City of Galt

WASTEWATER COLLECTION SYSTEM MASTER PLAN

FINAL

MAY 2010

carollo
Engineers...Working Wonders With Water™

2500 VENTURE OAKS WAY • SUITE 320 • SACRAMENTO, CALIFORNIA 95833 • (916) 565-4888 • FAX (916) 565-4880
pw://Carollo/Documents/Client/CA/Galt/8100A00/Deliverables/Sewer/TOC (FinalA)

City of Galt

WASTEWATER COLLECTION SYSTEM MASTER PLAN

TABLE OF CONTENTS

Page No.

EXECUTIVE SUMMARY

ES.1 INTRODUCTION 1
ES.2 STUDY AREA 1
ES.3 EXISTING AND FUTURE SERVICE AREA 1
ES.4 HISTORICAL AND FUTURE POPULATION 3
ES.5 HISTORICAL AND FUTURE SEWER FLOWS 3
ES.6 CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS 5
ES.6.1 Existing Versus Future Improvement 6
ES.6.2 Proposed Existing Sewer Trunk Improvements 6
ES.6.3 Proposed Future System Improvements 9
ES.6.4 Existing and Build-out Lift Station Improvements 9
ES.7 CAPITAL PROJECT PRIORITIZATION 10
ES.7.1 Phase 1 Projects (2009-2015) 10
ES.7.2 Phase 2 Projects (2016-2020) 11
ES.7.3 Phase 3 and 4 Projects (2021-2025 and 2026-2030) 11
ES.8 CAPITAL PROJECT COSTS 12
ES.9 COST OF SERVICE 12
ES.9.1 Existing Users Fee for Service 13
ES.9.2 Future User Development Impact Fees 13

CHAPTER 1

1.1 INTRODUCTION 1-1
1.2 BACKGROUND 1-1
1.3 SEWER SERVICE AREA 1-1
1.4 SCOPE AND AUTHORIZATION 1-1
1.5 REPORT ORGANIZATION 1-4
1.6 ACKNOWLEDGMENTS 1-4
1.7 ABBREVIATIONS AND DEFINITIONS 1-5
1.8 REFERENCE MATERIAL 1-7

CHAPTER 2

2.1 STUDY AREA 2-1
2.2 PLANNING PERIOD 2-1
2.3 CLIMATE 2-1
2.4 100-YEAR FLOODPLAIN 2-3
2.5 LAND USE 2-3
2.5.1 Wastewater Service Area by Land Use 2-7
2.6 HISTORICAL AND FUTURE POPULATION 2-7

CHAPTER 3

3.1 GRAVITY SEWERS 3-1
3.1.1 MANNING COEFFICIENT (N) 3-1
3.1.2 FLOW DEPTH CRITERIA (D/D) 3-1
3.1.3 DESIGN VELOCITIES AND MINIMUM SLOPES 3-2
3.1.4 CHANGES IN PIPE SIZE 3-3
3.1.5 LIFT STATIONS, AND FORCE MAINS 3-3

CHAPTER 4

4.1 FLOW MONITORING PROGRAM 4-1
4.2 FLOW MONITORING RESULTS 4-1
4.2.1 Average Day Flow Data 4-1
4.2.2 Rainfall Data 4-3
4.2.3 Wet Weather Flow Data 4-3
4.2.4 Design Storm Data 4-4
4.3 WASTEWATER FLOW COMPONENTS 4-4
4.3.1 Base Wastewater Flow 4-6
4.3.2 Groundwater Infiltration 4-6
4.3.3 Average Dry Weather Flow 4-6
4.3.4 Peak Dry Weather Flow 4-7
4.3.5 Maximum Day Wet Weather Flow 4-7
4.3.6 Peak Wet Weather Flow (Design Flow) 4-7
4.3.7 Inflow and Infiltration 4-9
4.4 WASTEWATER TREATMENT PLANT FLOWS 4-9
4.5 WASTEWATER FLOW PROJECTIONS 4-10
4.5.1 Existing Wastewater Flow Coefficients 4-10
4.5.2 Existing and Future (Build-out) Average Day Flow 4-11
4.5.3 Growth Horizon 4-14
4.6 DESIGN FLOWS 4-15
4.7 PLANNING CRITERIA SUMMARY 4-16

CHAPTER 5

5.1 WASTEWATER SERVICE AREA OVERVIEW 5-1
5.2 MODELED SEWER SYSTEM 5-3
5.2.1 Interceptors and Major Trunk Sewers 5-3
5.2.2 Lift Stations and Forcemains 5-5
5.3 SEWER SYSTEM HYDRAULIC MODEL 5-7
5.3.1 Quantifying Wastewater Flow 5-7
5.3.2 Sewer Tributary Areas (Existing and Future) 5-8
5.3.3 Model Calibration 5-11
5.3.4 Wet Weather Flow Calibration 5-11

CHAPTER 6

6.1 CAPACITY EVALUATION 6-1
6.2 COLLECTION SYSTEM IMPROVEMENTS 6-2

6.2.1 Sewer Main Rehabilitation and Replacement....6-2
6.2.2 Existing Versus Future Improvement....6-2
6.2.3 Proposed Existing System Improvements....6-6
6.2.4 Available Capacity in Existing Sewers....6-7
6.2.5 New Trunk Sewers, Interceptors, and Lift Stations....6-9
6.2.6 Existing and Build-out Lift Station Improvements....6-11
6.3 PROJECT PRIORITIZATION....6-13
6.3.1 Phase 1 Projects (2009-2015)....6-14
6.3.2 Phase 2 Projects (2016-2020)....6-14
6.3.3 Phase 3 and 4 Projects (2021-2025 and 2026-2030)....6-15
6.4 ADDITIONAL RECOMMENDATIONS....6-15
6.4.1 Sewer Main Replacement, Rehabilitation, and Maintenance....6-15
6.4.2 Lift Station Replacement or Rehabilitation....6-16
6.5 RURAL RESIDENTIAL DEVELOPMENT....6-16
6.5.1 Rural Residential Wastewater Flow....6-16
6.5.2 Rural Residential Collection System....6-17

CHAPTER 7

7.1 CAPITAL IMPROVEMENT PROJECT COSTS....7-1
7.2 COST ESTIMATING ACCURACY....7-1
7.3 CONSTRUCTION UNIT COSTS....7-1
7.3.1 Sewer Trunk Unit Costs....7-2
7.3.2 Lift and Pump Station Costs....7-3
7.3.3 Land Acquisition Costs....7-3
7.4 PROJECT COSTS AND CONTINGENCIES....7-3
7.4.1 Baseline Construction Cost....7-3
7.4.2 Estimated Construction Cost....7-3
7.4.3 Capital Improvement Cost....7-5
7.5 CAPITAL IMPROVEMENT IMPLEMENTATION....7-6
7.6 EXISTING VERSUS FUTURE USER COST SHARE....7-6
7.7 COST OF SERVICE....7-6
7.7.1 Existing Users Fee for Service....7-9
7.7.2 Future Users Development Impact Fees....7-11
7.8 FUTURE USER BENEFIT ZONES....7-12
7.8.1 Common Use Facilities....7-12
7.8.2 Zone 1: East of Highway 99....7-14
7.8.3 Zone 2: Industrial....7-14
7.8.4 Zone 3: North of Twin Cities Road....7-14
7.8.5 Future Infill Development....7-14

APPENDIX A Description of Different Land Uses (General Plan Excerpts)
APPENDIX B Diurnal Curves
APPENDIX C DWF Calibration Plots
APPENDIX D WWF Calibration Plots
APPENDIX E Project Memorandum (Location Summaries of Benefits/Drawbacks)
APPENDIX F Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study

LIST OF TABLES

Table ES.1 Existing and Projected Year 2030 Population 3
Table ES.2 Wastewater Flow Summary 4
Table ES.3 Wastewater Flow Projections 5
Table ES.4 Existing Versus Future User Cost Share 12
Table ES.5 Existing Versus Future Cost by Facility Type 12
Table 2.1 Study Area Land Use Designations 2-6
Table 2.2 Historical and Projected Population 2-8
Table 3.1 d/D Ratios for Design Flow Conditions 3-2
Table 3.2 Minimum Slope and Maximum Flow 3-3
Table 4.1 Average Day and Average Peak Dry Weather Flow 4-3
Table 4.2 I/I Results Summary 4-4
Table 4.3 Historical Monthly WWTP Influent Flows 4-8
Table 4.4 Existing Wastewater Flow Coefficients 4-12
Table 4.5 General Plan Build-out Flow Projections 4-13
Table 4.6 Average Day Flow Summary 4-14
Table 4.7 Wastewater Flow Projections 4-15
Table 4.8 Peaking Factor and Peak I/I Summary 4-16
Table 4.9 Existing and Projected Wastewater Flows 4-16
Table 4.10 Planning Criteria Summary 4-17
Table 5.1 Wastewater Collection System Pipeline Summary 5-1
Table 5.2 Modeled Sewer System Pipeline Summary 5-3
Table 5.3 Lift Station Design Data 5-6
Table 5.4 Average Day Flow Calibration Summary 5-14
Table 6.1 Proposed Collection System Improvements (2 pages) 6-5
Table 6.2 Lift Station Evaluation Table 6-12
Table 7.1 Pipeline Construction Unit Costs 7-2
Table 7.2 Capital Improvement Projects (2 pages) 7-7
Table 7.3 Existing Versus Future User Cost Share 7-9
Table 7.4 Existing Versus Future Cost by Facility Type 7-9

LIST OF FIGURES

Figure ES.1 Study Area 2
Figure ES.2 Proposed Wastewater Collection System Improvements (2 pages) 7
Figure 1.1 Location Map 1-2
Figure 1.2 Current Wastewater Collection System Service Area 1-3
Figure 2.1 Study Area 2-2
Figure 2.2 100-Year Floodplain 2-4
Figure 2.3 General Plan Land Use 2-5
Figure 2.4 Historical and Projected Population 2-9
Figure 4.1 Flow Meter Locations 4-2
Figure 4.2 10-year, 24-hour Design Storm Distribution 4-5
Figure 5.1 Existing Wastewater Collection System 5-2
Figure 5.2 Modeled Wastewater Collection System 5-4
Figure 5.3 Existing Sewer System Tributaries 5-9
Figure 5.4 Future System Sewer Tributaries 5-10
Figure 5.5 Flow Meter 4 Diurnal Pattern 5-12

Figure 5.6 Flow Meter 4 Average Day Flow Calibration 5-13
Figure 5.7 Flow Meter 5 Wet Weather Calibration 5-15
Figure 6.1 Proposed Collection System Improvements (2 pages) 6-3
Figure 6.2 B Street Trunk Sewer Improvements.....6-8
Figure 7.1 Lift Station Cost Versus Capacity Curve.....7-4
Figure 7.2 Common Use Facilities.....7-13

WASTEWATER COLLECTION SYSTEM MASTER PLAN

ES.1 INTRODUCTION

The City of Galt (City) is located along State Highway 99 in northern California's Central Valley, between the cities of Sacramento and Stockton, and near the Delta Recreation Area[1].

The City owns, maintains and operates gravity sewer pipelines and forcemains, sewer lift stations and pump stations, and the wastewater treatment plant (WWTP). The City's wastewater infrastructure includes over 80 miles of pipelines ranging in diameter from 6- to 24-inches, twelve lift/pump stations, and the WWTP located west of Highway 99 and north of Twin Cities Road. The City collects wastewater from residential, commercial, institutional, and industrial customers within the service area.

ES.2 STUDY AREA

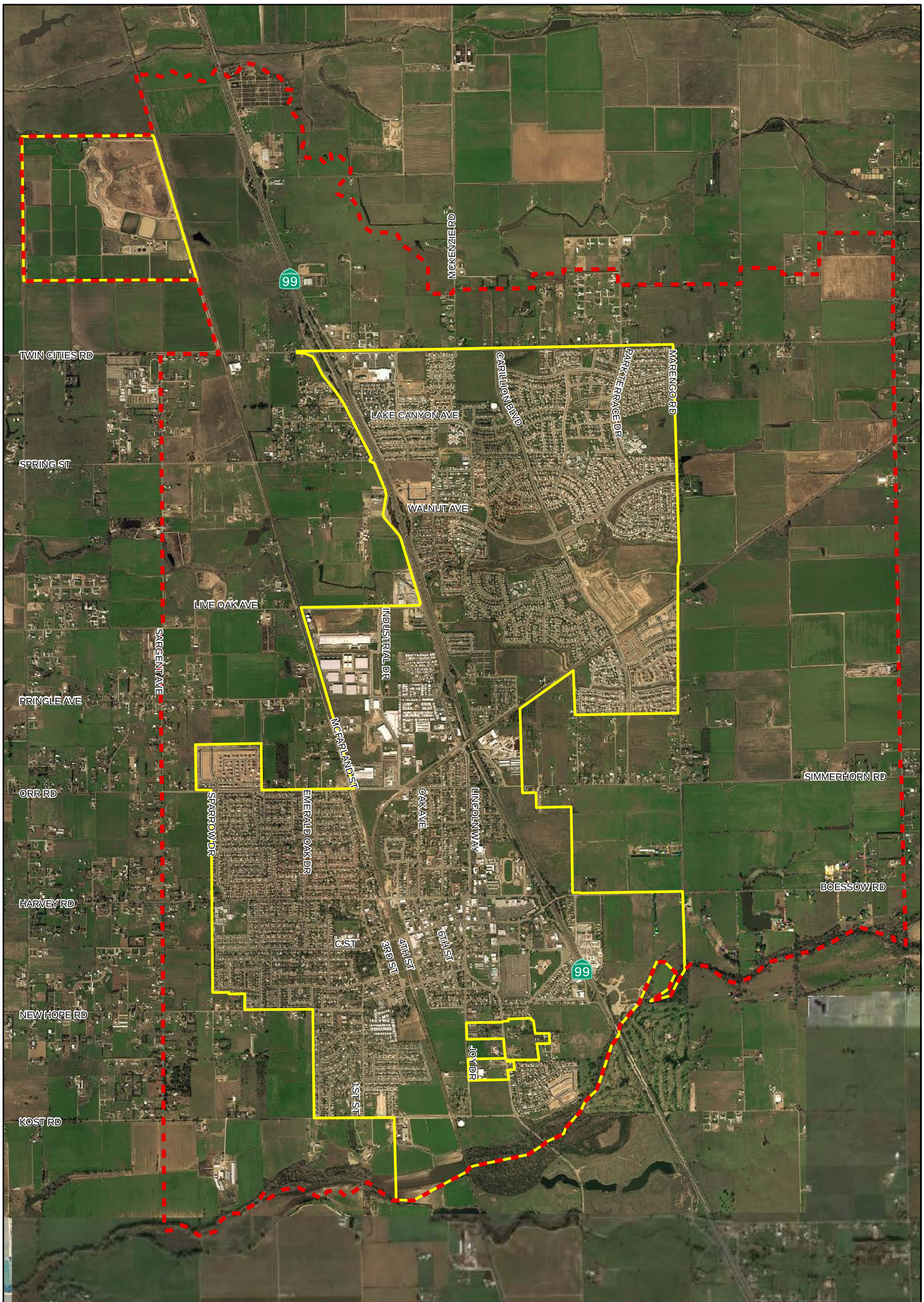
The City's 2030 General Plan Public Review Draft (General Plan) planning boundary is the study area boundary for this sewer system master plan (Master Plan). The Master Plan study boundary and the General Plan boundary are synonymous and will be used interchangeably throughout this report. The General Plan planning boundary extends beyond the current sewer system service area and is approximately 8,817 acres (13.8 square miles). The Master Plan contains a forecast of sewer system improvements in a large study area beyond the City's limits. Figure ES.1 shows the study area boundary and the current City limits.

Evaluating infrastructure needs beyond the current City limits is important because: there are pending conceptual development plans that are beyond the City limits; and historical cycles of rapid growth in the Sacramento metropolitan area indicates that significant development into the study area could occur within a short planning period.

ES.3 EXISTING AND FUTURE SERVICE AREA

The land use criteria used in this Master Plan (residential, commercial, etc.) are consistent with the 2030 General Plan Land Use. The type of land use in an area will affect the volume and character of the sewer flows. Adequately estimating the sewer generation from various land use types is important in sizing and maintaining sewer system facilities.

[1] City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al.



Legend

General Plan Boundary

City Limits

Feet

0 2,500 5,000

Figure ES.1
Study Area
Wastewater Collection System Master Plan
City of Galt

The City currently provides sewer service to approximately 3,763 acres (includes developed and undeveloped land) or 5.9 square miles. The largest land use category is residential (rural, low density, medium density, medium-high density, and high density), which accounts for approximately 1,758 acres, or approximately 47 percent of the total current City limit acreage. Commercial, office professional, and light industrial make up approximately 566 acres, or 15 percent of the total. Other land uses such as public/quasi-public, parks, streets, and open space account for 1,440 acres, or 38 percent of the total service area.

At build-out of the General Plan boundary, the City will serve approximately 8,817 acres. Build-out is defined as complete development of all lands outside the 100-year floodplain. There are approximately 8,059 acres within the study area that are outside the 100-year floodplain. Residential will continue to represent the largest land use category in the City and will make up approximately 49 percent of the total acreage.

ES.4 HISTORICAL AND FUTURE POPULATION

In 1990, the City's population began to grow rapidly and that growth continued through year 2000. From 1990 through 2007, the population grew from approximately 8,800 to 23,500. Over these 17 years, the City grew at an annual rate of about six percent.

The General Plan forecasts that the City's population will grow at an annual rate of 3.4 percent from 2002 to 2025, and will reach a 2030 population of 50,094 people. Table ES.1 summarizes the existing and projected year 2030 population.

Table ES.1 Existing and Projected Year 2030 Population
Wastewater Collection System Master Plan
City of Galt

Year	Population
2007	23,470
2030	50,094

Note:
1. Source: City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al. and City of Galt General Plan, Policy Document, Public Review Draft, July 2008, Mintier & Associates et al.

ES.5 HISTORICAL AND FUTURE SEWER FLOWS

The per capita wastewater generation rate is equal to the City's average daily flow (ADF) divided by its population. Between 2004 and 2008, the average per capita wastewater generation ranged between 89 gallons per capita per day (gpcd) (2008) to 97 gpcd (2007), and averaged 92 gpcd. The highest per capita year was 2007.

The ADF is the total annual sewer flow divided by number of days in the year. For example, in 2006, the ADF was approximately 2.17 million gallons per day (mgd), and in 2007, the ADF increased to approximately 2.3 mgd. The average dry weather flow (ADWF) is the average daily flow occurring during the dry weather months (June to September for this report). In 2007, the ADWF was approximately 2.3 mgd.

The Peak Wet Weather Flow (PWWF), or Design Flow, is the highest sewer flow rate during a 1-hour period of the year. In general, the PWWF is typically 2.0 to 3.0 times greater than the ADF in cities throughout Central California. PWWF for this master plan was developed by reviewing data collected by the temporary flow monitoring program, historical WWTP plant records, and rainfall data for the Galt area. Based on this analysis the existing PWWF for the City is 7.01 mgd. This results in a peaking factor of 3.0, which is on the higher side of the typical range.

Developing an accurate estimate of the sewer flows is an important step in determining the size of collection system facilities, for both existing conditions and future developments. The future ADF projections were developed based on the land use projections as described in the City's General Plan. Per City direction, the demand projections provided in this Master Plan assume that undeveloped areas within the 100-year floodplain will not be developed in the future. However, floodplain areas within the City limits that are currently developed will remain developed.

A summary of the existing and future ADF is presented in Table ES.2. In addition to the projected average flows, Table ES.2 includes estimates for the Design Flows and peaking factors through build-out of the 2030 General Plan boundary. Based on these projections, it is anticipated that the City's build-out ADF, and design flow will approach 5.60 mgd, and 14.45 mgd respectively. As the City develops, it is anticipated that the Design Flow to ADF peaking factor will decrease from 3.0 to 2.6. The decrease in peaking factor results from new development and improved sewer construction methods, which result in a decrease in inflow and infiltration.

Table ES.2 Wastewater Flow Summary
Wastewater Collection System Master Plan
City of Galt

	Average Day Flow[2] (mgd)	Design Flow (mgd)	Peaking Factor
Existing[1]	2.30	7.01	3.0
General Plan[2] Boundary	5.60	14.45	2.6

Notes:
1. Based on year 2007 flows.
2. Based on land use and acreage from the City's General Plan and build-out of all land within the General Plan 2030 boundary, excluding undeveloped land within the 100-year floodplain.

The discussion up to this point has focused on the flow projection when the General Plan boundary is fully built out. This Master Plan assumes that the study area will be completely built out by year 2030, with the exception of undeveloped areas within the 100-year floodplain boundary, which are assumed to be undeveloped through build-out. Based on this assumption and using a rate of increase consistent with the General Plan population increase, Table ES.3 provides wastewater flow projections through 2030.

Table ES.3 Wastewater Flow Projections
Wastewater Collection System Master Plan
City of Galt

Planning Year	Average Day Flow [1] (mgd)
2007	2.30
2010	2.70
2015	3.50
2020	4.16
2025	4.90
2030	5.60

Notes:
1. Assumes build-out of the study area by year 2030, except for undeveloped areas located within the 100-year floodplain.

ES.6 CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS

The capacity analysis entailed identifying areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey design flows. Sewers that lack sufficient capacity to convey design flows could produce backwater effects in the collection system and potentially cause sanitary sewer overflows (SSOs).

In general, the existing wastewater collection system has sufficient capacity to convey existing design flows. In some locations, such as the City's downtown, existing infiltration and inflow (I/I) cause sudden increases in wastewater flow during a storm event. The storm water inflow could cause a few sewers to surcharge and potentially overflow during a large storm.

The proposed improvements that will serve future users are sized for build-out conditions. As the City continues to grow beyond its current limits, it is recommended that the pipeline diameters and pump station capacities proposed in this Master Plan be constructed so that the facilities have sufficient capacity for build-out conditions. Building a smaller interim project with the plans of upsizing in the future to account for further growth is not recommended due to the extended useful life of the improvements proposed herein; in this

Master Plan, the proposed pipe diameter represents the ultimate diameter for build-out conditions.

Figure ES.2 illustrates the proposed sewer improvements required to correct existing deficiencies and to serve future users. The proposed pipeline diameter is also shown on the figures. Figure ES.2 shows the proposed improvements in different categories (colors). The different colors identify the implementation timeframe of the improvements and differentiate between near-term and long-term improvements.

A detailed inventory of the proposed improvements illustrated in Figure ES.2 is included in Table 7.2 of this report.

ES.6.1 Existing Versus Future Improvement

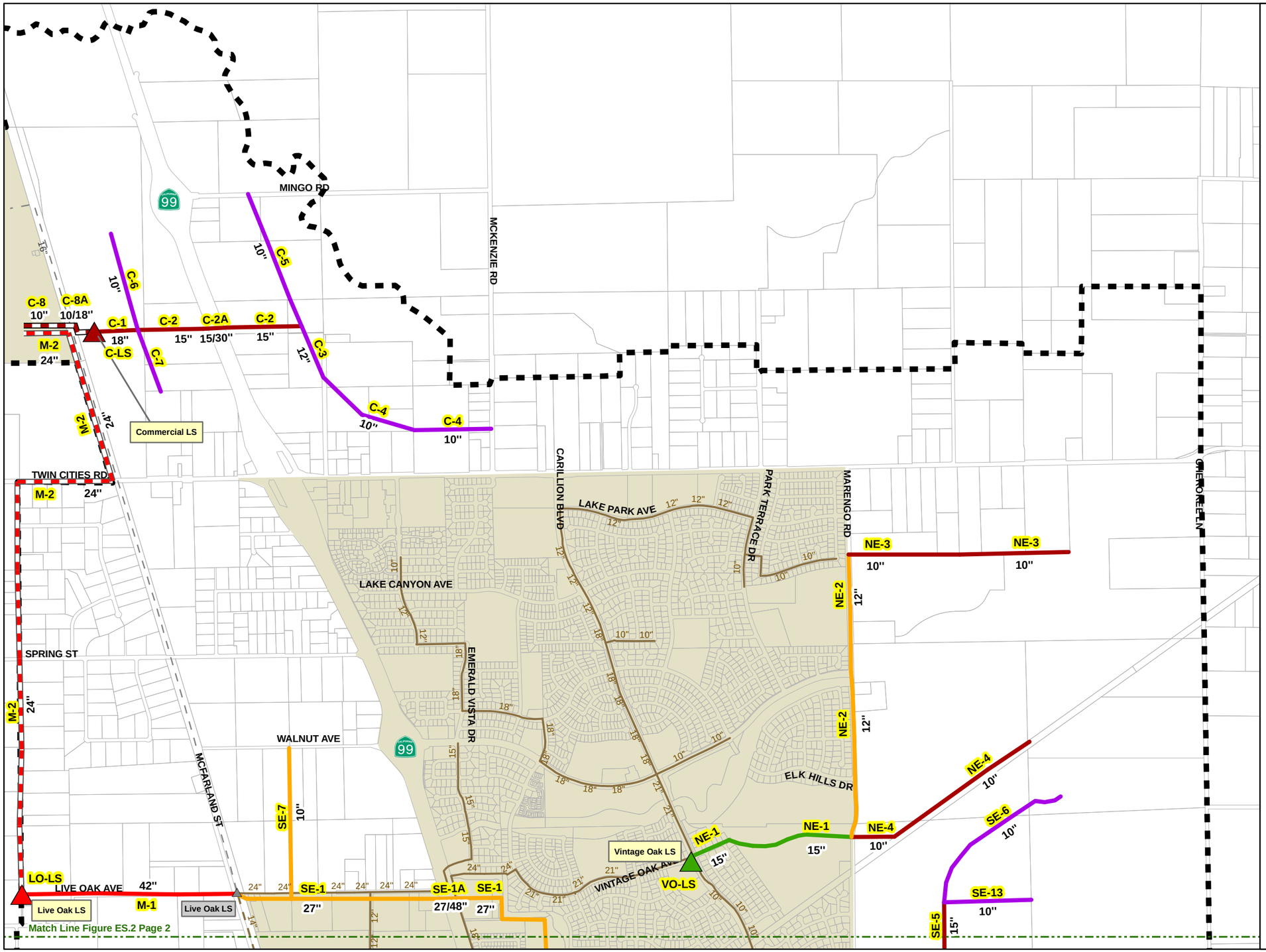
An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g. pipeline upgrades required to prevent severe surcharging during the design wet weather event) for existing users. If a project was proposed to correct an existing deficiency, then existing users were assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs.

The vast majority of the Master Plan improvements will serve future users, even when an improvement calls for the upgrade of an existing facility. In these cases, an existing sewer or lift station may have sufficient capacity to convey current design flows, but as growth continues and more users are added to the system, the increased flow results in capacity deficiencies. These are labeled future improvements. Future users were assigned 100 percent of the future project's benefit and 100 percent of the costs. More information on the breakdown in cost split between existing and future users and whether a proposed improvement is intended to correct an existing deficiency, to serve a future user, or both is provided in Chapter 6 and Chapter 7.

ES.6.2 Proposed Existing Sewer Trunk Improvements

For the majority of the City, the existing wastewater collection system contains sufficient capacity to convey peak dry weather flows (PDWF) and design flows without exceeding the capacity criteria discussed in Chapter 3 (e.g. maximum d/D). There are a few exceptions where existing sewers will need to be replaced by larger diameter sewers, or parallel sewers will need to be constructed to bypass flow around hydraulically deficient sewers. Detailed descriptions of the proposed existing system improvement projects are provided in Chapter 6.

The existing sewer trunk system improvements include the Elm, Quail Hollow, Live Oak, and B Street Trunk Sewers, and the Midway Force Main. The B Street, Live Oak trunk sewers, and the Midway force main, and their associated lift station improvements correct the most severe sewer capacity issue within the existing system. The 8-inch diameter



Legend

Modeled Wastewater Collection System

- Lift Station
- Abandoned Lift Station
- Pipelines
- Abandoned Pipelines

Existing System Improvements

Phase 1 (2009 - 2015)

- Lift Station

Pipelines

Phase 1 (2009 - 2015)

- New Pipeline
- Pipeline to be Replaced

Future System Improvements

Lift Station

- Phase 1 (2009 - 2015)
- Phase 3 (2021 - 2025)

Pipelines

- Phase 1 (2009 - 2015)
- Phase 2 (2016 - 2020)
- Phase 3 (2021 - 2025)
- Phase 4 (2026 - 2030)
- General Plan Boundary
- Parcels
- City Limits
- P-25 Figure Number
- 12" Proposed Pipeline Diameter
- 12/24" Proposed Pipeline/ Casing Pipe Diameter
- 8" Existing Pipeline Diameter

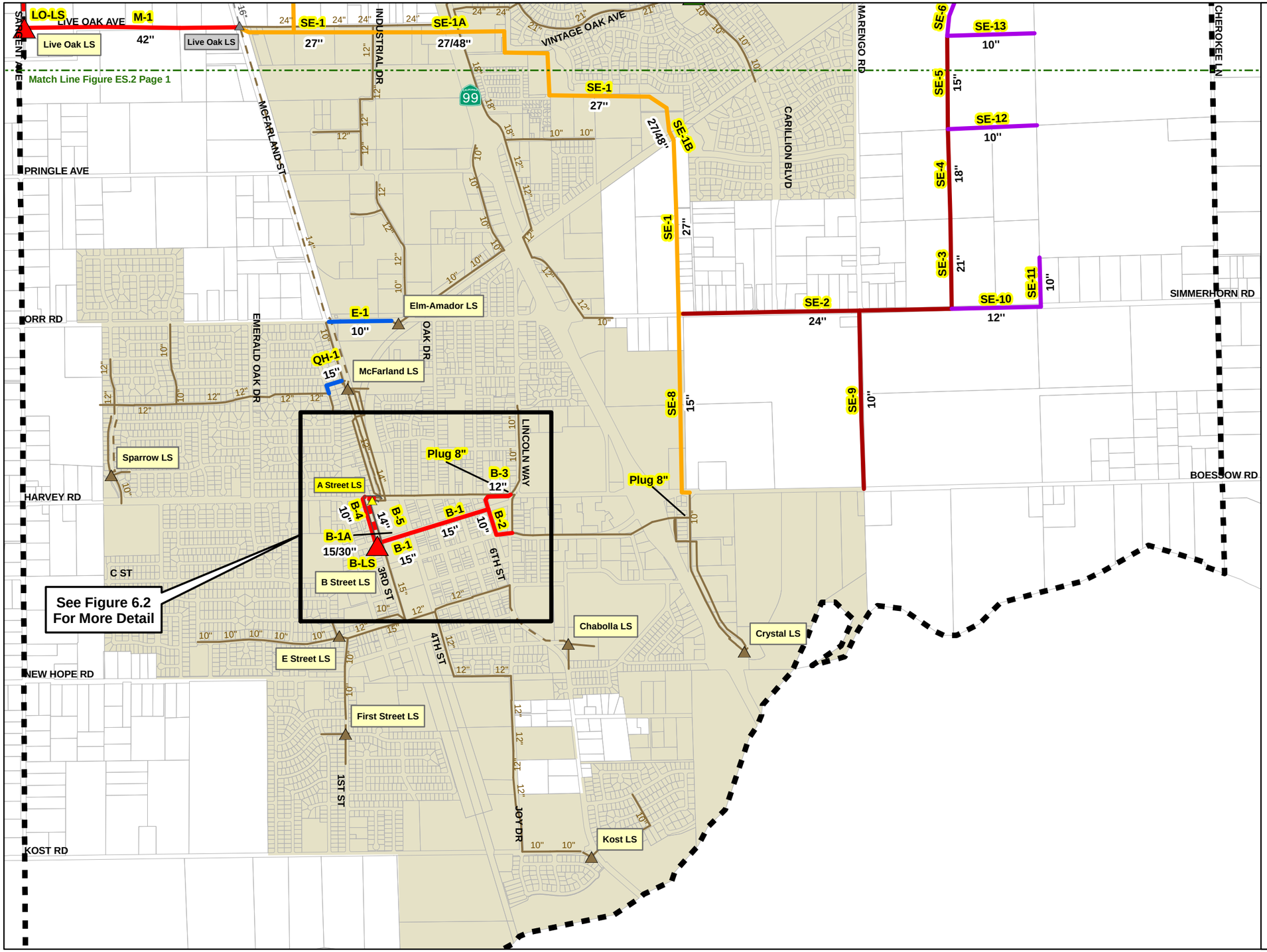
N

0 1,500 3,000 Feet

Figure ES.2
Proposed Capital Improvements (1 of 2)
Wastewater Collection System Master Plan
City of Galt

GALT

carollo



Legend

Modeled Wastewater Collection System

- Lift Station
- Abandoned Lift Station
- Relocated Lift Station
- Pipelines
- Abandoned Pipelines

Existing System Improvements

Phase 1 (2009 - 2015)

- Lift Station

Pipelines

Phase 1 (2009 - 2015)

- New Pipeline
- Pipeline to be Replaced

Future System Improvements

Lift Station

- Phase 1 (2009 - 2015)
- Phase 3 (2021 - 2025)

Pipelines

- Phase 1 (2009 - 2015)
- Phase 2 (2016 - 2020)
- Phase 3 (2021 - 2025)
- Phase 4 (2026 - 2030)
- Parcels
- City Limits
- General Plan Boundary
- P-25 Figure Number
- 12" Proposed Pipeline Diameter
- 12/24" Proposed Pipeline/ Casing Pipe Diameter
- 8" Existing Pipeline Diameter

N

0 1,500 3,000 Feet

Figure ES.2
Proposed Capital Improvements (2 of 2)
Wastewater Collection System Master Plan
City of Galt

GALT

carollo

sewer in A Street between the railroad and Lincoln Way experiences dramatic increases in flow following a storm event, due to rain runoff in the form of I/I. Hydraulic modeling results reveal that surcharging during large storms could lead to SSOs from manholes on this sewer.

ES.6.3 Proposed Future System Improvements

Chapter 6 summarizes the new trunk sewers and interceptors that will serve future users. Please note that the locations of the sewers are preliminary and will likely change during the design phase. The locations shown are possible alignments based on available information and are intended to assist in the development of probable construction costs. No investigation into the feasibility of these alignments has been conducted. However, an attempt was made to place future pipeline alignments within existing streets or proposed future streets identified in the General Plan's Land Use and Circulation Diagram.

The Southeast, Northeast and Commercial Trunk Sewers make up the majority of the new sewers that will serve future development. The City evaluated alternatives for relocating the Live Oak Pump Station. The alternative locations included the intersection at Live Oak and Midway Avenue or the WWTP. Although building an influent pump station at the WWTP along with a 42-inch diameter gravity trunk sewer had the highest capital cost, it was recommended as part of a project memorandum due to its overall lower life cycle costs compared to locating the pump station at Live Oak and Midway (Appendix E). After a long and careful consideration of estimated construction andoperation and maintenance costs, the impact to the improvements presented inthis master plan, and possible benefits to future development, City staff concluded that the force main alternative is the most cost effective and suitable improvement for the City.

ES.6.4 Existing and Build-out Lift Station Improvements

This Master Plan assumes that each lift station's firm capacity should be sufficient to pump 100 percent of the design flow rate. Each lift station's firm capacity was compared to the existing and build-out design flow conveyed to the lift station. If the design flow was greater than the lift station's firm capacity, then the lift station was considered deficient and required upgrade.

All the lift stations, with the exception of A Street, Vintage Oak, and Live Oak Lift Stations contain sufficient firm capacity to convey existing and future design flows. The A Street and Live Oak Lift Stations require immediate upgrades to convey existing design flows. The Vintage Oak lift station capacity will need to be increased to convey build-out design flows.

ES.7 CAPITAL PROJECT PRIORITIZATION

The majority of the proposed improvements are driven by future development. Most of the improvements are new sewers that serve future growth, but there are some improvements to existing facilities that are needed to serve existing users, or result from future growth.

Prioritizing the required capital improvements for the City's sewer system is an important aspect of the Master Plan. The improvement projects were prioritized based on the following factors:

- Upgrading existing sewers or lift stations to serve future users
- Building the sewers necessary to serve future users

Improvements to existing sewers and lift stations will provide sufficient capacity to mitigate existing issues and to convey increased flows resulting from new development. Future development will require the construction of sewers to serve new users. The implementation of these improvements will depend on the City's growth and selection of areas to be served with urban infrastructure. The City provided guidance on future development and phasing of infrastructure to serve future users. Based on this input, the projects were grouped into the following timeframes:

- Years 2009 through 2015
- Years 2016 through 2020
- Years 2021 through 2025
- Years 2026 through 2030

ES.7.1 Phase 1 Projects (2009-2015)

The highest priority project for the existing system includes the B Street Trunk Sewer (B-1, 2 and 3) and lift station project. Building the B Street Trunk Sewer will relieve the excess flows that currently overwhelm the 8-inch diameter sewer main in A Street and reduce the threat of SSOs. The A Street Lift Station should also be relocated to convey the design flows for this sewer tributary and to avoid conflicts with the City's planned railroad grade separation project.

The Live Oak Trunk Sewer and Midway Force Main projects (M-1 and M-2) and influent Live Oak pump station (LO-LS) are also high priority projects for the City. These projects are needed to replace the Live Oak Lift Station and force main and to serve future growth through the year 2030. Also, expansion of the Vintage Oak Lift Station (VO-LS) is needed before additional development within its tributary is approved.

- **Elm Trunk Sewer (E-1).** Mitigates existing capacity deficiency in existing 8-inch diameter sewer downstream of the Elm-Amador Lift Station.

- **Quail Hollow Trunk Sewer (QH-1).** Mitigates existing capacity deficiency in existing 10 and 12-inch diameter sewers around Oberlin and Trudy Way.

Projects serving new developments in the growth areas targeted by the City are shown in Phase 1. Some of these projects identified within Phase 1 are part of a larger capital project. For example, the first reach of the Northeast Trunk Sewer (NE-1) is targeted as the first segment of this sewer to be constructed.

In addition to these capital projects, the City will implement sewer main and lift station rehabilitation and replacement.

ES.7.2 Phase 2 Projects (2016-2020)

In general, the Phase 2 projects will serve future developments beyond year 2015. Lower priority system improvement projects were also targeted for construction in Phase 2. These projects are within the 10-year window identified by the City. Many of the projects are extensions of sewers that were started in Phase 1. For example, NE-2 represents the second phase of the Northeast Trunk Sewer. The Phase 2 projects include the following:

- **Northeast Trunk Sewer (NE-2).** Second reach of sewer constructed to service the area north of the railroad and east of Marengo Road.
- **Southeast Trunk Sewer (SE-1, SE-7, SE-8).** First reaches constructed to service new development in the area south of the railroad and east of Highway 99 and to redirect existing flows from the area around Crystal Way.
- On going sewer main and lift station rehabilitation and replacement.

ES.7.3 Phase 3 and 4 Projects (2021-2025 and 2026-2030)

Table 6.1 breaks the projects into Phase 3 and 4. For the purposes of prioritization, these are viewed as very long-term, development dependant projects, and will be grouped together. The Phase 3 and 4 projects include the following:

- **Northeast Trunk Sewer (NE-3, NE-4).** Final reaches constructed to service the area north of the railroad and east of Marengo Road.
- **Southeast Trunk Sewer (SE-2 through SE-9 through SE-13).** Final reaches constructed to service new development south of the railroad and east of Highway 99.
- **Commercial Trunk Sewer and Lift Station (C-1 through C-8A).** All reaches of sewers and lift station constructed to service the predominantly commercial land use areas north of Twin Cities Road.
- On going sewer main and lift station rehabilitation and replacement.

ES.8 CAPITAL PROJECT COSTS

A summary of the capital project costs and the implementation timeframe is presented in Chapter 7. The breakdown in existing and future user cost share by phase is summarized in Table ES.4. Table ES.5 summarizes the breakdown in cost for the different facility categories (e.g. trunk sewers and lift/pump stations).

Table ES.4 Existing Versus Future User Cost Share
Wastewater Collection System Master Plan
City of Galt

Reimbursement Category	Implementation Phase				
	2009-15 ($, mill.)	2016-20 ($, mill.)	2021-25 ($, mill.)	2026 - 30 ($, mill.)	Total ($, mill.)
Existing User (2)	17.6	3.7	3.7	3.7	**28.6**
Future User (3)	10.8	7.6	6.9	2.1	**27.3**
Total	**28.4**	**11.2**	**10.5**	**5.7**	**55.9**

Notes:
1. Costs are based on the ENR CCI 20 City average of 8534 (March 2009).
2. Projects are funded through user rates.
3. Projects are expected to be funded through sewer development impact fees collected by the City or by developers.

Table ES.5 Existing Versus Future Cost by Facility Type
Wastewater Collection System Master Plan
City of Galt

Reimbursement Category	Pipelines ($, mill.)	Lift Stations ($, mill.)	Total ($, mill.)
Existing Users(2)	13.3	15.2	**28.6**
Future Users(3)	17.4	10.0	**27.3**
Total	**30.7**	**25.2**	**55.9**

Notes:
1. Costs are based on the ENR CCI 20 City average of 8534 (March 2009).
2. Projects are funded through user rates.
3. Projects are expected to be funded through sewer development impact fees collected by the City or by developers.

ES.9 COST OF SERVICE

The existing and future user capital costs discussed above were used to determine a cost of service to existing rate payers and future customer connections. The following is not a rate study, fee program, or development impact fee analysis. It is a simplified assessment of the costs that the City might need to recover from existing rate payers and future development to pay for the proposed Master Plan projects. This analysis serves only to

assist the City in determining whether a rate or development impact fee increase might be needed to finance the proposed CIPs. This Master Plan analysis is simply a high level calculation that provides the potential order of magnitude assessment and brackets the possible costs. A more detailed rate/development impact study should be conducted to determine the magnitude of a possible increase to fund the proposed CIPs.

ES.9.1 Existing Users Fee for Service

The City collects sewer utility rates to pay for such services including but not limited to operations and maintenance, capital replacement and improvement, administration, and to establish a capital reserve. The capital costs to implement the proposed Master Plan projects fall under the capital replacement and improvement of existing system components. These costs are spread over approximately 7,200 existing service connections.

The total existing system Master Plan capital costs equal approximately $28.6 million. One possible scenario to finance these projects might include two phases of bond financing. One bond could cover the work for Phases 1 and 2 (2009 through 2020), and a second bond could finance projects for Phases 3 and 4 (2021 through 2030). For this simplified analysis, we assumed that the first bond would be for $21.3 million and the second for $7.4 million, both paid off over 30 years at an interest rate of 5 percent. The monthly increase necessary to finance the first bond to fund the proposed capital replacement projects could be approximately $7.25 per service connection. In year 2020 when the second bond is secured, the rate could increase to about $9 dollars per connection.

This simplified analysis does not take into account any existing City bonds (if any) that are being paid, the type of bond that would be used, alternative payment schedules, or class of service. It is recommended that a comprehensive rate analysis be completed to quantify the impacts to existing rate payers and to the City's proposed wastewater rates. The possible rate increase will likely vary depending on the class of service (e.g. single and multi-family residential, commercial, and industrial) and determining this level of detail for setting rates goes beyond the scope of this study.

ES.9.2 Future User Development Impact Fees

The City collects development impact fees to finance capital expenses associated with increased capacity of the wastewater collection system. The City's current development impact fee is approximately $8,700 and includes a sewer connection fee of approximately $4,900 and a Supplemental WWTP fee of about $3,800. The sewer connection fee portion finances sewers and lift stations. The total future system Master Plan capital costs equal approximately $27.3 million; $10.0 million would be needed for lift stations and $17.4 million for sewer pipeline construction. Depending upon the City's future development plan, the pipelines could be either funded through the development impact fees, or directly through the developers, with a separate reimbursement fund established for others who benefit from the improvement.

As mentioned above, based on the 2030 General Plan, the number of future sewer connections added to the City will likely increase above the projections summarized in Table 4-1 of the 2005 Urban Water Management Plan (UWMP). Based on the projected build-out wastewater demands, by year 2030 there could be between 18,000 to 20,000 connections within the General Plan boundaries. This represents an increase of 10,800 to 12,800 in future connections that would pay impact fees to finance future lift stations and sewer pipelines. Therefore, the possible capital cost of collection system improvements per future connection could range from $2,200 to $2,600. Note that these costs do not include impact fees associated with the WWTP.

The development impact fee is significantly influenced by the number of connections, and will likely vary based on the class of service. The City should complete a detailed development impact fee study to quantify the appropriate fee by class of service to finance increases in capacity for future users.

BACKGROUND

1.1 INTRODUCTION

This chapter presents a brief summary of the wastewater collection system service area, the need for this wastewater collection system master plan (Master Plan) and the objectives of the study. A list of abbreviations is also provided to assist the reader in understanding the information presented.

1.2 BACKGROUND

The City of Galt (City) is located along State Highway 99 in northern California's Central Valley, between the cities of Sacramento and Stockton, and near the Delta Recreation Area[1]. Figure 1.1 presents a location map of the City. The City owns, maintains and operates gravity sewer pipelines and forcemains, sewer lift stations and pump stations, and the wastewater treatment plant (WWTP). The City collects wastewater from residential, commercial, institutional, and industrial customers within the service area.

1.3 SEWER SERVICE AREA

Figure 1.2 illustrates the City's current wastewater service area. The City manages and maintains approximately 80 miles of sewer lines spanning 6 - to 24-inches in diameter, 12 lift/pump stations, and the WWTP.

The land use assumptions in this Master Plan were based on the City's 2030 General Plan Public Review Draft (General Plan) and projected future developments within the General Plan boundary. Should future planning conditions change from the assumptions stated in this Master Plan (i.e., accelerated growth, more intense developments, etc.), revisions and adjustments to the Master Plan recommendations would be necessary.

1.4 SCOPE AND AUTHORIZATION

The purpose of this Master Plan is to identify capacity deficiencies in the wastewater collection system, develop feasible alternatives to correct these deficiencies, and plan the infrastructure that will serve future development. On August 15, 2008, the City approved a professional service agreement with Carollo Engineers, P.C. (Carollo) to prepare this Master Plan for the wastewater collection system, which included the following main tasks:

[1] City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al.

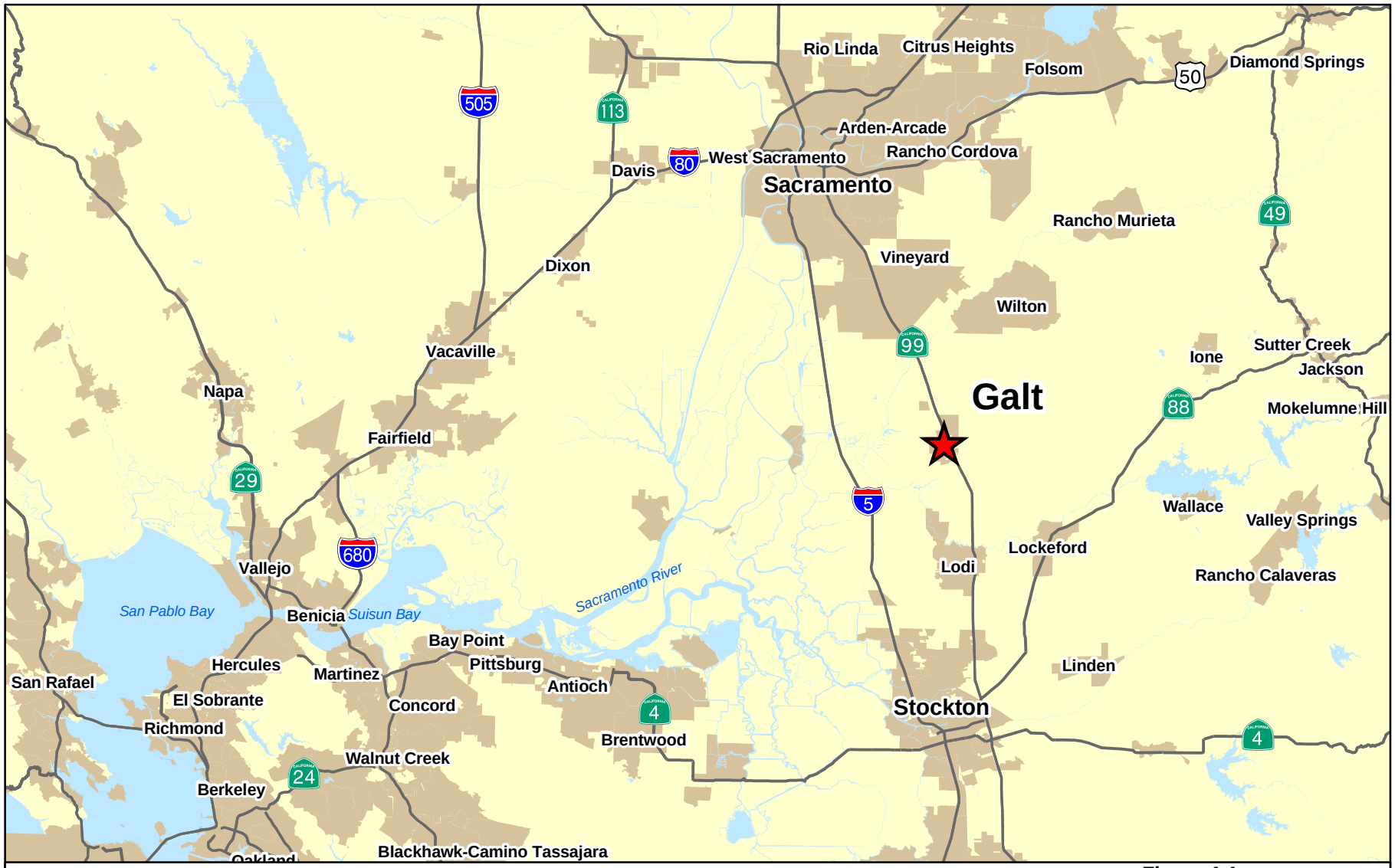
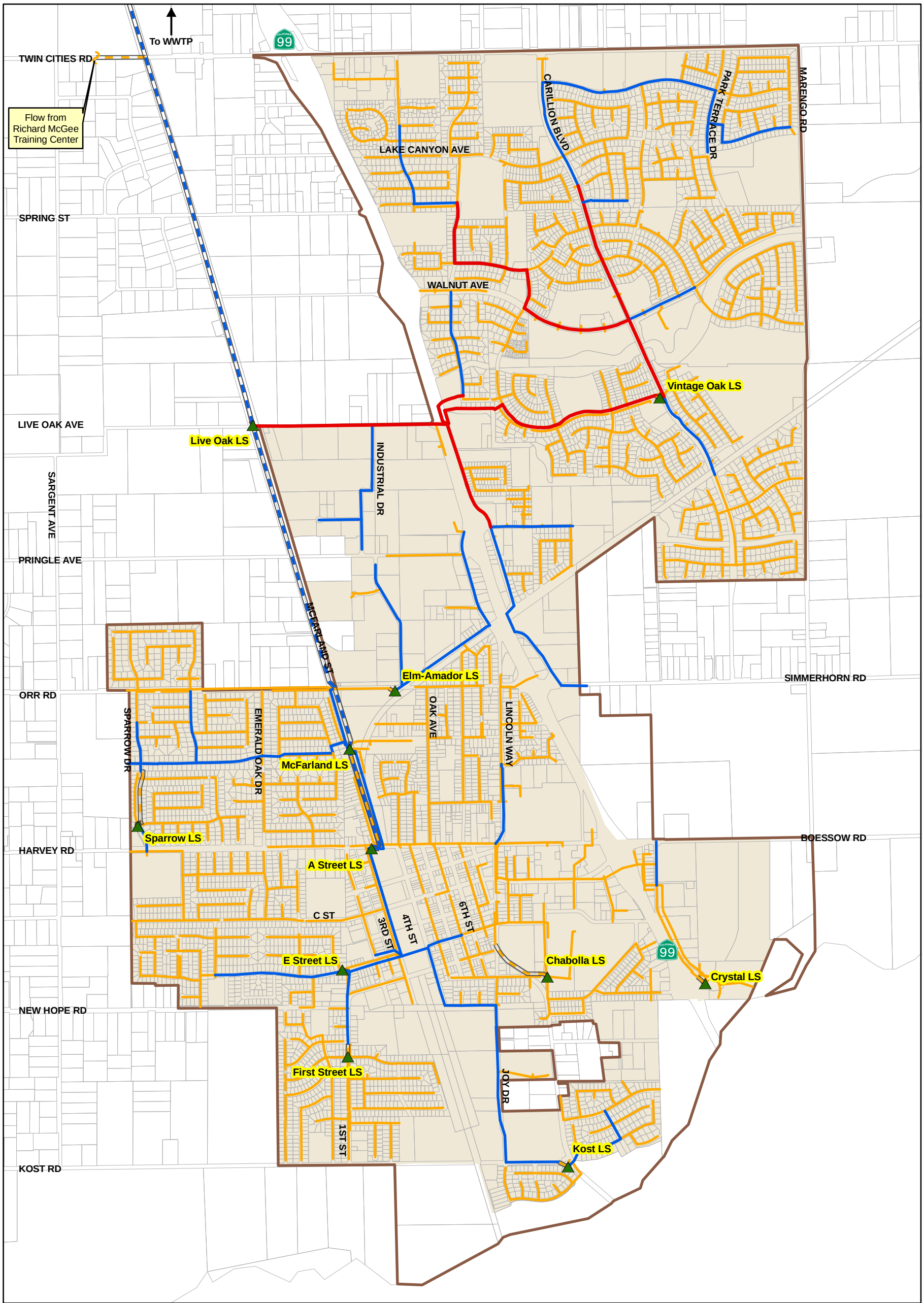


Figure 1.1
Location Map
Wastewater Collection System Master Plan
City of Galt



Legend

Existing Wastewater Collection System

- Lift Station

Sewer Pipeline Diameter

Gravity Main

- 8" and Smaller
- 10" - 16"
- 21" and Larger

Force Main

- 8" and Smaller
- 10" - 16"

- Current Wastewater Collection Service Area
- Parcels
- City Limits

N

0 1,250 2,500 Feet

Figure 1.2
Current Wastewater Collection System Service Area
Wastewater Collection System Master Plan
City of Galt

GALT

carollo

- Model development
- Collection system analysis and capital project development
- Master Plan preparation

1.5 REPORT ORGANIZATION

The Master Plan report contains seven chapters, followed by appendices that provide supporting documentation for the information presented in the report. The chapters are briefly described below:

Chapter 1 - Background. This chapter presents the need for this Master Plan and the objectives of the study. Lists of abbreviations and reference materials are also provided to assist the reader in understanding the information presented.

Chapter 2 - Study Area Description. This chapter presents a description of the study area, defines the land use classifications, and summarizes the historical population trends.

Chapter 3 - Planning Criteria. This chapter presents the planning criteria for evaluating the wastewater collection system. The planning criteria address the collection system capacity, gravity sewer slopes, maximum depth of flow within a sewer, average sewer flow coefficients, and sewer peaking factors.

Chapter 4 - Wastewater Design Flows. This chapter summarizes the flow-monitoring program and presents the calculation of the design flows used to model the existing and future sewer system.

Chapter 5 - Wastewater Collection System Facilities and Hydraulic Model. This chapter describes the development and calibration of the City's wastewater collection system hydraulic model.

Chapter 6 - Capacity Evaluation and Proposed Improvements. This chapter discusses the hydraulic evaluation of the collection system and the proposed projects that correct capacity deficiencies and serve future users.

Chapter 7 - Capital Improvement Projects. This chapter presents the capital improvement projects, a summary of the capital costs, and assessment of the costs that the City will need to recover from existing rate payers and future development. This chapter is organized to assist the City in making finance decisions.

1.6 ACKNOWLEDGMENTS

Carollo Engineers wishes to acknowledge and thank Mr. Gregg Halladay, Director of Public Works; Mr. Paul Cavanaugh, City Engineer; Mr. Bill Forrest, Project Manager/Senior Civil Engineer; Ms. Lisa Sanders, Senior Civil Engineer; Mr. Adin Selby, Streets Superintendent;

Mr. Bo Dahlberg, Wastewater Treatment Plant Supervisor; Mr. Reb Bisnett, Water Superintendent. Their cooperation and courtesy in obtaining a variety of necessary information were valuable components in completing and producing this report.

1.7 ABBREVIATIONS AND DEFINITIONS

To conserve space and to improve readability, the following abbreviations are used in this report.

AACE	Association for the Advancement of Cost Engineering
ASCE	American Society of Civil Engineers
ADF	average day flow
ADWF	average dry weather flow
ADMMF	average day maximum month flow
AFY	acre feet per year
AWWF	average wet weather flow
BEF	best efficiency point
BWF	base wastewater flow
CIP	capital improvement project
City	City of Galt
cfs	cubic feet per second
County	County of Sacramento
DOF	California Department of Finance
DU	dwelling unit
DWF	dry weather flow
EDU	equivalent dwelling unit
ENR CCI	Engineering News Record Construction Cost Index
FEMA	Federal Emergency Management Agency
FOG	fats, oil, and grease
fps	feet per second

GIS	geographic information system
gpda	gallons per day per acre
gpcd	gallons per capita per day
gpm	gallons per minute
GWI	groundwater infiltration
I/I	infiltration/inflow
LAFCo	Local Agency Formation Commission
MDDWF	maximum day dry weather flow
MDWWF	maximum day wet weather flow
MG	million gallons
mgd	million gallons per day
msl	mean sea level
NFIP	National Flood Insurance Program
PDWF	peak dry weather flow
PWWF	peak wet weather flow
RCP	reinforced concrete pipe
ROW	right-of-way
SASD	Sacramento Area Sewer District
SSMP	sanitary sewer management plan
SSO	Sanitary Sewer Overflow
UWMP	Urban Water Management Plan
VCP	vitrified clay pipe
WEF	Water Environment Federation
WRCC	Western Regional Climate Center
WWF	wet weather flow
WWTP	Wastewater Treatment Plant

1.8 REFERENCE MATERIAL

The following documents were referenced in the preparation of this master plan:

- City of Galt General Plan, Existing Conditions Report for the 2008 General Plan, Public Review Draft, November 2005, Mintier & Associates et al.
- City of Galt General Plan, Policy Document, Public Review Draft, July 2008, Mintier & Associates et al.
- City of Galt General Plan, Draft Environmental Impact Report, Draft, July 2008, Mintier & Associates et al.
- City of Galt Wastewater Collection System Capacity Analysis Phase I, October 2005, Boyle Engineering Corporation
- City of Galt Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, March 2009, V&A

STUDY AREA DESCRIPTION

This chapter presents a description of the study area, defines the land use classifications, and summarizes the historical population trends.

2.1 STUDY AREA

The 2030 General Plan Public Review Draft (General Plan) planning boundary is the study area boundary for this wastewater collection system master plan (Master Plan). The Master Plan study boundary and the 2030 General Plan planning boundary are synonymous and will be used interchangeably throughout this report. The General Plan boundary extends beyond the current sewer service area and is approximately 8,817 acres (13.8 square miles). The Master Plan contains a forecast of sewer system improvements in a large study area beyond the City of Galt's (City) limits. Figure 2.1 shows the study area boundary and the current City limits. The current City limits roughly extend from Dry Creek on the south to Twin Cities Road on the north; and from McFarland Street/Sparrow Drive on the west to Marengo Road on the east[1].

Evaluating infrastructure needs beyond the current City limits is important because:

- There are pending conceptual development plans that are beyond the City limits; and
- Historical rapid growth in the Sacramento metropolitan area indicates that significant development into the study area could occur within a short planning period, in the future.

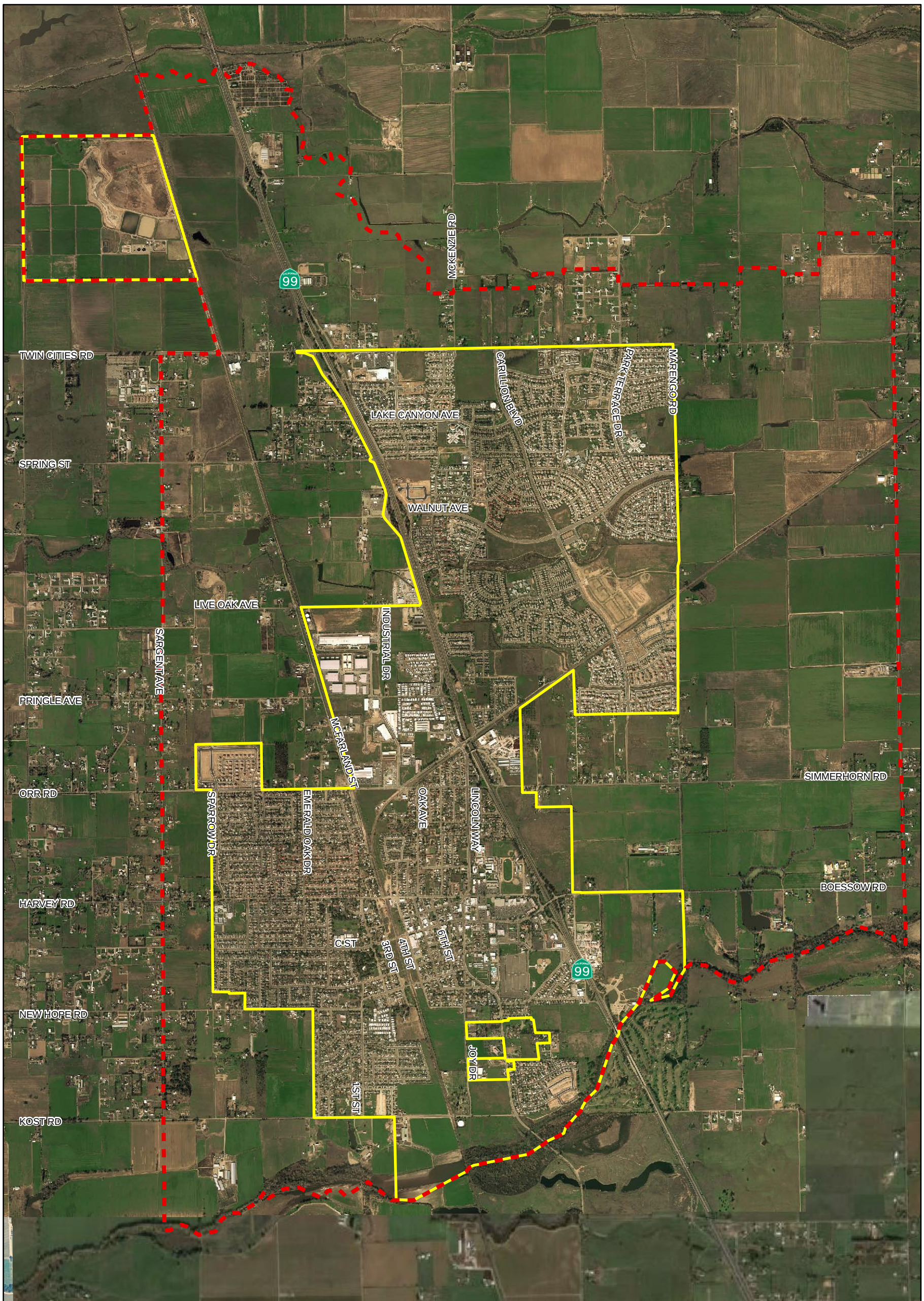
2.2 PLANNING PERIOD

The Master Plan study area is intended to include the existing City limits and development within the General Plan boundary that could occur through the year 2030. Existing and projected populations and land uses within the study area are discussed in this chapter.

