CITY OF GREENFIELD
WASTEWATER MASTER PLAN
July 2016

Greenfield
Where Historic El Camino Real
Meets Monterey Wine Country

<u>City Council</u>
Mayor John Huerta, Jr.
Mayor Pro-Tempore Raul Rodriguez
Council Member Leah Santibañez
Council Member Avelina Torres
Council Member Lance Walker

REGISTERED PROFESSIONAL ENGINEER
STEVEN G. TANAKA
C049779
CIVIL
STATE OF CALIFORNIA

Prepared By:

Steven G. Tanaka, P.E. 49779
Principal Civil Engineer

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List of Tables and Figures

List of Acronyms

ADF	Average Daily Flow
CIP	Capital Improvement Projects
City	City of Greenfield
County	Monterey County
d/D	Depth over Diameter
ENR	Engineering New Record
FOG	fats, oil, and grease
FPS	Feet per Second
Ft	Feet
Ft/Sec	Feet per Second
GIS	Geographic Information System
GPD	Gallons Per Day
GPM	Gallons Per Minute
HDPE	High Density Polyethylene
I/I	Infiltration and Inflow
LF	Linear Feet
MDDWF	Maximum Day Dry Weather Flow
MGD	Million Gallons Per Day
min	Minute
NA	Not Applicable
NAD	North American Datum
NAVD	North American Vertical Datum
O&M	Operation and Maintenance
P.E.	Professional Engineer
PF	Peaking Factor
PHDWF	Peak Hour Dry Weather Flow
PVC	Polyvinyl Chloride
S.F.	Square Foot
SCADA	Supervisory Control and Data Acquisition
VCP	Vitrified Clay Pipe
WWMP	Wastewater Master Plan

List of References

1. California Code of Regulations, Title 22.
2. City of Greenfield, 2005-2025 Wastewater System Capital Improvement Plan Update and Capacity Charge Study, June 2005
3. City of Greenfield, 2008 Update of the Wastewater System Capital Improvement Plan Update and Capacity Charge Study, July 2008
4. City of Greenfield, Sewer System Management Plan, October 2012
5. US^3 Flow Monitoring, September and October, 2015
6. Progress GIS Files of the Sewer Collection System
7. Greenfield Population – Historical and Projected, 2005 General Plan
8. McGraw Hill ENR Construction Cost Index of 10242 (March 2016)
9. Metcalf & Eddy design handbook "Wastewater Engineering, Treatment and Reuse, Fifth Edition" 2014
10. Personal Communication with Mic Steinman, Community Services Director and Arturo Felix, Public Works Manager
11. Smith and Loveless Engineering Orders for Lift Stations

1: Introduction

The City of Greenfield (City) is responsible for the maintenance and operation of the sewer collection system and wastewater treatment facilities serving the residences and businesses in the City. As older infrastructure is replaced and new development projects are constructed, it is the City's goal to construct sewer collection system and treatment improvements to meet the current and ultimate needs of the City. In order to facilitate this goal, and to adequately plan for the capital resources needed to meet this goal, the City commissioned a comprehensive Wastewater Master Plan (Plan or WWMP) that evaluates all aspects of the wastewater collection and treatment system and its ability to meet current and long-term needs of the City.

Purpose of the Project

Preparation of the Plan will assist the City in prioritizing both current and future wastewater needs and set forth a mechanism for addressing those needs. The Plan does the following:

1. Addresses existing deficiencies within the sewer collection system based on today's standards and requirements;
2. Addresses deficiencies within the sewer collection system to meet future build-out needs;
3. Updates the prior 2013 wastewater evaluation and identifies improvements needed at the wastewater treatment plant; and
4. Provides a prioritized list of recommendations with associated hard and soft costs to complete the projects.

Environmental Review

In accordance with Title 14, California Code of Regulations, Chapter 3, Article 18 (Statutory Exemptions), this Wastewater Master Plan is considered a planning study and therefore adoption of this document is exempt from the requirements to prepare Environmental Impact Reports (EIR) or Negative Declarations (ND).

Authorization and Scope of Work

On May 13, 2015, the City authorized Wallace Group to prepare a comprehensive Wastewater Master Plan. This WWMP was prepared in accordance with Wallace Group's proposal dated April 10, 2015. A summarized scope of work is as follows:

1. **Kick-Off Meeting, Project Review Meetings, Field Reviews and Operation Staff Interviews:** Coordinate and attend a kick-off meeting with key Team members and City staff, including interviews with the City's operations staff and an initial field investigation of the City's lift stations to understand layouts and system operations.
2. **Existing Data Collection:** Develop an information database from existing planning reports, documents, maps, existing system flows, and population growth projections. Review City wastewater data, maintenance records, and meet with City staff to identify areas of concern

(high maintenance areas, or HMAs) regarding sewer mains (both gravity and force) and lift stations.

3. **Preliminary Findings Memorandum:** Prepare a description and general inventory of the sanitary sewer system based on review of plans, reports, studies, and other City records, visits with staff and field inspections. Visit and document accessible existing facilities and prepare an accurate, up-to-date description of the system. Include existing collection, pumping, and treatment system, including facilities, conditions, and processes; document existing wastewater treatment plant design conditions and criteria; document capital improvements and system expansions completed over the past 10-20 years based on record drawings and other detailed information provided by City staff; and document compliance requirements for California Regional Water Quality Control Board, Central Coast Region, Waste Discharge Requirements Order No. R-3-2002-0062.

   Sewer Model Development and Calibration: Model and evaluate the existing sewer collection system to determine areas of deficiency including proper design flows and cavitation at lift stations. Document existing wastewater flows and projections of future requirements; based on historical wastewater consumption and population, land use, and economic growth projections, quantify sanitary flow and wastewater demand requirements; use infiltration/inflow characteristics from the existing system and accepted values for new construction, groundwater infiltration, and rainfall flow factors to develop infiltration/inflow values and wastewater demands for future requirements. Review “hot spots” or high maintenance areas (HMAs) with City staff, and including prior SSO reports, and collectively (with City staff) recommend specific areas for CCTV video by the City. Conduct in-line flow monitoring at select locations to evaluate wastewater flow trends in the collection system.

4. **Lift Station Evaluation:** Inspect all pump stations; inventory capabilities of each facility; and collect relevant as-built plans, maintenance records, pump curves, and run logs. Inspection of the existing lift stations will be limited to visual observation of overall conditions of the lift station pumps, wet well and visible piping. **Manhole Inspection:** Evaluate the condition of up to 5 typical problem area manholes identified by the City Staff. Develop general recommendations for sewer manhole rehabilitation, coating and/or replacement based on these observations, and make recommendations for on-going inspection of sewer manholes by City staff. Budgetary level costs will be included in the wastewater master plan as part of the recommended capital improvement program (CIP).

5. **Wastewater Treatment Plant Analysis:** Analyze Wastewater Treatment Plant Capacity Requirements. Utilize present and future flow information to determine capacity requirements to meet future needs and identify capital and system improvements and expansions to meet future wastewater flow demands and needs. Build upon the work already completed, the 2013 report entitled “City of Greenfield Wastewater Treatment Plant Evaluation”, and update this Report based on prior work completed since publication of this referenced report, including:
   - Update to State Board WWTP Re-Classification (from Class 3 to Class 2 plant);

- Current status of oxidation pond aeration improvements, currently under design by Wallace Group.
- Updates to current regulatory status, and future regulatory considerations, with the Regional Water Quality Control Board. We will include recent correspondence with Tom Kukol, Region 3 RWQCB, and Nicki Fowler, Monterey County Environmental Health Department relative to recent and on-going odor complaints received in the area of the treatment plant.
- Near-term WWTP improvements needed to support the 2.0 mgd design capacity that will be achieved when the 90 HP aeration project is completed. These recommendations will include improvements to sludge digestion, handling and drying; stormwater management practices on the WWTP premises; headworks improvements; facility/laboratory building upgrades.
- Long-term wastewater treatment considerations including general recommendations for the WWTP for wastewater flows beyond 2 mgd and through the 20-year planning horizon.

Develop Capital Improvement Program: Using data collected during Research and Field Investigations, develop a Wastewater Capital Improvement Program recommending improvements necessary to maintain a desired level of service for the City's wastewater assets such as mainlines, manholes, lift pump stations and wastewater treatment facilities.

6. **Staffing Recommendations:** Provide recommendations for improvements to the organizational structure of the Wastewater Treatment and Collection System staff, including suggestions for improvements to the City's general approach to operation of the system. Build upon prior work conducted for the City related to wastewater system staffing needs and requirements based on State of California Operator Certification requirements.

7. **Regulatory Update:** Identify present and future regulatory concerns for the treatment facilities and sewer collection system.

Acknowledgements

Wallace Group thanks and gratefully acknowledges the following for their efforts, involvements, input and assistance in preparing this Sewer Collection System Master Plan:

City of Greenfield City Council:

Mayor John Huerta, Jr.
Mayor Pro-Tempore Raul Rodriguez
Council Member Leah Santibanez
Council Member Avelina Torres
Council Member Lance Walker

City of Greenfield City Staff:

Susan Stanton	City Manager
Doug Pike (MNS)	City Engineer
Mic Steinmann	Acting Community Development Director
Arturo Felix	Public Works Utilities Manager
Carmen Lorenzana	Public Works Administrative Assistant

The following Wallace Group key team members were involved in the preparation of this Water Master Plan:

Steven G. Tanaka, PE	Principal Civil Engineer
Kari Wagner, PE	Director of Water Resources
Valerie Huff, PE	Senior Civil Engineer
Kyle Anderson, PE	Civil Engineer
Jeff LeNay	GIS Specialist

2: Sewer Collection System Overview

Chapter 2 describes the features of the City's sewer collection system. The details regarding the various sewer collection system features are then presented in subsequent chapters.

Sewer Collection System Background

The City owns and operates a sewer collection system that is comprised of approximately thirty-one miles of gravity sewer pipes ranging in size from 4-inch to 24-inch diameter, and six lift stations. The sewer collection system spans over 2.1 square miles to serve the City's 3,700 customers. For the purposes of this master plan, only those trunk main sewer lines that were modeled, were included in this exhibit. The existing (modeled) sewer collection system is shown in Figure 2-1. An inventory of existing sewer pipe diameters and materials that were analyzed/modeled for this master plan are provided in Tables 2-1 and 2-2.

Table 2-1 Modeled Pipeline Inventory by Material

Material	Length	
	Feet	Miles
ACP	2,414	0.5
HDPE	8,339	1.6
PVC	26,986	5.1
VCP	17,348	3.3

Table 2-2 Modeled Pipeline Inventory by Diameter

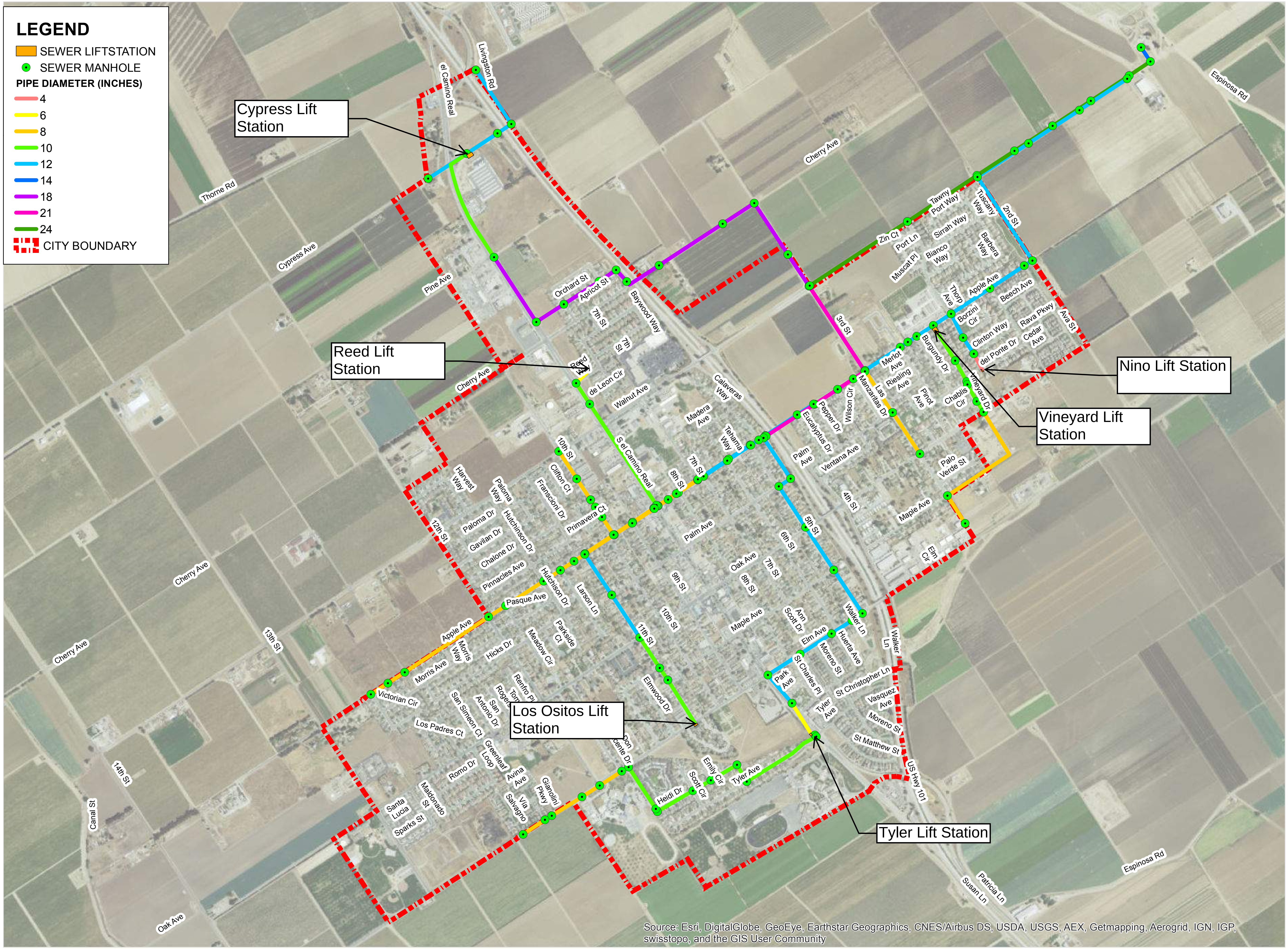
Diameter	Length	
	Feet	Miles
8	13,334	2.5
10	9,483	1.8
12	17,886	3.4
14	225	0.0
18	5,820	1.1
21	2,946	0.6
24	5,394	1.0

Lift Stations

The City owns six (6) lift stations (all Smith & Loveless wetpit/drypit lift stations) located throughout the collection system which are shown in Figure 2-1 and are briefly summarized in this chapter. Lift station tributary areas are shown on Figure 2-2.

- <u>Tyler Lift Station:</u> Tyler Lift Station is located at the intersection of El Camino Real and Tyler Avenue. The lift station discharges through a 6-inch diameter PVC force main to a manhole near the intersection of Huerta Avenue and El Camino Real.
- <u>Los Ositos Lift Station:</u> Los Ositos Lift Station is located at the intersection of 11th Street and Elm Avenue. The lift station discharges through an 8-inch diameter PVC force main to a manhole near the intersection of 11th Street and Maple Avenue.
- <u>Vineyard Lift Station:</u> Vineyard Lift Station is located on Vineyard Avenue, south of Apple Avenue. The lift station discharges through a 4-inch diameter PVC force main to a manhole to the northwest of the lift station in Apple Avenue.

- Nino Lift Station: Nino Lift Station is located at the intersection of Nino Lane and Del Ponte Drive. The lift station discharges through a 4-inch diameter PVC force main to a manhole to the northwest of the lift station near the intersection of Del Ponte Drive and Nino Lane.
- Reed Lift Station: Reed Lift Station is located near the intersection of Reed Lane and De Leon Drive. The lift station discharges through a 6-inch diameter PVC force main to a manhole near the intersection of Reed Way and El Camino Real.
- Cypress Lift Station: Cypress Lift Station is located near the intersection of Cypress Avenue and El Camino Real. The lift station discharges through a 10-inch diameter PVC force main to a manhole near the intersection of Pine Avenue and El Camino Real.



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Note: This exhibit only shows sewer lines that were modeled as part of this master plan but does not include the entire system.

FIGURE 2-1
SEWER COLLECTION SYSTEM OVERVIEW

1 inch = 1,200 feet
N
0 600 1,200
Feet

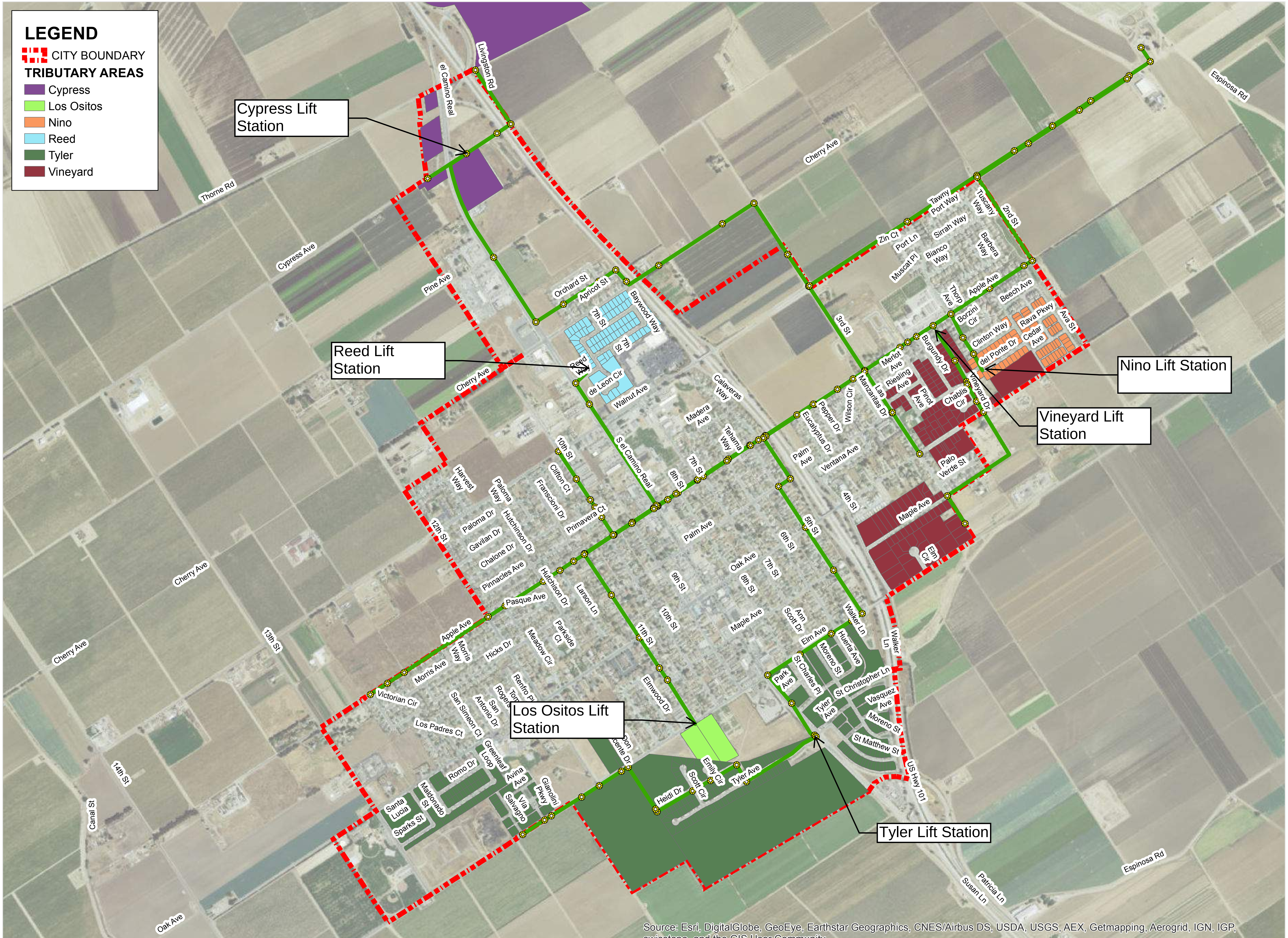

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FIGURE 2-2
LIFT STATION
TRIBUTARY MAP

N

1 inch = 1,200 feet

0 600 1,200

Feet

3: Study Area Characteristics

Chapter 3 describes the study area characteristics germane to this Sewer Master Plan for the City. Included in this chapter is a description of the various land uses in the service area, future development projections, and existing and future population projections. Future development is based on the 2005 General Plan Land Use Element and direction from City Staff.

Land Use and Future Development

The City of Greenfield is located in the Salinas Valley in Monterey County. Founded in 1905 and incorporated in 1947, Greenfield is centered in a highly productive agricultural region. Figure 3-1 illustrates the City's boundary, and the existing Land Use Designations per the 2005 General Plan. Table 3-1 summarizes the Land Use Designations and Projections (from the 2005 General Plan) and provides a breakdown of acreage designated for each land use. Figure 3-2 illustrates the future growth areas and land uses.

Population

For this master plan, historical and future population estimates were provided by the City. The reported population for the City for 2010-2015 is as follows:

- **2010:** 16,192 persons
- **2011:** 16,396 persons
- **2012:** 16,466 persons
- **2013:** 16,784 persons
- **2014:** 16,919 persons
- **2015:** 16,870 persons

For the purposes of this Sewer Master Plan, the City has provided a projected population growth rate of 2.5% from the base population of 16,870 in 2015. This growth rate results in a total population of 28,400 by 2035, which correlates to the 20 year planning horizon for this Sewer Master Plan.

Table 3-1. Existing and Future Land Use

General Plan Land Use	Total Acreage	Future Growth Area
Single Family Residential	380.60	190.74
Multiple Family Residential	220.37	113.61
High Density Residential	30.84	0.00
Residential Estate	0.00	149.05
Neighborhood Commercial	5.24	0.00
Downtown Commercial	29.69	0.00
Highway Commercial	103.43	234.13
Professional Office	20.92	0.00
Light Industrial	108.36	32.43
Heavy Industrial	0.00	154.03
Public Quasi Public	139.22	0.00
Recreation and Open Space	34.56	0.00
Artisan Agricultural Visitor Serving	0	168.29

Source: City of Greenfield General Plan and Zoning 2010 GIS Database from PMC.

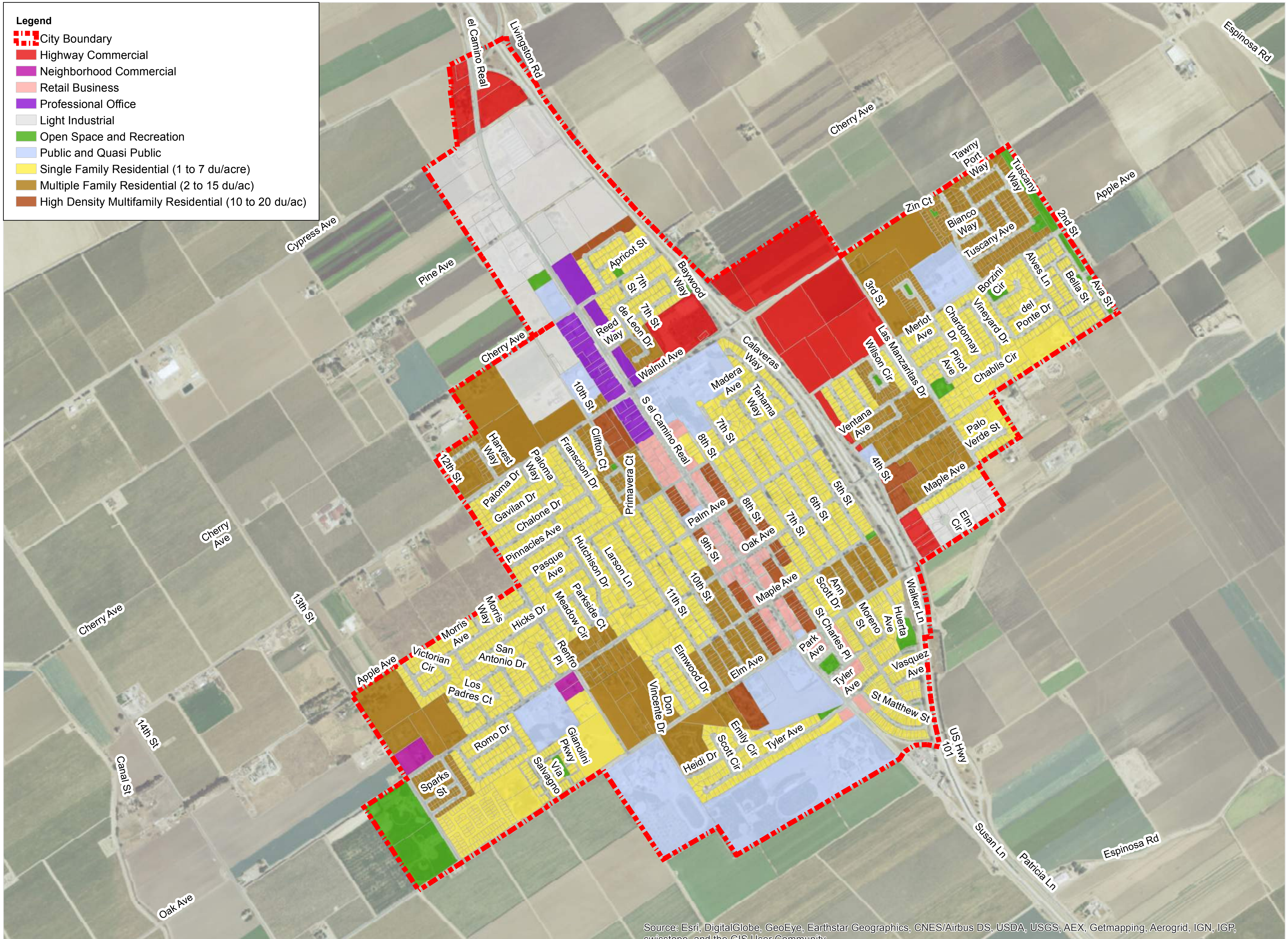

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FIGURE 3-1
EXISTING ZONING

N

1 inch = 1,200 feet

0 300 600 1,200

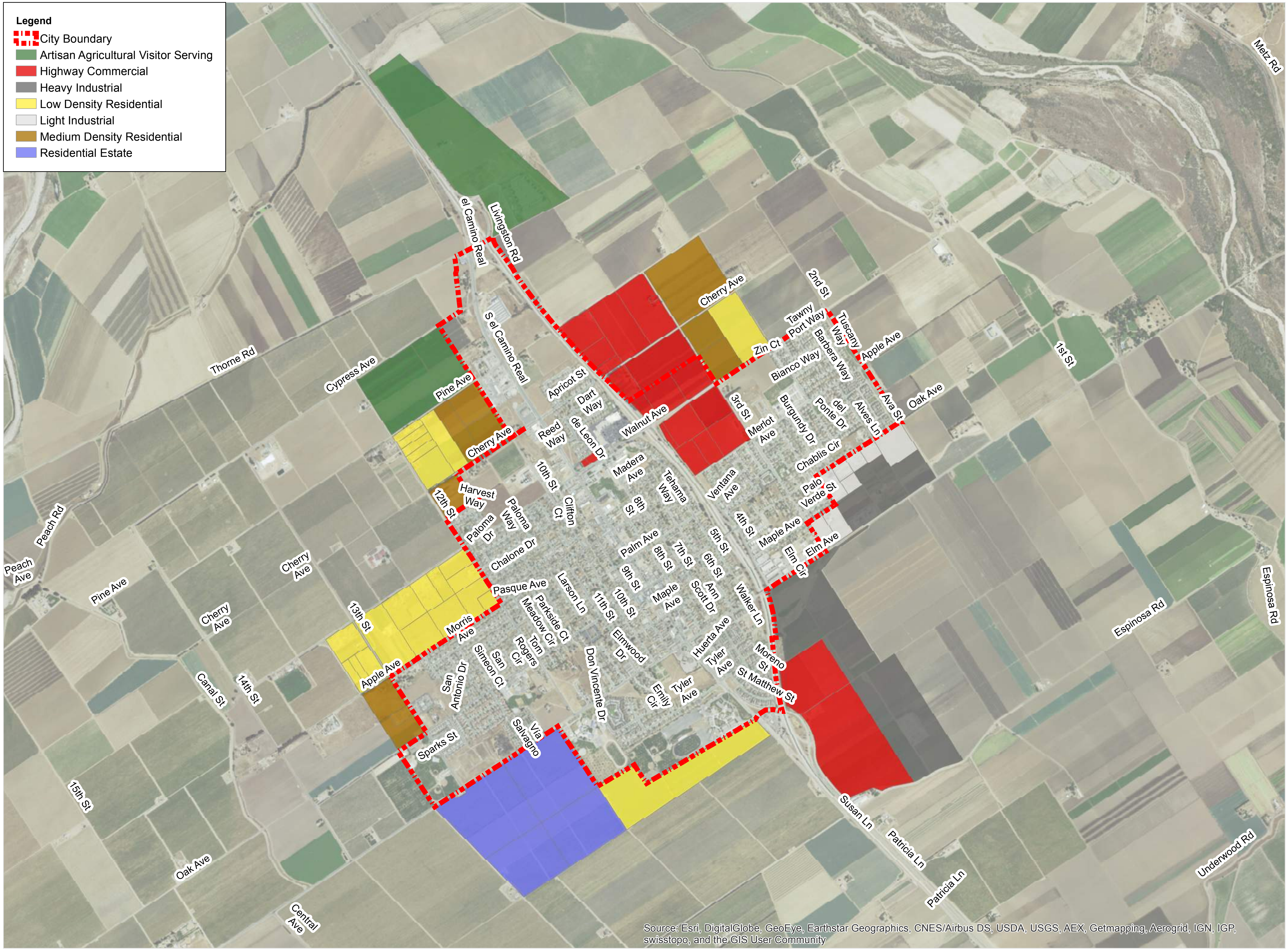

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FIGURE 3-2
FUTURE GROWTH AREA ZONING

4: Wastewater Flows

Chapter 4 describes the existing and projected sewer flows for the City. The sewer flow forecasts will form the basis for identifying existing and future system needs and analyzing deficiencies.

Wastewater Flow Monitoring

As part of this master plan effort, in conjunction with US^3, in-line sewer flow monitoring was conducted at three select locations, from September 24, 2015 through October 28, 2015. The main goal of this data is to develop peaking factors for the hydraulic model. The data can also provide very useful information on flow patterns and if/when sewers may be surcharging. Figure 4-1 shows the locations of the three flow monitoring locations chosen for this study. These locations are described as follows:

Site 1

The site 1 flow meter was installed in a manhole on the 12” sewer main at the intersection of Apple Avenue and 2nd Street. It collects flow from the area east of Highway 101 and south of Apple Avenue. Flow is to the north towards Walnut Avenue.

Site 2

The site 2 flow meter was installed in a manhole on the 12” sewer main at the intersection of Apple Avenue and Calaveras Way. It collects flow from a majority of the central downtown area west of Highway 101. Flow is to the east under the Freeway.

Site 3

The site 3 flow meter was installed in a manhole on the 12” sewer main at the intersection of Apple Avenue and Palm Avenue. It collects flow from the southern portion of the City west of Highway 101. Flow is to the north to Apple Avenue.

Flow Meter Results

The flow meters were used to evaluate wastewater flow contributions from the individual tributary areas in the City. There were no significant rain events during the flow monitoring period to evaluate the rainfall dependent infiltration and inflow. Therefore, flow data collected for the entire monitoring period was used to calculate a daily average, representative of dry weather flow. A summary of the average daily flow results from each of the flow monitoring stations is provided in Table 4-1 below. Detailed flow monitoring results are included in Appendix A.

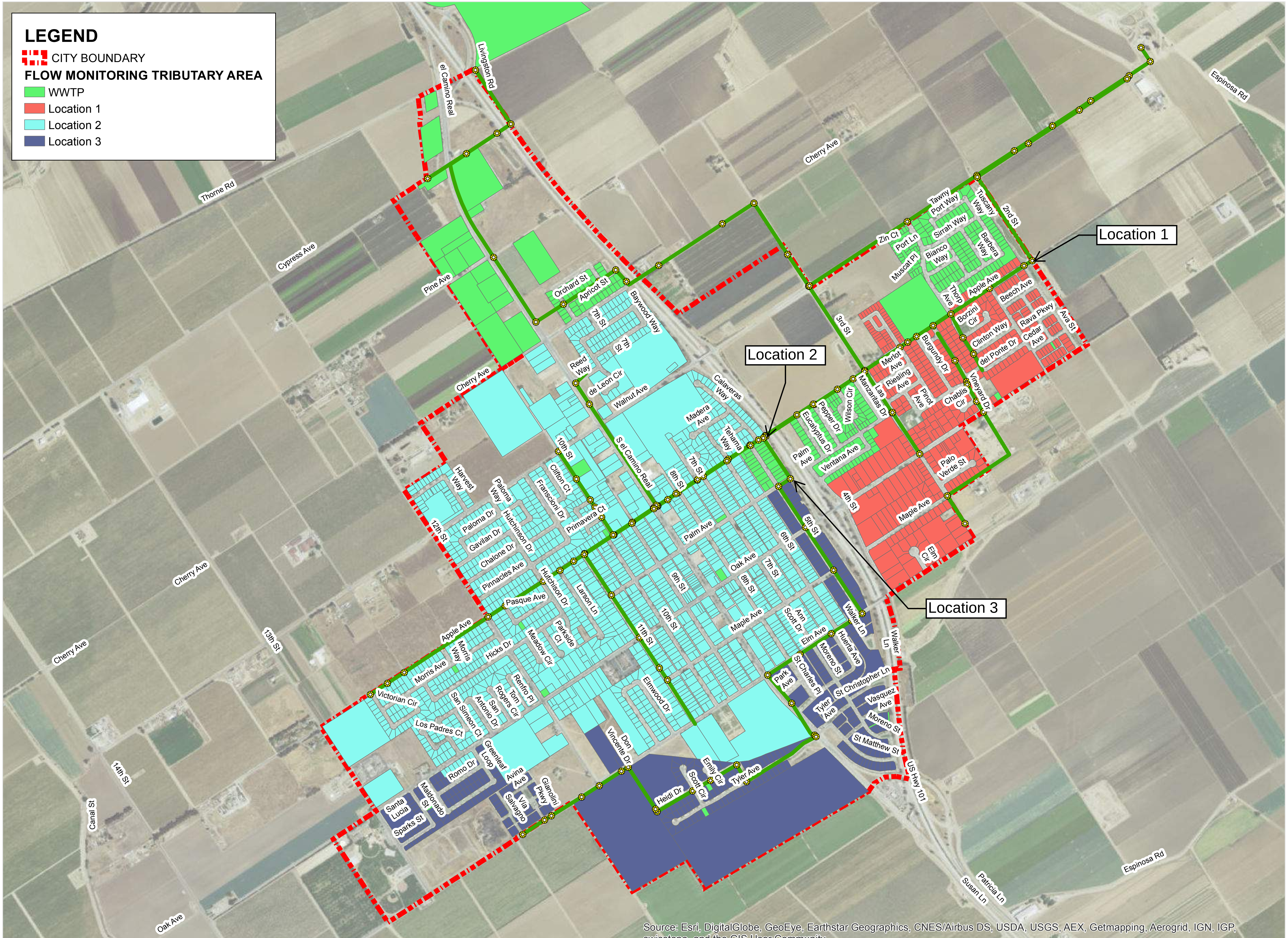

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FIGURE 4-1
FLOW MONITORING
BASIN MAP

1 inch = 1,200 feet
0 600 1,200
Feet
N

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Table 4-1. Flow Meter Results Summary

Location	Average Daily Flow[1]
Site 1	160,000
Site 2	440,000
Site 3	220,000
Total	820,000

[1]This table represents flow from the metered areas only, and does not include flow for the entire City of Greenfield.

The flow monitoring equipment used included Hach Flo-Dar flow meters, which use pressure transducers to sense liquid pressure at the point of monitoring (which translates into flow depth), and radar to measure liquid velocity, with data readings every 5 minutes. These readings are then calculated into flow. These reports, and more in-depth analysis of the data conducted by Wallace Group, are included in their entirety as Appendix A. The wastewater treatment plant influent flow monitoring data (provided by the City) was also reviewed, to compare the various flow characteristics at the plant relative to the collection system. It is most informative to view these diurnal charts on weekdays, Saturday and Sunday separately, as in some instances, some very distinct and repeatable flow patterns exist.

There are a number of interesting observations to the flow data, and a number of diurnal curves were developed and reviewed. Again, these results are included in the Appendices. The major finding is at Location 2, where the existing gravity sewer in Apple Avenue appears to be surcharging on a daily basis. This was evaluated in the sewer model; however, the model results did not show surcharging to the degree shown in the flow monitoring results. The City should conduct sewer videos in this reach to determine if physical observations match up to this data.

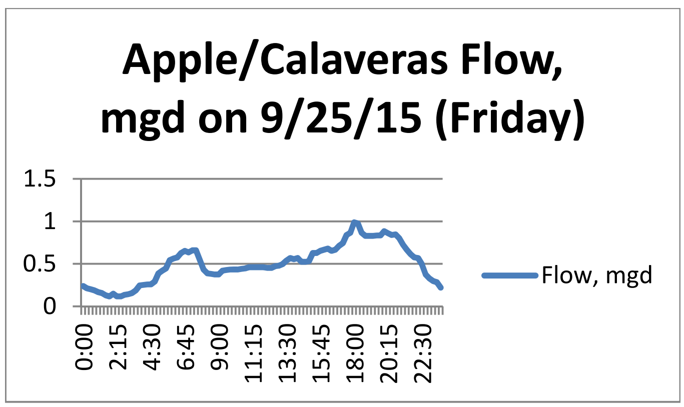

Figure 4-2. Diurnal Flow Pattern in City Sewer

Figure 4-2 depicts the flow pattern at Monitoring Site 2 (Apple Ave@Calaveras). This flow trend shows a fairly typical week day trend, with a morning peak as people get ready for work and school, and an evening peak when people come home after school and work.

Collection System Peaking Factors

As part of the flow analysis, peaking factors were derived from these diurnal curves and flow data, and were be used in the sewer collection system model. Table 4-2 summarizes the peaking factors derived at each monitoring location.

Table 4-2. Summary of Peaking Factors in Sewer Collection System

Location	Peaking Factor
2nd Street/Apple Avenue	2.0
Apple Avenue/Freeway 101	2.0
Apple Alley/Palm Avenue	2.0
WWTP	2.75

5: Collection System Analysis

This Chapter presents the analysis of the gravity wastewater collection system for the City of Greenfield. Refer to Chapter 2 for an overview of the City's wastewater collection system. Refer to Chapter 8 for the proposed capital improvements based on the analysis presented in this Chapter.

Introduction

The City's wastewater collection system consists of a network of 8-inch to 24-inch gravity sewer mains, and six (6) lift stations. The main trunk sewer system was analyzed using a computer based hydraulic model as part of this Sewer Master Plan project, to evaluate performance of the wastewater collection system under both existing and future conditions. Figure 2-1 provides an overview of the sewer mains that were included in the hydraulic model.

The analysis of the wastewater collection system is based on a sewer Geographic Information System (GIS) developed using survey data collected by Wallace Group.

Collection System analysis criteria

As described in the City's Sanitary Sewer Management Plan (SSMP), Element 5: Design and Performance Provisions, the City defers to the City of Salinas Standard Specifications, Design Standards and Standard Plans for standardized design for the wastewater collection system. The recommended design criteria are summarized in Table 5-1. These criteria provide capacity buffer to avoid surcharge conditions, for fluctuations in flows due to diurnal variations, and anticipated peak flows. Gravity pipe performance was analyzed based on maximum percent full (d/D ratio), defined as the depth of flow in a pipe divided by the diameter of the pipe.

Table 5-1. City Design and Performance Standards

Pipe Diameter	Maximum Allowed d/D
10-inch and smaller	0.67
12-inch and larger	0.8
Other Design Criteria	
Minimum Diameter	8-inch
Minimum Velocity	2.0 fps
Maximum Velocity	8.0 fps
Manning's Coefficient, n	0.013 for VCP, CIP & DIP, 0.011 for PVC & HDPE

Collection System Flows

Existing and future flows were analyzed in the sewer model for both dry weather and wet weather conditions. Flow rates were derived as described in Chapter 4 of this report. Flow parameters as utilized in this analysis are defined as follows.

