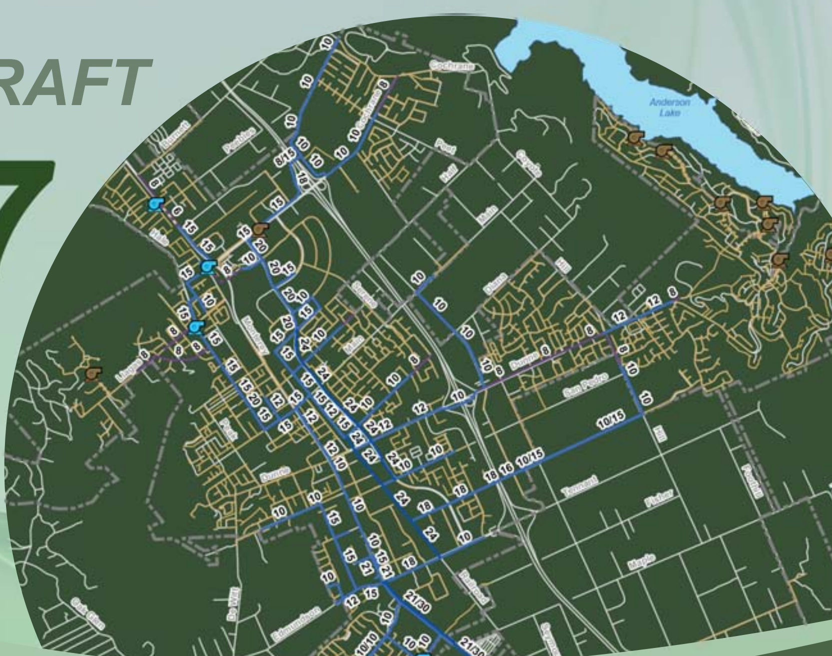
MORGAN HILL
California

Sewer System Master Plan

ADMINISTRATIVE DRAFT

AUGUST 2017



AKEL
ENGINEERING GROUP, INC.

CITY OF MORGAN HILL

2017

SEWER SYSTEM MASTER PLAN

Administrative Draft

August 2017

AKEL ENGINEERING GROUP, INC.

August 16, 2017

City of Morgan Hill
17575 Peak Avenue
Morgan Hill California, 95037

Attention: Karl Bjarke, P.E.
Director of Public Works/City Engineer

Subject: 2017 Sewer System Master Plan – Draft Report

Dear Karl:

We are pleased to submit the draft report for the City of Morgan Hill Sewer System Master Plan. This master plan is a standalone document, though it was prepared as part of the integrated infrastructure master plans for the water, sewer, and storm drainage master plans. The master plan documents the following:

- Existing collection system facilities, acceptable hydraulic performance criteria, and projected wastewater flows consistent with the Urban Planning Area
- Development and calibration of the City's GIS-based hydraulic sewer model.
- Capacity evaluation of the existing sewer system with improvements to mitigate existing deficiencies and to accommodate future growth.
- Capital improvement program (CIP) with an opinion of probable construction costs and suggestions for cost allocations to meet AB 1600.
- Morgan Hill-Gilroy Joint Trunk Analysis/Evaluation

We extend our thanks to you, Dan Repp, Deputy Director of Public Works, and other City staff whose courtesy and cooperation were valuable components in completing this study.

Sincerely,

AKEL ENGINEERING GROUP, INC.

Tony Akel, P.E.
Principal

Enclosure: Report

Acknowledgements

City Council

Steve Tate, Mayor

Rich Constantine, Mayor Pro Tempore

Rich Constantine

Rene Spring

Caitlin Robinett Jachimowicz

Management Personnel

Karl Bjarke, Public Works Director/ City Engineer

Dan Repp, Deputy Director of Utility Services

Scott Creer, Deputy Director for Engineering

John Baty, Senior Planner

Other City Engineering, Planning, and Operations Staff

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APPENDICES

EXECUTIVE SUMMARY

This executive summary presents a brief background of the City of Morgan Hill's sewer system, the planning area characteristics, the planning and design criteria, and the hydraulic model development.

The hydraulic model was used to evaluate the capacity adequacy of the existing sanitary sewer system and for recommending improvements to mitigate existing deficiencies and for servicing future growth. The prioritized capital improvement program accounts for growth through the Morgan Hill Planning Area.

ES.1 STUDY OBJECTIVES

The City of Morgan Hill recognizes the importance of planning, developing, and financing the City's water system infrastructure. The City retained the services of Placeworks to develop a comprehensive, long-term General Plan for the orderly development of the community, while integrating the City's social, economic, and environmental goals. On July 27, 2016, the City Council adopted the Morgan Hill 2035 General Plan, a comprehensive update of the City's General Plan.

As a part of the General Plan update, the City also initiated the update of the infrastructure master plans. These master plans, which were closely coordinated and paralleled the preparation of the General Plan, included:

- 2017 Water System Master Plan
- 2017 Sewer System Master Plan
- 2017 Storm Drainage System Master Plan

City Council approved the preparation of the General Plan in June of 2013, which included authorizing Akel Engineering Group Inc. to prepare this master plan. The 2017 Sewer System Master Plan (SSMP) is intended to serve as a tool for planning and phasing the construction of future sanitary sewer system facilities for the projected buildout of the City of Morgan Hill. This 2017 WSMP is intended to serve as a tool for planning and phasing the construction of future sewer system infrastructure for the projected buildout of the City of Morgan Hill.

The area and horizon for the master plan is stipulated in the City's General Plan. Should planning conditions change, and depending on their magnitude, adjustments to the master plan recommendations might be necessary.

This master plan included the following tasks:

- Summarize the City's existing sanitary sewer system facilities.

- Document growth planning assumptions and known future developments.
- Summarize the sewer system performance criteria and design storm event.
- Project future sewer flows.
- Develop and calibrate a new hydraulic model based on the City's Geographic Information Systems (GIS).
- Evaluate the adequacy of capacity for the sanitary sewer system facilities to meet existing and projected peak dry weather flows and peak wet weather flows.
- Recommend a capital improvement program (CIP) with an opinion of probable construction costs.
- Perform a capacity allocation analysis for cost sharing purposes between existing users and future growth.
- Develop the 2017 Sewer System Master Plan Report.

ES.2 INTEGRATED APPROACH TO MASTER PLANNING

The City implemented an integrated master planning approach and contracted the services of Akel Engineering Group to prepare the following documents:

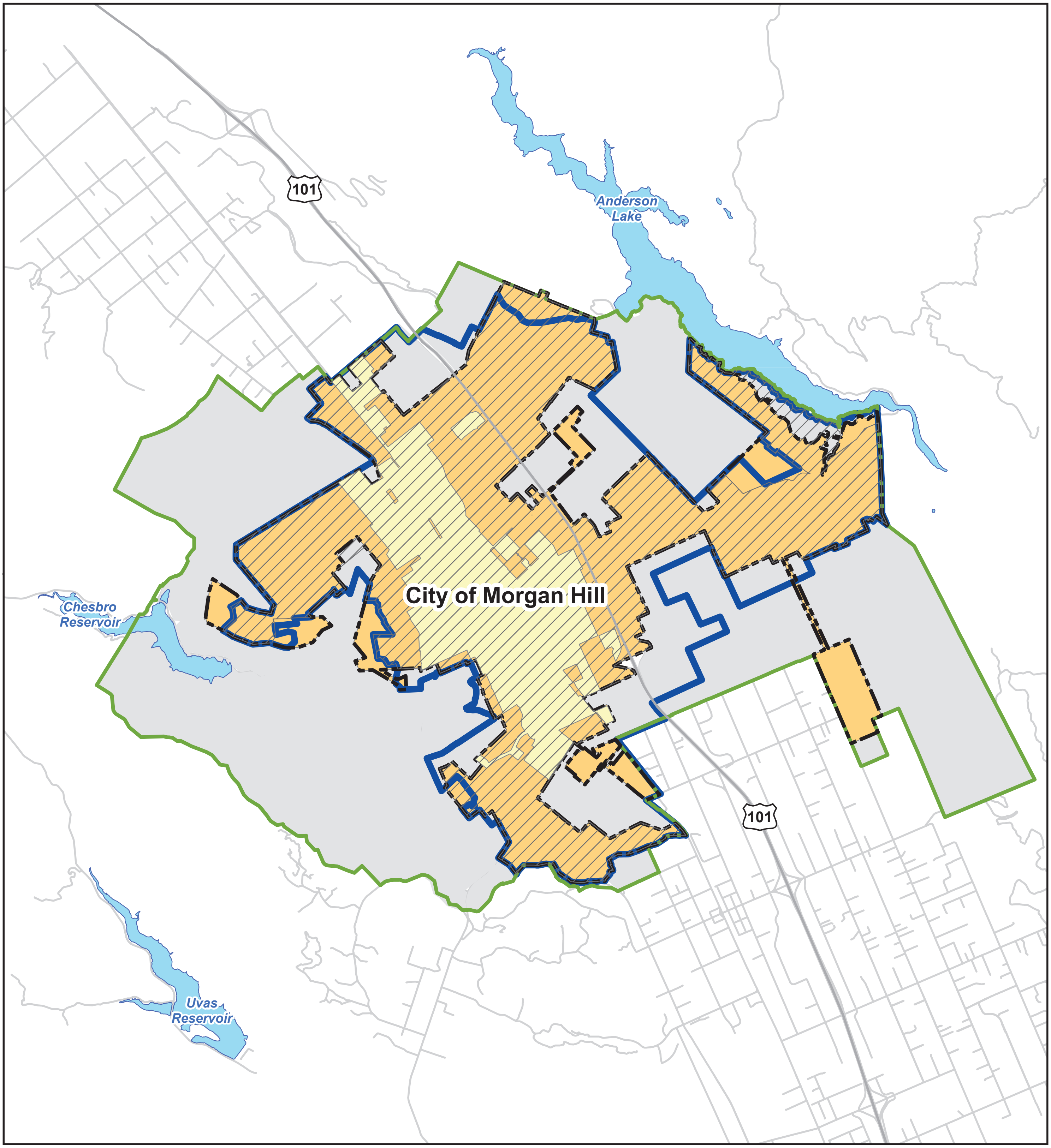
- Water System Master Plan
- Sewer System Master Plan
- Storm Drainage System Master Plan

While each of these reports is published as a standalone document, it has been coordinated for consistency with the City's General Plan document. Additionally, each document has been cross referenced to reflect relevant analysis results with the other documents.

ES.3 STUDY AREA DESCRIPTION

The City of Morgan Hill is located in Santa Clara County, approximately 22 miles southeast of the City of San Jose and 24 miles northwest of the city of Hollister. The City's closest neighbor, the City of Gilroy, is located 8 miles to the southeast. U.S. Route 101 bisects the eastern boundary of the City in the north-south direction. The City limits currently encompass 12.9 square miles, with an approximate population of 42,000 residents.

The City is generally bound to the north by Tilton Avenue, to the east by Anderson Lake, to the southeast by Foothill Avenue, to the west by Sunnyside Drive, and to the south by Middle Avenue. The unincorporated community of San Martin is located to the south of the City. The City's topography is generally flat in the center of the City with increasing slopes on the east and west. **Figure ES.1** displays the planning area showing city limits, the Urban Growth Boundary of the City and the City's Sphere of Influence Boundary.



Legend

City Limits	General Plan Area
City Limits Area	Roads
Redevelopment Area	Highways
Urban Service Area	Lakes
Urban Growth Boundary	
Sphere of Influence Boundary	

PRELIMINARY

Updated: February 24, 2017

0 0.25 0.5 1 Mile

N W GIS E S

ES. 1
Planning Area

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\ES\MH_ES-1Planning Area_022417.mxd

ES.4 SYSTEM PERFORMANCE AND DESIGN CRITERIA

Gravity sewer capacities depend on several factors including: material and roughness of the pipe, the limiting velocity and slope, and the maximum allowable depth of flow. Design criteria include capacity requirements for the sanitary sewer collection facilities, flow calculation methodologies for future users, flow peaking factors, and accounting for infiltration and inflows.

Partial Flow Criteria (d/D)

Partial flow in gravity sewers is expressed as a depth of flow to pipe diameter ratio (d/D). For circular gravity conduits, the highest capacity is generally reached at 92 percent of the full height of the pipe (d/D ratio of 0.92). This is due to the additional wetted perimeter and increased friction of a gravity pipe.

During max day dry weather flows (MDDWF), the maximum allowable d/D ratio for all proposed pipes (all diameters) is 0.75. The maximum allowable d/D ratio for all existing pipes (all diameters) is 0.90. The criterion for existing pipes is relaxed in order to maximize the use of the existing pipes before costly pipes improvements are required.

During max day wet weather flows (MDWWF), to avoid premature or unnecessary trunk line replacements, the capacity analysis allowed the d/D ratio to exceed the dry weather flow criteria and surcharge. This condition is evaluated using the dynamic hydraulic model criteria that stipulates that the hydraulic grade line (HGL), even during a surcharged condition, should be at least three feet below the manhole rim elevation

The City's design standards pertaining to the d/D criteria are summarized in **Table ES.1**.

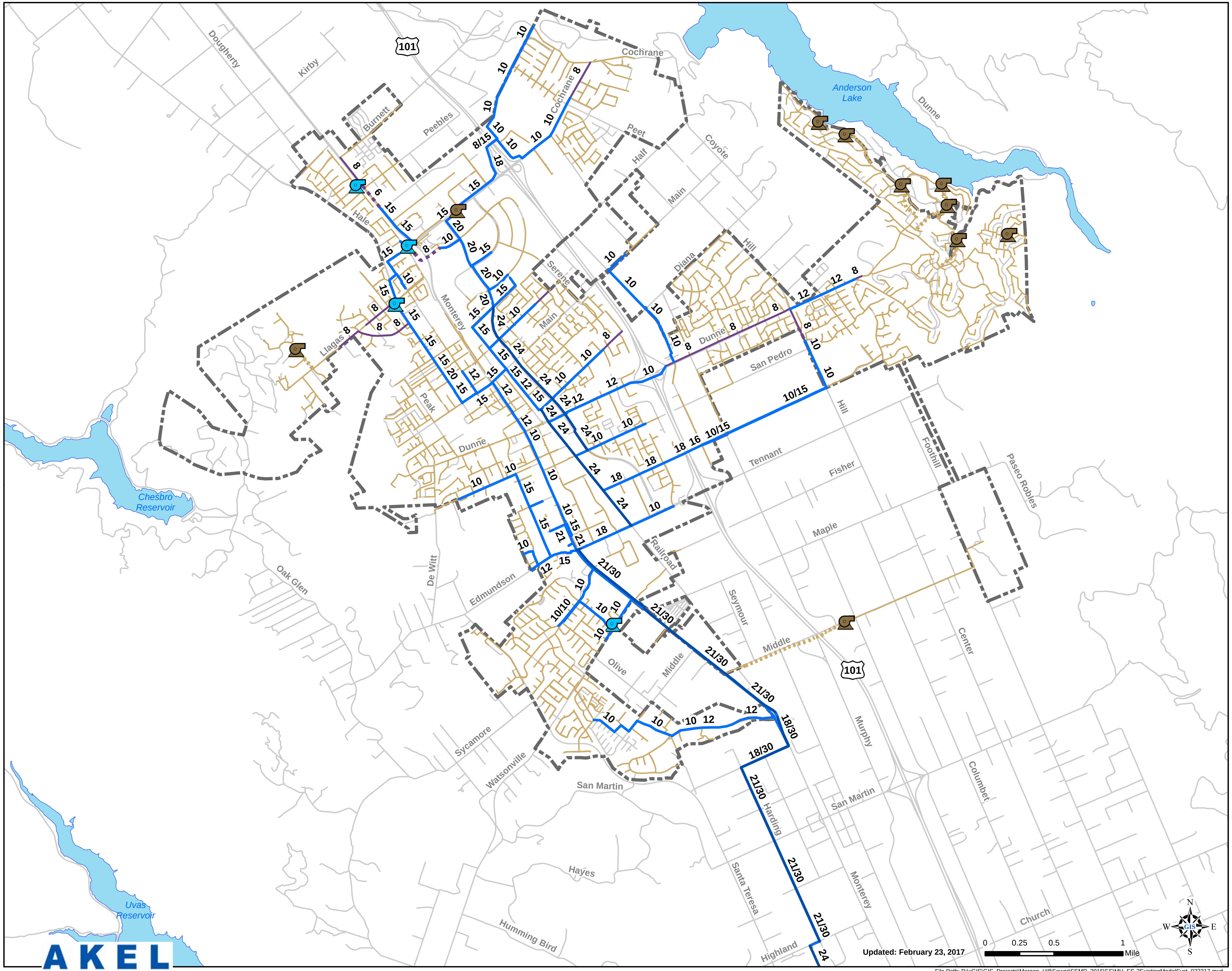
ES.5 EXISTING SEWER COLLECTION SYSTEM OERVIEW

The City provides sewer collection services to approximately 12,400 residential, commercial, industrial, and institutional accounts. The City's collection system consists of approximately 158 miles of up to 30-inch gravity sewer pipes, which includes part of the Morgan Hill-Gilroy Joint Sewer Trunk, that convey flows towards the South County Regional Wastewater Authority (SCRWA) Wastewater Treatment Plant (WWTP), located southeast of the City of Gilroy, as shown on **Figure ES.2**.

A system-wide modeled pipe inventory, listing the total length by pipe diameter, is shown on **Table ES.2**. This table is based on information extracted from the City's GIS and updated to reflect review of construction drawings provided by City staff. The 8-inch, 10-inch and 12-inch diameter pipes account for approximately 50 percent of the total sewer pipeline length.

ES.6 SANITARY SEWER FLOWS

The wastewater flows collected and treated at the SCRWA WWTP vary monthly, daily, and hourly. While the dry weather flows are influenced by customer uses, the wet weather flows are



Legend

Modeled System

Lift Stations

Modeled Gravity Pipes by Size

8"

10" - 21"

24" - 30"

Modeled Force Mains

8" or Smaller

Non-Modeled System

Lift Stations

Gravity Pipes

Force Mains

Roads

City Limits

Lakes

PRELIMINARY

ES. 2
Existing Modeled Sewer System

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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Table ES.1 Sewer System Performance and Design Criteria

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Pipeline Criteria			
Peak Dry Weather Flow Criteria			
Diameter		**Maximum Allowable d/D**	
(in)		**Existing Trunks**	**Proposed Trunks**
8 to 12		**0.90**	**0.75**
> 12		**0.90**	**0.75**
Peak Wet Weather Flow Criteria			
Hydraulic Grade Line (HGL) should be at least **3 feet** below the manhole rim			
Pipe Size	**Minimum Grade**	**Capacity** (n = 0.013)	
(in)	(ft/ft)	(mgd)	(cfs)
8	0.0026	0.36	0.55
10	0.0019	0.56	0.87
12	0.0015	0.79	1.23
15	0.0011	1.28	1.98
18	0.0009	1.78	2.75
21	0.0007	2.43	3.76
24	0.0006	3.27	5.05
27	0.0005	4.18	6.47
30	0.0004	5.13	7.94
33	0.0004	6.04	9.34
36	0.0004	7.61	11.78
42	0.0003	10.27	15.90

1/7/2016

Table ES.2 Existing Modeled Sewer Pipe Inventory

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Pipe Size	Length (feet)	Length (miles)
City Pipes		
≤ 8"	21,756	4.1
10"	51,293	9.7
12"	20,654	3.9
14"	425	0.1
15"	25,257	4.8
16"	4,126	0.8
18"	8,167	1.5
20"	4,244	0.8
21"	11,340	2.1
24"	11,869	2.2
30"	13,060	2.5
Total	172,191	32.6
Joint Trunk Pipes		
≤ 21"	8,436	1.6
24"	12,924	2.4
27"	4,407	0.8
30"	10,257	1.9
33"	22,132	4.2
42"	246	< 0.1
60"	96	< 0.1
Total	58,497	11.1

6/28/2016

influenced by the severity and length of storm events. **Table ES.3** shows the City flows recorded at the SCRWA WWTP have decreased from 2.85 mgd in 2010 to 2.37 mgd in 2015. In addition to listing the 2010-2015 flows, and for comparison purposes, the table calculates the peaking factors applied to the corresponding average annual flows for each year.

The land use methodology was used to estimate the buildout flows from the City's Planning Area and to be consistent with the General Plan. **Table ES.4** documents the total acreages for residential and non-residential land use categories, and the undeveloped lands designated for urbanization. The undeveloped lands were multiplied by the corresponding unit flow factor to estimate the wastewater flows. The existing sewer flows were increased to 2.7 mgd to account for 100% occupancy, and the ultimate buildout flows were calculated at 4.2 mgd.

ES.7 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

The City's hydraulic model combines information on the physical characteristics of the sanitary sewer system (pipelines, lift stations, force mains) and operational characteristics (how they operate). The hydraulic model then performs calculations and solves series of equations to simulate flows in pipes, including backwater calculations for surcharged conditions. Computer modeling requires the compilation of large numerical databases that enable data input into the model. Detailed physical aspects, such as pipe size, ground elevation, invert elevations, and pipe lengths contribute to the accuracy of the model.

The hydraulic modeling software used for evaluating the capacity adequacy of the Morgan Hill sewer system, InfoSWMM by Innovyze Inc., utilizes the fully dynamic St. Venant's equation which has a more accurate engine for simulating backwater and surcharge conditions, in addition to having the capability for simulating manifolded force mains. The software also incorporates the use of the Manning Equation in other calculations including upstream pipe flow conditions.

Model Development

The City of Morgan Hill's sanitary sewer system was skeletonized to reduce the model from approximately 156 miles of pipeline extracted from the GIS to 44 miles of pipeline modeled. Skeletonizing the model is useful in creating a system that accurately reflects the hydraulics of the pipes within the system while reducing the complexities of large models. This process reduces the time of analysis while maintaining accuracy, but will also comply with the limitations imposed by the computer program. The modeled pipes generally include pipes 10-inches in diameter and larger, and critical smaller diameter lines, as well as force mains. The modeled sewer system is shown on **Figure ES.2**.

Model Calibration

Calibration is intended to instill a level of confidence in the flows that are simulated in the hydraulic model. The calibration process was iterative as it involved calibrating each of the flow monitored sites in the 2014 V&A flow monitoring program and for the following three calibration conditions: 1)

Table ES.3 Historical Flow Data and Peaking Factors

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Year	Population	Average Annual Flow			Seasonal Average		Maximum Month		Maximum Day		Total SCRWA Plant Flow[1]	
		AAF (MGD)	Per Capita Flow (GPCD)	Percentage Change	ADWF (MGD)	AWWF (MGD)	MMDWF (MGD)	MMWWF (MGD)	MDDWF (MGD)	MDWWF (MGD)	MDDWF (MGD)	MDWWF (MGD)
2010	40,246	2.85	71	-	2.69	3.02	2.89	3.22	3.14	4.61	7.19	8.99
2011	38,309	2.85	74	0%	2.66	3.04	2.86	3.71	3.10	5.81	7.37	11.98
2012	39,127	2.69	69	-6%	2.60	2.78	2.66	2.97	2.77	4.61	7.13	9.68
2013	40,079	2.69	67	0%	2.66	2.73	2.70	2.77	2.90	3.09	7.18	7.67
2014	41,197	2.58	63	-4%	2.52	2.64	2.64	2.73	2.81	3.69	6.57	8.45
2015	42,382	2.37	56	-8%	2.31	2.40	2.35	2.64	2.42	3.77	6.02	8.24
Historical Peaking Factors (Applied to ADWF)												
2010	-	1.06	-	-	1.00	1.12	1.07	1.20	1.17	1.71	-	-
2011	-	1.07	-	-	1.00	1.14	1.08	1.39	1.17	2.18	-	-
2012	-	1.03	-	-	1.00	1.07	1.02	1.14	1.07	1.77	-	-
2013	-	1.01	-	-	1.00	1.03	1.02	1.04	1.09	1.16	-	-
2014	-	1.02	-	-	1.00	1.05	1.05	1.08	1.12	1.46	-	-
2015	-	1.03	-	-	1.00	1.04	1.02	1.15	1.05	1.64	-	-

Notes:

6/29/2016

1. Total SCRWA Plant Flow represents combined flow of cities of Morgan Hill and Gilroy.
2. Definitions are as follows:
   - AAF - Average Annual Flow (annual flow, expressed in daily or other time units)
   - ADWF - Average Dry Weather Flow (average flow that occurs on a daily basis during the dry weather season)
   - AWWF - Average Wet Weather Flow (average flow that occurs on a daily basis during the wet weather season)
   - MMDWF - Maximum Month Dry Weather Flow (maximum month flow during the dry weather season)
   - MMWWF - Maximum Month Wet Weather Flow (maximum month flow during the wet weather season)
   - MDDWF - Maximum Day Dry Weather Flow (highest measured daily flow that occurs during a dry weather season)
   - MDWWF - Maximum Day Wet Weather Flow (highest measured daily flow that occurs during a wet weather season)
   - PDWF - Peak Dry Weather Flow (highest measured hourly flow that occurs during a dry weather flow)
   - PWWF - Peak Hour Wet Weather Flow (highest measured hourly flow that occurs during wet weather)
3. Source:
   - 2010 and 2011 flows from South County Regional WasteWater Authority Community Development Report
   - 2012, 2013, 2014, and 2015 flows from South County Regional WasteWater Authority Public Works Report

Table ES.4 Average Daily Flows at Buildout of Project Area
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Land Use Classifications	Sewer Flows at 100% Occupancy													
	Existing Development within City Limits			Future Development within City Limits			Total Development within City Limits		Future Development Outside City Limits		Total	
	Existing Development within City Limits (net acres)	Sewer Unit Factor (gpd/net acre)	Existing Average Daily Flow (gpd)	Future Development (net acres)	Future Sewer Unit Factor (gpd/net acre)	Future Development Average Daily Flow (gpd)	Development (net acres)	Total Development Average Daily Flow (gpd)	Future Development (net acres)	Future Development Average Daily Flow (gpd)	Development (net acres)	Average Daily Flow (gpd)
Residential												
Single Family												
Residential Estate	508	150	76,184	198	150	29,670	706	105,854	321	48,208	1,027	154,062
Residential Detached Low	979	340	333,019	171	340	58,076	1,150	391,094	239	81,123	1,389	472,218
Residential Detached Medium	1,252	630	789,028	187	630	117,524	1,439	906,552	411	259,136	1,850	1,165,687
Residential Detached High	30	840	25,374	4	840	3,649	35	29,024	20	16,430	54	45,454
Multi-Family												
Residential Attached Low	340	1,100	374,280	114	1,100	125,902	455	500,182	2	2,384	457	502,566
Residential Attached Medium	100	1,700	169,290	53	1,700	89,953	152	259,243	7	12,494	160	271,737
Residential Attached High	1	2,930	2,344	5	2,930	16,065	6	18,409	0	0	6	18,409
Subtotal	3,211		1,769,519	732		440,839	3,943	2,210,358	1,000	419,775	4,943	2,630,133
Non-Residential												
General Commercial	24	1,340	32,131	0	1,340	0	24	32,131	0	0	24	32,131
Commercial	260	1,000	259,501	130	1,000	130,352	390	389,853	4	3,700	394	393,553
Commercial / Industrial[1]	501	900	451,041	230	900	207,281	731	658,322	220	197,918	951	856,240
Mixed Use	93	960	89,594	6	960	5,861	99	95,454	0	0	99	95,454
Mixed Use Flex	64	900	57,604	40	900	36,436	104	94,040	8	7,395	113	101,435
Public Facility	302	220	66,362	12	220	2,582	313	68,944	46	10,206	360	79,149
Subtotal	1,244		956,232	419		382,512	1,663	1,338,744	278	219,219	1,941	1,557,963
Other (Non-Flow Generating)												
Sports-Recreation-Leisure	0	0	0	0	0	0	0	0	251	0	251	0
Landscape Irrigation	201	0	0	0	0	0	201	0	0	0	201	0
Open Space	605	0	0	581	0	0	1,186	0	2,737	0	3,922	0
Subtotal	806		0	581		0	1,387	0	2,988	0	4,375	0
Totals	**5,260**		**2,725,751**	**1,732**		**823,351**	**6,992**	**3,549,102**	**4,267**	**638,993**	**11,259**	**4,188,095**

Notes
1. "Commercial / Industrial" combines land use types "Commercial / Institutional" and "Industrial"

9/19/2016

peak dry weather flow, 2) peak wet weather flows from storm rainfall Event No. 1, and 3) peak wet weather flows from storm rainfall Event No. 2

The calibrated hydraulic model was used as an established benchmark in the capacity evaluation of the existing sanitary sewer system. The model was also used to identify improvements necessary for mitigating existing system deficiencies and for accommodating future growth.

The hydraulic model is a valuable investment that will continue to prove its worth to the City as future planning issues or other operational conditions surface. It is recommended that the model be maintained and updated with new construction projects to preserve its integrity.

ES.8 CAPACITY EVALUATION

The calibrated hydraulic model was used for evaluating the sanitary sewer system for capacity deficiencies during max day dry weather flows (MDDWF) and max day wet weather flows (MDWWF). The system performance and design criterion was used as a basis to judge the adequacy of capacity for the existing sanitary sewer system. The design flows simulated in the hydraulic model for existing conditions and the general plan buildout include:

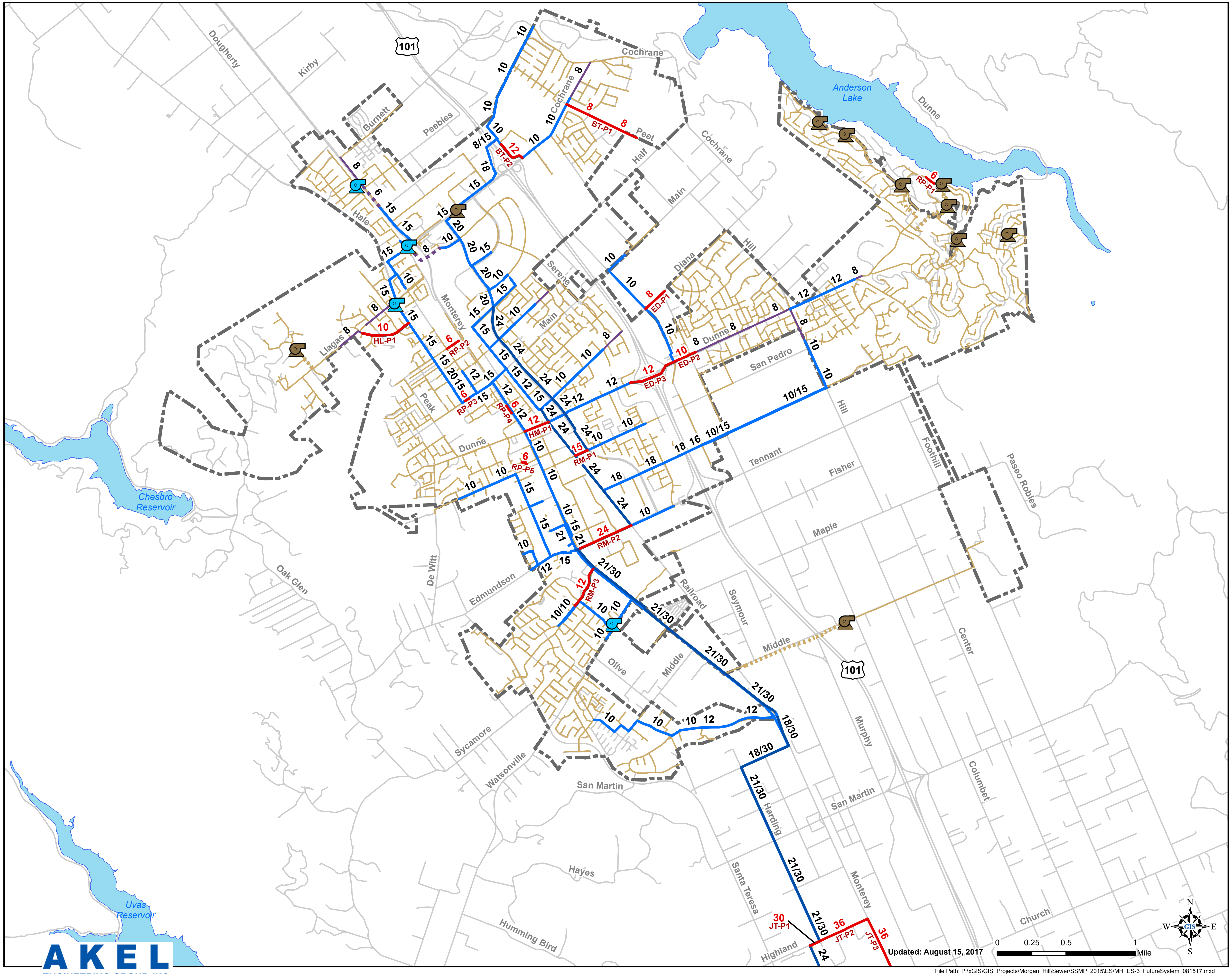
- Existing MDDWF = 2.8 mgd
- Existing MDWWF = 3.7 mgd
- Buildout MDDWF = 4.7 mgd
- Buildout MDWWF = 6.0 mgd

In general, the hydraulic model indicated that the sanitary sewer system exhibited acceptable performance to service the existing customers during both peak dry weather and peak wet weather flows. Future flows were then added to the hydraulic model and the existing system was expanded in order to serve these future customers. The proposed improvements for the future system are shown with pipe sizes on an overall exhibit on **Figure ES.3**. This master plan also included a capacity assessment of the Morgan Hill-Gilroy Joint Sewer Trunk.

ES.9 CAPITAL IMPROVEMENT PROGRAM

The Capital Improvement Program includes pipeline, lift station, and pipe rehabilitation projects recommended in this master plan (**Table ES.5**). Each improvement was assigned a uniquely coded identifier associated with its basin, and which is used for locating it on **Figure ES.3**.

The estimated construction costs include the baseline costs plus **30 percent** contingency allowance to account for unforeseen events and unknown field conditions. Capital improvement costs include the estimated construction costs plus **30 percent** project-related costs (engineering design, project administration, construction management and inspection, and legal costs).



Legend

Proposed Pipe Improvements

Existing Modeled System

Lift Stations

Modeled Gravity Pipes by Size

8"

10" - 21"

24" - 30"

Modeled Force Mains

8" or Smaller

Non-Modeled System

Lift Stations

Gravity Pipes

Force Mains

Roads

City Limits

Lakes

PRELIMINARY

ES. 3
Proposed Improvements

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

Table ES.5 Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements: Existing Diameter (in)	Pipeline Improvements: New/Parallel/ Replace	Infrastructure Costs: Diameter (in)	Infrastructure Costs: Length (ft)	Infrastructure Costs: Unit Cost ($)	Infrastructure Costs: Infr. Cost ($)	Baseline Constr. Costs [1] ($)	Estimated Const. Costs [2] ($)	Capacity Improv. Cost [3] ($)	Suggested Expenditure Budget	Construction Trigger (EDUs)	Suggested Cost Allocation: Existing Users (%)	Suggested Cost Allocation: Future Users (%)	Cost Allocation: Existing Users ($)	Cost Allocation: Future Users ($)
Pipeline Capacity Improvements																		
Butterfield Trunk																		
BT-P1	Gravity Main	Peet Rd	From approximately 3,000 ft e/o Cochrane Rd to Cochrane Rd	-	New	8	3,000	171	511,745	511,745	665,269	864,849	2026-2030	-	0%	100%	0	864,849
BT-P2	Gravity Main	Along NB US 101	From 900 ft n/o Cochrane Rd to intersection of Cochrane Rd and NB US 101	10	Replacement	12	1,200	199	238,814	238,814	310,459	403,596	2026-2030	1,900	51%	49%	204,471	199,125
									Subtotal - Butterfield Trunk	**750,560**	**975,727**	**1,268,446**						
Hale-Llagas Trunk																		
HL-P1	Gravity Main	Llagas Creek Dr	From Eagle Springs Ct to Hale Ave	8	Replacement	10	1,950	185	360,354	360,354	468,460	608,998	2021-2025	675	72%	28%	435,981	173,017
									Subtotal - Hale-Llagas Trunk	**360,354**	**468,460**	**608,998**						
East Dunne Trunk																		
ED-P1	Gravity Main	Diana Ave	From Murphy Ave to Condit Rd	-	New	8	1,000	171	170,582	170,582	221,756	288,283	2026-2030	-	0%	100%	0	288,283
ED-P2	Gravity Main	Dunne Ave	From 230 ft e/o Murphy Ave to Condit Rd	8	Replacement	10	950	185	175,557	175,557	228,224	296,691	2021-2025	1,525	83%	17%	247,639	49,052
ED-P3	Gravity Main	Dunne Ave	From Condit Rd to 530 ft e/o Walnut Grove Dr	8/10	Replacement	12	1,950	199	388,073	388,073	504,495	655,844	2021-2025	2,400	47%	53%	309,083	346,762
									Subtotal - East Dunne Trunk	**734,212**	**954,476**	**1,240,819**						
Hale-Monterey Trunk																		
HM-P1	Gravity Main	Dunne Ave	From Monterey Rd to Railroad Ave	-	New	12	1,000	199	199,012	199,012	258,716	336,330	2018-2020	0	80%	20%	270,210	66,121
RP-P2	Gravity Main	Wright Ave	From 230 ft e/o Garden Ave to Del Monte Ave	6	Replacement	6	550	156	86,002	86,002	111,802	145,343	2018-2020	0	100%	0%	145,343	0
RP-P3	Gravity Main	Main Ave	Frome 120 ft e/o Hale Ave to 300 ft e/o Hale Ave	6	Replacement	6	175	156	27,364	27,364	35,573	46,245	2018-2020	0	100%	0%	46,245	0
RP-P4	Gravity Main	Monterey Rd	From 3rd to 4th Street	6	Replacement	6	350	156	54,728	54,728	71,147	92,491	2018-2020	0	100%	0%	92,491	0
RP-P5	Gravity Main	ROW	Right of Way e/o Manor Ct to 450 ft w/o Monterey Rd and Bisceglia Ave	6	Replacement	6	200	156	31,273	31,273	40,655	52,852	2018-2020	0	100%	0%	52,852	0
									Subtotal - Hale-Monterey Trunk	**398,379**	**517,893**	**673,261**						
Railroad-Monterey Trunk																		
RM-P1	Gravity Main	San Pedro Ave	From Butterfield Blvd to Railroad Ave	10	Replacement	15	550	270	148,548	148,548	193,113	251,047	2026-2030	2,000	58%	42%	146,509	104,537
RM-P2 [4]	Gravity Main	Tennant Ave	From RailRoad Ave to Monterey Rd	18	Replacement	24	2,200	426	938,199	1,131,799	1,471,339	1,912,741	2018-2020	3,175	61%	39%	1,170,082	742,659
RM-P3	Gravity Main	La Crosse Dr / Vineyard Blvd	From La Mar Dr to Monterey Rd	10	Replacement	12	1,700	199	338,320	338,320	439,817	571,762	2021-2025	0	92%	8%	527,499	44,262
									Subtotal - Railroad-Monterey Trunk	**1,618,668**	**2,104,269**	**2,735,549**						
Hill-Barrett Trunk																		
RP-P1	Gravity Main	ROW	Along Holiday Dr to Oak Ln	6	Replacement	6	400	156	62,547	62,547	81,311	105,704	2018-2020	0	100%	0%	105,704	0
									Subtotal - Hill-Barrett Trunk	62,547	81,311	105,704						
Joint Trunk [5]																		
JT-P1	Gravity Main	Highland Ave	From Harding Ave to 400 ft w/o Harding Ave	21	Replacement	30	450	569	255,873	255,873	294,253	338,391	2018-2020	0	100%	0%	338,391	0
									Subtotal - Joint Trunk	255,873	294,253	338,391						
									Subtotal - Pipeline Capacity Improvements	**4,180,592**	**5,396,389**	**6,971,168**					**4,092,500**	**2,878,668**
Relief Trunk Improvements [5]																		
JT-P2	Gravity Main	Highland Ave	From Harding Ave to Monterey Rd	-	New	30	2,050	569	1,165,642	1,165,642	1,340,488	1,541,561	2018-2020	-	25%	75%	385,390	1,156,171
JT-P3	Gravity Main	Monterey Rd	From Highland Ave to Las Animas Ave	-	New	36	19,700	569	11,201,533	11,201,533	12,881,763	14,814,028	2018-2020	-	25%	75%	3,703,507	11,110,521
JT-P4	Gravity Main	Las Animas Ave	From Monterey Rd to Murray Ave	-	New	36	1,750	569	995,060	995,060	1,144,319	1,315,967	2018-2020	-	25%	75%	328,992	986,975
JT-P5	Gravity Main	Murray Ave	From Las Animas Ave to Chestnut St	-	New	36	7,550	569	4,292,973	4,292,973	4,936,919	5,677,457	2018-2020	-	25%	75%	1,419,364	4,258,093
JT-P6	Gravity Main	Chestnut St	From Murray Ave to Lewis St	-	New	36	400	569	227,442	227,442	261,559	300,792	2018-2020	-	25%	75%	75,198	225,594
JT-P7	Gravity Main	Chestnut St	From Chestnut St to 7th St	-	New	36	2,100	569	1,194,072	1,194,072	1,373,183	1,579,160	2018-2020	-	25%	75%	394,790	1,184,370

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements: Existing Diameter (in)	Pipeline Improvements: New/Parallel/ Replace	Infrastructure Costs: Diameter (in)	Infrastructure Costs: Length (ft)	Infrastructure Costs: Unit Cost ($)	Infrastructure Costs: Infr. Cost ($)	Baseline Constr. Costs [1] ($)	Estimated Const. Costs [2] ($)	Capacity Improv. Cost [3] ($)	Suggested Expenditure Budget	Construction Trigger (EDUs)	Suggested Cost Allocation: Existing Users (%)	Suggested Cost Allocation: Future Users (%)	Cost Allocation: Existing Users ($)	Cost Allocation: Future Users ($)
JT-P8	Gravity Main	7th Street	From Chestnut St to US Highway 101.	-	New	36	1,450	569	824,478	824,478	948,150	1,090,373	2018-2020	-	25%	75%	272,593	817,779
JT-P9	Gravity Main	Along US 101	Jogging from 7th St and US Highway 101 to Renz Lane	-	New	36	2,000	569	1,137,211	1,137,211	1,307,793	1,503,962	2018-2020	-	25%	75%	375,991	1,127,972
Subtotal - Joint Trunk Improvements										**21,038,413**	**24,194,175**	**27,823,301**					**6,955,825**	**20,867,476**
Joint Trunk Condition Assessment [6]																		
CCTV and Condition Assessment - Existing Joint Trunk							24,369	200,000		-	-	200,000	2018-2020		100%	0%	200,000	0
Subtotal - Joint Trunk Condition Assessment										**-**	**-**	**200,000**					**200,000**	**0**
Infiltration and Inflow Improvements																		
INI-P1	Gravity Main	Llagas Rd	From 80 ft e/o Hale Ave to 20 ft e/o Hale Ave (Group 5)	8	Trenchless Rehabilitation	8	100	41	4,127	4,127	5,365	6,975	2018-2020	-	100%	0%	6,975	0
INI-P2	Gravity Main	Llagas Rd	From Fox Hollow Cir to Murphy Springs Dr (Group 5)	8	Trenchless Rehabilitation	8	350	41	14,446	14,446	18,779	24,413	2018-2020	-	100%	0%	24,413	0
INI-P3	Gravity Main	Laurel Wood Ln	From 120 fts/o Almond Orchard Dr to 135 ft s/o Almond Orchard Dr (Grp 5)	6	Point Repair	6	15	162	2,429	2,429	3,157	4,104	2018-2020	-	100%	0%	4,104	0
INI-P4	Gravity Main	250 ft n/o Berkshire Ave	From 60 ft e/o Hale Ave to 115 ft e/o Hale Ave (Group 5)	15	Trenchless Rehabilitation	15	100	67	6,664	6,664	8,664	11,263	2018-2020	-	100%	0%	11,263	0
INI-P5	Gravity Main	110 ft s/o Wright Ave	From 180 ft w/o Crest Ave to 50 ft e/o Crest Ave (Group 4)	6	Trenchless Replacement	6	250	792	198,067	198,067	257,487	334,733	2018-2020	-	100%	0%	334,733	0
INI-P6	Gravity Main	Shady Lane Dr	From Trail Ridge Ln to Calico Ridge Trl (Group 2)	6	Trenchless Rehabilitation	6	150	46	6,965	6,965	9,054	11,771	2018-2020	-	100%	0%	11,771	0
INI-P7	Gravity Main	Trail Ridge Ln	From 150 ft w/o Shady Lane Dr to 70 ft e/o Shady Lane Dr (Group 2)	6	Trenchless Replacement	6	250	792	198,067	198,067	257,487	334,733	2018-2020	-	100%	0%	334,733	0
INI-P8	Gravity Main	50 ft n/o Copper Hill Pl	From 40 ft w/o Copper Hill Dr to 60 ft w/o Holiday Dr (Group 2)	6	Trenchless Rehabilitation	6	200	46	9,286	9,286	12,072	15,694	2018-2020	-	100%	0%	15,694	0
INI-P9	Gravity Main	Quail Ln	From 150 ft e/o Quail Ct to 110 ft w/o Quail Ct (Group 2)	6	Trenchless Rehabilitation	6	300	46	13,930	13,930	18,109	23,541	2018-2020	-	100%	0%	23,541	0
INI-P10	Gravity Main	175 ft s/o Oakridge Ct	From 180 ft n/o Oakridge Ln to Oakridge Ln (Group 1)	6	Trenchless Rehabilitation	6	200	46	9,286	9,286	12,072	15,694	2018-2020	-	100%	0%	15,694	0
Subtotal - Infiltration and Inflow Improvements										**463,267**	**602,247**	**782,921**					**782,921**	**0**
Rehabilitation Improvements																		
Group 1	Gravity Main	Various	See Group 1 Figure	Various	Various	Various	7,750	Various	2,426,606	2,426,606	3,154,588	4,100,964	2018	-	100%	0%	4,100,964	0
Group 2	Gravity Main	Various	See Group 2 Figure	Various	Various	Various	9,800	Various	1,167,715	1,167,715	1,518,029	1,973,438	2019	-	100%	0%	1,973,438	0
Group 3	Gravity Main	Various	See Group 3 Figure	Various	Various	Various	5,650	Various	363,053	363,053	471,968	613,559	2019	-	100%	0%	613,559	0
Group 4	Gravity Main	Various	See Group 4 Figure	Various	Various	Various	10,300	Various	907,288	907,288	1,179,475	1,533,317	2020	-	100%	0%	1,533,317	0
Group 5	Gravity Main	Various	See Group 5 Figure	Various	Various	Various	6,000	Various	371,370	371,370	482,781	627,615	2020	-	100%	0%	627,615	0
Group 6	Gravity Main	Various	See Group 6 Figure	Various	Various	Various	5,550	Various	597,377	597,377	776,590	1,009,566	2021	-	100%	0%	1,009,566	0
Group 7	Gravity Main	Various	See Group 7 Figure	Various	Various	Various	8,950	Various	1,784,493	1,784,493	2,319,841	3,015,794	2021	-	100%	0%	3,015,794	0
Group 8	Gravity Main	Various	See Group 8 Figure	Various	Various	Various	5,700	Various	653,074	653,074	848,996	1,103,695	2022	-	100%	0%	1,103,695	0
Group 9	Gravity Main	Various	See Group 9 Figure	Various	Various	Various	2,900	Various	356,669	356,669	463,670	602,771	2022	-	100%	0%	602,771	0
Subtotal - Rehabilitation Improvements										**8,627,644**	**11,215,937**	**14,580,719**					**14,580,719**	**0**
Comprehensive Plan Updates																		
Sewer System Master Plan Updates (Years 2021, 2026, 2031, 2036)								200,000		-	-	800,000	2021, 2026, 2031, 2036		65%	35%	520,000	280,000
Sewer System Management Plan Updates (Years 2021, 2026, 2031, 2036)								100,000		-	-	400,000	2021, 2026, 2031, 2036		65%	35%	260,000	140,000
Sewer Rate Study Updates (Years 2021, 2026, 2031, 2036)								100,000		-	-	400,000	2021, 2026, 2031, 2036		65%	35%	260,000	140,000
Subtotal - Comprehensive Plan Updates												**1,600,000**					**1,040,000**	**560,000**
On-Going CCTV Sewer System																		
CCTV of 16 miles of pipelines per year (From 2018 to 2035)							84,480	1.50	2,280,960	-	-	2,280,960	126,720 Annually		100%	0%	2,280,960	0
Subtotal - On-going CCTV System												**2,280,960**					**2,280,960**	**0**

Table ES.5 Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements: Existing Diameter (in)	Pipeline Improvements: New/Parallel/ Replace	Infrastructure Costs: Diameter (in)	Infrastructure Costs: Length (ft)	Infrastructure Costs: Unit Cost ($)	Infrastructure Costs: Infr. Cost ($)	Baseline Constr. Costs [1] ($)	Estimated Const. Costs [2] ($)	Capacity Improv. Cost [3] ($)	Suggested Expenditure Budget	Construction Trigger (EDUs)	Suggested Cost Allocation: Existing Users (%)	Suggested Cost Allocation: Future Users (%)	Cost Allocation: Existing Users ($)	Cost Allocation: Future Users ($)
Currently Planned Projects																		
Sewer Plant Expansion (SCRWA)												32,700,000	2017-2024		0%	100%	0	32,700,000
Sewer Plant Maintenance/ Improvements (SCRWA)												9,430,000	2017-2021		100%	0%	9,430,000	0
Holiday Lakes Gravity Line Feasibility Study												60,000	2018		100%	0%	60,000	0
Lift Station Condition Assessment												80,000	2019		100%	0%	80,000	0
Lift Station W Repair and Refurbish												1,000,000	2017-2018		100%	0%	1,000,000	0
Inflow and Infiltration Investigation and Cross Connection Elimination												300,000	2017-2020		100%	0%	300,000	0
Wastewater Collection System Compliance Inspection[7]												10,000,000	2017-2024		100%		10,000,000	0
Subtotal - Currently Planned Projects												**53,570,000**					**20,870,000**	**32,700,000**
Total Costs																		
Pipeline Capacity Improvements																		
Collection System Capacity Improvements										**4,180,592**	**5,396,389**	**6,971,168**					**4,092,500**	**2,878,668**
Relief Trunk Improvements										**21,038,413**	**24,194,175**	**27,823,301**					**6,955,825**	**20,867,476**
Subtotal										**25,219,005**	**29,590,564**	**34,794,469**					**11,048,325**	**23,746,144**
Condition Assessment Improvements																		
Joint Trunk Condition Assessment										**200,000**	**200,000**	**200,000**					**200,000**	**0**
Infiltration and Inflow Improvements										**463,267**	**602,247**	**782,921**					**782,921**	**0**
Rehabilitation Improvements										**8,627,644**	**11,215,937**	**14,580,719**					**14,580,719**	**0**
On-Going CCTV System										**2,280,960**	**2,280,960**	**2,280,960**					**2,280,960**	**0**
Subtotal										**11,571,871**	**14,299,145**	**17,844,600**					**17,844,600**	**0**
Plan Updates																		
Comprehensive Plan Updates										**1,600,000**	**1,600,000**	**1,600,000**					**1,040,000**	**560,000**
Planned Project (Including SCRWA Plant)																		
Currently Planned Projects										**53,570,000**	**53,570,000**	**53,570,000**					**20,870,000**	**32,700,000**
Total Improvement Costs										**91,960,876**	**99,059,708**	**107,809,069**					**50,802,925**	**57,006,144**

AKEL ENGINEERING GROUP, INC.

8/15/2017

Notes :

1. Cost estimates are based on the Engineering News Record (ENR) construction cost index (CCI) of 10532 for January 2017.
2. Baseline construction costs plus 30% to account for unforeseen events and unknown conditions.
3. Estimated construction cost plus 30% to cover other costs including: engineering design, project administration (developer and City staff), construction management and inspection, and legal costs.
4. Improvement RM-P2 will require a casing where crossing railroad. Casing length assumed to be equal to 200 ft.
5. The Joint Relief Trunk improvements are currently in the design process. As such, contingencies are reduced from 30% to 15% for this project.
6. Joint Trunk Condition Assessment extents start at from the intersection of Monterey Highway and California Avenue to Day Road.
7. This item estimates the potential overall cost for inspections and rehabilitations related to sewer collection system condition comliance.

The costs in this Sanitary Sewer System Master Plan were benchmarked using a 20-City national average ENR CCI of 10,532, reflecting a date of January 2017. In total, the CIP includes approximately 22 miles of pipeline, on-going CCTV, comprehensive plan updates, as well as currently planned projects with a cost totaling over $107 million dollars.

CHAPTER 1 - INTRODUCTION

This chapter provides a brief background of the City of Morgan Hill's (City) sewer system (also known as a wastewater collection system), the need for this master plan, and the objectives of the study. Abbreviations and definitions are also provided in this chapter.

1.1 BACKGROUND

The City of Morgan Hill (City) is located approximately 22 miles southeast of the City of San Jose, and 8 miles northwest of the City of Gilroy (**Figure 1.1**). The City provides sewer collection services to approximately 10,000 residential, commercial, industrial, and institutional accounts. The City owns, operates, and maintains the sanitary sewer collection system, which consists of over 34 miles of gravity trunks and force mains, with up to 30-inch pipe sizes, which convey the flow to the South County Regional Wastewater Authority (SCRWA) Wastewater Treatment Plant (WWTP). The WWTP has an average daily capacity rating of 10.1 million gallons per day (MGD).

In 2002, the City developed a Sewer System Master Plan that identified capacity deficiencies in the existing sewer system and recommended improvements to alleviate existing deficiencies and serve future developments in the Urban Growth Boundary.

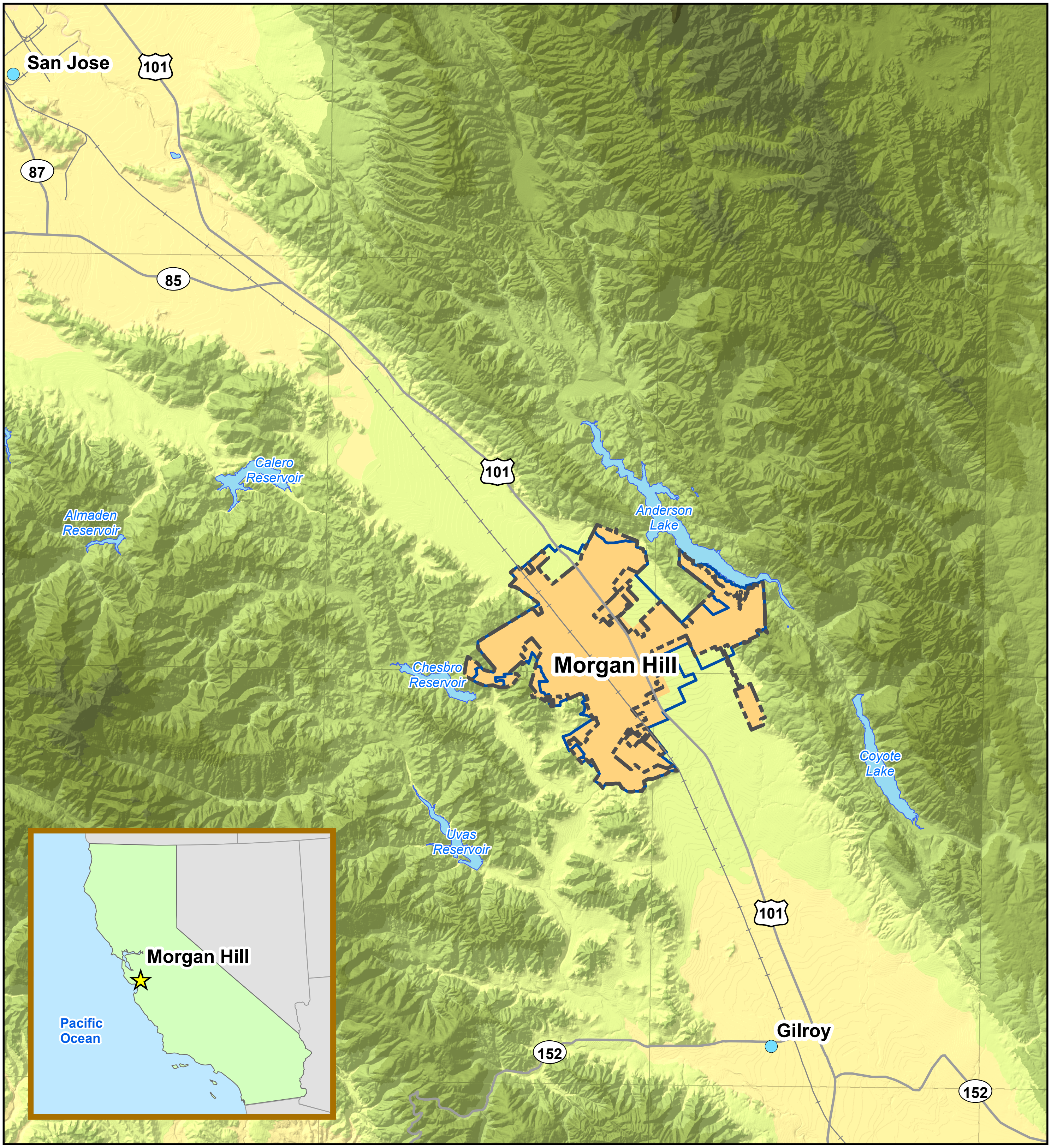
The City recognizes the importance of planning, developing, and financing the sewer system infrastructure. The City retained the services of Placeworks to develop a comprehensive, long-term General Plan for the orderly development of the community, while integrating the City's social, economic, and environmental goals. On July 27, 2016, the City Council adopted the Morgan Hill 2035 General Plan, a comprehensive update of the City's General Plan.

1.2 SCOPE OF WORK

As a part of the General Plan update, the City also initiated the update of the infrastructure master plans. These master plans, which were closely coordinated and paralleled the preparation of the General Plan, included:

- 2017 Water System Master Plan
- 2017 Sewer System Master Plan
- 2017 Storm Drainage System Master Plan

City Council approved the preparation of the General Plan in June of 2013, which included authorizing Akel Engineering Group Inc. to prepare this master plan. The 2017 SSMP evaluates the City's sanitary sewer system and recommends capacity improvements necessary to service the needs of existing users and for servicing the future growth of the City. This 2017 SSMP is



Legend

- Cities
- Study Area
- City Limits
- Urban Growth Boundary
- Highways
- Railroads
- Lakes

Elevation (ft)

- 100 - 250
- 251 - 500
- 501 - 1,000
- 1,001 - 2,000
- 2,001 - 3,000
- 3,001 - 4,000
- > 4,000

PRELIMINARY

0 0.5 1 2 Miles

Updated: January 6, 2016

AKEL ENGINEERING GROUP, INC.

Figure 1.1
Regional Location Map

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\MH_Fig1-1ProjLoc_010616.mxd intended to serve as a tool for planning and phasing the construction of future sewer system infrastructure for the projected buildout of the City of Morgan Hill.

The area and horizon for the master plan is stipulated in the City's General Plan. Should planning conditions change, and depending on their magnitude, adjustments to the master plan recommendations might be necessary.

This master plan included the following tasks:

- Summarize the City's existing sanitary sewer system facilities.
- Document growth planning assumptions and known future developments.
- Summarize the sewer system performance criteria and design storm event.
- Project future sewer flows.
- Develop and calibrate a new hydraulic model based on the City's Geographic Information Systems (GIS).
- Evaluate the adequacy of capacity for the sanitary sewer system facilities to meet existing and projected peak dry weather flows and peak wet weather flows.
- Recommend a capital improvement program (CIP) with an opinion of probable construction costs.
- Perform a capacity allocation analysis for cost sharing purposes between existing users and future growth.
- Develop a 2017 Sewer System Master Plan Report.

1.3 INTEGRATED APPROACH TO MASTER PLANNING

This City implemented an integrated master planning approach and contracted the services of Akel Engineering Group to prepare the following documents:

- Water System Master Plan
- Sewer System Master Plan
- Storm Drainage System Master Plan
- Morgan Hill-Gilroy Joint Trunk Study

While each of these reports is published as a standalone document, it has been coordinated for consistency with the City's General Plan document. Additionally, each document has been cross referenced to reflect relevant analysis results with the other documents.

1.4 RELEVANT REPORTS

The City has completed several special studies intended to evaluate localized growth. These reports were referenced and used during this capacity analysis. The following lists relevant reports that were used in the completion of this master plan, as well as a brief description of each document:

- **City of Morgan Hill Sewer System Master Plan, February 2002 (2002 SSMP).** This report documents the planning and performance criteria, evaluates the sewer system, recommends improvements, and provides an estimate of costs.

- **City of Morgan Hill 2035 General Plan, July 2016 (2035 General Plan).** The City's 2035General Plan provides future land use planning, and growth assumptions for the planning areas. Additionally, this report establishes the planning horizon for improvements in this master plan.

- **Morgan Hill-Gilroy Joint Trunk Relief Phasing.** This study evaluates the hydraulic capacity of the uppers segments of the Joint Trunk Sewer, between the Cities of Morgan Hill and Gilroy.

- **Morgan Hill-Gilroy Joint Trunk Capacity Allocation.** This study summarizes the capacity allocation between the Cities of Morgan Hill and Gilroy, in accordance with the 2008 capacity analysis.

- **2015 Urban Water Management Plan (2015 UWMP).** The 2015 Urban Water Management Plan (UWMP) establishes a benchmark per capita water usage and targets in order to achieve higher levels of water conservation for the sustainability of water supply sources. This includes adopting an updated water shortage contingency plan, defining supply sources, addressing supply reliability, and projecting sustainable supply yields and future demands.

1.5 REPORT ORGANIZATION

The Sewer System Master Plan report contains the following chapters:

Chapter 1 – Introduction. This chapter provides a brief background of the City of Morgan Hill's sewer system, the need for this master plan, and the objectives of the study. Abbreviations and definitions are also provided in this chapter.

Chapter 2 – Planning Area Characteristics. This chapter presents a discussion of the planning area characteristics for this master plan including a study area description; service areas land use; and population for the City of Morgan Hill.

Chapter 3 – System Performance and Design Criteria. This chapter presents the City's performance and design criteria, which were used in this master plan for evaluating the adequacy of capacity for the existing sanitary sewer system and for sizing improvements required to mitigate deficiencies and to accommodate future growth. The design criteria include: capacity requirements for the sanitary sewer facilities, flow calculation methodologies for future users, flow peaking factors, and accounting for infiltration and inflows.

Chapter 4 – Existing Sewer Collection Facilities. This chapter provides a description of the City's existing sanitary sewer system facilities including gravity trunks, force mains, lift stations, and sewer collection basins. The chapter also includes a brief description of the South County Regional Wastewater Authority Wastewater Treatment Plant.

Chapter 5 – Sanitary Sewer Flows. This chapter summarizes historical wastewater flows experienced at the SCRWA WWTP and defines flow terminologies relevant to this evaluation. This chapter discusses the wastewater flow distribution within the five basins, and identifies the design flows used in the hydraulic modeling effort and capacity evaluation. The design flows include the existing condition (existing customers) and the projected ultimate buildout scenario.

Chapter 6 – Hydraulic Model Development. This chapter describes the development and calibration of the City's sanitary sewer system hydraulic model. Hydraulic network analysis has become an effectively powerful tool in all aspects of sanitary sewer system planning, design, operation, management, and system reliability analysis. The City's hydraulic model was used to evaluate the capacity adequacy of the existing system and to plan its expansion to service anticipated future growth.

Chapter 7 – Evaluation and Proposed Improvements. This section presents a summary of the sanitary sewer system capacity evaluation during peak dry weather flows and peak wet weather flows for the existing and buildout flows. The recommended sanitary sewer system improvements needed to mitigate capacity deficiencies are also discussed in this chapter.

Chapter 8 – Capital Improvement Program. This chapter provides a summary of the recommended Capital Improvement Program (CIP) for the City of Morgan Hill's sanitary sewer system. The program is based on the evaluation of the City's sewer system, and on the recommended projects described in the previous chapters. The CIP has been prepared to assist the City in planning and constructing the collection system improvements through the ultimate buildout scenario. This chapter also presents the cost criteria and methodologies for developing the capacity improvement costs.

1.6 ACKNOWLEDGEMENTS

Obtaining the necessary information to successfully complete the analysis presented in this report, and developing the long-term strategy for mitigating the existing system deficiencies and for accommodating future growth, was accomplished with the strong commitment and very active input from dedicated team members including:

- Karl Bjarke, Public Works Director/City Engineer
- Dan Repp, Deputy Director of Utility Services
- Scott Creer, Deputy Director for Engineering
- John Baty, Senior Planner
- David Gittleson, Associate Engineer
- Mark Rauscher, Engineering Technician

As part of the preparation of this Sewer System Master Plan, Hydmet Consulting prepared reports for the design storm used in evaluating the existing and future sewer system for Max Day Wet Weather flows.

1.7 UNIT CONVERSIONS AND ABBREVIATIONS

Engineering units were used in reporting flow rates and volumes pertaining to the design and operation of various components of the sanitary sewer system. In some cases, different sets of units were used to describe the same parameter where it was necessary to report values in smaller or larger quantities. Values reported in one set of units can be converted to another set of units by applying a multiplication factor. A list of multiplication factors for units used in this report are shown on **Table 1.1**.

Various abbreviations and acronyms were also used in this report to represent relevant sewer system terminologies and engineering units. A list of abbreviations and acronyms is included in **Table 1.2**.

1.8 GEOGRAPHIC INFORMATION SYSTEMS

This master planning effort made extensive use of Geographic Information Systems (GIS) technology, for efficiently completing the following tasks:

- Develop the physical characteristics of the hydraulic model (gravity mains, force mains, and lift stations).
- Allocate existing wastewater loads, as calculated using the developed wastewater unit factors.
- Calculate and allocating future wastewater loads, based on the future developments land use.
- Extract ground elevations along the gravity and force mains from available contour maps.
- Generate maps and exhibits used in this master plan.

Table 1.1 Unit Conversions

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Volume Unit Calculations		
To Convert From:	**To:**	**Multiply by:**
acre feet	gallons	325,857
acre feet	cubic feet	43,560
acre feet	million gallons	0.3259
cubic feet	gallons	7.481
cubic feet	acre feet	2.296×10^{-5}
cubic feet	million gallons	7.481×10^{-6}
gallons	cubic feet	0.1337
gallons	acre feet	3.069×10^{-6}
gallons	million gallons	1×10^{-6}
million gallons	gallons	1,000,000
million gallons	cubic feet	133,672
million gallons	acre feet	3.069

Flow Rate Calculations		
To Convert From:	**To:**	**Multiply By:**
ac-ft/yr	mgd	8.93×10^{-4}
ac-ft/yr	cfs	1.381×10^{-3}
ac-ft/yr	gpm	0.621
ac-ft/yr	gpd	892.7
cfs	mgd	0.646
cfs	gpm	448.8
cfs	ac-ft/yr	724
cfs	gpd	646300
gpd	mgd	1×10^{-6}
gpd	cfs	1.547×10^{-6}
gpd	gpm	6.944×10^{-4}
gpd	ac-ft/yr	1.12×10^{-3}
gpm	mgd	1.44×10^{-3}
gpm	cfs	2.228×10^{-3}
gpm	ac-ft/yr	1.61
gpm	gpd	1,440
mgd	cfs	1.547
mgd	gpm	694.4
mgd	ac-ft/yr	1,120
mgd	gpd	1,000,000

2/11/2016

Table 1.2 Abbreviations and Acronyms

Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Abbreviation	Expansion	Abbreviation	Expansion
2016 SSMP	2016 Sewer System Master Plan	HGL	Hydraulic Grade Line
10Yr-24Hr	10-Year 24-Hour	in/hr	Inch per Hour
ADWF	Average Dry Weather Flow	I&I	Infiltration and Inflow
AAF	Annual Average Flow	LF	Linear Feet
Akel	Akel Engineering Group, Inc.	MDDWF	Maximum Day Dry Weather Flow
AWWF	Average Wet Weather Flow	MDWWF	Maximum Day Wet Weather Flow
BWF	Base Wastewater Flow	MGD	Million Gallons per Day
CCI	Construct Cost Index	MMDWF	Maximum Month Dry Weather Flow
CCTV	Closed Circuit Television	MMWWF	Maximum Month Wet Weather Flow
CDP	Census Designated Place	NOAA	National Oceanic and Atmospheric Administration
CIP	Capital Improvement Program	PWSS	Public Water System Statistics
City	City of Morgan Hill	PDWF	Peak Dry Weather Flow
DDF	Depth Duration Frequency	PWWF	Peak Wet Weather Flow
d/D	depth of flow to pipe diameter	ROW	Right of Way
EDUs	Equivalent Dwelling Units	SCADA	Supervisory Control and Data Acquisition
ENR	Engineering News Record	SCRWA	South County Regional Wastewater Authority
fps	Feet per Second	SCVWD	Santa Clara Valley Water District
FY	Fiscal Year	VCP	Vitrified Clay Pipe
GIS	Geographic Information Systems	V&A	Villalobos and Associates
gpd	Gallons per Day	WWTP	Wastewater Treatment Plant
gpm	Gallons per Minute		

CHAPTER 2 - PLANNING AREA CHARACTERISTICS

This chapter presents a discussion of the planning area characteristics for this master plan and includes a study area description, service area land use, and population for the City of Morgan Hill.

2.1 STUDY AREA DESCRIPTION

The City of Morgan Hill is located in Santa Clara County, approximately 22 miles southeast of the City of San Jose and 24 miles northwest of the city of Hollister. The City's closest neighbor, the City of Gilroy, is located 8 miles to the southeast. U.S. Route 101 bisects the eastern boundary of the City in the north-south direction. The City limits currently encompass 12.9 square miles, with an approximate population of 42,000 residents.

The City is generally bound to the north by Tilton Avenue, to the east by Anderson Lake, to the southeast by Foothill Avenue, to the west by Sunnyside Drive, and to the south by Middle Avenue. There are several creeks flowing through and along the boundaries of the City, including: Fisher Creek, West Little Llagas Creek, and Llagas Creek. The topography is generally flat in the valley portion of the city, with increasing slopes in east and west side of the city due to the Santa Cruz Mountain to the west and the Diablo Range to the east. The unincorporated community of San Martin is located to the south of the City. **Figure 2.1** displays the planning area showing City Limits, the Urban Growth Boundary of the City, and the City's Sphere of Influence Boundary.

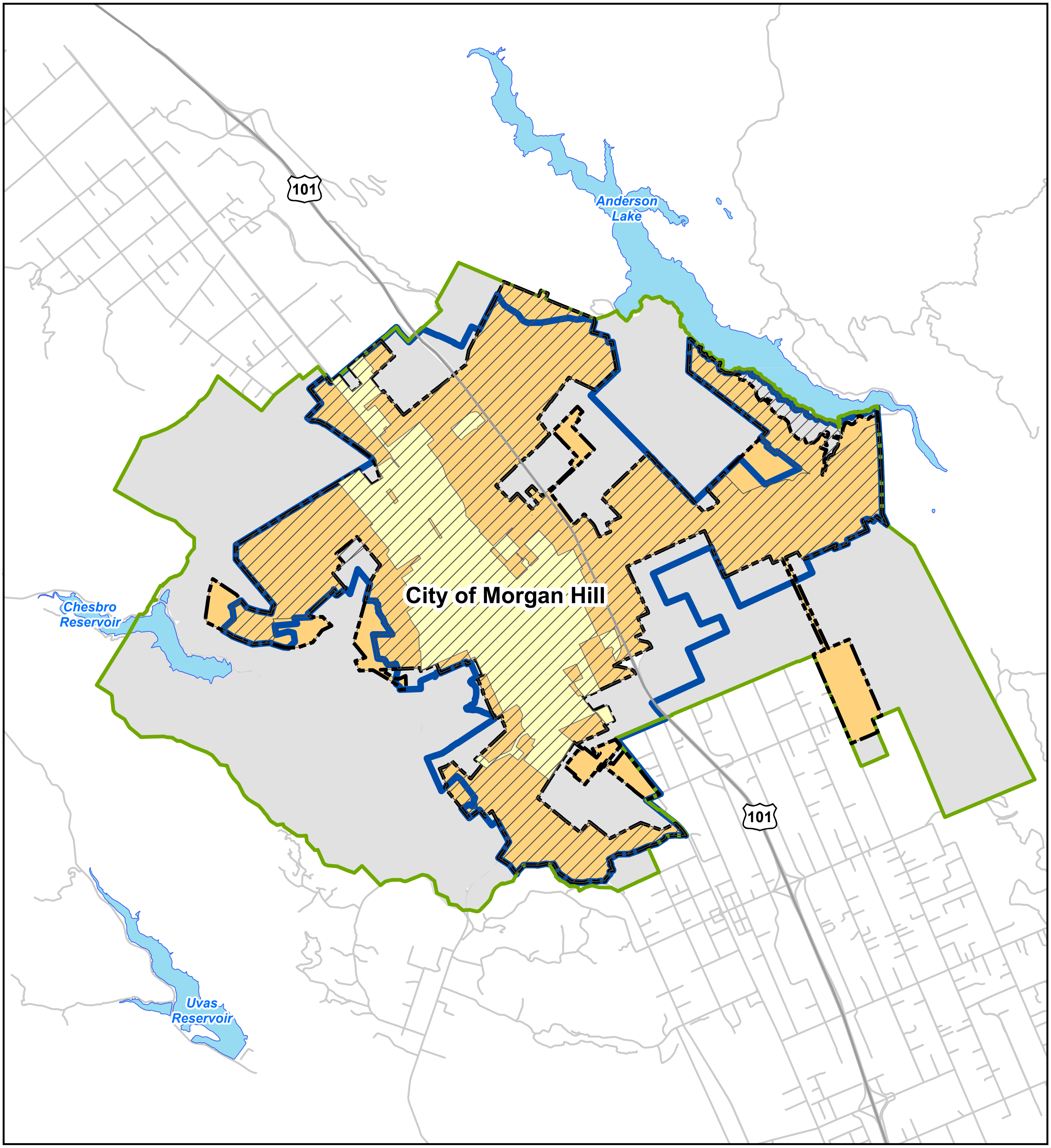
The City operates and maintains a sewer collection system that covers the majority of the area within the City Limits. Currently, the wastewater flows are conveyed to the South County Regional Wastewater Authority (SCRWA) Wastewater Treatment Plant (WWTP).

2.2 SEWER SERVICE AREAS AND LAND USE

The City of Morgan Hill's wastewater collection system services residential and non-residential lands within the City limits, as summarized on **Table 2.1**. This service area includes:

- 5,260 net acres of developed lands inside the City limits.
- 1,732 net acres of undeveloped lands inside the City limits.

The existing land use statistics were based on information received from Placeworks staff, the planning firm responsible for preparing the 2035 General Plan, and are shown on **Figure 2.2.**



Legend

- City Limits
- City Limits Area
- Redevelopment Area
- Urban Service Area
- Urban Growth Boundary
- Sphere of Influence Boundary
- General Plan Area
- Roads
- Highways
- Lakes

PRELIMINARY

Updated: February 24, 2017

0 0.25 0.5 1 Mile

N W E S

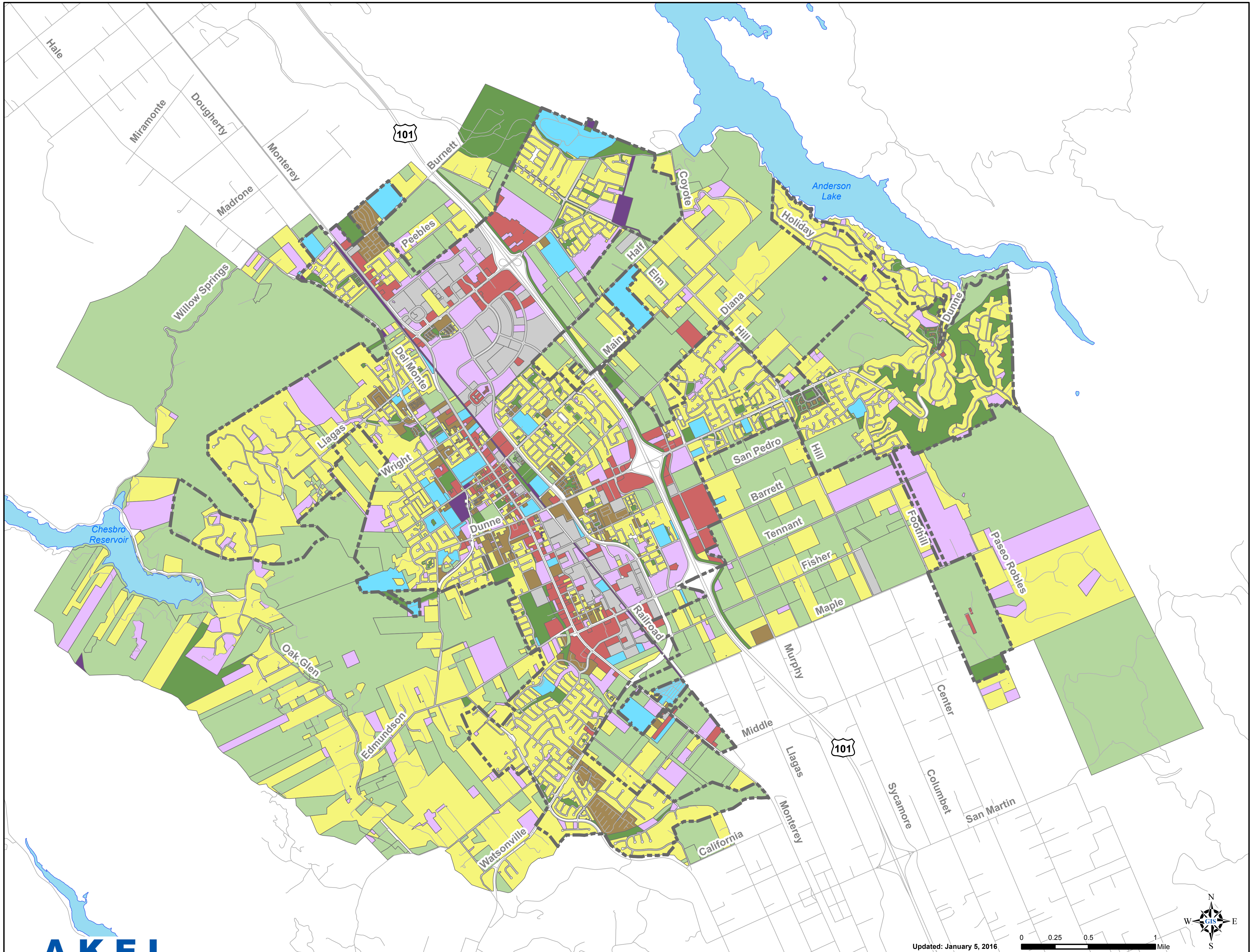
AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\MH_Fig2-1Planning Area_022417.mxd

Figure 2.1
Planning Area

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL



Legend

Existing Land Use

- Agriculture
- Parks
- Single-Family
- Multi-Family
- Commercial
- Industrial
- Public/Institutional
- Vacant
- Other
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 2.2
Existing Land Use

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

AKEL
ENGINEERING GROUP, INC.

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Table 2.1 Existing and Future Sewer Service Areas

Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Land Use Classification	Existing Service Area (City Limits)		Development Outside City Limits	
	Developed (net acres)	Undeveloped (net acres)	Developed (net acres)	Undeveloped (net acres)
Residential				
Rural County	0	0	3,966	2,435
Residential Estate	508	198	228	94
Single Family Low	979	171	169	70
Single Family Medium	1,252	187	294	117
Single Family High	30	4	7	12
Subtotal - Single Family Residential	2,770	560	4,664	2,728
Multi-Family Low	340	114	2	0
Multi-Family Medium	100	53	0	7
Multi-Family High	1	5	0	0
Subtotal - Multi-Family Residential	441	173	2	7
Subtotal - Residential	**3,211**	**732**	**4,666**	**2,736**
Non-Residential				
General Commercial	24	0	0	0
Commercial	260	130	4	0
Commercial / Industrial[1]	501	230	145	75
Mixed Use	93	6	0	0
Mixed Use Flex	64	40	8	0
Sports-Recreation-Leisure	0	0	212	39
Public Facility	302	12	46	0
Subtotal	1,244	**419**	**416**	**113**
Other (Non-Flow Generating)				
Landscape Irrigation	201	0	0	0
Open Space	605	581	1,409	1,328
Subtotal	806	**581**	**1,409**	**1,328**
Total	**5,260**	**1,732**	**6,491**	**4,177**

AKEL ENGINEERING GROUP, INC.

9/19/2016

Note:
1. "Commercial / Industrial" combines land use types "Commercial / Institutional" and "Industrial"

At ultimate development of the General Plan, the City's wastewater system is anticipated to service approximately 4,943 acres of residential land use, 2,192 acres of non-residential land use, and 4,124 acres of non-flow generating land use, for a total of approximately 11,259 acres (**Table 2.1**). The land use designations utilized in this master plan are consistent with the Land Use Element of the City's General Plan, and as received from the City's planning division and shown on **Figure 2.3**.

2.3 HISTORICAL AND PROJECTED POPULATION

The City is a growing community, with over 2 percent of the Santa Clara County population residing within the City limits. DOF records estimate the 2015 population at more than 42,000. Between 1970 and 1980 the City saw dramatic growth, with the population increasing from 5,579 to 16,924 at an average annual growth rate of approximately 18 percent. This rapid growth led to the City's adoption of a growth management system, known as the Residential Development Control System (RDCS), which regulates growth by limiting the number of new homes approved annually. Following the implementation of the RDCS the average annual growth rate between 1980 and 2000 fell to approximately 4.7 percent. From 2000 to present the City has observed an average annual growth rate of approximately 2.4 percent.

The General Plan Update anticipates a 2035 population of 58,200 and this 2017 SSMP is consistent with the General Plan projections. The current and projected service area population is summarized in **Table 2.2**; it should be noted that projected service area populations are consistent with the City's 2015 UWMP.

The City's RDCS sets a maximum number of annual housing allotments that would not be exceeded and can only be reduced. Furthermore, if the number of allotments is reduced in a given year, they cannot be added to a future year. The population limit, which is a ceiling and not a target, is then a function of the maximum number of allotments.

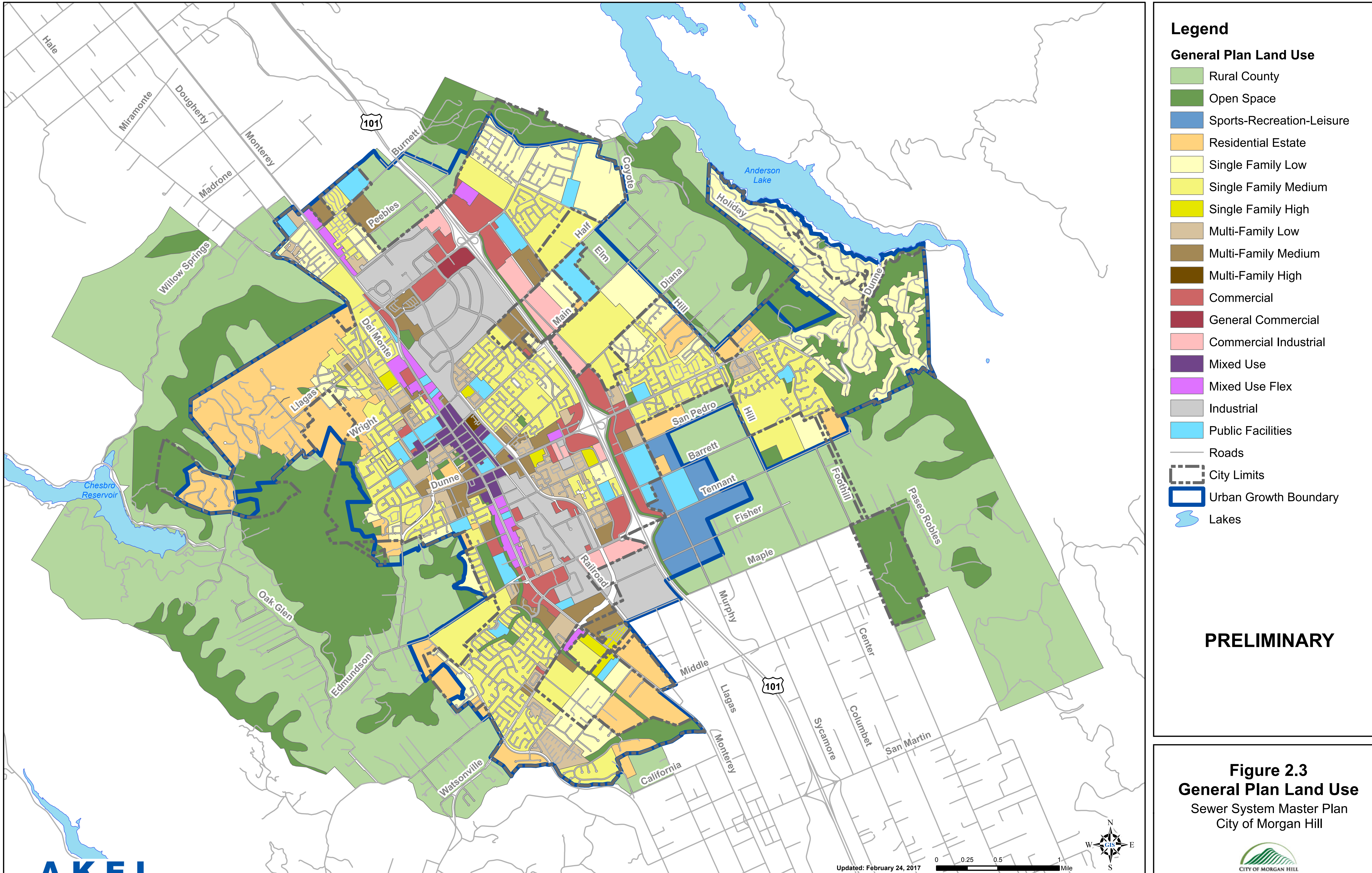

Figure 2.3
General Plan Land Use
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

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Table 2.2 Historical and Projected Population
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Year	Population[1]	Percent Growth (%)	Dwelling Units Added[2,3] (DU/year)
Historical			
2000	33,586	-	-
2001	33,914	1.0%	105
2002	34,210	0.9%	95
2003	34,109	-0.3%	-32
2004	34,618	1.5%	164
2005	35,011	1.1%	126
2006	35,535	1.5%	168
2007	36,467	2.6%	300
2008	37,107	1.8%	206
2009	37,653	1.5%	176
2010	37,882	0.6%	75
2011	38,456	1.5%	143
2012	39,432	2.5%	205
2013	40,486	2.7%	330
2014	41,562	2.7%	268
2015	42,382	2.0%	351
Projected			
2016 General Plan (RDCS Population Limit)			
	Population[2]		Dwelling Units Added (3.16 persons/DU) (DU/year)
2016	43,645	3.0%	275
2017	44,692	2.4%	275
2018	45,765	2.4%	275
2019	46,863	2.4%	275
2020	48,000	2.4%	275
2021	48,680	1.4%	215
2022	49,360	1.4%	215
2023	50,040	1.4%	215
2024	50,720	1.4%	215
2025	51,400	1.3%	215
2026	52,080	1.3%	215
2027	52,760	1.3%	215
2028	53,440	1.3%	215
2029	54,120	1.3%	215
2030	54,800	1.3%	215
2031	55,480	1.2%	215
2032	56,160	1.2%	215
2033	56,840	1.2%	215
2034	57,520	1.2%	215
2035	58,200	1.2%	215

Note: 10/26/2016

1. Historical Populations per California Department of Finance estimates.
2. Historical values received from City staff August 17, 2016.
3. People per dwelling unit at approximate historical averages.

CHAPTER 3 - SYSTEM PERFORMANCE AND DESIGN CRITERIA

This chapter presents the City's performance and design criteria, which were used in this master plan for evaluating the adequacy of capacity for the existing sanitary sewer system and for sizing improvements required to mitigate deficiencies and to accommodate future growth. The design criteria include: capacity requirements for the sanitary sewer facilities, flow calculation methodologies for future users, flow peaking factors, and accounting for infiltration and inflows.

3.1 HYDRAULIC CAPACITY CRITERIA

In addition to applying the City design standards for evaluating hydraulic capacities; this master plan included dynamic hydraulic modeling. The dynamic modeling was a critical and essential element in identifying surcharge conditions resulting from downstream bottlenecks in the gravity sewers.

3.1.1 Gravity Sewers

Gravity sewer capacities depend on several factors including: material and roughness of the pipe, the limiting velocity and slope, and the maximum allowable depth of flow. The hydraulic modeling software used for evaluating the capacity adequacy of the City's sewer system, InfoSWMM by Innovyze Inc., utilizes the fully dynamic St. Venant's equation which has a more accurate engine for simulating backwater and surcharge, in addition to manifolded force mains. The software also incorporates the use of the Manning Equation in other calculations including upstream pipe flow conditions.

Manning's Equation for Pipe Capacity

The Continuity equation and the Manning equation for steady-state flow are used for calculating pipe capacities in open channel flow. Open channel flow can consist of either open conduits or, in the case of gravity sewers, partially full closed conduits. Gravity full flow occurs when the conduit is flowing full but has not reached a pressure condition.

- Continuity Equation: $Q = V A$

  Where:
  Q = peak flow, in cubic feet per second (cfs)
  V = velocity, in feet per second (fps)
  A = cross-sectional area of pipe, in square feet (sq. ft.)

- Manning Equation: $V = (1.486\, R^{2/3}\, S^{1/2})/n$

  Where:
  V = velocity, fps
  n = Manning's roughness coefficient
  R = hydraulic radius (area divided by wetted perimeter), ft
  S = slope of pipe, in feet per foot

St. Venant's Equation for Pipe Capacity

Dynamic modeling facilitates the analysis of unsteady and non-uniform flows (dynamic flows) within a sewer system. Some hydraulic modeling programs have the ability to analyze these types of flows using the St. Venant equation, which take into account unsteady and non-uniform conditions that occur over changes in time and cross-section within system pipes.

The St. Venant equation is a set of two equations, a continuity equation and a dynamic equation, that are used to analyze dynamic flows within a system. The first equation, the continuity equation, relates the continuity of flow mass within the system pipes in terms of: (A) the change in the cross-sectional area of flow at a point over time and (B) The change of flow over the distance of piping in the system. The continuity equation is provided as follows:

- Continuity Equation: $$\underset{(A)}{\frac{\partial A}{\partial t}} + \underset{(B)}{\frac{\partial Q}{\partial x}} = 0$$

Where:
t = time
x = distance along the longitudinal direction of the channel
Q = discharge flow
A = flow cross-sectional area perpendicular to the x directional axis

The second equation, the dynamic equation, relates changes in flow to fluid momentum in the system using: (A) Changes in acceleration at a point over time, (B) Changes in convective flow acceleration, (C) Changes in momentum due to fluid pressure at a given point, (D) Changes in momentum from the friction slope of the pipe and (E) Fluid momentum provided by gravitational forces. The dynamic equation is provided as follows:

- Dynamic Equation: $$\underset{(A)}{\frac{\partial Q}{\partial t}} + \underset{(B)}{\frac{\partial}{\partial t}\left(\beta \frac{Q^2}{A}\right)} + \underset{(C)}{gA\frac{\partial y}{\partial x}} + \underset{(D)}{gAS_f} - \underset{(E)}{gAS_o} = 0$$

Where:
t = time
x = distance along the longitudinal direction of the channel
Q = discharge flow
A = flow cross-sectional area perpendicular to the x directional axis
y = flow depth measured from the channel bottom and normal to the x directional axis
S_f = friction slope
S_o = channel slope
β = momentum
g = gravitational acceleration

Use of this method of analysis provides a more accurate and precise analysis of flow conditions within the system compared to steady state flow analysis methods. It must be noted that two assumptions are made for use of St. Venant equations in the modeling software. First, flow is one dimensional. This means it is only necessary to consider velocities in the downstream direction and not in the transverse or vertical directions. Second, the flow is gradually varied. This means the vertical pressure distribution increases linearly with depth within the pipe.

Manning's Roughness Coefficient (n)

The Manning roughness coefficient 'n' is a friction coefficient that is used in the Manning formula for flow calculation in open channel flow. In sewer systems, the coefficient can vary between 0.009 and 0.017 depending on pipe material, size of pipe, depth of flow, root intrusion, smoothness of joints, and other factors.

For the purpose of this evaluation, and in accordance with City standards, an "n" value of 0.013 was used for both existing and proposed gravity sewer pipes unless directed otherwise by City staff based on pipe structural condition. This "n" value is an acceptable practice in planning studies.

Partial Flow Criteria (d/D)

Partial flow in gravity sewers is expressed as a depth of flow to pipe diameter ratio (d/D). For circular gravity conduits, the highest capacity is generally reached at 92 percent of the full height of the pipe (d/D ratio of 0.92). This is due to the additional wetted perimeter and increased friction of a gravity pipe.

When designing sewer pipelines, it is common practice to use variable flow depth criteria that allow higher safety factors in larger sizes. Thus, design d/D ratios may range between 0.5 and 0.92, with the lower values used for smaller pipes. The smaller pipes may experience flow peaks greater than planned or may experience blockages from debris. The City's design standards pertaining to the d/D criteria are summarized on **Table 3.1**.

During peak dry weather flows (PDWF), the maximum allowable d/D ratio for proposed pipes (all diameters) is 0.75. The maximum allowable d/D ratio for all existing pipes (all diameters) is 0.90. The criterion for existing pipes is relaxed in order to maximize the use of the existing pipes before costly pipes improvements are required.

During peak wet weather flows (PWWF), to avoid premature or unnecessary trunk line replacements, the capacity analysis allowed the d/D ratio to exceed the dry weather flow criteria and surcharge. This condition is evaluated using the dynamic hydraulic model and the criteria listed on **Table 3.1**, which stipulates that the hydraulic grade line (HGL), even during a surcharged condition, should be at least three feet below the manhole rim elevation.

Table 3.1 Sewer System Performance and Design Criteria

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Pipeline Criteria

Peak Dry Weather Flow Criteria

Diameter (in)	Maximum Allowable d/D	
	Existing Trunks	Proposed Trunks
8 to 12	0.90	0.75
> 12	0.90	0.75

Peak Wet Weather Flow Criteria

Hydraulic Grade Line (HGL) should be at least **3 foot** below the manhole rim

Pipe Size	Minimum Grade	Capacity (n = 0.013)	
(in)	(ft/ft)	(mgd)	(cfs)
8	0.0026	0.36	0.55
10	0.0019	0.56	0.87
12	0.0015	0.79	1.23
15	0.0011	1.28	1.98
18	0.0009	1.78	2.75
21	0.0007	2.43	3.76
24	0.0006	3.27	5.05
27	0.0005	4.18	6.47
30	0.0004	5.13	7.94
33	0.0004	6.04	9.34
36	0.0004	7.61	11.78
42	0.0003	10.27	15.90

Minimum Pipe Sizes and Design Velocities

In order to minimize the settlement of sewage solids, it is standard practice in the design of gravity sewers to specify that a minimum velocity of 2 feet per second (fps) be maintained when the pipeline is half-full. At this velocity, the sewer flow will typically result with self-cleaning of the pipe.

Due to the hydraulics of a circular conduit, velocity of half-full flows approaches the velocity of nearly full flows. **Table 3.1** lists the minimum slopes, varying by pipe size, in accordance with the City's design standards. The design standards also specify minimum pipe sizes, depending on the peak dry weather flows, as shown on **Table 3.1**.

Changes in Pipe Size

When a smaller gravity sewer pipe joins a larger pipe, the invert of the larger pipe is generally to maintain the same energy gradient. One of the methods used to approximate this condition includes placing the 80 percent depth point (d/D at 0.8) from both sewers at the same elevation. For master planning purposes, and in the absence of known field data, sewer crowns were matched at the manholes.

3.1.2 Force Mains and Lift Stations

The Hazen-Williams formula is commonly used for the design of force mains as follows:

- Hazen Williams Velocity Equation: $V = 1.32\, C\, R^{0.63}\, S^{0.54}$

Where:
V = mean velocity, fps
C = roughness coefficient
R = hydraulic radius, ft
S = slope of the energy grade line, ft/ft

The value of the Hazen-Williams 'C' varies and depends on the pipe material and is also influenced by the type of construction and pipe age. A 'C' value of 110 was used in this analysis.

The minimum recommended velocity in force mains is at 2 feet per second. The economical pumping velocity in force mains ranges between 3 and 5 fps. A maximum desired velocity is typically around 7 fps and a maximum not-to-exceed velocity is at 10 fps.

The capacities of pump stations are evaluated and designed to meet the peak wet weather flows with one standby pump having a capacity equal to the largest operating unit. The standby pump provides a safety factor in case the duty pump malfunctions during operations and allows for maintenance.

3.2 DRY WEATHER FLOW CRITERIA

Sewer unit flow factors are coefficients commonly used in planning level analysis to estimate future average daily sewer flows for areas with predetermined land uses. The unit factors are multiplied by the number of dwelling units or gross acreages for residential categories, and by the gross acreages for non-residential categories, to yield the average daily sewer flow projections.

3.2.1 Unit Flow Factors Methodology

Sewer unit factors are developed by using water consumption records and applying a return to sewer ratio for each land use to estimate sewer flow coefficients. There are several methods for developing the unit factors. This analysis relied on the use of the City's water consumption billing records, and the Public Water System Statistics (PWSS) report, which lists the monthly water consumption per customer account, by land use type, to estimate the unit factors within the service area.

3.2.2 Average Daily Wastewater Unit Flow Factors

Wastewater flow factors were based on water demands as extracted from the City's water consumption billing records. A return to sewer ratio was applied to each unadjusted water demand factor for individual land uses, and sewer flows were balanced to wastewater treatment plant flows. Generally, non-residential land uses return the majority of the water demand to the sewer system. These unit factors were estimated at ranging from 45 percent to 75 percent return to sewer ratio. The same concept can be applied to multi-family residential lots, which were estimated at ranging from 25 percent to 80 percent return to sewer ratio. Single family residential lots often have the lowest return to sewer ratio. This is largely due to water lost for landscape irrigation. Single family lots were estimated ranged from 25 percent to 40 percent return to sewer ratio. Lastly, unit factors were adjusted to 100 percent occupancy, and rounded.

This analysis generally indicates that existing residential land uses have higher flow generation factors than that of non-residential land uses. The existing unit factor analysis is shown on **Table 3.2**.

3.2.3 Peaking Factors

The sanitary sewer system is evaluated based on its ability to convey peak sewer flows. Peaking factors represent the increase in sewer flows experienced above the average dry weather flows (ADWF). The various peaking conditions are numerical values obtained from a review of historical data and, at times, tempered by engineering judgment.

The peaking conditions that are significant to hydraulic analysis of the sewer system include:

- peak dry weather flows (PDWF)
- peak wet weather flows (PWWF)

Typical values for peaking factors of 2.0 or less are generally used to estimate peak flows at treatment facilities where flow fluctuations are smoothed out during the time of travel in the sewer, while peaking factors between 3.0 and 4.0 are used to estimate peak flows in the smaller upstream areas of the system where low flow conditions are prone to greater fluctuations.

Table 3.2 Sewer Unit Flow Factor Analysis
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Land Use Classification	Development with City Limits		2012 Average Daily Water Demand Unit Factors: 2012 Consumption				2012 Production		2012 Production at 100% Occupancy				2012 Water Unit Factor		Revised 2016 Unit Factor		Average Daily Wastewater Flow Unit Factors	2012 Wastewater Flows		2012 Wastewater Flows at 100% Occupancy		2012 Wastewater Unit Factor		Revised 2016 Wastewater Unit Factor	
	Number of D.U.[1]	Existing	Annual Consumption	Unadjusted Water Unit Factors		Balance to 2012 Consumption	Unaccounted-For-Water Rate	Production (w/o Vacancy Rate)	Vacancy Rate[1]	Projected Production at 100% Occupancy			Recommended Water Unit Factor	Balance Using Recommended Unit Factor	Recommended Factor	Balance Using Recommended Unit Factor	Return to Sewer Ratio	Unadjusted Wastewater Unit Factor	Balance to Existing Conditions	Projected Flows at 100% Occupancy		Recommended Factor	Balance Using Recommended Unit Factors	Recommended Factor	Balance Using Recommended Unit Factors
		(net acre)	(gpd)	(gpd/DU)	(gpd/net acre)	(gpd)	(%)	(gpd)	(%)	(gpd/DU)	(gpd/net acre)	(gpd)	(gpd/net acre)	(gpd)	(gpd/net acre)	(gpd)		(gpd/net acre)	(gpd)	(gpd/net acre)	(gpd)	(gpd/net acre)	(gpd)	(gpd/net acre)	(gpd)
Residential																									
Residential Estate		508	320,127		630	320,127	12%	359,351	1.0%		714	362,553	700	355,525	560	284,420	25%	158	80,032	159	80,832	160	81,263	150	76,184
Residential Detached Low		979	1,124,021		1,148	1,124,021	12%	1,261,743	2.8%		1,320	1,293,216	1,325	1,297,793	1,050	1,028,440	30%	344	337,206	354	346,648	360	352,608	340	333,019
Residential Detached Medium		1,252	2,297,438		1,834	2,297,438	12%	2,578,934	6.3%		2,175	2,723,672	2,150	2,692,714	1,700	2,129,123	35%	642	804,103	682	854,762	675	845,387	630	789,028
Residential Detached High		30	0		0	0	12%	0	50.0%		0	0	2,140	64,644	2,140	64,644	40%	0	0	0	0	900	27,187	840	25,374
Subtotal Single Family Residential	10,672	2,770	3,741,586	351	1,351	3,741,586	12%	4,200,028	4.6%	410	1,578	4,370,914	1,592	4,410,677	1,266	3,506,627			1,221,341		1,282,242		1,306,445		1,223,605
Residential Attached Low		340	694,577		2,041	694,577	12%	779,681	4.3%		2,379	809,548	2,400	816,612	1,900	646,484	55%	1,123	382,017	1,171	398,444	1,175	399,800	1,100	374,280
Residential Attached Medium		100	333,687		3,351	333,687	12%	374,573	3.6%		3,882	386,586	2,900	288,788	2,300	229,039	70%	2,346	233,581	2,430	241,990	1,825	181,737	1,700	169,290
Residential Attached High		0.8	1,008		1,260	1,008	12%	1,132	95.0%		2,612	2,090	3,950	3,160	3,130	2,504	80%	1,008	807	1,966	1,573	3,150	2,520	2,930	2,344
Subtotal Multi-Family Residential	2,101	473	1,029,273	490	2,174	1,029,273	12%	1,155,386	4.0%	570	2,527	1,196,638	2,341	1,108,560	1,854	878,027			616,405		642,007		584,057		545,914
Non-Residential																									
General Commercial		24	43,123		1,798	43,123	12%	48,407	14.3%		2,276	54,574	2,275	54,550	1,800	43,161	70%	1,259	30,186	1,439	34,503	1,440	34,529	1,340	32,131
Commercial		260	351,322		1,354	351,322	12%	394,368	14.3%		1,713	444,607	1,700	441,151	1,350	350,326	70%	948	245,925	1,083	281,093	1,070	277,666	1,000	259,501
Commercial / Industrial[2]		501	546,049		1,090	546,049	12%	611,575	14.3%		1,376	689,660	1,410	706,631	1,120	561,296	75%	817	409,537	934	468,101	960	481,110	900	451,041
Mixed Use		93	120,928		1,296	120,928	12%	135,745	14.3%		1,640	153,037	1,700	158,655	1,350	125,991	65%	842	78,603	963	89,843	1,030	96,126	960	89,594
Mixed Use Flex		64	90,401		1,412	90,401	12%	101,478	14.3%		1,787	114,405	1,750	112,008	1,390	88,967	60%	847	54,241	969	61,997	960	61,445	900	57,604
Public Facility		302	134,428		446	134,428	12%	150,899	14.3%		564	170,123	500	112,008	400	120,657	45%	201	60,493	229	69,143	230	69,378	220	66,362
Subtotal Non-Residential		1,244	1,286,252		1,034	1,286,252	12%	1,442,473	14.3%		1,309	1,626,407	1,320	112,008	1,038	1,290,397			878,986		1,004,680		1,020,254		956,232
Other (Non-Flow Generating)																									
Landscape Irrigation		201	378,727		1,881	378,727	12%	425,131	0.0%		2,111	425,131	2,125	427,863	1,680	338,263	0%								
Open Space		605	0		0	0	0%	0			0	0	0	0	0	0	0%								
	12,773	5,293	6,435,839			6,435,839		**7,223,018**				7,619,089		**7,570,917**		**6,013,315**			**2,716,732**		**2,928,930**		**2,910,755**		**2,725,751**

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9/19/2016

Notes:
1. Source: Dwelling Unit counts and Residential Vacany rates US Census Bureau American Community Survey.
2. "Commercial / Industrial" combines land use types "Commercial / Institutional" and "Industrial"

Table 3.3 Average Daily Sewer Unit Flow Factors

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Land Use Classifications	Recommended Factor (gpd/net acre)
Residential	
Single Family	
Residential Estate	150
Residential Detached Low	340
Residential Detached Medium	630
Residential Detached High	840
Multi-Family	
Residential Attached Low	1,100
Residential Attached Medium	1,700
Residential Attached High	2,930
Non-Residential	
General Commercial	1,340
Commercial	1,000
Commercial / Industrial[1]	900
Mixed Use	960
Mixed Use Flex	900
Public Facility	220

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Note:

9/10/2016

1. "Commercial / Industrial" combines land use types "Commercial / Institutional" and "Industrial"

The City's 2002 master plan included a diurnal curve that was used for peaking dry weather flows. Similarly, this master plan used 24-hour diurnal patterns for dry weather flows tributary to each flow monitor, as shown on **Figure 3.1, Figure 3.2** and **Figure 3.3**.

3.3 WET WEATHER FLOW CRITERIA

The wet weather flow criteria accounts for the infiltration and inflows (I&I) that seep into the City's sewer system during storm events.

3.3.1 Infiltration and Inflow

Groundwater infiltration and inflow is associated with extraneous water entering the sewer through defects in pipelines and manholes. Infiltration occurs when groundwater rises or the soil is saturated due to seasonal factors such as a storm event which causes an increase in flows in the sewer system. The ground water will enter the sewer system through cracks in the pipes or deteriorating manholes. Inflow occurs when surface water enters the wastewater collection system from storm drain cross connections, manhole covers, or roof/footing drains. **Figure 3.4** was developed by King County, Washington and was included in this chapter to illustrate the typical causes of infiltration and inflow.

There are several accepted methodologies for estimating infiltration and inflows (I&I). These include:

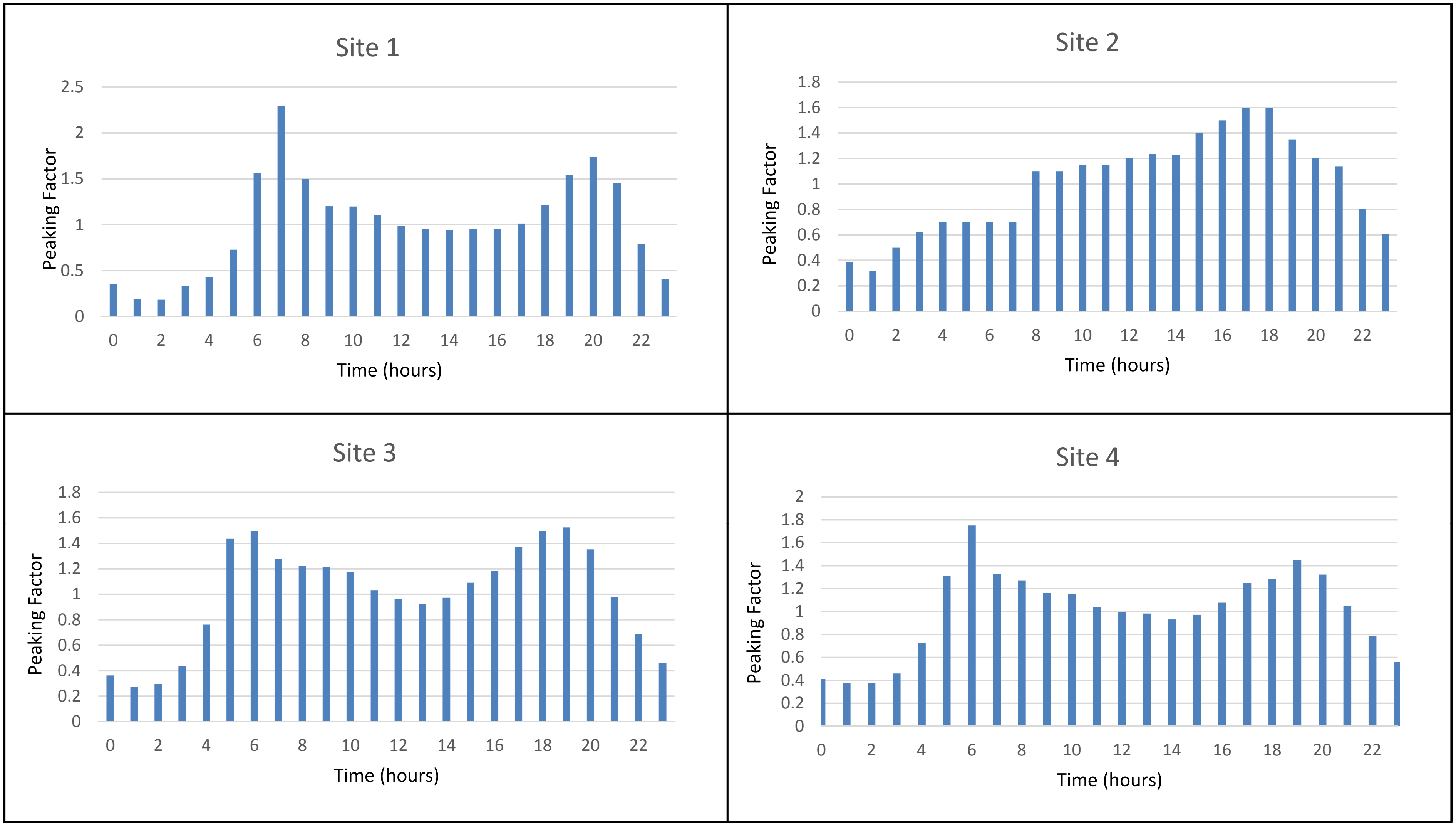
- **Methodology 1.** Based on Acreages. In this methodology, factors that may range between 400 and 1,500 gallons per day (gpd) or more are applied to acreages for estimating the I&I component.

- **Methodology 2.** Based on Linear Feet of Pipe. In this methodology, factors that may range between 12 and 30 or more gallons per day per inch diameter per 100 linear feet (gpd/inch diameter/100LF) are applied to linear feet of gravity sewers.

- **Methodology 3**. Based on a percentage of Average Dry Weather Flows. In this methodology, Infiltration and Inflows (I&I) are calculated based on a percentage of the average dry weather flow.

- **Methodology 4**. Based on flow monitoring data. In this methodology, infiltration and inflows are determined by analyzing flow monitoring data of current and past flow monitoring efforts.

This capacity analysis and master plan based the infiltration and inflow on specific flow monitoring data from the Villalobos and Associates (V&A) 2014 Flow Monitoring Program (**Appendix A**). Thus, the infiltration and inflows are reasonable and reflect the actual behavior of the sanitary sewer system.



LEGEND

Peaking Factor

PRELIMINARY

October 27, 2016

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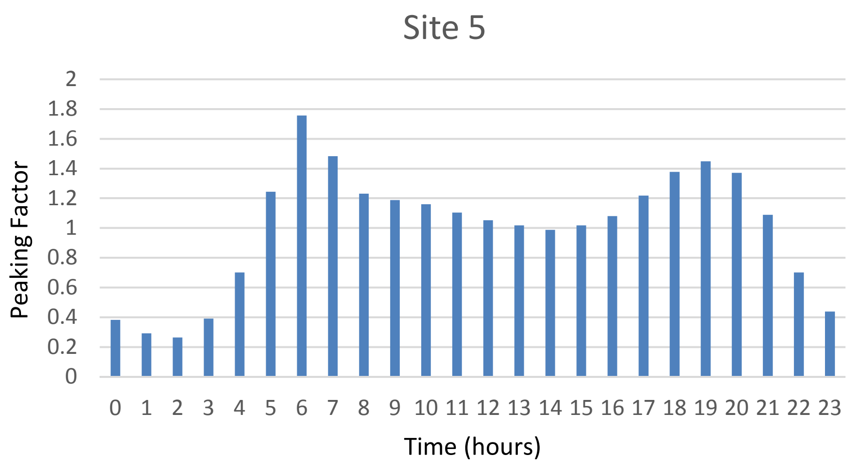
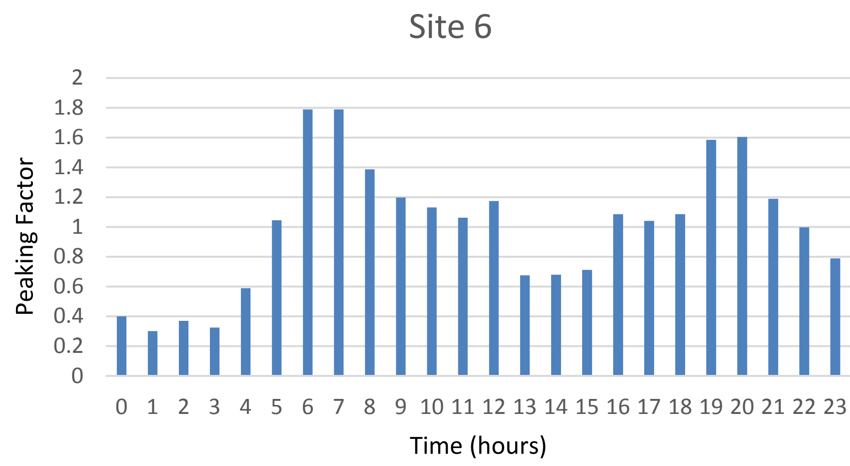
Figure 3.1

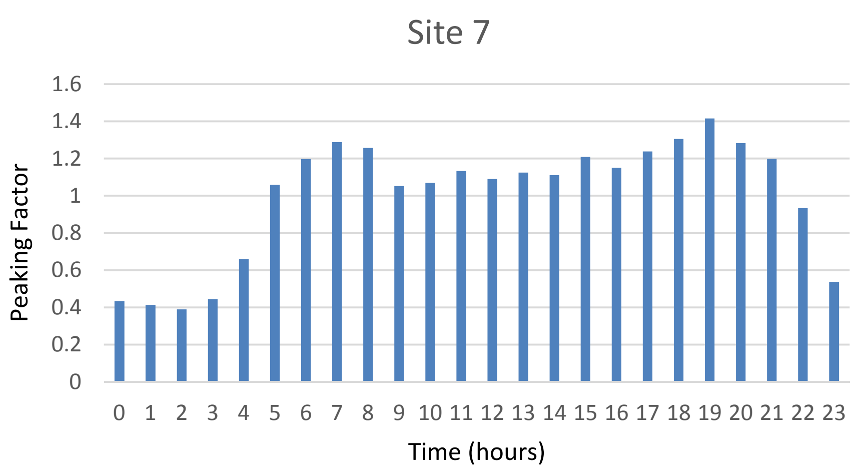
Hydraulic Model Diurnals

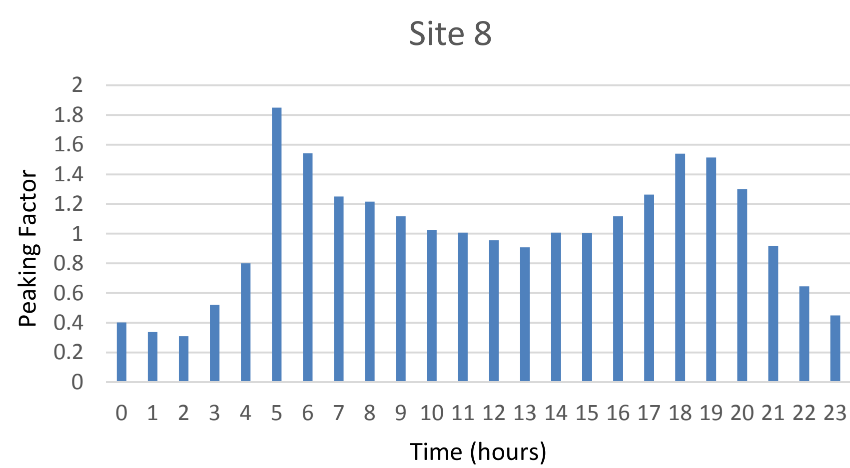
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL









LEGEND

Peaking Factor

PRELIMINARY

October 27, 2016

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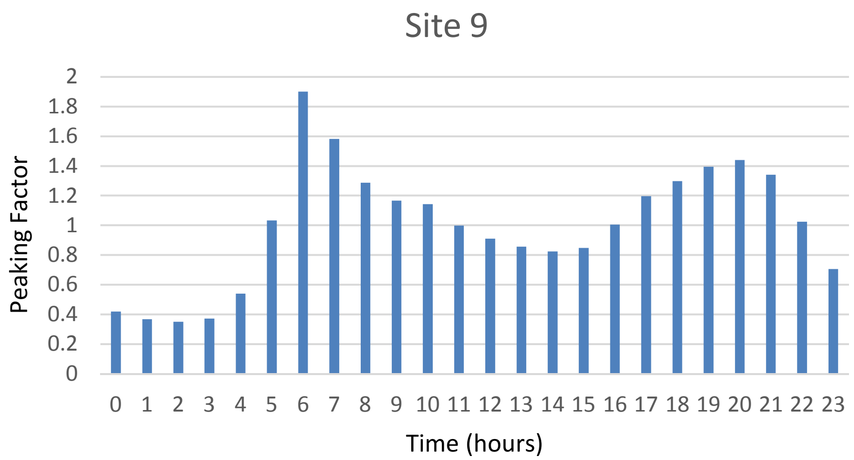
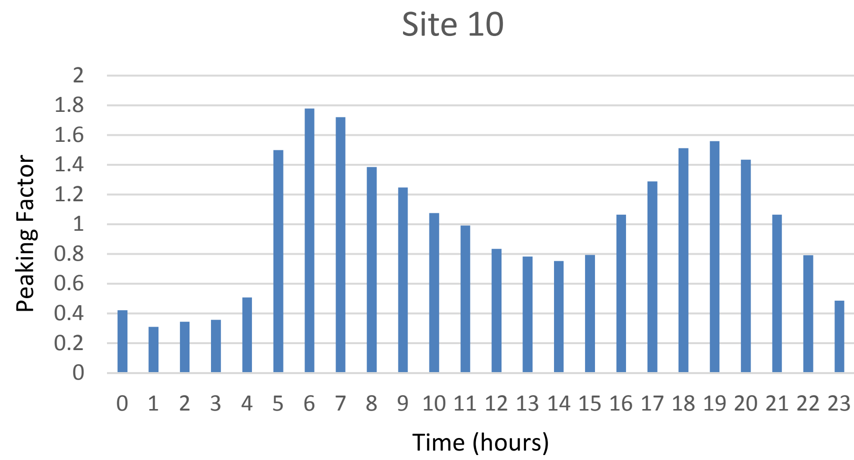
Figure 3.2

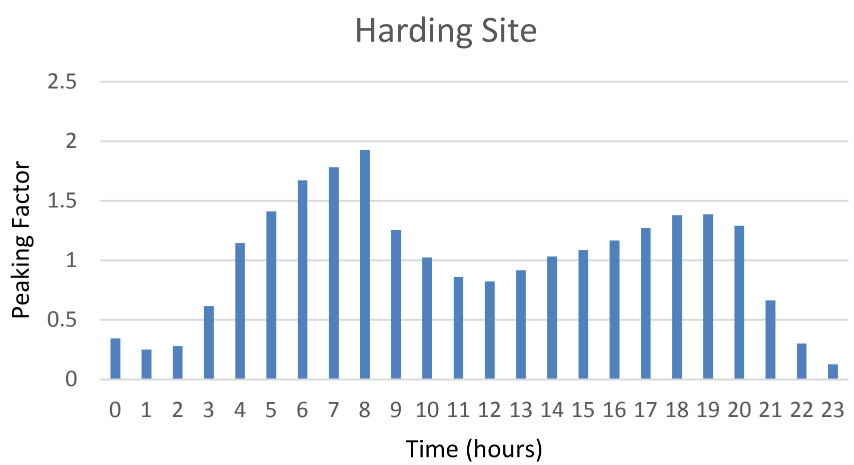
Hydraulic Model Diurnals

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL







LEGEND

Peaking Factor

PRELIMINARY

October 27, 2016

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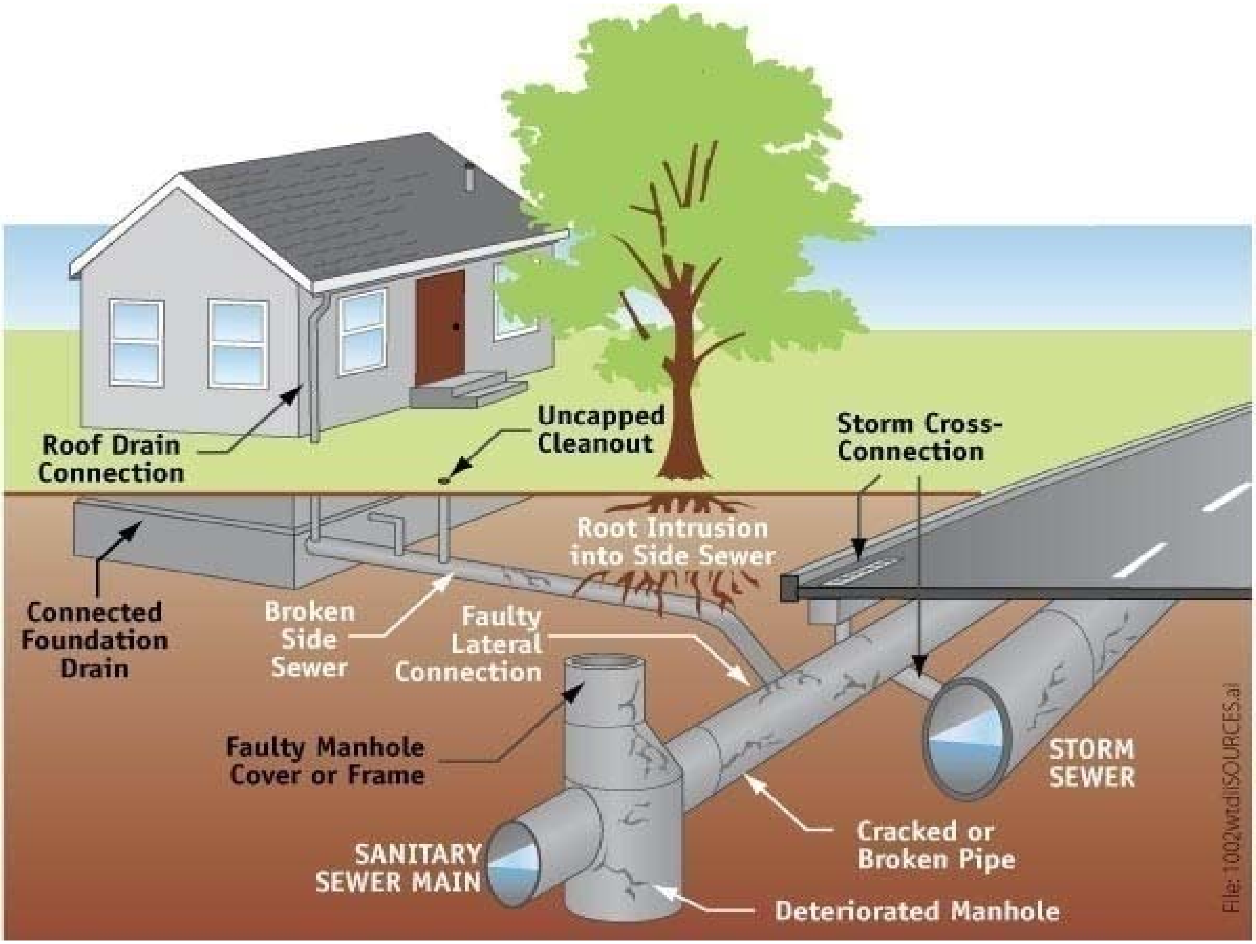
Figure 3.3

Hydraulic Model Diurnals

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL



Source: King County, WA
http://www.kingcounty.gov/environment/wastewater/II/What.aspx?print=1

LEGEND

Inflow Sources (Black Text)

Infiltration Sources (White Text)

PRELIMINARY

Figure 3.4
Infiltration and Inflow Sources

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

February 05, 2016

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3.3.2 Sewer System Flow Monitoring

In 2014 V&A's services were used for a temporary flow monitoring program to capture 10 sites during dry and wet weather flows. In addition to temporary flow monitors, flow monitored data from Harding meter and SCRWA was also incorporated into the analysis. The V&A flow monitored locations are shown on **Figure 3.5**.

