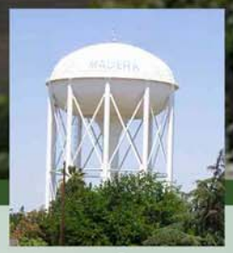
SEPTEMBER

2014

City of Madera



Water System

Sanitary Sewer System

Storm Drainage System

Sanitary Sewer System

Master Plan

FINAL

AKEL ENGINEERING GROUP, INC.

CITY OF MADERA

2014

SANITARY SEWER SYSTEM MASTER PLAN

Final

September 2014

REGISTERED PROFESSIONAL ENGINEER
ANTOINE A AKEL
C 041682
CIVIL
STATE OF CALIFORNIA
09/23/14

AKEL
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Smart Planning Our Water Resources

September 23, 2014

City of Madera
205 W. Fourth Street
Madera, CA 93637

Attention: Keith Helmuth, P.E.
City Engineer

Subject: 2014 Sanitary Sewer System Master Plan – Final Report

Dear Keith:

We are pleased to submit the final report for the City of Madera Sanitary 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.
- The Madera Wastewater Treatment Plant Evaluation and the Schnoor Avenue Sewer Trunk Condition Assessment completed by MWH Americas, Inc.

We extend our thanks to you, Dave Merchen, Community Development Director; Dave Randall, Public Works Director; 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

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Appendix B Hydraulic Model Calibration Exhibits
Appendix C Fairgrounds Lift Station Analysis
Appendix D Schnoor Avenue Trunk Sewer Replacement/Rehabilitation Evaluation (MWH)
Appendix E Madera Wastewater Treatment Plant Evaluation (MWH)

EXECUTIVE SUMMARY

This executive summary presents a brief background of the City of Madera's sanitary 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 Madera Planning Area.

ES.1 STUDY OBJECTIVES

The City of Madera recognizes the importance of planning, developing, and financing the City's sanitary sewer system facilities. In order to continue providing reliable and enhanced service to existing customers and to serve anticipated future developments, City staff initiated the preparation of this 2014 Sanitary Sewer System Master Plan (SSSMP).

City Council approved Akel Engineering Group Inc. to prepare this master plan in January of 2011. This 2014 SSSMP evaluates the City's sanitary sewer system using a hydraulic model and recommends capacity improvements necessary to service the needs of existing users and for servicing the future growth of the City. The 2014 SSSMP 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 Madera.

The Planning 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.

The City of Madera authorized Akel Engineering Group Inc. to complete 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 Sanitary 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
- Sanitary Sewer System Master Plan
- Storm Drainage System Master Plan
- Recycled Water Feasibility 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.

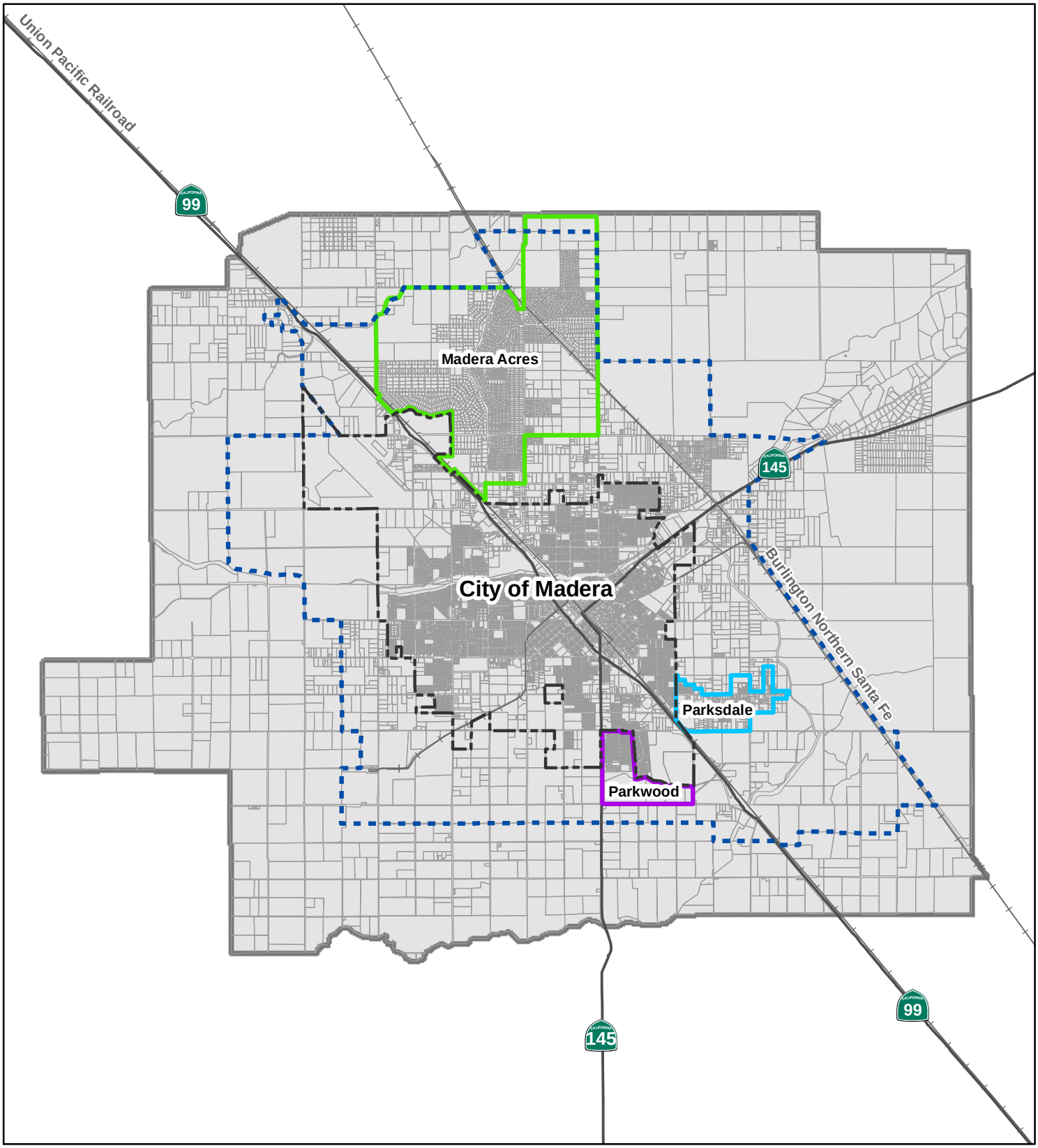
ES.3 STUDY AREA DESCRIPTION

The City of Madera is located in Madera County in the central part of California known as the San Joaquin Valley, approximately 33 miles west of the geographic center of California. The City of Madera serves as the county seat and is located approximately 70 miles southeast of the City of Modesto, 15 miles southeast of the City of Chowchilla, and 22 miles northwest of the City of Fresno. The City limits currently encompass 15.8 square miles, with an approximate population of 62,000 residents.

The City of Madera is generally bound to the north by Avenue 17, to the east by Road 28, to the west by Road 24, and to the south by Avenue 12. There are three unincorporated areas to the north, south, and east of the City respectively: Madera Acres, Parkwood, and Parksdale. State Route 99 bi-sects the City in a northwest to southeast direction and the Fresno River runs in an east-west direction in the northern half of the City. The general topography is generally flat, with very low slope from east to west. **Figure ES.1** displays the planning area showing city limits, the Urban Growth Boundary of the City and the Census Designated Places (CDP).

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.



Legend

- City Limits
- Urban Growth Boundary
- General Plan Area

Census Designated Places

- Parkwood
- Parksdale
- Madera Acres
- Highways
- Railroads
- Parcels

Updated: June 12, 2013

0 0.5 1 2 Miles

N W E S

AKEL ENGINEERING GROUP, INC.

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ES.1
Planning Area

Sanitary Sewer System
Master Plan
City of Madera

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 peak dry weather flows (PDWF), the maximum allowable d/D ratio for proposed pipes 12 inches in diameter and smaller is 0.5, and for proposed pipes larger than 12 inches in diameter, the maximum allowable d/D ratio is 0.75. The maximum allowable d/D ratio for all existing pipes (all diameters) is 0.92. 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 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

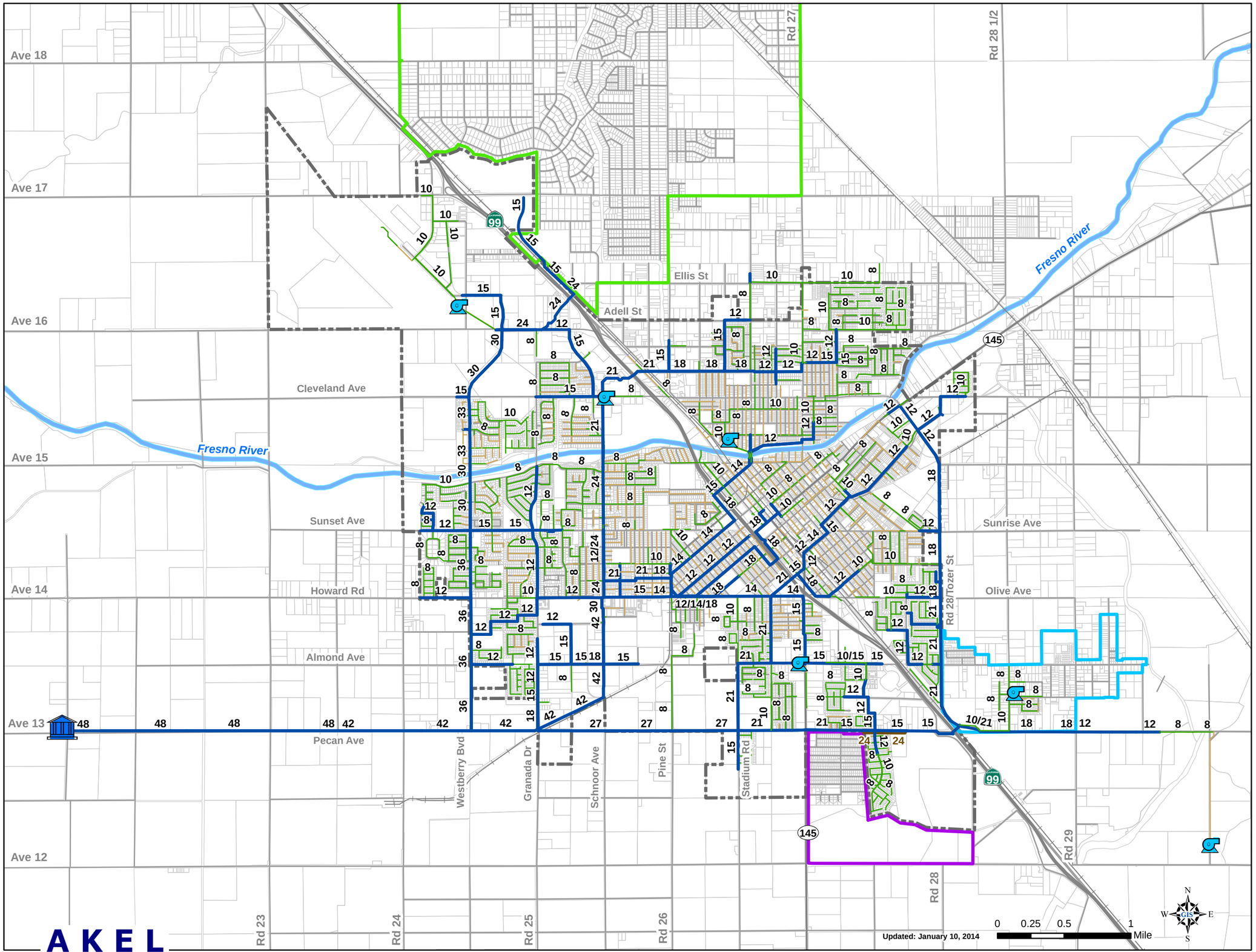
The City provides sewer collection services to approximately 10,000 residential, commercial, industrial, and institutional accounts. The City's collection system consists of approximately 176 miles of up to 48-inch gravity sewer pipes that convey flows towards the Madera Wastewater Treatment Plant (WWTP), on Road 21 ½ and Avenue 13, as shown on **Figure ES.2**.

A system-wide 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 6-inch and 8-inch diameter pipes account for 57 percent of the total sewer pipe lengths.

ES.6 SANITARY SEWER FLOWS

The wastewater flows collected and treated at the Madera 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 and length of storm events. **Table ES.3** shows the flows experienced at the Madera WWTP have increased from 5.05 MGD in 2006 to 5.58 MGD in 2011. In addition to listing the 2006-2011 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



Legend

Existing

- WWTP
- Lift Stations

Pipes

- 6" or Smaller
- 8" - 10"
- 12" or Larger
- Existing Pipe Not In Use
- Roads
- Highways
- Railroads
- Parcels
- City Limits

Census Designated Places

- Parkwood
- Parksdale
- Madera Acres
- Fresno River

ES.2
Existing Sanitary Sewer Collection System
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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Table ES.1 Sewer System Performance and Design Criteria

Sanitary Sewer System Master Plan
City of Madera

Pipeline Criteria

Peak Dry Weather Flow Criteria

Diameter (in)	Maximum Allowable d/D Existing Trunks	Maximum Allowable d/D Proposed Trunks
8 to 12	0.92	0.50
> 12	0.92	0.75

Peak Wet Weather Flow Criteria

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

Pipe Size (in)	Minimum Grade (ft/ft)	Capacity[1] (mgd)	Velocity[1] (fps)
8	0.0034	0.49	2.28
10	0.0025	0.76	2.26
12	0.0022	1.15	2.40
15	0.0015	1.73	2.30
18	0.0012	2.81	2.60
21	0.0010	3.46	2.35
24	0.00080	4.42	2.30
27	0.00080	6.05	2.48
30	0.00080	8.01	2.66
33	0.00080	10.33	2.84
36	0.00080	13.02	3.01
42	0.00080	19.65	3.33
48	0.00070	26.24	3.41

Unit Flow Factor Criteria

Land Use Classification	Unit	Recommended Factor Existing (gpd/unit)	Recommended Factor Future (gpd/unit)
Residential			
Single Family	Gross Acre	1,250	1,200
Multiple Family	Gross Acre	1,500	1,500
Non-Residential			
Commercial	Gross Acre	750	950
Industrial, Light	Gross Acre	750	950
Institutional	Gross Acre	750	950
Mixed Use / Village Reserve[2]	Gross Acre	n/a	950

Notes: 3/20/2013

1. Pipe friction factor assumed at 0.013.
2. The Village Reserve category is intended to incorporate neighborhood planning and village building, and as such has been conservatively estimated as equivalent to commercial use.

Table ES.2 Existing Sewer Pipe Inventory

Sanitary Sewer System Master Plan
City of Madera

Pipe Diameter	Total (ft)	Total (miles)	Percent
Data not available	90,534	17.1	10%
6"	162,054	30.7	17%
8"	375,973	71.2	40%
10"	59,768	11.3	6%
12"	77,144	14.6	8%
14"	12,799	2.4	1%
15"	40,075	7.6	4%
17"	529	0.1	0%
18"	24,296	4.6	3%
21"	26,861	5.1	3%
24"	9,924	1.9	1%
27"	7,985	1.5	1%
30"	9,259	1.8	1%
36"	7,909	1.5	1%
42"	14,382	2.7	2%
48"	10,845	2.1	1%
Total	930,335	**176**	**100%**

Note: 3/20/2013

1. The sewer system pipe inventory was extracted from the City's GIS-based hydraulic model developed in August 2012

Table ES.3 Historical Flows at The Madera Wastewater Treatment Plant

Sanitary Sewer System Master Plan
City of Madera

Year	Average Annual Flow		Seasonal Average		Maximum Month		Maximum Day		Peak Hour	
	AAF	Percent Change	ADWF	AWWF	MMDWF	MMWWF	MDDWF	MDWWF	PDWF	PWWF
Historical Flows										
	(MGD)		(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)
2006	5.05		5.10	5.03	5.12	5.09	6.18	8.23		
2007	5.51	8.3%	5.58	5.48	5.93	6.05	6.40	6.80		
2008	5.82	5.3%	5.87	5.75	6.02	5.80	6.32	6.51		
2009	5.72	-1.7%	5.82	5.68	5.86	5.79	6.19	6.62		
2010	5.68	-0.7%	5.70	5.67	5.79	5.74	6.42	6.57	9.61	11.36
2011	5.58	-1.8%						6.99		10.50
Historical Peaking Factors (applied to AAF)										
2006	1.00		1.01	1.00	1.01	1.01	1.22	1.63		
2007	1.00		1.01	0.99	1.08	1.10	1.16	1.23		
2008	1.00		1.01	0.99	1.03	1.00	1.09	1.12		
2009	1.00		1.02	0.99	1.02	1.01	1.08	1.16		
2010	1.00		1.00	1.00	1.02	1.01	1.13	1.16	1.69	2.00
2011	1.00							1.25		1.88

Notes: 6/5/2013

1. Peaking Factors are multipliers applied to the Average Annual Flow (AAF)
2. Flow Components Definitions

AADF = Average Annual Daily 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, with no evident reaction to rainfall)
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 Hour Dry Weather Flow (highest measured hourly flow that occurs during a dry weather day)
PWWF = Peak Hour Wet Weather Flow (highest measured hourly flow that occurs during a wet weather day)

Table ES.4 Average Daily Flows at Buildout of Project Area

Sanitary Sewer System Master Plan

City of Madera

Classifications	2010 Wastewater Flows at 100% Occupancy					Planning Area Wastewater Flows at 100% Occupancy														
	Existing Areas Currently Serviced within			Wastewater Unit Factor	2010 Average Annual Flows	Total Planning Area (including Madera Acres)		Planned Development within City Limits		Madera Acres Sub Area		Parkwood Sub Area		Parksdale Sub Area		Future Planning Area Development Excluding Sub Areas		Total Areas to be Serviced by City (Excluding Madera Acres		Total Average Annual Flow in Planning Area (Including Madera Acres)
	City Limits	Parkwood	Parksdale			Acreages	Factor	Acreages	Annual Flows	Acreages	Annual Flows	Acreages	Annual Flows	Acreages	Annual Flows[2]	Acreages	Annual Flows	Acreages	Annual Flows	
	(gr. acres)	(gr. acres)	(gr. acres)	(gpd/gr. acre)	(gpd)	(gr. acres)	(gpd/gr. acre)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. Acres)	(gpd)	(gpd)
Residential																				
Single Family Residential	2,995	66	230	1,250	4,114,836	12,867	1,200	881	1,057,287	3,151	3,781,614	66	79,674	147	146,500	5,330	6,395,615	9,716	11,793,912	15,575,526
Multi-Family Residential	272	17	46	1,500	503,549	2,365	1,500	21	32,118	173	259,157	17	25,705	38	38,400	1,779	2,669,038	2,192	3,268,809	3,527,966
Subtotal	*3,268*	*84*	*276*		*4,618,385*	*15,232*		*902*	*1,089,405*	*3,324*	*4,040,771*	*84*	*105,379*	*185*	*184,900*	*7,109*	*9,064,654*	*11,908*	*15,062,722*	*19,103,492*
Non-Residential																				
Commercial	917	3		750	690,187	1,988	950	471	447,775	82	78,027	3	3,266	0	0	511	485,533	1,906	1,626,761	1,704,789
Industrial	523			750	392,555	4,834	950	226	214,611	90	85,247	0	0	0	0	3,995	3,795,653	4,745	4,402,818	4,488,066
Institutional	410	0		750	307,480	580	950	13	12,546	13	12,768	0	326	13	13,400	130	123,153	567	456,905	469,673
Mixed Use / Village Reserve		136		500	67,960	5,893	950		0	786	746,611	136	129,123	0	0	4,835	4,593,021	5,107	4,790,104	5,536,714
Subtotal	*1,850*	*140*	*0*		*1,458,181*	*13,295*		*710*	*674,932*	*971*	*922,653*	*140*	*132,715*	*13*	*13,400*	*9,471*	*8,997,360*	*12,324*	*11,276,588*	*12,199,241*
Other (Non-Flow Generating)																				
Landscape Irrigation	1,276					1,435												1,435		
Other	1,337		6			37,367		308		360		0		0		35,355		37,006		
Subtotal	*2,612*	*0*	*6*			*38,802*		*308*		*360*		*0*		*0*		*35,355*		*38,442*		
Totals	7,730	223	283		**6,076,565**	67,329		1,921	1,764,337	4,656	4,963,424	223	238,094	198	198,300	51,935	18,062,014	62,673	**26,339,310**	31,302,733

Notes:

1. Irrigated landscape is non-flow generating and is categorized under Non-Flow Generating.
2. Parksdale future lands were assumed to develop at 1,000 gpd/acre.

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 2010 flows were increased to 6.1 MGD to account for 100% occupancy, and the ultimate buildout flows were calculated at 26.3 MGD. Madera Acres flows are also listed on this table for completeness though they were excluded from this capacity analysis of the collection system and wastewater treatment plant.

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) 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 Madera 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.

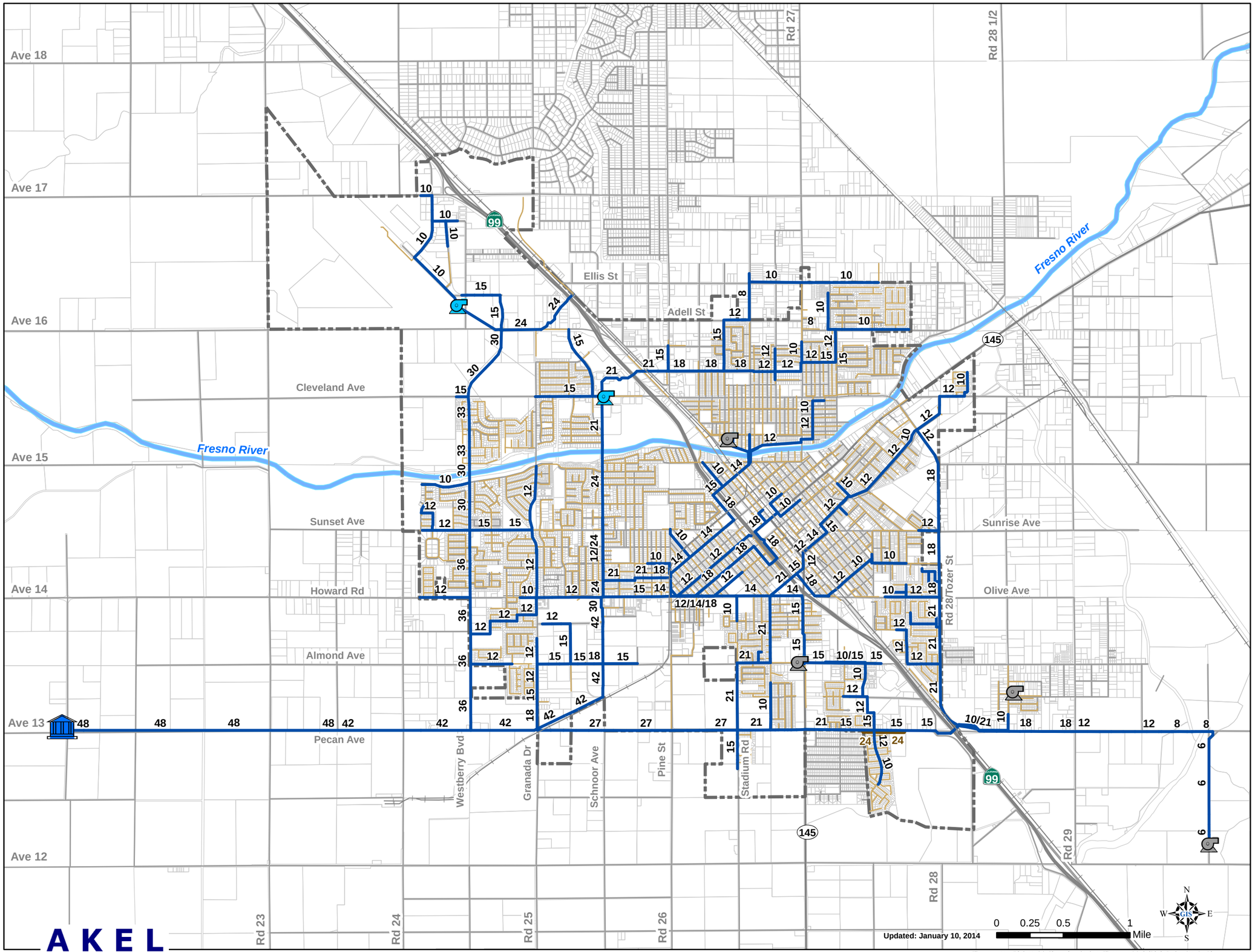
Model Development

City staff completed a GIS mapping project for the sanitary sewer system prior to initiating this master plan project and 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

The City of Madera's sanitary sewer system was skeletonized to reduce the model from approximately 3,180 pipes (176 miles) extracted from the GIS to 940 pipes (55 miles). The modeled pipes included pipes 10-inches in diameter and larger, in addition to some critical smaller gravity sewer pipes. The modeled sewer system is shown on **Figure ES.3**.

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 2011 V&A flow monitoring program and for the following 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



Legend

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes

- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

ES.3
Modeled Sewer System
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

File Name: MA_ES-3_ModeledSewerSys_011014 File Name: P:\xGIS\GIS_Projects\Madera\Sewer

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 peak dry weather flows (PDWF) and peak wet weather flows (PWWF). 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:

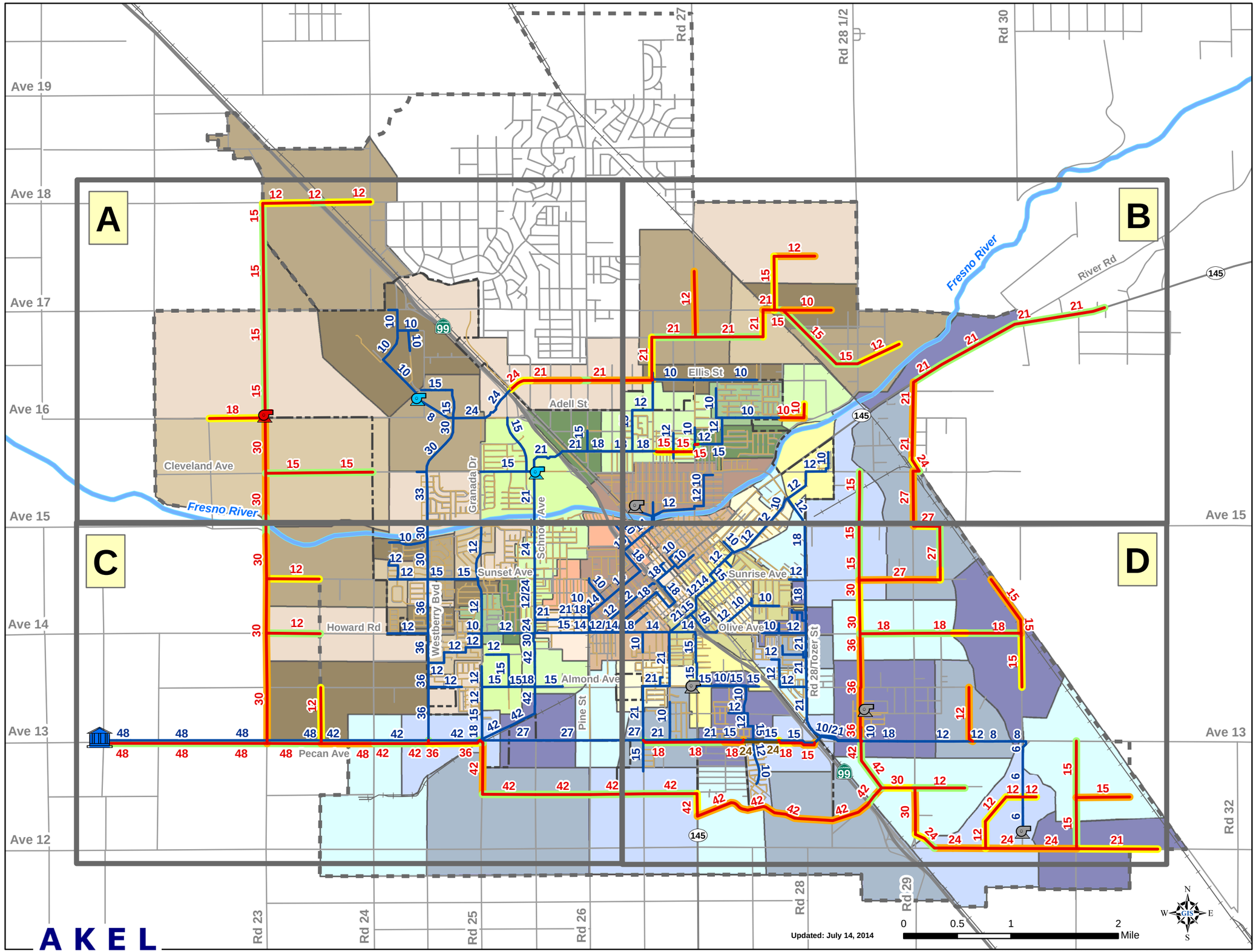
- Existing PDWF = 10.0 MGD
- Existing PWWF = 15.5 MGD
- Buildout PDWF = 44.1 MGD
- Buildout PWWF = 47.1 MGD

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.4**.

This master plan also included a condition assessment of the Schnoor Avenue Sewer Trunk by MHW Americas, Inc. and a wastewater treatment plant capacity evaluation, also completed by MHW Americas, Inc. The condition assessment found the concrete pipes in the Schnoor Trunk were in severe condition with varying degrees of exposed or missing aggregate. The wastewater treatment plant evaluation was completed to identify the current treatment capacity of the WWTP and the impact of the updated buildout sewer flows from the projected population with any recommended improvements.

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 the corresponding detail sheets.



Legend

Improvements

- Lift Station
- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- City Limits
- Urban Growth Boundary
- Fresno River

ES.4
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

Table ES.5 Capital Improvement Program

Sanitary Sewer System Master Plan

City of Madera

Itemized Cost Estimate														Construction Trigger	Suggested Phasing	Village/District	Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]				% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost								Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)						($)	($)
Pecan Basin																				
Pecan Ave Trunk																				
P-1	Pipe	Pecan Avenue	From w/o Highway 99 to 400 ft w/o Golden State Blvd	15	Parallel	15	1,025	132	135,044	135,044	162,053		202,566	1,045 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	202,566
P-1A	Pipe	Pecan Avenue	From 400ft w/o Golden State to 1,360 ft e/o Garnet Ave	15	Parallel	18	775	145	111,988	111,988	134,385		167,981	1,045 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	167,981
P-2	Pipe	Pecan Avenue	From 460ft w/o Garnet Ave to 40 ft w/o Raymond Thomas St	15	Parallel	18	920	145	132,940	132,940	159,528		199,410	1,550 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	199,410
P-3	Pipe	Pecan Avenue	From 40 ft w/o Raymond Thomas St to Stadium Rd (Road 26 1/2)	21	Parallel	18	6,230	145	900,235	900,235	1,080,282		1,350,353	3,710 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	1,350,353
P-4	Pipe	Pecan Avenue	From Granada Dr (Road 25) to Westberry Blvd (Road 24 1/2)	42	Parallel	36	2,670	372	993,240	993,240	1,191,888		1,489,860	22,320 EDUs	FY 2026 - 2030	N/A	0%	100%	0	1,489,860
P-5	Pipe	Pecan Avenue	From Westberry Blvd (Road 24 1/2) to Road 24	42	Parallel	42	2,740	443	1,213,820	1,213,820	1,456,584		1,820,730	33,675 EDUs	FY 2026 - 2030	N/A	0%	100%	0	1,820,730
P-6	Pipe	Pecan Avenue	From Road 24 to Road 23 1/2	42	Parallel	48	2,660	506	1,346,720	1,346,720	1,616,064		2,020,080	17,185 EDUs	FY 2026 - 2030	West Madera, N/A	0%	100%	0	2,020,080
P-7	Pipe	Pecan Avenue	From Road 24 to WWTP	48	Parallel	48	11,280	506	5,710,903	5,710,903	6,853,083		8,566,354	33,675 EDUs	FY 2026 - 2030	N/A	0%	100%	0	8,566,354
			Sub-Total				28,300		$10,544,889	$10,544,889	$12,653,867	**$0**	**$15,817,334**						**$0**	**$15,817,334**
Road 28 1/2 Trunk																				
P-8	Pipe	Road 28 1/2, Parallel to HWY 99	From Pecan to Avenue 12 1/2		New	42	2,560	443	1,134,080	1,134,080	1,360,896		1,701,120	5,570 EDUs or when Ave 12 1/2 trunk is constructed (IMP No. P-26 to P-32)	FY 2016 - 2020	Community College	0%	100%	0	1,701,120
P-9	Pipe	Road 29 1/2	From Almond Ave (Ave 13 1/2) to Pecan Avenue (Ave 13)		New	12	2,750	93	254,375	254,375	305,250		381,563	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	381,563
P-10	Pipe	Road 28 1/2	From Ave 14 to Pecan Ave (Ave 13)	8/10	New/Parallel	36	5,340	372	1,986,480	1,986,480	2,383,776		2,979,720	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	2,979,720
P-11	Pipe	Avenue 14	Road 30 to Road 28 1/2		New	18	7,740	145	1,118,430	1,118,430	1,342,116		1,677,645	Construct with development in Area	FY 2016 - 2050	Parksdale, N/A	0%	100%	0	1,677,645
P-12	Pipe/Casing[5]	Avenue 14	Crossing the Main Canal		New	18/42	150	840	126,000	126,000	151,200		189,000	Construct with development in Area	FY 2016 - 2050	Parksdale, N/A	0%	100%	0	189,000
P-13	Pipe	Road 30	From Ave 13 1/2 ext to Ave 14		New	15	2,640	132	347,820	347,820	417,384		521,730	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	521,730
P-14	Pipe	Santa Fe Railroad ROW	From Ave 14 1/2 ext to Ave 14		New	15	3,130	132	412,378	412,378	494,853		618,566	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	618,566
P-15	Pipe	Road 28 1/2	From Ave 14 1/2 to Ave 14		New	30	2,640	310	818,400	818,400	982,080		1,227,600	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,227,600
P-16	Pipe	Road 28 1/2	From Ave 15 1/2 to Ave 14 1/2		New	15	5,320	132	700,910	700,910	841,092		1,051,365	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,051,365
P-17	Pipe	Avenue 14 1/2	Road 29 1/4 ext to Road 28 1/2		New	27	3,990	279	1,113,210	1,113,210	1,335,852		1,669,815	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,669,815
P-18	Pipe/Casing[5]	Road 29 1/4 extension	Crossing the Main Canal		New	27/54	150	1,080	162,000	162,000	194,400		243,000	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	243,000
P-19	Pipe	Road 29 1/4 extension	Ave 15 to Ave 14 1/2		New	27	2,210	279	616,590	616,590	739,908		924,885	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	924,885
P-20	Pipe	Avenue 15	Ave 29 to Road 29 1/4 ext		New	27	1,290	279	359,910	359,910	431,892		539,865	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	539,865
P-21	Pipe	Road 29	From Ave 15 1/2 to Ave 15		New	27	2,700	279	753,300	753,300	903,960		1,129,950	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	1,129,950
P-22	Pipe	Avenue 15 1/2	From Santa Fe Dr to Road 29		New	24	160	218	34,880	34,880	41,856		52,320	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	52,320
P-23	Pipe/Casing[5]	Avenue 15 1/2	Railroad Crossing		New	24/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	216,000
P-24	Pipe	Road 29	From 660ft n/o Ave 15 1/2 to Ave 15 1/2		New	24	660	218	143,880	143,880	172,656		215,820	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	215,820
P-25	Pipe	Road 29	From River Rd to 660ft n/o Ave 15 1/2		New	21	3,810	180	684,657	684,657	821,588		1,026,986	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	1,026,986
P-26	Pipe	Highway 145	From Juanita Dr to Road 29		New	21	10,240	180	1,840,128	1,840,128	2,208,154		2,760,192	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	2,760,192
			Sub-Total				57,630		$12,751,428	$12,751,428	$15,301,713	**$0**	**$19,127,141**						**$0**	**$19,127,141**
Ave 12 1/2 Trunk																				
P-27	Pipe	Road 25	From Ave 12 1/2 to Pecan Ave (Ave 13)		New	42	2,700	443	1,196,100	1,196,100	1,435,320		1,794,150	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/2	FY 2016 - 2020	N/A	0%	100%	0	1,794,150
P-27A	Pipe/Casing[5]	Road 25	Crossing Railroad Tracks		New	42/66	50	1,320	66,000	66,000	79,200		99,000	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/3	FY 2016 - 2020	N/A	0%	100%	0	99,000
P-28	Pipe	Avenue 12 1/2	From Highway 145 to Road 25		New	42	10,560	443	4,678,080	4,678,080	5,613,696		7,017,120	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/4	FY 2016 - 2020	Parkwood, N/A	0%	100%	0	7,017,120
P-29	Pipe	Highway 145	From 1,110 ft s/o Avenue 12 1/2 to Avenue 12 1/2		New	42	1,110	443	491,730	491,730	590,076		737,595	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/5	FY 2016 - 2020	Parkwood	0%	100%	0	737,595
P-30	Pipe	Borden St/City ROW/Burges	From Ave 12 1/2 to Highway 145		New	42	9,710	443	4,301,426	4,301,426	5,161,711		6,452,139	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/6	FY 2016 - 2020	Parkwood	0%	100%	0	6,452,139
P-31	Pipe/Casing[5]	City ROW near Borden St	Crossing Highway 99 and RR Tracks		New	42/66	300	1,320	396,000	396,000	475,200		594,000	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/5	FY 2016 - 2020	Parkwood, Community College	0%	100%	0	594,000
P-32	Pipe	Avenue 12 1/2	From Road 29 to Borden St Ext		New	30	1,640	310	508,400	508,400	610,080		762,600	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	762,600
P-33	Pipe	Avenue 12 1/2	From Road 29 1/2 extension to Road 29		New	12	2,450	93	226,625	226,625	271,950		339,938	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	339,938
P-34	Pipe	Road 29	From 680 ft n/o Avenue 12 to Avenue 12 1/2		New	30	2,290	310	709,900	709,900	851,880		1,064,850	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,064,850

Table ES.5 Capital Improvement Program

Sanitary Sewer System Master Plan

City of Madera

Itemized Cost Estimate														Construction Trigger	Suggested Phasing	Village/District	Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]				% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost								Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)						($)	($)
P-35	Pipe/Casing[5]	City ROW near Ave 12 and Road 29	Crossing creek and canal		New	24/48	490	960	470,400	470,400	564,480		705,600	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	705,600
P-36	Pipe	City ROW near Ave 12 and Road 29	From 900 ft e/o Road 29 to 675 ft nw/o Ave 12		New	24	680	218	148,240	148,240	177,888		222,360	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	222,360
P-37	Pipe	Avenue 12	From Road 30 1/2 to 900 ft e/o Road 29		New	24	7,000	218	1,526,000	1,526,000	1,831,200		2,289,000	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	2,289,000
P-38	Pipe	Avenue 12 1/2 extension Road 29 1/2 Extension	From 660ft e/o Road 30 then SE to Ave 12		New	12	4,380	93	405,150	405,150	486,180		607,725	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	607,725
P-39	Pipe	Avenue 12	From 3,990 ft e/o Road 30 1/2 to Road 30 1/2		New	21	3,990	180	717,003	717,003	860,404		1,075,505	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,075,505
P-40	Pipe	Road 30 1/2	From Pecan Ave (Ave 13) to Ave 12		New	15	5,290	132	696,958	696,958	836,349		1,045,436	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,045,436
P-41	Pipe	Avenue 12 1/2 extension	From 2,630 ft e/o Road 30 1/2 to Road 30 1/2		New	15	2,630	132	346,503	346,503	415,803		519,754	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	519,754
			Sub-Total				55,270		$16,884,514	$16,884,514	$20,261,417	**$0**	**$25,326,771**						**$0**	**$25,326,771**
Schnoor Basin																				
Sherwood Way Trunk																				
S-1	Pipe	Wessmith Way	190ft e/o Lake St (Road 27) to Lake St (Road 27)	10	Replace	15	190	160	30,400	30,400	36,480		45,600	810 EDUs	FY 2016 - 2050	Central Madera	53%	47%	24,219	21,381
S-2	Pipe	Sherwood Way	Lake St (Road 27) to 220ft w/o Nebraska Ave	12	Replace	15	1,930	160	308,800	308,800	370,560		463,200	1,115 EDUs	FY 2016 - 2050	Central Madera	53%	47%	246,016	217,184
S-3	Pipe	Kennedy Street (Avenue 16)	Road 28 to Chapin St		New	10	1,120	65	73,080	73,080	87,696		109,620	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	109,620
S-4	Pipe	Road 28	680 ft n/o Kennedy St (Ave 16) to Kennedy St (Ave 16)		New	10	680	65	44,370	44,370	53,244		66,555	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	66,555
			Sub-Total				88,390		$22,587,942	$456,650	$547,980	**$0**	**$684,975**						**$270,235**	**$414,740**
Schnoor Trunk																				
S-5	Pipe Rehabilitation - CIPP[6]	Improvements Triggered by Structural Conditions		Clean/Remove Roots/Install Pipe Liner						437,511	482,690		603,363	Existing Structural Deficiency	FY 2013 - 2015	Northwest Madera	100%	0%	603,363	0
S-6	Lift Station[7,8]	Fairgrounds Lift Station Pump Capacity Upgrade			Replace	2,500 gpm				300,000	360,000		450,000	PWWF Deficient	FY 2013 - 2015	Northwest Madera	64%	36%	288,000	162,000
			Sub-Total				3,920		$456,650	$737,511	$842,690	**$0**	**$1,053,363**						**$891,363**	**$162,000**
Westberry Basin																				
Ellis Street Trunk																				
W-3	Pipe	Ellis Street	From Krohn St to Sharon Blvd		New	24	1,070	218	233,260	233,260	279,912		349,890	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	349,890
W-4	Pipe	Ellis Street	From D St to Krohn St		New	21	5,890	180	1,058,433	1,058,433	1,270,120		1,587,650	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	1,587,650
W-5	Pipe/Casing[5]	Ellis Street	Crossing Canal		New	21/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	216,000
W-6	Pipe	D Street	From Martin St to Ellis St		New	21	1,020	180	183,294	183,294	219,953		274,941	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	274,941
W-7	Pipe	Lake Street (Road 27)	From 2,120t n/o Martin St to Martin St		New	12	2,120	93	196,100	196,100	235,320		294,150	Construct with development in Area	FY 2016 - 2050	Central Madera, North Madera	0%	100%	0	294,150
W-8	Pipe	Martin Street Extension	From 3,330 e/o Lake St to D St		New	21	4,340	180	779,898	779,898	935,878		1,169,847	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	1,169,847
W-9	Pipe	Future City Road or ROW (3,330ft e/o Lake Street)	From Avenue 17 to Martin St Ext		New	21	1,290	180	231,813	231,813	278,176		347,720	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	347,720
W-10	Pipe/Casing[5]	Avenue 17	Crossing RR Tracks		New	21/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	216,000
W-11	Pipe	Avenue 17	From 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4) to 560ft w/o RR Tracks		New	21	410	180	73,677	73,677	88,412		110,516	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	110,516
W-12	Pipe	Future Road 27 3/4	From Avenue 17 1/2 Ext to Avenue 17		New	15	2,640	132	347,820	347,820	417,384		521,730	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	521,730
W-13	Pipe	Avenue 17 1/2 extension	Tuolumne St Ext to Future Road 27 3/4		New	12	2,000	93	185,000	185,000	222,000		277,500	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	277,500
W-14	Pipe	Avenue 17	From Harper Blvd to 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4)		New	15	420	132	55,335	55,335	66,402		83,003	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	83,003
W-15	Pipe	Avenue 17	From 2,370 ft e/o Harper Blvd to Harper Blvd		New	10	2,370	65	154,643	154,643	185,571		231,964	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	231,964
W-16	Pipe	Harper Boulevard	From Road 28 1/2 to Avenue 17		New	15	4,750	132	625,813	625,813	750,975		938,719	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	938,719
W-17	Pipe	Future Road or ROW	From Arizona Ave southwest to Road 28 1/2		New	12	2,260	93	209,050	209,050	250,860		313,575	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	313,575
			Sub-Total				30,880		$4,622,135	$4,622,135	$5,546,562	**$0**	**$6,933,203**						**$0**	**$6,933,203**
Road 23 Trunk																				
W-18	Pipe	Road 23	From Ave 16 to Pecan Ave (Ave 13)		New	30	14,950	310	4,634,500	4,634,500	5,561,400		6,951,750	Construct with development in Area	FY 2016 - 2050	Northwest Madera, N/A	0%	100%	0	6,951,750
W-19	Casing	Road 23	Crossing Fresno River		New	30/54	950	1,080	1,026,000	1,026,000	1,231,200		1,539,000	Construct with development in Area	FY 2016 - 2050	Northwest Madera, N/A	0%	100%	0	1,539,000
W-20	Lift Station[7,8]	Avenue 16 and Road 23			New	2,300 gpm				1,125,000	1,350,000	20,000	1,712,500	Construct with development in Area	FY 2016 - 2050	Airport North	0%	100%	0	1,712,500

Table ES.5 Capital Improvement Program

Sanitary Sewer System Master Plan

City of Madera

Itemized Cost Estimate													Construction Trigger	Suggested Phasing	Village/District	Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]			% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost							Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)					($)	($)
W-21	Pipe	Avenue 16	From Road 22 1/2 to Road 23		New	18	2,750	145	397,375	397,375	476,850		596,063	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	596,063
W-22	Pipe	Road 23	From Ave 18 to Ave 16		New	15	10,550	132	1,389,963	1,389,963	1,667,955		2,084,944	Construct with development in Area	FY 2016 - 2050	Northwest Madera	0%	100%	0	2,084,944
W-23	Pipe/Casing[5]	Avenue 18	Crossing HWY 99 and Railroad Tracks		New	12/36	550	720	396,000	396,000	475,200		594,000	Construct with development in Area	FY 2016 - 2050	Airport North	0%	100%	0	594,000
W-24	Pipe	Avenue 18	From Sharon Blvd/Road 24 to Road 23		New	12	4,760	93	440,300	440,300	528,360		660,450	Construct with development in Area	FY 2016 - 2050	Airport North, N/A	0%	100%	0	660,450
W-25	Pipe	Road 23 1/2	From Almond Ave (Ave 13 1/2) to Pecan Ave (Ave 13)		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-26	Pipe	Avenue 14	From Road 23 1/2 to Road 23		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-27	Pipe	Avenue 14 1/2	From Road 23 1/2 to Road 23		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-28	Pipe	Avenue 15 1/2	From Avenue 24 to Road 23		New	15	5,275	132	694,981	694,981	833,978		1,042,472	Construct with development in Area	FY 2016 - 2050	Northwest Madera	0%	100%	0	1,042,472
			Sub-Total				47,585		$9,700,619	$10,825,619	$12,990,743	**$20,000**	**$16,263,428**					**$0**	**$16,263,428**
								Total Improvement Costs					**$85,206,214**					**$1,161,598**	**$84,044,616**

Notes:

1. Cost estimates are based on the Engineering News Record (ENR) construction cost index (CCI) of 9545 for the 20 cities for August 2013.
2. Baseline construction costs plus 20% to account for unforeseen events and unknown conditions.
3. A land acquisition fee for the construction of lift stations was assumed based on recent land acquisitions. It was generally assumed that lift stations will require 0.5 acre.
4. Estimated construction cost plus 25% to cover other costs including: engineering design, project administration (developer and City staff), construction management and inspection, and legal costs.
5. Proposed casings size and carrier pipe size.
6. Pipe rehabilitation contingencies are baseline plus 10% for unforeseen events and 25% to cover other costs.
7. Lift Station capacity is given as capacity with largest pump out of service.
8. Lift station pricing can vary widely with site conditions.

6/7/2013

The estimated construction costs include the baseline costs plus **20 percent** contingency allowance to account for unforeseen events and unknown field conditions. Capital improvement costs include the estimated construction costs plus **25 percent** project-related costs (engineering design, project administration, construction management and inspection, and legal costs).

The costs in this Sanitary Sewer System Master Plan were benchmarked using a 20-City national average ENR CCI of 9545, reflecting a date of August 2013. In total, the CIP includes approximately 59 miles of pipeline improvements, 1 lift station capacity upgrade, 1 new lift station, and pipe rehabilitation along Schnoor Avenue based on structural condition, with a project cost totaling over $85.2 million dollars.

CHAPTER 1 - INTRODUCTION

This chapter provides a brief background of the City of Madera's (City) sanitary 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 Madera is located approximately 70 miles southeast of the City of Modesto, 15 miles southeast of the City of Chowchilla, and 22 miles northwest of the City of Fresno (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 175 miles of gravity trunks and force mains, with up to 48-inch pipe sizes, which convey the flow to the Madera Wastewater Treatment Plant (WWTP). The WWTP has an average daily capacity rating of 10.1 million gallons per day (MGD).

In 1997, the City of Madera 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 Madera Planning Area.

Recognizing the importance of planning, developing, and financing system facilities to provide reliable sanitary sewer service to existing customers and for servicing anticipated growth within the Madera Planning Area, the City initiated updating elements of the 1997 Sanitary Sewer Master Plan, to reflect current land use conditions.

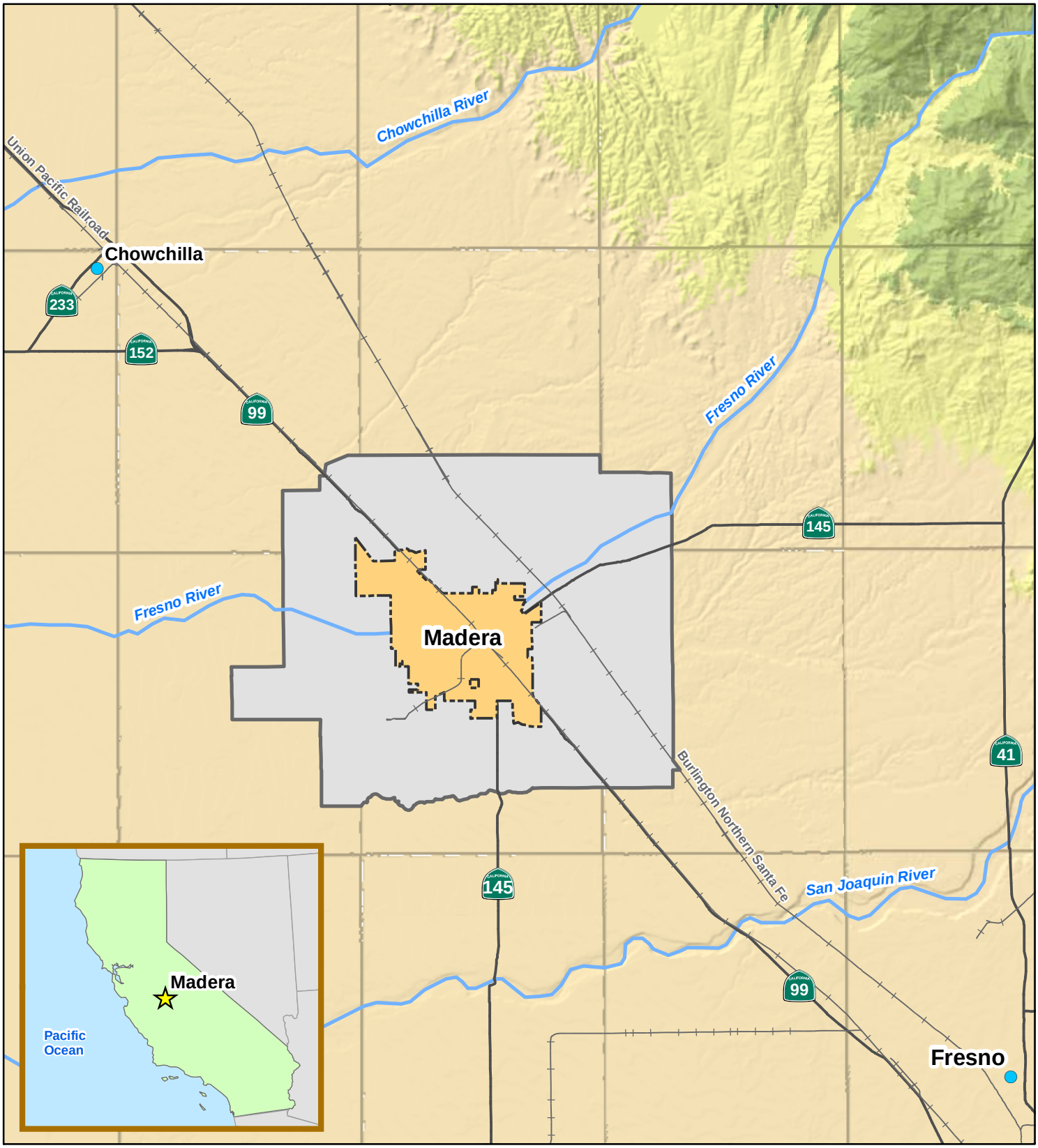
1.2 SCOPE OF WORK

City Council approved Akel Engineering Group Inc. to prepare this master plan in January of 2011. This 2014 Sanitary Sewer System Master Plan (SSSMP) 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 Madera. The 2014 SSSMP 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.

The Planning 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.

The City of Madera authorized Akel Engineering Group Inc. to complete the following tasks:

- Summarize the City's existing sanitary sewer system facilities.
- Document growth planning assumptions and known future developments.



Legend

- Cities
- City Limits
- General Plan Area

Elevation (ft)

- < 150
- 151 - 500
- 501 - 1,000
- 1,001 - 1,500
- 1,501 - 2,000
- > 2,000
- Highways
- Railroads
- Rivers

Updated: June 12, 2013

0 1.25 2.5 5 Miles

N W E S

AKEL ENGINEERING GROUP, INC.

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Figure 1.1
Regional Location Map
Sanitary Sewer System
Master Plan
City of Madera

- 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 Sanitary 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
- Sanitary Sewer System Master Plan
- Storm Drainage System Master Plan
- Recycled Water Feasibility 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 PREVIOUS MASTER PLANS

The City's most recent sewer master plan was completed in 1997. This master plan included evaluation of servicing growth to the planning area, evaluated existing sewer flows and projected future flows and recommended phased improvements to the sewer system for a horizon year of 2020. Additionally, the 1997 master plan included the development of the hydraulic model which was used for evaluating the sewer system. Improvements were recommended for servicing existing and future growth areas, and a corresponding Capital Improvement Program was developed to quantify the corresponding costs.

1.5 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:

- Sewer System Master Plan, February 1997 (1997 SSMP). This report documents the planning and performance criteria, evaluates the sewer system, recommends improvements, and provides an estimate of costs.
- City of Madera Draft Specific Plan #1 "Airport Area" Infrastructure Master Plan (2010 Airport Plan). This report was drafted in February of 2010 to master plan the airport area and relevant infrastructure necessary to accommodate potential growth. This report included sizing and pipeline alignments for the sanitary sewer system, and provided a basis for projected future sewer infrastructure improvements in the area.
- Sharon Boulevard Plan Line Alignment Alternatives for Water, Sanitary Sewer, and Storm Drain Facilities Planning (2010 Sharon Boulevard Report). This final draft report was completed in September 2010, and contained proposed improvements for the Sharon Boulevard area, and included sanitary sewer improvements. This report was referenced during the preparation of this master plan for potential pipeline improvements.
- City of Madera General Plan, October 2009 (2009 General Plan). The City's 2009 General Plan provides future land use planning, and growth assumptions for the Planning Area. Additionally, this report establishes the planning horizon for improvements in this master plan.

1.6 REPORT ORGANIZATION

The Sanitary Sewer System Master Plan report contains the following chapters:

Chapter 1 – Introduction. This chapter provides a brief background of the City of Madera's sanitary 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 Madera.

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 includes: 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 Madera Wastewater Treatment Plant.

Chapter 5 – Sanitary Sewer Flows. This chapter summarizes historical wastewater flows experienced at the Madera 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 Madera'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.

1.7 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:

- Dave Merchen, Community Development Director
- Keith Helmuth, City Engineer
- Dave Randall, Director of Public Works
- Ellen Bitter, Project Development Coordinator
- Eric Portnoff, Engineering Technician

- Ehab Ibrahim, Engineering Technician
- Jose Aguilar, Deputy City Engineer
- Danny Martin, Water & Sewer Operations Manager
- Bob Mack, Streets Operations Manager
- Matt Bullis, Former Public Works Director

As part of the preparation of this Sanitary Sewer System Master Plan, MWH Americas prepared reports for the Schnoor Avenue Condition Assessment and the Madera Wastewater Treatment Plant Evaluation. MWH Americas was a valuable partner in the preparation of this master plan.

1.8 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 wastewater system terminologies and engineering units. A list of abbreviations and acronyms is included in **Table 1.2**.

1.9 GEOGRAPHIC INFORMATION SYSTEMS

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

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

Table 1.1 Unit Conversions

Sanitary Sewer System Master Plan
City of Madera

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

3/20/2013

Table 1.2 Abbreviations and Acronyms

Sanitary Sewer System Master Plan
City of Madera

Abbreviation	Expansion	Abbreviation	Expansion
2014 SSSMP	2014 Sanitary Sewer System Master Plan	gpd	Gallons per Day
10Yr-24Hr	10-Year 24-Hour	gpm	Gallons per Minute
AACE International	Association for the Advancement of Cost Engineering	HGL	Hydraulic Grade Line
ADWF	Average Dry Weather Flow	in/hr	Inch per Hour
AAF	Annual Average Flow	I&I	Infiltration and Inflow
Akel	Akel Engineering Group, Inc.	LF	Linear Feet
AWWF	Average Wet Weather Flow	MDDWF	Maximum Day Dry Weather Flow
BWF	Base Wastewater Flow	MDWWF	Maximum Day Wet Weather Flow
CCI	Construct Cost Index	MGD	Million Gallons per Day
CCTV	Closed Circuit Television	MMDWF	Maximum Month Dry Weather Flow
CDP	Census Designated Place	MMWWF	Maximum Month Wet Weather Flow
CIP	Capital Improvement Program	MWH	MWH Americas Inc.
City	City of Madera	NOAA	National Oceanic and Atmospheric Administration
CSA3	County Service Area 3	PWSS	Public Water System Statistics
DDF	Depth Duration Frequency	PDWF	Peak Dry Weather Flow
d/D	depth of flow to pipe diameter	PWWF	Peak Wet Weather Flow
EDUs	Equivalent Dwelling Units	ROW	Right of Way
ENR	Engineering News Record	SCADA	Supervisory Control and Data Acquisition
fps	Feet per Second	VCP	Vitrified Clay Pipe
FY	Fiscal Year	V&A	Villalobos and Associates
GIS	Geographic Information Systems	WWTP	Wastewater Treatment Plant

6/17/2013

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 Madera.

2.1 STUDY AREA DESCRIPTION

The City of Madera is located in Madera County in the central part of California known as the San Joaquin Valley, approximately 33 miles west of the geographic center of California. The City of Madera serves as the county seat and is located approximately 140 miles southeast of the City of Sacramento and 220 miles northwest of the City of Los Angeles. State Route 99 runs in a north-south direction and the Fresno River runs in an east-west direction, dividing the City into four quadrants. The City Limits currently encompass 15.8 square miles, with an approximate population of 62,000 residents.

The City of Madera is generally bound to the north by Avenue 17, to the east by Road 28, to the west by Road 24, and to the south by Avenue 12. The City has several small creeks to the north and south of the City Limits, and the Fresno River bisects the City from the east. The topography is generally flat, with very low slope from east to west. There are three unincorporated, Census Designated Places (CDP) to the north, south, and east of the City respectively: Madera Acres, Parkwood, and Parksdale. **Figure 2.1** displays the planning area showing City Limits, the Urban Growth Boundary of the City, the Planning Area, and the Census Designated Places.

The City operates and maintains a sanitary sewer collection system that covers the majority of the area within the City Limits. Currently, the wastewater flows are conveyed to the Madera Wastewater Treatment Plant.

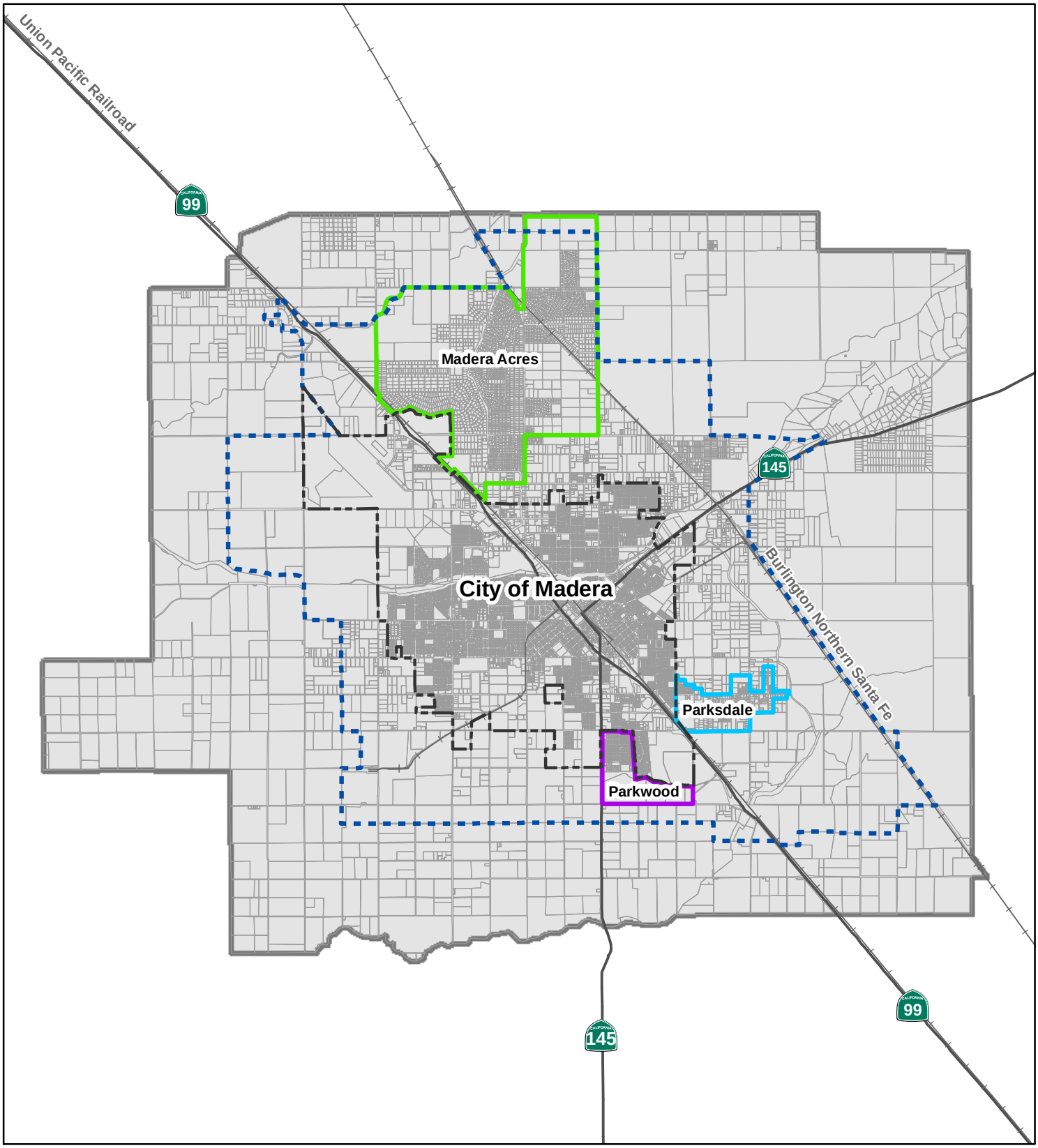
2.2 SEWER SERVICE AREAS AND LAND USE

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

- 7,730 acres of developed lands inside the City limits.
- 1,921 acres of undeveloped lands inside the City limits.

The existing land use statistics were based on the most recent assessor's parcel map, as shown on **Figure 2.2**.

There are three census designated places in the Planning Area located outside the current City limits. These areas, which consist of Parkwood, Parksdale, and Madera Acres, and are described as follows:



Legend

- City Limits
- Urban Growth Boundary
- General Plan Area

Census Designated Places

- Parkwood
- Parksdale
- Madera Acres
- Highways
- Railroads
- Parcels

N W E S

Updated: June 12, 2013

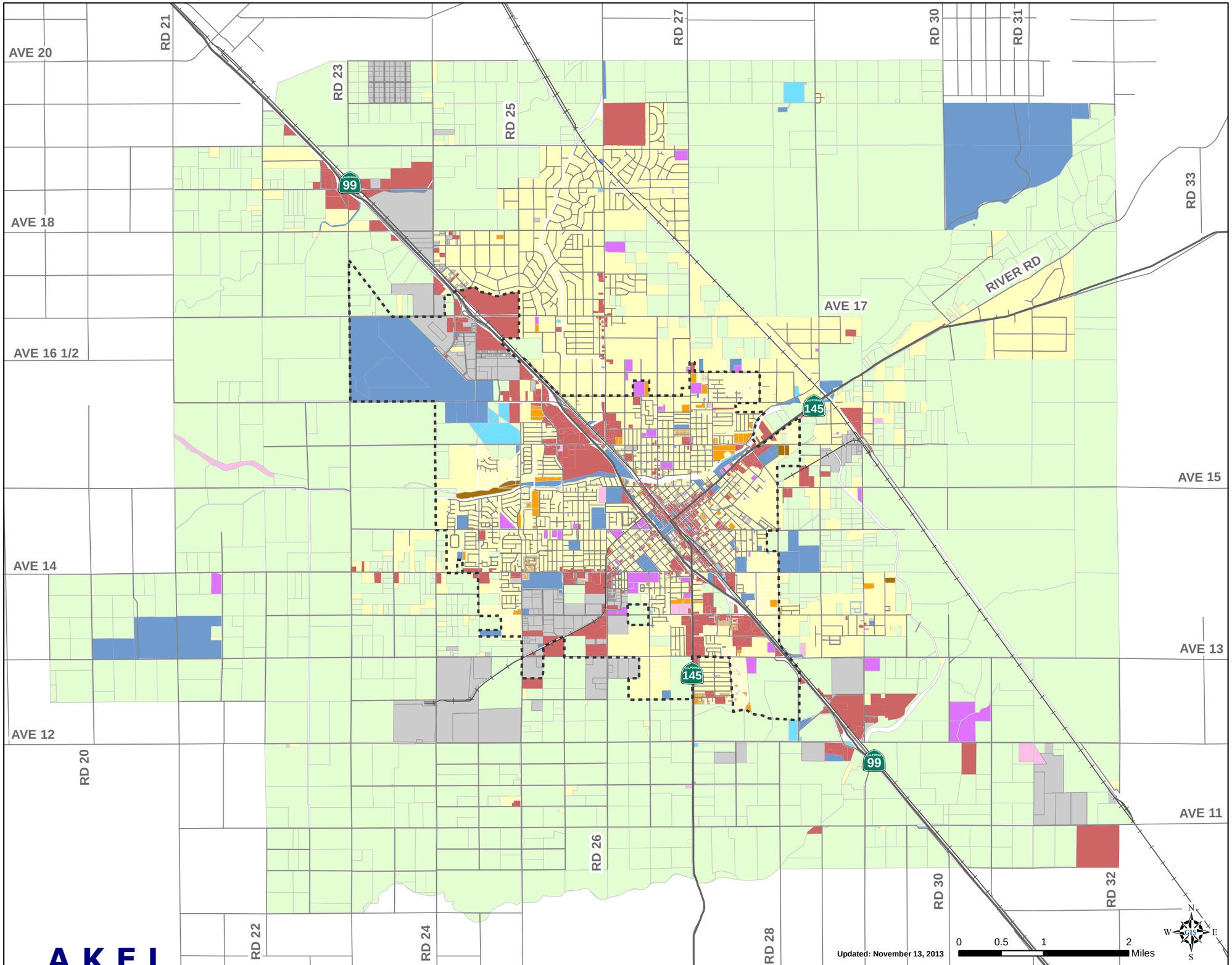
0 0.5 1 2 Miles

AKEL ENGINEERING GROUP, INC.

File Name: MA_Fig2-1PlanArea_061213 File Location: P:\xGIS\GIS_Projects\Madera\MP\Sewer

Figure 2.1
Planning Area

Sanitary Sewer System
Master Plan
City of Madera



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Legend

Existing Land Use

- Commercial
- Government
- Industrial
- Low Density Residential
- High Density Residential
- Institutional
- Recreational
- Agricultural
- Other
- Unknown
- Roads
- Highway
- Railroads
- City Limits

Figure 2.2
Existing Land Use
Sanitary Sewer System Master Plan
City of Madera

AKEL
ENGINEERING GROUP, INC.

Table 2.1 Existing and Future Sewer Service Areas

Sanitary Sewer System Master Plan
City of Madera

Land Use Classification	Existing Service Area					Currently Serviced by Others			Planning Area Boundary[4]	
	City Limits		Outside City Limits		Total	Sub Areas			Excluding Madera Acres	Including Madera Acres
	Developed (gross acres)	Undeveloped (gross acres)	Parkwood[2] (gross acres)	Parksdale[3] (gross acres)	(gross acres)	Madera Acres (gross acres)	Parkwood[2] (gross acres)	Parksdale[3,5] (gross acres)	(gross acres)	(gross acres)
Residential										
Very Low Density Residential	2,995	881	0	205		2,478		144		5,661
Low Density Residential			66	25		673	66	3		7,206
Subtotal - Single Family Residential	2,995	881	66	230	3,292	3,151	66	147	9,716	12,867
Med Density Residential	272	21	17	3		142	17	38		1,947
High Density Residential			0	44		31		0		418
Subtotal - Multi-Family Residential	272	21	17	46	336	173	17	38	2,192	2,365
Subtotal- Residential	***3,268***	***902***	***84***	***276***	***4,530***	***3,324***	***84***	***185***	11,908	***15,232***
Non-Residential										
Commercial	917	471	3			82	3		1,660	1,742
Office									247	247
Industrial	523	226				90			4,745	4,834
Neighborhood Mixed Use	0					9			54	63
Village Mixed Use	0								37	37
Public / Semi Public	410	13	0.3			13	0.3	13	567	580
Village Reserve	0		136			777	136		5,015	5,792
Subtotal	***1,850***	***710***	***140***	***0***	***2,700***	***971***	***140***	***13***	12,324	***13,295***
Other (Non-Flow Generating)										
Landscape Irrigation	1,276					0				1,435
Open Space[4]	12	41				159				1,881
Resource Conservation / Ag	1,021	151		6		177				35,005
Other	303	117				24				481
Subtotal	*2,612*	***308***	*0*	*6*		***360***	***0***	***0***	38,442	***38,802***
Total	**7,730**	**1,921**	**223**	**283**	***10,157***	**4,656**	**223**	**198**	**62,673**	**67,329**

Notes:

3/20/2013

1. Existing Public/Semi-Public land use includes parks and schools. Approximately 199 acres are considered to be schools.
2. Parkwood is assumed at half of the acreage serviced by private sewer, and half serviced by City sewer.
3. Parksdale acreages extracted from CSA3 -Infiltration and Inflow Sewer Capacity Study Executive Draft May 2010, completed by Carollo Engineers.
4. Other includes areas designated for Highway 99, and the Fresno River.

- **Parkwood** – The Parkwood CDP is located near the south-east corner of the City of Madera in Madera County just south of Avenue 13 and west of State Highway 99. The Parkwood area encompasses approximately 446 acres and approximately half of this area (223 acres) is served by the City of Madera WWTP while the other half is serviced by the small Parkwood WWTP.

- **Parksdale (County Service Area 3)** – The Parksdale CDP also known as County Service Area No. 3 (CSA 3) is located in Madera County on the south-east corner of the City of Madera, just east of State Highway 99 and north of Avenue 13. The Parksdale area encompasses approximately 481 acres with 283 developed acres and 198 undeveloped acres. Parksdale sewer is serviced by 8.4 miles of pipelines and one sewer lift station at the intersection of Road 28 ½ and Avenue 13 ½. The lift station discharges into the City of Madera's sanitary sewer system under an agreement with the City.

- **Madera Acres** – The Madera Acres CDP is north of the City of Madera on the east side of State Highway 99 in Madera County. Madera Acres consists of approximately 4,656 acres. Madera Acres sewer is not serviced by the City of Madera.

At ultimate development of the General Plan, the City's wastewater system is anticipated to service approximately 11,908 acres of residential land use, 12,324 acres of non-residential land use, and 38,442 of non-flow generating land use, for a total of 62,673 acres inside the planning area, and not including Madera 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**.

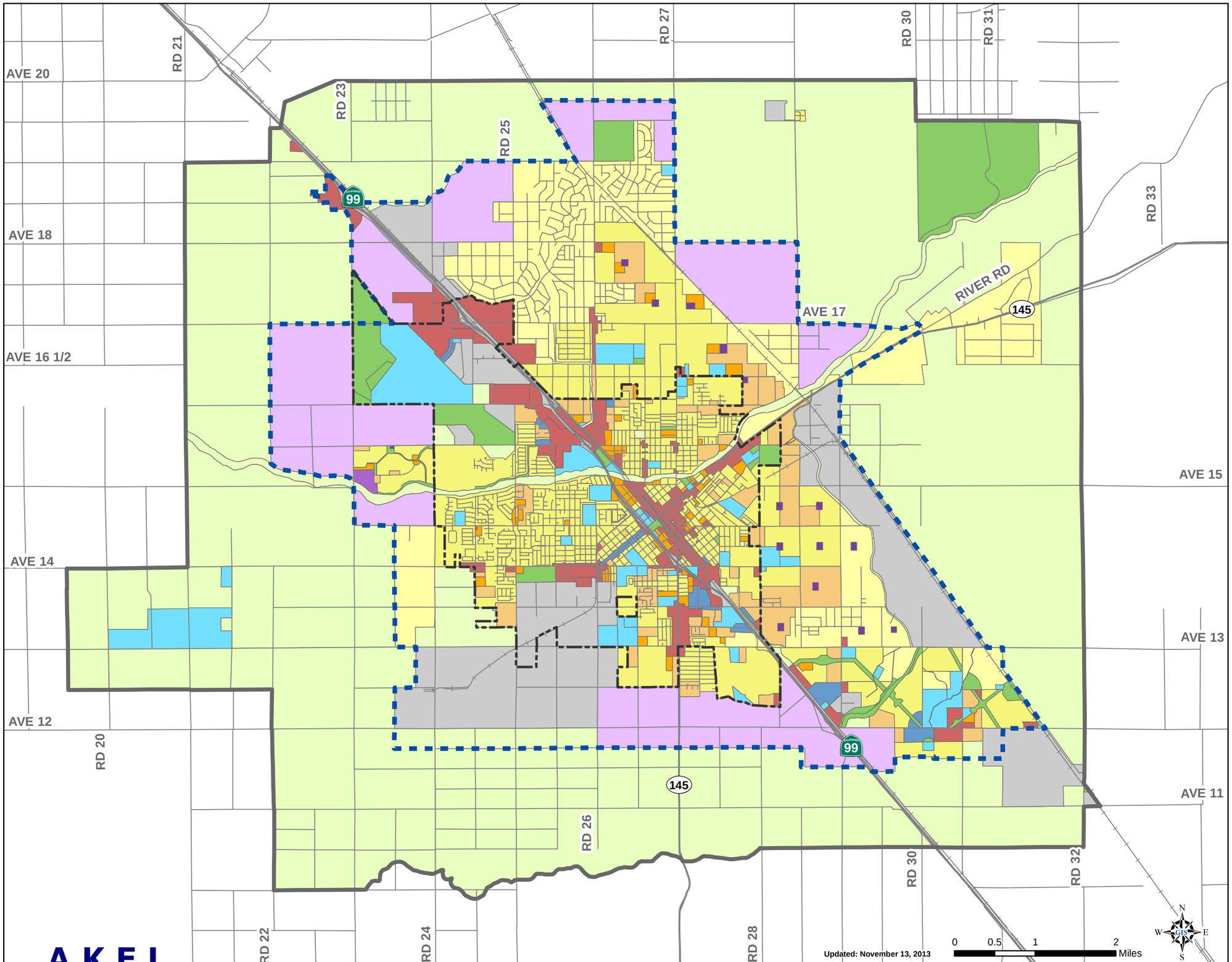
2.3 VILLAGES AND DISTRICTS

The General Plan has defined Village and District boundaries, and for consistency, this Master Plan will use the same naming convention and boundaries. **Figure 2.4** shows the Villages and Districts as defined by the City of Madera and a brief description of each is given as follows:

Village A – North Madera: This area includes most of the census designated place Madera Acres and is located north of Avenue 17 in the north central portion of the Planning Area. The land use in this area is typically considered rural with large single family residential lots. This village contains 4,313 total acres where more than half have been developed.

Village B – Northeast Madera: The Northeast Madera Village is located on the northeast side of the Burlington Northern Santa Fe Railroad Tracks with the Fresno River creating the southern border. This village contains 1,301 acres and approximately one-third of the area has been developed as low density residential units.

Village C – Central Madera: The Central Madera Village is located south of Avenue 17 with the Fresno River creating the southeast border. This village contains approximately 2,556 acres and



Legend

General Plan

Land Use Designations

- C - Commercial
- O - Office
- I - Industrial
- VLD - Very Low Density Residential
- LD - Low Density Residential
- MD - Medium Density Residential
- HD - High Density Residential
- NMU - Neighborhood Mixed Use
- VMU - Village Mixed Use
- VR - Village Reserve
- OS - Open Space
- RC - Resource Conservation/Agriculture
- P&SP - Other Public and Semi-Public Uses
- City Limits
- Urban Growth Boundary
- General Plan Boundary
- Roads
- Highway
- Railroads

Figure 2.3
General Plan Land Use
Sanitary Sewer System
Master Plan
City of Madera

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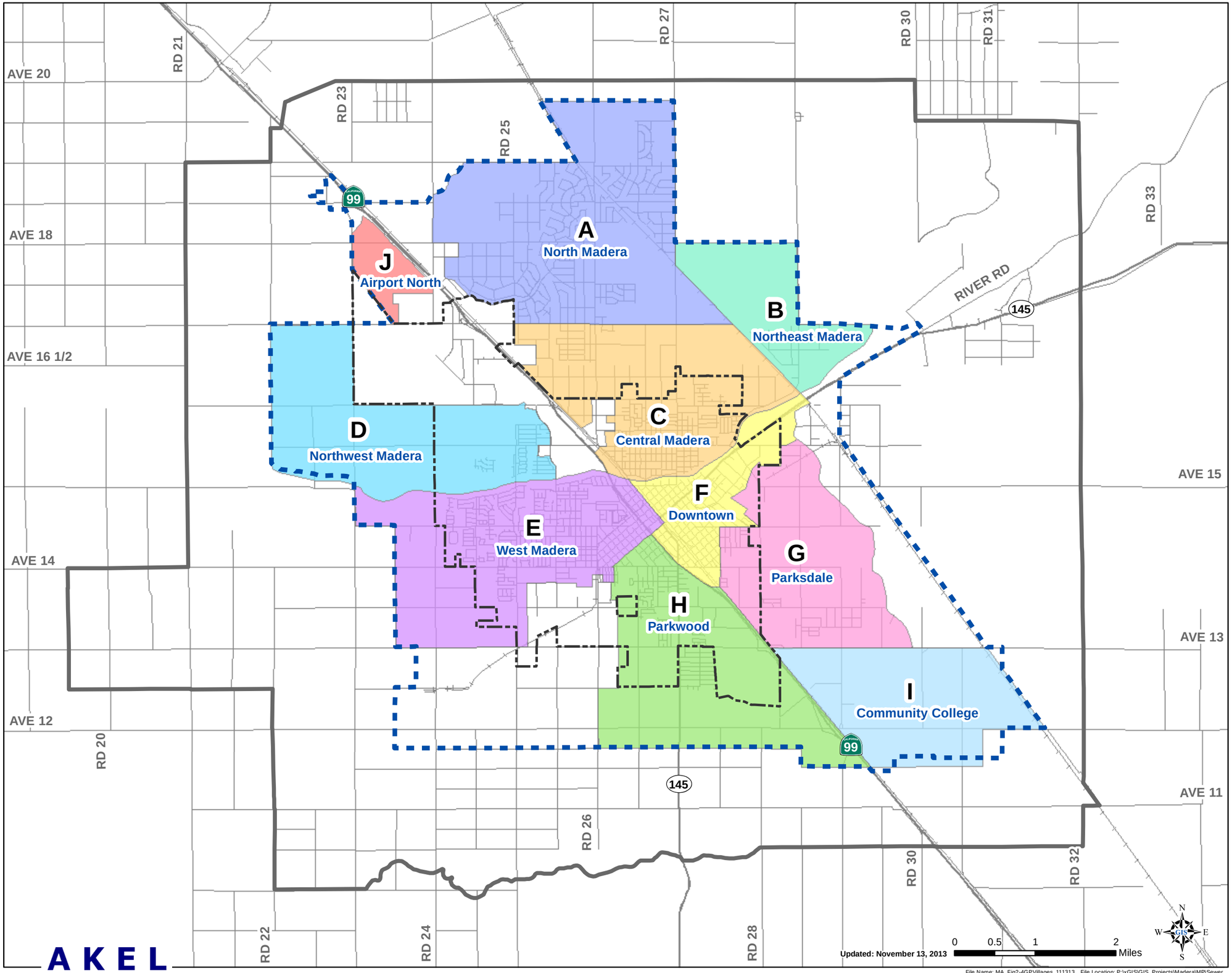

Legend

General Plan Villages

- A - North Madera
- B - Northeast Madera
- C - Central Madera
- D - Northwest Madera
- E - West Madera
- F - Downtown
- G - Parksdale
- H - Parkwood
- I - Community College
- J - Airport North
- City Limits
- Urban Growth Boundary
- General Plan Boundary
- Roads
- Highway
- Railroads

Figure 2.4
General Plan Villages
Sanitary Sewer System
Master Plan
City of Madera

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half of the area has been developed with residential and small retail developments. One-third of this village is in the City Limits.

Village D – Northwest Madera: The Northwest Madera Village is located north of the Fresno River in the northwest portion of the Planning Area just south of the Airport. This village contains 2,763 acres with most of the area undeveloped. One-third of the village is located in City Limits.

Village E – West Madera: The West Madera Village is located south of the Fresno River in the west central portion of the Planning Area. This village contains approximately 3,041 acres and most of the areas within the City Limits are developed with the area outside of the City Limits being less developed.

District F – Downtown District: The Downtown District is located in the Historic center of Madera with the north-western boundary being the Fresno River and the south-western boundary being State Route 99. This District contains approximately 1,160 acres and most of the area is within City Limits and is developed. The only undeveloped area is near the northeast corner along State Route 145. This district contains a mixture of residential and light commercial along with some industrial land use along the rail corridor.

Village G – Parksdale: The Parksdale Village is located north of Avenue 13 and State Highway 99 with the eastern border being an irrigation canal. This village contains approximately 2,414 acres with most of the area being undeveloped. About one-fourth of this village is within City Limits.

Village H – Parkwood: The Parkwood Village is located southwest of State Route 99 and north of Avenue 13 ¼ in the southern portion of the Planning Area. This village contains approximately 3,148 acres, with a little more than half of the area within City Limits.

Village I – Community College: The Community College Village is located south of Avenue 13, east of State Route 99, and west of the Burlington Northern Santa Fe Railroad Tracks in the southeast portion of the Planning Area. This Village contains approximately 2,161 acres and this entire village is located outside of the City Limits.

District J – Airport North District: The Airport North District is located south of State Route 99 and north of the Madera Airport. This District contains 433 acres and is outside of the City Limits.

2.4 HISTORICAL AND PROJECTED POPULATION

Projected population was developed through residential land use designations and projected population densities based on land use. Though historical populations were used in understanding the domestic wastewater flow behaviors and trends, population forecasts are presented for informational purposes only.

Estimates of future wastewater flows were not based on population, but rather on gross acreage for residential and non-residential land uses. Future population was used as a means for estimating the planning horizon of the wastewater system.

The City's historical and projected population data are presented in **Table 2.2**. The historical information was extracted from the previous master plan and California Department of Finance documents. **Table 2.2** documents the historical population from 1990 to 2010 and the projected population by year to the buildout planning horizon of 2047. The Madera Planning Area population has seen steady growth over the last 20 years resulting with an annual increase of 3.5%. Continuing this growth trend projects the population of the general plan area to increase from 61,416 to 252,039 people.

Table 2.2 Historical and Projected Population

Sanitary Sewer System Master Plan
City of Madera

Historical Population[1]

Year	Population	Percent Increase (%)
1990	29,281	-
1991	30,157	3.0%
1992	32,504	7.8%
1993	33,862	4.2%
1994	35,504	4.8%
1995	36,557	3.0%
1996	37,753	3.3%
1997	39,276	4.0%
1998	40,518	3.2%
1999	41,424	2.2%
2000	43,205	4.3%
2001	44,565	3.1%
2002	46,066	3.4%
2003	47,939	4.1%
2004	49,691	3.7%
2005	51,735	4.1%
2006	53,928	4.2%
2007	57,181	6.0%
2008	58,767	2.8%
2009	59,868	1.9%
2010	61,416	2.6%

Historical Averages

	(%)
Last 3 years	2.4%
Last 5 years	3.5%
Last 10 years	3.6%
Last 15 years	3.5%
Last 20 years	3.8%

Projected Population

Year	Population (excluding Madera Acres)	Madera Acres	Percent Increase (%)	Total Population within Planning Area
2010	61,416	9,163		
2011	63,566	9,484	3.5%	73,049
2012	65,790	9,816	3.5%	75,606
2013	68,093	10,159	3.5%	78,252
2014	70,476	10,515	3.5%	80,991
2015	72,943	10,883	3.5%	83,826
2016	75,496	11,264	3.5%	86,760
2017	78,138	11,658	3.5%	89,796
2018	80,873	12,066	3.5%	92,939
2019	83,704	12,488	3.5%	96,192
2020	86,633	12,925	3.5%	99,559
2021	89,666	13,378	3.5%	103,043
2022	92,804	13,846	3.5%	106,650
2023	96,052	14,331	3.5%	110,382
2024	99,414	14,832	3.5%	114,246
2025	102,893	15,351	3.5%	118,244
2026	106,494	15,889	3.5%	122,383
2027	110,222	16,445	3.5%	126,666
2028	114,080	17,020	3.5%	131,100
2029	118,072	17,616	3.5%	135,688
2030	122,205	18,232	3.5%	140,437
2031	126,482	18,871	3.5%	145,353
2032	130,909	19,531	3.5%	150,440
2033	135,491	20,215	3.5%	155,705
2034	140,233	20,922	3.5%	161,155
2035	145,141	21,654	3.5%	166,795
2036	150,221	22,412	3.5%	172,633
2037	155,479	23,197	3.5%	178,675
2038	160,920	24,009	3.5%	184,929
2039	166,553	24,849	3.5%	191,402
2040	172,382	25,719	3.5%	198,101
2041	178,415	26,619	3.5%	205,034
2042	184,660	27,550	3.5%	212,210
2043	191,123	28,515	3.5%	219,638
2044	197,812	29,513	3.5%	227,325
2045	204,736	30,546	3.5%	235,281
2046	211,902	31,615	3.5%	243,516
2047	219,318	32,721	3.5%	252,039
2048	226,994	33,867	3.5%	260,861
2049	234,939	35,052	3.5%	269,991
2050	243,162	36,279	3.5%	279,441

Note:

1. Source: California Department of Finance estimates E-4.

3/20/2013

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 includes: 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 Madera 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: $$\underbrace{\frac{\partial A}{\partial t}}_{(A)} + \underbrace{\frac{\partial Q}{\partial x}}_{(B)} = 0$$

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

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 in **Table 3.1**.

During peak dry weather flows (PDWF), the maximum allowable d/D ratio for proposed pipes 12 inches in diameter and smaller is 0.5, and for proposed pipes larger than 12 inches in diameter, the maximum allowable d/D ratio is 0.75. The maximum allowable d/D ratio for all existing pipes (all diameters) is 0.92. 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

Sanitary Sewer System Master Plan
City of Madera

Pipeline Criteria

Peak Dry Weather Flow Criteria

Diameter (in)	Maximum Allowable d/D: Existing Trunks	Maximum Allowable d/D: Proposed Trunks
8 to 12	0.92	0.50
> 12	0.92	0.75

Peak Wet Weather Flow Criteria

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

Pipe Size (in)	Minimum Grade (ft/ft)	Capacity[1] (mgd)	Velocity[1] (fps)
8	0.0034	0.49	2.28
10	0.0025	0.76	2.26
12	0.0022	1.15	2.40
15	0.0015	1.73	2.30
18	0.0012	2.81	2.60
21	0.0010	3.46	2.35
24	0.0008	4.42	2.30
27	0.0008	6.05	2.48
30	0.0008	8.01	2.66
33	0.0008	10.33	2.84
36	0.0008	13.02	3.01
42	0.0008	19.65	3.33
48	0.0007	26.24	3.41

Unit Flow Factor Criteria

Land Use Classification	Unit	Recommended Factor: Existing (gpd/unit)	Recommended Factor: Future (gpd/unit)
Residential			
Single Family	Gross Acre	1,250	1,200
Multiple Family	Gross Acre	1,500	1,500
Non-Residential			
Commercial	Gross Acre	750	950
Industrial, Light	Gross Acre	750	950
Institutional	Gross Acre	750	950
Mixed Use / Village Reserve[2]	Gross Acre	n/a	950

Notes: 3/20/2013

1. Pipe friction factor assumed at 0.013.
2. The Village Reserve category is intended to incorporate neighborhood planning and village building, and as such has been conservatively estimated as equivalent to commercial use.

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 2010 Public Water System Statistics (PWSS) report, which lists the monthly water consumption 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 by land use type, as annually recorded on PWSS reports. 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 a 95 percent return to sewer ratio. The same concept can be applied to multi-family residential lots, which were estimated at a 70 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 at 48 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**.

It should be noted that additional water conservation is expected for residential land uses with the completion and implementation of water metering. Water conservation was taken into account for residential land uses and the future water demand unit factors were decreased accordingly in the water system master plan. Anticipated increases in land use densities, as identified in the General Plan, are assumed to be offset by future water conservation efforts.

While the impact of water conservation may have some impact on residential flow generation, it is not expected to reduce flows significantly, as most of the conservation is expected to be realized in landscape irrigation. Therefore, minimal adjustments were made to future residential unit factors. Existing non-residential factors were slightly lower than normal and were adjusted for future scenarios to apply a level of conservancy when projecting future sewer flows. **Table 3.3** shows the 2010 flow factors as well as the ones used for estimating flows from future developments within the urban growth boundary.

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

Table 3.2 Unit Flow Factors Analysis

Sanitary Sewer System Master Plan

City of Madera

Land Use Classification	Existing Service Area				2010 Average Daily Water Demand Unit Factors										2010 Average Daily Wastewater Unit Flow Factors								
					2010 Consumption				2010 Production		Vacancy Rate[1,6]	2010 Water Unit Factor				2010 Wastewater Flows			2010 Wastewater Flows at 100% Occupancy			2010 Recommended Wastewater Unit Factor		
	Number of D.U.[1]	City of Madera (gr. acres)	Parksdale (Sewer only) (gr. acres)	Parkwood (Sewer only) (gr. acres)	2010 Annual Consumption[4] (gpd)	Unadjusted Water Unit Factors (gpd/DU)	Unadjusted Water Unit Factors (gpd/gr. acres)	Balance to 2010 Consumption (gpd)	Unaccounted-For-Water Rate[5] (%)	Production (w/o Vacancy Rate) (gpd)	(%)	Recommended Water Unit Factor (gpd/DU)	Recommended Water Unit Factor (gpd/gr. acres)	Balance Using Recommended Unit Factor (gpd)	Return to Sewer Ratio	Unadjusted Wastewater Unit Factor (gpd/DU)	Unadjusted Wastewater Unit Factor (gpd/gr. acres)	Balance to Existing Conditions[7] (gpd)	Projected Flows at 100% Occupancy (gpd/DU)	Projected Flows at 100% Occupancy (gpd/gr. acres)	Projected Flows at 100% Occupancy (gpd)	Recommended Wastewater Unit Factor (gpd/DU)	Recommended Wastewater Unit Factor (gpd/gr. acres)	Balance Using Recommended Unit Factors (gpd)
Residential																								
Single Family Residential	12,410	2,995	230	66	7,661,222	617	2,558	7,661,222	7%	8,197,507	4.0%	550	2,850	8,536,530	48%	296	1,187	3,908,677	308	1,235	4,065,024	310	1,250	4,114,836
Multi-Family Residential	4,224	272	46	17	421,342	100	1,547	421,342	7%	450,836	4.0%	280	2,850	776,233	70%	70	1,352	453,721	73	1,406	471,870	130	1,500	503,549
Non-Residential																								
Commercial / Industrial / Institutional		1,850	0	140	1,300,000		703	1,300,000	7%	1,391,000	11.0%		780	1,442,881	95%		668	1,328,267		741	1,474,376		750	1,492,161
Other (Non-Flow Generating)																								
Landscape Irrigation		1,276			391,922		307	391,922	7%	419,357	0.0%		330	421,031	0%									
Other		1,337	6	0																				
	16,634	7,730	283	223	9,774,486			9,774,486		10,458,700				11,176,675			**5,690,665**			6,011,270			**6,110,545**

Notes:

1. Source: Dwelling Unit counts and Non-Residential Vacancy rates US Census Bureau American Community Survey.
2. Source: Parcel Shapefile received 3/17/2011
3. Source: General Plan Land Use Shapefile received from City Planning Department 1/24/2011.
4. Source: 2010 Public Water System Statistics report provided by City staff.
5. Source: 2010 Draft Urban Water Management Plan July 6, 2011.
6. Source: Residential Vacancy Rate from Table I-10 in the City of Madera General Plan Housing Element.

2/22/2012

Table 3.3 Average Daily Wastewater Unit Flow Factors

Sanitary Sewer System Master Plan

City of Madera

Classifications	2010		Future Developments within Urban Growth Boundary	
	(gpd/DU)	(gpd/gr.acre)	(gpd/DU)	(gpd/gr. acre)
Residential				
Single Family Residential	310	1,250	310	1,200
Multi-family Residential	130	1,500	115	1,500
Non-Residential				
Commercial		750		950
Industrial		750		950
Institutional		750		950
Mixed Use / Village Reserve[1]				950
Commercial or Industrial, High Intensity[2]		*TBD*		*TBD*

Note:

11/11/2011

1. The Village Reserve category is intended to incorporate neighborhood planning and village building, and as such has been conservatively estimated as equivalent to commercial use.
2. High intensity commercial or industrial to be determined based on water use and building square footage.

(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 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.

The City's 1997 master plan included a peaking factor curve for dry weather flows and another curve for wet weather flows. This study developed 24-hour diurnal patterns and peaking factors for dry and wet weather flows for the tributary area to each flow monitor, as shown on **Figure 3.1**.

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 3.2** 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.

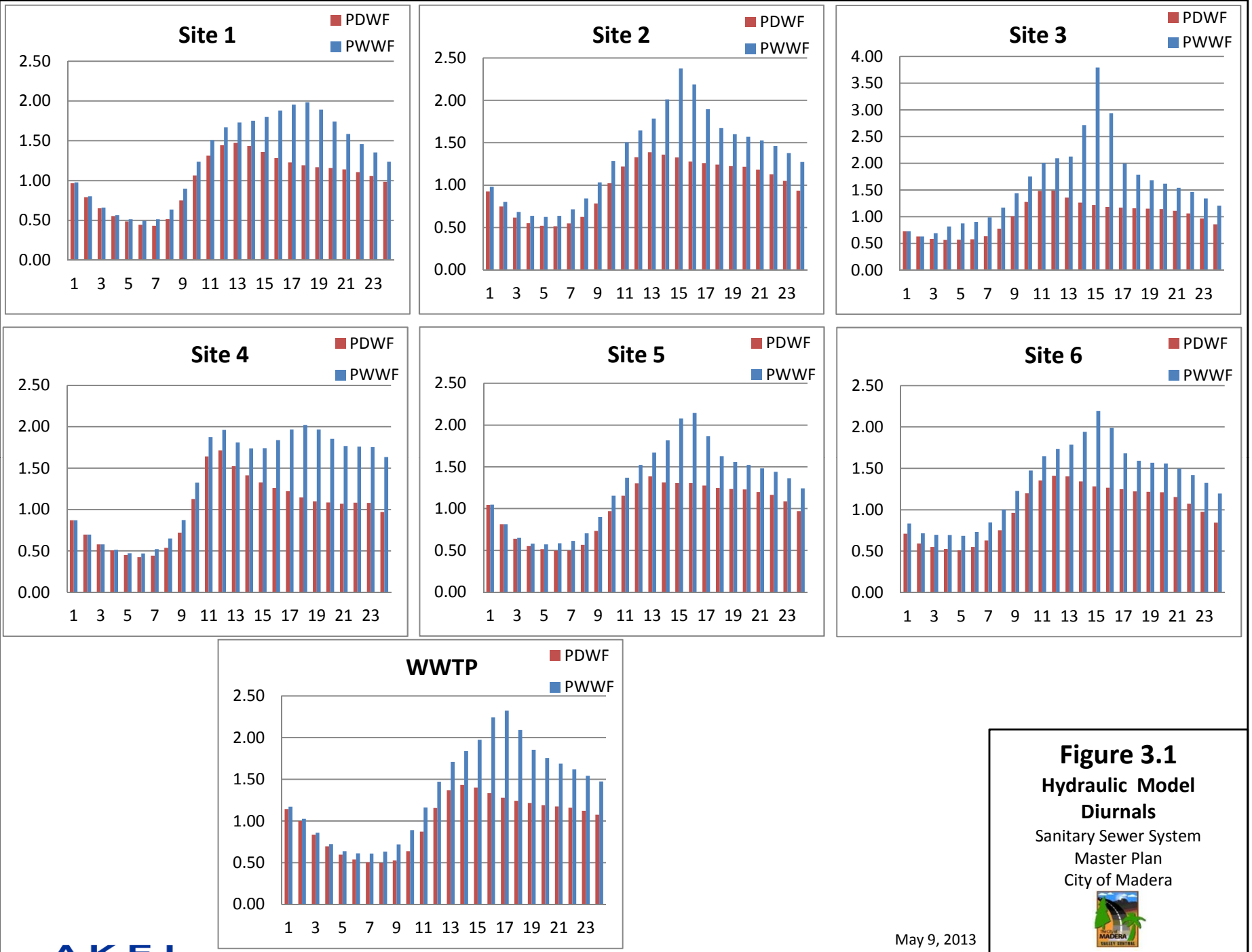
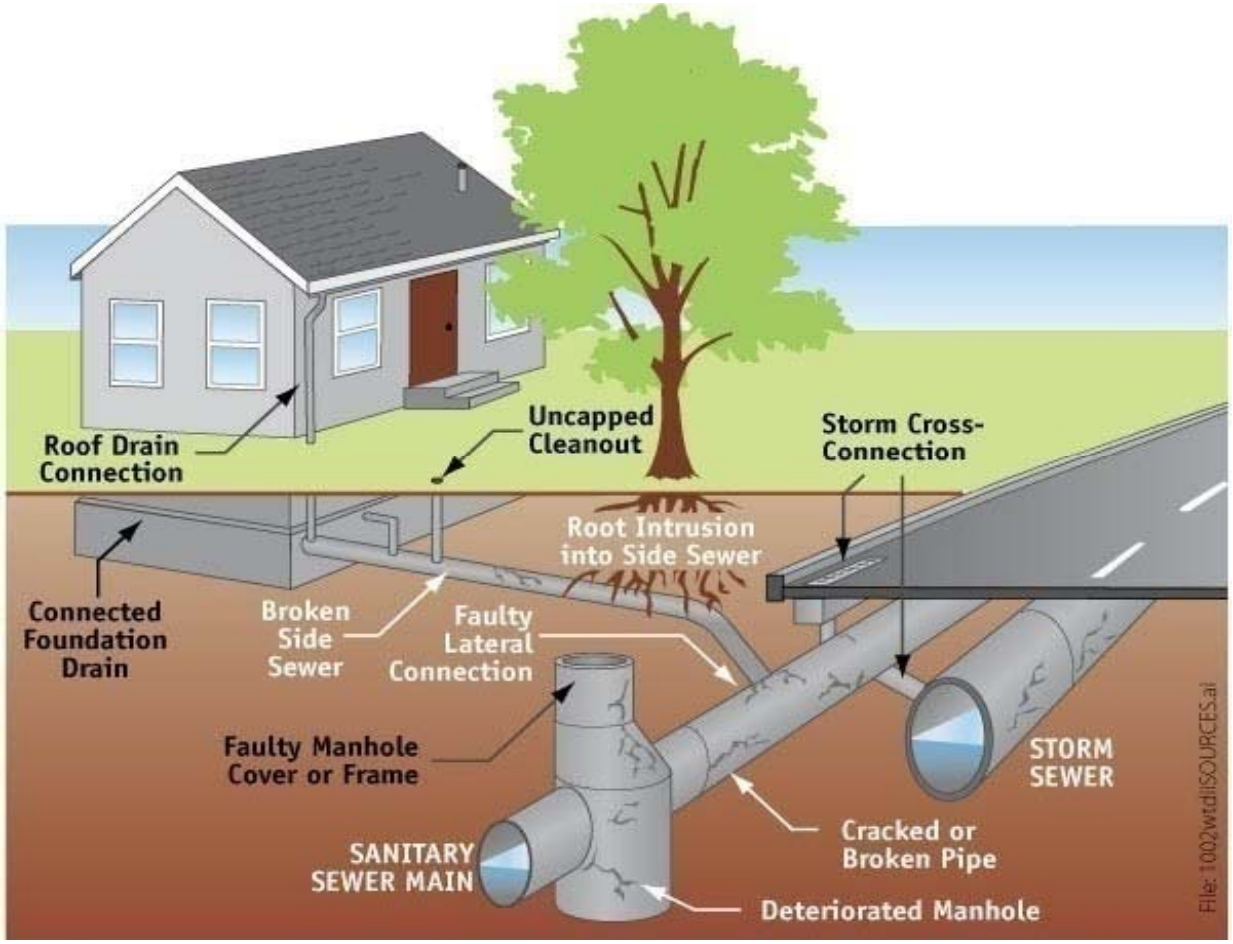


Figure 3.1
Hydraulic Model Diurnals
Sanitary Sewer System
Master Plan
City of Madera

May 9, 2013

AKEL ENGINEERING GROUP, INC.



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

February 26, 2013

AKEL ENGINEERING GROUP, INC.

LEGEND

Inflow Sources (Black Text)

Infiltration Sources (White Text)

Figure 3.2

Infiltration and Inflow Sources

Sanitary Sewer System Master Plan
City of Madera

- **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) 2011 Flow Monitoring Program (**Appendix A**). Thus, the infiltration and inflows are reasonable and reflect the actual behavior of the sanitary sewer system.

3.3.2 Sewer System Flow Monitoring

In 2011 V&A's services were used for a temporary flow monitoring program to capture six sites during wet weather flows. The City had also monitored 31 locations in the sewer system between 2009 and 2011. The locations were not monitored simultaneously and were monitored for approximately a week before the monitor was moved to a different location. These sites and the 6 locations V&A monitored are shown on **Figure 3.3**.

The City also provided flow data from the Madera WWTP. This flow data was used to analyze the seasonal flow patterns the City's sewer system experiences and to gauge the effects the wet weather season has on the sewer system.

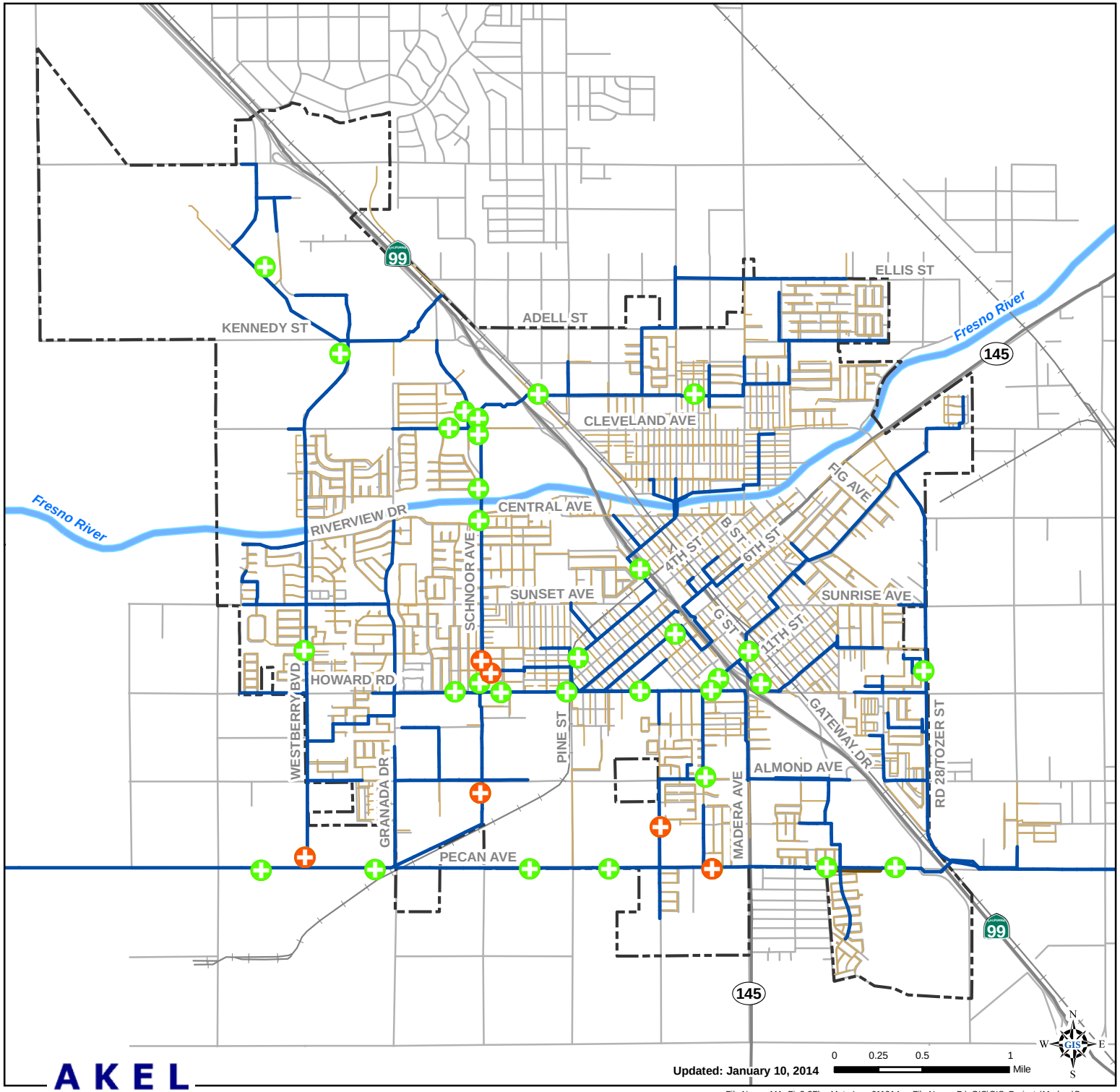
The rain gauge data for the V&A flow monitoring period was obtained from V&A. The V&A rain gauge was located in a City yard near the intersection of East Olive Avenue and South Gateway Drive. The flow monitoring and rain data was used in this analysis to calibrate the computer hydraulic model to average 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.

- **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 of Madera 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.



Legend

- 2011 V&A Flow Meters
- City Meters
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highway
- Railroads
- City Limits
- Fresno River

Figure 3.3
Flow Meter Locations
Sanitary Sewer System
Master Plan
City of Madera

AKEL
ENGINEERING GROUP, INC.

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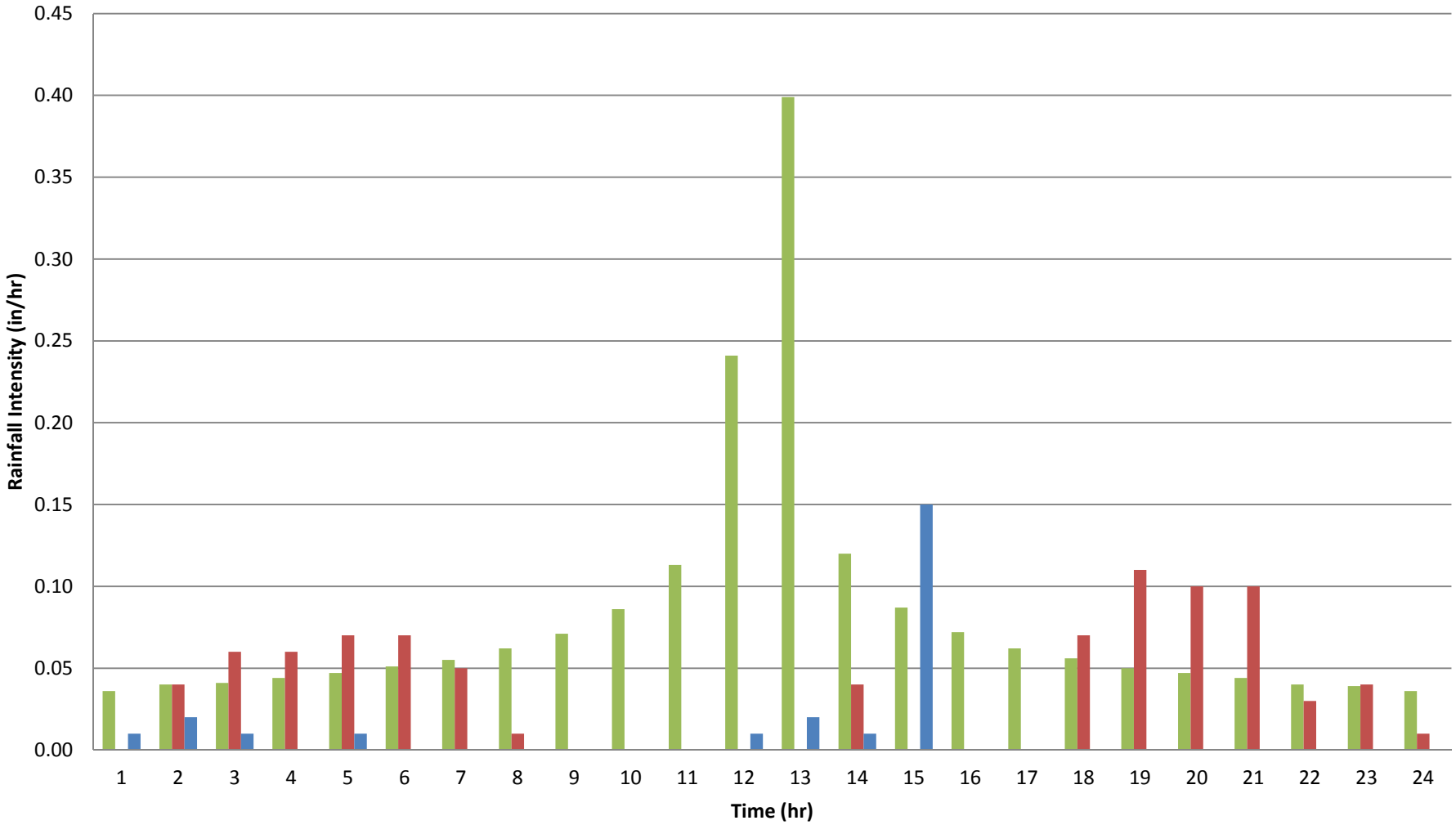
- **Balanced Rainfall Centered Distribution.** The National Resources Conservation Service, previously known as the Soil Conservation Service, has developed rainfall distributions for wide geographic regions based on traditional Depth-Duration-Frequency (DDF) rainfall data. In this methodology, the highest 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 January 2009 and December 2012, as shown on **Table 3.4**. The table lists the total rainfall volume, duration, peak hour intensity, and total monthly rainfall (if available) for each storm event.

Figure 3.4 is intended to show the diurnal comparison between the design storm and the two storm events experienced during February of 2011. The comparison indicates that, based on the balanced centered hyetograph, the design storm's peak hour value is at 0.40 inches per hour (in/hr), while the February 18th and 25th storms peak values are 0.11 and 0.15 in/hr respectively. This comparison illustrates the more conservative nature of the design storm and the relatively small storm events experienced in February 2011.



LEGEND

- Design Storm: 10 Year - 24 Hour (1.94 in)
- Historical Storm Event 1: February 18th, 2011 (0.86 in)
- Historical Storm Event 2: February 25th, 2011 (0.24 in)

March 27, 2013

AKEL ENGINEERING GROUP, INC.

Figure 3.4

10-Year 24-Hour Storm
(Design vs. 2011 Storms)

Sanitary Sewer System
Master Plan
City of Madera

Table 3.4 Storm Events Analysis

Sanitary Sewer System Master Plan
City of Madera

Storm Event	Date	Single Rainfall Event Volume and Duration			Total Monthly Rainfall
		Volume	Duration	Peak Intensity	
		(inches)	(hours)	(in/hr)	(inches)
Historical Rainfall Events					
1	October 13, 2009	1.28			1.39
2	December 11, 2009	0.84			2.35
3	February 26, 2010	0.76			2.94
4	April 20, 2010	0.77			2.19
5	October 5, 2011	0.74			0.76
6[1]	February 18, 2011	0.86	15	0.11	1.61
7[1]	February 25, 2011	0.24	8	0.15	1.61
8	March 17, 2012	1.90			3.18
9	November 30, 2012	1.15			2.07
Design Storm					
10-Year	**10-Year 24-Hour**	**1.94**	**24**	**0.40** ←	Selected Design Storm
5-Year	5-Year 24-Hour	1.63	24	0.34	
2-Year	2-Year 24-Hour	1.30	24	0.27	

3/11/2013

Note:

1. Rainfall Events during V&A flow metering period (February 2011)

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 Madera Wastewater Treatment Plant.

4.1 SANITARY SEWER COLLECTION SYSTEM OVERVIEW

The City provides sewer collection services to approximately 10,000 residential, commercial, industrial, and institutional accounts. The City's collection system consists of approximately 176 miles of up to 48-inch gravity sewer pipes that convey flows towards the Madera WWTP, on Road 21 ½ and Avenue 13, 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 and was updated to reflect the review of construction drawings provided by City Staff. The 6-inch and 8-inch diameter pipes account for 57 percent of the total sewer pipe lengths.

4.2 SEWER COLLECTION BASINS AND TRUNKS

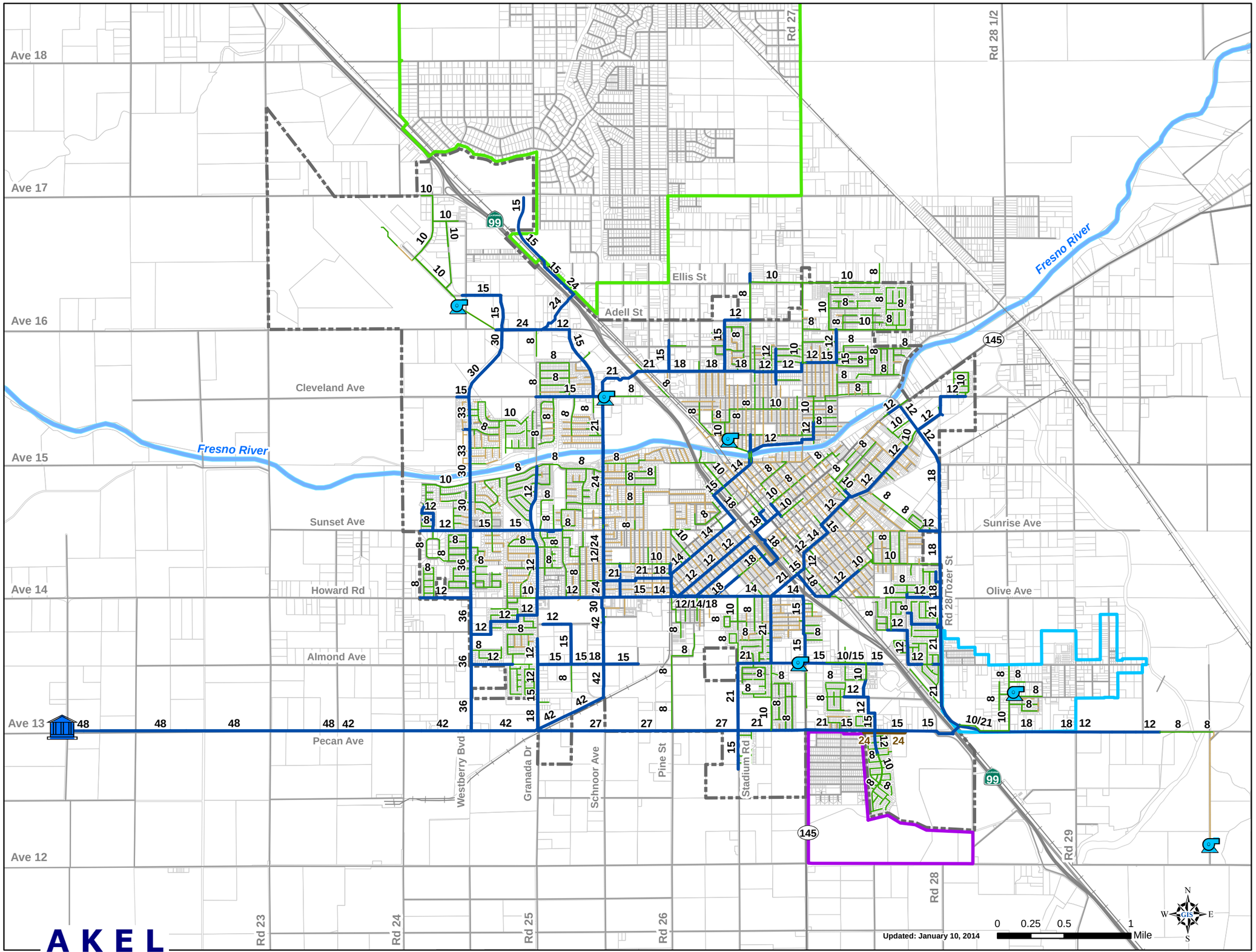
Due to topography, the sanitary sewer system is divided into five separate dendritic sewer collection basins, each defining the boundaries of a sewer collection trunk system. The following five major wastewater collection basins were created and shown on **Figure 4.2**: Westberry, Schnoor, Fourth Street, Stadium, and Pecan. 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 Westberry Collection Basin

The Westberry Collection Basin encompasses 3,142 acres in the western portion of the City. This basin includes the areas generally east of Road 24 and west of Granada Drive (Road 25) with Avenue 17 in the North and Pecan Avenue to the South. This basin includes the Airport and two trunk systems; the Airport Trunk and Westberry Trunk.

This basin collects flows from the Airport area with a 10-inch trunk that flows southeast to the Airport Lift Station located near Aviation Drive and Condor Road. The flows are pumped to the 30-inch Westberry Trunk at Avenue 16 and Westberry Boulevard.

The flow continues south in the 30-inch Westberry Trunk along Westberry Boulevard. The pipe increases to a 36-inch pipe at Sunset Avenue and continues south where it connects to the 42-inch Pecan Trunk in the Pecan Collection Basin.



