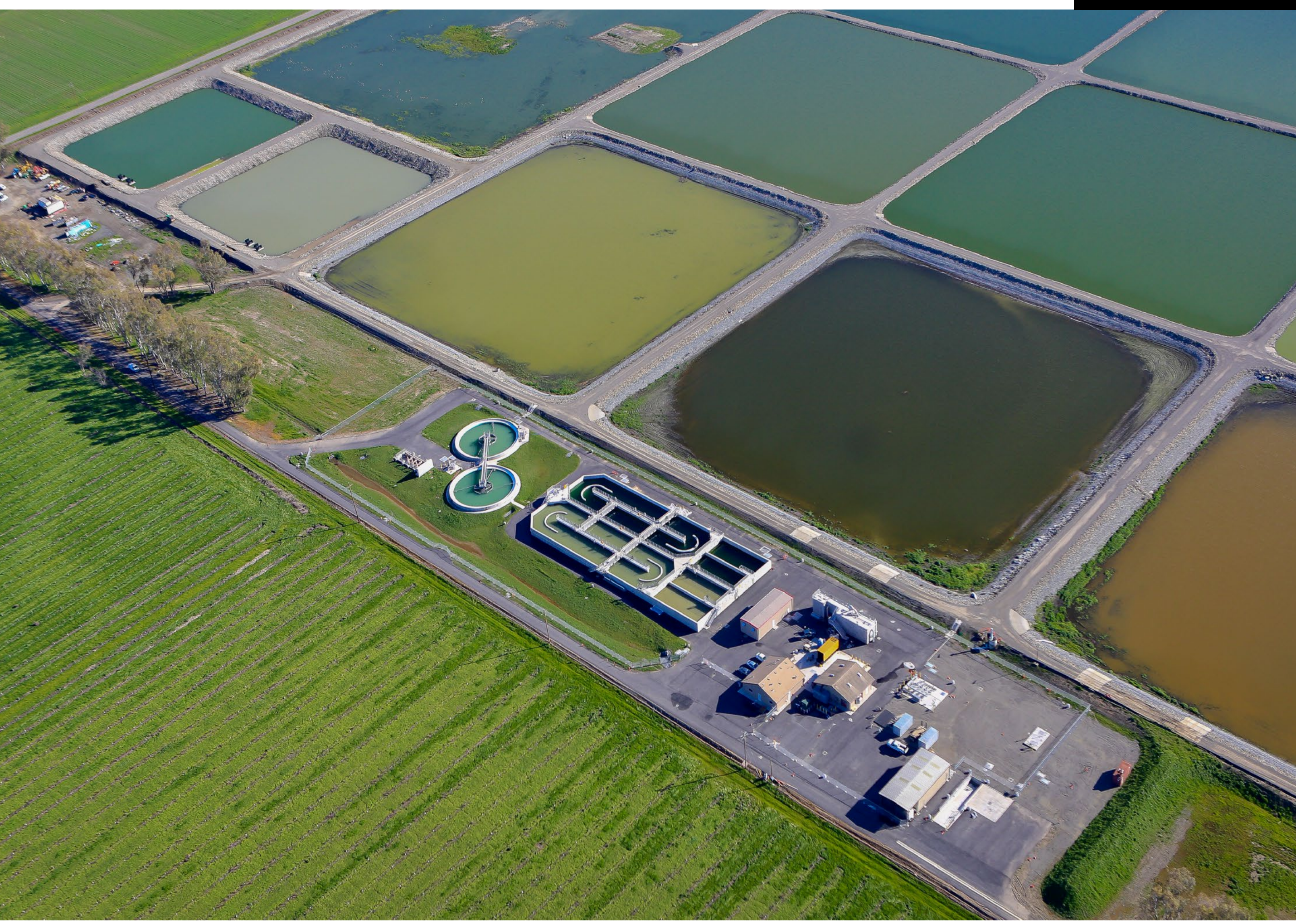
City of Dixon
Sewer Collection System Master Plan

March 2023



Prepared for:
City of Dixon

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Stantec

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Report Date

March 1, 2023

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Executive Summary

ES.1 Introduction

The purpose of this Sewer Collection System Master Plan (SCSMP) prepared for the City of Dixon (City), is to identify existing sewer collection system deficiencies and required collection system improvements based on wastewater flow projections and system condition assessment, and to formulate a comprehensive Capital Improvement Program (CIP) which meets the needs of the City's existing and future wastewater service area and users.

This SCSMP was completed based on information from the City's sewer collection system at the end of 2019 and early 2020. Flow monitoring data, collected at three locations within the sewer trunk network over February, March, and April of 2020, were used to calibrate the hydraulic model of the collection system developed as part of this SCSMP.

This effort was predominantly completed in February 2021. During the ensuing time period, initial project conditions changed including: 1) the decommissioning of the Pitt School Lift Station; 2) initial occupation of the Homestead Development; 3) completion of the East-West sewer trunk connection; and 4) completion of the General Plan 2040 adopted May 18, 2021. None of these updated conditions are material enough to change the recommendations of the study developed as of February 2021. The SCSMP contains recommendations for future efforts to update the system modeling with updated flow data that reflects these more recent conditions. Capital costs were updated to the 2022 time period.

The SCSMP provides the City with a comprehensive and prioritized road map for implementing improvements to address existing deficiencies, improve the reliability of the system, and planning for additional capacity to accommodate future growth. Specific objectives and tasks from the SCSMP project are listed below with references to specific chapters of the SCSMP, which provide additional details.

Table ES-1 SCSMP Chapter Tasks & Objectives

No.	Objective/Task	Chapter Title
1	Present SCSMP purpose and objectives.	Introduction
2	Describe and summarize the City's existing sewer service area and collection system facilities.	Existing Sewer System
3	Describe the City's existing and future service area characteristics and boundaries.	Planning Area Characteristics
4	Analyze flow data recorded within collection system and from the WWTF to determine typical flow patterns, distribution, and generation per capita.	Sanitary Sewer Flows
5	Review and refine the City's sewer collection system planning and design criteria for analyzing performance of the City's sewer collection system.	Planning & Design Criteria
6	Develop and calibrate the City's sewer collection system hydraulic model.	Hydraulic Model Development
7	Evaluate the ability of the City's sewer collection system to serve existing wastewater flows while meeting the City's planning and design criteria.	Existing Collection System Evaluation
8	Evaluate the ability of the City's sewer collection system to serve projected wastewater flows while meeting the City's planning and design criteria.	Future Collection System Evaluation
9	Develop a strategic asset management plan for the City's existing collection system facilities to provide guidance for the City's preventative maintenance and rehabilitation and replacement programs.	Condition Assessment & Asset Management
10	Develop a comprehensive Capital Improvement Program identifying the size and location of required improvements to address collection system deficiencies and future collection system needs.	Capital Improvement Program

The following sections provide a brief summary of key aspects of the SCSMP; details are provided within each of the individual chapters.

ES.2 Existing Sewer System

The City provides sanitary sewer service within its city limits and treats collected wastewater at its Wastewater Treatment Facility (WWTF). The City's existing collection system covers an area of approximately 2,500 acres and provides service to residential, industrial, and commercial users. The City of Dixon had a 2020 population of 19,972 according to the California Department of Finance (CDOF). The Association of Bay Area Governments (ABAG) projects that Dixon will grow by 30% between 2020 and 2040, an increase of approximately 5,000 to 6,000 residents.

The wastewater generated from these users is collected and conveyed to the WWTF by a network of sewer pipes, force mains, and pump stations. The collection system consists of approximately 75 miles of sanitary sewers (local sewers, trunk sewers, and force mains) and two lift stations, the Pitt School Lift Station (PSLS), and the Lincoln Street Lift Station (LSLS). At the beginning of the development of this SCSMP, the City's collection system had two wastewater lift stations in operation that provided service to approximately 20% of the sewer service area. The PSLS has since been abandoned. It was taken out of service with the construction of the East West Sewer Trunk Connector. The reliable pumping capacity is the capacity of the lift station with the largest pump out of service. The PSLS had a reliable pumping capacity of approximately 0.5 MGD. The LSLS has a reliable pumping capacity of approximately 0.8 MGD.

The City's WWTF has been in operation since 1952, but recently underwent a significant improvement project to comply with regulatory requirements and assumed to provide capacity for current and entitled projects within the City through 2040. The WWTF is permitted by the Central Valley Regional Water Quality Control Board (CVRWQCB) under WDR Order No. R5-2014-0098. On July 9, 2015 the City of Dixon celebrated the groundbreaking of its Wastewater Treatment Facility Construction Project (WWTF Improvements Project). The WWTF Improvements Project improvements included the construction of new secondary treatment facilities and abandoning thirteen wastewater treatment ponds. The WWTF Improvements Project was completed in 2017 to bring it into compliance with the permit issued by the Central Valley Regional Water Quality Control Board.

Figure ES-1 shows the City's existing sanitary sewer collection system network including pump station and force-mains, in addition to the future planning areas described in the following section.

ES.3 Planning Area Characteristics

The purpose of this chapter is to describe the City's existing sewer service area and present relevant planning data and information used to project wastewater flow and distribution from future development areas within the planning horizon of the City's General Plan 2040.

The City's existing wastewater collection system planning has primarily been driven by the on-going developments within City's planning area. Specific plans outlining the City's existing sewer planning are included in **Appendix B**. This information was reviewed and incorporated into this SCSMP.

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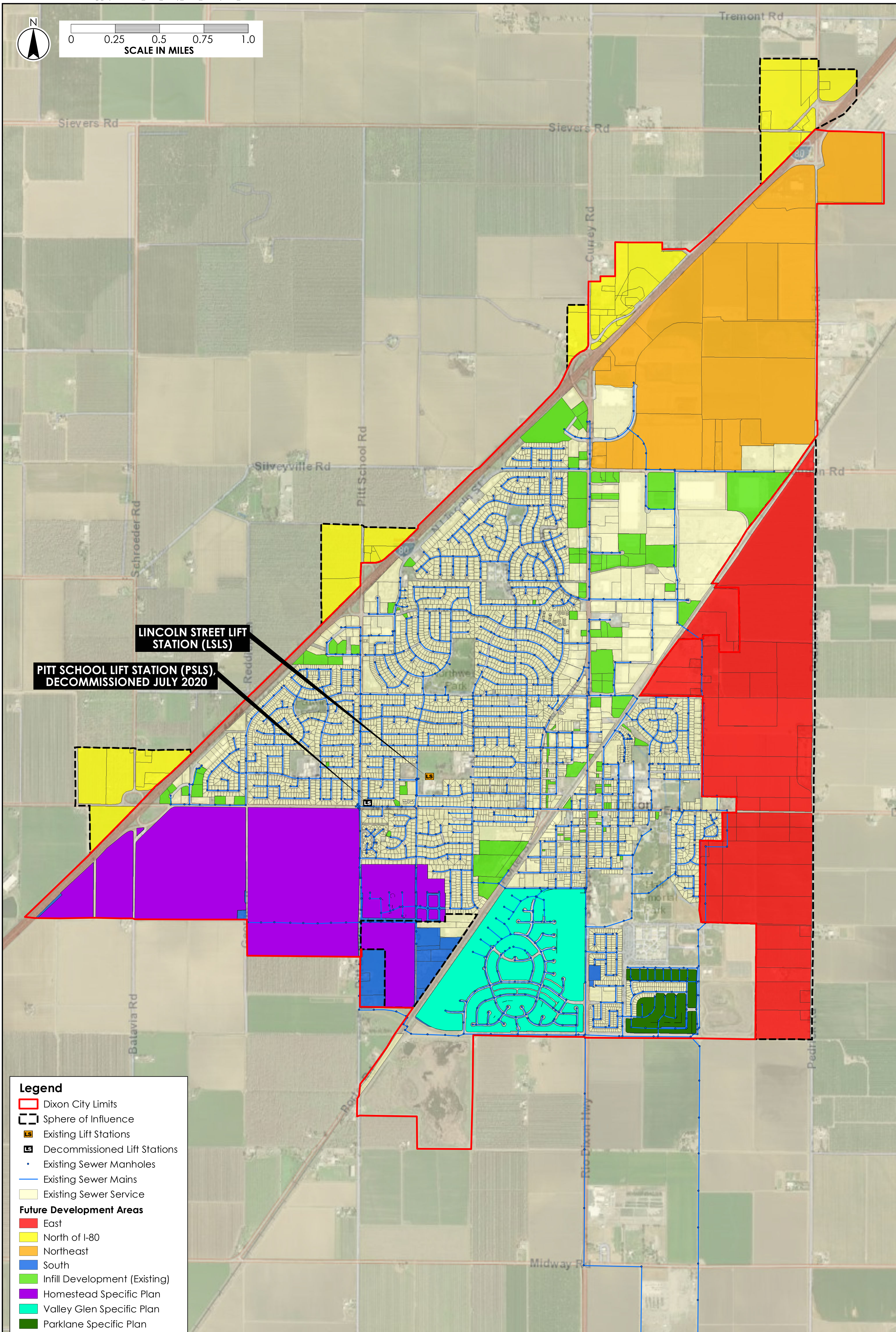

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City of Dixon
Sewer Collection System
Master Plan

Figure ES-1
City of Dixon Future Development Areas

The General Plan 2040 outlines the City's goals for future development, circulation, conservation of resources, and utilizes policies and actions necessary to achieve these goals. The General Plan 2040 is a collaborative community effort to create a vision and a blueprint for development through 2040. The GIS shapefile associated with the General Plan 2040 was provided by the City. The land uses within the shapefile serve as the basis for establishing future wastewater flow estimates associated with future development areas within city limits and the SOI.

Vacant parcels within the existing extent of the sewer service area were identified from information presented in the City's Water Master Plan (West Yost, 2018). Future development or occupancy of vacant parcels within the existing service area is referred to herein as infill, or infill development. This SCSMP assumes that the vacant or infill development parcels are assigned the right to available capacity before future upstream development flows are considered. There are approximately 193 acres of vacant developable or unoccupied land within the existing service area.

Wastewater flow estimates for future development areas were projected using the General Plan 2040 information and the City's wastewater generation rates. To provide consistency with existing City planning documents, this SCSMP divides the City into the seven development areas shown in **Table ES-2**. The correlation between the seven development areas outlined in the General Plan 2040 is also shown in the table. A map depicting the existing sewer collection system and the future development areas used in this SCSMP is presented as **Figure ES-1**.

The Existing, Downtown, and SR-113 Corridor development areas are collectively considered infill development. Specific plans were provided for the Homestead, Valley Glen, and Parklane development areas. Specific Plan areas are referred to as on-going development areas, all of which exist in the "South" development area.

Table ES-2 City of Dixon Development Areas

General Plan Development Areas	SCSMP Development Area	Specific Plan
Existing	Existing Service Area/Infill	
Downtown	Existing Service Area/Infill	
SR-113 Corridor	Existing Service Area/Infill	
South	South/On-going Development	Southwest Dixon (Homestead Development, Valley Glen, & Parklane)
Northeast	Northeast	Northeast Quadrant (1)
East	East	
North of I-80	North of I-80	

1) General Plan 2040 land use data (2019) was used to project flow from the Northeast Quadrant as opposed to information presented in its Specific Plan, which was adopted in 1995.

This SCSMP assesses collection system performance under existing development conditions and the following three future growth scenarios. The future development scenarios are cumulative and represent near-term, long-term, and build-out level of development planning. These scenarios and the associated future development areas added to each scenario are summarized in **Table ES-3**.

Table ES-3 SCSMP Level of Development Scenarios

Level of Development	Model Description	Development Areas Added
Existing	Scenarios 1 – 3	• Existing Services
Near-Term	Scenario 4	• Service Area Infill • On-going/South
Long-Term	Scenario 5	• Northeast • North of I-80 (portion)
Build-Out	Scenario 6	• East • North of I-80 (remaining)

ES.4 Sanitary Sewer Flows

Chapter 4.0 assesses the wastewater flow within the existing collection system under current development conditions in order to establish current capacity and as a basis for projecting flows under future conditions. Estimates of future wastewater flow developed for future planning areas described in the City's General Plan 2040, are developed using land use information and wastewater generation rates in **Chapter 5.0**. The projected flows are added to the existing collection system model to evaluate the capacity of the system under existing, near-term, long-term, and build-out levels of development.

Existing Wastewater Flows

Wastewater collection systems are designed to convey peak wet weather flow (PWWF), which is characterized by three elements: Base sanitary flow, groundwater infiltration (GWI), and rainfall dependent inflow and infiltration (RDII). The average dry weather flow (ADWF) or base flow is the component of PWWF generated directly through sewer system users. GWI and RDII are components associated with inflow and infiltration (I/I) of rainfall into the collection system.

Influent flow at the City's WWTF recorded an ADWF of 1.13 MGD, an average annual flow (AAF) 1.14 MGD, and a maximum monthly flow (MMF) of 1.19 MGD in 2019. Historically, influent flow recorded at the WWTF has been declining due to a number of factors. Which may include the City's successful I/I reduction efforts in 2006, water conservation efforts in response to the period of drought between 2011 to 2015, and declining groundwater levels.

The City has significantly reduced the volume of I/I entering the collection system since 2002. This was primarily achieved through improvement projects to reduce I/I in the 42-inch Main Trunk and by isolating the 27-inch Main Trunk from the upstream collection system. These efforts reduced the maximum daily flow recorded at the WWTF by approximately 1.0 MGD.

A temporary flow monitoring study was conducted to provide baseline sanitary sewer flow data from specific locations within the collection system to allow calibration of the hydraulic model. This work was conducted by V&A Consulting Engineers (V&A) and summarized in a technical report dated November 2020 included in **Appendix A**. Flow was monitored at three locations within the City's trunk sewer network over the period of February 7 to April 22, 2020. The flow monitoring sites were used to determine flow contributed from the upstream sewer-shed area. To distribute flow within the model, flows recorded at flow monitors in series are estimated by subtracting flow from upstream sewer-sheds. The location of these flow monitors and associated sewer-sheds are described in **Table ES-4**.

Table ES-4 Flow Monitoring Shed Characteristics

Flow Monitor	MH ID/ Location	Pipe Size (in)	Area (Acres)	Sewer-shed Description
1	SS0098N Yale Dr., 100 feet north of Parkway Blvd.	30	427	Flow collected from the North Dixon Trunk, North Industrial Area, and Connemara Subdivision.
2	SS1104 Parkway Blvd., 900 feet west of Yale Dr.	27	305	Flow collected from upstream of the Parklane Trunk (downstream of West Cherry St.) including Valley Glen, Collier Manner Subdivision, and flow conveyed through the North Interceptor Sewer.
3	SS1081 South 1st St., 525 feet south of W. Cherry St.	15	713	Flow collected from the PSLS and LSLS sewer-sheds, the majority of the "old town" portion of the City, and residential flows from the Hillview Drive Area.

The relatively low amount of rainfall experienced during the flow monitoring period corresponded to a limited wet-weather flow response within the collection system. Without a sufficient wet-weather response within the flow monitoring data set, wet-weather model calibration was performed using historically observed influent flow data recorded at the WWTF. Despite the limited response, V&A provided an analysis of I/I within the collection system which is summarized in **Table ES-5**. Additional information is provided in V&A's full Technical Report included in **Appendix A**.

Table ES-5 V&A Flow Monitoring and I/I Analysis Summary

Flow Monitoring Site	V&A's ADWF (1) (MGD)	Peak Measured Flow (MGD)	Peaking Factor	Max Depth/ Diameter Ratio	Total I/I per ADWF (2) (MG/MGD) *1-inch of Rainfall*	I/I Ranking (3)
1	0.273	0.57	2.1	0.36	0.055	2
2	0.200	1.56	2.0	0.24	0.162	1
3	0.577	2.07	3.6	0.68	0.048	3
Total:	**1.050**					

1) ADWF values are representative of ADWF conditions prior to the SIP order. ADWF at Site 2 has been adjusted to remove flow recorded at upstream Site 3, the total ADWF measured at Site 2 was 0.777 MGD.
2) This value represents the total volume of I/I that enters the collection system per 1 MGD of ADWF corresponding to 1-inch of rainfall over the sewer-shed.
3) A ranking of "1" represents the most observed I/I after normalization.

The following impacts to data quality should be considered with the results of the 2020 flow monitoring study and model evaluations presented in this SCSMP. Additional details are presented in **Chapter 4.0**.

- Less than ideal flow conditions during the flow monitoring period – debris blockage
- Changes in daily flow pattern during the flow monitoring period – shelter-in-place order
- Insignificant wet-weather flow response during the flow monitoring period – Lower than average rainfall
- Major system routing changes that have occurred since the flow monitoring period – East West Trunk Sewer/ PSLS decommissioned

The objectives of collecting additional flow monitoring data and *associated details* are summarized below:

- Validate flow data collected in the 2020 V&A study:
  Adjust locations slightly to measure flow under more ideal flow conditions.
- Validate the typical daily flow pattern used in the model, and confirm that this pattern is representative of typical post-pandemic daily flow pattern,
  Measure the daily pattern over a period that is representative of typical conditions and can be used for long-range planning.
- Determine the local peak wet-weather flow response within each sewer-shed and overall distribution of I/I within the collection system,
  Measure flow over the typical wet-weather season in hopes of collecting data over more significant rainfall events. Monitor groundwater in addition to flow under high groundwater conditions.
- Validate system routing and flow distribution related to connecting the East West Trunk Sewer and decommissioning the PSLS,
  Measure flow at key locations considering major system flow routing changes.

It is recommended that the City perform flow monitoring twice annually to capture dry weather and wet weather flow data.

Flow Per Capita Analysis

Historical flow and population data were used to calculate the typical wastewater flow per capita in the City of Dixon. The analysis uses the annual ADWF recorded at the WWTF and historical population data to calculate the annual average per capita wastewater flow. The historical per capita flow, or generation rate was compared to the standard design values presented in the City's Design Standards. The results of this analysis were used as a basis for identifying appropriate wastewater generation rates to project flow generated from future development areas.

The evaluation of historical data from 2005 to 2019 resulted in an average unit rate of 250 gpd/EDU. Overall, the historical trend in per capita flow within the City has been declining over the past 15-years. The City's average per capita flow calculated from 2005 to 2019 is approximately 68 gpcd. Multiplying the average per capita flow by the City's standard planning density of 3.7 persons/EDU, gives the wastewater unit rate of approximately 250 gpd/EDU. This wastewater unit rate is 100 gpd/EDU less than what is currently in the City's Design Standards. However, it appears to be a reasonable estimate of wastewater flow from similar sized communities in this region. (City of Woodlake at 232 gpd/EDU, City of Lincoln at 250 gpd/EDU, City of Benicia at 225 gpd/EDU, City of Merced at 257 gpd/EDU, City of Lodi at 211 gpd/EDU)

As a result of the analysis the wastewater generation rates were reduced by approximately 30% from those presented in the City's Design Standards (250/350 ~ 70%). These wastewater generation rates are used as a basis for projecting flow from future development areas in this SCSMP to reflect historically observed conditions.

