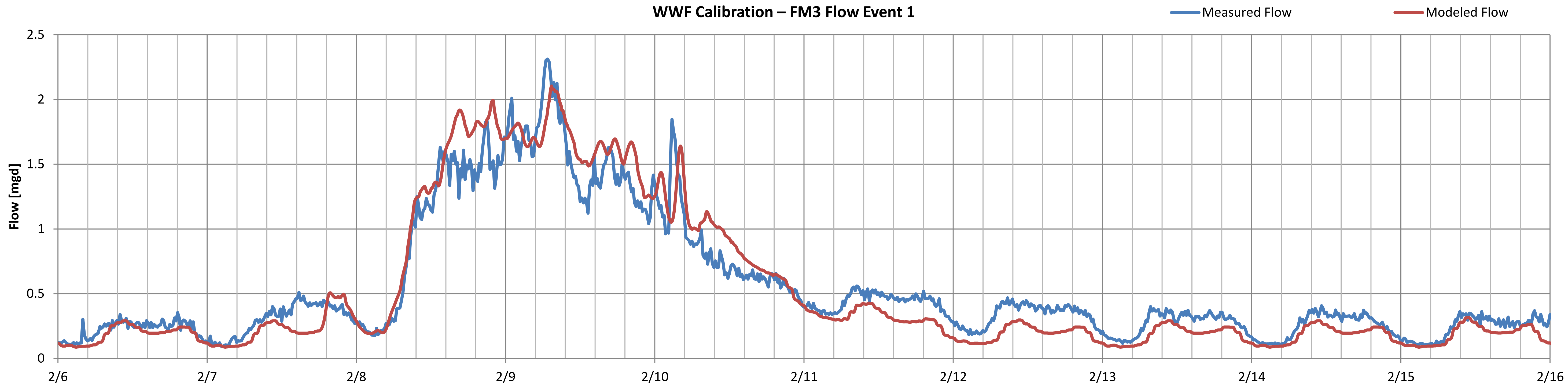
Figure No.
D-6

Title
WWF Calibration Hydrographs
Flow Monitor 2



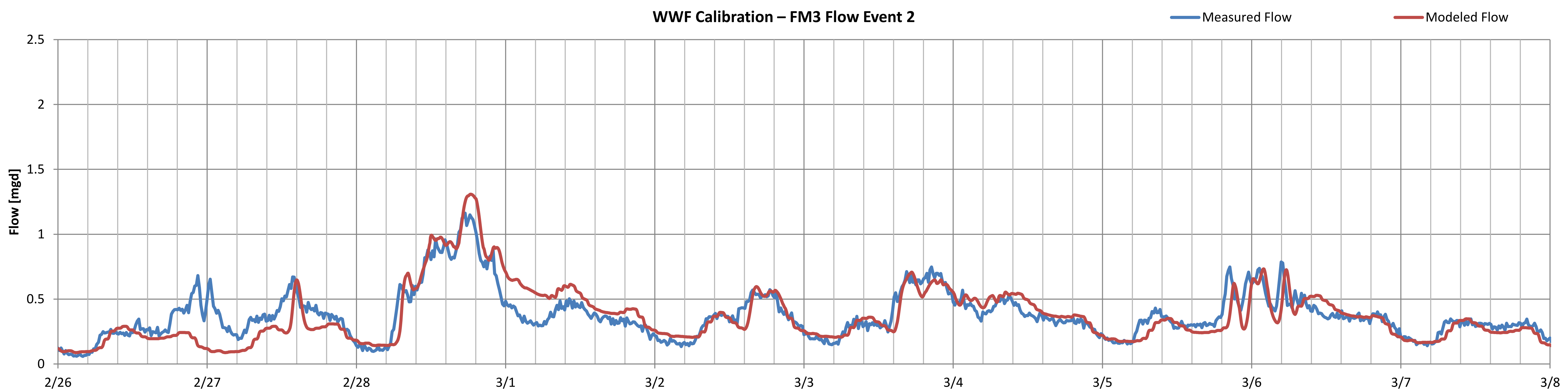


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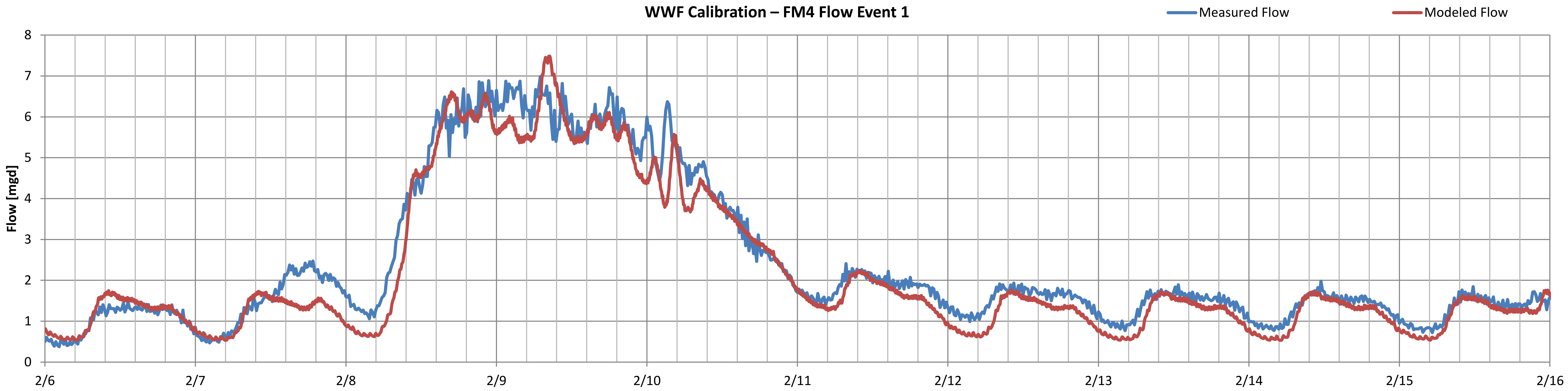
Figure No.
D-7

Title
WWF Calibration Hydrographs
Flow Monitor 3



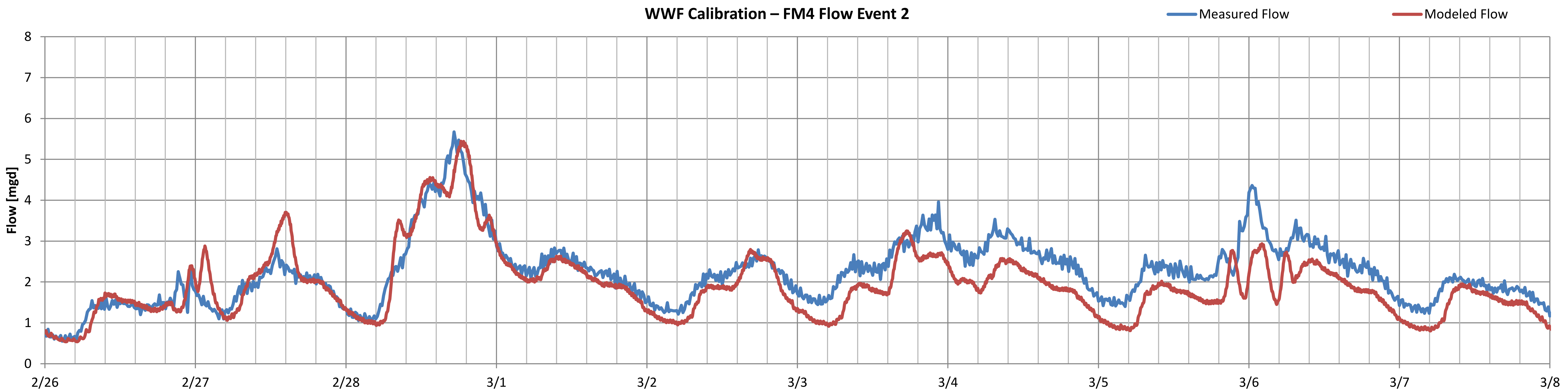


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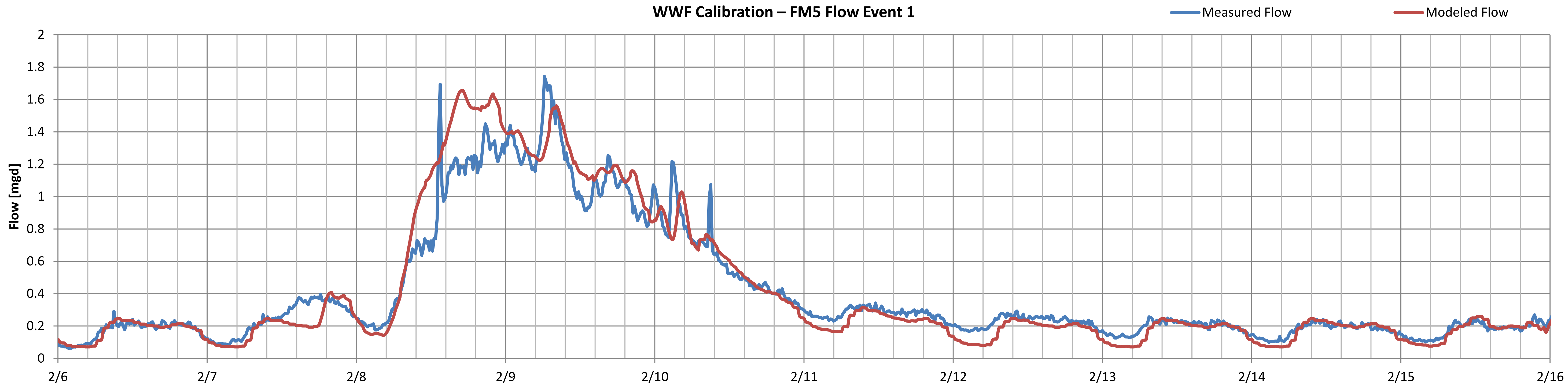
Figure No.
D-8

Title
WWF Calibration Hydrographs
Flow Monitor 4



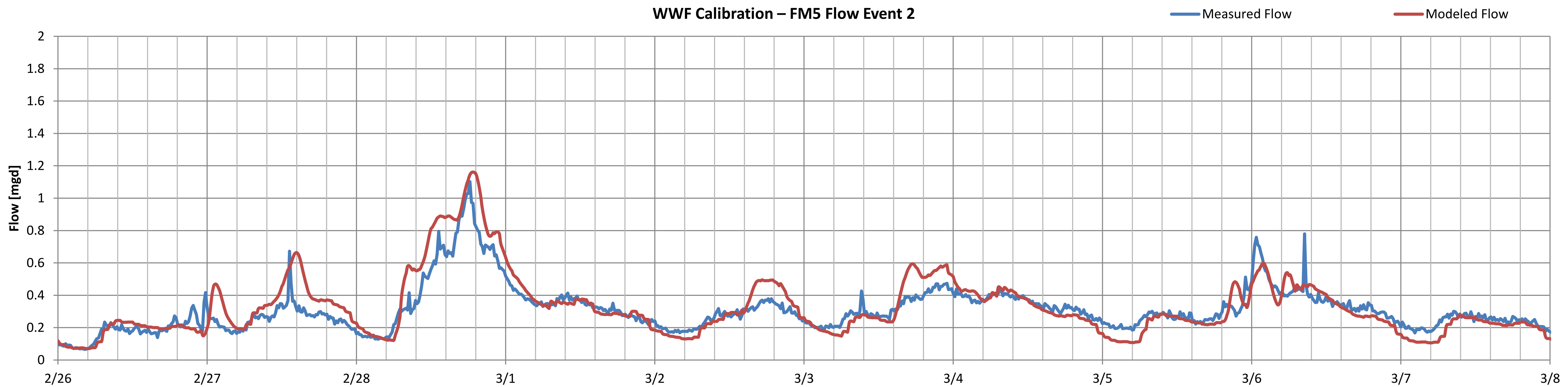


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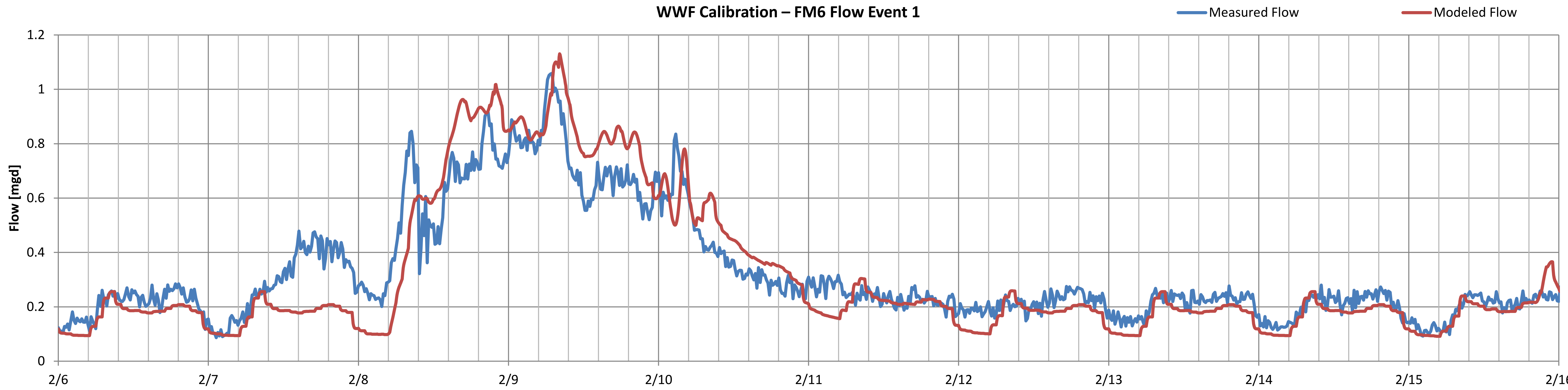
Figure No.
D-9

Title
WWF Calibration Hydrographs
Flow Monitor 5



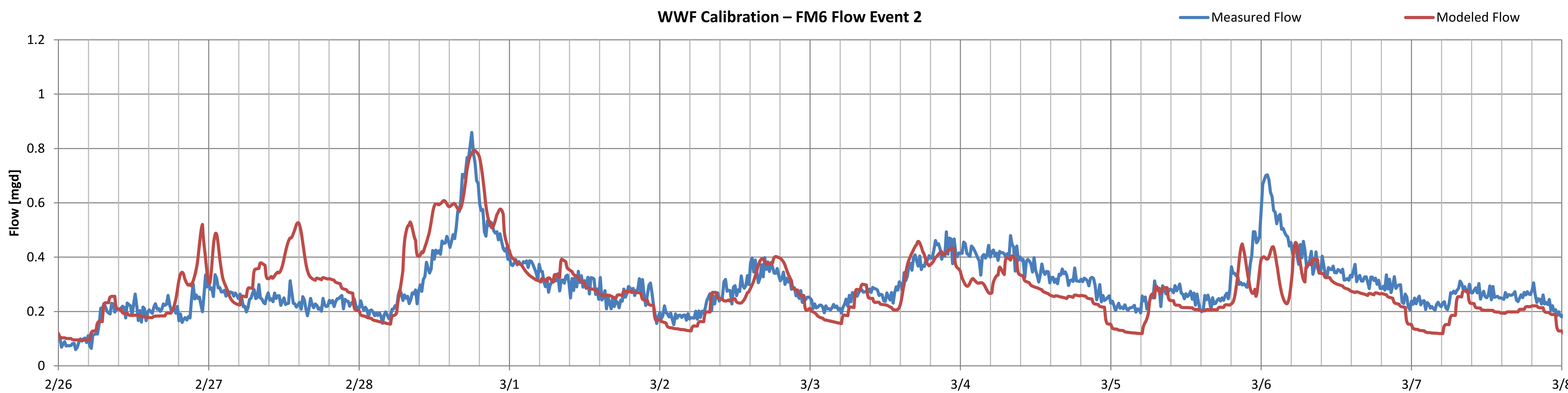


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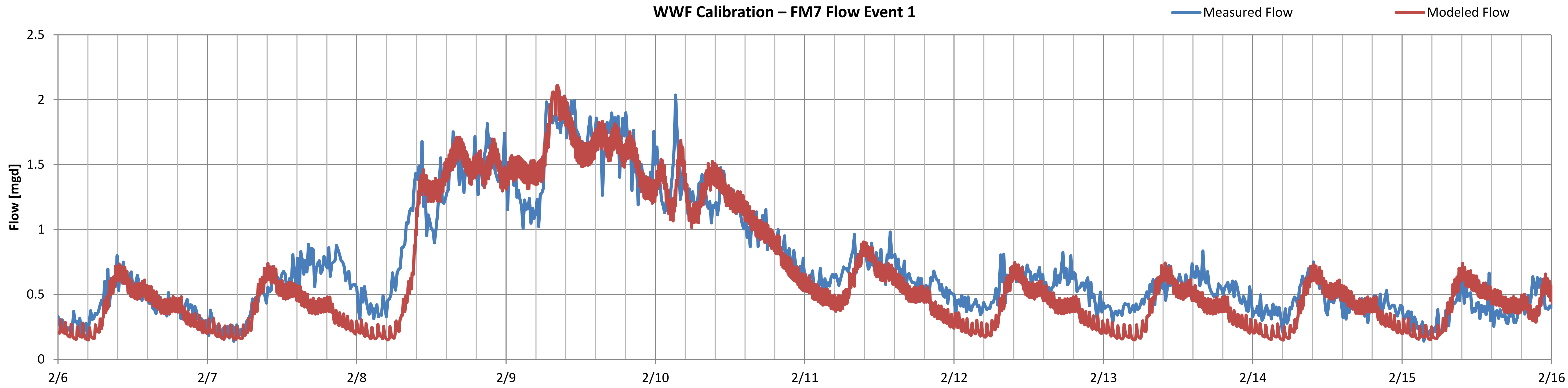
Figure No.
D-10

Title
WWF Calibration Hydrographs
Flow Monitor 6



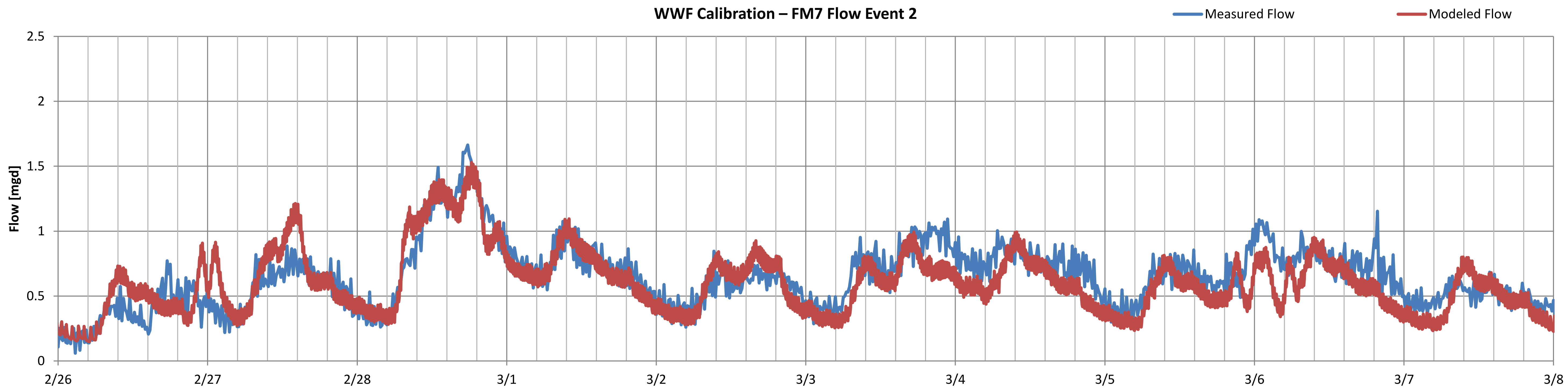


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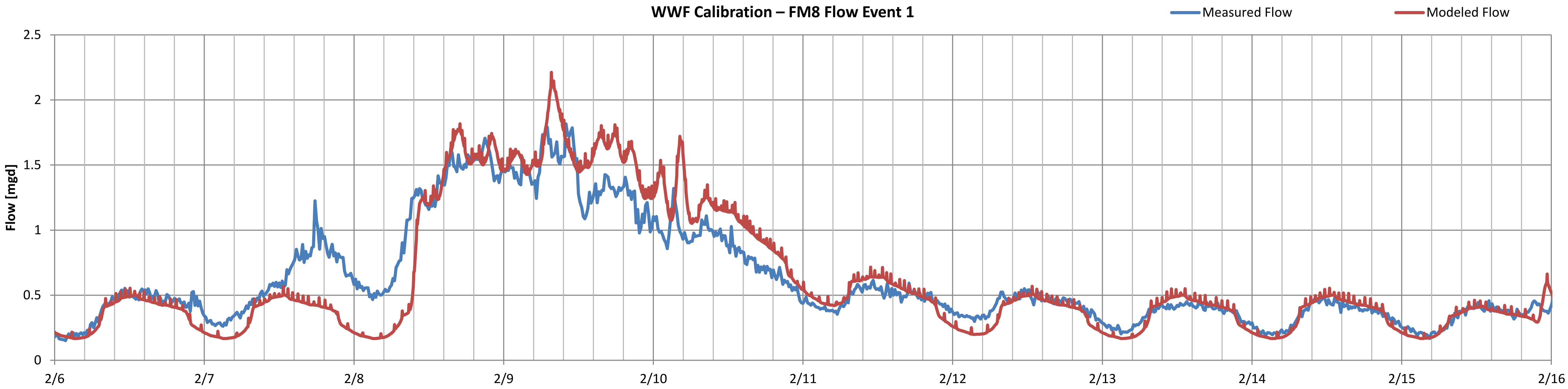
Figure No.
D-11

Title
WWF Calibration Hydrographs
Flow Monitor 7



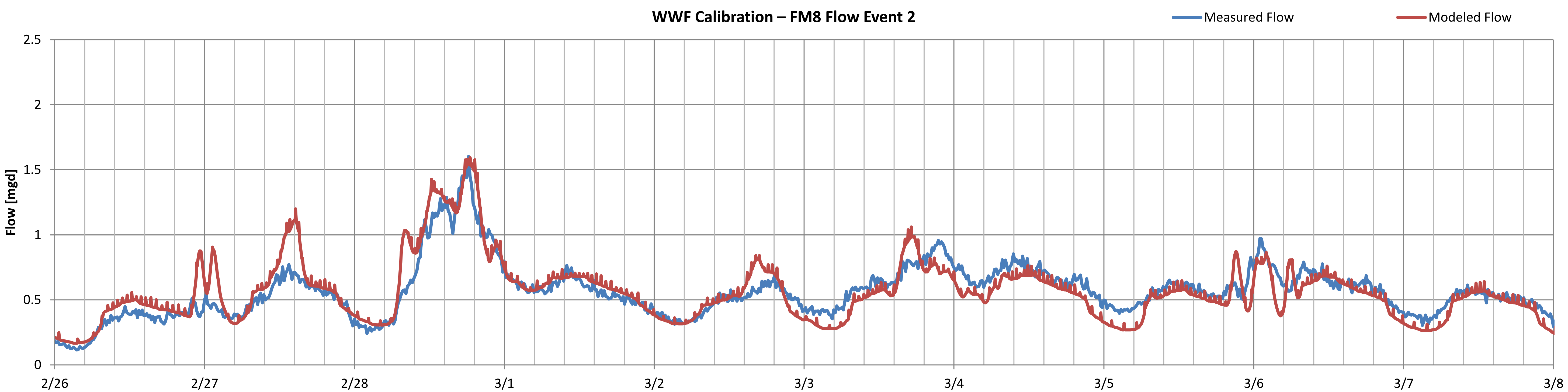


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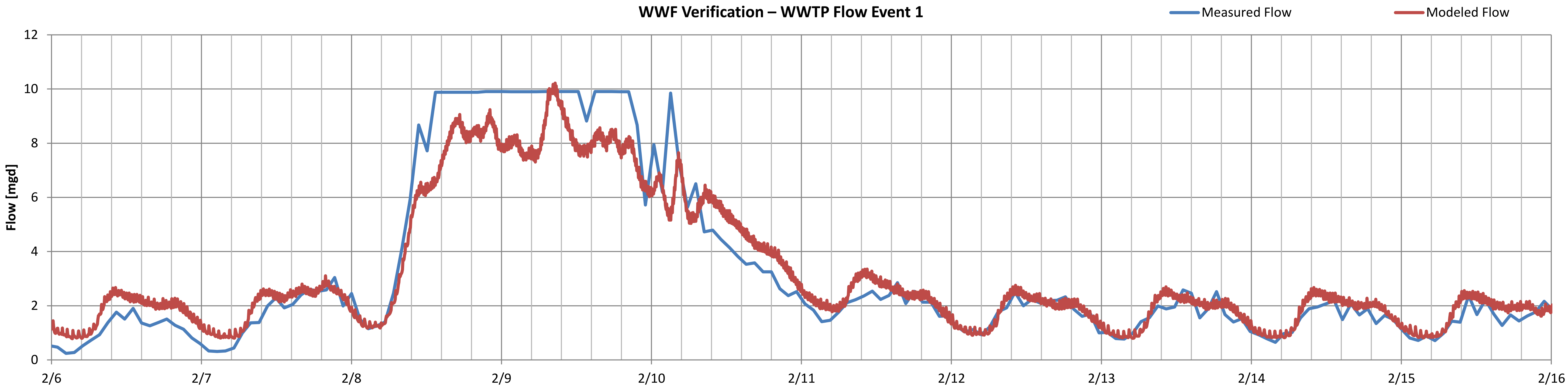
Figure No.
D-12

Title
WWF Calibration Hydrographs
Flow Monitor 8



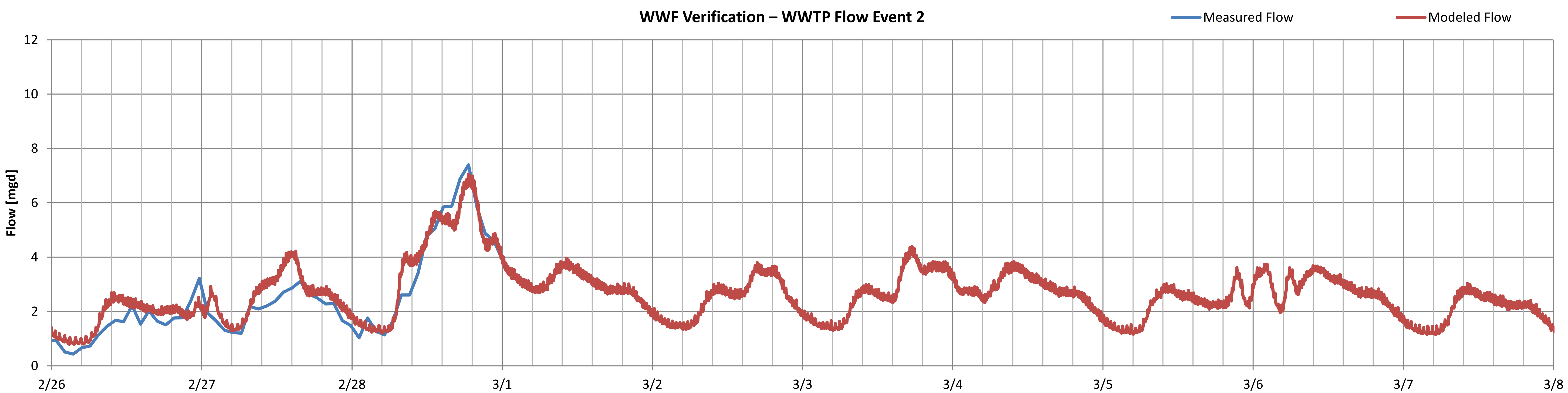


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Figure No.
D-13

Title
WWF Calibration Verification Hydrographs
WWTP Influent Flow Monitor

Appendix E HGL PROFILES - THE PEAK SURCHARGE ELEVATION ALONG EIGHT IDENTIFIED REACHES

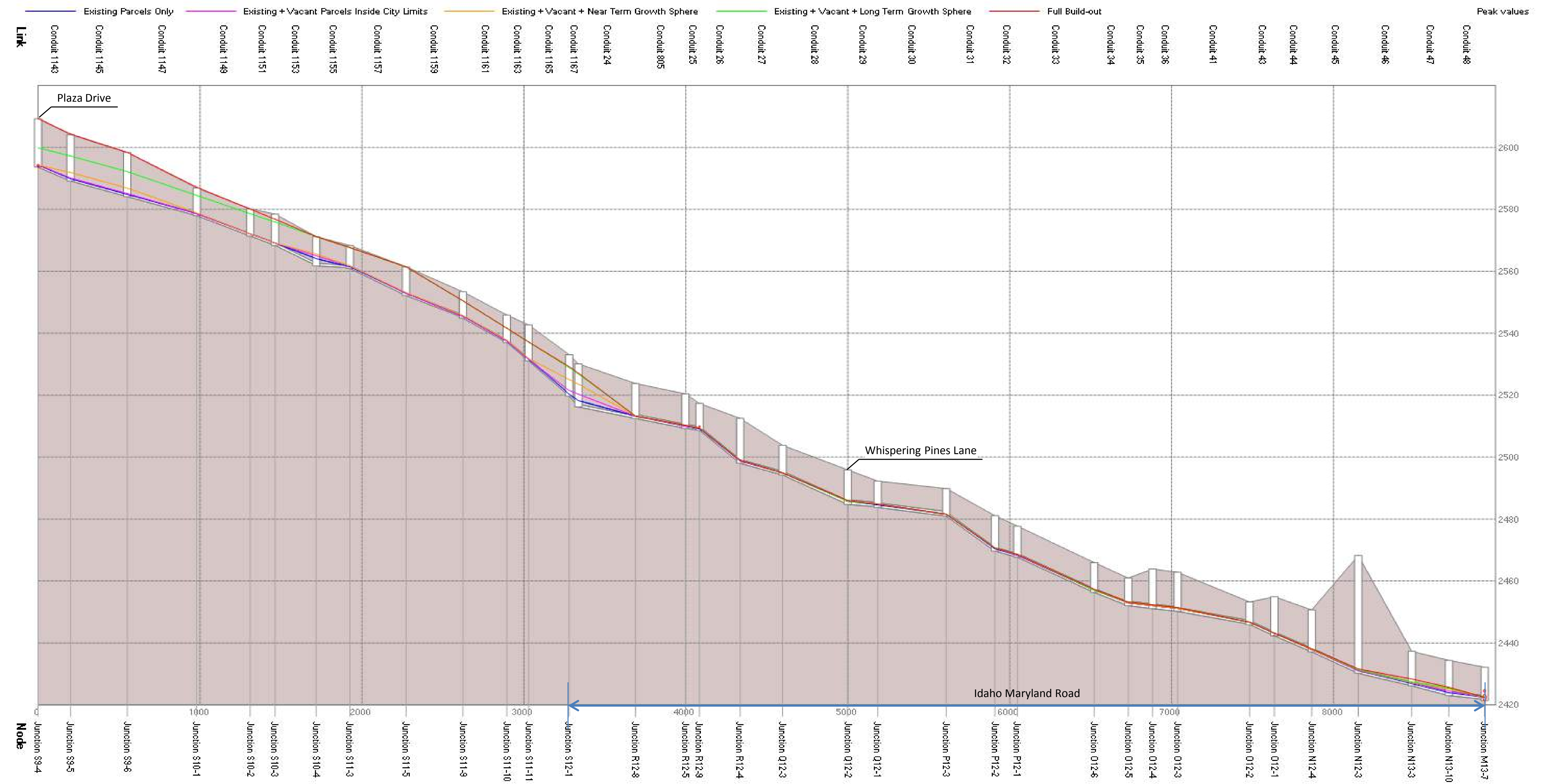

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Figure No.