2.3 CLIMATE

The City's study area is characterized by a Mediterranean-type climate with wet, cold winters, and warm, dry summers. Most of the rainfall occurs between November and April with an average annual rainfall of 17.5 inches[1]. The Study Area's elevation ranges from about 34 feet above mean sea level (msl) on the west side of the City, to 68 feet msl on the east.

[1] City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al.



Legend

General Plan Boundary

City Limits

N

0 2,500 5,000 Feet

Figure 2.1
Study Area
Wastewater Collection System Master Plan
City of Galt

2.4 100-YEAR FLOODPLAIN

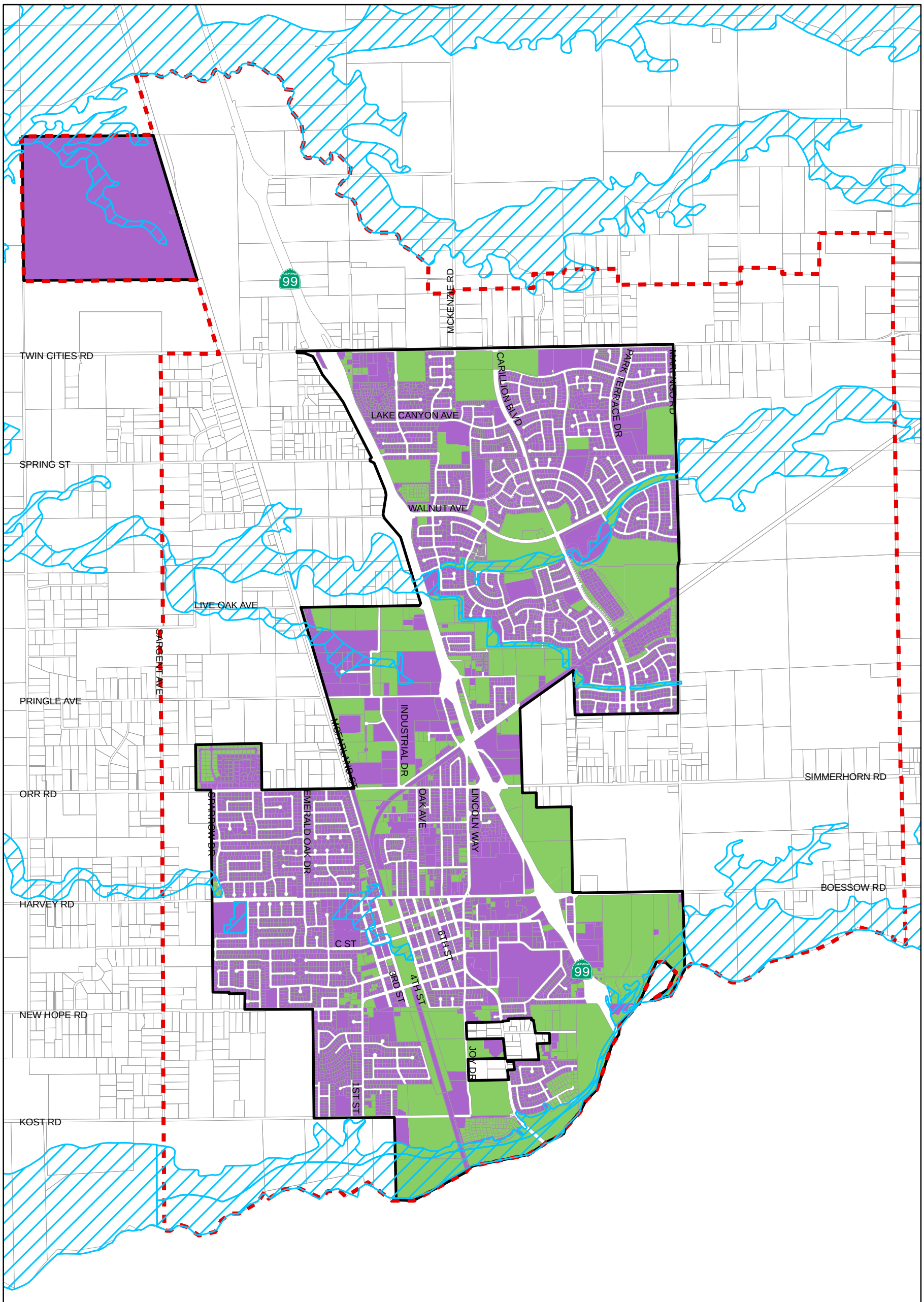
Maps indicating “special flood hazard areas” (i.e. floodplains) have been developed through the National Flood Insurance Program (NFIP), which is administered by the Federal Emergency Management Agency (FEMA). Areas of particular importance for insurance purposes are those that are subject to inundation by a 100-year flood event. Figure 2.2 shows the 100-year floodplain boundary for the current City limits and General Plan boundary.

This Master Plan assumes that currently undeveloped land within the 100-year floodplain will not be developed in the future. These areas, therefore, are assumed to generate no future wastewater flow through build-out. If any project is allowed to develop within the floodplain, the anticipated increase in wastewater flow generated from these project should have relatively little impact on the proposed sewer improvements in this Master Plan. If the floodplain area changes in size and results in more development, then the City might consider evaluating the impacts of downstream sewers.

2.5 LAND USE

At the time of writing this Master Plan, the General Plan Existing Conditions Report (November 2005), General Plan Policy Document Public Review Draft (July 2008), and General Plan Environmental Impact Report Draft (July 2008) were complete. The General Plan guides development within the planning boundary and establishes the long-range development policies. The General Plan also provides land use and population projections. Land use and population information are integral components in determining the amount of wastewater generation within the City. The type of land use in an area will affect the volume and character of the wastewater generation. Adequately estimating the generation of wastewater from various land use types is important in sizing and maintaining effective sewer system facilities.

Land use assumptions used in this study are consistent with the 2030 General Plan. Since the land use assumptions forecast the type of growth within the study area, this association to the Master Plan should ensure that the wastewater generation projections and facilities required to serve future growth are consistent with the City’s guiding document on development. Figure 2.3 illustrates the different land uses found in the General Plan. The study area’s land use designation and respective acreage totals are summarized in Table 2.1. Appendix A provides a description of the different land uses. The descriptions are excerpts from the General Plan.



Legend

- 100-Year FEMA Floodplain
- Developed Land (November 2008)
- Vacant Land (November 2008)
- Parcels
- General Plan Boundary
- City Limits

N

0 2,500 5,000 Feet

Figure 2.2
100-Year Floodplain
Wastewater Collection System Master Plan
City of Galt

Source: Mintier & Associates

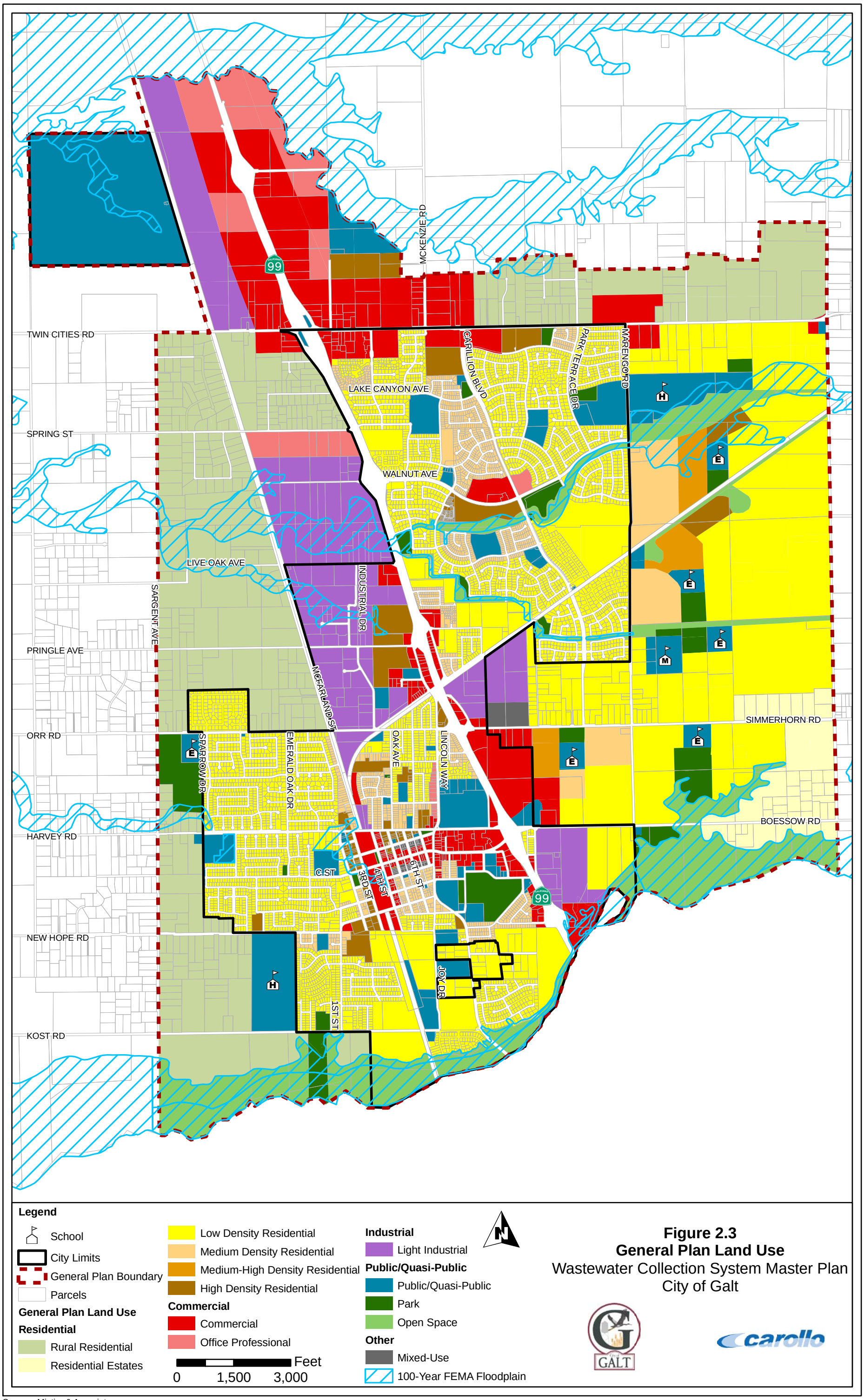


Figure 2.3
General Plan Land Use
Wastewater Collection System Master Plan
City of Galt

Source: Mintier & Associates

Table 2.1 Study Area Land Use Designations
Wastewater Collection System Master Plan
City of Galt

Land Use Designation	General Plan Boundary [1] (ac)	2007/2008 City Limits		
		Total (ac)	Developed (ac)	Undeveloped (ac)
Residential				
Rural Residential (A-RR)	1,329	0.4	0.1	0.3
Residential Estates (K-RE)	172	0	0	0
Low Density Residential (B-LDR)	2,246	1,401	990	411
Medium Density Residential (C-MDR)	335	216	208	8
Medium High Density Residential (L-MHD)	77	0	0	0
High Density Residential (D-HDR)	181	140	101	39
Employment Related				
Commercial (E-C)	623	271	101	169
Office Professional (F-OP)	179	11	3	8
Light Industrial (G-LI)	599	284	120	164
Others				
Public/Quasi-Public (H-PQP)	437	218	162	56
Open Space (I-OS)	561	152	11	141
Park (I-PK)	191	70	70	0.2
Mixed Use (M-MU)	19	5	4	0.5
Roads/Railroads/Canals [2]	1,572	702	702	0
Wastewater Treatment Plant [3]	296	293	293	0
Total	**8,817**	**3,763**	**2,766**	**997**

Notes:
1. Galt General Plan boundary is the study area boundary. Land use totals exclude roads, canals, and railroads. Adjustments to land use totals were made in order to maintain a Roads/Railroads/Canals subtotal that is approximately 20 percent.
2. Roads not identified in the General Plan as a land use, but separated here for flow calculations.
3. Included with Public/Quasi-Public total land use in General Plan, but separated here for flow calculations.

2.5.1 Wastewater Service Area by Land Use

2.5.1.1 Existing Service Area Land Use

The City provides wastewater collection service to residents, businesses, and other institutions within its City limits. Table 2.1 provides the acreage totals by land use classification within the General Plan boundary. Also included in Table 2.1 are the land use totals for the 2007/08 sewer service area, and the breakdown between developed land, which generates wastewater flow, and undeveloped land that will be developed in the future. The City currently provides sewer service to approximately 3,763 acres (includes developed and undeveloped land) or 5.9 square miles.

The largest land use category is residential (rural density, low density, medium density, medium-high density, and high density), which accounts for approximately 1,758 acres, or approximately 47 percent of the total current City limit acreage. Commercial, office professional, and light industrial make up approximately 566 acres, or 15 percent of the total. Other land uses such as public/quasi-public, parks, streets, and open space account for 1,440 acres, or 38 percent of the total service area.

2.5.1.2 Future Service Area Land Use

At build-out of the General Plan boundary, the City will encompass approximately 8,817 acres. Build-out is defined as complete development of all lands outside the 100-year floodplain. There are approximately 8,059 acres within the study area that are outside the 100-year floodplain. The breakdown of the different land use categories is provided in Table 2.1.

2.6 HISTORICAL AND FUTURE POPULATION

The City was historically an agriculture based community, and has become an important transportation hub for rail and trucking. It has also evolved into a bedroom community for the growing regional centers of Sacramento and San Joaquin Counties[2]. In 1990, the City's population began to grow rapidly and that growth continued through year 2000. From 1990 through 2007, the population grew from approximately 8,800 to 23,500. Over these 17 years, the City grew at an annual rate of about six percent.

The General Plan forecasts that the City's population will grow at an annual rate of 3.4 percent from 2002 to 2025, and will reach a 2030 population of 50,094 people. Table 2.2 and Figure 2.4 summarize the City's historical and projected population to year 2030.

[2] City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al.

Table 2.2	**Historical and Projected Population Wastewater Collection System Master Plan City of Galt**		
Year	**Population**	**Year**	**Population**
1970	3,200	2007	23,470
1980	5,514	2015	32,779
1990	8,775	2020	38,000
1995	14,800	2025	44,150
2000	19,472	2030	50,094

Note:
1. Source: City of Galt 2030 General Plan, Existing Conditions Report, November 2005, Mintier & Associates et al. and City of Galt General Plan, Policy Document, Public Review Draft, July 2008, Mintier & Associates et al.

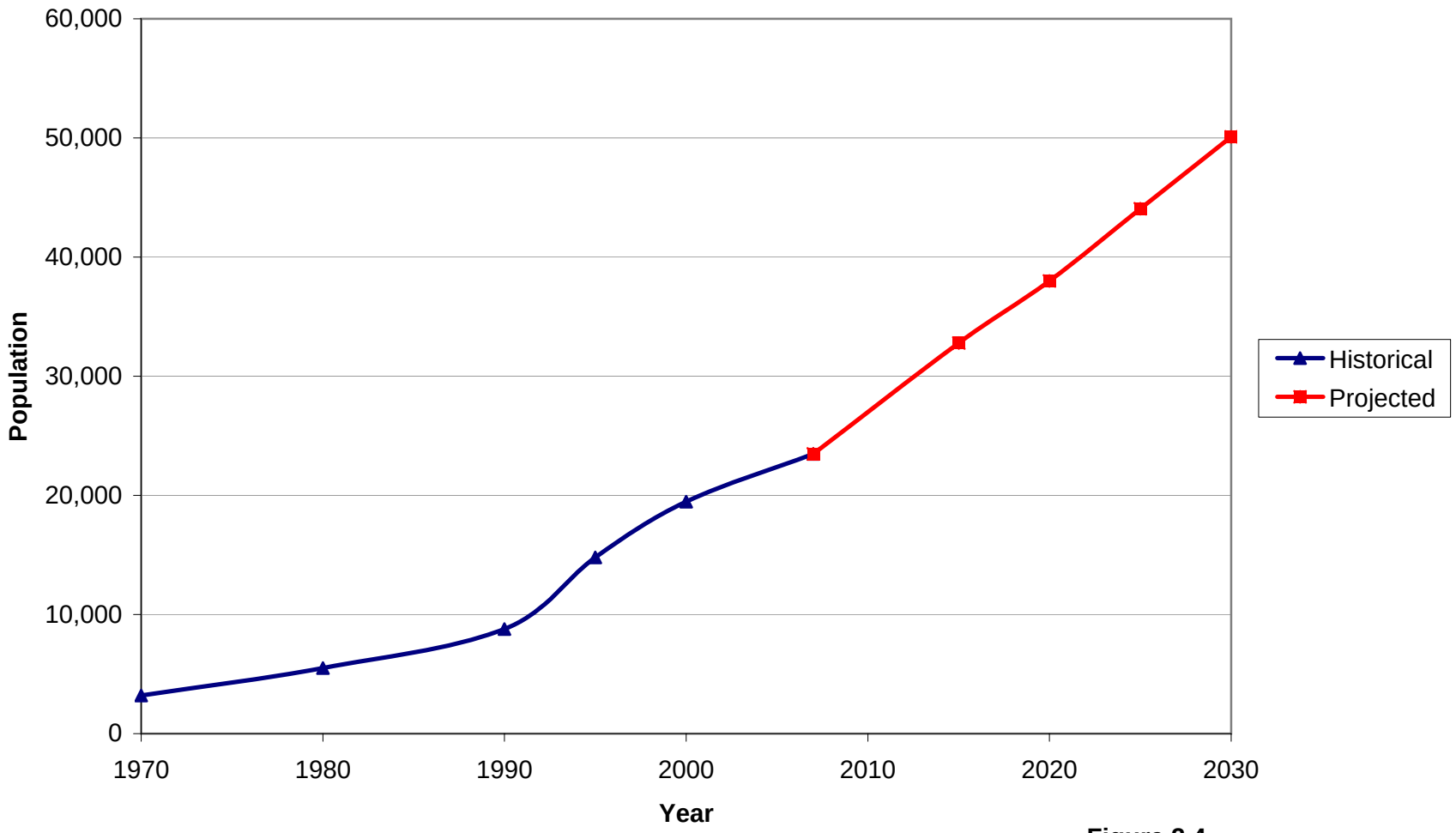


Figure 2.4
Historical and Projected Population
Wastewater Collection System Master Plan
City of Galt

carollo
Engineers...Working Wonders With Water™

PLANNING CRITERIA

The capacity of the City of Galt's (City) wastewater collection system was evaluated based on the planning criteria defined in this chapter. The criteria include standards from the City's Improvement Standards and Specifications (Improvement Standards), the Sacramento Area Sewer District (SASD) standards, and other planning criteria developed by Carollo based on engineering judgment and past experience. The planning criteria address the collection system capacity, gravity sewer slopes, maximum depth of flow within a sewer, average wastewater flow coefficients, and wastewater peaking factors.

3.1 GRAVITY SEWERS

Capacity analysis of the wastewater collection system was performed in accordance with the criteria established in this chapter. The City's Improvement Standards stipulate general policies of the City and outline sewer design criteria. Some of these criteria are discussed below. If not discussed in this wastewater collection system master plan (Master Plan), the reader should assume that the design criteria conform to the Improvement Standards. Sewer pipe capacities are dependent on many factors, including roughness of the pipe, the maximum allowable depth of flow, minimum velocity, and slope of pipe.

3.1.1 Manning Coefficient (n)

The Manning coefficient 'n' is a friction coefficient and varies with respect to pipe material, size of pipe, depth of flow, smoothness of pipe and joints, and extent of root intrusion. For sewer pipes, the Manning coefficient typically ranges between 0.011 and 0.017, with 0.013 being a representative value used for sewer system master planning, which is consistent with the SASD standards.

3.1.2 Flow Depth Criteria (d/D)

The primary criterion used to identify capacity deficient trunk sewers or to size new improvements is the maximum flow depth to pipe diameter ratio (d/D). This approach is consistent with the 2005 Boyle Engineering capacity analysis report. The d/D value is defined as the depth (d) of flow in a pipe during peak flow conditions divided by the pipe's diameter (D). The City's Improvement Standards do not define the acceptable d/D values for various pipe diameters.

3.1.2.1 Flow Depth for Existing Sewers

Using a conservative d/D ratio when evaluating existing sewers may lead to unnecessary replacement of existing pipelines. Therefore, a d/D ratio of 0.92 (pipe flowing at maximum capacity) was used to evaluate the City's existing trunk sewer system for peak dry weather flow (PDWF). During peak wet weather flow (PWWF) (this is typically the maximum hourly flow in the collection system) water levels were allowed to rise to within three feet of the

manhole rim. Sewers were allowed to surcharge under these maximum flow conditions. If the flow depth was greater than the maximum allowed, then the sewer was deemed deficient and a larger sewer was proposed to provide greater flow capacity.

3.1.2.2 Flow Depth for New Sewers

When designing new sewers, it is common practice to adopt variable flow depth criteria for different pipe sizes. Design d/D ratios typically range from 0.5 to 0.92, with the lower values used for smaller pipes, which may experience flow peaks greater than design flow or may experience blockages from debris, paper or rags.

Sewers less than 12-inches in diameter shall be designed to flow half full at peak flow rates (peak flow rates will be discussed later). Sewers 12 to 18-inches in diameter shall be designed to flow at two-thirds depth at peak flow rate. Sewers larger than 18-inches diameter shall be designed to flow at a d/D of 0.75 at peak flow rate. The maximum allowable d/D ratios for design flow conditions are summarized in Table 3.1.

SASD Standards specify that pipelines 12 inches and larger shall be allowed to flow at a d/D of 1.0. Carollo recommends the d/D ratios in Table 3.1 to provide a conservative recommendation in the event that the projected flows change as a result of modifications in flow generation assumptions or land uses after the Master Plan has been published.

Table 3.1 d/D Ratios for Design Flow Conditions
Wastewater Collection System Master Plan
City of Galt

Sewer Diameter	Design Flow Maximum d/D Ratio
Less than 12-inches	0.50
D=12-inches up to D=18-inches	0.67
Greater than 18-inches	0.75

3.1.3 Design Velocities and Minimum Slopes

In order to minimize the settlement of sewage solids, sewer velocity should be equal to or greater than 2 feet per second (fps) for all sewers when flowing at design flow d/D (based on roughness coefficient of 0.013). At this velocity, the sewer flow will typically provide self-cleaning for the pipe. Table 3.2 lists the recommended minimum slopes and their corresponding maximum flows for maintaining self-cleaning velocities (equal to or greater than 2 fps) when the pipe is flowing at its maximum depth.

The recommended minimum slopes presented in Table 3.2 are consistent with those presented in the SASD standards for sewers up to 18-inches in diameter. For sewers larger than 18-inches in diameter, the recommended minimum pipes slopes are less than those presented in the SASD standards. The reason for the deviation is that the SASD minimum slopes are steeper than what is required to maintain a minimum velocity of 2 fps.

3.1.4 Changes in Pipe Size

When a smaller sewer joins a large one, the soffit elevation of both pipes is matched to maintain the same energy gradient.

Table 3.2 Minimum Slope and Maximum Flow
Wastewater Collection System Master Plan
City of Galt

Pipe Diameter (inches)	Minimum Slope[1] (feet/feet)	Calculated Flow at Maximum d/D Criteria	
		d/D	Maximum Flow (mgd)
8	0.0035[2]	0.5	0.23
10	0.0025[2]	0.5	0.35
12	0.0020[2]	0.67	0.81
15	0.0015[2]	0.67	1.28
18	0.0012[2]	0.67	1.86
21	0.0009	0.75	2.80
24	0.0008	0.75	3.77
27	0.0007	0.75	4.83
30	0.0006	0.75	5.92
36	0.0006	0.75	9.63
42	0.0006	0.75	14.53
48	0.0006	0.75	20.74

Notes:
1. Recommended minimum slope for maximum pipe flow at various d/D values and velocity greater than or equal to 2 fps.
2. Sacramento Area Sewer District Standards

3.1.5 Lift Stations, and Force Mains

According to SASD Standards, the required flow rates from ADWF to PWWF shall be 60 to 115 percent of the capacity of the selected pump at its best efficiency point (BEF). Therefore, all sewage lift stations should have sufficient firm capacity (capacity with the largest pump out of service) to pump the design flow.

Force main piping shall be sized to provide a minimum velocity of 3 fps at the design flow rate of the lift station and no more than 8 fps. There are no slope requirements for force mains in the Improvement Standards.

For the determination of head loss, the Hazen Williams Equation was used with a C factor of 100, consistent with SASD standards.

Chapter 4

SEWER DESIGN FLOWS

This chapter summarizes the flow monitoring program and presents the calculation of the design flows used to model the existing and future sewer system.

4.1 FLOW MONITORING PROGRAM

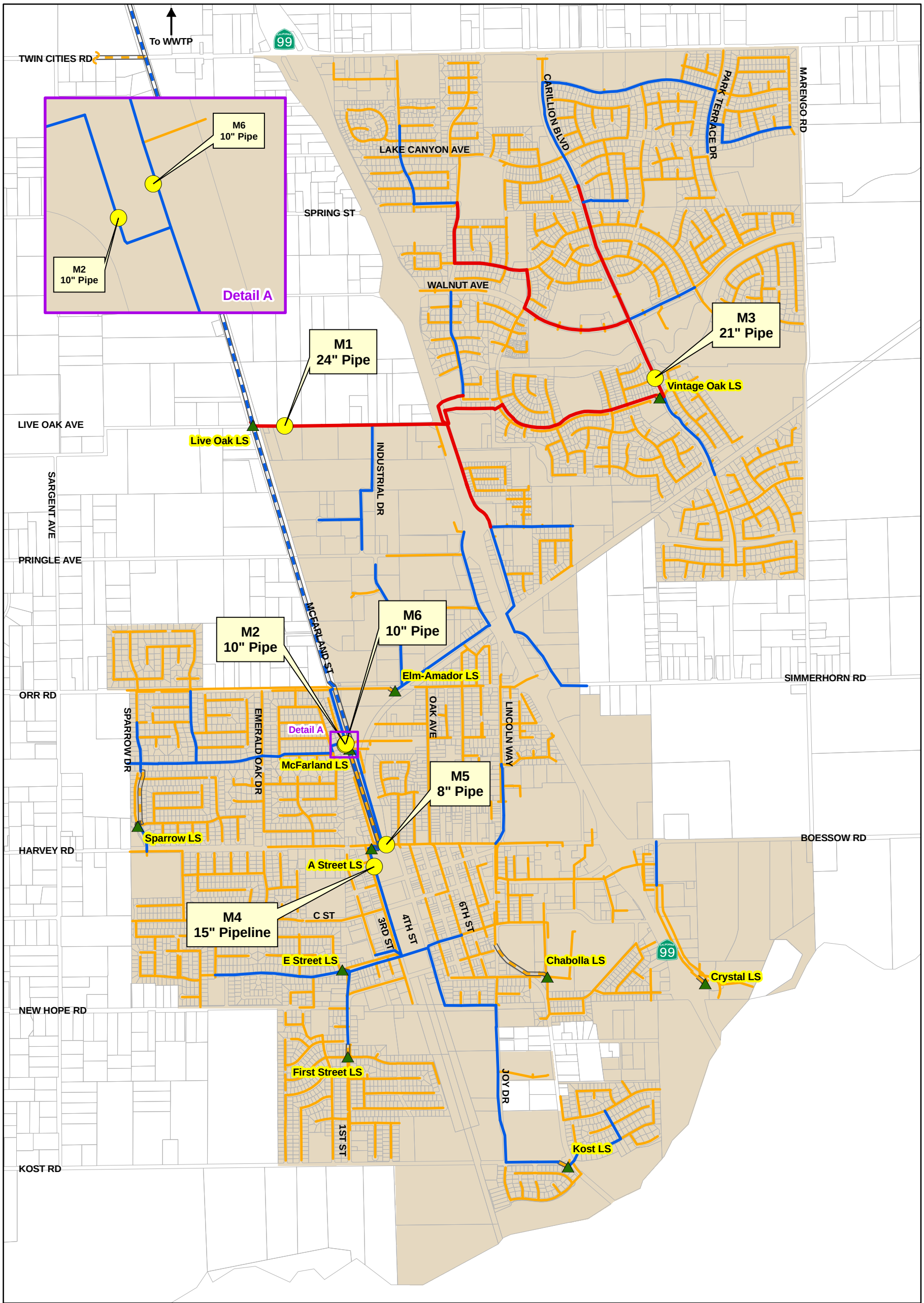
Temporary flow monitoring was conducted to assist in the development of design flow criteria, and to correlate actual sewer system flows to the hydraulic model predicted flows. Flow monitoring data are used to calibrate the wastewater collection system hydraulic model for dry weather and wet weather flow. The “Sanitary Sewer Flow Monitoring, and Inflow/Infiltration Study, March 2009” prepared by V&A Consulting Engineers, Inc. summarizes the flow monitoring program and was submitted to the City as a separate report. A copy of the report is included in Appendix F.

The primary purpose of flow monitoring is to measure flow contributions from different areas of the collection system. The flow monitoring period was conducted for approximately four weeks from December 13, 2008 through January 15, 2009 at 6 monitoring sites, at locations selected by Carollo Engineers, P.C. (Carollo). Figure 4.1 illustrates the flow meter locations. The meter sites were selected to best model the sewer areas and multiple sub-areas within the sewer system.

4.2 FLOW MONITORING RESULTS

4.2.1 Average Day Flow Data

During the flow monitoring period, depth and velocity data were collected at each meter at 5-minute intervals. The 5-minute data was then aggregated to 15-minute and hourly data for the dry weather flow calibration effort. Characteristic dry weather 24-hour diurnal flow patterns for each site were developed based on the hourly data. This hourly flow data was then used to calibrate the hydraulic model for average day flow. A summary of the average day flow for the 6 flow meters is presented in Table 4.1. The flow data indicated that the average weekday flows were generally equal to the average weekend flows, however, the diurnal patterns occasionally differed. For this wastewater collection system master plan (Master Plan), the weekday flow averages and diurnal patterns were used for calibration and analysis.



Legend

- Flow Meter Location
- **Existing Wastewater Collection System**
- Lift Station

Sewer Pipeline Diameter

Gravity Main
- 8" and Smaller
- 10" - 16"
- 21" and Larger

Force Main
- 8" and Smaller
- 10" - 16"

- Parcels
- City Limits

N

0 1,250 2,500 Feet

Figure 4.1
Flow Meter Locations
Wastewater Collection System Master Plan
City of Galt

Table 4.1	Average Day and Average Peak Dry Weather Flow Wastewater Collection System Master Plan City of Galt			
Flow Meter	**Average Day Flow[1] (mgd)**		**Average Day Peak Flow[2] (mgd)**	
	Weekday	Weekend	Weekday	Weekend
1	0.98	1.01	1.44	1.58
2	0.298	0.30	0.47	0.43
3	0.67	0.73	1.11	1.07
4	0.51	0.52	0.74	0.83
5	0.25	0.23	0.37	0.40
6	0.18	0.18	0.23	0.24

Source: Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, March 2009
Notes:
1. Average daily flows calculated from data.
2. Average of the daily peak daily flows.

4.2.2 Rainfall Data

One rain gauge was installed within the City to record rainfall data during the flow monitoring period. Rainfall data was also collected from a gauge in the City of Lodi, just south of Galt. The historical average rainfall for the flow-monitoring period (December 13 to January 15) is 3.27-inches. The historical data was taken from the Western Regional Climate Center (WRCC) at station 045032 in Lodi, California. The measured rainfall for the flow monitoring periods totaled 1.96-inches, which is approximately 60 percent of the historical average for the Galt area. Three rainfall events were captured during this time period, with the largest totaling 0.87-inches of rainfall for a 24-hour period. The flow monitoring report classifies this event as less than a 2-year storm. For more details on the rainfall analysis see the Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, March 2009.

4.2.3 Wet Weather Flow Data

Flow monitoring data was also evaluated to determine how the collection system responds to wet weather events. As mentioned above, the flow monitoring program captured three rainfall events. The event that occurred on December 24th and 25 caused a sufficient inflow and infiltration (I/I) response to be used for I/I analysis.

The parameters from the I/I analysis used for the master plan are the peak I/I rate and the R-Value. Table 4.2 summarizes the results from the I/I analysis. As can be seen in Table 4.2, basin M6 showed the highest I/I response with an R-Value of 1.01 percent.

The R-Value is the percentage of rainfall volume that makes it into the collection system as I/I. The distribution of I/I is adjusted until the modeled flows match the peak I/I rate. More discussion on model calibration is presented in Chapter 5.

Table 4.2 I/I Results Summary
Wastewater Collection System Master Plan
City of Galt

Flow Meter	ADF(1) (mgd)	Estimated Total I/I(3) (MG)	R-Value(3) (%)	Peak I/I Rate(3) (mgd)	Peak I/I to ADF Ratio
1	0.98	0.30	0.85%	0.73	0.74
2	0.29	0.04	0.47%	0.14	0.49
3	0.67	0.16	0.79%	0.45	0.66
4	0.51	0.11	0.53%	0.39	0.75
5	0.25	0.08	0.67%	0.36	1.46
6	0.18	0.05	1.01%	0.13	0.71

Source: Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study, March 2009
Notes:
1. Average daily flow calculated from data.
2. Average of the daily peak daily flows.
3. Results presented are taken from the December 24 - 25, 2008 wet weather event.

4.2.4 Design Storm Data

Design storms are rainfall events used to analyze the performance of a collection system under peak flows and volumes, and have specific recurrence intervals and rainfall duration. The development of a design storm can be accomplished in different ways. If data is not available, a synthetic design storm can be developed. A 10-year, 24-hour design storm is typical for modeling collection systems. For this Master Plan, a 10-year, 24-hour design storm was developed using the methods for developing short duration design storms as described in the City and County of Sacramento Drainage Manual, Volume 2: Hydrology Standards (December 1996). Based on the methods in the drainage manual, the depth of a 10-year, 24-hour storm totaled 3.26-inches, with a peak intensity of 0.62 inches/hour. Figure 4.2 shows the distribution of the 10-year, 24-hour design storm.

4.3 WASTEWATER FLOW COMPONENTS

Wastewater consists of dry weather flow (DWF) and wet weather flow (WWF). DWF (or base flow) is flow generated by routine water usage in the residential, commercial, business and industrial sectors of the sewer system. The other component of DWF is the contribution of dry weather groundwater infiltration (GWI) into the sewer system. Dry weather GWI will enter the sewer system when the relative depth of the groundwater table is higher than the

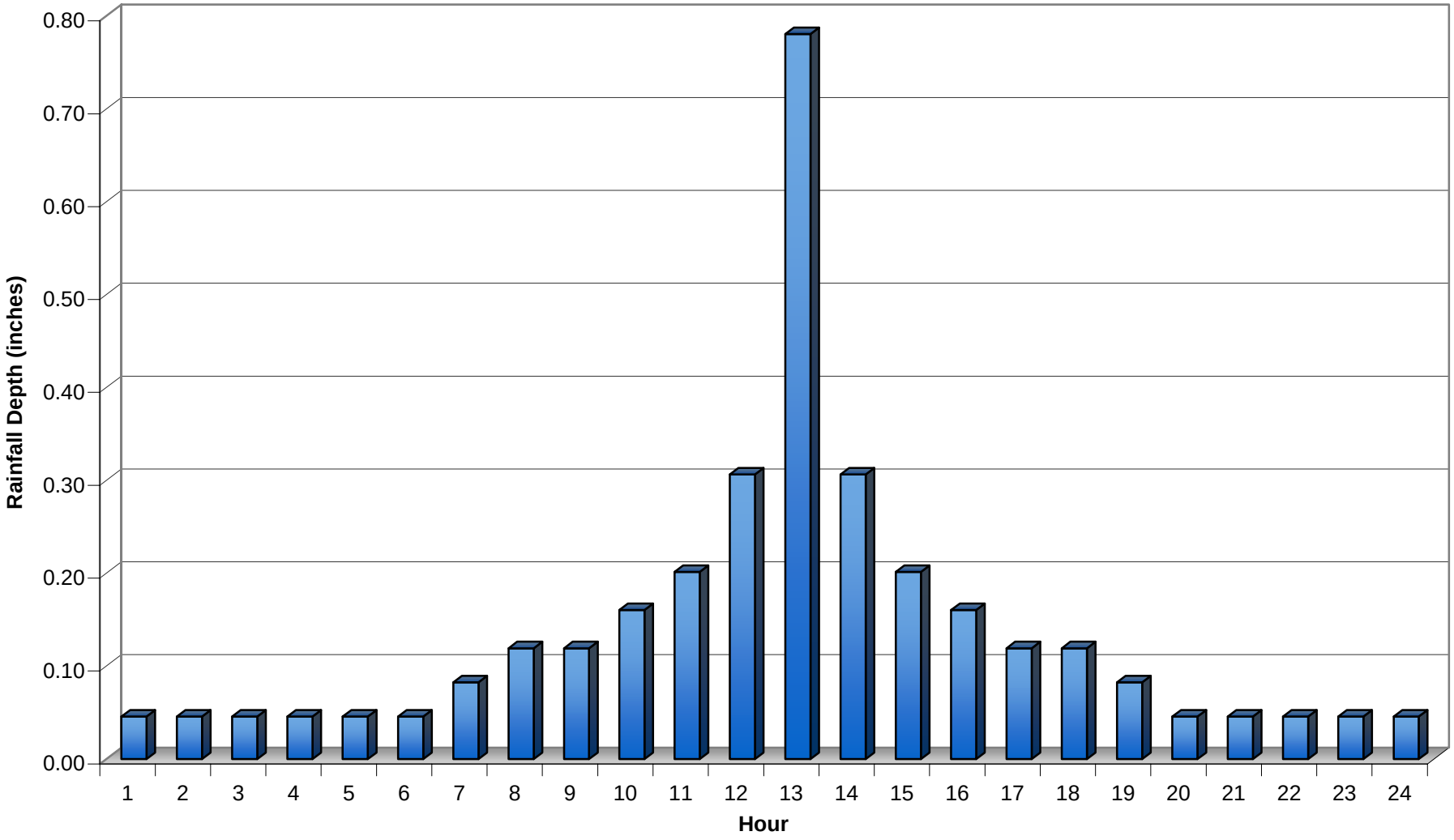


Figure 4.2
10 year 24-Hour Design Storms
Wastewater Collection System Master Plan
City of Galt

carollo
Engineers...Working Wonders With Water™

depth of the pipeline and when the susceptibility of the sanitary sewer pipe allows infiltration through defects such as cracks, misaligned joints and broken pipelines. In the City's sewer service area, dry weather GWI may be considerable because the shallow depth to perched groundwater in some parts of town.

WWF includes storm water inflow, trench infiltration, and GWI. The storm water inflow and trench infiltration comprise the WWF component termed I/I. The response in the sewer system to rainfall is seen immediately (as with inflow) or within hours after the storm (as with infiltration).

The third element of WWF is GWI, which is not specific to a single rainfall event, but rather to the effects on the sewer system over the entire wet weather season. The depth of the groundwater table rising above the pipe invert elevation causes GWI.

Sewer pipes within close proximity to a body of water can be greatly influenced by groundwater effects. As the groundwater table fluctuates over the wet weather season, this fluctuation is seen as a mounding effect in flow monitoring data. A description of each flow component is detailed in the following sections.

4.3.1 Base Wastewater Flow

The base wastewater flow (BWF) is the flow generated by the City's residential, commercial, and industrial customers. The flow has a diurnal pattern that varies with land use categories. Typically, a residential diurnal pattern has two peaks with the more pronounced peak following the wake-up hours of the day, and a less pronounced peak occurring in the evening. In Galt, the more pronounced peak of the diurnal pattern occurred in the evening. Commercial and industrial patterns, though they vary depending on the type of use, typically have more consistent higher flow patterns during business hours, and lower flows at night. Furthermore, the diurnal flow pattern experienced during a weekend may vary from the diurnal flow experienced during a weekday.

4.3.2 Groundwater Infiltration

Ground water infiltration, one of the components of I/I, is associated with extraneous water entering the sewer system through defects in pipes and manholes. GWI is related to the condition of the sewer pipes, manholes, and groundwater levels. GWI may occur throughout the year, although rates are typically higher in the late winter and early spring. Dry weather GWI (or base infiltration) cannot easily be separated from BWF by flow measurement techniques. Therefore, dry weather GWI is typically grouped with BWF.

4.3.3 Average Dry Weather Flow

Average dry weather flow (ADWF) is the average flow that occurs on a daily basis during the dry weather season, defined as June through September for this Master Plan. The

ADWF includes the BWF generated by the City's residential, commercial, and industrial users, plus the dry weather GWI component. See Table 4.3 for a summary of ADWF.

4.3.4 Peak Dry Weather Flow

Peak dry weather flow (PDWF) is the highest observed hourly flow that occurs during the dry weather season. The collection system's ability to convey the PDWF without surcharging is a criterion of this Master Plan. In Galt, the highest dry weather flow typically occurs during the dry weather months occurs in May or June. The PDWF will also occur during one of these two months.

4.3.5 Maximum Day Wet Weather Flow

Maximum day wet weather flow (MDWWF) is the highest daily flow that occurs during the wet weather season (defined as October through April for this Master Plan). The Water Environment Federation (WEF) Manual of Practice FD-6 and the American Society of Civil Engineers (ASCE) Manual and Report on Engineering Practice No. 62 suggest that the MDWWF to ADWF ratio typically ranges between 2 and 3, even in well constructed systems. Higher values usually indicate a more pronounced I/I problem. Review of daily flows in from 2004 to 2008 indicate that the MDWWF occurred on May 23, 2007 and was 4.62 million gallons per day (mgd), which translates to a 2007 MDDWF to ADWF ratio of 2.0 (2007 ADWF of 2.30). The calculated ratio is on the lower end of the typical range.

4.3.6 Peak Wet Weather Flow (Design Flow)

Peak wet weather flow is the highest observed hourly flow that occurs following the design storm event. Wet weather I/I causes flows in the collection system to increase. PWWF is typically used for designing sewers and lift stations. Therefore, PWWF will be referred to as the design flow in this Master Plan. The trunk sewers and lift stations were evaluated based on their capacity to convey the design flow (PWWF). If the sewers violated the flow depth criterion, then they were considered capacity deficient for which improvements were proposed.

The WEF Manual of Practice FD-6 and ASCE Manual No. 62 recommends maintaining design flow to ADWF ratios below 3 to 4, with higher values indicative of pronounced I/I. The WWTP does not measure hourly flow variations, however, based on the modeling results, we were able to derive a peak flow. The model derived peak hourly flow (or model derived design flow) at the WWTP was estimated at 7.01 mgd for existing conditions. Therefore, the PWWF to ADWF factor was 3.0, which is within the recommended range. Peak to average ratios measured at the treatment plant are generally lower than those measured in the collection system, due to flow attenuation. In the downtown area of the City where flows are influenced by storm drain connections, the ratio of PWWF to ADWF ranged between 3 and 5.

.

Table 4.3 Historical Monthly WWTP Influent Flows
Wastewater Collection System Master Plan
City of Galt

Year	Population[1]	Per Capita Flow (gpcd)	Average Day Flow (mgd)	ADWF (mgd)	AWWF (mgd)	MDDWF (mgd)		MDWWF (mgd)	
2004	22,773	90	2.00	1.96	2.03	N/A[2]	N/A[2]	3.95[2]	October
2005	23,003	92	2.10	2.08	2.12	2.86[3]	October	4.62	December
2006	23,235	93	2.17	2.13	2.20	2.51[4]	May	3.75	April
2007	23,470	97	2.27	2.30	2.24	3.09	May	3.68	March
2008	24,474	89	2.19	2.11	2.25	2.78	June	4.60	February
	Average	**92**							

Note:
1. Population for 2007 from 2030 General Plan EIR. Other years interpolated.
2. Daily data prior to October 2004 was not available. MDWWF was for the months of October, November and December 2004.
3. Daily data for July, August and September of 2005 was not available.
4. Daily data for June, July, August and September of 2006 was not available.

4.3.7 Inflow and Infiltration

Infiltration is defined as storm water flows that enter the sewer system by percolating through the soil and then through defects in pipelines, manholes, and joints. Examples of infiltration entry points are cracks in pipelines, misaligned joints, and root penetration. Inflow is defined as storm water that enters the sewer system via a storm drain cross connection to the sewer system. Examples of inflow entry points are roof drain and downspout connections, leaky manhole covers, and illegal storm drain connections.

The adverse effects of I/I entering the sewer system is that it increases both the flow volume and peak flows such that the sewer system is operating at or above its capacity. If too much I/I enters the sewer system, sanitary sewer overflows (SSO's) could occur. The modeling results indicate that the City's collection system does have infiltration.

The flow monitoring data from Meter 5 indicates a rapid increase in sewer flow volume and a peak concentration that typically occurs after a storm event if the system has storm drain cross connections. For example, the level of flow at Meter 5 surcharged 19.5-inches above the pipe crown after a storm event, where other areas of the City measured considerably less increase in flow. These cross connections create considerable inflow during storm events that use capacity within the existing system that may cause SSO's. These deficiencies will require extensive capital projects to mitigate. The City should consider implementing an I/I study to locate the main sources of runoff into the collection system. Disconnecting these storm drain cross connections will result in a decrease in flow and reduce the threat of SSOs.

4.4 WASTEWATER TREATMENT PLANT FLOWS

In addition to the flow-monitoring program, this project reviewed historical daily WWTP influent flow data since January 2004 to establish wastewater flow criteria.

Flow data from January 2004 through December 2008 are summarized in Table 4.3. Table 4.3 lists for each year, the following parameters:

- Population and per capita flow.
- Total annual flow.
- Average day flow (ADF): Total annual flow divided by the number of days in the year. Average flow entering the plant over the entire year, without consideration of season (dry or wet).
- Average dry weather flow (ADWF): Average WWTP influent flow during the months of June through September.
- Average wet weather flow (AWWF): Average WWTP influent flow during the months of January through May, and October through December.

- Maximum day dry weather flow (MDDWF): Average day flow for the highest dry weather day of a given year.
- Average day maximum wet weather flow (MDWWF): Average day flow for the wet weather months of a given year.

As shown in Table 4.3, the ADF in 2007 was 2.27 mgd, the ADWF was 2.30 mgd, and the AWWF was 2.24 mgd. When this project started in August 2008, the last year's worth of completed data was for 2007. Even though data for 2008 became available, the base year for analysis and future projections was 2007. This is consistent with the approach used in calculating water demands in the Water Distribution System Master Plan. Typically, the wet weather season contributes higher than normal sewer flows due to rainfall dependent I/I following storms. However, from 2004 through 2008, the dry weather averages were consistent with the wet weather averages. For this study, since there is little difference between average day, average dry and average wet weather flows, the average day flow was used in calculations.

For 2007 average day flow of approximately 2.3 mgd and sewer service area population was 23,470, the calculated gallons per capita per day (gpcd) equal 97. The average per capita flow from 2004 through 2008 is 93 gpcd. In comparison, the City's water use for 2007 averaged approximately 5.5 mgd (~6,200 acre-feet per year [AFY]), and the per capita water use equaled 236 gpcd. Therefore, the City's wastewater generation represents about 41 percent of the City's total water use

4.5 WASTEWATER FLOW PROJECTIONS

For sewer system master plans, developing relationships between land use and flow is the most common method employed to develop average wastewater flow predictions. The flow planning criteria are typically based on standard practice values, but then refined based on actual flow data. The land use flow coefficients were established to estimate average day flow within the study area.

4.5.1 Existing Wastewater Flow Coefficients

Average wastewater flow coefficients are rates, usually expressed in gallons per day per acre (gpda), applied to either gross or net acres for calculating average day flow generated from a particular land use. A flow coefficient was developed for each land use classification discussed previously. The flow coefficient provides a means to transform a land use category from acreage into wastewater flow. The resulting flow is then input into the appropriate sewer area in the sewer system model. Wastewater flow coefficients for residential areas can range between 1,000 to 4,000 gpda, and commercial and industrial areas might range from 500 to 2,500 gpda, with typical values averaging approximately 800 to 1,000 gpda. Land uses designated as open space and agriculture are assumed to generate negligible amounts of sewage flow.

The coefficients were estimated by evaluating the flows measured from the flow-monitoring program and analyzing the tributary service area generating flow to each meter. For example, for Flow Meter No. 5, the average day flow during the flow-monitoring period was approximately 0.25 mgd. The tributary service area upstream of this monitoring station is developed with buildings falling into one of the following land use categories (excluding undeveloped land): low density residential, public/quasi public, commercial and other non-wastewater generating lands (e.g. roads and parks). Flow coefficients were multiplied by the acreages for each land use designation until the flows equaled 0.25 mgd. This process was repeated for each flow meter.

The flow coefficients were then adjusted to balance the calculated flows for the entire developed sewer service area to 2.3 mgd, which was the 2007 average day flow. The average day flow is the basis for calibrating the average sewer flow coefficients. Table 4.4 presents the wastewater flow coefficients calculated from the flow monitoring program and the balancing to match 2.3 mgd.

As with most cities throughout California, residential land use makes up the majority of developed land and wastewater flow. For Galt, residential customers make up approximately 86 percent of the current flow.

4.5.2 Existing and Future (Build-out) Average Day Flow

Developing an accurate estimate of the quantity of wastewater is an important step in maintaining and sizing sewer system facilities, for both existing conditions and future developments. The future average day flow was generated in a similar manner as the existing average day flow. Future flow projections were developed based on the land use projections as described in the City's general plan.

As previously reported, build-out of the General Plan boundary will nearly triple the current developed acreage. The build-out sewer flows were calculated by multiplying each land use area by its corresponding flow coefficient. The resulting flow projections are summarized in Table 4.5. Per City direction, the flows summarized in Table 4.5 assume that undeveloped areas within the 100-year floodplain will not be developed in the future. However, floodplain areas within the City limits that are currently developed will remain developed. In addition, the flow projections also assume that approximately 20 percent of the future developed land use area will consist of roads, railroads, and canals, which do not generate wastewater. Twenty percent was assumed because based on Carollo's experience with previous projects. Calculations by others that use the coefficients in Table 4.5 should be aware of the percent of land assumed for roads and other right-of-ways.

A summary of the existing and future average day flows is presented in Table 4.6. If the City achieves build-out of the General Plan boundary by 2030, then wastewater flow could increase at an annual rate of 2.6 to 4.3 percent between now and build-out of the study area. Based on these projections, it is anticipated that the City build-out ADF will approach 5.6 mgd.

Table 4.4 Existing Wastewater Flow Coefficients
Wastewater Collection System Master Plan
City of Galt

Land Use Designation	2007 Wastewater Service Area			
	Developed [1] (acres)	Sewer Coefficient (gpda)	Average Day Flow (gpd)	Percent of Total Flow (%)
Residential				
Rural Residential (A-RR)	0.1	150	0	0
Residential Estates (K-RE)	0	300	0	0
Low Density Residential (B-LDR)	990	1,385	1,371,000	59
Medium Density Residential (C-MDR)	208	1,900	396,000	17
Medium High Density Residential (D-HDR)	0	2,200	0	0
High Density Residential (D-HDR)	101	2,300	232,000	10
				86
Employment Related				
Commercial (E-C)	101	800	81,000	4
Office Professional (F-OP)	3	800	2,000	0
Light Industrial (G-LI)	120	800	96,000	4
				8
Others				
Public/Quasi-Public (H-PQP)	162	800	130,000	6
Open Space (I-OS)	11	0	0	0
Park (I-PK)	70	0	0	0
Mixed Use (M-MU)	4	800	3,000	0
Roads/Railroads/Canals	702	0	0	0
WWTP	293	0	0	0
				6
Total	**2,766**		**2,311,000**	**100**

Notes:
1. Area totals exclude roads, railroads, and canals, which account for approximately 24 percent of the developed City limits.

Table 4.5 General Plan Build-out Flow Projections
Wastewater Collection System Master Plan
City of Galt

Land Use Designation	Projected Build-out of General Plan Boundary				
	Area Outside 100-Yr Floodplain [1] (acres)	**Flow Coefficient[2] (gpd/acre)**	**Average Flow (gpd)**	**Average Flow (mgd)**	**Percent of Total Flow**
Residential					
Rural Residential (A-RR)	1,236	Note 3			
Residential Estates (K-RE)	172	300	51,000	0.1	1%
Low Density Residential (B-LDR)	2,200	1,385	3,047,000	3.0	54%
Medium Density Residential (C-MDR)	321	1,900	609,000	0.6	11%
Medium High Density Residential (L-MHD)	63	2,200	138,000	0.1	2%
High Density Residential (D-HDR)	170	2,300	391,000	0.4	7%
Employment Related					
Commercial (E-C)	616	800	491,000	0.5	9%
Office Professional (F-OP)	179	800	143,000	0.1	3%
Light Industrial (G-LI)	528	800	420,000	0.4	7%
Others					
Public/Quasi-Public (H-PQP)	419	800	335,000	0.3	6%
Open Space (I-OS)	78	0	0	0.0	0%
Park (I-PK)	186	0	0	0.0	0%
Meadowview Park	4	0	0	0.0	0%
Mixed Use (M-MU)	19	800	15,000	0.0	0%
Roads/Railroads/Canals	1,572	0	0	0.0	0%
WWTP [2]	296	0	0	0.0	0%
Total	**8,059**		**5,645,000**	**5.6**	**100%**

Notes:
1. Adjustments to the acreage totals for the land uses were made in order to maintain a Roads/Railroads/Canals sub-total that is approximately 20 percent of the total.
2. Included with Public/Quasi-Public total land use in General Plan.
3. Per the General Plan, Rural Residential designated lands are not anticipated to be annexed into the City limits during this General Plan.

Table 4.6 Average Day Flow Summary Wastewater Collection System Master Plan City of Galt	
Planning Year	**WWTP Average Day Flow (mgd)**
Existing [1]	2.30
General Plan Build-out [2]	5.60

Notes:
1. Based on City's WWTP 2007 flow data.
2. Based on land use and acreage from the City's 2030 General Plan.

To validate the results generated using land use information, the sewer flows were projected using the population method. Based on a projected 2030 population of 50,094 and an average per capita flow of 97 gpcd (average for 2007), the projected average day flow could approach 4.9 mgd. The population method for calculating wastewater flows results in a slightly lower projection when compared to the land use method. One reason for the lower estimate using the population approach is that employment related lands (e.g. commercial, office, and industrial) will make up a larger percentage of the land use and wastewater flow in the future. For example, employment related lands currently make up eight percent of the total developed land and had a year 2007 flow of 0.18 mgd. At build-out, their percentage of the developed land doubles to 16 and their projected flows increase to 1.05 mgd. The wastewater flows generated by these employment related lands will increase the future per capita flow, which averaged 92 gpcd over the last five years. Currently, many local residents commute to areas outside of the City for work. Therefore, using land use to calculate demands provides a more accurate estimate when compared to population.

4.5.3 Growth Horizon

The discussion up to this point has focused on the flow projection when the General Plan boundary is fully built out. This Master Plan assumes that the study area will be completely built out by year 2030, with the exception of undeveloped areas within the 100-year floodplain boundary, which are assumed to be undeveloped through build-out. Based on this assumption and using a rate of increase consistent with the General Plan population increase, Table 4.7 provides wastewater flow projections through 2030.

Table 4.7 Wastewater Flow Projections Wastewater Collection System Master Plan City of Galt	
Planning Year	**Average Day Flow [1] (mgd)**
2007	2.30
2010	2.70
2015	3.50
2020	4.16
2025	4.90
2030	5.60

Notes:
1. Assumes build-out of the study area by year 2030, except for undeveloped areas located within the 100-year floodplain.

4.6 DESIGN FLOWS

The design flow is the maximum hourly flow rate under selected design storm and growth conditions. The design flow includes the average daily flow and the peak I/I rate. Typically, a design storm is routed through a hydraulic model to estimate the PWWF in a sewer system and to quantify the system's capacity. This Master Plan utilized the 10-year, 24-hour design storm rainfall pattern for generating the design flow in the sewer system ("design flow" is synonymous to peak wet weather flow). The 10-year, 24-hour storm was derived based on the methods described in Volume 2 of the Sacramento County Drainage Manual.

To predict the peak I/I rate in the collection system at build-out of the General Plan boundary, the I/I parameters from tributaries of the existing collection system that would best represent future developments were used. For this Master Plan, the northeast area of the City tributary to Flow Meter 3 was assumed to represent new development. Based on this assumption, the I/I parameters measured from Flow Meter 3 were assigned to future development within the General Plan boundary and input into the hydraulic model. The hydraulic model was then used to simulate the design flow for the buildout condition. Table 4.8 summarizes the design flow for each flow meter tributary area, the peaking factors and the peak I/I rate associated with the design storm event.

Table 4.8	Peaking Factor and Peak I/I Summary Wastewater Collection System Master Plan City of Galt			
Meter Site	**Average Day Flow[1] (mgd)**	**Design Flow (mgd)**	**Design Flow: Average Day Peaking Factor**	**Maximum I/I (gpad)**
1	1.04	3.16	3.03	1,412
2	0.31	0.67	2.16	1,167
3	0.81	2.12	2.61	1,563
4	0.54	1.77	3.27	1,330
5	0.24	0.98	4.08	1,200
6	0.28	0.64	2.28	2,319

Notes:
1. Based on meter data from Sanitary Sewer Flow Monitoring, December 2008.

In summary, the design flow consists of two components:

- Average day flow
- Wet weather inflow and infiltration

Table 4.9 presents the current average day and design flow for the service area. Also shown are the forecast average day and design flow at build-out of the City's General Plan boundary.

Table 4.9	Existing and Projected Wastewater Flows Wastewater Collection System Master Plan City of Galt		
	Average Day Flow[2] (mgd)	**Design Flow (mgd)**	**Peaking Factor**
Existing [1]	2.30	7.01	3.0
General Plan Build-out [2]	5.60	14.45	2.6

Notes:
1. Based on year 2007 flows.
2. Based on land use and acreage from the City's General Plan and build-out of all land within the General Plan 2030 boundary, excluding undeveloped land within the 100-year floodplain.

4.7 PLANNING CRITERIA SUMMARY

The recommended planning criteria for this Master Plan are summarized in Table 4.10.

Table 4.10 Planning Criteria Summary
Wastewater Collection System Master Plan
City of Galt

Minimum Slopes for New Pipes

Pipe Size (inches)	Minimum Slope[1] (ft/ft)	Pipe Capacity d/D	Pipe Capacity Maximum Flow (mgd)
8	0.0035[2]	0.5	0.23
10	0.0025[2]	0.5	0.35
12	0.0020[2]	0.67	0.81
15	0.0015[2]	0.67	1.28
18	0.0012[2]	0.67	1.86
21	0.0009	0.75	2.80
24	0.0008	0.75	3.77
27	0.0006	0.75	4.83
30	0.0006	0.75	5.92
36	0.0006	0.75	9.63
42	0.0006	0.75	14.53
48	0.0006	0.75	20.74

Notes:
1. Recommended minimum slope for maximum pipe flow at various d/D values and velocity greater than or equal to 2 feet/second.
2. Sacramento Area Sewer District Standards.

Maximum Flow Depth, d/D

The following flow depth criteria was used in the analysis:

Maximum d/D for Existing Sewers

Dry Weather Flows: 0.92

Peak Wet Weather Flow: Pipes will be allowed to surcharge 3 feet below manhole rim

Maximum d/D for Planning New Sewers

Pipe Diameter (inches)	Maximum d/D Ratio (during Peak Flows)
Less than 12	0.50
12 to 18	0.67
Larger than 18	0.75

Headloss in Existing Pipes

Headloss in existing sewer pipes shall be calculated based on the following:

Gravity Pipes Manning's n = 0.013

Pressure Pipes Hazen Willam's C = 120

Changes in Pipe Size

When a smaller sewer joins a larger one, sewer crowns will be matched.

Average Sewer Flow Coefficients

Land Use Category	Coefficients (gpd/acre)
Rural Residential (A-RR)	(Note 1)
Residential Estates (K-RE)	300
Low Density Residential (B-LDR)	1,385
Medium Density Residential (C-MDR)	1,900
Medium High Density Residential (L-MHD)	2,200
High Density (Residential (D-HDR)	2,300
Commercial (E-C)	800
Office Professional (F-OP)	800
Light Industrial (G-LI)	800
Public/Quasi-Public (H-PQP)	800
Open Space (I-OS)	0
Park (I-PK)	0
Mixed Use (M-MU)	800

Note:

1. Per the General Plan, Rural Residential designated lands are not anticipated to be annexed into the City limits during this General Plan.

COLLECTION SYSTEM FACILITIES AND HYDRAULIC MODEL

This chapter describes the development and calibration of the City of Galt (City) wastewater collection system hydraulic model. This model was used to identify the sewer system's capacity deficiencies and to develop improvements to correct those deficiencies.