Projected Wastewater Flow

As described in **Chapter 5.0** of this SCSMP, future wastewater flows were projected through build-out of the City's sewer service area using a unit demand methodology based on land uses incrementally for each development scenario presented in the General Plan 2040 and remaining EDU counts within specific plan areas. The City's standard wastewater generation rates were adjusted based on the per-capita flow analysis presented in **Chapter 4.0**. **Table ES-6** provides a summary of the existing and projected wastewater flow in each development scenario.

Table ES-6 SCSMP Development Scenarios – ADWF Projection

General Plan 2040 Land Use Designation	Generation Rate (gpd/acre)	Development Scenario ADWF Projection (MGD)				
		Existing	Near-Term	Long-Term	Build-Out	Total
Existing Service Area						
Existing Sewer-sheds	-	1.09				1.09
Open Space						
Agricultural	100		0.00	0.00	0.00	0.00
Parks	0		0.02	0.00	0.00	0.02
Mixed Use						
Campus Mixed Use	3,000		0.05	0.63	0.00	0.68
Corridor Mixed Use	3,000		0.23	0.00	0.00	0.23
Downtown Mixed Use	3,000		0.01	0.00	0.00	0.01
Residential						
Low Density Residential	800		0.35	0.00	0.39	0.75
Medium Density Residential	3,600		0.23	0.00	0.00	0.23
High Density Residential	5,000		0.00	0.00	0.00	0.00
Commercial						
Neighborhood Commercial	1,100		0.00	0.00	0.00	0.00
Regional Commercial	1,100		0.01	0.13	0.18	0.32
Service Commercial	1,100		0.01	0.00	0.00	0.01
Industrial						
Industrial	1,400		0.07	0.29	0.00	0.37
Public						
Public Facilities	0		0.00	0.00	0.00	0.00
Total ADWF:		**1.09**	**0.98**	**1.06**	**0.57**	**3.70**

ES.5 Existing Collection System Evaluation & Findings

The existing collection system was evaluated using the recommended planning and design criteria presented in **Chapter 5.0** along with the newly developed collection system hydraulic model described in **Chapter 6.0**. **Chapter 7.0** presents the existing collection system evaluation and explains each of the recommended existing system improvements.

The existing system capacity evaluation includes an analysis of existing trunk sewer flow and pumping capacity. The upstream and downstream full pipe flow capacity of the City's primary trunk sewers is summarized in **Table ES-7**.

The hydraulic performance evaluation assesses the existing collection system's ability to meet recommended performance standards under current ADWF and PWWF conditions. The capacity evaluation of the City's only operating pump station was found to be sufficient under current and future conditions. It has been identified as an asset that is near the end of its useful life and is already identified by the City as an existing CIP project.

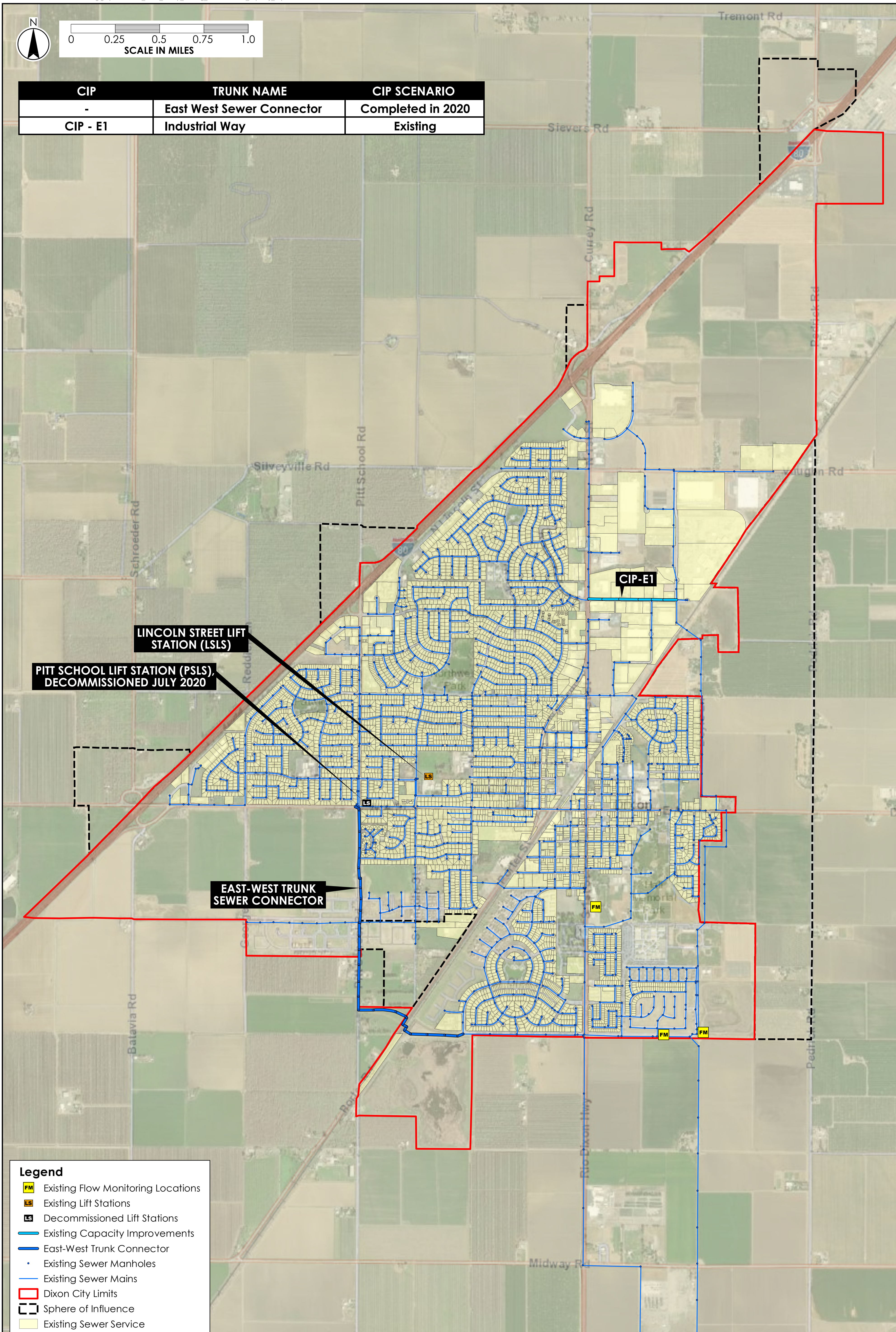
The only capacity constraint in the existing system was identified in the Industrial Way Trunk (CIP-E1) sewer. This 10-inch sewer is undersized based on modeled flows and the presence of larger upstream sewers. Surcharging under existing conditions is predicted to be less than 1-foot above the pipe crown and the available freeboard (depth between the rim elevation and the pipe crown) is greater than 15-feet along the length of the trunkline, reducing potential for a sewer system overflow (SSO). It is recommended that the City address this capacity constraint before adding additional flow or allowing new development to occur in the upstream service area.

Chapter 7.0 also presents an evaluation of the impacts of the City's recent East West Sewer Connector project on system capacity and flow routing identified as Scenario 3. Under Scenario 3, it was found that that the addition of the East West Trunk Sewer Connector diverted approximately 0.5 MGD of PWWF from trunk sewers in Sewer-Shed 3 by directly routing flow to the Parkway Blvd trunk. These trunk sewers were nearing their full pipe capacity in Scenario 2 and the connection of the new trunk provided an additional 0.5 MGD of residual or available capacity for infill development within the sewer-shed.

The existing trunk sewer network, the capacity constraint in the Industrial Way Trunk (CIP-E1), and the East West Sewer Connector are shown in **Figure ES-2**.

Table ES-7 Existing Collection System Trunk Sewer Capacity

Primary Trunk Sewer	Pipe Size (in)	Length (LF)	Sewer-shed	Capacity (MGD) Upstream	Capacity (MGD) Downstream
North Dixon Trunk	27 to 30	11,800	1	5.0	6.1
Fitzgerald Dr.	21	3,000	1	2.4	3.1
Vaughn Rd/Dorset Drive	10 to 12	3,000	1	0.7	1.2
Vaughn Rd - East	15 to 18	2,300	1	1.5	2.5
Industrial Way Trunk	10	2,100	1	0.4	0.5
North 1st Street - Industrial	10 to 12	4,200	1	0.6	0.9
Connemara Trunk	10	3,300	1	0.6	0.8
Parkway Blvd	27	5,300	2	6.4	12.2
South 1st Street - Part 1	15 to 27	3,400	2/3	2.2	6.0
North Interceptor Sewer	15 to 27	10,000	2	1.2	5.5
Collier Manor Trunk	10 to 12	3,800	2	0.5	1.2
East-West Trunk Connector	15 to 27	7,800	2	1.8	6.0
E-W Branch 1	18	2,600	2	2.2	2.2
E-W Branch 2	10	3,100	2	0.7	0.7
PSLS/Rehrmann Dr	10 to 15	3,300	2/3	0.8	1.6
Pheasant Run Dr.	10	1,900	2/3	0.7	0.8
Manning Way	10	700	2/3	0.8	0.8
South 1st St - Part 2	14	1,800	3	1.5	1.6
North 1st Street	10	2,300	3	0.6	0.9
East A Street	12	1,500	3	0.6	1.3
Cherry St./Porter Road Crossing	15	2,700	3	1.6	1.9
South Almond/Hillview Dr.	15	2,000	3	1.2	1.6
South Lincoln/West A St	15	2,500	3	1.3	1.8
North Lincoln Street	10	3,100	3	0.7	1.1
Pitt School Rd	12	3,200	3	0.8	0.8
27-inch Main Trunk	27	15,200	Main	5.6	5.8
42-inch Main Trunk	42	12,700	Main	15.2	15.6



CIP	TRUNK NAME	CIP SCENARIO
-	East West Sewer Connector	Completed in 2020
CIP - E1	Industrial Way	Existing

Figure ES-2
Existing Collection System – Recommended Capacity Improvements

ES.6 Future Collection System Evaluation and Findings

The City's future collection system was evaluated based on near-term, long-term, and build-out development conditions using the collection system planning and design criteria presented in **Chapter 5.0**, along with the newly developed collection system hydraulic model described in **Chapter 6.0**. **Chapter 8.0** presents the future sewer collection system evaluation and explains each of the recommended future collection system improvements. The future system capacity evaluation includes an analysis of facility capacity needs and hydraulic performance.

The system capacity evaluation assesses the sewer flow and pumping capacity needs under future development conditions to provide preliminary sizing and recommendations of new infrastructure needed to expand the service area.

The hydraulic performance evaluation assesses the collection system's ability to meet the recommended LOS performance standards under future PWWF conditions, to identify capacity limitations within the existing system and recommend improvements.

This SCSMP evaluated three future scenarios including near-term, long-term, and build-out levels of development. A summary of the assumptions and planning parameters associated with each level of development is provided in **Table ES-8**.

Table ES-8 Summary of Future Level of Development Scenarios

Level of Development	Model Scenario	Development Areas Added	Modeled Flow (MGD)	Assumptions
Existing	Scenarios 1-3	• Existing Services	ADWF: 1.09 PWWF: 4.30	Current state of development and sewer service area.
Near-Term	Scenario 4	• Service Area Infill • On-going/South	ADWF: 2.07 PWWF: 7.53	Full development of the existing service area and on-going development areas
Long-Term	Scenario 5	• Northeast • North of I-80 (portion)	ADWF: 3.13 PWWF: 10.34	Full build-out of the City's existing city limits boundary, and on-going development areas, including the Northeast area and the adjacent portion of the SOI west of I-80.
Build-Out	Scenario 6	• East • North of I-80 (remaining)	ADWF: 3.70 PWWF: 12.69	Full buildout development of the City's SOI boundary.

Future System Facility Capacity Evaluation

The future system capacity evaluation recommendations of new infrastructure required to adequately convey flows from the future development areas are included in each of the following scenarios.

Near-Term

The only new system trunk needed to expand the existing service area to serve infill and on-going development areas is to extend an existing branch of the City's new E-W Trunk Connector further west to serve the remaining portion of the Homestead Development area. The 18-inch E-W Branch 1 trunk was extended west to Batavia Road where the new trunk turns north until it meets I-80. The new trunk sewer, referred to as the E-W Sewer Extension, ranges in diameter from 15-inch at its downstream point of connection to the existing system, to 10-inches in diameter at its upstream end where it meets I-80. The capacity of the proposed trunk extension ranges from 1.84 MGD at its downstream end to 0.76 MGD at its upstream end.

Long-Term

The long-term development scenario extends the existing collection system to the remaining undeveloped portion of city limits. The proposed collection system improvements extend from the existing trunk sewers in Sewer-Shed 1 to reach parcels within the Northeast development area and the portion of the north of I-80 development area that lies within the current extent of city limits. There are three primary regions of the expanded service area that will require new collection system infrastructure:

- North of I-80
- Gravity Service (Southern Northeast Quad)
- Lift Station Service (Northern Northeast Quad)

The three regions are proposed to connect to the existing system at two points of connection. The Northeast Quad will connect to the upstream end of the Fitzgerald trunk and the area North of I-80 will tie into the existing system at the upstream end of the Vaughn Rd/Dorset Drive trunk.

The proposed trunk serving the area north of I-80 extends the Vaughn Road/Dorset Drive trunk to cross I-80. The new Milk Farm Rd. – I-80 Crossing trunk will need to be 10-inches at its downstream end and 8-inches at the upstream end, with a capacity of 0.45 MGD upstream and 0.78 MGD downstream.

The Northeast Quad is divided by the portion that can be served by gravity sewers and the portion requiring a lift station. The main trunk running through the gravity service area is referred to as the Main NE Quad trunk and ranges from 21-inches to 12-inches in diameter, requiring a capacity of 1.0 MGD at the upstream end and 3.3 MGD at the downstream end. It connects to the existing system at the intersection of Vaughn Road and Fitzgerald Drive and serves the southern portion of the Northeast Quad.

There are two main branches proposed to extend from the Main NE Quad trunk to the east. There are two main trunk branches proposed to extend from the Main NE Quad trunk to the east. The southern branch, Branch 1 ranges from 12 to 10 inches in diameter, with two 8-inch collector sewers. The northern branch, Branch 2 extends from the NE Quad Trunk connecting at its transition from 18 to 12-inches in diameter.

The proposed lift station will serve approximately 115 acres of the Northeast Quad under long-term development conditions. Under full buildout conditions, it is proposed to also serve the area north of I-80 for full buildout service area of approximately 195 acres. The reliable capacity of the lift station under near-term development conditions is approximately 0.45 MGD, to be expanded to 0.65 MGD at buildout. The proposed lift station discharges at the upstream end of the Main NE Quad trunk. It is recommended that the City install dual 4-inch force mains to accommodate the phasing of development in the area.

Build-out

The proposed build-out improvements are needed to expand the service area to include the remaining City planning area, including the remaining areas North of I-80 and the East development area. The proposed improvements include three I-80 crossings and the East Area Main trunk.

The E-W Trunk I-80 crossing and the North Lincoln St. I-80 crossing are proposed to be 8-inch sewers at the minimum slope required to ensure a minimum full pipe flow velocity 2.5 fps. The Sparling Lane I-80 crossing extends from the proposed North East Quad lift station service area. It is proposed to be a 10-inch sewer at the downstream end and reduce to an 8-inch sewer at the upstream end after crossing I-80. The expansion of the lift station service area will also require that the reliable pumping capacity of the lift station be expanded from 0.45 MGD to 0.65 MGD.

A summary of the recommended future sewer collection system is shown in **Figure ES-3** and summarized in **Table ES-9**.

Table ES-9 Summary of Proposed New Infrastructure

CIP ID	Scenario	Name	Diameter (in)	Length (LF)	Downstream Capacity (MGD)	Upstream Capacity (MGD)
N1	Near-term	E-W Sewer Extension	10 to 15	4,590	1.84	0.76
N2	Long-term	Milk Farm Rd., I-80 Crossing	8 to 10	3,010	0.78	0.45
N3	Long-term	Main NE Quad Trunk	12 to 21	5,220	3.30	1.00
N4	Long-term	Main NE Quad Trunk - Branch 1	8 to 12	3,730	1.03	0.57
N5	Long-term	Main NE Quad Trunk - Branch 2	8 to 12	2,130	1.03	0.57
N6a	Long-term	NE Quad LS Sewer-shed	8 to 10	1,850	1.03	0.57
N6b	Long-term	NE Quad LS (0.65 MGD) (1)	Dual 4-in	3,140	0.71	0.57
N7	Build-out	E-W Trunk, I-80 Crossing	8	1,980	0.57	0.57
N8	Build-out	N Lincoln St., I-80 Crossing	8	2,100	0.57	0.57
N9	Build-out	Sparling Ln., I-80 Crossing	8 to 10	2,410	0.71	0.50
N10	Build-out	East Area Main Trunk	8 to 21	14,720	3.25	0.57

1) Reliable pump station capacity of 0.65 MGD at build-out and 0.45 MGD under long-term development conditions.

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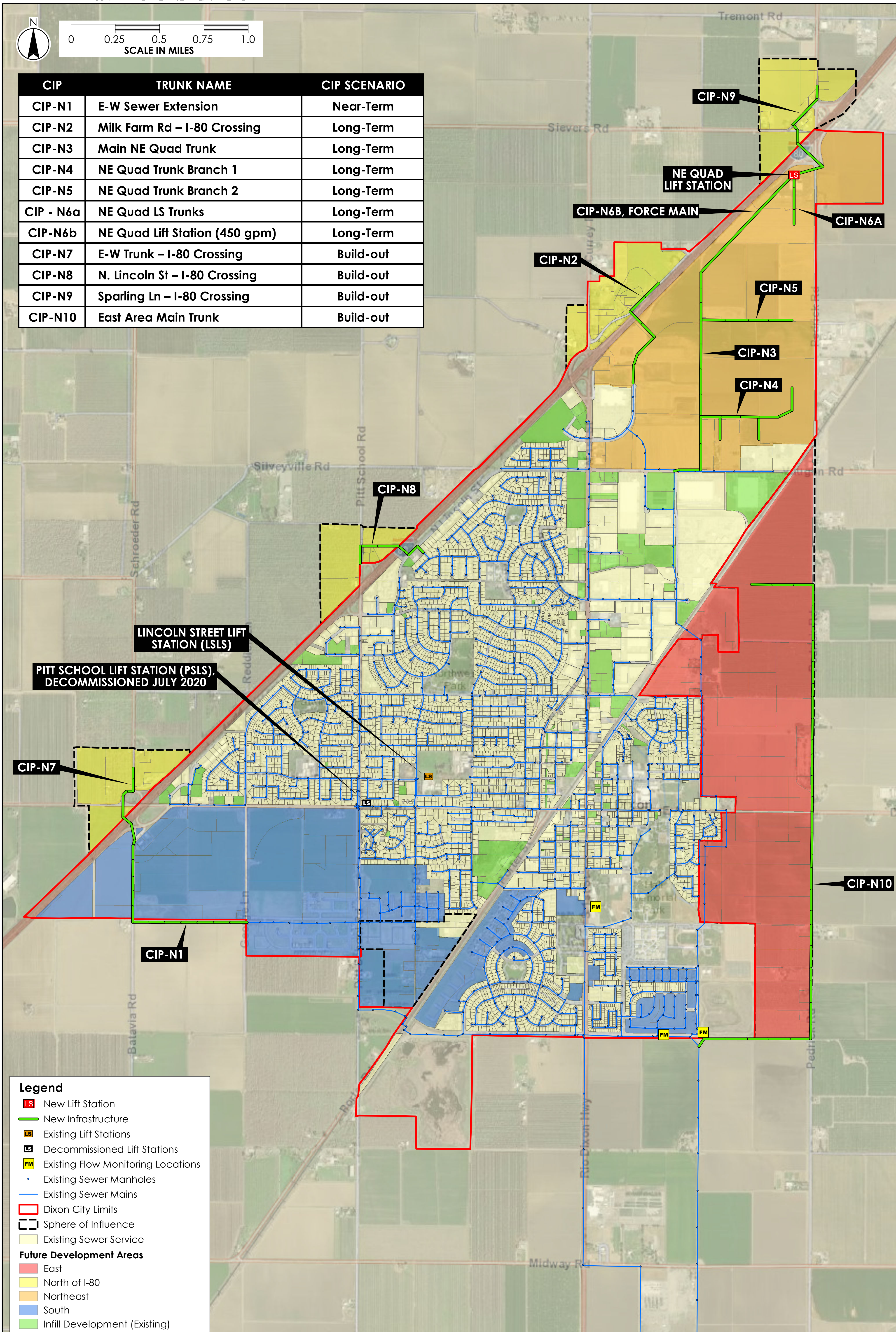

CIP	TRUNK NAME	CIP SCENARIO
CIP-N1	E-W Sewer Extension	Near-Term
CIP-N2	Milk Farm Rd – I-80 Crossing	Long-Term
CIP-N3	Main NE Quad Trunk	Long-Term
CIP-N4	NE Quad Trunk Branch 1	Long-Term
CIP-N5	NE Quad Trunk Branch 2	Long-Term
CIP - N6a	NE Quad LS Trunks	Long-Term
CIP-N6b	NE Quad Lift Station (450 gpm)	Long-Term
CIP-N7	E-W Trunk – I-80 Crossing	Build-out
CIP-N8	N. Lincoln St – I-80 Crossing	Build-out
CIP-N9	Sparling Ln – I-80 Crossing	Build-out
CIP-N10	East Area Main Trunk	Build-out

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Figure ES-3
Future Collection System – Proposed New Infrastructure

Future System Hydraulic Performance Evaluation

The hydraulic performance of the existing collection system was evaluated for each future development scenario considering the added flow from the expanded service area. The hydraulic performance evaluation of the future collection system identified the capacity constraints and recommended improvements summarized in **Table ES-10**.

Table ES-10 Summary of Capacity Related Improvements

CIP ID	Trunk Sewer	Scenario Identified	Length (LF)	Current Size (in)	Proposed Size (in)	Notes
E1	Industrial Way	Existing PWWF	2,100	10	15	A 12-inch sewer is required under existing conditions, but a 15-inch will be needed for future scenarios.
E2	Fitzgerald Dr.	Long-Term	2,550	21	27	Surcharging is not expected to exceed the pipe crown until build-out.
E3	North Dixon Trunk	Build-out	300	30	36	Surcharging is not expected to exceed the pipe crown without improvements.

The evaluation also identifies capacity concerns within the system, which are sewers predicted to be approaching their maximum capacity at the specified level of development. Areas of capacity concern are summarized in **Table ES-11**. These sewers are predicted to have a hydraulic loading ratio (HLR) approaching 100% but do not exceed level of service criteria or cause surcharging in the system. No improvements are recommended for the sewers noted below, however, they should be monitored and re-assessed periodically to confirm modeled flows and available capacity as the City grows.