The rain gauge data for the V&A flow monitoring period was obtained from V&A. There were three rain gauges used for the wet weather analysis. The V&A rain gauges, were located in the East, West and South portion of the City. The east rain gauge was located near Tennant Avenue and Foothill Avenue, the west rain gauge was located near West Dunne Avenue and Dewitt Avenue, and the south rain gauge was located near Santa Teresa Boulevard and West Middle Avenue. The flow monitoring and rain data was used in this analysis to calibrate the computer hydraulic model to average dry weather flow and wet weather flow conditions.

3.3.3 10-Year 24-Hour Design Storm

A synthetic design storm is typically used to evaluate the sewer collection system's response during wet weather flow conditions. The design storm information was collected from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Volume 6 (**Table 3.4**).

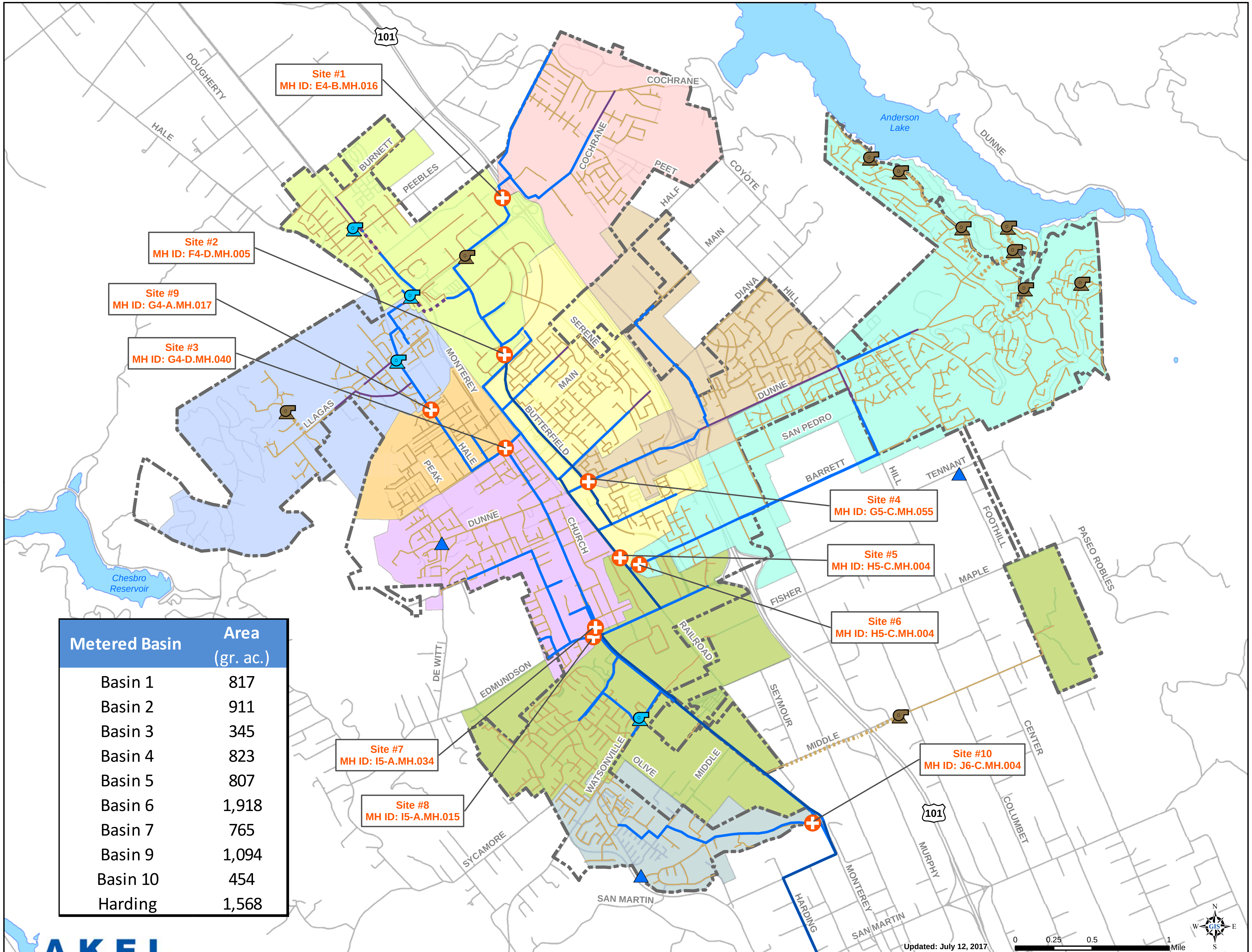
- **10-Year Frequency.** Industry standards include design storms that range between 5-year and 20-year events. Based on current regulatory trends, a 10-year storm event was chosen for the City to evaluate the capacity adequacy of the sanitary sewer system.

- **24-Hour Duration.** Peak flows from a storm event are usually cause by brief intense rains, that can happen as part of an individual event or as a portion of a larger storm. The 24-hour storm duration is longer than needed to determine peak flow but aids in identifying infiltration and inflows a sewer system may experience during a storm event.

- **Balanced Rainfall Centered Distribution.** The National Resources Conservation Service, previously known as the Soil Conservation Service, has developed rainfall distributions for wide geographic regions based on traditional Depth-Duration-Frequency (DDF) rainfall data. In this methodology, the highest rainfall intensity is placed at the center of the storm. Incrementally lower intensities are placed on alternating sides of the peak.

Thus, the NOAA Atlas 14 Depth Duration Frequency (DDF), 10-year 24-hour (10yr-24hr) design storm, with a balanced rainfall distribution, was used to evaluate the capacity adequacy of the City's sanitary sewer system during wet weather flow conditions.

The selected 10-year 24-hour design storm was further compared to historical storm events, between February 2014 and March 2014, as shown on **Table 3.5**. The table lists the total rainfall volume, duration, peak hour intensity, and total monthly rainfall (if available) for each storm event.



Metered Basin	Area (gr. ac.)
Basin 1	817
Basin 2	911
Basin 3	345
Basin 4	823
Basin 5	807
Basin 6	1,918
Basin 7	765
Basin 9	1,094
Basin 10	454
Harding	1,568

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Legend

Rain Gauge
Flow Meters

Flow Meter Basins
- Basin 1
- Basin 2
- Basin 3
- Basin 4
- Basin 5
- Basin 6
- Basin 7
- Basin 9
- Basin 10
- Harding Basin

Modeled System

Lift Stations

Gravity Pipes by Size
- 8"
- 10" - 21"
- 24" - 30"

Force Mains
- 8" or Smaller

Non-Modeled System

- Lift Stations
- Gravity Pipes
- Force Mains
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 3.5
Flow Meter Locations
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

Table 3.4 Precipitation Depth-Duration-Frequency

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Duration	2-Year		5-Year		10-Year		25-Year		100-Year	
	(in)	(in/hr)	(in)	(in/hr)	(in)	(in/hr)	(in)	(in/hr)	(in)	(in/hr)
5-min	0.15	1.80	0.20	2.40	0.23	2.76	0.29	3.48	0.37	4.44
10-min	0.22	1.32	0.28	1.68	0.34	2.04	0.41	2.46	0.53	3.18
15-min	0.26	1.04	0.34	1.36	0.41	1.64	0.50	2.00	0.65	2.60
30-min	0.36	0.72	0.47	0.94	0.56	1.12	0.69	1.38	0.90	1.80
1-hr	0.55	0.55	0.71	0.71	0.85	0.85	1.04	1.04	1.35	1.35
2-hr	0.85	0.43	1.10	0.55	1.31	0.66	1.61	0.81	2.09	1.05
3-hr	1.07	0.36	1.39	0.46	1.65	0.55	2.02	0.67	2.62	0.87
6-hr	1.53	0.26	1.98	0.33	2.37	0.40	2.90	0.48	3.78	0.63
12-hr	2.06	0.17	2.69	0.22	3.22	0.27	3.97	0.33	5.22	0.44
24-hr	2.69	0.11	3.51	0.15	4.22	0.18	5.22	0.22	6.91	0.29

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Note:

2/11/2016

1. Source: NOAA Atlas 14 Volume 6 version 2 for station MORGAN HILL 2E.

Table 3.5 Storm Events Analysis

Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Storm Event	Estimated Return Interval	Single Rainfall Event Volume and Intensity	
		Volume (in)	Peak Intensity (in//hr)
February 26- February 27, 2014	< 1-year	1.38	0.24
February 28 - March 1, 2014	2-Year 6 Hour	2.04	0.45
Design Storm	10-Year 24-Hour	4.22	0.84

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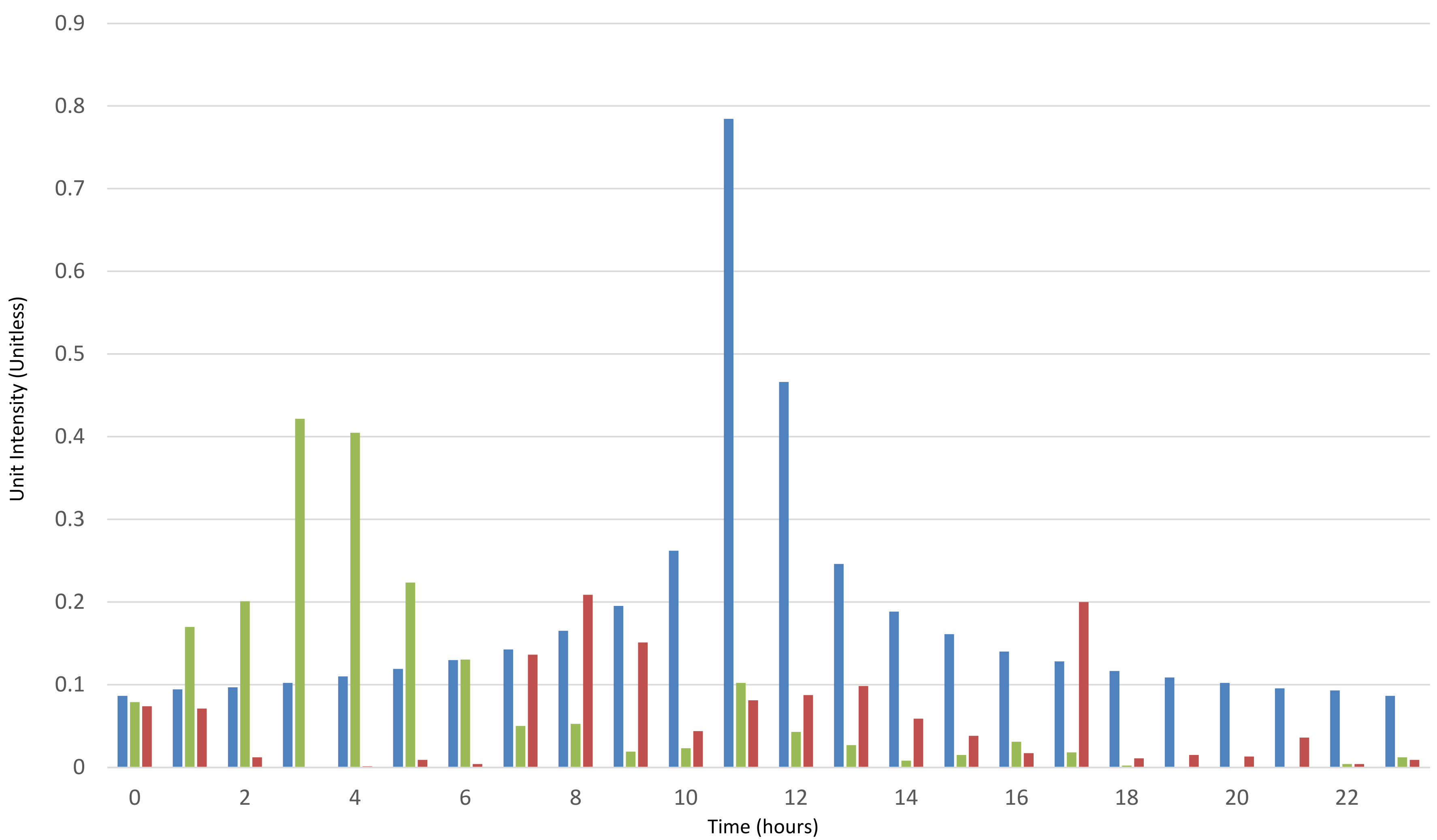
5/5/2017

Figure 3.6 is intended to show the diurnal comparison between the design storm and the two storm events experienced during February of 2014. The comparison indicates that, based on the balanced centered hyetograph, the design storm's peak hour value is at 0.78 inches per hour (in/hr), while the February 26th and 28th storms peak values are 0.24 and 0.45 in/hr respectively. This comparison illustrates the more conservative nature of the design storm and the relatively small storm events experienced in February 2014.



LEGEND

PRELIMINARY

Design Storm: 10 Year - 24 Hour (4.22 in)

Historical Storm Event : February 28, 2014 (2.04 in)

Historical Storm Event: February 28, 2014 (1.38 in)

June 5, 2017

Figure 3.6

10-Year 24-Hour Storm
(Design vs Historical Storm)

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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CHAPTER 4 - EXISTING SEWER COLLECTION FACILITIES

This chapter provides a description of the City's existing sewer system facilities including gravity trunks, force mains, lift stations, and sewer collection basins. The chapter also includes a brief description of the SCRWA Wastewater Treatment Plant.

4.1 SEWER COLLECTION SYSTEM OVERVIEW

The City provides sewer collection services to approximately 10,000 residential, commercial, industrial, and institutional accounts. The City's modeled collection system consists of approximately 160 miles of up to 30-inch gravity sewer pipes that convey flows, via the Morgan Hill-Gilroy Joint Trunk, towards the South County Regional Water Authority WWTP, on Southside Drive in the City of Gilroy, as shown on **Figure 4.1**.

A system-wide pipe inventory, listing the total length by pipe diameter, is shown on **Table 4.1**. This table is based on information extracted from the City's GIS. The 8-inch to 15-inch diameter pipes account for 55 percent of the total sewer pipe lengths.

4.2 SEWER COLLECTION BASINS AND TRUNKS

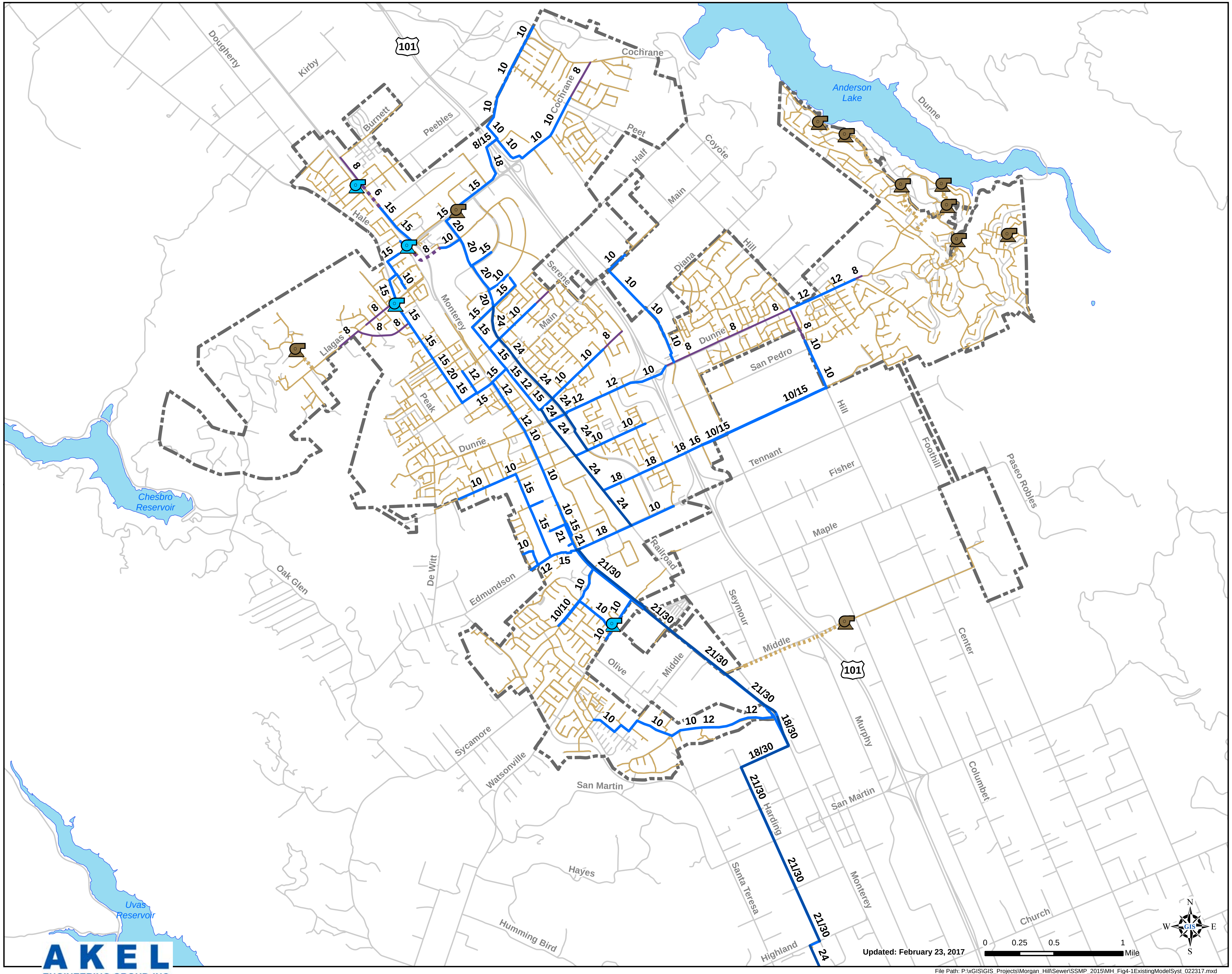
Due to topography, the sanitary sewer system is divided into five separate dendritic sewer collection basins, each defining the boundaries of a sewer collection trunk system. The following 6 major sewer collection basins were created and shown on **Figure 4.2**: the Butterfield Trunk, the East Dunne Trunk, the Hale-Llagas Trunk, the Hale-Monterey Trunk, the Hill—Barrett Trunk, the Llagas Trunk and the Railroad-Monterey Trunk. The sewer trunk system for each collection basin is shown on **Figure 4.3**, and a schematic diagram intended to simplify the connectivity between the basins and trunks is shown on **Figure 4.4**.

4.2.1 Butterfield Trunk

This trunk starts at the intersection of Cochrane Road and Butterfield Boulevard as a 20-inch gravity main in a southbound direction. The main then follows Butterfield Boulevard before increasing to a 24-inch diameter main just south of the intersection of Jarvis Drive and Butterfield Boulevard, and continues south along Butterfield Boulevard. The Butterfield Trunk ends at the intersection of San Pedro Avenue where it turns west and discharges into the Railroad-Monterey Trunk.

4.2.2 Llagas Trunk

This trunk starts west of the intersection of Sanchez Drive and Monterey Road as 15-inch gravity main. The main continues westerly before reaching Del Monte Avenue where it turns south



Legend

Existing Modeled System

- Lift Stations

Modeled Gravity Pipes by Size

- 8"
- 10" - 21"
- 24" - 30"

Modeled Force Mains

- 8" or Smaller

Non-Modeled System

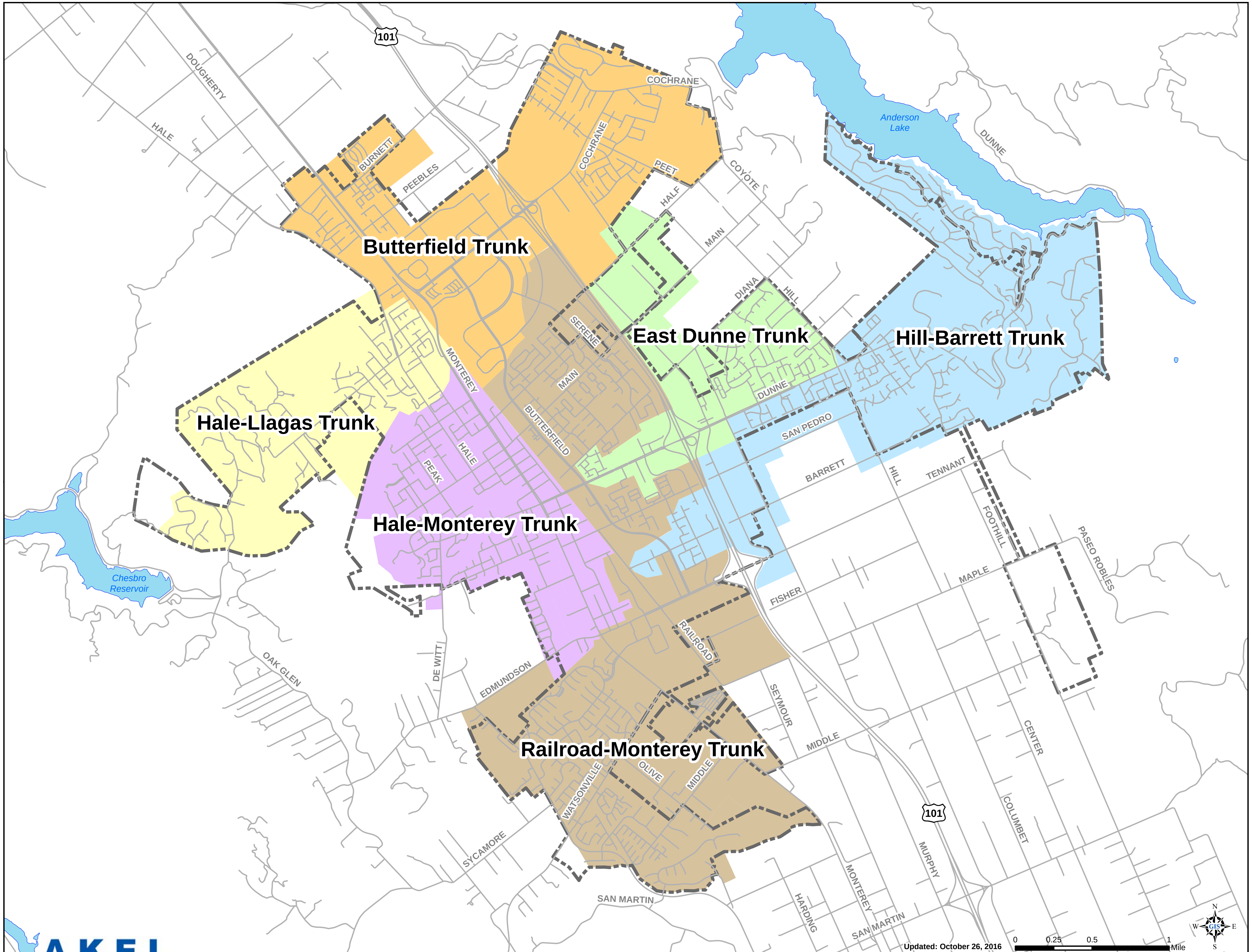
- Lift Stations
- Gravity Pipes
- Force Mains
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 4.1
Existing Modeled Sewer System

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL



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Legend

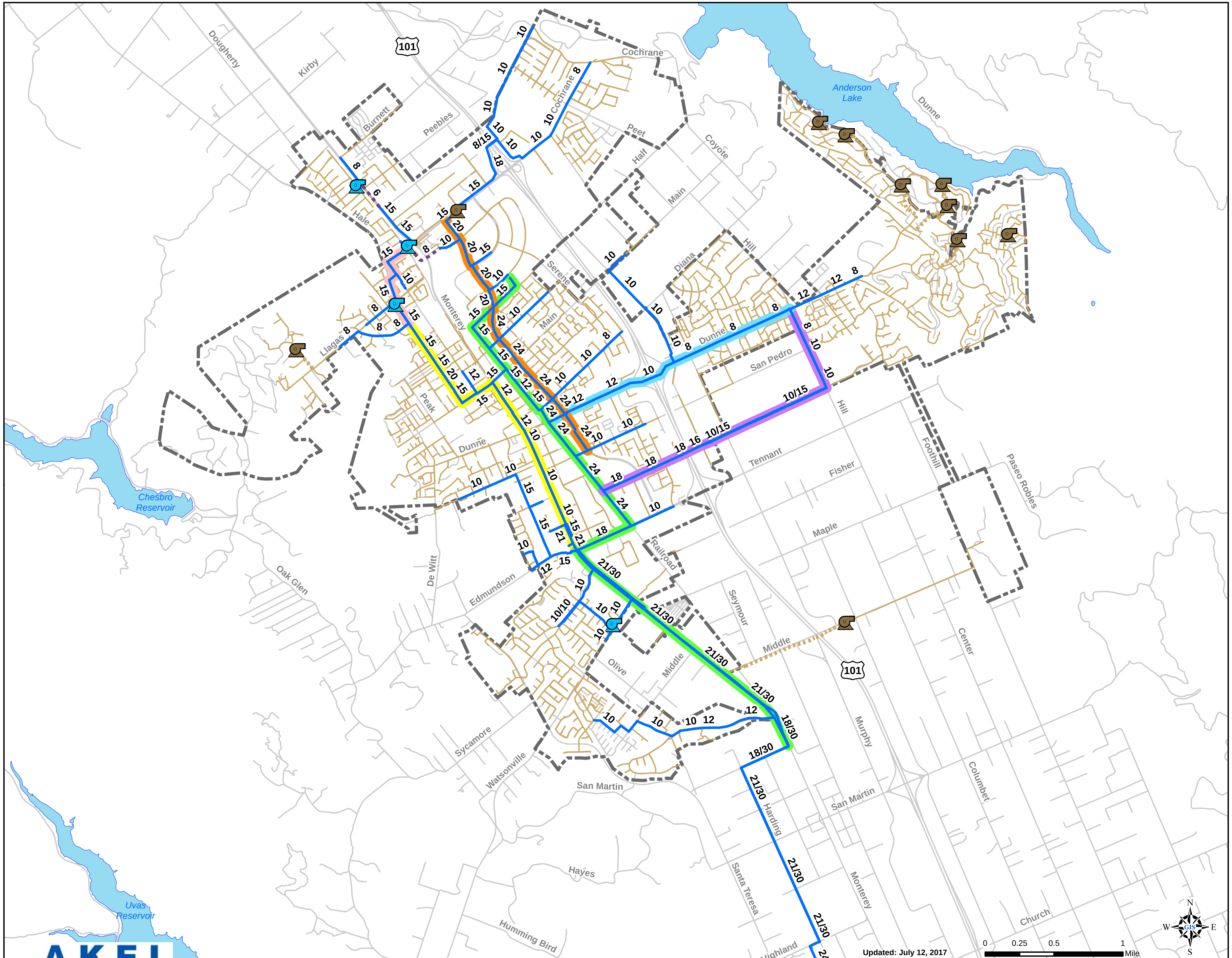
Collection Basins

- Butterfield Trunk
- East Dunne Trunk
- Hale-Llagas Trunk
- Hale-Monterey Trunk
- Hill-Barrett Trunk
- Railroad-Monterey Trunk
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 4.2
Existing Sewer Collection System Basins
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL



Legend

Modeled System

- Lift Stations
- Gravity Pipes
- Force Mains

Sewer Collection Trunks

- Butterfield Trunk
- East Dunne Trunk
- Hale-Monterey Trunk
- Hill-Barrett Trunk
- Llagas Trunk
- Railroad-Monterey Trunk

Non-Modeled System

- Lift Stations
- Gravity Pipes
- Force Mains
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 4.3
Existing Modeled Trunk System

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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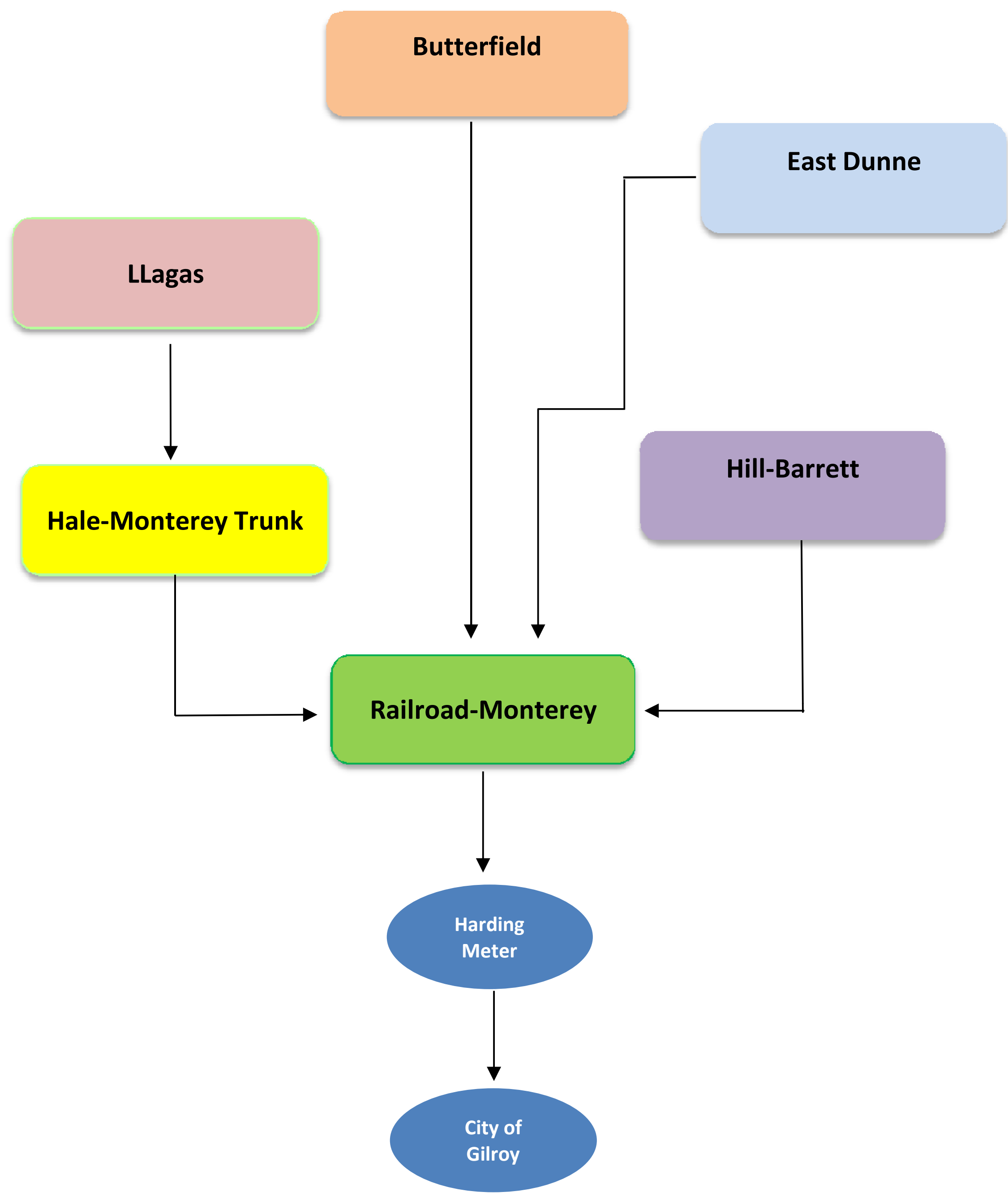
File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\MH_Fig4-3_SewerTrunks_022317.mxd



PRELIMINARY

LEGEND

Sewer Tributary Area

Sewer Trunk

October 25, 2016

Figure 4.4
Trunk Connectivity
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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Table 4.1 Existing GIS Pipe Inventory

Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Diameter (in)	Number of lines [1]	Pipe Length (miles) [1]	Portion of Sewer System [1] (%)
Gravity Main			
4	7	0.32	0.2%
6	1,487	56.03	34.8%
8	1,533	63.11	39.2%
10	355	16.44	10.2%
12	74	3.15	2.0%
14	3	0.11	0.1%
15	124	6.47	4.0%
16	1	0.09	0.1%
18	37	1.42	0.9%
20	14	0.70	0.4%
21	36	3.25	2.0%
24	34	2.35	1.5%
30	49	3.91	2.4%
Unknown	8	0.50	0.3%
SubTotal	3,762	157.87	98.2%
Force Main			
4	11	0.86	0.5%
6	10	1.03	0.6%
8	2	0.26	0.2%
12	9	0.79	0.5%
SubTotal	32	2.93	1.8%
Grand Total	**3,794**	160.80	**100%**

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Note:

5/12/2017

1. Information extracted from GIS shapefiles provided by City Staff on 01/25/17.

following Del Monte Avenue. The trunk then turns west, north of Berskhire Drive where it will continue towards the intersection of Llagas Road and Hale Avenue. At this intersection, the trunk turns south and follows Hale Avenue, until it reaches Christine Lynn Drive. At this intersection, the Llagas Trunk discharges into the Hale-Monterey Trunk.

4.2.3 Hale-Monterey Trunk

The trunk starts at the intersection of Hale Avenue and Christine Lynn Drive in a southerly direction following Hale Avenue as a 15-inch gravity main. The trunk increases into a 20-inch diameter main for a short section of pipeline 300 feet south of Longview Drive, and then decreases into a 15-inch diameter main. At the intersection of Main Avenue and Hale Avenue, the trunk turns east on Main Avenue as a 15-inch diameter main. When the trunk reaches the intersection of Main Avenue and Monterey Road, the trunk turns south and decreases in a 12-inch gravity main. The trunk follows the alignment of Monterey Road, and decreases in size into a 10-inch diameter main at the intersection of Dunne Avenue. From this intersection, the trunk continues south along Monterey Road. At the intersection of Edes Street, the trunk increases into a 21-inch diameter main, before consolidating into the Railroad-Monterey Trunk.

4.2.4 Hill-Barrett Trunk

This trunk starts south of the intersection of Dunne Avenue and Hill Avenue as an 8-inch gravity main. The main continues south following Hill Avenue before increasing to a 10-inch diameter main when it reaches San Pedro Avenue. The sewer main then turns west at Barrett Avenue, where it increases in size to a 10-inch sewer main in parallel with a 15-inch sewer main. The trunk then continues in a westerly direction and increases in size to an 18-inch at the intersection of Highway 101 and Barrett Avenue. Then trunk then ends at the intersection of Barrett Avenue and the Railroad where it consolidates into a 24-inch sewer main.

4.2.5 East Dunne Trunk

This trunk starts west at the intersection of Dunne Avenue and Hill Avenue as an 8-inch gravity main. The main continues westerly along Dunne Avenue before increasing to a 10-inch diameter main west of the intersection of Dunne Avenue and Highway 101. The trunk continues westerly along Dunne Avenue until it reaches Laurel Road, where it increases into a 12-inch diameter main. The East Dunne Trunk continues west and discharges into the Butterfield Trunk, at the intersection of Butterfield Boulevard.

4.2.6 Railroad-Monterey Trunk

This trunk starts south-east of the intersection of Butterfield Boulevard and Jarvis Drive as a 15-inch gravity main. The main continues westerly in an easement alignment before turning south at the railroad. The main continues along the railroad alignment before increasing to a 24-inch diameter main at Dunne Avenue, and continuing to Tennant Avenue. The sewer main turns west at Tennant Avenue, where is decreases in size to an 18-inch and continues to Monterey Road.

The trunk then turns south on Monterey Road as a parallel 21-inch and 30-inch to California Avenue, where it consolidates into the Joint Trunk sewer.

4.3 JOINT MORGAN HILL – GILROY SEWER TRUNK

The City of Morgan Hill's sewer flows are collected via the major trunk systems described in the previous sections, and ultimately flow is conveyed to the intersection of California Avenue and Monterey Road. At this point, flows enter the Joint Morgan Hill-Gilroy Sewer Trunk, and proceed west on California Avenue to Harding Avenue. From this intersection, flow continues south on Harding Avenue, jogging along Highland Avenue and easements in agricultural lands to the city of Gilroy at Day Road. Flow then jogs through the city of Gilroy and is ultimately discharged to the SCRWA WWTP.

The Joint Trunk is maintained by a Joint Exercise of Powers Agreement between the City of Gilroy and the City of Morgan Hill (Appendix A) and dated May 19th, 1992. This agreement establishes the formation of the South County Regional Wastewater Authority (SCRWA), and includes an exhibit that documents the pipeline capacities and the capacity allocation for each segment of the Joint Trunk. The agreement dictates that a 4.0 MGD capacity allocation exist in the Joint Trunk and a 7.5 MGD capacity reservation in the City of Gilroy trunk system to accommodate Morgan Hill's flows.

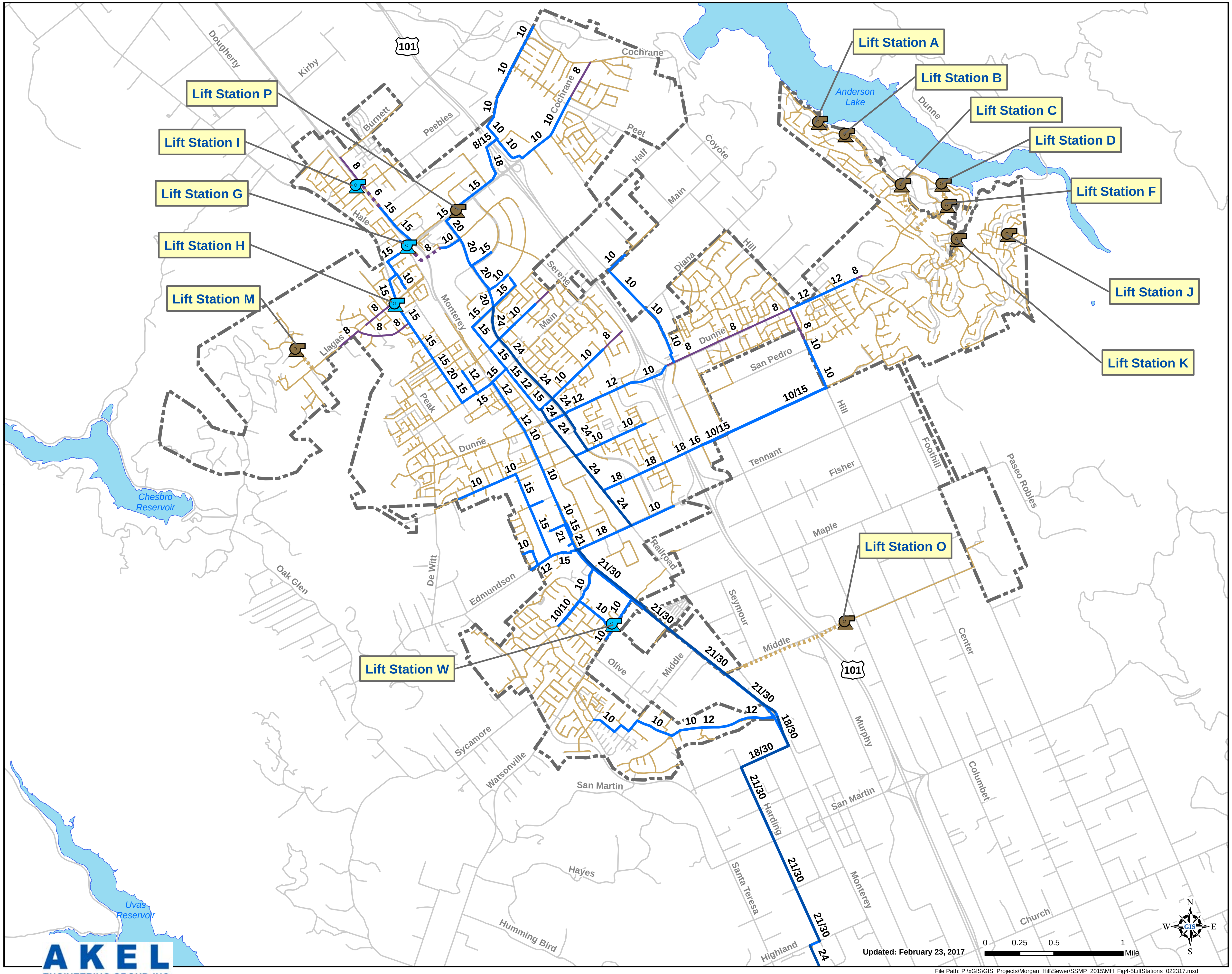
There have been several studies since the 2002 Sewer System Master Plan that evaluated the capacity adequacy of the existing Joint Trunk, and the need for construction of the Relief Trunk. The City of Morgan Hill has initiated construction of the Relief Trunk, and is currently in the design-build process to connect the Relief Trunk to the trunk system in the City of Gilroy.

4.4 LIFT STATIONS

When routing flows by gravity is not possible due to adverse grades, lift stations are used to pump flows. The City currently maintains fourteen lift stations in the sewer collection system, as summarized on **Table 4.2** and shown on **Figure 4.5**. **Table 4.2** also includes wet well dimensions, capacity, and holding time of each lift station. Only four of the fourteen lift stations were included in the hydraulic modeling effort, and due to their location on major trunk systems. The modeled lift stations are listed as follows:

- **Lift Station G:** This lift station services the area located along Old Monterey and along Sanchez Drive to the west. The lift station is located at 8615 Monterey Road. This lift station includes a duty pump that is rated at 800 gallons per minute (gpm). The pump discharges into an 8-inch force main that heads south along Butterfield Boulevard.

- **Lift Station H.** This lift station services the area that is along Llagas Road, east of Enderson Court, and west of Sanchez Drive. The lift station is located at 320 Llagas Road. The lift station includes a duty pump that is rated at 500 gpm. The pump discharges into a 4-inch force main that heads southeast along Hale Avenue.



Legend

Modeled System

Lift Stations

Modeled Gravity Pipes by Size

8"

10" - 21"

24" - 30"

Modeled Force Mains

8" or Smaller

Non-Modeled System

Lift Stations

Gravity Pipes

Force Mains

Roads

City Limits

Lakes

PRELIMINARY

Figure 4.5
Existing Lift Stations

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\MH_Fig4-5LiftStations_022317.mxd

Table 4.2 Existing System Lift Station Inventory

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Lift Station Information		Wet Well Dimensions		Capacity		Holding Time
Station	Location	Diameter (ft)	Depth (ft)	Per Tank (gal)	Total (gal)	(hrs)
Modeled Lift Stations						
G	8615 Monterey Road	10	16	9,400	9,400	
H	320 Llagas Road	5	18	2,644	2,644	0.5
I	19160 Saffron Drive	8	29	10,904	10,904	
W	15505 Watsonville Road	8	21	7,896	12,549	3.5
		6	22	4,653		
Non-Modeled Lift Stations						
A	17670 Racoon Court	8	11.5	4,324	9,024	14
		4	8	752		
		8	10.5	3,948		
B	17558 Holiday Drive	8	19	7,144	9,823	
		4	15	1,410		
		6	6	1,269		
C	3272 Quail Lane	12.5	10	9,180	12,118	1.5
		5	12	1,763		
		5	8	1,175		
D	17110-B Shady Lane	6	16	3,384	6,944	15.5
		6	11.5	2,432		
		4	12	1,128		
F	17109 Holiday Drive	8	18	6,768	14,847	
		6x6	10	2,693		
		6x6	10	2,693		
		6x6	10	2,693		
J	16035 Jackson Oaks Drive	6	15	3,173	4,865	
		4	10	940		
		4	8	752		
K	3300 East Dunne Avenue	8	12	4,512	4,512	13.5
M	1162 Llagas Road	4	12	1,128	1,128	
O	952 East Middle Avenue	5	15	2,203	2,203	
P	320 Woodview Avenue	4	19	1,786	1,786	14.25

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Notes:

1/7/2016

1. Source: Lift Station Information received from City of Morgan Hill Staff on 08/14/2014

- **Lift Station I.** This lift station services the area that is north of the intersection of Tarragon Avenue and Saffron Drive, yet bounded below U.S. State Highway 101. The lift station is located at 19160 Saffron Drive. The lift station includes a duty pump that is rated at 500 gpm. The pump discharges into a 6-inch force main that heads southeast along Monterey Road.

- **Lift Station W.** This lift station services the area that is along south of La Crosse Drive, south of La Jolla Drive, north of Watsonville Drive, and north of Calle Enrique. The lift station is located at 15505 Watsonville Drive. The lift station includes a duty pump that is rated at 500 gpm. The pump discharges into a 6-inch force main that heads into Watsonville Drive.

4.5 FLOW DIVERSIONS

The City's sewer collection system contains diversion structures that are intended to provide opportunities to route flow away from sewer trunks with capacity limitations to sewer pipelines that may have excess capacity. The City has two identified flow diversions that are included in the hydraulic model:

- **East Dunne Avenue Diversion.** The East Dunne Avenue diversion is located at the intersection of Hill Road and East Dunne Avenue. Flows from the foothills in the east near Anderson Lake flow west to the intersection of East Dunne Avenue and Hill Road, where City staff have the option to continue flows west down East Dunne Avenue, or to the south on Hill Road.

- **West Main Avenue Diversion.** The West Main Avenue diversion is located at the intersection of West Main Avenue and Monterey Road. Flow in the Hale-Monterey Trunk may be diverted at that intersection to the east to the Railroad-Monterey Trunk or continue south down Monterey Road.

4.6 SOUTH COUNTY REGIONAL WASTEWATER AUTHORTY WASTEWATER TREATMENT PLANT

The South County Regional Wastewater Authority WWTP is an 8.5 million gallons per day (mgd) primary, secondary and tertiary treatment facility. The treatment facility is located at the end of Southside Drive. The original plant was completed in 1990 with a plant expansion occurring in 2007 to provide the plants current capacity and technology. The SCRWA WWTP has a design capacity of 9 mgd, but is limited to 8 mgd due to the chlorine contact basin capacity and it can accommodate a design peak dry weather flow of up to 15.1mgd. The plant is currently operating at an average flow of 6 mgd with low of approximately 3 mgd and a peak of approximately 9 mgd.

CHAPTER 5 – SANITARY SEWER FLOWS

This chapter summarizes historical wastewater flows experienced at the South Country Regional Wastewater Authority (SCRWA) WWTP and defines flow terminologies relevant to this evaluation. This chapter discusses the wastewater flow distribution within the ten flow monitored basins, and identifies the design flows used in the hydraulic modeling effort and capacity evaluation. The design flows include the existing condition (existing customers) and the projected ultimate buildout scenario.

5.1 FLOWS AT THE SCRWA WWTP

The wastewater flows collected and treated at the SCRWA WWTP vary monthly, daily, and hourly. While the dry weather flows are influenced by customer uses, the wet weather flows are influenced by the severity of storm events and the condition of the system. **Figure 5.1** shows the monthly flows versus rainfall at the SCRWA WWTP for 2012. April and December were the maximum months during 2012, with December also being higher than average due to the considerable amount of rain received that month.

Flow data influent to the SCRWA WWTP was obtained from City operation staff. The flow data covered a period from 2010 to 2015. From this data monthly, daily, peak daily flows, and peak hourly flows (if available), were determined as summarized on **Table 5.1**.

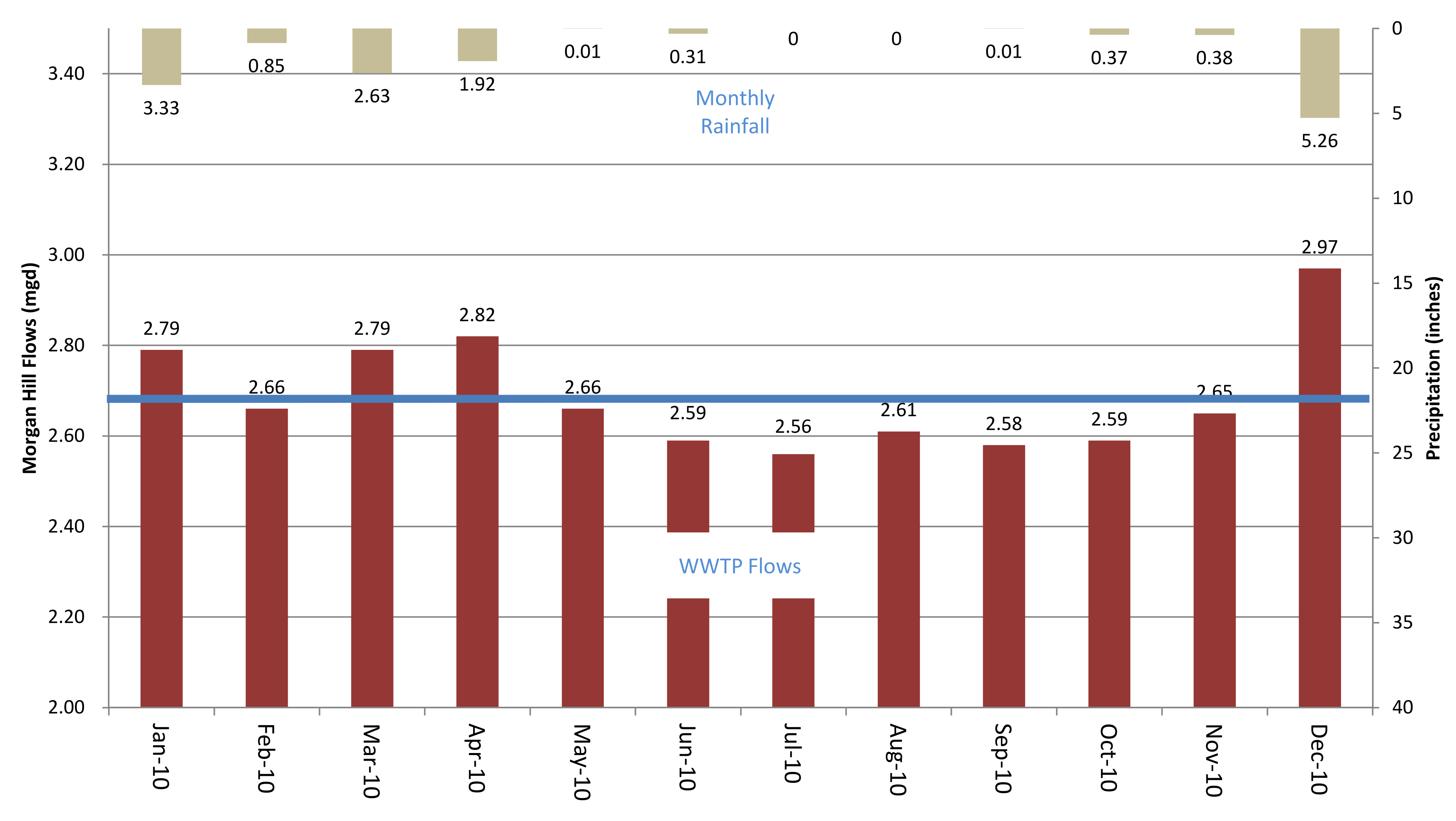
The following definitions are intended to document relevant terminologies shown on **Table 5.1**:

- **Average Annual Flow (AAF).** The average annual flow is the total annual flow, or average monthly flow, for a given year, expressed in daily or other time units. This flow includes the combined average of the average dry weather flow (ADWF) and average wet weather flow (AWWF).

- **Average Dry Weather Flow (ADWF).** The average dry weather flow occurs on a daily basis during the dry weather season, with no evident reaction to rainfall. The ADWF also includes the Base Wastewater Flow (BWF). The base wastewater flow is the average flow that is generated by residential, commercial, and industrial users. The flow pattern from these users varies depending on land use types.

- **Average Wet Weather Flow (AWWF).** This average wet weather flow occurs on a daily basis during the wet weather season. In addition to the flow components in the ADWF, the AWWF includes infiltration and inflow from storm rainfall events.

- **Maximum Month Dry Weather Flow (MMDWF).** This maximum month flow occurs during the dry weather season.



LEGEND

Monthly Flows

Rainfall

2012 Average Flow

PRELIMINARY

Note : Flows shown measured at the Harding Meter.
-Rainfall data extracted from CIMIS Station in Gilroy (Year 2012)
-Morgan Hill flow values provided by SCWRA (Year 2012)

October 26, 2016

Figure 5.1

2012 City Flow

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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Table 5.1 Historical Flow Data and Peaking Factors

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Year	Population	Average Annual Flow			Seasonal Average		Maximum Month		Maximum Day		Total SCRWA Plant Flow[1]	
		AAF (MGD)	Per Capita Flow (GPCD)	Percentage Change	ADWF (MGD)	AWWF (MGD)	MMDWF (MGD)	MMWWF (MGD)	MDDWF (MGD)	MDWWF (MGD)	MDDWF (MGD)	MDWWF (MGD)
2010	40,246	2.85	71	-	2.69	3.02	2.89	3.22	3.14	4.61	7.19	8.99
2011	38,309	2.85	74	0%	2.66	3.04	2.86	3.71	3.10	5.81	7.37	11.98
2012	39,127	2.69	69	-6%	2.60	2.78	2.66	2.97	2.77	4.61	7.13	9.68
2013	40,079	2.69	67	0%	2.66	2.73	2.70	2.77	2.90	3.09	7.18	7.67
2014	41,197	2.58	63	-4%	2.52	2.64	2.64	2.73	2.81	3.69	6.57	8.45
2015	42,382	2.37	56	-8%	2.31	2.40	2.35	2.64	2.42	3.77	6.02	8.24
Historical Peaking Factors (Applied to ADWF)												
2010	-	1.06	-	-	1.00	1.12	1.07	1.20	1.17	1.71	-	-
2011	-	1.07	-	-	1.00	1.14	1.08	1.39	1.17	2.18	-	-
2012	-	1.03	-	-	1.00	1.07	1.02	1.14	1.07	1.77	-	-
2013	-	1.01	-	-	1.00	1.03	1.02	1.04	1.09	1.16	-	-
2014	-	1.02	-	-	1.00	1.05	1.05	1.08	1.12	1.46	-	-
2015	-	1.03	-	-	1.00	1.04	1.02	1.15	1.05	1.64	-	-

Notes:

6/29/2016

1. Total SCRWA Plant Flow Represents combined flow of cities of Morgan Hill and Gilroy.
2. Definitions are as follows:
   - AAF - Average Annual Flow (annual flow, expressed in daily or other time units)
   - ADWF - Average Dry Weather Flow (average flow that occurs on a daily basis during the dry weather season)
   - AWWF - Average Wet Weather Flow (average flow that occurs on a daily basis during the wet weather season)
   - MMDWF - Maximum Month Dry Weather Flow (maximum month flow during the dry weather season)
   - MMWWF - Maximum Month Wet Weather Flow (maximum month flow during the wet weather season)
   - MDDWF - Maximum Day Dry Weather Flow (highest measured daily flow that occurs during a dry weather season)
   - MDWWF - Maximum Day Wet Weather Flow (highest measured daily flow that occurs during a wet weather season)
   - PDWF - Peak Dry Weather Flow (highest measured hourly flow that occurs during a dry weather flow)
   - PWWF - Peak Hour Wet Weather Flow (highest measured hourly flow that occurs during wet weather)
3. Source:
   - 2010 and 2011 flows from South County Regional WasteWater Authority Community Development Report
   - 2012, 2013, 2014, and 2015 flows from South County Regional WasteWater Authority Public Works Report

- **Maximum Month Wet Weather Flow (MMWWF).** This maximum month flow occurs during the wet weather season.
- **Maximum Day Dry Weather Flow (MDDWF).** This is the highest measured daily flow that occurs during a dry weather season.
- **Maximum Day Wet Weather Flow (MDWWF).** This is the highest measured daily flow that occurs during a wet weather season.
- **Peak Dry Weather Flow (PDWF).** This is the highest measured hourly flow that occurs during a dry weather season.
- **Peak Wet Weather Flow (PWWF).** This is the highest measured hourly flow that occurs during a wet weather season.

Table 5.1 shows the average annual flows (AAF) contributed by the City at the SCRWA WWTP have decreased from 2.85 mgd in 2010 to 2.37 mgd in 2015, which is a decrease of approximately 17%. In general, the AAF flows have decreased slowly from 2010 to 2015, and decreased by 12 % between 2014 and 2015. Table 5.1 also indicates that the City's AAF is generally close to the Maximum Month Dry Weather Flows (MMDWF).

In addition to listing the 2010-2015 flows, and for comparison purposes, the table calculates the peaking factors applied to the corresponding average dry weather flows (ADWF) for each year. During wet weather flows in 2015, the maximum daily volume (MDWWF) contributed by the City at the SCRWA WWTP was 1.64 times higher than the average dry weather flow for the same year.

5.2 EXISTING SEWER FLOWS BY MONITORING BASIN

The existing sewer flows represented in this Master Plan were based on the City's water consumption billing records. The number of acres and corresponding sewer flows, for sewer flow metering basin, are summarized on Table 5.2. The sewer flow monitoring basins are also shown in Chapter 3 on Figure 3.5

- **Basin 1.** This basin includes 9 percent of the total acres and 7 percent of the existing dry weather flows.
- **Basin 2.** This basin includes 10 percent of the total acres and 9 percent of the existing dry weather flows.
- **Basin 3.** This basin includes 4 percent of the total acres and 6 percent of the existing dry weather flows.

Table 5.2 Sewer Flow Distribution

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Basin	Area		Average Dry Weather Flows [1]	
	Acres	Percent of Total (%)	Flows (gpm)	Percent of Total (%)
Basin 1	817	9%	127	7%
Basin 2	911	10%	169	9%
Basin 3	345	4%	117	6%
Basin 4	823	9%	147	8%
Basin 5	807	8%	218	11%
Basin 6	1,918	20%	345	18%
Basin 7	367	4%	121	6%
Basin 8	398	4%	136	7%
Basin 9	1,094	12%	157	8%
Basin 10	454	5%	145	8%
Harding	1,568	17%	234	12%
Total	**9,500**	100%	**1,916**	100%

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10/31/2016

Note:
1. Based on 2012 water billing records and adequate return to sewer factor.

- **Basin 4.** This basin includes 9 percent of the total acres and 8 percent of the existing dry weather flows.
- **Basin 5.** This basin includes 8 percent of the total acres and 11 percent of the existing dry weather flows.
- **Basin 6.** This basin includes 20 percent of the total acres and 18 percent of the existing dry weather flows.
- **Basin 7.** This basin includes 4 percent of the total acres and 6 percent of the existing dry weather flows.
- **Basin 8.** This basin includes 4 percent of the total acres and 7 percent of the existing dry weather flows.
- **Basin 9.** This basin includes 12 percent of the total acres and 8 percent of the existing dry weather flows.
- **Basin 10.** This basin includes 5 percent of the total acres and 8 percent of the existing dry weather flows.
- **Harding Basin.** This basin includes 17 percent of the total acres and 12 percent of the existing dry weather flows.

5.3 BUILDOUT WASTEWATER FLOWS

The land use methodology was used to estimate the buildout flows from the City's Planning Area and to be consistent with the General Plan. Table 5.3 documents the total acreages for residential and non-residential land use, and the undeveloped lands designated for urbanization. The undeveloped lands were multiplied by the corresponding unit flow factor to estimate the sewer flows. The 2014 flows were increased to 2.8 mgd to account for 100% occupancy, and the ultimate buildout flows were calculated at 4.2 mgd.

5.4 SEWER COLLECTION SYSTEM DESIGN FLOWS

The design flows most relevant in this capacity analysis of the sewer system, in addition to the Maximum Day Dry Weather Flows (MDDWF), include the peak dry weather flow (PDWF) and peak wet weather flow (PWWF).

- **Peak Dry Weather Flow (PDWF).** The PDWF is used for evaluating the capacity adequacy of the sanitary sewer system, and to meet the criteria set forth in the previous chapter and in the City standards.

Table 5.3 Average Daily Flows at Buildout of Project Area
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Land Use Classifications	Sewer Flows at 100% Occupancy											
	Existing Development within City Limits			Future Development within City Limits			Total Development within City Limits		Future Development Outside City Limits		Total	
	Existing Development within City Limits (net acres)	Sewer Unit Factor (gpd/net acre)	Existing Average Daily Flow (gpd)	Future Development (net acres)	Future Sewer Unit Factor (gpd/net acre)	Future Development Average Daily Flow (gpd)	Development (net acres)	Total Development Average Daily Flow (gpd)	Future Development (net acres)	Future Development Average Daily Flow (gpd)	Development (net acres)	Average Daily Flow (gpd)
Residential												
Single Family												
Residential Estate	508	150	76,184	198	150	29,670	706	105,854	321	48,208	1,027	154,062
Residential Detached Low	979	340	333,019	171	340	58,076	1,150	391,094	239	81,123	1,389	472,218
Residential Detached Medium	1,252	630	789,028	187	630	117,524	1,439	906,552	411	259,136	1,850	1,165,687
Residential Detached High	30	840	25,374	4	840	3,649	35	29,024	20	16,430	54	45,454
Multi-Family												
Residential Attached Low	340	1,100	374,280	114	1,100	125,902	455	500,182	2	2,384	457	502,566
Residential Attached Medium	100	1,700	169,290	53	1,700	89,953	152	259,243	7	12,494	160	271,737
Residential Attached High	1	2,930	2,344	5	2,930	16,065	6	18,409	0	0	6	18,409
Subtotal	3,211		1,769,519	732		440,839	3,943	2,210,358	1,000	419,775	4,943	2,630,133
Non-Residential												
General Commercial	24	1,340	32,131	0	1,340	0	24	32,131	0	0	24	32,131
Commercial	260	1,000	259,501	130	1,000	130,352	390	389,853	4	3,700	394	393,553
Commercial / Industrial[1]	501	900	451,041	230	900	207,281	731	658,322	220	197,918	951	856,240
Mixed Use	93	960	89,594	6	960	5,861	99	95,454	0	0	99	95,454
Mixed Use Flex	64	900	57,604	40	900	36,436	104	94,040	8	7,395	113	101,435
Public Facility	302	220	66,362	12	220	2,582	313	68,944	46	10,206	360	79,149
Subtotal	1,244		956,232	419		382,512	1,663	1,338,744	278	219,219	1,941	1,557,963
Other (Non-Flow Generating)												
Sports-Recreation-Leisure	0	0	0	0	0	0	0	0	251	0	251	0
Landscape Irrigation	201	0	0	0	0	0	201	0	0	0	201	0
Open Space	605	0	0	581	0	0	1,186	0	2,737	0	3,922	0
Subtotal	806		0	581		0	1,387	0	2,988	0	4,375	0
Totals	**5,260**		**2,725,751**	**1,732**		**823,351**	**6,992**	**3,549,102**	**4,267**	**638,993**	**11,259**	**4,188,095**

Notes
1. "Commercial / Industrial" combines land use types "Commercial / Institutional" and "Industrial"

9/19/2016

- **Peak Wet Weather Flow (PWWF).** The PWWF is used for designing the capacity of the collection system, while allowing acceptable amounts of surcharging in the system.

The design flows used in evaluating the capacity adequacy of the sewer collection system are summarized on **Table 5.4**. The table lists the maximum day and peak hour flows for dry and wet weather conditions. PDWF and PWWF used for evaluating the existing collection system were estimated at 5.4 mgd and 7.7 mgd, respectively. The PDWF and PWWF used for designing the General Plan buildout system, including growth, were estimated at 8.0 mgd and 10.5 mgd, respectively.

Table 5.4 Design Flows

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Description	Flow	
	Maximum Day (mgd)	Peak Hour (mgd)
2014 Existing Condition Scenarios		
Existing DWF	2.8	5.4
Existing WWF (10Yr-24Hr Design Storm)	3.7	7.7
Ultimate Buildout Scenarios		
Buildout DWF	4.7	8.0
Buildout WWF (10Yr-24Hr Design Storm)	6.0	10.5

CHAPTER 6 - HYDRAULIC MODEL DEVELOPMENT

This chapter describes the development and calibration of the City's sewer system hydraulic model. Hydraulic network analysis has become an effectively powerful tool in all aspects of sanitary sewer system planning, design, operation, management, and system reliability analysis. The City's hydraulic model was used to evaluate the capacity adequacy of the existing system and to plan its expansion to service anticipated future growth.

6.1 HYDRAULIC MODEL SOFTWARE SELECTION

The City's hydraulic model combines information on the physical characteristics of the sewer system (pipelines, lift stations) and operational characteristics (how they operate). The hydraulic model then performs calculations and solves series of equations to simulate flows in pipes, including backwater calculations for surcharged conditions.

There are several network analysis software products released by different manufacturers that can equally perform the hydraulic analysis satisfactorily. The selection of a particular software depends on user preferences, the sanitary sewer system's unique requirements, and the costs for purchasing and maintaining the software.

The hydraulic modeling software used for evaluating the capacity adequacy of the City's sewer system, InfoSWMM by Innovyze Inc., utilizes the fully dynamic St. Venant's equation which has a more accurate engine for simulating backwater and surcharge conditions, in addition to having the capability for simulating manifolded force mains. The software also incorporates the use of the Manning Equation in other calculations including upstream pipe flow conditions. The St Venant's and Manning's equations are discussed in the System Performance and Design Criteria chapter.

6.2 HYDRAULIC MODEL DEVELOPMENT

Computer modeling requires the compilation of large numerical databases that enable data input into the model. Detailed physical aspects, such as pipe size, ground elevation, invert elevations, and pipe lengths contribute to the accuracy of the model.

Pipes and manholes represent the physical aspect of the system within the model. A manhole is a computer representation of a place where sewer flows may be allocated into the hydraulic system, while a pipe represents the conveyance aspect of the sewer flows. In addition, selected lift station capacity and design head settings were also included into the hydraulic model.

Developing the hydraulic model included system skeletonization, digitizing and quality control, developing pipe and manhole databases, and sewer loading allocation.

6.2.1 Skeletonization

Skeletonizing the model refers to the process where pipes not essential to the hydraulic analysis of the system are stripped from the model. Skeletonizing the model is useful in creating a system that accurately reflects the hydraulics of the pipes within the system. In addition, skeletonizing the model will reduce complexities of large models, which will also reduce the time of analysis while maintaining accuracy, but will also comply with the limitations imposed by the computer program.

The hydraulic model for the City of Morgan Hill was skeletonized to include the major trunk system. By comparison, the total system includes approximately 160 miles of pipe, whereas the hydraulic model includes approximately 44 miles of pipelines. The modeled pipes included pipes 10-inches in diameter and larger, in addition to some critical smaller gravity sewer pipes.

Table 6.1 documents the inventory of pipelines included in the hydraulic model by diameter and is approximately 28 percent of the overall system. The modeled sewer system is shown on **Figure 6.1**.

6.2.2 Digitizing and Quality Control

City staff completed a GIS mapping project for the sanitary sewer system prior to initiating this master plan project. City staff also conducted manhole field surveys that recorded the rim elevations, pipe invert elevations, as well as the physical manhole location. This GIS data was the basis for developing the hydraulic model used in the capacity evaluation of the sewer system.

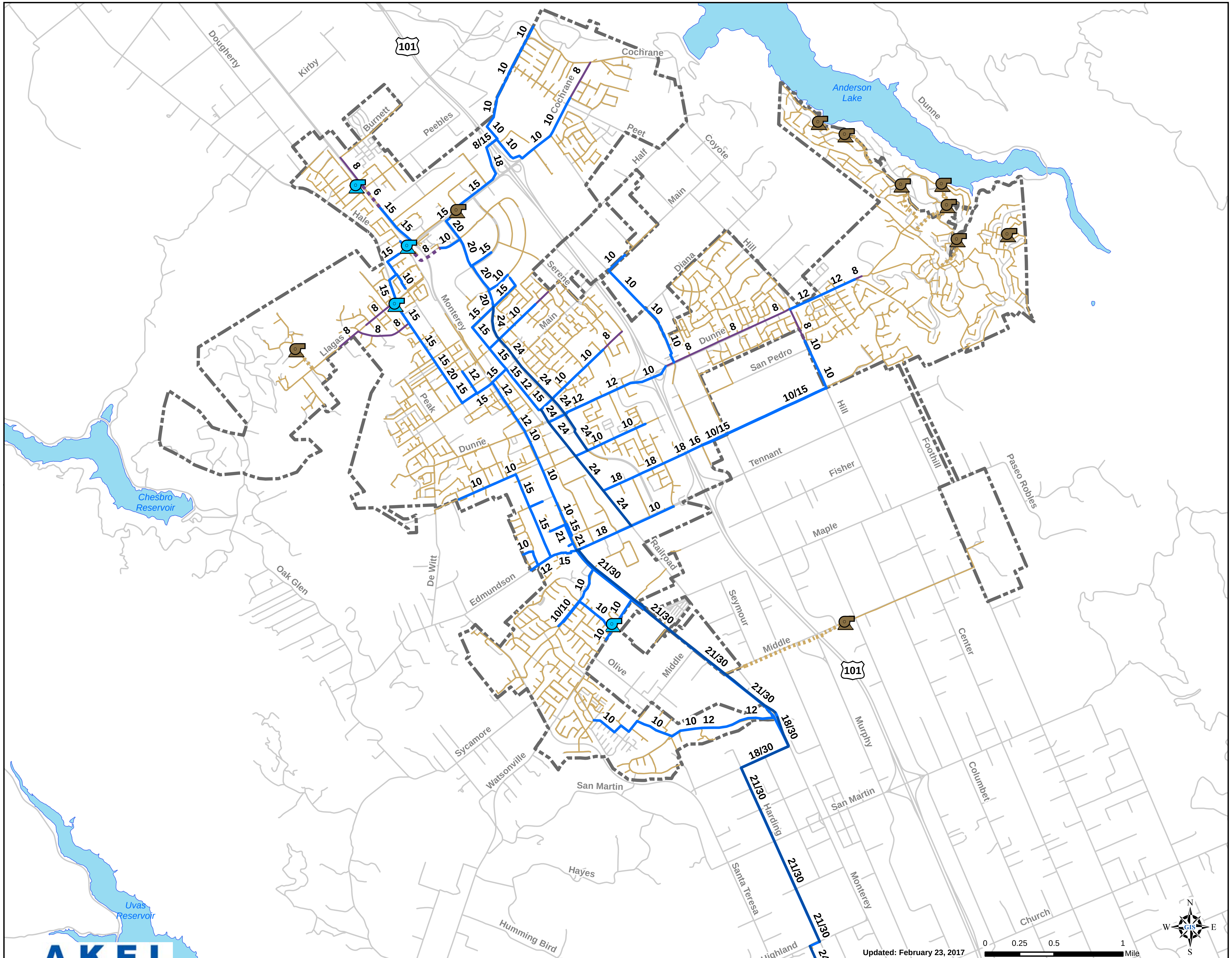
During the development of the new hydraulic model, the project team consisting of City staff and Akel Engineering staff implemented a thorough quality control program to resolve discrepancies. The quality control program included the following:

- The 2002 Sewer System Master Plan hydraulic model and the subsequent revisions
- As-Built or construction drawings
- GIS database provided by City Staff
- Available closed circuit television (CCTV)

6.2.3 Load Allocation

Load allocation consists of assigning sewer flow to the appropriate manholes (nodes) in the model. The goal is to distribute the loads throughout the model to best represent actual system response.

Allocating loads to manholes within the hydraulic model required multiple steps, incorporating the efficiency and capabilities of GIS and the hydraulic modeling software.



Legend

Modeled System

- Lift Stations
- Modeled Gravity Pipes by Size
  - 8"
  - 10" - 21"
  - 24" - 30"
- Modeled Force Mains
  - 8" or Smaller

Non-Modeled System

- Lift Stations
- Gravity Pipes
- Force Mains
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 6.1
Existing Modeled Sewer System
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

Table 6.1 Modeled Sewer Pipe Inventory

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Pipe Size	Length (feet)	Length (miles)
City Pipes		
≤ 8"	21,756	4.1
10"	51,293	9.7
12"	20,654	3.9
14"	425	0.1
15"	25,257	4.8
16"	4,126	0.8
18"	8,167	1.5
20"	4,244	0.8
21"	11,340	2.1
24"	11,869	2.2
30"	13,060	2.5
Total	172,191	32.6
Joint Trunk Pipes		
≤ 21"	8,436	1.6
24"	12,924	2.4
27"	4,407	0.8
30"	10,257	1.9
33"	22,132	4.2
42"	246	< 0.1
60"	96	< 0.1
Total	58,497	11.1

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6/28/2016

Sewer loads were developed by combining the flow factors developed in Chapter 3 with the assessor's parcel data for the City, including acreage and land use. The loads calculated were allocated to the nearest manhole that serves the corresponding parcel.

6.3 MODEL CALIBRATION

Calibration is intended to instill a level of confidence in the flows that are simulated, and it generally consists of comparing model predictions to the 2014 V&A flow monitoring program, and making necessary adjustments.

6.3.1 Calibration Plan

Calibration can be performed for steady state conditions, which model the peak hour flows, or for dynamic conditions (24 hours or more). Dynamic calibration consists of comparing the model predictions to diurnal operational changes in the wastewater flows. The City's hydraulic model was calibrated for dynamic conditions.

In sanitary sewer systems, and when using dynamic hydraulic modeling to evaluate the impact of wet weather flows, it is common practice to calibrate the model to the following three conditions:

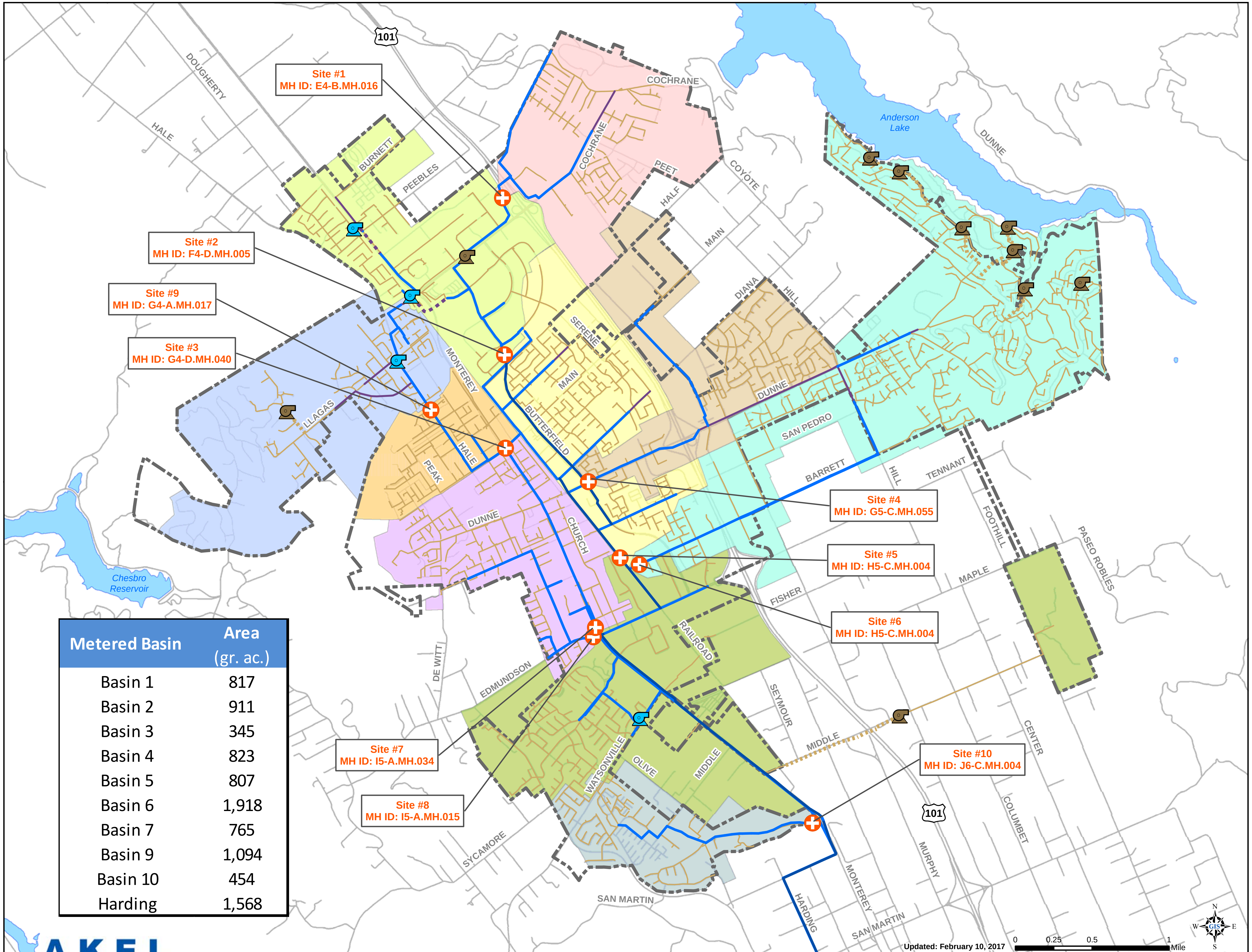
- Peak dry weather flows.
- Peak wet weather flows from storm rainfall Event No. 1.
- Peak wet weather flows from storm rainfall Event No. 2.

After the model is calibrated to these conditions, it is benchmarked and used for evaluating the capacity adequacy of the sanitary sewer system, under dry and wet weather conditions.

6.3.2 2014 V&A Temporary Flow Monitoring Program

A temporary flow monitoring program was included in this project to validate the existing dry and wet weather flows from each sewer basin. The program consisted of installing 10 flow meters, for a period of 20 days, from February 25, 2014 to March 17, 2014. Villalobos and Associates (V&A) was retained to install the flow meters, monitor rainfall, and perform an Infiltration and Inflow analysis. The selected flow monitoring sites are listed on **Table 6.2** and shown on **Figure 6.2**. Additionally, **Table 6.3** provided a calibration result summary for each of the respective sites monitored.

The 2014 V&A Flow Monitoring Program captured two rainfall events and included a summary report identifying areas of the City that were most affected by rain dependent infiltration and inflows. The two rainfall events experienced during the flow monitoring period varied in duration and intensity (**Table 3.4**), and provided an insight into the sewer system response to storm conditions.



Metered Basin	Area (gr. ac.)
Basin 1	817
Basin 2	911
Basin 3	345
Basin 4	823
Basin 5	807
Basin 6	1,918
Basin 7	765
Basin 9	1,094
Basin 10	454
Harding	1,568

Figure 6.2
Flow Meter Locations
Sewer System Master Plan
City of Morgan Hill

Table 6.2 Flow Monitoring Sites

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Site No.	GIS Manhole ID	Location	Pipe Information		Tributary Areas	
			Size (in)	Upstream Pipe Slope (ft/ft)	Metered Basins	Area (gr. ac.)
1	E4-B.MH.016	Behind Thomas Kinkade Co. along HWY 101 South off ramp (Cochrane Rd, Exit 367)	15	0.0012	Basin 1	817
2	F4-D.MH.005	Butterfield Blvd south of Digital Dr	20 from N	0.0014	Basins 1, 2	1,727
3	G4-D.MH.040	Main Ave and Monterey Rd	15 from SW	0.0046	Basins 3, 9	1,439
4	G5-C.MH.055	Dunne Ave east of Butterfield Blvd	12	0.0021	Basin 4	823
5	H5-C.MH.004	Railroad Ave and Barrett Ave	24 from NW	0.0020	Basins 1, 2, 3, 4, 5, 9	4,796
6	H5-C.MH.004	Railroad Ave and Barrett Ave	18 from E	0.0025	Basin 6	1,918
7	I5-A.MH.034	Monterey Rd and Edmundson Ave/Tennant Ave	21 from N	0.0018	Basin 7	765
8	I5-A.MH.014	Monterey Rd and Edmundson Ave/Tennant Ave	15 from W	0.0014		
9	G4-A.MH.017	Hale Ave, SE of Hillwood Ln	15 from NW	0.0017	Basin 9	1,094
10	J6-C.MH.004	Monterey Ave north of California Ave	12	0.0019	Basin 10	454
Harding			21	0.0029	Basins 1-10, Harding	9,500

Table 6.3 Calibration Results Summary
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Meter		Units	Row No.	Dry Period (Weekday)			Dry Period (Weekend)			Wet Period (Event 1)			Wet Period (Event 2)		
				Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Site 1	Flow Monitored	(GPM)	1	24.67	281.98	140.59	39.07	241.11	133.73	28.49	226.67	129.17	48.24	338.17	153.46
	Model	(GPM)	2	25.54	276.95	125.36	53.23	223.28	128.01	25.54	290.32	134.60	27.85	359.11	144.50
	Difference	(GPM)	3	-0.87	5.02	15.23	-14.16	17.84	5.72	2.94	-63.66	-5.43	20.39	-20.95	8.96
		(%)	4	4%	-2%	-11%	36%	-7%	-4%	-10%	28%	4%	-42%	6%	-6%
Site 2	Flow Monitored	(GPM)	5	69.54	388.17	245.15	52.33	376.95	235.49	48.21	421.14	252.82	63.55	512.04	279.90
	Model	(GPM)	6	91.16	384.44	246.55	111.32	366.92	236.21	91.17	407.36	264.81	110.74	517.62	275.86
	Difference	(GPM)	7	-21.62	3.73	-1.40	-58.99	10.03	-0.72	-42.95	13.78	-11.98	-47.19	-5.58	4.04
		(%)	8	31%	-1%	1%	113%	-3%	0%	89%	-3%	5%	74%	1%	-1%
Site 3	Flow Monitored	(GPM)	9	61.79	337.23	216.81	73.39	402.32	240.18	64.15	415.20	241.21	124.80	621.90	336.70
	Model	(GPM)	10	77.01	320.91	218.16	83.98	400.26	240.91	77.01	442.86	258.52	113.23	630.41	314.26
	Difference	(GPM)	11	-15.21	16.31	-1.35	-10.59	2.07	-0.72	-12.86	-27.65	-17.30	11.57	-8.50	22.43
		(%)	12	25%	-5%	1%	14%	-1%	0%	20%	7%	7%	-9%	1%	-7%
Site 4	Flow Monitored	(GPM)	13	52.11	226.80	145.11	55.71	257.41	161.40	51.30	260.76	150.69	63.95	337.48	175.16
	Model	(GPM)	14	52.65	218.75	138.24	66.00	245.33	152.65	52.65	242.98	153.59	59.08	357.00	174.14
	Difference	(GPM)	15	-0.54	8.05	6.86	-10.29	12.08	8.75	-1.35	17.78	-2.89	4.87	-19.52	1.02
		(%)	16	1%	-4%	-5%	18%	-5%	-5%	3%	-7%	2%	-8%	6%	-1%
Site 5	Flow Monitored	(GPM)	17	273.86	1162.44	795.75	283.48	1305.25	833.66	277.08	1326.59	839.41	391.97	1510.57	976.23
	Model	(GPM)	18	301.04	1146.42	839.38	362.99	1318.12	845.09	301.04	1381.11	913.71	357.28	1587.53	985.85
	Difference	(GPM)	19	-27.18	16.02	-43.63	-79.50	-12.87	-11.43	-23.96	-54.52	-74.30	34.69	-76.96	-9.62
		(%)	20	10%	-1%	5%	28%	1%	1%	9%	4%	9%	-9%	5%	1%
Site 6	Flow Monitored	(GPM)	21	87.04	426.04	245.89	85.34	451.42	259.88	92.28	437.94	248.16	106.39	549.94	291.15
	Model	(GPM)	22	80.70	427.42	246.57	108.09	442.19	257.04	80.70	464.97	264.51	84.78	601.07	286.08
	Difference	(GPM)	23	6.34	-1.39	-0.68	-22.75	9.23	2.84	11.58	-27.03	-16.35	21.61	-51.13	5.07
		(%)	24	-7%	0%	0%	27%	-2%	-1%	-13%	6%	7%	-20%	9%	-2%
Site 7	Flow Monitored	(GPM)	25	40.52	235.27	149.60	75.07	292.00	188.70	63.32	333.90	188.88	92.75	522.38	236.41
	Model	(GPM)	26	63.08	223.56	159.16	77.54	289.32	189.05	63.08	324.58	187.62	78.39	524.23	236.12
	Difference	(GPM)	27	-22.56	11.71	-9.56	-2.47	2.68	-0.36	0.24	9.31	1.26	14.36	-1.84	0.29
		(%)	28	56%	-5%	6%	3%	-1%	0%	0%	-3%	-1%	-15%	0%	0%
Site 8	Flow Monitored	(GPM)	29	41.65	167.47	107.80	40.85	203.78	124.31	29.32	175.95	102.74	48.35	216.14	133.22
	Model	(GPM)	30	35.15	169.92	102.41	41.36	193.91	119.93	35.15	181.36	110.03	44.30	216.92	129.32
	Difference	(GPM)	31	6.50	-2.45	5.39	-0.51	9.87	4.38	-5.84	-5.42	-7.29	4.05	-0.78	3.90
		(%)	32	-16%	1%	-5%	1%	-5%	-4%	20%	3%	7%	-8%	0%	-3%
Site 9	Flow Monitored	(GPM)	33	21.68	188.31	93.37	14.53	181.62	102.58	32.13	182.01	104.33	56.69	272.48	138.84
	Model	(GPM)	34	33.26	171.02	93.43	33.61	176.71	102.88	33.26	199.28	109.56	37.72	290.32	128.73
	Difference	(GPM)	35	-11.58	17.29	-0.06	-19.08	4.91	-0.30	-1.13	-17.28	-5.22	18.97	-17.84	10.11
		(%)	36	53%	-9%	0%	131%	-3%	0%	4%	9%	5%	-33%	7%	-7%
Site 10	Flow Monitored	(GPM)	37	23.10	143.88	81.42	26.95	166.61	94.81	24.92	162.19	84.47	27.16	194.42	93.38
	Model	(GPM)	38	26.58	141.09	81.08	20.78	172.16	91.41	26.58	148.40	85.66	27.03	180.10	96.39
	Difference	(GPM)	39	-3.49	2.79	0.33	6.17	-5.55	3.40	-1.66	13.79	-1.19	0.13	14.32	-3.01
		(%)	40	15%	-2%	0%	-23%	3%	-4%	7%	-9%	1%	0%	-7%	3%
Harding	Flow Monitored	(GPM)	41	700.61	2634.02	1820.89	679.60	3163.05	1903.23	720.67	2748.66	1864.16	790.73	3405.34	2243.25
	Model	(GPM)	42	724.67	2606.60	1796.38	678.57	3117.82	1900.58	724.67	2826.59	1944.01	779.20	3673.07	2144.68
	Difference	(GPM)	43	-24.07	27.43	24.51	1.04	45.23	2.65	-4.00	-77.93	-79.85	11.53	-267.74	98.57
		(%)	44	3%	-1%	-1%	0%	-1%	0%	1%	3%	4%	-1%	8%	-4%

During the V&A flow monitoring program; three rain gauges was set up in the City to record storm events during the monitoring period shown on **Figure 6.2**. Data from the V&A flow monitoring effort, as documented in the 2014 V&A Flow Monitoring Program, was used in this analysis to calibrate the computer hydraulic model to average dry weather flow (ADWF) and peak wet weather flow (PWWF) conditions.

6.3.3 Dynamic Model Calibration

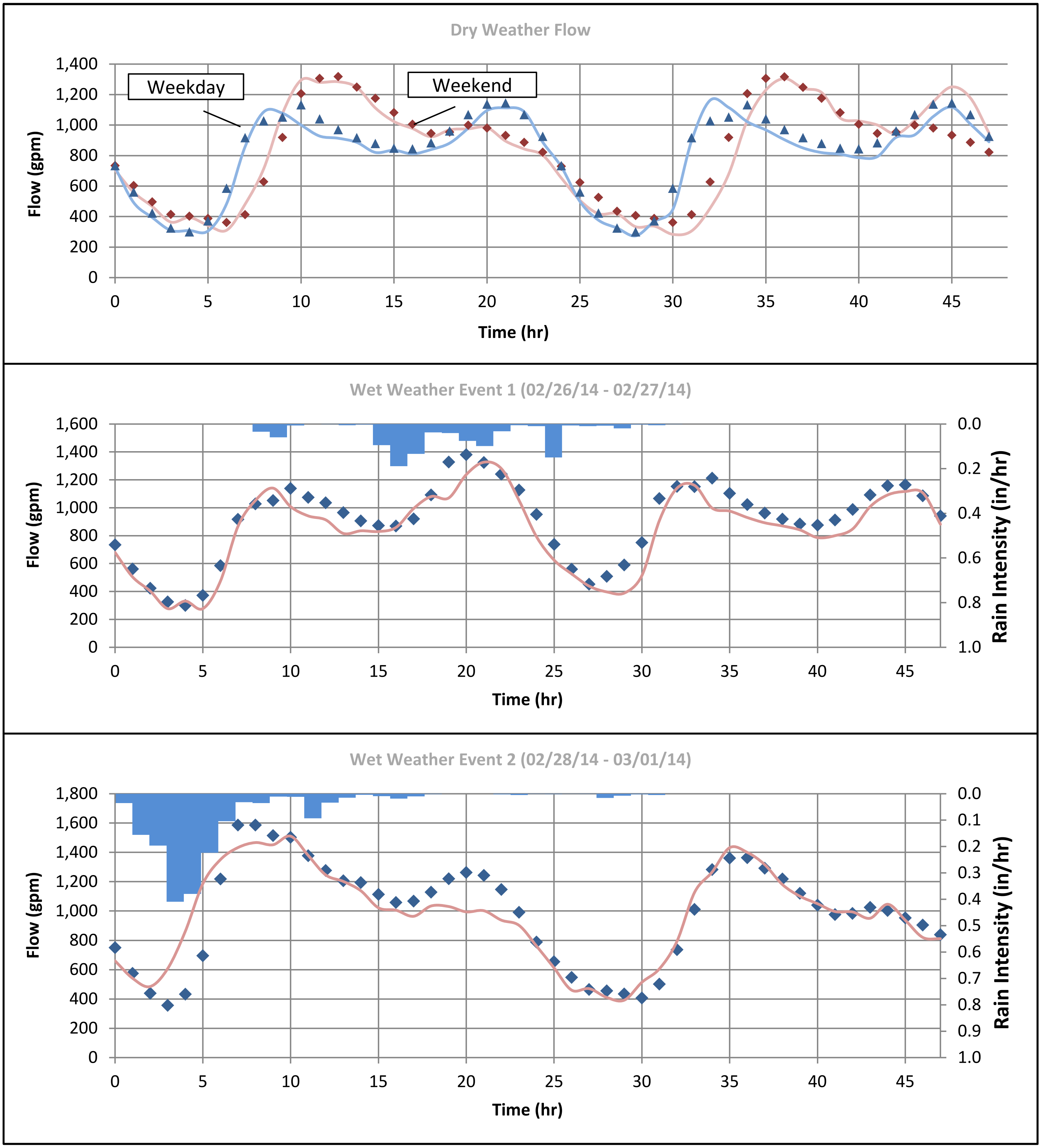
The calibration process was iterative as it involved calibrating each of the 10 flow monitored sites and for the three calibration conditions: 1) peak dry weather flow, 2) peak wet weather flows from storm rainfall Event No. 1, and 3) peak wet weather flows from storm rainfall Event No. 2.

The rain events of February 26, 2014 to February 27, 2014 (Event No. 1) and February 28, 2014 to March 1, 2014 (Event No. 2), as listed on **Table 3.4**, were used to calibrate the hydraulic model to the wet weather conditions. The diurnal curves for each of the 10 sites were extracted from the 2014 V&A Flow Monitoring Program and the data was used for comparison purposes with the hydraulic model predictions. The calibration effort continued until it yielded acceptable results for each site and for each of the three calibration conditions.

The calibration results for each flow monitoring site are documented in **Appendix B**. These results indicate the calibration effort yielded reasonable comparisons between the flow monitoring data and the hydraulic model predictions at the 10 sites. Representative extracts from **Appendix B** are shown on **Figures 6.3** and **6.4**. After each of the calibration process has been completed, the hydraulic model was benchmarked for further analysis and evaluation.

6.3.4 Use of the Calibrated Model

The calibrated hydraulic model was used as an established benchmark in the capacity evaluation of the existing sanitary sewer system. The model was also used to identify improvements necessary for mitigating existing system deficiencies and for accommodating future growth. The hydraulic model is a valuable investment that will continue to prove its worth to the City as future planning issues or other operational conditions surface. It is recommended that the model be maintained and updated with new construction projects to preserve its integrity.



LEGEND

Rain Event

Hydraulic Model

V&A Flow Monitoring

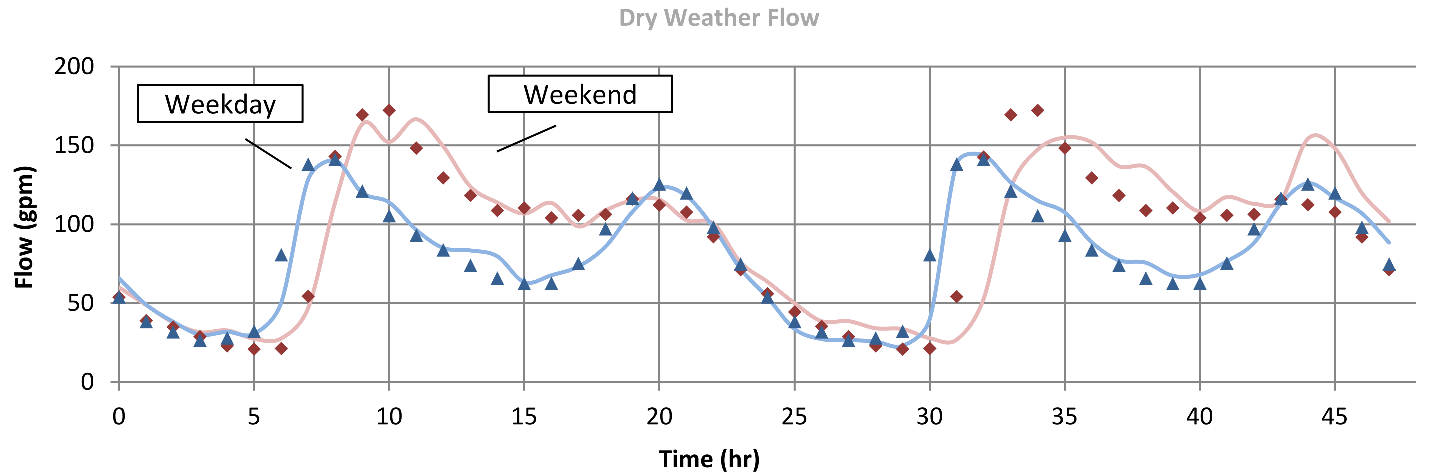
V&A Flow Monitoring

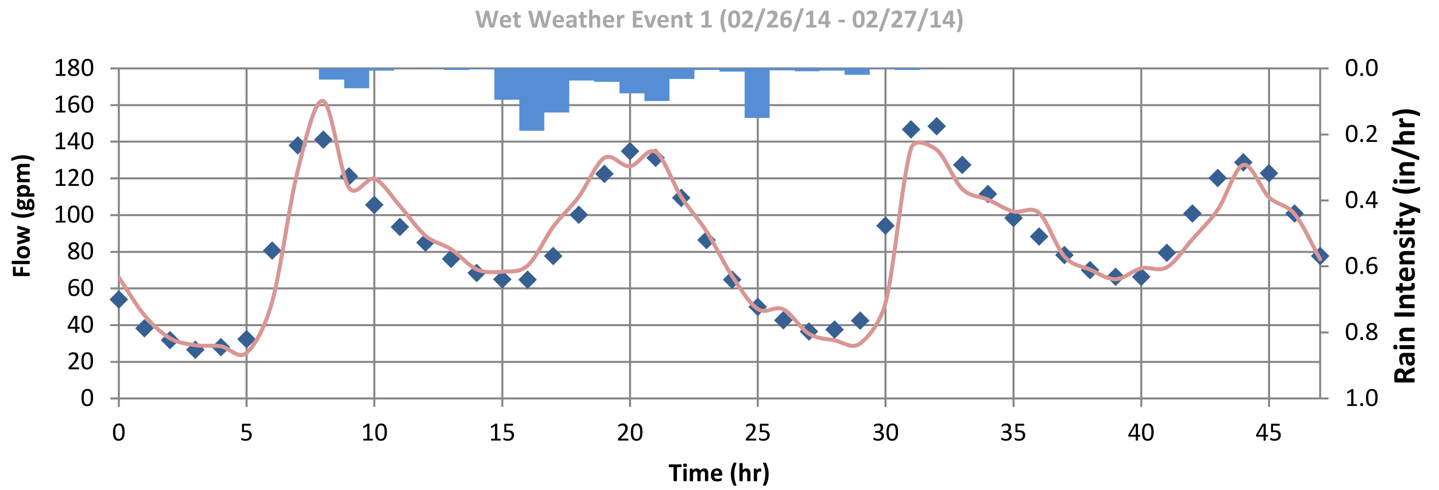
PRELIMINARY

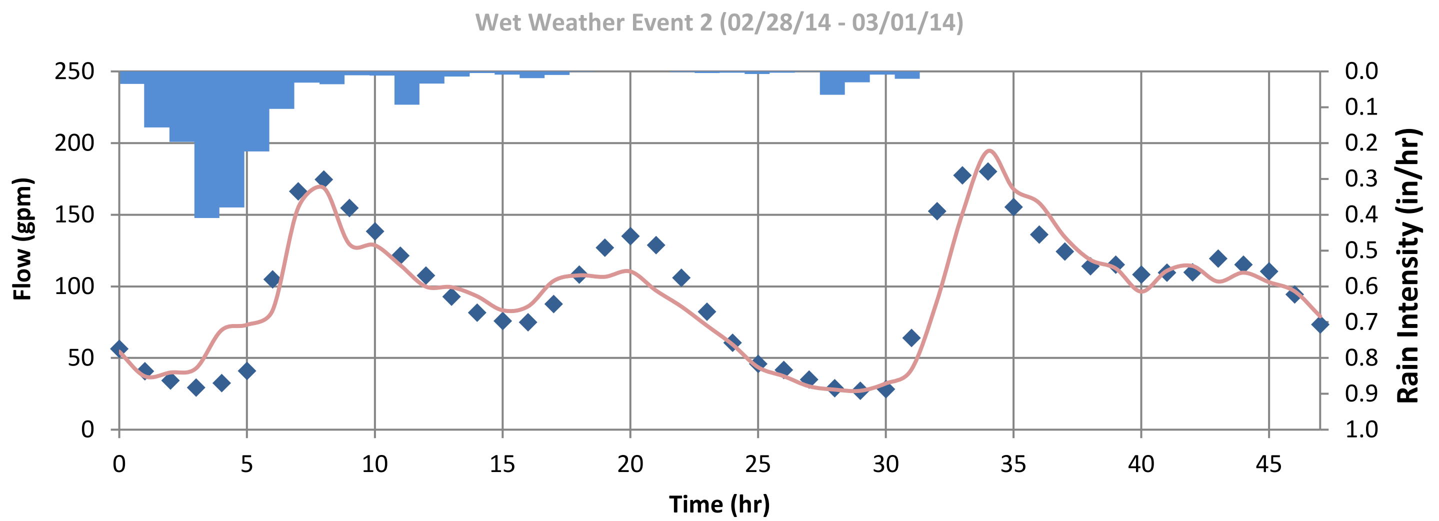
June 29, 2016

Figure 6.3
Site 5 Calibration
Barrett Ave and Railroad Ave.
Sewer System Master Plan
City of Morgan Hill

AKEL ENGINEERING GROUP, INC.







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

AKEL ENGINEERING GROUP, INC.

Figure 6.4
Site 10 Calibration
w/o Monterey Rd. and n/o California Ave.

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

CHAPTER 7 - EVALUATION AND PROPOSED IMPROVEMENTS

This section presents a summary of the sanitary sewer system capacity evaluation during peak dry weather flows and peak wet weather flows for the existing and buildout flows. The recommended sanitary sewer system improvements needed to mitigate capacity deficiencies are also discussed in this chapter.

7.1 OVERVIEW

The calibrated hydraulic model was used for evaluating the sanitary sewer system for capacity deficiencies during peak dry weather flows (PDWF) and peak wet weather flows (PWWF). Since the hydraulic model was calibrated for dynamic modeling, the analysis duration was established at 24 hours for most analyses.

The criteria used for evaluating the capacity adequacy of the wastewater collection system facilities (gravity mains, force mains, and lift stations) were discussed and summarized in the System Performance and Design Criteria chapter.

7.2 EXISTING SEWER SYSTEM CAPACITY EVALUATION

The system performance and design criteria summarized, on **Table 3.1**, were used as a basis to judge the adequacy of capacity for the existing sanitary sewer system. The design flows simulated in the hydraulic model for existing conditions were summarized on **Table 5.4** and are listed as follows:

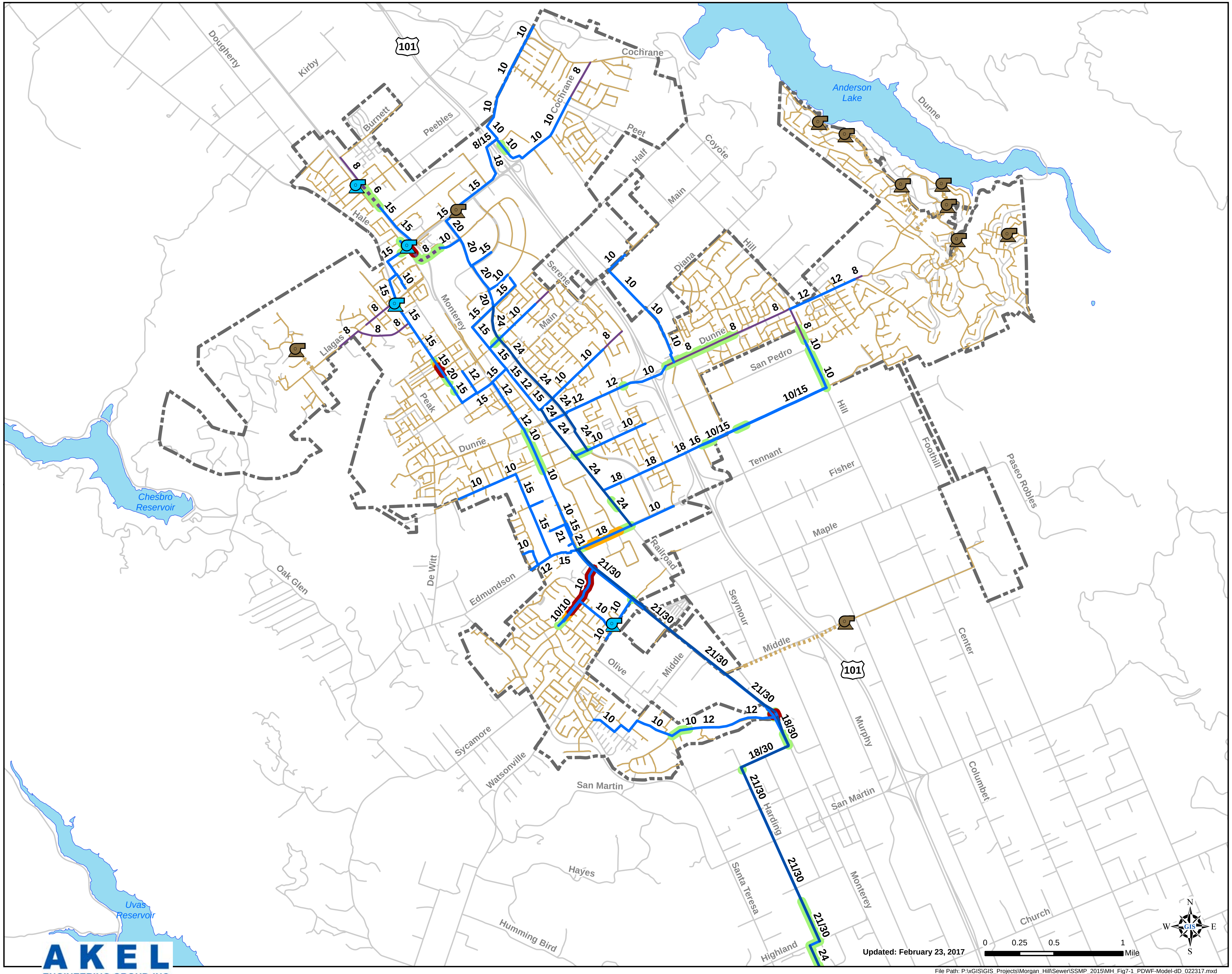
- Existing PDWF = 5.4 mgd
- Existing PWWF = 7.7 mgd

During the peak dry weather simulations, the maximum allowable pipe d/D criteria for new pipes (d/D ratio of 0.75) for was used. For existing pipes, the criteria was relaxed to allow a maximum d/D ratio of 0.90 (full pipe capacity) to prevent unnecessary pipe replacements. During the peak wet weather simulations, capacity deficiencies included pipe segments with a hydraulic grade line (HGL) that rises within three feet of the manhole rim elevation.

In general, the hydraulic model indicated that the sanitary sewer system exhibited acceptable performance to service the existing customers during both peak dry weather flows (**Figure 7.1**) and peak wet weather flows (**Figure 7.2**), with exceptions noted in the following sections.

7.2.1 Existing Peak Dry Weather Flows Capacity Evaluation

The existing dry weather flow analysis indicated several areas where pipelines experienced deficiencies, which are documented on **Figure 7.1**. Additionally, this figure documents pipelines



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Legend

Modeled System

Lift Stations

Pipe d/D

d/D > 0.90

d/D 0.75 - 0.90

d/D 0.50 - 0.75

Modeled Gravity Pipes by Size

8"

10" - 21"

24" - 30"

Modeled Force Mains

8" or Smaller

Non-Modeled System

Lift Stations

Gravity Pipes

Force Mains

Roads

City Limits

Lakes

PRELIMINARY

Figure 7.1
Existing Modeled System
Analysis for PDWF
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

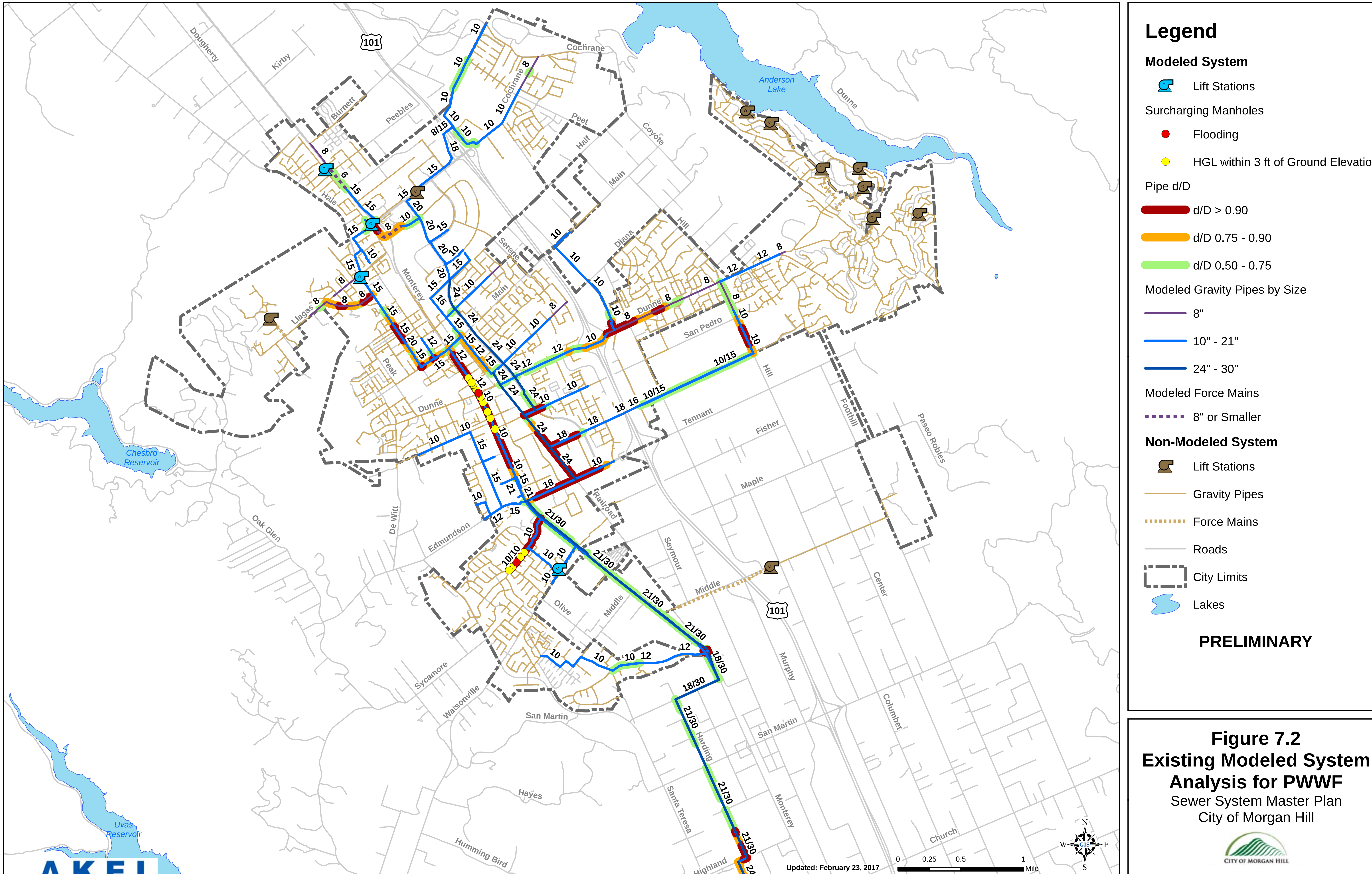

Figure 7.2
Existing Modeled System
Analysis for PWWF
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

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File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP_2015\MH_Fig7-2_PWWF-Model-dD_022317.mxd that, while not deficient, may be approaching full capacity. Deficient pipelines are highlighted in red on the figure and discussed as follows:

- Vineyard Boulevard from La Cross Drive to Monterey Road. This segment experiences d/D ratios above 0.9.
- Hale Avenue from Longview Drive to Main Avenue. This segment experiences d/D ratios under 0.5 excepting one portion experiencing a d/D ratio above 0.9.

7.2.2 Existing Peak Wet Weather Flows Capacity Evaluation

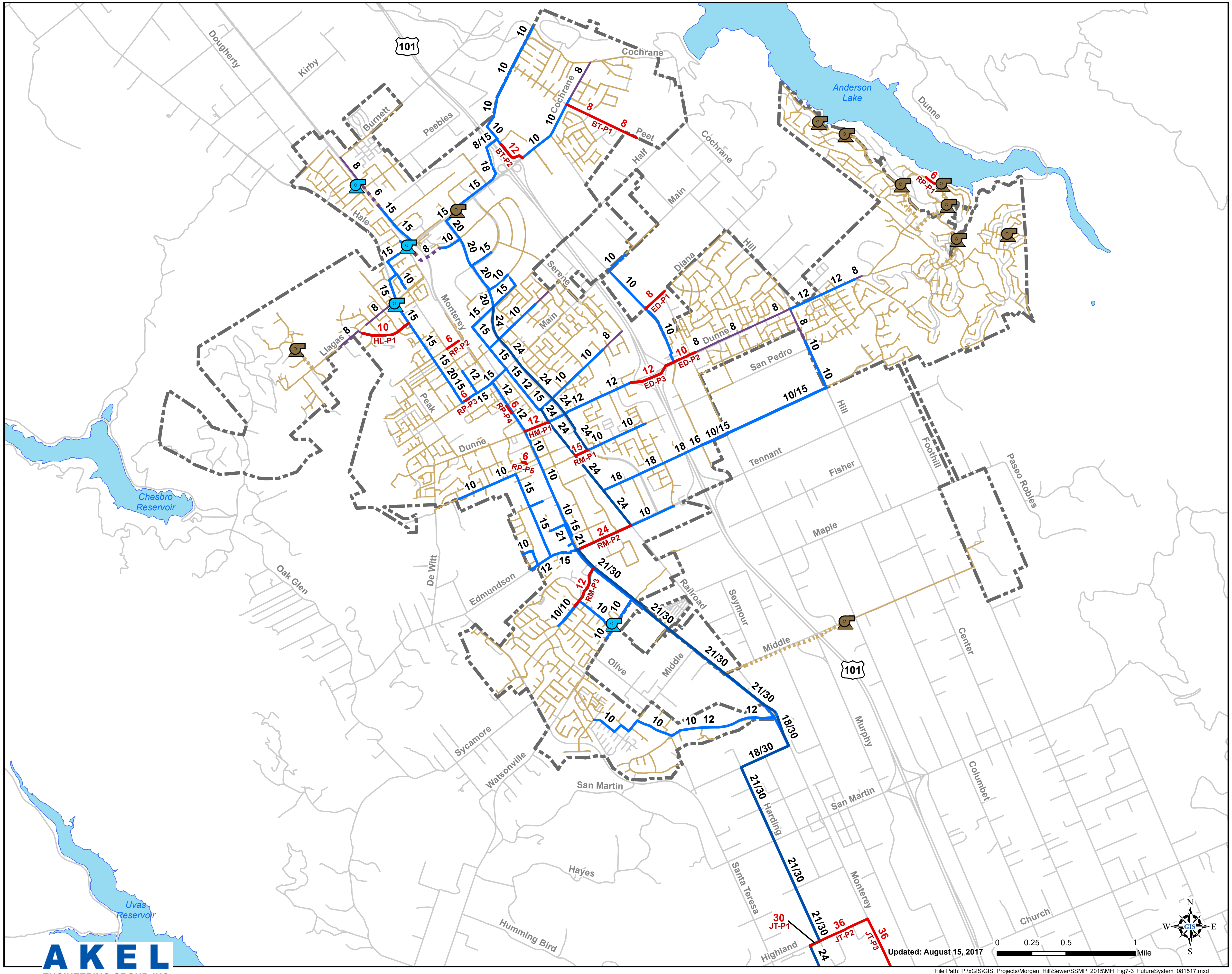
The wet weather flow analysis is intended to document the impact of rainfall events on the existing system, and to identify the improvements necessary to limit sanitary sewer overflows. The design criteria for wet weather events allows pipeline surcharging in the manhole to within three feet of the rim elevation. The hydraulic analysis indicates two areas of deficiencies, as shown on **Figure 7.2**, and documented in the following:

- Monterey Road from Main Avenue to approximately Cosmo Avenue. This segment experiences surcharging conditions where the hydraulic grade line raises within 3 feet of the rim elevation.
- La Cross Drive from La Grande Drive to Monterey Road. This segment experiences surcharging conditions within 3 feet of the rim elevation until it reaches Vineyard Boulevard. From Vineyard Boulevard, the segment of pipeline experiences d/D ratios above 0.9.

7.3 ULTIMATE BUILDOUT CAPACITY IMPROVEMENTS

The system performance and design criteria summarized on **Table 3.1**, was used as a basis to evaluate the capacity adequacy of the existing sanitary sewer system. The design flows simulated in the hydraulic model for the General Plan buildout were summarized on **Table 5.4** and are documented as follows:

- Buildout PDWF = 8.0 mgd
- Buildout PWWF = 10.5 mgd
- Sewer pipelines are recommended to serve future growth inside the City and increase the reliability of the sewer collection system as well. The proposed capacity improvements for the sanitary sewer system are listed on **Table 7.1**. This table lists the master plan assigned improvement number (e.g,HM-P1), along with other relevant information including alignment description, pipe size, and pipe length. The improvement number is further defined in the Capital Improvement Program chapter (Chapter 8). The improvements are described in detail on the following pages and shown on **Figure 7.3.**



Legend

Proposed Pipe Improvements

Existing Modeled System

Lift Stations

Modeled Gravity Pipes by Size

8"

10" - 21"

24" - 30"

Modeled Force Mains

8" or Smaller

Non-Modeled System

Lift Stations

Gravity Pipes

Force Mains

Roads

City Limits

Lakes

PRELIMINARY

Figure 7.3
Proposed Improvements

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

Table 7.1 Schedule of Improvements
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Existing Diameter (in)	Pipeline Improvements New/Parallel/Replace	Pipeline Improvements Diameter (in)	Pipeline Improvements Length (ft)
Pipeline Capacity Improvements							
Butterfield Trunk							
BT-P1	Gravity Main	Peet Rd	From approximately 3,000 ft e/o Cochrane Rd to Cochrane Rd	-	New	8	3,000
BT-P2	Gravity Main	Along NB US 101	From 900 ft n/o Cochrane Rd to intersection of Cochrane Rd and NB US 101	10	Replacement	12	1,200
Hale-Llagas Trunk							
HL-P1	Gravity Main	Llagas Creek Dr	From Eagle Springs Ct to Hale Ave	8	Replacement	10	1,950
East Dunne Trunk							
ED-P1	Gravity Main	Diana Ave	From Murphy Ave to Condit Rd	-	New	8	1,000
ED-P2	Gravity Main	Dunne Ave	From 230 ft e/o Murphy Ave to Condit Rd	8	Replacement	10	950
ED-P3	Gravity Main	Dunne Ave	From Condit Rd to 530 ft e/o Walnut Grove Dr	8/10	Replacement	12	1,950
Hale-Monterey Trunk							
HM-P1	Gravity Main	Dunne Ave	From Monterey Rd to Railroad Ave	-	New	12	1,000
Railroad-Monterey Trunk							
RM-P1	Gravity Main	San Pedro Ave	From Butterfield Blvd to Railroad Ave	10	Replacement	15	550
RM-P2 [1]	Gravity Main	Tennant Ave	From RailRoad Ave to Monterey Rd	18	Replacement	24	2,200
RM-P3	Gravity Main	La Crosse Dr / Vineyard Blvd	From La Mar Dr to Monterey Rd	10	Replacement	12	1,700
Joint Trunk							
JT-P1	Gravity Main	Highland Ave	From Harding Ave to 400 ft w/o Harding Ave	21	Replacement	30	450
Relief Trunk Improvements							
JT-P2	Gravity Main	Highland Ave	From Harding Ave to Monterey Rd	-	New	36	2,050
JT-P3	Gravity Main	Monterey Rd	From Highland Ave to Las Animas Ave	-	New	36	19,700
JT-P4	Gravity Main	Las Animas Ave	From Monterey Rd to Murray Ave	-	New	36	1,750
JT-P5	Gravity Main	Murray Ave	From Las Animas Ave to Chestnut St	-	New	36	7,550
JT-P6	Gravity Main	Chestnut St	From Murray Ave to Lewis St	-	New	36	400
JT-P7	Gravity Main	Chestnut St	From Chestnut St to 7th St	-	New	36	2,100
JT-P8	Gravity Main	7th Street	From Chestnut St to US Highway 101.	-	New	36	1,450
JT-P9	Gravity Main	Along US 101	Jogging from 7th St and US Highway 101 to Renz Lane	-	New	36	2,000
Infiltration and Inflow Improvements							
INI-P1	Gravity Main	Llagas Rd	From 80 ft e/o Hale Ave to 20 ft e/o Hale Ave (Group 5)	8	Trenchless Rehabilitatior	8	100
INI-P2	Gravity Main	Llagas Rd	From Fox Hollow Cir to Murphy Springs Dr (Group 5)	8	Trenchless Rehabilitatior	8	350
INI-P3	Gravity Main	Laurel Wood Ln	From 120 fts/o Almond Orchard Dr to 135 ft s/o Almond Orchard Dr (Grp 5)	6	Point Repair	6	15
INI-P4	Gravity Main	250 ft n/o Berkshire Ave	From 60 ft e/o Hale Ave to 115 ft e/o Hale Ave (Group 5)	15	Trenchless Rehabilitatior	15	100
INI-P5	Gravity Main	110 ft s/o Wright Ave	From 180 ft w/o Crest Ave to 50 ft e/o Crest Ave (Group 4)	6	Trenchless Replacement	6	250
INI-P6	Gravity Main	Shady Lane Dr	From Trail Ridge Ln to Calico Ridge Trl (Group 2)	6	Trenchless Rehabilitatior	6	150
INI-P7	Gravity Main	Trail Ridge Ln	From 150 ft w/o Shady Lane Dr to 70 ft e/o Shady Lane Dr (Group 2)	6	Trenchless Replacement	6	250
INI-P8	Gravity Main	50 ft n/o Copper Hill Pl	From 40 ft w/o Copper Hill Dr to 60 ft w/o Holiday Dr (Group 2)	6	Trenchless Rehabilitatior	6	200
INI-P9	Gravity Main	Quail Ln	From 150 ft e/o Quail Ct to 110 ft w/o Quail Ct (Group 2)	6	Trenchless Rehabilitatior	6	300
INI-P10	Gravity Main	175 ft s/o Oakridge Ct	From 180 ft n/o Oakridge Ln to Oakridge Ln (Group 1)	6	Trenchless Rehabilitatior	6	200
Rehabilitation Improvements							
Group 1	Gravity Main	Various	See Group 1 Figure	Various	Various	Various	7,750
Group 2	Gravity Main	Various	See Group 2 Figure	Various	Various	Various	9,800
Group 3	Gravity Main	Various	See Group 3 Figure	Various	Various	Various	5,650
Group 4	Gravity Main	Various	See Group 4 Figure	Various	Various	Various	10,300
Group 5	Gravity Main	Various	See Group 5 Figure	Various	Various	Various	6,000
Group 6	Gravity Main	Various	See Group 6 Figure	Various	Various	Various	5,550
Group 7	Gravity Main	Various	See Group 7 Figure	Various	Various	Various	8,950
Group 8	Gravity Main	Various	See Group 8 Figure	Various	Various	Various	5,700
Group 9	Gravity Main	Various	See Group 9 Figure	Various	Various	Various	2,900

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Note:

1. Improvement RM-P2 will require a casing where crossing railroad.

7.3.1 Butterfield Trunk

This section documents pipeline improvements within the Butterfield Trunk sewer service area.

- **BT-P1**: Construct a new 8-inch gravity sewer in Peet Road from 3,000 feet east of Cochrane Road to Cochrane Road.
- **BT-P2:** Replace the existing 10-inch gravity sewer along North-Bound US 101 from 900 feet north of Cochrane Road to intersection of Cochrane Road and North-Bound US 101.

7.3.2 Hale-Llagas Trunk

This section documents pipeline improvements within the Hale-Llagas Trunk sewer service area.

- **HL-P1**: Replace the existing 8-inch gravity sewer along Llagas Creek Drive from Eagle Springs Court to Hale Avenue. It should be noted that the condition assessment report included in Appendix C of this 2017 Master Plan also recommends replacement of this 8-inch trunk due to its adverse conditions.

7.3.3 East Dunne Trunk

This section documents pipeline improvements within the Hale-Llagas Trunk sewer service area.

- **ED-P1**: Construct a new 8-inch gravity sewer in Diana Avenue from Murphy Avenue to Condit Road.
- **ED-P2**: Replace the existing 8-inch gravity sewer along Dunne Avenue from 230 feet east of Murphy Avenue to Condit Road.
- **ED-P3**: Replace the existing 8-inch and 10-inch gravity sewer along Dunne Avenue from Condit Road to 530 feet east of Walnut Grove Drive.

7.3.4 Hale-Monterey Trunk

This section documents pipeline improvements within the Hale-Monterey Trunk sewer service area.

- **HM-P1**: Construct a new 12-inch gravity sewer along Dunne Avenue from Monterey Road to Railroad Avenue.

7.3.5 Railroad-Monterey Trunk

This section documents pipeline improvements within the Railroad-Monterey Trunk sewer service area.

- **RM-P1**: Replace the existing 10-inch gravity sewer along San Pedro Avenue from Butterfield Boulevard to Railroad Avenue.

- **RM-P2**: Replace the 18-inch gravity sewer along Tennant Ave from Railroad Ave to Monterey Road.
- **RM-P3**: Replace the 10-inch gravity sewer along La Crosse Drive / Vineyard Boulevard from La Mar Drive to Monterey Road.

7.4 JOINT MORGAN HILL-GILROY SEWER TRUNK CAPACITY EVALUATION

As part of this Master Plan, the Joint Morgan Hill-Gilroy Sewer Trunk (Joint Trunk) was evaluated for capacity adequacy. The Joint Trunk has historically had capacity constraints in the portion along California Avenue and Harding Avenue. Since the previous Master Plan, the City has surveyed the Joint Trunk, developed additional improvement alternatives to evaluate the feasibility of prioritizing construction, and constructed portions of a relief trunk.

The Joint Relief Trunk (Relief Trunk) was identified in the 2002 Master Plan, and subsequent studies as a required improvement intended to mitigate existing capacity deficiencies and to service future growth. The City has constructed Relief Trunk along California Avenue, and south along Harding Avenue to Highland Avenue as a 30-inch parallel pipeline. The City is currently in the process of designing the remaining 30-inch Relief Trunk from Highland Avenue to the City of Gilroy (**Figure 7.4**).

7.5 PIPELINE CONDITION ASSESSMENT

As additional task added to the scope of the master plan, City staff requested Akel Engineering Group to utilize existing closed-circuit television (CCTV) data to assess the condition of the existing sewer system infrastructure. This section documents the findings and recommendations of the condition assessment.

7.5.1 Background and Purpose

City staff have been proactively involved in the condition assessment of their pipelines, including performing CCTV, and rating those pipelines against the NASSCO Pipeline Assessment Certification Program (PACP). This rating system assesses pipelines based on observations from the CCTV, and scores them from 1 to 5 based on the criticality of the observations, with 5 being most critical. As such, City staff would like to prioritize improvement recommendations and costs in this master plan based on the physical condition of the sewer collection system. The pipelines that have been included in the CCTV, and which were reviewed as part of this condition assessment, are shown graphically on **Figure 7.5**.