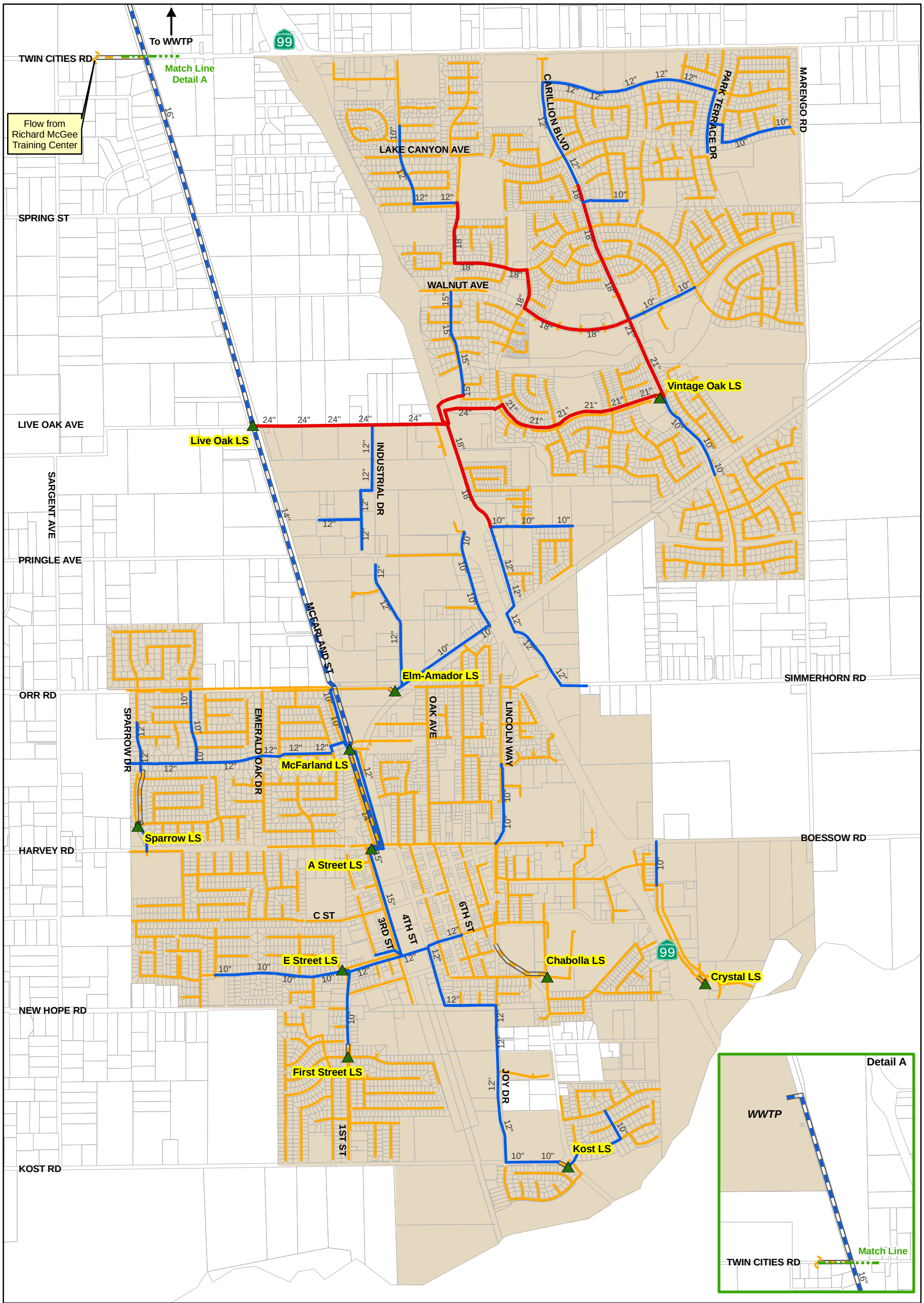
5.1 WASTEWATER SERVICE AREA OVERVIEW

The City's collection system consists of sewer mains and trunk sewers that collect and convey wastewater to the City's wastewater treatment plant (WWTP). All wastewater is collected and conveyed to the Live Oak Pump Station located at Live Oak Avenue and McFarland Street. Flow from the Live Oak Pump Station is conveyed to the WWTP in a 16-inch diameter forcemain that runs parallel to the railroad. The City's system is broken up into two main tributaries. The northeast tributary flows directly to the Live Oak Pump Station, while sewers in the south flow to either the McFarland Pump Station or the A-Street Lift Station. The forcemains from A Street and McFarland pump stations combine into a single 14-inch diameter forcemain that conveys flow to the Live Oak Pump Station wet well. Figure 5.1 shows the existing sewer collection system, including sewer diameters and lift station locations. More detail on the City's trunk sewers is provided below. Table 5.1 presents a summary by diameter of the known sewers in the collection system. In total, there are approximately 79 miles of sewer mains and trunk sewers in the City's collection system.

Table 5.1 Wastewater Collection System Pipeline Summary
Wastewater Collection System Master Plan
City of Galt

Diameter (inch)	Length (feet)	Diameter (inch)	Length (feet)
4	1,481	15	3,970
6	130,967	16	11,143
8	190,801	18	10,153
10	28,198	21	4,870
12	24,822	24	4,383
14	7,431	**Total**	**418,219**

Source: City provided AutoCAD data.



Legend

Existing Wastewater Collection System

- Lift Station

Sewer Pipeline Diameter

Gravity Main

- 8" and Smaller
- 10" - 16"
- 21" and Larger

Force Main

- 8" and Smaller
- 10" - 16"

- Parcels
- City Limits

N

0 1,250 2,500 Feet

Figure 5.1
Existing Wastewater Collection System
Wastewater Collection System Master Plan
City of Galt

5.2 MODELED SEWER SYSTEM

The modeled sewer system consists of approximately 22 miles of sanitary sewer lines ranging in diameter from 4-inches to 24-inches, and 11 lift/pump stations. Figure 5.2 presents the City's modeled wastewater collection system. The larger trunk sewers range in diameter from 10-inches to 24-inches. Table 5.2 presents a summary of the modeled sewer system by diameter and length of pipe. Not included in these totals are the smaller sewer mains that were not modeled (typically 8-inch diameter and smaller).

Table 5.2 Modeled Sewer System Pipeline Summary
Sewer System Master Plan
City of Galt

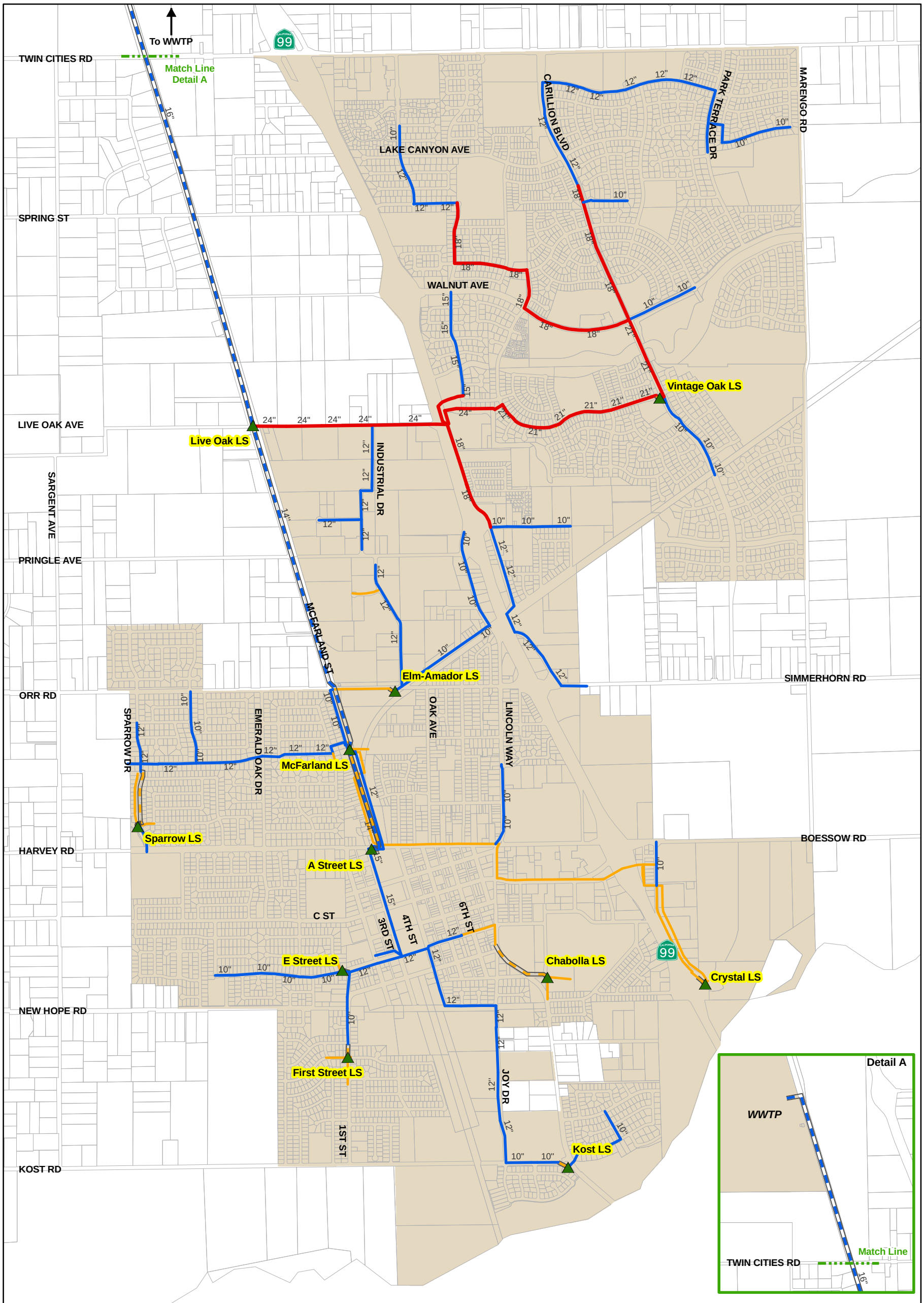
Diameter (inch)	Length (feet)	Diameter (inch)	Length (feet)
4	1,188	15	3,970
6	4,320	16	11,143
8	19,056	18	10,153
10	28,198	21	4,870
12	24,822	24	4,383
14	7,431	**Total**	**119,534**

Source: City provided AutoCAD data.

5.2.1 Interceptors and Major Trunk Sewers

5.2.1.1 Live Oak/Vintage Oak Trunk Sewer

The Live Oak/Vintage Oak Trunk Sewer conveys flow from approximately half of the City. It starts at Vintage Oak Avenue and Carillion Boulevard at the discharge of the Vintage Oak Lift Station as a 21-inch diameter pipeline and continues west to Live Oak Avenue where it increases to a 24-inch diameter pipeline and discharges to the wet well at the Live Oak Pump Station. The sewer was constructed of reinforced concrete pipe (RCP) material in the early 1990's as part of the Northeast Community Service District.



Legend

Modeled Wastewater Collection System

- Lift Station

Sewer Pipeline Diameter

Gravity Main

- 8" and Smaller
- 10" - 16"
- 21" and Larger

Force Main

- 8" and Smaller
- 10" - 16"

- Parcels
- City Limits

N

0 1,250 2,500 Feet

Figure 5.2
Modeled Wastewater Collection System
Wastewater Collection System Master Plan
City of Galt

GALT

carollo

5.2.1.2 Carillion Trunk Sewer

The Carillon Trunk is a 21,18 and 12-inch diameter sewer that conveys flow from residential and commercial users north of Deadman's Gulch in the northeast part of the City. It conveys flow to the Vintage Oak Lift Station, and intercepts flow from the Walnut Trunk Sewer at the intersection of Walnut Avenue and Carillion Boulevard. The sewer was constructed in the early 1990's as a part of the Northeast Area Community Service District. The 21 and 18-inch diameter segments are constructed of RCP, while the 12-inch diameter segments are vitrified clay pipe (VCP).

5.2.1.3 Walnut Trunk Sewer

The 15 and 12-inch diameter Walnut Trunk Sewer conveys flow from the northern part of the City, between Highway 99 and Carillion Boulevard, to the Carillion Trunk Sewer. The Walnut Trunk Sewer runs primarily west to east in Walnut Avenue and Elk Hills Drive before discharging into the Carillion Trunk Sewer near the intersection of Walnut Avenue and Carillion Boulevard.

5.2.1.4 Stockton/Carol Trunk Sewer

The 18 and 12-inch diameter Stockton/Carol Trunk Sewer conveys flow from the area just east of Highway 99, between Live Oak Avenue and Simmerhorn Road. This sewer runs primarily south to north and discharges into the Live Oak/Vintage Oak Trunk Sewer.

5.2.1.5 Quail Hollow Trunk Sewer

The 12-inch diameter Quail Hollow Trunk Sewer conveys flow from west to east for the entire length of Quail Hollow Drive, from Sparrow Drive to the McFarland Pump Station. This trunk sewer serves the entire area from Elm Avenue to A Street, including some residential areas south of A Street.

5.2.1.6 3rd Street Trunk Sewer

The 3rd Street Trunk Sewer serves the portion of the City south of A Street and conveys flow to the A Street Pump Station. The 15 and 12-inch diameter trunk sewer runs from south to north from the Kost Lift Station to the A Street Pump Station. The E Street, First Street, Kost and Chabolla Lift Stations are all tributary to this trunk sewer.

5.2.2 Lift Stations and Forcemains

There are 12 lift/pump stations in the collection system, 11 of which were included in the hydraulic model. The general locations of each lift station are shown on Figure 5.1. Table 5.3 summarizes the design data for the 11 lift stations.

Table 5.3 Lift Station Design Data
Wastewater Collection System Master Plan
City of Galt

Lift Station	Pump Design Data					
	Pump No.	Pump Capacity (gpm)	Firm Capacity (gpm)	Total Capacity (gpm)	Total Discharge Head (ft)	Horsepower (hp)
A Street	1	1,800	1,800	3,600	--(1)	40
	2	1,800			--(1)	40
Chabolla	1	300	300	600	--(1)	3
	2	300			--(1)	3
Crystal	1	300	300	600	--(1)	3
	2	300			--(1)	3
E Street	1	500	500	1,000	--(1)	53
	2	500			--(1)	53
Elm/Almador	1	500	500	1,000	--(1)	6
	2	500			--(1)	3
First Street	1	250	250	500	--(1)	7.5
	2	250			--(1)	5
Kost	1	380	380	760	--(1)	5
	2	380			--(1)	5
Live Oak	1	3,000	3,000	6,000	70	77
	2	3,000			70	77
McFarland	1	1,000	1,000	2,000	80	20
	2	1,000			80	20
Sparrow	1	700	700	1,400	--(1)	15
	2	700			--(1)	15
Vintage Oak	1	1,500	1,500	3,000	33	18
	2	1,500			33	18

Notes:
(1) TDH data unavailable.

There are three primary pump stations that convey all the flow in the City to the WWTP. Flow from the A Street Pump Station and the McFarland Pump Station join in a common 14-inch diameter forcemain that conveys flow north in McFarland Street to the Live Oak

Pump Station. These two pump stations convey flow from the entire southern portion of the City, south of Pringle Avenue and west of Highway 99. From the Live Oak Pump Station, wastewater from the entire City is pumped over two miles to the WWTP in a 16-inch diameter forcemain.

5.3 SEWER SYSTEM HYDRAULIC MODEL

A wastewater collection system model is a simplified representation of the real sewer system. Sewer system models can assess the conveyance capacity for a collection system. Also, sewer system models can perform “what if” scenarios to assess the impacts of future developments and land use changes.

The City’s hydraulic model combines information on the physical and operational characteristics of the wastewater system, and performs calculations to solve a series of mathematical equations to simulate flows in pipes. The hydraulic model was developed based on data collected from the City’s AutoCAD database, developer design drawings, lift station design summaries, and field surveys.

Elements comprising the computer modeling process are: skeletonizing the sewer system, defining pipes and nodes, and identifying the service areas. Skeletonizing is the process by which collection systems are stripped of pipelines not considered essential for the intended analysis purpose. The sewer system model generally includes pipelines with a diameter of 10-inches or greater, all associated manholes, and lift stations. In some instances, 8-inch diameter pipelines and smaller were included in the model to further define a specific area of interest or to maintain connectivity in the system. Modeling sewers that are 10-inch diameter and greater serves the purpose of minimizing model analysis run time while retaining the hydraulic integrity of the collection system as a whole.

Computer modeling requires gathering information of the sewer system, such as pipe diameters, lengths; invert elevations at the upstream and downstream manholes, pipe slope, manhole rim elevations, and wet well dimensions.

5.3.1 Quantifying Wastewater Flow

Determining the quantity of wastewater flow generated by a City and loaded into a hydraulic model is typically based on land use designations, flow coefficients, and land use area. The land use type will affect the volume and diurnal flow pattern of wastewater being generated from the study area. Adequately estimating this volume of wastewater is an important process in maintaining and sizing sewer system facilities, both for present conditions and future developments.

Land use data provided by the City, wastewater flow coefficients discussed in Chapter 4, and customer billing records were the basis for estimating wastewater loads input into the hydraulic model. The flow coefficients and billing records provided a means to transform a specific land use category into an average day flow. The loads were calculated in the

geographic information systems (GIS) software program by multiplying the flow coefficient by the land use acreage. The model's load allocation assigned the calculated average day flow to an appropriate sewershed and corresponding node in the sewer system model.

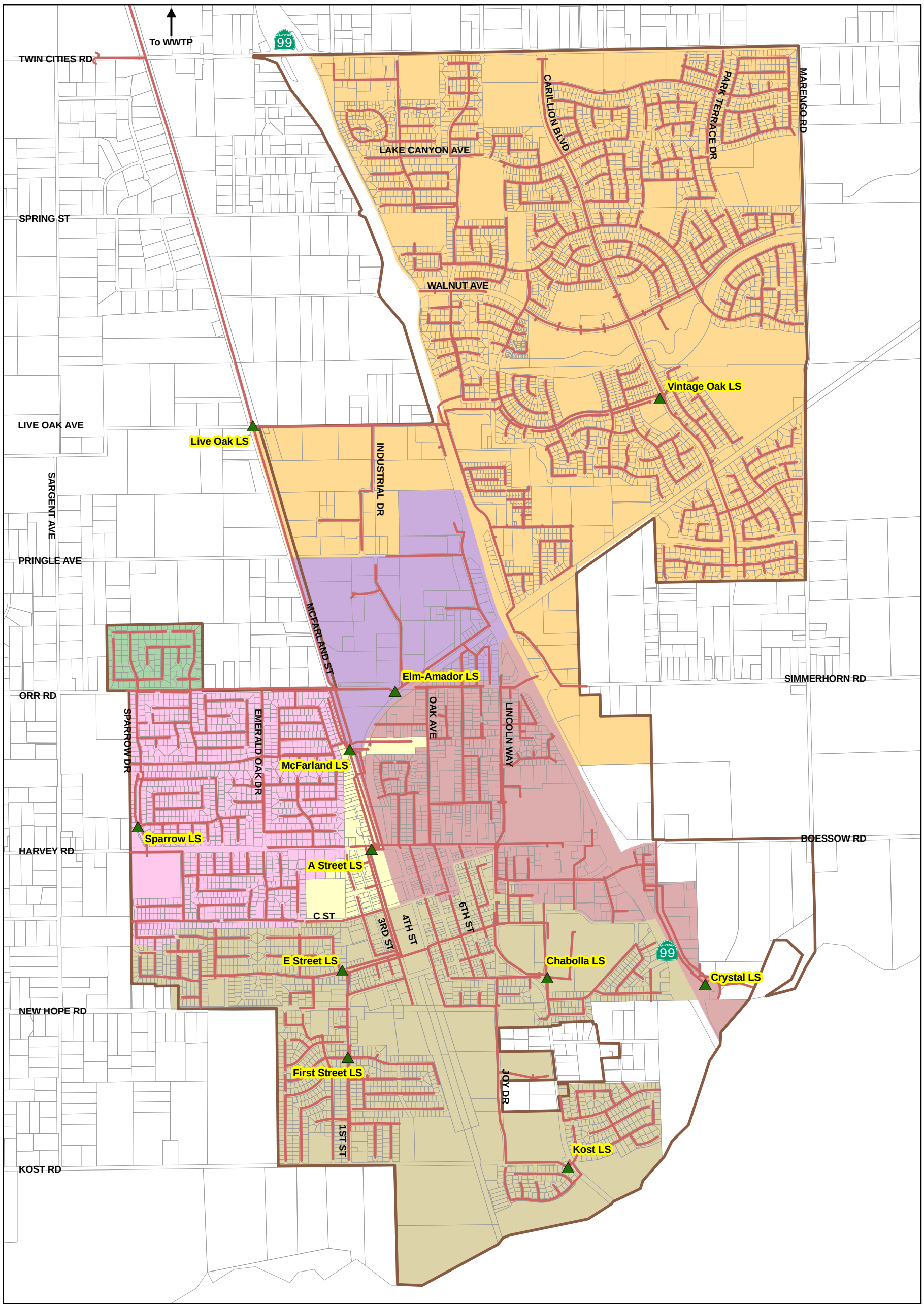
5.3.2 Sewer Tributary Areas (Existing and Future)

The City's existing and future sewer service area were divided into tributary areas. Each of these tributary areas was further broken down into "sewersheds" to facilitate the assignment of wastewater flow to appropriate sewers. A sewershed is a geographic area within the sewer system where wastewater generated in the area will be injected into a single node in the model. Usually a sewershed will encompass a particular subdivision or grouping of lots.

Figure 5.3 presents the major sewer tributaries for the existing collection system. The tributaries represent the assembly of smaller sewersheds into larger tributaries. As shown in the figure, the Live Oak tributary is the largest of the major tributaries and collects flow from several trunk sewers throughout the northeast part of the City. The other sewer tributaries are named after the major trunk sewer that conveys flow within the tributary. For example, all flow generated within the 3rd Street sewer tributary will be conveyed by the 3rd Street Trunk Sewer.

Figure 5.3 is useful because City staff can quickly inspect the figure and determine where a proposed development's flow will discharge. Although Figure 5.3 represents the existing collection system and is useful for infill development within the current City limits, as the City continues to grow through build-out of the sphere of influence (SOI), the tributaries and flow routing scheme will shift. Figure 5.4 illustrates the future sewer tributaries through build-out of the SOI and the anticipated shift in flow routing. As shown in Figure 5.4, some of the future flows in the southern part of town were routed to existing sewers. However, a majority of future development was routed to new sewers.

As shown in Figure 5.4, as the City expands beyond the current City limits, new trunk sewers will convey future flow, including the Marengo, and Simmerhorn Trunk Sewers. The Marengo Trunk Sewer and tributary are located in the northeast perimeter of the study area. The southwest part of the City (south of the railroad and east of Highway 99) will be conveyed in the proposed Simmerhorn Trunk Sewer. All existing and future flows south of Twin Cities Road will be conveyed in the proposed Midway Trunk Sewer. More detailed discussion of proposed future trunk sewers and their tributaries is provided in Chapter 6.



Legend

Existing Wastewater Collection System

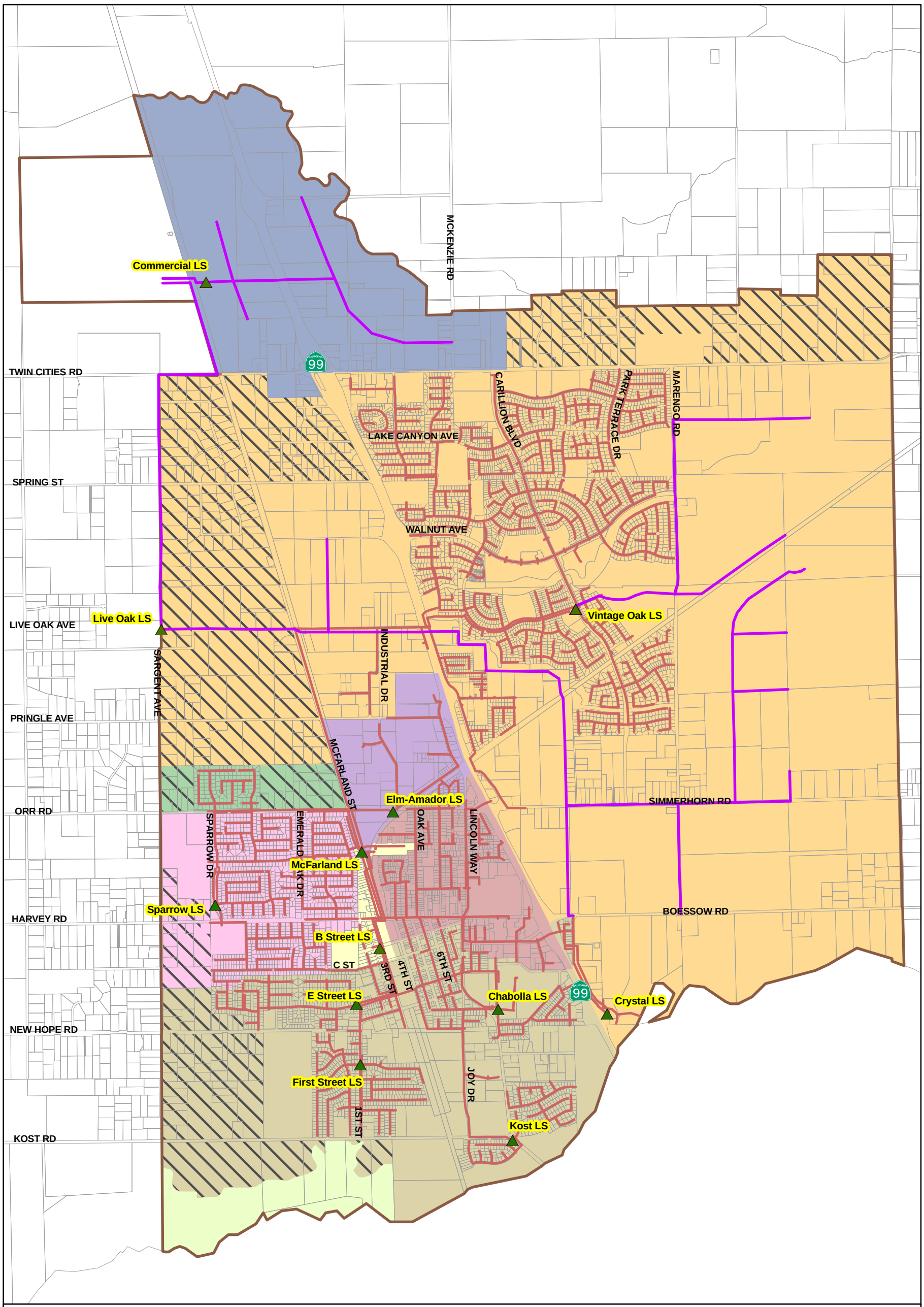
- Lift Station
- Sewer Pipelines
- City Limits
- Parcels

Tributary Areas

- 3rd Street Sewer
- A Street Sewer
- Elm-Amador Lift Station
- Live Oak/Vintage Oak Sewer
- McFarland Lift Station
- Orr Sewer
- Quail Hollow Sewer

0 1,250 2,500 Feet

Figure 5.3
Existing Sewer System Tributaries
Wastewater Collection System Master Plan
City of Galt



Legend

Proposed Wastewater Collection System

- Lift Station
- Existing Sewer
- Future Sewer
- General Plan Boundary
- Parcels

Tributary Areas

- 3rd Street Sewer
- A Street Sewer
- Commercial
- Elm-Amador Lift Station
- Live Oak/Vintage Oak Sewer
- McFarland Lift Station
- Orr Sewer
- Quail Hollow Sewer
- Parks/Open Space
- Rural Residential

Feet
0 1,250 2,500

Figure 5.4
Future Sewer System Tributaries
Wastewater Collection System Master Plan
City of Galt

5.3.3 Model Calibration

Model calibration is a crucial component of the hydraulic modeling effort. Calibrating the model to match data collected during the flow-monitoring program ensures the most accurate results possible. The calibration process consists of calibrating to both dry and wet weather conditions. For this project, both dry and wet weather flow monitoring were conducted. Dry weather flow (DWF) calibration ensures an accurate depiction of base wastewater flow generated within the study area. The wet weather flow (WWF) calibration consists of calibrating the hydraulic model to a specific storm event to quantify the peak and volume of infiltration/inflow (I/I) into the sewer system. The amount of I/I is essentially the difference between the WWF and DWF components.

5.3.3.1 Average Day Flow Calibration

The average day flow calibration consists of several elements: 1) dividing the sewer system into areas tributary to each of the flow meter stations; 2) defining the flow volumes within each area; and 3) creating diurnal patterns to match the temporal distribution of flow. The diurnal curve is the variation of flow throughout the day.

The first step in the calibration process was to divide the City's service area into flow meter tributary areas. Six tributary areas were created, one for each flow meter. The next step was to define the flow volumes within each area, which was accomplished in the flow-loading step discussed above. Diurnal curves based on the flow monitoring data were created for nodes tributary to a specific flow meter. Usually, the peak in a diurnal curve will occur in the morning, between 7 a.m. and 10 a.m. and again in the evening between 5 p.m. and 8 p.m. Figure 5.5 displays the diurnal curve for the area tributary to Flow Meter 3, which measured flows from a residential area in the northeastern part of the City. Similar diurnal curves were developed for each of the remaining meters and their tributary areas. These additional curves are available in Appendix C.

The calibration process compared the meter data with the model output. Comparisons were made for minimum, maximum, and average flows as well as the temporal distribution of flow. Table 5.4 summarizes the ADF calibration using minimum, maximum, and average flow results. A sample of the ADF calibration for Flow Meter 3 is presented in Figure 5.6. This figure shows the measured flow at the meter versus the model predicted flows over a 24-hour period. The remaining DWF calibration plots are provided in Appendix D. As shown in Appendix D, the correlation between the modeled and the measured results is good.

5.3.4 Wet Weather Flow Calibration

The WWF calibration enables the hydraulic model to accurately measure I/I entering a sewer system during a large storm. WWF calibration consists of two steps: 1) determining a rainfall event that characterizes the most significant impact on the sewer system facilities, preferably during wet antecedent soil moisture conditions; and 2) creating a database of I/I

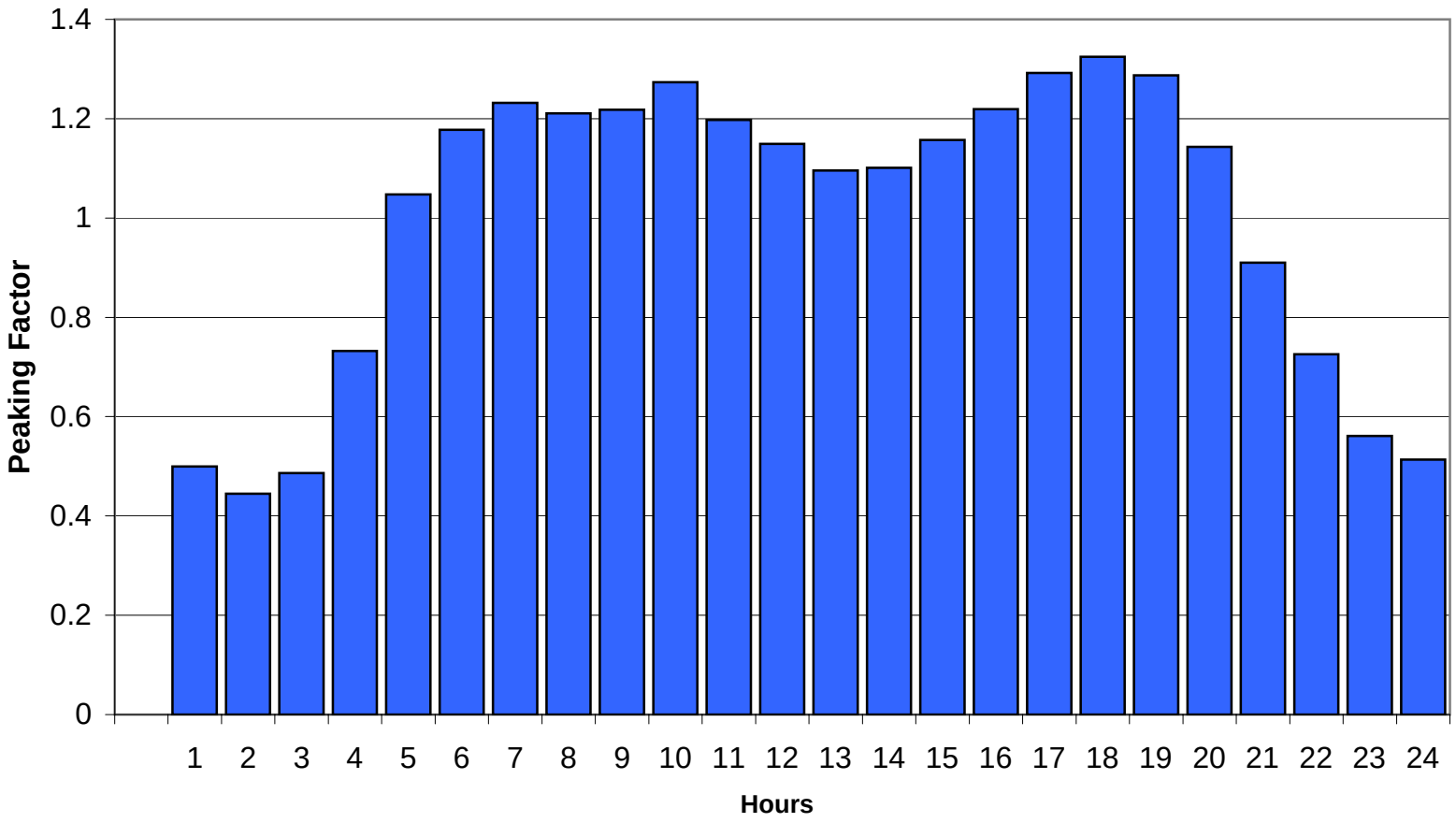


Figure 5.5
Flow Meter 4 Diurnal Pattern
Wastewater Collection System Master Plan
City of Galt

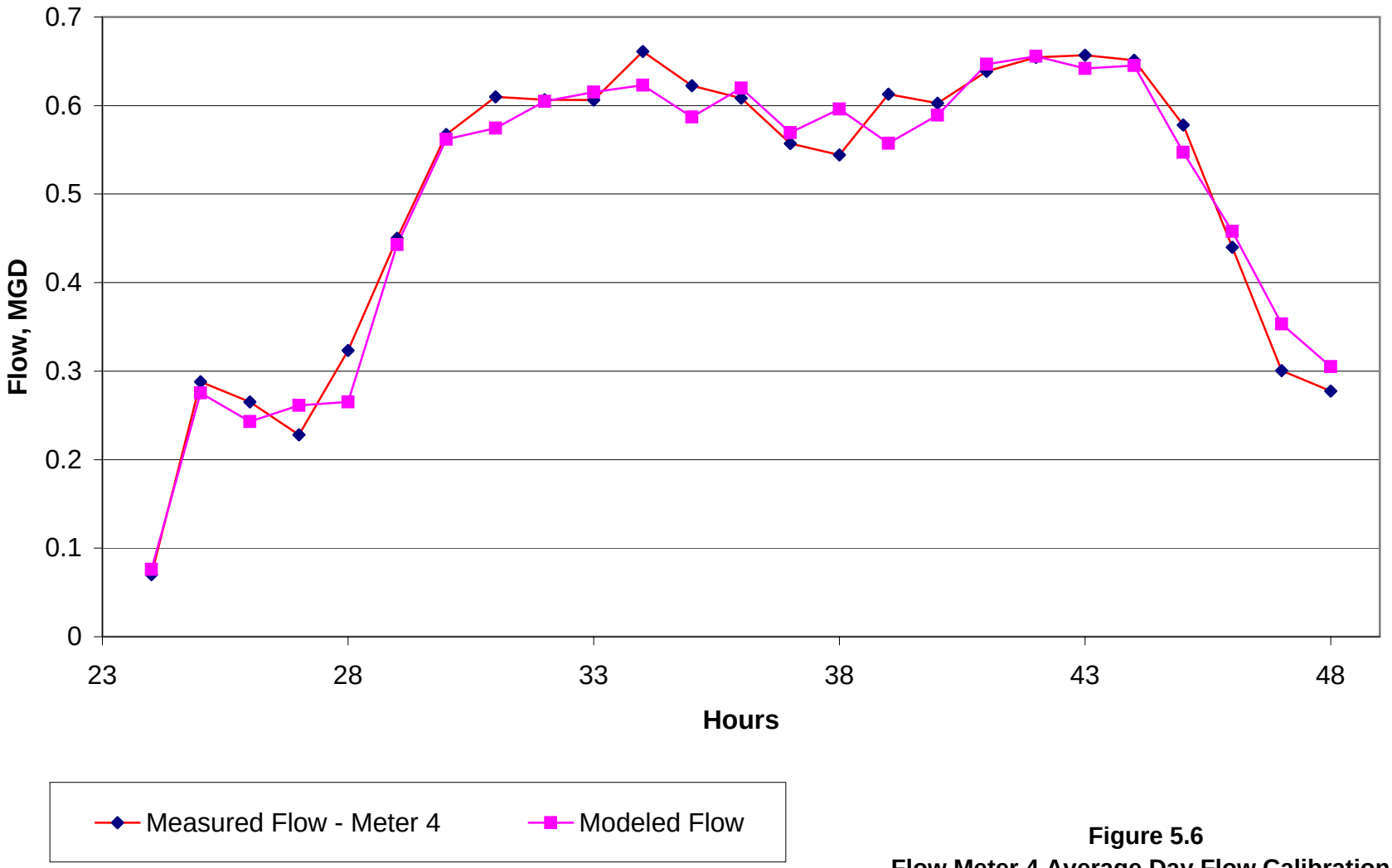


Figure 5.6
Flow Meter 4 Average Day Flow Calibration
Wastewater Collection System Master Plan
City of Galt

carollo
Engineers...Working Wonders With Water™

parameters for this rainfall event. The December 24th to 25th rainfall event was used to characterize the system's response to wet weather events. The hydraulic model was calibrated to this event, and then the parameters were verified with the rain event that occurred on December 15th.

The wet weather calibration process involves creating custom unit hydrographs for each flow meter tributary. The hydrographs utilize the R-Values (percent of rainfall that enters collection system) to simulate I/I. The R-Values are input into the model and the parameters are adjusted until the peak I/I rate measured during the flow monitoring program are simulated for the first rainfall event, and that the modeled sewer flows match the measured data from the flow monitoring program.

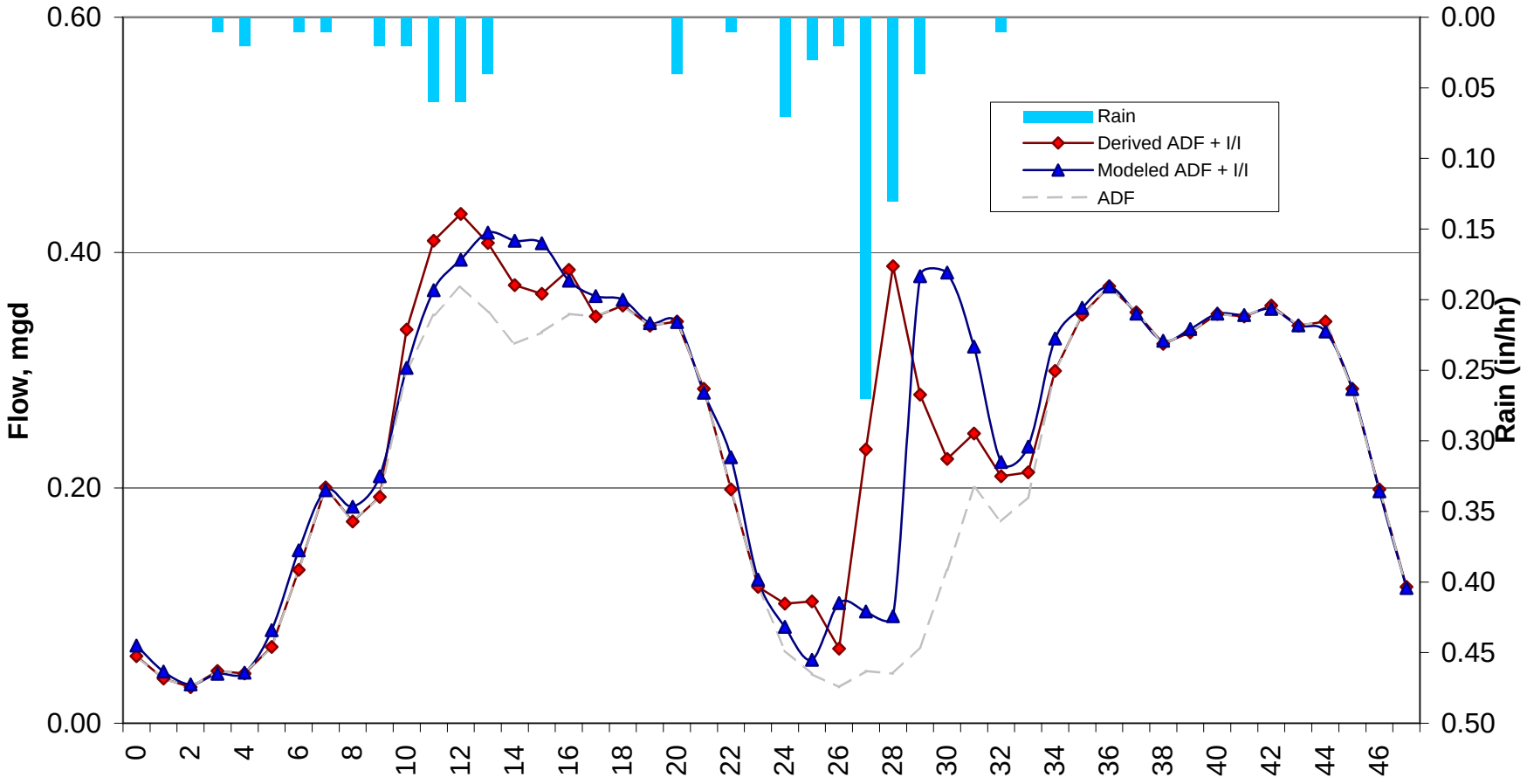
Table 5.4 Average Day Flow Calibration Summary
Sewer System Master Plan
City of Galt

	Meter Average Day Flow[1]			Model Average Day Flow			
Meter Site	**Average (mgd)**	**Max. (mgd)**	**Min. (mgd)**	**Average (mgd)**	**Max. (mgd)**	**Min. (mgd)**	**Percent Diff.[2]**
1	0.96	1.38	0.38	0.97	1.42	0.39	1.7%
2	0.29	0.46	0.12	0.29	0.46	0.12	1.2%
3	0.75	1.21	0.28	0.75	1.21	0.29	0.6%
4	0.51	0.66	0.23	0.51	0.66	0.24	-0.9%
5	0.22	0.37	0.03	0.22	0.37	0.03	-0.4%
6	0.18	0.23	0.08	0.18	0.24	0.08	-1.8%

Source: Meter data from Sanitary Sewer Flow Monitoring and Inflow/Infiltration Study March 2008

Notes:
1. Average day flow calculated from flow monitoring data, not including weekend flow. Maximum flow is the maximum hourly flow measured over the monitoring period for dry weather conditions.
2. Percent difference between the average flow for the meter collected data and the modeled derived results.

Once the model results matched the measured data from the first rain event, the same parameters were used to route the second rain event and check the model results versus the flow monitoring data. Using two events shows that the model will respond accurately to the simulated design storm event. Figure 5.7 illustrates the results for the wet weather calibration for Meter 5.



Note: M5 Wet Weather Calibraiton for December 24-25 2008 rainfall event

Figure 5.7
Flow Meter 5 Wet Weather Flow Calibration
Wastewater Collection System Master Plan
City of Galt

carollo
Engineers...Working Wonders With Water™

CAPACITY EVALUATION AND PROPOSED IMPROVEMENTS

This chapter discusses the hydraulic evaluation of the wastewater collection system and the proposed projects that correct capacity deficiencies and serve future users.

6.1 CAPACITY EVALUATION

Following the dry and wet weather flow calibration, a capacity analysis of the modeled collection system was performed. The capacity analysis entailed identifying areas in the sewer system where flow restrictions occur or where pipe capacity is insufficient to convey design flows. Sewers that lack sufficient capacity to convey design flows could produce backwater effects in the collection system and potentially cause sanitary sewer overflows (SSOs). The sewer system was evaluated based on planning criteria presented in this report.

This chapter discusses the locations of existing and future hydraulic deficiencies resulting from flows exceeding the maximum flow depth (d) to pipe diameter ratio (D) (d/D) criteria and from lift stations lacking the firm capacity to convey design flow. When an increase to capacity is required, existing sewers can be upgraded or a parallel sewer can be constructed. For the purposes of this wastewater collection system master plan (Master Plan), unless otherwise stated, we assumed that a capacity deficient sewer would be upgraded to a larger diameter. An exception to this is the parallel Live Oak trunk sewers. The upgraded pipeline generally followed the same slope as the existing pipeline, with the exception where survey data revealed negative or flat slopes in an existing alignment. In essence there are two alternatives for every trunk sewer project, but the decision to replace or construct a parallel sewer should be made during the preliminary design phase.

During the preliminary design phase, the existing sewer should be inspected by closed circuit television (CCTV) to determine its structural condition. If severely deteriorated, the existing sewer should be upgraded. If moderately deteriorated, slip lining or cured-in-place pipe lining can rehabilitate the existing sewer.

In general, the existing wastewater collection system has sufficient capacity to convey existing design flows. In some locations, such as the City of Galt's (City) downtown, existing infiltration and inflow (I/I) cause sudden increases in wastewater flow during a storm event. The storm water inflow could cause a few sewers to surcharge and potentially overflow during a large storm.

The proposed improvements that will serve future users are sized for build-out conditions. As the City continues to grow beyond its current limits, it is recommended that the pipeline diameters and pump station capacities proposed in this Master Plan be constructed so that the facilities have sufficient capacity for build-out conditions. Building a smaller interim project with the plans of upsizing in the future to account for further growth is not recommended due to the extended useful life of the improvements proposed herein; in this Master Plan, the proposed pipe diameter represents the ultimate diameter for build-out conditions.

6.2 COLLECTION SYSTEM IMPROVEMENTS

Figure 6.1 illustrates the proposed sewer improvements required to correct existing deficiencies and to serve future users. The proposed pipeline diameter is also shown on the figures. Other projects that will benefit existing users include proposed sewer main rehabilitation projects discussed below.

Table 6.1 was created to show more detail of each improvement. For future sewers, the proposed diameter is shown along with the length of sewer. This table also shows the improvement identification number. Both Figure 6.1 and Table 6.1 should be used together to locate the proposed improvement on the map and to gain details of the improvement (length, diameter, street location, etc.). The improvement identification number links the figure and table. For example, E-1 in Table 6.1 refers to the proposed Elm Trunk Sewer upgrade, which will increase the existing 8-inch diameter sewer main to a 10-inch diameter sewer, and can be found in Figure 6.1, downstream of the Elm-Amador Lift Station.

6.2.1 Sewer Main Rehabilitation and Replacement

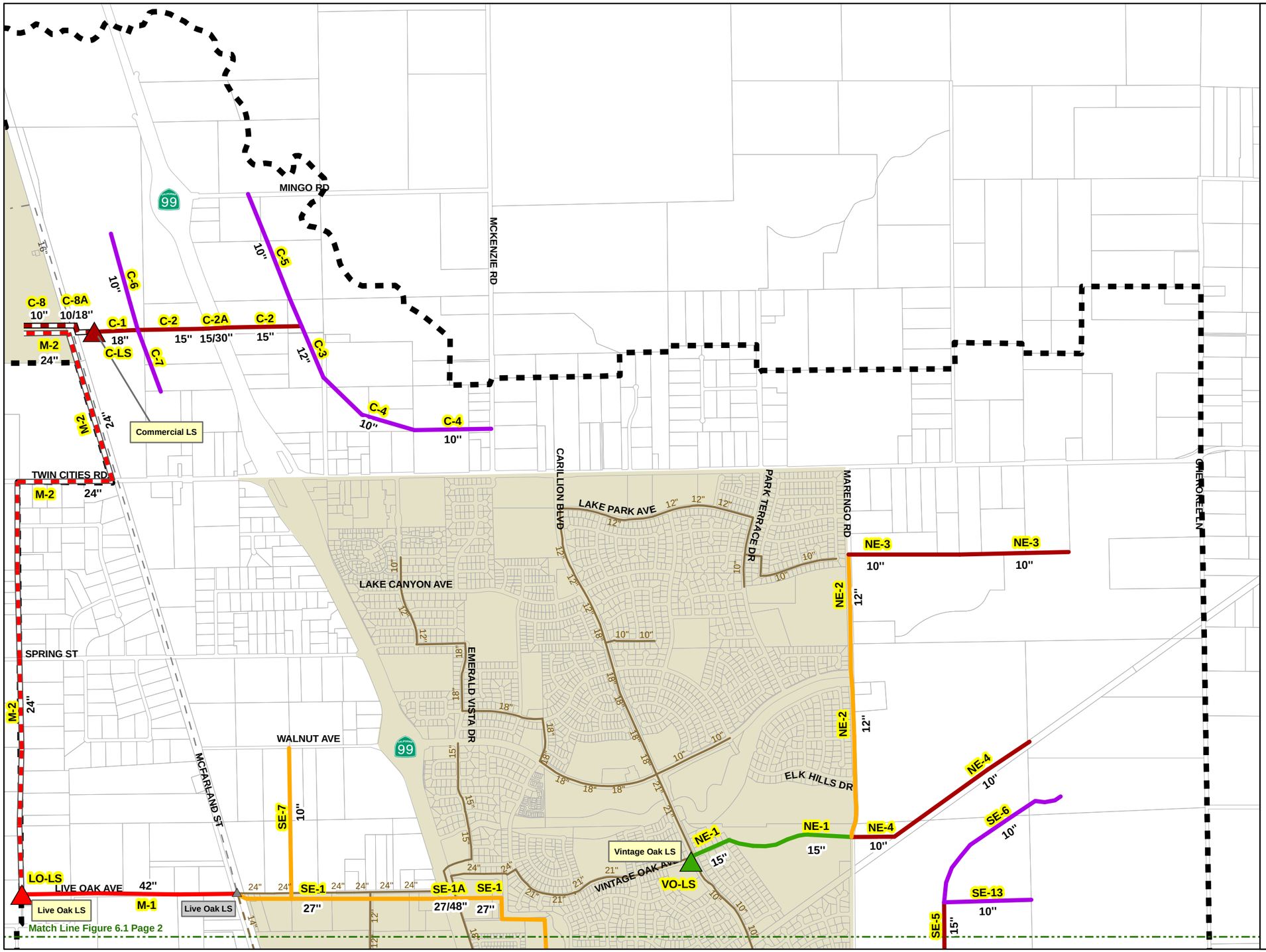
In addition to the pipeline and lift station improvements recommended to mitigate capacity deficiencies, the City may implement a sewer main condition assessment and rehabilitation program. The sewer main rehabilitation program consists of replacing older sewers that are susceptible to failure, showing signs of deterioration, or lining sewers that are exhibiting signs of corrosion but still possess structural integrity. This program will extend the useful life of existing sewers and maintain the operation of the collection system.

Postponing rehabilitation and replacement of corroded sewers could result in pipe failure and disruption of sewer service. Rehabilitation and replacement projects are recommended capital improvement projects (CIP). The scope of rehabilitation and replacement projects would occur at the same rate as the City's current program. This may need to be adjusted in the future, pending a comprehensive condition assessment. Based on work in other cities, the sewers that should be targeted for this work include those that are over 30 years in age or unlined.

6.2.2 Existing Versus Future Improvement

An existing deficiency is one where the existing facility's capacity is insufficient to meet the planning criteria (e.g. pipeline upgrades required to prevent severe surcharging during the design wet weather event) for existing users. If a project was proposed to correct an existing deficiency, then existing users were assigned 100 percent of the project's benefit, and therefore, 100 percent of the costs.

The vast majority of the Master Plan improvements will serve future users, even when an improvement calls for the upgrade of an existing facility. In these cases, an existing sewer or lift station may have sufficient capacity to convey current design flows, but as growth continues and more users are added to the system, the increased flow results in capacity deficiencies. These are labeled future improvements. Future users were assigned 100 percent of the future project's benefit and 100 percent of the costs. More information on the breakdown in cost split between



Legend

Modeled Wastewater Collection System

- Lift Station
- Abandoned Lift Station
- Pipelines
- Abandoned Pipelines

Existing System Improvements

Phase 1 (2009 - 2015)

- Lift Station

Pipelines

Phase 1 (2009 - 2015)

- New Pipeline
- Pipeline to be Replaced

Future System Improvements

Lift Station

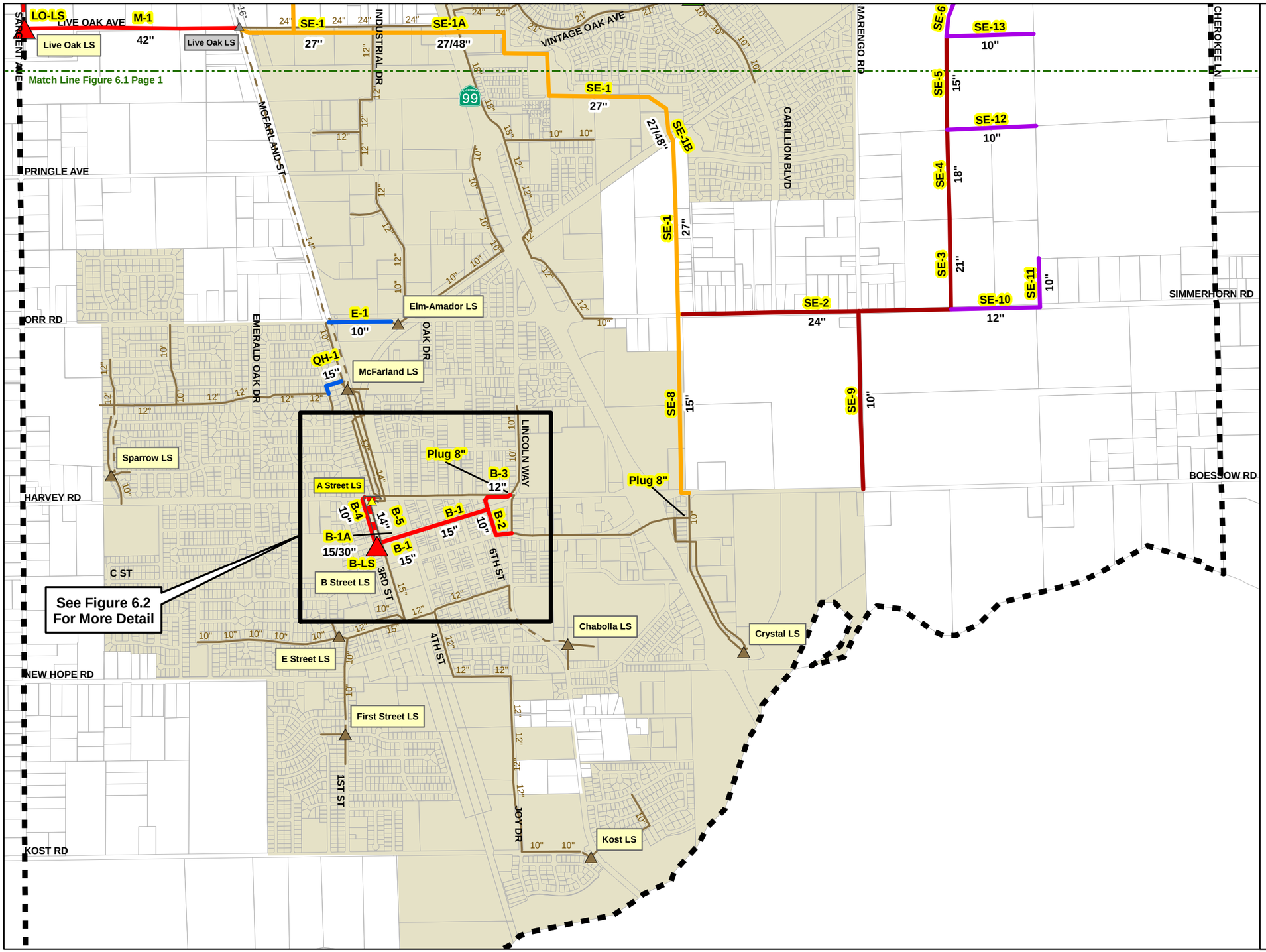
- Phase 1 (2009 - 2015)
- Phase 3 (2021 - 2025)

Pipelines

- Phase 1 (2009 - 2015)
- Phase 2 (2016 - 2020)
- Phase 3 (2021 - 2025)
- Phase 4 (2026 - 2030)
- General Plan Boundary
- Parcels
- City Limits
- P-25 Figure Number
- 12" Proposed Pipeline Diameter
- 12/24" Proposed Pipeline/ Casing Pipe Diameter
- 8" Existing Pipeline Diameter

0 1,500 3,000 Feet

Figure 6.1
Proposed Capital Improvements (1 of 2)
Wastewater Collection System Master Plan
City of Galt



Legend

Modeled Wastewater Collection System

- Lift Station
- Abandoned Lift Station
- Relocated Lift Station
- Pipelines
- Abandoned Pipelines

Existing System Improvements

Phase 1 (2009 - 2015)

- Lift Station

Pipelines

Phase 1 (2009 - 2015)

- New Pipeline
- Pipeline to be Replaced

Future System Improvements

Lift Station

- Phase 1 (2009 - 2015)
- Phase 3 (2021 - 2025)

Pipelines

- Phase 1 (2009 - 2015)
- Phase 2 (2016 - 2020)
- Phase 3 (2021 - 2025)
- Phase 4 (2026 - 2030)
- Parcels
- City Limits
- General Plan Boundary
- P-25 Figure Number
- 12" Proposed Pipeline Diameter
- 12/24" Proposed Pipeline/ Casing Pipe Diameter
- 8" Existing Pipeline Diameter

N

0 1,500 3,000 Feet

Figure 6.1
Proposed Capital Improvements (2 of 2)
Wastewater Collection System Master Plan
City of Galt

GALT

carollo

Table 6.1 Proposed Collection System Improvements
Wastewater Collection System Master Plan
City of Galt

Figure No.	Type of Improvement	Description/ Street	Description / Limits	Pipeline Cost Schedule (A or B)	Project Length/Size: Ex. Size/ Diam. (in)	New Size/ Diam. (in)	Replace/ New	Length (ft)	Capital Improvement Phasing: Phase 1 2009-2015	Phase 2 2016-2020	Phase 3 2021-2025	Phase 4 2026-2030
Existing System Improvements												
Pipelines												
Live Oak Trunk Sewer and Midway Force Main												
M-1	Pipe	Live Oak Avenue	McFarland Street to Sargent Avenue	B	-	42	New	3,200	Phase 1			
M-2	Force Main	Midway Avenue	Live Oak Avenue to WWTP	B	-	24	New	10,700	Phase 1			
B Street Trunk Sewer												
B-1	Pipe	B Street	3rd Street to 7th Street	A	-	15	New	1,550	Phase 1			
B-1A	Pipe/Casing(1)	B Street	UPRR Crossing	A	-	15/30	New	100	Phase 1			
B-2	Pipe	7th Street/C Street	B Street to Lincoln Way	A	-	10	New	600	Phase 1			
B-3	Pipe	7th Street/A Street	B Street to Lincoln Way	A	-	12	New	500	Phase 1			
B-4	Pipe	3rd Street	A Street to B Street	A	-	10	New	660	Phase 1			
B-5	Force Main	A Street/3rd Street	McFarland Street to B Street	A	-	14	New	850	Phase 1			
Elm Trunk Sewer												
E-1	Pipe	Elm Avenue	McFarland Street to west of Industrial Way	A	8	10	Replace	900	Phase 1			
Quail Hollow Trunk Sewer												
QH-1	Pipe	Oberlin Way/Trudy Way	Lorna Way to McFarland Street	A	10/12	15	Replace	400	Phase 1			
Sewer Main Replacement Program												
-	Pipe	Various Locations	Sewer Main Replacement Program. Average size is 10".	A	10	10	Replace	50,000	Phase 1	Phase 2	Phase 3	Phase 4
Lift Stations(2)												
B-LS	Lift Station	B Street/4th Street	A Street LS Relocation (3.6 mgd Firm Capacity)	-	5.2 mgd	5.4 mgd	Relocate/Upgrade	-	Phase 1			
	Land Acquisition				-	0.25 acres	Relocate/Upgrade	-	Phase 1			
LO-LS	Lift Station	Live Oak and Sargent Road	Live Oak Lift Station (14.0 mgd Firm Capacity)	-	-	17.5 mgd	New	-	Phase 1			
	Land Acquisition				-	0.25 acres	New	-	Phase 1			
-	Lift Station	Various Locations	Lift Station Rehab. Program. Average Capacity is 2.6 mgd (Total).	-	2.6 mgd	2.6 mgd	Rehab/Replace	1 Project/Phase	Phase 1	Phase 2	Phase 3	Phase 4
Future System Improvements												
Pipelines												
Southeast Trunk Sewer												
SE-1	Pipe	Live Oak Avenue/Undeveloped Area near Deadman Gulch	McFarland Street to Simmerhorn Road	A	-	27	New	10,200		Phase 2		
SE-1A	Pipe/Casing(1)	Live Oak Avenue	Highway 99 Crossing	A	-	27/48	New	300		Phase 2		
SE-1B	Pipe/Casing(1)	Undeveloped Area west of Carillion Boulevard	UPRR Crossing	A	-	27/48	New	100		Phase 2		
SE-2	Pipe	Simmerhorn Road	West of Carillion Boulevard to east of Marengo Road	A	-	24	New	4,000			Phase 3	
SE-3	Pipe	East of Marengo Road	Simmerhorn Road to north of Simmerhorn Road	B	-	21	New	1,300			Phase 3	
SE-4	Pipe	East of Marengo Road	North of Simmerhorn Road to south of UPRR	B	-	18	New	1,300			Phase 3	
SE-5	Pipe	East of Marengo Road	North of Simmerhorn Road to south of UPRR	B	-	15	New	1,400			Phase 3	
SE-6	Pipe	Southeast of UPRR line	North of Simmerhorn Road to south of UPRR	B	-	10	New	2,500				Phase 4
SE-7	Pipe	East of McFarland Street	Live Oak Avenue to north of Live Oak Avenue	B	-	10	New	2,200		Phase 2		
SE-8	Pipe	Undeveloped Area west of Carillion Boulevard	Simmerhorn Road to Boessow Road	B	-	15	New	2,900		Phase 2		
SE-9	Pipe	Marengo Road Extension	Simmerhorn Road to Boessow Road	B	-	10	New	2,700			Phase 3	
SE-10	Pipe	Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	12	New	1,300			Phase 3	Phase 4
SE-11	Pipe	West of Cherokee Lane	Simmerhorn Road to north of Simmerhorn Road	B	-	10	New	700				Phase 4
SE-12	Pipe	North of Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	10	New	1,300				Phase 4
SE-13	Pipe	North of Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	10	New	1,300				Phase 4
Northeast Trunk Sewer												
NE-1	Pipe	Undeveloped Area near Deadman Gulch	Carillion Boulevard to Marengo Road	B	-	15	New	2,400	Phase 1			
NE-2	Pipe	Marengo Road	South of Elk Hills Drive to South of Lake Park Avenue	A	-	12	New	3,600		Phase 2		
NE-3	Pipe	South of Twin Cities Road	Marengo Road to east of Marengo Road	B	-	10	New	3,300			Phase 3	
NE-4	Pipe	North of UPRR	Marengo Road to east of Marengo Road	B	-	10	New	2,800			Phase 3	
Commercial Trunk Sewer												
C-1	Pipe	North of Twin Cities Road	McFarland Street to West of Highway 99	B	-	18	New	800			Phase 3	
C-2	Pipe	North of Twin Cities Road	West of Highway 99 to East of Highway 99	B	-	15	New	2,100			Phase 3	
C-2A	Pipe/Casing(1)	North of Twin Cities Road	Highway 99 Crossing	A	-	15/30	New	400			Phase 3	
C-3	Pipe	North of Twin Cities Road	East of Highway 99 to Bergeron Road	B	-	12	New	800				Phase 4
C-4	Pipe	North of Twin Cities Road	Bergeron Road to McKenzie Road	B	-	10	New	2,800				Phase 4
C-5	Pipe	East of Highway 99	North of Twin Cities Road to Mingo Road	B	-	10	New	2,100				Phase 4
C-6	Pipe	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	1,500				Phase 4
C-7	Pipe	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	1,000				Phase 4
C-8	Force Main	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	200			Phase 3	
C-8A	Pipe/Casing(1)	North of Twin Cities Road	UPRR Crossing	A	-	10/18	New	100			Phase 3	
Lift Stations(2)												
VO-LS	Lift Station	Vintage Oak Avenue and Carrilion Boulevard	Vintage Oak Lift Station Upgrade (3.8 mgd Firm Capacity)	-	4.3 mgd	5.7 mgd	Upgrade	-	Phase 1			
C-LS	Lift Station	East of Railraod Tracks near WWTP	Commercial Lift Station (1.4 mgd Firm Capacity)	-	-	2.8 mgd	New	-			Phase 3	
	Land Acquisition				-	0.25 acres	New	-			Phase 3	

Notes:
1. Proposed casing size and carrier pipe size.
2. Lift station capacities refer to the total capacity, unless otherwise noted.

existing and future users and whether a proposed improvement is intended to correct an existing deficiency, to serve a future user, or both is provided in Table 7.3.