Table ES-11 Summary of Capacity Concerns (95 – 100% HLR)

Area of Concern	Trunk Sewer	Scenario Identified	Length (LF)	Current Size (in)	Proposed Size (in)	Notes
1	Parkway Blvd.	Near-term	800	27	NA	Shallow sloped, s = 0.0007 ft/ft. Mitigated by restoring the 27-inch Main Trunk.
2	South 1st Street	Near-term	800	15	NA	W Cherry St to Silveyville Cemetery/North Interceptor Sewer
-	North Dixon Trunk	Long-term	300	30	36	Recommended as CIP-E3 under build-out conditions.
3	North Dixon Trunk	Build-out	3,100	27	NA	Doyle Ln., E. A Street to E. H Street.
4	Dorset Drive	Build-out	770	10	NA	E Dorset Drive, two segments flowing south
5	42-inch Main Trunk	Build-out	NA	42	NA	Records show a slope of 0.00055 ft/ft. Capacity concern only exists at build-out if slopes are actually < 0.0004 ft/ft

Capacity improvements and areas of concern identified in the existing system are shown on **Figure ES-4**.

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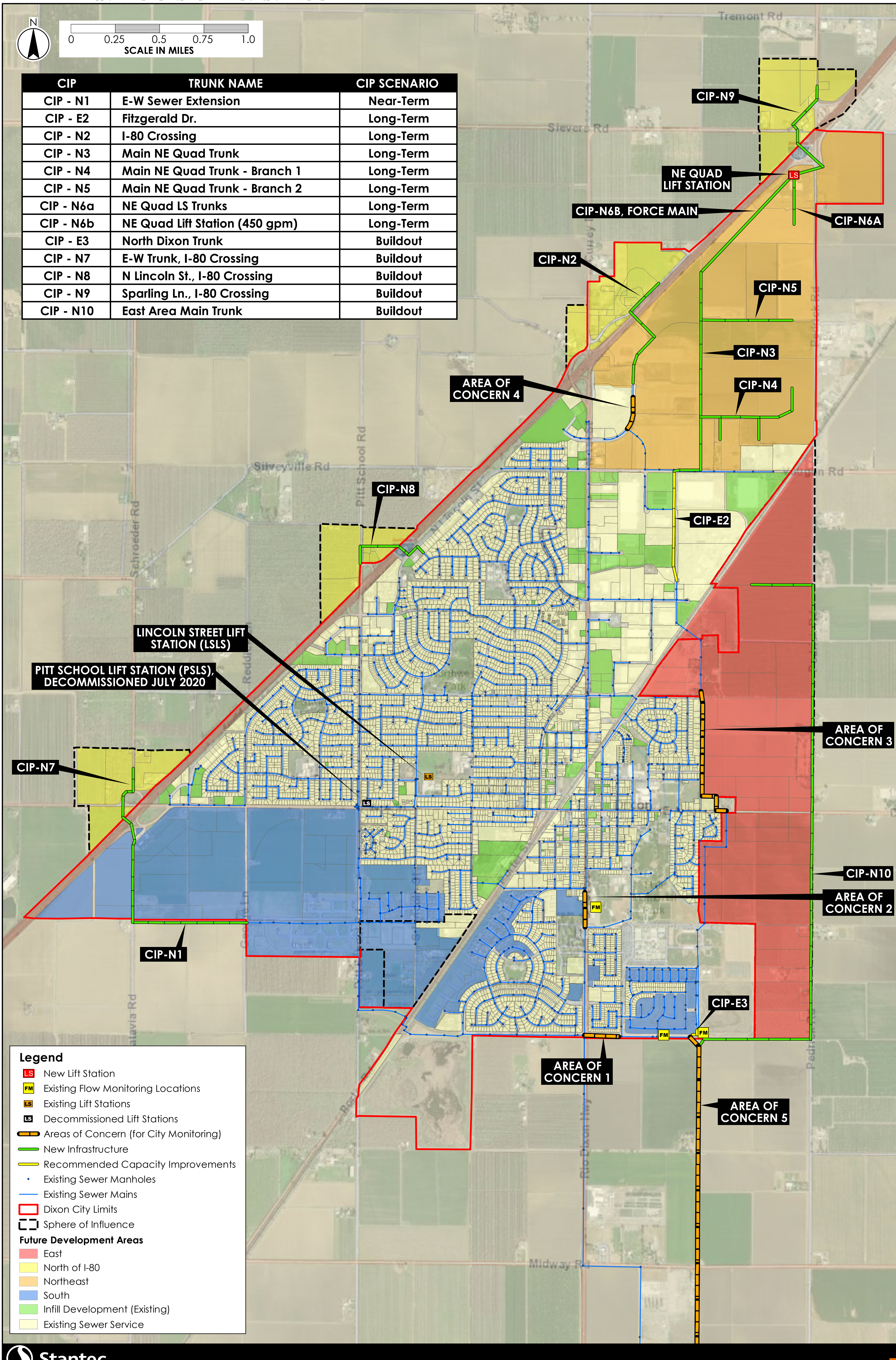

CIP	TRUNK NAME	CIP SCENARIO
CIP - N1	E-W Sewer Extension	Near-Term
CIP - E2	Fitzgerald Dr.	Long-Term
CIP - N2	I-80 Crossing	Long-Term
CIP - N3	Main NE Quad Trunk	Long-Term
CIP - N4	Main NE Quad Trunk - Branch 1	Long-Term
CIP - N5	Main NE Quad Trunk - Branch 2	Long-Term
CIP - N6a	NE Quad LS Trunks	Long-Term
CIP - N6b	NE Quad Lift Station (450 gpm)	Long-Term
CIP - E3	North Dixon Trunk	Buildout
CIP - N7	E-W Trunk, I-80 Crossing	Buildout
CIP - N8	N Lincoln St., I-80 Crossing	Buildout
CIP - N9	Sparling Ln., I-80 Crossing	Buildout
CIP - N10	East Area Main Trunk	Buildout

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Figure ES-4
Future Collection System – Recommended Capacity Improvements and Areas of Concern

ES.7 Condition Assessment

As described in **Chapter 9.0**, a detailed inventory of the City's existing wastewater system and its condition was developed from the City's GIS database, City improvement plans, interviews with City Staff, and City inspection records. The City's closed-circuit television (CCTV) sewer inspection records were reviewed. The records indicate 39 of the City's 1,464 sewer pipeline assets have critical (poor) condition scores due to structural and O&M defects.

The O&M defects (14 pipes) were caused by roots, grease buildup, or blockages; the assets with O&M defects should be added to the hotspot maintenance lists for more frequent maintenance and monitoring for further defects. The structural defects (25 pipes) were caused by cracks, breaks, offsets, or holes in the pipe and need to be replaced. For the 25 sewer assets with structural defects, the condition scores were used to prioritize those replacements.

The City has also conducted CCTV inspections of the 27-inch sewer trunk constructed in the 1950s. The sewer is nearing the typical useful life of VCP pipe. While not reviewed as part of this analysis, the CCTV inspections of the 27-inch trunk were reported to show widespread deterioration of the pipe. A review of potential solutions including a plastic liner system is recommended to extend the VCP's life. This trunk system is a high priority restoration project.

The City's Lincoln Street Sewer Lift Station has been in operation for over 35 years, lacks alarms and automation expected with stations of this size, shows signs of corrosion, and requires significant maintenance and oversight from City crews. The station is a package station meaning it is comprised of a steel "can" with equipment inside and is not easily rehabilitated. As a result, the station is a high priority replacement project and should be replaced with a new station consistent with other City wastewater facilities that have been designed for a 50+ year life.

Approximately $7.4 million of sewer and pump station replacement and rehabilitation projects have been identified as part this evaluation. They have been generally grouped into sewer replacements ($925,000), installation of a lining system in the 27-inch trunk ($5.1 million), and replacement of the Lincoln Street Sewer Lift Station ($1.4 million). It is recommended to complete these projects over the next five years.

An estimate of annual funding needs for sewer replacement was developed based on CCTV scores and the sewer's age and assumed life. The projection was completed over the next 25 years to capture larger projects such as the City's 42-inch trunk sewer. Those projects total about $23 million (expressed in current dollars) over the 25-year period.

ES.8 Capital Improvement Program

Based on the evaluations performed for this SCSMP, several improvement projects have been recommended for the City's existing, and build-out collection system. The locations of the recommended collection system improvements are shown on **Figure ES-5**. The estimated costs for the recommended Capital Improvement Program (CIP) are described in **Chapter 10.0** and summarized in **Table ES-12** below. Additional details on the assumptions used in the development of the estimated costs are provided in **Appendix E.**

Table ES-12 Summary of Recommended Sewer Collection System CIP Cost (1) (2)

Improvement Cost Estimate	Existing System	Near-Term System	Long-Term System	Build-out System	Total Capital Costs
Repair & Replacement Program (3)	$5,750,000	$5,750,000	$5,750,000	$5,750,000	$23,000,000
Existing System Improvement (4)	$617,000	$0	$1,350,000	$227,000	$2,194,000
New Infrastructure	$0	$1,202,000	$5,409,000	$6,062,000	$12,673,000
Total:	**$6,367,000**	**$6,952,000**	**$12,509,000**	**$12,039,000**	**$37,867,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.
3) Repair and replacement program projects are identified based on the existing assets physical condition. Repair and replacement plans are developed for a 5-year and 25-year period. The 5-year plan has a total cost of approximately $7.4 million and prioritizes improvements needed within the next 5-years. The 25-year plan has a total cost of $23 million and identifies projects needed to replace critical assets, including those in the 5-year plan, and those that meet the end of their useful life between now and 2047.
4) Existing system improvements are needed to address capacity deficiencies in the existing sewer system that occur under PWWF conditions at the specified level of development.
5) A unit rate of $1,000/LF was assumed for pipe segments crossing I-80 to provide an allowance for working in a CalTrans ROW.

The recommended existing system improvements for the Industrial Way Trunk should be completed as soon as possible to ensure adequate capacity to meet existing PWWF conditions. The construction of the recommended future system improvements should be coordinated with the proposed schedules of future development to ensure that the required infrastructure will be in place to serve future users.

It should be noted that the recommended sewer collection system improvements are identified at a master planning level and subsequent, more detailed evaluations may be needed prior to the design and construction of these improvements to confirm the sizing and locations that will also meet the City's future sewer collection system requirements.

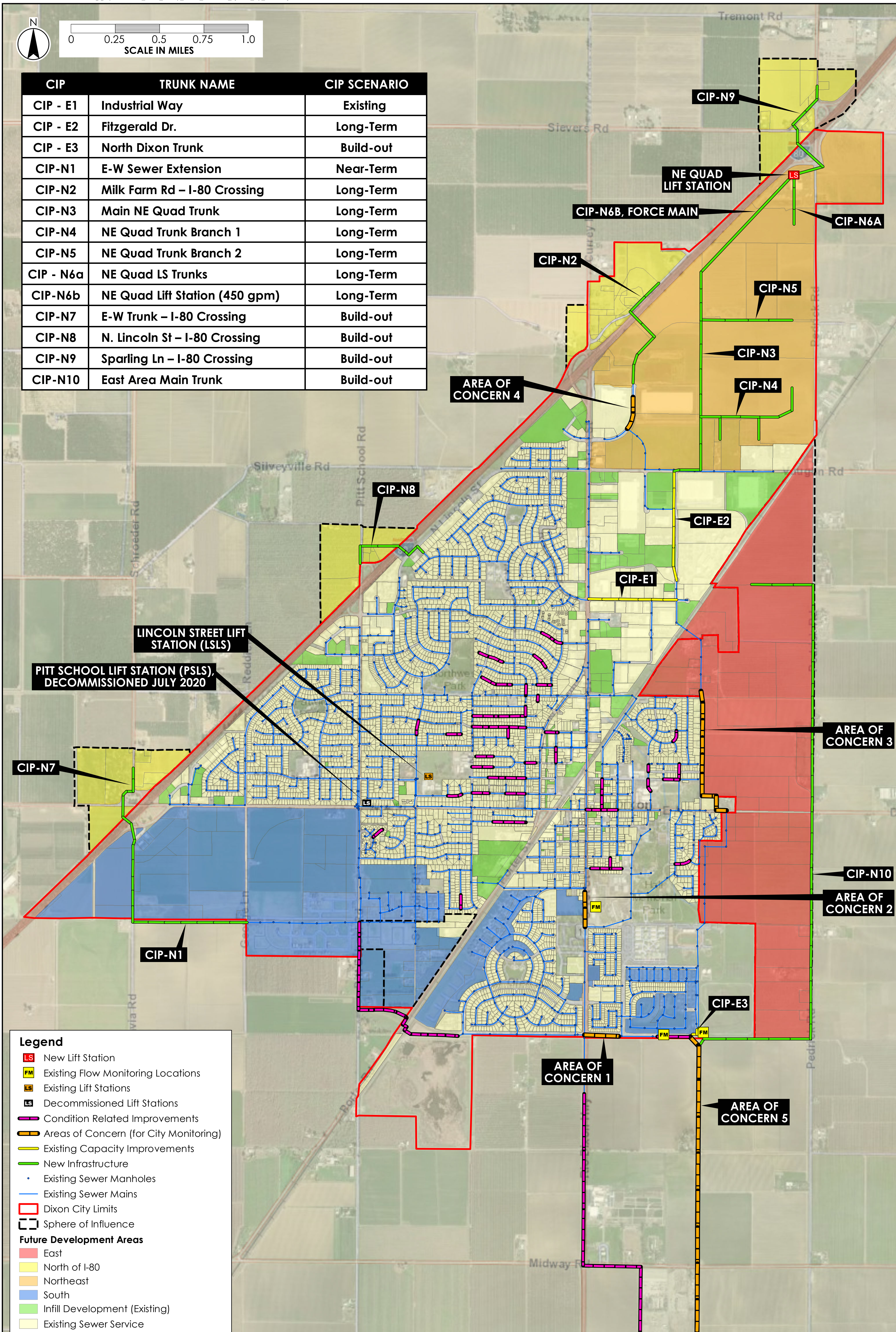
ES.9 Summary of Overall Recommendations & CIPs

This section of the executive summary provides a brief overview of the overall recommendations provided in this Master Plan.

- On-going I/I reduction program, flow monitoring, and model calibration: Perform flow monitoring twice annually to capture dry weather and wet weather flow data as part of an on-going I/I program. Additional details are provided in Section 4.3.
- It is recommended that the City monitor areas identified as "Areas of Concern" at the described levels of development as described in Sections 9.0.
- Monitor groundwater levels as they relate to the 27-inch main trunk sewer and consider the alternatives analysis described in Section 9.3.
- Implement the identified CIPs in Section 10.0 and summarized below.

Table ES- 1 Collection System CIP Summary

ID	Scenario	Name/Description	Length (LF)	Proposed Size (in)	Existing Size (in)	Capital Cost
CIP - E1	Existing	Industrial Way	2,100	15	10	$617,000
CIP - N1	Near-Term	E-W Sewer Extension	4,590	10 to 15	NA	$1,202,000
CIP - E2	Long-Term	Fitzgerald Dr.	2,550	27	21	$1,350,000
CIP - N2	Long-Term	I-80 Crossing	3,010	8 to 10	NA	$745,000
CIP - N3	Long-Term	Main NE Quad Trunk	5,220	12 to 21	NA	$1,819,000
CIP - N4	Long-Term	Main NE Quad Trunk - Branch 1	3,730	8 to 12	NA	$647,000
CIP - N5	Long-Term	Main NE Quad Trunk - Branch 2	2,130	8 to 12	NA	$391,000
CIP - N6a	Long-Term	NE Quad LS Trunks	4,990	8 to 10	NA	$517,000
CIP - N6b	Long-Term	NE Quad Lift Station (450 gpm)	NA	Dual 4-inch force main	NA	$1,290,000
CIP - E3	Buildout	North Dixon Trunk	300	36	30	$227,000
CIP - N7	Buildout	E-W Trunk, I-80 Crossing	1,980	8	NA	$478,000
CIP - N8	Buildout	N Lincoln St., I-80 Crossing	2,100	8	NA	$462,000
CIP - N9	Buildout	Sparling Ln., I-80 Crossing	2,410	8 to 10	NA	$1,020,000
CIP - N10	Buildout	East Area Main Trunk	14,720	8 to 21	NA	$4,102,000
					Total:	**$14,867,000**



CIP	TRUNK NAME	CIP SCENARIO
CIP - E1	Industrial Way	Existing
CIP - E2	Fitzgerald Dr.	Long-Term
CIP - E3	North Dixon Trunk	Build-out
CIP-N1	E-W Sewer Extension	Near-Term
CIP-N2	Milk Farm Rd – I-80 Crossing	Long-Term
CIP-N3	Main NE Quad Trunk	Long-Term
CIP-N4	NE Quad Trunk Branch 1	Long-Term
CIP-N5	NE Quad Trunk Branch 2	Long-Term
CIP - N6a	NE Quad LS Trunks	Long-Term
CIP-N6b	NE Quad Lift Station (450 gpm)	Long-Term
CIP-N7	E-W Trunk – I-80 Crossing	Build-out
CIP-N8	N. Lincoln St – I-80 Crossing	Build-out
CIP-N9	Sparling Ln – I-80 Crossing	Build-out
CIP-N10	East Area Main Trunk	Build-out

Figure ES-5
City of Dixon Sewer Collection System CIP Summary

Abbreviations

ABAG	Association of Bay Area Governments
ADWF	Average Dry Weather Flow (observed during the dry season)
BMP	Best Management Practice
CalTrans	California Department of Transportation
CDOF	California Department of Finance
CIP	Capital Improvement Program
CVRWQCB	Central Valley Regional Water Quality Control Board
DU	Dwelling Unit
du/ac	dwelling units per acre
DWF	Dry Weather Flow (Observed during the flow monitoring period, used in model simulations)
EDU	Equivalent Dwelling Unit
GIS	Geographic Information System
gpcd	Gallons per Capita Per Day
gpd, gal/d	Gallons per Day
GWI	Ground Water Infiltration

HGL	Hydraulic Grade Line
HLR	Hydraulic Loading Ratio
I/I	Inflow and Infiltration
I-80	Interstate 80
IDM	Inch-diameter-mile
IDW	Inverse Distance Weighting
in	inches
LF	Linear Feet
LiDAR	Light Detection and Ranging
LOS	Level of Service
LSLS	Lincoln Street Lift Station
MG	Million Gallons
MGD	Million Gallons per Day
NEQSP	Northeast Quad Specific Plan
PCSWMM	Personal Computer Storm Water Management Model
PS	Pump Station
PSLS	Pitt School Lift Station

PWWF	Peak Wet Weather Flow
RDI	Rainfall Dependent Infiltration
RDII	Rainfall Dependent Inflow and Infiltration
ROW	Right of Way
SCSMP	Sewer Collection System Master Plan
SIP	Shelter-In-Place
SOI	Sphere of Influence
SR-113	State Route 113
SRTC	Sensitivity-based Radio Tuning Calibration
SWDSP	Southwest Dixon Specific Plan
V&A	V&A Consulting Engineers, Inc.
Water Year	October to September (i.e. Water Year 2020 = 10/19 – 11/20)
WDR	Waste Discharge Requirements
WWTF	Wastewater Treatment Facility

1.0 INTRODUCTION

The City of Dixon (City) currently collects and treats wastewater from an area of approximately 2,500 acres within the city limits, serving a population of approximately 20,000 residents as well as several industrial and commercial users. The City is located approximately 19 miles west of the City of Sacramento and 67 miles northeast of San Francisco. A vicinity map showing the location of the City of Dixon is provided as **Figure 1-1**.

This chapter is divided into the following sections:

- Sewer System Master Plan Purpose
- Sewer System Master Plan Objectives
- Authorization
- Report Organization
- Acknowledgments

1.1 SEWER SYSTEM MASTER PLAN PURPOSE

The purpose of this Sewer Collection System Master Plan (SCSMP) is to identify existing wastewater collection system deficiencies and required system improvements to formulate a comprehensive Capital Improvement Program (CIP) that meets the needs of the City's existing and future service area defined by the boundaries presented in the City's General Plan 2040. The system capacity assessment is performed using simulated results of the hydraulic model built based on updated wastewater flow projections, GIS data, system evaluations, survey data, and calibrated using wastewater system flow monitoring data.

This SCSMP was completed based on information for the City's sewer collection system at the end of 2019 and early 2020. The system updates and operational changes incorporated in 2020 considered in this SCSMP are described, changes outside those outlined in this SCSMP are not considered. This effort was predominantly completed in February 2021. During the ensuing time period, initial project conditions changed including: 1) the decommissioning of the Pitt School Lift Station; 2) initial occupation of the Homestead Development; 3) completion of the East-West sewer trunk connection; and 4) completion of the General Plan 2040 for the City in May 2021.

None of these updated conditions are material enough to change the recommendations of the study developed as of February 2021. The SCSMP contains recommendations for future efforts to update the system modeling with updated flow data that reflects these more recent conditions. Capital costs were updated to the 2022 time period.