Legend

Existing

- WWTP
- Lift Stations

Pipes

- 6" or Smaller
- 8" - 10"
- 12" or Larger
- Existing Pipe Not In Use
- Roads
- Highways
- Railroads
- Parcels
- City Limits

Census Designated Places

- Parkwood
- Parksdale
- Madera Acres
- Fresno River

Figure 4.1
Existing Sanitary Sewer Collection System
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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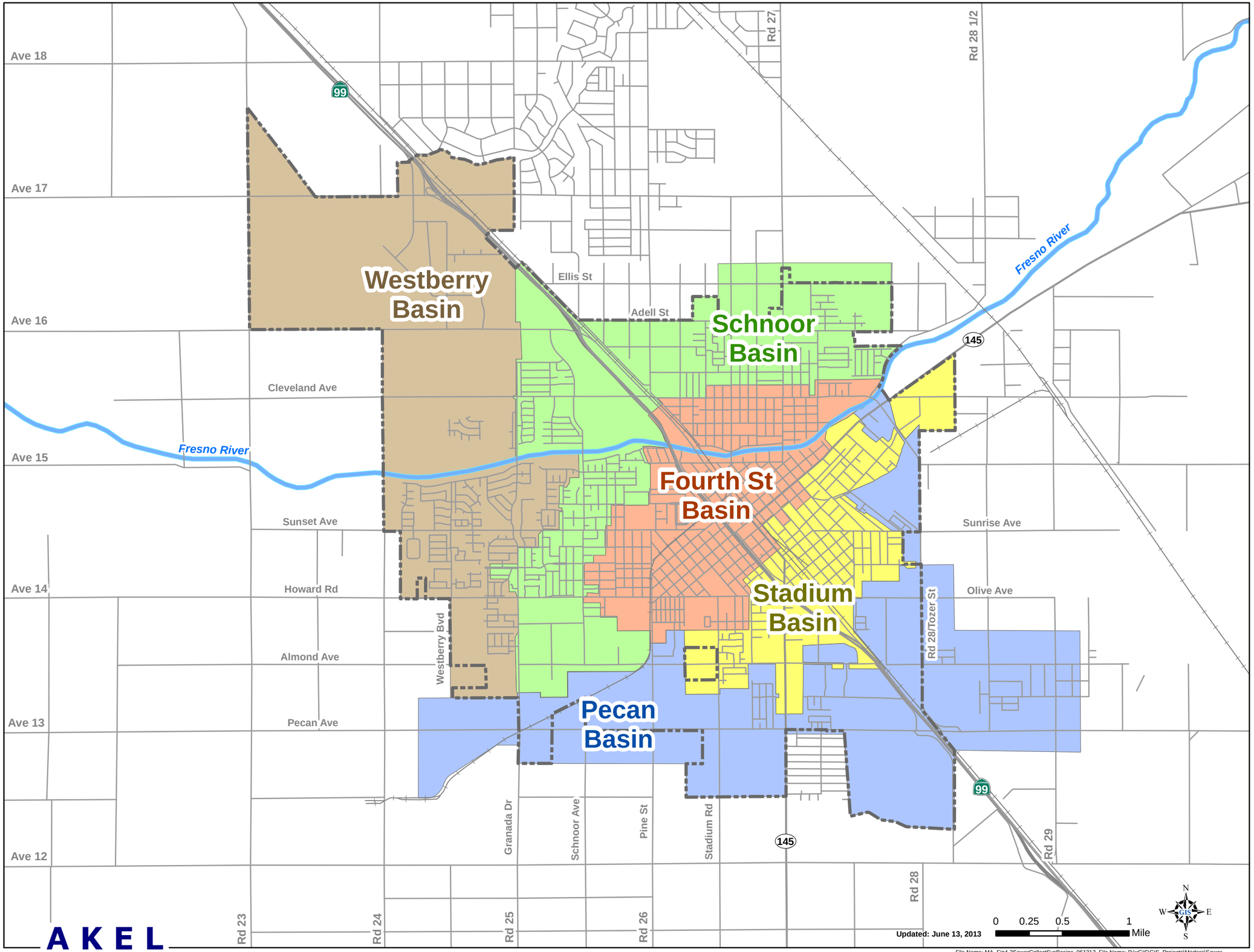


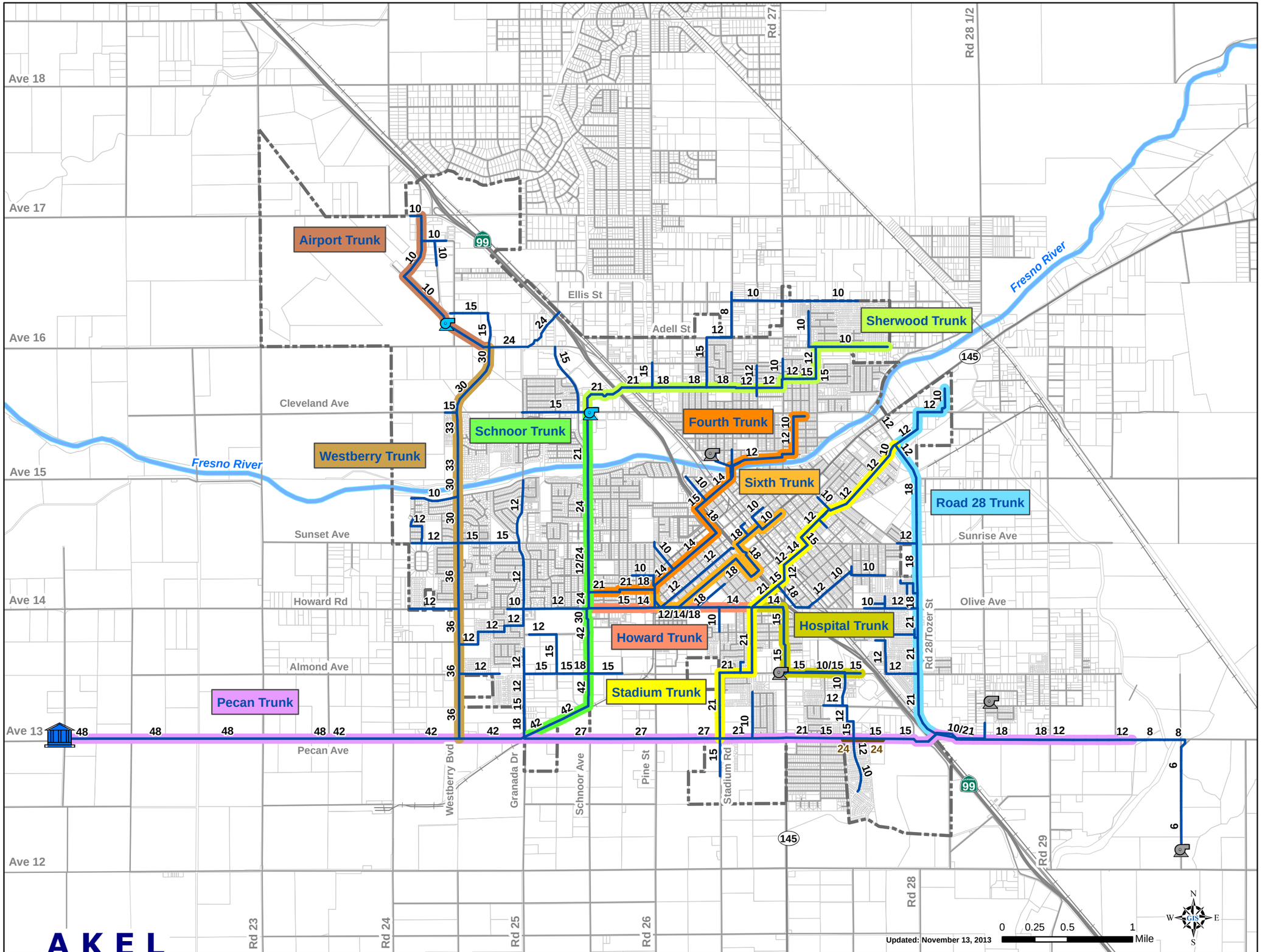
Legend

Existing Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- City Limits
- Fresno River

Figure 4.2
Sewer Collection System Basins
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.



Legend

Existing

- WWTP
- Lift Stations
- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use

Trunks By Basins

Pecan Basin
- Pecan
- Road 28

Schnoor Basin
- Schnoor
- Sherwood

Fourth Street Basin
- Fourth
- Sixth
- Howard

Stadium Basin
- Stadium
- Hospital

Westberry Basin
- Westberry
- Airport

- Roads
- Highways
- Railroads
- City Limits
- Parcels
- Fresno River

Figure 4.3
Existing Trunk System
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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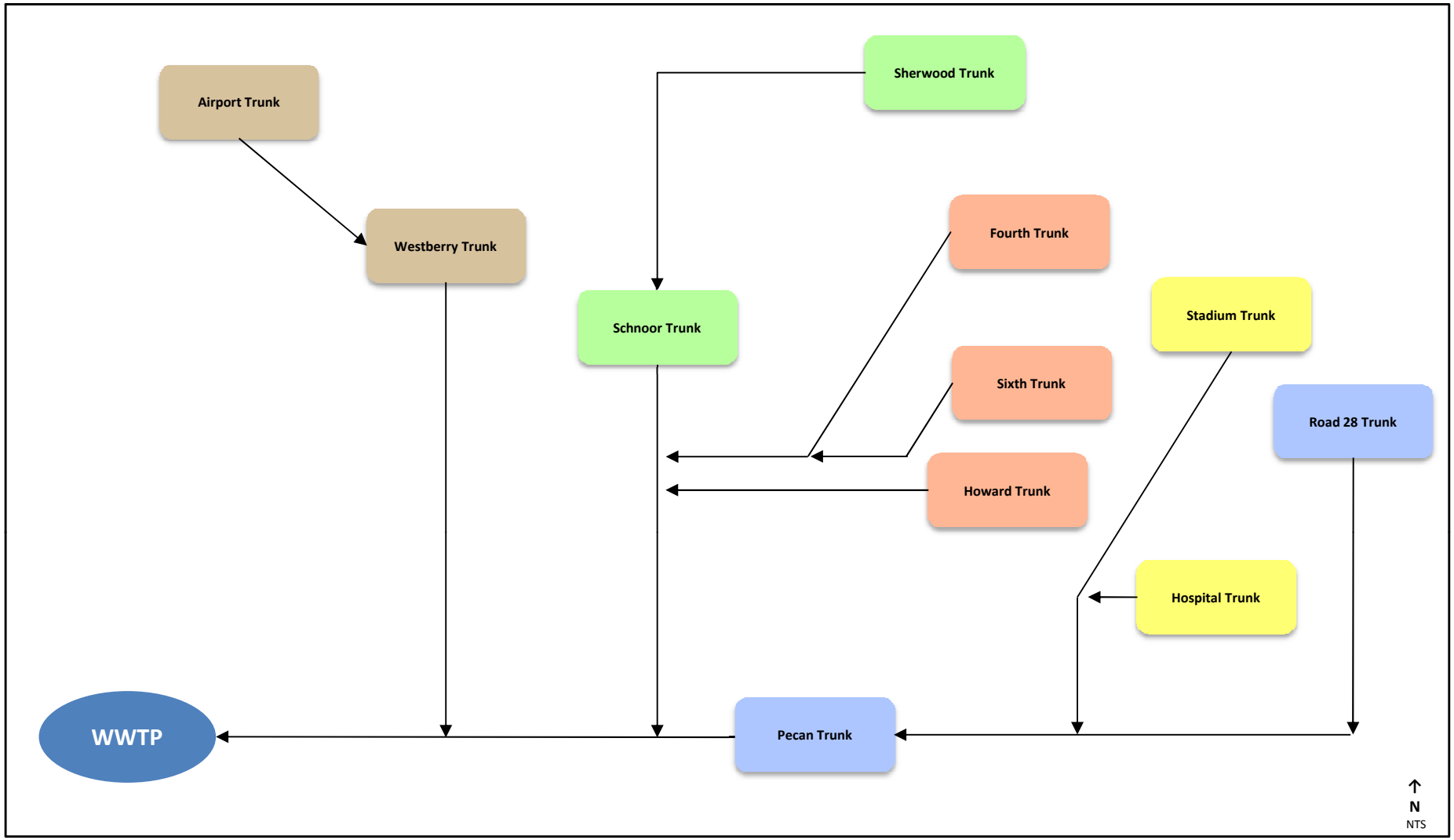


LEGEND

- Westberry Basin
- Schnoor Basin
- Fourth Street Basin
- Stadium Basin
- Pecan Basin
- Wastewater Trunks

June 10, 2013

AKEL ENGINEERING GROUP, INC.

Figure 4.4

Sewer Basin and Trunk Connectivity

Sanitary Sewer System
Master Plan
City of Madera

Table 4.1 Existing Sewer Pipe Inventory

Sanitary Sewer System Master Plan
City of Madera

Pipe Diameter	Total (ft)	Total (miles)	Percent
Data not available	90,534	17.1	10%
6"	162,054	30.7	17%
8"	375,973	71.2	40%
10"	59,768	11.3	6%
12"	77,144	14.6	8%
14"	12,799	2.4	1%
15"	40,075	7.6	4%
17"	529	0.1	0%
18"	24,296	4.6	3%
21"	26,861	5.1	3%
24"	9,924	1.9	1%
27"	7,985	1.5	1%
30"	9,259	1.8	1%
36"	7,909	1.5	1%
42"	14,382	2.7	2%
48"	10,845	2.1	1%
Total	930,335	**176**	**100%**

Note: 3/20/2013

1. The sewer system pipe inventory was extracted from the City's GIS-based hydraulic model developed in August 2012

4.2.2 Schnoor Collection Basin

The Schnoor Collection Basin encompasses 2,385 acres in the east-central and northern parts of the City. The two main trunks in this basin are the Sherwood Trunk which collects flows from the northern part or the City and the Schnoor Trunk which collects flows along Schnoor Avenue.

This basin begins collecting flows in the northeast corner of the City and directs them to the 21-inch Sherwood Trunk in Sherwood Way which flows east toward Highway 99. The trunk crosses Highway 99 and continues south to the Fairgrounds Lift Station located near Cleveland and Schnoor Avenues.

The Fairgrounds Lift Station pumps into the 21-inch Schnoor Trunk and flows south. The Schnoor Trunk crosses the Fresno River with a dual 12-inch siphon and then increases in size to a 24-inch. At Schnoor Avenue and Fourth Street the Fourth Street Collection Basin flows enter the Schnoor Trunk. The 24-inch continues south increasing in size up to a 42-inch pipe as it connects to the Pecan Collection Basin.

4.2.3 Fourth Street Collection Basin

The Fourth Street Collection Basin encompasses 1,489 acres in central Madera. This basin includes the northern portion of the Downtown area. The upper limit of this basin is Cleveland Avenue and the southern boundary is formed by Olive Avenue. This basin contains three major trunk systems: the Fourth Street Trunk, Sixth Trunk and Howard Trunk.

The Fourth Street Trunk starts collecting sewer flows with a network of pipes between Cleveland Avenue and the Fresno River with Highway 145 and Highway 99 creating the east and west borders. This network drains south towards a parallel 15-inch and 8-inch pipe that crosses the Fresno River at D Street. This trunk continues southwest down 1st Street crossing Highway 99 and then jogs southeast to Fourth Street where it continues southwest in Fourth Street to Schnoor Avenue increasing in size up to a 21-inch pipe. This trunk connects to the 24-inch Schnoor Trunk in the Schnoor Collection Basin.

The flows from the southern portion of the downtown area are collected and drain towards the Sixth Trunk via the 15-inch pipe near Yosemite Avenue and Gateway Drive. This trunk jogs to Sixth Street after crossing Highway 99 and continues southwest to Pine Street. The flows from this trunk intersect with the Fourth Street Trunk at Pine and Fourth Streets.

The Howard Trunk collects flows from the southern part of this basin with a 15-inch trunk along Olive Avenue and Howard Road. This trunk flows west from Monterey Street and Olive Avenue to the 30-inch Schnoor Trunk in the Schnoor Collection Basin.

4.2.4 Stadium Collection Basin

The Stadium Collection Basin encompasses 1,148 acres in the south-central part of the City. This basin includes the southern portion of the downtown area and the Madera Community Hospital.

The upper limit of this basin is approximately Highway 145 and the southern boundary is formed by Almond Avenue. This basin contains two trunk systems; the Stadium Trunk and Hospital Trunk.

The Stadium Trunk starts near Highway 145 and Tozer Street with a 10-inch pipe in Clinton Street. The flow heads southwest in Clinton Street toward Highway 99 where it jogs down to 10th Street and crosses Highway 99 with a 21-inch pipe. At Olive Avenue the flows from the Hospital Trunk are intersected and then the 21-inch pipe continues south along Monterey Street, west along Almond Avenue and finally south along Stadium Road where it connects to the Pecan Collection Basin.

The Hospital Trunk collects flows from the Madera Community Hospital on Almond Avenue near Highway 99. The trunk flows west in a 15-inch pipe to Madera Avenue where it turns north to Olive Avenue. The flow continues west in Olive Avenue where it then connects to the Stadium Trunk at Olive Avenue and Monterey Street.

4.2.5 Pecan Collection Basin

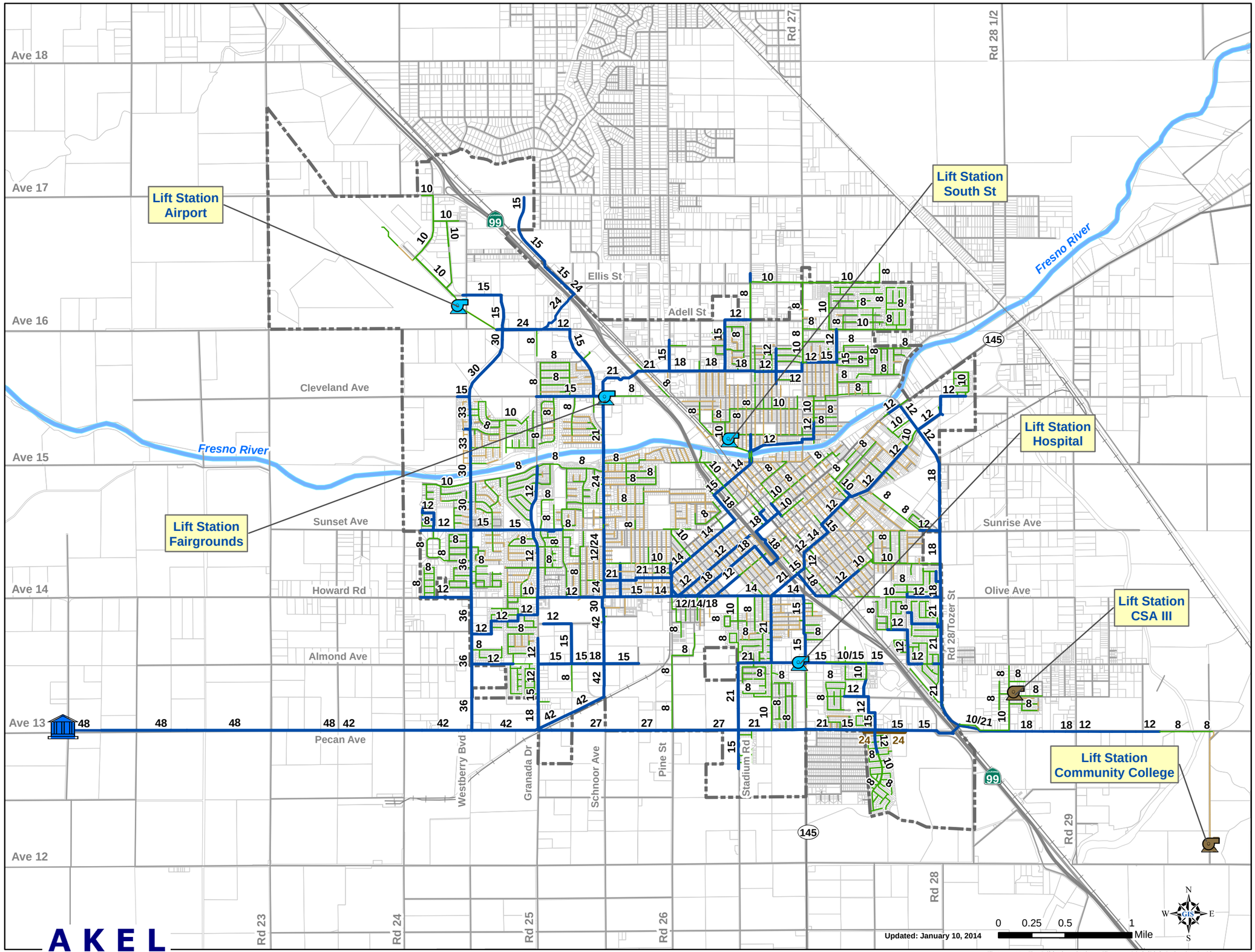
The Pecan Avenue Collection Basin encompasses 3,030 acres in the southern area of the City. This basin runs east-west along Pecan Avenue and north south along Road 28 from Pecan Avenue to Highway 145. This basin includes the Parkwood and Parksdale areas. Sewage flows from the Community College enter this basin from a force main near Avenue 13 (Pecan Avenue) and Road 29 ½. This basin contains two major trunk systems; the Road 28 Trunk and Pecan Trunk.

The Pecan basin collects flows along Road 28 starting near Highway 145 with the Road 28 trunk. The Road 28 trunk heads south from Highway 145 increasing in size to a 21-inch pipe where it connects to the Pecan Avenue Trunk.

The Pecan Trunk collects the Parksdale flows at Road 28½ with an 18-inch pipe that heads west and crosses Highway 99 with a parallel 30 and 15-inch pipe. After crossing Highway 99, a 15-inch trunk continues west to Stadium Road where the Stadium Collection Basin flows are intersected. The flows continue west in a 27-inch pipe to Granada Drive where the pipe increases to a 42-inch pipe and the Schnoor Collection Basin flows are intersected. The Westberry Collection Basin flows are then intersected at Westberry Boulevard and then the trunk continues west along Pecan Avenue increasing in size to a 48-inch pipe that discharges to the Madera Wastewater Treatment Plant at Pecan Avenue and Road 21½.

4.3 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 four lift stations in the sewer collection system, as summarized on **Table 4.2** and shown on **Figure 4.5**. There are two other lift stations that pump into the sewer



Legend

Existing

- WWTP
- City-Owned Lift Stations
- County/Other Lift Stations

Pipes

- 6" or Smaller
- 8" - 10"
- 12" or Larger
- Existing Pipe Not In Use
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 4.5
Lift Stations
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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Table 4.2 Lift Station Inventory

Sanitary Sewer System Master Plan
City of Madera

No.	Lift Station Name	Location	Year Built	Type	Pumps				Force Main	
					No.	Hp	Capacity of Each Pump (gpm)	TDH (ft)	Diameter (in)	Length (ft)
1	Fairground Lift Station[1]	Cleveland Ave 375ft e/o Schnoor Ave	1976	VFD Dry-Pit Non-Clog Pumps	3	7 1/2	875, 820, 957 (speed at 100%)	16	12	60
2	South Street Lift Station[1]	South St between Sonora St and Columbia St	Overhauled in 1994	Variable Speed, Non-Clog Centrifugal Pumps	2	7 1/2	410	23.87	6	43
3	Airport Lift station	Southwest of the Falcon Dr and Aviation Dr Elbow	n/a	Non-Clog Centrifugal	3	7 1/2	410	34	8	2,100
4	Hospital Lift Station[1]	Madera Ave and Almond Ave	1975	Centrifugal Pumps	2	5	-	-	6	67

Note:

1. Source: 1997 Sewer System Master Plan.

2/5/2014

System; they are from the Parksdale (CSA3) area and the Madera Community College Center. These lift stations are not owned by the City of Madera.

Table 4.2 lists each lift station with relevant information obtained from the City's records including: location, year built, type, number of pumps, and horse power if data was provided. The lift stations are operated to turn "on" or "off" based on the levels in their wet wells. Table 4.2 also documents the size and length of the force main downstream from each lift station.

Two of these lift stations were included in the hydraulic model and an individual capacity evaluation and was included along with the hydraulic modeling effort. The following lift stations were included:

- **Fairgrounds Lift Station.** This lift station services the areas north of Cleveland Avenue, not including the airport area. This lift station is located on Cleveland Avenue 375 feet east of Schnoor Avenue near the Madera Fairgrounds and was built in 1976. The lift station includes two duty pumps and one standby pump rated. The pump station firm capacity is rated at 1,653 gallons per minute (gpm) and its total capacity at 2,551 gpm. The pumps discharge into a 12-inch force main in Schnoor Drive.

- **Airport Lift Station.** This lift station serves the airport area in the northwest corner of the City. This lift station is located southwest of the Falcon Drive and Aviation Drive Elbow. This lift station includes two duty pumps rated at 410 gpm and one standby pump rated at 410 gpm. The lift station discharges into an 8-inch force main to the Westberry trunk.

4.4 MADERA WASTEWATER TREATMENT PLANT

The Madera WWTP is a 10.1 million gallons per day (MGD) primary and secondary treatment facility. The treatment facility is located on the corner of Road 21½ and Avenue 13 with a street address of 13048 Road 21½. The original plant was completed in 1972 with a plant expansion occurring in 2007 to provide the plants current capacity and technology. The Madera WWTP has a design capacity of 10.1 MGD and it can accommodate a design peak dry weather flow of up to 15.1 MGD. The plant is currently operating at an average flow of 5.7 MGD.

CHAPTER 5 – SANITARY SEWER FLOWS

This chapter summarizes historical wastewater flows experienced at the Madera 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.

5.1 FLOWS AT THE MADERA WWTP

The wastewater flows collected and treated at the Madera 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 and length of storm events. Figure 5.1 shows the monthly flows verses rainfall at the WWTP for 2010. August and September were the maximum months during 2010, with December also being higher than average due to the considerable amount of rain received that month.

Flow data influent to the Madera WWTP was obtained from City operation staff. The flow data covered a period from 2006 to 2011. From this data monthly, daily, peak daily flows, and peak hourly flows (if available), were determined as summarized on Table 5.1.

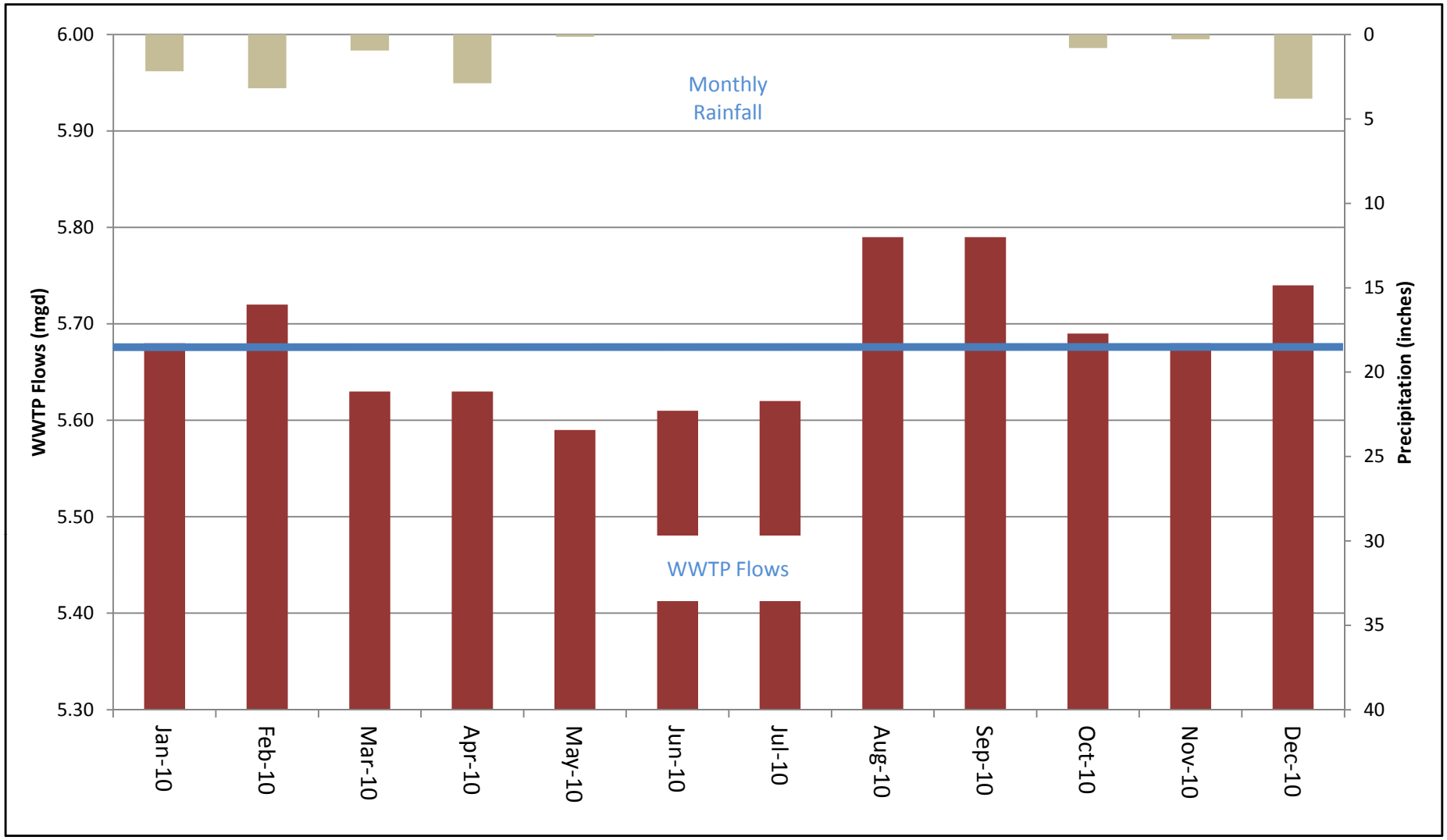
The following definitions are intended to document relevant terminologies shown on Table 5.1:

- **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
- 2010 Average Flow

March 27, 2013

AKEL ENGINEERING GROUP, INC.

Figure 5.1

2010 Madera WWTP Flows

Sanitary Sewer System Master Plan
City of Madera

Table 5.1 Historical Flows at The Madera Wastewater Treatment Plant

Sanitary Sewer System Master Plan
City of Madera

Year	Average Annual Flow		Seasonal Average		Maximum Month		Maximum Day		Peak Hour	
	AAF	Percent Change	ADWF	AWWF	MMDWF	MMWWF	MDDWF	MDWWF	PDWF	PWWF
Historical Flows										
	(MGD)		(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)	(MGD)
2006	5.05		5.10	5.03	5.12	5.09	6.18	8.23		
2007	5.51	8.3%	5.58	5.48	5.93	6.05	6.40	6.80		
2008	5.82	5.3%	5.87	5.75	6.02	5.80	6.32	6.51		
2009	5.72	-1.7%	5.82	5.68	5.86	5.79	6.19	6.62		
2010	5.68	-0.7%	5.70	5.67	5.79	5.74	6.42	6.57	9.61	11.36
2011	5.58	-1.8%						6.99		10.50
Historical Peaking Factors (applied to AAF)										
2006	1.00		1.01	1.00	1.01	1.01	1.22	1.63		
2007	1.00		1.01	0.99	1.08	1.10	1.16	1.23		
2008	1.00		1.01	0.99	1.03	1.00	1.09	1.12		
2009	1.00		1.02	0.99	1.02	1.01	1.08	1.16		
2010	1.00		1.00	1.00	1.02	1.01	1.13	1.16	1.69	2.00
2011	1.00							1.25		1.88

Notes: 6/5/2013

1. Peaking Factors are multipliers applied to the Average Annual Flow (AAF)
2. Flow Components Definitions

AADF = Average Annual Daily 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, with no evident reaction to rainfall)
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 Hour Dry Weather Flow (highest measured hourly flow that occurs during a dry weather day)
PWWF = Peak Hour Wet Weather Flow (highest measured hourly flow that occurs during a wet weather day)

- **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 flows experienced at the Madera WWTP have increased from 5.05 MGD in 2006 (AAF) to 5.58 MGD in 2011. The flows increased by 13.2% from 2006 to 2008 and have decreased by about 4.3% since 2008. Table 5.1 also indicates that, in the City of Madera, the average annual flow (AAF) is generally close to the Maximum Month Dry Weather Flows (MMDWF) and the Maximum Month Wet Weather Flows (MMWWF).

In addition to listing the 2006-2011 flows, and for comparison purposes, the table calculates the peaking factors applied to the corresponding average annual flows for each year. During wet weather flows, the maximum daily volume (MDWWF) experienced at the Madera WWTP was 1.25 times higher than the average annual flow. The maximum hourly peak flow (PWWF) experienced at the Madera WWTP was 2.00 times higher than the average annual flow.

5.2 EXISTING SEWER FLOWS BY COLLECTION BASIN

The existing wastewater flows represented in this Master Plan were based on the City's existing land use. The number of acres and corresponding wastewater flows, for each sewer collection basin, are summarized on Table 5.2. These basins correspond to sewer trunk systems as discussed in Chapter 2.

- **Westberry Basin.** This basin includes 28 percent of the total acres and 18 percent of the existing dry weather flows.
- **Schnoor Basin.** This basin includes 22 percent of the total acres and 27 percent of the existing dry weather flows.
- **Fourth Street Basin.** This basin includes 13 percent of the total acres and 25 percent of the existing dry weather flows.
- **Stadium Basin.** This basin includes 10 percent of the total acres and 14 percent of the existing dry weather flows.

Table 5.2 2010 Wastewater Flows by Basin

Sanitary Sewer System Master Plan

City of Madera

Basin	Area		Average Annual Flows	
	Acres	Percent of Total (%)	Flows (gpm)	Percent of Total (%)
Westberry	3,142	28%	724	18%
Schnoor	2,385	22%	1,053	27%
Fourth Street	1,489	13%	972	25%
Stadium	1,148	10%	556	14%
Pecan	3,030	27%	639	16%
Total	**11,194**	100%	**3,944**	100%

6/7/2013

- **Pecan Basin.** This basin includes 27 percent of the total acres and 16 percent of the existing dry weather flows.

Figure 5.2 shows that the Fourth Street Basin, which is located in central Madera, produces higher per acre sewer flows than other basins, largely due to more developed areas in the City's central corridor.

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 wastewater flows. The 2010 flows were increased to 6.1 MGD to account for 100% occupancy, and the ultimate buildout flows were calculated at 26.3 MGD. Madera Acres flows are also listed on this table for completeness though they were excluded from this capacity analysis of the collection system and the wastewater treatment plant.

Table 5.4 includes annual wastewater flow projections, based on the population methodology, and a unit flow factor of 120 gallons per day per capita (gpdc). It should be noted that this unit factor was increased, from the current 100 gpdc, to account for the large acreages of industrial developments identified in the General Plan. This table documents the population projection at an rate of increase of 3.5%, from 2010 (61,416 people) to the planning horizon of 2047 (219,318 people).

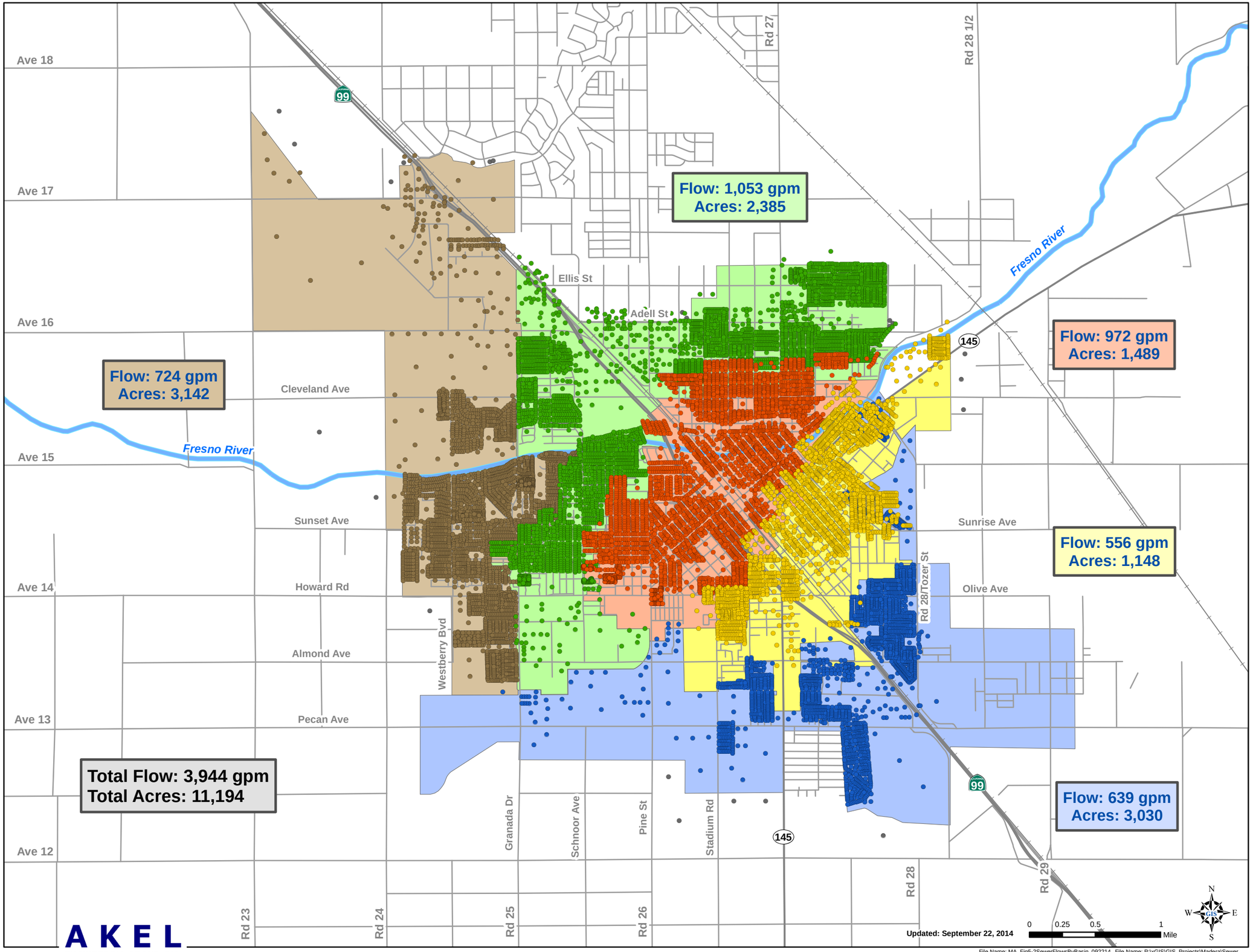
5.4 SANITARY SEWER COLLECTION SYSTEM DESIGN FLOWS

The design flows most relevant in this capacity analysis of the sanitary 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.

- **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. During PWWF a relaxed criteria was used compared to PDWFs. The hydraulic analysis allowed surcharging to occur during wet weather conditions with the hydraulic grade line (HGL) rising up to three feet below the manhole rim. If the HGL at any time was less than three feet from the manhole rim, the pipe was considered deficient.

The design flows used in evaluating the capacity adequacy of the sewer collection system are summarized on Table 5.5. The table lists the maximum day and peak hour flows for dry and wet



Legend

Accounts By Basin

- Stadium
- Fourth St
- Schnoor
- Westberry
- Pecan
- Other

Existing Basins

- Stadium Basin
- Fourth St Basin
- Schnoor Basin
- Westberry Basin
- Pecan Basin
- Roads
- Highways
- Railroads
- Parcels
- Fresno River

Figure 5.2
Sewer Flows By Basin
Sanitary Sewer System Master Plan
City of Madera

AKEL
ENGINEERING GROUP, INC.

Table 5.3 Average Daily Flows at Buildout of Project Area
Sanitary Sewer System Master Plan
City of Madera

Classifications	2010 Wastewater Flows at 100% Occupancy					Planning Area Wastewater Flows at 100% Occupancy														
	Existing Areas Currently Serviced within			Wastewater Unit Factor	2010 Average Annual Flows	Total Planning Area (including Madera Acres)		Planned Development within City Limits		Madera Acres Sub Area		Parkwood Sub Area		Parksdale Sub Area		Future Planning Area Development Excluding Sub Areas		Total Areas to be Serviced by City (Excluding Madera Acres		Total Average Annual Flow in Planning Area (Including Madera Acres)
	City Limits	Parkwood	Parksdale			Acreages	Factor	Acreages	Annual Flows	Acreages	Annual Flows	Acreages	Annual Flows	Acreages	Annual Flows[2]	Acreages	Annual Flows	Acreages	Annual Flows	
	(gr. acres)	(gr. acres)	(gr. acres)	(gpd/gr. acre)	(gpd)	(gr. acres)	(gpd/gr. acre)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. acres)	(gpd)	(gr. Acres)	(gpd)	(gpd)
Residential																				
Single Family Residential	2,995	66	230	1,250	4,114,836	12,867	1,200	881	1,057,287	3,151	3,781,614	66	79,674	147	146,500	5,330	6,395,615	9,716	11,793,912	15,575,526
Multi-Family Residential	272	17	46	1,500	503,549	2,365	1,500	21	32,118	173	259,157	17	25,705	38	38,400	1,779	2,669,038	2,192	3,268,809	3,527,966
Subtotal	*3,268*	*84*	*276*		*4,618,385*	*15,232*		*902*	*1,089,405*	*3,324*	*4,040,771*	*84*	*105,379*	*185*	*184,900*	*7,109*	*9,064,654*	*11,908*	*15,062,722*	*19,103,492*
Non-Residential																				
Commercial	917	3		750	690,187	1,988	950	471	447,775	82	78,027	3	3,266	0	0	511	485,533	1,906	1,626,761	1,704,789
Industrial	523			750	392,555	4,834	950	226	214,611	90	85,247	0	0	0	0	3,995	3,795,653	4,745	4,402,818	4,488,066
Institutional	410	0		750	307,480	580	950	13	12,546	13	12,768	0	326	13	13,400	130	123,153	567	456,905	469,673
Mixed Use / Village Reserve		136		500	67,960	5,893	950		0	786	746,611	136	129,123	0	0	4,835	4,593,021	5,107	4,790,104	5,536,714
Subtotal	*1,850*	*140*	*0*		*1,458,181*	*13,295*		*710*	*674,932*	*971*	*922,653*	*140*	*132,715*	*13*	*13,400*	*9,471*	*8,997,360*	*12,324*	*11,276,588*	*12,199,241*
Other (Non-Flow Generating)																				
Landscape Irrigation	1,276					1,435												1,435		
Other	1,337		6			37,367		308		360		0		0		35,355		37,006		
Subtotal	*2,612*	*0*	*6*			*38,802*		*308*		*360*		*0*		*0*		*35,355*		*38,442*		
Totals	7,730	223	283		**6,076,565**	67,329		1,921	1,764,337	4,656	4,963,424	223	238,094	198	198,300	51,935	18,062,014	62,673	**26,339,310**	31,302,733

Notes:
1. Irrigated landscape is non-flow generating and is categorized under Non-Flow Generating.
2. Parksdale future lands were assumed to develop at 1,000 gpd/acre.

Table 5.4 Projected Future Sewer Flows (100% Occupancy)

Sanitary Sewer System Master Plan
City of Madera

Year	Population[1]	Annual Growth	gpdc	Average Day	
		(%)		(MGD)	(gpm)
2010	**61,416**		99	6.1	4,222
2011	63,566	3.5%	101	6.4	4,463
2012	65,790	3.5%	103	6.8	4,715
2013	68,093	3.5%	105	7.2	4,979
2014	70,476	3.5%	107	7.6	5,256
2015	**72,943**	3.5%	110	8.0	5,547
2016	75,496	3.5%	112	8.4	5,851
2017	78,138	3.5%	114	8.9	6,170
2018	80,873	3.5%	116	9.4	6,504
2019	83,704	3.5%	118	9.9	6,853
2020	**86,633**	3.5%	120	10.4	7,219
2021	89,666	3.5%	120	10.8	7,472
2022	92,804	3.5%	120	11.1	7,734
2023	96,052	3.5%	120	11.5	8,004
2024	99,414	3.5%	120	11.9	8,284
2025	**102,893**	3.5%	120	12.3	8,574
2026	106,494	3.5%	120	12.8	8,875
2027	110,222	3.5%	120	13.2	9,185
2028	114,080	3.5%	120	13.7	9,507
2029	118,072	3.5%	120	14.2	9,839
2030	**122,205**	3.5%	120	14.7	10,184
2031	126,482	3.5%	120	15.2	10,540
2032	130,909	3.5%	120	15.7	10,909
2033	135,491	3.5%	120	16.3	11,291
2034	140,233	3.5%	120	16.8	11,686
2035	**145,141**	3.5%	120	17.4	12,095
2036	150,221	3.5%	120	18.0	12,518
2037	155,479	3.5%	120	18.7	12,957
2038	160,920	3.5%	120	19.3	13,410
2039	166,553	3.5%	120	20.0	13,879
2040	**172,382**	3.5%	120	20.7	14,365
2041	178,415	3.5%	120	21.4	14,868
2042	184,660	3.5%	120	22.2	15,388
2043	191,123	3.5%	120	22.9	15,927
2044	197,812	3.5%	120	23.7	16,484
2045	**204,736**	3.5%	120	24.6	17,061
2046	211,902	3.5%	120	25.4	17,658
2047	219,318	3.5%	120	**26.3**	18,277
2048	226,994	3.5%	120	27.2	18,916
2049	234,939	3.5%	120	28.2	19,578
2050	**243,162**	3.5%	120	**29.2**	20,263

Note:

1. Population excludes projected population for Madera Acres.

Table 5.5 Design Flows

Sanitary Sewer System Master Plan
City of Madera

Description	Flow	
	Maximum Day (mgd)	Peak Hour (mgd)
2010 Existing Condition Scenarios		
Existing DWF	7.3	10.0
Existing WWF (10Yr-24Hr Design Storm)	9.4	15.5
Ultimate Buildout Scenarios		
Buildout DWF	30.5	44.8
Buildout WWF (10Yr-24Hr Design Storm)	32.7	47.1

6/11/2013

weather conditions. PDWF and PWWF used for evaluating the existing collection system were estimated at 10.0 MGD and 15.5 MGD, respectively. The PDWF and PWWF used for designing the General Plan buildout system, including growth, were estimated at 44.8 MGD and 47.1 MGD, respectively.

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.

6.1 HYDRAULIC MODEL SOFTWARE SELECTION

The City's hydraulic model combines information on the physical characteristics of the sanitary 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 Madera 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.

In the City of Madera's case, skeletonizing was necessary to reduce the model from approximately 3,180 pipes extracted from the GIS to 940 pipes. The modeled pipes included pipes 10-inches in diameter and larger, in addition to some critical smaller gravity sewer pipes.

Table 4.1 lists the wastewater collection system total length of pipes at 176 miles, compared to Table 6.1 listing the total length of modeled pipes at 55 miles. Thus, approximately 31 percent of the total length of gravity sewers was modeled. 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.

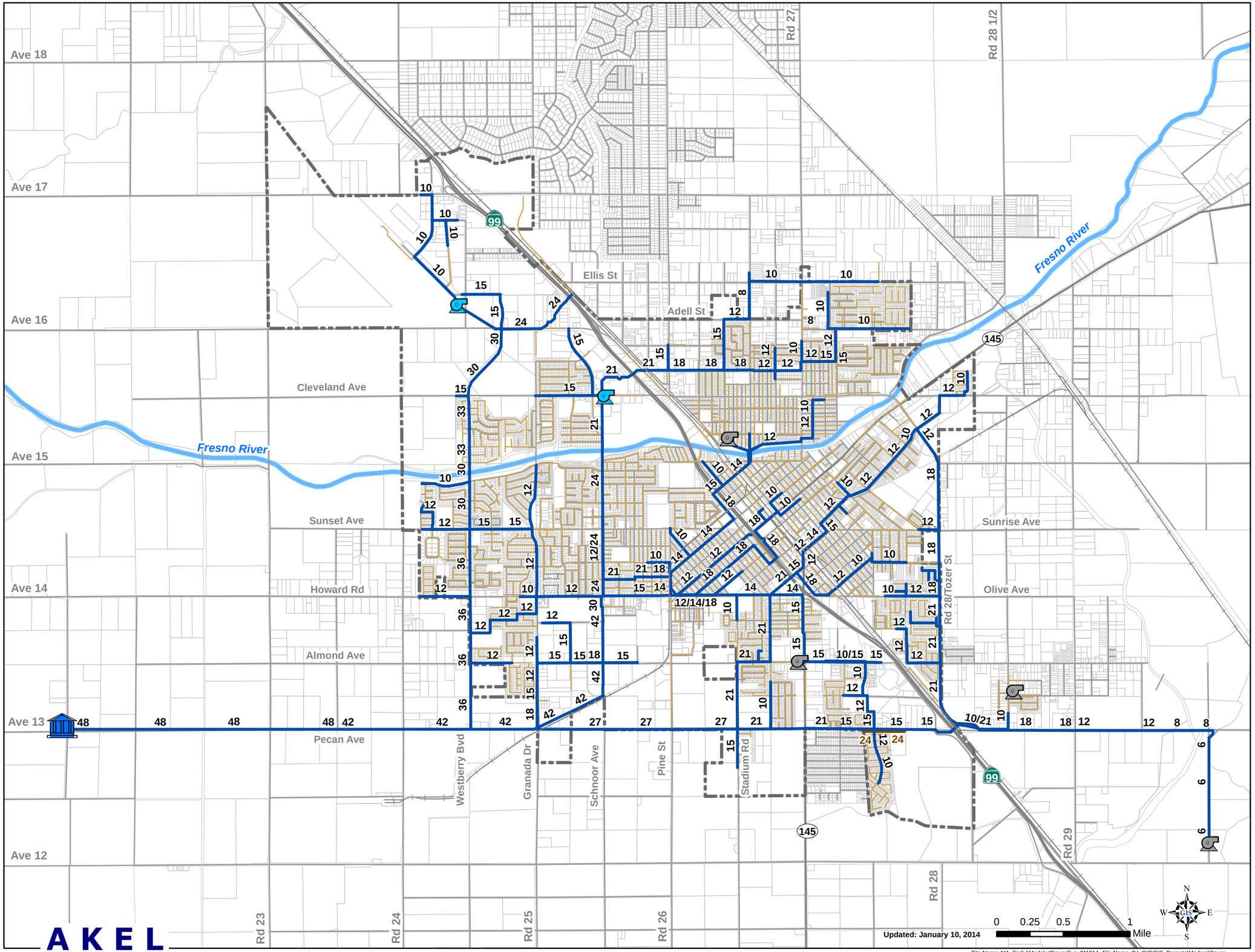
During the development of the new hydraulic model, the project team consisting of City staff and Akel Engineering staff implemented a thorough quality control program to resolve discrepancies. The quality control program included the following:

- The previous hydraulic model, developed in Hydra, and used in the 1997 master plan
- Supplemental field surveys
- As-Built or construction drawings
- Available closed circuit television (CCTV) along major trunks

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. Determining the sewer loads was accomplished by using the wastewater flow factors developed for this master plan and presented in chapter 3, and parcel data including acreage and land use. The loads calculated



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Legend

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes

- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 6.1
Modeled Sewer System
Sanitary Sewer System Master Plan
City of Madera

AKEL
ENGINEERING GROUP, INC.

Table 6.1 Modeled Sewer Pipe Inventory

Sanitary Sewer System Master Plan
City of Madera

Pipe Diameter	Total (ft)	Total (miles)	Percent
6"	4,635	0.9	2%
8"	6,940	1.3	2%
10"	46,942	8.9	16%
12"	68,355	12.9	24%
14"	12,764	2.4	4%
15"	39,836	7.5	14%
18"	24,324	4.6	8%
21"	26,861	5.1	9%
24"	8,174	1.5	3%
27"	7,985	1.5	3%
30"	9,181	1.7	3%
36"	7,909	1.5	3%
42"	14,382	2.7	5%
48"	10,849	2.1	4%
Total	289,137	**55**	**100%**

Note: 3/20/2013

1. The sewer system pipe inventory was extracted from the City's GIS-based hydraulic model developed in August 2012

were allocated to the nearest manhole that serves the corresponding parcel using the capabilities the hydraulic model has for allocating loads.

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 2011 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.

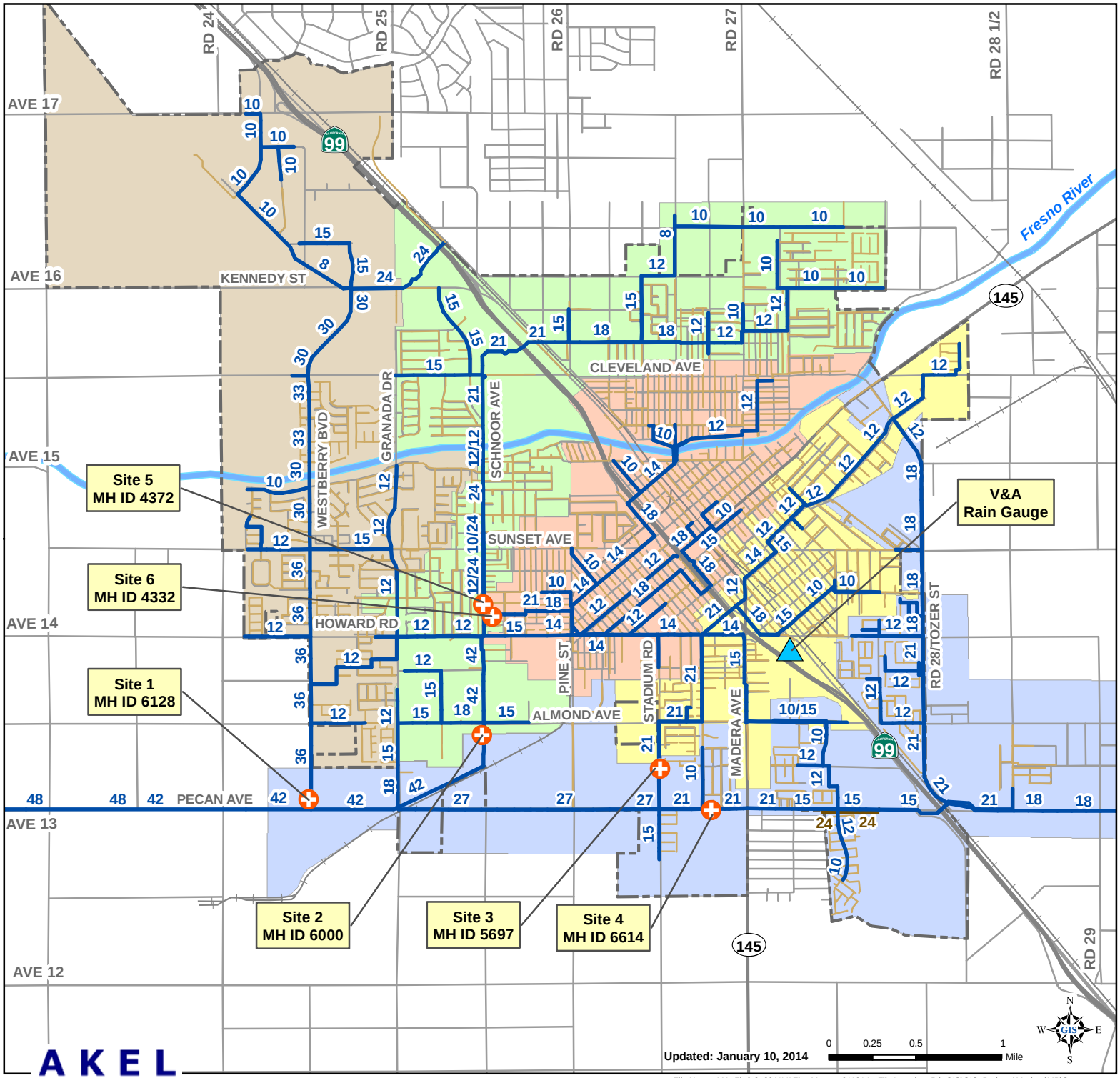
After the model is calibrated to these conditions, it is benchmarked and used for evaluating the capacity adequacy of the sanitary sewer system, under dry and wet weather conditions.

6.3.2 2011 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 6 flow meters, for a period of 18 days, from February 15, 2011 to March 4, 2011. 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**.

The 2011 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.

During the V&A flow monitoring program; one rain gauge 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 2011 V&A Flow Monitoring Program, was used in this analysis to



Legend

- V&A Rain Gauge
- 2011 V&A Flow Meters
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

Basins

- Fourth St
- Pecan
- Schnoor
- Stadium
- Westberry
- Roads
- Railroads
- Highways
- City Limits
- Fresno River

Figure 6.2
2011 V&A Flow Monitoring Program
Sanitary Sewer Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

File Name: MA_Fig6-2_2011VAFlowMeters_011014 File Location: P:\xGIS\GIS_Projects\Madera\MP\Sewer

Table 6.2 Flow Monitoring Sites

Sanitary Sewer System Master Plan
City of Madera

Site No.	GIS Manhole ID	Location	Metered Pipe Size	Monitoring Periods	
			(in)	From Date	To Date
1	6128	Westberry Rd north of Pecan Ave	36	2/15/2011	3/4/2011
2	6000	Schnoor Ave south of Almond Ave	42	2/15/2011	3/4/2011
3	5697	Stadium Rd at Gary Lane	21	2/15/2011	3/4/2011
4	6614	Pecan Ave east of Stadium Rd	21	2/15/2011	3/4/2011
5	4372	Schnoor Ave north of Jennings St	24	2/15/2011	3/4/2011
6	4332	West Fourth St east of Schnoor Ave	21	2/15/2011	3/4/2011

3/26/2013

calibrate the computer hydraulic model to average dry weather flow (ADWF) and peak wet weather flow (PWWF) conditions.

6.3.3 Dynamic Model Calibration

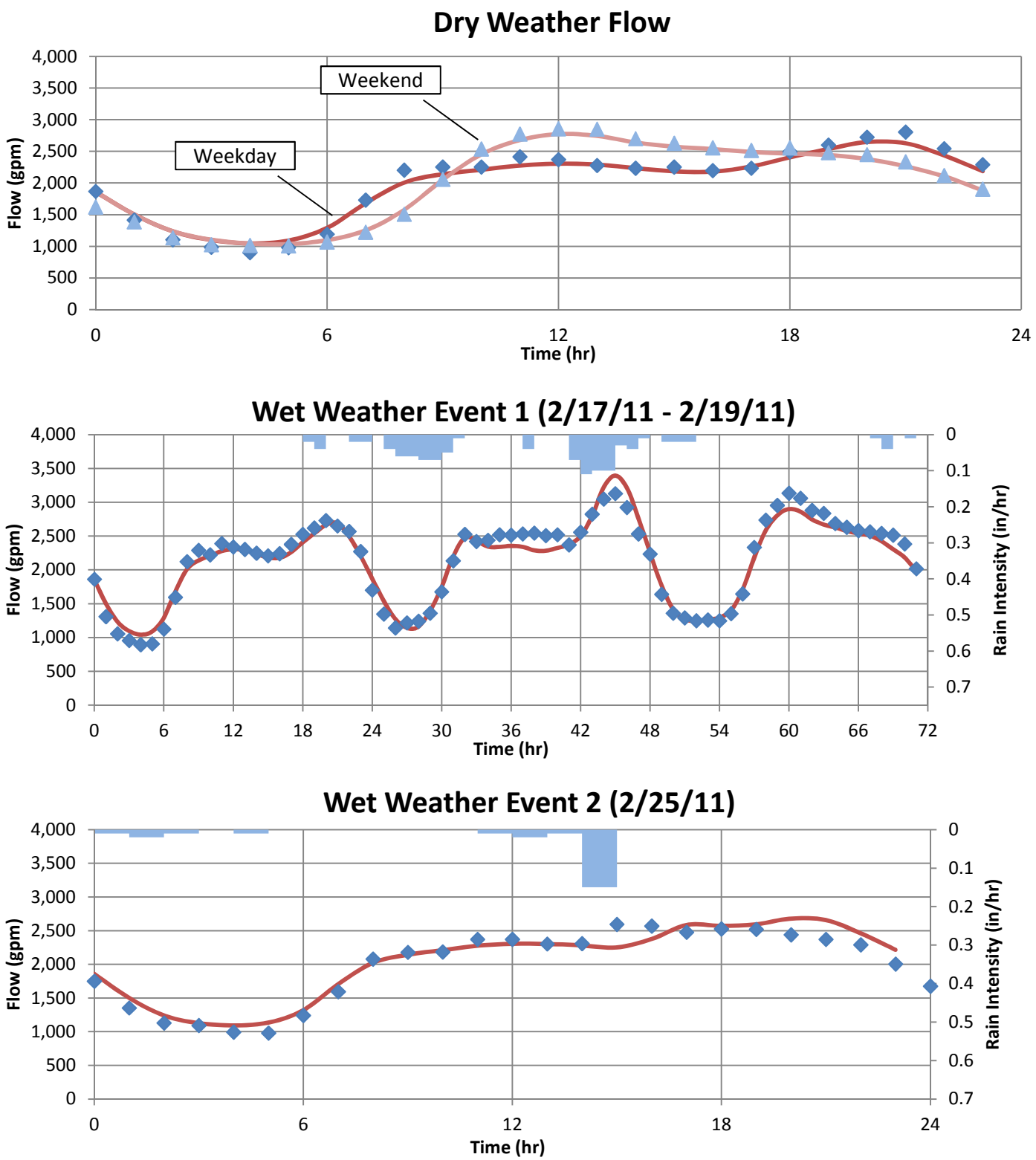
The calibration process was iterative as it involved calibrating each of the 6 flow monitored sites and for the three calibration conditions: 1) peak dry weather flow, 2) peak wet weather flows from storm rainfall Event No. 1, and 3) peak wet weather flows from storm rainfall Event No. 2.

The rain events of February 17, 2011 (Event No. 1) and February 25, 2011 (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 6 sites were extracted from the 2011 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 6 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
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

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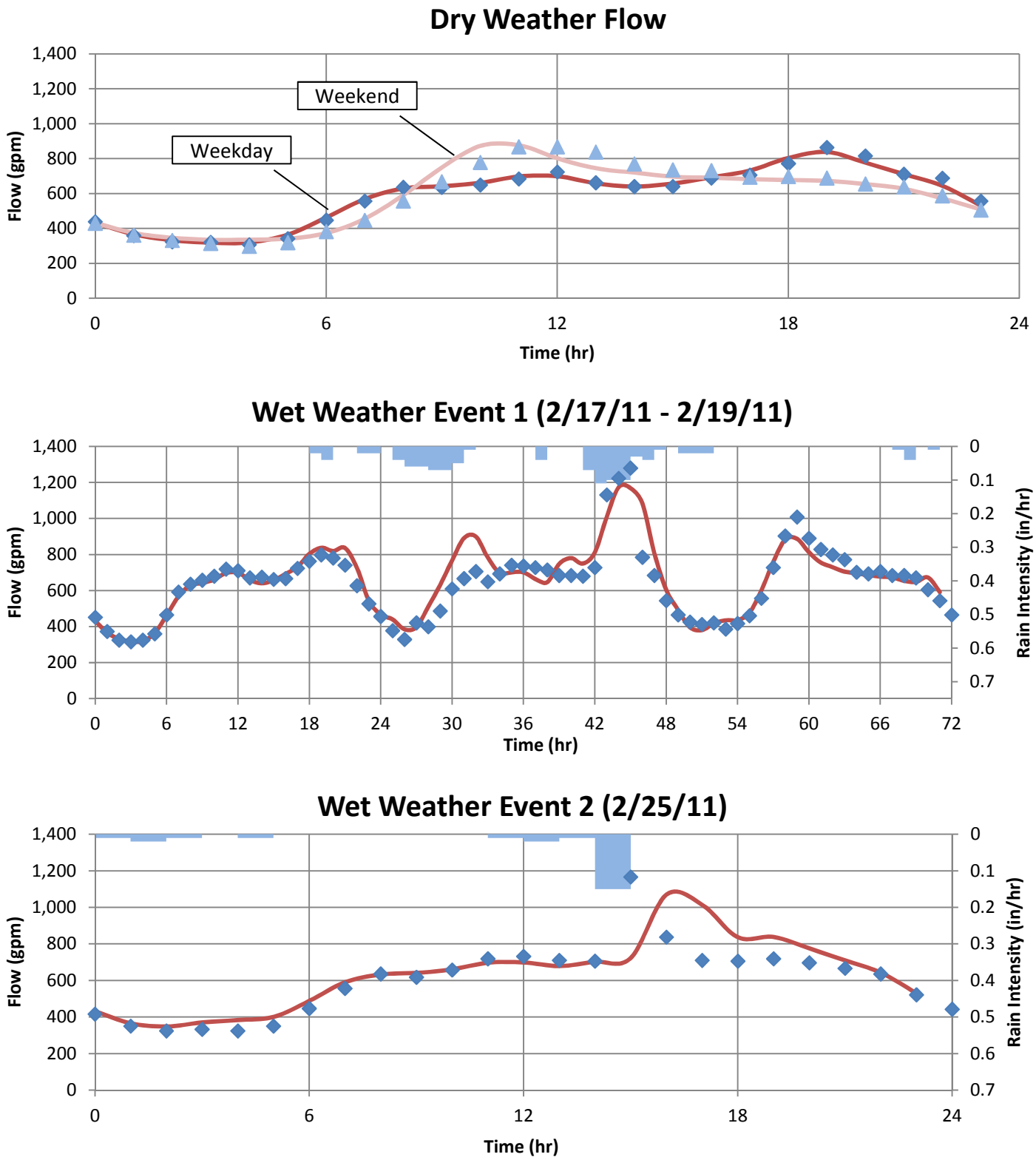
Figure 6.3

Site 2 Calibration

Schnoor Ave, South of Almond Ave

Sanitary Sewer Master Plan
City of Madera



LEGEND

- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

AKEL ENGINEERING GROUP, INC.

Figure 6.4
Site 3 Calibration
Stadium Rd at Gary Lane, North of Pecan Ave
Sanitary Sewer Master Plan
City of Madera

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.

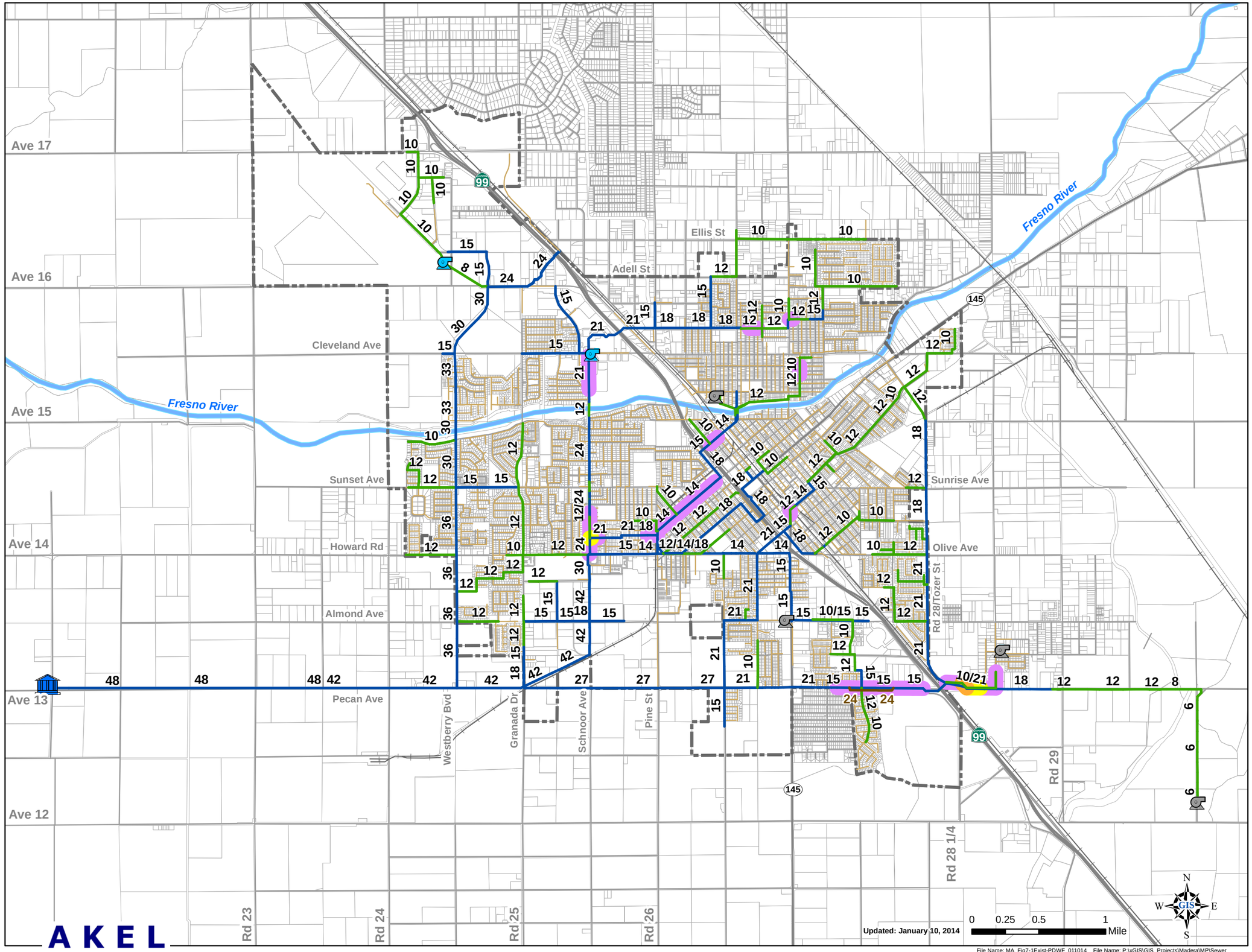
7.2 EXISTING SEWER SYSTEM CAPACITY EVALUATION

The system performance and design criteria summarized, on Table 3.1, was thus 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 5.5 and they include:

- Existing PDWF = 10.0 MGD
- Existing PWWF = 15.5 MGD

During the peak dry weather simulations, the maximum allowable pipe d/D criteria for new pipes (0.5 for 12-inch or smaller and 0.75 for larger than 12-inches) was used. For existing pipes, the criteria was relaxed to allow a maximum d/D ratio of 0.92 (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).



Legend

Analysis Results

Manholes

- HGL within 3 ft of Ground Elevation

Pipe d/D

- d/D ≥ 0.9
- d/D ≥ 0.75
- d/D ≥ 0.6

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes by Size

- 12" and Smaller
- 14" and Larger
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 7.1
Existing System Analysis for PDWF
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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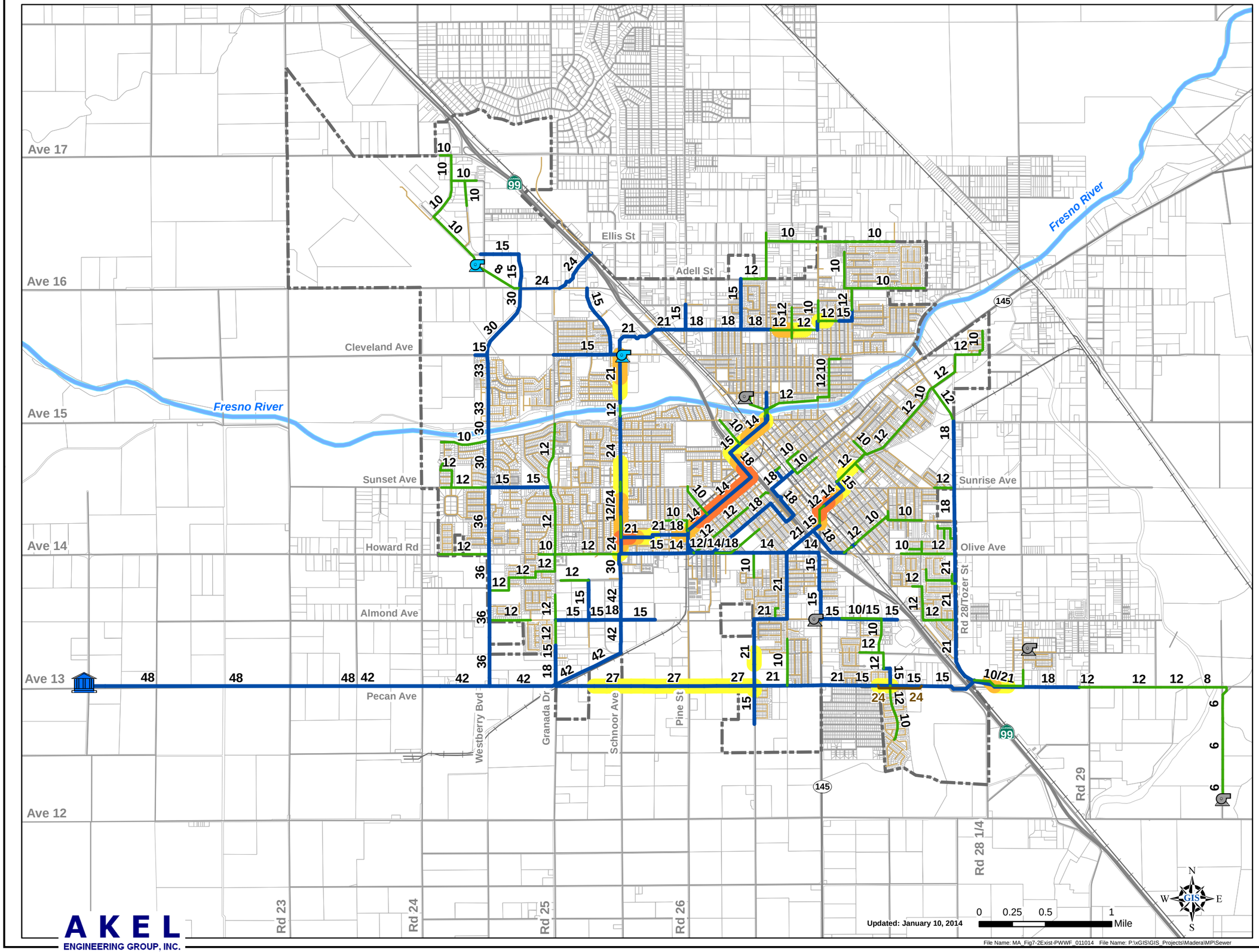


Legend

Analysis Results

Manholes

- HGL within 3 ft of Ground Elevation

Pipe d/D

- d/D ≥ 1.0
- d/D ≥ 0.9
- d/D ≥ 0.75

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes by Size

- 12" and Smaller
- 14" and Larger
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 7.2
Existing System Analysis for PWWF
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

File Name: MA_Fig7-2Exist-PWWF_011014 File Name: P:\xGIS\GIS_Projects\Madera\MP\Sewer

7.2.1 Existing Peak Dry Weather Flows Capacity Evaluation

The design DWF at the Madera Wastewater Treatment Plant for this scenario was estimated at 7.3 MGD while the PDWF was estimated at 10.0 MGD, as documented on **Table 5.5**. In general, the hydraulic model indicated that the sanitary sewer system exhibited acceptable performance to service the existing customers during peak dry weather flows.

The hydraulic model does not predict pipe deficiencies in the existing system during peak dry weather flows, although several trunk reaches approach the maximum capacity criteria (**Figure 7.1**). Several pipe reaches that approach the maximum allowable capacity criteria (d/D Ratio of 0.92) were identified and highlighted on this figure, including:

- Schnoor Avenue (Road 25 ½), from the Fairground Lift Station to Dutra Way. This segment experiences d/D ratios between 0.60 and 0.75.
- Schnoor Avenue (Road 25 ½), from Julius Street to Howard Road (Avenue 14). This segment experiences d/D ratios between 0.60 and 0.75 with a few sections near Fourth Street experiencing d/D ratios approaching 0.92.
- Fourth Street, from Rotan Avenue to I Street. This segment experiences d/D ratios between 0.60 and 0.75.
- Pecan Avenue (Avenue 13), from Golden State Boulevard (Road 28 ¼) to Hickory Avenue. This segment experiences d/D ratios between 0.60 and 0.75.
- Pecan Avenue (Avenue 13), from Road 28 ½ (Raymond Road) to Golden State Highway (State Highway 99). This segment experiences d/D ratios between 0.60 and 0.92.

7.2.2 Existing Peak Wet Weather Flows Capacity Evaluation

The design WWF at the Madera Wastewater Treatment Plant for this scenario was estimated at 9.4 MGD while the PWWF was estimated at 15.5 MGD, as documented on **Table 5.5**. In general, the hydraulic model indicated that the sanitary sewer system exhibited acceptable performance to service the existing customers during peak wet weather flows.

The hydraulic model does not predict pipe deficiencies in the existing system during peak wet weather flows, although several trunk reaches approach the maximum capacity criteria (**Figure 7.2**). Several pipe reaches that approach the maximum allowable capacity criteria (3 feet below manhole rim) were identified and highlighted on this figure, including:

- Schnoor Avenue (Road 25 ½), from the Fairground Lift Station to Dutra Way. This segment experiences d/D ratios between 0.75 and 0.92.
- Schnoor Avenue (Road 25 ½), from Park Drive to Howard Road (Avenue 14). This segment experiences d/D ratios between 0.75 and 0.92.

- Fourth Street, from Schnoor Avenue (Road 25 ½) to Williams Avenue. This segment experiences d/D ratios between 0.75 and 1.0.
- Fourth Street, from Rotan Avenue to I Street. This segment experiences surcharging conditions.
- I Street, from Fourth Street to First Street. This segment experiences d/D ratios between 0.75 and 1.0.
- First Street, from I Street to Central Avenue. This segment experiences d/D ratios between 0.75 and 0.92.
- Madera Avenue (Highway 145/Road 27) and Ninth Street, from Tenth Street to B Street. This segment experiences d/D ratios between 0.75 and 1.0.
- Pecan Avenue (Avenue 13), from Stadium Road (Road 26 ½) to Schnoor Street (Road 25 ½). This segment experiences d/D ratios between 0.75 and 0.92.
- Sherwood Way, from Nebraska Avenue to Bloker Street. This segment experiences d/D ratios between 0.75 and 0.92.
- Pecan Avenue (Avenue 13), at Road 28. This segment experiences d/D ratios between 0.75 and 0.92.

7.3 ULTIMATE BUILDOUT CAPACITY IMPROVEMENTS

The system performance and design criteria summarized on **Table 3.1**, was used as a basis to judge 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 5.5** and they include:

- Buildout PDWF = 44.8 MGD
- Buildout PWWF = 47.1 MGD

During the peak dry weather simulations, the maximum allowable pipe d/D criteria for new pipes (0.5 for 12-inch or smaller and 0.75 for larger than 12-inches) was used. For existing pipes, the criteria was relaxed to allow a maximum d/D ratio of 0.92 (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.

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., P-1), along with other relevant information including alignment description, pipe size, pipe length, and a suggested phasing. The

Table 7.1 Proposed Capacity Improvements

Table 7.1 Proposed Capacity Improvements

Sanitary Sewer System Master Plan
City of Madera

Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter (in)	Parallel, Replace, or New	New Pipe Diameter (in)	Pipe Length (ft)	Village/District	Suggested Phasing
Pecan Basin									
Pecan Ave Trunk									
P-1	Pipe	Pecan Avenue	From w/o Highway 99 to 400 ft w/o Golden State Blvd	15	Parallel	15	1,025	Parkwood	FY 2013 - 2015
P-1A	Pipe	Pecan Avenue	From 400ft w/o Golden State to 1,360 ft e/o Garnet Ave	15	Parallel	18	775	Parkwood	FY 2013 - 2015
P-2	Pipe	Pecan Avenue	From 460ft w/o Garnet Ave to 40 ft w/o Raymond Thomas St	15	Parallel	18	920	Parkwood	FY 2013 - 2015
P-3	Pipe	Pecan Avenue	From 40 ft w/o Raymond Thomas St to Stadium Road (Road 26 1/2)	21	Parallel	18	6,230	Parkwood	FY 2013 - 2015
P-4	Pipe	Pecan Avenue	From Granada Dr (Road 25) to Westberry Blvd (Road 24 1/2)	42	Parallel	36	2,670	N/A	FY 2026 - 2030
P-5	Pipe	Pecan Avenue	From Westberry Blvd (Road 24 1/2) to Road 24	42	Parallel	42	2,740	N/A	FY 2026 - 2030
P-6	Pipe	Pecan Avenue	From Road 24 to Road 23 1/2	42	Parallel	48	2,660	West Madera, N/A	FY 2026 - 2030
P-7	Pipe	Pecan Avenue	From Road 24 to WWTP	48	Parallel	48	11,280	N/A	FY 2026 - 2030
Road 28 1/2 Trunk									
P-8	Pipe	Road 28 1/2, Parallel to HWY 99	From Pecan Ave to Avenue 12 1/2		New	42	2,560	Community College	FY 2016 - 2020
P-9	Pipe	Road 29 1/2	From Avenue 13 1/2 to Pecan Avenue		New	12	2,750	Parksdale	FY 2016 - 2050
P-10	Pipe	Road 28 1/2	From Ave 14 to Pecan Ave	8/10	New/Parallel	36	5,340	Parksdale	FY 2016 - 2050
P-11	Pipe	Avenue 14	Road 30 to Road 28 1/2		New	18	7,740	Parksdale, N/A	FY 2016 - 2050
P-12	Pipe/Casing[1]	Avenue 14	Crossing the Main Canal		New	18/42	150	Parksdale, N/A	FY 2016 - 2050
P-13	Pipe	Road 30	From Ave 13 1/2 ext to Ave 14		New	15	2,640	N/A	FY 2016 - 2050
P-14	Pipe	Santa Fe Railroad ROW	From Ave 14 1/2 ext to Ave 14		New	15	3,130	N/A	FY 2016 - 2050
P-15	Pipe	Road 28 1/2	From Ave 14 1/2 to Ave 14		New	30	2,640	Parksdale	FY 2016 - 2050
P-16	Pipe	Road 28 1/2	From Ave 15 1/2 to Ave 14 1/2		New	15	5,320	Parksdale	FY 2016 - 2050
P-17	Pipe	Avenue 14 1/2	Road 29 1/4 ext to Road 28 1/2		New	27	3,990	Parksdale	FY 2016 - 2050
P-18	Pipe/Casing[1]	Road 29 1/4 extension	Crossing the Main Canal		New	27/54	150	N/A	FY 2016 - 2050
P-19	Pipe	Road 29 1/4 extension	Ave 15 to Ave 14 1/2		New	27	2,210	N/A	FY 2016 - 2050
P-20	Pipe	Avenue 15	Ave 29 to Road 29 1/4 ext		New	27	1,290	N/A	FY 2016 - 2050
P-21	Pipe	Road 29	From Ave 15 1/2 to Avenue 15		New	27	2,700	N/A	FY 2016 - 2050
P-22	Pipe	Avenue 15 1/2	From Santa Fe Dr to Road 29		New	24	160	N/A	FY 2016 - 2050
P-23	Pipe/Casing[1]	Avenue 15 1/2	Crossing Railroad Tracks		New	24/48	150	N/A	FY 2016 - 2050
P-24	Pipe	Road 29	From 660ft n/o Ave 15 1/2 to Ave 15 1/2		New	24	660	N/A	FY 2016 - 2050
P-25	Pipe	Road 29	From River Rd to 660ft n/o Ave 15 1/2		New	21	3,810	N/A	FY 2016 - 2050
P-26	Pipe	Highway 145	From Juanita Dr to Road 29		New	21	10,240	N/A	FY 2016 - 2050
Ave 12 1/2 Trunk									
P-27	Pipe	Granada Dr (Road 25)	From Ave 12 1/2 to Pecan Ave (Ave 13)		New	42	2,700	N/A	FY 2016 - 2020
P-27A	Pipe/Casing[1]	Granada Dr (Road 25)	Crossing Railroad Tracks		New	42/66	50	N/A	FY 2016 - 2020
P-28	Pipe	Avenue 12 1/2	From Highway 145 to Granada Dr (Road 25)		New	42	10,560	Parkwood, N/A	FY 2016 - 2020
P-29	Pipe	Highway 145	From 1,110 ft s/o Avenue 12 1/2 to Avenue 12 1/2		New	42	1,110	Parkwood	FY 2016 - 2020
P-30	Pipe	Borden St/City ROW/Burges	From Ave 12 1/2 to Highway 145		New	42	9,710	Parkwood	FY 2016 - 2020
P-31	Pipe/Casing[1]	City ROW near Borden St	Crossing Highway 99 and Railroad Tracks		New	42/66	300	Parkwood, Community College	FY 2016 - 2020
P-32	Pipe	Avenue 12 1/2	From Road 29 to Borden St Ext		New	30	1,640	Community College	FY 2016 - 2050
P-33	Pipe	Avenue 12 1/2	From Road 29 1/2 extension to Road 29		New	12	2,450	Community College	FY 2016 - 2050
P-34	Pipe	Road 29	From 680 ft n/o Avenue 12 to Avenue 12 1/2		New	30	2,290	Community College	FY 2016 - 2050
P-35	Pipe/Casing[1]	City ROW near Ave 12 and Road 29	Crossing creek and canal		New	24/48	490	Community College	FY 2016 - 2050
P-36	Pipe	City ROW near Ave 12 and Road 29	From 900 ft e/o Road 29 to 675 ft nw/o Ave 12		New	24	680	Community College	FY 2016 - 2050
P-37	Pipe	Avenue 12	From Road 30 1/2 to 900 ft e/o Road 29		New	24	7,000	Community College	FY 2016 - 2050
P-38	Pipe	Avenue 12 1/2 extension Road 29 1/2 Extension	From 660ft e/o Road 30 then SE to Ave 12		New	12	4,380	Community College	FY 2016 - 2050
P-39	Pipe	Avenue 12	From 3,990 ft e/o Road 30 1/2 to Road 30 1/2		New	21	3,990	Community College	FY 2016 - 2050
P-40	Pipe	Road 30 1/2	From Pecan Ave to Ave 12		New	15	5,290	Community College	FY 2016 - 2050
P-41	Pipe	Avenue 12 1/2 extension	From 2,630 ft e/o Road 30 1/2 to Road 30 1/2		New	15	2,630	Community College	FY 2016 - 2050

Table 7.1 Proposed Capacity Improvements

Sanitary Sewer System Master Plan
City of Madera

Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter (in)	Parallel, Replace, or New	New Pipe Diameter (in)	Pipe Length (ft)	Village/District	Suggested Phasing
Schnoor Basin									
Sherwood Way Trunk									
S-1	Pipe	Wessmith Way	190 ft e/o Lake St. to Lake St. (Road 27)	10	Replace	15	190	Central Madera	FY 2016 - 2050
S-2	Pipe	Sherwood Way	Lake St. to 220ft w/o Nebraska Ave.	12	Replace	15	1,930	Central Madera	FY 2016 - 2050
S-3	Pipe	Kennedy Street (Avenue 16)	Road 28 to Chapin St.		New	10	1,120	Central Madera	FY 2016 - 2050
S-4	Pipe	Road 28	680 ft n/o Kennedy St (Ave 16) to Kennedy St (Ave 16)		New	10	680	Central Madera	FY 2016 - 2050
Schnoor Trunk									
S-6	Lift Station[2]	Fairgrounds Lift Station Pump Capacity Upgrade			Replace	2,500 gpm		Northwest Madera	FY 2013 - 2015
Westberry Basin									
Ellis Street Trunk									
W-3	Pipe	Ellis Street	From Krohn St to Sharon Blvd		New	24	1,070	Central Madera	FY 2016 - 2050
W-4	Pipe	Ellis Street	From D St to Krohn St		New	21	5,890	Central Madera	FY 2016 - 2050
W-5	Pipe/Casing[1]	Ellis Street	Crossing Canal		New	21/48	150	Central Madera	FY 2016 - 2050
W-6	Pipe	D Street	From Martin St to Ellis St		New	21	1,020	Central Madera	FY 2016 - 2050
W-7	Pipe	Lake Street (Road 27)	From 2,120ft n/o Martin St to Martin St		New	12	2,120	Central Madera, North Madera	FY 2016 - 2050
W-8	Pipe	Martin Street Extension	From 3,330 e/o Lake St (Road 27) to D St		New	21	4,340	Central Madera	FY 2016 - 2050
W-9	Pipe	Future City Road or ROW (3,330ft e/o Lake Street)	From Avenue 17 to Martin St Ext		New	21	1,290	Central Madera	FY 2016 - 2050
W-10	Pipe/Casing[1]	Avenue 17	Crossing RR Tracks		New	21/48	150	Central Madera	FY 2016 - 2050
W-11	Pipe	Avenue 17	From 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4) to 560ft w/o RR Tracks		New	21	410	Northeast Madera	FY 2016 - 2050
W-12	Pipe	Future Road 27 3/4	From Avenue 17 1/2 Ext to Avenue 17		New	15	2,640	Northeast Madera	FY 2016 - 2050
W-13	Pipe	Avenue 17 1/2 extension	Tuolumne St Ext to Future Road 27 3/4		New	12	2,000	Northeast Madera	FY 2016 - 2050
W-14	Pipe	Avenue 17	From Harper Blvd to 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4)		New	15	420	Northeast Madera	FY 2016 - 2050
W-15	Pipe	Avenue 17	From 2,370 ft e/o Harper Blvd to Harper Blvd		New	10	2,370	Northeast Madera	FY 2016 - 2050
W-16	Pipe	Harper Boulevard	From Road 28 1/2 to Avenue 17		New	15	4,750	Northeast Madera	FY 2016 - 2050
W-17	Pipe	Future Road or ROW	From Arizona Ave southwest to Road 28 1/2		New	12	2,260	Northeast Madera	FY 2016 - 2050
Road 23 Trunk									
W-18	Pipe	Road 23	From Ave 16 to Pecan Ave (Ave 13)		New	30	14,950	Northwest Madera, N/A	FY 2016 - 2050
W-19	Pipe/Casing[1]	Road 23	Crossing Fresno River		New	30/54	950	Northwest Madera, N/A	FY 2016 - 2050
W-20	Lift Station[2]	Avenue 16 and Road 23			New	2,300 gpm		Airport North	FY 2016 - 2050
W-21	Pipe	Avenue 16	From Road 22 1/2 to Road 23		New	18	2,750	N/A	FY 2016 - 2050
W-22	Pipe	Road 23	From Ave 18 to Ave 16		New	15	10,550	Northwest Madera	FY 2016 - 2050
W-23	Pipe/Casing[1]	Avenue 18	Crossing HWY 99 and Railroad Tracks		New	12/36	550	Airport North	FY 2016 - 2050
W-24	Pipe	Avenue 18	From Sharon Blvd/Road 24 to Road 23		New	12	4,760	Airport North, N/A	FY 2016 - 2050
W-25	Pipe	Road 23 1/2	From Almond Ave (Ave 13 1/2) to Pecan Ave (Ave 13)		New	12	2,600	N/A	FY 2016 - 2050
W-26	Pipe	Avenue 14	From Road 23 1/2 to Road 23		New	12	2,600	N/A	FY 2016 - 2050
W-27	Pipe	Avenue 14 1/2	From Road 23 1/2 to Road 23		New	12	2,600	N/A	FY 2016 - 2050
W-28	Pipe	Avenue 15 1/2	From Avenue 24 to Road 23		New	15	5,275	Northwest Madera	FY 2016 - 2050

Notes:
1. Proposed casings size and carrier pipe size.
2. Lift Station capacity is given as capacity with largest pump out of service.

6/12/2013

proposed improvements are shown with pipe sizes on an overall exhibit on Figure 7.3. This exhibit was further divided into four quadrants to show a greater level of detail, including the improvement numbers, as shown on Figures 7.4 - 7.7.

Figure 7.8 shows the future trunk system that was developed for organizing the future system improvements in Table 7.1.

7.3.1 Capacity Improvement Phasing

The Capacity Improvements for the future flows are divided into the following phases:

- **Near Term:** This short-term phase consists of improvements for the fiscal years (FY) 2013 through 2015 for improvements that are required to resolve existing deficiencies and other critical pipes in the sanitary sewer system.
- **Intermediate Term:** This intermediate term phase includes improvements that are required to be completed for fiscal years 2016 through 2020.
- **Long Term:** This long term phase includes improvements that are required to be completed for fiscal years 2021 through 2030.
- **Other Long Term and Extended Range:** This phase includes the rest of the improvements for the future system. As it is difficult to impossible to guess which direction the City will develop, these improvements are labeled as FY 2031-2050, which indicate these improvements will be constructed as new developments necessitate them.

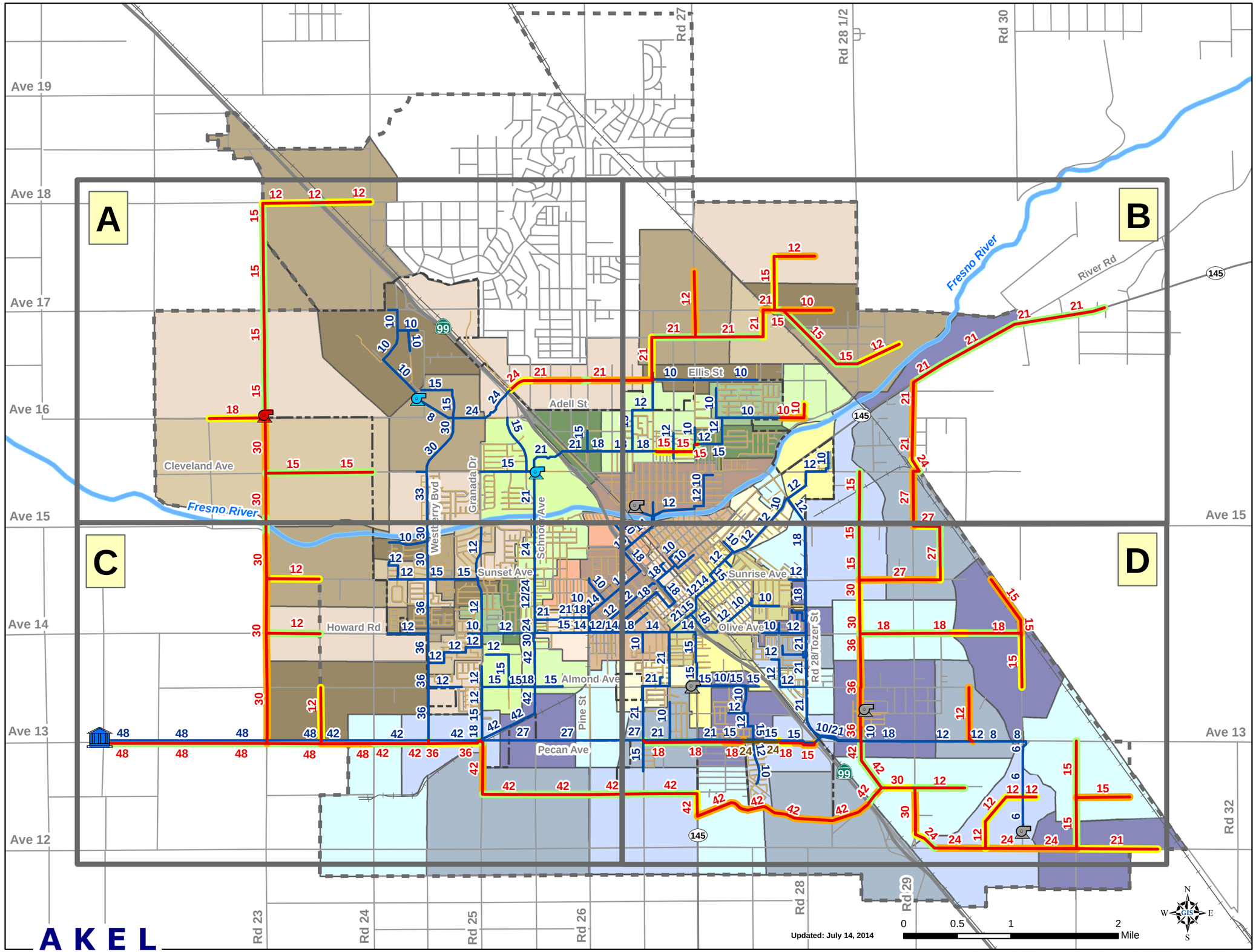
7.3.2 Near-Term Future Capacity Improvements

The existing system evaluation revealed no pipe capacity deficiencies for either the existing peak dry weather flow or the existing peak wet weather flow; however, there is one stretch of pipes along Pecan Avenue between Highway 99 and Stadium Road that is nearing capacity. Capacity issues in this stretch would cause a bottleneck for any new developments in the Road 28 Trunk and CSA3 areas. City staff has indicated that these improvements should be considered as near-term improvements.

The following improvements shown on Figure 7.7 are considered near term improvements:

Pecan Basin – Pecan Avenue Trunk Improvements:

- Improvement P-1: Parallel the existing 15-inch gravity sewer in Pecan Avenue (Avenue 13) from Golden State Highway (State Highway 99) to 400 feet west of Golden State Boulevard (Road 28 ¼) with a new 15-inch pipe.
- Improvement P-1A: Parallel the existing 15-inch gravity sewer in Pecan Avenue (Avenue 13) from 400 feet west of Golden State Boulevard (Road 28 ¼) to 1,360 feet east of Garnet Avenue with a new 18-inch pipe.



Legend

Improvements

- Lift Station
- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 7.3
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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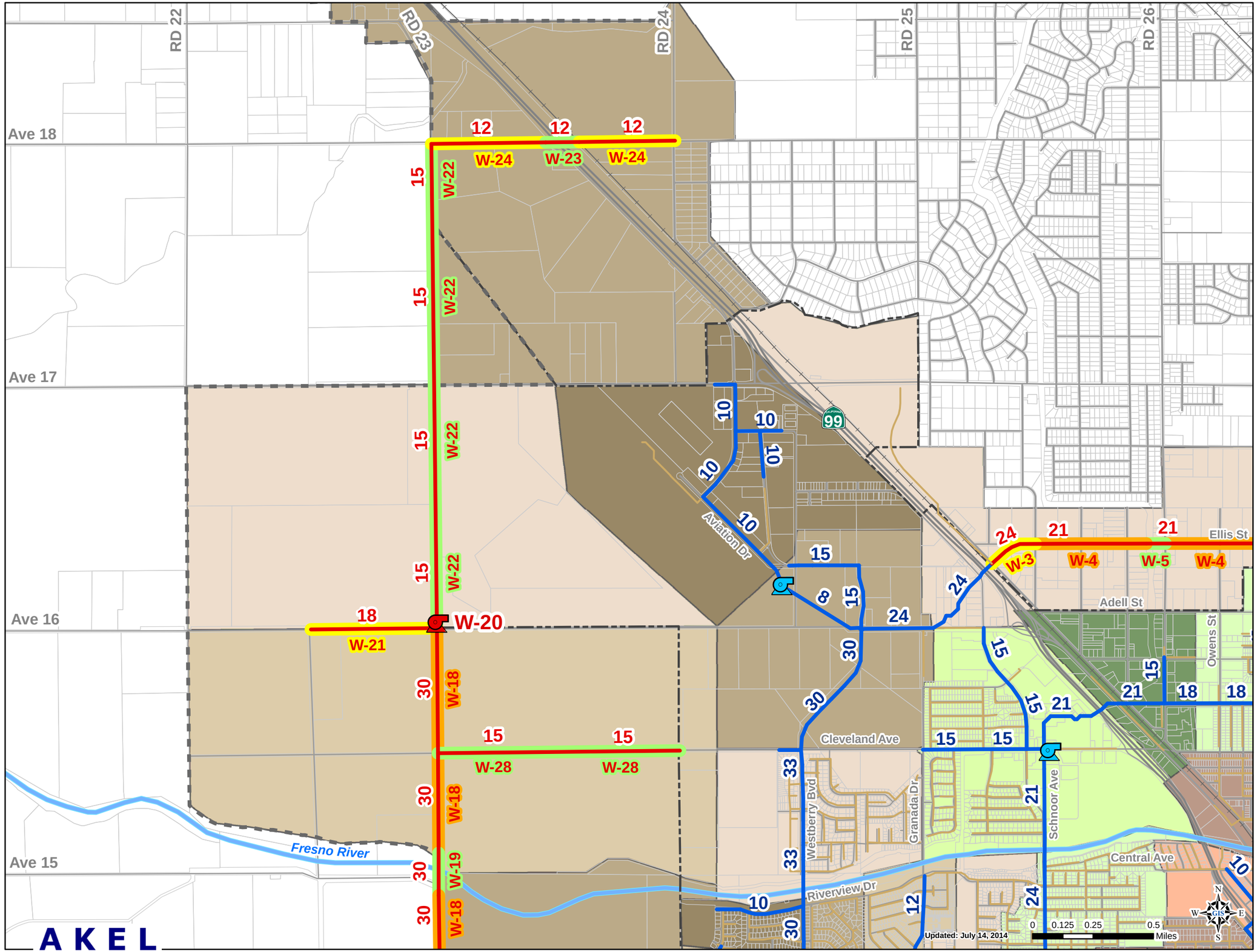


Legend

Improvements

- Lift Station
- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

- Modeled Lift Stations
- Modeled Pipes
- Non-Modeled Pipes

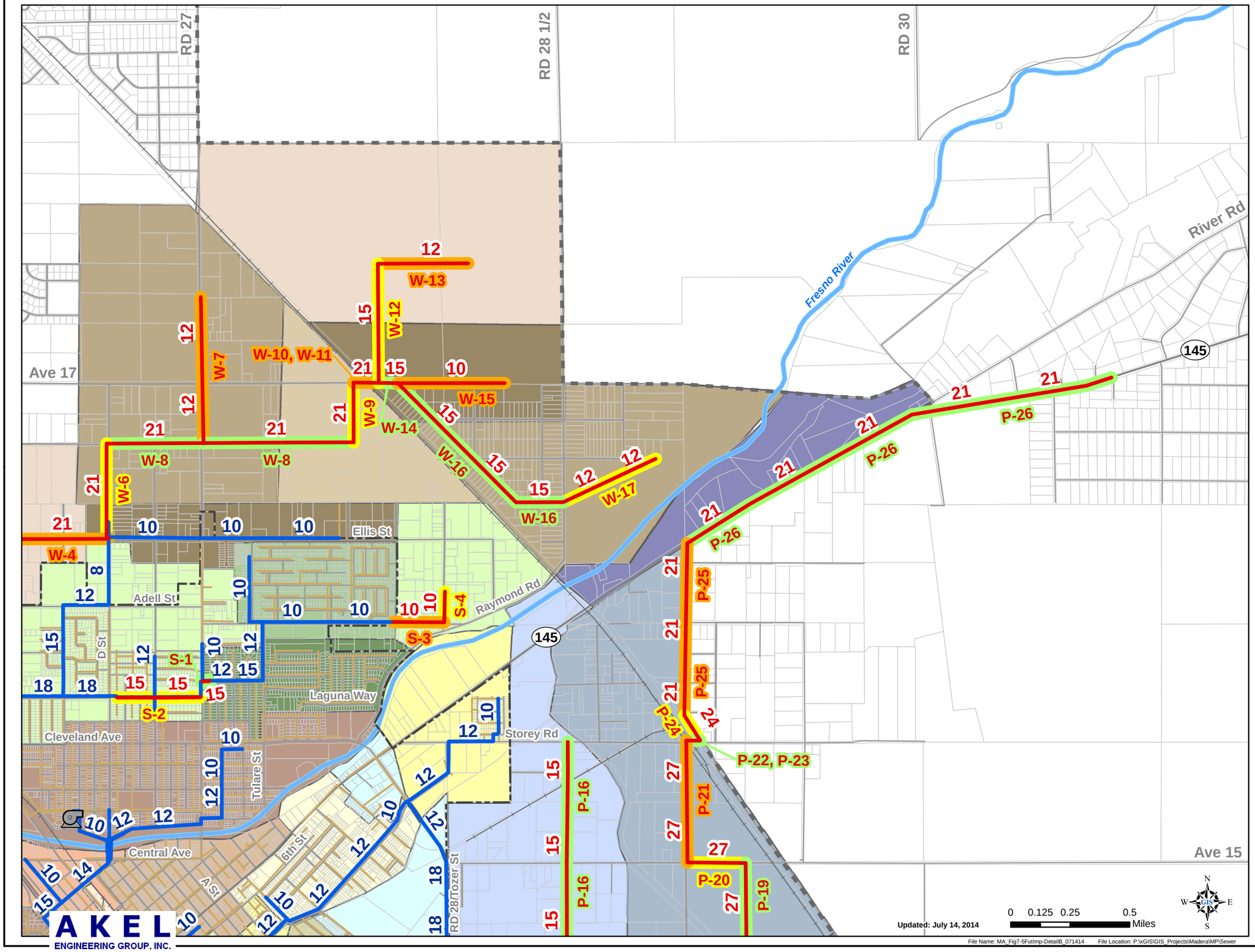
Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 7.4 - Detail A
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

File Name: MA_Fig7-4FutImp-DetailA_071414 File Location: P:\xGIS\GIS_Projects\Madera\MP\Sewer



Legend

Improvements

- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

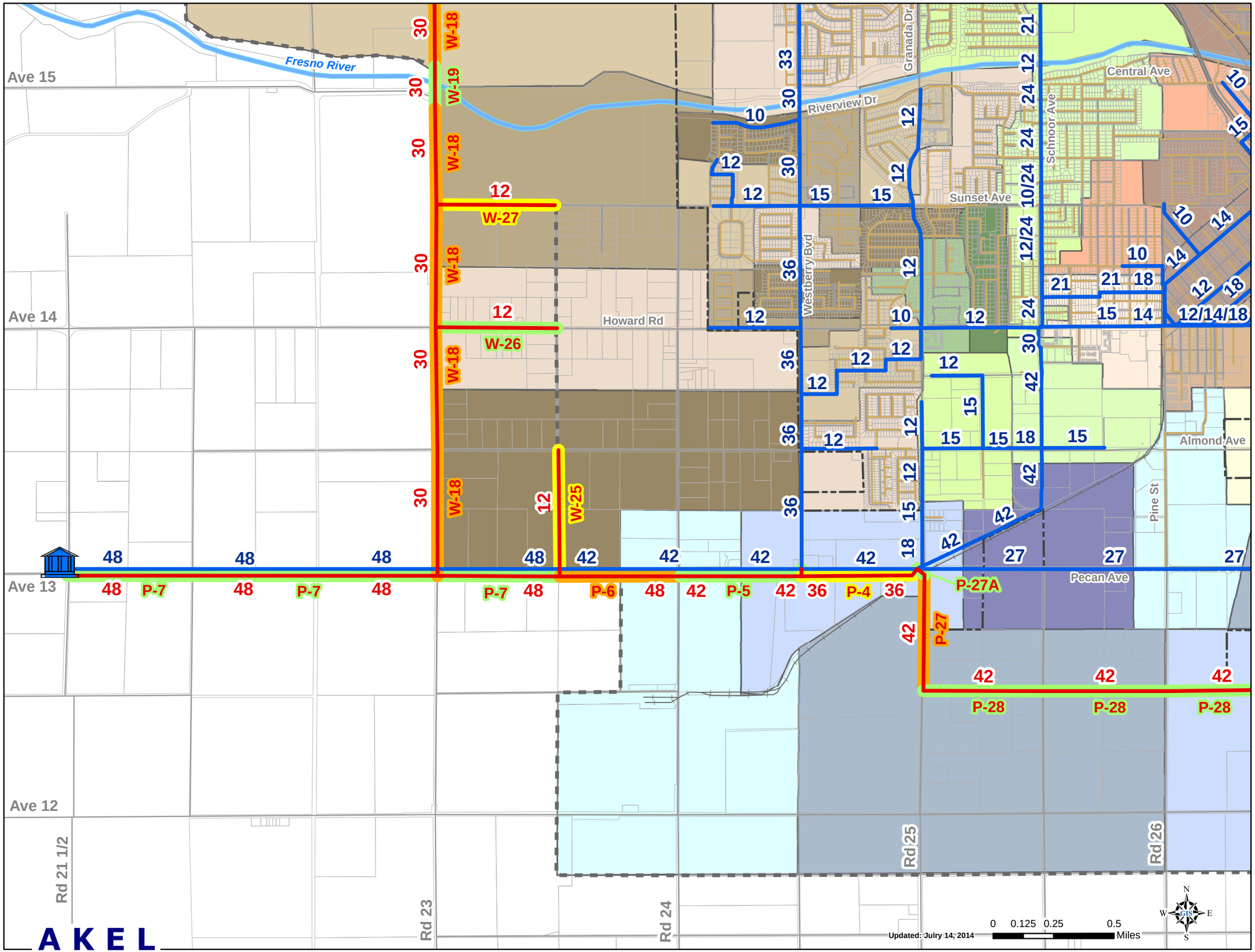
- Non-Modeled Lift Stations
- Modeled Pipes
- Non-Modeled Pipes

Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 7.5 - Detail B
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.



Legend

Improvements

- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

- WWTP
- Modeled Pipes
- Non-Modeled Pipes

Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 7.6 - Detail C
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

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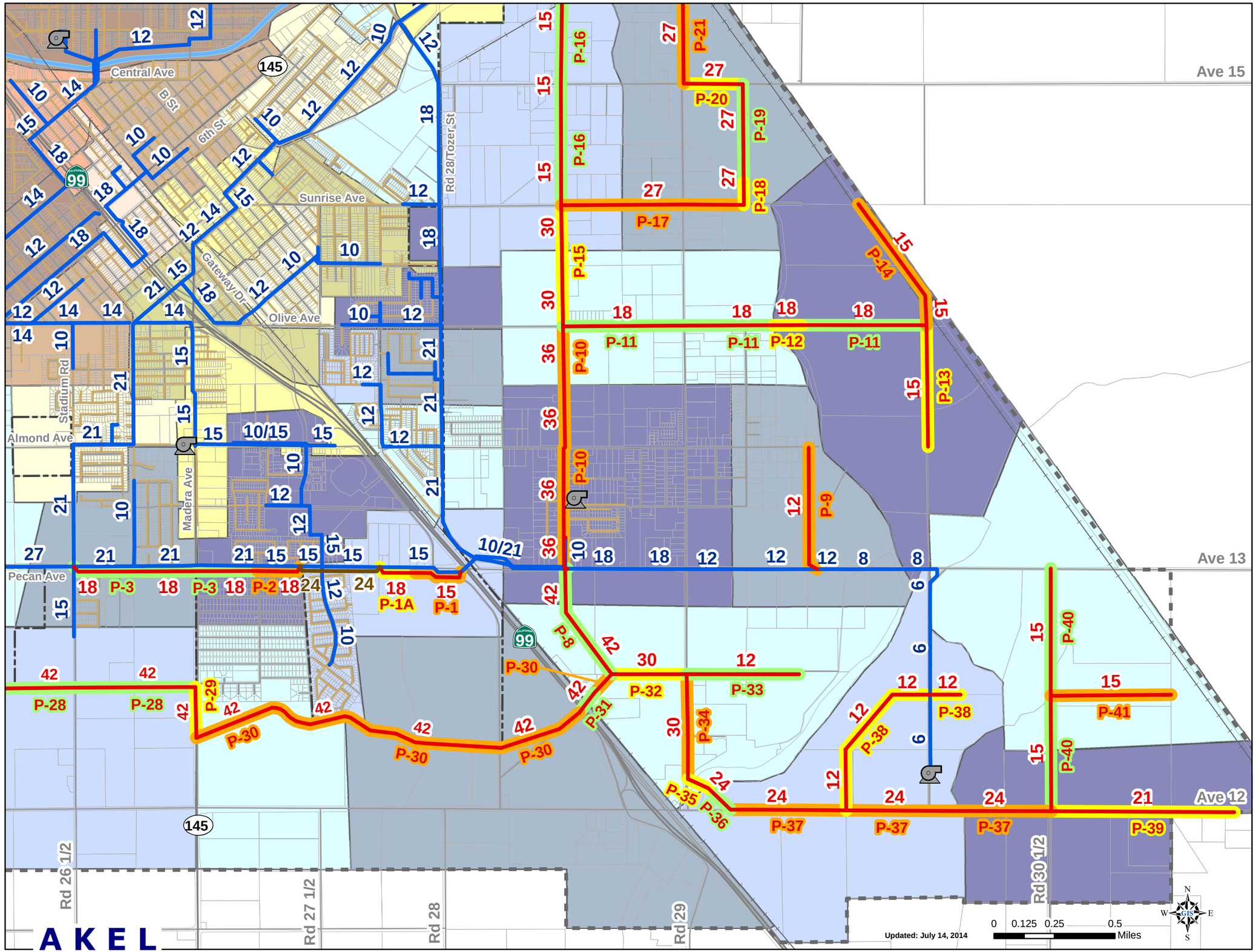

Legend

Improvements

- Pipes
- CIP Pipe Segments
- CIP Pipe Segments
- CIP Pipe Segments

Existing

- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

Basins

- Fourth St Basin
- Pecan Basin
- Schnoor Basin
- Stadium Basin
- Westberry Basin
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 7.7 - Detail D
Capacity Improvements
Sanitary Sewer System Master Plan
City of Madera

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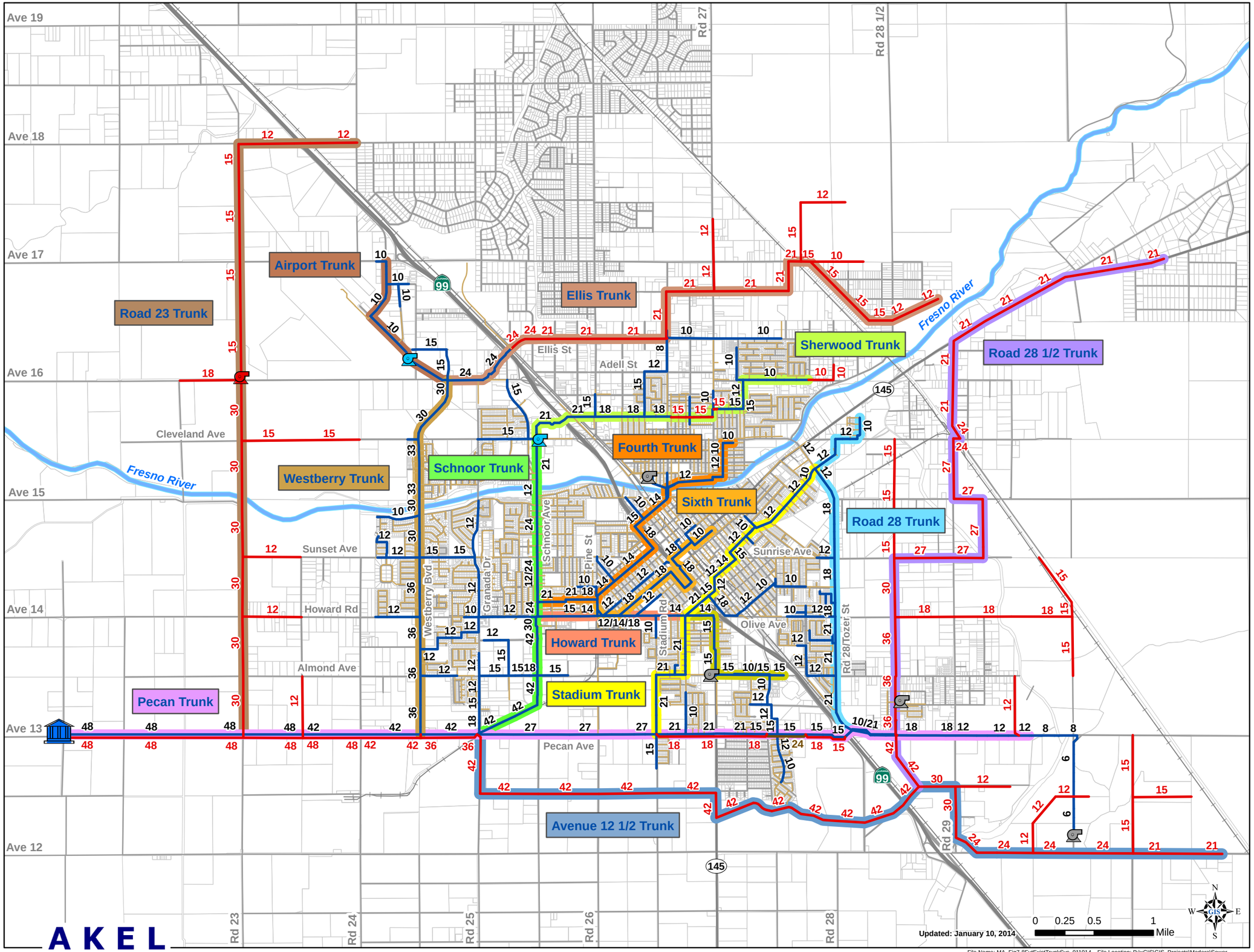


Legend

Future Improvements

- Lift Station
- Pipes

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

Trunks By Basins

Pecan Basin
- Pecan
- Road 28
- Avenue 12 1/2 (Future)
- Road 28 1/2 (Future)

Schnoor Basin
- Schnoor
- Sherwood

Fourth Street Basin
- Fourth
- Sixth
- Howard

Stadium Basin
- Stadium
- Hospital

Westberry Basin
- Westberry
- Airport
- Road 23 (Future)
- Ellis (Future)

- Roads
- Highways
- Railroads
- Parcels
- Fresno River

Figure 7.8
Existing and Future Trunk System
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

- Improvement P-2: Parallel the existing 15-inch gravity sewer in Pecan Avenue (Avenue 13) from 460 feet west of Garnet Avenue to 40 feet west of Raymond Thomas Street with a new 18-inch pipe.

- Improvement P-3: Parallel the existing 21-inch gravity sewer in Pecan Avenue (Avenue 13) from 40 feet west of Raymond Thomas Street to Stadium Road (Road 26 ½) with a new 18-inch pipe.

There is an existing 1,700 feet of 24-inch parallel relief sewer that was constructed in Pecan Avenue (Avenue 13) from 1,360 feet east of Garnet Ave and 460 feet west of Garnet Avenue (between improvements P-1A and P-2 on Figure 7.7)and which seems to have been capped. This segment can be used in conjunction with improvements P-1, P-1A, P-2, and P-3 to eliminate the bottleneck in this area and increase the capacity along this portion of the Pecan Trunk.

7.3.3 Intermediate-Term Future Capacity Improvements

Due to the lack of existing sewer capacity and infrastructure for new developments on the east side of Madera including the villages of Parkwood, Parksdale, and the Community College; these capacity improvements have been identified as intermediate-term.

The following improvements shown on Figure 7.6 and 7.7 are considered intermediate-term improvements:

Pecan Basin – Road 28 ½ Trunk Improvements:

- Improvement P-8: Construct a new 42-inch gravity sewer in Raymond Road (Road 28 ½) parallel to Golden State Highway (State Highway 99) from Pecan Avenue (Avenue 13) to Pecan Lane (Avenue 12 ½).

Pecan Basin – Avenue 12 ½ Trunk Improvements:

- Improvement P-27: Construct a new 42-inch gravity sewer in Granada Drive (Road 25) from Pecan Lane (Avenue 12 ½) to Pecan Avenue (Avenue 13).

- Improvement P-27A: Construct a new 42-inch gravity sewer and 66-inch casing in Granada Drive (Road 25) for the railroad crossing.

- Improvement P-28: Construct a new 42-inch gravity sewer in Pecan Lane (Avenue 12 ½) from Madera Avenue (Highway 145/Road 27) to Granada Drive (Road 25).

- Improvement P-29: Construct a new 42-inch gravity sewer in Madera Avenue (Highway 145/Road 27) from 1,110 feet south of Pecan Lane (Avenue 12 ½) to Pecan Lane (Avenue 12 ½).

- Improvement P-30: Construct a new 42-inch gravity sewer in Borden Street, City ROW, and Burges from Pecan Lane (Avenue 12 ½) to Madera Avenue (Highway 145/Road 27).

- Improvement P-31: Construct a new 42-inch gravity sewer and 66-inch casing in the City ROW near Borden Street for the Golden State Highway (State Highway 99) and railroad crossing.

7.3.4 Long-Term Future Capacity Improvements

The following have been identified as long-term improvements for the fiscal years 2021-2030 based on the general growth of the City. These improvements will add capacity along the Pecan Trunk to the wastewater treatment plant for all of the future flows.

The following improvements shown on Figure 7.6 are considered long-term improvements:

Pecan Basin – Pecan Avenue Trunk Improvements:

- Improvement P-4: Parallel the existing 42-inch gravity sewer in Pecan Avenue (Avenue 13) from Granada (Road 25) to Westberry Bolevard (Road 24 ½) with a new 36-inch pipe.

- Improvement P-5: Parallel the existing 42-inch gravity sewer in Pecan Avenue (Avenue 13) from Westberry Boulevard (Road 24 ½) to Road 24 with a new 42-inch pipe.

- Improvement P-6: Parallel the existing 42-inch gravity sewer in Pecan Avenue (Avenue 13) from Road 24 to Road 23 ½ with a new 48-inch pipe.

- Improvement P-7: Parallel the existing 48-inch gravity sewer in Pecan Avenue (Avenue 13) from Road 24 to the Madera Wastewater Treatment Plant Headwork's with a new 48-inch pipe.

7.3.5 Other Long-Term and Extended Range Future Capacity Improvements

The future sanitary sewer system for the buildout flows extends to the Urban Growth Boundary to service the anticipated General Plan growth. These remaining improvements are labeled as FY 2016-2050 as it is difficult to impossible to guess which direction the City will develop. These improvements will be constructed as new developments necessitate them.