- **ADF**: Average daily dry weather system flow
- **PHDWF**: Peak hour dry weather system flow

Collection System Model Development

A hydraulic model of the sewer collection system was developed with the Innovyze® InfoSWMM sewer modeling program. InfoSWMM utilizes Manning's Equation for open channel flow (gravity pipes), Dynamic Wave analysis for flow routing through the collection system, and the Hazen-Williams Equation for pressurized flow conditions (force mains or surcharged pipes). Model results were evaluated for pipeline capacity, flow velocity, and maximum d/D ratio under various flow conditions.

Flow Allocation

Wastewater flows were assigned to the sewer model utilizing estimated flows as described in Chapter 4. Flows were allocated to individual sewer manholes based on the actual location of City customers. Tributary areas for each modeled manhole were developed and shown on Figure 5-1. Each tributary area represents the total residential, commercial, and institutional customers contained within the tributary boundary.

Future wastewater flows were allocated to the sewer model based on the location of the parcels in relation to the tributary areas for the modeled manholes. The impact to the collection system from future flows and the proposed land uses, sewer system layout, and demands should be re-evaluated for each project in the planning stage to confirm assumptions made for the purpose of this Sewer Master Plan are accurate and confirm that no additional upgrades will be required.

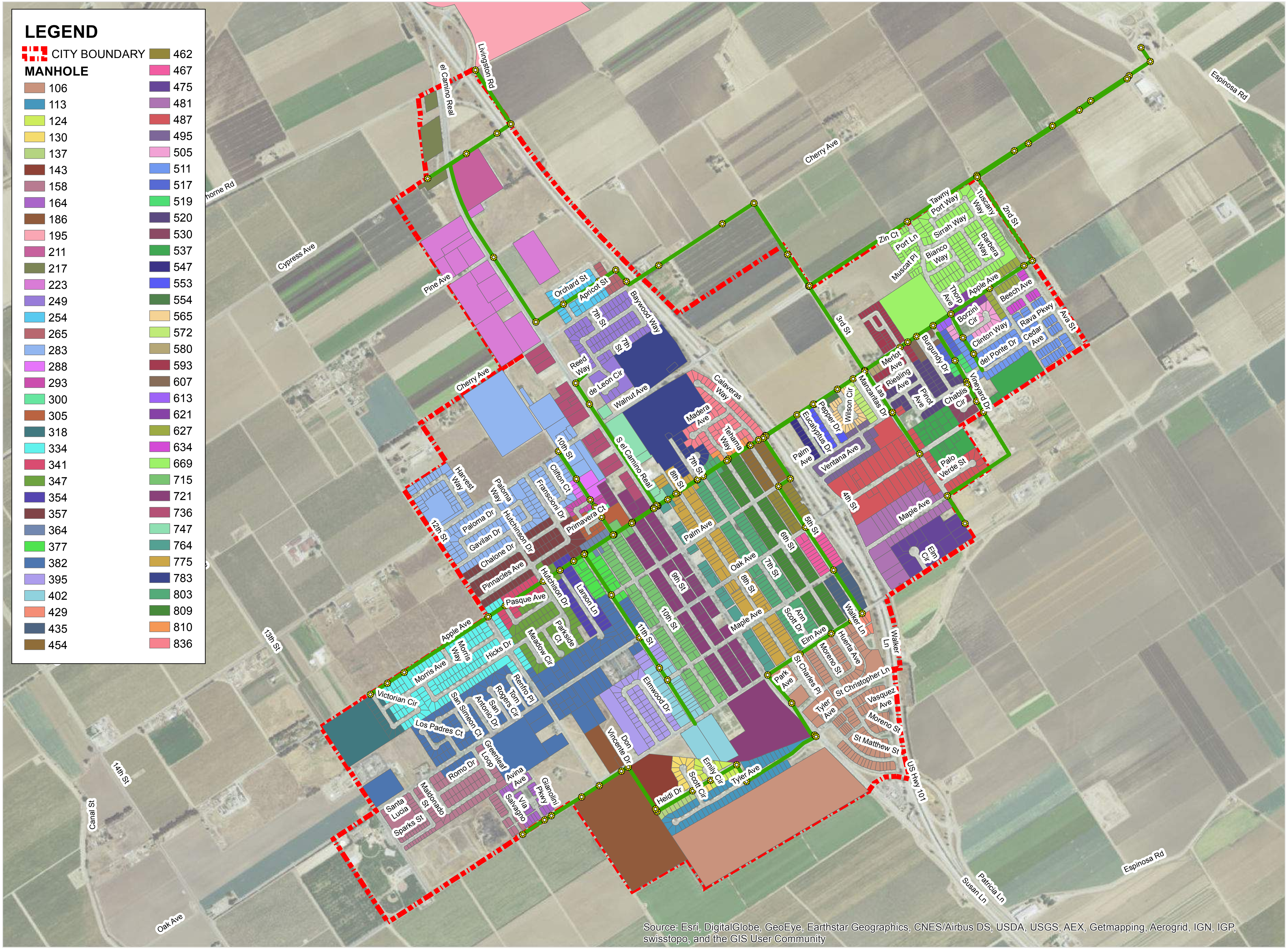

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FIGURE 5-1
MANHOLE TRIBUTARY AREA MAP

1 inch = 1,200 feet

0 600 1,200

Feet

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Model Calibration

Approximately five weeks of sewer flow data was collected in support of the hydraulic model development, as described in Chapter 4 of this report. Representative data for each flow monitoring location was compared to model results and demand allocations were adjusted to match the flow monitoring results for average daily flow conditions. However, the surcharging described in Chapter 4 at flow monitoring location 2 was not replicated.

System Conditions Analyzed

The hydraulic model was utilized to analyze system flow for both existing and future flow conditions. Within the model, multiple scenarios were developed that represent these various conditions. Existing and Future scenarios were utilized to identify system upgrades required in order to meet performance criteria as specified, and to identify areas recommended for high priority maintenance operations. Scenarios developed consist of the following:

- *Existing PHDWF Scenario:* This scenario represents the trunk sewer system under existing peak hour dry weather flow conditions. This scenario includes estimated flow contributions from groundwater infiltration.

- *Future PHDWF Scenario:* This scenario represents the trunk sewer system under future peak hour dry weather flow conditions, with all potential development as described in Chapter 2 contributing to the existing collection system. This scenario includes estimated flow contributions from groundwater infiltration.

Collection System Model Results – Existing Flow Conditions

This section provides a summary of model results for the existing flow conditions modeled.

Deficient System Capacity

The following locations were identified through the analysis as having insufficient capacity to meet City performance standards under existing system flow conditions. Recommended pipe upgrades identified for existing conditions may have the potential to further increase in diameter for future conditions, as described later in this chapter. Thus, when making recommendations to correct existing deficiencies, the future condition must also be considered in the overall recommendation to upsize sewer mains. Refer to Figure 5-2 for a system-wide map showing whether existing modeled sewer mains meet the maximum d/D criteria under existing PHDWF conditions. Refer to Figure 5-3 for an overall map of the location of recommended system upgrades for existing conditions. Within the sewer model all gravity sewer upgrades were designated as PVC.

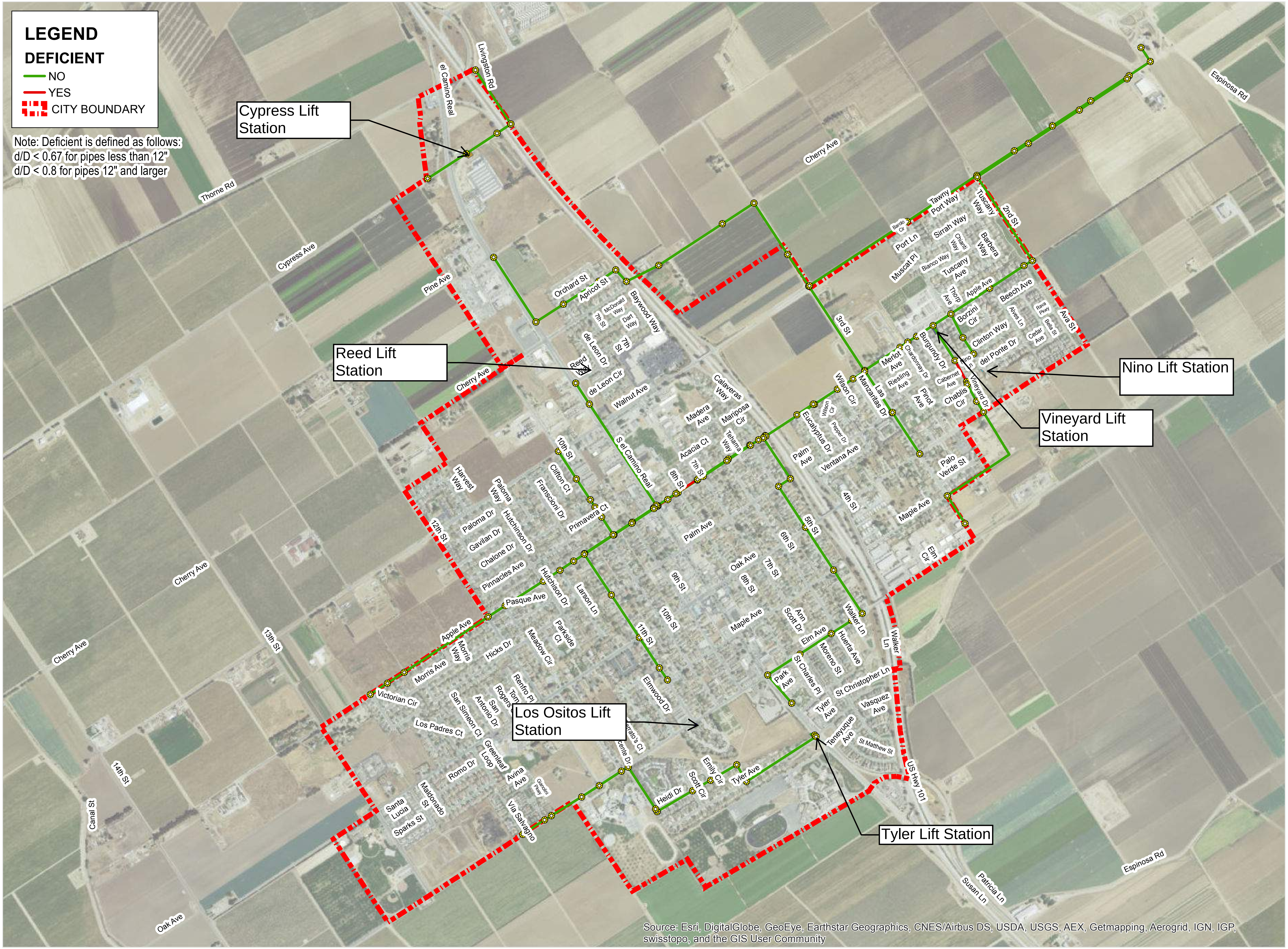

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FIGURE 5-2
DEFICIENT GRAVITY SEWER MAINS UNDER EXISTING FLOW CONDITIONS

1 inch = 1,200 feet
N
0 600 1,200
Feet

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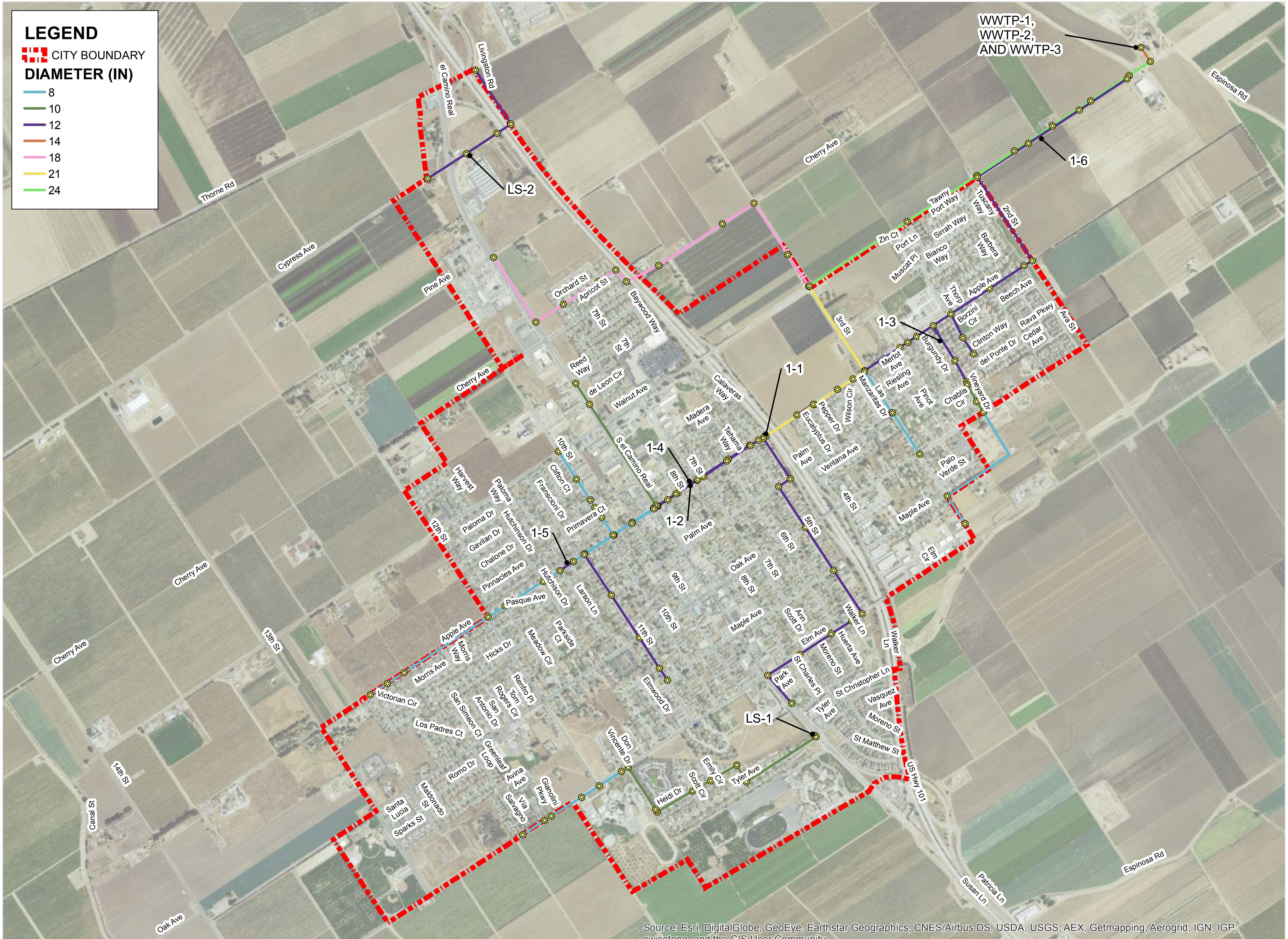

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FIGURE 5-3
EXISTING CIPS

1 inch = 1,200 feet
0 600 1,200
Feet
N

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Apple Avenue

- Location Extents: South line between 7th Street and El Camino Real, north line between 7th Street and 8th Street and between Larson Lane and the mid-block manhole east of Larson Lane.

There are two parallel sewer mains in Apple Avenue, both of which are currently 8-inch VCP. Under PHDWF conditions, d/D values up to 1.00 were modeled, thus indicating potential or actual surcharge conditions. It is recommended to upgrade the identified gravity sewer mains to 12-inch PVC to reduce the maximum d/D to acceptable levels (below 0.8 for upsized 12" PVC gravity main). These upgrades require 1,200 lineal feet of 12-inch PVC.

Vineyard Drive

- Location Extents: Cabernet Avenue to Vineyard Drive Lift Station.

The Vineyard Drive sewer main is currently 10-inch VCP, and the sewer model indicates d/D values up to 1.00 under PHDWF. Upgrading to 12-inch PVC reduces the maximum d/D to acceptable levels (below 0.8) for existing and just over 0.8 for future flow conditions. Because the future maximum d/D is just over 0.8, the City will want to monitor and continue to reassess the capacity of this pipe to determine if further upgrades are necessary. This upgrade requires 780 lineal feet of 12-inch PVC.

Low Pipe Velocity

Low pipe velocity results in the increased likelihood for solids to settle out of wastewater flow, leading to pipe backups and blockages. It is recommended to maintain a minimum pipe velocity of 2.0 feet per second (fps) during average flow conditions, to maintain solids in suspension. A total of 77 modeled pipes were identified with a velocity below 2.0 fps under existing average day conditions. It is recommended that pipes identified with a maximum velocity of less than 2.0 fps be flushed and/or vacuumed on a regular basis. Total length of pipe is 6.4 miles. These pipes are depicted in Figure 5-4. These recommendations should be considered for incorporation into the City's SSMP (subsequent update following the 2014 Update) list of high maintenance areas (HMAs).

Pipe Travel Time

Excessive pipe travel time is a result of low velocity and/or long pipe runs, and can lead to problems with hydrogen sulfide attack and odor at downstream manholes. Typically wastewater is oxygenated as it flows through a manhole, decreasing likelihood of hydrogen sulfide generation. Travel time exceeding thirty minutes through a single pipe (manhole to manhole) is undesirable. All pipes included in the hydraulic model have an existing average day travel time of 5 minutes or less; therefore pipe travel time is not anticipated to cause maintenance issues for the City’s collection system.

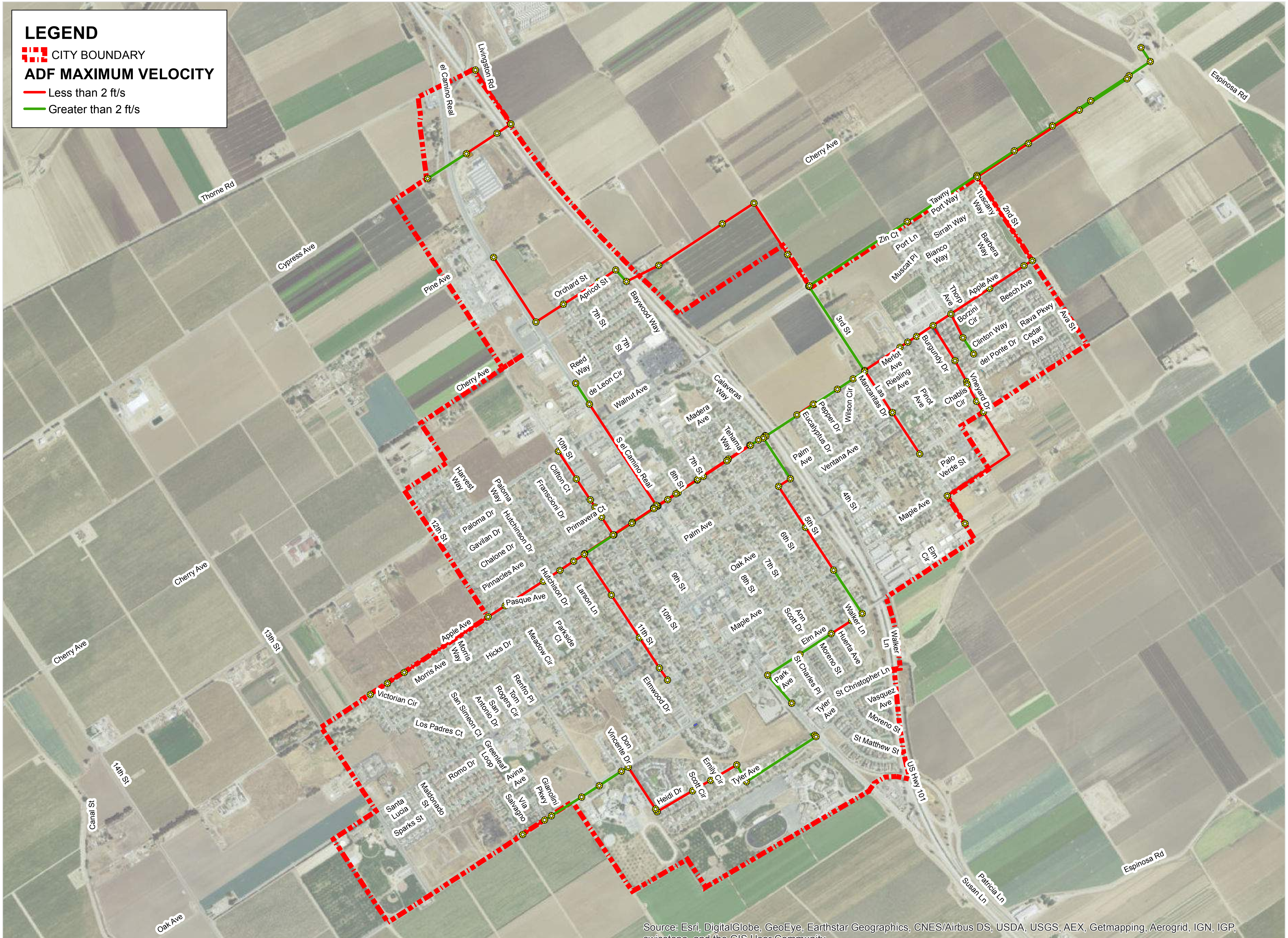

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FIGURE 5-4
EXISTING AVERAGE DAILY FLOW MAXIMUM VELOCITY

1 inch = 1,200 feet

N

0 600 1,200
Feet

Collection System Model Results – Future Flow Conditions

This section provides a summary of model results for the future flow conditions modeled.

Deficient System Capacity

The following locations were identified through the analysis as having insufficient capacity to meet City performance standards while conveying future system flows. Refer to Figure 5-5 for a system-wide map showing where the collection system does and does not meet maximum d/D criteria under future flow conditions. Refer to Figure 5-6 for an overall map of recommended system upgrades to address future wastewater flow conditions.

Recommendations for future upgrades to the City sewer collection system are based on the assumptions that the all of the upgrades recommended for existing conditions have been completed, and/or that recommended upgrades to address existing deficiencies already anticipate the future upgrades.

Apple Avenue

- Location Extents: Apple Avenue north line from 5th Street to 7th Street, Apple Avenue from 5th Street to 5th Street Alley, and Apple Avenue from 11th Street to 12th Street.

The sewer main in Apple Avenue is currently 12-inch that is projected to have a d/D greater than 0.80 under future PHDWF conditions. Upgrading to 18-inch PVC reduces the maximum d/D to acceptable levels. Total affected pipe length is 840 lineal feet.

WWTP

- Location Extents: End of 24" line on Walnut Avenue to WWTP headworks.

The sewer main at the WWTP is currently 14-inch and is projected to have a d/D greater than 0.80 under future PHDWF conditions. Upgrading to 24-inch PVC reduces the maximum d/D to acceptable levels. Total affected pipe length is 220 lineal feet.

Elm Avenue

- Location Extents: Heidi Drive to Via Salvgano.

The sewer main in Elm Avenue is currently 8-inch that has a d/D greater than 0.67 under future PHDWF. Upgrading to 10-inch PVC reduces the maximum d/D to acceptable levels. Total affected pipe length is 1,650 lineal feet.

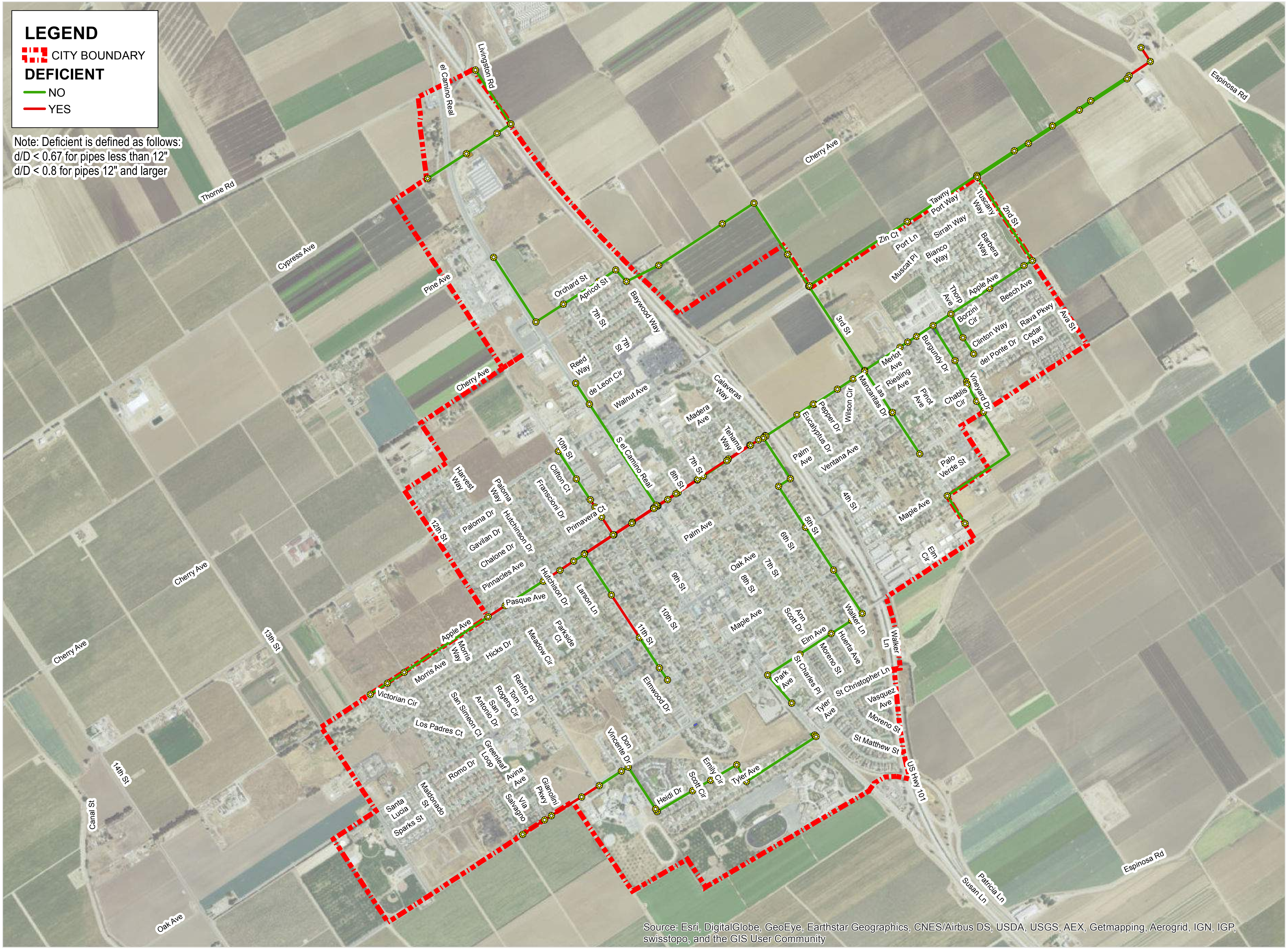

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FIGURE 5-5
DEFICIENT GRAVITY SEWER MAINS UNDER FUTURE FLOW CONDITIONS

1 inch = 1,200 feet

N

0 600 1,200
Feet

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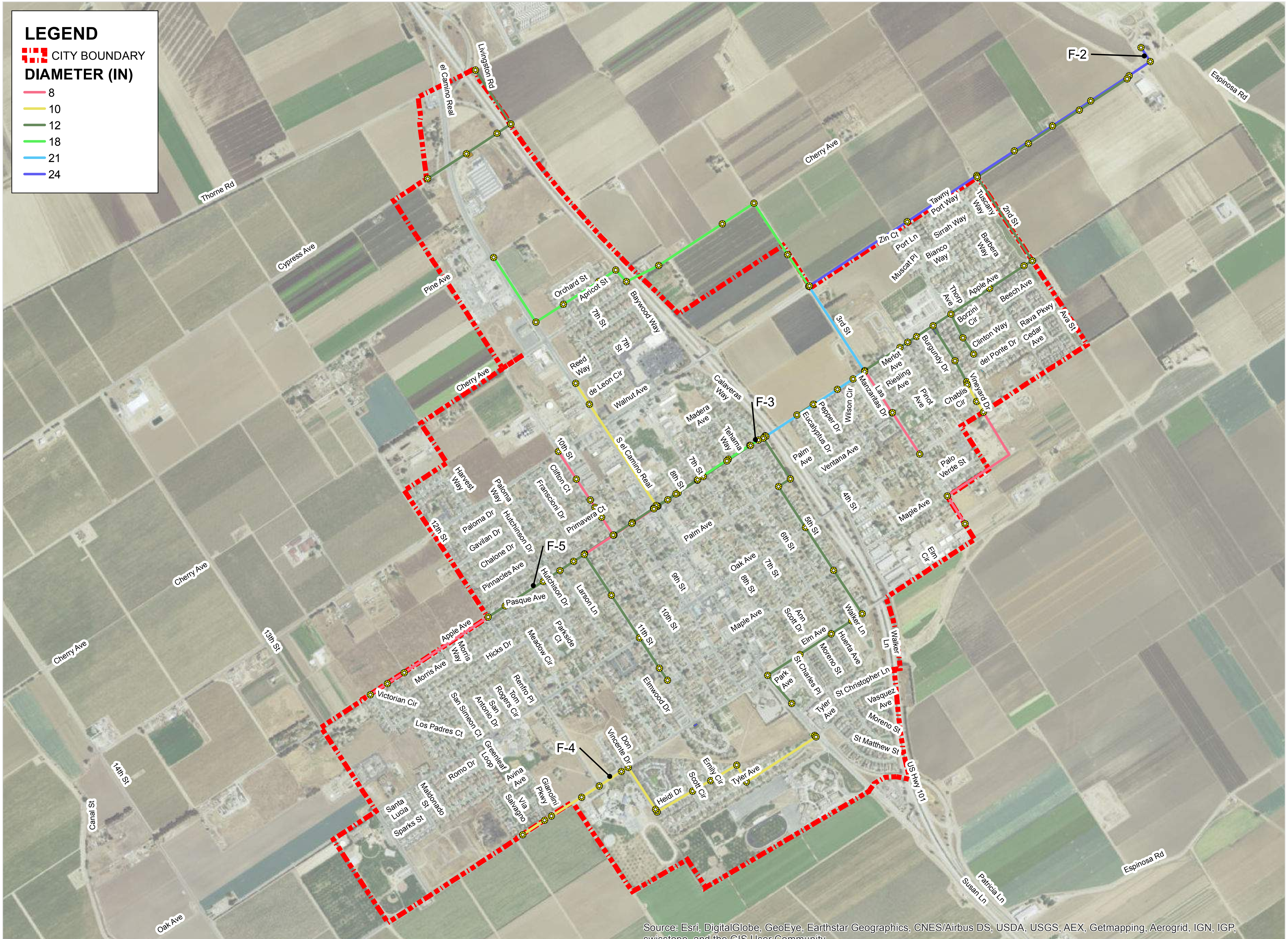

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FIGURE 5-6
FUTURE CIP
DIAMETER

1 inch = 1,200 feet

N
0 600 1,200
Feet

6: Evaluation of Sewage Lift Stations

The City of Greenfield owns and operates six sewage lift stations as part of the City's overall sewage collection system. All six lift stations are Smith & Loveless wetpit/drypit lift stations. This section provides a detailed evaluation of each of the six lift stations. The lift stations are evaluated from a general operational standpoint, and then from a hydraulic/operations standpoint. These lift stations and corresponding tributary areas are depicted on Figure 2-2 in Chapter 2. Details of the hydraulic capacity, equipment and other details of the lift stations will be provided later in this Chapter.

Lift Station General Evaluation (non-hydraulic)

The six lift stations were evaluated based on non-hydraulic parameters. This evaluation included review of existing information, as-built drawings, and a site visit to each lift station with City staff on December 2, 2015. A summary of the pertinent non-hydraulic parameters of the lift stations is presented in Table 6-1.

Tyler Lift Station

Tyler Lift Station is located at the intersection of El Camino Real and Tyler Avenue. The lift station services the portion of the City to the south of Elm Avenue and west of Highway 101. The lift station discharges through a 6-inch diameter PVC force main to a manhole near the intersection of Huerta Avenue and El Camino Real.


Tyler Lift Station

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station was installed in 1990 with two 10-hp pumps and was upgraded in 2007 with two new 20-hp pumps. According to City staff, this upgrade to 20-hp pumps maximizes the space within the drypit area, thus larger pumps cannot be accommodated in the existing drypit if needed in future years.

Wetwell: The wetwell is a circular unlined concrete wetwell. The wetwell is in good condition, with no visible signs of corrosion.

Site Conditions: The lift station site area is paved and fenced, with a driveway/access off of El Camino Real. Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site has lighting for night-time emergency maintenance and there is potable water available for sanitation and washdown purposes.

Los Ositos Lift Station

The Los Ositos Lift Station is located at the intersection of 11th Street and Elm Avenue. The lift station services the Los Ositos residential development on Elm Avenue near the intersection of Elm Avenue and 11th Street. The lift station discharges through a 6-inch diameter PVC force main to a manhole near the intersection of 11th Street and Maple Avenue.

Table 6-1. Summary of Lift Station Conditions (Non-Hydraulic)

	Tyler	Los Ositos	Vineyard
Year Built	1990	1979	1983
Lift Station Type	Smith and Loveless - Wet Pit/Dry Pit	Smith and Loveless - Wet Pit/Dry Pit	Smith and Loveless - Wet Pit/Dry Pit
Standby Power	Yes	No, quick connect.	No, quick connect.
Electrical Service	Unknown	Unknown	Unknown
Alarms	Known Problems	Good	Good
Wetwell Material	Concrete	Concrete	Concrete
Wetwell Coating	No	Tar around Manway	No
Wetwell Condition	Good	Good	Deteriorating in manway, unable to see lower.
Chemical Feed (Ferrous Chloride) Tanks/Piping	Unknown	Unknown	Unknown
Site Drainage	Good	Good	Good
Potable Water at Site	Yes	No	No
Site Lighting	Yes	No, street lights nearby.	No, street lights nearby.
Site Security/Fencing	Yes	No	Yes

	Nino	Reed	Cypress
Year Built	2004	1985	2004
Lift Station Type	Smith and Loveless - Wet Pit/Dry Pit	Smith and Loveless - Wet Pit/Dry Pit	Smith and Loveless - Wet Pit/Dry Pit
Standby Power	Yes	No, quick connect.	No, quick connect.
Electrical Service	Unknown	Unknown	Unknown
Alarms	Good	Good	Good
Wetwell Material	Concrete	Concrete	Concrete
Wetwell Coating	No	No	Yes
Wetwell Condition	Good	Good	Good
Chemical Feed (Ferrous Chloride) Tanks/Piping	Unknown	Unknown	Unknown
Site Drainage	Good	Good	Good
Potable Water at Site	No	No	Yes
Site Lighting	No, street lights nearby.	No	Yes
Site Security/Fencing	Yes	Yes	Yes


Los Ositos Lift Station

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station has two 3-hp pumps and was installed in 1979.

Wetwell: The wetwell is a circular concrete wetwell with tar around the manway portion to prevent root intrusion from the tree adjacent to the lift station. According to City staff, root intrusion is not currently an issue. The wetwell is in good condition, with no visible signs of corrosion however the manhole ring is beginning to deteriorate.

Site Conditions: The lift station is in a grassy area near Elm Avenue and is open (not fenced), with vehicle access for maintenance. Hatches are padlocked for security. The lift station is situated between two multi-family residential areas on the south side of Elm Avenue, with walkways on both sides of the station. There is good drainage in the area, and the site is not prone to flooding. The site does not have lighting for night-time emergency maintenance and there is not potable water available for sanitation and washdown purposes.

Vineyard Lift Station


Vineyard Lift Station

The Vineyard Lift Station is located on Vineyard Avenue, south of Apple Avenue. The lift station services portions of the City south of Apple Avenue and north of Elm Avenue and between Las Manzanitas Drive to the west and Alves Lane to the east. The lift station discharges through a 4-inch diameter PVC force main to a manhole to the northwest of the lift station in Apple Avenue.

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station has two 3-hp pumps and was installed in 1983.

Wetwell: The wetwell is a circular concrete wetwell with no coating. There are noticeable signs of deterioration in the manway, but due to access constraints the condition of the wet well below the manway is unknown.

Site Conditions: The lift station is in a dirt lot between Apple Avenue and Vineyard Drive near Elm Avenue and is fenced, with vehicle access for maintenance. Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site does not have lighting for night-time emergency maintenance and there is not potable water available for sanitation and washdown purposes.

Nino Lift Station

Nino Lift Station

The Nino Lift Station is located at the intersection of Nino Lane and Del Ponte Drive. The lift station services portions of the City south of Apple Avenue and north of Oak Avenue and between Las Ava Street to the west and Del Ponte Drive to the east. The lift station discharges through a 4-inch diameter PVC force main to a manhole to the northwest of the lift station near the intersection of Del Ponte Drive and Nino Lane.

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station has two 3-hp pumps and was installed in 2004.

Wetwell: The wetwell is a circular concrete wetwell with no coating. The wetwell is in good condition, with no visible signs of corrosion. The wetwell has a large manhole lid that requires two operators to open.

Site Conditions: The lift station is in a dirt lot and is fenced, with vehicle access for maintenance. The wetwell is located outside of the fenced area in Del Ponte Drive. Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site does not have lighting for night-time emergency maintenance and there is not potable water available for sanitation and washdown purposes.

Reed Lift Station

Reed Lift Station

The Reed Lift Station is located near the intersection of Reed Lane and De Leon Drive. The lift station services portions of the City south of Apricot Avenue and north of Walnut Avenue and between Highway 101 to the west and El Camino Real to the east. The lift station discharges through a 6-inch diameter PVC force main to a manhole near the intersection of Reed Way and El Camino Real.

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station has two 3-hp pumps and was installed in 1985.

Wetwell: The wetwell is a circular concrete wetwell with no coating. The wetwell is in good condition, with no visible signs of corrosion.

Site Conditions: The lift station is paved and is fenced, with vehicle access for maintenance. Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site does not have lighting for night-time emergency maintenance and there is not potable water available for sanitation and washdown purposes.

Cypress Lift Station

The Cypress Lift Station is located near the intersection of Cypress Avenue and El Camino Real. The lift station services portions of the City north of Cypress Avenue. The lift station discharges through a 10-inch diameter PVC force main to a manhole near the intersection of Pine Avenue and El Camino Real.

Cypress Lift Station

Lift Station/Pumps: The lift station has a wetwell with suction piping and a drypit that houses the two pumps and valving. The lift station has two 20-hp pumps and was installed in 2004.

Wetwell: The wetwell is a circular concrete wetwell with a protective coating. The wetwell is in good condition, with no visible signs of corrosion.

Site Conditions: The lift station is paved and is fenced, with vehicle access for maintenance and the wetwell is located outside of the fenced area. Hatches are padlocked for security. There is good drainage in the area, and the site is not prone to flooding. The site does not have lighting for night-time emergency maintenance and there is not potable water available for sanitation and washdown purposes.

Lift Station Hydraulic Performance Evaluation

The hydraulic characteristics of each lift station were analyzed and deficiencies were noted. Design criteria that apply to the lift stations and force mains are summarized below. Table 6-2 summarizes the hydraulic parameters of each lift station.

- Force main velocities should be greater than 2.0 feet per second to maintain self-cleansing properties but less than 5.0 feet per second to minimize head loss and water hammer.
- Lift stations should be able to convey peak flows with the largest pump out of service. Station “capacity” is therefore calculated with the largest pump out of service.
- Lift station wet wells should be sized to limit the number of pump starts per hour to acceptable limits as defined by the pump manufacturer. Traditionally this is in the range of 6 starts per hour.
- Lift stations should have a means of conveying peak flows during a power outage. Lift stations serving a small number of customers could use wet well storage to meet this requirement.

Lift Station Flows

This subsection describes details of the existing lift stations and tributary flows (existing and future) relative to the pumping capacities of the existing lift stations. Flow parameters for each lift station are summarized in Table 6-3.

The peak hour wet weather flow is calculated as follows:

The average wastewater flow is multiplied by the diurnal peaking factor measured during the flow monitoring described previously, to obtain peak hour flow (dry weather).