E-1

Title

Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 1

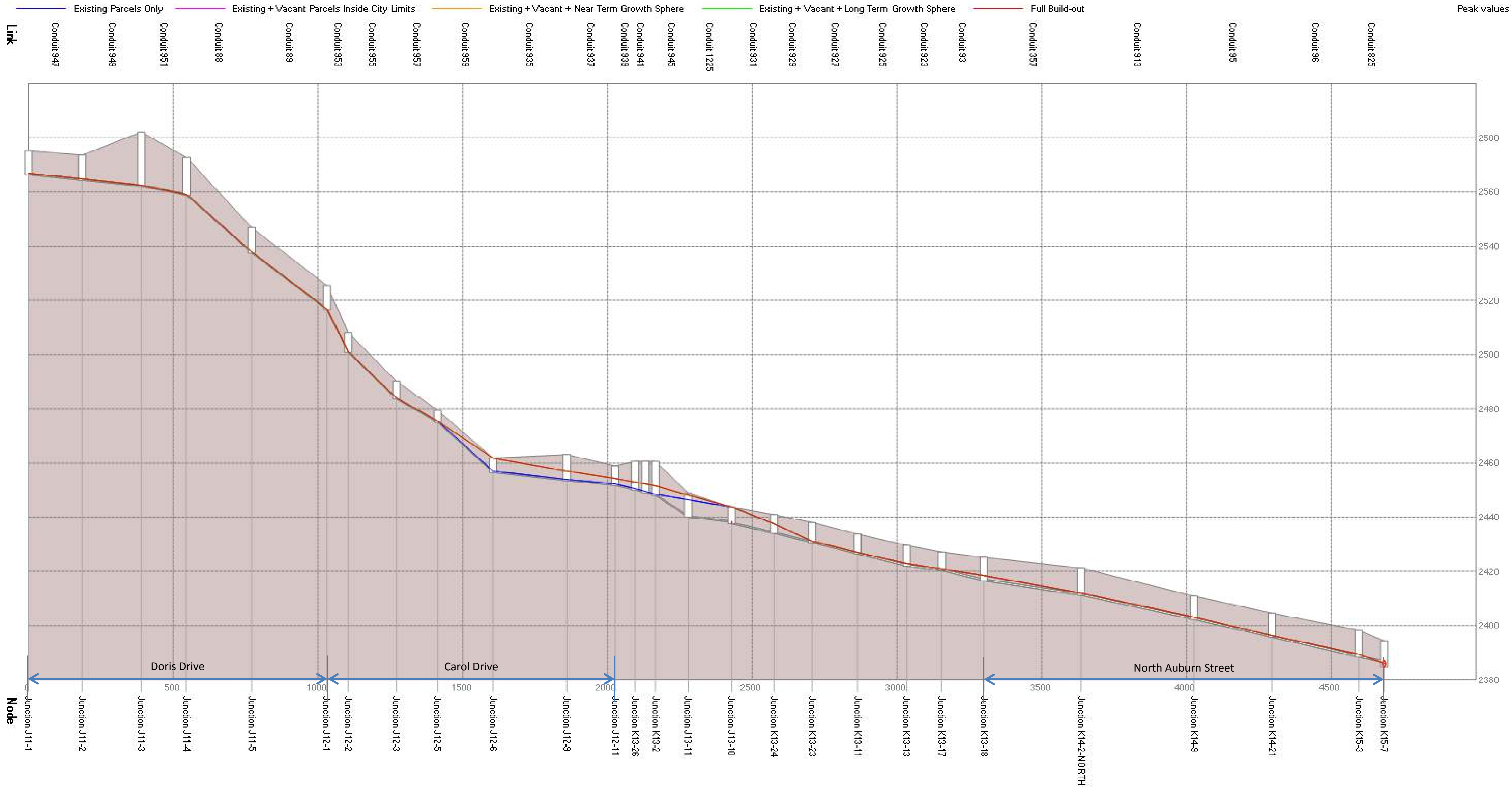

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Figure No.
E-2

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 2

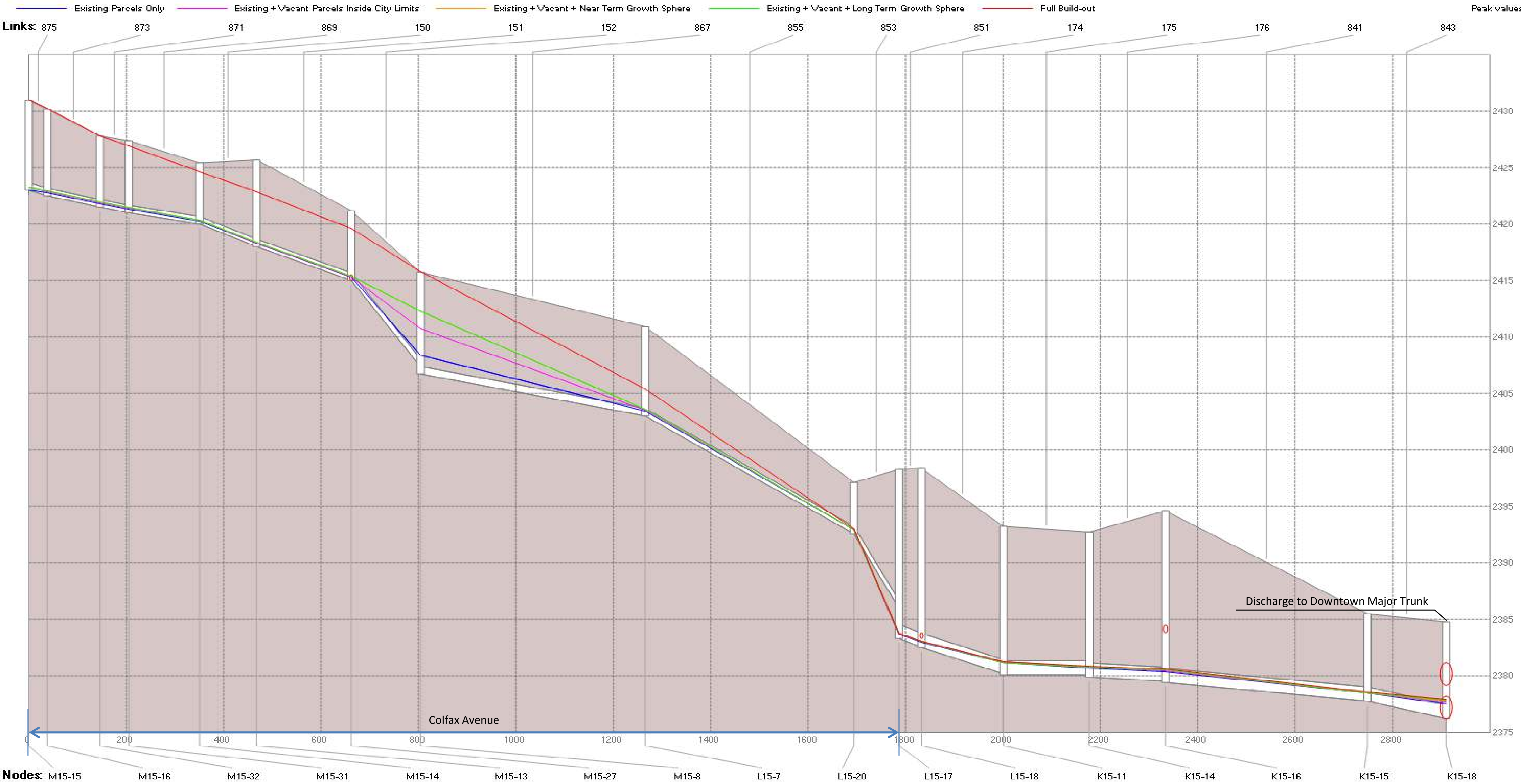

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Figure No.
E-3

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 3

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Figure No.
E-4

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 4

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Figure No.
E-5

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 5

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Figure No.
E-6

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 6

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Figure No.
E-7

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 7

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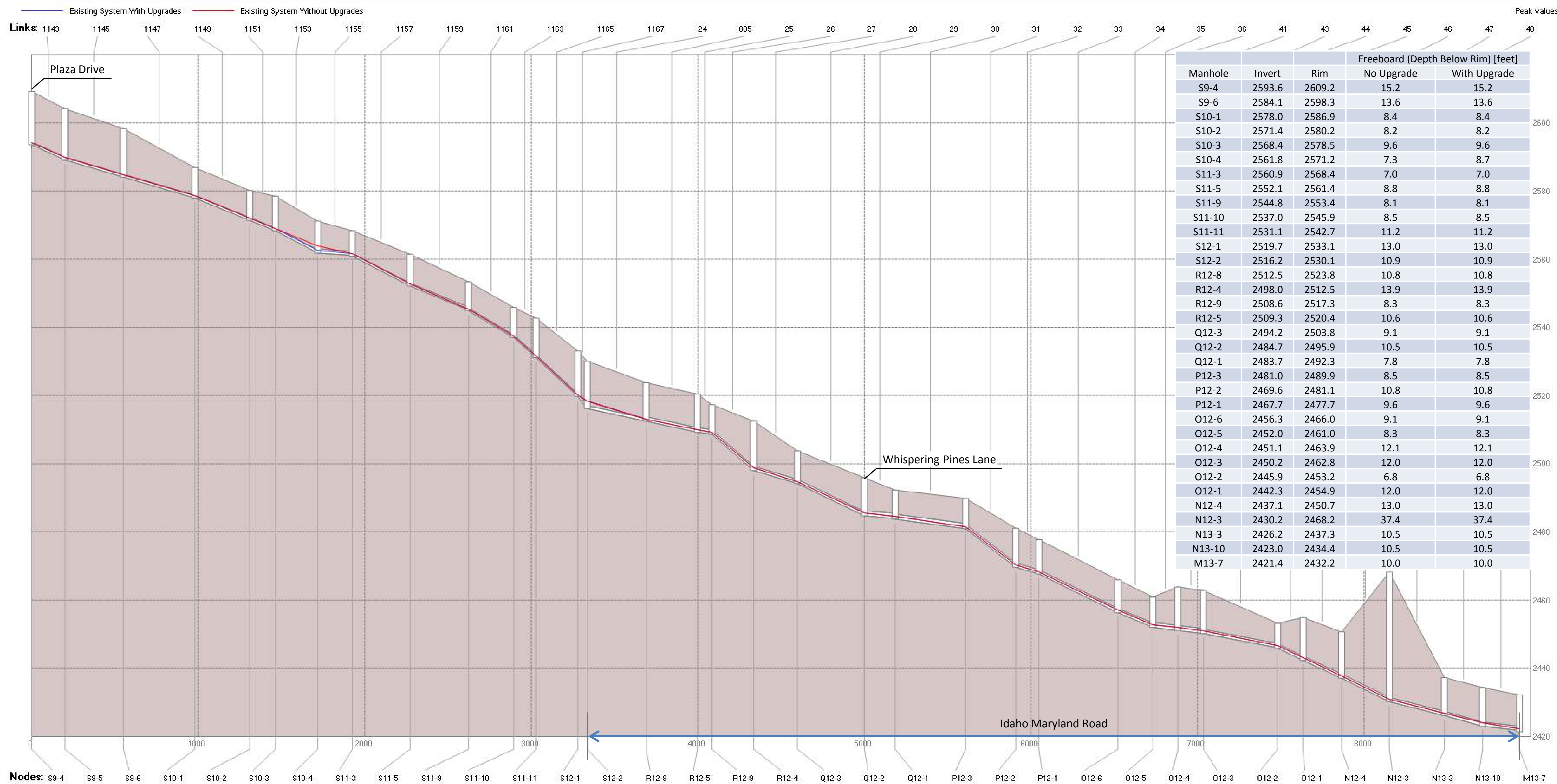
Figure No.
E-8

Title
Existing System Pre-improvement Results – 1:10 Year Design Rainfall HGL Profile 8



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
S9-4	2593.6	2609.2	15.2	15.2
S9-6	2584.1	2598.3	13.6	13.6
S10-1	2578.0	2586.9	8.4	8.4
S10-2	2571.4	2580.2	8.2	8.2
S10-3	2568.4	2578.5	9.6	9.6
S10-4	2561.8	2571.2	7.3	8.7
S11-3	2560.9	2568.4	7.0	7.0
S11-5	2552.1	2561.4	8.8	8.8
S11-9	2544.8	2553.4	8.1	8.1
S11-10	2537.0	2545.9	8.5	8.5
S11-11	2531.1	2542.7	11.2	11.2
S12-1	2519.7	2533.1	13.0	13.0
S12-2	2516.2	2530.1	10.9	10.9
R12-8	2512.5	2523.8	10.8	10.8
R12-4	2498.0	2512.5	13.9	13.9
R12-9	2508.6	2517.3	8.3	8.3
R12-5	2509.3	2520.4	10.6	10.6
Q12-3	2494.2	2503.8	9.1	9.1
Q12-2	2484.7	2495.9	10.5	10.5
Q12-1	2483.7	2492.3	7.8	7.8
P12-3	2481.0	2489.9	8.5	8.5
P12-2	2469.6	2481.1	10.8	10.8
P12-1	2467.7	2477.7	9.6	9.6
O12-6	2456.3	2466.0	9.1	9.1
O12-5	2452.0	2461.0	8.3	8.3
O12-4	2451.1	2463.9	12.1	12.1
O12-3	2450.2	2462.8	12.0	12.0
O12-2	2445.9	2453.2	6.8	6.8
O12-1	2442.3	2454.9	12.0	12.0
N12-4	2437.1	2450.7	13.0	13.0
N12-3	2430.2	2468.2	37.4	37.4
N13-3	2426.2	2437.3	10.5	10.5
N13-10	2423.0	2434.4	10.5	10.5
M13-7	2421.4	2432.2	10.0	10.0

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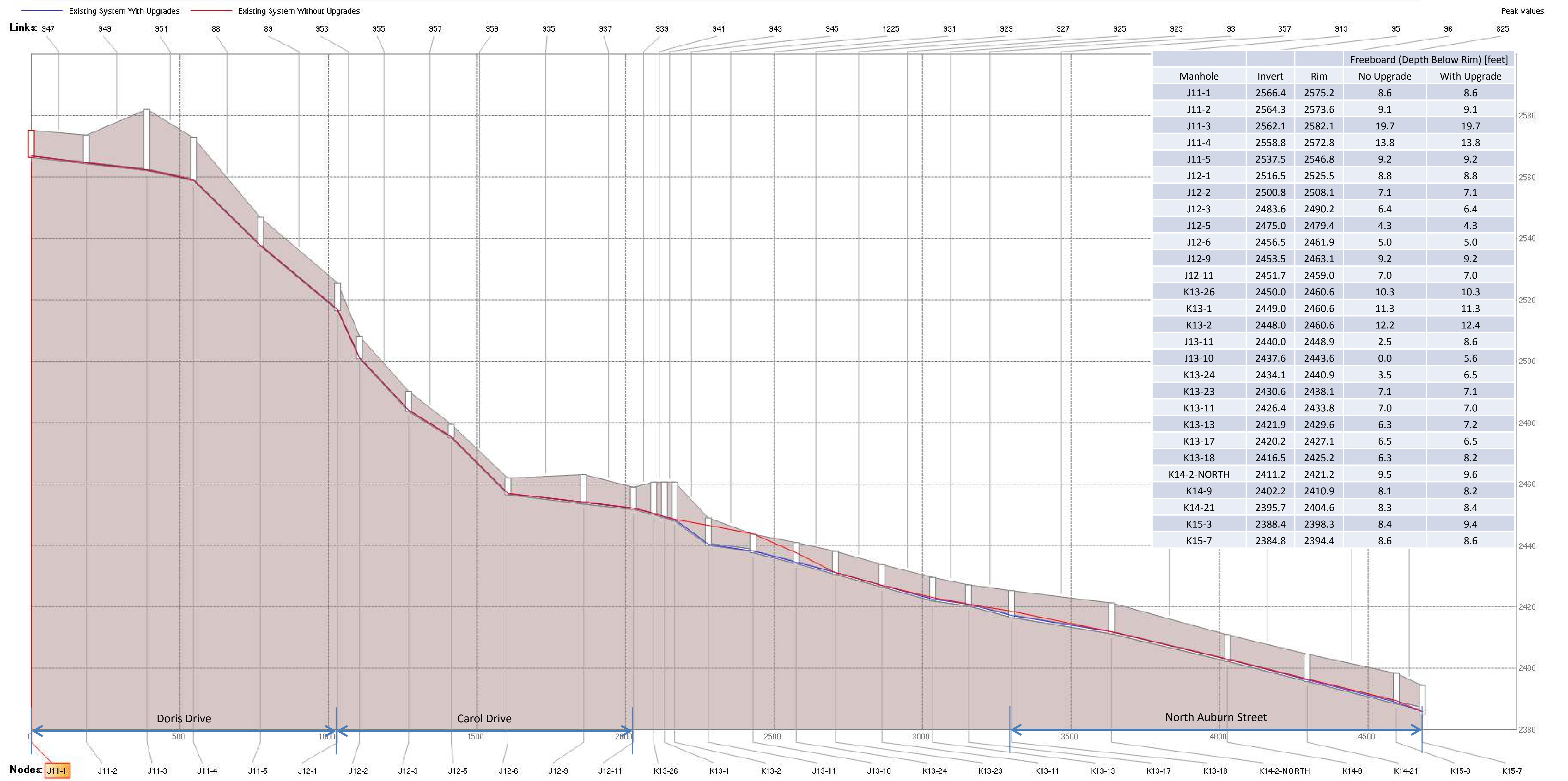
Figure No.
E-9

Title
Existing System Pre- and Post –improvement Results
HGL Profile 1



Manhole	Invert	Rim	Freeboard (Depth Below Rim) [feet] No Upgrade	With Upgrade
J11-1	2566.4	2575.2	8.6	8.6
J11-2	2564.3	2573.6	9.1	9.1
J11-3	2562.1	2582.1	19.7	19.7
J11-4	2558.8	2572.8	13.8	13.8
J11-5	2537.5	2546.8	9.2	9.2
J12-1	2516.5	2525.5	8.8	8.8
J12-2	2500.8	2508.1	7.1	7.1
J12-3	2483.6	2490.2	6.4	6.4
J12-5	2475.0	2479.4	4.3	4.3
J12-6	2456.5	2461.9	5.0	5.0
J12-9	2453.5	2463.1	9.2	9.2
J12-11	2451.7	2459.0	7.0	7.0
K13-26	2450.0	2460.6	10.3	10.3
K13-1	2449.0	2460.6	11.3	11.3
K13-2	2448.0	2460.6	12.2	12.4
J13-11	2440.0	2448.9	2.5	8.6
J13-10	2437.6	2443.6	0.0	5.6
K13-24	2434.1	2440.9	3.5	6.5
K13-23	2430.6	2438.1	7.1	7.1
K13-11	2426.4	2433.8	7.0	7.0
K13-13	2421.9	2429.6	6.3	7.2
K13-17	2420.2	2427.1	6.5	6.5
K13-18	2416.5	2425.2	6.3	8.2
K14-2-NORTH	2411.2	2421.2	9.5	9.6
K14-9	2402.2	2410.9	8.1	8.2
K14-21	2395.7	2404.6	8.3	8.4
K15-3	2388.4	2398.3	8.4	9.4
K15-7	2384.8	2394.4	8.6	8.6

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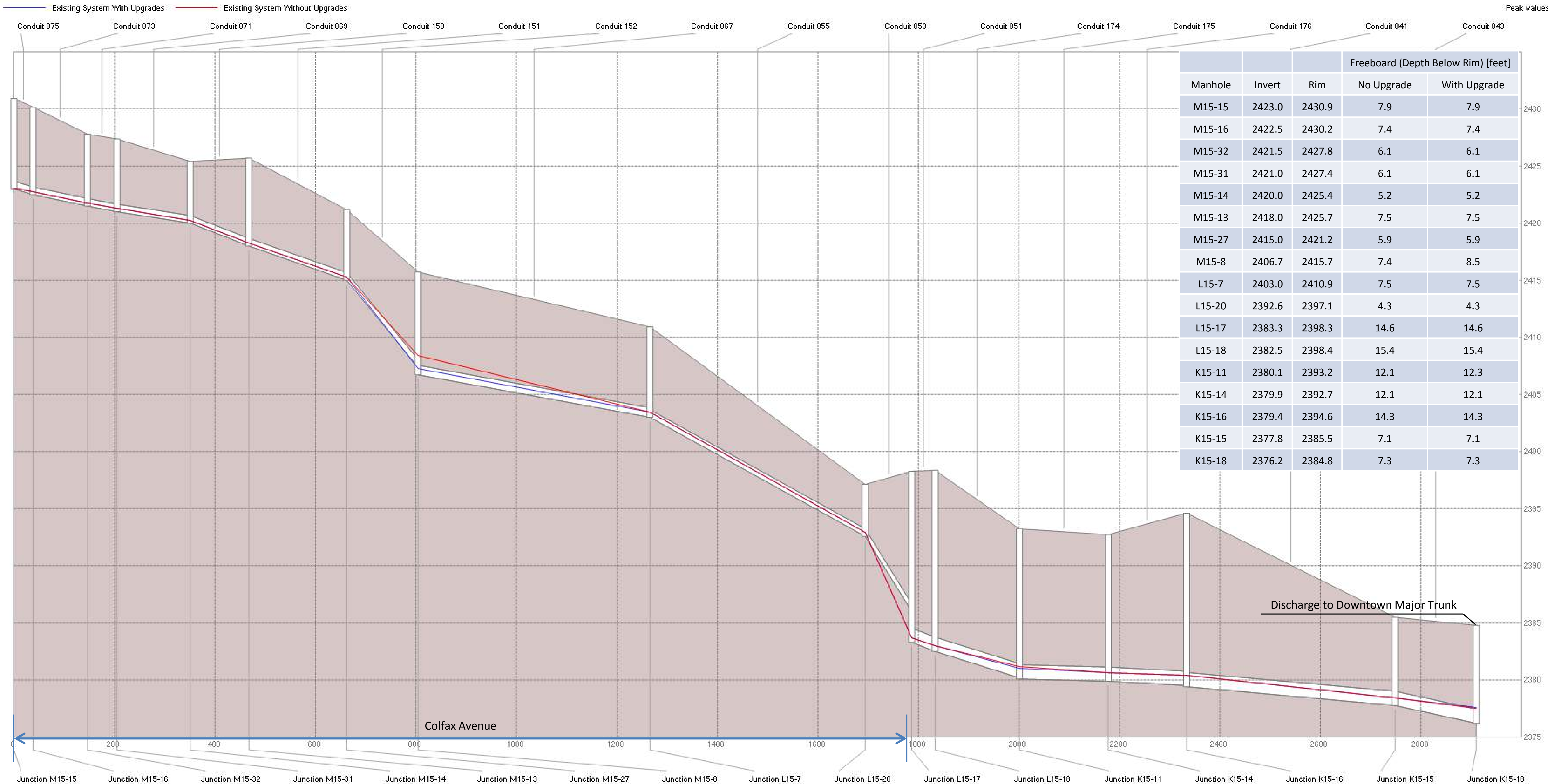
Figure No.
E-10

Title
Existing System Pre- and Post –improvement Results
HGL Profile 2



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
M15-15	2423.0	2430.9	7.9	7.9
M15-16	2422.5	2430.2	7.4	7.4
M15-32	2421.5	2427.8	6.1	6.1
M15-31	2421.0	2427.4	6.1	6.1
M15-14	2420.0	2425.4	5.2	5.2
M15-13	2418.0	2425.7	7.5	7.5
M15-27	2415.0	2421.2	5.9	5.9
M15-8	2406.7	2415.7	7.4	8.5
L15-7	2403.0	2410.9	7.5	7.5
L15-20	2392.6	2397.1	4.3	4.3
L15-17	2383.3	2398.3	14.6	14.6
L15-18	2382.5	2398.4	15.4	15.4
K15-11	2380.1	2393.2	12.1	12.3
K15-14	2379.9	2392.7	12.1	12.1
K15-16	2379.4	2394.6	14.3	14.3
K15-15	2377.8	2385.5	7.1	7.1
K15-18	2376.2	2384.8	7.3	7.3

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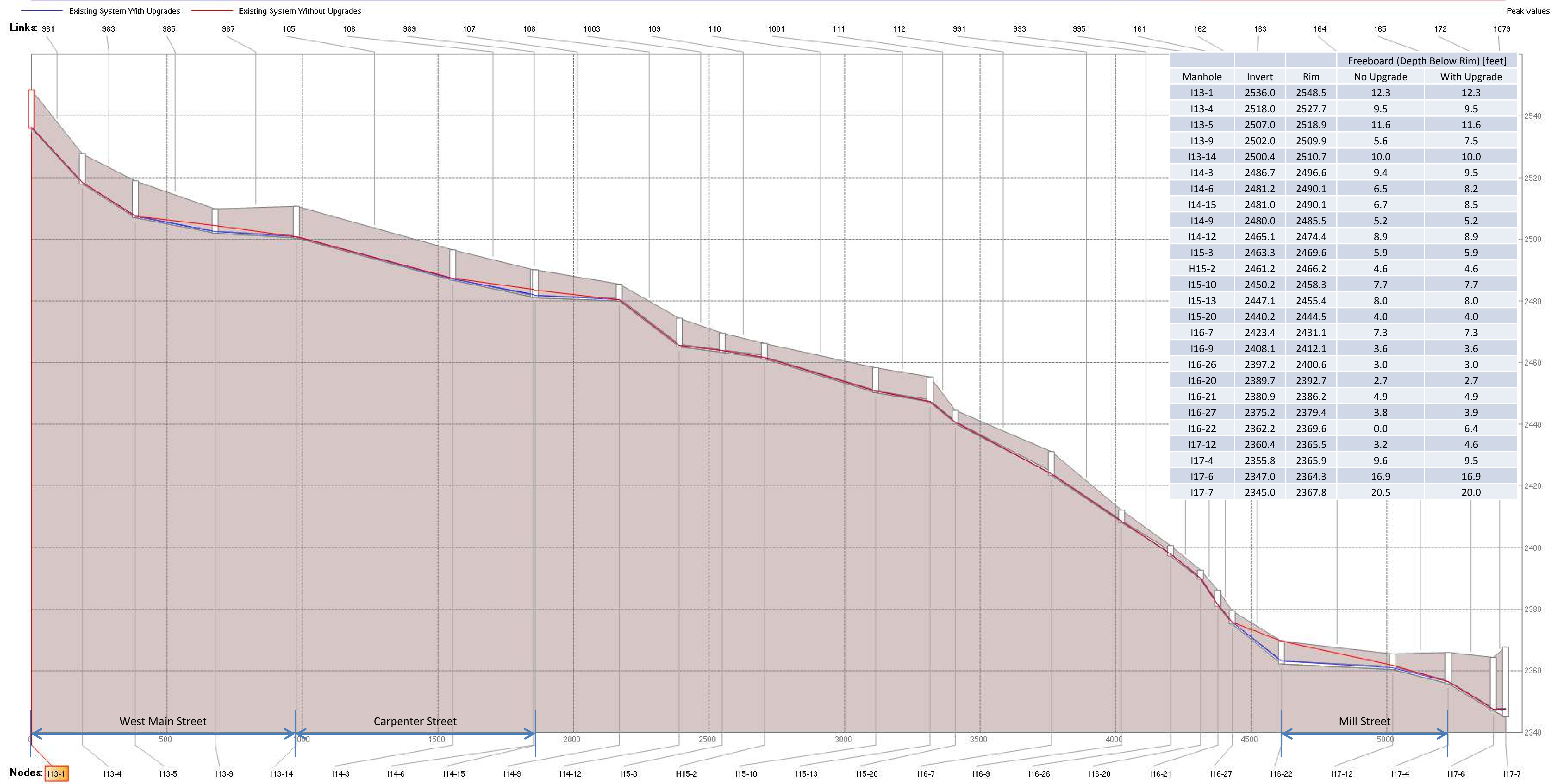
Figure No.
E-11

Title
Existing System Pre- and Post –improvement Results
HGL Profile 3



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
I13-1	2536.0	2548.5	12.3	12.3
I13-4	2518.0	2527.7	9.5	9.5
I13-5	2507.0	2518.9	11.6	11.6
I13-9	2502.0	2509.9	5.6	7.5
I13-14	2500.4	2510.7	10.0	10.0
I14-3	2486.7	2496.6	9.4	9.5
I14-6	2481.2	2490.1	6.5	8.2
I14-15	2481.0	2490.1	6.7	8.5
I14-9	2480.0	2485.5	5.2	5.2
I14-12	2465.1	2474.4	8.9	8.9
I15-3	2463.3	2469.6	5.9	5.9
H15-2	2461.2	2466.2	4.6	4.6
I15-10	2450.2	2458.3	7.7	7.7
I15-13	2447.1	2455.4	8.0	8.0
I15-20	2440.2	2444.5	4.0	4.0
I16-7	2423.4	2431.1	7.3	7.3
I16-9	2408.1	2412.1	3.6	3.6
I16-26	2397.2	2400.6	3.0	3.0
I16-20	2389.7	2392.7	2.7	2.7
I16-21	2380.9	2386.2	4.9	4.9
I16-27	2375.2	2379.4	3.8	3.9
I16-22	2362.2	2369.6	0.0	6.4
I17-12	2360.4	2365.5	3.2	4.6
I17-4	2355.8	2365.9	9.6	9.5
I17-6	2347.0	2364.3	16.9	16.9
I17-7	2345.0	2367.8	20.5	20.0

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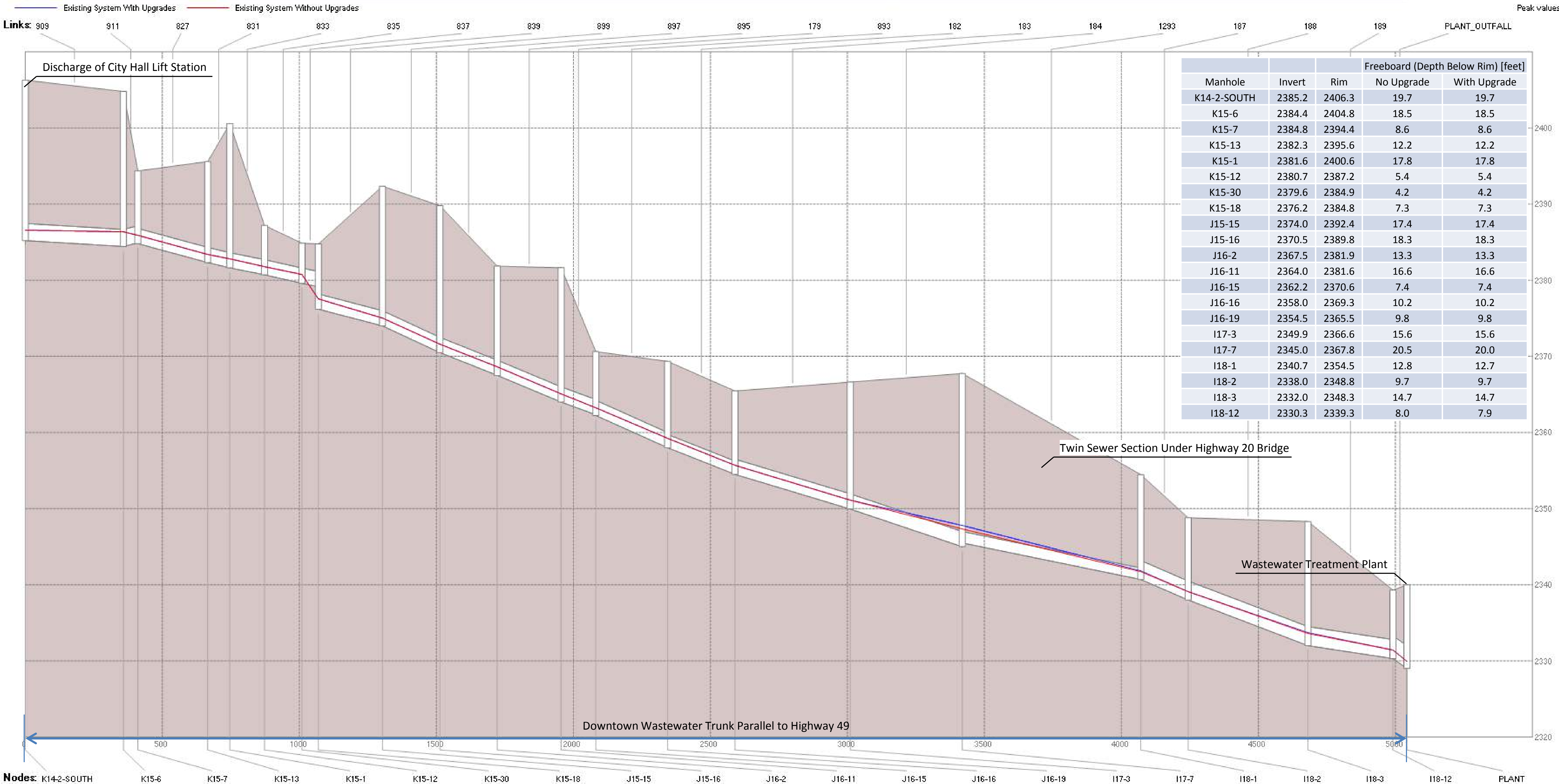
Figure No.
E-12