The following improvements shown on Figure 7.4 - 7.7 are considered other long-term and extended range improvements:

Pecan Basin – Road 28 ½ Trunk Improvements:

- Improvement P-9: Construct a new 12-inch gravity sewer in Road 29 ½ from Almond Avenue (Avenue 13 ½) to Pecan Avenue (Avenue 13).

- Improvement P-10: Construct a new 36-inch gravity sewer in Raymond Road (Road 28 ½) from Olive Avenue (Avenue 14) to Pecan Avenue (Avenue 13) that parallels the 8 and 10-inch gravity sewer.
- Improvement P-11: Construct a new 18-inch gravity sewer in Olive Avenue (Avenue 14) from Road 30 to Raymond Road (Road 28 ½).
- Improvement P-12: Construct a new 18-inch gravity sewer and a 42-inch casing in Olive Avenue (Avenue 14) for the Main Canal crossing.
- Improvement P-13: Construct a new 15-inch gravity sewer in Road 30 from Almond Avenue (Avenue 13 ½) extension to Olive Avenue (Avenue 14).
- Improvement P-14: Construct a new 15-inch gravity sewer in the Santa Fe Railroad right of way (ROW) from Sunrise Avenue (Avenue 14 ½) extension to Olive Avenue (Avenue 14).
- Improvement P-15: Construct a new 30-inch gravity sewer in Raymond Road (Road 28 ½) from Sunrise Avenue (Avenue 14 ½) to Olive Avenue (Avenue 14).
- Improvement P-16: Construct a new 15-inch gravity sewer in Raymond Road (Road 28 ½) from Storey Road (Avenue 15 ½) to Sunrise Avenue (Avenue 14 ½).
- Improvement P-17: Construct a new 27-inch gravity sewer in Sunrise Avenue (Avenue 14 ½) from Road 29 ¼ extension to Raymond Road (Road 28 ½).
- Improvement P-18: Construct a new 27-inch gravity sewer and a 54-inch casing in Road 29 ¼ for the Main Canal crossing.
- Improvement P-19: Construct a new 27-inch gravity sewer in Road 29 ¼ extension from Avenue 15 to Sunrise Avenue (Avenue 14 ½).
- Improvement P-20: Construct a new 27-inch gravity sewer in Avenue 15 from Avenue 29 to Road 29 ¼ extension.
- Improvement P-21: Construct a new 27-inch gravity sewer in Road 29 from Storey Road (Avenue 15 ½) to Avenue 15.
- Improvement P-22: Construct a new 24-inch gravity sewer in Storey Road (Avenue 15 ½) from Santa Fe Drive to Road 29.
- Improvement P-23: Construct a new 24-inch gravity sewer with a 48-inch casing in Storey Road (Avenue 15 ½) for the railroad crossing.
- Improvement P-24: Construct a new 24-inch gravity sewer in Road 29 from 660 feet north of Storey Road (Avenue 15 ½) to Storey Road (Avenue 15 ½).

- Improvement P-25: Construct a new 21-inch gravity sewer in Road 29 from Yosemite Avenue (Highway 145) to 660 feet north of Storey Road (Avenue 15 ½).
- Improvement P-26: Construct a new 21-inch gravity sewer in Yosemite Avenue (Highway 145) from Juanita Drive to Road 29.

Pecan Basin – Avenue 12 ½ Trunk Improvements:

- Improvement P-32: Construct a new 30-inch gravity sewer in Pecan Lane (Avenue 12 ½) from Road 29 to Borden Street extension.
- Improvement P-33: Construct a new 12-inch gravity sewer in Pecan Lane (Avenue 12 ½) from Road 29 ½ extension to Road 29.
- Improvement P-34: Construct a new 30-inch gravity sewer in Road 29 from 680 feet north of Avenue 12 to Pecan Lane (Avenue 12 ½).
- Improvement P-35: Construct a new 24-inch gravity sewer with a 48-inch casing in the City ROW for the creek and canal crossing near Avenue 12 and Road 29.
- Improvement P-36: Construct a new 24-inch gravity sewer in the City ROW near Avenue 12 and Road 29 from 900 feet east of Road 29 to 675 feet northwest of Avenue 12.
- Improvement P-37: Construct a new 24-inch gravity sewer in Avenue 12 from Road 30 ½ to 900 feet east of Road 29.
- Improvement P-38: Construct a new 12-inch gravity sewer in Avenue 12 ½ extension and Road 29 ½ extension from 660 feet east of Road 30 then southeast to Avenue 12.
- Improvement P-39: Construct a new 21-inch gravity sewer in Avenue 12 from 3,990 feet east of Road 30 ½ to Road 30 ½.
- Improvement P-40: Construct a new 15-inch gravity sewer in Road 30 ½ from Pecan Avenue (Avenue 13) to Avenue 12.
- Improvement P-41: Construct a new 15-inch gravity sewer in Avenue 12 ½ extension from 2,630 feet east of Road 30 ½ to Road 30 ½.

Schnoor Basin – Sherwood Trunk Improvements:

- Improvement S-1: Replace the existing 10-inch gravity sewer with a new 15-inch gravity sewer in Wessmith Way from 190 feet east of Lake Street (Road 27) to Lake Street (Road 27).
- Improvement S-2: Replace the existing 12-inch gravity sewer with a new 15-inch gravity sewer in Sherwood Way from Lake Street (Road 27) to 220 feet west of Nebraska Avenue.

- Improvement S-3: Construct a new 10-inch gravity sewer in Kennedy Street (Avenue 16) from Road 28 to Chapin Street.
- Improvement S-4: Construct a new 10-inch gravity sewer in Road 28 from 680 feet north of Kennedy Street (Avenue 16) to Kennedy Street (Avenue 16).

Schnoor Basin – Schnoor Trunk Improvements:

- Improvement S-6: Upgrade the existing Fairgrounds Lift Station to a firm capacity of 2,500 gpm with variable speed motors.

Westberry Basin – Ellis Street Trunk Improvements:

- Improvement W-2: Parallel the existing 12-inch gravity sewer in Kennedy Street (Avenue 16) from Road 25 to Condor Drive with a new 30-inch pipe.
- Improvement W-3: Construct a new 24-inch gravity sewer in Ellis Street from Krohn Street to Sharon Boulevard.
- Improvement W-4: Construct a new 21-inch gravity sewer in Ellis Street from D Street to Krohn Street.
- Improvement W-5: Construct a new 21-inch gravity sewer and 48-inch casing in Ellis Street for the canal crossing.
- Improvement W-6: Construct a new 21-inch gravity sewer in D Street from Martin Street to Ellis Street.
- Improvement W-7: Construct a new 12-inch gravity sewer in Lake Street (Road 27) from 2,120 feet north of Martin Street to Martin Street.
- Improvement W-8: Construct a new 21-inch gravity sewer in Martin Street extension from 3,330 feet east of Lake Street (Road 27) to D Street.
- Improvement W-9: Construct a new 21-inch gravity sewer in a future City road or ROW (3,330 feet east of Lake Street (Road 26)) from Avenue 17 to Martin Street extension.
- Improvement W-10: Construct a new 21-inch gravity sewer and 48-inch casing in Avenue 17 for the railroad crossing.
- Improvement W-11: Construct a new 21-inch gravity sewer in Avenue 17 from 425 feet west of Harper Boulevard at the railroad tracks (future Road 27+ ¾) to 560 feet west of the railroad tracks.
- Improvement W-12: Construct a new 15-inch gravity sewer in future Road 27 ¾ from Avenue 17 1/2 extension to Avenue 17.

- Improvement W-13: Construct a new 12-inch gravity sewer in Avenue 17 ½ extension from Tuolumne Street extension to future Road 27 ¾.
- Improvement W-14: Construct a new 15-inch gravity sewer in Avenue 17 from Harper Boulevard to 425 feet west of Harper Boulevard at the railroad tracks (future Road 27 ¾).
- Improvement W-15: Construct a new 10-inch gravity sewer in Avenue 17 from 2,370 feet east of Harper Boulevard to Harper Boulevard.
- Improvement W-16: Construct a new 15-inch gravity sewer in Harper Boulevard from Raymond Road (Road 28 ½) to Avenue 17.
- Improvement W-17: Construct a new 12-inch gravity sewer in a future road or City ROW from Arizona Avenue southwest to Raymond Road (Road 28 ½).

Westberry Basin – Road 23 Trunk Improvements:

- Improvement W-18: Construct a new 30-inch gravity sewer in Road 23 from Avenue 16 to Pecan Avenue (Avenue 13).
- Improvement W-19: Construct a new 30-inch gravity sewer and 54-inch casing in Road 23 crossing the Fresno River.
- Improvement W-20: Construct a 2,300 gpm firm capacity lift station at Avenue 16 and Road 23.
- Improvement W-21: Construct a new 18-inch gravity sewer in Avenue 16 from Road 22 ½ to Road 23.
- Improvement W-22: Construct a new 15-inch gravity sewer in Road 23 from Avenue 18 to Avenue 16.
- Improvement W-23: Construct a new 12-inch gravity sewer and 36-inch casing in Avenue 18 crossing Golden State Highway (State Highway 99) and the railroad tracks.
- Improvement W-24: Construct a new 12-inch gravity sewer in Avenue 18 from Sharon Boulevard (Road 24) to Road 23.
- Improvement W-25: Construct a new 12-inch gravity sewer in Road 23 ½ from Almond Avenue (Avenue 13 ½) to Pecan Avenue (Avenue 13).
- Improvement W-26: Construct a new 12-inch gravity sewer in Howard Road (Avenue 14) from Road 23 ½ to Road 23
- Improvement W-27: Construct a new 12-inch gravity sewer in Sunset Avenue (Avenue 14 ½) from Road 23 ½ to Road 23.

- Improvement W-28: Construct a new 15-inch gravity sewer in Cleveland Avenue (Avenue 15 ½) from Road 24 to Road 23.

7.3.6 Buildout Peak Dry Weather Flow Critical Areas

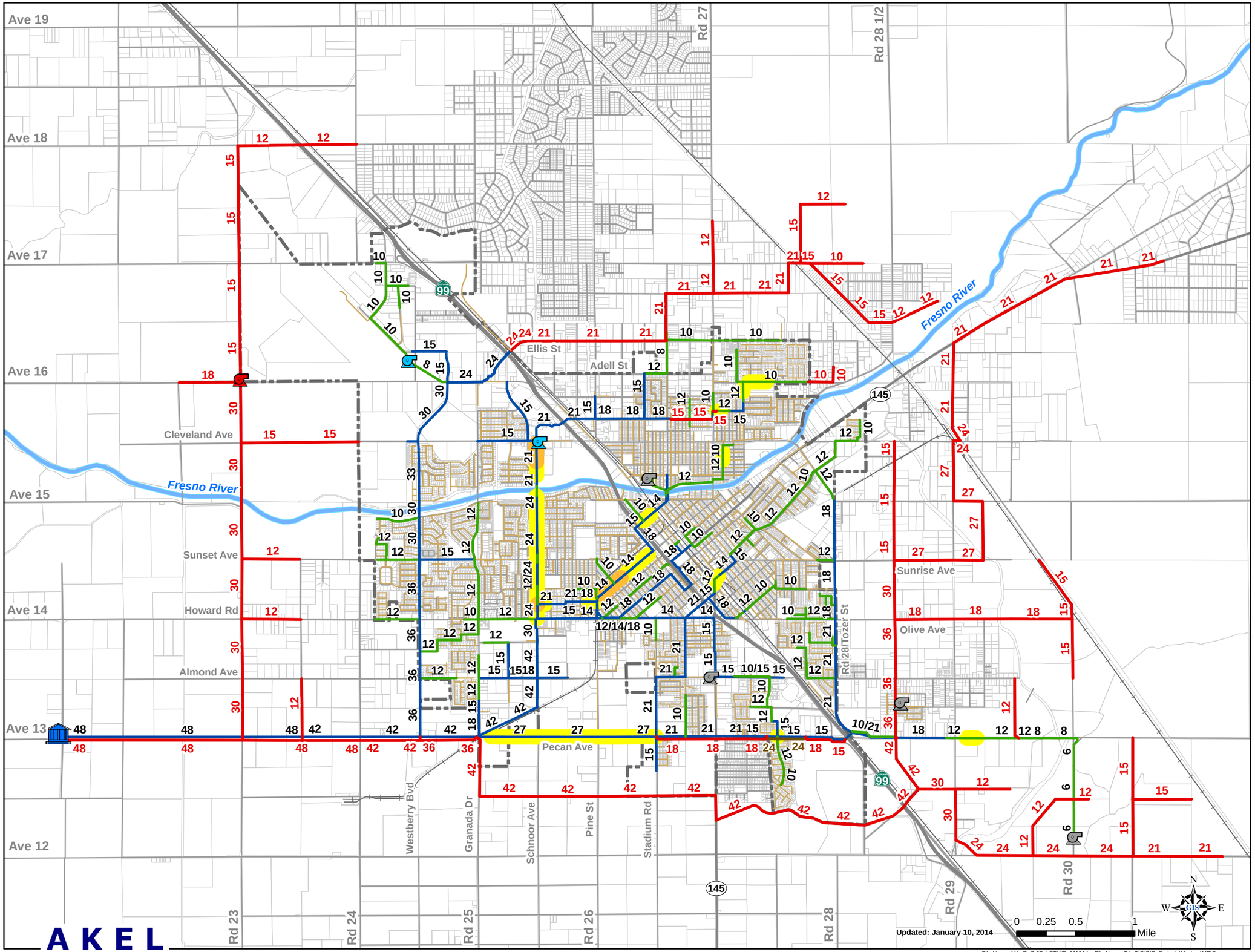
The hydraulic model identifies several trunk reaches that approach the maximum capacity criteria during future peak dry weather flows. While Figure 7.9, shows the pipe reaches that approach the maximum allowable criteria for capacity (d/D Ratio of 0.92), this section discusses the most critical ones.

- Schnoor Avenue (Road 25 ½), from Fairground Lift Station to Dutra Way. This segment experiences d/D ratios between 0.75 and 0.92.
- Schnoor Avenue (Road 25 ½), from Riverview Drive to Howard Road (Avenue 14). This segment experiences d/D ratios between 0.75 and 0.92.
- Fourth Street, from Rotan Avenue to I Street. This segment experiences d/D ratios between 0.75 and 0.92.
- Pecan Avenue (Avenue 13), Stadium Road (Road 26 ½) to Granada Drive (Road 25). This segment experiences d/D ratios between 0.75 and 0.92.

7.3.7 Buildout Peak Wet Weather Flow Critical Areas

The hydraulic model indicates that the existing sanitary sewer system will exhibit some surcharging approaching maximum allowable criteria during a 10yr-24hr storm event with future flows. While Figure 7.10, shows the pipe reaches that approach the maximum allowable criteria for capacity (3 feet below manhole rim), this section discusses the most critical ones.

- Schnoor Avenue (Road 25 ½), from Fairground Lift Station to Howard Road. This segment experiences surcharging.
- Fourth Street, from Schnoor Avenue (Road 25 ½) to I Street. This segment experiences surcharging.
- I Street, from Fourth Street to First Street. This segment experiences surcharging.
- First Street, from I Street to Central Avenue. This segment experiences surcharging.
- Madera Avenue/Ninth Street/Clinton Street, from Tenth Street to Lake Street (Road 25). This segment experiences surcharging.
- Pecan Avenue (Avenue 13), from Conrad Street to Granada Drive (Road 25). This segment experiences surcharging.



Legend

Analysis Results

Manholes

- HGL within 3 ft of Ground Elevation

Pipe d/D

- d/D ≥ 0.9
- d/D ≥ 0.75

Future Improvements

- Lift Station
- Pipes

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes by Size

- 12" and Smaller
- 14" and Larger
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 7.9
Proposed Future System PDWF
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

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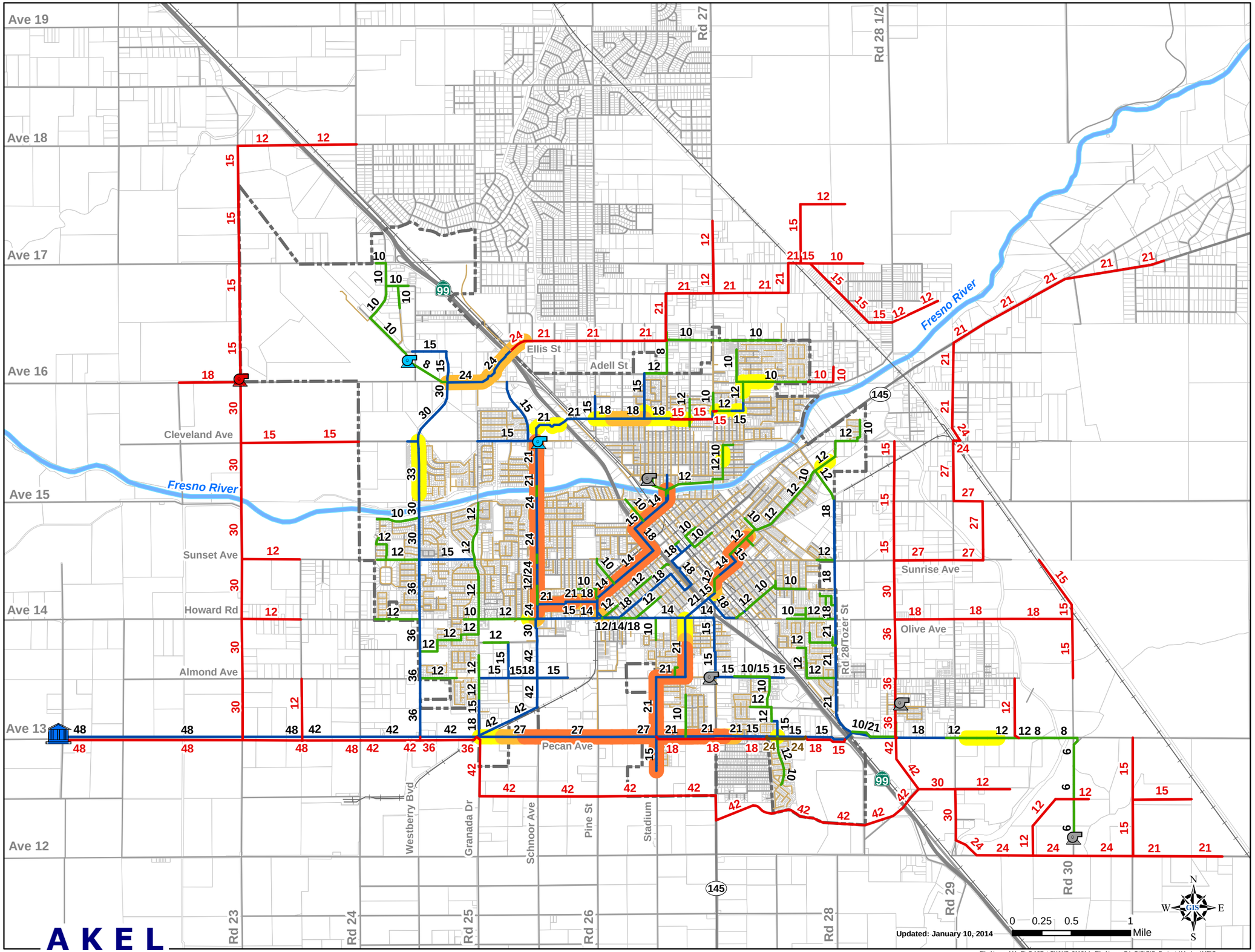


Legend

Analysis Results

Manholes

- HGL within 3 ft of Ground Elevation

Pipe d/D

- d/D ≥ 1.0
- d/D ≥ 0.9
- d/D ≥ 0.75

Future Improvements

- Lift Station
- Pipes

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations

Modeled Pipes by Size

- 12" and Smaller
- 14" and Larger
- Existing Pipe Not In Use
- Non-Modeled Pipes
- Roads
- Highways
- Railroads
- Parcels
- City Limits
- Fresno River

Figure 7.10
Proposed Future System PWWF

Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

- Stadium Road (Road 26 ½)/Almond Avenue (Avenue 13 ½)/Monterey Street, from Olive Avenue (Avenue 14) to Pecan Avenue (Avenue 13). This segment experiences surcharging.
- Sherwood Way, from Nebraska Avenue to Country Club Drive (Road 26). This segment experiences d/D ratios between 0.75 and 0.92.

7.4 LIFT STATIONS AND FORCE MAINS CAPACITY EVALUATION

The lift stations evaluated in this master plan include the Fairgrounds Lift Station and the Airport Lift Station.

7.4.1 Fairgrounds Lift Station

The 1997 Master Plan documented the lift stations pump capacities at 1,060 gpm each. Using the 1997 Master Plan capacities, the firm capacity of the lift station is 2,120 gpm (two duty pumps with one standby pump) and a total capacity of 3,180 gpm (all three pumps). During the analysis of the Fairgrounds Lift Station, City staff indicated that the actual flow output was less than the rated capacity and occasionally all three pumps would be in operation.

Supervisory control and data acquisition (SCADA) was received from City staff for the lift station and used to analyze the lift stations existing capacity. The analysis showed that all three pumps do occasionally operate at the same time but operate at a lower capacity than the 1997 Master Plan documented. **Appendix C** includes the analysis of the SCADA showing the number of pumps and the total flow from the lift station and the Schnoor Sewer Trunk profiles using the 1997 Master Plan pump capacities. The analysis concluded the lift station does not operate at 1,060 gpm per pump but closer to 500 gpm per pump. The City did some additional review and took further measurements to determine the existing capacities for the Fairgrounds lift station:

- Pump 1 = 540 gpm (speed at 85%) and 875 gpm (speed at 100%)
- Pump 2 = 580 gpm (speed at 85%) and 820 gpm (speed at 100%)
- Pump 3 = 592 gpm (speed at 85%) and 957 gpm (speed at 100%)
- Pump 1, 2 = 1,600 gpm (speed at 100%)
- Pump 1, 3 = 1,625 gpm (speed at 100%)
- Pump 2, 3 = 1,653 gpm (speed at 100%)
- Pump 1, 2, 3 = 2,551 gpm (speed at 100%)

Thus, for the capacity evaluation and hydraulic modeling of the Fairgrounds lift station, a firm capacity of 1,653 gpm and a total capacity of 2,551 gpm was used.

Hydraulic analysis indicates that the highest flow experienced at this lift station during existing conditions is approximately 1,300 gallons per minute (gpm) during peak dry weather flow and 1,900 gpm during peak wet weather flow. The existing pump station has a firm capacity of 1,653 gpm (two pumps running with one pump for standby) and a total capacity of 2,551 gpm (all three pumps running). The pump station has adequate firm capacity for the peak dry weather flows but is considered deficient for peak wet weather flows. The total capacity for the pump station is adequate but does not provide a provision for a standby pump in case of a pump failure.

The Fairgrounds Lift Station analysis is shown on **Table 7.2** and lists the existing and future flows for the lift station and the firm and total capacity for the lift station. This analysis recommends that the Fairgrounds Lift Station be upgraded to a firm capacity of 2,500 gpm with variable speed pumps (Improvement Number S-6). This pump station upgrade will provide the required firm capacity for existing and future buildout flows.

The highest velocity experienced by the 12-inch Fairgrounds force main is 4.7 feet per second (fps) for the existing lift stations firm capacity, and 7.1 fps for the upgraded lift station firm capacity (**Table 7.3**).

7.4.2 Airport Lift Station

Hydraulic analysis indicates that the highest flow experienced at this lift station during existing conditions is approximately 185 gpm during peak dry weather flow and 200 gpm during peak wet weather flow. The existing pump station has a firm capacity of 820 gpm (two pumps running with one pump for standby) and a total capacity of 1,230 gpm (all three pumps running). The pump station has enough firm capacity for the peak dry weather flows and peak wet weather flows.

The Airport Lift Station analysis is shown on **Table 7.4** and lists the existing and future flows for the lift station and the firm and total capacity for the lift station. This analysis shows that the Airport Lift Station has an adequate capacity for existing and future buildout flows.

The velocities experienced by the 8-inch Fairgrounds force main is 2.6 fps for one pump operating (410 gpm) and 5.2 fps with two pumps operating (820 gpm) (**Table 7.3**).

7.5 SCHNOOR AVENUE TRUNK SEWER REPLACEMENT/ REHABILITATION EVALUATION

A condition assessment of the Schnoor Avenue Sewer Trunk was completed for this Master Plan by MWH Americas, Inc. (MWH) and included in **Appendix D**. The condition assessment included a review of the following:

- 2007 CCTV
- Previous Condition Assessment (Schnoor Avenue Trunk Sewer Evaluation, by Blair Church and Flynn, May 2008)

Table 7.2 Capacity Evaluation for Lift Station PS-1 (Fairgrounds)

Sanitary Sewer System Master Plan

City of Madera

	Design Flows (gpm)	Pump Capacity: Firm	Pump Capacity: Total	Comments
Existing Pump Station Capacity				
3 Existing Pumps (capacities vary[1])		**1,653 gpm** Includes Pumps 2+3 (Pump 1 as standby)	**2,551 gpm** Pumps 1+2+3	Capacity of Existing 24-inch Sewer Gravity Trunk on Schnoor: Pump discharge at 2,551 gpm during existing Peak Wet Weather Flow will result with minor surcharging.
Capacity Analysis				
Existing Flows				
Peak Dry Weather Flow	1,300	Adequate	Adequate	
Peak Wet Weather Flow	1,900	Deficient	Adequate	
Future Flows (Buildout)				
Peak Dry Weather Flow	2,000	Deficient	Adequate	
Peak Wet Weather Flow	2,500	Deficient	Adequate	
Recommended Pump Station Improvements				
3 New Pumps at 1,250 gpm		**2,500 gpm** Includes Pumps 1+2 (Pump 3 as standby)	**3,750 gpm** Pumps 1+2+3 (running all three new pumps will cause deficiency in 24-inch on Schnoor)	Capacity of Existing 24-inch Sewer Gravity Trunk on Schnoor: Pump discharge during future Peak Dry Weather Flow or future Peak Wet Weather Flow over 2,500 gpm will result with excessive surcharging and pipe deficiencies.

Notes:

6/3/2013

1. Existing pump capacities, as provided by City staff on 12/7/12
   - Pump 1 = 540 gpm (speed at 85%) and 875 gpm (speed at 100%)
   - Pump 2 = 580 gpm (speed at 85%) and 820 gpm (speed at 100%)
   - Pump 3 = 592 gpm (speed at 85%) and 957 gpm (speed at 100%)
   - Pumps 1,2 = 1,600 gpm (speed at 100%)
   - Pumps 1,3 = 1,625 gpm (speed at 100%)
   - Pumps 2,3 = 1,653 gpm (speed at 100%)
   - Pumps 1,2,3 = 2,551 gpm (speed at 100%)

Table 7.3 Force Mains Analysis

Sanitary Sewer System Master Plan
City of Madera

Flow Conditions	Lift Station Firm Capacity (gpm)	Force Main Diameter (in)	Force Main Length (ft)	Force Main Velocity (ft/s)
Fairgrounds				
Existing	1,653	12	60	4.7
Ultimate Buildout	2,500			7.1
Airport				
Existing	820	8	2,100	2.6 (410 gpm)
Ultimate Buildout				5.2 (820 gpm)

3/26/2013

Table 7.4 Capacity Evaluation for the Airport Lift Station

Sanitary Sewer System Master Plan

City of Madera

	Design Flows (gpm)	Pump Capacity Firm	Pump Capacity Total
Existing Pump Station Capacity			
Existing Pumps		**820 gpm**	**1,230 gpm**
Capacity Analysis			
Existing Flows			
Peak Dry Weather Flow	185	Adequate	Adequate
Peak Wet Weather Flow	200	Adequate	Adequate
Future Flows (Buildout)			
Peak Dry Weather Flow	450	Adequate	Adequate
Peak Wet Weather Flow	470	Adequate	Adequate

6/14/2013

- This master plan's capacity evaluation of the Schnoor Trunk

A condition evaluation and rehabilitation methodology was included which summarize the rehabilitation techniques used to extend the useful life of a sewer trunk system. The review of the 2007 CCTV of the Schnoor Avenue Trunk found that the vitrified clay pipes (VCP) were in good condition with some root intrusion and a few longitudinal cracks near joints. The concrete pipes, however were found to be in severe conditions with varying degrees of exposed or missing aggregate. The condition assessment recommended that 505 feet of pipe have roots removed, 145 feet of pipes be cleaned, and 2,435 feet of pipe have a cured in place pipe (CIPP) liner installed. The detailed condition assessment with pipe segments, descriptions, and a cost estimate are included in the evaluation located in **Appendix D**.

7.6 WASTEWATER TREATMENT PLANT EVALUATION

Previous wastewater treatment plant evaluations were completed in 1998 and 1994. Since the 1994 evaluation, several treatment plant upgrades were completed and resulted with a current daily flow capacity of up to 10.1 MGD.

This master plan also included a wastewater treatment plant capacity evaluation to identify the impact of future sewer flows from the ultimate buildout of the General Plan. This wastewater treatment plant evaluation (2012 WWTP evaluation) was completed in 2012 by MWH Americas, Inc. (**Appendix E**), and provides the City with a road map in planning WWTP improvements for the near-term and ultimate buildout wastewater flows.

This 2012 WWTP evaluation report includes a description of the wastewater treatment plant processes, waste discharge requirements, near-term improvements, and alternatives for ultimate developments within the City Planning Area.

This existing treatment plant was evaluated and the following near-term capital improvements were recommended (**Table 7.5**).

- Preliminary treatment improvements consisting of the additional mechanical bar screen, the influent lift station capacity, and the primary effluent pump station capacity.
- New solids handling improvements consisting of a new sludge thickening system and an additional anaerobic digester.

The evaluation included future regulations, treatment technologies, regulatory requirements, and viable technologies. Alternatives for a future centralized or decentralized WWTP were also evaluated, and though the satellite plants introduce significant advantages for reusing the treated water, they also proved to be cost prohibitive. For this reason, the 2012 WWTP evaluation recommended the continued expansion of the existing plant with a Membrane Bioreactor (MBR) upgrade for the buildout flows, as summarized on **Table 7.6**. This table lists the number of existing treatment units and the number of additional units that are required to treat the buildout flows.

Table 7.5 Recommended WWTP Capital Improvement Projects - Near-Term

Sanitary Sewer System Master Plan

City of Madera

Unclassified Sludge		Class B Biosolids	
Year Improvement Needed	**Recommended Capital Improvement Projects**	**Year Improvement Needed**	**Recommended Capital Improvement Projects**
Near-Term	Preliminary Treatment Improvements · Mechanical Bar Screen Addition · Influent Lift Station Expansion · Primary Effluent Pump Station Expansion	**Near-Term**	Preliminary Treatment Improvements · Mechanical Bar Screen Addition · Influent Lift Station Expansion · Primary Effluent Pump Station Expansion
Y2013-Y2017	Solids Handling Improvements without WAS Co-Settling · Sludge Thickener	**Y2013-Y2020**	Solids Handling Improvements without WAS Co-Settling · Sludge Thickener · 1 – 45’ diameter Anaerobic Digesters
Review By	**Potential Capital Improvement Projects**	**Review By**	**Potential Capital Improvement Projects**
Y2017	Oxidation Ditch Improvements	**Y2017**	Oxidation Ditch Improvements
Y2019	Evaporation / Percolation Ponds	**Y2019**	Evaporation / Percolation Ponds

Note:

1. Table extracted from MWH Wastewater Treatment Plant Evaluation report dated 2013

9/23/2013

Table 7.6 Recommended Wastewater Treatment Plant Expansion with MBR

Sanitary Sewer System Master Plan
City of Madera

	Existing Number of Units	WWTP expansion for Urban Growth Boundary	
		Additional Units[2]	Total
Headwork's	1	1	2
Primary Treatment			
Primary Clarifiers	3	3	6
Primary Effluent Pump Station	1	1	2
Secondary Treatment			
Oxidation Ditches	3	0	3
Secondary Clarifiers	4	0	4
Secondary/Tertiary Treatment			
MBR Trains[3]	0	4	4
Effluent Disposal			
Evaporation / Percolation Ponds	16	20	36
Potential Tertiary Filters[4]	0	4	4
Solids Handling			
Sludge Thickening Units	2	2	4
Anaerobic Digesters[5]	4	2[6]	6
Dewatering Centrifuges	2	2	4

Notes: 10/8/2013

1. Table extracted from MWH Wastewater Treatment Plant Evaluation report dated 2013
2. Additional facilities are assumed to be same size as existing unless otherwise noted.
3. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.
4. Space allocation for potential filters for tertiary treatment of 10.1 mgd oxidation ditch effluent (e.g. Title 22 recycled water)
5. Assumes largest digester unit is out of service
6. Additional digesters are assumed to be 70-feet diameter

CHAPTER 8 - CAPITAL IMPROVEMENT PROGRAM

This chapter provides a summary of the recommended Capital Improvement Program (CIP) for the City of Madera'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.

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.

The unit costs include:

- **Pipeline Unit Costs.** In the process of developing unit costs for the City of Madera, public and private cost of construction was taken into account. Public costs are projects completed by the City of Madera, and private costs are associated with developer costs. Private costs are based on recent bid tab information and were provided by City staff.

  Mixed unit costs were developed based on the public versus private costs and percent responsibilities provided by City staff. These costs vary by pipe sizes (up to 48 inches) and are based on the length of pipes, in feet.

- **Pump Station Costs.** These costs are based on a lift station project equation, and were adjusted to reflect the current ENR CCI.

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 Storm Drainage System Master Plan were benchmarked using a 20-City national average ENR CCI of 9545, reflecting a date of August 2013.

Table 8.1 Capacity Improvement Unit Costs

Sanitary Sewer System Master Plan

City of Madera

Pipe Size (in)	Public Costs[1,2] ($/Lineal Foot)	Private Costs[1,3] ($/Lineal Foot)	Mixed Public/Private Costs[4] Percent Private Cost Responsibility[5]	Mixed Public/Private Costs[4] ($/Lineal Foot)
Gravity Pipes				
10	**$135**	**$42**	75%	**$65**
12	**$140**	**$45**	50%	**$93**
15	**$160**	**$47**	25%	**$132**
18	**$175**	**$53**	25%	**$145**
21	**$193**	**$60**	10%	**$180**
24	**$218**	**$66**	0%	**$218**
27	**$279**	**$74**	0%	**$279**
30	**$310**	**$79**	0%	**$310**
36	**$372**	**$95**	0%	**$372**
42	**$443**	**$111**	0%	**$443**
48	**$506**	**$126**	0%	**$506**
Casings				
Estimated casing costs are based on $20 per inch diameter per linear foot				
Lift Stations				
Estimated Lift Station Project Cost = $7943Q^2+221869Q+300541$, where Q is in mgd				

6/11/2013

Notes:

1. Public costs are associated with construction projects completed by the City of Madera, while Private costs are associated with construction projects completed by developers.
2. Public Unit Costs were escalated from the 1997 Master Plan (ENR 20-City Average CCI 5726), and reflect an ENR 20-City Average CCI of 9545 (August 2013).
3. Private Unit Costs were provided by City of Madera and are based on recent construction bid tabs
4. Mixed Costs reflect the weighted Unit Costs, based on applying the percentages for Public and Private
5. Percentage of Cost attributed to Developers (Private)

8.2.3 Land Acquisition

Construction of pipelines is assumed to generally be within existing or future street right-of-ways. The construction of a new lift station was assumed to require 0.5 acres and a land acquisition fee was assumed based on recent land acquisitions.

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 **20 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 **25 percent** to the estimated construction costs.

8.3 CAPITAL IMPROVEMENT PROGRAM

The Capacity Improvement Costs for the previously identified projects are summarized on **Table 8.2**. The Capital Improvement Program lists the type of improvement, location, cost, construction trigger, suggested phasing, village/district, 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**.

Table 8.2 Capital Improvement Program
Sanitary Sewer System Master Plan
City of Madera

Itemized Cost Estimate														Construction Trigger	Suggested Phasing	Village/District	Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]				% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost								Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)						($)	($)
Pecan Basin																				
Pecan Ave Trunk																				
P-1	Pipe	Pecan Avenue	From w/o Highway 99 to 400 ft w/o Golden State Blvd	15	Parallel	15	1,025	132	135,044	135,044	162,053		202,566	1,045 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	202,566
P-1A	Pipe	Pecan Avenue	From 400ft w/o Golden State to 1,360 ft e/o Garnet Ave	15	Parallel	18	775	145	111,988	111,988	134,385		167,981	1,045 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	167,981
P-2	Pipe	Pecan Avenue	From 460ft w/o Garnet Ave to 40 ft w/o Raymond Thomas St	15	Parallel	18	920	145	132,940	132,940	159,528		199,410	1,550 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	199,410
P-3	Pipe	Pecan Avenue	From 40 ft w/o Raymond Thomas St to Stadium Rd (Road 26 1/2)	21	Parallel	18	6,230	145	900,235	900,235	1,080,282		1,350,353	3,710 EDUs	FY 2013 - 2015	Parkwood	0%	100%	0	1,350,353
P-4	Pipe	Pecan Avenue	From Granada Dr (Road 25) to Westberry Blvd (Road 24 1/2)	42	Parallel	36	2,670	372	993,240	993,240	1,191,888		1,489,860	22,320 EDUs	FY 2026 - 2030	N/A	0%	100%	0	1,489,860
P-5	Pipe	Pecan Avenue	From Westberry Blvd (Road 24 1/2) to Road 24	42	Parallel	42	2,740	443	1,213,820	1,213,820	1,456,584		1,820,730	33,675 EDUs	FY 2026 - 2030	N/A	0%	100%	0	1,820,730
P-6	Pipe	Pecan Avenue	From Road 24 to Road 23 1/2	42	Parallel	48	2,660	506	1,346,720	1,346,720	1,616,064		2,020,080	17,185 EDUs	FY 2026 - 2030	West Madera, N/A	0%	100%	0	2,020,080
P-7	Pipe	Pecan Avenue	From Road 24 to WWTP	48	Parallel	48	11,280	506	5,710,903	5,710,903	6,853,083		8,566,354	33,675 EDUs	FY 2026 - 2030	N/A	0%	100%	0	8,566,354
			Sub-Total				28,300		$10,544,889	$10,544,889	$12,653,867	**$0**	**$15,817,334**						**$0**	**$15,817,334**
Road 28 1/2 Trunk																				
P-8	Pipe	Road 28 1/2, Parallel to HWY 99	From Pecan to Avenue 12 1/2		New	42	2,560	443	1,134,080	1,134,080	1,360,896		1,701,120	5,570 EDUs or when Ave 12 1/2 trunk is constructed (IMP No. P-26 to P-32)	FY 2016 - 2020	Community College	0%	100%	0	1,701,120
P-9	Pipe	Road 29 1/2	From Almond Ave (Ave 13 1/2) to Pecan Avenue (Ave 13)		New	12	2,750	93	254,375	254,375	305,250		381,563	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	381,563
P-10	Pipe	Road 28 1/2	From Ave 14 to Pecan Ave (Ave 13)	8/10	New/Parallel	36	5,340	372	1,986,480	1,986,480	2,383,776		2,979,720	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	2,979,720
P-11	Pipe	Avenue 14	Road 30 to Road 28 1/2		New	18	7,740	145	1,118,430	1,118,430	1,342,116		1,677,645	Construct with development in Area	FY 2016 - 2050	Parksdale, N/A	0%	100%	0	1,677,645
P-12	Pipe/Casing[5]	Avenue 14	Crossing the Main Canal		New	18/42	150	840	126,000	126,000	151,200		189,000	Construct with development in Area	FY 2016 - 2050	Parksdale, N/A	0%	100%	0	189,000
P-13	Pipe	Road 30	From Ave 13 1/2 ext to Ave 14		New	15	2,640	132	347,820	347,820	417,384		521,730	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	521,730
P-14	Pipe	Santa Fe Railroad ROW	From Ave 14 1/2 ext to Ave 14		New	15	3,130	132	412,378	412,378	494,853		618,566	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	618,566
P-15	Pipe	Road 28 1/2	From Ave 14 1/2 to Ave 14		New	30	2,640	310	818,400	818,400	982,080		1,227,600	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,227,600
P-16	Pipe	Road 28 1/2	From Ave 15 1/2 to Ave 14 1/2		New	15	5,320	132	700,910	700,910	841,092		1,051,365	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,051,365
P-17	Pipe	Avenue 14 1/2	Road 29 1/4 ext to Road 28 1/2		New	27	3,990	279	1,113,210	1,113,210	1,335,852		1,669,815	Construct with development in Area	FY 2016 - 2050	Parksdale	0%	100%	0	1,669,815
P-18	Pipe/Casing[5]	Road 29 1/4 extension	Crossing the Main Canal		New	27/54	150	1,080	162,000	162,000	194,400		243,000	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	243,000
P-19	Pipe	Road 29 1/4 extension	Ave 15 to Ave 14 1/2		New	27	2,210	279	616,590	616,590	739,908		924,885	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	924,885
P-20	Pipe	Avenue 15	Ave 29 to Road 29 1/4 ext		New	27	1,290	279	359,910	359,910	431,892		539,865	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	539,865
P-21	Pipe	Road 29	From Ave 15 1/2 to Ave 15		New	27	2,700	279	753,300	753,300	903,960		1,129,950	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	1,129,950
P-22	Pipe	Avenue 15 1/2	From Santa Fe Dr to Road 29		New	24	160	218	34,880	34,880	41,856		52,320	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	52,320
P-23	Pipe/Casing[5]	Avenue 15 1/2	Railroad Crossing		New	24/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	216,000
P-24	Pipe	Road 29	From 660ft n/o Ave 15 1/2 to Ave 15 1/2		New	24	660	218	143,880	143,880	172,656		215,820	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	215,820
P-25	Pipe	Road 29	From River Rd to 660ft n/o Ave 15 1/2		New	21	3,810	180	684,657	684,657	821,588		1,026,986	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	1,026,986
P-26	Pipe	Highway 145	From Juanita Dr to Road 29		New	21	10,240	180	1,840,128	1,840,128	2,208,154		2,760,192	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	2,760,192
			Sub-Total				57,630		$12,751,428	$12,751,428	$15,301,713	**$0**	**$19,127,141**						**$0**	**$19,127,141**
Ave 12 1/2 Trunk																				
P-27	Pipe	Road 25	From Ave 12 1/2 to Pecan Ave (Ave 13)		New	42	2,700	443	1,196,100	1,196,100	1,435,320		1,794,150	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/2	FY 2016 - 2020	N/A	0%	100%	0	1,794,150
P-27A	Pipe/Casing[5]	Road 25	Crossing Railroad Tracks		New	42/66	50	1,320	66,000	66,000	79,200		99,000	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/3	FY 2016 - 2020	N/A	0%	100%	0	99,000
P-28	Pipe	Avenue 12 1/2	From Highway 145 to Road 25		New	42	10,560	443	4,678,080	4,678,080	5,613,696		7,017,120	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/4	FY 2016 - 2020	Parkwood, N/A	0%	100%	0	7,017,120
P-29	Pipe	Highway 145	From 1,110 ft s/o Avenue 12 1/2 to Avenue 12 1/2		New	42	1,110	443	491,730	491,730	590,076		737,595	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/5	FY 2016 - 2020	Parkwood	0%	100%	0	737,595
P-30	Pipe	Borden St/City ROW/Burges	From Ave 12 1/2 to Highway 145		New	42	9,710	443	4,301,426	4,301,426	5,161,711		6,452,139	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/6	FY 2016 - 2020	Parkwood	0%	100%	0	6,452,139
P-31	Pipe/Casing[5]	City ROW near Borden St	Crossing Highway 99 and RR Tracks		New	42/66	300	1,320	396,000	396,000	475,200		594,000	With Development in Area, or 5,570 EDUs tributary to Ave 13 and Road 28 1/5	FY 2016 - 2020	Parkwood, Community College	0%	100%	0	594,000
P-32	Pipe	Avenue 12 1/2	From Road 29 to Borden St Ext		New	30	1,640	310	508,400	508,400	610,080		762,600	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	762,600
P-33	Pipe	Avenue 12 1/2	From Road 29 1/2 extension to Road 29		New	12	2,450	93	226,625	226,625	271,950		339,938	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	339,938
P-34	Pipe	Road 29	From 680 ft n/o Avenue 12 to Avenue 12 1/2		New	30	2,290	310	709,900	709,900	851,880		1,064,850	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,064,850

Table 8.2 Capital Improvement Program

Sanitary Sewer System Master Plan
City of Madera

Itemized Cost Estimate														Construction Trigger	Suggested Phasing	Village/District	Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]				% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost								Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)						($)	($)
P-35	Pipe/Casing[5]	City ROW near Ave 12 and Road 29	Crossing creek and canal		New	24/48	490	960	470,400	470,400	564,480		705,600	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	705,600
P-36	Pipe	City ROW near Ave 12 and Road 29	From 900 ft e/o Road 29 to 675 ft nw/o Ave 12		New	24	680	218	148,240	148,240	177,888		222,360	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	222,360
P-37	Pipe	Avenue 12	From Road 30 1/2 to 900 ft e/o Road 29		New	24	7,000	218	1,526,000	1,526,000	1,831,200		2,289,000	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	2,289,000
P-38	Pipe	Avenue 12 1/2 extension Road 29 1/2 Extension	From 660ft e/o Road 30 then SE to Ave 12		New	12	4,380	93	405,150	405,150	486,180		607,725	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	607,725
P-39	Pipe	Avenue 12	From 3,990 ft e/o Road 30 1/2 to Road 30 1/2		New	21	3,990	180	717,003	717,003	860,404		1,075,505	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,075,505
P-40	Pipe	Road 30 1/2	From Pecan Ave (Ave 13) to Ave 12		New	15	5,290	132	696,958	696,958	836,349		1,045,436	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	1,045,436
P-41	Pipe	Avenue 12 1/2 extension	From 2,630 ft e/o Road 30 1/2 to Road 30 1/2		New	15	2,630	132	346,503	346,503	415,803		519,754	Construct with development in Area	FY 2016 - 2050	Community College	0%	100%	0	519,754
			Sub-Total				55,270		$16,884,514	$16,884,514	$20,261,417	**$0**	**$25,326,771**						**$0**	**$25,326,771**
Schnoor Basin																				
Sherwood Way Trunk																				
S-1	Pipe	Wessmith Way	190ft e/o Lake St (Road 27) to Lake St (Road 27)	10	Replace	15	190	160	30,400	30,400	36,480		45,600	810 EDUs	FY 2016 - 2050	Central Madera	53%	47%	24,219	21,381
S-2	Pipe	Sherwood Way	Lake St (Road 27) to 220ft w/o Nebraska Ave	12	Replace	15	1,930	160	308,800	308,800	370,560		463,200	1,115 EDUs	FY 2016 - 2050	Central Madera	53%	47%	246,016	217,184
S-3	Pipe	Kennedy Street (Avenue 16)	Road 28 to Chapin St		New	10	1,120	65	73,080	73,080	87,696		109,620	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	109,620
S-4	Pipe	Road 28	680 ft n/o Kennedy St (Ave 16) to Kennedy St (Ave 16)		New	10	680	65	44,370	44,370	53,244		66,555	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	66,555
			Sub-Total				88,390		$22,587,942	$456,650	$547,980	**$0**	**$684,975**						**$270,235**	**$414,740**
Schnoor Trunk																				
S-5	Pipe Rehabilitation - CIPP[6]	Improvements Triggered by Structural Conditions		Clean/Remove Roots/Install Pipe Liner						437,511	482,690		603,363	Existing Structural Deficiency	FY 2013 - 2015	Northwest Madera	100%	0%	603,363	0
S-6	Lift Station[7,8]	Fairgrounds Lift Station Pump Capacity Upgrade			Replace	2,500 gpm				300,000	360,000		450,000	PWWF Deficient	FY 2013 - 2015	Northwest Madera	64%	36%	288,000	162,000
			Sub-Total				3,920		$456,650	$737,511	$842,690	**$0**	**$1,053,363**						**$891,363**	**$162,000**
Westberry Basin																				
Ellis Street Trunk																				
W-3	Pipe	Ellis Street	From Krohn St to Sharon Blvd		New	24	1,070	218	233,260	233,260	279,912		349,890	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	349,890
W-4	Pipe	Ellis Street	From D St to Krohn St		New	21	5,890	180	1,058,433	1,058,433	1,270,120		1,587,650	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	1,587,650
W-5	Pipe/Casing[5]	Ellis Street	Crossing Canal		New	21/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	216,000
W-6	Pipe	D Street	From Martin St to Ellis St		New	21	1,020	180	183,294	183,294	219,953		274,941	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	274,941
W-7	Pipe	Lake Street (Road 27)	From 2,120t n/o Martin St to Martin St		New	12	2,120	93	196,100	196,100	235,320		294,150	Construct with development in Area	FY 2016 - 2050	Central Madera, North Madera	0%	100%	0	294,150
W-8	Pipe	Martin Street Extension	From 3,330 e/o Lake St to D St		New	21	4,340	180	779,898	779,898	935,878		1,169,847	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	1,169,847
W-9	Pipe	Future City Road or ROW (3,330ft e/o Lake Street)	From Avenue 17 to Martin St Ext		New	21	1,290	180	231,813	231,813	278,176		347,720	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	347,720
W-10	Pipe/Casing[5]	Avenue 17	Crossing RR Tracks		New	21/48	150	960	144,000	144,000	172,800		216,000	Construct with development in Area	FY 2016 - 2050	Central Madera	0%	100%	0	216,000
W-11	Pipe	Avenue 17	From 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4) to 560ft w/o RR Tracks		New	21	410	180	73,677	73,677	88,412		110,516	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	110,516
W-12	Pipe	Future Road 27 3/4	From Avenue 17 1/2 Ext to Avenue 17		New	15	2,640	132	347,820	347,820	417,384		521,730	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	521,730
W-13	Pipe	Avenue 17 1/2 extension	Tuolumne St Ext to Future Road 27 3/4		New	12	2,000	93	185,000	185,000	222,000		277,500	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	277,500
W-14	Pipe	Avenue 17	From Harper Blvd to 425ft w/o Harper Blvd at RR Tracks (Future Road 27 3/4)		New	15	420	132	55,335	55,335	66,402		83,003	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	83,003
W-15	Pipe	Avenue 17	From 2,370 ft e/o Harper Blvd to Harper Blvd		New	10	2,370	65	154,643	154,643	185,571		231,964	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	231,964
W-16	Pipe	Harper Boulevard	From Road 28 1/2 to Avenue 17		New	15	4,750	132	625,813	625,813	750,975		938,719	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	938,719
W-17	Pipe	Future Road or ROW	From Arizona Ave southwest to Road 28 1/2		New	12	2,260	93	209,050	209,050	250,860		313,575	Construct with development in Area	FY 2016 - 2050	Northeast Madera	0%	100%	0	313,575
			Sub-Total				30,880		$4,622,135	$4,622,135	$5,546,562	**$0**	**$6,933,203**						**$0**	**$6,933,203**
Road 23 Trunk																				
W-18	Pipe	Road 23	From Ave 16 to Pecan Ave (Ave 13)		New	30	14,950	310	4,634,500	4,634,500	5,561,400		6,951,750	Construct with development in Area	FY 2016 - 2050	Northwest Madera, N/A	0%	100%	0	6,951,750
W-19	Casing	Road 23	Crossing Fresno River		New	30/54	950	1,080	1,026,000	1,026,000	1,231,200		1,539,000	Construct with development in Area	FY 2016 - 2050	Northwest Madera, N/A	0%	100%	0	1,539,000
W-20	Lift Station[7,8]	Avenue 16 and Road 23			New	2,300 gpm				1,125,000	1,350,000	20,000	1,712,500	Construct with development in Area	FY 2016 - 2050	Airport North	0%	100%	0	1,712,500

Table 8.2 Capital Improvement Program

Sanitary Sewer System Master Plan

City of Madera

Itemized Cost Estimate															Capacity Allocation			
Improvement Number	Type of Improvement	Street	Limits	Existing Pipe Diameter	Parallel, Replace, or New	Pipeline and Appurtenances Costs				Baseline Constr. Cost	Estimated Constr. Cost[2]	Land Acquisition[3]	Capacity Improv. Cost[4]	Construction Trigger	Suggested Phasing	Village/District	% Benefit		Cost Sharing	
						Diameter	Length	Unit Cost[1]	Pipe Cost								Existing	Future	Existing	Future
				(in)		(in)	(ft)	($/LF)	($)	($)	($)	($)	($)						($)	($)
W-21	Pipe	Avenue 16	From Road 22 1/2 to Road 23		New	18	2,750	145	397,375	397,375	476,850		596,063	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	596,063
W-22	Pipe	Road 23	From Ave 18 to Ave 16		New	15	10,550	132	1,389,963	1,389,963	1,667,955		2,084,944	Construct with development in Area	FY 2016 - 2050	Northwest Madera	0%	100%	0	2,084,944
W-23	Pipe/Casing[5]	Avenue 18	Crossing HWY 99 and Railroad Tracks		New	12/36	550	720	396,000	396,000	475,200		594,000	Construct with development in Area	FY 2016 - 2050	Airport North	0%	100%	0	594,000
W-24	Pipe	Avenue 18	From Sharon Blvd/Road 24 to Road 23		New	12	4,760	93	440,300	440,300	528,360		660,450	Construct with development in Area	FY 2016 - 2050	Airport North, N/A	0%	100%	0	660,450
W-25	Pipe	Road 23 1/2	From Almond Ave (Ave 13 1/2) to Pecan Ave (Ave 13)		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-26	Pipe	Avenue 14	From Road 23 1/2 to Road 23		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-27	Pipe	Avenue 14 1/2	From Road 23 1/2 to Road 23		New	12	2,600	93	240,500	240,500	288,600		360,750	Construct with development in Area	FY 2016 - 2050	N/A	0%	100%	0	360,750
W-28	Pipe	Avenue 15 1/2	From Avenue 24 to Road 23		New	15	5,275	132	694,981	694,981	833,978		1,042,472	Construct with development in Area	FY 2016 - 2050	Northwest Madera	0%	100%	0	1,042,472
			Sub-Total				47,585		$9,700,619	$10,825,619	$12,990,743	**$20,000**	**$16,263,428**						**$0**	**$16,263,428**
								Total Improvement Costs					**$85,206,214**						**$1,161,598**	**$84,044,616**

Notes:

1. Cost estimates are based on the Engineering News Record (ENR) construction cost index (CCI) of 9545 for the 20 cities for August 2013.
2. Baseline construction costs plus 20% to account for unforeseen events and unknown conditions.
3. A land acquisition fee for the construction of lift stations was assumed based on recent land acquisitions. It was generally assumed that lift stations will require 0.5 acre.
4. Estimated construction cost plus 25% to cover other costs including: engineering design, project administration (developer and City staff), construction management and inspection, and legal costs.
5. Proposed casings size and carrier pipe size.
6. Pipe rehabilitation contingencies are baseline plus 10% for unforeseen events and 25% to cover other costs.
7. Lift Station capacity is given as capacity with largest pump out of service.
8. Lift station pricing can vary widely with site conditions.

6/7/2013

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.

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,
- or as new villages are developed by the City.

8.3.3 Construction Phasing

The Capacity 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) 2013 through 2015 for improvements that are required to resolve existing deficiencies and other critical pipes in the sanitary sewer system.
- **Intermediate Term:** This intermediate term phase includes improvements that are required to be completed for fiscal years 2016 through 2020.
- **Long Term:** This long term phasing includes improvements that are required to be completed for fiscal years 2021 through 2030.

Beyond General Plan Horizon

- **Extended Range:** This extended range phase consists of improvements for the fiscal years 2031 through 2050 for improvements beyond the General Plan Horizon.

As it is difficult to impossible to guess which direction the City will develop, it should be noted that most items have a phasing of FY 2016-2050, which indicate these improvements will be constructed as new developments necessitate them.

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 Improvements Located within Planning Villages

Sanitary sewer system improvements are documented on **Figure 8.1** with City Planning Village boundaries overlaid. This figure is intended to provide general guidance documenting improvements occurring in each of the Planning Villages. **Table 8.2** lists improvements, and documents the Planning Village in which the improvement occurs, if applicable.

8.3.5 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.

8.3.6 Wastewater Treatment Plant Capital Improvement Costs

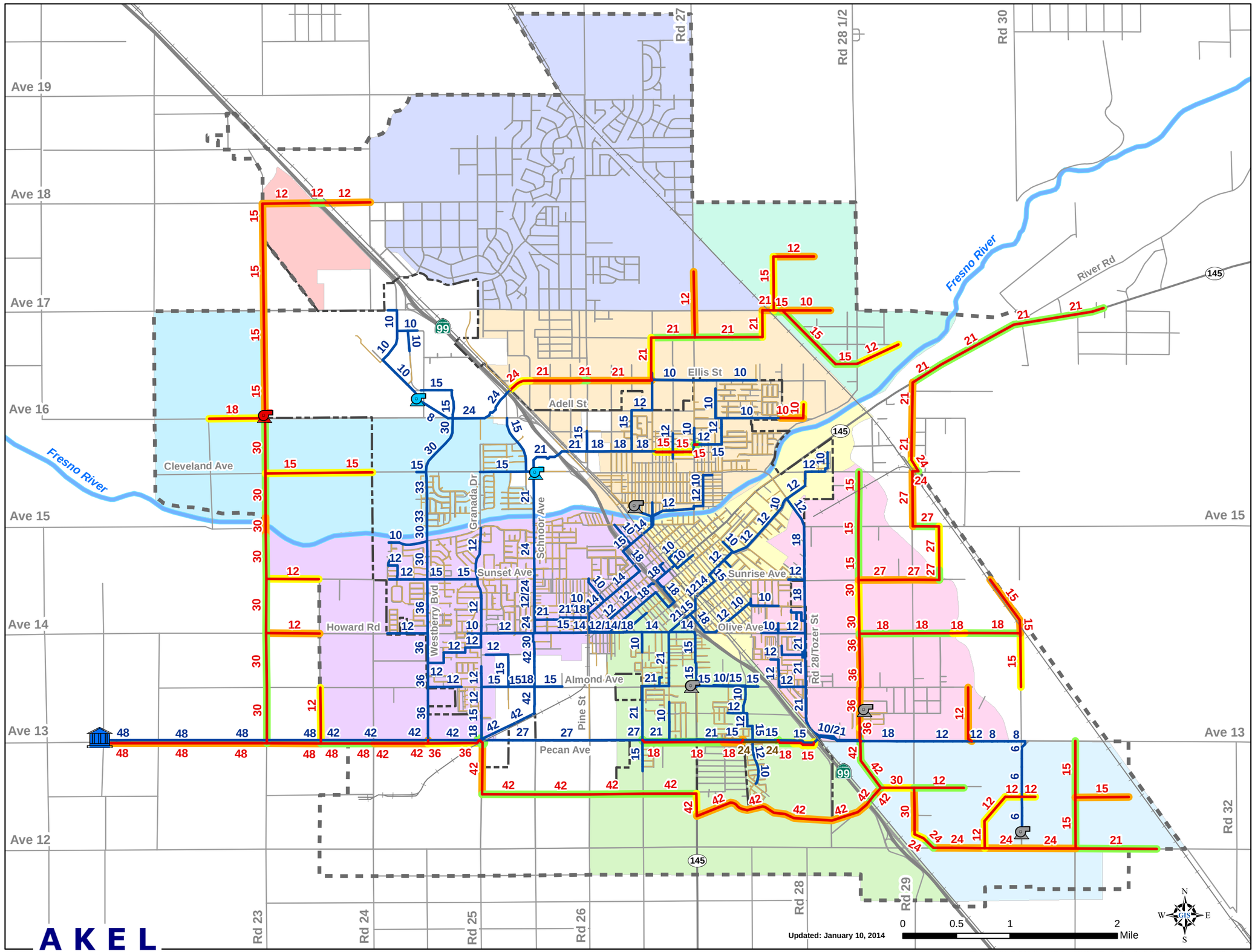
The 2012 Wastewater Treatment Plant Evaluation (**Appendix E**) included capital improvement project costs for the near-term (**Table 8.3**) and for the ultimate buildout of the General Plan (**Table 8.4**). It should be noted that the costs in **Tables 8.3** and **8.4** do not include planning, engineering, administration, permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, and unforeseen site conditions.

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 Suggested Expenditure Budget

The suggested expenditure budget is shown on **Table 8.5**, and includes the total costs for pipelines and pump stations phased by 5-year fiscal period through the year 2050. Costs are categorized through the General Plan horizon of 2030 for near-term, immediate term, and long term planning. Improvements beyond the General Plan horizon were considered extended range improvements and consist mostly of buildout improvements intended to service future users outside the existing system.



Legend

Improvements

- Lift Station
- Pipes

Existing

- WWTP
- Modeled Lift Stations
- Non-Modeled Lift Stations
- Modeled Pipes
- Existing Pipe Not In Use
- Non-Modeled Pipes

General Plan Villages

- A - North Madera
- B - Northeast Madera
- C - Central Madera
- D - Northwest Madera
- E - West Madera
- F - Downtown
- G - Parksdale
- H - Parkwood
- I - Community College
- J - Airport North

- Roads
- Highways
- Railroads
- City Limits
- Urban Growth Boundary
- Fresno River

Figure 8.1
Capacity Improvements with Planning Villages
Sanitary Sewer System Master Plan
City of Madera

AKEL ENGINEERING GROUP, INC.

File Name: MA_Fig8-1FutImp-wVillages_011014 File Location: P:\xGIS\GIS_Projects\Madera\MP\Sewer

Table 8.3 Recommended WWTP Capital Improvement Project Costs - Near-Term

Sanitary Sewer System Master Plan
City of Madera

Unclassified Sludge			Class B Biosolids		
Year Improvement Needed	Recommended Capital Improvement Projects	Estimated Construction Cost[2]	Year Improvement Needed	Recommended Capital Improvement Projects	Estimated Construction Cost[2]
Near-Term	Preliminary Treatment Improvements · Mechanical Bar Screen Addition · Influent Lift Station Expansion · Primary Effluent Pump Station Expansion	$1,500,000 - $2,600,000 [3]	Near-Term	Preliminary Treatment Improvements · Mechanical Bar Screen Addition · Influent Lift Station Expansion · Primary Effluent Pump Station Expansion	$1,500,000 - $2,600,000 [3]
Y2014-2017	Solids Handling Improvements without WAS Co-Settling[4] · Sludge Thickener	$3,100,000	Y2014-2020	Solids Handling Improvements without WAS Co-Settling[4] · Sludge Thickener · 1 – 45' diameter Anaerobic Digesters	$9,900,000 [3]
Review By	**Potential Capital Improvement Projects**	**Comments**	**Review By**	**Potential Capital Improvement Projects**	**Comments**
Y2017	Oxidation Ditch Improvements	Unlikely to be required	Y2017	Oxidation Ditch Improvements	Unlikely to be required
Y2019	Evaporation / Percolation Ponds	Proactive monitoring is critical	Y2019	Evaporation / Percolation Ponds	Proactive monitoring is critical

Notes:

10/8/2013

1. Table extracted from MWH Wastewater Treatment Plant Evaluation report dated 2013

2. Estimated construction cost does not include planning, engineering, administration, permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, unforeseen site conditions.
3. Cost dependent on influent lift station expansion
4. Solids handling improvements based on Option 2 but does not preclude interim implementation of Option 3

Table 8.4 Recommended WWTP Capital Improvement Project Costs - Buildout

Sanitary Sewer System Master Plan
City of Madera

Description	Estimated Construction Costs[2]
Pretreatment	$20.5M
Primary Treatment	$6.1M
Tertiary Treatment[3]	$10.1M
MBR Trains	$114.8M
Effluent Disposal	$32.8M
Solids Handling	$17.3M
Total	**$202M**

Notes: 10/8/2013

1. Table extracted from MWH Wastewater Treatment Plant Evaluation report dated 2013

2. Estimated construction cost does not include escalation, planning, engineering, administration, permits, major utility, relocation, disposal of contaminated soil or ground water, land/easement purchases, unforeseen site conditions.

3. Cost of tertiary treatment for existing oxidation ditch effluent is optional, but included for purposes of comparison

Table 8.5 Suggested Sewer Collection System Expenditure Budget

Sanitary Sewer System Master Plan

City of Madera

Project Type	Suggested Sewer Collection System Expenditure Budget[1,2]							
	General Plan Horizon				Beyond General Plan Horizon[3,4]			
	Near-Term	Intermediate Term	Long-Term		Extended Range			
	FY 2013 - 2015	FY 2016 - 2020	FY 2021 - 2025	FY 2026 - 2030	FY 2031 - 2035	FY 2036 - 2040	FY 2041 - 2045	FY 2046 - 2050
Pipelines	$2,523,672	$18,395,124	$7,427,397	$13,897,024	$10,155,750	$11,172,266	$8,485,941	$10,986,540
Lift Stations	$450,000	$0	$0	$0	$0	$0	$0	$1,712,500
Total	**$2,973,672**	**$18,395,124**	**$7,427,397**	**$13,897,024**	**$10,155,750**	**$11,172,266**	**$8,485,941**	**$12,699,040**
Cumulative Cost	$2,973,672	$21,368,796	$28,796,193	$42,693,217	$52,848,967	$64,021,233	$72,507,174	$85,206,214

Notes: 6/7/2013

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.
2. Wastewater treatment plant upgrade/expansion project costs are not include in this suggested expenditure budget.
3. Phasing on this table is based on the extended horizon of available land use within the Planning Area.
4. The extended horizon is based on expected population growth and available residential land uses within the Planning Area.

8.4.2 Sequence of Construction

Suggested expenditure budget phasing is intended to provide general guidance for implementing the Capital Improvement Projects listed in this Master Plan. Construction of pipeline improvements, within any collection basin identified in this Master Plan, is recommended only after the completion of any recommended downstream improvements. Developing a sequence of construction for the capital improvement projects is subject to the approval of the City Engineer.

APPENDICES

APPENDIX A

Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, 2011 (V&A)

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

City of Madera

March 2011

V&A

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

City of Madera

Prepared for

AKEL ENGINEERING GROUP, INC.

Prepared by

V&A

March 2011

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APPENDIX

Appendix A: Flow Monitoring Sites: Data, Graphs, Information

EXECUTIVE SUMMARY

V&A has completed sanitary sewer flow monitoring, rainfall monitoring and inflow/infiltration (I&I) analysis within the City of Madera, California (City). Flow monitoring occurred over the 18-day period from February 15, 2011 to March 4, 2011 at six open channel flow monitoring sites.

Flow Monitoring and Inflow/Infiltration Analysis Results

Table 1 summarizes the flow monitoring and I&I results for the flow monitoring sites. Infiltration and inflow rankings are shown such that 1 = highest infiltration or inflow contribution, to 6 = least infiltration or inflow contribution. Figure 1 shows a flow chart of the peak flows. I&I values shown are from Storm Event 1 (February 16 – 20, 2011).

Table 1.
Flow Monitoring and I&I Results Summary[1]

Site	ADWF (mgd)	Peak Flow (mgd)	Total Infiltration and Inflow (gallons)	Inflow Ranking	Infiltration Ranking		Combined I&I Ranking	Peaking Factor	d/D Ratio
					RDI	GWI			
Site 1	1.01	1.84	113,000	3	3	3(tie)	6	1.82	0.34
Site 2	2.93	4.68	619,000	5	4	3(tie)	3	1.60	0.24
Site 3	0.85	2.26	202,000	1	2	2	1	2.65	0.50
Site 4	0.61	1.43	119,000	2	1	3(tie)	4	2.35	0.45
Site 5	1.26	2.16	237,000	4	5	3(tie)	5	1.71	0.70
Site 6	1.32	2.10	303,000	6	6	1	2	1.59	0.52

The following results from this project are noted:

- **Inflow:** Site 3 had the highest inflow factor of the flow monitoring sites.
- **Rain Dependent Infiltration (RDI):** Site 4 had the highest RDI factor of the flow monitoring sites.
- **Groundwater Infiltration (GWI):** Sites 3 and 6 had groundwater infiltration rates that were higher than typical.
- **Combined I&I:** Site 3 had the highest combined I&I factor of the flow monitoring sites; however, Sites 2 through 6 were relatively similar in response. Site 1 had the lowest combined I&I factor.

[1] Notes: (a) ADWF = average dry weather flow. (b) Peak flow as measured during entire flow monitoring period. (c) RDI = Rain Dependent Infiltration, GWI = Groundwater Infiltration (refer to pages 13, 14 for additional information) (d) PF = Peaking Factor = Peak Measured Flow / ADWF. (e) d/D Ratio = peak measured depth / pipe diameter. Value shown is the highest d/D ratio as measured during entire flow monitoring period.

- **Capacity (Peaking Factor):** All locations had peaking factors less than typical design threshold limits for the peak flow to average dry weather flow ratio. Site 3 had the largest peaking factor.
- **Capacity (d/D Ratio):** All locations had d/D ratios below common threshold values for d/D ratio. None of the flow monitoring sites surcharged during this study. Site 5 had the highest d/D ratio.

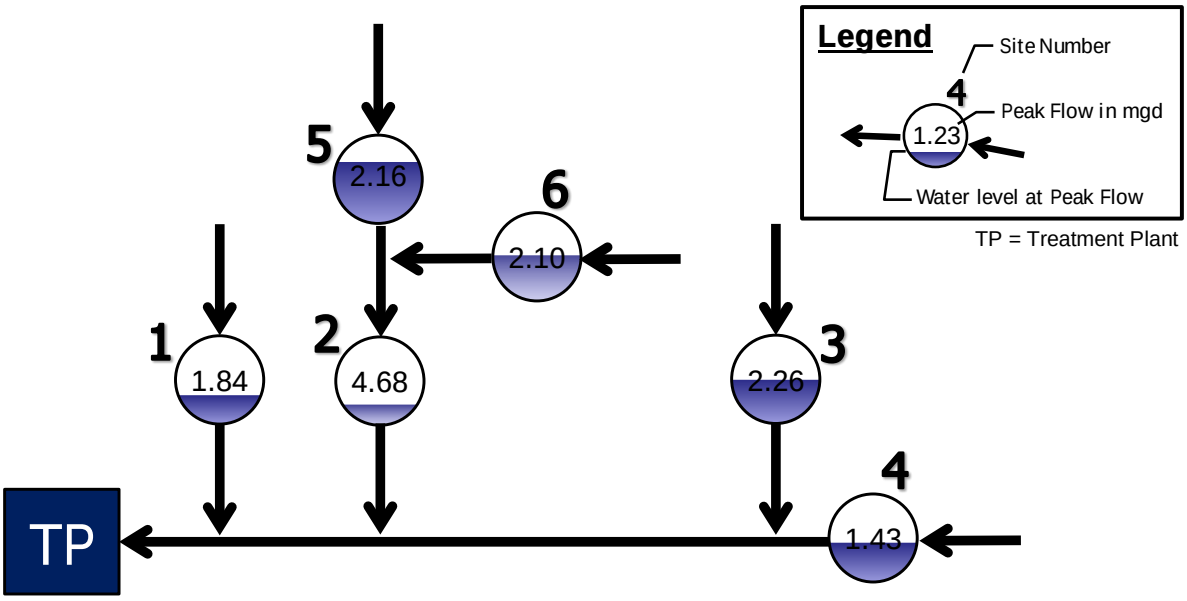


Figure 1. Peak Measured Flow (Flow Schematic)

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 occurs in the basin upstream from Site 3.
   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. The highest normalized infiltration was occurring in the basins upstream from Site 4.
2. **I&I Investigation Methods:** Potential I&I investigation methods include the following:
   a. Smoke testing
   b. Mini-basin flow monitoring

c. Night-time 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.

d. CCTV inspection

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/infiltration and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional rainfall dependent I&I flow.

INTRODUCTION

V&A has completed sanitary sewer flow monitoring, rainfall monitoring and inflow/infiltration (I&I) analysis within the City of Madera sanitary sewer collection system. Flow monitoring occurred over the 18-day period from February 15, 2011 to March 4, 2011 at six open channel flow monitoring sites. Rainfall data was captured from one rain gauge placed at the City corporation yard. The flow monitoring sites and rain gauge site are listed in Table 2 and illustrated in Figure 2. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in *Appendix A*.

Table 2.
List of Flow Monitoring and Rainfall Monitoring Sites

Monitoring Site	Diameter (in)	Location
Site 1	35 ¼	MH ID 6128: Westberry Road (Road 24 1/2), north of Pecan Avenue (Ave 13)
Site 2	42	MH ID 6000: Schnoor Avenue, south of Almond Avenue
Site 3	21	MH ID 5697: Stadium Road (Road 26 1/2) at Gary Lane, north of Pecan Avenue (Ave 13)
Site 4	21	MH ID 6614: Pecan Avenue (Ave 13), east of Stadium Road (Road 26 1/2) between Monterey St. and Concord Ct.
Site 5	24	MH ID 4372: Schnoor Avenue, north of Jennings Street
Site 6	21	MH ID 4332: West Fourth Street, east of Schnoor Avenue
RG 1	City Yard: near intersection of East Olive Avenue and South Gateway Dr.	

Figure 2. Flow Monitoring and Rain Gauge Sites

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. Title 8, Section 5158 of the California Code of Regulations provides the guidelines and rules for working in these environments. 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 4-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 entrant(s) to monitor the atmosphere on another 4-gas monitor and maintaining records of all Entrants, if there is 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 4-Gas Monitor

Flow Meter Installation

Six Isco 2150 flow meters were installed by V&A in the sewer lines shown in Figure 2. Isco meters use a pressure transducer to collect depth readings and ultrasonic Doppler sensors on the probe to determine the average fluid velocity. Figure 3 shows a sketch of a typical flow meter installation.

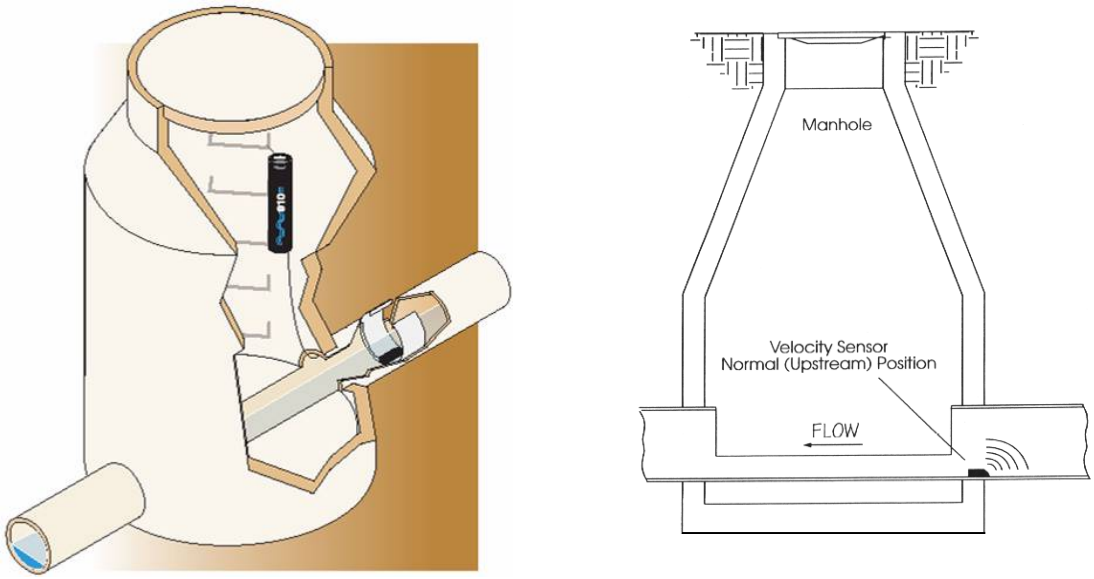


Figure 3. Typical Flow Meter Installation

Continuous depth and velocity readings were recorded by the flow meters in 5-minute increments and downloaded into a computer spreadsheet program where the data could be analyzed and made report ready. Manual level and velocity readings were taken in the field during the flow meter installation and again when the flow meters were removed. These readings were compared to the readings of the flow meters to ensure proper calibration and accuracy.

RAINFALL RESULTS

There was one significant rainfall event over the flow monitoring period, summarized in Table 3. Figure 4 graphically displays the rainfall events recorded over the flow monitoring period.

Table 3.
Rainfall Events used for I&I Analysis

Rainfall Event	RG 1 (in)
Event 1: February 16 - 21, 2011	1.20
Total over Monitoring Period:	*1.52*

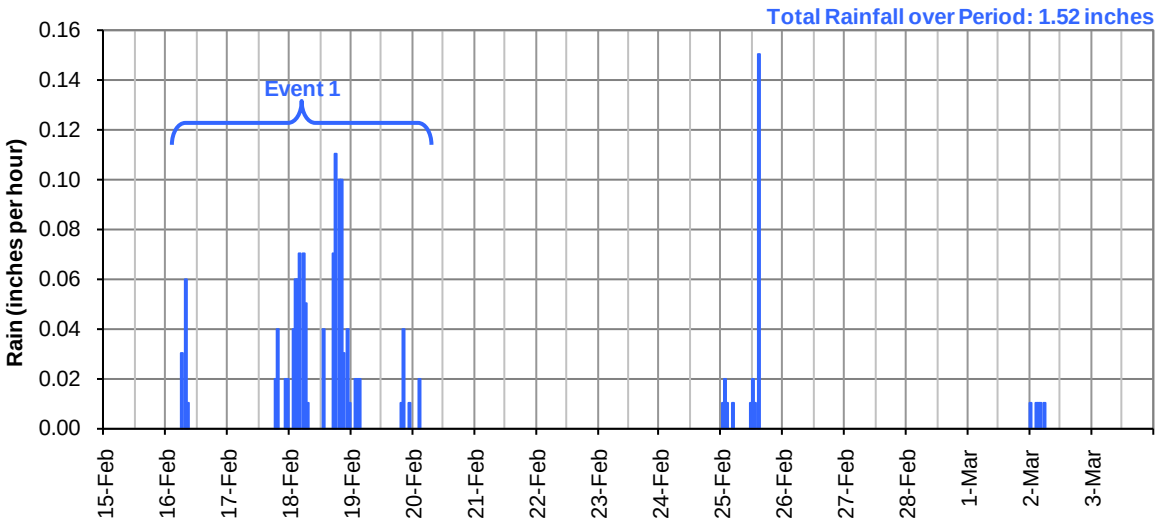


Figure 4. Rainfall Activity over Flow Monitoring Period (RG1 Data shown)

Figure 5 shows the rain accumulation plot of the rain gauge, as well as the historical average rainfall[2] in the City during this project duration. Rainfall totals for Rain Gauge 1 were 119% of historical normal levels during this time period.

[2] Historical data taken from the WRCC: http://www.wrcc.dri.edu/summary/climsmnca.html

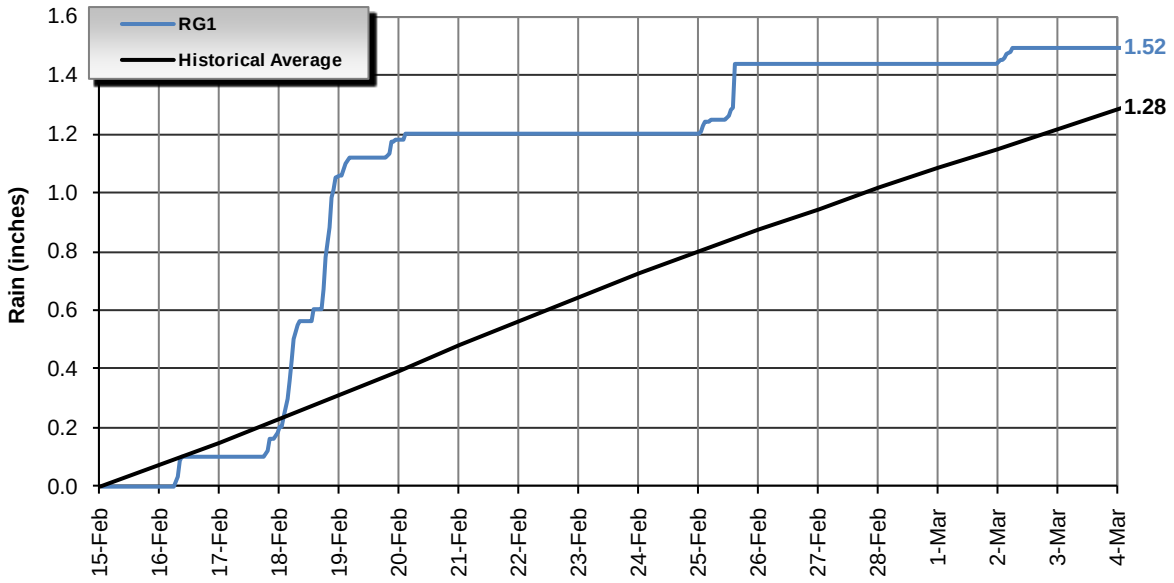


Figure 5. Rainfall Accumulation Plots

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[3]. Storm events are classified by intensity and duration. Based on historical data, frequency contour maps for given intensity and duration storm events 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[4] classifies a 10-year, 24-hour storm event in Madera (at the coordinates of Rain Gauge 1) as 1.82 inches (Figure 6). This means that in any given year, there is a 10% chance that 1.82 inches of rain will fall in any 24-hour period.

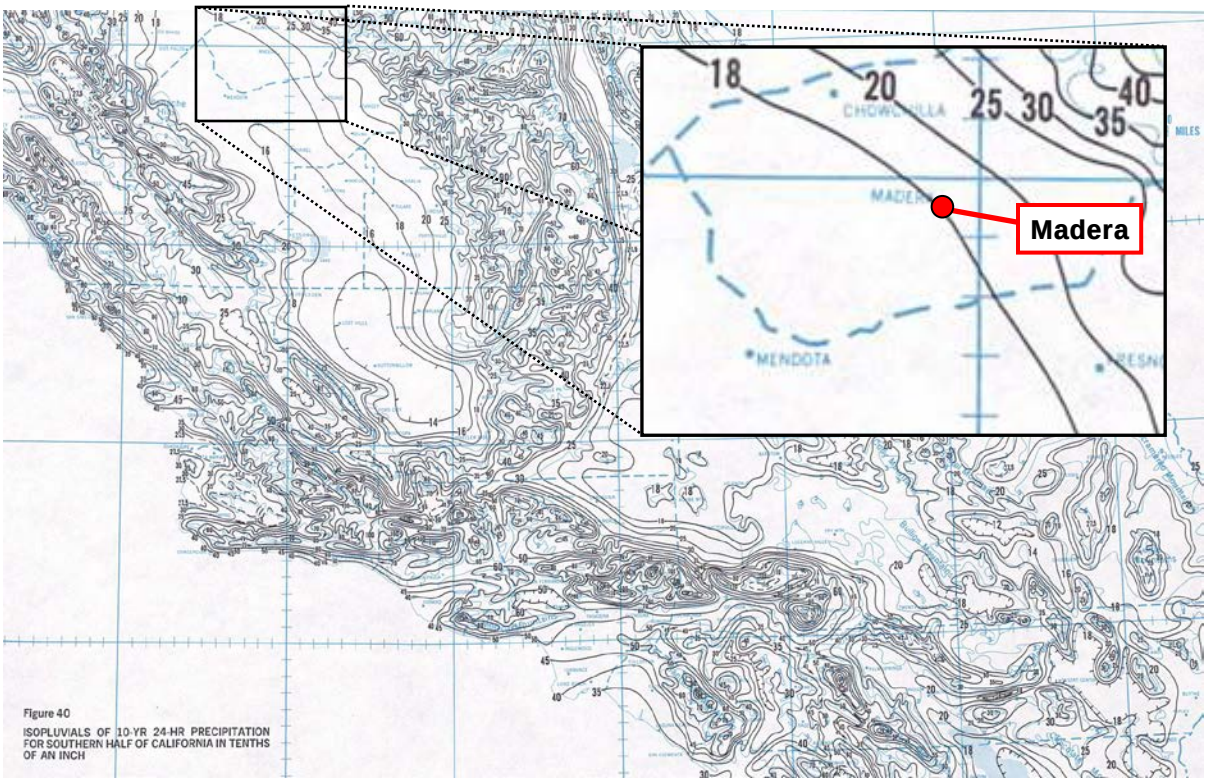


Figure 6. NOAA Northern California Rainfall Frequency Map

From the NOAA frequency maps, the rainfall totals for 1-hour, 6-hour and 24-hour period durations, and 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year period intensities, were plotted to develop a rain event frequency map specific to the rainfall monitoring site, shown in Figure 7.

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

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

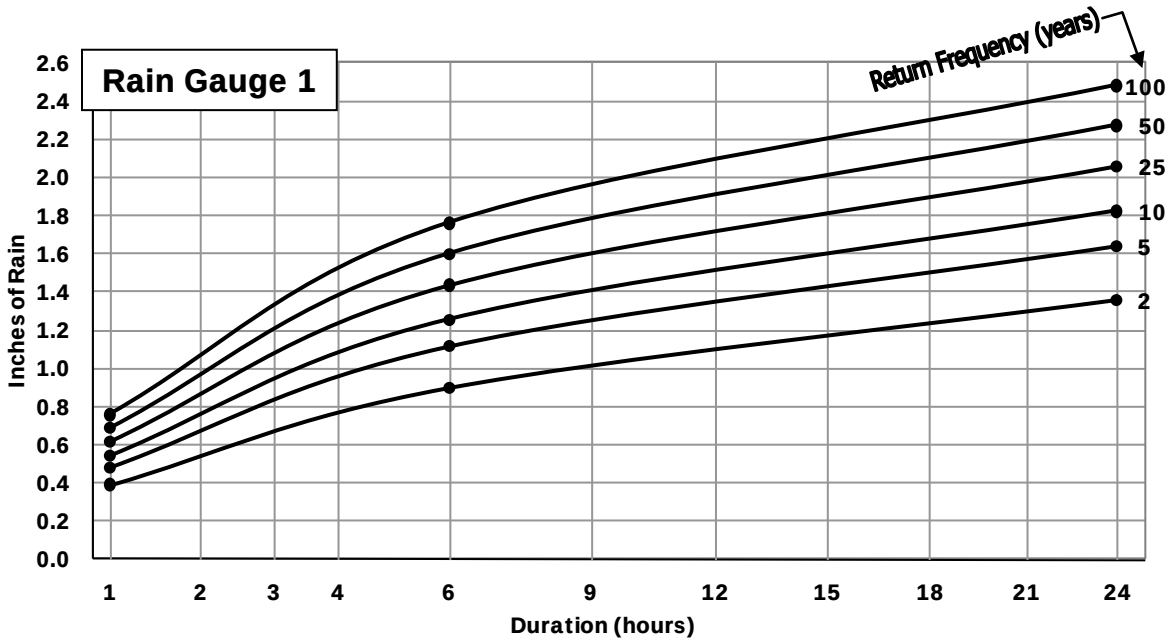


Figure 7. Rain Event Frequency Plot (Rain Gauge 1 Location)

Peak measured densities per hourly periods were calculated for the rain gauge location, summarized in Table 4[5].

Table 4.
Peak Measured Rainfall Densities per Hourly Period (Event 2)

Rainfall Duration	Rain Gauge 1 Peak Densities (inches)
1 hr	0.11
3 hr	0.31
6 hr	0.45
12 hr	0.52
18 hr	0.68
24 hr	0.87

Superimposing the peak measured densities for the storm events on the Rain Event Frequency Plot determines the classification of each storm event, shown in Figure 8.

[5] For example, the highest rainfall in a 1-hour period was 0.11 inches and in any consecutive 6-hour period was 0.45 inches.

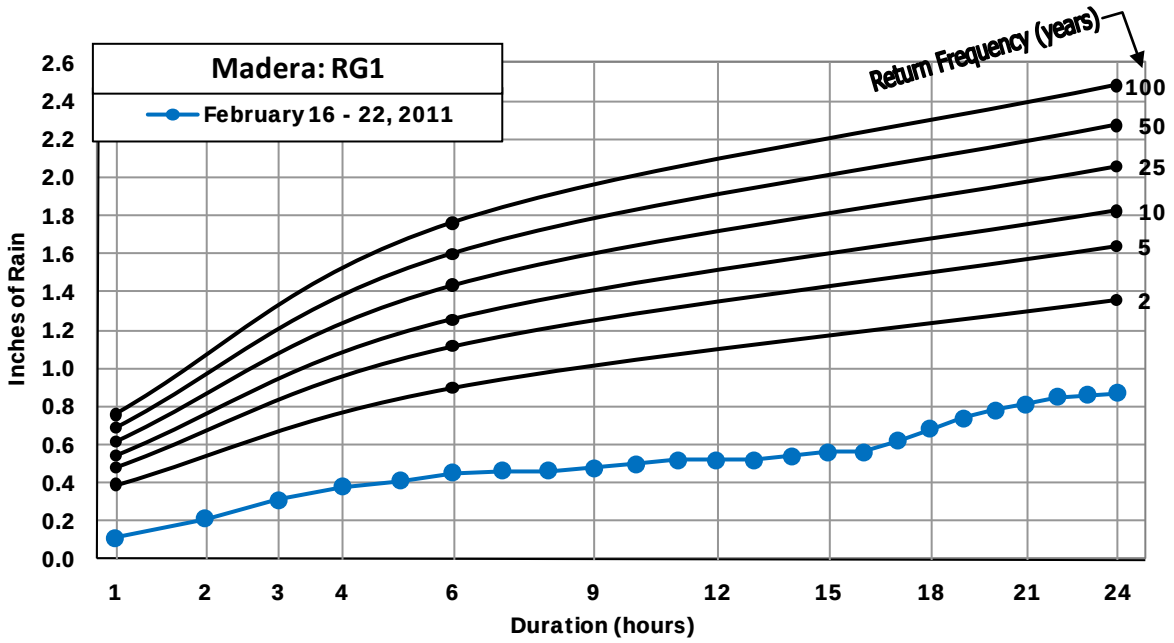


Figure 8. Storm Event Classifications

For Rain Gauge 1, Event 1 was rated as less than a 2-year event.

FLOW MONITORING RESULTS SUMMARY

Average Dry Weather Flows

Weekday and weekend flow patterns vary and must be separated when determining average dry weather flows. Days least affected by rainfall were used to estimate weekend and weekday average flows[6]. Table 5 lists the average dry weather flow (ADWF) recorded during this study for the flow monitoring sites. Detailed graphs of the flow monitoring data on a site-by-site basis are included in *Appendix A*.

Table 5.
Dry Weather Flow Summary

Monitoring Site	Weekday ADWF (mgd)	Weekend ADWF (mgd)	Overall ADWF (mgd)	Weekend/ Weekday Ratio
Site 1	0.99	1.05	1.01	1.06
Site 2	2.90	3.01	2.93	1.04
Site 3	0.85	0.86	0.85	1.00
Site 4	0.60	0.62	0.61	1.04
Site 5	1.24	1.33	1.26	1.08
Site 6	1.31	1.34	1.32	1.02

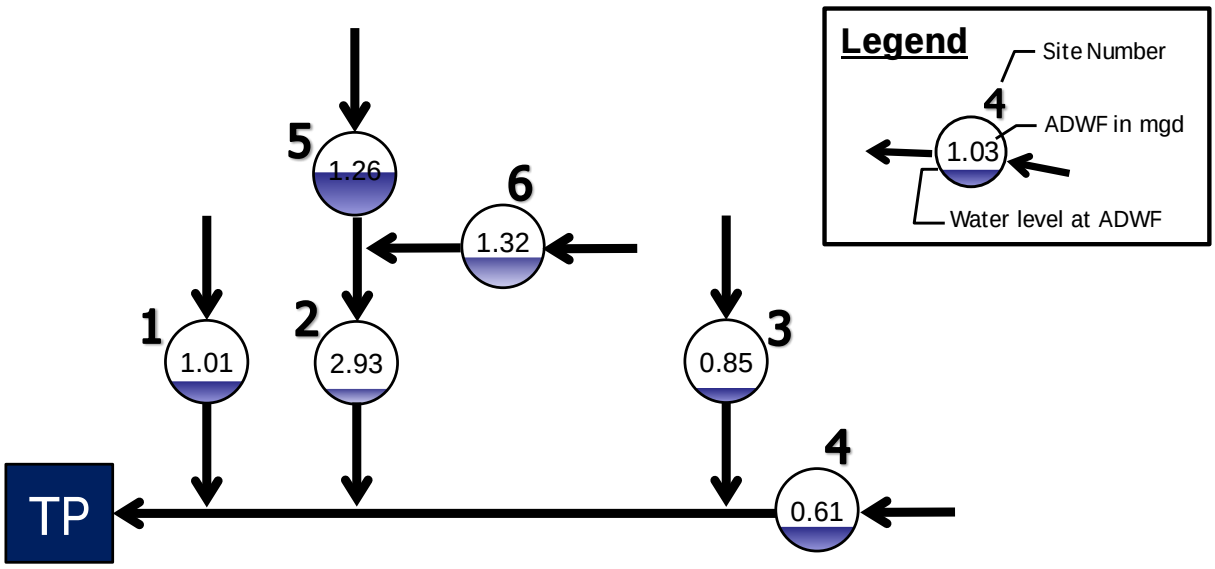


Figure 9. Average Dry Weather Flow (Flow Schematic)

[6] Average days taken from February 26 – March 3, 2011.

Inflow / Infiltration: Methods

Inflow / Infiltration (I&I) consists of storm water and groundwater which enters the sewer system through pipe defects and improper storm drainage connections, 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 immediately following a rain event.

Infiltration

- **Definition:** Infiltration is defined as water entering the sanitary sewer system through defective pipes, pipe joints, and manhole walls, and 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 and/or manholes, and may include cracks, offset joints, root intrusion points, and broken pipes. The sources may be wide-spread 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.

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 on groundwater levels and subsequent groundwater infiltration rates are

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 similar to 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 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. Note: this type of infiltration can have a quick response and graphically can look very similar to inflow.

Figure 10 illustrates the possible locations and components of I&I.

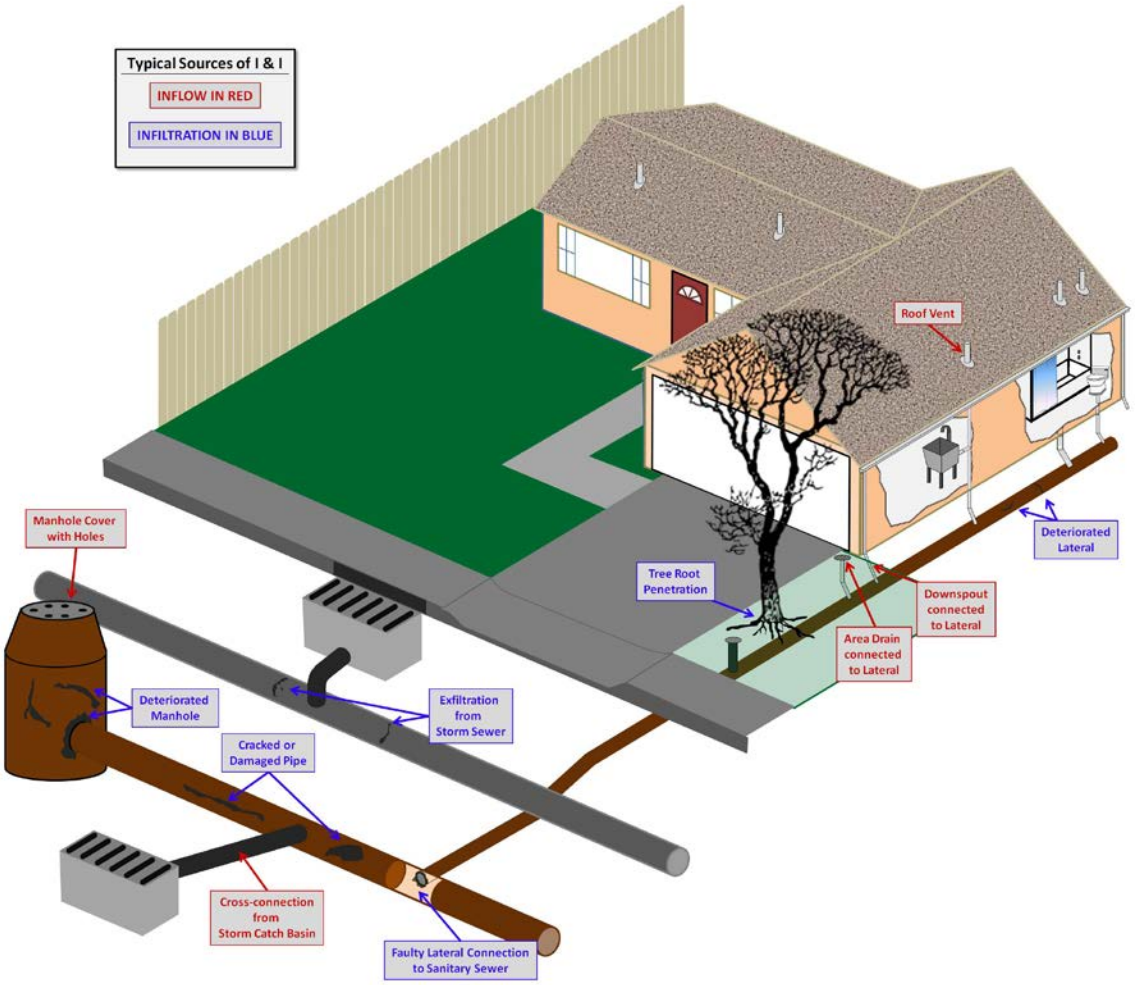
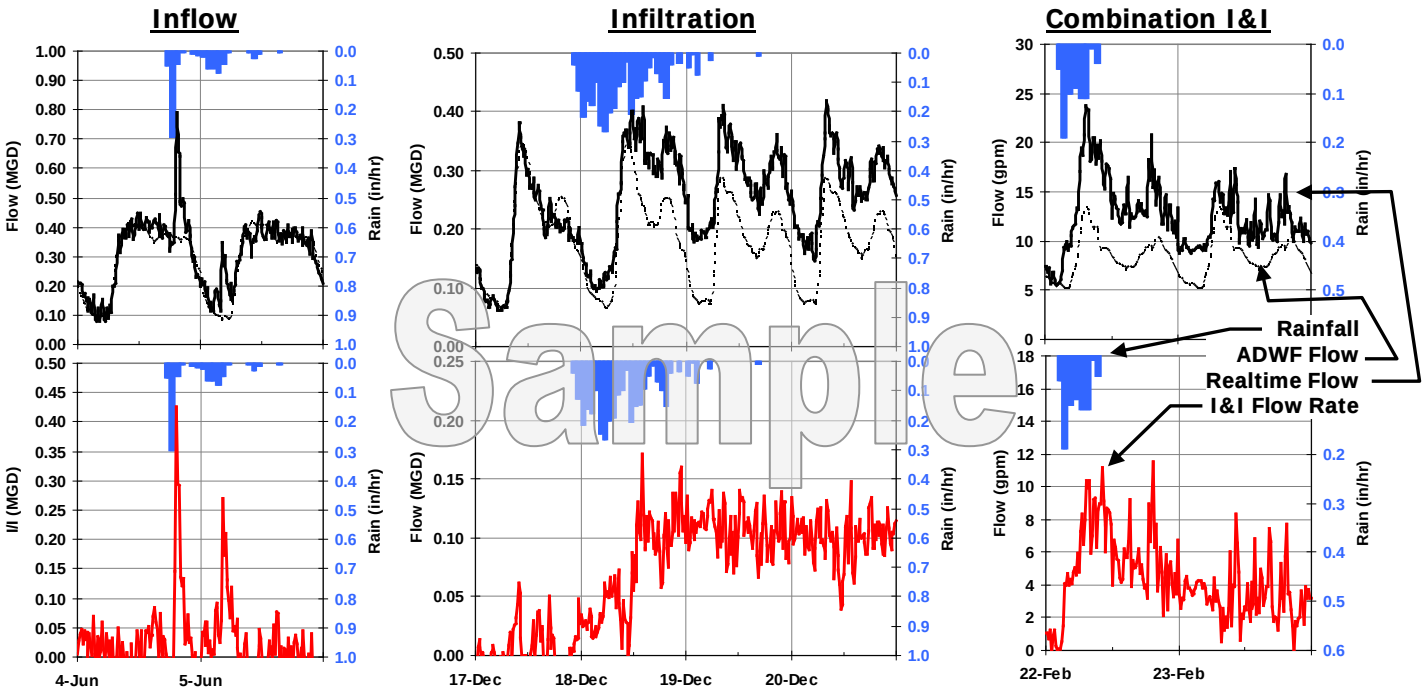


Figure 10. Typical Sources of Infiltration and Inflow

Graphical Identification of I&I Components

Figure 11 shows sample graphs indicating the typical graphical response patterns for inflow/infiltration.



Response Pattern
Sharp Spike
Short Duration

Response Pattern
Gradual Increase
Gradual Recession

Response Pattern
Combination of Inflow and Infiltration

Figure 11. Inflow / Infiltration Graphical Response Patterns

Analysis Techniques

After differentiating I&I flows from ADWF flows, various calculations can be made to determine which I&I component is more prevalent at a particular site, and to compare the relative magnitude of the I&I components between drainage basins and between storm events, summarized as follows:

Inflow Indicators

Peak I&I Flow Rate: It is preferable for I&I analysis to look strictly at I&I flow rates rather than peak flow rates; peak flow rates can be skewed higher or lower depending on whether the storm event I&I response occurs during low flow or high flow hours. 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 basin size of the drainage basin, basin units expressed 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.

The Peak I&I Rate per ADWF Ratio method was used in this report.

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, which may result in quantifying the rate of excess ground water infiltration. Similar to the discussion on inflow, GWI infiltration rates can be normalized by means of pipe length (IDM), basin area (acres) and/or 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 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.

For this project, the infiltration rates were taken approximately 6 hours after the conclusion of the storm event. Infiltration rates were normalized by dividing by ADWF rates.

Combined I&I Indicators

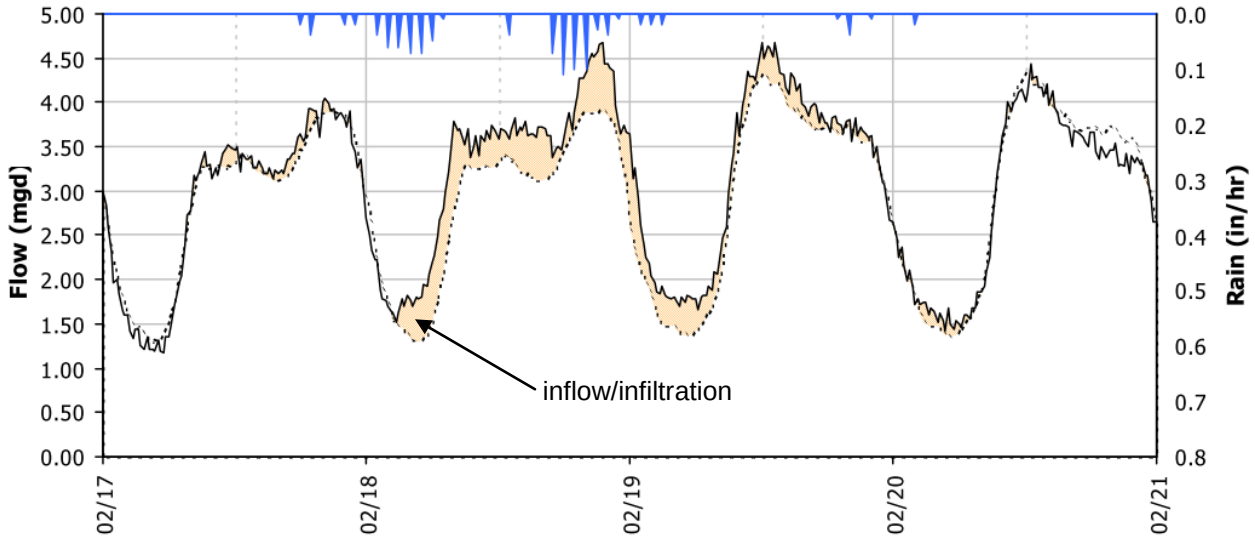
Total Infiltration: The total inflow and infiltration is measured in gallons per site and per storm event. Because it is based on total I&I volume, it is an indicator of combined inflow/infiltration and is used to identify the overall volumetric influence of I&I within the monitoring basin. As with inflow,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

- **Combined I&I Flow Rate per IDM:** Total infiltration (gallons) divided by length of pipe (IDM) and divided by storm event rainfall (inches of rain). Final units are gallons per day (gpd) per IDM per inch-rain.
- **R-Value:** Total infiltration (gallons) divided by the total rainfall that fell within the acreage of that basin (gallons of rainfall). This is expressed as a percentage and is explained as "the percent of rain that falls that enters the sanitary sewer collection system." Systems with R-Values less than 5%[7] 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 gallons (x 10^6) per MGD of ADWF per inch-rain.

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

The combined I&I Flow Rate per ADWF method was used in this report.

Figure 12 below shows a sample I&I graph that illustrates and summarizes the I&I response and I&I calculations made per site per storm event. Similar graphs for each site and storm event are located in *Appendix A*.



Storm Event #1 I/I Analysis

Rainfall:	1.08	*inches*			
Peak Flow:	4.68	*mgd*	**PF:**	1.60	
Peak I/I Rate:	1.02	*mgd*	**Pk I/I:ADWF:**	0.35	
Peak Level:	10.20	*inches*	**d/D Ratio:**	0.24	
Total I/I:	619,000	*gallons*	**I/I per ADWF:**	0.20	*per in-rain*
Infiltration Rate: ***(at 2/19 8:00)***	0.29	*mgd*	**RDI (% of BL):**	10%	

Figure 12. I&I Flow Graph for Site 2, Event 1

INFLOW ANALYSIS

Table 6 summarizes the peak measured I&I flows and inflow analysis results for the flow monitoring locations for Storm Event 1. Figure 13 shows a bar graph of the inflow indicators.

Table 6.
Inflow Analysis Summary

Site	ADWF (mgd)	Peak I&I Rate[A] (mgd)	Peak I&I to ADWF Ratio	Inflow Ranking
Site 1	1.01	0.38	0.37	3
Site 2	2.93	1.02	0.35	5
Site 3	0.85	1.12	1.31	1
Site 4	0.61	0.28	0.46	2
Site 5	1.26	0.47	0.37	4
Site 6	1.32	0.40	0.30	6

Ranking: 1 = most inflow after normalization

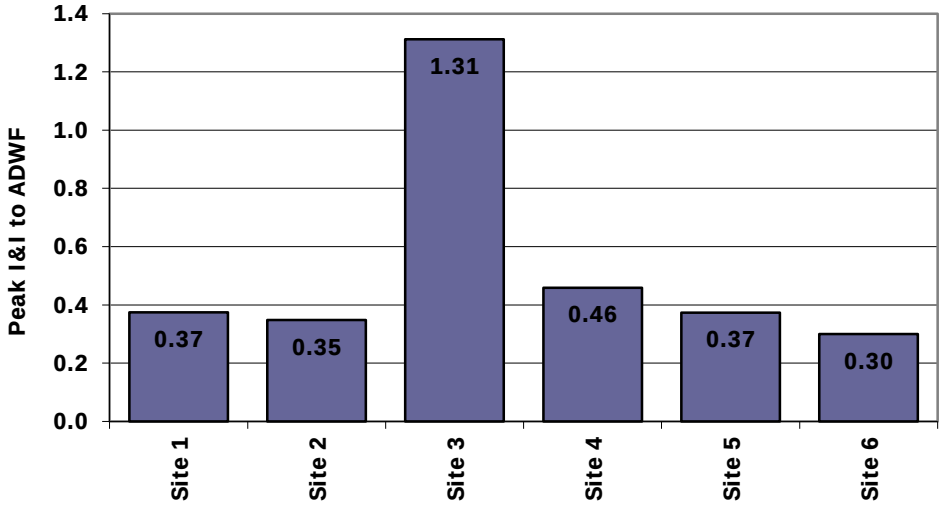


Figure 13. Inflow Indicators by Site

The following inflow analysis results are noted:

- Site 3 had the highest inflow factor of the flow monitoring sites.

RDI (Rain Dependent Infiltration) Analysis

Table 6 summarizes the RDI flows and infiltration component analysis results for the flow monitoring locations. Infiltration rates were taken approximately 6 hours after the conclusion of Event 1. Figure 14 shows bar graphs of the RDI indicators.

Table 7.
RDI Analysis Summary

Site	ADWF (mgd)	Infiltration Rate 6 hours post rain event (mgd)	Infiltration Rate (% of ADWF)	RDI Ranking
Site 1	1.01	0.10	10.0%	3
Site 2	2.93	0.29	9.8%	4
Site 3	0.85	0.10	11.6%	2
Site 4	0.61	0.11	17.4%	1
Site 5	1.26	0.10	8.1%	5
Site 6	1.32	0.07	5.5%	6

Ranking: 1 = most rain dependent infiltration after normalization

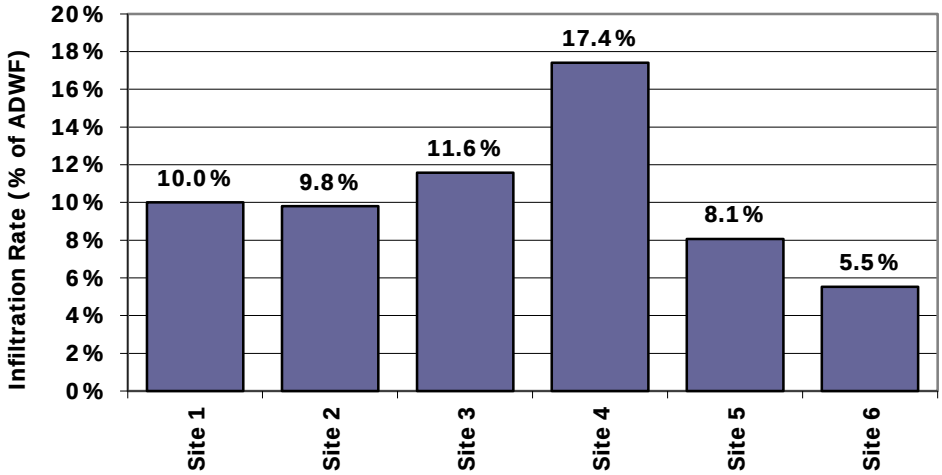


Figure 14. RDI Indicators by Site

The following RDI analysis results are noted:

- Site 4 had the highest RDI factor of the flow monitoring sites.

GWI (Groundwater Infiltration) Analysis

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 lows fall into a predictable range when compared to the daily average flow. If a site has a large percentage of groundwater infiltration occurring during the periods of dry weather flow measurement, the amplitudes of the peak and low flows will be dampened[8]. Figure 15 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.

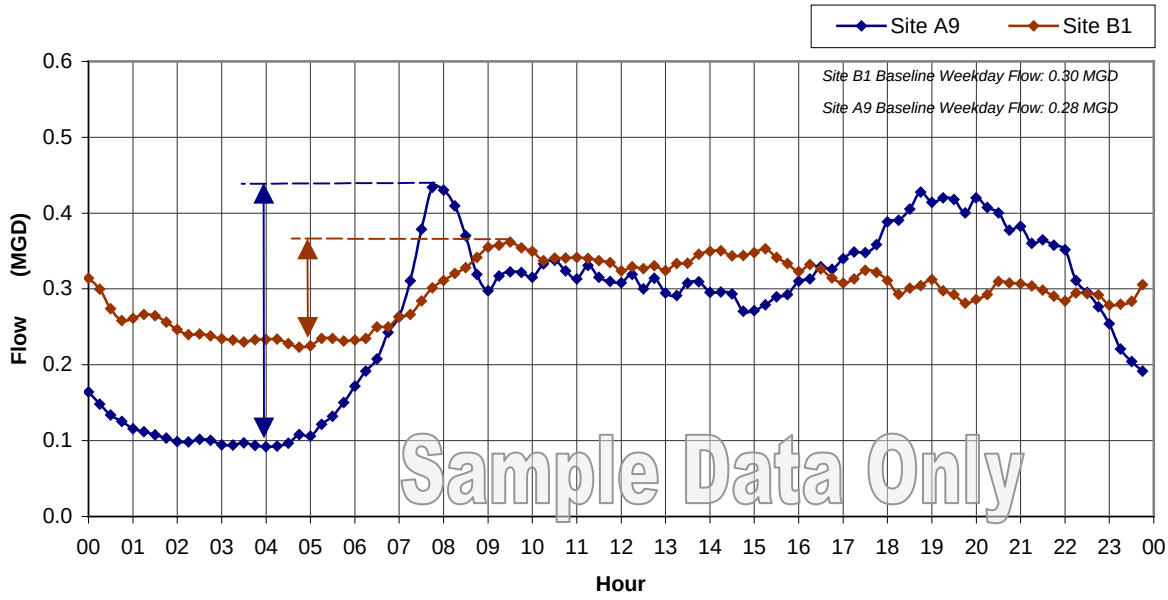


Figure 15. Groundwater Infiltration Sample Figure

It can be useful to compare the low-to-ADWF flow ratios for all flow metering sites. A site with abnormal ratios, and with no other reasons to suspect abnormal flow patterns (such as proximity to pump station, treatment facilities, etc.), has a distinct possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system. Figure 16 plots the min-to-ADWF flow ratios against the ADWF flows for the sites monitored during this study. The dotted line shows "typical" min-to-ADWF flow ratios per the Water Environment Federation (WEF)[9].

Sites 3 and 6 had min-to-ADWF flow ratios that were slightly **above** the WEF typical Low-to-Average Ratio, indicating excessive groundwater infiltration. The rates of groundwater infiltration above typical groundwater infiltration standards (as set forth by WEF) for each site were calculated and are shown

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

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

in Table 8. Please note: The stated groundwater rates are not **total** groundwater infiltration rates, but groundwater rates above typical rates.

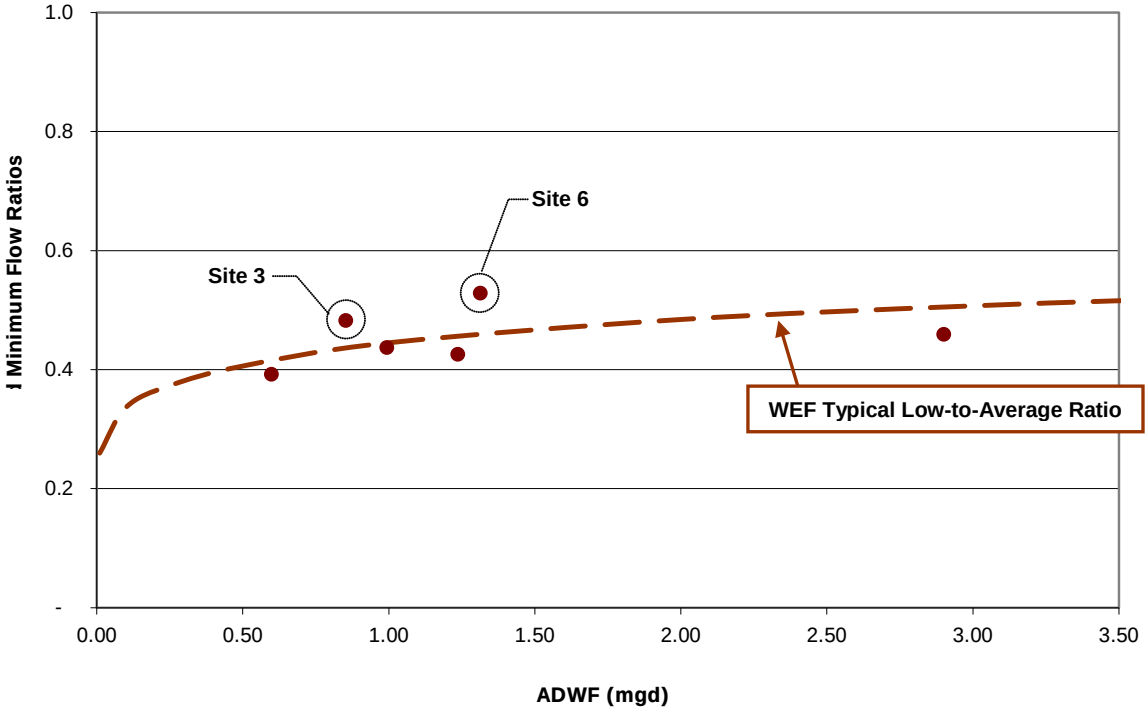


Figure 16. Minimum Flow Ratios vs. ADWF[10]

Table 8.
Excess Groundwater Infiltration Rates above Typical Rate

Location	WkDay ADWF[11] (mgd)	Excess GWI Rate (mgd)	GWI Rate to Revised ADWF Ratio[12] (%)	GWI Ranking
Site 1	0.99	-	-	3(tie)
Site 2	2.90	-	-	3(tie)
Site 3	0.85	0.08	11%	2
Site 4	0.60	-	-	3(tie)
Site 5	1.24	-	-	3(tie)
Site 6	1.31	0.18	19%	1

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

[11] Weekday ADWF is used for this calculation to avoid atypical results during weekend flows which may exist in basins with a large commercial/business service area.

[12] Revised ADWF is calculated as the ADWF after subtracting out the excess GWI. For Site 3, the revised ADWF = 1.27 – 0.11 = 1.16 mgd. Note: the revised ADWF is <u>only</u> used for this analysis. For other analyses and calculations in this study, the measured ADWF (not revised) is used.

Combined I&I Results Summary

Table 9 summarizes the combined I&I analysis results for the flow monitoring locations. Figure 17 shows bar graphs of the inflow indicators.

Table 9.
Combined I&I Analysis Summary

Site	ADWF (mgd)	Total Infiltration (gallons)	I&I per ADWF per In-Rain	Combined I&I Ranking
Site 1	1.01	113,000	0.104	6
Site 2	2.93	619,000	0.196	3
Site 3	0.85	202,000	0.219	1
Site 4	0.61	119,000	0.182	4
Site 5	1.26	237,000	0.173	5
Site 6	1.32	303,000	0.212	2

Ranking: 1 = most combined infiltration/inflow after normalization

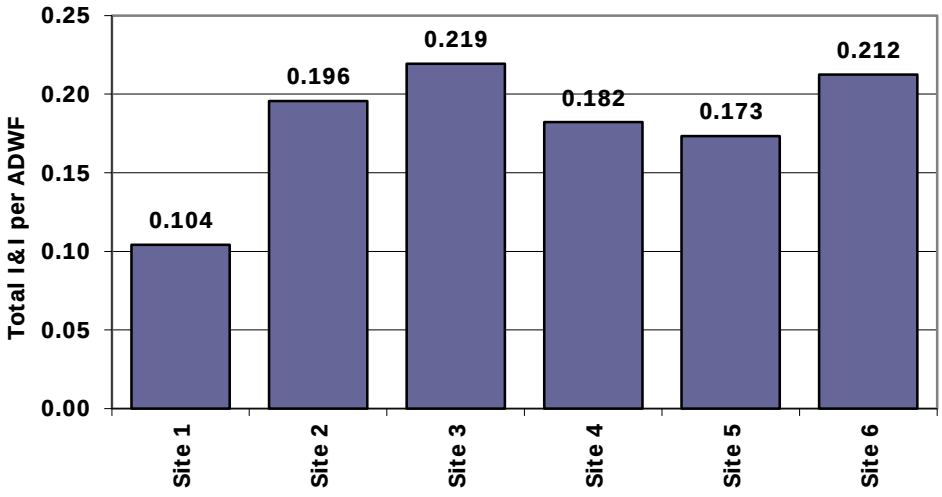


Figure 17. Combined I&I Indicator by Site

The following combined I&I analysis results are noted:

- Site 3 had the highest combined I&I factor of the flow monitoring sites; however, Sites 2 through 6 were relatively similar in response. Site 1 had the lowest combined I&I factor.

Pipeline Capacity Analysis

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 divided by the pipe diameter. A d/D ratio less than 0.75 is a common threshold value used for pipe design. The d/D ratio for each site was computed based on the maximum depth of flow during the rainfall events.

Table 10 summarizes the peak recorded d/D ratios and peaking factors per site during the flow monitoring period. Figure 18 shows a flow chart of the peak flows. Figure 19 shows a flow chart of the d/D ratios. Figure 20 shows bar graphs of the capacity results.