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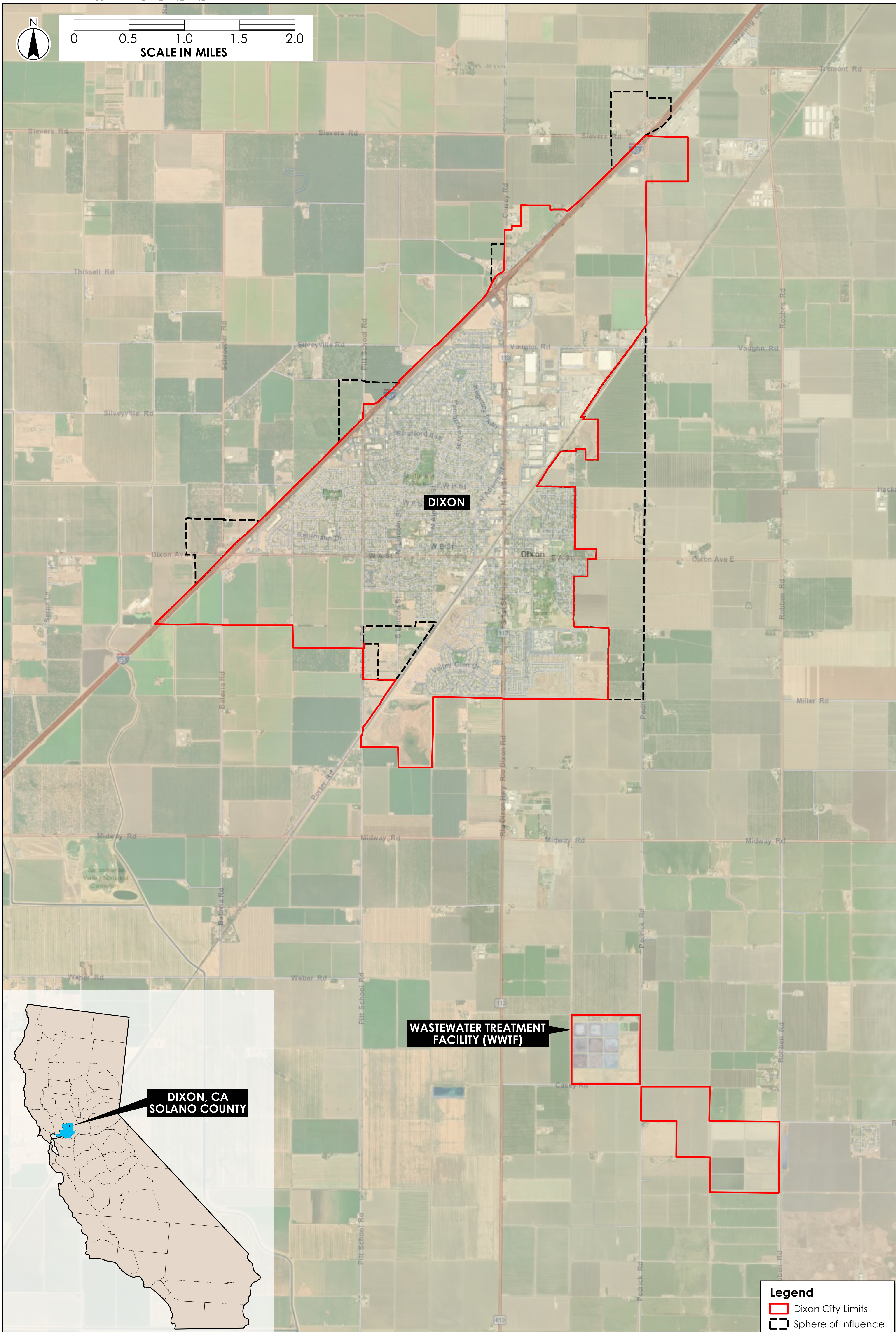

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Figure 1-1
City of Dixon Vicinity Map

1.2 SEWER SYSTEM MASTER PLAN OBJECTIVES

The objectives of the SCSMP are to:

- Perform gap analysis and review of the City's geographic information system (GIS) data and use that data to develop, calibrate, and expand a hydraulic model of the existing collection system and update the City's NEXGEN AM asset management database;
- Develop level of service (LOS) and design criteria under which the existing wastewater collection system will be evaluated, and future facilities will be planned and designed;
- Determine existing wastewater flows within the collection system through a temporary flow monitoring study, the data from which will be used to perform hydraulic analysis and project future flows that will occur with future development;
- Assess the capacity and condition of the existing collection system, including main trunk sewer lines and lift stations;
- Identify improvements, if any, needed to provide adequate capacity and LOS within the existing wastewater collection system;
- Identify potential new wastewater collection system infrastructure needed to serve future development;
- Prepare a CIP and identify priority and strategic projects including expansion and upgrades to existing infrastructure and construction of new infrastructure to meet system needs, including preliminary cost estimates.

1.3 AUTHORIZATION

Stantec Consulting Services Inc. (Stantec) was authorized to prepare this SCSMP by the City of Dixon on August 21, 2019 per City of Dixon City Council Resolution 19-132.

1.4 REPORT ORGANIZATION

This SCSMP is organized into the following chapters:

- Chapter 1: Introduction
- Chapter 2: Existing Sewer System
- Chapter 3: Planning Area Characteristics
- Chapter 4: Sanitary Sewer Flows
- Chapter 5: Planning & Design Criteria
- Chapter 6: Hydraulic Model Development
- Chapter 7: Existing Collection System Evaluation
- Chapter 8: Future Collection System Evaluation
- Chapter 9: Condition Assessment
- Chapter 10: Capital Improvement Program

The following appendices to this SCSMP contain data collected as part of this effort, additional technical information, assumptions, and calculations:

- Appendix A: V&A Flow Monitoring Report
- Appendix B: Previous City of Dixon Sewer Studies
- Appendix C: Model Calibration Details
- Appendix D: Model Results LOS Plan View Figures
- Appendix E: Model Results HGL Profile View Figures
- Appendix F: NexGEN Asset Management TM

1.5 ACKNOWLEDGEMENTS

The development of this SCSMP would not have been possible without the key involvement and assistance of City staff. The following staff provided comprehensive information, significant input, and key insights throughout the development of this SCSMP:

Deborah Barr, PE
City Engineer/Director of Utilities
City of Dixon

Jordan Santos
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City of Dixon

Evan “Sandy” Jones
Chief Plant Operator
City of Dixon

Leland Markusen
Junior Civil Engineer
City of Dixon

Brian Spindor, PE
Project Manager
Harris & Associates

Ann Hajnosz, PE
Vice President
Harris & Associates

This SCSMP was developed in association with NEXGEN Utility Management. V&A Consulting Engineers, Inc. performed wastewater collection system flow monitoring and provided data essential to development of this SCSMP.

2.0 EXISTING SEWER SYSTEM

The purpose of this chapter is to describe the City's existing sewer service area and sewer collection system facilities. System information was obtained through the review of previous reports, maps, record drawings, operating records, and other available data provided by the City. Surveying was conducted to collect rim and invert elevations at critical points in the collection system where necessary. The information was assembled and updated in a geographic information system (GIS) database before being incorporated into the hydraulic model of the collection system used in the development of this SCSMP. The development of sewer system GIS database is described in **Section 6.1** of this SCSMP.

This chapter is divided into the following sections:

- Existing Sewer Service Area
- Existing Connections & Population Served
- Dixon Wastewater Treatment Facility
- Existing Collection System Facilities

2.1 EXISTING SEWER SERVICE AREA

The City of Dixon is located along the Interstate 80 (I-80) corridor in central Solano County, as shown in **Figure 1-1**. The City covers an area of approximately five square miles and is bisected by State Route 113 (SR-113), which runs north-south through the center of the City. The City is bordered on all sides by agricultural lands and the City has maintained close ties to its agricultural heritage.

Dixon is known as a distinctive community with small-town charm. The older portion of the City, commonly referred to as "Old Town", contains numerous century-old buildings. At the same time, the City has also developed many new neighborhoods, employment centers, and shopping destinations.

The City provides sanitary sewer service within its city limits and treats collected wastewater at its Wastewater Treatment Facility (WWTF). The City's existing collection system covers an area of approximately 2,500 acres and provides service to residential, industrial, and commercial users. The wastewater generated from these users is collected and conveyed to the WWTF by a network of sewer pipes, force mains, and lift stations. **Figure 2-1** shows the City's existing service area and sanitary sewer collection system network including pump stations and force mains.

2.2 EXISTING CONNECTIONS & POPULATION SERVED

The following describes the existing number of sewer service connections and industrial dischargers which contribute various industrial waste streams to the City's collection system. This section also presents the current population served, and future population projections based on existing data and planning documents.

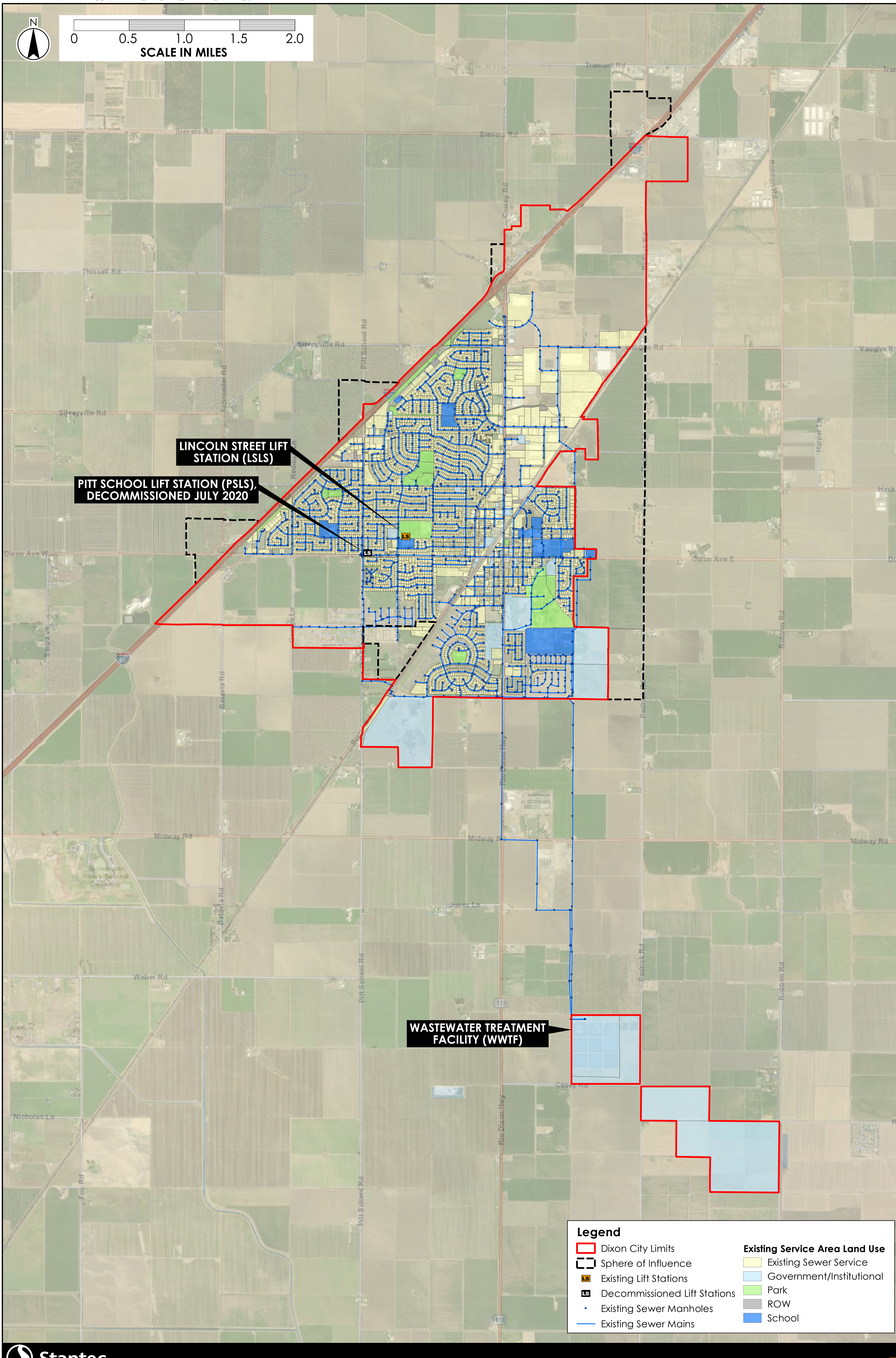

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Figure 2-1
City of Dixon Wastewater Collection System

2.2.1 User Connections & Industrial Dischargers

The existing wastewater collection system provides service to over 5,000 residential and commercial connections. The influent wastewater includes flows from five industrial dischargers that make up approximately seven percent of the annual flow (1.13 MGD).

Table 2-1 Industrial Wastewater Users

Industry	Address	Average Flow (MGD)
Genetech Meter 1	2727 Fitzgerald Drive	0.051
Former Gymboree Meter 1	2299 Kids Way	0.005
Milgard Meter 1	1320 Business Park Drive	0.012
Western Insulfoam Meter 1	1155 A Business Park Drive	0.013
Western Insulfoam Meter 2	1155 B Business Park Drive	0.005

2.2.2 Historical & Projected Population

The City of Dixon is the second smallest city in Solano County, with a 2020 population of 19,972 according to the California Department of Finance (CDOF).

Table 2-2 City of Dixon Historical Population Data (2005-2020)

Year	Total Historical Population (1, 2)	Annual Percent Change in Total Population
2006	17,914	2.7%
2007	18,105	1.1%
2008	18,148	0.2%
2009	18,293	0.8%
2010	18,441	0.8%
2011	18,293	-0.8%
2012	18,388	0.5%
2013	18,525	0.7%
2014	18,986	2.5%
2015	19,080	0.5%
2016	19,229	0.8%
2017	19,485	1.3%
2018	19,686	1.0%
2019	19,920	1.2%
2020	19,972	0.3%

1) CDOF, E-4 Population Estimates for Cities, Counties, and the State, 2001-2010, with 2000 and 2010 Census Counts, November 2012.
2) CDOF, E-4 Population Estimates for Cities, Counties, and the State, 2011-2020, with 2010 Benchmark.

Table 2-2 shows the population total for the City of Dixon and the annual change in population, which is assumed to equate to the approximate population within the City's sewer service area. Based on the CDOF population data, the City experienced a slight decline in population after 2010. This decline in population is most likely a result of the Great Recession. The City has seen a slow increase in population since 2010. The City's estimated average annual growth rate between 2000 and 2015 is approximately 1%. The Association of Bay Area Governments (ABAG) projects that Dixon will grow by 30% between 2020 and 2040, an increase of approximately 5,000 to 6,000 residents.

2.3 DIXON WASTEWATER TREATMENT FACILITY

The City of Dixon owns and operates the wastewater treatment and collection system facilities serving the City. The existing wastewater treatment facility (WWTF) is located at 6915 Pedrick Road, as shown in **Figure 2-1.** The City's WWTF has been in operation since 1952, but recently underwent a significant improvement project to comply with regulatory requirements and provide capacity for current and entitled projects within the City through 2040. The WWTF is permitted by the Central Valley Regional Water Quality Control Board (CVRWQCB) under WDR Order No. R5-2014-0098.

On July 9, 2015 the City of Dixon celebrated the groundbreaking of its Wastewater Treatment Facility Construction Project (WWTF Improvements Project). The WWTF Improvements Project improvements included the construction of new secondary treatment facilities and abandoning thirteen wastewater treatment ponds. The new system contains two treatment trains operated in parallel, each including a concrete oxidation ditch and a secondary clarifier. The new secondary facilities include an activated sludge treatment process including denitrification. Treated wastewater effluent is disposed of using eight percolation basins with a total surface area of 160 acres.

The WWTF Improvements Project was completed in 2017 to bring it into compliance with the permit issued by the Central Valley Regional Water Quality Control Board. A flow schematic of the existing wastewater treatment system process is shown in **Figure 2-2**.

2.4 EXISTING COLLECTION SYSTEM FACILITIES

The City's existing wastewater collection system facilities are shown in **Figure 2-1**. The collection system consists of approximately 75 miles of sanitary sewers (local sewers, trunk sewers, and force mains) and two lift stations, the Pitt School Lift Station (PSLS), and the Lincoln Street Lift Station (LSLS). It should be noted that the PSLS was taken out of service shortly after model development. The existing collection system facilities are further described below. The evaluation of facility capacities and their ability to meet existing and future sewer service needs is discussed in **Chapter 7.0** of this SCSMP, titled *Existing Collection System Evaluation*.

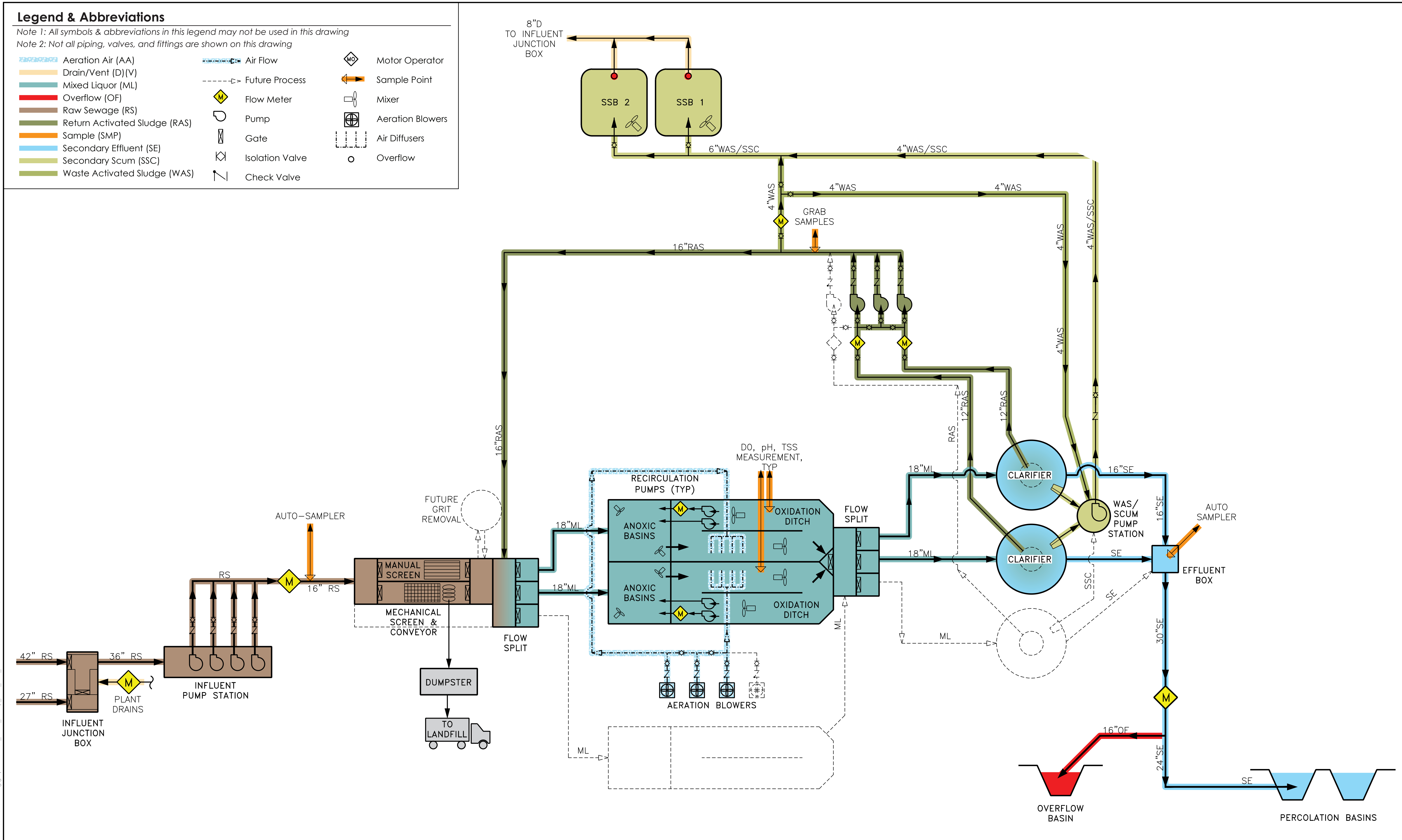


Figure 2-2
City of Dixon Wastewater Treatment Facility Flow Schematic

2.4.1 Primary Trunk Sewers

The oldest portions of the City's existing collection system were constructed in 1952, along with the original WWTF. This includes the 27-inch main trunk line conveying flow collected within the City to the WWTF. The sewer system has since expanded to accommodate growth. A new 42-inch trunk line was constructed in 2003. Both trunks suffered from inflow and infiltration until the 42-inch trunk line was repaired and the 27-inch trunk line was isolated from service in April 2005. The City plans to fully repair the 27-inch trunk line before bringing it back into service. As a result of fixing the 42-inch trunk line and temporarily removing the 27-inch trunk line, inflow and infiltration has significantly improved.

For purposes of this SCSMP, sewers larger than 8-inches in diameter are referred to as primary trunk sewers and sewers less than or equal to 8-inches in diameter are referred to as local collector sewers. Approximately one third of the City's collection system consists of primary trunk sewers, equating to approximately 25 miles of the system. The hydraulic model constructed in the development of this SCSMP includes all of the City's existing sewers, but of the hydraulic capacity evaluation is focused on the City's primary trunk sewers.

Table 2-3 summarizes the size, length, and location of the primary trunk sewers within the City's collection system.

A new gravity sewer line from the southwest corner of Pitt School Road and West A Street to the existing sewer trunk system on South First Street at Parkway Boulevard (southeast corner of the Valley Glenn Subdivision) was recently constructed and commissioned in July of 2020. The new 15 to 27-inch East-West Trunk Connector provides gravity service to Pitt School Lift Station (PSLS) service area and allowed the lift station to be decommissioned. A cost-benefit analysis over a 20-year period showed that abandoning the PSLS and constructing the East West Sewer Trunk Connector would cost approximately half of the cost of repairing, operating, and maintaining the existing lift station.

The hydraulic model scenarios include bringing the new East West Trunk Connector online and eliminating the PSLS, but it should be noted that this change was incorporated into the model post-calibration. The flow monitoring data used to calibrate the hydraulic model described in this SCSMP was collected immediately before this new trunk was brought into service and the lift station was decommissioned.

Table 2-3 Primary Trunk Sewers

ID	Primary Trunk Sewer	Diameter (in)	Length (LF)	Sewer-shed (1)
1	27-inch Main Trunk	27	15,200	Main
2	42-inch Main Trunk	42	12,700	Main
3	Cherry St./Porter Road Crossing	15	2,700	3
4	Collier Manor Trunk	10	3,800	2
5	Connemara Trunk	10	3,300	1
6	East A Street	12	1,500	3
7	East-West Trunk Connector	15 to 27	7,800	2
8	E-W Branch 1	18	2,600	2
9	E-W Branch 2	10	3,100	2
10	Fitzgerald Dr.	21	3,000	1
11	Industrial Way Trunk	10	2,100	1
12	Manning Way	10	700	3
13	North 1st Street – Part 1	10	2,300	3
14	North 1st Street - Industrial	10 to 12	4,200	1
15	North Dixon Trunk	27 to 30	11,800	1
16	North Interceptor Sewer	15	10,000	2
17	North Lincoln Street	10	3,100	3
18	Parkway Blvd	27	5,300	2
19	Pheasant Run Dr.	10	1,900	3
20	Pitt School Rd	12	3,200	3
21	PSLS/Rehrmann Dr	15	3,300	3
22	South 1st St – Part 2	14	1,800	3
23	South 1st Street	15 to 27	3,400	3
24	South Almond/Hillview Dr.	15	2,000	3
25	South Lincoln/West A St	15	2,500	3
26	Vaughn Rd/Dorset Drive	10 to 12	3,000	1
27	Vaughn Rd - East	15 to 18	2,300	1

1) Sewer-sheds within the collection system are defined as the portion of the collection system that contributes flow to each of the flow monitoring locations. Flow monitoring data is used to calibrate the hydraulic model, additional information is provided in **Chapter 4.0**.

2.4.2 Lift Stations

At the beginning of the development of this SCSMP, City's collection system had two wastewater lift stations that provided service to approximately 20% of the sewer service area. As previously discussed, the Pitt School Lift Station (PSLS) was recently abandoned and taken out of service with the construction of the East West Sewer Trunk Connector. The PSLS had a reliable pumping capacity of approximately 0.5 MGD. The reliable capacity assumes that one of the two pumps is out of service, although records indicate that they operate in a lead-lag fashion. This lift station was included in the hydraulic model for purposes of calibration.