Title
Existing System Pre- and Post –improvement Results
HGL Profile 4



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K14-2-SOUTH	2385.2	2406.3	19.7	19.7
K15-6	2384.4	2404.8	18.5	18.5
K15-7	2384.8	2394.4	8.6	8.6
K15-13	2382.3	2395.6	12.2	12.2
K15-1	2381.6	2400.6	17.8	17.8
K15-12	2380.7	2387.2	5.4	5.4
K15-30	2379.6	2384.9	4.2	4.2
K15-18	2376.2	2384.8	7.3	7.3
J15-15	2374.0	2392.4	17.4	17.4
J15-16	2370.5	2389.8	18.3	18.3
J16-2	2367.5	2381.9	13.3	13.3
J16-11	2364.0	2381.6	16.6	16.6
J16-15	2362.2	2370.6	7.4	7.4
J16-16	2358.0	2369.3	10.2	10.2
J16-19	2354.5	2365.5	9.8	9.8
I17-3	2349.9	2366.6	15.6	15.6
I17-7	2345.0	2367.8	20.5	20.0
I18-1	2340.7	2354.5	12.8	12.7
I18-2	2338.0	2348.8	9.7	9.7
I18-3	2332.0	2348.3	14.7	14.7
I18-12	2330.3	2339.3	8.0	7.9

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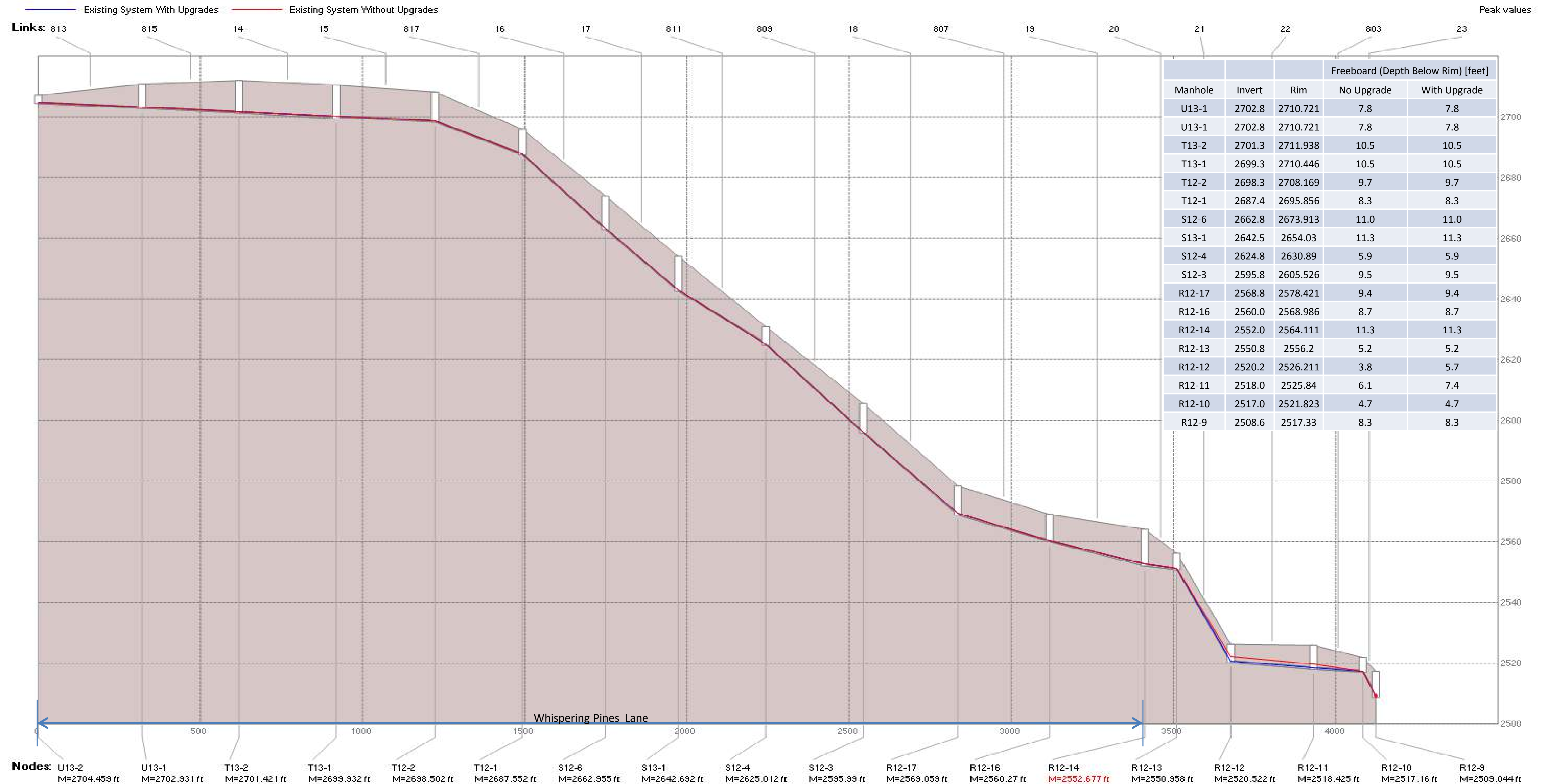
Figure No.
E-13

Title
Existing System Pre- and Post –improvement Results
HGL Profile 5



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
U13-1	2702.8	2710.721	7.8	7.8
U13-1	2702.8	2710.721	7.8	7.8
T13-2	2701.3	2711.938	10.5	10.5
T13-1	2699.3	2710.446	10.5	10.5
T12-2	2698.3	2708.169	9.7	9.7
T12-1	2687.4	2695.856	8.3	8.3
S12-6	2662.8	2673.913	11.0	11.0
S13-1	2642.5	2654.03	11.3	11.3
S12-4	2624.8	2630.89	5.9	5.9
S12-3	2595.8	2605.526	9.5	9.5
R12-17	2568.8	2578.421	9.4	9.4
R12-16	2560.0	2568.986	8.7	8.7
R12-14	2552.0	2564.111	11.3	11.3
R12-13	2550.8	2556.2	5.2	5.2
R12-12	2520.2	2526.211	3.8	5.7
R12-11	2518.0	2525.84	6.1	7.4
R12-10	2517.0	2521.823	4.7	4.7
R12-9	2508.6	2517.33	8.3	8.3

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Figure No.
E-14

Title
Existing System Pre- and Post –improvement Results
HGL Profile 6

			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K9-1	2665.0	2673.06	6.1	6.1
K10-1	2664.3	2668.47	3.7	3.7
K10-4	2663.0	2670.194	6.6	6.6
K10-5	2662.7	2670.222	6.9	6.9
K10-14	2662.0	2670.31	8.0	8.0
K10-15	2656.0	2663.995	7.8	7.8
L11-1	2646.7	2653.954	7.0	7.0
L11-2	2640.0	2646.245	6.0	6.0
L11-9	2629.5	2635.213	5.5	5.5
L11-10	2623.2	2631.488	8.1	8.1
L11-12	2617.7	2625.972	7.9	7.9
L11-5	2617.4	2623.197	5.4	5.4
L11-6	2616.0	2619.932	3.7	3.7
L11-8	2611.5	2615.067	3.2	3.2
M11-2	2609.0	2616.007	6.6	6.6
M11-3	2607.0	2611.282	3.9	3.9
M11-1	2592.4	2597.35	4.4	4.4
M10-6	2591.5	2596.926	5.0	5.0
M10-4	2590.1	2596.139	5.7	5.7
M10-5	2579.1	2584.659	5.3	5.3
M10-7	2551.0	2556.648	5.3	5.3
N11-1	2527.3	2539.357	11.4	11.4
N11-4	2526.4	2539.6	12.4	12.4
N11-5	2523.0	2530.537	7.1	7.1
M11-4	2514.0	2521.517	7.1	7.1
M11-7	2496.0	2503.392	7.0	7.0
M11-10	2483.0	2490.047	6.6	6.6
M12-2	2472.0	2477.608	5.0	5.0
M12-5	2468.0	2468	0.6	0.6
M12-15	2429.0	2436.522	6.3	6.9
M13-3	2428.0	2434.574	5.9	5.9
M13-7	2421.4	2432.18	10.0	10.0

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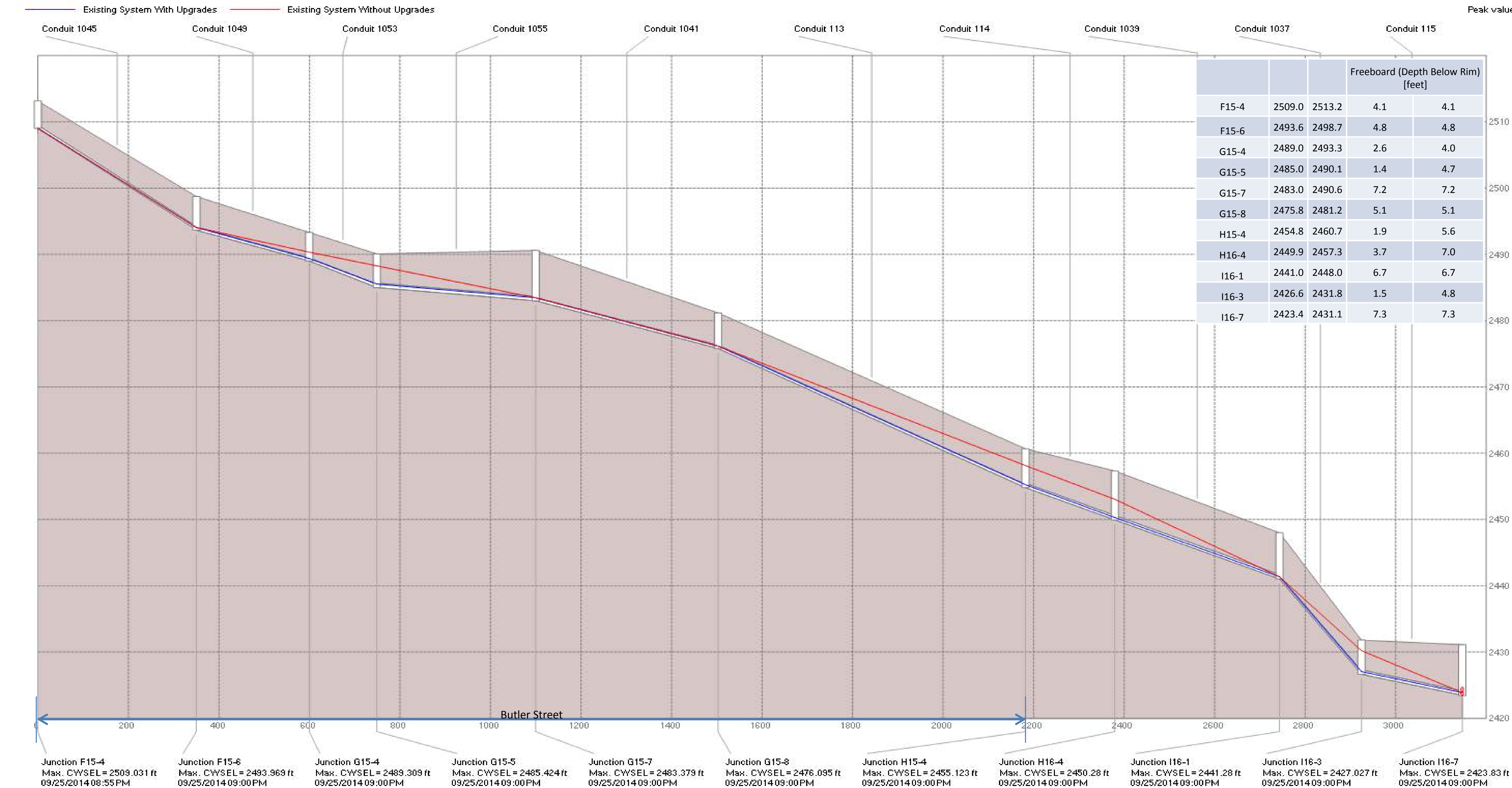
Figure No.
E-15

Title
Existing System Pre- and Post –improvement Results
HGL Profile 7



			Freeboard (Depth Below Rim) [feet]	
F15-4	2509.0	2513.2	4.1	4.1
F15-6	2493.6	2498.7	4.8	4.8
G15-4	2489.0	2493.3	2.6	4.0
G15-5	2485.0	2490.1	1.4	4.7
G15-7	2483.0	2490.6	7.2	7.2
G15-8	2475.8	2481.2	5.1	5.1
H15-4	2454.8	2460.7	1.9	5.6
H16-4	2449.9	2457.3	3.7	7.0
I16-1	2441.0	2448.0	6.7	6.7
I16-3	2426.6	2431.8	1.5	4.8
I16-7	2423.4	2431.1	7.3	7.3

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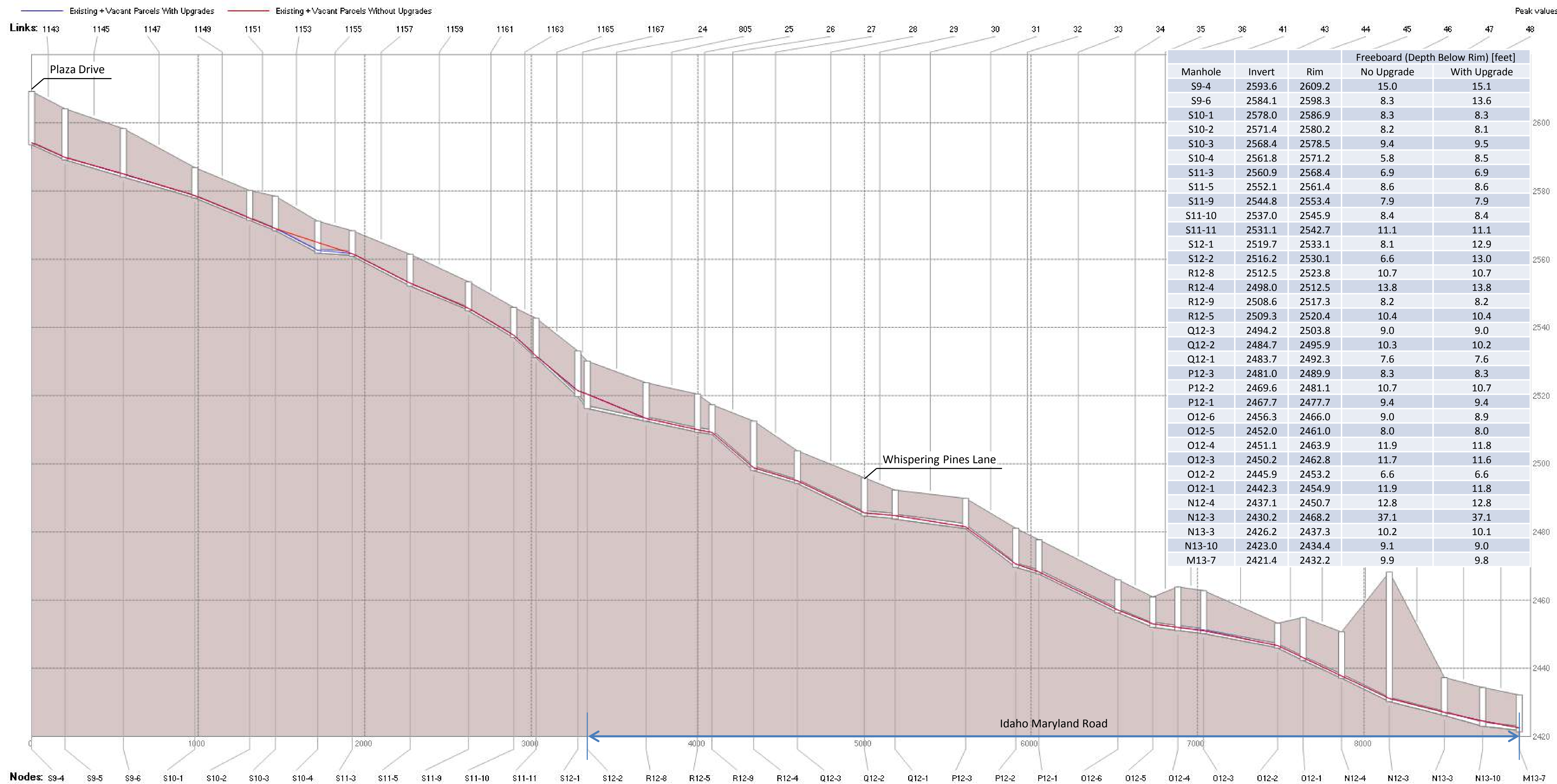
Figure No.
E-16

Title
Existing System Pre- and Post – improvement Results
HGL Profile 8



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
S9-4	2593.6	2609.2	15.0	15.1
S9-6	2584.1	2598.3	8.3	13.6
S10-1	2578.0	2586.9	8.3	8.3
S10-2	2571.4	2580.2	8.2	8.1
S10-3	2568.4	2578.5	9.4	9.5
S10-4	2561.8	2571.2	5.8	8.5
S11-3	2560.9	2568.4	6.9	6.9
S11-5	2552.1	2561.4	8.6	8.6
S11-9	2544.8	2553.4	7.9	7.9
S11-10	2537.0	2545.9	8.4	8.4
S11-11	2531.1	2542.7	11.1	11.1
S12-1	2519.7	2533.1	8.1	12.9
S12-2	2516.2	2530.1	6.6	13.0
R12-8	2512.5	2523.8	10.7	10.7
R12-4	2498.0	2512.5	13.8	13.8
R12-9	2508.6	2517.3	8.2	8.2
R12-5	2509.3	2520.4	10.4	10.4
Q12-3	2494.2	2503.8	9.0	9.0
Q12-2	2484.7	2495.9	10.3	10.2
Q12-1	2483.7	2492.3	7.6	7.6
P12-3	2481.0	2489.9	8.3	8.3
P12-2	2469.6	2481.1	10.7	10.7
P12-1	2467.7	2477.7	9.4	9.4
O12-6	2456.3	2466.0	9.0	8.9
O12-5	2452.0	2461.0	8.0	8.0
O12-4	2451.1	2463.9	11.9	11.8
O12-3	2450.2	2462.8	11.7	11.6
O12-2	2445.9	2453.2	6.6	6.6
O12-1	2442.3	2454.9	11.9	11.8
N12-4	2437.1	2450.7	12.8	12.8
N12-3	2430.2	2468.2	37.1	37.1
N13-3	2426.2	2437.3	10.2	10.1
N13-10	2423.0	2434.4	9.1	9.0
M13-7	2421.4	2432.2	9.9	9.8

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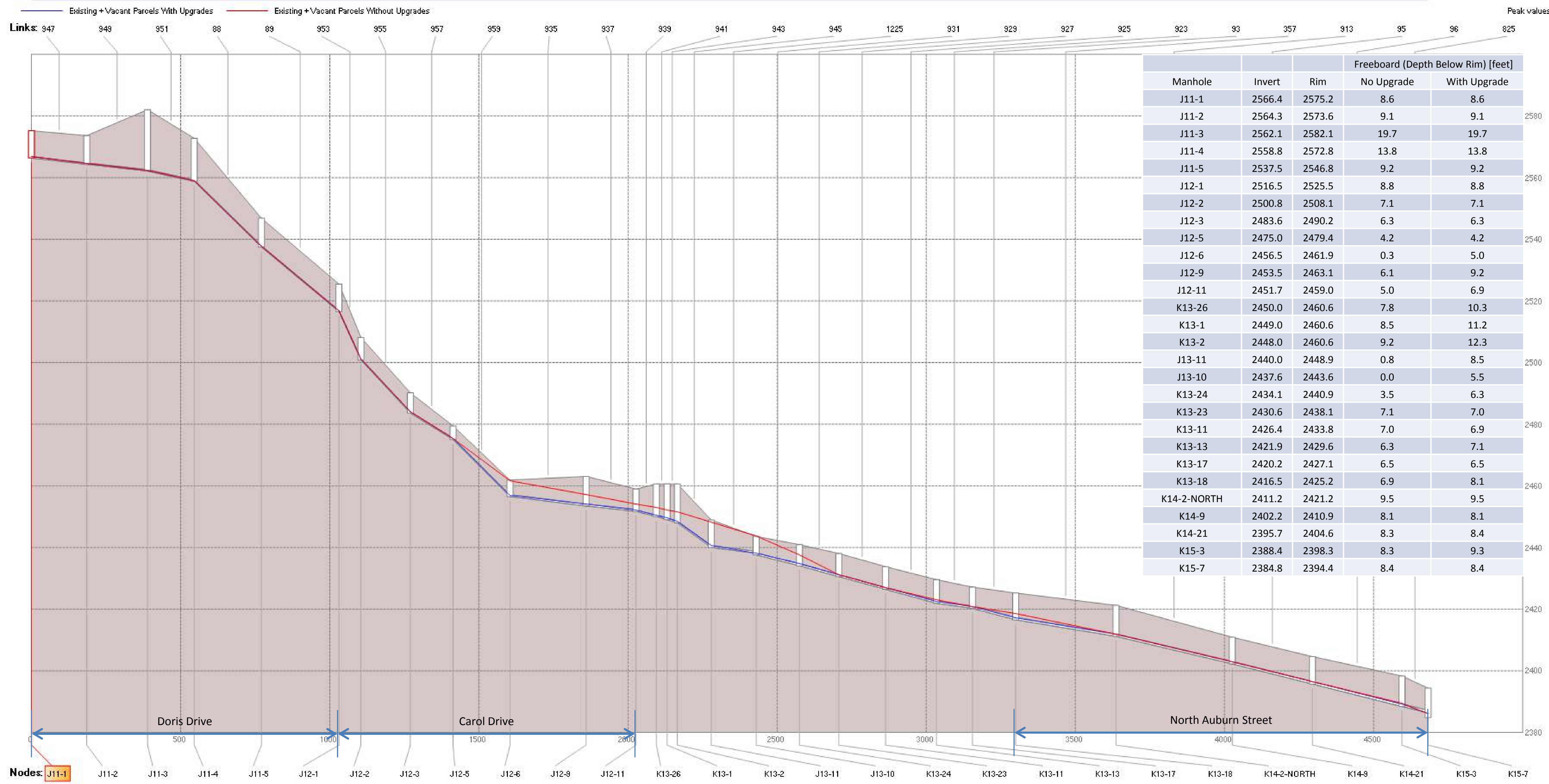
Figure No.
E-17

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 1



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
J11-1	2566.4	2575.2	8.6	8.6
J11-2	2564.3	2573.6	9.1	9.1
J11-3	2562.1	2582.1	19.7	19.7
J11-4	2558.8	2572.8	13.8	13.8
J11-5	2537.5	2546.8	9.2	9.2
J12-1	2516.5	2525.5	8.8	8.8
J12-2	2500.8	2508.1	7.1	7.1
J12-3	2483.6	2490.2	6.3	6.3
J12-5	2475.0	2479.4	4.2	4.2
J12-6	2456.5	2461.9	0.3	5.0
J12-9	2453.5	2463.1	6.1	9.2
J12-11	2451.7	2459.0	5.0	6.9
K13-26	2450.0	2460.6	7.8	10.3
K13-1	2449.0	2460.6	8.5	11.2
K13-2	2448.0	2460.6	9.2	12.3
J13-11	2440.0	2448.9	0.8	8.5
J13-10	2437.6	2443.6	0.0	5.5
K13-24	2434.1	2440.9	3.5	6.3
K13-23	2430.6	2438.1	7.1	7.0
K13-11	2426.4	2433.8	7.0	6.9
K13-13	2421.9	2429.6	6.3	7.1
K13-17	2420.2	2427.1	6.5	6.5
K13-18	2416.5	2425.2	6.9	8.1
K14-2-NORTH	2411.2	2421.2	9.5	9.5
K14-9	2402.2	2410.9	8.1	8.1
K14-21	2395.7	2404.6	8.3	8.4
K15-3	2388.4	2398.3	8.3	9.3
K15-7	2384.8	2394.4	8.4	8.4

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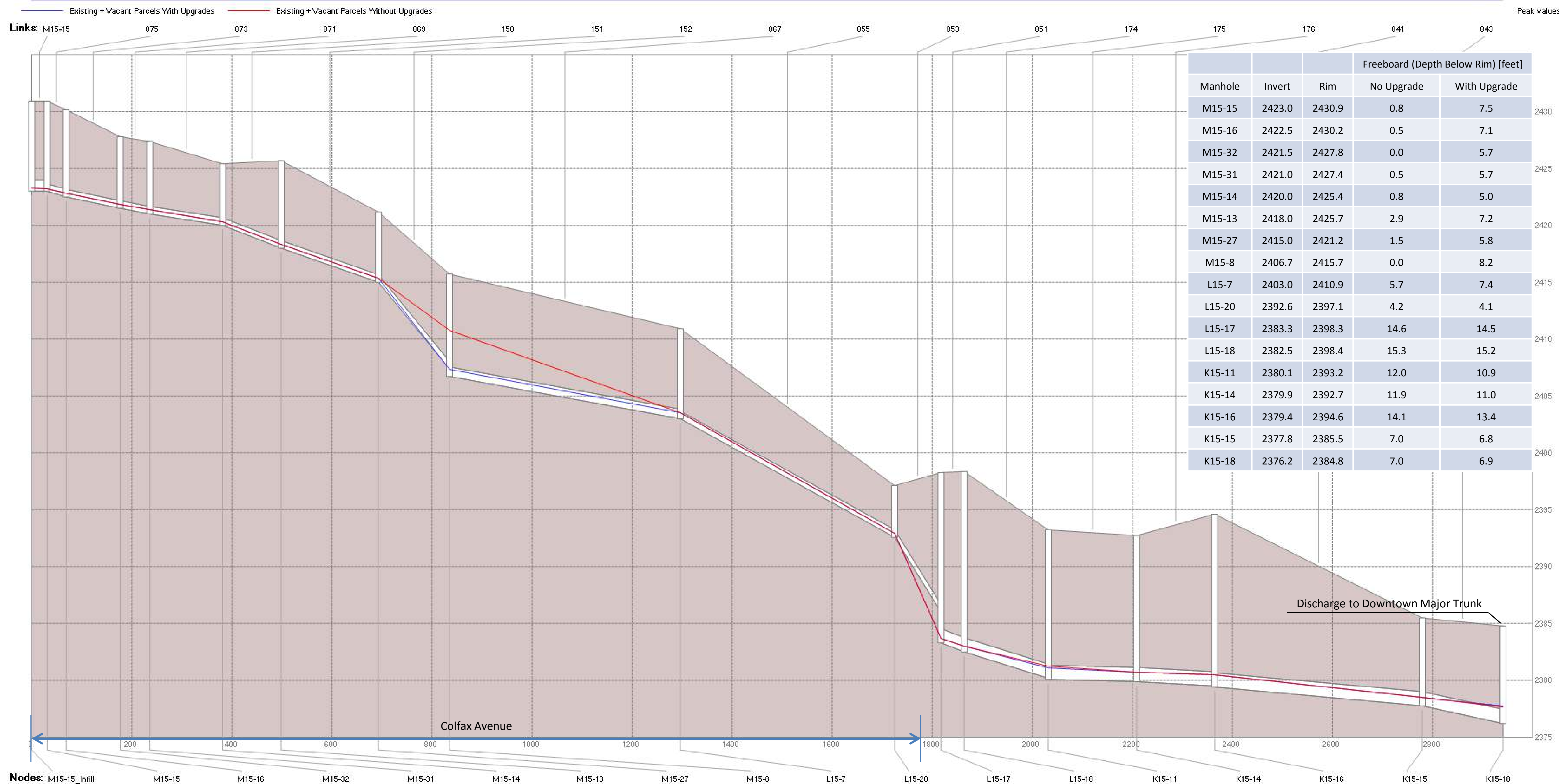
Figure No.
E-18

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 2



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
M15-15	2423.0	2430.9	0.8	7.5
M15-16	2422.5	2430.2	0.5	7.1
M15-32	2421.5	2427.8	0.0	5.7
M15-31	2421.0	2427.4	0.5	5.7
M15-14	2420.0	2425.4	0.8	5.0
M15-13	2418.0	2425.7	2.9	7.2
M15-27	2415.0	2421.2	1.5	5.8
M15-8	2406.7	2415.7	0.0	8.2
L15-7	2403.0	2410.9	5.7	7.4
L15-20	2392.6	2397.1	4.2	4.1
L15-17	2383.3	2398.3	14.6	14.5
L15-18	2382.5	2398.4	15.3	15.2
K15-11	2380.1	2393.2	12.0	10.9
K15-14	2379.9	2392.7	11.9	11.0
K15-16	2379.4	2394.6	14.1	13.4
K15-15	2377.8	2385.5	7.0	6.8
K15-18	2376.2	2384.8	7.0	6.9