6.2.3 Proposed Existing System Improvements

For the majority of the City, the existing wastewater collection system contains sufficient capacity to convey peak dry weather flows (PDWF) and design flows without exceeding the capacity criteria discussed in Chapter 3 (e.g. maximum d/D). There are a few exceptions where existing sewers will need to be replaced by larger diameter sewers, or parallel sewers will need to be constructed to bypass flow around hydraulically deficient sewers.

6.2.3.1 Elm Trunk Sewer

The existing 8-inch diameter sewer downstream of the Elm-Amador Lift Station lacks the capacity to convey design flows. This has been substantiated by field observations collected by City maintenance staff documenting that the existing sewer “backs-up” during high flows. A 10-inch diameter trunk sewer (E-1 in Figure 6.1) is recommended to replace this capacity deficient sewer.

6.2.3.2 Quail Hollow Trunk Sewer

The existing Quail Hollow Trunk Sewer lacks the hydraulic capacity to convey design flows around Oberlin and Trudy Way. A 15-inch diameter trunk sewer (QH-1) is recommended to replace the existing 10 and 12-inch diameter sewers.

6.2.3.3 B Street Trunk Sewer and A Street Lift Station

The proposed sewer and lift station improvements discussed in the following paragraphs should be considered one comprehensive project that would be implemented together. The most severe sewer capacity issues within the existing system occur in the downtown area, specifically the 8-inch diameter sewer in A Street between the railroad and Lincoln Way. Based on results from the flow monitoring program, this sewer experiences dramatic increases in flow following a storm event, due to rain runoff in the form of I/I. Hydraulic modeling results reveal that surcharging during large storms could lead to SSOs from manholes on this sewer. Also, during dry weather periods, this sewer will often surcharge during PDWF, which indicates that the pipeline is undersized.

The A Street Lift Station is also undersized to pump the design flow. The lift station’s existing firm capacity is 2.59 million gallons per day (mgd), and the existing design flow is 4.00 mgd, resulting in a capacity deficiency of 1.41 mgd. It also appears that the operating controls on the lift station cause the wet well wastewater level to remain higher than the tributary sewers, triggering the wastewater to back up into the 15-inch diameter trunk sewer in 3rd Street. This was observed during the flow monitoring program, where the upstream sewers flowing to the wet well were submerged, even during low flow dry weather periods. However, there are other recommendations in this Master Plan that should reduce the peak flows reaching the lift station. Wastewater generated on the east side of Highway 99 (around Crystal Way) that currently flows

to this lift station will be routed to the proposed Southeast Trunk Sewer (SE-8) as described below. This re-routing of flow will reduce the build-out design flows to 3.64 mgd.

The proposed solution to this problem not only corrects the sewer capacity issue, but also takes into account a planned grade separation project at A Street, the utility conflicts in A street, and the disruption to downtown businesses in C Street. The proposed sewer alignments avoid A Street and C Street, to the extent possible. There are feasible variations to this proposed solution, but this recommendation should be sufficient for developing a comprehensive capital improvement project (CIP) and for providing adequate funding regardless if there are slight variations in the final alternative.

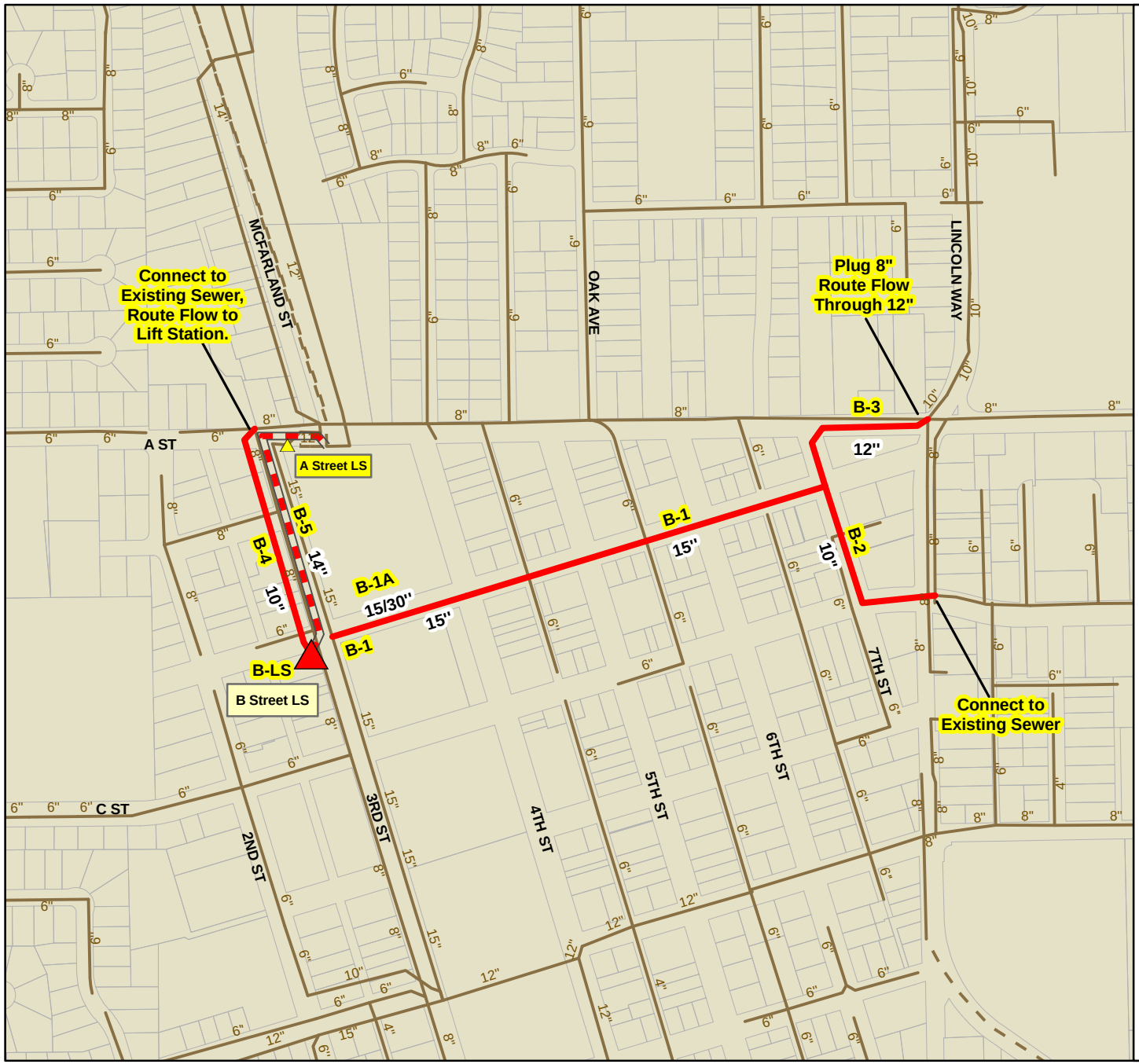
Since the City is planning to replace the at grade railroad crossing at A Street, upgrading the A Street Lift Station at its current location is not recommended, since it might be relocated as part of the railroad crossing project. Instead, the City should consider moving the lift station to a new location. One possible site is near the intersection of B and 4th Street. This site is suggested because the proposed sewer needed to relieve the A Street sewer would run in B Street, as shown in Figure 6.2. This new lift station would have a firm and total capacity of 3.6 and 5.4 mgd, respectively, to serve existing and future customers. Although the preliminary work compiled as part of this master plan does not constitute a design, a possible configuration of this lift station would be three pumps, each with a capacity of 1.8 mgd (1,250 gpm).

The proposed B Street relief sewer (B-1, 2, and 3) would divert flow away from the A Street sewer. This is accomplished by connecting new 10 and 12-inch diameter pipelines to existing sewers at the locations shown in Figure 6.2. This configuration would require a plug on the existing 8-inch diameter A Street sewer, immediately downstream of the connection point. Under this arrangement, all flow east of Lincoln Way that flowed down A Street would now be conveyed down the proposed 15-inch diameter sewer in B Street, to the relocated lift station. The A Street sewer would still convey flow from a sizeable tributary north of A Street, but by diverting a portion of the upstream flow down B Street, the risk of SSOs is reduced.

Due to the lift station relocation, the 8-inch diameter sewer in 3rd Street that currently flows to the A Street Lift Station would need to be rerouted to the new lift station location. Figure 6.2 shows the new alignment going from A Street to B Street.

6.2.4 Available Capacity in Existing Sewers

Where applicable, this Master Plan summarizes the available capacity remaining in existing sewers before an improvement is necessary. In some cases, the capacity for existing sewers was increased only to accommodate more flow resulting from future development. To calculate the timing of these improvements or the growth triggers that initiate a capacity expansion, it was necessary to determine the available capacity in an existing sewer or lift station. The available capacity is presented in terms of equivalent single-family dwelling units (EDU) values. In other words, this value defines the number of additional EDUs that can convey wastewater to a sewer before exceeding its capacity.



Legend

Existing Wastewater Collection System

- Lift Station
- Relocated Lift Station
- Pipelines

Existing System Improvements Phase 1 (2009 - 2015)

- Lift Station
- New Pipeline
- Parcels
- City Limits
- P-25 Figure Number
- 12" Proposed Pipeline Diameter
- 12/24" Proposed Pipeline/ Casing Pipe Diameter
- 8" Existing Pipeline Diameter

N

0 300 600 Feet

Figure 6.2
B Street Improvements
Wastewater Collection System Master Plan
City of Galt

GALT

carollo

The City's Improvement Standards do not include a recommendation on estimating flow from a typical single family residential dwelling unit. For other Central Valley communities, this value has ranged from 250 to 300 gallons per unit per day (gpud). For the purposes of estimating the remaining sewer capacity in terms of EDUs, this study assumed 326 gpud. This valve is used to approximate sewage flows for a single family detached residential home. In order to calculate the available capacity in terms of EDUs, the difference between a sewer's maximum capacity (based on size, slope and maximum d/D) and the current design flow was divided by an appropriate peaking factor to derive an average flow. This average flow was then divided by 326 gpud to quantify the EDU. This calculation does not apply to new sewers serving future users, only to existing sewers that will also serve future users. The number of EDUs that can be served before a capacity expansion is triggered is discussed below. Note that only a few existing facilities need to be improved to serve future users.

For the purposes of calculating available capacity, this Master Plan assumed a peaking factor of 2.6.

6.2.5 New Trunk Sewers, Interceptors, and Lift Stations

The following discussion summarizes the new trunk sewers and interceptors that will serve future users. Please note that the locations of the sewers are preliminary and will likely change during the design phase. The locations shown are possible alignments based on available information and are intended to assist in the development of probable construction costs. No investigation into the feasibility of these alignments has been conducted. However an attempt was made to place future pipeline alignments within existing streets or proposed future streets presented in the General Plan's Land Use and Circulation Diagram.

This Master Plan assumes that all sewage lift stations will have two non-clog sewer pumps, each capable of pumping 100 percent of the design flowrate. In other words, each lift station's firm capacity should be sufficient to pump 100 percent of the design flowrate. Each lift station's firm capacity was compared to the existing and build-out design flow conveyed to the lift station. If the design flow was greater than the lift station's firm capacity, then the lift station was considered deficient and required upgrade.

6.2.5.1 Live Oak Pump Station, Trunk Sewer, and Midway Forcemain

The City evaluated the alternatives for relocating the Live Oak Pump Station. The alternative locations included either the intersection at Live Oak and Midway Avenue or the wastewater treatment plant (WWTP). A project memorandum that summarized the benefits and drawbacks of each location was prepared for the City to assist in the selection of the preferred alternative. This memorandum is included in Appendix E. Although building an influent pump station at the WWTP along with a 42-inch diameter gravity trunk sewer had the highest capital cost, it was recommended by the project memorandum due to its overall life cycle costs. After long and careful consideration of estimated construction and operation and maintenance costs, the impact to the improvements presented in this master plan, and possible benefits to future

development, City staff concluded that the forcemain alternative is the most cost effective and suitable improvement for the City.

The Live Oak Pump Station would have a firm and total capacity of 14.0 and 17.5 mgd, respectively. Although this does not constitute a design, a possible configuration of this lift station would be five pumps, each with a capacity of 3.5 mgd (2,400 gpm). This capacity is sufficient to serve existing and future users through year 2030.

The new 10,700 feet, 24-inch diameter forcemain in Midway Avenue (M-2 in Figure 6.1) would extend from the New Live Oak Pump Station (LO-LS) located at Live Oak Avenue and Sargent Avenue to the WWTP. The new 3,200 feet, 42-inch trunk sewer in Live Oak from McFarland Street to Sargent Avenue will connect the new Live Oak Pump Station to the existing 24-inch diameter Live Oak Trunk Sewer at McFarland Street. The existing Live Oak Pump Station will no longer be needed.

6.2.5.2 Southeast Trunk Sewer

The existing 24-inch diameter trunk sewer in Live Oak Avenue has sufficient capacity to serve approximately 3,800 additional EDUs, or 700 to 900 acres of low density residential housing (depending on the number of units per acre). These numbers represent the amount of additional infill development west of Highway 99, or new development east of the highway that could be built within the Live Oak/Vintage Oak Sewer Tributary shown in Figure 5.4, and served by the existing 24-inch diameter pipeline. However, as development continues, a parallel sewer will need to be constructed to provide sufficient capacity.

Carollo developed a gravity sewer alternative that can convey flow from future users in the southeast part of the study area across Highway 99 to the Midway Trunk Sewer, without lift stations. As shown in Figure 6.1 (SE-1 through SE-6), the proposed sewer starts as a 27-inch diameter pipeline in Live Oak Avenue, crosses Highway 99 and the railroad, reduces to a 24-inch diameter pipeline in Simmerhorn Road, and continues north until its terminus east of Marengo Road. This proposed sewer would connect to the 42-inch diameter Midway Trunk Sewer at McFarland Street and run parallel to the existing 24-inch diameter Live Oak Trunk Sewer. The new 27-inch diameter sewer will probably not be needed until after year 2020. However, the City should track the pace of new development within the Live Oak/Vintage Oak Sewer Tributary, monitor the flow in the 24-inch diameter sewer, and regularly calculate the remaining capacity in this sewer.

In addition to this primary trunk sewer, there are also smaller sewers that branch off this main pipeline and will serve future users. These smaller trunk sewers (SE-7 through SE-13) flow tributary to the Southeast Trunk Sewer. Note that improvement SE-8 from Boessow Road to Simmerhorn Road is also sized to serve the existing users around Crystal Way, east of Highway 99. The purpose is to redirect wastewater that currently flows through the City's downtown sewers into this new 15-inch diameter trunk sewer, provide some relief and reduce the threat of SSOs. Therefore, after this 15-inch diameter sewer is installed and connected to the Southeast Trunk Sewer, the existing 8-inch diameter sewer main that crosses Highway 99 can be plugged (as shown in Figure 6.1).

6.2.5.3 Northeast Trunk Sewer

The Northeast Trunk Sewer will serve future development primarily from the area east of Marengo Road, north of the railroad, and south of Twin Cities Road. This sewer will connect to the existing 21-inch diameter Carillion Trunk Sewer and run east to Marengo Road where it branches off and continues north and east (NE-1 through NE-4, Figure 6.1). The Northeast Trunk Sewer ranges in diameter from 10 to 15-inches.

6.2.5.4 Commercial Trunk Sewer and Lift Station

The Commercial Trunk Sewer will serve the study area north of Twin Cities Road that is dominated by commercial, office, and industrial land uses. Since a new WWTP influent pump station alternative was not recommended, wastewater generated in this tributary will require a lift station (C-LS) located east of the railroad tracks.

The Commercial Trunk Sewer will require a 10-inch forcemain (C-8 and C-8A) from the proposed lift station to the headworks of the WWTP. From the lift station the sewer would run east, cross Highway 99, branch off and extend as far east as McKenzie Road (C-1 through C-4). This trunk sewer ranges in diameter from 10 to 18-inches. There are also 10-inch diameter trunk sewers (C-5, 6, and 7) that will serve future users to the north and south of this primary line. The alignments shown were intended to follow possible future streets shown in the City's General Plan. The sewer tributary for this area is shown in Figure 5.4.

6.2.5.5 Vintage Oak Lift Station

The Vintage Oak Lift Station currently has a firm and total capacity of 2.16 and 4.32 mgd, respectively. When the Northeast Trunk Sewer starts conveying flow to this lift station, its capacity will be exceeded. The current design flows reaching the lift station are essentially equal to the existing firm capacity. The lift station should be upgraded before significant developments within its tributary are completed. At build-out, the firm capacity for the Vintage Oak Lift Station should be increased to 3.80 mgd to serve future growth. The City will have the option to replace the existing pumps (if the wet well size can support larger pumps) or replace the entire lift station. These options can be evaluated during the preliminary design phase.

6.2.6 Existing and Build-out Lift Station Improvements

All the lift stations, with the exception of A Street, Vintage Oak, and Live Oak Lift Stations contain sufficient firm capacity to convey existing and future design flows. The A Street and Live Oak Lift Stations require immediate upgrades to convey existing design flows. Vintage Oak is the only lift station requiring an increase in capacity to convey build-out design flows. Table 6.2

Table 6.2 Lift Station Evaluation Table
Wastewater Collection System Master Plan
City of Galt

	Pump Design Data									
Lift Station	**Pump Number**	**Pump Capacity (gpm)**	**Total Capacity (gpm)**	**Firm Pump Capacity (gpm)**	**Firm Pump Capacity (mgd)**	**Existing Modeled PWWF Influent to LS (mgd)**	**Existing Capacity Deficiency (mgd)**	**Buildout Modeled PWWF Influent to LS (mgd)**	**Buildout Capacity Deficiency (mgd)**	**Available Capacity[4] (EDUs)**
A Street	1	1,800	3,600	1,800	2.59	4.00[1]	**1.41**	3.64[3]	1.05	0
	2	1,800								
Chabolla	1	300	600	300	0.43	0.04	0	0.04	0.00	462
	2	300								
Crystal	1	300	600	300	0.43	0.03	0	0.03	0.00	471
	2	300								
E Street	1	500	1,000	500	0.72	0.27	0	0.33	0.00	531
	2	500								
Elm/Amador	1	500	1,000	500	0.72	0.32	0	0.36	0.00	472
	2	500								
First Street	1	250	500	250	0.36	0.36	0	0.36	0.00	0
	2	250								
Kost	1	380	760	380	0.55	0.18	0	0.21	0.00	433
	2	380								
Live Oak	1	3,000	6,000	3,000	4.32	9.20	**4.88**	13.7[2]	**9.38**	0
	2	3,000								
McFarland	1	1,000	2,000	1,000	1.44	1.42	0	1.51	0.00	27
	2	1,000								
Sparrow	1	700	1,400	700	1.01	0.16	0	0.16	0.00	1,000
	2	700								
Vintage Oak	1	1,500	3,000	1,500	2.16	2.29	0	3.80	**1.64**	0
	2	1,500								

Notes:
1. PWWF at A Street with improvements to mitigate deficiencies in A Street Sewer.
2. PWWF are taken at Midway Road and Live Oak Avenue.
3. PWWF lower at buildout because flow from east side of Highway 99 are routed north of the new 24-inch trunk sewer.
4. Available Capacity = (Firm Pump Capacity - Existing Modeled PWWF)/(2.6x300)

summarizes each lift station's current design data. The design data includes the number of pumps, individual pump capacity, firm capacity and total capacity.

Table 6.2 also summarizes the model derived build-out design flows that will be conveyed to each existing lift station. The build-out design flows were compared to each lift station's capacity to determine whether a capacity increase would be necessary. Based on the build-out design flow and existing capacity, the number of EDUs that can be served before an increase in capacity is required was calculated. For example, based on existing design flows, the Sparrow Lift Station can serve approximately 1,100 additional EDUs before an upgrade is necessary. This is important to know if the City is considering redevelopment of existing areas that would intensify the number of EDUs within a lift station's tributary. Quick examination of Table 6.2 would provide information on whether a lift station possesses sufficient capacity to serve higher density of housing.

6.3 PROJECT PRIORITIZATION

The majority of improvements listed in Table 6.1 are driven by future development. Most of the improvements are new sewers that serve future growth, but there are some improvements to existing facilities that are needed to serve existing users, or resulted from future growth. When fully implemented, the capital projects will allow the conveyance of design flows to the WWTP during build-out conditions.

Prioritizing the required capital improvements for the City's sewer system is an important aspect of the Master Plan. The improvement projects were prioritized based on the following factors:

- Upgrading existing sewers or lift stations to serve future users
- Building the sewers necessary to serve future users

Improvements to existing sewers and lift stations will provide sufficient capacity to mitigate existing issues and to convey increased flows resulting from new development. Future development will require the construction of sewers to serve new users. The implementation of these improvements will depend on the City's growth and selection of areas to be served with urban infrastructure. The City provided guidance on future development and phasing of infrastructure to serve future users. Based on this input, the projects were grouped into the following timeframes:

- Years 2009 through 2015
- Years 2016 through 2020
- Years 2021 through 2025
- Years 2026 through 2030

Table 6.1 indicates the phasing timeframe for each capital project. As shown in the table, the City is planning the Midway Trunk Sewer (M-1) as a near term project and should be constructed within the next five years (as indicated by the Phase 1 entry in the 2009 - 2015) column). This is a high priority project.

Proposed improvements within areas identified for early development were assigned a higher priority. Areas within an approved tentative map tract received the highest priority. The actual implementation of the improvements serving future users depends on growth. The priorities presented below are estimates based on available information provided by the City. Changes in the City's planning assumptions or growth projections could increase or decrease the priority of each improvement.

6.3.1 Phase 1 Projects (2009-2015)

The highest priority project for the existing system includes the B Street Trunk Sewer (B-1, 2 and 3) and lift station project. Building the B Street Trunk Sewer will relieve the excess flows that currently overwhelm the 8-inch diameter sewer main in A Street and reduce the threat of SSOs. The A Street Lift Station should also be relocated to convey the design flows for this sewer tributary and to avoid conflicts with the City's planned railroad grade separation project.

The Live Oak Trunk Sewer and Midway Forcemain projects (M-1 and M-2) and Live Oak Pump Station (LO-LS) are also high priority projects for the City. These projects are needed to replace the Live Oak Pump Station and forcemain and to serve future growth through the year 2030. Also, expansion of the Vintage Oak Lift Station (VO-LS) is needed before significant development within its tributary is completed.

- **Elm Trunk Sewer (E-1).** Mitigates existing capacity deficiency in an existing 8-inch diameter sewer downstream of the Elm-Amador Lift Station.
- **Quail Hollow Trunk Sewer (QH-1).** Mitigates existing capacity deficiency in existing 10 and 12-inch diameter sewers around Oberlin and Trudy Way.

Projects serving new developments in the growth areas targeted by the City are shown in Phase 1. Some of these projects identified within Phase 1 are part of a larger capital project. For example, the first reach of the Northeast Trunk Sewer (NE-1) is targeted as the first segment of this sewer to be constructed.

In addition to these capital projects, the City will implement sewer main and lift station rehabilitation and replacement projects.

6.3.2 Phase 2 Projects (2016-2020)

Lower priority system improvement projects were grouped into Phase 2 construction. In general, all of Phase 2 projects will serve future development beyond year 2015. These projects are within the 10-year window identified by the City. Many of the projects are

extensions of sewers that were started in Phase 1. For example, NE-2 represents the second phase of the Northeast Trunk Sewer. The Phase 2 projects include the following:

- **Northeast Trunk Sewer (NE-2).** Second reach of sewer constructed to service the area north of the railroad and east of Marengo Road.
- **Southeast Trunk Sewer (SE-1, SE-7, SE-8).** First reaches constructed to service new development in the area south of the railroad and east of Highway 99 and to redirect existing flows from the area around Crystal Way.
- On going sewer main and lift station rehabilitation and replacement.

6.3.3 Phase 3 and 4 Projects (2021-2025 and 2026-2030)

Table 6.1 breaks the projects into Phase 3 and 4. For the purposes of prioritization, these are viewed as very long-term projects exclusively driven by development at the fringes of the planning area, and will be grouped together. The Phase 3 and 4 projects include the following:

- **Northeast Trunk Sewer (NE-3, NE-4).** Final reaches constructed to service the area north of the railroad and east of Marengo Road.
- **Southeast Trunk Sewer (SE-2 through SE-9 through SE-13).** Final reaches constructed to service new development south of the railroad and east of Highway 99.
- **Commercial Trunk Sewer and Lift Station (C-1 through C-8A).** All reaches of sewers and lift station constructed to service the predominantly commercial land use areas north of Twin Cities Road.
- On going sewer main and lift station rehabilitation and replacement.

6.4 ADDITIONAL RECOMMENDATIONS

In addition to the improvements discussed above, the City should consider the following recommendations.

6.4.1 Sewer Main Replacement, Rehabilitation, and Maintenance

Deteriorated sewers are often corroded and susceptible to collapse. This Master Plan effort did not include a condition assessment to evaluate the structural integrity of existing sewers, nor did it evaluate rehabilitation alternatives for existing sewers. However, based on input from City staff, we are including rehabilitation of sewers as capital projects. For the purposes of budgeting, we assumed that the City would implement various rehabilitation projects through the year 2030 at more or less the same rate as currently completed. A sewer main rehabilitation allowance was provided for Phases 1 through 4.

The capital improvements assumed that the City would replace 0.5 miles of sewer every year. The weighted average diameter of all sewers in the collection system is 10-inches. Therefore, in order to develop the capital program impact, we assumed that 0.5 miles of 10-inch diameter sewer would be replaced every year through 2030.

According to City staff, there are neighborhoods that experience a significant amount of fats, oil and grease (FOG) problems. The neighborhoods included the area south of New Hope Road around Meadowview Drive, and neighborhoods around the downtown area. The City will need to continue routine maintenance like pipe cleaning to preventing clogging and SSOs.

6.4.2 Lift Station Replacement or Rehabilitation

Lift station replacement or rehabilitation will be necessary to maintain proper operation of existing facilities. We assumed that one lift station would be rehabilitated or replaced every five years, starting in Phase 1, for a total of four lift stations by year 2030. For budgeting purposes, we assumed replacement of the four oldest lift stations, which range in capacity from 500 to 6,000 gpm, and average approximately 1,900 gpm.

6.5 RURAL RESIDENTIAL DEVELOPMENT

Up to this point, one of the assumptions in the Master Plan is that Rural Residential designated lands are not anticipated to be annexed into the City limits during the timeframe of the 2030 General Plan. Therefore, wastewater flows imparted onto the system from existing and future customers within this land use were not accounted in the analysis. A question was raised about the impacts that extending service to rural customers would have on the proposed system. The following discussion addresses this question.

6.5.1 Rural Residential Wastewater Flow

The first step in assessing the impacts on the sewer system was calculating the increase in wastewater flow from rural residential customers. Assuming that rural wastewater flows per dwelling unit would be similar to low density residential flows per dwelling unit, a reasonable flow coefficient for Rural Residential would be 150 gallons per day per acre (gpda). Since the Rural Residential density is 0.5 dwelling units per acre (per the General Plan), the resulting flow is 300 gallons per day per dwelling unit (gpd/du). This per dwelling unit flow is typical for other Central Valley cities. For the purpose of this discussion and quantifying the impacts to sewer and lift stations, we assumed a wastewater flow coefficient of 150 gpda.

There are approximately 1,236 net acres (excluding public right-of-way) of Rural Residential land within the General Plan boundary and outside the 100-year floodplain. Therefore, the total average day flow (ADF) from Rural Residential equals approximately 0.19 mgd, which is distributed throughout the northern and western perimeter of the study area.

6.5.2 Rural Residential Collection System

Since the wastewater flows generated by rural residential customers will be low compared to the rest of the study area, and the rural users are spread out over a large area, these potential customers do not require large diameter pipelines for service. Rural residential customers could be served by extending 8-inch diameter sewer mains from existing trunk sewers. This Master Plan does not include capital costs for pipelines less than 10-inches in diameter. Sewer mains are typically funded by developers and are not included in development impact fee calculations.

Chapter 7

CAPITAL IMPROVEMENT PROJECTS

This chapter presents the recommended capital improvement projects (CIP) for the City of Galt (City) sewer system, a summary of the capital costs, and a basic assessment of the possible financial impact on individual existing and future users. This chapter is organized to assist the City in making finance decisions, and to plan the sewer system improvements through build-out of the 2030 General Plan (General Plan). The CIP is based on the evaluation of the City's sewer system, planning area, and land use, as detailed in the recommended projects described in the previous chapters.

7.1 CAPITAL IMPROVEMENT PROJECT COSTS

The capacity upgrades and new service projects set the foundation for the City's sewer system CIP. The cost estimates presented in this study are opinions developed from bid tabulations, cost curves, information obtained from previous studies, and Carollo Engineers, P.C.'s (Carollo) experience on other projects. The costs are based on an Engineering News Record Construction Cost Index (ENR CCI) 20-City Average of 8,534 (March 2009).

7.2 COST ESTIMATING ACCURACY

The cost estimates presented in the CIP have been prepared for general master planning purposes and for guidance in project evaluation and implementation. Final costs of a project will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors such as preliminary alignment generation, investigation of alternative routings, and detailed utility and topography surveys.

The Association for the Advancement of Cost Engineering (AACE) defines an Order of Magnitude Estimate, deemed appropriate for master plan studies, as an approximate estimate made without detailed engineering data. It is normally expected that an estimate of this type would be accurate within plus 50 percent to minus 30 percent. This section presents the assumptions used in developing order of magnitude cost estimates for recommended facilities.

7.3 CONSTRUCTION UNIT COSTS

The construction costs are representative of sewer system facilities under normal construction conditions and schedules. Costs have been estimated for public works construction, as either new construction in existing developed areas, or new construction in undeveloped areas.

7.3.1 Sewer Trunk Unit Costs

Sewer trunk line improvements range in size from 10-inches to 42-inches in diameter. Pipe casings up to 48-inches in diameter are included for major crossings (e.g. creeks, canals, highways, and railroad) of the trunk sewers. Pipeline unit costs are shown in Table 7.1. The construction cost estimates are based upon these unit costs. The unit costs are for "typical" field conditions with construction in stable soil at a depth ranging between 10 to 15 feet.

Construction of pipelines in undeveloped areas is anticipated to cost less than those constructed in developed areas, such as downtown. The unit costs in Table 7.1 are discounted by 30 percent for pipelines that will be built in undeveloped areas. This discount is based on review of bid tabulations from recent projects that were constructed in developed and undeveloped areas. Pipelines built in undeveloped areas ranged from 30 to 50 percent less than pipelines built in developed areas.

Table 7.1 Pipeline Construction Unit Costs
Wastewater Collection System Master Plan
City of Galt

	Pipeline Unit Cost ($/Linear Foot)	
Pipe Size (inches)	**Schedule A (Developed Areas)**	**Schedule B (Undeveloped Areas)**
10	116	81
12	139	97
15	174	122
18	190	133
21	222	155
24	253	177
27	285	199
30	316	222
33	348	244
36	380	266
39	411	288
42	443	310
Pipeline Casing for Major Crossings		
15/30	1,108	--
18/30	1,108	--
24/42	1,551	--
27/48	1,772	--

Note:
1. ENR CCI 20 City average used for estimating (March 2009) = 8,534

7.3.2 Lift and Pump Station Costs

Lift station improvements include increasing the firm capacity to convey design flows. The lift station cost versus capacity curve shown in Figure 7.1 was developed based on projects of similar size in California. Costs were generated by inputting the appropriate capacity and calculating the corresponding costs.

7.3.3 Land Acquisition Costs

Acquisition of property, easements, and right-of-way (ROW) may be required for some of the recommended projects, but not all. For the majority of sewers, pipeline corridor or easements are assumed to be in public ROW, and therefore do not require land acquisition. However, land may be required for the lift stations recommended in this Master Plan and are noted in the CIP table. The land costs were assumed to equal $240,000 per acre.

7.4 PROJECT COSTS AND CONTINGENCIES

7.4.1 Baseline Construction Cost

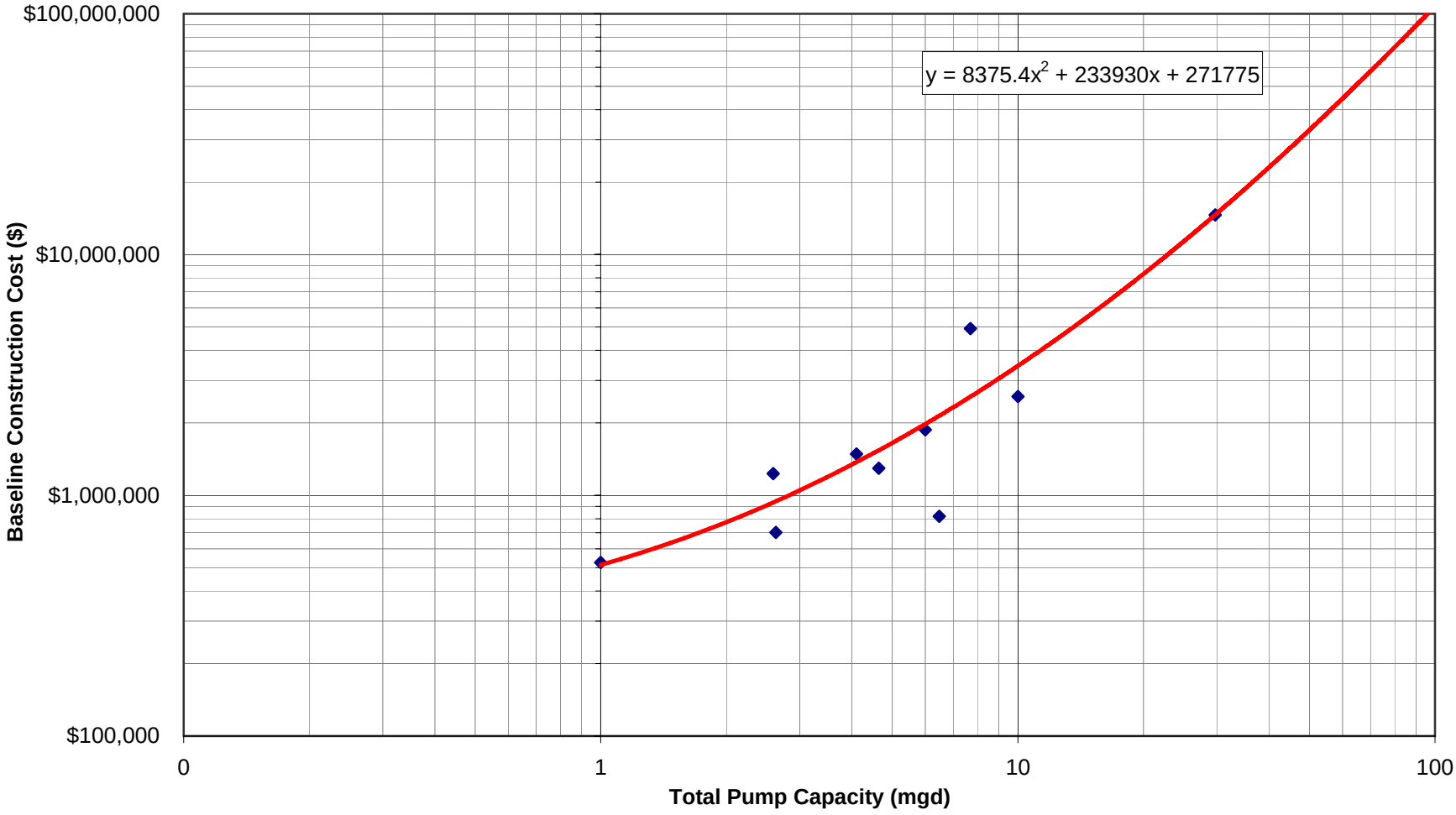
This is the total estimated construction cost, in dollars, of the proposed improvement. Pipeline and lift station. Baseline Construction Costs were developed using the following criteria:

- Pipeline: Calculated by multiplying the estimated length by the unit cost.
- Lift and Pump Stations: Calculated by using the total capacity in the cost curve equation.

7.4.2 Estimated Construction Cost

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project. Consequently, it is appropriate to allow for uncertainties associated with the preliminary layout of a project. Such factors as unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are a few of the items that can increase project costs for which it is wise to make allowances in preliminary estimates. To assist the City in making financial decisions for these future construction projects, contingency costs will be added to the planning budget as percentages of the total construction cost, divided into two categories: Estimated Construction Cost and Capital Improvement Cost.

Since knowledge about site-specific conditions of each proposed project is limited at the master planning stage, a 25 percent contingency was applied to the Baseline Construction Cost to account for unforeseen events and unknown conditions. A 25 percent contingency to account for unknown site conditions such as poor soils, unforeseen conditions,



ENR CCI 20 City Average used for estimating (March 2009) = 8,534

Figure 7.1
Lift Station Cost Versus Capacity Curve
Wastewater Collection System Master Plan
City of Galt

Engineers...Working Wonders With Water™

pw:/Galt/810A00/Cost Estimate/LSCostCurve.xls/LS Cost Curve

environmental mitigations and other unknowns are typical for master planning projects. The Estimated Construction Cost for the proposed sewer system improvement consists of the Baseline Construction Cost plus the 25 percent construction contingency.

7.4.3 Capital Improvement Cost

Other project construction contingency costs are divided into three subcategories, totaling 30 percent: 10 percent engineering, 10 percent construction phase professional services, and 10 percent project administration. Engineering services associated with new facilities include preliminary investigations and reports, ROW acquisition, foundation explorations, preparation of drawings and specifications during construction, surveying and staking, sampling of testing material, and start-up services. For this study, engineering costs are assumed to equal 10 percent of the Estimated Construction Cost.

Construction phase professional services cover such items as construction management, engineering services, materials testing, and inspection during construction. The cost of these items can also vary, but for the purpose of this study, it is assumed that construction phase professional services expenses will equal approximately 10 percent of the Estimated Construction Cost.

Finally, there are project administration costs, which cover such items as legal fees, environmental/CEQA compliance requirements, financing expenses, administrative costs, and interest during construction. The cost of these items can also vary, but for the purpose of this Master Plan, it is assumed that project administration costs will equal 10 percent of the Estimated Construction Cost.

The Capital Improvement Cost is the total of the Estimated Construction Cost (including contingency) plus the other costs discussed in the previous paragraphs.

As shown in the following sample calculation of the Capital Improvement Cost, the total cost of all project construction contingencies (construction, engineering services, construction management, and project administration) is 62.5 percent of the Baseline Construction Cost. Note that contingencies were not applied to land acquisition costs. Calculation of the 62.5 percent is the overall mark-up on the baseline construction cost to arrive at the capital improvement cost. It is not an additional contingency.

Example:

Baseline Construction Cost	$1,000,000
Construction Contingency (25%)	250,000
Estimated Construction Cost	1,250,000
Engineering Cost (10%)	125,000
Construction Management (10%)	125,000
Project Administration (10%)	125,000
Capital Improvement Cost	$1,625,000

A summary of the capital project costs is presented in Table 7.2. This table identifies the projects, provides a brief description of the project, identifies facility size (e.g. pipe diameter and length), and the capital improvement cost. The table also shows the probable phase in which the project would be implemented. The implementation timeframe was based on the priority of each project to correct existing deficiencies or to serve future users.

7.5 CAPITAL IMPROVEMENT IMPLEMENTATION

As discussed in Chapter 6, the CIPs are prioritized based on their urgency to mitigate existing deficiencies and for servicing anticipated growth. It is recommended that improvements to mitigate existing deficiencies be constructed as soon as possible. The deficiencies in the future system have a significant total capital cost that is best distributed based on the order in which the City develops.

The implementation phases are separated into 5-year increments, except for the first phase, which runs from 2009 through 2015. Each project is itemized by phase in Table 7.2 and a summary by phase is provided in Table 7.3. The need for new sewers was based on the City's projection for extending urban services to new developments.

7.6 EXISTING VERSUS FUTURE USER COST SHARE

The improvements proposed in this Master Plan either benefit existing users, or are required to service new development and future users. Some of the projects provide benefit to both existing and future users. An opinion of benefit to future users, based on preliminary project information, was included in Table 7.2. It was assumed that projects intended to correct existing deficiencies provide no benefit to future users, and would be required regardless of future development. It was also assumed that future users would benefit entirely from the construction of facilities necessary to serve future development.

A breakdown in existing and future user cost share of the proposed projects by phase is summarized in Table 7.3. Table 7.4 summarizes the breakdown in cost for the different facility categories (e.g. trunk sewers and lift stations).

7.7 COST OF SERVICE

The existing and future user capital costs discussed above were used to determine a cost of service to existing ratepayers and future customer connections. The following is not a rate study, fee program, or development impact fee analysis. It is a simplified assessment of the costs that the City might need to recover from existing ratepayers and future development to pay for the proposed Master Plan projects. This analysis serves only to

Table 7.2 Capital Improvement Projects
Wastewater Collection System Master Plan
City of Galt

Figure No.	Type of Improvement	Description/ Street	Description / Limits	Pipeline Cost Schedule (A or B)	Project Length/Size and Cost: Ex. Size/ Diam. (in)	New Size/ Diam. (in)	Replace/ New	Length (ft)	Capital Improvement Cost[2],[3] ($)	Capital Improvement Phasing: Phase 1 2009-2015 ($)	Phase 2 2016-2020 ($)	Phase 3 2021-2025 ($)	Phase 4 2026-2030 ($)	Future Users Benefit (%)	Reimbursement Category: Existing Improvements ($)	Future Improvements ($)
Existing System Improvements																
Pipelines																
Live Oak Trunk Sewer and Midway Force Main																
M-1	Pipe	Live Oak Avenue	McFarland Street to Sargent Avenue	B	-	42	New	3,200	$ 1,612,000	$ 1,612,000				46%	$ 870,480	$ 741,520
M-2	Force Main	Midway Avenue	Live Oak Avenue to WWTP	B	-	24	New	10,700	$ 2,922,000	$ 2,922,000				46%	$ 1,577,880	$ 1,344,120
B Street Trunk Sewer																
B-1	Pipe	B Street	3rd Street to 7th Street	A	-	15	New	1,550	$ 439,000	$ 439,000				0%	$ 439,000	$ -
B-1A	Pipe/Casing[1]	B Street	UPRR Crossing	A	-	15/30	New	100	$ 180,000	$ 180,000				0%	$ 180,000	$ -
B-2	Pipe	7th Street/C Street	B Street to Lincoln Way	A	-	10	New	600	$ 114,000	$ 114,000				0%	$ 114,000	$ -
B-3	Pipe	7th Street/A Street	B Street to Lincoln Way	A	-	12	New	500	$ 114,000	$ 114,000				0%	$ 114,000	$ -
B-4	Pipe	3rd Street	A Street to B Street	A	-	10	New	660	$ 125,000	$ 125,000				0%	$ 125,000	$ -
B-5	Force Main	A Street/3rd Street	McFarland Street to B Street	A	-	14	New	850	$ 213,000	$ 213,000				0%	$ 213,000	$ -
Elm Trunk Sewer																
E-1	Pipe	Elm Avenue	McFarland Street to west of Industrial Way	A	8	10	Replace	900	$ 169,000	$ 169,000				0%	$ 169,000	$ -
Quail Hollow Trunk Sewer																
QH-1	Pipe	Oberlin Way/Trudy Way	Lorna Way to McFarland Street	A	10/12	15	Replace	400	$ 114,000	$ 114,000				0%	$ 114,000	$ -
Sewer Main Replacement Program																
-	Pipe	Various Locations	Sewer Main Replacement Program. Average size is 10".	A	10	10	Replace	50,000	$ 9,429,000	$ 3,000,000	$ 2,143,000	$ 2,143,000	$ 2,143,000	0%	$ 9,429,000	$ -
Lift Stations[4]																
B-LS	Lift Station	B Street/4th Street	A Street LS Relocation (3.6 mgd Firm Capacity)	-	5.2 mgd	5.4 mgd	Relocate/Upgrade	-	$ 2,891,000	$ 2,891,000				0%	$ 2,891,000	$ -
	Land Acquisition				-	0.25 acres	Relocate/Upgrade	-	$ 98,000	$ 98,000				0%	$ 98,000	$ -
LO-LS	Lift Station	Live Oak and Sargent Road	Live Oak Lift Station (14.0 mgd Firm Capacity)	-	-	17.5 mgd	New	-	$ 11,263,000	$ 11,263,000				46%	$ 6,082,020	$ 5,180,980
	Land Acquisition				-	0.25 acres	New	-	$ 98,000	$ 98,000				46%	$ 52,920	$ 45,080
-	Lift Station	Various Locations	Lift Station Rehab. Program. Average Capacity is 2.6 mgd (Total).	-	2.6 mgd	2.6 mgd	Rehab/Replace	1 Project/Phase	$ 6,088,000	$ 1,522,000	$ 1,522,000	$ 1,522,000	$ 1,522,000	0%	$ 6,088,000	$ -
								Existing Improvements Subtotal	**$ 35,869,000**	**$ 24,874,000**	**$ 3,665,000**	**$ 3,665,000**	**$ 3,665,000**		**$ 28,557,300**	**$ 7,311,700**
Future System Improvements																
Pipelines																
Southeast Trunk Sewer																
SE-1	Pipe	Live Oak Avenue/Undeveloped Area near Deadman Gulch	McFarland Street to Simmerhorn Road	A	-	27	New	10,200	$ 4,721,000		$ 4,721,000			100%	$ -	$ 4,721,000
SE-1A	Pipe/Casing[1]	Live Oak Avenue	Highway 99 Crossing	A	-	27/48	New	300	$ 865,000		$ 865,000			100%	$ -	$ 865,000
SE-1B	Pipe/Casing[1]	Undeveloped Area west of Carillion Boulevard	UPRR Crossing	A	-	27/48	New	100	$ 288,000		$ 288,000			100%	$ -	$ 288,000
SE-2	Pipe	Simmerhorn Road	West of Carillion Boulevard to east of Marengo Road	A	-	24	New	4,000	$ 1,646,000			$ 1,646,000		100%	$ -	$ 1,646,000
SE-3	Pipe	East of Marengo Road	Simmerhorn Road to north of Simmerhorn Road	B	-	21	New	1,300	$ 328,000			$ 328,000		100%	$ -	$ 328,000
SE-4	Pipe	East of Marengo Road	North of Simmerhorn Road to south of UPRR	B	-	18	New	1,300	$ 281,000			$ 281,000		100%	$ -	$ 281,000
SE-5	Pipe	East of Marengo Road	North of Simmerhorn Road to south of UPRR	B	-	15	New	1,400	$ 278,000			$ 278,000		100%	$ -	$ 278,000
SE-6	Pipe	Southeast of UPRR line	North of Simmerhorn Road to south of UPRR	B	-	10	New	2,500	$ 330,000				$ 330,000	100%	$ -	$ 330,000
SE-7	Pipe	East of McFarland Street	Live Oak Avenue to north of Live Oak Avenue	B	-	10	New	2,200	$ 291,000		$ 291,000			100%	$ -	$ 291,000
SE-8	Pipe	Undeveloped Area west of Carillion Boulevard	Simmerhorn Road to Boessow Road	B	-	15	New	2,900	$ 574,000		$ 574,000			100%	$ -	$ 574,000
SE-9	Pipe	Marengo Road Extension	Simmerhorn Road to Boessow Road	B	-	10	New	2,700	$ 356,000			$ 356,000		100%	$ -	$ 356,000
SE-10	Pipe	Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	12	New	1,300	$ 206,000				$ 206,000	100%	$ -	$ 206,000
SE-11	Pipe	West of Cherokee Lane	Simmerhorn Road to north of Simmerhorn Road	B	-	10	New	700	$ 93,000				$ 93,000	100%	$ -	$ 93,000
SE-12	Pipe	North of Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	10	New	1,300	$ 172,000				$ 172,000	100%	$ -	$ 172,000
SE-13	Pipe	North of Simmerhorn Road	East of Marengo Road to west of Cherokee Lane	B	-	10	New	1,300	$ 172,000				$ 172,000	100%	$ -	$ 172,000
Northeast Trunk Sewer																
NE-1	Pipe	Undeveloped Area near Deadman Gulch	Carillion Boulevard to Marengo Road	B	-	15	New	2,400	$ 475,000	$ 475,000				100%	$ -	$ 475,000
NE-2	Pipe	Marengo Road	South of Elk Hills Drive to South of Lake Park Avenue	A	-	12	New	3,600	$ 814,000		$ 814,000			100%	$ -	$ 814,000
NE-3	Pipe	South of Twin Cities Road	Marengo Road to east of Marengo Road	B	-	10	New	3,300	$ 436,000			$ 436,000		100%	$ -	$ 436,000

Table 7.2 Capital Improvement Projects
Wastewater Collection System Master Plan
City of Galt

Figure No.	Type of Improvement	Description/ Street	Description / Limits	Pipeline Cost Schedule (A or B)	Project Length/Size and Cost: Ex. Size/ Diam. (in)	New Size/ Diam. (in)	Replace/ New	Length (ft)	Capital Improvement Cost[2],[3] ($)	Capital Improvement Phasing: Phase 1 2009-2015 ($)	Phase 2 2016-2020 ($)	Phase 3 2021-2025 ($)	Phase 4 2026-2030 ($)	Future Users Benefit (%)	Reimbursement Category: Existing Improvements ($)	Future Improvements ($)
NE-4	Pipe	North of UPRR	Marengo Road to east of Marengo Road	B	-	10	New	2,800	$ 369,000			$ 369,000		100%	$ -	$ 369,000
Commercial Trunk Sewer																
C-1	Pipe	North of Twin Cities Road	McFarland Street to West of Highway 99	B	-	18	New	800	$ 172,000			$ 172,000		100%	$ -	$ 172,000
C-2	Pipe	North of Twin Cities Road	West of Highway 99 to East of Highway 99	B	-	15	New	2,100	$ 416,000			$ 416,000		100%	$ -	$ 416,000
C-2A	Pipe/Casing[1]	North of Twin Cities Road	Highway 99 Crossing	A	-	15/30	New	400	$ 720,000			$ 720,000		100%	$ -	$ 720,000
C-3	Pipe	North of Twin Cities Road	East of Highway 99 to Bergeron Road	B	-	12	New	800	$ 127,000				$ 127,000	100%	$ -	$ 127,000
C-4	Pipe	North of Twin Cities Road	Bergeron Road to McKenzie Road	B	-	10	New	2,800	$ 369,000				$ 369,000	100%	$ -	$ 369,000
C-5	Pipe	East of Highway 99	North of Twin Cities Road to Mingo Road	B	-	10	New	2,100	$ 278,000				$ 278,000	100%	$ -	$ 278,000
C-6	Pipe	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	1,500	$ 198,000				$ 198,000	100%	$ -	$ 198,000
C-7	Pipe	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	1,000	$ 132,000				$ 132,000	100%	$ -	$ 132,000
C-8	Force Main	West of Highway 99	North of Twin Cities Road to South of Mingo Road	B	-	10	New	200	$ 24,000			$ 24,000		100%	$ -	$ 24,000
C-8A	Pipe/Casing[1]	North of Twin Cities Road	UPRR Crossing	A	-	10/18	New	100	$ 141,000			$ 141,000		100%	$ -	$ 141,000
Lift Stations[4]																
VO-LS	Lift Station	Vintage Oak Avenue and Carrilion Boulevard	Vintage Oak Lift Station Upgrade (3.8 mgd Firm Capacity)	-	4.3 mgd	5.7 mgd	Upgrade	-	$ 3,050,000	$ 3,050,000				100%	$ -	$ 3,050,000
C-LS	Lift Station	East of Railraod Tracks near WWTP	Commercial Lift Station (1.4 mgd Firm Capacity)	-	-	2.8 mgd	New	-	$ 1,612,000			$ 1,612,000		100%	$ -	$ 1,612,000
	Land Acquisition				-	0.25 acres	New	-	$ 98,000			$ 98,000		100%	$ -	$ 98,000
								Future Improvements Subtotal	**$ 20,032,000**	**$ 3,525,000**	**$ 7,553,000**	**$ 6,877,000**	**$ 2,077,000**		**$ -**	**$ 20,032,000**
								CIP Total (Existing and Future)	**$ 55,901,000**	**$ 28,399,000**	**$ 11,218,000**	**$ 10,542,000**	**$ 5,742,000**		**$ 28,557,300**	**$ 27,343,700**

Notes:
1. Proposed casings size and carrier pipe size.
2. Baseline Construction Cost plus 25% to account for unforeseen events and unknown conditions.
3. Estimated Construction Cost plus 30% to cover other costs including Engineering, Construction Management, and Project Administration.
4. Lift station capacities refer to the total capacity unless noted otherwise.
5. Costs are based on the Engingeering News Record Construction Cost Index 20-city average of 8534 (March 2009).

Table 7.3	Existing Versus Future User Cost Share Wastewater Collection System Master Plan City of Galt				
	Implementation Phase				
Reimbursement Category	2009-15 ($, mill.)	2016-20 ($, mill.)	2021-25 ($, mill.)	2026 - 30 ($, mill.)	Total ($, mill.)
Existing User [2]	17.6	3.7	3.7	3.7	**28.6**
Future User [3]	10.8	7.6	6.9	2.1	**27.3**
Total	**28.4**	**11.2**	**10.5**	**5.7**	**55.9**

Notes:
1. Costs are based on ENR CCI 20 City average of 8534 (March 2009)
2. Projects are funded through user rates.
3. Projects are expected to be funded through sewer development impact fees collected by the City or by developers.

Table 7.4	Existing Versus Future Cost by Facility Type Wastewater Collection System Master Plan City of Galt		
Reimbursement Category	Pipelines ($, mill.)	Lift Stations ($, mill.)	Total ($, mill.)
Existing Users[2]	13.3	15.2	**28.6**
Future Users[3]	17.4	10.0	**27.3**
Total	**30.7**	**25.2**	**55.9**

Notes:
1. Costs are based on the ENR CCI 20 City average of 8534 (March 2009)
2. Projects are funded through user rates.
3. Projects are expected to be funded through sewer development impact fees collected by the City or by developers.

assist the City in determining whether a rate or development impact fee increase might be needed to finance the proposed CIPs. This Master Plan analysis is simply a high level calculation that provides the potential order of magnitude assessment and brackets the possible costs. A more detailed rate/development impact study should be conducted to determine the magnitude of a possible increase to fund the proposed CIPs.

7.7.1 Existing Users Fee for Service

The City collects sewer utility rates to pay for such services including but not limited to operations and maintenance, capital replacement and improvement, administration, and to establish a capital reserve. The capital costs to implement the proposed Master Plan projects fall under the capital replacement and improvement of existing system components. These costs are spread over approximately 7,200 existing service connections.

In the 2008/09 Wastewater Fund, the City's capital improvements budget equaled about $3.4 million. The proposed Phase 1 (2009 through 2015) Master Plan improvements average $4.5 million per year, which is a significant increase above the City's currently planned $0.88 million 2009/10 budget for capital improvements. As the City implements the proposed Master Plan projects, an increase in the rates could be necessary to finance them if they are implemented per the phasing schedule assumed in this Master Plan.

The total existing system Master Plan capital costs equal approximately $28.6 million. One possible scenario to finance these projects might include two phases of bond financing. One bond could cover the work for Phases 1 and 2 (2009 through 2020), and a second bond could finance projects for Phases 3 and 4 (2021 through 2030). For this simplified analysis, we assumed that the first bond would be for $21.3 million and the second for $7.4 million, both paid off over 30 years at an interest rate of 5 percent. Although the number of current service connections is approximately 7,200, this number is expected to increase.

Since the 2005 Urban Water Management Plan (UWMP) forecasts the possible increase in future water service connections, we used this report to estimate the increase in sewer service connections. We assumed that the number of future water connections was equal to the number of future sewer connections. Table 4-1 of the 2005 UWMP provides guidance for the increase in connections to 2030, and future growth was assumed at three percent after year 2030. However, we revised this information to reflect the anticipated growth reported in the 2030 General Plan, which based on population and commercial development, effectively increased the projections of future connections above those reported in the 2005 UWMP. In other words, using the 2030 General Plan guidance, we have estimated that there will be more ratepayers available to pay for these two bonds than is anticipated in the UWMP, thereby reducing the increase on the individual ratepayers to finance this debt. We assumed the mid-point of each repayment term to estimate the number of ratepayers available to pay back the debt. Below is a summary of the assumptions regarding the number of future connections used to estimate the possible increase in rates necessary to finance the bond debt:

- 7,200 service connections in Year 2008
- 13,540 service connections in Year 2020 (Table 4.1 from 2005 UWMP reported 11,393 service connections)
- 18,385 service connections in Year 2030 (Table 4.1 from 2005 UWMP reported 16,484 service connections)
- Three percent annual increase in service connections after Year 2030
- Average number of service connections to pay Bond 1 was 15,950
- Average number of service connections to pay Bond 2 was 23,350

Based on these assumptions, the monthly increase necessary to finance the first bond to fund the proposed capital replacement projects could be approximately $7.25 per service

connection. In year 2020 when the second bond is secured, the rate could increase to about $9 dollars per connection.

In 2009, the City's wastewater utility rate for single family residential (without a water meter) was a flat rate of $25.81 per month (excludes the wastewater treatment plant [WWTP] Upgrade Account). A preliminary rate study completed by the City indicated that a 14 percent rate increase was required to pay for off-hauling biosolids from the WWTP and to implement capital projects identified in the City's sanitary sewer management plan (SSMP). However, City staff recommended a 4.1 percent increase to $26.87 per month. The projects identified in this Master Plan would replace those recommended in the SSMP and could increase the utility rates to nearly $35 per month to finance the projects.

This simplified analysis does not take into account any existing City bonds (if any) that are being paid, the type of bond that would be used, alternative payment schedules, or class of service. It is recommended that a comprehensive rate analysis be completed to quantify the impacts to existing ratepayers and to the City's proposed wastewater rates. The possible rate increase will likely vary depending on the class of service (e.g. single and multi-family residential, commercial, and industrial) and determining this level of detail for setting rates goes beyond the scope of this study.

7.7.2 Future Users Development Impact Fees

The City collects development impact fees to finance capital expenses associated with increased capacity of the wastewater collection system. The City's current development impact fee is approximately $8,700 and includes a sewer connection fee of approximately $4,900 and a Supplemental WWTP fee of about $3,800. The sewer connection fee portion finances sewers and lift stations. The total future system Master Plan capital costs equal approximately $27.3 million; $10.0 million would be needed for lift stations and $17.4 million for sewer pipeline construction. Depending upon the City's future development plan, the pipelines could be either funded through the development impact fees, or directly through the developers, with a separate reimbursement fund established for others who benefit from the improvement.

As mentioned above, based on the 2030 General Plan, the number of future sewer connections added to the City will likely increase above the projections summarized in Table 4-1 of the 2005 UWMP. Based on the projected build-out wastewater demands, by year 2030 there could be between 18,000 to 20,000 connections within the General Plan boundaries. This represents an increase of 10,800 to 12,800 in future connections that would pay impact fees to finance future lift stations and sewer pipelines. Therefore, the possible capital cost of collection system improvements per future connection could range from $2,200 to $2,600. Note that these costs do not include impact fees associated with the WWTP.

The development impact fee is significantly influenced by the number of connections, and will likely vary based on the class of service. The City should complete a detailed development impact fee study to quantify the appropriate fee by class of service to finance increases in capacity for future users.

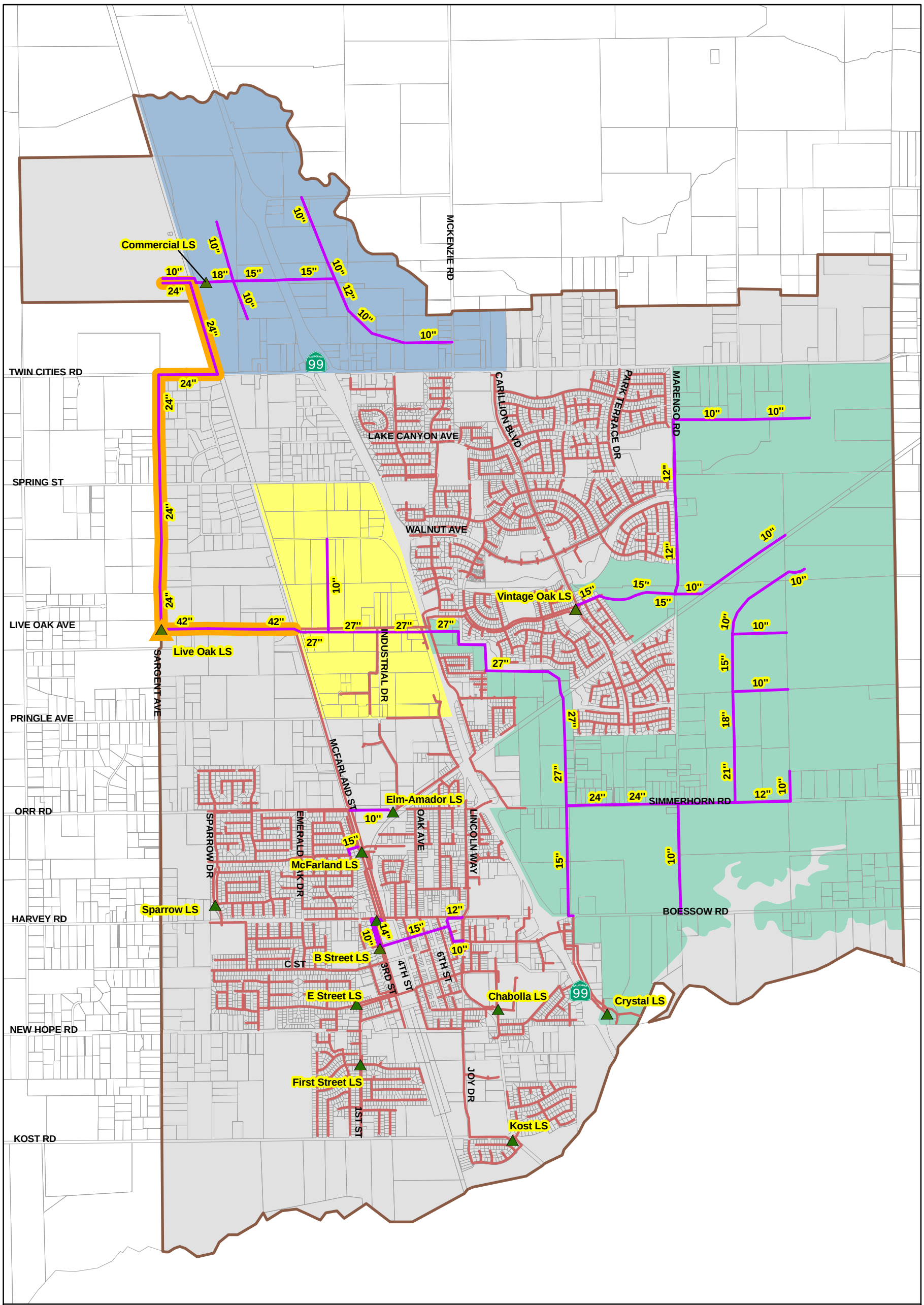
7.8 FUTURE USER BENEFIT ZONES

The City requested that Carollo evaluate the feasibility of establishing “benefit zones” for the sewer system where the development impact fee would be based on the development location. The rationale is that development located farthest from urban services should pay a greater impact fee since the costs to extend services out to these users is greater when compared to an infill development or development adjacent to existing services.