Table 6-2. Summary of Hydraulic Characteristics

Lift Station		Tyler	Los Ositos	Vineyard
Pump Type		Vertical Non-Clog	Vertical Non-Clog	Vertical Non-Clog
Pump Manufacturer/Model		Smith & Loveless, Model 4C2A	Smith & Loveless, Model 4B2A	Smith & Loveless, Model 4B2A
No. of Pumps		2	2	2
Pump Motor HP		20	3	3
Motor Speed, RPM		1800	1170	1170
Date of Last Pump Upgrade/Overhaul		2007	N/A	N/A
Design Flow/Head (GPM@TDH)		600 GPM @ 80' TDH	450 GPM @ 17' TDH	140 GPM @ 25' TDH
Pump Design Flow Condition		Simplex	Simplex	Simplex
Approximate Pump Operating Efficiency at Design Point, %		70	70	45
Wet Well Diameter		6	6	6
Wet Well Depth		31	15.66	24
Operating Depth (ft)	**High (Pump On)**	6.50	4.00	7.00
	Low (Pump Off)	2.00	3.00	4.00
Wetwell Operating Volume, Gallons[1]		952	211	441
Force Main Diameter, Inches		6	8	4
Force Main Material		PVC	PVC	PVC
Force Main Velocity, ft/s, Simplex		2.20	1.59	1.65

[1]*Wetwell operating volume calculated based on existing operational set points.*

Lift Station		Nino	Reed	Cypress
Pump Type		Vertical Non-Clog	Vertical Non-Clog	Vertical Non-Clog
Pump Manufacturer/Model		Smith & Loveless, Model 4B2	Smith & Loveless, Model 4B2A	Smith & Loveless, Model 6C4C
No. of Pumps		2	2	2
Pump Motor HP		3	3	20
Motor Speed, RPM		1200	875	1200
Date of Last Pump Upgrade/Overhaul		N/A	N/A	N/A
Design Flow/Head (GPM@TDH)		180 GPM @ 25' TDH	200 GPM @ 19' TDH	1000 GPM @ 42' TDH
Pump Design Flow Condition		Simplex	Simplex	Simplex
Approximate Pump Operating Efficiency at Design Point, %		55	70	65
Wet Well Diameter		6	4	8
Wet Well Depth		20	22.33	30
Operating Depth (ft)	High (Pump On)	6.75	5.50	1.50
	Low (Pump Off)	4.50	3.25	1.00
Wetwell Operating Volume, Gallons[1]		330	211	188
Force Main Diameter, Inches		4	6	10
Force Main Material		PVC	PVC	PVC
Force Main Velocity, ft/s, Simplex		3.12	1.73	4.08

[1]*Wetwell operating volume calculated based on existing operational set points.*

Force Main Velocities

For three of the six lift stations (Tyler, Nino and Cypress), the force main velocities under simplex pump mode are within generally accepted criteria for self-cleansing and for minimizing headloss. The remaining three lift stations (Los Ositos, Vineyard and Reed) have force main velocities under simplex pump mode that are below generally accepted criteria for self-cleansing and for minimizing headloss. It is recommended that these force mains be maintained on a regular basis by occasionally running the pump station in duplex mode to increase the force main velocity.

Table 6-3. Summary of Lift Station Flows

Lift Station	Tyler	Los Ositos	Vineyard	Nino	Reed	Cypress
Existing Average Daily Flow, gpd	189,821	7,058	75,312	28,944	19,829	28,541
Existing Average Daily Flow, gpm	132	5	52	20	14	20
Peaking Factor	2	2	2	2	2	2
Existing Peak Hour Flow, gpm	264	10	105	40	28	40
Future Average Daily Flow, gpd	602,237	7,058	181,642	28,944	19,829	151,790
Future Average Daily Flow, gpm	418	5	126	20	14	105
Future Peak Hour Flow, gpm	836	10	252	40	28	211
Lift Station Design Capacity, gpm, Simplex	600	450	140	180	200	1,000
Lift Station Tested Capacity, gpm, Simplex [1]	193	249	64	122	153	1,000

[1]Tested lift station capacity was calculated using the time required to run a full pump down cycle (i.e., between the high and low operational set points) and does not account for inflow into the wetwell during the test. Therefore, flows calculated based on this testing methodology are most likely lower than the actual flow.

Tyler Lift Station

The Tyler Lift Station has a design capacity of 600 gpm but a tested operating capacity of 193 gpm. Considering the design operating flow, the lift station is adequately sized to meet existing peak hour flow conditions with one pump running but will not have adequate capacity to meet future peak hour flow conditions. Although the design operating flow is adequate, a telephone interview with Arturo Felix on 6/20/16 revealed the following ongoing concerns with the Tyler lift station:

- The City has had difficulty with operating the lift station since the pumps were upgraded to 20 HP in 2007 and the new pumps do not seem to fit perfectly with the existing lift station configuration.
- The existing lift station physically cannot handle larger pumps than the existing 20 HP pumps.
- The generator at the site is not rated large enough to handle the startup amperage draw from the pumps when there is a power failure. When there is a power outage, the auto-transfer switch to the generator "trips", triggers an alarm, and the pumps do not run. When this happens, City staff has to manually turn the generator back on to allow the pumps to run.
- The two gate valves and the check valve on pump #1 are currently not functional and the City is actively working to replace them.

Because of the future capacity deficiency and the items identified above, it is recommended that the City consider a full replacement of the Tyler lift station with a new triplex submersible or wet pit/dry pit lift station.

Los Ositos Lift Station
The Los Ositos Lift Station has a design capacity of 450 gpm but a tested operating capacity of 249 gpm. Based on existing and future peak hour flows, and despite the lower field tested pumping capacity noted, the lift station will be capable of pumping existing and future peak hydraulic flow in Simplex mode of operation.

Vineyard Lift Station
The Vineyard Lift Station has a design capacity of 140 gpm but a tested operating capacity of 64 gpm. Considering the design operating flow, the lift station has adequate capacity to meet the existing peak hour flow conditions with one pump running but will not have adequate capacity to meet future peak hour flow conditions. As development occurs upstream of the Vineyard lift station, the City should reanalyze the lift station and upsize the pumps as necessary.

Nino Lift Station
The Nino Lift Station has a design capacity of 180 gpm but a tested operating capacity of 122 gpm. Considering the design operating flow, the lift station has adequate capacity to meet both existing and future peak hour flow conditions with one pump running.

Reed Lift Station
The Reed Lift Station has a design capacity of 200 gpm but a tested operating capacity of 153 gpm. Considering the design operating flow, the lift station has adequate capacity to meet both existing and future peak hour flow conditions with one pump running.

Cypress Lift Station
The Cypress Lift Station has a design capacity of 1,000 gpm. Considering the design operating flow, the lift station has adequate capacity to meet both existing and future peak hour flow

conditions with one pump running. This lift station is over-sized, and recommendations to address this will be provided later in this section.

Lift Station Wetwell Capacity

The lift station volumes were calculated, and pump cycle times were computed for each station, based on average day and peak hour flows (running in simplex mode). Cycle times were not able to be computed for the wetwells in duplex mode, as duplex curves were not available. Table 6-4 summarizes the wetwell cycle time calculations.

Table 6-4. Summary of Lift Station Cycles per Hour

	Tyler	Los Ositos	Vineyard	Nino	Reed	Cypress
Wetwell Operating Volume, gallons	952	211	441	330	211	188
Cycles per Hour at Existing ADF	6.5	1.4	4.5	3.2	3.6	6.2
Cycles per Hour at Existing MDF	9.3	2.7	3.6	5.7	6.7	12.2
Cycles per Hour at Future ADF[2]	8.0	1.4	1.7	3.2	3.6	30.1
Cycles per Hour at Future MDF[1,2]	-20.8	2.7	-27.6	5.7	6.7	53.1

[1]Negative values indicate that the MDF exceeds current pump flow rate in simplex mode.

[2]Cypress lift station is currently using a small portion of the total wetwell capacity. Therefore, the calculated number of cycles per hour shown in this table for future flow conditions are higher than they would be if the full wetwell capacity was in use. It is anticipated that the City will adjust the operating levels of the wetwell as the ADF and MDF increase in the future.

Lift station pumps should typically cycle not more than 5 to 6 times per hour at average flow conditions, to limit pump starts. This recommendation, however, should be based on the actual pump manufacturer's information. Pump motors and starters have improved significantly over the years, and thus can withstand more frequent starts than in years past. Pump cycling in excess of the manufacturer's recommendation can lead to increased wear and tear, increased maintenance requirements and premature pump failure. If pumps do not cycle frequently enough, raw sewage is allowed to sit in the wetwell for longer, increasing the likelihood of off-gassing and sulfuric acid attack to the wetwell.

Tyler Lift Station

Although the Tyler lift station has a relatively large operating volume, it appears to be undersized, causing the pumps to cycle too frequently under existing peak hour and future average day conditions. Under future peak hour conditions, the inflow rate exceeds the outflow rate so the calculated number of cycles per hour is negative. Capital improvement recommendations related to the Tyler lift station are included in the next section.

Los Ositos Lift Station

Although the Los Ositos lift station has a relatively small operating volume, it appears to be oversized, causing the pumps to cycle too infrequently under all conditions. It is recommended that the City assess the feasibility of adjusting pump on/off levels to marginally decrease the operating volume.

Vineyard Lift Station

The Vineyard lift station appears to be appears to be oversized, causing the pumps to cycle too infrequently under existing average day and peak hour conditions and future average day conditions. It is recommended that the City assess the feasibility of adjusting pump on/off levels to marginally decrease the operating volume. Under future peak hour conditions, the inflow rate exceeds the outflow rate so the calculated number of cycles per hour is negative.

Nino Lift Station

The Nino lift station appears to be properly sized based on the current operating volume for conditions analyzed.

Reed Lift Station

The Nino lift station appears to be properly sized based on the current operating volume for conditions analyzed.

Cypress Lift Station

Although the operating volume for the Cypress lift station is currently small because of the operational set points, the lift station has a very large wetwell with adequate capacity to accommodate increased flows. The wetwell is sized too large for the design flows, and thus even with adjusting the wetwell operating volume, wastewater tends to sit in the wetwell for extended periods of time and turn septic. This has the potential to cause significant sulfide build up in the wetwell, cause potential odor problems, and create these same problems downstream of the lift station and force main. The City should consider the following recommendations to address the Cypress Lift Station:

- Reduce the horsepower/size of the pumps.
- If reducing the horsepower/size of the pumps does not fully address the concerns described above, compartmentalize the wetwell to reduce its volume, leaving half of the wetwell reserved only for emergency storage. This will ensure better throughput of sewage from the wetwell and keep wastewater fresher. This may be challenging

however, as it creates a maintenance problem to maintain this vacant portion of wetwell, and to clean it out if/when it is used for emergency storage.

Lift Station Capital Improvements

Recommended capital improvements with corresponding capital costs are presented in Chapter 8. A summary of the recommended capital improvements and their justification is included in this section.

Tyler Lift Station

Priority 1 Capital Improvements:

- Upgrade lift station alarms and controls to address lift station problems related to malfunctioning alarms and controls.
- Replace the lift station with either a new triplex wet pit/dry pit lift station or a triplex submersible pump station with self-cleaning wetwell, with shallow valve vault (eliminates confined space entry, except for any future wetwell interior repairs), and with sufficient hydraulic capacity/redundancy to meet future peak flows in the simplex mode of operation. This new station could be located in the City's nearby park located to the west on Tyler Avenue.

Los Ositos Lift Station

Priority 1 Capital Improvements:

- Adjust operational set-points (if possible) to decrease operating volume and increase the number of starts to reduce the likelihood of off-gassing and sulfuric acid attack to the wetwell.
- Consider constructing a fence around the lift station to improve security.

Nino Lift Station

Priority 1 Capital Improvements:

- Adjust operational set-points (if possible) to increase operating volume.

Reed Lift Station

Priority 1 Capital Improvements:

- Adjust operational set-points (if possible) to increase operating volume.

Vineyard Lift Station

Priority 1 Capital Improvements:

- Thorough inspection and repair of the entire lift station. The goal of this inspection and repair process would be to identify and remedy the cause of the discrepancy between the pump design flow and its tested operating flow. If, after completing the inspection and repair process, the operating flow is still significantly lower than the design flow, the City will need to consider additional lift station improvement options.

- Replace pumps if inspection and repair process does not correct design vs. actual flow discrepancy.
- As development occurs upstream of the Vineyard lift station, the City should reanalyze the lift station and make upgrades as necessary.

Cypress Lift Station

Priority 1 Capital Improvements:

- Reduce the horsepower/size of the pumps.
- Compartmentalize the wetwell to reduce its volume, leaving half of the wetwell reserved only for emergency storage. This will ensure better throughput of sewage from the wetwell and keep wastewater fresher. This may be challenging however, as it creates a maintenance problem to maintain this vacant portion of wetwell, and to clean it out if/when it is used for emergency storage.

7: Wastewater Treatment Plant Evaluation and Update

This Chapter presents the wastewater treatment plant evaluation and update. In 2013, Wallace Group, in conjunction with Kennedy Jenks Consultants, prepared a Wastewater Treatment Plant (WWTP) evaluation. This evaluation covered a regulatory review of existing waste discharge requirements (WDRs), wastewater flow and organic loading analyses (and projection of future flows and loadings), field review of the overall plant and individual plant processes, review of the effluent disposal facilities, review of plant staffing needs, and preparation of a report/technical memorandum summarizing recommendations for the plant. This April 2013 Report is on file with the City. The majority of detailed information contained in this April 2013 Report is not duplicated in this Master Plan document, but rather specific items that are updated are described herein.

Definitions

- **BOD5** – biochemical oxygen demand, a measure of the organic waste strength of a wastewater.
- **TSS** – total suspended solids, a measure of the solids suspended in wastewater
- **TDS** – total dissolved solids – minerals and salts that exist in solution state in a water or wastewater

WWTP General Process Description

The treatment process is generally described as follows:

Raw wastewater enters the influent headworks by gravity via a 24" diameter circular gravity sewer pipe from Walnut Avenue, which discharges into a concrete rectangular channel. Raw wastewater then flows through a mechanical rake/screen, and control of the rake is actuated based on ultrasonic level measurement immediately upstream of the mechanical rake. Flow then passes through a coarse manual bar screen, where the channel then splits into two channels. At this point, wastewater flows through one of the two channels, each equipped with a comminutor before passing through a 6" Parshall Flume (in one of the channels, the second channel being the bypass channel) and primary clarifier flow splitter box. Raw wastewater then flows to three primary clarifiers operated in parallel.

Primary waste sludge and scum is pumped to two aerobic digesters for digestion, followed by discharge to sludge drying beds for drying/dewatering and ultimate disposal off-site. Sludge collected from Clarifier #1 and #2 is conveyed to Digester #1, and sludge collected from Clarifier #3 is conveyed to Digester#2.

Primary effluent flows by gravity to a splitter box, where flow may split between Oxidation Pond 1 and 2. Flow from Oxidation Ponds 1 and 2 then go to Oxidation Pond 3; Oxidation Pond 2 may discharge directly to Percolation Pond 4. Flow from Oxidation Pond 3 flows to Percolation Pond 5. Finally, effluent is pumped by a 60-HP pump station through manually maneuvered irrigation piping and is spray disposed on 26 acres of spray disposal fields. Refer to Figure 7-1 for a depiction of this overall plant process. An aerial view of the City's WWTP is shown as Figure 7-2. Figure 7-3 shows the flow patterns at the oxidation and percolation ponds, and Figure 7-4 shows the effluent spray disposal area.

WWTP Design Criteria

The following table (Table 7-1) summarizes the design parameters for the City's existing wastewater treatment plant. This criteria is consistent with that provided in the 2013 Wastewater Evaluation.

Table 7-1. Greenfield WWTP Design Criteria

<table>
<tr><th>Process/Plant</th><th>Criteria, Units</th><th>1.0 MGD CAPACITY</th><th>2.0 MGD CAPACITY</th></tr>
<tr><td rowspan="5">Flows and Loading</td><td>ADWF, mgd</td><td>1.0</td><td>2.0</td></tr>
<tr><td>Peak Flow, process, mgd</td><td>3.0</td><td>6.0[e]</td></tr>
<tr><td>Peak Hydraulic Flow, mgd</td><td>5.0</td><td>10.0[e, q]</td></tr>
<tr><td>BOD_5, mg/L (lb/day)</td><td>240 (2,000)[p]</td><td>240 (4,000)[p]</td></tr>
<tr><td>TSS, mg/L (lb/day)</td><td>240 (2,000)[p]</td><td>240 (4,000)[p]</td></tr>
<tr><td rowspan="3">Headworks</td><td>Headworks Channel, mgd</td><td colspan="2">0.1 to 2.5[n]</td></tr>
<tr><td>Number of Channels, dimensions (inches)</td><td colspan="2">2@31”Wx32”D[l]</td></tr>
<tr><td>Chain & Rake Monster™, quantity</td><td colspan="2">1 each (3.5 mgd)</td></tr>
</table>

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
	(peak hydraulic capacity, mgd)		
	Coarse Bar Screen, number	1	
	Comminutor, mgd	0.1 to 2.5[n]	
	Comminutor, quantity@HP	2@5HP each[l]	
	Flow Measuring/Parshall Flume, mgd	0.1 to 2.5 6" Throat[l, n]	
Primary Sedimentation	Number of Units	2@0.5 mgd	2@0.5 mgd, 1@1.0 mgd
	Diameter, ft	2@30'	2@30' 1@45'
	Removal Rate, %SS	60[a]	
	Effective Volume, ft3	2@6,126 ft^3	2@6,126 ft^3 1@12,253 ft^3
	Surface Loading, gpd/sf	707[a]	
	Detention Time, hours	2.2[b]	
	Weir Overflow Rate, gpd/LF	5,300	
Sludge Digestion (aerobic)	VSS (% of TSS)	75	
	VSS Reduction, %	40	

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
	Volume Treated Per Day, ft^3	347	694
	Number of Units	1	2
	Size, each unit (ft)	30’ Dia x 13.5’ Deep	
	Solids Retention Time, days	30	
	Rotary Lobe Blower, HP, each unit	10	
	Blower capacity, cfm	500	
	Loading Rate, lb VSS/ft^3-day	0.04 (0.06)[l]	
Sludge Drying (Lagoons/beds)	Number of beds	6[c, n]	
	Loading, lb/year	315,360[d]	630,720[e]
	Area, each bed, sf	62,500[n]	
	Volume, ft^3	125,000[d, n]	
	Loading Rate, lb dry solids/SF/day	0.006[l]	0.012[e, n]
	Loading Rate, lb/ft^3/year	2.52[e]	5.04[e, n]
Oxidation Ponds	Number	3	
	Surface Area, Total	6.25[j], 7.6[k]	
	Depth, ft	5	
	Detention Time, days	5.1[j], 14.9[k]	2.5[j], 7.4[k]

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
	BOD_5 Loading Rate, lbs/acre-day	200[j]	400[j]
	BOD_5 Loading Rate, kg/acre-day	78[k]	156[k]
	Aerators	None	6@15 HP each[f]
Percolation Ponds	Number	2	
	Area, total, acres	4.21	
	Depth, each, ft	5	
	Percolation Rate, gal/acre-day	47,850	
	Application Rate, inches per day	2.3	
	Disposal Capacity, mgd	0.2[g]	
Spray Disposal Fields	Total Area, acres	13	26
	Application rate, inches/day (inches/year)	2.3 (70)	
	Capacity, mgd	0.812	1.62[h]

[a]*Stated for original two 0.5 mgd clarifiers only. Design % Removal of BOD is not stated.*
[b]*Detention time assumed to be based on ADWF.*
[c]*Only three beds were observed during February 2013 site visit.*
[d]*Need to verify if this is loading per bed, or total.*
[e]*Estimated value, based on same ratio for 1.0 mgd criteria.*
[f]*Proposed in June 4, 2004 letter, but not installed.*
[g]*It is noted that in the June 4, 2004 letter, it was stated that the oxidation and disposal ponds had never been cleaned or dredged since their construction in the 1970s, and that these ponds effectively do not percolate (currently).*
[h]*This means total effluent disposal capacity is 1.8 mgd with percolation ponds and spray disposal. See note (g) above also. In order to yield 0.2 mgd disposal capacity in the percolation ponds, they will need to be properly rotated, dried, ripped and solids removed. Winter storage or redundancy/buffer should also be considered.*

[j]Based on 2003 Freitas Report.
[k]Based on June 2004 Freitas Letter Report.
[l]Section VI, RM Associates Report of Waste Discharge Report, July 2001.
[m]The O&M Manual does not indicate peak hydraulic capacity of this equipment.
[n]April 5, 2013 letter from Freitas+Freitas indicating original design criteria is sufficient for 2.0 mgd capacity.
[p]Refer to Section on "Wastewater Characteristics". City will need to re-evaluate organic loading based on most recent Annual Report data.
[q]Observed peaking factors from flow chart recorders suggest this peak value may not be realized at the plant. Further evaluation is warranted.

Waste Discharge Requirements and Supporting Documents

The City of Greenfield WWTP is regulated by the Regional Water Quality Control Board (Regional Board), Central Coast Region, by Waste Discharge Requirements (WDR) Order No. R3-2002-0062. These WDRs were adopted May 31, 2002. Key aspects of the City's WDRs are summarized as follows:

- Current plant capacity is stated as 1.0 mgd, with City plans for expansion to "at least 1.5 mgd". Flows to the WWTP in 2002 were reported at 0.91 mgd.
- Specification B.1, wastewater flows shall not exceed 1.0 mgd until certain facility improvements are completed and supporting design documentation is submitted to and accepted by the Regional Board.
- Specification B.4, effluent disposal operations shall not cause downgradient monitoring wells to exceed 8 mg/L nitrates (as N).
- Specification B.5, effluent disposal operations shall not cause downgradient monitoring wells to see "significant increases" in mineral quality.
- Specification B.11, effluent disposal ponds shall be alternated to permit emptying for maintenance purposes.
- Specification B.12, disposal ponds shall be dried and disked at least annually.
- Specification B.13, wastewater application to spray irrigate disposal areas shall be managed to prevent ponding.
- Specification B.14, wastewater application to spray disposal areas shall not take place during rains.
- Specification B.16, spray disposal areas shall be operated using a regular rotation. Rotation from one irrigation area to another shall occur at least weekly. Between applications, irrigated areas shall be allowed to dry at approximately the field moisture condition of non-irrigated areas.
- Specification B.17, all solids generated must be reclaimed or disposed of in an acceptable manner.

Figure 7-1. Aerial View of City of Greenfield WWTP

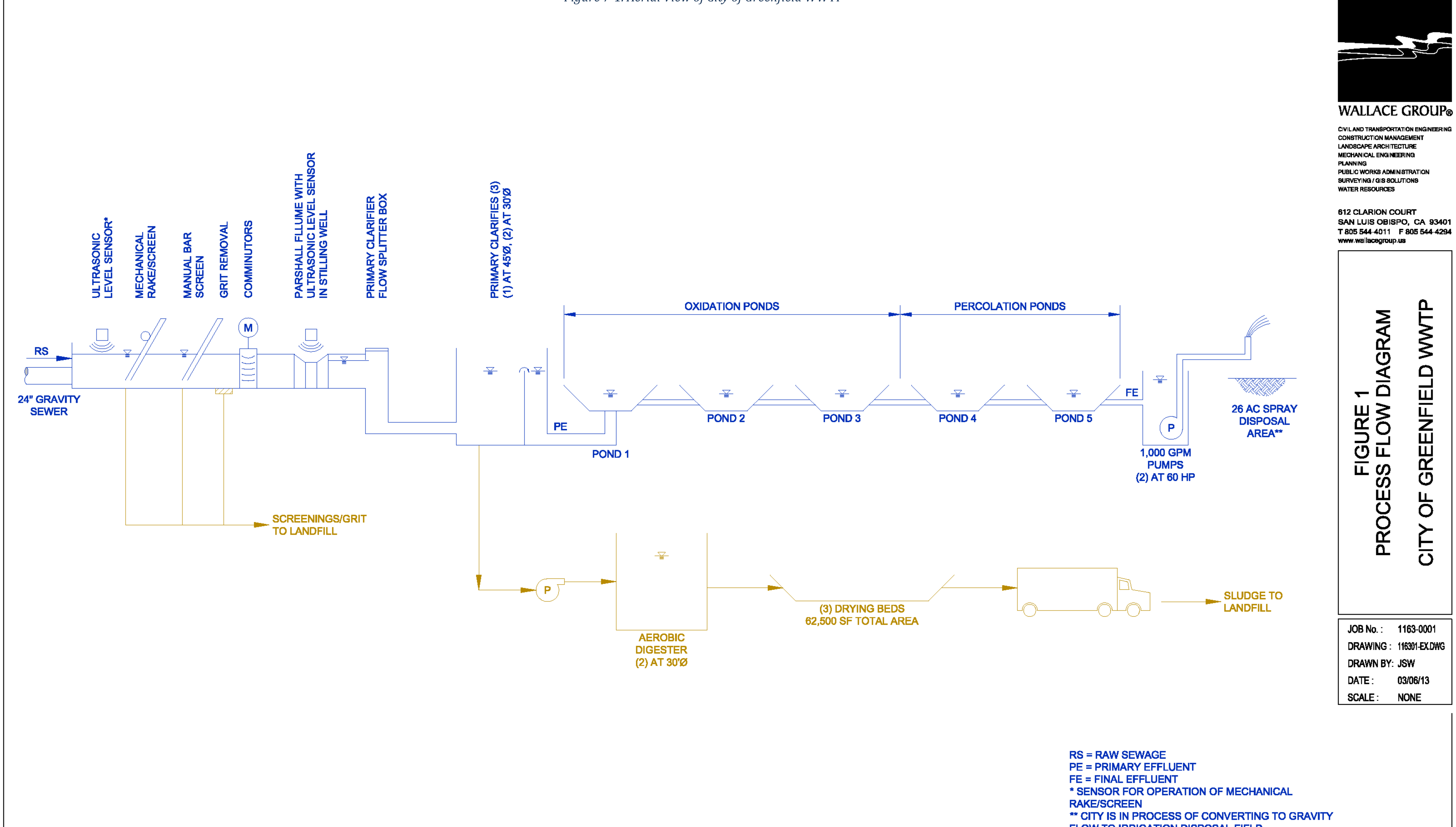


Figure 7-2. Aerial View of City of Greenfield WWTP

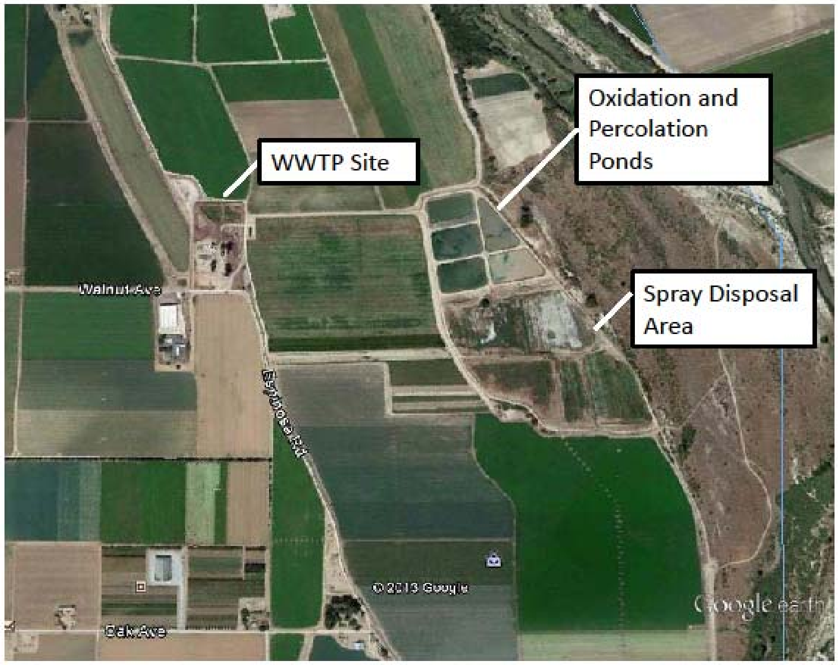


Figure 7-3. Oxidation and Percolation Pond Layout

Figure 7-4. Effluent Spray Disposal Fields

- Specification B.18, all storm water contacting domestic wastewater shall be contained on site.
- Specification B.19, best management practices shall be implemented to minimize the inflow and infiltration of storm water into the facility.
- Provision C.5, City shall evaluate salt management practices and implement a long term salt management program. City shall submit report to Regional Board by March 1, 2003.
- Provision C.6 and C.7, City shall submit a report to the Regional Board by November 30, 2002 addressing groundwater monitoring wells and hydraulic gradient in the area of the facility. If disposal system is insufficient, City shall submit engineering report by March 1, 2003 evaluating various wastewater disposal options and shall consider water recycling as an option.

It is noted that there are no specific effluent treatment standards imposed in these WDRs.

Key aspects of the WDR monitoring requirements are as follows:

- Influent wastewater monitoring includes:
    - Daily flow metering, maximum daily flow metered, and mean daily flow (calculated).

  - Quarterly BOD_5 and TSS (24-hour composite), settleable solids and pH (grab).
  - Annual TDS, sodium, chloride, sulfate, boron (24 hour composite)
- Pond monitoring, weekly grab samples for pH and dissolved oxygen.
- Effluent monitoring (discharged to spray disposal area):
  - Quarterly grab samples for pH, BOD_5, TSS, settleable solids, TDS, sodium, chloride, boron, sulfate, nitrite (as N), nitrate (as N), total Kjeldahl nitrogen (as N), total nitrogen (as N).
  - Annual grab sample for heavy metals.
  - Once every 5 years, grab sample for volatile organics and pesticides.
- Solids/biosolids monitoring:
  - Reported tonnage or yardage of sludge removed, each load.
  - Representative samples during transport/removal, for moisture content, nitrate (as N), pH, oils and grease, heavy metals
  - At least once every 5 years prior to transport or disposal, pesticides, organic lead and PCBs.

Prior Observations and Plant Conditions

A brief summary of prior plant conditions is summarized as follows:

- Screenings Device. The automatic bar screen which precedes the comminutors may be the source of rags and debris entering the plant. These rags and debris cause maintenance issues at the primary clarifiers and sludge pumps.
- Clarifiers. Significant accumulation of scum and grease has been observed on the clarifier surfaces, and could be a reflection of ineffective initial treatment units and possibly excess fats, oil and grease (FOG) in the collection system.
- Aerobic Digesters. The digesters operate on a fill and draw basis. Air blowers to the digesters are operated on a time clock sequence. It was uncertain whether there is any monitored program and a basis for such a program such as dissolved oxygen levels within the digesters. It was also uncertain as to the basis for when sludge is withdrawn from the digesters and sent to the sludge storage/drying lagoons.
- Sludge Lagoons. Odors from these lagoons seem to indicate that solids digestion may only be partial. In addition, an abundance of weeds was observed on the embankments, along with observed bank erosion.
- Oxidation Ponds. The major issue with the oxidation ponds is that they are being overloaded organically. A number of odor complaints have been received over the last several years, and the City is in the process of adding aeration to the ponds (described later in this Chapter).
- Percolation Ponds. The ponds seemed to remain full of water most if not all of the time, with little time to dry and scarify the bottoms. These ponds and the oxidation ponds had a lot of weed growth on the perimeters.

- Effluent Spray Disposal Area. Treated wastewater in excess of that disposed of from the infiltration ponds, is pumped to 26 acres of spray disposal fields. This system is comprised of a pumping station and irrigation pipe to spray heads within the disposal area. It was reported that the spray system utilizes 1,000 gpm 60 H.P. pumps and that the irrigation pipe is manually changed and moved to rotate use of the spray disposal fields. Operations have since been changed to thin spread effluent on the disposal fields.

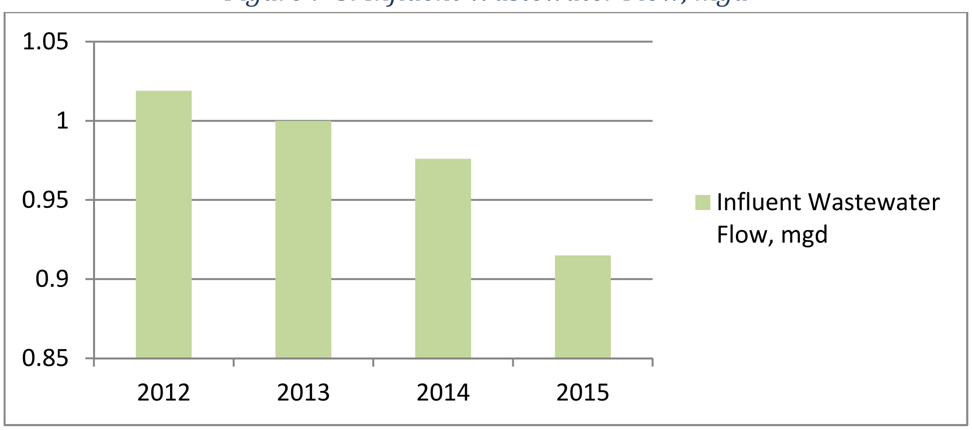
Existing and Projected Future Wastewater Characteristics

The following subsections summarize influent wastewater flows, influent organic waste strength (expressed in BOD_5 and TSS) and effluent quality (also expressed in BOD_5 and TSS).

Existing Plant Influent Flows

Influent wastewater flows over the past four years has decreased, while population continued to marginally increase. Year 2015 showed the most notable drop in measured flow, due to significant water conservation measures mandated by the drought. Figure 7-5 shows the wastewater influent flow trend for the past 4 years.

Figure 7-5: Influent Wastewater Flow, mgd



Future Plant Flows

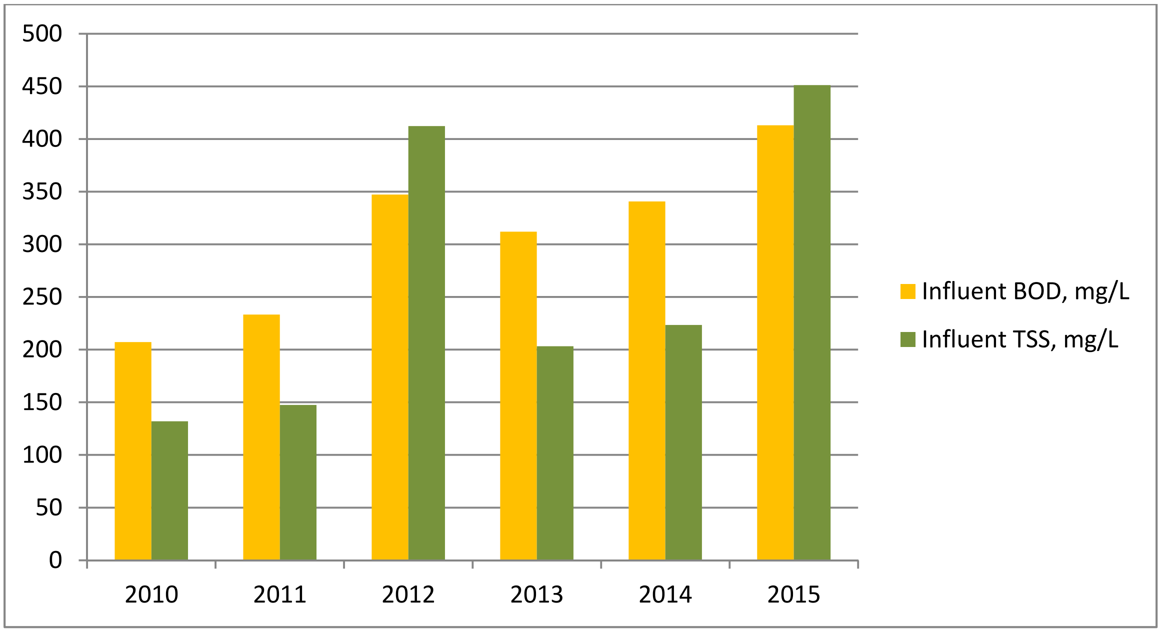
For the purposes of this Master Planning process, the City has provided a projected population growth rate of 2.5%. This growth rate results in a total population of 28,400 by 2035, which correlates to the 20 year planning horizon for the City's water and wastewater master planning effort.

Current wastewater treatment plant flows have dropped to below 0.95 mgd recently due to drought/water conservation efforts. This results in a per capita wastewater flow rate of 60 gallons per capita per day (gpcd). Using the 2012 flow and population data, the per capita wastewater flow was 62 gpcd, just slightly higher than the 2015 per capita flow rate. Using the build-out population of 28,400, the Year 2035 wastewater flow rate is expected to reach 1.7 mgd (average dry weather flow) to 1.8 mgd.

Existing Influent BOD5 and TSS

Influent waste strength is typically measured as organic waste strength expressed in BOD_5 and TSS. These two values typically define the municipal wastewater strength, by which most biological wastewater processes are designed and compared against for treatment effectiveness. Figure 7-6 portrays the annual average BOD_5 and TSS results for Years 2010 through 2015. Based on these annual average results, there appears to be a clear upward trend in the organic waste strength. This upward trend is due mainly to water/wastewater reductions that concentrate organic matter in wastewater.

Figure 7-6: Summary of Organic Waste Strength at WWTP



Tables 7-2 and 7-3 summarize the quarterly influent BOD_5 and TSS results, respectively. These results are presented to show the variability in values each quarter. The wide variation in results suggests possible anomalies in sampling, and the City should review sampling protocol and ensure that 24-hour composite samples are being taken per established protocol, stored properly (refrigerated) during the sampling period. If and when anomalies arise (for example, 684 mg/L BOD during the 4th quarter of 2015), the City should consider re-sampling to verify results. It is also recommended that additional composite sampling be performed on varying days of the week including weekends, to ascertain if there are any trends and variations in waste strength based on calendar day of the week. Additional analysis can sometimes identify sources of large loading demands from area restaurants or other commercial establishments.

Table 7-2. Summary of Influent BOD_5 Sampling at WWTP

Quarter	2015	2014	2013	2012	2011	2010
1st	421	256	361	576	178	272
2nd	309	218	379	228	255	187
3rd	238	528	266	365	212	200
4th	684	361	242	220	288	170
Annual Average	413	341	312	347	233	207

Table 7-3. Summary of Influent TSS Sampling at WWTP

Quarter	2015	2014	2013	2012	2011	2010
1st	544	58	250	477	122	146
2nd	590	43	234	210	72	208
3rd	38	658	57	684	182	106
4th	633	135	272	278	214	68
Annual Average	451	224	203	412	148	132

Future Influent BOD5 and TSS

It is expected that this upward trend in organic waste strength has reached a plateau, due to the fact that further reduction in per capita wastewater flows are not likely to occur. However, it is important to understand the relationship between wastewater strength and organic loading to the plant. In the 2013 WWTP Evaluation Report, the following future values were proposed for wastewater planning purposes:

- Influent BOD5, 300 mg/L (5,000 lbs/day@2.0 mgd flow)
- Influent TSS, 275 mg/L (4,600 lbs/day@2.0 mgd flow)

For this planning period, and projecting 1.8 mgd wastewater flows, if the prior projected loading of 5,000 pounds per day BOD_5 is maintained, this results in a projected influent BOD_5 of 333 mg/L. The point made here, is that the overall organic loading to the plant at build-out will not change appreciably, but the flows and corresponding concentrations of organic matter in the wastewater will continue to vary over the years. As mentioned earlier, the City should continue to monitor waste strength at the plant, as it will be important from an operations standpoint.

Wastewater Flow Trends

In 2013, Wallace Group evaluated flow trends relative to inflow and infiltration (I/I). Additional evaluation was not warranted at this time. Prior results are summarized in this section. A select interval of chart recordings were reviewed during winter 2012. This interval was selected as there were mostly dry weather days preceding this interval, with one rain event on November 30, 2012. Influent wastewater flows were evaluated, and no unusual spikes in flow were observed during the rain days. The largest peaking factor observed was 2.3. Thus, the City does not appear to have an I/I problem that warrants further investigation. The peak flow values and peaking factors in Table 7-1 address the existing plant flow trends adequately.

Wastewater Pond Aeration Improvements

The construction of the wastewater pond improvements is now under way. Consistent with the 2013 recommendations, plans were prepared to improve the oxidation ponds by deepening and re-conditioning the ponds, adding synthetic liners to Ponds 1, 2 and 3, adding 90 HP of surface aerators. Once these improvements are completed, the ponds will be capable of processing 2.0 mgd capacity. These improvements are anticipated to be complete by late 2016 or early 2017. The City was awarded CDBG grant monies of approximately $1.4 million. The total project cost exceeds this amount by approximately $500,000. The additional costs stem from recommending deepening and lining of the ponds, and significant improvements to yard piping and hydraulic control structures. The latter will afford the City more flexibility in operating the ponds. Once completed, the improvements will also address the on-going odor complaints being received by local residents. The additional cost not covered by CDBG grant monies should be considered a near-term capital improvement.