Table 10.
Capacity Analysis Summary

Monitoring Site	ADWF (mgd)	Peak Flow (mgd)	Peaking Factor	Diameter (in)	Peak Level (in)	d/D Ratio	Surcharged Level above Pipe Crown (ft)
Site 1	1.01	1.84	1.82	35.25	11.9	0.34	n/a
Site 2	2.93	4.68	1.60	42	10.2	0.24	n/a
Site 3	0.85	2.26	2.65	21	10.4	0.50	n/a
Site 4	0.61	1.43	2.35	21	9.6	0.45	n/a
Site 5	1.26	2.16	1.71	24	16.7	0.70	n/a
Site 6	1.32	2.10	1.59	21	10.9	0.52	n/a

The following capacity analysis results are noted:

- **Peaking Factor:** All locations had peaking factors less than typical design threshold limits for peak flow to average dry weather flow ratio. Site 3 had the largest peaking factor.
- **d/D Ratio:** All locations had d/D ratios that were less common threshold values for d/D ratio. None of the flow monitoring sites surcharged during this study. Site 5 had the highest d/D ratio.

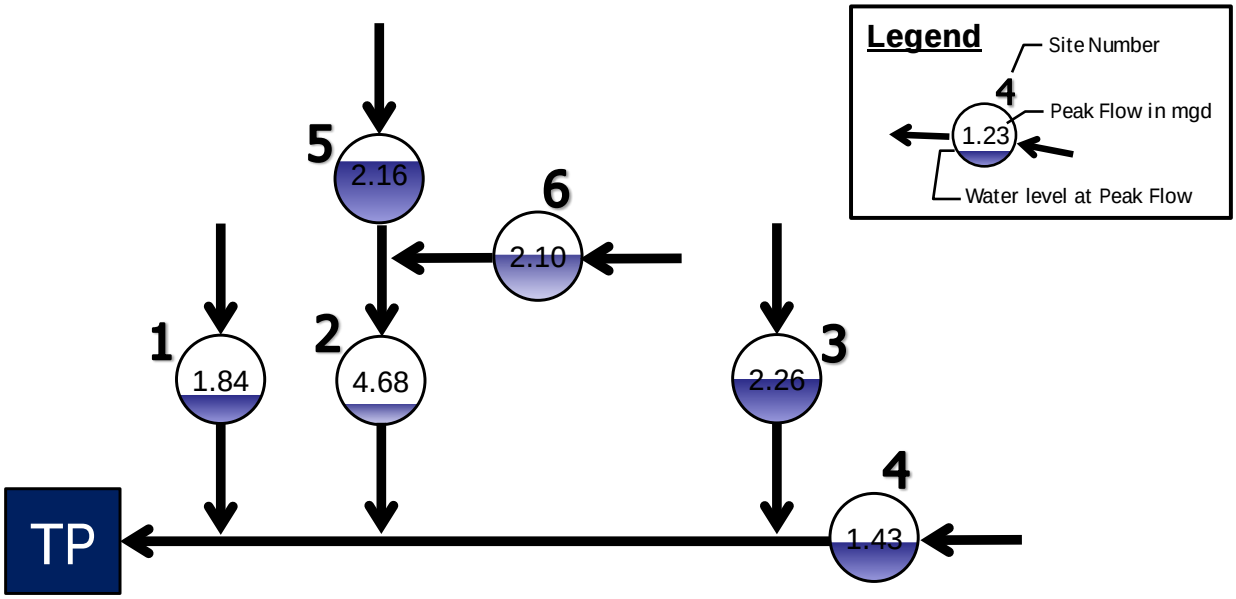


Figure 18. Peak Measured Flow (Flow Schematic)

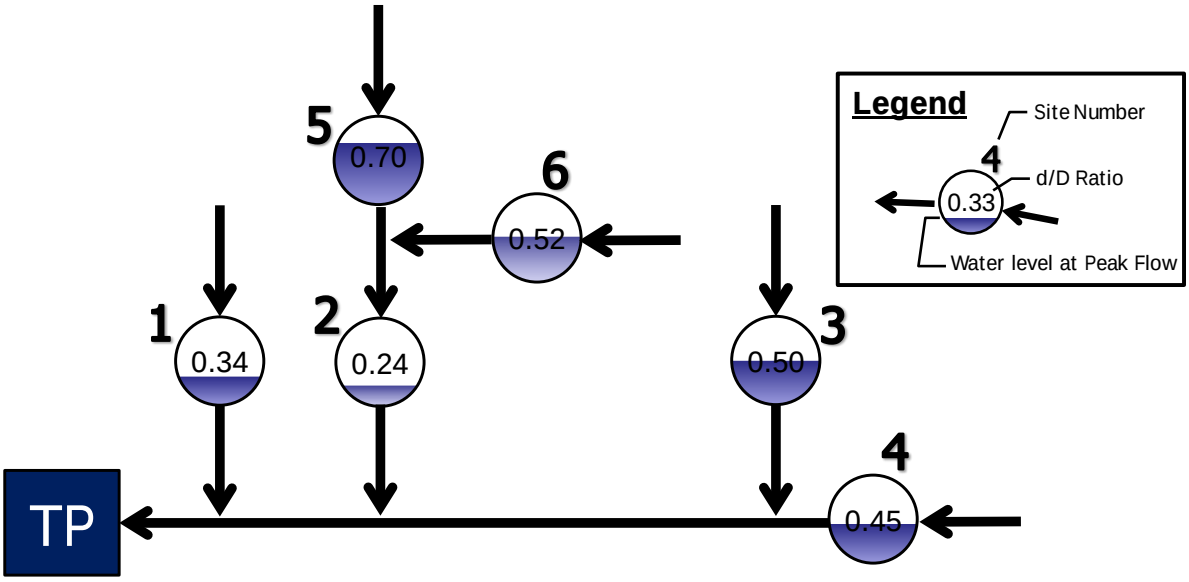


Figure 19. d/D Ratios (Flow Schematic)

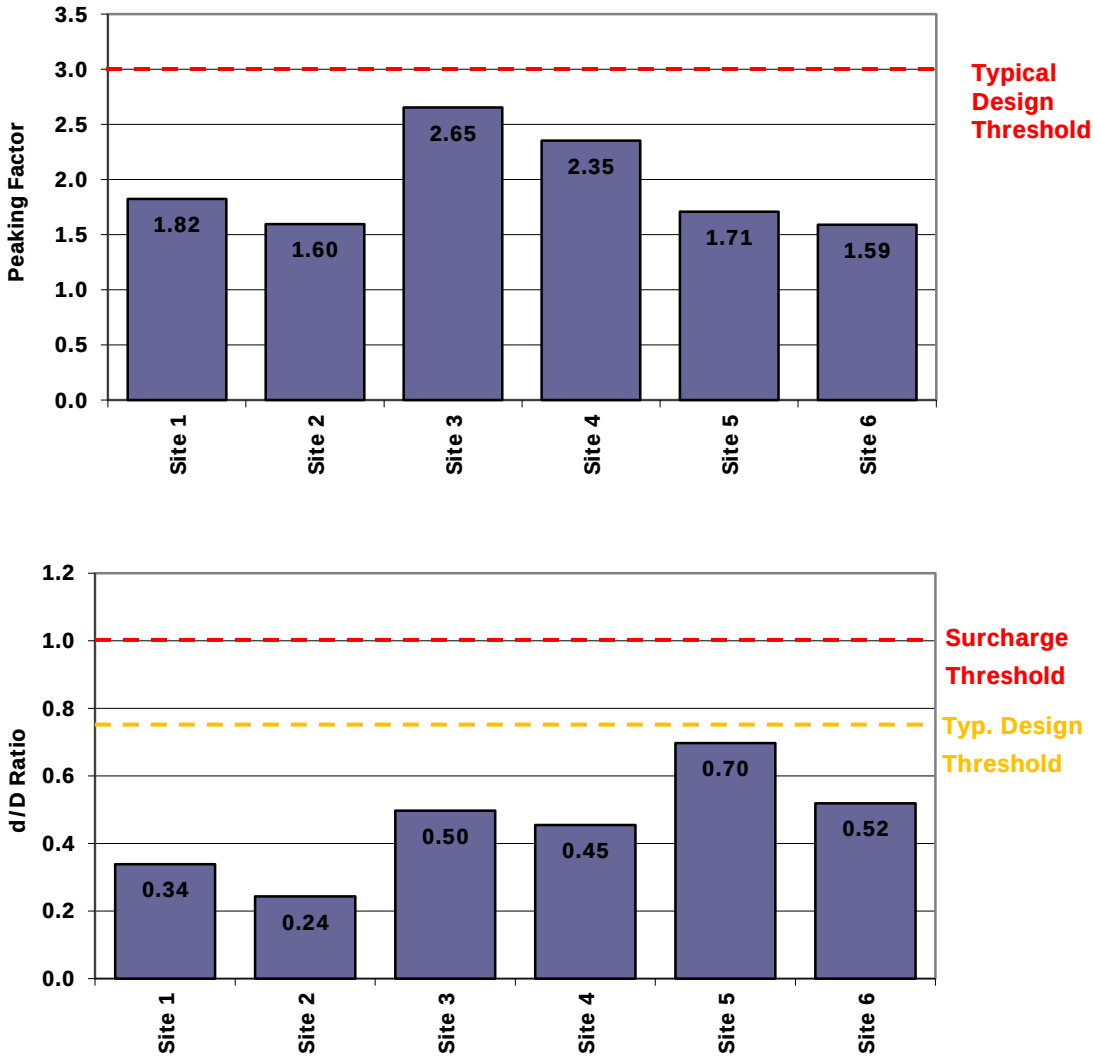


Figure 20. Capacity Bar Graphs: Peaking Factor and d/D Ratio by Site

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 occurs in the basin upstream from Site 6.
   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. The highest normalized infiltration was occurring in the basins upstream from Sites 3 and 4.
2. **I&I Investigation Methods:** Potential I&I investigation methods include the following:
   a. Smoke testing
   b. Mini-basin flow monitoring
   c. Night-time 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.
   d. CCTV inspection
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/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

Temporary Flow Monitoring Study

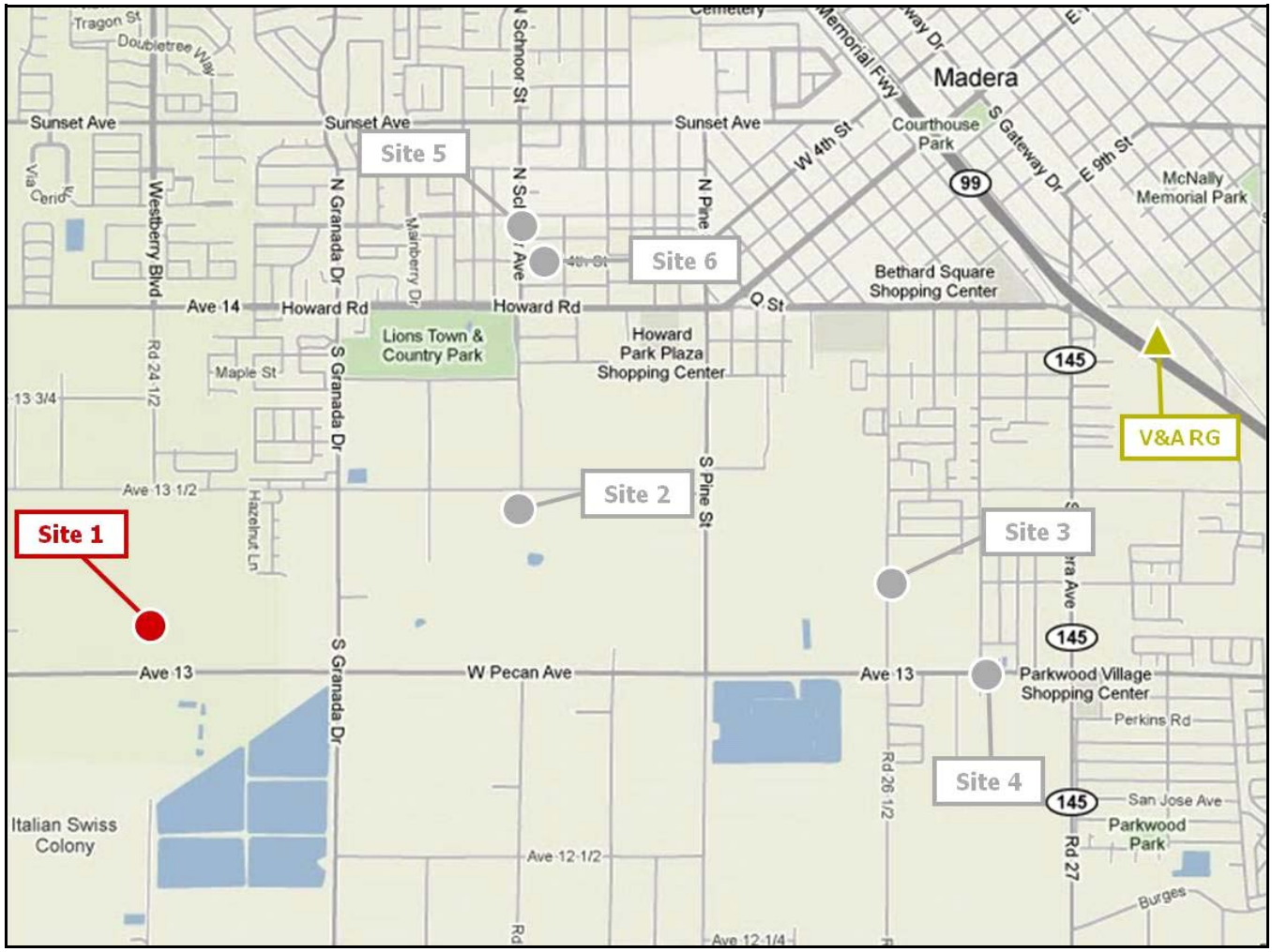
Sanitary Sewer Collection System

Monitoring Site: Site 1

Location: MH ID 6128: Westberry Road (Road 24 1/2), north of Pecan Avenue (Ave 13)

Size / Type Line: 35.25-inch Sanitary Sewer Pipe

Data Summary Report



Site Information Report

Monitoring Site: Site 1

Location: MH ID 6128: Westberry Road (Road 24 1/2), north of Pecan Avenue (Ave 13)

Latitude: 36.940175°

Longitude: -120.101161°

Rim Elevation: 251 *feet above sea level*

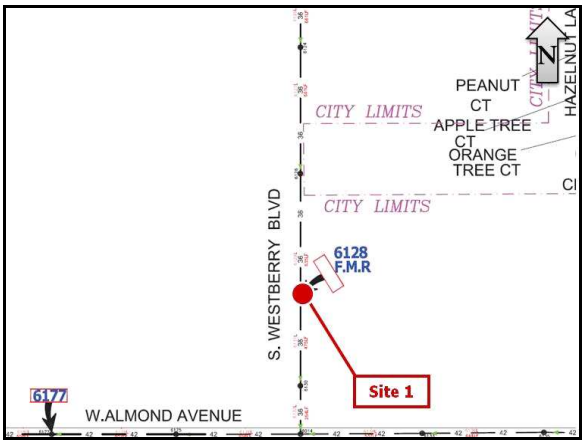
Diameter: 35 *inches*

Average Dry Weather Flow: 1.009 *mgd*

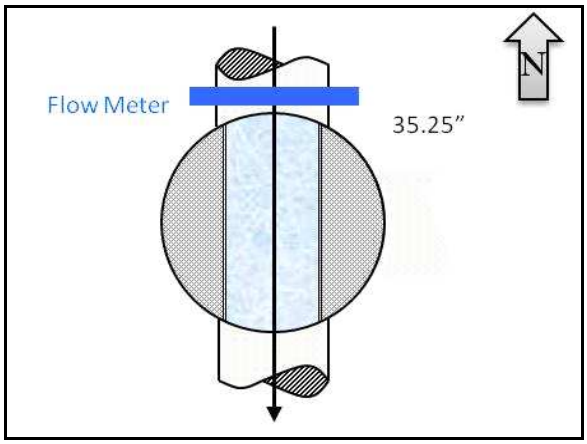
Peak Measured Flow: 1.840 *mgd*

Satellite Map

Sanitary Map

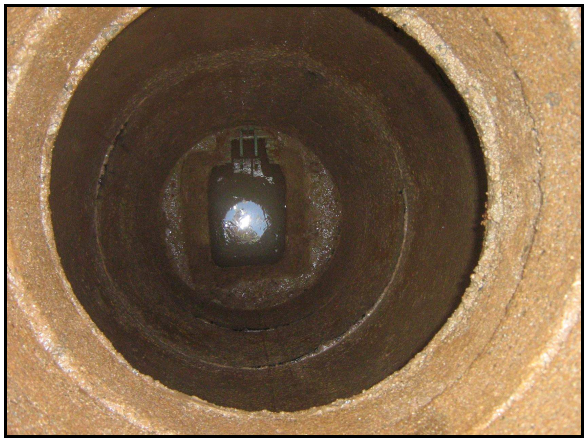


Flow Schematic



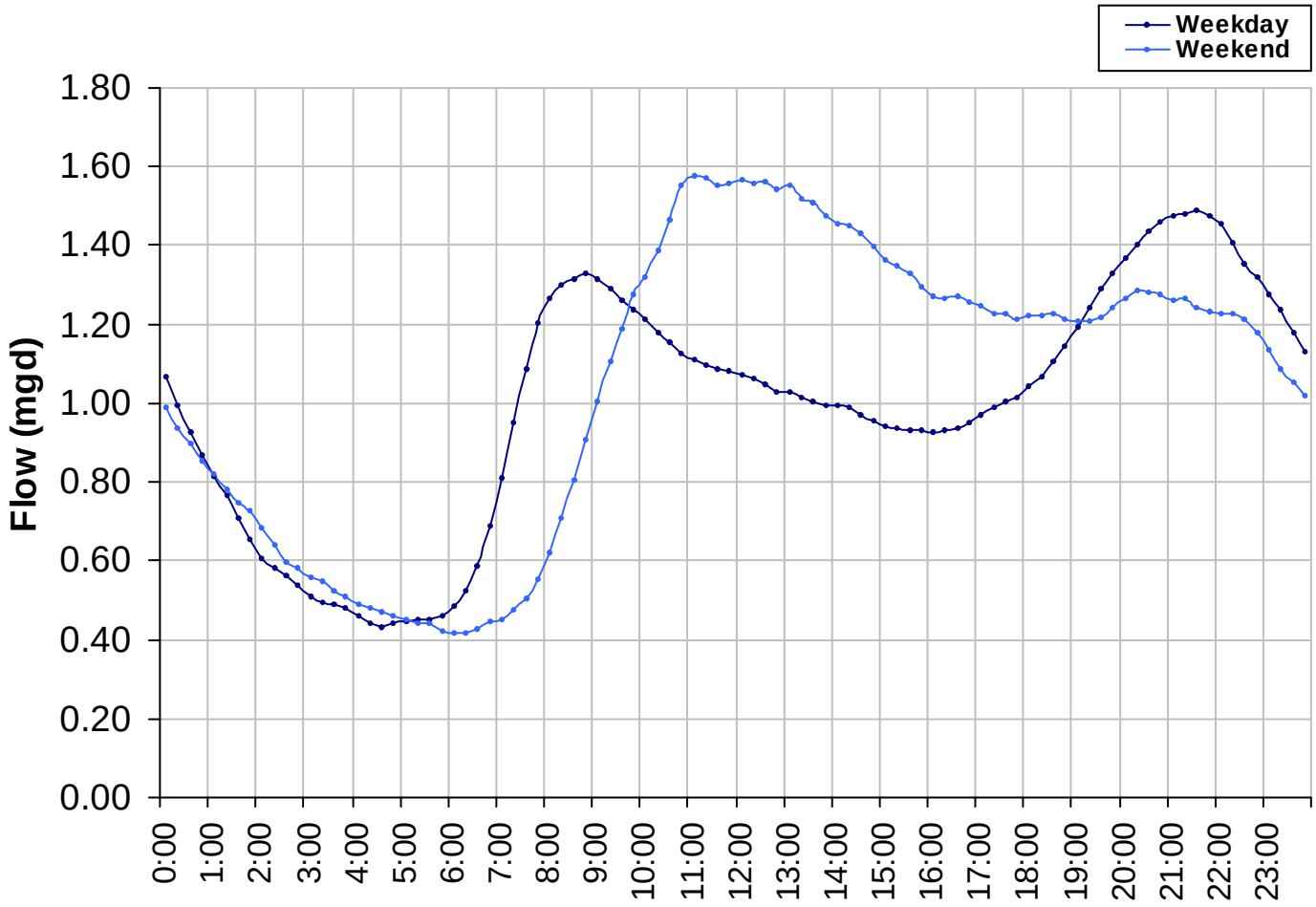
Street View Photo

Plan View Photo

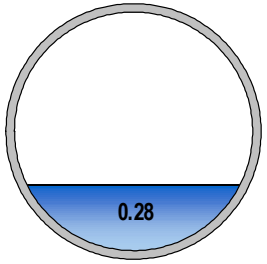


Average Dry Weather Flow

Monitoring Site:
Site 1



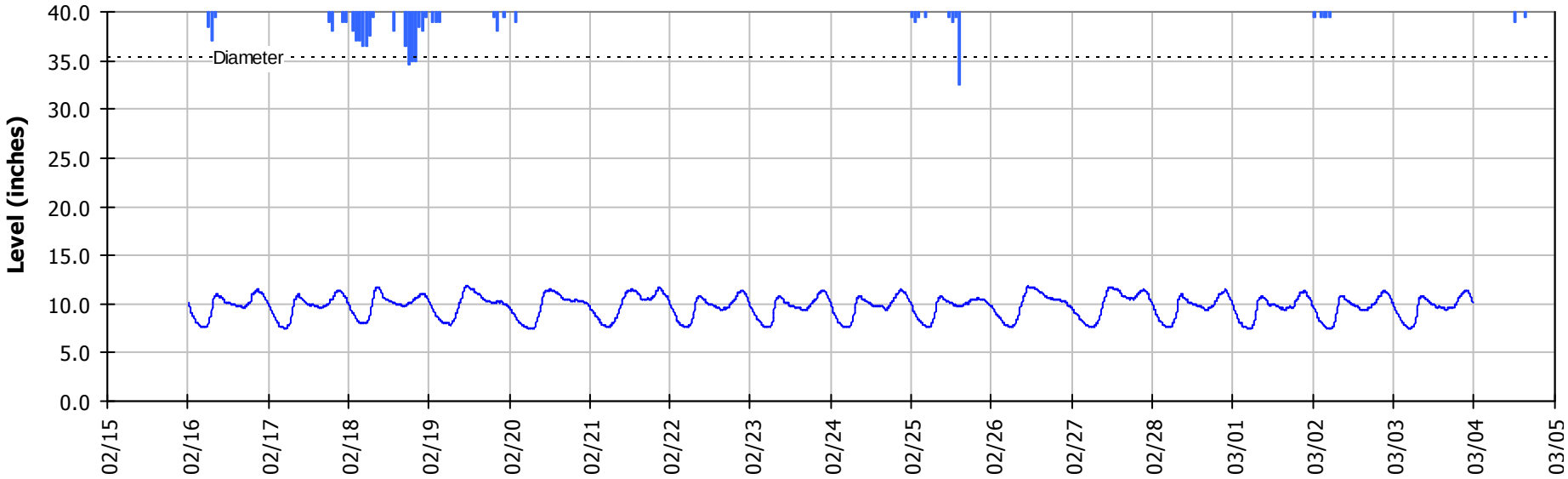
Average Dry Weather Flow:
1.009 *mgd*

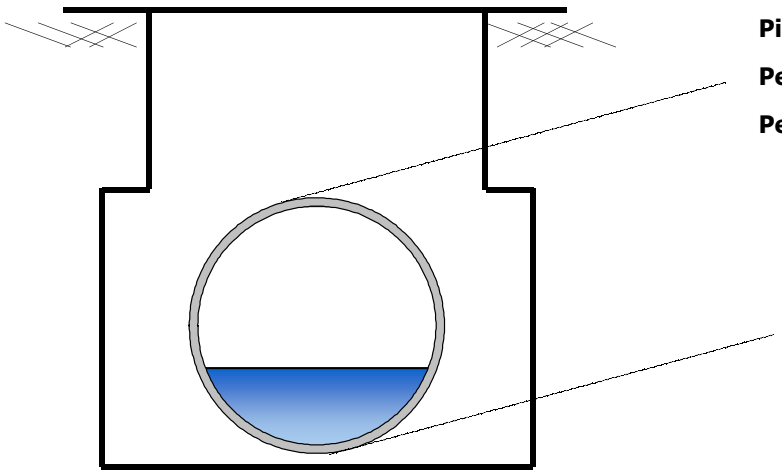


Site Capacity / Surcharge Summary

Monitoring Site:
Site 1

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period

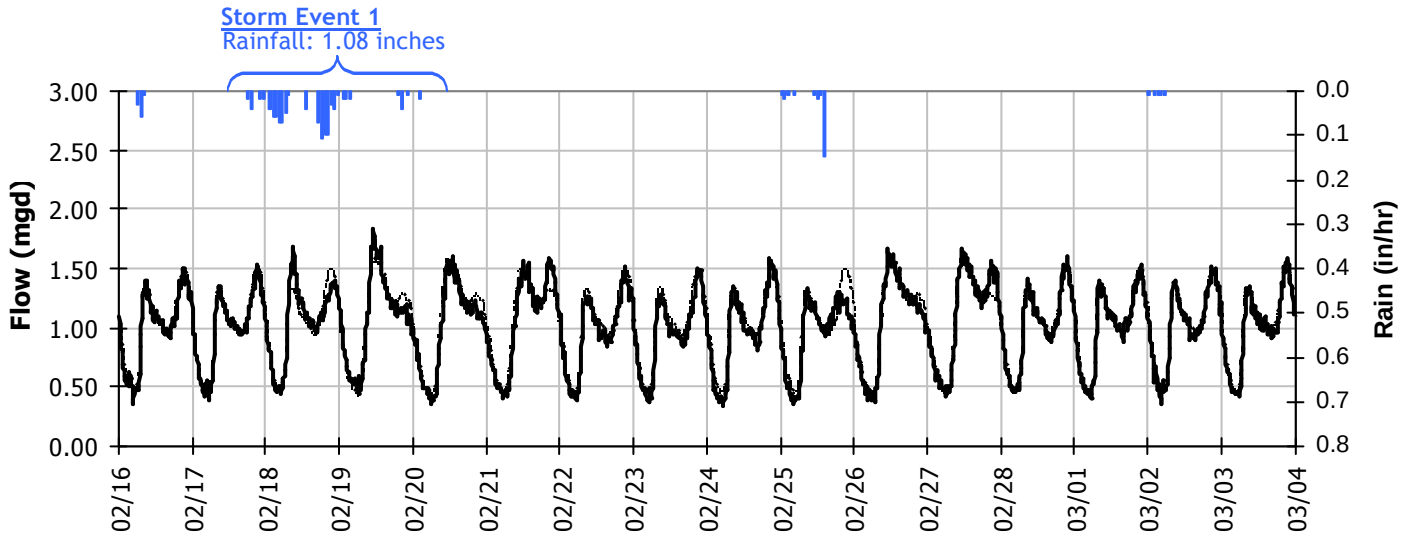




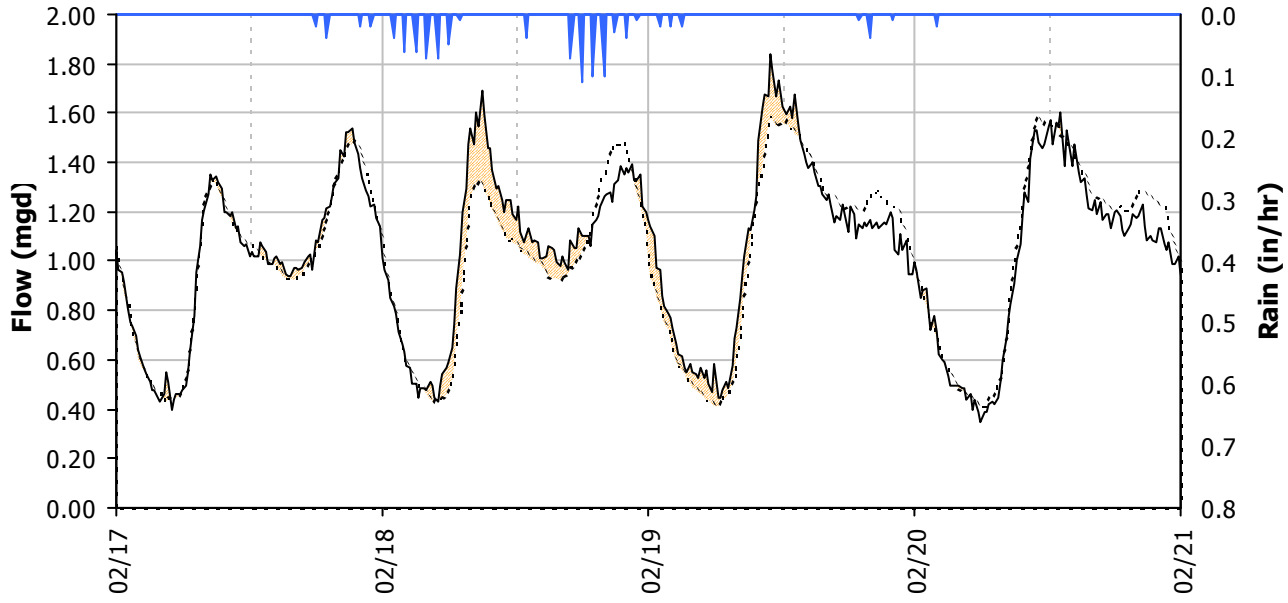
Pipe Diameter:	35.3 *inches*
Peak Measured Level:	11.9 *inches*
Peak d/D Ratio:	0.34

	I/I Summary Event 1	**Monitoring Site:** Site 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

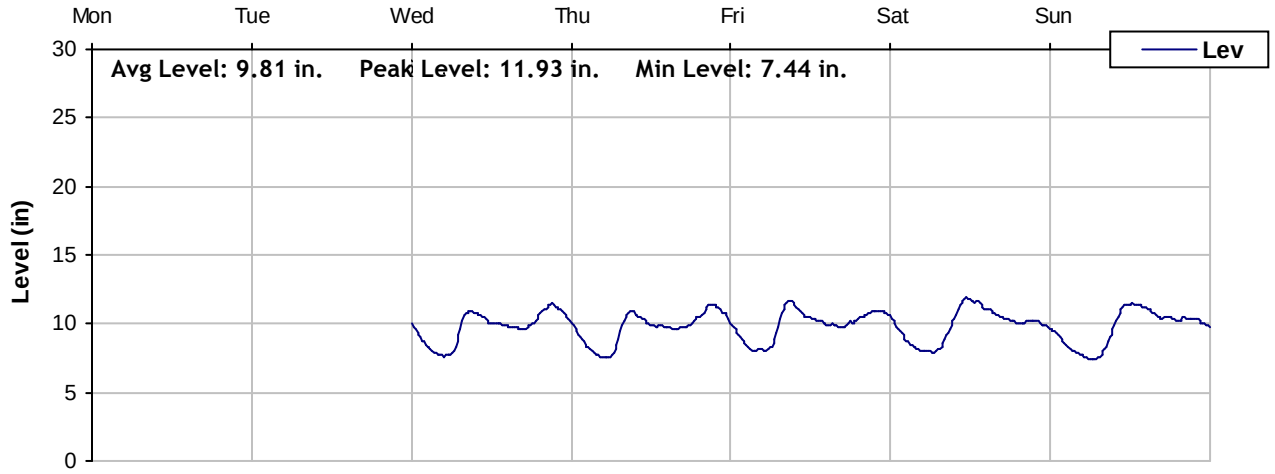


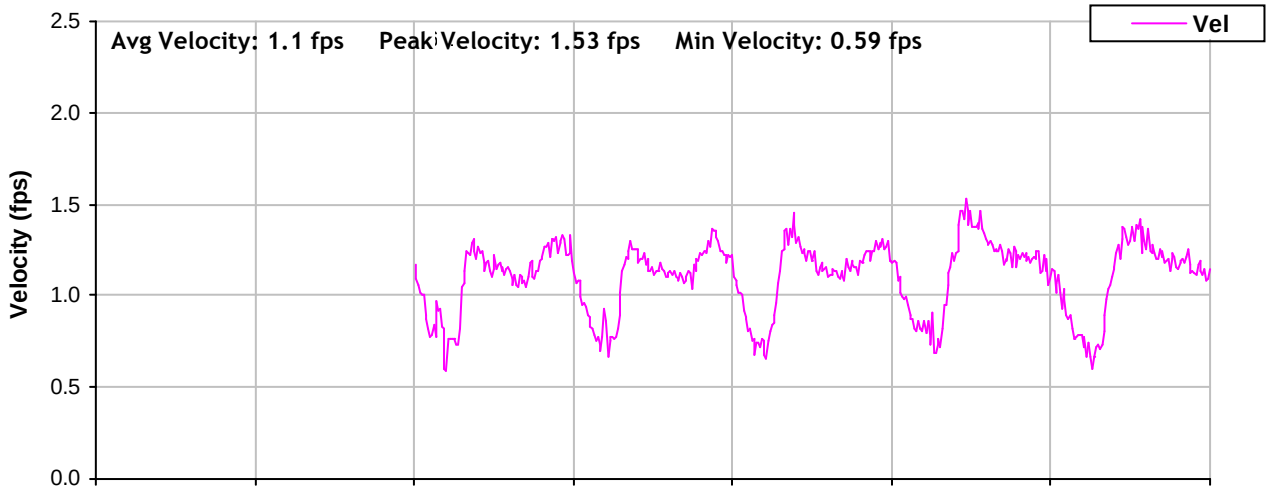
Storm Event #1 I/I Analysis

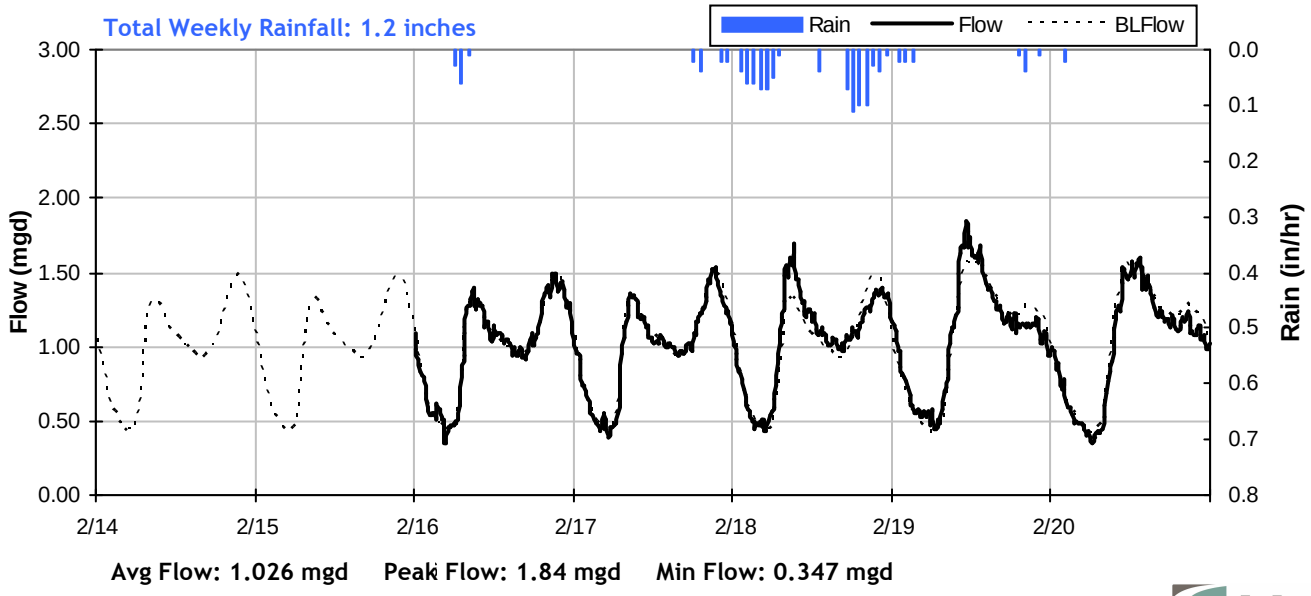
Rainfall:	1.08	*inches*			
Peak Flow:	1.84	*mgd*	**PF:**	1.82	
Peak I/I Rate:	0.38	*mgd*	**Pk I/I:ADWF:**	0.37	
Peak Level:	11.93	*inches*	**d/D Ratio:**	0.34	
Total I/I:	113,000	*gallons*	**I/I per ADWF:**	0.10	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.10	*mgd*	**RDI (% of BL):**	10%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 1

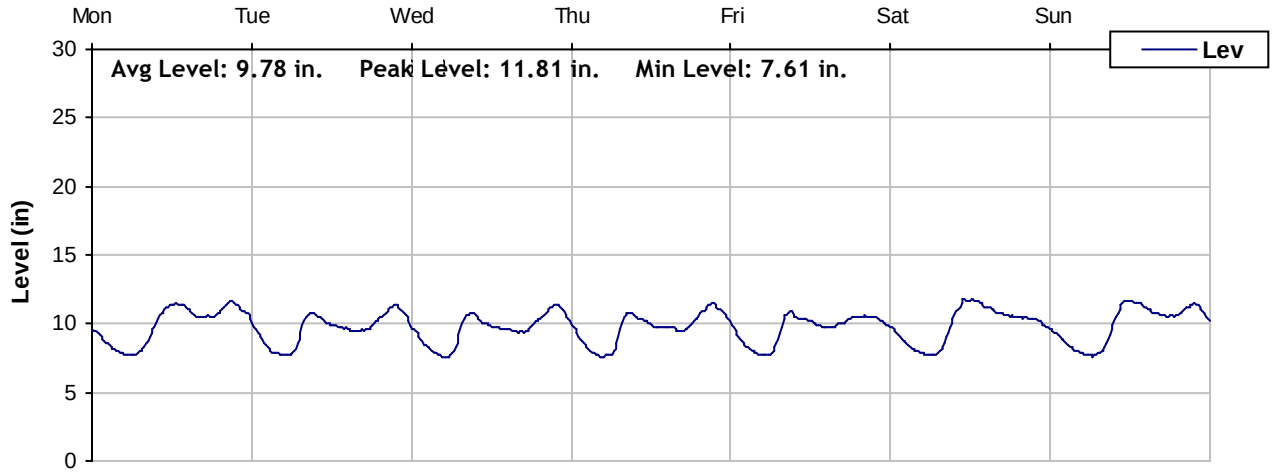


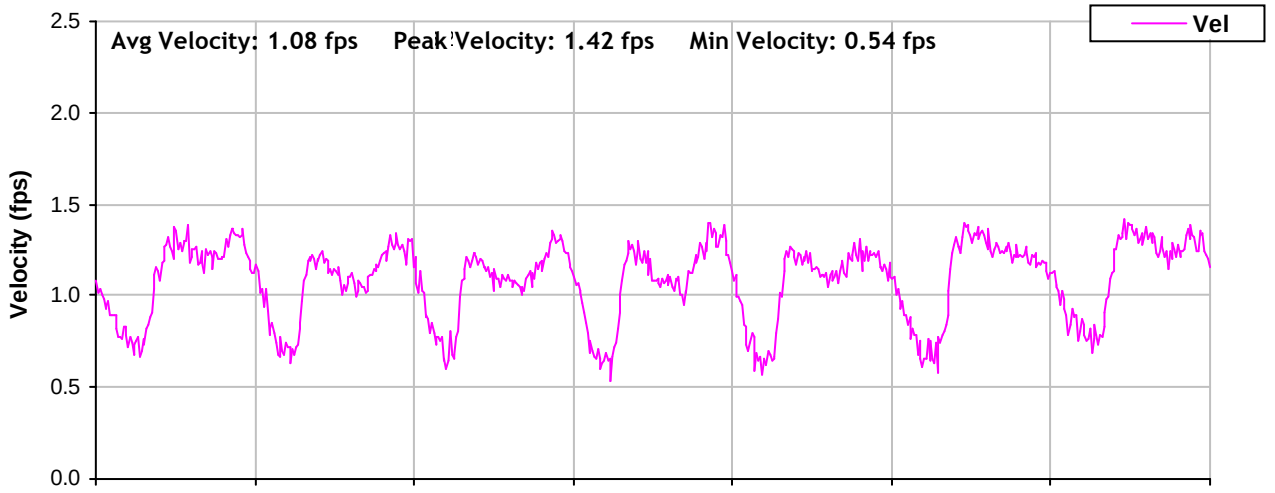


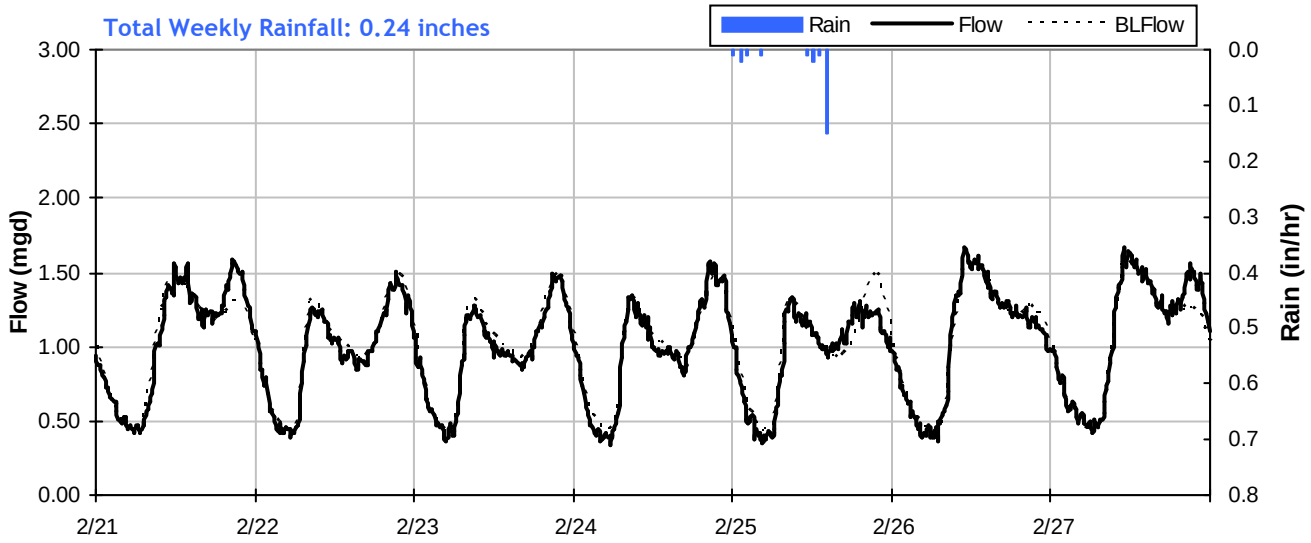


Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 1



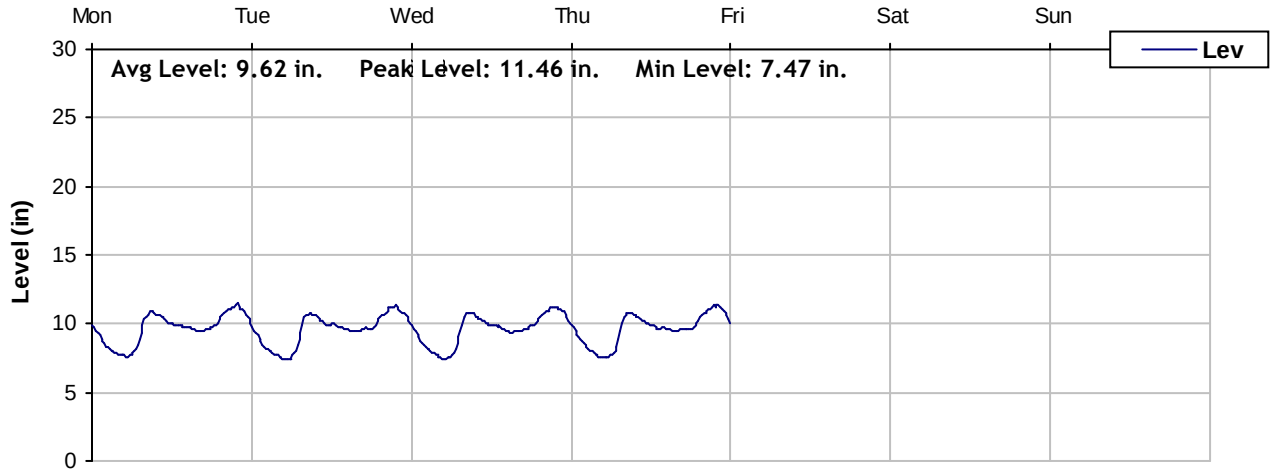


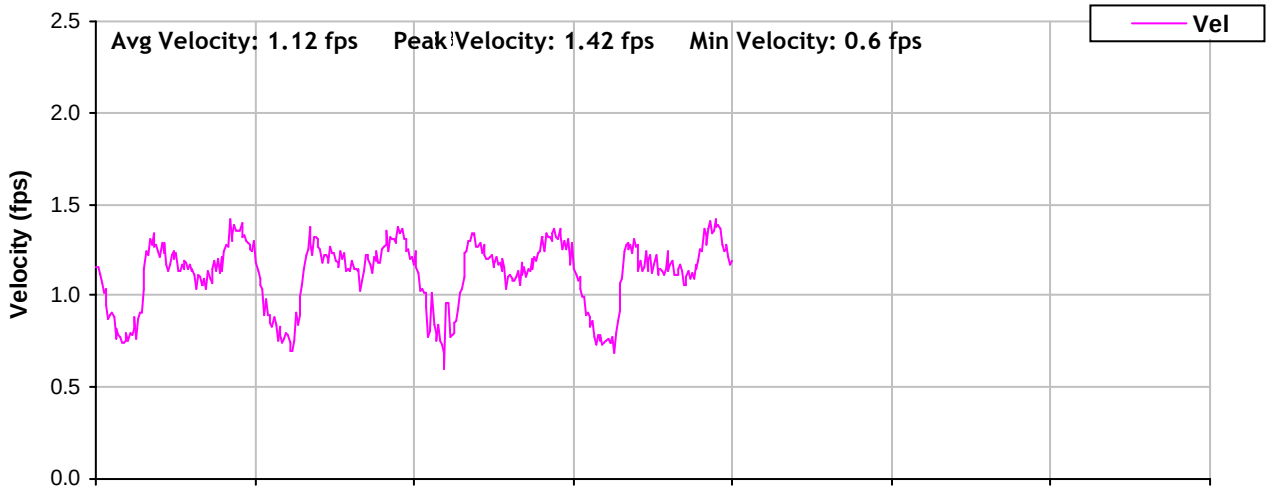


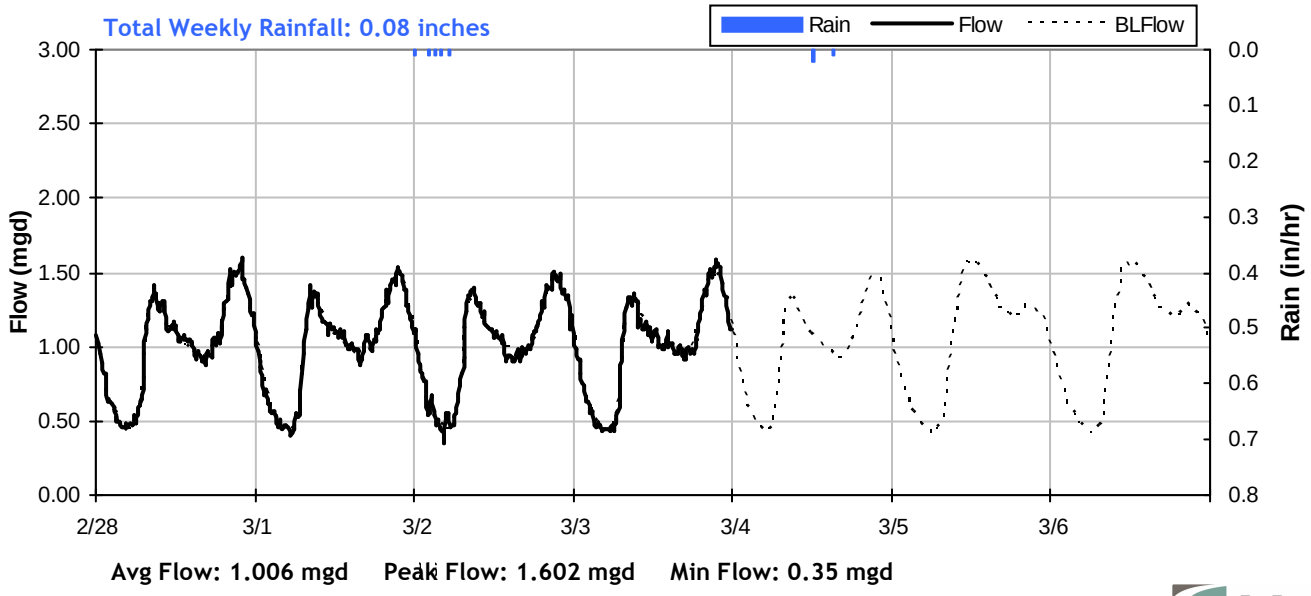
Avg Flow: 1.007 mgd Peak Flow: 1.67 mgd Min Flow: 0.33 mgd

Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 1







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 1

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**9.81**	**1.10**	**1.03**
Weekly Minimum:	**7.47**	**0.67**	**0.39**
Weekly Maximum:	**11.84**	**1.46**	**1.75**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00							9.70	1.08	0.96	9.73	1.09	0.97	9.64	1.07	0.93	10.11	1.16	1.09	9.43	1.09	0.92	**0:00**
1:00							8.91	0.94	0.73	8.95	0.96	0.75	8.96	0.95	0.74	9.36	1.02	0.86	8.96	1.02	0.80	**1:00**
2:00							8.31	0.81	0.56	8.29	0.86	0.59	8.36	0.81	0.57	8.73	0.93	0.70	8.33	0.91	0.63	**2:00**
3:00							7.86	0.90	0.57	7.84	0.76	0.48	8.06	0.73	0.48	8.29	0.84	0.58	7.97	0.78	0.50	**3:00**
4:00							7.69	0.71	0.43	7.60	0.82	0.49	8.08	0.73	0.48	8.07	0.83	0.55	7.72	0.77	0.48	**4:00**
5:00							7.71	0.76	0.47	7.56	0.74	0.44	8.12	0.77	0.51	8.03	0.79	0.52	7.53	0.71	0.42	**5:00**
6:00							8.24	0.76	0.52	8.06	0.81	0.54	8.81	0.94	0.72	7.95	0.76	0.49	7.47	0.67	0.39	**6:00**
7:00							9.88	1.13	1.03	9.59	1.13	0.99	10.49	1.25	1.25	8.32	0.81	0.57	7.68	0.75	0.45	**7:00**
8:00							10.89	1.26	1.33	10.72	1.25	1.29	11.56	1.33	1.54	9.24	1.07	0.88	8.55	0.99	0.72	**8:00**
9:00							10.79	1.24	1.29	10.76	1.24	1.29	11.45	1.36	1.55	10.28	1.22	1.18	9.76	1.16	1.04	**9:00**
10:00							10.56	1.19	1.19	10.38	1.20	1.18	10.81	1.25	1.30	11.37	1.43	1.62	10.89	1.29	1.37	**10:00**
11:00							10.20	1.14	1.09	10.07	1.16	1.09	10.52	1.22	1.22	11.84	1.46	1.75	11.38	1.32	1.48	**11:00**
12:00							10.06	1.18	1.10	9.88	1.13	1.02	10.34	1.17	1.14	11.62	1.39	1.61	11.42	1.36	1.54	**12:00**
13:00							9.98	1.14	1.05	9.89	1.14	1.04	10.16	1.16	1.10	11.44	1.38	1.57	11.34	1.35	1.51	**13:00**
14:00							9.83	1.11	1.00	9.80	1.12	1.00	9.93	1.12	1.03	11.05	1.30	1.40	11.14	1.29	1.41	**14:00**
15:00							9.76	1.08	0.97	9.59	1.10	0.96	9.93	1.13	1.04	10.79	1.26	1.31	10.79	1.24	1.29	**15:00**
16:00							9.61	1.08	0.94	9.72	1.09	0.97	9.77	1.11	0.99	10.48	1.23	1.23	10.44	1.23	1.22	**16:00**
17:00							9.81	1.14	1.03	9.88	1.10	1.00	9.97	1.17	1.08	10.32	1.22	1.19	10.45	1.18	1.17	**17:00**
18:00							10.15	1.14	1.08	10.21	1.18	1.13	10.14	1.15	1.09	10.21	1.21	1.16	10.35	1.20	1.18	**18:00**
19:00							10.85	1.24	1.30	10.61	1.24	1.25	10.44	1.19	1.18	10.10	1.22	1.15	10.30	1.18	1.14	**19:00**
20:00							11.26	1.28	1.43	11.24	1.29	1.43	10.69	1.23	1.26	10.19	1.20	1.15	10.38	1.21	1.18	**20:00**
21:00							11.33	1.29	1.44	11.33	1.33	1.49	10.93	1.26	1.34	10.19	1.22	1.16	10.30	1.12	1.09	**21:00**
22:00							10.97	1.29	1.38	10.94	1.24	1.32	10.94	1.28	1.37	9.99	1.18	1.09	10.17	1.15	1.10	**22:00**
23:00							10.41	1.24	1.23	10.42	1.21	1.19	10.71	1.24	1.27	9.77	1.12	1.00	9.88	1.12	1.02	**23:00**
Average:							**9.78**	**1.09**	**1.01**	**9.71**	**1.09**	**1.00**	**9.95**	**1.11**	**1.05**	**9.91**	**1.13**	**1.07**	**9.69**	**1.09**	**1.00**	**Ave**
Minimum:							**7.69**	**0.71**	**0.43**	**7.56**	**0.74**	**0.44**	**8.06**	**0.73**	**0.48**	**7.95**	**0.76**	**0.49**	**7.47**	**0.67**	**0.39**	**Min**
Maximum:							**11.33**	**1.29**	**1.44**	**11.33**	**1.33**	**1.49**	**11.56**	**1.36**	**1.55**	**11.84**	**1.46**	**1.75**	**11.42**	**1.36**	**1.54**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 1

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**9.78**	**1.08**	**1.01**
Weekly Minimum:	**7.64**	**0.63**	**0.38**
Weekly Maximum:	**11.70**	**1.38**	**1.62**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	9.44	1.03	0.88	9.72	1.09	0.97	9.50	1.11	0.95	9.64	1.07	0.94	9.58	1.04	0.91	9.50	1.06	0.91	9.43	1.09	0.92	**0:00**
1:00	8.96	0.96	0.75	8.92	0.99	0.77	8.79	0.96	0.72	8.82	0.94	0.71	8.78	0.90	0.68	8.93	0.94	0.73	8.98	1.00	0.78	**1:00**
2:00	8.48	0.90	0.64	8.21	0.82	0.55	8.30	0.83	0.57	8.19	0.74	0.50	8.28	0.74	0.51	8.54	0.85	0.61	8.50	0.86	0.62	**2:00**
3:00	8.07	0.78	0.52	7.85	0.72	0.45	7.88	0.76	0.48	7.82	0.67	0.42	7.93	0.67	0.43	8.07	0.77	0.51	8.12	0.90	0.60	**3:00**
4:00	7.84	0.79	0.50	7.74	0.71	0.44	7.66	0.67	0.41	7.66	0.64	0.39	7.73	0.64	0.39	7.88	0.67	0.43	7.88	0.82	0.52	**4:00**
5:00	7.70	0.73	0.44	7.74	0.69	0.42	7.64	0.70	0.42	7.67	0.63	0.38	7.72	0.66	0.40	7.69	0.70	0.43	7.74	0.78	0.48	**5:00**
6:00	7.77	0.72	0.45	8.19	0.78	0.53	8.17	0.81	0.55	8.14	0.80	0.54	8.19	0.75	0.51	7.71	0.65	0.40	7.69	0.77	0.47	**6:00**
7:00	8.10	0.79	0.52	9.62	1.09	0.95	9.80	1.13	1.02	9.78	1.11	1.00	9.56	1.09	0.95	8.13	0.78	0.52	7.96	0.78	0.50	**7:00**
8:00	8.90	0.99	0.77	10.68	1.21	1.24	10.67	1.19	1.22	10.74	1.27	1.31	10.72	1.25	1.29	9.15	0.98	0.80	8.77	0.99	0.75	**8:00**
9:00	9.93	1.14	1.04	10.66	1.19	1.22	10.66	1.20	1.23	10.60	1.25	1.26	10.62	1.21	1.23	10.45	1.28	1.27	10.07	1.20	1.12	**9:00**
10:00	10.79	1.27	1.32	10.37	1.20	1.18	10.20	1.16	1.11	10.31	1.20	1.17	10.34	1.20	1.17	11.41	1.31	1.48	11.20	1.33	1.46	**10:00**
11:00	11.29	1.27	1.42	10.03	1.13	1.06	9.99	1.10	1.02	10.11	1.16	1.10	10.23	1.20	1.15	11.69	1.36	1.60	11.67	1.38	1.62	**11:00**
12:00	11.43	1.29	1.46	9.90	1.12	1.02	9.78	1.10	0.98	9.81	1.08	0.97	10.05	1.15	1.08	11.70	1.34	1.57	11.58	1.37	1.58	**12:00**
13:00	11.36	1.32	1.49	9.72	1.04	0.92	9.66	1.10	0.96	9.77	1.07	0.96	9.82	1.11	0.99	11.51	1.33	1.53	11.47	1.31	1.49	**13:00**
14:00	11.07	1.23	1.33	9.61	1.10	0.96	9.59	1.08	0.94	9.77	1.08	0.97	9.74	1.11	0.98	11.20	1.29	1.42	11.16	1.34	1.47	**14:00**
15:00	10.64	1.22	1.24	9.46	1.04	0.89	9.42	1.07	0.90	9.65	1.08	0.94	9.82	1.10	0.99	10.99	1.26	1.35	10.84	1.30	1.36	**15:00**
16:00	10.51	1.20	1.20	9.52	1.04	0.89	9.39	1.04	0.88	9.47	1.00	0.85	10.03	1.16	1.08	10.74	1.25	1.29	10.67	1.26	1.29	**16:00**
17:00	10.53	1.21	1.21	9.64	1.12	0.98	9.56	1.10	0.95	9.71	1.10	0.97	10.22	1.18	1.13	10.61	1.27	1.29	10.57	1.21	1.23	**17:00**
18:00	10.61	1.21	1.23	10.08	1.16	1.09	10.10	1.15	1.09	10.22	1.20	1.14	10.51	1.23	1.23	10.54	1.22	1.23	10.58	1.24	1.25	**18:00**
19:00	10.98	1.25	1.34	10.48	1.22	1.21	10.45	1.19	1.18	10.76	1.26	1.31	10.49	1.22	1.22	10.44	1.22	1.20	10.81	1.25	1.30	**19:00**
20:00	11.54	1.35	1.55	11.02	1.29	1.38	11.03	1.29	1.39	11.27	1.34	1.49	10.53	1.23	1.23	10.41	1.22	1.20	11.18	1.32	1.44	**20:00**
21:00	11.46	1.33	1.51	11.31	1.29	1.45	11.34	1.31	1.46	11.38	1.31	1.48	10.51	1.23	1.23	10.33	1.21	1.17	11.42	1.32	1.50	**21:00**
22:00	11.06	1.27	1.37	11.01	1.22	1.31	11.11	1.28	1.39	11.02	1.34	1.44	10.20	1.16	1.11	10.11	1.18	1.12	11.15	1.32	1.44	**22:00**
23:00	10.64	1.15	1.17	10.29	1.27	1.23	10.43	1.17	1.16	10.43	1.19	1.17	9.91	1.13	1.03	9.76	1.13	1.01	10.43	1.21	1.20	**23:00**
Average:	**9.96**	**1.10**	**1.06**	**9.66**	**1.06**	**0.96**	**9.63**	**1.06**	**0.96**	**9.70**	**1.06**	**0.98**	**9.64**	**1.06**	**0.95**	**9.89**	**1.09**	**1.04**	**9.99**	**1.14**	**1.10**	**Ave**
Minimum:	**7.70**	**0.72**	**0.44**	**7.74**	**0.69**	**0.42**	**7.64**	**0.67**	**0.41**	**7.66**	**0.63**	**0.38**	**7.72**	**0.64**	**0.39**	**7.69**	**0.65**	**0.40**	**7.69**	**0.77**	**0.47**	**Min**
Maximum:	**11.54**	**1.35**	**1.55**	**11.31**	**1.29**	**1.45**	**11.34**	**1.31**	**1.46**	**11.38**	**1.34**	**1.49**	**10.72**	**1.25**	**1.29**	**11.70**	**1.36**	**1.60**	**11.67**	**1.38**	**1.62**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

Monitoring Site:

Site 1

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**9.62**	**1.12**	**1.01**
Weekly Minimum:	**7.50**	**0.72**	**0.42**
Weekly Maximum:	**11.31**	**1.39**	**1.55**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	9.63	1.13	0.99	9.45	1.12	0.95	9.54	1.14	0.98	9.55	1.11	0.96										**0:00**
1:00	8.84	0.97	0.74	8.62	0.96	0.70	8.72	1.00	0.75	8.78	0.98	0.74										**1:00**
2:00	8.21	0.86	0.58	8.08	0.87	0.57	8.13	0.86	0.58	8.20	0.87	0.59										**2:00**
3:00	7.84	0.78	0.49	7.75	0.80	0.50	7.76	0.79	0.49	7.82	0.76	0.47										**3:00**
4:00	7.71	0.76	0.47	7.53	0.77	0.46	7.52	0.75	0.44	7.58	0.75	0.45										**4:00**
5:00	7.66	0.82	0.50	7.50	0.72	0.42	7.54	0.83	0.49	7.57	0.76	0.45										**5:00**
6:00	8.12	0.86	0.58	8.00	0.91	0.59	8.10	0.92	0.61	8.01	0.82	0.54										**6:00**
7:00	9.59	1.16	1.01	9.65	1.17	1.04	9.72	1.15	1.03	9.67	1.17	1.04										**7:00**
8:00	10.76	1.30	1.35	10.71	1.30	1.34	10.77	1.32	1.37	10.75	1.27	1.31										**8:00**
9:00	10.73	1.26	1.30	10.61	1.29	1.31	10.63	1.29	1.32	10.62	1.25	1.27										**9:00**
10:00	10.45	1.21	1.20	10.20	1.21	1.15	10.27	1.23	1.19	10.30	1.18	1.15										**10:00**
11:00	10.10	1.21	1.14	9.90	1.23	1.12	9.99	1.20	1.11	10.02	1.17	1.09										**11:00**
12:00	9.94	1.17	1.08	9.91	1.20	1.09	9.86	1.18	1.07	9.83	1.16	1.05										**12:00**
13:00	9.83	1.17	1.06	9.71	1.18	1.04	9.71	1.14	1.00	9.68	1.14	1.00										**13:00**
14:00	9.73	1.14	1.01	9.53	1.16	1.00	9.44	1.10	0.93	9.63	1.19	1.04										**14:00**
15:00	9.56	1.08	0.93	9.45	1.12	0.95	9.44	1.10	0.93	9.48	1.12	0.96										**15:00**
16:00	9.46	1.08	0.92	9.57	1.16	1.01	9.48	1.14	0.97	9.54	1.11	0.95										**16:00**
17:00	9.65	1.13	0.99	9.62	1.18	1.03	9.74	1.16	1.03	9.62	1.11	0.97										**17:00**
18:00	9.92	1.17	1.07	9.88	1.20	1.09	10.04	1.22	1.14	9.83	1.15	1.04										**18:00**
19:00	10.55	1.24	1.25	10.51	1.29	1.29	10.56	1.31	1.32	10.53	1.30	1.31										**19:00**
20:00	11.07	1.36	1.47	11.03	1.30	1.40	10.99	1.32	1.41	10.99	1.35	1.45										**20:00**
21:00	11.31	1.36	1.53	11.24	1.33	1.47	11.22	1.34	1.48	11.30	1.39	1.55										**21:00**
22:00	11.08	1.31	1.42	10.89	1.31	1.38	11.00	1.29	1.38	11.16	1.29	1.41										**22:00**
23:00	10.42	1.27	1.25	10.32	1.21	1.18	10.35	1.24	1.21	10.44	1.21	1.20										**23:00**
Average:	**9.67**	**1.12**	**1.01**	**9.57**	**1.12**	**1.00**	**9.60**	**1.12**	**1.01**	**9.62**	**1.11**	**1.00**										**Ave**
Minimum:	**7.66**	**0.76**	**0.47**	**7.50**	**0.72**	**0.42**	**7.52**	**0.75**	**0.44**	**7.57**	**0.75**	**0.45**										**Min**
Maximum:	**11.31**	**1.36**	**1.53**	**11.24**	**1.33**	**1.47**	**11.22**	**1.34**	**1.48**	**11.30**	**1.39**	**1.55**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Temporary Flow Monitoring Study

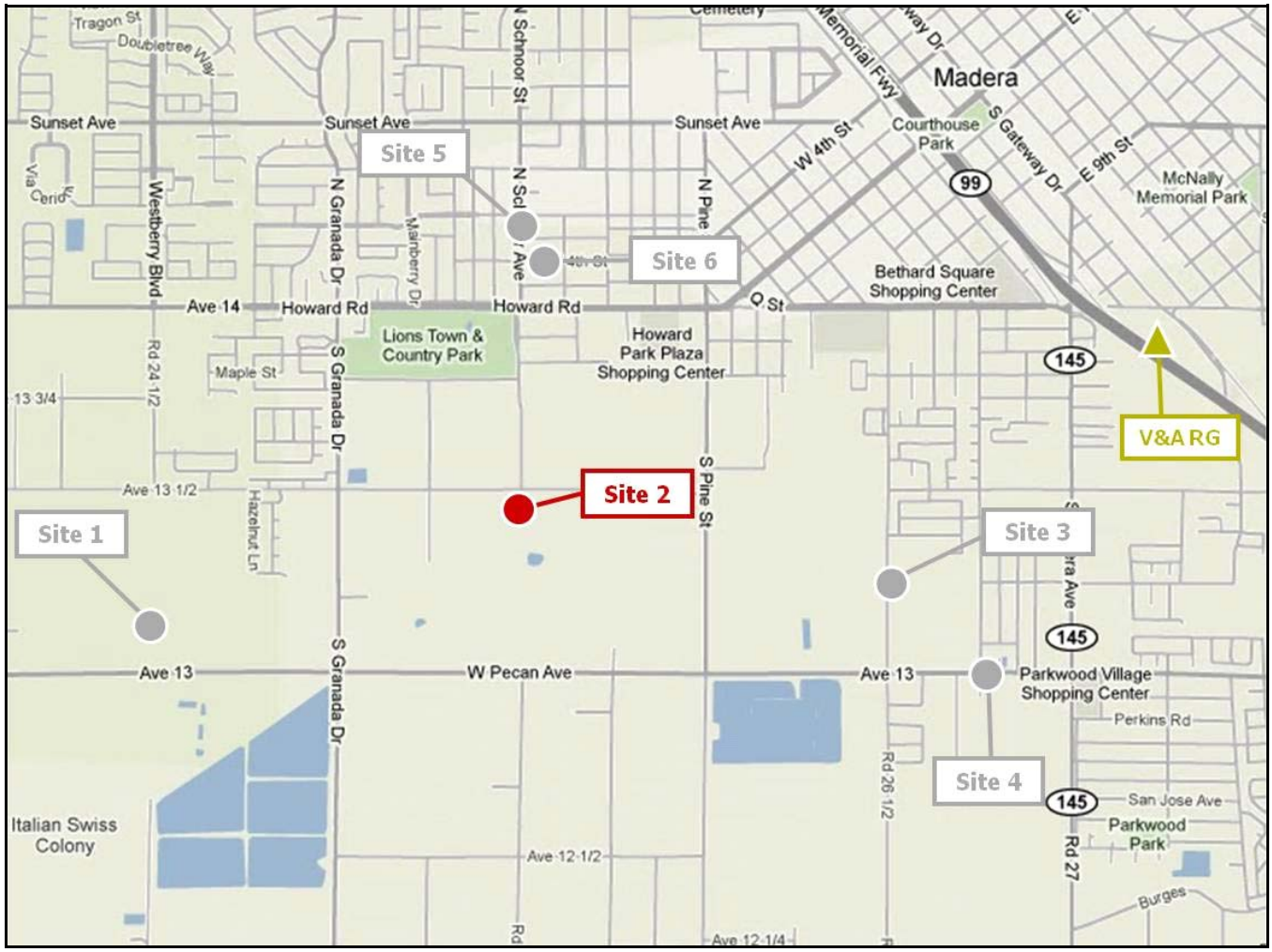
Sanitary Sewer Collection System

Monitoring Site: Site 2

Location: MH ID 6000: Schnoor Avenue, south of Almond Avenue

Size / Type Line: 42-inch Sanitary Sewer Pipe

Data Summary Report



Site Information Report

Monitoring Site: Site 2

Location: MH ID 6000: Schnoor Avenue, south of Almond Avenue

Latitude: 36.944845°

Longitude: -120.083243°

Rim Elevation: 258 *feet above sea level*

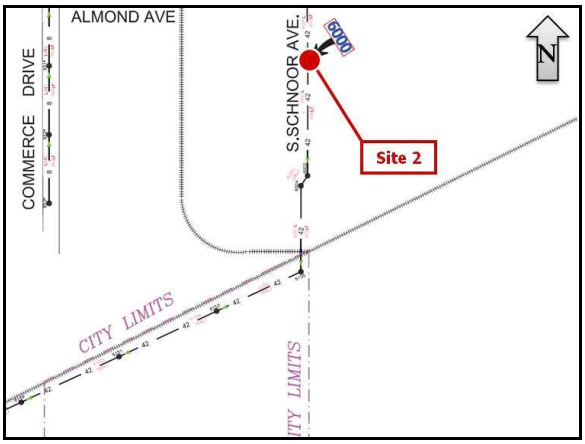
Diameter: 42 *inches*

Average Dry Weather Flow: 2.931 *mgd*

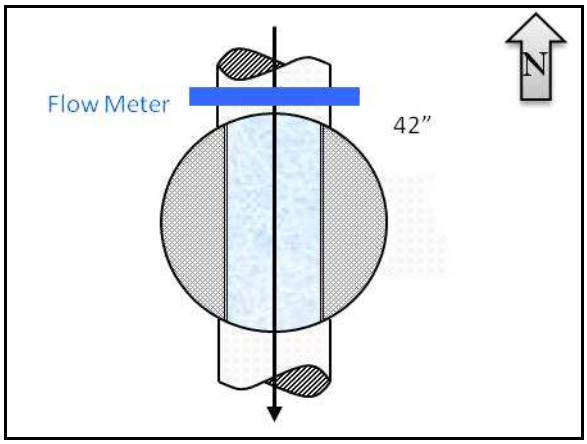
Peak Measured Flow: 4.680 *mgd*

Satellite Map

Sanitary Map

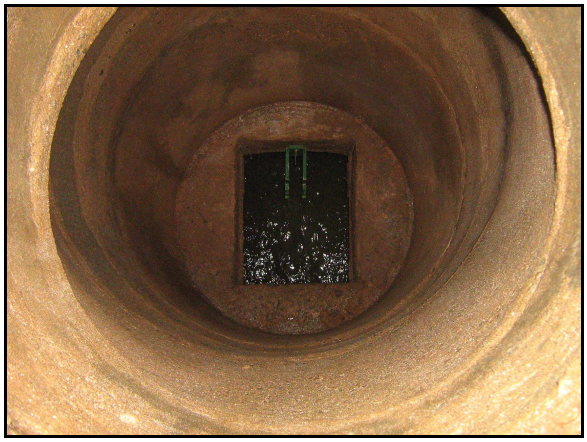


Flow Schematic



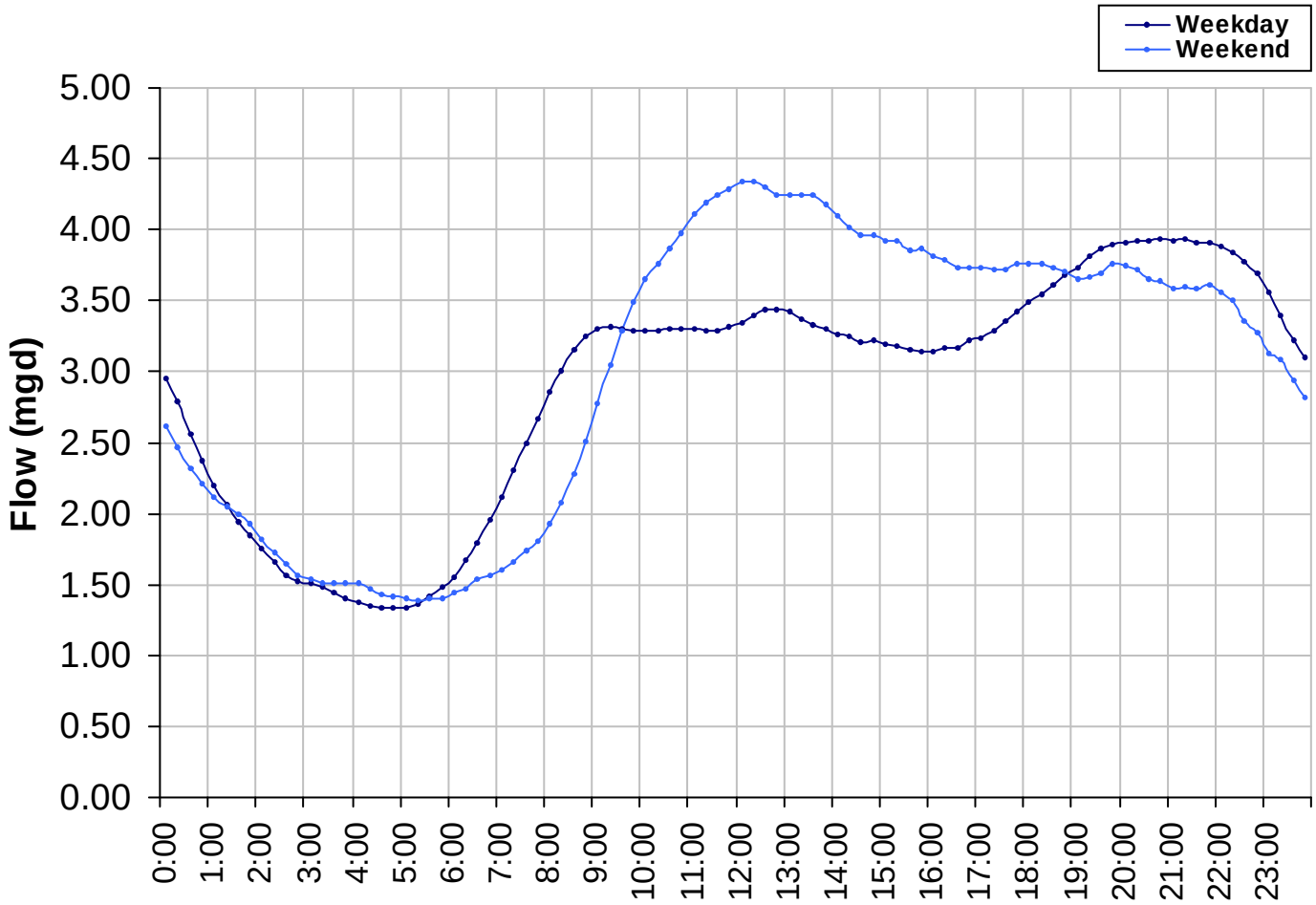
Street View Photo

Plan View Photo

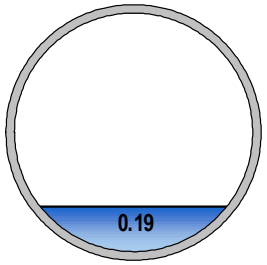


Average Dry Weather Flow

Monitoring Site:
Site 2



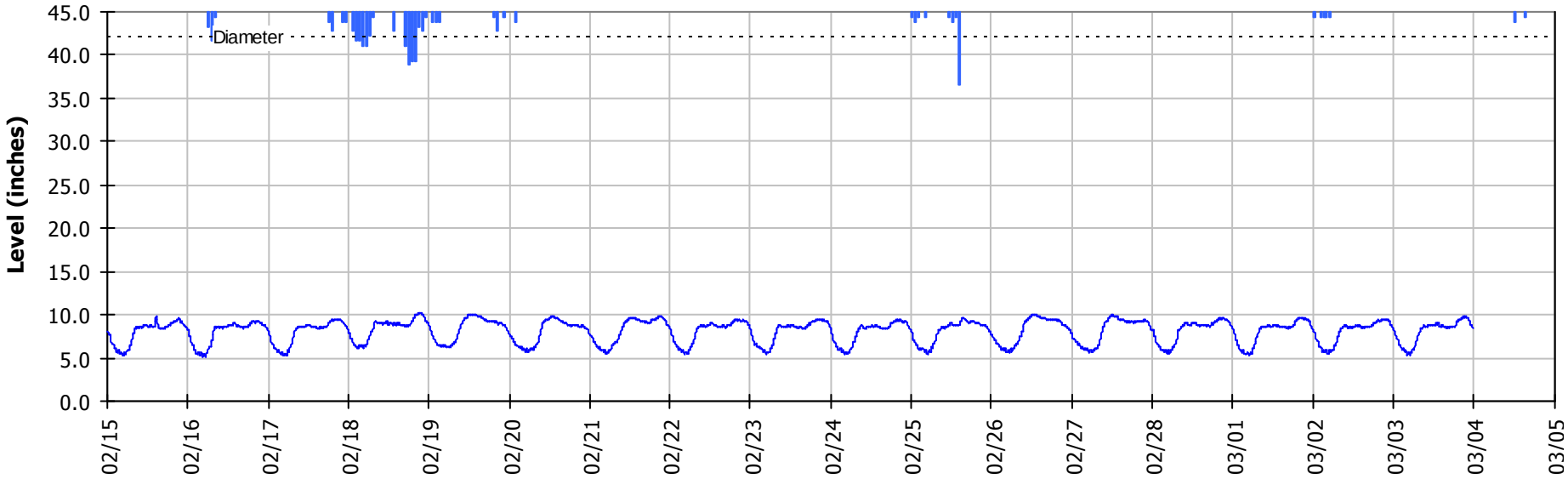
Average Dry Weather Flow:
2.931 *mgd*

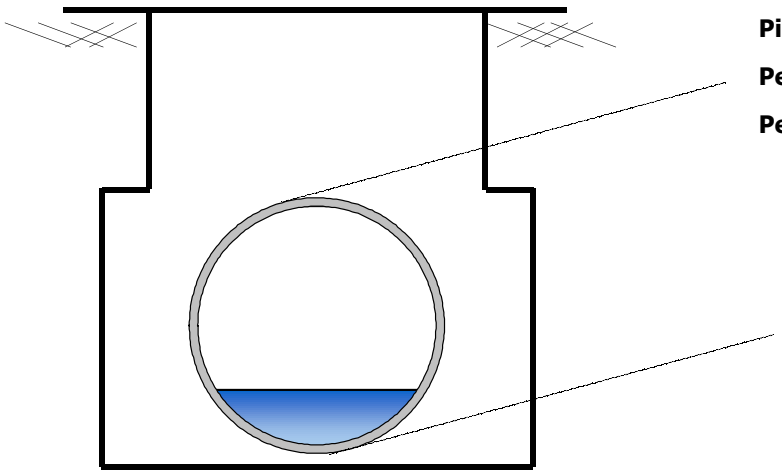


Site Capacity / Surcharge Summary

Monitoring Site:
Site 2

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 42 *inches*

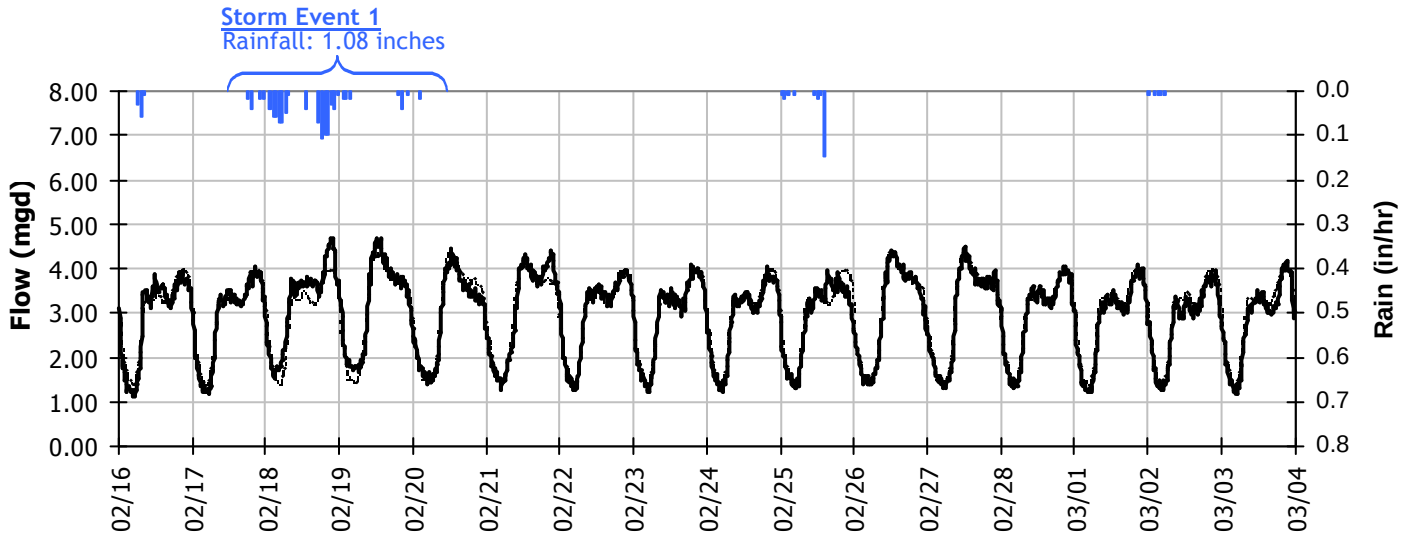
Peak Measured Level: 10.2 *inches*

Peak d/D Ratio: 0.24

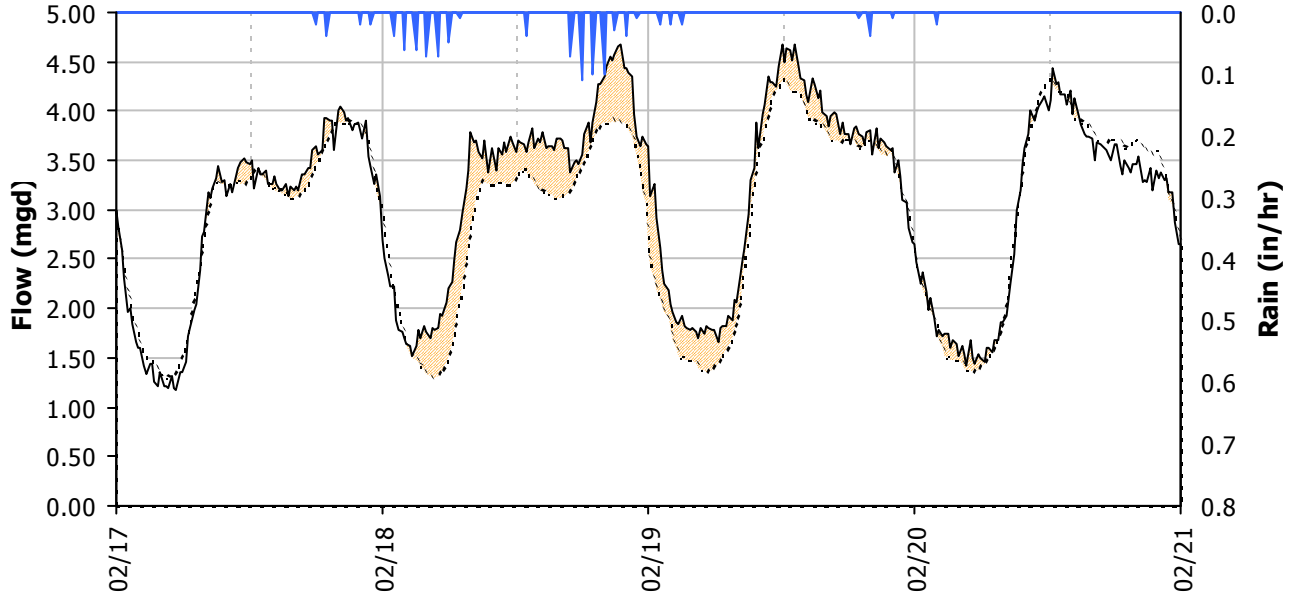
I/I Summary
Event 1

Monitoring Site:
Site 2

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

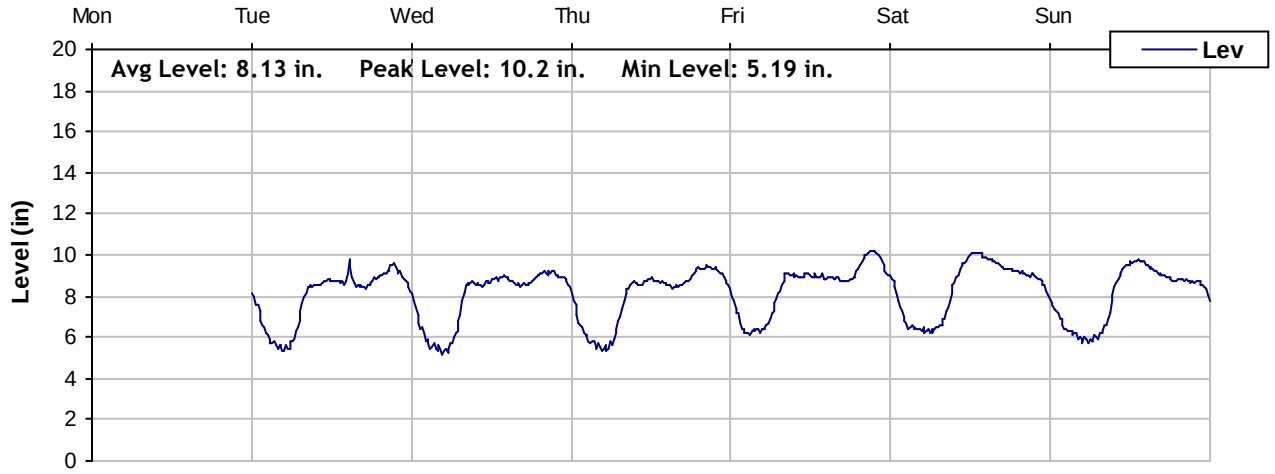


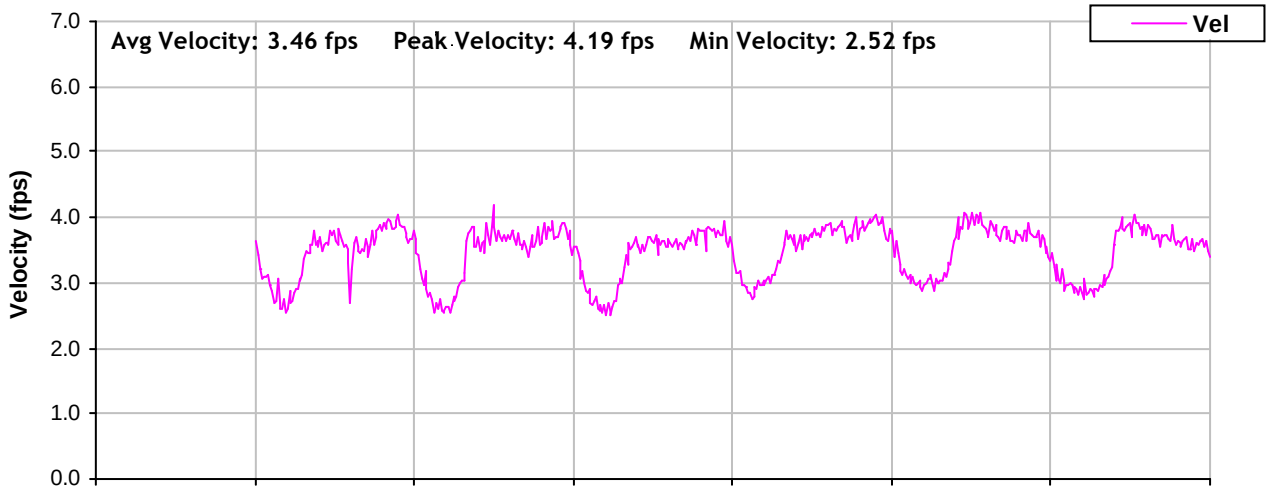
Storm Event #1 I/I Analysis

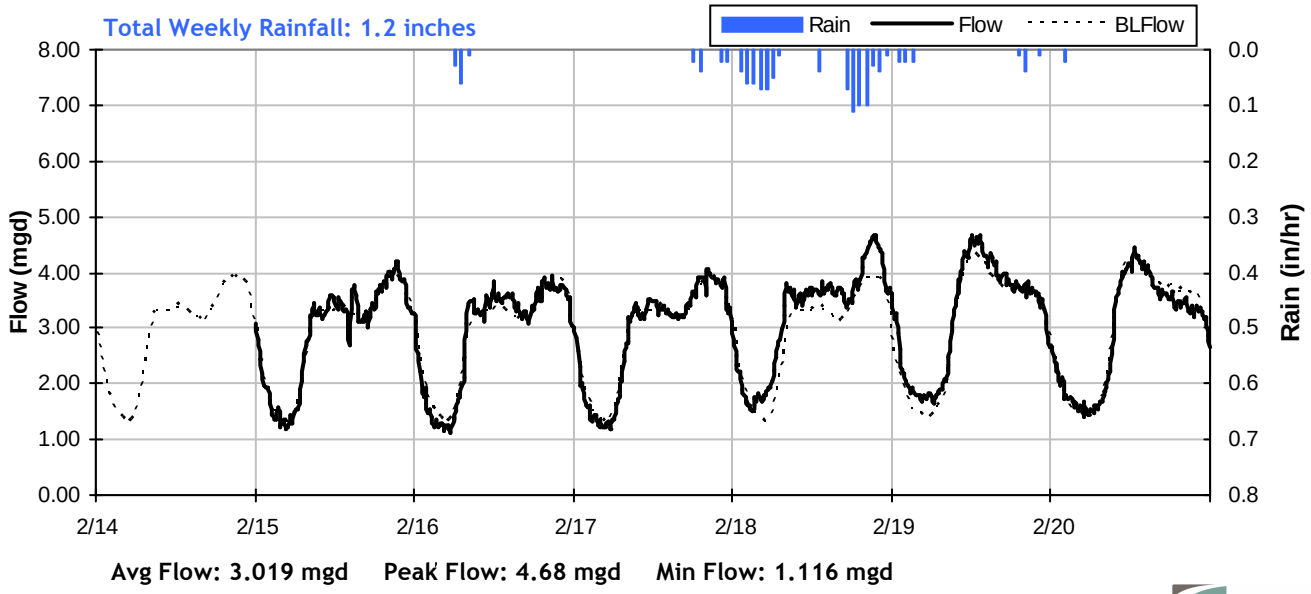
Rainfall:	1.08	*inches*			
Peak Flow:	4.68	*mgd*	**PF:**	1.60	
Peak I/I Rate:	1.02	*mgd*	**Pk I/I:ADWF:**	0.35	
Peak Level:	10.20	*inches*	**d/D Ratio:**	0.24	
Total I/I:	619,000	*gallons*	**I/I per ADWF:**	0.20	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.29	*mgd*	**RDI (% of BL):**	10%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 2

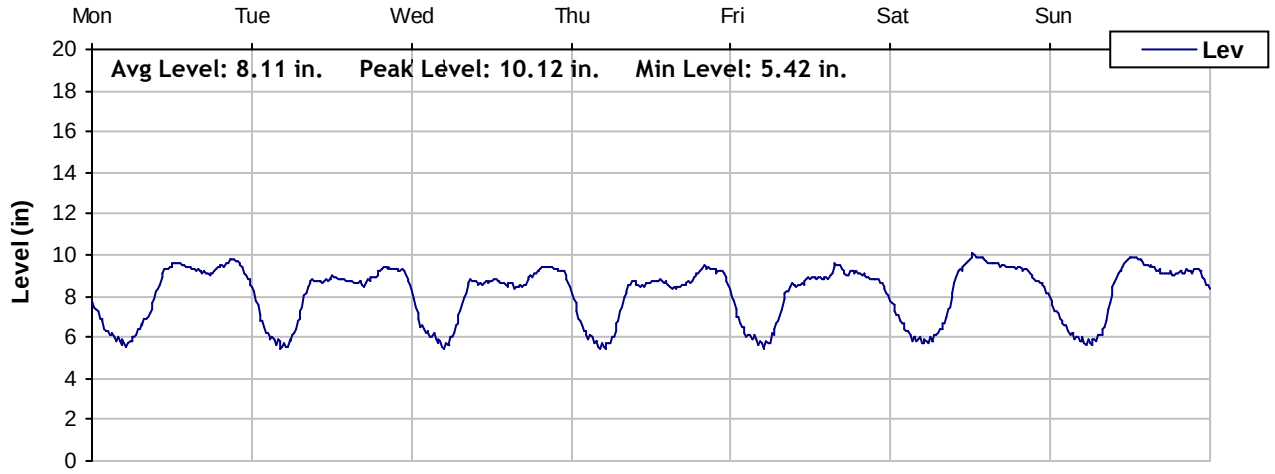


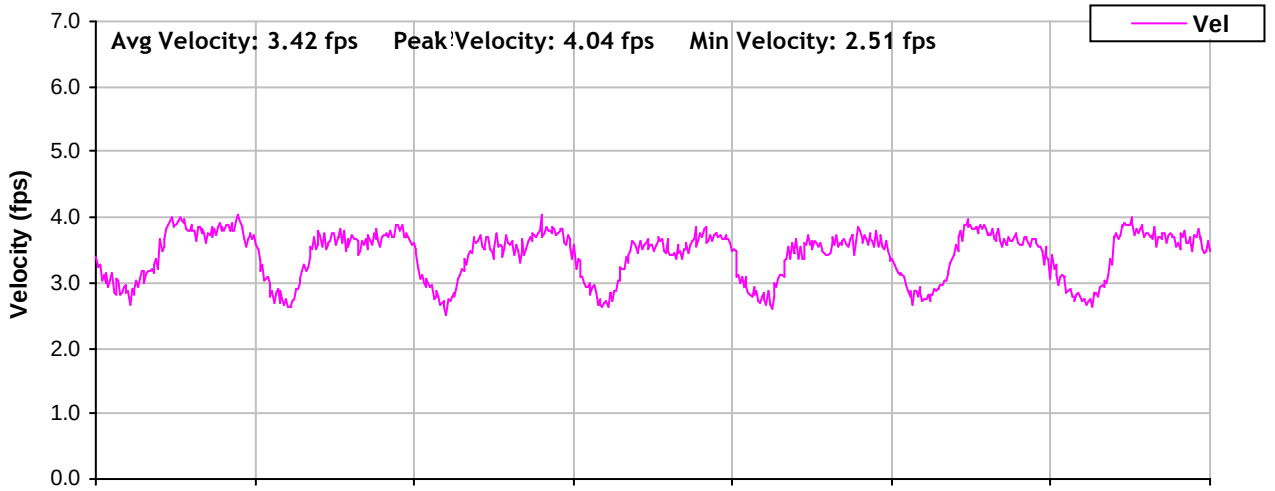


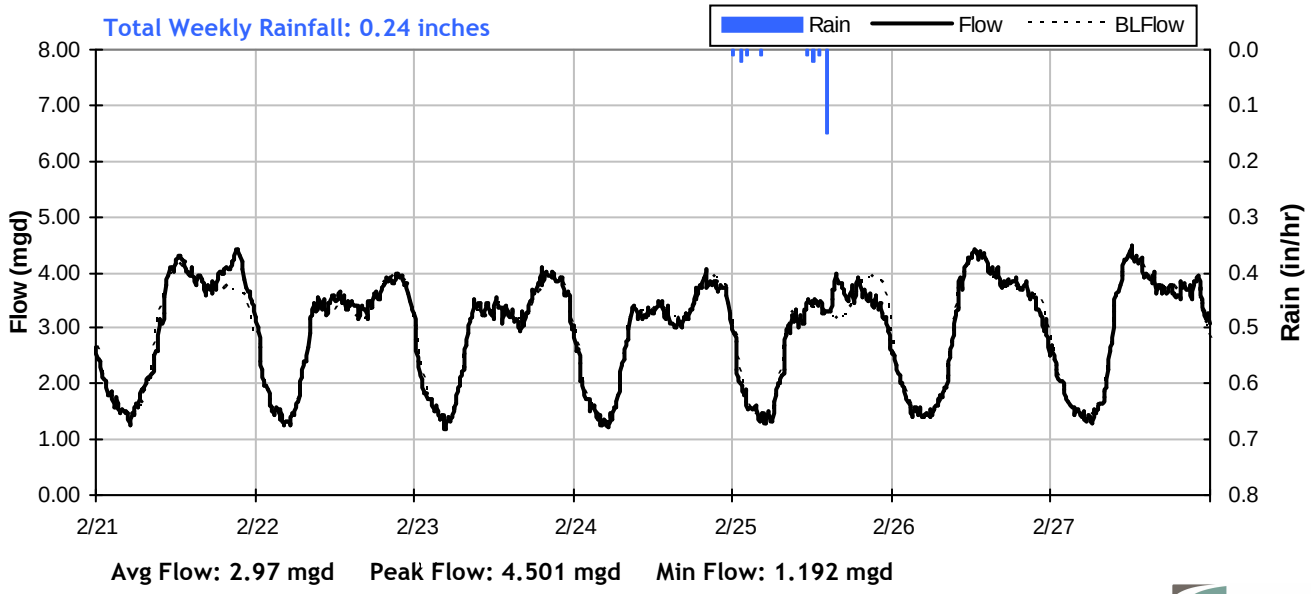


Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 2

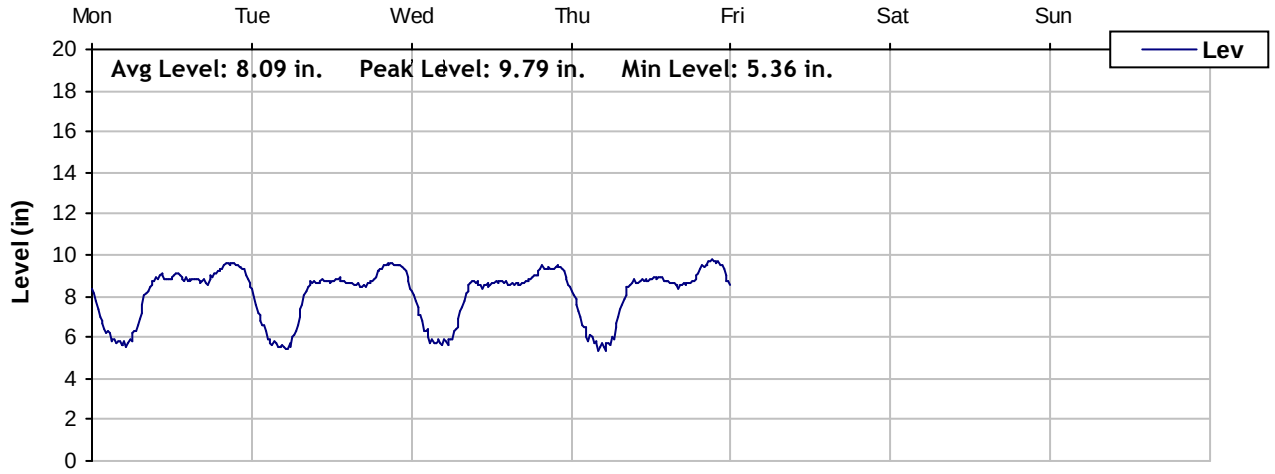


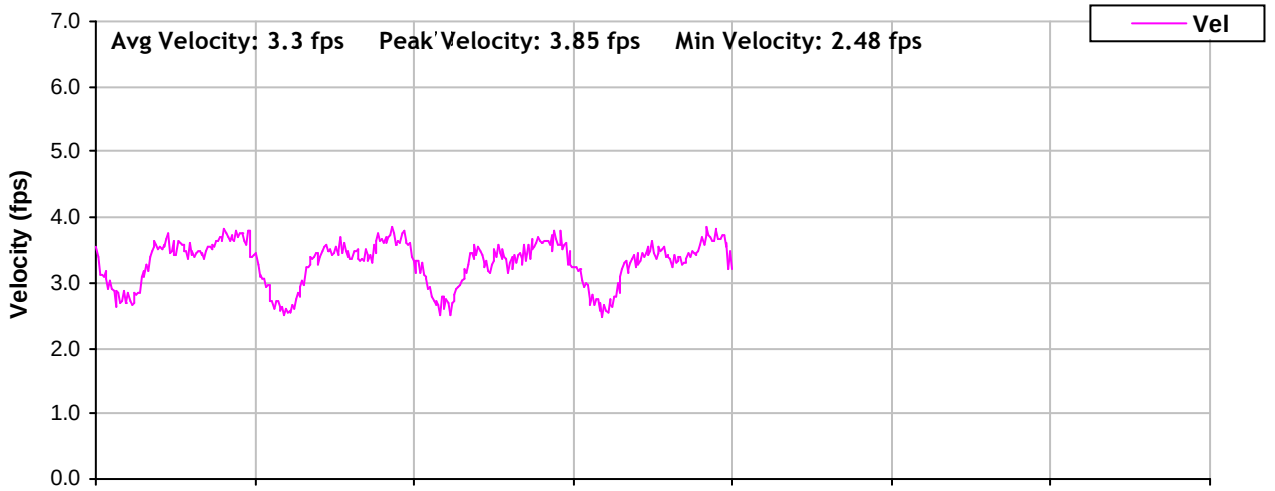


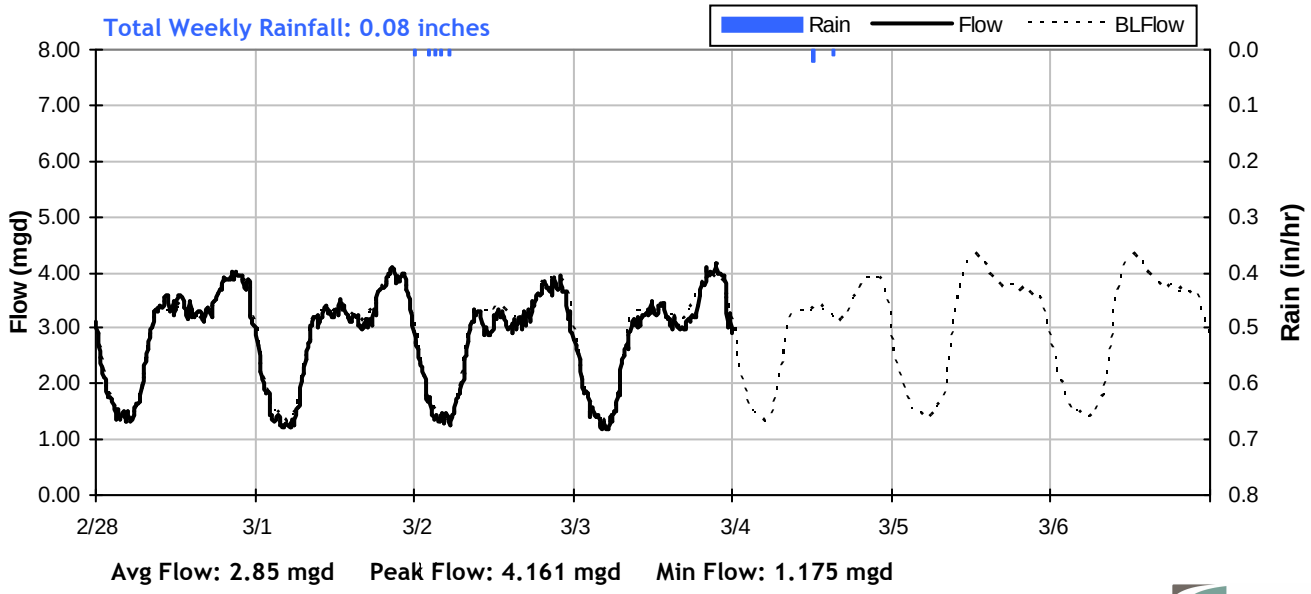


Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 2







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 2

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**8.13**	**3.46**	**3.02**
Weekly Minimum:	**5.39**	**2.58**	**1.20**
Weekly Maximum:	**10.15**	**4.00**	**4.60**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00				7.82	3.36	2.69	7.62	3.43	2.65	7.64	3.46	2.68	7.59	3.20	2.45	8.54	3.57	3.24	7.48	3.25	2.43	**0:00**
1:00				6.80	3.09	2.02	6.46	3.03	1.84	6.52	3.03	1.86	6.67	3.01	1.91	7.39	3.19	2.35	6.88	3.08	2.05	**1:00**
2:00				5.97	2.88	1.56	5.77	2.73	1.41	5.93	2.77	1.49	6.19	2.84	1.62	6.61	3.08	1.93	6.35	2.95	1.74	**2:00**
3:00				5.65	2.77	1.38	5.52	2.67	1.29	5.67	2.66	1.34	6.35	2.92	1.72	6.46	3.02	1.83	6.21	2.91	1.66	**3:00**
4:00				5.49	2.63	1.26	5.39	2.58	1.20	5.52	2.60	1.25	6.34	2.99	1.76	6.41	2.95	1.77	5.92	2.84	1.52	**4:00**
5:00				5.59	2.78	1.37	5.44	2.65	1.25	5.54	2.61	1.26	6.64	3.06	1.93	6.30	3.06	1.79	5.89	2.90	1.54	**5:00**
6:00				6.16	2.97	1.69	6.07	2.86	1.59	6.05	2.86	1.58	7.50	3.20	2.41	6.36	2.98	1.77	5.95	2.87	1.54	**6:00**
7:00				7.41	3.35	2.49	7.46	3.22	2.42	7.23	3.18	2.28	8.42	3.48	3.10	6.64	3.05	1.93	6.18	2.97	1.69	**7:00**
8:00				8.42	3.60	3.20	8.57	3.81	3.47	8.37	3.49	3.08	9.11	3.71	3.69	7.36	3.22	2.36	6.83	3.08	2.03	**8:00**
9:00				8.54	3.61	3.28	8.63	3.58	3.29	8.65	3.61	3.33	9.03	3.59	3.53	8.57	3.71	3.39	8.23	3.55	3.05	**9:00**
10:00				8.61	3.57	3.27	8.54	3.66	3.32	8.60	3.52	3.23	8.96	3.66	3.55	9.40	3.85	4.00	9.08	3.87	3.82	**10:00**
11:00				8.79	3.73	3.52	8.68	3.85	3.57	8.80	3.68	3.48	9.09	3.71	3.67	9.80	3.92	4.32	9.49	3.86	4.07	**11:00**
12:00				8.72	3.69	3.45	8.84	3.74	3.56	8.78	3.61	3.40	8.99	3.75	3.66	10.11	4.00	4.60	9.69	3.88	4.21	**12:00**
13:00				8.69	3.57	3.31	8.91	3.67	3.53	8.68	3.60	3.34	8.97	3.79	3.68	10.05	3.93	4.49	9.72	3.83	4.17	**13:00**
14:00				9.26	3.20	3.26	8.87	3.75	3.59	8.56	3.59	3.27	8.90	3.85	3.70	9.86	3.80	4.22	9.54	3.83	4.06	**14:00**
15:00				8.64	3.55	3.28	8.64	3.65	3.36	8.47	3.59	3.21	8.90	3.80	3.65	9.71	3.82	4.15	9.29	3.70	3.78	**15:00**
16:00				8.49	3.55	3.19	8.57	3.58	3.26	8.54	3.59	3.25	8.80	3.88	3.67	9.47	3.75	3.93	9.09	3.68	3.64	**16:00**
17:00				8.53	3.58	3.24	8.61	3.55	3.26	8.74	3.69	3.46	8.76	3.67	3.45	9.33	3.74	3.85	8.95	3.69	3.58	**17:00**
18:00				8.86	3.80	3.63	8.88	3.66	3.51	9.16	3.68	3.68	8.99	3.83	3.73	9.28	3.70	3.77	8.78	3.74	3.52	**18:00**
19:00				9.05	3.85	3.79	9.15	3.72	3.72	9.35	3.71	3.83	9.62	3.85	4.13	9.17	3.73	3.74	8.84	3.62	3.44	**19:00**
20:00				9.31	3.89	3.98	9.12	3.85	3.83	9.45	3.81	3.99	10.05	3.91	4.47	9.03	3.78	3.70	8.77	3.60	3.39	**20:00**
21:00				9.42	3.93	4.10	9.12	3.73	3.71	9.34	3.76	3.87	10.15	3.96	4.59	9.06	3.71	3.66	8.70	3.57	3.33	**21:00**
22:00				9.00	3.79	3.71	8.93	3.87	3.74	9.10	3.77	3.74	9.77	3.90	4.29	8.77	3.68	3.46	8.61	3.63	3.33	**22:00**
23:00				8.52	3.68	3.32	8.64	3.61	3.33	8.58	3.62	3.30	9.11	3.72	3.69	8.17	3.43	2.92	8.13	3.51	2.97	**23:00**
Average:				**7.99**	**3.43**	**2.92**	**7.93**	**3.43**	**2.90**	**7.97**	**3.39**	**2.88**	**8.45**	**3.55**	**3.25**	**8.41**	**3.53**	**3.22**	**8.02**	**3.43**	**2.94**	**Ave**
Minimum:				**5.49**	**2.63**	**1.26**	**5.39**	**2.58**	**1.20**	**5.52**	**2.60**	**1.25**	**6.19**	**2.84**	**1.62**	**6.30**	**2.95**	**1.77**	**5.89**	**2.84**	**1.52**	**Min**
Maximum:				**9.42**	**3.93**	**4.10**	**9.15**	**3.87**	**3.83**	**9.45**	**3.81**	**3.99**	**10.15**	**3.96**	**4.59**	**10.11**	**4.00**	**4.60**	**9.72**	**3.88**	**4.21**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 2

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**8.11**	**3.42**	**2.97**
Weekly Minimum:	**5.57**	**2.63**	**1.31**
Weekly Maximum:	**10.00**	**3.94**	**4.34**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	7.48	3.23	2.42	8.02	3.40	2.82	7.71	3.32	2.61	7.74	3.37	2.66	7.54	3.30	2.51	7.41	3.25	2.40	7.47	3.25	2.44	**0:00**
1:00	6.77	3.05	1.98	6.79	3.11	2.03	6.62	3.03	1.90	6.65	3.03	1.92	6.68	3.02	1.92	6.81	3.13	2.04	6.90	3.07	2.04	**1:00**
2:00	6.23	2.95	1.70	6.11	2.84	1.59	6.19	2.92	1.66	6.11	2.91	1.63	6.12	2.84	1.60	6.25	2.88	1.66	6.38	2.93	1.75	**2:00**
3:00	6.00	2.94	1.60	5.88	2.83	1.50	5.97	2.78	1.50	5.87	2.80	1.48	5.98	2.84	1.54	5.92	2.80	1.50	6.08	2.77	1.54	**3:00**
4:00	5.79	2.89	1.49	5.63	2.69	1.34	5.68	2.63	1.32	5.57	2.67	1.31	5.69	2.76	1.39	5.85	2.81	1.48	5.92	2.77	1.48	**4:00**
5:00	5.71	2.81	1.42	5.72	2.68	1.36	5.77	2.77	1.43	5.66	2.76	1.38	5.73	2.70	1.38	5.88	2.76	1.46	5.76	2.72	1.40	**5:00**
6:00	6.13	3.04	1.71	6.37	2.96	1.76	6.39	2.96	1.77	6.27	2.95	1.72	6.39	2.95	1.76	5.97	2.88	1.55	5.84	2.78	1.46	**6:00**
7:00	6.67	3.13	1.99	7.43	3.22	2.39	7.47	3.25	2.44	7.39	3.27	2.41	7.27	3.17	2.29	6.45	2.97	1.79	6.19	2.90	1.65	**7:00**
8:00	7.12	3.21	2.25	8.48	3.55	3.19	8.55	3.51	3.19	8.36	3.46	3.04	8.30	3.46	3.01	7.16	3.17	2.23	7.08	3.12	2.17	**8:00**
9:00	8.03	3.44	2.86	8.76	3.66	3.43	8.72	3.66	3.41	8.68	3.53	3.27	8.55	3.48	3.17	8.44	3.46	3.08	8.52	3.60	3.26	**9:00**
10:00	8.98	3.73	3.64	8.71	3.60	3.35	8.61	3.57	3.28	8.49	3.59	3.22	8.58	3.48	3.17	9.28	3.75	3.83	9.26	3.79	3.85	**10:00**
11:00	9.36	3.92	4.05	8.82	3.69	3.50	8.72	3.51	3.28	8.63	3.59	3.30	8.83	3.63	3.45	9.65	3.89	4.20	9.78	3.89	4.27	**11:00**
12:00	9.64	3.94	4.24	8.94	3.70	3.58	8.77	3.59	3.38	8.74	3.61	3.38	8.92	3.58	3.46	10.00	3.82	4.33	9.92	3.87	4.34	**12:00**
13:00	9.56	3.87	4.11	8.83	3.64	3.46	8.65	3.55	3.27	8.73	3.56	3.33	8.86	3.51	3.35	9.93	3.85	4.32	9.77	3.82	4.20	**13:00**
14:00	9.42	3.82	3.97	8.72	3.65	3.41	8.58	3.61	3.29	8.62	3.50	3.21	8.92	3.48	3.36	9.70	3.76	4.09	9.49	3.73	3.93	**14:00**
15:00	9.30	3.78	3.86	8.66	3.56	3.28	8.46	3.45	3.08	8.42	3.46	3.07	9.42	3.64	3.79	9.60	3.72	3.97	9.40	3.71	3.85	**15:00**
16:00	9.20	3.74	3.76	8.62	3.60	3.31	8.50	3.53	3.18	8.50	3.47	3.12	9.37	3.63	3.75	9.51	3.66	3.87	9.19	3.71	3.73	**16:00**
17:00	9.11	3.77	3.74	8.78	3.68	3.47	8.82	3.69	3.50	8.62	3.57	3.28	9.10	3.63	3.61	9.46	3.63	3.80	9.12	3.72	3.70	**17:00**
18:00	9.30	3.82	3.91	8.97	3.67	3.57	9.12	3.74	3.73	8.98	3.68	3.58	9.19	3.66	3.69	9.41	3.72	3.87	9.12	3.68	3.66	**18:00**
19:00	9.43	3.85	4.02	9.28	3.73	3.80	9.35	3.84	3.95	9.29	3.78	3.87	9.11	3.70	3.67	9.35	3.63	3.74	9.18	3.65	3.67	**19:00**
20:00	9.73	3.82	4.17	9.36	3.72	3.84	9.40	3.81	3.95	9.36	3.66	3.78	8.94	3.67	3.55	9.24	3.64	3.69	9.25	3.68	3.73	**20:00**
21:00	9.77	3.94	4.32	9.31	3.85	3.95	9.35	3.77	3.89	9.26	3.73	3.79	8.81	3.63	3.45	8.91	3.65	3.51	9.27	3.65	3.72	**21:00**
22:00	9.42	3.68	3.83	9.25	3.77	3.83	9.23	3.71	3.75	9.19	3.71	3.73	8.64	3.61	3.32	8.63	3.46	3.19	9.19	3.68	3.70	**22:00**
23:00	8.81	3.72	3.53	8.80	3.63	3.44	8.75	3.58	3.36	8.65	3.61	3.33	8.07	3.46	2.90	8.18	3.34	2.85	8.57	3.52	3.20	**23:00**
Average:	**8.20**	**3.50**	**3.11**	**8.09**	**3.43**	**2.97**	**8.06**	**3.41**	**2.92**	**7.99**	**3.38**	**2.87**	**8.04**	**3.37**	**2.88**	**8.21**	**3.40**	**3.02**	**8.19**	**3.42**	**3.03**	**Ave**
Minimum:	**5.71**	**2.81**	**1.42**	**5.63**	**2.68**	**1.34**	**5.68**	**2.63**	**1.32**	**5.57**	**2.67**	**1.31**	**5.69**	**2.70**	**1.38**	**5.85**	**2.76**	**1.46**	**5.76**	**2.72**	**1.40**	**Min**
Maximum:	**9.77**	**3.94**	**4.32**	**9.36**	**3.85**	**3.95**	**9.40**	**3.84**	**3.95**	**9.36**	**3.78**	**3.87**	**9.42**	**3.70**	**3.79**	**10.00**	**3.89**	**4.33**	**9.92**	**3.89**	**4.34**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

Monitoring Site:

Site 2

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**8.09**	**3.30**	**2.85**
Weekly Minimum:	**5.53**	**2.57**	**1.25**
Weekly Maximum:	**9.73**	**3.76**	**4.08**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	7.91	3.30	2.69	7.78	3.24	2.57	7.88	3.25	2.62	7.94	3.21	2.62										**0:00**
1:00	6.73	3.06	1.97	6.76	3.01	1.96	6.82	3.14	2.06	6.86	3.05	2.02										**1:00**
2:00	6.15	2.86	1.62	5.96	2.75	1.49	6.10	2.83	1.58	6.12	2.80	1.58										**2:00**
3:00	5.86	2.79	1.47	5.68	2.66	1.34	5.83	2.64	1.38	5.83	2.68	1.40										**3:00**
4:00	5.74	2.77	1.42	5.56	2.57	1.26	5.78	2.73	1.41	5.53	2.60	1.25										**4:00**
5:00	5.76	2.73	1.40	5.53	2.60	1.26	5.84	2.65	1.39	5.61	2.65	1.31										**5:00**
6:00	6.36	2.90	1.72	6.28	2.83	1.65	6.31	2.90	1.70	6.29	2.85	1.66										**6:00**
7:00	7.42	3.17	2.35	7.48	3.12	2.35	7.36	3.11	2.28	7.45	3.24	2.42										**7:00**
8:00	8.37	3.51	3.10	8.48	3.35	3.01	8.40	3.39	3.00	8.34	3.26	2.86										**8:00**
9:00	8.79	3.54	3.35	8.66	3.39	3.13	8.74	3.51	3.29	8.69	3.34	3.10										**9:00**
10:00	9.00	3.64	3.56	8.72	3.53	3.29	8.50	3.33	2.99	8.75	3.41	3.20										**10:00**
11:00	8.85	3.50	3.34	8.71	3.47	3.23	8.55	3.23	2.93	8.80	3.52	3.33										**11:00**
12:00	9.06	3.56	3.51	8.81	3.53	3.35	8.67	3.49	3.23	8.90	3.46	3.32										**12:00**
13:00	8.90	3.48	3.35	8.78	3.49	3.29	8.74	3.42	3.20	8.91	3.49	3.36										**13:00**
14:00	8.82	3.47	3.30	8.65	3.43	3.17	8.61	3.26	2.99	8.72	3.35	3.13										**14:00**
15:00	8.81	3.46	3.28	8.60	3.42	3.13	8.62	3.39	3.12	8.60	3.36	3.07										**15:00**
16:00	8.75	3.46	3.25	8.48	3.41	3.06	8.60	3.38	3.10	8.53	3.31	3.00										**16:00**
17:00	8.75	3.55	3.33	8.62	3.41	3.14	8.79	3.53	3.33	8.62	3.42	3.14										**17:00**
18:00	9.10	3.67	3.64	8.94	3.62	3.51	9.01	3.62	3.54	8.80	3.45	3.26										**18:00**
19:00	9.35	3.73	3.85	9.37	3.65	3.77	9.32	3.63	3.72	9.36	3.65	3.77										**19:00**
20:00	9.58	3.67	3.92	9.58	3.76	4.01	9.34	3.61	3.72	9.63	3.73	4.01										**20:00**
21:00	9.54	3.75	3.97	9.54	3.61	3.83	9.40	3.65	3.80	9.73	3.74	4.08										**21:00**
22:00	9.34	3.69	3.80	9.43	3.70	3.86	9.34	3.63	3.73	9.48	3.68	3.87										**22:00**
23:00	8.76	3.50	3.30	8.81	3.48	3.30	8.70	3.32	3.09	8.78	3.38	3.19										**23:00**
Average:	**8.15**	**3.37**	**2.94**	**8.05**	**3.29**	**2.83**	**8.05**	**3.28**	**2.80**	**8.09**	**3.27**	**2.83**										**Ave**
Minimum:	**5.74**	**2.73**	**1.40**	**5.53**	**2.57**	**1.26**	**5.78**	**2.64**	**1.38**	**5.53**	**2.60**	**1.25**										**Min**
Maximum:	**9.58**	**3.75**	**3.97**	**9.58**	**3.76**	**4.01**	**9.40**	**3.65**	**3.80**	**9.73**	**3.74**	**4.08**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Temporary Flow Monitoring Study