7.5.2 Condition Assessment Findings

The condition assessment focused on documenting major structural defects (PACP Rating 4 or 5) and infiltration and inflow defects. Major structural defects can lead to costly pipeline failures, while infiltration and inflow defects may contribute to sanitary sewer overflows downstream.

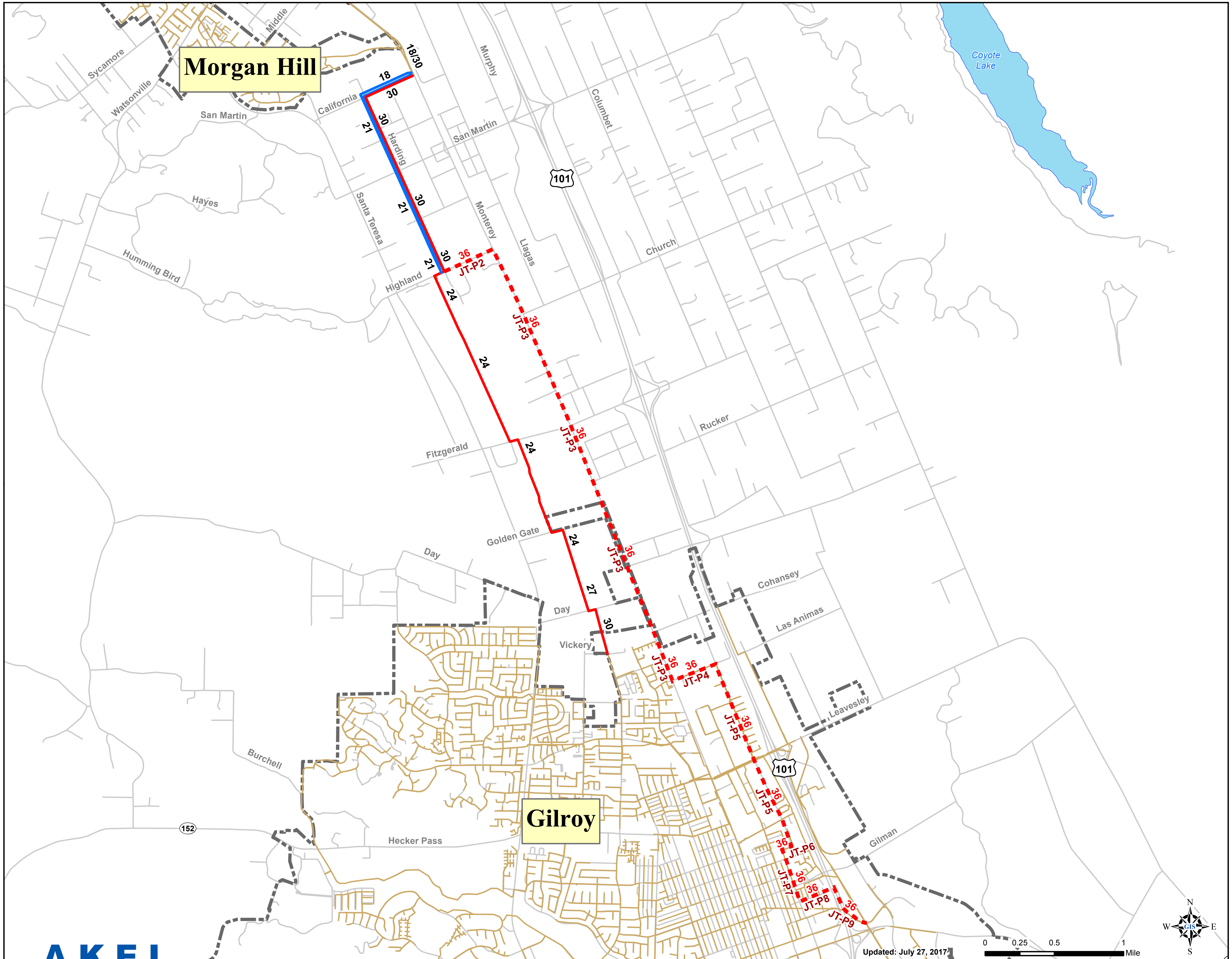
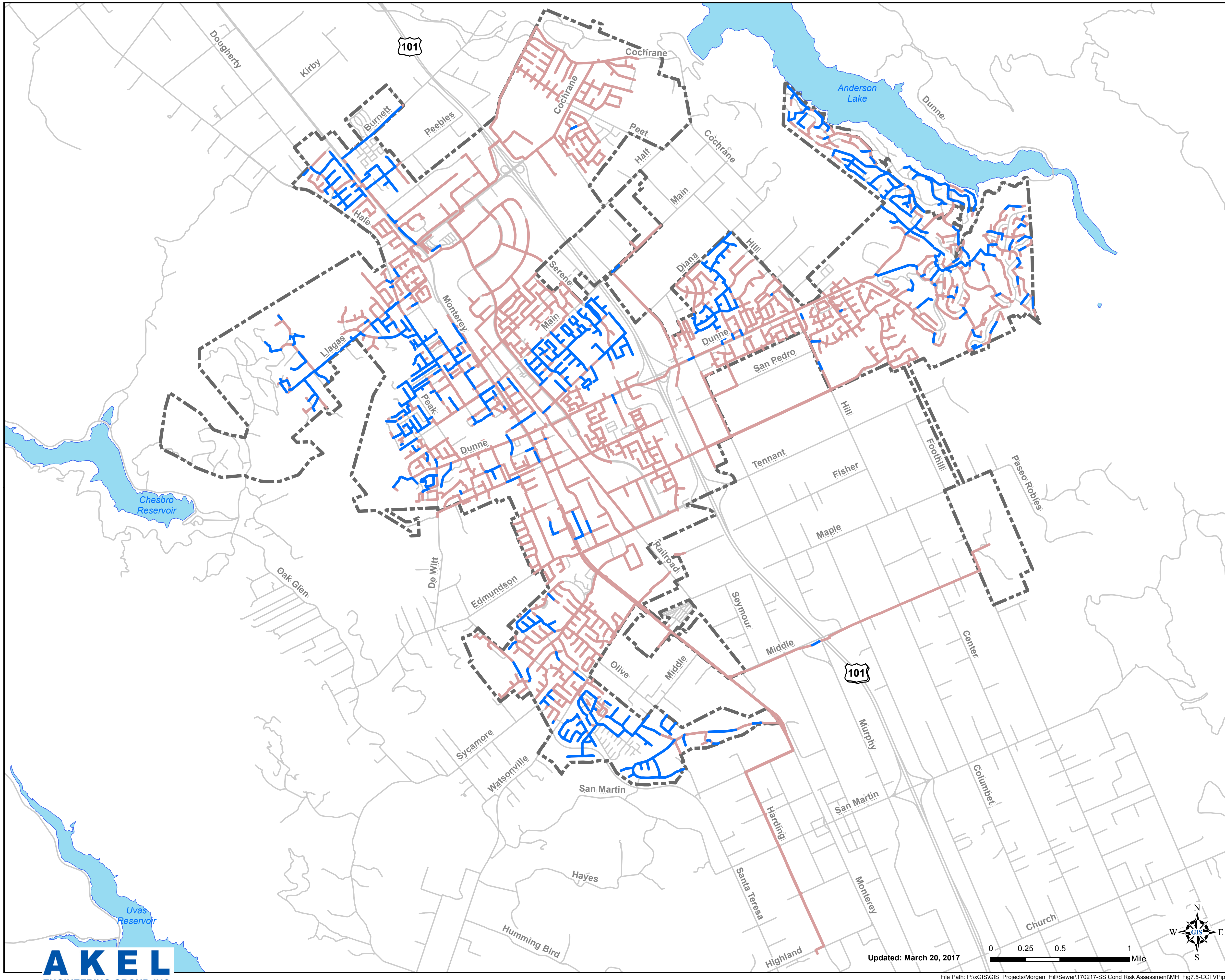


Figure 7.4
Joint Trunk
Sewer System
Sewer System Master Plan
City of Morgan Hill



Legend

- CCTV Pipes
- Pipes
- Roads
- City Limits
- Lakes

PRELIMINARY

Figure 7.5
CCTV Extent
Sewer System
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\MH_Fig7.5-CCTVPipes_081517.mxd

Other structural defects (PACP Rating 1-3), were used in the process of evaluating how critical the individual pipe segments were.

The major structural defects were documented on **Figure 7.6**. The individual defects included broken or deformed pipes, holes in the pipe, collapsed pipes, or obstructions located within the pipe. These defects are indicators that pipelines have already failed or are in imminent danger of failing.

Infiltration and inflow defects were also documented as part of this analysis and included on **Figure 7.7** and **Figure 7.8**. These defects can lead to sanitary sewer overflows, premature pipeline upsizing and higher treatment costs.

7.5.3 Improvement Recommendations

Improvements recommendations were developed based on the findings of the condition assessment. As pipelines fail to varying degrees and in various locations, the improvement recommendations were grouped by the following criteria:

- **Risk.** This category is based on the failure type and location to critical places, such as schools or creeks.
- **Location.** To avoid costly mobilization and adverse public interference, improvements were grouped into confined neighborhoods.
- **Cost.** Pipelines were also grouped by cost. As it may be cost prohibitive to complete a City-wide rehabilitation and replacement program, improvements were grouped to maintain an annual reasonable cost for budgeting purposes.

The project groupings are documented in **Appendix C**. Once groups were created, they were prioritized by the overall risk of the group, and proximity to other groups.

Additionally, City staff identified priority sections of pipeline intended to be replaced in the near-term due to deteriorated conditions or operational considerations (**Table 7.2**). It should be noted that, for planning purposes, the operational improvements generally involve replacing pipes in kind. These improvements are described as follows:

- **RP-P1**: Replace the existing 6-inch gravity sewer located on right-of-way along Holiday Drive to Oak Lane. This improvement is meant to mitigate the existing sag in the gravity main that could cause flows to spill into the adjacent lake. It should be noted that this pipeline is planned for rehabilitation and is shown on the condition assessment exhibits included in **Appendix C**.
- **RP-P2**: Replace the existing 6-inch gravity sewer along Wright Avenue from 230 feet east of Garden Avenue to Del Monte Avenue. This improvement is meant to repair the existing gravity main that is currently broken.

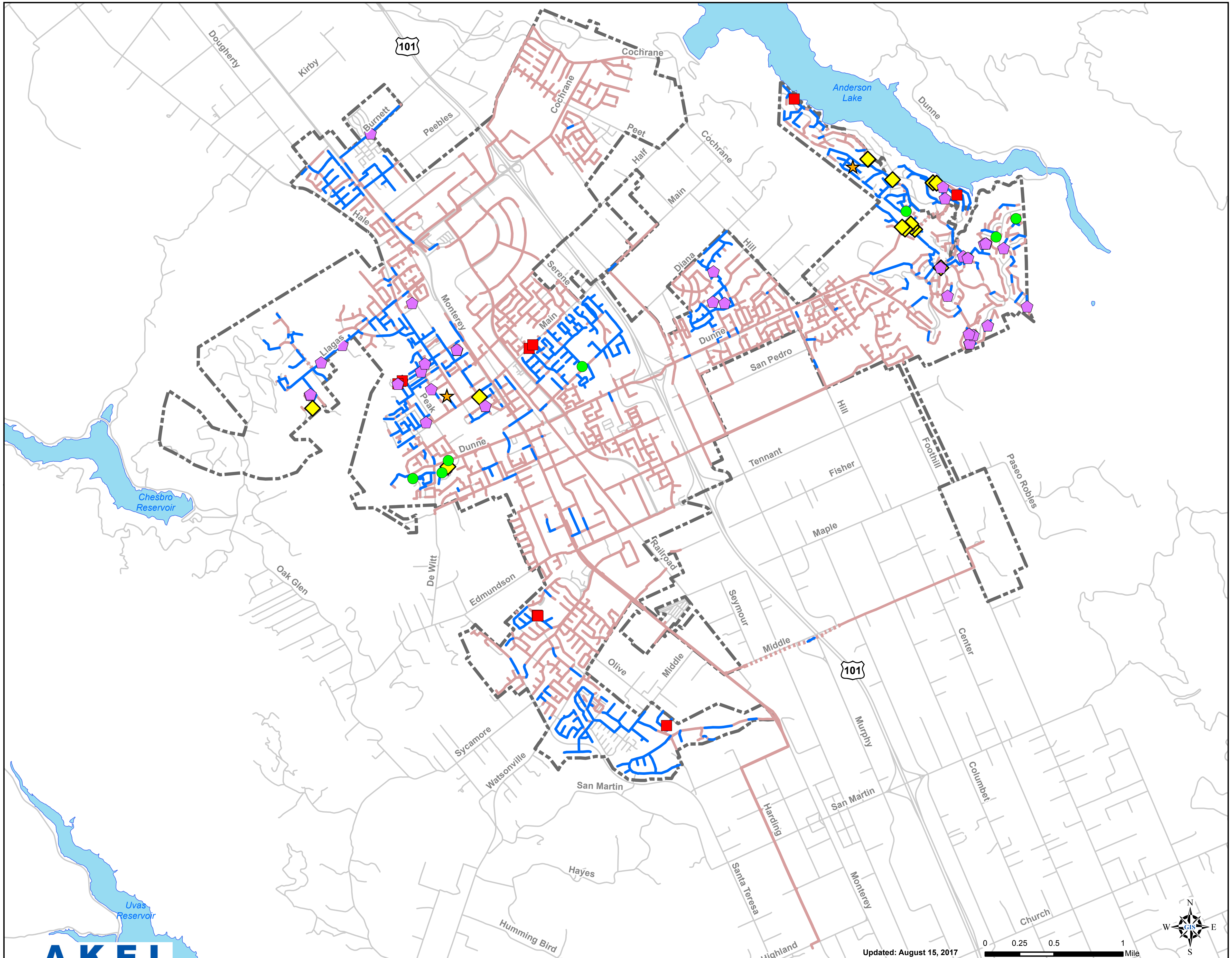


Figure 7.6
Major Structural Defects
Sewer System
Sewer System Master Plan
City of Morgan Hill

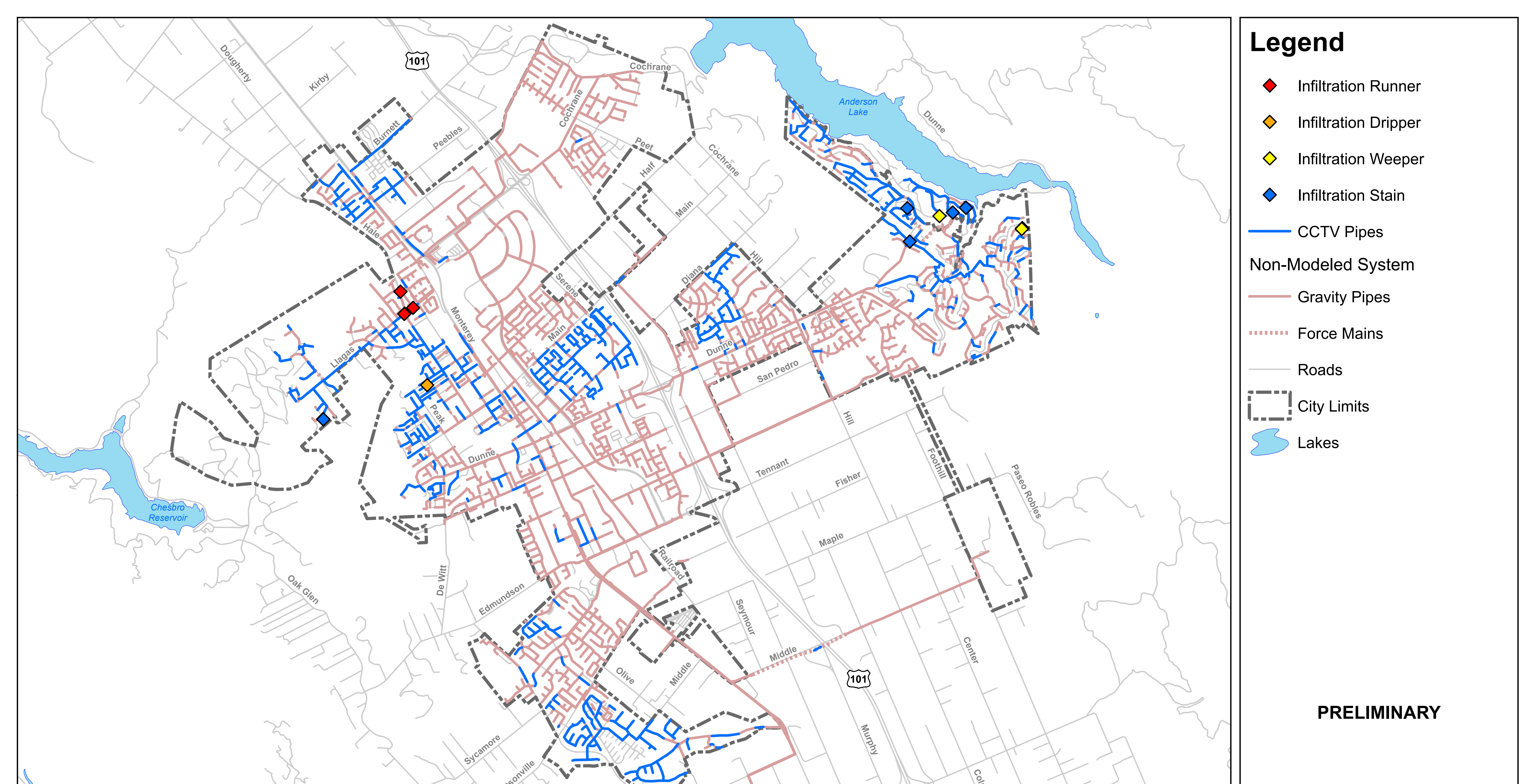


Figure 7.7
Major Infiltration Defects
Sewer System
Sewer System Master Plan
City of Morgan Hill

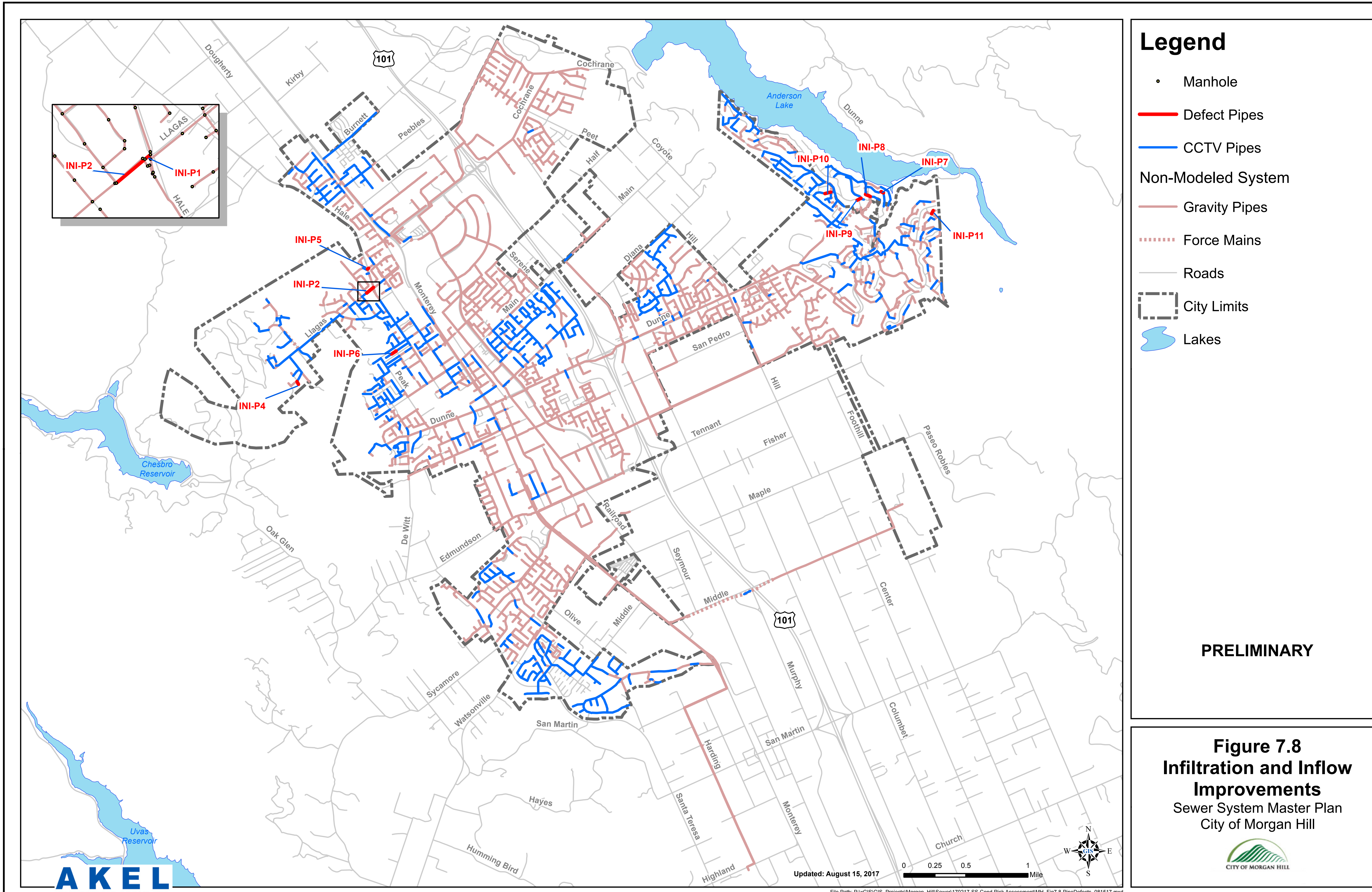


Figure 7.8
Infiltration and Inflow Improvements
Sewer System Master Plan
City of Morgan Hill

Table 7.2 Planned Operational Improvements

Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Existing Diameter (in)	Pipeline Improvements New/Parallel/ Replace	Pipeline Improvements Diameter (in)	Pipeline Improvements Length (ft)
RP-1	Gravity Main	ROW	Along Holiday Dr to Oak Ln	6	Replacement	6	400
RP-2	Gravity Main	Wright Ave	From 230 ft e/o Garden Ave to Del Monte Ave	6	Replacement	6	550
RP-3	Gravity Main	Main Ave	Frome 120 ft e/o Hale Ave to 300 ft e/o Hale Ave	6	Replacement	6	175
RP-4	Gravity Main	Monterey Rd	From 3rd to 4th Street	6	Replacement	6	350
RP-5	Gravity Main	ROW	Right of Way e/o Manor Ct to 450 ft w/o Monterey Rd and Bisceglia Ave	6	Replacement	6	200

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2/16/2017

- **RP-P3**: Replace the existing 6-inch gravity sewer along Main Avenue from 120 feet east of Hale Avenue to 300 feet east of Hale Avenue. This improvement is meant to mitigate the existing sag in the gravity main. The existing sag prevents the flushing of the gravity main.

- **RP-P4**: Replace the existing 6-inch gravity sewer along Monterey Road from 3rd Street to 4th Street. This improvement is meant to mitigate. This improvement is meant to mitigate the existing sag in the gravity main that could cause potential traffic hazard.

- **RP-P5**: Replace the existing 6-inch gravity sewer along Right of Way east of Manor Court to 450 feet west of Monterey Road and Bisceglia Avenue. This improvement is meant to mitigate the existing sag in the gravity main preventing access.

7.6 INFILTRATION AND INFLOW INVESTIGATION IMPROVEMENTS

City staff suspect there may be infiltration and inflow related deficiencies in the sewer collection system. As such, this master plan recommends a future Infiltration and Inflow (I/I) Investigation and Reduction program be initiated to identify, track, and mitigate I/I deficiencies. Should I/I defects be found and mitigated, it will reduce peak flow events during the wet weather season, help mitigate sanitary sewer overflows, and reduce the need for costly improvements to increase capacity in the collection system and the treatment plant.

The following elements may be included in a potential future I/I study:

- Pre-rehabilitation flow monitoring
- Basin delineation for high priority basins
- CCTV for high priority basins
- Rehabilitation pilot projects
- Hydraulic Model Calibration
- Post rehabilitation flow monitoring

As part of this Master Plan, preliminary I/I mitigation projects were identified from the City's existing CCTV data. While these projects were included in the condition assessment improvements discussed in a previous section, they are documented on **Figure 7.7** and **Figure 7.8** and individually in this section should City staff choose to prioritize these improvements before the condition assessment groupings. These improvements include the following:

- **INI-P1**: This improvement consists in 50 feet of trenchless rehabilitation for the 8-inch gravity main located along Llagas Road from 80 feet east of Hale Avenue to 20 feet east of Hale Avenue.

- **INI-P2**: This improvement consists in 300 feet of trenchless rehabilitation for the 8-inch gravity main located along the Llagas Road from Fox Hollow Drive to Murphy Springs Drive.

- **INI-P3**: This improvement consists of a point repair for the 6-inch gravity main located along Laurel Wood Lane from 120 feet south of Almond Orchard Drive to 135 feet south of Almond Orchard Drive.
- **INI-P4**: This improvement consists in 50 feet of trenchless rehabilitation for the 15-inch gravity main aligned 250 feet north of Berkshire Avenue from 60 feet east of Hale Avenue to 115 east of Hale Avenue.
- **INI-P5**: This improvement consists in 250 feet of trenchless rehabilitation for the 6-inch gravity main aligned 110 feet south of Wright Avenue from 180 feet west of Crest Avenue to 50 feet east of Crest Avenue.
- **INI-P6**: This improvement consists in 100 feet of trenchless rehabilitation for the 6-inch gravity main located along Shady Lane Drive from Trail Ridge Lane to Calico Ridge.
- **INI-P7**: This improvement consists in 200 feet of trenchless rehabilitation for the 6-inch gravity main located along Trail Ridge Lane from 150 feet west of Shady Lane Drive to 70 feet east of Shady Lane Drive.
- **INI-P8**: This improvement consists in 200 feet of trenchless rehabilitation for the 6-inch gravity main aligned 50 feet north of Copper Hill Place from 40 feet west of Copper Hill Drive to 60 feet west of Holiday Drive.
- **INI-P9**: This improvement consists in 250 feet of trenchless rehabilitation for the 6-inch gravity main located along Quail Lane from 150 feet east of Quail Court to 110 feet west of Quail Court.
- **INI-P10**: This improvement consists 150 feet of trenchless rehabilitation for the 6-inch gravity main aligned 175 feet south of Oakridge Court from 180 feet north of Oakridge Lane to Oakridge Lane.

7.7 MORGAN HILL/GILROY INTERCEPTOR CONDITION ASSESSMENT AND MAINTENANCE PROJECT

As discussed in a previous section, the city of Morgan Hill and city of Gilroy maintain a Joint Powers Agreement for the shared capacity of a sewer interceptor that begins south of Morgan Hill and terminates at the South County Regional Wastewater Authority Wastewater Treatment Plant in the city of Gilroy. While there have been several capacity studies for this sewer interceptor, the physical condition of the pipeline is unknown, and there have been no recorded preventative maintenance tasks completed. As this sewer interceptor is critical to the city of Morgan Hill and city of Gilroy, it is recommended that further investigation of the physical condition of the trunk be completed, and necessary preventative maintenance measures be taken to extend its useful life.

7.8 MONTEREY ROAD TRUNK IMPROVEMENTS

City staff indicate that the sewer trunk pipeline in Monterey Road south of Dunne Avenue has recently experienced sanitary sewer overflows and grease build up. Additionally, there are known physical limitations, including pipeline sags, that are contributing to capacity limitations. As such, it is recommended that field investigations be performed to properly identify potential rehabilitation methodologies that will increase the useful life of these assets. This may include up to replacing sections of the pipe that are beyond their physical life.

CHAPTER 8 - CAPITAL IMPROVEMENT PROGRAM

This chapter provides a summary of the recommended Capital Improvement Program (CIP) for the City of Morgan Hill's sewer system. The program is based on the evaluation of the City's sewer system and on the recommended projects described in the previous chapters. The CIP has been prepared to assist the City in planning and constructing the collection system improvements through the ultimate buildout scenario. This chapter also presents the cost criteria and methodologies for developing the capacity improvement costs.

8.1 COST ESTIMATE ACCURACY

Cost estimates presented in the capacity improvement costs were prepared for general master planning purposes and, where relevant, for further project evaluation. Final costs of a project will depend on several factors including the final project scope, costs of labor and material, and market conditions during construction.

The Association for the Advancement of Cost Engineering (AACE International), formerly known as the American Association of Cost Engineers, has defined three classifications. These classifications are presented in order of increasing accuracy: Order of Magnitude, Budget, and Definitive.

- **Order of Magnitude Estimate.** This classification is also known as an "original estimate", "study estimate", or "preliminary estimate", and is generally intended for master plans and studies.

  This estimate is not supported with detailed engineering data about the specific project, and its accuracy is dependent on historical data and cost indices. It is generally expected that this estimate would be accurate within -30 percent to +50 percent.

- **Budget Estimate.** This classification is also known as an "official estimate" and generally intended for pre-design studies. This estimate is prepared to include flow sheets and equipment layouts and details. It is generally expected that this estimate would be accurate within -15 percent to +30 percent.

- **Definitive Estimate.** This classification is also known as a "final estimate" and prepared during the time of contract bidding. The data includes complete plot plans and elevations, and equipment data sheets, and complete specifications. It is generally expected that this estimate would be accurate within -5 percent to +15 percent.

Costs developed in this study should be considered "Order of Magnitude" and have an expected accuracy range of **-30 percent** and **+50 percent**.

8.2 COST ESTIMATE METHODOLOGY

Cost estimates presented in this chapter are opinions of probable construction and other relevant costs developed from several sources including cost curves, Akel experience on other master planning projects, and input from City staff on the development of public and private cost sharing. Where appropriate, costs were escalated to reflect the more current Engineering News Records (ENR) Construction Cost Index (CCI).

This section documents the unit costs used in developing the opinion of probable construction costs, the Construction Cost Index, the land acquisition costs, and markups to account for construction contingency and other project related costs.

8.2.1 Unit Costs

The unit cost estimates used in developing the Capital Improvement Program are summarized on **Table 8.1**. The unit costs are intended for developing the Order of Magnitude estimate, and do not account for site specific conditions, labor or material costs during the time of construction, final project scope, implementation schedule, detailed utility and topography surveys, investigation of alternative routings for pipes, and other various factors. These factors are assumed included in the contingencies applied to the final capital improvement cost.

Unit costs were developed based on Akel Engineering Group experience, and included the following:

- Pipelines
- Cured-in-Place Pipe
- Cleaning
- CCTV

8.2.2 Construction Cost Index

Costs estimated in this study are adjusted utilizing the Engineering News Record (ENR) Construction Cost Index (CCI), which is widely used in the engineering and construction industries.

The costs in this Sewer System Master Plan were benchmarked using a 20-City national average ENR CCI of 10532, reflecting a date of January 2017.

8.2.3 Land Acquisition

Construction of pipelines is assumed to generally be within existing or future street right-of-ways. Lift station's land acquisition costs are included in the lift station unit cost.

Table 8.1 Unit Costs

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Pipelines				
Pipe Size	Improvement Type Unit Cost			
	New/Parallel/Replacement	CIPP	Cleaning	CCTV
(in)	($/unit length)	($/unit length)	($/unit length)	($/unit length)
6	156	24	1.03	1.29
8	171	37	1.03	1.29
10	185	51	1.03	1.29
12	199	64	1.03	1.29
15	270	84	1.03	1.29
18	313	105	1.03	1.29
21	370	125	1.03	1.29
24	426	145	1.03	1.29
27	498	--	--	1.29
30	569	185	--	1.29
36	711	--	--	1.29

Pipeline Casings
22$ per inch diameter per linear foot

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Notes :

2/17/2017

1. Unit costs are based on an ENR CCI Index Value of 10,532 (01/2017)

8.2.4 Construction Contingency Allowance

Knowledge about site-specific conditions for each proposed project is limited at the master planning stage; therefore, construction contingencies were used. The estimated construction costs in this master plan include a **30 percent** contingency allowance to account for unforeseen events and unknown field conditions.

8.2.5 Project Related Costs

The capital improvement costs also account for project-related costs, comprising of engineering design, project administration (developer and City staff), construction management and inspection, and legal costs. The project related costs in this master plan were estimated by applying an additional **30 percent** to the estimated construction costs.

8.3 CAPITAL IMPROVEMENT PROGRAM

The Capacity Improvement Costs for the previously identified projects in this master plan for mitigating existing system deficiencies and for serving anticipated future growth throughout the City are summarized on **Table 8.2**. The Capital Improvement Program lists the type of improvement, location, cost, construction trigger, suggested phasing, and cost sharing.

8.3.1 Pipelines

The recommended pipeline improvements are grouped by collection basin and listed on **Table 8.2**. Each improvement includes a general description of the street alignment and limits as well as existing pipe diameter and length.

The following three pipeline improvements categories were identified:

- **New Pipeline.** The new pipeline is proposed where none exists.
- **Replacement Pipeline.** This improvement is intended as a replacement to an existing pipeline and along the same alignment. The existing pipeline should be abandoned when the replacement pipeline has been constructed.
- **Parallel Pipeline.** This improvement is intended as a parallel to an existing pipeline. The existing pipeline should remain in service, even when this new improvement is constructed.

The opinion of probable construction costs, for the projects included in this master plan, are based on the pipe unit costs summarized on **Table 8.1**.

It is assumed that any replacement pipes will be in the same alignment and at the same slope as the existing pipe. However, this study recommends an investigation of the alignment during the pre-design stage of each project.

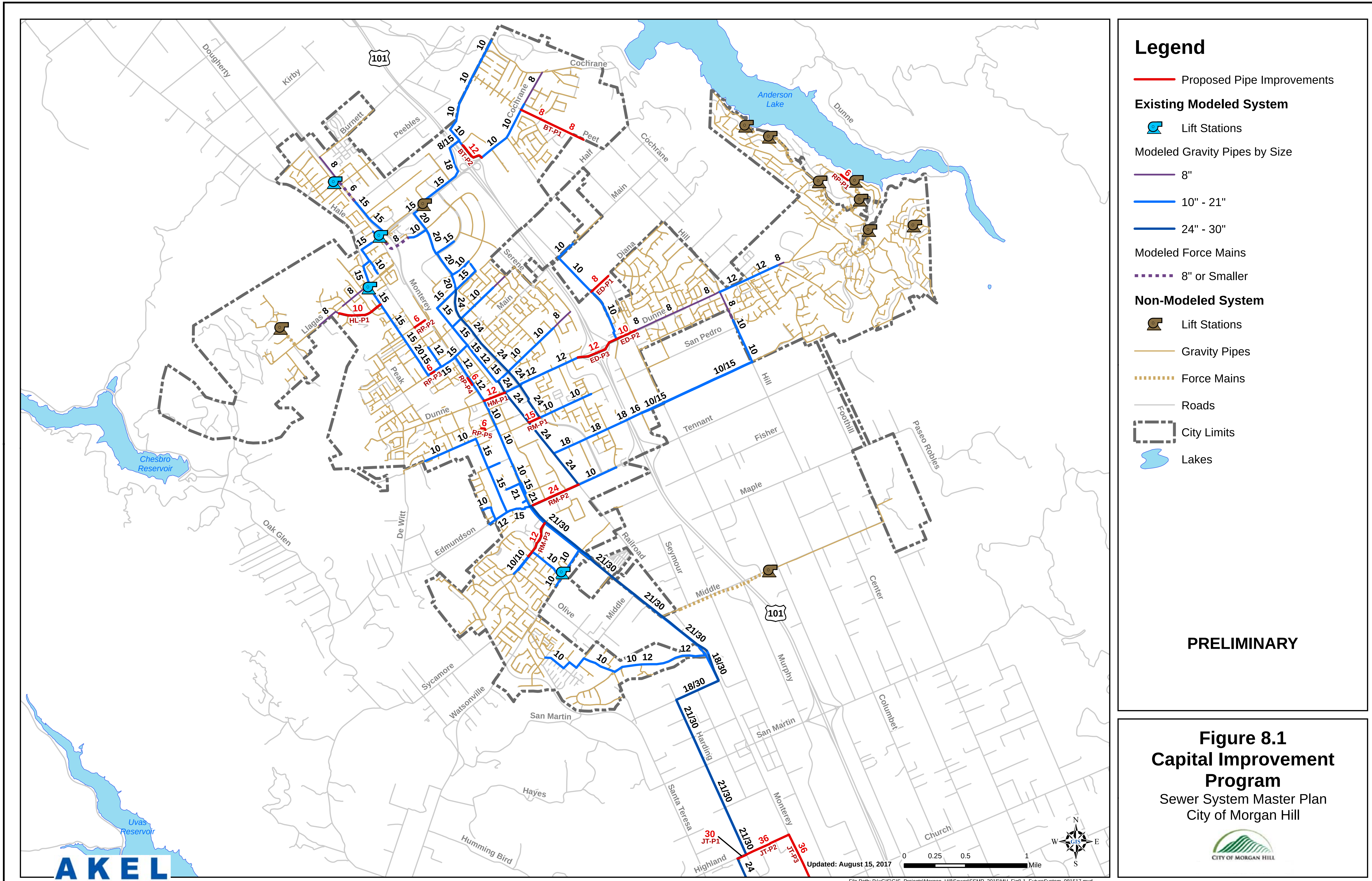

Figure 8.1
Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

Table 8.2 Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements: Existing Diameter (in)	Pipeline Improvements: New/Parallel/Replace	Infrastructure Costs: Diameter (in)	Infrastructure Costs: Length (ft)	Infrastructure Costs: Unit Cost ($)	Infrastructure Costs: Infr. Cost ($)	Baseline Constr. Costs [1] ($)	Estimated Const. Costs [2] ($)	Capacity Improv. Cost [3] ($)	Suggested Expenditure Budget	Construction Trigger (EDUs)	Suggested Cost Allocation: Existing Users (%)	Suggested Cost Allocation: Future Users (%)	Cost Allocation: Existing Users ($)	Cost Allocation: Future Users ($)
Pipeline Capacity Improvements																		
Butterfield Trunk																		
BT-P1	Gravity Main	Peet Rd	From approximately 3,000 ft e/o Cochrane Rd to Cochrane Rd	-	New	8	3,000	171	511,745	511,745	665,269	864,849	2026-2030	-	0%	100%	0	864,849
BT-P2	Gravity Main	Along NB US 101	From 900 ft n/o Cochrane Rd to intersection of Cochrane Rd and NB US 101	10	Replacement	12	1,200	199	238,814	238,814	310,459	403,596	2026-2030	1,900	51%	49%	204,471	199,125
									Subtotal - Butterfield Trunk	**750,560**	**975,727**	**1,268,446**						
Hale-Llagas Trunk																		
HL-P1	Gravity Main	Llagas Creek Dr	From Eagle Springs Ct to Hale Ave	8	Replacement	10	1,950	185	360,354	360,354	468,460	608,998	2021-2025	675	72%	28%	435,981	173,017
									Subtotal - Hale-Llagas Trunk	**360,354**	**468,460**	**608,998**						
East Dunne Trunk																		
ED-P1	Gravity Main	Diana Ave	From Murphy Ave to Condit Rd	-	New	8	1,000	171	170,582	170,582	221,756	288,283	2026-2030	-	0%	100%	0	288,283
ED-P2	Gravity Main	Dunne Ave	From 230 ft e/o Murphy Ave to Condit Rd	8	Replacement	10	950	185	175,557	175,557	228,224	296,691	2021-2025	1,525	83%	17%	247,639	49,052
ED-P3	Gravity Main	Dunne Ave	From Condit Rd to 530 ft e/o Walnut Grove Dr	8/10	Replacement	12	1,950	199	388,073	388,073	504,495	655,844	2021-2025	2,400	47%	53%	309,083	346,762
									Subtotal - East Dunne Trunk	**734,212**	**954,476**	**1,240,819**						
Hale-Monterey Trunk																		
HM-P1	Gravity Main	Dunne Ave	From Monterey Rd to Railroad Ave	-	New	12	1,000	199	199,012	199,012	258,716	336,330	2018-2020	0	80%	20%	270,210	66,121
RP-P2	Gravity Main	Wright Ave	From 230 ft e/o Garden Ave to Del Monte Ave	6	Replacement	6	550	156	86,002	86,002	111,802	145,343	2018-2020	0	100%	0%	145,343	0
RP-P3	Gravity Main	Main Ave	Frome 120 ft e/o Hale Ave to 300 ft e/o Hale Ave	6	Replacement	6	175	156	27,364	27,364	35,573	46,245	2018-2020	0	100%	0%	46,245	0
RP-P4	Gravity Main	Monterey Rd	From 3rd to 4th Street	6	Replacement	6	350	156	54,728	54,728	71,147	92,491	2018-2020	0	100%	0%	92,491	0
RP-P5	Gravity Main	ROW	Right of Way e/o Manor Ct to 450 ft w/o Monterey Rd and Bisceglia Ave	6	Replacement	6	200	156	31,273	31,273	40,655	52,852	2018-2020	0	100%	0%	52,852	0
									Subtotal - Hale-Monterey Trunk	**398,379**	**517,893**	**673,261**						
Railroad-Monterey Trunk																		
RM-P1	Gravity Main	San Pedro Ave	From Butterfield Blvd to Railroad Ave	10	Replacement	15	550	270	148,548	148,548	193,113	251,047	2026-2030	2,000	58%	42%	146,509	104,537
RM-P2 [4]	Gravity Main	Tennant Ave	From RailRoad Ave to Monterey Rd	18	Replacement	24	2,200	426	938,199	1,131,799	1,471,339	1,912,741	2018-2020	3,175	61%	39%	1,170,082	742,659
RM-P3	Gravity Main	La Crosse Dr / Vineyard Blvd	From La Mar Dr to Monterey Rd	10	Replacement	12	1,700	199	338,320	338,320	439,817	571,762	2021-2025	0	92%	8%	527,499	44,262
									Subtotal - Railroad-Monterey Trunk	**1,618,668**	**2,104,269**	**2,735,549**						
Hill-Barrett Trunk																		
RP-P1	Gravity Main	ROW	Along Holiday Dr to Oak Ln	6	Replacement	6	400	156	62,547	62,547	81,311	105,704	2018-2020	0	100%	0%	105,704	0
									Subtotal - Hill-Barrett Trunk	62,547	81,311	105,704						
Joint Trunk [5]																		
JT-P1	Gravity Main	Highland Ave	From Harding Ave to 400 ft w/o Harding Ave	21	Replacement	30	450	569	255,873	255,873	294,253	338,391	2018-2020	0	100%	0%	338,391	0
									Subtotal - Joint Trunk	255,873	294,253	338,391						
									Subtotal - Pipeline Capacity Improvements	**4,180,592**	**5,396,389**	**6,971,168**					**4,092,500**	**2,878,668**
Relief Trunk Improvements [5]																		
JT-P2	Gravity Main	Highland Ave	From Harding Ave to Monterey Rd	-	New	30	2,050	569	1,165,642	1,165,642	1,340,488	1,541,561	2018-2020	-	25%	75%	385,390	1,156,171
JT-P3	Gravity Main	Monterey Rd	From Highland Ave to Las Animas Ave	-	New	36	19,700	569	11,201,533	11,201,533	12,881,763	14,814,028	2018-2020	-	25%	75%	3,703,507	11,110,521
JT-P4	Gravity Main	Las Animas Ave	From Monterey Rd to Murray Ave	-	New	36	1,750	569	995,060	995,060	1,144,319	1,315,967	2018-2020	-	25%	75%	328,992	986,975
JT-P5	Gravity Main	Murray Ave	From Las Animas Ave to Chestnut St	-	New	36	7,550	569	4,292,973	4,292,973	4,936,919	5,677,457	2018-2020	-	25%	75%	1,419,364	4,258,093
JT-P6	Gravity Main	Chestnut St	From Murray Ave to Lewis St	-	New	36	400	569	227,442	227,442	261,559	300,792	2018-2020	-	25%	75%	75,198	225,594
JT-P7	Gravity Main	Chestnut St	From Chestnut St to 7th St	-	New	36	2,100	569	1,194,072	1,194,072	1,373,183	1,579,160	2018-2020	-	25%	75%	394,790	1,184,370

Table 8.2 Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements		Infrastructure Costs				Baseline Constr. Costs [1]	Estimated Const. Costs [2]	Capacity Improv. Cost [3]	Suggested Expenditure Budget	Construction Trigger	Suggested Cost Allocation		Cost Allocation	
				Existing Diameter	New/Parallel/ Replace	Diameter	Length	Unit Cost	Infr. Cost						Existing Users	Future Users	Existing Users	Future Users
				(in)		(in)	(ft)	($)	($)	($)	($)	($)		(EDUs)	(%)	(%)	($)	($)
JT-P8	Gravity Main	7th Street	From Chestnut St to US Highway 101.	-	New	36	1,450	569	824,478	824,478	948,150	1,090,373	2018-2020	-	25%	75%	272,593	817,779
JT-P9	Gravity Main	Along US 101	Jogging from 7th St and US Highway 101 to Renz Lane	-	New	36	2,000	569	1,137,211	1,137,211	1,307,793	1,503,962	2018-2020	-	25%	75%	375,991	1,127,972
									Subtotal - Joint Trunk Improvements	**21,038,413**	**24,194,175**	**27,823,301**					**6,955,825**	**20,867,476**
Joint Trunk Condition Assessment [6]																		
CCTV and Condition Assessment - Existing Joint Trunk							24,369		200,000	-	-	200,000	2018-2020		100%	0%	200,000	0
									Subtotal - Joint Trunk Condition Assessment	**-**	**-**	**200,000**					**200,000**	**0**
Infiltration and Inflow Improvements																		
INI-P1	Gravity Main	Llagas Rd	From 80 ft e/o Hale Ave to 20 ft e/o Hale Ave (Group 5)	8	Trenchless Rehabilitation	8	100	41	4,127	4,127	5,365	6,975	2018-2020	-	100%	0%	6,975	0
INI-P2	Gravity Main	Llagas Rd	From Fox Hollow Cir to Murphy Springs Dr (Group 5)	8	Trenchless Rehabilitation	8	350	41	14,446	14,446	18,779	24,413	2018-2020	-	100%	0%	24,413	0
INI-P3	Gravity Main	Laurel Wood Ln	From 120 fts/o Almond Orchard Dr to 135 ft s/o Almond Orchard Dr (Grp 5)	6	Point Repair	6	15	162	2,429	2,429	3,157	4,104	2018-2020	-	100%	0%	4,104	0
INI-P4	Gravity Main	250 ft n/o Berkshire Ave	From 60 ft e/o Hale Ave to 115 ft e/o Hale Ave (Group 5)	15	Trenchless Rehabilitation	15	100	67	6,664	6,664	8,664	11,263	2018-2020	-	100%	0%	11,263	0
INI-P5	Gravity Main	110 ft s/o Wright Ave	From 180 ft w/o Crest Ave to 50 ft e/o Crest Ave (Group 4)	6	Trenchless Replacement	6	250	792	198,067	198,067	257,487	334,733	2018-2020	-	100%	0%	334,733	0
INI-P6	Gravity Main	Shady Lane Dr	From Trail Ridge Ln to Calico Ridge Trl (Group 2)	6	Trenchless Rehabilitation	6	150	46	6,965	6,965	9,054	11,771	2018-2020	-	100%	0%	11,771	0
INI-P7	Gravity Main	Trail Ridge Ln	From 150 ft w/o Shady Lane Dr to 70 ft e/o Shady Lane Dr (Group 2)	6	Trenchless Replacement	6	250	792	198,067	198,067	257,487	334,733	2018-2020	-	100%	0%	334,733	0
INI-P8	Gravity Main	50 ft n/o Copper Hill Pl	From 40 ft w/o Copper Hill Dr to 60 ft w/o Holiday Dr (Group 2)	6	Trenchless Rehabilitation	6	200	46	9,286	9,286	12,072	15,694	2018-2020	-	100%	0%	15,694	0
INI-P9	Gravity Main	Quail Ln	From 150 ft e/o Quail Ct to 110 ft w/o Quail Ct (Group 2)	6	Trenchless Rehabilitation	6	300	46	13,930	13,930	18,109	23,541	2018-2020	-	100%	0%	23,541	0
INI-P10	Gravity Main	175 ft s/o Oakridge Ct	From 180 ft n/o Oakridge Ln to Oakridge Ln (Group 1)	6	Trenchless Rehabilitation	6	200	46	9,286	9,286	12,072	15,694	2018-2020	-	100%	0%	15,694	0
									Subtotal - Infiltration and Inflow Improvements	**463,267**	**602,247**	**782,921**					**782,921**	**0**
Rehabilitation Improvements																		
Group 1	Gravity Main	Various	See Group 1 Figure	Various	Various	Various	7,750	Various	2,426,606	2,426,606	3,154,588	4,100,964	2018	-	100%	0%	4,100,964	0
Group 2	Gravity Main	Various	See Group 2 Figure	Various	Various	Various	9,800	Various	1,167,715	1,167,715	1,518,029	1,973,438	2019	-	100%	0%	1,973,438	0
Group 3	Gravity Main	Various	See Group 3 Figure	Various	Various	Various	5,650	Various	363,053	363,053	471,968	613,559	2019	-	100%	0%	613,559	0
Group 4	Gravity Main	Various	See Group 4 Figure	Various	Various	Various	10,300	Various	907,288	907,288	1,179,475	1,533,317	2020	-	100%	0%	1,533,317	0
Group 5	Gravity Main	Various	See Group 5 Figure	Various	Various	Various	6,000	Various	371,370	371,370	482,781	627,615	2020	-	100%	0%	627,615	0
Group 6	Gravity Main	Various	See Group 6 Figure	Various	Various	Various	5,550	Various	597,377	597,377	776,590	1,009,566	2021	-	100%	0%	1,009,566	0
Group 7	Gravity Main	Various	See Group 7 Figure	Various	Various	Various	8,950	Various	1,784,493	1,784,493	2,319,841	3,015,794	2021	-	100%	0%	3,015,794	0
Group 8	Gravity Main	Various	See Group 8 Figure	Various	Various	Various	5,700	Various	653,074	653,074	848,996	1,103,695	2022	-	100%	0%	1,103,695	0
Group 9	Gravity Main	Various	See Group 9 Figure	Various	Various	Various	2,900	Various	356,669	356,669	463,670	602,771	2022	-	100%	0%	602,771	0
									Subtotal - Rehabilitation Improvements	**8,627,644**	**11,215,937**	**14,580,719**					**14,580,719**	**0**
Comprehensive Plan Updates																		
Sewer System Master Plan Updates (Years 2021, 2026, 2031, 2036)									200,000	-	-	800,000	2021, 2026, 2031, 2036		65%	35%	520,000	280,000
Sewer System Management Plan Updates (Years 2021, 2026, 2031, 2036)									100,000	-	-	400,000	2021, 2026, 2031, 2036		65%	35%	260,000	140,000
Sewer Rate Study Updates (Years 2021, 2026, 2031, 2036)									100,000	-	-	400,000	2021, 2026, 2031, 2036		65%	35%	260,000	140,000
									Subtotal - Comprehensive Plan Updates			**1,600,000**					**1,040,000**	**560,000**
On-Going CCTV Sewer System																		
CCTV of 16 miles of pipelines per year (From 2018 to 2035)							84,480	1.50	2,280,960	-	-	2,280,960	126,720 Annually		100%	0%	2,280,960	0
									Subtotal - On-going CCTV System			**2,280,960**					**2,280,960**	**0**

Table 8.2 Capital Improvement Program
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

Improv. No.	Type of Improvement	Alignment	Limits	Pipeline Improvements: Existing Diameter (in)	Pipeline Improvements: New/Parallel/Replace	Infrastructure Costs: Diameter (in)	Infrastructure Costs: Length (ft)	Infrastructure Costs: Unit Cost ($)	Infrastructure Costs: Infr. Cost ($)	Baseline Constr. Costs [1] ($)	Estimated Const. Costs [2] ($)	Capacity Improv. Cost [3] ($)	Suggested Expenditure Budget	Construction Trigger (EDUs)	Suggested Cost Allocation: Existing Users (%)	Suggested Cost Allocation: Future Users (%)	Cost Allocation: Existing Users ($)	Cost Allocation: Future Users ($)
Currently Planned Projects																		
Sewer Plant Expansion (SCRWA)												32,700,000	2017-2024		0%	100%	0	32,700,000
Sewer Plant Maintenance/ Improvements (SCRWA)												9,430,000	2017-2021		100%	0%	9,430,000	0
Holiday Lakes Gravity Line Feasibility Study												60,000	2018		100%	0%	60,000	0
Lift Station Condition Assessment												80,000	2019		100%	0%	80,000	0
Lift Station W Repair and Refurbish												1,000,000	2017-2018		100%	0%	1,000,000	0
Inflow and Infiltration Investigation and Cross Connection Elimination												300,000	2017-2020		100%	0%	300,000	0
Wastewater Collection System Compliance Inspection[7]												10,000,000	2017-2024		100%		10,000,000	0
									Subtotal - Currently Planned Projects			**53,570,000**					**20,870,000**	**32,700,000**
Total Costs																		
									Pipeline Capacity Improvements									
									Collection System Capacity Improvements	**4,180,592**	**5,396,389**	**6,971,168**					**4,092,500**	**2,878,668**
									Relief Trunk Improvements	**21,038,413**	**24,194,175**	**27,823,301**					**6,955,825**	**20,867,476**
									Subtotal	**25,219,005**	**29,590,564**	**34,794,469**					**11,048,325**	**23,746,144**
									Condition Assessment Improvements									
									Joint Trunk Condition Assessment	**200,000**	**200,000**	**200,000**					**200,000**	**0**
									Infiltration and Inflow Improvements	**463,267**	**602,247**	**782,921**					**782,921**	**0**
									Rehabilitation Improvements	**8,627,644**	**11,215,937**	**14,580,719**					**14,580,719**	**0**
									On-Going CCTV System	**2,280,960**	**2,280,960**	**2,280,960**					**2,280,960**	**0**
									Subtotal	**11,571,871**	**14,299,145**	**17,844,600**					**17,844,600**	**0**
									Plan Updates									
									Comprehensive Plan Updates	**1,600,000**	**1,600,000**	**1,600,000**					**1,040,000**	**560,000**
									Planned Project (Including SCRWA Plant)									
									Currently Planned Projects	**53,570,000**	**53,570,000**	**53,570,000**					**20,870,000**	**32,700,000**
									Total Improvement Costs	**91,960,876**	**99,059,708**	**107,809,069**					**50,802,925**	**57,006,144**

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Notes :

1. Cost estimates are based on the Engineering News Record (ENR) construction cost index (CCI) of 10532 for January 2017.
2. Baseline construction costs plus 30% to account for unforeseen events and unknown conditions.
3. Estimated construction cost plus 30% to cover other costs including: engineering design, project administration (developer and City staff), construction management and inspection, and legal costs.
4. Improvement RM-P2 will require a casing where crossing railroad. Casing length assumed to be equal to 200 ft.
5. The Joint Relief Trunk improvements are currently in the design process. As such, contingencies are reduced from 30% to 15% for this project.
6. Joint Trunk Condition Assessment extents start at from the intersection of Monterey Highway and California Avenue to Day Road.
7. This item estimates the potential overall cost for inspections and rehabilitations related to sewer collection system condition comliance.

8.3.2 Construction Triggers

The CIP improvements are prioritized based on their urgency to mitigate existing deficiencies and to serve future growth. The construction triggers for each improvement are as follows:

Existing Users

- It is recommended that improvements for the existing deficiencies be constructed as soon as possible.

Future Users

- The amount of equivalent dwelling units (EDUs) that the existing pipe can handle before a replacement or parallel pipe will have to be constructed.

8.3.3 Construction Phasing

The Capital Improvement Program was divided into the following phases:

General Plan Horizon:

- **Near Term:** This short-term phase consists of improvements for the fiscal years (FY) 2018 through 2020 for improvements that are required to resolve existing deficiencies and other critical pipes in the sewer system.

- **Intermediate Term:** This intermediate term phase includes improvements that are required to be completed for fiscal years 2021 through 2025.

- **Long Term:** This long-term phasing includes improvements that are required to be completed for fiscal years 2026 through 2035.

City staff has cited certain improvements to have more specificity and those have been assigned certain Phases based on growth assumptions. This phasing plan is subject to revisions by City staff based on how new developments occur. The City is capable of allocating larger resources based on the necessity of the projects and will perform updated reassessments as necessary.

8.3.4 Recommended Cost Allocation Analysis

Capacity allocation analysis is needed to identify improvement funding sources, and to establish a nexus between development impact fees and improvements needed to service growth. In compliance with the provisions of Assembly Bill AB 1600, the analysis differentiates between the project needs of servicing existing users and for those required to service anticipated future developments. Table 8.2 lists each improvement and separates the cost by responsibility between existing and future users. The cost responsibility is based on model parameters for existing and future land use, and may change depending on the nature of development.

8.4 SUGGESTED EXPENDITURE BUDGET

This section discusses the suggested expenditure budget for the capital improvement plan horizon, and the recommended sequence of construction for capital improvement planning.

8.4.1 5-year Capital Improvement Costs and Phasing

The capital improvement program costs and phasing for the next five fiscal years are summarized on **Table 8.3**, this plan includes the total costs for pipelines improvements, I&I improvement and rehabilitation improvements to be constructed in the next five fiscal years (FY). The improvements listed are also categorized by improvement classification, indicating whether the improvement is intended to upgrade, expand, or replace the existing water distribution system infrastructure.

8.4.2 Suggested Expenditure Budget

The suggested expenditure budget is shown on **Table 8.4**, and includes the total costs for pipelines and pump stations phased by 5-year fiscal period through the year 2035. Costs are categorized through the General Plan horizon of 2035 for near-term, immediate term, and long-term planning.

8.4.3 Sequence of Construction

Suggested expenditure budget phasing is intended to provide general guidance for implementing the capital improvement projects listed in this master plan. The sequence of construction on **Table 8.4** for the near-term and intermediate term improvements accounts for projects that City Staff has identified as having immediate benefit. Additional improvements may be constructed as developments occurs and the phasing and implementation of a sequence of construction is subject to the approval of the City Engineer.

Table 8.3 5-year Improvement Phasing
Sewer System Master Plan
City of Morgan Hill

PRELIMINARY

CIP ID	Year Range	Upgrade	Expansion	Repair & Replacement	Fiscal Year Improvement Phasing FY 2017	FY 2018	FY 2019	FY 2020	FY 2021	FY 2022	FY 2017-2022 Total
		%	%	%	($)	($)	($)	($)	($)	($)	($)
Pipeline Improvements											
Operational Improvements											
RP-P1	2017	0%	0%	100%	105,704	0	0	0	0	0	105,704
RP-P2	2017	0%	0%	100%	145,343	0	0	0	0	0	145,343
RP-P3	2017	0%	0%	100%	46,245	0	0	0	0	0	46,245
RP-P4	2017	0%	0%	100%	92,491	0	0	0	0	0	92,491
RP-P5	2017	0%	0%	100%	52,852	0	0	0	0	0	52,852
		Subtotal -Operational Improvements			442,635	0	0	0	0	0	442,635
Capacity Improvements											
HM-P1	2017	80%	20%	0%	312,307	0	0	0	0	0	312,307
RM-P2 4	2018	61%	39%	0%		1,912,741					1,912,741
RM-P3	2017	92%	8%	0%	571,762	0	0	0	0	0	571,762
HL-P1	2019	72%	28%	0%	0	0	608,998	0	0	0	608,998
ED-P2	2021	83%	17%	0%	0	0	0	0	296,691	0	296,691
BT-P2	2021	51%	49%	0%	0	0	0	0	403,596	0	403,596
JT-P1	2018	0%	100%	0%	0	338,391	0	0	0	0	338,391
		Subtotal - Pipeline Capacity Improvements			884,068	2,251,133	608,998	0	700,288	0	4,444,487
		Subtotal - Pipeline Improvements			**1,326,703**	**2,251,133**	**608,998**	**0**	**700,288**	**0**	**4,887,121**
Relief Trunk Improvements											
	2018	25%	75%		0	27,823,301	0	0	0	0	27,823,301
		Subtotal-Joint Trunk Improvements			**0**	**27,823,301**	**0**	**0**	**0**	**0**	**27,823,301**
Joint Trunk Condition Assessment											
CCTV and Condition Assessment	2017				200,000	0	0	0	0	0	200,000
		Subtotal - Joint Trunk Condition Assessment			**200,000**	**0**	**0**	**0**	**0**	**0**	**200,000**
Infiltration and Inflow											
I&I Improvements	2017	0%	0%	100%	782,921	0	0	0	0	0	782,921
		Subtotal - Infiltration and Inflow Improvements			**782,921**	**0**	**0**	**0**	**0**	**0**	**782,921**
Rehabilitation											
Group 1	2018	0%	0%	100%	0	4,100,964	0	0	0	0	4,100,964
Group 2	2019	0%	0%	100%	0	0	1,973,438	0	0	0	1,973,438
Group 3	2019	0%	0%	100%	0	0	613,559	0	0	0	613,559
Group 4	2020	0%	0%	100%	0	0	0	1,533,317	0	0	1,533,317
Group 5	2020	0%	0%	100%	0	0	0	627,615	0	0	627,615
Group 6	2020	0%	0%	100%	0	0	0	1,009,566	0	0	1,009,566
Group 7	2021	0%	0%	100%	0	0	0	0	3,015,794	0	3,015,794
Group 8	2022	0%	0%	100%	0	0	0	0	0	1,103,695	1,103,695
Group 9	2022	0%	0%	100%	0	0	0	0	0	602,771	602,771
		Subtotal - Rehabilitation Improvements			**0**	**4,100,964**	**2,586,996**	**3,170,499**	**3,015,794**	**1,706,466**	**14,580,719**
Comprehensive Plan Updates											
Sewer System Master Plan Updates	2022				0	0	0	0	0	200,000	200,000
Sewer System Management Plan Updates	2022				0	0	0	0	0	100,000	100,000
Sewer Rate Study Updates	2022				0	0	0	0	0	100,000	100,000
		Subtotal - Comprehensive Plan Updates			**0**	**0**	**0**	**0**	**0**	**400,000**	**400,000**
On-Going CCTV Sewer System											
	2017-2022				126,270	126,270	126,270	126,270	126,270	126,270	126,270
		Subtotal - On-going CCTV Sewer System			**126,270**	**126,270**	**126,270**	**126,270**	**126,270**	**126,270**	**757,620**
Currently Planned Projects											
Sewer Plant Expansion (SCRWA) [2]	2017-2024	0%	100%	0%	560,000	420,000	420,000	10,500,000	8,400,000	8,400,000	32,700,000
Sewer Plant Maintenance/ Improvements	2017-2021	0%	0%	100%	4,500,000	2,600,000	830,000	750,000	750,000	0	9,430,000
Holiday Lakes Gravity Line Feasibility Study	2018	0%	0%	100%	0	60,000	0	0	0	0	60,000
Lift Station Condition Assessment	2019	0%	0%	100%			80,000	0	0	0	80,000
Lift Station W Repair and Refurbish	2017-2018	0%	0%	100%	750,000	250,000		0	0	0	1,000,000
Inflow and Infiltration Investigation and Cross Connection Elimination	2017-2020	0%	0%	100%	75,000	75,000	75,000	75,000	0	0	300,000
Wastewater Collection System Compliance Inspection	2017-2021	0%	0%	100%	1,000,000	1,000,000	1,100,000	2,300,000	2,300,000	2,300,000	10,000,000
		Subtotal - Currently Planned Projects			6,885,000	4,405,000	2,505,000	13,625,000	11,450,000	10,700,000	**53,570,000**
Total Improvement Costs											
		Fiscal Year Total			$9,320,894	$38,706,667	$5,827,264	$16,921,769	$15,292,351	$12,932,736	$103,001,682
		Cumulative Total			$9,320,894	$48,027,562	$53,854,826	$70,776,595	$86,068,946	$99,001,682	**$103,001,682**

8/15/2017

Notes:
1. This short-term and expenditure budget is not set, and is dependent on the City's rate of growth. The City is not bound by this budget and may implement capital improvement projects as funding is available.
2. Currently Planned Project "Sewer Plant Expansion" doesn't include additional costs of 4,000,000 USD in FY 2023 for a total project cost of 32,700,000USD.

Table 8.4 Suggested Expenditure Budget

Sewer System Master Plan

City of Morgan Hill

PRELIMINARY

Project Type	Relief Trunk	Suggested Expenditure Budget[1]			
		General Plan Horizon			
		Near-Term	Intermediate Term	Long-Term	
		2018-2020	2021-2025	2026-2030	2031-2035
Pipeline Capacity	$27,823,301	$3,030,098	$2,133,295	$1,807,775	$0
Infiltration and Inflow		$782,921	$0	$0	$0
Rehabilitation		$8,848,893	$5,731,826	$0	$0
Joint Trunk Condition Assessment		$200,000	$0	$0	$0
Comprehensive Plan Updates		$400,000	$400,000	$400,000	$400,000
CCTV-Sewer System		$380,160	$633,600	$633,600	$633,600
Currently Planned Projects		$27,420,000	$26,150,000	$0	$0
Total	**$27,823,301**	**$41,062,072**	**$35,048,721**	**$2,841,375**	**$1,033,600**
Cumulative Cost	$27,823,301	$68,885,373	$103,934,093	$106,775,469	$107,809,069

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Notes:

1. This expenditure budget is suggested, and is dependent on the City's rate of growth. The City is not bound by this budget and may implement capital improvement projects as funding is available.

APPENDICES

APPENDIX A

Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, 2014 (V&A)



SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

City of Morgan Hill, CA

May 2014

V&A

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

CITY OF MORGAN HILL

Prepared for

Akel Engineering Group, Inc.
7433 N. First Street, Suite 103
Fresno, CA 93720

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
QA/QC	quality assurance/quality control
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
Attenuation	Flow attenuation in a sewer collection system is the natural process of the reduction of the peak flow rate through redistribution of the same volume of flow over a longer period of time. This occurs as a result of friction (resistance), internal storage and a tendency to reach a steady state along the sewer pipes. As the flows from the basins combine within the trunk sewer lines, (a) the peaks from each basin will not necessary coincide at the same time, and (b) due to the length and time of travel through the trunk sewers, peak flows will ***attenuate*** as the peak flows move downstream. The sum of the peak flows of individual basins upstream will generally be greater than the measured peak flows observed at points downstream.
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 the 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. **Combined 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 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

Term	Definition
	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 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 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:

Term	Definition
	❖ 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 (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 of 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 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 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 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 was retained by Akel Engineering Group to perform sanitary sewer flow monitoring and inflow and infiltration (I/I) analysis within the City of Morgan Hill, California (City). Flow monitoring was performed over a 20-day period at ten open-channel flow monitoring sites within the City. The flow monitoring period began on February 25, 2014, and ended on March 17, 2014. The purpose of this study was to measure sanitary sewer flows at the flow monitoring sites and estimate available sewer capacity and infiltration and inflow (I/I) occurring in the basins upstream from the flow monitoring sites.

V&A had access to flow monitoring data from the Harding Avenue Flow Meter. This meter measures the total flow from the City of Morgan Hill through a single 21-inch transmission sewer line sewer prior to entering into the City of Gilroy collection system. Flows from this meter during the flow monitoring period are also presented in this report and used to establish system totals.

Site Flow Monitoring and Capacity Results

Peak measured flows and the corresponding flow levels (depths) are important to understand the capacity limitations of a collection 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 site locations; hydraulic conditions in other areas of the collection system will differ.

Table 1. Capacity Analysis Summary

Metering Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	Peak *d*/*D* Ratio	Level Surcharged above Crown (ft)
Site 1	0.19	0.56	2.93	18	3.73	0.21	-
Site 2	0.36	0.88	2.45	19.75	7.94	0.40	-
Site 3	0.32	0.93	2.95	15	13.13	0.88	-
Site 4	0.21	0.53	2.49	12	6.04	0.50	-
Site 5	1.15	2.29	1.98	24	15.95	0.66	-
Site 6	0.35	0.91	2.60	17.75	12.52	0.71	-
Site 7	0.24	0.77	3.17	21	6.82	0.32	-
Site 8	0.16	0.43	2.73	17.5	6.76	0.39	-
Site 9	0.13	0.41	3.03	15	6.36	0.42	-
Site 10	0.12	0.31	2.45	11.75	4.57	0.39	-

The following capacity analysis results are noted:

- **Peaking Factor:** Sites 7 and 9 had peaking factors that exceeded typical design threshold limits for the ratio of peak flow to average dry weather flow.
- ***d/D* Ratio:** Only Site 3 had a *d/D* ratio that exceeded the common design threshold for *d/D* ratio. None of the flow monitoring sites reached surcharge conditions.

Figure 1 shows bar graphs of the capacity results. Figure 2 shows a schematic diagram of the peak measured flows with peak flow levels.

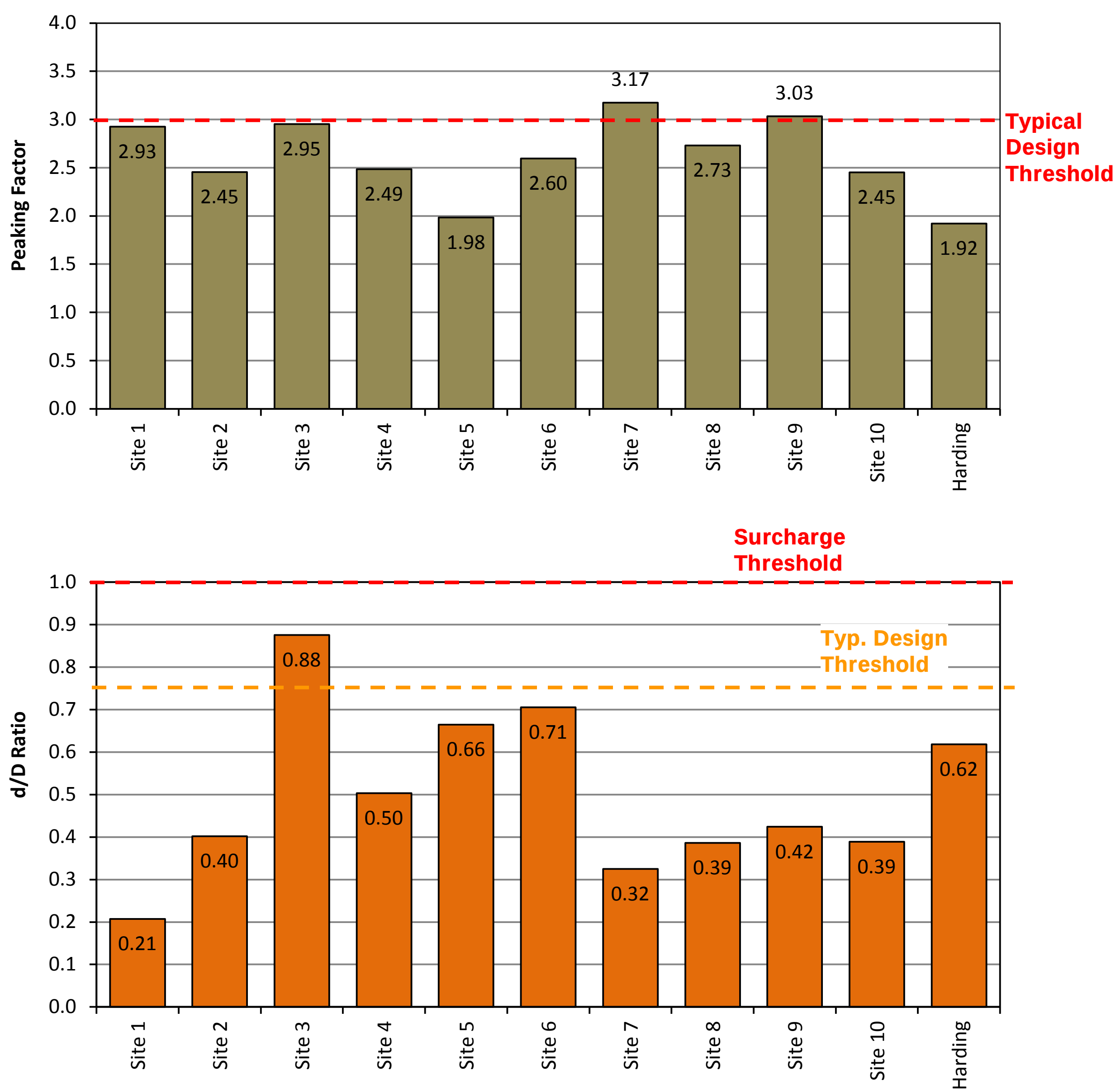

Figure 1. Capacity Summary Bar Graphs: Peaking Factors and Peak *d/D* Ratios

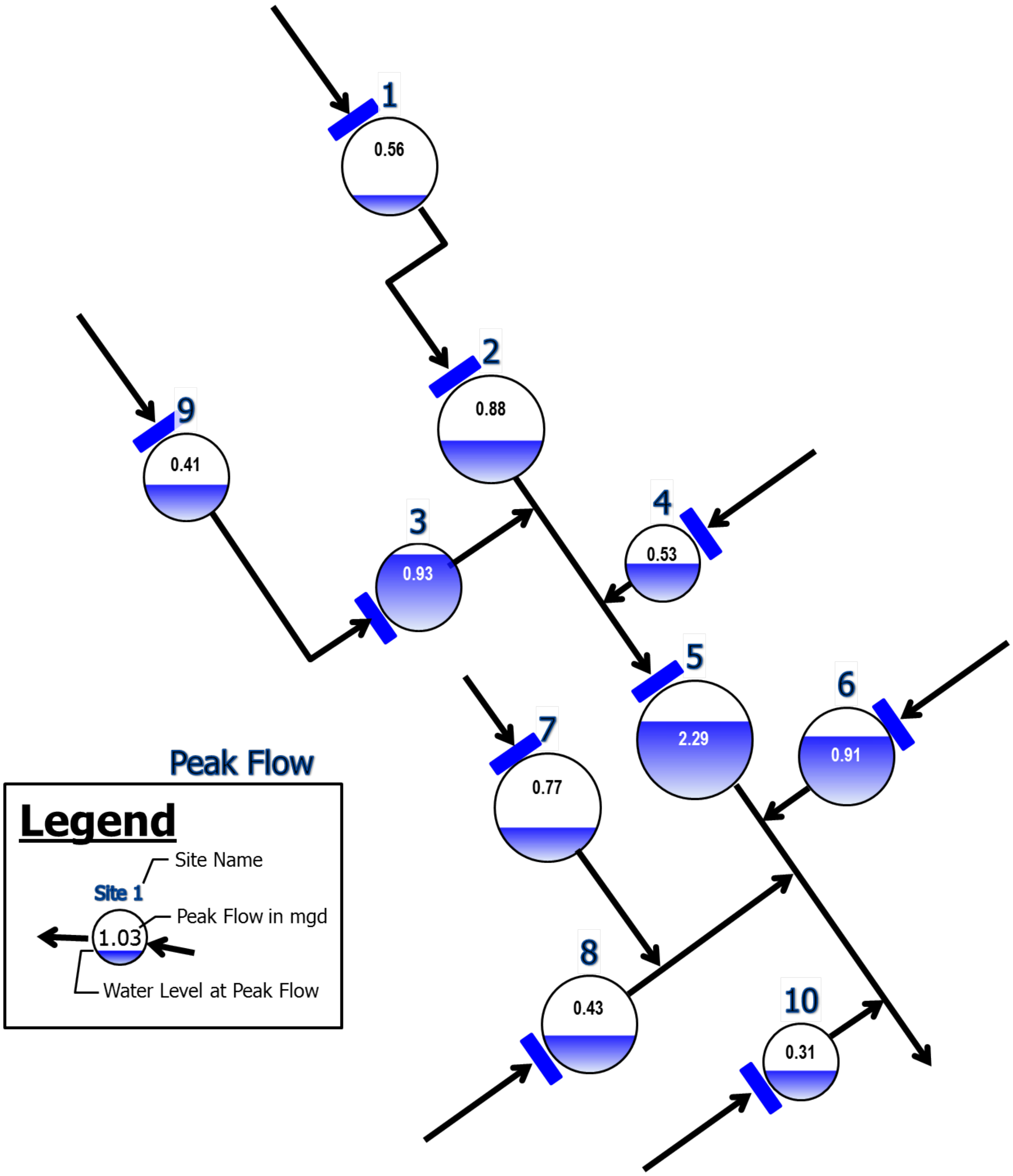

Figure 2. Peak Measured Flow (Flow Schematic)

Basin Inflow and Infiltration Analysis Results

Due to potential cross-connections between basins, the I/I results for Basins 7 and 8 are shown together, labelled "Basin 7/8". The individual results for Basin 7 and 8 are presented as sub-headers and for informational purposes only to be used at the reader's discretion.

Table 2 summarizes the flow monitoring and I/I results for the nine 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 9 represents the least. Basins that ranked 1, 2 or 3 in a category are color coded red. Please refer to the *I/I Methods* section for more information on inflow and infiltration analysis methods and ranking methods.

Table 2. I/I Analysis Summary

Metering Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Combined I/I (gallons)	Inflow Ranking	RDI Ranking	Evidence of High GWI?	Combined I/I Ranking
Basin 1	0.19	0.23	56,400	7	4	No	7
Basin 2	0.17	0.13	47,200	9	T7	No	8
Basin 3	0.18	0.46	166,200	1	3	No	1
Basin 4	0.21	0.41	114,100	2	5	No	4
Basin 5	0.27	0.18	90,900	6	2	No	5
Basin 6	0.35	0.57	136,300	5	T7	Yes	6
Basin 7/8	0.40	0.67	230,300	3	T7	No	2
Basin 7	*0.24*	*0.57*	*177,700*			*No*	
Basin 8	*0.16*	*0.21*	*52,000*			*No*	
Basin 9	0.13	0.34	133,300	4	1	No	3
Basin 10	0.12	0.11	16,400	8	6	No	9
System	***2.65***	***2.97***	***1,135,500***				

The following inflow/infiltration analysis results are noted:

- **Inflow:** Basins 3, 4, 7/8 and 9 ranked highest for normalized inflow contribution.
  - If isolated, Basin 7 ranks highly within the Basin 7/8 basin and highly within the collection system.
- **Rainfall-Dependent Infiltration:** Basins 3, 5 and 9 ranked highest for normalized RDI contribution.
- **Groundwater Infiltration:** Basin 6 had GWI rates that were **above** the WEF typical low-to-average ratio, indicating excessive groundwater infiltration.
- **Combined I/I:** Basins 3, 7/8, 9 and 4 ranked highest for normalized combined I/I contribution.
  - If isolated, Basin 7 ranks highly within the Basin 7/8 basin and highly within the collection system.

Figure 3 through Figure 6 show temperature maps of the overall rankings for each inflow and infiltration component.

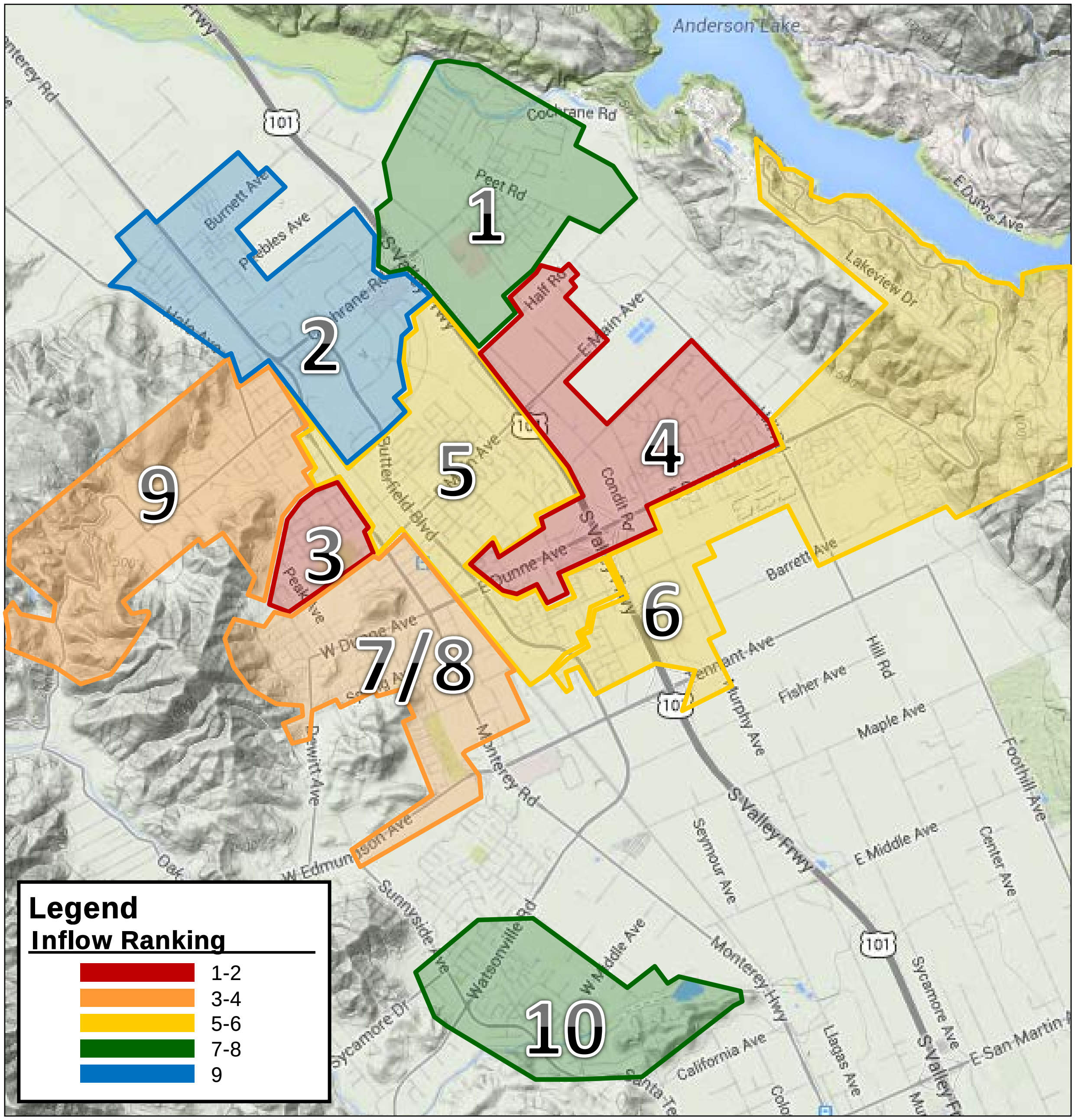

Figure 3. Inflow Temperature Map (by Rank)

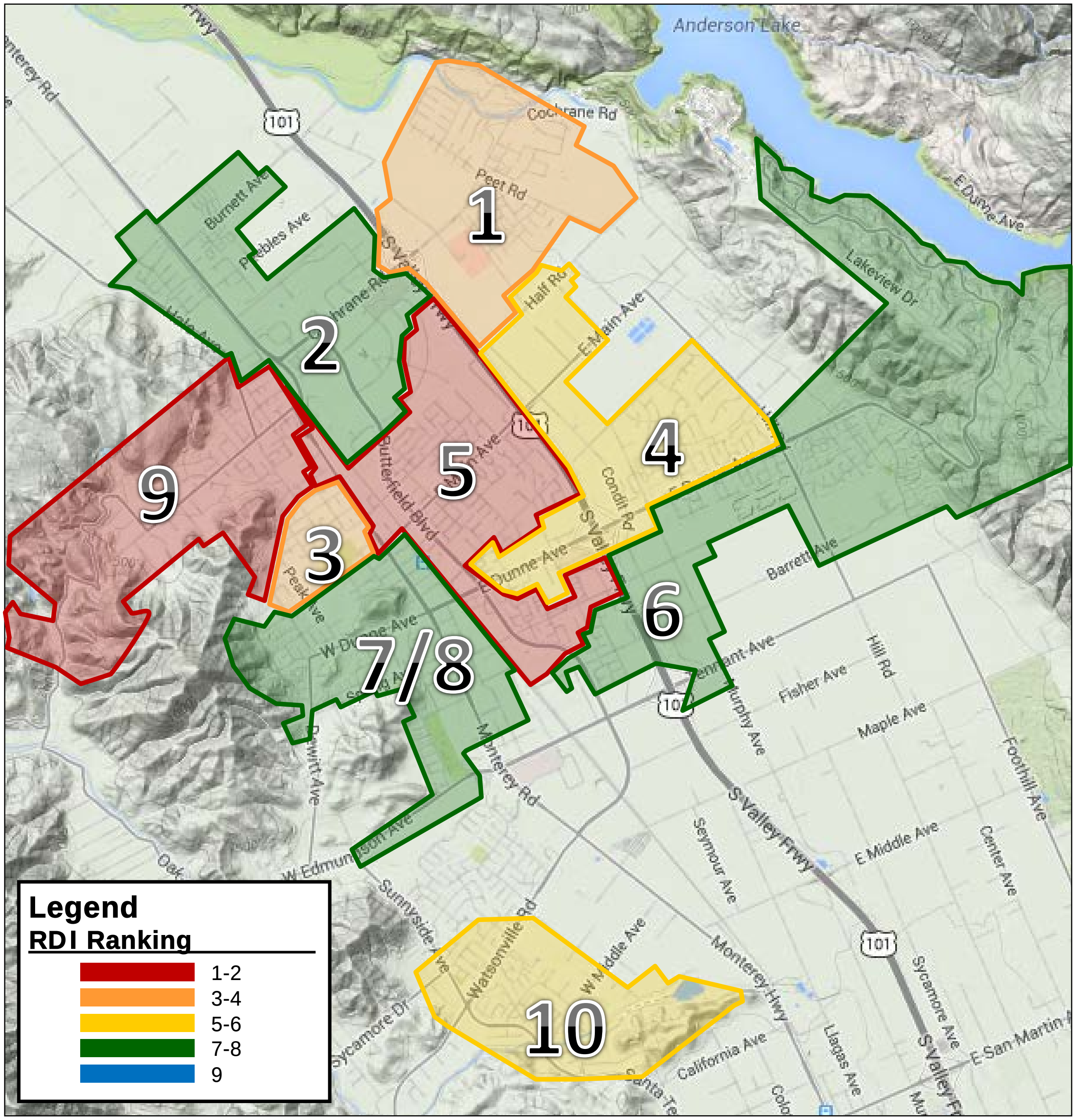

Figure 4. RDI Temperature Map (by Rank)

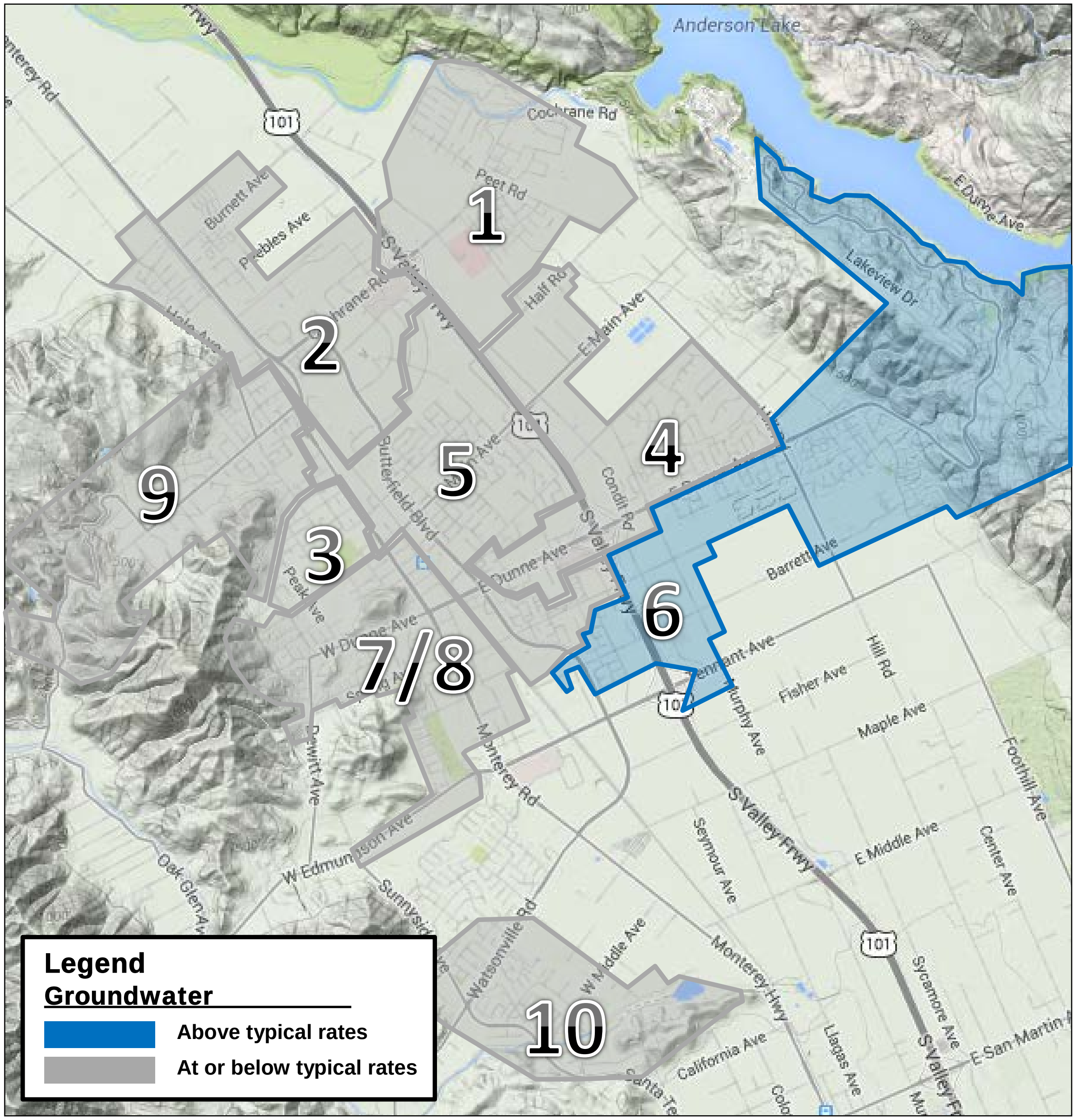

Figure 5. Basins with Groundwater Infiltration

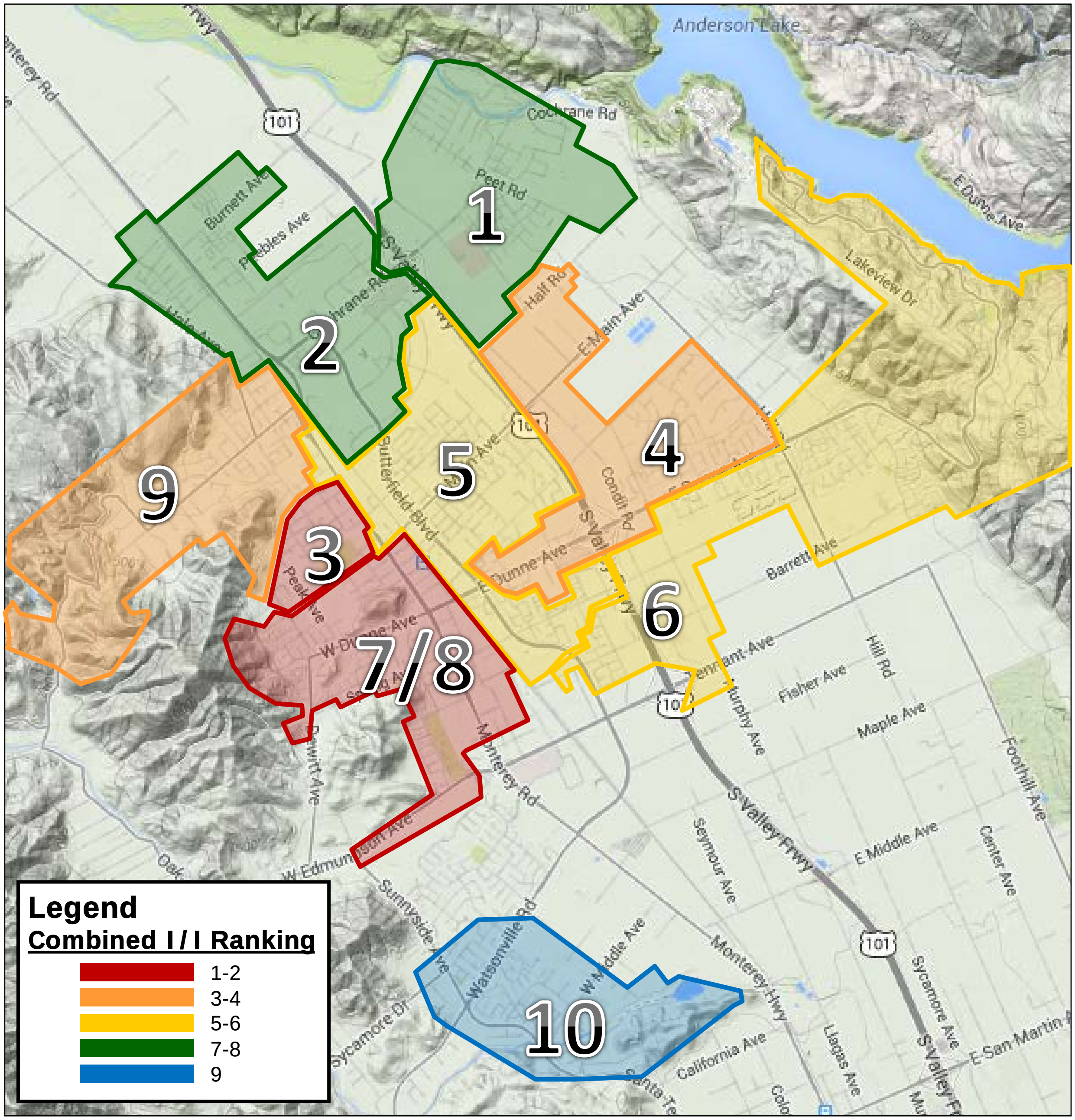

Figure 6. Combined I/I Temperature Map (by Rank)

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 occurred in Basins 3, 7/8 and 9.
    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 occurred in Basins 3, 5 and 9.
        ii. The highest groundwater infiltration occurred in Basin 6.
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.

INTRODUCTION

Scope and Purpose

V&A was retained by Akel Engineering Group to perform sanitary sewer flow monitoring and inflow and infiltration (I/I) analysis within the City of Morgan Hill, California (City). Flow monitoring was performed over a 20-day period at ten open-channel flow monitoring sites within the City. The flow monitoring period began on February 25, 2014, and ended on March 17, 2014. The purpose of this study was to measure sanitary sewer flows at the flow monitoring sites and estimate available sewer capacity and infiltration and inflow (I/I) occurring in the basins upstream from the flow monitoring sites.

Flow Monitoring Sites

Flow monitoring sites are the manholes 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. The locations and other information for the flow monitoring sites are shown in Table 3 and illustrated in Figure 7.

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 the pipelines[3]. A basin may refer to the entire collection system upstream from a flow meter or may exclude separately monitored basins upstream. I/I analysis in this report will be conducted on a basin-by-basin basis. The isolated basins of this project are illustrated in Figure 8. Due to potential cross-connections between basins, the I/I results for Basins 7 and 8 are shown together, labelled “Basin 7/8”. The individual results for Basin 7 and 8 will be presented in future analyses tables as sub-headers and for informational purposes only.

For this study subtraction of flows was required to isolate the drainage areas of some flow monitoring basins. Shown in Table 4 are the equations (in which Q refers to flow rate) used to calculate the flow rate results for each basin from the flow rates recorded at the monitoring sites. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in *Appendix A*.

Harding Flow Meter

V&A had access to flow monitoring data from the Harding Avenue Flow Meter. This meter measures the total flow from the City of Morgan Hill through a single 21-inch transmission sewer line sewer prior to entering into the City of Gilroy collection system. Flows from this meter during the flow monitoring period are also presented in this report and used to establish system totals.

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

[3] The basin areas (in acres) were provided by Akel Engineering Group.

Table 3. List of Flow Monitoring Sites

Metering Site	Pipe Diameter (in)	Location
Site 1	18	Behind Residence Inn, off Madrone Pkwy.
Site 2	19.75	Butterfield Blvd., south of Jarvis Dr.
Site 3	15	Intersection of Main Ave. and Monterey Rd.
Site 4	12	E. Dunne Ave, just east of Butterfield Blvd.
Site 5	24	Intersection of Barrett Ave. and Railroad Ave.
Site 6	17.75	Intersection of Barrett Ave. and Railroad Ave.
Site 7	21	Intersection of Edes St. and Monterey Rd.
Site 8	17.5	W. Edmundson Ave., just west of Monterey Rd.
Site 9	15	Hale Ave., north of Wright Ave.
Site 10	11.75	Easement west of Monterey Rd., north of California Ave.

Table 4. Flow Monitoring Basin Information

Flow Metering Basin	Metering Basin Size (acres)	Basin Flow Calculation
Basin 1	817	$Q_{1(Basin)} = Q_{1(Site)}$
Basin 2	910	$Q_{2(Basin)} = Q_{2(Site)} - Q_{1(Site)}$
Basin 3	345	$Q_{3(Basin)} = Q_{3(Site)} - Q_{9(Site)}$
Basin 4	823	$Q_{4(Basin)} = Q_{4(Site)}$
Basin 5	807	$Q_{5(Basin)} = Q_{5(Site)} - Q_{2(Site)} - Q_{3(Site)} - Q_{4(Site)}$
Basin 6	1,918	$Q_{6(Basin)} = Q_{6(Site)}$
Basin 7/8	765	$Q_{7/8(Basin)} = Q_{7(Site)} + Q_{8(Site)}$
Basin 7[4]	*284*	$Q_{7(Basin)} = Q_{7(Site)}$
Basin 8[4]	*481*	$Q_{8(Basin)} = Q_{8(Site)}$
Basin 9	1,094	$Q_{9(Basin)} = Q_{9(Site)}$
Basin 10	454	$Q_{10(Basin)} = Q_{10(Site)}$
City of Morgan Hill Collection System	9,500	$Q_{System} = Q_{Harding}$

[4] Basin 7 and 8 sizes in acres assume that cross-connections between the basins were not active during this flow monitoring study. Report analyses assume non-active cross-connections and analysis results are shown for informational purposes only and should be used per the discretion of the reader.

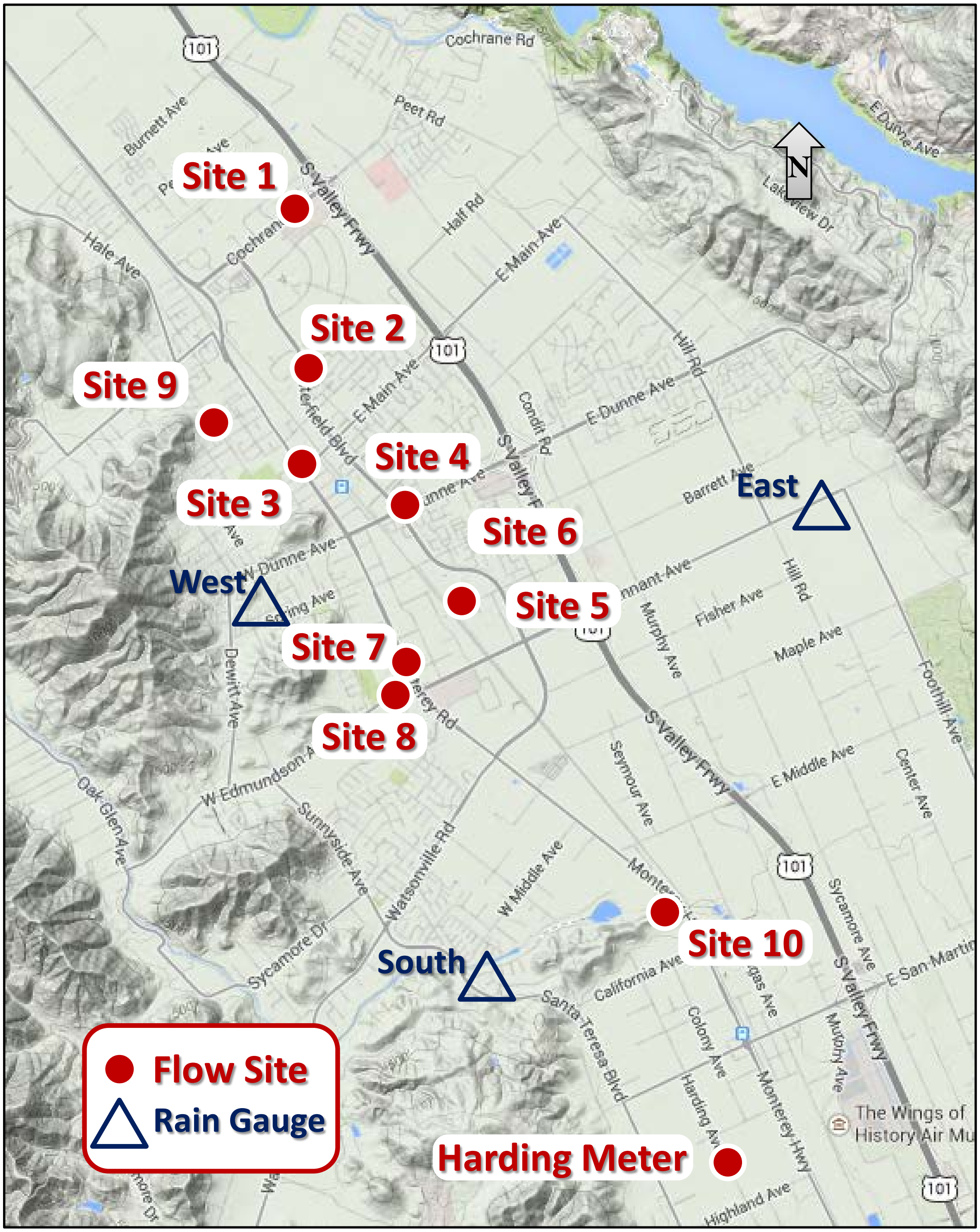


Figure 7. Site Location Map

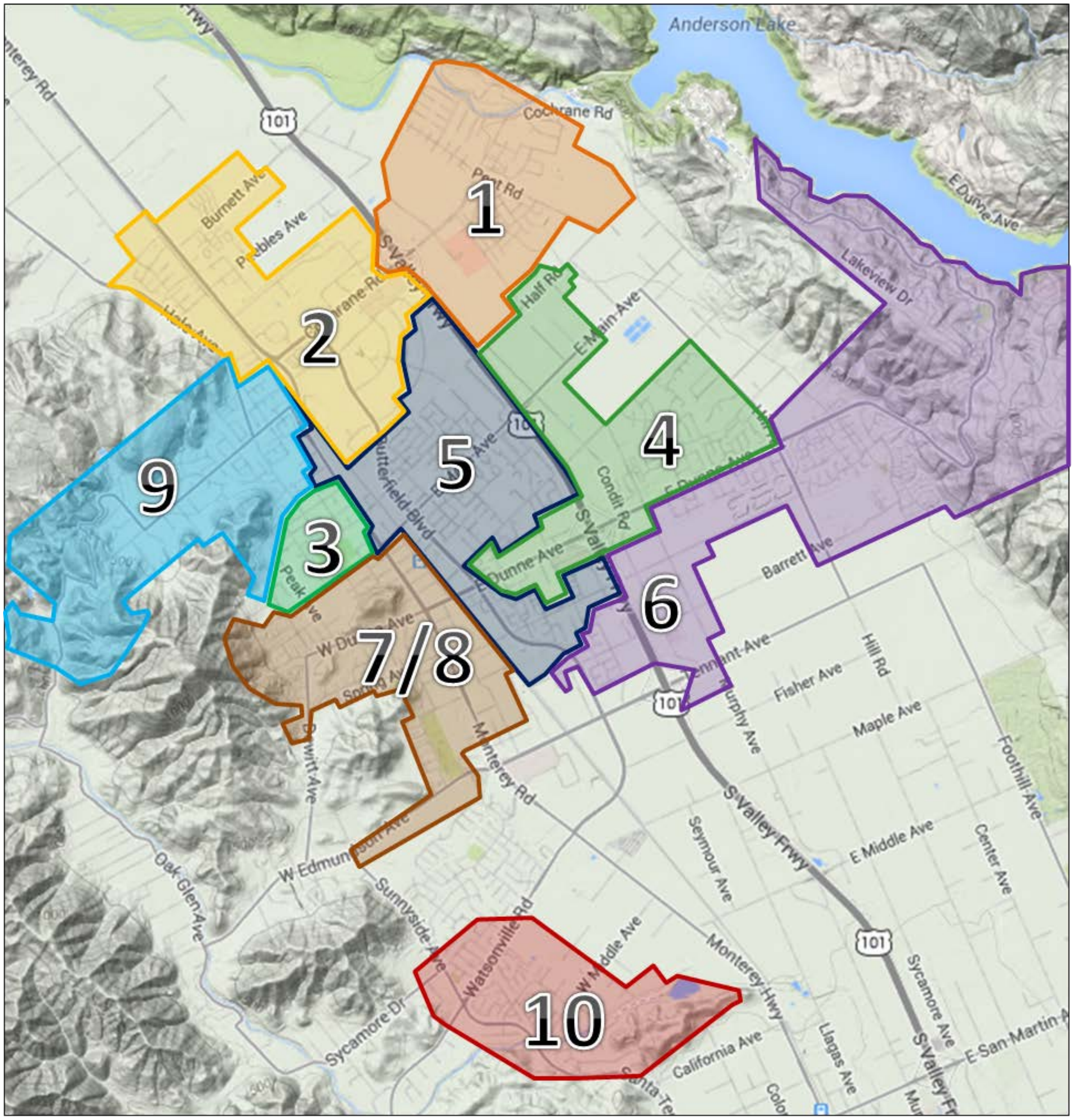


Figure 8. Basin Location Map

METHODS AND PROCEDURES

Confined Space Entry

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 on another four-gas monitor and maintaining records of all Entrants, if there are more than one. The Supervisor develops 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 ten Isco 2150 area-velocity flow meters at the metering locations referenced 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 (500 kHz) 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 9 shows a typical installation for a flow meter with a submerged sensor.

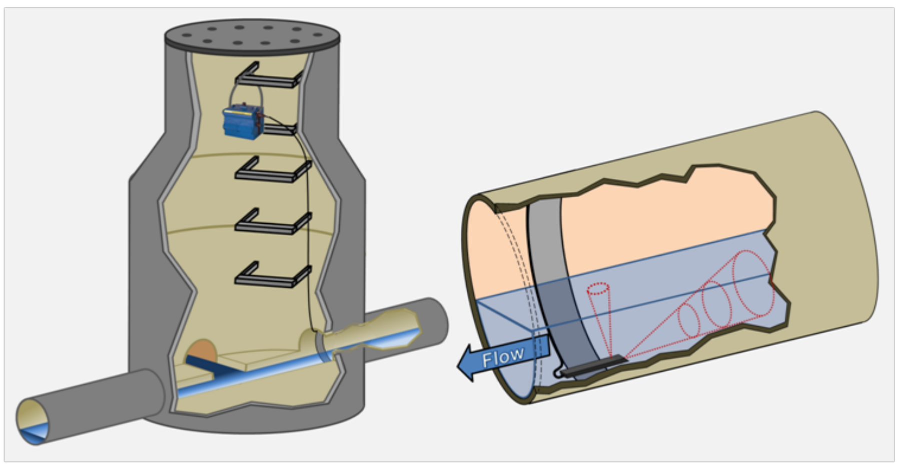


Figure 9. Typical Installation for Flow Meter with Submerged Sensor

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.

RESULTS AND ANALYSIS

Rainfall: Rain Gauge Data

V&A utilized rain data from rain gauges maintained by local weather enthusiasts. While V&A performed QA/QC analysis to ensure, to the extent possible, the quality of the rainfall data from the three rain gauges used, it is noted that V&A has no direct control over these gauges.

There were five rain events stacked over seven days from February 26 to March 5, 2014. The largest individual rain event occurred on February 28, 2014 and this event was used for infiltration and inflow analysis for this study. Table 5 summarizes the rainfall data collected for this study. Figure 10 graphically displays the rainfall activity recorded over the flow monitoring period (average of the three rain gauges shown).

Table 5. Rainfall Events Used for I/I Analysis

Rainfall Event	RG WEST Event Rainfall (in)	RG EAST Event Rainfall (in)	RG SOUTH Event Rainfall (in)
Event 1: February 28 – March 1, 2014	1.74	2.16	2.26
Total over Monitoring Period	*3.16*	*3.34*	*4.06*

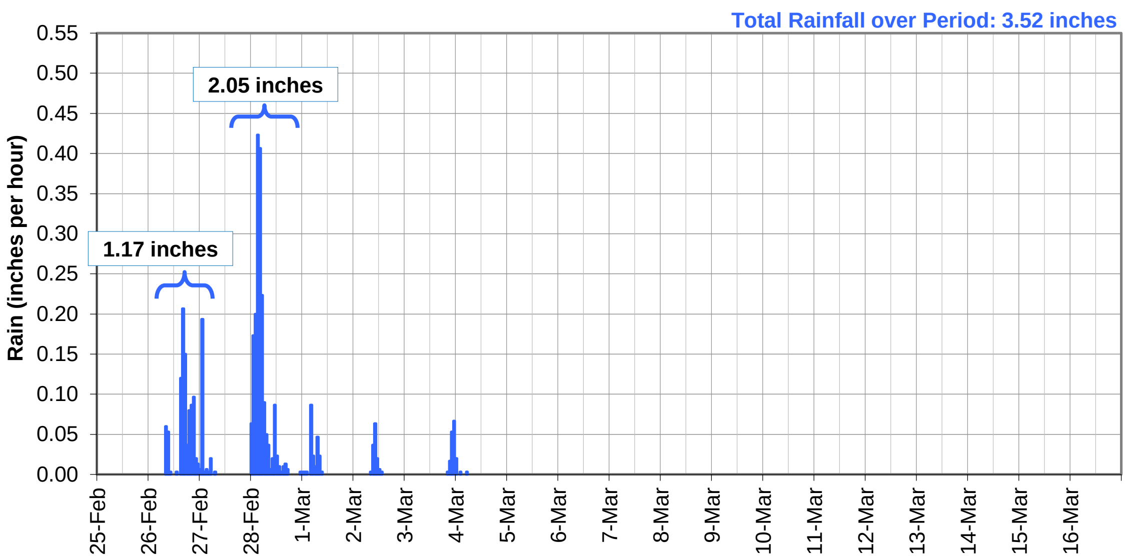


Figure 10. Rainfall Activity over Flow Monitoring Period (Avg. of Rain Gauges)

Figure 11 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[5] in Morgan Hill during this project duration. Rainfall totals for Morgan Hill ranged between 158% and 203% of historical normal levels during this 20-day time period.

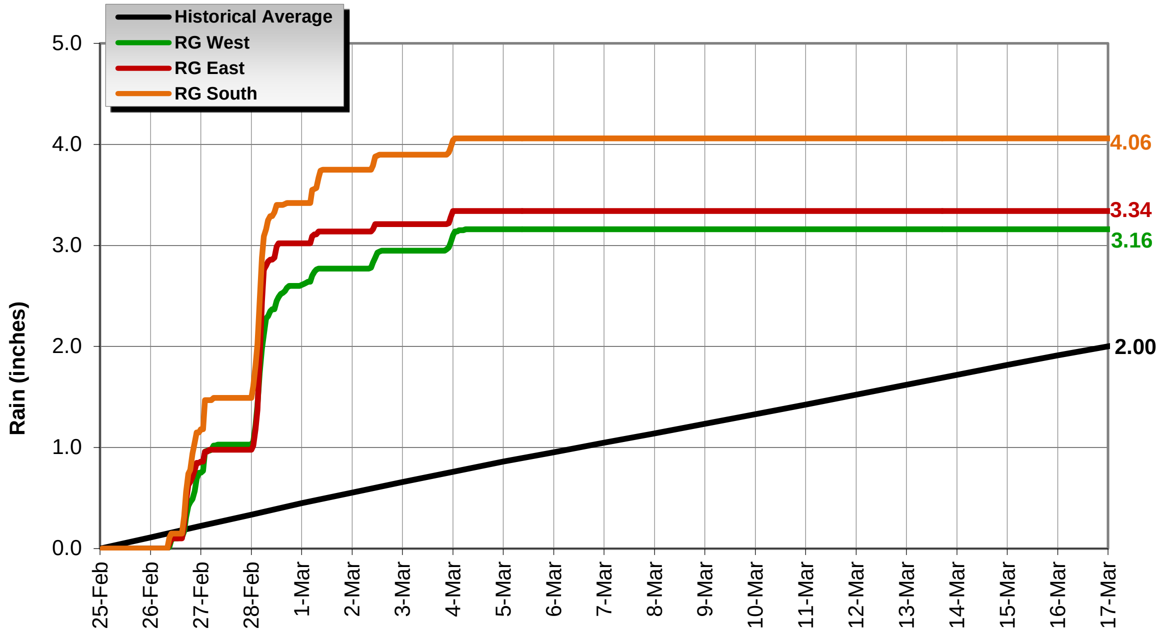


Figure 11. Rainfall Accumulation Plot

Rain Gauge Triangulation Distribution

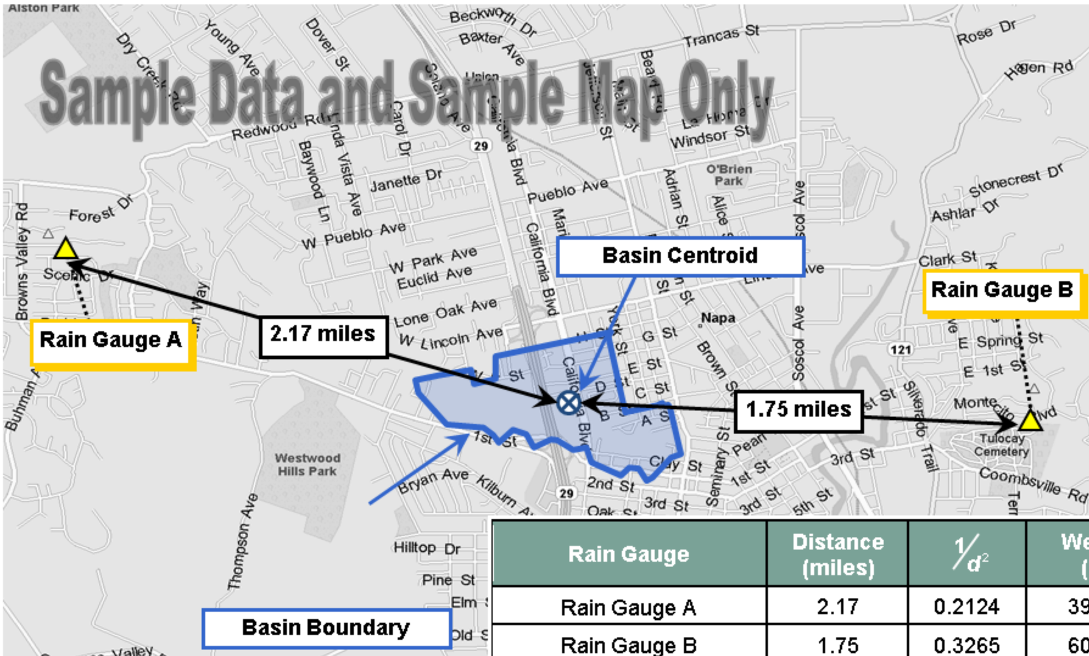
The rainfall affecting the sanitary sewer collection system basins must be calculated based on the proximity to the rain gauge locations. The mean precipitation for each site was calculated by taking data from the rain gauges and using the inverse distance weighting (IDW) method. IDW is an interpolation method that assumes the influence of each rain gauge location diminishes with distance. The approximate geographic coordinates of each site were determined and a weighted average was taken of the precipitation data from nearby rain gauge locations.

IDW is performed using the equation

$$w = \frac{1/d^p}{\sum 1/d^p}$$

where the weight, *w*, depends on the distance, *d*, from the rain gauge to the monitoring site and *p*, a user-selected power ($p > 0$). The most common choice of *p* in hydrological studies of watershed areas is 2. Figure 12 illustrates the IDW method with sample data.

[5] Historical data taken from the WRCC (Station 043417 in Morgan Hill): http://www.wrcc.dri.edu/summary/climsmnca.html



Rain Gauge	Distance (miles)	$1/d^2$	Weight (%)
Rain Gauge A	2.17	0.2124	39.4%
Rain Gauge B	1.75	0.3265	60.6%
Totals		**0.5389**	**100%**

Figure 12. Rainfall Inverse Distance Weighting Method

The rain gauge distribution as calculated for each metering site for this project is shown in Table 6.

Table 6. Rain Gauge Distribution by Basin

Metering Basin	RG West (%)	RG East (%)	RG South (%)
Basin 1	54.3%	45.7%	0.0%
Basin 2	65.2%	34.8%	0.0%
Basin 3	92.2%	7.8%	0.0%
Basin 4	48.2%	51.8%	0.0%
Basin 5	79.0%	21.0%	0.0%
Basin 6	7.4%	92.6%	0.0%
Basin 7/8	97.6%	2.4%	0.0%
Basin 9	89.4%	10.6%	0.0%
Basin 10	3.2%	0.0%	96.8%
System	53.5%	31.4%	15.1%

Rainfall: Storm Event Classification

It is important to classify the relative size of the 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 Morgan Hill (at the coordinates of the RG East rain gauge) as 4.23 inches (Figure 13). This means that in any given year, there is a 10% chance that 4.23 inches of rain will fall in any 24-hour period.

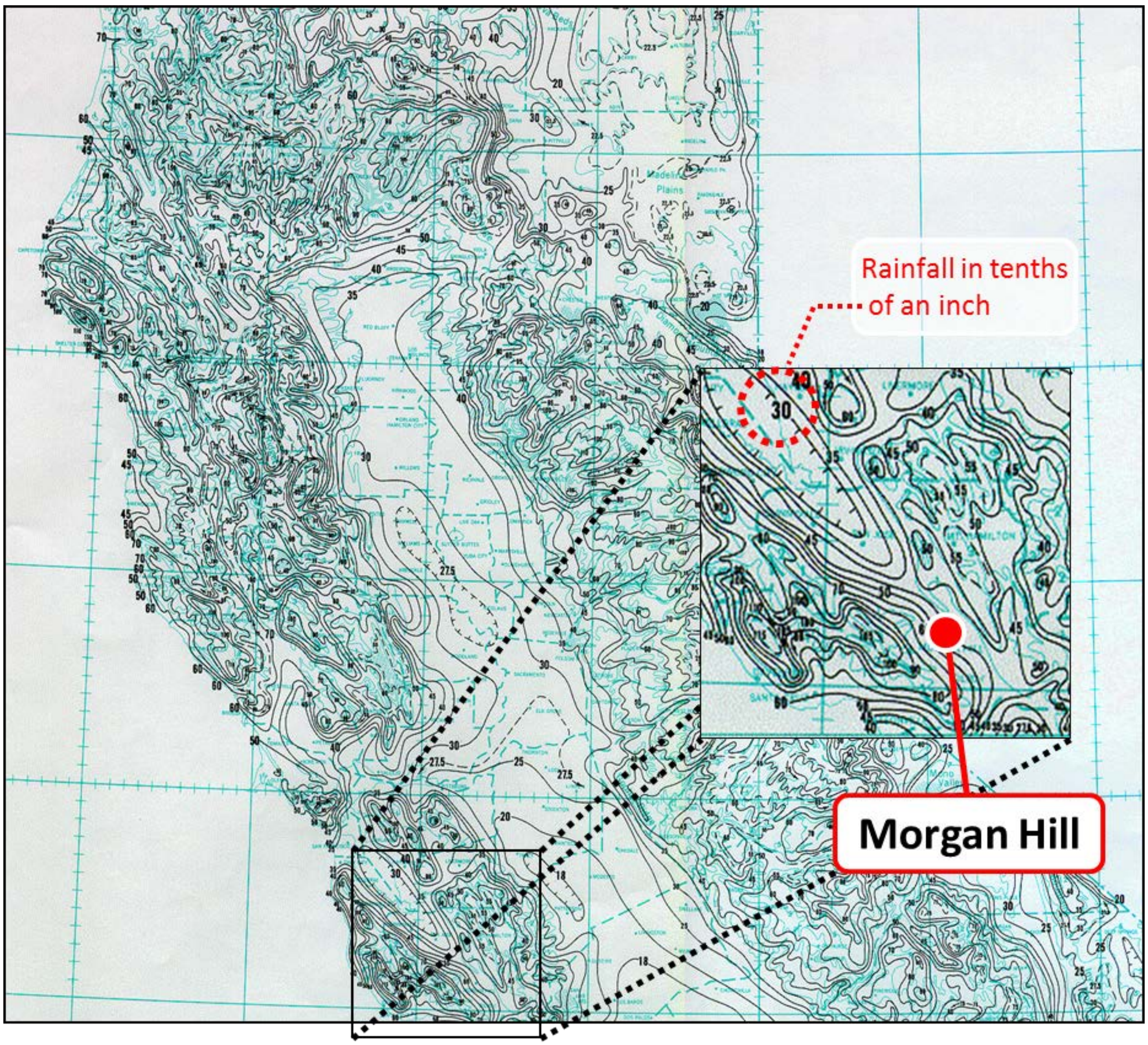


Figure 13. NOAA Northern California Rainfall Frequency Map

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

[7] NOAA Western U.S. Precipitation Frequency Maps Atlas 2, 1973: http://www.wrcc.dri.edu/pcpnfreq.html

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 15 minutes to 60 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 Event 1 on the rain event frequency plot determines the classification of the storm event, as shown in Figure 14 for RG East.

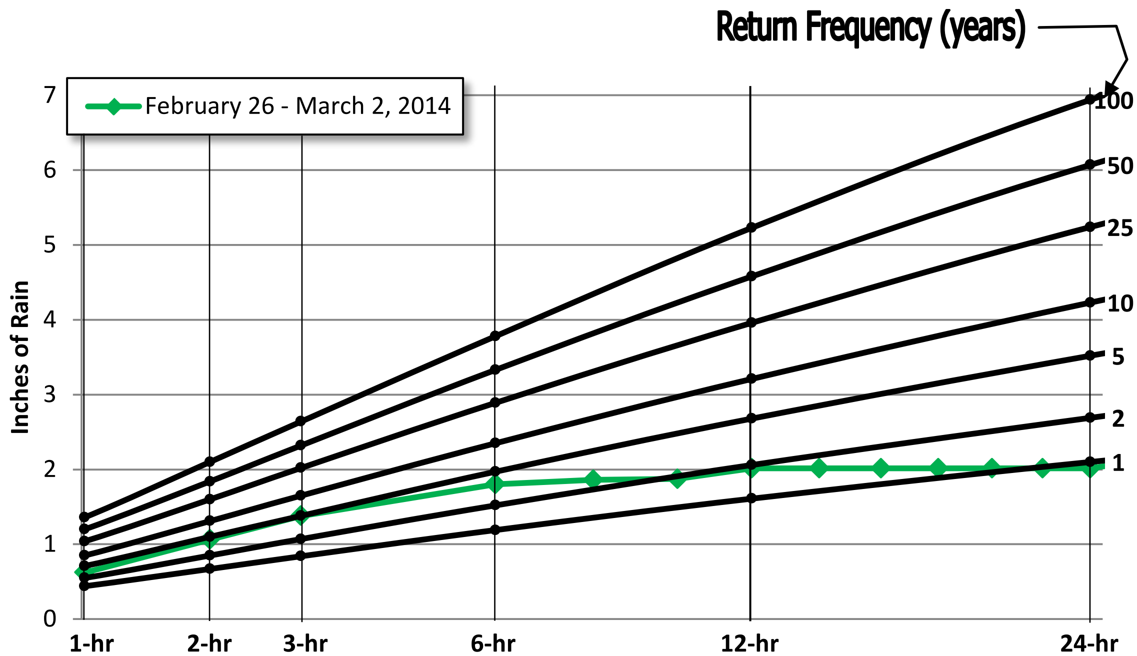


Figure 14. Storm Event Classification (RG East)

Table 7 summarizes the classification of the rainfall events that occurred during the flow monitoring period.

Table 7. Classification of Rainfall Events

Rain Gauge	Event 1: February 28-March 1, 2013		
	Duration: 3 Hour	Duration: 6 Hours	Duration: 12 Hours
RG West	<1 Year	1 Year	<1 Year
RG East	5 Year	4 Year	2 Year
RG South	2 Year	2 Year	1.5 Year

Flow Monitoring: Average Dry Weather Flows

Weekday and weekend diurnal flow patterns differ and can be separated when establishing average dry weather flow rates. Within weekdays, the average dry weather flow (ADWF) patterns for Friday will vary from the Monday through Thursday patterns, particularly in the evening hours as people prepare for the weekend. Similarly, Sunday flow patterns typically vary in the evenings from Saturday flow patterns as people prepare for the work week. Figure 15 illustrates the varying flow patterns within a work week (Site 7 shown). Graphs of the ADWF flow patterns for each site may be found in *Appendix A*.