Other Major Plant Improvements

In addition to the pond aeration project, there are a number of other key plant improvements will be needed to be addressed as part of the 10-year CIP period. These improvements include addressing the plant headworks, sludge management/sludge drying, and modernization of the existing administration building.

Headworks

The existing headworks to the wastewater plant is in fair to poor condition, and only rated at 1.0 mgd. It is anticipated that the headworks should be replaced within the next 5 years. The headworks replacement or overhaul should include screenings/rake, influent grinder, Parshall Flume, bypass channel, add grit chamber. The construction cost is estimated at $1,000,000, or $1.4 million total.

Sludge Management

The City's existing sludge drying ponds are substandard and in need of significant upgrades, or the sludge drying area should be moved to a different location all together. On December 11 and 12, 2014, a major storm pelted the entire State of California. The Central Coast was hit hard by this storm, and the resulting runoff in the City of Greenfield caused the unlined ditch which conveys stormwater to the storm ponds within this same sludge drying area, to breach upstream of Stormwater Pond No. 1 (immediately upstream of the connecting pipe to Stormwater Pond No. 1).

Wallace Group assisted the City with interim repairs, which were completed just prior to the start of the rainy season in 2015. Although the stormwater repairs were completed, it is recommended that the City consider separating the stormwater management from sludge management on the WWTP site. From a long-term perspective, it is recommended that new sludge drying facilities be constructed outside of this existing sludge drying area, as the City will need to consider lined sludge beds (concrete or pavement) with sufficient surface area to for effective drying of sludge. The contours and grades of the existing sludge pond area are not conducive to providing flat and level sludge drying beds. A more detailed study is needed to determine the sludge bed area required, and suitable location(s) for the new sludge drying facilities.

It is recommended that conventional paved sludge drying beds be added to the plant, and that the existing sludge drying bed area be re-purposed and/or be dedicated to stormwater management only. It is estimated that new sludge facilities will be $500,000 construction, or a total of $700,000 (hard and soft costs).

Administration Building

The City's existing administration building at the WWTP is old and reaching the end of its useful life. A new administration building should include amenities such as laboratory facilities and sanitation/shower facilities, and provisions for the City to centralize SCADA and treatment plant controls in this building. It is estimated that a new modern administration with laboratory and other amenities, will cost $750,000 to construct, or approximately $1.1 million total.

Projects Beyond the 10-Year CIP Period

One key project that may be required in the future, is to address the existing aerobic sludge digesters. As part of the 2013 WWTP study, it was noted that use of aerobic digestion for primary clarifier solids is unusual, and that addition study should be conducted to evaluate the conversion to anaerobic digestion. It may be prudent to conduct this study as part of further study of the sludge drying beds. Regardless, for future planning purposes, the City should consider employing anaerobic digestion in lieu of current aerobic digestion process, estimated at $2 million construction, or $2.8 million total.

Wastewater Treatment Plant Classification

In early 2014, the City requested Wallace Group to assist the City with classification of the wastewater treatment plant. In 2013, the plant had been inadvertently classified as a Class III wastewater treatment plant, due to the City's application providing conflicting information with regards to the type of treatment processes at the plant. The prior application had indicated the City uses an activated sludge (conventional) process, which led State Board Staff to classify the WWTP as a Class III facility. Wallace Group prepared a technical letter and revised application clarifying the treatment processes at the WWTP, and the plant was subsequently re-classified by the State Board as a Class II treatment facility. This re-classification further allows the City's current Class II operator, Arturo Felix, to oversee operations of the WWTP as a Grade II WWTP Operator, in compliance with Title 23, Division 3, Chapter 26, Section 3670.1(b)(1), Certification Requirements for Operating Wastewater Treatment Plants. The re-classification letter, dated March 7, 2014, is on file with the City.

Wastewater Treatment Plant Staffing Needs

The City's current staffing requirements were reviewed as part of the 2013 WWTP report. The City currently employs four full-time operators, including Arturo Felix, Grade 2 (chief plant operator), and one Grade 1 operator. The other two operators do not have wastewater operator certification, and thus tasks and responsibilities are limited with respect to wastewater operations. In 2014, Wallace Group assisted the City with preparation of a standard operating procedure (SOP) for WWTP staffing (included as Appendix C). The purpose of this procedure is to ensure that the City of Greenfield remains in compliance with Title 23 California Code of Regulations (CCR) Sections 3670 to 3719 which governs the classification of wastewater treatment plants (WWTP) and operator certification requirements.

Based on the size and complexity of the City's wastewater facility, the CPO should be a Grade 2 operator, as is currently provided. Other plant staff may be Grade 2 or lower.

Using an excel spreadsheet program (dated August 2006) based on a USEPA Publication "Estimating Staffing for Municipal Wastewater Treatment Facilities", dated 1973, staffing needs for the City's wastewater plant were calculated. Consideration was given to current wastewater flows (~1.0 mgd) and current plant improvements, and desired plant rated capacity of 2.0 mgd and anticipated near-term improvements to include pond aeration.

A "sensitivity" analysis was conducted, using variable inputs to the program for flows ranging from 1.5 to 2.0 mgd, and with the current oxidation pond operation and expected future addition of aeration to the ponds. Both variables resulted in a recommended range of 3 to 4 full-time staff to meet all plant operational needs for a wastewater plant of the City's size and complexity.

Thus, it is recommended that the City staff a minimum of four operations staff for current and near-term future plant needs. Based on the recommendations that will be made regarding needed plant maintenance in the short-term (such as weed abatement, re-condition pond embankments, sludge removal, etc.), the City's current planned staffing level of four staff is likely a minimum requirement to achieve the short-term needs in a reasonable time frame. It is noted that the City currently has two vacant positions for Utility Worker, to be allocated to the wastewater treatment plant. These positions should be filled as soon as is practicable.

8: Capital Improvement Projects

This Chapter presents the proposed Capital Improvement Projects (CIP), with a brief description of the proposed projects and a preliminary cost estimate for each proposed improvement for the City of Greenfield (City). Basis of Capital Improvement Program Costs

The CIP costs were developed based on engineering judgment, confirmed bid prices for similar work in the area, consultation with vendors and contractors, established budgetary unit prices for the work, and other reliable sources. Hard construction costs are typically escalated by a factor of 1.4, to allow budget for "soft costs" that include preliminary engineering, engineering, administration, construction management and inspection costs. Some projects may have factors other than 1.4 depending on project type. All CIP costs are expressed in 2016 dollars, using McGraw-Hill ENR Construction Cost Index of **10242**, and will need to be escalated to the year or years scheduled for the work. The unit cost for new gravity sewers includes the proposed pipelines, manholes, lateral re-connections, sewer bypassing, traffic control, etc., and all other aspects of sewer system construction.

Timing of Recommended Improvements

There are some projects triggered by existing deficiencies and some projects triggered by future development. The existing deficiencies are considered near-term projects, and are recommended to be completed within the next 10 years.

There are also projects that are triggered by potential future development, for which timing is difficult to ascertain. These long-term projects are listed in no particular order as they will be prioritized based on timing and location of future development.

Table 8-1 provides a summary of all the existing recommended CIPs, or Near Term Projects. Table 8-1 also provides an estimate of the construction and "soft" costs for each project. Table 8-2 provides a summary of the future recommended CIPs, or Long Term Projects, and their estimated costs.

Table 8-1: Existing CIPs

Project #	Title	Description	Quantity	Length (ft)	Existing Diameter (in)	New Diameter (in)	Street	Location	Construction Cost ($)		Total Project Cost ($)
1-1	Apple Avenue 1	Connect 12" line to 21" line upstream of Highway 101 crossing	1	20	12	12	Apple Avenue	Apple Avenue at 5th Street Alley	25,000	LS	40,000
1-2	Apple Avenue 2	Replace 8" VCP with 12" due to capacity deficiency	1	640	8	12	Apple Avenue	Apple Avenue south line between 7th Street and El Camino Real	112,000	LS	200,000
1-3	Vineyard Drive	Replace 10" VCP with 12" due to capacity deficiency	1	780	10	12	Vineyard Drive	Vineyard Drive from Cabernet Avenue to Vineyard Lift Station	136,500	LS	200,000
1-4	Apple Avenue 3	Replace 8" with 12" due to capacity deficiency	1	350	8	12	Apple Avenue	Apple Avenue north line from 7th Street to 8th Street	61,250	LS	90,000
1-5	Apple Avenue 4	Replace 8" with 12" due to capacity deficiency	1	220	8	12	Apple Avenue	Apple Avenue from Larson Lane to mid-block manhole east of Larson Lane	38,500	LS	60,000
1-6	Walnut Avenue	Abandon 12" gravity sewer from Walnut/2nd to the WWTP	1	NA	12	---	Walnut Avenue	2nd Street to WWTP	25,000	LS	40,000
LS-1	Tyler Lift Station	New Lift Station	1	LS	---	---	Tyler Lift Station	Tyler Lift Station	355,000	LS	500,000
LS-2	Cypress Lift Station	Operational Improvements - Decrease Pump size	1	LS	-	-	Cypress Lift Station	Cypress Lift Station	25,000	LS	40,000
LS-3	Tyler Lift Station	Operational Improvements	1	LS	-	-	Tyler Lift Station	Tyler Lift Station	10,000	LS	20,000
WWTP-1	WWTP	Complete Pond Aerator Addition Project	1	LS	---	---	WWTP	WWTP	500,000	LS	700,000
WWTP-2	WWTP	Upgrade Headworks	1	LS	---	---	WWTP	WWTP	1,000,000	LS	1,400,000
WWTP-3	WWTP	New Administration/Laboratory Building	1	LS	---	---	WWTP	WWTP	750,000	LS	1,050,000
WWTP-4	WWTP	Sludge Beds	1	LS	---	---	WWTP	WWTP	500,000	LS	700,000
										Total Existing Project Costs	$ 4,320,000

Table 8-2: Future CIPs

Project #	Title	Description	Quantity	Length (ft)	Existing Diameter (in)	New Diameter (in)	Street	Location	Construction Cost ($)		Total Project Cost ($)
F-1	Apple Avenue 5	Replace 12" with 18" due to capacity deficiency as a result of future development	1	840	12	18	Apple Avenue	Apple Avenue north line from 5th Street to 7th Street	189,000	LS	270,000
F-2	WWTP	Replace 14" with 24" due to capacity deficiency as a result of future development	1	220	14	24	Walnut Avenue	End of 24" line on Walnut Avenue to WWTP headworks	60,500	LS	90,000
F-3	Apple Avenue 6	Replace 12" with 18" due to capacity deficiency as a result of future development	1	250	12	18	Apple Avenue	Apple Avenue from 5th Street to 5th Street Alley	56,250	LS	100,000
F-4	Elm Avenue	Replace 8" with 10" due to capacity deficiency as a result of future development	1	1,650	8	10	Elm Avenue	Elm Avenue from Heidi Drive to Via Salvgano	247,500		350,000
F-5	Apple Avenue 7	Replace 8" with 12" due to capacity deficiency as a result of future development	1	1,500	8	12	Apple Avenue	Apple Avenue from 11th Street to 12th Street	262,500		370,000
										Total Future Project Costs	$ 910,000

9: Appendix A – Flow Monitoring Results

Site Report

11-02-2015

Confidential Proprietary Information

Wallace Group

399 2nd St
Greenfield,CA 93927

Site 1

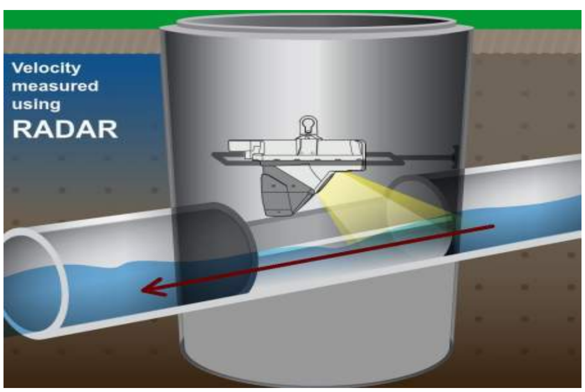
Access:
Manhole within intersection of Apple Av & 2nd St

System Type:
Sanitary [X] Storm [ ]

Install Date: 9/24/2015

Map

Technology



Traffic Plan

Flow Meter

Meter Depth: 79.25"

Meter SN:*

Slow & steady hydraulics

Avg Velocity	Avg Measured Level	Multiplier
1.1 fps	4.19"	1.0

Gas

O2	H2S	CO	LEL
20.9	0	0	0

Notes

*

Traffic Safety

Used cones, signs & vehicle.

Land Use

Residential	Commercial	Industrial	Trunk
X			

Manhole Depth	97"
Pipe Size	12"
Inner Pipe Size (In/Out)	12"/12"
Pipe Shape	Round
Pipe Condition	Good
Manhole Material	Concrete
Silt (inches)	1"
Velocity Profile Data	*
Velocity Profile Taken	
Sensor Offset	17.75"
Sensor Dist. to Crown	5.75"
Flow Direction	Upstream
Flow Heading	North

Meter Site Document

Wallace Group

Site 1

399 2nd St
Greenfield,CA 93927

Site

Manhole Before Install

Installation Process

Installed

Upstream

Downstream

Utility Systems Science and Software

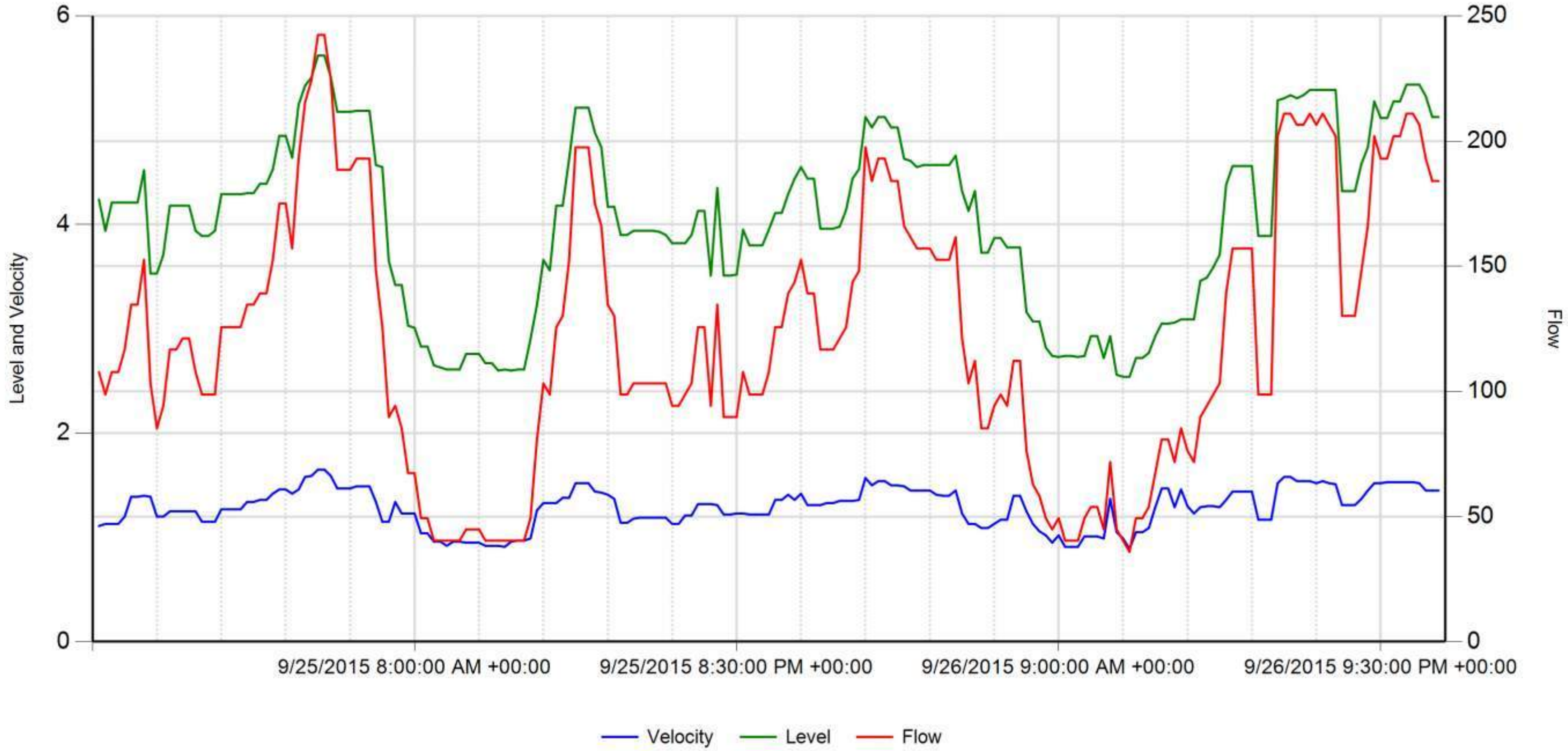
Report Date: 11/02/2015
Customer: Wallace Group
Group: WebFlow
SiteID: 2295

Statistics from Site 1 (2nd & Apple): 09/24/2015 thru 10/28/2015

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Rain
9/24/2015	141.43	242.37	67.32	0.20	0.35	0.10	1.34	1.65	1.11	4.41	5.62	3.03	203,656	
9/25/2015	111.41	197.49	40.39	0.16	0.28	0.06	1.25	1.57	0.91	3.89	5.12	2.60	160,435	
9/26/2015	122.63	210.95	35.91	0.18	0.30	0.05	1.28	1.58	0.83	4.06	5.34	2.54	176,593	
9/27/2015	122.21	224.42	40.39	0.18	0.32	0.06	1.26	1.61	0.92	4.07	5.40	2.55	175,987	
Week:	**124.42**	**242.37**	**35.91**	**0.18**	**0.35**	**0.05**	**1.28**	**1.65**	**0.83**	**4.11**	**5.62**	**2.54**	**716,670**	
9/28/2015	110.66	215.44	35.91	0.16	0.31	0.05	1.25	1.61	0.82	3.89	5.35	2.50	159,358	
9/29/2015	109.36	233.39	35.91	0.16	0.34	0.05	1.23	1.62	0.91	3.89	5.51	2.52	157,472	
9/30/2015	104.31	228.90	26.93	0.15	0.33	0.04	1.22	1.66	0.83	3.77	5.47	2.35	150,201	
10/1/2015	105.52	201.97	35.91	0.15	0.29	0.05	1.21	1.68	0.87	3.83	5.16	2.44	151,952	
10/2/2015	108.23	210.95	35.91	0.16	0.30	0.05	1.25	1.60	0.89	3.84	5.21	2.49	155,857	
10/3/2015	120.90	215.44	35.91	0.17	0.31	0.05	1.27	1.69	0.86	4.04	5.33	2.53	174,097	
10/4/2015	134.18	264.81	35.91	0.19	0.38	0.05	1.31	1.76	0.81	4.15	5.74	2.41	193,222	
Week:	**113.31**	**264.81**	**26.93**	**0.16**	**0.38**	**0.04**	**1.25**	**1.76**	**0.81**	**3.92**	**5.74**	**2.35**	**1,142,159**	
10/5/2015	106.18	233.39	35.91	0.15	0.34	0.05	1.20	1.66	0.90	3.87	5.38	2.44	152,894	
10/6/2015	113.38	210.95	35.91	0.16	0.30	0.05	1.25	1.55	0.86	3.93	5.35	2.52	163,262	
10/7/2015	118.57	255.83	31.42	0.17	0.37	0.05	1.29	1.72	0.91	3.95	5.62	2.43	170,735	
10/8/2015	111.41	269.30	35.91	0.16	0.39	0.05	1.21	1.79	0.86	3.95	5.69	2.43	160,435	
10/9/2015	105.66	188.51	35.91	0.15	0.27	0.05	1.18	1.57	0.91	3.95	5.11	2.53	152,154	
10/10/2015	121.61	210.95	35.91	0.18	0.30	0.05	1.26	1.56	0.93	4.05	5.32	2.44	175,111	
10/11/2015	116.60	210.95	35.91	0.17	0.30	0.05	1.25	1.70	0.90	3.98	5.33	2.45	167,908	
Week:	**113.34**	**269.30**	**31.42**	**0.16**	**0.39**	**0.05**	**1.23**	**1.79**	**0.86**	**3.95**	**5.69**	**2.43**	**1,142,500**	

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	**Avg**	**Max**	**Min**	**Avg**	**Max**	**Min**	**Avg**	**Max**	**Min**	**Avg**	**Max**	**Min**	**Total Gal**	**Rain**
10/12/2015	106.97	251.35	31.42	0.15	0.36	0.05	1.17	1.69	0.78	3.95	5.61	2.50	154,039	
10/13/2015	116.70	251.35	26.93	0.17	0.36	0.04	1.24	1.70	0.81	3.99	5.64	2.33	168,042	
10/14/2015	115.99	255.83	35.91	0.17	0.37	0.05	1.24	1.77	0.94	3.96	5.49	2.33	167,033	
10/15/2015	104.63	242.37	35.91	0.15	0.35	0.05	1.17	1.68	0.85	3.85	5.52	2.52	150,673	
10/16/2015	105.15	193.00	31.42	0.15	0.28	0.05	1.21	1.59	0.77	3.80	5.06	2.45	151,413	
10/17/2015	119.88	233.39	31.42	0.17	0.34	0.05	1.26	1.63	0.81	3.99	5.52	2.47	172,620	
10/18/2015	126.75	251.35	35.91	0.18	0.36	0.05	1.26	1.70	0.85	4.10	5.62	2.50	182,517	
Week:	**113.72**	**255.83**	**26.93**	**0.16**	**0.37**	**0.04**	**1.22**	**1.77**	**0.77**	**3.95**	**5.64**	**2.33**	**1,146,337**	
10/19/2015	115.15	215.44	26.93	0.17	0.31	0.04	1.28	1.58	0.76	3.88	5.34	2.45	165,821	
10/20/2015	108.37	206.46	35.91	0.16	0.30	0.05	1.22	1.73	0.55	3.86	5.23	2.55	156,059	
10/21/2015	108.89	246.86	31.42	0.16	0.36	0.05	1.22	1.67	0.79	3.86	5.59	2.41	156,799	
10/22/2015	112.39	233.39	31.42	0.16	0.34	0.05	1.26	1.61	0.72	3.87	5.63	2.43	161,849	
10/23/2015	107.02	197.49	26.93	0.15	0.28	0.04	1.23	1.49	0.80	3.82	5.24	2.34	154,106	
10/24/2015	123.90	219.93	31.42	0.18	0.32	0.05	1.25	1.58	0.77	4.09	5.46	2.46	178,410	
10/25/2015	123.24	224.42	26.93	0.18	0.32	0.04	1.26	1.55	0.78	4.05	5.58	2.39	177,468	
Week:	**114.14**	**246.86**	**26.93**	**0.16**	**0.36**	**0.04**	**1.24**	**1.73**	**0.55**	**3.92**	**5.63**	**2.34**	**1,150,511**	
10/26/2015	126.75	273.79	35.91	0.18	0.39	0.05	1.32	1.71	0.83	4.03	6.03	2.56	182,517	
10/27/2015	117.96	264.81	35.91	0.17	0.38	0.05	1.22	1.77	0.85	4.04	5.68	2.55	169,860	
10/28/2015	80.09	152.60	31.42	0.12	0.22	0.05	1.11	1.35	0.62	3.42	4.66	2.82	115,327	
Week:	**108.26**	**273.79**	**31.42**	**0.16**	**0.39**	**0.05**	**1.22**	**1.77**	**0.62**	**3.83**	**6.03**	**2.55**	**467,705**	
Totals:	**114.42**	**273.79**	**26.93**	**0.16**	**0.39**	**0.04**	**1.24**	**1.79**	**0.55**	**3.94**	**6.03**	**2.33**	**5,765,882**	

Site 1



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.287	4.039	122.022	**RainFall**	Inches
Maximum	1.650	5.620	242.369		
Minimum	0.890	2.540	35.906		

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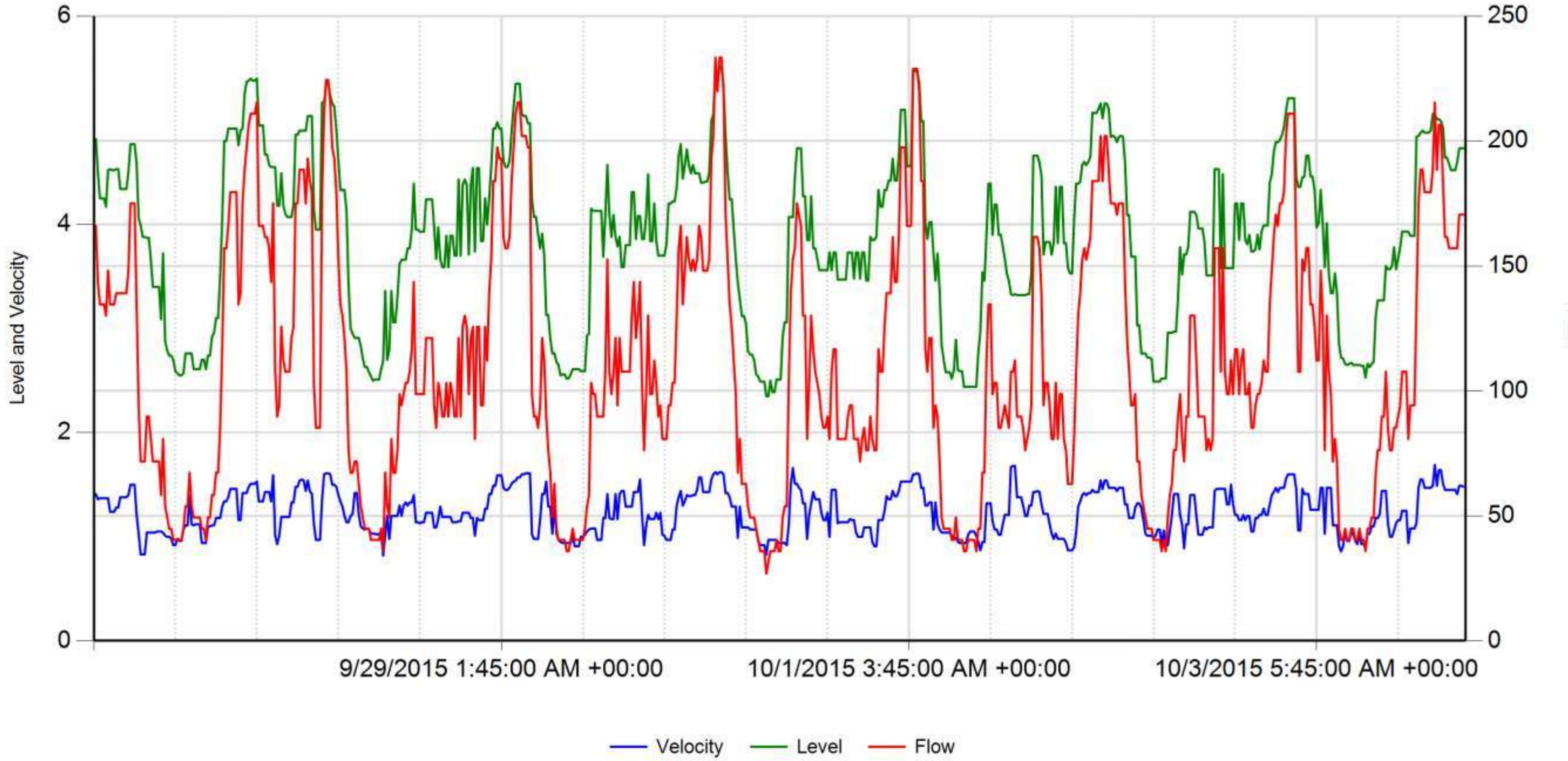
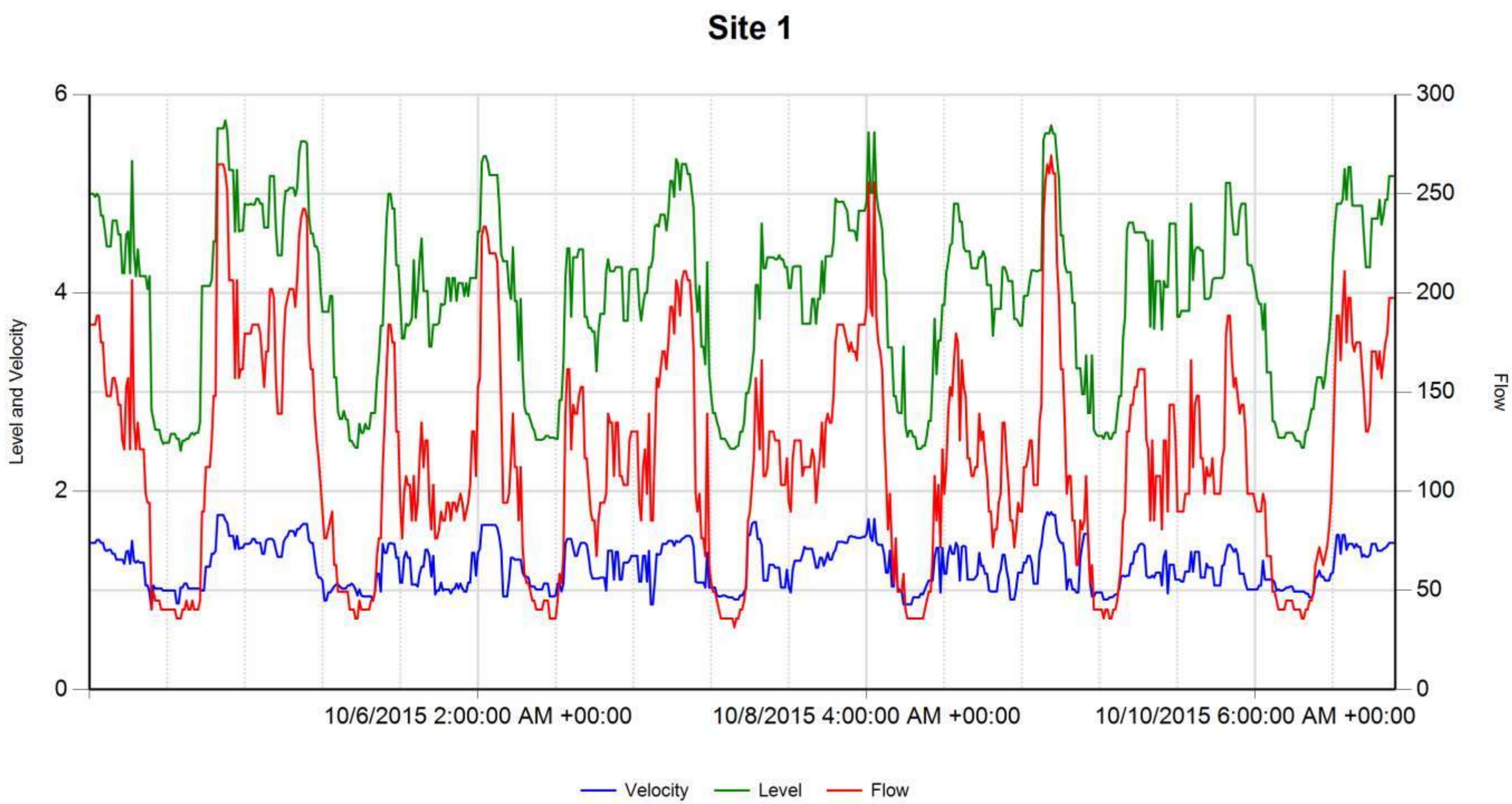
Site 1



	Velocity (fps)	Level (in)	Flow (gpm)			
Average	1.237	3.889	110.431	**RainFall**	Inches	
Maximum	1.690	5.510	233.392			
Minimum	0.820	2.350	26.930			11/2/2015 1:29:42 PM



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.243	3.979	116.061	**RainFall**	Inches
Maximum	1.790	5.740	269.299		
Minimum	0.810	2.410	31.418		

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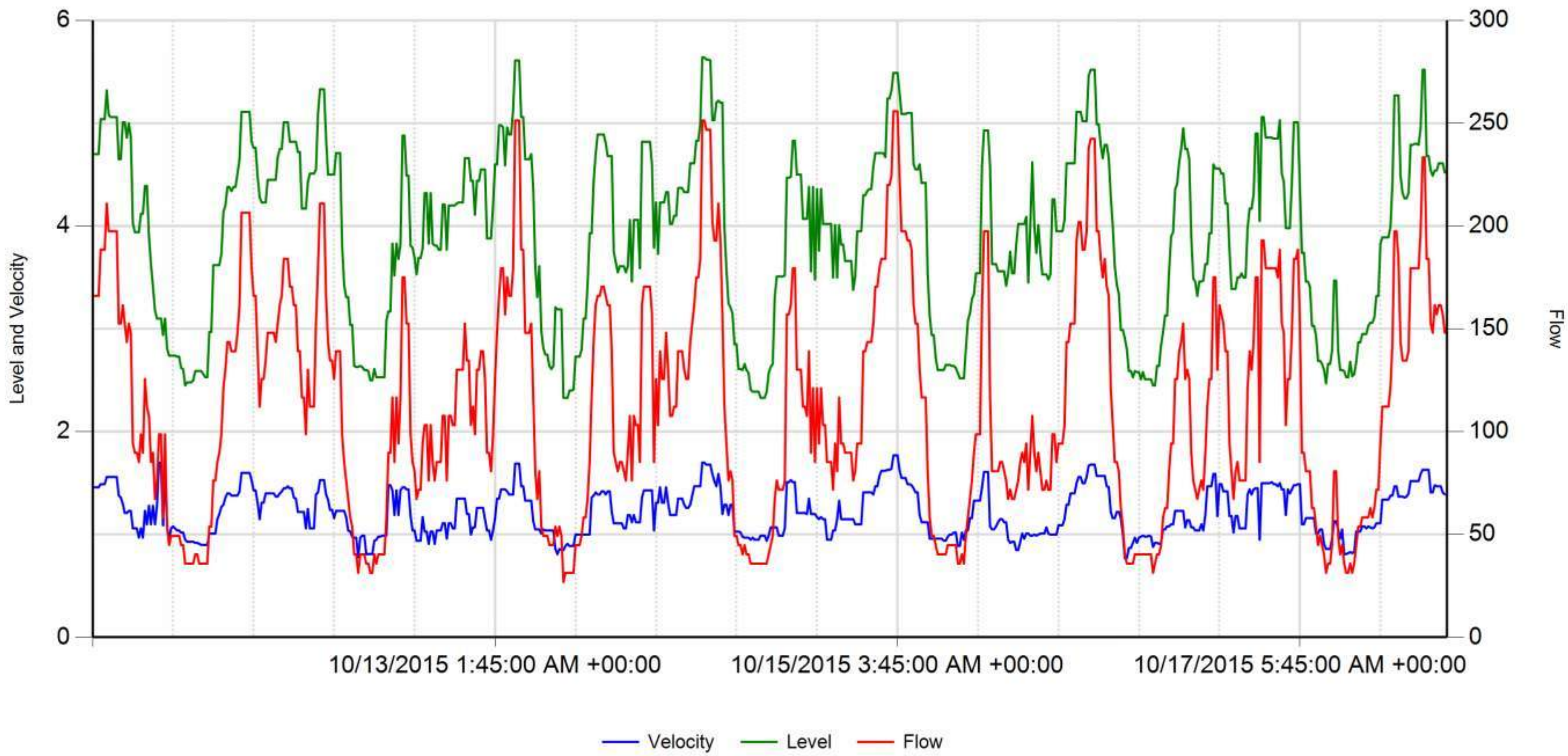
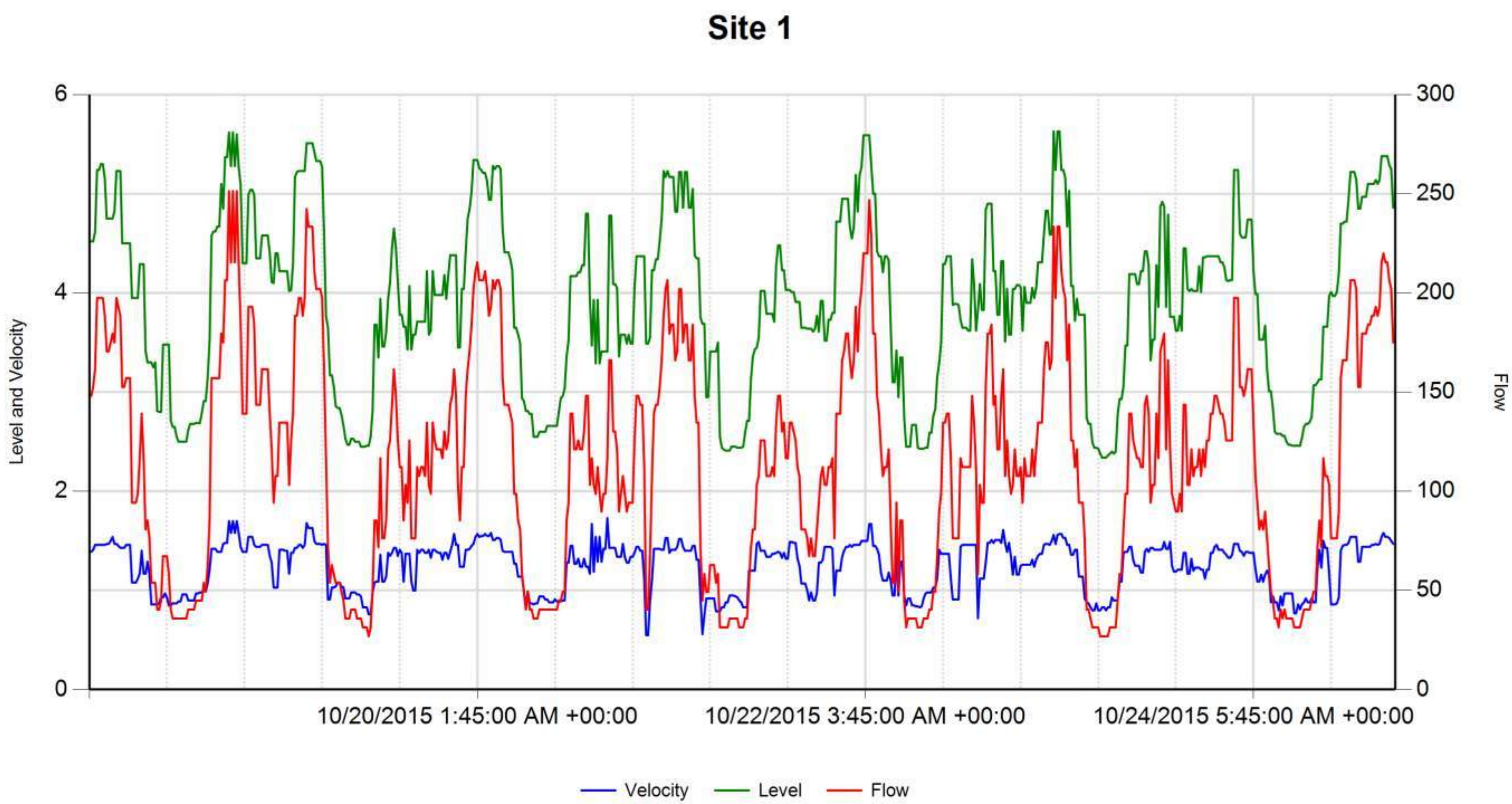
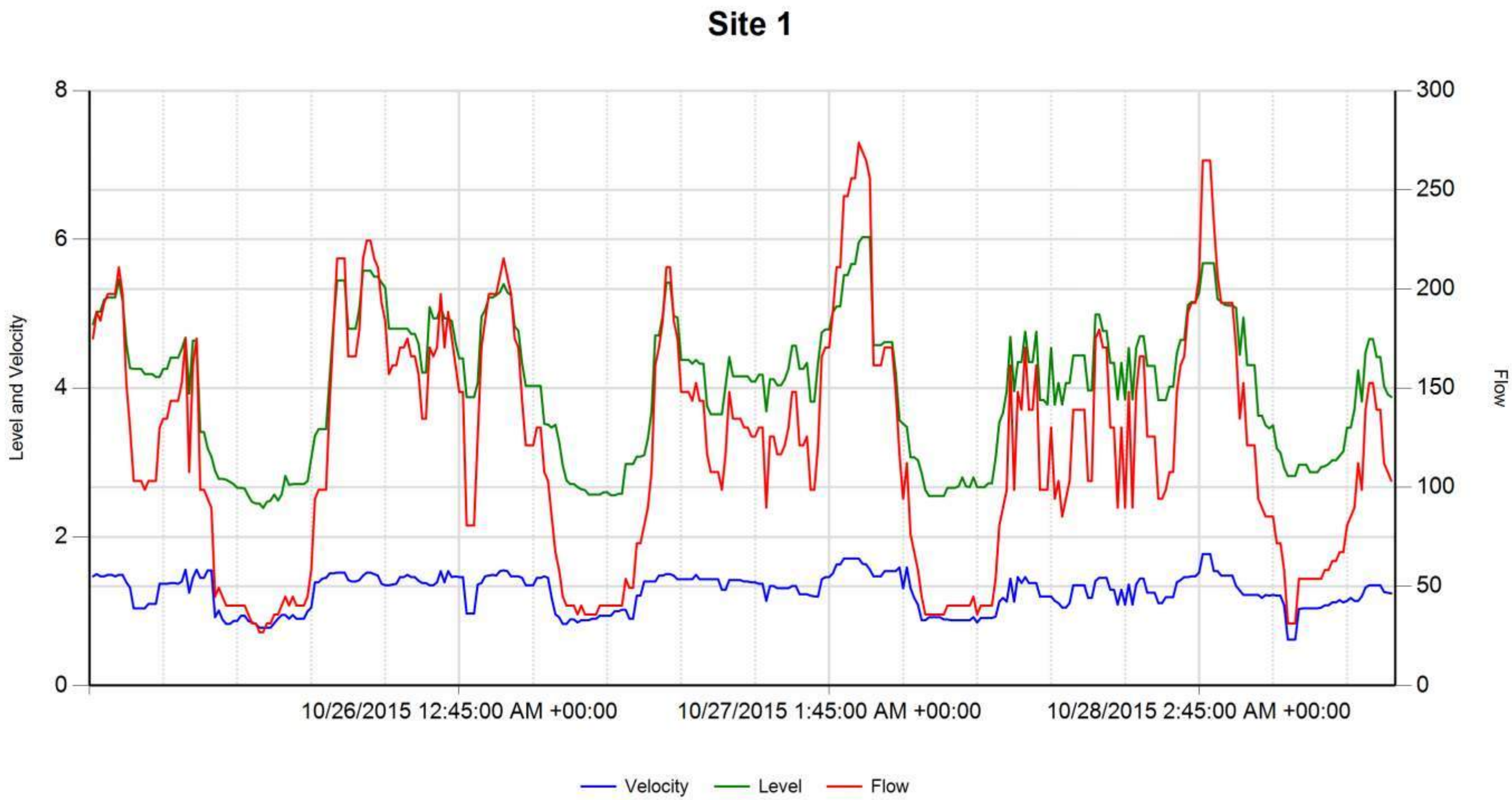
Site 1