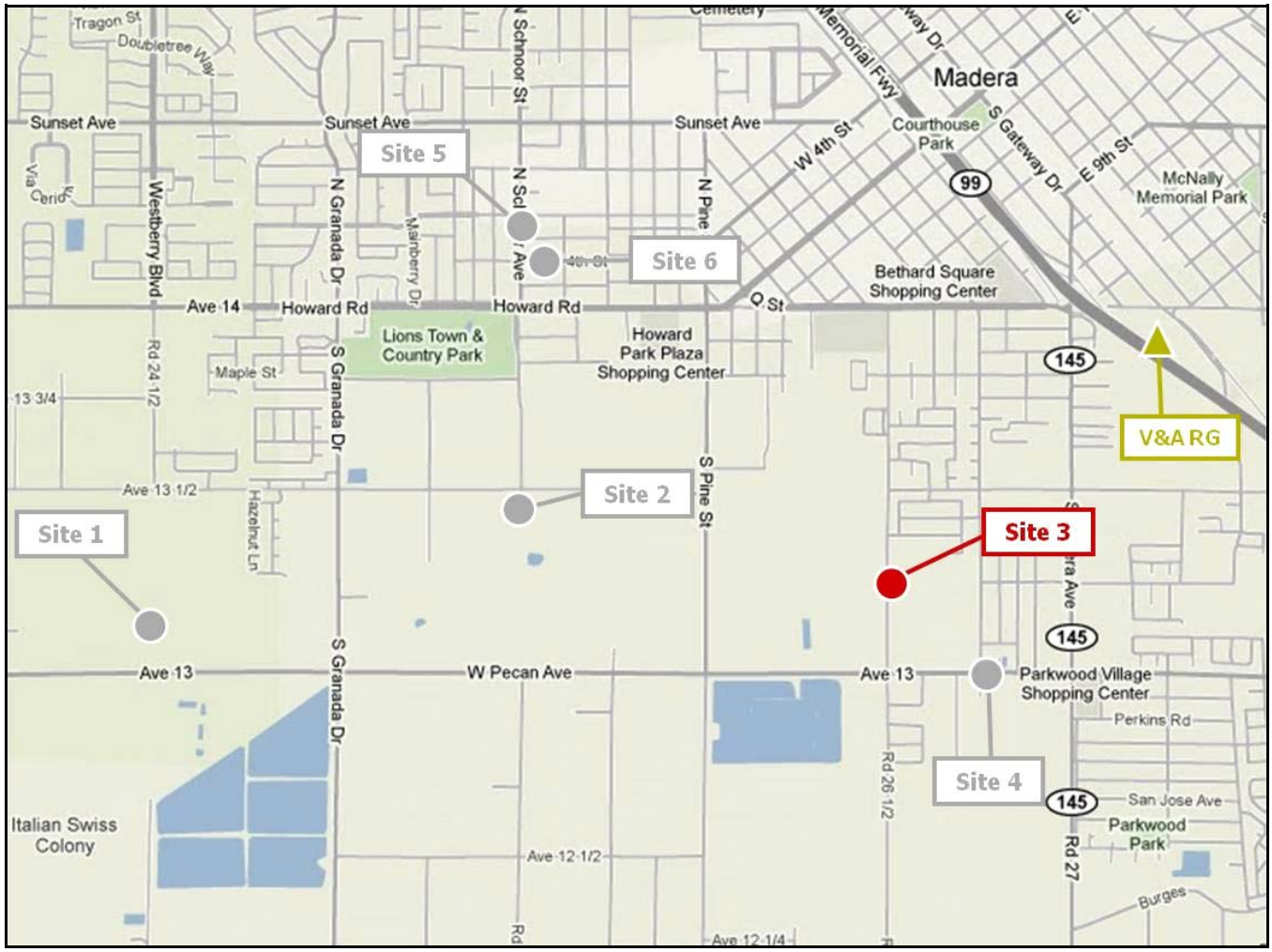
Sanitary Sewer Collection System

Monitoring Site: Site 3

Location: MH ID 5697: Stadium Road (Road 26 1/2) at Gary Lane, north of Pecan Avenue (Ave 13)

Size / Type Line: 21-inch Sanitary Sewer Pipe

Data Summary Report



Site Information Report

Monitoring Site: Site 3

Location: MH ID 5697: Stadium Road (Road 26 1/2) at Gary Lane, north of Pecan Avenue (Ave 13)

Latitude: 36.941935°

Longitude: -120.065069°

Rim Elevation: 263 *feet above sea level*

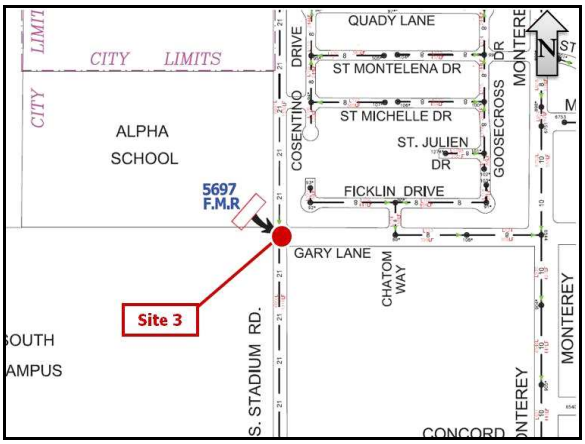
Diameter: 21 *inches*

Average Dry Weather Flow: 0.854 *mgd*

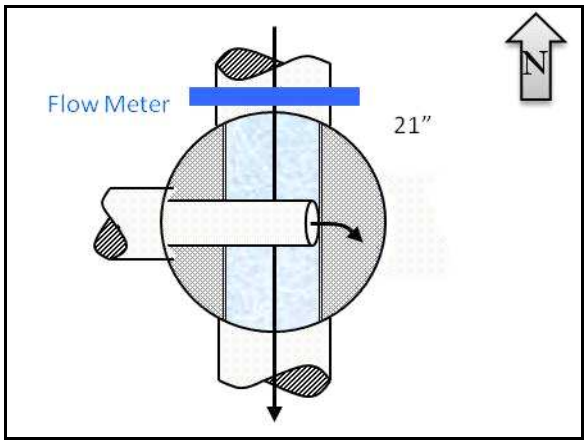
Peak Measured Flow: 2.277 *mgd*

Satellite Map

Sanitary Map

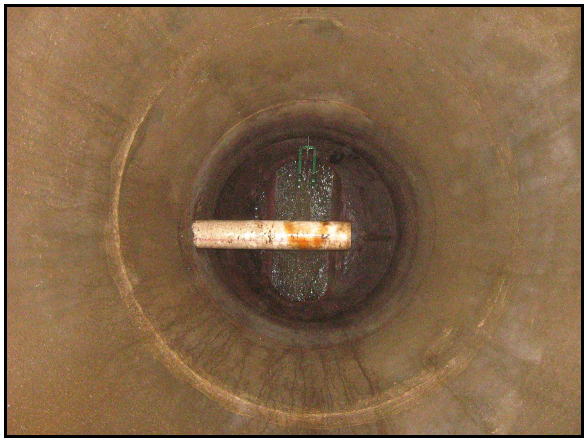


Flow Schematic



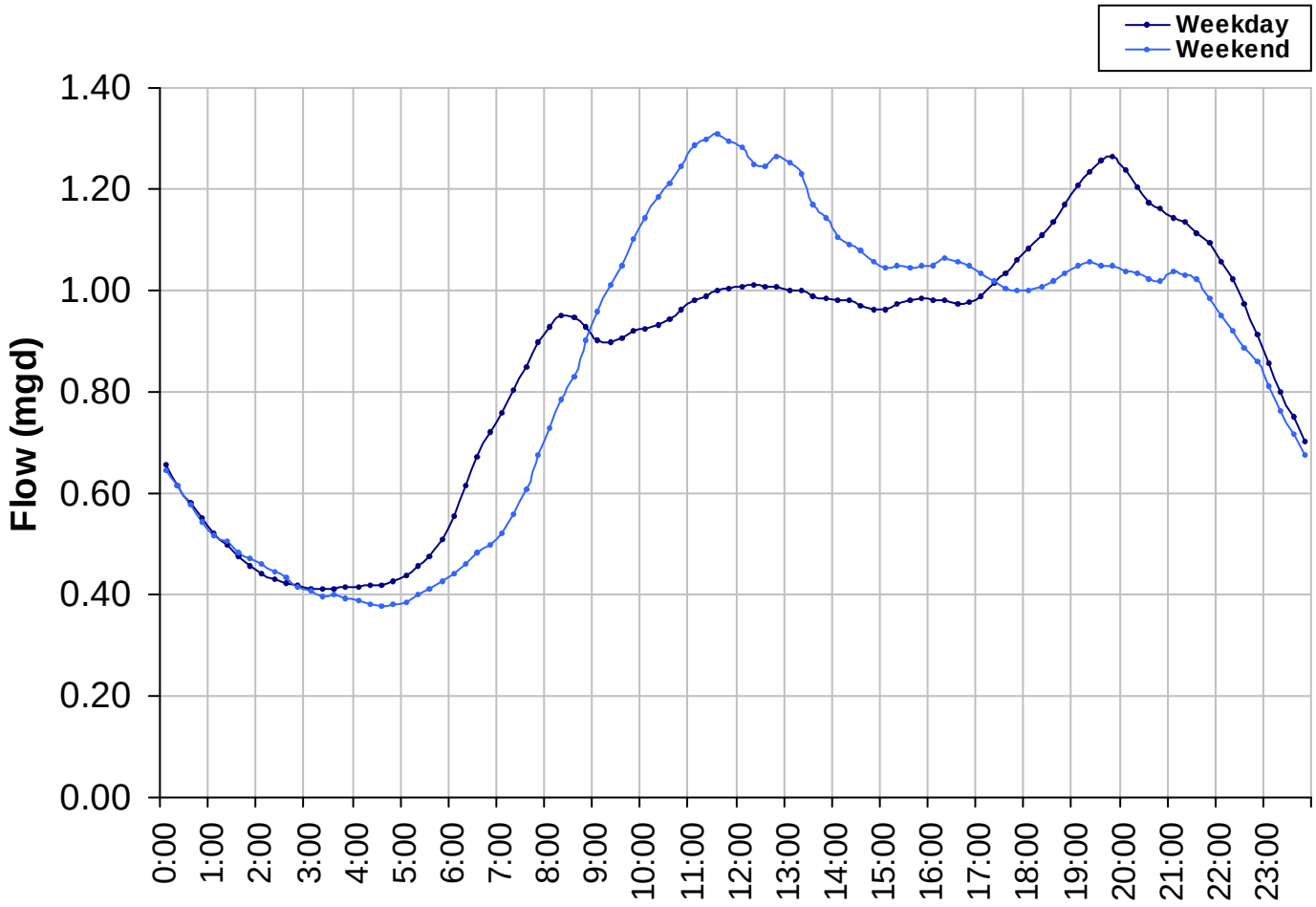
Street View Photo

Plan View Photo

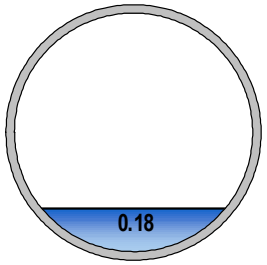


Average Dry Weather Flow

Monitoring Site:
Site 3



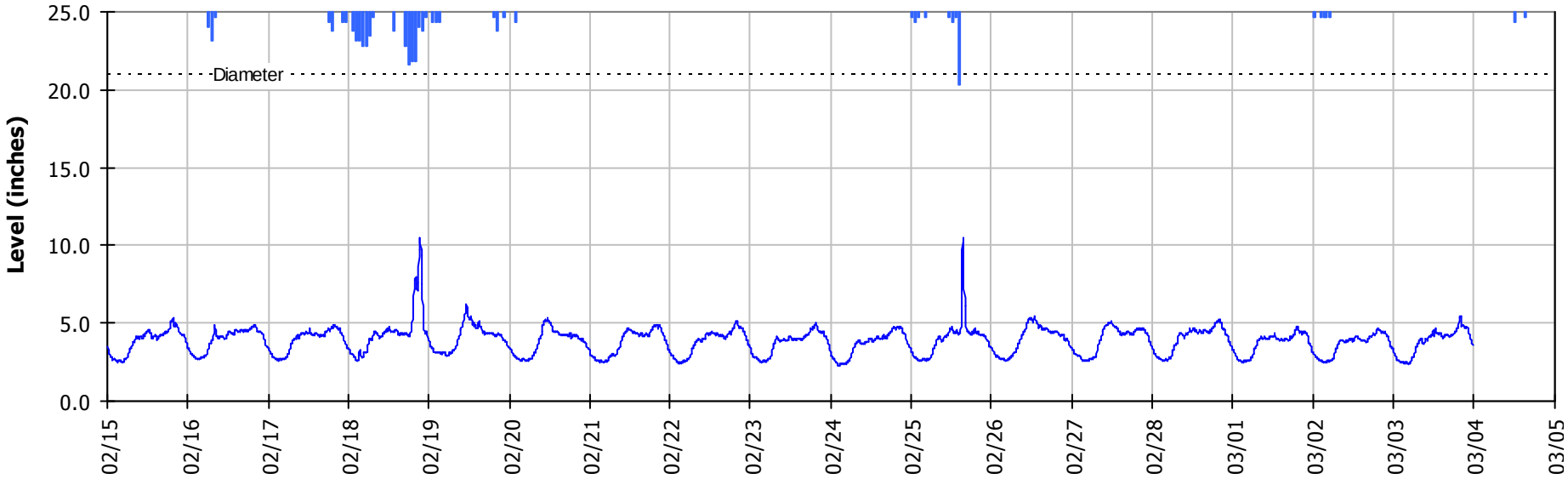
Average Dry Weather Flow:
0.854 *mgd*

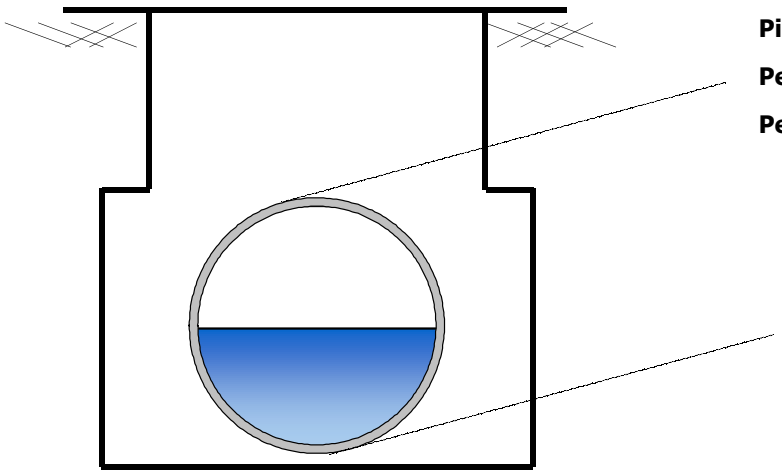


Site Capacity / Surcharge Summary

Monitoring Site:
Site 3

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 21 *inches*

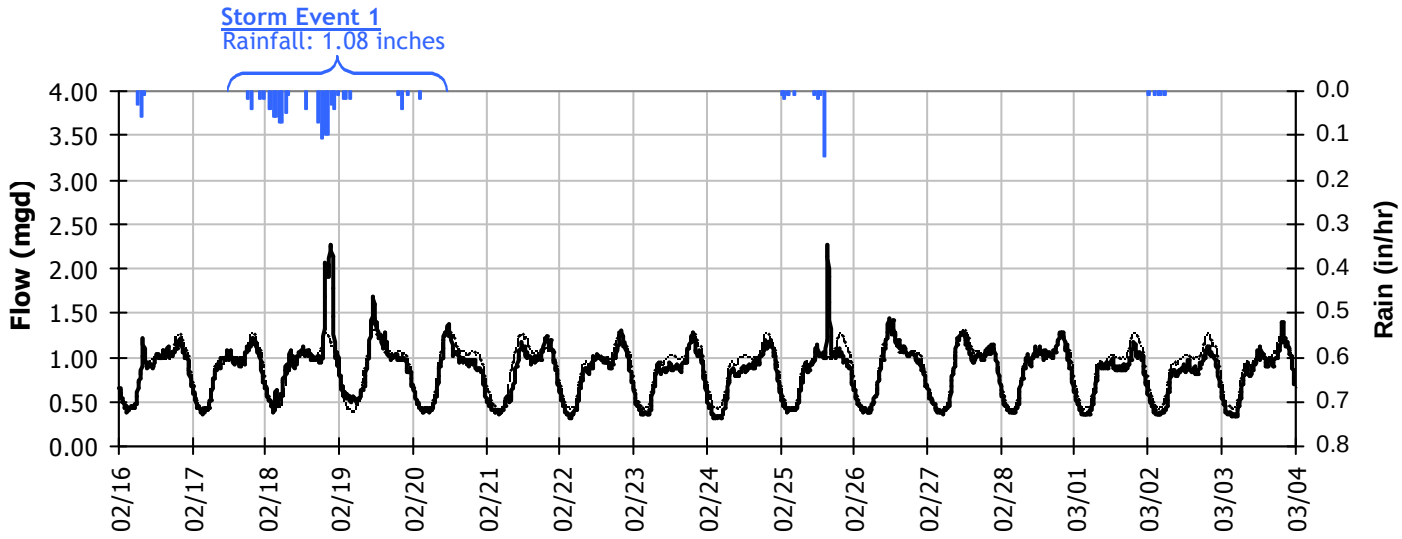
Peak Measured Level: 10.4 *inches*

Peak d/D Ratio: 0.50

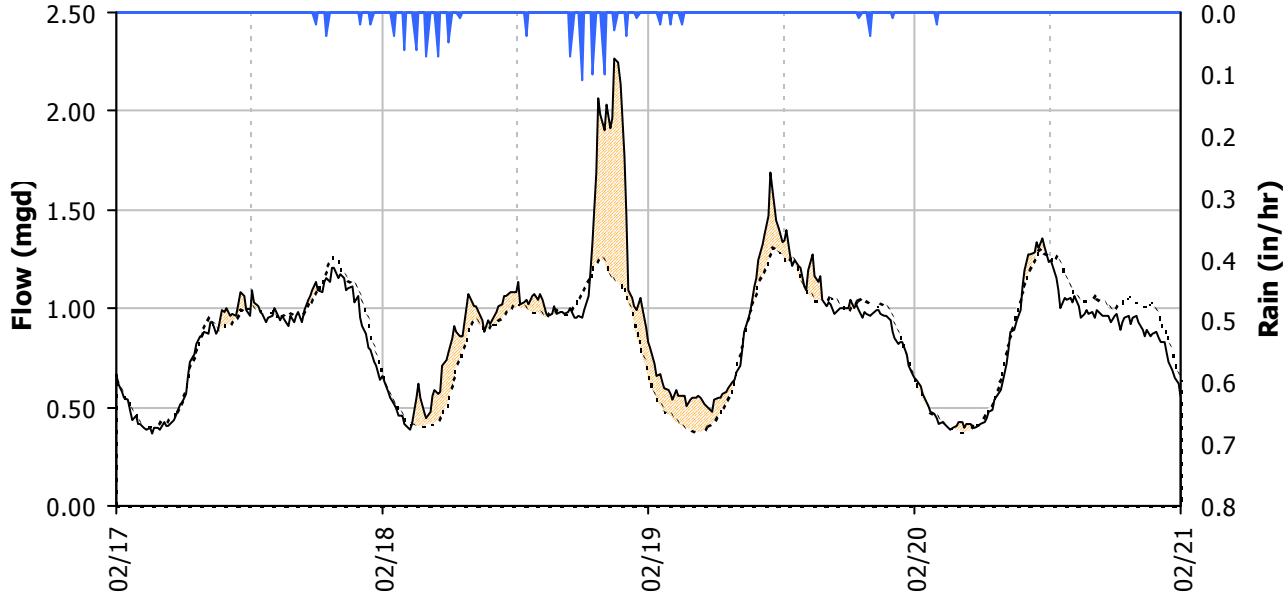
I/I Summary
Event 1

Monitoring Site:
Site 3

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

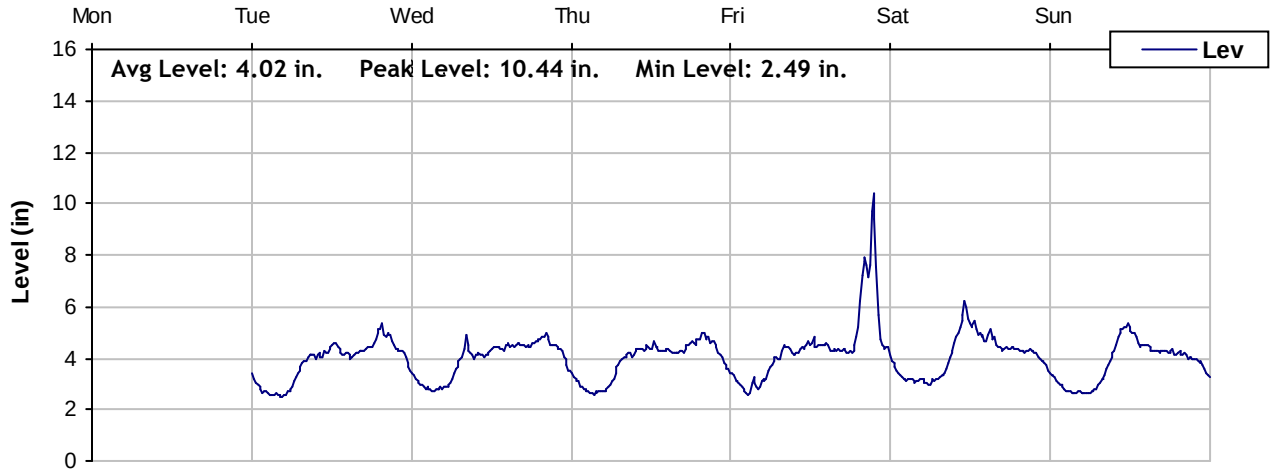


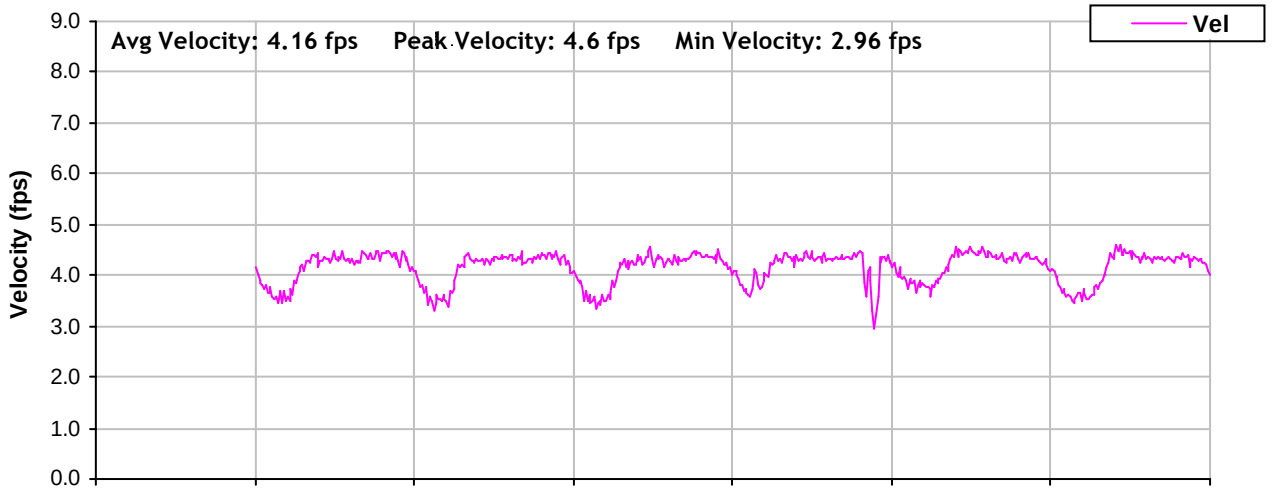
Storm Event #1 I/I Analysis

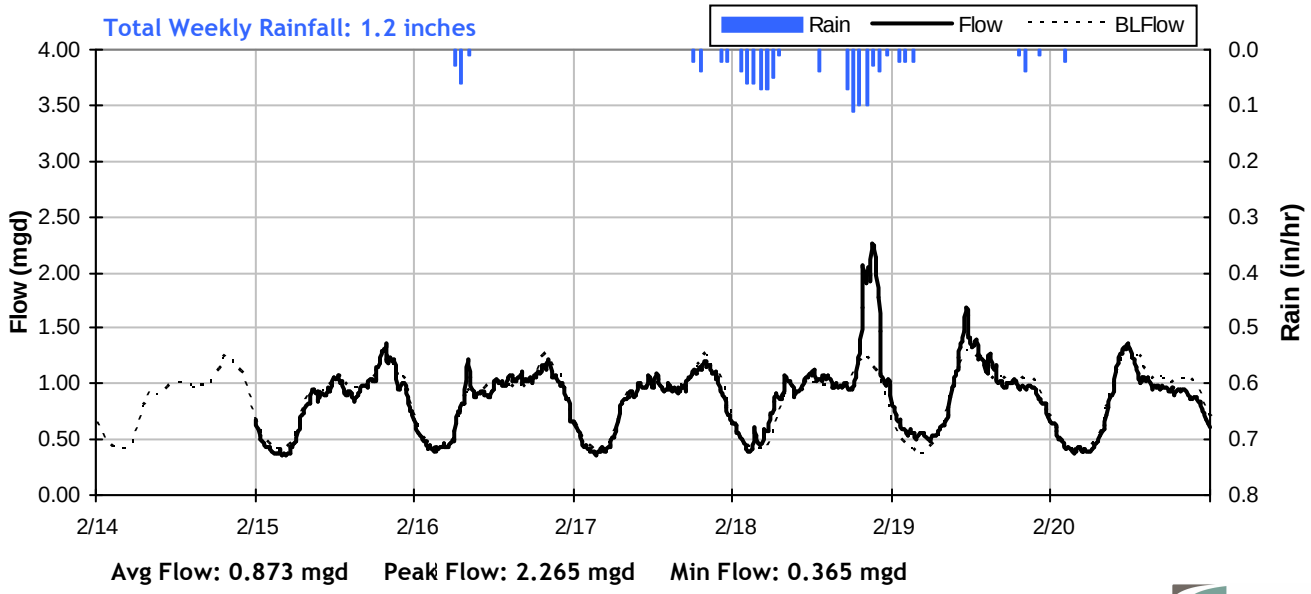
Rainfall:	1.08	*inches*			
Peak Flow:	2.26	*mgd*	**PF:**	2.65	
Peak I/I Rate:	1.12	*mgd*	**Pk I/I:ADWF:**	1.31	
Peak Level:	10.44	*inches*	**d/D Ratio:**	0.50	
Total I/I:	202,000	*gallons*	**I/I per ADWF:**	0.22	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.10	*mgd*	**RDI (% of BL):**	12%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 3

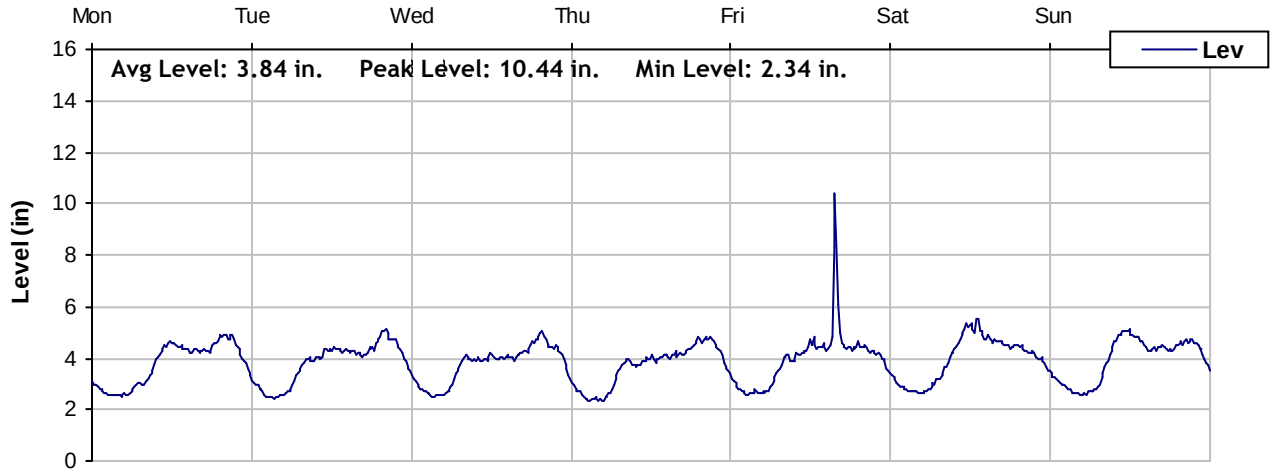


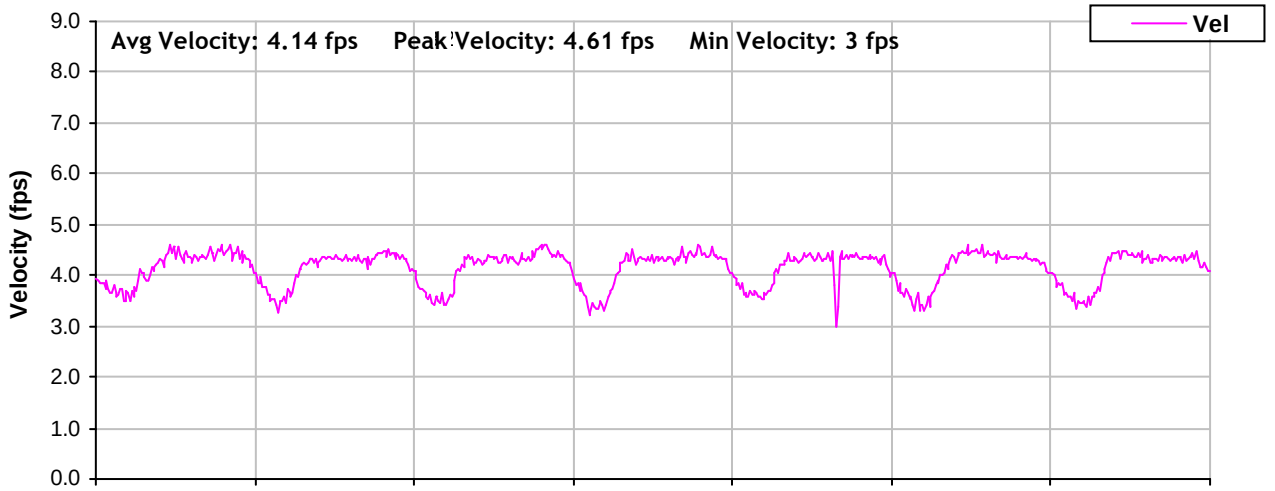


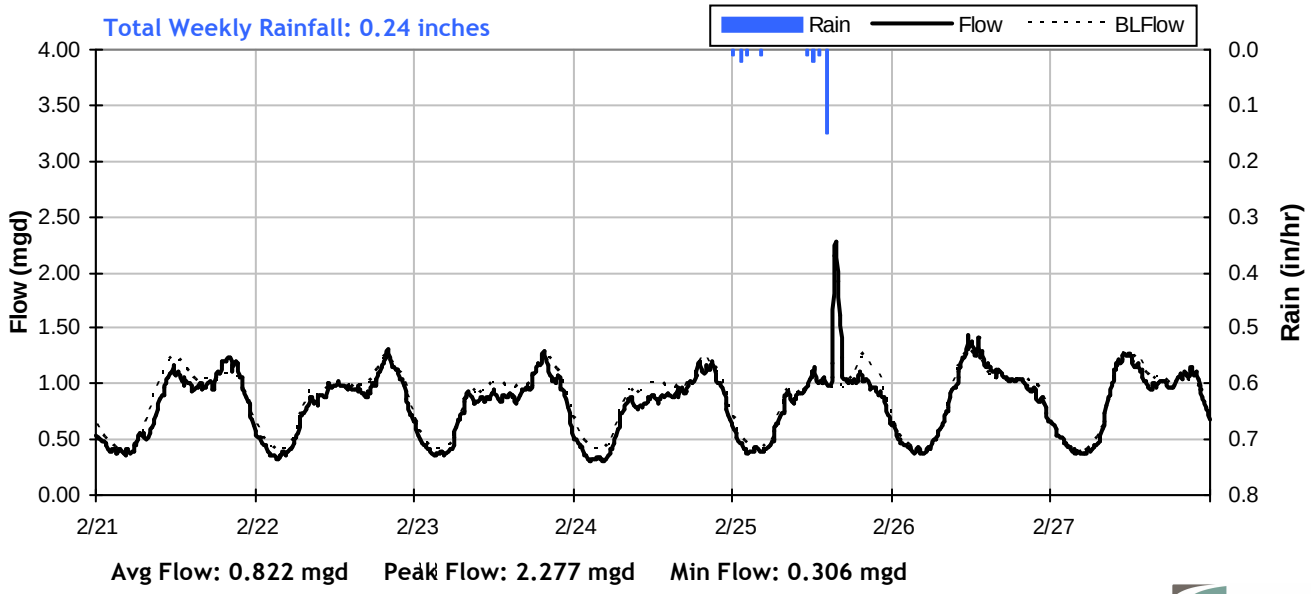


Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 3

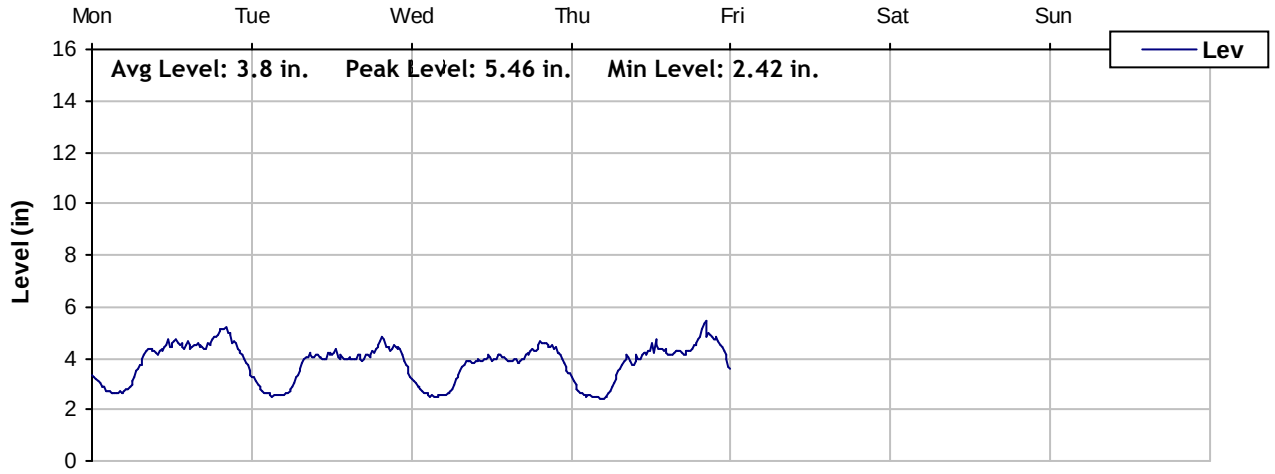


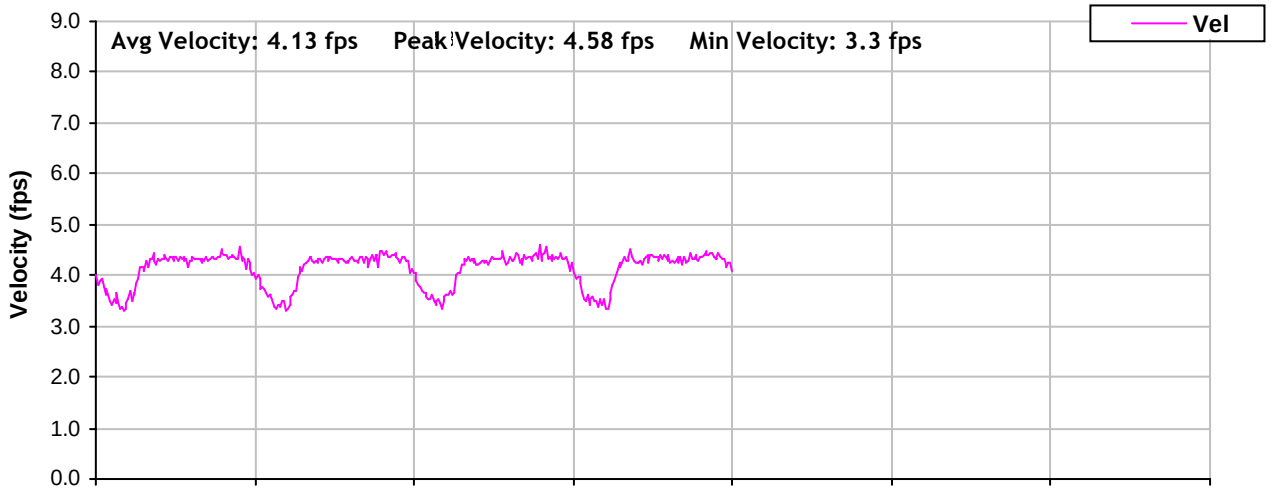


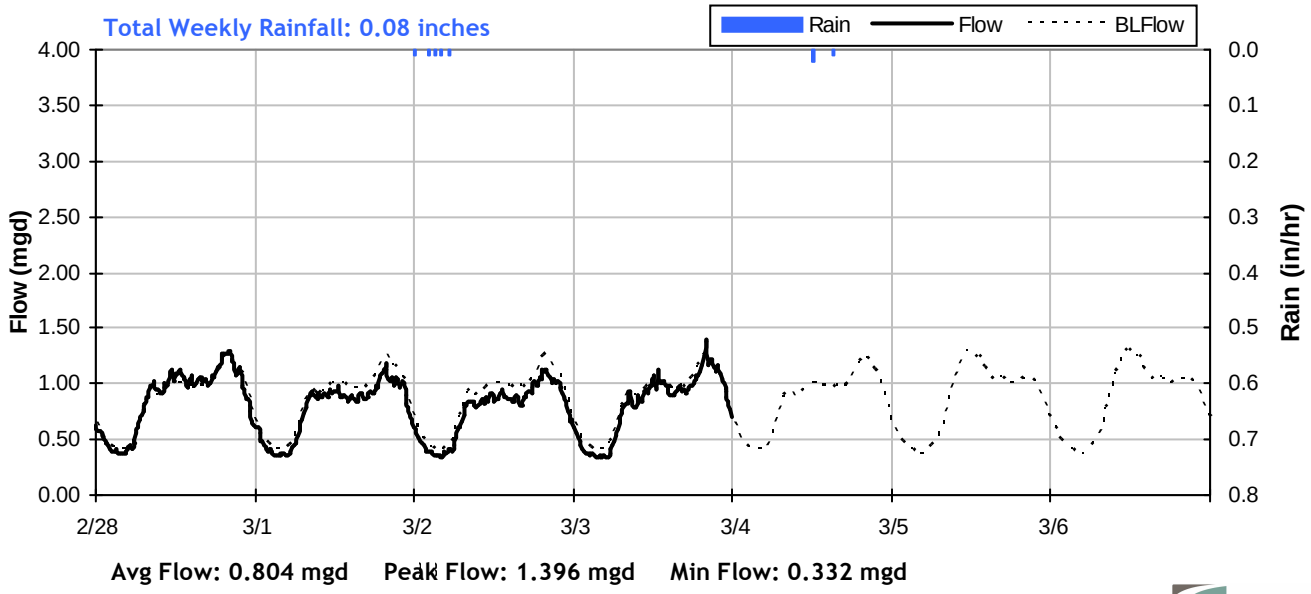


Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 3







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 3

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**4.02**	**4.16**	**0.87**
Weekly Minimum:	**2.52**	**3.27**	**0.37**
Weekly Maximum:	**9.26**	**4.49**	**2.10**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00				3.16	3.99	0.58	3.25	3.95	0.60	3.26	3.96	0.60	3.23	4.03	0.60	3.70	4.12	0.75	3.28	4.04	0.62	**0:00**
1:00				2.78	3.76	0.45	2.97	3.74	0.50	2.91	3.71	0.48	2.90	3.76	0.48	3.32	4.01	0.62	2.96	3.74	0.49	**1:00**
2:00				2.62	3.63	0.40	2.80	3.44	0.42	2.70	3.52	0.41	2.63	3.65	0.41	3.17	3.86	0.56	2.74	3.61	0.43	**2:00**
3:00				2.61	3.59	0.39	2.75	3.54	0.42	2.64	3.48	0.39	3.03	4.00	0.55	3.11	3.80	0.54	2.65	3.51	0.39	**3:00**
4:00				2.52	3.56	0.37	2.84	3.54	0.44	2.71	3.51	0.41	2.98	3.86	0.52	3.15	3.84	0.55	2.69	3.64	0.42	**4:00**
5:00				2.71	3.68	0.43	2.94	3.61	0.47	2.84	3.67	0.46	3.35	4.10	0.65	3.00	3.72	0.50	2.65	3.57	0.40	**5:00**
6:00				3.17	4.02	0.59	3.46	4.07	0.67	3.32	3.98	0.62	3.92	4.31	0.85	3.13	3.84	0.55	2.76	3.70	0.44	**6:00**
7:00				3.70	4.22	0.77	4.06	4.29	0.89	3.89	4.23	0.83	4.19	4.36	0.95	3.28	3.99	0.61	3.09	3.87	0.54	**7:00**
8:00				4.05	4.33	0.90	4.47	4.34	1.04	4.13	4.24	0.90	4.40	4.36	1.02	3.71	4.19	0.77	3.64	4.22	0.75	**8:00**
9:00				4.07	4.31	0.90	4.09	4.29	0.90	4.20	4.28	0.93	4.14	4.31	0.92	4.47	4.42	1.05	4.25	4.42	0.98	**9:00**
10:00				4.13	4.32	0.92	4.14	4.28	0.91	4.32	4.29	0.97	4.33	4.35	0.99	5.30	4.46	1.35	5.01	4.48	1.26	**10:00**
11:00				4.30	4.34	0.98	4.15	4.28	0.92	4.40	4.44	1.04	4.53	4.42	1.07	5.77	4.49	1.54	5.22	4.47	1.33	**11:00**
12:00				4.53	4.32	1.05	4.42	4.35	1.02	4.46	4.30	1.02	4.57	4.33	1.07	5.27	4.43	1.33	4.95	4.40	1.21	**12:00**
13:00				4.18	4.36	0.94	4.34	4.36	1.00	4.28	4.28	0.96	4.51	4.35	1.05	4.90	4.49	1.22	4.52	4.33	1.05	**13:00**
14:00				4.12	4.28	0.91	4.51	4.36	1.05	4.31	4.28	0.97	4.48	4.31	1.03	4.82	4.41	1.17	4.47	4.36	1.04	**14:00**
15:00				4.12	4.26	0.90	4.51	4.33	1.05	4.20	4.31	0.94	4.31	4.33	0.98	4.73	4.37	1.13	4.29	4.32	0.97	**15:00**
16:00				4.26	4.42	0.98	4.53	4.30	1.05	4.25	4.30	0.95	4.31	4.34	0.98	4.38	4.35	1.01	4.27	4.35	0.97	**16:00**
17:00				4.41	4.36	1.02	4.48	4.31	1.03	4.48	4.37	1.05	4.26	4.35	0.97	4.38	4.31	1.00	4.23	4.33	0.95	**17:00**
18:00				4.68	4.42	1.13	4.62	4.36	1.09	4.62	4.46	1.11	4.46	4.41	1.05	4.38	4.40	1.02	4.21	4.30	0.94	**18:00**
19:00				5.11	4.46	1.29	4.77	4.40	1.15	4.84	4.40	1.18	6.61	4.30	1.77	4.29	4.34	0.98	4.18	4.36	0.95	**19:00**
20:00				4.90	4.43	1.21	4.74	4.40	1.14	4.76	4.38	1.14	7.58	3.94	1.95	4.26	4.39	0.98	4.10	4.32	0.91	**20:00**
21:00				4.47	4.31	1.03	4.48	4.34	1.04	4.57	4.37	1.08	9.26	3.27	2.10	4.23	4.34	0.96	3.99	4.33	0.88	**21:00**
22:00				4.28	4.39	0.98	4.25	4.31	0.96	4.05	4.26	0.88	4.83	4.33	1.16	3.94	4.25	0.84	3.83	4.29	0.82	**22:00**
23:00				3.76	4.13	0.77	3.68	4.12	0.75	3.57	4.15	0.72	4.33	4.31	0.98	3.63	4.19	0.74	3.41	4.13	0.67	**23:00**
Average:				**3.86**	**4.16**	**0.83**	**3.97**	**4.14**	**0.85**	**3.90**	**4.13**	**0.83**	**4.46**	**4.17**	**1.00**	**4.09**	**4.21**	**0.91**	**3.81**	**4.13**	**0.81**	**Ave**
Minimum:				**2.52**	**3.56**	**0.37**	**2.75**	**3.44**	**0.42**	**2.64**	**3.48**	**0.39**	**2.63**	**3.27**	**0.41**	**3.00**	**3.72**	**0.50**	**2.65**	**3.51**	**0.39**	**Min**
Maximum:				**5.11**	**4.46**	**1.29**	**4.77**	**4.40**	**1.15**	**4.84**	**4.46**	**1.18**	**9.26**	**4.42**	**2.10**	**5.77**	**4.49**	**1.54**	**5.22**	**4.48**	**1.33**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 3

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**3.84**	**4.14**	**0.82**
Weekly Minimum:	**2.37**	**3.33**	**0.32**
Weekly Maximum:	**7.97**	**4.57**	**1.88**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	2.96	3.89	0.51	3.02	3.93	0.53	3.16	3.89	0.56	2.86	3.83	0.48	3.08	3.89	0.54	3.22	3.92	0.58	3.27	3.94	0.60	**0:00**
1:00	2.76	3.80	0.45	2.73	3.74	0.44	2.80	3.68	0.45	2.60	3.69	0.40	2.75	3.73	0.44	2.91	3.70	0.48	3.01	3.80	0.51	**1:00**
2:00	2.58	3.69	0.40	2.47	3.56	0.36	2.64	3.50	0.39	2.38	3.33	0.32	2.62	3.62	0.40	2.76	3.62	0.43	2.86	3.60	0.45	**2:00**
3:00	2.57	3.67	0.39	2.47	3.40	0.34	2.54	3.53	0.37	2.41	3.39	0.33	2.68	3.63	0.41	2.70	3.49	0.40	2.65	3.51	0.39	**3:00**
4:00	2.54	3.59	0.38	2.57	3.56	0.38	2.57	3.48	0.37	2.37	3.41	0.32	2.66	3.58	0.40	2.66	3.36	0.38	2.61	3.48	0.38	**4:00**
5:00	2.62	3.65	0.40	2.72	3.69	0.43	2.72	3.60	0.42	2.56	3.66	0.39	2.76	3.71	0.44	2.75	3.49	0.41	2.67	3.46	0.39	**5:00**
6:00	2.93	3.94	0.51	3.20	4.06	0.60	3.27	4.05	0.62	3.08	4.01	0.56	3.18	4.03	0.59	3.00	3.77	0.51	2.77	3.62	0.43	**6:00**
7:00	2.98	3.95	0.53	3.65	4.22	0.76	3.80	4.26	0.81	3.63	4.31	0.77	3.69	4.22	0.77	3.28	3.99	0.61	3.13	3.88	0.56	**7:00**
8:00	3.26	4.10	0.62	3.96	4.30	0.86	4.02	4.32	0.89	3.90	4.38	0.86	4.04	4.33	0.89	3.79	4.22	0.80	3.87	4.25	0.83	**8:00**
9:00	3.80	4.27	0.81	3.97	4.25	0.85	3.91	4.28	0.84	3.72	4.29	0.79	3.97	4.30	0.87	4.30	4.35	0.98	4.53	4.45	1.08	**9:00**
10:00	4.32	4.33	0.98	4.10	4.35	0.92	3.94	4.26	0.85	3.78	4.34	0.81	4.14	4.36	0.93	4.81	4.45	1.18	4.93	4.45	1.22	**10:00**
11:00	4.59	4.54	1.12	4.32	4.35	0.99	4.02	4.37	0.90	3.96	4.34	0.87	4.43	4.40	1.03	5.24	4.48	1.34	5.06	4.44	1.26	**11:00**
12:00	4.52	4.41	1.07	4.36	4.34	1.00	4.02	4.30	0.88	3.95	4.31	0.86	4.55	4.36	1.06	5.23	4.49	1.34	4.85	4.39	1.18	**12:00**
13:00	4.39	4.38	1.02	4.29	4.33	0.98	4.00	4.34	0.88	4.03	4.31	0.89	4.45	4.33	1.03	5.11	4.46	1.29	4.70	4.38	1.12	**13:00**
14:00	4.27	4.33	0.97	4.27	4.32	0.97	4.04	4.33	0.89	4.05	4.34	0.90	4.37	4.38	1.01	4.78	4.44	1.16	4.36	4.33	1.00	**14:00**
15:00	4.30	4.35	0.98	4.23	4.34	0.96	4.01	4.26	0.87	4.12	4.30	0.91	7.97	3.67	1.88	4.66	4.37	1.11	4.32	4.30	0.98	**15:00**
16:00	4.26	4.38	0.98	4.09	4.26	0.89	4.18	4.35	0.94	4.13	4.41	0.94	5.05	4.38	1.24	4.64	4.38	1.10	4.42	4.34	1.02	**16:00**
17:00	4.25	4.43	0.98	4.26	4.31	0.96	4.32	4.36	0.99	4.28	4.40	0.99	4.40	4.38	1.02	4.45	4.36	1.03	4.36	4.32	0.99	**17:00**
18:00	4.58	4.49	1.11	4.49	4.42	1.06	4.67	4.49	1.14	4.55	4.45	1.09	4.37	4.40	1.02	4.46	4.37	1.04	4.34	4.33	0.99	**18:00**
19:00	4.87	4.49	1.21	4.92	4.45	1.22	4.90	4.57	1.24	4.71	4.44	1.14	4.49	4.35	1.04	4.45	4.34	1.03	4.58	4.33	1.07	**19:00**
20:00	4.80	4.45	1.17	4.88	4.46	1.21	4.53	4.44	1.08	4.76	4.43	1.15	4.34	4.36	1.00	4.25	4.36	0.97	4.65	4.34	1.09	**20:00**
21:00	4.64	4.46	1.12	4.67	4.38	1.11	4.44	4.41	1.04	4.56	4.37	1.07	4.22	4.33	0.95	4.19	4.31	0.94	4.65	4.41	1.11	**21:00**
22:00	4.08	4.33	0.91	4.23	4.34	0.96	4.13	4.36	0.93	4.15	4.33	0.93	4.06	4.32	0.89	3.95	4.25	0.85	4.33	4.20	0.96	**22:00**
23:00	3.52	4.17	0.71	3.65	4.13	0.74	3.48	4.19	0.70	3.55	4.12	0.71	3.59	4.08	0.71	3.61	4.08	0.72	3.74	4.14	0.77	**23:00**
Average:	**3.77**	**4.17**	**0.81**	**3.81**	**4.14**	**0.81**	**3.75**	**4.15**	**0.79**	**3.67**	**4.13**	**0.77**	**3.99**	**4.13**	**0.86**	**3.97**	**4.13**	**0.86**	**3.94**	**4.11**	**0.85**	**Ave**
Minimum:	**2.54**	**3.59**	**0.38**	**2.47**	**3.40**	**0.34**	**2.54**	**3.48**	**0.37**	**2.37**	**3.33**	**0.32**	**2.62**	**3.58**	**0.40**	**2.66**	**3.36**	**0.38**	**2.61**	**3.46**	**0.38**	**Min**
Maximum:	**4.87**	**4.54**	**1.21**	**4.92**	**4.46**	**1.22**	**4.90**	**4.57**	**1.24**	**4.76**	**4.45**	**1.15**	**7.97**	**4.40**	**1.88**	**5.24**	**4.49**	**1.34**	**5.06**	**4.45**	**1.26**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

Monitoring Site:

Site 3

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**3.80**	**4.13**	**0.80**
Weekly Minimum:	**2.44**	**3.39**	**0.34**
Weekly Maximum:	**5.11**	**4.45**	**1.27**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	3.23	3.91	0.58	3.17	3.90	0.57	3.04	3.88	0.53	3.04	3.97	0.54										**0:00**
1:00	2.90	3.69	0.47	2.76	3.68	0.44	2.74	3.64	0.43	2.64	3.64	0.41										**1:00**
2:00	2.70	3.46	0.40	2.62	3.54	0.39	2.59	3.55	0.39	2.53	3.53	0.37										**2:00**
3:00	2.64	3.46	0.39	2.54	3.39	0.36	2.52	3.47	0.36	2.47	3.48	0.35										**3:00**
4:00	2.69	3.41	0.39	2.58	3.40	0.37	2.55	3.51	0.37	2.44	3.45	0.34										**4:00**
5:00	2.85	3.61	0.45	2.70	3.58	0.41	2.62	3.65	0.40	2.58	3.59	0.39										**5:00**
6:00	3.37	4.02	0.64	3.12	3.97	0.57	3.06	3.94	0.55	3.10	4.07	0.58										**6:00**
7:00	3.88	4.18	0.81	3.71	4.21	0.77	3.57	4.26	0.74	3.66	4.27	0.77										**7:00**
8:00	4.34	4.34	0.99	4.08	4.32	0.90	3.90	4.31	0.85	3.99	4.38	0.89										**8:00**
9:00	4.21	4.29	0.94	4.06	4.29	0.89	3.82	4.25	0.81	3.86	4.26	0.82										**9:00**
10:00	4.34	4.34	0.99	4.04	4.33	0.89	3.92	4.28	0.85	4.07	4.29	0.89										**10:00**
11:00	4.53	4.34	1.06	4.12	4.32	0.92	4.03	4.30	0.89	4.22	4.37	0.96										**11:00**
12:00	4.63	4.34	1.09	4.17	4.31	0.93	3.96	4.32	0.87	4.51	4.35	1.05										**12:00**
13:00	4.46	4.28	1.02	4.02	4.29	0.88	4.09	4.35	0.92	4.36	4.38	1.01										**13:00**
14:00	4.49	4.32	1.04	3.97	4.30	0.87	3.93	4.29	0.85	4.18	4.30	0.93										**14:00**
15:00	4.50	4.31	1.04	3.99	4.30	0.87	3.92	4.35	0.86	4.21	4.29	0.94										**15:00**
16:00	4.44	4.31	1.02	3.99	4.34	0.88	3.89	4.32	0.85	4.22	4.31	0.95										**16:00**
17:00	4.50	4.35	1.05	4.10	4.30	0.91	4.17	4.34	0.94	4.24	4.33	0.96										**17:00**
18:00	4.80	4.39	1.16	4.33	4.36	0.99	4.34	4.45	1.02	4.50	4.34	1.05										**18:00**
19:00	5.11	4.40	1.27	4.68	4.45	1.13	4.61	4.42	1.10	4.99	4.40	1.23										**19:00**
20:00	5.01	4.37	1.23	4.40	4.39	1.02	4.49	4.35	1.04	5.04	4.42	1.25										**20:00**
21:00	4.56	4.38	1.08	4.42	4.34	1.02	4.38	4.35	1.01	4.77	4.37	1.14										**21:00**
22:00	4.14	4.30	0.92	4.17	4.33	0.94	4.03	4.37	0.90	4.45	4.30	1.02										**22:00**
23:00	3.60	4.10	0.72	3.51	4.14	0.70	3.49	4.18	0.70	3.82	4.19	0.80										**23:00**
Average:	**4.00**	**4.12**	**0.86**	**3.72**	**4.11**	**0.78**	**3.65**	**4.13**	**0.76**	**3.83**	**4.14**	**0.82**										**Ave**
Minimum:	**2.64**	**3.41**	**0.39**	**2.54**	**3.39**	**0.36**	**2.52**	**3.47**	**0.36**	**2.44**	**3.45**	**0.34**										**Min**
Maximum:	**5.11**	**4.40**	**1.27**	**4.68**	**4.45**	**1.13**	**4.61**	**4.45**	**1.10**	**5.04**	**4.42**	**1.25**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Temporary Flow Monitoring Study

Sanitary Sewer Collection System

Monitoring Site: Site 4

Location: MH ID 6614: Pecan Avenue (Ave 13), east of Stadium Road (Road 26 1/2) between Monterey Street and Concord Court

Size / Type Line: 21-inch Sanitary Sewer Pipe

Data Summary Report

Site Information Report

Monitoring Site: Site 4

Location: MH ID 6614: Pecan Avenue (Ave 13), east of Stadium Road (Road 26 1/2) between Monterey Street and Concord Court

Latitude: 36.938293°

Longitude: -120.059918°

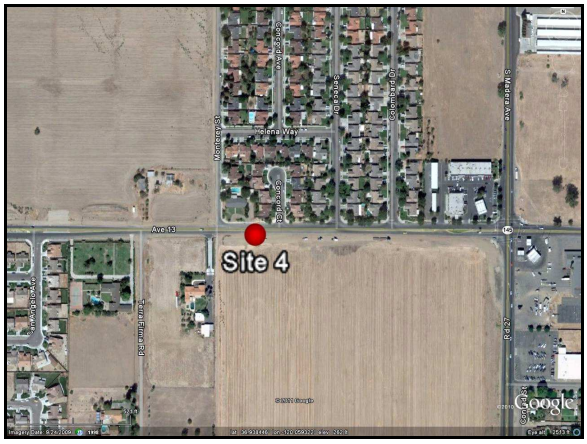
Rim Elevation: 262 *feet above sea level*

Diameter: 21 *inches*

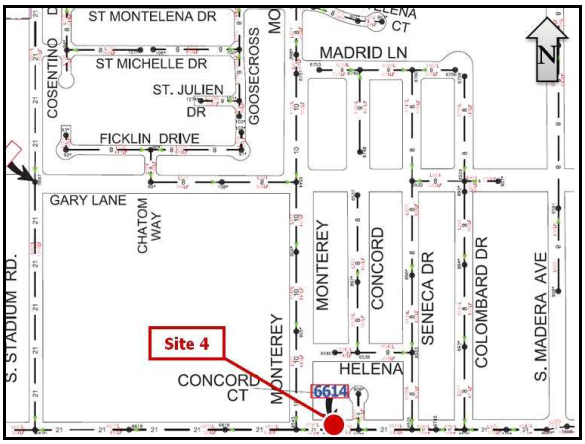
Average Dry Weather Flow: 0.606 *mgd*

Peak Measured Flow: 1.426 *mgd*

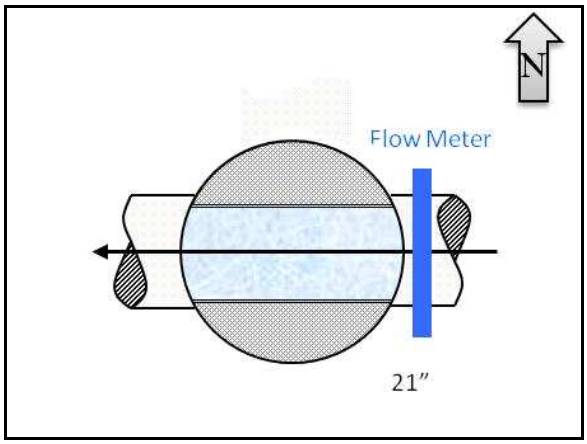
Satellite Map



Sanitary Map



Flow Schematic

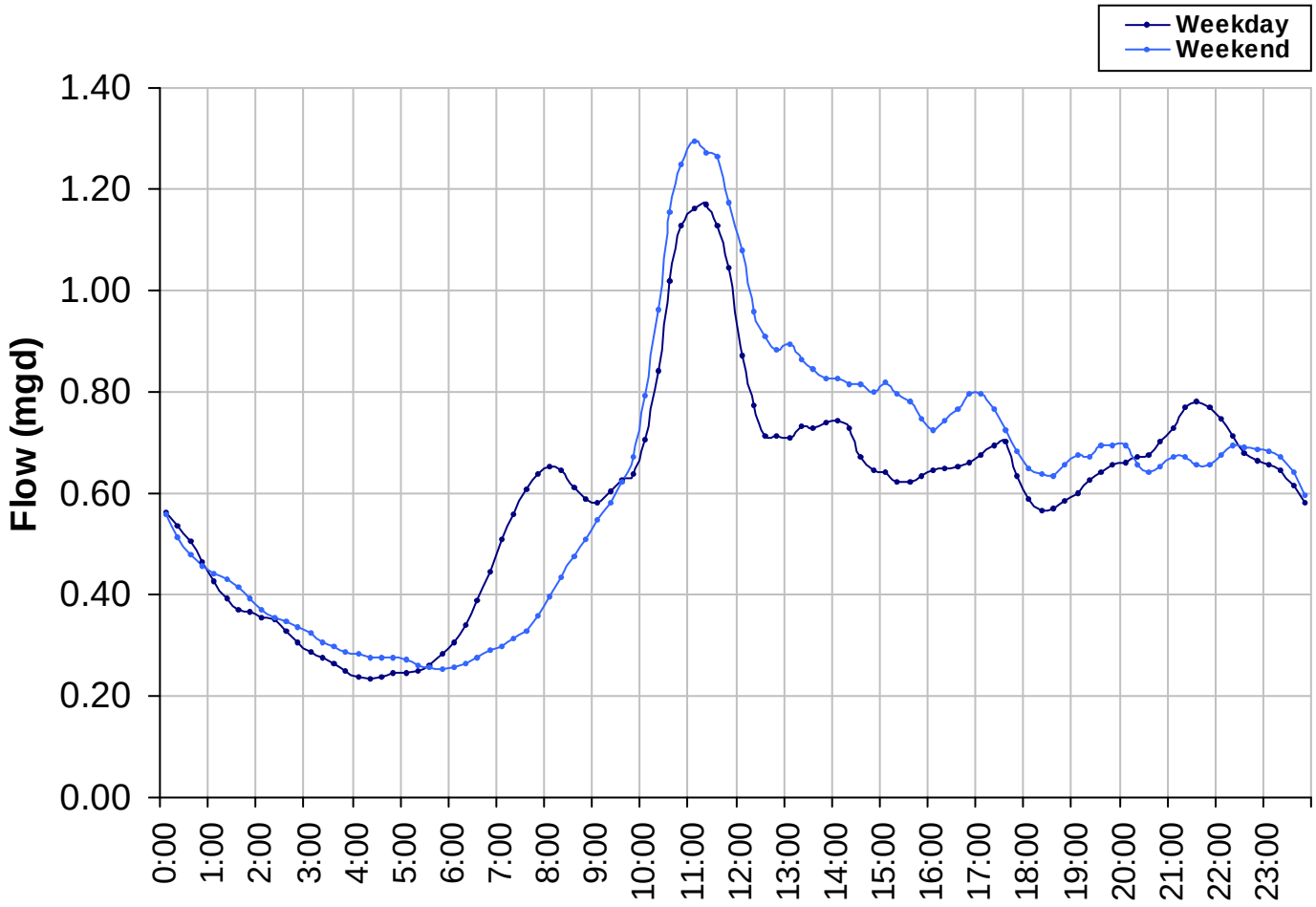


Street View Photo

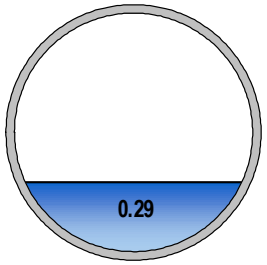
Plan View Photo

Average Dry Weather Flow

Monitoring Site:
Site 4



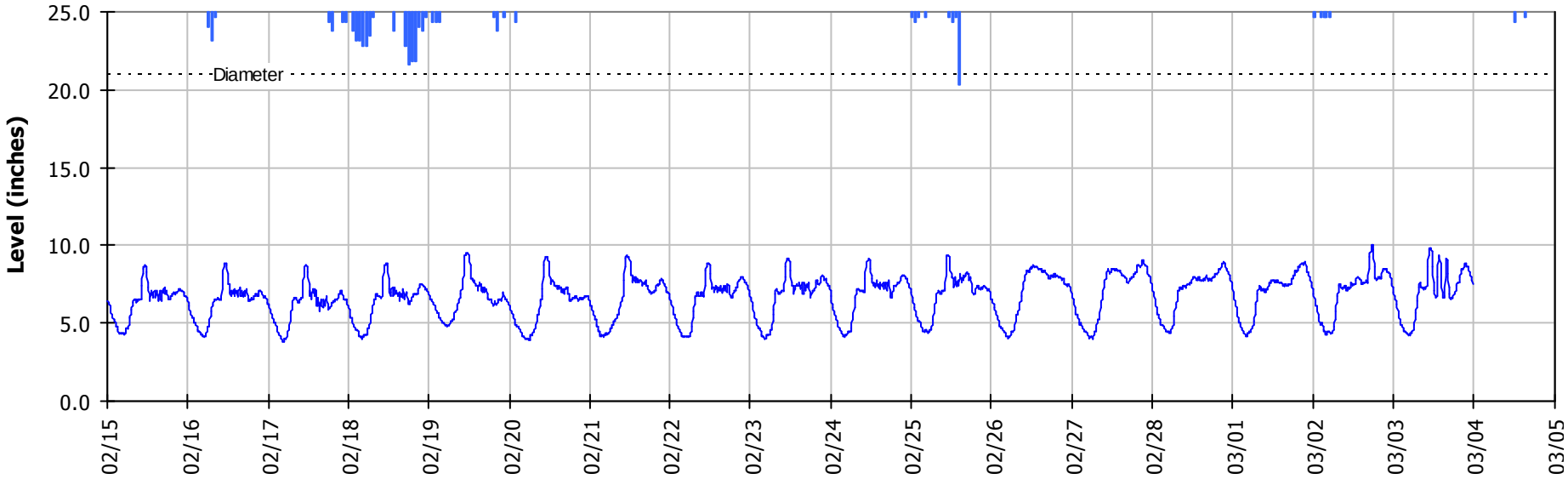
Average Dry Weather Flow:
0.606 *mgd*

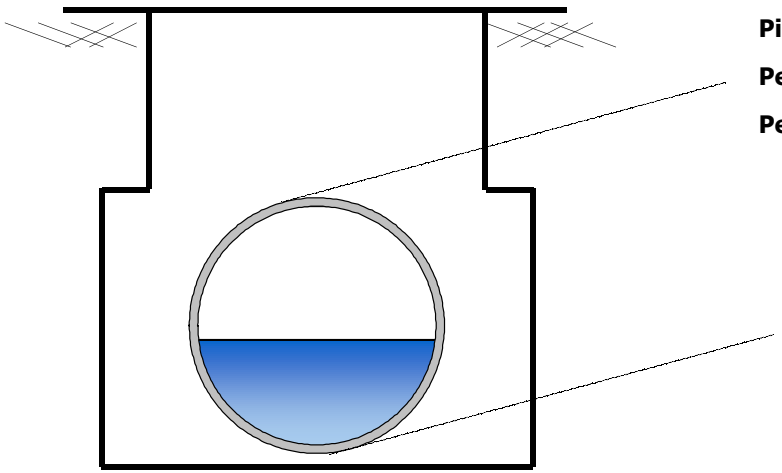


Site Capacity / Surcharge Summary

Monitoring Site:
Site 4

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 21 *inches*

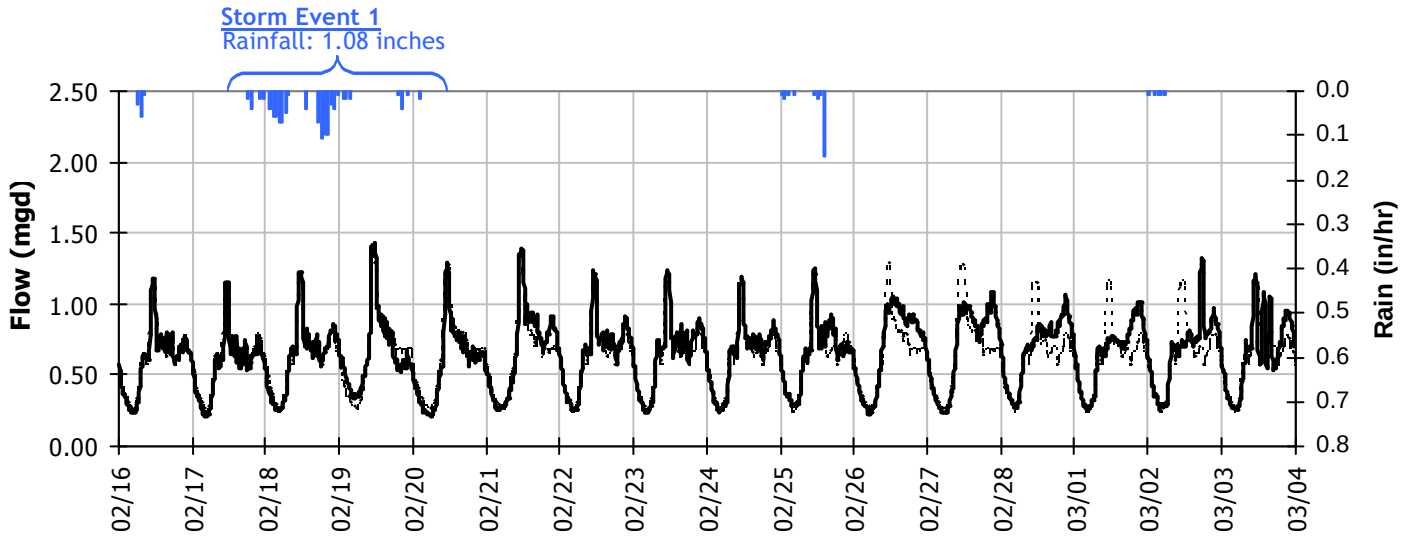
Peak Measured Level: 9.55 *inches*

Peak d/D Ratio: 0.45

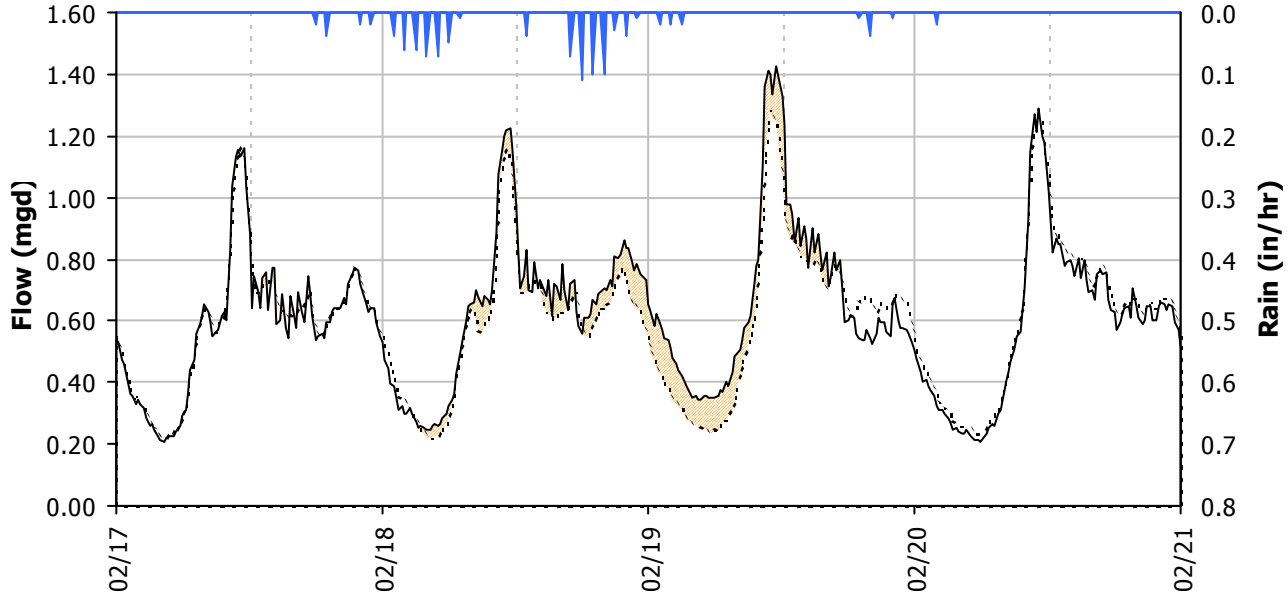
I/I Summary
Event 1

Monitoring Site:
Site 4

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

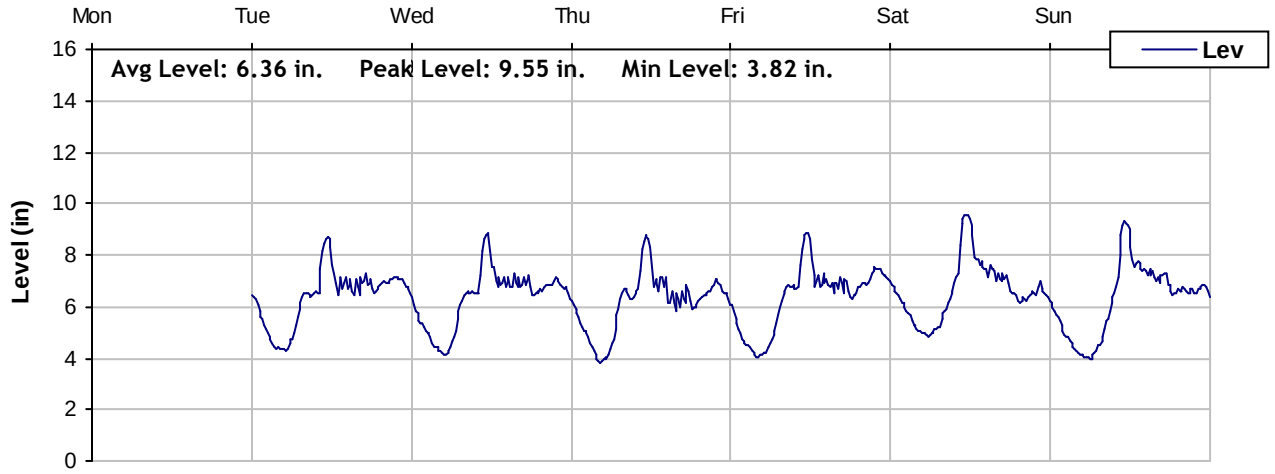


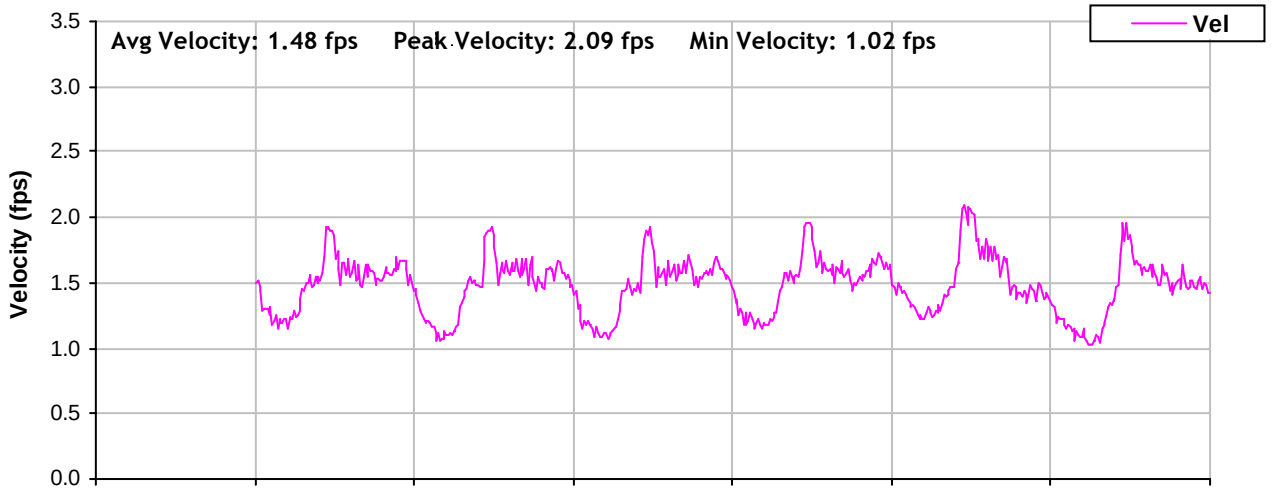
Storm Event #1 I/I Analysis

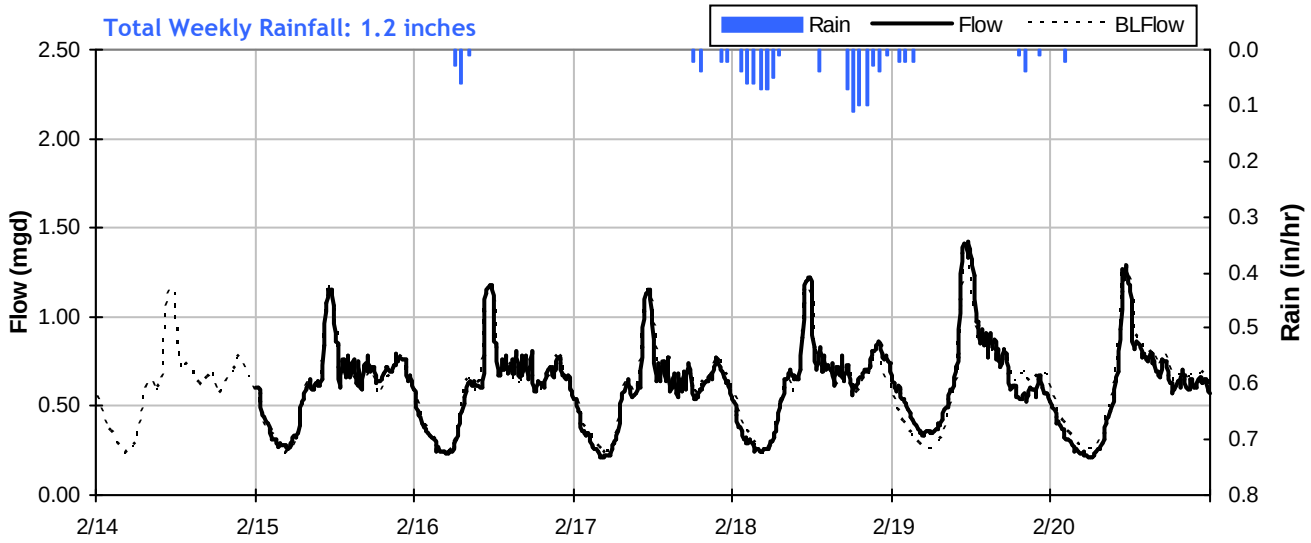
Rainfall:	1.08	*inches*			
Peak Flow:	1.43	*mgd*	**PF:**	2.35	
Peak I/I Rate:	0.28	*mgd*	**Pk I/I:ADWF:**	0.46	
Peak Level:	9.55	*inches*	**d/D Ratio:**	0.45	
Total I/I:	119,000	*gallons*	**I/I per ADWF:**	0.18	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.11	*mgd*	**RDI (% of BL):**	17%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 4



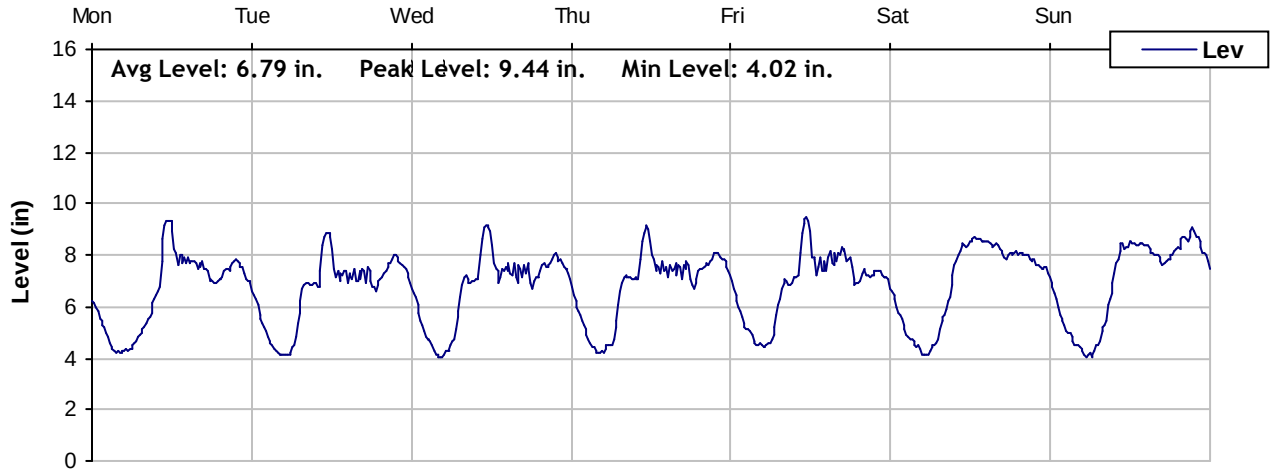


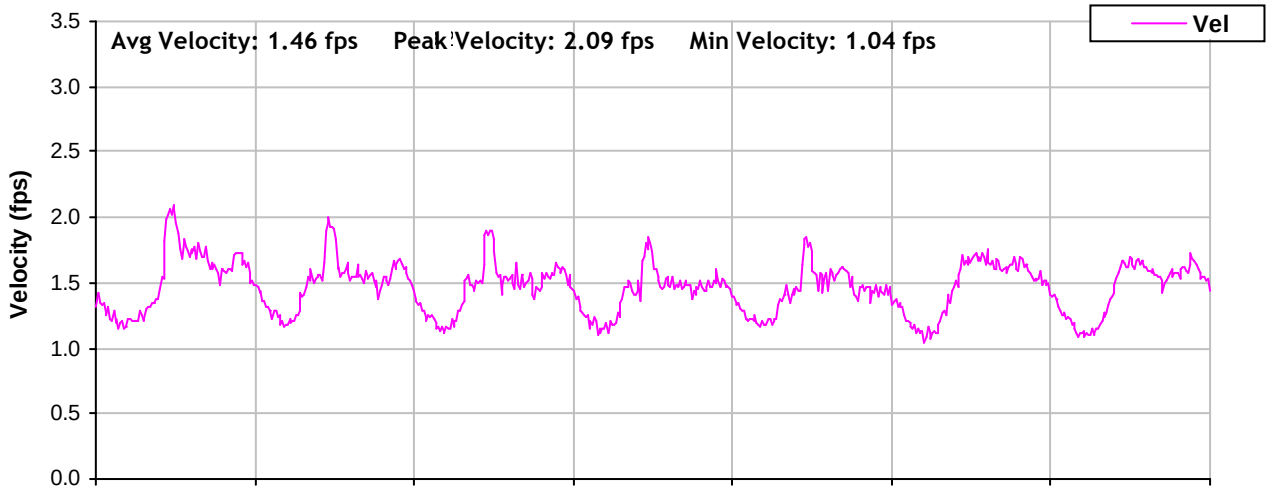


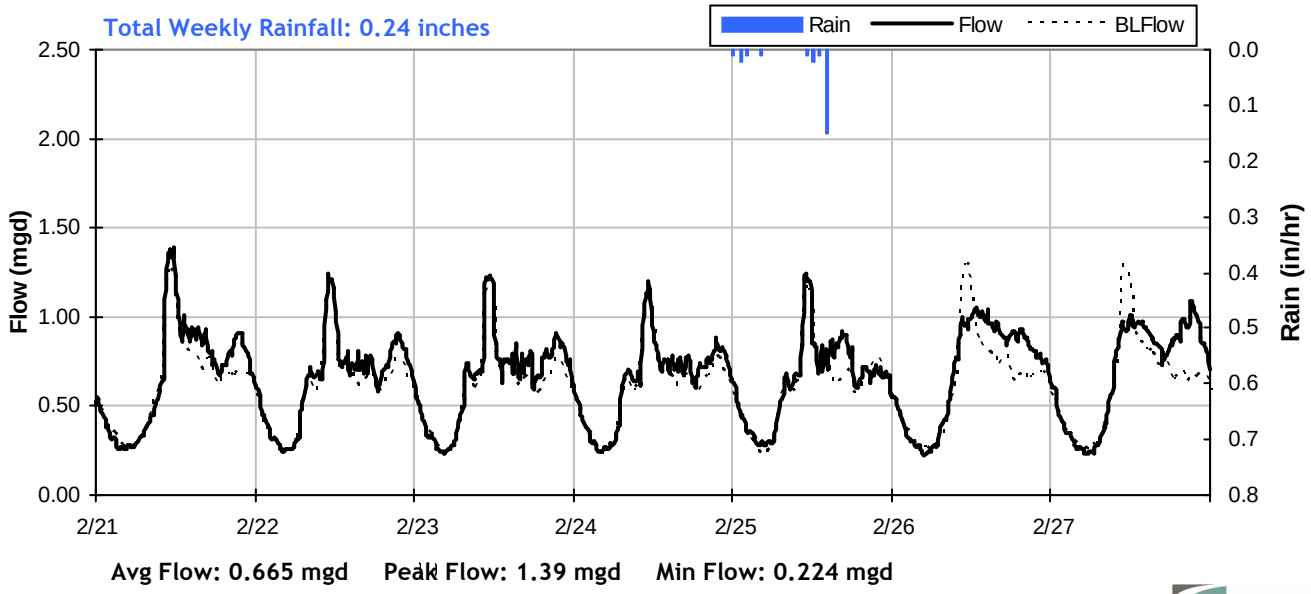
Avg Flow: 0.613 mgd Peak Flow: 1.426 mgd Min Flow: 0.209 mgd

Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 4

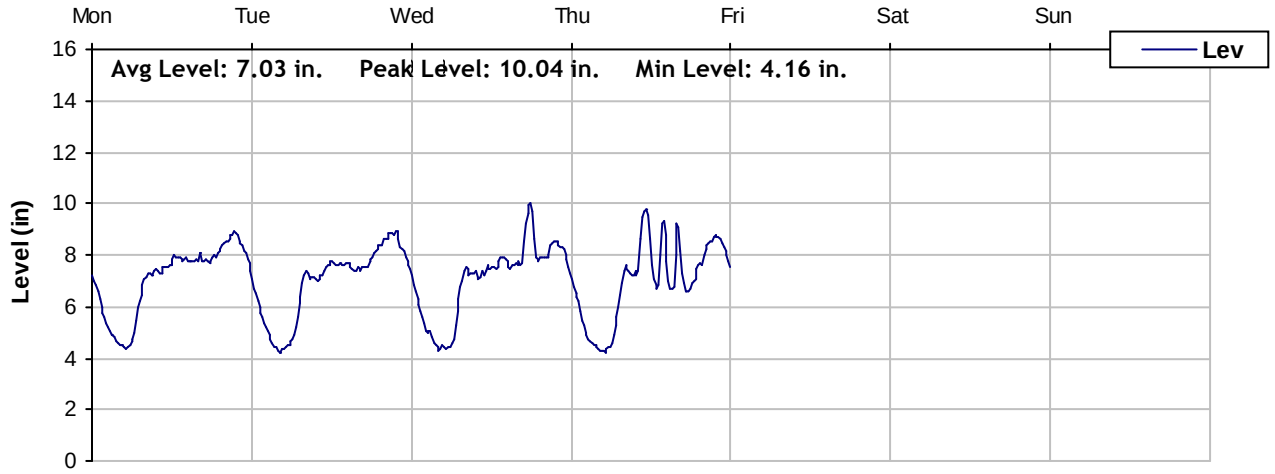


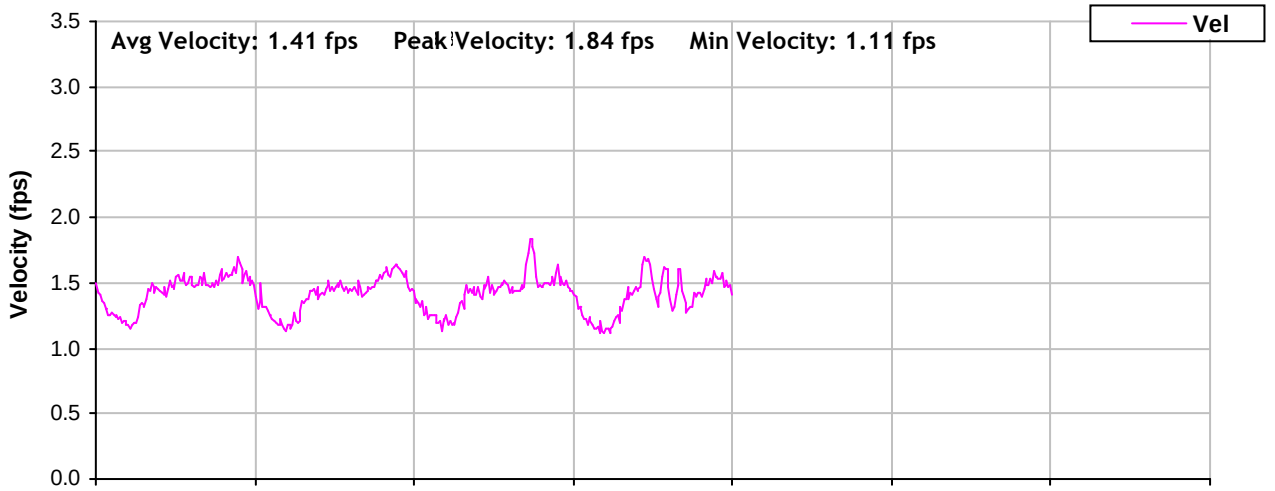


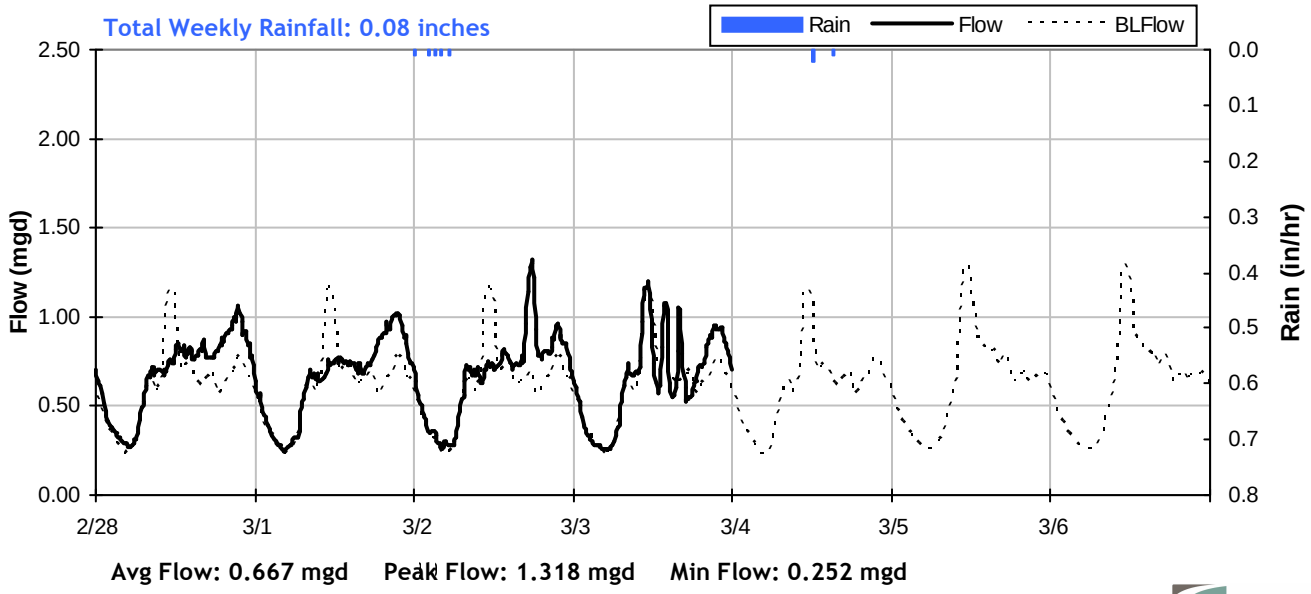


Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 4







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 4

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**6.36**	**1.48**	**0.61**
Weekly Minimum:	**3.90**	**1.05**	**0.22**
Weekly Maximum:	**9.50**	**2.03**	**1.39**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00				6.30	1.48	0.58	5.99	1.37	0.50	6.00	1.37	0.50	5.70	1.35	0.46	6.68	1.46	0.62	5.92	1.27	0.46	**0:00**
1:00				5.56	1.30	0.43	5.35	1.22	0.38	5.30	1.18	0.36	4.88	1.26	0.34	6.26	1.45	0.56	5.33	1.23	0.38	**1:00**
2:00				4.92	1.23	0.34	4.93	1.18	0.33	4.84	1.17	0.32	4.53	1.22	0.30	5.78	1.36	0.47	4.79	1.16	0.31	**2:00**
3:00				4.44	1.20	0.29	4.47	1.09	0.26	4.21	1.11	0.25	4.16	1.18	0.26	5.27	1.29	0.40	4.39	1.12	0.26	**3:00**
4:00				4.35	1.19	0.28	4.23	1.09	0.24	3.90	1.10	0.22	4.14	1.17	0.25	4.99	1.23	0.35	4.12	1.10	0.24	**4:00**
5:00				4.48	1.24	0.30	4.26	1.11	0.25	4.12	1.10	0.24	4.34	1.20	0.28	4.88	1.29	0.35	4.01	1.05	0.22	**5:00**
6:00				5.14	1.29	0.38	5.00	1.20	0.34	5.06	1.21	0.35	4.99	1.29	0.36	5.04	1.26	0.36	4.15	1.06	0.23	**6:00**
7:00				6.22	1.47	0.57	6.13	1.41	0.53	6.34	1.41	0.56	6.11	1.51	0.57	5.42	1.34	0.43	4.61	1.12	0.28	**7:00**
8:00				6.46	1.51	0.61	6.53	1.52	0.63	6.51	1.48	0.61	6.75	1.56	0.67	6.05	1.42	0.53	5.47	1.28	0.41	**8:00**
9:00				6.52	1.53	0.63	6.55	1.49	0.62	6.46	1.47	0.60	6.73	1.54	0.66	6.80	1.55	0.68	6.47	1.39	0.56	**9:00**
10:00				7.65	1.78	0.92	7.64	1.71	0.89	7.70	1.71	0.90	7.81	1.80	0.95	8.59	1.93	1.17	8.26	1.76	1.01	**10:00**
11:00				8.58	1.89	1.13	8.64	1.89	1.14	8.59	1.87	1.12	8.76	1.95	1.20	9.50	2.03	1.39	9.18	1.87	1.22	**11:00**
12:00				7.07	1.63	0.75	7.21	1.62	0.77	7.09	1.60	0.74	7.22	1.71	0.81	8.43	1.93	1.13	7.84	1.71	0.91	**12:00**
13:00				6.92	1.62	0.73	7.03	1.63	0.74	6.91	1.55	0.69	7.04	1.65	0.75	7.69	1.73	0.89	7.58	1.61	0.82	**13:00**
14:00				6.94	1.62	0.72	6.91	1.60	0.71	6.64	1.61	0.68	6.90	1.59	0.71	7.40	1.76	0.86	7.36	1.61	0.78	**14:00**
15:00				6.72	1.54	0.66	6.94	1.60	0.72	6.28	1.59	0.62	6.81	1.59	0.70	7.32	1.71	0.82	7.17	1.58	0.74	**15:00**
16:00				6.88	1.55	0.69	6.96	1.63	0.74	6.30	1.60	0.63	6.82	1.59	0.70	7.18	1.64	0.77	7.14	1.55	0.72	**16:00**
17:00				7.02	1.60	0.73	6.98	1.60	0.72	6.43	1.64	0.66	6.74	1.54	0.67	6.97	1.57	0.71	7.07	1.53	0.70	**17:00**
18:00				6.62	1.52	0.64	6.51	1.50	0.61	5.98	1.50	0.55	6.44	1.48	0.60	6.50	1.45	0.60	6.52	1.47	0.60	**18:00**
19:00				6.88	1.57	0.69	6.65	1.51	0.64	6.33	1.56	0.61	6.84	1.54	0.68	6.25	1.42	0.55	6.67	1.53	0.65	**19:00**
20:00				6.96	1.57	0.71	6.84	1.61	0.71	6.55	1.58	0.65	6.93	1.59	0.71	6.33	1.41	0.56	6.56	1.48	0.61	**20:00**
21:00				7.14	1.64	0.76	7.01	1.61	0.73	6.94	1.68	0.75	7.42	1.67	0.82	6.52	1.40	0.58	6.61	1.49	0.62	**21:00**
22:00				7.02	1.65	0.75	6.81	1.58	0.69	6.71	1.58	0.68	7.36	1.66	0.81	6.76	1.44	0.63	6.74	1.51	0.65	**22:00**
23:00				6.66	1.49	0.63	6.57	1.51	0.63	6.37	1.53	0.61	7.11	1.63	0.75	6.39	1.39	0.56	6.64	1.46	0.62	**23:00**
Average:				**6.39**	**1.50**	**0.62**	**6.34**	**1.47**	**0.60**	**6.15**	**1.47**	**0.58**	**6.35**	**1.51**	**0.63**	**6.63**	**1.52**	**0.66**	**6.27**	**1.41**	**0.58**	**Ave**
Minimum:				**4.35**	**1.19**	**0.28**	**4.23**	**1.09**	**0.24**	**3.90**	**1.10**	**0.22**	**4.14**	**1.17**	**0.25**	**4.88**	**1.23**	**0.35**	**4.01**	**1.05**	**0.22**	**Min**
Maximum:				**8.58**	**1.89**	**1.13**	**8.64**	**1.89**	**1.14**	**8.59**	**1.87**	**1.12**	**8.76**	**1.95**	**1.20**	**9.50**	**2.03**	**1.39**	**9.18**	**1.87**	**1.22**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 4

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**6.79**	**1.46**	**0.66**
Weekly Minimum:	**4.06**	**1.10**	**0.24**
Weekly Maximum:	**9.33**	**2.06**	**1.37**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	6.03	1.35	0.50	6.35	1.45	0.57	6.40	1.35	0.54	6.33	1.38	0.54	6.59	1.35	0.56	6.33	1.35	0.53	6.64	1.39	0.58	**0:00**
1:00	5.37	1.30	0.41	5.46	1.34	0.43	5.37	1.25	0.39	5.55	1.26	0.42	5.70	1.28	0.43	5.55	1.31	0.43	5.67	1.27	0.43	**1:00**
2:00	4.69	1.23	0.32	4.79	1.27	0.34	4.74	1.25	0.33	4.83	1.20	0.32	5.13	1.22	0.36	4.88	1.19	0.33	5.06	1.24	0.36	**2:00**
3:00	4.24	1.19	0.27	4.33	1.24	0.29	4.25	1.16	0.26	4.33	1.17	0.27	4.65	1.20	0.31	4.59	1.15	0.29	4.62	1.16	0.29	**3:00**
4:00	4.23	1.17	0.26	4.13	1.18	0.25	4.06	1.14	0.24	4.22	1.15	0.26	4.50	1.18	0.29	4.26	1.10	0.25	4.38	1.11	0.26	**4:00**
5:00	4.34	1.21	0.28	4.15	1.21	0.26	4.32	1.17	0.27	4.45	1.17	0.28	4.53	1.20	0.29	4.16	1.11	0.24	4.11	1.10	0.24	**5:00**
6:00	4.64	1.24	0.32	4.77	1.31	0.35	5.01	1.25	0.36	4.98	1.22	0.35	5.13	1.25	0.37	4.53	1.12	0.28	4.24	1.13	0.25	**6:00**
7:00	5.06	1.26	0.36	6.37	1.45	0.58	6.55	1.43	0.60	6.68	1.41	0.60	6.50	1.39	0.57	5.23	1.24	0.38	4.85	1.20	0.33	**7:00**
8:00	5.54	1.34	0.44	6.87	1.54	0.68	7.02	1.50	0.68	7.16	1.48	0.69	6.95	1.41	0.63	6.05	1.35	0.50	5.77	1.31	0.46	**8:00**
9:00	6.36	1.44	0.57	6.87	1.55	0.68	7.04	1.49	0.68	7.08	1.44	0.66	7.08	1.44	0.66	7.35	1.47	0.71	7.12	1.46	0.68	**9:00**
10:00	8.07	1.84	1.03	7.79	1.75	0.93	8.21	1.73	0.98	8.08	1.63	0.91	8.34	1.68	0.98	8.25	1.63	0.92	8.23	1.63	0.92	**10:00**
11:00	9.33	2.06	1.37	8.83	1.94	1.20	9.07	1.87	1.20	8.95	1.79	1.13	9.27	1.80	1.19	8.42	1.66	0.97	8.34	1.65	0.95	**11:00**
12:00	8.22	1.82	1.03	7.56	1.65	0.83	7.73	1.61	0.84	7.72	1.57	0.81	7.75	1.54	0.81	8.62	1.70	1.02	8.47	1.65	0.96	**12:00**
13:00	7.91	1.79	0.96	7.22	1.60	0.76	7.23	1.50	0.71	7.48	1.48	0.73	7.55	1.52	0.77	8.58	1.70	1.01	8.44	1.67	0.97	**13:00**
14:00	7.76	1.74	0.91	7.20	1.54	0.73	7.47	1.53	0.76	7.43	1.51	0.74	7.80	1.55	0.81	8.53	1.67	0.99	8.35	1.61	0.92	**14:00**
15:00	7.66	1.73	0.89	7.13	1.57	0.73	7.24	1.51	0.72	7.43	1.47	0.72	7.79	1.56	0.82	8.39	1.64	0.95	8.04	1.58	0.86	**15:00**
16:00	7.61	1.71	0.87	7.32	1.54	0.74	7.44	1.52	0.75	7.47	1.49	0.74	8.14	1.61	0.90	8.22	1.62	0.91	7.83	1.51	0.80	**16:00**
17:00	7.24	1.62	0.77	7.20	1.54	0.73	7.35	1.51	0.73	7.49	1.46	0.73	7.94	1.55	0.83	7.97	1.61	0.87	7.78	1.53	0.80	**17:00**
18:00	6.97	1.57	0.71	6.75	1.45	0.62	6.99	1.43	0.65	6.88	1.44	0.64	7.09	1.41	0.65	8.08	1.64	0.90	8.08	1.58	0.87	**18:00**
19:00	7.15	1.60	0.74	7.12	1.53	0.71	7.46	1.53	0.76	7.45	1.46	0.72	7.18	1.46	0.69	8.04	1.65	0.90	8.48	1.59	0.93	**19:00**
20:00	7.48	1.63	0.81	7.59	1.57	0.80	7.60	1.55	0.79	7.65	1.48	0.76	7.25	1.43	0.68	7.97	1.61	0.87	8.65	1.61	0.97	**20:00**
21:00	7.78	1.73	0.90	7.94	1.65	0.89	7.98	1.61	0.87	8.01	1.54	0.84	7.33	1.43	0.69	7.78	1.53	0.80	8.92	1.68	1.06	**21:00**
22:00	7.52	1.64	0.82	7.65	1.61	0.82	7.73	1.59	0.82	7.92	1.50	0.80	7.26	1.44	0.68	7.51	1.53	0.76	8.42	1.57	0.91	**22:00**
23:00	6.99	1.52	0.69	7.21	1.49	0.71	7.31	1.49	0.72	7.49	1.45	0.72	7.02	1.45	0.66	7.39	1.48	0.72	7.84	1.50	0.80	**23:00**
Average:	**6.59**	**1.53**	**0.68**	**6.61**	**1.50**	**0.65**	**6.73**	**1.46**	**0.65**	**6.79**	**1.42**	**0.64**	**6.85**	**1.43**	**0.65**	**6.94**	**1.46**	**0.69**	**7.01**	**1.45**	**0.69**	**Ave**
Minimum:	**4.23**	**1.17**	**0.26**	**4.13**	**1.18**	**0.25**	**4.06**	**1.14**	**0.24**	**4.22**	**1.15**	**0.26**	**4.50**	**1.18**	**0.29**	**4.16**	**1.10**	**0.24**	**4.11**	**1.10**	**0.24**	**Min**
Maximum:	**9.33**	**2.06**	**1.37**	**8.83**	**1.94**	**1.20**	**9.07**	**1.87**	**1.20**	**8.95**	**1.79**	**1.13**	**9.27**	**1.80**	**1.19**	**8.62**	**1.70**	**1.02**	**8.92**	**1.68**	**1.06**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

Monitoring Site:

Site 4

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**7.03**	**1.41**	**0.67**
Weekly Minimum:	**4.25**	**1.14**	**0.26**
Weekly Maximum:	**9.72**	**1.79**	**1.26**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	6.92	1.42	0.63	6.59	1.39	0.58	6.72	1.35	0.58	6.67	1.35	0.57										**0:00**
1:00	5.88	1.30	0.46	5.66	1.30	0.44	5.72	1.30	0.45	5.68	1.26	0.43										**1:00**
2:00	5.13	1.26	0.37	4.97	1.24	0.35	5.02	1.25	0.36	4.82	1.20	0.32										**2:00**
3:00	4.74	1.23	0.32	4.42	1.19	0.28	4.56	1.21	0.30	4.51	1.14	0.28										**3:00**
4:00	4.49	1.19	0.29	4.25	1.16	0.26	4.39	1.20	0.28	4.29	1.15	0.26										**4:00**
5:00	4.50	1.18	0.29	4.53	1.19	0.29	4.40	1.19	0.28	4.35	1.14	0.27										**5:00**
6:00	5.28	1.28	0.40	5.11	1.22	0.36	5.12	1.26	0.37	5.09	1.22	0.36										**6:00**
7:00	6.66	1.37	0.58	6.68	1.36	0.58	6.87	1.39	0.62	6.70	1.34	0.57										**7:00**
8:00	7.24	1.45	0.69	7.23	1.44	0.68	7.37	1.44	0.70	7.42	1.42	0.70										**8:00**
9:00	7.35	1.44	0.70	7.10	1.42	0.65	7.30	1.42	0.68	7.26	1.44	0.69										**9:00**
10:00	7.41	1.43	0.70	7.24	1.44	0.68	7.20	1.45	0.69	8.78	1.62	1.00										**10:00**
11:00	7.58	1.48	0.75	7.63	1.46	0.74	7.47	1.47	0.73	9.46	1.63	1.11										**11:00**
12:00	7.93	1.54	0.83	7.66	1.49	0.76	7.53	1.44	0.72	7.05	1.38	0.63										**12:00**
13:00	7.83	1.52	0.80	7.67	1.45	0.75	7.88	1.50	0.80	8.35	1.55	0.90										**13:00**
14:00	7.81	1.51	0.79	7.65	1.44	0.74	7.65	1.45	0.74	7.57	1.44	0.73										**14:00**
15:00	7.80	1.51	0.80	7.43	1.46	0.72	7.64	1.44	0.73	7.67	1.45	0.76										**15:00**
16:00	7.92	1.51	0.81	7.48	1.41	0.70	7.91	1.51	0.81	7.88	1.45	0.78										**16:00**
17:00	7.77	1.48	0.77	7.69	1.46	0.75	9.72	1.79	1.26	6.62	1.30	0.55										**17:00**
18:00	7.99	1.54	0.84	8.10	1.53	0.84	8.54	1.56	0.93	7.10	1.41	0.65										**18:00**
19:00	8.29	1.54	0.88	8.44	1.58	0.92	7.88	1.48	0.79	7.74	1.45	0.75										**19:00**
20:00	8.63	1.58	0.95	8.72	1.59	0.97	8.06	1.51	0.83	8.41	1.50	0.87										**20:00**
21:00	8.82	1.63	1.01	8.85	1.63	1.01	8.52	1.54	0.91	8.70	1.55	0.94										**21:00**
22:00	8.39	1.54	0.89	8.34	1.56	0.90	8.32	1.51	0.86	8.51	1.52	0.89										**22:00**
23:00	7.74	1.51	0.79	7.69	1.45	0.75	7.68	1.45	0.74	7.85	1.47	0.78										**23:00**
Average:	**7.09**	**1.43**	**0.68**	**6.96**	**1.41**	**0.65**	**7.06**	**1.42**	**0.67**	**7.02**	**1.39**	**0.66**										**Ave**
Minimum:	**4.49**	**1.18**	**0.29**	**4.25**	**1.16**	**0.26**	**4.39**	**1.19**	**0.28**	**4.29**	**1.14**	**0.26**										**Min**
Maximum:	**8.82**	**1.63**	**1.01**	**8.85**	**1.63**	**1.01**	**9.72**	**1.79**	**1.26**	**9.46**	**1.63**	**1.11**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Temporary Flow Monitoring Study

Sanitary Sewer Collection System

Monitoring Site: Site 5

Location: MH ID 4372: Schnoor Avenue, north of Jennings Street

Size / Type Line: 42-inch Sanitary Sewer Pipe

Data Summary Report

Site Information Report

Monitoring Site: Site 5

Location: MH ID 4372: Schnoor Avenue, north of Jennings Street

Latitude: 36.955933°

Longitude: -120.083154°

Rim Elevation: 262 *feet above sea level*

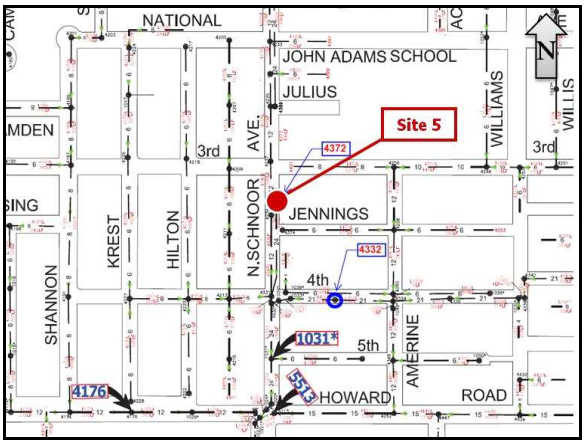
Diameter: 24 *inches*

Average Dry Weather Flow: 1.264 *mgd*

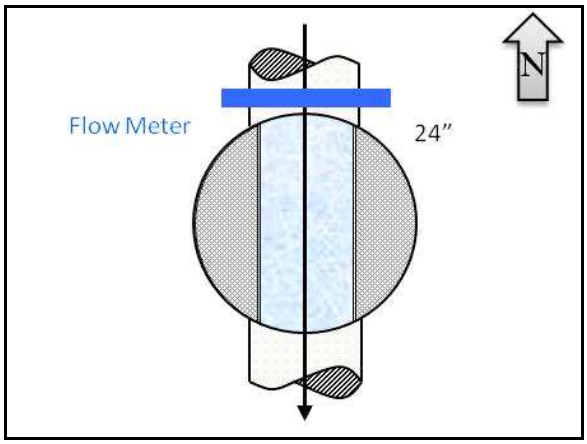
Peak Measured Flow: 2.162 *mgd*

Satellite Map

Sanitary Map



Flow Schematic

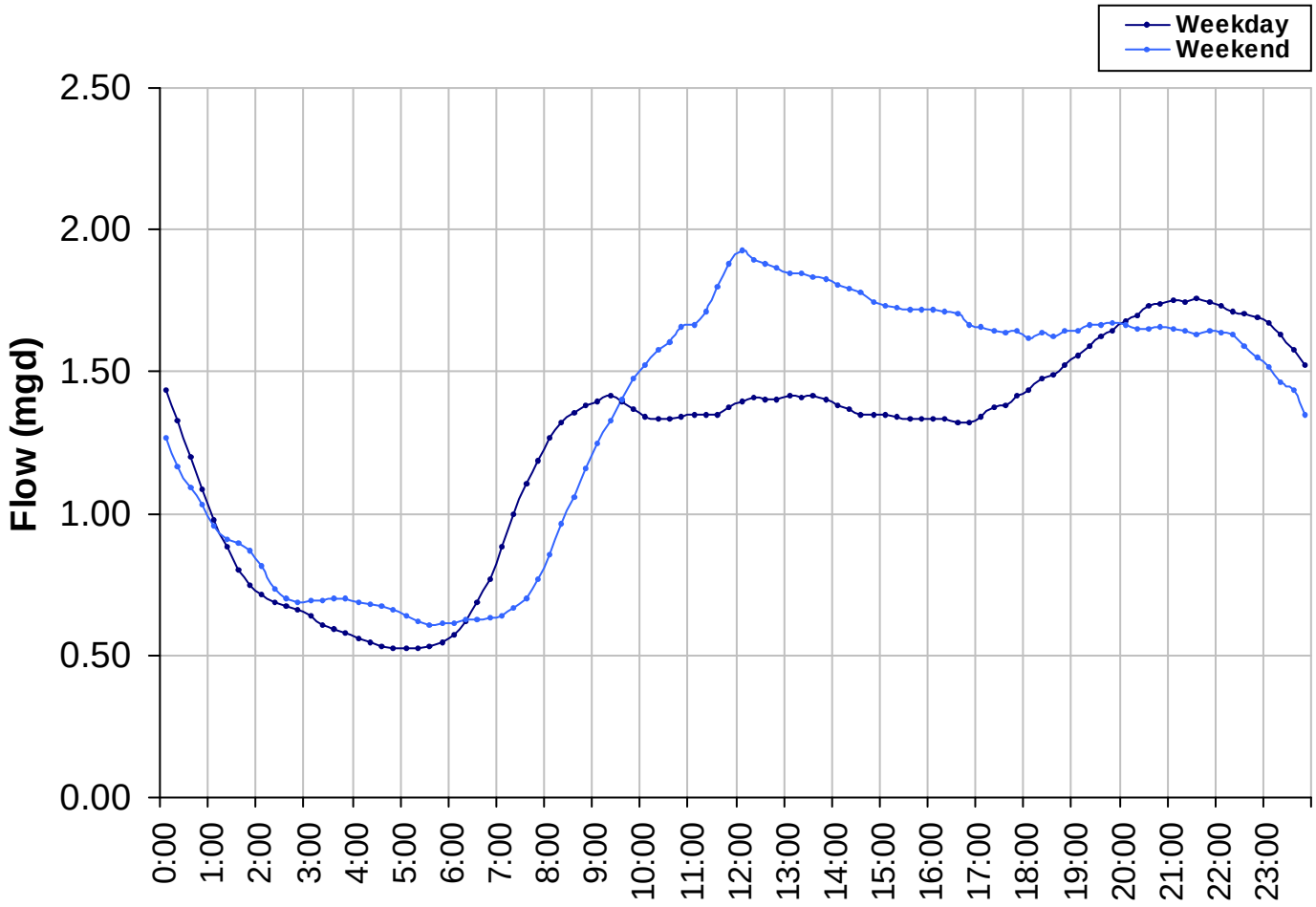


Street View Photo

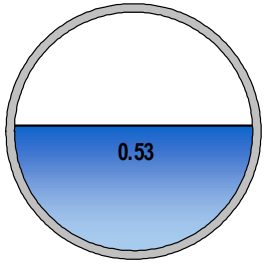
Plan View Photo

Average Dry Weather Flow

Monitoring Site:
Site 5



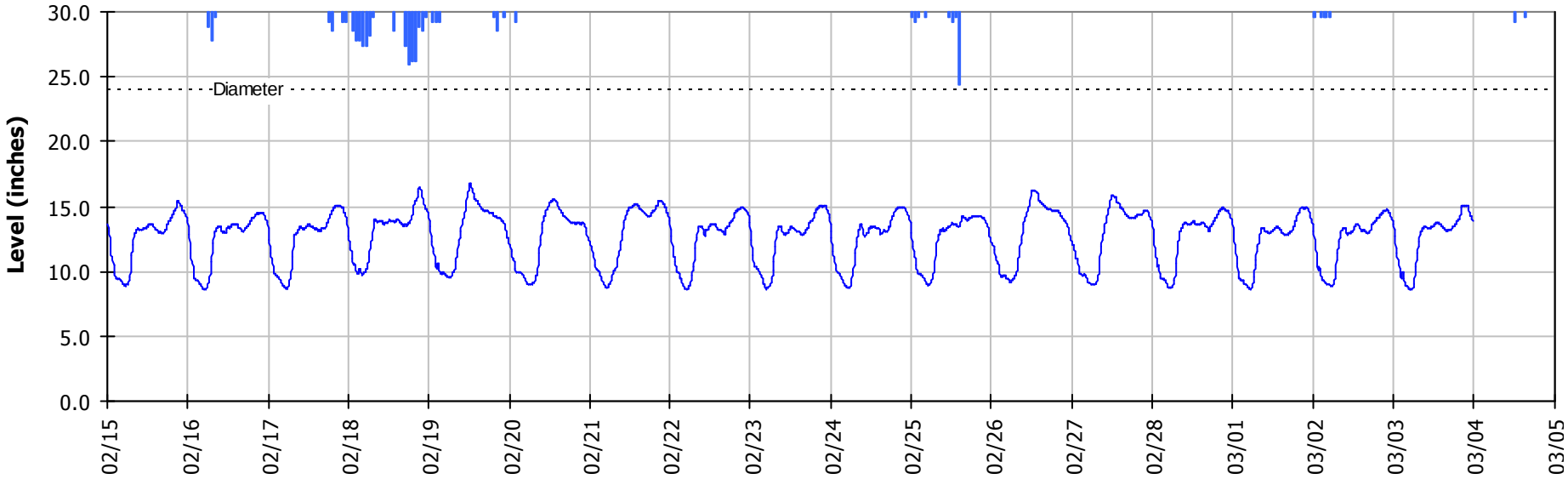
Average Dry Weather Flow:
1.264 *mgd*

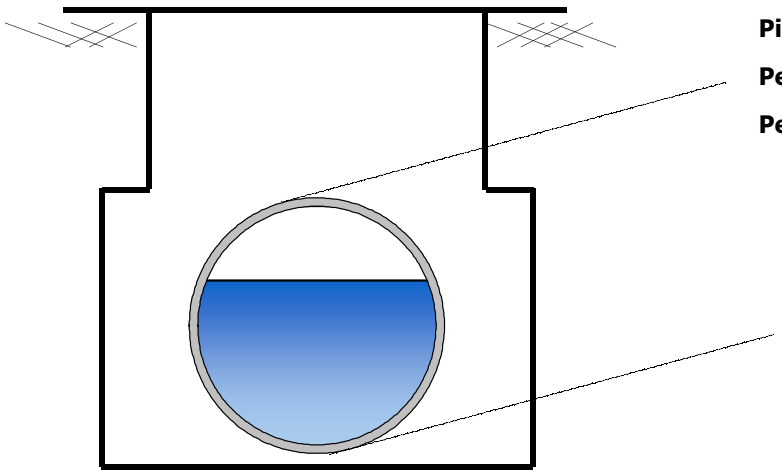


Site Capacity / Surcharge Summary

Monitoring Site:
Site 5

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 24 *inches*

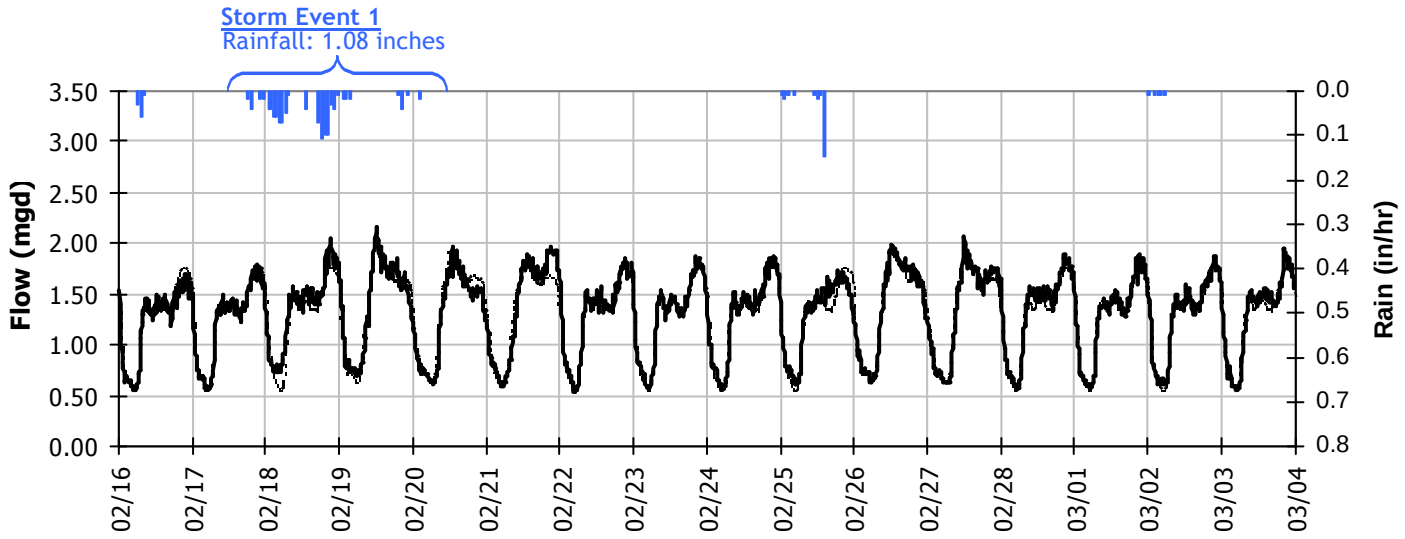
Peak Measured Level: 16.7 *inches*

Peak d/D Ratio: 0.70

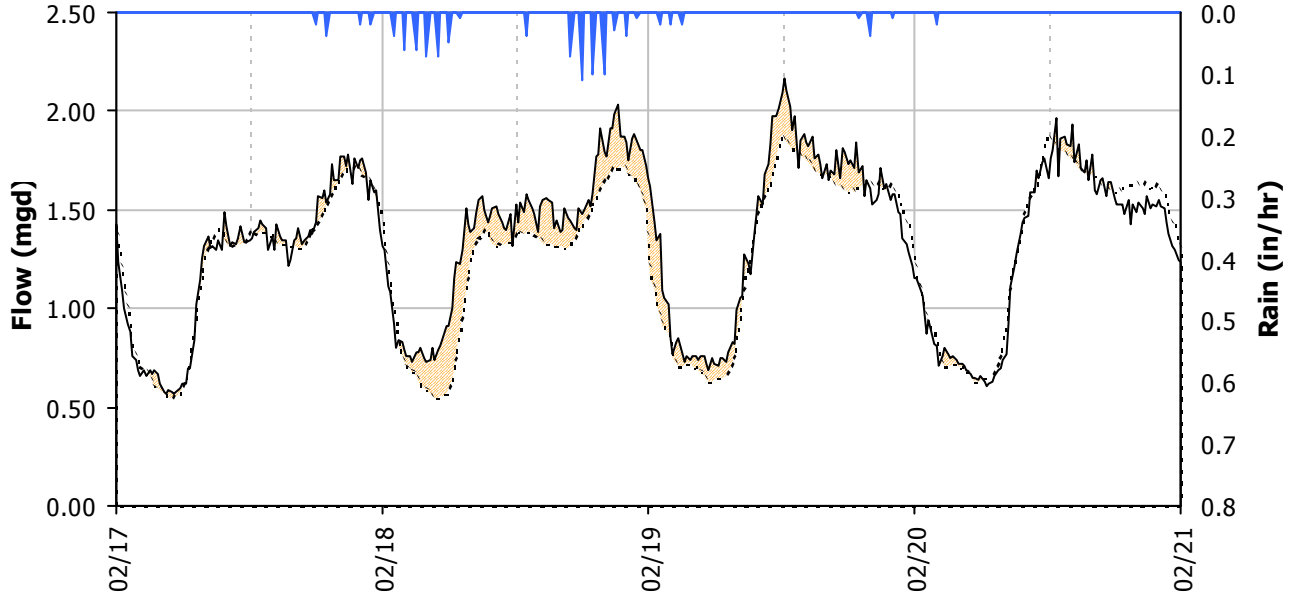
I/I Summary
Event 1

Monitoring Site:
Site 5

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

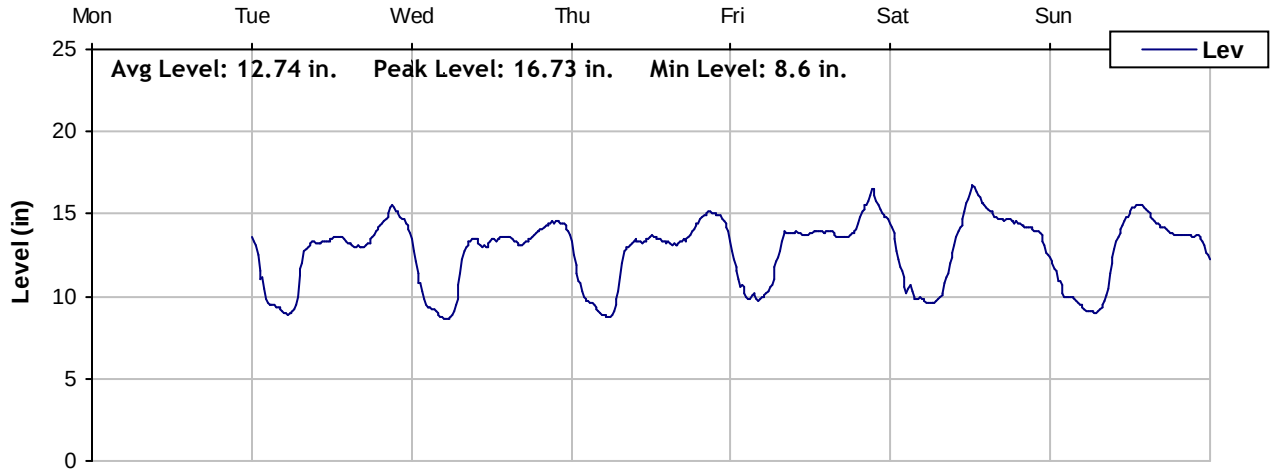


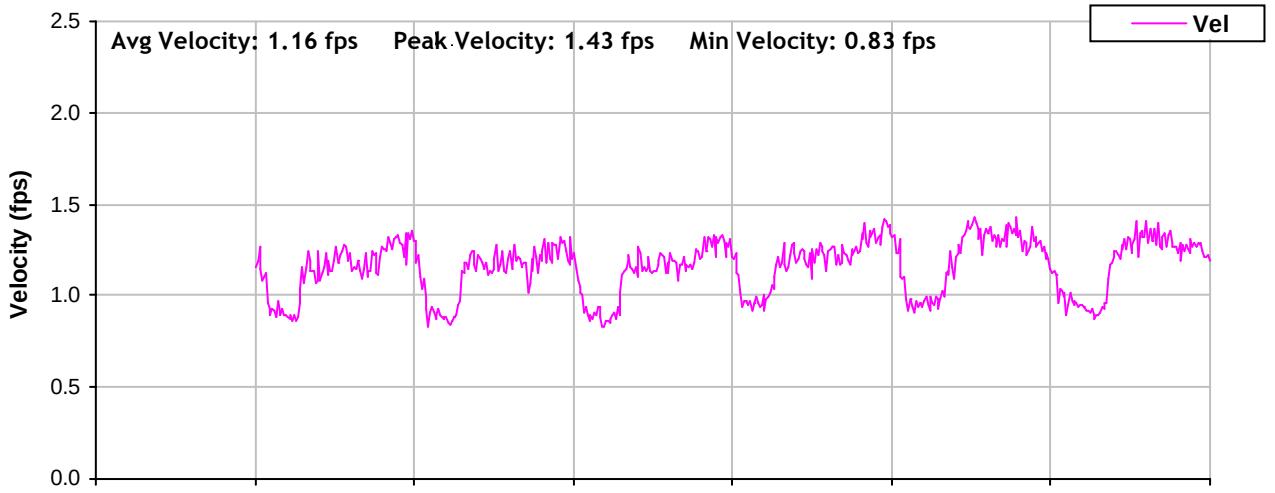
Storm Event #1 I/I Analysis

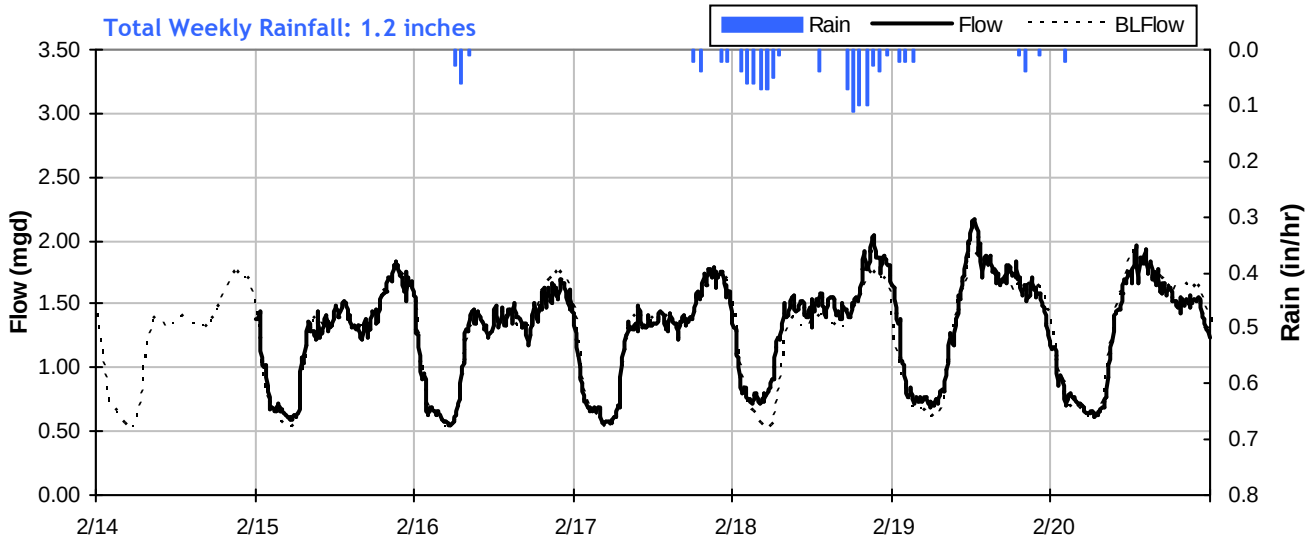
Rainfall:	1.08	*inches*			
Peak Flow:	2.16	*mgd*	**PF:**	1.71	
Peak I/I Rate:	0.47	*mgd*	**Pk I/I:ADWF:**	0.37	
Peak Level:	16.73	*inches*	**d/D Ratio:**	0.70	
Total I/I:	237,000	*gallons*	**I/I per ADWF:**	0.17	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.10	*mgd*	**RDI (% of BL):**	8%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 5