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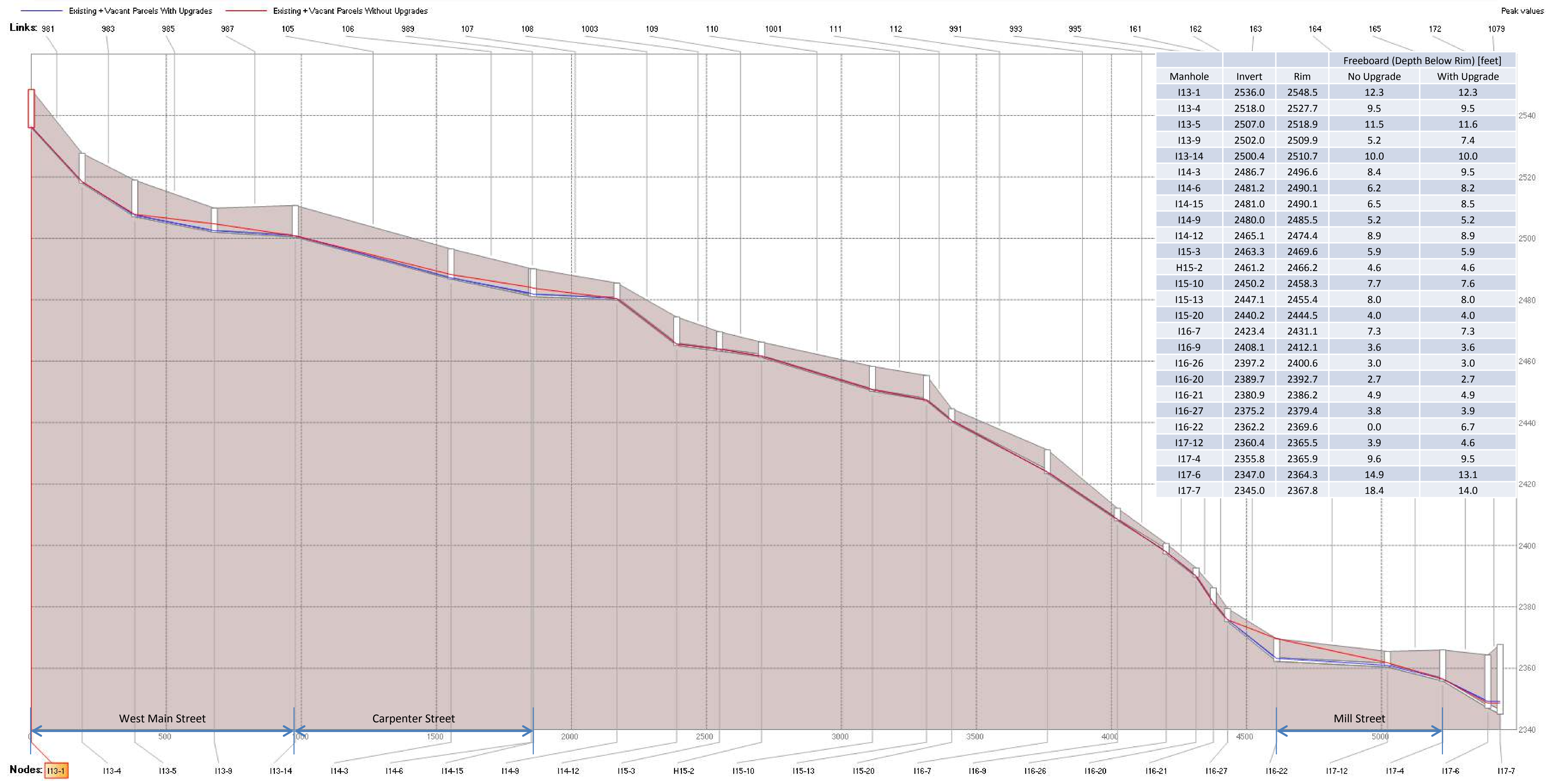
Figure No.
E-19

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 3



Manhole	Invert	Rim	Freeboard (Depth Below Rim) [feet] No Upgrade	With Upgrade
I13-1	2536.0	2548.5	12.3	12.3
I13-4	2518.0	2527.7	9.5	9.5
I13-5	2507.0	2518.9	11.5	11.6
I13-9	2502.0	2509.9	5.2	7.4
I13-14	2500.4	2510.7	10.0	10.0
I14-3	2486.7	2496.6	8.4	9.5
I14-6	2481.2	2490.1	6.2	8.2
I14-15	2481.0	2490.1	6.5	8.5
I14-9	2480.0	2485.5	5.2	5.2
I14-12	2465.1	2474.4	8.9	8.9
I15-3	2463.3	2469.6	5.9	5.9
H15-2	2461.2	2466.2	4.6	4.6
I15-10	2450.2	2458.3	7.7	7.6
I15-13	2447.1	2455.4	8.0	8.0
I15-20	2440.2	2444.5	4.0	4.0
I16-7	2423.4	2431.1	7.3	7.3
I16-9	2408.1	2412.1	3.6	3.6
I16-26	2397.2	2400.6	3.0	3.0
I16-20	2389.7	2392.7	2.7	2.7
I16-21	2380.9	2386.2	4.9	4.9
I16-27	2375.2	2379.4	3.8	3.9
I16-22	2362.2	2369.6	0.0	6.7
I17-12	2360.4	2365.5	3.9	4.6
I17-4	2355.8	2365.9	9.6	9.5
I17-6	2347.0	2364.3	14.9	13.1
I17-7	2345.0	2367.8	18.4	14.0

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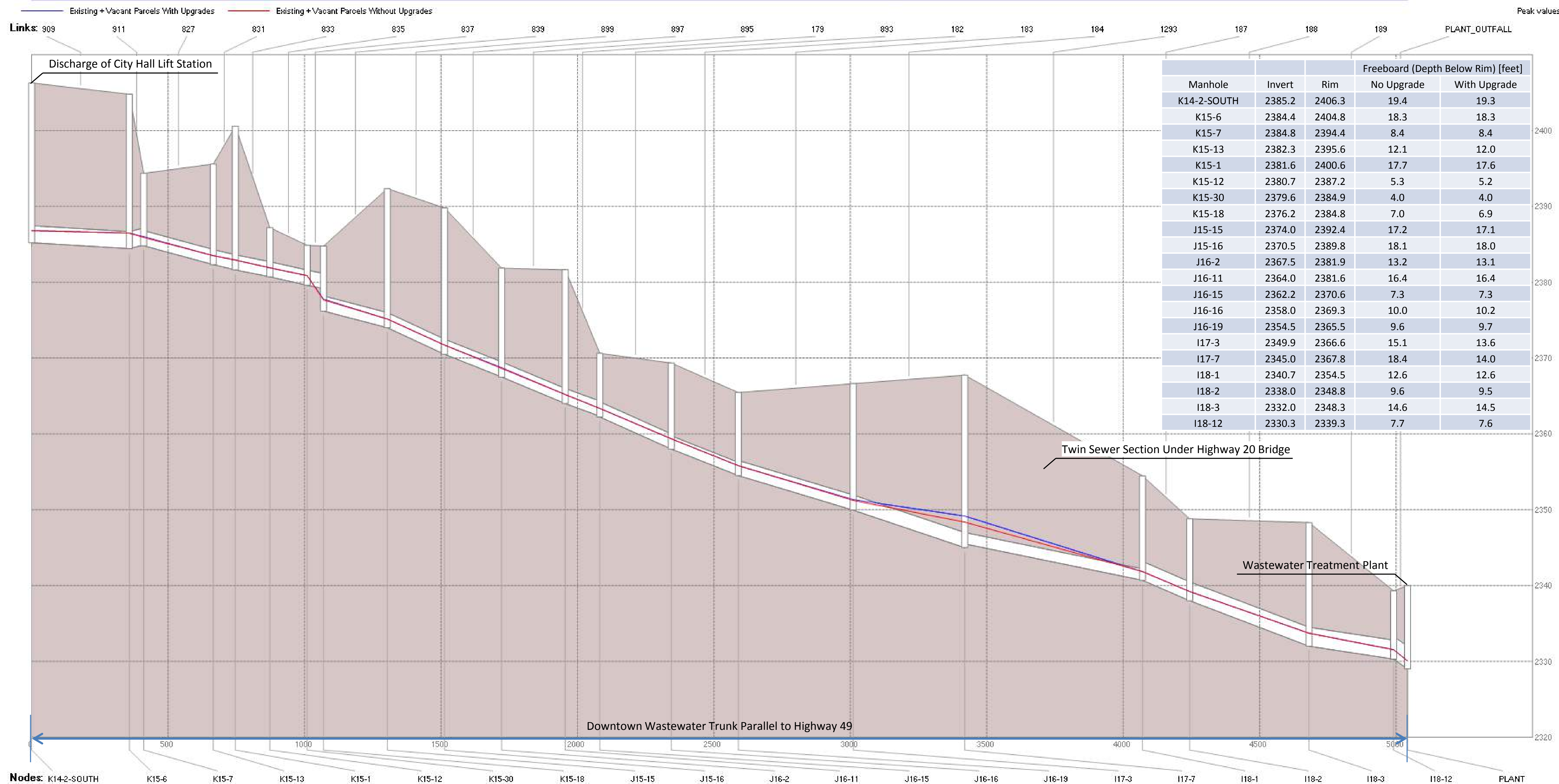
Figure No.
E-20
Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 4



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K14-2-SOUTH	2385.2	2406.3	19.4	19.3
K15-6	2384.4	2404.8	18.3	18.3
K15-7	2384.8	2394.4	8.4	8.4
K15-13	2382.3	2395.6	12.1	12.0
K15-1	2381.6	2400.6	17.7	17.6
K15-12	2380.7	2387.2	5.3	5.2
K15-30	2379.6	2384.9	4.0	4.0
K15-18	2376.2	2384.8	7.0	6.9
J15-15	2374.0	2392.4	17.2	17.1
J15-16	2370.5	2389.8	18.1	18.0
J16-2	2367.5	2381.9	13.2	13.1
J16-11	2364.0	2381.6	16.4	16.4
J16-15	2362.2	2370.6	7.3	7.3
J16-16	2358.0	2369.3	10.0	10.2
J16-19	2354.5	2365.5	9.6	9.7
I17-3	2349.9	2366.6	15.1	13.6
I17-7	2345.0	2367.8	18.4	14.0
I18-1	2340.7	2354.5	12.6	12.6
I18-2	2338.0	2348.8	9.6	9.5
I18-3	2332.0	2348.3	14.6	14.5
I18-12	2330.3	2339.3	7.7	7.6

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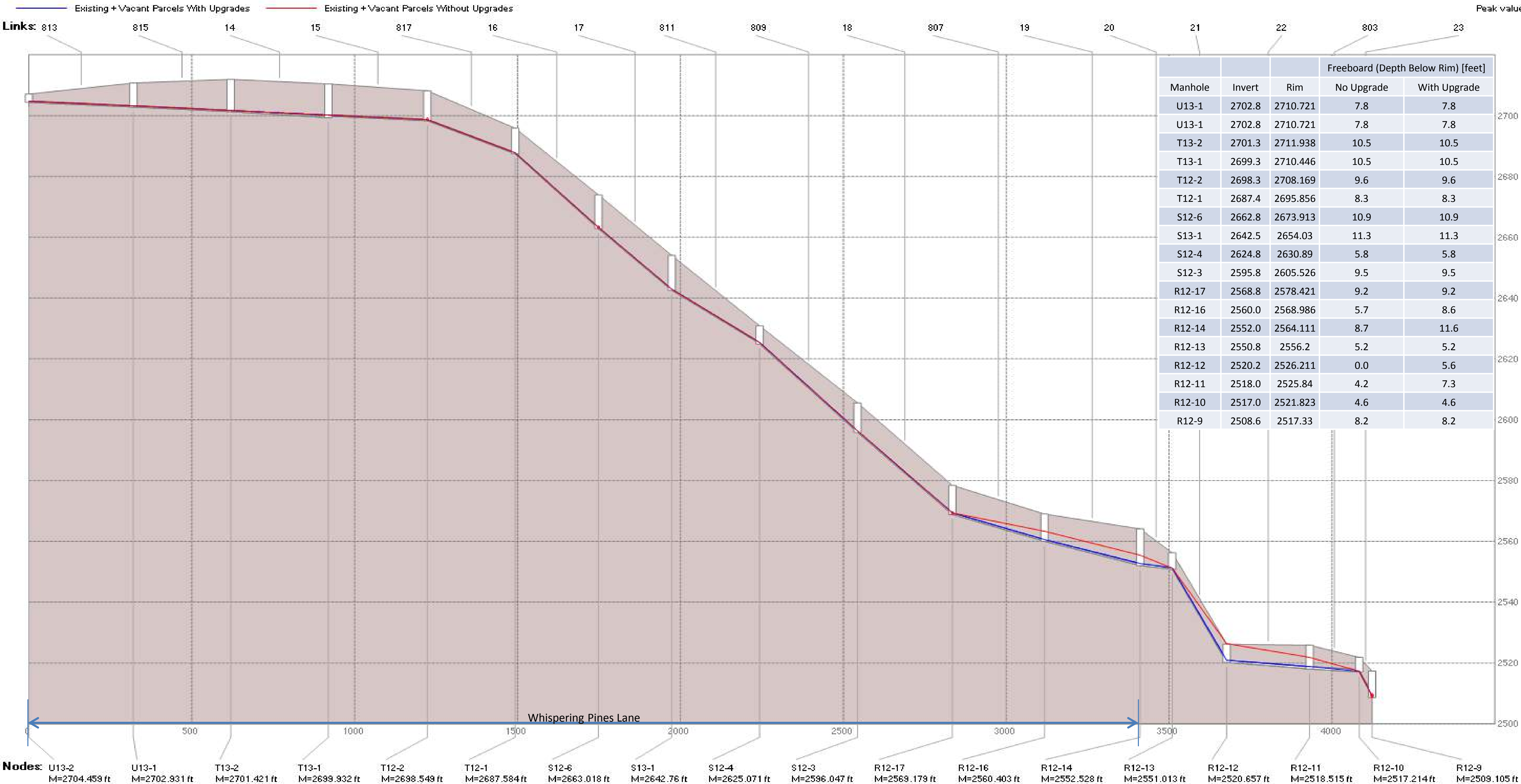
Figure No.
E-21

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 5



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
U13-1	2702.8	2710.721	7.8	7.8
U13-1	2702.8	2710.721	7.8	7.8
T13-2	2701.3	2711.938	10.5	10.5
T13-1	2699.3	2710.446	10.5	10.5
T12-2	2698.3	2708.169	9.6	9.6
T12-1	2687.4	2695.856	8.3	8.3
S12-6	2662.8	2673.913	10.9	10.9
S13-1	2642.5	2654.03	11.3	11.3
S12-4	2624.8	2630.89	5.8	5.8
S12-3	2595.8	2605.526	9.5	9.5
R12-17	2568.8	2578.421	9.2	9.2
R12-16	2560.0	2568.986	5.7	8.6
R12-14	2552.0	2564.111	8.7	11.6
R12-13	2550.8	2556.2	5.2	5.2
R12-12	2520.2	2526.211	0.0	5.6
R12-11	2518.0	2525.84	4.2	7.3
R12-10	2517.0	2521.823	4.6	4.6
R12-9	2508.6	2517.33	8.2	8.2

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Figure No.
E-22

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 6

Manhole	Invert	Rim	Freeboard (Depth Below Rim) [feet] No Upgrade	Freeboard (Depth Below Rim) [feet] With Upgrade
K9-1	2665.0	2673.06	7.1	7.1
K10-1	2664.3	2668.47	3.6	3.6
K10-4	2663.0	2670.194	6.5	6.5
K10-5	2662.7	2670.222	6.8	6.8
K10-14	2662.0	2670.31	8.0	8.0
K10-15	2656.0	2663.995	7.7	7.7
L11-1	2646.7	2653.954	7.0	7.0
L11-2	2640.0	2646.245	6.0	6.0
L11-9	2629.5	2635.213	5.5	5.5
L11-10	2623.2	2631.488	8.1	8.1
L11-12	2617.7	2625.972	7.8	7.8
L11-5	2617.4	2623.197	5.4	5.4
L11-6	2616.0	2619.932	3.7	3.7
L11-8	2611.5	2615.067	3.2	3.2
M11-2	2609.0	2616.007	6.6	6.6
M11-3	2607.0	2611.282	3.9	3.9
M11-1	2592.4	2597.35	4.4	4.4
M10-6	2591.5	2596.926	5.0	5.0
M10-4	2590.1	2596.139	5.7	5.7
M10-5	2579.1	2584.659	5.3	5.3
M10-7	2551.0	2556.648	5.3	5.3
N11-1	2527.3	2539.357	11.4	11.4
N11-4	2526.4	2539.6	12.3	12.3
N11-5	2523.0	2530.537	7.1	7.1
M11-4	2514.0	2521.517	7.1	7.1
M11-7	2496.0	2503.392	7.0	7.0
M11-10	2483.0	2490.047	6.6	6.6
M12-2	2472.0	2477.608	5.0	5.0
M12-5	2468.0	2468	0.5	0.5
M12-15	2429.0	2436.522	6.2	6.8
M13-3	2428.0	2434.574	5.9	5.9
M13-7	2421.4	2432.18	9.9	9.9

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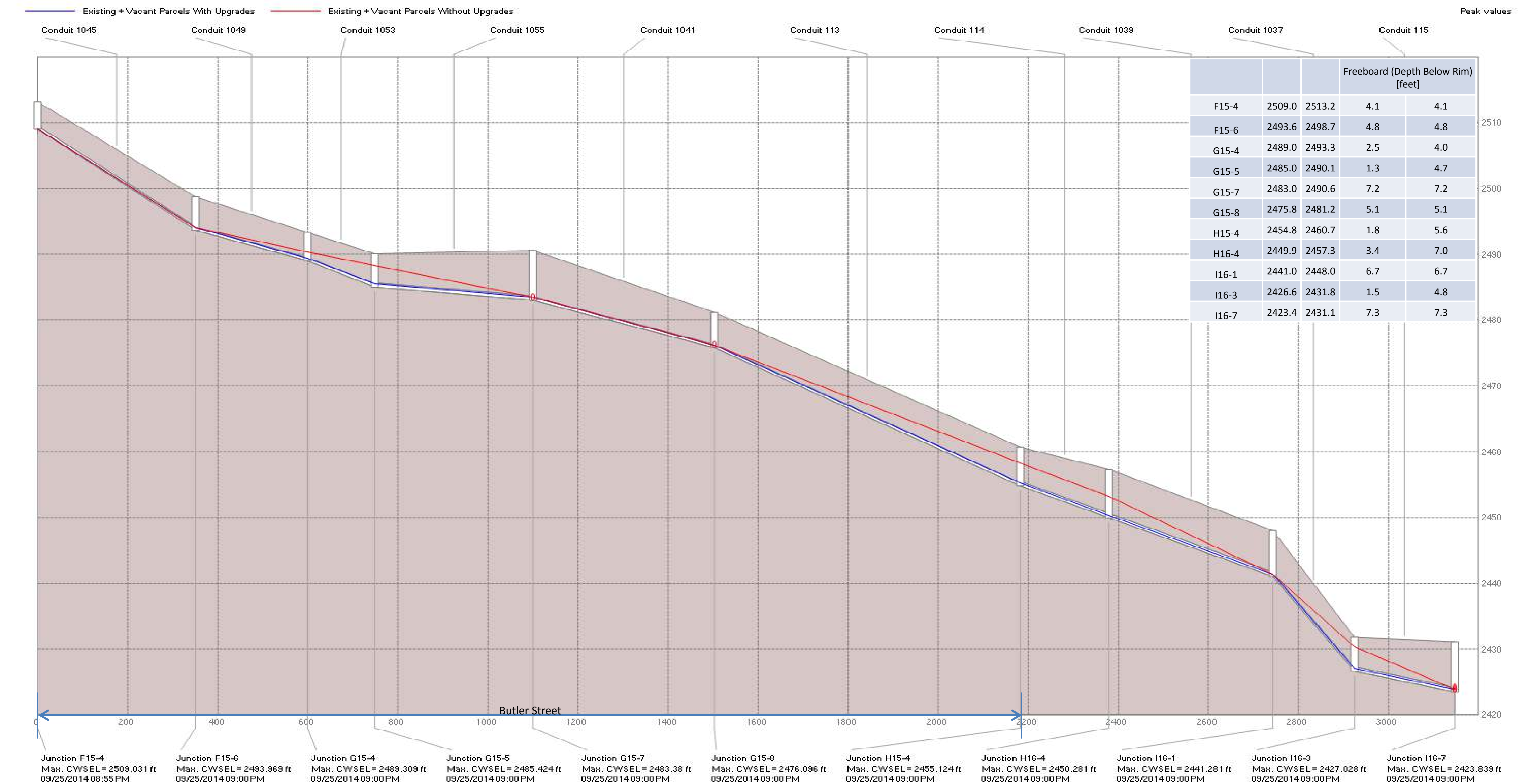
Figure No.
E-23

Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 7



			Freeboard (Depth Below Rim) [feet]	
F15-4	2509.0	2513.2	4.1	4.1
F15-6	2493.6	2498.7	4.8	4.8
G15-4	2489.0	2493.3	2.5	4.0
G15-5	2485.0	2490.1	1.3	4.7
G15-7	2483.0	2490.6	7.2	7.2
G15-8	2475.8	2481.2	5.1	5.1
H15-4	2454.8	2460.7	1.8	5.6
H16-4	2449.9	2457.3	3.4	7.0
I16-1	2441.0	2448.0	6.7	6.7
I16-3	2426.6	2431.8	1.5	4.8
I16-7	2423.4	2431.1	7.3	7.3

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Figure No.
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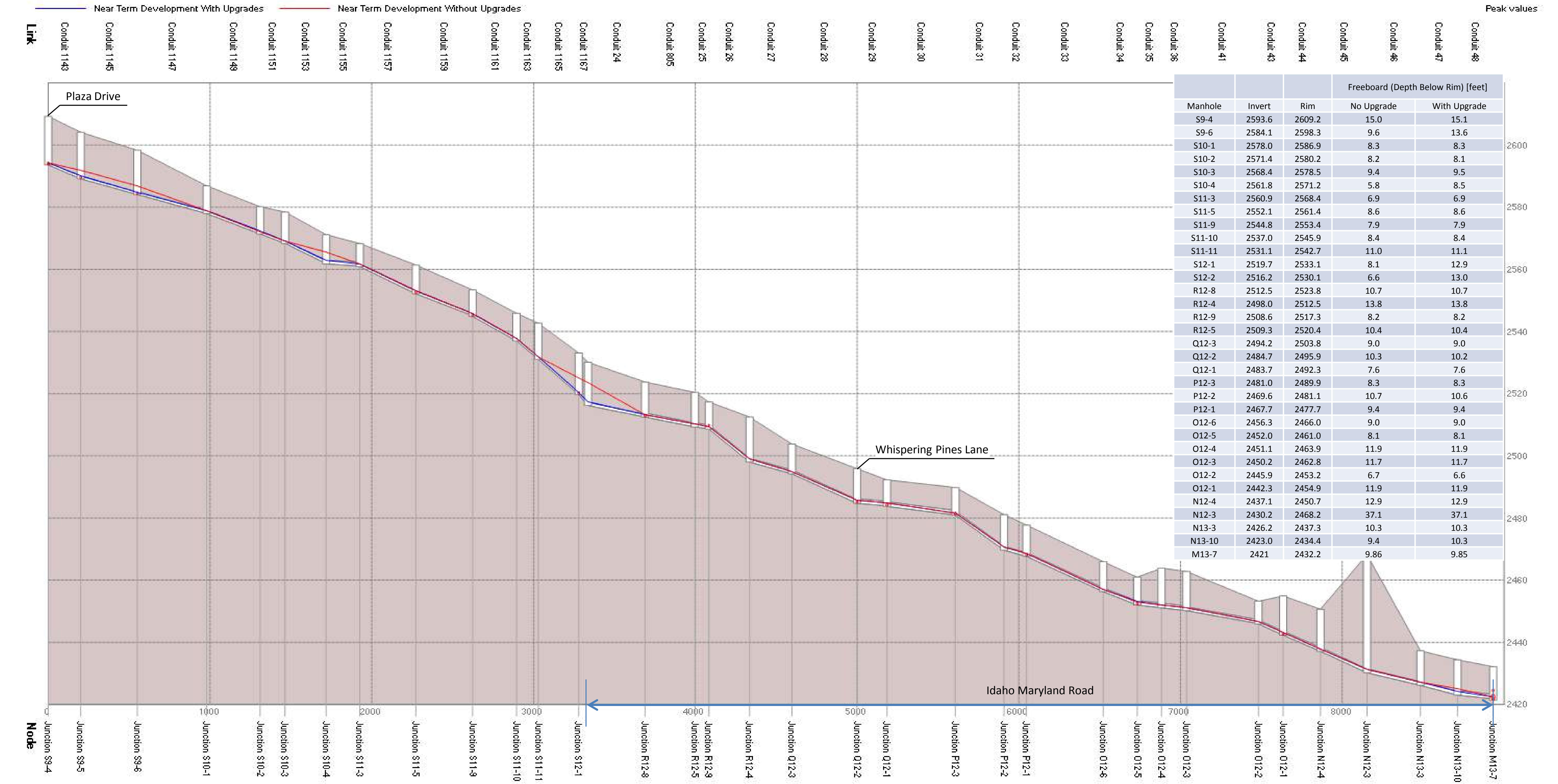
Title
Existing Service Area Build-out
Pre- and Post –improvement Results HGL Profile 8



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
S9-4	2593.6	2609.2	15.0	15.1
S9-6	2584.1	2598.3	9.6	13.6
S10-1	2578.0	2586.9	8.3	8.3
S10-2	2571.4	2580.2	8.2	8.1
S10-3	2568.4	2578.5	9.4	9.5
S10-4	2561.8	2571.2	5.8	8.5
S11-3	2560.9	2568.4	6.9	6.9
S11-5	2552.1	2561.4	8.6	8.6
S11-9	2544.8	2553.4	7.9	7.9
S11-10	2537.0	2545.9	8.4	8.4
S11-11	2531.1	2542.7	11.0	11.1
S12-1	2519.7	2533.1	8.1	12.9
S12-2	2516.2	2530.1	6.6	13.0
R12-8	2512.5	2523.8	10.7	10.7
R12-4	2498.0	2512.5	13.8	13.8
R12-9	2508.6	2517.3	8.2	8.2
R12-5	2509.3	2520.4	10.4	10.4
Q12-3	2494.2	2503.8	9.0	9.0
Q12-2	2484.7	2495.9	10.3	10.2
Q12-1	2483.7	2492.3	7.6	7.6
P12-3	2481.0	2489.9	8.3	8.3
P12-2	2469.6	2481.1	10.7	10.6
P12-1	2467.7	2477.7	9.4	9.4
O12-6	2456.3	2466.0	9.0	9.0
O12-5	2452.0	2461.0	8.1	8.1
O12-4	2451.1	2463.9	11.9	11.9
O12-3	2450.2	2462.8	11.7	11.7
O12-2	2445.9	2453.2	6.7	6.6
O12-1	2442.3	2454.9	11.9	11.9
N12-4	2437.1	2450.7	12.9	12.9
N12-3	2430.2	2468.2	37.1	37.1
N13-3	2426.2	2437.3	10.3	10.3
N13-10	2423.0	2434.4	9.4	10.3
M13-7	2421	2432.2	9.86	9.85

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Figure No.

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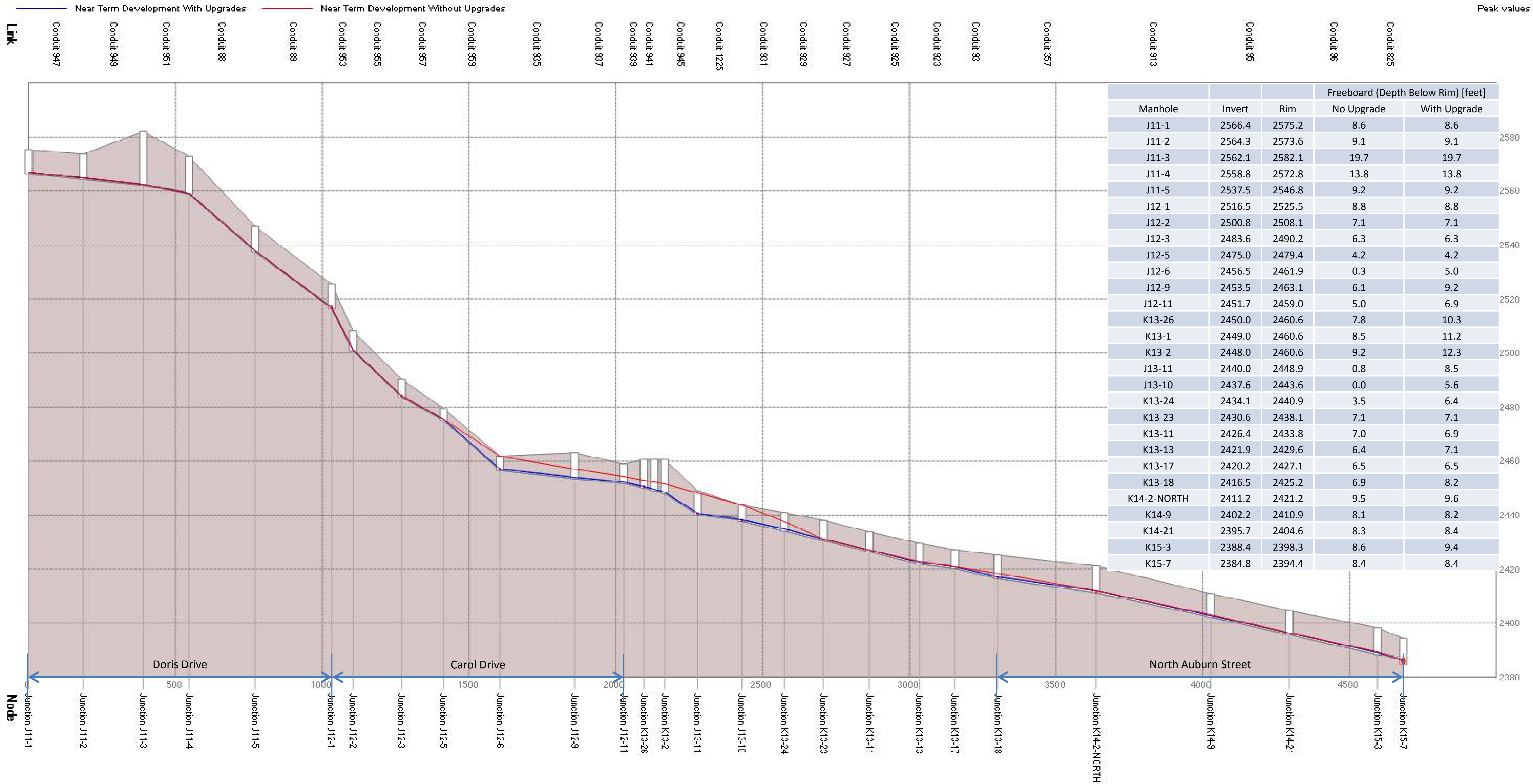
Title

Existing Build-out plus Near Term

Pre- and Post –improvement Results HGL Profile 1



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
J11-1	2566.4	2575.2	8.6	8.6
J11-2	2564.3	2573.6	9.1	9.1
J11-3	2562.1	2582.1	19.7	19.7
J11-4	2558.8	2572.8	13.8	13.8
J11-5	2537.5	2546.8	9.2	9.2
J12-1	2516.5	2525.5	8.8	8.8
J12-2	2500.8	2508.1	7.1	7.1
J12-3	2483.6	2490.2	6.3	6.3
J12-5	2475.0	2479.4	4.2	4.2
J12-6	2456.5	2461.9	0.3	5.0
J12-9	2453.5	2463.1	6.1	9.2
J12-11	2451.7	2459.0	5.0	6.9
K13-26	2450.0	2460.6	7.8	10.3
K13-1	2449.0	2460.6	8.5	11.2
K13-2	2448.0	2460.6	9.2	12.3
J13-11	2440.0	2448.9	0.8	8.5
J13-10	2437.6	2443.6	0.0	5.6
K13-24	2434.1	2440.9	3.5	6.4
K13-23	2430.6	2438.1	7.1	7.1
K13-11	2426.4	2433.8	7.0	6.9
K13-13	2421.9	2429.6	6.4	7.1
K13-17	2420.2	2427.1	6.5	6.5
K13-18	2416.5	2425.2	6.9	8.2
K14-2-NORTH	2411.2	2421.2	9.5	9.6
K14-9	2402.2	2410.9	8.1	8.2
K14-21	2395.7	2404.6	8.3	8.4
K15-3	2388.4	2398.3	8.6	9.4
K15-7	2384.8	2394.4	8.4	8.4