	Velocity (fps)	Level (in)	Flow (gpm)			
Average	1.218	3.931	112.007	**RainFall**	Inches	
Maximum	1.770	5.640	255.834			
Minimum	0.770	2.330	26.930			11/2/2015 1:29:42 PM



	Velocity (fps)	Level (in)	Flow (gpm)			
Average	1.244	3.928	114.759	**RainFall**	Inches	
Maximum	1.730	5.630	251.345			
Minimum	0.550	2.340	26.930			11/2/2015 1:29:42 PM



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.261	4.021	120.802	**RainFall**	Inches
Maximum	1.770	6.030	273.787		
Minimum	0.620	2.390	26.930		

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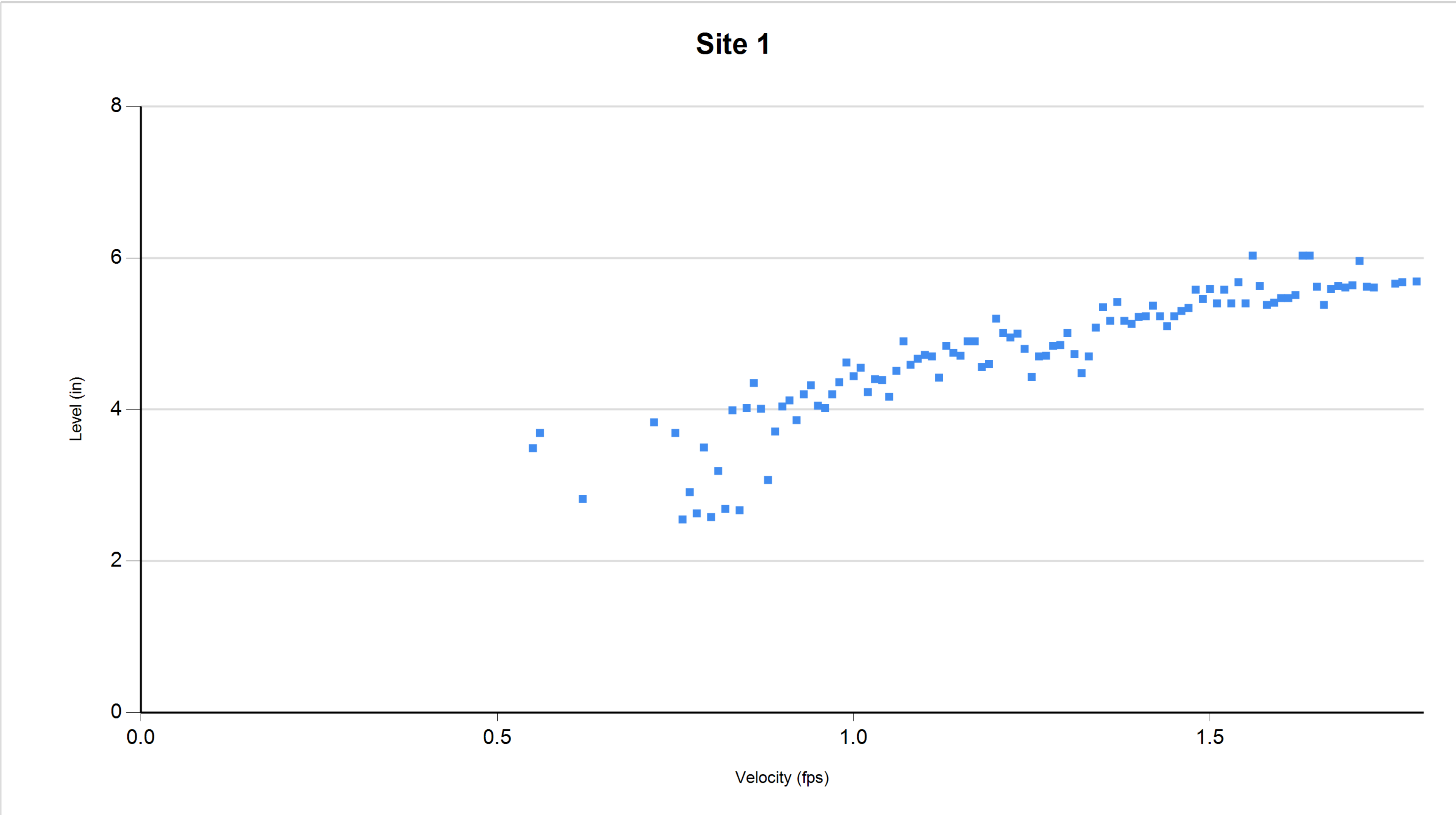


9/24/2015 thru 10/28/2015

11/2/2015 1:29:42 PM

Site Report 11-02-2015

Confidential Proprietary Information

Wallace Group

580 Apple Av
Greenfield,CA 93927

Site 2

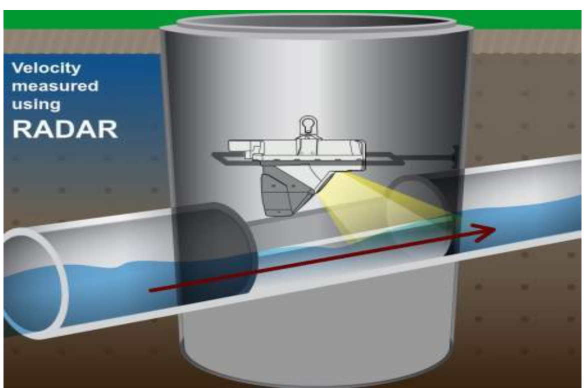
Access:
Manhole within intersection of Apple Av & Calaveras Way

System Type:
Sanitary [X] Storm [ ]

Install Date: 9/24/2015

Map

Technology



Traffic Plan

Flow Meter

Meter Depth: 82"

Meter SN:*

Slow & steady hydraulics

Avg Velocity	Avg Measured Level	Multiplier
2.25 fps	6.50"	1.0

Gas

O2	H2S	CO	LEL
20.9	0	0	0

Notes

*

Traffic Safety

Used cones, signs & vehicle.

Land Use

Residential	Commercial	Industrial	Trunk
X			

Manhole Depth	100"
Pipe Size	12"
Inner Pipe Size (In/Out)	12"/12"
Pipe Shape	Round
Pipe Condition	Fair
Manhole Material	Concrete
Silt (inches)	0
Velocity Profile Data	*
Velocity Profile Taken	
Sensor Offset	17.92"
Sensor Dist. to Crown	5.92"
Flow Direction	Downstream
Flow Heading	North

Meter Site Document

Wallace Group

Site 2

580 Apple Av
Greenfield,CA 93927

Site

Manhole Before Install

Installation Process

Installed

Upstream

Downstream

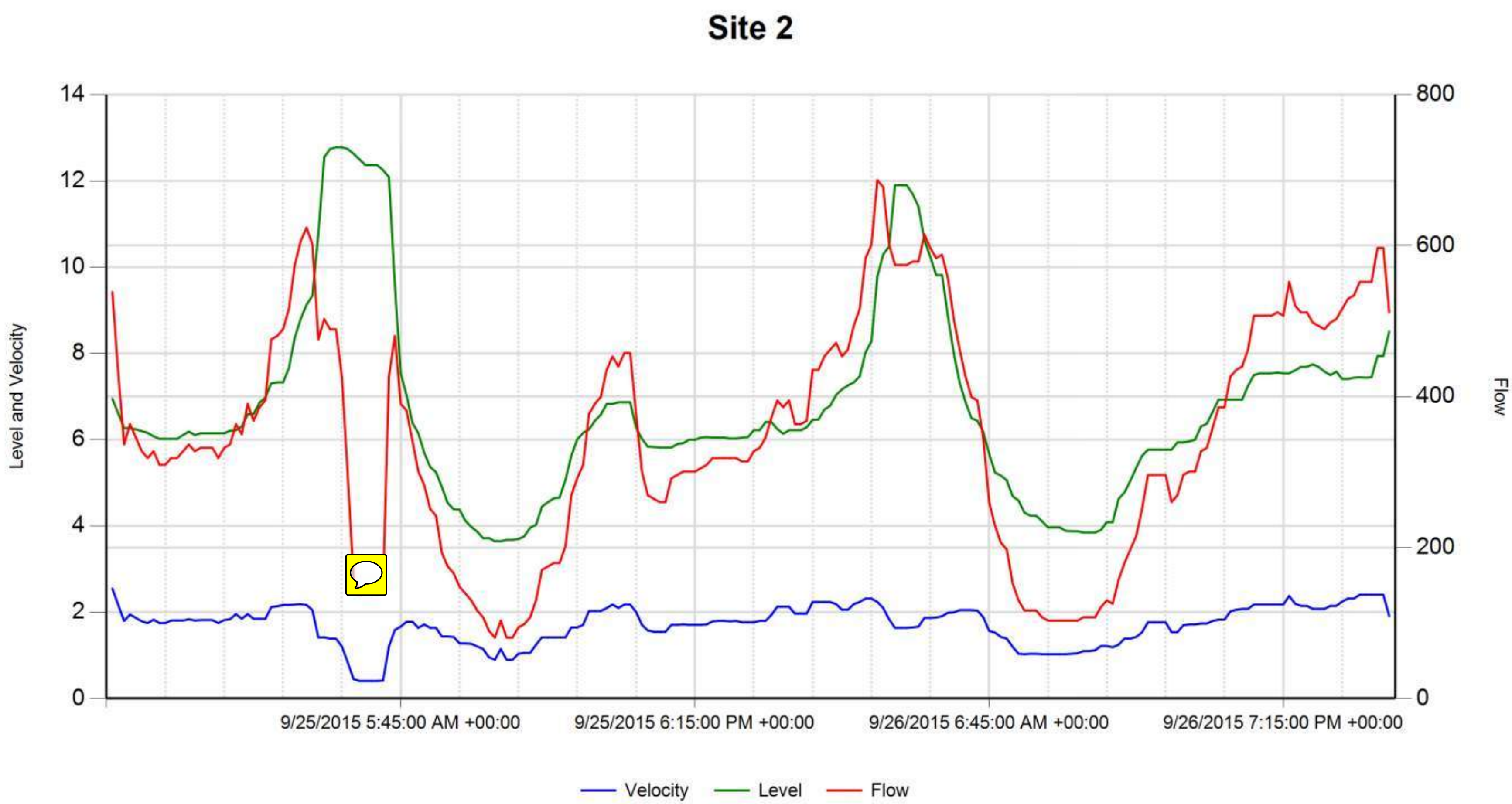
Utility Systems Science and Software

Report Date: 11/17/2015
Customer: US3
Group: WebFlow
SiteID: 2296

Statistics from Site 2 (Calaveras & Apple): 09/24/2015 thru 10/28/2015

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Rain
9/24/2015	359.76	623.88	143.63	0.52	0.90	0.21	1.64	2.55	0.41	7.90	12.78	4.53	518,056	
9/25/2015	346.91	686.71	80.79	0.50	0.99	0.12	1.74	2.32	0.90	6.46	11.90	3.65	499,549	
9/26/2015	384.92	664.27	103.23	0.55	0.96	0.15	1.77	2.41	1.03	6.98	12.57	3.85	554,284	
9/27/2015	377.06	632.85	121.18	0.54	0.91	0.17	1.56	2.18	0.55	8.13	12.86	3.77	542,974	
Week:	**367.16**	**686.71**	**80.79**	**0.53**	**0.99**	**0.12**	**1.68**	**2.55**	**0.41**	**7.37**	**12.86**	**3.65**	**2,114,863**	
9/28/2015	312.41	570.02	94.25	0.45	0.82	0.14	1.64	2.32	0.42	6.78	12.82	3.64	449,863	
9/29/2015	317.88	543.09	98.74	0.46	0.78	0.14	1.65	2.13	0.56	6.74	12.60	3.37	457,740	
9/30/2015	291.65	574.50	112.21	0.42	0.83	0.16	1.52	2.11	0.66	6.77	12.46	3.95	419,971	
10/1/2015	301.65	525.13	85.28	0.43	0.76	0.12	1.51	1.96	0.63	6.88	12.57	3.70	434,379	
10/2/2015	333.82	668.76	98.74	0.48	0.96	0.14	1.66	2.30	0.77	6.69	12.95	3.94	480,698	
10/3/2015	335.50	561.04	121.18	0.48	0.81	0.17	1.53	2.07	0.71	7.63	12.96	3.86	483,122	
10/4/2015	315.21	543.09	85.28	0.45	0.78	0.12	1.28	2.02	0.44	8.28	12.66	3.87	453,903	
Week:	**315.44**	**668.76**	**85.28**	**0.45**	**0.96**	**0.12**	**1.54**	**2.32**	**0.42**	**7.11**	**12.96**	**3.37**	**3,179,677**	
10/5/2015	247.89	547.57	76.30	0.36	0.79	0.11	1.28	2.20	0.48	6.80	12.89	3.65	356,955	
10/6/2015	228.16	502.69	80.79	0.33	0.72	0.12	1.21	1.69	0.41	6.73	12.62	3.63	328,544	
10/7/2015	254.66	507.18	67.32	0.37	0.73	0.10	1.28	1.73	0.72	6.78	12.87	3.66	366,718	
10/8/2015	235.68	457.81	62.84	0.34	0.66	0.09	1.22	1.68	0.57	6.67	12.35	3.48	339,384	
10/9/2015	267.76	484.74	67.32	0.39	0.70	0.10	1.34	1.85	0.73	6.53	12.38	3.71	385,568	
10/10/2015	305.91	507.18	71.81	0.44	0.73	0.10	1.35	1.79	0.78	7.41	12.68	3.62	440,505	
10/11/2015	299.13	493.71	85.28	0.43	0.71	0.12	1.30	1.90	0.44	7.65	12.82	3.87	430,743	
Week:	**262.74**	**547.57**	**62.84**	**0.38**	**0.79**	**0.09**	**1.28**	**2.20**	**0.41**	**6.94**	**12.89**	**3.48**	**2,648,418**	

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Rain
10/12/2015	288.47	668.76	80.79	0.42	0.96	0.12	1.44	2.07	0.48	6.89	12.66	3.77	415,393	
10/13/2015	271.45	543.09	103.23	0.39	0.78	0.15	1.40	1.99	0.54	6.77	12.37	3.56	390,887	
10/14/2015	264.02	498.20	103.23	0.38	0.72	0.15	1.37	1.99	0.48	6.68	12.92	3.62	380,182	
10/15/2015	249.57	462.30	103.23	0.36	0.67	0.15	1.31	1.97	0.52	6.77	12.66	3.62	359,379	
10/16/2015	310.44	493.71	116.70	0.45	0.71	0.17	1.59	2.07	0.90	6.58	11.93	3.64	447,036	
10/17/2015	313.62	561.04	112.21	0.45	0.81	0.16	1.45	2.30	0.64	7.42	12.59	3.89	451,614	
10/18/2015	339.71	614.90	103.23	0.49	0.89	0.15	1.51	2.30	0.55	7.75	12.71	3.73	489,181	
Week:	**291.04**	**668.76**	**80.79**	**0.42**	**0.96**	**0.12**	**1.44**	**2.30**	**0.48**	**6.98**	**12.92**	**3.56**	**2,933,673**	
10/19/2015	305.63	587.97	103.23	0.44	0.85	0.15	1.55	2.07	0.48	6.97	12.57	3.71	440,101	
10/20/2015	307.17	565.53	107.72	0.44	0.81	0.16	1.59	2.30	0.55	6.81	12.86	3.59	442,323	
10/21/2015	313.34	561.04	103.23	0.45	0.81	0.15	1.58	2.30	0.55	6.85	12.49	3.73	451,210	
10/22/2015	315.63	668.76	107.72	0.45	0.96	0.16	1.61	2.20	0.55	6.65	12.48	3.63	454,509	
10/23/2015	315.63	587.97	98.74	0.45	0.85	0.14	1.65	2.07	0.91	6.40	12.19	3.37	454,509	
10/24/2015	359.95	614.90	125.67	0.52	0.89	0.18	1.66	2.30	1.11	7.04	11.45	4.06	518,333	
10/25/2015	346.68	614.90	107.72	0.50	0.89	0.16	1.42	2.20	0.48	8.29	12.76	3.84	499,212	
Week:	**323.43**	**668.76**	**98.74**	**0.47**	**0.96**	**0.14**	**1.58**	**2.30**	**0.48**	**7.00**	**12.86**	**3.37**	**3,260,197**	
10/26/2015	295.39	547.57	103.23	0.43	0.79	0.15	1.55	2.07	0.48	6.81	12.57	3.77	425,357	
10/27/2015	307.92	565.53	107.72	0.44	0.81	0.16	1.61	2.30	0.65	6.61	12.32	3.53	443,400	
10/28/2015	185.72	403.95	103.23	0.27	0.58	0.15	1.35	1.99	1.09	4.66	6.78	3.65	267,441	
Week:	**263.01**	**565.53**	**103.23**	**0.38**	**0.81**	**0.15**	**1.50**	**2.30**	**0.48**	**6.03**	**12.57**	**3.53**	**1,136,199**	
Totals:	**303.15**	**686.71**	**62.84**	**0.44**	**0.99**	**0.09**	**1.49**	**2.55**	**0.41**	**6.96**	**12.96**	**3.37**	**15,273,028**	



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.715	6.741	347.762	**RainFall**	Inches
Maximum	2.550	12.780	686.712		
Minimum	0.410	3.650	80.790		

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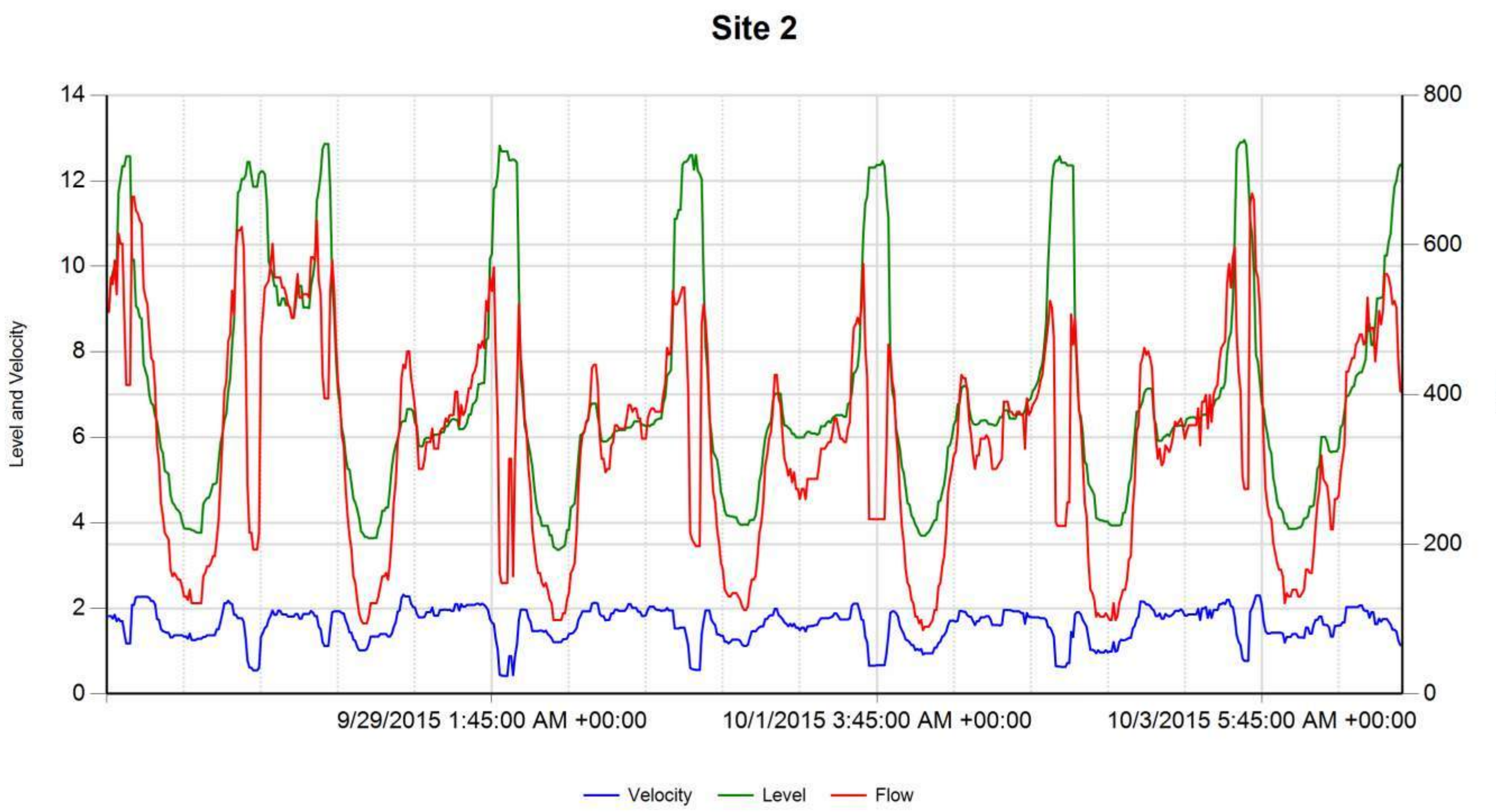

	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.606	7.058	330.746	**RainFall**	Inches
Maximum	2.320	12.950	668.758		
Minimum	0.420	3.370	85.278		

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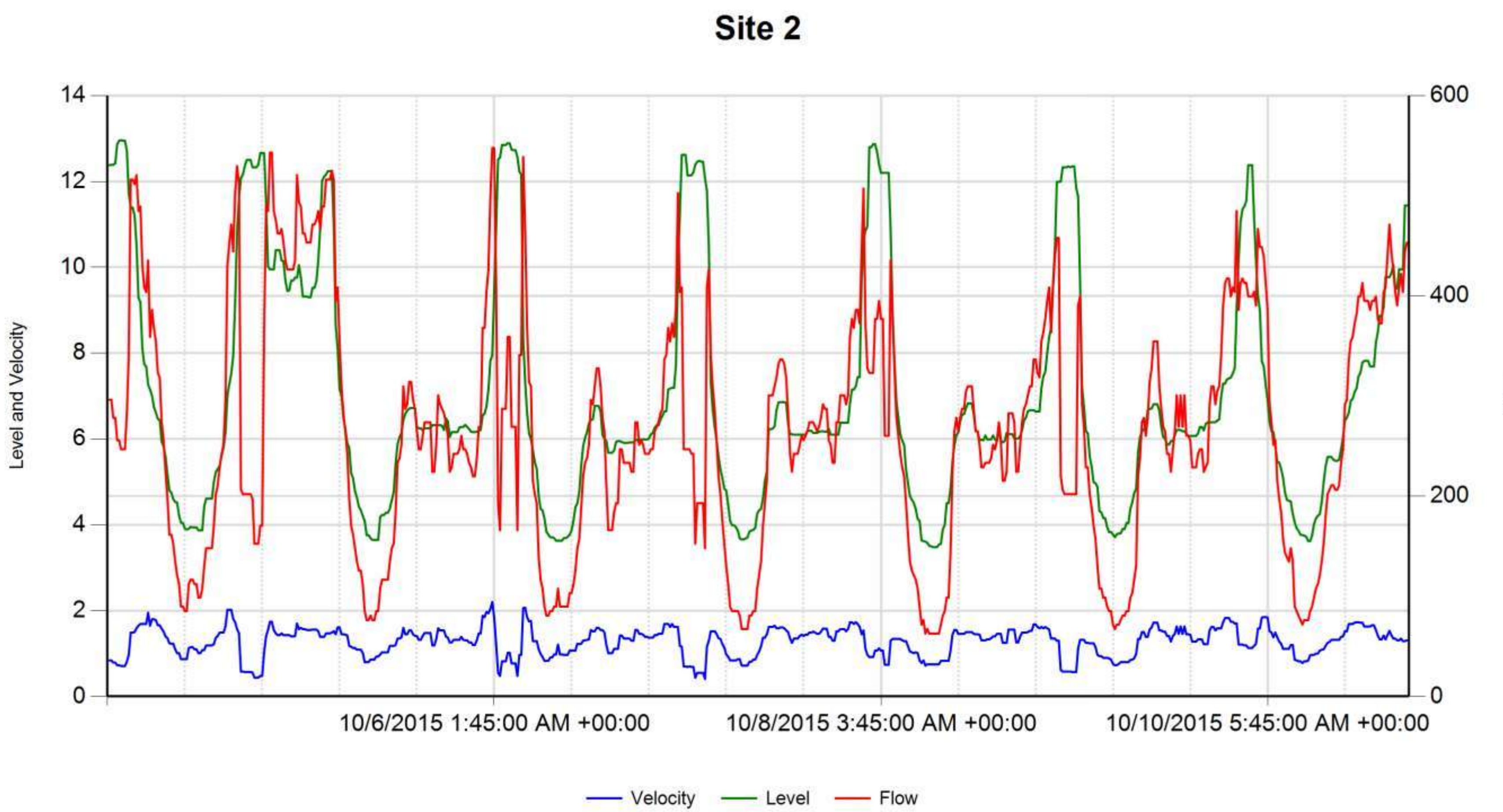

	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.274	7.047	263.481	**RainFall**	Inches
Maximum	2.200	12.960	547.574		
Minimum	0.410	3.480	62.836		

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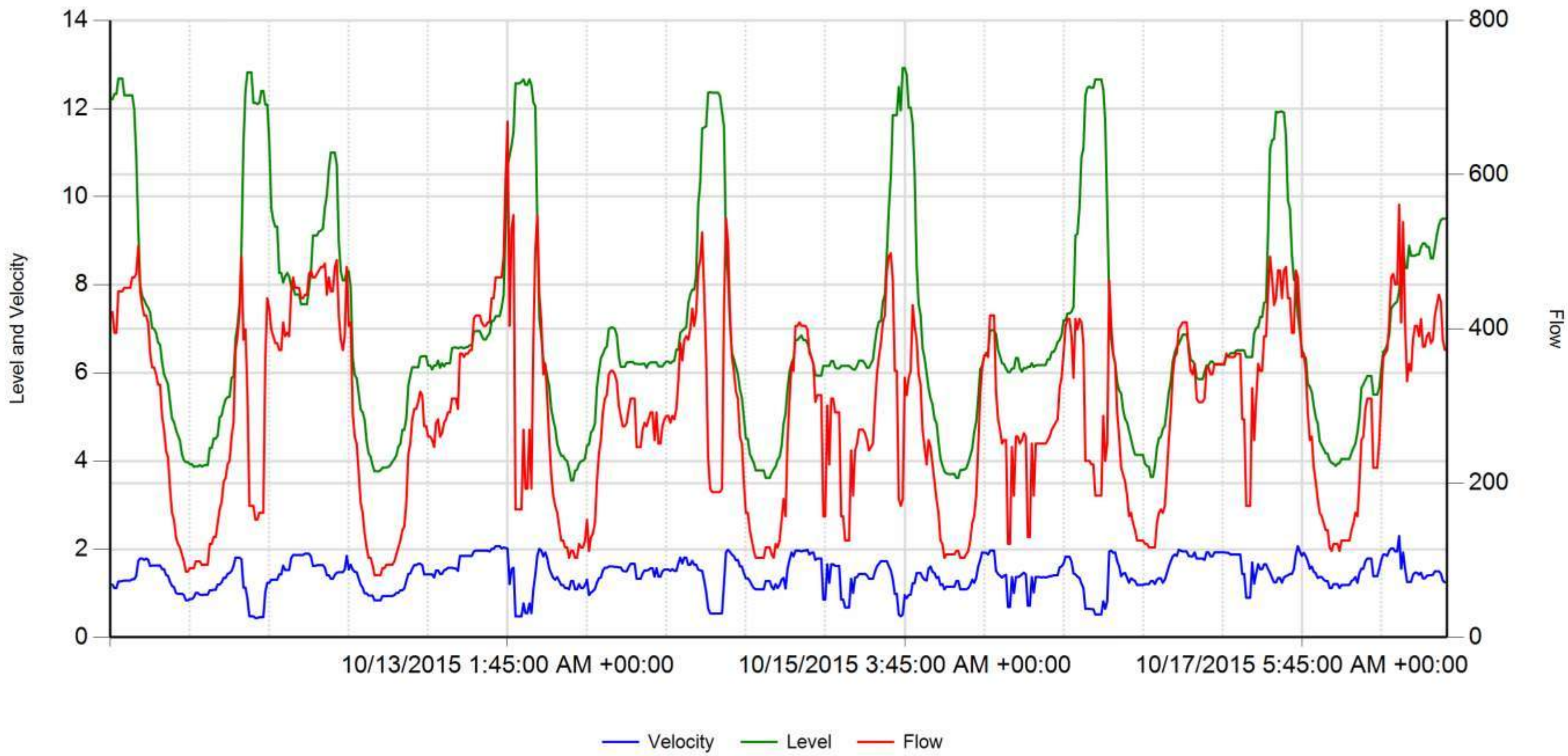
Site 2



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.412	6.958	286.531	**RainFall**	Inches
Maximum	2.300	12.920	668.758		
Minimum	0.440	3.560	80.790		

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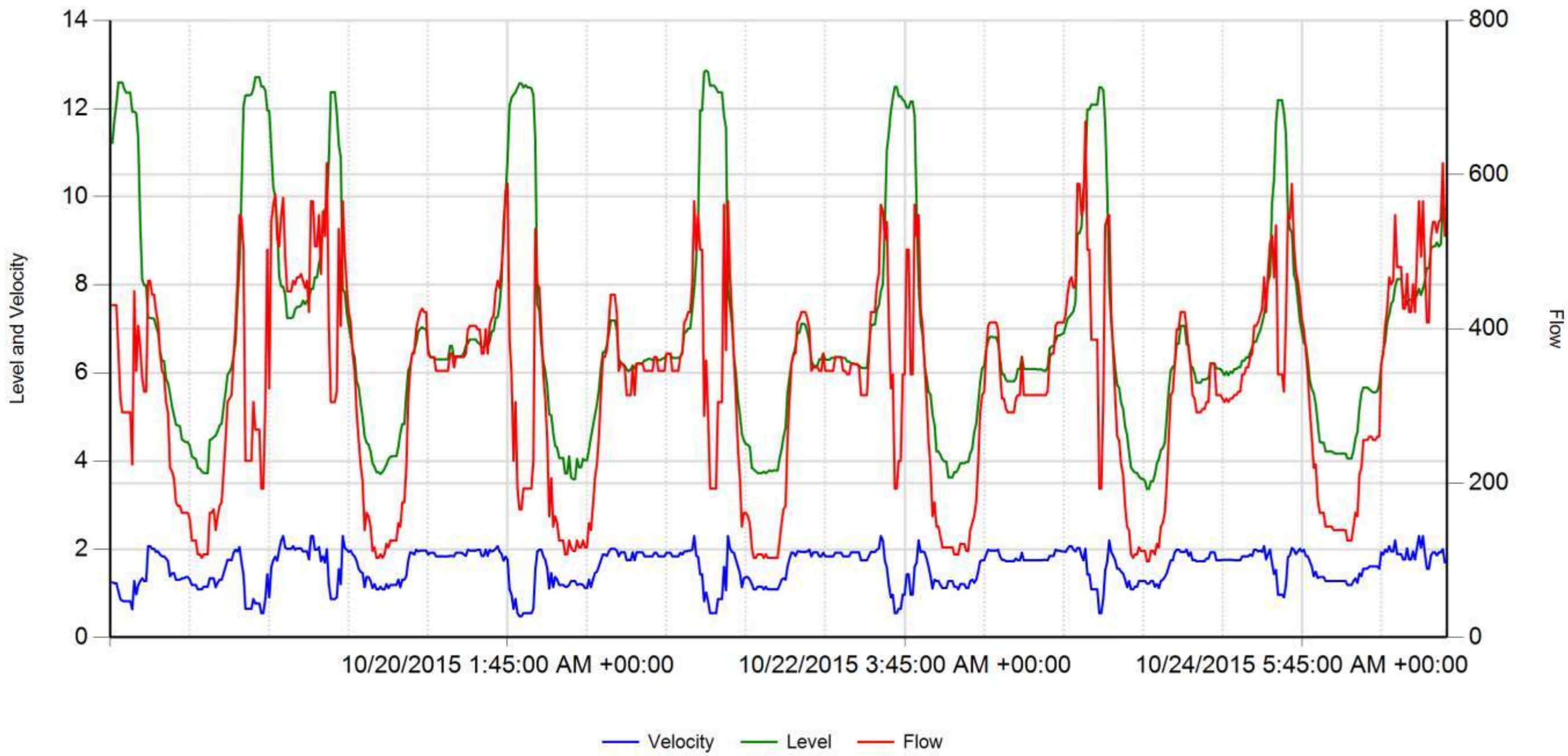
Site 2



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.580	6.964	319.371	**RainFall**	Inches
Maximum	2.300	12.860	668.758		
Minimum	0.480	3.370	98.743		

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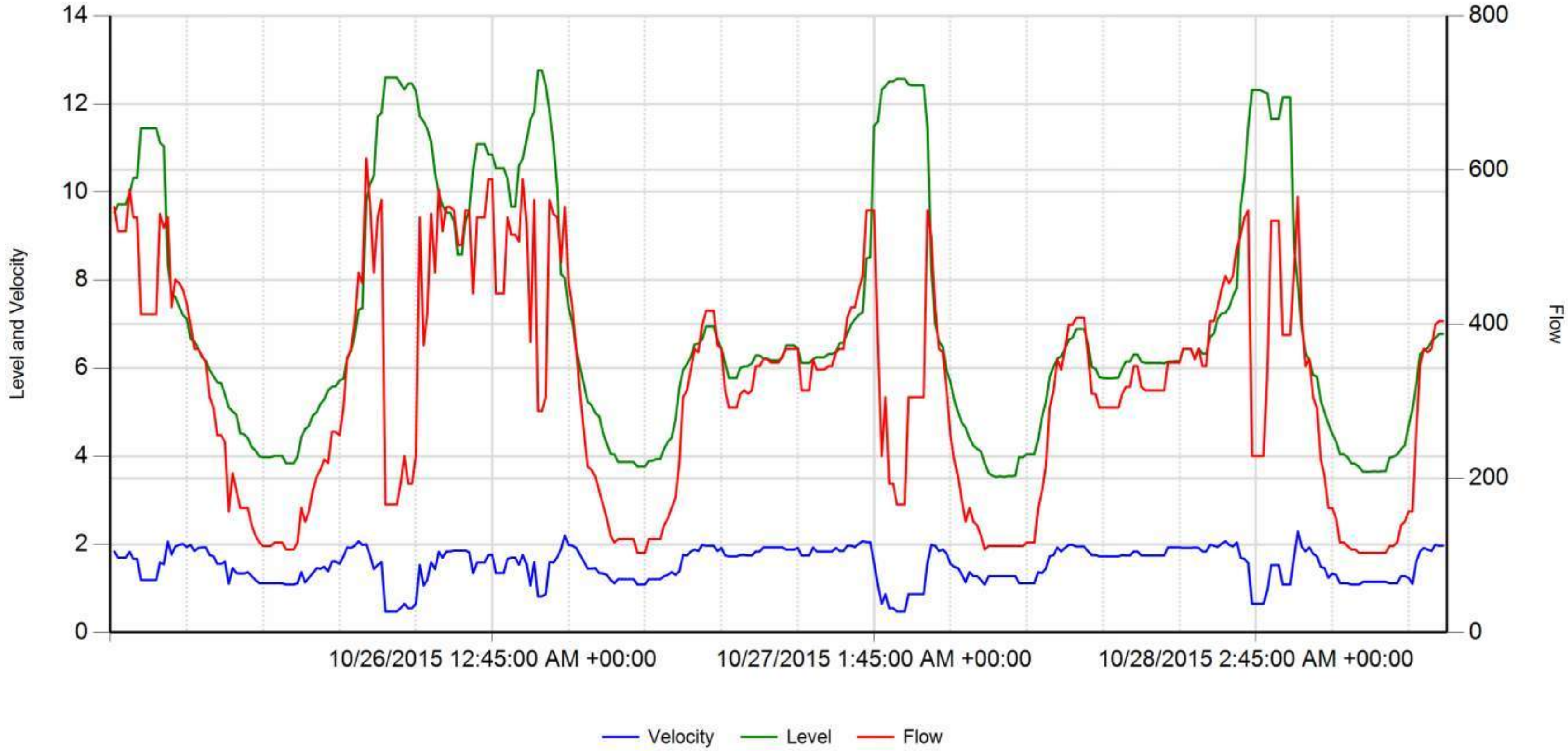
Site 2



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.524	7.119	314.580	**RainFall**	Inches
Maximum	2.300	12.760	614.899		
Minimum	0.480	3.530	103.231		

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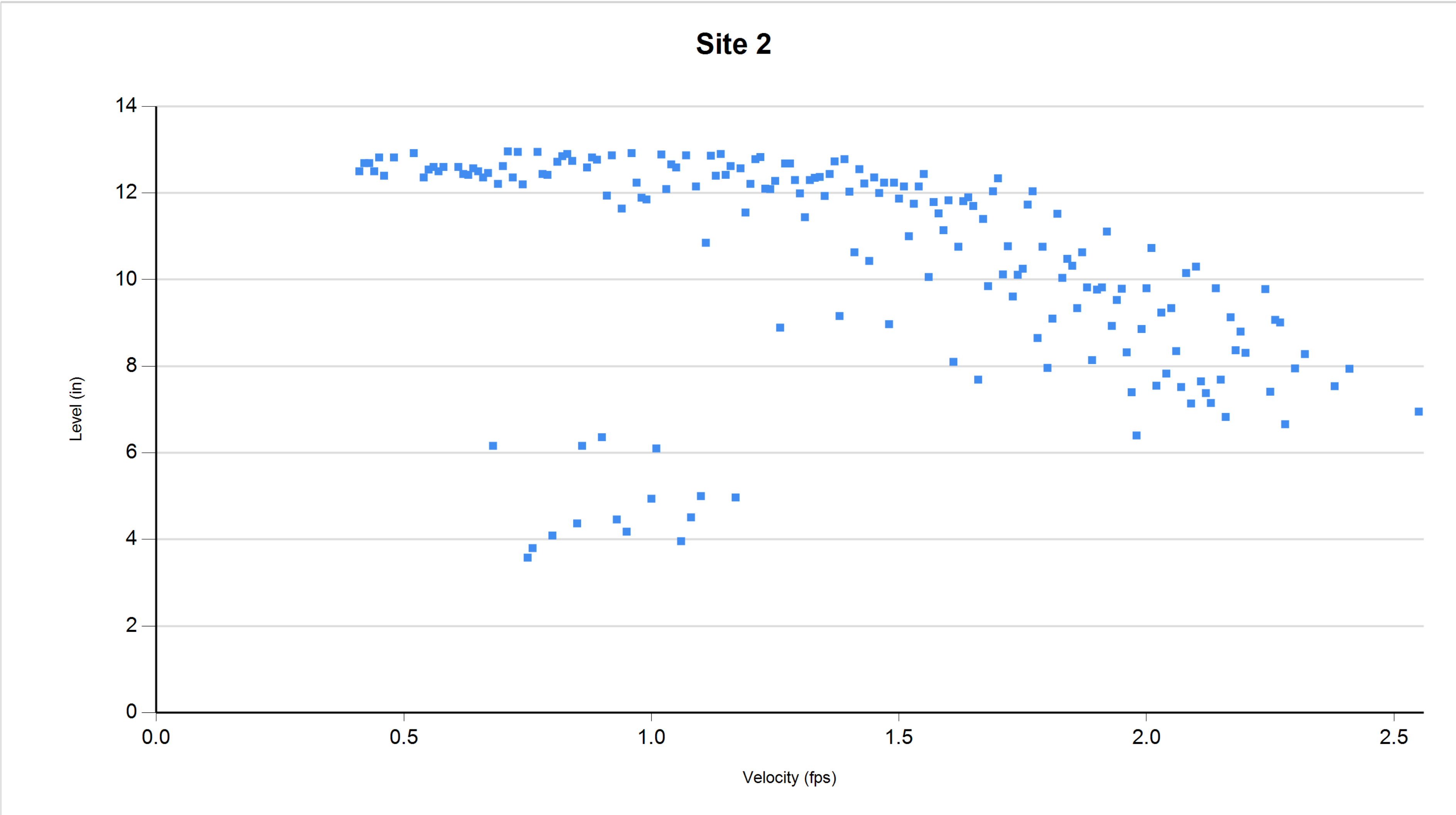

9/24/2015 thru 10/28/2015

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Site Report 11-02-2015

Confidential Proprietary Information

Wallace Group

502 Palm Av
Greenfield,CA 93927

Site 3

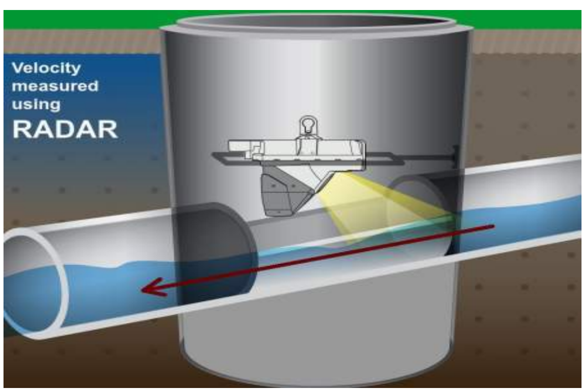
Access:
Manhole within intersection of Apple Av & Palm Av

System Type:
Sanitary [X] Storm [ ]

Install Date: 9/24/2015

Map

Technology



Traffic Plan

Flow Meter

Meter Depth: 91"

Meter SN:*

Slow & steady hydraulics

Avg Velocity	Avg Measured Level	Multiplier
1.36 fps	4.77"	1.0

Gas

O2	H2S	CO	LEL
20.9	3	0	0

Notes

Minor H2S production.