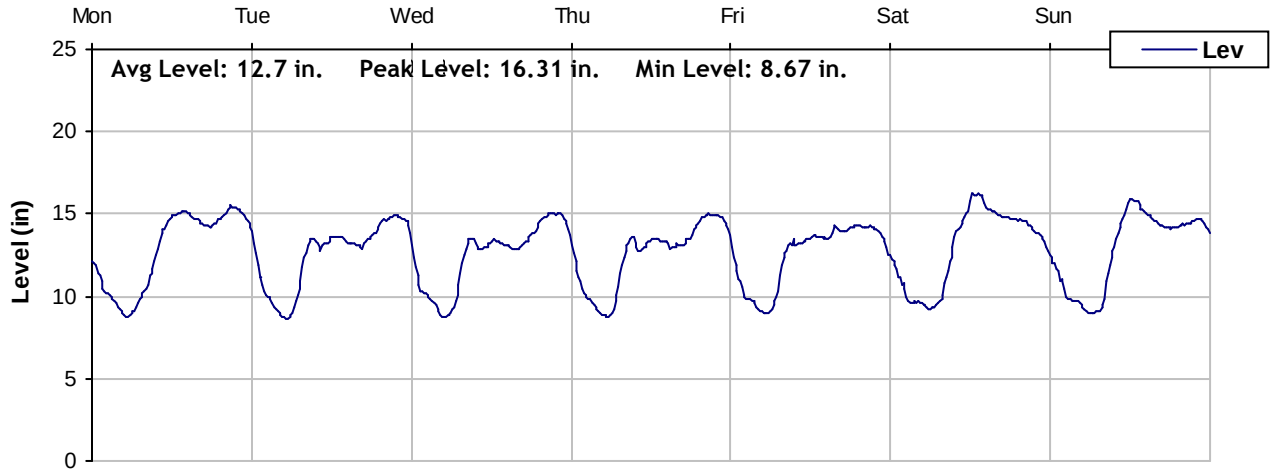


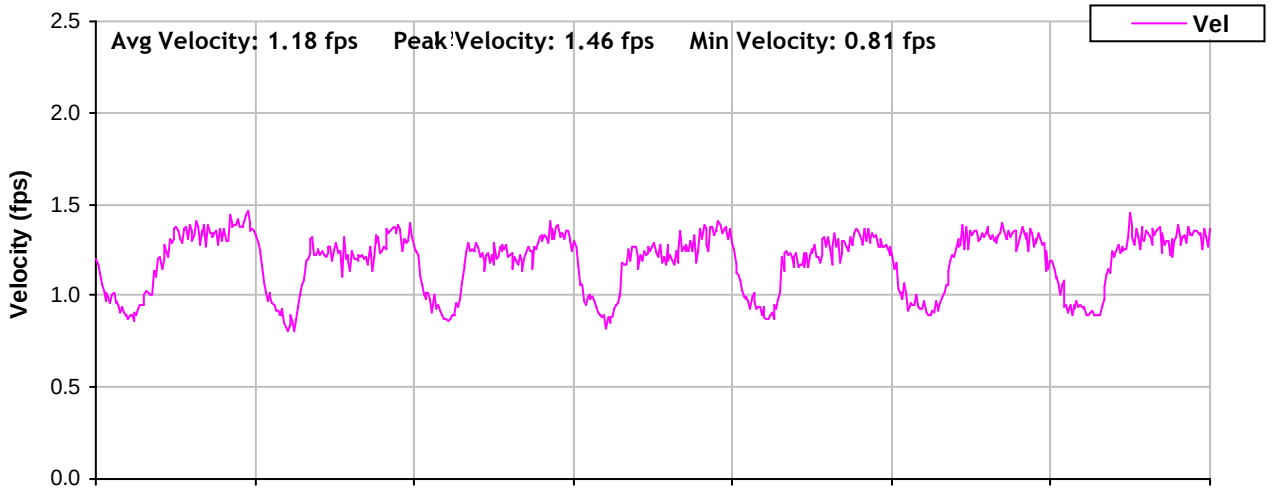


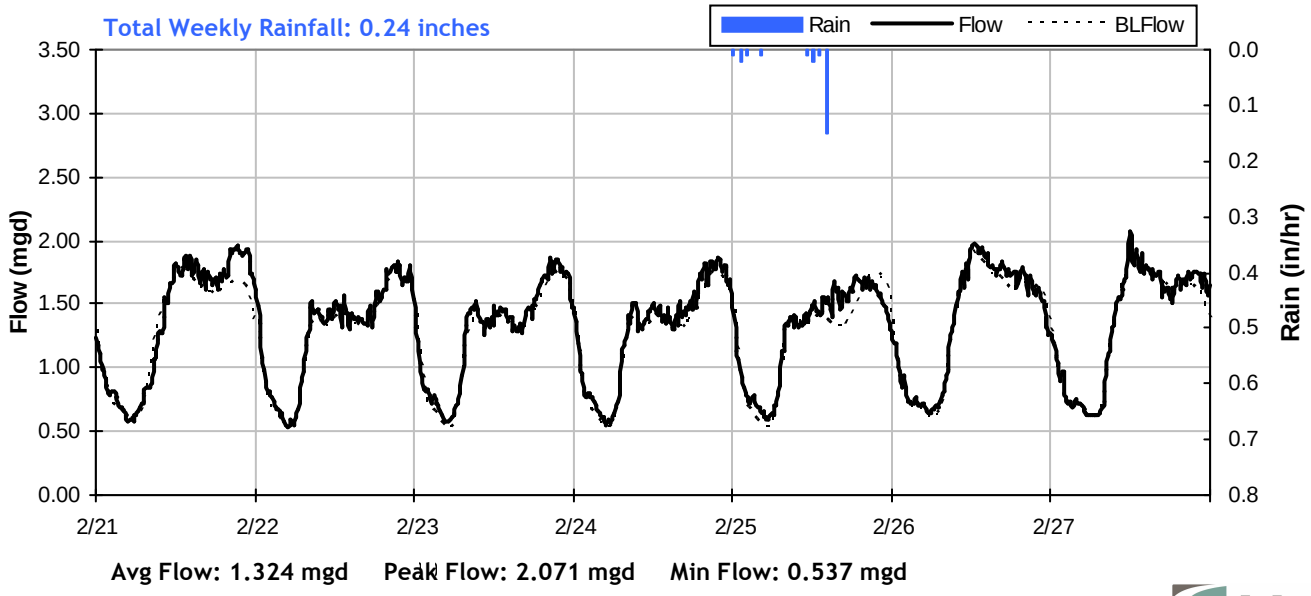
Avg Flow: 1.297 mgd Peak Flow: 2.162 mgd Min Flow: 0.547 mgd

Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 5

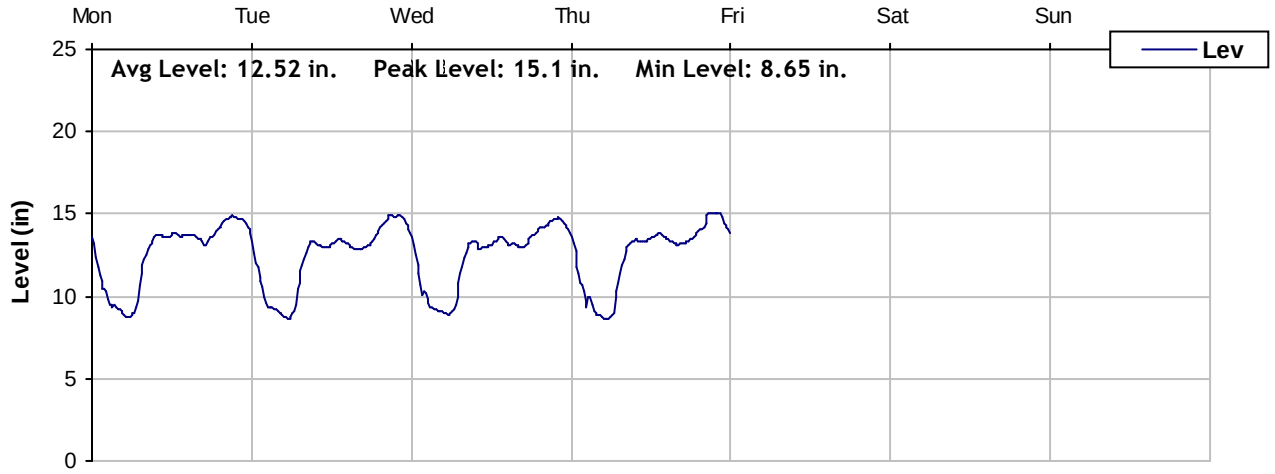


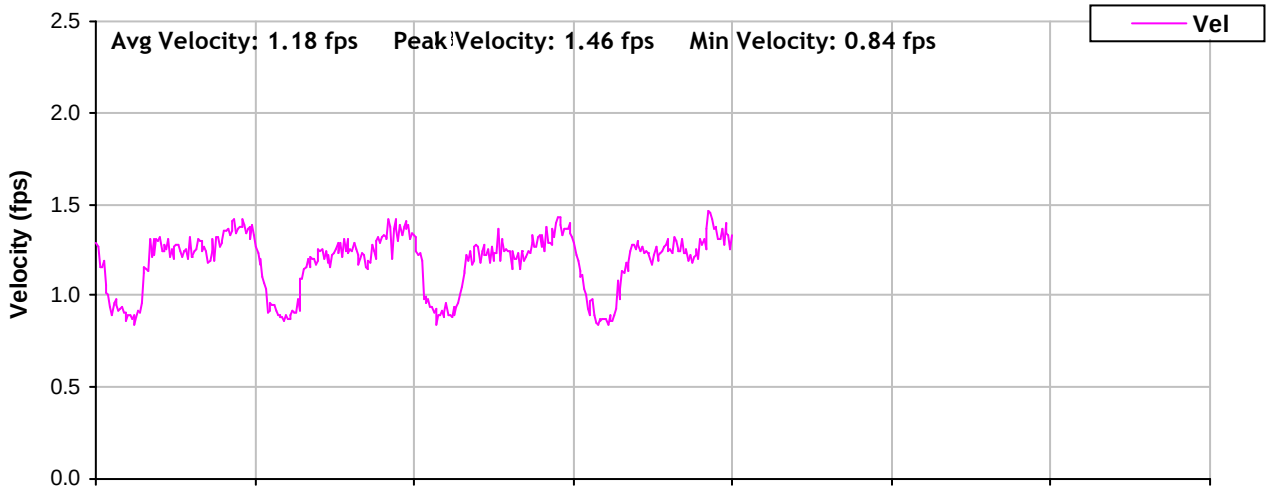


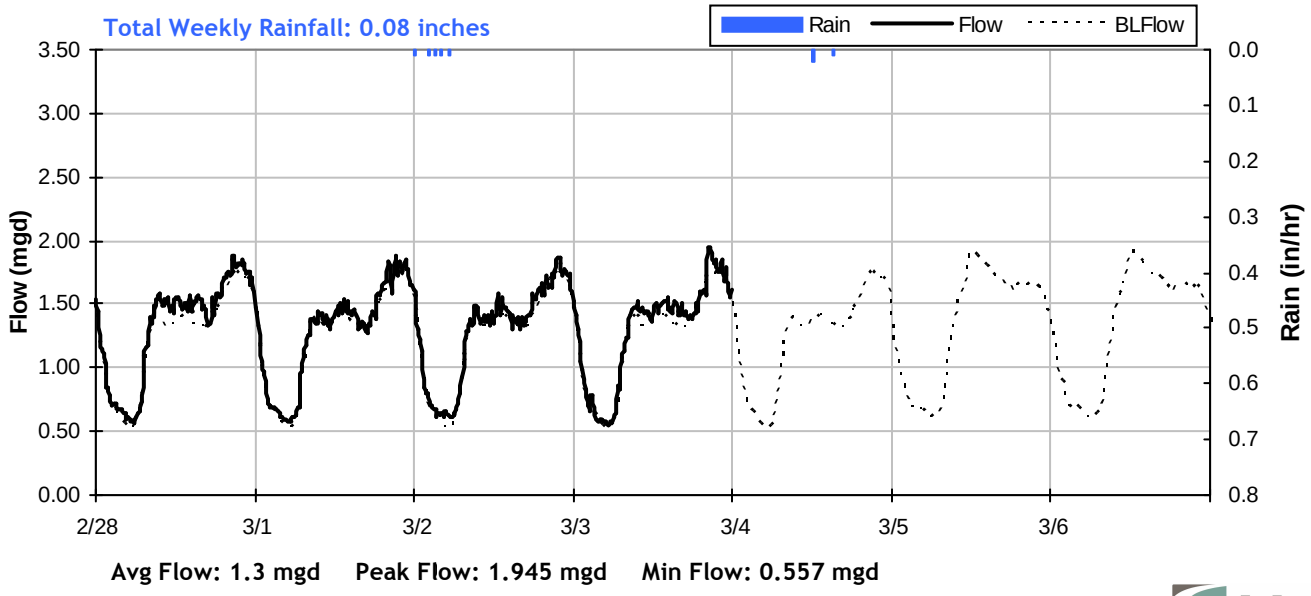


Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 5







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 5

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**12.74**	**1.16**	**1.30**
Weekly Minimum:	**8.64**	**0.85**	**0.56**
Weekly Maximum:	**16.60**	**1.40**	**2.10**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00				13.15	1.19	1.36	12.47	1.20	1.28	12.30	1.13	1.19	12.11	1.17	1.20	13.58	1.28	1.52	11.83	1.12	1.12	**0:00**
1:00				11.05	1.07	0.97	10.42	1.01	0.86	10.51	0.97	0.83	10.67	0.97	0.85	11.70	1.15	1.13	10.69	1.01	0.88	**1:00**
2:00				9.57	0.92	0.69	9.39	0.90	0.66	9.70	0.88	0.68	9.95	0.95	0.76	10.49	0.97	0.82	9.94	0.97	0.77	**2:00**
3:00				9.40	0.91	0.67	9.13	0.90	0.64	9.38	0.92	0.68	10.04	0.97	0.78	9.98	0.94	0.75	9.86	0.96	0.76	**3:00**
4:00				9.15	0.90	0.64	8.75	0.88	0.59	8.93	0.85	0.59	9.86	0.95	0.75	9.86	0.96	0.75	9.47	0.95	0.70	**4:00**
5:00				8.94	0.88	0.61	8.64	0.86	0.56	8.75	0.87	0.58	10.28	1.01	0.84	9.61	0.96	0.73	9.10	0.92	0.65	**5:00**
6:00				9.48	0.94	0.70	9.32	0.92	0.67	9.33	0.90	0.66	11.42	1.13	1.08	9.59	0.97	0.73	9.06	0.90	0.63	**6:00**
7:00				11.90	1.16	1.17	11.78	1.16	1.15	11.52	1.10	1.06	13.11	1.20	1.36	10.01	0.98	0.79	9.30	0.92	0.67	**7:00**
8:00				13.04	1.15	1.30	13.20	1.22	1.39	13.00	1.18	1.33	13.87	1.20	1.46	11.35	1.16	1.10	10.39	1.05	0.89	**8:00**
9:00				13.30	1.12	1.29	13.49	1.18	1.39	13.38	1.16	1.36	13.87	1.23	1.49	12.95	1.17	1.31	12.68	1.22	1.33	**9:00**
10:00				13.27	1.16	1.33	13.10	1.17	1.32	13.32	1.17	1.35	13.75	1.24	1.49	14.27	1.29	1.63	13.98	1.24	1.52	**10:00**
11:00				13.37	1.16	1.35	13.16	1.13	1.28	13.51	1.16	1.36	13.75	1.16	1.40	15.77	1.38	1.95	14.83	1.30	1.71	**11:00**
12:00				13.55	1.21	1.43	13.43	1.20	1.40	13.59	1.16	1.37	13.92	1.23	1.50	16.60	1.40	2.10	15.38	1.31	1.80	**12:00**
13:00				13.60	1.25	1.48	13.57	1.19	1.40	13.42	1.20	1.40	13.90	1.26	1.53	15.87	1.31	1.86	15.52	1.31	1.81	**13:00**
14:00				13.27	1.18	1.36	13.57	1.19	1.41	13.30	1.18	1.36	13.95	1.23	1.51	15.35	1.36	1.86	15.25	1.35	1.83	**14:00**
15:00				13.03	1.17	1.31	13.37	1.22	1.42	13.10	1.14	1.29	13.78	1.23	1.49	14.94	1.30	1.73	14.64	1.31	1.70	**15:00**
16:00				13.00	1.14	1.28	13.15	1.17	1.34	13.34	1.17	1.36	13.57	1.22	1.44	14.66	1.30	1.69	14.28	1.31	1.65	**16:00**
17:00				13.24	1.19	1.37	13.42	1.11	1.29	13.55	1.16	1.37	13.62	1.21	1.44	14.64	1.36	1.77	14.02	1.32	1.63	**17:00**
18:00				13.85	1.17	1.42	13.75	1.21	1.45	14.18	1.23	1.54	13.92	1.24	1.51	14.53	1.37	1.76	13.78	1.28	1.54	**18:00**
19:00				14.34	1.26	1.59	14.07	1.23	1.53	14.71	1.25	1.63	14.94	1.32	1.76	14.39	1.30	1.66	13.71	1.25	1.50	**19:00**
20:00				14.97	1.30	1.72	14.40	1.24	1.58	15.08	1.29	1.73	15.75	1.31	1.85	14.17	1.26	1.57	13.71	1.25	1.50	**20:00**
21:00				15.29	1.32	1.79	14.51	1.27	1.63	15.03	1.29	1.72	16.24	1.33	1.94	14.05	1.32	1.62	13.66	1.29	1.53	**21:00**
22:00				14.81	1.25	1.64	14.50	1.29	1.65	14.77	1.29	1.69	15.20	1.35	1.83	13.71	1.28	1.53	13.51	1.28	1.50	**22:00**
23:00				14.19	1.33	1.65	14.02	1.22	1.50	13.96	1.27	1.56	14.68	1.38	1.80	12.65	1.20	1.30	12.62	1.21	1.31	**23:00**
Average:				**12.61**	**1.14**	**1.26**	**12.44**	**1.13**	**1.22**	**12.57**	**1.12**	**1.24**	**13.17**	**1.19**	**1.38**	**13.11**	**1.21**	**1.40**	**12.55**	**1.17**	**1.29**	**Ave**
Minimum:				**8.94**	**0.88**	**0.61**	**8.64**	**0.86**	**0.56**	**8.75**	**0.85**	**0.58**	**9.86**	**0.95**	**0.75**	**9.59**	**0.94**	**0.73**	**9.06**	**0.90**	**0.63**	**Min**
Maximum:				**15.29**	**1.33**	**1.79**	**14.51**	**1.29**	**1.65**	**15.08**	**1.29**	**1.73**	**16.24**	**1.38**	**1.94**	**16.60**	**1.40**	**2.10**	**15.52**	**1.35**	**1.83**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 5

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**12.70**	**1.18**	**1.32**
Weekly Minimum:	**8.74**	**0.85**	**0.57**
Weekly Maximum:	**16.22**	**1.42**	**1.95**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	11.84	1.14	1.13	12.94	1.28	1.42	12.16	1.20	1.24	12.34	1.20	1.26	12.18	1.17	1.21	11.97	1.15	1.16	12.01	1.13	1.15	**0:00**
1:00	10.74	1.00	0.88	10.84	1.06	0.94	10.39	1.01	0.85	10.70	1.01	0.89	10.67	1.02	0.89	10.94	1.02	0.92	11.09	1.05	0.97	**1:00**
2:00	10.10	0.98	0.80	9.96	0.98	0.78	10.00	0.98	0.78	9.91	0.99	0.78	9.86	0.97	0.76	9.91	0.96	0.76	10.02	0.94	0.75	**2:00**
3:00	9.62	0.94	0.72	9.33	0.91	0.66	9.48	0.94	0.70	9.40	0.93	0.69	9.59	0.96	0.73	9.63	0.96	0.73	9.74	0.94	0.72	**3:00**
4:00	9.02	0.89	0.62	8.78	0.86	0.57	8.84	0.88	0.59	8.93	0.87	0.60	9.14	0.92	0.65	9.59	0.94	0.71	9.54	0.94	0.70	**4:00**
5:00	8.83	0.89	0.60	8.74	0.85	0.57	8.87	0.88	0.60	8.79	0.87	0.59	9.03	0.89	0.62	9.29	0.90	0.65	9.10	0.91	0.64	**5:00**
6:00	9.29	0.93	0.68	9.51	0.95	0.72	9.61	0.95	0.72	9.50	0.97	0.72	9.59	0.93	0.71	9.38	0.93	0.69	9.02	0.91	0.63	**6:00**
7:00	10.05	1.00	0.81	11.37	1.13	1.08	11.66	1.16	1.14	11.55	1.13	1.10	11.48	1.15	1.11	9.80	0.97	0.76	9.27	0.93	0.67	**7:00**
8:00	10.94	1.07	0.96	13.12	1.27	1.44	13.31	1.26	1.45	13.12	1.24	1.41	12.95	1.23	1.37	11.42	1.11	1.06	11.03	1.11	1.01	**8:00**
9:00	12.28	1.17	1.22	13.38	1.24	1.44	13.32	1.26	1.45	13.36	1.23	1.42	13.22	1.19	1.36	13.22	1.25	1.43	12.88	1.23	1.37	**9:00**
10:00	13.85	1.23	1.50	13.02	1.23	1.38	12.88	1.19	1.33	12.84	1.22	1.35	13.26	1.18	1.36	14.33	1.30	1.64	14.25	1.26	1.58	**10:00**
11:00	14.59	1.32	1.70	13.33	1.25	1.44	13.11	1.20	1.36	13.21	1.26	1.44	13.47	1.22	1.43	15.20	1.32	1.79	15.41	1.32	1.83	**11:00**
12:00	14.95	1.33	1.77	13.59	1.25	1.48	13.41	1.24	1.45	13.44	1.24	1.44	13.65	1.23	1.47	16.22	1.34	1.95	15.81	1.34	1.90	**12:00**
13:00	15.16	1.36	1.84	13.57	1.21	1.43	13.22	1.27	1.45	13.39	1.21	1.41	13.57	1.26	1.49	16.13	1.32	1.91	15.48	1.32	1.82	**13:00**
14:00	15.03	1.34	1.80	13.29	1.20	1.38	13.08	1.20	1.36	13.18	1.22	1.39	13.53	1.25	1.48	15.40	1.34	1.85	14.98	1.33	1.77	**14:00**
15:00	14.73	1.36	1.77	13.18	1.21	1.38	12.83	1.20	1.32	13.04	1.21	1.37	14.13	1.30	1.61	15.15	1.31	1.77	14.68	1.33	1.73	**15:00**
16:00	14.48	1.35	1.72	13.04	1.20	1.35	13.05	1.19	1.34	13.09	1.28	1.45	14.01	1.25	1.54	14.90	1.36	1.79	14.33	1.33	1.68	**16:00**
17:00	14.27	1.34	1.68	13.47	1.20	1.41	13.55	1.23	1.45	13.41	1.27	1.48	14.05	1.28	1.58	14.77	1.32	1.74	14.20	1.28	1.60	**17:00**
18:00	14.46	1.33	1.69	13.87	1.29	1.57	13.97	1.29	1.58	14.07	1.24	1.53	14.25	1.34	1.68	14.69	1.32	1.72	14.15	1.29	1.60	**18:00**
19:00	14.76	1.32	1.72	14.44	1.29	1.64	14.61	1.32	1.71	14.68	1.36	1.76	14.25	1.32	1.66	14.63	1.33	1.72	14.29	1.33	1.67	**19:00**
20:00	15.30	1.40	1.92	14.70	1.36	1.77	14.95	1.35	1.79	14.96	1.32	1.76	14.24	1.32	1.65	14.44	1.31	1.67	14.42	1.33	1.70	**20:00**
21:00	15.42	1.39	1.92	14.86	1.37	1.80	15.02	1.35	1.81	14.94	1.36	1.81	14.14	1.32	1.64	14.06	1.30	1.61	14.56	1.34	1.73	**21:00**
22:00	15.07	1.42	1.91	14.76	1.29	1.69	14.93	1.34	1.77	14.78	1.37	1.80	13.69	1.28	1.53	13.69	1.31	1.56	14.66	1.32	1.71	**22:00**
23:00	14.46	1.37	1.75	14.32	1.33	1.68	14.21	1.30	1.63	14.14	1.33	1.66	12.84	1.26	1.39	12.89	1.18	1.31	14.12	1.33	1.65	**23:00**
Average:	**12.88**	**1.20**	**1.38**	**12.56**	**1.17**	**1.29**	**12.52**	**1.17**	**1.29**	**12.53**	**1.18**	**1.30**	**12.53**	**1.17**	**1.29**	**12.98**	**1.19**	**1.37**	**12.88**	**1.19**	**1.36**	**Ave**
Minimum:	**8.83**	**0.89**	**0.60**	**8.74**	**0.85**	**0.57**	**8.84**	**0.88**	**0.59**	**8.79**	**0.87**	**0.59**	**9.03**	**0.89**	**0.62**	**9.29**	**0.90**	**0.65**	**9.02**	**0.91**	**0.63**	**Min**
Maximum:	**15.42**	**1.42**	**1.92**	**14.86**	**1.37**	**1.80**	**15.02**	**1.35**	**1.81**	**14.96**	**1.37**	**1.81**	**14.25**	**1.34**	**1.68**	**16.22**	**1.36**	**1.95**	**15.81**	**1.34**	**1.90**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

Monitoring Site:

Site 5

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**12.52**	**1.18**	**1.30**
Weekly Minimum:	**8.67**	**0.86**	**0.57**
Weekly Maximum:	**15.07**	**1.42**	**1.88**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	12.72	1.22	1.34	12.45	1.22	1.29	12.76	1.25	1.38	12.86	1.21	1.34										**0:00**
1:00	10.77	1.07	0.94	10.67	1.03	0.90	10.62	1.03	0.90	10.83	1.06	0.95										**1:00**
2:00	9.75	0.95	0.73	9.41	0.95	0.70	9.73	0.94	0.73	9.73	0.95	0.73										**2:00**
3:00	9.35	0.94	0.68	9.21	0.90	0.65	9.23	0.89	0.64	9.10	0.86	0.61										**3:00**
4:00	9.00	0.90	0.62	8.87	0.88	0.60	9.05	0.92	0.64	8.77	0.87	0.58										**4:00**
5:00	8.76	0.87	0.59	8.70	0.90	0.60	8.93	0.91	0.62	8.67	0.86	0.57										**5:00**
6:00	9.26	0.92	0.66	9.51	0.98	0.74	9.43	0.95	0.70	9.54	0.97	0.73										**6:00**
7:00	11.54	1.14	1.10	11.63	1.15	1.13	11.61	1.13	1.10	11.61	1.12	1.09										**7:00**
8:00	12.94	1.26	1.41	13.04	1.19	1.35	13.00	1.20	1.34	13.02	1.21	1.36										**8:00**
9:00	13.58	1.30	1.53	13.25	1.22	1.40	13.34	1.25	1.45	13.41	1.28	1.49										**9:00**
10:00	13.67	1.28	1.52	13.04	1.22	1.38	12.91	1.24	1.38	13.38	1.25	1.45										**10:00**
11:00	13.65	1.24	1.47	13.03	1.21	1.36	13.05	1.23	1.38	13.42	1.21	1.41										**11:00**
12:00	13.82	1.26	1.52	13.30	1.27	1.46	13.34	1.25	1.45	13.65	1.23	1.46										**12:00**
13:00	13.65	1.24	1.48	13.41	1.27	1.48	13.58	1.28	1.51	13.73	1.26	1.51										**13:00**
14:00	13.69	1.25	1.49	13.17	1.26	1.43	13.22	1.22	1.40	13.48	1.26	1.48										**14:00**
15:00	13.65	1.29	1.54	12.87	1.21	1.35	13.13	1.20	1.36	13.24	1.28	1.47										**15:00**
16:00	13.35	1.24	1.43	12.87	1.19	1.32	12.99	1.22	1.37	13.23	1.25	1.44										**16:00**
17:00	13.42	1.23	1.44	13.11	1.23	1.39	13.38	1.27	1.48	13.34	1.21	1.40										**17:00**
18:00	13.85	1.30	1.58	13.58	1.28	1.51	13.82	1.30	1.57	13.63	1.22	1.45										**18:00**
19:00	14.35	1.34	1.70	14.24	1.32	1.66	14.15	1.31	1.63	14.07	1.29	1.60										**19:00**
20:00	14.75	1.38	1.80	14.77	1.34	1.76	14.42	1.31	1.66	14.87	1.42	1.88										**20:00**
21:00	14.84	1.37	1.81	14.85	1.37	1.81	14.68	1.40	1.81	15.07	1.36	1.82										**21:00**
22:00	14.66	1.38	1.79	14.80	1.37	1.80	14.63	1.37	1.77	14.86	1.34	1.77										**22:00**
23:00	14.12	1.35	1.68	14.19	1.34	1.68	14.12	1.36	1.68	14.13	1.32	1.63										**23:00**
Average:	**12.63**	**1.20**	**1.33**	**12.41**	**1.18**	**1.28**	**12.46**	**1.18**	**1.29**	**12.57**	**1.18**	**1.30**										**Ave**
Minimum:	**8.76**	**0.87**	**0.59**	**8.70**	**0.88**	**0.60**	**8.93**	**0.89**	**0.62**	**8.67**	**0.86**	**0.57**										**Min**
Maximum:	**14.84**	**1.38**	**1.81**	**14.85**	**1.37**	**1.81**	**14.68**	**1.40**	**1.81**	**15.07**	**1.42**	**1.88**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Temporary Flow Monitoring Study

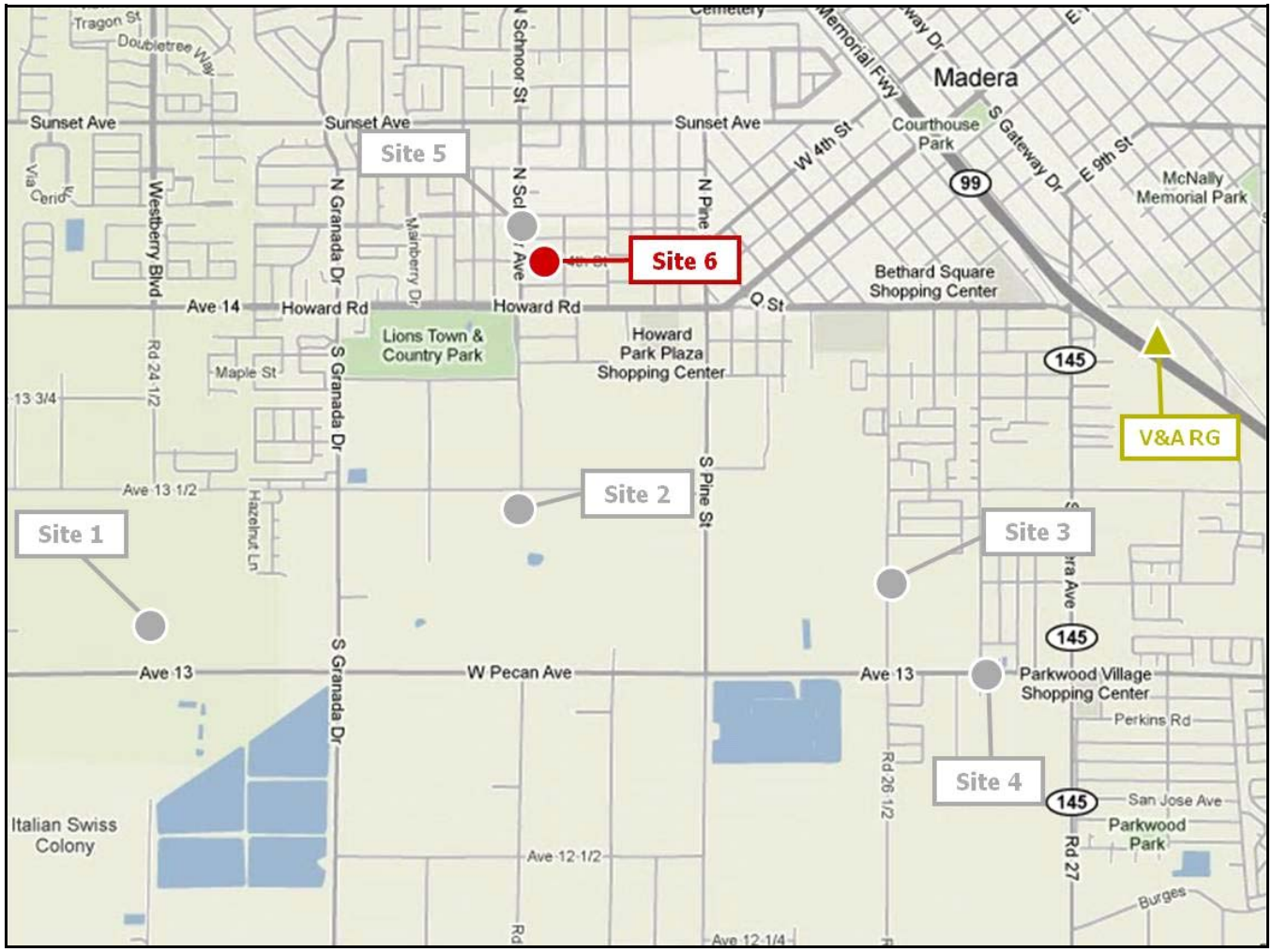
Sanitary Sewer Collection System

Monitoring Site: Site 6

Location: MH ID 4332: West Fourth Street, east of Schnoor Avenue

Size / Type Line: 21-inch Sanitary Sewer Pipe

Data Summary Report



Site Information Report

Monitoring Site: Site 6

Location: MH ID 4332: West Fourth Street, east of Schnoor Avenue

Latitude: 36.954596°

Longitude: -120.082036°

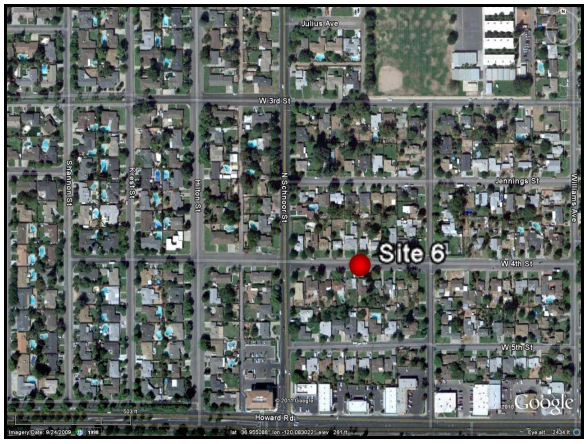
Rim Elevation: 261 *feet above sea level*

Diameter: 21 *inches*

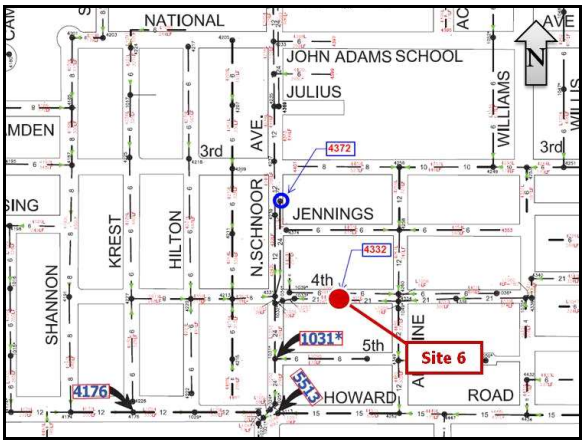
Average Dry Weather Flow: 1.320 *mgd*

Peak Measured Flow: 2.100 *mgd*

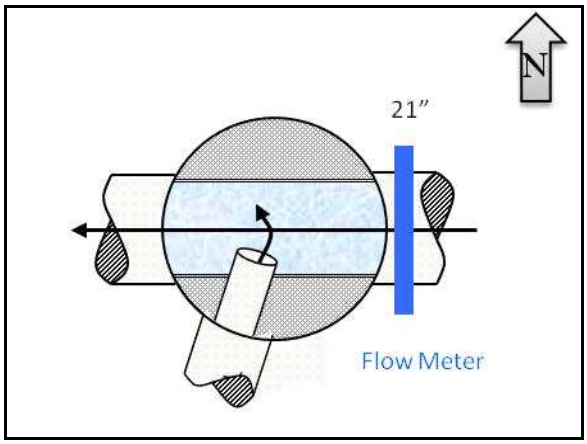
Satellite Map



Sanitary Map

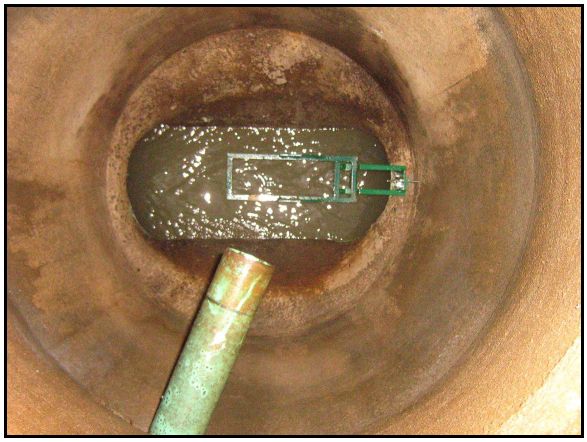


Flow Schematic



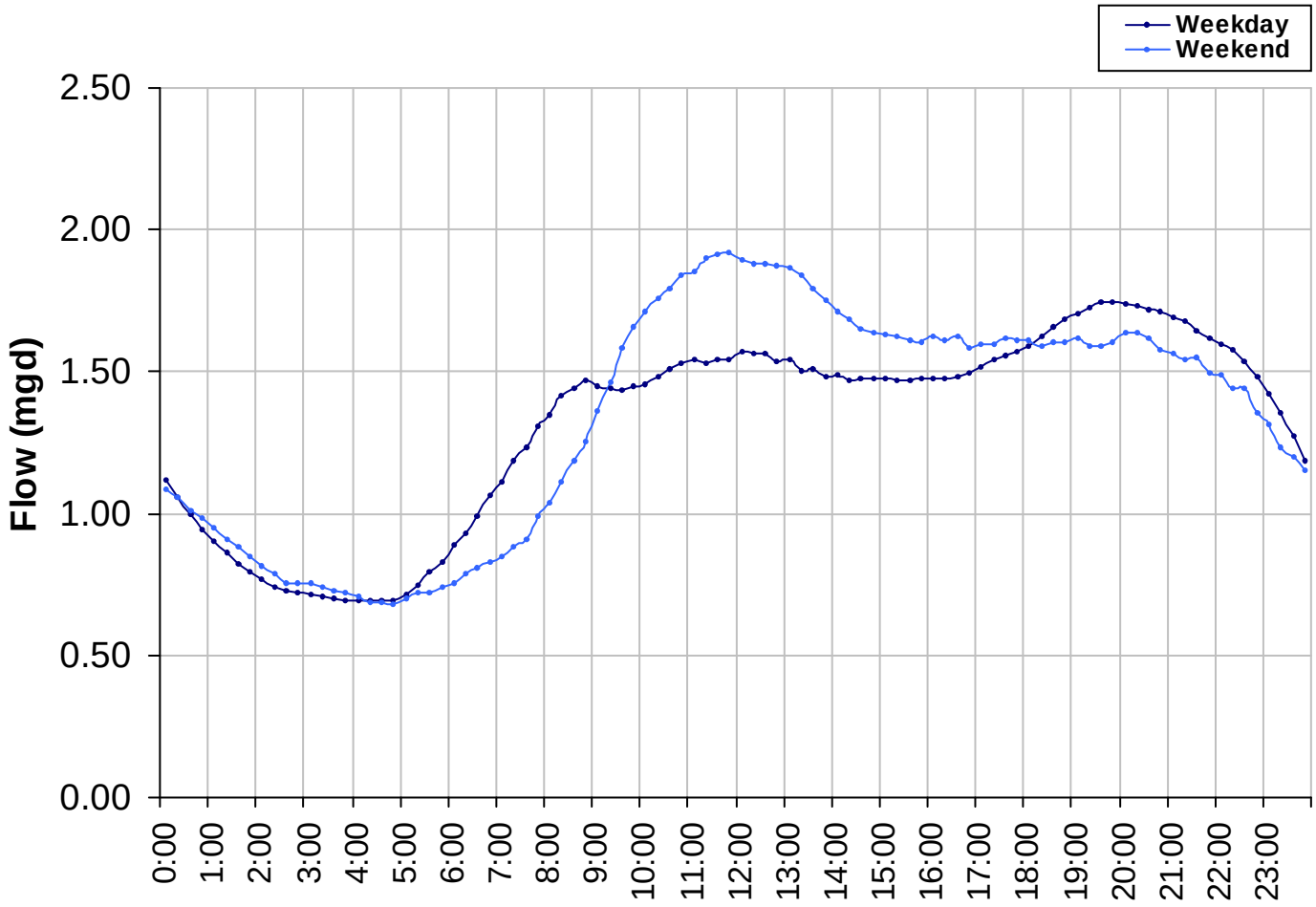
Street View Photo

Plan View Photo

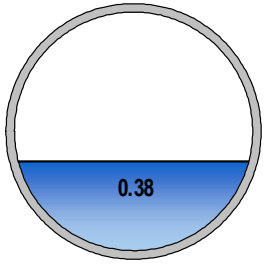


Average Dry Weather Flow

Monitoring Site:
Site 6



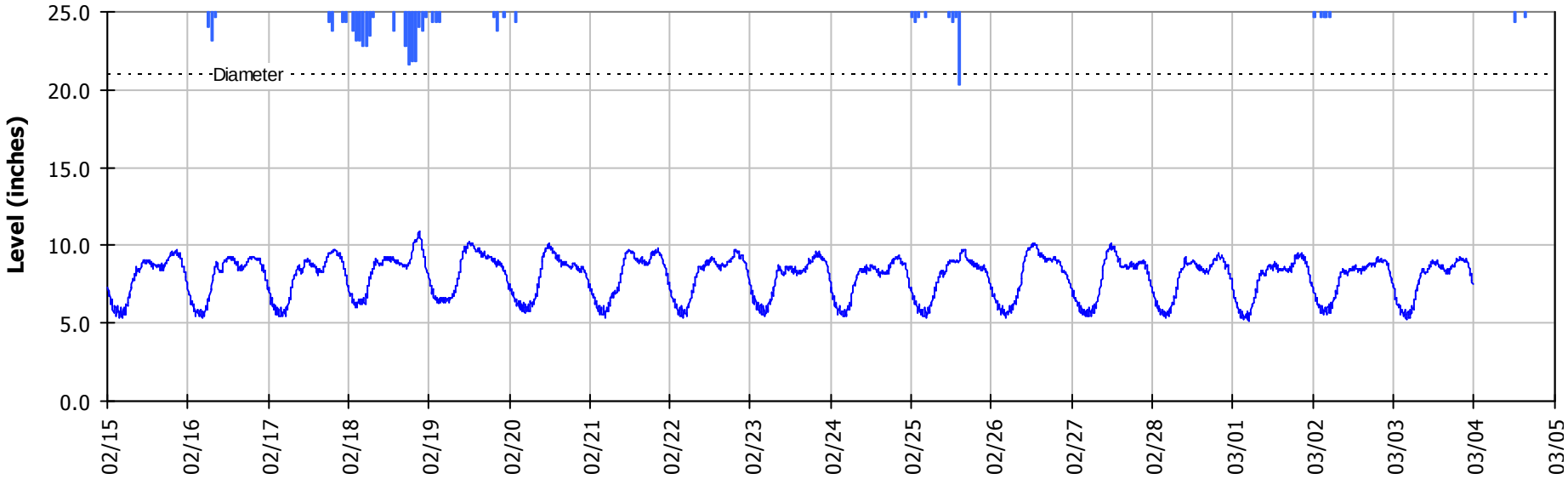
Average Dry Weather Flow:
1.320 *mgd*

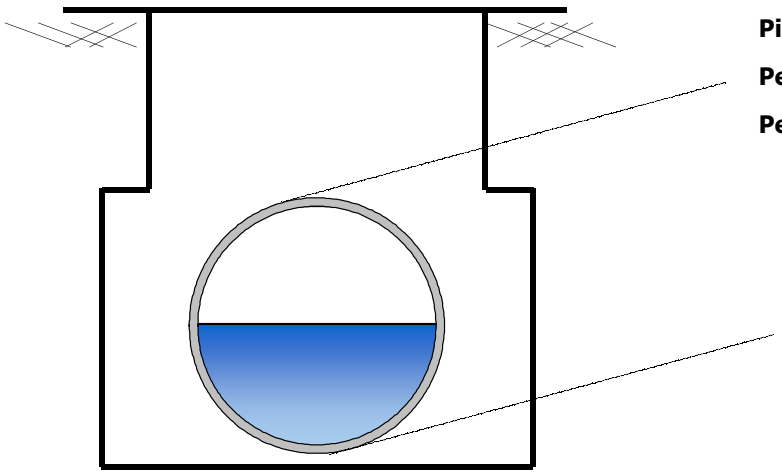


Site Capacity / Surcharge Summary

Monitoring Site:
Site 6

Baseline and Realtime Flow Levels with Rainfall Data over Monitoring Period





Pipe Diameter: 21 *inches*

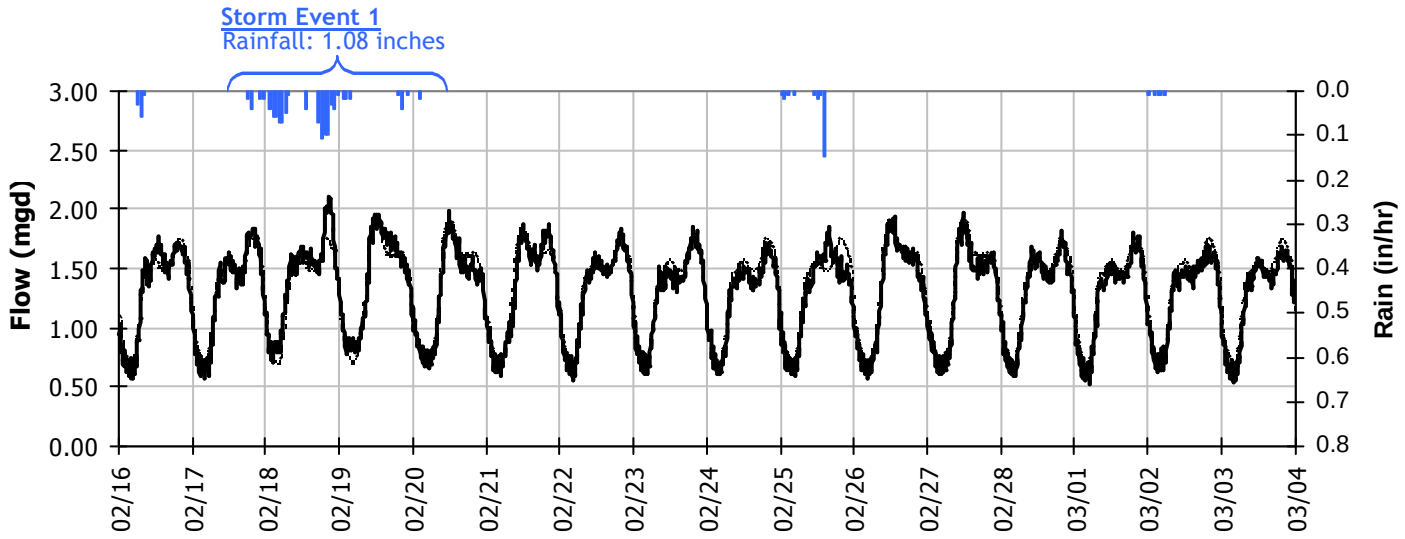
Peak Measured Level: 10.9 *inches*

Peak d/D Ratio: 0.52

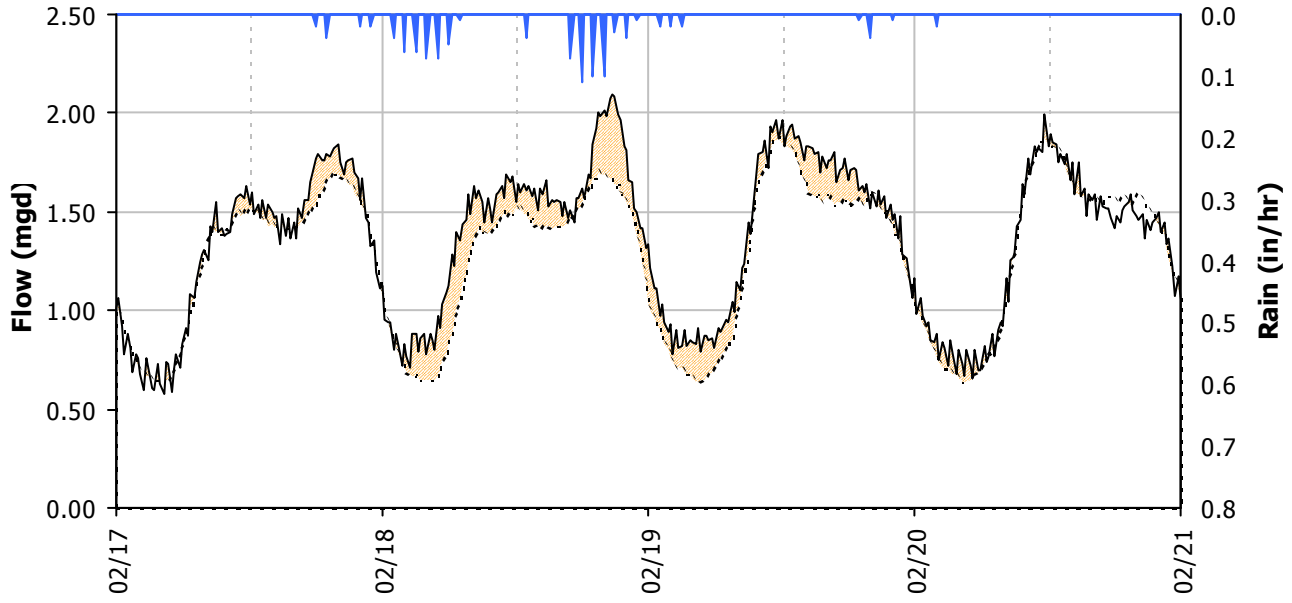
I/I Summary
Event 1

Monitoring Site:
Site 6

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Storm Event #1 Detail I/I Graph

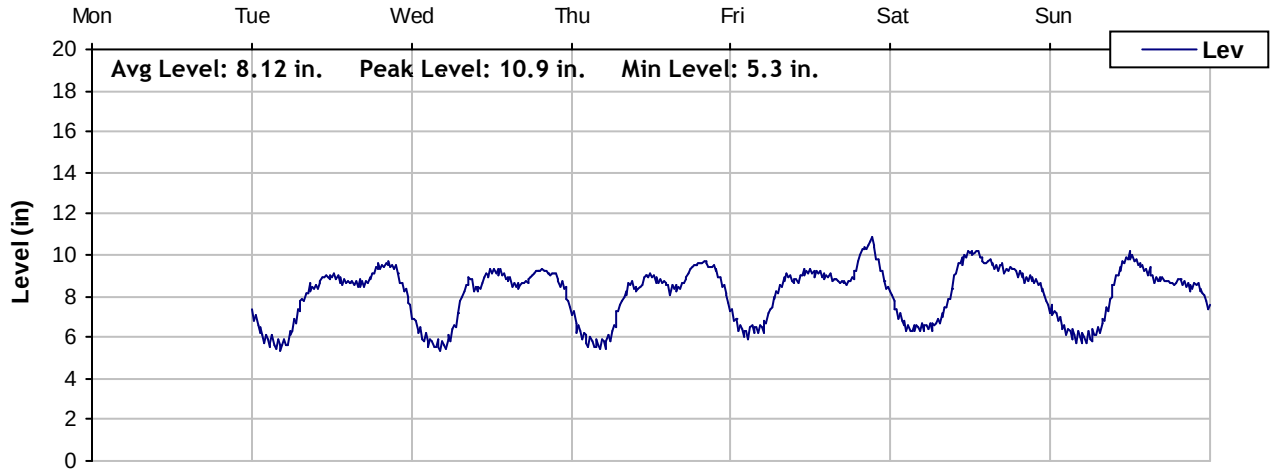


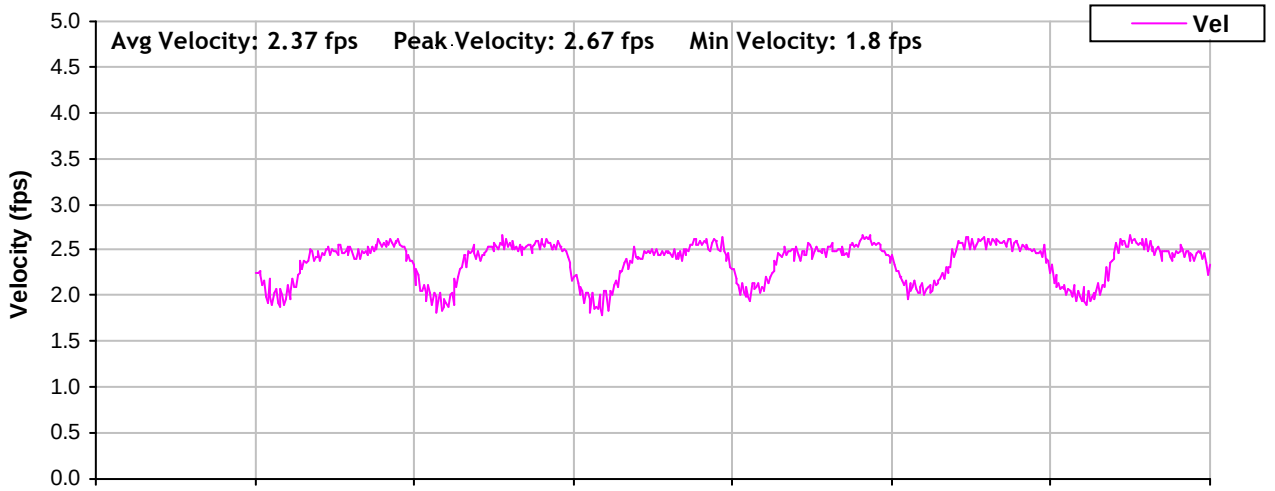
Storm Event #1 I/I Analysis

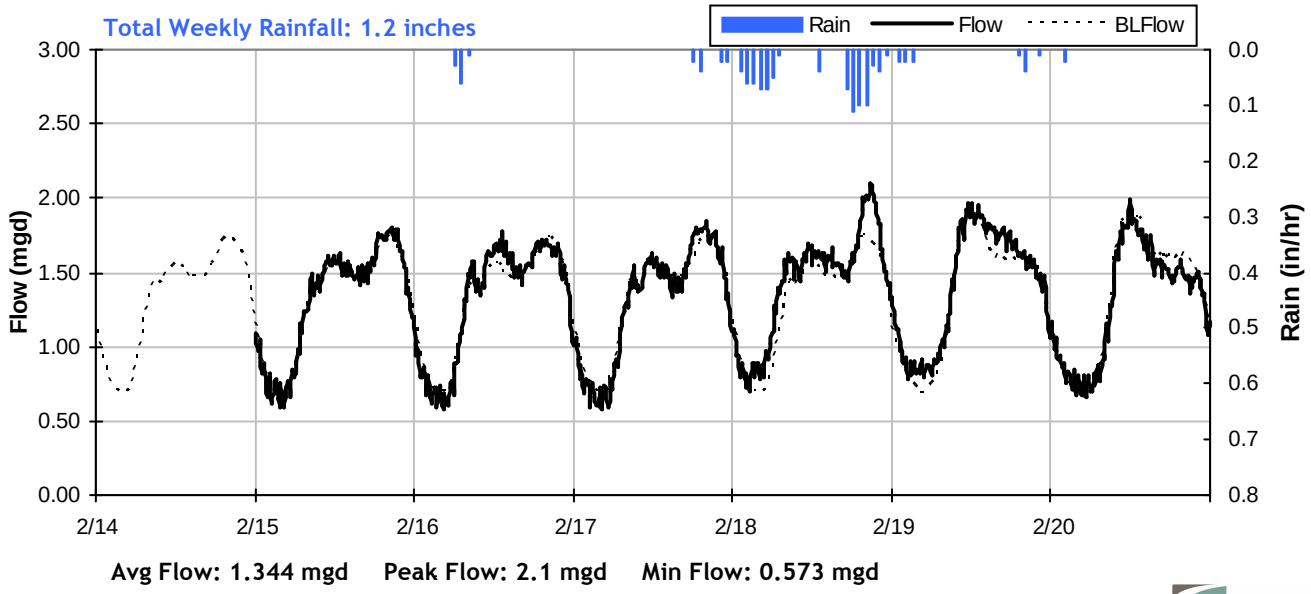
Rainfall:	1.08	*inches*			
Peak Flow:	2.10	*mgd*	**PF:**	1.59	
Peak I/I Rate:	0.40	*mgd*	**Pk I/I:ADWF:**	0.30	
Peak Level:	10.90	*inches*	**d/D Ratio:**	0.52	
Total I/I:	303,000	*gallons*	**I/I per ADWF:**	0.21	*per in-rain*
Infiltration Rate: *(at 2/19 8:00)*	0.07	*mgd*	**RDI (% of BL):**	6%	

Level, Velocity and Flow
From 2/14/2011 to 2/21/2011

Monitoring Site:
Site 6

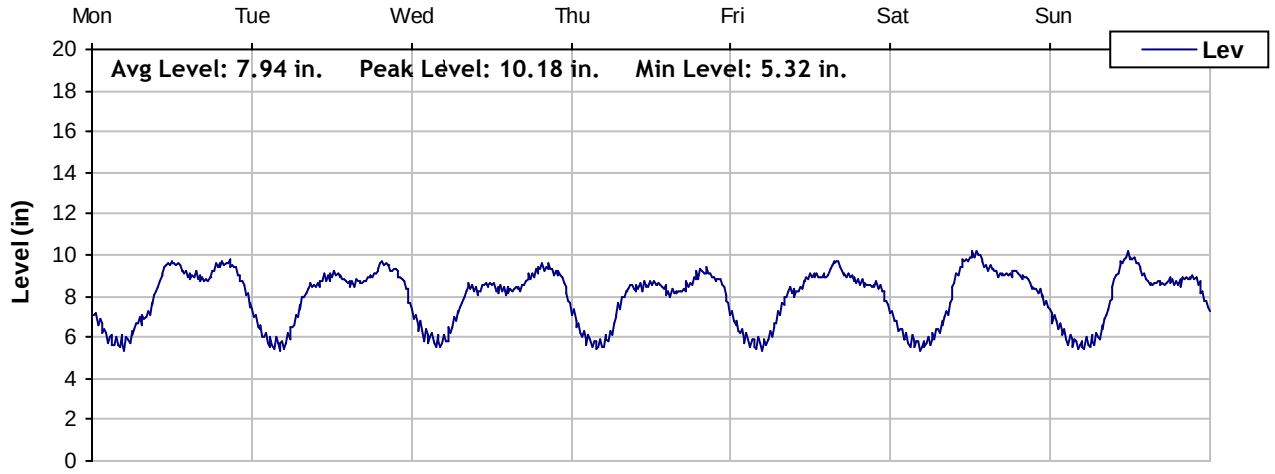


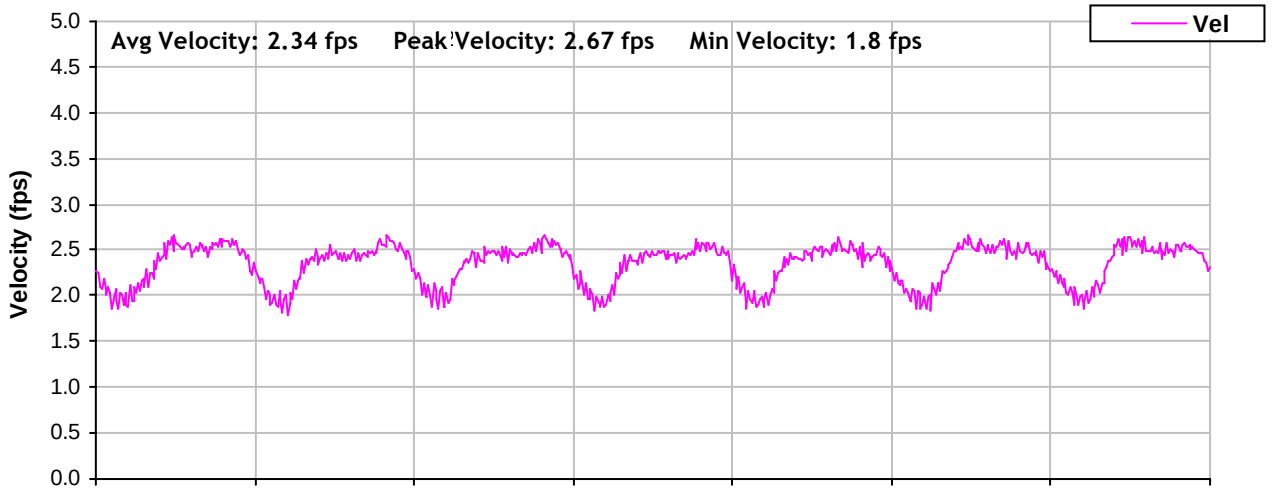


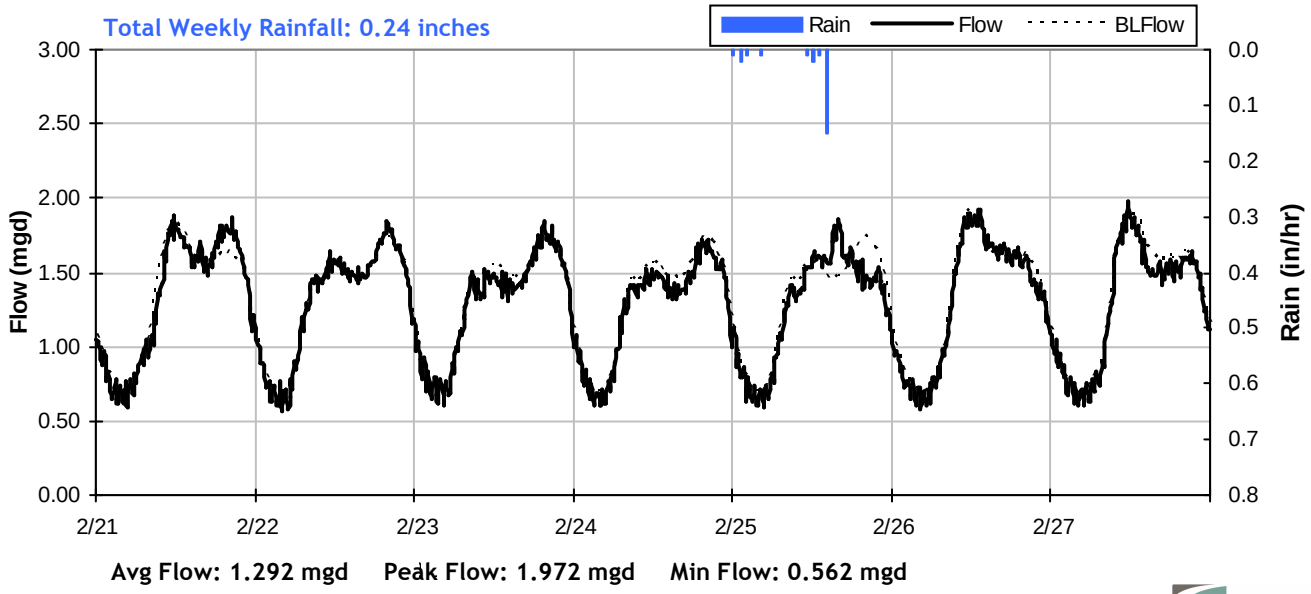


Level, Velocity and Flow
From 2/21/2011 to 2/28/2011

Monitoring Site:
Site 6

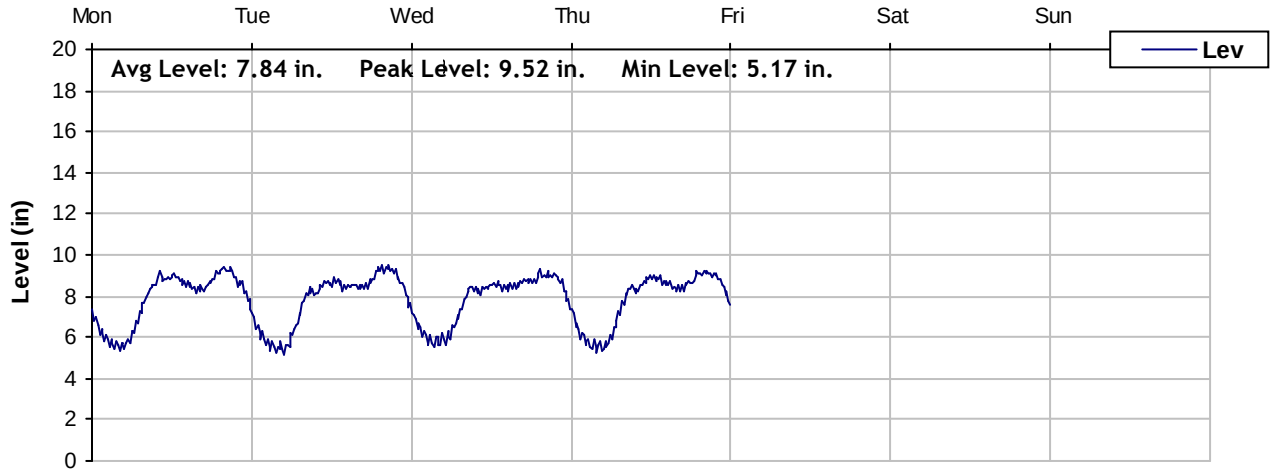


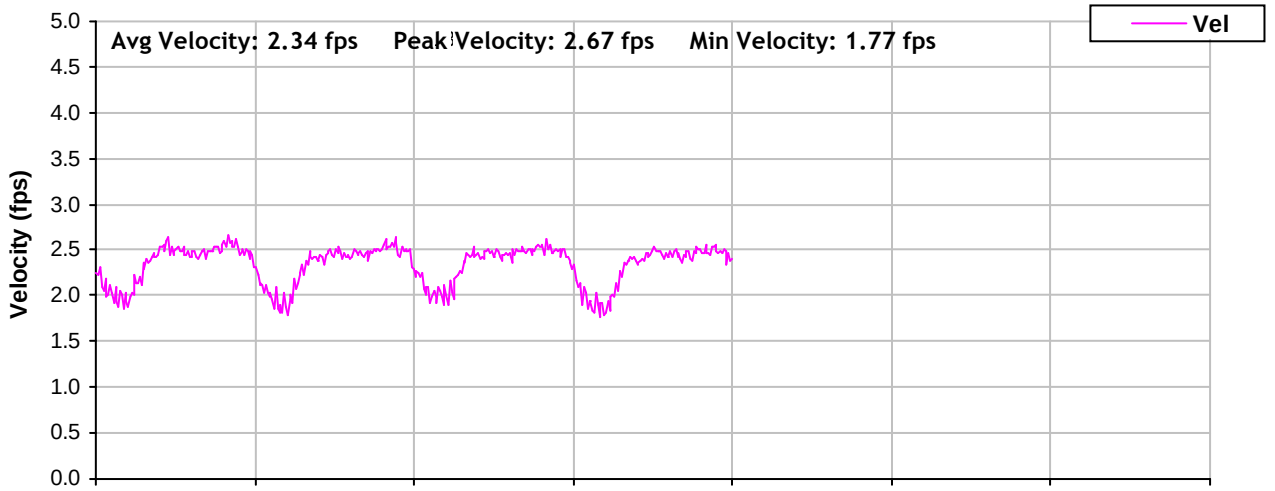


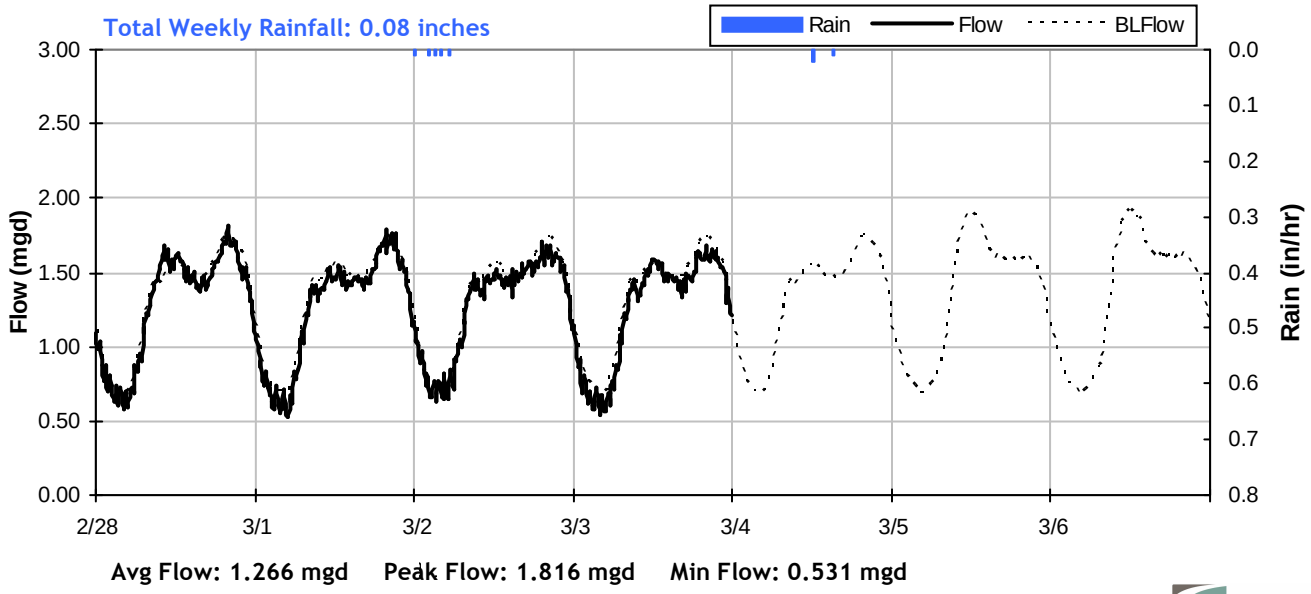


Level, Velocity and Flow
From 2/28/2011 to 3/7/2011

Monitoring Site:
Site 6







Hourly Data: Depth, Velocity and Flow

From 2/14/2011 to 2/21/2011

Monitoring Site:

Site 6

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**8.12**	**2.37**	**1.34**
Weekly Minimum:	**5.58**	**1.90**	**0.63**
Weekly Maximum:	**10.46**	**2.64**	**2.05**

	Monday 2/14/2011			Tuesday 2/15/2011			Wednesday 2/16/2011			Thursday 2/17/2011			Friday 2/18/2011			Saturday 2/19/2011			Sunday 2/20/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00				6.94	2.25	1.01	6.72	2.18	0.93	6.82	2.14	0.94	6.98	2.18	0.99	7.64	2.34	1.19	7.25	2.20	1.05	**0:00**
1:00				6.14	2.05	0.78	6.11	2.03	0.77	6.25	2.03	0.79	6.34	2.04	0.81	6.86	2.17	0.96	6.66	2.10	0.89	**1:00**
2:00				5.84	2.03	0.72	5.84	2.03	0.72	5.85	1.97	0.70	6.28	2.04	0.80	6.51	2.09	0.86	6.37	2.06	0.82	**2:00**
3:00				5.76	1.99	0.69	5.71	1.95	0.67	5.71	1.90	0.65	6.45	2.11	0.85	6.46	2.09	0.85	6.06	2.03	0.75	**3:00**
4:00				5.64	2.00	0.67	5.58	1.90	0.63	5.70	1.95	0.67	6.40	2.07	0.83	6.46	2.07	0.84	6.05	2.02	0.75	**4:00**
5:00				5.90	2.08	0.75	5.85	1.95	0.69	5.82	1.98	0.69	7.01	2.20	1.00	6.57	2.08	0.87	5.98	1.97	0.72	**5:00**
6:00				6.80	2.24	0.98	6.67	2.20	0.93	6.69	2.16	0.92	7.87	2.36	1.26	6.57	2.12	0.88	6.10	2.01	0.76	**6:00**
7:00				7.67	2.35	1.20	7.80	2.38	1.25	7.64	2.32	1.19	8.51	2.47	1.46	6.94	2.20	0.98	6.57	2.09	0.87	**7:00**
8:00				8.29	2.47	1.41	8.73	2.50	1.53	8.36	2.33	1.35	8.95	2.49	1.57	7.65	2.30	1.17	7.37	2.24	1.09	**8:00**
9:00				8.43	2.44	1.42	8.46	2.44	1.43	8.45	2.45	1.43	8.78	2.47	1.52	8.71	2.51	1.53	8.51	2.46	1.45	**9:00**
10:00				8.80	2.48	1.53	8.58	2.46	1.47	8.58	2.44	1.46	9.01	2.45	1.57	9.69	2.56	1.79	9.37	2.56	1.72	**10:00**
11:00				8.98	2.50	1.58	9.10	2.53	1.63	9.05	2.49	1.59	9.17	2.53	1.65	10.03	2.61	1.92	9.88	2.58	1.85	**11:00**
12:00				8.97	2.51	1.59	9.23	2.55	1.68	8.87	2.47	1.54	9.13	2.48	1.61	10.12	2.57	1.90	9.83	2.60	1.85	**12:00**
13:00				8.74	2.49	1.52	9.15	2.59	1.68	8.72	2.47	1.51	9.09	2.50	1.61	9.91	2.62	1.89	9.54	2.58	1.77	**13:00**
14:00				8.68	2.47	1.50	8.84	2.55	1.58	8.46	2.46	1.44	8.96	2.52	1.60	9.69	2.58	1.81	9.25	2.56	1.69	**14:00**
15:00				8.62	2.45	1.47	8.55	2.53	1.50	8.42	2.45	1.43	8.83	2.50	1.55	9.48	2.60	1.77	8.91	2.49	1.56	**15:00**
16:00				8.60	2.48	1.48	8.55	2.51	1.49	8.50	2.46	1.45	8.69	2.49	1.51	9.47	2.59	1.76	8.87	2.48	1.55	**16:00**
17:00				8.80	2.51	1.55	8.79	2.53	1.56	9.05	2.52	1.61	8.70	2.50	1.52	9.28	2.58	1.71	8.70	2.48	1.51	**17:00**
18:00				9.27	2.57	1.70	9.16	2.57	1.67	9.52	2.59	1.77	9.08	2.55	1.64	9.28	2.57	1.70	8.61	2.44	1.46	**18:00**
19:00				9.47	2.56	1.74	9.25	2.59	1.71	9.65	2.58	1.80	10.07	2.63	1.94	9.05	2.53	1.62	8.78	2.51	1.54	**19:00**
20:00				9.55	2.57	1.77	9.13	2.55	1.65	9.54	2.56	1.76	10.46	2.64	2.05	8.84	2.52	1.57	8.52	2.45	1.45	**20:00**
21:00				9.37	2.59	1.74	9.05	2.55	1.63	9.35	2.58	1.73	10.39	2.57	1.97	8.78	2.49	1.53	8.50	2.45	1.44	**21:00**
22:00				8.68	2.49	1.51	8.57	2.50	1.49	8.73	2.51	1.54	9.26	2.51	1.66	8.44	2.48	1.45	8.36	2.46	1.42	**22:00**
23:00				7.88	2.38	1.27	7.88	2.32	1.24	7.81	2.37	1.25	8.44	2.42	1.41	7.66	2.35	1.20	7.69	2.34	1.21	**23:00**
Average:				**7.99**	**2.37**	**1.32**	**7.97**	**2.37**	**1.31**	**7.98**	**2.34**	**1.30**	**8.45**	**2.40**	**1.43**	**8.34**	**2.40**	**1.41**	**7.99**	**2.34**	**1.30**	**Ave**
Minimum:				**5.64**	**1.99**	**0.67**	**5.58**	**1.90**	**0.63**	**5.70**	**1.90**	**0.65**	**6.28**	**2.04**	**0.80**	**6.46**	**2.07**	**0.84**	**5.98**	**1.97**	**0.72**	**Min**
Maximum:				**9.55**	**2.59**	**1.77**	**9.25**	**2.59**	**1.71**	**9.65**	**2.59**	**1.80**	**10.46**	**2.64**	**2.05**	**10.12**	**2.62**	**1.92**	**9.88**	**2.60**	**1.85**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/21/2011 to 2/28/2011

Monitoring Site:

Site 6

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**7.94**	**2.34**	**1.29**
Weekly Minimum:	**5.60**	**1.90**	**0.63**
Weekly Maximum:	**10.04**	**2.61**	**1.88**

	Monday 2/21/2011			Tuesday 2/22/2011			Wednesday 2/23/2011			Thursday 2/24/2011			Friday 2/25/2011			Saturday 2/26/2011			Sunday 2/27/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	6.99	2.17	0.98	6.98	2.20	1.00	6.99	2.20	1.00	6.94	2.16	0.97	6.86	2.20	0.97	6.93	2.21	0.99	6.97	2.25	1.01	**0:00**
1:00	6.53	2.09	0.86	6.25	2.08	0.81	6.30	2.03	0.80	6.34	2.14	0.85	6.21	2.04	0.79	6.27	2.12	0.83	6.38	2.15	0.86	**1:00**
2:00	6.02	1.99	0.73	5.86	1.96	0.70	6.06	2.04	0.76	5.88	2.00	0.71	5.86	1.95	0.69	5.85	2.00	0.71	6.04	2.07	0.77	**2:00**
3:00	5.73	1.96	0.67	5.72	1.94	0.67	5.91	1.99	0.71	5.64	1.92	0.65	5.70	1.96	0.67	5.81	2.01	0.71	5.79	2.01	0.70	**3:00**
4:00	5.69	1.92	0.66	5.60	1.90	0.63	5.80	1.97	0.69	5.62	1.91	0.64	5.68	1.95	0.66	5.62	1.94	0.65	5.61	1.94	0.65	**4:00**
5:00	6.00	2.02	0.74	6.10	2.03	0.76	6.08	2.05	0.77	6.06	2.02	0.75	5.96	2.00	0.73	5.76	1.92	0.67	5.65	1.97	0.67	**5:00**
6:00	6.48	2.09	0.86	6.82	2.16	0.95	6.90	2.23	0.99	6.74	2.16	0.94	6.50	2.18	0.89	6.13	2.09	0.79	5.93	2.08	0.75	**6:00**
7:00	6.88	2.16	0.96	7.70	2.34	1.21	7.56	2.36	1.19	7.74	2.32	1.21	7.43	2.32	1.14	6.51	2.18	0.90	6.24	2.10	0.82	**7:00**
8:00	7.25	2.26	1.08	8.41	2.40	1.40	8.44	2.43	1.42	8.33	2.40	1.38	8.16	2.40	1.34	7.26	2.32	1.11	7.28	2.33	1.12	**8:00**
9:00	8.10	2.39	1.32	8.55	2.42	1.44	8.35	2.41	1.39	8.43	2.37	1.38	8.20	2.41	1.36	8.50	2.49	1.47	8.56	2.49	1.48	**9:00**
10:00	9.10	2.51	1.62	8.74	2.44	1.49	8.42	2.43	1.42	8.48	2.44	1.44	8.43	2.42	1.41	9.42	2.55	1.72	9.49	2.56	1.74	**10:00**
11:00	9.61	2.59	1.80	8.91	2.47	1.55	8.56	2.48	1.47	8.52	2.45	1.45	8.90	2.45	1.53	9.79	2.60	1.85	9.91	2.61	1.88	**11:00**
12:00	9.60	2.55	1.76	9.09	2.44	1.57	8.47	2.47	1.45	8.62	2.47	1.48	9.02	2.50	1.60	10.04	2.53	1.86	9.79	2.56	1.82	**12:00**
13:00	9.27	2.55	1.68	8.90	2.42	1.52	8.37	2.47	1.42	8.47	2.46	1.44	8.98	2.50	1.58	9.89	2.55	1.84	9.33	2.55	1.70	**13:00**
14:00	9.02	2.48	1.58	8.66	2.46	1.48	8.26	2.44	1.39	8.22	2.45	1.38	9.07	2.50	1.61	9.41	2.50	1.69	8.83	2.53	1.57	**14:00**
15:00	9.03	2.54	1.62	8.68	2.42	1.47	8.34	2.45	1.40	8.25	2.43	1.38	9.64	2.55	1.77	9.28	2.52	1.67	8.61	2.50	1.50	**15:00**
16:00	8.85	2.50	1.56	8.70	2.44	1.48	8.45	2.46	1.44	8.28	2.43	1.38	9.35	2.52	1.69	9.05	2.57	1.65	8.64	2.49	1.50	**16:00**
17:00	8.84	2.55	1.58	8.92	2.48	1.56	8.90	2.50	1.57	8.50	2.46	1.45	9.00	2.53	1.61	9.09	2.50	1.61	8.70	2.50	1.52	**17:00**
18:00	9.40	2.57	1.73	9.14	2.54	1.65	9.14	2.57	1.67	8.90	2.54	1.59	8.82	2.50	1.55	9.16	2.45	1.60	8.69	2.49	1.51	**18:00**
19:00	9.54	2.60	1.79	9.54	2.58	1.77	9.40	2.59	1.74	9.14	2.54	1.65	8.69	2.45	1.49	9.00	2.51	1.59	8.68	2.52	1.53	**19:00**
20:00	9.60	2.57	1.78	9.47	2.60	1.77	9.43	2.57	1.74	9.20	2.55	1.67	8.57	2.42	1.45	8.81	2.54	1.57	8.89	2.55	1.60	**20:00**
21:00	9.34	2.51	1.67	9.26	2.53	1.67	9.17	2.56	1.67	8.85	2.51	1.56	8.55	2.47	1.46	8.38	2.46	1.43	8.91	2.52	1.58	**21:00**
22:00	8.74	2.47	1.51	8.85	2.48	1.54	8.96	2.48	1.57	8.68	2.48	1.50	8.33	2.46	1.41	8.14	2.44	1.36	8.42	2.46	1.44	**22:00**
23:00	7.84	2.29	1.21	8.01	2.37	1.29	8.01	2.41	1.31	7.65	2.36	1.21	7.57	2.35	1.18	7.56	2.31	1.16	7.58	2.33	1.18	**23:00**
Average:	**8.06**	**2.35**	**1.32**	**8.03**	**2.34**	**1.31**	**7.93**	**2.36**	**1.29**	**7.81**	**2.33**	**1.25**	**7.89**	**2.33**	**1.27**	**7.98**	**2.35**	**1.31**	**7.87**	**2.36**	**1.29**	**Ave**
Minimum:	**5.69**	**1.92**	**0.66**	**5.60**	**1.90**	**0.63**	**5.80**	**1.97**	**0.69**	**5.62**	**1.91**	**0.64**	**5.68**	**1.95**	**0.66**	**5.62**	**1.92**	**0.65**	**5.61**	**1.94**	**0.65**	**Min**
Maximum:	**9.61**	**2.60**	**1.80**	**9.54**	**2.60**	**1.77**	**9.43**	**2.59**	**1.74**	**9.20**	**2.55**	**1.67**	**9.64**	**2.55**	**1.77**	**10.04**	**2.60**	**1.86**	**9.91**	**2.61**	**1.88**	**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.

Hourly Data: Depth, Velocity and Flow

From 2/28/2011 to 3/7/2011

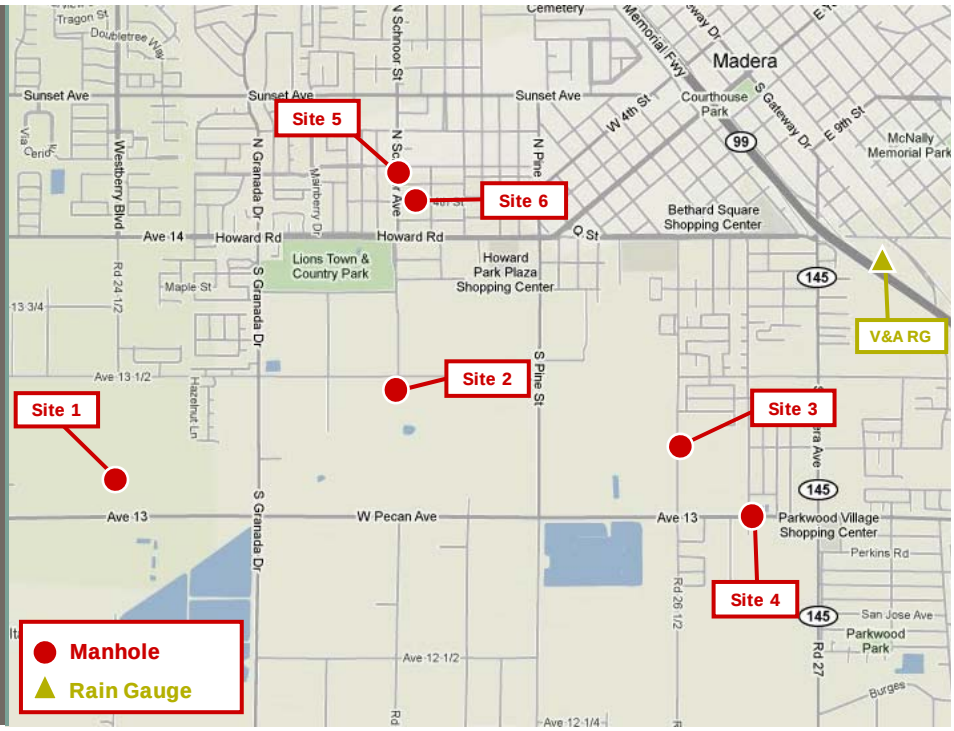
Monitoring Site:

Site 6

	Level (in)	Vel (fps)	Flow (mgd)
Weekly Average:	**7.84**	**2.34**	**1.27**
Weekly Minimum:	**5.45**	**1.87**	**0.60**
Weekly Maximum:	**9.35**	**2.59**	**1.72**

	Monday 2/28/2011			Tuesday 3/1/2011			Wednesday 3/2/2011			Thursday 3/3/2011			Friday 3/4/2011			Saturday 3/5/2011			Sunday 3/6/2011			
Hour	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Level (in)**	**Vel (fps)**	**Flow (mgd)**	**Hour**
0:00	6.92	2.22	0.99	6.80	2.20	0.96	6.90	2.24	1.00	6.96	2.18	0.98										**0:00**
1:00	6.13	2.06	0.78	6.10	2.08	0.78	6.38	2.11	0.84	6.17	2.04	0.78										**1:00**
2:00	5.87	2.04	0.73	5.61	1.96	0.65	5.90	2.01	0.72	5.72	1.89	0.65										**2:00**
3:00	5.67	1.97	0.67	5.51	1.92	0.63	5.83	2.01	0.71	5.54	1.88	0.62										**3:00**
4:00	5.57	1.93	0.64	5.45	1.88	0.60	5.85	2.00	0.71	5.55	1.87	0.61										**4:00**
5:00	5.94	2.06	0.75	5.74	2.02	0.70	5.99	2.02	0.74	5.83	1.95	0.69										**5:00**
6:00	6.51	2.15	0.88	6.44	2.17	0.88	6.73	2.22	0.95	6.44	2.10	0.85										**6:00**
7:00	7.46	2.36	1.17	7.42	2.33	1.14	7.38	2.36	1.15	7.43	2.29	1.13										**7:00**
8:00	8.23	2.42	1.37	8.22	2.43	1.37	8.25	2.46	1.39	8.23	2.40	1.35										**8:00**
9:00	8.64	2.49	1.50	8.20	2.41	1.35	8.32	2.44	1.40	8.36	2.38	1.37										**9:00**
10:00	8.97	2.57	1.63	8.53	2.41	1.43	8.31	2.45	1.40	8.51	2.41	1.42										**10:00**
11:00	8.86	2.48	1.55	8.63	2.47	1.49	8.47	2.48	1.45	8.83	2.46	1.52										**11:00**
12:00	9.00	2.51	1.59	8.70	2.50	1.52	8.60	2.47	1.48	8.89	2.50	1.56										**12:00**
13:00	8.70	2.48	1.51	8.51	2.43	1.44	8.47	2.44	1.43	8.79	2.45	1.51										**13:00**
14:00	8.57	2.46	1.46	8.43	2.45	1.43	8.45	2.45	1.43	8.63	2.44	1.47										**14:00**
15:00	8.37	2.45	1.41	8.49	2.47	1.45	8.49	2.48	1.46	8.34	2.48	1.42										**15:00**
16:00	8.33	2.48	1.42	8.45	2.46	1.44	8.66	2.50	1.51	8.40	2.41	1.40										**16:00**
17:00	8.63	2.50	1.50	8.61	2.46	1.48	8.77	2.50	1.54	8.63	2.44	1.47										**17:00**
18:00	9.00	2.51	1.60	9.02	2.51	1.60	8.75	2.53	1.55	8.90	2.50	1.57										**18:00**
19:00	9.29	2.59	1.72	9.31	2.56	1.70	9.11	2.54	1.64	9.13	2.50	1.62										**19:00**
20:00	9.26	2.56	1.69	9.35	2.55	1.70	9.04	2.50	1.60	9.14	2.48	1.61										**20:00**
21:00	8.90	2.52	1.58	9.16	2.52	1.64	9.00	2.51	1.59	9.05	2.51	1.61										**21:00**
22:00	8.55	2.49	1.48	8.66	2.51	1.51	8.63	2.47	1.49	8.68	2.49	1.51										**22:00**
23:00	7.81	2.41	1.27	7.80	2.40	1.27	7.82	2.36	1.25	7.90	2.40	1.28										**23:00**
Average:	**7.88**	**2.36**	**1.29**	**7.80**	**2.34**	**1.26**	**7.84**	**2.36**	**1.27**	**7.83**	**2.31**	**1.25**										**Ave**
Minimum:	**5.57**	**1.93**	**0.64**	**5.45**	**1.88**	**0.60**	**5.83**	**2.00**	**0.71**	**5.54**	**1.87**	**0.61**										**Min**
Maximum:	**9.29**	**2.59**	**1.72**	**9.35**	**2.56**	**1.70**	**9.11**	**2.54**	**1.64**	**9.14**	**2.51**	**1.62**										**Max**

Note: Values shown are hourly averages of the recorded 15-minute data. Stated minimum and maximum flows will not necessarily correspond to the values stated in the remainder of the report, which were reported based on the recorded 15-minute data.



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San Diego, CA 92123
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Houston, TX 77065
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713.840.6491 **Fax**

vaengineering.com

APPENDIX B

Hydraulic Model Calibration Exhibits

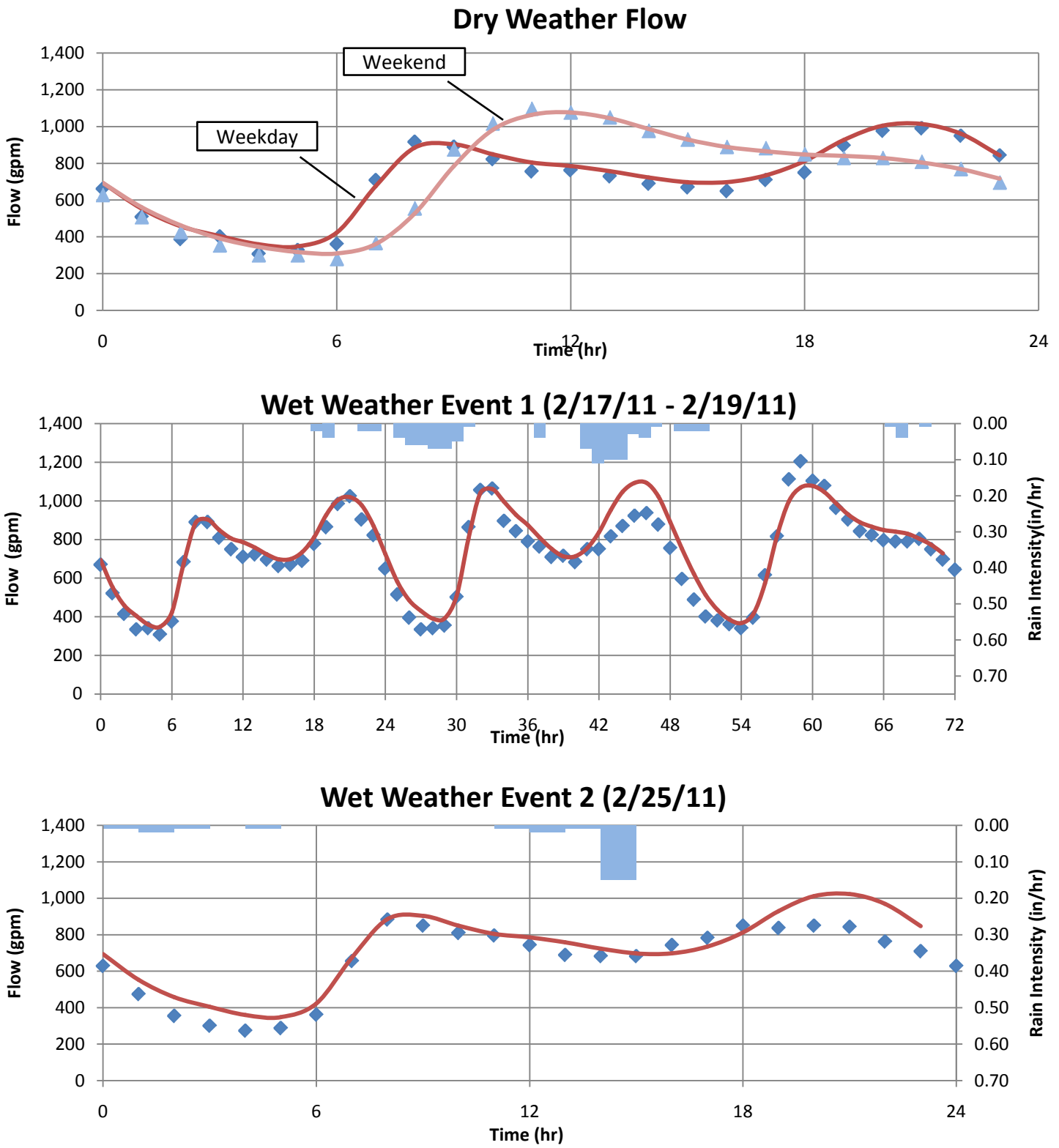
CITY OF MADERA

SANITARY SEWER MASTER PLAN

Sanitary Sewer
Hydraulic Model Calibration

April 10, 2012

AKEL
ENGINEERING GROUP, INC.



LEGEND

Rain Event

V&A Flow Monitoring

Hydraulic Model

April 10, 2012

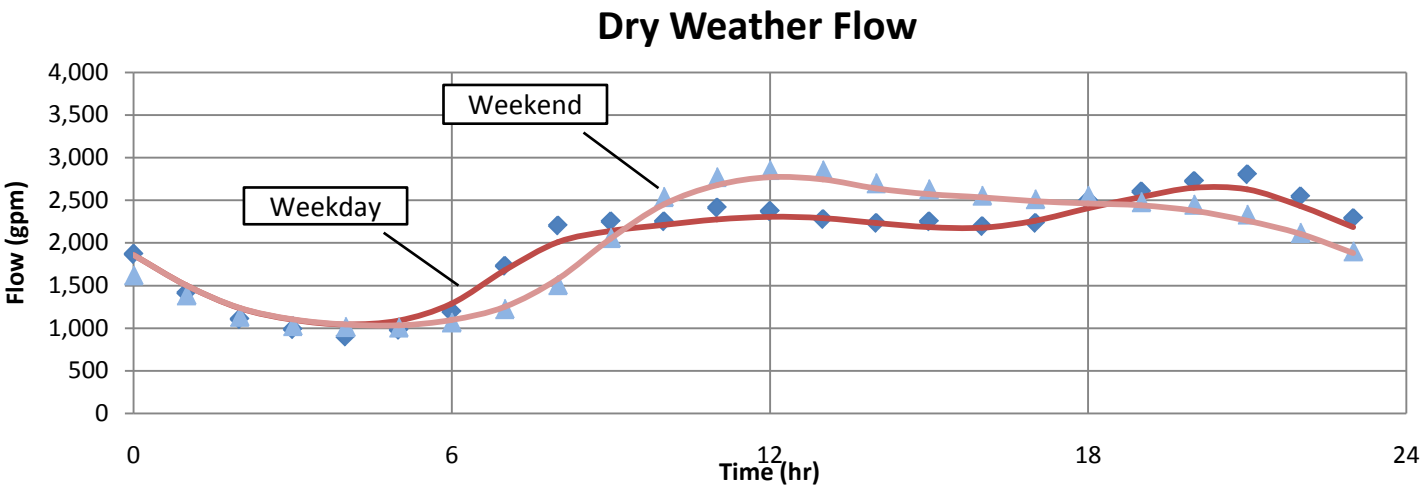
AKEL ENGINEERING GROUP, INC

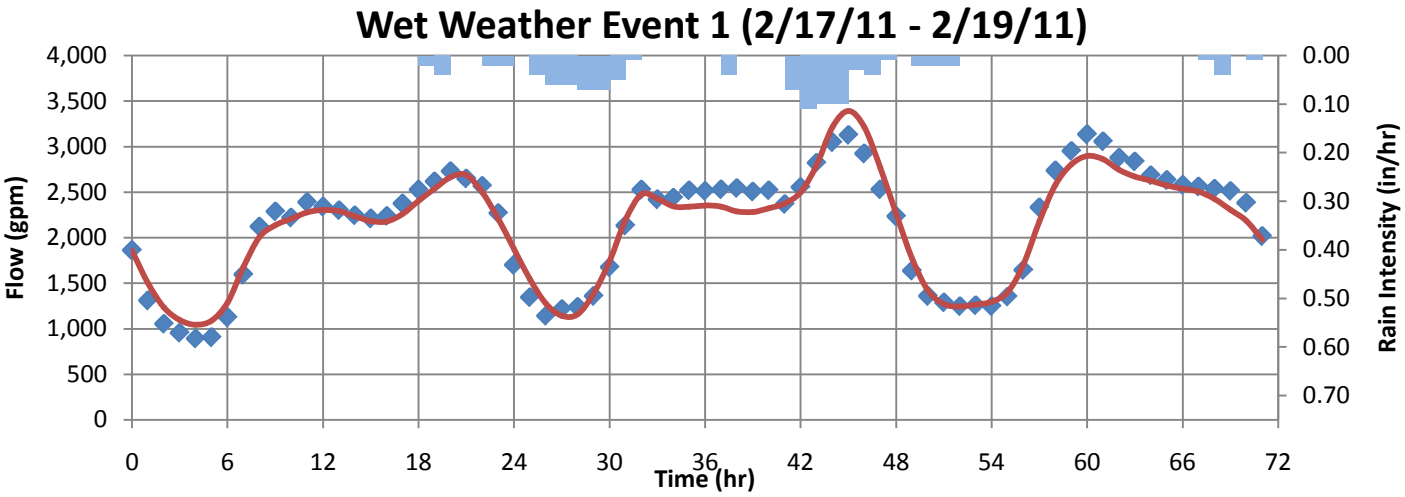
Figure 2

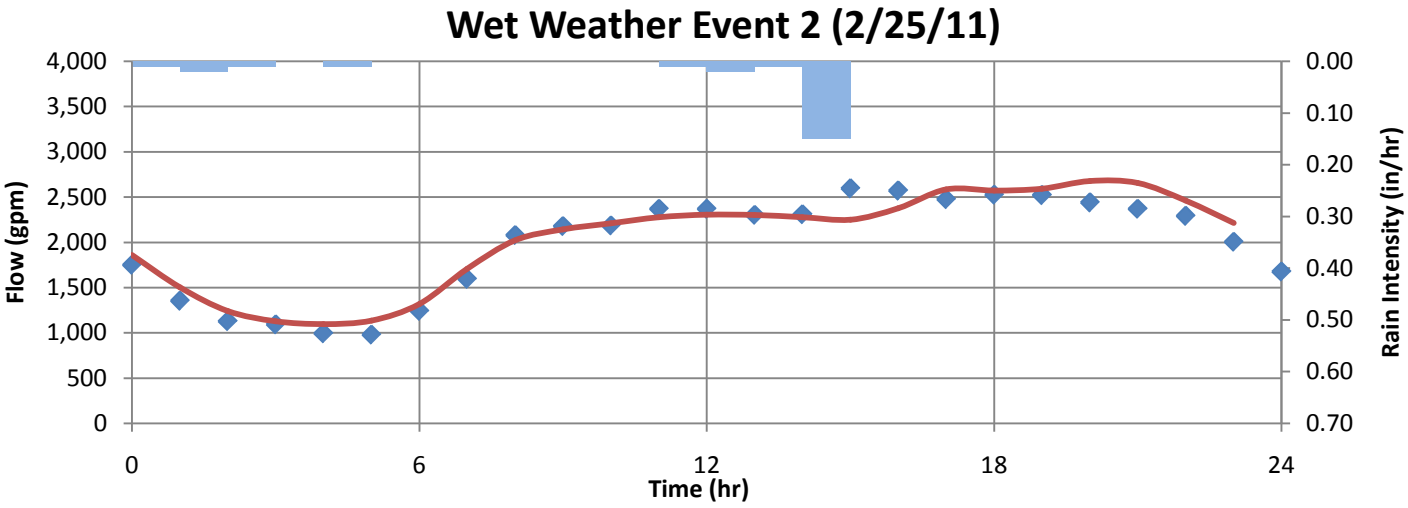
Site 1 Calibration

Westberry Rd, north of Pecan Ave

Sanitary Sewer Master Plan
City of Madera







LEGEND

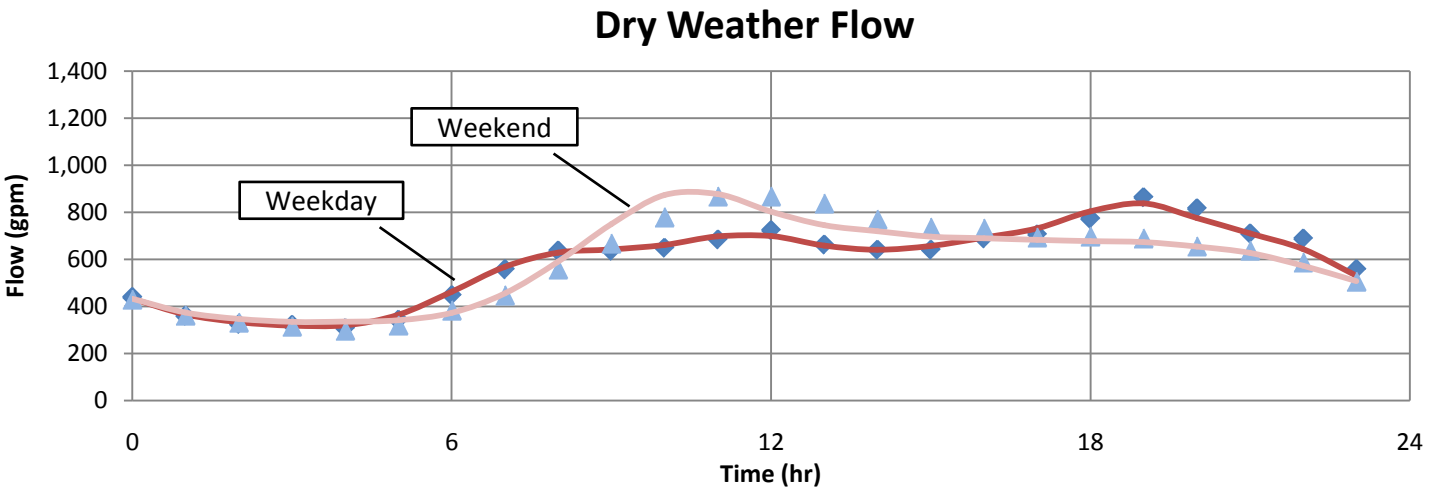
- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

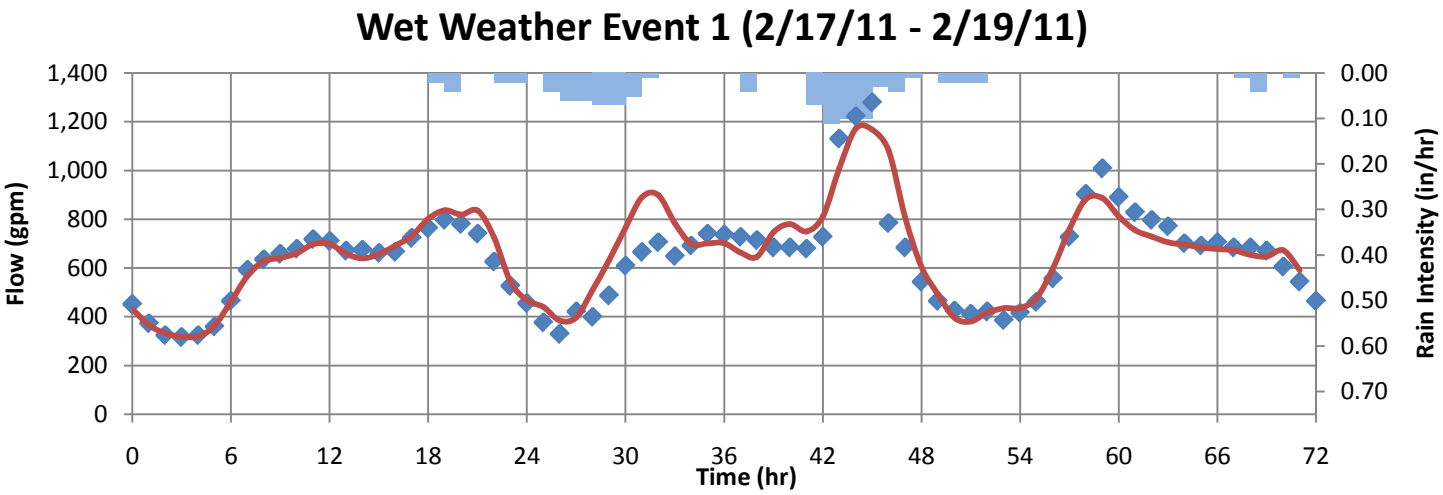
April 10, 2012

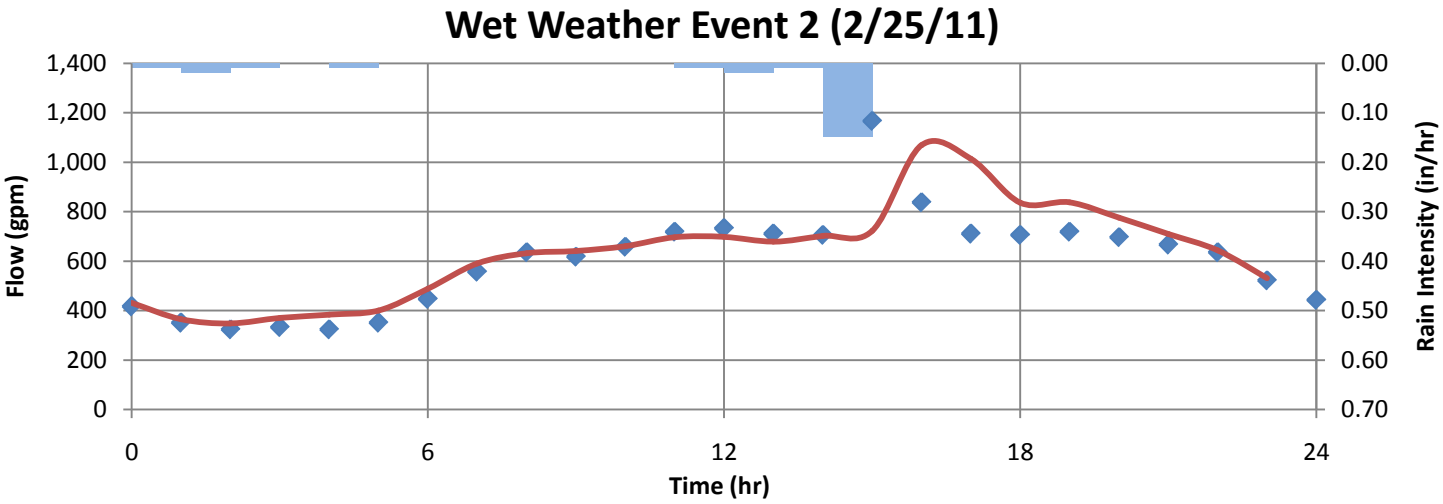
AKEL ENGINEERING GROUP, INC

Figure 3
Site 2 Calibration
Schnoor Ave, south of Almond Ave

Sanitary Sewer Master Plan
City of Madera







LEGEND

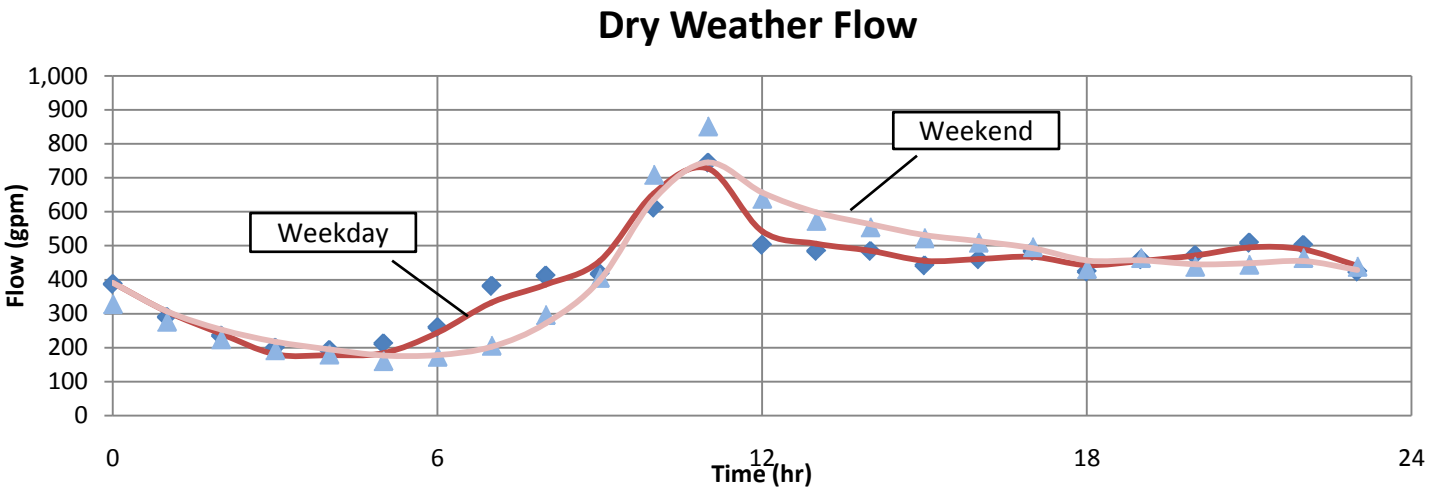
- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

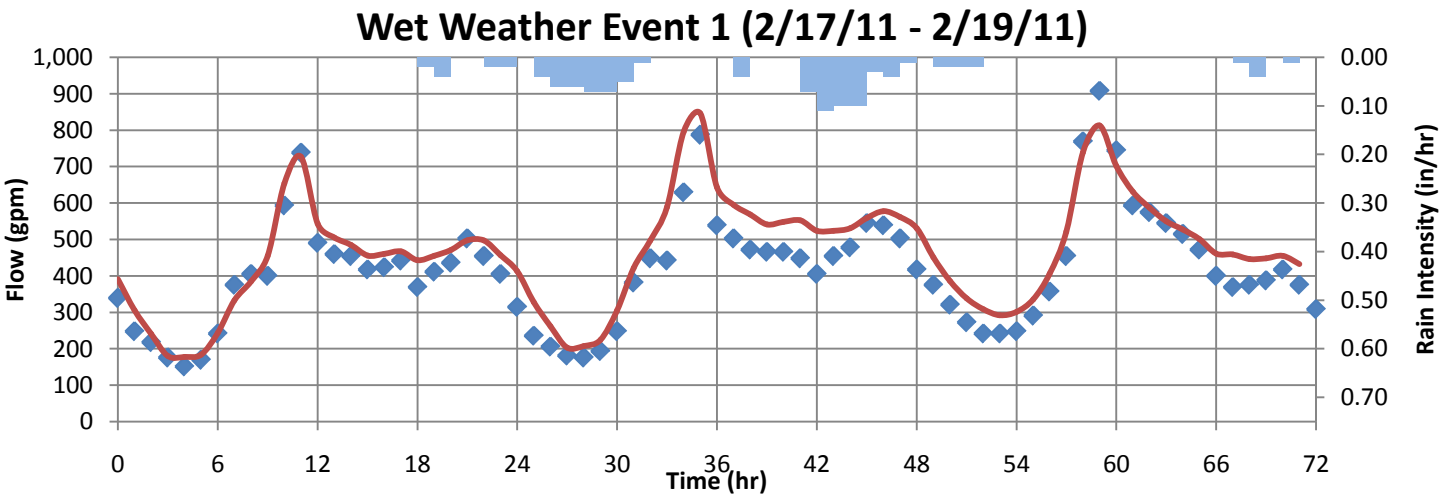
April 10, 2012

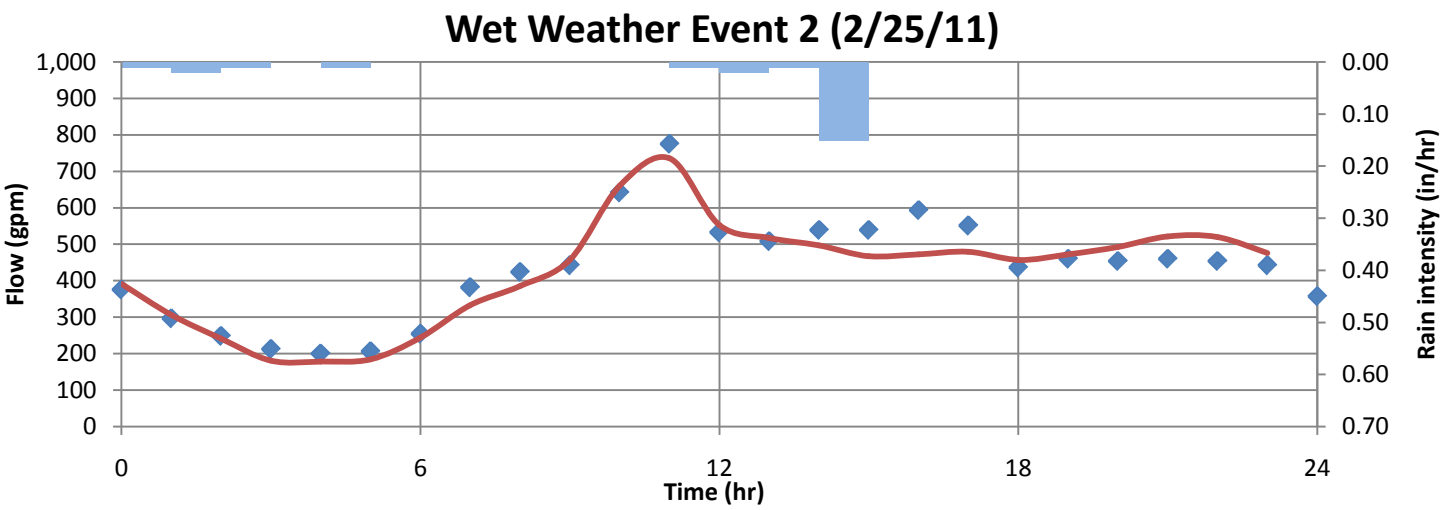
AKEL ENGINEERING GROUP, INC

Figure 4
Site 3 Calibration
Stadium Rd at Gary Lane, north of Pecan Ave

Sanitary Sewer Master Plan
City of Madera







LEGEND

- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

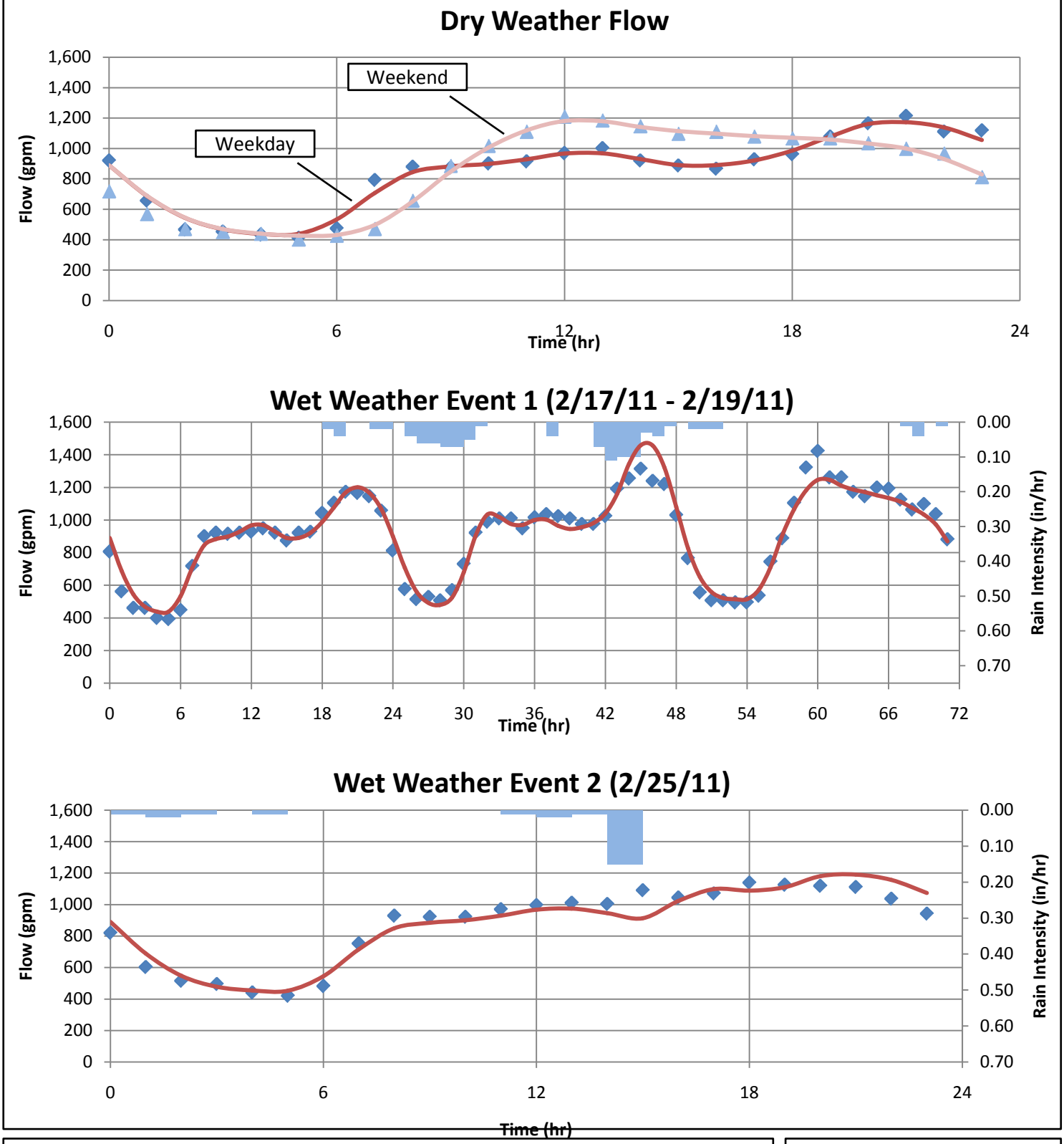
AKEL ENGINEERING GROUP, INC

Figure 5

Site 4 Calibration

Pecan Ave, between Monterey St. and Concord Ct.