The City has an existing Capital Improvement Project (CIP 315) to completely reconstruct the Lincoln Street Lift Station (LSLS) located on North Lincoln Street. The improvement project includes new piping, overflow controls, below grade pumps, and an emergency generator. The existing lift station is deteriorating and requires a significant amount of maintenance. The original lift station was located in the roadway and was relocated to its current location when North Lincoln Street was widened. The piping system was constructed to divert flow to the current lift station and the sewer pipe is cracked and needs to be replaced. The placement of the existing pumps in the wet-well need maintenance staff to enter the confined space for routine and emergency service to the pumps. The LSLS has a reliable pumping capacity of approximately 0.8 MGD and operates in a lead-lag fashion.

Physical characteristics and information regarding the lift stations that was incorporated into the hydraulic model is summarized in **Table 2-4**.

Table 2-4 Lift Station Data

Parameter	PSLS (1)	LSLS (2)	Units
Pump Station Capacity	350	550	gpm
Pump Station Capacity	0.5	0.8	MGD
Wet-Well Diameter	60.0	72.0	in
Cross Section Area	19.6	28.3	ft^2
Volume (total)	3084.5	4335.9	gal
Volume (total)	412.3	579.6	ft^3
Inlet Elevation	50.2	49.3	ft
Invert Elevation	46.0	43.5	ft
Rim Elevation	67.0	64.0	ft
Total Depth	21.0	20.5	ft
Pump Off Elevation	48.0	45.0	ft
Pump Off Depth	2.0	1.5	ft
Pump On Elevation	50.0	49.0	ft
Pump On Depth	4.0	5.5	ft
Second Pump On Elevation	52.0	49.5	ft
Second Pump On Depth	6.0	6.0	ft
High Water Level Elevation	59.0	50.0	ft
High Water Level Depth	13.0	6.5	ft
Operating Depth, P1	2.0	4.0	ft
Operating Depth, P2	4.0	4.5	ft
Operating Depth, Below Inlet	2.2	4.3	ft
Operating Depth, Before P2 On	4.0	4.5	ft
Operating Depth, High Water Level	11.0	5.0	ft
Depth, Inlet to High Water	8.8	0.7	ft
Depth, Both Pumps to High Water Level	7.0	0.5	ft
P1 Operating Volume	39.3	113.1	ft^3
P1 Operating Volume	293.8	846.0	gal
High Water Volume	255.3	183.8	ft^3
High Water Volume	1909.4	1374.8	gal
Force main Diameter	6.0	8.0	in
Exit Elevation	58.0	60.1	ft

1) Information for the Pitt School Lift Station was extracted from the record drawing set titled: “Dixon-West "A" Street Assessment District, dated 1988”.
2) Information for the Lincoln Street Lift Station was extracted from the record drawing set titled: “Wastewater Treatment Plant Improvements, dated 1985”.

3.0 PLANNING AREA CHARACTERISTICS

The purpose of this chapter is to describe the City's existing sewer service area and present relevant planning data and information used to project wastewater flow and distribution from future development areas within the planning horizon of the City's General Plan 2040. This data was used as the basis for the development of the hydraulic model of the existing and future collection system. The hydraulic model was used to evaluate hydraulic conditions and the capacity of the existing system to identify infrastructure needed to provide service to future development areas. The following presents a summary of the City's existing and future development areas as described in the City's General Plan 2040, specific on-going development planning information, and future planning areas defined in the City's Water Master Plan (West Yost, 2018). These planning areas define the extent of the sewer service area at each level of development scenario evaluated in this SCSMP.

This chapter is divided into the following sections:

- Previous Sewer Planning
- General Plan 2040 Land Use
- Existing Service Area
- Future Service Area
- Level of Development Scenarios

3.1 PREVIOUS SEWER PLANNING

The City's existing wastewater collection system planning has primarily been driven by the on-going developments within the City's planning area. A sewer system Master Plan was prepared for the Southwest Dixon Specific Plan Area in 2005 by Nolte Associates, Inc. The most recent addendum to this document, the Homestead Development Sanitary Sewer Report prepared by Carlson, Barbee, & Gibson, Inc. (CBG) dated July 2019 was provided by the City for use in preparing this SCSMP in addition to the Master Sewer Study prepared by Wood Rodgers for the Valley Glen South Area dated April 2015. The Homestead Development Sewer Report identifies that it is consistent with the underlying principles and sanitary sewer system design concept presented in the August 2005 Master Plan, which was reviewed by the City and included as a supporting document to the Southwest Dixon Specific Plan. Information for the Parklane subdivision was presented in the Homestead Development Sewer Report and confirmed using aerial imagery, tentative subdivision maps, and record drawings. These previous sewer planning documents are attached as **Appendix B**.

3.2 GENERAL PLAN 2040 LAND USE

The City's General Plan 2040 was adopted in 2021 and is considered the guiding document relative to growth and development of land and services within its municipal boundaries. The General Plan 2040 outlines the City's goals for future development, circulation, conservation of resources, and utilizes policies and actions necessary to achieve these goals.

The goals and objectives of the General Plan update include:

- Directing and managing Dixon's future growth
- Assessing the need to expand Dixon's Sphere of Influence, especially west of I-80
- Strengthening Dixon's Downtown
- Attracting high quality businesses and jobs
- Improving mobility and transportation options
- Maintaining public safety and municipal services
- Protecting natural resources, including valuable farmland
- Maintaining and enhancing quality of life for residents

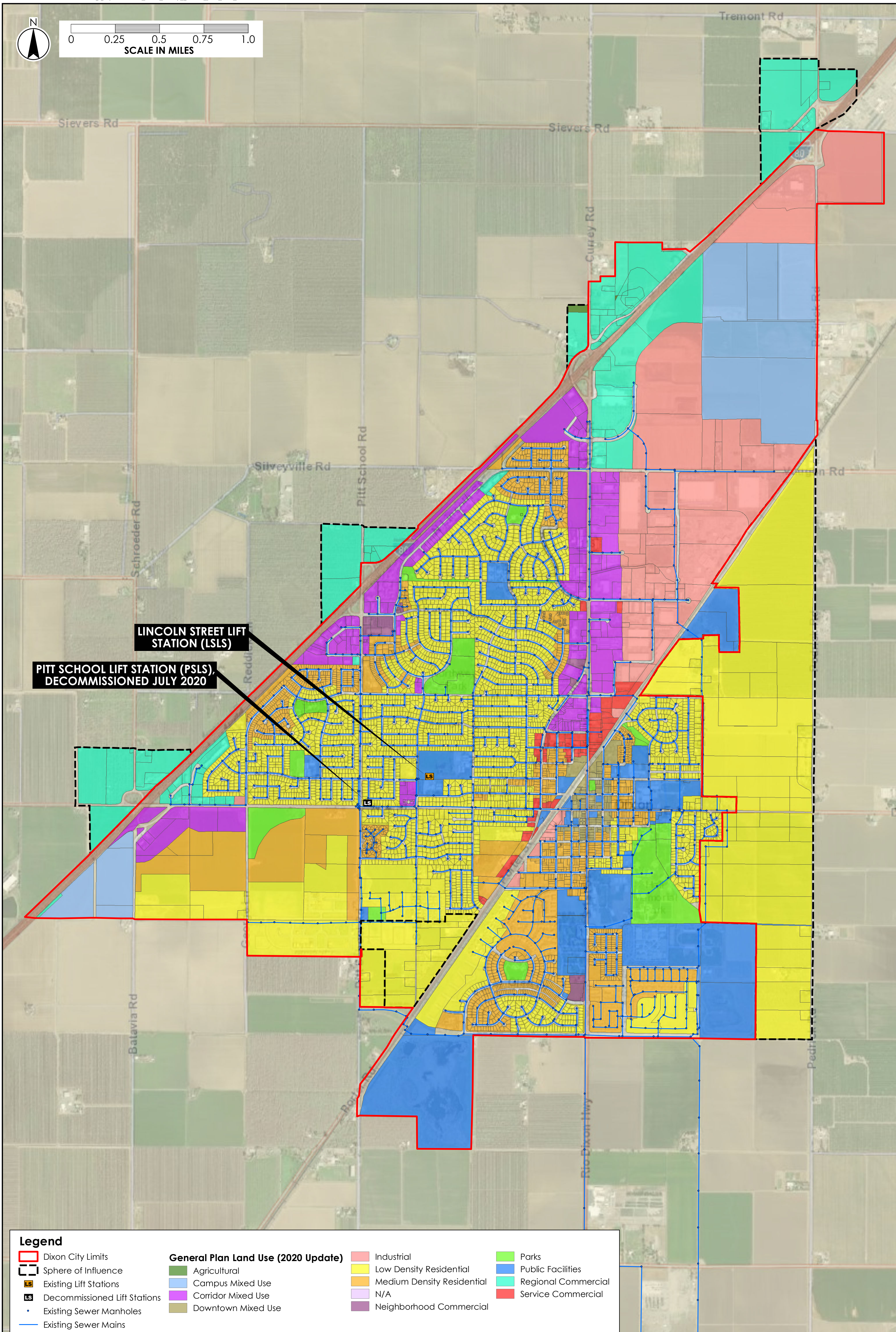
The General Plan 2040 outlines the City's current extents or city limits, as well as its ultimate planning area boundary, referred to as the sphere of influence (SOI). **Figure 3-1** illustrates designated land uses contained in the GIS shapefile associated with the General Plan 2040, which was provided by the City. The land use quantities from the General Plan are summarized in **Table 3-1**.

Table 3-1 General Plan 2040 Land Use [(1)]

Land Use Designation	Area (Acres)
Regional Commercial	580
Corridor Mixed Use	303
Medium Density Residential	513
Low Density Residential	1,790
Parks	130
Public Facilities	449
Downtown Mixed Use	317
Service Commercial	61
Campus Mixed Use	318
Industrial	549
Neighborhood Commercial	30
Agricultural	5
N/A	1
“Blank”	433
Total Area	**5,479**

1) This GIS data was provided as a draft during development of the General Plan 2040.

The General Plan 2040 provides the City with the opportunity to address priorities for the next era of development, enhance the economy, and the community's quality of life. The land uses established in the General Plan 2040 serve as the basis for establishing future wastewater flow estimates associated with future development areas within city limits and the SOI. These estimates are incorporated into the hydraulic model used to evaluate the future planning scenarios considered in this SCSMP.



Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 3-1
City of Dixon General Plan 2040 Land Use

3.3 EXISTING SERVICE AREA

The City currently collects and treats wastewater from an area of approximately 2,500 acres within city limits, serving a population of more than 19,000 residents as well as number of industrial and commercial users. The existing sewer service area was delineated using City parcel data, sewer system GIS data, and record drawings associated with recent and on-going developments. The assumptions and extent of the existing sewer service area was presented and confirmed by the City.

Vacant parcels within the existing extent of the sewer service area were identified from information presented in the City's Water Master Plan (West Yost, 2018). Future development or occupancy of vacant parcels within the existing service area is referred to herein as infill, or infill development. This SCSMP assumes vacant or infill development areas should get any available capacity within the existing system to limit the extents of existing system improvements. Although it is not likely that all infill areas will develop or become occupied simultaneously and/or before new construction occurs outside the extents of the existing system, flow estimates from infill areas are added to the existing system model as the first level of future development phasing to reserve any available capacity within the existing system in subsequent scenarios.

Evaluating the impacts of infill development first, distinguishes between the sewer system capacity impacts and improvements associated with providing service to the existing service area and those associated with expanding the existing service area and collection system. New development outside of the extent of the existing service area is evaluated assuming the existing system has been built out, in other words, all infill, on-going, and existing development areas contribute flow to the existing system.

Development outside of the existing service area will require new construction to extend and improve the existing system. The land uses within existing sewer service area and parcels identified as vacant/infill development areas are shown in **Figure 3-2**. **Table 3-2** presents a summary of the land use data for parcels within the existing extents of the wastewater collection system. Parcels identified as infill were assumed to equate to those identified as vacant in the City's Water System Master Plan.

Table 3-2 Existing Sewer Service Area Land Use

Land Use Designation	Existing Area (Acres)
Existing Sewer Service Area	1,443
Government/Institutional	702
Park	138
Right-of-way (ROW)	13
School	101
Vacant/Infill Development	*193*
Total	**2,590**

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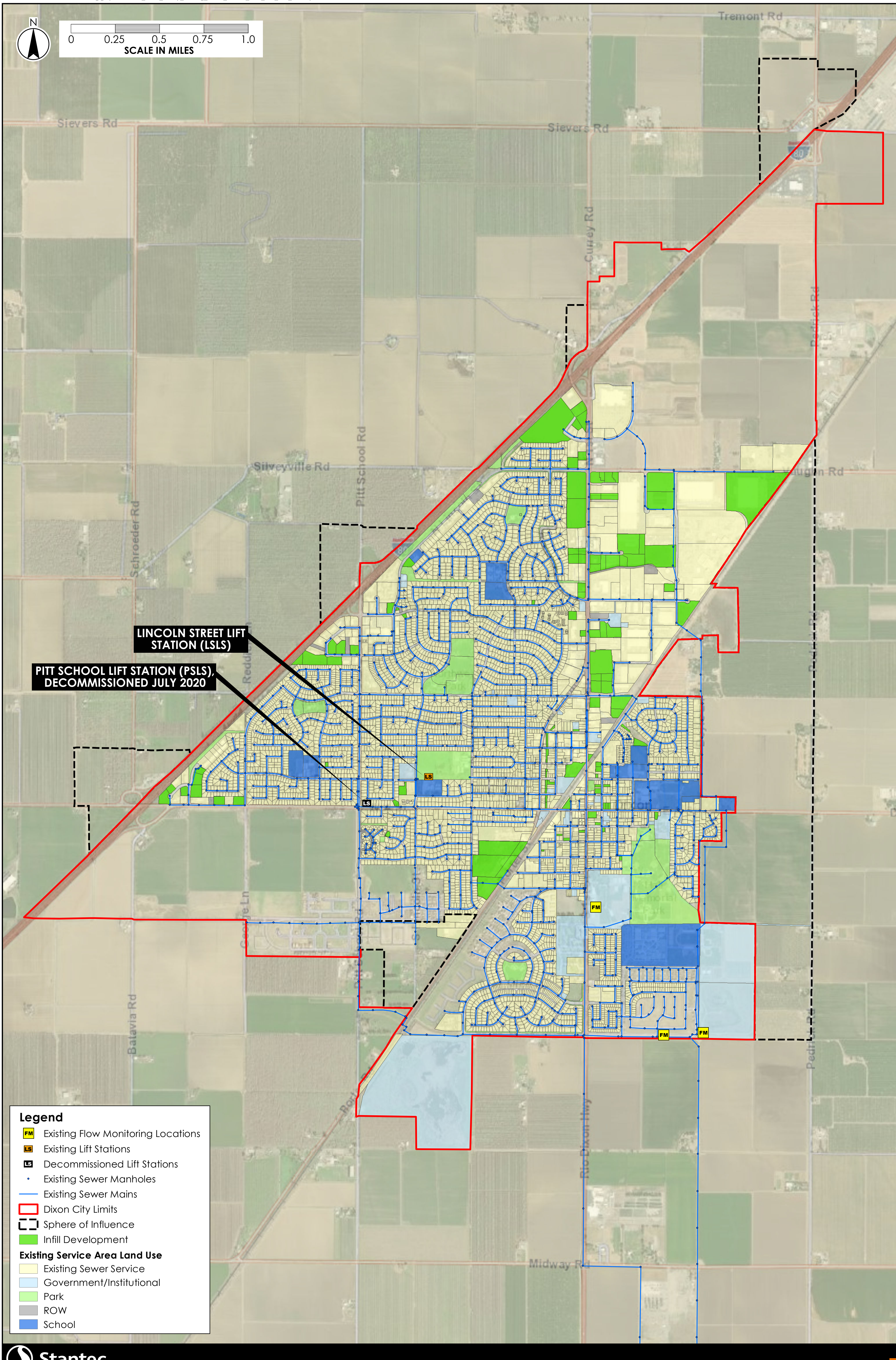

Figure 3-2
City of Dixon Existing Sewer Service Area & Infill Development

3.4 FUTURE SERVICE AREA

The following summarizes the planning information used to project future wastewater flows associated with planned development areas within the City's planning area. General Plan 2040 land use information and equivalent dwelling units (EDU) projections from Specific Plan are used as the basis to estimate future wastewater flow rates for each development area. The development areas are used to define the level of development for each future planning scenario considered in this SCSMP and simulated within the hydraulic model. The wastewater generation rates used to translate land use and EDU data into wastewater flow estimates is presented in **Chapter 4.0** of this SCSMP.

3.4.1 Planned Development Areas

To provide consistency with existing City planning documents, this SCSMP divides the City into the seven development areas outlined in the City's General Plan 2040 and Water Master Plan (West Yost, 2018). Future wastewater flow estimates for these areas were developed using the General Plan 2040 information provided by the City and the City's wastewater generation rates.

The "South" development area includes three on-going developments for which detailed planning information was provided. This information is summarized in the following section and used to project future wastewater estimates for these developments, as opposed to using the associated General Plan land use designation. Specific plans were provided for the Homestead Development, Valley Glen, and Parklane development areas.

The correlation between the seven development areas outlined in the General Plan 2040 and those used in this SCSMP is summarized in **Table 3-3**. A map depicting the development areas used in this SCSMP is presented as **Figure 3-3**.

The Existing, Downtown, and SR-113 Corridor development areas are collectively considered infill development. Specific Plan areas are referred to as on-going development areas.

Table 3-3 City of Dixon Development Areas

General Plan Development Areas	SCSMP Development Area	Specific Plan
Existing	Existing Service Area/Infill	
Downtown	Existing Service Area/Infill	
SR-113 Corridor	Existing Service Area/Infill	
South	South/On-going Development	Southwest Dixon (Homestead Development, Valley Glen, & Parklane)
Northeast	Northeast	Northeast Quadrant (1)
East	East	
North of I-80	North of I-80	

1) Updated General Plan land use data (2019) was used to project flow from the Northeast Quadrant as opposed to information presented in its Specific Plan, which was adopted in 1995.

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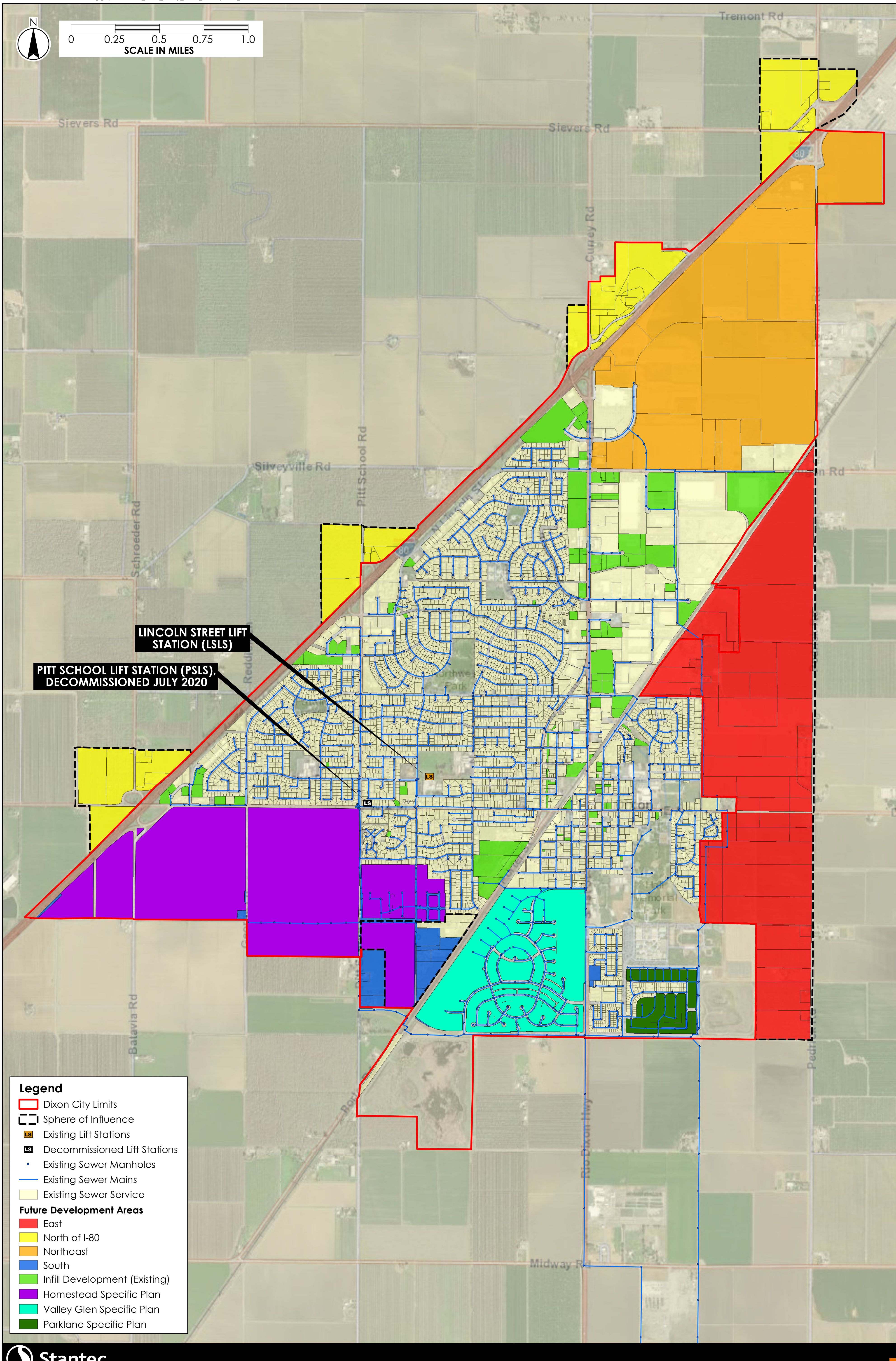


Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 3-3
City of Dixon Future Development Areas

3.4.2 Development Area Land Use

The General Plan 2040 GIS file provided by the City was used to assign land uses to parcels within each development area. This information is summarized for each future development in **Table 3-4**.

Table 3-4 Development Area Land Use

	Development Area (Gross Acres)					
Land Use	**Infill [(1)]**	**Northeast**	**East**	**South [(2)]**	**North of I-80**	**Total**
Open Space						
Agricultural	0	0	1	0	2	3
Parks	1	0	0	20	0	21
Mixed Use						
Campus Mixed Use	0	263	0	54	0	318
Corridor Mixed Use	82	0	0	37	0	119
Downtown Mixed Use	5	0	0	0	0	5
Residential						
Low Density Residential	1	0	616	371	0	988
Medium Density Residential	23	0	0	161	0	184
High Density Residential	0	0	0	0	0	0
Commercial						
Neighborhood Commercial	0	0	0	3	0	3
Regional Commercial	10	76	0	1	276	362
Service Commercial	6	0	0	0	0	6
Industrial						
Industrial	64	262	0	0	0	326
Public						
Public Facilities	2	0	29	9	0	39
Total:	**193**	**601**	**645**	**655**	**278**	**2,373**

1) Infill represents infill development within the existing service area, vacant parcels were identified as those presented in the City's Water Master Plan (West Yost, 2018).
2) Data for the south development area includes on-going development areas for which specific planning data was available and used to project flows. Specific Plan information includes: *Ongoing Development Area (remaining equivalent EDUs to be connected)*: Homestead Development (1,880 EDUs), Parklane (210 EDUs), and Valley Glen (259 EDUs). Specific Plans are further described in the following section of this SCSMP.