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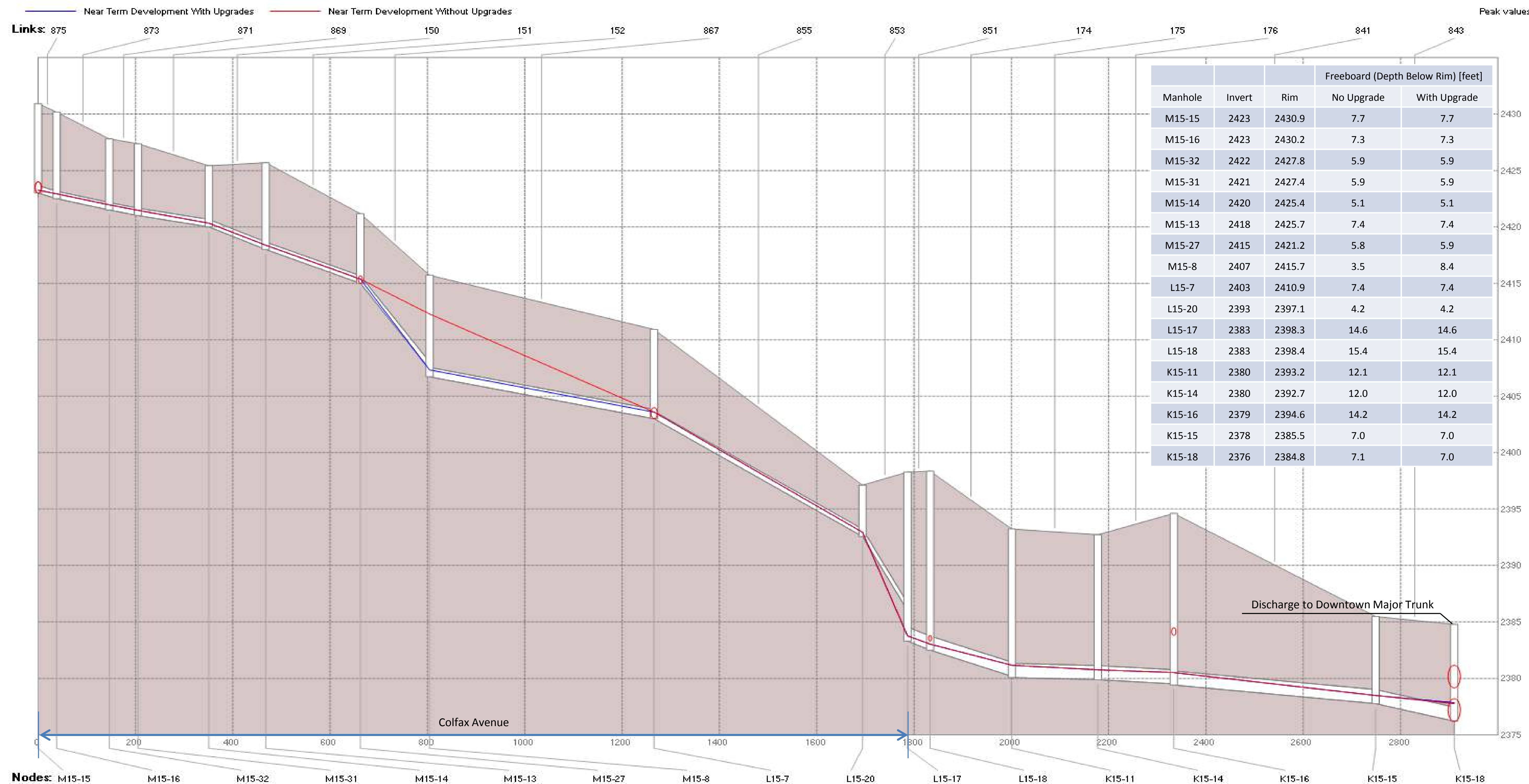
Figure No.
E-26

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 2



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
M15-15	2423	2430.9	7.7	7.7
M15-16	2423	2430.2	7.3	7.3
M15-32	2422	2427.8	5.9	5.9
M15-31	2421	2427.4	5.9	5.9
M15-14	2420	2425.4	5.1	5.1
M15-13	2418	2425.7	7.4	7.4
M15-27	2415	2421.2	5.8	5.9
M15-8	2407	2415.7	3.5	8.4
L15-7	2403	2410.9	7.4	7.4
L15-20	2393	2397.1	4.2	4.2
L15-17	2383	2398.3	14.6	14.6
L15-18	2383	2398.4	15.4	15.4
K15-11	2380	2393.2	12.1	12.1
K15-14	2380	2392.7	12.0	12.0
K15-16	2379	2394.6	14.2	14.2
K15-15	2378	2385.5	7.0	7.0
K15-18	2376	2384.8	7.1	7.0

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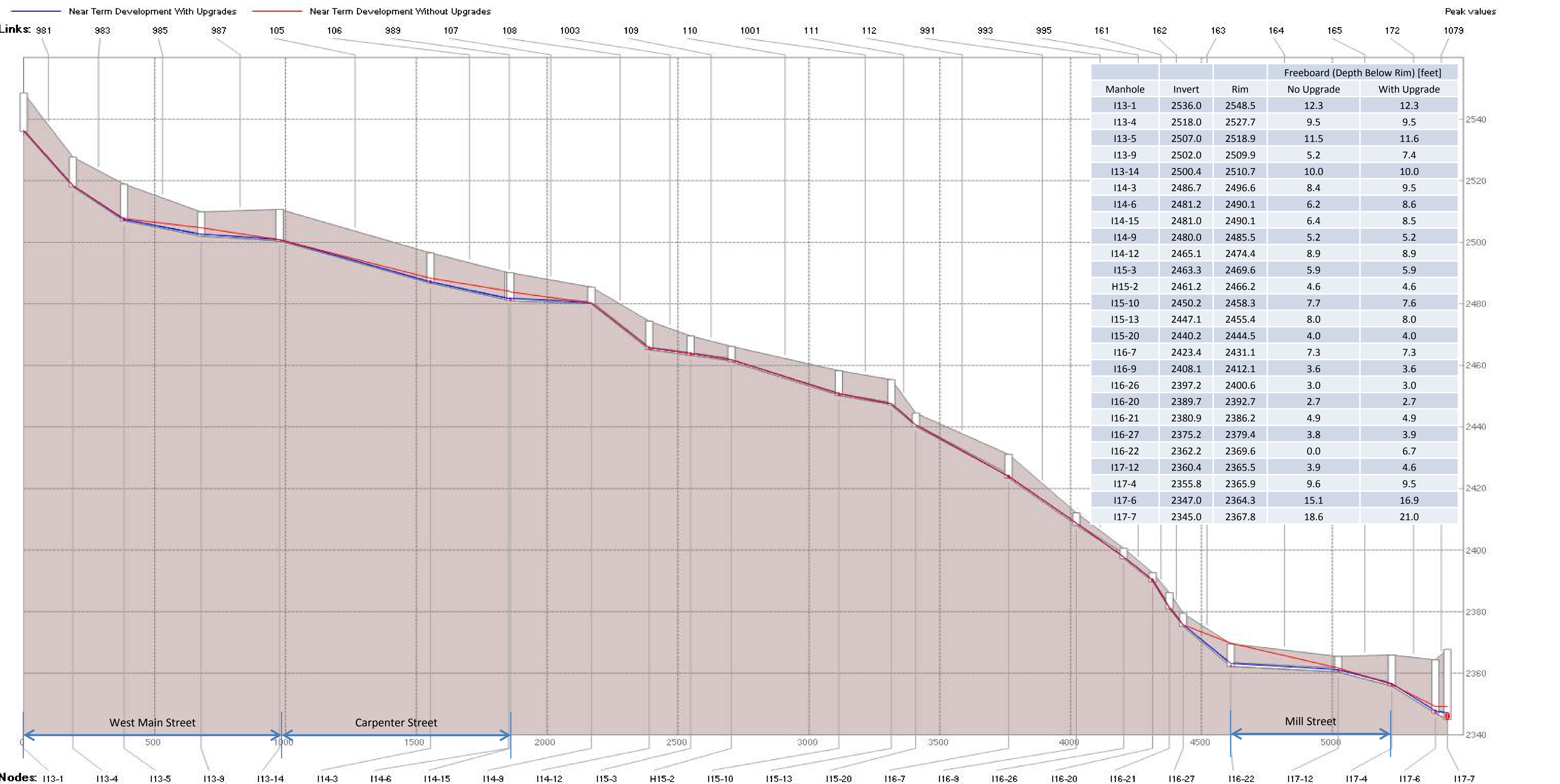
Figure No.
E-27

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 3



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
I13-1	2536.0	2548.5	12.3	12.3
I13-4	2518.0	2527.7	9.5	9.5
I13-5	2507.0	2518.9	11.5	11.6
I13-9	2502.0	2509.9	5.2	7.4
I13-14	2500.4	2510.7	10.0	10.0
I14-3	2486.7	2496.6	8.4	9.5
I14-6	2481.2	2490.1	6.2	8.6
I14-15	2481.0	2490.1	6.4	8.5
I14-9	2480.0	2485.5	5.2	5.2
I14-12	2465.1	2474.4	8.9	8.9
I15-3	2463.3	2469.6	5.9	5.9
H15-2	2461.2	2466.2	4.6	4.6
I15-10	2450.2	2458.3	7.7	7.6
I15-13	2447.1	2455.4	8.0	8.0
I15-20	2440.2	2444.5	4.0	4.0
I16-7	2423.4	2431.1	7.3	7.3
I16-9	2408.1	2412.1	3.6	3.6
I16-26	2397.2	2400.6	3.0	3.0
I16-20	2389.7	2392.7	2.7	2.7
I16-21	2380.9	2386.2	4.9	4.9
I16-27	2375.2	2379.4	3.8	3.9
I16-22	2362.2	2369.6	0.0	6.7
I17-12	2360.4	2365.5	3.9	4.6
I17-4	2355.8	2365.9	9.6	9.5
I17-6	2347.0	2364.3	15.1	16.9
I17-7	2345.0	2367.8	18.6	21.0

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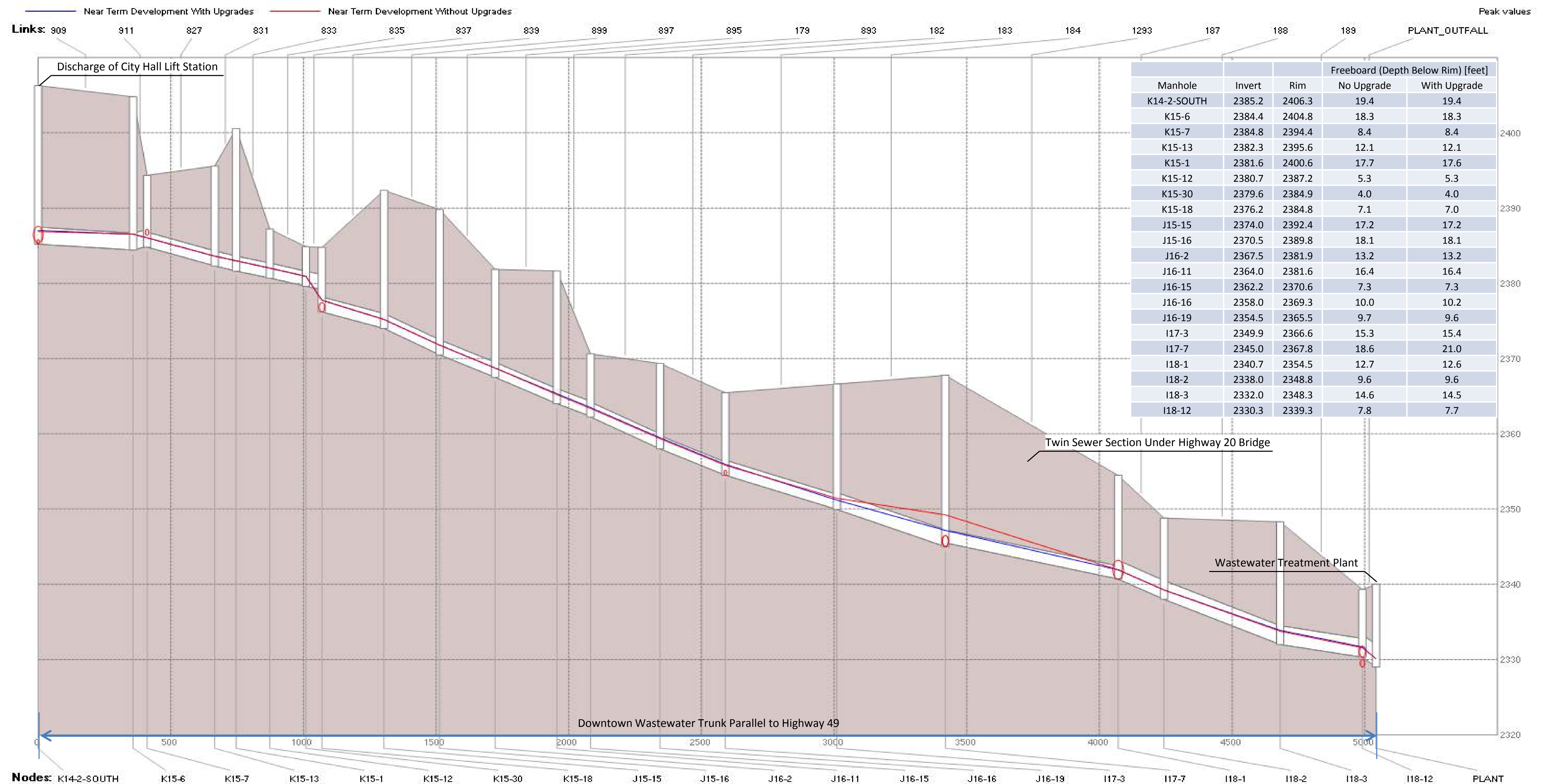
Figure No.
E-28

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 4



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K14-2-SOUTH	2385.2	2406.3	19.4	19.4
K15-6	2384.4	2404.8	18.3	18.3
K15-7	2384.8	2394.4	8.4	8.4
K15-13	2382.3	2395.6	12.1	12.1
K15-1	2381.6	2400.6	17.7	17.6
K15-12	2380.7	2387.2	5.3	5.3
K15-30	2379.6	2384.9	4.0	4.0
K15-18	2376.2	2384.8	7.1	7.0
J15-15	2374.0	2392.4	17.2	17.2
J15-16	2370.5	2389.8	18.1	18.1
J16-2	2367.5	2381.9	13.2	13.2
J16-11	2364.0	2381.6	16.4	16.4
J16-15	2362.2	2370.6	7.3	7.3
J16-16	2358.0	2369.3	10.0	10.2
J16-19	2354.5	2365.5	9.7	9.6
I17-3	2349.9	2366.6	15.3	15.4
I17-7	2345.0	2367.8	18.6	21.0
I18-1	2340.7	2354.5	12.7	12.6
I18-2	2338.0	2348.8	9.6	9.6
I18-3	2332.0	2348.3	14.6	14.5
I18-12	2330.3	2339.3	7.8	7.7

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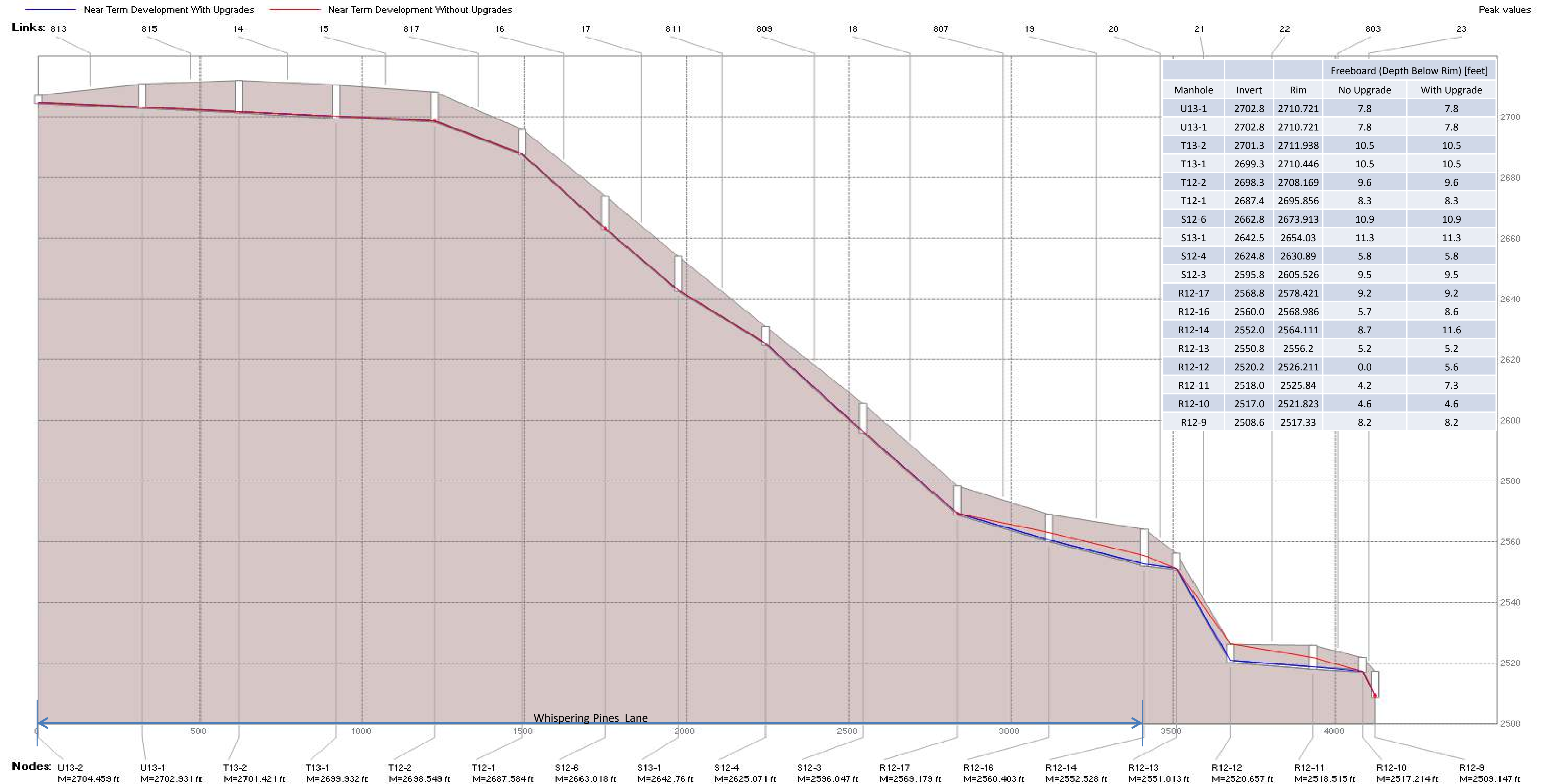
Figure No.
E-29

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 5



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
U13-1	2702.8	2710.721	7.8	7.8
U13-1	2702.8	2710.721	7.8	7.8
T13-2	2701.3	2711.938	10.5	10.5
T13-1	2699.3	2710.446	10.5	10.5
T12-2	2698.3	2708.169	9.6	9.6
T12-1	2687.4	2695.856	8.3	8.3
S12-6	2662.8	2673.913	10.9	10.9
S13-1	2642.5	2654.03	11.3	11.3
S12-4	2624.8	2630.89	5.8	5.8
S12-3	2595.8	2605.526	9.5	9.5
R12-17	2568.8	2578.421	9.2	9.2
R12-16	2560.0	2568.986	5.7	8.6
R12-14	2552.0	2564.111	8.7	11.6
R12-13	2550.8	2556.2	5.2	5.2
R12-12	2520.2	2526.211	0.0	5.6
R12-11	2518.0	2525.84	4.2	7.3
R12-10	2517.0	2521.823	4.6	4.6
R12-9	2508.6	2517.33	8.2	8.2

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Figure No.
E-30

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 6

			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K9-1	2665.0	2673.06	7.1	7.1
K10-1	2664.3	2668.47	3.6	3.6
K10-4	2663.0	2670.194	6.5	6.5
K10-5	2662.7	2670.222	6.8	6.8
K10-14	2662.0	2670.31	8.0	8.0
K10-15	2656.0	2663.995	7.7	7.7
L11-1	2646.7	2653.954	7.0	7.0
L11-2	2640.0	2646.245	6.0	6.0
L11-9	2629.5	2635.213	5.5	5.5
L11-10	2623.2	2631.488	8.1	8.1
L11-12	2617.7	2625.972	7.8	7.8
L11-5	2617.4	2623.197	5.4	5.4
L11-6	2616.0	2619.932	3.7	3.6
L11-8	2611.5	2615.067	3.2	3.2
M11-2	2609.0	2616.007	6.6	6.6
M11-3	2607.0	2611.282	3.9	3.9
M11-1	2592.4	2597.35	4.4	4.4
M10-6	2591.5	2596.926	5.0	5.0
M10-4	2590.1	2596.139	5.7	5.7
M10-5	2579.1	2584.659	5.3	5.3
M10-7	2551.0	2556.648	5.3	5.3
N11-1	2527.3	2539.357	11.3	11.3
N11-4	2526.4	2539.6	12.3	12.3
N11-5	2523.0	2530.537	7.1	7.1
M11-4	2514.0	2521.517	7.1	7.1
M11-7	2496.0	2503.392	7.0	7.0
M11-10	2483.0	2490.047	6.6	6.6
M12-2	2472.0	2477.608	5.0	5.0
M12-5	2468.0	2468	0.5	0.5
M12-15	2429.0	2436.522	6.1	6.8
M13-3	2428.0	2434.574	5.9	5.9
M13-7	2421.4	2432.18	9.9	9.9

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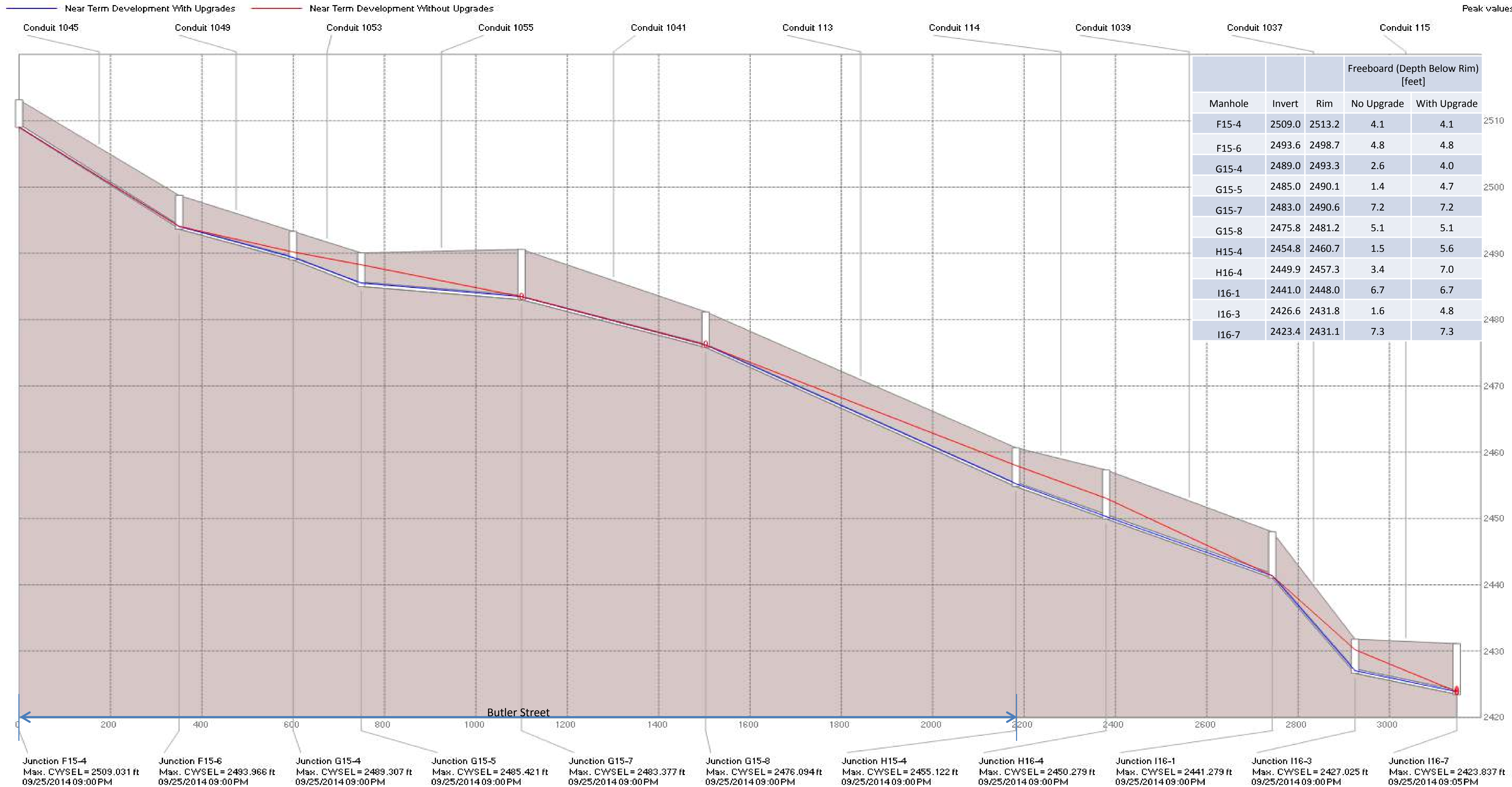
Figure No.
E-31

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 7



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
F15-4	2509.0	2513.2	4.1	4.1
F15-6	2493.6	2498.7	4.8	4.8
G15-4	2489.0	2493.3	2.6	4.0
G15-5	2485.0	2490.1	1.4	4.7
G15-7	2483.0	2490.6	7.2	7.2
G15-8	2475.8	2481.2	5.1	5.1
H15-4	2454.8	2460.7	1.5	5.6
H16-4	2449.9	2457.3	3.4	7.0
I16-1	2441.0	2448.0	6.7	6.7
I16-3	2426.6	2431.8	1.6	4.8
I16-7	2423.4	2431.1	7.3	7.3

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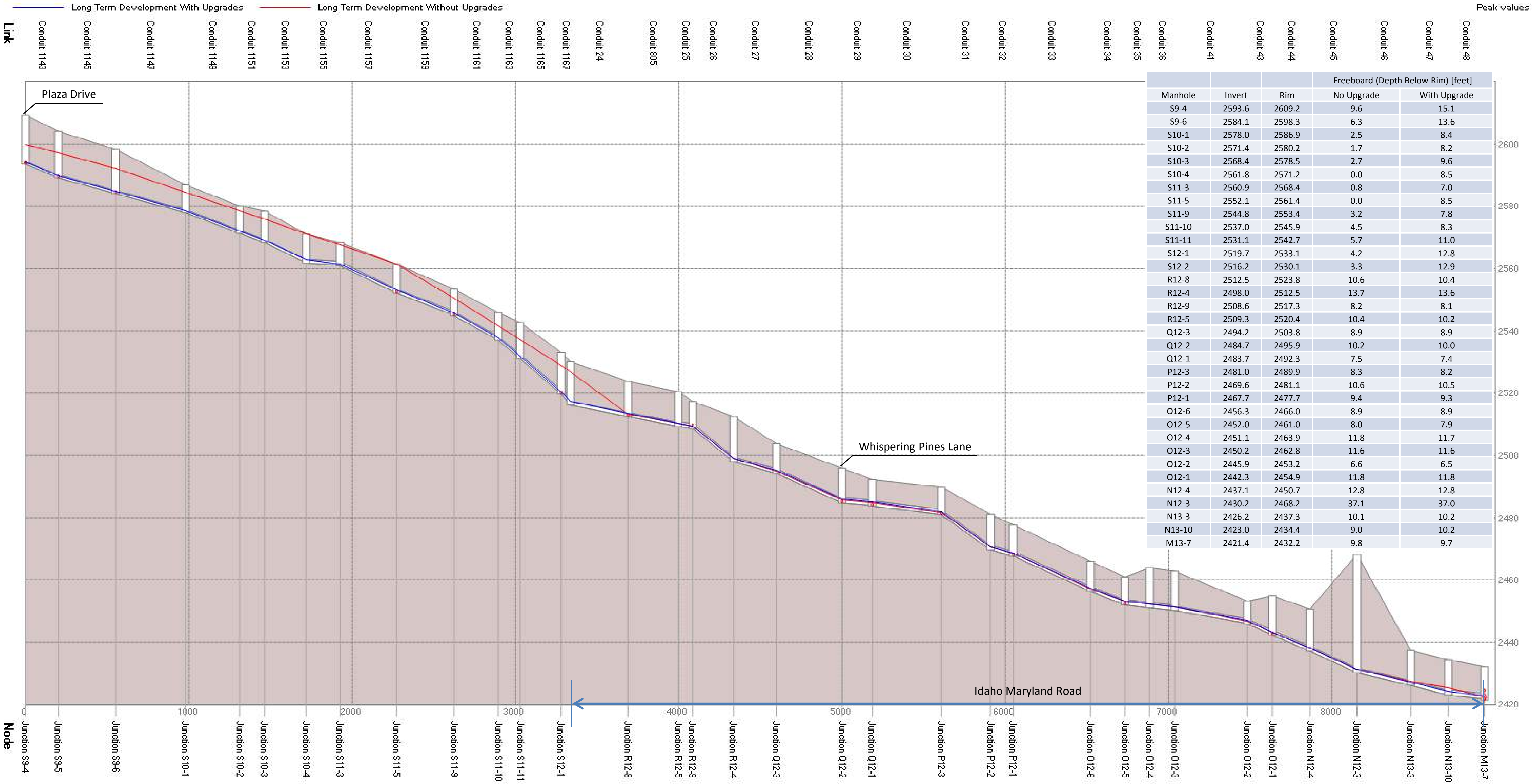
Figure No.
E-32

Title
Existing Build-out plus Near Term
Pre- and Post –improvement Results HGL Profile 8



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
S9-4	2593.6	2609.2	9.6	15.1
S9-6	2584.1	2598.3	6.3	13.6
S10-1	2578.0	2586.9	2.5	8.4
S10-2	2571.4	2580.2	1.7	8.2
S10-3	2568.4	2578.5	2.7	9.6
S10-4	2561.8	2571.2	0.0	8.5
S11-3	2560.9	2568.4	0.8	7.0
S11-5	2552.1	2561.4	0.0	8.5
S11-9	2544.8	2553.4	3.2	7.8
S11-10	2537.0	2545.9	4.5	8.3
S11-11	2531.1	2542.7	5.7	11.0
S12-1	2519.7	2533.1	4.2	12.8
S12-2	2516.2	2530.1	3.3	12.9
R12-8	2512.5	2523.8	10.6	10.4
R12-4	2498.0	2512.5	13.7	13.6
R12-9	2508.6	2517.3	8.2	8.1
R12-5	2509.3	2520.4	10.4	10.2
Q12-3	2494.2	2503.8	8.9	8.9
Q12-2	2484.7	2495.9	10.2	10.0
Q12-1	2483.7	2492.3	7.5	7.4
P12-3	2481.0	2489.9	8.3	8.2
P12-2	2469.6	2481.1	10.6	10.5
P12-1	2467.7	2477.7	9.4	9.3
O12-6	2456.3	2466.0	8.9	8.9
O12-5	2452.0	2461.0	8.0	7.9
O12-4	2451.1	2463.9	11.8	11.7
O12-3	2450.2	2462.8	11.6	11.6
O12-2	2445.9	2453.2	6.6	6.5
O12-1	2442.3	2454.9	11.8	11.8
N12-4	2437.1	2450.7	12.8	12.8
N12-3	2430.2	2468.2	37.1	37.0
N13-3	2426.2	2437.3	10.1	10.2
N13-10	2423.0	2434.4	9.0	10.2
M13-7	2421.4	2432.2	9.8	9.7