Sanitary Sewer Master Plan
City of Madera



LEGEND

- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

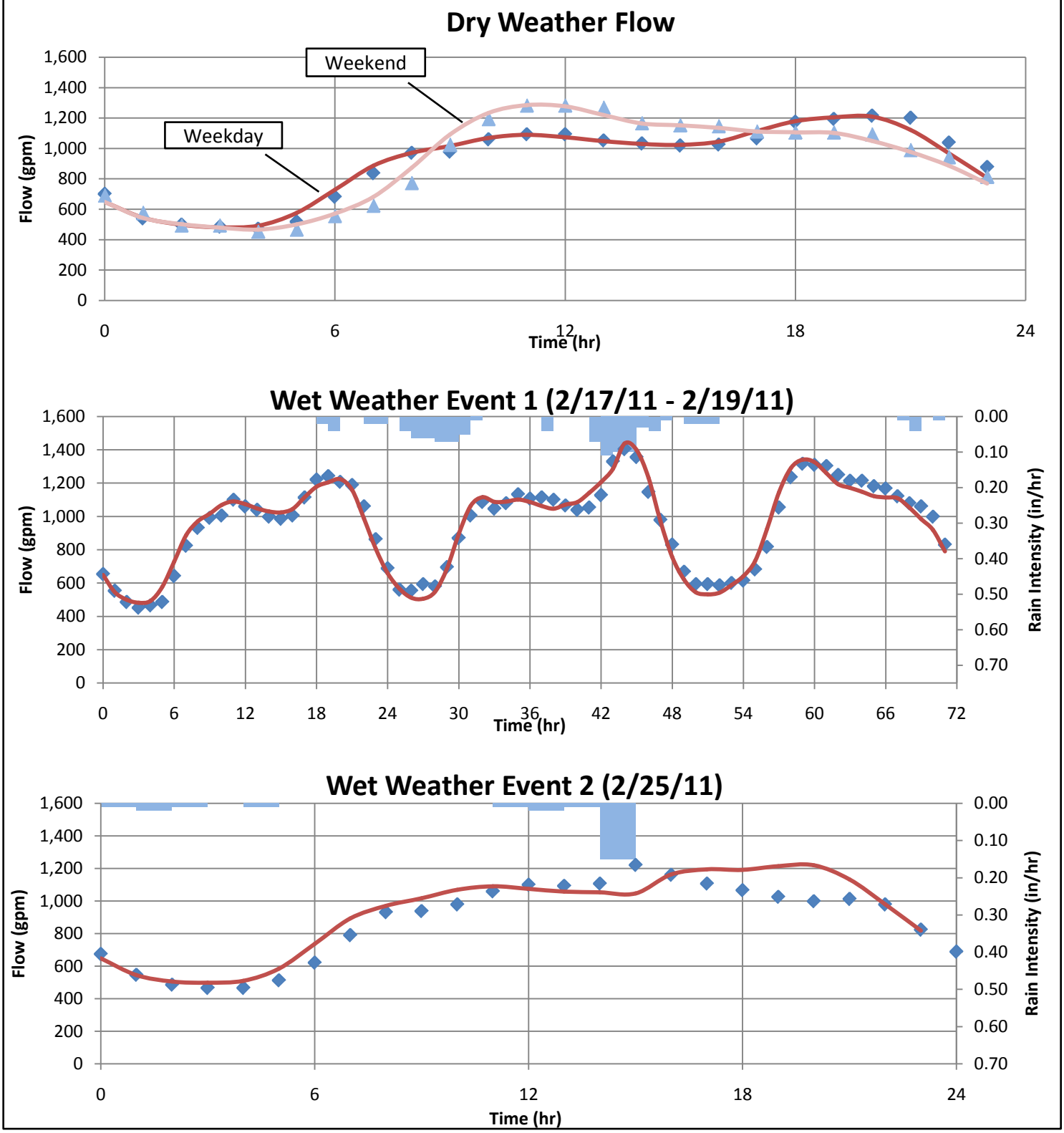
AKEL ENGINEERING GROUP, INC

Figure 6

Site 5 Calibration

Schnoor Ave, north of Jennings St

Sanitary Sewer Master Plan
City of Madera



LEGEND

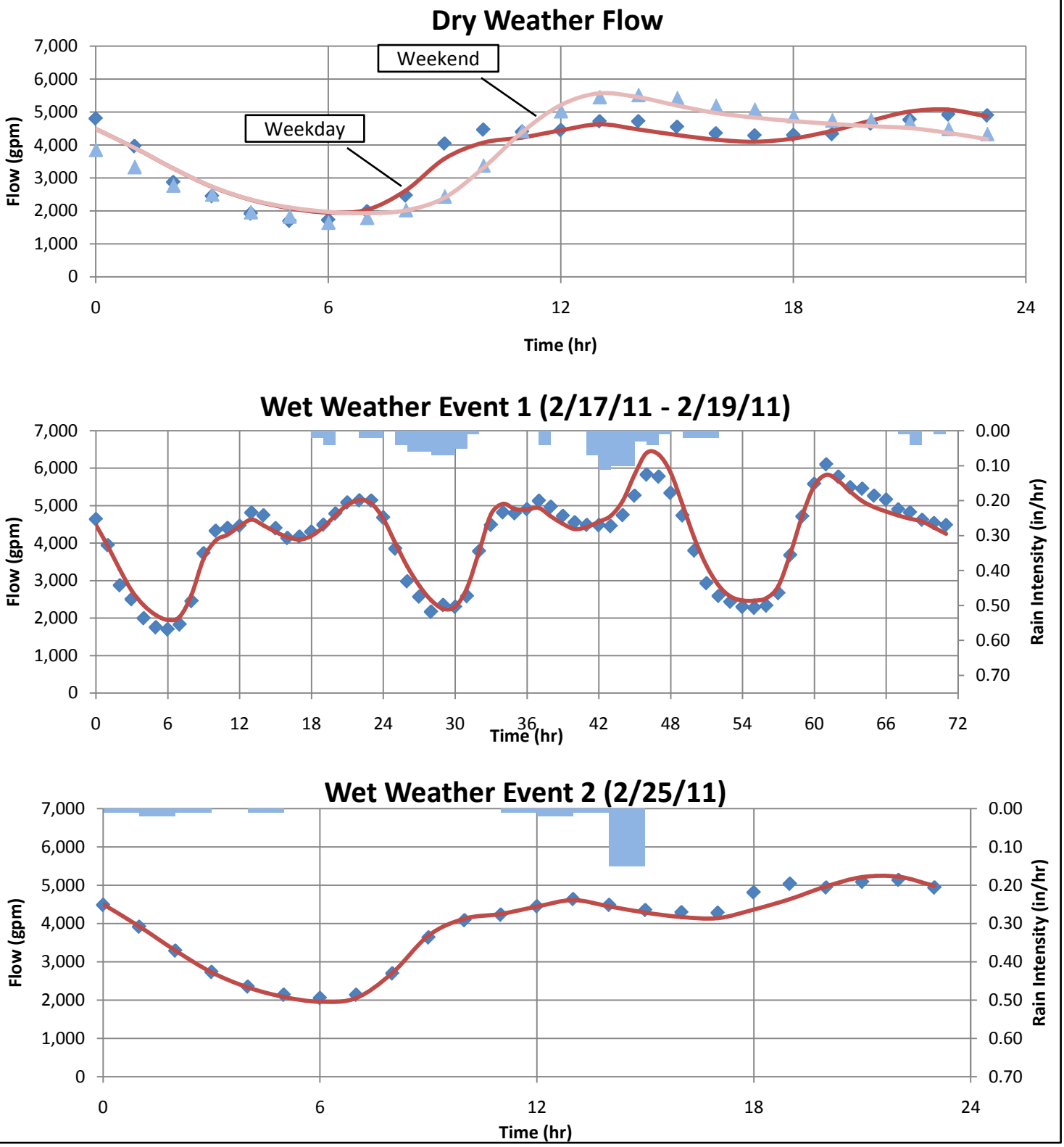
- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

AKEL ENGINEERING GROUP, INC

Figure 7
Site 6 Calibration
West Fourth St, east of Schnoor Ave

Sanitary Sewer Master Plan
City of Madera



LEGEND

- Rain Event
- V&A Flow Monitoring
- Hydraulic Model

April 10, 2012

AKEL ENGINEERING GROUP, INC

Figure 8
WWTP Calibration
WWTP Flow Meter
Sanitary Sewer Master Plan
City of Madera

APPENDIX C

Fairgrounds Lift Station Analysis

CITY OF MADERA

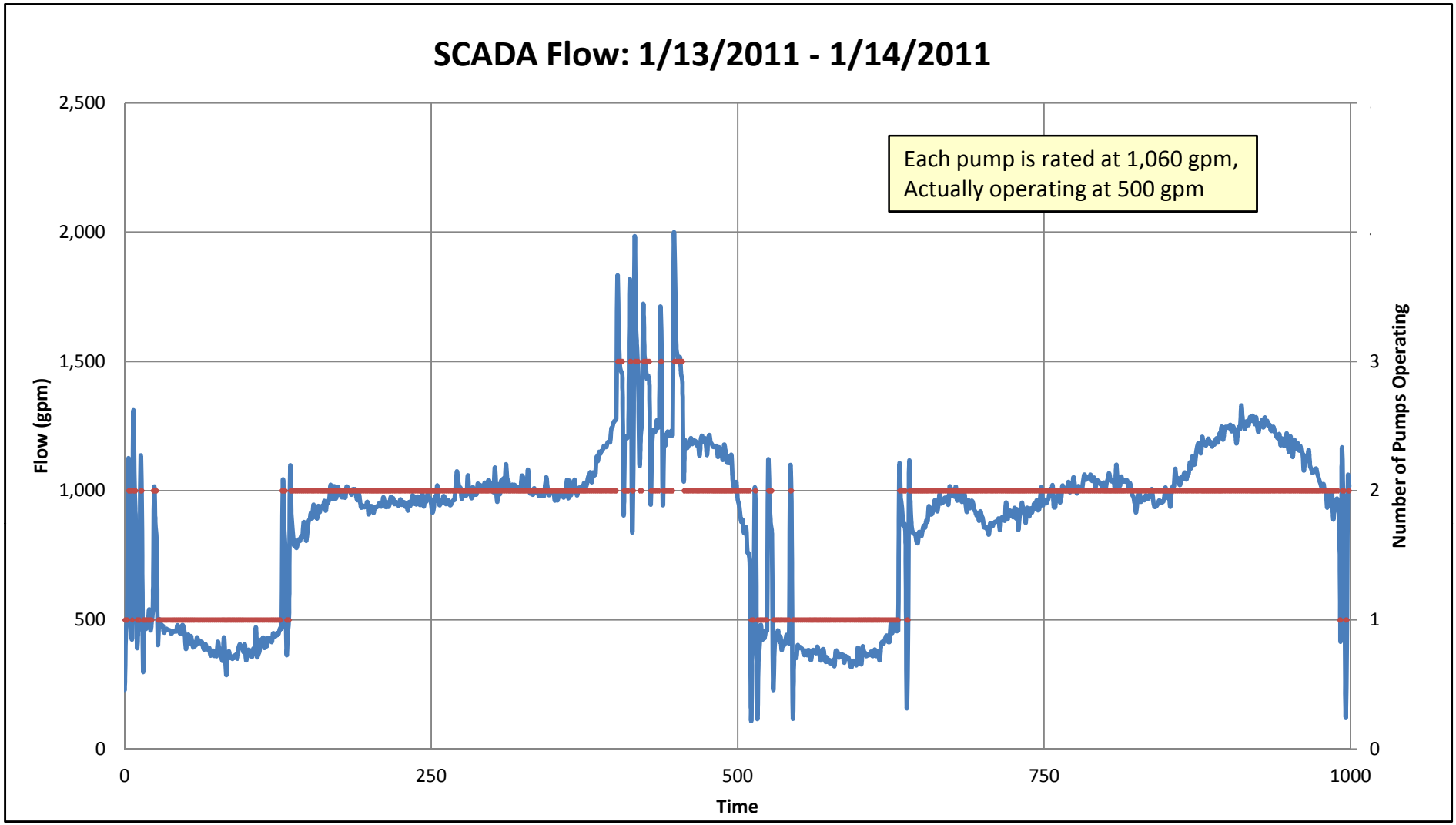
FAIR GROUNDS LIFT STATION ANALYSIS

SANITARY SEWER SYSTEM MASTER PLAN

PRELIMINARY

November 2012

AKEL ENGINEERING GROUP, INC.



LEGEND

Flow

Number of Pumps Operating

November 21, 2011

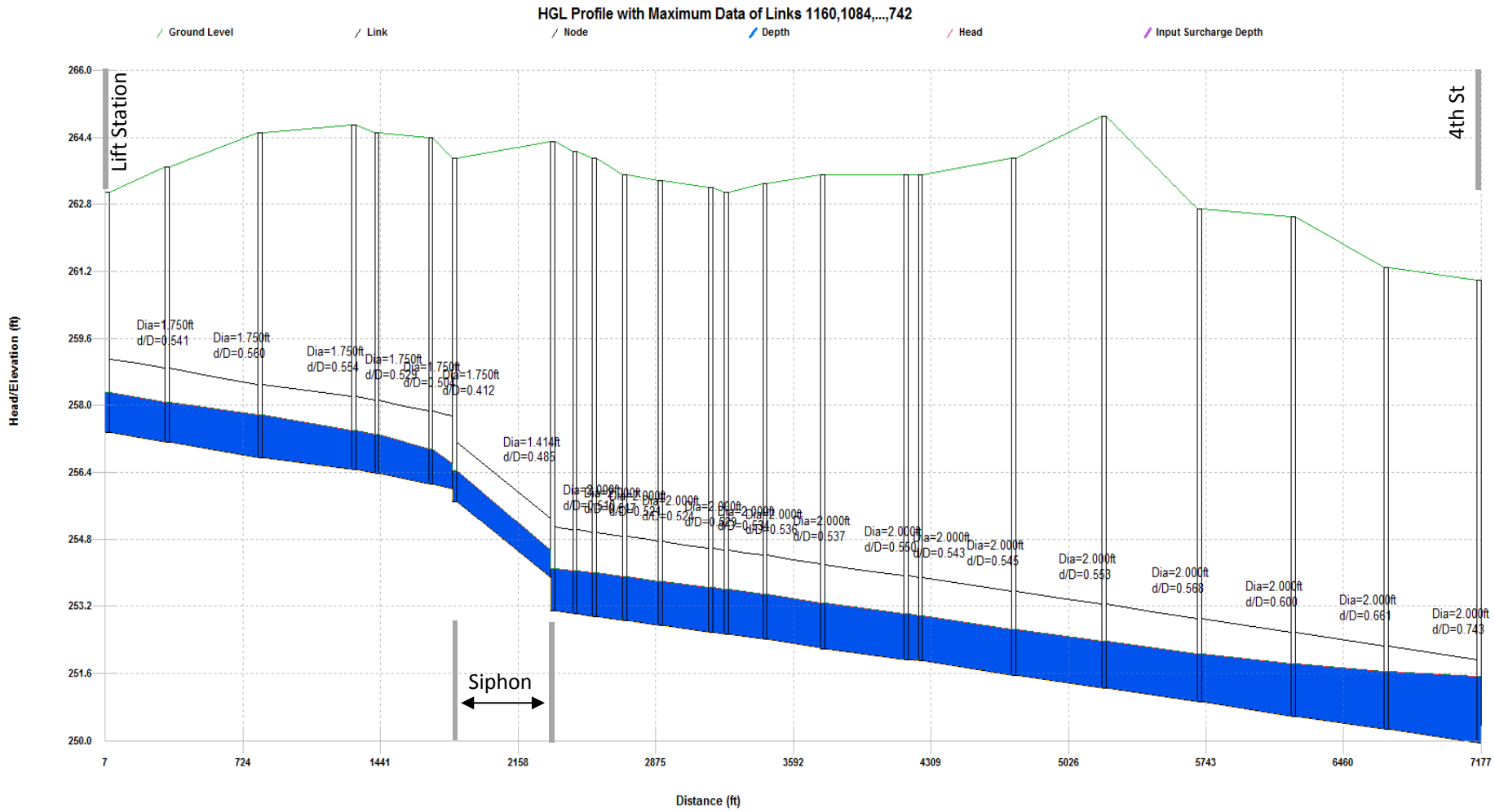
AKEL ENGINEERING GROUP, INC.

Figure 1

Fair Grounds Lift Station Operation Analysis

Sanitary Sewer System Master Plan

City of Madera



Notes

Profile shows Schnoor Ave
from Fair Grounds Lift Station to 4th St

Each pump at Fair Grounds Lift Station is modeled at 1,060 gpm (Total Flow = 1,060 gpm)

November 21, 2011

AKEL ENGINEERING GROUP, INC.

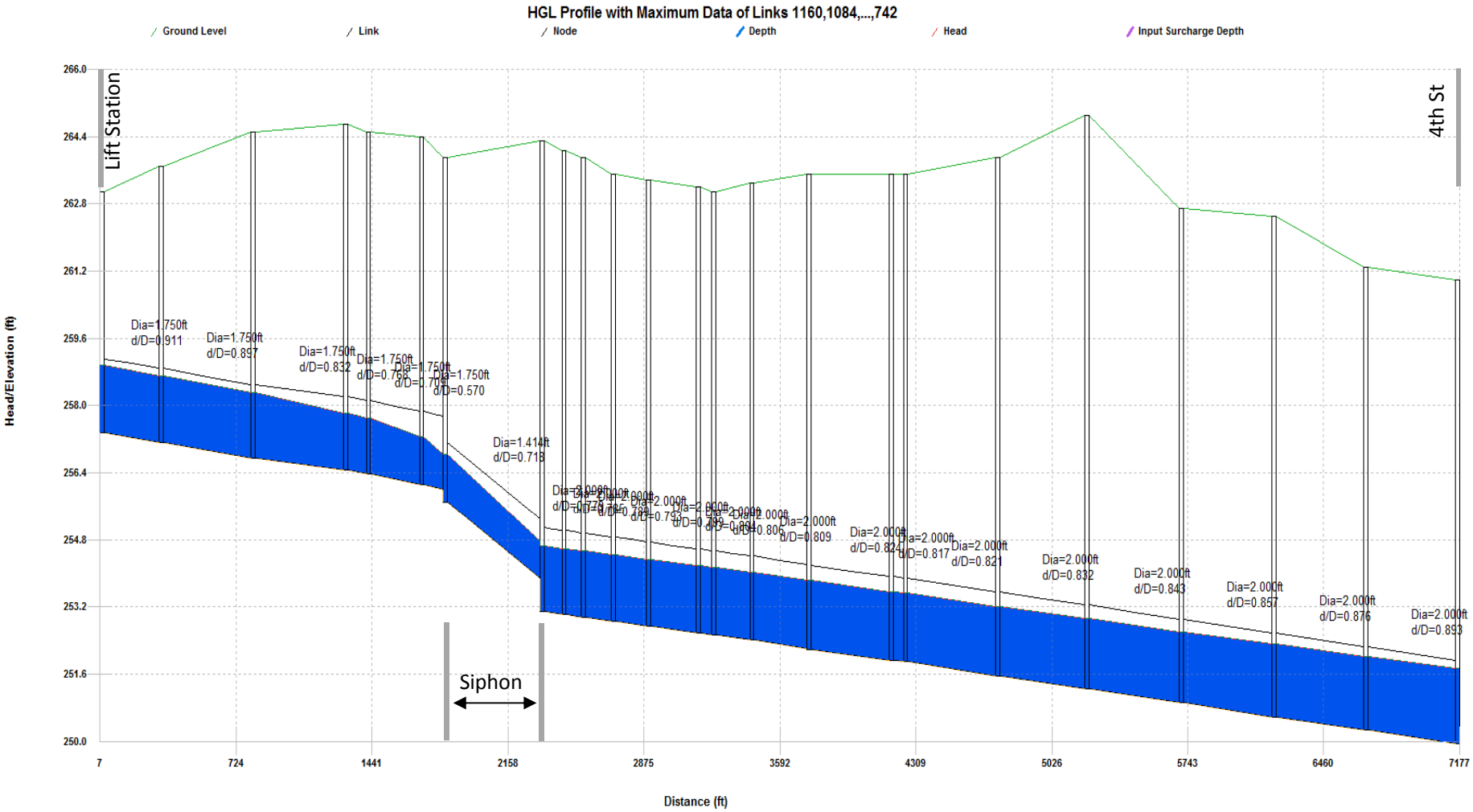
Figure 2

Schnoor Ave Profile with

1 Pump Active

at Fair Ground LS

Sanitary Sewer System
Master Plan
City of Madera



Notes

Profile shows Schnoor Ave
from Fair Grounds Lift Station to 4th St

Each pump at Fair Grounds Lift Station is modeled at 1,060 gpm (Total Flow = 2,120 gpm)

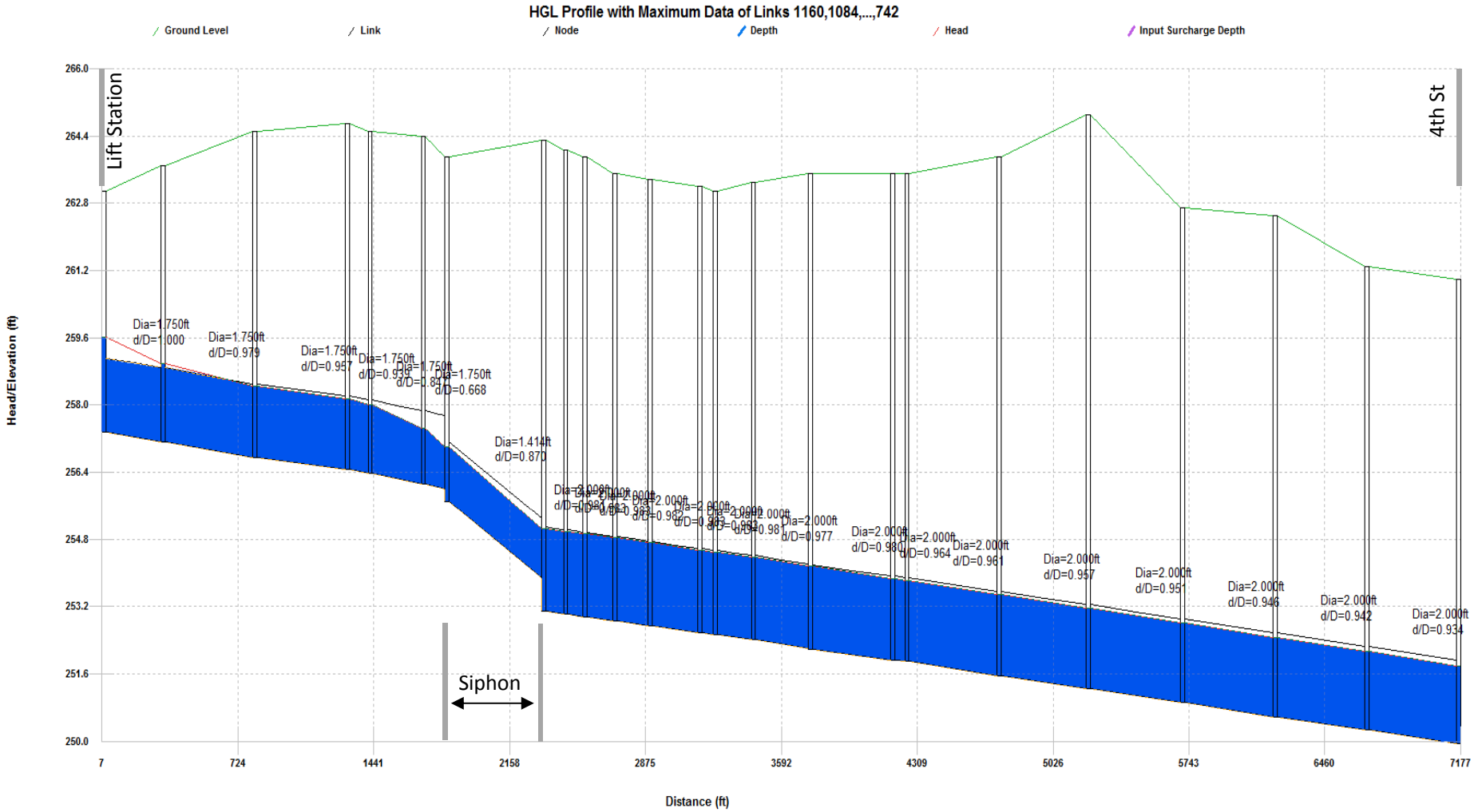
November 21, 2011

AKEL ENGINEERING GROUP, INC.

Figure 3

Schnoor Ave Profile with
2 Pumps Active
at Fair Ground LS

Sanitary Sewer System
Master Plan
City of Madera



Notes

Profile shows Schnoor Ave
from Fair Grounds Lift Station to 4th St

Each pump at Fair Grounds Lift Station is modeled at 1,060 gpm (Total Flow = 3,180 gpm)

November 21, 2011

AKEL ENGINEERING GROUP, INC.

Figure 4

Schnoor Ave Profile with
3 Pumps Active
at Fair Ground LS

Sanitary Sewer System
Master Plan
City of Madera

APPENDIX D

Schnoor Avenue Trunk Sewer Replacement/Rehabilitation Evaluation (MWH)

CITY OF MADERA

Schnoor Ave. Trunk Sewer Replacement/Rehabilitation Evaluation

Final

October 2013

1 Introduction

1.1 Purpose

This TM documents the evaluation of the structural integrity and hydraulic capacity of Schnoor Ave. Trunk Sewer and provides a methodology for rehabilitation, replacement and/or repair of the sewer pipelines and manholes to extend the useful life of the sewer. The TM also provides a preliminary budget level Opinion of Probable Construction Cost (OPCC).

1.2 Background

The Schnoor Ave. Trunk Sewer was televised using closed circuit television (CCTV) in May of 2007. Digital Versatile Disks (DVDs) of the inspection were provided by the City of Madera. In May of 2008 Blair Church and Flynn (BCF) conducted an evaluation of the sewers utilizing the CCTV inspections, analyzed the capacity for current and future flows and provided recommendations for improvement. Pipeline Assessment Certification Program (PACP) inspection reports were included as Appendix B of the BCF Schnoor Ave. Trunk Sewer Evaluation. In general the BCF evaluation found the following:

1. Under full build out conditions, with the recommended improvements to Pump Station 1, all segments of the sewer would be surcharged and recommended that the sewer be upsized or that a relief sewer be constructed.
2. In regards to the condition of the existing pipe, it found the vitrified clay pipe (VCP) to be in good condition and recommended no action for those segments.
3. It considered the concrete pipe to be in fair condition due to exposed aggregate of varying degree and recommended that if a relief sewer was constructed and the existing sewer remained in service it should be rehabilitated.

In the BCF evaluation the project area was broken into 4 segments:

- From PS1 to the Fresno River
- Across the Fresno River
- From the Fresno River to Fourth Street
- From Fourth Street to Howard Road

Currently Akel Engineering is re-analyzing the flows for this project area. Flow data provided by Akel Engineering will be used to evaluate the hydraulic capacity.

2 Condition Evaluation and Rehabilitation Methodology

The overall methodology used to determine recommended improvements to the sewer system is highlighted in Table 1. The primary objective of the rehabilitation recommendation is to extend the useful life of the sewer system. Areas with significant debris or roots are recommended for cleaning or root removal to restore the hydraulic capacity of the sewer. Areas with severe corrosion are recommended for cleaning followed by installation of a cured in place liner to extend the useful life of the sewer.

Table 1. Rehabilitation Methodology

Defect	Degree	Recommendation
Roots	Fine	Monitor
	Medium	Deploy root cutter
	Tap	Monitor
Settled Deposits	Fine	Monitor
	Medium	Clean
	Heavy	Clean
Attached Deposits	Ragging	Monitor
	Fouling	Clean
Longitudinal Crack	Minor	Monitor
Exposed Aggregate		Clean and CIPP line
Missing Aggregate		Clean and CIPP line
Broken		Point Repair
Sag		Point Repair

3 Condition Evaluation

In general the VCP was found to be in good condition with roots in several locations and three longitudinal cracks observed near joints. However, the concrete pipe was found to be in severe condition with varying degrees of exposed aggregated and missing aggregate in some locations. There were also settled deposits ranging from fine to medium and attached deposits causing ragging in both the VCP and concrete pipelines. Shown on left side Table 2 are the assessments of the condition of the pipelines in each segment of the Schnoor Avenue sewer.

4 Rehabilitation Recommendations

This TM updates the recommendations based upon our review of the 2007 CCTV DVD's and the 2008 Blair Church and Flynn (BCF) report. The updated recommendations are provided in accordance with the four major segments listed above. MWH has the condition of the Schnoor Ave Trunk Sewer and provided the condition assessment on the left hand side of Table 2. Recommendations are provided for the additional pipeline segments extending to Industrial Ave (included in the 2007 CCTV inspections) that were not discussed in the BCF evaluation.

4.1 From PS1 to Fresno River

This reach of the Schnoor Ave. Trunk Sewer is 21" VCP. The pipeline is in good condition. MWH recommends that the reach be monitored for settled and attached deposits from MH1 (MH2124) to MH3 (MH2121) and for roots from MH3 (MH2121) to MH5 (MH2011). From MH3 (MH2121) to MH4 (MH2050) the roots were designated as medium therefore, MWH recommends a root cutter be deployed to this segment.

Table 2. Condition and Capacity Evaluation

(MH ID)	LN_ID	Segments	Length	Condition Assessment								Capacity Assessment (MGD)												
				BCF		MWH		BCF		MWH		Hydraulic Model Flow Capacity (St. Venant Equation)	Existing PS1 (AKEL 2013)				Existing PS1 (BCF 2007)				Improved PS1 (BCF Future Project)			
													DWF		WWF		DWF		WWF		DWF		WWF	
				Material	Defect	Material	Defect	Assessment	Recommendation	Assessment	Recommendation		Existing	Future (Improved PS-1)	Existing	Future (Improved PS-1)	Existing	Future	Existing	Future	Existing	Future	Existing	Future
PS1 to MH1098		**PS1 to MH7**																						
PS1 - MH2124	2126L	PS1 - MH1		21" VCP	none	No Video Available		Good	none	Unknown	none	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2124 - MH2122	2125L	MH1 - MH2	316	21" VCP	Deposits Settled: Fine @ 292.2 ft.	21" VCP	Deposits Settled: Fine @ 292.2 ft.	Good	none	Good	Monitor	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2122 - MH2121	2123L	MH2 - MH3	503	21" VCP	Deposits Settled: Fine @ 83.6 ft. to 120.1 ft., 177.2 ft. to 220.8 ft. and 325.7 ft. to 334.7 ft. Deposits Attached: Ragging @ 315.4 ft. to 364.5 ft.	21" VCP	Deposits Settled: Fine @ 83.6 ft. to 120.1 ft., 163 ft. to 168 ft., 250 ft., 288 ft., 177.2 ft. to 220.8 ft. and 325.7 ft. to 334.7 ft. Deposits Attached: Ragging @ 315.4 ft. to 364.5 ft.	Good	none	Good	Monitor	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2121 - MH2050	2053L	MH3 - MH4	505	21" VCP	Roots, Medium: Joint @ 396 ft.	21" VCP	Roots, Medium: Joint @ 396 ft.	Good	none	Good	Deploy root cutter	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2050 - MH2011	L763	MH4 - MH5	111	21" VCP	Roots, Fine: Joint @ 90.0 ft. & 104 ft.	21" VCP	Roots, Fine: Joint @ 91.0 ft., 98 ft. & 104 ft.	Good	none	Good	Monitor	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2011 - MH1093	2051L	MH5 - MH6	282	21" VCP	none	21" VCP	none	Good	none	Good	none	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH1093 - MH1098	L2071	MH6 - MH7	111	21" VCP	none	21" VCP	none	Good	none	Good	none	3.31	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH1098 - MH2046	2128L / 2129L	MH7 - MH8		Dual 12" (	Unknown	Dual 12" (	Unknown	Unknown	none	Unknown	none	No Data Available	2.05	3.23	2.40	3.60	2.54	2.67	3.45	3.58	4.96	5.09	5.01	5.14
MH2046 to MH4331		**MH8 to MH24**																						
MH2046 - MH2044	2047L	MH8 - MH9	101	24" VCP	none	24" VCP	none	Good	none	Good	none	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH2044 - MH2042	2045L	MH9 - MH10	92	24" VCP	Roots, Tap: Joint @ 0.0 ft.	24" VCP	Root, Fine: Joint @ 45.9 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH2042 - MH3394	3395L	MH10 - MH11	145	24" VCP	none	24" VCP	Deposits Settled: Medium @ 85.2 ft. to 91 ft.	Good	none	Good	Clean	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3394 - MH1136*	3369L	MH11 - MH12	178	24" VCP	none	24" VCP	none	Good	none	Good	none	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH1136* - MH3365	3370L	MH12 - MH13	261	24" VCP	none	24" VCP	none	Good	none	Good	none	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3365 - MH3357	3371L	MH13 - MH14	67	24" VCP	none	24" VCP	Deposits Attached: Ragging @ 8 ft. to 66.6 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3357 - MH1137*	3358L	MH14 - MH15	195	24" VCP	none	24" VCP	Deposits Attached: Ragging @ 0 ft. to 194 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH1137* - MH3306	3308L	MH15 - MH16	304	24" VCP	none	24" VCP	Deposits Attached: Ragging @ 0 ft. to 299 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3306 - MH3392	3393L	MH16 - MH17	453	24" VCP	Longitudinal Crack @ 119.6 ft.	24" VCP	Longitudinal Crack @ 119.6 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3392 - MH1138*	L1095	MH17 - MH18	55	24" VCP	none	24" VCP	none	Good	none	Good	none	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH1138* - MH3304	3421L	MH18 - MH19	504	24" VCP	2 Longitudinal Cracks @ 503.0 ft.	24" VCP	2 Longitudinal Cracks @ 503.0 ft.	Good	none	Good	Monitor	3.67	2.15	3.45	2.72	3.83	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3304 - MH3327	3305L	MH19 - MH20	491	24" VCP	none	24" VCP	none	Good	none	Good	none	3.74	2.28	3.66	3.00	4.29	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH3327 - MH4366	3303L	MH20 - MH21	513	24" VCP	none	24" VCP	none	Good	none	Good	none	3.74	2.28	3.66	3.00	4.29	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH4366 - MH4369	L2099	MH21 - MH22	501	24" VCP	none	24" VCP	none	Good	none	Good	none	3.74	2.28	3.66	3.00	4.29	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH4369 - MH4372	4373L	MH22 - MH23	501	24" VCP	none	24" VCP	none	Good	none	Good	none	3.74	2.28	3.66	3.00	4.29	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH4372 - MH4331	4375L	MH23 - MH24	500	24" VCP	none	24" VCP	none	Good	none	Good	none	3.74	2.28	3.66	3.00	4.29	2.64	2.76	3.61	3.77	5.06	5.18	5.17	5.33
MH4235 - MH4237	4238L	MH9 - MH10	419	Not Included in Report but Video provided. Location corresponds with	Surface Aggregate Projecting @ Entire Segment. Roots, fine: joint @ 109 ft.	12" RCP	Not Applicable			Severe	Clean and install CIPP Liner	No Data Available												
MH4237 - MH4331	L4242	MH10 - MH11	201	Not Included in Report but Video provided. Location corresponds with line parallel to 24" at MH 23 to 24.	Surface Aggregate Projecting @ Entire Segment. Sag @ 7 ft. to 48 ft. Deposits Attached: Ragging @107 ft. to 120 ft.	12" RCP	Not Applicable			Severe	Clean and install CIPP Liner													
MH4331 to MH30		**MH24 to MH30**																						
MH4331 - MH1032*	L1054	MH24 - MH25	34	24" RCP	Surface Aggregate Projecting @ 5.0 ft. to 28.1 ft.	24" RCP	Surface Aggregate Projecting @ 5.0 ft. to 33.9 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1032* - MH1031*	L1053	MH25 - MH26	293	24" RCP	Surface Aggregate Projecting @ 5.0 ft. to 288.9 ft.	24" RCP	Surface Aggregate Projecting @ 5.0 ft. to 293 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1031* - MH1025*	L1052	MH26 - MH27	277	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 277.4 ft.	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 277.4 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1025* - MH1024*	L1051	MH27 - MH28	42	24" RCP	Surface Aggregate Missing @ 5.0 ft. to 11.3 ft. Surface Aggregate Projecting @ 11.3 ft. to 36.9 ft.	24" RCP	Surface Aggregate Missing @ 5.0 ft. to 18.3 ft. Surface Aggregate Projecting @ 18.3 ft. to 41.7 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1024* - MH1021*	L1050	MH28 - MH29	58	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 58.4 ft.	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 58.4 ft. Root Tap: Joint @58.4 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1021* - MH1023*	L1049	MH29 - MH30	15	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 14.7 ft.	24" RCP	Surface Aggregate Projecting @ 0.0 ft. to 14.7 ft.	Fair	If relief sewer is constructed, cured in place (CIPP) liner could be installed. (Section 4.8.2)	Severe	Clean and install CIPP Liner	6.91	4.44	6.48	6.43	8.33	4.62	4.71	6.6	6.73	7.04	7.13	8.16	8.29
MH1023* - MH5089	5090L	MH30 - MH31	369	Not Included in Report but Video provided. Location continues 24" line to next MH		30" RCP	hole in patched section of Pipe @7.3 ft. Surface Aggregate Projecting & Deposits Attached: Raging@ 5.0 ft. to 368.5 ft. Roots Medium: Side of Pipe @77 ft. &161 ft.	Not Applicable		Severe	Clean, Point Repair and install CIPP Liner	15.98	4.71	6.88	7.29	9.45	No Data Available							
MH5089 - MH5087	L953	MH31 - MH32	5	Not Included in Report but Video		42" RCP	Surface Aggregate Projecting @ 5 ft.	Not Applicable		Poor	Clean and install CIPP Liner	21.74	4.71	6.88	7.29	9.45								
MH5087 - MH1005*	L951	MH32 - MH33	319	Not Included in Report but Video		42" RCP	Surface Aggregate Projecting @ 5 ft. to 319 ft.	Not Applicable		Poor	Clean and install CIPP Liner	21.74	4.71	6.88	7.29	9.45								
MH1005* - MH1003*	L949	MH33 - MH34	403	Not Included in Report but Video		42" RCP	Surface Aggregate Projecting @ 5 ft. to 403 ft.	Not Applicable		Poor	Clean and install CIPP Liner	21.74	4.71	6.88	7.29	9.45								
MH5133 - MH1023*	L1048	MH30A - MH30	96	Not Included in Report but Video		15" VCP	none	Not Applicable		Good	none	No Data Available												

For the condition assessment highlighted areas denote differences in the BCF and MWH assessments. For the capacity assessment highlighted areas denote capacity deficiency

4.2 Across Fresno River

This reach is dual 12" cast iron (CI) pipes and could not be inspected by CCTV; therefore the condition of this reach is unknown. It is MWH's recommendation that that these dual 12 inch cast iron pipes be televised.

4.3 From the Fresno River to Fourth Street

This reach is 24" VCP. The pipeline is generally in good condition. There were three minor longitudinal cracks found: one at 109-ft between MH16 (MH3306) and MH17 (MH3392) and two at 503-ft between MH18 (MH1138*) and MH19 (MH3304). Because the degree of the cracks was considered minor, MWH recommends these segments be monitored. MWH also recommends that reach be monitored for settled and attached deposits from MH13 (MH3365) to MH16 (MH3306) and for roots at MH9 (MH2044). From MH10 (MH2042) to MH11 (MH3394) the settled deposits were designated as medium therefore, MWH recommends cleaning this segment.

Two 12" concrete segments were included in the CCTV of this reach, MH4235 to MH4237 and MH4237 to MH4331. These segments were found to have projecting surface aggregate and in severe condition. MWH recommends that the lines be cleaned and CIPP lined.

4.4 From Fourth Street to Howard Road (continuing to Industrial Avenue)

This reach is 24" concrete up to Howard Rd, it increases to 30" for one segment and continues and 42" in the following segments to Industrial Ave. The pipeline from MH24 (MH4331) to MH31 (MH5089) was found to have projecting surface aggregate and in severe condition. In a segment from MH27 (MH1025*) to MH28 (MH1024*) aggregate was missing from 5ft to 18.3-ft and a hole was found in a patch of pipe at 7.3ft of the segment for MH30 (MH1023*) to MH31 (MH5089). The pipeline from MH31 (MH5089) to MH34 (MH1003*) was found to have projecting surface aggregate and in poor condition. MWH recommends that the lines be cleaned and CIPP lined.

One segment of 15" VCP was included in the CCTV Inspection from MH5133 to MH1023*. This segment of pipe was found to be in good condition; therefore, no further action is recommended.

5 Capacity Assessment

This TM updates the recommendations based upon our review of the 2008 Blair Church and Flynn (BCF) report and more recent flow information provided by Akel Engineering. MWH has assessed three flow scenarios for the Schnoor Ave Trunk Sewer which are presented on the right hand side of Table 2. The two scenarios found in the BCF evaluations were based on county flow data and rainfall data acquired in 2007 and included the following:

- Existing PS1 (AKEL 2013) - Existing and future peak dry weather flows (PDWF) and peak wet weather flows (PWWF) from Akel Engineering based upon modeling completed in 2013 for the existing and improved PS1.
- Existing PS1 (BCF 2007) - Existing and future peak dry weather flows (PDWF) and peak wet weather flows (PWWF) for the existing PS1
- Improved PS1 (BCF Future Project) - Existing and future PDWF and PWWF for an improved Pump Station Number 1 (PS1) as described in the PS1 Report

Based on the analysis of both the Akel Engineering modeling data and BCF evaluation, the pipeline has adequate capacity from PS1 to MH30 (MH 4331) for existing and future PDWF and existing PWWF.

According to the BCF evaluation the pipelines from PS1 to MH30 (MH1023*) do not have capacity for existing or future PDWF or PWWF under the improved PS1 scenario. If PS1 is improved as described in the BCF evaluation, this may present capacity deficiencies which would require replacing the existing pipeline with large pipes or providing relief sewers in addition to rehabilitating the portions of existing lines as recommended above. However the Akel modeling data shows a minor capacity deficiency for the future PWWF only, which can be accommodated with some surcharging. Based on this assessment MWH recommends the pipelines not be replaced but rehabilitated as recommended above to extend the useful life of the system.

6 Cost

MWH is recommending CIPP for rehabilitation of the concrete pipe. A Parametric Class 5 Cost Estimate was prepared for the recommended rehabilitation described above. Shown on Table 3 are the estimated costs for each segment of the project. The estimate calculates a total cost of $604K including costs for mobilization, demobilization, bypass pumping, traffic control and contingencies. Shown in Appendix A is the cost detail breakdown for each element of the project.

Table 3. Cost Estimate

(MH ID)	LN_ID	Length (ft.)	Diameter (in)	Description	Cost
Mob. and De-Mob.					$30,867
MH2121 - MH2050	2035L	505	21	Remove Roots	$8,995
MH2042 - Mh3394	3395L	145	24	Clean	$5,320
MH4235 - MH4237	4238L	419	12	Install CIPP Liner	$25,586
MH4237 - MH4331	L4242	201	12	Install CIPP Liner	$15,388
MH4331 - MH1032*	L1054	34	24	Install CIPP Liner	$8,816
MH1032* - MH1031*	L1053	293	24	Install CIPP Liner	$31,474
MH1031* - MH1025*	L1052	277	24	Install CIPP Liner	$30,127
MH1025* - MH1024*	L1051	42	24	Install CIPP Liner	$9,489
MH1024* - MH1021*	L1050	58	24	Install CIPP Liner	$10,836
MH1021* - MH1023*	L1049	15	24	Install CIPP Liner	$7,216
MH1023* - MH5089	5090L	369	30	Install CIPP Liner	$67,628
MH5089 - MH5087	L953	5	42	Install CIPP Liner	$7,098
MH5087 - MH1005*	L951	319	42	Install CIPP Liner	$79,632
MH1005* - MH1003*	L949	403	42	Install CIPP Liner	$99,078
Subtotal					$437,511
Estimating Contingency					$45,179
Scope Contingency					$120,673
Total					$603,363

Appendix A

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Sub Amount	Equip Amount	Other Amount	Total Cost/Unit	Total Amount
001 Schnoor Ave Trunk Sewer Replacement/Rehabilitation								
001 Mobilization and Demobilization								
001 Mobilization/Demobilization/General Conditions	1.00 ls	21,685	1,136		7,672	375	30,867.45 /ls	30,867
001 Mobilization and Demobilization	**1.00 ls**	**21,685**	**1,136**		**7,672**	**375**	**30,867.45 /ls**	**30,867**
004 MH2121 - MH2050, 21" VCP								
101 Bypass Pumping	505.00 ls	530	58		263		1.69 /ls	851
102 Bypass Pumping Setup/Teardown	505.00 ls	1,855	97		452		4.76 /ls	2,404
103 Normal Cleaning of Sewer	505.00 lf			1,529			3.03 /lf	1,529
104 Video Inspection of Sewer	505.00 lf			2,447			4.85 /lf	2,447
105 Mechanical Cleaning of Sewer	110.00 lf			800			7.27 /lf	800
111 Traffic Control Maintenance	505.00 lf	199		636			1.65 /lf	835
112 Traffic Control Labor	505.00 lf	88					0.18 /lf	88
004 MH2121 - MH2050, 21" VCP	**505.00 lf**	**2,672**	**155**	**5,412**	**715**		**17.73 /lf**	**8,955**
011 MH2042 - MH3394, 24" VCP								
101 Bypass Pumping	145.00 ls	530	58		263		5.87 /ls	851
102 Bypass Pumping Setup/Teardown	145.00 ls	1,855	97		452		16.58 /ls	2,404
103 Normal Cleaning of Sewer	145.00 lf			439			3.03 /lf	439
104 Video Inspection of Sewer	145.00 lf			703			4.85 /lf	703
111 Traffic Control Maintenance	145.00 lf	199		636			5.76 /lf	835
112 Traffic Control Labor	145.00 lf	88					0.61 /lf	88
011 MH2042 - MH3394, 24" VCP	**145.00 lf**	**2,672**	**155**	**1,778**	**715**		**36.69 /lf**	**5,320**
025 MH4235 - MH4237, 12" RCP								
101 Bypass Pumping	419.00 ls	1,590	174		790		6.10 /ls	2,554
102 Bypass Pumping Setup/Teardown	419.00 ls	1,855	97		452		5.74 /ls	2,404
103 Normal Cleaning of Sewer	419.00 lf			1,015			2.42 /lf	1,015
104 Video Inspection of Sewer	419.00 lf			2,030			4.85 /lf	2,030
106 CIPP Liner	419.00 lf			15,736			37.56 /lf	15,736
111 Traffic Control Maintenance	419.00 lf	398		1,272			3.99 /lf	1,670
112 Traffic Control Labor	419.00 lf	177					0.42 /lf	177
025 MH4235 - MH4237, 12" RCP	**419.00 lf**	**4,020**	**270**	**20,053**	**1,242**		**61.06 /lf**	**25,586**
026 MH4237 - MH4331, 12" RCP								
101 Bypass Pumping	201.00 ls	1,325	145		659		10.59 /ls	2,128
102 Bypass Pumping Setup/Teardown	201.00 ls	1,855	97		452		11.96 /ls	2,404
103 Normal Cleaning of Sewer	201.00 lf			487			2.42 /lf	487
104 Video Inspection of Sewer	201.00 lf			974			4.85 /lf	974
106 CIPP Liner	201.00 lf			7,549			37.56 /lf	7,549
111 Traffic Control Maintenance	201.00 lf	398		1,272			8.31 /lf	1,670
112 Traffic Control Labor	201.00 lf	177					0.88 /lf	177
026 MH4237 - MH4331, 12" RCP	**201.00 lf**	**3,754**	**241**	**10,282**	**1,111**		**76.56 /lf**	**15,388**

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Sub Amount	Equip Amount	Other Amount	Total Cost/Unit	Total Amount
027 MH4331 - MH1032, 24" RCP								
101 Bypass Pumping	34.00 ls	1,060	116		527		50.08 /ls	1,703
102 Bypass Pumping Setup/Teardown	34.00 ls	1,855	97		452		70.71 /ls	2,404
103 Normal Cleaning of Sewer	34.00 lf			103			3.03 /lf	103
104 Video Inspection of Sewer	34.00 lf			165			4.85 /lf	165
106 CIPP Liner	34.00 lf			2,595			76.32 /lf	2,595
111 Traffic Control Maintenance	34.00 lf	398		1,272			49.11 /lf	1,670
112 Traffic Control Labor	34.00 lf	177					5.20 /lf	177
027 MH4331 - MH1032, 24" RCP	**34.00 lf**	**3,489**	**213**	**4,135**	**979**		**259.29 /lf**	**8,816**
028 MH1032 - MH1031, 24" RCP								
101 Bypass Pumping	293.00 ls	1,590	174		790		8.72 /ls	2,554
102 Bypass Pumping Setup/Teardown	293.00 ls	1,855	97		452		8.21 /ls	2,404
103 Normal Cleaning of Sewer	293.00 lf			887			3.03 /lf	887
104 Video Inspection of Sewer	293.00 lf			1,420			4.85 /lf	1,420
106 CIPP Liner	293.00 lf			22,362			76.32 /lf	22,362
111 Traffic Control Maintenance	293.00 lf	398		1,272			5.70 /lf	1,670
112 Traffic Control Labor	293.00 lf	177					0.60 /lf	177
028 MH1032 - MH1031, 24" RCP	**293.00 lf**	**4,019**	**270**	**25,942**	**1,242**		**107.42 /lf**	**31,474**
029 MH1031 - MH1025, 24" RCP								
101 Bypass Pumping	277.00 ls	1,590	174		790		9.22 /ls	2,554
102 Bypass Pumping Setup/Teardown	277.00 ls	1,855	97		452		8.68 /ls	2,404
103 Normal Cleaning of Sewer	277.00 lf			839			3.03 /lf	839
104 Video Inspection of Sewer	277.00 lf			1,342			4.85 /lf	1,342
106 CIPP Liner	277.00 lf			21,141			76.32 /lf	21,141
111 Traffic Control Maintenance	277.00 lf	398		1,272			6.03 /lf	1,670
112 Traffic Control Labor	277.00 lf	177					0.64 /lf	177
029 MH1031 - MH1025, 24" RCP	**277.00 lf**	**4,019**	**270**	**24,595**	**1,242**		**108.76 /lf**	**30,127**
030 MH1025 - MH1024, 24" RCP								
101 Bypass Pumping	42.00 ls	1,060	116		527		40.54 /ls	1,703
102 Bypass Pumping Setup/Teardown	42.00 ls	1,855	97		452		57.24 /ls	2,404
103 Normal Cleaning of Sewer	42.00 lf			127			3.03 /lf	127
104 Video Inspection of Sewer	42.00 lf			204			4.85 /lf	204
106 CIPP Liner	42.00 lf			3,206			76.32 /lf	3,206
111 Traffic Control Maintenance	42.00 lf	398		1,272			39.75 /lf	1,670
112 Traffic Control Labor	42.00 lf	177					4.21 /lf	177
030 MH1025 - MH1024, 24" RCP	**42.00 lf**	**3,489**	**213**	**4,808**	**979**		**225.94 /lf**	**9,489**
031 MH1024 - MH1021, 24" RCP								
101 Bypass Pumping	58.00 ls	1,060	116		527		29.36 /ls	1,703
102 Bypass Pumping Setup/Teardown	58.00 ls	1,855	97		452		41.45 /ls	2,404

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Sub Amount	Equip Amount	Other Amount	Total Cost/Unit	Total Amount
103 Normal Cleaning of Sewer	58.00 lf			176			3.03 /lf	176
104 Video Inspection of Sewer	58.00 lf			281			4.85 /lf	281
106 CIPP Liner	58.00 lf			4,427			76.32 /lf	4,427
111 Traffic Control Maintenance	58.00 lf	398		1,272			28.79 /lf	1,670
112 Traffic Control Labor	58.00 lf	177					3.05 /lf	177
031 MH1024 - MH1021, 24" RCP	**58.00 lf**	**3,489**	**213**	**6,155**	**979**		**186.84 /lf**	**10,836**
032 MH1021 - MH1023, 24" RCP								
101 Bypass Pumping	15.00 ls	1,060	116		527		113.52 /ls	1,703
102 Bypass Pumping Setup/Teardown	15.00 ls	1,855	97		452		160.27 /ls	2,404
103 Normal Cleaning of Sewer	15.00 lf			45			3.03 /lf	45
104 Video Inspection of Sewer	15.00 lf			73			4.85 /lf	73
106 CIPP Liner	15.00 lf			1,145			76.32 /lf	1,145
111 Traffic Control Maintenance	15.00 lf	398		1,272			111.31 /lf	1,670
112 Traffic Control Labor	15.00 lf	177					11.78 /lf	177
032 MH1021 - MH1023, 24" RCP	**15.00 lf**	**3,489**	**213**	**2,535**	**979**		**481.06 /lf**	**7,216**
033 MH1023 - MH5089, 30" RCP								
101 Bypass Pumping	369.00 ls	1,458	159		725		6.35 /ls	2,341
102 Bypass Pumping Setup/Teardown	369.00 ls	1,855	97		452		6.52 /ls	2,404
103 Normal Cleaning of Sewer	369.00 lf			1,788			4.85 /lf	1,788
104 Video Inspection of Sewer	369.00 lf			1,788			4.85 /lf	1,788
106 CIPP Liner	369.00 lf			42,468			115.09 /lf	42,468
107 Point Repair	1.00 ea			14,992			14,991.91 /ea	14,992
111 Traffic Control Maintenance	369.00 lf	398		1,272			4.53 /lf	1,670
112 Traffic Control Labor	369.00 lf	177					0.48 /lf	177
033 MH1023 - MH5089, 30" RCP	**369.00 lf**	**3,887**	**256**	**62,308**	**1,177**		**183.27 /lf**	**67,628**
034 MH5089 - MH5087, 42" RCP								
101 Bypass Pumping	5.00 ls	1,060	116		527		340.54 /ls	1,703
102 Bypass Pumping Setup/Teardown	5.00 ls	1,855	97		452		480.80 /ls	2,404
103 Normal Cleaning of Sewer	5.00 lf			61			12.11 /lf	61
104 Video Inspection of Sewer	5.00 lf			24			4.84 /lf	24
106 CIPP Liner	5.00 lf			1,060			212.01 /lf	1,060
111 Traffic Control Maintenance	5.00 lf	398		1,272			333.91 /lf	1,670
112 Traffic Control Labor	5.00 lf	177					35.33 /lf	177
034 MH5089 - MH5087, 42" RCP	**5.00 lf**	**3,489**	**213**	**2,417**	**979**		**1,419.56 /lf**	**7,098**
035 MH5087 - MH1005, 42" RCP								
101 Bypass Pumping	319.00 ls	1,458	159		725		7.34 /ls	2,341
102 Bypass Pumping Setup/Teardown	319.00 ls	1,855	97		452		7.54 /ls	2,404
103 Normal Cleaning of Sewer	319.00 lf			3,865			12.12 /lf	3,865
104 Video Inspection of Sewer	319.00 lf			1,546			4.85 /lf	1,546
106 CIPP Liner	319.00 lf			67,630			212.01 /lf	67,630
111 Traffic Control Maintenance	319.00 lf	398		1,272			5.23 /lf	1,670

Spreadsheet Level	Takeoff Quantity	Labor Amount	Material Amount	Sub Amount	Equip Amount	Other Amount	Total Cost/Unit	Total Amount
112 Traffic Control Labor	319.00 lf	177					0.55 /lf	177
035 MH5087 - MH1005, 42" RCP	**319.00 lf**	**3,887**	**256**	**74,313**	**1,177**		**249.63 /lf**	**79,632**
036 MH1005 - MH1003, 42" RCP								
101 Bypass Pumping	403.00 ls	1,590	174		790		6.34 /ls	2,554
102 Bypass Pumping Setup/Teardown	403.00 ls	1,855	97		452		5.97 /ls	2,404
103 Normal Cleaning of Sewer	403.00 lf			4,882			12.12 /lf	4,882
104 Video Inspection of Sewer	403.00 lf			1,953			4.85 /lf	1,953
106 CIPP Liner	403.00 lf			85,439			212.01 /lf	85,439
111 Traffic Control Maintenance	403.00 lf	398		1,272			4.14 /lf	1,670
112 Traffic Control Labor	403.00 lf	177					0.44 /lf	177
036 MH1005 - MH1003, 42" RCP	**403.00 lf**	**4,020**	**270**	**93,546**	**1,242**		**245.85 /lf**	**99,078**
001 Schnoor Ave Trunk Sewer Replacement/Rehabilitation	**1.00 ls**	**72,083**	**4,344**	**338,279**	**22,430**	**375**	**437,510.71 /ls**	**437,511**

Estimate Totals

Description	Amount	Totals	Percent of Total
Labor	72,083		11.95%
Material	4,344		0.72%
Subcontract	338,279		56.07%
Equipment	22,430		3.72%
Other	375		0.06%
Subtotal	**437,511**	**437,511**	**72.51**
Estimating Contingency - CL4	45,179		7.49%
Subtotal	**45,179**	**482,690**	**7.49**
Scope Contingency - CL4	120,673		20.00%
Escalation			
Sutotal	**120,673**	**603,363**	**20.00**
Total		**603,363**	

APPENDIX E

Madera Wastewater Treatment Plant Evaluation (MWH)

CITY OF MADERA

SANITARY SEWER MASTER PLAN

WASTEWATER TREATMENT PLANT EVALUATION

Final

October 2013

MWH

AKEL ENGINEERING GROUP, INC.

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Attachment

Attachment 1: Waste Discharge Requirements for the City of Madera

List of Abbreviations and Acronyms

ADF	average daily flow
Akel	Akel Engineering, Inc.
ASCE	American Society of Civil Engineers
BOD	biochemical oxygen demand
CEPT	chemically enhanced primary treatment
City	City of Madera
d	day
Dia.	diameter
EPA	Environmental Protection Agency
ft/yr	feet/year
F:M	food:microorganism ratio
gpm	gallons per minute
GBT	gravity belt thickener
HVAC	heating, ventilation, and air conditioning
I&I	infiltration and inflow
lb	pounds
mg/L	milligrams per liter
MLSS	mixed liquor suspended solids
MM	maximum month
MWH	MWH Americas, Inc.
PHF	peak hour flow
PS	pump station
RAS	Return Activated Sludge
RWQCB	Regional Water Quality Control Board
SRT	solids retention time
sf	square feet
TSS	total suspended solids
WAS	Waste Activated Sludge
WEF	Water Environment Federation
WDR	Waste Discharge Requirements
WWTP	wastewater treatment plant

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1 Introduction

A previous master plan for the City of Madera's (City's) Wastewater Treatment Plant (WWTP) was completed in 1988 and updated in 1994. Since 1994, the WWTP has been expanded and upgraded with improvements designed to provide a treatment capacity for an average daily flow (ADF) of 10.1 million gallon per day (mgd).

New population growth projections have prompted the need for this update of the City's Sanitary Sewer Master Plan. The City executed an agreement dated January 5, 2011 with the consulting team of Akel Engineering, Inc. (Akel) and MWH Americas, Inc. (MWH) to evaluate the capacity of the WWTP and to evaluate means to accommodate projected flows and loadings.

The purpose of the WWTP Evaluation is to provide the City with a tool for planning WWTP improvements and expanded capacity. The recommended WWTP improvements developed and described herein as follows:

- **Near-term:** Evaluate effective capacity of existing treatment facilities and recommend improvements, where needed, to provide secondary treatment for an ADF of 10.1 mgd. This capacity was selected to match the existing oxidation ditch treatment capacity and existing evaporation/percolation pond capacity. Note that if influent flow increases from the current ADF of 5.6 mgd to 10.1 mgd at the rate of 3.5% per year to 7.5% per year, this would provide an 8 to 17 year period for planning, design and construction of future facilities needed for flows greater than 10.1 mgd.

- **Urban growth boundary:** Evaluate reconnaissance-level collection system and treatment plant improvements to convey and treat urban growth boundary flow (ADF = 26.5 mgd). Develop baseline alternative to meet this objective at the existing WWTP location. Develop reconnaissance-level alternatives for comparison to the baseline alternative that utilize one or more additional treatment plant location(s) and associated collection system improvements to convey flow to each treatment plant location.

2 Wastewater Treatment Plant Description

The existing WWTP consists of a headworks with two mechanical bar screens, an influent lift station, and two grit chambers; three rectangular primary clarifiers and primary effluent pump station; and biological treatment with three oxidation ditches, four circular secondary clarifiers, and a Return Activated Sludge (RAS)/ Waste Activated Sludge (WAS) splitter. The effluent disposal facilities consist of the equivalent of 16 evaporation/percolation ponds at 20-acres each[1]. Solids handling includes three anaerobic sludge digesters (two primary and one secondary) and two sludge dewatering centrifuges.

The original treatment plant and disposal facilities were constructed in 1972. The plant was expanded in 1990 with the addition of a third primary clarifier and then upgraded in 2007 with the installation of three oxidation ditches and four secondary clarifiers, which replaced the original trickling filters. The influent mechanical screens at the headworks were replaced in 2011. The current plant layout is shown in Figure 1. The current process flow diagram for the plant is shown on Figure 2.

[1] The existing evaporation/percolation ponds actually consist of 14 20-acre ponds and 1 40-acre pond, or the equivalent of 16 20-acre ponds, and are discussed herein as 16 20-acre ponds for simplicity.

Figure 1
Madera WWTP
Existing Layout
Sanitary Sewer
Master Plan
City of Madera

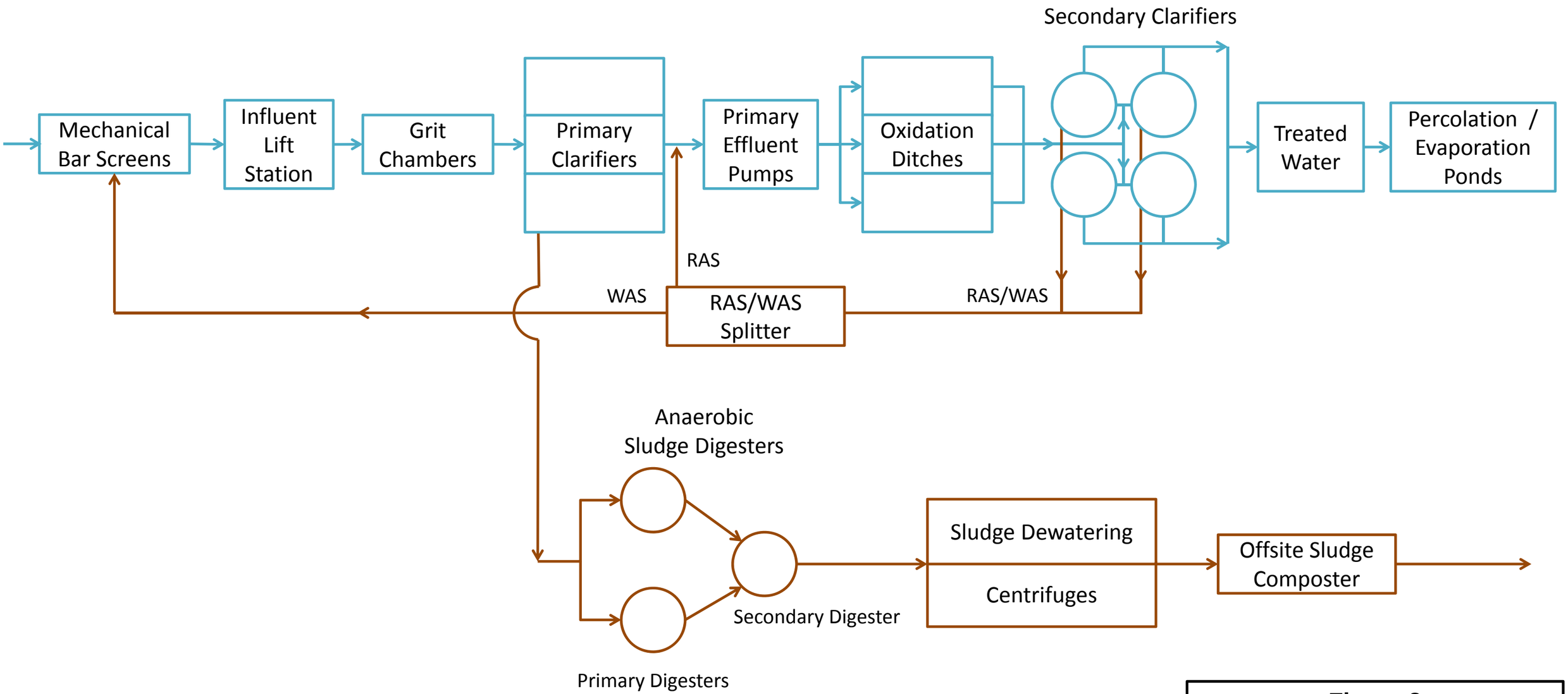


Figure 2
Madera WWTP
Existing Process Flow Diagram
Sanitary Sewer
Master Plan
City of Madera

3 Waste Discharge Requirements

The City of Madera WWTP operates fall under Waste Discharge Requirements (WDRs) Order No. 95-046 of the California Regional Water Quality Control Board (RWQCB), Central Valley Region, which was adopted in 1989. The treated effluent from the existing WWTP is discharged to existing evaporation/percolation ponds. The WDRs prohibit the Madera WWTP from discharging wastes to surface waters or surface water drainage courses; from bypassing or overflowing untreated or partially treated waste; and from discharging waste or sludge classified as 'hazardous.' The discharge specifications for the plant stipulate that the monthly average daily dry weather (average dry flow, ADF) discharge not exceed 7 million gallons per day (mgd). Plant effluent limits are presented in Table 1.

Table 1: City of Madera WWTP Effluent Limits

Water Quality Parameter[1]	Monthly Average	Daily Maximum
Biochemical Oxygen Demand (mg/L)	40	80
Total Suspended Solids (mg/L)	40	80
Settleable Solids (mg/L)	0.2	0.5

1. mg/L = milligrams per liter

No bacterial requirements for the treated effluent are listed in the permit.

Prior to the 2007 plant expansion, the City was experiencing permit violations. However, since the upgrade, there have been no discharge violations.

The current WDR (Order No. 95-046) appears to be based on the trickling filter technology in operation before the 2007 plant upgrade and permits the design capacity of the WWTP at 7 mgd. The current ADF is 5.6 mgd. Note that the oxidation ditch technology installed with the recent upgrade will provide significantly better effluent quality than needed for the current discharge limitations. For the purposes of this WWTP Evaluation, we have assumed that the level of treatment achieved with the oxidation ditch technology will be maintained.

We understand that the City has initiated a renewal of the WDR over a year ago, but the Regional Board has been unable to take any action to date on its execution. It is recommended that the City continue to request that the RWQCB update the WDR in advance of reaching 7 mgd.

4 Near-Term Improvements

4.1 Flow and Loadings

An evaluation of recent flow and loading parameters was conducted to develop per capita flow and loading values. The per capita loading values were used to predict the anticipated biochemical oxygen demand (BOD) and total suspended solids (TSS) loading for the design capacity of an ADF of 10.1 mgd.

A summary of the average annual flow, BOD loading, and TSS loading for the past three-year period of 2009 to 2011 is presented in Table 2. The reported flow data is based on daily flow measurements. The reported BOD and TSS loadings are calculated from concentrations (mg/L) analyzed from 24-hour samples taken approximately once per week and the reported flow for the day the sample was taken. The annual service

populations for this three-year period are also included. A "basis of design" value was established for each flow and loading parameter from this three-year period as follows:

- Average daily flow (ADF) is the mean value of the reported daily flows.
- Peak hour flow (PHF) is the highest reported daily flow multiplied by an assumed diurnal peaking factor of 1.5.
- The maximum month (MM) BOD loading is the highest 4-week running average value.
- The maximum month (MM) TSS loading is the highest 4-week running average value.
- The population value is the mean value of the annual populations.

Table 2: Historical Flow, BOD, and TSS

Parameter	Unit	Y2009	Y2010	Y2011	Basis of Design
ADF	mgd	5.72	5.68	5.58	5.66
PDF	mgd	6.62	6.57	6.99	6.99
AA BOD	lb/d	10,605	9,549	9,462	9,900
AA TSS	lb/d	10,270	8,505	8,211	9,000
MM BOD	lb/d	---	---	---	12,700
MM TSS	lb/d	---	---	---	12,100
Population	capita	59,900	61,400	62,700	61,300

Utilizing the Basis of Design parameters presented in Table 2, the PHF was determined by using a 1.5 multiplier to estimate diurnal PHF applied to the PDF value. The "per capita" loading values were calculated as summarized in Table 3.

Table 3: Per Capita Parameters

Parameter	Unit	Basis of Design
ADF	mgd	5.66
PDF	mgd	6.99
PHF[1]	mgd	10.5
PHF/ADF	ratio	1.86
AA BOD	lb/d	9,900
AA TSS	lb/d	9,000
MM BOD	lb/d	12,700
MM TSS	lb/d	12,100
Population	capita	61,300
AA Unit BOD	lb BOD/cap/d	0.16
AA Unit TSS	lb TSS/cap/d	0.15
MM Unit BOD	lb BOD/cap/d	0.21
MM Unit TSS	lb TSS/cap/d	0.20

1. The PHF/PDF ratio is assumed to be 1.5

The "per capita" loading parameters from Table 3 are applied to the future ADF of 10.1 mgd, which corresponds to a projected service population of approximately 85,000 based on the population and flow projections developed in Chapter 5. The projected population of 85,000 was used to develop the design criteria for design loadings presented in Table 4. Note that the BOD and TSS design loading parameters are based on the highest 4-week running average to represent the maximum month condition. The 4-week running average is considered more conservative than taking the highest average monthly loading from the historical record although it is possible the two periods can coincide. The maximum month condition is utilized because compliance with discharge permit conditions is driven by monthly averages and removals.

Table 4: Design Criteria for Flow, BOD, and TSS

Parameter	Per Capita Values (2009 - 2011)		Design Criteria (ADF = 10.1 mgd)	
	Unit	Basis of Design	Unit	Value
Population	capita	61,300	capita	85,000
ADF	---	---	mgd	10.1
PHF	PHF/ADF	1.86	mgd	19
AA BOD	lb BOD/cap/d	0.16	lb/d	13,700
AA TSS	lb TSS/cap/d	0.15	lb/d	12,500
MM BOD	lb BOD/cap/d	0.21	lb/d	17,600
MM TSS	lb TSS/cap/d	0.20	lb/d	16,800

4.2 Evaluation of Treatment Capacities

Each wastewater treatment process was evaluated to determine its effective capacity to meet the design wastewater flows and loadings. Descriptions and evaluations of each individual process are provided in this section. The evaluation of WWTP capacity is guided by comparison of existing facilities with design criteria and guidelines commonly used in the wastewater industry and identified herein as "Industry Practice". The design criteria and guidelines cited as Industry Practice are based on MWH experience with related facilities and the following technical references:

- *Design of Municipal Wastewater Treatment Plants, 4th Edition*, Water Environment Federation (WEF) and American Society of Civil Engineers (ASCE), 1998.
- *Wastewater Treatment Plant Design*, Edited by P. Aarne Vesilind, WEF, 2003.
- *Wastewater Engineering Treatment and Reuse, 4th Edition*, Metcalf and Eddy, 2003.
- *MWH Best Practice Design Guides.*

4.3 Mechanical Bar Screens

Raw sewage from the wastewater collection system reaches the WWTP by gravity sewer and passes through two below-grade mechanical bar screens before entering the influent lift station. The previous mechanical

bar screens were ineffective in removing material, such as rags, that have historically caused operational and maintenance issues. The bar screens have recently been replaced with two front-cleaning, front-return, link-driven mechanically cleaned bar screens (Duperon FlexRake® Model Full Penetration Fine Screens). The specified capacity of each screen is 14 million gallons per day (mgd) and matches the firm pumping capacity of the influent lift station as described below (assumes one duty screen and one standby screen). Additional screening capacity will be needed to meet the future design condition PHF of 19 mgd.

Additional mechanical bar screen capacity can be achieved by adding another channel with equivalent hydraulic capacity of 14 mgd. This will provide a firm capacity of 28 mgd with one unit out of service. Although 28 mgd exceeds the design capacity of 19 mgd, the incremental cost for the extra capacity is minor and it is of greater value to have equal sized units for ease of operation and maintenance. Another alternative would be to provide a bypass channel with a manual bar screen, but this approach requires operator attendance to manually clean the bar screen. A manual bar screen is not recommended because the plant is not staffed on a full time basis and would risk surcharging the collection system.

4.4 Influent Lift Station

The influent lift station consists of three pumps, each with a capacity of 5,000 gallons per minute (gpm) or 7.2 mgd. The pumps appear to be in good condition. At current influent flows, the WWTP runs one pump an average of 12 hours a day. The firm pumping capacity (two duty units and one standby unit) is 14.4 mgd. Additional pump capacity will be required to meet the future design condition PHF of 19 mgd with firm pumping capacity.

One potential option to increase the influent lift station capacity is replacement of the three existing pumps with three larger pumps (6,600 gpm each) to provide firm influent lift station capacity of 19 mgd. Another option is the construction of a new influent lift station with a 5,000 gpm pump (to match existing pump size) that would operate in parallel with the existing influent lift station. The combination of the two lift stations would provide a firm pumping capacity of 22 mgd. Although 22 mgd exceeds the design capacity of 19 mgd, the incremental cost for the extra capacity is minor and the installation of equal sized pumps provides greater value due to interchangeability and commonality of spare parts.

4.5 Grit Chambers

The headworks includes two mechanical vortex grit chambers, each with a rated capacity of 15 mgd. The operations staff report that the grit chambers have historically operated effectively, with one exception. The exception occurred when a large storm coincided with significant construction at the plant, resulting in ineffective operation. This is considered a one-time event under special circumstances.

The grit chambers are equipped with a bypass channel and can be operated in complete bypass with both units out of service, with one unit in service, or both units in service. The grit chambers are generally reliable and operation of both grit chambers in parallel provides a PHF capacity of 30 mgd, which meets the future design conditions. If one unit is out of service, the duty chamber can operate at a higher flow capacity with reduction of grit removal efficiency. This condition can typically be tolerated and is reasonable practice for short durations. However, the installation of a third grit chamber could be considered by the City for redundancy in the future before peak hour flow reaches 15 mgd (ADF = 8.0 mgd). This consideration is recommended to be coupled with the companion review of the mechanical bar screens and influent lift station improvements.

4.6 Primary Clarifiers

Two of the primary clarifiers were installed during the original construction in 1972, while the third was added during the 1990 plant expansion. Each unit is 36-feet wide by 100-feet long with a side water depth of 7.5-feet. The effective capacity of the existing primary clarifiers based on surface loading rates is presented below in Table 5 for the existing process condition (WAS co-settling), and potential upgraded process conditions such as separate WAS settling and chemical enhancement.

Table 5: Primary Clarifier Effective Capacity

Design Condition	Primary Treatment with WAS Co-Settling[1]		Primary Treatment without WAS Co-Settling[2]		Chemically Enhanced Primary Treatment	
	Industry Practice	Capacity of Existing Units (without standby)	Industry Practice	Capacity of Existing Units (without standby)	Industry Practice	Capacity of Existing Units (without standby)
ADF	600 – 800 gpd/sf	6.5 – 8.6 mgd	800 – 1,200 gpd/sf	8.6 – 13 mgd	---	---
PHF	1,200 – 1,700 gpd/sf	13 – 18 mgd	2,000 – 3,000 gpd/sf	22 – 32 mgd	4000 gpd/sf	43 mgd

1. Assumptions: 25% BOD removal, 50% TSS removal, and sludge at 3.5% dry solids.
2. Assumptions: 30% BOD removal, 65% TSS removal, and sludge at 6.0% dry solids.

If the existing primary clarifiers continue to operate with WAS co-settling, additional primary clarifier capacity would be needed to meet the future design conditions. Options to achieve additional primary clarifier capacity include the following:

- Addition of 1 new primary clarifier of similar size to existing units (36 feet wide, 100 feet long, and 7.5 feet deep).
- Addition of dedicated WAS sludge thickening process, allowing the primary clarifiers to operate without co-settling.
- Addition of a chemical feed system such as ferric or polymer to provide enhanced coagulation and settling.

A matrix of inter-related solids handling alternatives will be evaluated later in this section that address the addition of a new primary clarifier and/or the addition of a dedicated WAS thickening or dewatering process. Note the significant increase in effective capacity of the existing primary clarifiers is achieved if dedicated WAS thickening or dewatering is employed and co-settling of primary sludge and WAS is eliminated. Dedicated WAS thickening or dewatering offsets the need for additional primary clarifier capacity to meet future design conditions.

The possible addition of a chemical feed system to provide chemically enhanced primary treatment (CEPT) was considered as a potential option as mentioned above. However, after further review, this option was dropped. CEPT is generally considered for treatment of wet weather flows in excess of dry weather flows for wastewater collection systems that have high levels of infiltration and inflow (I&I). In high I&I scenarios, the CEPT provides an economical way to achieve performance for short durations at higher surface overflow rates (with dilute wastewater) and can be automatically activated when a wet weather event occurs and deactivated when the storm event subsides. The Madera wastewater collection system does not encounter

significant I&I contributions during wet weather, so application of CEPT would imply continuous operation. CEPT would not be advisable for routine operation because of annual chemical costs and additional labor requirements.

4.7 Primary Effluent Pump Station

After passing through the primary clarifiers, primary effluent is pumped by the primary effluent pump station into the oxidation ditches. In addition, the RAS from the activated sludge system flows by gravity from the secondary clarifiers to the primary effluent pump station and is returned to the oxidation ditches in combination with the primary effluent. The primary effluent pump station consists of 3 vertical pumps, each rated for 8,700 gpm to provide a firm primary effluent pumping capacity of 25 mgd (PHF).

The RAS is routed to the wet well of the Primary Effluent Pump Station where it is combined with the primary effluent. The combined flow is then delivered by the primary effluent pumps to the inlet chamber of a distribution box equipped with three downward-opening slide gates that discharge to individual outlet chambers that in turn are connected via yard piping to their respective oxidation ditches.

The current firm pumping capacity of the influent lift station is 14.4 mgd (PHF), and as previously discussed, this corresponds to an ADF of 7.7 mgd. The difference between the capacities of the two stations would represent the available hydraulic capacity for RAS and would be (25 mgd - 14.4 mgd =) 10.6 mgd. The RAS/ADF ratio is then (10.6 mgd / 7.7 mgd =) 139%. The industry practice for oxidation ditch RAS capacity ranges from 75% to 150% of ADF. So for an ADF of 7.7 mgd, equivalent to the influent pumping capacity, the available RAS capacity at the primary effluent pump station appears to be adequate.

The lowest feasible RAS/ADF ratio that would be expected to be practical is 100%. The effective capacity of the primary effluent pump station utilizing an RAS/ADF ratio of 100% corresponds to 8.8 mgd. This capacity is towards the lower end of industry practice and may be adequate, but should be monitored as flows approach an ADF of 7.7 mgd. Additional capacity would be required to meet the ADF = 10.1 mgd design condition.

4.8 Oxidation Ditches

The facility was upgraded with the installation of three WesTech oxidation ditches in 2007. The plant currently operates two oxidation ditches with the third unit on standby. Mixed liquor from the oxidation ditches overflows by gravity to the secondary clarifiers for solids separation.

The oxidation ditches currently operate with mixed liquor suspended solids (MLSS) concentrations ranging from about 1,200 mg/L to more than 3,500 mg/L, with an average of 1,700 mg/L based on plant records. The two clarifiers in service were observed to be producing clear effluent at the current flow of 5.6 mgd.

The effective capacity of the oxidation ditches based on the existing process condition (WAS co-settling), and a potential upgrade process condition (separate WAS settling) is presented in Table 6. Note that the effective capacity of the most critical parameters, solids retention time (SRT) and food:microorganism ratio (F:M), straddle the design conditions for ADF = 10.1 mgd. Operation of primary clarifiers without co-settling of WAS improves the BOD and TSS removal performance, reduces loadings to the oxidation ditches, and thereby provides a modest increase in the effective capacity of the oxidation ditches.

Table 6: Oxidation Ditch Effective Capacity

Parameter	Industry Practice	Capacity of Existing Units with WAS Co-Settling	Capacity of Existing Units without WAS Co-Settling
HRT	15-30 hr	9.0 MGD	9.0 MGD
MLSS	2500-4000 mg/L	10.7 MGD	11.4 MGD
SRT	15-30 d		
F:M	0.04-0.08 lb BOD / lb MLSS-d	9.0 MGD	9.6 MGD

For the key parameters of SRT and F:M, the effective capacity is towards the lower end of industry practice, but may be adequate and should be monitored as influent wastewater flow increases. The effective capacity of the oxidation ditches may require an increase in the future, but it is premature at this juncture to assume it will be necessary. An assessment of the adequacy of the oxidation ditch capacity is recommended to be evaluated as the ADF approaches 9.0 mgd. This assessment could be conducted in conjunction with the review of the primary effluent pump station's RAS capacity.

4.9 Secondary Clarifiers

There are four secondary clarifiers, which are 90 feet in diameter with an average sidewater depth of 15 feet. The plant is currently operated with only two clarifiers in service and an overflow rate of 440 gpd/sf. The secondary clarifiers were observed to be operating effectively at current flow rates, with the sludge blanket down and clear effluent. The effective capacity of the existing secondary clarifiers based on surface and solids loading rates is presented below in Table 7.

Table 7: Secondary Clarifier Effective Capacity

Design Condition	Hydraulic Surface Loading		Solids Loading (MLSS = 3500 mg/l)	
	Industry Practice	**Capacity of Existing Units (without standby)**	**Industry Practice**	**Capacity of Existing Units (without standby)**
ADF	400-700 gpd/sf	10 – 18 mgd	0.8 -1.2 lb/sf-hr	17 – 25 mgd
PHF	1,000-1600 gpd/sf	25 – 41 mgd	1.6 lb/sf-hr	33 mgd

The existing four clarifiers are adequately sized for the ADF = 10.1 mgd design condition.

4.10 Evaporation / Percolation Ponds

The WWTP has the equivalent of 16 20-acre evaporation/percolation ponds available for secondary effluent disposal. Under current practice, 5 to 6 ponds are needed to handle current effluent flows of 5 to 6 mgd over a heavy wet weather season. At a nominal capacity of 1 mgd per pond, this translates to an annual loading rate equivalent to approximately 56 ft/year.

Current practice is to take selected ponds out of service during each summer season for drying and ripping to reinstate their percolation capacity after two years of service. Consequently, 5 to 6 ponds can be taken out of service in any given year leaving 10 to 11 ponds available for service. This provides an effective ADF capacity of 10 to 11 mgd as shown in Table 8. It appears the existing evaporation/percolation ponds have

adequate capacity for the foreseeable future, but may become marginal as the future design conditions are reached. Continued monitoring of pond performance, which may have a tendency to deteriorate with time, is recommended to anticipate the need for additional secondary effluent capacity as flow approaches the future ADF design condition of 10.1 mgd.

Table 8: Evaporation/Percolation Ponds Effective Capacity

Design Condition	Annual Evaporation/Percolation Rate	
	Historical	Capacity of Existing Units (with 10 to 11 ponds in service)
ADF	56 ft/y	10 to 11 mgd

4.11 Solids Handling

Primary sludge is produced by settling solids from the influent wastewater in the primary clarifiers. The settled sludge is thickened in the primary clarifier sludge hopper and pumped on an intermittent basis to the primary digesters.

WAS is produced from the biological treatment of the primary effluent in the oxidation ditches. As previously discussed, The WAS flows by gravity from the RAS/WAS splitter to the headworks where it combines with the influent wastewater and is delivered to the primary clarifiers. The WAS is then co-settled and co-thickened with the primary sludge in the primary clarifiers and pumped on an intermittent basis to the primary digesters.

For the purposes of this Evaluation and the anticipated operation at design conditions, the WAS withdrawal is assumed to occur one shift per day, 5 days per week. The extended activated sludge process of the oxidation ditches has sufficient flexibility to handle this type of wasting cycle (i.e., the MLSS concentration can increase without negative consequences over a 2 to 3 day period without wasting sludge).

4.11.1 Anaerobic Digesters

The facility has two primary anaerobic digesters and one secondary anaerobic digester for sludge stabilization. The primary digesters have fixed covers, equipped with heating and pump mixing, and provide a combined digester volume for the two primary digesters of 410,000 gallons. The secondary digester is equipped with a floating cover, heating, and pump mixing; and provides a volume of 205,000 gallons. The secondary digester can be used as a primary digester when a primary digester is taken out of service.

The estimated co-settled (primary sludge and WAS) sludge production is 3,500 lb dry solids (DS)/day under current conditions. The primary digesters, not including the secondary digester, operate with an SRT of approximately 16 days. This is sufficient detention time to achieve Class B biosolids.

The Environmental Protection Agency (EPA) requires a minimum 15 days SRT of anaerobic digestion to produce Class B Biosolids.[2] MWH recommends a 15 day SRT with 1 digester out of service, to provide reliability when a digester unit must be taken out of service for inspection, maintenance and/or repair. Current operating conditions comply with the Class B biosolids criteria as described above.

2 U.S. Environmental Protection Agency, Office of Wastewater Management. *A Plain English Guide to the EPA Part 503 Biosolids Rule.* Chapter 5: Pathogen and Vector Attraction Reduction Requirements. By John M. Walker, Lynn Knight, and Linda Stein. Washington, DC, 1994. Print.

As influent wastewater flow and loadings increase in the future and the attendant sludge production increases, the above criteria for Class B biosolids will not be met with the existing digesters. However, under the current arrangement for the transportation and off-site disposal of dewatered sludge by a contractor (Synagro), Class B biosolids for off-site composting may not be required. A reasonable level of stabilization to consider if Class B biosolids is not required is operation with a SRT of 10 days with one digester out of service. Under this scenario, if all digesters are in service, then a SRT of 15 days is provided and Class B biosolids are achieved. For any period of time that one digester unit is out of service, the sludge would be unclassified but could be handled under the current hauling and disposal arrangement.

The effective capacities of the existing digesters based on an SRT of 10 days and 15 days are presented below in Table 9. Note that the effective capacities of the existing digesters for the scenario where WAS is separately thickened and the primary clarifiers are operated without co-settling are also shown for comparison and for subsequent evaluation of solids handling alternatives.

Table 9: Anaerobic Digesters Effective Capacity

Solids Retention Time (SRT)	Capacity of Existing Digester Units	
	With WAS Co-Settling	**Without WAS Co-Settling**
10 days (Unclassified Sludge) (1 digester out of service)	12,800 lb DS/d (ADF = 9.0 mgd equivalent)	18,200 lb DS/d (ADF = 10.5 mgd equivalent)
15 days (Class B Biosolids) (1 digester out of service)	8,500 lb DS/d (ADF = 6.0 mgd equivalent)	12,200 lb DS/d (ADF = 7.4 mgd equivalent)

1. DS = dry solids

For the purposes of this planning effort and potential space allocation, the future addition of digester capacity to achieve Class B Biosolids is evaluated. Construction of future digesters can be phased to match both the actual increase in flow and loadings and to maintain Class B Biosolids, if desired.

Additional anaerobic digester capacity is needed to reach the ADF = 10.1 mgd design condition for either unclassified sludge or Class B biosolids with WAS co-settling. Addition of new digester units of similar size to existing (45-feet diameter, 18-feet side water depth) was assumed for the digester phasing shown in Table 10. Note that capacities shown assume that one digester is out of service.

Table 10: Anaerobic Digesters Phasing with WAS Co-Settling

Digester Phasing	Unclassified[1] (SRT = 10 d)	Class B[1] (SRT = 15 d)
3 – 45' Dia (E)	ADF = 9.0 mgd	ADF = 6.0 mgd
1 – 45' Dia[2]	ADF = 12.0 mgd	ADF = 8.8 mgd
1 – 45' Dia[2]	N/A	ADF = 11.0 mgd

2. Capacities shown assume that the largest digester is out of service.
3. Assumes 45-feet diameter anaerobic digesters to match existing diameters.
4. DS = dry solids

If operation of unclassified sludge is acceptable, the construction of one 45-feet diameter digester would be sufficient to meet the future design condition with continued WAS co-settling. If continued operation to

produce Class B biosolids is desired, the construction of two 45-feet diameter digesters would be sufficient to meet the future design condition with continued WAS co-settling.

Another option to achieve additional anaerobic digester capacity is based on the installation of sludge thickening equipment for dedicated thickening of WAS. This allows the primary clarifiers to operate without co-settling of WAS and increases the primary sludge solids concentrations from 3.5% DS to 6.0% DS. A dedicated WAS thickener can also achieve a 6.0% DS content, so the net sludge solids concentration to the anaerobic digesters increases from 3.5% DS to 6.0% DS concentration. This in turn increases the effective capacity of the existing digesters and decreases the volume needed for future digester capacity. The additional digester capacity without WAS co-settling to meet the future design condition can be achieved by the addition of new digester units of same size as the existing (45-feet diameter with 17-feet sidewater depth).

Refer to Table 11 to see the range of digester phasing possibilities. Note that capacities shown assume that one digester is out of service.

Table 11: Anaerobic Digesters Phasing without WAS Co-Settling

	Unclassified[1]	Class B[1]
3 – 45' Dia (E)	10.4 mgd	7.4 mgd
1 – 45' Dia[2]	N/A	10.5 mgd

1. Capacities shown assume that one digester is out of service.
2. Assumes 45-feet diameter anaerobic digesters with sidewater depth of 17-feet to match existing units.

If operation of unclassified sludge is acceptable, then the existing digesters would be sufficient to meet the future design condition with WAS thickening (without WAS co-settling). If continued operation to produce Class B biosolids is desired, the construction of one 45-feet diameter digesters would be sufficient to meet the future design condition with WAS thickening (without WAS co-settling).

4.11.2 Sludge Dewatering Centrifuges

The existing facility has two Centrisys centrifuges with a capacity of 100 gpm each for digested sludge dewatering. The centrifuges are currently operated 4 to 6 hours per day, 5 days a week to dewater the digested sludge. For the range of digested sludge solids content (say 2.0 % to 3.0% dry solids content), the centrifuge capacity will not be liquid limited (90 to 100 gpm), not solids limited.

A range of operational scenarios can be utilized for planning future centrifuge dewatering. Typical practice at one end of the spectrum would suggest operation of both centrifuges in parallel 5 days per week, 8 hours per day for routine operation. If one centrifuge is taken out of service, then the remaining one would operate additional hours per day and/or additional days per week until the other centrifuge is returned to service. At the other end of the spectrum with less conservative criteria, both centrifuges could operate in parallel up to 7 days per week, 12 hours per day for routine operation. If one centrifuge is taken out of service, then the remaining unit would be required to operate 7 days per week at 24 hours per day. The capacity of the existing units is presented in Table 12 below to show the range of these two operating scenarios.

Table 12: Sludge Dewatering Centrifuge Capacity

Design Condition	Operational Hours	Capacity of Existing Units (2 in service)
5 d/wk	8 hr/d	80,000 lb DS/wk (11.4 mgd ADF equivalent)
7 d/wk	12 hr/d	170,000 lb DS/wk (24 mgd ADF equivalent)

As can be seen, the existing two centrifuges are adequately sized for future design conditions.

4.12 Summary of Near-Term Needs

The effective capacities of the existing wastewater treatment facilities previously evaluated are summarized in Table 13. The effective capacities are shown with the continuation of WAS co-settling and with the elimination of WAS co-settling by installing a dedicated WAS thickening facility. The year the effective capacities are expected to be reached are included in Table 13 and are based on population and flow projections developed in previous chapters. The effective capacities are also shown graphically in Figure 3 and Figure 4.

As can be seen from the graph demonstrating the effective capacity of the treatment plant facilities, additional capacity will be needed in the future to meet the ADF = 10.1 mgd design capacity for the following unit processes:

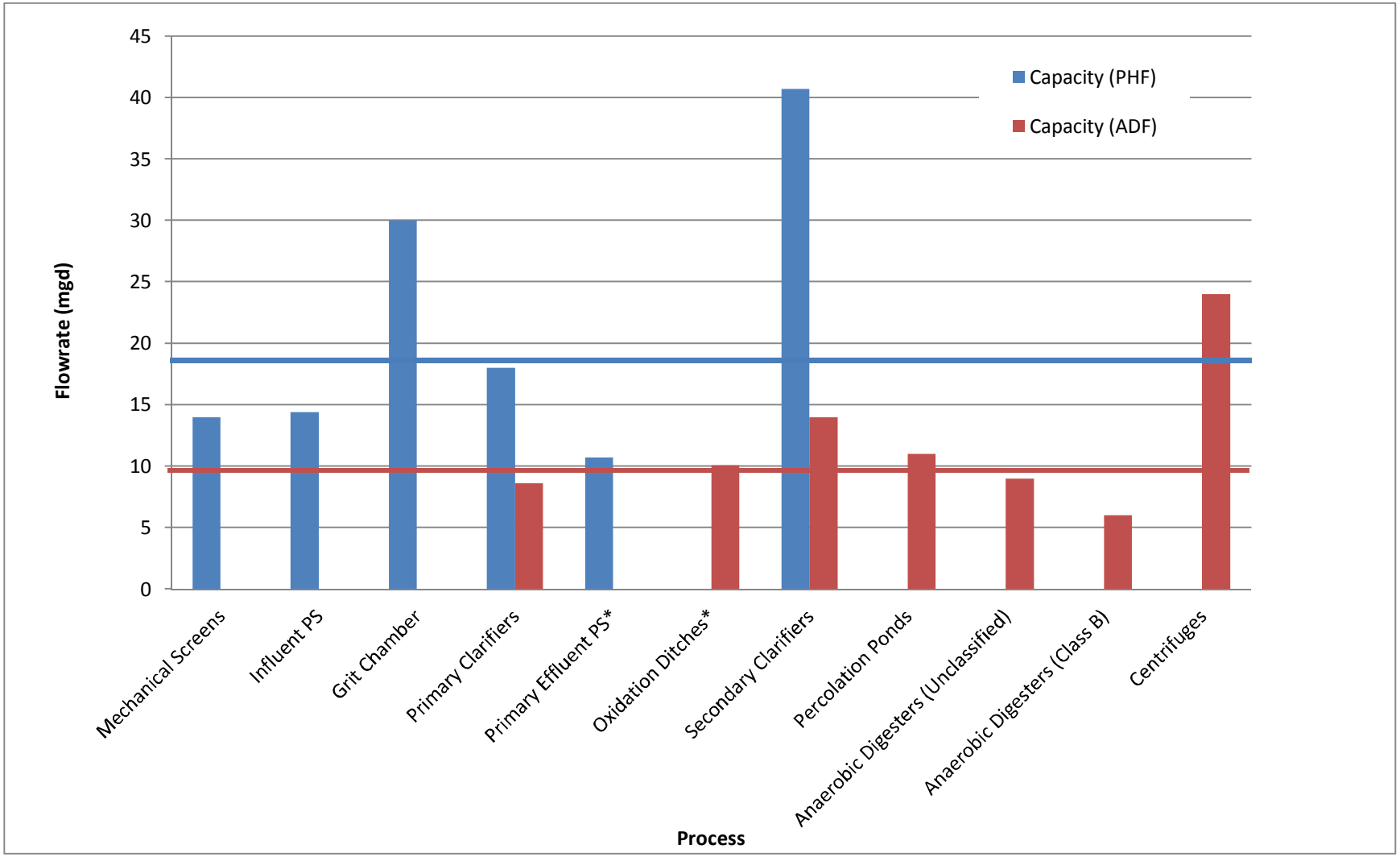
- Preliminary Treatment
  - Mechanical Screens
  - Influent Lift Station
  - Primary Effluent Pump Station
- Solids Handling
  - Primary Clarifiers
  - Anaerobic Digesters
  - WAS Thickening

Alternatives to increase the effective capacity of these treatment processes are evaluated in the following section.

Table 13: Effective Capacities of Treatment Facilities

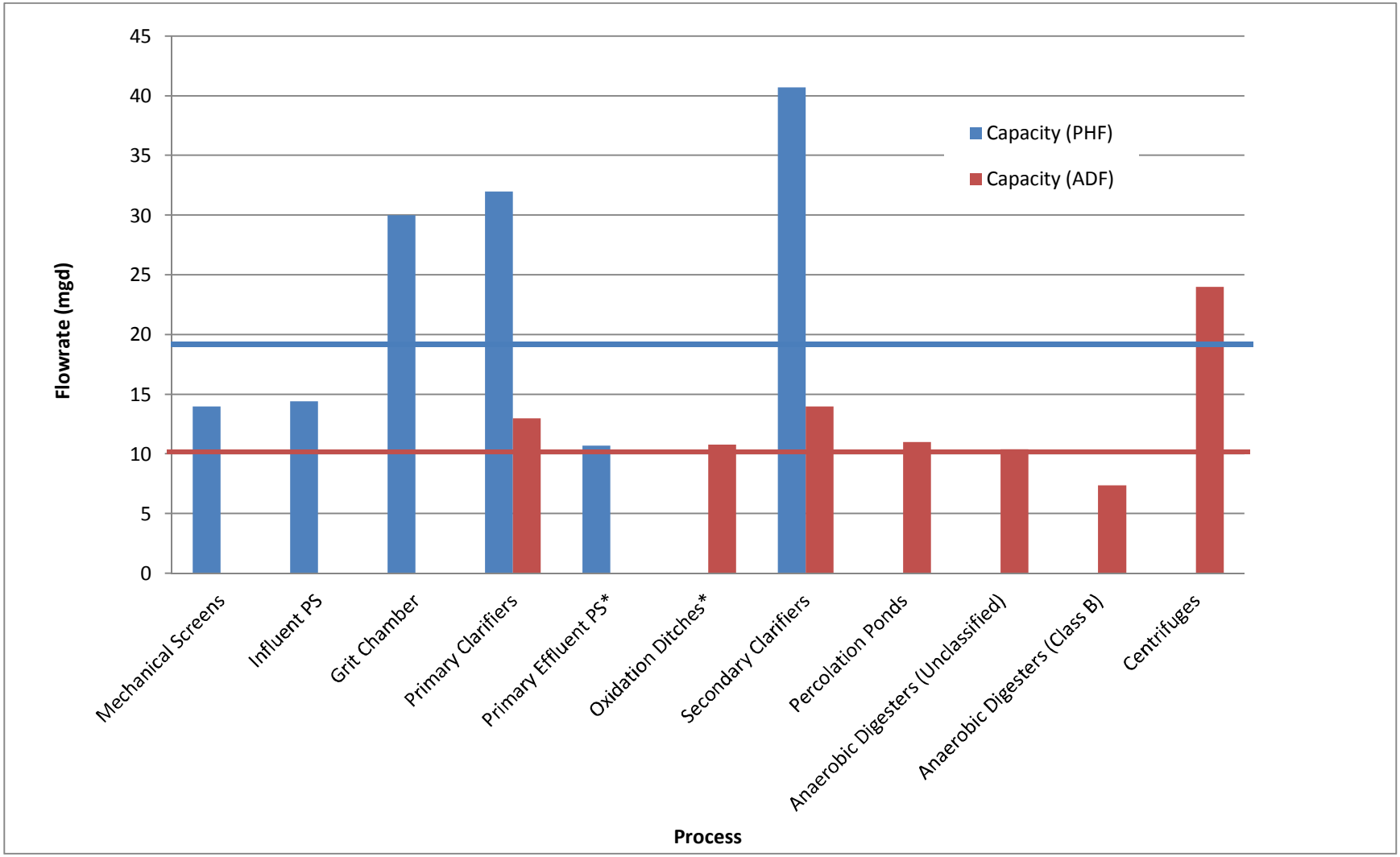
			With WAS Co-Settling		Without WAS Co-Settling	
	Existing Facility Processes		Effective Capacity	Year Effective Capacity Reached	Effective Capacity	Year Effective Capacity Reached
	Duty	Standby				
Headworks						
Mechanical Bar Screens						
PHF Capacity (mgd)	1	1	14	2013	14	2013
Influent Lift Station						
PHF Capacity (mgd)	2	1	14.4	2013	14.4	2013
Grit Chambers						
PHF Capacity (mgd)	2	0	30	2032	30	2032
Primary Treatment						
Primary Clarifiers						
ADF Capacity (mgd)	3	0	8.6	2015	13	2026
PHF Capacity (mgd)	3	0	18	2018	32	2034
Primary Effluent Pump Station						
ADF Capacity (mgd)	2	1	8.8	2016	8.8	2016
Secondary Treatment						
Oxidation Ditches						
ADF Capacity (mgd)	3	0	10.1	2019	10.8	2021
Secondary Clarifiers						
ADF Capacity (mgd)	4	0	18	2035	18	2035
PHF Capacity (mgd)	4	0	41	2041	41	2041
Effluent Disposal						
Evaporation/Percolation Ponds						
ADF Capacity (mgd)	10 – 11	5 – 6	10 – 11	2019-2021	10 – 11	2019-2021
Solids Handling						
Anaerobic Digesters						
ADF Capacity (mgd) Unclassified Sludge	2	1	9.0	2017	10.4	2020
ADF Capacity (mgd) Class B Sludge	2	1	6.0	2012	7.4	2013
Dewatering Centrifuges						
ADF Capacity (mgd) 5d/wk 8hr/d	2	0	11.4	2022	11.4	2022
ADF Capacity (mgd) 7d/wk 12hr/d	2	0	24	2044	24	2044

Figure 3: Effective Capacities with WAS Co-Settling



* Effective capacity shown is based on the upper end of predicted performance range. Refer to text for further discussion.

Figure 4: Effective Capacities without WAS Co-Settling



* Effective capacity shown is based on the upper end of predicted performance range. Refer to text for further discussion.

4.13 Near-Term Improvement Alternatives

Alternatives for increasing capacity of the headworks and solids handling processes and the associated costs are described in this section. Reconnaissance level construction cost estimates were prepared that include civil, structural, architectural, mechanical, HVAC, electrical, and instrumentation elements. The construction cost estimates include amounts for mobilization/demobilization, general conditions, bonds and insurance, and overhead and profit. The opinion of probable cost provided is a Class 5 Estimate in accordance with the Association for the Advancement of Cost Engineering. The expected accuracy range is -30 percent / +50 percent.

MWH has prepared the opinions of probable construction costs in good faith based on its experience and applicable professional industry standards. However, costs of labor and materials, competitive bidding environments and procedures, unknown field conditions, financial and/or market conditions, and other factors are beyond the reasonable control of the City and MWH and may impact the actual cost. MWH makes no warranty, promise, or representation, either express or implied, that the estimated costs of construction presented herein will not vary from subsequent proposals, bids, and/or actual costs.

Costs for the various treatment alternatives were obtained from approximate quantity takeoffs, based on conceptual configurations, unit costs based on recent construction projects, and allowances based on experience with past projects. The estimated construction costs do not explicitly include any costs associated with permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, unforeseen site conditions, and the like.

4.13.1 Preliminary Treatment

Preliminary treatment improvements include increasing the capacity of the headworks facilities and the primary effluent pump station. Note that although it may be desirable to increase grit removal capacity in the future as previously discussed, increased grit removal capacity is not critical and is not included in this analysis. The existing headworks including mechanical screens, influent lift station, and grit chambers has a firm hydraulic capacity (one unit out of service) ranging from 14 mgd to 15 mgd depending upon the particular process as previously discussed. Options to increase the headworks capacity to meet the design condition PHF of 19 mgd include the following:

- Addition of one mechanical screen channel to match the capacity of the two existing channels (14 mgd each) to provide firm screening capacity of 28 mgd.
- Potential replacement of the three existing pumps with three larger pumps (6,600 gpm each) to provide firm influent lift station capacity of 19 mgd.
- Construction of new influent lift station with a 5000 gpm pump that would operate in parallel with the existing influent lift station. The combination of the two lift stations would provide a firm pumping capacity of 22 mgd.

A reconnaissance-level cost estimate for headworks improvements to add one mechanical screen and increase firm pumping capacity of the influent lift station to 19 mgd ranges from $1,000,000 to $2,100,000 depending whether existing pumps are replaced with larger units or a second influent lift station is constructed. The replacement of existing pumps with larger pumps is expected to be more economical than construction of a second lift station, if the existing size of hydraulic components and electrical gear can accommodate the

increased capacity. The assessment of which option should be conducted during preliminary design of the headworks improvements.

Primary effluent pump station capacity will need to be increased from its current PHF + RAS = 25 mgd to PHF + RAS = 35 mgd in order to meet the ADF = 10.1 mgd design criteria and to provide RAS/ADF ratio of 150%. The addition of one 8,700 gpm primary effluent pump to match the additional pumps would provide a capacity of PHF + RAS = 37.5 mgd. Although 37.5 mgd exceeds the needed PHF + RAS = 35 mgd, the incremental cost for the extra capacity is minor and the installation of equal sized pumps provides greater value due to interchangeability and commonality of spare parts. The estimated construction cost to increase the capacity of the primary effluent pump station capacity is $500,000.

4.13.2 Solids Handling Improvements

A matrix of potential improvements that are inter-related are evaluated to address improvements to meet design condition flows and loadings for the primary clarifiers, sludge thickening, and sludge stabilization (anaerobic digestion) processes as indicated in Table 14. The basis of this matrix was refined in previous discussions regarding the effective capacity of the unit processes listed.

Table 14: Solids Handling Improvement Matrix

Description	Option 1: With WAS Co-Settling	Option 2: Without WAS Co-Settling	Option 3: Without WAS Co-Settling
Primary Clarifier	1 Additional Unit (matching existing)	N/A	N/A
WAS Thickening	N/A	Gravity Belt Thickeners or equal	N/A
WAS Dewatering	N/A	N/A	1 Additional Centrifuge (matching existing)
Anaerobic Digesters	2 – 45 ft Dia.	1 – 45 ft Dia.	N/A

The performance of the primary clarifiers with co-settling of WAS (current operation) influences to a minor extent the performance of the oxidation ditches because of differences in BOD and TSS removals as previously discussed, but more importantly influences the sizing of anaerobic digesters. The latter is affected by the significant difference in primary sludge and WAS solids concentration that is obtained with co-settling primary sludge and WAS (3.5% DS) versus primary sludge without WAS co-settling (6.0% DS) and dedicated WAS thickening (6.0% DS) is installed.

A potential solids handling option that could defer construction of WAS thickening and new anaerobic digesters would consist of the following:

- Utilize the existing anaerobic digesters for primary sludge only. Dewater the digested primary sludge with the existing centrifuges.
- Provide no further stabilization of waste activated sludge (WAS) from the oxidation ditches and dewater the undigested WAS with centrifuges. This approach would likely require the addition of one dewatering centrifuge.

This solids handling option would have the following advantages:

- Eliminate co-settling of primary sludge and WAS, thereby increasing the effective hydraulic capacity and BOD removal of the existing primary clarifiers. This would reduce loading to the biological treatment system and provide energy savings.
- Would not require the installation of the WAS thickening. The undigested WAS could be directly dewatered (e.g., current practice with belt presses at South County Regional Wastewater Agency for Gilroy - Morgan Hill CA, Ojai CA, and Lake Elsinore CA; current practice with centrifuges at Western Riverside CA and other installations).
- The existing digesters operating with only primary sludge would have a SRT = 15 days with one unit out of service with the solids loading predicted at an ADF = 11.3 mgd.

This solids handling option would be predicated on the following:

- One potential implication of improved BOD removal by the primary sedimentation tanks is that the reduced loading to the biological treatment system could inhibit denitrification if insufficient carbon was available. The implementation of this option should be predicated on a careful evaluation of the biological treatment process with a process model that would be calibrated to existing performance and then used to simulate performance with the proposed changes to determine if biological treatment would be inhibited
- Utilization of a third party (should be a viable option with current contractor - Synagro) to accept and handle unclassified dewatered biological sludge (WAS).
- Suitability of existing centrifuges to dewater digested primary sludge and undigested WAS. Discussion with existing centrifuge manufacturer's representative[3] indicates that the existing units can successfully and separately dewater digested primary sludge and undigested WAS. The most significant features of this approach are summarized as follows:
    - The centrifuge dewatering of digested primary sludge would be anticipated to be relatively easy (very similar, if not better than the current digested co-settled sludge) with higher dewatered cake solids (on the order of 30% DS content).
    - The centrifuge dewatering of undigested WAS would be anticipated to be more difficult than the current digested co-settled sludge and produce lower dewatered cake solids (in the range of 15% to 18% DS content) at a similar sludge feed rate. However, the solids loading with 1% DS WAS feed would be considerable lower and an additional centrifuge would be needed for adequate dewatering capacity.

The installation of WAS feed pumps would be needed to deliver WAS to the dewatering units. Note that if the option to implement WAS thickening was implemented, the same pumps could likely be used to deliver WAS to the thickening units.

A summary of the construction cost estimates for the solids handling alternatives are presented in

Table 15. Option 2 without WAS co-settling (dedicated WAS thickening) is $2,600,000 lower in estimated construction cost than Option 1 with WAS co-settling. Note that this comparison was based on the use of gravity belt thickeners (GBTs) for dedicated WAS thickening. Other WAS thickening options such as centrifuges (may be desirable to match equipment used for existing sludge dewatering) or rotary drum screens are feasible and should evaluated during preliminary design if this alternative is selected. Option 3 without WAS co-settling and without anaerobic digestion of primary sludge is $2,100,000 lower in construction cost than Option 2.

[3] Telephone conversation between Chris Gatewood – Centrysis and Steve Hyland – MWH on April 17, 2012.