This approach could be applied to assessing impact fees for wastewater collection systems. The reason is that the City’s wastewater collection system varies in size and depth depending on where future development is proposed. In other words, large diameter trunk sewers and lift stations may be necessary to serve future developments on the perimeter of the sphere of influence, whereas an infill development near existing infrastructure may only need a small sewer main to provide service. Some might argue that developments at the farthest reaches of study area should pay higher impact fees when compared to developments adjacent to or within City limits because the type of infrastructure required to extend urban service to undeveloped areas is not uniform for all future users and depends on location. This is a policy decision that will not be discussed in this Master Plan. However, basing development impact fees on future user benefit zones or on location is a possible approach for the City, in lieu of a uniform sewer connection fee for the entire City. The discussion below is not a recommendation for the City to pursue future user benefit zones when establishing impact fees, but simply a possible scenario for the City to investigate during a more detailed rate/development impact fee study.

7.8.1 Common Use Facilities

There are proposed sewer facilities that will serve all users (existing and future) regardless of location in the collection system. These facilities are labeled common use facilities in Figure 7.2 and include the proposed influent pump station at the WWTP and the new 42-inch diameter trunk sewer. These projects are currently in the planning/design phase and would be built regardless of future development. However, they will be designed with sufficient capacity to serve future users because it is necessary and practical to account for planned growth outlined in the 2030 General Plan. For existing ratepayers, sewer improvements have historically been paid for through capital improvement fees included in the rates, as a benefit to the existing system. For future users, their share of the costs for these common use facilities would be paid through the sewer connection fees. Due to the location of these facilities in the collection system, there is uniform benefit to all future users. Therefore, all future users should support the costs for these facilities equally.



Legend

Proposed Wastewater Collection System

- Lift Station
- Future Sewer
- Existing Sewer
- Common Use Facilities
- General Plan Boundary
- Parcels

Benefit Zone

- Zone 1 (East of Highway 99)
- Zone 2 (Industrial)
- Zone 3 (North of Twin Cities Road (TCR))

Feet
0 1,250 2,500

Figure 7.2
Common Use Facilities
Wastewater Collection System Master Plan
City of Galt

7.8.2 Zone 1: East of Highway 99

Future development east of Highway 99 was grouped into future user benefit Zone 1. Future sewer facilities shown in Figure 7.2 that are east of Highway 99 and south of Twin Cities Road will benefit future development within Zone 1. Note that the boundary for Zone 1 is simply for discussion purposes and would likely be modified if a more detailed rate/development impact fee study were undertaken. The boundary would likely be modified to account for existing community facility districts, City limits, and existing developed areas already receiving sewer service. Therefore, the boundaries discussed in this section should be considered provisional and subject to change.

7.8.3 Zone 2: Industrial

Future development within the Zone 2 boundary will benefit from the construction of proposed sewers, including a 27-inch diameter sewer shown in Live Oak Avenue. Note that this 27-inch diameter sewer will also benefit future users in Zone 1 because their wastewater will be conveyed in this sewer also.

7.8.4 Zone 3: North of Twin Cities Road

Future development north of Twin Cities Road was grouped into future user benefit Zone 3. The sewer facilities shown in Figure 7.2 that are north of Twin Cities Road will benefit future development within Zone 3.

7.8.5 Future Infill Development

Future infill development throughout the City will likely require the installation of smaller sewer mains (8-inch diameter) to extend urban service. Planning the layout of these sewer mains goes beyond the scope of this study. Even though infill development might not be included in one of the zones discussed above, it would still be subject to sewer impact fees.

APPENDIX A - DESCRIPTION OF DIFFERENT LAND USES (GENERAL PLAN EXCERPTS)

Land Use Designations and Standards

The Land Use and Circulation Diagram (Figure LU-1) shows 13 residential and non-residential land use designations. These 13 land use designations are described, with photo examples, in Table LU-1. State law mandates that general plans include standards of population density and building intensity for all of the territory covered by the plan. To satisfy this requirement, the General Plan includes such standards for each of the land use designations appearing on the Land Use and Circulation Diagram. These standards are stated differently for residential and non-residential development.

Residential Uses

Standards of population density for residential uses can be derived by multiplying the maximum allowable number of dwelling units per gross acre by the average number of persons per dwelling unit assumed for the applicable residential designation. Standards of building intensity for residential uses are stated as the allowable range of dwelling units per gross acre.

The assumed average number of persons per dwelling unit for each residential designation has been extrapolated from population and housing unit estimates prepared by the Sacramento Area Council of Governments (SACOG) and the State of California Department of Finance. These are summarized in Table LU-2.

Non-Residential Uses

Standards of building intensity for non-residential uses in the General Plan are stated as maximum floor-area ratios (FARs). A floor-area ratio is the ratio of the gross building square footage on a lot to the net square footage of the lot.

For example, on a lot with 10,000 net square feet of land area, a FAR of 1.00 will allow 10,000 square feet of gross building floor area to be built, regardless of the number of stories in the building (e.g., 5,000 square feet per floor on two floors or 10,000 square feet on one floor). On the same lot, a FAR of 0.50 would allow 5,000 square feet of floor area, and a FAR of 0.25 would allow 2,500 square feet. The diagram below illustrates conceptually how buildings of one, two, and four stories could be developed on a given lot with a FAR of 1.00.

Standards of population density for non-residential uses can be derived by multiplying one acre (43,560 square feet) by the applicable FAR and then dividing by the assumed average square footage of building area per employee. The assumed average square footage of nonresidential building floor area per employee is based on historic averages and market studies. These are summarized in Table LU-2.

Various Building Configurations Representing a Floor-Area Ratio of 1.00 on the Same Lot

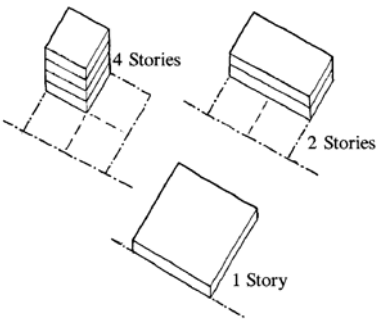
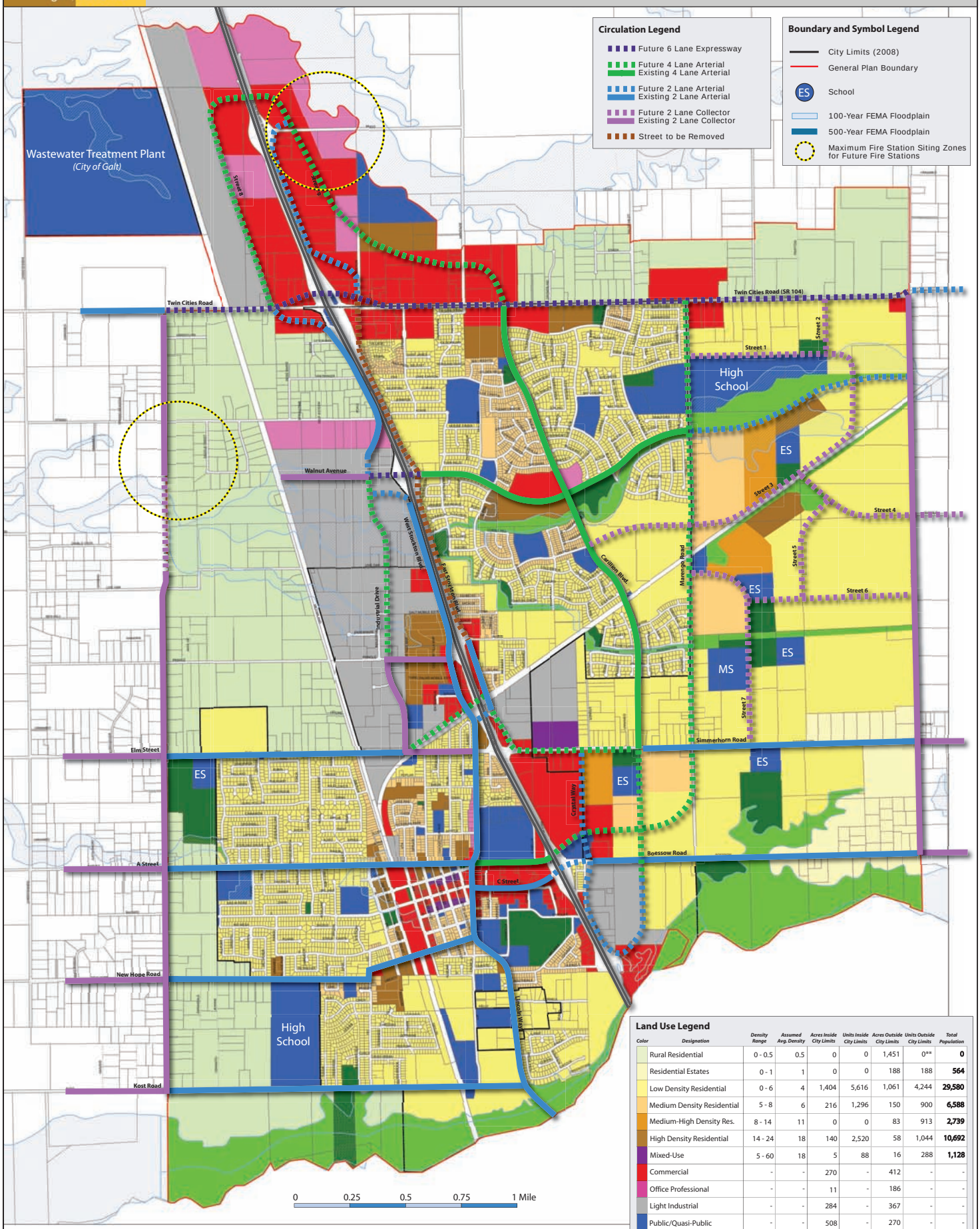


Table LU-1 Land Uses Categories	
	Rural Residential (RR) Provides for single family detached homes and secondary residential units on 2 acre minimum lots without full urban services and with limited agricultural uses. This use is in the Planning Area but outside of the 2007 city limits. This use is typically located on the far western and northern parts of the Planning Area to provide transition between urban and rural uses.
	Residential Estates (RE) Provides for single family detached executive housing opportunities, secondary residential units, and public and quasi-public uses on large lots with limited urban services.
	Low-Density Residential (LDR) Provides for single family detached homes, secondary residential units, public and quasi-public uses, and similar and compatible uses. This use is typically located in areas which include full urban services, and away from industrial, intensive commercial, and large-scale infrastructure (i.e., power substations, wastewater treatment plant).
	Medium-Density Residential (MDR) Provides for single family detached homes, secondary residential units, duplexes, public and quasi-public uses, and similar and compatible uses. Attached single- and multi-family homes are also allowed with a conditional use permit. This use is typically located adjacent to low-density residential areas and provides a transition between low-density and medium-high density residential.
	Medium-High Density Residential (MHDR) Provides for single family detached and attached homes, secondary residential units, public and quasi-public uses, and similar, compatible uses. This use provides a transition from lower density residential areas and is often close to commercial/office professional areas, and arterial streets.
	High-Density Residential (HDR) Provides for single-family attached homes, multi-family residential units, and similar and compatible uses. This use is typically located near medium-high density and/or near commercial/office professional uses or arterial streets and highways.

	Mixed-Use (MU) Provides for residential uses combined with compatible uses such as retail, service, restaurants, banks, entertainment uses, professional and administrative offices, and public and quasi-public uses. This use is typically located in the downtown and other parts of the city that serve as community centers with residential, commercial, and employment uses in the immediate vicinity.
	Commercial (C) Provides primarily for regional, neighborhood, and locally-oriented retail and service uses, restaurants, banks, entertainment uses, public and quasi-public uses, and similar and compatible uses. This use is typically located downtown and in areas of good visibility, such as arterials or major intersections.
	Office Professional (OP) Provides for office parks, office buildings, and quasi-public uses. This use is typically located on arterial and collector streets, and in downtown if it is in scale with existing buildings.
	Industrial (I) Provides for research and development, warehouses, and manufacturing, and quasi-public uses. This use is typically located away from residential uses and in the immediate vicinity of State Route 99 and/or the Union Pacific mainline railroad tracks.
	Public/Quasi-Public (PQ) Provides for public facilities such as schools, fire stations, hospitals, sanitariums, libraries, museums, government offices and courts, churches, meeting halls, cemeteries and mausoleums, public facilities, and similar and compatible uses. This use is typically located throughout the community.
	Parks (P) Provides for active and passive recreational uses, habitat protection, and public/quasi-public uses. This use is located throughout the community.
	Open Space (OS) Provides for passive outdoor recreational uses, habitat protection, watershed management, public and quasi-public uses, areas that contain public health and safety hazards such as floodways, and areas containing environmentally-sensitive features. This use is located throughout the community.

general plan 2030 a vision for the future galt, california

FIGURE LU-1: LAND USE AND CIRCULATION DIAGRAM



Land Use Legend

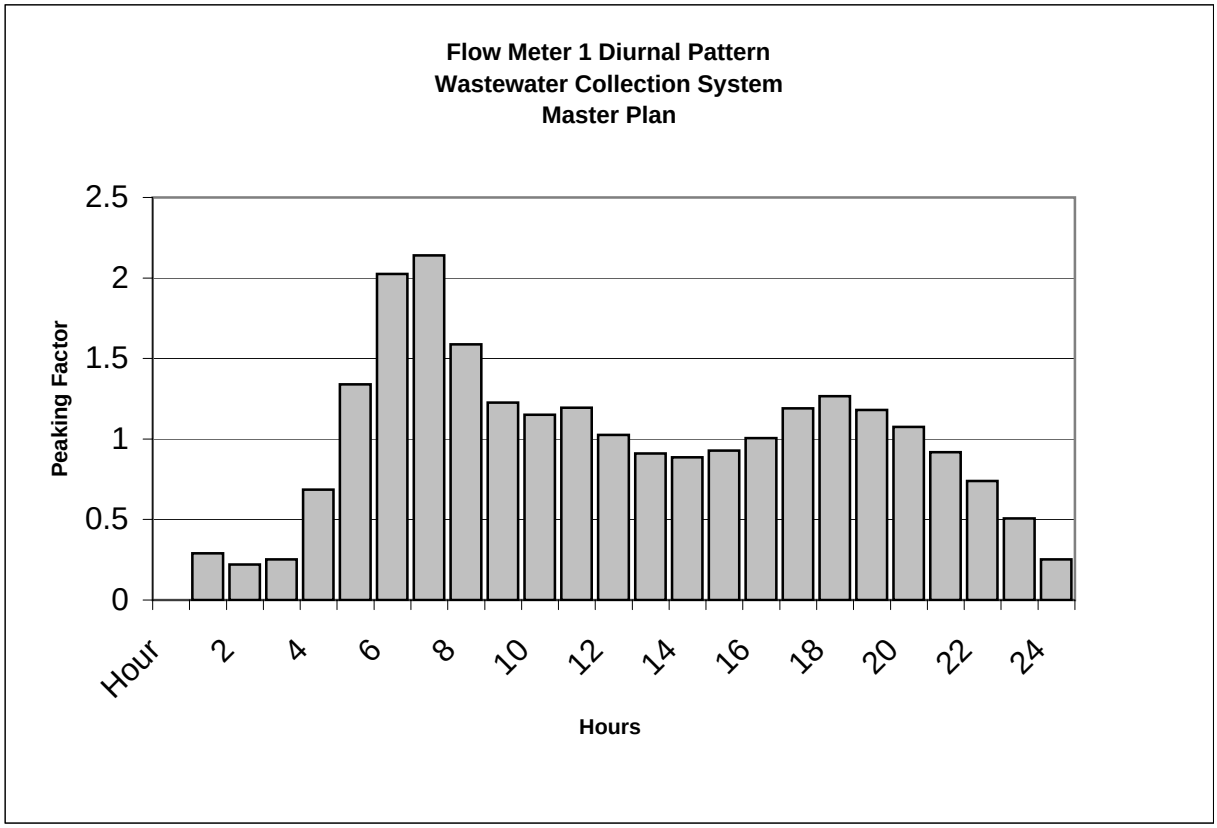
Color	Designation	Density Range	Assumed Avg. Density	Acres Inside City Limits	Units Inside City Limits	Acres Outside City Limits	Units Outside City Limits	Total Population
	Rural Residential	0 - 0.5	0.5	0	0	1,451	0**	**0**
	Residential Estates	0 - 1	1	0	0	188	188	**564**
	Low Density Residential	0 - 6	4	1,404	5,616	1,061	4,244	**29,580**
	Medium Density Residential	5 - 8	6	216	1,296	150	900	**6,588**
	Medium-High Density Res.	8 - 14	11	0	0	83	913	**2,739**
	High Density Residential	14 - 24	18	140	2,520	58	1,044	**10,692**
	Mixed-Use	5 - 60	18	5	88	16	288	**1,128**
	Commercial	-	-	270	-	412	-	-
	Office Professional	-	-	11	-	186	-	-
	Light Industrial	-	-	284	-	367	-	-
	Public/Quasi-Public	-	-	508	-	270	-	-
	Parks	-	-	65	-	140	-	-
	Open Space	-	-	152	-	420	-	-
	Total			*3,055*	*9,522*	*4,802*	*7,577*	***51,291***

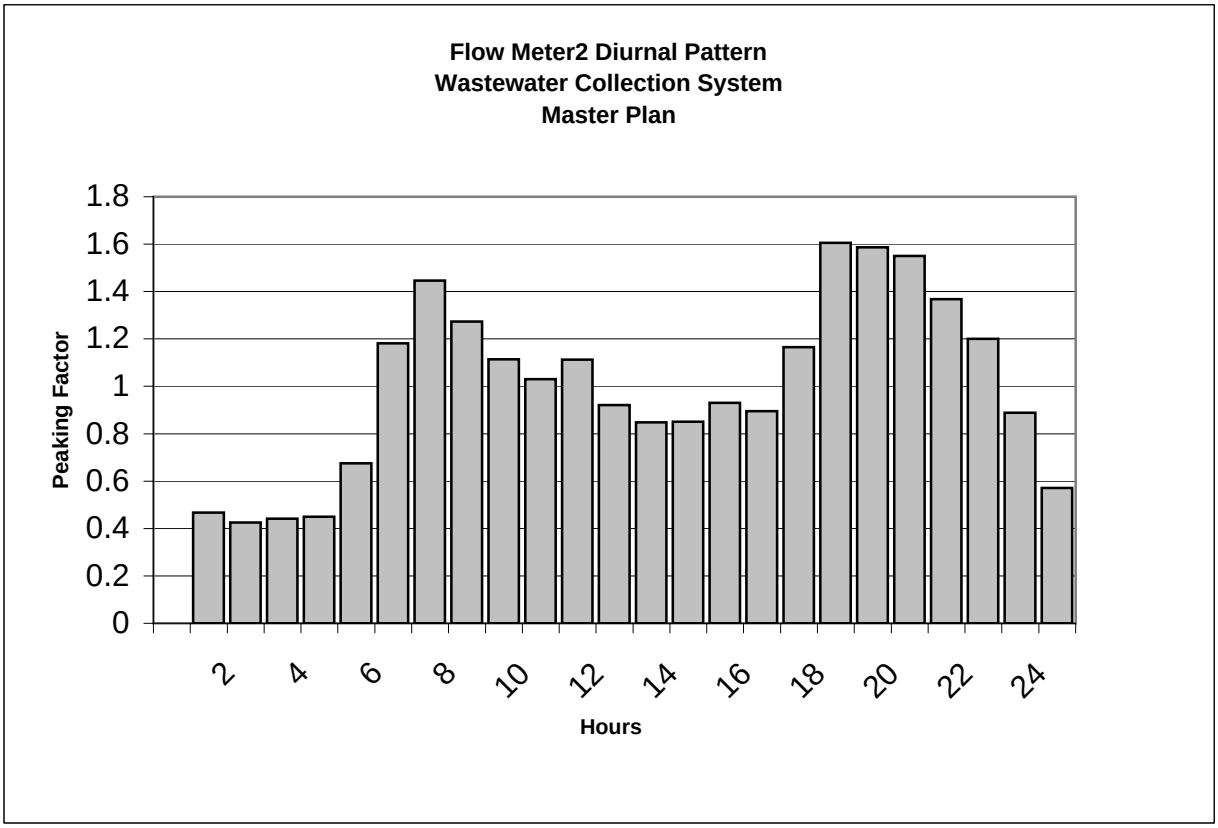
Notes:
1) The population associated with this diagram is approximately 51,291 total.
2) This diagram includes minor changes from assumptions contained in the Final EIR, including removal of designations for three fire stations, resulting in de minimis increases in residential, park, and commercial land uses.
** Units per Gross Acre*
*** Rural Residential-designated lands are not anticipated to be annexed into the City limits during this General Plan*