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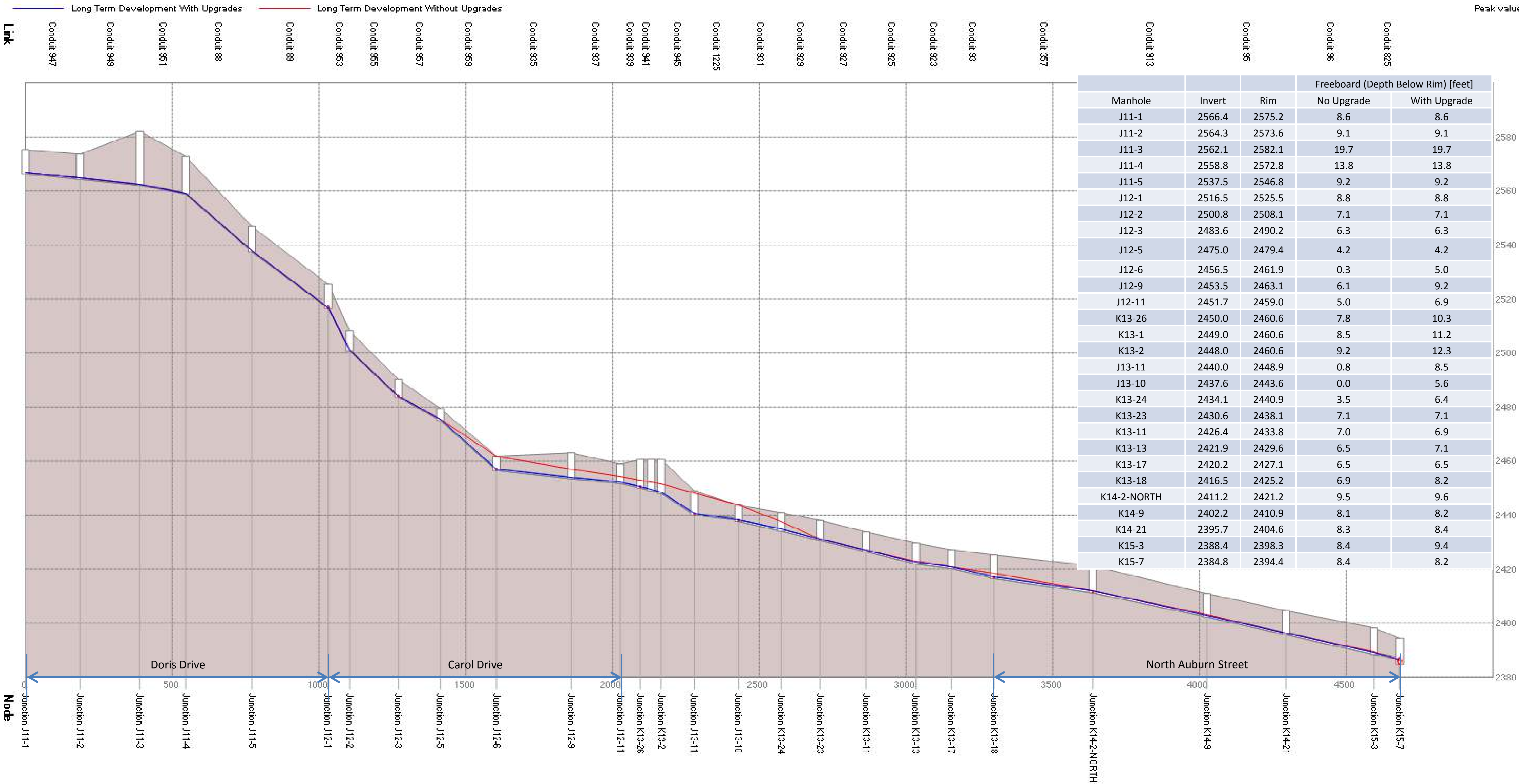
Figure No.
E-33

Title
Existing Build-out plus Both Near and Long Term Pre- and Post –improvement Results HGL Profile 1



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
J11-1	2566.4	2575.2	8.6	8.6
J11-2	2564.3	2573.6	9.1	9.1
J11-3	2562.1	2582.1	19.7	19.7
J11-4	2558.8	2572.8	13.8	13.8
J11-5	2537.5	2546.8	9.2	9.2
J12-1	2516.5	2525.5	8.8	8.8
J12-2	2500.8	2508.1	7.1	7.1
J12-3	2483.6	2490.2	6.3	6.3
J12-5	2475.0	2479.4	4.2	4.2
J12-6	2456.5	2461.9	0.3	5.0
J12-9	2453.5	2463.1	6.1	9.2
J12-11	2451.7	2459.0	5.0	6.9
K13-26	2450.0	2460.6	7.8	10.3
K13-1	2449.0	2460.6	8.5	11.2
K13-2	2448.0	2460.6	9.2	12.3
J13-11	2440.0	2448.9	0.8	8.5
J13-10	2437.6	2443.6	0.0	5.6
K13-24	2434.1	2440.9	3.5	6.4
K13-23	2430.6	2438.1	7.1	7.1
K13-11	2426.4	2433.8	7.0	6.9
K13-13	2421.9	2429.6	6.5	7.1
K13-17	2420.2	2427.1	6.5	6.5
K13-18	2416.5	2425.2	6.9	8.2
K14-2-NORTH	2411.2	2421.2	9.5	9.6
K14-9	2402.2	2410.9	8.1	8.2
K14-21	2395.7	2404.6	8.3	8.4
K15-3	2388.4	2398.3	8.4	9.4
K15-7	2384.8	2394.4	8.4	8.2

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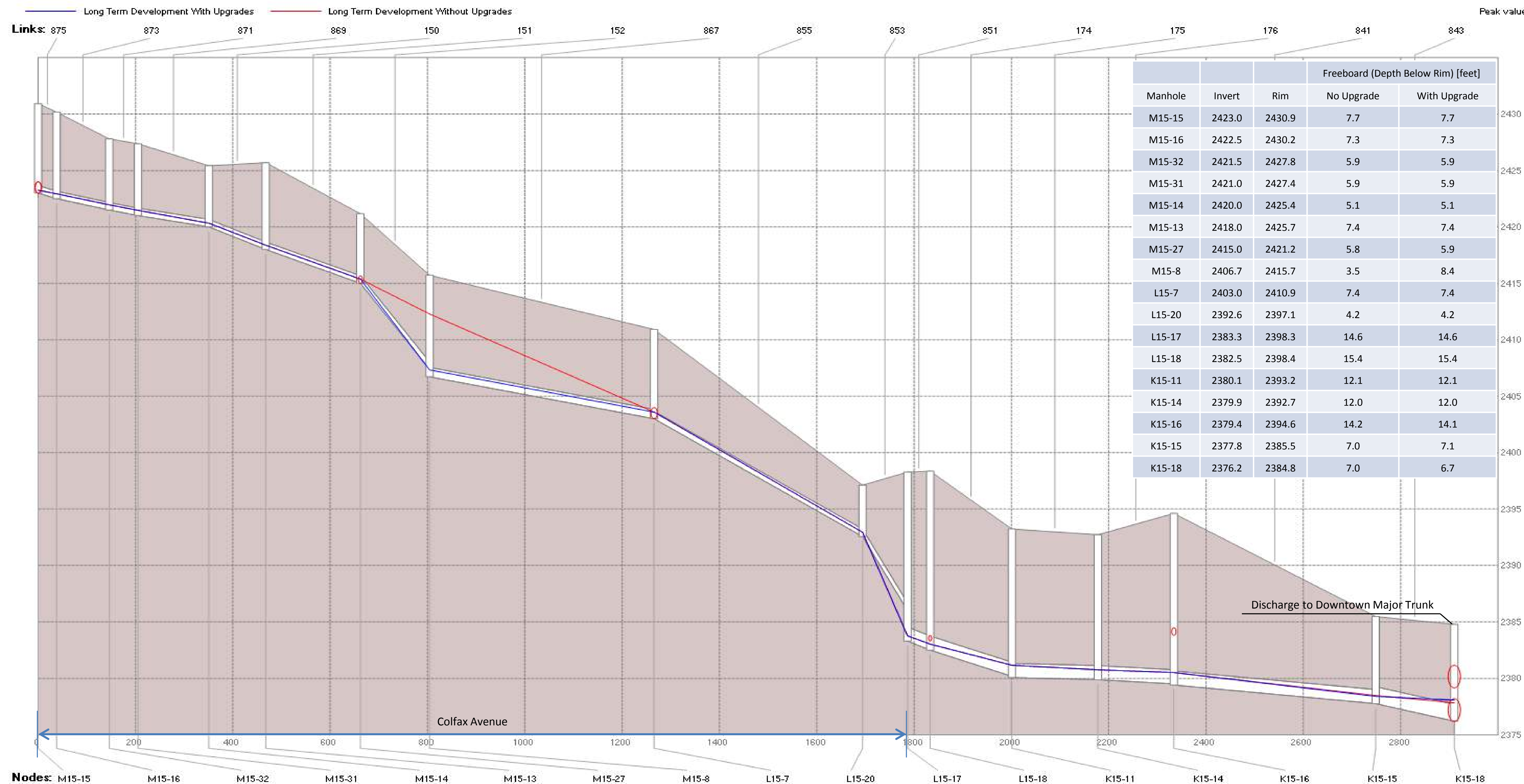
Figure No.
E-34

Title
Existing Build-out plus Both Near and Long Term
Pre- and Post –improvement Results HGL Profile 2



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
M15-15	2423.0	2430.9	7.7	7.7
M15-16	2422.5	2430.2	7.3	7.3
M15-32	2421.5	2427.8	5.9	5.9
M15-31	2421.0	2427.4	5.9	5.9
M15-14	2420.0	2425.4	5.1	5.1
M15-13	2418.0	2425.7	7.4	7.4
M15-27	2415.0	2421.2	5.8	5.9
M15-8	2406.7	2415.7	3.5	8.4
L15-7	2403.0	2410.9	7.4	7.4
L15-20	2392.6	2397.1	4.2	4.2
L15-17	2383.3	2398.3	14.6	14.6
L15-18	2382.5	2398.4	15.4	15.4
K15-11	2380.1	2393.2	12.1	12.1
K15-14	2379.9	2392.7	12.0	12.0
K15-16	2379.4	2394.6	14.2	14.1
K15-15	2377.8	2385.5	7.0	7.1
K15-18	2376.2	2384.8	7.0	6.7

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Figure No.
E-35

Title
Existing Build-out plus Both Near and Long Term Pre- and Post –improvement Results HGL Profile 3

Manhole	Invert	Rim	Freeboard (Depth Below Rim) [feet] No Upgrade	Freeboard (Depth Below Rim) [feet] With Upgrade
I13-1	2536.0	2548.5	12.3	12.3
I13-4	2518.0	2527.7	9.5	9.5
I13-5	2507.0	2518.9	11.5	11.6
I13-9	2502.0	2509.9	5.2	7.4
I13-14	2500.4	2510.7	10.0	10.0
I14-3	2486.7	2496.6	8.4	9.5
I14-6	2481.2	2490.1	6.2	8.6
I14-15	2481.0	2490.1	6.4	8.5
I14-9	2480.0	2485.5	5.2	5.2
I14-12	2465.1	2474.4	8.9	8.9
I15-3	2463.3	2469.6	5.9	5.9
H15-2	2461.2	2466.2	4.6	4.6
I15-10	2450.2	2458.3	7.7	7.6
I15-13	2447.1	2455.4	8.0	8.0
I15-20	2440.2	2444.5	4.0	4.0
I16-7	2423.4	2431.1	7.3	7.3
I16-9	2408.1	2412.1	3.6	3.6
I16-26	2397.2	2400.6	3.0	3.0
I16-20	2389.7	2392.7	2.7	2.7
I16-21	2380.9	2386.2	4.9	4.9
I16-27	2375.2	2379.4	3.8	3.9
I16-22	2362.2	2369.6	0.0	6.7
I17-12	2360.4	2365.5	3.6	4.6
I17-4	2355.8	2365.9	9.7	9.6
I17-6	2347.0	2364.3	15.3	16.9
I17-7	2345.0	2367.8	18.7	20.8

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Figure No.
E-36

Title
Existing Build-out plus Both Near and Long Term
Pre- and Post –improvement Results HGL Profile 4

			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K14-2-SOUTH	2385.2	2406.3	19.3	18.8
K15-6	2384.4	2404.8	18.3	18.1
K15-7	2384.8	2394.4	8.4	8.2
K15-13	2382.3	2395.6	12.1	11.9
K15-1	2381.6	2400.6	17.6	17.4
K15-12	2380.7	2387.2	5.3	5.1
K15-30	2379.6	2384.9	4.0	3.8
K15-18	2376.2	2384.8	7.0	6.7
J15-15	2374.0	2392.4	17.2	17.1
J15-16	2370.5	2389.8	18.1	18.0
J16-2	2367.5	2381.9	13.2	13.0
J16-11	2364.0	2381.6	16.4	16.3
J16-15	2362.2	2370.6	7.3	7.1
J16-16	2358.0	2369.3	9.9	10.0
J16-19	2354.5	2365.5	9.6	9.4
I17-3	2349.9	2366.6	15.2	15.2
I17-7	2345.0	2367.8	18.7	20.8
I18-1	2340.7	2354.5	12.6	12.5
I18-2	2338.0	2348.8	9.6	9.5
I18-3	2332.0	2348.3	14.6	14.4
I18-12	2330.3	2339.3	7.7	7.6

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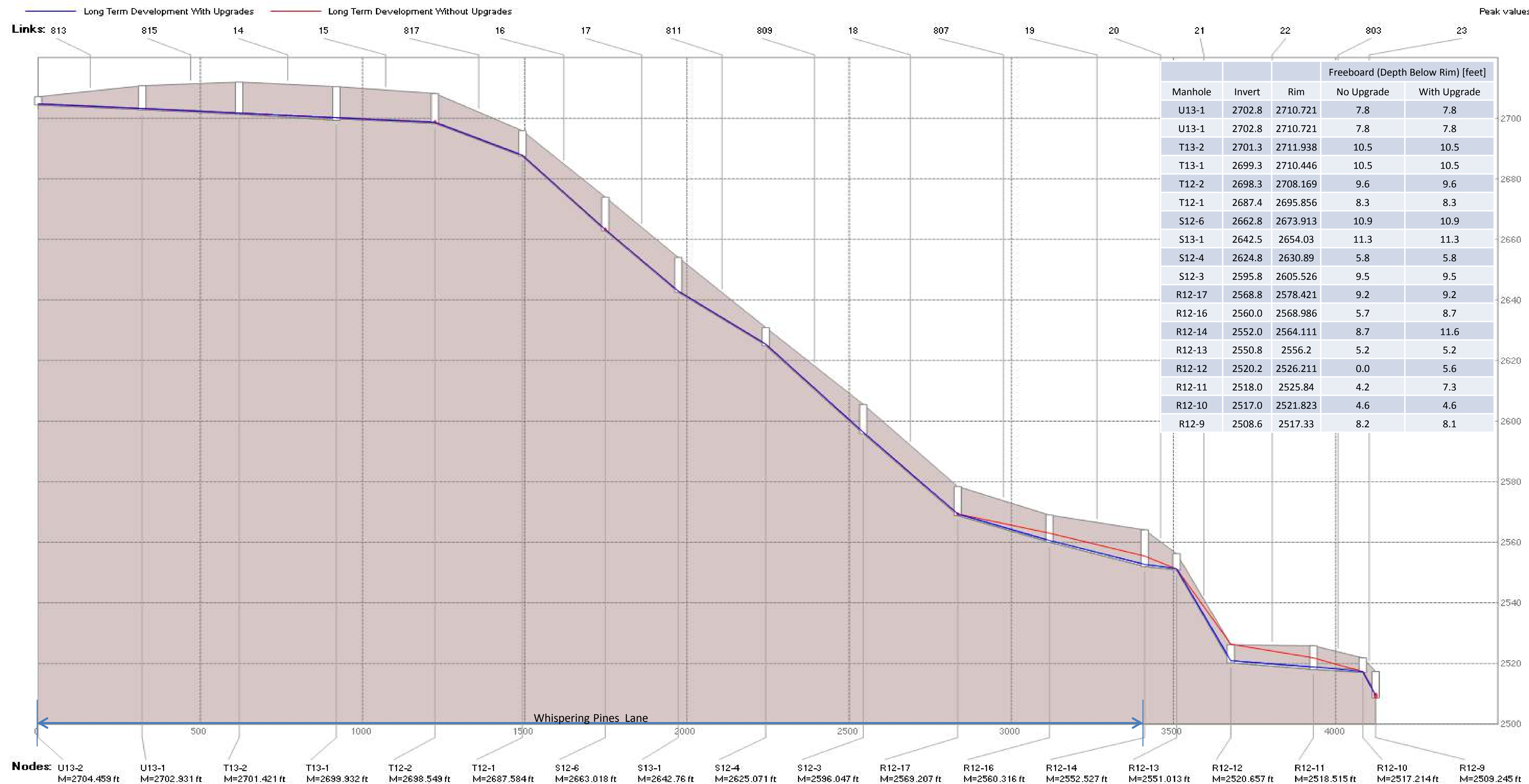
Figure No.
E-37
Title
Existing Build-out plus Both Near and Long Term
Pre- and Post –improvement Results HGL Profile 5



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
U13-1	2702.8	2710.721	7.8	7.8
U13-1	2702.8	2710.721	7.8	7.8
T13-2	2701.3	2711.938	10.5	10.5
T13-1	2699.3	2710.446	10.5	10.5
T12-2	2698.3	2708.169	9.6	9.6
T12-1	2687.4	2695.856	8.3	8.3
S12-6	2662.8	2673.913	10.9	10.9
S13-1	2642.5	2654.03	11.3	11.3
S12-4	2624.8	2630.89	5.8	5.8
S12-3	2595.8	2605.526	9.5	9.5
R12-17	2568.8	2578.421	9.2	9.2
R12-16	2560.0	2568.986	5.7	8.7
R12-14	2552.0	2564.111	8.7	11.6
R12-13	2550.8	2556.2	5.2	5.2
R12-12	2520.2	2526.211	0.0	5.6
R12-11	2518.0	2525.84	4.2	7.3
R12-10	2517.0	2521.823	4.6	4.6
R12-9	2508.6	2517.33	8.2	8.1

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Figure No.
E-38

Title
Existing Build-out plus Both Near and Long Term Pre- and Post –improvement Results HGL Profile 6

Manhole	Invert	Rim	Freeboard (Depth Below Rim) [feet] No Upgrade	Freeboard (Depth Below Rim) [feet] With Upgrade
K9-1	2665.0	2673.06	7.1	7.5
K10-1	2664.3	2668.47	3.6	3.6
K10-4	2663.0	2670.194	6.5	6.5
K10-5	2662.7	2670.222	6.8	6.8
K10-14	2662.0	2670.31	8.0	8.0
K10-15	2656.0	2663.995	7.7	7.7
L11-1	2646.7	2653.954	7.0	7.0
L11-2	2640.0	2646.245	6.0	6.0
L11-9	2629.5	2635.213	5.5	5.5
L11-10	2623.2	2631.488	8.1	8.1
L11-12	2617.7	2625.972	7.8	7.8
L11-5	2617.4	2623.197	5.4	5.4
L11-6	2616.0	2619.932	3.7	3.6
L11-8	2611.5	2615.067	3.2	3.2
M11-2	2609.0	2616.007	6.6	6.6
M11-3	2607.0	2611.282	3.9	3.9
M11-1	2592.4	2597.35	4.4	4.4
M10-6	2591.5	2596.926	5.0	5.0
M10-4	2590.1	2596.139	5.7	5.7
M10-5	2579.1	2584.659	5.3	5.3
M10-7	2551.0	2556.648	5.3	5.3
N11-1	2527.3	2539.357	11.4	11.3
N11-4	2526.4	2539.6	12.3	12.3
N11-5	2523.0	2530.537	7.1	7.1
M11-4	2514.0	2521.517	7.1	7.1
M11-7	2496.0	2503.392	7.0	7.0
M11-10	2483.0	2490.047	6.6	6.6
M12-2	2472.0	2477.608	5.0	5.0
M12-5	2468.0	2468	0.5	0.5
M12-15	2429.0	2436.522	6.1	6.9
M13-3	2428.0	2434.574	5.9	5.9
M13-7	2421.4	2432.18	9.8	9.7

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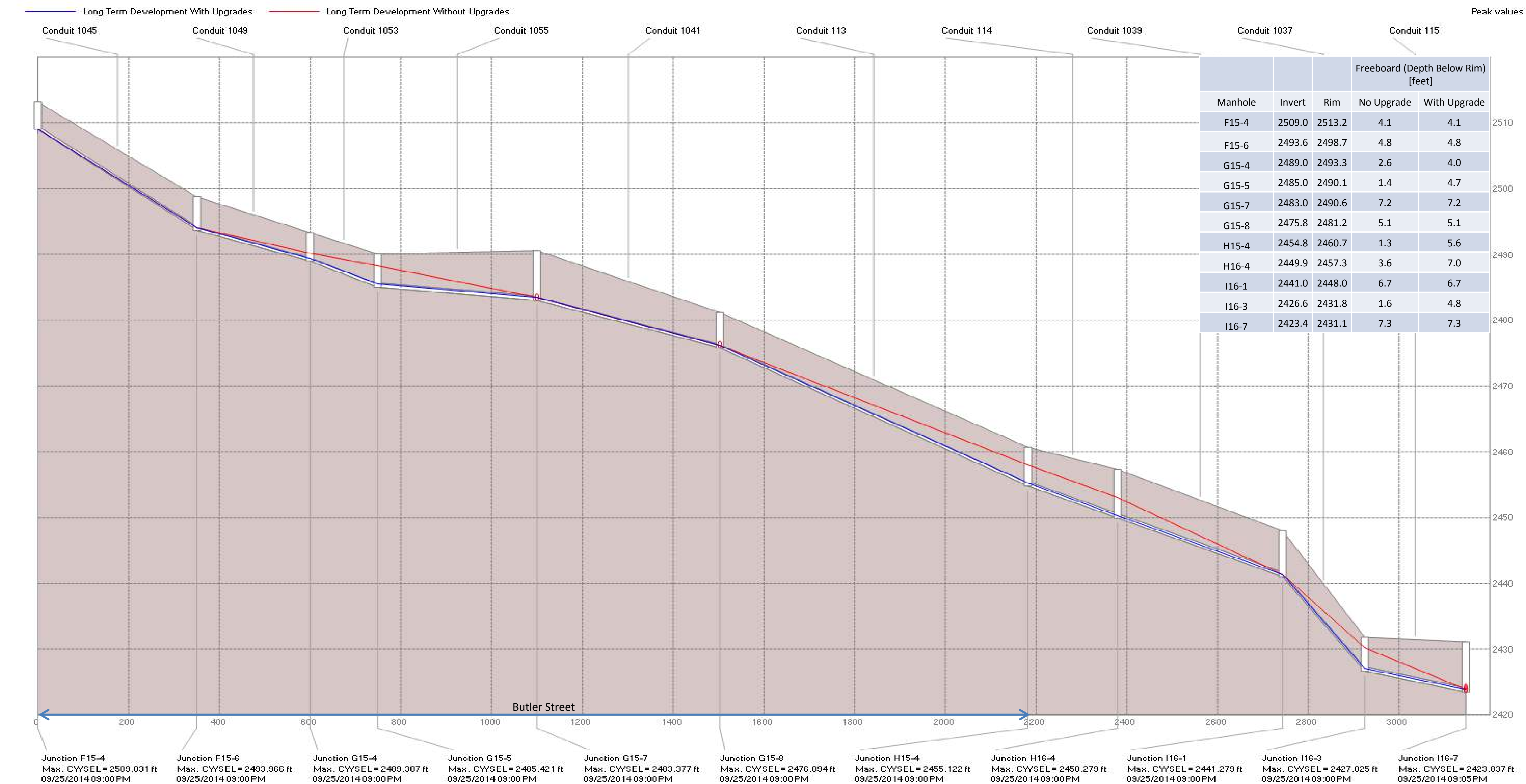
Figure No.
E-39

Title
Existing Build-out plus Both Near and Long Term
Pre- and Post –improvement Results HGL Profile 7



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
F15-4	2509.0	2513.2	4.1	4.1
F15-6	2493.6	2498.7	4.8	4.8
G15-4	2489.0	2493.3	2.6	4.0
G15-5	2485.0	2490.1	1.4	4.7
G15-7	2483.0	2490.6	7.2	7.2
G15-8	2475.8	2481.2	5.1	5.1
H15-4	2454.8	2460.7	1.3	5.6
H16-4	2449.9	2457.3	3.6	7.0
I16-1	2441.0	2448.0	6.7	6.7
I16-3	2426.6	2431.8	1.6	4.8
I16-7	2423.4	2431.1	7.3	7.3

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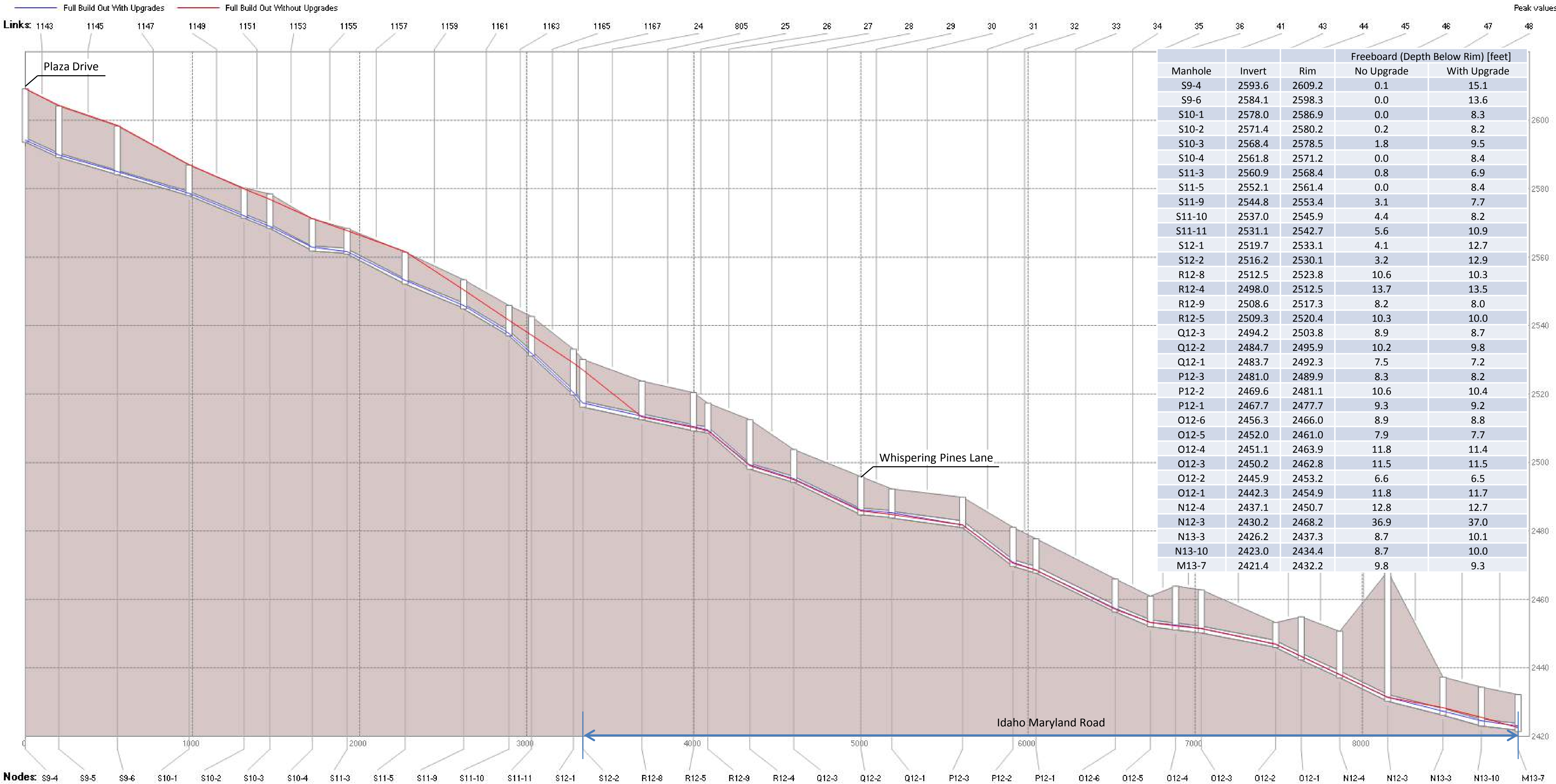
Figure No.
E-40

Title
Existing Build-out plus Both Near and Long Term Pre- and Post –improvement Results HGL Profile 8



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
S9-4	2593.6	2609.2	0.1	15.1
S9-6	2584.1	2598.3	0.0	13.6
S10-1	2578.0	2586.9	0.0	8.3
S10-2	2571.4	2580.2	0.2	8.2
S10-3	2568.4	2578.5	1.8	9.5
S10-4	2561.8	2571.2	0.0	8.4
S11-3	2560.9	2568.4	0.8	6.9
S11-5	2552.1	2561.4	0.0	8.4
S11-9	2544.8	2553.4	3.1	7.7
S11-10	2537.0	2545.9	4.4	8.2
S11-11	2531.1	2542.7	5.6	10.9
S12-1	2519.7	2533.1	4.1	12.7
S12-2	2516.2	2530.1	3.2	12.9
R12-8	2512.5	2523.8	10.6	10.3
R12-4	2498.0	2512.5	13.7	13.5
R12-9	2508.6	2517.3	8.2	8.0
R12-5	2509.3	2520.4	10.3	10.0
Q12-3	2494.2	2503.8	8.9	8.7
Q12-2	2484.7	2495.9	10.2	9.8
Q12-1	2483.7	2492.3	7.5	7.2
P12-3	2481.0	2489.9	8.3	8.2
P12-2	2469.6	2481.1	10.6	10.4
P12-1	2467.7	2477.7	9.3	9.2
O12-6	2456.3	2466.0	8.9	8.8
O12-5	2452.0	2461.0	7.9	7.7
O12-4	2451.1	2463.9	11.8	11.4
O12-3	2450.2	2462.8	11.5	11.5
O12-2	2445.9	2453.2	6.6	6.5
O12-1	2442.3	2454.9	11.8	11.7
N12-4	2437.1	2450.7	12.8	12.7
N12-3	2430.2	2468.2	36.9	37.0
N13-3	2426.2	2437.3	8.7	10.1
N13-10	2423.0	2434.4	8.7	10.0
M13-7	2421.4	2432.2	9.8	9.3