Traffic Safety

Used cones & vehicle.

Land Use

Residential	Commercial	Industrial	Trunk
X			

Manhole Depth	109"
Pipe Size	12"
Inner Pipe Size (In/Out)	12"/12"
Pipe Shape	Round
Pipe Condition	Fair
Manhole Material	Concrete
Silt (inches)	0.5"
Velocity Profile Data	*
Velocity Profile Taken	
Sensor Offset	17.92"
Sensor Dist. to Crown	5.92"
Flow Direction	Upstream
Flow Heading	West

Meter Site Document

Wallace Group

Site 3

502 Palm Av
Greenfield,CA 93927

Site

Manhole Before Install

Installation Process

Installed

Upstream

Downstream

Utility Systems Science and Software

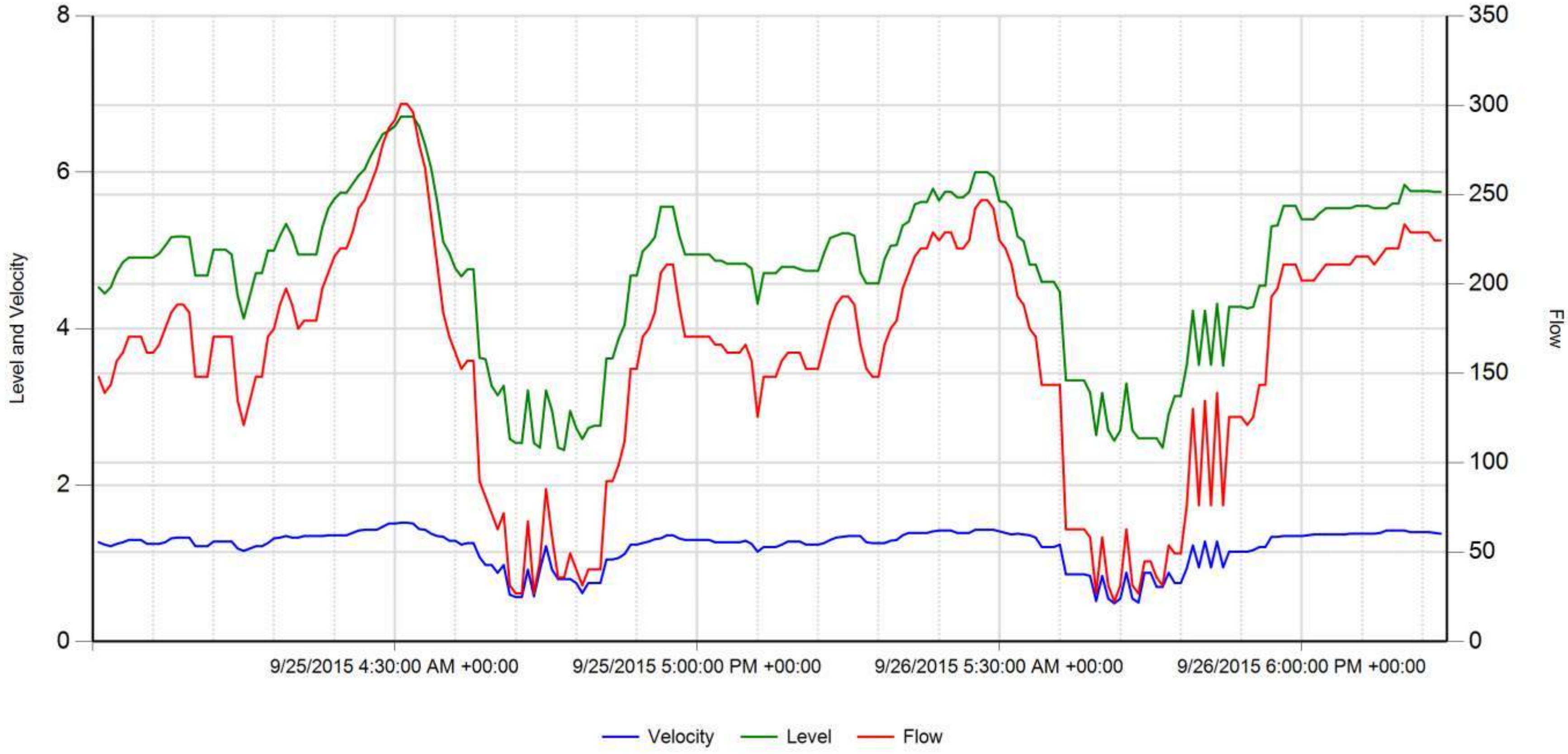
Report Date: 11/02/2015
Customer: Wallace Group
Group: WebFlow
SiteID: 2297

Statistics from Site 3 (Palm & Apple): 09/24/2015 thru 10/28/2015

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Rain
9/24/2015	192.78	300.72	121.18	0.28	0.43	0.17	1.32	1.52	1.16	5.27	6.71	4.13	277,608	
9/25/2015	150.36	246.86	26.93	0.22	0.36	0.04	1.20	1.43	0.57	4.57	6.00	2.45	216,516	
9/26/2015	156.39	237.88	22.44	0.23	0.34	0.03	1.18	1.42	0.49	4.70	6.04	2.48	225,201	
9/27/2015	167.94	273.79	22.44	0.24	0.39	0.03	1.20	1.44	0.56	4.84	6.53	2.27	241,830	
Week:	**166.87**	**300.72**	**22.44**	**0.24**	**0.43**	**0.03**	**1.22**	**1.52**	**0.49**	**4.85**	**6.71**	**2.27**	**961,156**	
9/28/2015	154.85	291.74	17.95	0.22	0.42	0.03	1.16	1.50	0.45	4.72	6.61	2.36	222,979	
9/29/2015	153.63	296.23	22.44	0.22	0.43	0.03	1.16	1.46	0.46	4.69	6.88	2.32	221,229	
9/30/2015	142.97	278.28	22.44	0.21	0.40	0.03	1.11	1.44	0.42	4.63	6.63	2.48	205,879	
10/1/2015	152.14	282.76	22.44	0.22	0.41	0.03	1.15	1.49	0.48	4.73	6.48	2.44	219,074	
10/2/2015	141.66	219.93	22.44	0.20	0.32	0.03	1.12	1.33	0.51	4.56	5.89	2.33	203,994	
10/3/2015	156.34	246.86	17.95	0.23	0.36	0.03	1.15	1.43	0.43	4.70	6.12	2.27	225,134	
10/4/2015	164.34	273.79	26.93	0.24	0.39	0.04	1.15	1.45	0.53	4.86	6.49	2.41	236,646	
Week:	**152.28**	**296.23**	**17.95**	**0.22**	**0.43**	**0.03**	**1.14**	**1.50**	**0.42**	**4.70**	**6.88**	**2.27**	**1,534,935**	
10/5/2015	158.31	291.74	44.88	0.23	0.42	0.06	1.22	1.61	0.92	4.77	6.74	2.37	227,961	
10/6/2015	150.31	309.69	22.44	0.22	0.45	0.03	1.15	1.54	0.54	4.65	6.77	2.19	216,449	
10/7/2015	155.22	291.74	26.93	0.22	0.42	0.04	1.22	1.94	0.52	4.69	6.69	2.41	223,518	
10/8/2015	149.80	278.28	22.44	0.22	0.40	0.03	1.15	1.42	0.48	4.71	6.65	2.56	215,708	
10/9/2015	143.06	215.44	26.93	0.21	0.31	0.04	1.13	1.32	0.57	4.62	5.85	2.53	206,014	
10/10/2015	157.00	228.90	22.44	0.23	0.33	0.03	1.18	3.94	0.45	4.78	6.03	2.43	226,076	
10/11/2015	162.19	269.30	17.95	0.23	0.39	0.03	1.19	3.75	0.42	4.83	6.44	2.32	233,549	
Week:	**153.70**	**309.69**	**17.95**	**0.22**	**0.45**	**0.03**	**1.18**	**3.94**	**0.42**	**4.72**	**6.77**	**2.19**	**1,549,275**	

	Flow (GPM)			Flow (MGD)			Velocity (FPS)			Level (inches)				
Date	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Total Gal	Rain
10/12/2015	150.59	273.79	26.93	0.22	0.39	0.04	1.13	1.43	0.54	4.74	6.56	2.48	216,853	
10/13/2015	144.00	260.32	31.42	0.21	0.38	0.05	1.14	1.39	0.68	4.61	6.43	2.30	207,360	
10/14/2015	146.01	269.30	31.42	0.21	0.39	0.05	1.12	1.41	0.62	4.70	6.54	2.53	210,255	
10/15/2015	155.50	264.81	35.91	0.22	0.38	0.05	1.35	5.25	0.59	4.63	6.30	2.45	223,922	
10/16/2015	148.49	219.93	31.42	0.21	0.32	0.05	1.29	4.40	0.62	4.56	5.77	2.37	213,823	
10/17/2015	156.39	237.88	26.93	0.23	0.34	0.04	1.24	4.58	0.54	4.68	6.03	2.30	225,201	
10/18/2015	161.67	255.83	22.44	0.23	0.37	0.03	1.20	1.44	0.48	4.76	6.20	2.45	232,809	
Week:	**151.81**	**273.79**	**22.44**	**0.22**	**0.39**	**0.03**	**1.21**	**5.25**	**0.48**	**4.67**	**6.56**	**2.30**	**1,530,223**	
10/19/2015	153.91	287.25	22.44	0.22	0.41	0.03	1.17	1.49	0.45	4.66	6.57	2.35	221,633	
10/20/2015	152.60	278.28	22.44	0.22	0.40	0.03	1.18	1.49	0.52	4.59	6.45	2.37	219,748	
10/21/2015	152.28	269.30	17.95	0.22	0.39	0.03	1.14	1.48	0.42	4.67	6.48	2.30	219,276	
10/22/2015	153.12	273.79	26.93	0.22	0.39	0.04	1.19	1.47	0.52	4.66	6.41	2.36	220,488	
10/23/2015	151.20	260.32	26.93	0.22	0.38	0.04	1.29	5.72	0.68	4.55	5.92	2.31	217,728	
10/24/2015	154.99	233.39	22.44	0.22	0.34	0.03	1.17	1.37	0.47	4.69	6.00	2.42	223,181	
10/25/2015	146.34	269.30	26.93	0.21	0.39	0.04	1.16	1.52	0.52	4.54	7.12	2.47	210,726	
Week:	**152.06**	**287.25**	**17.95**	**0.22**	**0.41**	**0.03**	**1.19**	**5.72**	**0.42**	**4.62**	**7.12**	**2.30**	**1,532,781**	
10/26/2015	167.98	309.69	17.95	0.24	0.45	0.03	2.43	7.04	0.46	3.95	6.99	1.89	241,898	
10/27/2015	146.90	287.25	26.93	0.21	0.41	0.04	2.25	7.17	0.58	3.78	6.06	1.89	211,534	
10/28/2015	146.13	184.02	116.70	0.21	0.27	0.17	5.73	6.08	5.45	1.70	1.97	1.55	210,433	
Week:	**153.67**	**309.69**	**17.95**	**0.22**	**0.45**	**0.03**	**3.47**	**7.17**	**0.46**	**3.14**	**6.99**	**1.55**	**663,865**	
Totals:	**154.34**	**309.69**	**17.95**	**0.22**	**0.45**	**0.03**	**1.41**	**7.17**	**0.42**	**4.55**	**7.12**	**1.55**	**7,772,235**	

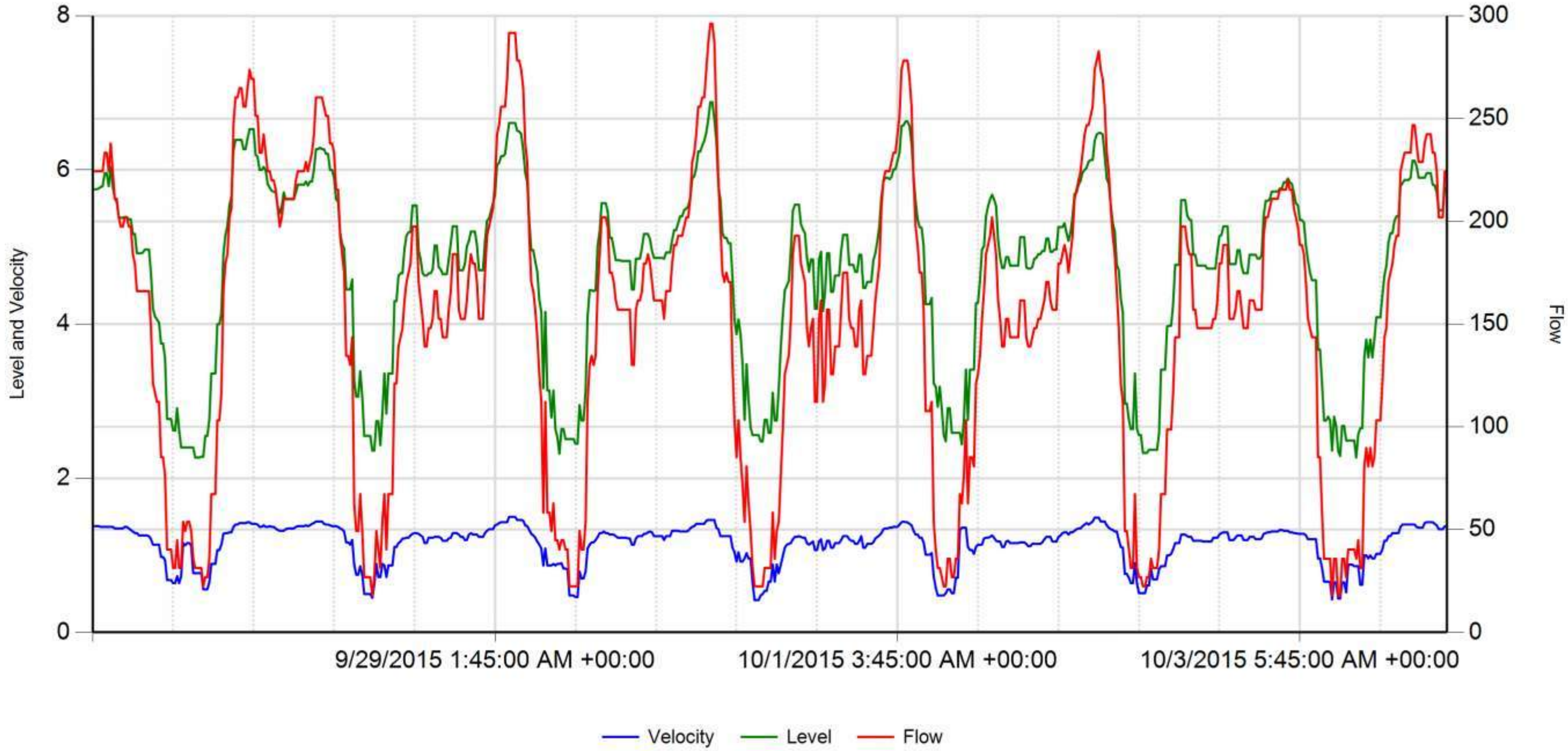
Site 3



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.210	4.721	159.083	**RainFall**	Inches
Maximum	1.520	6.710	300.717		
Minimum	0.490	2.450	22.442		

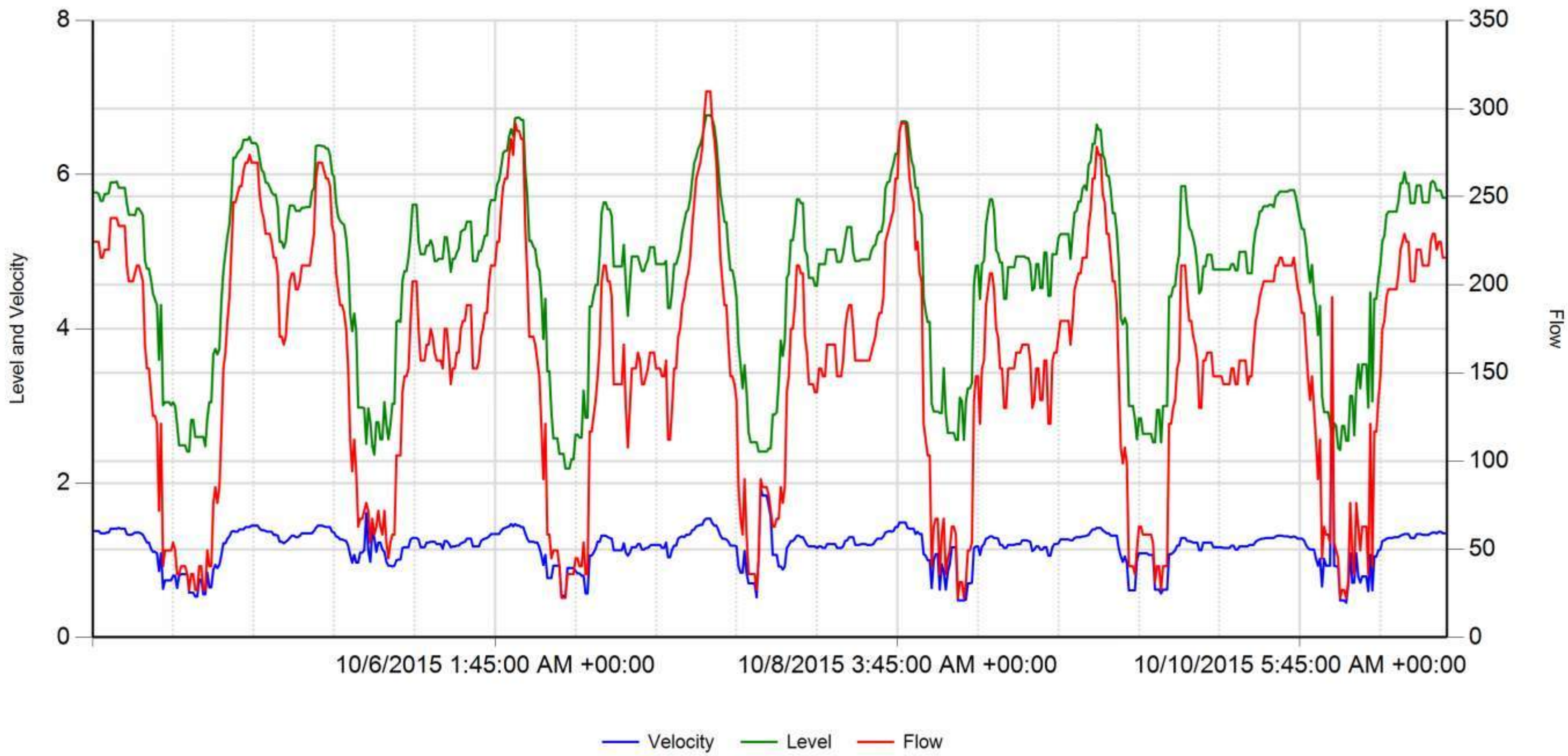
11/2/2015 1:31:23 PM

Site 3



	Velocity (fps)	Level (in)	Flow (gpm)			
Average	1.149	4.685	152.242	**RainFall**	Inches	
Maximum	1.500	6.880	296.229			
Minimum	0.420	2.270	17.953			11/2/2015 1:31:23 PM

Site 3



	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.172	4.731	154.560	**RainFall**	Inches
Maximum	3.940	6.770	309.694		
Minimum	0.450	2.190	22.442		

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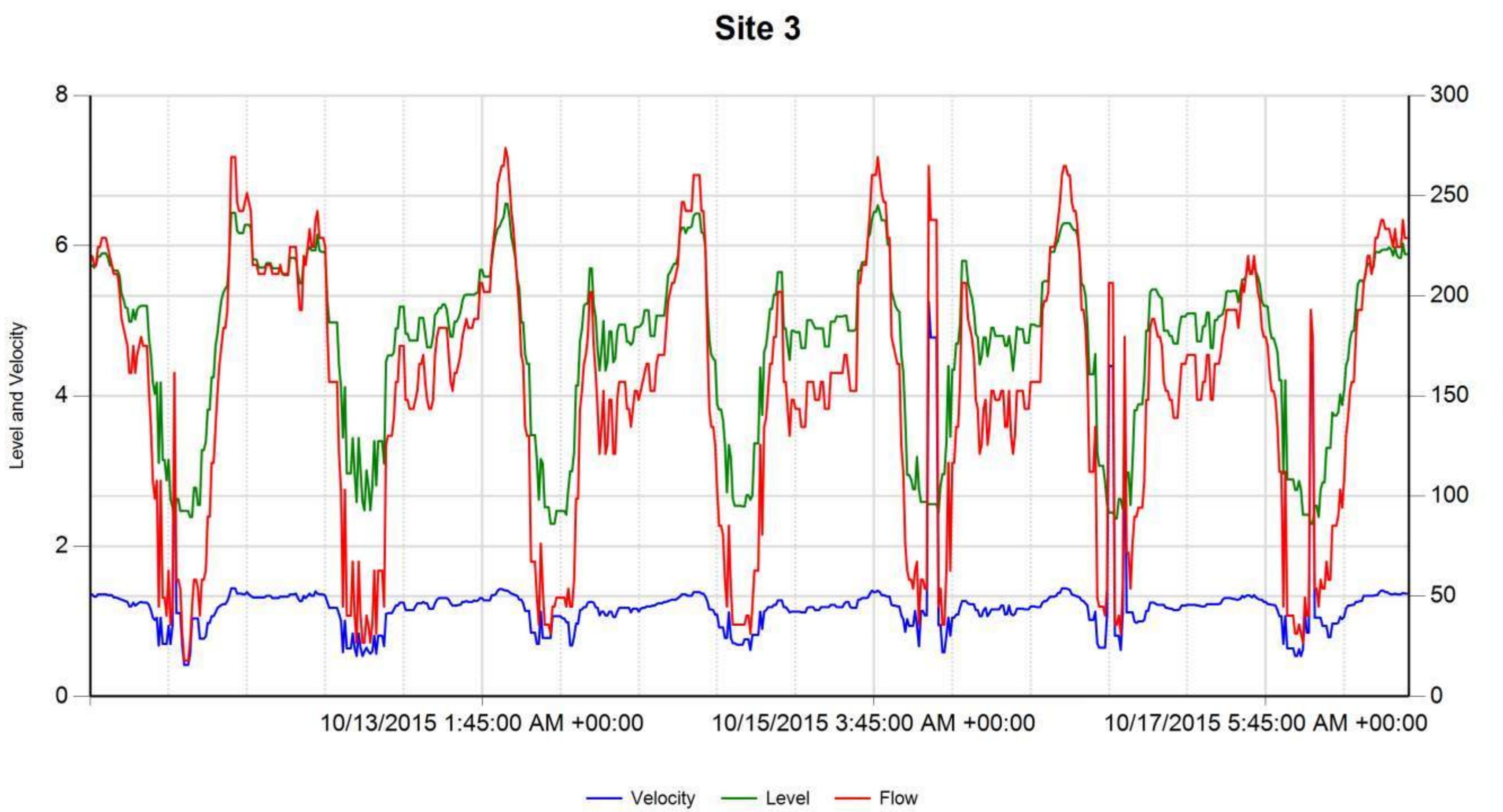

	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.206	4.679	151.781	**RainFall**	Inches
Maximum	5.250	6.560	273.787		
Minimum	0.420	2.300	17.953		

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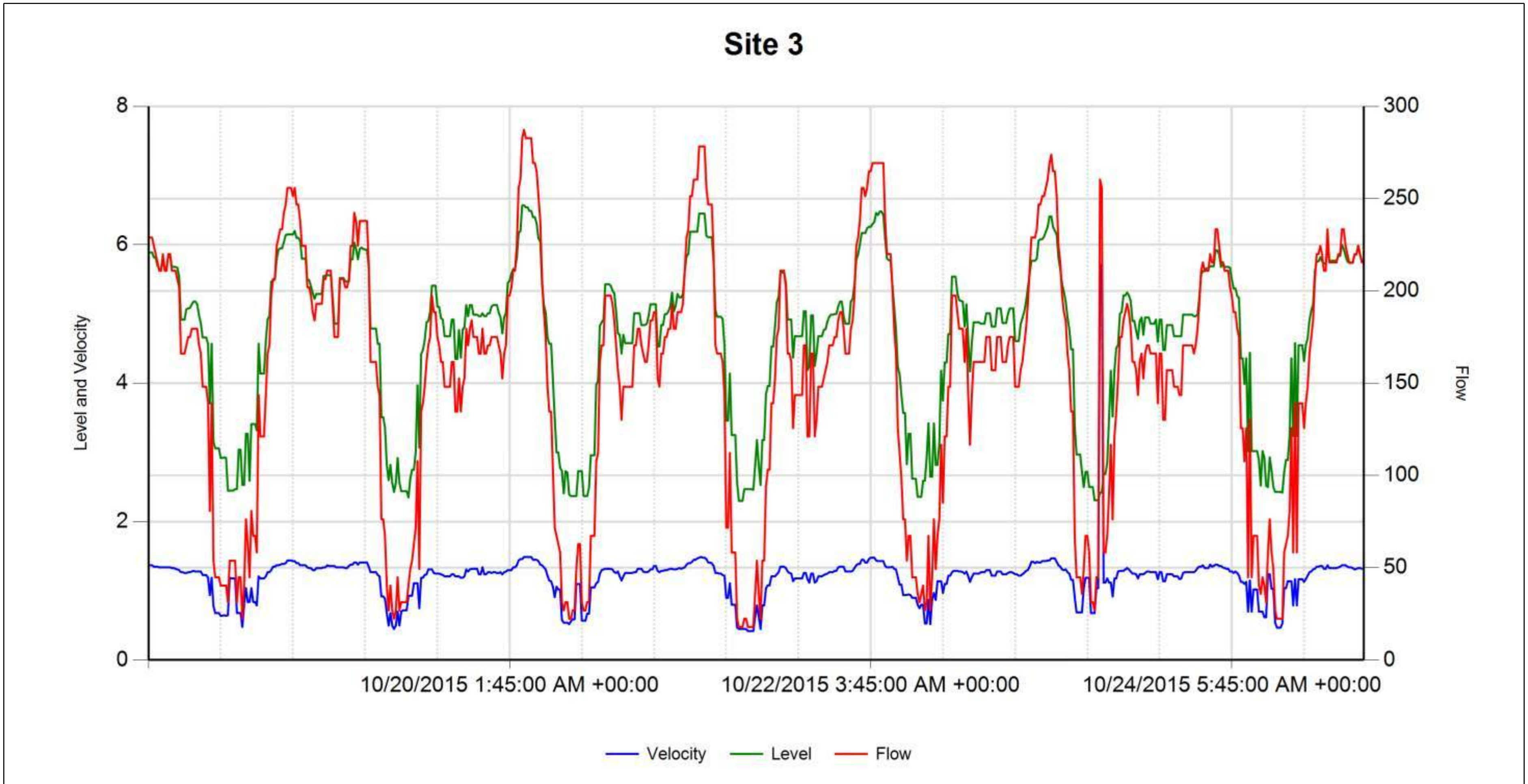
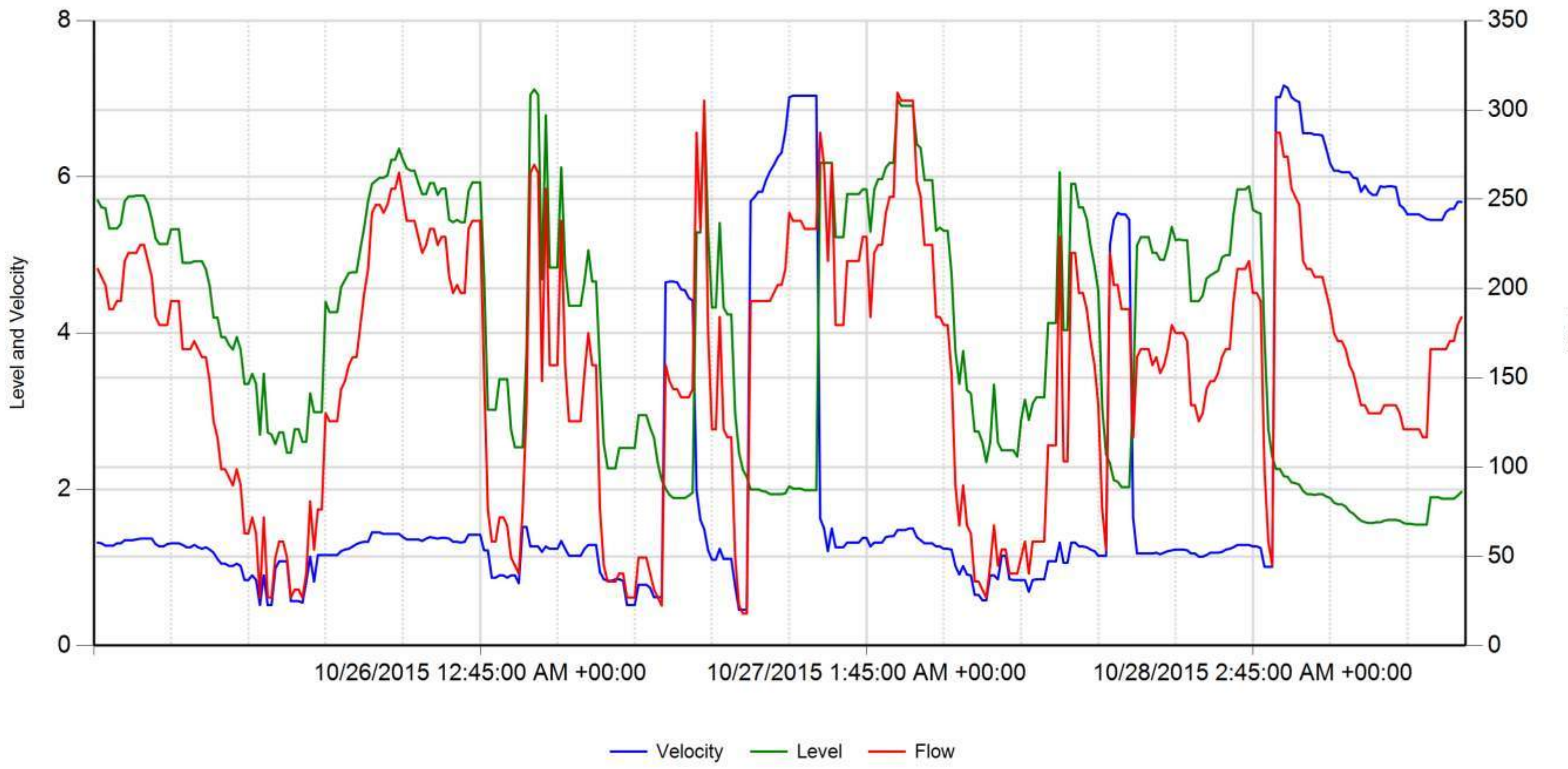


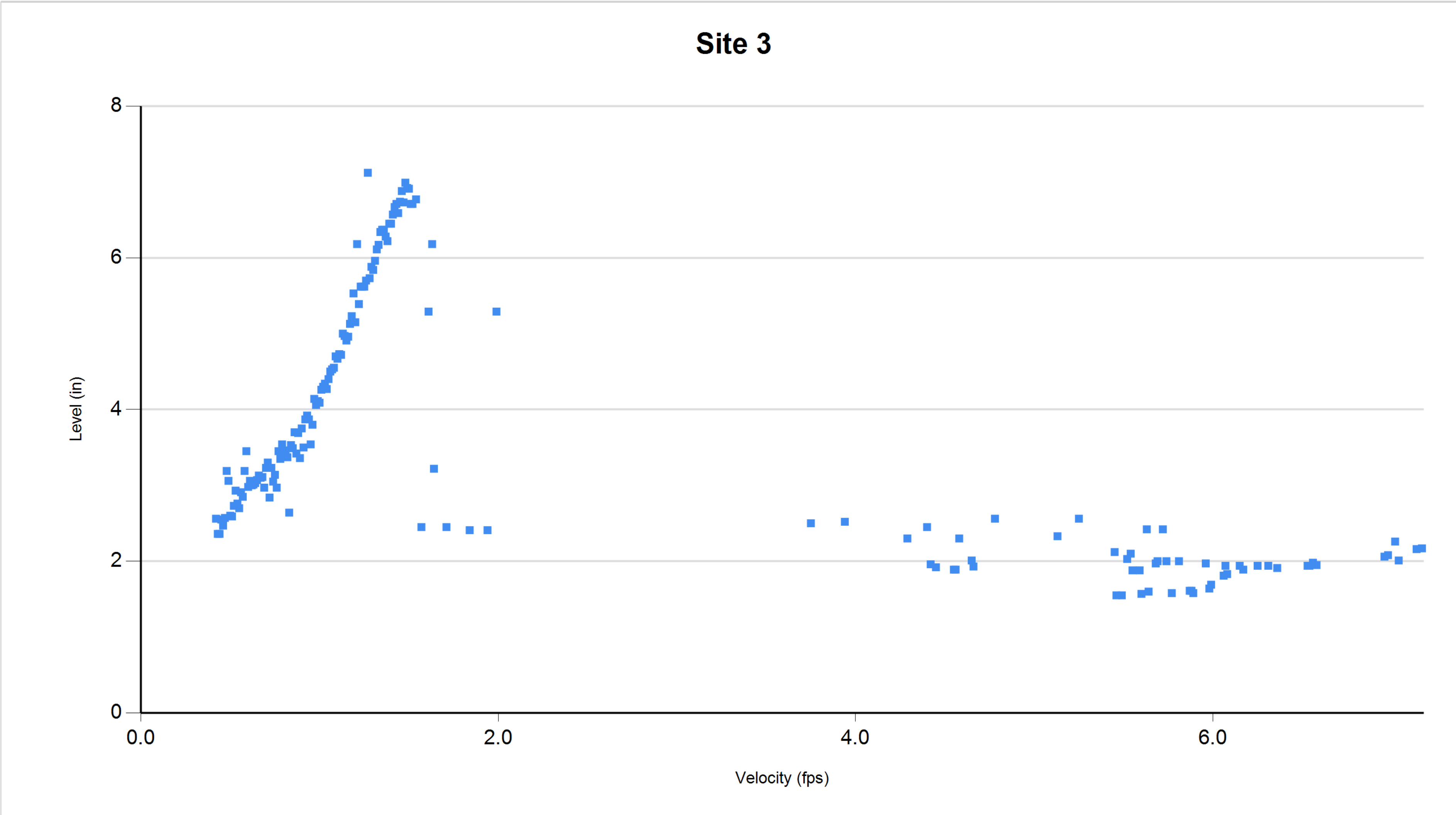
	Velocity (fps)	Level (in)	Flow (gpm)		
Average	1.193	4.660	154.553	**RainFall**	Inches
Maximum	5.720	6.570	287.252		
Minimum	0.420	2.300	17.953		

11/2/2015 1:31:23 PM

Site 3



	Velocity (fps)	Level (in)	Flow (gpm)			
Average	2.249	3.965	156.001	**RainFall**	Inches	
Maximum	7.170	7.120	309.694			
Minimum	0.460	1.550	17.953			11/2/2015 1:31:23 PM



9/24/2015 thru 10/28/2015

11/2/2015 1:31:23 PM

10: Appendix B - Expansion Report for City of Greenfield WWTP

CITY OF GREENFIELD

WASTEWATER TREATMENT PLANT EVALUATION

May 10, 2013

Prepared by:

Kennedy/Jenks Consultants
Engineers & Scientists

WALLACE GROUP®

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Kennedy/Jenks Consultants
Engineers & Scientists

WALLACE GROUP®

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LANDSCAPE ARCHITECTURE
MECHANICAL ENGINEERING
PLANNING
PUBLIC WORKS ADMINISTRATION
SURVEYING / GIS SOLUTIONS
WATER RESOURCES

MEMORANDUM

Date: May 10, 2013

To: Dale Lipp, City of Greenfield Public Works Director

From: Steven G. Tanaka, PE, Wallace Group
John Jenks, PE, Kennedy/Jenks Consultants

Subject: City of Greenfield Wastewater Treatment Plant Evaluation

INTRODUCTION AND SCOPE OF SERVICES

Wallace Group, in conjunction with Kennedy/Jenks Consultants, was retained by the City of Greenfield to provide a review of the City of Greenfield's wastewater treatment plant (WWTP), to provide City Staff with a technical memorandum describing the status and capabilities of the existing WWTP. The evaluation includes the following tasks:

1. Review of existing waste discharge requirements (WDRs), Order No. R3-2002-0062 (see Appendix A – Waste Discharge Requirements).
2. Collect and review pertinent wastewater data including influent BOD_5/TSS, effluent BOD_5/TSS, plant operational data including dissolved oxygen levels in ponds, and other available data to assess the biological treatment capability and efficiency of the plant. This will be a general overview of plant performance, not a detailed extensive review of overall plant performance.
3. Conduct a kickoff meeting/site visit and interview plant operations staff, review records, discuss issues and concerns, review plant operations staff recommendations for process/operational changes, and visually observe plant operations during a site visit.
4. Review effluent disposal operations and assess the adequacy of current effluent disposal facilities.
5. Provide a cursory review of the current staffing requirements and needs, and make recommendations for any changes or increased levels and/or quantity of staff.
6. Prepare a technical memorandum (TM) summarizing the findings of this study. We will attend one meeting (Wallace Group in person; Kennedy/Jenks via teleconference) to discuss draft TM comments prior to finalizing the TM.