Updated April 2009

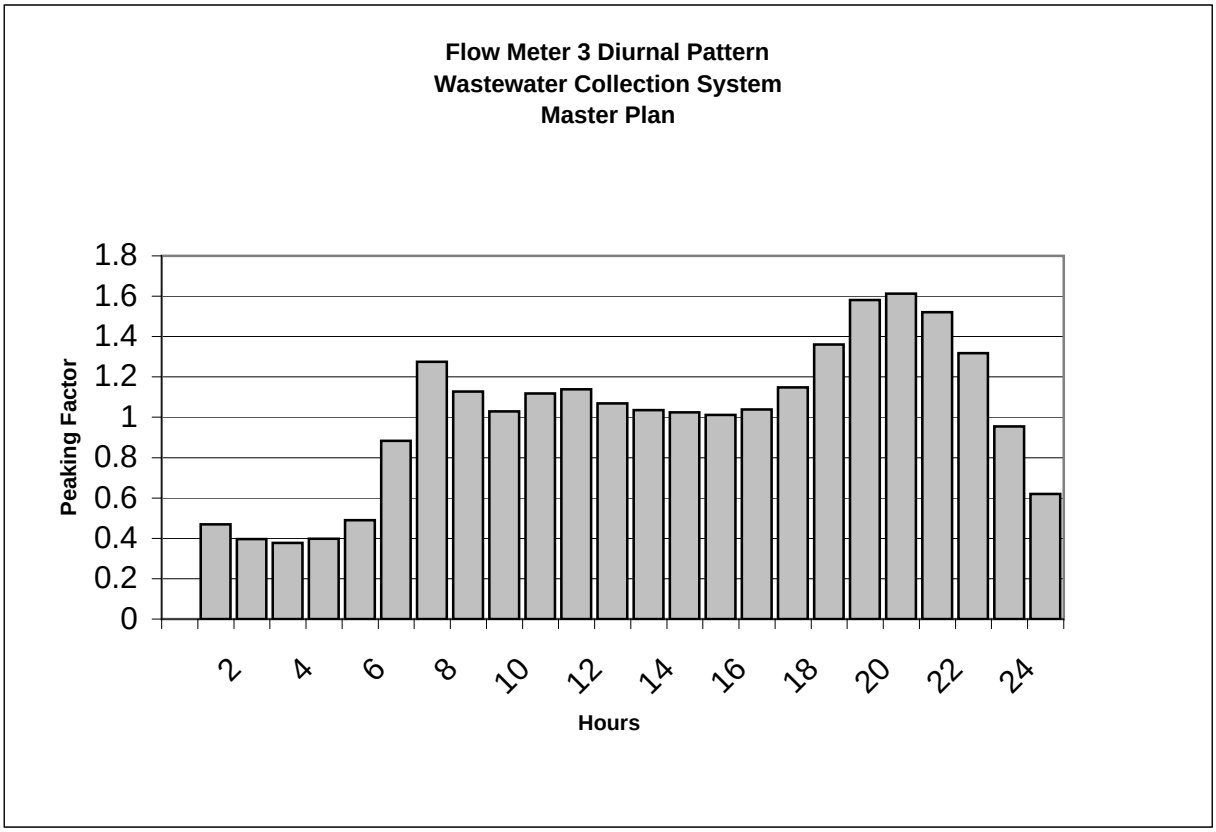
APPENDIX B - DIURNAL CURVES

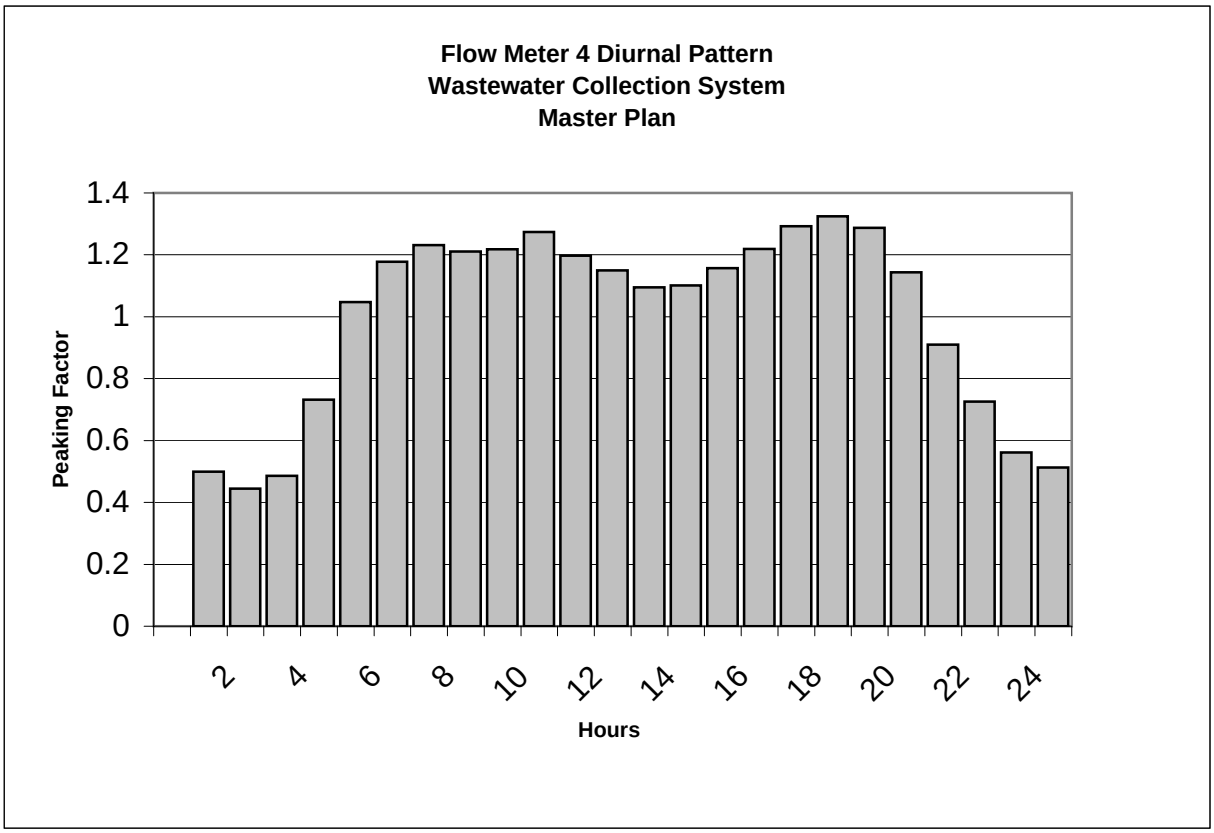
Diurnal Patterns



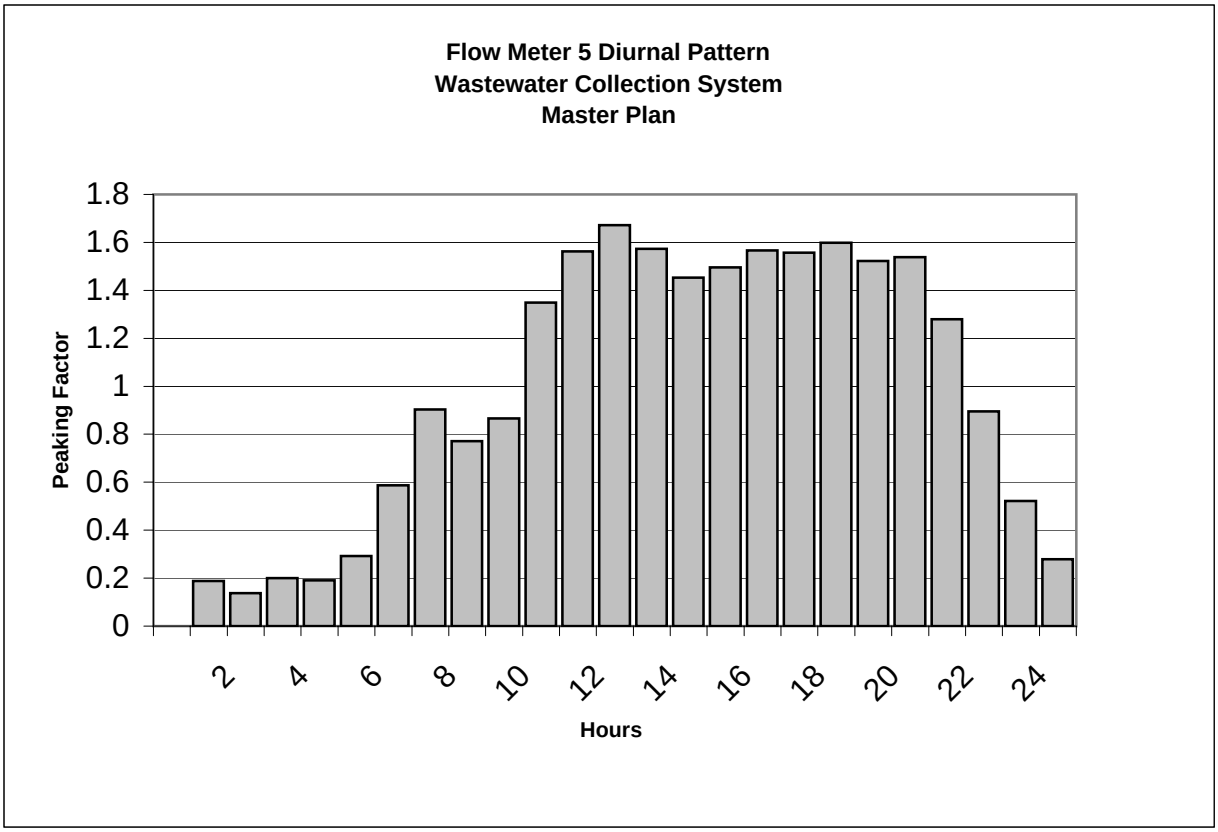


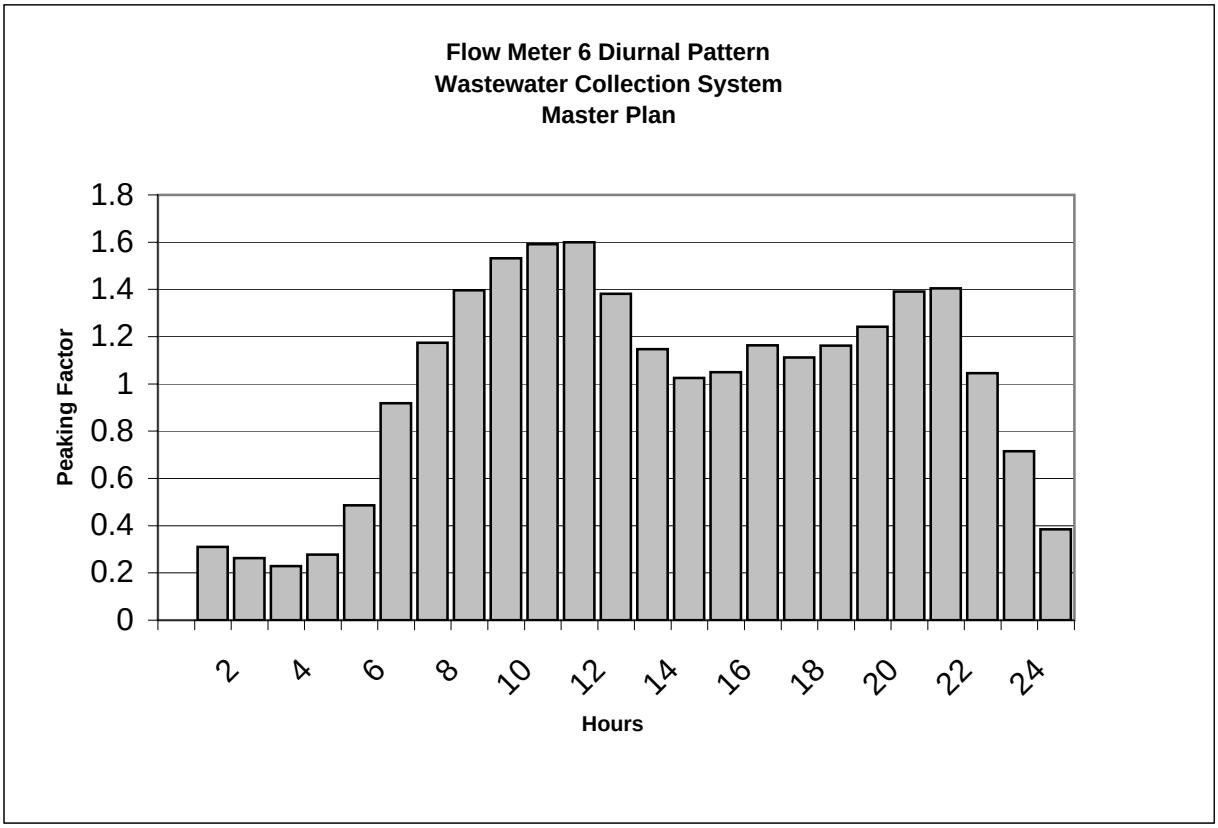
Diurnal Patterns





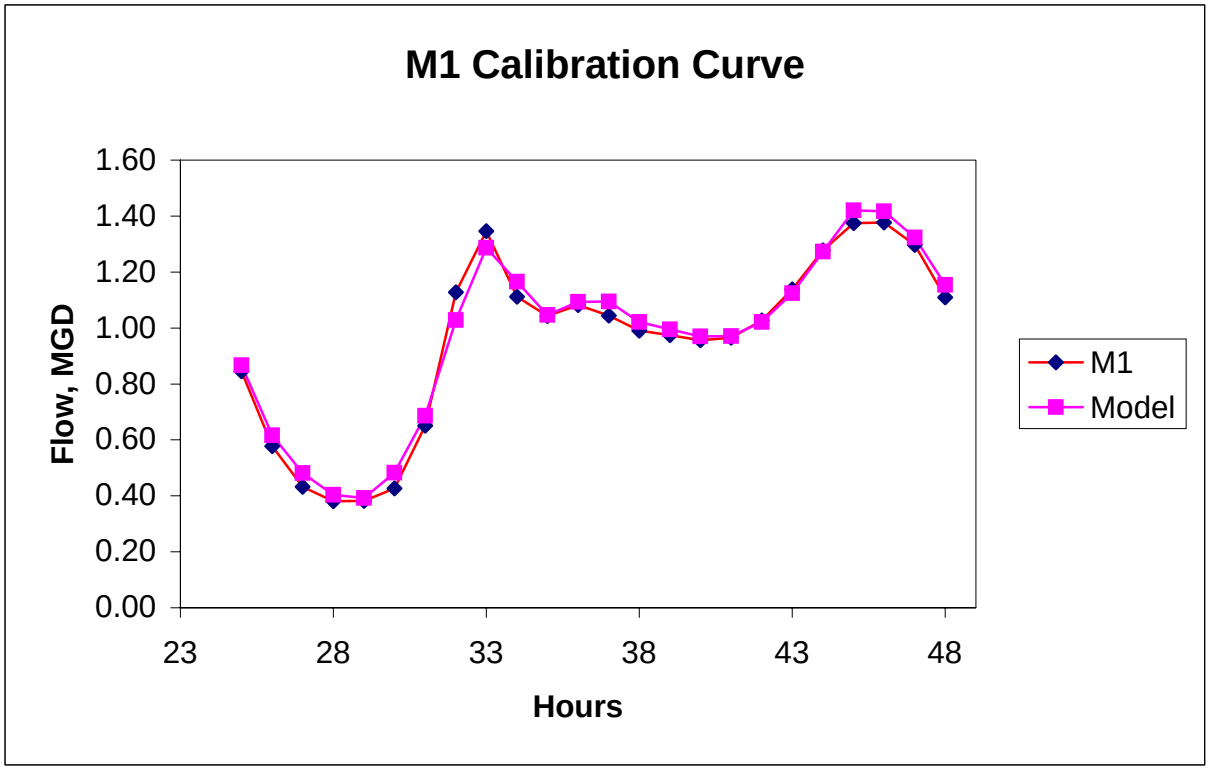
Diurnal Patterns

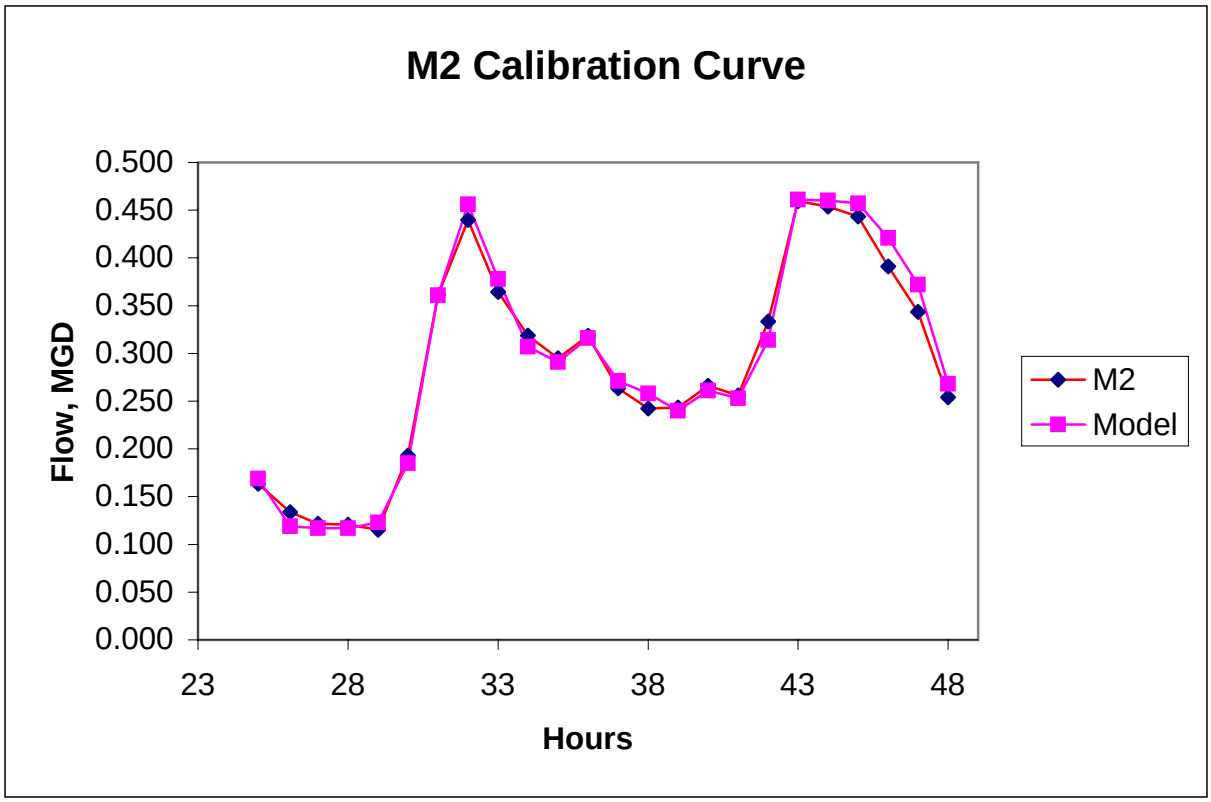




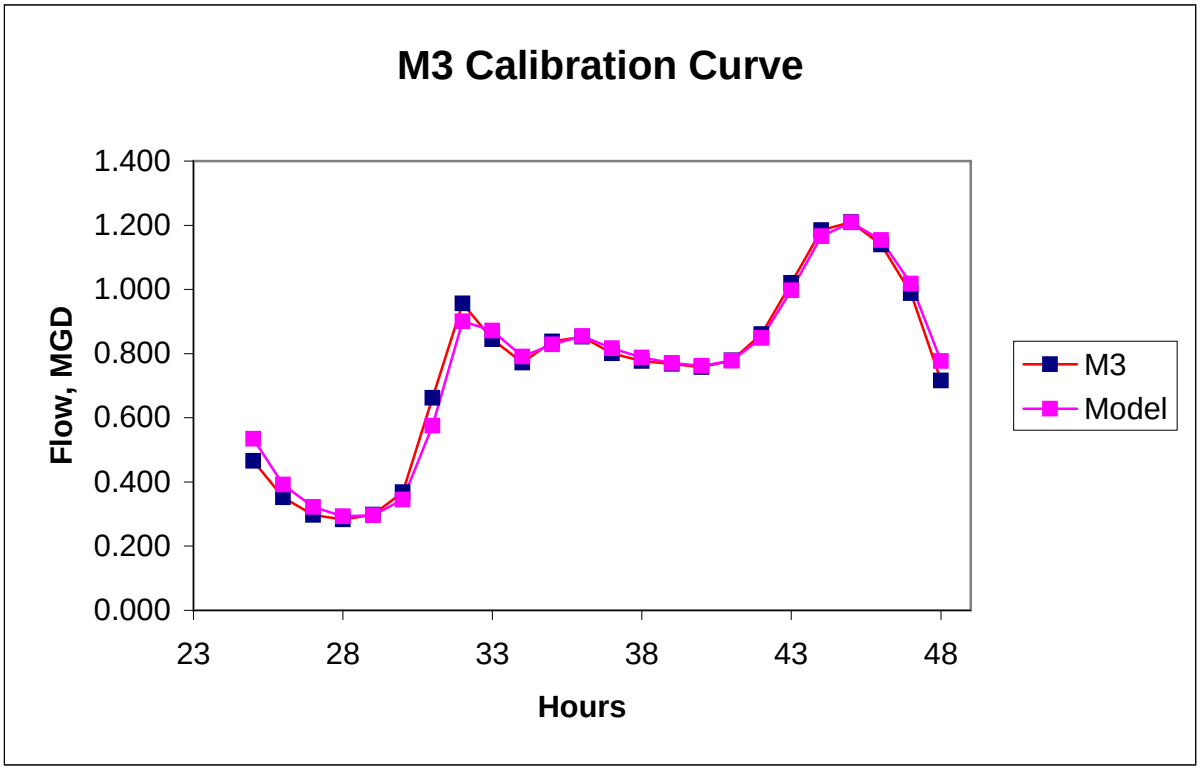
APPENDIX C - DWF CALIBRATION PLOTS

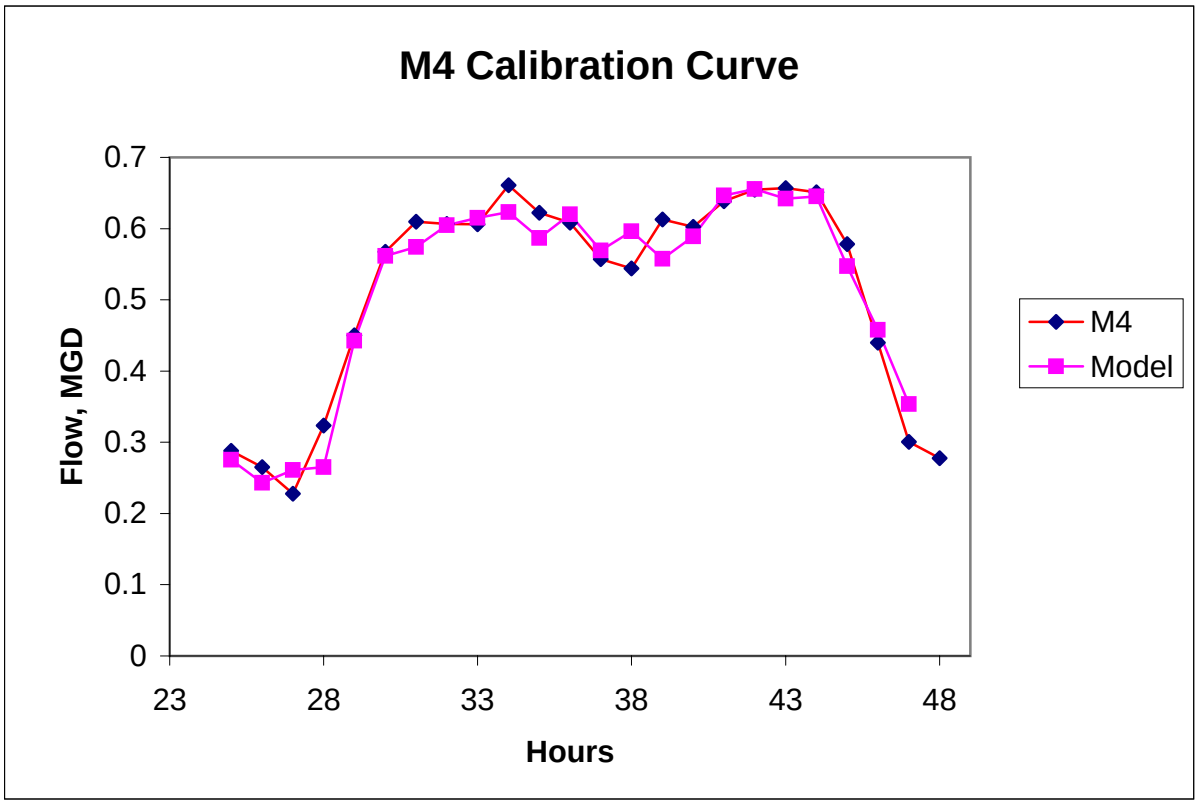
Dry Weather Flow Calibration Plots



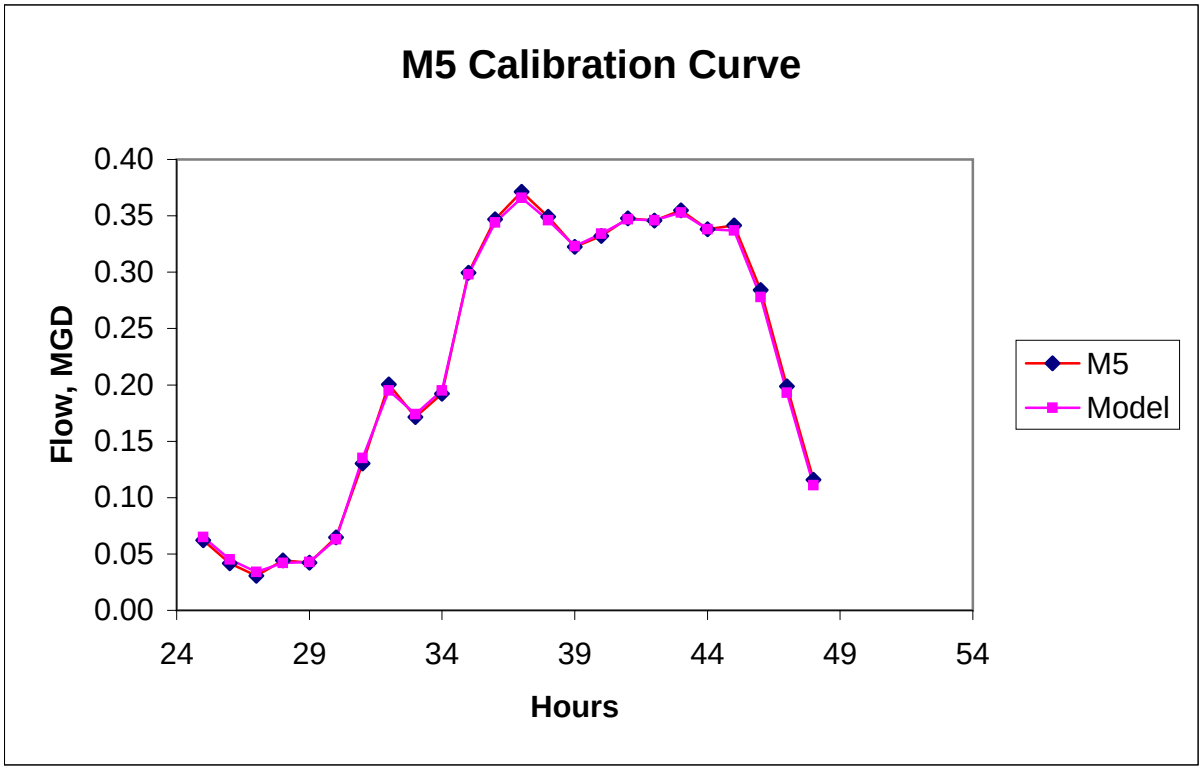


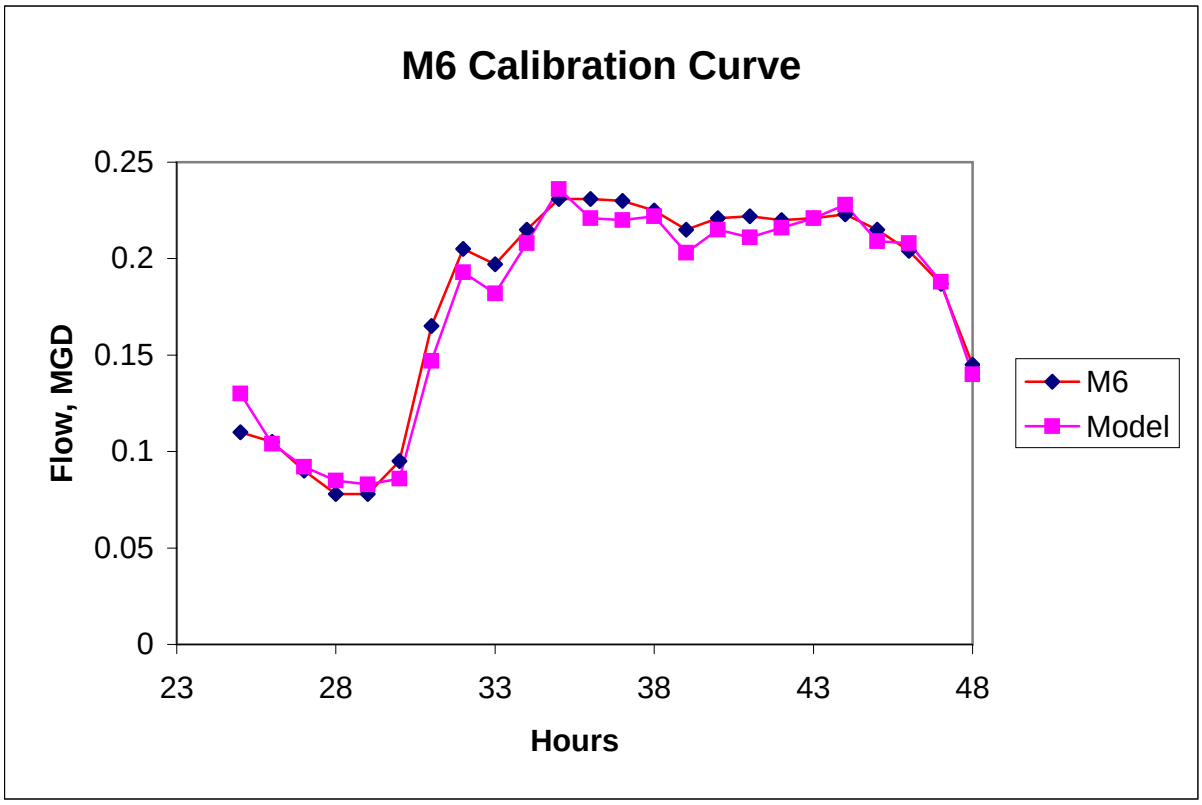
Dry Weather Flow Calibration Plots



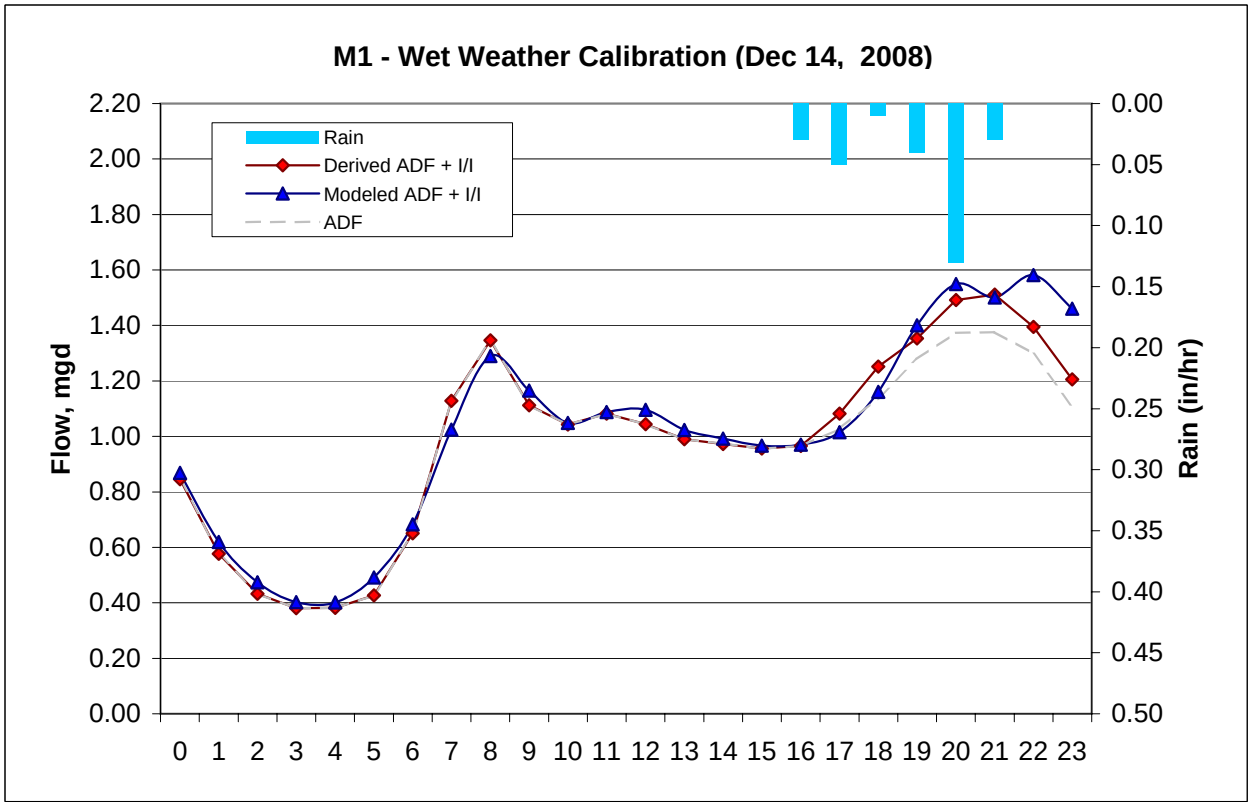


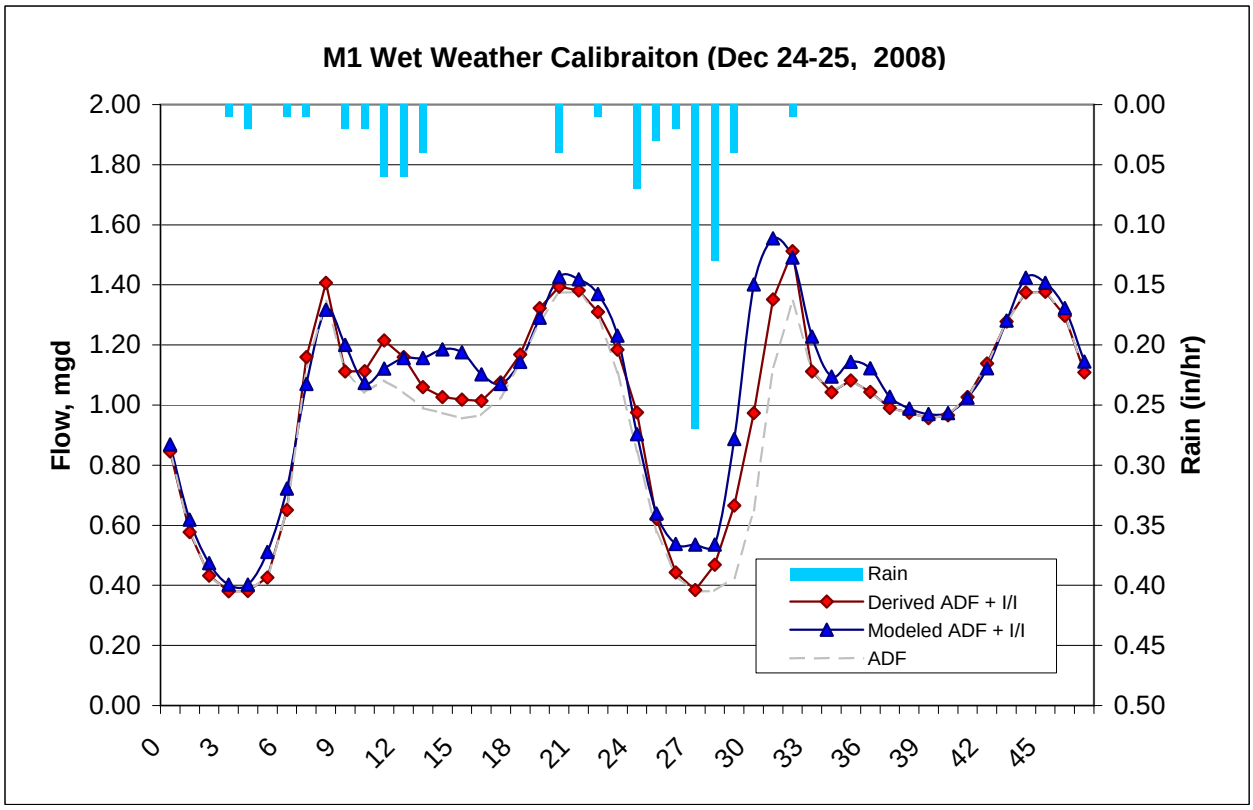
Dry Weather Flow Calibration Plots

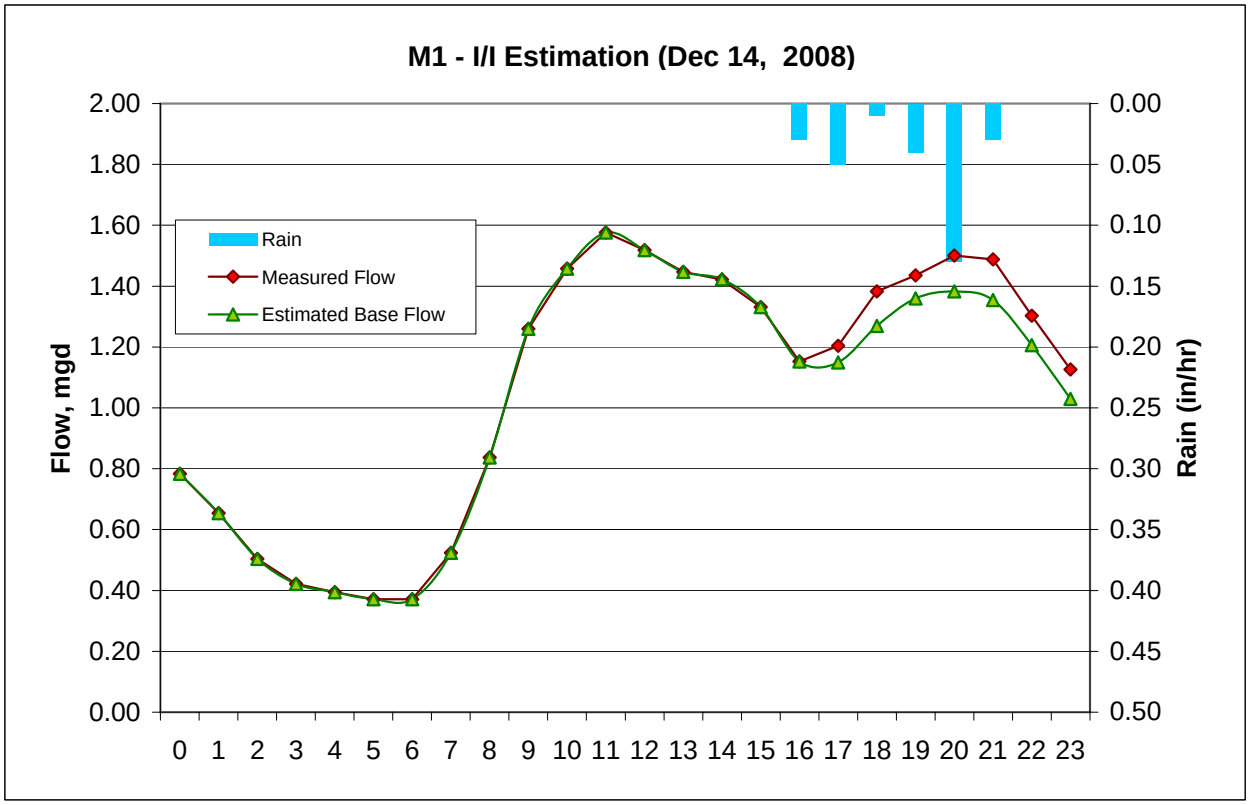


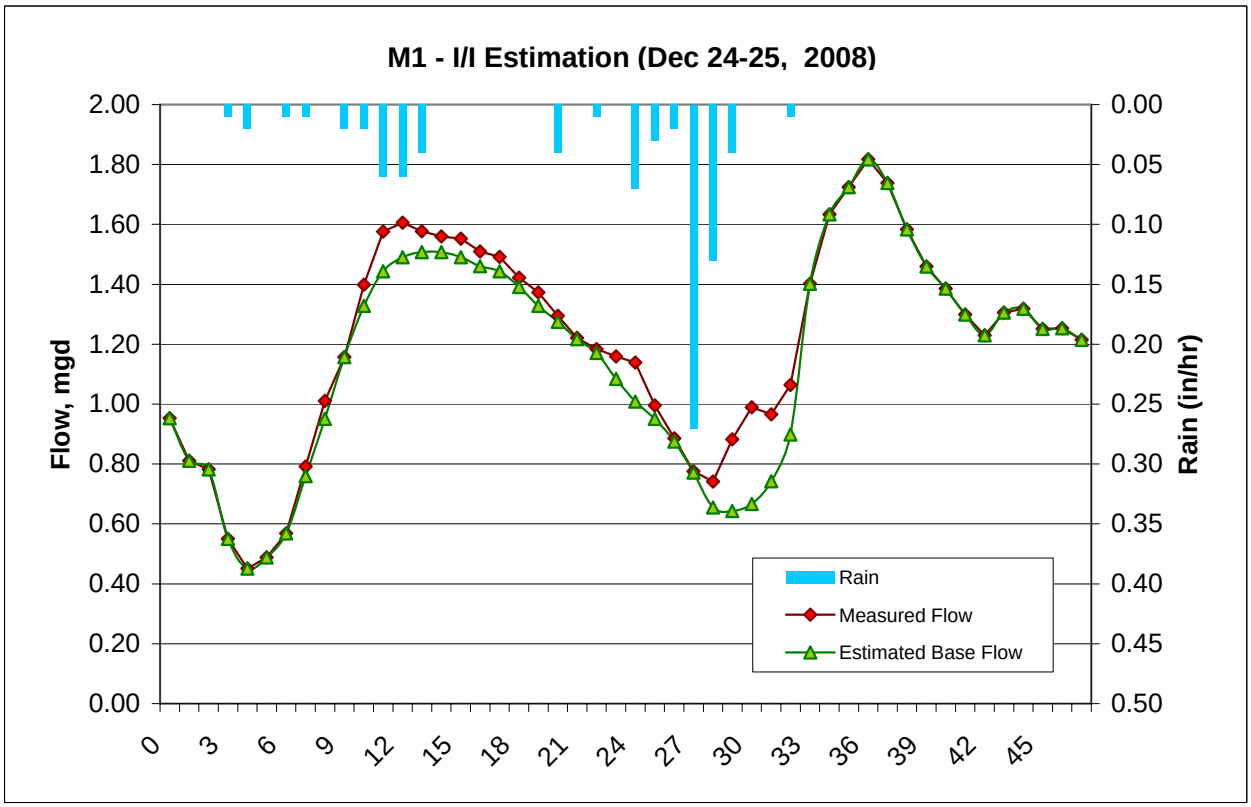


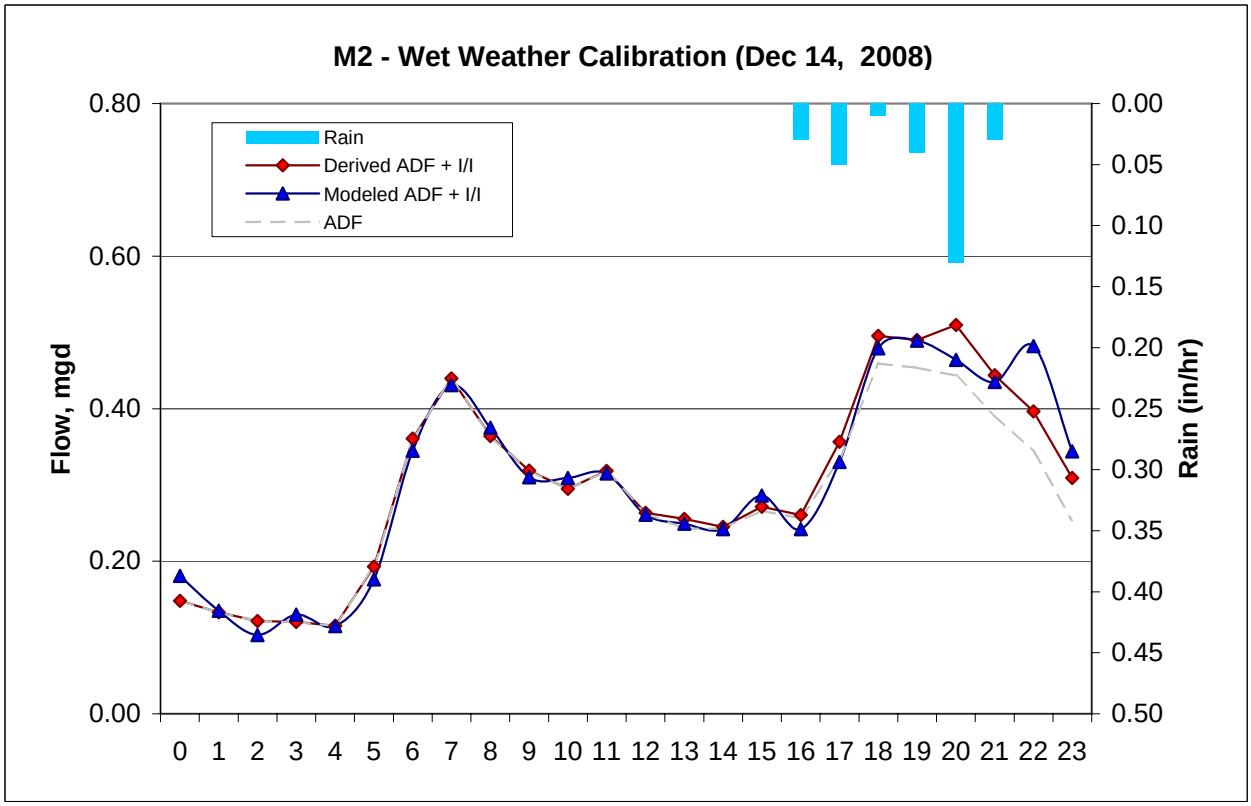
APPENDIX D - WWF CALIBRATION PLOTS

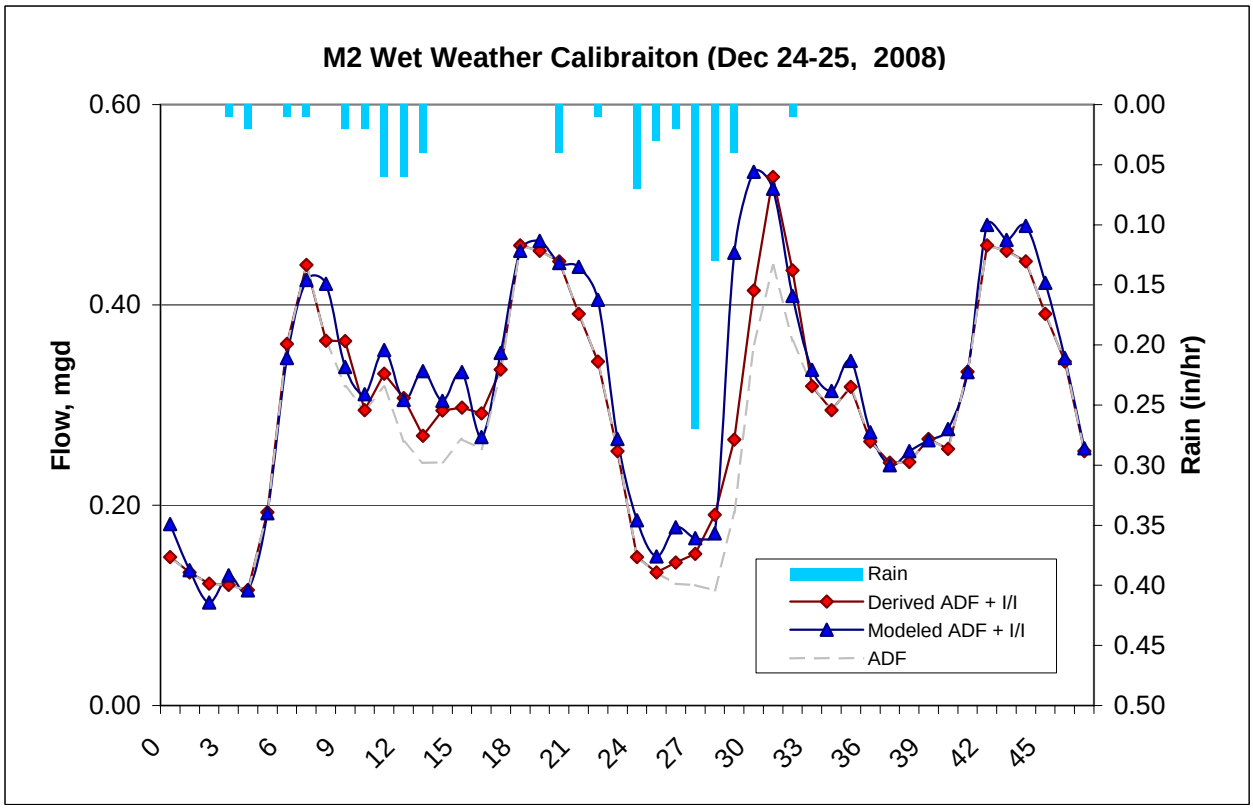

M1 - Wet Weather Calibration (Dec 14, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

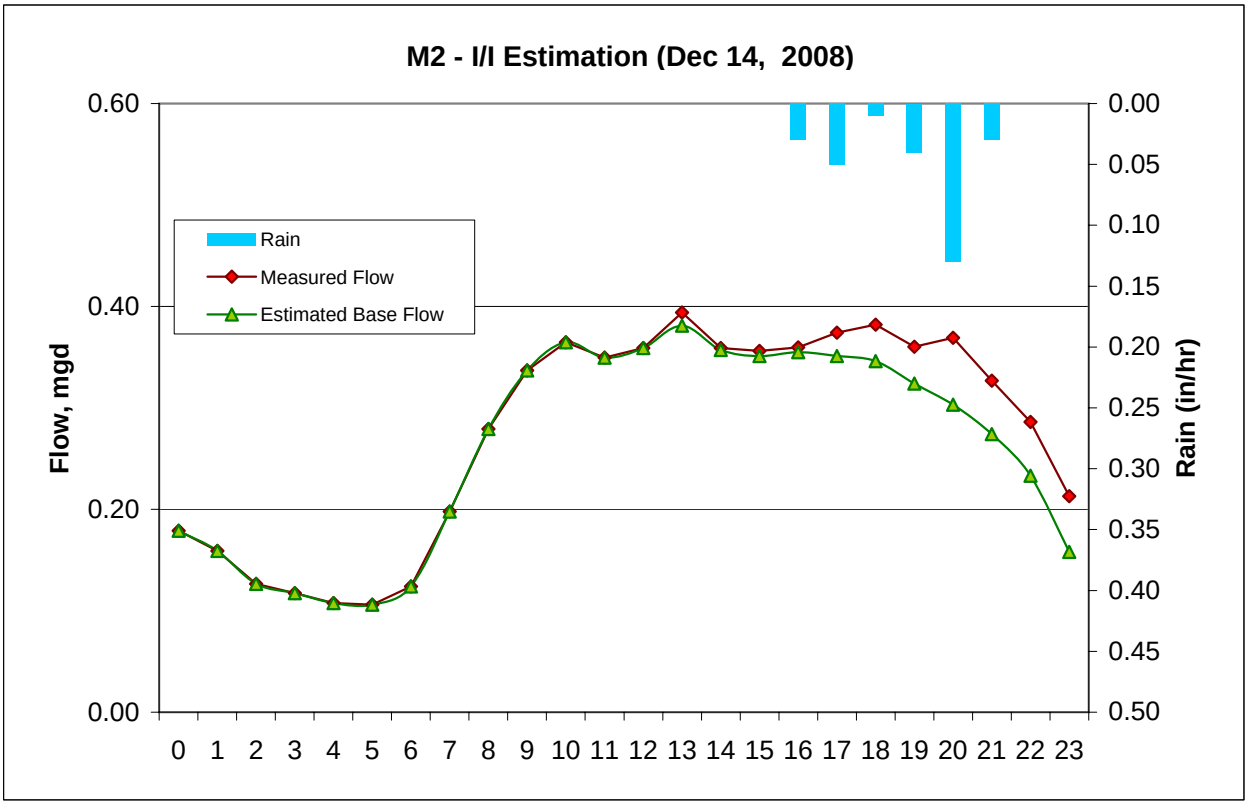

M1 Wet Weather Calibraiton (Dec 24-25, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

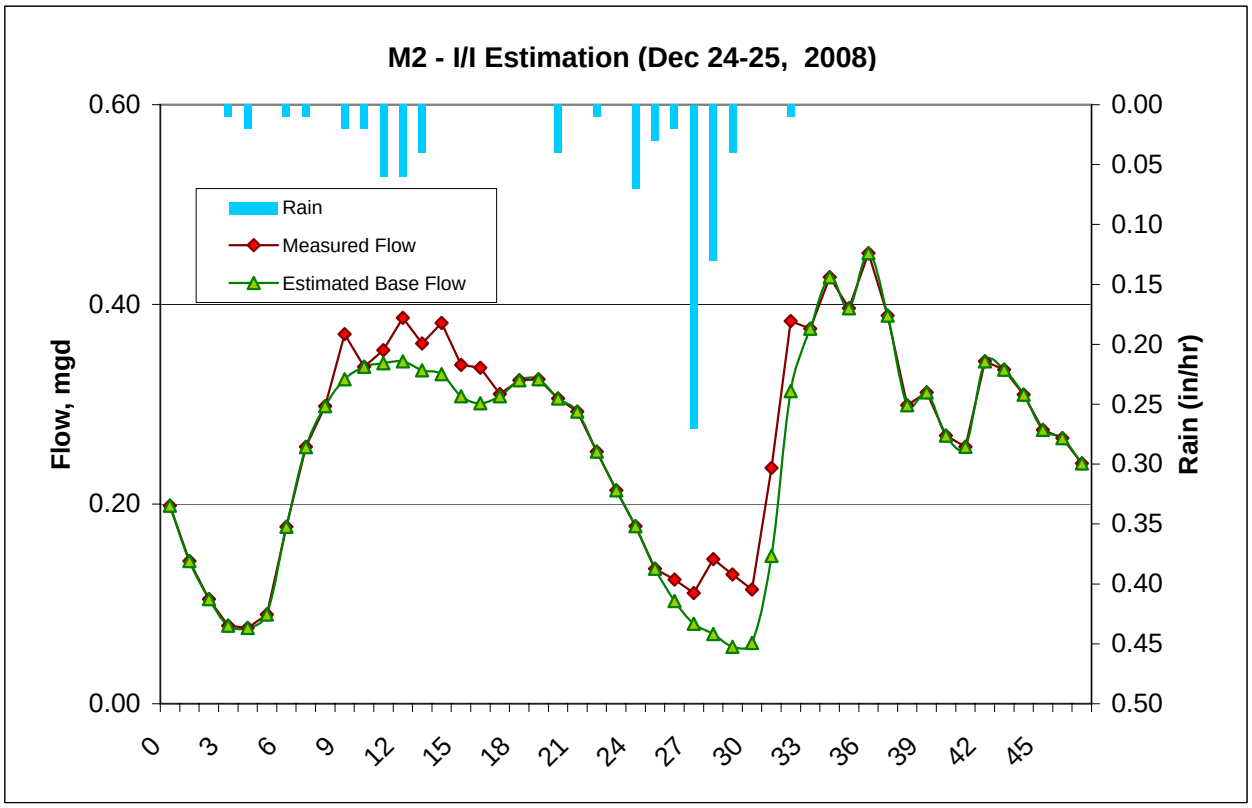

M1 - I/I Estimation (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

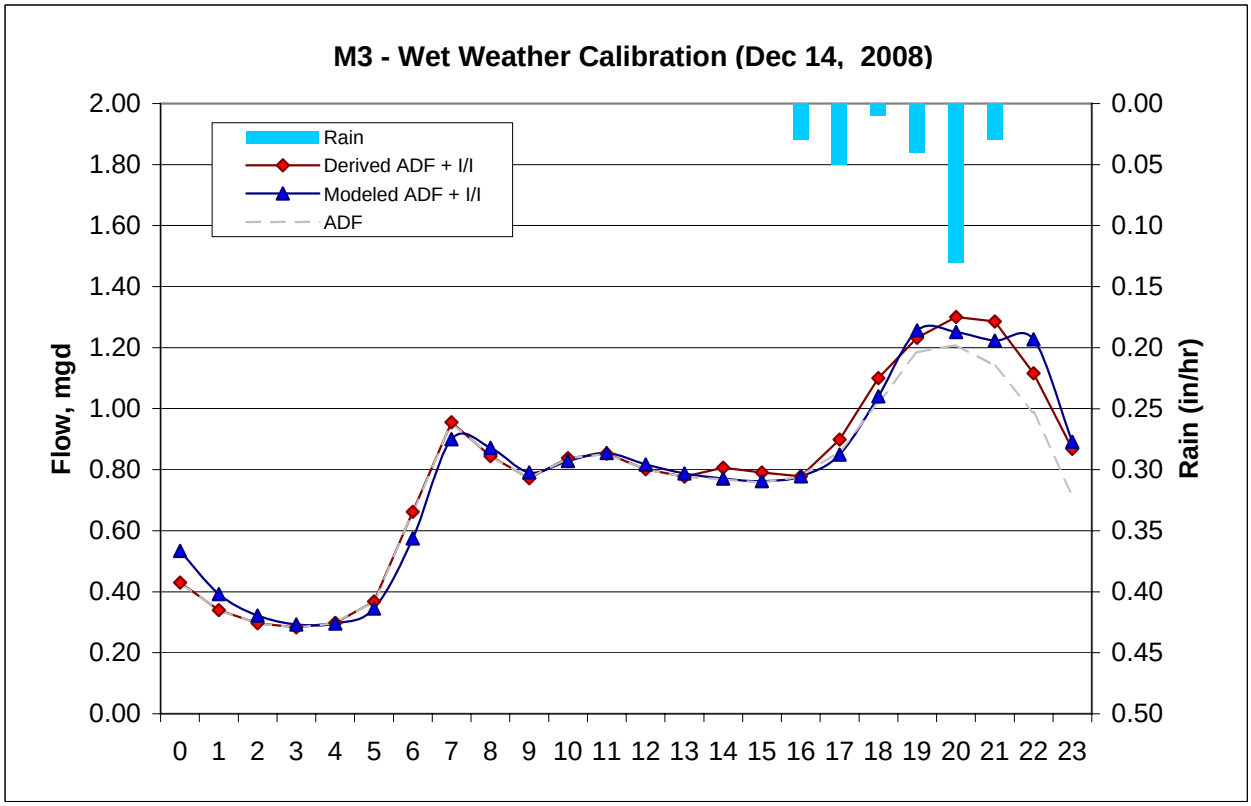

M1 - I/I Estimation (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

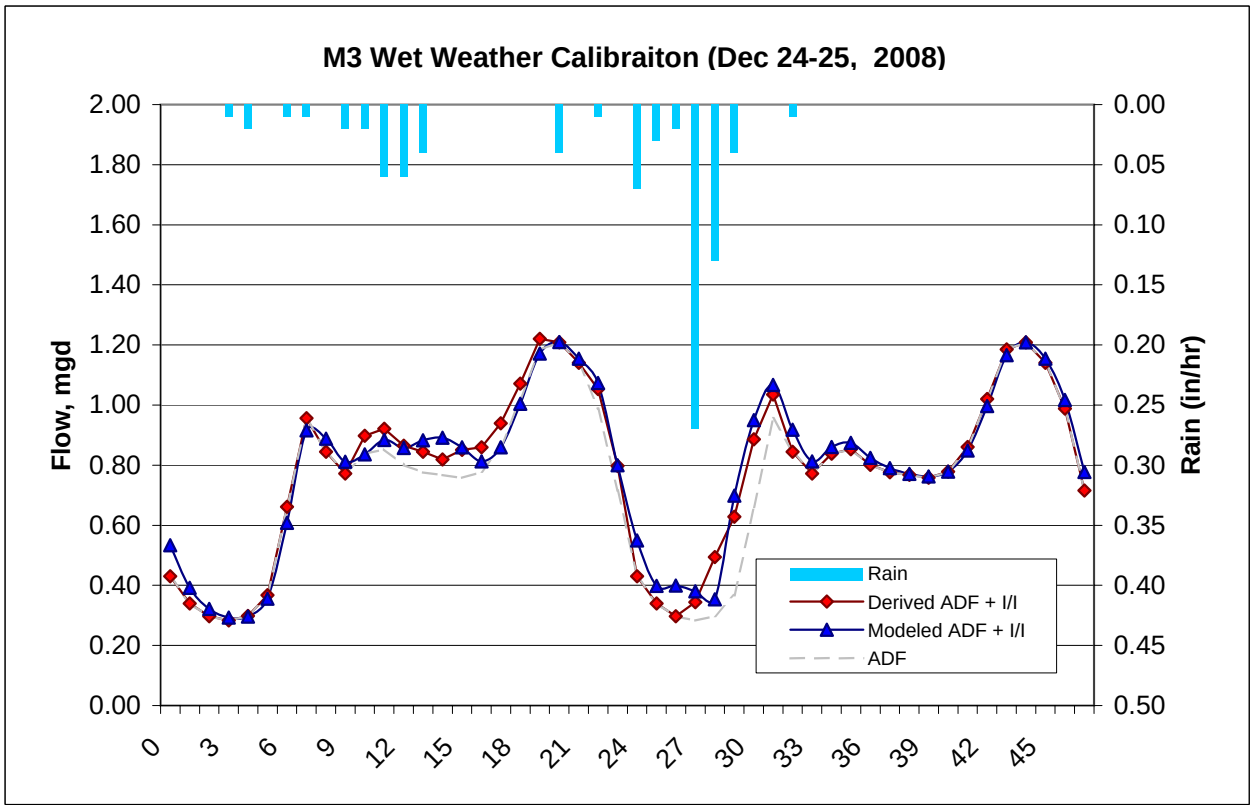

M2 - Wet Weather Calibration (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

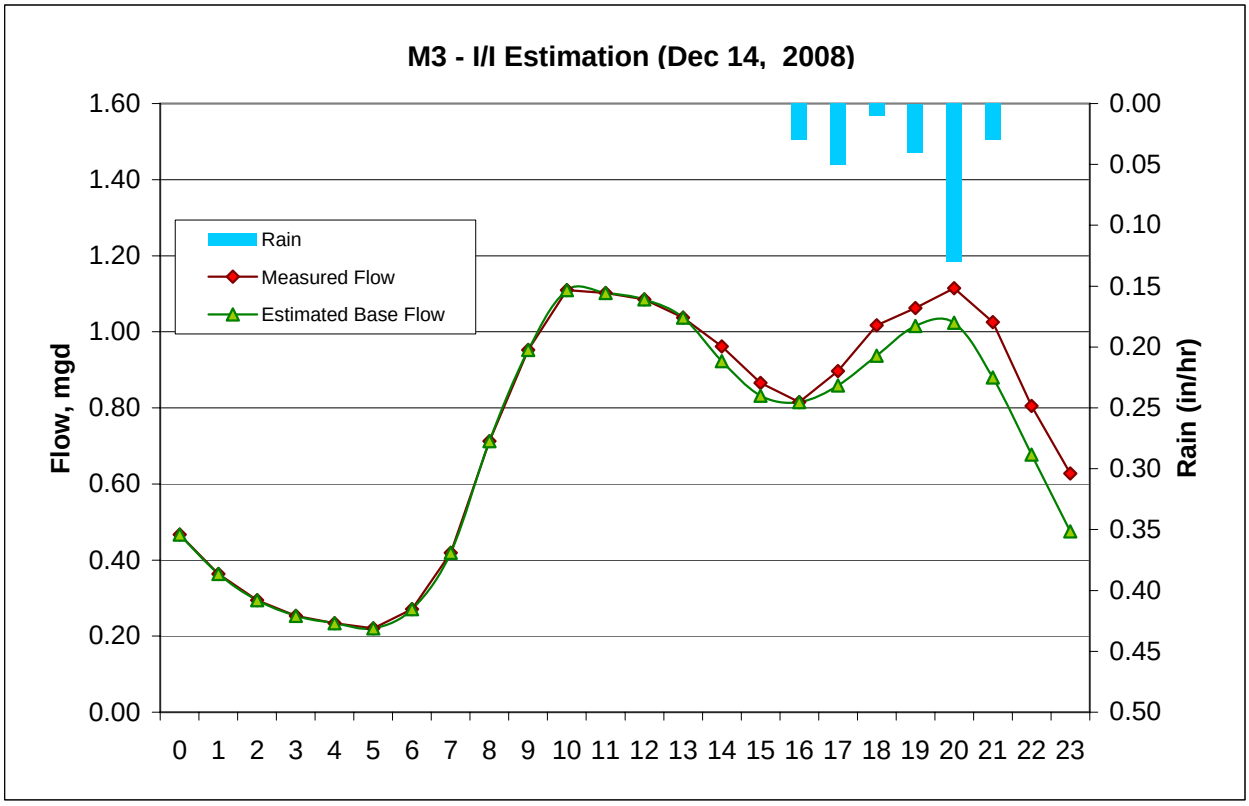

M2 Wet Weather Calibraiton (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

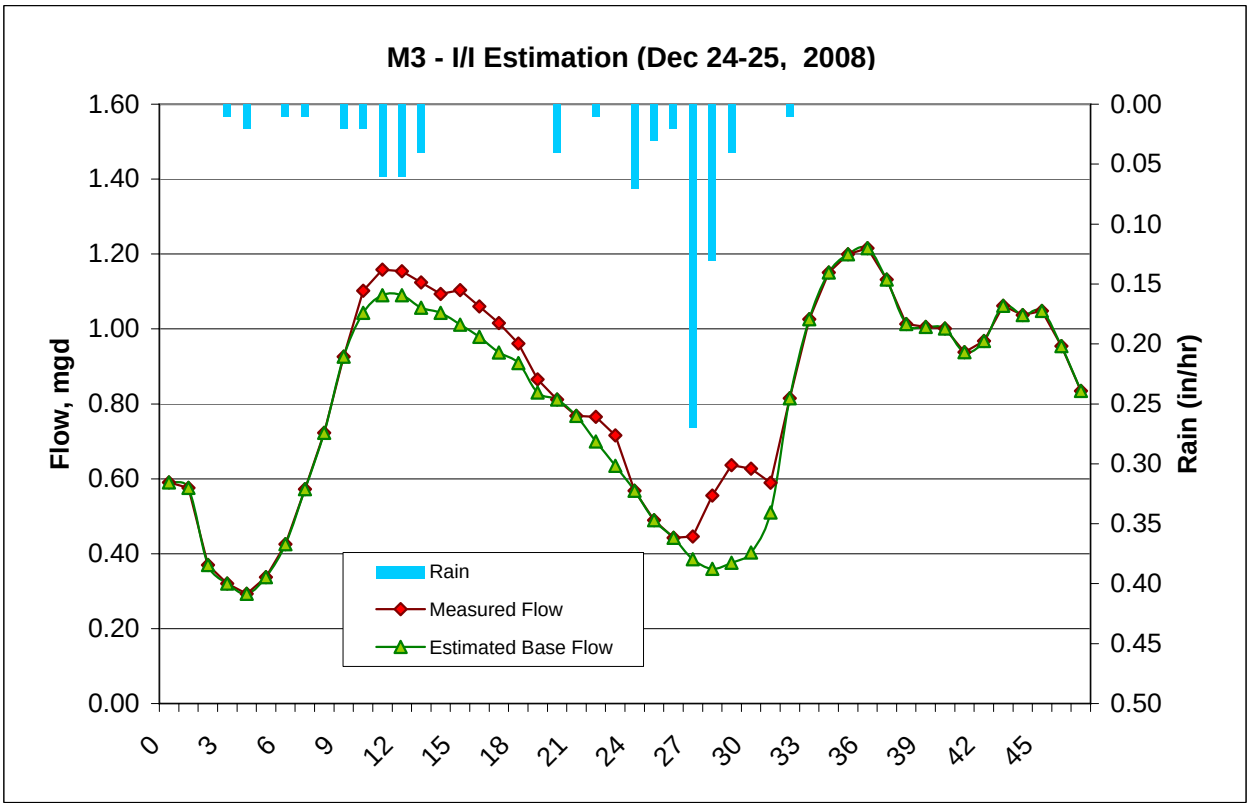

M2 - I/I Estimation (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

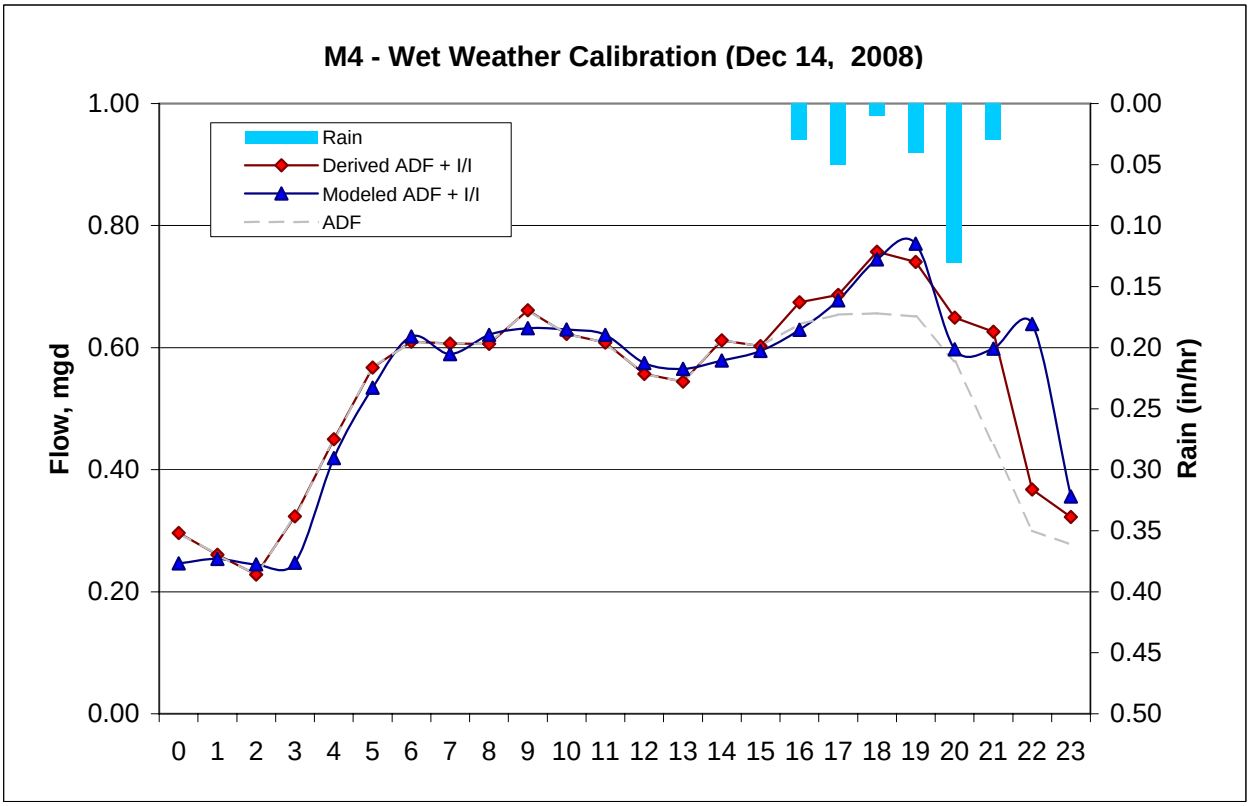

M2 - I/I Estimation (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

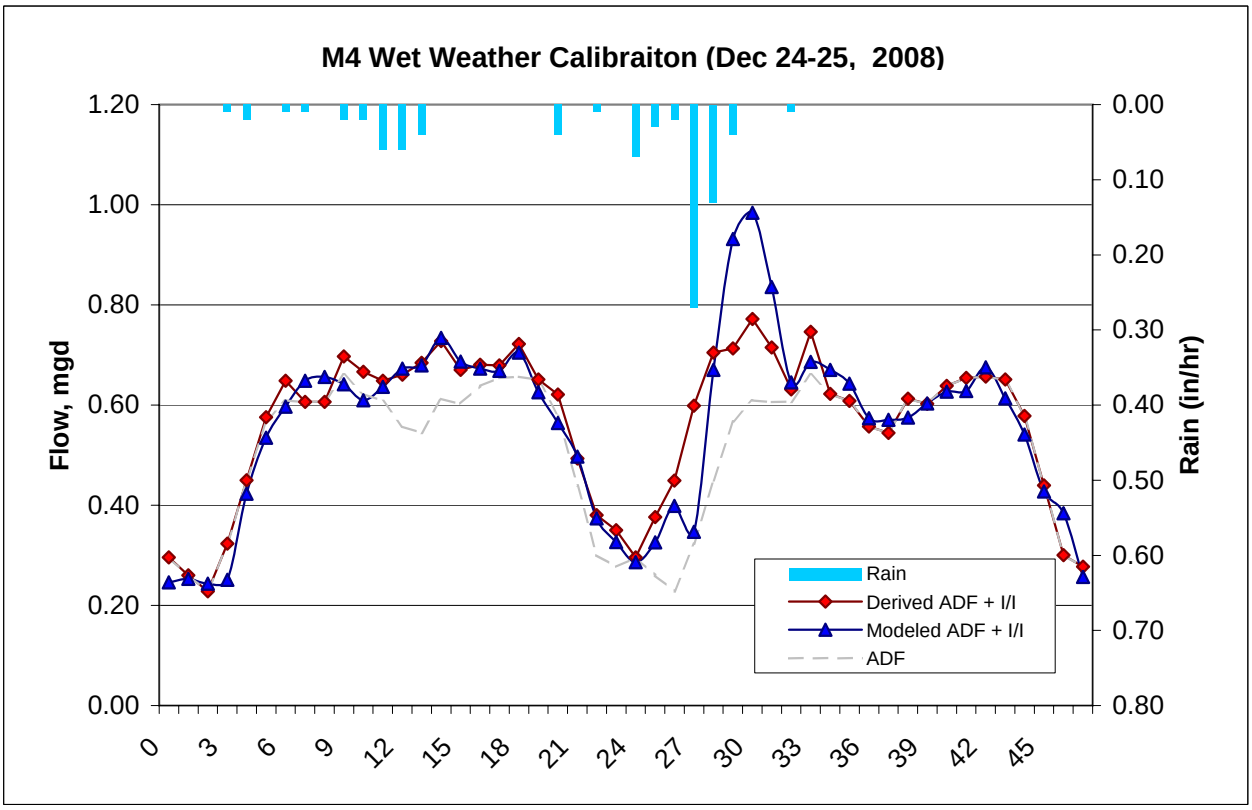

M3 - Wet Weather Calibration (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

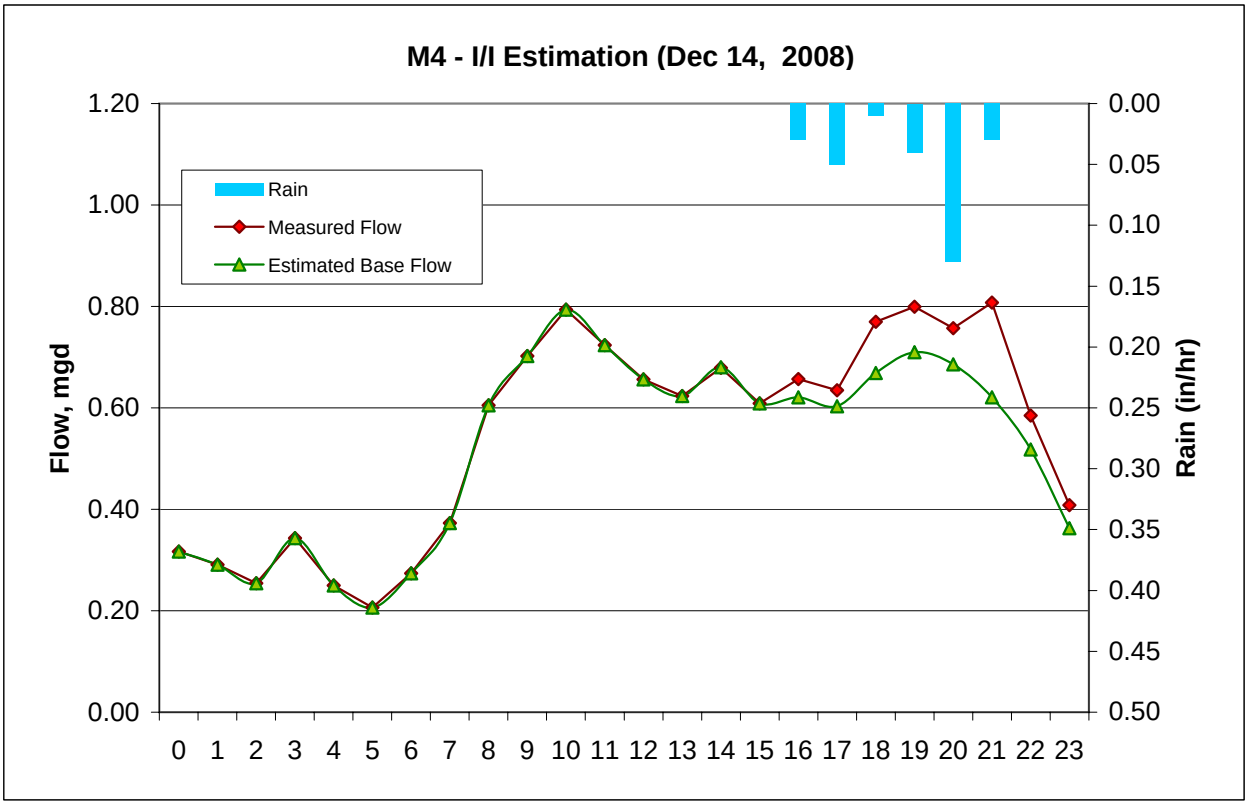

M3 Wet Weather Calibraiton (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

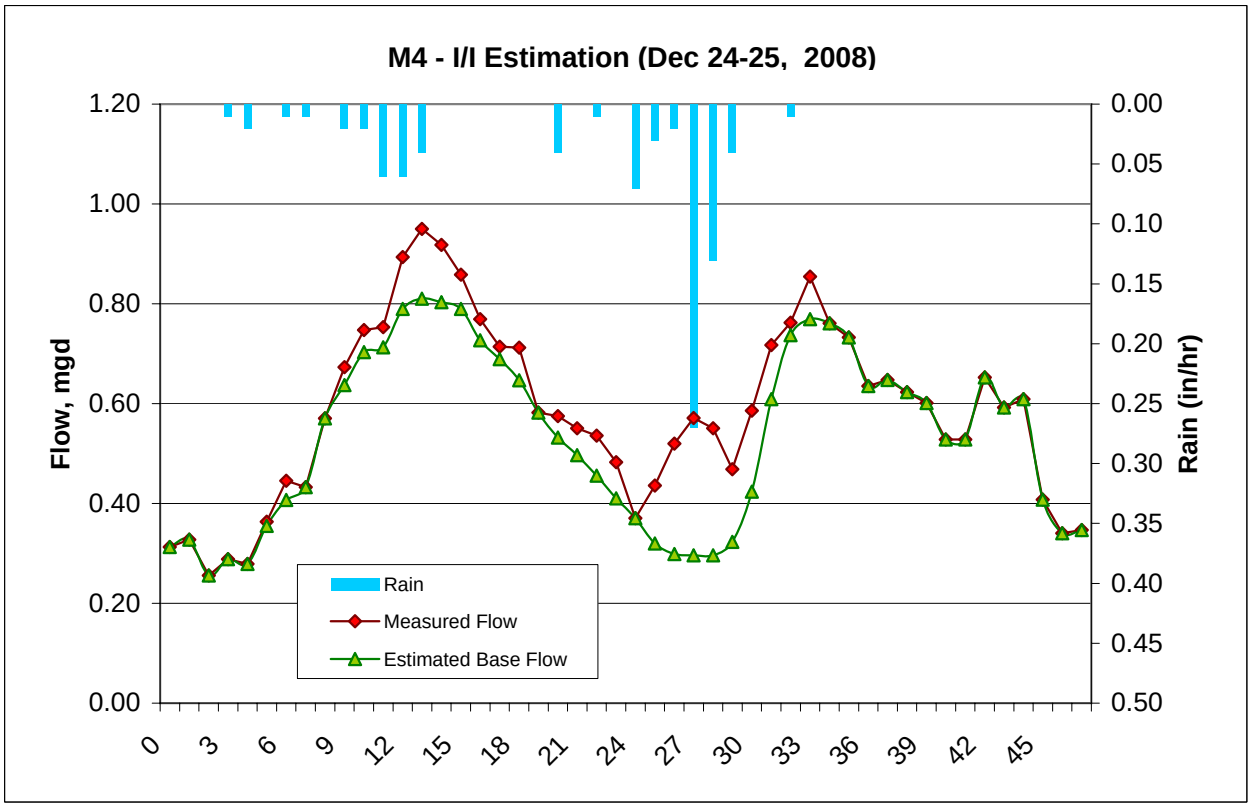

M3 - I/I Estimation (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

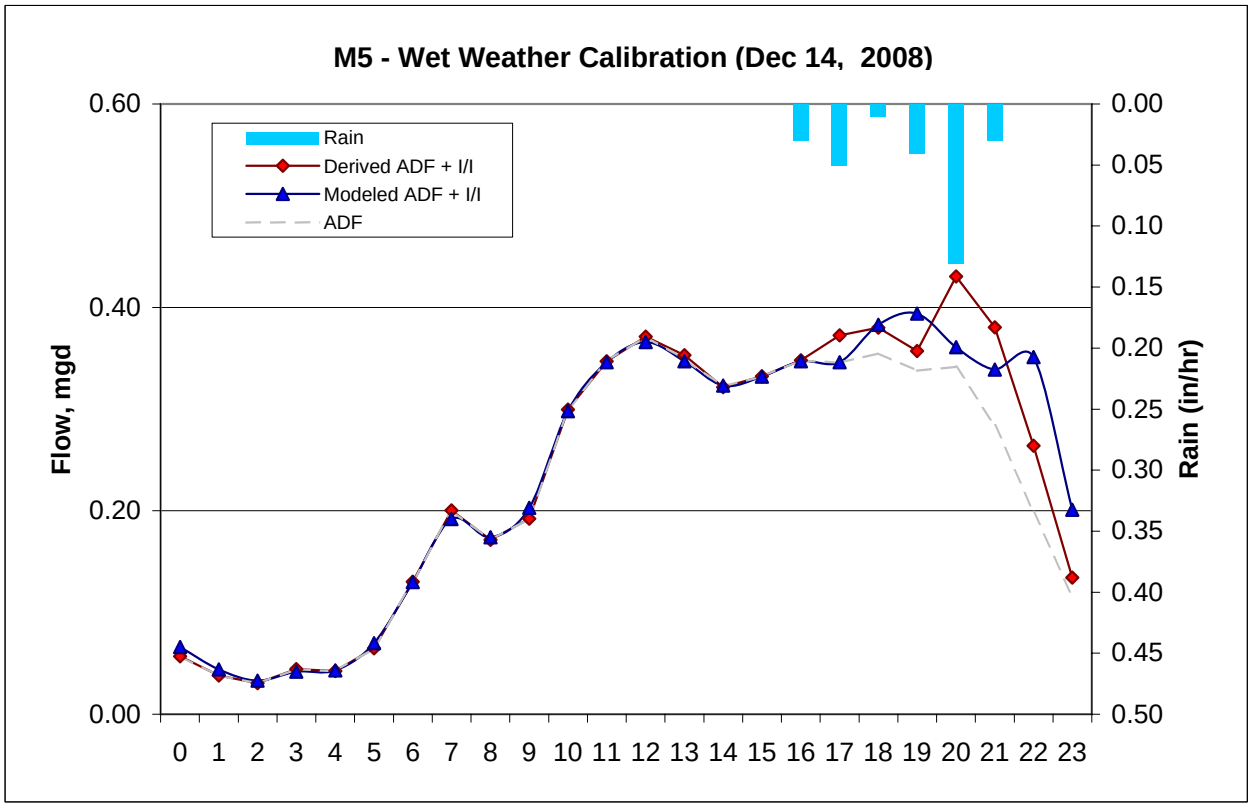

M3 - I/I Estimation (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

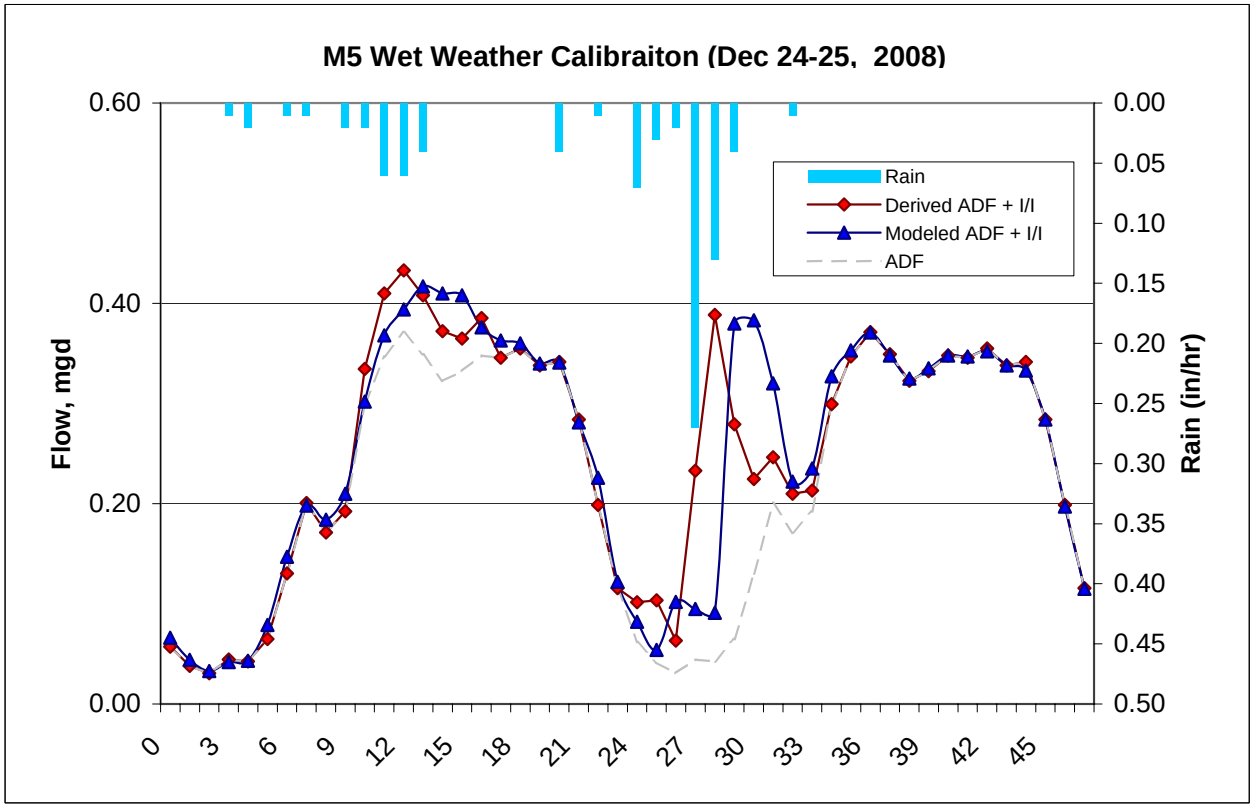

M4 - Wet Weather Calibration (Dec 14, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

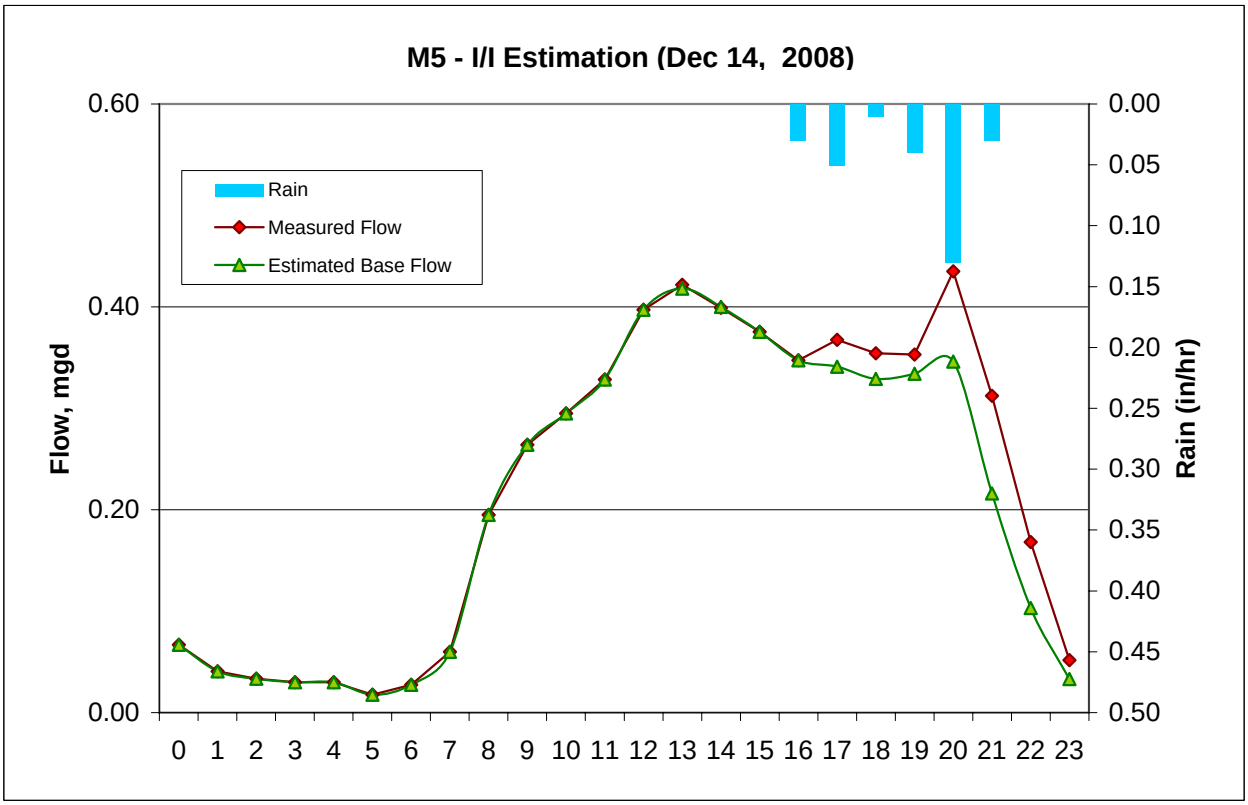

M4 Wet Weather Calibraiton (Dec 24-25, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

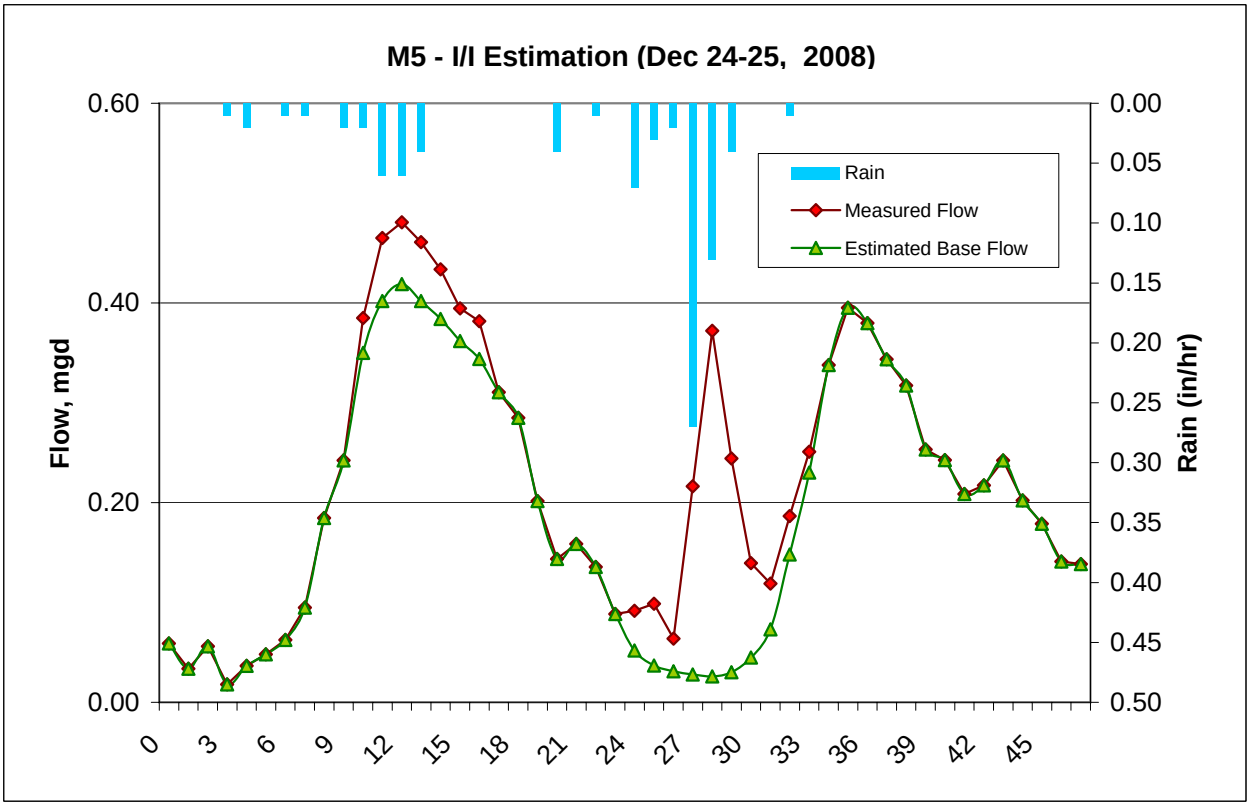

M4 - I/I Estimation (Dec 14, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

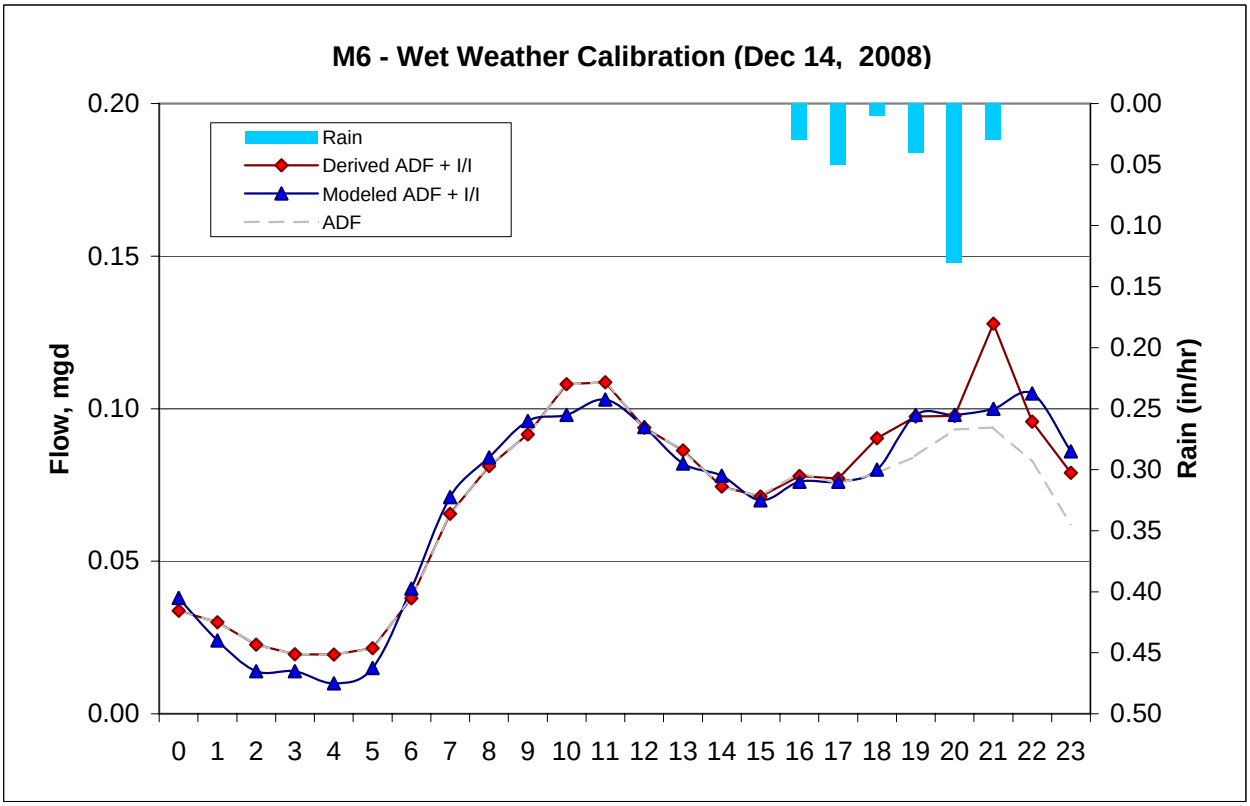

M4 - I/I Estimation (Dec 24-25, 2008)
Flow, mgd
Rain (in/hr)
Rain
Measured Flow
Estimated Base Flow
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

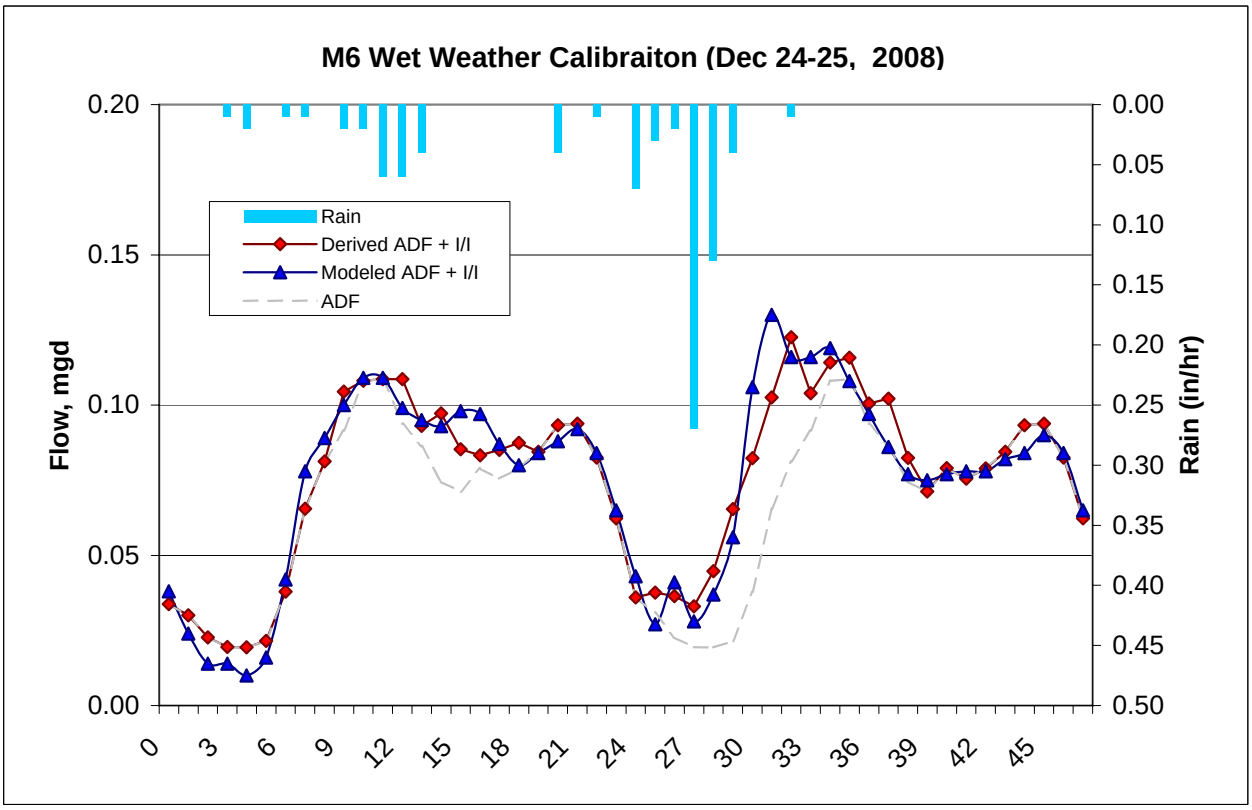

M5 - Wet Weather Calibration (Dec 14, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

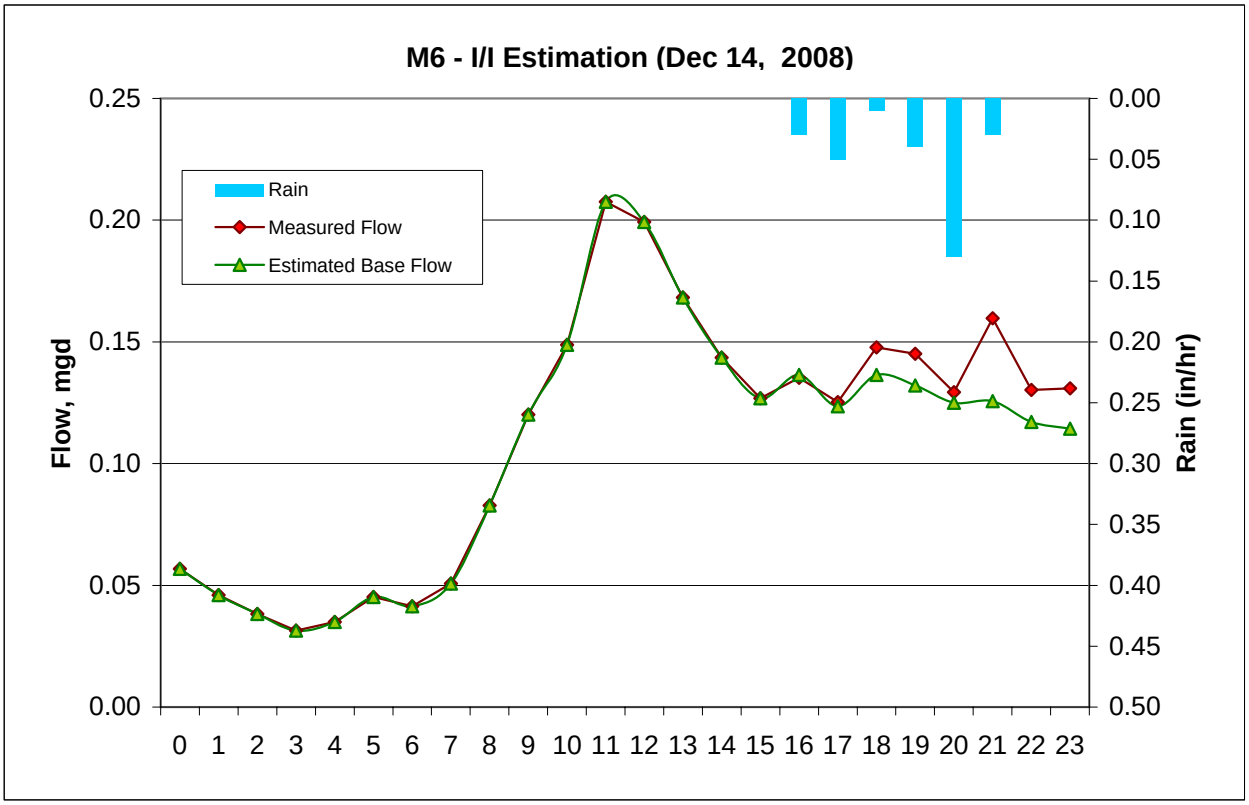

M5 Wet Weather Calibraiton (Dec 24-25, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

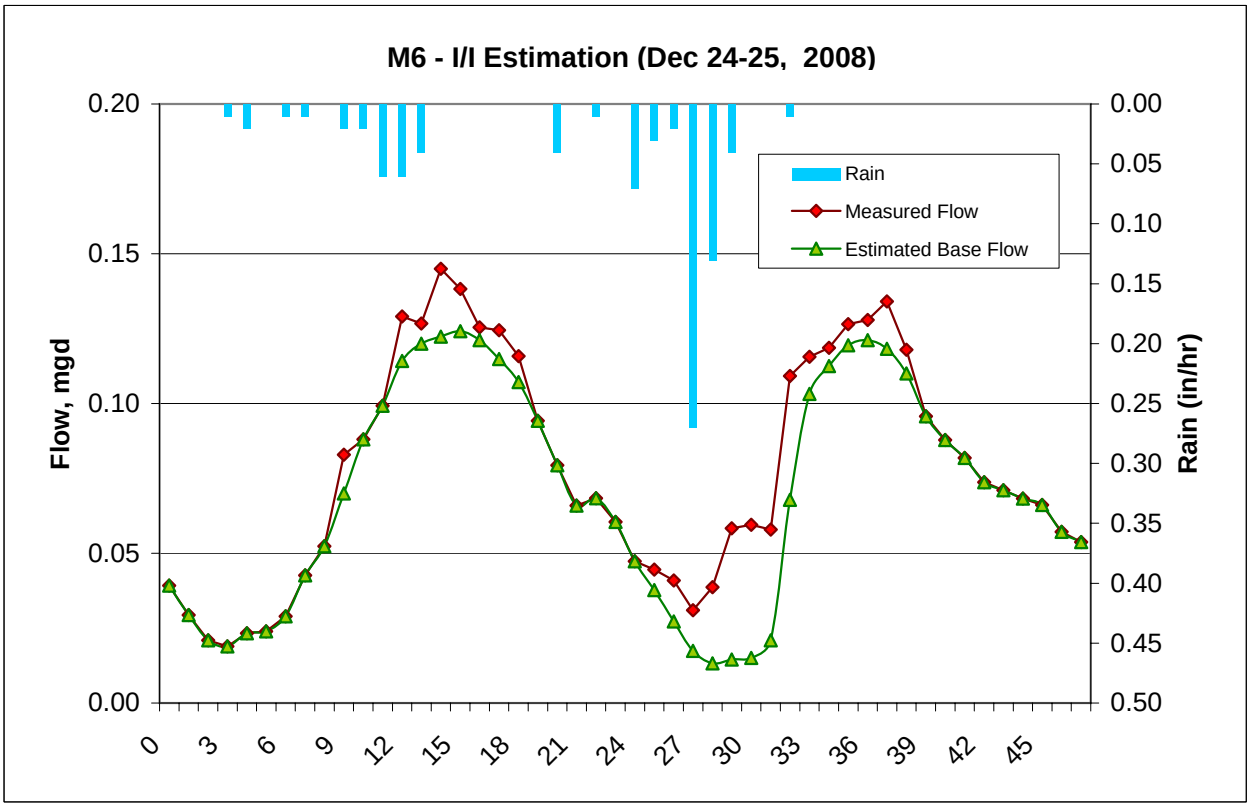

M5 - I/I Estimation (Dec 14, 2008)
Rain
Measured Flow
Estimated Base Flow
Flow, mgd
Rain (in/hr)
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23


M5 - I/I Estimation (Dec 24-25, 2008)
Rain
Measured Flow
Estimated Base Flow
Flow, mgd
Rain (in/hr)
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45


M6 - Wet Weather Calibration (Dec 14, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
0.20
0.15
0.10
0.05
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23


M6 Wet Weather Calibraiton (Dec 24-25, 2008)
Rain
Derived ADF + I/I
Modeled ADF + I/I
ADF
Flow, mgd
Rain (in/hr)
0.20
0.15
0.10
0.05
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45


M6 - I/I Estimation (Dec 14, 2008)
Rain
Measured Flow
Estimated Base Flow
Flow, mgd
Rain (in/hr)
0.25
0.20
0.15
0.10
0.05
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23


M6 - I/I Estimation (Dec 24-25, 2008)
Rain
Measured Flow
Estimated Base Flow
Flow, mgd
Rain (in/hr)
0.20
0.15
0.10
0.05
0.00
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45

APPENDIX E - PROJECT MEMORANDUM (LOCATION SUMMARIES OF BENEFITS/DRAWBACKS)

PROJECT MEMORANDUM

Project Name:	Sewer Collection System Master Plan	**Date:**	07/15/2009
Client:	City of Galt	**Project Number:**	8100A.00
Prepared By:	Tim Loper		
Reviewed By:	Jose Gutierrez, Scott Parker		
Subject:	Wastewater Collection Alternative Alignment Analysis		
Distribution:	Bill Forrest, Senior Civil Engineer		

This memorandum summarizes the analysis of the sewer alignment alternatives for serving future development in the southeast portion of the City, and for selecting the future location of the Live Oak pump station.