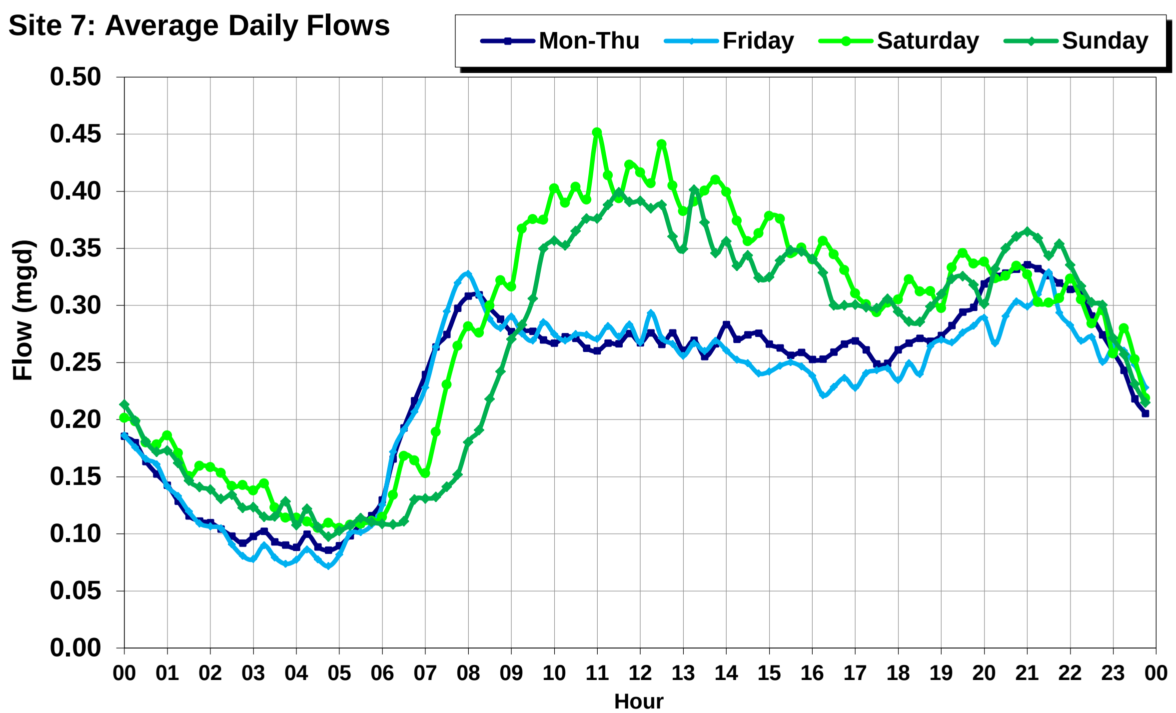


Figure 15. Sample ADWF Diurnal Flow Patterns (Site 7)

The overall average dry weather flow (ADWF) is calculated per the following equation:

$$ADWF = \left(ADWF_{Mon-Thu} \times \frac{4}{7}\right) + \left(ADWF_{Fri} \times \frac{1}{7}\right) + \left(ADWF_{Sat} \times \frac{1}{7}\right) + \left(ADWF_{Sun} \times \frac{1}{7}\right),$$

Table 8 lists the average dry weather flow (ADWF) recorded during this study for the flow monitoring sites. Figure 16 shows a schematic diagram of the average dry weather flows and flow levels.

Table 8. Dry Weather Flow Summary

Monitoring Site	Mon-Thu ADWF (mgd)	Friday ADWF (mgd)	Saturday ADWF (mgd)	Sunday ADWF (mgd)	Overall ADWF (mgd)
Site 1	0.19	0.20	0.19	0.21	0.19
Site 2	0.36	0.36	0.34	0.34	0.36
Site 3	0.31	0.30	0.33	0.34	0.32
Site 4	0.21	0.20	0.23	0.23	0.21
Site 5	1.15	1.12	1.18	1.19	1.15
Site 6	0.35	0.34	0.36	0.37	0.35
Site 7	0.23	0.22	0.28	0.26	0.24
Site 8	0.15	0.15	0.18	0.17	0.16
Site 9	0.13	0.13	0.14	0.15	0.13
Site 10	0.12	0.12	0.13	0.14	0.12
Harding	2.62	2.62	2.70	2.73	2.65

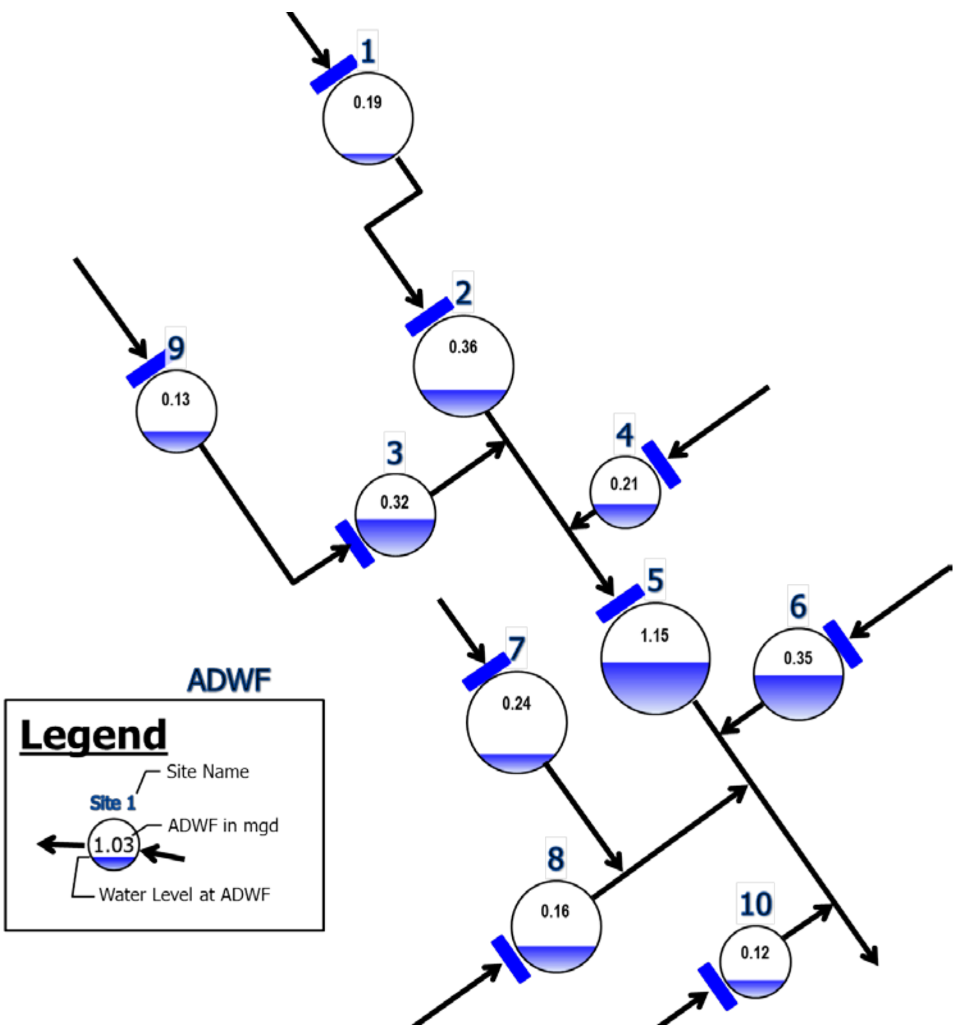


Figure 16. Average Dry Weather Flow (Flow Schematic)

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.

Two key 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 from the flow monitoring study.

Table 9 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 site locations; hydraulic conditions in other areas of the collection system will differ.

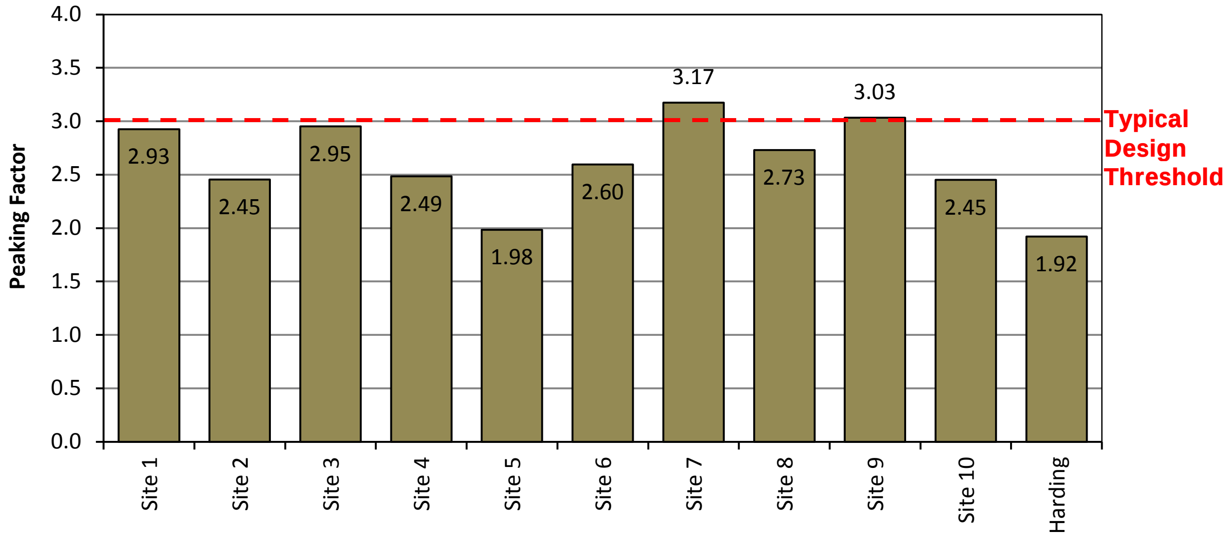
Table 9. Capacity Analysis Summary

Metering Site	ADWF (mgd)	Peak Measured Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	Peak *d/D* Ratio	Level Surcharged above Crown (ft)
Site 1	0.19	0.56	2.93	18	3.73	0.21	-
Site 2	0.36	0.88	2.45	19.75	7.94	0.40	-
Site 3	0.32	0.93	2.95	15	13.13	0.88	-
Site 4	0.21	0.53	2.49	12	6.04	0.50	-
Site 5	1.15	2.29	1.98	24	15.95	0.66	-
Site 6	0.35	0.91	2.60	17.75	12.52	0.71	-
Site 7	0.24	0.77	3.17	21	6.82	0.32	-
Site 8	0.16	0.43	2.73	17.5	6.76	0.39	-
Site 9	0.13	0.41	3.03	15	6.36	0.42	-
Site 10	0.12	0.31	2.45	11.75	4.57	0.39	-

The following capacity analysis results are noted:

- **Peaking Factor:** Sites 7 and 9 had peaking factors that slightly exceeded typical design threshold limits for the ratio of peak flow to average dry weather flow.
- ***d/D* Ratio:** Only Site 3 had a *d/D* ratio that exceeded the common design threshold for *d/D* ratio. None of the flow monitoring sites reached surcharge conditions.

Figure 17 shows bar graphs of the capacity results. Figure 18 shows a schematic diagram of the peak measured flows with peak flow levels.



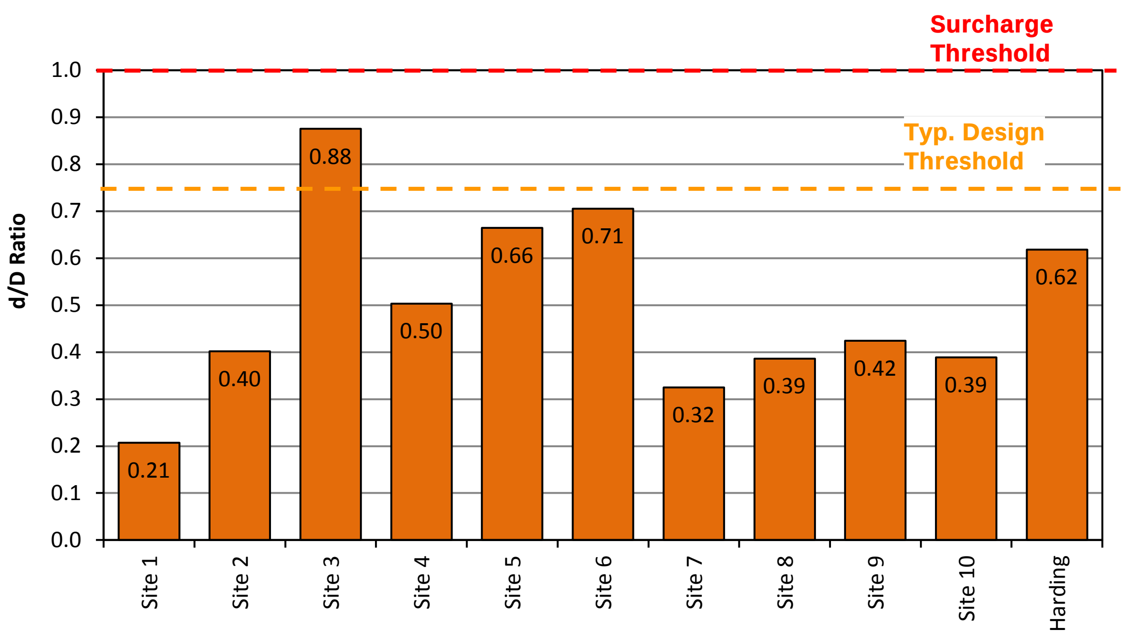


Figure 17. Capacity Summary Bar Graphs: Peaking Factors and Peak *d/D* Ratios

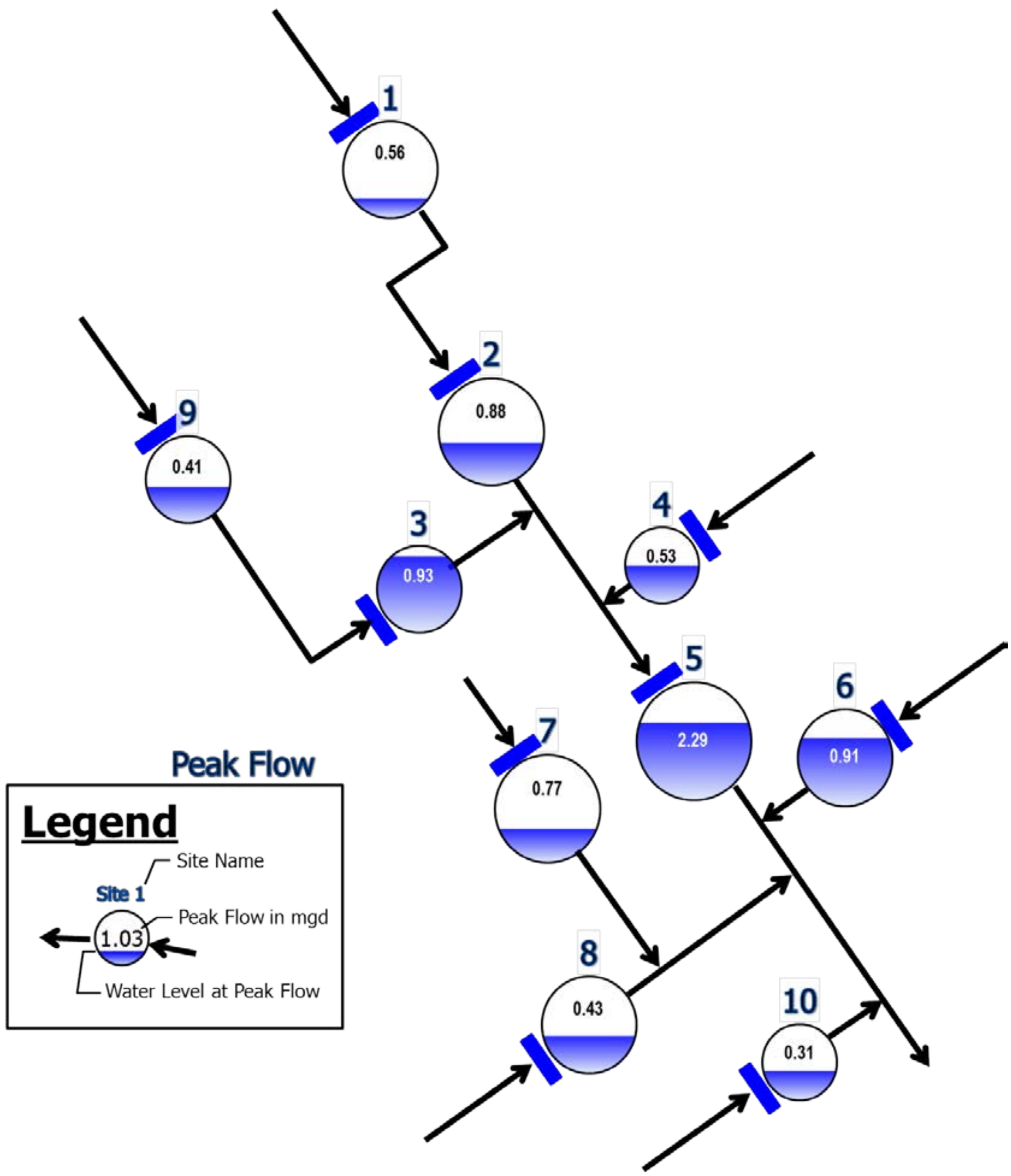


Figure 18. Peak Measured Flow (Flow Schematic)

Inflow / Infiltration Analysis: Definitions and Identification

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:

Inflow

- **Definition:** 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.
- **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.
- **Graphical Identification:** Inflow is usually recognized graphically by large-magnitude, short-duration spikes in flow immediately following a rain event.

Infiltration

- **Definition:** 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.
- **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:** 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.

Figure 19 shows sample graphs indicating the typical graphical response patterns for inflow and infiltration.

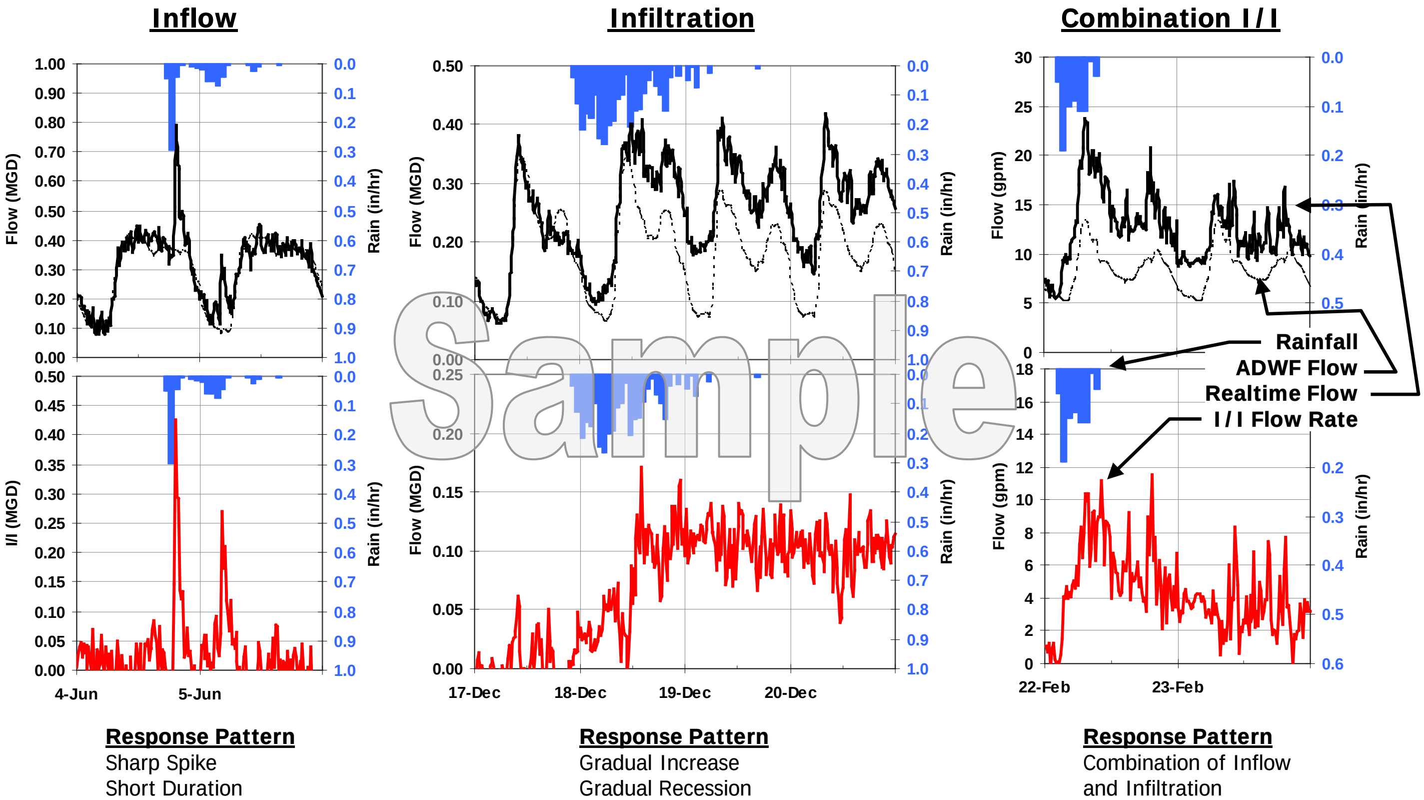


Figure 19. Inflow and Infiltration: Graphical Response Patterns

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.

Figure 20 illustrates the possible sources and components of I/I.

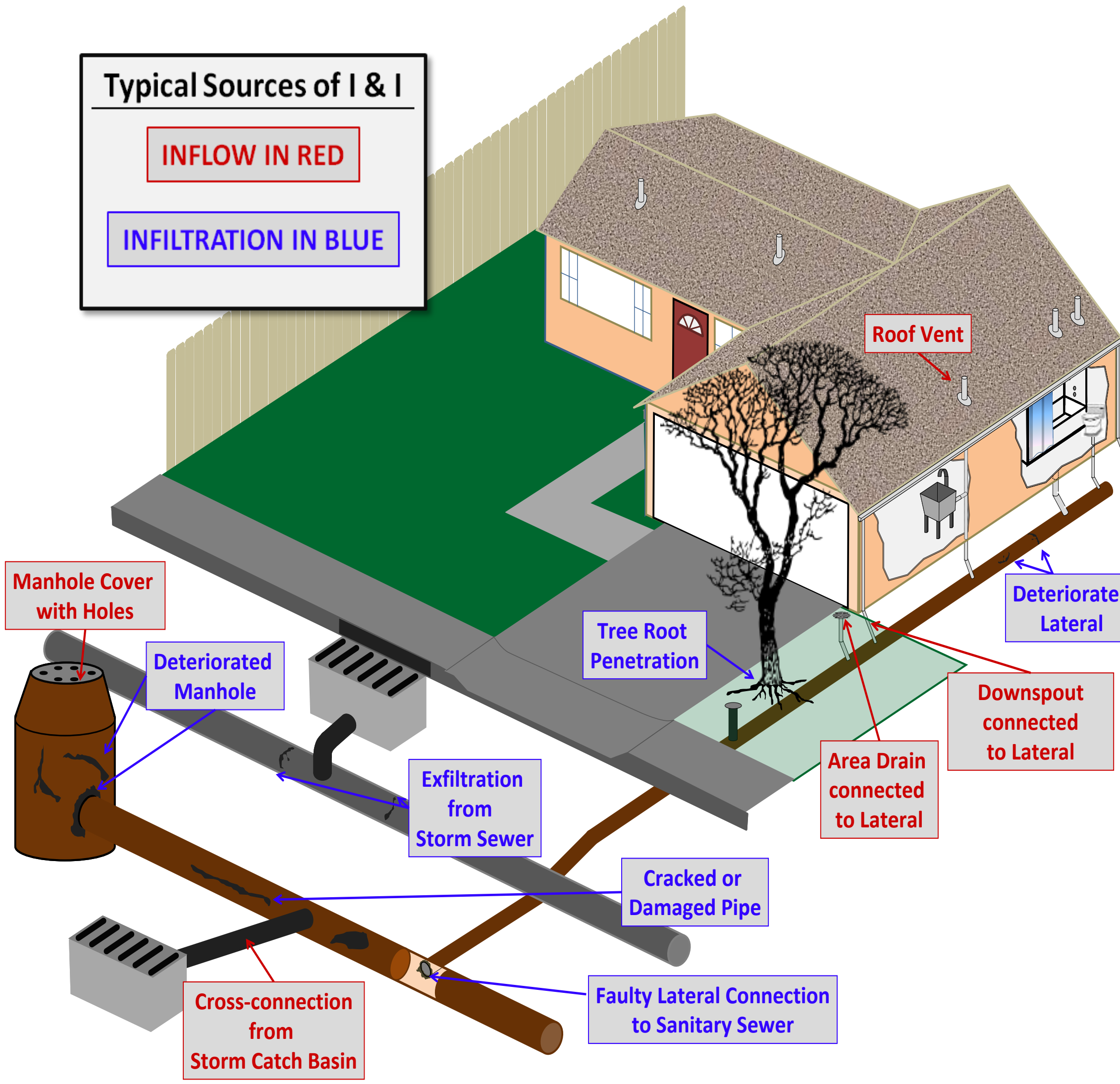


Figure 20. Typical Sources of Infiltration and Inflow

Inflow / Infiltration: Analysis Methods

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

Inflow Indicators

Peak I/I Flow Rate: Inflow is characterized by sharp, direct spikes occurring during a rainfall event. Peak I/I rates are used for inflow analysis[8]. 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.
- **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

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.

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 particular collection 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.

Combined I/I Indicators

Total Infiltration: 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 of rain.

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[8] 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.

- **R-Value:** Total infiltration (gallons) divided by the total rainfall that fell within the acreage of that basin (gallons of rainfall). This is expressed as a percentage and is explained as "the percentage of rain that enters the sanitary sewer collection system." Systems with R-values less than 5%[9] 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 of rain.

Instantaneous flows were plotted against ADWF flows to analyze the I/I response to rainfall events. Figure 21 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*.

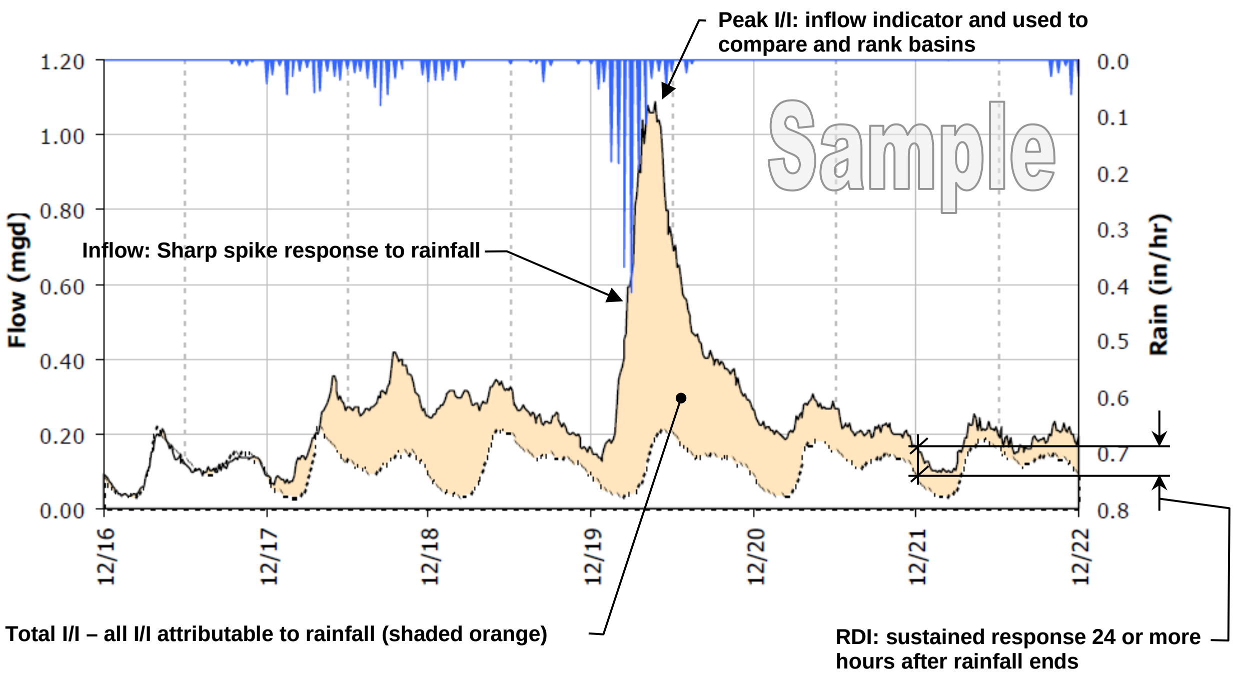


Figure 21. Sample Infiltration and Inflow Isolation Graph

The infiltration and inflow indicators were normalized by basin area and by ADWF in this report. Final rankings were determined by weighting the normalization methods by 50% for ADWF, and 50% for basin area, with ties broken by ADWF. The per-ADWF method is given the tie-break because it is normalized by actual sanitary waste usage. The per-acre method was not given the tie-breaker because the catchment area per each flow monitoring basin is estimated but requires a thorough hydrologic study to determine the true watershed.

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

Inflow / Infiltration: Results

Inflow Results Summary

Table 10 summarizes the peak measured I/I flows and inflow analysis results for Storm Event 1, which elicited the highest peak I/I response (refer to the *I/I Methods* section for more information on inflow analysis methods and ranking procedures). Basins that ranked 1, 2 or 3 in a category are color coded red.

Table 10. Basin Inflow Analysis Summary

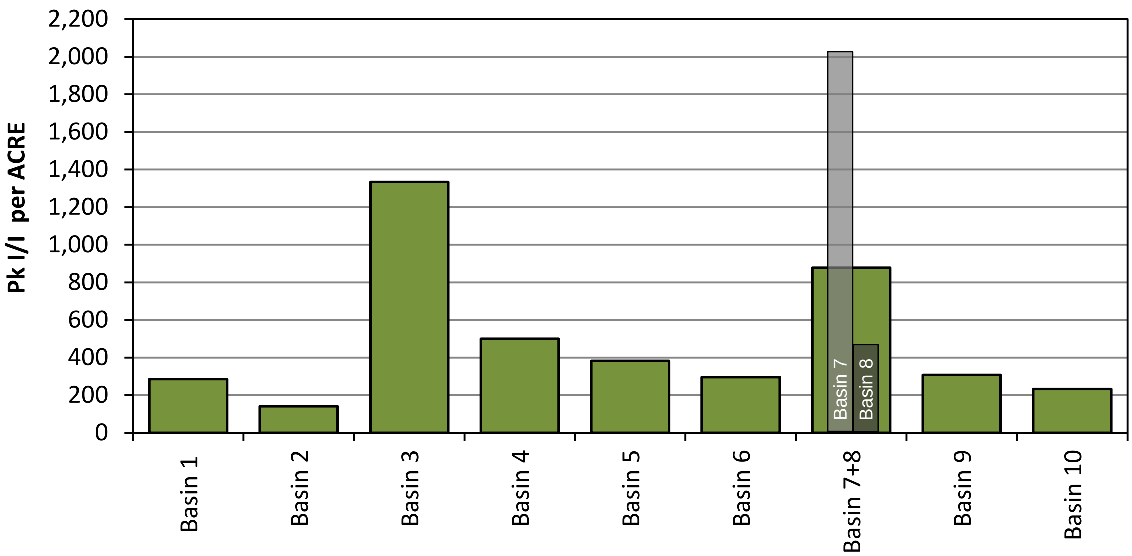
Metering Basin	ADWF (mgd)	Peak I/I Rate (mgd)	Peak I/I per Acre (gpd/acre)	Peak I/I per ADWF	Overall Inflow Ranking[A]
Basin 1	0.19	0.23	286 (7)[B]	1.22 (6)	7
Basin 2	0.17	0.13	140 (9)	0.77 (9)	9
Basin 3	0.18	0.46	1,334 (1)	2.55 (1)	1
Basin 4	0.21	0.41	500 (3)	1.92 (3)	2
Basin 5	0.27	0.18	219 (4)	0.66 (7)	6
Basin 6	0.35	0.57	295 (6)	1.61 (5)	5
Basin 7/8	0.40	0.67	878 (2)	1.68 (4)	3
Basin 7	*0.24*	*0.57*	*2,010*	*2.36*	
Basin 8	*0.16*	*0.21*	*434*	*1.31*	
Basin 9	0.13	0.34	308 (5)	2.50 (2)	4
Basin 10	0.12	0.11	233 (8)	0.85 (8)	8
System	***2.65***	***2.97***	***312***	***1.12***	

[A] Ranking of 1 represents most inflow after normalization.
[B] The number in parenthesis shows the ranking within the individual Category.

The following inflow analysis results are noted:

- Basins 3, 4, 7/8 and 9 ranked highest for normalized inflow contribution.
  - If isolated, Basin 7 ranks highly within the Basin 7/8 basin and highly within the collection system.

Figure 22 shows bar graph summaries of the inflow analysis. Figure 23 shows a temperature map summary of the inflow analysis results per basin.



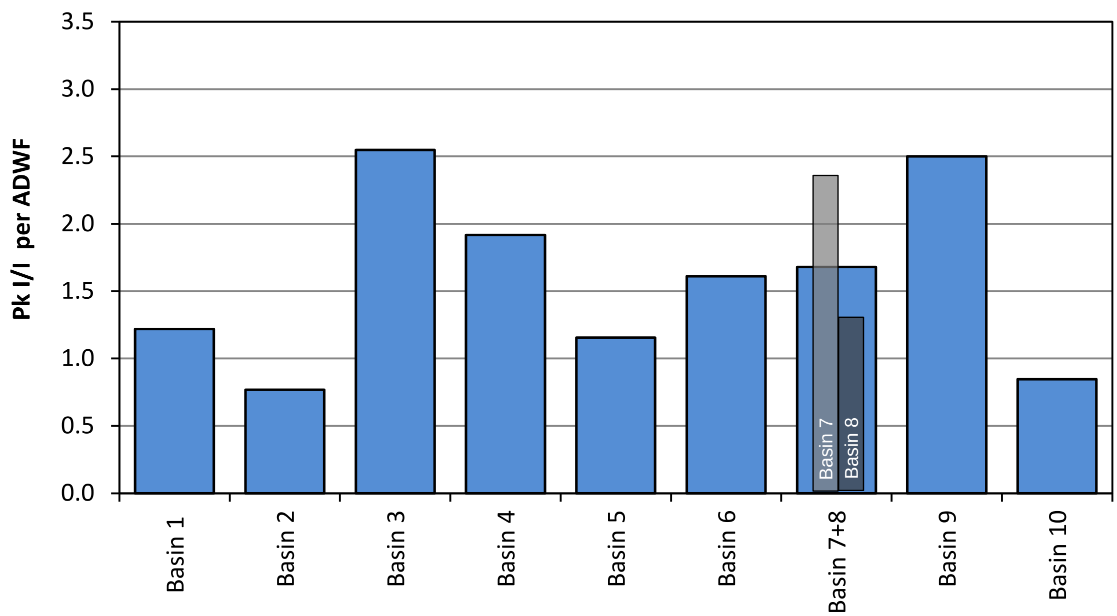


Figure 22. Bar Graphs: Inflow Analysis Summary

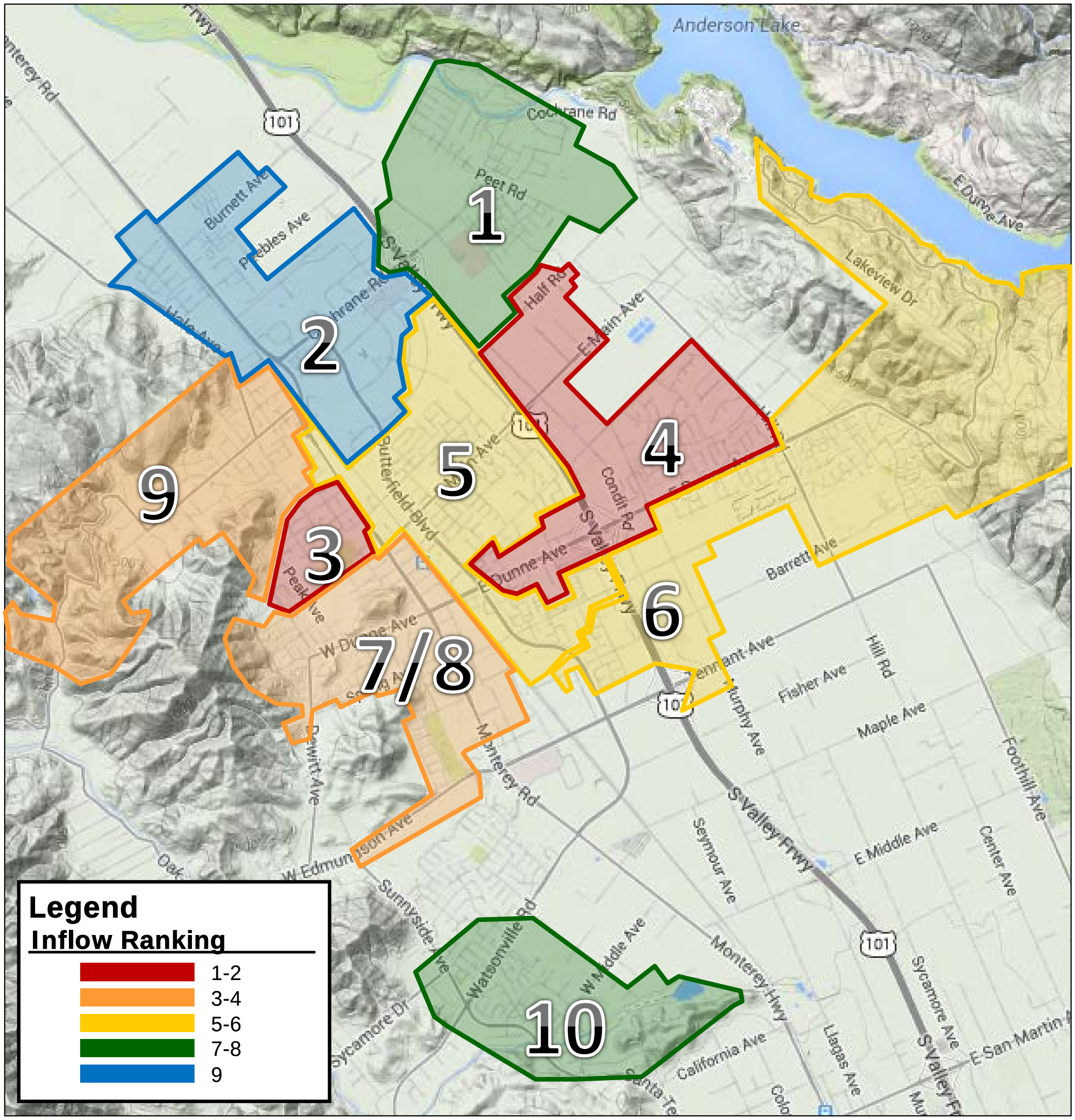


Figure 23. Inflow Temperature Map (by Rank)

Rainfall-Dependent Infiltration Results Summary

Table 11 summarizes the calculated average RDI flow rate during the low-flow hours immediately following the rainfall event (refer to the *I/I Methods* section for more information on RDI analysis methods and ranking methods). Basins that ranked 1, 2 or 3 in a category are color coded red.

Table 11. Basin RDI Analysis Summary

Metering Basin	ADWF (mgd)	RDI Rate (mgd)	RDI per Acre (GPAD)	RDI per ADWF	Overall RDI Ranking[A]
Basin 1	0.19	0.023	28 (4)[B]	12% (4)	4
Basin 2	0.17	0	0 (T7)	0% (T7)	T7
Basin 3	0.18	0.029	85 (1)	16% (3)	3
Basin 4	0.21	0.014	17 (5)	6% (5)	5
Basin 5	0.27	0.051	63 (2)	19% (2)	2
Basin 6	0.35	0	0 (T7)	0% (T7)	T7
Basin 7/8	0.40	0	0 (T7)	0% (T7)	T7
Basin 7	*0.24*	*0*	*0*	*0%*	
Basin 8	*0.16*	*0*	*0*	*0%*	
Basin 9	0.13	0.040	37 (3)	30% (1)	1
Basin 10	0.12	0.002	4 (6)	1% (6)	6
System	***2.65***	***0.210***	***22***	***8%***	

[A] Ranking of 1 represents most RDI after normalization.
[B] The number in parenthesis shows the ranking within the individual Category.

The following RDI analysis results are noted:

- Basins 3, 5 and 9 ranked highest for normalized RDI contribution.

Figure 24 shows bar graph summaries of the RDI analysis. A temperature map by overall ranking is shown in Figure 25.

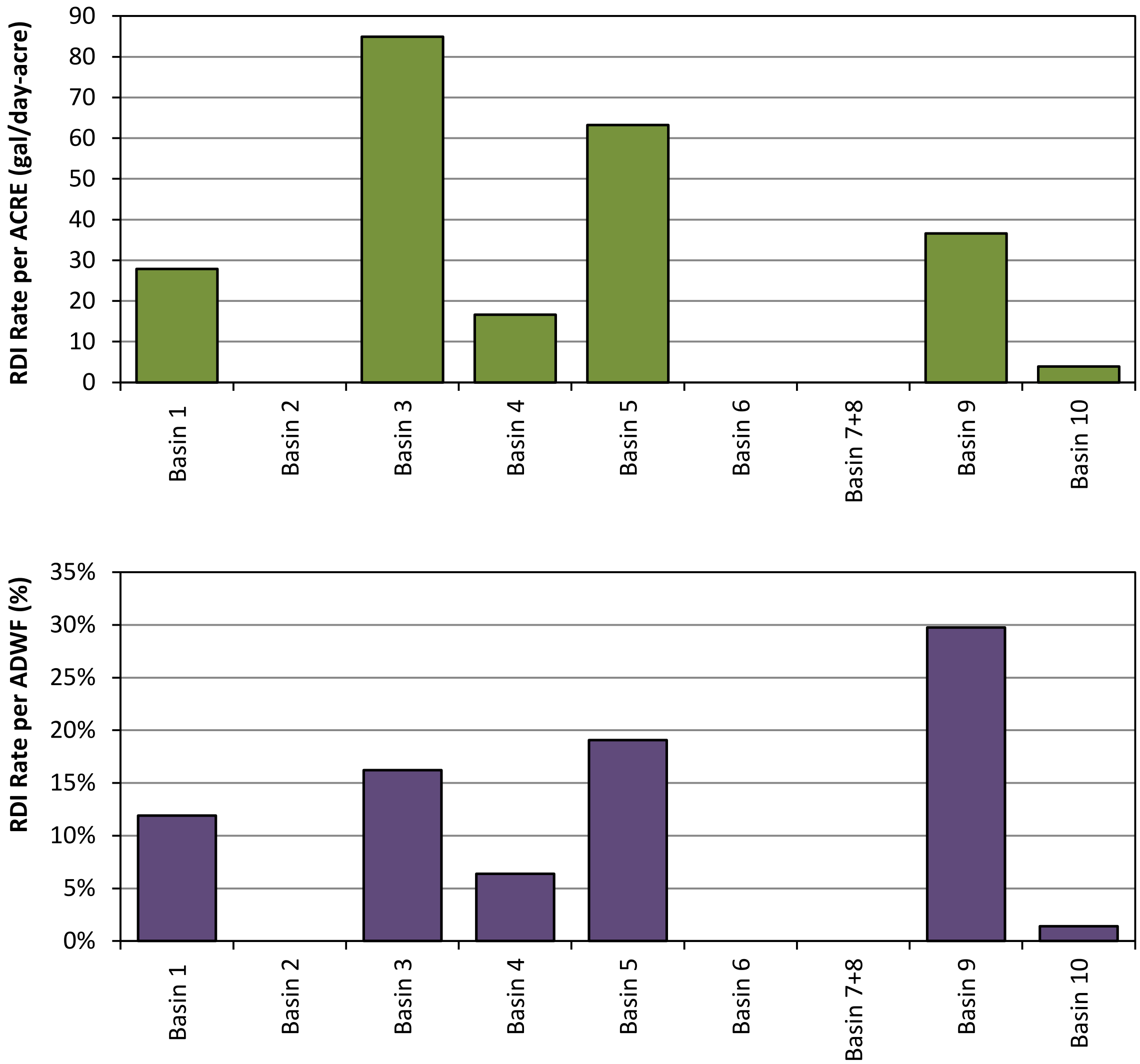


Figure 24. Bar Graphs: RDI Analysis Summary

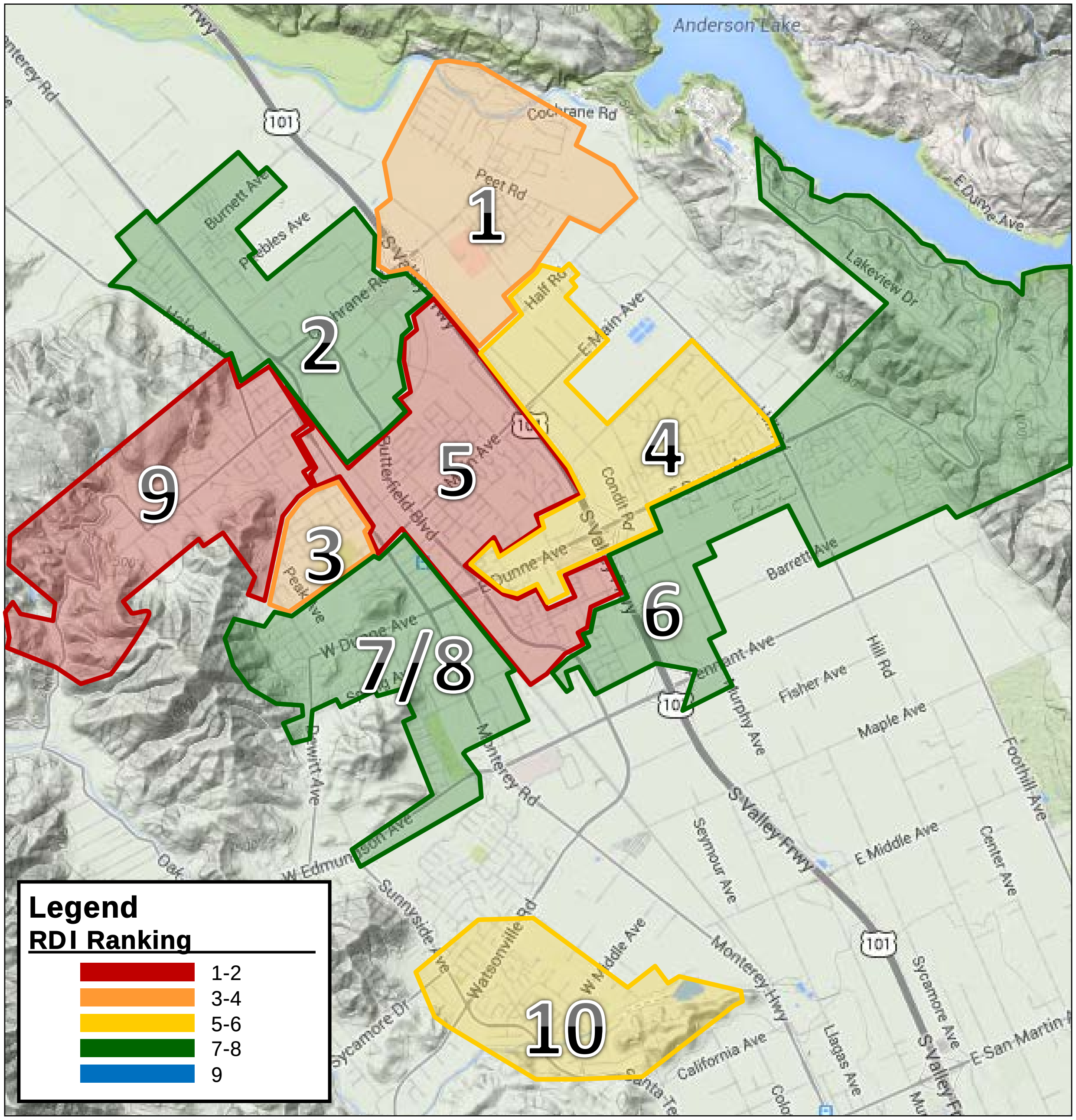


Figure 25. RDI Temperature Map (by Rank)

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[10]. Figure 26 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.

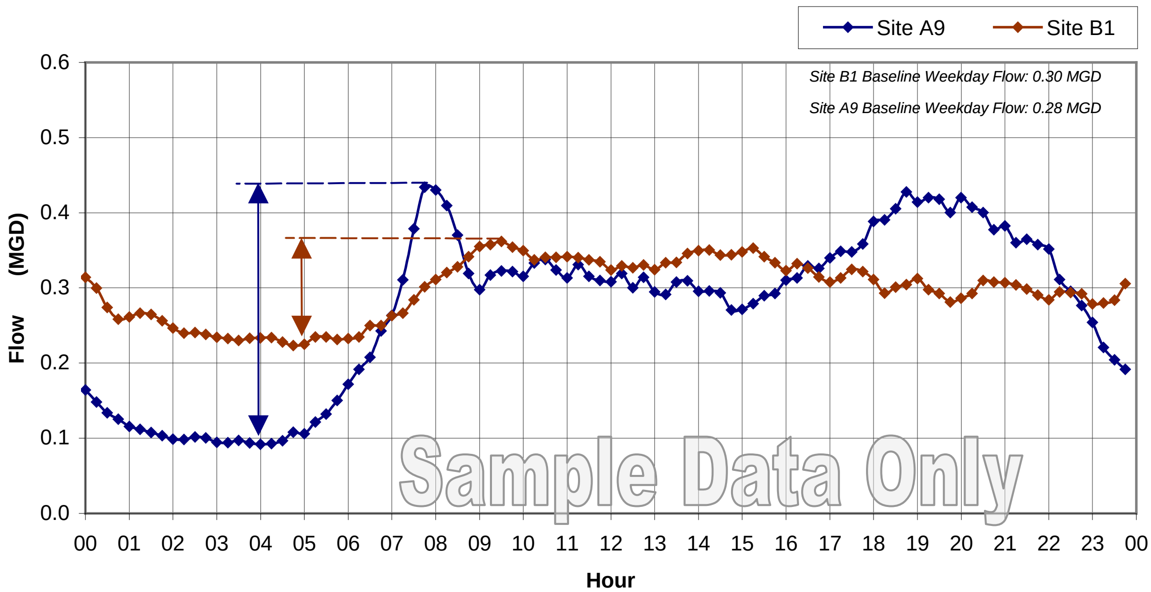


Figure 26. Groundwater Infiltration Sample Figure

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 reasons to suspect abnormal flow patterns (such as proximity to a pump station, treatment facilities, etc.), has a possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system. Figure 27 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)[11]. The following GWI results are noted:

- ❖ Basin 6 had GWI rates that were slightly **above** the WEF typical low-to-average ratio, indicating excessive groundwater infiltration.

Figure 28 shows a color-coded map of the basins with rates of groundwater infiltration considerably above typical groundwater infiltration standards (as set forth by WEF).

[10] In an extreme case, perhaps 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.

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

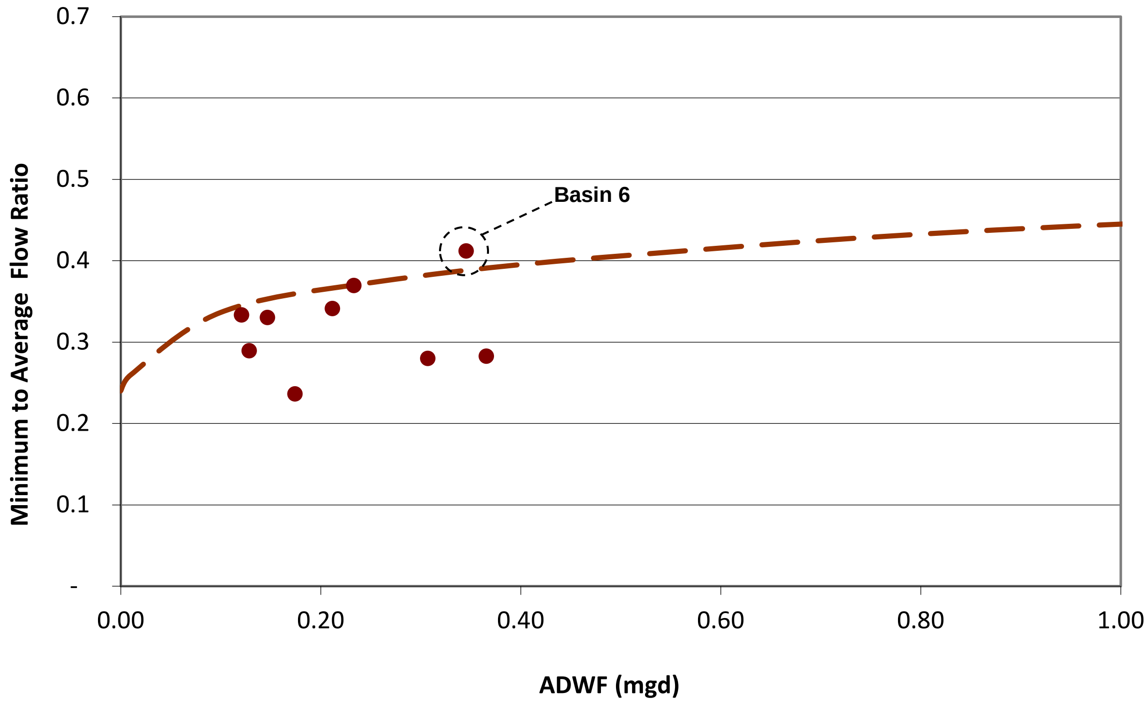


Figure 27. Minimum Flow Ratios vs. ADWF[12]

[12] 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.

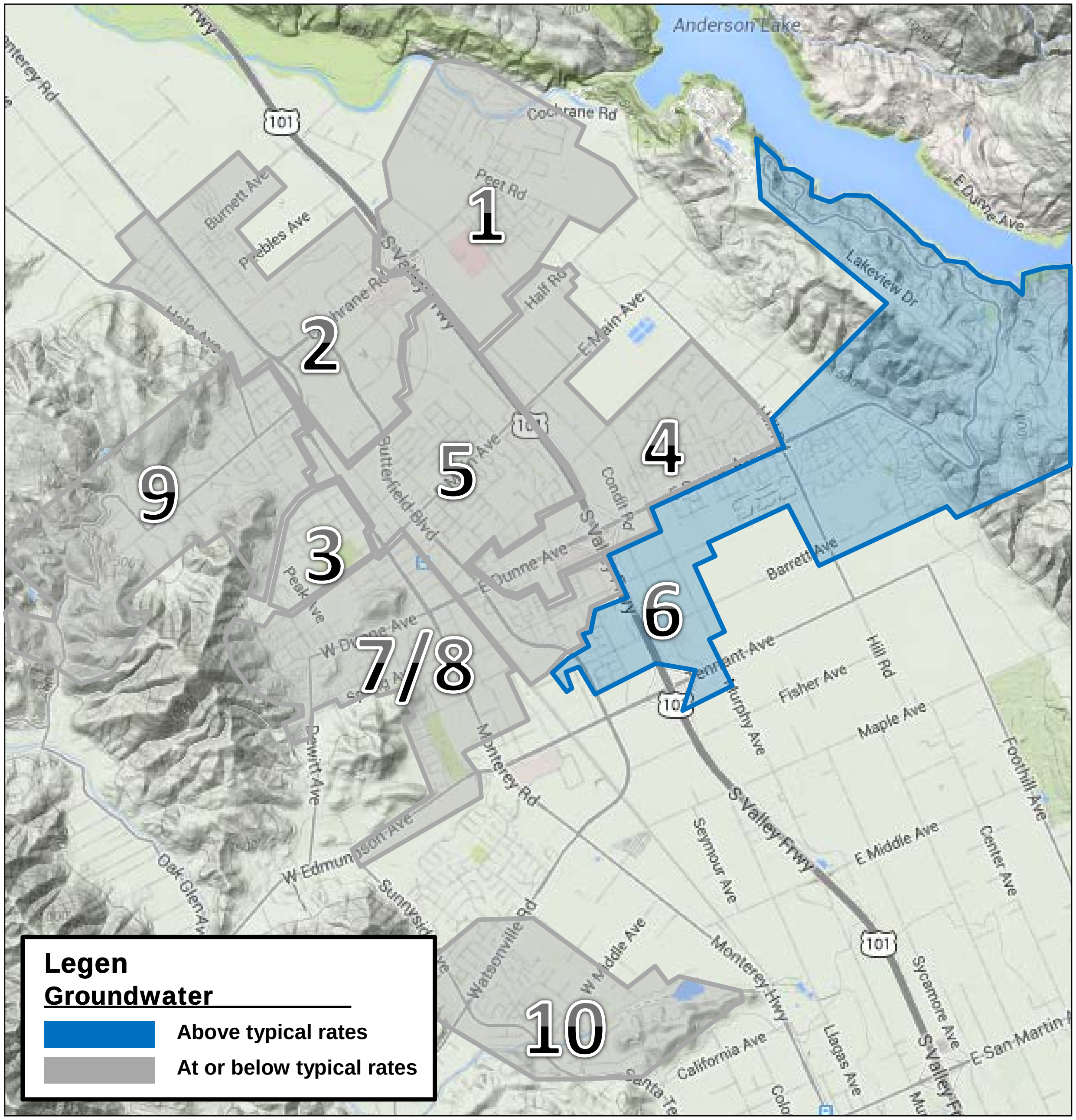


Figure 28. Basins with Groundwater Infiltration

Combined I/I Results Summary

Combined I/I analysis considers the totalized volume (in gallons) of both inflow and rainfall-dependent infiltration over the course of a storm event. Table 12 summarizes the combined I/I flow results for the storm event (refer to the *I/I Methods* section for more information on combined I/I analysis methods and ranking methods). Basins that ranked 1, 2 or 3 in a category are color coded red.

Table 12. Basin Combined I/I Analysis Summary

Metering Basin	ADWF (mgd)	Combined I/I (gallons)	R-Value (%)	Combined I/I per ADWF	Overall Combined I/I Ranking[A]
Basin 1	0.19	56,400	0.13% (6)[B]	0.15 (7)	7
Basin 2	0.17	47,200	0.10% (8)	0.15 (8)	8
Basin 3	0.18	166,200	1.00% (1)	0.52 (1)	1
Basin 4	0.21	114,100	0.26% (3)	0.27 (4)	4
Basin 5	0.27	90,900	0.22% (4)	0.18 (6)	5
Basin 6	0.35	136,300	0.12% (7)	0.18 (5)	6
Basin 7/8	0.40	230,300	0.63% (2)	0.33 (3)	2
Basin 7	*0.24*	*177,700*	*1.31%*	*0.42*	
Basin 8	*0.16*	*52,000*	*0.23%*	*0.19*	
Basin 9	0.13	133,300	0.22% (5)	0.49 (2)	3
Basin 10	0.12	16,400	0.06% (9)	0.06 (9)	9
System	***2.65***	***1,135,500***	***0.23%***	***0.22***	

[A] Ranking of 1 represents most inflow after normalization.
[B] The number in parenthesis shows the ranking within the individual Category.

The following combined I/I analysis results are noted:

- Basins 3, 7/8, 9 and 4 ranked highest for normalized combined I/I contribution.
  - If isolated, Basin 7 ranks highly within the Basin 7/8 basin and highly within the collection system.

Figure 29 shows bar graph summaries of the combined I/I analysis. A temperature map by overall ranking is shown in Figure 30.

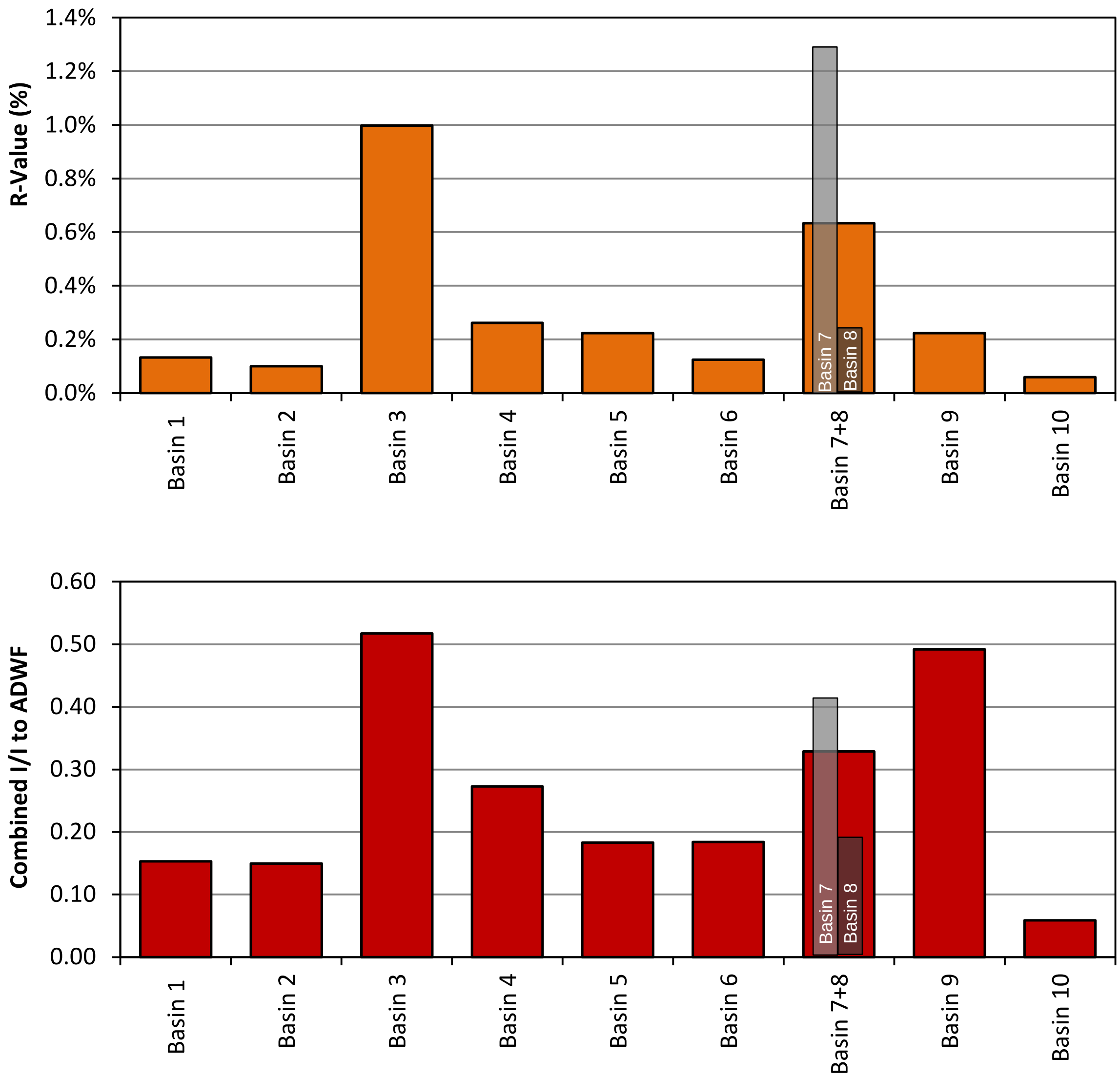


Figure 29. Bar Graphs: Combined I/I Analysis Summary

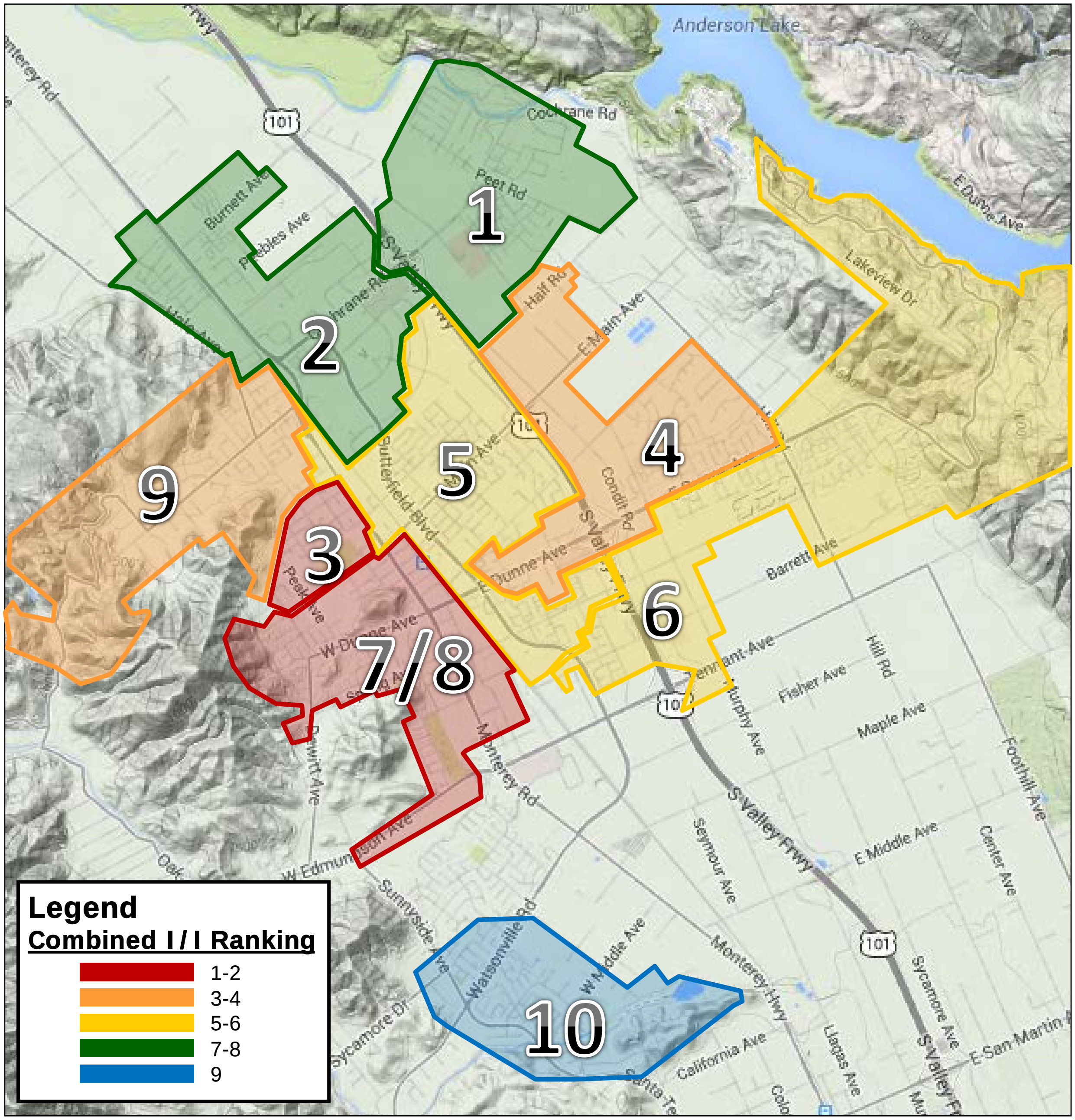


Figure 30. Combined I/I Temperature Map (by Rank)

Inflow / Infiltration: Synthetic Hydrographs

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 Storm Event 1, for Site 3 is shown in Figure 31.

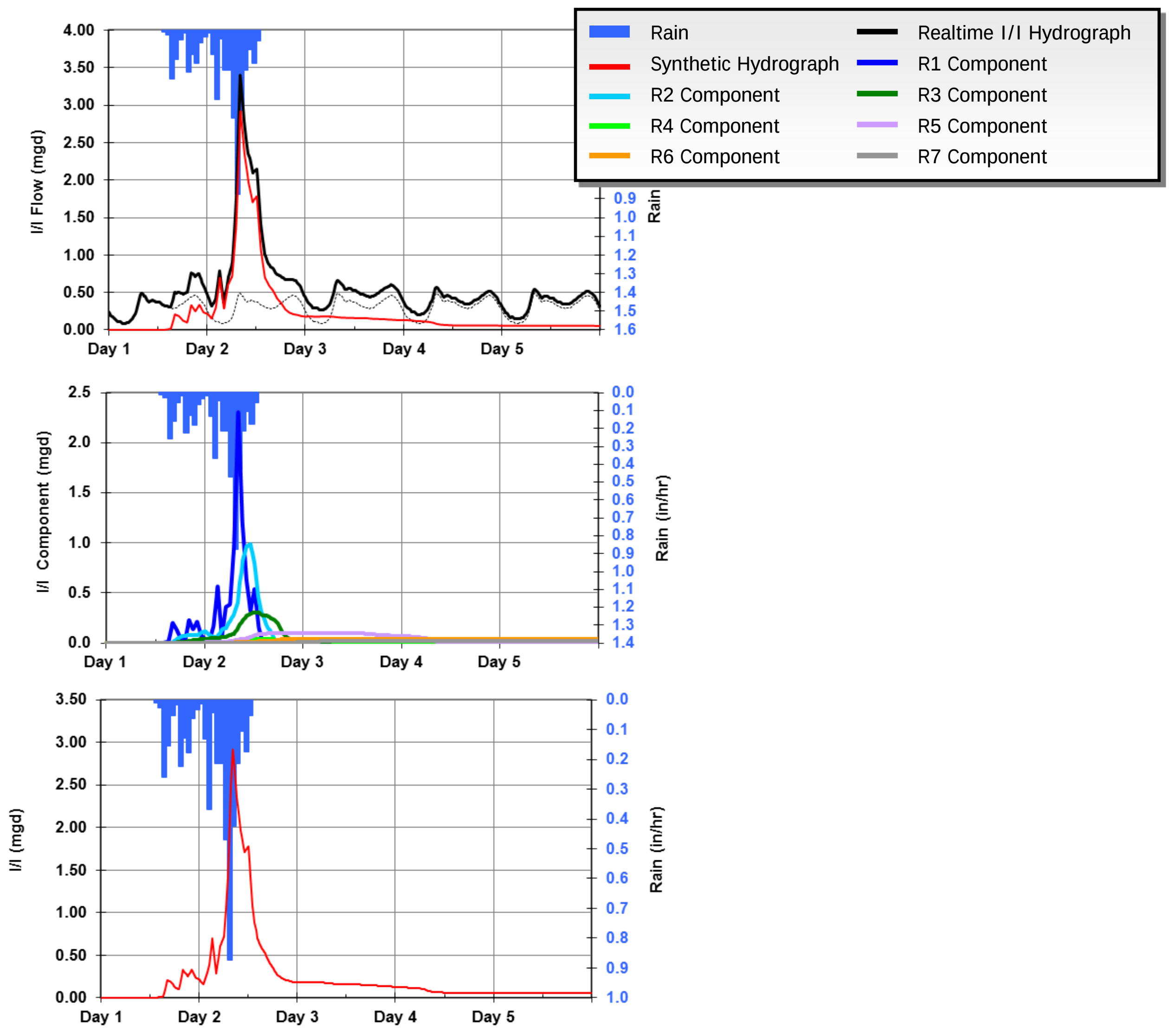


Figure 31. Site 3, Storm Event 1: Synthetic Hydrograph

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 32 summarizes the design storm magnitude and profile. This particular profile distribution also fits the NOAA criterion for 2-hour and 6-hour durations, in addition to the 24-hour duration.

10-Year, 24-hour Design Storm	
Hour	**Inches of Rain**
1	0.010
2	0.026
3	0.259
4	0.156
5	0.052
6	0.016
7	0.223
8	0.127
9	0.178
10	0.064
11	0.032
12	0.013
13	0.131
14	0.368
15	0.044
16	0.213
17	0.213
18	0.471
19	0.875
20	0.425
21	0.213
22	0.105
23	0.175
24	0.053
Total:	**4.44**

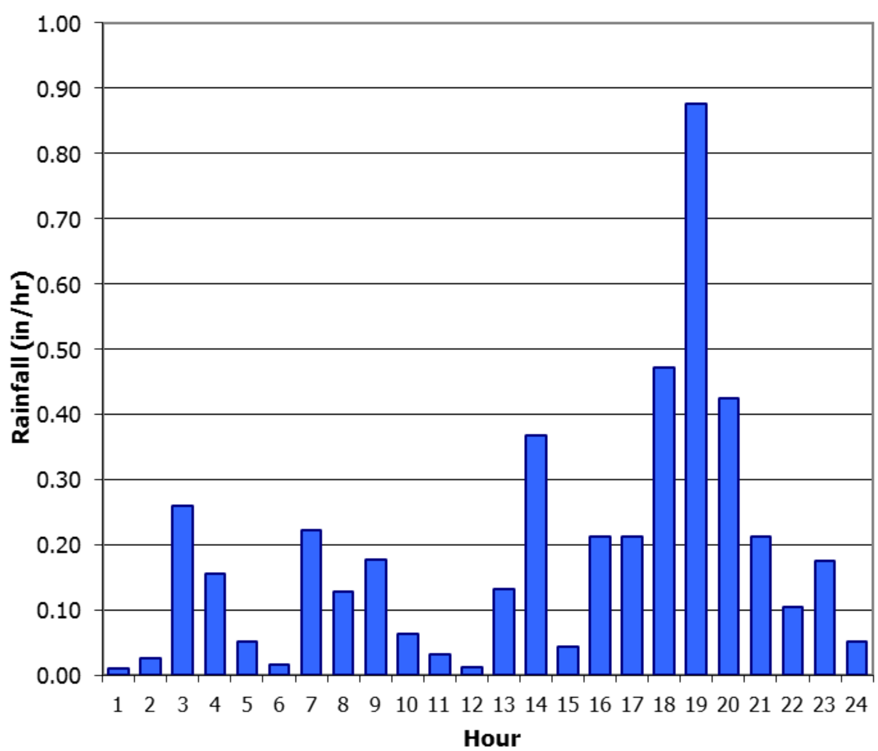


Figure 32. 10-Year, 24-Hour Design Storm Values and Profile (MORGA30)

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 that the peak I/I flows from the design storm coincide with peak sanitary flows to produce a "worst-case" scenario of peak wet weather flows. Table 13 summarizes the final results for the design storm on a site-by-site basis.

Table 13. Design Storm I/I Analysis Summary

Metering Site	Predicted Peak Dry Weather Flow (mgd)	Predicted Peak I/I Rate (mgd)	Predicted Peak Flow (mgd)	Predicted Total I/I (gallons)
Site 1	0.34	0.75	1.09	285,000
Site 2	0.58	1.42	2.00	451,000
Site 3	0.56	3.13	3.69	1,134,000
Site 4	0.37	1.02	1.39	279,000
Site 5	1.89	5.26	7.15	2,128,000
Site 6	0.67	1.22	1.89	468,000
Site 7	0.38	2.41	2.80	749,000
Site 8	0.29	0.70	0.99	214,000
Site 9	0.25	1.16	1.40	371,000
Site 10	0.24	0.14	0.39	52,000
Harding	4.46	8.74	13.20	4,075,000

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 occurred in Basins 3, 7/8 and 9.
    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 occurred in Basins 3, 5 and 9.
        ii. The highest groundwater infiltration occurred in Basins 5 and 6.
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.

APPENDIX A

FLOW MONITORING SITES: DATA, GRAPHS, INFORMATION

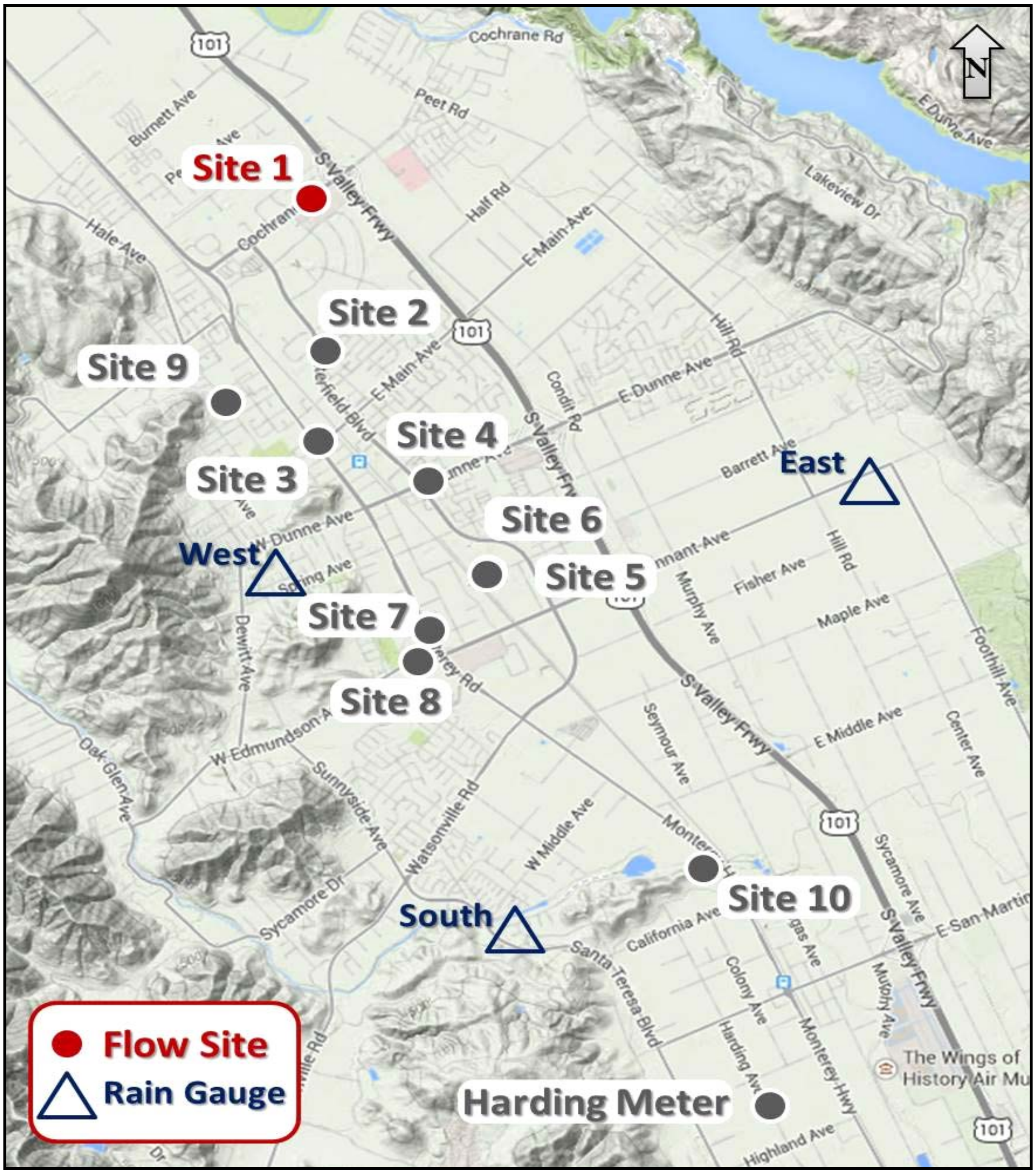
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 1

Location: Behind Residence Inn, off Madrone Parkway

Data Summary Report

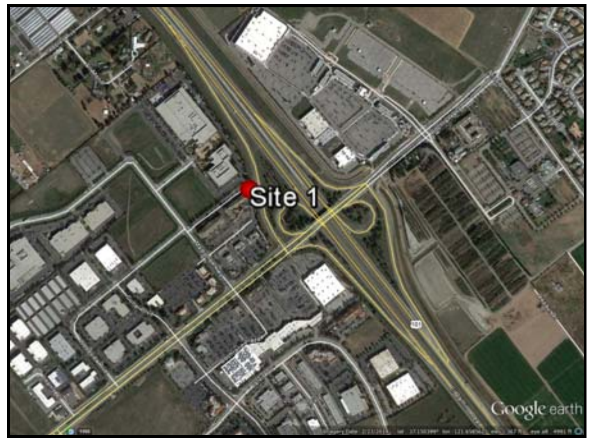


Vicinity Map: Site 1

SITE 1

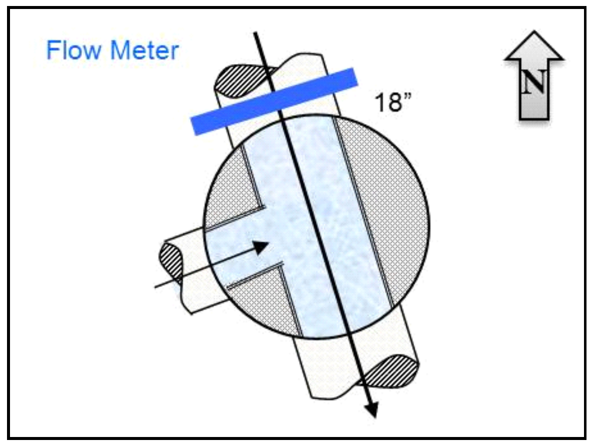
Site Information

Location:	Behind Residence Inn, off Madrone Parkway
Coordinates:	121.6549° W, 37.1537° N
Rim Elevation:	377 feet
Pipe Diameter:	18 inches
Baseline Flow:	0.192 mgd
Peak Measured Flow:	0.560 mgd



Satellite Map

Sanitary Map



Flow Sketch

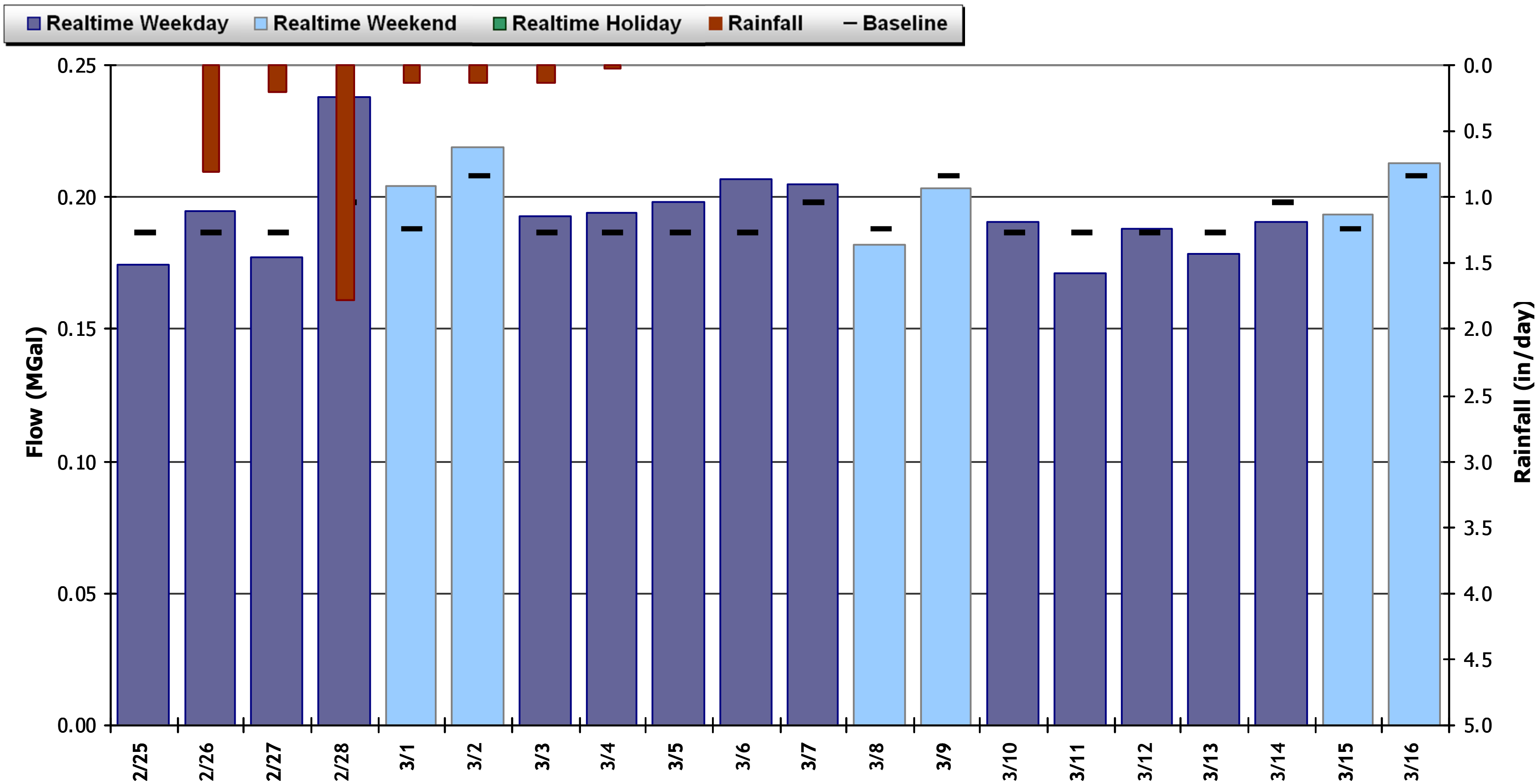
Street View

Plan View

SITE 1
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.196 MGal **Peak Daily Flow: 0.238 MGal** **Min Daily Flow: 0.171 MGal**

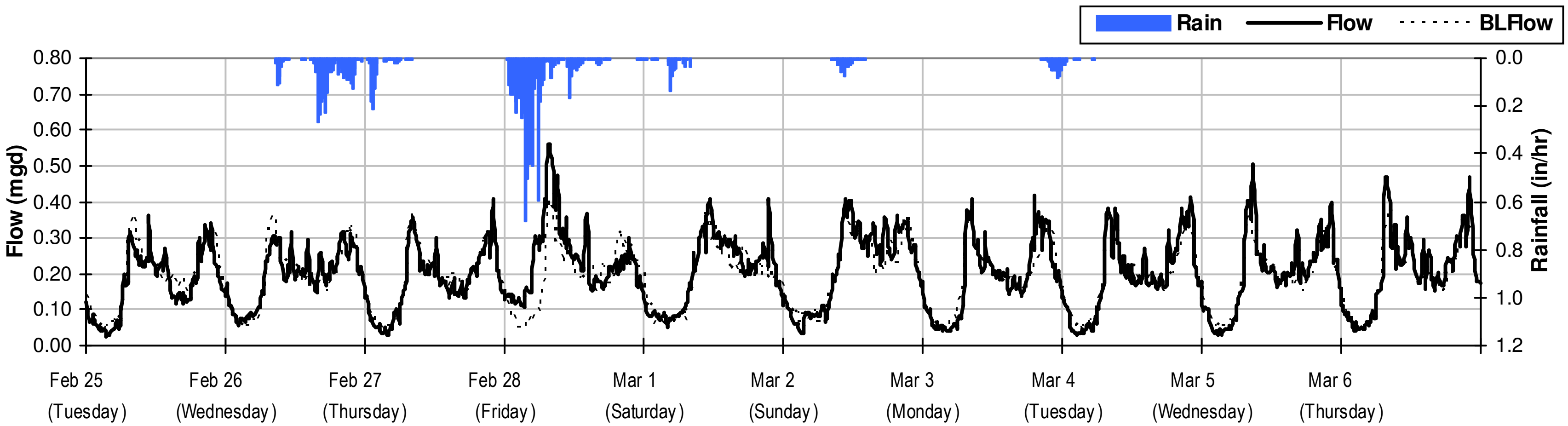
Total Period Rainfall: 3.25 inches

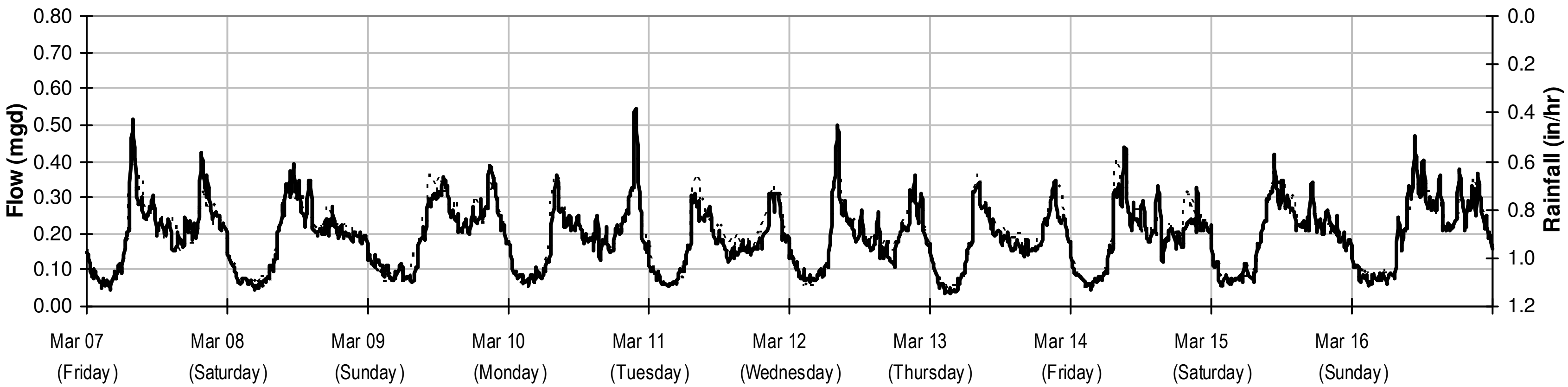


SITE 1

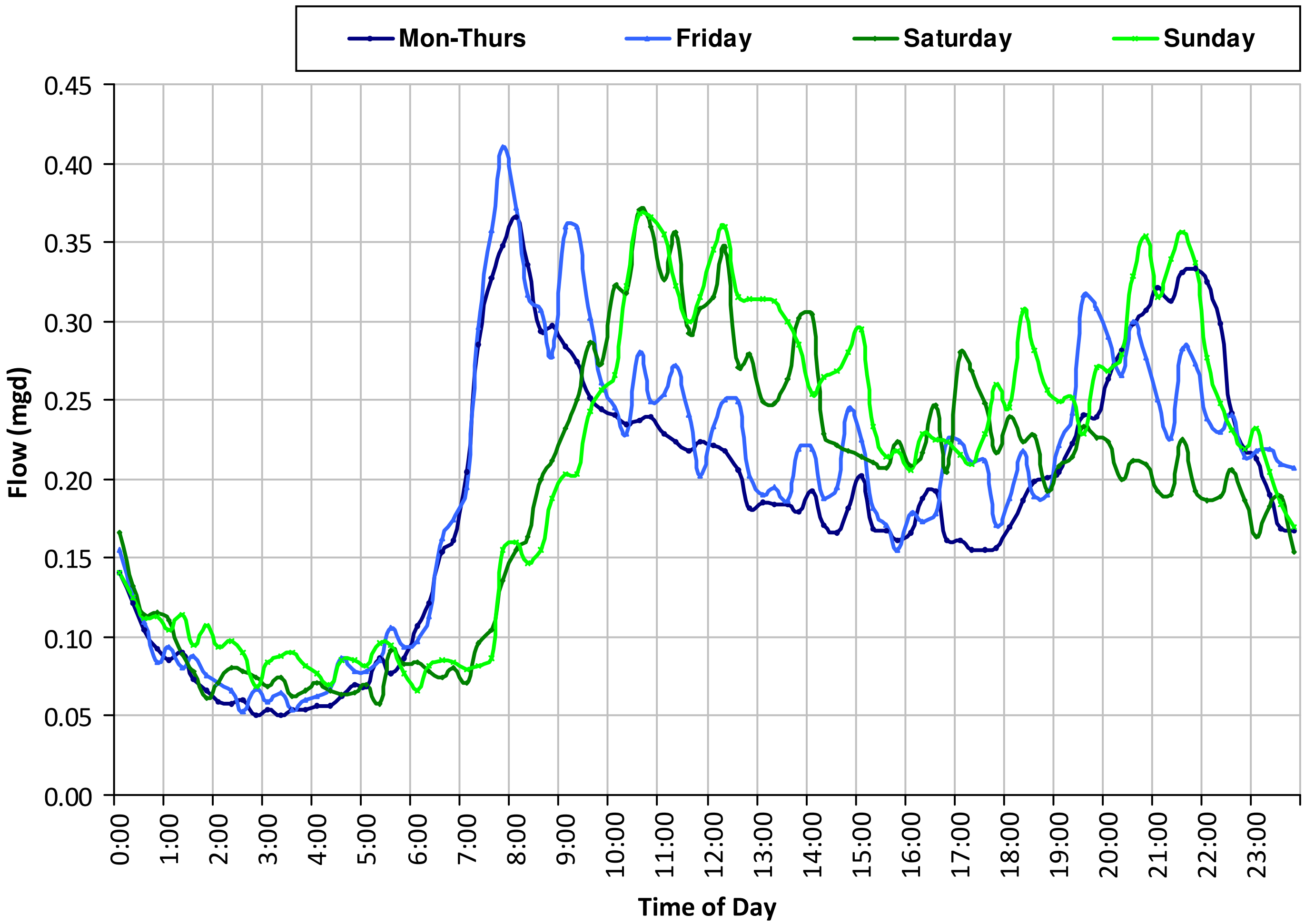
Flow Summary: 2/25/2014 to 3/17/2014

Total Period Rainfall: 3.25 inches **Avg Flow: 0.196 mgd** **Peak Flow: 0.560 mgd** **Min Flow: 0.028 mgd**



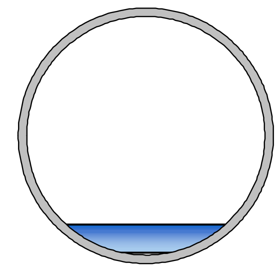


SITE 1
Baseline Flow Hydrographs



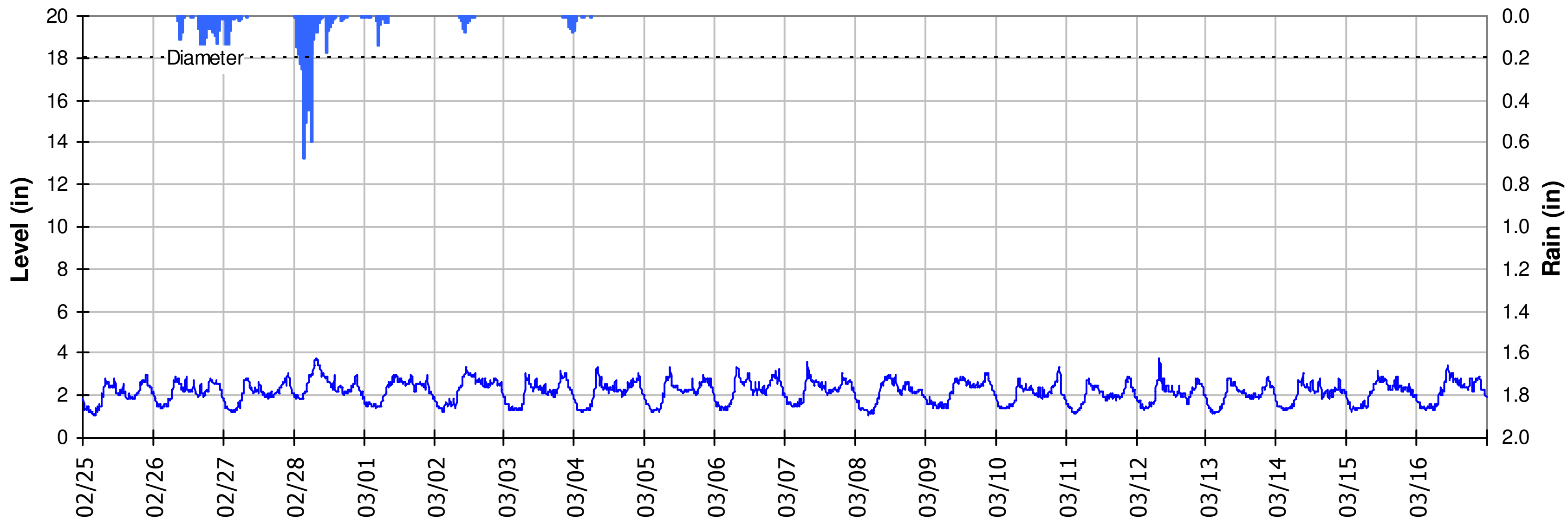
Baseline Flow:

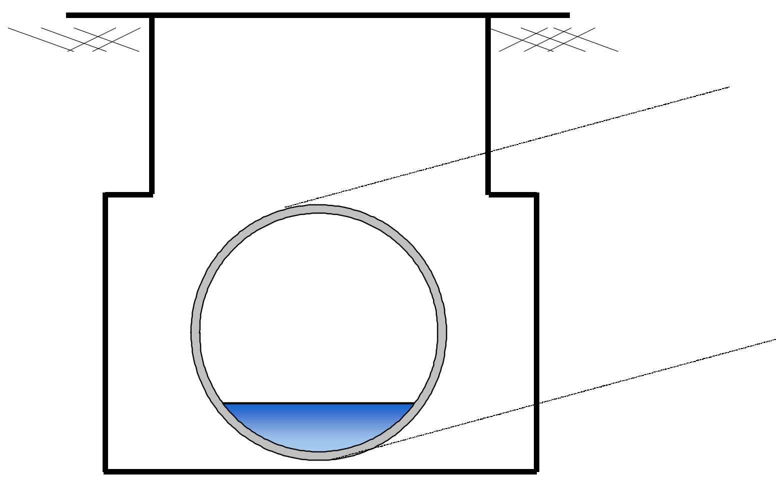
0.192 *mgd*



SITE 1
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

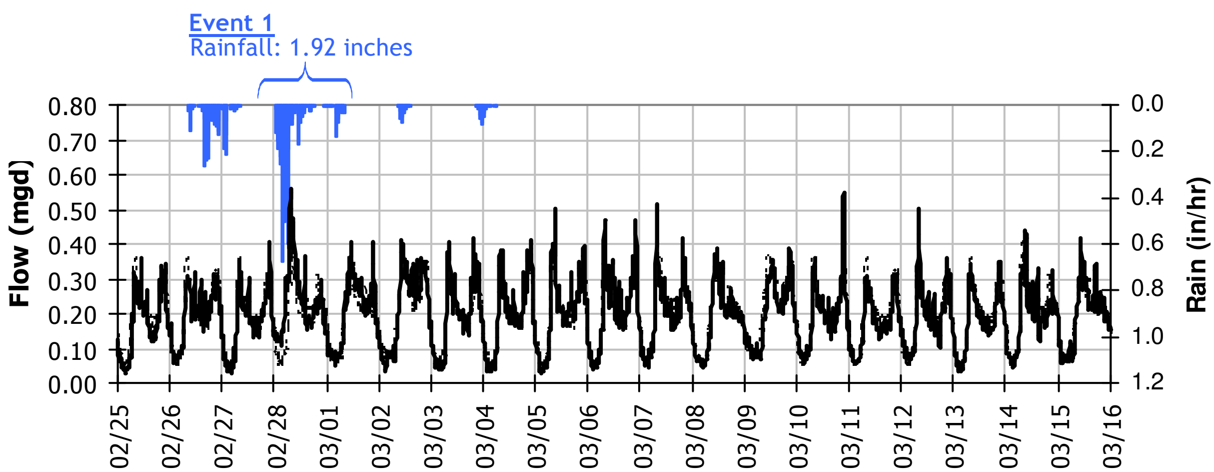




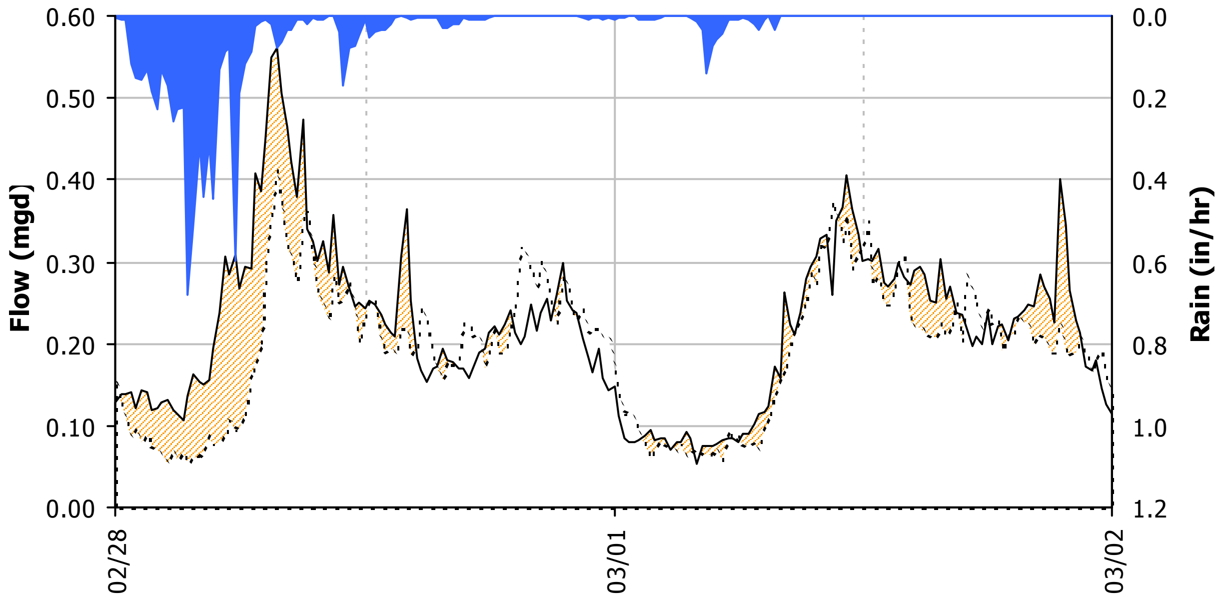
Pipe Diameter:	18 *inches*
Peak Measured Level:	3.73 *inches*
Peak d/D Ratio:	0.21

SITE 1
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



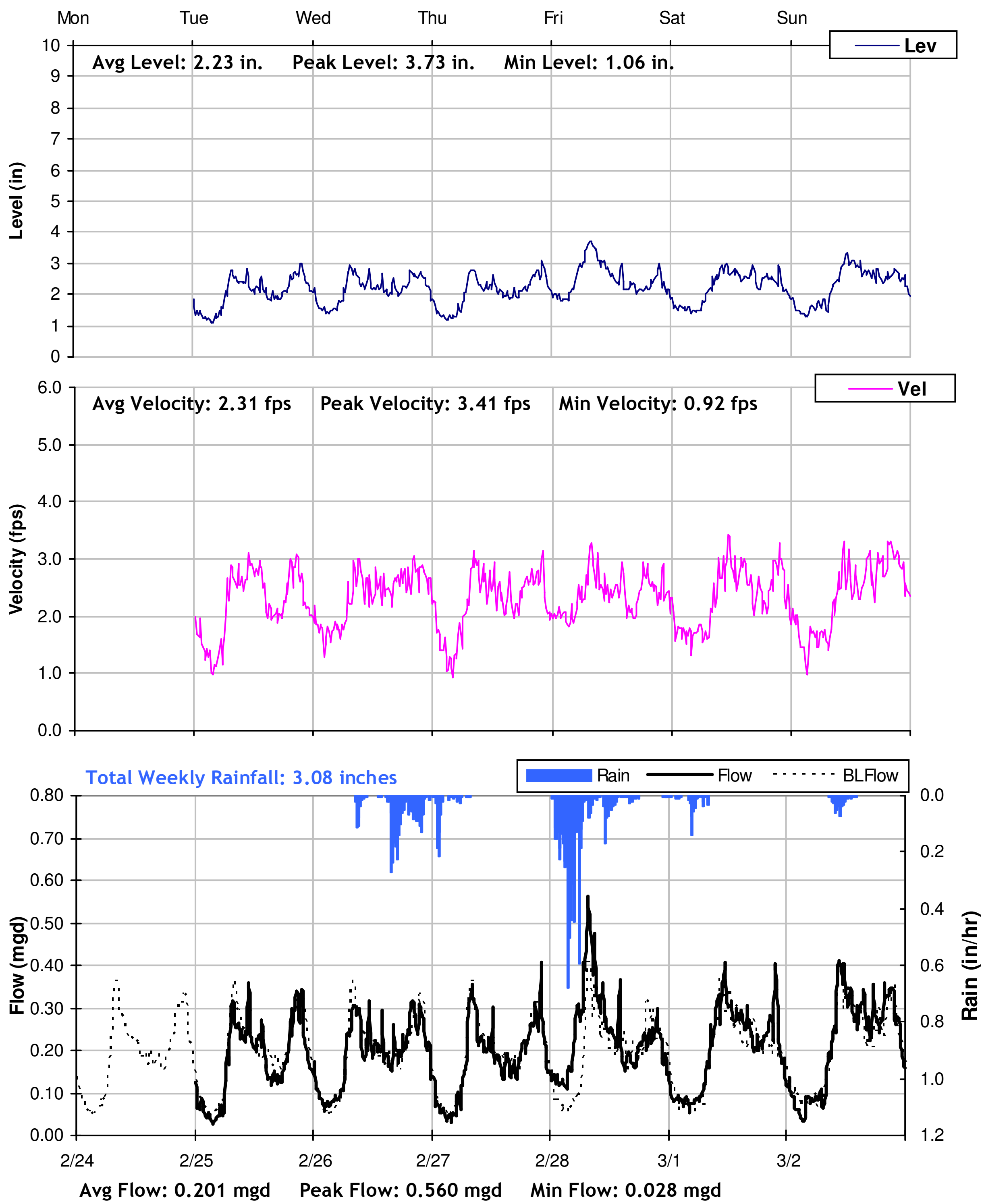
Event 1 Detail Graph



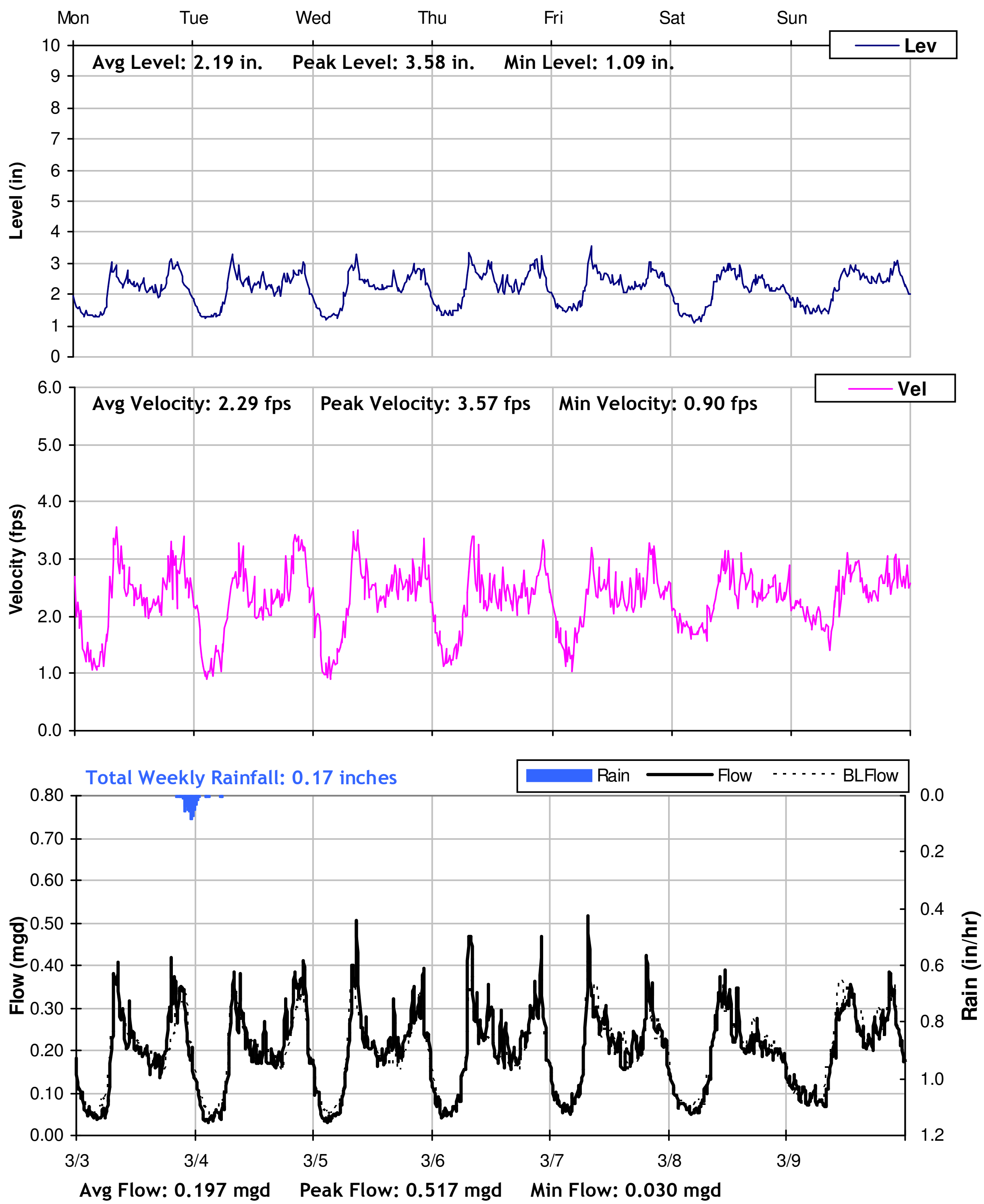
Storm Event I/I Analysis (Rain = 1.92 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.56 *mgd*	**Peak I/I Rate:**	0.23 *mgd*
PF:	2.93	**Total I/I:**	56,000 *gallons*

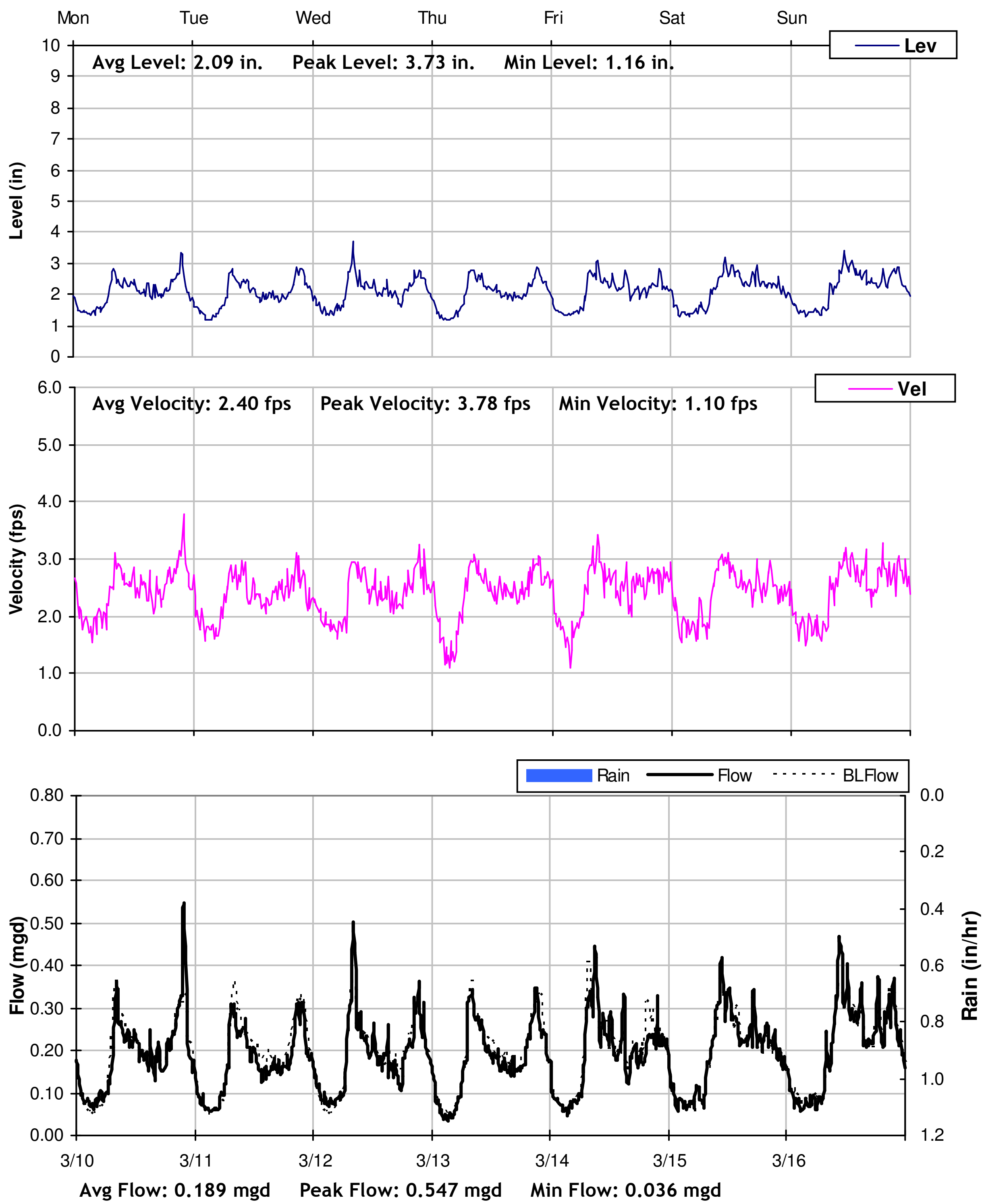
SITE 1
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



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



SITE 1
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014



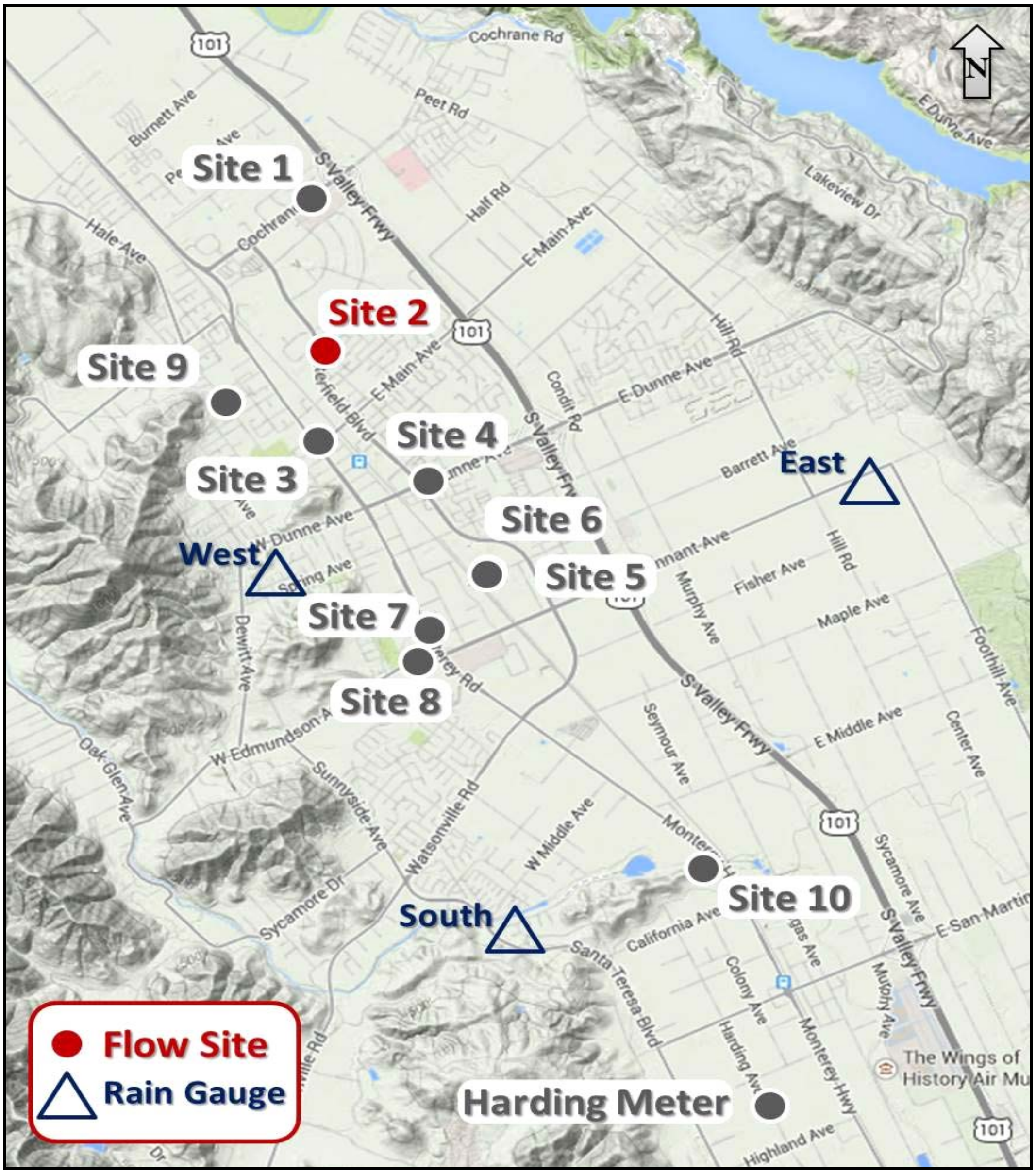
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 2

Location: Butterfield Boulevard, south of Jarvis Drive

Data Summary Report



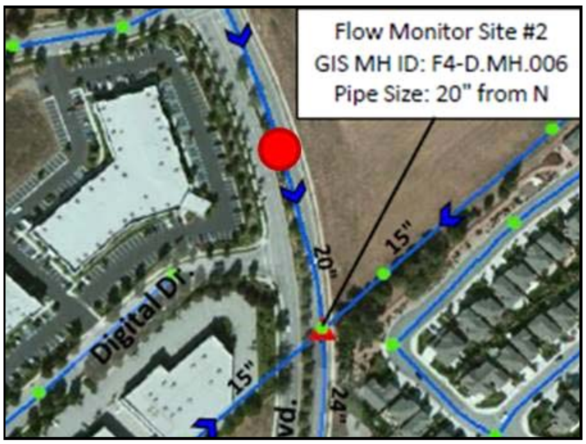
Vicinity Map: Site 2

SITE 2

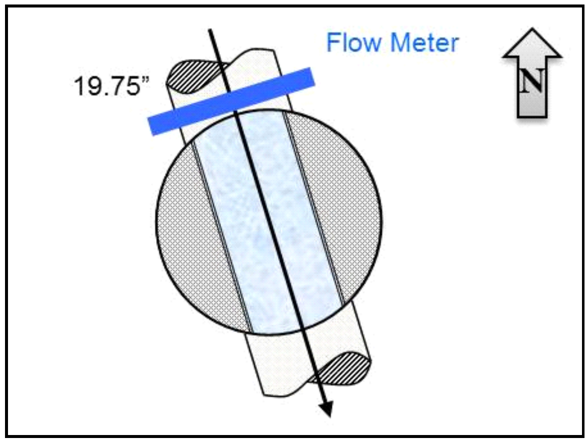
Site Information

Location:	Butterfield Boulevard, south of Jarvis Drive
Coordinates:	121.6546° W, 37.1393° N
Rim Elevation:	361 feet
Pipe Diameter:	19.75 inches
Baseline Flow:	0.357 mgd
Peak Measured Flow:	0.877 mgd

Satellite Map



Sanitary Map



Flow Sketch

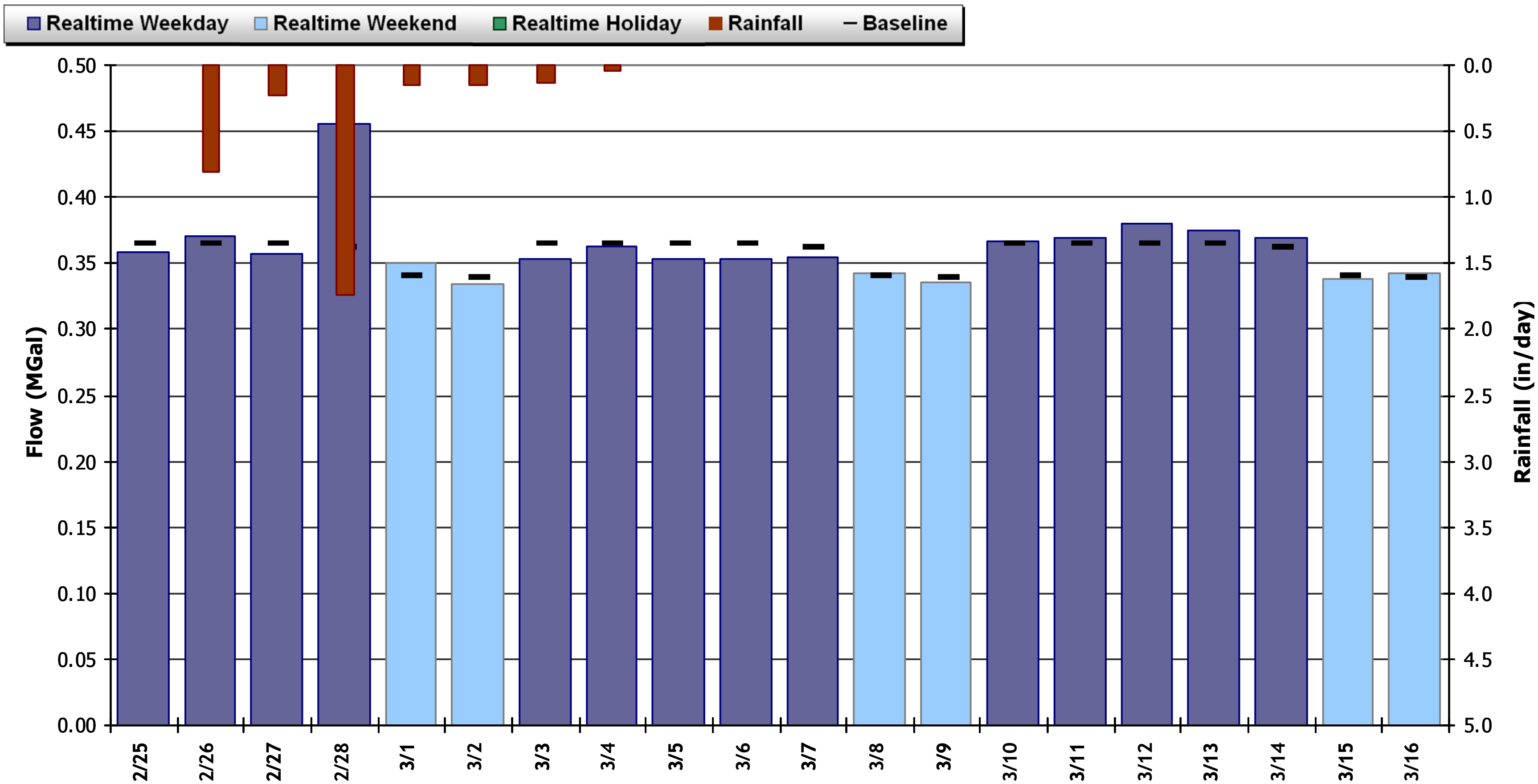
Street View

Plan View

SITE 2
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.361 MGal **Peak Daily Flow: 0.456 MGal** **Min Daily Flow: 0.334 MGal**