3.4.3 Specific Plan Information

As previously discussed, information extracted from specific planning documents was used to project future wastewater flow for on-going development areas within the City. The City of Dixon has two areas for which it has adopted specific plans, the Southwest Dixon and the Northeast Quadrant. Southwest Dixon is planned to include large residential components, while the Northeast Quadrant is primarily planned for commercial, light industrial, and office development.

The Southwest Dixon Specific Plan (SWDSP) and remaining south development area includes the three on-going developments (Homestead Development, Valley Glen, and Parklane) for which updated planning documents and specific plan addendums had been adopted and approved by the City as recently as July 2019. These on-going developments are primarily residential and include a variety of housing types to be developed at densities ranging from less than 2 du/ac to more than 24 du/ac.

The Northeast Quad Specific Plan (NEQSP) was adopted in 1995 and does not include any on-going developments. Therefore, the General Plan 2040 land use information is considered to be more recent than the specific plan information and was used to project flow from the area.

In summary, Specific Plan information was used to project future wastewater flow from on-going developments within the City's south development area. The remaining area to be developed within each on-going development area, including the associated land use and EDU projections within each on-going development area is further described below.

Homestead Development

The SWDSP area consists of approximately 269 acres and is located west of Porter Road and east of Interstate 80. Most of the site is presently in agricultural use. Approximately 61 percent of the land is designated for residential use, while the remainder is for commercial uses and public facilities. The Specific Plan contains three residential land use designations that provide for housing ranging from low-density single-family units to townhomes, cluster homes, and apartments.

The Southwest Dixon Specific Plan was adopted by the City Council in 1995 but has been updated and amended numerous times prior to its approval and development. Addendums and final specific planning information for the SWDSP included further refinement of the development area including tentative subdivision maps of the proposed Homestead Development. The new information required that the original Nolte Sewer Master Plan developed in 2005 be reconsidered and system capacity be re-evaluated. The resulting SWDSP Sewer Master Plan is presented in the Homestead Development Sanitary Sewer Report (CBG 2019).

The Homestead Development Sanitary Sewer Report describes the planned and approved sewer system improvements required to provide sewer service capacity for the SWDSP area and surrounding development areas. The evaluation focuses on the East West Trunk Sewer Connector, which was recently brought into service and allowed the PSLS to be decommissioned. The evaluation included an assessment of all on-going and planned development that will contribute to this portion of the collection system and included flow diverted from the PSLS.

The Homestead Development area is completely undeveloped under existing conditions. It is planned to include approximately 1,234 single family homes (EDUs), 10.4 acres of multi-family dwelling units, 69.5 acres of commercial/public areas, and 49.9 acres of Industrial areas. To quantify the stage of development and the amount of existing and future wastewater flow, the number of single-family homes that would produce the equivalent amount of flow (EDUs) as the multi-family, commercial, public, and industrial land use areas was determined, as shown in **Table 3-5**. Multi-family, commercial, public, and industrial flow projections were divided by the City's standard unit rate (250 gpd/EDU) to determine the equivalent number of EDUs.

This SCSMP assumes that the Homestead Development is entirely vacant or not currently contributing to the sewer system. The Homestead Development will add approximately 1,880 EDUs to the existing collection system in the future.

Valley Glen

The 210-acre Valley Glen residential development is in South Dixon, located adjacent to South First Street (SR-113) on the west side. The proposed 210-acre project is located within the City of Dixon and is bound to the north by existing residential homes along West Cherry Street, to the east by SR-113, south by rural farmland adjacent Parkway Boulevard, and west by the Union Pacific Railroad. The project consists of seven single-family residential villages containing 676 single family dwelling units, a 3.75-acre commercial site, a 4.4-acre condominium site, a 4.7-acre apartment site, and a 5.0-acre park.

The Master Sewer Study prepared by Wood Rodgers for the Valley Glen South Area (April 2015) describes the final sewer servicing plan for the area which was approved by the City. The purpose of the study was to summarize sewer infrastructure requirements for the Valley Glen South sewer-shed at full build out of the community. Several iterations of sewer studies had been prepared in the past for the entire development area, identifying infrastructure needs and connection points in support of splitting service into a north and south shed area. Due to the relatively flat topography across the site and in the surrounding areas, the sewerage serviceability for the site is provided from both the north and south, in West Cherry Street and Parkway Boulevard, respectively.

The Valley Glen development area is considered to be partially developed under existing conditions with 259 EDUs remaining to be connected to the sewer system. Approximately 360 single-family homes have been built, as well as the Bristol Apartment complex and Valley Glen Apartments, equating to approximately 578 EDUs. The Valley Glen development plans to add an additional 259 EDUs, resulting in a buildout EDU count of 837 EDUs.

Parklane

Parklane Subdivision Development is a residential community on approximately 94 acres immediately south of the existing Country Faire neighborhood and east of State Route 113 and was annexed to the City in 2005. The conditions of the development agreement included dedication of 40 acres for the new high school, completed in 2006, and construction of infrastructure, including streets, as well as a high volume water well and storage system (completed in 2007) for the southeast Dixon area. The development proposes four housing types, including alley-loaded, medium-density, and low-density single-family homes.

Planning Area Characteristics

The Parklane Subdivision is considered to be partially developed with 311 existing EDUs and plans to add an additional 210 EDUs, resulting in a buildout EDU count of 521 EDUs.

Table 3-5 summarizes the specific plan information used to project flow from the on-going development areas.

Table 3-5 Specific Plan Land Use Information

Specific Plan Data	Homestead Development	Valley Glen	Parklane	Total
Single Family (EDUs)	1,245	837	521	2,603
Multi-Family (Acres)	10	-	-	10
Commercial/Public (Acres)	70	-	-	70
Industrial (Acres)	50	-		50
Non-Single-Family EDU Equivalent (1)	*635*			*635*
Development Status	*Completely Undeveloped*	*Partially Developed*	*Partially Developed*	
Existing EDUs	0	578	311	889
Remaining to be Developed	1,880	259	210	2,349

1) The non-single-family EDU equivalent converts the multi-family, commercial/public, and industrial development areas into the equivalent amount of single family homes or EDUs using the wastewater generation rates and densities described in **Chapter 4.0**.
2) Sources: Homestead Sanitary Sewer Report (July 2019), and Dixon Housing Element Update (Feb 2015), confirmed with City Staff.
3) This information may be updated in the final version of the City's General Plan 2040.

3.5 LEVEL OF DEVELOPMENT SCENARIOS

This SCSMP assesses collection system performance under existing development conditions and the following three future growth scenarios. The future development scenarios are cumulative and represent near-term, long-term, and build-out level of development planning. These scenarios and the associated future development areas added to each scenario are summarized in **Table 3-6**. The wastewater flow projections for each scenario are presented in **Section 6.4** of this SCSMP. Each level of development generally represents increments of five years of development for budgeting purposes. Actual implementation of improvements should be correlated with the actual amount of development occurring in the areas listed in the development areas added column of the table below.

Table 3-6 SCSMP Level of Development Scenarios

Level of Development	Model Description	Development Areas Added
Existing	Scenarios 1 – 3	• Existing Services
Near-Term	Scenario 4	• Service Area Infill • On-going/South
Long-Term	Scenario 5	• Northeast • North of I-80 (portion)
Build-Out	Scenario 6	• East • North of I-80 (remaining)

4.0 SANITARY SEWER FLOWS

The purpose of this chapter is to assess the existing wastewater flow within the collection system under current development conditions to establish current capacity and as a basis for projecting flows under future conditions. This chapter presents the basis of wastewater flow characterization, reviews the City's historical wastewater flow, summarizes the temporary collection system flow monitoring study, and present the results of a per-capita-flow analysis for comparison with the City's existing Design Standards.

This chapter is divided into the following sections:

- Wastewater Flow Characterization
- Historical WWTF Wastewater Flow
- Collection System Flow Monitoring
- Flow Per Capita Analysis

4.1 WASTEWATER FLOW CHARACTERIZATION

Wastewater collection systems are designed to convey peak wet weather flow (PWWF), which is characterized by three elements: Base sanitary flow, groundwater infiltration (GWI), and rainfall dependent inflow and infiltration (RDII).

Average Dry Weather Flow (ADWF) or Base Sanitary Flow

ADWF or base flow is the component of PWWF generated directly through sewer service connections; flow contributed directly by the public, residential, commercial, and industrial users. The ADWF is the average wastewater flow measured during periods of dry weather, without influence of GWI or RDII. In California, the ADWF is typically defined as the average flow recorded during the months of July, August, and September, when there is little rainfall.

Groundwater Infiltration (GWI)

GWI is groundwater that enters the collection system through cracks in sewer pipes and manholes, leaky joints, and damaged sewer lateral connections. GWI tends to vary seasonally depending on groundwater depth in relation to the depth of sewer pipelines. GWI has a higher impact on flow during the wet season when groundwater elevations are high. GWI is also more significant in sewers built in low-lying areas near creeks and drainages, where groundwater elevations may be high due to surface water conditions.

Rainfall Dependent Inflow and Infiltration (RDII)

RDII is flow that enters the collection system as a result of precipitation events. Inflow enters the sewer system directly often through leaky manhole covers, improperly connected roof leaders, and clean-outs. Infiltration is an indirect introduction of rainfall into the collection system through cracked sewer pipes, leaky joints, and manhole walls.

4.2 HISTORICAL WWTF WASTEWATER FLOW

The average daily influent flow measured at the City's WWTF from May 2003 through April 2020 is presented in **Figure 4-1** and a summary of historical WWTF influent flow data is presented as **Table 4-1**. The following flow parameters are used to describe influent flow at the WWTF.

WWTF Flow Parameters:

Average Annual Flow (AAF):	The annual average of the total flow per day.
Average Dry Weather Flow (ADWF):	The average flow over the dry weather season, considered to include June, July, and August.
Maximum/Peak Month Flow (MMF):	Maximum average monthly flow measured on an annual basis.
Maximum/Peak Day Flow (MDF):	Maximum average daily flow measured on an annual basis.
Peak Hour Flow (PHF):	The peak hour flow measured in recent years 2018 or 2019, since the WWTF upgrade project was completed in 2017. The peak hour flow assessment is presented in **Table 4-2**.

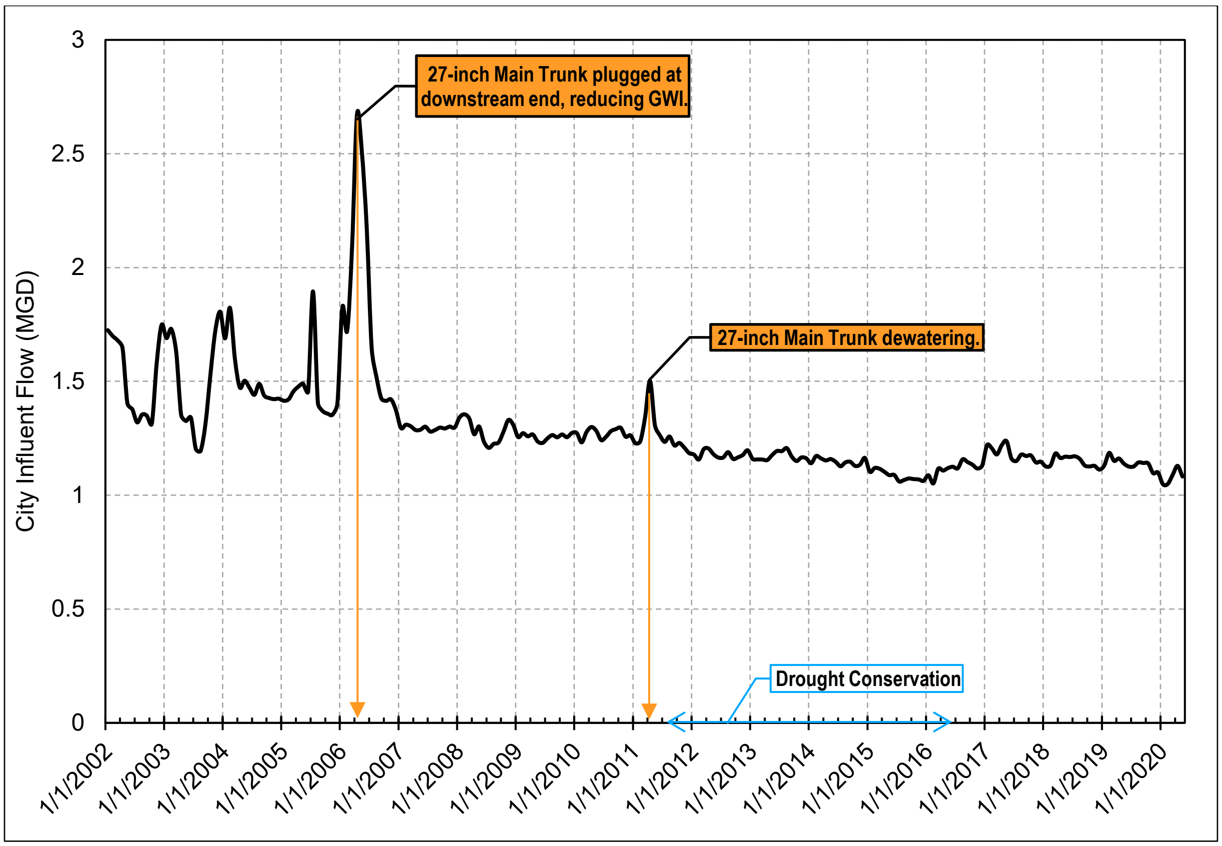


Figure 4-1 Historical WWTF Influent Flow

Table 4-1 City of Dixon WWTF Historical Flows

Year	AAF (MGD)	ADWF (MGD)	MMF (MGD)	MDF (MGD)	AAF/ADWF	MMF/AAF	MDF/AAF
2003	1.43	1.25	1.81	2.00	1.15	1.27	1.40
2004	1.52	1.47	1.82	2.26	1.03	1.20	1.49
2005	1.46	1.59	1.90	2.45	0.92	1.30	1.68
2006 (1)	1.82	1.78	2.67	3.16	1.02	1.47	1.74
2007	1.29	1.29	1.31	1.98	1.00	1.01	1.53
2008	1.29	1.22	1.36	1.97	1.05	1.05	1.53
2009	1.26	1.25	1.27	1.90	1.01	1.01	1.51
2010	1.27	1.26	1.30	1.71	1.01	1.02	1.35
2011	1.27	1.25	1.50	1.75	1.01	1.18	1.38
2012	1.18	1.17	1.20	1.51	1.00	1.02	1.28
2013	1.17	1.20	1.21	1.32	0.98	1.03	1.13
2014	1.15	1.14	1.17	1.69	1.01	1.02	1.48
2015	1.08	1.07	1.12	1.41	1.01	1.03	1.30
2016	1.12	1.13	1.16	1.45	0.99	1.04	1.30
2017 (2)	1.18	1.16	1.24	1.84	1.02	1.05	1.56
2018	1.15	1.17	1.18	1.40	0.98	1.03	1.22
2019	1.14	1.13	1.19	1.66	1.00	1.04	1.46

1) The 27-inch main trunk line was plugged and taken out of service on 04/06/2006, this significantly reduced GWI and PHF. (GWI was found to be significant if the local ground water level reaches 6-feet below the surface).
2) Construction of the WWTF Improvements Project was completed in 2017, which may have impacted flow quantity and/or data quality.

Influent flow at the City's WWTF recorded an ADWF of 1.13 MGD, an average annual flow (AAF) 1.14 MGD, and a maximum monthly flow (MMF) of 1.19 MGD in 2019. The average daily influent flow measured at the City's WWTF has decreased since 2002, this is due to a combination of factors impacting water resources in the region. Some of which may include the City's successful I/I reduction efforts in 2006, water conservation efforts in response to the period of drought between 2011 to 2015, and historical groundwater levels.

On April 4, 2006, the City decided to plug the 27-inch Main Trunk at the manhole closest to the main influent gate at the WWTF as an I/I reduction measure. Even though the 27-inch Main Trunk had already been plugged upstream at the manhole in the intersection of South 1st St. and Parkway Blvd. when the Parkway Blvd Trunk was constructed, approximately 2.5 miles of the 27-inch Main Trunk remained between the existing plug and the WWTF was found to be contributing excessive I/I to the WWTF. Operators at the WWTF estimated that between 200,000 and 400,000 gallons per day of GWI was contributed by the unused 2.5-mile section of pipe.

At the time that the City decided to plug the downstream end of the 27-inch Main Trunk, influent flows at the WWTF had been steadily increasing until they reached a high of 2.6 MGD. After the downstream end of the 27-inch Main Trunk was plugged, flows dropped about by about 200,000 gpd each day until returning to normal conditions. Routine manhole level checks revealed that manholes along the 27-inch Main Trunk were still filling with groundwater after it was plugged and isolated from the existing collection system. Measurements from groundwater wells in the area indicated that the groundwater level was still rising, reaching four feet three inches from the surface. It had appeared that groundwater was forcing itself into manholes and raising water in the manholes to higher elevations than the groundwater level. During this time, it was noted that groundwater levels had risen approximately two feet in one week.

After making this discovery the City decided that the only solution was to pull the plug at the downstream end of the Main Trunk to allow it to drain into the headworks. This added approximately 300,000 gpd to the influent flow on April 12, 2006 and 200,000 gpd thereafter until groundwater levels dropped significantly. Based on this event, the City concluded that when groundwater reaches less than 6-feet below the ground surface, excess GWI will occur in the lowest manholes in the system. It was also confirmed that the 2.5-mile section of the 27-inch Main Trunk leaks at approximately 200,000 gpd when submerged by groundwater.

The next large spike in flow shown in **Figure 4-1** occurred during the spring of 2011 and can also be attributed GWI in the 27-inch Main Trunk. The City staff reported that the old 27-inch Main Trunk line was still isolated from both ends from the existing collection system and in the spring of 2011 the groundwater elevation rose again to a level that allowed it to fill the trunk line. The City needed to dewater the trunk to avoid GWI surfacing at the downstream manholes. GWI from the 27-inch Main Trunk was dewatered into the 42-inch Main Trunk (parallel trunk line) conveying flow to the WWTF. The City has built up the rim elevations on some of the critical manholes on the 27-in trunk line and the pipes will be repaired before the line is brought back into service.

The PHF measured at the City's WWTF was assessed for the purposes of validating model calibration and identifying the peak influent flow at the WWTF. The results of PHF analysis were compared to those presented in the WWTF Facilities Plan Report (Stantec, 2014). The peak hourly influent flow criteria used for purposes of designing improvements to the WWTF headworks was a PHF/AAF ratio of 2.80.

Table 4-2 Dixon WWTF PHF Analysis

PHF Event	Date	AFF (MGD) 2019	Total Daily Flow (MGD)	PHF (MGD)	PHF/AAF	24-hr Rainfall Total (in) CIMIS #121
1	1/7/2019	1.14	1.42	2.67	2.35	1.84
2	1/16/2019	1.14	1.51	3.18	2.80	1.49
3 (1)	2/26/2019	1.14	1.53	2.45	2.16	2.17
4 (1)	12/1/2019	1.14	1.52	2.28	2.01	1.25
5	12/2/2019	1.14	1.67	2.28	2.00	0.97
6	1/8/2018	1.15	1.58	2.20	1.92	2.97

1) Event > 24-hours, total rainfall = 3.54 in over 2.3 days on 2/26; total rainfall = 2.51 inches over 2.0 days on 12/1.

Historical influent flow data provided by the City and hourly precipitation data available on the Department of Water Resources (DWR) California Irrigation Management Information Systems (CIMIS) website was used to identify peak rainfall events and the associated PHFs measured at the WWTF. The precipitation data used in the assessment was recorded at the Dixon Station (121). Time periods were selected corresponding to the largest influent flow measurements and 24-hour rainfall totals. The most recent event with the highest PHF/AAF ratio occurred on January 16, 2019. The 24-hour rainfall total equated to 1.49-inches and the PHF recorded at the WWTF was 3.18 MGD.

4.3 COLLECTION SYSTEM FLOW MONITORING

The City monitored flow at three locations within the collection system from February 7 to April 22, 2020. This work was conducted by V&A Consulting Engineers (V&A) and summarized in a technical report dated November 2020. Open channel flow monitoring was performed at three sites within the City's trunk sewer network. Flow monitoring was performed to provide baseline sanitary sewer flow data from specific locations within the collection system to allow calibration of the hydraulic model. Calibration allows the actual distribution of dry weather flows to be assessed as well as allowing a system specific distribution of wet weather flow. The flow monitoring report and inflow and infiltration study provided by V&A is included in **Appendix A** (*City of Dixon 2020 Flow Monitoring and Inflow/Infiltration Study, November 2020, V&A Consulting Engineers*).

4.3.1 Sewer-sheds & Monitoring Locations

To distribute flow within the hydraulic model, flows from flow monitors in series are estimated by subtracting flow from upstream sewer-sheds. The flow monitoring basin, or sewer-shed refers to localized areas of the sanitary sewer system upstream of each flow monitoring location. There are inherent errors introduced when subtracting flow monitors in series due to variations in data quality and travel time between monitors. Flow from sewer-shed 3 was subtracted from that recorded at sewer-shed 2 to estimate flow from this isolated sewer-shed. Flow recorded at sites 1 and 3 did not have upstream flow monitors and are considered to be representative of the entire upstream system and service area. The data for these three flow monitoring sites were used for model development and calibration.

The location of these flow monitors and associated sewer-sheds (flow monitoring sheds) are presented in **Figure 4-2** and described in **Table 4-3**.