At the workshop conducted on April 27, 2009, Carollo Engineers, P.C. (Carollo) presented preliminary sewer alignments to serve users in the east and southeast areas of the planning area. Based on these preliminary alternatives, the City requested that Carollo revise the sewer alignments and develop a decision matrix to assist the selection process.

Alternatives to Serve Future Users in the Southeast Area

Alternative 1 relies on gravity sewers to convey flow from future users in the southeast part of the study area across Highway 99 to the Live Oak pump station. Alternative 1 could serve the entire southeast area with a gravity trunk sewer (Figure 1). Alternative 2 follows a similar alignment to serve the southeast area, but relies on a pump station located near the intersection of Carillion Boulevard and Simmerhorn Road (Figure 2) to convey flow across Highway 99. The pump station would have a total capacity of approximately 4.6 mgd and convey wastewater in a 12-inch forcemain north and then west to a new 27-inch gravity trunk sewer at Live Oak Avenue near Highway 99. The new 27-inch gravity line in Live Oak Avenue is proposed in both alternatives.

Carollo developed a decision matrix to help determine the relative value of each alternative. Evaluation criteria were developed and assigned an importance value of High, Medium or Low, where a High importance had a value of 3, medium had a value of 2, and low a value of 1. The alternatives were then assigned a score between 1 and 3 for each criterion based on how well the alternatives performed against the criteria. Below is a discussion of the criteria and their importance. The performance was multiplied by the importance and the values were summed to determine the total score for each alternative.

The criteria address several areas including Sacramento Area Sewer District (SASD) minimum slope standards, required facilities, operation and maintenance, and reliability. A complete list of the criteria is as follows:

1. **SASD Minimum Slope Standards** - SASD criteria specify minimum slopes for sewer pipelines that achieve minimum velocities greater than 2 feet per second (fps). For large pipelines flatter slopes can be used and still meet the minimum velocity of 2 fps. The gravity alternative uses minimum slopes that are less than SASD standards, but still provide velocities greater than 2 fps. An alternative scored lower if pipeline slopes were less than SASD minimum slopes.

2. **South East Lift Station Required** - This criterion specifies if the alternative requires an additional lift station to serve users in southeast area. An alternative scored higher if a lift station was not required.

3. **Operations & Maintenance** - Impact to operations and maintenance staff. An alternative scored higher if anticipated O&M costs were lower.

4. **Highway 99 & Railroad Crossings** - Relative differences between crossings for each alternative. An alternative scored higher if it resulted in simplified crossings at highways and railroads.

5. **Future User Connection Options** - This criteria evaluates the relative difference between the alternative's flexibility for connecting future users along the pipeline alignment. An alternative scored higher if it provided more opportunities for connecting future users.

6. **Connect Existing and Future Users East of Highway 99** - Ability of existing and future users south of Simmerhorn Road and east of Highway 99 to connect.

7. **Reliability** - Reliability of each alternative. Alternative scored higher if it was viewed as being more reliable.

8. **Pipeline Costs** - Relative comparison of pipeline costs. Alternative scored higher if it was more economical.

The benefits of Alternative 1 include reduced O&M costs since City staff will not be servicing a large pump station, the reliability of a gravity sewer versus a long forcemain, and the flexibility for future users to connect to a gravity sewer. The benefits of Alternative 2 are the ease of construction of the forcemain versus a gravity sewer, however, those benefits are out weighed by the added costs of the pump station.

As shown in Table 1, the gravity alternative has a total weighted score of 40, while the forcemain alternative has a total weighted score of 29. Therefore, based on a qualitative analysis of the two alternatives, the gravity alternative is preferred.

Cost Comparison of Alternatives 1 and 2 for Serving Southeast Area

Table 2 provides an estimate of capital project costs based on the contingency assumptions and unit costs that will be presented in the Master Plan. The costs include a 25 percent construction contingency as well as 10 percent contingencies each for engineering, construction management, and project administration. The costs presented are for the reaches of pipeline that run from the intersection of Simmerhorn Road and Carillion Boulevard to Highway 99 and Live Oak Avenue. The alignments downstream of Highway 99 are the same for both alternatives.

As shown in Table 2, the total capital cost for the forcemain alternative is $5.96 million, where the gravity alternative totals approximately $3.96 million. Therefore the gravity alternative provides a cost savings to the City. Based on the decision matrix and cost comparison, Alternative 1, the gravity trunk is the recommended alternative.

Table 1 Southeast Sewer Service Alternatives
Sewer System Master Plan
City of Galt

Criteria	Importance (High = 3, Medium = 2, Low = 1)	Alternative - 1 Gravity Trunk	Alternative - 2 Forcemain Alternative	Weighted Score: Gravity Alternative	Weighted Score: Forcemain Alternative
1. SASD Min Slope Standards	2	1	3	2	6
2. South East Lift Station Required	3	3	1	9	3
3. Operations & Maintenance	3	3	1	9	3
4. Highway 99 & Railroad Crossings	1	2	2	2	2
5. Future User Connection Options	1	3	1	3	1
6. Connect Existing Users East of Highway 99	1	3	2	3	2
7. Reliability	3	3	1	9	3
8. Pipeline Costs	3	1	3	3	9
			Total	**40**	**29**

Description of Criteria

1. Sacramento Area Sewer District (SASD) criteria specify minimum slopes for sewer pipelines that achieve minimum velocities greater than 2 feet per second (fps). For large pipelines flatter slopes can be used and still meet the minimum velocity of 2 fps. The gravity alternative uses min slopes that are less than SASD standards, but still provide minimum velocities of 2 fps. An alternative scored lower if pipeline slopes were less than SASD minimum slopes.

2. This criterion specifies if the alternative requires an additional lift station to serve users in southeast area. An alternative scored higher if a lift station was not required.

3. Impact to operations and maintenance staff. An alternative scored higher if anticipated O&M costs were lower.

4. Relative differences between crossings for each alternative. An alternative scored higher if it resulted in simplified crossings at highways and railroads.

5.This criteria evaluates the relative difference between the alternatives flexibility for connecting future users along the pipeline alignment. An alternative scored higher if it provided more opportunities for connecting future users.

6. Ability of existing and future users south of Simmerhorn Road and east of Highway 99 to connect.

7. Reliability of each alternative. Alternative scored higher if it was viewed as being more reliable.

8. Relative comparison of pipeline costs. Alternative scored higher if it was more economical.

Table 2	Alternative Cost Comparison Sewer System Master Plan City of Galt		
	Construction Cost[1] (Million $)	**Engineering, CM, and Project Administration Cost (Million $)**	**Capital Improvement Project Cost (Million $)**
Alternative 1 - 27-inch Gravity Trunk[2]			
27-inch Gravity Trunk	$3.05	$0.91	$3.96
Alternative 2 - Pump Station and 12-inch Forcemain			
12-inch Forcemain	$1.04	$0.31	$1.35
5.7 mgd Pump Station	$3.54	$1.06	$4.60
Total Cost	**$4.58**	**$1.37**	**$5.96**

Note:
1. ENR CCI 20 City average used for estimating (March 2009) = 8,534
2. These cost only account for the reach of pipeline from the intersection of Simmerhorn Road and Carillion Boulevard to Highway 99 and Live Oak Avenue.
3. Total Capacity. Design flow of 4.3 mgd. Assumes three 1.4 mgd pumps.

Live Oak Lift Station Location Alternatives

Carollo also evaluated the two alternative locations to replace the Live Oak pump station. Alternative 1 (WWTP Influent Pump Station) would place the pump station at the WWTP and flow would be conveyed in a new 42-inch diameter gravity trunk sewer in Midway Road. Locating the pump station at the WWTP would allow flow from future commercial and industrial users north of Twin Cities Road to be conveyed by gravity directly to the WWTP.

Alternative 2 proposes to locate the pump station at the intersection of Live Oak Avenue and Midway Road. Flow would be conveyed to the WWTP in a new 20-inch diameter forcemain. If the Live Oak pump station were located at the intersection of Live Oak Avenue and Midway Road, a second lift station would be required east of the railroad tracks near the WWTP to convey flow that originates north of Twin Cities Road.

The criteria developed to evaluate the two alternative pump station locations address several areas including required facilities, operation and maintenance, pipeline costs, and reliability. A complete list of the criteria is as follows:

1. **Twin Cities Lift Station** - If the Live Oak pump station is located at the WWTP, future users in the area north of Twin Cities Road can be served by this pump station. Alternative 2 will require a second pump station to convey flow from users north of Twin Cities road to the WWTP. An alternative scored higher if only one pump station was required.

2. **Flexibility for Future Development** - The location of the Live Oak pump station will have an impact on the relative ease with which future users can be served. Conveying flow by gravity to the

WWTP would allow future users around Midway Road greater ease of connection. An alternative scored higher if it provided more opportunities for connecting future users.

3. **Reliability** - Reliability of each alternative. Alternative scored higher if it was viewed as being more reliable.

4. **Pipeline Costs** - Relative comparison of pipeline costs. Alternative scored higher if it was more economical.

5. **Live Oak Pump Station Costs** - Compares the difference in cost of the pump station for both locations. An alternative scored higher if the estimated costs of the pump station were lower.

6. **Operations & Maintenance** - Impact to operations and maintenance staff. An alternative scored higher if it had less impact on O&M staff.

Table 3 summarizes the evaluation of the alternatives in a similar fashion to the southeast trunk evaluation. As shown, locating the pump station at the WWTP (Alternative 1) has a weighted total of 42, while Alternative 2 has a weighted score of 29.

Cost Comparison of Live Oak Alternatives

Carollo also developed preliminary cost estimates for both alternatives. Table 4 summarizes the estimated construction and total project cost for each alternative. While the criteria provide a qualitative standard by which to judge the alternatives, the Flexibility for Future Development and O&M criteria also have a cost component that will impact the outcome. Since Alternative 2 requires a forcemain to convey flow to the WWTP, future rural residential development north of Live Oak Avenue and west of Highway 99 (Figure 1) would need a gravity trunk sewer to convey flow south to the future pump station. With Alternative 1, development could connect directly to the proposed 42-inch diameter gravity sewer and avoid a significant capital project. Carollo determined that the total project cost for a 10-inch diameter gravity sewer extending from Van Parker Lane to the intersection of Midway Road and Live Oak Avenue is estimated to cost approximately $0.98 million.

In addition to the costs for the 10-inch diameter sewer, locating the pump station at Midway Road and Live Oak Avenue would require a second lift station located near Twin Cities Road to serve future development. The addition of this second lift station along with the associated O&M costs for two lift stations located in the collection system would increase the overall O&M costs for Alternative 2. Based on experience with other municipalities, Carollo estimated that 16 additional man-hours per week are required to maintain two lift stations in the field versus one located at the WWTP. If we assume a compensation of $40 per hour (salary and benefits) and assuming an escalation of staff salaries at 3 percent per year and an interest rate of 5 percent, the net present worth of the additional O&M costs over 20 years totals $0.54 million dollars. These cost, along with the cost for the 10-inch sewer are included in Table 4 for Alternative 2.

As shown in Table 4, locating the lift station at the WWTP (Alternative 1) would cost approximately $22.98 million, while Alternative 2 would cost approximately $23.29 million. Therefore, based on a preliminary cost assessment, Alternative 1 would result in lower capital cost by $0.31 million, which is less than two percent of the total. The total capital and O&M costs for both alternatives are essentially the same.

The benefits of locating the pump station at the WWTP and conveying flow with a gravity sewer would be considerable. City staff could better maintain and manage pumping operations. The gravity sewer can be regularly inspected, while regular inspection of a forcemain would be difficult and would require temporary shut down of the pump station. The gravity sewer would also provide a greater level of reliability. Based on the decision matrix, and the evaluation of costs, Carollo recommends locating the pump station at the WWTP.

Table 3 - Live Oak Pump Station Location Alternatives
Sewer System Master Plan
City of Galt

Criteria	Importance (High = 3, Medium = 2, Low = 1)	Location of Lift Station		Weighted Score	
		WWTP	Midway Road	WWTP Alternative	Midway Alternative
1. Twin Cities Lift Station	3	3	1	9	3
2. Flexibility for Future Development	2	3	1	6	2
3. Reliability	3	3	1	9	3
4. Pipeline Costs	3	1	3	3	9
5. Live Oak LS Costs	3	2	3	6	9
6. Operations & Maintenance	3	3	1	9	3
			Total	**42**	**29**

Description of Criteria

1. If the Live Oak LS is located at the WWTP future users in the area north of Twin Cities Road can be served without an additional lift station. The Midway location will require a lift station to convey flow from users north of Twin Cities road to the WWTP.

2. The location of the Live Oak lift station will have an impact on the relative ease that future users in the area can be served.

3. Reliability of each alternative. Alternative scored higher if it was viewed as being more reliable.

4. Relative comparison of pipeline costs. Alternative scored higher if it was more economical.

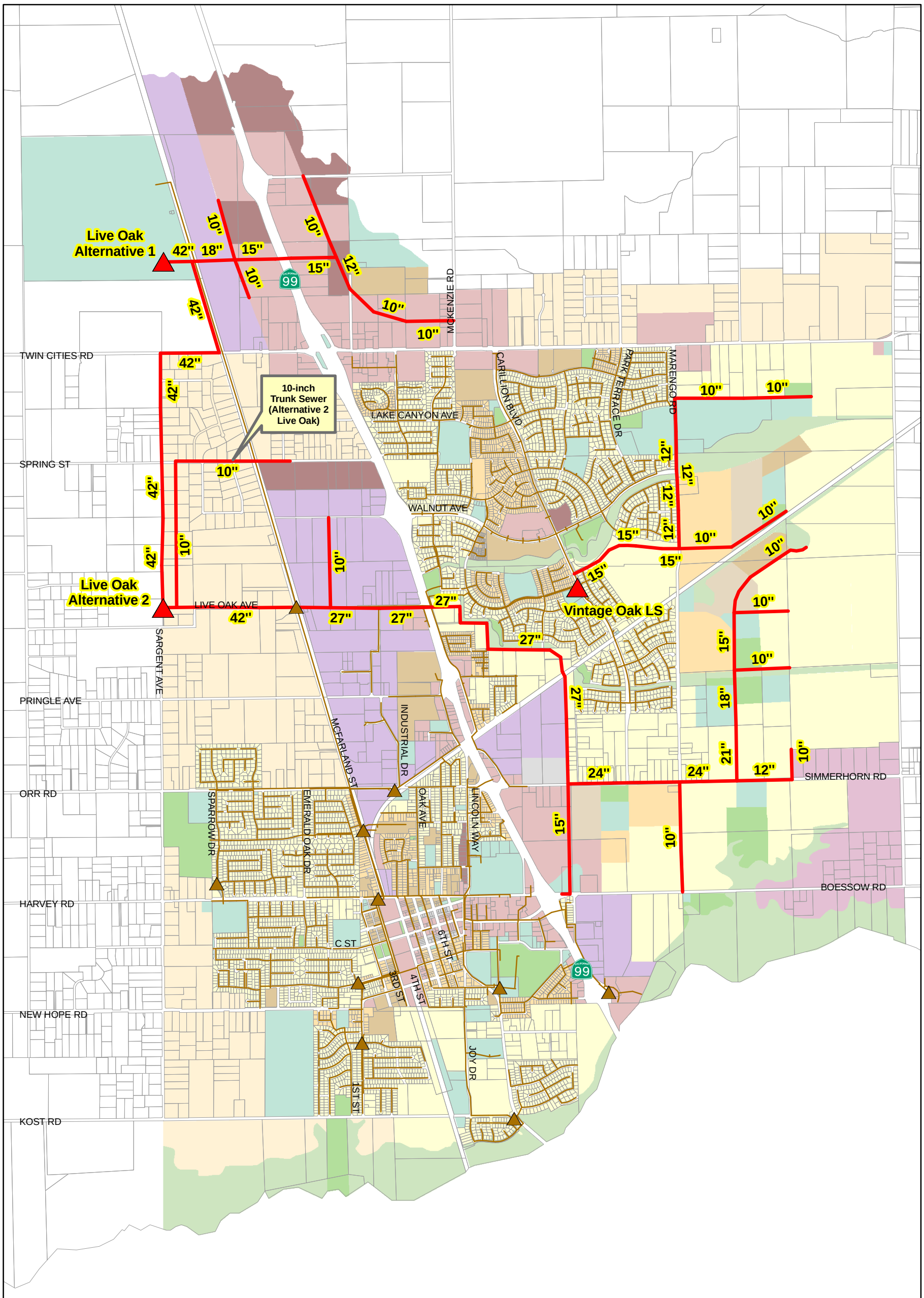
5. Compares the difference in cost of the lift station for both locations. An alternative scored higher if the estimated costs of the lift station were lower.

6. Impact to operations and maintenance staff. An alternative scored higher if it had less impact on O&M staff.

Table 4 Alternative Cost Comparison
Sewer System Master Plan
City of Galt

	Construction Cost[1] (Million $)	Engineering, CM, and Project Administration (Million $)	Pump Station O&M Cost (Million $)	Total Project Cost[7] (Million $)
Alternative 1 - WWTP Influent PS[2]				
WWTP Pump Station (18.2 mgd[4])	$13.66	$4.10	--	$17.76
42-inch Gravity Trunk	$4.01	$1.20	--	$5.21
Total Cost	**$17.67**	**$5.30**	**--**	**$22.97**
Alternative 2 - Live Oak PS[2]				
Live Oak Pump Station (17.5 mgd[5])	$13.06	$3.92	--	$16.98
20-inch Forcemain	$1.81	$0.55	--	$2.36
Twin Cities LS (2.8 mgd[4])	$1.87	$0.56	--	$2.43
10-inch Gravity Sewer	$0.75	$0.23	--	$0.98
Additional O&M for Twin Cities Pump Station[(3)]	--	--	$0.54	$0.54
Total Cost	**$17.49**	**$5.25**	**$0.54**	**$23.29**

Note:
1. ENR CCI 20 City average used for estimating (March 2009) = 8,534
2. These cost only account for the reach of pipeline from the intersection of Live Oak Avenue and Midway Road to the WWTP.
3. Assumes 3 percent per year and an interest rate of 5 percent, the net present worth of the additional O&M costs over 20 years totals $0.54 million dollars.
4. Total Capacity of 18.2 mgd and a firm capacity of14.55 mgd, for a design flow of 14.55 mgd. Assumes five 3.64 mgd pumps.
5. Total Capacity of 17.5 mgd and a firm capacity of 14.00 mgd, for a design flow of 14.00 mgd. Assumes five 3.50 mgd pumps.
6. Total Capacity of 2.80 mgd and a firm capacity of1.40 mgd, for a design flow of 1.40 mgd. Assumes two 1.40 mgd pumps.
7. Sum of Construction, Engineering, CM, Administration, and O&M Costs



Legend

Existing Collection System
- Lift Station
- Pipelines

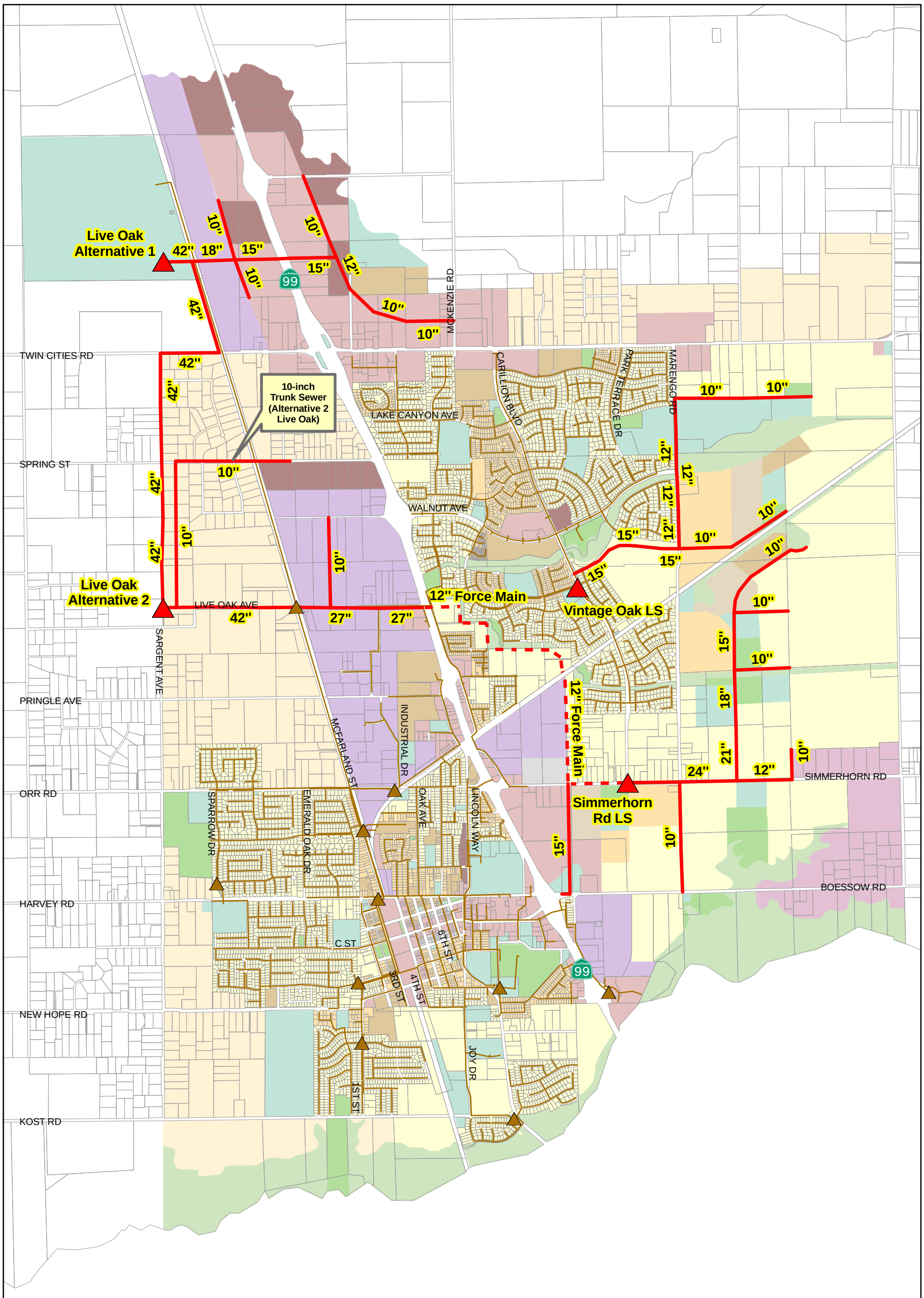
Proposed Improvements to Serve Future Users
- Lift Station
- Pipelines
- Parcels

General Plan Land Use
- A-Rural Residential
- B-Low Density Residential
- C-Medium Density Residential
- D-High Density Residential
- E-Commercial
- F-Office Professional
- G-Light Industrial
- H-Public/Quasi-Public
- I-Open Space
- I-Park
- K-Rural Estate
- L-Medium High Density Residential
- M-Mixed Use

Text:
- 12" CIP Diameter

0 2,500 5,000 Feet

Figure 1
Proposed Improvements To Serve Future Users - Alternative 1
Sewer Collection System Master Plan
City of Galt



Legend

Existing Collection System
- Lift Station
- Pipelines

Proposed Improvements to Serve Future Users
- Lift Station
- Gravity Pipeline
- Force Main
- Parcels

General Plan Land Use
- A-Rural Residential
- B-Low Density Residential
- C-Medium Density Residential
- D-High Density Residential
- E-Commercial
- F-Office Professional
- G-Light Industrial
- H-Public/Quasi-Public
- I-Open Space
- I-Park
- K-Rural Estate
- L-Medium High Density Residential
- M-Mixed Use

Text:
12" CIP Diameter

0 2,500 5,000 Feet

Figure 2
Proposed Improvements To Serve Future Users - Alternative 2
Sewer Collection System Master Plan
City of Galt

APPENDIX F – SANITARY SEWER FLOW MONITORING AND INFLOW/INFILTRATION STUDY

SANITARY SEWER FLOW MONITORING AND INFLOW / INFILTRATION STUDY

City of Galt

March 2009

V&A

CITY OF GALT

Sanitary Sewer Flow Monitoring
and
Inflow / Infiltration Study

Prepared For:

CAROLLO ENGINEERS
7580 North Ingram Ave, Suite 112
Fresno, CA 93711

Prepared By:

V&A
Lake Merritt Plaza
1999 Harrison Street, Suite 975
Oakland, CA 94612

March 2009

TABLE OF CONTENTS

DESCRIPTION PAGE NO.

1 EXECUTIVE SUMMARY 1
1.1 Flow Monitoring and I&I Results 1
2 INTRODUCTION 6
3 METHODS AND PROCEDURES 9
3.1 Confined Space Entry 9
3.2 Flow Meter Installation 10
4 RAINFALL RESULTS 11
5 STORM EVENT CLASSIFICATION 13
6 FLOW MONITORING RESULTS 16
6.1 Average Dry Weather Flows 16
6.2 Inflow/Infiltration: Methods 18
6.2.1 Definitions 18
6.2.2 Analysis Techniques 21
6.3 Inflow Analysis 23
6.4 GWI Analysis 25
6.5 Combined I&I Results Summary 27
6.6 Pipeline Capacity Analysis 29
7 RECOMMENDATIONS 32

TABLES

Table 1-1 Flow Monitoring and I&I Results Summary 1
Table 2-1 Sewer Basins Information 6
Table 4-1 Summary of Storm Events 11
Table 5-1 Peak Measured Rainfall Densities per Hourly Period (Rain Gauge 1) 14
Table 6-1 Dry Weather Flow Summary 17
Table 6-2 Inflow Analysis Summary 23
Table 6-3 Excess Groundwater Infiltration Rates above Typical Rates 26
Table 6-4 Combined I&I Analysis Summary 27
Table 6-5 Capacity Analysis Summary 29

FIGURES

Figure 1-1. Bar Graphs Summary of Analysis Markers 2
Figure 1-2. Temperature Map: Peak Inflow Rate to ADWF Ratio 3
Figure 1-3. Temperature Map: R-Value (Combined I&I) 4
Figure 2-1. Street Map of Flow Monitoring Locations 6
Figure 2-2. Sewer Base Map of Flow Monitoring Locations 7

Figure 2-3. Basin Map 8
Figure 3-1. Typical Sigma 910 Installation 10
Figure 4-1. Rainfall Activity over Flow Monitoring Period 11
Figure 4-2. Rainfall Accumulation Plots 12
Figure 5-1. NOAA Northern California Rainfall Frequency Map 13
Figure 5-2. City of Galt Rain Event Frequency Map 14
Figure 5-3. City of Galt Storm Event Classification 15
Figure 6-1. Site 1: Average Dry Weather Flow 16
Figure 6-2. Measured Average Dry Weather Flow Schematic 17
Figure 6-3. Infiltration / Inflow Locations and Components 19
Figure 6-4. Infiltration/Inflow Graphical Response Patterns 20
Figure 6-5. Sample I&I Flow Graph 22
Figure 6-6. Inflow Markers by Site 23
Figure 6-7. Temperature Map: Peak Inflow Rate to ADWF Ratio 24
Figure 6-8. Groundwater Infiltration Sample Figure 25
Figure 6-9. Peak and Minimum Flow Ratios vs. ADWF 26
Figure 6-10. Bar Graph of R-Value by Site 27
Figure 6-11. Temperature Map: R-Value 28
Figure 6-12. Capacity Rankings: Peaking Factor and d/D Ratio by Site 30
Figure 6-13. Peak Measured Flow Schematic 31

APPENDIX A. FLOW MONITORING SITES: DATA, GRAPHS, INFORMATION

1 EXECUTIVE SUMMARY

V&A has completed sanitary sewer flow monitoring and inflow and infiltration (I&I) analysis at six locations within the City of Galt (City) collection system. Flow monitoring was conducted over a 1-month period from December 13, 2008 to January 15, 2009. The flow monitoring sites are shown in Figure 2-1 (Page 6) of this report.

1.1 Flow Monitoring and I&I Results

Table 1-1 summarizes the flow monitoring results and infiltration and inflow results for each flow monitoring site. The infiltration and inflow (I&I) results shown in this table are taken from a wet weather event that occurred from December 24, 2008 to December 25, 2008.

Table 1-1
Flow Monitoring and I&I Results Summary

Meter	ADWF (MGD)	Estimated Total I&I (gallons)	R-Value (%)	Peak I&I Rate (MGD)	Peak I&I to ADWF Ratio	Inflow Rank	Combined I&I Rank
M1	0.99	301,000	0.85%	0.73	0.74	**3**	**2**
M2	0.29	39,000	0.47%	0.14	0.49	**6**	**6**
M3	0.69	155,000	0.79%	0.45	0.66	**5**	**3**
M4	0.52	108,000	0.53%	0.39	0.75	**2**	**5**
M5	0.24	83,000	0.67%	0.36	1.46	**1**	**4**
M6	0.18	45,000	1.01%	0.13	0.71	**4**	**1**

The following results from this project are noted:

- **Inflow:** Basin 5 had higher inflow rates compared to the other flow monitoring sites.
- **Combined I&I:** Basin 6 had the highest combined I&I contribution relative to basin size.
- **Groundwater Infiltration:** There may be slightly higher-than-normal groundwater infiltration occurring in Basin 6 during periods of dry weather flow.
- **Capacity (Peaking Factor):** All sites had peaking factors below the typical design threshold limits for peak flow to average dry weather flow ratio during this study.
- **Capacity (d/D Ratio):** Sites M4 and M5 had peak d/D Ratios greater than 1, indicating a surcharged condition. Sites M4 and M5 surcharged approximately 11.1 and 19.5 inches above the pipe crown, respectively. This is due to the downstream lift station “A Street” Lift Station which backs up flow into incoming sewer lines.

Figure 1-1 graphically summarizes various infiltration, inflow and capacity results. Figure 1-2 and Figure 1-3 show temperature maps of the Peak I&I to ADWF Ratio and R-Value markers, respectively.

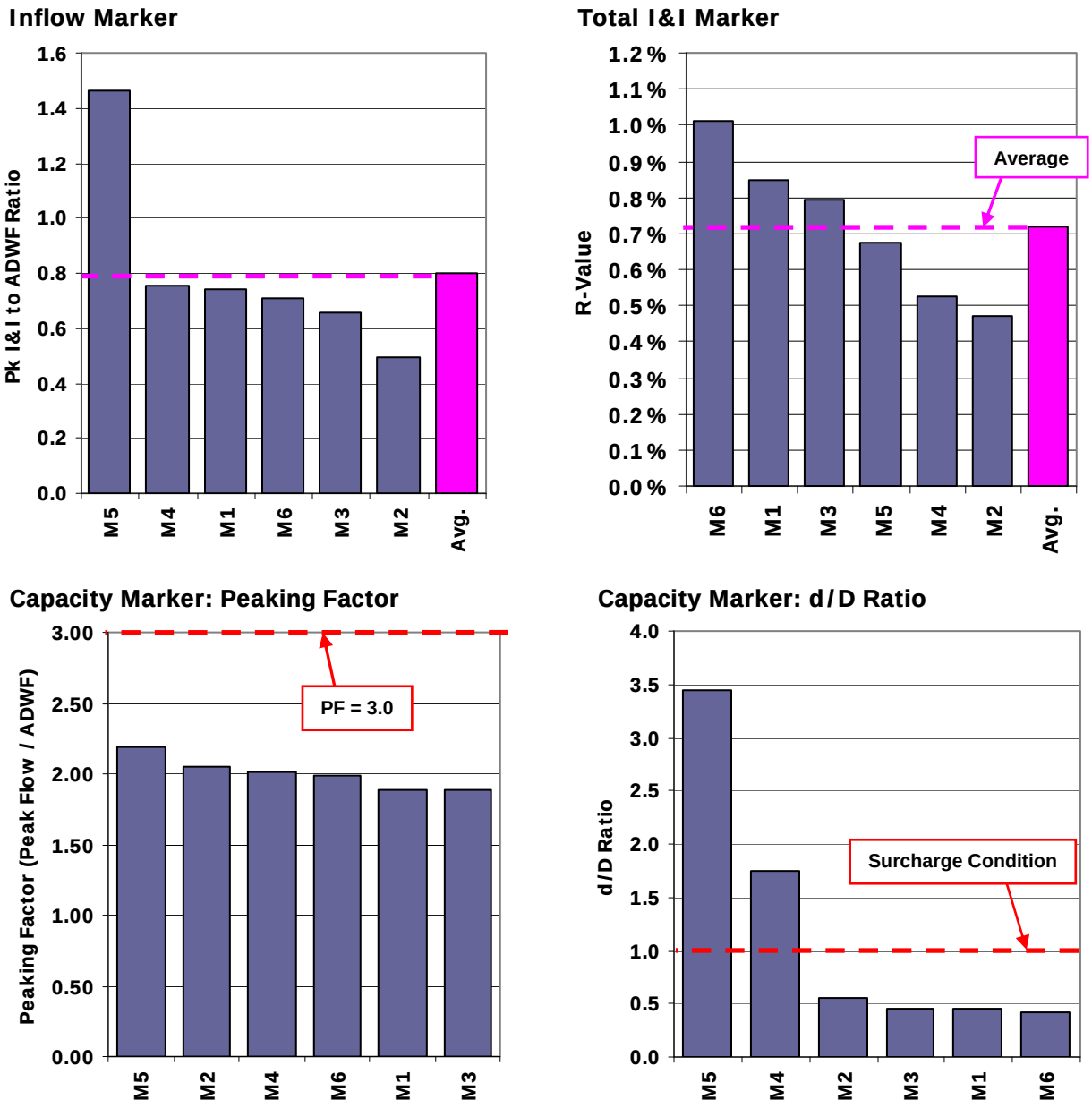


Figure 1-1. Bar Graphs Summary of Analysis Markers

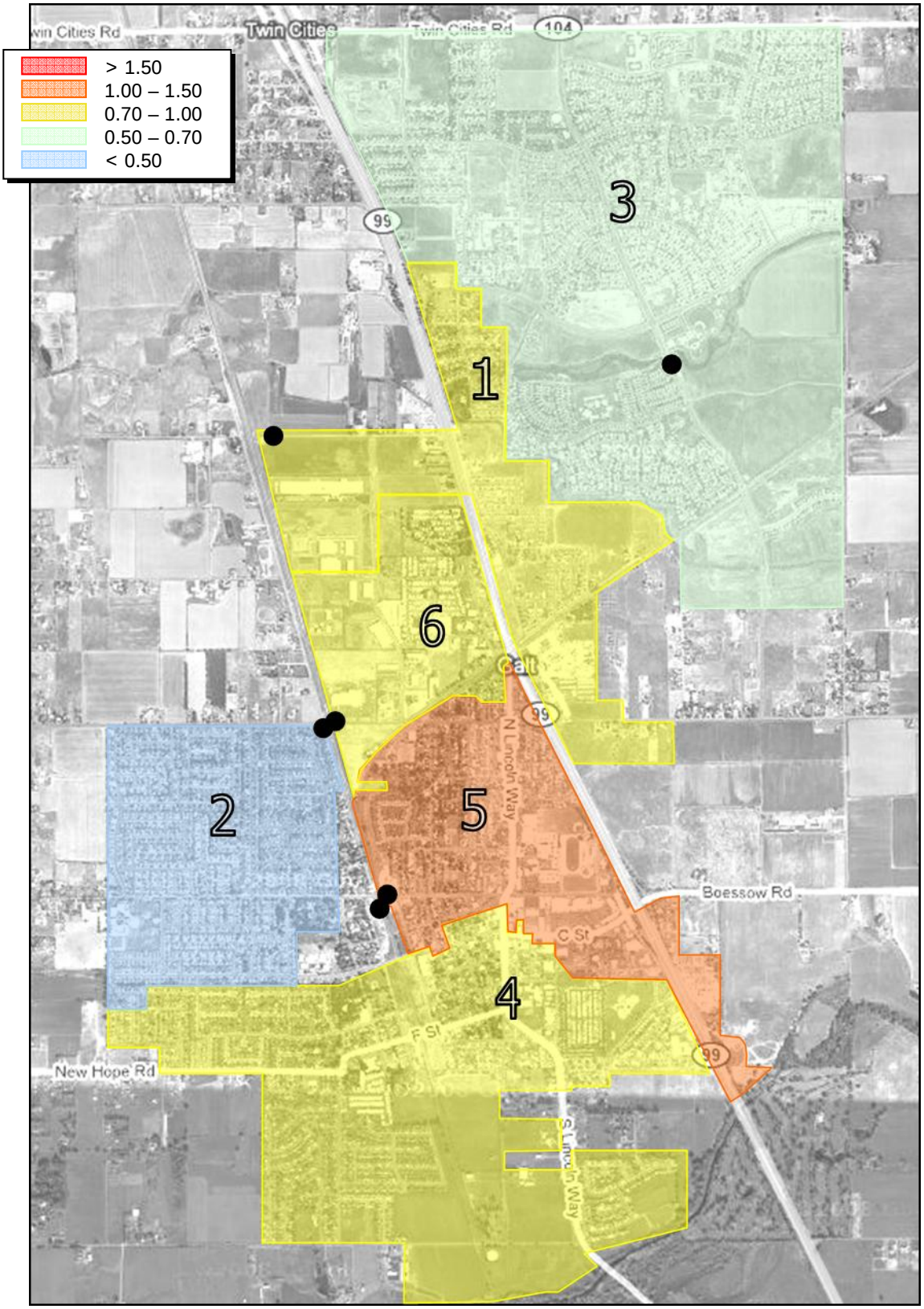


Figure 1-2. Temperature Map: Peak Inflow Rate to ADWF Ratio

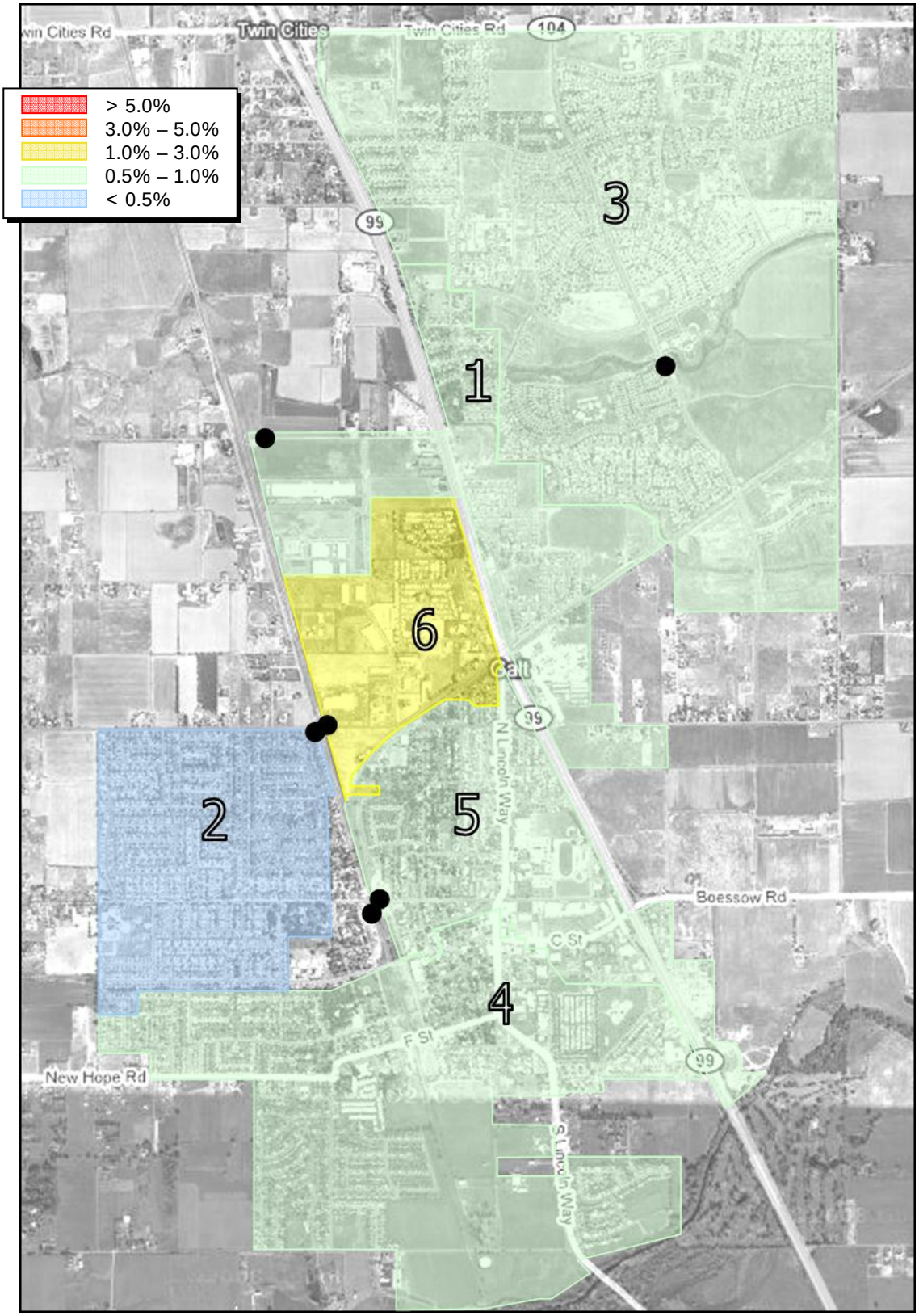


Figure 1-3. Temperature Map: R-Value (Combined I&I)

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

1. **Determine an I&I Reduction Program:** The City should examine its I&I reduction needs to determine a future I&I reduction program.
    a. If peak flows, sanitary sewer overflows and pipeline capacity issues are of greater concern, then the program can be weighted to investigate and reduce sources of inflow within the basins with the greatest inflow problems.
    b. If total infiltration and general pipeline deterioration is of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems.
2. **I&I Reduction Methods:** Potential I&I reduction methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
    d. CCTV inspection
3. **I&I Reduction Cost Effective Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of infiltration and inflow and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional storm water I&I flow.
4. **Future Flow Monitoring:** The City should consider volumetric state logger flow monitoring at lift stations as a cost-effective way to conduct long-term flow monitoring. Besides tracking flows and flow responses to rainfall events, it would also increase understanding of pump station efficiency as well. A review of the number of pump starts per hour may assist the City in determining if a more efficient hydraulic operation would improve overall pump station life cycle costs. Note: There are twelve lift stations shown on Figure 2-2 (Page 7).

    Should the City decide to install permanent open channel flow meters at the sites monitored for this study, two metering types would be recommended. Sites M4 and M5 surcharge above the pipe crown due to the downstream lift station operation which backs up flow into incoming sewer lines. For this reason, an area-velocity flow monitor with submerged probe such as Teledyne Isco 2150 model should be used at Sites M4 and M5.

    A non-contact open channel flow monitor such as the Hach Marsh-McBirney Flo-Dar can be used at the remaining flow meter sites. The Flo-Dar typically requires less maintenance than submerged probe meters, saving on recurring maintenance costs.

2 INTRODUCTION

V&A Consulting Engineers (V&A) has completed sanitary sewer flow monitoring and inflow and infiltration (I&I) analysis within the City of Galt. Flow monitoring occurred over a 1-month period from December 13, 2008 to January 15, 2009 at six flow monitoring sites. Table 2-1 describes the flow monitoring locations and upstream sewerage basins. Figure 2-1, Figure 2-2 and Figure 2-3 show the street, sewer and basin maps of the project, respectively. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in *Appendix A*.

Table 2-1
Sewer Basins Information

Basin	Acres	Flow Monitor	Pipe Diameter (in)
Basin 1	1,504	M1	24
Basin 2	353	M2	10
Basin 3	828	M3	21
Basin 4	870	M4	15
Basin 5	520	M5	8
Basin 6	188	M6	10

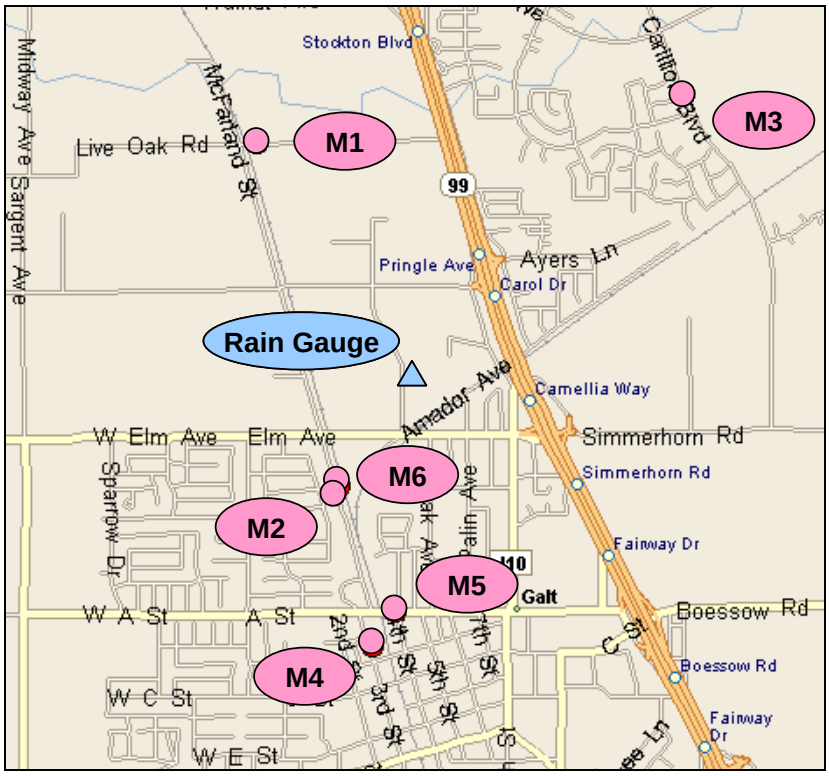


Figure 2-1. Street Map of Flow Monitoring Locations

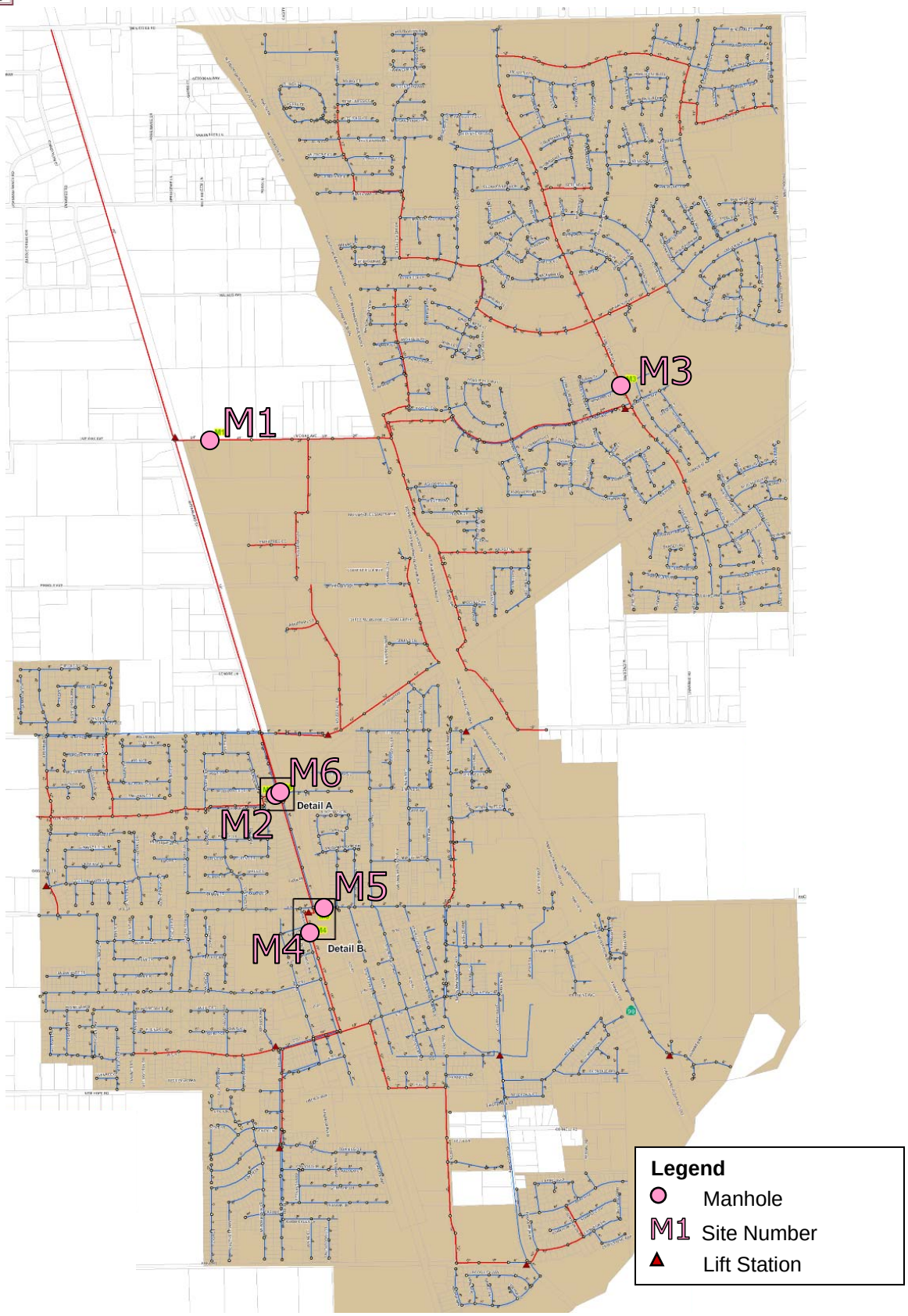


Figure 2-2. Sewer Base Map of Flow Monitoring Locations

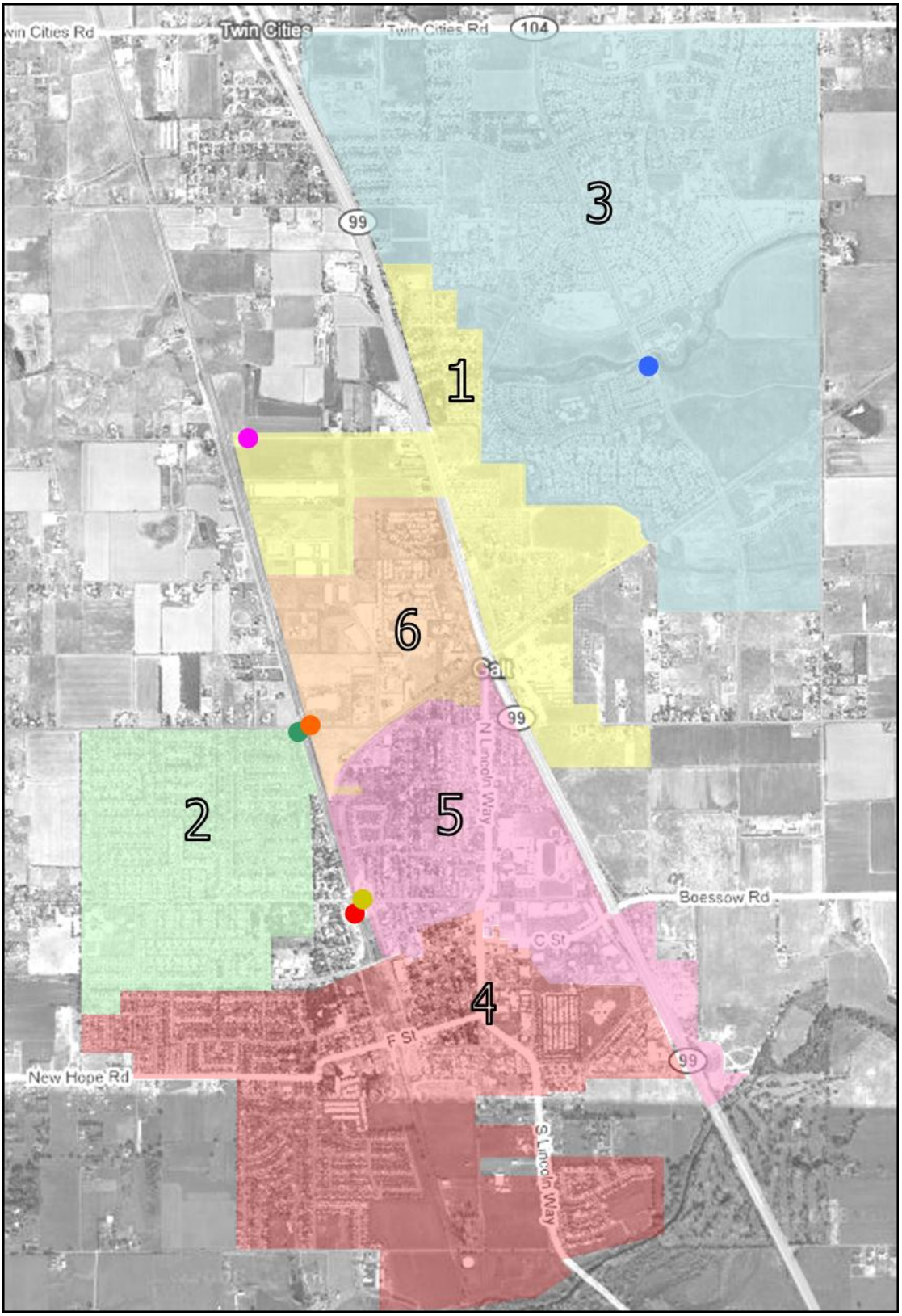


Figure 2-3. Basin Map

3 METHODS AND PROCEDURES

3.1 Confined Space Entry

A confined space (Photo 3.1) is defined as any space that is large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit and is not designed for continuous employee occupancy. Title 8, Section 5158 of the California Code of Regulations provides the guidelines and rules for working in these environments. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5 to 23.0%), and the absence of Hydrogen Sulfide (H_2S) gas, Carbon Monoxide (CO) gas and LEL levels. A typical confined space entry crew has at least three members; the entrant, the attendant and the supervisor. The entrant is the individual performing the work. He/she is equipped with all of the necessary personal protective equipment needed to perform the job safely, including a personal 4-gas monitor (Photo 3.2). If it is not possible to maintain line-of-sight with the entrant, then more entrants are required until line-of-sight can be maintained. The attendant is responsible for maintaining contact with the entrant(s) to monitor the atmosphere on another 4-gas monitor. He/she is also responsible for maintaining records of all entrants, if there is more than one. The supervisor develops the safe work plan for the job at hand.

Photo 3.1 – Confined Space Entry

Photo 3.2 – Typical Personal 4-Gas Monitor

3.2 Flow Meter Installation

Six Sigma 910 flow meters were installed by V&A in the sewer lines shown in Figure 2-1. Sigma meters use a pressure transducer to collect depth readings. Ultrasonic Doppler sensors on the probe determine the average fluid velocity. Figure 3-1 shows a sketch of a typical flow meter installation.

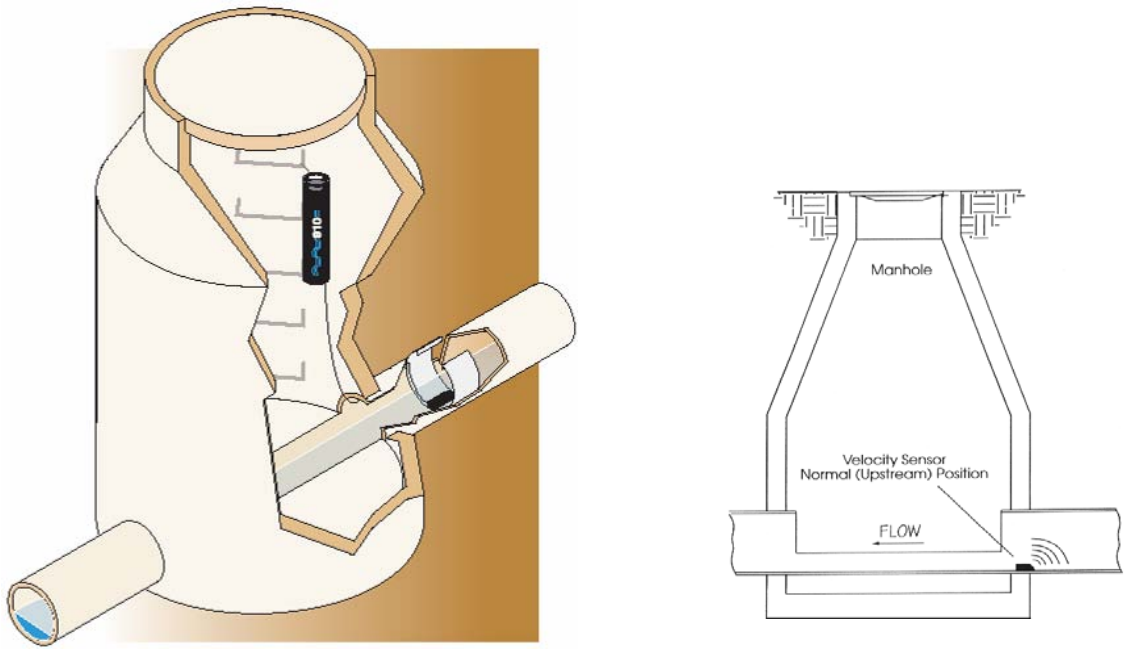


Figure 3-1. Typical Sigma 910 Installation

Continuous depth and velocity readings were recorded by the flow meters in 15-minute increments and downloaded into a computer spreadsheet program where the data could be analyzed and made report-ready. Manual level and velocity readings were taken in the field during the flow meter installation and again when removed. These were compared to the readings of the flow meters to ensure proper calibration and accuracy.