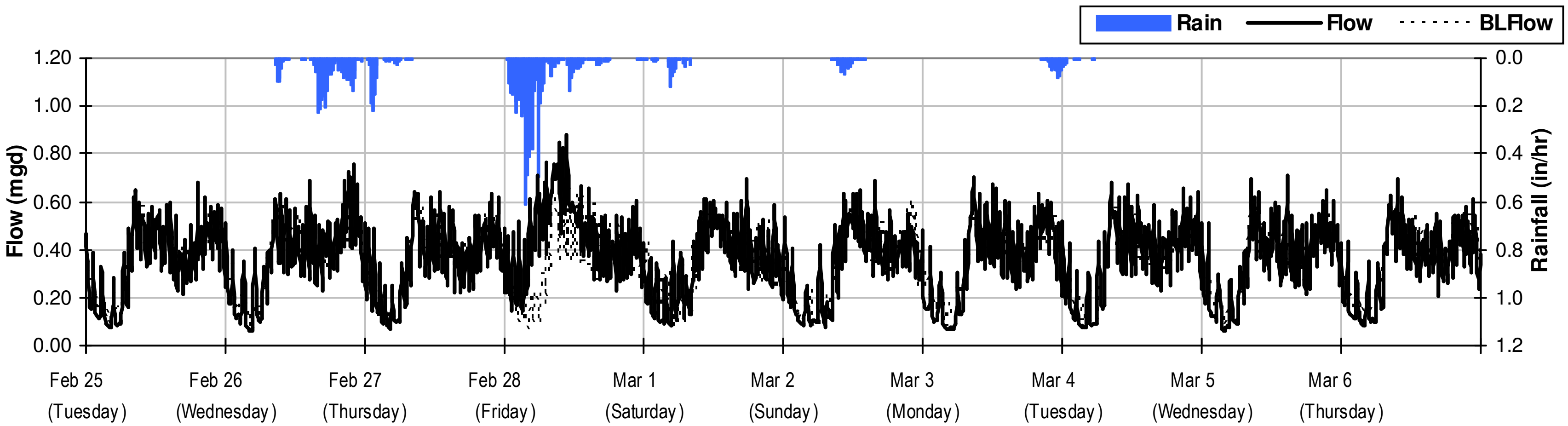
Total Period Rainfall: 3.24 inches

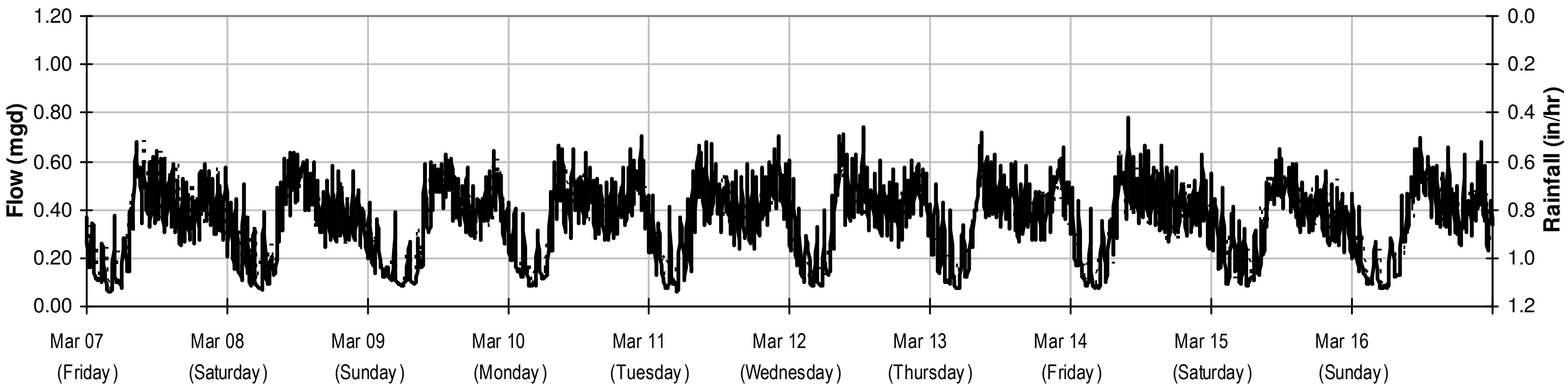


SITE 2

Flow Summary: 2/25/2014 to 3/17/2014

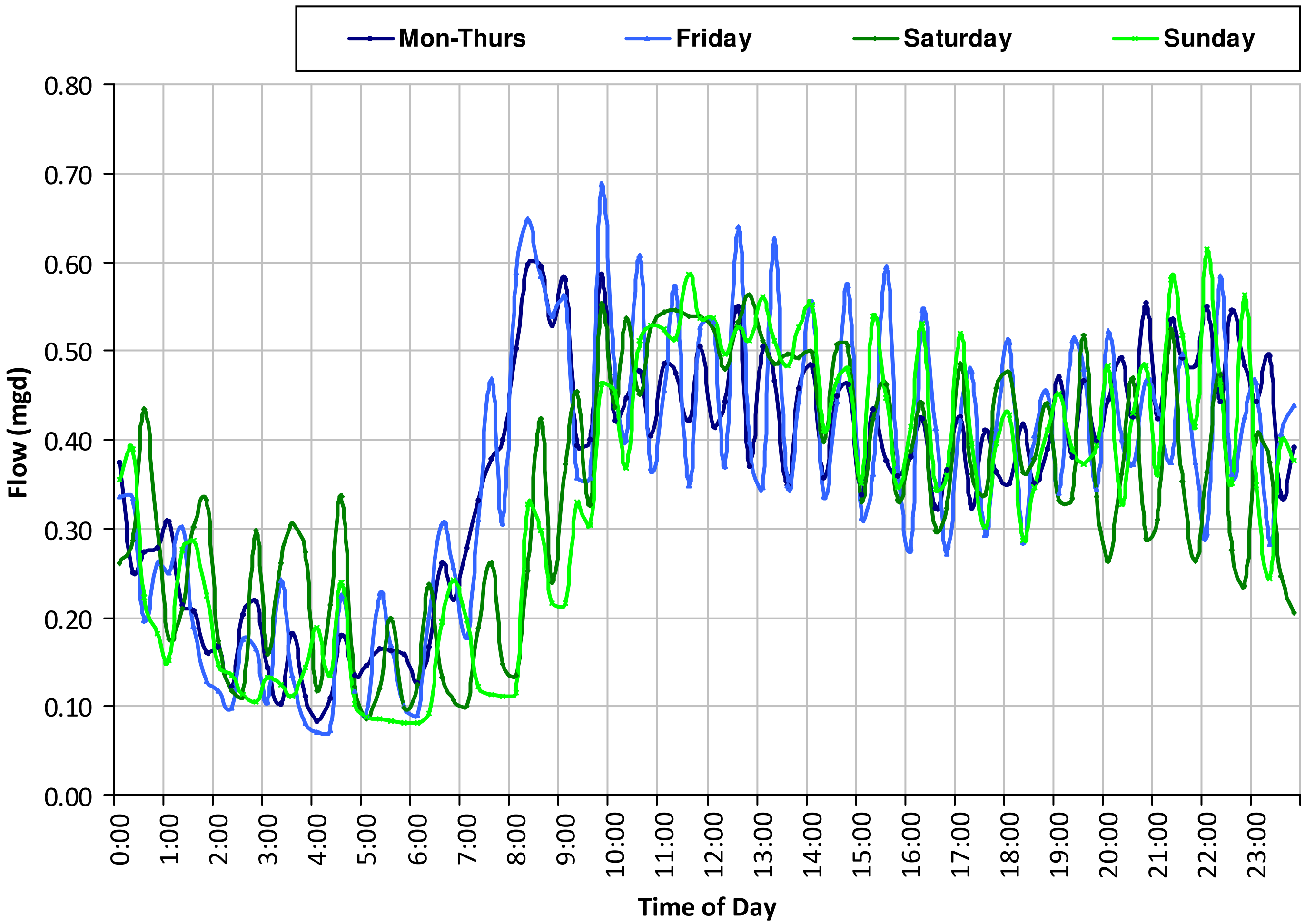
Total Period Rainfall: 3.24 inches   Avg Flow: 0.361 mgd   Peak Flow: 0.877 mgd   Min Flow: 0.062 mgd





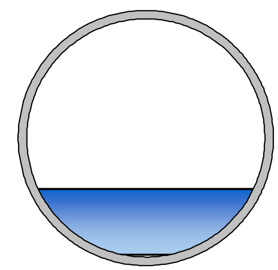
SITE 2

Baseline Flow Hydrographs



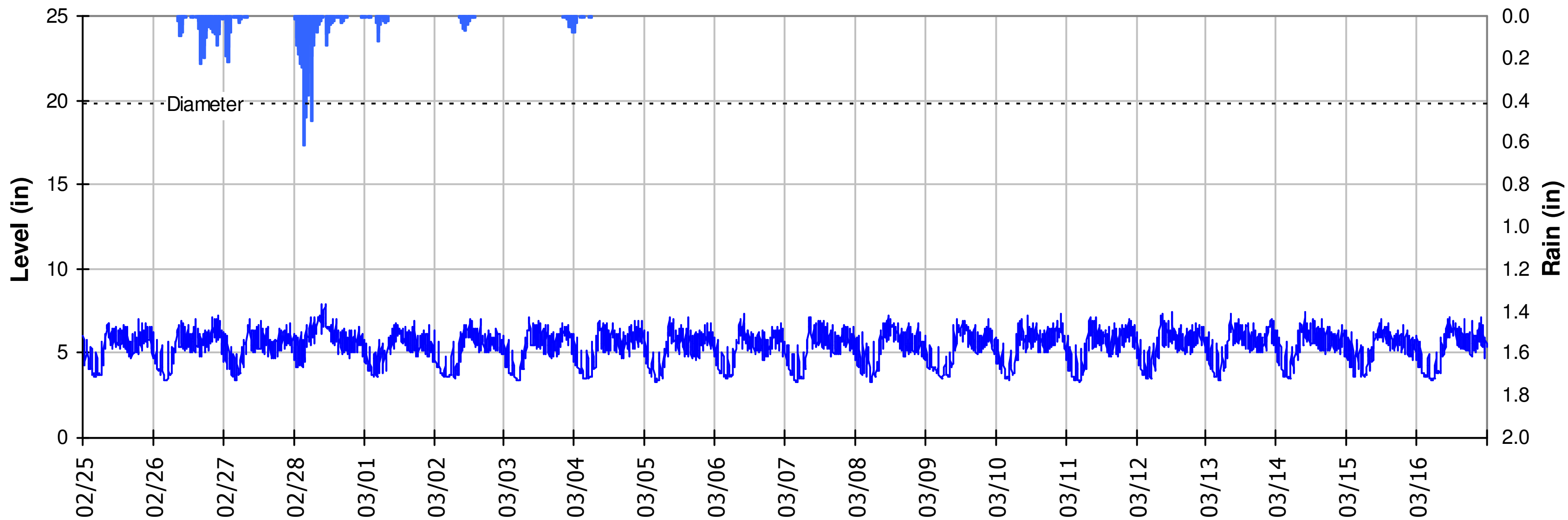
Baseline Flow:

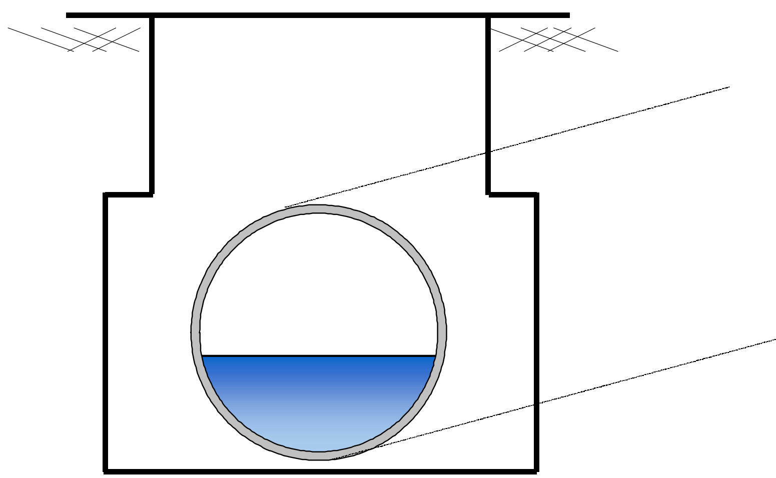
0.357 *mgd*



SITE 2
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





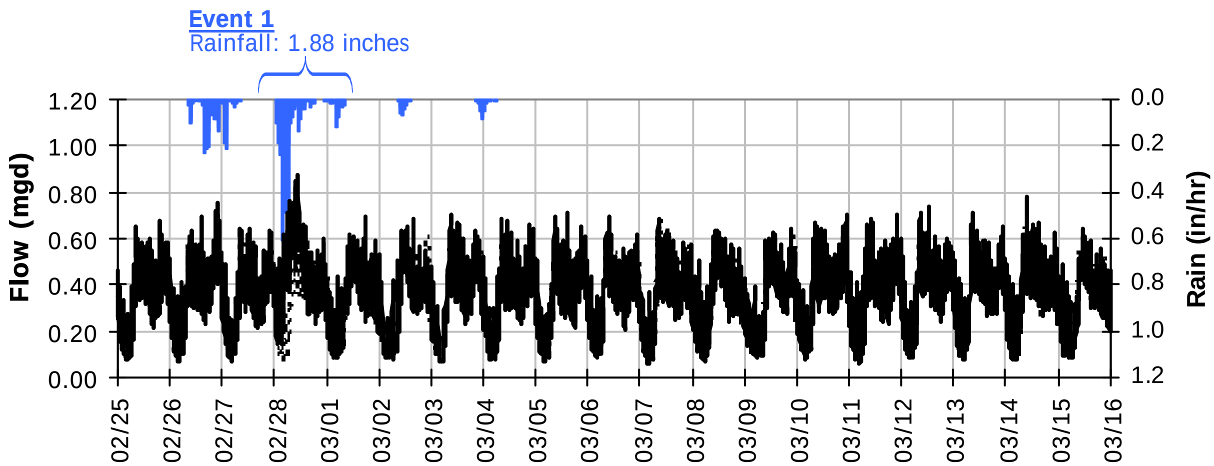
Pipe Diameter: 19.8 *inches*

Peak Measured Level: 7.94 *inches*

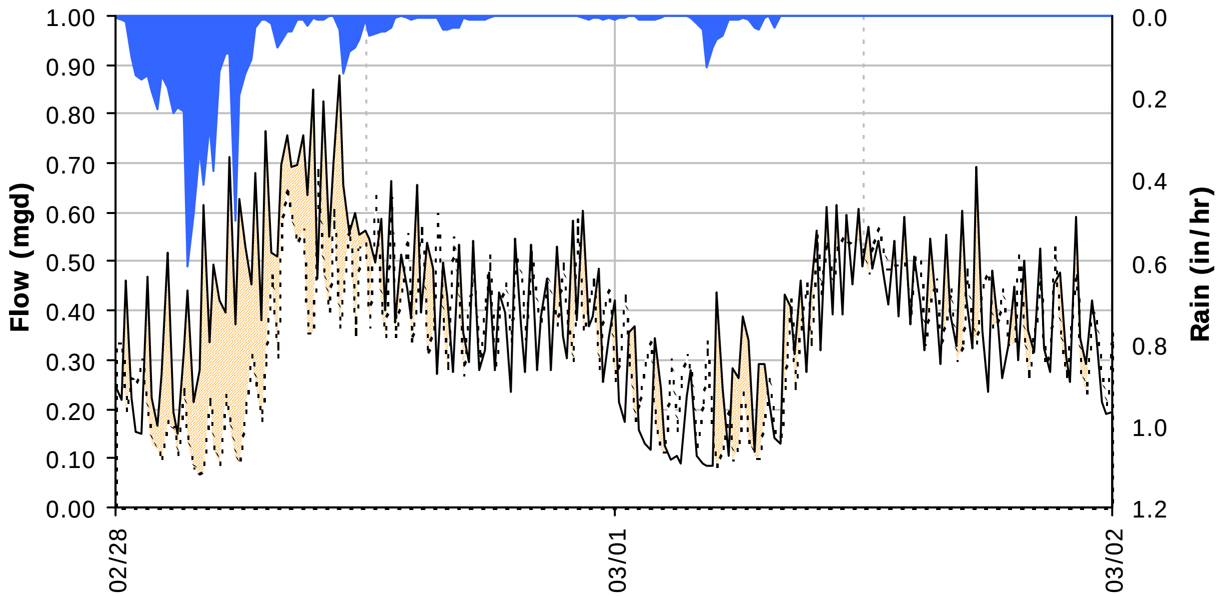
Peak d/D Ratio: 0.40

SITE 2
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



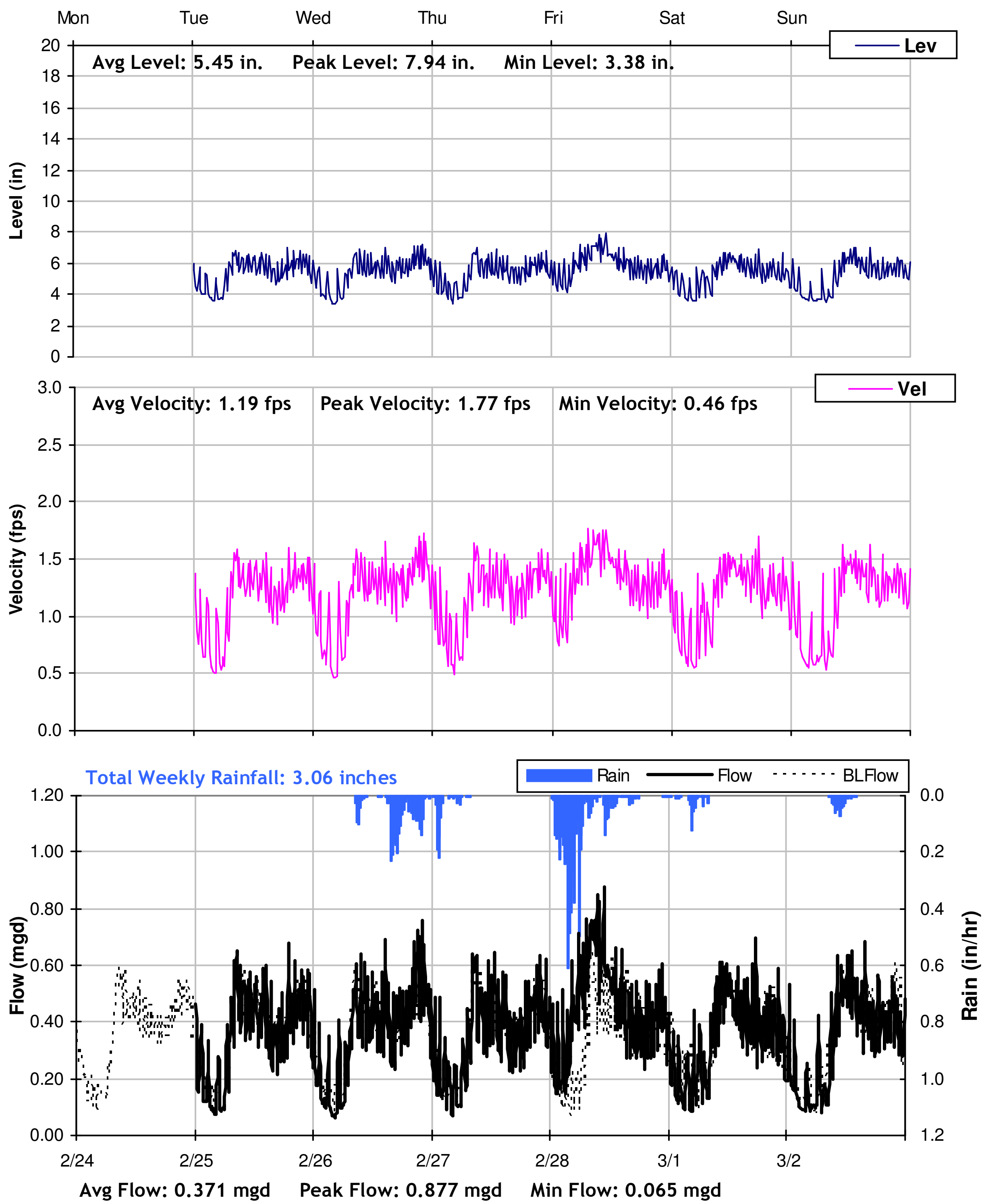
Event 1 Detail Graph



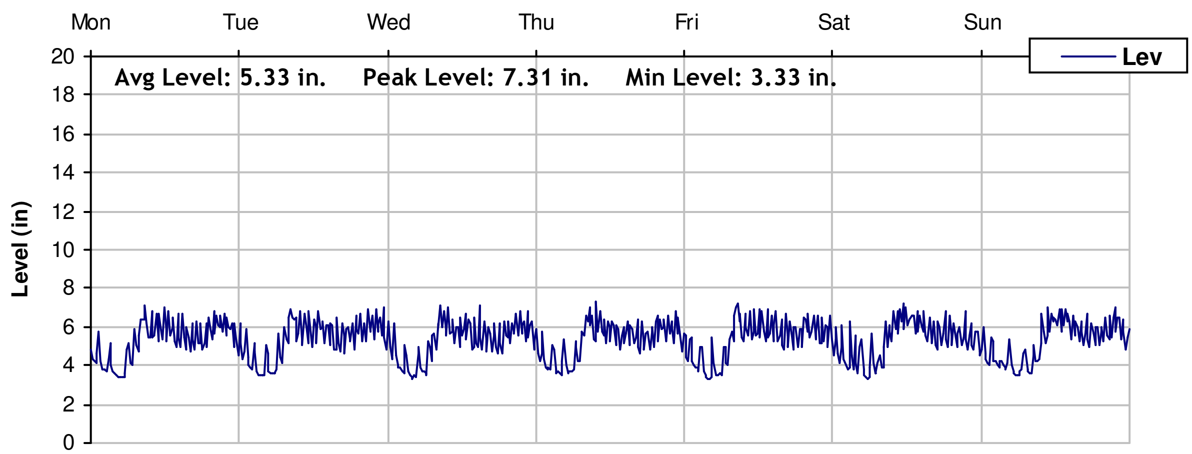
Storm Event I/I Analysis (Rain = 1.88 inches)

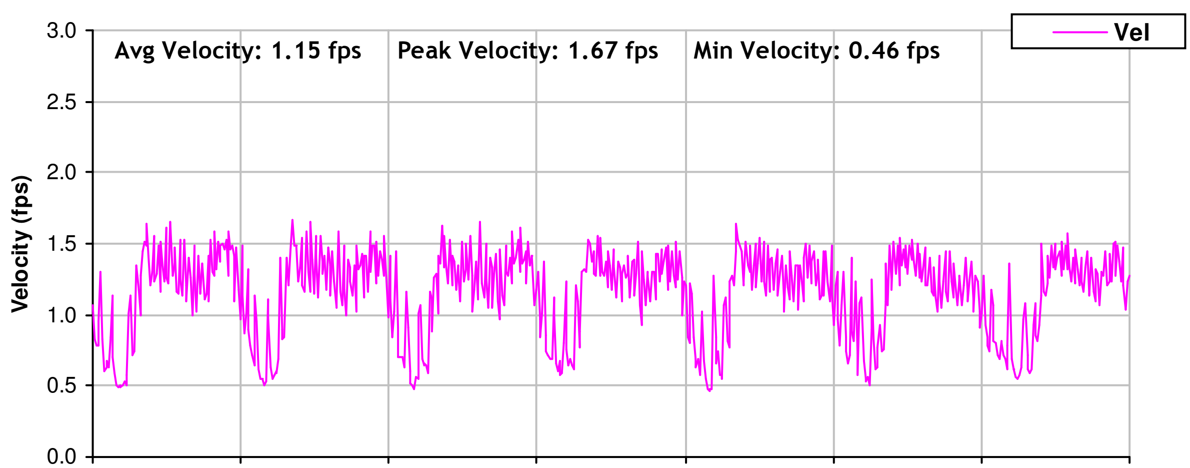
Capacity		Inflow / Infiltration	
Peak Flow:	0.88 *mgd*	**Peak I/I Rate:**	0.35 *mgd*
PF:	2.45	**Total I/I:**	104,000 *gallons*

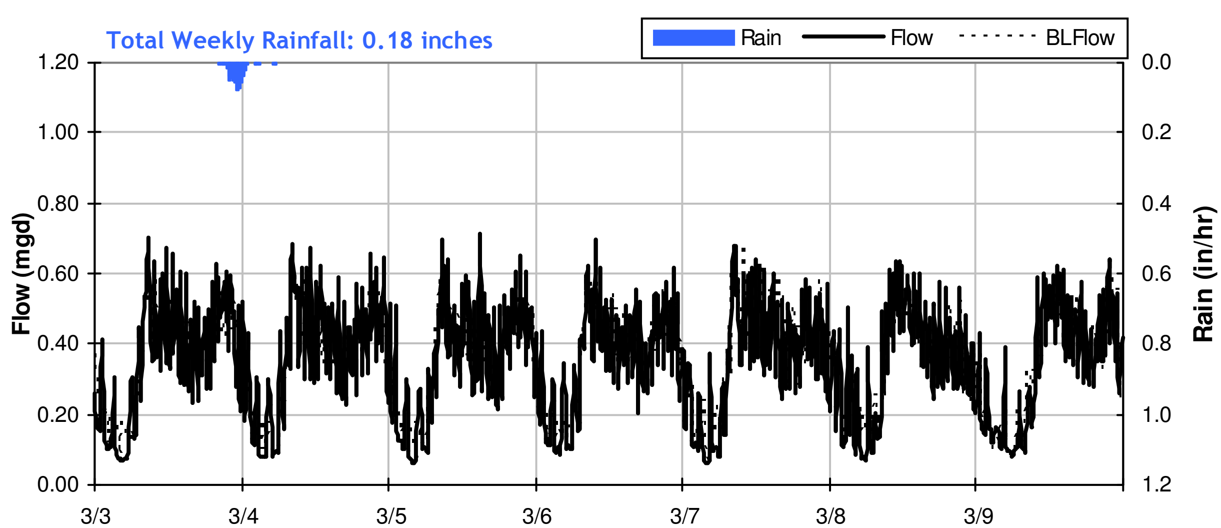
SITE 2
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



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

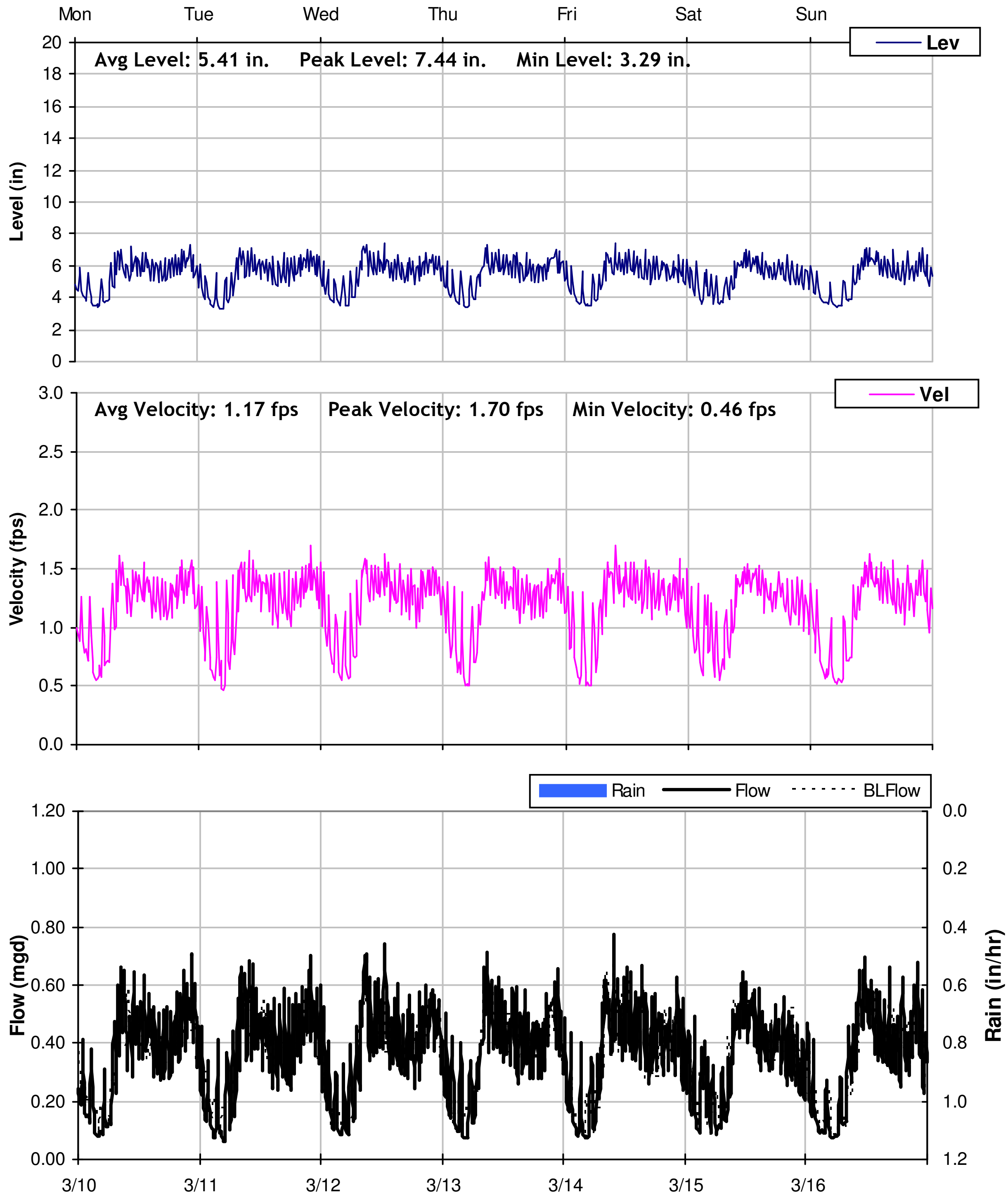






Avg Flow: 0.351 mgd Peak Flow: 0.711 mgd Min Flow: 0.062 mgd

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



Avg Flow: 0.363 mgd Peak Flow: 0.776 mgd Min Flow: 0.062 mgd

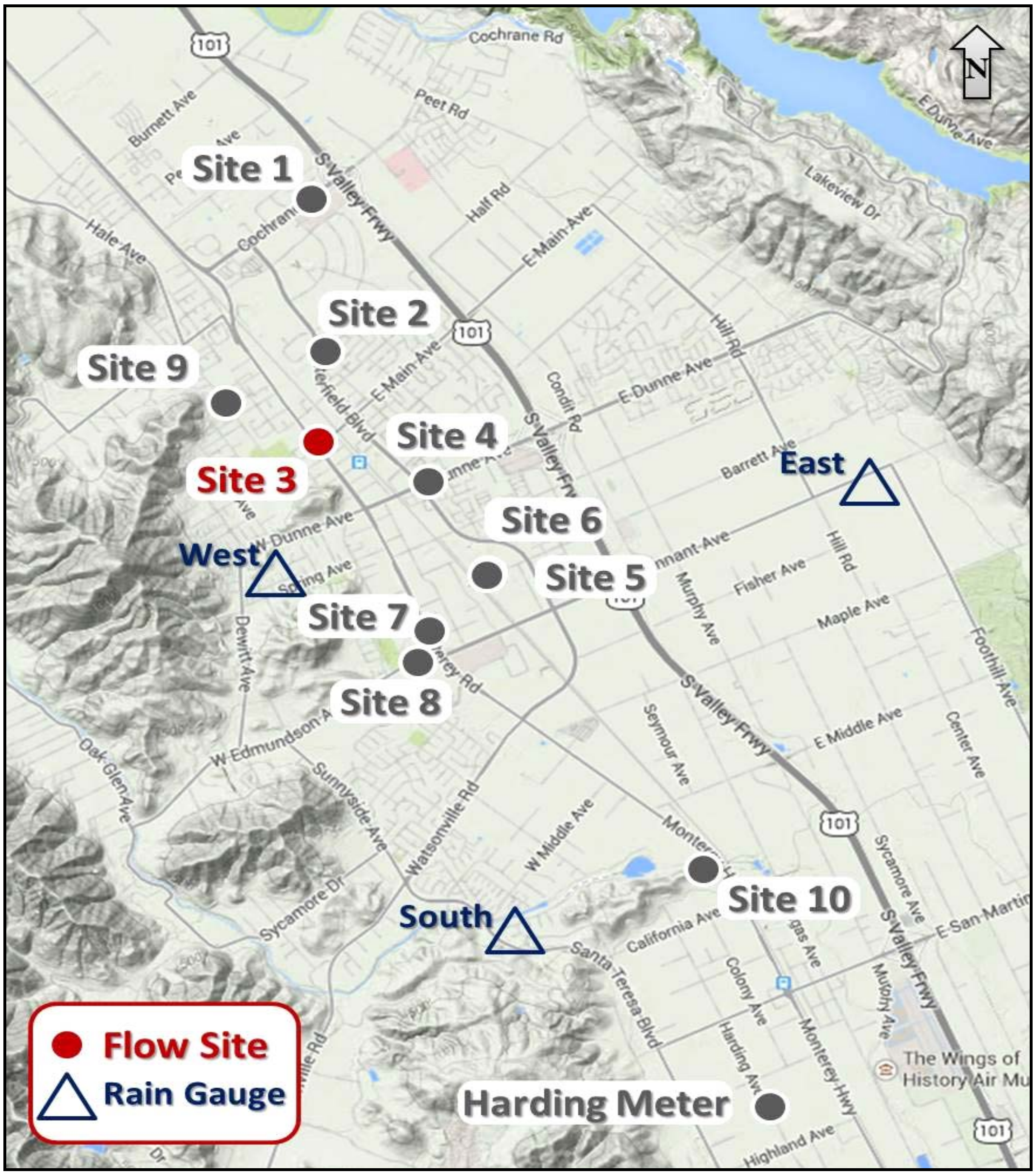
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 3

Location: Intersection of Main Avenue and Monterey Road

Data Summary Report

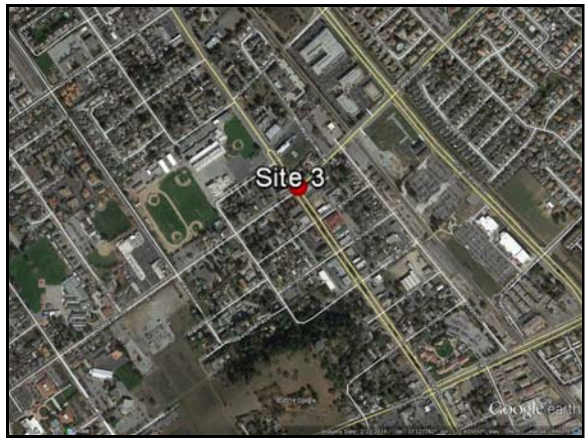


Vicinity Map: Site 3

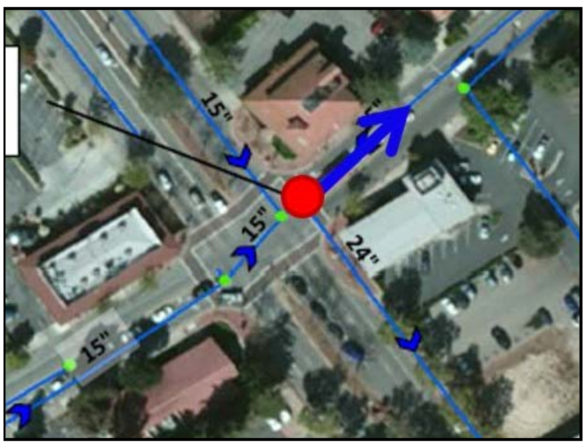
SITE 3

Site Information

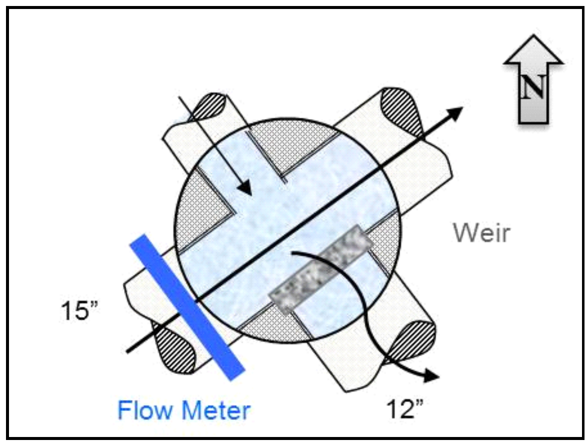
Location:	Intersection of Main Avenue and Monterey Road
Coordinates:	121.6544° W, 37.1305° N
Rim Elevation:	349 feet
Pipe Diameter:	15 inches
Baseline Flow:	0.315 mgd
Peak Measured Flow:	0.931 mgd



Satellite Map



Sanitary Map



Flow Sketch

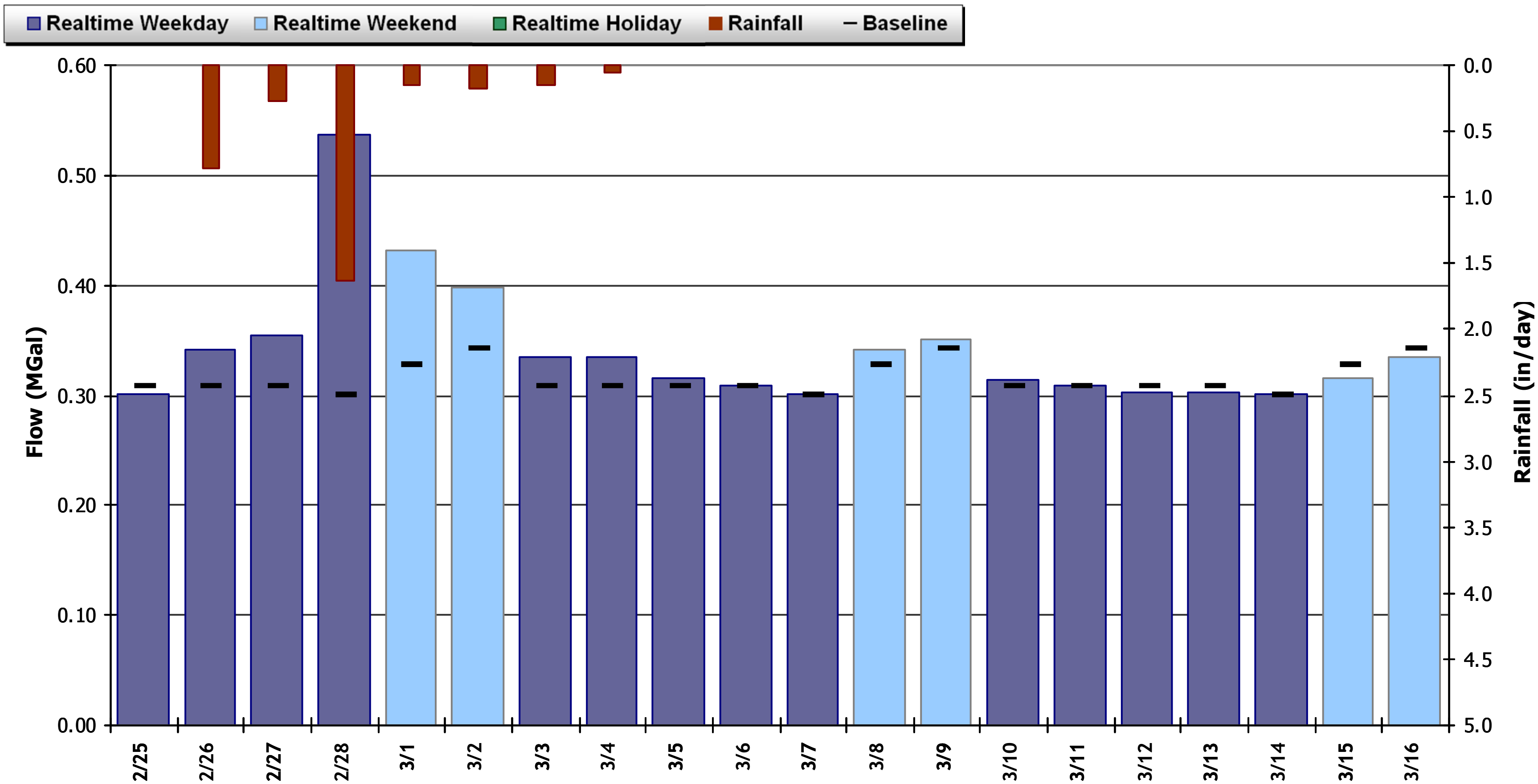
Street View

Plan View

SITE 3
Period Flow Summary: Daily Flow Totals

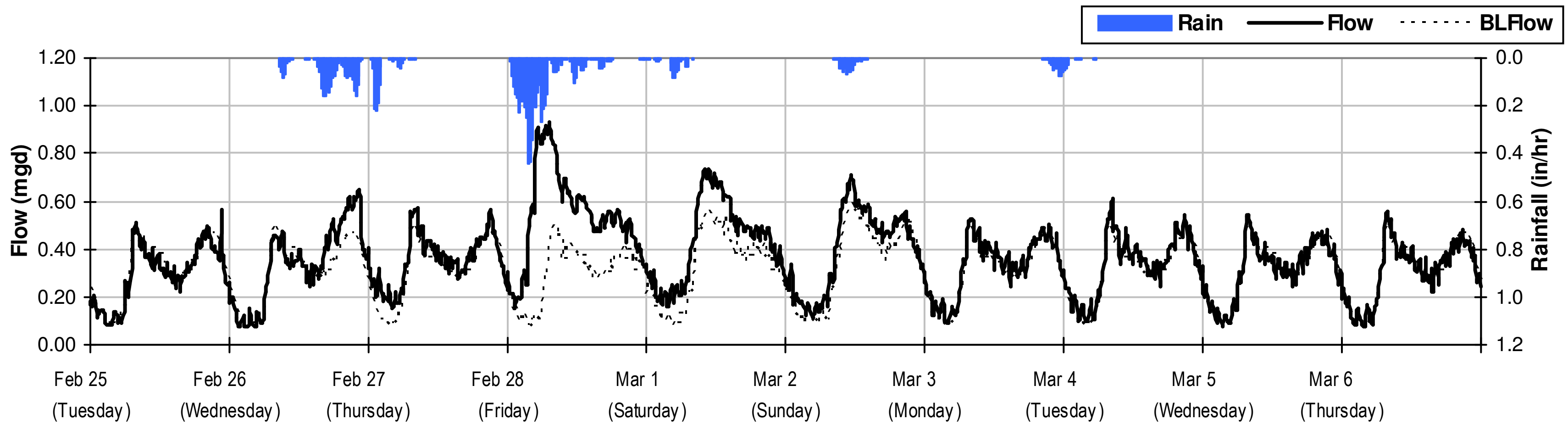
Avg Period Flow: 0.341 MGal **Peak Daily Flow: 0.538 MGal** **Min Daily Flow: 0.301 MGal**

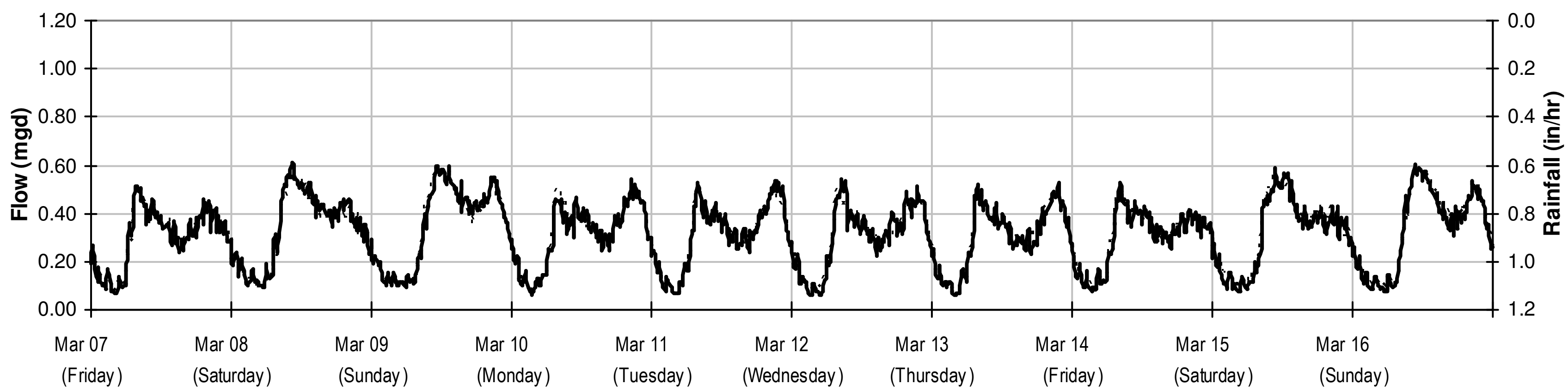
Total Period Rainfall: 3.20 inches



SITE 3
Flow Summary: 2/25/2014 to 3/17/2014

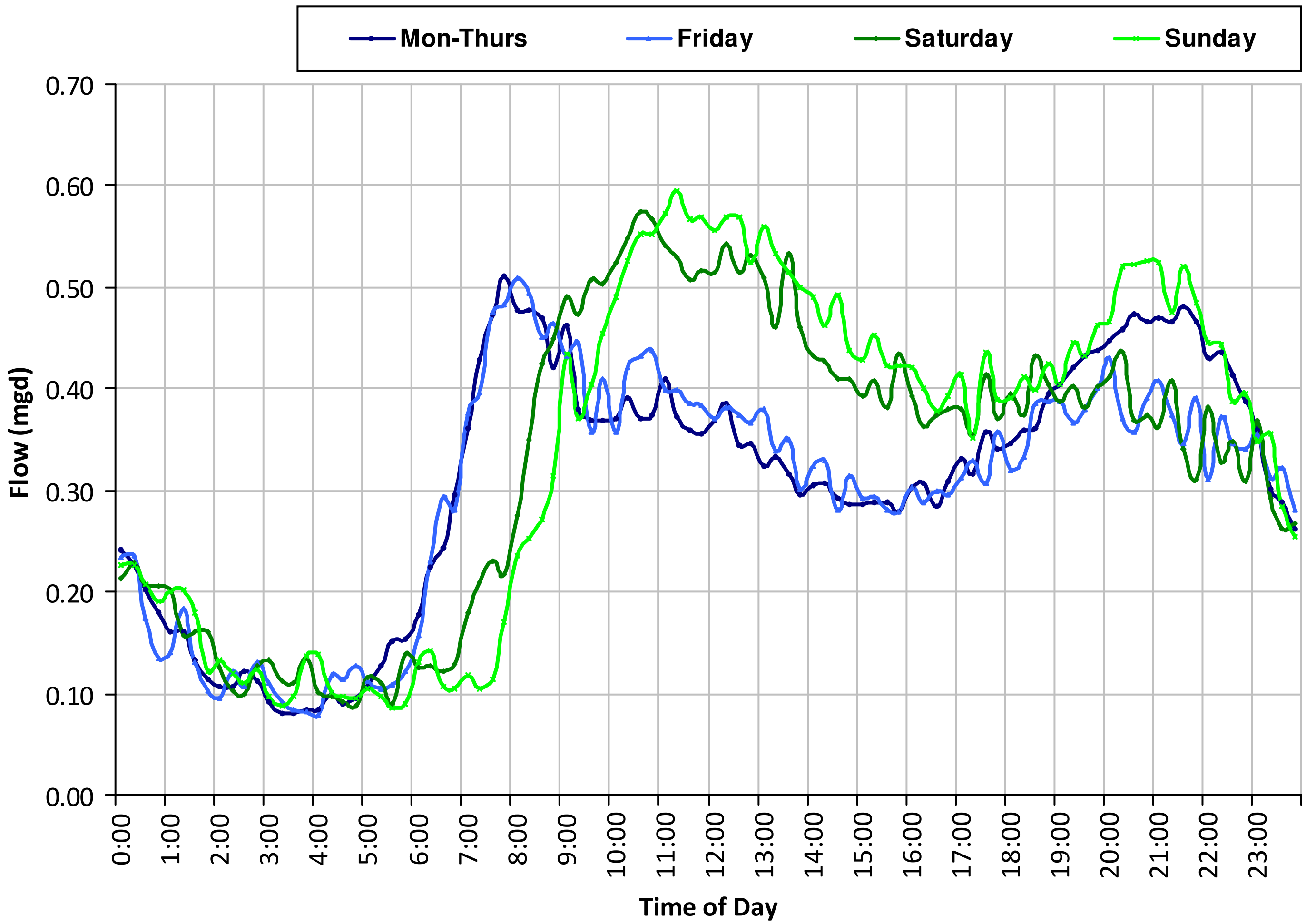
Total Period Rainfall: 3.20 inches Avg Flow: 0.341 mgd Peak Flow: 0.931 mgd Min Flow: 0.057 mgd

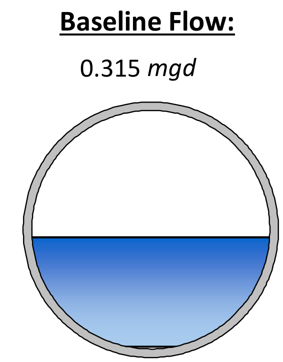




SITE 3

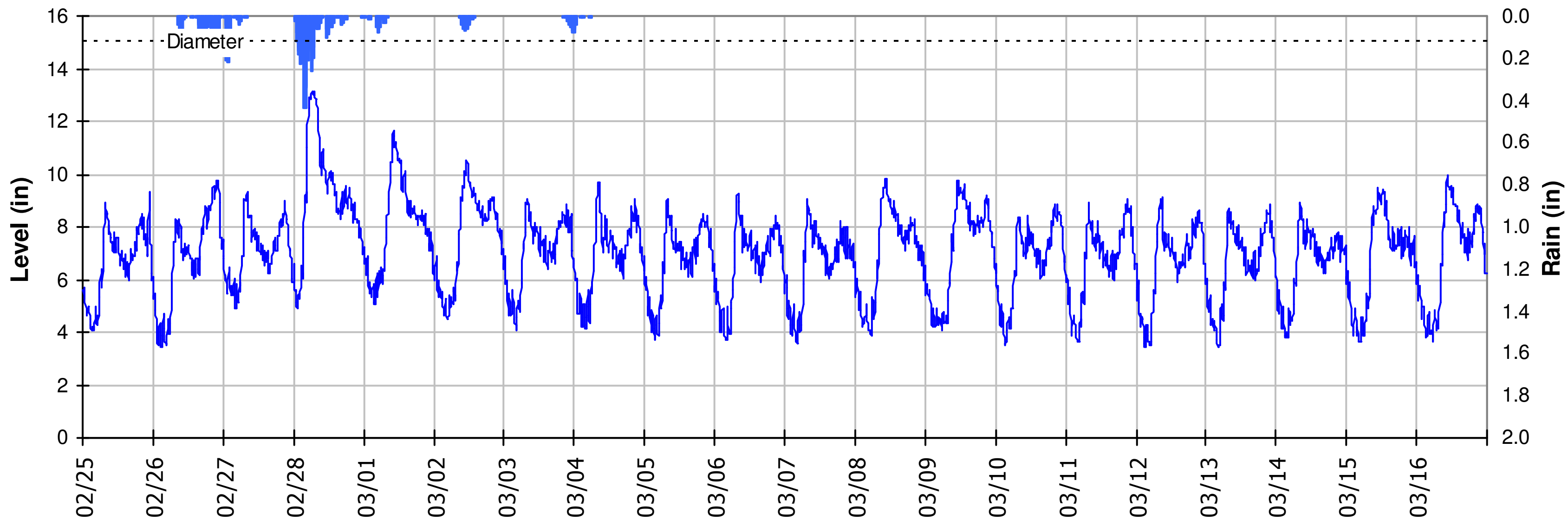
Baseline Flow Hydrographs

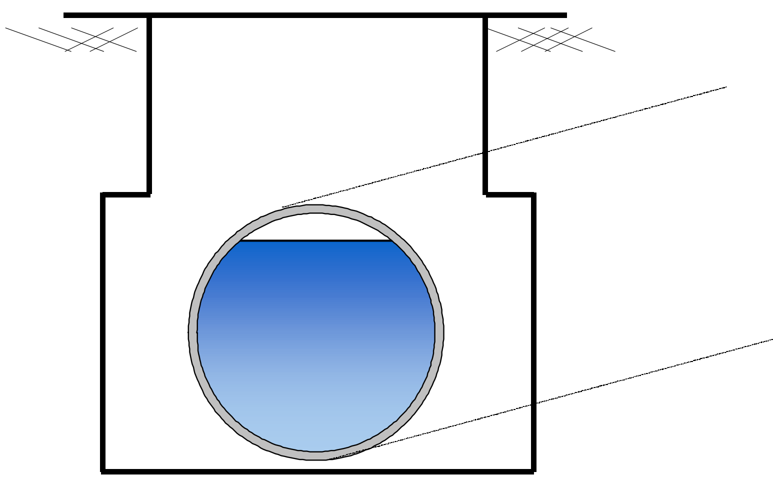




SITE 3
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





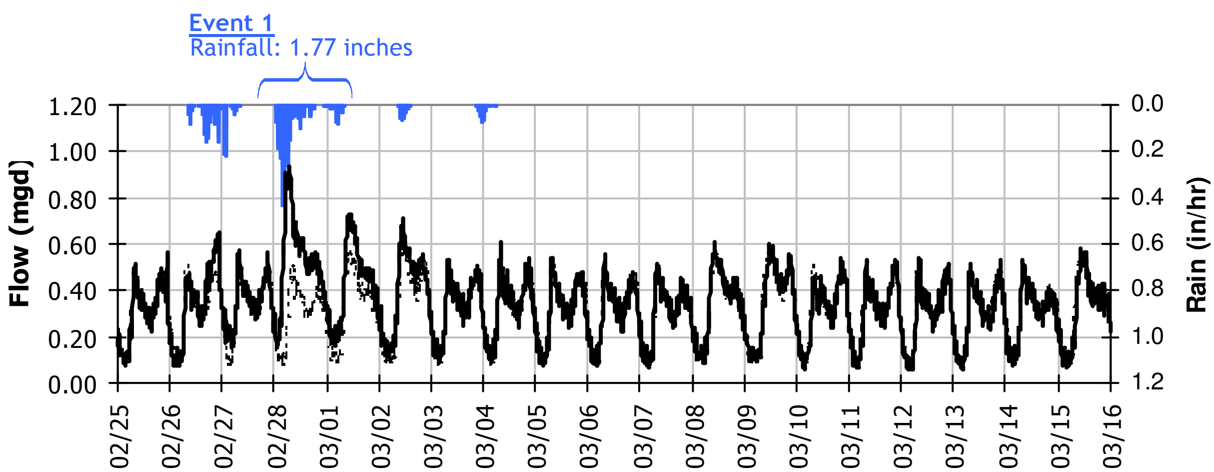
Pipe Diameter: 15 *inches*

Peak Measured Level: 13.1 *inches*

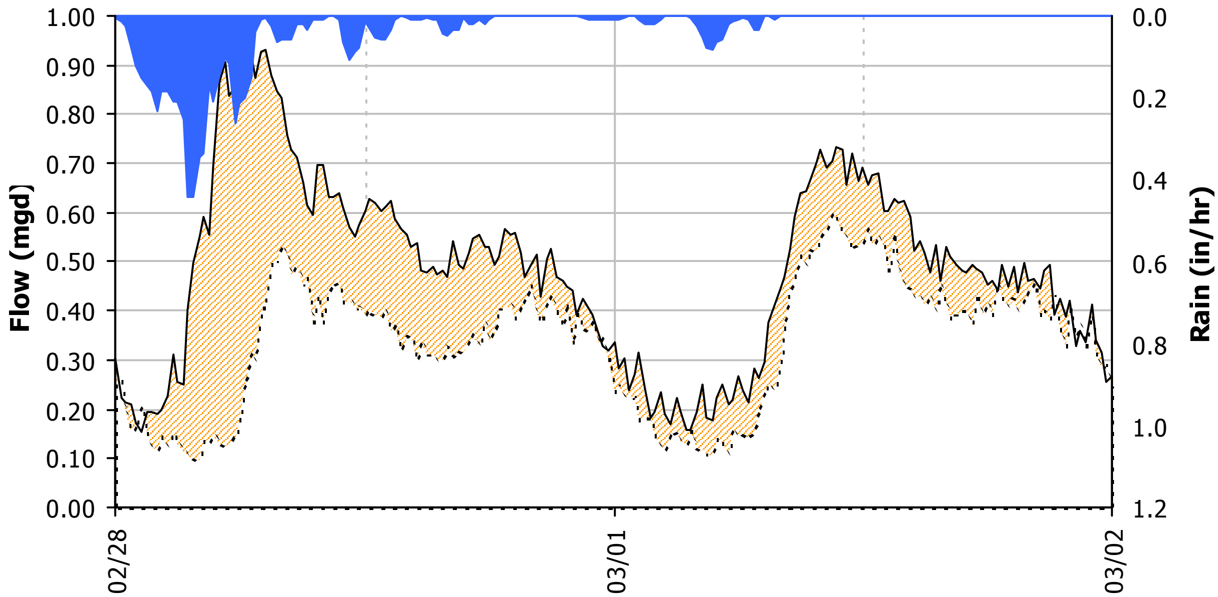
Peak d/D Ratio: 0.88

SITE 3
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



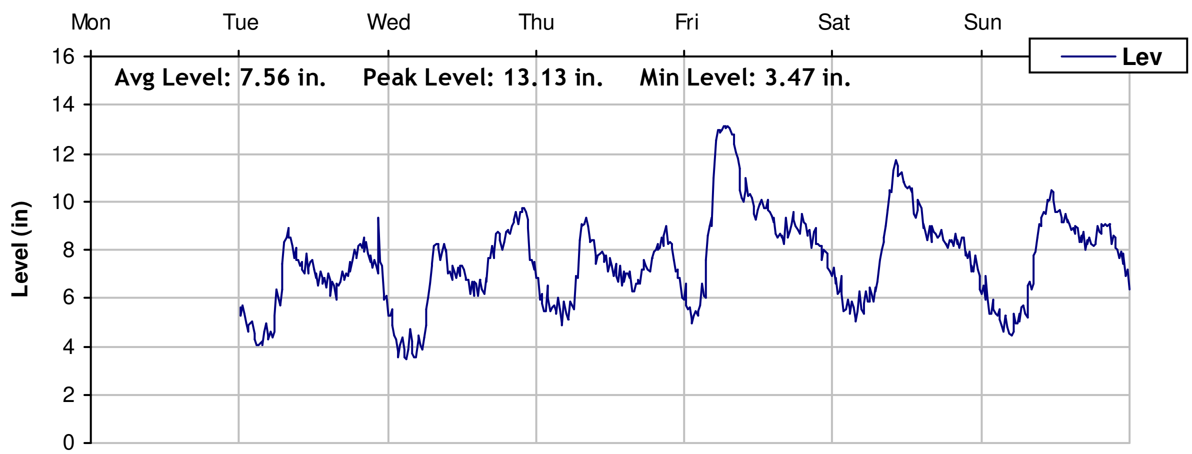
Event 1 Detail Graph

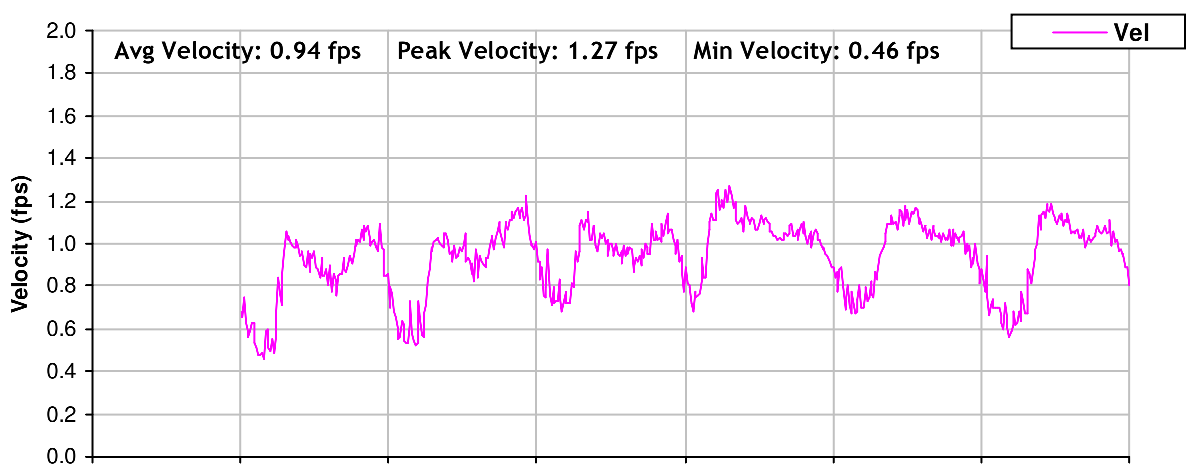


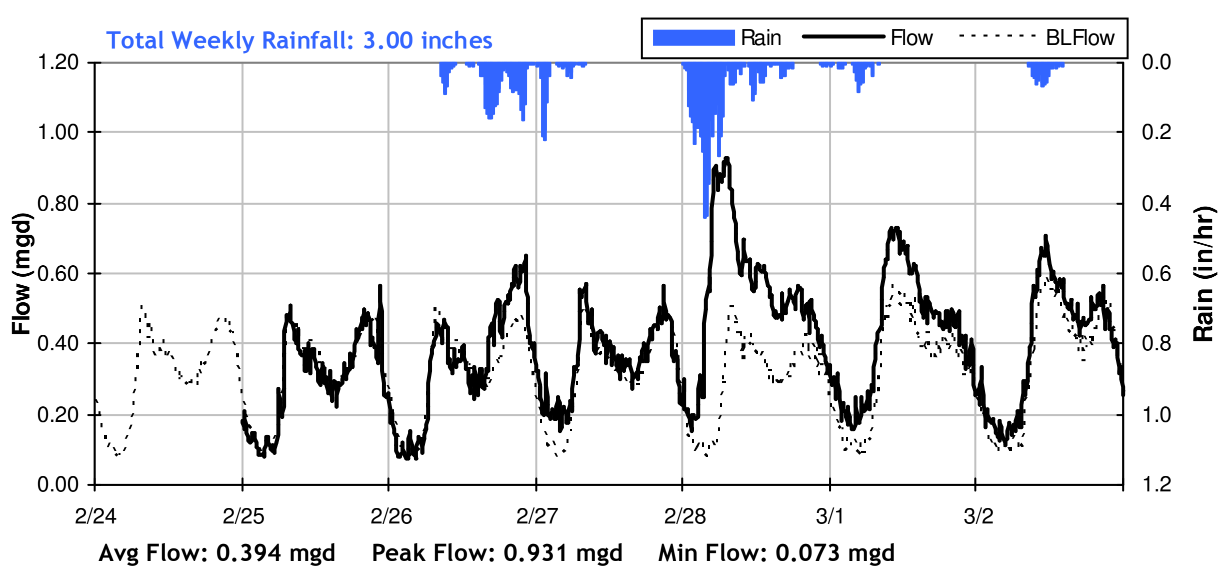
Storm Event I/I Analysis (Rain = 1.77 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.93 *mgd*	**Peak I/I Rate:**	0.78 *mgd*
PF:	2.95	**Total I/I:**	299,000 *gallons*

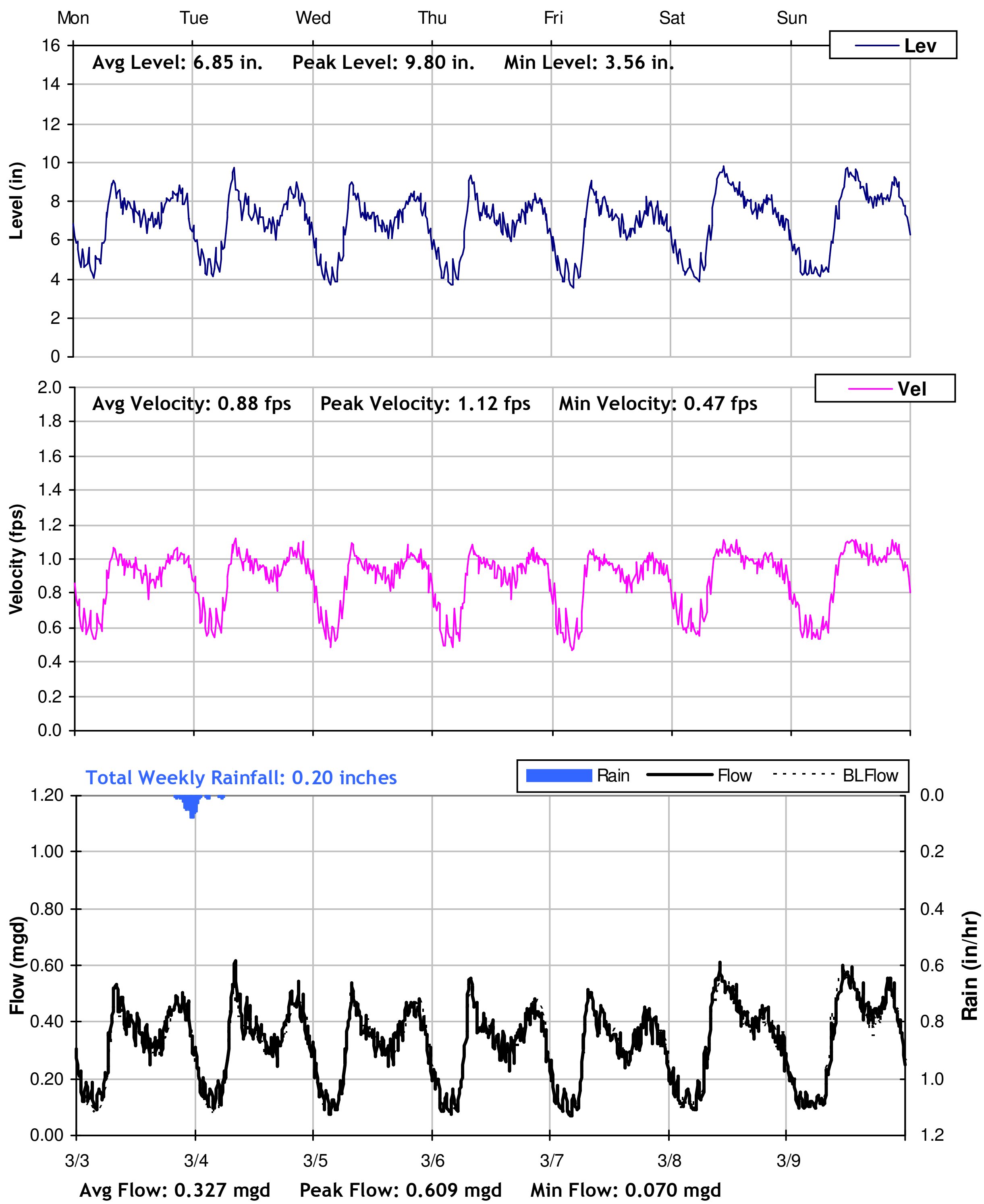
SITE 3
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



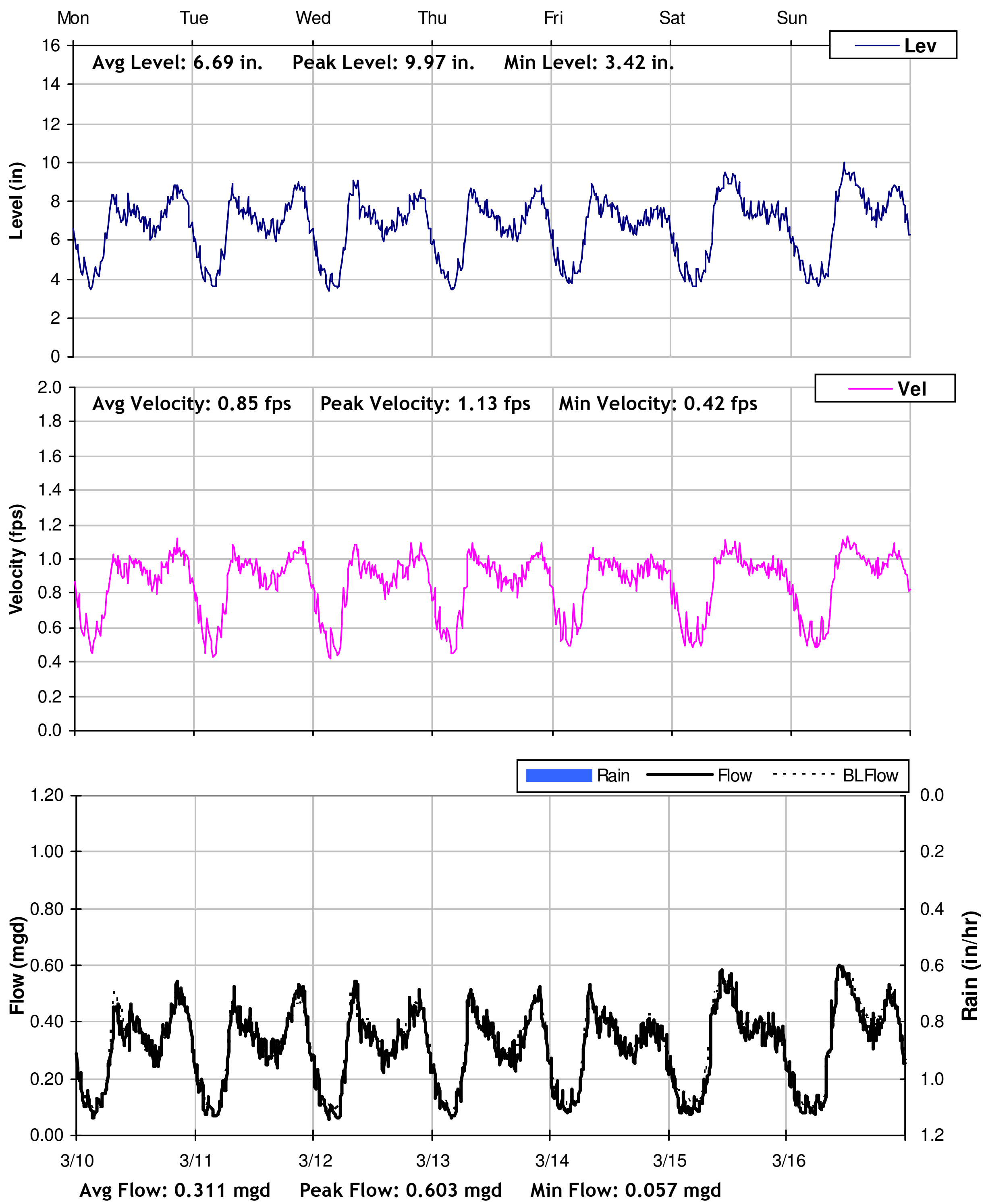




SITE 3
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



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



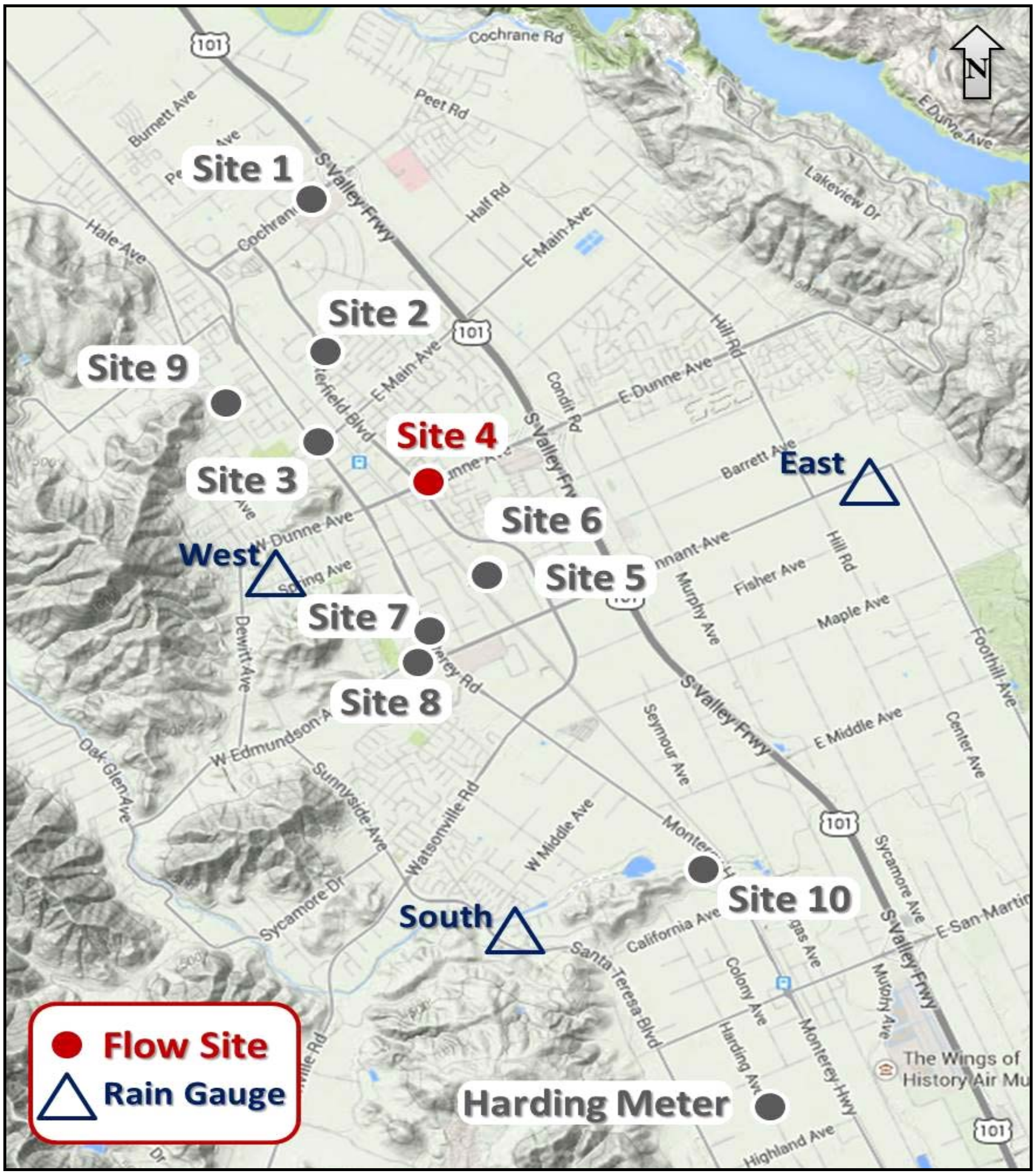
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 4

Location: E. Dunne Avenue, just east of Butterfield Boulevard

Data Summary Report



Vicinity Map: Site 4

SITE 4

Site Information

Location: E. Dunne Avenue, just east of Butterfield Boulevard

Coordinates: 121.6446° W, 37.1274° N

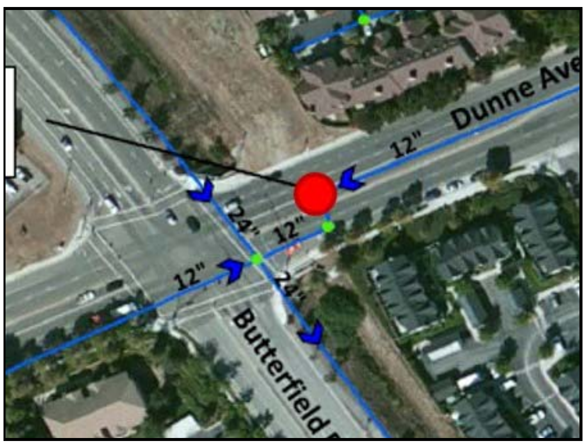
Rim Elevation: 350 feet

Pipe Diameter: 12 inches

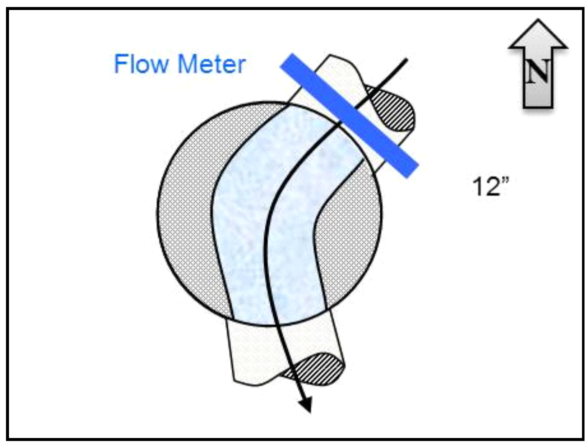
Baseline Flow: 0.215 mgd

Peak Measured Flow: 0.534 mgd

Satellite Map



Sanitary Map



Flow Sketch

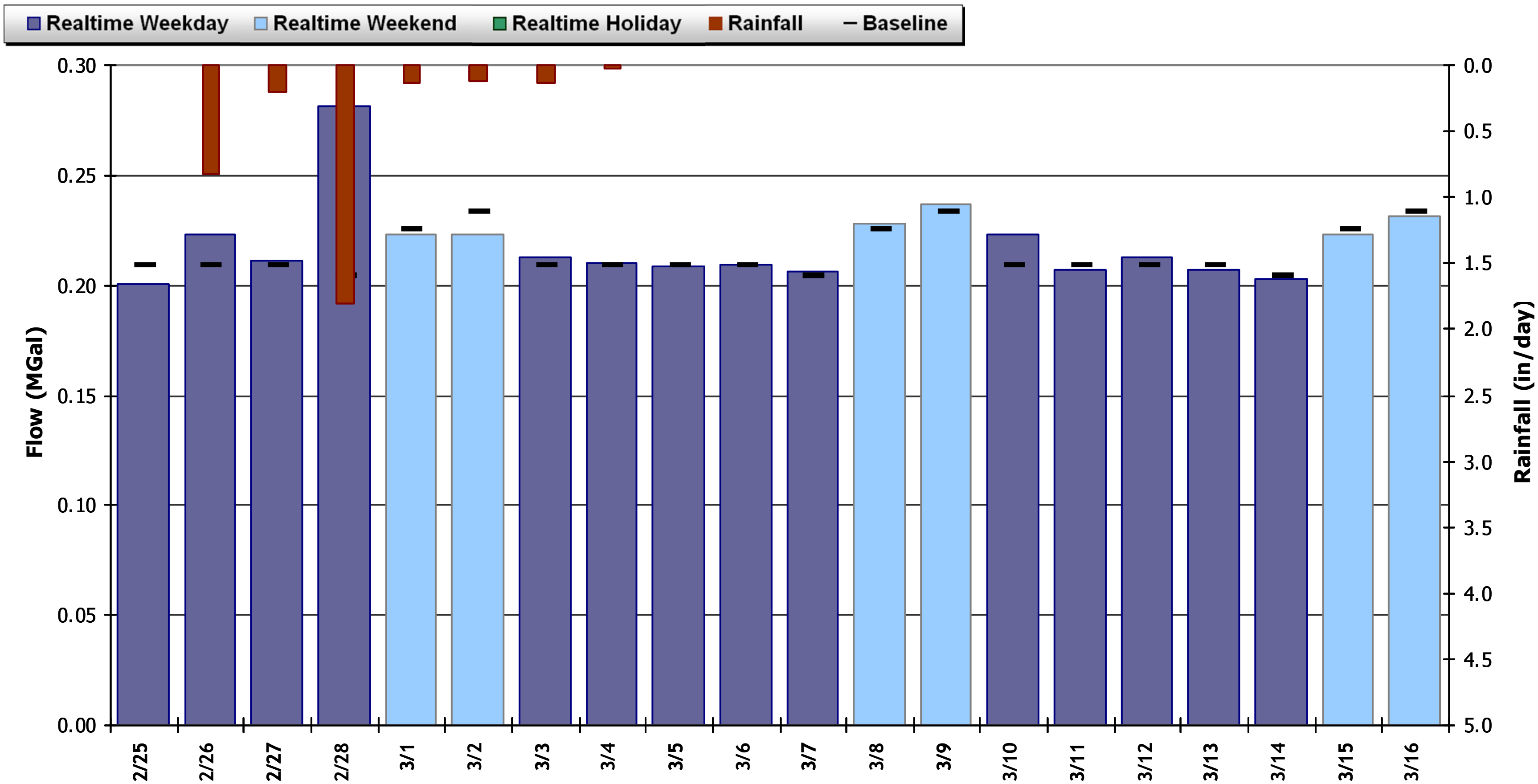
Street View

Plan View

SITE 4
Period Flow Summary: Daily Flow Totals

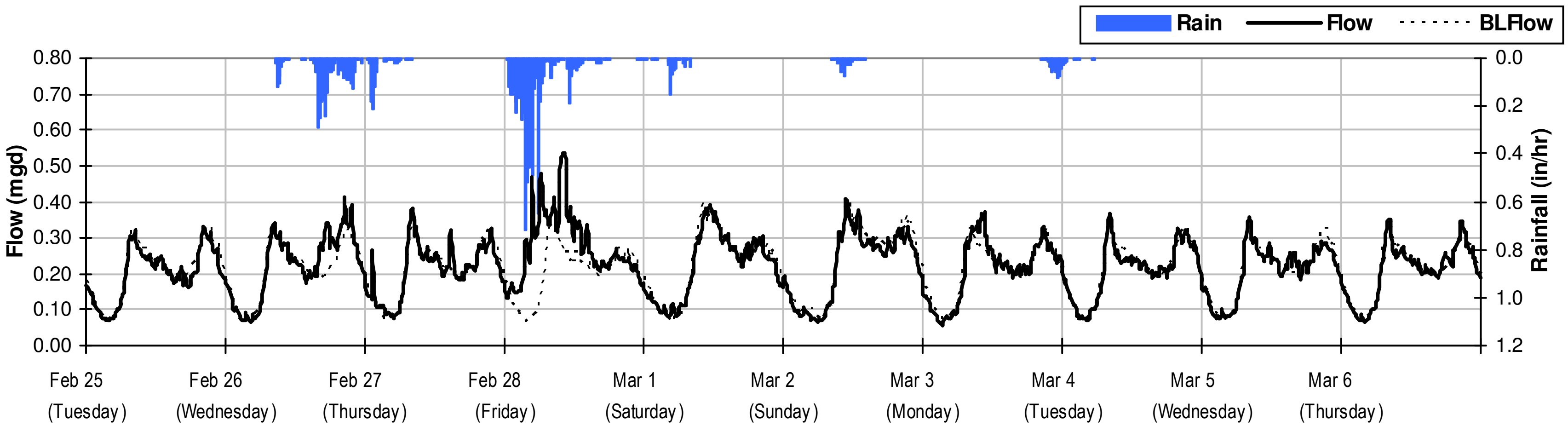
Avg Period Flow: 0.219 MGal **Peak Daily Flow: 0.281 MGal** **Min Daily Flow: 0.201 MGal**

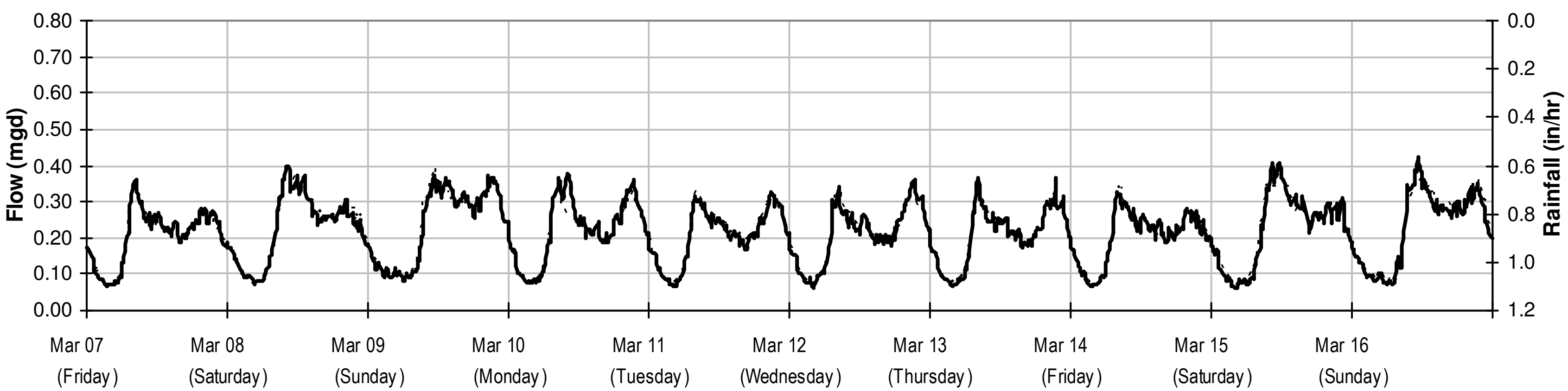
Total Period Rainfall: 3.25 inches



SITE 4
Flow Summary: 2/25/2014 to 3/17/2014

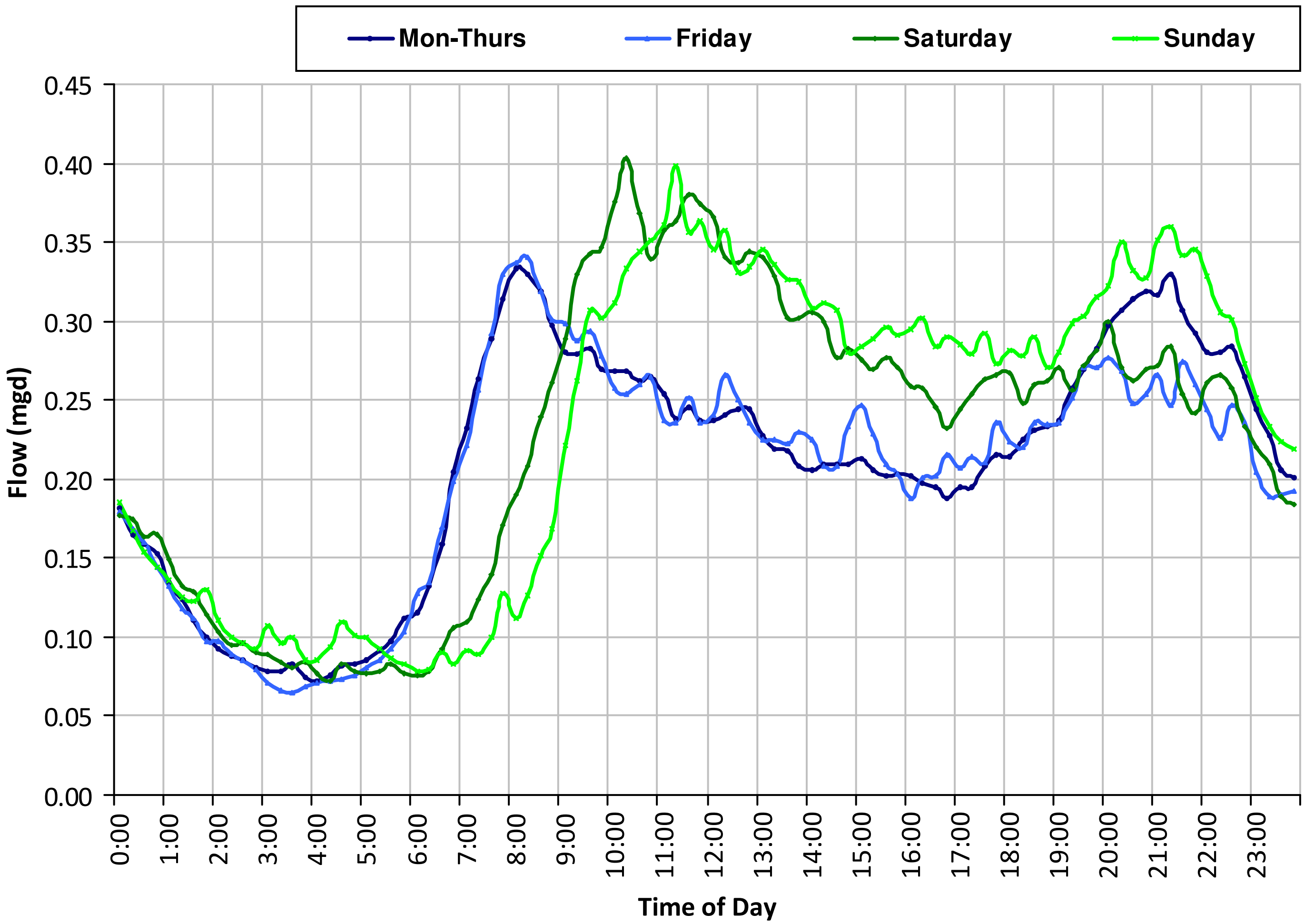
Total Period Rainfall: 3.25 inches   Avg Flow: 0.219 mgd   Peak Flow: 0.534 mgd   Min Flow: 0.057 mgd





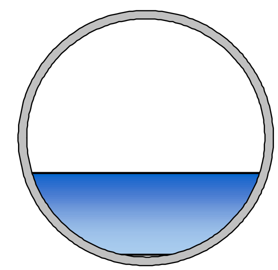
SITE 4

Baseline Flow Hydrographs



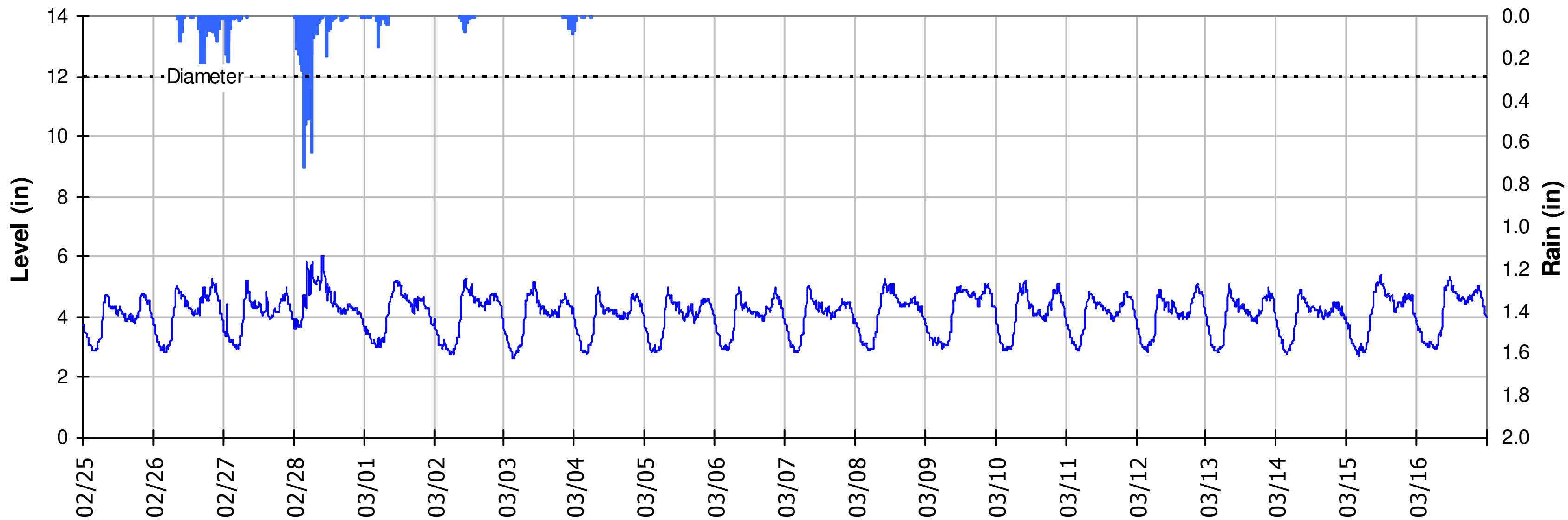
Baseline Flow:

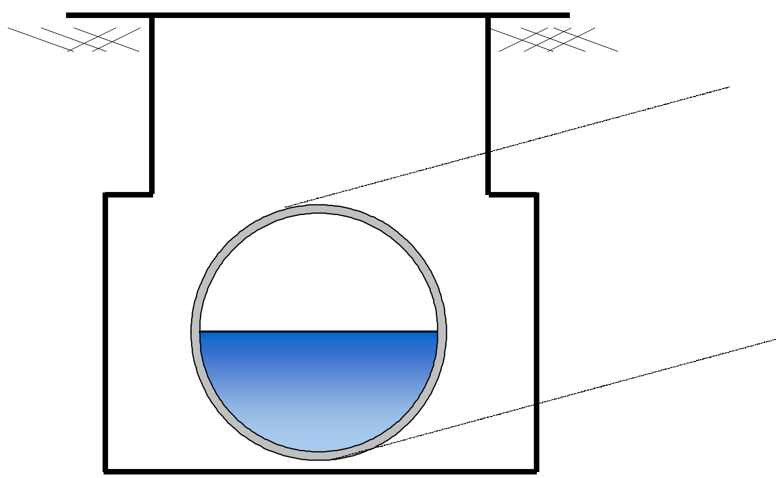
0.215 *mgd*



SITE 4
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





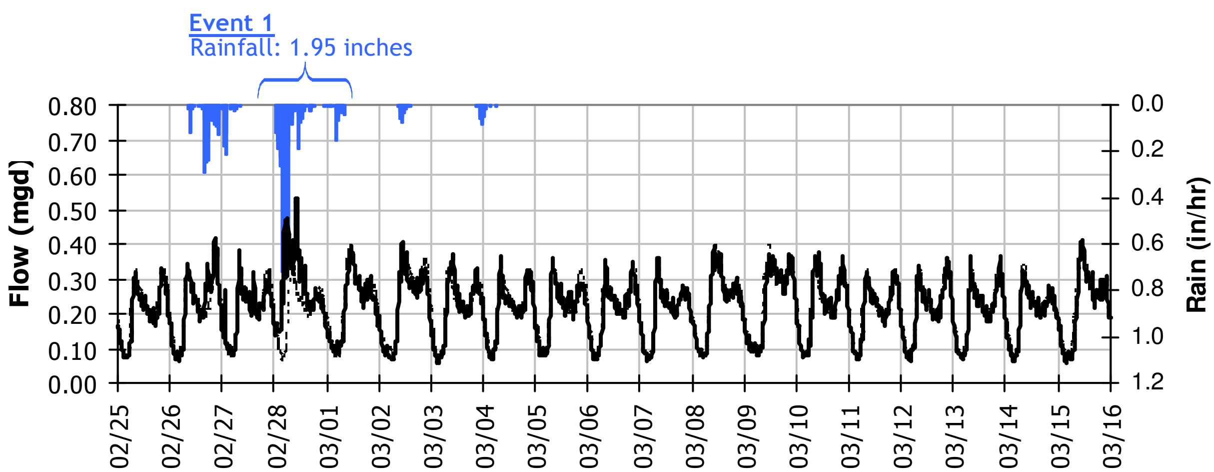
Pipe Diameter: 12 *inches*

Peak Measured Level: 6.04 *inches*

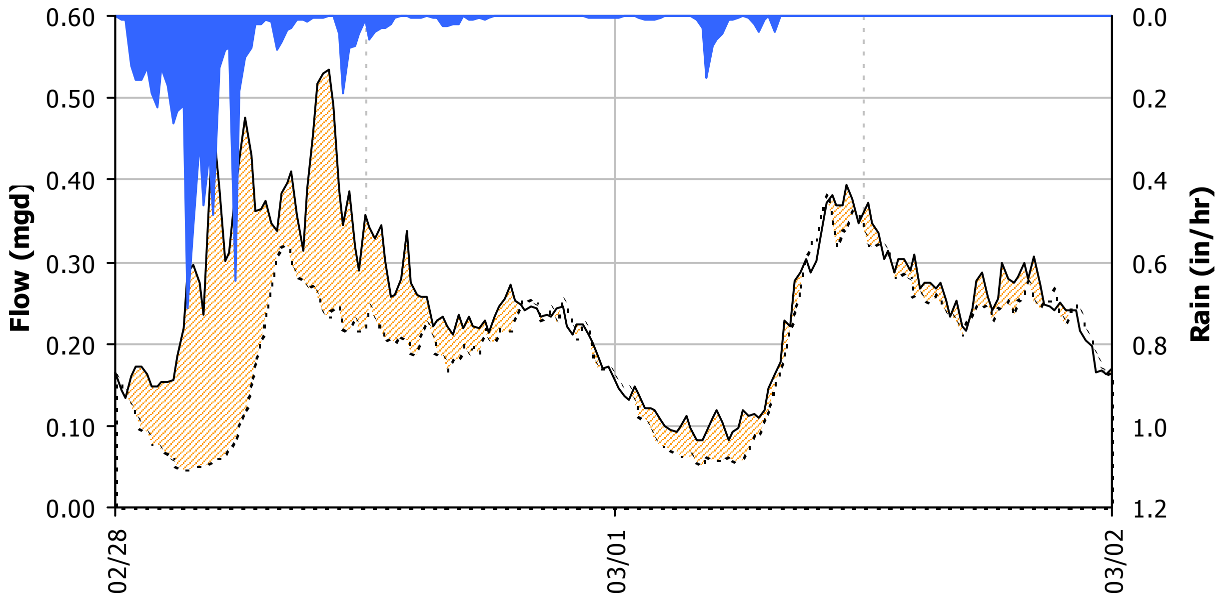
Peak d/D Ratio: 0.50

SITE 4
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



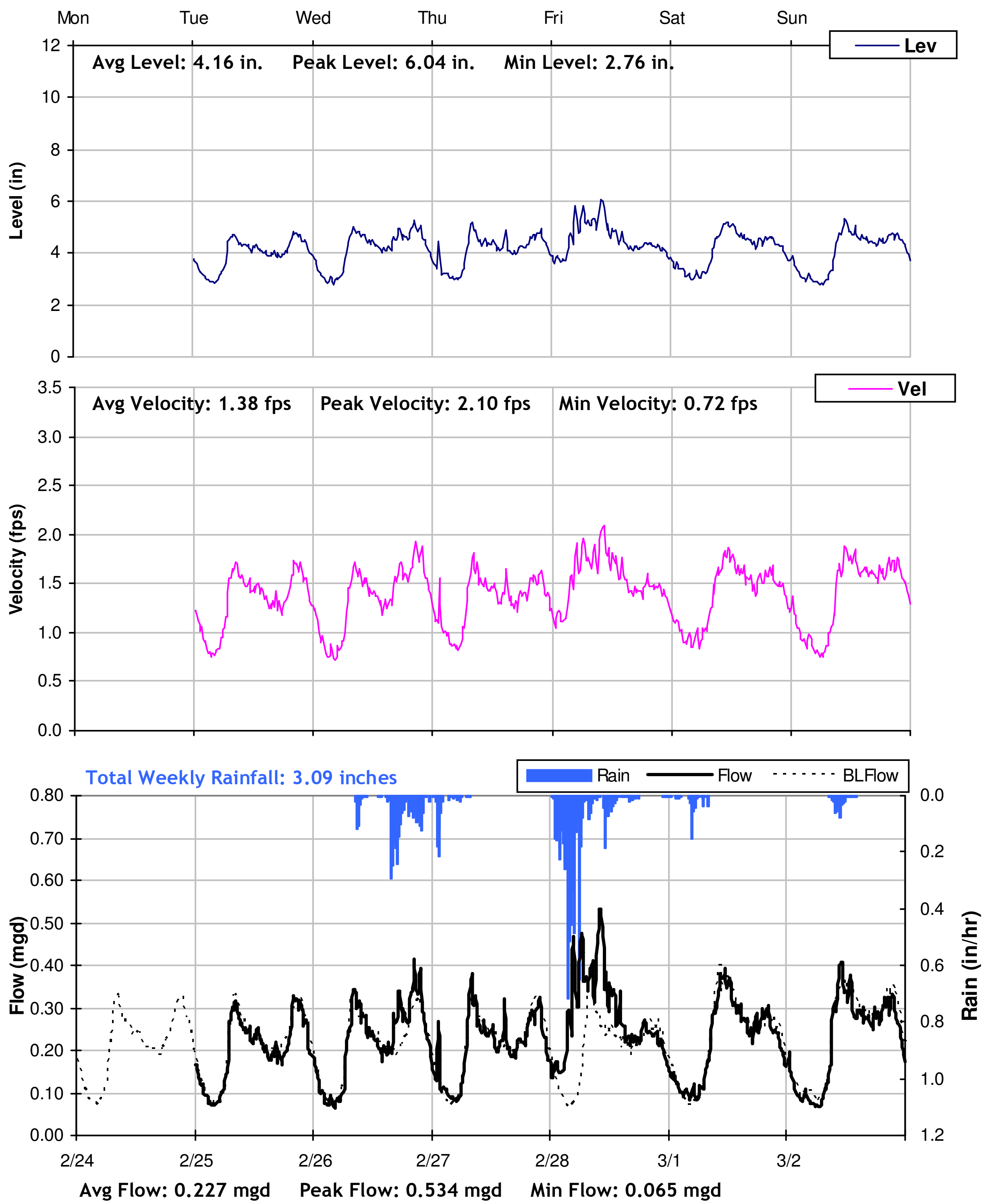
Event 1 Detail Graph



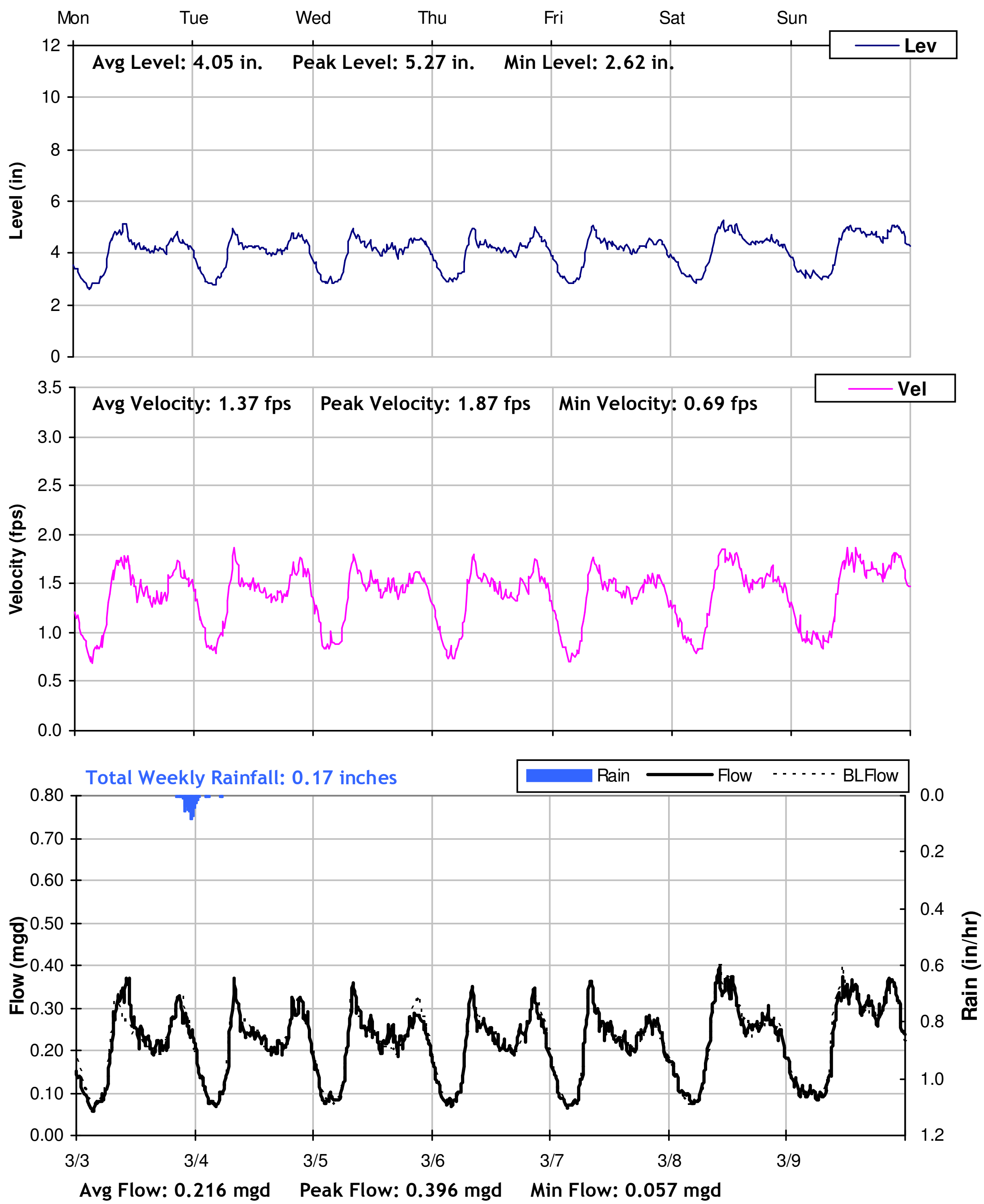
Storm Event I/I Analysis (Rain = 1.95 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.53 *mgd*	**Peak I/I Rate:**	0.41 *mgd*
PF:	2.49	**Total I/I:**	114,000 *gallons*

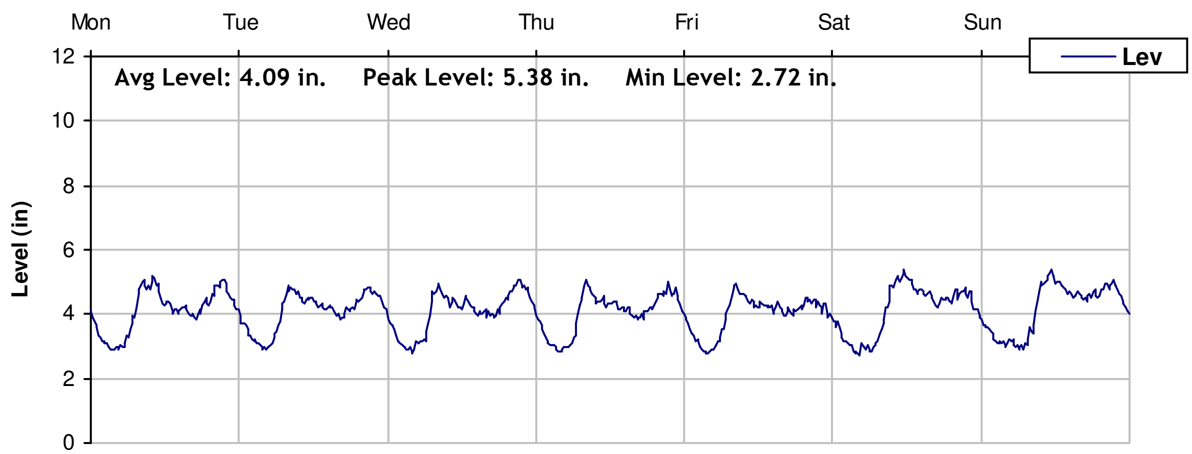
SITE 4
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014

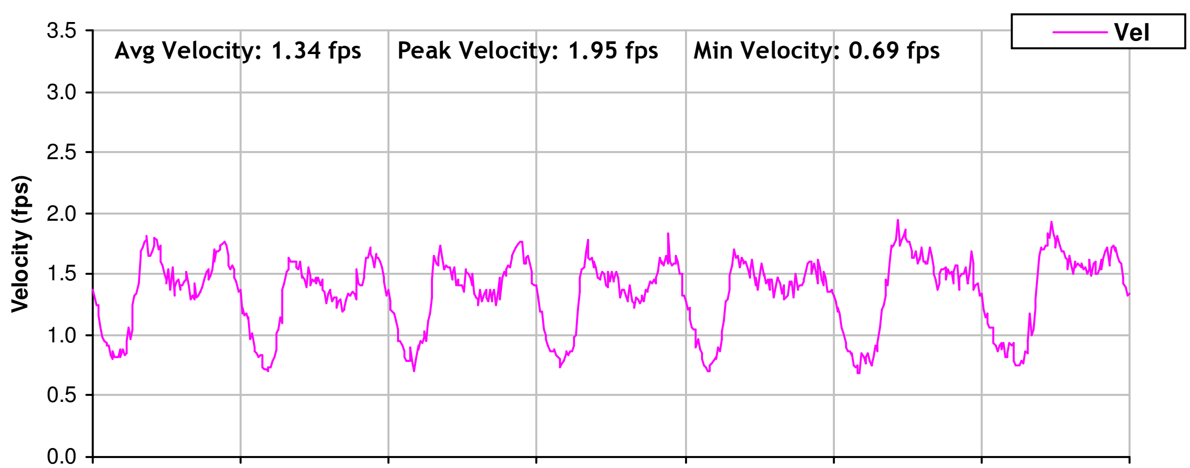


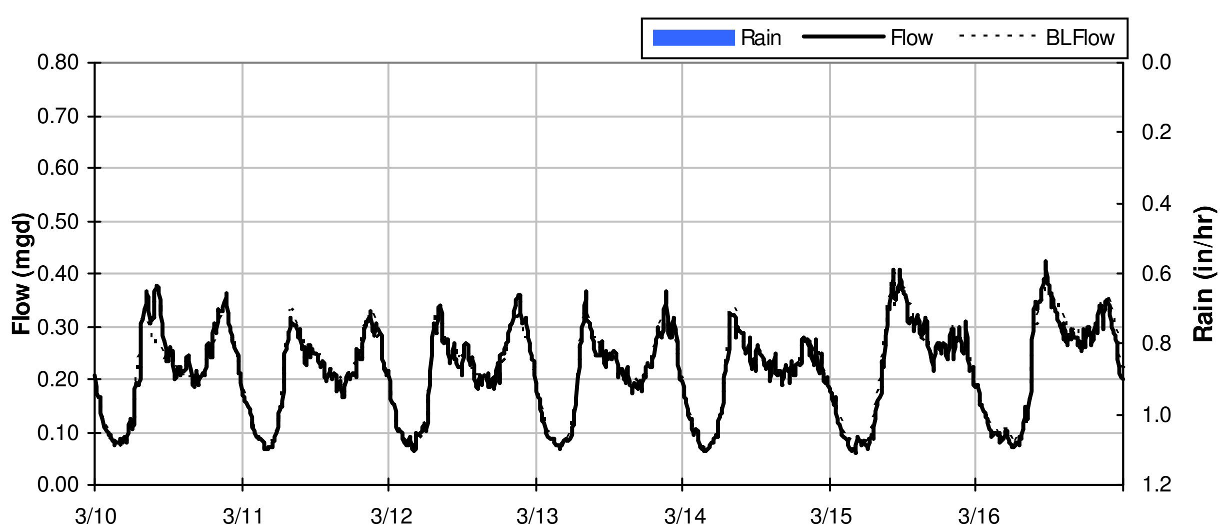
SITE 4
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



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







Avg Flow: 0.215 mgd   Peak Flow: 0.424 mgd   Min Flow: 0.060 mgd

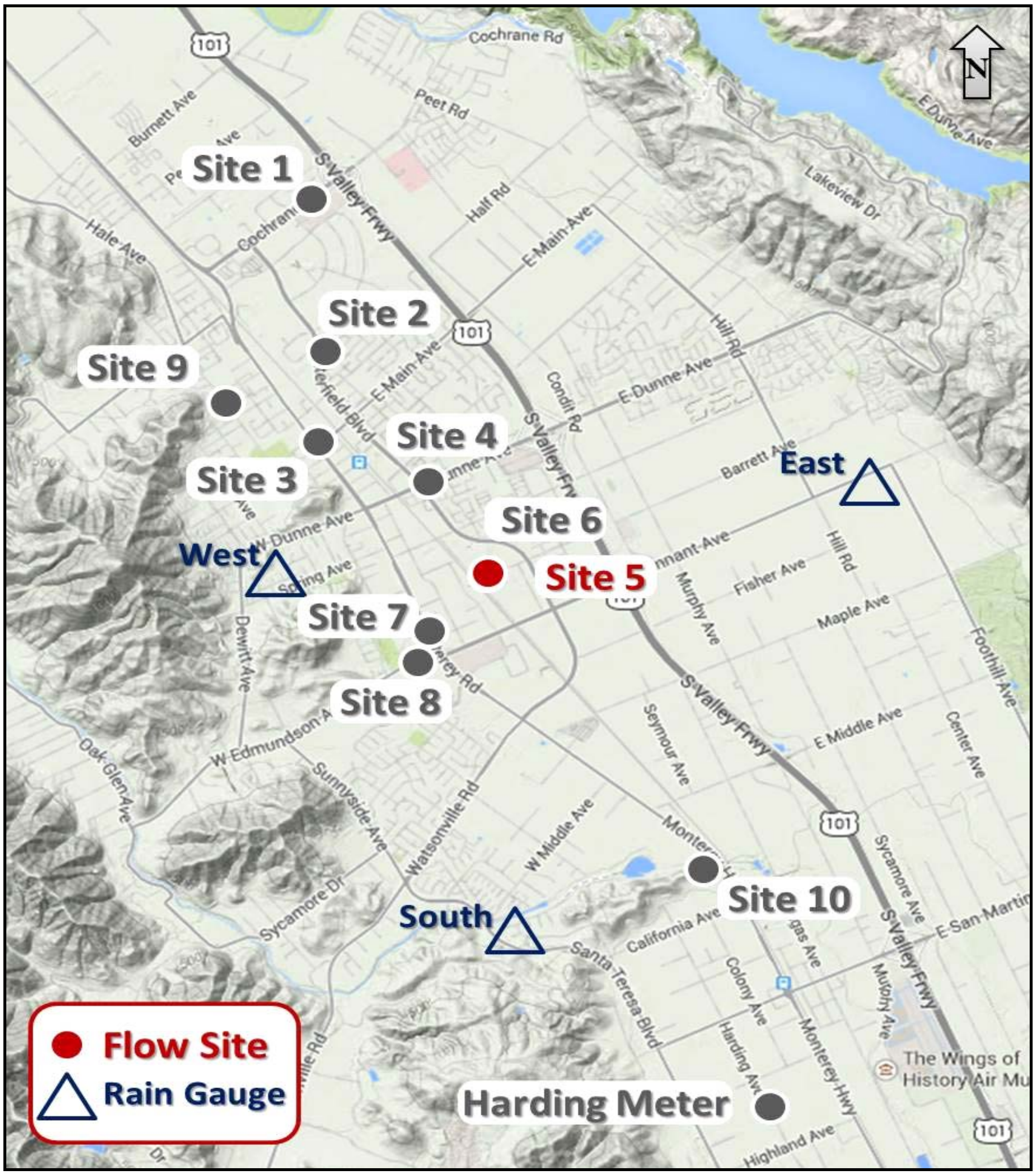
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 5

Location: Intersection of Barrett Avenue and Railroad Avenue

Data Summary Report



Vicinity Map: Site 5

SITE 5

Site Information

Location: Intersection of Barrett Avenue and Railroad Avenue

Coordinates: 121.6396° W, 37.1192° N

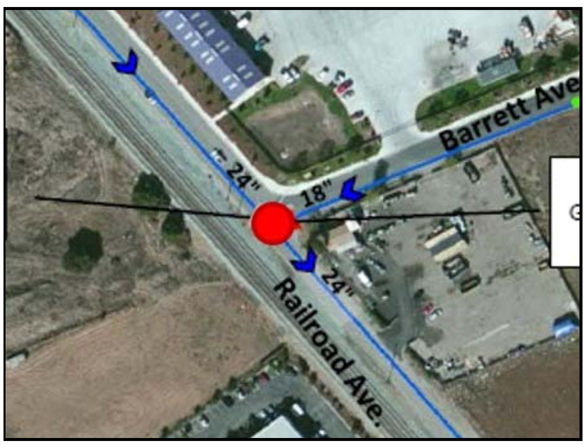
Rim Elevation: 338 feet

Pipe Diameter: 24 inches

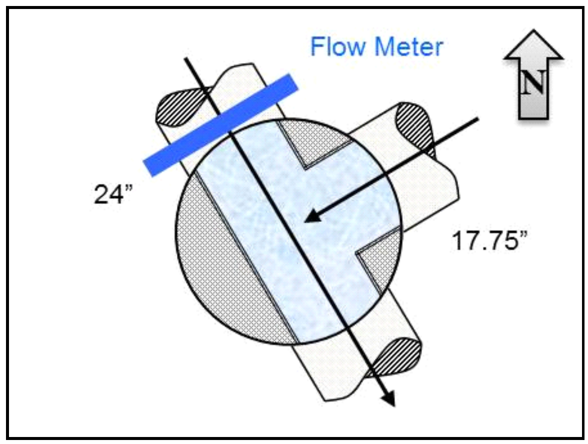
Baseline Flow: 1.155 mgd

Peak Measured Flow: 2.291 mgd

Satellite Map



Sanitary Map



Flow Sketch

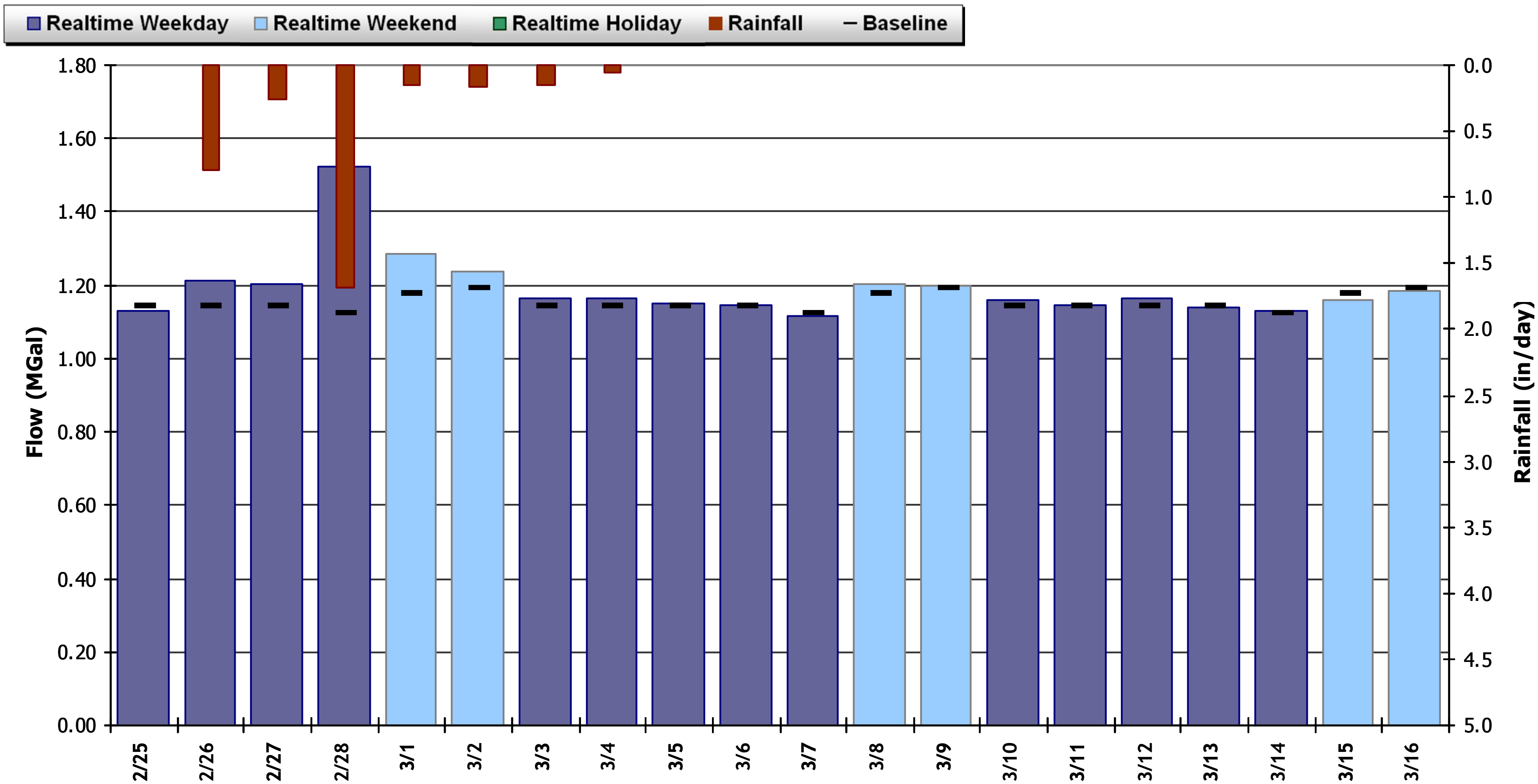
Street View

Plan View

SITE 5
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 1.191 MGal **Peak Daily Flow: 1.525 MGal** **Min Daily Flow: 1.116 MGal**

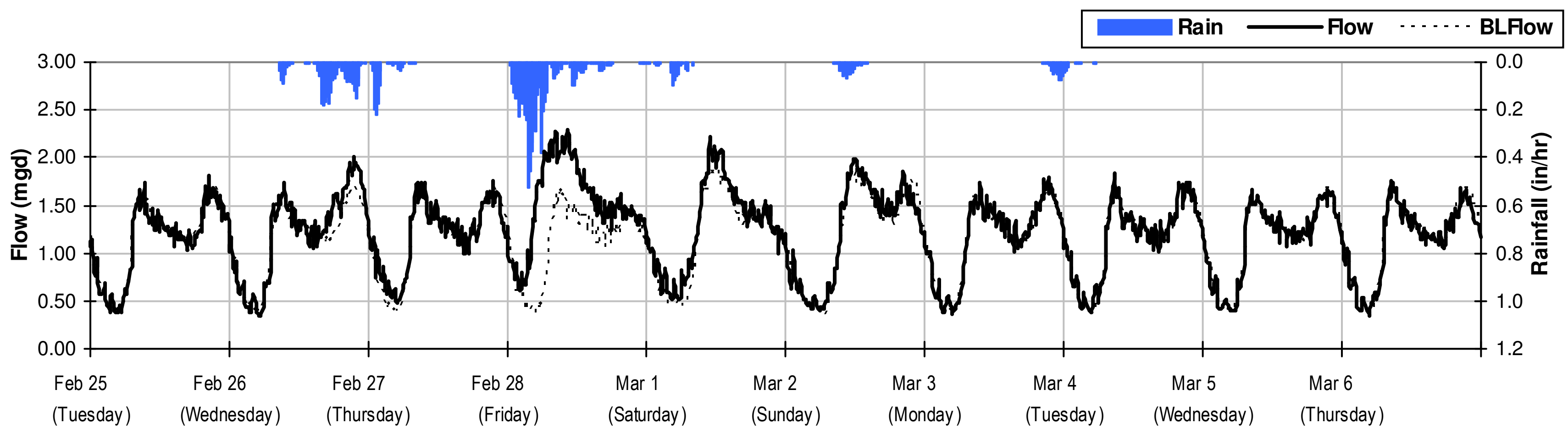
Total Period Rainfall: 3.22 inches

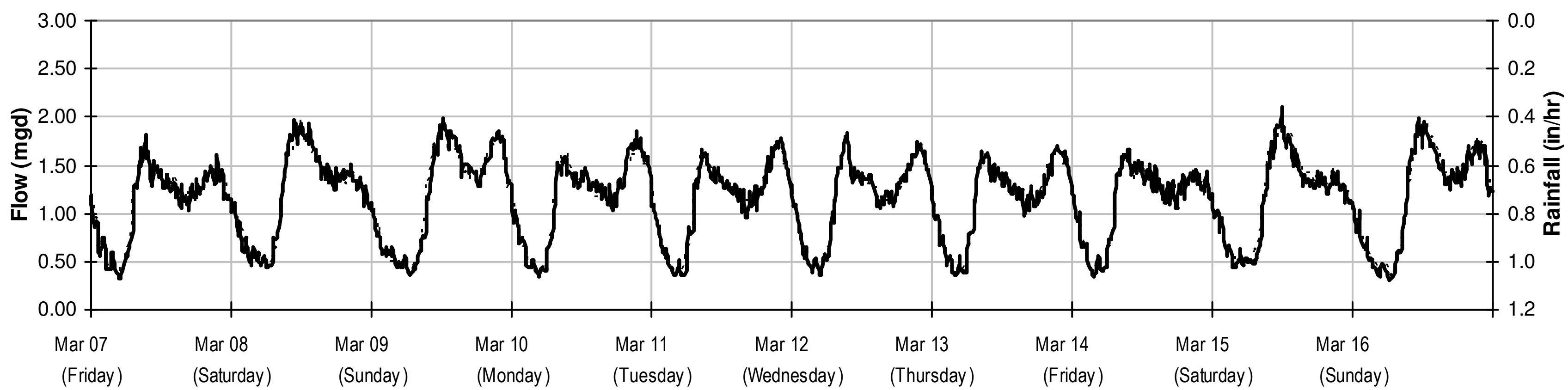


SITE 5

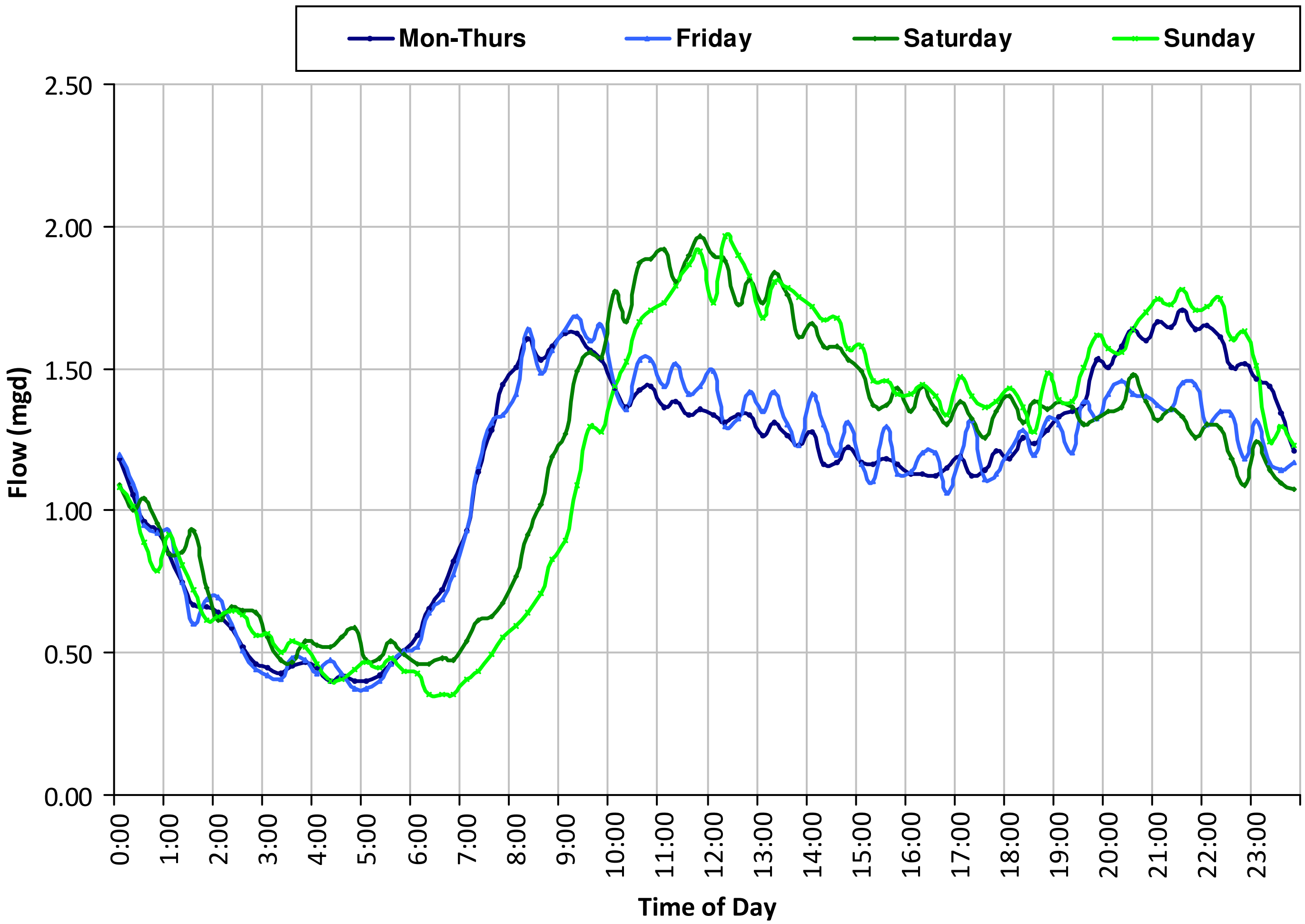
Flow Summary: 2/25/2014 to 3/17/2014

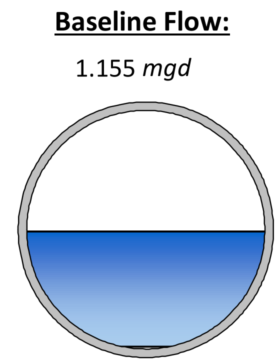
Total Period Rainfall: 3.22 inches   Avg Flow: 1.191 mgd   Peak Flow: 2.291 mgd   Min Flow: 0.303 mgd