DEFINITIONS

- BOD_5 – biochemical oxygen demand, a measure of the organic waste strength of a wastewater.

- TSS – total suspended solids, a measure of the solids suspended in wastewater

WALLACE GROUP
A California Corporation

612 CLARION CT
SAN LUIS OBISPO
CALIFORNIA 93401

T 805 544-4011
F 805 544-4294

www.wallacegroup.us

- TDS – total dissolved solids – minerals and salts that exist in solution state in a water or wastewater

CITY OF GREENFIELD DEMOGRAPHICS OVERVIEW

The City of Greenfield had a 2010 population reported at 16,330 (2010 UWMP). With growth noted to be at approximately 0.3% per year, current (Year 2013) population is estimated to around 16,500. The 2010 UWMP estimates that "build out" could be reached by around Year 2050, resulting in a projected population of 36,000. Refer to this 2010 UWMP for further details on City of Greenfield demographic factors.

A breakdown of current sewer connections, as provided by the City, is summarized as follows:

- Single Family Residential (SFR) – 2,815
- Multi-Family Residential (MFR) – 88
- Commercial – 98
- Industrial – 7
- Public Facility – 7
- TOTAL SEWER CONNECTIONS - 3015

In 2010, with the reported population of 16,330 and total annual water sales of 1,837 AFY (excluding metered landscaping), total water usage was calculated to be 101 gallons per capita per day (gpcd) on an annual average basis. For many central California communities, with low flow fixtures and increased water conservation, wastewater flows can be estimated at 75 gpcd or less. This would mean that the City of Greenfield's indoor water usage is approximately 75% of total potable water usage (indoor and outdoor usage), which seems appropriate.

Based on an estimated 75 gpcd wastewater flow, and an estimated current-day population of 16,500, it is believed that the City of Greenfield WWTP could be receiving up to 1.2 mgd total wastewater flows. However, with larger percentages of low flow fixtures and toilets, and larger per household densities, per capita flows of 60 gpcd and less are not uncommon.

WWTP GENERAL PROCESS DESCRIPTION

The treatment process is generally described as follows:

Raw wastewater enters the influent headworks by gravity via a 24" diameter circular gravity sewer pipe from Walnut Avenue, which discharges into a concrete rectangular channel. Raw wastewater then flows through a mechanical rake/screen, and control of the rake is actuated based on ultrasonic level measurement immediately upstream of the mechanical rake. Flow then passes through a coarse manual bar screen, where the channel then splits into two channels. At this point, wastewater flows through one of the two channels, each equipped with a comminutor before passing through a 6" Parshall Flume (in one of the channels, the second channel being the bypass channel) and primary clarifier flow splitter box. Raw wastewater then flows to three primary clarifiers operated in parallel.

Primary waste sludge and scum is pumped to two aerobic digesters for digestion, followed by discharge to sludge drying beds for drying/dewatering and ultimate disposal off-site. Sludge collected from Clarifier #1 and #2 is conveyed to Digester #1, and sludge collected from Clarifier #3 is conveyed to Digester#2.

Primary effluent flows by gravity to a splitter box, where flow may split between Oxidation Pond 1 and 2. Flow from Oxidation Ponds 1 and 2 then go to Oxidation Pond 3; Oxidation Pond 2 may discharge directly to Percolation Pond 4. Flow from Oxidation Pond 3 flows to Percolation Pond 5. Finally, effluent is pumped by a 60-HP pump station through manually maneuvered irrigation piping and is spray disposed on 26 acres of spray disposal fields. Refer to Figure 1 for a depiction of this overall plant process. An aerial view of the City's WWTP is shown as Figure 2. Figure 3 shows the flow patterns at the oxidation and percolation ponds, and Figure 4 shows the effluent spray disposal area.

GREENFIELD WWTP DESIGN CRITERIA

The following table (Table 1) summarizes the design parameters for the City's existing wastewater treatment plant. Unless otherwise noted, the design criteria is based on the Expansion Report for City of Greenfield WWTP, September 2003 (see Appendix B), Freitas+Freitas, and as supplemented by the June 11, 2004 Letter, Freitas+Freitas, regarding facility capacity factors and hydraulic and organic loading of the disposal ponds.

Table 1. Greenfield WWTP Design Criteria

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
Flows and Loading	ADWF, mgd	1.0	2.0
	Peak Flow, process, mgd	3.0	6.0[e]
	Peak Hydraulic Flow, mgd	5.0	10.0[e, q]
	BOD_5, mg/L (lb/day)	240 (2,000)[p]	240 (4,000)[p]
	TSS, mg/L (lb/day)	240 (2,000)[p]	240 (4,000)[p]
Headworks	Headworks Channel, mgd	0.1 to 2.5[n]	
	Number of Channels, dimensions (inches)	2@31"Wx32"D[l]	
	Chain & Rake Monster™, quantity (peak hydraulic capacity, mgd)	1 each (3.5 mgd)	
	Coarse Bar Screen, number	1	

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
	Comminutor, mgd	0.1 to 2.5[n]	
	Comminutor, quantity@HP	2@5HP each[l]	
	Flow Measuring/Parshall Flume, mgd	0.1 to 2.5 6" Throat[l, n]	
Primary Sedimentation	Number of Units	2@0.5 mgd	2@0.5 mgd, 1@1.0 mgd
	Diameter, ft	2@30'	2@30' 1@45'
	Removal Rate, %SS	60[a]	
	Effective Volume, ft3	2@6,126 ft^3	2@6,126 ft^3 1@12,253 ft^3
	Surface Loading, gpd/sf	707[a]	
	Detention Time, hours	2.2[b]	
	Weir Overflow Rate, gpd/LF	5,300	
Sludge Digestion (aerobic)	VSS (% of TSS)	75	
	VSS Reduction, %	40	
	Volume Treated Per Day, ft^3	347	694
	Number of Units	1	2
	Size, each unit (ft)	30' Dia x 13.5' Deep	
	Solids Retention Time, days	30	
	Rotary Lobe Blower, HP, each unit	10	
	Blower capacity, cfm	500	
	Loading Rate, lb VSS/ft^3-day	0.04 (0.06)[l]	
Sludge Drying (Lagoons/beds)	Number of beds	6[c, n]	
	Loading, lb/year	315,360[d]	630,720[e]
	Area, each bed, sf	62,500[n]	
	Volume, ft^3	125,000[d, n]	
	Loading Rate, lb dry solids/SF/day	0.006[l]	0.012[e, n]
	Loading Rate, lb/ft^3/year	2.52[e]	5.04[e, n]
Oxidation Ponds	Number	3	
	Surface Area, Total	6.25[j], 7.6[k]	
	Depth, ft	5	
	Detention Time, days	5.1[j], 14.9[k]	2.5[j], 7.4[k]

Process/Plant	Criteria, Units	1.0 MGD CAPACITY	2.0 MGD CAPACITY
	BOD_5 Loading Rate, lbs/acre-day	200[j]	400[j]
	BOD_5 Loading Rate, kg/acre-day	78[k]	156[k]
	Aerators	None	6@15 HP each[f]
Percolation Ponds	Number	2	
	Area, total, acres	4.21	
	Depth, each, ft	5	
	Percolation Rate, gal/acre-day	47,850	
	Application Rate, inches per day	2.3	
	Disposal Capacity, mgd	0.2[g]	
Spray Disposal Fields	Total Area, acres	13	26
	Application rate, inches/day (inches/year)	2.3 (70)	
	Capacity, mgd	0.812	1.62[h]

[a]*Stated for original two 0.5 mgd clarifiers only. Design % Removal of BOD is not stated.*
[b]*Detention time assumed to be based on ADWF.*
[c]*Only three beds were observed during February 2013 site visit.*
[d]*Need to verify if this is loading per bed, or total.*
[e]*Estimated value, based on same ratio for 1.0 mgd criteria.*
[f]*Proposed in June 4, 2004 letter, but not installed.*
[g]*It is noted that in the June 4, 2004 letter, it was stated that the oxidation and disposal ponds had never been cleaned or dredged since their construction in the 1970s, and that these ponds effectively do not percolate (currently).*
[h]*This means total effluent disposal capacity is 1.8 mgd with percolation ponds and spray disposal. See note (g) above also. In order to yield 0.2 mgd disposal capacity in the percolation ponds, they will need to be properly rotated, dried, ripped and solids removed. Winter storage or redundancy/buffer should also be considered.*
[j]*Based on 2003 Freitas Report.*
[k]*Based on June 2004 Freitas Letter Report.*
[l]*Section VI, RM Associates Report of Waste Discharge Report, July 2001.*
[m]*The O&M Manual does not indicate peak hydraulic capacity of this equipment.*
[n]*April 5, 2013 letter from Freitas+Freitas indicating original design criteria is sufficient for 2.0 mgd capacity.*
[p]*Refer to Section on “Wastewater Characteristics”. City will need to re-evaluate organic loading based on most recent Annual Report data.*
[q]*Observed peaking factors from flow chart recorders suggest this peak value may not be realized at the plant. Further evaluation is warranted.*

Figure 1. Process Flow Diagram

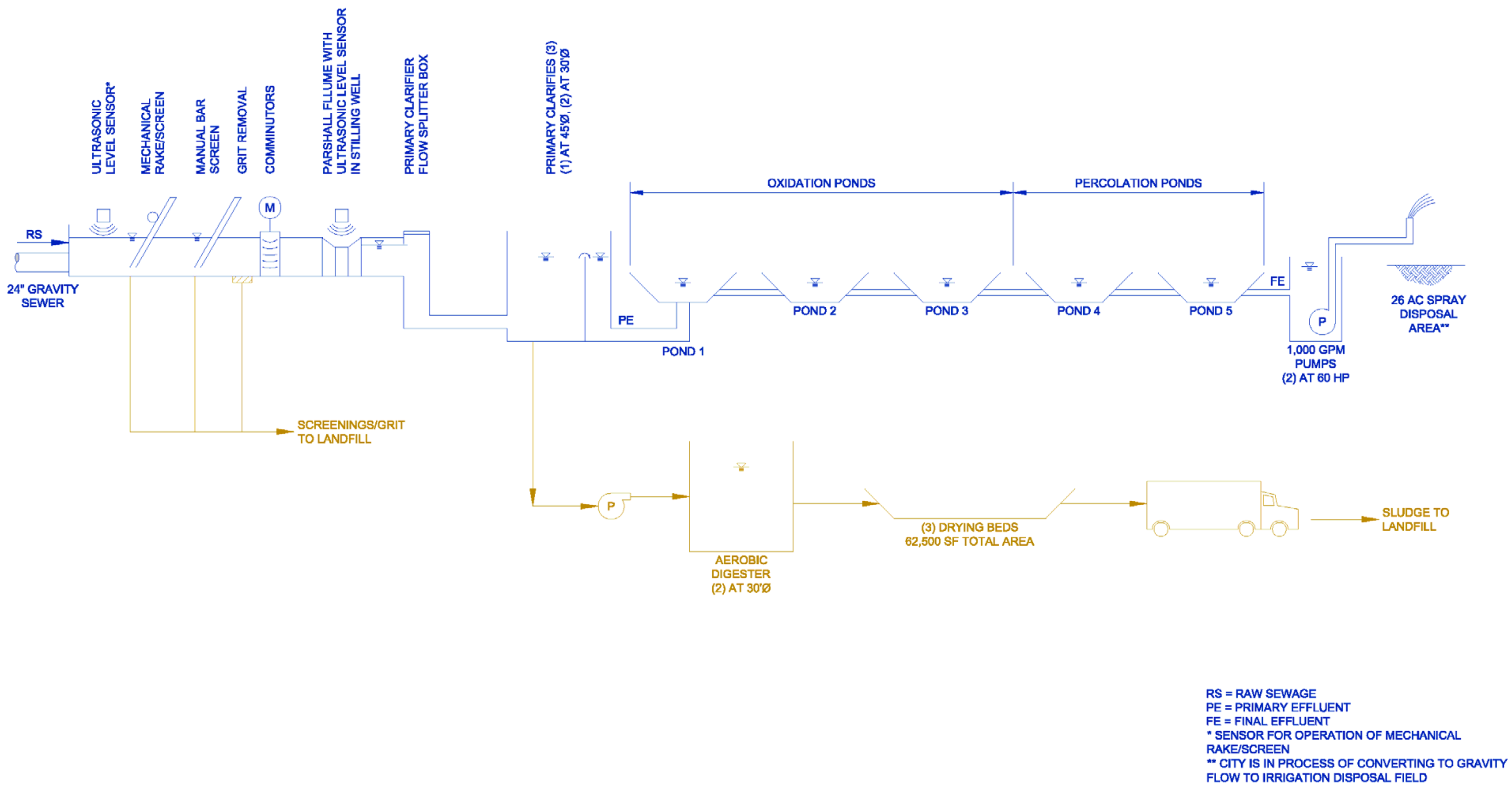

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FIGURE 1
PROCESS FLOW DIAGRAM
CITY OF GREENFIELD WWTP

JOB No. :	1163-0001
DRAWING :	116301-EX.DWG
DRAWN BY:	JSW
DATE :	03/06/13
SCALE :	NONE

Figure 2. Aerial View of City of Greenfield WWTP

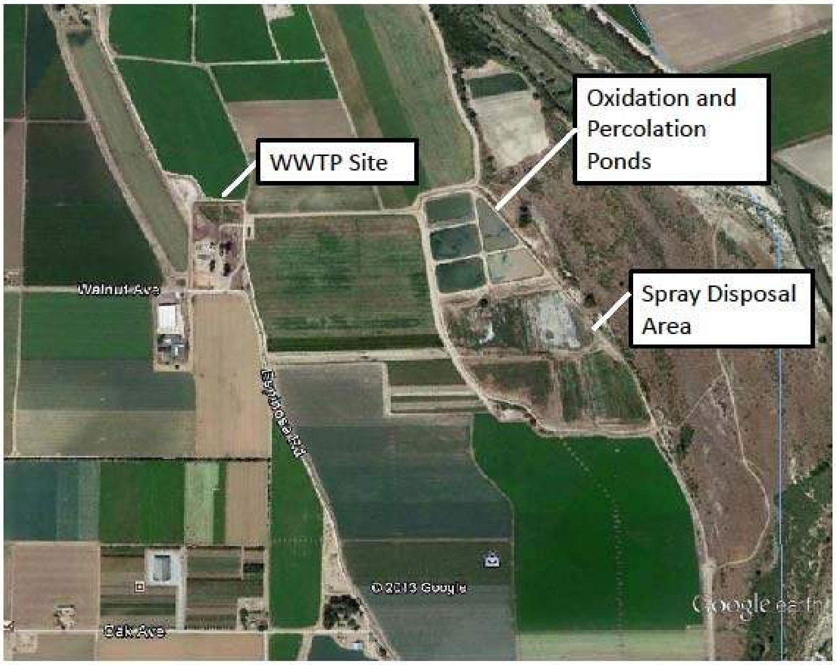

Figure 3. Oxidation and Percolation Pond Layout

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Figure 4. Effluent Spray Disposal Fields

WASTE DISCHARGE REQUIREMENTS AND SUPPORTING DOCUMENTS

The City of Greenfield WWTP is regulated by the Regional Water Quality Control Board (Regional Board), Central Coast Region, by Waste Discharge Requirements (WDR) Order No. R3-2002-0062. These WDRs were adopted May 31, 2002. Refer to Appendix A for a copy of these WDRs.

Key aspects of the City's WDRs are summarized as follows:

- Current plant capacity is stated as 1.0 mgd, with City plans for expansion to "at least 1.5 mgd". Flows to the WWTP in 2002 were reported at 0.91 mgd.
- Specification B.1, wastewater flows shall not exceed 1.0 mgd until certain facility improvements are completed and supporting design documentation is submitted to and accepted by the Regional Board.
- Specification B.4, effluent disposal operations shall not cause downgradient monitoring wells to exceed 8 mg/L nitrates (as N).
- Specification B.5, effluent disposal operations shall not cause downgradient monitoring wells to see "significant increases" in mineral quality.
- Specification B.11, effluent disposal ponds shall be alternated to permit emptying for maintenance purposes.
- Specification B.12, disposal ponds shall be dried and disked at least annually.

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- Specification B.13, wastewater application to spray irrigate disposal areas shall be managed to prevent ponding.
- Specification B.14, wastewater application to spray disposal areas shall not take place during rains.
- Specification B.16, spray disposal areas shall be operated using a regular rotation. Rotation from one irrigation area to another shall occur at least weekly. Between applications, irrigated areas shall be allowed to dry at approximately the field moisture condition of non-irrigated areas.
- Specification B.17, all solids generated must be reclaimed or disposed of in an acceptable manner.
- Specification B.18, all storm water contacting domestic wastewater shall be contained on site.
- Specification B.19, best management practices shall be implemented to minimize the inflow and infiltration of storm water into the facility.
- Provision C.5, City shall evaluate salt management practices and implement a long term salt management program. City shall submit report to Regional Board by March 1, 2003.
- Provision C.6 and C.7, City shall submit a report to the Regional Board by November 30, 2002 addressing groundwater monitoring wells and hydraulic gradient in the area of the facility. If disposal system is insufficient, City shall submit engineering report by March 1, 2003 evaluating various wastewater disposal options and shall consider water recycling as an option.

It is noted that there are no specific effluent treatment standards imposed in these WDRs.

Key aspects of the WDR monitoring requirements are as follows:

- Influent wastewater monitoring includes:
  - daily flow metering, maximum daily flow metered, and mean daily flow (calculated).
  - Quarterly BOD_5 and TSS (24-hour composite), settleable solids and pH (grab).
  - Annual TDS, sodium, chloride, sulfate, boron (24 hour composite)
- Pond monitoring, weekly grab samples for pH and dissolved oxygen.
- Effluent monitoring (discharged to spray disposal area):
  - Quarterly grab samples for pH, BOD_5, TSS, settleable solids, TDS, sodium, chloride, boron, sulfate, nitrite (as N), nitrate (as N), total Kjeldahl nitrogen (as N), total nitrogen (as N).
  - Annual grab sample for heavy metals.
  - Once every 5 years, grab sample for volatile organics and pesticides.
- Solids/biosolids monitoring:
  - Reported tonnage or yardage of sludge removed, each load.
  - Representative samples during transport/removal, for moisture content, nitrate (as N), pH, oils and grease, heavy metals
  - At least once every 5 years prior to transport or disposal, pesticides, organic lead and PCBs.

March 11, 2004 Letter from Regional Board to City (John Alves).

This letter from the Regional Board (see Appendix D) acknowledged receipt of the Expansion Report for City of Greenfield WWTP (see Appendix B) and the Wastewater Disposal Report (see Appendix E) for City of Greenfield WWTP on October 24, 2003. The letter indicated the submitted reports were insufficient in addressing certain aspects of the proposed WWTP expansion from 1.0 mgd to 2.0 mgd, regarding effluent disposal capacity and organic loading to the oxidation ponds. The Regional Board expressed concerns that the organic loading criteria used for the oxidation ponds exceeded typical design values, and that the effluent disposal capacity was not adequately addressed.

At the time of the original submission of the Freitas' reports, the following plant expansion items were proposed to increase capacity to 2.0 mgd:

1. Installation of a 1 mgd, 45-foot diameter circular clarifier
2. Installation of a 30-foot diameter aerobic digester
3. Installation of a small pump building to house the new sludge and scum pumps
4. Installation of new interconnecting piping
5. Expansion of the existing spray irrigation fields to 26 acres

Supplemental information, at the request of the Regional Board, was submitted on June 22, 2004.

June 22, 2004 Supplemental Information (transmittal dated June 11, 2004 from Freitas + Freitas).

At the request of the Regional Board, supplemental information was submitted to the Regional Board (see Appendix F) to address concerns expressed regarding organic loading to the oxidation ponds, and effluent disposal capacity to meet 2.0 mgd rated plant capacity.

Effluent Disposal Capacity. Freitas noted in this letter, that the oxidation ponds had not been dredged since the 1970s, and it is believed that these ponds do not effectively percolate. They substantiated this claim by comparing pumping records to the effluent disposal field to influent wastewater readings. Freitas indicated that the data at that time supported an effluent disposal rate of 80,000 gallons per acre per day (0.24 feet per day).

Oxidation Ponds. Freitas noted that the existing oxidation ponds, rated at 1.0 mgd, have the following loading rates:

- BOD_5 loading rate = 78 kg/acre-day (220 lb/acre-day)
- Detention Time=14.9 days

Freitas concluded that by increasing the plant flows to 2.0 mgd, the following loading rates would be observed in the oxidation ponds (without aeration):

- BOD_5 loading rate = 156 kg/acre/day (440 lb/acre-day)

- Detention Time=7.4 days

Freitas recommended that six 15-HP aerators (a total of 90 HP) be added to the oxidation ponds. As of the date of this Report, these improvements have not been completed.

August 30, 2004 Letter from Regional Board to City (John Alves).

This letter from the Regional Board (see Appendix G) acknowledged receipt of the Expansion Report for the City of Greenfield WWTP and the Wastewater Disposal Report for City of Greenfield WWTP on October 24, 2003, and supplemental information, at the request of the Regional Board, on June 22, 2004.

The Regional Board conditionally approved expansion to 2.0 mgd rated capacity, from current rated capacity of 1.0 mgd per WDR Order R3-2002-0062, upon successful completion of the previously identified improvements, including the addition of six 15-HP floating aerators on the oxidation ponds. The City was also required to submit status reports quarterly on the status of these above facility improvements.

February 22, 2013 Site Visit

On February 22, 2013, Wallace Group (Steve Tanaka) and Kennedy/Jenks (John Jenks) conducted a site visit and tour of the wastewater facilities. Ed (Sonny) Vaughn, Grade III contract operator, and Dale Lipp, Public Works Director, also attended this site visit.

<u>Observations</u>

<u>Initial Treatment Units/Headworks</u>

Influent raw sewage enters the plant by gravity, and the mechanical rake/screen is controlled by measurement of influent channel liquid depth, as measured by an ultrasonic meterFollowing the mechanical rake, the flow channel splits into two channels, where the flow passes through a coarse bar screen, followed by two comminutors (one in each channel), a Parshall Flume equipped with a stilling well and ultrasonic level sensor, then a flow splitter box which controls flow to three primary clarifiers operated in parallel.

<u>Flow Metering</u>. Flow metering is accomplished with a Parshall Flume (6" throat) and ultrasonic level sensor installed in a stilling well. The flow measurement should be calibrated at least annually and verified for accuracy. Given current estimated population, wastewater flows are suspected to be slightly higher than metered flows.

<u>Screenings</u>. The automatic bar screen followed by comminutors were seen as possibly being relatively ineffective as to the function of reducing the volume of rag materials in particular. Sonny Vaughn suggested considering an alternative means of screening such as a rotary screen in place of the comminutors.

Clarifiers

Two of the three clarifiers were in operation at the time of the visit. These clarifiers appear to be functioning as intended. There seemed to be more accumulation of scum and grease on the clarifier surfaces than might be expected, and could be a reflection of ineffective initial treatment units and possibly excess fats, oil and grease (FOG) in the collection system.

Solids Handling

Sludge is pumped with use of plunger-type pumps, from the clarifiers to one of two aerobic digesters operating in parallel. Digestion occurs through introduction of air to the sludge mixture. Operation is on a fill and draw basis. Air blowers to the digesters are operated on a time clock sequence. It was uncertain whether there is any monitored program and a basis for such a program such as dissolved oxygen levels within the digesters. It was also uncertain as to the basis for when sludge is withdrawn from the digesters and sent to the sludge storage/drying lagoons.

Odors from these lagoons seem to indicate that solids digestion may only be partial. In addition, an abundance of weeds was observed on the embankments, along with observed bank erosion. A third sludge drying lagoon is present; it is connected “hydraulically” to the second sludge lagoon by pipe connection. Other noted poor housekeeping included observations of screenings dumped on the ground in this vicinity of the sludge lagoons.

Oxidation Ponds

Primary clarifier effluent flows by gravity to one of three oxidation ponds, two of which were in operation at the time of our visit. The third oxidation pond was dry and had a significant accumulation of dried sludge deposited on the bottom. The two operating oxidation ponds had “Solarbees” installed for purposes of providing some mixing and aeration. The two operating oxidation ponds appeared to be significantly overloaded from both appearance and odor.

Infiltration/Percolation Ponds

Oxidation pond effluent then flows to one of two percolation/infiltration ponds, both of which were in operation at the time of visit. There apparently is no effort to operate these ponds on the basis of intermittent operation of use, then drying and scarifying the bottoms.

Effluent Spray Disposal System

Treated wastewater in excess of that disposed of from the infiltration ponds, is pumped to 26 acres of spray disposal fields. This system is comprised of a pumping station and irrigation pipe to spray heads within the disposal area. It was reported that the spray system utilizes 1,000 gpm 60 H.P. pumps and that the irrigation pipe is manually changed and moved to rotate use of the spray disposal fields. It was reported that the spraying operation is sometimes thought to be a source of odors. It is not clear how irrigation application rates are tracked and controlled. The spray disposal fields are surrounded by agricultural fields/food crops. Refer to Figure 4 for a view of the effluent spray disposal ponds.

February 26, 2013 File Review at Regional Board Office

On February 26, 2013, Wallace Group conducted a brief file review at the Regional Board office. As part of this review of the file, several past inspection reports were reviewed. A summary of recent inspection reports is as follows:

- 05/23/08. No violations noted on this visit.
- 02/03/06. Inspector noted all five ponds had standing water, as well as noticeable odors at the ponds.
- 11/10/04. City was cited for significant ponding in the lower spray disposal area.

06/11/03. Inspector noted inadequate pond freeboard, and pond vegetation accumulations.

Based on a review of this information, it is apparent that the noted concerns described above stem mainly from plant operational issues, with the exception possibly the issue of pond odors (which is likely due to organic overloading of the oxidation ponds).

Annual Reports

The past three annual reports (2012, 2011, 2010) submitted to the Regional Board, were reviewed as part of this study.

The 2012 annual report indicates that in regards to plant capacity, the "modifications to the wastewater plant are 100% complete". Based on our review of the current plant status, additional steps will be required to ensure 2.0 mgd capacity. Once the City concurs with these updated recommendations, we recommend the City send the Regional Board a letter describing a plan and schedule for implementing the necessary means of achieving 2.0 mgd rated capacity.

The 2012 annual report indicates that the City's O&M manual for the WWTP was updated in 1993 and reflects current plant operations. It is recommended that the City review the status of their most recent O&M manual (dated 2008), and make reference to the most current O&M manual. A cursory review of this O&M manual indicates a need for the manual to be updated and expanded upon, to include at a minimum, the following:

- Complete and up to date summary of all plant elements, design criteria, including maps and drawings of existing plant headworks, primary clarifiers, digesters, controls building, electrical and controls, oxidation and percolation ponds, effluent pump station, spray disposal field.
- Description of how each process is to be operated, optimized, maintained, with corresponding log sheets for recording and documenting operation and maintenance activities.
- Equipment and process trouble-shooting.
- Mechanical equipment maintenance requirements, logs and records.

- Plan for continual maintenance and upkeep of the oxidation and percolation ponds, including procedures and schedules for emptying, drying and disking ponds, weed and embankment maintenance.
- Operational procedures for the effluent spray disposal area, effluent pump station.

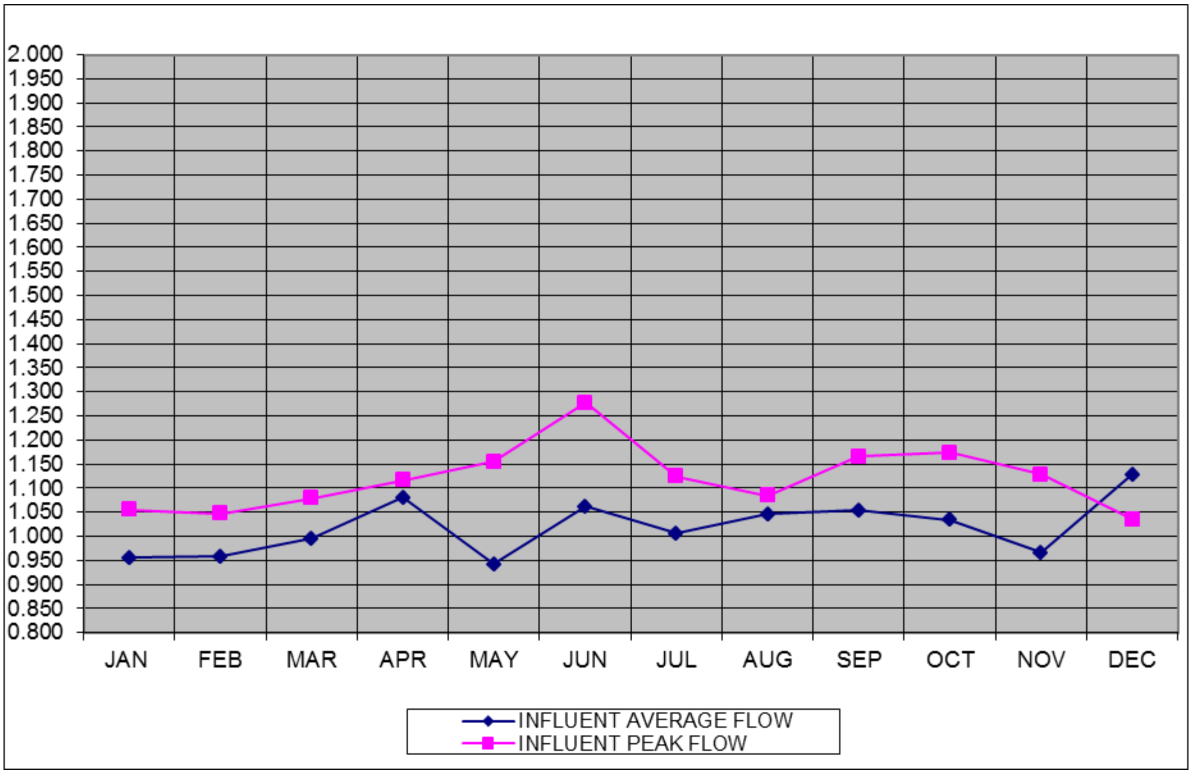
A detailed review of the City's O&M manual was beyond the scope of this study; however, based on our cursory review, we recommend the City immediately take steps to update this Manual.

WASTEWATER CHARACTERISTICS

The following subsections summarize influent wastewater flows, influent organic waste strength (expressed in BOD_5 and TSS) and effluent quality (also expressed in BOD_5 and TSS).

Plant Influent Flows. Figure 5 shows the influent wastewater flows measured, metered and reported to the Regional Board for calendar year 2012. Flows are expressed in million gallons per day (mgd). As noted earlier in this report, it is suspected that influent wastewater flows may be slightly higher than reported, due to possible flow metering inaccuracies. With current population estimated at 16,500 people, it is expected that influent flows average around 1.2 mgd (based on a per capital wastewater flow of 75 gallons per capita per day [gpcd]). However, it is also noted that per capita wastewater flows may decrease with household size, and given the demographics of Greenfield, the persons per household may be between 3 and 4 persons per household. Based on this,the Metcalf & Eddy resource indicates a wastewater flow per capita in the 40 to 70 gpcd range. Regardless, it will be important for the City to confirm the accuracy of influent flow meteringto determine actual wastewater flows to the facilities.

Figure 5. Calendar Year 2012 Influent Wastewater Flows (from City of Greenfield Annual Report to the Regional Board)



A check of 2012 winter-time water consumption was reviewed. Based on the 2012 water production data, the lowest winter-time consumption of water occurred in February 2012. Daily water demand averaged 1.26 mgd for this month, which may indicate the actual plant flows of 1.0 mgd may be reasonable. However, a verification of plant flows is still recommended to verify accuracy of flows.

Review of Influent Flow Charts. A select interval of chart recordings were reviewed, November 24, 2012 through December 5, 2012. This interval was selected as there were mostly dry weather days preceding this interval, with one rain event on November 30, 2012. This interval captured weekdays, weekends, and captured the Thanksgiving holiday weekend. A review of the California Irrigation Management Information System (CIMIS) website showd only inactive weather startions in the Greenfield area; however, there was believed to be precipitation in Greenfield on November 30, 2012, based on review of the closest available data to Greenfield (Arroyo Seco Station #114). Based on this limited review of influent data, the City's wastewater flow pattern exhibits a typical diurnal peak mid-morning, and again in the early evening. Weekend peak hour flows tended to crest in the early to mid afternoon, with a gradual increase from morning hours to the afternoon. It is interesting to note, that in reviewing weekday flow trends in September 2012, the morning peak hour flows were measured around 8 am, as opposed to 10 am in November 2012. This is likely due to a higher level of farming activity in the warmer months, with workers getting ready for work much earlier in the day. The largest peak hour flow peaking factor was calculated at 2.3, and coincided with weekend days. Whether any inflow/infiltration is occurring in the system is inconclusive given the limited data available. The largest peaking factors occurring on the two weekend days following the recorded rain day are likely a coincidence, given that ADWFs for those days did not increase any. A summary of the flow monitoring data is included in Table 2.

Table 2. Summary of Influent Flow Data

		Flow, mgd[1]			
Day	**Date**	**Avg**	**Max**	**PF[2]**	**Comment**
Sat	11/24/12	0.94	1.71	1.8	Weekend Day of Thanksgiving Weekend
Sun	11/2512	1.01	2.00	2.0	Weekend Day of Thanksgiving Weekend
Mon	11/26/12	1.04	2.04	2.0	
Tue	11/27/12	1.01	1.70	1.7	
Wed	11/28/12	0.94	1.87	2.0	
Thu	11/29/12	0.95	1.70	1.8	
Fri	11/30/12	0.95	1.82	1.9	1.2 Inches rain[3]
Sat	12/01/12	1.00	2.31	2.3	
Sun	12/02/12	1.00	2.27	2.3	
Mon	12/03/12	1.04	1.88	1.8	
Tue	12/04/12	0.97	1.70	1.8	
Wed	12/05/12	0.93	1.68	1.8	

[1]Based on City-provided flow chart recorder sheets
[2]Diurnal Peaking Factor
[3]Recorded at Arroyo Seco Monterey Bay Station 114 (CIMIS Website).
No active weather stations could be located in vicinity of Greenfield.

Influent BOD$_5$ and TSS. Influent waste strength is typically measured as organic waste strength expressed in BOD$_5$ and TSS. These two values typically define the municipal wastewater strength, by which most biological wastewater processes are designed and compared against for treatment effectiveness. Tables 3 and 4 summarize the quarterly influent wastewater sampling results (24-hours composite sample results) for calendar years 2010 through 2012. Based on the results for both parameters, Year 2012 seems to show a significant increase in both BOD$_5$ and TSS. This should be evaluated in further detail to verify the trend in wastewater strength, which could be a very significant consideration for current and future wastewater plant loadings. Based on

Table 3. Influent BOD5 (mg/L) for Calendar Years 2010 through 2012

2012	2011	2010
576	178	272
228	255	187
365	212	200
220	288	170
347.25	233.25	207.25

the Year 2012 BOD_5 results, this equates to approximately 0.17 lb/capita/day, which is in the expected range of organic loading (Table 3-12, Metcalf & Eddy, see Reference 7) for a community such as Greenfield.

For the purposes of assessing the current treatment plant organic loading, the following BOD and TSS values will be used:

- Influent BOD5, 300 mg/L (5,000 lbs/day@2.0 mgd flow)
- Influent TSS, 275 mg/L (4,600 lbs/day@2.0 mgd flow)

Table 4. Influent TSS (mg/L) for Calendar Years 2010 through 2012

2012	2011	2010
477	122	146
210	72	208
684	182	106
278	214	68
412.25	147.5	132

PLANT POWER CONSUMPTION

Fiscal year 2010/2011 and 2011/2012 power bills were reviewed and summarized for the main plant meter, and the effluent pump station (spray disposal). Figure 6 shows the power consumption (in Kilowatt hours, or Kwh) for the plant, which includes the headworks, clarifiers, aerobic digesters. Figure 7 shows the power consumption for the effluent pump station and spray disposal operations. It is noted that the significant power usage at the plant is for effluent spray disposal.

Figure 6. FY 2010/11 and 2011/12 Main Plant Power Consumption

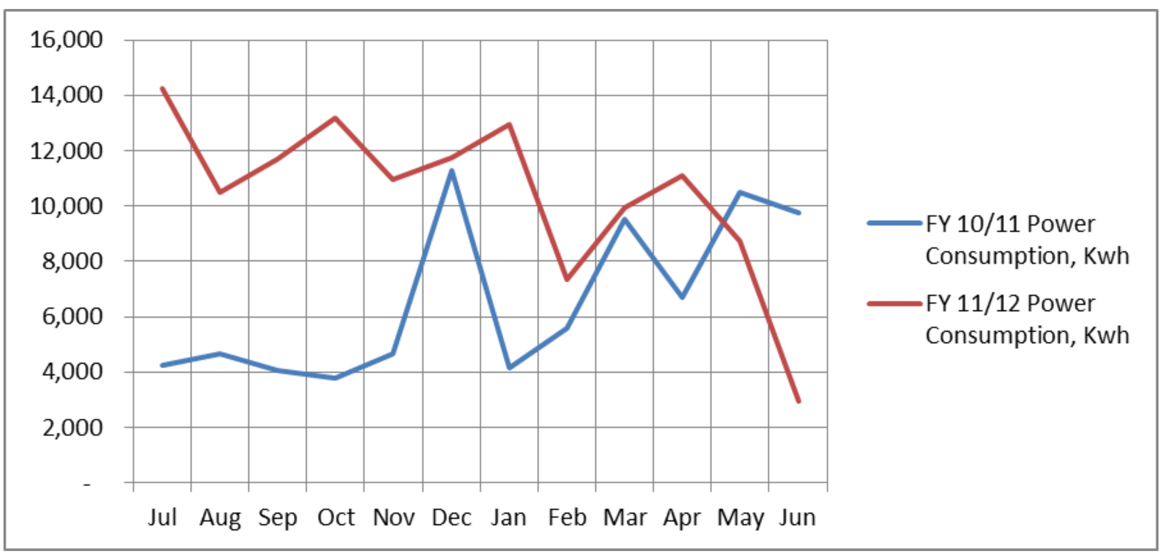
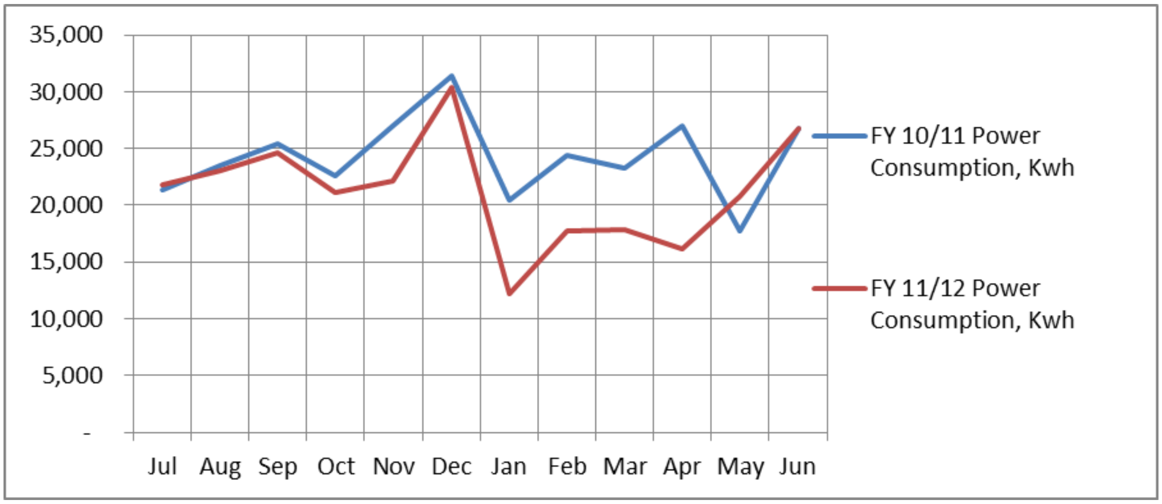


Figure 7. FY 2010/11 and 2011/12 Effluent Pump Station Power Consumption



Main Plant Power Usage. Power consumption at the plant averaged 6,600 Kwh per month during FY 2010/11. However, power consumption jumped to 10,500 Kwh per month, a 60% increase from the prior fiscal year. It is surmised that the increase in power consumption was due to the second aerated digester coming on line. Review of the first six months of power usage in FY 2012/13 (not shown in the above figures) indicates that power consumption this current fiscal year will match that of FY 2011/12. The power consumption over the year does not appear to follow any established trend, and the variations may in part be due to the timing of billing and reading of meters.