4 RAINFALL RESULTS

Over the course of this study, one rainfall event caused sufficient infiltration and inflow response to be used for I&I analysis. Table 4-1 summarizes the storm event used for I&I analysis during the monitoring period.

Table 4-1
Summary of Storm Events

Storm Event No.	Total Rain	Event Period
Event 1	0.87 inches	December 24 – 25, 2008*

* Rain event occurred on a holiday. Weekend ADWF curves were used to determine I&I

Figure 4-1 graphically displays the rainfall events recorded over the flow monitoring period.

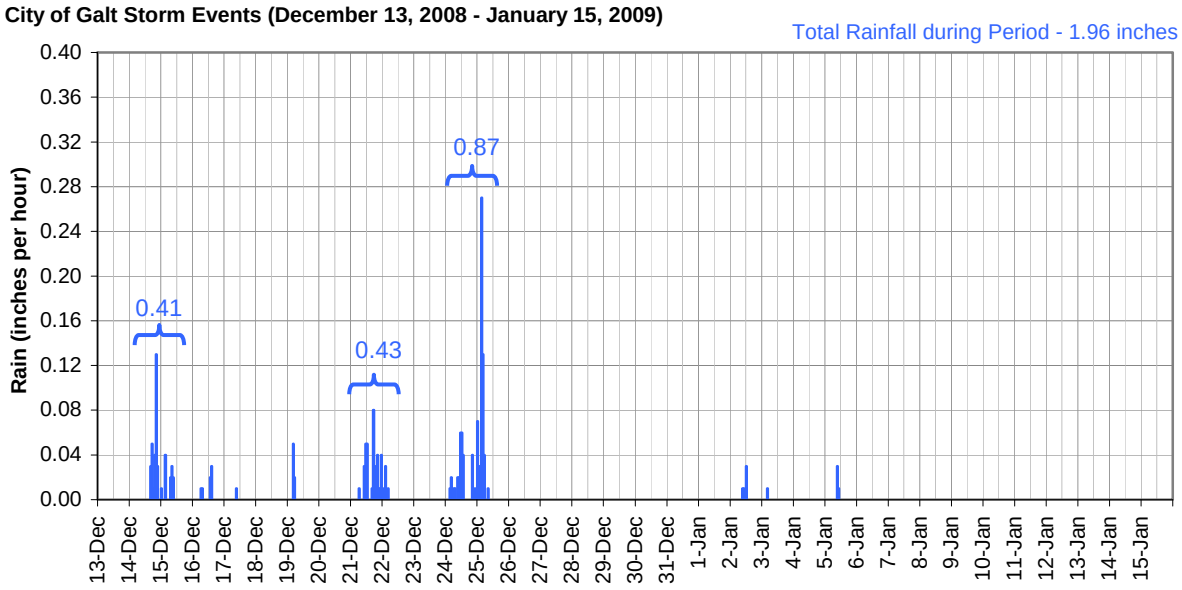


Figure 4-1. Rainfall Activity over Flow Monitoring Period

Figure 4-2 shows the rain accumulation plot of the rain gauges, as well as the historical average rainfall for the City during the duration of this project.

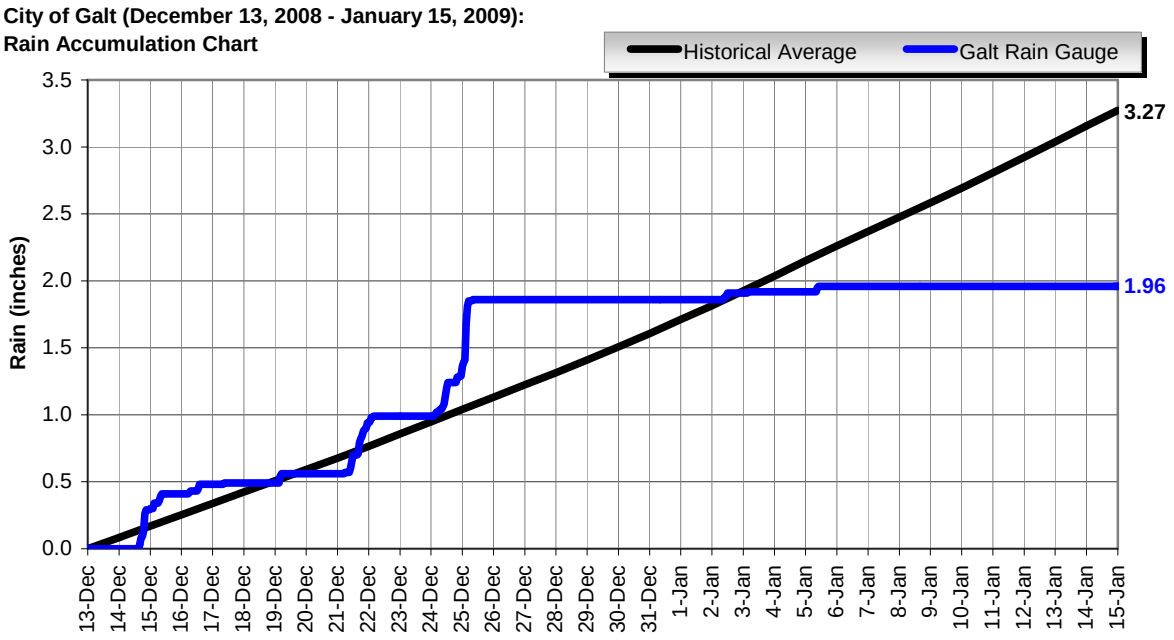


Figure 4-2. Rainfall Accumulation Plots

The historical average rainfall is shown for comparison to the rainfall that occurred over the course of the flow monitoring period (December 13, 2008 through January 15, 2009). The historical data was taken from the Western Regional Climate Center (WRCC) at Station 045032 in Lodi, California. Rainfall data from the years 1970 through 2000 were used to determine these averages. The historical average over the monitoring period is 3.27 inches. The rain gauge indicated a rainfall total of 1.96 inches, approximately 60% of normal levels for this period of time.

5 STORM EVENT CLASSIFICATION

It is important to classify the relative size of the major storm event that occurs over the course of a flow monitoring period[1]. Storm events are classified by intensity and duration. Based on historical data, frequency contour maps for given intensity and duration storm events have been developed by the National Oceanic and Atmospheric Administration (NOAA) for all areas within the continental United States. For example, the NOAA Rainfall Frequency Atlas[2] classifies a 10-year, 24-hour storm event in Galt as 2.62 inches (Figure 5-1). This means that in any given year, there is a 10% chance (1/10) 2.62 inches of rain will fall in any 24-hour period.

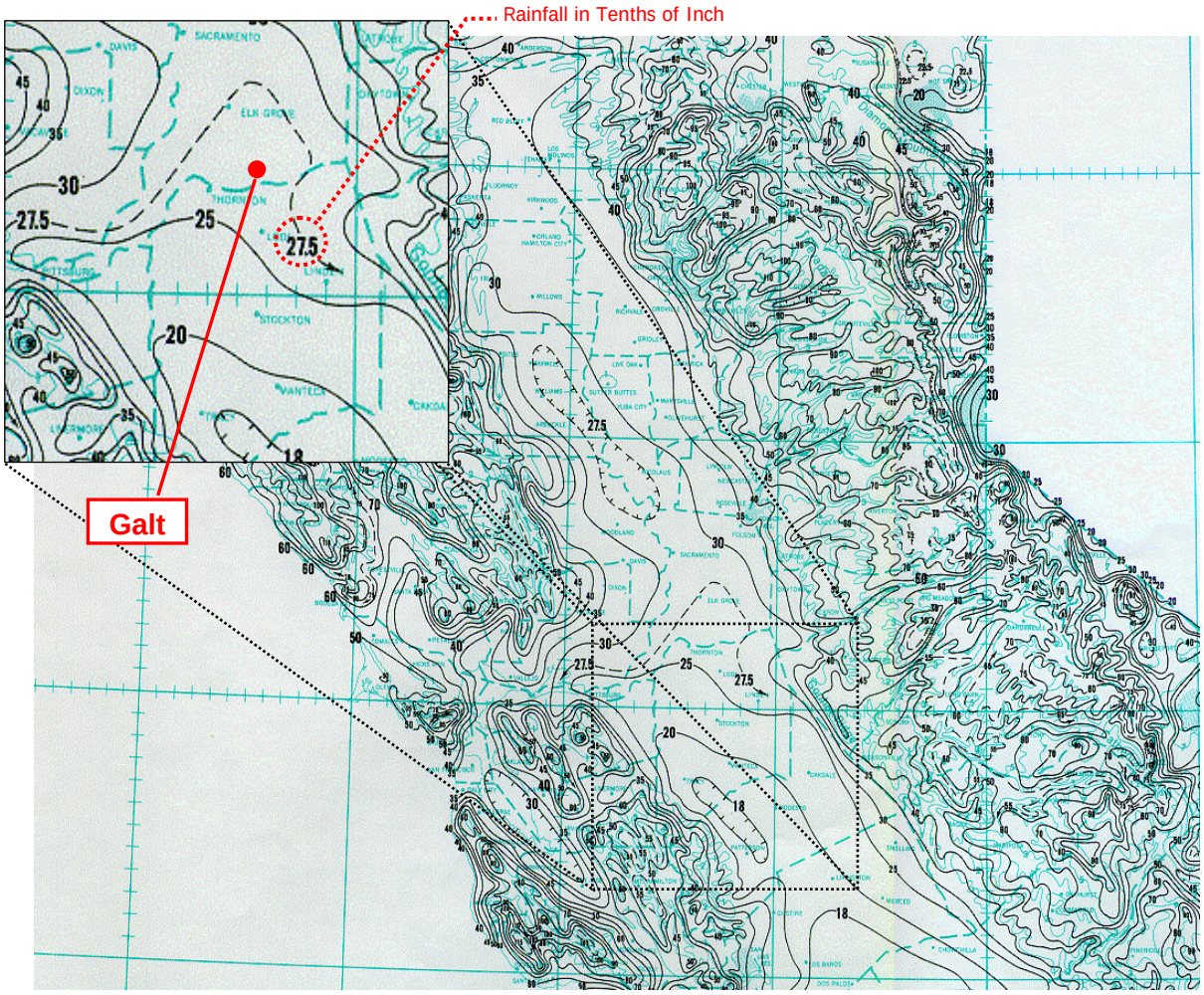


Figure 5-1. NOAA Northern California Rainfall Frequency Map

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

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

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

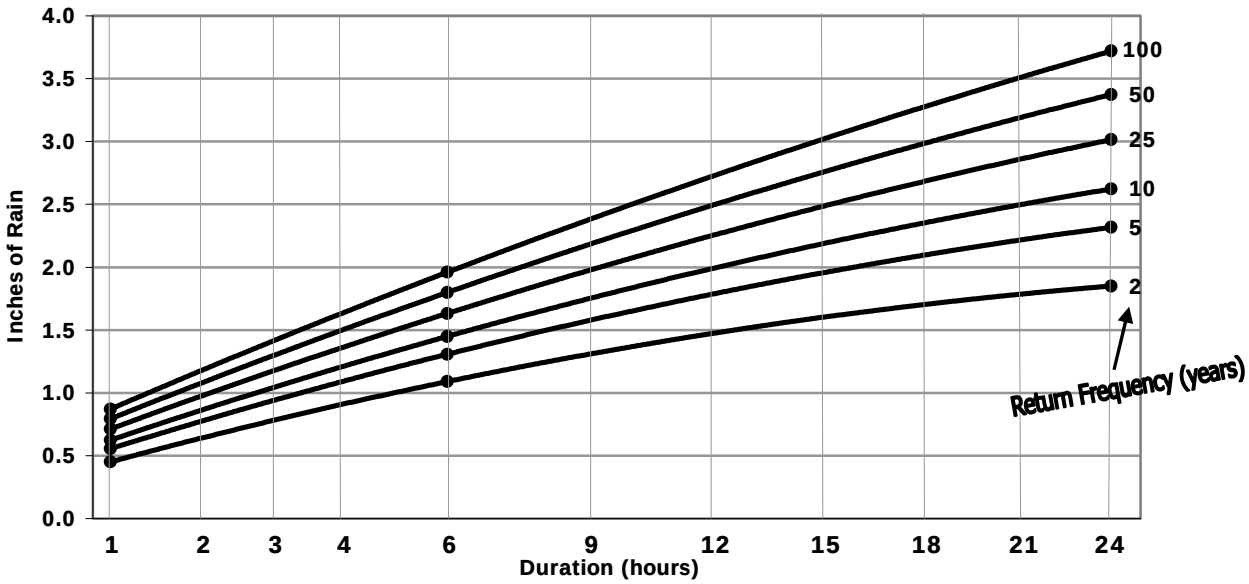


Figure 5-2. City of Galt Rain Event Frequency Map

The highest rainfall in any 1-hour period during this flow monitoring project was 0.27 inches and occurred during Storm Event 1. The highest consecutive 6-hour period was 0.56 inches, also occurring during Storm Event 1. Peak measured densities per hourly periods were calculated for both rainfall events, as summarized in Table 5-1. Superimposing the peak measured densities for the storm events on the Galt Rainfall Storm Event Classification Chart will determine the classification of the storm event, shown in Figure 5-3.

Table 5-1
Peak Measured Rainfall Densities per Hourly Period (Rain Gauge 1)

Rainfall Duration	Storm Event 1 (inches)
1 hr	0.27
3 hr	0.44
6 hr	0.56
12 hr	0.61
18 hr	0.73
24 hr	0.83

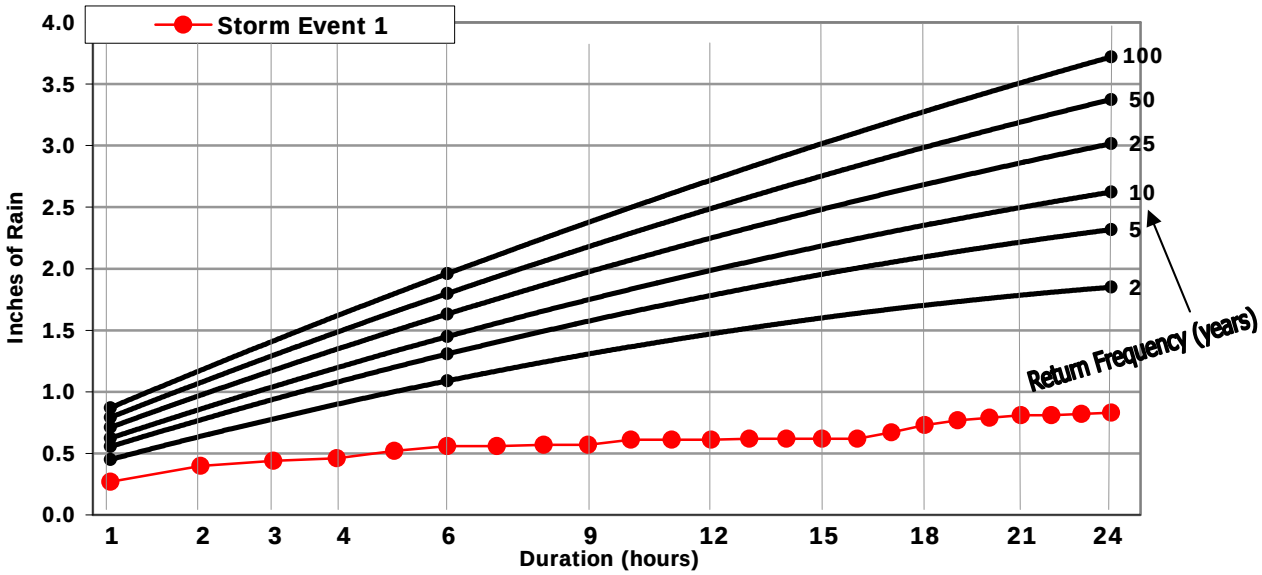


Figure 5-3. City of Galt Storm Event Classification

Storm Event 1 is classified as less than a 2-year storm event.

6 FLOW MONITORING RESULTS

6.1 Average Dry Weather Flows

Weekday and weekend flow patterns vary and must be separated when determining average dry weather flows. For this project, the following days were least affected by rainfall and were used to determine weekend and weekday average flows:

- Weekdays: January 5 – 9
- Weekends: January 3, 4, 10, 11

Figure 6-1 shows a sample of the average dry weather flow graph that was generated for each flow monitoring site. Graphs for each site are located in *Appendix A*.

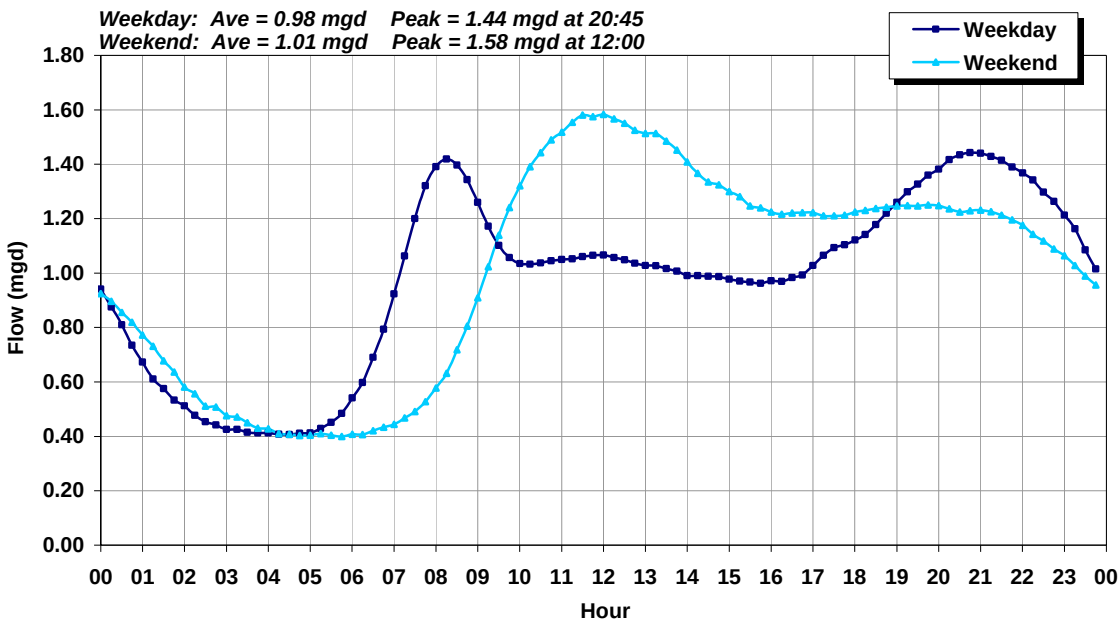


Figure 6-1. Site 1: Average Dry Weather Flow

Table 6-1 lists the average dry weather flow (ADWF) and average peak dry weather flows (PDWF) recorded during this study for the sites that were directly monitored. Figure 6-2 shows the system flow schematic presented as a pipe snapshot.

Table 6-1
Dry Weather Flow Summary

Location	Average Dry Weather Flow (MGD)		Weekend/ Weekday Ratio	Average Peak Dry Weather Flow (MGD)		PDWF/ADWF Ratio	
	Weekday	Weekend		Weekday	Weekend	Weekday	Weekend
M1	0.98	1.01	1.03	1.44	1.58	1.48	1.57
M2	0.28	0.30	1.06	0.47	0.43	1.66	1.45
M3	0.67	0.73	1.09	1.11	1.07	1.65	1.46
M4	0.51	0.52	1.01	0.74	0.83	1.43	1.60
M5	0.25	0.23	0.95	0.37	0.40	1.49	1.71
M6	0.18	0.18	0.98	0.23	0.24	1.28	1.37

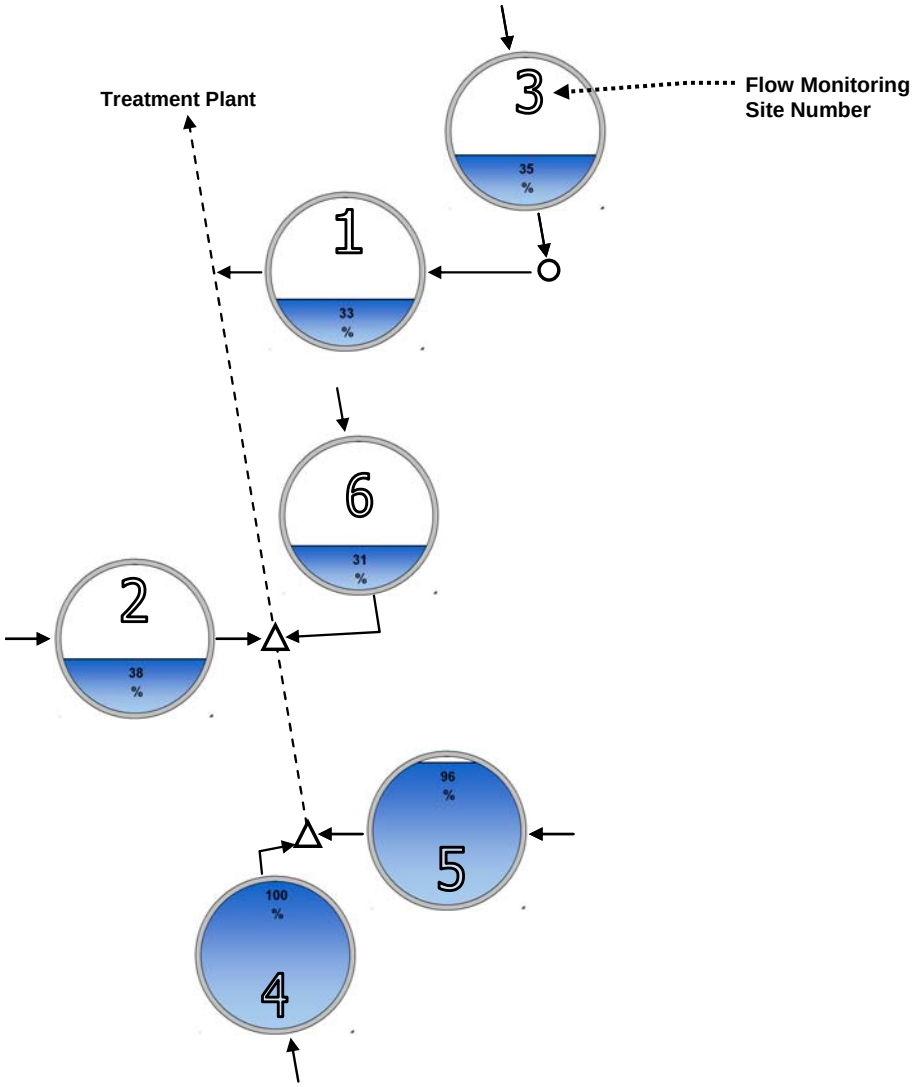


Figure 6-2. Measured Average Dry Weather Flow Schematic

6.2 Inflow/Infiltration: Methods

6.2.1 Definitions

Infiltration/inflow (I&I) consists of storm water and groundwater which enters the sewer system through pipe defects and improper storm drainage connections:

Inflow

Definition: Storm water inflow (SWI) is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard and area drains, holes in manhole covers, cross connections from storm drains or catch basins.

Impact: This component of I&I creates a **peak** flow problem in the sewer system, and together with RDI (explained below), dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow is tied closely to the intensity of the storm event, the short-term peak instantaneous flows may result in surcharging and overflows within a collection system. Severe inflow may result in sewage dilution, resulting in the upset of the biological treatment (secondary treatment) at the treatment facility.

Cost of Source Identification and Removal: Compared to infiltration sources, SWI locations are usually less difficult to find and usually less expensive to correct. These sources include direct and indirect cross connections with storm drainage systems, roof downspouts and various types of surface drains. Generally, the costs to identify and remove sources of SWI are low compared to the potential benefits to public health and safety or the costs of new facilities to transport the resulting peak flows.

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

Infiltration

Definition: Infiltration is defined as water entering the sanitary sewer system through defective pipes, pipe joints and manhole walls. These defects may include cracks, offset joints, root intrusion points and broken pipes.

Impact: Infiltration typically creates long-term annual **volumetric** problems. The major impacts of infiltration are the cost of pumping and treating the additional volume of water and paying for treatment (for municipalities that are billed strictly on flow volume).

Cost of Source Detection and Removal: Infiltration sources are usually harder to find and more expensive to correct than inflow sources. Infiltration sources include defects in deteriorated sewer pipes and/or manholes. These defects may include cracks, offset joints, root intrusion points and broken pipes. The sources may be wide-spread throughout a sanitary sewer system.

Graphical Identification: Infiltration is often recognized graphically by a gradual increase in flow after a wet weather event. The increased flow typically sustains for a period after the rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels.

Infiltration can be further subdivided into the following components:

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

Figure 6-3 illustrates the possible locations and components of I&I.

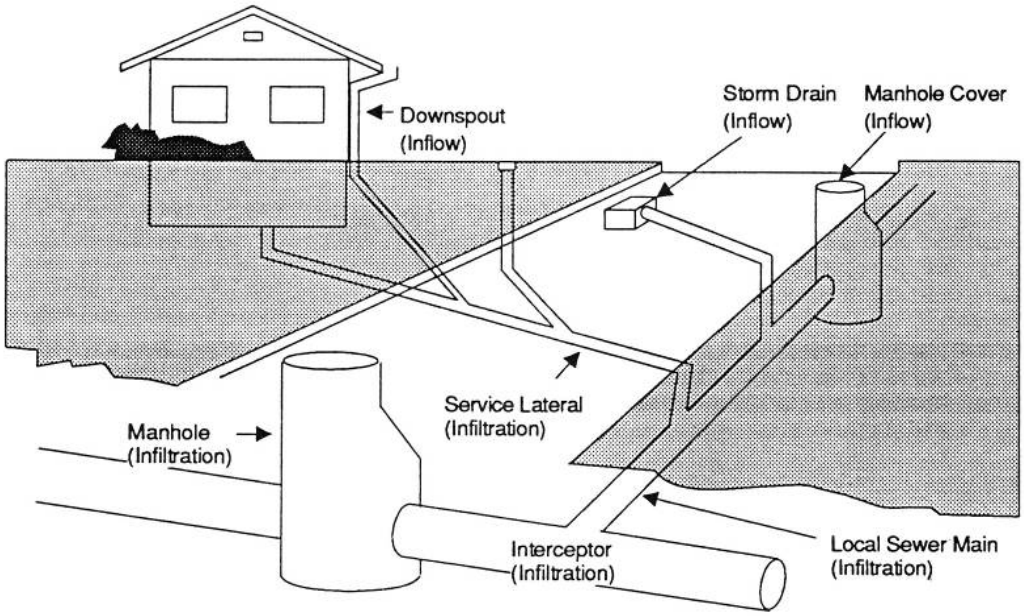


Figure 6-3. Infiltration / Inflow Locations and Components

Graphical Identification of I&I Components

Figure 6-4 shows sample graphs indicating the typical graphical response patterns for inflow and infiltration.

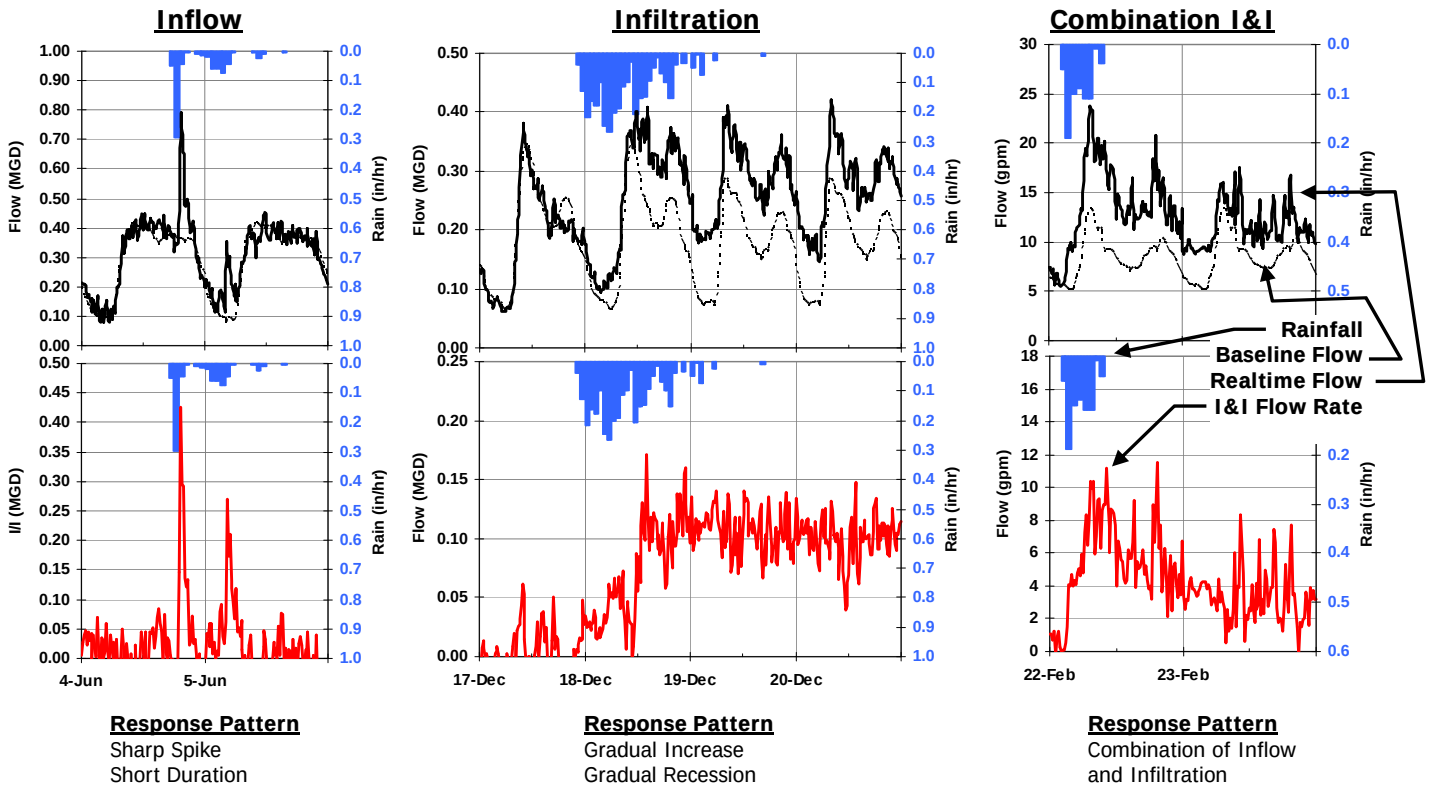


Figure 6-4. Infiltration/Inflow Graphical Response Patterns

6.2.2 Analysis Techniques

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

Inflow Indicators

Peak I&I Flow to ADWF Ratio: Peak I&I to ADWF Ratio is the peak measured I&I rate divided by ADWF. This ratio is a preferable comparative tool for I&I analysis to peaking factor because it looks strictly at I&I flow rates. Peaking factor (defined as peak flow divided by ADWF) can be skewed higher or lower depending on whether the storm event I&I response occurs during low flow or high flow hours. Peaking factor is a useful tool when looking at capacity issues (see the next section on *Capacity*).

Infiltration Indicators

Dry Weather Groundwater Infiltration: GWI analysis is conducted by looking at minimum dry weather flow to average dry weather flow ratios and comparing them to established standards. These methods are discussed in further detail in the *Groundwater Analysis* section later in this report. When quantified, units are typically stated in gallons per acre day (gpad) or gallons per day per Inch-Diameter-Mile (gpd/IDM).

Combined I&I Indicators

R-Value: When the acreages of each basin and total I&I attributable to a storm event is known, the percentage of rainfall that permeates into each basin can be calculated. This is called the R-Value. The R-Value method provides a means to compare the relative magnitude and severity of total I&I volume between different basins and different storm events. Systems with R-Values less than 5%[3] are often considered to be performing well. This criterion will be used for this study.

Figure 6-5 shows a sample I&I graph that illustrates and summarizes the I&I response and I&I calculations made per site per storm event. Similar graphs for each site and storm event are located in *Appendix A*.

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

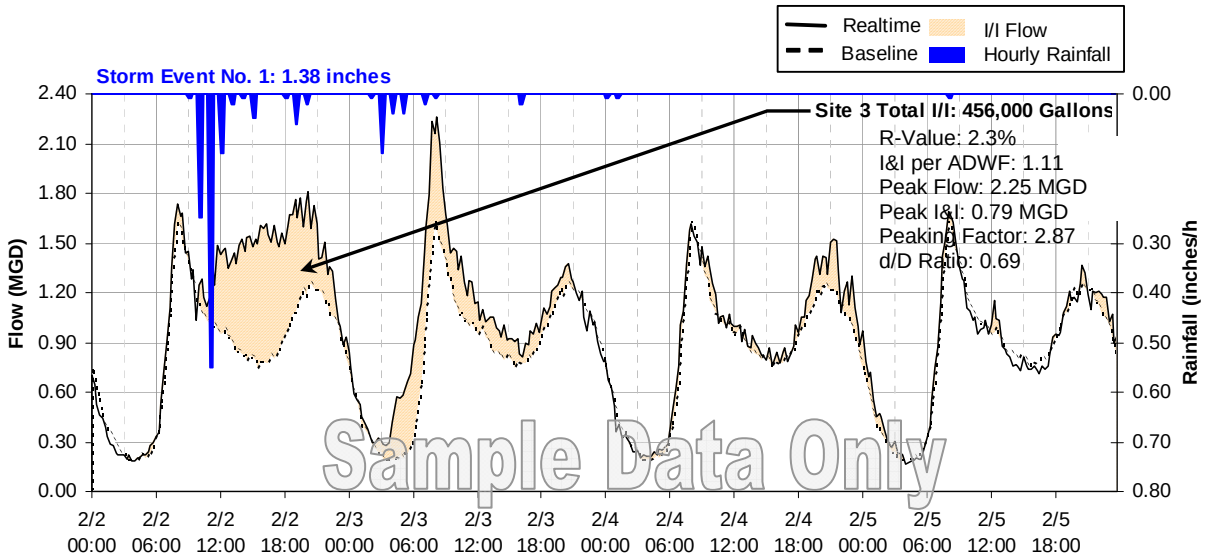


Figure 6-5. Sample I&I Flow Graph

6.3 Inflow Analysis

Table 6-2 summarizes the peak I&I flows and inflow analysis results for the flow monitoring locations. Inflow results were taken from the December 24 – 25, 2008 wet weather event.

Table 6-2
Inflow Analysis Summary

Location	ADWF (mgd)	Peak I&I Flow (mgd)	Peak I&I to ADWF Ratio	Inflow Ranking
M1	0.99	0.73	0.74	**3**
M2	0.29	0.14	0.49	**6**
M3	0.69	0.45	0.66	**5**
M4	0.52	0.39	0.75	**2**
M5	0.24	0.36	1.46	**1**
M6	0.18	0.13	0.71	**4**

The following inflow analysis results are noted:

- Basin 5 had a higher inflow marker compared to the other flow monitoring sites.

Figure 6-6 shows a bar graph of the Peak I&I to ADWF Ratio. Figure 6-7 shows a temperature map of the Peak I&I to ADWF Ratio.

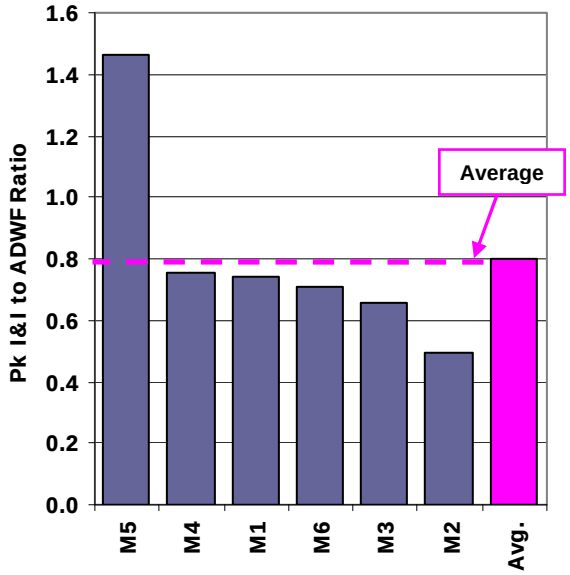


Figure 6-6. Inflow Markers by Site

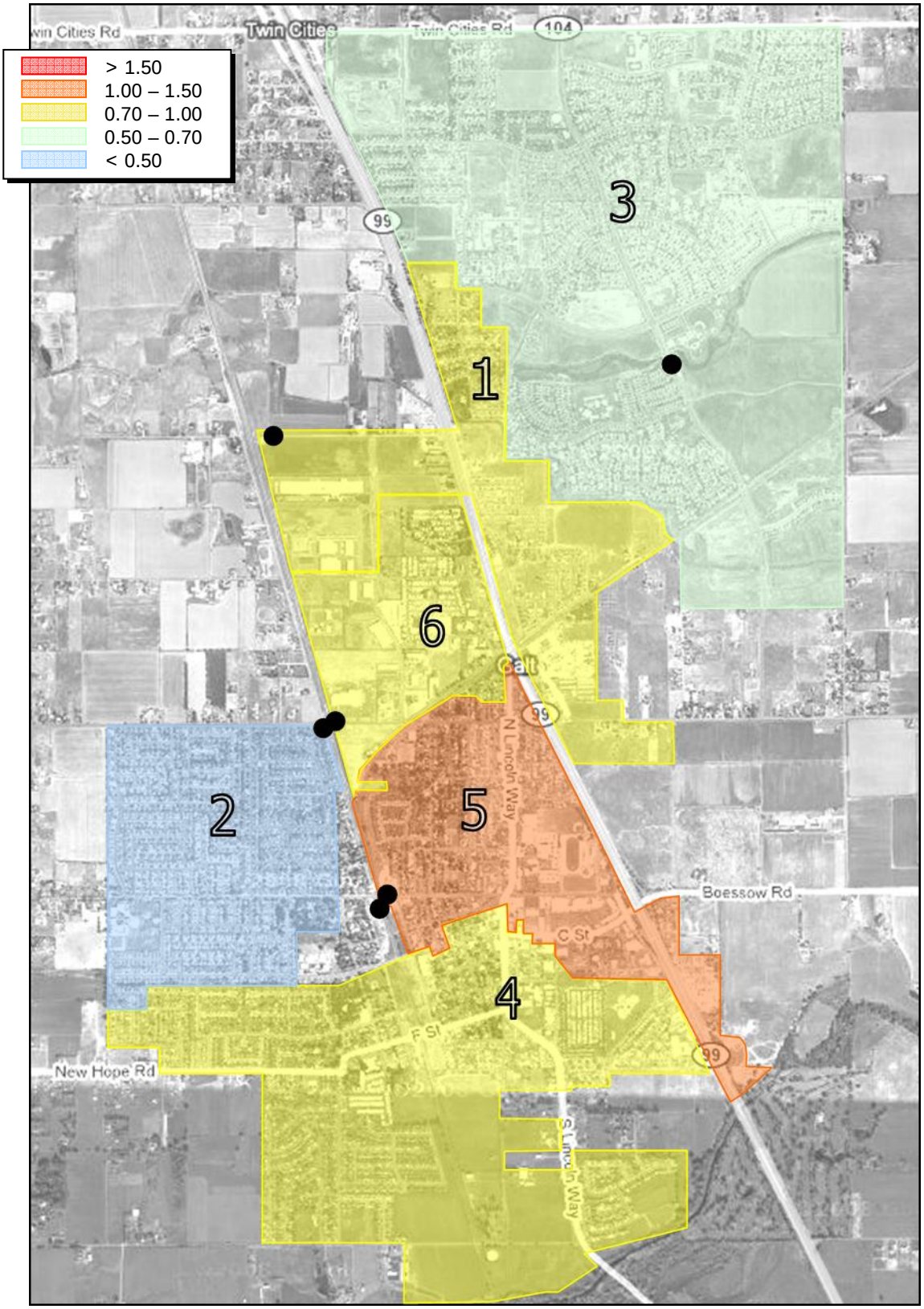


Figure 6-7. Temperature Map: Peak Inflow Rate to ADWF Ratio

6.4 GWI Analysis

Dry weather (baseline) flow can be expected to have a predictable diurnal flow pattern. While each site is unique, experience has shown that, given a reasonable volume of flow and typical loading conditions, the daily peaks and lows fall into a predictable range when compared to the daily average flow. If a site has a large percentage of groundwater infiltration occurring during the periods of dry weather flow measurement, the amplitudes of the peak and low flows will be dampened[4]. Figure 6-8 shows a sample of two flow monitoring sites, both with nearly the same average daily flow, but with considerably different peak and low flows. In this *sample* case, Site B1 may have a considerable volume of groundwater infiltration.

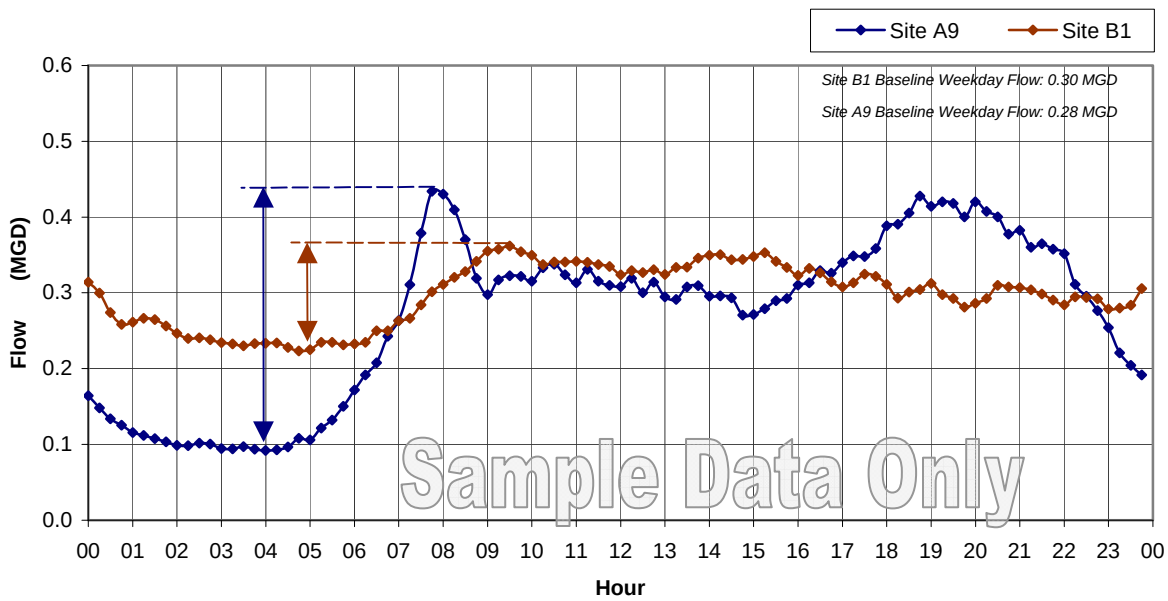


Figure 6-8. Groundwater Infiltration Sample Figure

It can be useful to compare the peak-to-baseline and low-to-baseline flow ratios for all flow metering sites. A site with abnormal ratios, and with no other reasons to suspect abnormal flow patterns, such as proximity to pump station, treatment facilities, etc., has a distinct possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system. Figure 6-9 plots the peak-to-baseline and minimum-to-baseline flow ratios against the baseline flows for all sites monitored during this study. The dotted line shows "typical" minimum-to-baseline flow ratios per the Water Pollution Control Federation[5]. There are no established peak-to-baseline ratios, but a system trendline has been drawn to better distinguish sites that fall outside the system trends. The minimum-to-baseline ratio should be given more credibility as low flows during early morning hours are generally more predictable than peak flows.

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

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

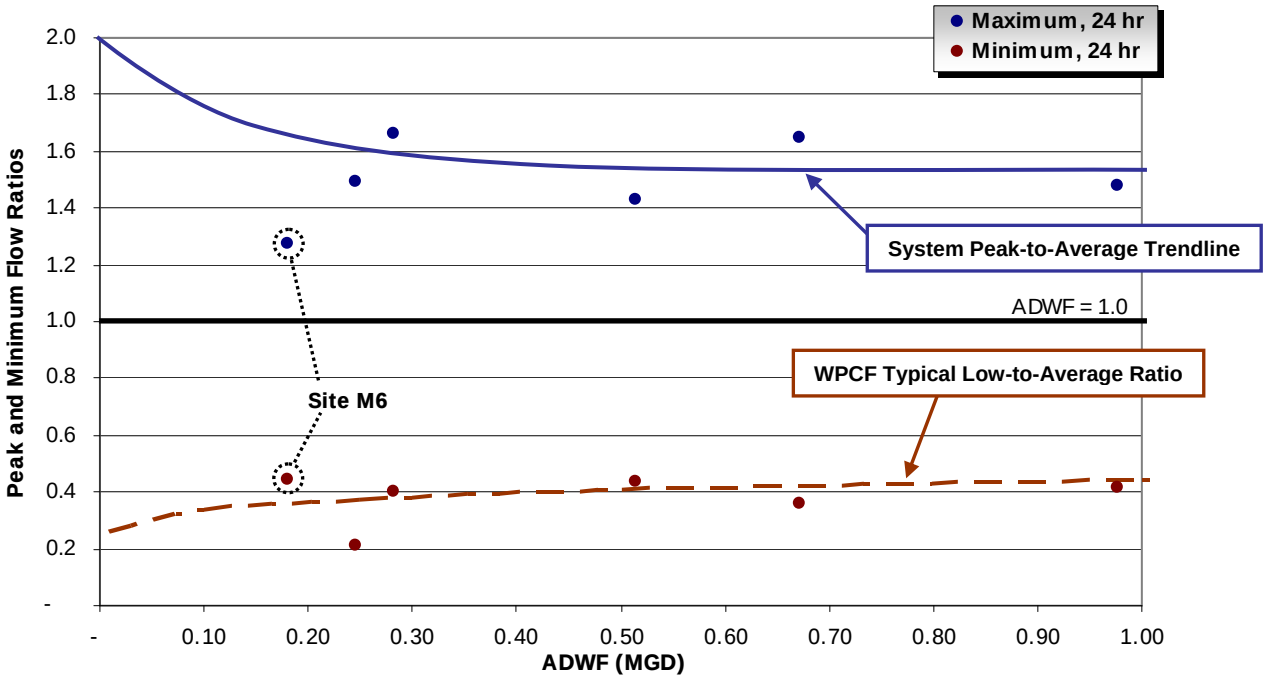


Figure 6-9. Peak and Minimum Flow Ratios vs. ADWF[6]

The rate of groundwater infiltration above typical groundwater infiltration standards (as set forth by WPCF) for Site M6 was calculated in Table 6-3. Please note: The stated groundwater rates are not **total** groundwater infiltration rates, but groundwater rates above typical rates.

Table 6-3
Excess Groundwater Infiltration Rates above Typical Rates

Site Name	Excess Groundwater Infiltration Rate (mgd)	Excess GWI Rate per Acre (gpad)
M6	0.025	134

The following groundwater infiltration analysis results are noted:

- Site M6 had minimum-to-baseline ratios that fell slightly outside of the typical minimum-to-baseline ratios as defined by WPCF. There may be higher-than-normal groundwater infiltration occurring in the basins upstream from this site during periods of dry weather flow.

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

6.5 Combined I&I Results Summary

Table 6-4 summarizes the combined I&I analysis results for the flow monitoring locations. The infiltration and inflow (I&I) results were taken from the December 24 – 25, 2008 wet weather event.

Table 6-4
Combined I&I Analysis Summary

Location	ADWF (mgd)	Combined Inflow/ Infiltration (gallons)	R-Value	R-Value Ranking
M1	0.99	301,000	0.85%	**2**
M2	0.29	39,000	0.47%	**6**
M3	0.69	155,000	0.79%	**3**
M4	0.52	108,000	0.53%	**5**
M5	0.24	83,000	0.67%	**4**
M6	0.18	45,000	1.01%	**1**

The following combined infiltration/inflow analysis results are noted:

- Basin 6 had the highest combined infiltration/inflow contributions relative to basin size.

Figure 6-10 shows a bar graph of the R-Value results. Figure 6-11 shows a temperature map of the R-Value results.

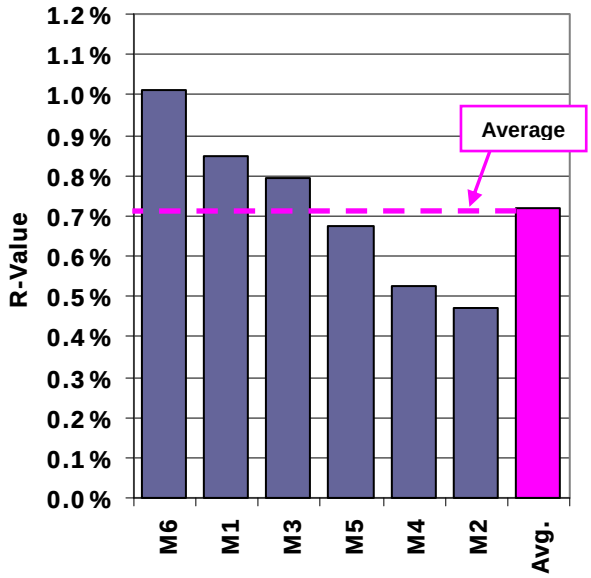


Figure 6-10. Bar Graph of R-Value by Site

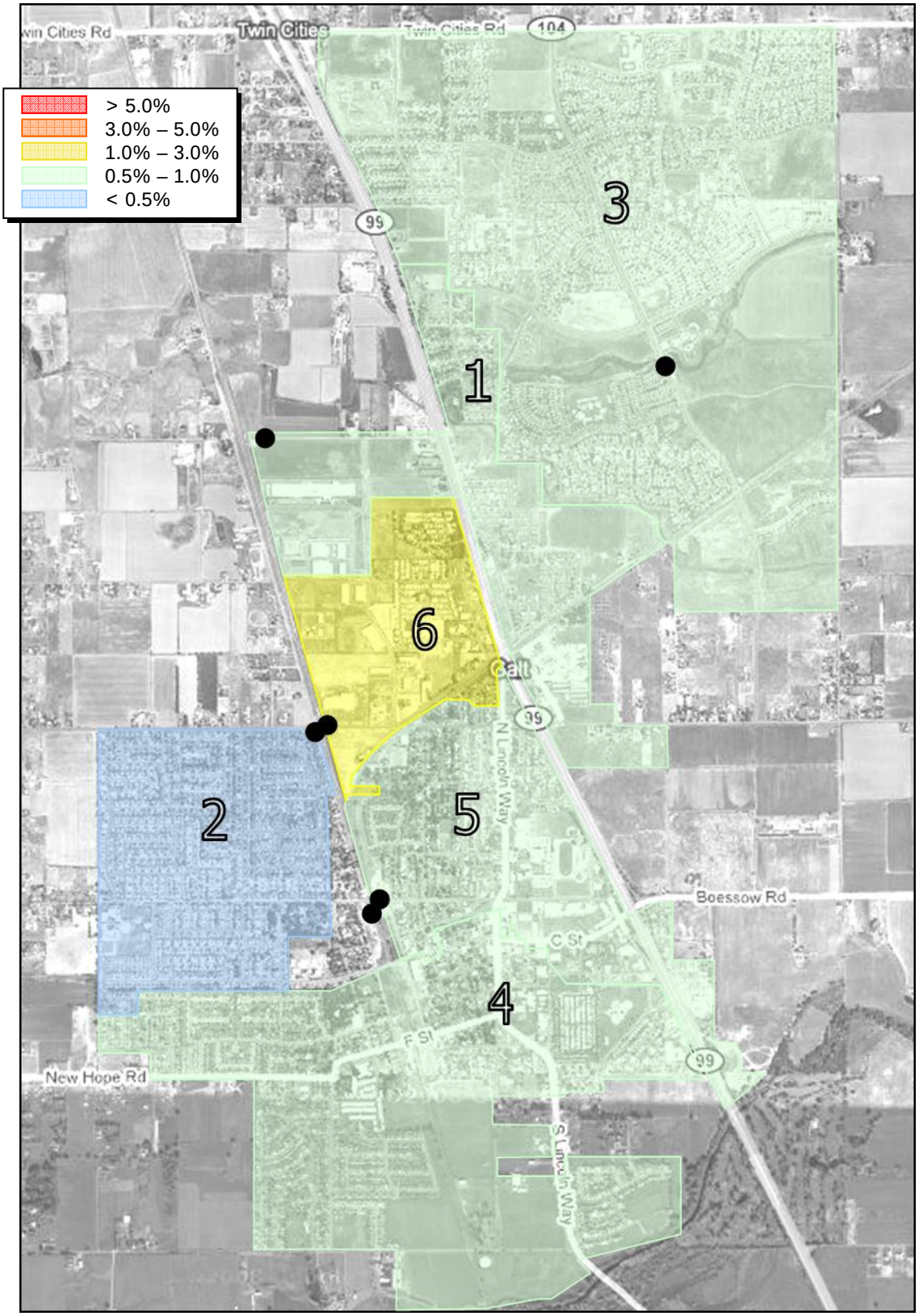


Figure 6-11. Temperature Map: R-Value

6.6 Pipeline Capacity Analysis

Peaking Factor: Peaking Factor is defined as the Peak Wet Weather Flow divided by the Average Dry Weather Flow. A peaking factor threshold value of 3.0 is commonly used for sanitary sewer design.

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

Table 6-5 summarizes the peak recorded d/D ratios and peaking factors over the course of the flow monitoring study period. Figure 6-12 shows a bar graph of the capacity results. Figure 6-13 shows the system flow schematic presented as a pipe snapshot of peak wet weather flows.

Table 6-5
Capacity Analysis Summary

Monitoring Site	Peaking Factor	d/D Ratio	Surcharged Level (in)
M1	1.89	0.45	n/a
M2[7]	2.05	0.56	n/a
M3	1.88	0.46	n/a
M4	2.01	**1.74**	**11.13**
M5	2.19	**3.44**	**19.50**
M6	1.99	0.42	n/a

The following capacity analysis results are noted:

- **Peaking Factor**
  - All sites had peaking factors below the typical design threshold limits for peak flow to average dry weather flow ratio during this study.
- **d/D Ratio**
  - Sites M4 and M5 had peak d/D Ratios greater than 1, indicating a surcharged condition. Sites M4 and M5 surcharged approximately 11.1 and 19.5 inches above the pipe crown, respectively. This is due to the downstream pump station operations.

[7] Site M2 had a peak flow event on December 30 at 2:45am, with a peak flow of 0.92 mgd and a peak level of 8.10 inches (resulting in PF = 3.21 and d/D = 0.81); however, this appeared to be an isolated peak flow event not related to rainfall or I&I contribution and was considered to be an anomaly in terms of this analysis.

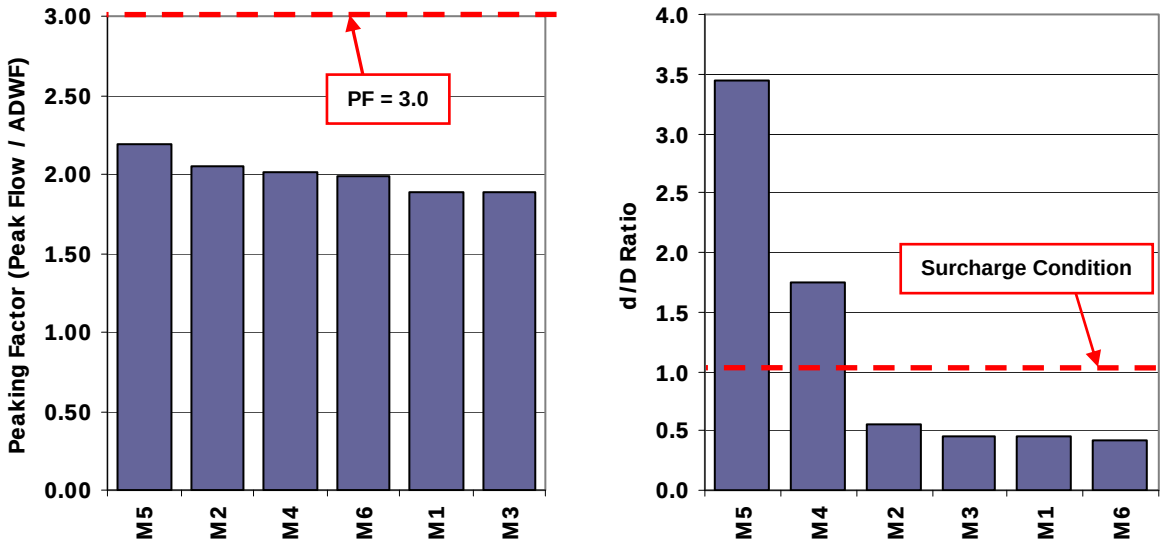


Figure 6-12. Capacity Rankings: Peaking Factor and d/D Ratio by Site

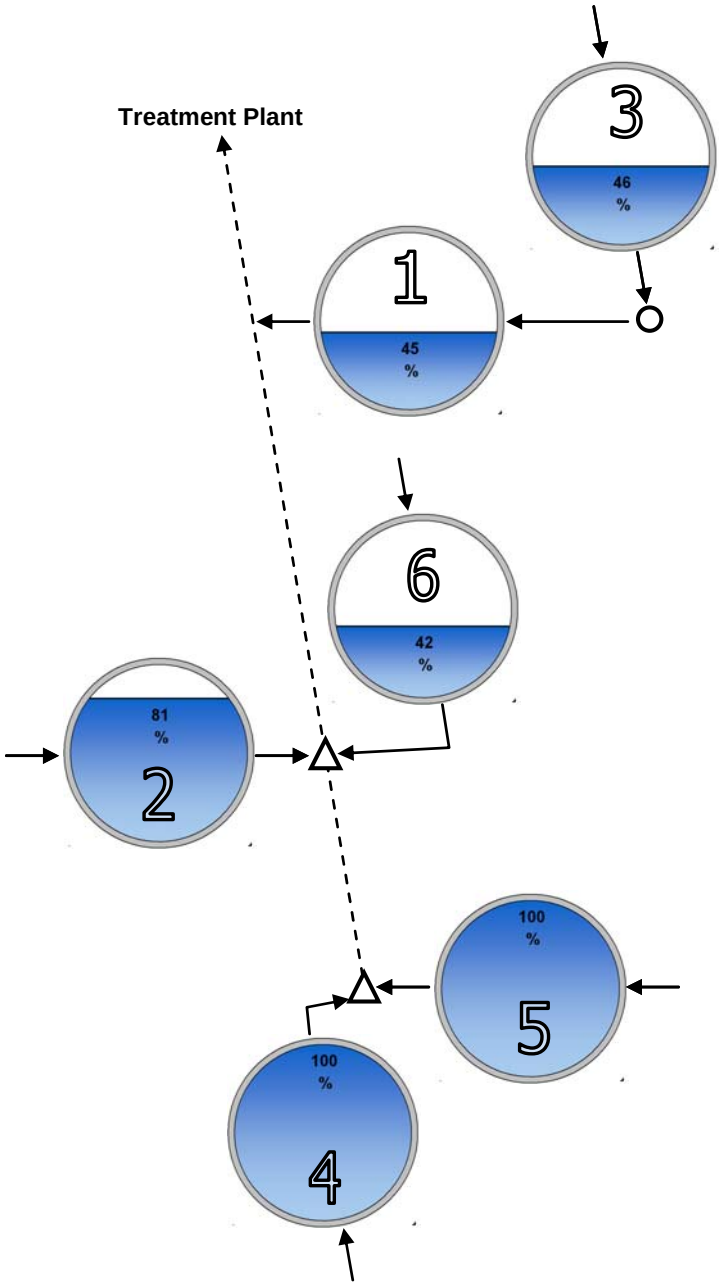


Figure 6-13. Peak Measured Flow Schematic

7 RECOMMENDATIONS

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

1. **Determine an I&I Reduction Program:** The City should examine its I&I reduction needs to determine a future I&I reduction program.
    a. If peak flows, sanitary sewer overflows and pipeline capacity issues are of greater concern, then the program can be weighted to investigate and reduce sources of inflow within the basins with the greatest inflow problems.
    b. If total infiltration and general pipeline deterioration is of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems.
2. **I&I Reduction Methods:** Potential I&I reduction methods include the following:
    a. Smoke testing
    b. Mini-basin flow monitoring
    c. Night-time reconnaissance work to (1) investigate and determine direct point sources of inflow, and (2) determine the areas and/or pipe reaches responsible for high levels of infiltration contribution.
    d. CCTV inspection
3. **I&I Reduction Cost Effective Analysis:** The City should conduct a study to determine which is more cost-effective: (1) locating the sources of infiltration and inflow and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional storm water I&I flow.
4. **Future Flow Monitoring:** The City should consider volumetric state logger flow monitoring at lift stations as a cost-effective way to conduct long-term flow monitoring. Besides tracking flows and flow responses to rainfall events, it would also increase understanding of pump station efficiency as well. A review of the number of pump starts per hour may assist the City in determining if a more efficient hydraulic operation would improve overall pump station life cycle costs. Note: There are twelve lift stations shown on Figure 2-2 (Page 7).

    Should the City decide to install permanent open channel flow meters at the sites monitored for this study, two metering types would be recommended. Sites M4 and M5 surcharge above the pipe crown due to the downstream lift station operation which backs up flow into incoming sewer lines. For this reason, an area-velocity flow monitor with submerged probe such as Teledyne Isco 2150 model should be used at Sites M4 and M5.

    A non-contact open channel flow monitor such as the Hach Marsh-McBirney Flo-Dar can be used at the remaining flow meter sites. The Flo-Dar typically requires less maintenance than submerged probe meters, saving on recurring maintenance costs.