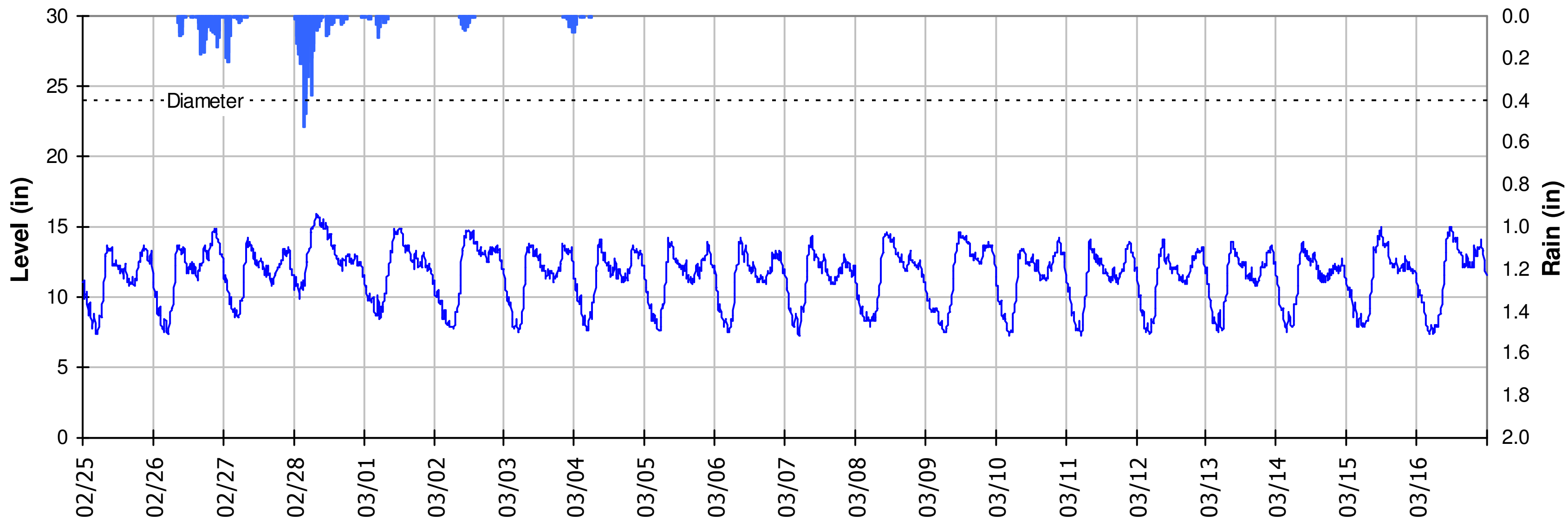
SITE 5
Baseline Flow Hydrographs

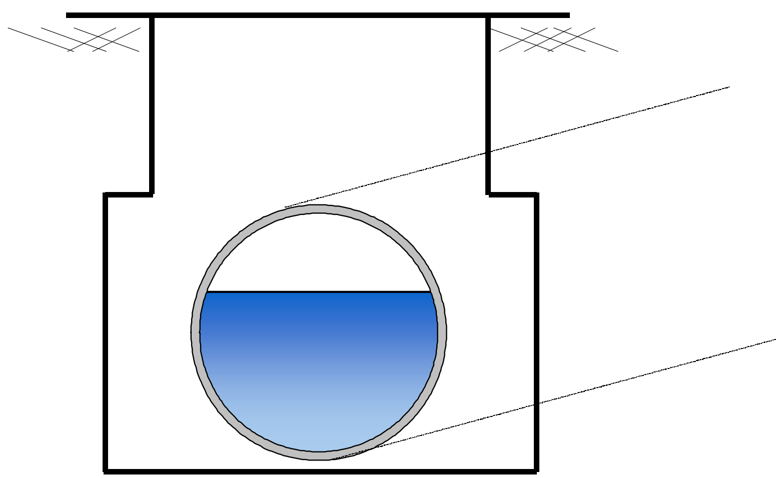




SITE 5
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





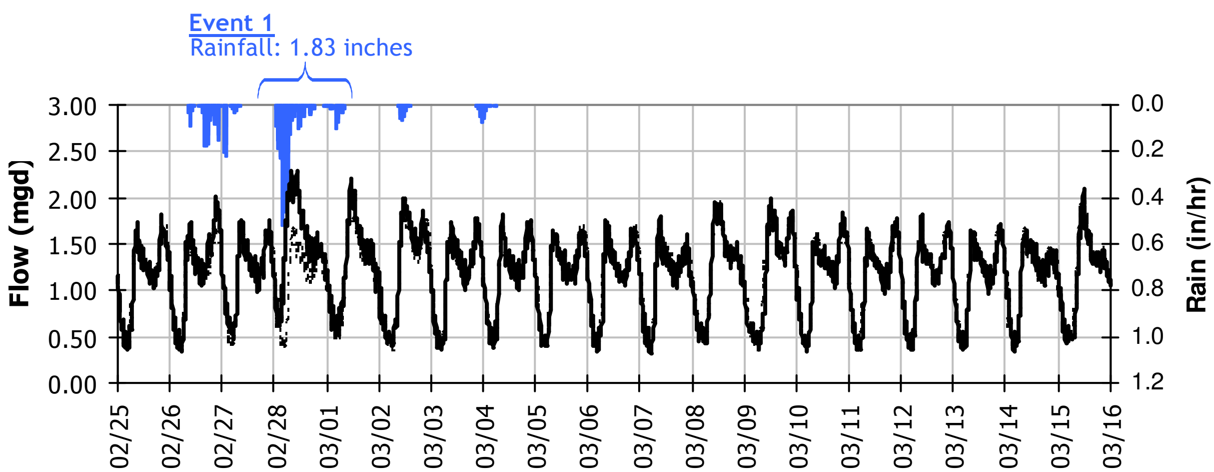
Pipe Diameter: 24 *inches*

Peak Measured Level: 16 *inches*

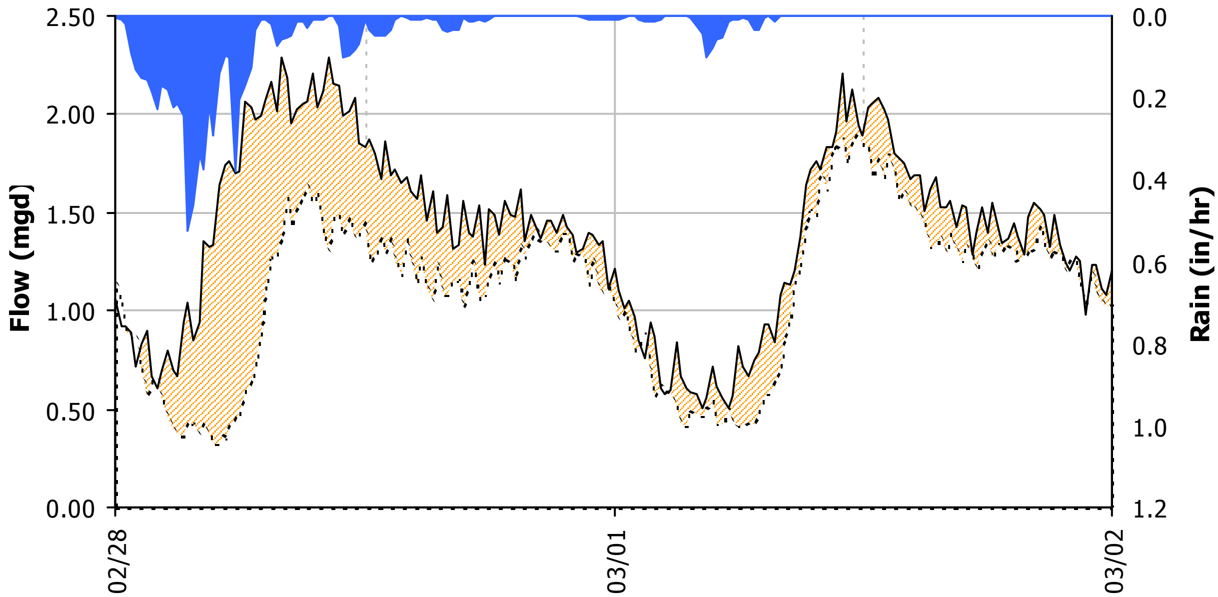
Peak d/D Ratio: 0.66

SITE 5
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



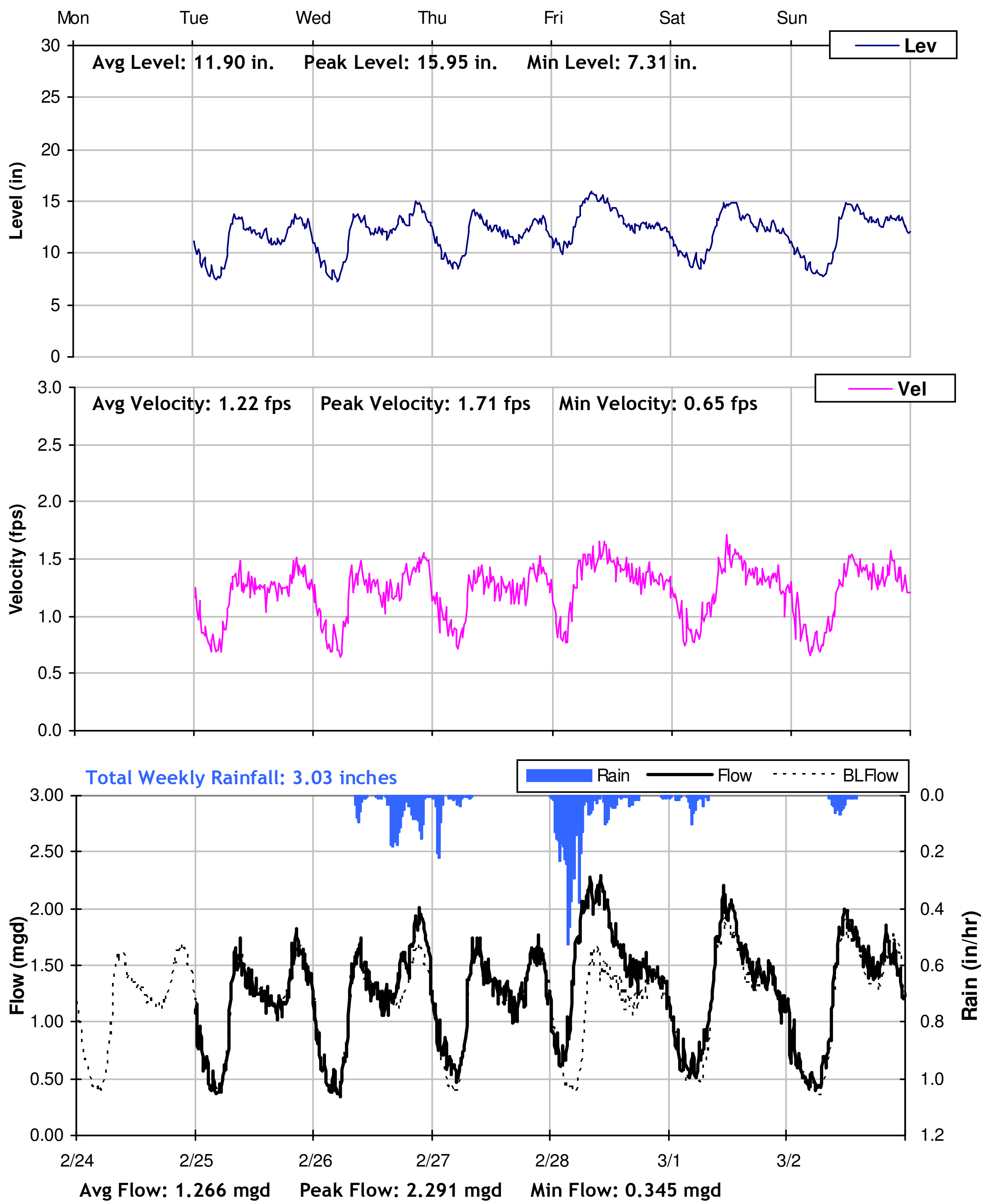
Event 1 Detail Graph



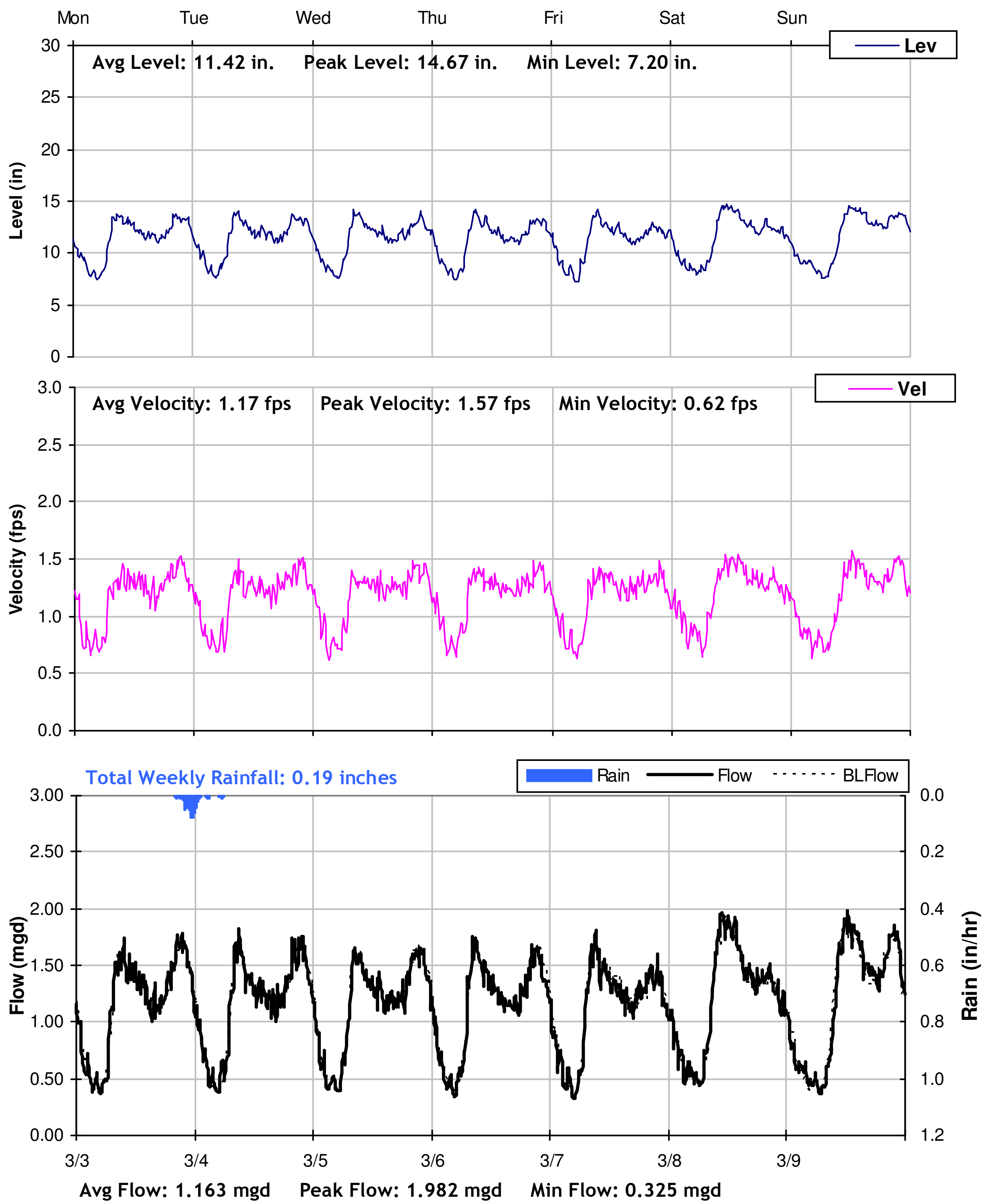
Storm Event I/I Analysis (Rain = 1.83 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	2.29 *mgd*	**Peak I/I Rate:**	1.47 *mgd*
PF:	1.98	**Total I/I:**	608,000 *gallons*

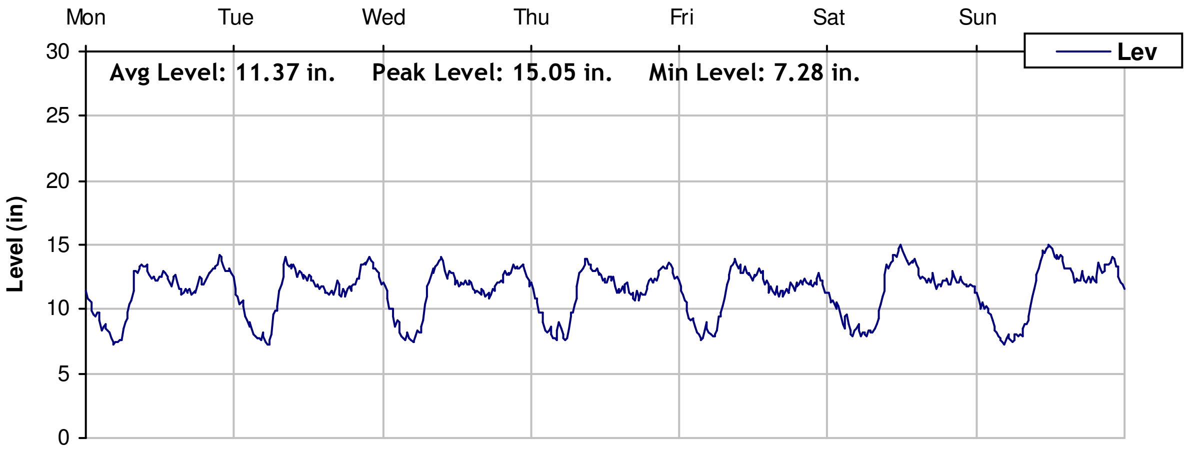
SITE 5
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014

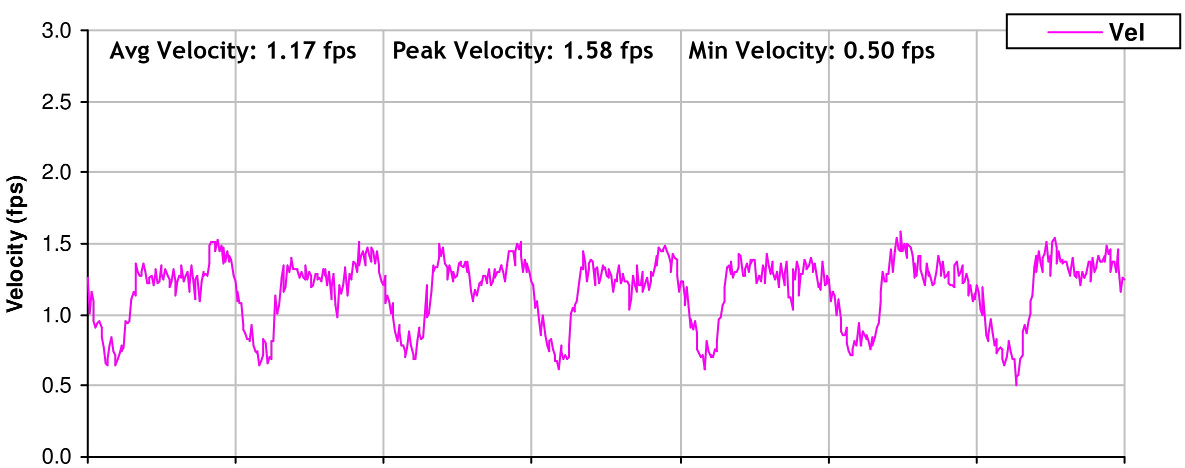


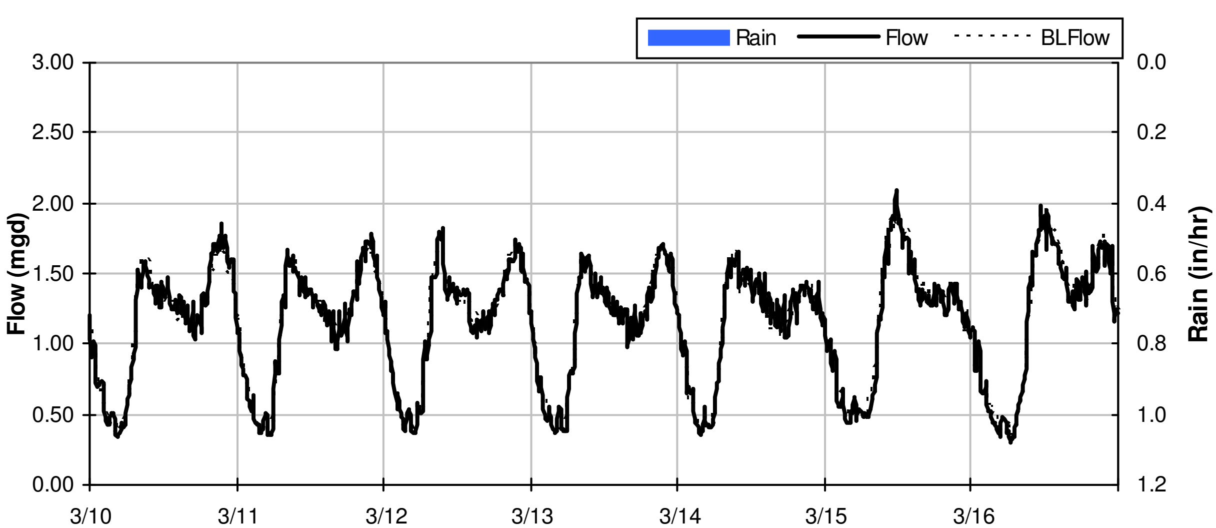
SITE 5
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



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







Avg Flow: 1.154 mgd Peak Flow: 2.101 mgd Min Flow: 0.303 mgd

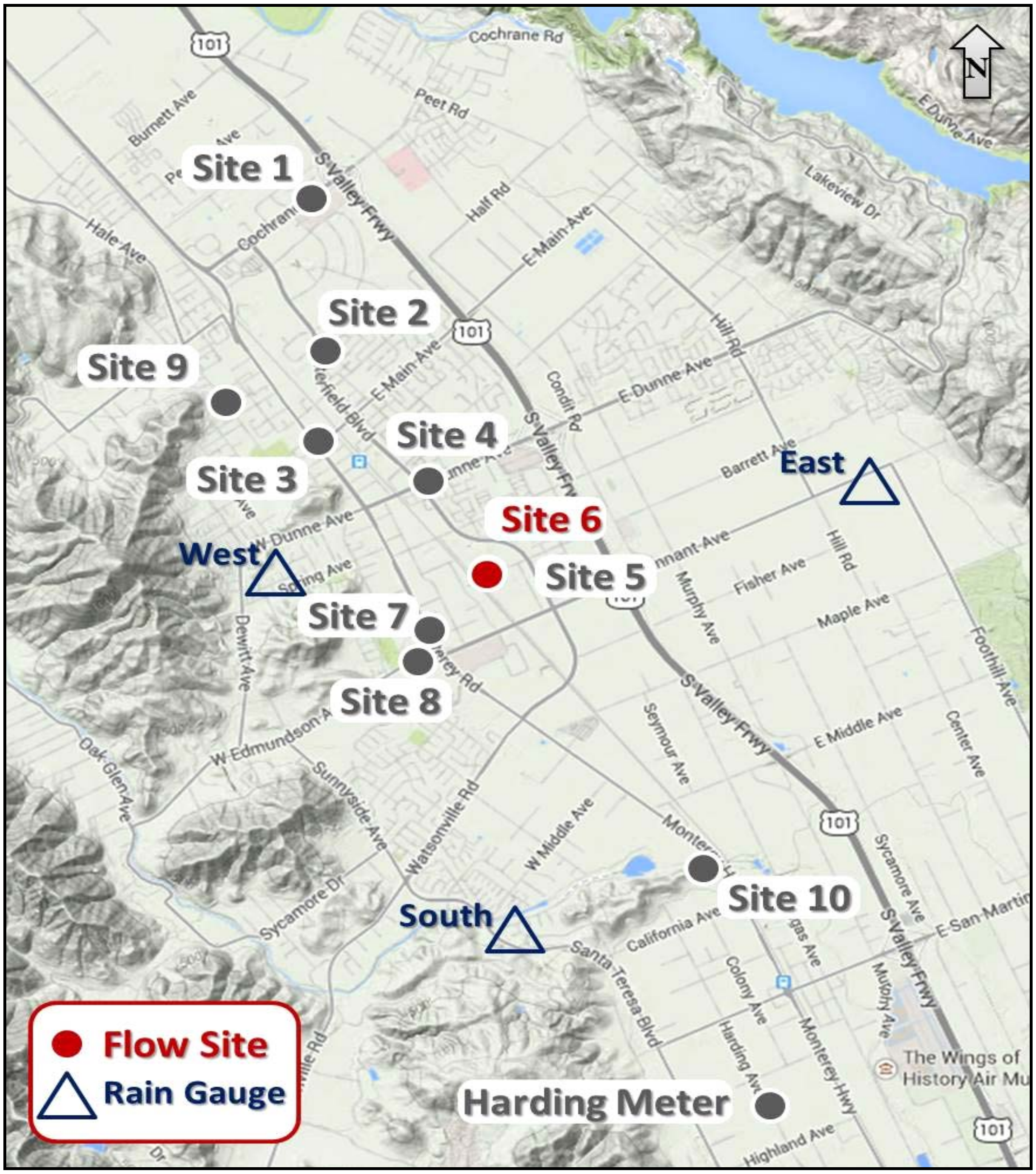
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 6

Location: Intersection of Barrett Avenue and Railroad Avenue

Data Summary Report



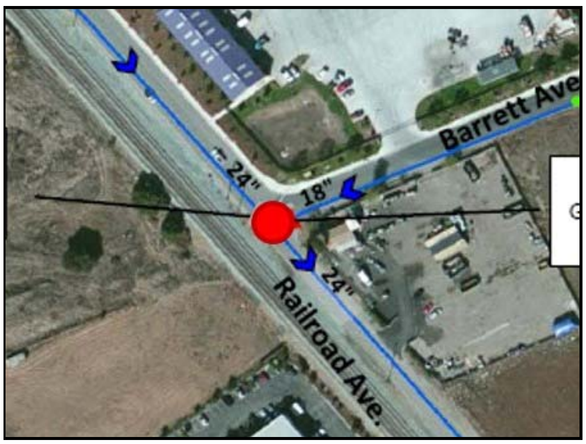
Vicinity Map: Site 6

SITE 6

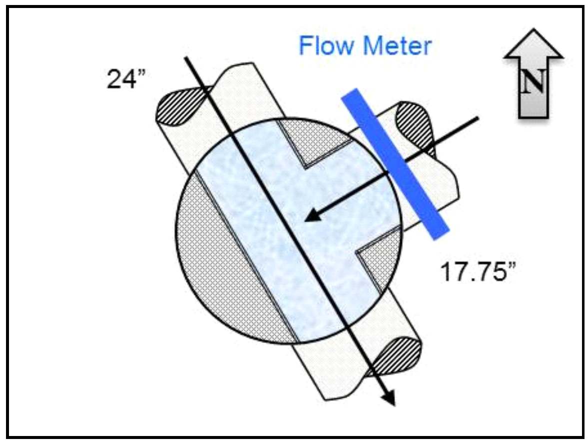
Site Information

Location:	Intersection of Barrett Avenue and Railroad Avenue
Coordinates:	121.6396° W, 37.1192° N
Rim Elevation:	338 feet
Pipe Diameter:	17.75 inches
Baseline Flow:	0.351 mgd
Peak Measured Flow:	0.912 mgd

Satellite Map



Sanitary Map



Flow Sketch

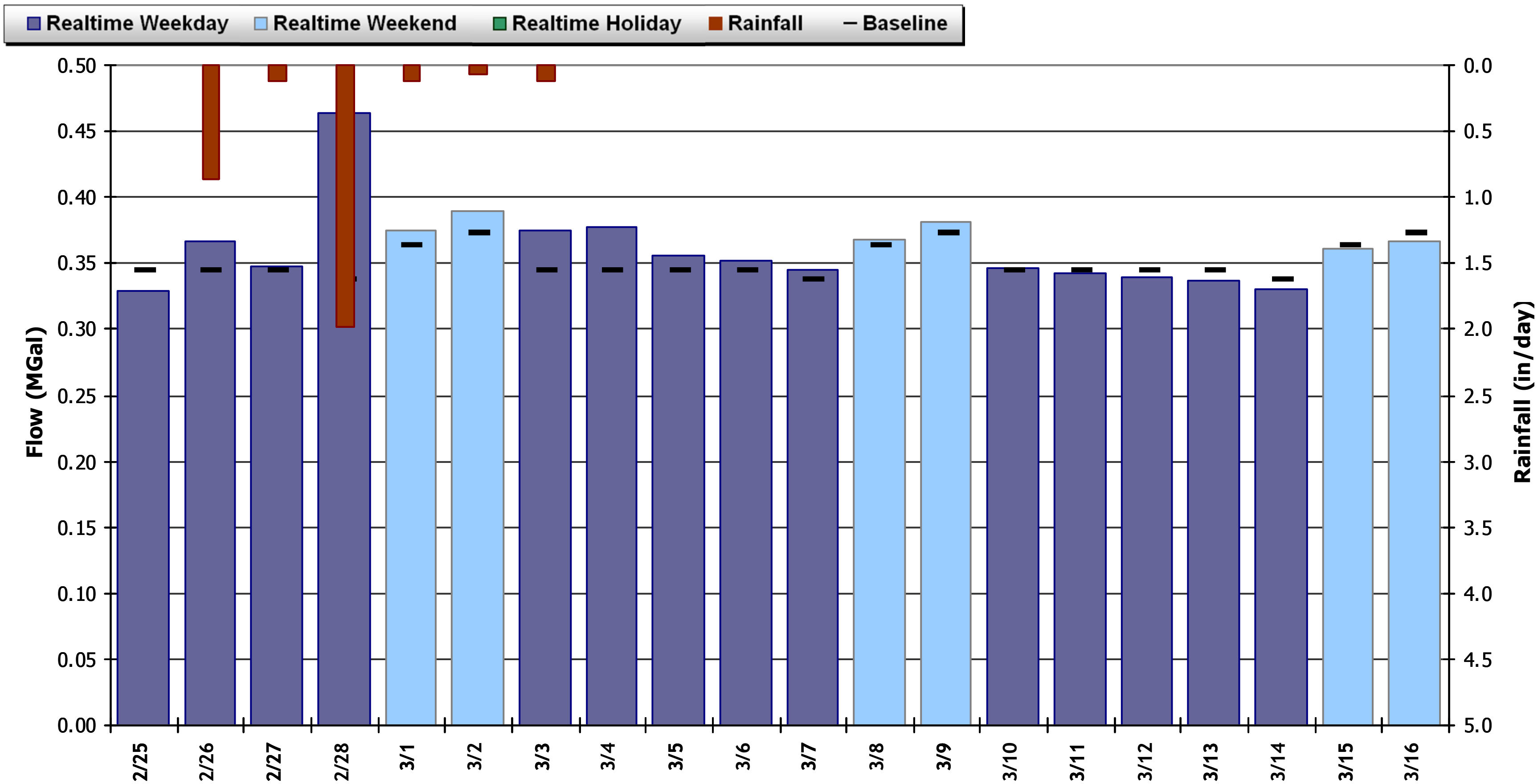
Street View

Plan View

SITE 6
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.362 MGal **Peak Daily Flow: 0.464 MGal** **Min Daily Flow: 0.329 MGal**

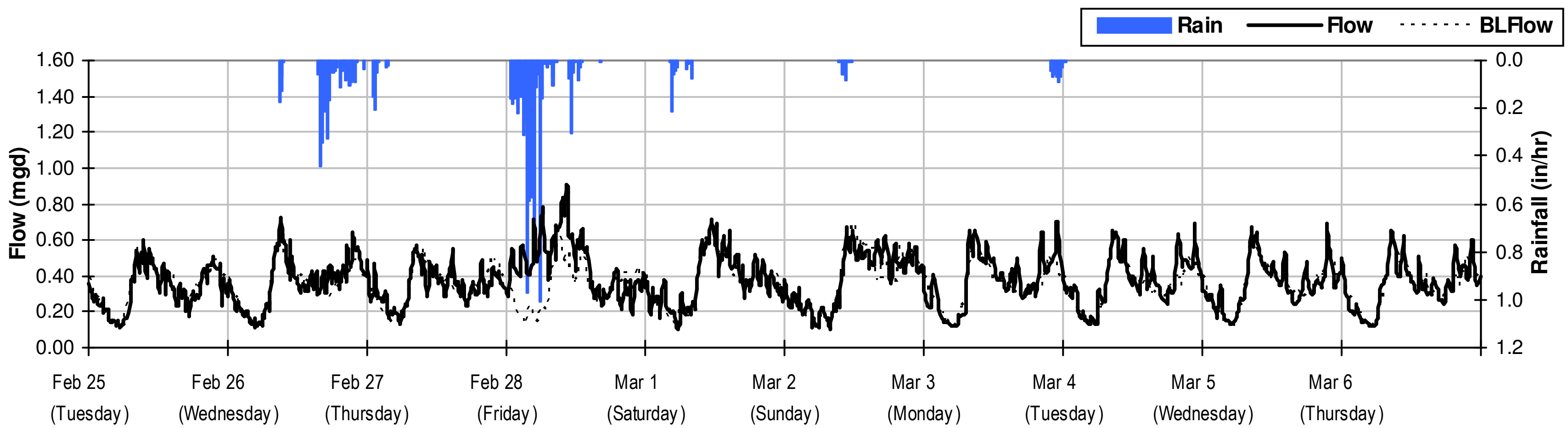
Total Period Rainfall: 3.30 inches

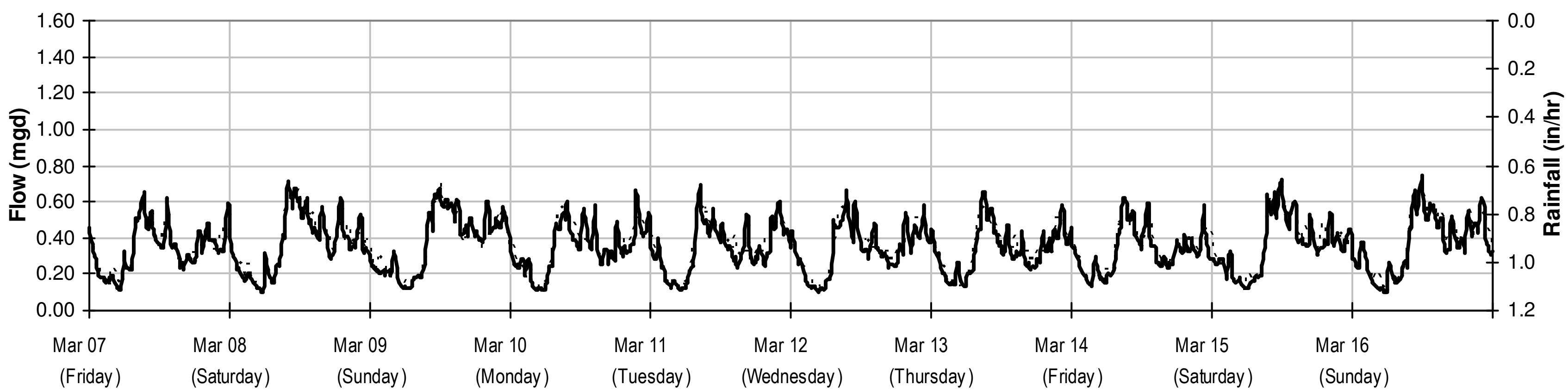


SITE 6

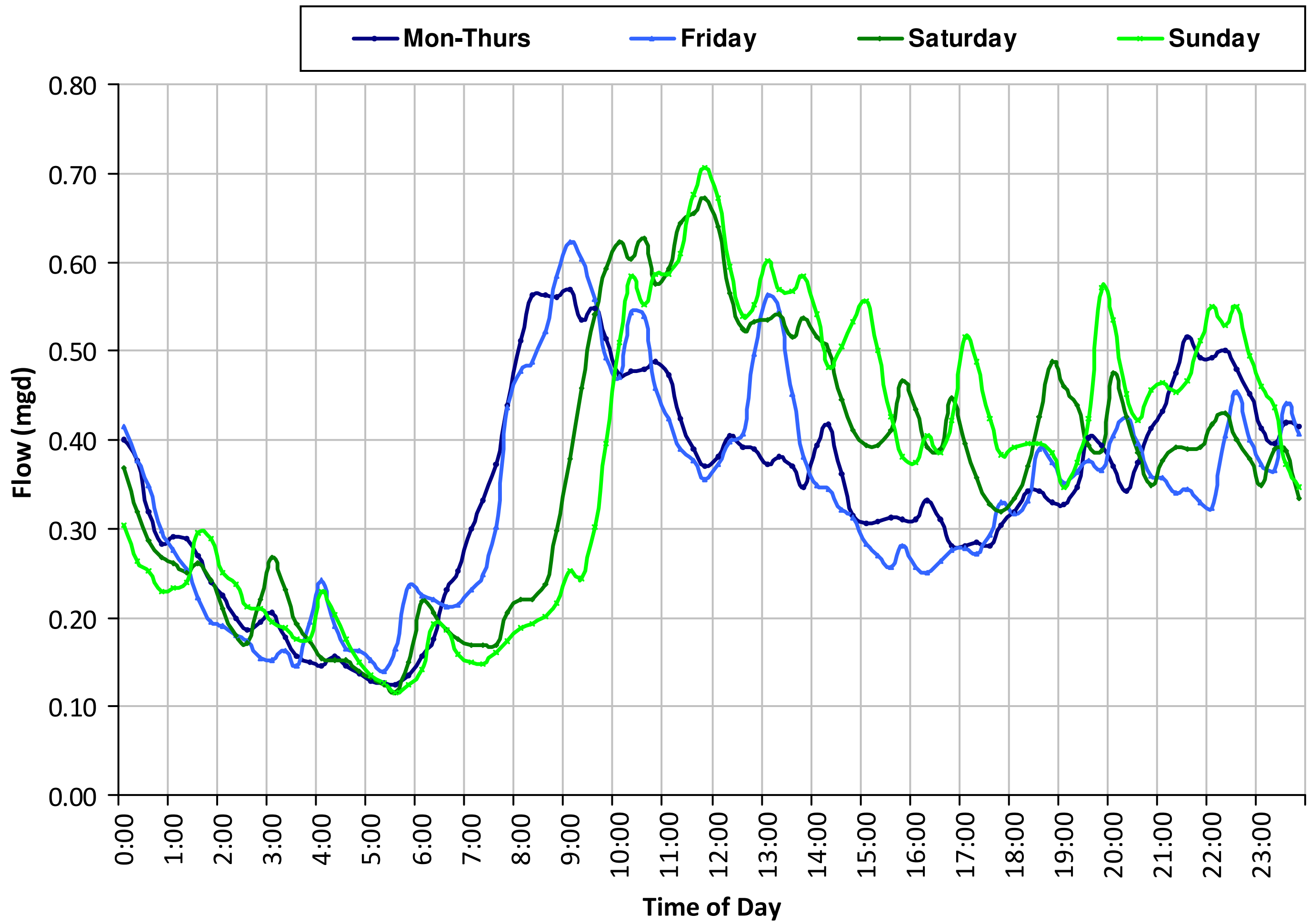
Flow Summary: 2/25/2014 to 3/17/2014

Total Period Rainfall: 3.30 inches **Avg Flow: 0.362 mgd** **Peak Flow: 0.912 mgd** **Min Flow: 0.098 mgd**



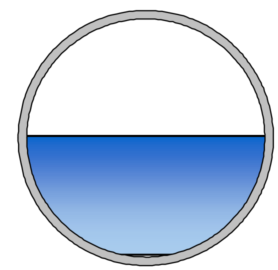


SITE 6
Baseline Flow Hydrographs



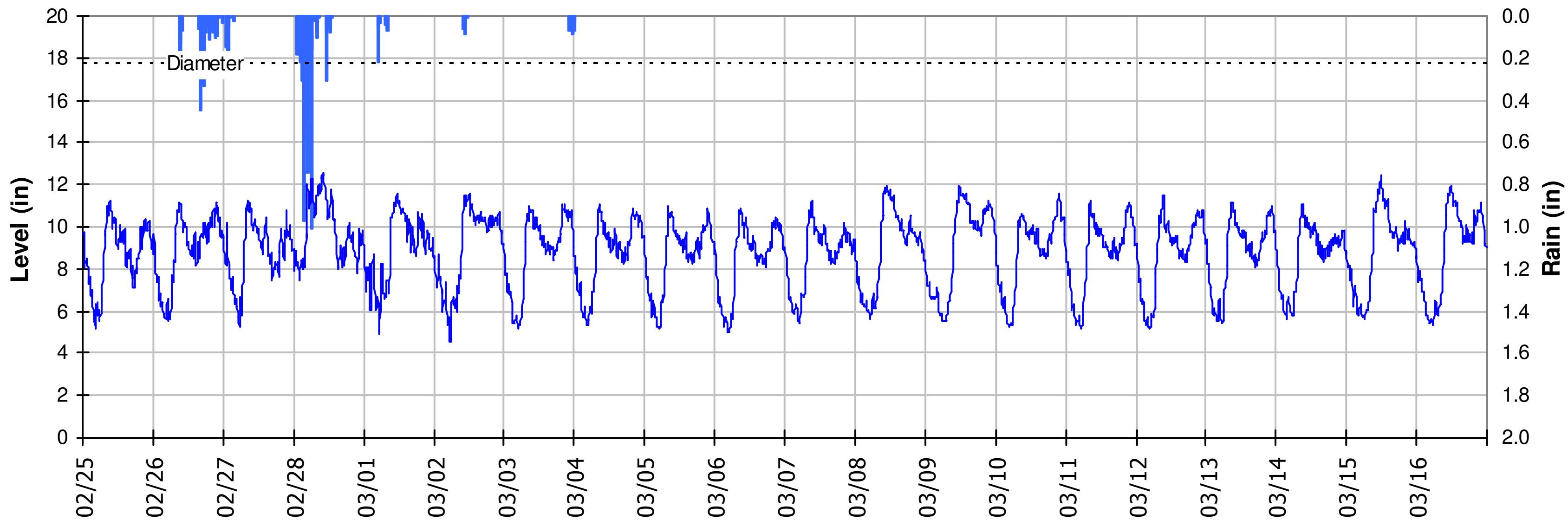
Baseline Flow:

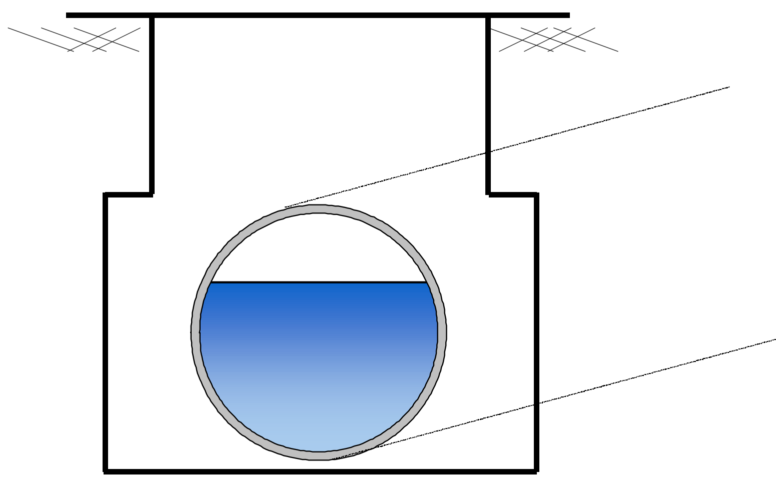
0.351 *mgd*



SITE 6
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





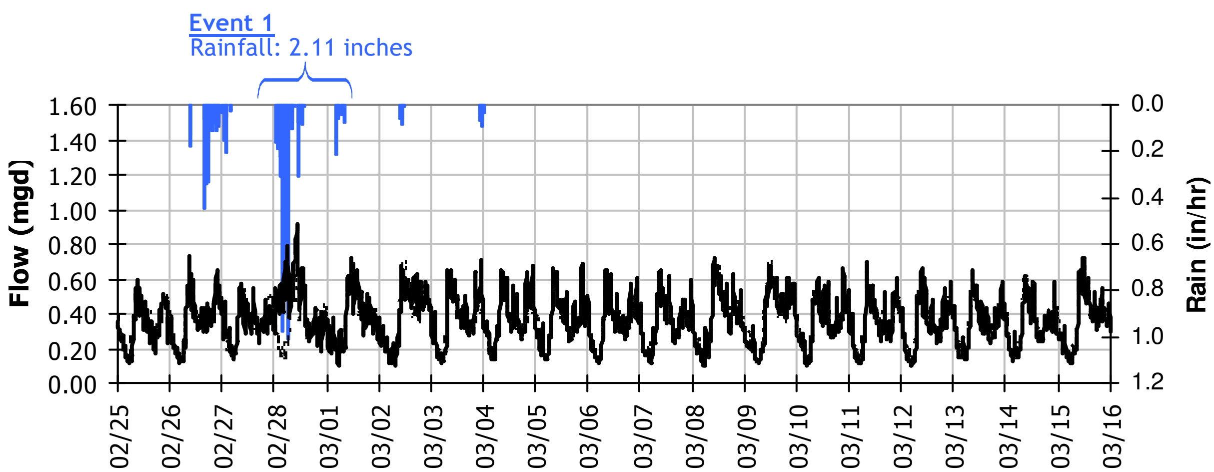
Pipe Diameter: 17.8 *inches*

Peak Measured Level: 12.5 *inches*

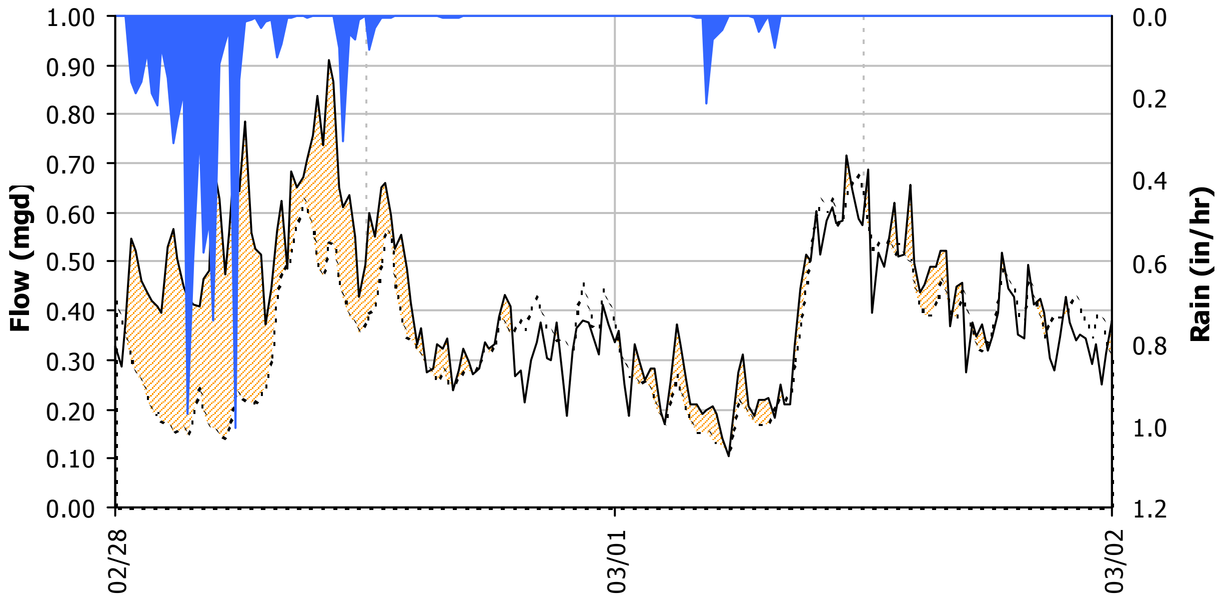
Peak d/D Ratio: 0.71

SITE 6
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph



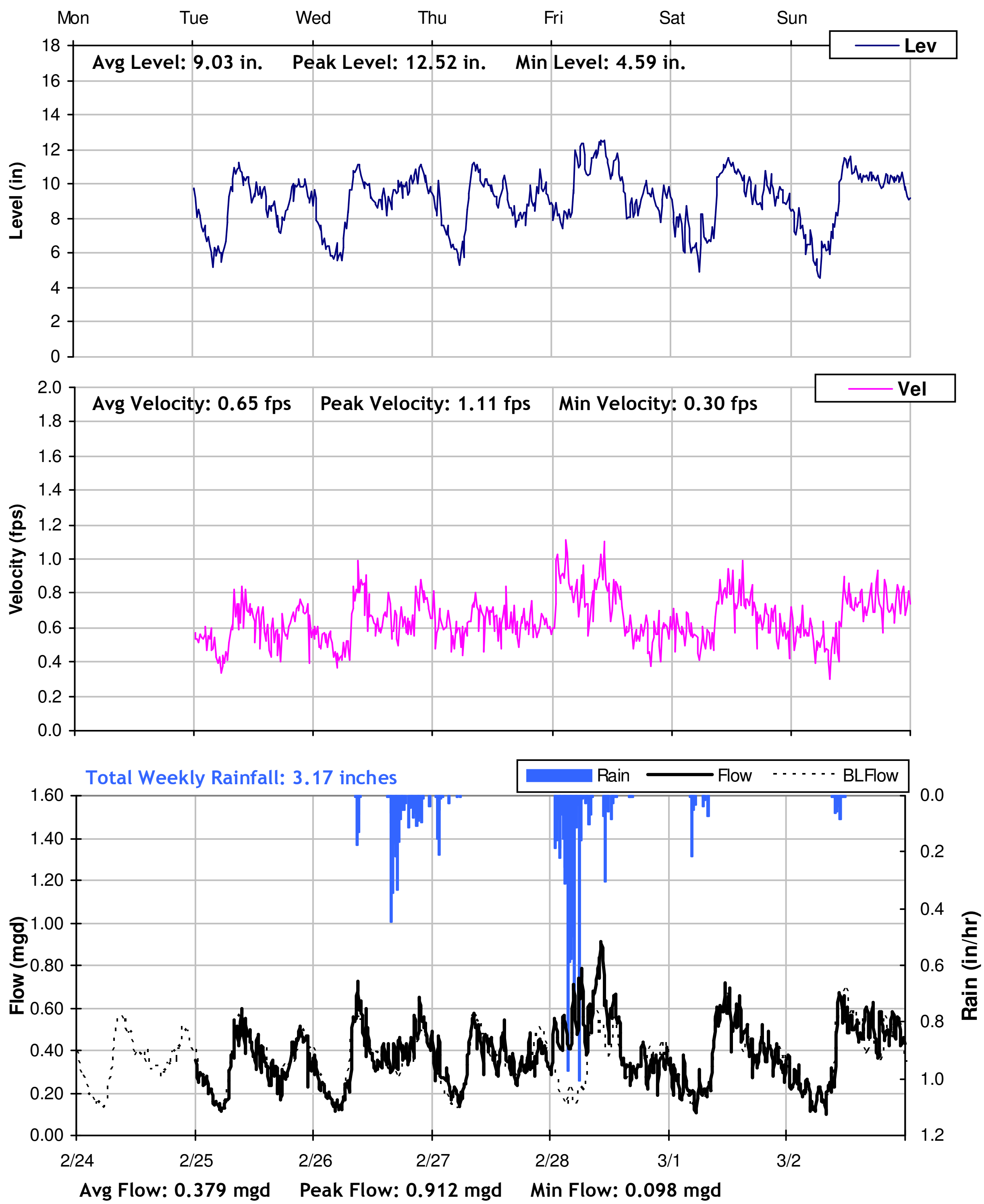
Storm Event I/I Analysis (Rain = 2.11 inches)

Capacity			Inflow / Infiltration		
Peak Flow:	0.91	*mgd*	**Peak I/I Rate:**	0.57	*mgd*
PF:	2.60		**Total I/I:**	136,000	*gallons*

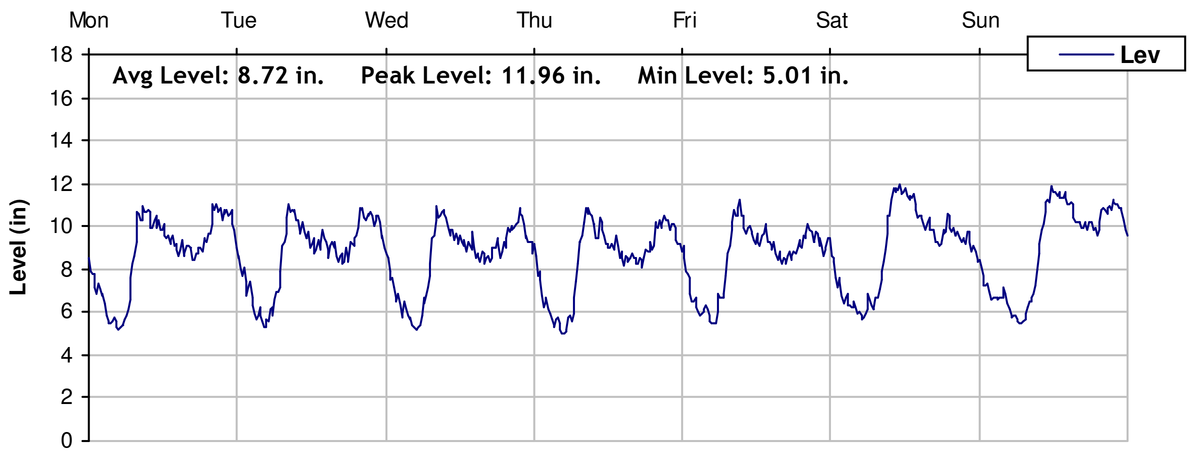
SITE 6

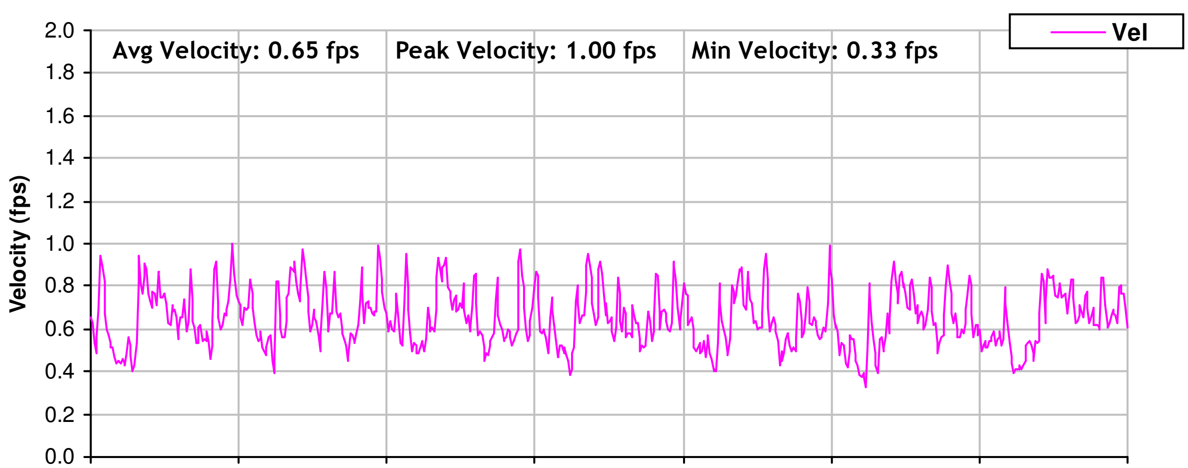
Weekly Level, Velocity and Flow Hydrographs

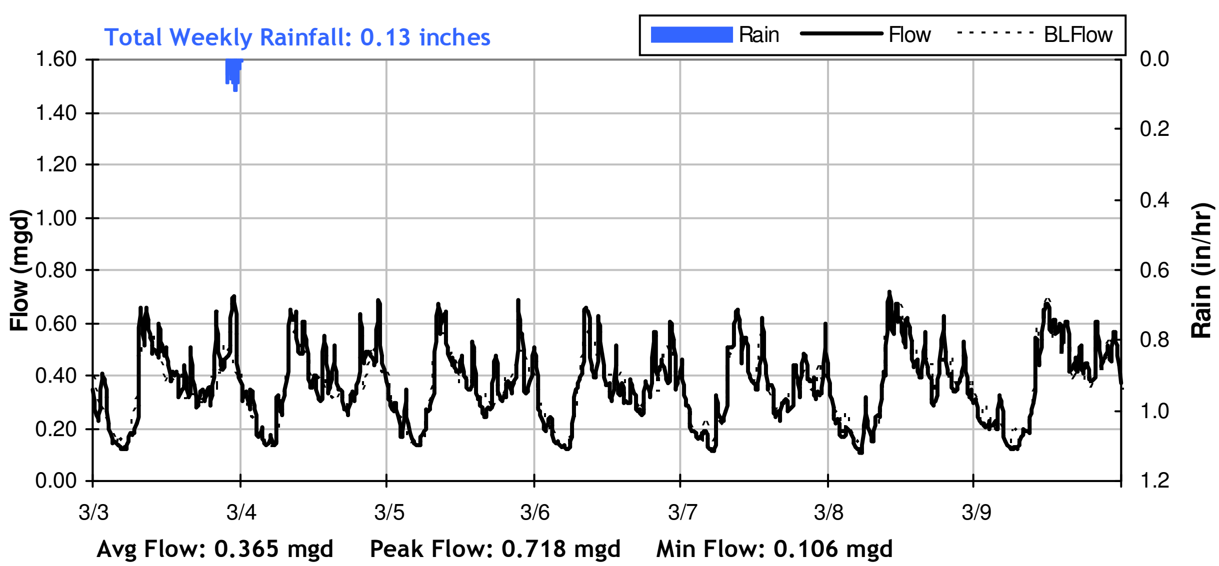
2/24/2014 to 3/3/2014



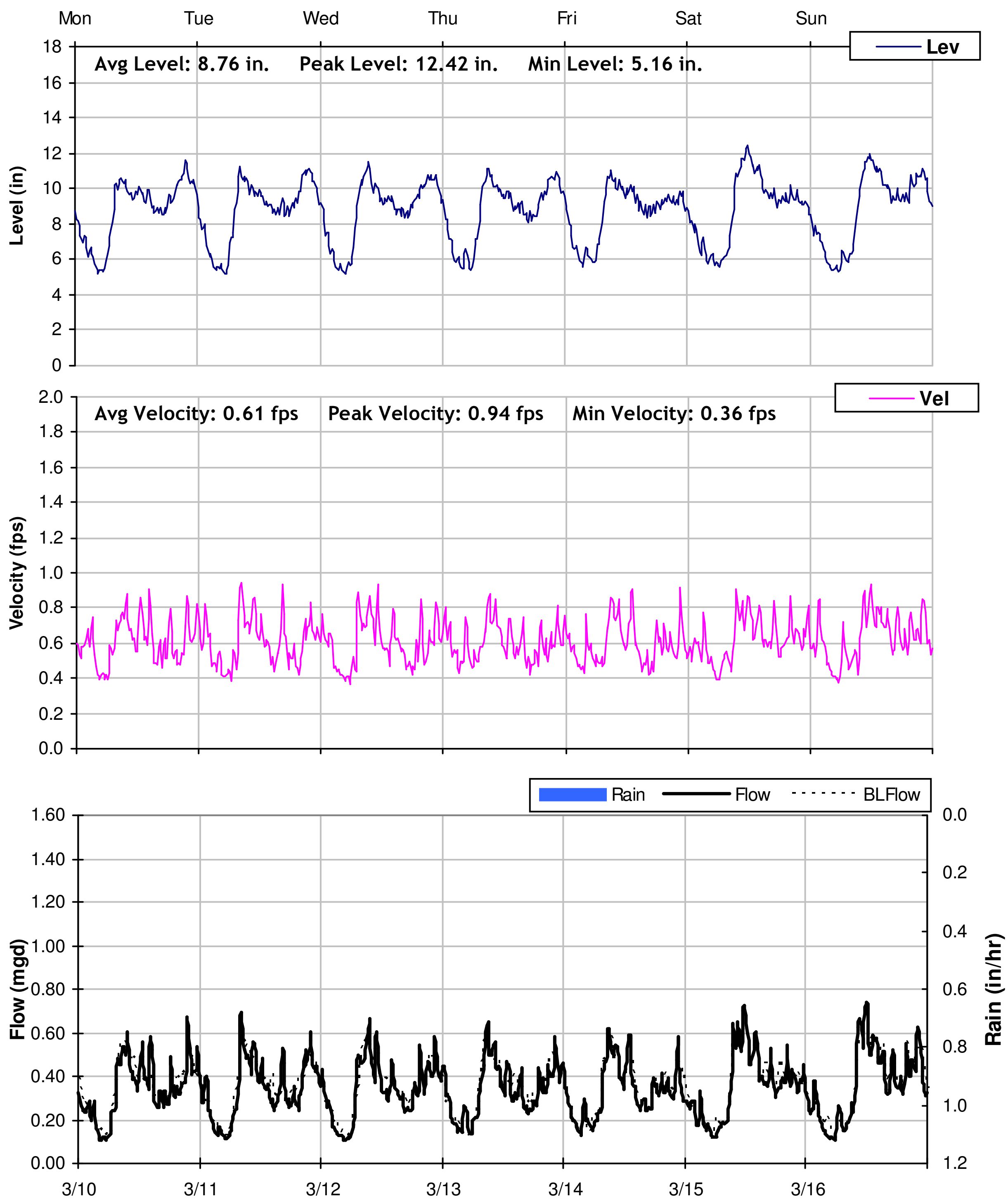
SITE 6
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014







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



Avg Flow: 0.346 mgd Peak Flow: 0.741 mgd Min Flow: 0.105 mgd

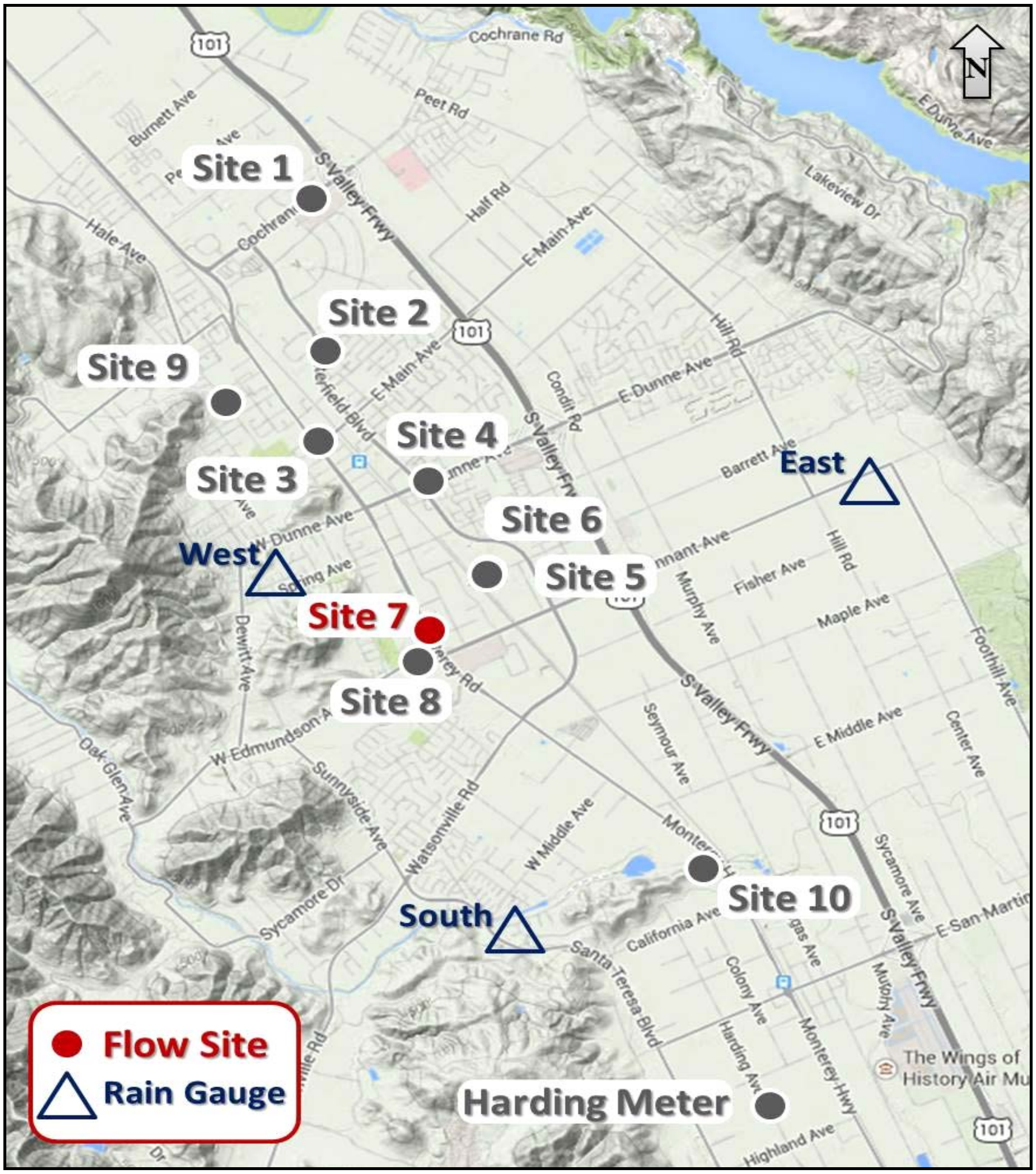
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 7

Location: Intersection of Edes Street and Monterey Road

Data Summary Report



Vicinity Map: Site 7

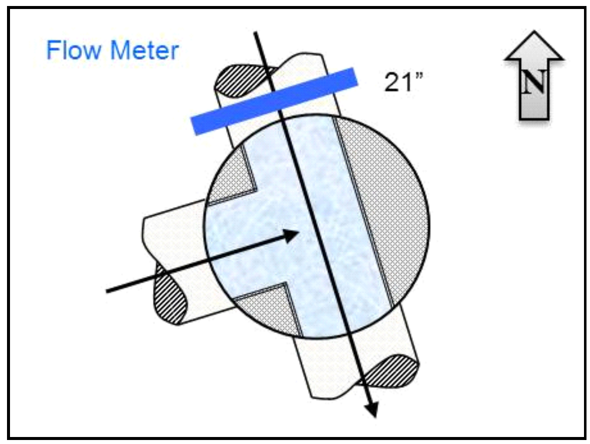
SITE 7

Site Information

Location:	Intersection of Edes Street and Monterey Road
Coordinates:	121.6436° W, 37.1137° N
Rim Elevation:	332 feet
Pipe Diameter:	21 inches
Baseline Flow:	0.241 mgd
Peak Measured Flow:	0.766 mgd

Satellite Map

Sanitary Map



Flow Sketch

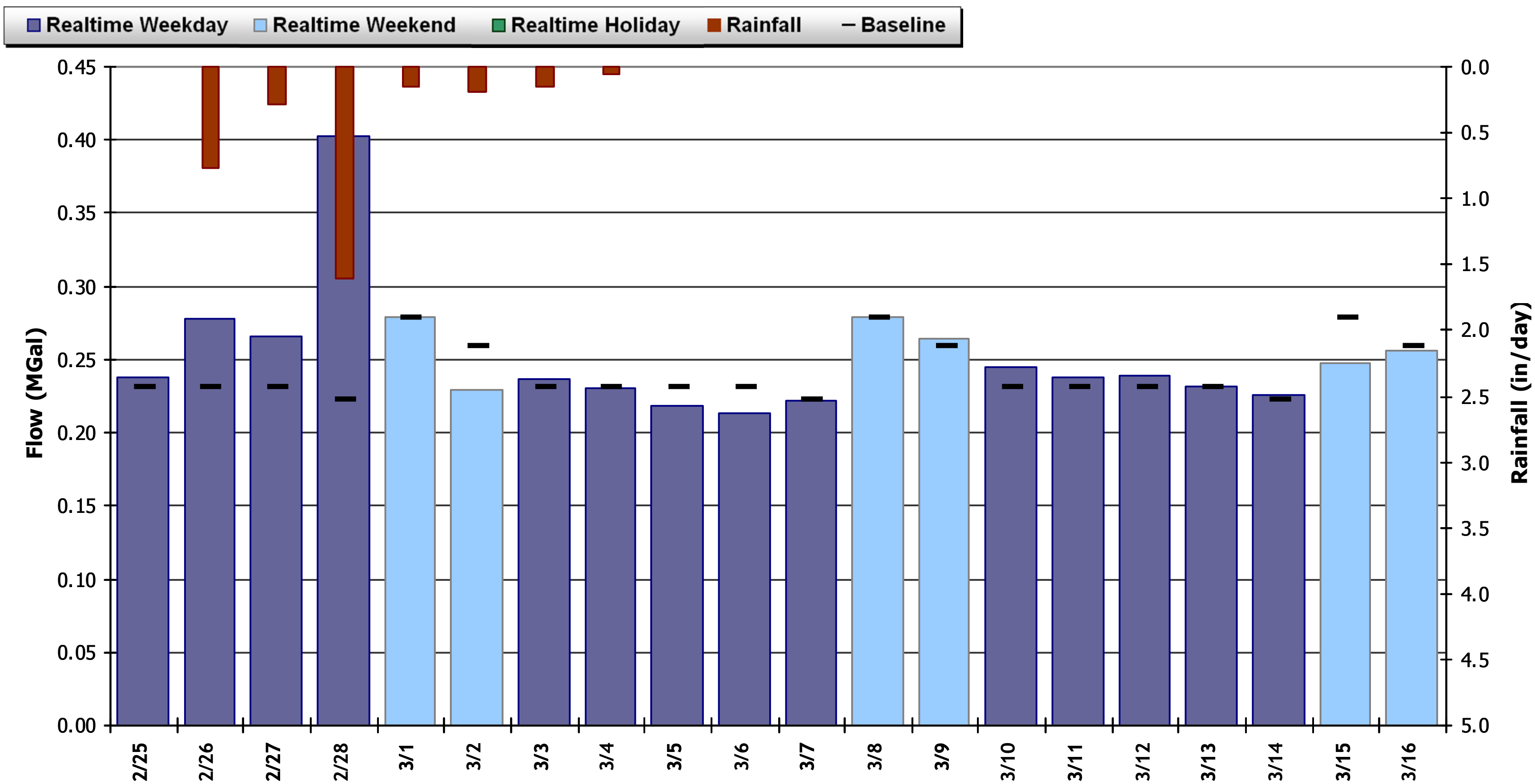
Street View

Plan View

SITE 7
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.252 MGal **Peak Daily Flow: 0.402 MGal** **Min Daily Flow: 0.213 MGal**

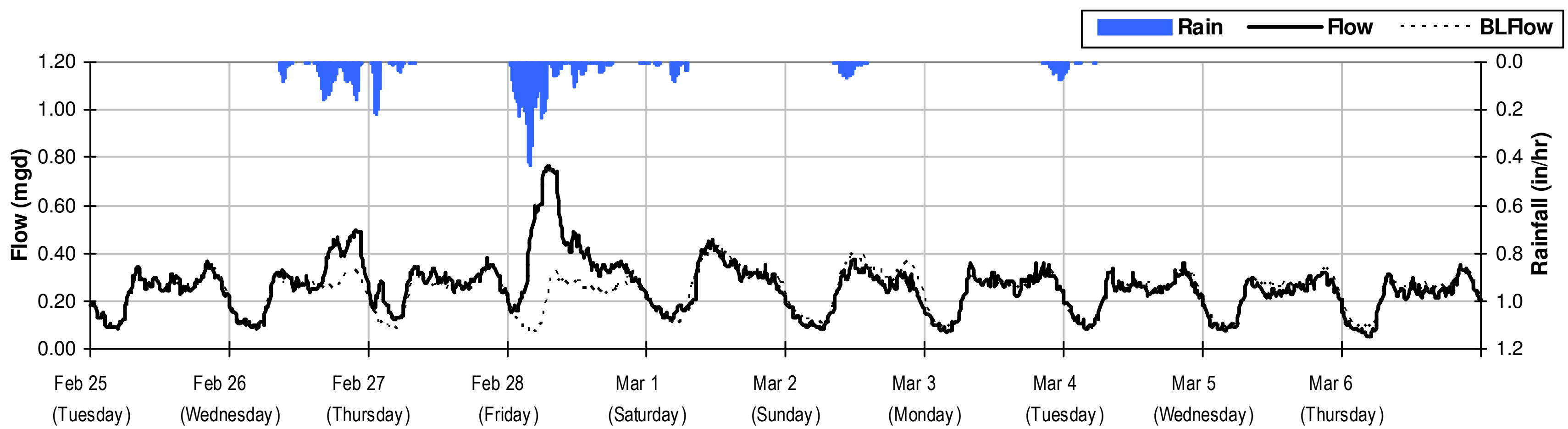
Total Period Rainfall: 3.20 inches

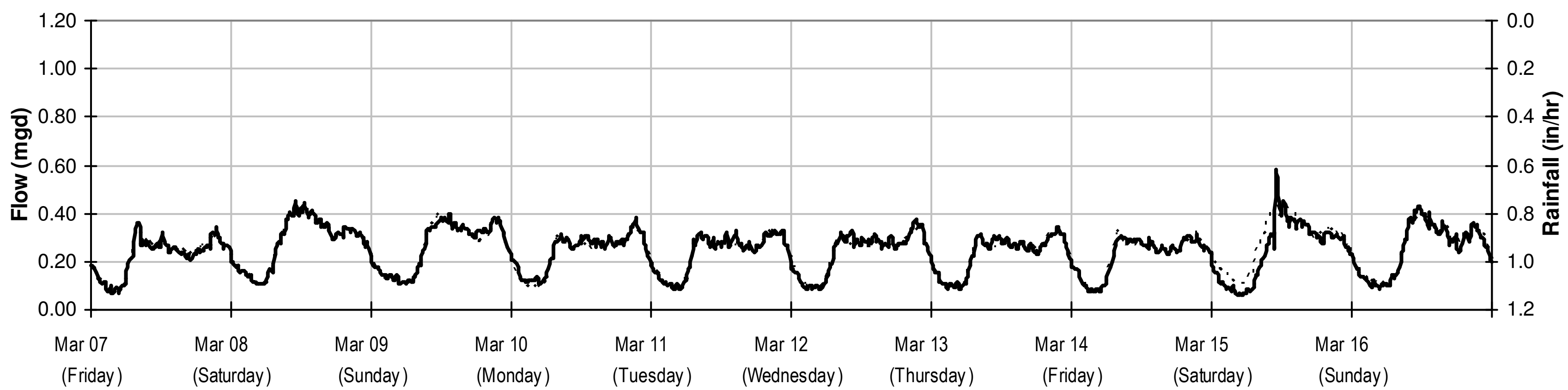


SITE 7

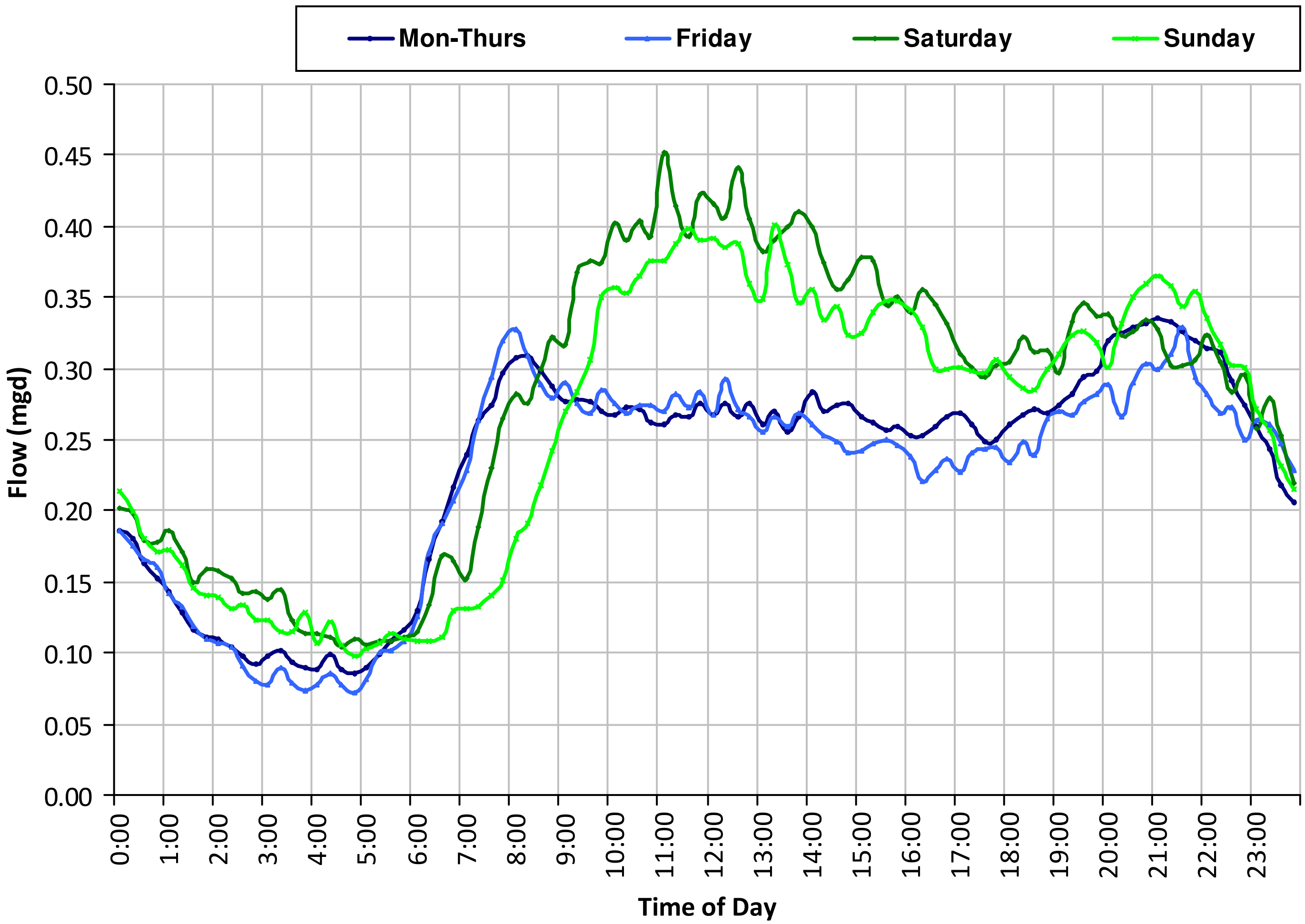
Flow Summary: 2/25/2014 to 3/17/2014

Total Period Rainfall: 3.20 inches Avg Flow: 0.252 mgd Peak Flow: 0.766 mgd Min Flow: 0.054 mgd



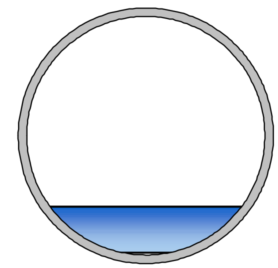


SITE 7
Baseline Flow Hydrographs



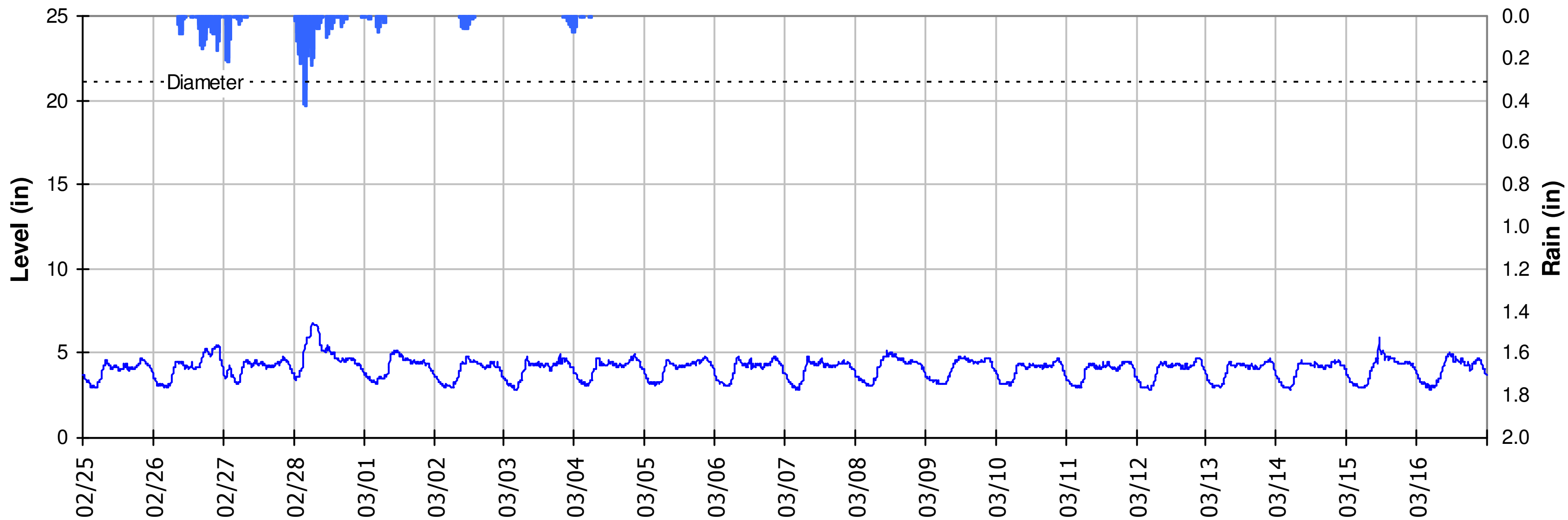
Baseline Flow:

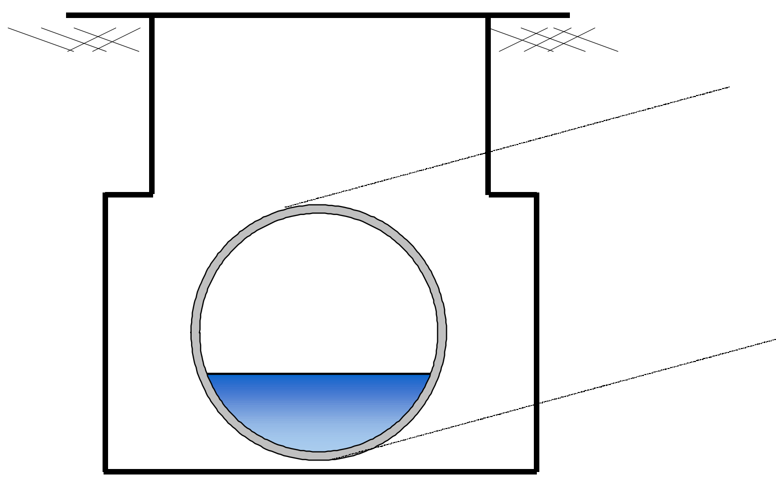
0.241 *mgd*



SITE 7
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





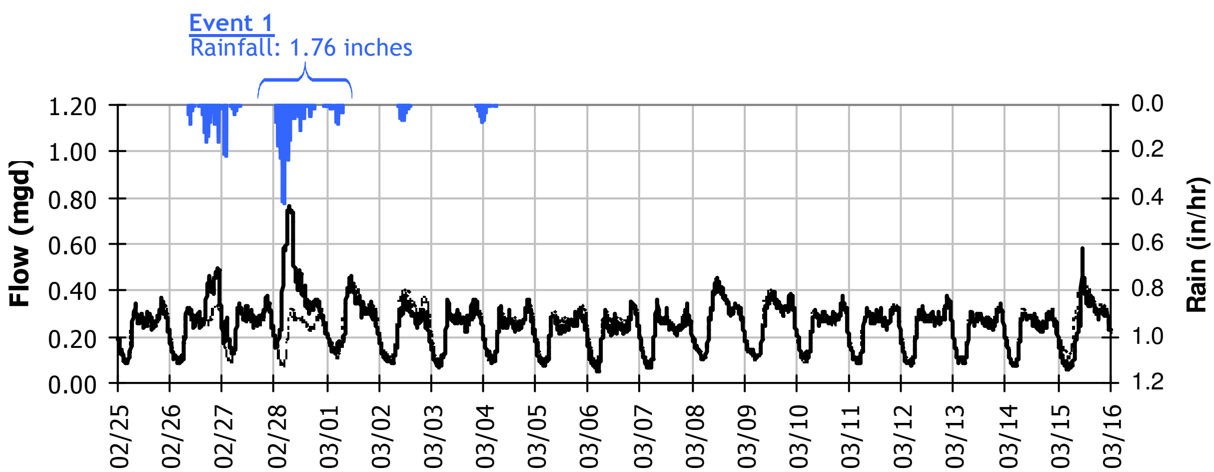
Pipe Diameter: 21 *inches*

Peak Measured Level: 6.82 *inches*

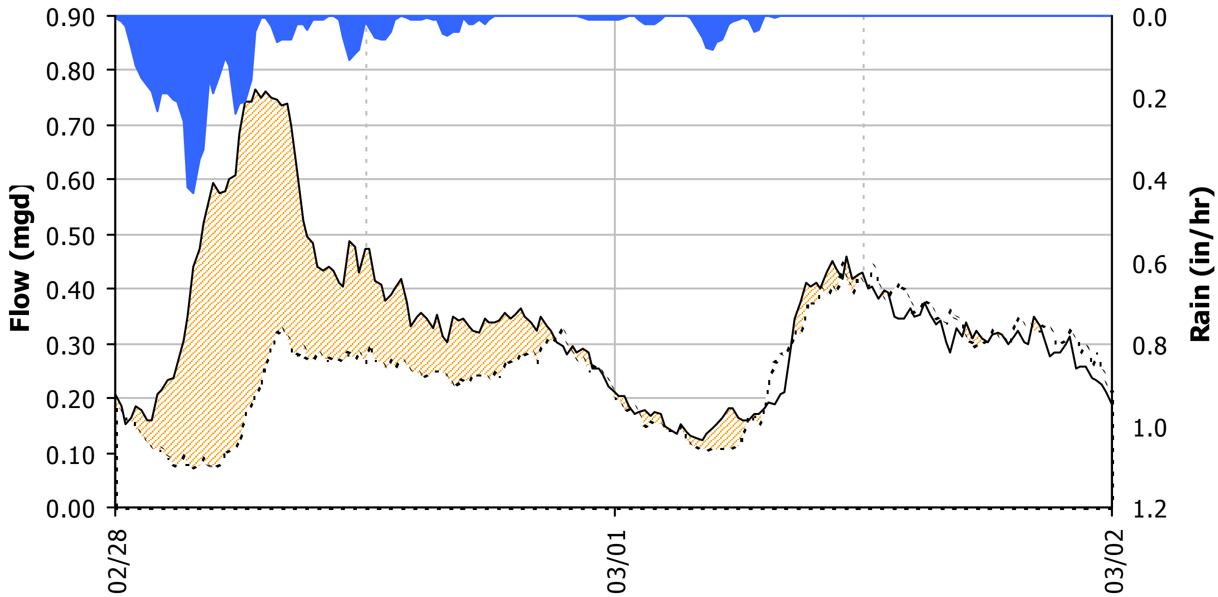
Peak d/D Ratio: 0.32

SITE 7
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



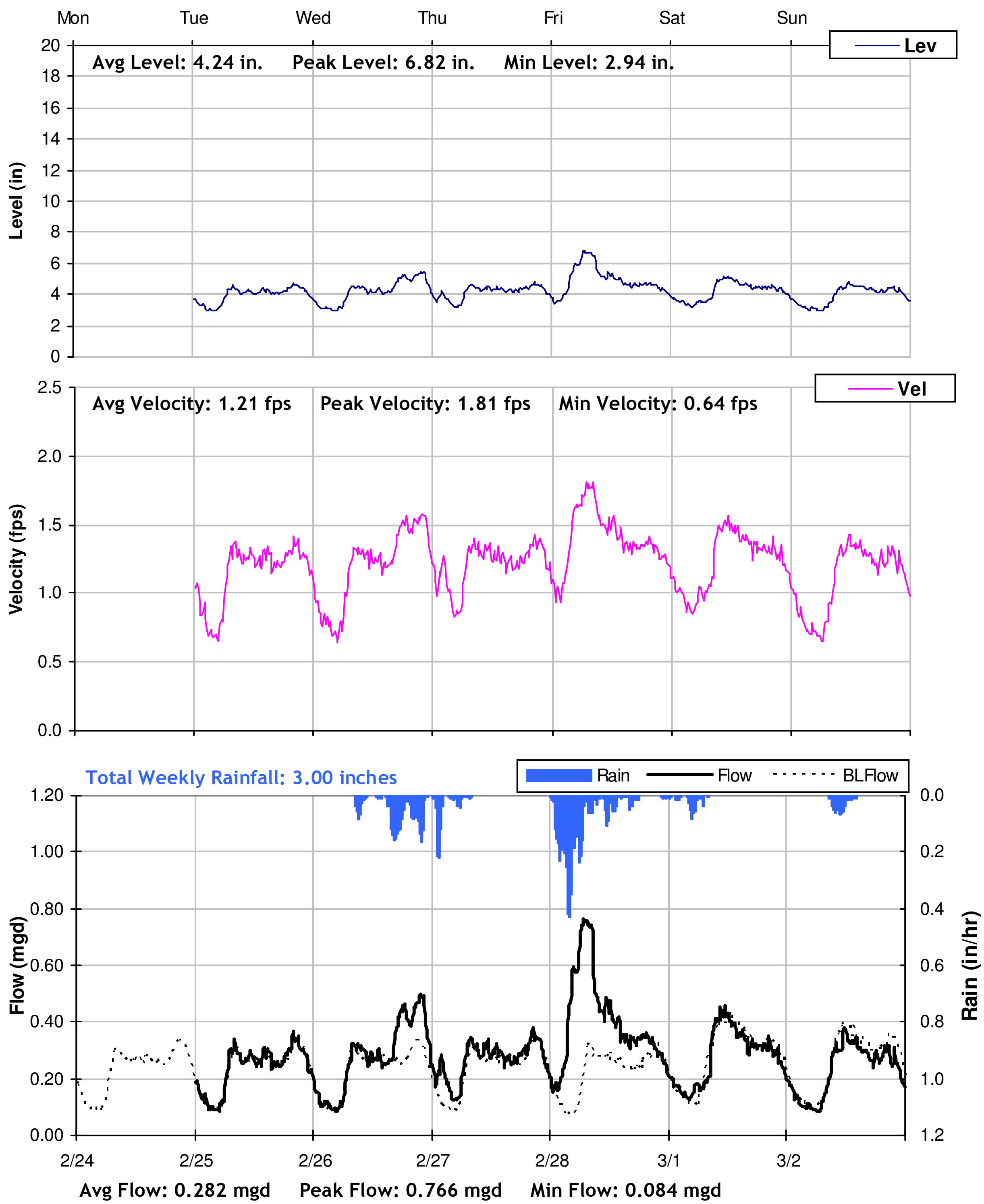
Event 1 Detail Graph



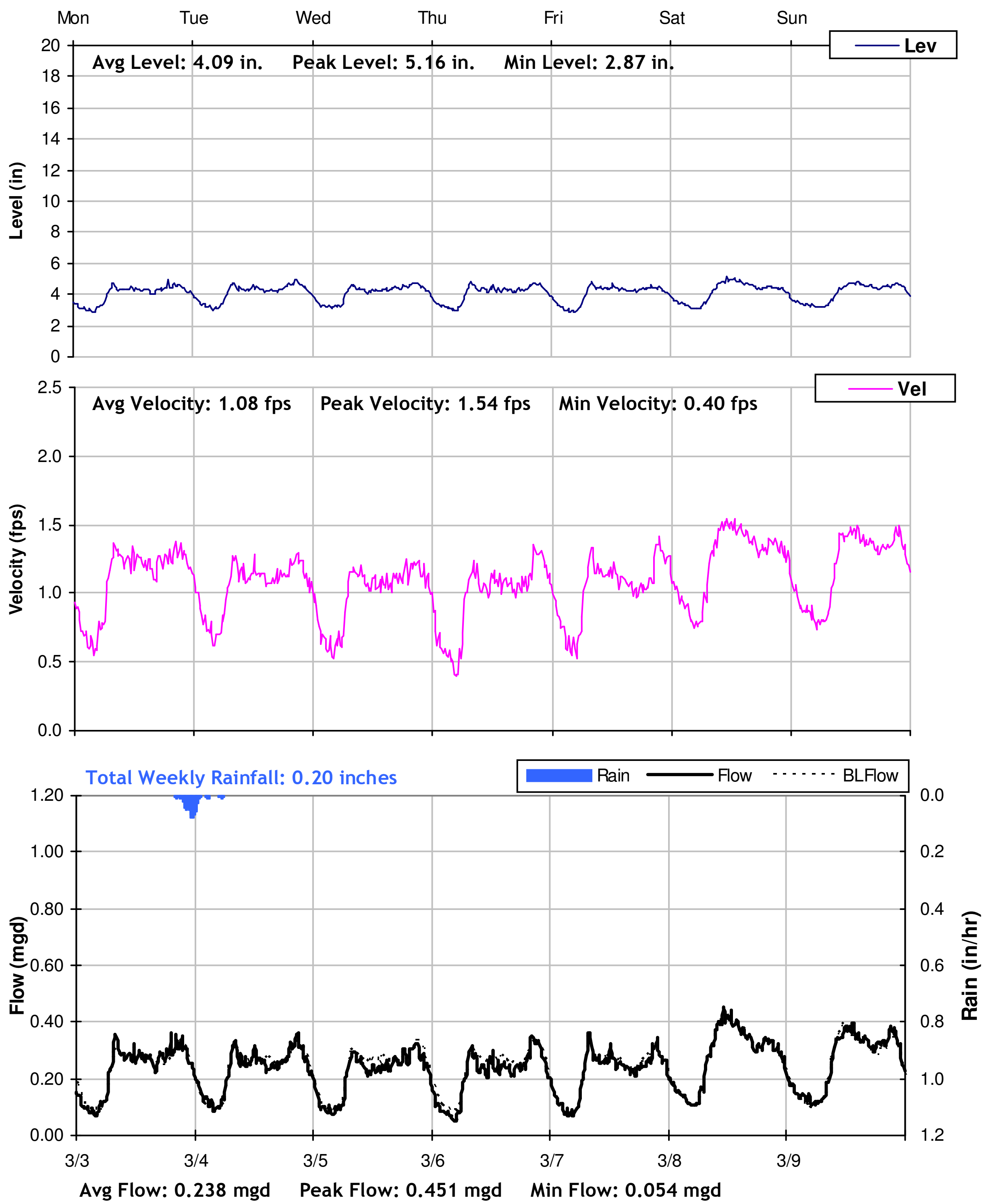
Storm Event I/I Analysis (Rain = 1.76 inches)

Capacity		**Inflow / Infiltration**	
Peak Flow:	0.77 *mgd*	**Peak I/I Rate:**	0.57 *mgd*
PF:	3.17	**Total I/I:**	178,000 *gallons*

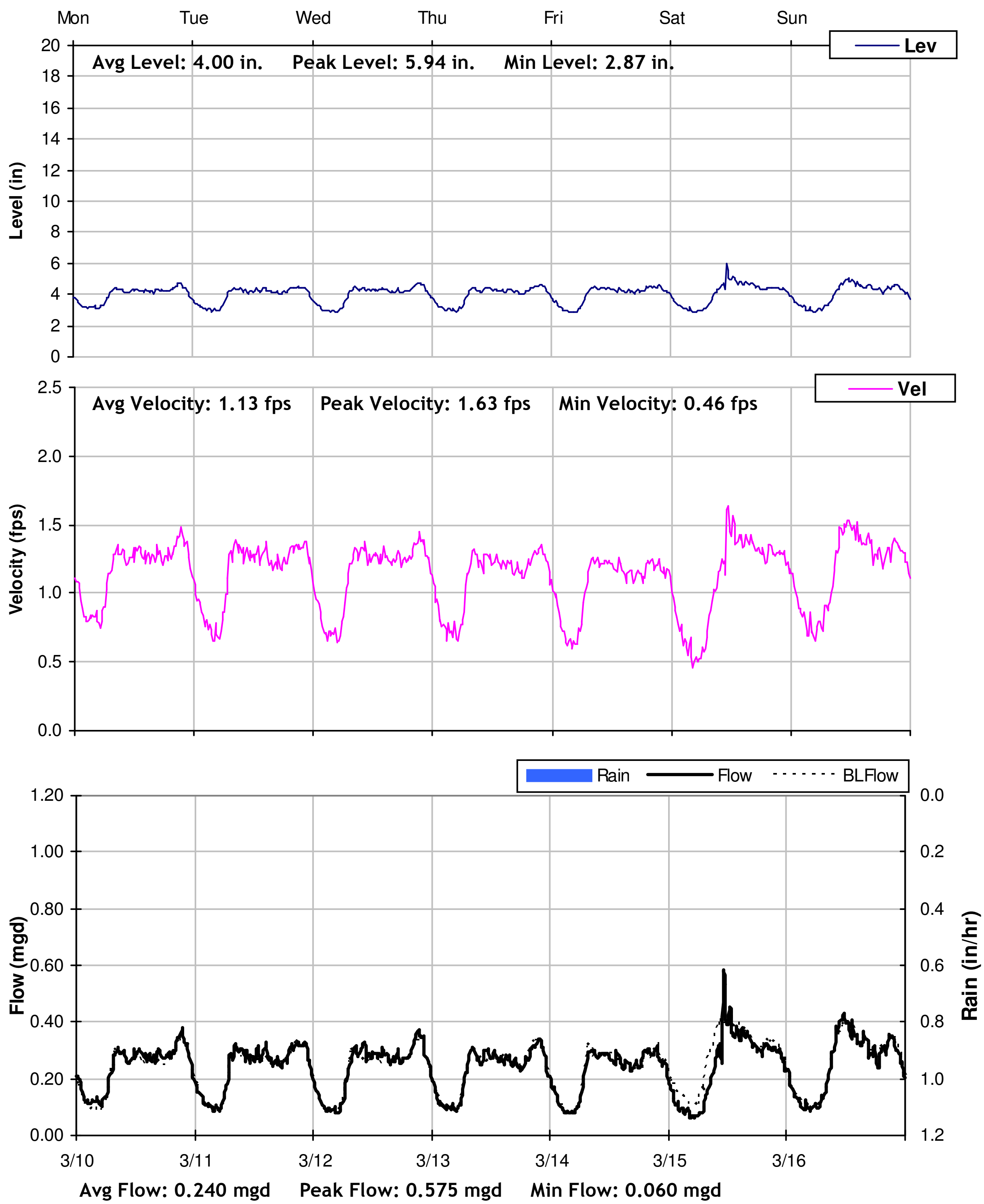
SITE 7
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



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



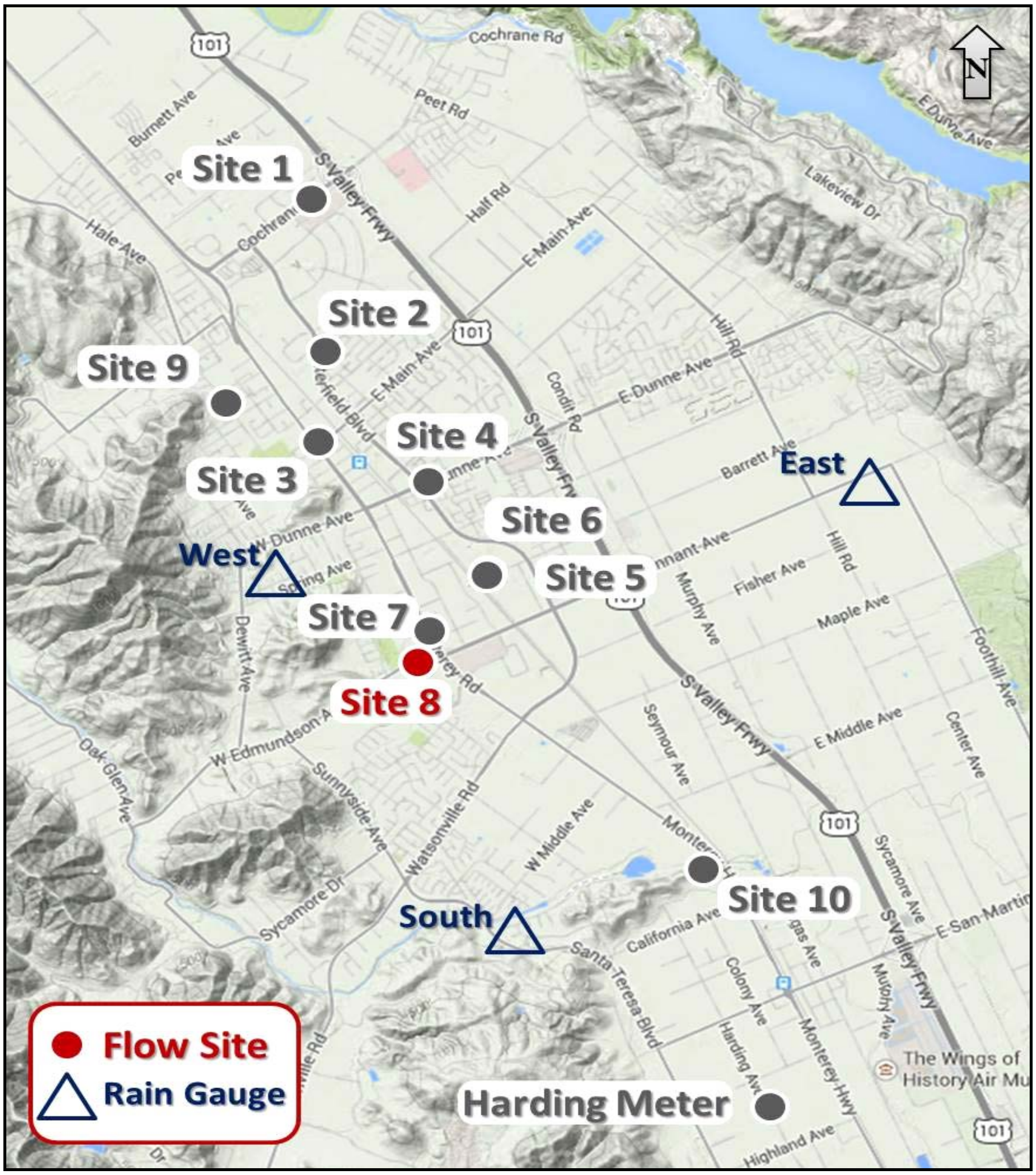
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 8

Location: W Edmundson Avenue, just west of Monterey Road

Data Summary Report



Vicinity Map: Site 8

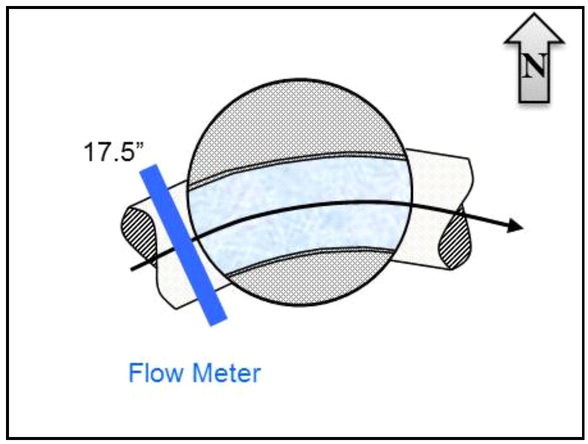
SITE 8

Site Information

Location:	W Edmundson Avenue, just west of Monterey Road
Coordinates:	121.6437° W, 37.1127° N
Rim Elevation:	331 feet
Pipe Diameter:	17.5 inches
Baseline Flow:	0.159 mgd
Peak Measured Flow:	0.435 mgd

Satellite Map

Sanitary Map



Flow Sketch

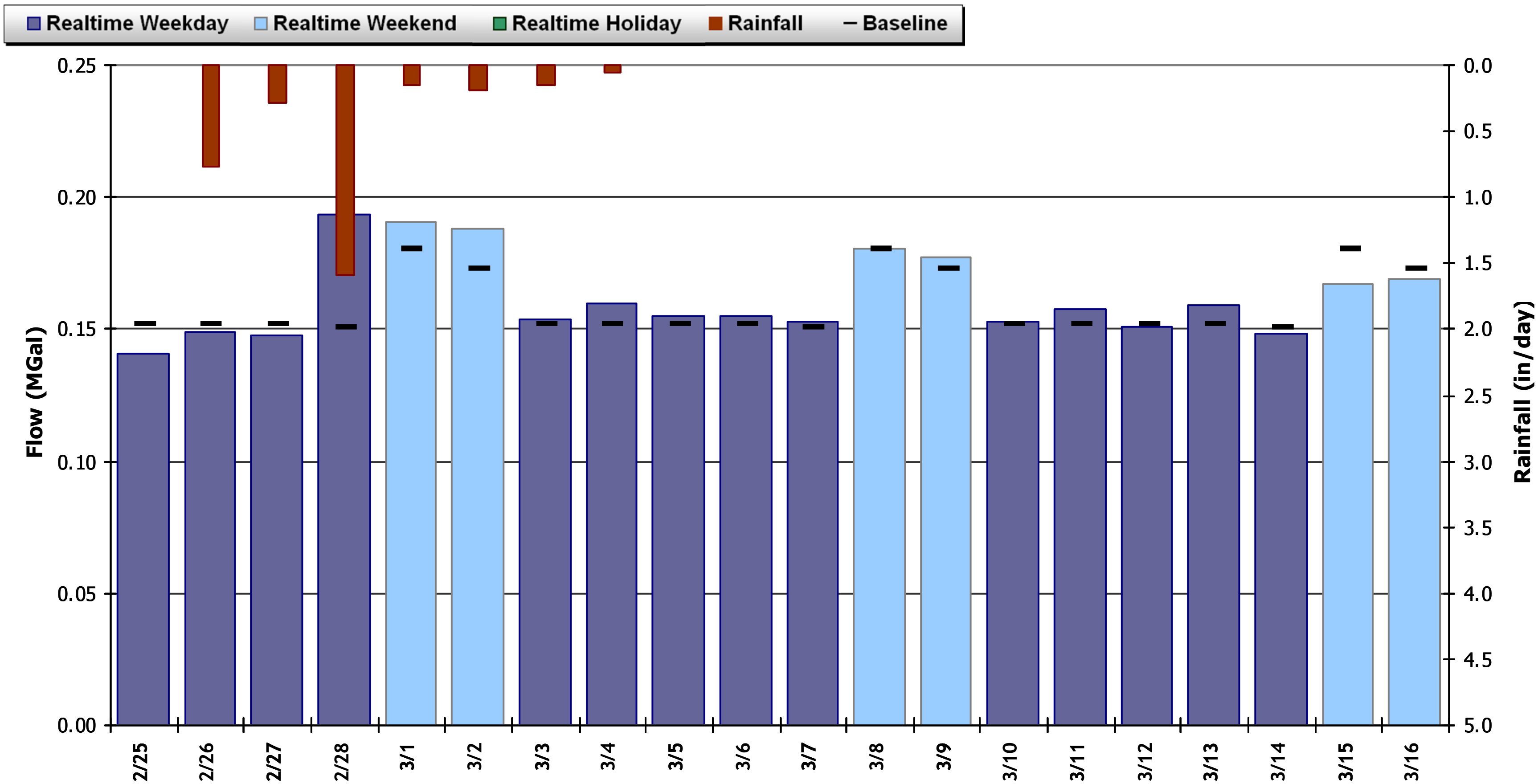
Street View

Plan View

SITE 8
Period Flow Summary: Daily Flow Totals

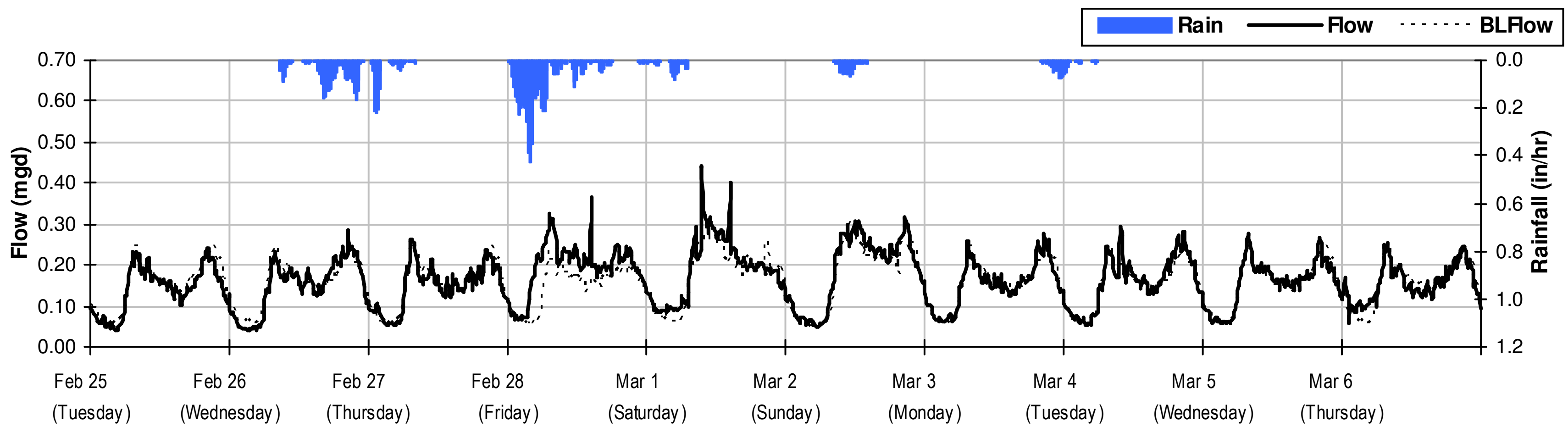
Avg Period Flow: 0.162 MGal **Peak Daily Flow: 0.193 MGal** **Min Daily Flow: 0.141 MGal**

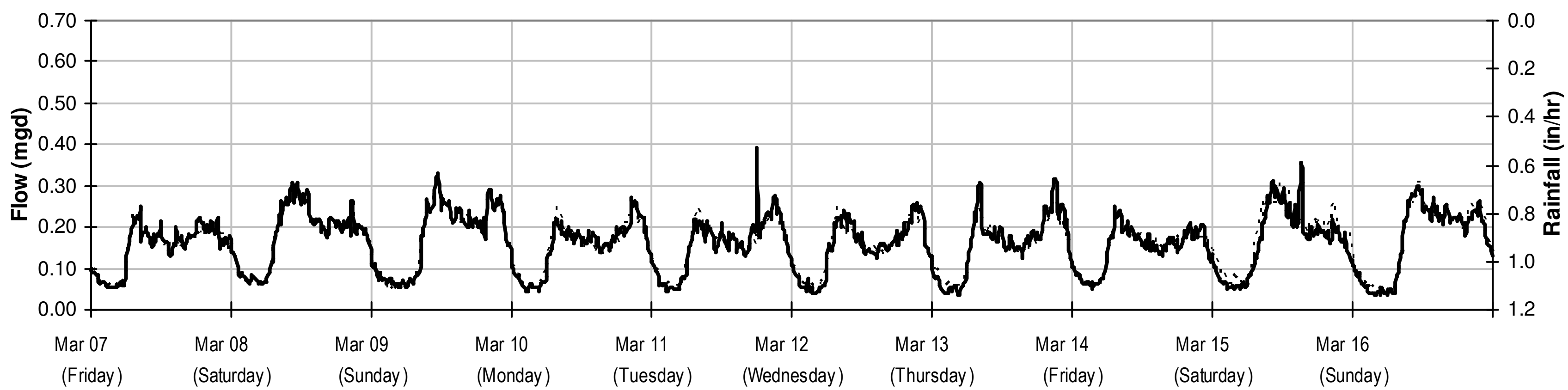
Total Period Rainfall: 3.20 inches



SITE 8
Flow Summary: 2/25/2014 to 3/17/2014

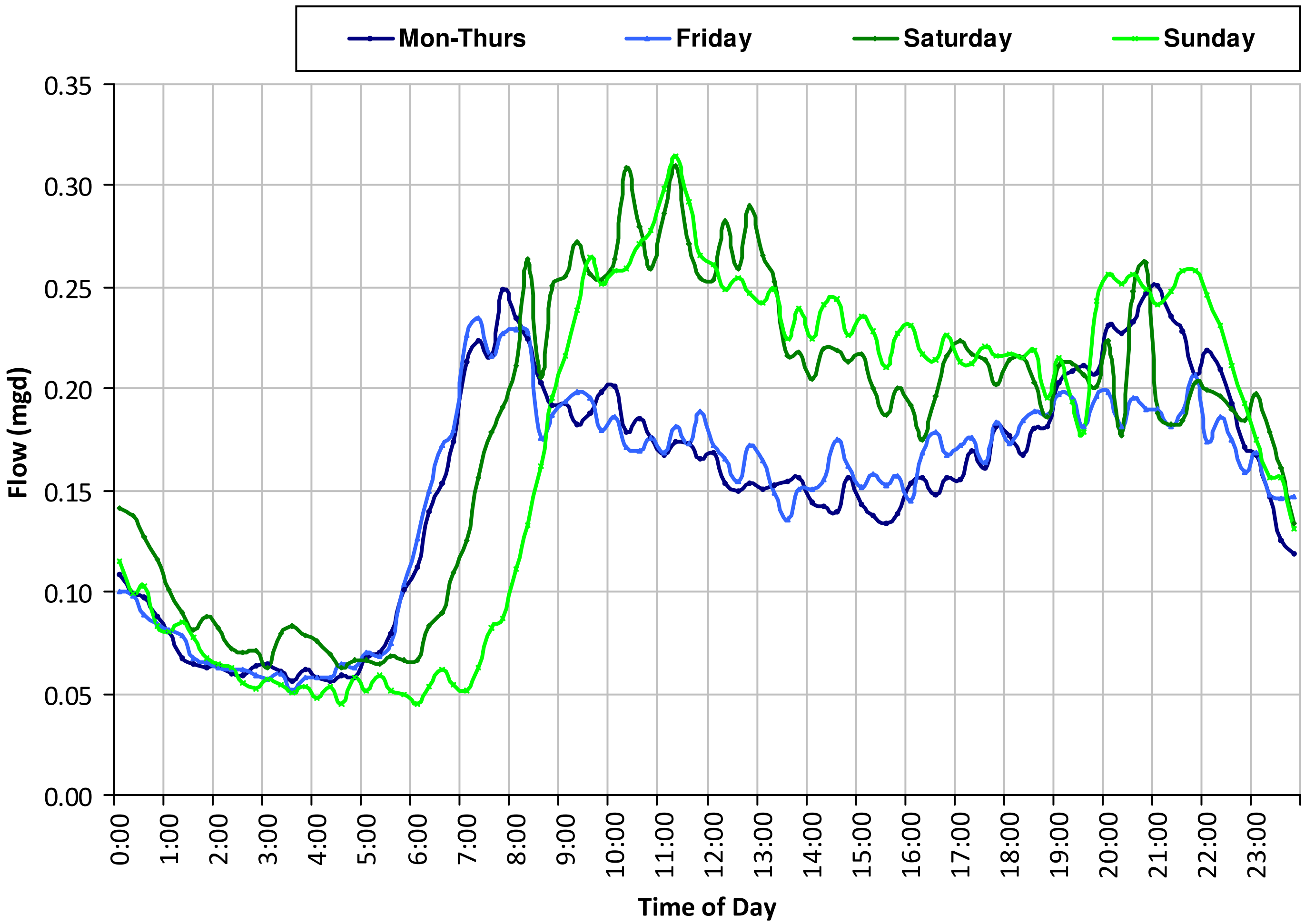
Total Period Rainfall: 3.20 inches   Avg Flow: 0.162 mgd   Peak Flow: 0.435 mgd   Min Flow: 0.034 mgd





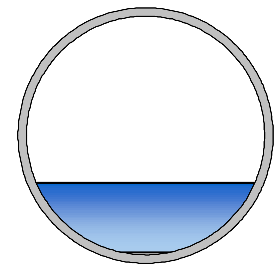
SITE 8

Baseline Flow Hydrographs



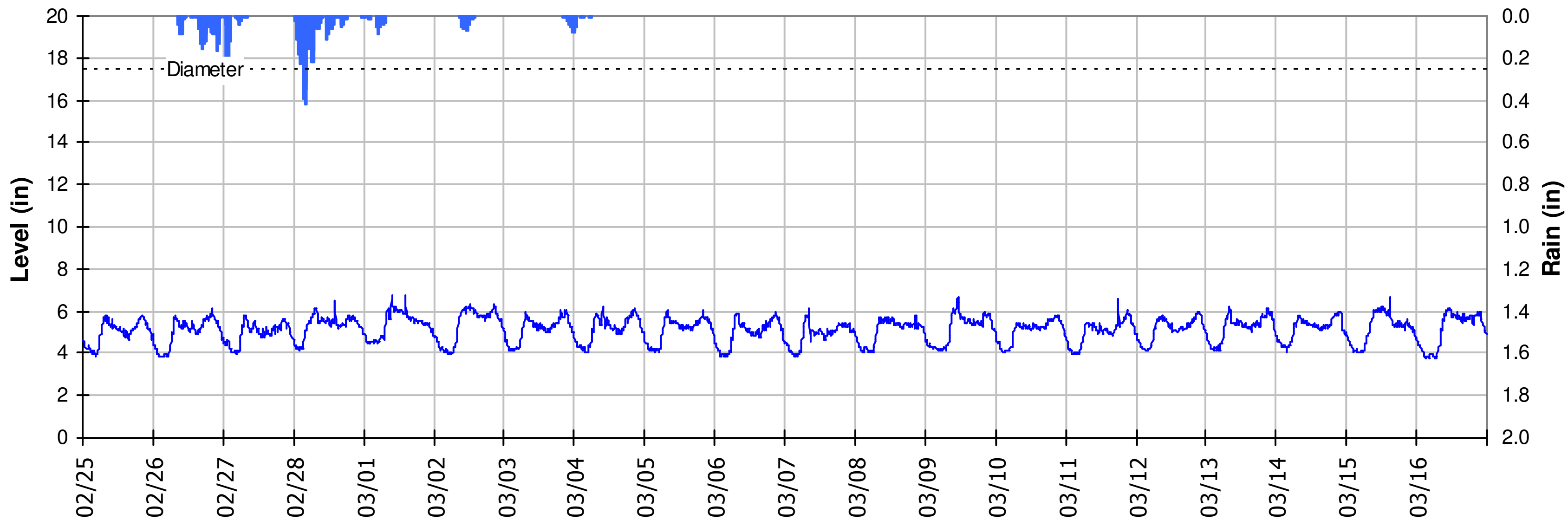
Baseline Flow:

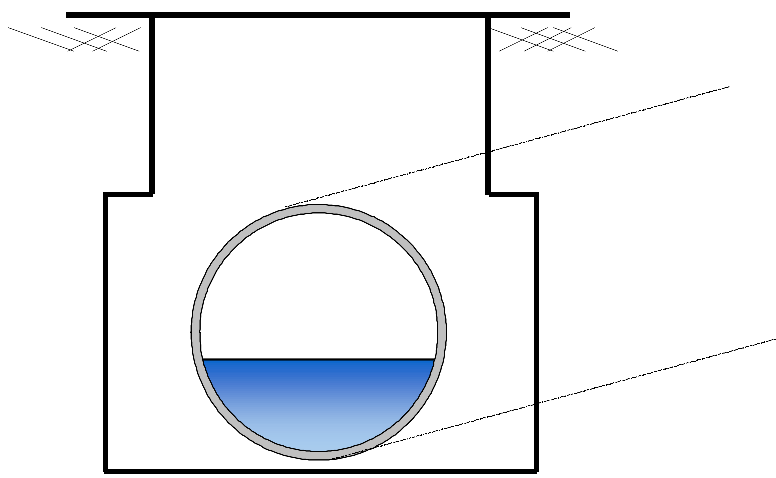
0.159 *mgd*



SITE 8
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





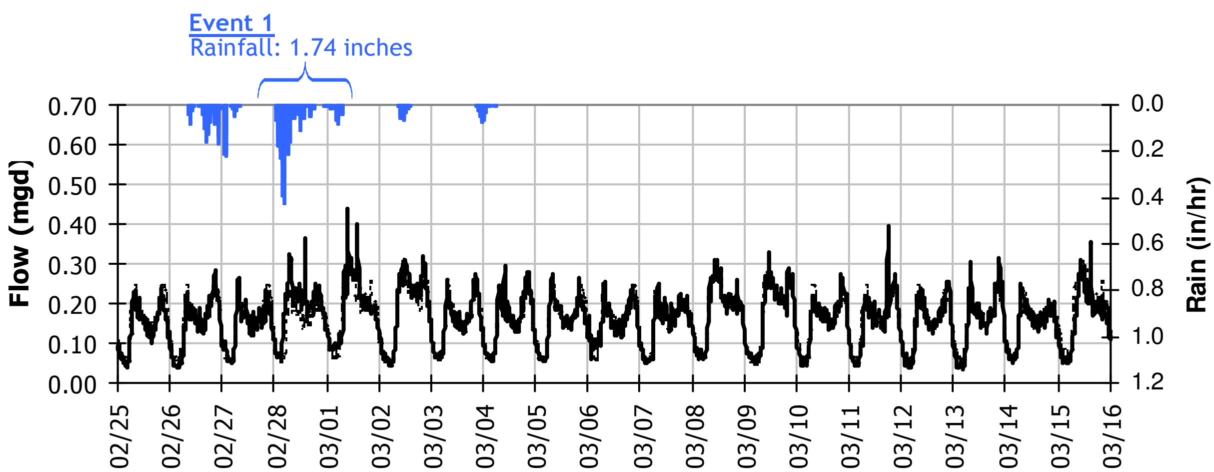
Pipe Diameter: 17.5 *inches*

Peak Measured Level: 6.76 *inches*

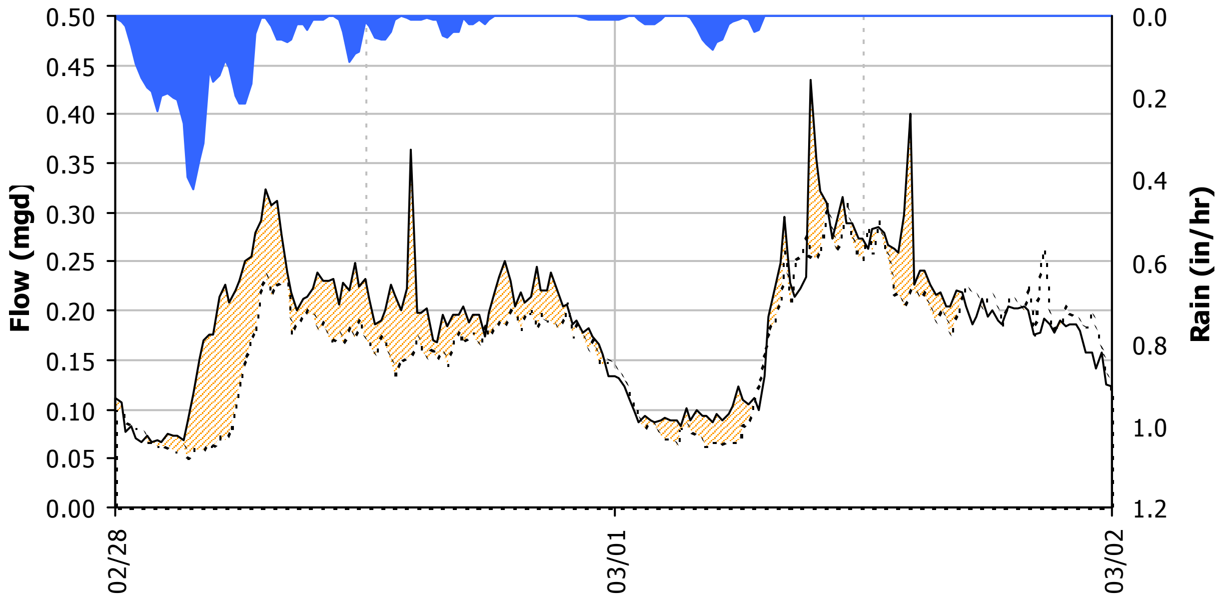
Peak d/D Ratio: 0.39

SITE 8
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



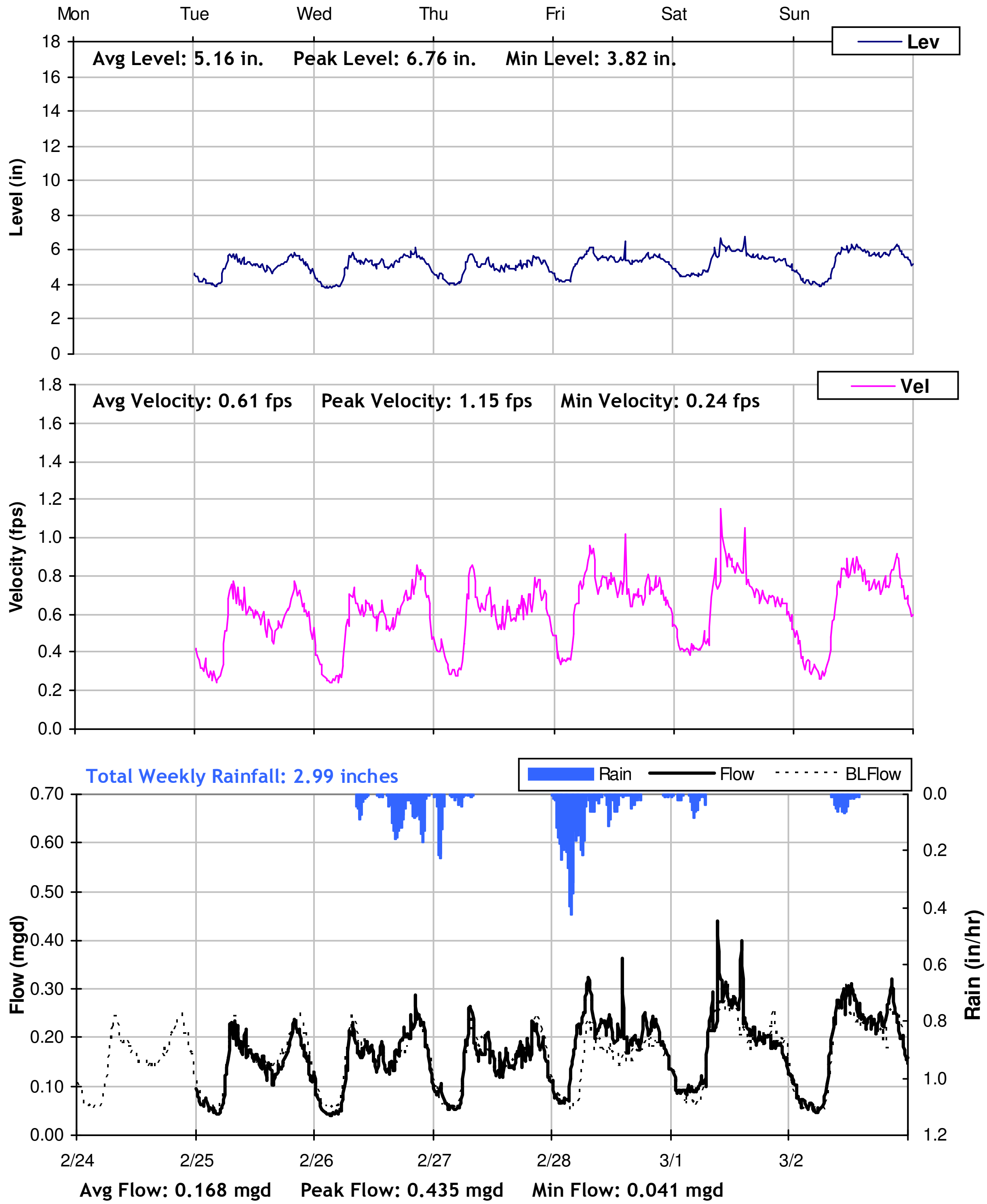
Event 1 Detail Graph



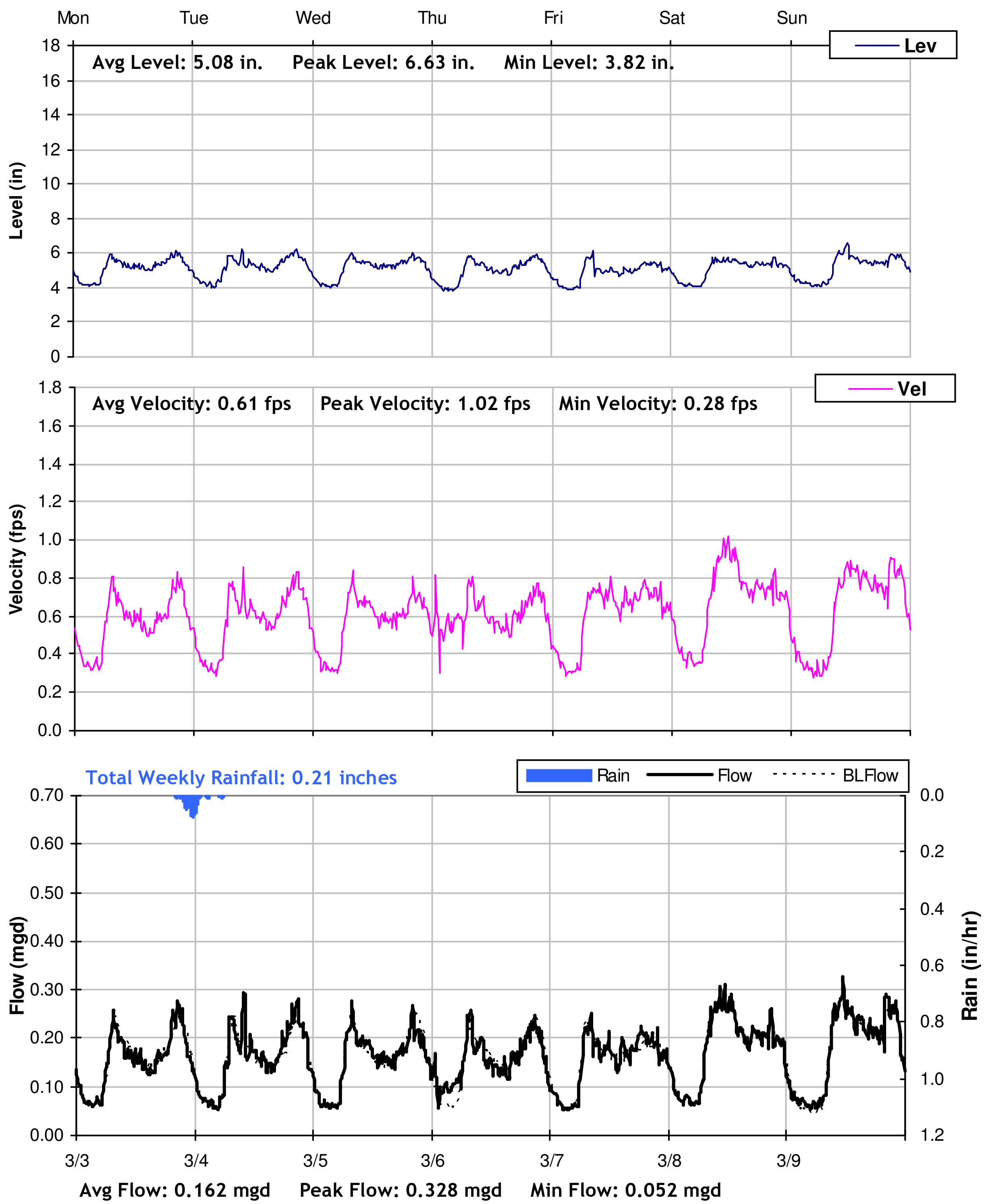
Storm Event I/I Analysis (Rain = 1.74 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.43 *mgd*	**Peak I/I Rate:**	0.21 *mgd*
PF:	2.73	**Total I/I:**	53,000 *gallons*

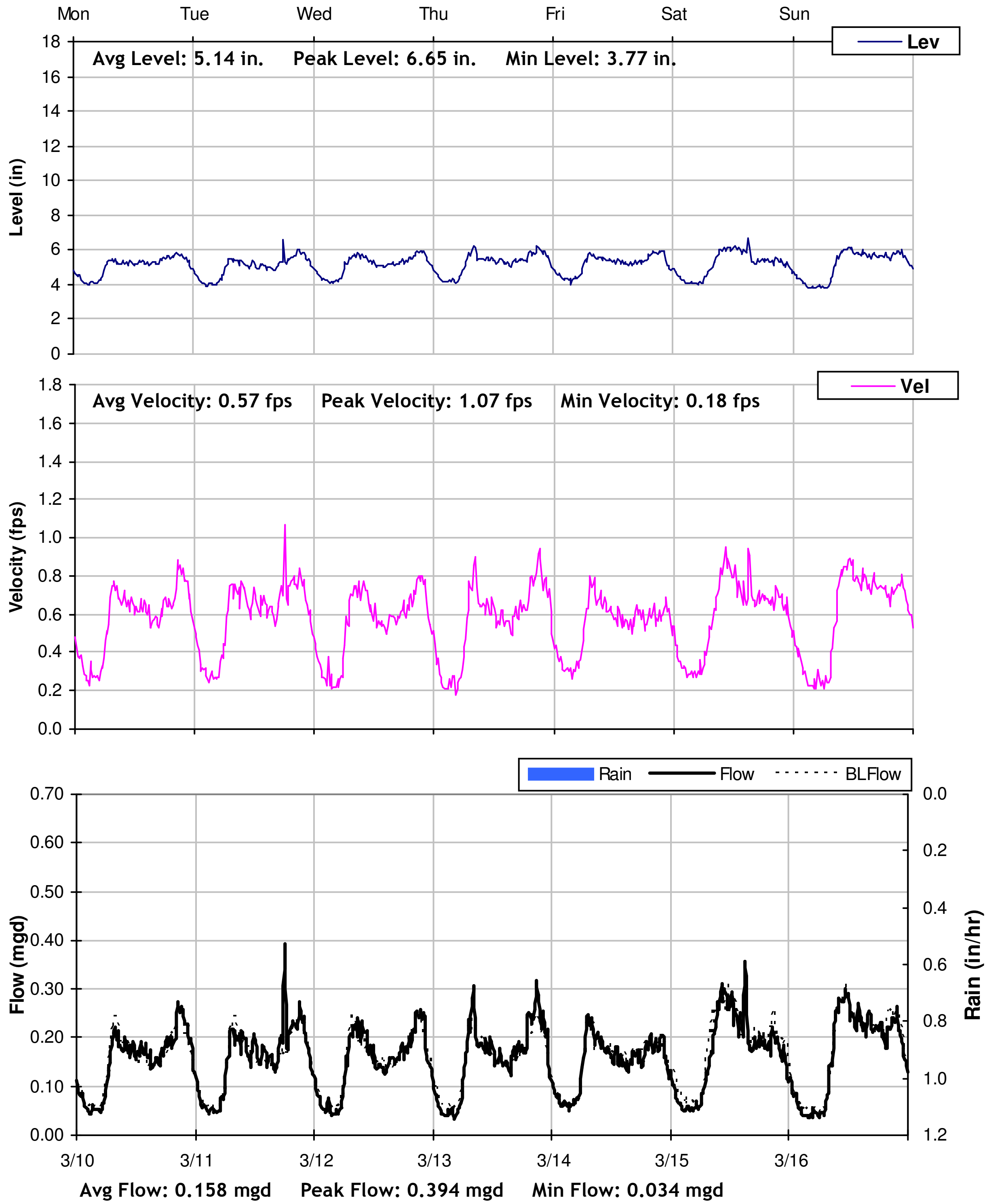
SITE 8
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



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



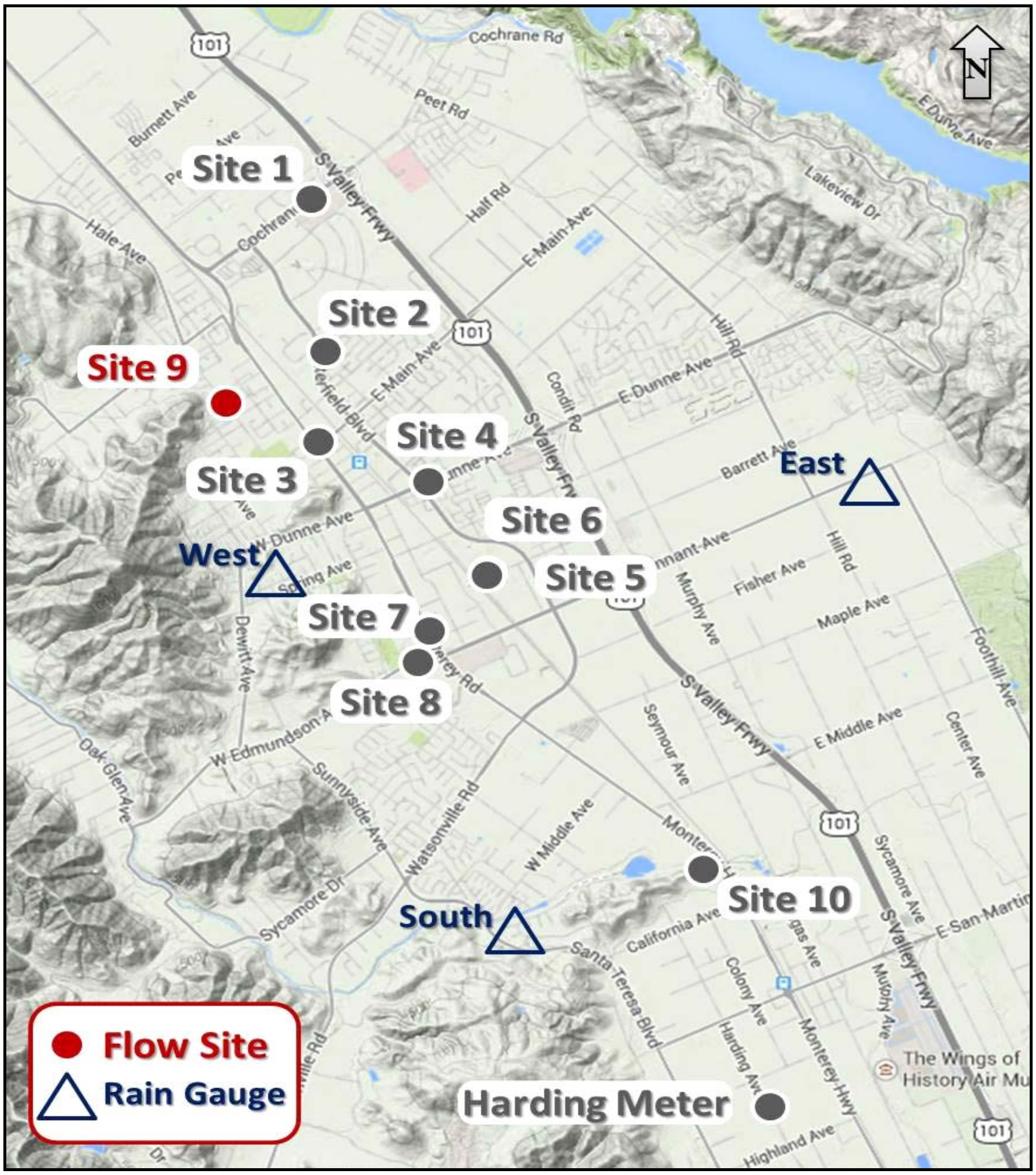
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 9

Location: Hale Avenue, north of Wright Avenue

Data Summary Report



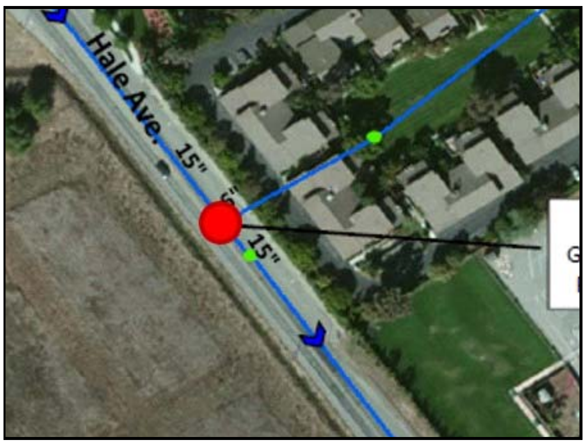
Vicinity Map: Site 9

SITE 9

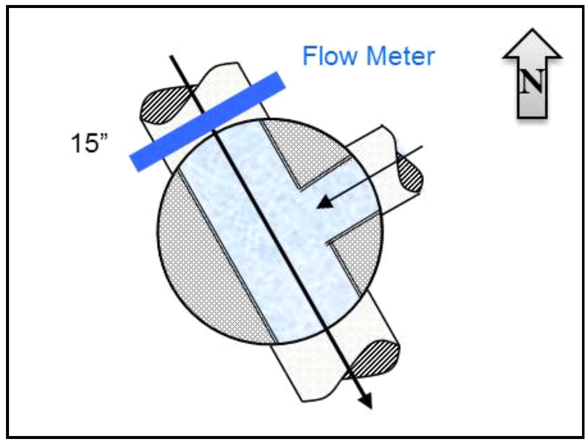
Site Information

Location:	Hale Avenue, north of Wright Avenue
Coordinates:	121.6633° W, 37.1340° N
Rim Elevation:	354 feet
Pipe Diameter:	15 inches
Baseline Flow:	0.135 mgd
Peak Measured Flow:	0.408 mgd

Satellite Map



Sanitary Map



Flow Sketch

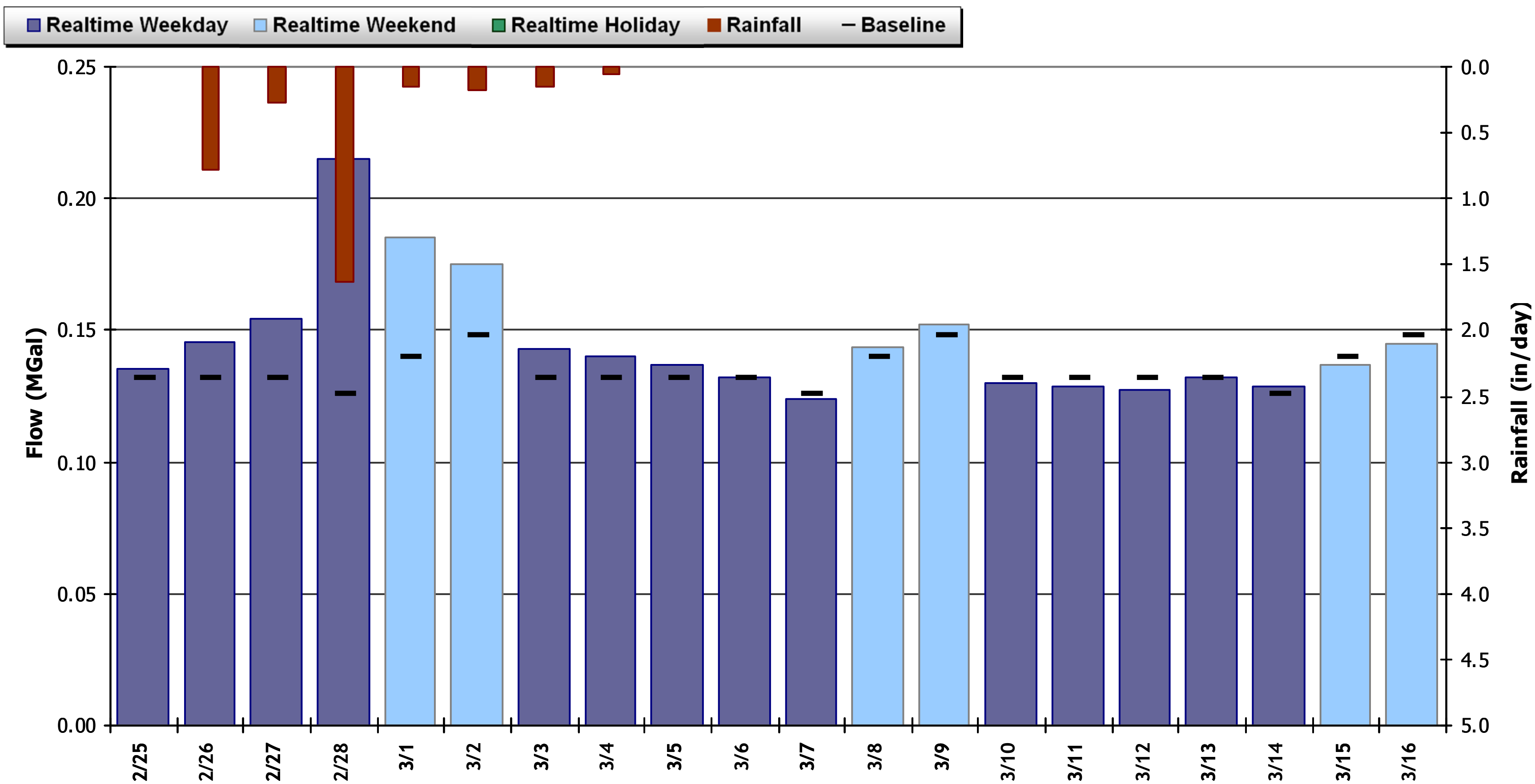
Street View

Plan View

SITE 9
Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.146 MGal **Peak Daily Flow: 0.215 MGal** **Min Daily Flow: 0.124 MGal**

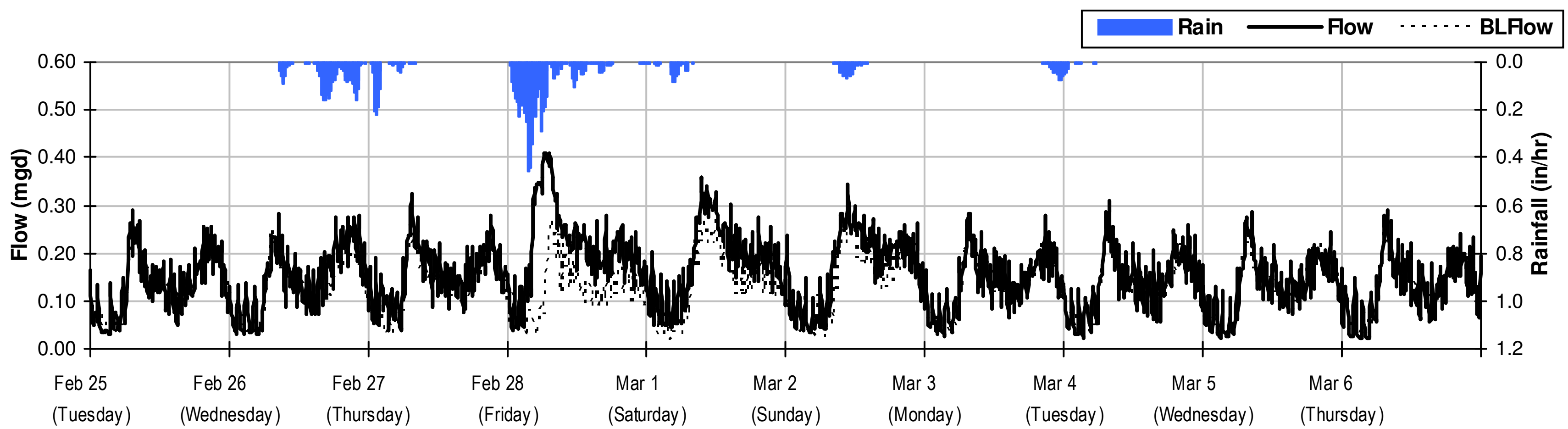
Total Period Rainfall: 3.21 inches

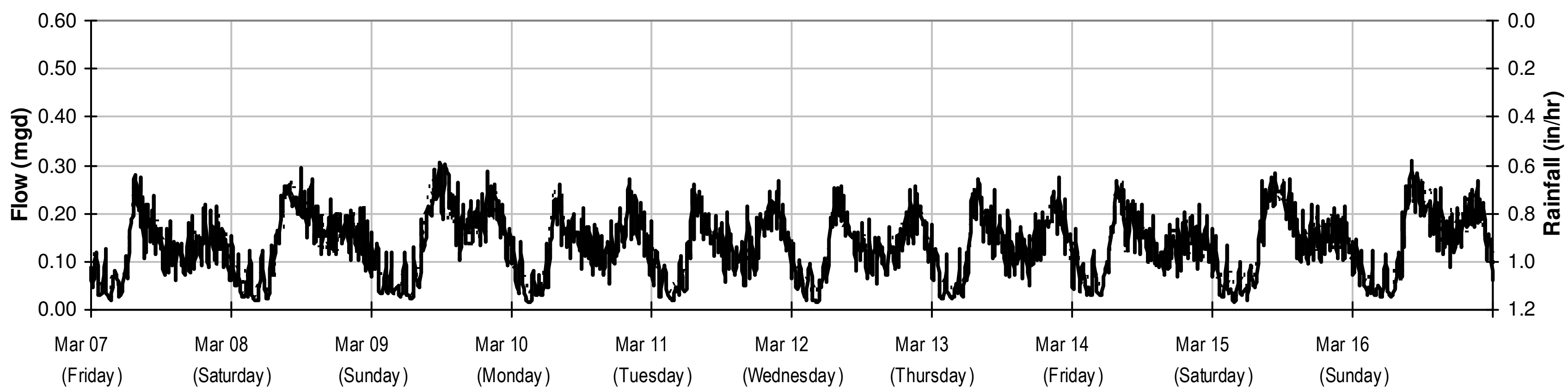


SITE 9

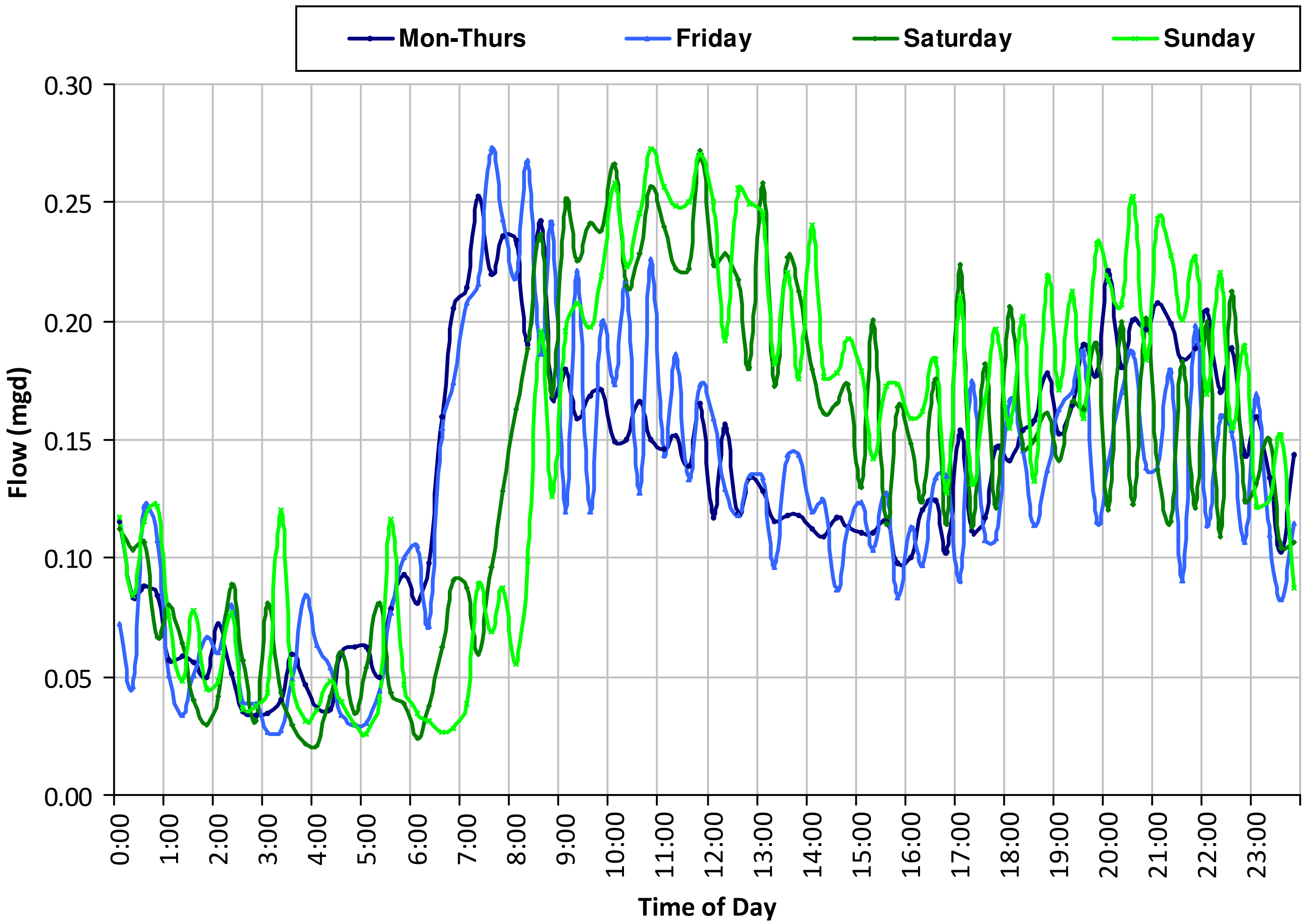
Flow Summary: 2/25/2014 to 3/17/2014

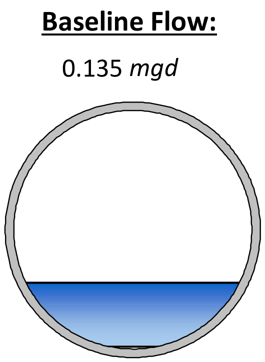
Total Period Rainfall: 3.21 inches **Avg Flow: 0.146 mgd** **Peak Flow: 0.408 mgd** **Min Flow: 0.015 mgd**





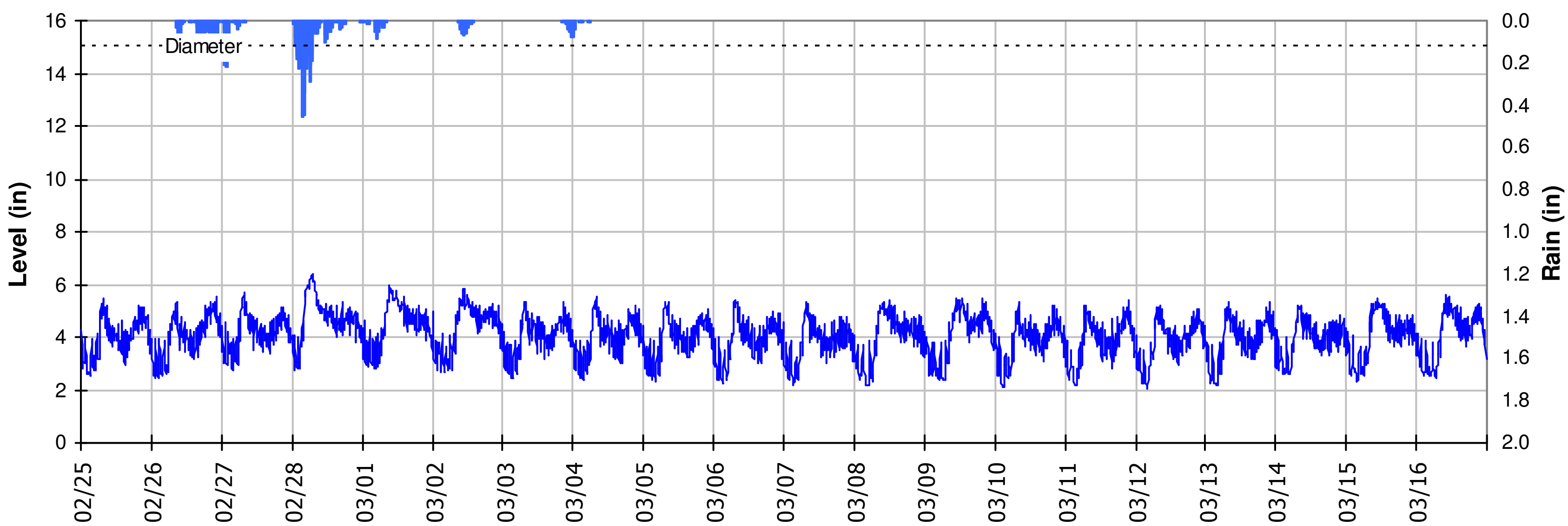
SITE 9
Baseline Flow Hydrographs

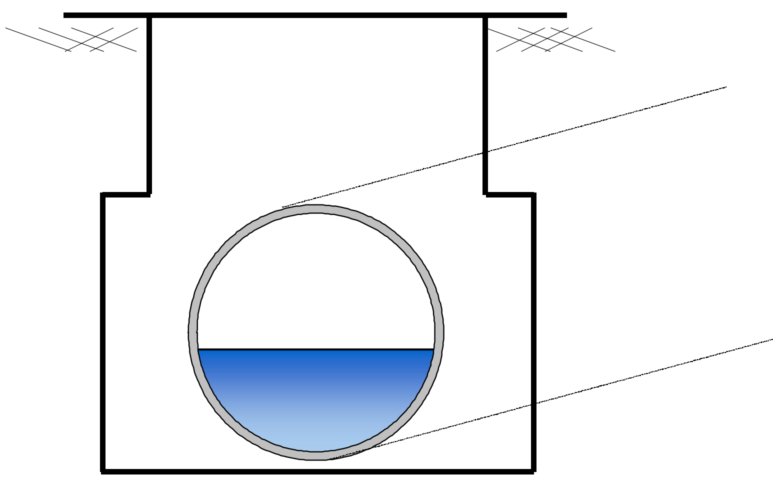




SITE 9
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

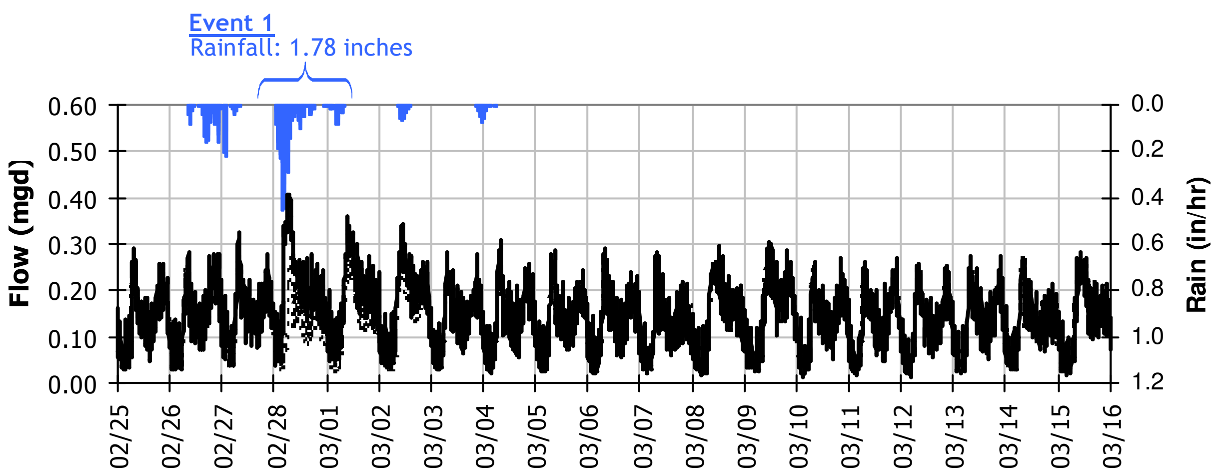




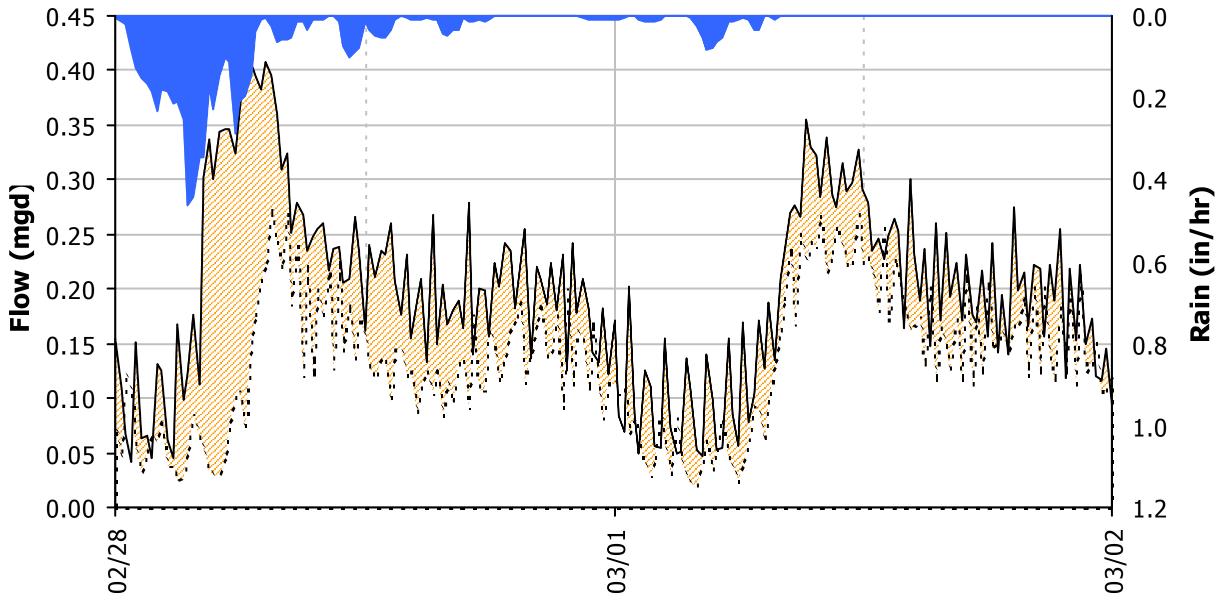
Pipe Diameter:	15 *inches*
Peak Measured Level:	6.36 *inches*
Peak d/D Ratio:	0.42

SITE 9
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



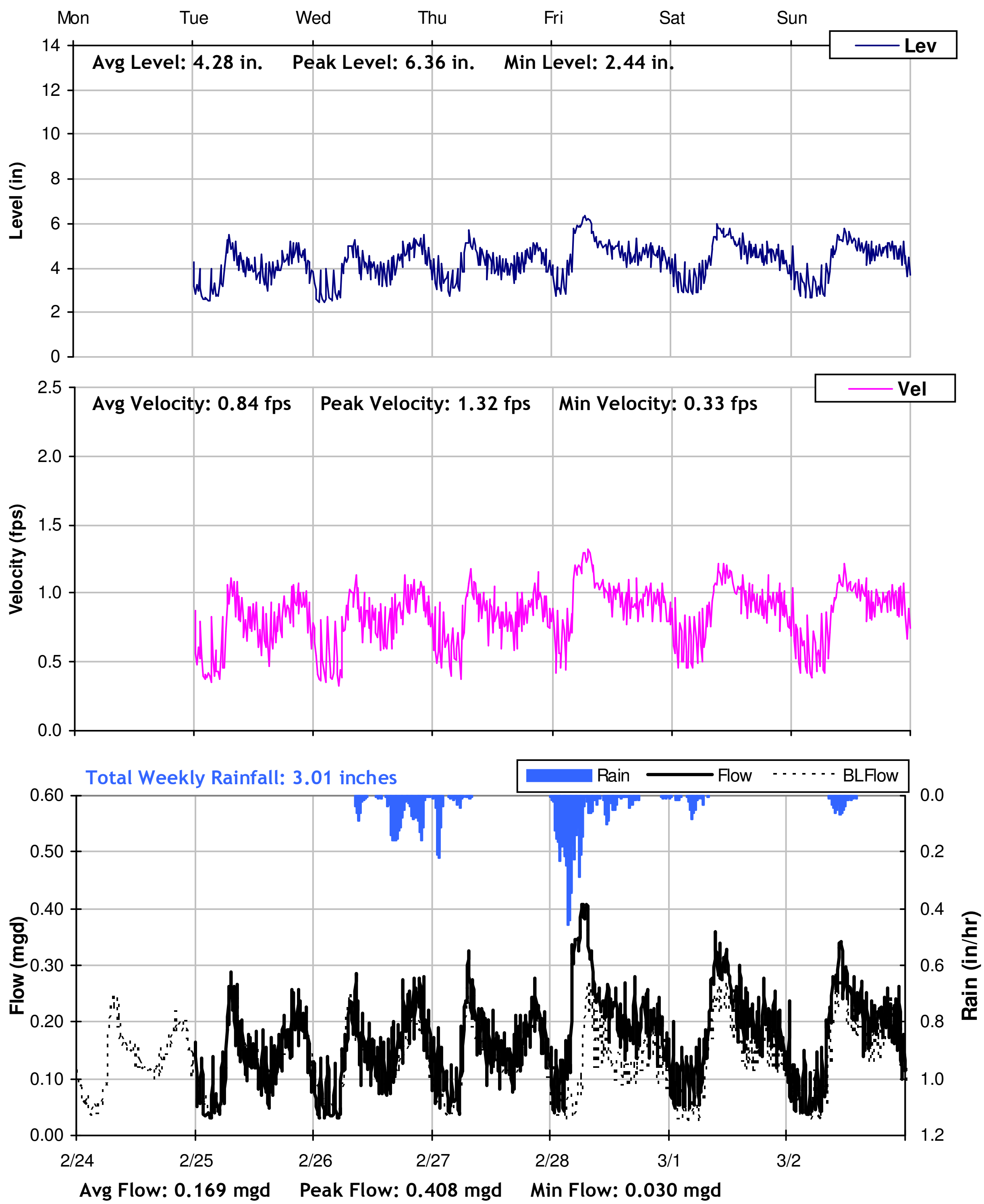
Event 1 Detail Graph



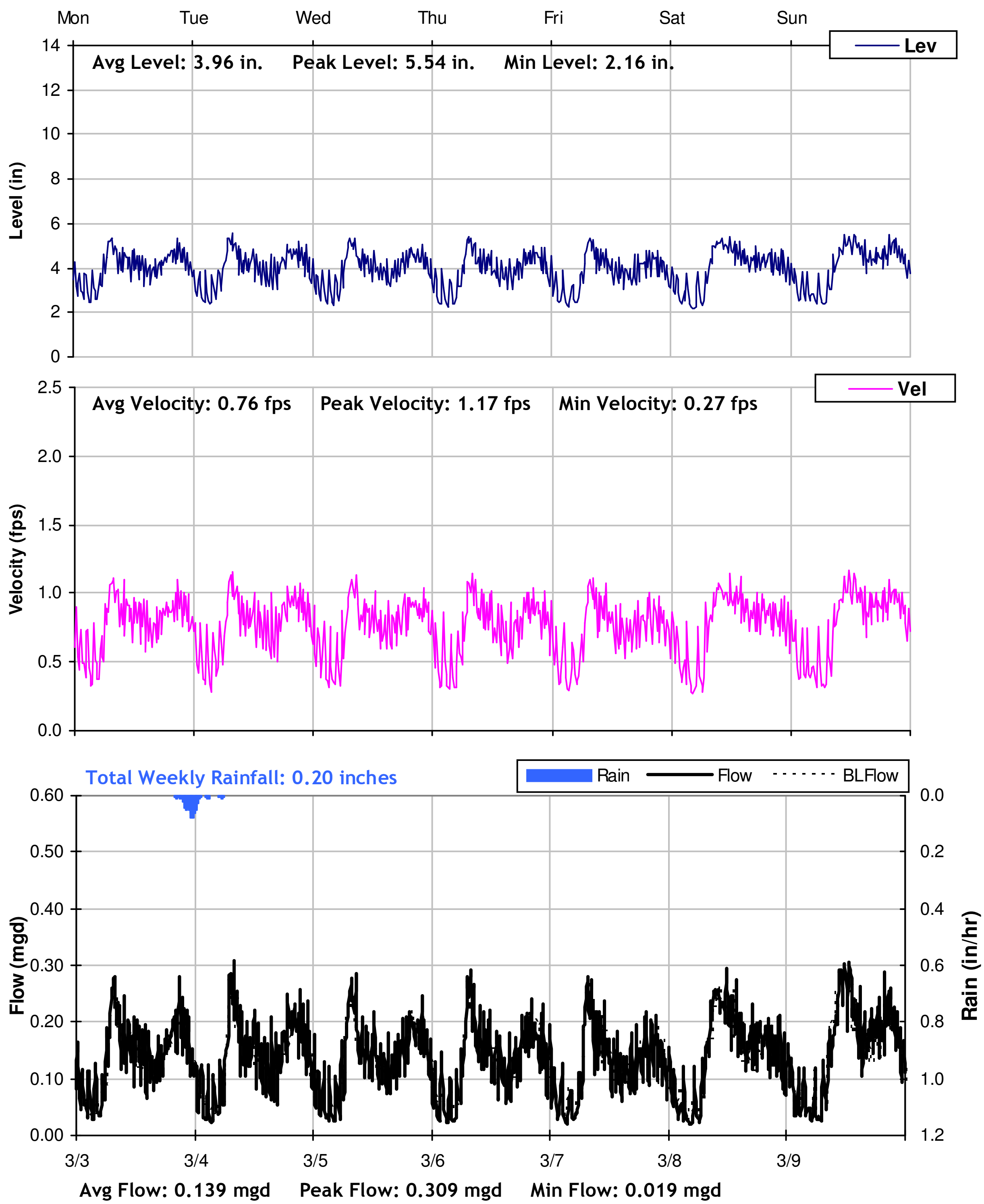
Storm Event I/I Analysis (Rain = 1.78 inches)

Capacity			Inflow / Infiltration		
Peak Flow:	0.41	*mgd*	**Peak I/I Rate:**	0.34	*mgd*
PF:	3.03		**Total I/I:**	133,000	*gallons*

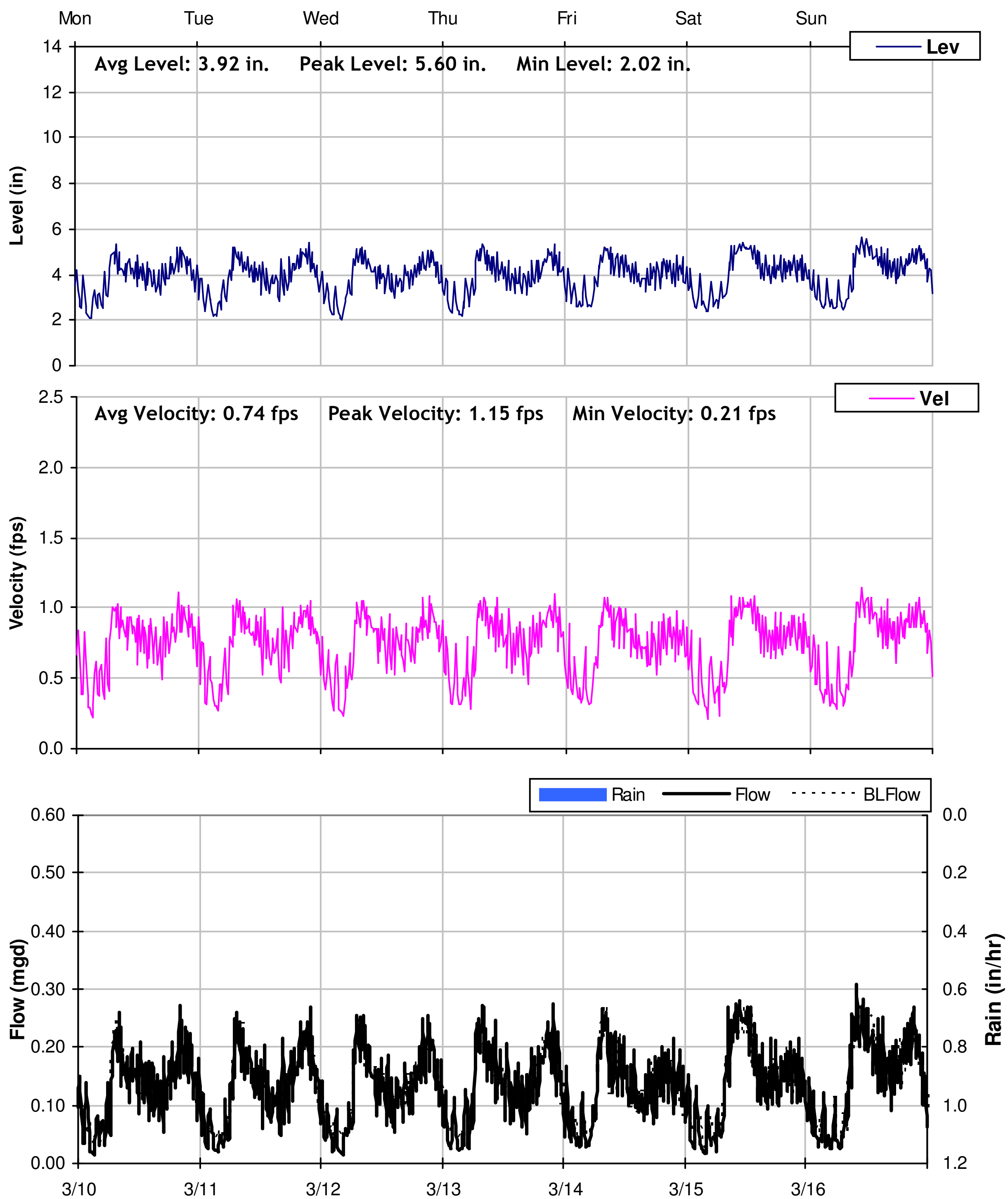
SITE 9
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



SITE 9
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



SITE 9
Weekly Level, Velocity and Flow Hydrographs
3/10/2014 to 3/17/2014



Avg Flow: 0.133 mgd Peak Flow: 0.309 mgd Min Flow: 0.015 mgd

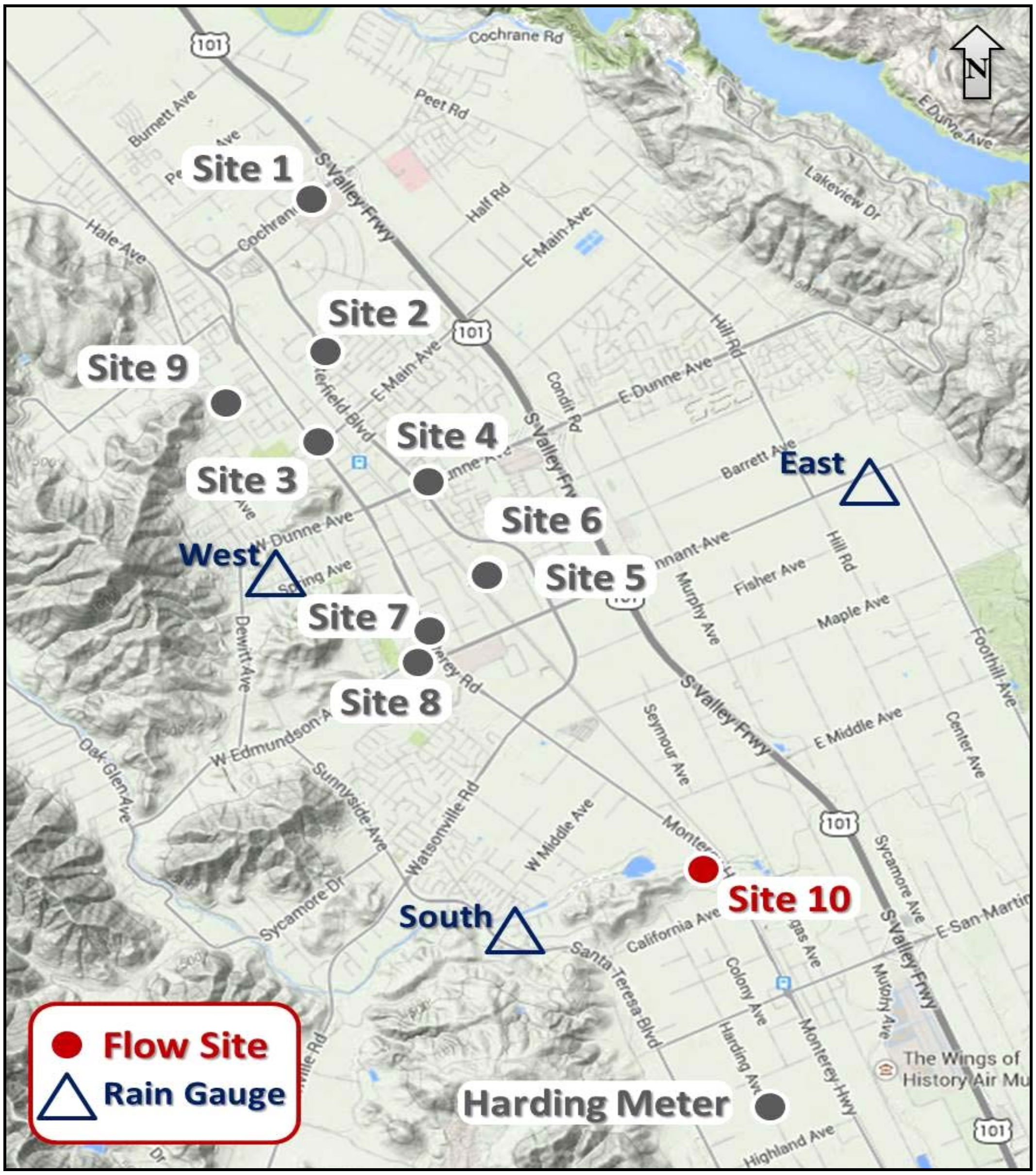
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Site 10

Location: Easement west of Monterey Road, north of California Avenue

Data Summary Report



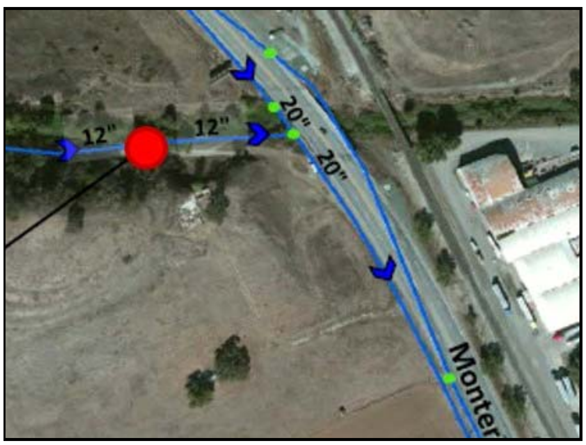
Vicinity Map: Site 10

SITE 10

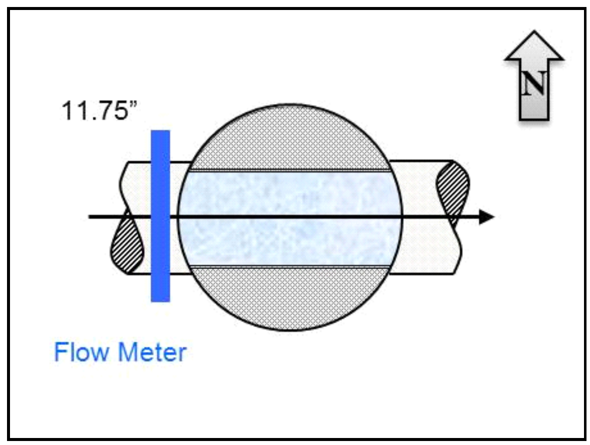
Site Information

Location:	Easement west of Monterey Road, north of California Avenue
Coordinates:	121.6174° W, 37.0955° N
Rim Elevation:	303 feet
Pipe Diameter:	11.75 inches
Baseline Flow:	0.125 mgd
Peak Measured Flow:	0.306 mgd

Satellite Map



Sanitary Map



Flow Sketch

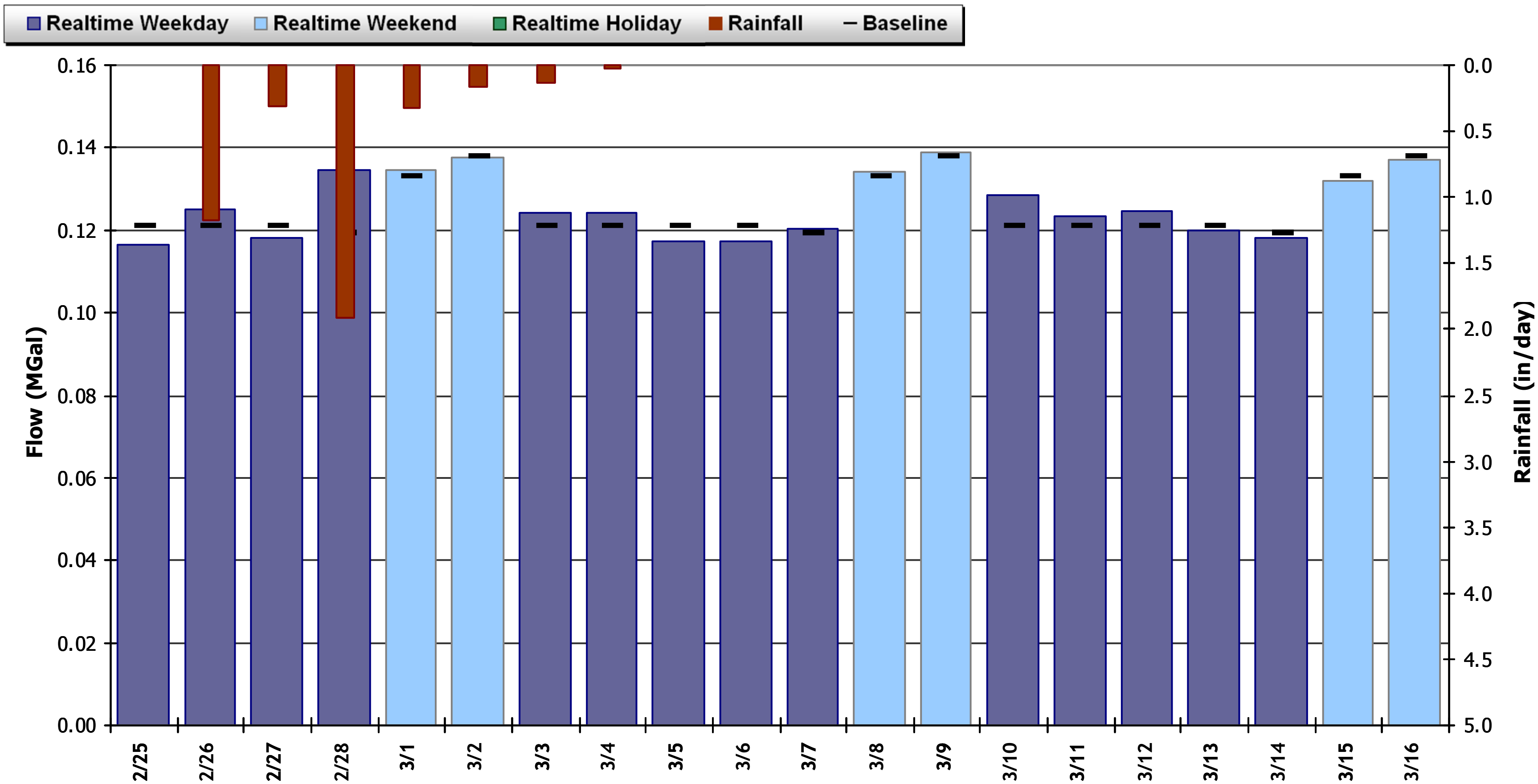

Street View


Plan View

SITE 10
Period Flow Summary: Daily Flow Totals

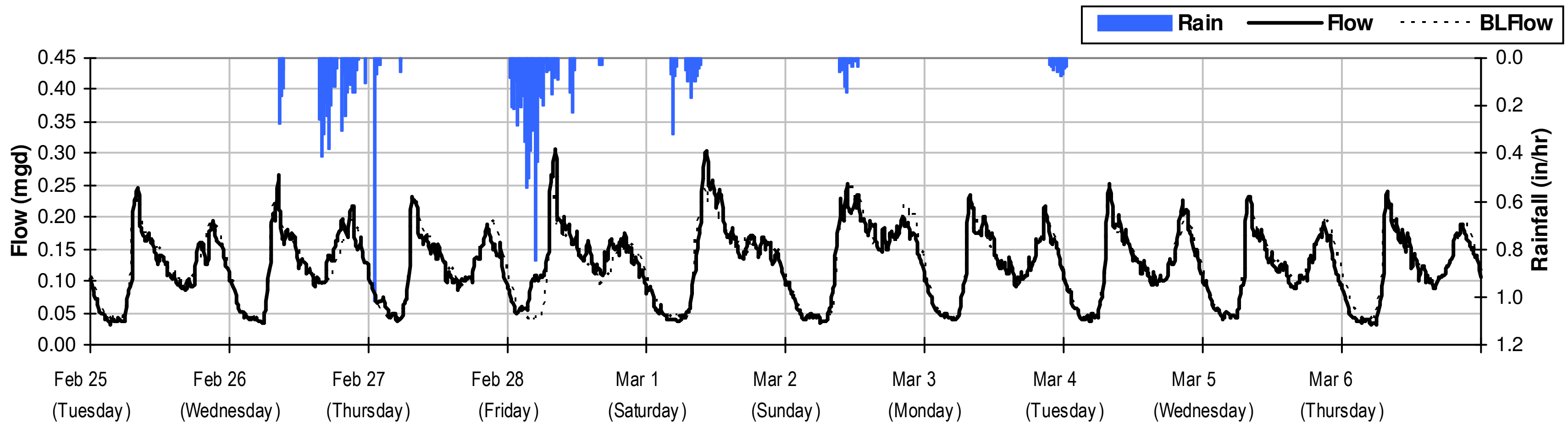
Avg Period Flow: 0.126 MGal **Peak Daily Flow: 0.139 MGal** **Min Daily Flow: 0.117 MGal**

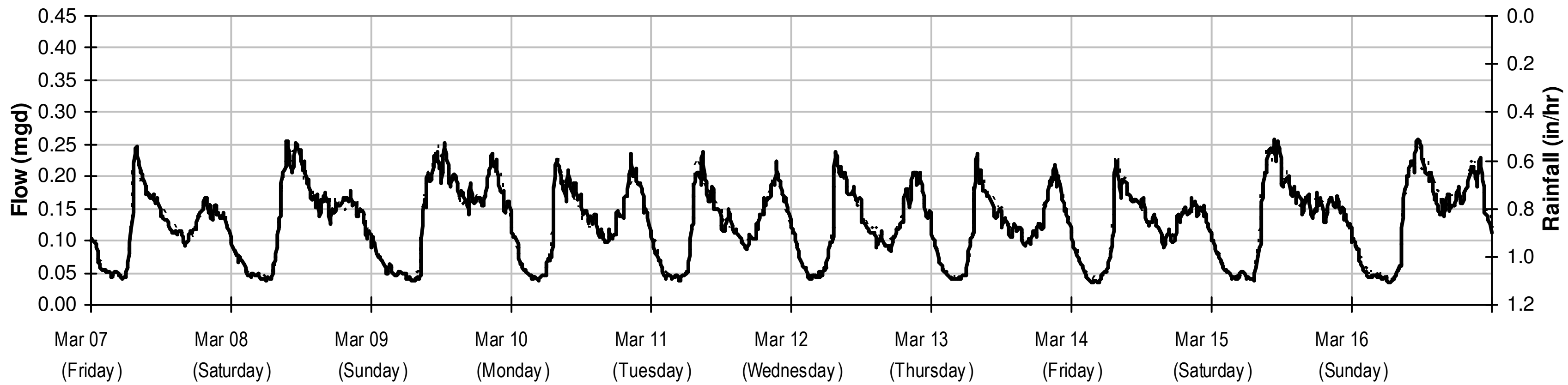
Total Period Rainfall: 4.04 inches



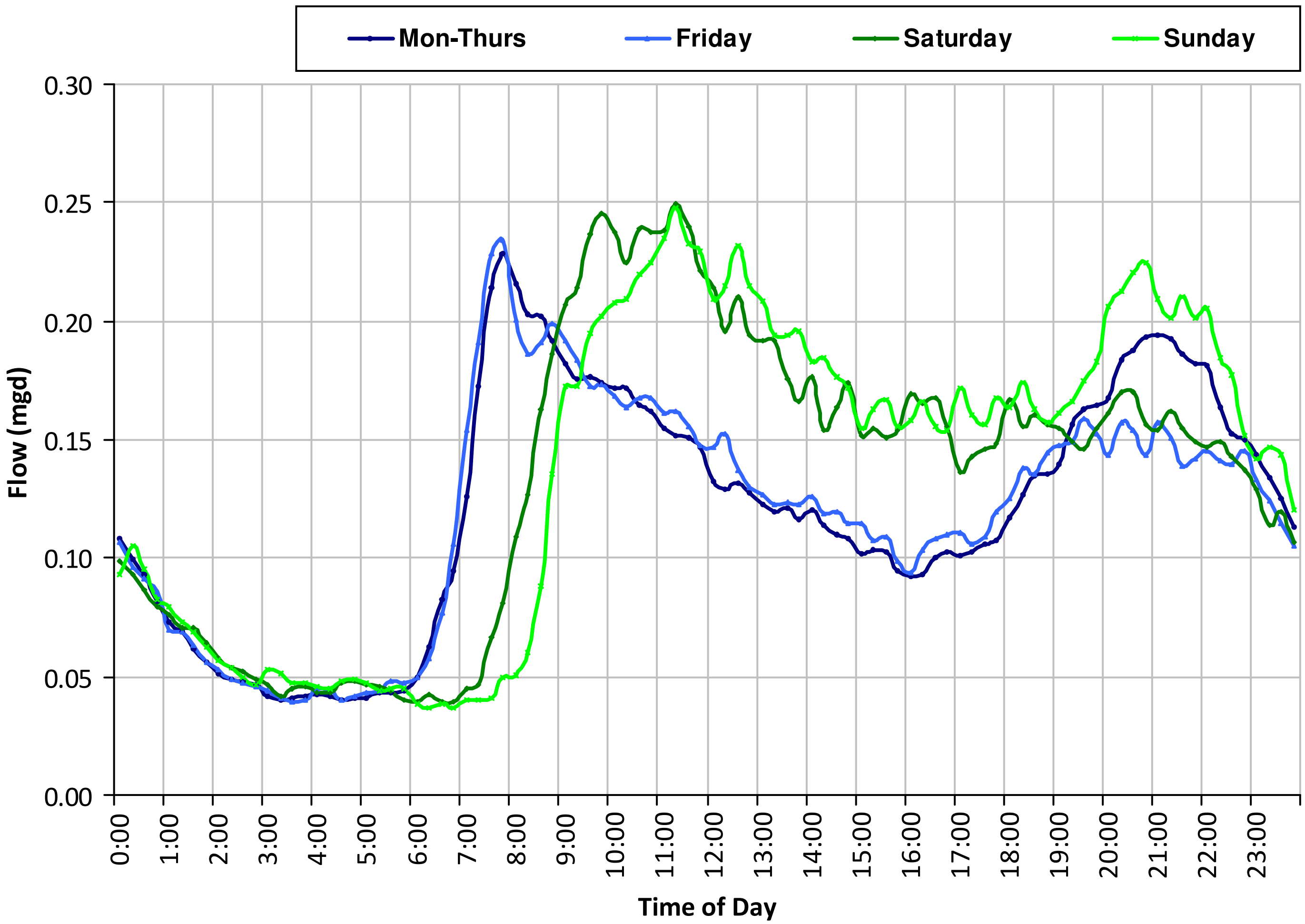
SITE 10
Flow Summary: 2/25/2014 to 3/17/2014

Total Period Rainfall: 4.04 inches   Avg Flow: 0.126 mgd   Peak Flow: 0.306 mgd   Min Flow: 0.031 mgd



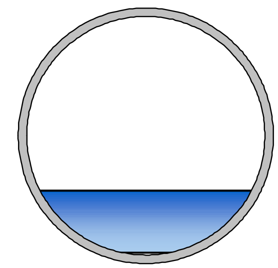


SITE 10
Baseline Flow Hydrographs



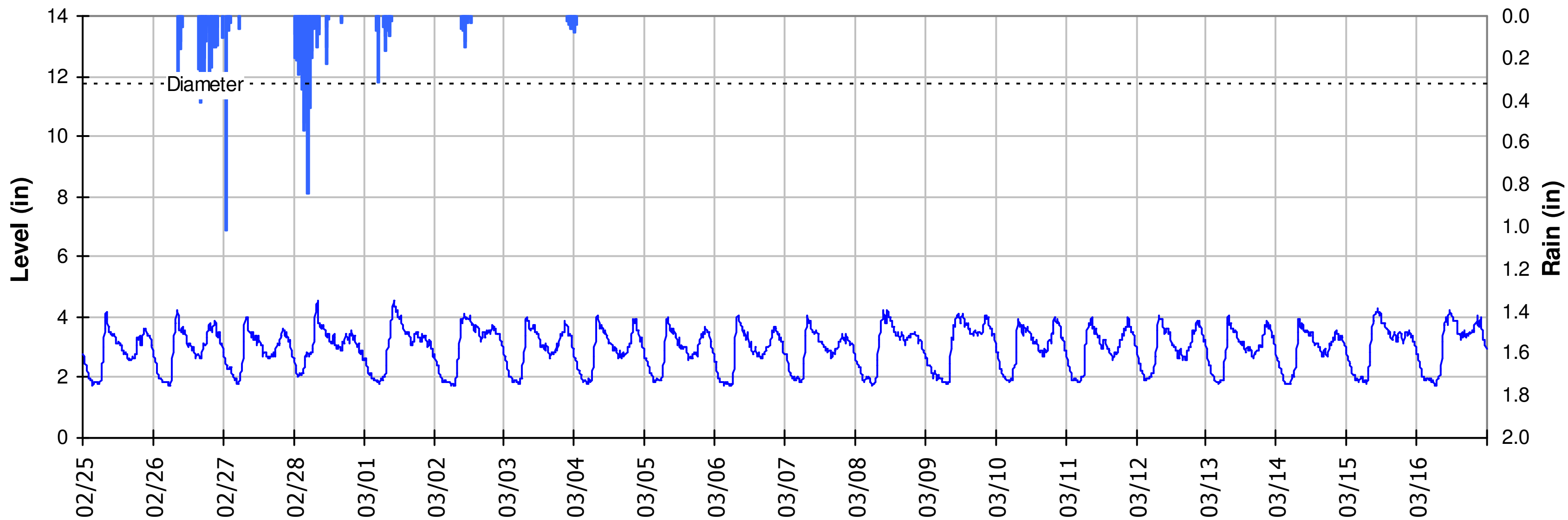
Baseline Flow:

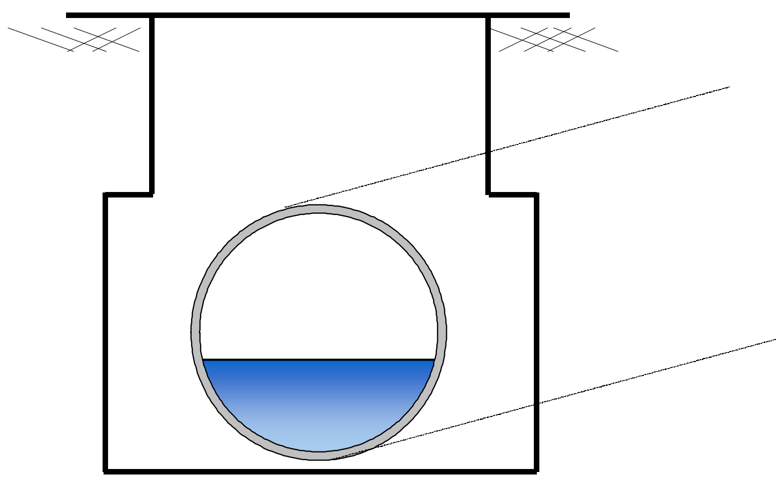
0.125 *mgd*



SITE 10
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





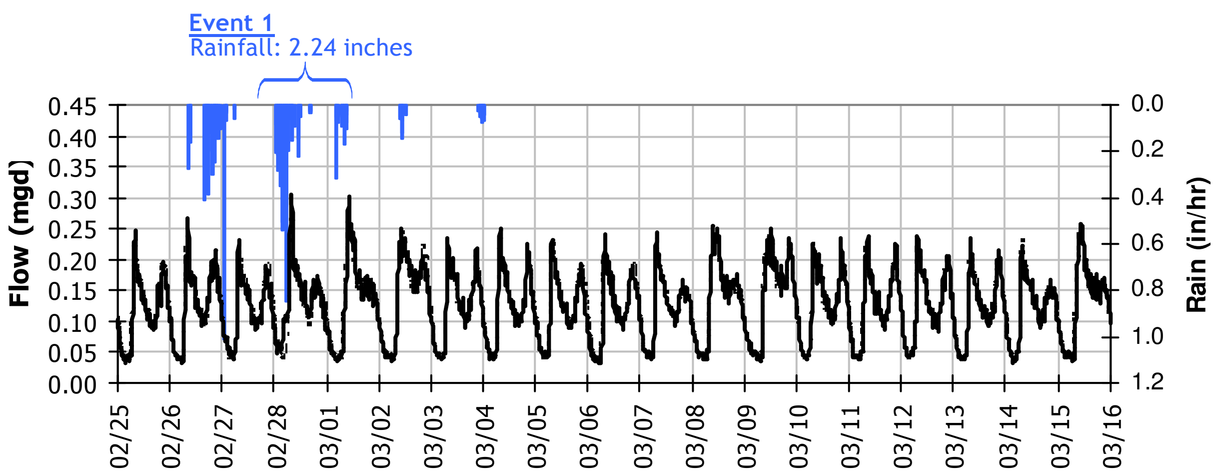
Pipe Diameter: 11.8 *inches*

Peak Measured Level: 4.57 *inches*

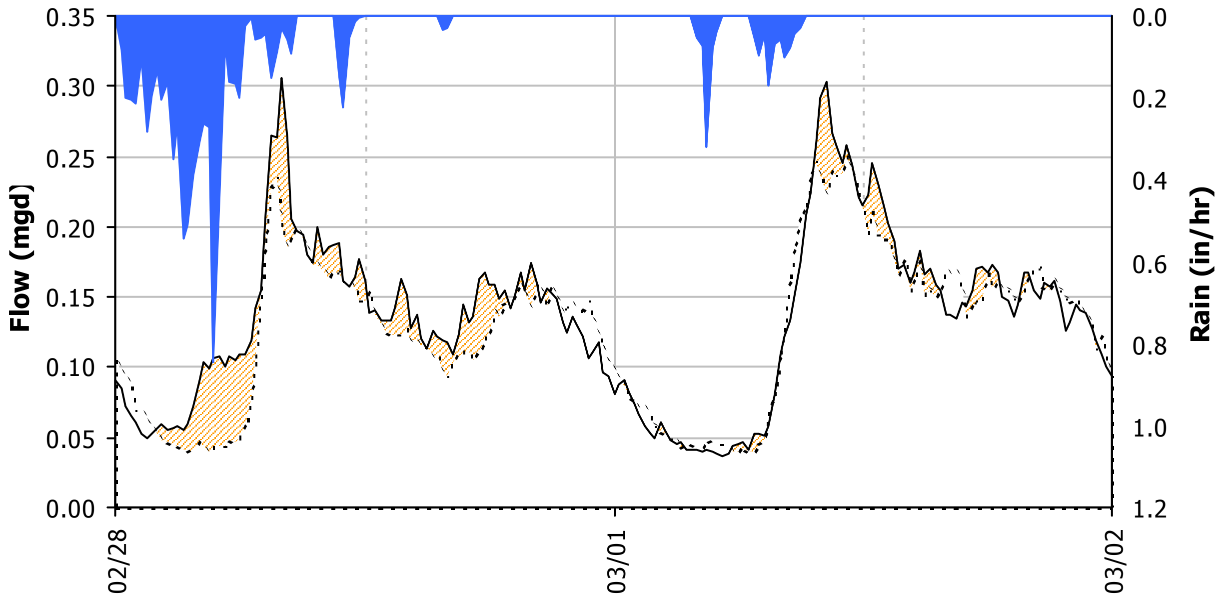
Peak d/D Ratio: 0.39

SITE 10
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



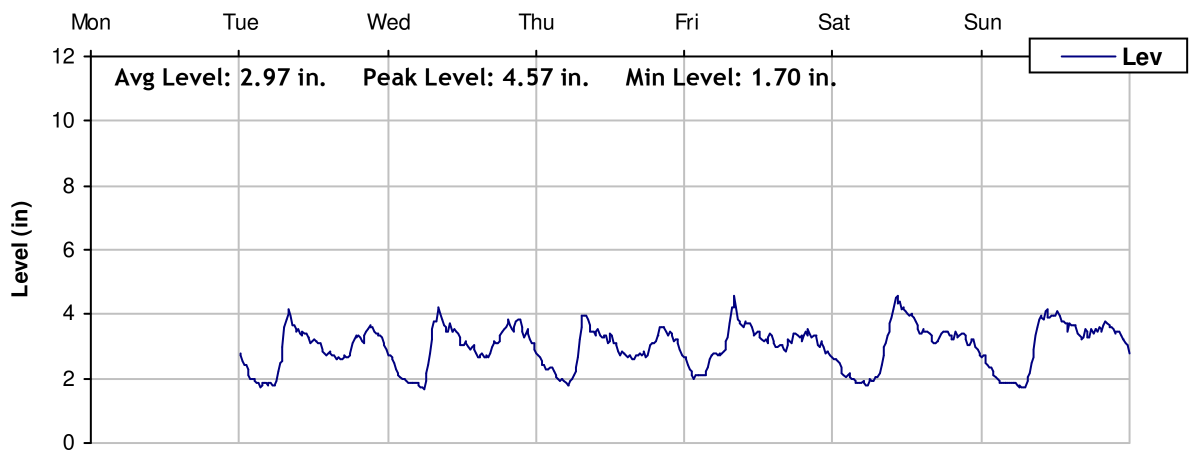
Event 1 Detail Graph

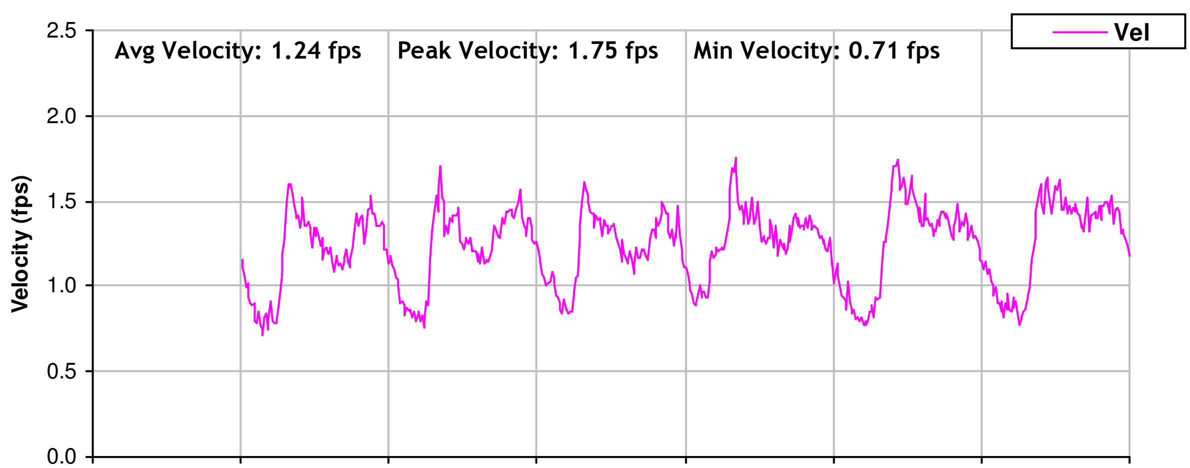


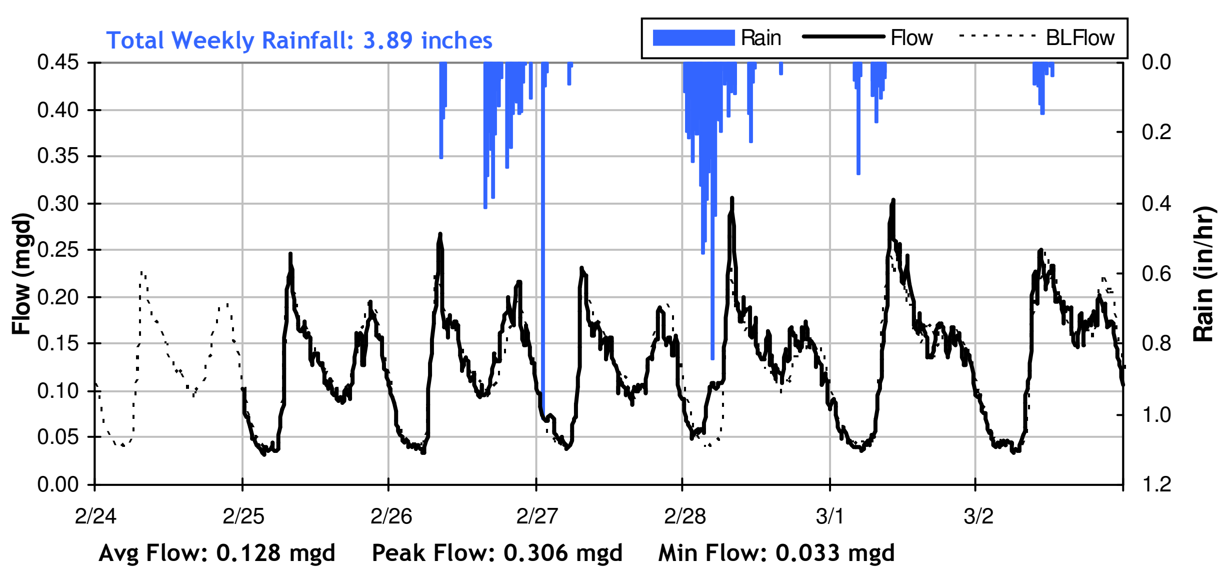
Storm Event I/I Analysis (Rain = 2.24 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	0.31 *mgd*	**Peak I/I Rate:**	0.11 *mgd*
PF:	2.45	**Total I/I:**	16,000 *gallons*

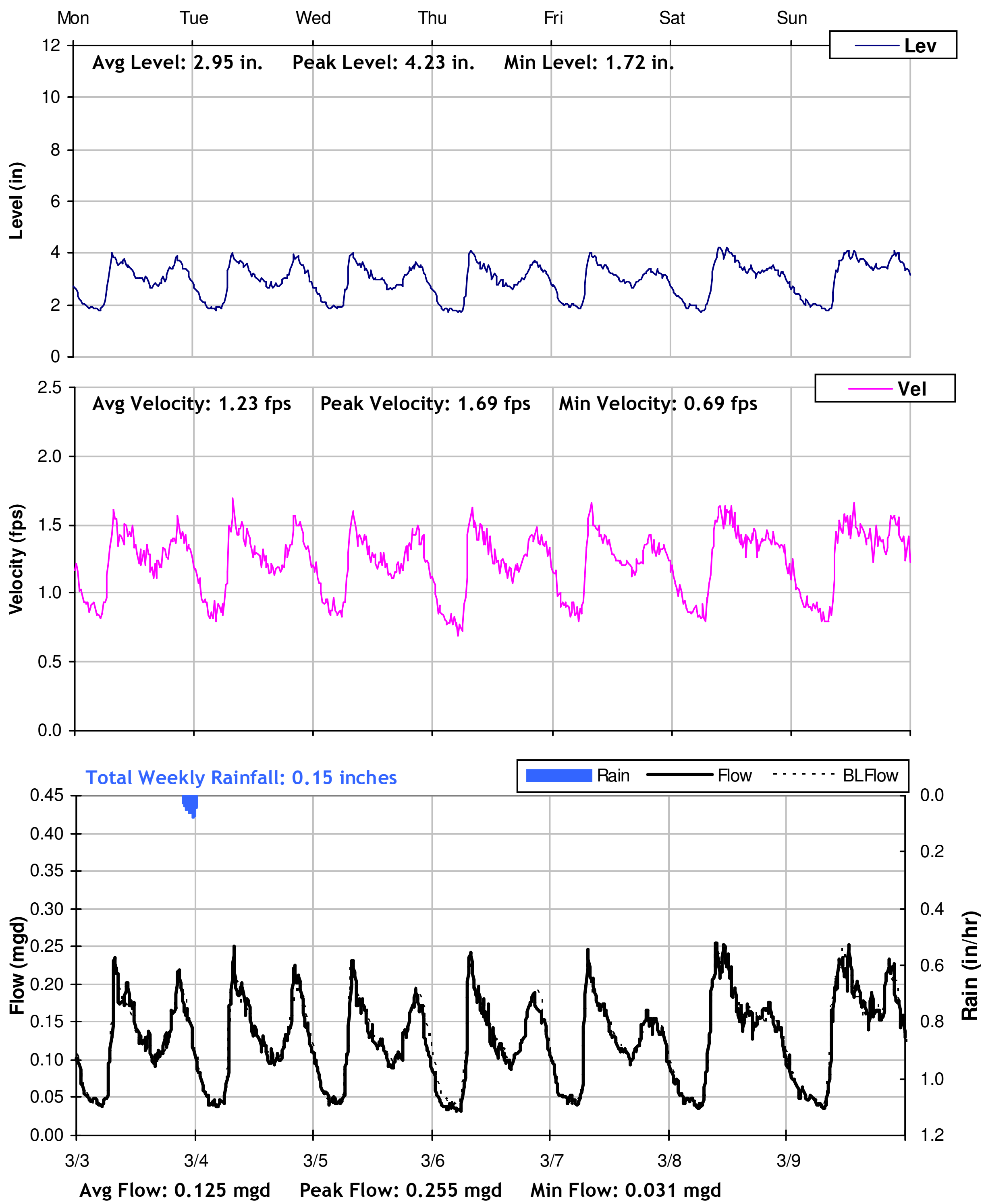
SITE 10
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



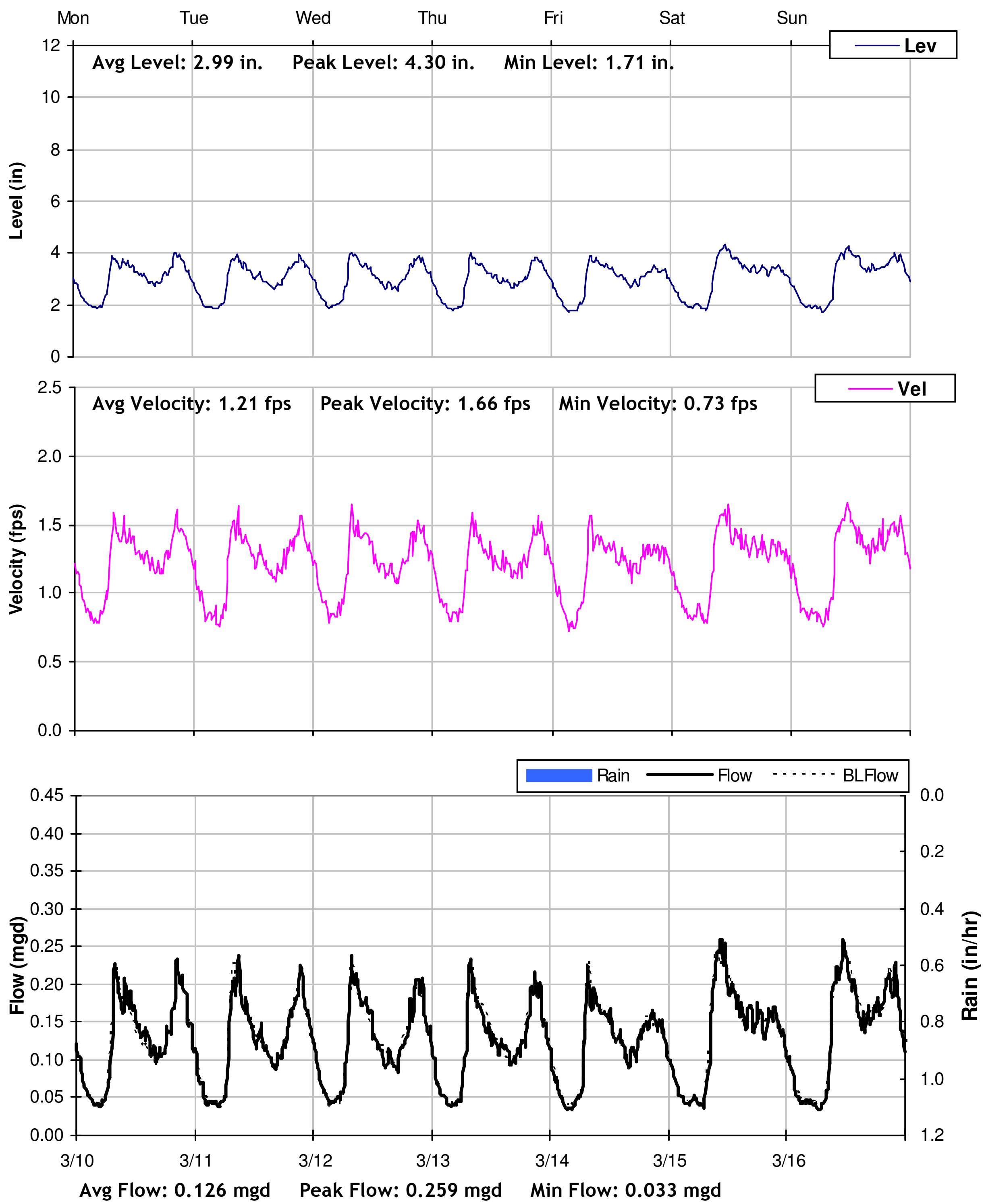




SITE 10
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



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



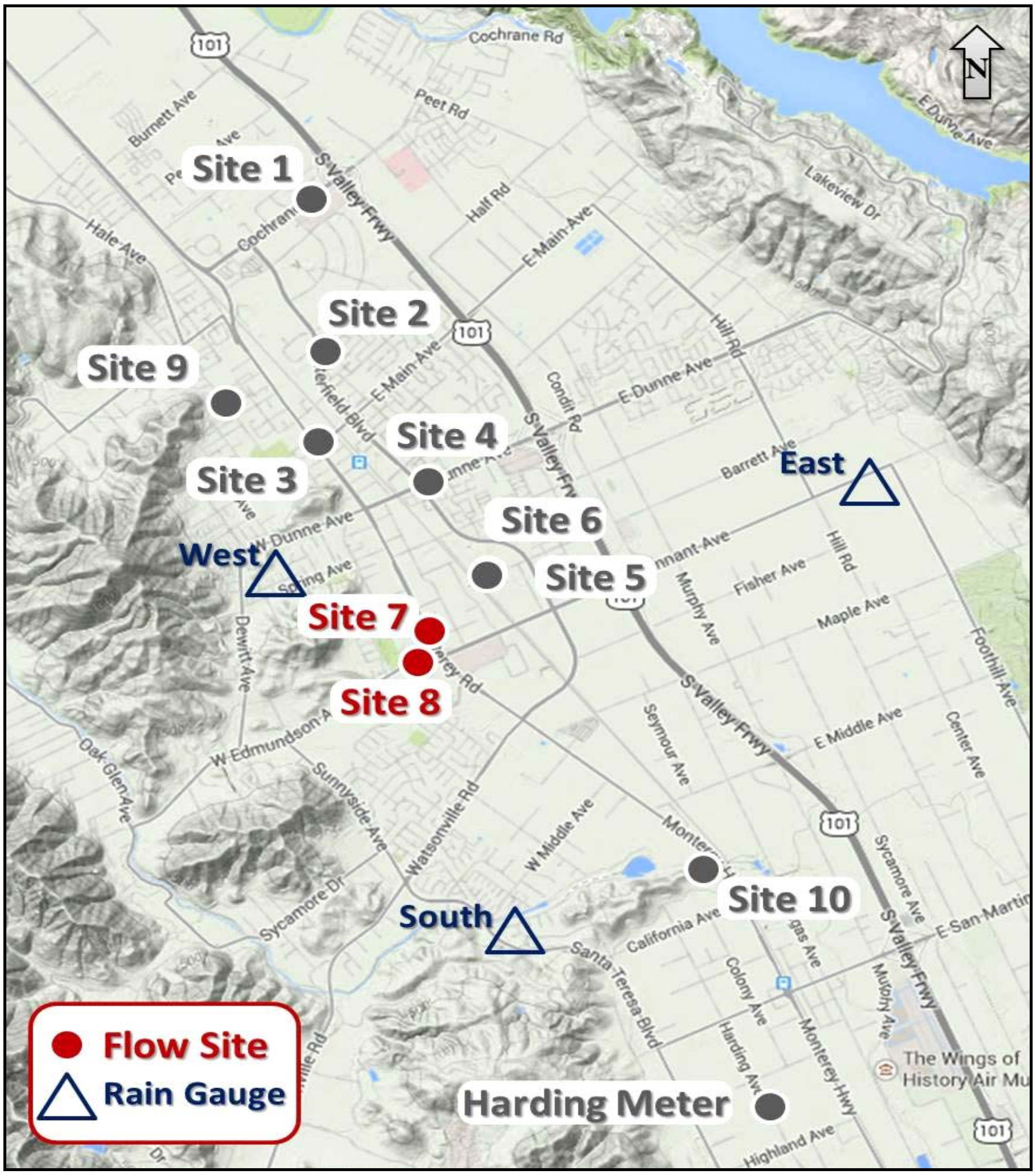
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Basin 7+8

Location: Sum of Flows from Sites 7 and 8

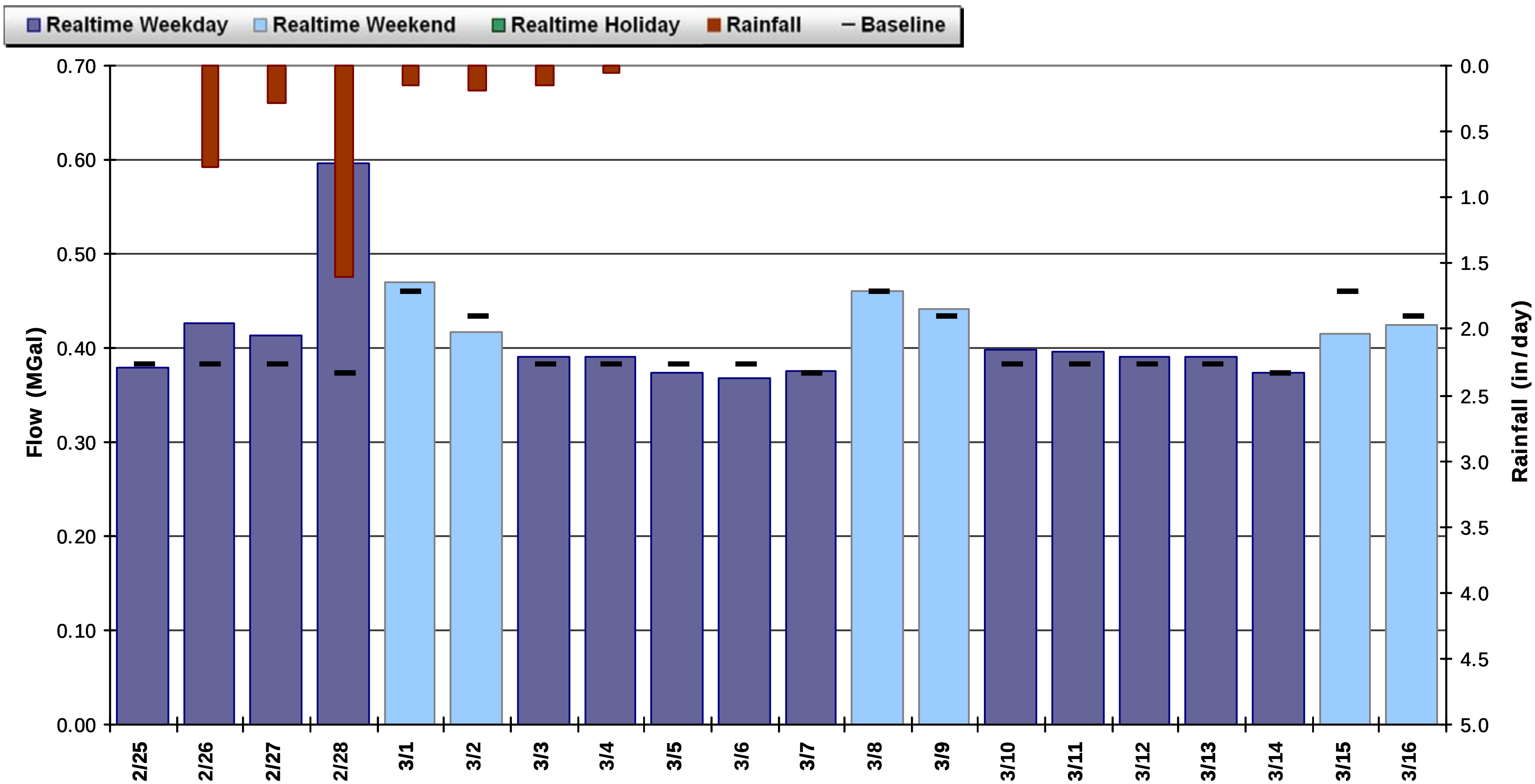
Data Summary Report



Vicinity Map: Basin 7+8

BASIN 7+8

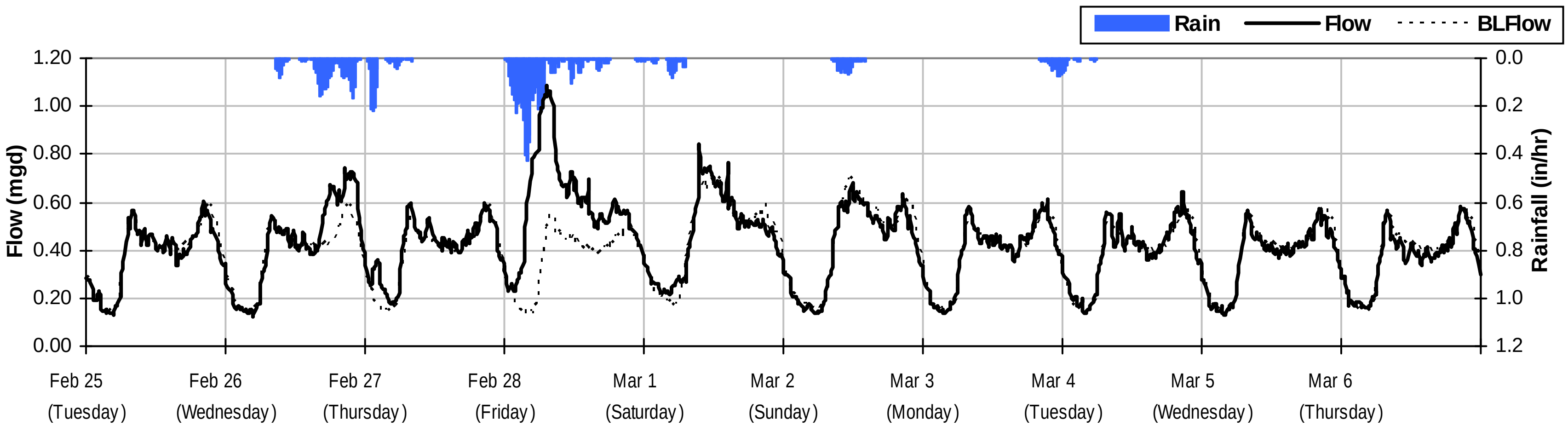
Period Flow Summary: Daily Flow Totals

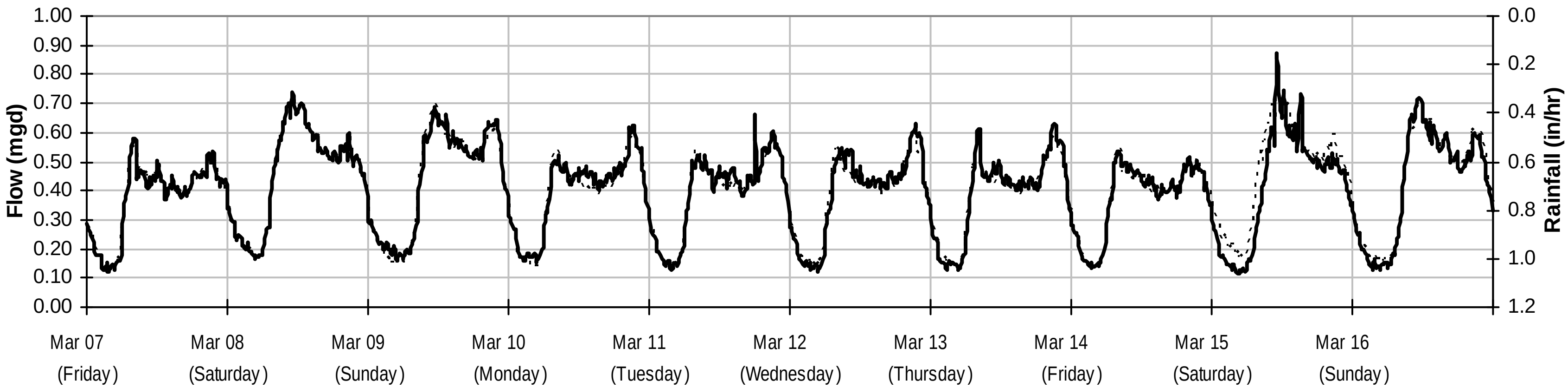


BASIN 7+8

Flow Summary: 2/25/2014 to 3/17/2014

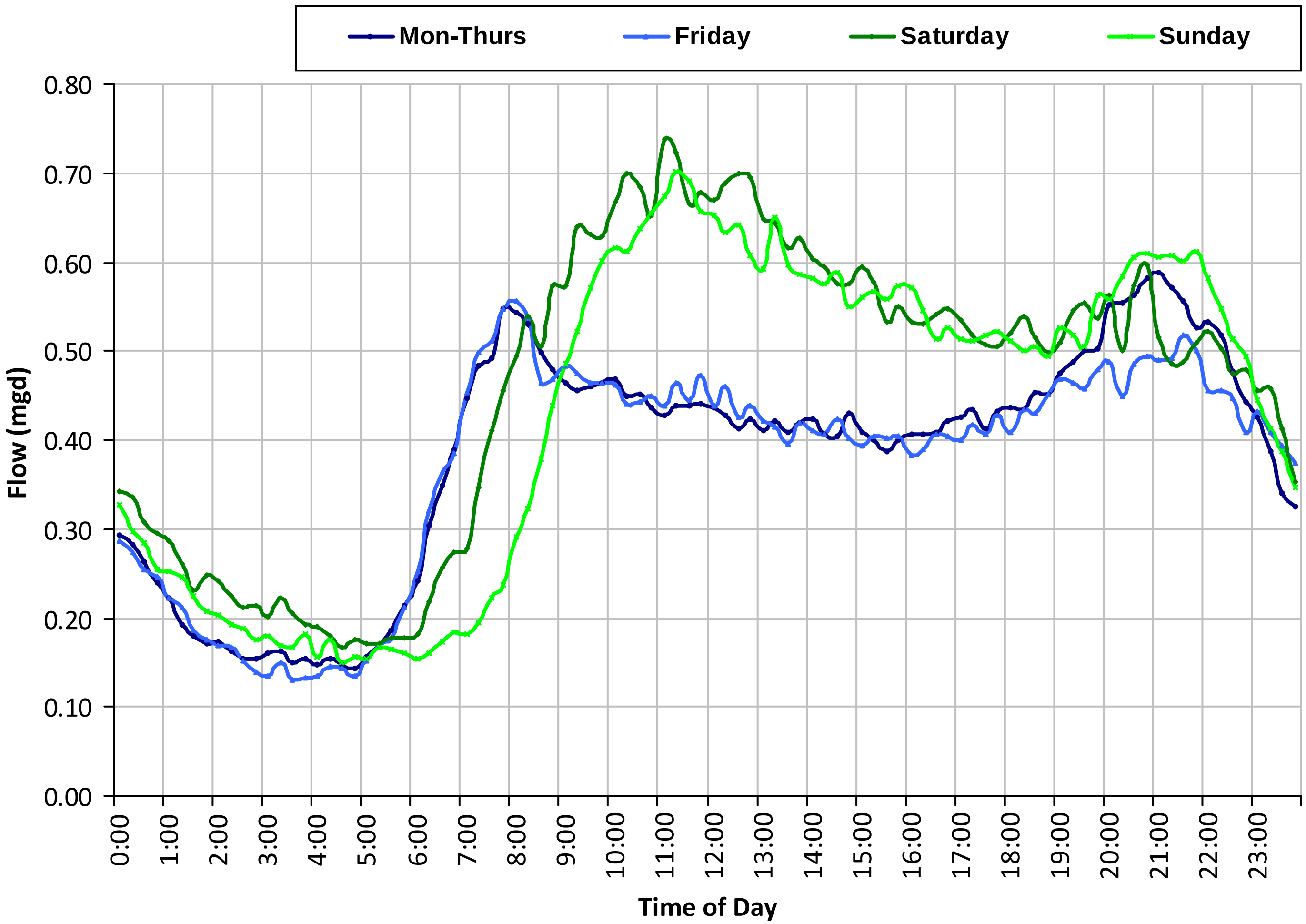
Total Period Rainfall: 3.20 inches **Avg Flow: 0.414 mgd** **Peak Flow: 1.085 mgd** **Min Flow: 0.113 mgd**





BASIN 7+8

Baseline Flow Hydrographs

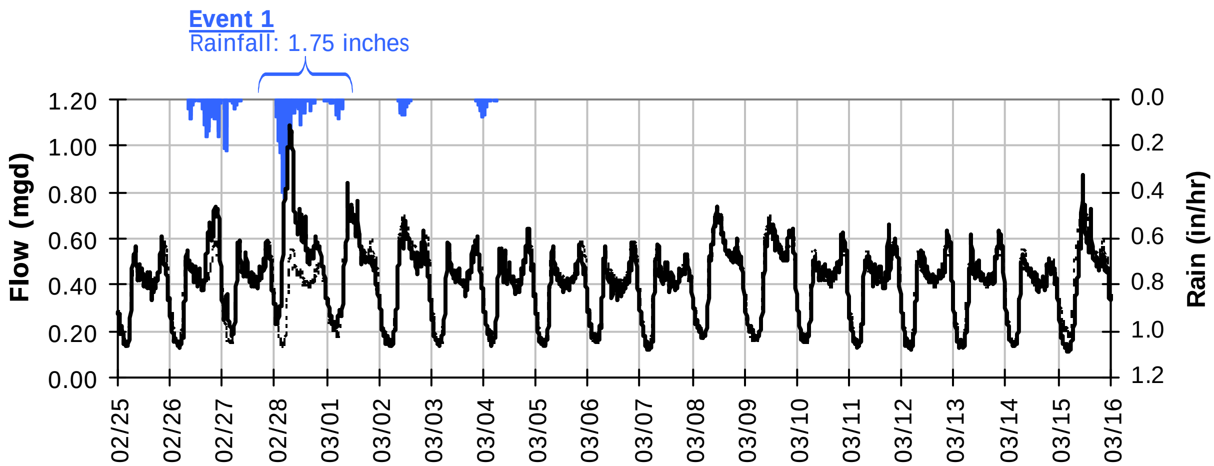


Baseline Flow:

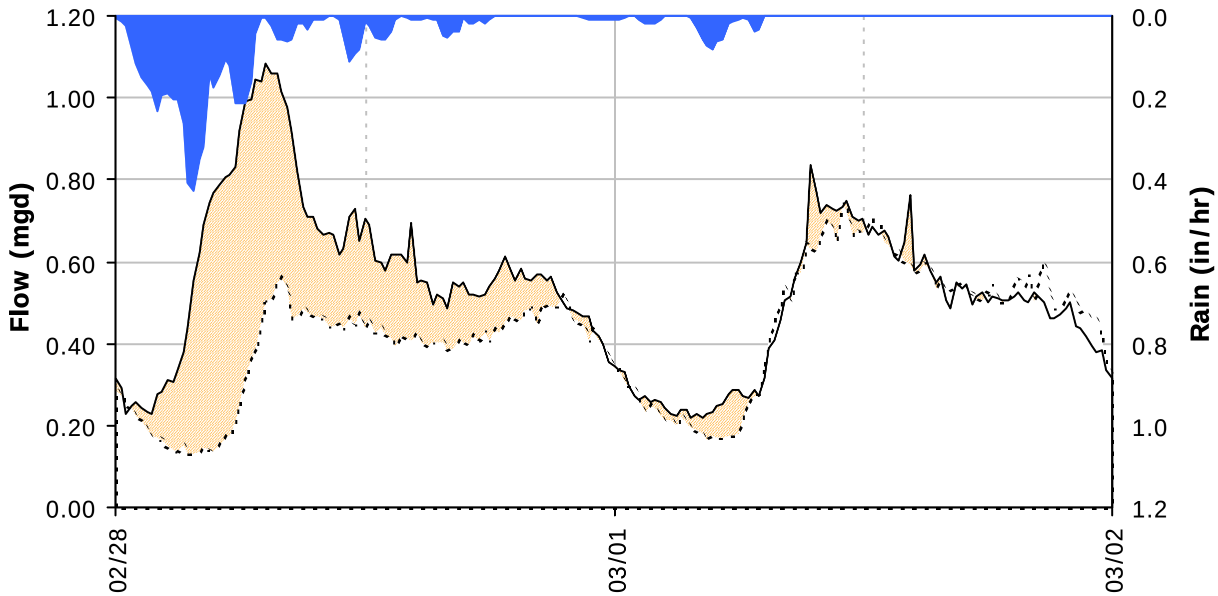
0.400 *mgd*

BASIN 7+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 = 1.75 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	1.08 *mgd*	**Peak I/I Rate:**	0.67 *mgd*
PF:	2.71	**Total I/I:**	230,000 *gallons*

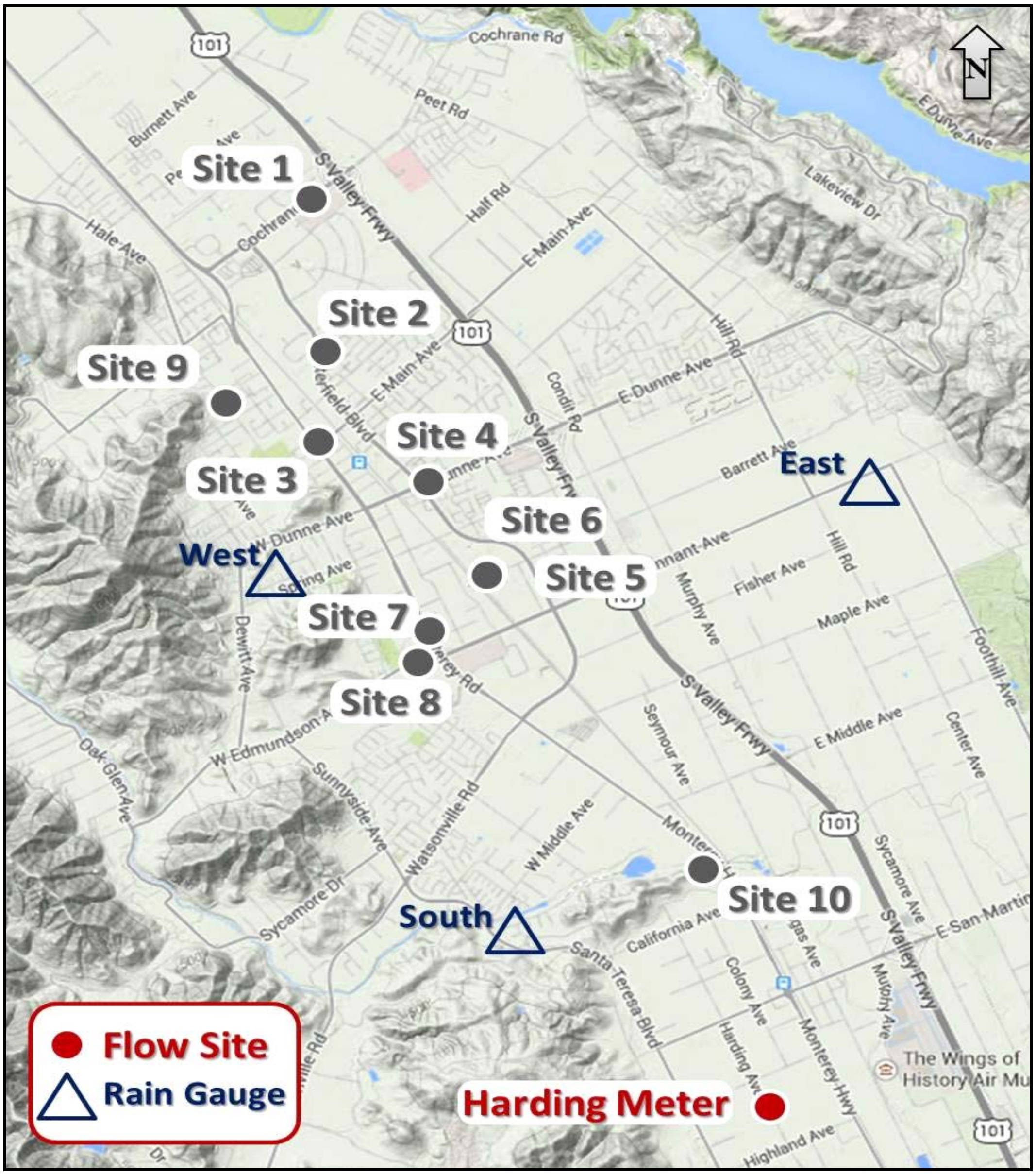
City of Morgan Hill

Sanitary Sewer Flow Monitoring
Temporary Monitoring: February and March, 2014

Monitoring Site: Harding

Location: Harding Avenue, north of Highland Avenue

Data Summary Report



Vicinity Map: Harding

HARDING

Site Information

Location: Harding Avenue, north of Highland Avenue

Coordinates: 121.6121° W, 37.0749° N

Rim Elevation: 269 feet

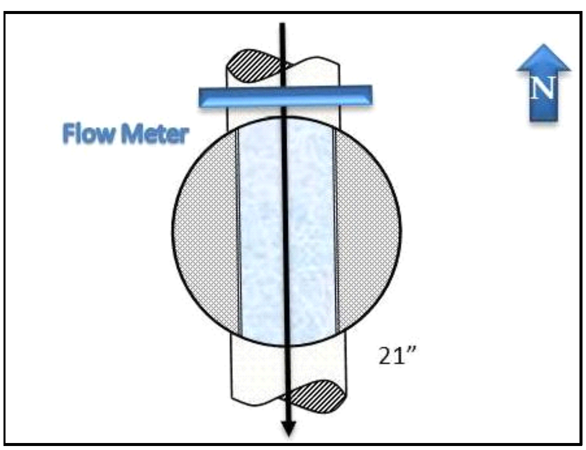
Pipe Diameter: 21 inches

Baseline Flow: 2.649 mgd

Peak Measured Flow: 5.093 mgd

Satellite Map

Street View



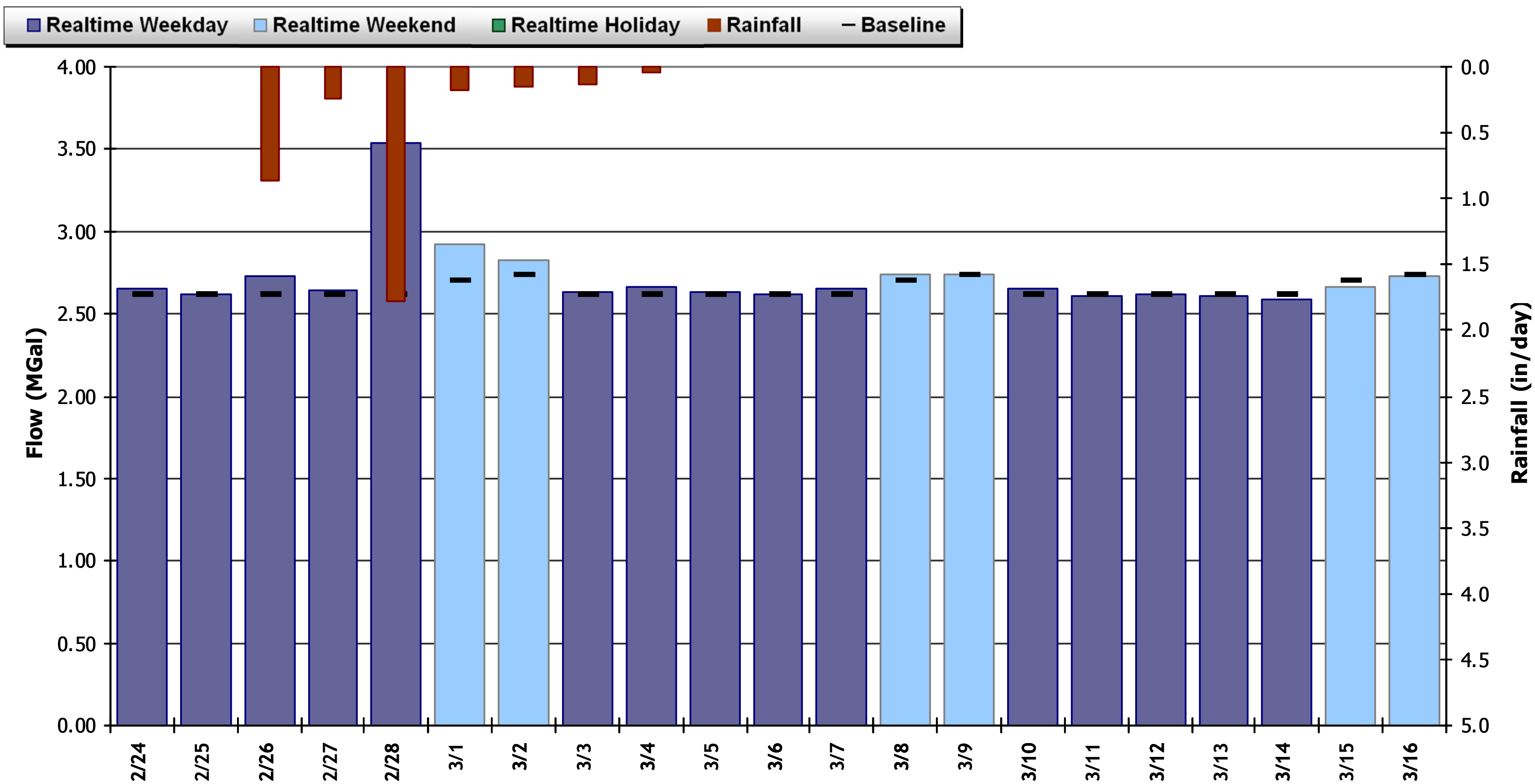
Flow Sketch

Plan View

HARDING
Period Flow Summary: Daily Flow Totals

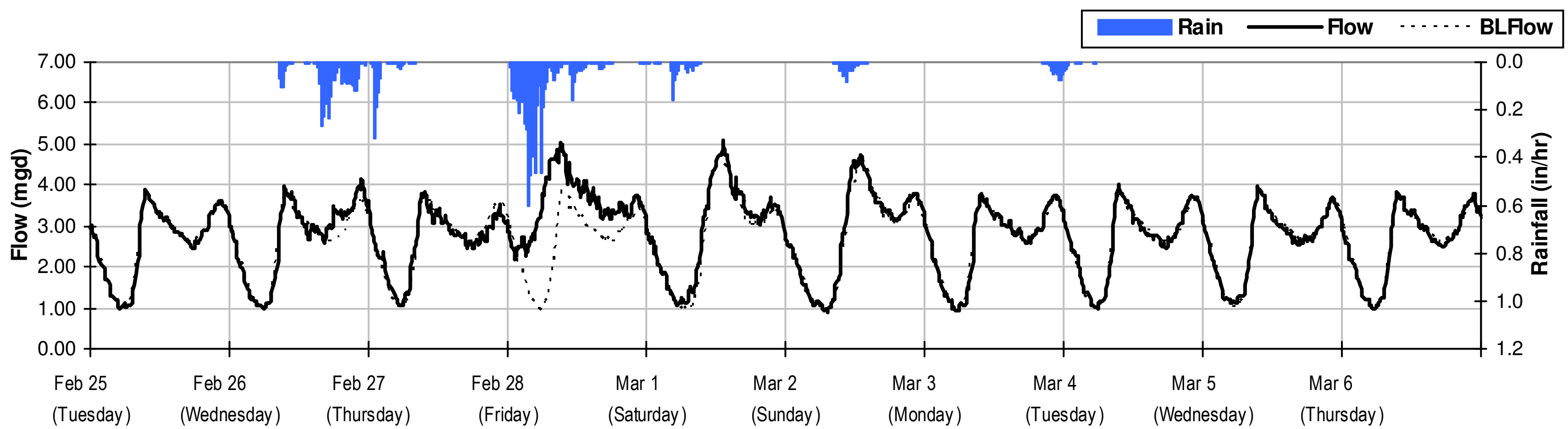
Avg Period Flow: 2.717 MGal **Peak Daily Flow: 3.539 MGal** **Min Daily Flow: 2.591 MGal**

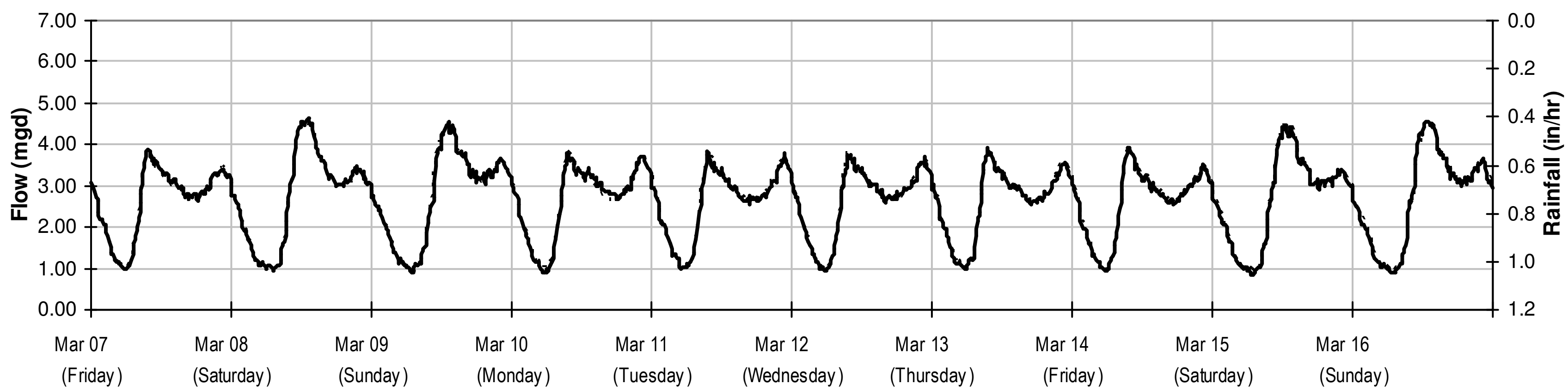
Total Period Rainfall: 3.36 inches



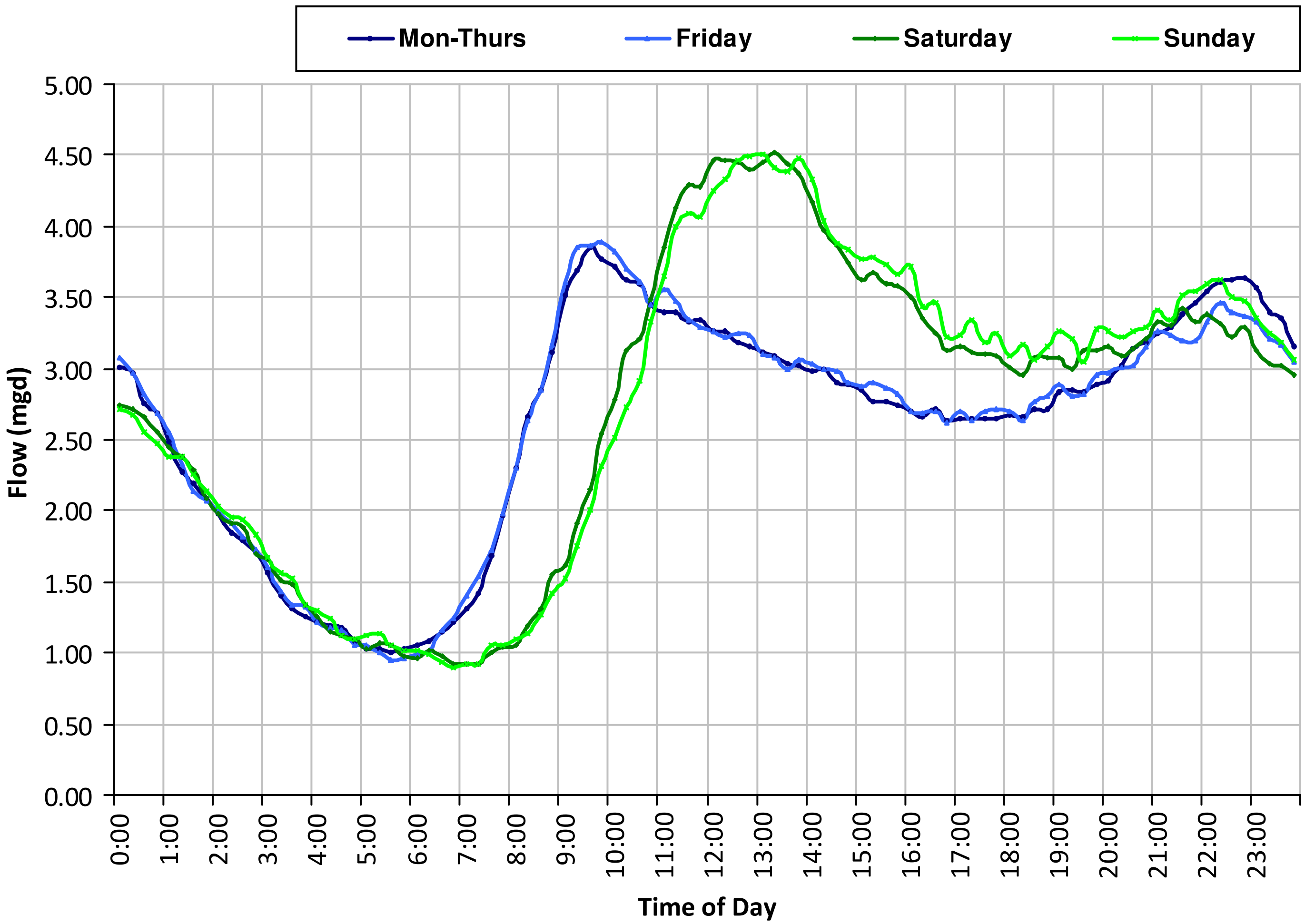
HARDING
Flow Summary: 2/25/2014 to 3/17/2014