Table 4-3 Flow Monitoring Shed Characteristics

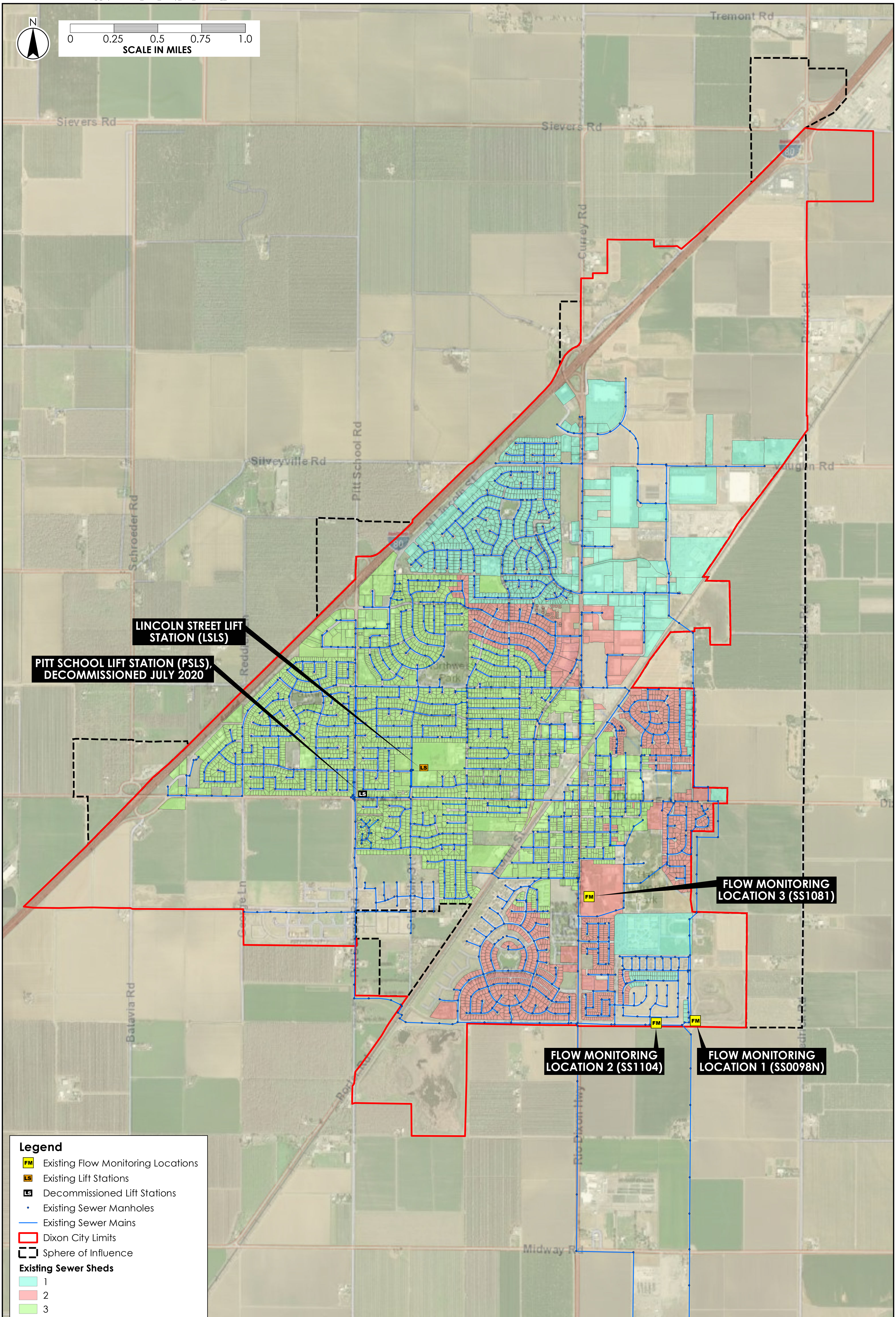
Flow Monitor	MH ID/ Location	Pipe Size (in)	Area (Acres)	Sewer-shed Description
1	• SS0098N *Yale Dr., 100 feet north of Parkway Blvd.*	30	427	Flow collected from the North Dixon Trunk, North Industrial Area, and Connemara Subdivision.
2	• SS1104 *Parkway Blvd., 900 feet west of Yale Dr.*	27	305	Flow collected from upstream of the Parklane Trunk (downstream of West Cherry St.) including Valley Glen, Collier Manner Subdivision, and flow conveyed through the North Interceptor Sewer.
3	• SS1081 *South 1st St., 525 feet south of W. Cherry St.*	15	713	Flow collected from the PSLS and LSLS sewer-sheds, the majority of the "old town" portion of the City, and residential flows from the Hillview Drive Area.

4.3.2 Rainfall Data

V&A installed rain gauges (RG) at three locations within the City to collect rainfall data during the 2020 flow monitoring period. RG North was located within Sewer-shed 1, RG Central was located within Sewer-shed 3, and RG South was located just outside of Sewer-shed 2. There were only two main rainfall events that occurred during the flow monitoring period, totaling approximately 2.8 inches of rain. A summary of the rainfall data collected for these events is presented as **Table 4-4**. Based on data presented by V&A, this amount of cumulative rainfall is between 28% and 39% of the typical historical precipitation totals for the duration of the flow monitoring period. Historically, between February 3 and April 23 the City accumulates approximately 8.10 inches of rainfall, almost three times the volume observed during the study.

Table 4-4 Rainfall Data Summary

Rain Gauge (RG)	Storm Event 1 *3/14 to 3/18* Total Rainfall (in)	Storm Event 2 *4/4 to 4/8* Total Rainfall (in)	Monitoring Period *2/7 to 4/22* Total Rainfall (in)
RG North	1.51	1.59	3.14
RG Central	1.06	1.20	2.30
RG South	1.70	1.13	2.91



Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 4-2
Flow Monitoring Locations

4.3.3 Flow Monitoring Data Summary

The relatively low amount of rainfall experienced during the flow monitoring period corresponded to a limited wet-weather flow response within the collection system. Without a sufficient wet-weather response within the collection system flow monitoring data, wet-weather model calibration had to be completed using historically observed influent flow data recorded at the WWTF. Despite the limited response to wet-weather, V&A provided an analysis of I/I within the collection system. The results of V&A's analysis are summarized in **Table 4-5**.

Table 4-5 V&A Flow Monitoring and I/I Analysis Summary

Flow Monitoring Site	V&A's ADWF (1) (MGD)	Peak Measured Flow (MGD)	Peaking Factor	Max Depth/ Diameter Ratio	Total I/I per ADWF (2) (MG/MGD) *1-inch of Rainfall*	I/I Ranking (3)
1	0.273	0.57	2.1	0.36	0.055	2
2	0.200	1.56	2.0	0.24	0.162	1
3	0.577	2.07	3.6	0.68	0.048	3
Total:	**1.050**					

1) ADWF values are representative of ADWF conditions prior to the SIP order. ADWF at Site 2 has been adjusted to remove flow recorded at upstream Site 3, the total ADWF measured at Site 2 was 0.777 MGD.
2) This value represents the total volume of I/I that enters the collection system per 1 MGD of ADWF, corresponding to 1-inch of rainfall over the sewer-shed.
3) A ranking of "1" represents the highest amount of I/I observed after normalization.

V&A differentiated I/I flow from ADWF to determine which components of I/I were more prevalent in each sewer-shed. After separating flow components, the I/I analysis metrics were normalized for an "apples-to-apples" comparison of each flow monitoring shed. Flows were normalized per-ADWF, and sewer-shed 2 had the highest normalized total I/I rate. In addition to information presented by V&A, the City should consider the historically observed GWI infiltration rates and consider additional flow monitoring to further refine and isolate additional sources of I/I within the system.

A summary of the results of the flow monitoring study is provided as **Table 4-5**. Additional information is provided in V&A's full Technical Report included in **Appendix A**.

4.3.4 Data Quality & Monitoring Period

In the case of the 2020 City of Dixon flow monitoring study, the flow depth and velocity readings measured at flow monitoring sites 1 and 2 were impacted by a downstream fat, oil, & grease (FOG) blockage that existed downstream of the confluence of the Parklane and North Dixon Trunks, in the upstream portion of the 42-inch Main Trunk. This unknown blockage resulted in a flow data that was inconsistent with the volumetric flow recorded at the WWTF. After consulting with the City and WWTF operations staff, the manholes were inspected, and the blockage was identified. V&A adjusted the flow monitoring data to reflect the reduced hydraulic capacity associated with the downstream FOG blockage.

The quality of the adjusted flow monitoring data was considered acceptable for use as a foundation in developing the hydraulic model of the City's collection system, but it is recommended that the City conduct additional flow monitoring data at more locations within the system to recalibrate the existing system model if it is to be used for more detailed purposes. The accuracy and quality of the hydraulic model results and subsequent recommendations depend on the to the quality of the 2020 flow monitoring data from which it was built.

The following impacts to data quality should be considered with the results of the 2020 flow monitoring study and model evaluations presented in this SCSMP.

- Unideal flow conditions during the flow monitoring period – FOG blockage
- Changes in daily flow pattern during the flow monitoring period – SIP order
- Insignificant wet-weather flow response during the flow monitoring period – Lower than average rainfall
- Major system routing changes that have occurred since the flow monitoring period – East West Trunk Sewer/ PSLS decommissioned

The impacts of the FOG blockage created flow conditions that are not ideal for flow monitoring and incur a higher level of error in the associated flow data. Ideal flow conditions are generally defined as laminar flow in a straight-through, constant-slope pipeline with no disturbances ten diameters upstream and five diameters downstream from the flow monitoring location. If ideal flow conditions are met, an expected uncertainty of final flow calculation from an open-channel flow meter may be approximately ±5%.

In addition to data quality concerns, the state of California issued a mandatory, statewide, shelter-in-place order on March 19, 2020, in response to the COVID-19 outbreak. The mandate shut down non-essential services and directed 40 million residents to generally stay at home and avoid social gatherings. The flow monitoring data shows evidence that the mandate altered the daily life for people in the City with an abrupt change in the daily flow pattern observed. This abrupt change in lifestyle impacted the amount of time that people spend in various parts of the City. The overall ADWF increased approximately 8%, but peak flows remained approximately the same. This attenuation and increase in ADWF is likely due to a decrease in the number of people preparing for work in the morning, and less commuters leaving the City to jobs outside the City during the day.

Therefore, collecting additional flow monitoring to validate the results of the hydraulic model and subsequent CIP is recommended. This is also recommended to account for the significant changes to flow routing that have since occurred within the system when the East West Connector sewer was brought online and the PSLS was decommissioned. The removal of the pump station and redirection of flow within the system should be monitored and the existing system should be updated within the model to reflect these impacts.

The objectives of collecting additional flow monitoring *data and associated details are summarized below:*

- Validate flow data collected in the 2020 V&A study:
  - Adjust locations slightly to measure flow under more ideal flow conditions.
- Validate the typical daily flow pattern used in the model, and confirm that this pattern is representative of typical post-pandemic daily flow pattern,
  - Measure the daily pattern over a period that is representative of typical conditions and can be used for long-range planning.
- Determine the local peak wet-weather flow response within each sewer-shed and overall distribution of I/I within the collection system,
  - Measure flow over the typical wet-weather season in hopes of collecting data over more significant rainfall events. Monitor groundwater in addition to flow under high groundwater conditions.
- Validate system routing and flow distribution related to connecting the East West Trunk Sewer and decommissioning the PSLS,
  - Measure flow at key locations considering major system flow routing changes.

4.4 FLOW PER CAPITA ANALYSIS

Historical flow and population data were used to calculate the typical wastewater flow per capita in the City of Dixon. The analysis uses the annual ADWF recorded at the WWTF and historical population data to calculate the annual average per capita wastewater flow. The historical per capita flow, or generation rate was compared to the standard design values presented in the City's Design Standards. The results of this analysis were used as a basis for identifying appropriate wastewater generation rates to project flow generated from future development areas. The wastewater generation rates used in this SCSMP are further discussed in **Section 5.1**.

Historical influent flow data from the WWTF Facilities Plan Report (Stantec, 2014) and the WWTF Improvements Project Design Report (Stantec, 2013) were used along with more recent data provided by the City. The population data used in this analysis was described in **Section 2.2**. The City's housing element and General Plan identify a standard development density of approximately 3.7 persons per household, or persons per EDU. This standard density was used to translate the per capita flow into the standard wastewater unit rate per EDU. The results of the evaluation of the wastewater flow per capita analysis are presented in **Table 4-6**.

As discussed, wastewater flow recorded at the WWTF has been declining over the past 20 years reflecting the City's I/I reduction efforts, historical groundwater levels, drought water conservation measures and other considerations. This declining trend can also be observed in the City's historical per capita flow rate.

After the 27-inch Main Trunk was isolated from the collection system in 2006 the per capita wastewater flow dropped sharply from approximately 90 gpcd to 70 gpcd. The per capita flow stabilizes around 70 gpcd between 2007 and 2011, until the City's water conservation measures took effect in 2012 in response to drought conditions. Between 2012 and 2015 the per capita flow continues to drop, going from approximately 65 gpcd in 2012, to less than 60 gpcd in 2015. The data from 2015 to 2019 appears to stabilize around approximately 60 gpcd.

Overall, the historical trend in per capita flow within the City has been declining over the past 15-years. The City's average per capita flow calculated from 2005 to 2019 is approximately 68 gpcd. Wastewater flow per capita in California typically ranges from 65-85 gpcd but has been declining due to drought conditions and water conservation efforts implemented throughout the State in recent years. Multiplying the average per capita flow by the City's standard planning density of 3.7 persons/EDU, gives a wastewater unit rate of approximately 250 gpd/EDU. This wastewater unit rate is 100 gpd/EDU less than what is currently in the City's Design Standards. However, it appears to be a reasonable estimate of wastewater flow from similar sized communities in this region. (City of Woodlake at 232 gpd/EDU, City of Lincoln at 250 gpd/EDU, City of Benicia at 225 gpd/EDU, City of Merced at 257 gpd/EDU, City of Lodi at 211 gpd/EDU).

The resulting unit rate of 250 gpd/EDU is used as a basis for projecting flow from future development areas in this SCSMP to reflect historically observed conditions. The following chapter, **Chapter 5.0**, presents a summary of the City's current sanitary sewer Design Standards.

Table 4-6 Historical Flow, Population, and per Capita Flow Analysis

Year	AAF (MGD) (1)	ADWF (MGD) (1)	Population (2)	Per Capita Flow (gpcd) (3)
2003	1.43	1.25		
2004	1.52	1.47		
2005	1.46	1.59 (4)	17,449	91
2006	1.82	1.78	17,914	99
2007	1.29	1.29	18,105	71
2008	1.29	1.22	18,148	67
2009	1.26	1.25	18,293	68
2010	1.27	1.26	18,441	68
2011	1.27	1.25	18,293	68
2012	1.18	1.17	18,388	64
2013	1.17	1.20	18,525	65
2014	1.15	1.14	18,986	60
2015	1.08	1.07	19,080	56
2016	1.12	1.13	19,229	59
2017	1.18	1.16	19,485	60
2018	1.15	1.17	19,686	59
2019	1.14	1.13	19,920	57
Wastewater Generation Rate Analysis, (gpd per EDU)				**Value**
			Average Per Capita Flow (gpcd):	68
			Persons/EDU:	3.7
			Average (gpd/EDU):	250
			Design Standard (gpd/EDU):	350
			Difference:	-100

1) Flow data from WWTF Facilities Plan Report (Stantec, 2014)
2) Population Data from City of Dixon Water Master Plan (West Yost, 2018) Source: CDOF, E-4 Population Estimates for Cities, Counties, and the State, 2001-2010, with 2000 and 2010 Census Counts, November 2012. CDOF, E-4 Population Estimates for Cities, Counties, and the State, 2011-2020, with 2010 Benchmark.
3) The per capita flow is calculated as the annual ADWF/population.
4) The 2005 ADWF is higher than the 2005 AAF due to an unusually high average monthly flow of 1.90 MGD record in July 2005. Likely due to high GWI described in 2006.

5.0 PLANNING & DESIGN CRITERIA

The purpose of this chapter is to define the planning and design criteria for analyzing the performance of the City's sewer collection system. This chapter presents a summary of the City's sanitary sewer design standards and review. It also presents the assumptions used to approximate future wastewater flow from areas within the planning horizon of their General Plan 2040, as well as the capacity evaluation criteria used to assess the model simulation results.

The system performance and facility sizing criteria used for this SCSMP are based on the City of Dixon Engineering Design Standards for Sanitary Sewer Design (August 2014, pages DS6.1 to DS6.5) and the design exceptions approved by the City in existing sewer system planning documents. It should be noted that the City updated the design standards in 2022 to reflect the recommendations of this report.

This chapter is divided into the following sections:

- Design Flow Criteria
- Facility Sizing
- Collection System Performance Criteria
- Planning Scenarios & Projected Wastewater Flow

5.1 DESIGN FLOW CRITERIA

Stantec reviewed the design flow criteria of similar sized communities in the region to assess how the City's criteria compares with other agencies and municipalities in the general vicinity to determine if the City should update their criteria. The City's criteria were found to be more conservative than other nearby agencies and the historical per-capita flow analysis discussed in **Section 4.4** of this SCSMP.

5.1.1 City Design Standards – Flow Methodology

The City's wastewater design flow criteria are presented in their Design Standards, which outlines the required method for approximating design sanitary sewer flow, pipe capacity, velocity, and sewer main sizing procedures. The City's design flow projection methodology is defined by the City Standard DS6-03. This standard outlines the methodology and unit rates that should be used in calculating the projected flow from future development areas. The future flow projection is to be used to design collection system needs for a development area.

The design flow is calculated by first determining the average daily flow, peaking factor, and inflow and infiltration (I/I) factor based on the land use designation and size of the proposed development. Once these parameters are determined the design flow can be calculated by multiplying the average flow by the peaking factor to get the resulting Peak Flow (Q_p) and adding the I/I factor to determine total PWWF design flow (Q_d). City Standard DS6-03 for design flow is summarized below.

The design sewer flow in gallons per day (gpd) shall be calculated with the following formula:

$$Q_d = Q_p + I/I$$

Where,

Q_d = Design Flow
Q_p = Peak Flow =Average Daily Flow X Peaking Factor
I/I = Inflow & Infiltration Factor

The average daily flow or Average Dry Weather Flow (ADWF) was determined using land use-based unit flow factors, also referred to as wastewater generation rates. ADWF projections for infill and future development areas are calculated as the product of the unit flow factors and the net area of the development or the number of EDUs described in the specific land use plan. The net area of the development is defined as 80% of the total gross development area. An EDU is a unit of measure that normalizes all land use types to the equivalent wastewater demand of one single-family residential unit.

Unit flow factors provided in Section 6 of the City's design standards as shown in **Table 5-1**. These values were adjusted for purposes of this Master Plan based on the per-capita flow analysis presented in **Section 4.3.4**, which resulted in reducing these values by approximately 30% to reflect current conditions.

Table 5-1 Wastewater Unit Flow Factors – City Design Standards (1)

Land Use	Average Daily Unit Flow	I+I Factor
Single-Family	350 gpd per EDU (2)	500 gpd per gross acre (4)
Multi-Family	5,000 gpd per net acre (3)	500 gpd per gross acre
Commercial/Public	1,500 gpd per net acre	500 gpd per gross acre
Industrial	2,000 gpd per net acre	500 gpd per gross acre
Schools	5,000 gpd per net acre	500 gpd per gross acre

1) This SCSMP uses adjusted wastewater unit flow factors developed based on a historical per capita wastewater flow evaluation. The evaluation of historical data gave an average unit rate of 250 gpd/EDU between 2005 and 2019. The values presented in this table were scaled by the ratio of historical and design unit rates to give the unit flow factors used in this SCSMP (250/350 ~ 70%).
2) EDU = An EDU is a unit of measure that normalizes all land use types to the equivalent wastewater demand of one single-family residential unit.
3) Net acre = Net acres is assumed to be 80% of gross acres.
4) Gross acre = Total acreage of the proposed development area.
5) These design standards were updated in 2022 to reflect the recommendations of this report.

The methodology outlined in the City's design standards requires that a peaking factor be used to convert the average daily flow to the peak daily flow. The peak daily flow or peak ADWF can be described as the peak flow experienced throughout the day as flow fluctuates without I/I, this is also referred to as the diurnal peak. Peaking factors, as presented in the City's Design Standards, are determined by the total size of the development, and summarized in **Table 5-2**.

Table 5-2 Peaking Factors – City Design Standards

Shed Area	Peaking Factor
Shed Area < 500 acres	2.5
500 acres ≤ Shed Area ≤ 1,500 acres	2.2
Shed Area > 1,500 acres	2.1

The City's Design Standards define the PWWF or Design Flow (Q_d) used for design purposes as the sum of the peak flow (Q_p) and the I/I flow factor. The I/I flow factors are determined by multiplying the gross area by the I/I unit rate, 500 gpd/acre, as shown in **Table 5-1**.

5.1.2 SCSMP – Flow Methodology

This SCSMP uses a hydraulic model to simulate peak flows in the collection system under existing and future development conditions. The methodology used by the model to determine the PWWF or the Design Flow (Q_d) defined in the City's Design Standards, can be broken out into the following equivalent components:

- Average Daily Flow: ADWF is distributed throughout the collection system model based on service area. Existing wastewater flows are determined from flow monitoring data and future wastewater flow projections are determined using unit flow factors and land use information.
- Peak Flow (Q_p): Flow pattern multipliers are calculated and applied to the dynamic model to determine the Peak Flow (Q_p) or peak daily flow as opposed to applying a peaking factor.
- Inflow and Infiltration Factor (I/I): RTK unit hydrographs are used to determine the portion of rainfall entering the collection system as inflow and infiltration or RDII during a 10-year, 24-hour design storm event.

The existing system model is calibrated to observed conditions in the collection system using flow monitoring data. The ADWF distributed throughout the model is equivalent to the Average Daily Flow defined by the City's Design Standards. It is calculated for future development areas using the same unit factor methodology presented in the City's Design Standards using the updated wastewater generation rates (250 gpd/EDU) as discussed in following section.

A diurnal flow pattern describes the variation in wastewater flow occurring over the course of a full day. Wastewater flow can vary significantly over a 24-hour period, with maximum flow typically occurring in the morning and early evening. During the flow monitoring study conducted by V&A for purposes of this SCSMP, flow was measured over 15-minute intervals. This monitoring provided detailed data allowing the City to evaluate these daily patterns at locations throughout its service area. Each area of the City has its own unique pattern, which also varies between weekdays and weekends.

Diurnal flow patterns are applied within the hydraulic model to determine the peak daily flow, which is equivalent to the Peak Flow (Q_p) defined by the City's Design Standards. The model incorporates the average daily flow pattern multipliers to represent hourly and daily flow variations. These patterns are calculated based on the observed data at each flow monitoring location and include the hourly flow variation on weekdays, the hourly flow variation on weekends, and the daily flow variations that occur throughout the week.

The RDII component of PWWF simulated in the model is equivalent to the Inflow and Infiltration Factor (I/I) defined by the City's Design Standards. The model projects the RDII in the system using calibrated unit hydrographs. The calibrated parameters determine the portion and rate of rainfall entering the collection system. After calibration, the observed rainfall data is exchanged for a design rainfall event representing a reasonable worst-case wet-weather flow condition. The design storm used to evaluate the system in this SCSMP has a statistical 10-year return frequency and 24-hour duration. The model results equate to PWWF conditions in the collection system used for the system evaluation.