Table 15: Solids Handling Improvements Estimated Life-Cycle Cost

Description	Option 1		Option 2		Option 3	
	With WAS Co-Settling	**Amount**	**Without WAS Co-Settling**	**Amount**	**Without WAS Co-Settling**	**Amount**
Primary Clarifier	1 Additional Unit (matching existing)	$1,500,000	N/A	$0	N/A	$0
WAS Thickening	N/A	$0	Gravity Belt Thickeners or equal	$3,100,000	N/A	$0
WAS Dewatering	N/A	$0	N/A	$0	1 Additional Centrifuge (matching existing)	$3,600,000
Anaerobic Digesters	2 – 45' Dia.	$6,800,000	1 – 45' Dia.	$2,600,000	N/A	$0
Construction		$8,300,000		$5,700,000		$3,600,000
Incremental O&M Costs		$6,000/yr		$73,000/yr		$230,000/yr
PW of O&M Costs		$100,000		$1,000,000		$3,200,000
Total		**$8,400,000**		**$6,700,000**		**$6,800,000**

The implementation of Option 1 would have little impact on existing annual O&M costs. The incremental annual cost for Option 1 to operate one additional primary clarifier is minor and the difference in annual cost to operate two additional 45-feet diameter digesters versus one additional 45-feet diameter digester is minimal.

The incremental annual cost to operate a sludge thickening facility for Option 2 would be similar, but lower, than the annual cost to operate the existing sludge dewatering facility (equivalent labor and repair cost, but lower chemical and electrical power cost). The annual cost for dedicated sludge thickening may be on the order of $73,000 per year and has a present worth value of $1,000,000 (PWF = 13.59, 4% interest rate, 20 year life). The lifecycle cost evaluation favors Option 2 without WAS co-settling by approximately $1,700,000 and would be preferred to enhance wastewater treatment (lower loading to oxidation ditch). Also note that the practice of co-settling primary sludge and WAS has fallen out of favor and is generally not used with current wastewater practice.

The incremental annual cost to operate on-site WAS dewatering facility for Option 3 is similar to the cost of the dedicated WAS thickening facility for Option 2, but the off-site disposal costs for dewatered raw, biological sludge will be significantly higher than the equivalent cost of offsite disposal of dewatered, digested sludge. The undigested WAS will not benefit from solids destruction from sludge digestion and the WAS cake will have a lower DS percentage (more water) that will increase cake quantities and associated disposal costs.

Option 3 was developed as a potential near-term approach to defer capital investment in dedicated WAS thickening and additional digester capacity as influent wastewater flows increased if circumstances allowed. However, the long-term expectation is that dedicated WAS thickening and stabilization of all sludge production will be needed, and perhaps sludge stabilization to produce Class B biosolids will be desirable in the future. Consequently, the development of alternatives for master planning purposes is based on Option 2, but the interim use of Option 3 is feasible and does not compromise the ultimate adoption of Option 2. That is, the interim adoption of Option 3 would not result in stranded assets.

Also note that the cost comparisons presented above are based on continuing to achieve Class B biosolids. If the production of unclassified sludge is accepted, the capital cost for solids handling improvements can be deferred until influent flow approaches ADF = 9.0 mgd.

4.14 Near-Term Findings and Recommendations

MWH has evaluated the existing treatment facilities to determine their effective capacities, and compared them to the design conditions of ADF = 10.1 mgd. Two recommended capital improvement projects that will be needed in the near future include the preliminary treatment improvements (additional mechanical bar screen, influent lift station capacity, and primary effluent pump station capacity) and solids handling improvements (new sludge thickening system and additional anaerobic digester). Note that the latter recommendation is based on eliminating WAS co-settling, utilizing dedicated WAS thickening, and continued production of Class B biosolids.

In addition, the effective capacity of several treatment process facilities may be become marginal as influent wastewater into the WWTP increases in the future. The performance of the oxidation ditches and evaporation / percolation ponds should be monitored at periodic intervals to anticipate the potential for future deficiencies.

The timing and estimated construction costs of recommended improvements for both unclassified sludge and Class B biosolids are summarized in Table 16. The timing of potential improvements is also summarized in Table 16. The year the effective capacities are expected to be reached are included in Table 13 and are based on population and flow projections developed in previous chapters. A preliminary site layout of the recommended facilities to reach ADF = 10.1 mgd design capacity with unclassified sludge is shown in Figure 5.

Table 16: Recommended Capital Improvement Projects for ADF = 10.1 mgd Design Conditions

Unclassified Sludge			Class B Biosolids		
Year Improvement Needed	Recommended Capital Improvement Projects	Estimated Construction Cost[1]	Year Improvement Needed	Recommended Capital Improvement Projects	Estimated Construction Cost[1]
Short Term	Preliminary Treatment Improvements • Mechanical Bar Screen Addition • Influent Lift Station Expansion • Primary Effluent Pump Station Expansion	$1,500,000 - $2,600,000[2]	Short Term	Preliminary Treatment Improvements • Mechanical Bar Screen Addition • Influent Lift Station Expansion • Primary Effluent Pump Station Expansion	$1,500,000 - $2,600,000[2]
Y2014 - Y2017	Solids Handling Improvements without WAS Co-Settling[3] • Sludge Thickener	$3,100,000	Y2014 – Y2020	Solids Handling Improvements without WAS Co-Settling[3] • Sludge Thickener • 1 – 45' diameter Anaerobic Digester	$9,900,000[2]
Review By	Potential Capital Improvement Projects	Comments	Review By	Potential Capital Improvement Projects	Comments
Y2017	Oxidation Ditch Improvements	Unlikely to be required	Y2017	Oxidation Ditch Improvements	Unlikely to be required
Y2019	Evaporation / Percolation Ponds	Proactive monitoring is critical	Y2019	Evaporation / Percolation Ponds	Proactive monitoring is critical

1. Estimated construction cost does not include escalation, planning, engineering, administration, permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, unforeseen site conditions, and the like.
2. Cost range dependent on influent lift station expansion option.
3. Solids handlng improvements based on Option 2 but does not preclude interim implementation of Option 3.

5 Urban Growth Boundary Alternatives

5.1 Introduction

The long-term horizon for the Sanitary Sewer Master Plan is based on the ultimate population and associated wastewater flow within the Urban Growth Boundary. The ultimate flow projections have been based on the urban growth boundary service area and population projections developed in Chapter 5. The population projections assume the following:

- Full occupancy of currently vacant buildings.
- New commercial and industrial growth.

- 3.5% annual population growth.

The above results in a predicted ADF of 26.5 mgd for complete development of the urban growth boundary service area.

In addition, the development of alternatives to provide long-term wastewater treatment must consider future regulations and the treatment technologies best suited for the anticipated future regulations. A discussion of potential regulatory requirements and viable treatment technologies are presented below. This discussion is followed by the development and evaluation of four Urban Growth Boundary Alternatives. This evaluation concludes with a single recommended alternative for collection system and treatment plant improvements in order to provide for the long-term needs of the City.

5.2 Future Regulations

The long-term forecast of wastewater treatment needs is very difficult to predict because of the constantly changing landscape of regulatory requirements. Perhaps the only certainty based on historical experience is to expect more restrictive discharge requirements in the future. The recent Strategic Plans and permitting activities of the State Water Resources Control Board and Central Valley Regional Water Quality Control Board suggests a future emphasis on regulating the following categories of pollutants:

- Salts (total dissolved solids, or TDS) and nitrate
- Coliforms and viruses
- Metals
- Emerging constituents (personal care products, pharmaceuticals, endocrine disruptors)

The City is fortunate to have a low TDS potable water supply that reduces the likelihood of potential issues with TDS regulations in the future, but increasing demand for water resources with higher utilization of groundwater and the increase in TDS for domestic use (200 – 300 mg/L increase from residential water supply to WWTP effluent) creates a subtle, but inexorable trend of increasing TDS beneath the existing evaporation/percolation ponds. No TDS related regulations are anticipated in the near future, but some degree of salt management may be inevitable.

The City is already providing biological treatment with the anoxic cells in the oxidation ditches that provide denitrification. Continued biological denitrificaion (conversion of nitrate to nitrogen gas) is assumed for all treatment alternatives considered in the Sanitary Sewer Master Plan.

Disinfection to reduce coliforms and/or viruses is not currently required with discharge to the existing evaporation/percolation ponds, but increasing utilization of groundwater resources may translate to a heightened concern and future disinfection requirements. Further, if evaporation/percolation pond capacity is limited and increased wastewater flow leads to surface water discharge alternatives for future portions of treated effluent, then disinfection requirements would certainly be expected.

The regulatory attention regarding the impact of dissolved metals on drinking water quality and aquatic life is becoming increasingly significant. In most instances, the effective metal limitations to protect aquatic life can be more restrictive than those for drinking water standards.

And finally, concerns about emerging constituents such as personal care products, pharmaceuticals, and endocrine disruptors in the environment increase as new analytical techniques become available to measure more and more compounds at lower and lower concentrations. These measurements promote increasing public concern about potential health and environmental impacts and require attention by the regulatory agencies.

In summary, the long-term forecast of how future regulations of the pollutant categories discussed above will impact the wastewater treatment requirements is difficult to predict. But the potential for more restrictive requirements in the future is considered in the following discussion regarding treatment technologies in order to best position the City for possible future requirements.

5.3 Potential Treatment Technologies

A reconnaissance-level review of potential treatment technologies was conducted for candidate processes for future WWTP expansions to meet the long-term Urban Growth Boundary flow capacity (ADF = 26.5 mgd) as well as to best position the City for future discharge regulations. A summary of potential treatment technologies for WWTP expansions with relative advantages and disadvantages is presented in Table 17.

A comparison of the biological treatment technologies to the existing oxidation ditch process presented above shows that with the exception of the membrane bioreactor (MBR), the remaining processes do not demonstrate any clear-cut advantages. The alternative technologies to the oxidation ditch process (other than MBR) are typically more complicated, would require greater operator attention, and would require training for operation of a second technology. Consequently, the remaining treatment technologies are not recommended for WWTP expansion.

MBR technology offers many advantages that better position the City for regulations that may promulgate in the future. In particular, any advanced treatment that may be contemplated for removal of TDS, metals, coliforms and viruses, and emerging constituents would be better served with membrane effluent than conventional tertiary media filtration. In addition, the production of Title 22 unrestricted recycled water and requiring the smallest footprint for WWTP sites will likely provide the City with significant side benefits in the future. Consequently, the utilization of MBR technology for long-term planning to meet the Urban Growth Boundary flow (ADF = 26.5 mgd) is recommended.

Under current conditions disinfection is not required for discharge to the evaporation/percolation ponds. However, future disinfection may be needed for the following scenarios:

- Future coliform limitations for discharge to evaporation/percolation ponds.
- Recycled water production.
- Potential discharge to receiving waters if additional evaporation/percolation for increased treated effluent flow is not available.

Disinfection with chlorine has historically been the predominant means of disinfection in the water and wastewater industry with a shift from chlorine gas to liquid hypochlorite (bleach) over the past two decades for safety and public perception considerations. However, ultraviolet (UV) disinfection has gained increasing prominence in recent years to minimize chemical handling, reduce the footprint for disinfection facilities, and avoid the potential generation of chlorinated byproducts. In addition, UV disinfection is generally favored

Table 17: Summary of Treatment Technologies

Treatment Technology	Advantages	Disadvantages
Oxidation Ditch	• Existing technology • Extended aeration • Robust, reliable operation	• Relatively larger footprint • Advanced treatment upgrade would require tertiary filtration
Vertical Loop Reactor (VLR)	• Variation of oxidation ditch technology • More energy efficient • Smaller footprint	• Not as tried and proven • Access for service more difficult • Advanced treatment upgrade would require tertiary filtration
Convention Activated Sludge (CAS)	• Tried and proven technology • Robust, reliable operation • Fine bubble aeration more energy efficient • Provides more precise operational control	• Greater operator attention required • Blower and fine bubble aeration maintenance • Advanced treatment upgrade would require tertiary filtration
Sequencing Batch Reactor (SBR)	• Smaller footprint with basins that combine biological treatment and solids separation • Provides more precise operational control	• Greater operator attention required • Blower and fine bubble aeration maintenance • Decanter operation can be problematic • Advanced treatment upgrade would require tertiary filtration • Generally not considered for WWTP capacities above 5 mgd
Biological Aerated Filtration (BAF) **Integrated Fixed Film Activated Sludge (IFAS)** **Moving Bed Bioreactor (MBBR)**	• Combination of fixed growth and suspended growth biological treatment • Reduces footprint • More energy efficient • Improved nitrogen removal	• Not as tried and proven • Access for service more difficult • Retaining fixed growth media can be problematic • Advanced treatment upgrade would require tertiary filtration
Membrane Bioreactor (MBR)	• State of the art technology • Smallest footprint • Provides higher quality effluent than conventional tertiary sand filters • Removes all coliforms • Provides Title 22 unrestricted use recycled water • Recommended treatment step upstream of advanced treatment	• Requires fine screening (< 2mm) • Higher energy consumption for air scour and recirculation • Chemical use for membrane cleaning • Membrane replacement every 7 to 10 years
Chlorine Disinfection	• Conventional disinfection technology	• Chemical use
Ultraviolet Disinfection	• Smaller footprint • No known byproducts • Meshes well with MBR	• Higher energy use

for MBR applications because the UV dose for membrane effluent disinfection is low (high transmittance effluent and membrane removal of coliforms). Consequently, UV disinfection is the recommended disinfection technology for the Sanitary Sewer Master Plan.

5.4 Urban Growth Boundary Alternatives

Four alternatives have been developed to evaluate the preferred means of accommodating the future flow (ADF = 26.5 mgd) associated with ultimate development within the urban growth boundary as follows:

- Baseline Alternative 1A (Existing WWTP)
- Baseline Alternative 1B (Existing WWTP)
- Alternative 2 (Existing WWTP & East WWTP)
- Alternative 3 (Existing WWTP, East WWTP, & North WWTP)

These alternatives are combinations of collection system improvements and provisions for one to three treatment plant location(s).

Baseline Alternative 1 preserves the existing WWTP location for treatment of all flow and was established for comparison with the other alternatives. Baseline Alternative 1 is used as a planning tool to represent the alternative best known to the City and the alternative expected to have the fewest complications and impediments for implementation. The expectation is that compelling reasons, such as significant cost savings or effluent disposal availability, would be needed to select an alternative that would decentralize wastewater treatment (i.e., provide treatment plant capacity at more than one location).

The development of Baseline Alternative 1 is predicated on the following assumptions:

- Collection system infrastructure provided to deliver influent wastewater to existing WWTP location;
- Elimination of WAS co-settling and use of WAS thickening;
- New headworks with mechanical screens, influent pump station, and grit chambers to supplement existing facilities;
- Continued use of primary clarifiers and biological treatment with oxidation ditches and secondary clarifiers provided for near-term capacity (ADF = 10.1 mgd);
- Additional primary clarifiers and additional biological treatment with either oxidation ditches and secondary clarifiers or MBR technology to reach the Urban Growth Boundary capacity (ADF = 26.5 mgd);
- Space allocation for potential tertiary treatment facilities to improve the quality of secondary effluent from the oxidation ditch and secondary clarifier facilities if needed in the future;
- Continued effluent disposal with additional evaporation/percolation pond capacity (note that the location of additional evaporation/percolation pond capacity is unknown and therefore not shown); and
- Sludge stabilization with anaerobic digestion to achieve Class B Biosolids.

The preceding assumptions were used to develop a conceptual configuration to determine potential land requirements. However, an expansion of wastewater treatment capacity of this magnitude (26.5 mgd) should also include a higher-level perspective that would consider the tradeoffs between single centralized WWTP location and multiple WWTP locations. These tradeoffs include collection system improvements (sewer interceptors and pump stations) associated with the WWTP location(s).

Consequently, two additional alternatives were developed that utilize alternate treatment plant locations that may reduce the cost of collection system improvements or effluent disposal. Alternative 2 utilizes one additional treatment plan in conjunction with the existing WWTP. Alternative 3 utilizes two additional treatment plants in conjunction with the existing WWTP.

The development of the decentralized WWTP(s) for Alternative 2 and Alternative 3 is predicated on the following assumptions:

- Collection system infrastructure provided to deliver influent wastewater to new WWTP location(s);
- Headworks with influent pump station and first-stage mechanical screens, grit removal, and second-stage fine screens;
- Primary clarifiers;
- MBR treatment train with anoxic basins, aeration basins, submerged membrane basins, filtrate pumps, backwash pumps, mixed liquor return and recirculation pumps, chemical cleaning systems, UV disinfection, and final effluent pumps;
- Effluent disposal to evaporation/percolation ponds (note that the location of additional evaporation/percolation ponds is unknown and therefore not shown);
- Sludge thickening with gravity belt thickeners and transportation of thickened sludge to the existing WWTP for sludge stabilization, dewatering, and off-site disposal;
- Maintenance and standby power building; and
- Administration building

The development of Alternative 2 and Alternative 3 assumes the use of additional evaporation/percolation ponds. The location of these ponds cannot be established at this time. If additional land for evaporation/percolation pond capacity is limited or not proximate to treatment plant locations, alternative effluent disposal systems may need to be investigated. Alternative effluent disposal systems may include the following:

- Potential year-round recycled water customers.
- Disposal to receiving waters that would require an NPDES Permit. Effluent discharge limitations for receiving water discharge would likely require a higher level of treatment, e.g. tertiary treatment.
- Potential utilization of seasonal effluent disposal options. Wet weather discharge to receiving waters that have less restrictive discharge requirements than year-round discharge requirements. Dry weather discharge to evaporation/percolation ponds and/or recycled water customers. The wet weather discharge option would offset the reduced evaporation/percolation pond capacity and reduce recycled water demand during the wet weather season.

A space allocation for the tertiary treatment of secondary effluent from the oxidation ditches and secondary clarifiers provided for the near-term capacity (ADR = 101 mgd) has also been included. The future installation of tertiary treatment should be considered for the following reasons:

- Tertiary treatment may improve disposal capacity of the existing evaporation/percolation ponds and/or prolong their service life.
- Tertiary treatment may be considered to accommodate potential future requirements for advanced treatment such as TDS, metals, and emerging constituents removal and disinfection as previously discussed.

In addition, a more detailed evaluation for the project alternatives described herein should include externalities such as land acquisition, permitting, public acceptance, environmental assessment, and financial planning.

A brief description, summary of advantages and disadvantages, and order of magnitude costs for each alternative are presented below.

5.5 Baseline Alternative 1

Baseline Alternative 1 would continue to convey all wastewater flow to the existing WWTP location. The collection system improvements necessary to convey the urban growth boundary flow (ADF = 26.5 mgd) to the existing WWTP location would necessitate construction of trunk sewers that parallel existing infrastructure as shown in Figure 6. A summary of the collection system improvements for Baseline Alternative 1 is presented in Chapter 7.

Two reconnaissance-level treatment options for Baseline Alternative 1 have been developed to expand the existing WWTP from the near-term capacity of 10.1 mgd to the ultimate growth boundary capacity of 26.5 mgd. The first option, Baseline Alternative 1A, is based on the continued use of oxidation ditch (OD) technology and the second option, Baseline Alternative 1B, utilizes MBR technology. Although MBR technology is the recommended means for providing the plant expansion capacity, the expansion of oxidation ditch technology has been shown for comparison and to represent the maximum space that could potentially be required. For both treatment options, two plant configurations have been developed based on adding future facilities to the west and based on adding future facilities to the north as shown on Figures 7A, 7B, 8A, and 8B.

Note that a space allocation has been included for tertiary treatment of secondary effluent for the near-term ADF capacity of 10.1 mgd. Summaries of the additional facilities at the existing WWTP location for the continued utilization of the oxidation ditch technology and the MBR technology are presented in Table 18 and Table 19, respectively.

Table 18: Urban Growth Boundary Expansion with OD Technology for Baseline Alt. 1A (26.5 mgd)

	Number of Units		
	ADF = 10.1 mgd	Additional[1]	Total
Headworks	1	1	2
Primary Treatment			
Primary Clarifiers	3	3	6
Primary Effluent Pump Station	1	1	2
Secondary Treatment			
Oxidation Ditches	3	5	8
Secondary Clarifiers	4	4	8
Potential Tertiary Filters[2]	0	10	10
Effluent Disposal			
Evaporation/Percolation Ponds	16	23	39
Solids Handling			
Sludge Thickening Units	2	2	4
Anaerobic Digesters[3]	4	2[4]	6
Dewatering Centrifuges	2	2	4

1. Additional facilities are assumed to be same size as existing unless otherwise noted.
2. Space allocation for potential filters for tertiary treatment of 26.5 mgd oxidation ditch effluent (e.g. Title 22 recycled water).
3. Assumes largest digester unit is out of service.
4. Additional digesters are assumed to be 70-feet diameter units.

Table 19: Urban Growth Boundary Expansion with MBR for Baseline Alternative 1B (26.5 mgd)

	Number of Units		
	ADF = 10.1 mgd	Additional[1]	Total
Headworks	1	1	2
Primary Treatment			
Primary Clarifiers	3	3	6
Primary Effluent Pump Station	1	1	2
Secondary Treatment			
Oxidation Ditches	3	0	3
Secondary Clarifiers	4	0	4
Secondary/Tertiary Treatment			
MBR Trains[2]	0	4	4
Effluent Disposal			
Evaporation/Percolation Ponds	16	20	36
Potential Tertiary Filters[3]	0	4	4
Solids Handling			
Sludge Thickening Units	2	2	4
Anaerobic Digesters[4]	4	2[5]	6
Dewatering Centrifuges	2	2	4

1. Additional facilities are assumed to be same size as existing unless otherwise noted.
2. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.
3. Space allocation for potential filters for tertiary treatment of 10.1 mgd oxidation ditch effluent (e.g. Title 22 recycled water).
4. Assumes largest digester unit is out of service.
5. Additional digesters are assumed to be 70-feet diameter units.

5.6 Alternative 2

Alternative 2 is based on the conveyance of a portion of the Urban Growth Boundary flow (ADF = 14.1 mgd) to the existing WWTP location. The remaining portion of the Urban Growth Boundary flow (ADF = 12.4 mgd) would be conveyed to a new treatment site (East WWTP). The collection system improvements (trunk sewers) necessary to convey the urban growth boundary flow to the existing WWTP location and the new East WWTP location are shown in Figure 9.

The selected treatment technology for the expansion of the existing WWTP and the East WWTP is MBR technology as previously discussed. The proposed configuration for the existing WWTP location is based on adding future MBR facilities as shown on Figure 10. A process flow schematic and an estimate of space allocation for the treatment plant technology at the new East WWTP are shown in Figure 11.

Note that a space allocation has been included for tertiary treatment of secondary effluent from the existing WWTP for the near-term ADF capacity of 10.1 mgd. Summaries of the additional facilities at the existing WWTP location for the addition of MBR technology the new facilities for MBR treatment technology at the new East WWTP are presented in Table 20 and Table 21, respectively.

Table 20: Additional WWTP Facilities for Alternative 2

	Number of Units		
Existing WWTP (14.1 MGD)	**ADF = 10.1 mgd**	**Additional[1]**	**Total**
Headworks	1	1	2
Primary Treatment			
Primary Clarifiers	3	1	4
Primary Effluent Pump Station	1	1	2
Secondary Treatment			
Oxidation Ditches	3	0	3
Secondary Clarifiers	4	0	4
Secondary/Tertiary Treatment			
MBR Trains[2]	0	1	1
Effluent Disposal			
Evaporation/Percolation Ponds	16	5	21
Potential Tertiary Filters[3]	0	1	1
Solids Handling			
Sludge Thickening Units	2	0	2
Anaerobic Digesters[4]	4	2[5]	6
Dewatering Centrifuges	2	2	4

1. Additional facilities are assumed to be same size as existing unless otherwise noted.
2. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.
3. Space allocation for potential filters for tertiary treatment of 10.1 mgd oxidation ditch effluent (e.g. Title 22 recycled water).
4. Assumes largest digester unit is out of service.
5. Additional digesters are assumed to be 70-feet diameter units.

Table 21: New WWTP Facilities for Alternative 2

	Number of Units
	East WWTP (12.4 mgd)
Headworks and Fine Screening	1
Primary Treatment	
Primary Clarifiers	3
Primary Effluent Pump Station	1
Secondary/Tertiary Treatment	
MBR Trains[1]	3
Effluent Disposal	
Evaporation/Percolation Ponds	15
Solids Handling	
Thickening Facilities	1
Facilities	
Administration Building	1
Maintenance/Standby Power Building	1

1. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.

5.7 Alternative 3

Alternative 3 is based on the conveyance of a portion of the Urban Growth Boundary flow (ADF = 14.1 mgd) to the existing WWTP location as in Alternative 2. The remaining portion of the Urban Growth Boundary flow (ADF = 12.4 mgd) would be conveyed to two new treatment plant locations. One location would be the East WWTP (ADF = 8.3 mgd) as described in Alternative 2. The second location would be the North WWTP (ADF = 4.1 mgd). The collection system improvements (trunk sewers) necessary to convey the urban growth boundary flow to the existing WWTP, the new East WWTP, and the new North WWTP locations are shown in Figure 12.

The selected treatment technology for the expansion of the existing WWTP, the East WWTP, and the North WWTP is MBR technology as previously discussed. The proposed configuration for the existing WWTP location is based on adding future MBR facilities as shown on Figure 10. A process flow schematic and an estimate of space allocation for the treatment plant technology at the new East WWTP and the new North WWTP are shown in Figures 13A and 13B, respectively.

Note that a space allocation has been included for tertiary treatment of secondary effluent from the existing WWTP for the near-term ADF capacity of 10.1 mgd. Summary of the additional facilities at the existing WWTP location for the addition of MBR technology is presented in Table 18. Summaries of the new facilities for the MBR treatment technology at the new East WWTP and North WWTP are presented in Table 23.

Table 22: Additional Facilities for Existing WWTP for Alternative 3 (14.1 mgd)

	Number of Units		
Existing WWTP (14.1 MGD)	ADF = 10.1 mgd	Additional[1]	Total
Headworks	1	1	2
Primary Treatment			
Primary Clarifiers	3	1	4
Primary Effluent Pump Station	1	1	2
Secondary Treatment			
Oxidation Ditches	3	0	3
Secondary Clarifiers	4	0	4
Secondary/Tertiary Treatment			
MBR Trains[2]	0	1	1
Effluent Disposal			
Evaporation/Percolation Ponds	16	5	21
Potential Tertiary Filters[3]	0	1	1
Solids Handling			
Sludge Thickening Units	2	0	2
Anaerobic Digesters[4]	4	2[5]	6
Dewatering Centrifuges	2	2	4

1. Additional facilities are assumed to be same size as existing unless otherwise noted.
2. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.
3. Space allocation for potential filters for tertiary treatment of 10.1 mgd oxidation ditch effluent (e.g. Title 22 recycled water).
4. Assumes largest digester unit is out of service.
5. Additional digesters are assumed to be 70-feet diameter units.

Table 23: New WWTP Facilities for Alternative 3

	Number of Units	
	East WWTP (8.3 mgd)	North WWTP (4.2 mgd)
Headworks and Fine Screening	1	1
Primary Treatment		
Primary Clarifiers	2	1
Primary Effluent Pump Station	1	1
Secondary/Tertiary Treatment		
MBR Trains[1]	2	1
Effluent Disposal		
Evaporation/Percolation Ponds	10	5
Solids Handling		
Thickening Facilities	1	1
Facilities		
Administration Building	1	1
Maintenance/Standby Power Building	1	1

1. MBR train consists of aeration basin, submerged membrane basin, UV disinfection, aeration and air scour blowers, filtrate and backwash pumps and associated support systems.

5.8 Estimated Construction Costs

The estimated construction costs for Baseline Alternative 1A, Baseline Alternative 1B, Alternative 2, and Alternative 3 are presented in Table 24 below. Reconnaissance level construction cost estimates were prepared for the four alternatives previously described. The construction cost estimates include allowances for mobilization/demobilization, general conditions, bonds and insurance, overhead and profit, and contingency. The opinion of probable cost provided is a Class 5 Estimate in accordance with the Association for the Advancement of Cost Engineering. The expected accuracy range is -30 percent / +50 percent.

MWH has prepared the opinions of probable construction costs in good faith based on its experience and applicable professional industry standards. However, costs of labor and materials, competitive bidding environments and procedures, unknown field conditions, financial and/or market conditions, and other factors are beyond the reasonable control of the City and MWH and will impact the actual cost. MWH makes no warranty, promise, or representation, either expressed or implied, that the estimated costs of construction presented herein will not vary from subsequent proposals, bids, and/or actual costs.

Costs for the various treatment alternatives were based on conceptual configurations, unit costs, and allowances based on past planning and design projects. The estimated construction costs do not explicitly include any costs associated with permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, unforeseen site conditions, and the like.

Table 24: Estimated Construction Costs for Alternatives

Description	Estimated Construction Costs[1]			
	Alternative 1		Alternative 2	Alternative 3
	1A – OD Technology``	1B - MBR		
Service Area				
Collection System	$31.9M	$31.9M	$20.0M	$18.9M
Recycled Water	$4.2M	$4.2M	$0M	$0M
Total	$36M	$36M	$20M	$19M
Existing WWTP				
Pretreatment	$16.4M	$20.5M	$5.0M	$5.0M
Primary Treatment	$6.1M	$6.1M	$1.9M	$1.9M
Oxidation Ditches/Secondary Clarifiers	$91.6M			
Tertiary Treatment[2]	$26.5M	$10.1M	$10.1M	$10.1M
MBR Trains		$114.8M	$48.0M	$48.0M
Effluent Disposal	$32.8M	$32.8M	$8.0M	$8.0M
Solids Handling	$17.3M	$17.3M	$14.2M	$14.2M
Total	$191M[3]	$202M	$87M	$87M
East WWTP	---	---	---	---
Pretreatment			$15.5M	$10.4M
Primary Treatment			$5.7M	$3.8M
MBR Trains			$99.2M	$83.0M
Effluent Disposal			$24.8M	$16.6M
Solids Handling			$3.1M	$3.1M
Buildings			$4.3M	$4.3M
Total			$153M	$121M
North WWTP	---	---	---	---
Pretreatment				$5.1M
Primary Treatment				$1.9M
MBR Trains				$49.2M
Effluent Disposal				$8.2M
Solids Handling				$2.1M
Buildings				$3.3M
Total				$70M
WWTP Total[3]	$191M[3]	$202M	$240M	$278M
Estimated Construction Cost	$ 227M	$238M	$260M	$297M

1. Estimated construction cost does not include escalation, planning, engineering, administration, permits, major utility relocations, disposal of contaminated soil or groundwater, land/easement purchases, unforeseen site conditions, and the like.
2. Cost of tertiary treatment for existing oxidation ditch effluent is optional, but included for purposes of comparison.
3. Range accounts for potential installation for tertiary treatment.

5.9 Recommended Urban Growth Boundary Alternative

A summary comparison of key considerations for Urban Growth Boundary Alternatives has been prepared as presented in Table 25 below. Key considerations include capital costs, operation and maintenance (O&M), property, recycled water, and effluent disposal. Capital cost considerations are quantified and the remaining considerations are rated as more favorable, neutral, or less favorable in the comparison of alternatives.

Table 25: Summary Comparison of Urban Growth Boundary Alternatives

	Centralized Alternatives		Decentralized Alternatives	
Alternative Components	**Alternative 1A**	**Alternative 1B**	**Alternative 2**	**Alternative 3**
Existing WWTP – Oxidation Ditch Expansion	**26.5 mgd**			
Existing WWTP – MBR Expansion		**26.5 mgd**	**14.1 mgd**	**14.1 mgd**
New East WWTP			**12.4 mgd**	**8.3 mgd**
New North WWTP				**4.1 mgd**
Considerations[1]				
Capital Costs				
Service Area Improvements	$36M	$36M	$20M	$19M
WWTP Construction	$191M	$202M	$240M	$278M
Admin/Engr	$80M	$83M	$91M	$104M
Land Acquisition/Easements	$10M	$10M	$11M	$12M
Total	**$317M**	**$331M**	**$362M**	**$413M**
Operation & Maintenance				
Annual costs	+	0	-	-
Common treatment technology	+	-	-	-
Positioned for advanced treatment	-	+	+	+
Property				
Permitting	0	0	-	-
WWTP acreage requirements	0	+	-	-
Recycled Water				
Title 22 Unrestricted Use readiness	-	+	+	+
Proximity to recycled water customers	-	-	+	+
Effluent Disposal				
Proximity to more Evap/Perc Ponds	0	0	0	0
Proximity to alternate discharge	-	-	0	0

1. + is more favorable, 0 is neutral, - is less favorable

The initial evaluation of Urban Growth Boundary Alternatives compared and selected the preferred centralized alternative (Alternative 1A or Alternative 1B) and the preferred decentralized alternative (Alternative 2 and Alternative 3). This evaluation is followed by a comparison and selection of the preferred Urban Boundary Growth alternative - centralized or decentralized.

5.9.1 Evaluation of Centralized Alternatives (Alternative 1A and Alternative 1B)

The major advantages of Alternative 1A compared to Alternative 1B are:

- Lower construction cost
- Lower annual O&M costs
- Common treatment technology

The major advantages of Alternative 1B are:

- Higher quality effluent
- Reduced property requirements
- Immediate Title 22 Unrestricted Use recycled water
  Better position for more restrictive discharge requirements in the future

The initial life-cycle cost advantage of Alternative 1A is expected to diminish with time as treatment requirements become more restrictive and the scarcity of water resources places a higher premium on reuse. Alternative 1B is preferred and recommended to optimize the use of the City's land and water resources.

5.9.2 Evaluation of Decentralized Alternatives (Alternative 2 and Alternative 3)

The advantages and disadvantages of Alternative 2 and Alternative 3 are similar.

The major advantage of Alternative 2 compared to Alternative 3 is:

- More cost effective (capital, O&M, and administration/management)

The major advantage of Alternative 3 is:

- The North WWTP has closer proximity for future inclusion of Madera Acres (currently outside the Urban Growth Boundary).

The additional capital cost, O&M cost, and administration/management of two decentralized WWTPs versus one decentralized WWTP favors Alternative 2. Consequently, Alternative 2 is recommended as the decentralized treatment alternative.

5.9.3 Evaluation of Centralized and Decentralized Alternatives (Alternative 1B and Alternative 2)

The major advantages of Alternative 1B compared to Alternative 2 are:

- Lower capital cost inherent with centralized facility.
- Lower O&M cost inherent with centralized facility.
- Permitting expansion at existing WWTP site less onerous than permitting a new WWTP site.
- Property acquisition /easements less difficult.

The major advantages of Alternative 2 are:

- Lower cost of collection system improvements to convey future wastewater to WWTP locations.
- Better proximity to some recycled water customers.

All of the advantages associated with Alternative 2 are related to reducing the cost of infrastructure to convey wastewater to WWTP locations or transmit treated effluent to recycled water customers. However, these infrastructure cost savings do not offset the higher capital cost, higher annual O&M cost, and property acquisition/permitting issues related to decentralized treatment facilities. Consequently, Alternative 1B is recommended as the basis serving the Urban Growth Boundary development.

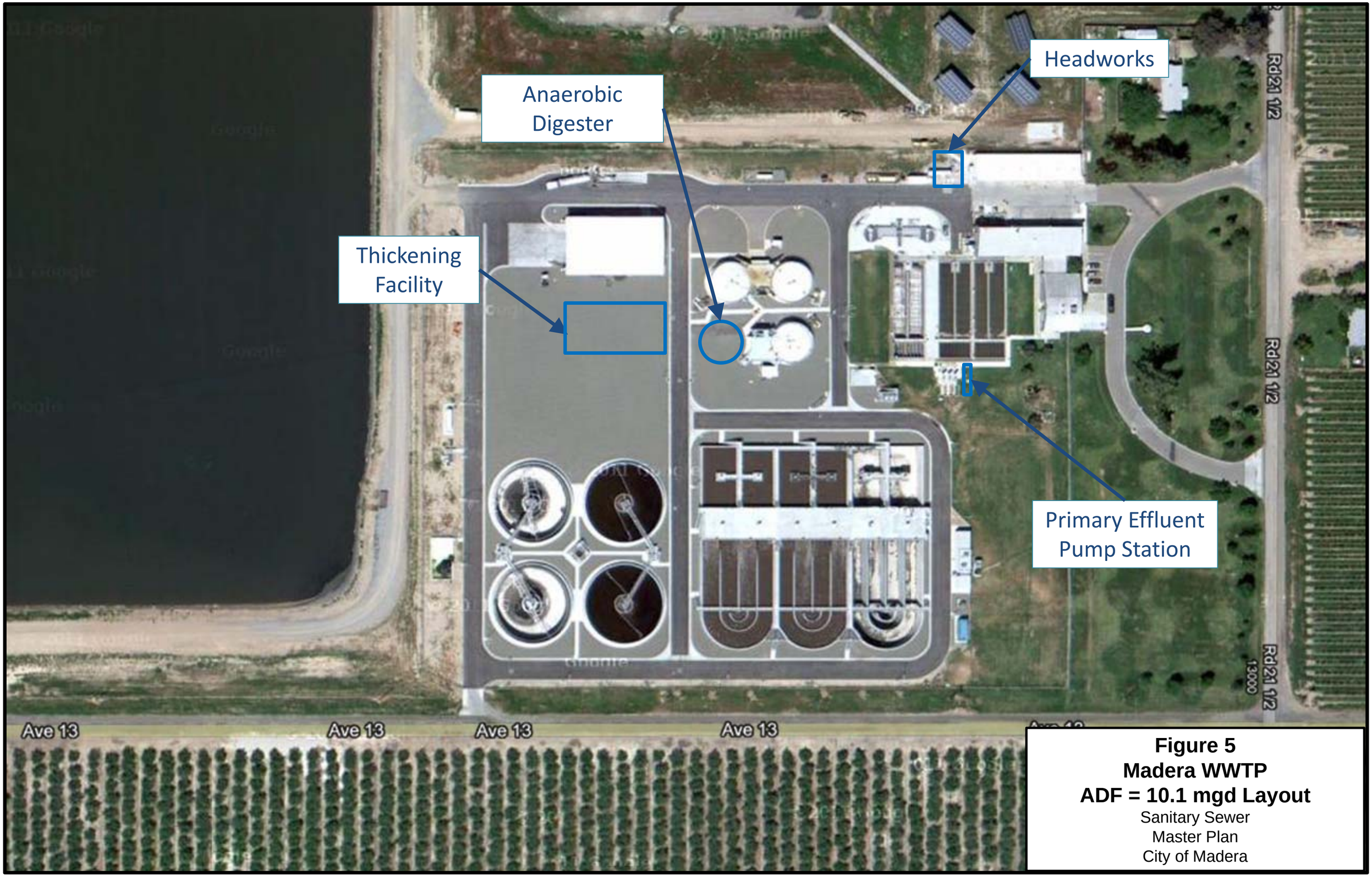
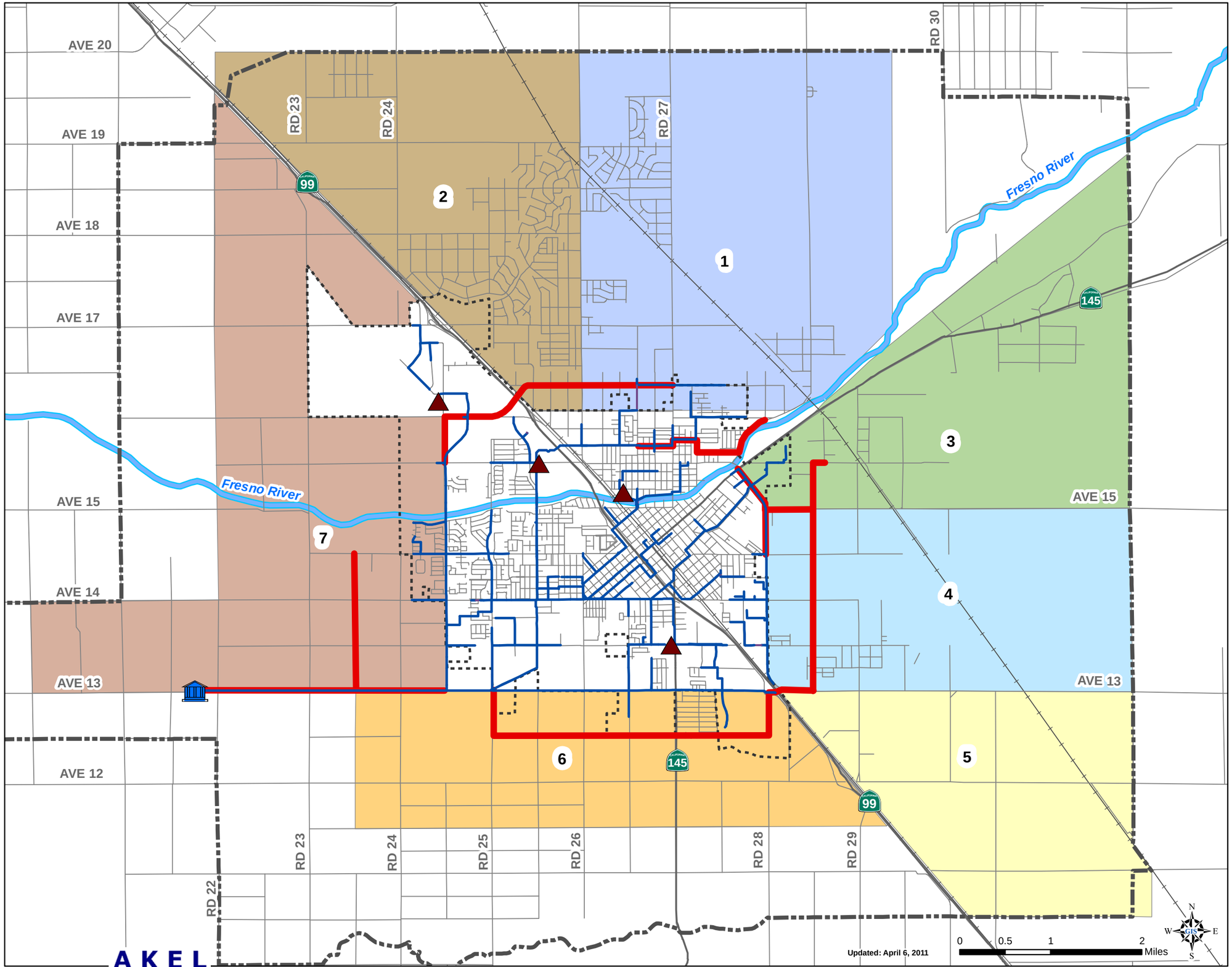


Figure 5
Madera WWTP
ADF = 10.1 mgd Layout
Sanitary Sewer
Master Plan
City of Madera



Legend

- Existing WWTP - 26.5 mgd
- Lift Stations

Modeled Pipes

Diameter

- 8" and Smaller
- 10" and Larger
- Proposed Pipes
- City Limits
- Planning Area Boundary
- Roads
- Railroads
- Fresno River

PRELIMINARY

Figure 6
Baseline Alternative 1
Existing WWTP
Sanitary Sewer
Master Plan Update
City of Madera

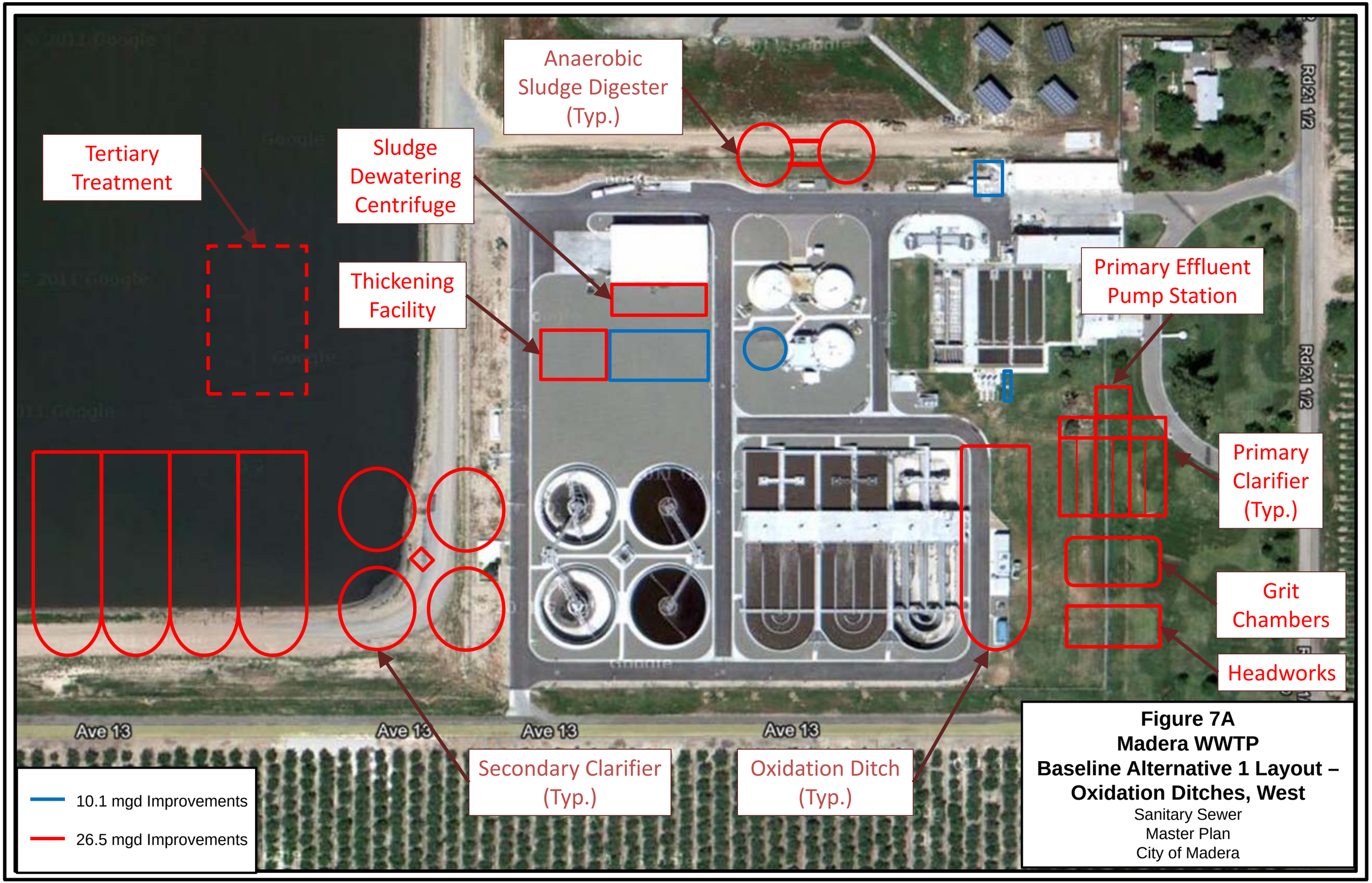


Figure 7A
Madera WWTP
Baseline Alternative 1 Layout – Oxidation Ditches, West
Sanitary Sewer
Master Plan
City of Madera

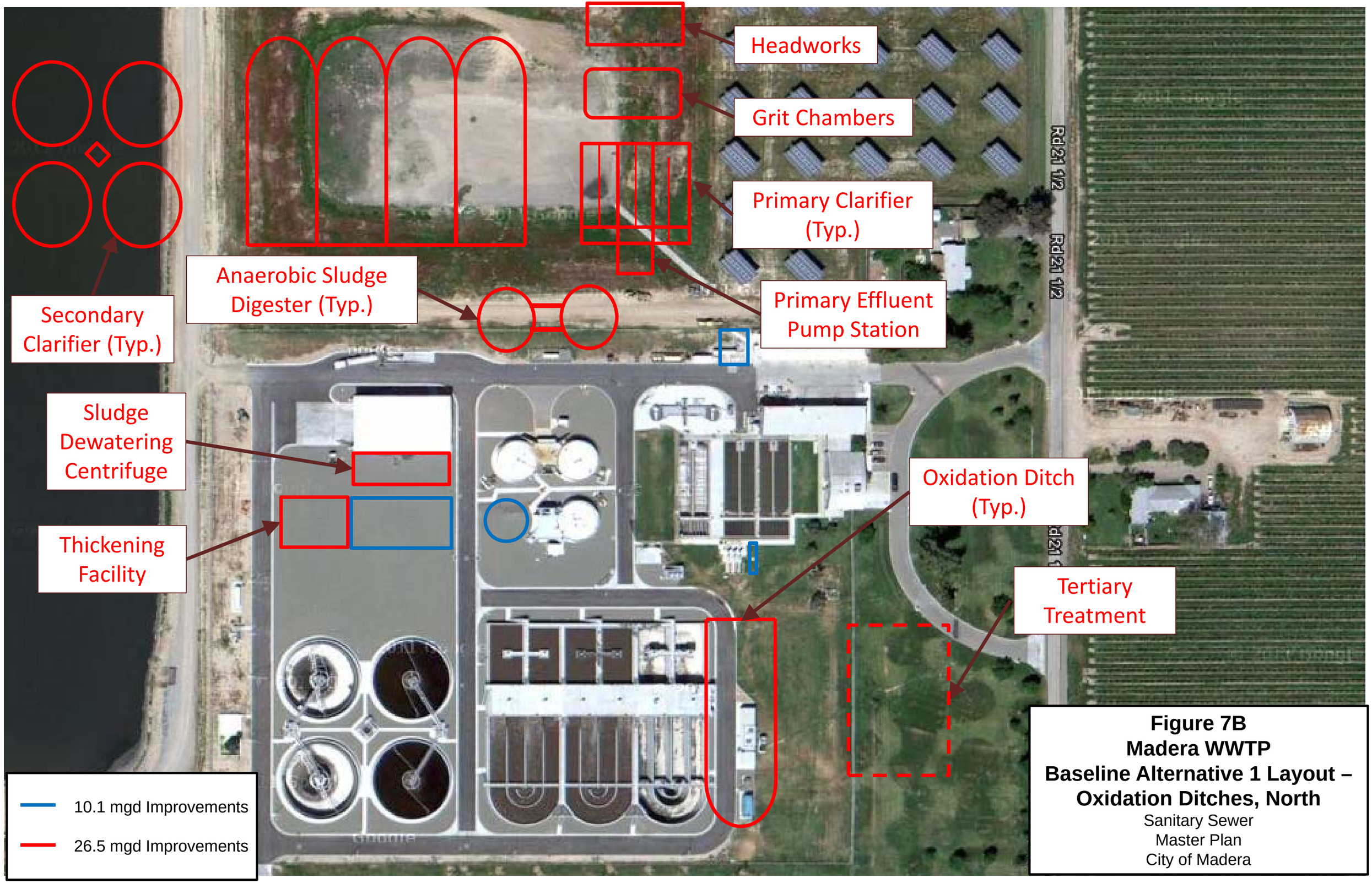


Figure 7B
Madera WWTP
Baseline Alternative 1 Layout – Oxidation Ditches, North
Sanitary Sewer
Master Plan
City of Madera

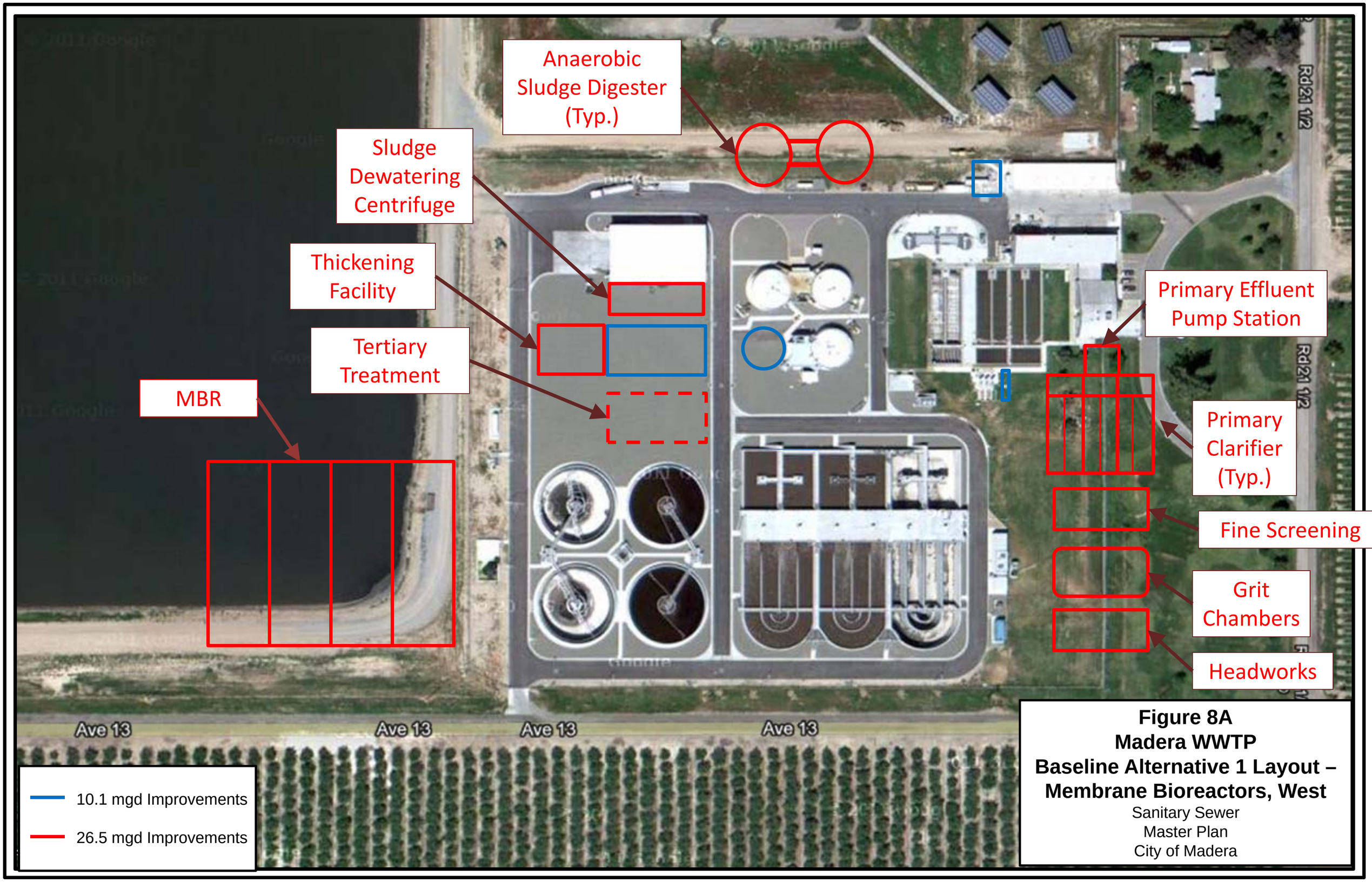


Figure 8A
Madera WWTP
Baseline Alternative 1 Layout – Membrane Bioreactors, West
Sanitary Sewer
Master Plan
City of Madera

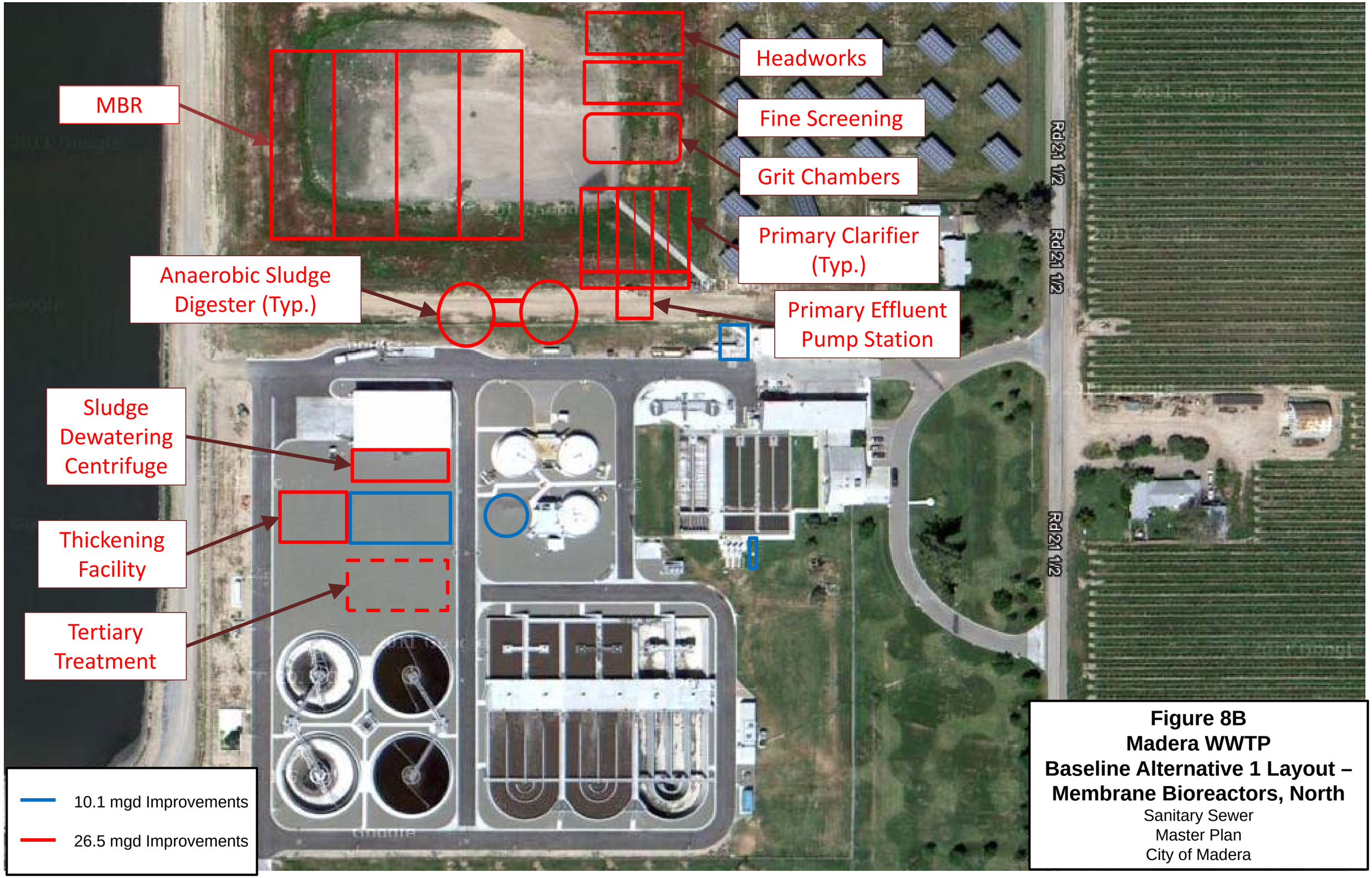
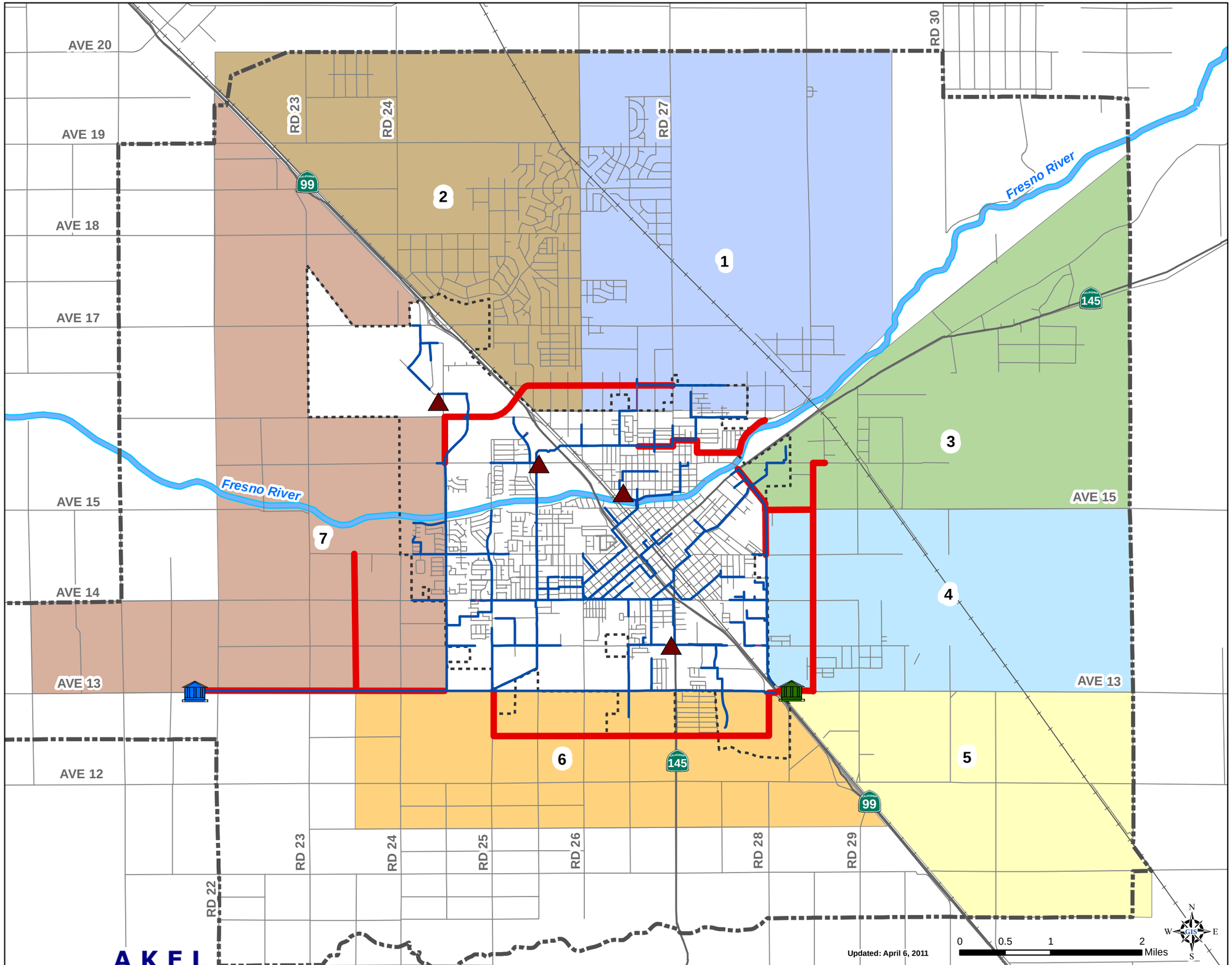


Figure 8B
Madera WWTP
Baseline Alternative 1 Layout – Membrane Bioreactors, North
Sanitary Sewer
Master Plan
City of Madera



Legend

- Existing WWTP - 14.1 mgd
- East WWTP - 12.4 mgd
- Lift Stations

Modeled Pipes

Diameter

- 8" and Smaller
- 10" and Larger
- Proposed Pipes
- City Limits
- Planning Area Boundary
- Roads
- Railroads
- Fresno River

PRELIMINARY

Figure 9
Alternative 2
Existing and East WWTP
Sanitary Sewer
Master Plan Update
City of Madera

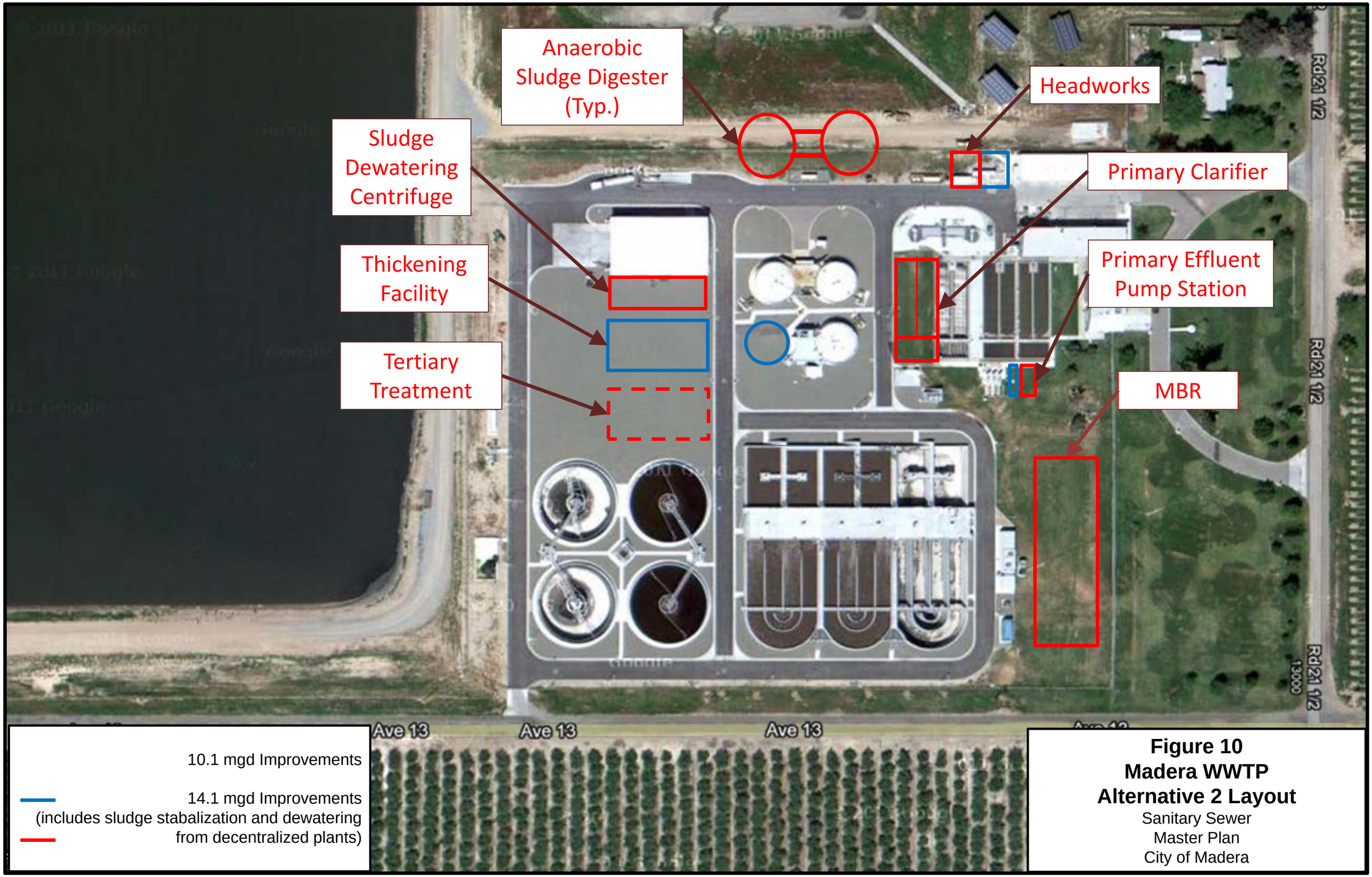


Figure 10
Madera WWTP
Alternative 2 Layout
Sanitary Sewer
Master Plan
City of Madera

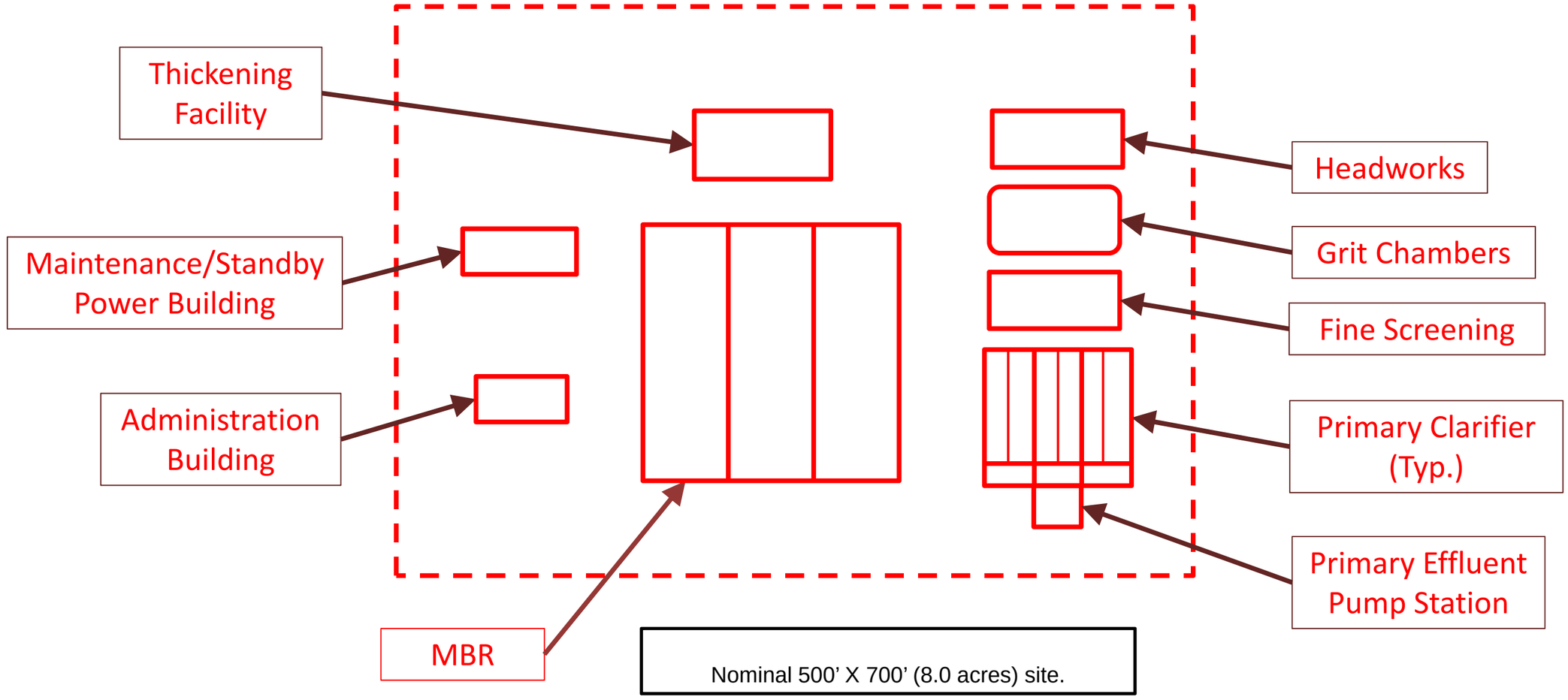
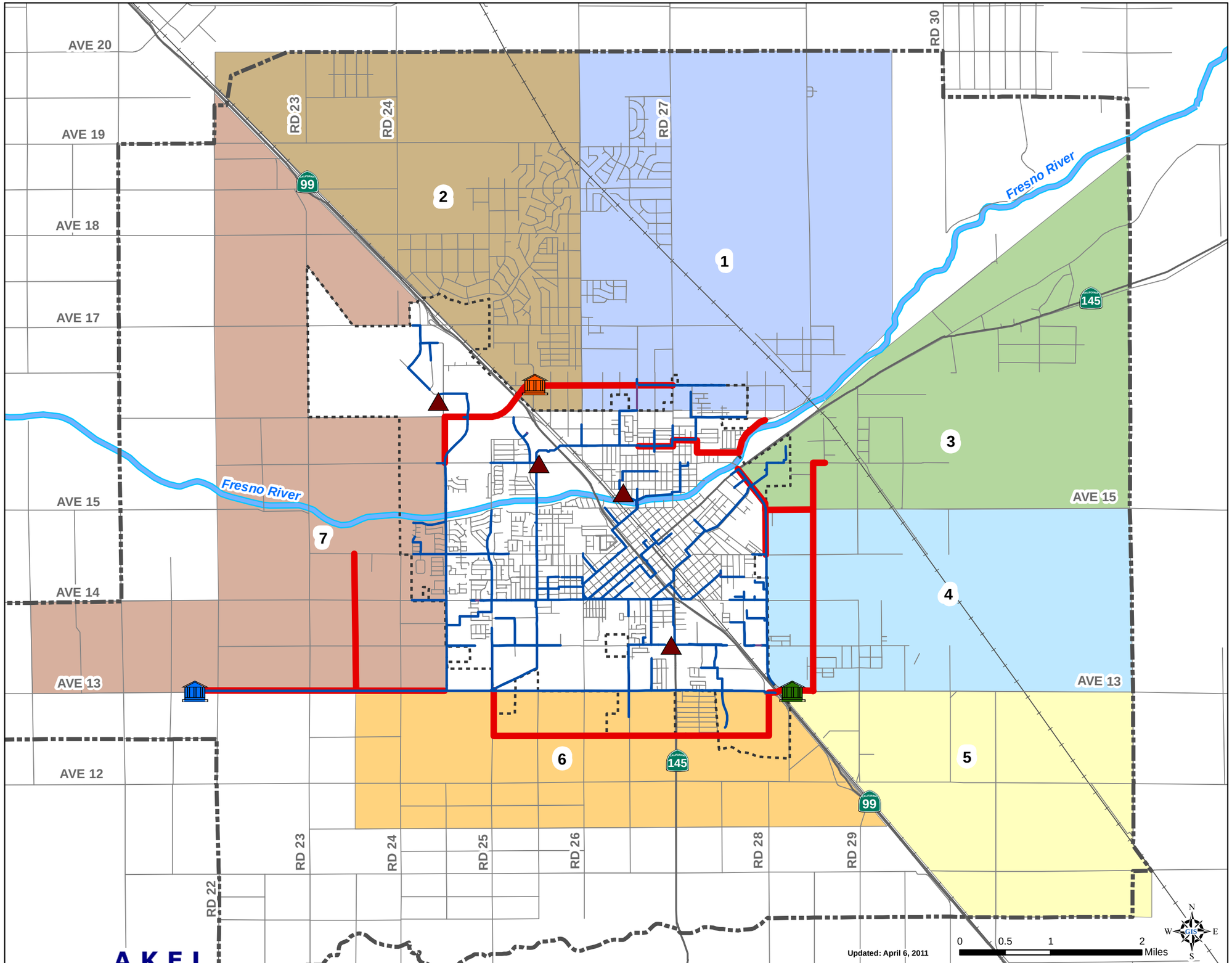


Figure 11
Madera WWTP
Alternative 2 –
12.4 mgd Decentralized, East
Sanitary Sewer
Master Plan
City of Madera



AKEL ENGINEERING GROUP, INC.

Legend

- Existing WWTP - 14.1 mgd
- East WWTP - 8.3 mgd
- North WWTP - 4.1 mgd
- Lift Stations

Modeled Pipes

Diameter

- 8" and Smaller
- 10" and Larger
- Proposed Pipes
- City Limits
- Planning Area Boundary
- Roads
- Railroads
- Fresno River

PRELIMINARY

Figure 12
Alternative 3
Existing, East, and North WWTP
Sanitary Sewer
Master Plan Update
City of Madera

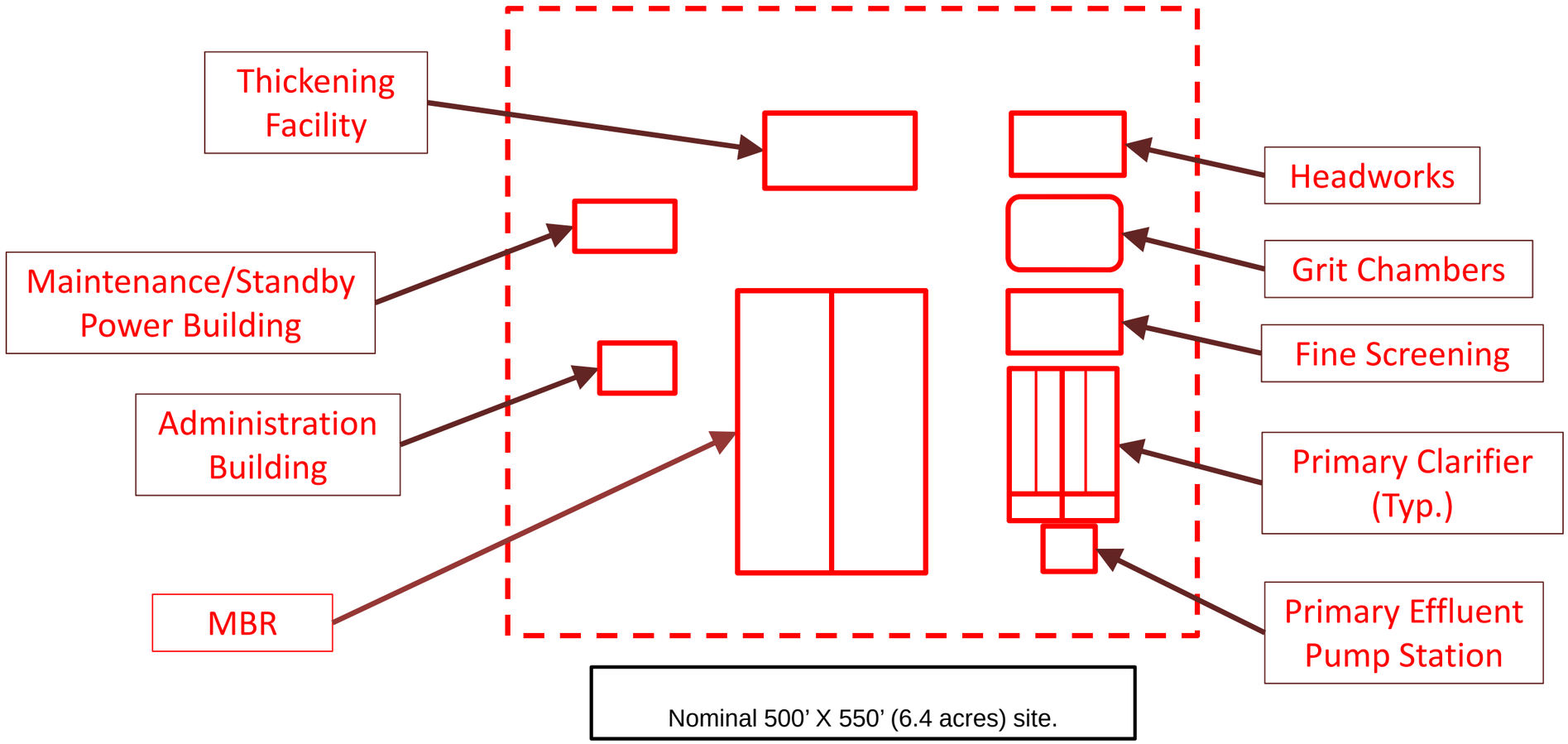


Figure 13A
Madera WWTP
Alternative 3 –
8.3 mgd Decentralized, East
Sanitary Sewer
Master Plan
City of Madera

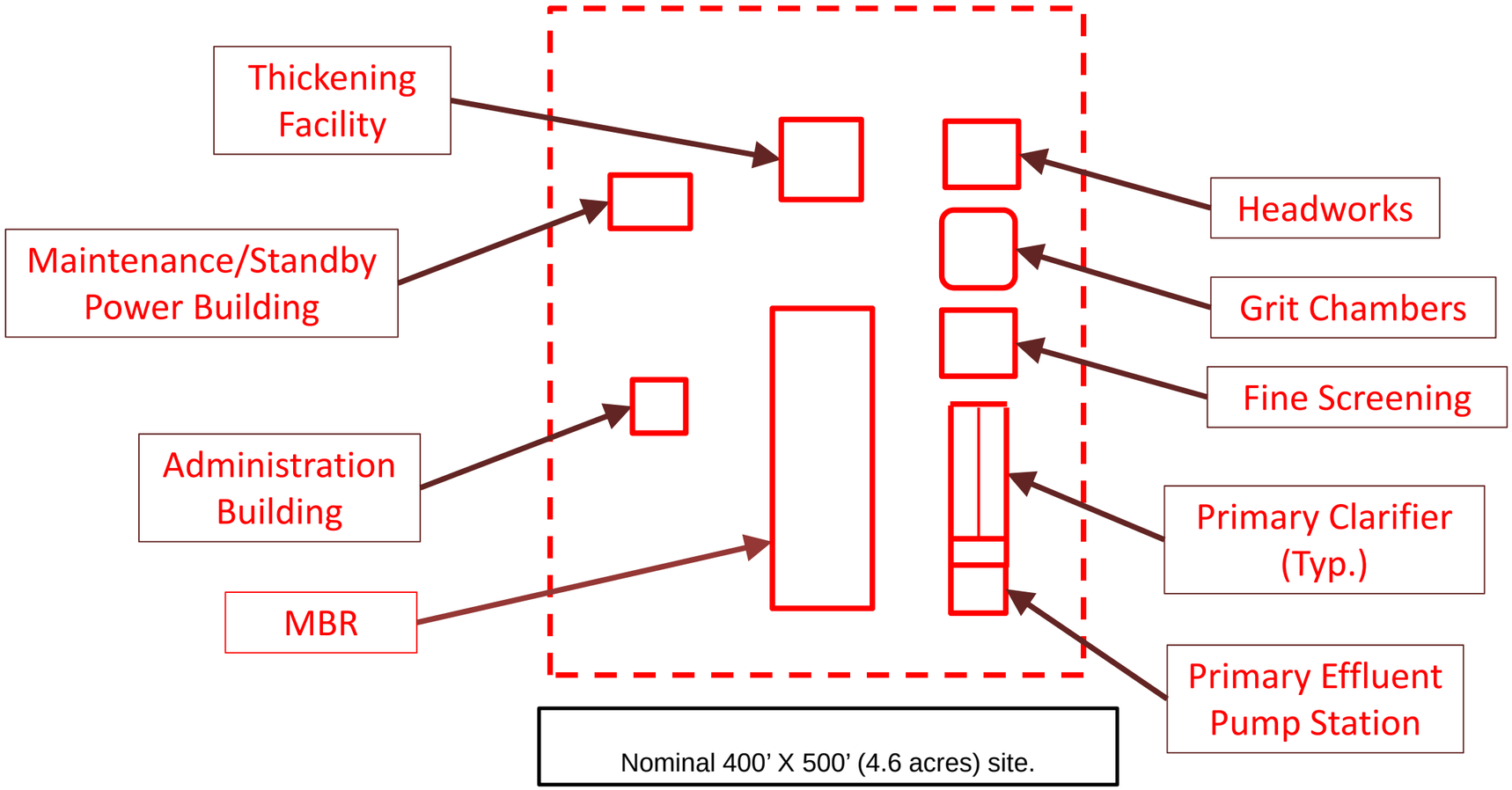


Figure 13B
Madera WWTP
Alternative 3 –
4.1 mgd Decentralized, North
Sanitary Sewer
Master Plan
City of Madera

ATTACHMENTS

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION

ORDER NO. 95-046

WASTE DISCHARGE REQUIREMENTS FOR CITY OF MADERA WASTEWATER TREATMENT FACILITY MADERA COUNTY

The California Regional Water Quality Control Board, Central Valley Region, (hereafter Board) finds that:

1. The City of Madera (hereafter Discharger) owns and operates a Wastewater Treatment Facility (WWTF). The WWTF is at Avenue 13 and Road 21-1/2 and serves the City and surrounding areas within the County of Madera's jurisdiction. The property (Assessor's Parcel Nos. 045-112-004, -005, -006, and 045-121-022) is owned by the Discharger.

2. The discharge is presently regulated by Waste Discharge Requirements (WDRs) Order No. 89-051, adopted by the Board on 31 March 1989. Order No. 89-051 is neither adequate nor consistent with current operations and plans and policies of the Board.

3. The design capacity of the WWTF is 7 million gallons per day (mgd). The average monthly flow is about 5 mgd. The WWTF consists of headworks; two primary clarifiers in parallel (Nos. 1 and 2), a back-up clarifier (No. 3); two redwood media trickling filters in parallel; two secondary clarifiers in parallel; two anaerobic sludge digesters in parallel; and four sludge drying beds. Each trickling filter is designed for an average hydraulic loading of 2,920 gallons/square feet/day and an average organic loading of 75 lbs BOD/1000 cubic feet/day. The filter recirculation ratio is 2:1.

4. Fourteen 20-acre evaporation-percolation ponds are available for wastewater disposal. Each pond is about three to five feet deep. The Discharger utilizes four evaporation-percolation ponds with a total surface area of 80 acres for wastewater disposal. The remaining ten ponds (200 acres) are kept dry and leased to a farmer for reclamation. The farmer plants several crops on the bottom of the subject ponds, including cotton, winter forage, corn (for cattle feed), and other crops not used for human consumption. Wastewater effluent is used to irrigate the crops via border irrigation. Every two years the dry ponds are replaced with those previously used for wastewater disposal to allow nutrient uptake by the crops. Crop yield is reportedly high. The dry ponds are hereafter referred to as the 'reclamation area'.

5. The produced sludge is used exclusively on-site. Sludge from the anaerobic digester is normally removed to the sludge drying beds before it is applied on-site as a soil amendment. However, the Discharger occasionally by-passes the drying beds and applies the digested sludge directly to the reclamation area and/or to pond levees and other open areas within the boundary of the WWTF.

6. The Discharger upgraded several components of the WWTF in spring 1993 by replacement of two raw sludge pumps, installation of new cavity pumps, installation of new effluent control valves for the trickling filters, and construction of a new back-up primary clarifier (No. 3).

7. The WWTF is in the south half of Section 25, T11S, R16E, MDB&M, and in the southwest 1/4 of Section 30, T11S, R17E, MDB&M, with surface water drainage toward the San Joaquin River between Friant Dam and Mendota Pool, as shown in Attachment A, which is attached hereto and part of this Order by reference. The site lies within the Madera Hydrologic Area (No. 545.20), as depicted on interagency hydrologic maps prepared by the Department of Water Resources in August 1986.

8. The soils in the area are sandy alluvium fan deposits with moderate permeability.

9. Depth to the uppermost ground water (unconfined) is about 130 feet below ground surface (bgs). According to the fall 1987 ground water contour map prepared by the Department of Water Resources, ground water in the area flows westerly and northwesterly.

10. An on-site well supplies water for rinsing WWTF equipment and landscape irrigation. The well was constructed at least 25 years ago and extends 230 feet bgs. The well log and construction details of the on-site well are not available. The well is about 60 feet north of the WWTF buildings and roughly 200 yards east of the evaporation-percolation ponds.

11. The electrical conductivity (EC) of ground water samples collected from the on-site well exceed the EC of the Madera public water supply, which ranges from 250 to 350 μmhos/cm. On 12 October 1994, staff collected two samples from the on-site well and field analyzed them for EC and pH. The average EC and pH were 900 μmhos/cm and 6.1, respectively. The employees at the WWTF use bottled water for drinking. The EC of wastewater effluent, prior to discharge to the evaporation-percolation ponds, ranges from 450 to 600 μmhos/cm.

12. The WWTF operates without a ground water monitoring network and ground water samples collected solely form the on-site well may not be representative of the ground water quality of the unconfined aquifer beneath the site.

13. Average precipitation and evaporation rates are 10 and 65 inches per year, respectively.

14. Pursuant to Title 23, California Code of Regulations (CCR), Section 2233, Publicly-Owned Treatment Works (POTWs) treating or designed to treat an average dry weather flow of 5 mgd or greater are required to have and enforce a Pretreatment Program.

15. The Discharger developed a Pretreatment Program in conformance with Code of Federal Regulations (CFR), Title 40, Part 403. The program was approved on 23 September 1983.

16. Currently, eight industrial users (IUs) discharge to the WWTF. The IUs are classified as non-categorical. The City issued pretreatment permits for the IUs in the early 80's and plans to update the permits in 1995. Below is a list of the existing IUs, types of industry and discharge, and the approximate wastewater discharge rates to the WWTF.

Industrial User	Industry	Discharge	Discharge Rate (gpd)
Georgia Pacific Corp. Nos.1 & 2	Box and paper manufacturer	Wash and boiler water	11, 400
Regency Thermographers	Film processor and Printer	Rinse water	9,000
Zarn Inc.	Plastic trash can manufacturer	Cooling water	1,000
Classic Rosters	Cashew and almond roster	Wash water	920
FMC Corporation	Machinery fabricator	Process and cooling water	869
Thomas Products	Vitamin manufacturer	Wash water	100
Florestone	Fiberglass manufacturer	Boiler water	15

17. Following a February 1992 staff pretreatment compliance inspection, the Board requested the Discharger to provide a technical report concerning the City's legal authority to implement its enforcement response plan. The Discharger submitted a technical report with regards to the City's legal authority. However, during a September 1993 inspection, Board staff determined that the Discharger did not adequately implement the monitoring and the enforcement response plan against IUs in noncompliance. The Discharger re-inspected the subject IUs in June 1994 and found the IUs in compliance with the City pretreatment permits.

18. On 27 September 1985, the Board adopted Special WDRs Order No. 85-244 to amend the pretreatment provisions of the WDRs adopted for the City of Madera WWTF and five other publicly owned WWTFs. Pretreatment provisions need updating to reflect subsequent changes in law.

19. The Board adopted a Water Quality Control Plan for the San Joaquin River Basin, Second Edition, (hereafter Basin Plan) which contains water quality objectives for all waters of the Basin. These requirements implement the Basin Plan.

20. The beneficial uses of the San Joaquin River between Friant Dam and Mendota Pool are municipal, industrial and agricultural supply; water-contact and noncontact water recreation; ground water recharge; fresh water habitat; warm water spawning; and preservation and enhancement of fish, wildlife, and other aquatic resources.

21. The beneficial uses of underlying ground water are domestic, industrial, and agricultural supply.

22. The proposed action is an update of previously adopted requirements and the volume and the character of the discharge remains the same. The permitted discharge is consistent with the antidegradation provisions of 40 CFR 131.12 and State Water Resources Control Board Resolution 68-16. The impact on water quality is insignificant. There is no evidence to indicate that the discharge will cause significant impacts on the beneficial uses of the underlying ground water most likely to be affected by the pollutants discharged.

23. Storm water at the WWTF is contained on-site, as there is no storm water discharge from the WWTF to a surface water of the United States. The Discharger is exempt from the storm water regulations promulgated pursuant to Section 402(p) of the Clean Water Act.

24. The California Department of Health Services (DHS) has established statewide reclamation criteria in Title 22, California Code of Regulations, Section 60301, et seq. (hereafter Title 22) for the use of reclaimed water and has developed guidelines for specific uses.

25. The Board consulted with the DHS and considered their recommendations regarding public health aspects for use of reclaimed water.

26. The action to update waste discharge requirements for this existing discharge is exempt from the provisions of the California Environmental Quality Act (CEQA) (Public Resources Code Section 21000, et seq.), in accordance with Title 14, CCR, Section 15301, of the California Water Code.

27. The Board has notified the Discharger and interested agencies and persons of its intent to prescribe waste discharge requirements for this discharge and has provided them with an opportunity for a public hearing and an opportunity to submit their written views and recommendations.

28. The Board, in a public meeting, heard and considered all comments pertaining to the discharge.

IT IS HEREBY ORDERED that Order No. 89-051 is rescinded and the City of Madera, its agents, successors, and assigns, in order to meet the provisions contained in Division 7 of the California Water Code and regulations adopted thereunder, shall comply with the following:

A. Discharge Prohibitions:

1. Discharge of wastes to surface waters or surface water drainage courses is prohibited.

2. Bypass or overflow of untreated or partially treated waste is prohibited.

3. Discharge of waste classified as 'hazardous' or 'designated', as defined in Sections 2521(a) and 2522(a) of Title 23, CCR, Section 2510, et seq., (Chapter 15) is prohibited.

4. Discharge of sludge classified as 'hazardous', as described in Title 22, CCR, Division 4.5, Chapter 11, Article 3, is prohibited.

B. Discharge Specifications:

1. The monthly average daily dry weather (May through October) discharge shall not exceed 7 million gallons.

2. Objectionable odors originating at this facility shall not be perceivable beyond the limits of the wastewater treatment and reclamation areas.

3. As a means of discerning compliance with Discharge Specification No. B.2, the dissolved oxygen content in the upper zone (1 foot) of wastewater in ponds shall not be less than 1.0 mg/l.

4. **The effluent from the treatment facility shall not exceed the following limits:**

Constituent	Units	Monthly Average	Daily Maximum
BOD_5[1]	mg/l	40	80
Suspended Solids	mg/l	40	80
Settleable Solids	ml/l	0.2	0.5

[1] 5-day, 20° Celsius biochemical oxygen demand.

5. Ponds shall not have a pH less than 6.5 or greater than 9.5.

6. Ponds shall be managed to prevent breeding of mosquitos. In particular:

   a. An erosion control program should assure that small coves and irregularities are not created around the perimeter of the water surface.

   b. Weeds shall be minimized through control of water depth, harvesting, or herbicides.

   c. Dead algae, vegetation, and debris shall not accumulate on the water surface.

7. Public contact with wastewater shall be precluded through such means as fences, signs, and other acceptable alternatives.

8. Ponds shall have sufficient capacity to accommodate allowable wastewater flow and design seasonal precipitation and ancillary inflow and infiltration during the nonirrigation season. Design seasonal precipitation shall be based on total annual precipitation using a return period of 100 years, distributed monthly in accordance with historical rainfall patterns. Freeboard shall never be less than two feet (measured vertically).

9. On or about 1 October of each year, available pond storage capacity shall at least equal the volume necessary to comply with Discharge Specification B.8 above.

C. Reclamation Area Specifications:

1. Reclaimed wastewater shall meet the criteria contained in Title 22, Division 4, CCR, Section 60301, et seq., and, if used in construction, the most current edition of "Guidelines for Use of Reclaimed Water for Construction Purposes," published by the California Department of Health Services.

2. The direct reuse of effluent shall be limited to construction purposes; and irrigation of crops limited to nonfood chain crops and animal feed crops not used to feed lactating dairy animals. Other uses for reclaimed wastewater shall be subject to the prior approval of the Executive Officer.

3. Application of reclaimed water to the reclamation area shall not exceed what is reasonably necessary for the crop, soil, and climate.

4. An adequate number of signs shall be posted indicating that reclaimed water is used for irrigation and is not safe for drinking.

5. No physical connection shall exist between reclaimed water piping and any domestic water supply or domestic well, or between reclaimed water piping and any irrigation well that does not have air gap or reduced pressure principle device.

6. Standing water shall not be present on the reclamation area 24 hours after each application has ceased.

7. The reclamation area shall be managed so as to prevent breeding of mosquitoes. More specifically:

   a. Tail water must be returned and the applied wastewater must infiltrate completely within a 48-hour period.

   b. Reclamation area shall be maintained free of floating vegetation.

   c. Low-pressure and unpressurized pipelines accessible to mosquitoes shall not be used to store wastewater.

8. Irrigation or impoundment of wastewater shall not occur within 100 feet of any domestic well or within 50 feet of any irrigation well unless it is demonstrated to the satisfaction of the Executive Officer that a shorter distance is justified.

D. Sludge Disposal Specifications:

1. Collected screenings, sludges, and other solids removed from liquid wastes shall be disposed of in a manner that is consistent with the requirements of Chapter 15 and approved by the Executive Officer.

2. Any proposed change in sludge use or disposal practice from disposal on the reclamation area shall be reported to the Executive Officer at least 90 days in advance of the change.

3. Use and disposal of sewage sludge shall comply with existing federal and state laws and regulations, including permitting requirements and technical standards included in 40 CFR 503.

   The State Water Resources Control Board (SWRCB) has not accepted primacy for implementing all regulations contained in 40 CFR 503. If the SWRCB assumes primacy, this Order may be reopened to incorporate appropriate time schedules and technical standards. Until then, the U.S Environmental Protection Agency (EPA) will regulate the Discharger for compliance with the standards and time schedules contained in 40 CFR 503, whether or not they have been incorporated into this Order.

4. Pollutant concentrations of sludge to be discharged within the reclamation area shall not exceed the following:

Pollutant	Concentration, mg/kg[1]
Arsenic	75
Cadmium	85
Chromium	3000[2]
Copper	4300[2]
Lead	840
Mercury	57[2]
Molybdenum	75
Nickel	420
Selenium	100
Zinc	7500[2]

[1] Dry weight basis

[2] Total Threshold Limit Concentrations (TTLC) prescribed in the California Code of Regulations are as follows: Chromium 2500 mg/kg, Copper 2500 mg/kg, Mercury 20 mg/kg, and Zinc 5000 mg/kg on a wet weight basis. Sludge sample which contains metals at or above the TTLC wet weight concentrations are defined as "hazardous" and may not be discharged under this Order. If any sludge sample contains chromium, copper, mercury or zinc in dry weight concentrations exceeding the TTLC wet weight concentrations, the sample concentration must be recalculated on a wet weight basis and compared to the TTLC values to verify whether the sludge is "hazardous".

5. The cumulative loading for pollutants at each sludge application site shall not exceed the following:

Pollutant	Cumulative Pollutant Loading Limits, Kg/ha
Arsenic	41
Cadmium	39
Chromium	3000
Copper	1500
Lead	300
Mercury	17
Molybdenum	18
Nickel	420
Selenium	100
Zinc	2800

6. Applied sludge shall be Class A or B with regard to pathogens, as described in 40 CFR 503.32(a) and (b), respectively.

7. Sludge application shall be limited to once per year, between crop plantings.

8. Sludge shall not be applied to soils with, or cause soils to have, a pH of less than 6.5.

9. Sludge shall not be staged or applied within 100 feet of domestic water supply wells or 50 feet of any irrigation well.

10. Sludge shall not be applied to wastewater treatment facility levees.

E. Ground Water Limitations:

The discharge, in combination with other sources, shall not cause underlying ground water to:

1. Contain waste constituents in concentrations statistically greater than receiving water limits, where specified below, or background water quality where not specified. (For purposes of comparison, background water quality shall be determined when background monitoring provides sufficient data. Quality determined in this manner establishes "water quality protection standards.")

2. Contain chemicals, heavy metals, or trace elements in concentrations that adversely affect beneficial uses or exceed maximum contaminant levels specified in the CCR, Division 4, Chapter 15.

3. Exceed a most probable number of total coliform organisms of 2.2/100 ml over any seven-day period.

4. Exceed concentrations of radionuclides specified in the CCR, Title 22, Division 4, Chapter 15.

5. Contain taste or odor-producing substances in concentrations that cause nuisance or adversely affect beneficial uses.

F. Provisions:

1. The Discharger shall comply with Monitoring and Reporting Program No. 95-046 , which is part of this Order, and any revisions thereto as ordered by the Executive Officer.

2. The Discharger shall comply with the "Standard Provisions and Reporting Requirements for Waste Discharge Requirements," dated 1 March 1991, which are attached hereto and by reference a part of this Order. This attachment and its individual paragraphs are commonly referenced as "Standard Provision(s)."

3. In the event of any change in control or ownership of land or waste discharge facilities described herein, the Discharger shall notify the succeeding owner or operator of the existence of this Order by letter, a copy of which shall be immediately forwarded to this office.

   To assume operation under this Order, the succeeding owner or operator must apply in writing to the Executive Officer requesting transfer of the Order. The request must contain the requesting entity's full legal name, the State of incorporation if a corporation, the name and address and telephone number of the persons responsible for contact with the Board, and a statement. The statement shall comply with the signatory paragraph of Standard Provision B.3 and state that the new owner or operator assumes full responsibility for compliance with this Order. failure to submit the request shall be considered a discharge without requirements, a violation of the California Water Code. Transfer shall be approved or disapproved by the Executive Officer.

4. At least 90 days prior to termination or expiration of any lease, contract, or agreement involving disposal or reclamation areas or off-site reuse of effluent, used to justify the capacity authorized herein and assure compliance with this Order, the Discharger shall notify the Board in writing of the situation and of what measures have been taken or are being taken to assure full compliance with this Order.

5. The Discharger shall comply with all pretreatment requirements contained in 40 CFR Part 403 and shall be subject to enforcement actions, penalties, fines, and other remedies by the U.S Environmental Protection Agency (EPA) or other appropriate parties, as provided in the Clean Water Act, as amended. The Discharger shall implement and enforce its approved POTW Pretreatment Program, which is hereby made an enforcement condition of these requirements. The U.S. EPA may initiate enforcement action against an industrial user for noncompliance with applicable standards and requirements as provided in the Clean Water Act.

6. The Discharger shall enforce the requirements promulgated under Sections 307(b), (c), (d), and 402(b) of the Clean Water Act. The Discharger shall cause industrial users subject to federal categorical standards to achieve compliance no later than the date specified in those requirements or, in the case of a new industrial user, upon commencement of the discharge.

7. The Discharger shall perform the pretreatment functions required in 40 CFR Part 403, including, but not limited to:

   a. Implementing the necessary legal authorities as provided in 40 CFR 403.8(f)(1);

   b. Enforcing the pretreatment requirements under 40 CFR 403.5 and 403.6;

   c. Implementing the programmatic functions as provided in 40 CFR 403.8(f)(2);

   d. Providing the requisite funding and personnel to implement the pretreatment program as provided in 40 CFR 403.8(f)(3); and

   e. Publishing a list of significant violators as required by 40 CFR 403.8(f)(2)(vii), where "significant violators" and "significant noncompliance" are as defined by EPA in Pretreatment Compliance Monitoring and Enforcement Guidance, PP 3-48 through 3.52.

8. The Discharger must comply with all conditions of this Order, including timely submittal of technical and monitoring reports as directed by the Executive Officer. Violations may result in enforcement action, including Regional Board or court orders requiring corrective action or imposing civil monetary liability, or in revision or rescission of this Order.

9. Special WDRs Order No. 85-244 is no longer applicable to this discharge, as the pretreatment terms of this Order supersede the requirements of Order No. 85-244.

10. A copy of this Order shall be kept at the WWTF for reference by operating personnel. Key operating personnel shall be familiar with its contents.

11. The Board will review this Order periodically and will revise requirements when necessary.

I, WILLIAM H. CROOKS, Executive Officer, do hereby certify the foregoing is a full, true, and correct copy of an Order adopted by the California Regional Water Quality Control Board, Central Valley Region, on 24 February 1995..

William H. Crooks

WILLIAM H. CROOKS, Executive Officer

RA:ra/fmc:2/14/95

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION

MONITORING AND REPORTING PROGRAM NO. 95-046
FOR
CITY OF MADERA
WASTEWATER TREATMENT FACILITY
MADERA COUNTY

Specific sample station locations shall be established with concurrence of the Board's staff, and a description of the stations shall be submitted to Board and attached to this Program.

INFLUENT MONITORING

Samples shall be collected at the headworks and should be representative of the influent for the period sampled. Time and date of collection of samples shall be recorded. Influent monitoring shall include at least the following:

Constituents	Units	Type of Sample	Sampling Frequency
Settleable Solids	ml/l	Grab	Daily
pH	pH units	Grab	Daily
Average Daily Flow	mgd	Continuous	Daily
BOD_5[1]	mg/l	8-hr Composite	Weekly
Total Suspended Solids	mg/l	8-hr Composite	Weekly

[1] Five-day, 20° Celsius biochemical oxygen demand.

EFFLUENT MONITORING

Effluent samples shall be collected just prior to discharge to the disposal facility and should be representative of the volume and nature of the discharge. Time of collection of composite and grab samples shall be recorded. Effluent monitoring shall include at least the following:

Constituents	Units	Type of Sample	Sampling Frequency[1]
Settleable Solids	ml/l	Grab	Daily
BOD_5[2]	mg/l	8-hr composite	Weekly
Total Suspended Solids	mg/l	8-hr composite	Weekly

Constituents	Units	Type of Sample	Sampling Frequency[1]
Specific Conductivity	μmhos/cm	8-hr composite	Weekly
Total Dissolved Solids	mg/l	8-hr composite	Monthly
Total Nitogen	mg/l	Grab	Quarterly
Kjeldahl Nitrogen	mg/l	Grab	Quarterly
Standard Minerals[3]	mg/l	Grab	Quarterly
Metals[4]	mg/l	8-hr composite	Quarterly

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1 If results of monitoring a pollutant appear to violate effluent limitations, but monitoring frequency is not sufficient to validate violation (e.g., the monthly average for BOD), or indicate a violation and potential upset of the treatment process (e.g., less than minimum D.O.), the frequency of sampling shall be increased to confirm the magnitude and duration of violation, if any, and aid in identification and resolution of the problem.

2 Five-day, 20° Celsius biochemical oxygen demand.

3 Standard mineral analyses shall include bicarbonate, calcium, carbonate, chloride, fluoride, iron, magnesium, nitrate, potassium, sodium, sulfate, and total phosphorous.

4 Metal analyses shall include aluminum, arsenic, barium, copper, cadmium, chromium, lead, mercury, molybdenum, selenium, silver, zinc, and nickel.

SLUDGE MONITORING

Annually, The Discharger shall collect and analyze composite representative sludge samples from the treatment facilities prior to final disposal. Sampling and analytical procedures shall be in accordance with the criteria specified in 22 CCR and by the U.S. EPA in POTW Sludge Sampling and Analysis Guidance Document, August 1989, and *test methods for Evaluating Solid Waste, Physical Chemical Methods (SW-846)*. Sludge samples shall be analyzed for the following total and soluble metal concentrations and other constituents:

	Constituent[1]	
pH	Chromium	Cadmium
Total Nitrogen	Cobalt	Nickel
Nitrate-Nitrogen	Copper	Selenium
Phosphorous	Lead	Silver
Potassium	Mercury	Zinc
Arsenic	Molybdenum	

[1] In metal analyses, ICAP may not be used for arsenic, mercury, and selenium.

By 28 February of each year, the Discharger shall submit a technical report including the analytical results, area of application and application rates, quantity of sludge disposed, and sludge disposal procedures relative to Department of Health Services' Manual of Good Practices for Landspreading of Sewage Sludge and EPA's *Process Design Manual for Land Application of Municipal Sludges* and Title 23, California Code of Regulations, Section 2511(f). If Sludge is applied to the reclamation area, the report shall also indicate types of crop grown.

Sampling records shall be retained for a minimum of five years. A log shall be kept of sludge quantities produced and of handling and disposal activities. The frequency of entries is discretionary; however, the log should be complete enough to serve as a basis for part of the annual report.

POND MONITORING

Pond water monitoring shall include the following:

Constituent	Unit	Type of Sample	Frequency
Dissolved Oxygen[1]	mg/l	Grab	Daily
pH	pH Units	Grab	Daily
Freeboard[2]	feet	Observation	Weekly

* See footnotes next page

[1] Samples shall be collected at a depth of one foot from each evaporation-percolation pond, opposite the inlet, and analyzed for dissolved oxygen. Samples shall be collected between 0800 and 0900 hours.

[2] Freeboard shall be monitored on the evaporation-percolation ponds in use to the nearest tenth foot.

Permanent markers shall be placed in the pond with calibration indicating the water level at design capacity and available operational freeboard.

In addition, the Discharger shall inspect the condition of the evaporation-percolation ponds once per week and write visual observation in a bound log book. Notations shall include observations of whether weeds are developing in the water or along the bank, and their location; whether dead algae, vegetation, scum, or debris are accumulating on the pond surface and their location; whether burrowing animals or insects are present; and the color of the pond (e.g., dark sparkling green, dull green, yellow, grey, tan, brown, etc.). A copy of the entries made in the log during each month shall be submitted along with the monitoring report the following month. Where the O&M manual indicates remedial action is necessary, the Discharger shall briefly explain in the transmittal what action has been taken or is scheduled to be taken.

WATER SUPPLY MONITORING

A sampling station shall be established where a representative sample of the Madera water supply can be obtained. Water supply monitoring shall include at least the following:

Constituents[1]	Units	Sampling Frequency
Electrical Conductivity @ 25°C	μmhos/cm	Quarterly
Standard Minerals[2]	mg/l	Annually

[1] Constituents shall be reported as a flow weighted average from a representative number of supply wells and the monitoring report shall include copies of supporting calculations.

[2] Standard mineral analyses shall include bicarbonate, calcium, carbonate, chloride, fluoride, iron, magnesium, manganese, nitrate, potassium, sodium, sulfate, total dissolved solids, and total phosphorous.

GROUND WATER MONITORING

By 1 May 1995, the Discharger shall submit a work plan for a ground water monitoring network consisting of one or more background monitoring wells and three or more downgradient wells. All well locations and construction features are subject to the prior approval of the Executive Officer. The well network must be sufficient to monitor potential impacts of the disposal operation on the uppermost ground water aquifer at the property boundaries. By (four months after approval of the ground water monitoring network), the Discharger shall implement the approved monitoring network.

The downgradient wells shall constitute "points of compliance (POC)". In conjunction with background monitoring, monitoring of POC's will enable one to determine the areal and vertical extent of noncompliance with water quality protection standards. The ground water surface elevation (in feet and hundredths, M.S.L.) in all wells shall be measured on a quarterly basis and used to determine the gradient and direction of ground water flow. This information shall be displayed on a water flow net diagram for the site. Water samples shall be collected from wells in the approved monitoring network an analyzed as follows:

Constituent	Units	Type of sample	Sampling Frequency
Total Coliform	MPN/100 ml	Grab	Monthly
Specific Electrical Conductance @ 25° C.	μmhos/cm	Grab	Quarterly
pH	pH units	Grab	Quarterly
Standard Minerals[1]	mg/l	Grab	Quarterly
Total Nitrogen	mg/l	Grab	Quarterly
Metals[2]	mg/l	Grab	Semi-annually

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1 Standard mineral analyses shall include bicarbonate, calcium, carbonate, chloride, fluoride, iron, magnesium, manganese, nitrate, potassium, sodium, sulfate, total dissolved solids, and total phosphorous.

2 Metal analyses shall include aluminum, arsenic, barium, cadmium, chromium, copper, lead, mercury, nickel, selenium, silver, and zinc.

If the Discharger, through a verification monitoring program, or the Board verifies that water quality protection standards have been exceeded at or beyond the POC's, the Discharger shall notify the Board, or acknowledge the Board's findings, and submit a technical report **within 90 days**. The report must contain a plan and time schedule for implementing a corrective action program designed to achieve compliance with water quality protection standards.

PRETREATMENT MONITORING

The Discharger shall submit annually a report to the Regional Board, with copies to EPA Region 9 and the State Board, describing the Discharger's pretreatment activities over the previous 12 months. In the event that the Discharger is not in compliance with any conditions or requirements of this Order, including noncompliance with pretreatment audit/compliance inspection requirements, the Discharger shall also include the reasons for noncompliance and state how and when the Discharger shall comply with such conditions and requirements. The annual report is due by **28 February 1995**.

In addition to the information required in the annual pretreatment report, the Discharger shall submit a **quarterly report within thirty days after the end of each calendar quarter.** The report shall contain, but not be limited to, the items listed in item E.7 of the Standard Provisions.

If none of the items in item E.7 exists, at a minimum a letter indicating that all industries are in compliance and no violations or changes to the pretreatment program have occurred during the quarter must be submitted. The information required in the fourth quarter report shall be included as part of the annual report.

Signed copies of the reports shall also be submitted to the EPA Regional Administrator and the State Board at the following addresses, or as advised in writing subsequent to adoption of this Order:

Regional Administrator
U. S. Environmental Protection Agency
Region 9
Water Management Division (W-5-2)
75 Hawthorne Street
San Francisco, CA 94105

Pretreatment Program Manager
Division of Water Quality
State Water Resources Control Board
P. O. Box 944213
Sacramento, CA 94244-2130

REPORTING

In reporting the monitoring data, the Discharger shall arrange the data in tabular form so that the date, the constituents, and the concentrations are readily discernible. The data shall be summarized in such a manner to illustrate clearly whether the Discharger complies with waste discharge requirements.

Monthly monitoring reports shall be submitted to the Board by the **20th day of the following month.** Quarterly monitoring results shall be submitted by the **20th day of the month following each calendar quarter.** Annual monitoring results shall be submitted by **31 January of each year.**

In reporting the monitoring data, the Discharger shall arrange the data in tabular form so that the date, the constituents, and the concentrations are readily discernible. The data shall be summarized in such a manner to illustrate clearly whether the discharge complies with waste discharge requirements. The highest daily maximum for the month, monthly and weekly averages, and medians, and removal efficiencies (%) for BOD and Suspended Solids, should be determined and recorded.

If the Discharger monitors any pollutant at the locations designated herein more frequently than is required by this Order, the results of such monitoring shall be included in the discharge monitoring report.

By **31 January of each year**, the Discharger shall submit a written report to the Executive Officer containing the following:

a. The names, titles, certificate grade (if any) and general responsibilities of persons operating and maintaining the wastewater treatment plant.

b. The names and telephone numbers of persons to contact regarding the plant for emergency and routine situations.

c. A certified statement of when the flow meter and other monitoring instruments and devices were last calibrated, including identification of who did the calibration (Standard Provision C.4).

d. A statement whether the current operation and maintenance manual, and contingency plan, reflect the wastewater treatment plant as currently constructed and operated, and the dates when these documents were last reviewed for adequacy.

The Discharger may also be requested to submit an annual report to the Board with tabular and graphical summaries of the monitoring data obtained during the previous year. Any such request shall be made in writing. The report shall discuss the corrective actions taken and planed to bring the discharge into full compliance with the waste discharge requirements.

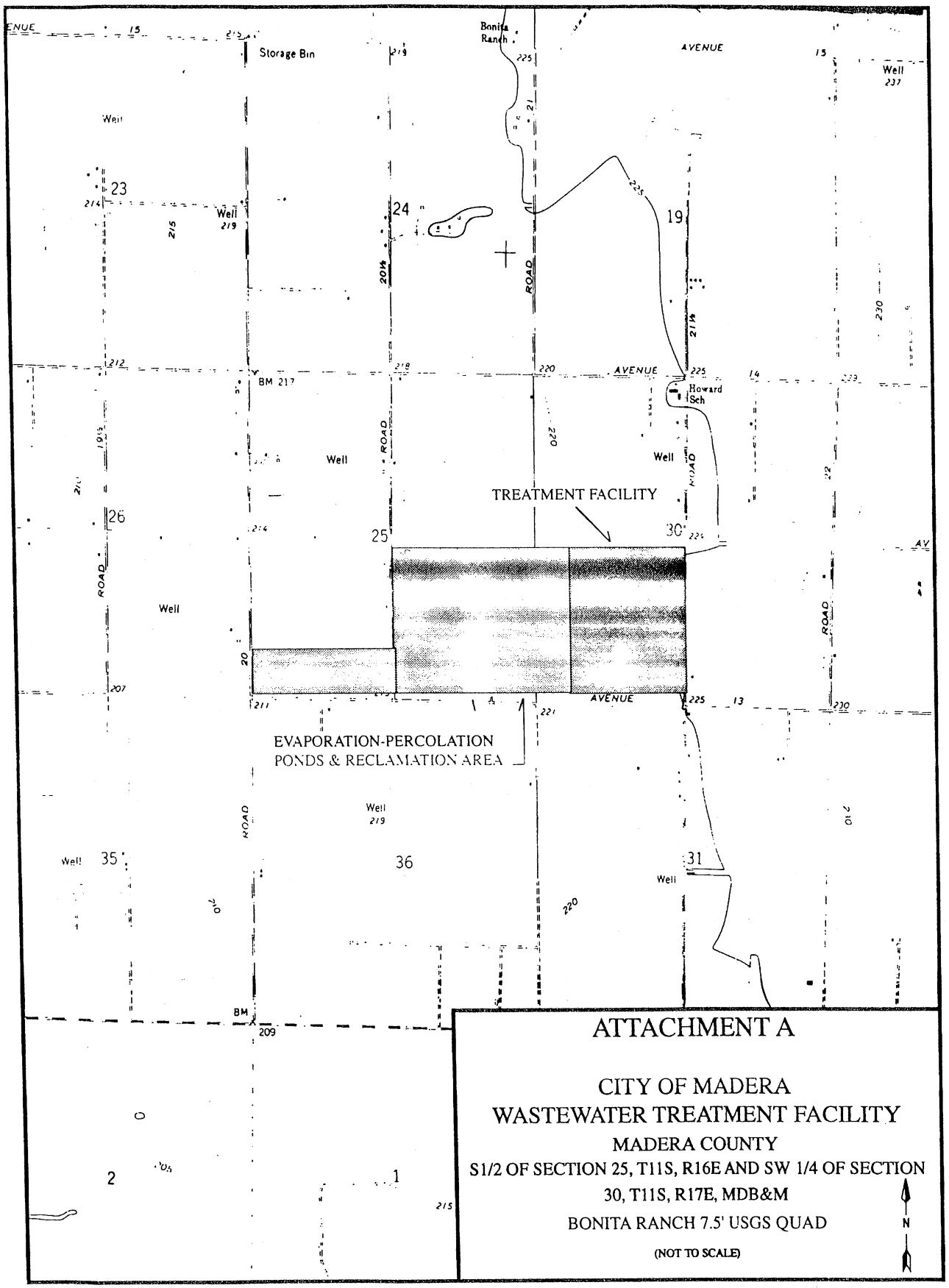
All reports submitted in response to this Order shall comply with the signatory requirements in Standard Provision B.3.

The Discharger shall implement the above monitoring program on the first day of the month following adoption of this Order.

Ordered by: William H. Crooks
WILLIAM H. CROOKS, Executive Officer

24 February 1995
(Date)

RA:ra/fmc



ATTACHMENT A

CITY OF MADERA
WASTEWATER TREATMENT FACILITY
MADERA COUNTY
S1/2 OF SECTION 25, T11S, R16E AND SW 1/4 OF SECTION 30, T11S, R17E, MDB&M
BONITA RANCH 7.5' USGS QUAD

(NOT TO SCALE)

N

INFORMATION SHEET

CITY OF MADERA
WASTEWATER TREATMENT FACILITY
MADERA COUNTY

The City of Madera (hereafter Discharger) owns and operates a Wastewater Treatment Facility (WWTF). The existing Waste Discharge Requirements (WDRs), Order No. 89-051 is neither adequate nor consistent with current operation and plans and policies of the Board.

The WWTF is a secondary treatment system using redwood media trickling filters. Effluent from the WWTF is discharged to four evaporation-percolation ponds with a total surface area of 80 acres. Ten other evaporation-percolation ponds (200 acres) are kept dry and leased to a farmer for reclamation. The farmer plants several crops on the bottom of the subject ponds. Wastewater effluent is used to irrigate the crops via border irrigation. Every two years the use of dry ponds is rotated with those previously used for wastewater disposal to allow nutrient uptake by the crops. The dry ponds are referred to as the 'reclamation area'.

Title 23, California Code of Regulations (CCR), Section 2233, requires that Publicly owned treatment works treating or designed to treat and average dry weather flow of 5 mgd or greater to have and enforce a Pretreatment Program. The Discharger developed a Pretreatment Program in conformance with Code of Federal Regulations (CFR), Title 40, Part 403. The program was approved on 23 September 1983.

Eight industrial users (IUs) discharge to the WWTF. The City issued pretreatment permits for the IUs in early 1983 and plans to update the permits in 1995. In June 1994, the Discharger inspected the IUs and determined the IUs in compliance with the City pretreatment permits.

The generated sludge is used on-site as soil amendment. Sludge from the anaerobic digesters is normally applied on the sludge drying beds, before it is applied on-site as soil amendment and/or fertilizer supplement on the reclamation area. Discharges by the IUs may elevate pollutant concentrations in the sludge. Currently, the Discharger is not required to test the sludge. This Order requires the Discharger to annually collect and analyze composite sludge samples and submit a technical report that includes analytical results and other information.

The WDRs incorporate 40 CFR Part 503, *Standards for the use or Disposal of Sewage Sludge*, promulgated by EPA on 19 February 1993. Some restrictions from the Department of Health Services (DHS) *Manual of Good Practice for Landspreading of Sewage Sludge* are also included. The WDRs require that the life time sludge loading rates are limited by the allowable cumulative heavy metals loadings established by Table 2 of 40 CFR 503.13. Metals concentrations in the sludge may not exceed those specified in Table 1 of 40 CFR 503.13.

Depth to the uppermost ground water is about 130 feet below ground surface (bgs). Ground water occurs in an unconfined zone and flows westerly and northwesterly.

CITY OF MADERA
WASTEWATER TREATMENT FACILITY
MADERA COUNTY

An on-site well supplies water for rinsing WWTF equipment and landscape irrigation. The well was constructed at least 25 years ago and extends 230 feet bgs and is roughly 200 yards east of the evaporation-percolation ponds. The electrical conductivity (EC) of ground water samples collected from the on-site well exceeds the EC of the Madera public water supply and the effluent wastewater. The EC of the Madera public water supply ranges from 250 to 300 μmhos/cm. On 12 October 1994, staff collected ground water samples from the on-site well. The average EC and pH were 900 μmhos/cm and 6.1, respectively. Effluent EC ranges from 450 to 600 μmhos/cm.

The well log and the construction detail of the on-site well are not available. The WWTF operates without a ground water monitoring network and ground water samples collected solely from the on-site well may not be representative of the ground water quality of the unconfined aquifer. This Order requires the Discharger to implement a ground water monitoring network consisting of one or two upgradient wells and three or more downgradient wells.

Soils in the area are sandy alluvium fan deposits with moderate permeability.

Average precipitation and evaporation rates are 10 and 65 inches per year, respectively

The action to update waste discharge requirements for this existing discharge is exempt from the provisions of the California Environmental Quality Act (CEQA), in accordance with Title 14, CCR, Section 15301.