Total Period Rainfall: 3.36 inches   Avg Flow: 2.721 mgd   Peak Flow: 5.093 mgd   Min Flow: 0.839 mgd



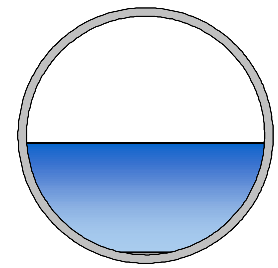


HARDING
Baseline Flow Hydrographs



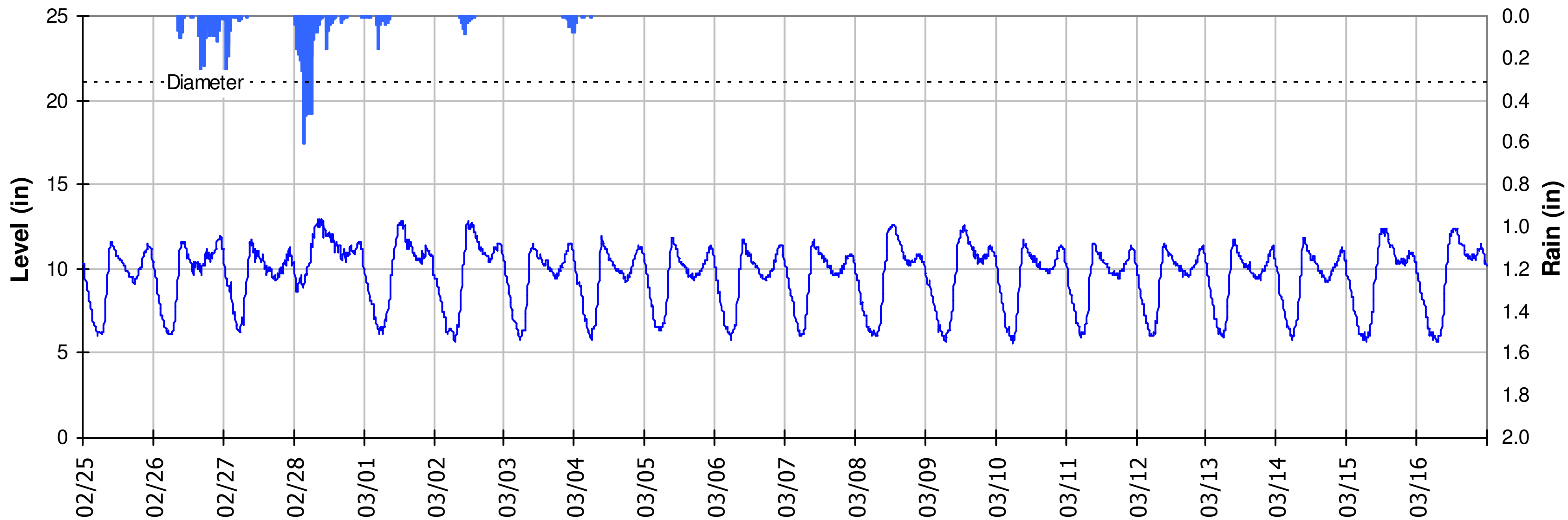
Baseline Flow:

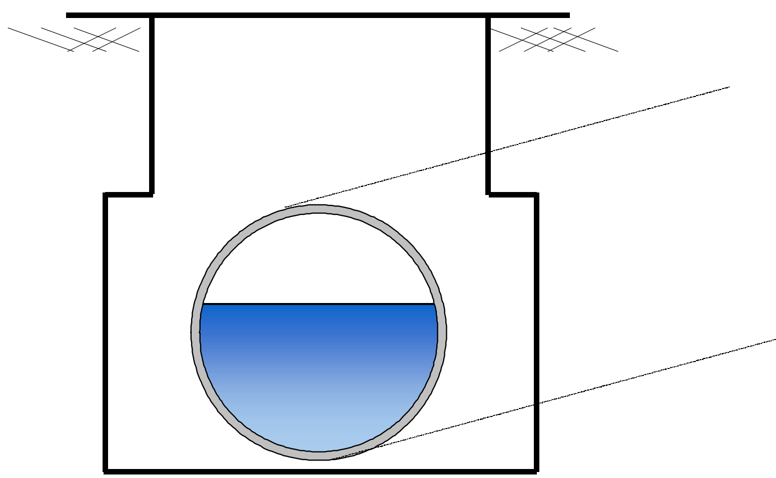
2.649 *mgd*



HARDING
Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period





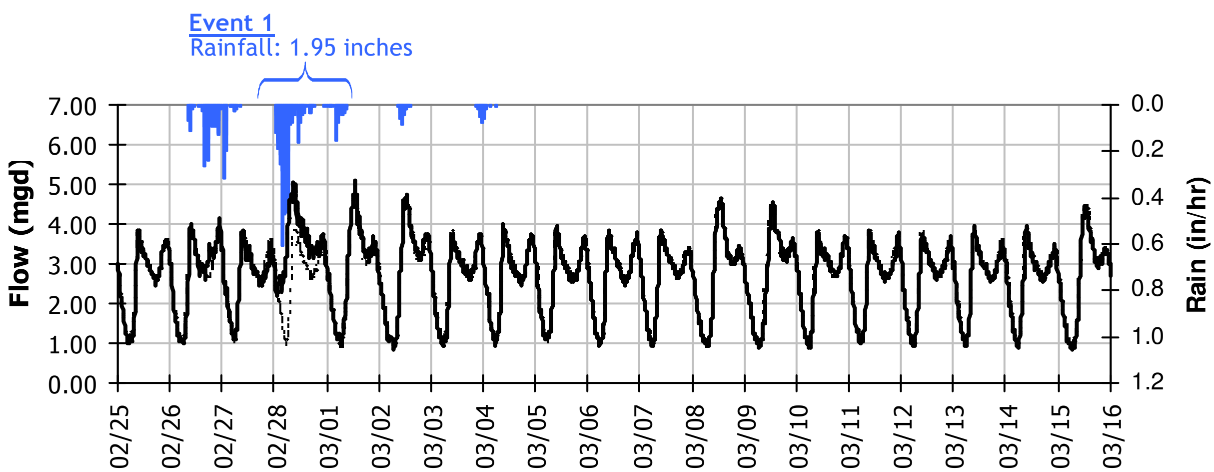
Pipe Diameter: 21 *inches*

Peak Measured Level: 13 *inches*

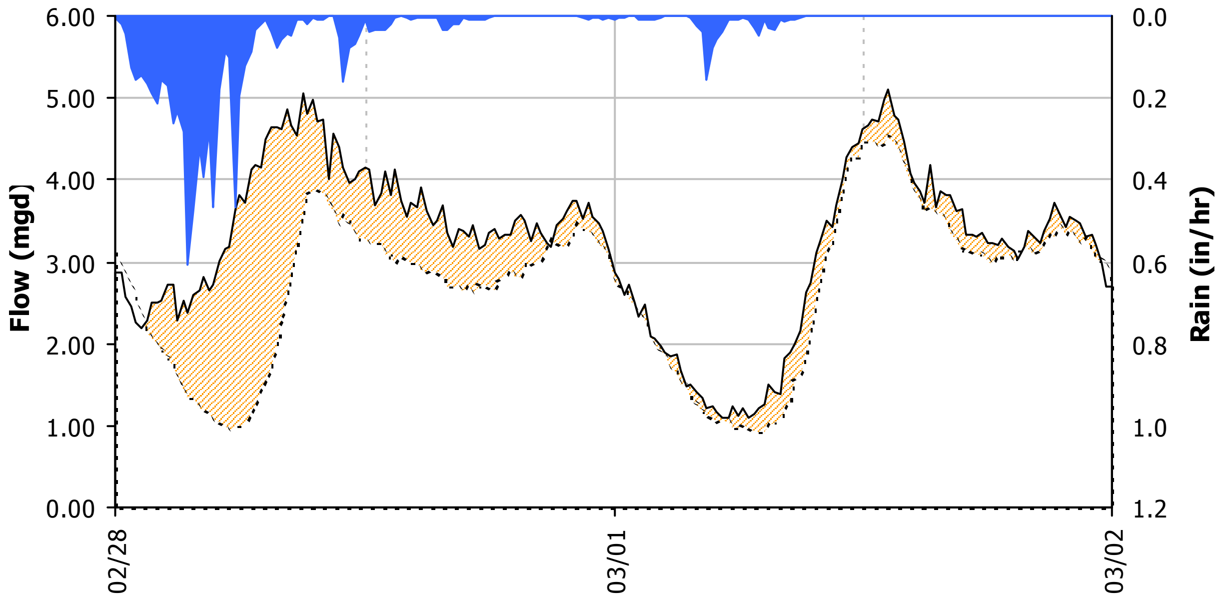
Peak d/D Ratio: 0.62

HARDING
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



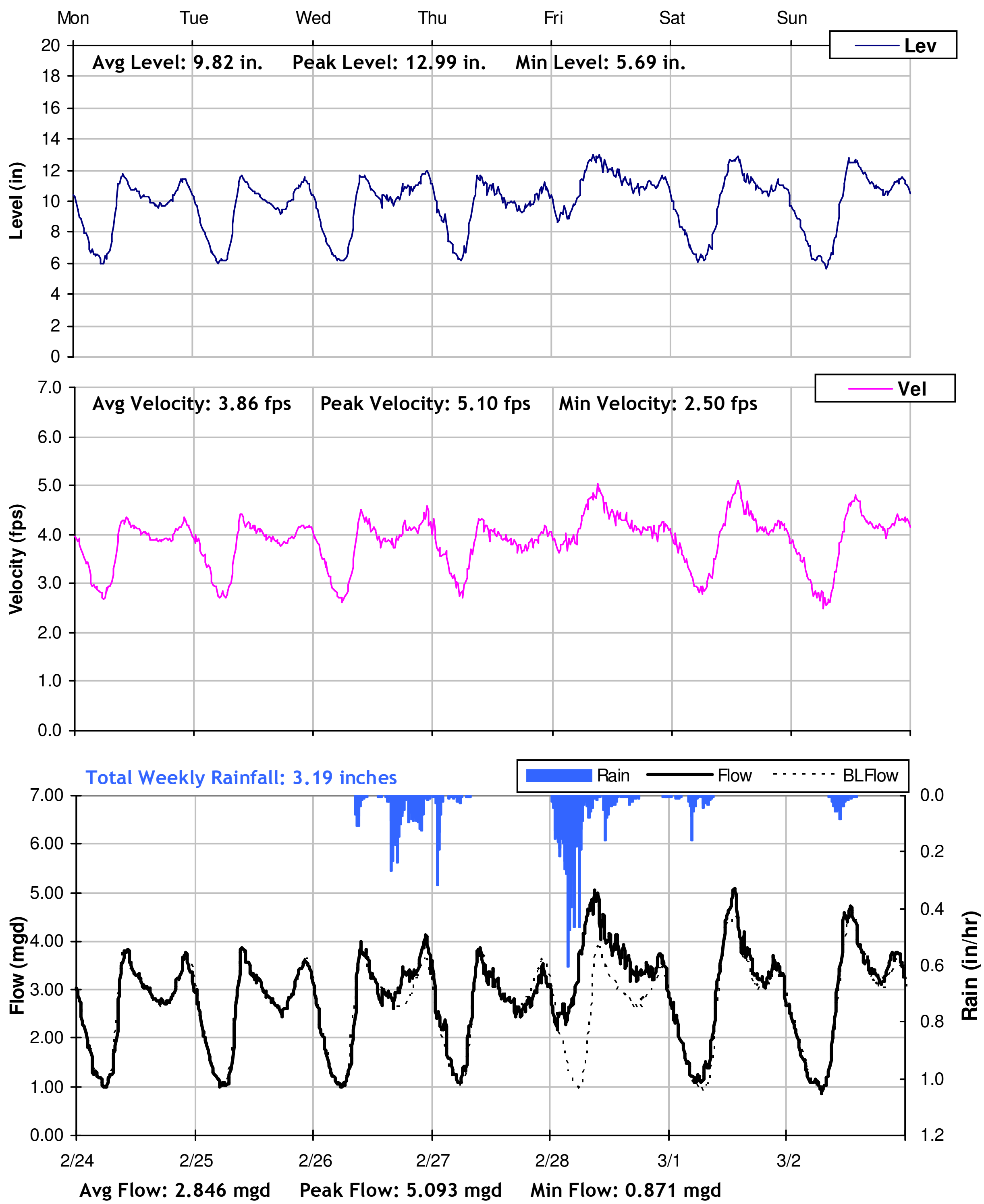
Event 1 Detail Graph



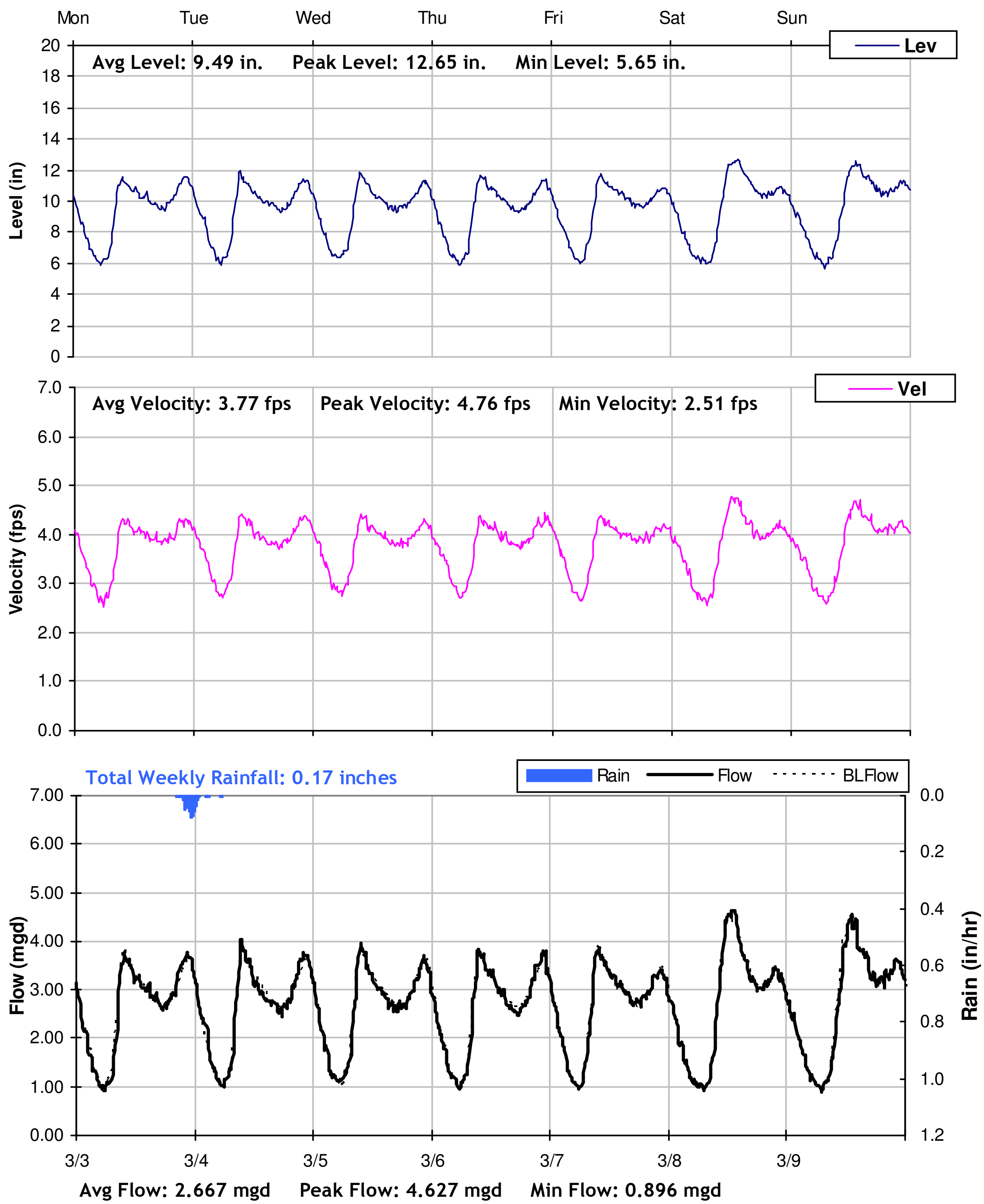
Storm Event I/I Analysis (Rain = 1.95 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	5.09 *mgd*	**Peak I/I Rate:**	2.97 *mgd*
PF:	1.92	**Total I/I:**	1,135,000 *gallons*

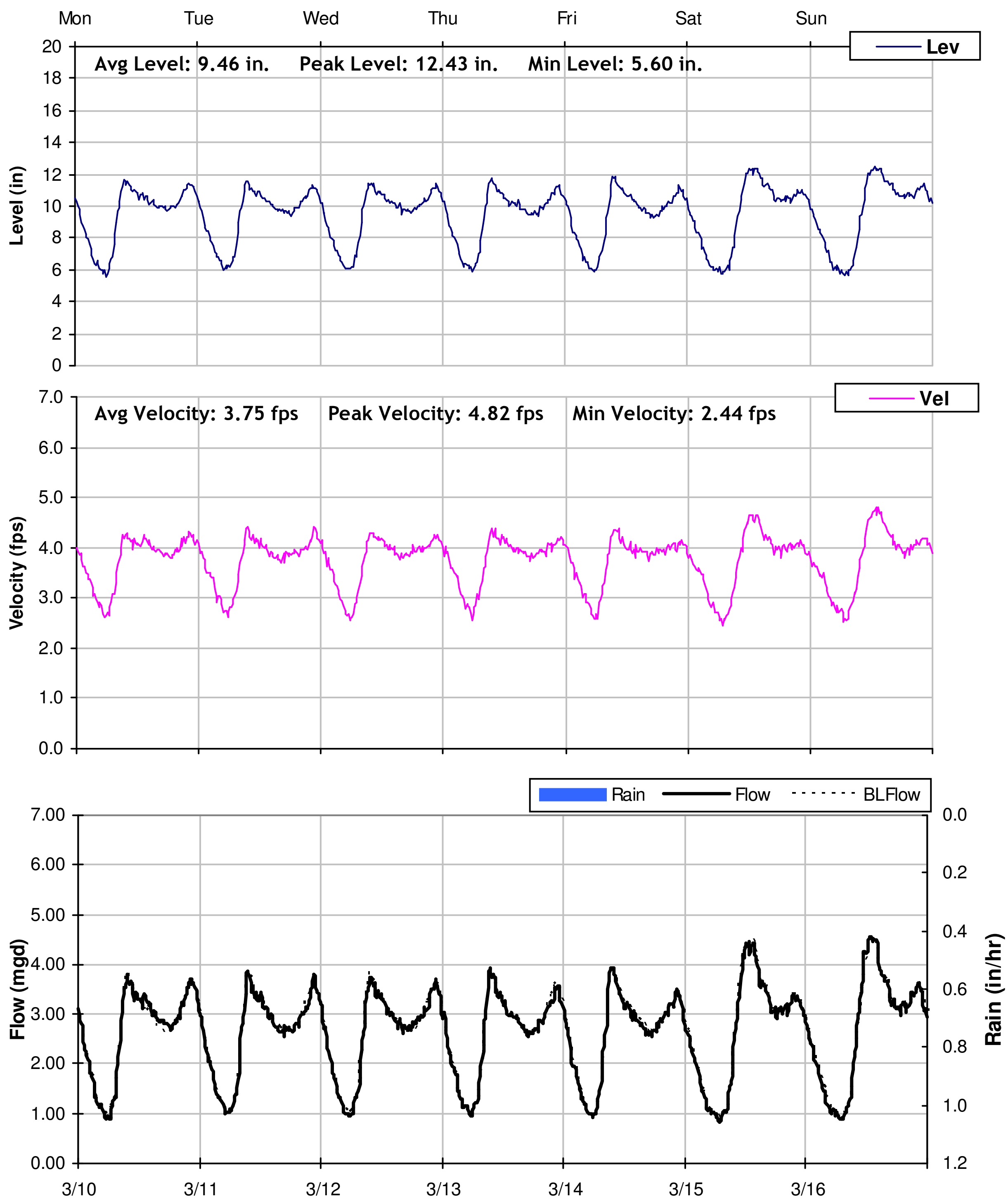
HARDING
Weekly Level, Velocity and Flow Hydrographs
2/24/2014 to 3/3/2014



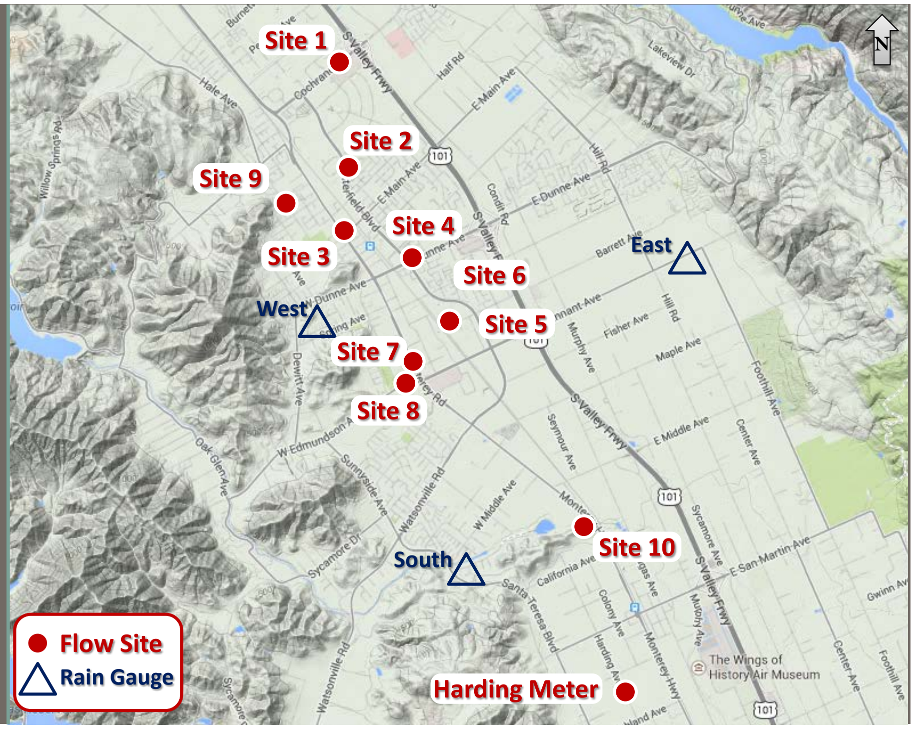
HARDING
Weekly Level, Velocity and Flow Hydrographs
3/3/2014 to 3/10/2014



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



Avg Flow: 2.639 mgd Peak Flow: 4.541 mgd Min Flow: 0.839 mgd



Oakland
155 Grand Avenue, Suite 700
Oakland, CA 94612
510.903.6600 **Tel**
510.903.6601 **Fax**

San Diego
11011 Via Frontera, Suite C
San Diego, CA 92127
858.576.0226 **Tel**

Houston
8220 Jones Road, Suite 500
Houston, TX 77065
713.568.9067 **Tel**

Las Vegas
3430 East Russell Road, Suite 316
Las Vegas, NV 89120
702.522.7967 **Tel**
702.553.4694 **Fax**

vaengineering.com

APPENDIX B

Hydraulic Model Calibration Exhibits

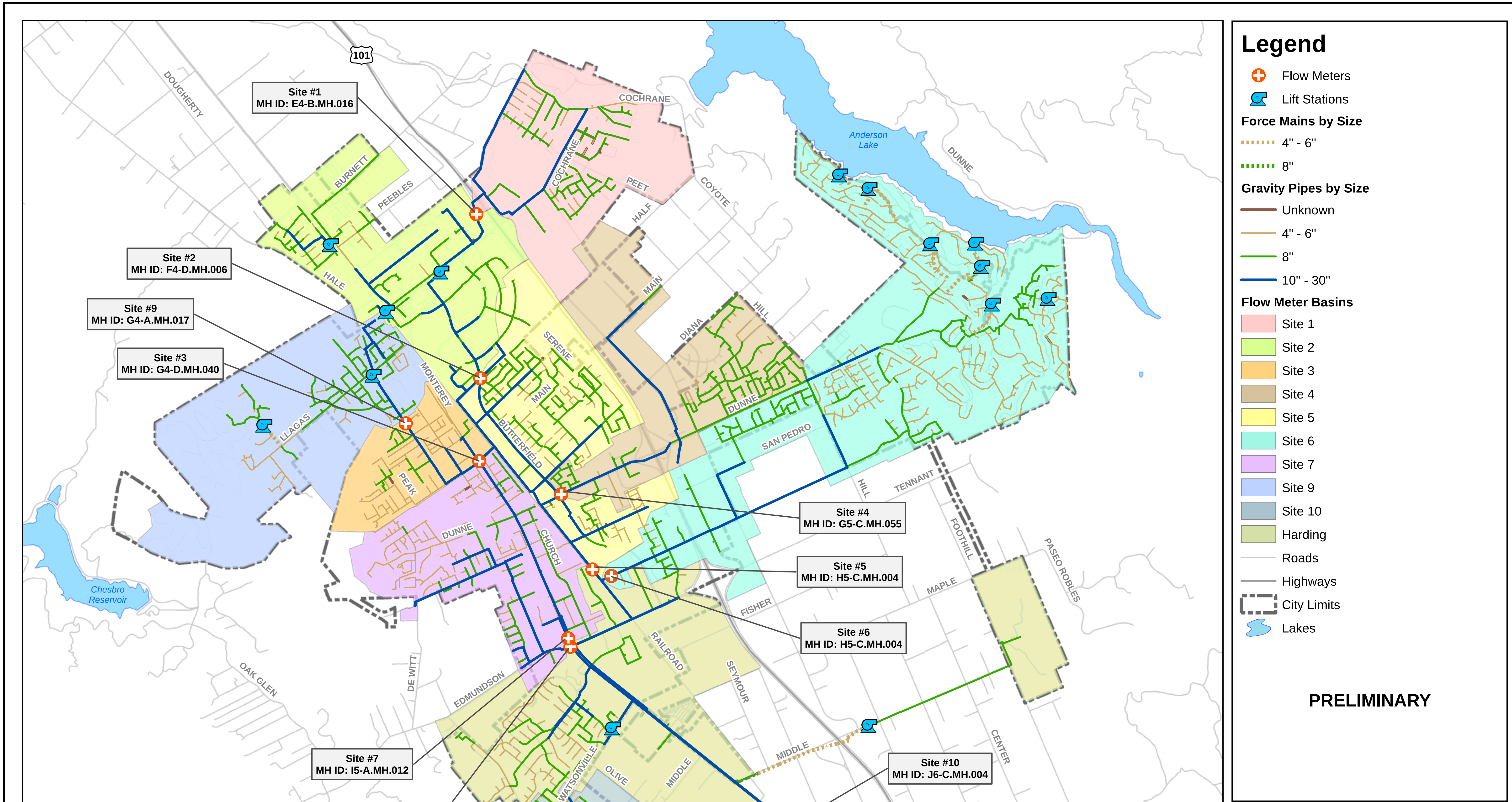
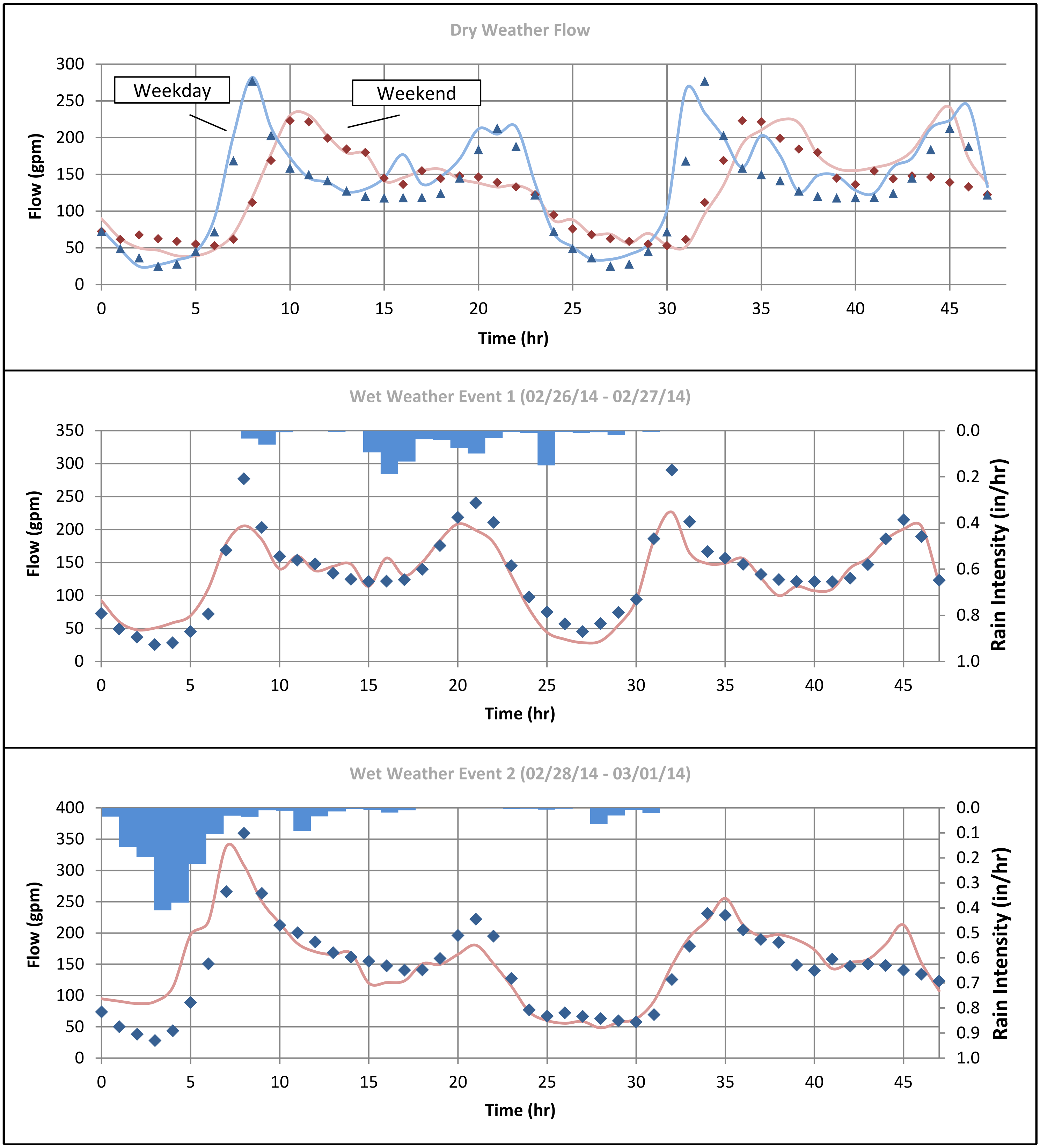


Figure 1
Flow Meter Locations
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

File Name: MH_FlowMeters_022014v2 File Location: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\SSMP



LEGEND

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

PRELIMINARY

June 29, 2016

AKEL ENGINEERING GROUP, INC.

Figure 2

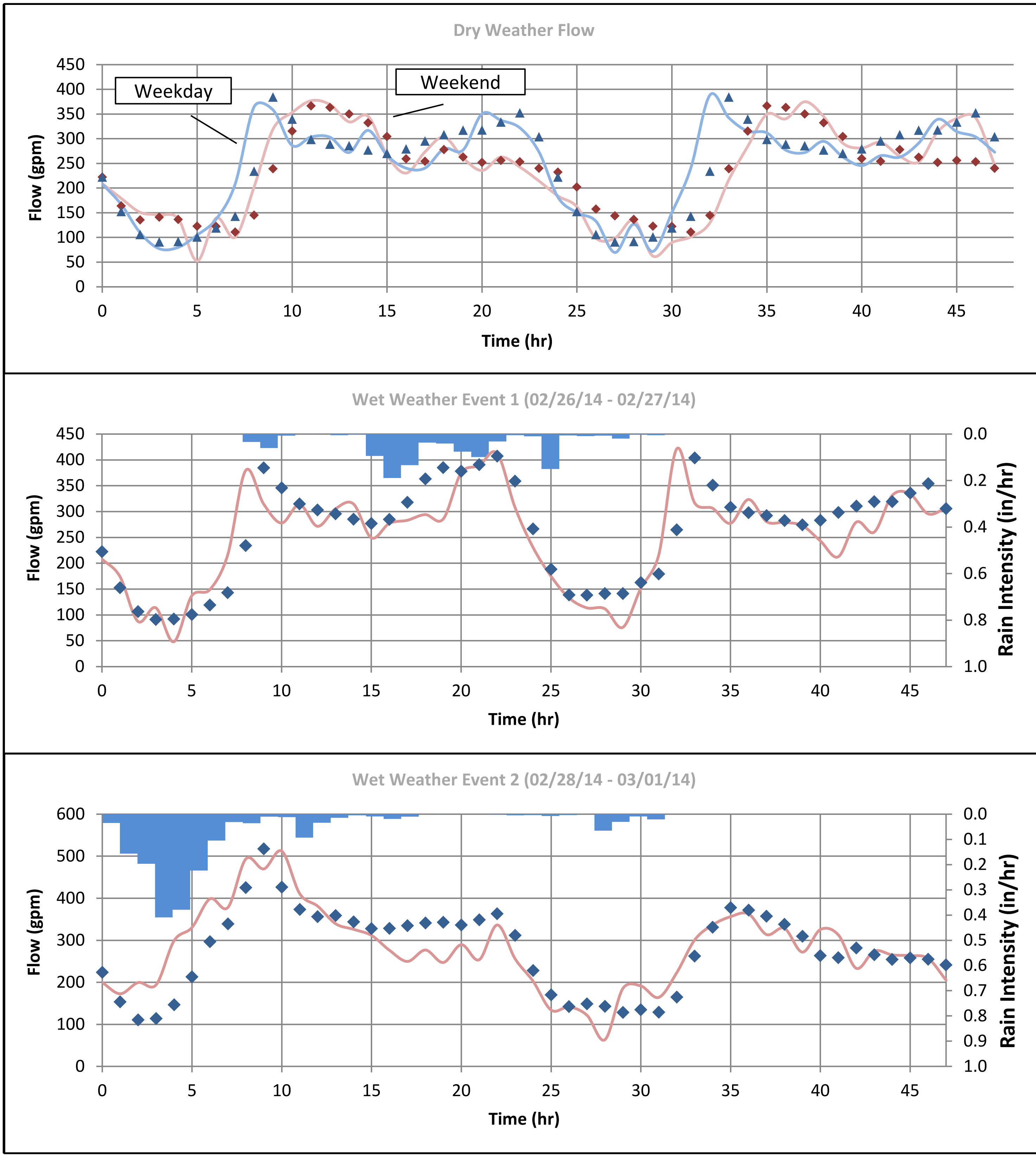
Site 1 Calibration

e/o Madrone Pwky.

Sewer System Master Plan

City of Morgan Hill

CITY OF MORGAN HILL



LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

Figure 3

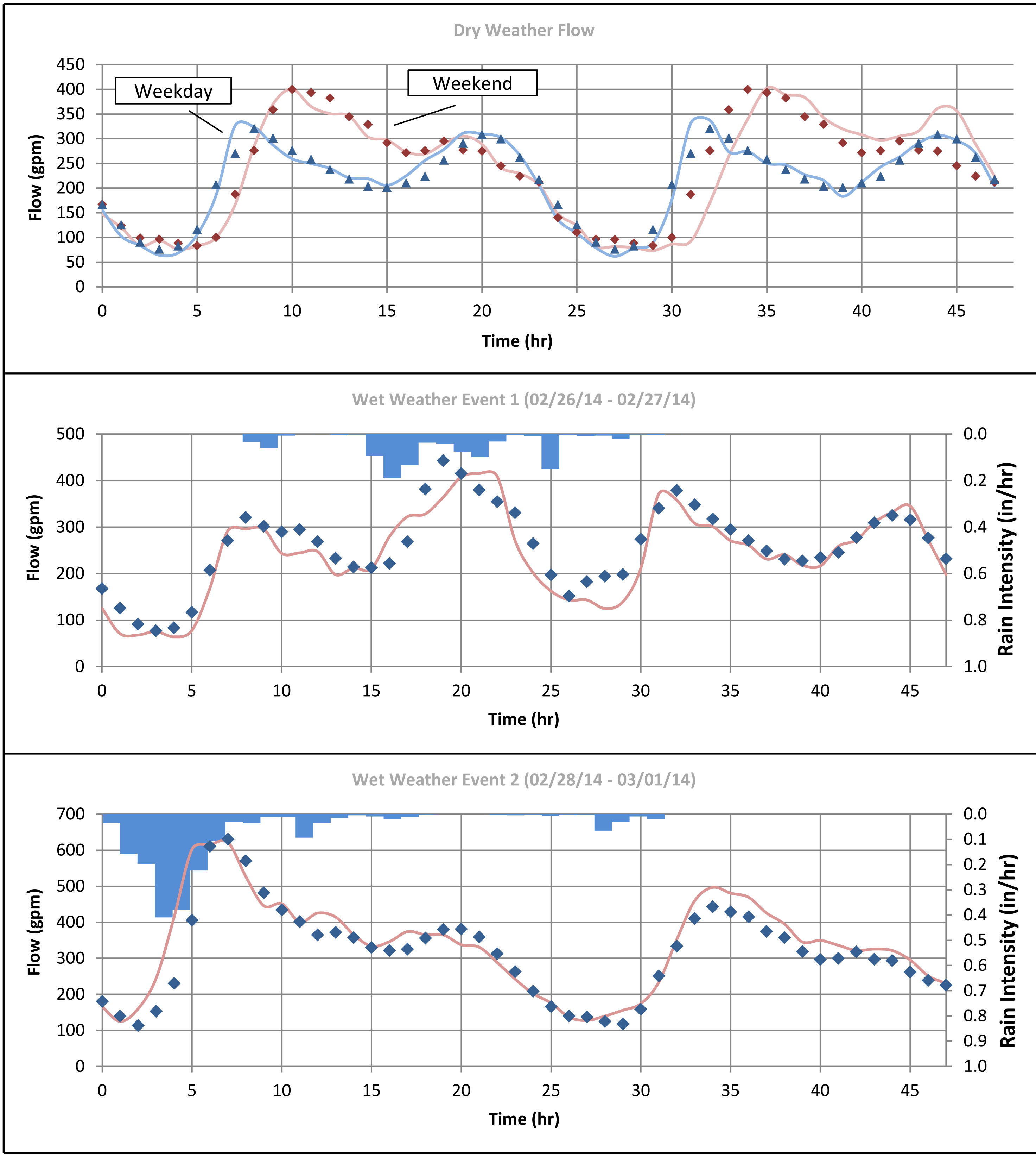
Site 2 Calibration

Butterfield Blvd. and s/o Jarvis Dr.

Sewer System Master Plan

City of Morgan Hill

AKEL ENGINEERING GROUP, INC.



LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

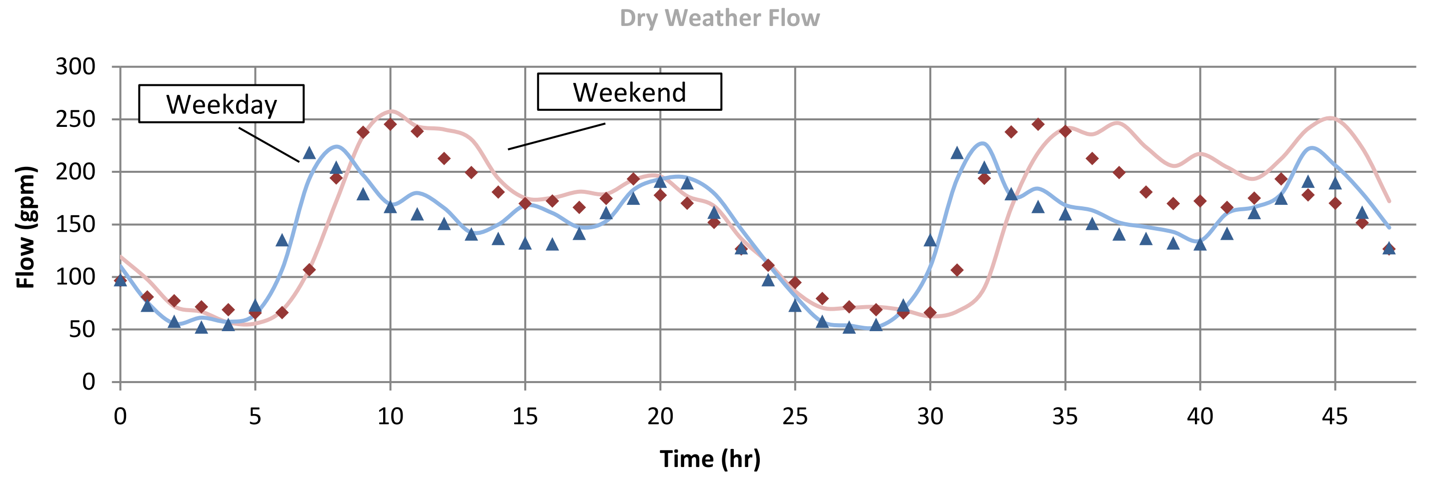
Figure 4

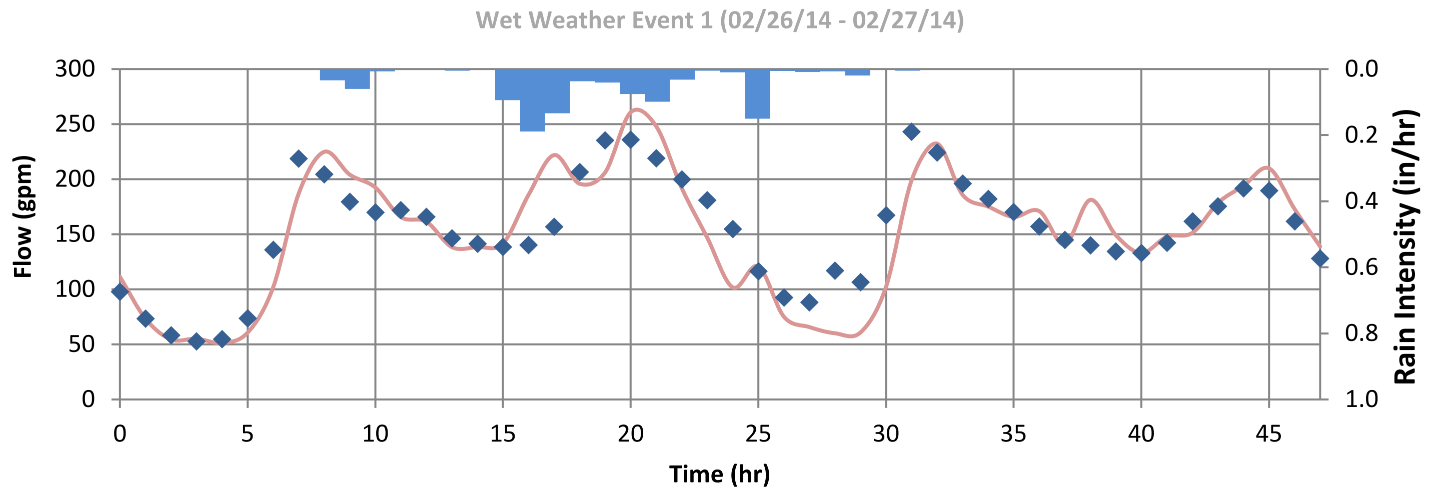
Site 3 Calibration

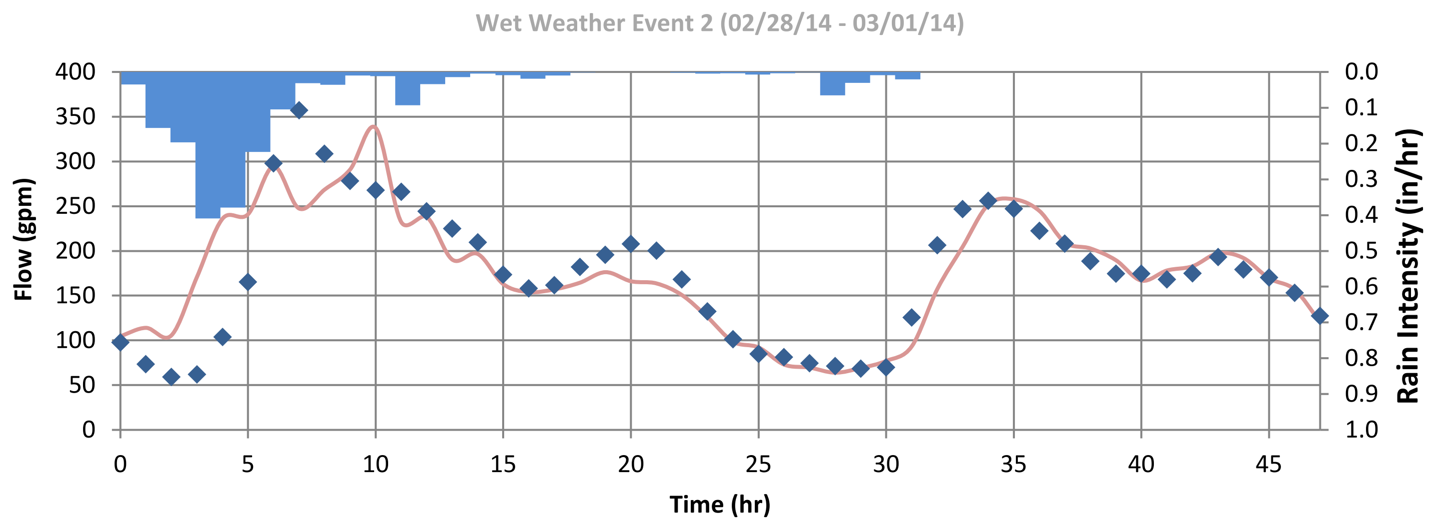
Main Ave. and Monterey Rd.

Sewer System Master Plan

City of Morgan Hill







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

Figure 5

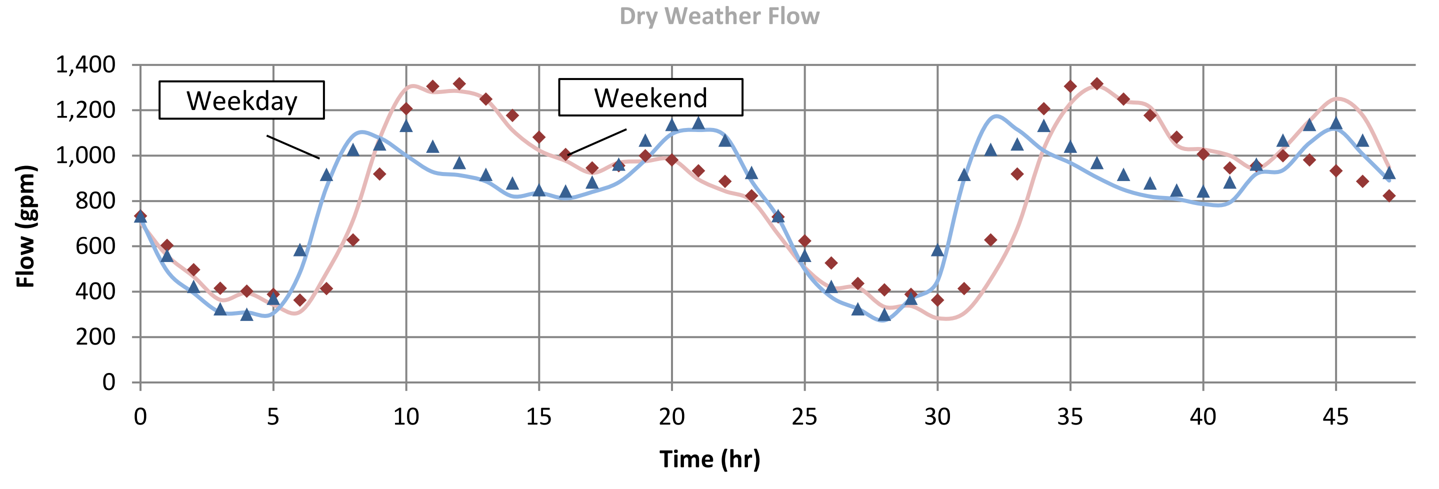
Site 4 Calibration

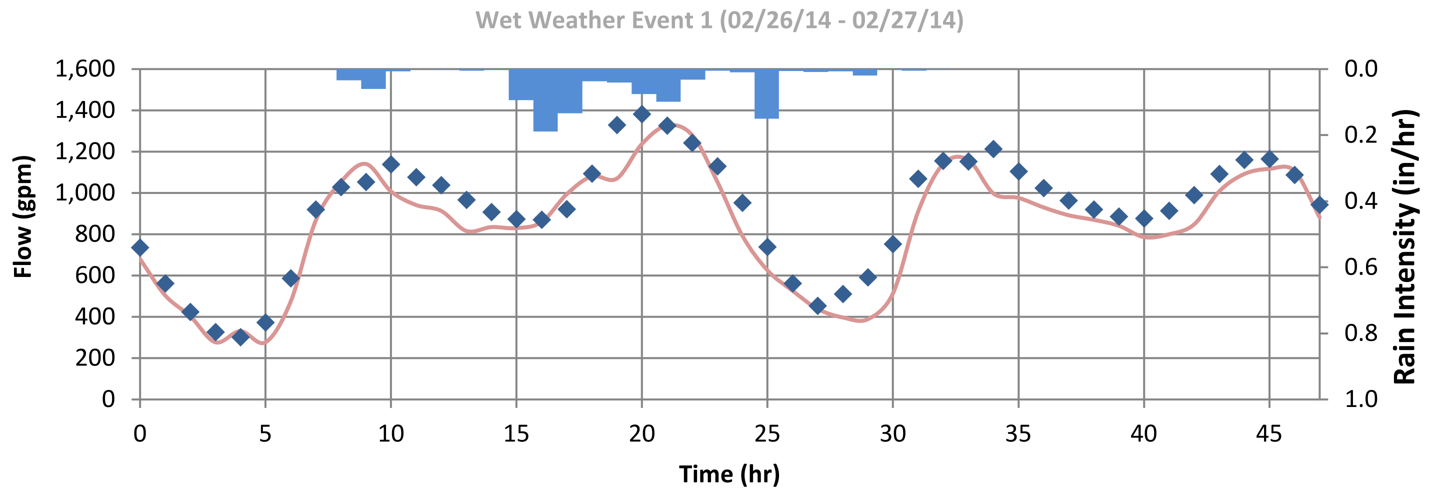
E. Dunne Ave. and e/o Butterfield Blvd.

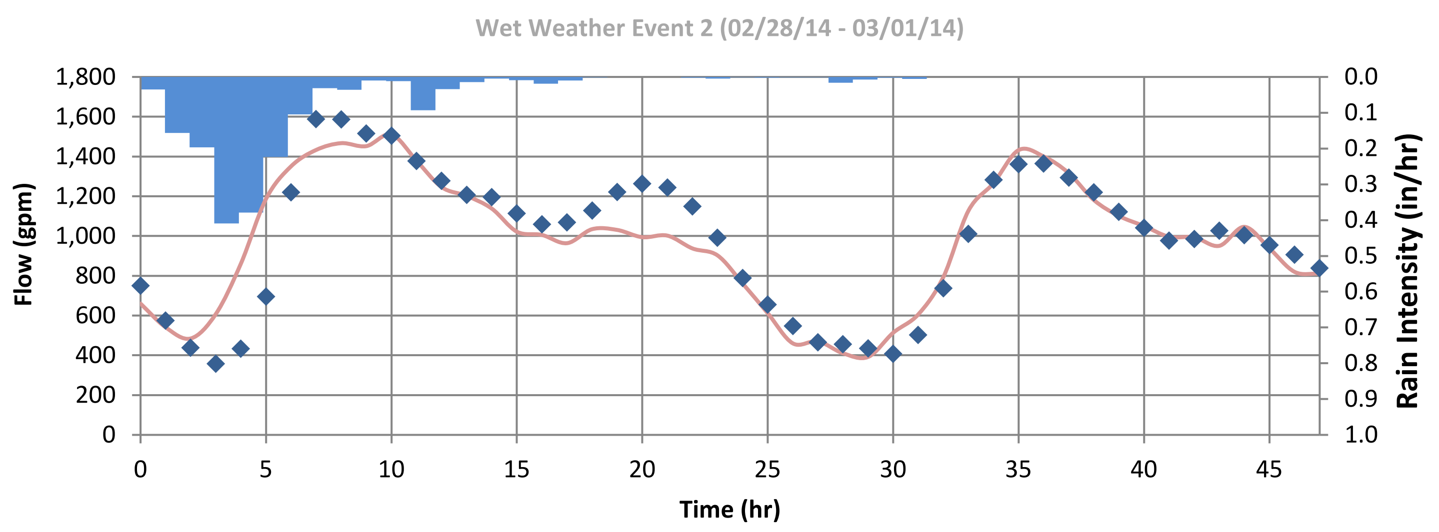
Sewer System Master Plan

City of Morgan Hill

AKEL ENGINEERING GROUP, INC.







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

Figure 6

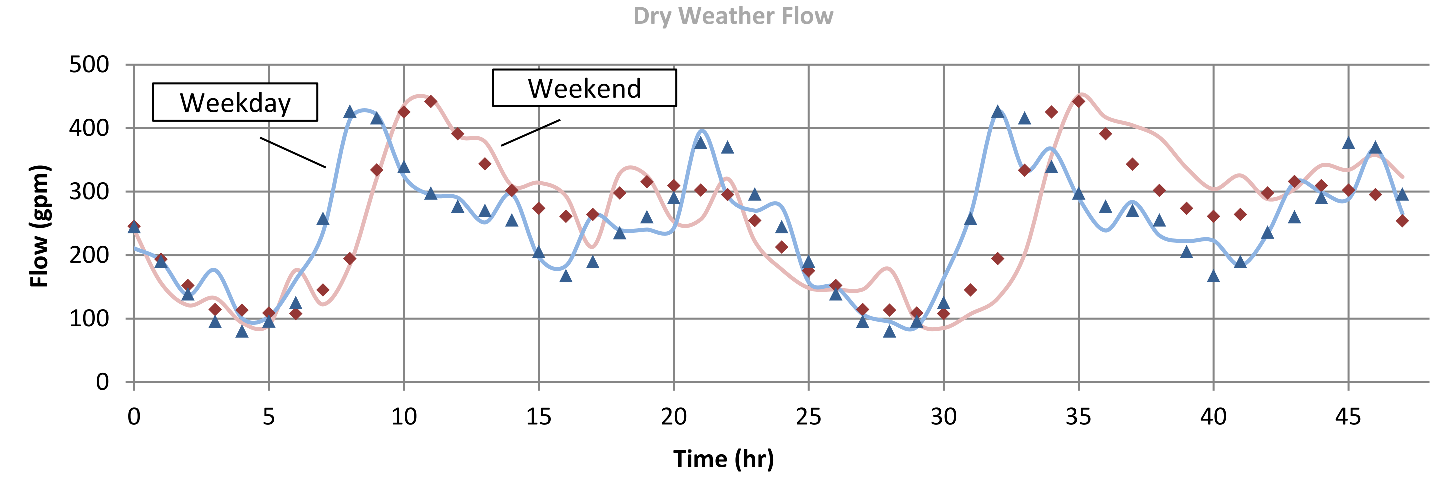
Site 5 Calibration

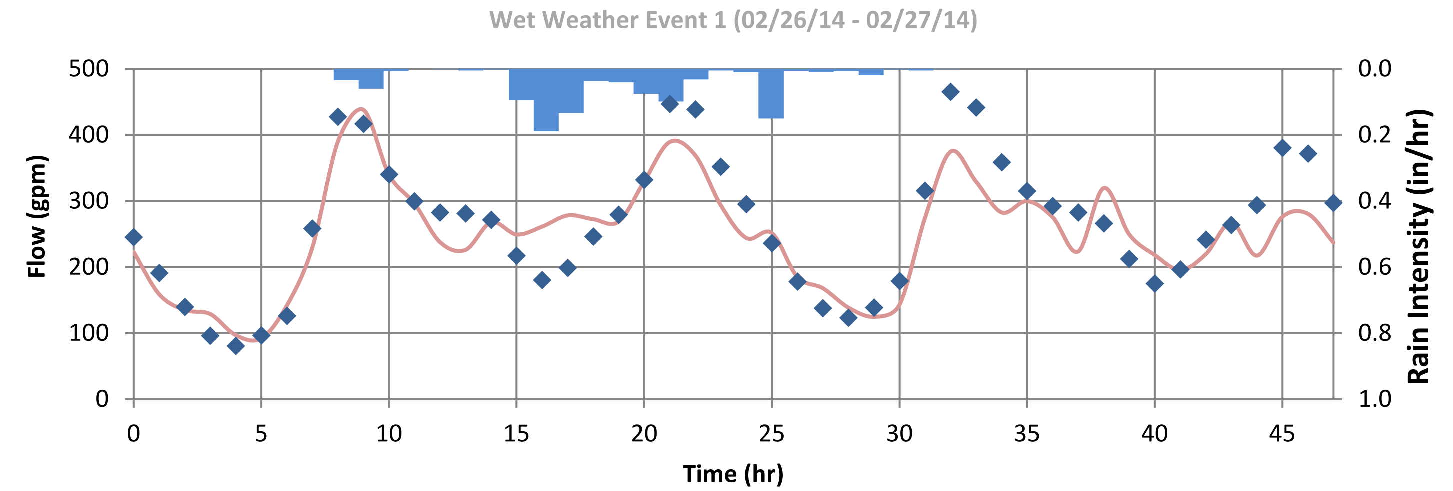
Barrett Ave and Railroad Ave.

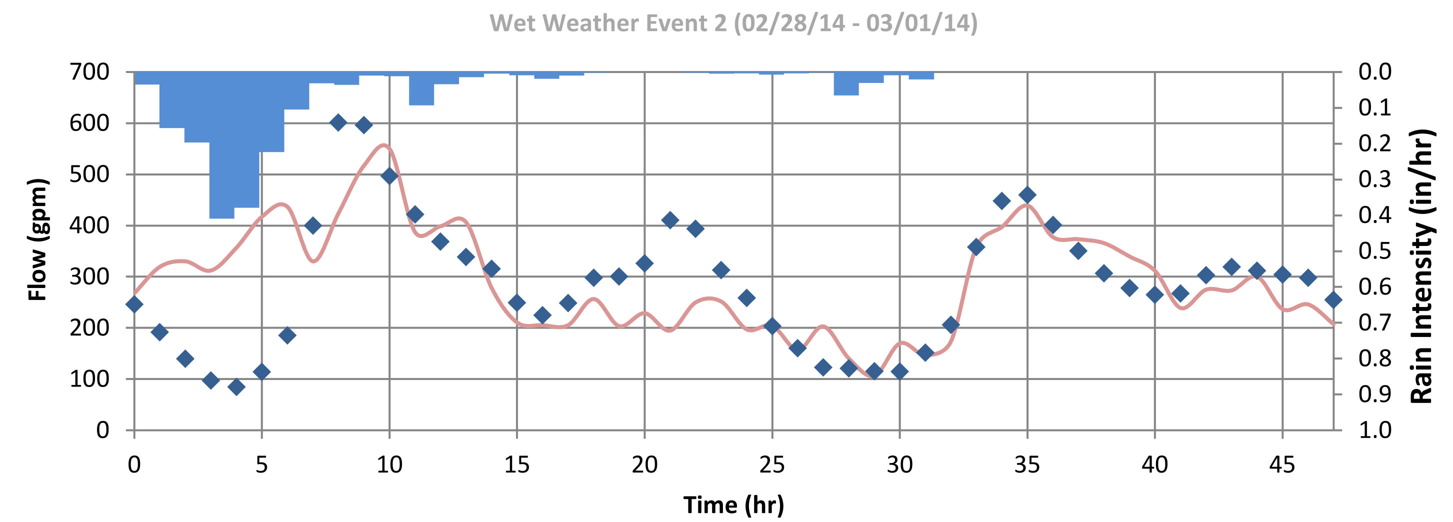
Sewer System Master Plan

City of Morgan Hill

AKEL ENGINEERING GROUP, INC.







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

AKEL ENGINEERING GROUP, INC.

Figure 7

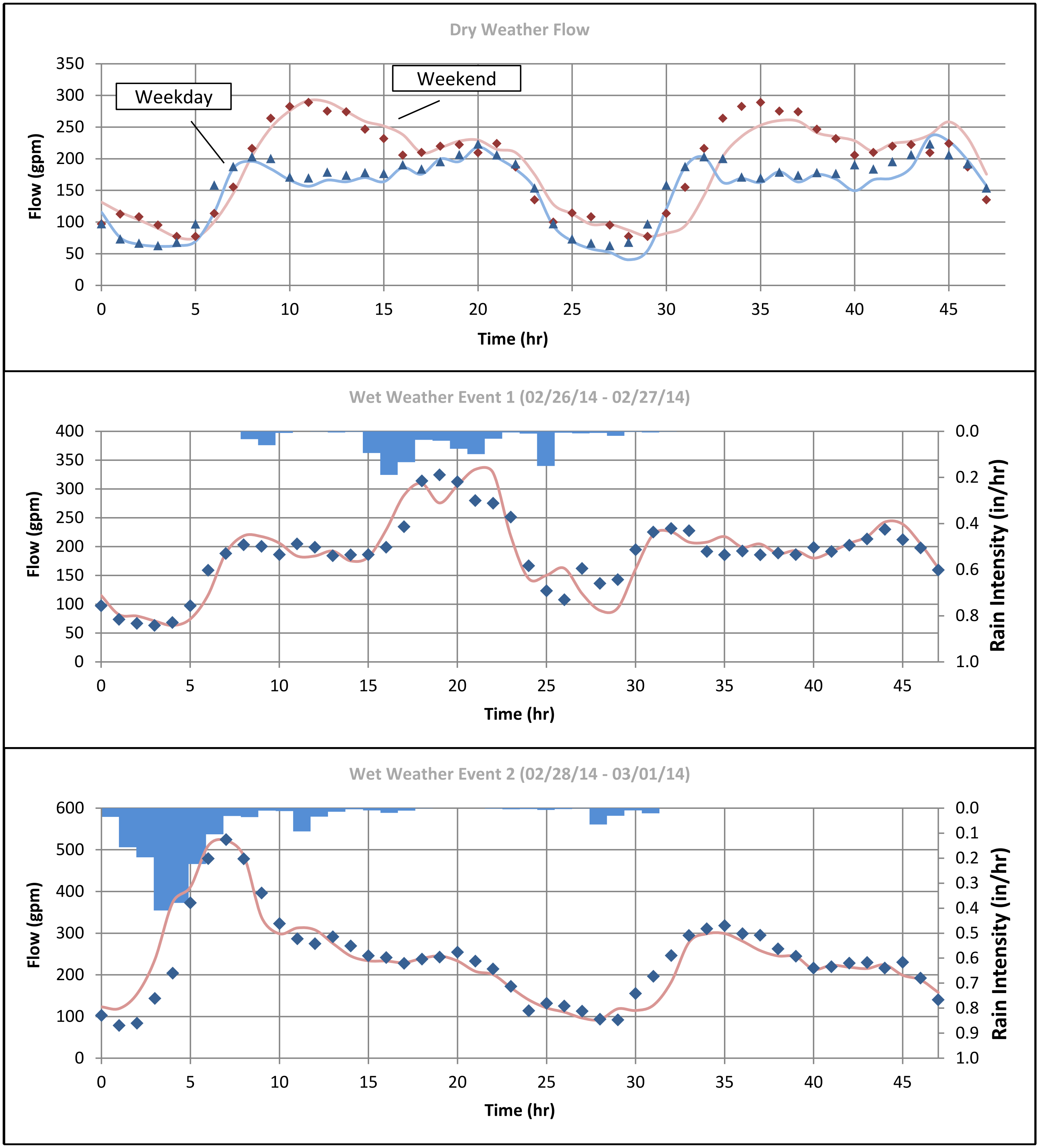
Site 6 Calibration

Barrett Ave. and Railroad Ave.

Sewer System Master Plan

City of Morgan Hill

CITY OF MORGAN HILL



LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

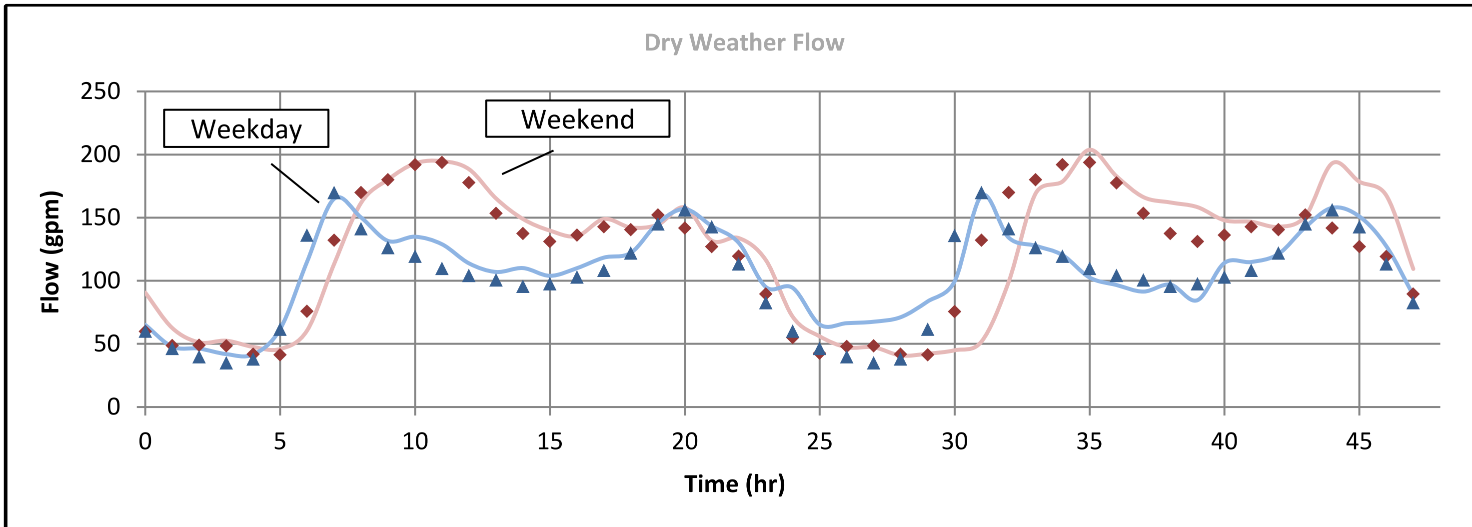
V&A Flow Monitoring

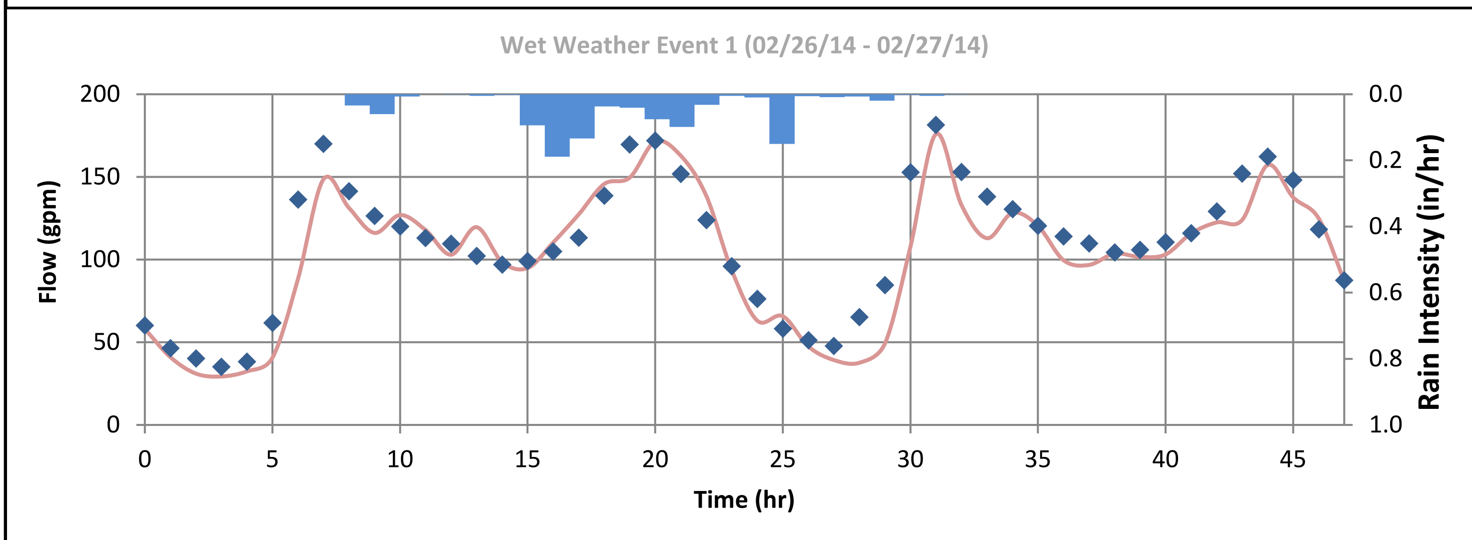
June 29, 2016

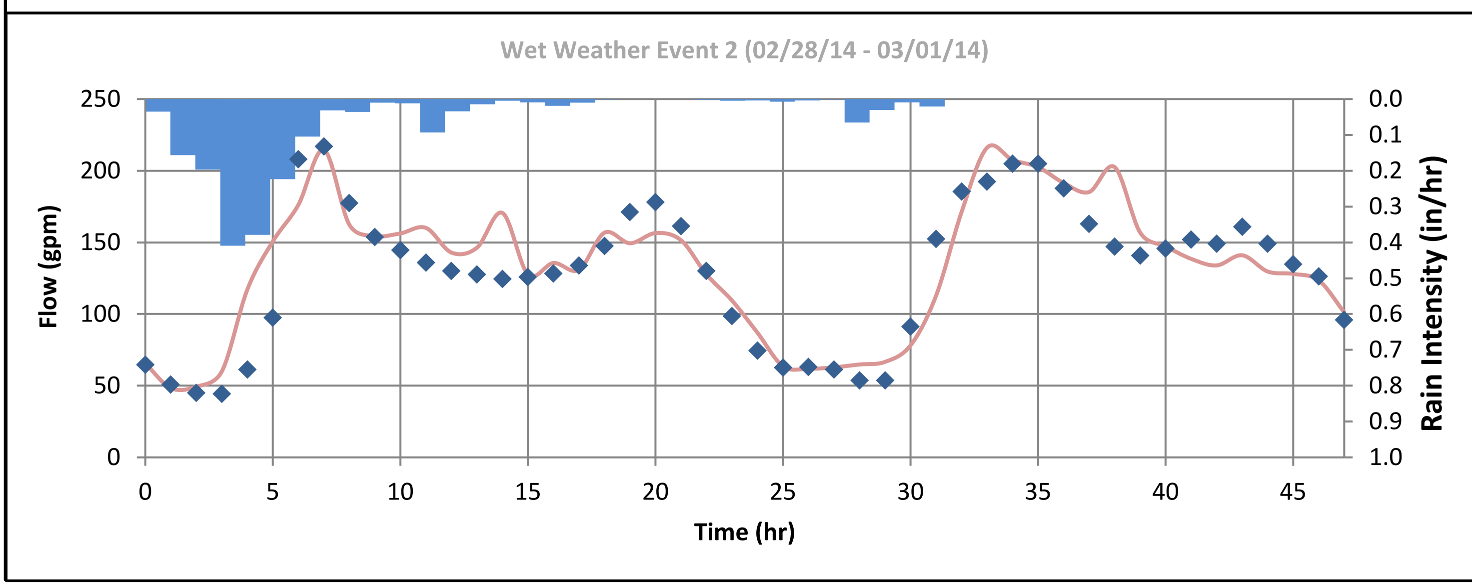
AKEL ENGINEERING GROUP, INC.

Figure 8
Site 7 Calibration
Edes St. and Monterey Rd.
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

Figure 9

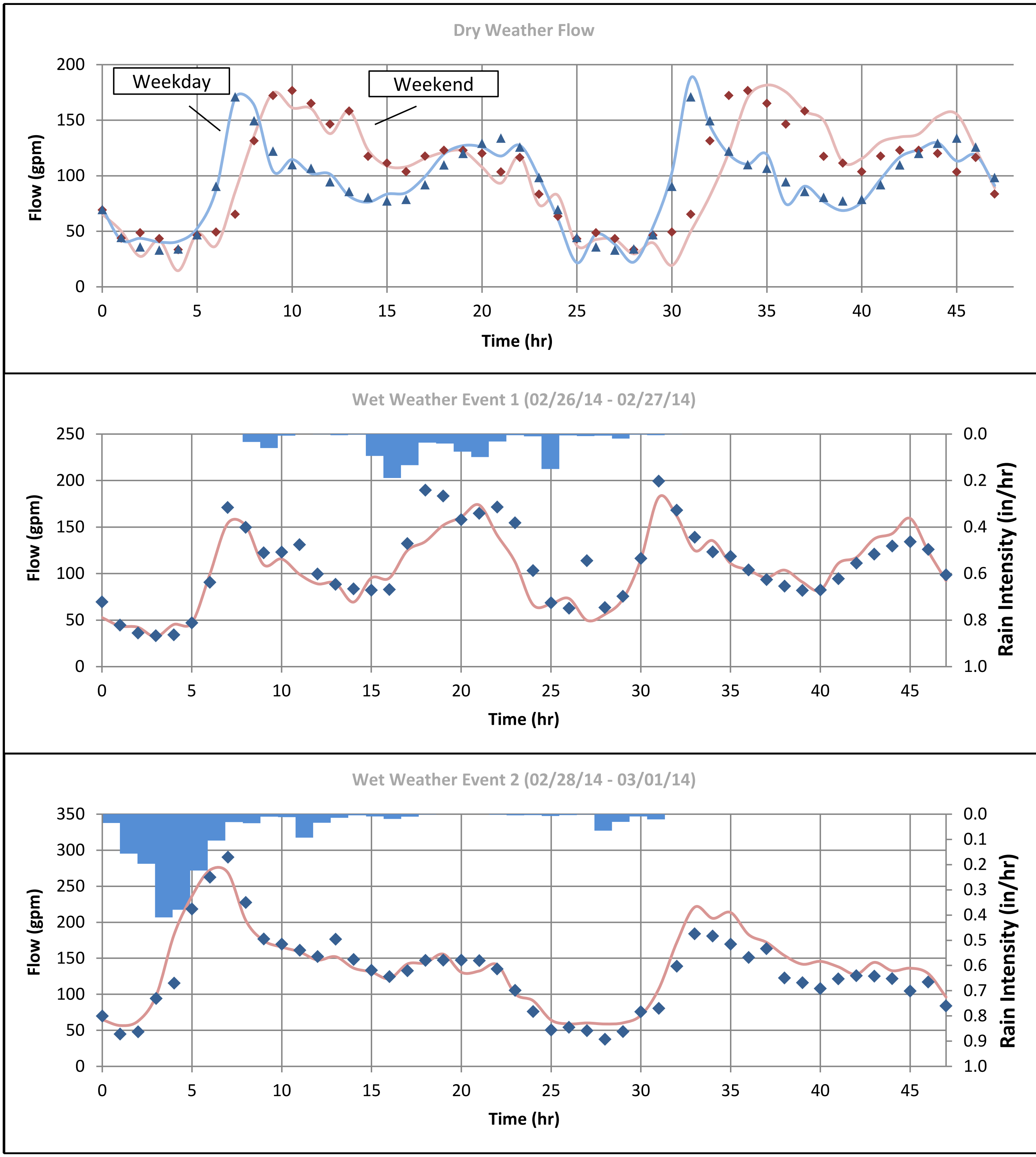
Site 8 Calibration

W. Edmundson Ave and w/o Monterey Rd.

Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.



LEGEND

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

PRELIMINARY

June 29, 2016

Figure 10

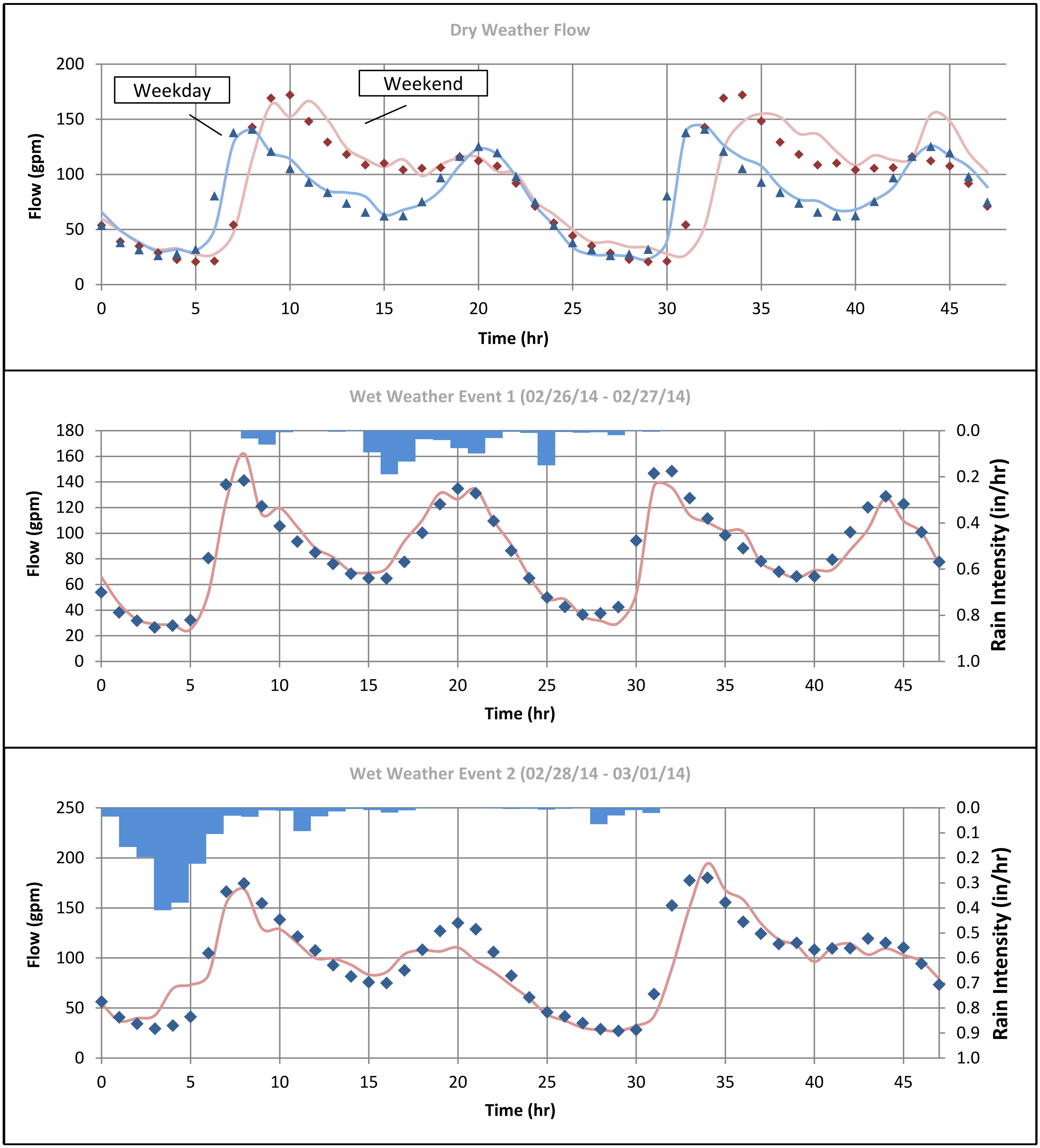
Site 9 Calibration

Hale Ave and n/o Wright Ave.

Sewer System Master Plan

City of Morgan Hill

AKEL ENGINEERING GROUP, INC.



LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

Figure 11

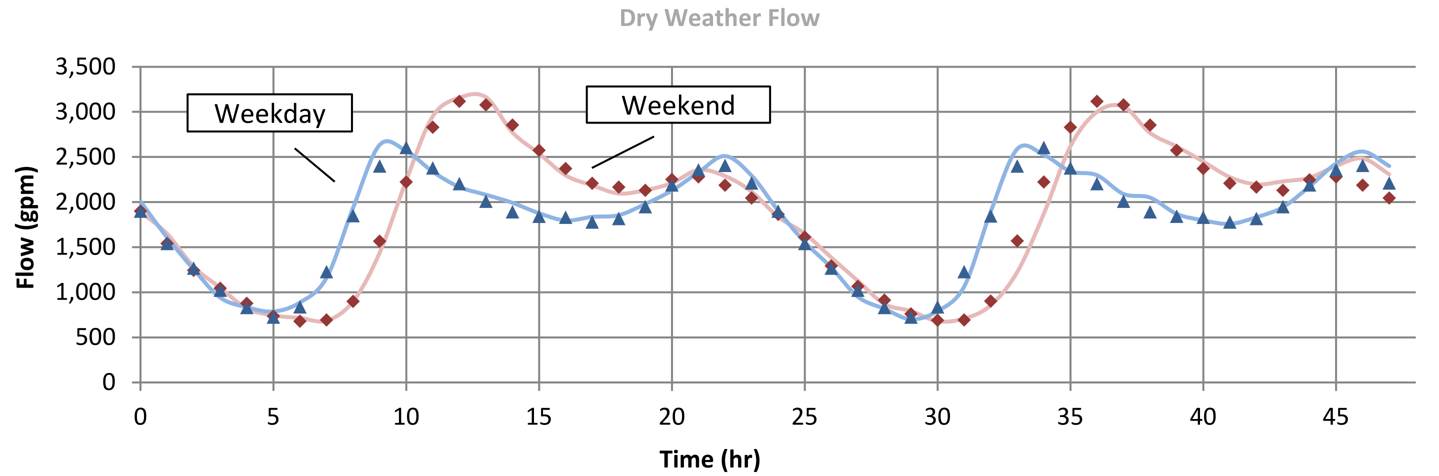
Site 10 Calibration

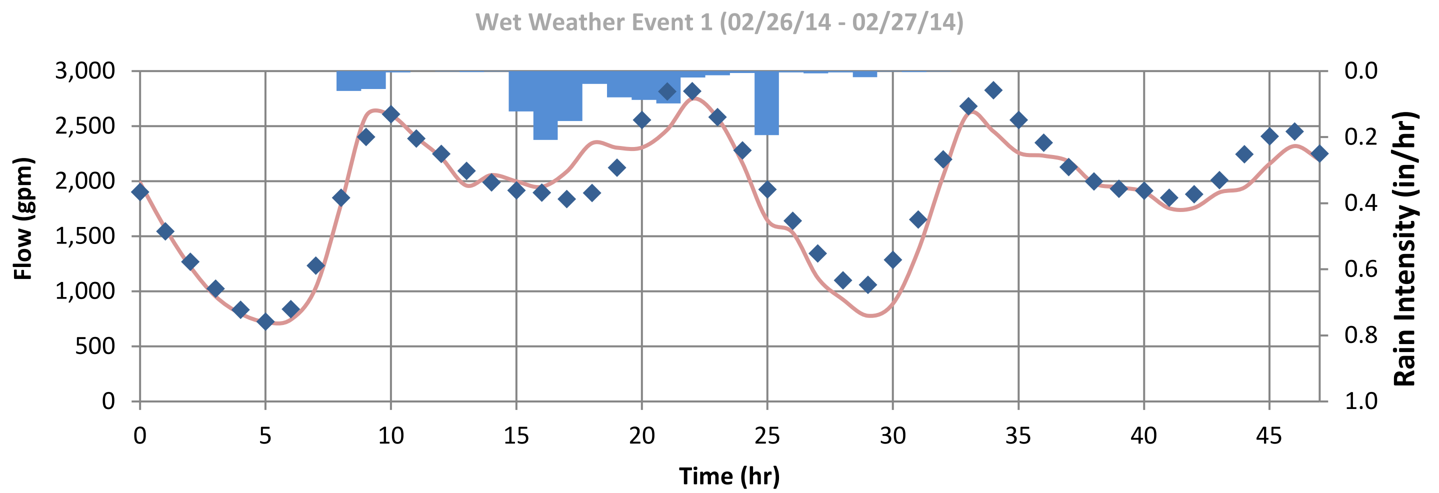
w/o Monterey Rd. and n/o California Ave.

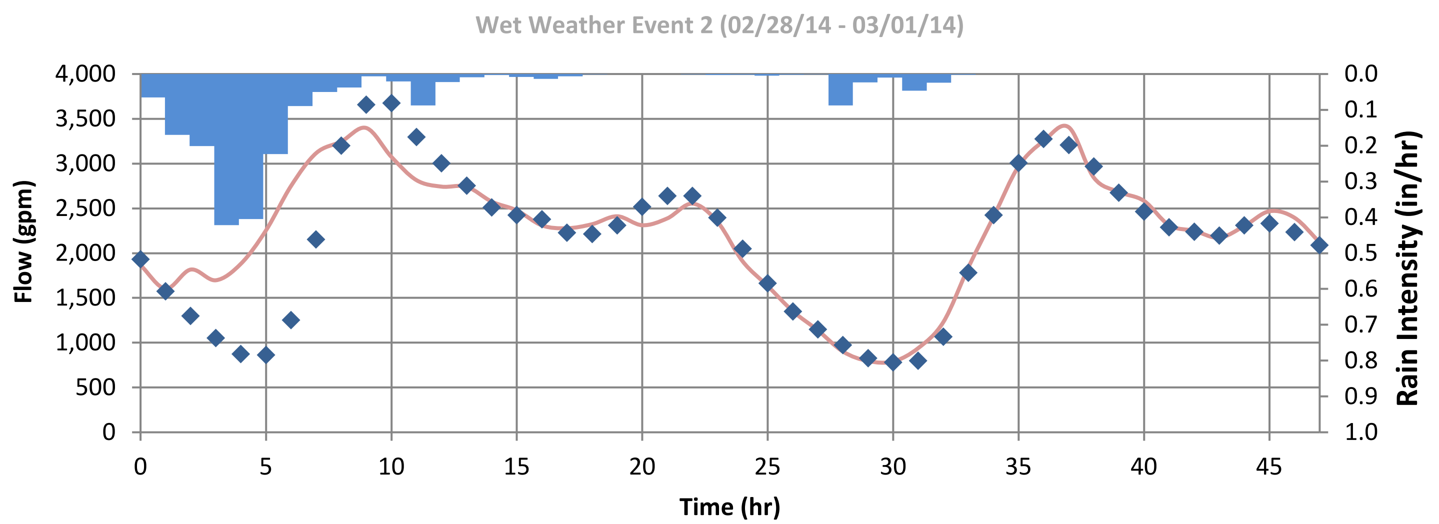
Sewer System Master Plan
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.







LEGEND

PRELIMINARY

Rain Event

Hydraulic Model

V&A Flow Monitoring

V&A Flow Monitoring

June 29, 2016

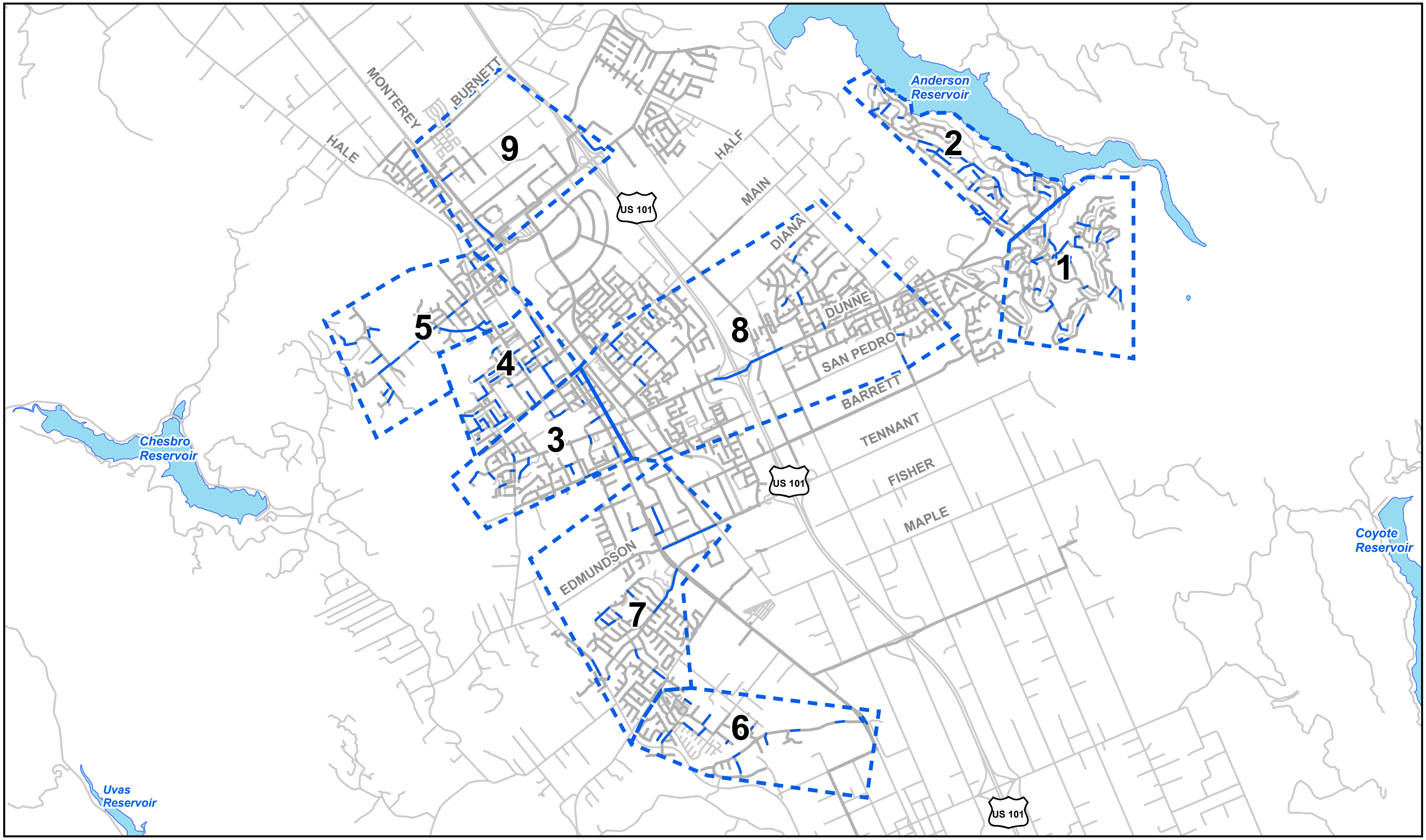
Figure 12
Harding Calibration

Harding Ave. and n/o Highland Ave.

Sewer System Master Plan
City of Morgan Hill

APPENDIX C

Condition Assessment Exhibits



Legend

- R & R Pipes
- Grouping Areas
- Pipes
- Roads

PRELIMINARY

Sewer System Key Map

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

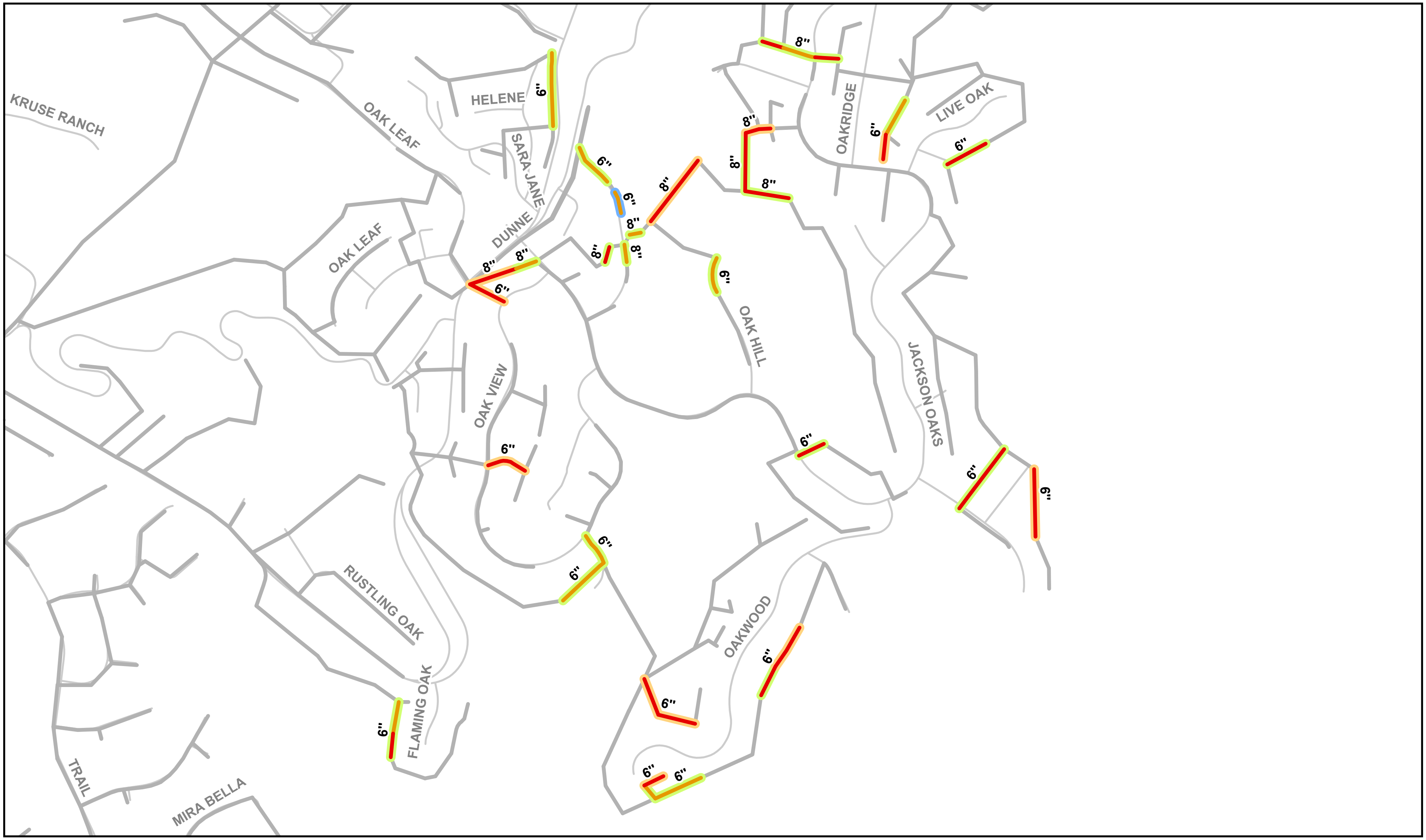
Updated: June 14, 2017

0 0.25 0.5 1 Miles

N W GIS E S

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\Rehab Costs\MH_Fig6-RehabCosts_KeyMap_051517.mxd



Legend

- Extreme Risk
- High Risk
- Trenchless Rehabilitation
- Trenchless Replacement
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	0 LF	Open Cut Replacement	$0
Trenchless Rehabilitation	4,886 LF	Trenchless Rehabilitation	$370,690
Trenchless Replacement	2,739 LF	Trenchless Replacement	$3,694,750
Point Repair	129 LF	Point Repair	$35,390
Grand Total	7,754 LF	Grand Total	$4,101,000

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

0 250 500 1,000 Feet

N W E S GIS

Updated: June 14, 2017

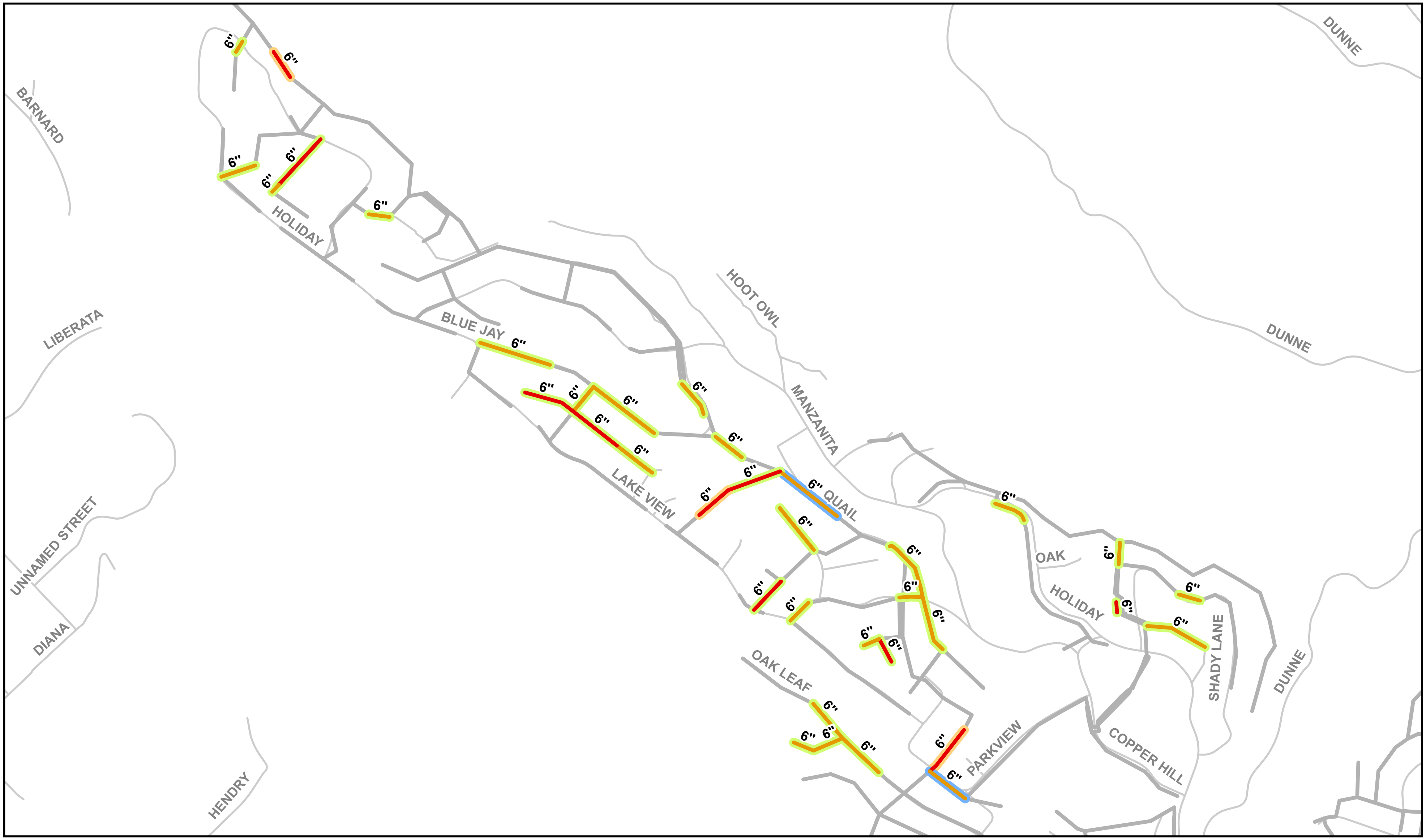
AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\Rehab Costs\MH_Fig6-RehabCosts_Group1_061417.mxd

Group 1
Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL



Legend

- Extreme Risk
- High Risk
- Trenchless Rehabilitation
- Trenchless Replacement
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	0 LF	Open Cut Replacement	$0
Trenchless Rehabilitation	8,192 LF	Trenchless Rehabilitation	$642,860
Trenchless Replacement	831 LF	Trenchless Replacement	$1,113,000
Point Repair	795 LF	Point Repair	$217,590
Grand Total	9,818 LF	Grand Total	$1,973,450

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

Updated: July 14, 2017

0 250 500 1,000 Feet

N W E S GIS

Group 2
Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\Rehab Costs\MH_Fig6-RehabCosts_Group2_061417.mxd

Legend

- Extreme Risk
- High Risk
- Trenchless Rehabilitation
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	0 LF	Open Cut Replacement	$0
Trenchless Rehabilitation	4,759 LF	Trenchless Rehabilitation	$375,380
Trenchless Replacement	0 LF	Trenchless Replacement	$0
Point Repair	871 LF	Point Repair	$238,170
Grand Total	5,630 LF	Grand Total	$613,600

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

N W E S GIS

0 250 500 1,000 Feet

Updated: May 19, 2017

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\MH_Fig6-RehabCosts_041917.mxd

Group 3
Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

Legend

- Extreme Risk
- High Risk
- Moderate Risk
- Open Cut Replacement
- Trenchless Rehabilitation
- Trenchless Replacement
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	567 LF	Open Cut Replacement	$206,840
Trenchless Rehabilitation	8,714 LF	Trenchless Rehabilitation	$375,380
Trenchless Replacement	325 LF	Trenchless Replacement	$434,500
Point Repair	692 LF	Point Repair	$198,120
Grand Total	10,299 LF	Grand Total	$1,533,400

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

0 250 500 1,000 Feet

N E S W GIS

Updated: June 14, 2017

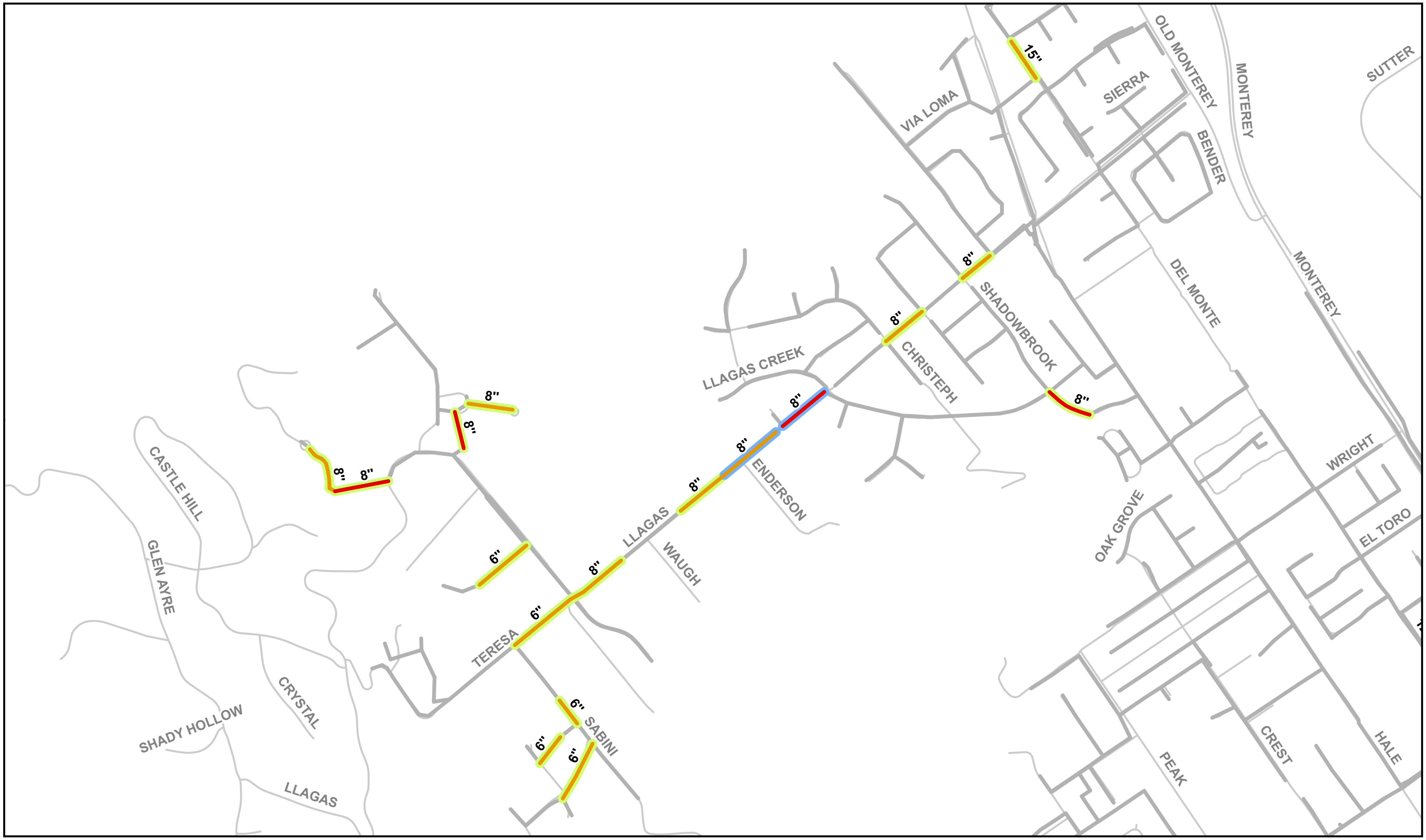
Group 4 Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\Rehab Costs\MH_Fig6-RehabCosts_Group4_061417.mxd



Legend

- Extreme Risk
- High Risk
- Moderate Risk
- Very Low Risk
- Point Repair
- Trenchless Rehabilitation
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	0 LF	Open Cut Replacement	$0
Trenchless Rehabilitation	5,137 LF	Trenchless Rehabilitation	$387,190
Trenchless Replacement	0 LF	Trenchless Replacement	$0
Point Repair	839 LF	Point Repair	$240,400
Grand Total	5,976 LF	Grand Total	$627,590

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

0 250 500 1,000 Feet

N W E S GIS

Updated: July 14, 2017

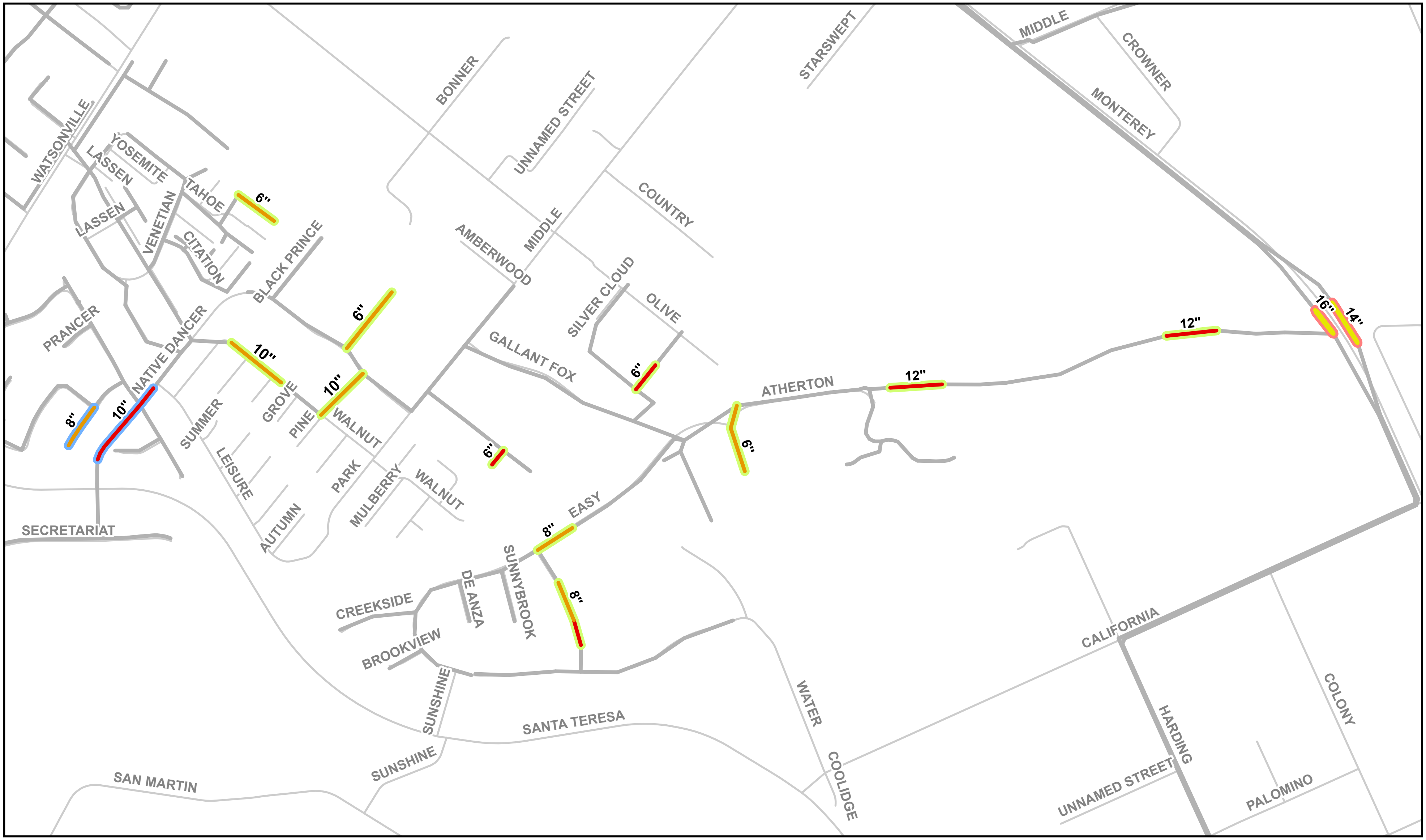
Group 5 Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\Rehab Costs\MH_Fig6-RehabCosts_Group5_061417.mxd



Legend

- Extreme Risk
- High Risk
- Moderate Risk
- Open Cut Replacement
- Trenchless Rehabilitation
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	1,245 LF	Open Cut Replacement	$482,140
Trenchless Rehabilitation	3,447 LF	Trenchless Rehabilitation	$282,560
Trenchless Replacement	0 LF	Trenchless Replacement	$0
Point Repair	833 LF	Point Repair	$244,840
Grand Total	5,525 LF	Grand Total	$1,009,600

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

N W E S GIS

0 250 500 1,000 Feet

Updated: May 19, 2017

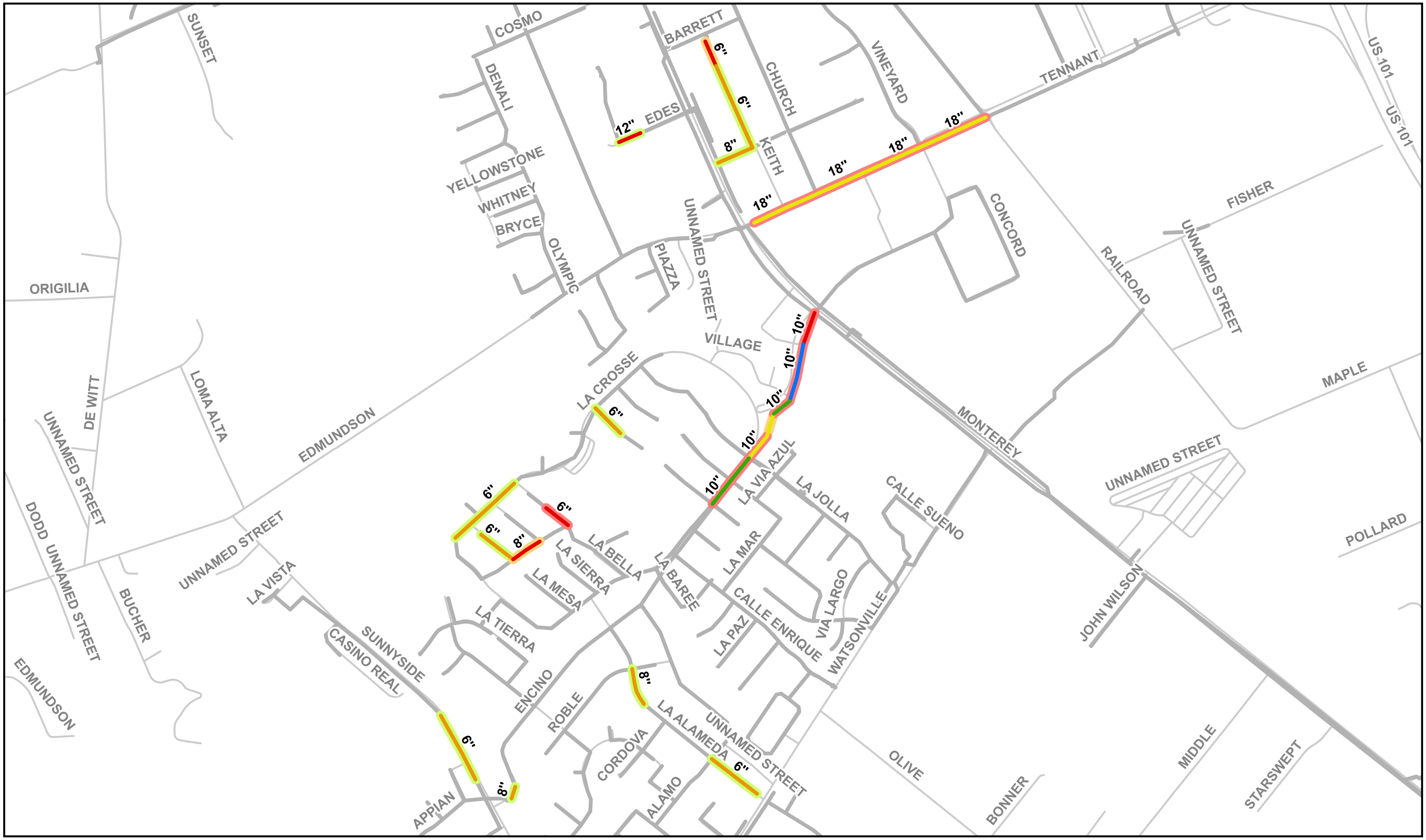
AKEL ENGINEERING GROUP, INC.

Group 6 Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

File Path: P:\xGIS\GIS_Projects\Morgan_Hill\Sewer\170217-SS Cond Risk Assessment\MH_Fig6-RehabCosts_041917.mxd



Legend

- Extreme Risk
- High Risk
- Moderate Risk
- Low Risk
- Very Low Risk
- Open Cut Replacement
- Trenchless Rehabilitation
- Trenchless Replacement
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	4,091 LF	Open Cut Replacement	$1,733,910
Trenchless Rehabilitation	4,138 LF	Trenchless Rehabilitation	$322,440
Trenchless Replacement	702 LF	Trenchless Replacement	$959,500
Point Repair	0 LF	Point Repair	$0
Grand Total	8,931 LF	Grand Total	$3,015,900

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

0 375 750 1,500 Feet

N W E S GIS

Updated: May 19, 2017

AKEL ENGINEERING GROUP, INC.

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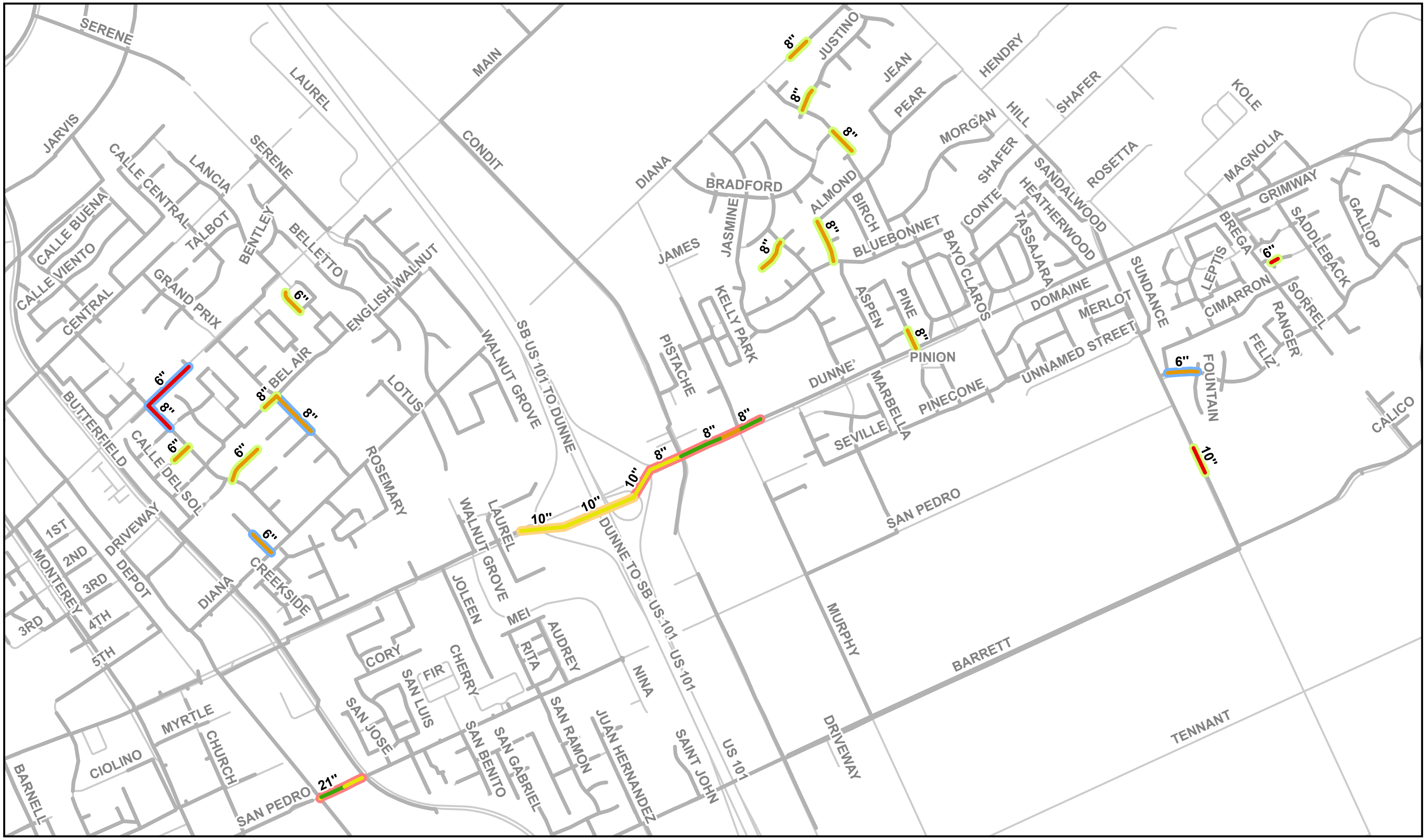
Group 7 Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL



Legend

- Extreme Risk
- High Risk
- Moderate Risk
- Low Risk
- Open Cut Replacement
- Trenchless Rehabilitation
- Trenchless Replacement
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	2,102 LF	Open Cut Replacement	$779,040
Trenchless Rehabilitation	3,178 LF	Trenchless Rehabilitation	$234,260
Trenchless Replacement	1,242 LF	Trenchless Replacement	$1,731,800
Point Repair	2,017 LF	Point Repair	$562,900
Grand Total	8,539 LF	Grand Total	$3,308,000

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

0 375 750 1,500 Feet

N S E W GIS

Updated: May 19, 2017

AKEL ENGINEERING GROUP, INC.

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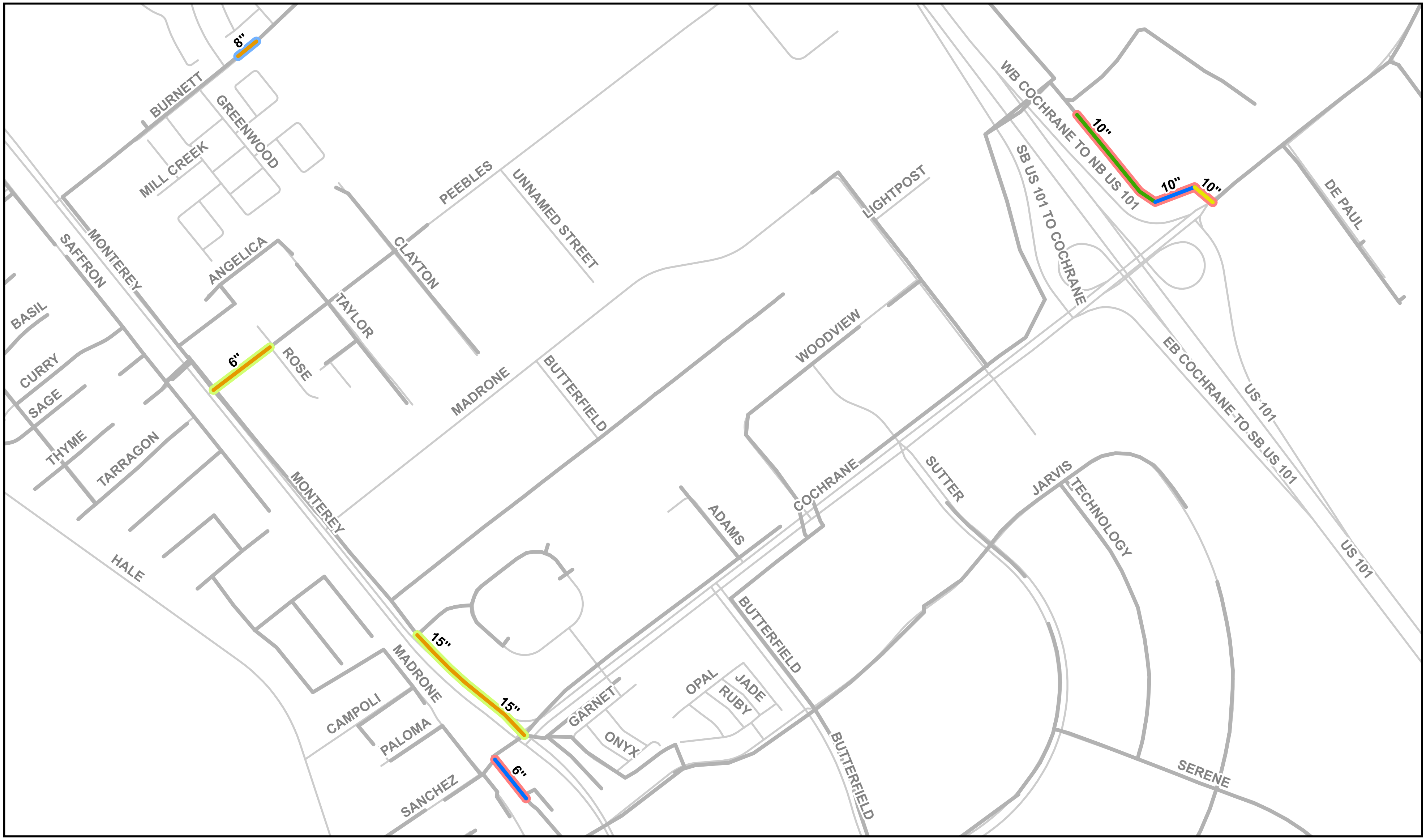
Group 8 Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL



Legend

- High Risk
- Moderate Risk
- Low Risk
- Very Low Risk
- Open Cut Replacement
- Trenchless Rehabilitation
- Point Repair
- Pipes
- Roads

PRELIMINARY

Open Cut Replacement	1,416 LF	Open Cut Replacement	$427,760
Trenchless Rehabilitation	1,321 LF	Trenchless Rehabilitation	$134,010
Trenchless Replacement	0 LF	Trenchless Replacement	$0
Point Repair	143 LF	Point Repair	$40,970
Grand Total	2,880 LF	Grand Total	$602,800

Notes:
1. Costs include master plan contingencies of 30% for unforeseen events and 30% for other costs including: design, environmental, legal, construction management, and project administration.
2. Costs and lengths are preliminary and intended for basic planning purposes. Improvements are subject to revision for both alignment, and rehab method based on more detailed review for design.

N W E S GIS

0 250 500 1,000 Feet

Updated: May 19, 2017

Group 9
Sewer System

Sewer Condition Assessment
City of Morgan Hill

CITY OF MORGAN HILL

AKEL ENGINEERING GROUP, INC.

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