Effluent Spray Disposal Power Usage. The power consumption for the effluent spray disposal fields averaged 24,250 Kwh per month during FY 2010/11 and 21,200 Kwh per month during FY 2011/12. It is uncertain as to why power consumption decreased during FY 2011/12 from the prior year. Although the power consumption seems to vary significant over the months, there are strikingly similar trends in power usage from month to month, for FY 2010/11 and 2011/12. The winter-time peak in power consumption in December appears to be from extended spray disposal operations during the wet winter months. However, the significant drop in consumption in January during both fiscal years is not clear. Again, part of this anomaly may be due to timing of reading of the meter.

Total power cost for effluent disposal in FY 2011/12 was $38,960. Using these power bills, and the 70HP pumps, it was calculated that the effluent pumps ran approximately 12 to 13 hours daily. Assuming that these pumps could be substituted with low-head pumps to discharge to a standpipe for flood irrigation to the spray fields (converted to percolation beds), the effluent disposal area modifications could yield a $25,000 annual power savings. This power savings would not be “pocketed” in the sense that the added 90 HP aeration requirements for the oxidation ponds will consume the power that the 70HP effluent pumps were consuming, plus additional power consumption. However, such revisions to the effluent pump station could yield significant savings over the years. Also, some of the existing spray disposal fields can likely be flood irrigated by gravity, yielding even more power savings.

SLUDGE DIGESTION

There is a significant limitation as to ability to analyze effectiveness of the existing solids handling system, notably the two existing aerobic digesters due to a lack of confirmed data. What is known is that the aeration system is operated on a timed basis and is not operating continuously. What is known is that the digesters are operated on a partially filled basis with periodic settling and withdrawal with unknown timing. Also, it is known that withdrawn digested sludge is to one of two, or three nearby lagoons for storage and drying. There is no information as to effectiveness of digestion, reduction of volatile solids and ability to meet Federal 503 requirements for digested sludge disposal. In order to gain a better understanding of solids digestion adequacy, a program of monitoring essential operating results should be established. Such program should include monitoring sludge for %volatile solids, %non-volatile solids, % volatile solids reduction, and overall sludge production from the aerobic digesters on a dry weight basis (dry tons).

It should be noted that use of aerobic digestion for primary clarifier solids is unusual and should be questioned as to continuing practice at the Greenfield installation. In this further regard, a preliminary determination of solids balance in the event that the existing two digesters were converted for anaerobic operation indicates adequate capacity for something more than 1 mgd of incoming wastewater. A separate study should be undertaken as to best means of solids handling, digestion and disposal.

It is noted that Freitas+Freitas developed a sludge digester operation plan for City operation staff, in 2008. Plant staff should employ these operational procedures and monitor the effectiveness of implementing these recommendations, which should also be included in the City's overall O&M Manual.

The two existing aerated digesters are likely using more energy than necessary to digest primary sludge. This in part, is due to the current digester operation of setting the aeration blowers on timers. In the near-term, transitioning to anaerobic digestion is not feasible. The digesters are not sufficiently large in size/volume to accommodate such a conversion.

BIOSOLIDS REMOVAL

There is currently a significant accumulation of sludge at the City's plant. There is sludge deposited in the existing sludge lagoons, ranging from very dry to wet. Oxidation Pond No. 1 also contains a significant layer of dried sludge; depth of this sludge blanket and quantity is not known. There is likely an accumulation of sludge in all of the remaining oxidation and percolation ponds.

The City should manage, dry and remove sludge from the WWTP site on a routine basis to avoid accumulation of biosolids. Biosolids removal can be contracted out to a sludge hauling/composting operation such as Liberty Composting in Kern County, among others, or taken to the local landfill (Johnson Canyon Landfill, Gonzales). The decision as to which option to select will in part be based on cost. According to Freitas+Freitas, the local landfill option may be more economical.

Wallace Group recently assisted a municipal client in San Luis Obispo County for dewatering, removal and disposal of biosolids from a wastewater treatment plant operation. The services of Liberty Composting (contact person, Drew Kolosky) were utilized, and the following summarizes the most recent understanding of requirements and costs:

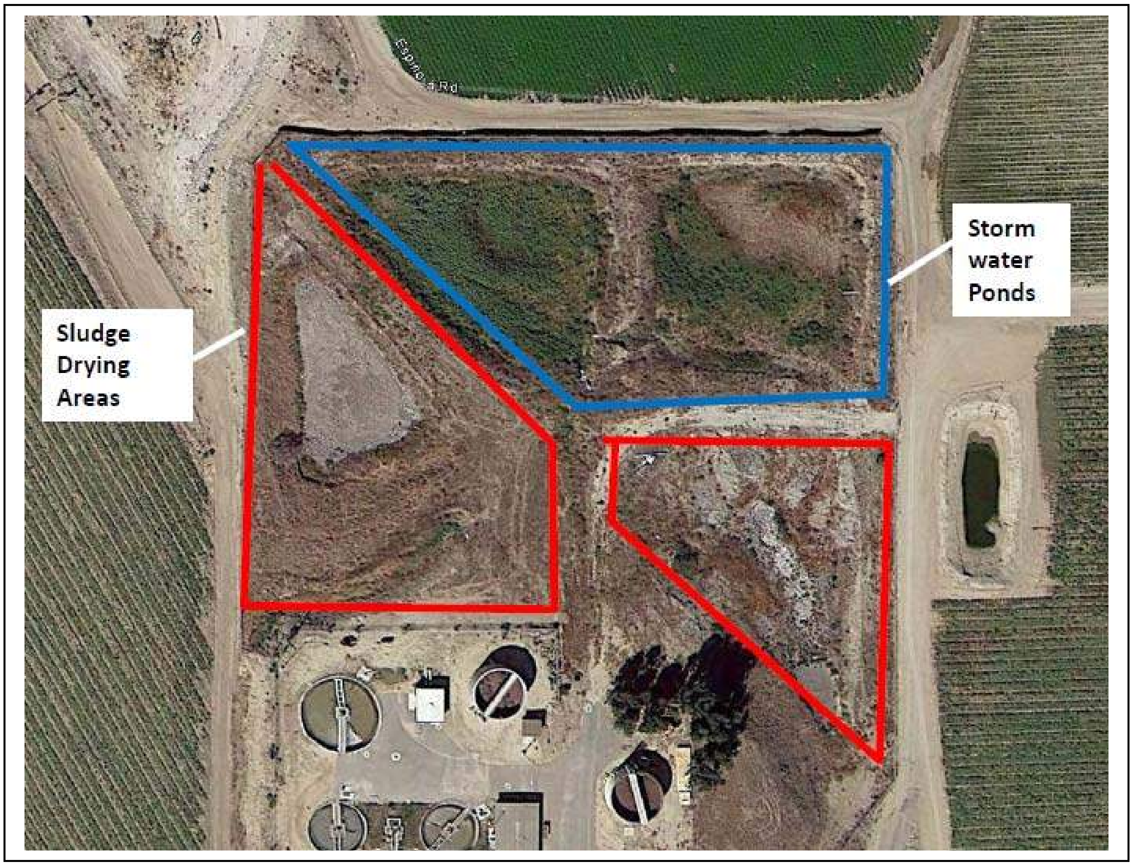
- Owner to stockpile sludge in accessible area
- Tipping Fees based on wet ton weight (thus the drier the sludge, the more economic the price for Owner)
- $30.00 per wet ton tip fee;
- $4.00 per wet ton to screen out plastics, etc; (with headworks/screening, this would likely not be required for City of Greenfield)
- $3.00 per wet ton to mob/demob Liberty's loader, plus labor to load onto Liberty's trucks;
- ~$20 per wet ton freight (depends on wet weight of sludge and limitation of weight per truck load).

In summary, the City should budget on the order of $60/wet ton for a contract composting facility to load, haul and land apply biosolids. This cost does not include City staff efforts to manage, dry remove and stockpile sludge for pickup by an outside service. An operation such as Liberty Composting has a number of accounts in this general area, and scheduling pickup of sludge should be relatively easy to plan for. Should the City elect to continue with disposal at the local landfill, the budgeted $60/wet should also be more than sufficient to cover sludge disposal related costs.

The City should quantify the amount of biosolids on site, dry the sludge to the extent possible, and stockpile sludge for future disposal. Such disposal should be on an annual and/or as needed basis to minimize stockpiling of sludge and to minimize potential nuisance conditions and "re-wetting" of sludge during rainy seasons.

Sludge Beds. The sludge beds should be cleared of weed accumulation, and the embankments touched up and re-graded where needed. As part of the overall sludge management plan, the sludge drying lagoons should be considered to be operated as drying beds, thus avoiding large discharge volumes of ponded sludge. Instead, sludge should be applied in small layers, quickly dried and removed. The City has suggested to consider employing mechanical dewatering such as a belt press. At this time, and given the land availability around the plant site (approximately 2 acres, including the area where the stormwater ponds are located), mechanical dewatering would be expensive. This would include building enclosure, odor scrubbing, belt presses, piping and other required ancillary equipment. Final cake from a belt press would be in the range of 20% solids, and would still require further drying prior to landfilling (direct haul for composting would be acceptable, but payment for disposal is on a wet ton basis and would be costly, if directly hauled to a composting site). At this time, it is recommended the City improve the existing sludge drying area to better accommodate thin spreading and effective drying, and in the near future consider the design of conventional sludge drying beds. This is envisioned to cost $300,000. Refer to Figure 8 for a depiction of the existing sludge drying area. The City should also consider relocating the stormwater ponds to another site and use this available space for future sludge drying.

Figure 8. Existing Sludge Drying Area



EVALUATION OF OXIDATION PONDS

Currently there do not appear to be any effluent discharge limitations specifically related to the existing oxidation ponds. Through earlier correspondence between the City and the RWQCB, the City proposed as part of the improvements for achieving 2.0 mgd plant capacity, the addition of floating aerators in each of the three existing oxidation ponds. The stated objective of floating aerator additions was to ensure maintaining a minimum oxygen concentration in the ponds of 2.0 mg/L. To achieve this objective it was proposed that six new 15 HP floating aerators be installed, two in each of the existing three oxidation ponds. It was also indicated that pond depth would be 6 feet rather than the currently 5 feet (as noted in prior design documents) of depth.

In the absence of stated oxidation pond effluent treatment limitations, it may be assumed that oxidation pond operation at the WWTP would relate directly to the "nuisance" objectives contained in all waste discharge requirements throughout the State. Regional Water Quality Control Boards have translated the avoidance of "nuisance" conditions in different ways including minimum pond concentration of dissolved oxygen at all times, maximum soluble BOD (SBOD) or carbonaceous BOD (CBOD) in pond effluent or maximum total BOD (TBOD).

For the purpose of preliminary estimates for oxidation pond improvements, it has been assumed that design objectives should include maintaining 2 mg/L dissolved oxygen concentration at all times, and oxidation pond effluent directed to disposal should contain no more than 50 mg/L CBOD.

To achieve the maximum CBOD objective of 50 mg/L, a standard aerated oxidation pond formula has been utilized:

$Cn/Co = 1/[1 + (kt/n)\ ]n$ where:

Cn = Effluent BOD = 50 mg/L
Co = Influent BOD = 170 mg/L
k = First –order reaction rate constant = 0.4
t = detention time, days
n = number of ponds in series = 3

From which, t = 6 days

At 2.0 mgd, the required total oxidation pond volume to achieve 10 days detention time would be about 12 million gallons (mg). The current total existing oxidation pond volume (based on 6.24 acres by 5 ft. depth) provides about 10 mg. The needed 12 mg could be provided by deepening the existing ponds by about one foot, or possibly it could be demonstrated through future operation that additional volume may not be needed.

In respect to needed supplemental oxygen provided through use of floating aerators, going through standard calculations and assumed design factors, it can be show as follows:

- Assuming oxidation pond influent BOD_5 from primary clarifiers is at a concentration of 170 mg/L, total incoming BOD would be 2 x 9.34 x 170 = 2,800 lbs/day.

- Assuming oxidation pond effluent CBOD concentration is 50 mg/L, total effluent BOD_5 removed through oxidation ponds would be 2,800 – (2 x 8.34 x 50) = 1,966 lbs/day.

- At total oxygen required at 1.5 lbs/lb BOD, O_2 required = 1,966 x 1.5 = 3,000 lbs/day

- Assuming AOR transfer rate of l.4 lbs O_2 per day per HP-hr. HP needed = 3,000/1.4/24 = 90 HP total, or say two 15 HP floating aerators in each of the three existing oxidation ponds. Also to be considered might be four 10 HP aerators strategically combined with 5 HP aerators to be determined as part of a final improvements design project. Such addition of a larger number of small HP aerators may minimize erosion to the pond embankments.

It is noted that basically, the foregoing confirms the prior engineering studies identifying the need for 90 HP of floating aerators (two new 15 horsepower floating aerators in each of the three existing ponds).

The foregoing also suggested that there is a theoretical deficiency in needed total detention time at the design 2.0 mgd capacity.

As part of preliminary recommendations, Oxidation Pond No. 1 should be de-sludged, and pond embankments abated of weeds and and re-conditioned. The City should maintain a program to minimize the presence of burrowing animals that could endanger pond embankment integrity. At this time, the City may also consider deepening Oxidation Pond No. 1, and also lining the pond. Although not currently a requirement of the Regional Board, at some point in the future, it is expected that the Regional Board will require pond lining of the process treatment ponds. Such lining would also minimize embankment erosion. It is noted that the City will need to establish a pond sludge removal program, to avoid large accumulations of biosolids in the ponds. If the oxidation pond is lined, the de-sludging operation would need to be accomplished by periodic dredging of the pond, and on-site dewatering/drying.

It is noted that Freitas+Freitas designed aeration improvement plans in 2012 for the City. It was proposed that five 25-HP floating aerators be installed, one each in the three oxidation ponds and two effluent percolation ponds. It is presumed that the intent was to convert the two effluent percolation ponds to aerated lagoons, thus relying on the effluent spray disposal area for all effluent disposal. To date, this work has not been implemented. Although we concur with the aeration requirements, we would be concerned that with the depth of the ponds, a single 25-HP aerator in each pond may raise potential issues for localized erosion in the pond bottoms, and possibly the embankments. A larger number of smaller HP aerators may want to be considered.

City operations staff indicates that floating aerators come with high maintenance costs, safety issues, and possible dead spots in the ponds. A detailed life cycle cost analysis and comparison of various aeration alternatives was beyond the scope of this study. However, as the City plans to move forward with aeration, additional focused design study should be conducted dto finalize a selection on type of aeration to be employed at the site.

Mazzei Aeration System

At the request of the City, Wallace Group evaluated an option to utilize the Mazzei Wastewater AirJection® Aeration System. This system works by circulating wastewater through venture injectors creating a vacuum to draw air/oxygen into the wastewater. The aerated process water is then discharged back into the process lagoon. Water discharged through patented Mazzei Nozzles for additional oxygen/air transfer and mixing before discharging back to the lagoon.

Technical representatives of Mazzei Corporation were contacted, and provided the same organic loading criteria applied to the surface aerator option (for a 6-foot deep wastewater lagoon). The following summarizes the operating parameters suggested by the manufacturer:

Number of Injector Units:	6 (12”)
Total recirculation rate:	15,000 gpm (2,500 gpm per injector/nozzle)
Brake Horsepower Required:	206
Aerator efficiency:	0.69 lbs O_2/BHP-hr

The total cost of the injectors and nozzles would be approximately $134,000 (equipment cost only). This cost excludes the pumping requirement, pump/controls building, and possible electrical service upgrade. The pumping system is estimated to cost $500,000, for a total capital cost of $650,000 or more. Due to the injection/recirculation flow rates (two injectors per lagoon), there would be the potential for localized erosion at the injection sites. Consideration may need to be given to fortifying the pond bottom and embankments in the immediate area of each injector unit.

The total energy requirements for this application are more than double that of the surface aerator option. The addition of pumping facilities and possible electrical service upgrade may also increase capital costs to implement this option. Power requirements and monthly energy bills would be significant more expensive than the surface aeration alternative. As noted above for the floating aerator discussion, additional detailed engineering studies can be conducted to better refine this alternative and other aeration options in lieu of conventional floating aerators.

Other Considerations (future). Another alternative to adding aeration to the oxidation ponds is to consider the addition of a fixed film reactor (FFR), also known as trickling filter, ahead of the oxidation ponds. Depending on timing and immediate needs to upgrade the plant, this may not be viable for a near-term improvement. However, it is estimated that a 2.0 mgd FFR process followed by oxidation ponds, could operate on 25 horsepower as compared to the 90 HP recommended to aerate the existing oxidation ponds. Assuming the FFR pumps operate 24 hours per day, 365 days per year, compared to the oxidation pond aerators that would operate between 12 and 24 hours per day, expected annual power savings could be on the order of $45,000 to $65,000 per year. If the FFR capital cost is approximately $1M to construct, at a 40 year life and 5% interest factor, the annualized cost (capital plus power) of a FFR is $83,000. The annualized cost for lined oxidation ponds with 90 HP aeration is approximately $165,000. The cost/benefit of adding an FFR as a long-term solution should be considered. Additional engineering studies can be performed to refine this alternative. This alternative maybe considered when expanding to meet future build-out capacity of 4.0 mgd.

WASTEWATER TREATMENT PLANT STAFFING NEEDS

The City’s current staffing requirements were reviewed as part of this letter report. As we understand it, the City currently employs three full-time operators (Arturo Felix, Grade 2; Ivan Barron, Grade 1; Alejandro Alvarez, OIT), and one temporary Grade 3 operator (Edward “Sonny” Vaughn), currently acting a chief plant operator (CPO). Based on the size and complexity of the City’s wastewater facility, the CPO should be a Grade 3 operator, as is currently provided. Other plant staff may be Grade 3 or lower.

Using an excel spreadsheet program (dated August 2006) based on a USEPA Publication "Estimating Staffing for Municipal Wastewater Treatment Facilities", dated 1973, we calculated staffing needs for the City's wastewater plant. Consideration was given to current wastewater flows (~1.3 mgd) and current plant improvements, and desired plant rated capacity of 2.0 mgd and anticipated near-term improvements to include pond aeration.

A "sensitivity" analysis was conducted, using variable inputs to the program for flows ranging from 1.5 to 2.0 mgd, and with the current oxidation pond operation and expected future addition of aeration to the ponds. Both variables resulted in a recommended range of 3 to 4 full-time staff to meet all plant operational needs for a wastewater plant of the City's size and complexity.

Thus, it is recommended that the City staff a minimum of four operations staff for current and near-term future plant needs. Based on the recommendations that will be made regarding needed plant maintenance in the short-term (such as weed abatement, re-condition pond embankments, sludge removal, etc.), the City's current planned staffing level of four staff is likely a minimum requirement to achieve the short-term needs in a reasonable time frame.

RECOMMENDATIONS

The following recommendations are near-term improvements and activities to be undertaken immediately and in the near future to achieve the desired WWTP capacity of 2.0 mgd, and to improve the overall performance of the City's WWTP.

Initial Treatment Units

The metering system for incoming flows should be checked as to accuracy. This flume may not be hydraulically sufficient to handle peak wastewater flows above 2.5 mgd, and thus this needs to be verified also.

The automatic bar screen as well as comminutors should be inspected to determine if there is a need for major overhaul to ensure effectiveness.

The overall ability of the existing headworks to handle peak hydraulic loading at 2.0 mgd ADWF capacity should be verified. It is recommended that the headworks be assessed in more detail, including development of a hydraulic profile for the treatment plant. Based on review of the chart recordings, a daily/diurnal peaking factor of at least 2.3 should be expected. We again note that according to Freitas+Freitas, the headworks was deemed adequate for the 2.0 mgd ADWF flow.

City operations staff suggests that addidtional focus be placed on potential addition of a fine screening device in the headworks channel, to enhance removal of organic material upstream of the primary clarifiers. We concur that this may further enhance WWTP performance and reduce organic loading to the plant. However, this report focused on immediate near-term improvements to bring the plant into compliance with WDRs and a 2.0 mgd rated capacity. It is recommended that this be considered at some future time, after the City has implemented other near-term improvements to provide the necessary treatment plant capacity. Further consideration of such equipment should occur after it is confirmed that the existing headworks channel and facilities are adequate for 2.0 mgd ADWF capacity.

The above items can be considered incidental O&M costs, and not a capital budget item.

Primary Clarifiers

Normal maintenance and repairs should ensure continuing satisfactory operation of the existing three clarifiers. No immediate actions are required other than routine servicing and maintenance. It is noted that at the current some 1 mgd incoming flow rate, the overflow rate of the clarifier operating in parallel is 210 gpd/sf/day and at 2 mgd, 420 gpd/sf/day. Similarly, detention time at 1 mgd is about 6.8 hours and at 2 mgd, 3.4 hours. These figures verify that the existing clarifiers have more than sufficient capacity for handling 2 mgd of incoming wastewater flows.

Solids Handling/Sludge Beds

It was noted earlier that Freitas+Freitas developed a sludge digester operation plan for City operation staff, in 2008. Plant staff should employ these operational procedures and monitor the effectiveness of implementing these recommendations, which should also be included in the City's overall O&M Manual.

Near-term improvements could consider automating aeration to the digesters, to maintain dissolved oxygen levels while minimizing blower run times to conserve energy, thus turning blowers off if not needed (in addition to when sludge is being decanted). This can be accomplished with the installation of dissolved oxygen probes in the digesters, and programming the blowers to operate based on a set range of measured dissolved oxygen levels.

The above item can be designed in-house by City staff, equipment purchased and installed. Some assistance may be needed with SCADA programming. Budgeted cost, $10,000.

Sludge Beds. The sludge beds should be cleared of weed accumulation, and the embankments touched up and re-graded where needed. As part of the overall sludge management plan, the sludge drying lagoons should be considered to be operated as drying beds, thus avoiding large discharge volumes of ponded sludge. Instead, sludge should be applied in small layers, quickly dried and removed. The City has suggested to consider employing mechanical dewatering such as a belt press. At this time, and given the land availability around the plant site (approximately 2 acres), mechanical dewatering would be expensive. This would include building enclosure, odor scrubbing, belt presses, piping and other required ancillary equipment. Final cake from a belt press would be in the range of 20% solids, and would still require further drying prior to landfilling (direct haul for composting would be acceptable, but payment for disposal is on a wet ton basis and would be costly, if directly hauled to a composting site). At this time, it is recommended the City improve the existing sludge drying area to better accommodate thin spreading and effective drying, and in the near future consider the design of conventional paved sludge drying beds. This is envisioned to cost $300,000. In addition to sludge drying area improvements, the City should evaluate drainage in and around this entire area, and make necessary improvements to make sure sludge drying/ponding is separated from any stormwater management.

O&M Recommendations. In order to gain a better understanding of solids digestion adequacy, a program of monitoring essential operating results should be established immediately. Such program should include monitoring sludge for %volatile solids, %non-volatile solids, % volatile solids reduction, and overall sludge production from the aerobic digesters on a dry weight basis (dry tons). This data can then be used for future planning for solids digestion and handling, and reviewing the adequacy of the existing sludge drying beds.

Long-term future considerations for plant expansion beyond 2 mgd, should include additional engineering studies and consideration of conversion to anaerobic digestion.

Oxidation Ponds

Near-term improvements should include the addition of a minimum of 90 HP of aeration to the three oxidation ponds. Whichever aeration system is ultimately chosen must be capable of providing the calculated required oxygen as stated earlier in this report and by the prior Freitas+Freitas report. All ponds should be de-sludged, and pond embankments abated of weeds and re-conditioned. Employ a program to continually maintain weed abatement programs, and also take measures to minimize the presence of burrowing animals in the area.

Oxidation Pond No. 1. Remove the dried sludge from the pond, abate weeds and re-condition the banks. At the City's option, provide design plans for deepening of the pond, and adding a pond lining system. The design plans will also address the sizing, placement and positioning of floating aerators on all three oxidation lagoons.

Oxidation Ponds 2 and 3. After the improvements at Pond 1 are complete, decommission one of these two remaining oxidation ponds, abate weeds and re-condition the pond. Similar to Pond No. 1, this pond could be deepened and possibly lined. After these improvements are complete, then the final oxidation pond can be improved in the same manner.

The above improvements can be completed in a single construction contract.

At this point, we recommend the City move forward immediately with the addition of aerators to the oxidation ponds, and monitor the effectiveness of the aeration addition. Should the City decide to further evaluate alternative aeration equipment, consideration should be given to capital cost investment, schedule and timing, ease of installation, O&M costs, safety, and other considerations, in light of the need to immediately upgrade the existing oxidation pond system. Lining and deepening of the ponds should be deferred until it is determined how the oxidation ponds are operating in conjunction with the supplemental aeration.

Estimated probable costs:

Design Plans and Specifications[a]	$10,000
Construction Mgt/Admin	$40,000
Construction	
Aerators, Electrical Service	$250,000
TOTAL:	$300,000

[a]*Update to 2012 Freitas+Freitas Design Plans, assumes floating aerator design is selected.*

Percolation Ponds

Current operation of the two infiltration ponds does not allow for a regular routine of drying and scarifying the bottom. This results in very little percolation/infiltration through the pond bottoms.

It is recommended that each infiltration pond be taken out of service individually, dried, disked. If underlying soils do not percolate effectively, scarify, scrape and remove the bottom layer of pond bottom and replace with clean granular material. The depth of removal of soil, if required, will need to be determined by field observations and tests. Abate weeds and re-condition the pond embankments. Once placed back in service, these percolation ponds should be alternated in use, dried and disked on periodic basis. However, should the City decide to operate these percolation ponds while maintaining water depth, there may be some added benefit of additional settling prior to discharge to the effluent spray disposal fields. The Solarbees may be placed back in service if desired; they are providing some circulation of water, but no effective aeration of the ponds.

The above items can be considered incidental O&M costs, and not a capital budget item. However, depending on whether some topsoil needs to be replaced or not, may add some capital cost to this project.

EFFLUENT SPRAY DISPOSAL FIELD

As noted earlier, the City utilizes a signficant amount of energy/power to spray dispose of effluent. The City should consider the installation of permanent distribution piping to convey effluent to the 26 acre disposal field. We understand at this time, that the City is moving forward with converting water application to the disposal fields by gravity (flood irrigation), thus eliminating the need for the effluent pump station. The field can be “partitioned” into individual percolation beds, and flood irrigated in small increments, allowed to percolate, dry and then disk, thus rotating use of such beds. A cursory review in Google Earth indicates that distribution fully by gravity may not be viable; however, the City should confirm the ability to disseminate flow by gravity to all portions of the disposal area. If not, the City can use low-head pumps to pump effluent to standpipes for distribution by gravity. Either way, it is likely the City can save a significant amount on energy costs each month, not to mention the labor savings in not having to manually move irrigation piping to portions of the spray disposal field.

The capital cost of improvements for site area grading, and installation of irrigation piping is estimated at $200,000 by City staff, for improvements to the disposal area.

O&M Considerations and Development of Standard Operating Procedures

The City should immediately begin updating the current O&M manual to reflect all plant operations, and develop standard operating procedures. Budgeting for this task is difficult; however, it is estimated that $15,000 to $20,000 should be budgeted if performed by consultants; however, we understand that the City will proceed with this task using in-house by City staff.

Regional Board Updates and Report of Waste Discharge

The City should prepare a schedule for implementation of these improvements, and perform the following:

1. Write a letter to the Regional Board informing the agency of the above recommendations, and a time frame to complete such Work.
2. Continue with quarterly updates to the Regional Board, and update the Board as necessary following further studies, such as the Effluent Disposal Study.

After completion of this Work, the Regional Board may request the City to submit an updated Report of Waste Discharge, or the Board could opt to issue a letter of concurrence (similar to their prior letter conditionally approving the 2.0 mgd capacity). It is difficult to determine the Board direction at this time.

SUMMARY

A summary of near-term improvements and action items are included in Table 5.

Table 5. Summary of Near-term Improvements at City of Greenfield WWTP

Project	Comments	Capital or Outside Costs, $
Calibrate Influent Flows and Parshall Flume	Considered O&M Cost	
Inspect automatic bar screen and comminutors	Considered O&M Cost	
Evaluate Plant Hydraulics/Hydraulic Profile, Headworks		$5,000
Employ Freitas+Freitas Recommended Sludge Digester Operations	Considered O&M Cost	
Employ Digested Sludge Monitoring Program	Considered O&M Cost	
Conventional Sludge Beds (optional)		$300,000
Add Aerators to Oxidation Ponds	Defer lining and deepening of ponds at this time	$300,000

Project	**Comments**	**Capital or Outside Costs, $**
Re-Condition Percolation Ponds	Considered O&M Cost (unless import material req'd)	
Effluent Disposal Area Construction Improvements (26 acre area)	Estimated Budget	$200,000
Prepare O&M Manual Updates and SOPs	Considered O&M Cost (in-house)	
Updated Report of Waste Discharge	Unknown at this time	$10,000
TOTAL ESTIMATED CAPITAL OUTLAY TO ACHIEVE 2.0 MGD CAPACITY		$515,000 (excluding addition of new sludge beds)

REFERENCES

1. Price Consulting Firm. City of Greenfield 2010 Urban Water Management Plan, February 21, 2013.
2. Terra Engineering, Inc. and Freitas + Freitas Engineering and Planning. City of Greenfield 2008 Update of the Wastewater System, Capital Improvement Plan and Capacity Charge Study, July 2008.
3. RM Associates. Report of Waste Discharge, 2001.
4. Terra Engineering, Inc. and Freitas + Freitas Engineering and Planning. Wastewater Disposal Report for City of Greenfield Wastewater Treatment Plant, September 2003.
5. Terra Engineering, Inc. and Freitas + Freitas Engineering and Planning. Expansion Report for City of Greenfield Wastewater Treatment Plant, September 2003.
6. Terra Engineering, Inc. and Freitas + Freitas Engineering and Planning. Wastewater Treatment Plant Expansion (Phase 3, Completed 2008) Operation & Maintenance Manual, 2008.
7. Metcalf & Eddy. Wastewater Engineering Treatment and Reuse, Fourth Edition, 2003.

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APPENDICES – On Compact Disk in pocket

Appendix A – City of Greenfield Waste Discharge Requirements

Appendix B – Expansion Report for City of Greenfield Wastewater Treatment Plant, September 2003, Freitas+Freitas

Appendix C – Additional Information Requested by RWQCB, June 11, 2004 Letter, Freitas+Freitas

Appendix D – RWQCB Letter Dated March 11, 2004, to Mr. John Alves

Appendix E – Wastewwater Disposal Report for City of Greenfield Wastewater Treatment Plant, September 2003, Freitas+Freitas

Appendix F – RWQCB Letter Dated August 30, 2004, to Mr. John Alves (Permitted Capacity Letter)

11: Appendix C - Standard Operating Procedure (SOP) for WWTP Staffing

Wastewater Treatment Plant Standard Operating Procedure City of Greenfield		Document No: W-SOP-01
Title: WASTE WATER TREATMENT PLANT STAFFING		Revision: 0
Approved by: Mic Steinmann *Community Development Director* City of Greenfield	Prepared by: Arturo Felix *Utility Manager* City of Greenfield	Page: 1 of 6 Effective Date: 11/13/2014

1. **Purpose**

   The purpose of this procedure is to ensure that the City of Greenfield (City or Greenfield) remains in compliance with Title 23 California Code of Regulations (CCR) Sections 3670 to 3719 which governs the classification of wastewater treatment plants (WWTP) and operator certification requirements.

   This SOP is applicable to the WWTP only.

2. **Definitions: 23 CCR Section 3671**

Chief Plant Operator (CPO)	The operator responsible for the overall operation of a wastewater treatment plant including compliance with effluent limitations established in the wastewater treatment plant's waste discharge requirements and ensuring that operators-in-training are supervised directly as required by section 3682.
Contract Operator	A person who enters into a contract with an owner to operate one or more wastewater treatment plants.
Designated Operator-In-Charge	An operator appointed by the chief plant operator pursuant to section 3680(b) to be responsible for the overall operation of a wastewater treatment plant, including compliance with the applicable waste discharge requirements, when the chief plant operator is unable to carry out the responsibilities of the position of "chief plant operator" as defined in this section. The designated operator-in-charge shall report to the chief plant operator.
Direct Supervision	The supervising operator shall oversee and inspect the work performed by an operator-in-training and provide training to ensure the safe and proper execution of the operator-in-training's duties. Direct supervision shall be carried out by an operator at the same or higher grade level as the operator-in-training. The supervising operator shall be present at the wastewater treatment plant or otherwise available to consult with, and provide assistance to, the operator-in-training in order to ensure the safe and proper execution of the operator-in-training's duties.
Lone Operator	An operator, at a grade level lower than the designated operator-in-charge, approved by the Office of Operator Certification pursuant to section 3681 to work alone at a wastewater treatment plant. An operator-in-training shall not be a "lone operator."
Maintenance	Those activities which are limited to the day-to-day servicing, adjustment or regulation of equipment which are performed by an operator and are necessary to maintain reliable operation of major treatment processes.
Operates	Actions or decisions to control the performance or outcome of one or more wastewater treatment processes. The term also includes the supervision of other operators acting or making decisions to control the performance or outcome of one or more wastewater treatment processes.

Operator	A person who operates a wastewater treatment plant and who possesses a valid, unexpired operator certificate. The term “operator” includes a person who possesses a valid, unexpired operator certificate, but who is not currently employed in a position for which an operator certificate is required.
Operator-in-training (OIT)	A person who has been issued an operator-in-training certificate by the State Water Board and who is acquiring qualifying experience at a wastewater treatment plant under the direct supervision of an operator at a higher grade level as the operator-in-training.
Wastewater treatment plant (WWTP)	A facility owned by a state, local, or federal agency and used in the treatment or reclamation of sewage and industrial wastes.

3. Health and Safety Warnings

1. This section is not applicable to this SOP.

4. Cautions

1. None.

5. City and Personnel Responsibilities and Qualifications

1. The Greenfield WWTP is a Class II Modified Treatment Pond treatment facility.
   a. The minimum Grade Level of Chief Plant Operator for a Class II facility under 23 CCR 3680(a) is a Grade II.
   b. The minimum Grade Level of the Designated Operator-in-Charge for a Class II facility under 23 CCR 3680(a) is a Grade I.

2. City of Greenfield Community Development Director Responsibilities
   a. If the person designated as the CPO changes, the City shall notify the State Water Board Office of Operator Certification in writing within 30 days and shall provide a signed statement from the new CPO acknowledging and accepting the responsibilities of the position of “chief plant operator” as defined in section 3671 above in Definitions.

   b. The City shall notify the Office of Operator Certification in writing within 30 days of any final disciplinary action taken by the owner against an operator, provisional operator, operator-in-training, or contract operator. Disciplinary action includes reprimanding or placing on probation, suspending, demoting, or discharging an operator, provisional operator, operator-in-training, or contract operator for performing, or allowing or causing another to perform, any act in violation of this chapter. The notice shall include the name of the operator, provisional operator, operator-in-training, or contract operator, the specific violations, and the disciplinary action taken. The notice also shall include the operator's certificate number or the contract operator's registration number.

c. The City shall ensure that all operator and OIT valid and original (not copies) certificates are posted in an area accessible to the public either at the WWTP or at an area accessible to the public at the City headquarters if the WWTP does not have an area accessible to the public.

3. Chief Plant Operator (CPO)
   a. The CPO is responsible for setting the WWTP staffing schedule , the schedule is set weekly
      i. The WWTP is staffed Monday to Friday from 8:00 AM to 3:00 PM.
      ii. The WWTP is staffed Saturday and Sunday from 8:00 AM to12:00 pm AM.
   b. The CPO is responsible for all WWTP operational decisions and the maintenance schedule.
   c. The CPO is responsible for implementation of and compliance with the Central Coast Regional Water Quality Control Board (RWQCB) Waste Discharge Requirements and Monitoring and Reporting Program No. R3-2002-0062 (2002 WDR/MRP).

4. Public Works Service Worker I
   a. Public Works Service Workers (PWSW) typically are also Utility Maintenance workers at the Greenfield WWTP and support maintenance of the sewage collection system. PWSWs are required to obtain and possess a valid Grade I Operator certificate within a time frame determined by the City or maintain a valid Operator-in-Training (OIT) certificate.
   b. Responsible for carrying out the directions given by the CPO in the operation and maintenance of the WWTP.
   c. Responsible for supervising OITs if a Grade 1 Operator.
   d. A Public Works Service Worker I may also be an OIT.

6. Equipment and Supplies

1. Computer
2. Cell Phone
3. Truck
4. Tools

7. Procedure

WWTP Staffing

1. The City of Greenfield has historically maintained a staff of two (2) Full Time Employees (FTE) at the WWTP. Public Works Service Workers are required to have

(or acquire within a specified time frame) a valid operator certificate as an OIT or Grade I operator; they are also responsible for sewer collection system operation and maintenance.

i. OIT's are under the direct supervisor of a Grade I licensed operator.

WWTP Staff Schedule

1. Chief Plant Operator
   a. Updates the City Manager and Community Development Director daily on the status of WWTP staffing.
   b. Reviews the WWTP Daily Schedule for flow, etc.
   c. Plans the work schedule on a monthly basis on a Calendar in the WWTP office; and
   d. Directs the Public Works Service Workers.

2. Public Works Service Worker I
   a. Required to login daily to the WWTP Log Book. Routine and non-routine activities are required to be written into the WWTP Log Book. The Wastewater Treatment Daily Schedule Form is also required to be completed on a daily basis.
   b. Operational decisions that are non-routine are to be communicated immediately by phone to the CPO.
   c. May act as the Designated Operator-in-Charge, and must possess a Grade 1 Operator Certificate if directed to do so, when the CPO is unavailable
   d. Performs sampling required by the WDR/MRP.

3. Public Works Office Specialist
   a. Assists the CPO in the generation of the quarterly and annual WWTP influent and effluent monitoring reports required by the 2002 WDR/MRP.

8. Data and Records Management

1. All required records shall be maintained for a minimum of five (5) years and shall be made available for review by the State Water Resources Control Board (SWRCB) and/or RWQCB during an onsite inspection or through an information request.
2. Records documenting compliance with all provisions of the WDR and MRP including any required records generated by contractors such as the State Certified Laboratory used to generate monitoring results.

9. Quality Control and Quality Assurance

1. Not Applicable to this SOP

10. References

1. USEPA Guidance for Preparing Standard Operating Procedures

2. Title 23 California Code of Regulations Chapter 26 Wastewater Treatment Plant Classification, Operator Certification, and Contractor Operator Registration
3. City of Greenfield Position Description Manual
4. MRP for the WDR: Order No. R3-2002-0062
5. SWRCB Enrollee's Guide to the SSO Database Sanitary Sewer Overflow Reduction Program, Last Updated August 2013

11. Attachments

1. City of Greenfield Wastewater Treatment Plan Organization Chart

Public Works Department
Wastewater Division
Organization Chart

MIC STEINMANN
SUBSTAINABILITY RESOURSE
DIRECTOR
LRO
(LEGAL RESPONSIBLE OFFICIAL)
(831)674-5591

CARMEN LORENZANA
PUBLIC WORKS DEPARTMENT
OFFICE SPECIALIST II
(831)674-2635

ARTURO FELIX
UTILITY SYSTEM
ASSISTANT SUPERINTENDENT
GRADE 2 # II-10611
(831)580-7989

IVAN BARRON
MAINTENANCE WORKER
GRADE 1 #I-28450
(831)674-2635

ALEJANDRO ALVAREZ
WW OIT
MAINTENANCE WORKER
(831)674-2635

* IVAN & ALEJANDRO CROSS OVER TO WATER DIVISION AS NEED