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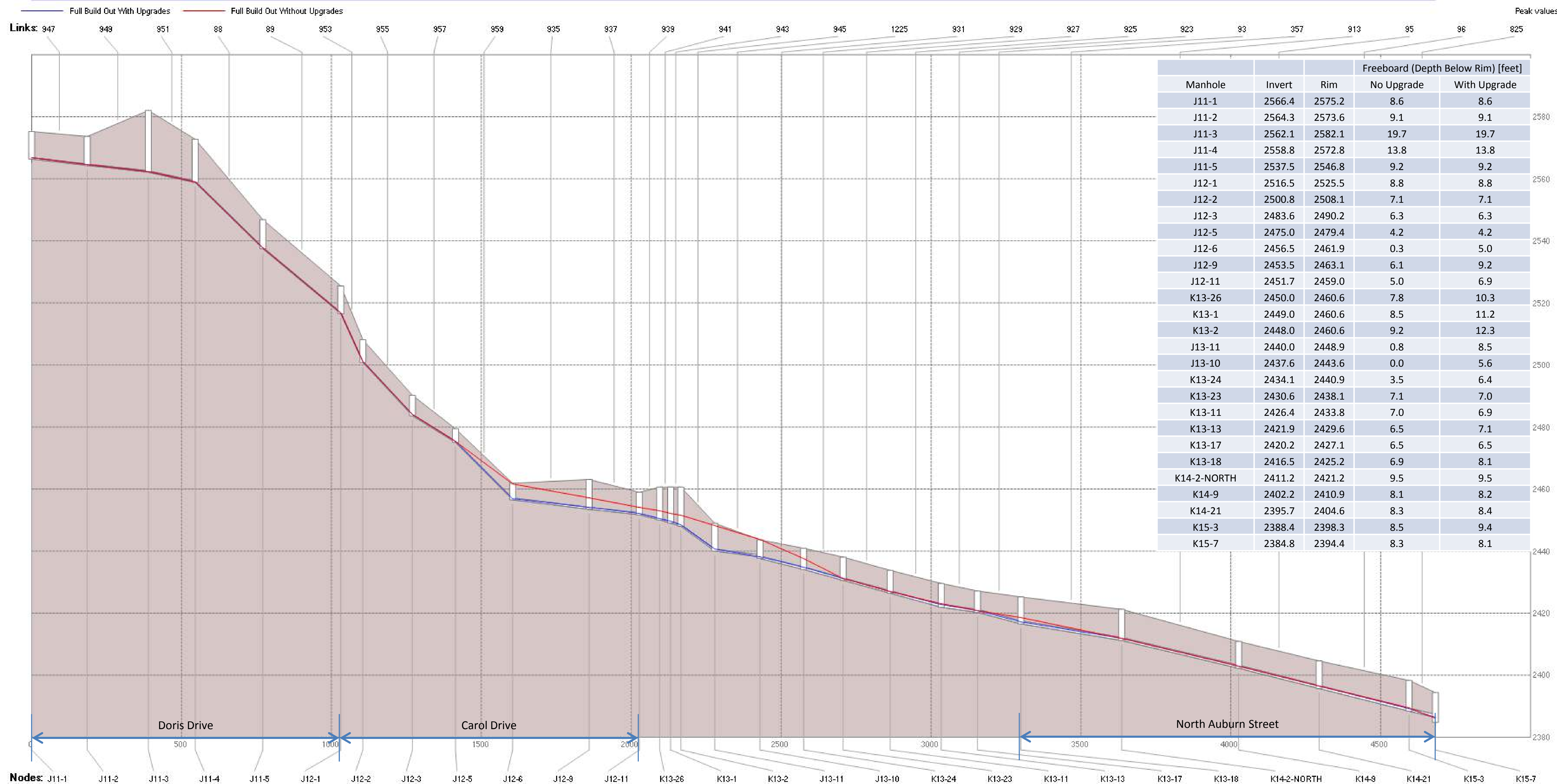
Figure No.
E-41

Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 1



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
J11-1	2566.4	2575.2	8.6	8.6
J11-2	2564.3	2573.6	9.1	9.1
J11-3	2562.1	2582.1	19.7	19.7
J11-4	2558.8	2572.8	13.8	13.8
J11-5	2537.5	2546.8	9.2	9.2
J12-1	2516.5	2525.5	8.8	8.8
J12-2	2500.8	2508.1	7.1	7.1
J12-3	2483.6	2490.2	6.3	6.3
J12-5	2475.0	2479.4	4.2	4.2
J12-6	2456.5	2461.9	0.3	5.0
J12-9	2453.5	2463.1	6.1	9.2
J12-11	2451.7	2459.0	5.0	6.9
K13-26	2450.0	2460.6	7.8	10.3
K13-1	2449.0	2460.6	8.5	11.2
K13-2	2448.0	2460.6	9.2	12.3
J13-11	2440.0	2448.9	0.8	8.5
J13-10	2437.6	2443.6	0.0	5.6
K13-24	2434.1	2440.9	3.5	6.4
K13-23	2430.6	2438.1	7.1	7.0
K13-11	2426.4	2433.8	7.0	6.9
K13-13	2421.9	2429.6	6.5	7.1
K13-17	2420.2	2427.1	6.5	6.5
K13-18	2416.5	2425.2	6.9	8.1
K14-2-NORTH	2411.2	2421.2	9.5	9.5
K14-9	2402.2	2410.9	8.1	8.2
K14-21	2395.7	2404.6	8.3	8.4
K15-3	2388.4	2398.3	8.5	9.4
K15-7	2384.8	2394.4	8.3	8.1

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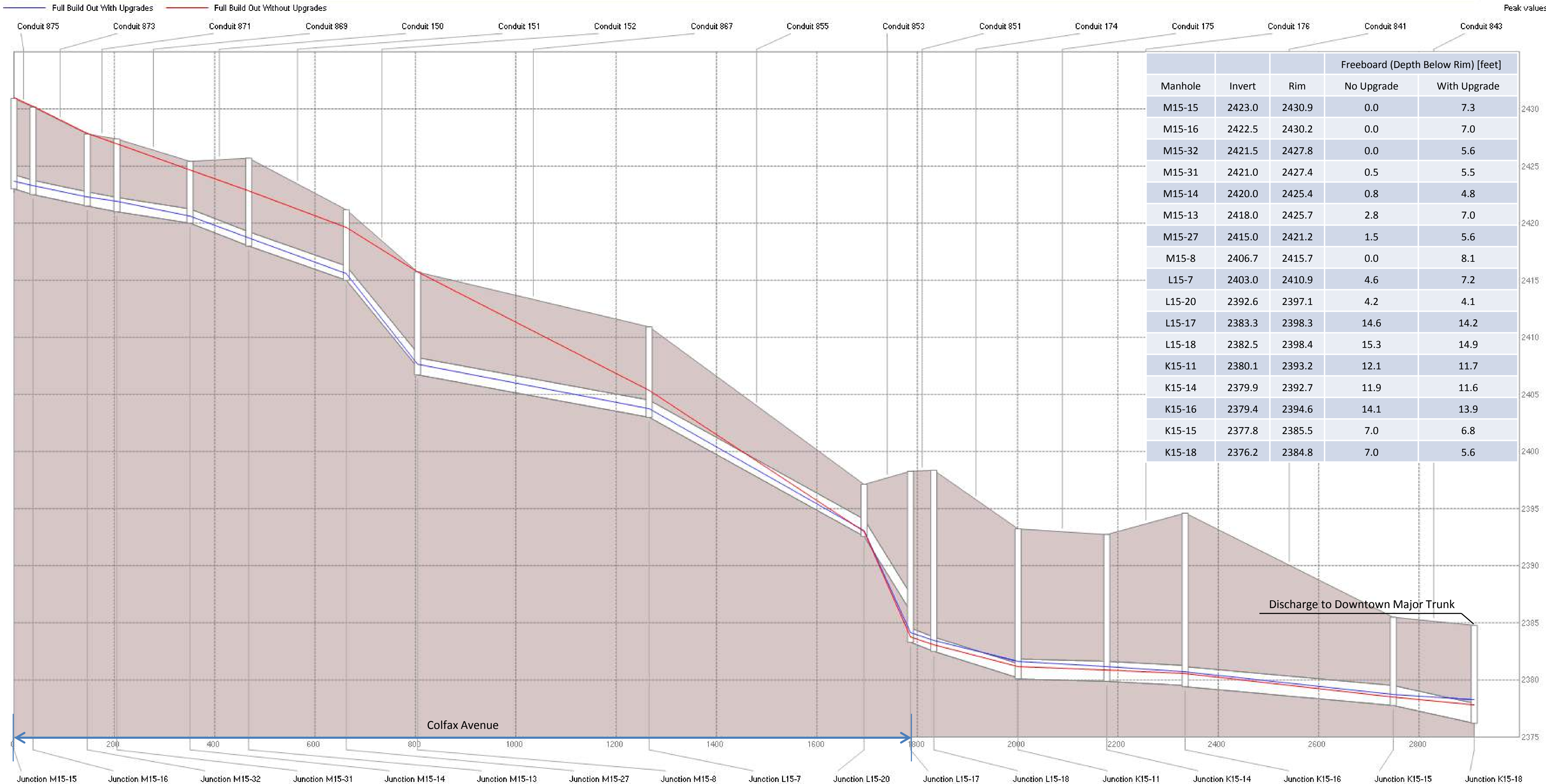
Figure No.
E-42

Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 2



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
M15-15	2423.0	2430.9	0.0	7.3
M15-16	2422.5	2430.2	0.0	7.0
M15-32	2421.5	2427.8	0.0	5.6
M15-31	2421.0	2427.4	0.5	5.5
M15-14	2420.0	2425.4	0.8	4.8
M15-13	2418.0	2425.7	2.8	7.0
M15-27	2415.0	2421.2	1.5	5.6
M15-8	2406.7	2415.7	0.0	8.1
L15-7	2403.0	2410.9	4.6	7.2
L15-20	2392.6	2397.1	4.2	4.1
L15-17	2383.3	2398.3	14.6	14.2
L15-18	2382.5	2398.4	15.3	14.9
K15-11	2380.1	2393.2	12.1	11.7
K15-14	2379.9	2392.7	11.9	11.6
K15-16	2379.4	2394.6	14.1	13.9
K15-15	2377.8	2385.5	7.0	6.8
K15-18	2376.2	2384.8	7.0	5.6

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Figure No.
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Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 3

			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
I13-1	2536.0	2548.5	12.2	12.2
I13-4	2518.0	2527.7	2.6	9.4
I13-5	2507.0	2518.9	0.0	11.5
I13-9	2502.0	2509.9	0.0	7.3
I13-14	2500.4	2510.7	6.8	9.9
I14-3	2486.7	2496.6	5.0	9.4
I14-6	2481.2	2490.1	5.8	8.6
I14-15	2481.0	2490.1	6.2	8.4
I14-9	2480.0	2485.5	5.2	5.2
I14-12	2465.1	2474.4	8.9	8.8
I15-3	2463.3	2469.6	5.8	5.8
H15-2	2461.2	2466.2	4.6	4.6
I15-10	2450.2	2458.3	7.6	7.6
I15-13	2447.1	2455.4	8.0	8.0
I15-20	2440.2	2444.5	4.0	3.9
I16-7	2423.4	2431.1	7.2	7.0
I16-9	2408.1	2412.1	3.6	3.3
I16-26	2397.2	2400.6	3.0	2.7
I16-20	2389.7	2392.7	2.6	2.5
I16-21	2380.9	2386.2	4.9	4.7
I16-27	2375.2	2379.4	0.7	3.6
I16-22	2362.2	2369.6	0.0	6.2
I17-12	2360.4	2365.5	0.0	4.4
I17-4	2355.8	2365.9	2.5	9.5
I17-6	2347.0	2364.3	3.1	16.6
I17-7	2345.0	2367.8	6.6	20.7

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Figure No.
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Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 4

			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K14-2-SOUTH	2385.2	2406.3	19.2	18.7
K15-6	2384.4	2404.8	18.2	19.0
K15-7	2384.8	2394.4	8.3	7.7
K15-13	2382.3	2395.6	12.0	11.8
K15-1	2381.6	2400.6	17.6	17.3
K15-12	2380.7	2387.2	5.2	4.5
K15-30	2379.6	2384.9	3.9	3.4
K15-18	2376.2	2384.8	6.9	5.5
J15-15	2374.0	2392.4	17.2	16.8
J15-16	2370.5	2389.8	18.0	17.7
J16-2	2367.5	2381.9	13.1	12.9
J16-11	2364.0	2381.6	16.4	16.1
J16-15	2362.2	2370.6	7.2	7.0
J16-16	2358.0	2369.3	7.8	9.8
J16-19	2354.5	2365.5	7.6	9.2
I17-3	2349.9	2366.6	12.2	15.1
I17-7	2345.0	2367.8	16.7	20.9
I18-1	2340.7	2354.5	12.6	12.0
I18-2	2338.0	2348.8	9.6	8.9
I18-3	2332.0	2348.3	14.6	14.1
I18-12	2330.3	2339.3	7.2	6.5

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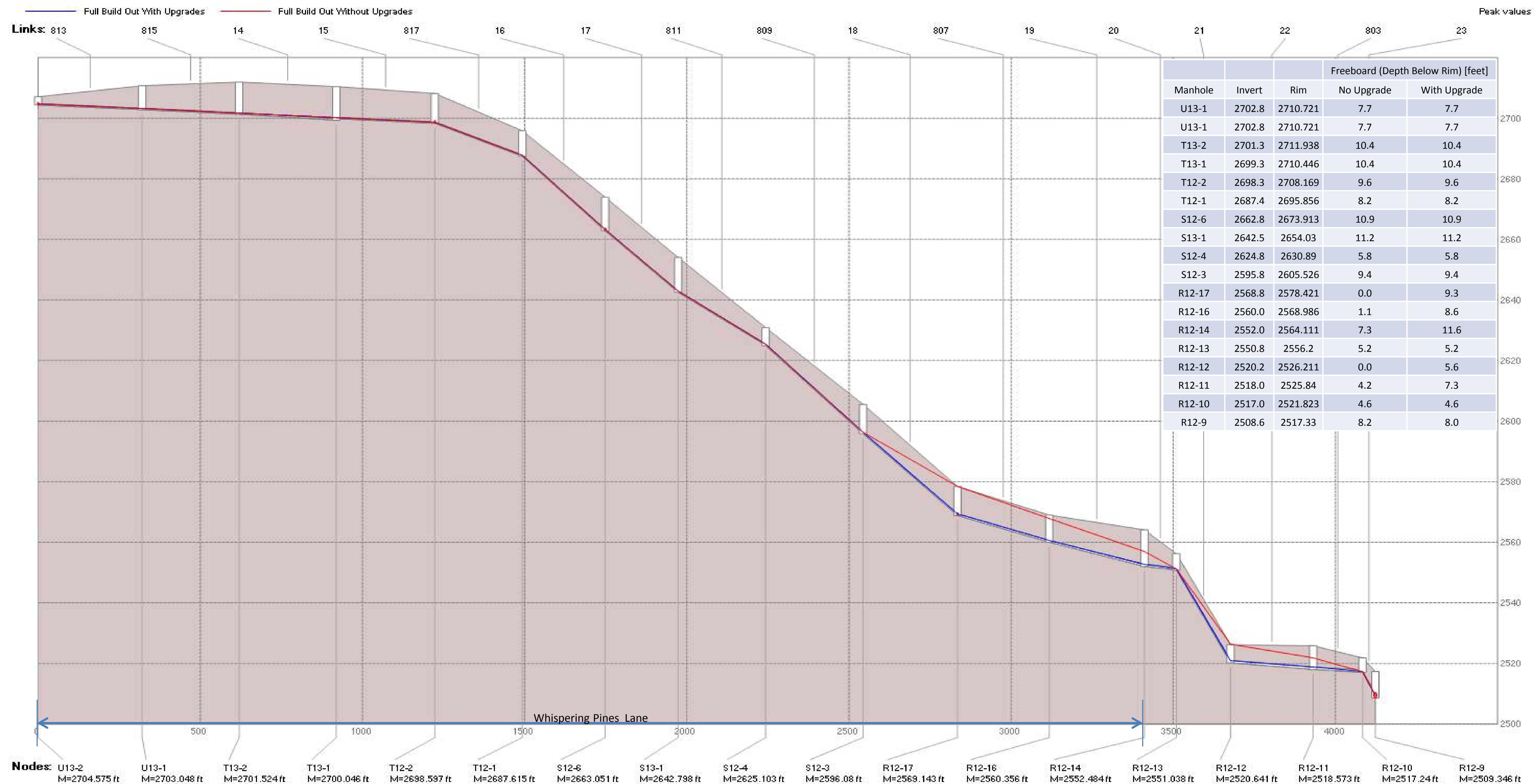
Figure No.
E-45
Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 5



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
U13-1	2702.8	2710.721	7.7	7.7
U13-1	2702.8	2710.721	7.7	7.7
T13-2	2701.3	2711.938	10.4	10.4
T13-1	2699.3	2710.446	10.4	10.4
T12-2	2698.3	2708.169	9.6	9.6
T12-1	2687.4	2695.856	8.2	8.2
S12-6	2662.8	2673.913	10.9	10.9
S13-1	2642.5	2654.03	11.2	11.2
S12-4	2624.8	2630.89	5.8	5.8
S12-3	2595.8	2605.526	9.4	9.4
R12-17	2568.8	2578.421	0.0	9.3
R12-16	2560.0	2568.986	1.1	8.6
R12-14	2552.0	2564.111	7.3	11.6
R12-13	2550.8	2556.2	5.2	5.2
R12-12	2520.2	2526.211	0.0	5.6
R12-11	2518.0	2525.84	4.2	7.3
R12-10	2517.0	2521.823	4.6	4.6
R12-9	2508.6	2517.33	8.2	8.0

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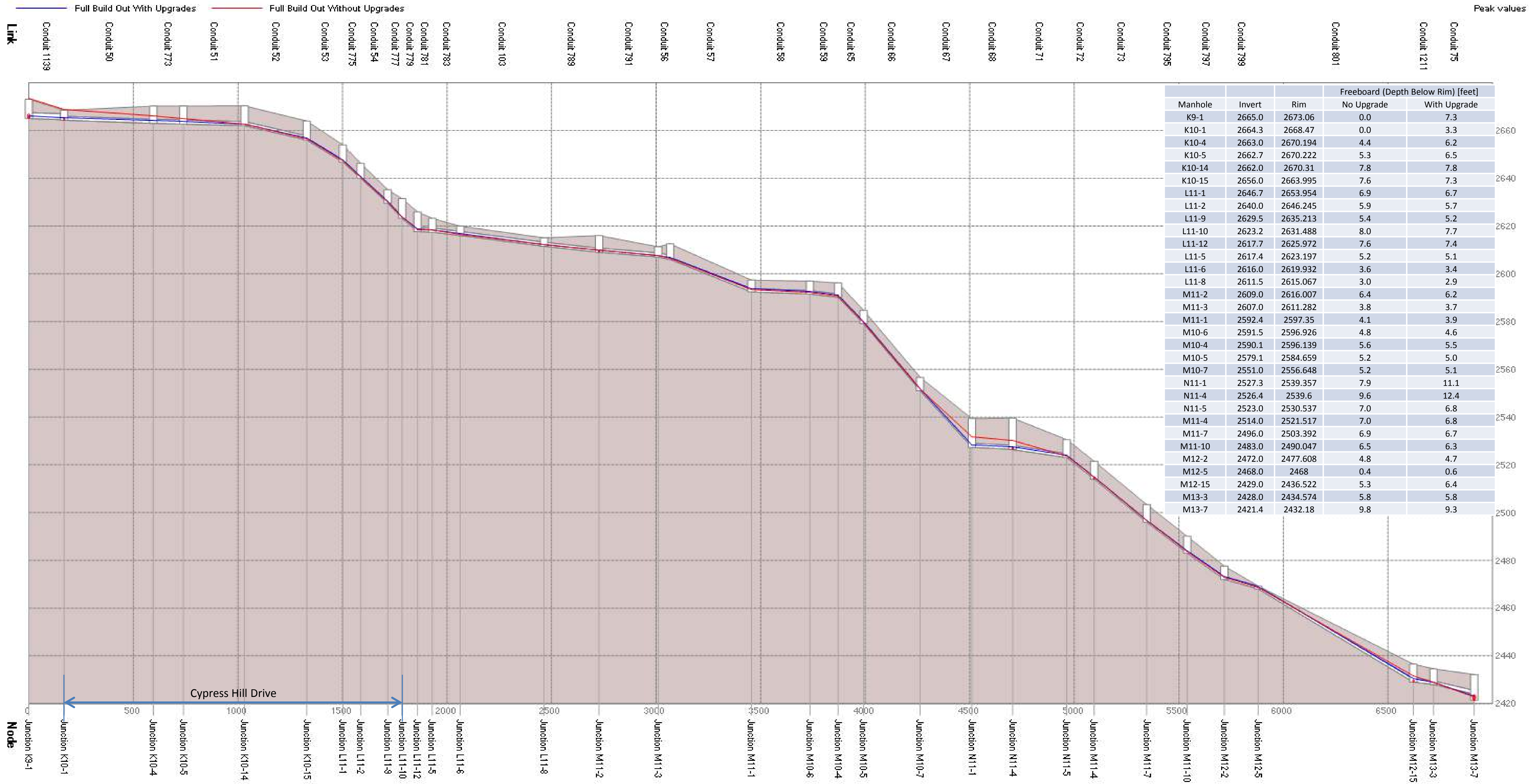
Figure No.
E-46

Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 6



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
K9-1	2665.0	2673.06	0.0	7.3
K10-1	2664.3	2668.47	0.0	3.3
K10-4	2663.0	2670.194	4.4	6.2
K10-5	2662.7	2670.222	5.3	6.5
K10-14	2662.0	2670.31	7.8	7.8
K10-15	2656.0	2663.995	7.6	7.3
L11-1	2646.7	2653.954	6.9	6.7
L11-2	2640.0	2646.245	5.9	5.7
L11-9	2629.5	2635.213	5.4	5.2
L11-10	2623.2	2631.488	8.0	7.7
L11-12	2617.7	2625.972	7.6	7.4
L11-5	2617.4	2623.197	5.2	5.1
L11-6	2616.0	2619.932	3.6	3.4
L11-8	2611.5	2615.067	3.0	2.9
M11-2	2609.0	2616.007	6.4	6.2
M11-3	2607.0	2611.282	3.8	3.7
M11-1	2592.4	2597.35	4.1	3.9
M10-6	2591.5	2596.926	4.8	4.6
M10-4	2590.1	2596.139	5.6	5.5
M10-5	2579.1	2584.659	5.2	5.0
M10-7	2551.0	2556.648	5.2	5.1
N11-1	2527.3	2539.357	7.9	11.1
N11-4	2526.4	2539.6	9.6	12.4
N11-5	2523.0	2530.537	7.0	6.8
M11-4	2514.0	2521.517	7.0	6.8
M11-7	2496.0	2503.392	6.9	6.7
M11-10	2483.0	2490.047	6.5	6.3
M12-2	2472.0	2477.608	4.8	4.7
M12-5	2468.0	2468	0.4	0.6
M12-15	2429.0	2436.522	5.3	6.4
M13-3	2428.0	2434.574	5.8	5.8
M13-7	2421.4	2432.18	9.8	9.3

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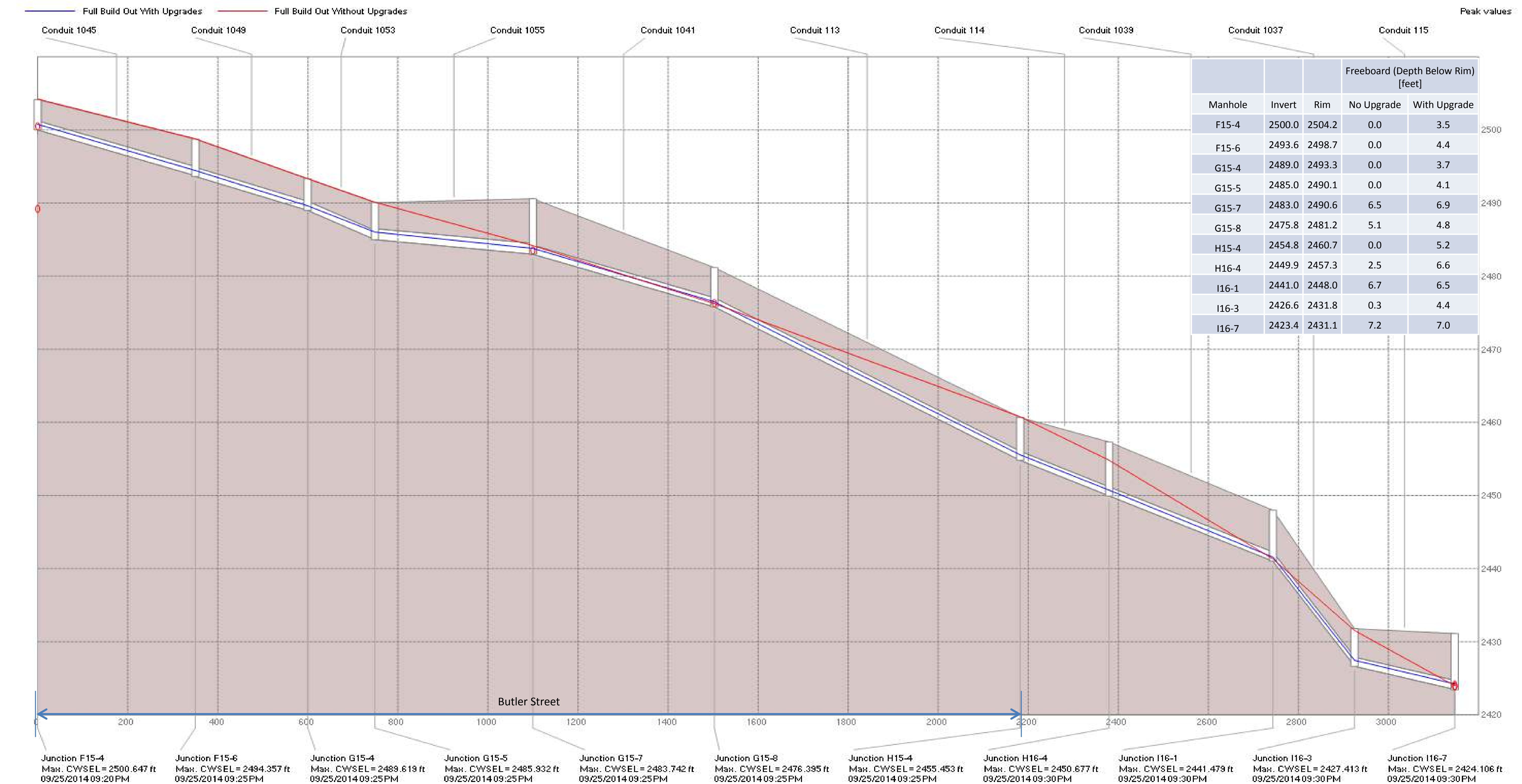
Figure No.
E-47

Title
Full Build-out Results – 1:10 Year Design Rainfall
HGL Profile 7



			Freeboard (Depth Below Rim) [feet]	
Manhole	Invert	Rim	No Upgrade	With Upgrade
F15-4	2500.0	2504.2	0.0	3.5
F15-6	2493.6	2498.7	0.0	4.4
G15-4	2489.0	2493.3	0.0	3.7
G15-5	2485.0	2490.1	0.0	4.1
G15-7	2483.0	2490.6	6.5	6.9
G15-8	2475.8	2481.2	5.1	4.8
H15-4	2454.8	2460.7	0.0	5.2
H16-4	2449.9	2457.3	2.5	6.6
I16-1	2441.0	2448.0	6.7	6.5
I16-3	2426.6	2431.8	0.3	4.4
I16-7	2423.4	2431.1	7.2	7.0

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Figure No.
E-48

Title
Full Build-out Pre- and Post –improvement Results
HGL Profile 8

Appendix F RECOMMENDED PIPE IMPROVEMENTS

F.1 RECOMMENDED PIPE IMPROVEMENTS FOR EXISTING SYSTEM

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
1	1155	10	15	S10-4	S11-3	207.9	Profile 1
2	1225	6	8	J13-11	J13-10	150.7	Profile 2
3	357	8	10	K13-18	K14-2-NORTH	336.5	Profile 2
4	825	8	10	K15-3	K15-7	87.8	Profile 2
5	913	8	10	K14-2-NORTH	K14-9	390.0	Profile 2
6	923	8	10	K13-13	K13-17	121.0	Profile 2
7	929	6	8	K13-24	K13-23	131.8	Profile 2
8	93	8	10	K13-17	K13-18	144.6	Profile 2
9	931	6	8	J13-10	K13-24	145.5	Profile 2
10	95	8	10	K14-9	K14-21	269.0	Profile 2
11	96	8	10	K14-21	K15-3	299.8	Profile 2
12	867	8	10	M15-8	L15-7	461.5	Profile 3
13	107	8	10	I14-15	I14-9	309.5	Profile 4
14	164	8	12	I16-22	I17-12	410.0	Profile 4
15	165	8	12	I17-12	I17-4	204.9	Profile 4
16	987	6	8	I13-9	I13-14	299.8	Profile 4
17	22	6	8	R12-12	R12-11	255.0	Profile 6
18	1211	12	15	M12-15	M13-3	96.1	Profile 7
19	1039	6	8	H16-4	I16-1	363.9	Profile 8
20	1055	6	8	G15-5	G15-7	350.8	Profile 8
21	114	6	8	H15-4	H16-4	197.5	Profile 8
22	115	6	8	I16-3	I16-7	222.7	Profile 8
23	387	6	8	M15-25	M15-27	483.7	-
24	849	6	8	L15-10	L15-9	167.4	-

F.2 RECOMMENDED PIPE IMPROVEMENTS FOR EXISTING BUILD-OUT SYSTEM

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
1	1155	10	15	S10-4	S11-3	207.9	Profile 1
2	1225	6	8	J13-11	J13-10	150.7	Profile 2
3	357	8	10	K13-18	K14-2-NORTH	336.5	Profile 2
4	825	8	10	K15-3	K15-7	87.8	Profile 2
5	913	8	10	K14-2-NORTH	K14-9	390.0	Profile 2
6	923	8	10	K13-13	K13-17	121.0	Profile 2
7	929	6	8	K13-24	K13-23	131.8	Profile 2
8	93	8	10	K13-17	K13-18	144.6	Profile 2
9	931	6	8	J13-10	K13-24	145.5	Profile 2
10	935	6	8	J12-6	J12-9	254.9	Profile 2
11	937	6	8	J12-9	J12-11	167.5	Profile 2
12	95	8	10	K14-9	K14-21	269.0	Profile 2
13	96	8	10	K14-21	K15-3	299.8	Profile 2
14	867	8	10	M15-8	L15-7	461.5	Profile 3
15	107	8	10	I14-15	I14-9	309.5	Profile 4
16	164	8	15	I16-22	I17-12	410.0	Profile 4
17	165	8	12	I17-12	I17-4	204.9	Profile 4
18	987	6	8	I13-9	I13-14	299.8	Profile 4
19	20	6	8	R12-14	R12-13	98.9	Profile 6
20	22	6	8	R12-12	R12-11	255.0	Profile 6
21	803	6	10	R12-11	R12-10	151.9	Profile 6
22	1211	12	15	M12-15	M13-3	96.1	Profile 7
23	1039	6	8	H16-4	I16-1	363.9	Profile 8
24	1055	6	8	G15-5	G15-7	350.8	Profile 8
25	114	6	8	H15-4	H16-4	197.5	Profile 8
26	115	6	8	I16-3	I16-7	222.7	Profile 8
27	387	6	8	M15-25	M15-27	483.7	-
28	849	6	8	L15-10	L15-9	167.4	-

F.3 RECOMMENDED PIPE IMPROVEMENTS FOR EXISTING BUILD-OUT SYSTEM PLUS NEAR-TERM DEVELOPMENT

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
1	1147	10	12	S9-6	S10-1	428.7	Profile 1
2	1155	10	15	S10-4	S11-3	207.9	Profile 1
3	24	10	12	S12-2	R12-8	353.2	Profile 1
4	48	15	18	N13-10	M13-7	220.3	Profile 1
5	1225	6	8	J13-11	J13-10	150.7	Profile 2
6	357	8	10	K13-18	K14-2-NORTH	336.5	Profile 2
7	825	8	10	K15-3	K15-7	87.8	Profile 2
8	913	8	10	K14-2-NORTH	K14-9	390.0	Profile 2
9	923	8	10	K13-13	K13-17	121.0	Profile 2
10	929	6	8	K13-24	K13-23	131.8	Profile 2
11	93	8	10	K13-17	K13-18	144.6	Profile 2
12	931	6	8	J13-10	K13-24	145.5	Profile 2
13	935	6	8	J12-6	J12-9	254.9	Profile 2
14	937	6	8	J12-9	J12-11	167.5	Profile 2
15	95	8	10	K14-9	K14-21	269.0	Profile 2
16	96	8	10	K14-21	K15-3	299.8	Profile 2
17	867	8	10	M15-8	L15-7	461.5	Profile 3
18	107	8	10	I14-15	I14-9	309.5	Profile 4
19	164	8	15	I16-22	I17-12	410.0	Profile 4
20	165	8	12	I17-12	I17-4	204.9	Profile 4
21	987	6	8	I13-9	I13-14	299.8	Profile 4
22	989	6	8	I14-6	I14-15	7.3	Profile 4
23	1293	18	24	I17-7	I18-1	651.5	Profile 5
24	182	21	24	J16-16	J16-19	245.9	Profile 5
25	20	6	8	R12-14	R12-13	98.9	Profile 6
26	22	6	8	R12-12	R12-11	255.0	Profile 6
27	803	6	10	R12-11	R12-10	151.9	Profile 6
28	1211	12	15	M12-15	M13-3	96.1	Profile 7

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
29	115	6	8	I16-3	I16-7	222.7	Profile 8
30	1039	6	8	H16-4	I16-1	363.9	Profile 8
31	114	6	8	H15-4	H16-4	197.5	Profile 8
32	1055	6	8	G15-5	G15-7	350.8	Profile 8
33	387	6	8	M15-25	M15-27	483.7	-
34	849	6	8	L15-10	L15-9	167.4	-

F.4 RECOMMENDED PIPE IMPROVEMENTS FOR EXISTING BUILD-OUT SYSTEM PLUS LONG-TERM DEVELOPMENT

No.	Pipe ID	Diameter (in)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
1	1145	10	12	S9-5	S9-6	350.2	Profile 1
2	1147	10	12	S9-6	S10-1	428.7	Profile 1
3	1149	10	12	S10-1	S10-2	329.9	Profile 1
4	1151	10	12	S10-2	S10-3	154.1	Profile 1
5	1153	10	12	S10-3	S10-4	254.5	Profile 1
6	1155	10	15	S10-4	S11-3	207.9	Profile 1
7	1157	10	15	S11-3	S11-5	347.1	Profile 1
8	1159	10	15	S11-5	S11-9	350.5	Profile 1
9	1161	10	15	S11-9	S11-10	271.0	Profile 1
10	1163	10	15	S11-10	S11-11	134.6	Profile 1
11	1165	10	15	S11-11	S12-1	250.6	Profile 1
12	1167	10	15	S12-1	S12-2	56.8	Profile 1
13	24	10	15	S12-2	R12-8	353.2	Profile 1
14	25	16	18	R12-5	R12-9	86.8	Profile 1
15	26	15	18	R12-9	R12-4	250.7	Profile 1
16	27	15	18	R12-4	Q12-3	262.9	Profile 1
17	28	15	18	Q12-3	Q12-2	401.5	Profile 1
18	29	18	21	Q12-2	Q12-1	185.0	Profile 1
19	30	18	21	Q12-1	P12-3	423.2	Profile 1
20	31	15	18	P12-3	P12-2	301.7	Profile 1

No.	Pipe ID	Diameter (in)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
21	32	15	18	P12-2	P12-1	138.4	Profile 1
22	33	15	18	P12-1	O12-6	473.8	Profile 1
23	34	15	18	O12-6	O12-5	210.1	Profile 1
24	35	18	21	O12-5	O12-4	150.8	Profile 1
25	36	18	21	O12-4	O12-3	154.1	Profile 1
26	41	15	18	O12-3	O12-2	444.9	Profile 1
27	43	15	18	O12-2	O12-1	152.9	Profile 1
28	44	15	18	O12-1	N12-4	230.5	Profile 1
29	45	15	18	N12-4	N12-3	287.3	Profile 1
30	46	15	18	N12-3	N13-3	330.3	Profile 1
31	47	15	18	N13-3	N13-10	229.6	Profile 1
32	48	15	21	N13-10	M13-7	220.3	Profile 1
33	1225	6	8	J13-11	J13-10	150.7	Profile 2
34	357	8	10	K13-18	K14-2-NORTH	336.5	Profile 2
35	825	8	10	K15-3	K15-7	87.8	Profile 2
36	913	8	10	K14-2-NORTH	K14-9	390.0	Profile 2
37	923	8	10	K13-13	K13-17	121.0	Profile 2
38	929	6	8	K13-24	K13-23	131.8	Profile 2
39	93	8	10	K13-17	K13-18	144.6	Profile 2
40	931	6	8	J13-10	K13-24	145.5	Profile 2
41	935	6	8	J12-6	J12-9	254.9	Profile 2
42	937	6	8	J12-9	J12-11	167.5	Profile 2
43	95	8	10	K14-9	K14-21	269.0	Profile 2
44	96	8	10	K14-21	K15-3	299.8	Profile 2
45	843	15	18	K15-15	K15-18	161.3	Profile 3
46	867	8	10	M15-8	L15-7	461.5	Profile 3
47	107	8	10	I14-15	I14-9	309.5	Profile 4
48	164	8	15	I16-22	I17-12	410.0	Profile 4
49	165	8	12	I17-12	I17-4	204.9	Profile 4
50	987	6	8	I13-9	I13-14	299.8	Profile 4
51	989	6	8	I14-6	I14-15	7.3	Profile 4
52	1293	18	24	I17-7	I18-1	651.5	Profile 5

No.	Pipe ID	Diameter (in)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
53	182	21	24	J16-16	J16-19	245.9	Profile 5
54	19	6	8	R12-16	R12-14	293.3	Profile 6
55	20	6	8	R12-14	R12-13	98.9	Profile 6
56	22	6	8	R12-12	R12-11	255.0	Profile 6
57	803	6	10	R12-11	R12-10	151.9	Profile 6
58	1139	8	10	K9-1	K10-1	168.7	Profile 7
59	1211	12	18	M12-15	M13-3	96.1	Profile 7
60	1039	6	8	H16-4	I16-1	363.9	Profile 8
61	1055	6	8	G15-5	G15-7	350.8	Profile 8
62	114	6	8	H15-4	H16-4	197.5	Profile 8
63	115	6	8	I16-3	I16-7	222.7	Profile 8
64	387	6	8	M15-25	M15-27	483.7	-
65	849	6	8	L15-10	L15-9	167.4	-

F.5 RECOMMENDED PIPE IMPROVEMENTS FOR FUTURE BUILD-OUT SYSTEM

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
1	1143	10	15	S9-4	S9-5	200.2	Profile 1
2	1145	10	15	S9-5	S9-6	350.2	Profile 1
3	1147	10	15	S9-6	S10-1	428.7	Profile 1
4	1149	10	15	S10-1	S10-2	329.9	Profile 1
5	1151	10	15	S10-2	S10-3	154.1	Profile 1
6	1153	10	15	S10-3	S10-4	254.5	Profile 1
7	1155	10	18	S10-4	S11-3	207.9	Profile 1
8	1157	10	18	S11-3	S11-5	347.1	Profile 1
9	1159	10	18	S11-5	S11-9	350.5	Profile 1
10	1161	10	18	S11-9	S11-10	271.0	Profile 1
11	1163	10	18	S11-10	S11-11	134.6	Profile 1
12	1165	10	18	S11-11	S12-1	250.6	Profile 1
13	1167	10	18	S12-1	S12-2	56.8	Profile 1

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
14	24	10	21	S12-2	R12-8	353.2	Profile 1
15	25	16	21	R12-5	R12-9	86.8	Profile 1
16	26	15	21	R12-9	R12-4	250.7	Profile 1
17	27	15	21	R12-4	Q12-3	262.9	Profile 1
18	28	15	21	Q12-3	Q12-2	401.5	Profile 1
19	29	18	24	Q12-2	Q12-1	185.0	Profile 1
20	30	18	24	Q12-1	P12-3	423.2	Profile 1
21	31	15	24	P12-3	P12-2	301.7	Profile 1
22	32	15	24	P12-2	P12-1	138.4	Profile 1
23	33	15	24	P12-1	O12-6	473.8	Profile 1
24	34	15	24	O12-6	O12-5	210.1	Profile 1
25	35	18	24	O12-5	O12-4	150.8	Profile 1
26	36	18	24	O12-4	O12-3	154.1	Profile 1
27	41	15	24	O12-3	O12-2	444.9	Profile 1
28	43	15	24	O12-2	O12-1	152.9	Profile 1
29	44	15	24	O12-1	N12-4	230.5	Profile 1
30	45	15	24	N12-4	N12-3	287.3	Profile 1
31	46	15	24	N12-3	N13-3	330.3	Profile 1
32	47	15	24	N13-3	N13-10	229.6	Profile 1
33	48	15	24	N13-10	M13-7	220.3	Profile 1
34	805	16	21	R12-8	R12-5	308.3	Profile 1
35	1225	6	8	J13-11	J13-10	150.7	Profile 2
36	357	8	12	K13-18	K14-2-NORTH	336.5	Profile 2
37	825	8	12	K15-3	K15-7	87.8	Profile 2
38	913	8	12	K14-2-NORTH	K14-9	390.0	Profile 2
39	923	8	12	K13-13	K13-17	121.0	Profile 2
40	925	8	10	K13-11	K13-13	170.3	Profile 2
41	927	8	10	K13-23	K13-11	157.0	Profile 2
42	929	6	10	K13-24	K13-23	131.8	Profile 2
43	93	8	12	K13-17	K13-18	144.6	Profile 2
44	931	6	10	J13-10	K13-24	145.5	Profile 2

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
45	935	6	8	J12-6	J12-9	254.9	Profile 2
46	937	6	8	J12-9	J12-11	167.5	Profile 2
47	95	8	12	K14-9	K14-21	269.0	Profile 2
48	96	8	12	K14-21	K15-3	299.8	Profile 2
49	150	8	15	M15-14	M15-13	117.0	Profile 3
50	151	8	15	M15-13	M15-27	194.6	Profile 3
51	152	8	15	M15-27	M15-8	142.0	Profile 3
52	175	15	21	K15-11	K15-14	176.7	Profile 3
53	176	15	21	K15-14	K15-16	156.1	Profile 3
54	841	15	21	K15-16	K15-15	415.0	Profile 3
55	843	15	21	K15-15	K15-18	161.3	Profile 3
56	853	8	18	L15-20	L15-17	92.5	Profile 3
57	855	8	18	L15-7	L15-20	428.4	Profile 3
58	867	8	18	M15-8	L15-7	461.5	Profile 3
59	869	8	15	M15-31	M15-14	145.6	Profile 3
60	871	8	15	M15-32	M15-31	59.0	Profile 3
61	873	8	15	M15-16	M15-32	108.2	Profile 3
62	875	8	15	M15-15	M15-16	37.9	Profile 3
63	1001	8	10	I15-10	I15-13	200.4	Profile 4
64	105	6	8	I13-14	I14-3	576.6	Profile 4
65	106	6	8	I14-3	I14-6	297.8	Profile 4
66	107	8	10	I14-15	I14-9	309.5	Profile 4
67	110	8	10	H15-2	I15-10	410.7	Profile 4
68	161	8	12	I16-20	I16-21	64.3	Profile 4
69	162	8	12	I16-21	I16-27	52.8	Profile 4
70	163	8	12	I16-27	I16-22	181.9	Profile 4
71	164	8	21	I16-22	I17-12	410.0	Profile 4
72	165	8	21	I17-12	I17-4	204.9	Profile 4
73	172	10	21	I17-4	I17-6	166.9	Profile 4
74	985	6	8	I13-5	I13-9	294.6	Profile 4
75	987	6	10	I13-9	I13-14	299.8	Profile 4
76	989	6	10	I14-6	I14-15	7.3	Profile 4

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
77	991	8	12	I16-7	I16-9	259.2	Profile 4
78	993	8	12	I16-9	I16-26	180.4	Profile 4
79	995	8	12	I16-26	I16-20	110.7	Profile 4
80	179	24	30	J16-11	J16-15	126.5	Profile 5
81	182	21	30	J16-16	J16-19	245.9	Profile 5
82	183	24	30	J16-19	I17-3	421.0	Profile 5
83	184	24	30	I17-3	I17-7	408.4	Profile 5
84	189	30	36	I18-3	I18-12	310.2	Profile 5
85	827	24	30	K15-7	K15-13	255.0	Profile 5
86	831	24	30	K15-13	K15-1	80.8	Profile 5
87	833	24	30	K15-1	K15-12	126.5	Profile 5
88	835	24	30	K15-12	K15-30	136.7	Profile 5
89	837	24	30	K15-30	K15-18	59.8	Profile 5
90	839	24	30	K15-18	J15-15	234.0	Profile 5
91	893	24	30	J16-15	J16-16	262.0	Profile 5
92	895	24	30	J16-2	J16-11	233.0	Profile 5
93	897	24	30	J15-16	J16-2	209.5	Profile 5
94	899	24	30	J15-15	J15-16	209.6	Profile 5
95	909	27	30	K14-2-SOUTH	K15-6	357.8	Profile 5
96	911	27	30	K15-6	K15-7	52.6	Profile 5
97	1293	18	30	I17-7	I18-1	646.0	Profile 5
98	19	6	8	R12-16	R12-14	293.3	Profile 6
99	20	6	10	R12-14	R12-13	98.9	Profile 6
100	22	6	10	R12-12	R12-11	255.0	Profile 6
101	803	6	10	R12-11	R12-10	151.9	Profile 6
102	807	6	8	R12-17	R12-16	282.9	Profile 6
103	1139	8	30	K9-1	K10-1	168.7	Profile 7
104	1211	12	18	M12-15	M13-3	96.1	Profile 7
105	50	10	21	K10-1	K10-4	426.4	Profile 7
106	51	10	21	K10-5	K10-14	292.6	Profile 7
107	52	10	21	K10-14	K10-15	297.2	Profile 7

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
108	56	12	21	M11-3	M11-12	57.1	Profile 7
109	58	12	18	M11-1	M10-6	280.0	Profile 7
110	59	12	18	M10-6	M10-4	134.5	Profile 7
111	68	12	21	N11-1	N11-4	194.8	Profile 7
112	71	12	21	N11-4	N11-5	258.7	Profile 7
113	75	12	18	M13-3	M13-7	194.8	Profile 7
114	773	10	21	K10-4	K10-5	143.8	Profile 7
115	781	12	21	L11-12	L11-5	70.6	Profile 7
116	783	12	21	L11-5	L11-6	134.4	Profile 7
117	789	12	21	L11-8	M11-2	262.9	Profile 7
118	791	12	21	M11-2	M11-3	283.0	Profile 7
119	799	12	15	M12-2	M12-5	162.0	Profile 7
120	801	12	15	M12-5	M12-15	742.	Profile 7
121	113	6	15	G15-8	H15-4	679.2	Profile 8
122	114	6	15	H15-4	H16-4	197.5	Profile 8
123	115	6	15	I16-3	I16-7	222.7	Profile 8
124	1037	6	15	I16-1	I16-3	181.0	Profile 8
125	1039	6	15	H16-4	I16-1	363.9	Profile 8
126	1041	6	15	G15-7	G15-8	403.0	Profile 8
127	1045	6	15	F15-4	F15-6	350.2	Profile 8
128	1049	6	15	F15-6	G15-4	249.3	Profile 8
129	1053	6	15	G15-4	G15-5	149.5	Profile 8
130	1055	6	18	G15-5	G15-7	350.8	Profile 8
131	2	6	12	Future_LS10	Buildout_28	1774.9	-
132	49	6	27	H6-2	9012	50.0	-
133	80	24	27	L13-14	L14-4	40.6	-
135	81	24	27	L14-4	L14-6	324.3	-
136	82	24	27	L14-6	L14-7	261.8	-
137	83	24	27	L14-7	L14-8	228.0	-
138	166	8	10	H17-9	H18-2	175.7	-
139	170	10	12	I17-9	I17-8	69.9	-

No.	Pipe ID	Diameter (inches)		From MH	To MH	Length (feet)	Profile No.
		Pre-development	Post-development				
140	171	10	12	I17-8	I17-4	347.4	-
141	387	6	15	M15-25	M15-27	483.7	-
142	103	12	21	L11-6	L11-8	400.3	-
143	793	6	8	L9-7	L9-8	117.3	-
144	849	6	8	L15-10	L15-9	167.4	-
145	877	6	10	M16-13	M15-25	413.4	-
146	879	6	10	M16-5	M16-13	210.8	-
147	903	24	30	L14-10	L14-11	76.8	-
148	905	27	30	L14-11	K14-2-SOUTH	320.1	-
149	933	6	8	J13-5	J13-10	107.2	-
150	961	6	8	J13-4	J13-5	82.5	-
151	963	6	8	J13-3	J13-4	183.9	-
152	1007	8	10	H18-2	H17-10	289.0	-
153	1009	8	10	I17-11	I17-9	166.7	-
154	1023	6	8	H17-2	H17-6	334.6	-
155	1029	6	8	H17-1	H17-2	25.4	-
156	1181	12	15	S8-4	S8-6	210.7	-
157	1183	12	15	S8-6	S8-7	156.7	-
158	1185	12	15	S8-7	S9-2	365.3	-
159	1207	8	10	R9-6	R9-5	23.8	-

Appendix G STORMBLOX™ PROPOSAL

Akram Botrous, Ph.D., PE
Stantec
3875 Atherton Road
Rocklin, California
95765-3716

August 13th, 2015

RE: Grass Valley, CA—Preliminary Proposal #032315-1-BEC-R0, Ovivo stormBLOX™ System

Mr. Botrous,

Thank you once again for your continued interest in the Ovivo stormBLOX™ System for Grass Valley, CA. Since our initial proposal we have done some additional evaluation in an effort to reduce both the capital and operating system economics. In addition, we have also done a preliminary plant layout drawing to ensure the system can fit within the provided space available, which is at a premium.

Cost Estimates

Two significant cost components of the stormBLOX process are the adsorption media beds, which are used to remove BOD and ammonia. Therefore we felt that it was a good idea to develop an economic model that evaluated different possibilities in terms of system configuration. Since the activated carbon, in particular, has a tremendous influence on operating costs, we looked at different reduction levels of BOD through the UF and GAC bed. The combination of alum addition followed by ultrafiltration (UF) is particularly effective at removing BOD, but exact removal rates can be difficult to predict.

While we have seen BOD removal rates through the UF greater than 90% in previous studies and are confident the same can be realized at Grass Valley, it is recommended that this removal rate be verified through pilot testing. Wastewater characteristics can vary drastically, particularly when peak flows are involved, impacting BOD removal rates through a UF membrane.

Below you will three different design scenarios for the stormBLOX system:

- 90% BOD removal through UF with activated carbon
- 75% BOD removal through UF with activated carbon
- 90% BOD removal through UF without activatred carbon

We have also revised our design to incorporate on-site regeneration of the zeolite beds, which further reduces costs. A summary of these different design configurations is shown in Table 1.

Table 1. Economic evaluation of various stormBLOX system designs

Parameter	UF + GAC + Zeolite	UF + GAC + Zeolite	UF + Zeolite
Peak Flow	6.0 MGD	6.0 MGD	6.0 MGD
No. Peak Events per Year	10	10	10
No. Days in Peak Event	2	2	2
Hours Operation per Peak Day	24	24	24
Influent BOD	50 mg/l	50 mg/l	50 mg/l
Influent TSS	50 mg/l	50 mg/l	50 mg/l
Influent NH4	10 mg/l	10 mg/l	10 mg/l
Pre-Treatment	2 mm fine screen	2 mm fine screen	2 mm fine screen
Alum Addition	0.7 mg alum/mg TSS	0.7 mg alum/mg TSS	0.7 mg alum/mg TSS
BOD Reduction Through UF	75%	75%	90%
BOD Reduction Through GAC	50%	25%	-
NH_4 Reduction Through Zeolite	>90%	>90%	>90%
Effluent BOD	2.5 mg/l	9.4 mg/l	5.0 mg/l
Effluent NH_4	<1.0 mg/l	<1.0 mg/l	<1.0 mg/l
Effluent TSS	<1.0 mg/l	<1.0 mg/l	<1.0 mg/l
Alum Costs	1,753 lb/day; $2,103/year	1,753 lb/day; $2,103/year	1,753 lb/day; $2,103/year
Sodium Hypochlorite Costs	6.8 lb/day; $20/year	6.8 lb/day; $20/year	6.8 lb/day; $20/year
Oxalic Acid Costs	<0.1 lbs/day; $480/year	<0.1 lbs/day; $480/year	<0.1 lbs/day; $480/year
GAC Costs	777,600 lbs/year; $2,332,800/year	423,360 lbs/year; $1,270,000/year	-
Zeolite On-site Regeneration Costs	92,320 lbs K_2SO_4; $37,851/year	92,320 lbs K_2SO_4; $37,851/year	92,320 lbs K_2SO_4; $37,851/year
Cleaning Labor	1,310 hrs/year; $65,500/year	1,310 hrs/year; $65,500/year	670 hrs/year; $33,500/year
Plant Maintenance	271 hrs/year; $13,550/year	271 hrs/year; $13,550/year	237 hrs/year; $11,850/year
Power	7,403 kWhr/day; $14,806/year	7,403 kWhr/day; $14,806/year	5,341 kWhr/day; $10,682/year
CAPEX	**$13,928,929**	**$11,832,328**	**$8,969,685**
OPEX	**$2,467,111/year**	**$1,404,391/year**	**$96,486/year**

Scope of Supply

The preliminary cost estimates were based on the following Ovivo scope of supply:

- Fine Screens
- UF System
    - UF membranes
    - UF cassettes
    - Feed, permeate, and backwash pumps
    - Chemical dosing pumps
        - Alum
        - Chlorine
        - Oxalic acid
    - Instrumentation, valves, and skid piping
- Granular Activated Carbon System
    - Carbon media
    - Carbon bed (tank)
    - Instrumentation, valves, and skid piping
- Zeolite System
    - Zeolite media
    - Zeolite bed/tank
    - Instrumentation, valves, and skid piping
    - Backwash pump
- Effluent Storage Tank (for UF backwashing and Zeolite regeneration)
- Control System
    - PLC
    - HMI
    - SCADA
- Project Management
- Inspection
- Commissioning & Training

Plant Layout

In order to develop a realistic footprint estimate we felt it was best to do a preliminary CAD design to ensure the system could fit in the allotted space. Figure 1 shows the preliminary system layout superimposed on a satellite photo of the plant. We have also included the preliminary layout drawings for your review.

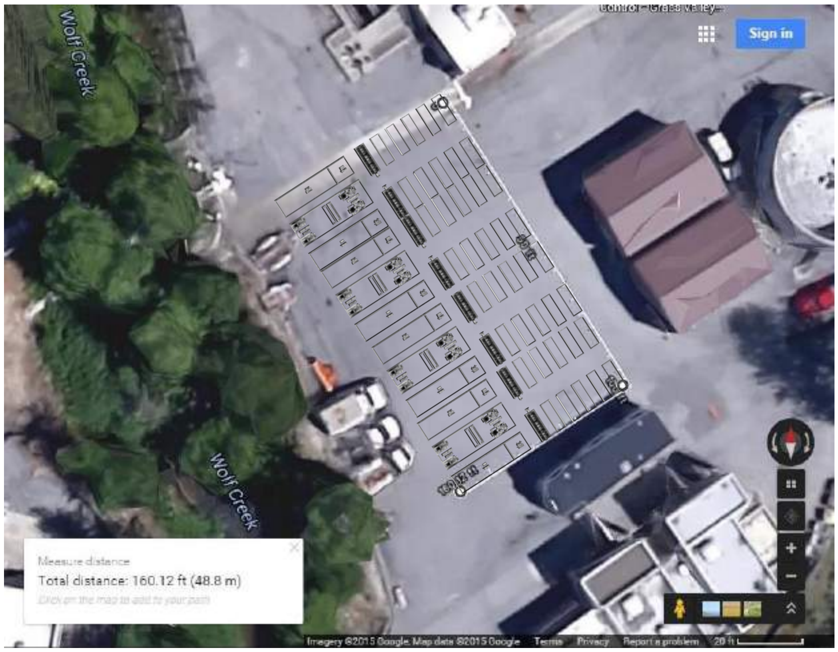

Figure 1: Proposed stormBLOX layout

It should be noted that the layout does include the carbon beds, so if it is determined that the UF is capable of meeting the <10 mg/l BOD effluent limit, then the overall footprint would be reduced.

Please do not hesitate to contact me or our local representative, Jim Gleason of Treatment Equipment Company, at (425) 641-4306 or jim@tec-nw.com if you have any questions.

Regards,

Brian Codianne
Regional Manager, MBR Systems

Appendix H SCENARIOS TO INCREASE SECONDARY TREATMENT CAPACITY AT DIFFERENT GROWTH HORIZONS

Scenario	Plant Capacity Mgal/d	Flow to Storm Blox Mgal/d	Equalized Flow to Secondary Mgal/d	Percent I/I reduction %	Required Equalization(1) Mgal
1	ADWF 1.6 mgd	0.0	7	0	8.4
2	ADWF 1.6 mgd	0.0	7	5	7.3
3	ADWF 1.6 mgd	0.0	7	10	6.3
4	ADWF 1.6 mgd	0.0	7	15	5.4
5	ADWF 1.6 mgd	0.0	7	20	4.6
6	ADWF 1.6 mgd	0.0	7	40	1.8
7	ADWF 1.6 mgd	0.5	7	0	7.3
8	ADWF 1.6 mgd	0.5	7	5	6.3
9	ADWF 1.6 mgd	0.5	7	10	5.4
10	ADWF 1.6 mgd	0.5	7	15	4.7
11	ADWF 1.6 mgd	0.5	7	20	3.9
12	ADWF 1.6 mgd	0.5	7	25	3.2
13	ADWF 1.6 mgd	1.0	7	0	6.3
14	ADWF 1.6 mgd	1.0	7	5	5.5
15	ADWF 1.6 mgd	1.0	7	10	4.7
16	ADWF 1.6 mgd	1.0	7	15	4.0
17	ADWF 1.6 mgd	1.0	7	20	3.3
18	ADWF 1.6 mgd	1.0	7	25	2.6
19	ADWF 1.6 mgd	1.5	7	0	5.5
20	ADWF 1.6 mgd	1.5	7	5	4.8
21	ADWF 1.6 mgd	1.5	7	10	4.1
22	ADWF 1.6 mgd	1.5	7	15	3.4
23	ADWF 1.6 mgd	1.5	7	20	2.7
24	ADWF 1.6 mgd	1.5	7	25	2.1
25	ADWF 1.6 mgd	4.0	7	0	2.6
26	ADWF 1.6 mgd	2.0	7	5	4.2
27	ADWF 1.6 mgd	2.0	7	10	3.5
28	ADWF 1.6 mgd	2.0	7	15	2.8
29	ADWF 1.6 mgd	2.0	7	20	2.2
30	ADWF 1.6 mgd	2.0	7	25	1.6

(1) Including existing 6 Mgal equalization storage

Scenario	Plant Capacity Mgal/d	Flow to Storm Blox Mgal/d	Equalized Flow to Secondary Mgal/d	Percent I/I reduction %	Required Equalization(1) Mgal
1	ADWF 1.9 mgd	0.0	7	0	12.0
2	ADWF 1.9 mgd	0.0	7	5	10.7
3	ADWF 1.9 mgd	0.0	7	10	9.6
4	ADWF 1.9 mgd	0.0	7	15	8.5
5	ADWF 1.9 mgd	0.0	7	20	7.5
6	ADWF 1.9 mgd	0.0	7	40	3.9
7	ADWF 1.9 mgd	0.5	7	0	10.5
8	ADWF 1.9 mgd	0.5	7	5	9.5
9	ADWF 1.9 mgd	0.5	7	10	8.4
10	ADWF 1.9 mgd	0.5	7	15	7.4
11	ADWF 1.9 mgd	0.5	7	20	6.4
12	ADWF 1.9 mgd	0.5	7	25	5.5
13	ADWF 1.9 mgd	1.0	7	0	9.4
14	ADWF 1.9 mgd	1.0	7	5	8.4
15	ADWF 1.9 mgd	1.0	7	10	7.4
16	ADWF 1.9 mgd	1.0	7	15	6.4
17	ADWF 1.9 mgd	1.0	7	20	5.6
18	ADWF 1.9 mgd	1.0	7	25	4.8
19	ADWF 1.9 mgd	1.5	7	0	8.3
20	ADWF 1.9 mgd	1.5	7	5	7.3
21	ADWF 1.9 mgd	1.5	7	10	6.4
22	ADWF 1.9 mgd	1.5	7	15	5.6
23	ADWF 1.9 mgd	1.5	7	20	4.9
24	ADWF 1.9 mgd	1.5	7	25	4.2
25	ADWF 1.9 mgd	4.0	7	0	4.6
26	ADWF 1.9 mgd	2.0	7	5	6.5
27	ADWF 1.9 mgd	2.0	7	10	5.7
28	ADWF 1.9 mgd	2.0	7	15	5.0
29	ADWF 1.9 mgd	2.0	7	20	4.2
30	ADWF 1.9 mgd	2.0	7	25	3.5

(1) Including existing 6 Mgal equalization storage

Scenario	Plant Capacity Mgal/d	Flow to Storm Blox Mgal/d	Equalized Flow to Secondary Mgal/d	Percent I/I reduction %	Required Equalization[1] Mgal
1	ADWF 2.1 mgd	0.0	7	0	14.9
2	ADWF 2.1 mgd	0.0	7	5	13.6
3	ADWF 2.1 mgd	0.0	7	10	12.2
4	ADWF 2.1 mgd	0.0	7	15	10.9
5	ADWF 2.1 mgd	0.0	7	20	9.7
6	ADWF 2.1 mgd	0.0	7	40	5.6
7	ADWF 2.1 mgd	0.5	7	0	13.3
8	ADWF 2.1 mgd	0.5	7	5	11.9
9	ADWF 2.1 mgd	0.5	7	10	10.7
10	ADWF 2.1 mgd	0.5	7	15	9.6
11	ADWF 2.1 mgd	0.5	7	20	8.6
12	ADWF 2.1 mgd	0.5	7	25	7.5
13	ADWF 2.1 mgd	1.0	7	0	11.7
14	ADWF 2.1 mgd	1.0	7	5	10.6
15	ADWF 2.1 mgd	1.0	7	10	9.5
16	ADWF 2.1 mgd	1.0	7	15	8.5
17	ADWF 2.1 mgd	1.0	7	20	7.5
18	ADWF 2.1 mgd	1.0	7	25	6.5
19	ADWF 2.1 mgd	1.5	7	0	10.5
20	ADWF 2.1 mgd	1.5	7	5	9.5
21	ADWF 2.1 mgd	1.5	7	10	8.4
22	ADWF 2.1 mgd	1.5	7	15	7.4
23	ADWF 2.1 mgd	1.5	7	20	6.5
24	ADWF 2.1 mgd	1.5	7	25	5.7
25	ADWF 2.1 mgd	4.0	7	0	6.0
26	ADWF 2.1 mgd	2.0	7	5	8.4
27	ADWF 2.1 mgd	2.0	7	10	7.4
28	ADWF 2.1 mgd	2.0	7	15	6.5
29	ADWF 2.1 mgd	2.0	7	20	5.7
30	ADWF 2.1 mgd	2.0	7	25	5.0

(1) Including existing 6 Mgal equalization storage