5.1.3 Wastewater Generation Rates

Land use-based wastewater generation rates were used to project dry weather flow from future development areas in this SCSMP. The City's standard wastewater generation rates are outlined in the City's Design Standards for each general land use designation, as shown in **Table 5-1**. These values were reviewed and updated based on the assessment of historical per capita wastewater flow described in **Section 4.3.4** of this SCSMP. The City's standard wastewater generation rates were assigned to each land use designation presented in the City's General Plan 2040.

The City's existing Design Standards define the standard wastewater unit rate per EDU as 350 gpd, which equates to a per capita flow of approximately 95 gpcd, assuming a standard unit density of 3.7 persons/EDU. The per capita flow analysis described in **Section 4.3.4**, resulted in a historical average per capita flow of approximately 68 gpcd between 2005 and 2019. Multiplying this value by the City's standard unit density of 3.7 persons/EDU, gives a lower wastewater unit rate per EDU which equates to approximately 250 gpd. Therefore, this Master Plan uses a wastewater unit rate of 250 gpd per EDU to reflect these historical conditions, 100 gpd less than the City's standard wastewater unit rate.

The City's standard land use-based wastewater generation rates are presented in in **Table 5-1**. These values were scaled by the ratio of the historical unit rate and the standard design unit rate to give the unit flow factors used in this SCSMP. The wastewater generation rates were reduced by approximately 30% from those presented in the City's Design Standards to reflect the same relative decrease in unit flows.

The general land use descriptions and the associated wastewater generation rates shown in **Table 5-1** were correlated to each of the more specific land use designations described in the City's General Plan. For example, it was assumed that all "commercial" land use designations included in the City's General Plan (neighborhood, regional, and service commercial) correlate to the Commercial/Public wastewater unit rate presented in the Design Standards (shown in **Table 5-1**). Industrial and school land use designations correlated directly with their respective wastewater generation rates.

Residential land use designations are defined differently in the General Plan than in the City's Design Standards. The City of Dixon Housing Element (2015) was reviewed to determine the development density of each General Plan 2040 land use designation. The development density is defined as the number of equivalent dwelling units (EDUs) per acre and is used to convert the residential unit rate into per acre wastewater generation rates for each specific residential land use designation. The Housing Element development densities and those used in this SCSMP are summarized in **Table 5-3**.

Table 5-3 2015 Housing Element Land Use Densities

2015 Housing Element		Density (EDU/Acre)			
Land Use Designation	**Label**	**Low**	**High**	**Average**	**SCSMP (1)**
Very Low Density	VLD	1.64	2.20	1.92	**3.1**
Low Density	LD	2.19	6.22	4.21	
Medium Density (Low)	MDL	6.23	14.52	10.38	**14.2**
Medium Density (High)	MDH	14.30	21.78	18.04	
High Density	HD	21.78	29.04	25.41	**20.0**
Mixed Use	PMU	6.30	21.78	14.04	**12.0**
Planned Multiple Residential District	PMR	10.00	10.00	10.00	
Agriculture	A	0.40	0.40	0.40	**0.4**

1) The value in this column represents the development density used to determine the wastewater generation rate on a per-acre basis for each land use designation. Although future development areas do not include High Density Residential planning areas, a density of 20.0 EDU/acre is recommended for future planning purposes.

The updated wastewater unit rate (250 gpd/EDU) along with the development densities presented in the City's 2015 Housing Element were used to determine the approximate flow per acre from residential, mixed use, and agricultural development areas. The wastewater generation rates for industrial and commercial land use designations were scaled proportionally to the reduction in flow-per-EDU.

The development density used for each residential land use designation was taken as the average of the density range presented in the City's Housing Element. Although the General Plan 2040 does not include any high-density development areas, a density of 20.0 EDU/acre is recommended for future planning purposes. Using the housing element density of 25.4 EDU/acre may result in an overly conservative flow estimate as high-density EDUs are typically observed to have lower unit rates (<250 gpd/EDU).

The Housing Element presents a density of 0.4 EDU/acre for agricultural areas. Campus, corridor, and downtown mixed-use designations were assumed to include a mix of both residential and commercial development and were assumed to have a unit density of approximately 12 EDU/acres. Public facilities and parks were assumed to consist of primarily open space and have no wastewater contribution.

The resulting wastewater generation rates used in this SCSMP and those presented in the City's Design Standards are summarized in **Table 5-4**. The wastewater generation rates used in this SCSMP are approximately 70% of those presented in the Design Standards.

Table 5-4 Wastewater Generation Rate Assessment

UNIT RATE (gpd/EDU):		***350***	***250***
Land Use Designation	**Density (1) (EDU/acre)**	**Design Standard Wastewater Generation Rate (gpd/acre)**	**Scaled SCSMP Wastewater Generation Rate (gpd/acre)**
Open Space			
Agricultural	0.40	140	100
Parks	-	0	0
Mixed Use			
Campus Mixed Use	12.0	4,200	3,000
Corridor Mixed Use	12.0	4,200	3,000
Downtown Mixed Use	12.0	4,200	3,000
Residential			
Low Density Residential	3.1	1,100	800
Medium Density Residential	14.2	5,000	3,600
High Density Residential	20.0	7,000	5,000
Commercial			
Neighborhood Commercial	-	1,500	1,100
Regional Commercial	-	1,500	1,100
Service Commercial	-	1,500	1,100
Industrial			
Industrial	-	2,000	1,400
Public			
Public Facilities	-	0	0
Schools	-	5,000	3,600

1) The City of Dixon Housing Element was reviewed to determine the development density of each General Plan 2040 land use designation, the density was assumed to be the approximate mid-point in the range of densities presented.

For purposes of this SCSMP the unit rate of 250 gpd/EDU was used to project flow generated by future development areas based on the per-capita flow analysis described in **Section 4.3.4**.

5.2 FACILITY SIZING

The following provides a description of the criteria used to size future system improvements in relation to the City's existing Design Standards and planning exceptions.

5.2.1 Sewer Main Design Criteria

City's Design Standards outline the basic design criteria for gravity sewers. The primary criteria can be summarized by the following, each of which is further described below:

- Minimum pipe size of 8-inches.
- Maximum flow capacity of 70% (peak flow/full pipe flow = 70%) using manning's equation.
- Minimum flow velocity of 2 fps (when flowing full), for sewers that will exceed 50% capacity at buildout.
- Minimum flow velocity of 2.5 fps (when flowing full), for sewers that will not exceed 50% capacity at buildout.

The City's Design Standards outline typical pipe capacity requirements for sewer design as the following:

> Typically, sewer mains shall be sized based upon the sewer flowing at 70% of pipe capacity using the following formula:
>
> **Manning's Formula**: $Q = A\ (1.49/n)\ (R^{2/3})\ (S^{1/2})$
>
> **Where,**
>
> **Q = Flow, in cubic feet per second (cfs)**
> **A = Area of the pipe cross section in square feet (sf)**
> **R = Hydraulic Radius (Area/ Wetted Perimeter)**
> **S = Slope of Pipe**
> **n = Roughness of 0.013 or as recommended by the pipe manufacturer whichever is greater.**
>
> Pipe capacity, in all cases, shall be adequate to carry the design flow from the entire tributary area, even though said tributary area is not located within the project boundaries. Sewer trunk line design criteria shall be done on a case by case basis as approved by the City engineer.

5.2.2 Design Exceptions

Recent sewer collection system planning documents and as-built record drawings for on-going and recent developments were provided by the City for review and consideration in the development of this SCSMP. As discussed in the provided sewer system planning documents, the following design variances have historically been accepted by the City in sewer system planning. These exemptions are outlined in the "Master Sewer Study – Valley Glen South Area (April 2015)" prepared by Wood Rodgers and included in **Appendix B**.

5.2.3 Collector Pipe Slope Design Exception

City design standards require that sewers that do not achieve 50% capacity Hydraulic Loading Ratio (HLR) under buildout conditions be designed to a flow velocity of 2.5 fps. Using Manning's formula, this would require that an 8-inch sewer have a minimum slope of 0.0052 ft/ft. Due to the relatively flat topography that exists within the City, meeting this slope requirement limits the extent of the service area to be served by gravity.

A slope of 0.0035 ft/ft for 8-inch sewers, resulting in an approximate full flow velocity of 2.0 fps, has historically been agreed upon for sewers where 50% capacity will not be achieved at buildout with the exception that the most upstream "dead end" sewer must still be designed to a flow velocity of 2.5 fps.

This decision was agreed upon (as cited in Section 5.2.2) in the name of expanding gravity flow potential in the service area and minimizing the number of lift stations in the collection system. This design exception was used in this SCSMP where necessary to expand the gravity service potential of the buildout service area.

The most upstream sewers, of those proposed to expand the collection system under future development scenarios in this SCSMP, are planned to be constructed at the slope required to achieve a minimum flow velocity of 2.5 fps, when an HLR of less than 50% is predicted by the model. Sewers predicted to have an HLR greater than 50% under buildout conditions are planned to be constructed at a slope greater than or equal to that required to achieve a minimum flow velocity of 2.0 fps. Steeper slopes corresponding to higher flow velocities were preferred and used where sufficient depth exists.

A summary of the standard slopes used to layout the future collection system is provided as **Table 5-5**. These values were determined using Manning's formula for full pipe flow with a friction constant of n = 0.013 for pipe roughness.

Table 5-5 Minimum Sewer Slope Criteria

Pipe Size (in)	Min. Slope (ft/ft), v = 2.5 fps	Min. Slope (ft/ft), v = 2.0 fps
8	0.0052	0.0035
10	0.0039	0.0025
12	0.0030	0.0020
15	0.0022	0.0015
18	0.0018	0.0011
21	0.0014	0.0009
24	0.0012	0.0008
27	0.0010	0.0007
30	0.0009	0.0006

1) These values are determined using Manning's formula with a friction constant of n = 0.013.
2) Minimum flow velocity of 2 fps (when flowing full), for sewers that will exceed 50% capacity at buildout.
3) Minimum flow velocity of 2.5 fps (when flowing full), for sewers that will NOT exceed 50% capacity at buildout.
4) To extend the gravity service area a minimum flow velocity of 2.0 fps may be used for sewers that will NOT exceed 50% capacity at buildout, as long as the most upstream sewers remain at the slope required to achieve a flow velocity of 2.5 fps.

5.2.4 House Service Design Exception

The vertical constraints imposed on the collection system due to the relatively flat topography and shallow stormwater drainage system within the City has historically posed pipe cover problems when attempting to maximize gravity sewer service, even when implementing the collector pipe slope design exception described above. An alternate shallow service detail has been agreed upon by the City, where the minimum cover over the service lateral, as measured from finish grade at the back of the sidewalk, could be reduced to 36-inches. This detail is shown in the "Master Sewer Study – Valley Glen South Area (April 2015)" prepared by Wood Rodgers and included as **Appendix B.**

5.3 COLLECTION SYSTEM PERFORMANCE CRITERIA

The purpose of this section is to present the capacity evaluation criteria is used to assess the collection system hydraulic model simulation results. The 10-year, 24-hour, design storm was applied to the model to simulated PWWF and evaluate the systems level of service (LOS) performance in meeting the following criteria.

Capacity evaluation criteria used in the model assessment include:

- Level of manhole surcharging
- Sewer flow velocity
- Pipe capacity

Separate LOS criteria are defined for existing system improvements and new infrastructure proposed to provide service to future development areas.

5.3.1 Hydraulic Loading Ratio (HLR)

Collection system performance is assessed based upon the hydraulic loading ratio (HLR) within each sewer under dry and peak wet weather conditions. The HLR is a commonly used metric in evaluating the capacity and performance of the collection system. The HLR is mathematically defined as the peak modeled flow divided by the full pipe capacity derived from Manning's Formula.

The existing level of development model was evaluated under dry weather conditions and wet weather conditions. Under dry weather flow conditions, sewers with a HLR of 50% or less is considered to meet LOS criteria. Sewers having HLRs greater than 50% indicates that there may not be capacity for flow under peak conditions and sewers should be considered for improvement.

The remaining model simulations evaluate the level of service within the system under PWWF conditions. Improvements identified under PWWF conditions are proposed based on a HLR nearing 100 percent in existing sewers and 70% in new sewers.

LOS criteria used to evaluate HLR in gravity sewer lines are presented in **Table 5-6**.

Table 5-6 Allowable Hydraulic Loading Ratio (HLR)

Improvement Area	Flow Conditions	Acceptable HLR (Max Flow/Full Flow)
Existing Collection System	Average Dry Weather	Less than 50%
Existing Collection System (Existing System Improvements)	Peak Wet Weather	Less than or equal to 100%
Future Collection System (Proposed Infrastructure)	Peak Wet Weather	Less than or equal to 70%

5.3.2 Residual Capacity

The residual capacity is the remaining capacity within a sewer when subjected to PWWF conditions. The residual capacity is mathematically defined as Manning's full pipe flow capacity minus the peak modeled flow. This performance indicator is useful for illustrating the relative remaining capacity throughout the collection system for use in evaluating future servicing strategies.

5.3.3 Velocity

The flow velocity within the collection system shall be considered in planning future system improvements and reviewed to identify O&M needs within the existing collection system. Existing system improvements will not be recommended based on minimum and maximum velocity parameters. Gravity trunk sewers are typically designed to maintain a minimum flow velocity of 2.0 fps when flowing full, with a maximum velocity of 10 fps. Sewers which will exceed 50% of their capacity under build-out conditions shall have their minimum design slope determined using a minimum velocity flowing full of 2.0 fps. Sewers which will NOT exceed 50% capacity under build-out conditions shall have their minimum design slope determined using a minimum velocity flowing full of 2.5 fps. Additional discussion of slope and velocity requirements for future system improvements is provided in **Section 5.2** of this SCSMP.

5.3.4 Surcharging

Freeboard in a manhole is defined as the distance between the rim elevation and the hydraulic grade line (HGL). Surcharging occurs when the HGL exceeds the pipe crown elevation. The maximum allowable surcharge in the gravity portion of the existing collection system is 1-foot. Manholes in the existing system must also maintain at least 10-feet of freeboard during a design storm event. No surcharging will be allowed in existing manholes under 10-feet deep or for new manholes proposed under future development scenarios. Proposed sewer improvements and new sewers are designed to have no surcharging under peak design flow conditions.

5.3.5 Depth Over Diameter Ratio (d/D)

The maximum sewer depth under PWWF conditions is an important factor in understanding capacity limitations of a collection system. The d/D ratio is the peak modeled depth of flow (d) divided by the pipe diameter (D). Typical LOS criteria allow a maximum d/D ratio of 0.70-0.85.

5.3.6 Force Mains and Lift Stations

This criterion compares peak inflow to the lift station to the reliable pumping capacity of the lift station to identify potential capacity constraints. The reliable capacity of the lift station is defined as the pumping capacity with the largest pump out of service. Force mains shall be designed to have a minimum flow velocity of 4 fps and a maximum flow velocity of 10 fps under the full range of pumping conditions.

5.4 PLANNING SCENARIOS & PROJECTED WASTEWATER FLOW

The level of development scenarios and the associated dry weather wastewater flow projections simulated in the hydraulic model and are summarized below.

5.4.1 Existing Level of Development

The existing level of development scenarios represent the model iterations of the existing system and existing level of development condition.

Scenario 1 – Existing Dry Weather Flow

This modeled scenario simulates the existing collection system under dry weather flow conditions (ADWF = 1.09 MGD, PDWF = 1.76 MGD), calibrated to flow data from the flow monitoring study. Dry weather flow is distributed in the model within each respective sewer-shed. Although recent groundwater elevation data was not considered in the development of this SCSMP, historical observations related to the 27-inch Main Trunk indicate that groundwater levels were relatively low during the flow monitoring period. Therefore, the dry weather flow component is not considered to be impacted by significant GWI.

It should also be noted that the flow monitoring period occurred prior to the decommissioning of the PSLS and connection of the East West Sewer Connector, although this condition is simulated in Scenario 3.

Scenario 2 – Existing Wet Weather Flow

This modeled scenario simulates flow in the existing collection system during a 10-year, 24-hour design storm event (PWWF = 4.30 MGD). This model was constructed using the calibrated wet-weather flow model and changing the simulated rainfall condition to represent the design storm. The results of this simulation evaluate the existing collection system under PWWF conditions.

Scenario 3 – Existing Wet Weather Flow (without PSLS)

This model scenario is the existing wet weather flow model evaluated in Scenario 2 (PWWF = 4.30 MGD), but infrastructure represented in the model is adjusted to reflect the decommissioning of the PSLS and incorporation of the new East West Trunk Sewer Connector. No additional flow was added to the model in this scenario, only flow routing and physical infrastructure parameters are modified. The results of this simulation evaluate the existing collection under PWWF conditions with recent system improvements that include the East West Trunk Sewer Connector and decommissioning of the PSLS.

5.4.2 Future Levels of Development

Each scenario in the sequence is built from preceding scenarios by cumulatively adding flow from future development areas. The three levels of development include near-term, long-term, and build-out development phases of the City's General Plan 2040. The scenarios add flow from future developments and include new infrastructure required to expand the service area and convey flow from future development areas to the existing system. All future scenarios include the East West Trunk Sewer Collector.

Each model simulates PWWF flow within the collection system and represents cumulative level of development conditions using diurnal patterns corresponding to the closest existing sewer-shed within the vicinity of each future development area. The unit hydrograph developed for the existing system was used to estimate PWWFs under design storm conditions for each level of development.

Scenario 4 – Near-Term Development

This modeled scenario is the existing wet-weather flow model evaluated in Scenario 3 with the addition of flow from vacant parcels within the existing service area and the remaining EDUs to be connected in on-going development areas. Generally, this includes all of the South development area and infill parcels within city limits excluding the Northeast development area. The sanitary sewer flow projections for these areas were determined using the updated wastewater generation rates. The results of this simulation represent PWWF conditions in the collection system if all vacant parcels within the existing service area and on-going development areas are connected to the existing system (PWWF = 7.53 MGD).

Scenario 5 – Long-Term Development

This modeled scenario represents full development of the extent of city limits by adding flow from the Northeast development area and the immediately adjacent portion of the North of I-80 development area to the conditions simulated in Scenario 4. The sanitary sewer flows projected from these areas were determined using the updated wastewater generation rates and added to the near-term development model. The results of this simulation represent PWWF conditions in the collection system upon full development of current extent of city limits (PWWF = 10.34 MGD).

Scenario 6 – Build-Out Development

This modeled scenario represents full build-out of the City's SOI. It builds on the Scenario 5 model by adding flow from the East development area and the remaining portions of the North of I-80 development area. The sanitary sewer flows projected from these areas were determined using the updated wastewater generation rates. The results of this simulation represent the projected PWWF in the collection system and the new trunk sewers needed to serve the added future development areas (PWWF = 12.69 MGD).

The development areas and their associated flow projections included in each model scenario evaluated in this SCSMP are presented as **Table 5-7**, **Table 5-8**, and **Table 5-9**. The development areas added to each level of development scenario simulated in the hydraulic model are depicted in **Figure 5-1**.

Table 5-7 Development Scenarios - Development Area

General Plan 2040 Land Use Designation	Development Scenario Gross Area (Acres) (1)				
	Existing (2)	Near-Term (3)	Long-Term	Build-Out	Total
Existing Service Area					
Existing Sewer-sheds	1,443				1,443
Open Space					
Agricultural	0	0.2	2	1	3
Parks	138	21	0	0	159
Mixed Use					
Campus Mixed Use	-	54	263	0	318
Corridor Mixed Use	-	119	0	0	119
Downtown Mixed Use	-	5	0	0	5
Residential					
Low Density Residential	-	372	0	616	988
Medium Density Residential	-	184	0	0	184
High Density Residential	-	0	0	0	0
Commercial					
Neighborhood Commercial	-	3	0	0	3
Regional Commercial	-	11	147	205	362
Service Commercial	-	6	0	0	6
Industrial					
Industrial	-	64	262	0	326
Public					
Public Facilities	817	10	0	29	856
Total:	**2,398**	**849**	**675**	**850**	**4,772**

1) Gross area represents the total acreage of the development, the net area is 80% of the gross area and is used to calculate average daily wastewater flow projections, as described in the City's Design Standards.
2) The existing sewer service area is the area that contributes base flow and I/I to the collection system and is not broken out into the same land use categories as the future development areas. The existing sewer sheds are comprised of mixed use, residential, commercial, and industrial land uses. The hydraulic model distributes flow to each parcel using a weighted area proportion relative to each sewer-shed. Public facilities and parks were excluded as they do not contribute flow at the same rate (gpd/acre) as other land use types.
3) The Near-Term scenario uses the remaining EDUs outlined in specific planning documents for on-going developments to project future flows for 620 acres of the South development area at a rate of 250 gpd/EDU. Remaining EDUs associated with on-going development areas equates to approximately 1,880 EDUs. See **Table 3-5** for specific plan data. The remaining 230 acres included in the Near-Term scenario consists of infill within the existing system and a small portion of the South development area. The land use of the remaining area was used to determine its associated flow projection and consists of approximately 40% mixed use, 30% industrial, 20% residential, and 10% commercial land use designations.

Table 5-8 Future Development Scenarios – ADWF Projection

General Plan 2040 Land Use Designation	Generation Rate (gpd/acre)	Development Scenario ADWF Projection (MGD)			
		Near-Term	Long-Term	Build-Out	Total
Open Space					
Agricultural	100	0.00	0.00	0.00	0.00
Parks	0	0.02	0.00	0.00	0.02
Mixed Use					
Campus Mixed Use	3,000	0.05	0.63	0.00	0.68
Corridor Mixed Use	3,000	0.23	0.00	0.00	0.23
Downtown Mixed Use	3,000	0.01	0.00	0.00	0.01
Residential					
Low Density Residential	800	0.35	0.00	0.39	0.75
Medium Density Residential	3,600	0.23	0.00	0.00	0.23
High Density Residential	5,000	0.00	0.00	0.00	0.00
Commercial					
Neighborhood Commercial	1,100	0.00	0.00	0.00	0.00
Regional Commercial	1,100	0.01	0.13	0.18	0.32
Service Commercial	1,100	0.01	0.00	0.00	0.01
Industrial					
Industrial	1,400	0.07	0.29	0.00	0.37
Public					
Public Facilities	0	0.00	0.00	0.00	0.00
	Total ADWF:	**0.98**	**1.06**	**0.57**	**2.61**

Table 5-9 Model Scenarios – ADWF Summary

Level of Development	Model Description	Development Areas Added	ADWF Added (MGD)	Total Modeled ADWF (MGD)	Modeled PWWF (MGD)
Existing	Scenarios 1-3	• Existing Services	1.09	1.09	4.30
Near-Term	Scenario 4	• Service Area Infill • On-going/South	0.98	2.07	7.53
Long-Term	Scenario 5	• Northeast • North of I-80 (portion)	1.06	3.13	10.34
Build-Out	Scenario 6	• East • North of I-80 (remaining)	0.57	3.70	12.69

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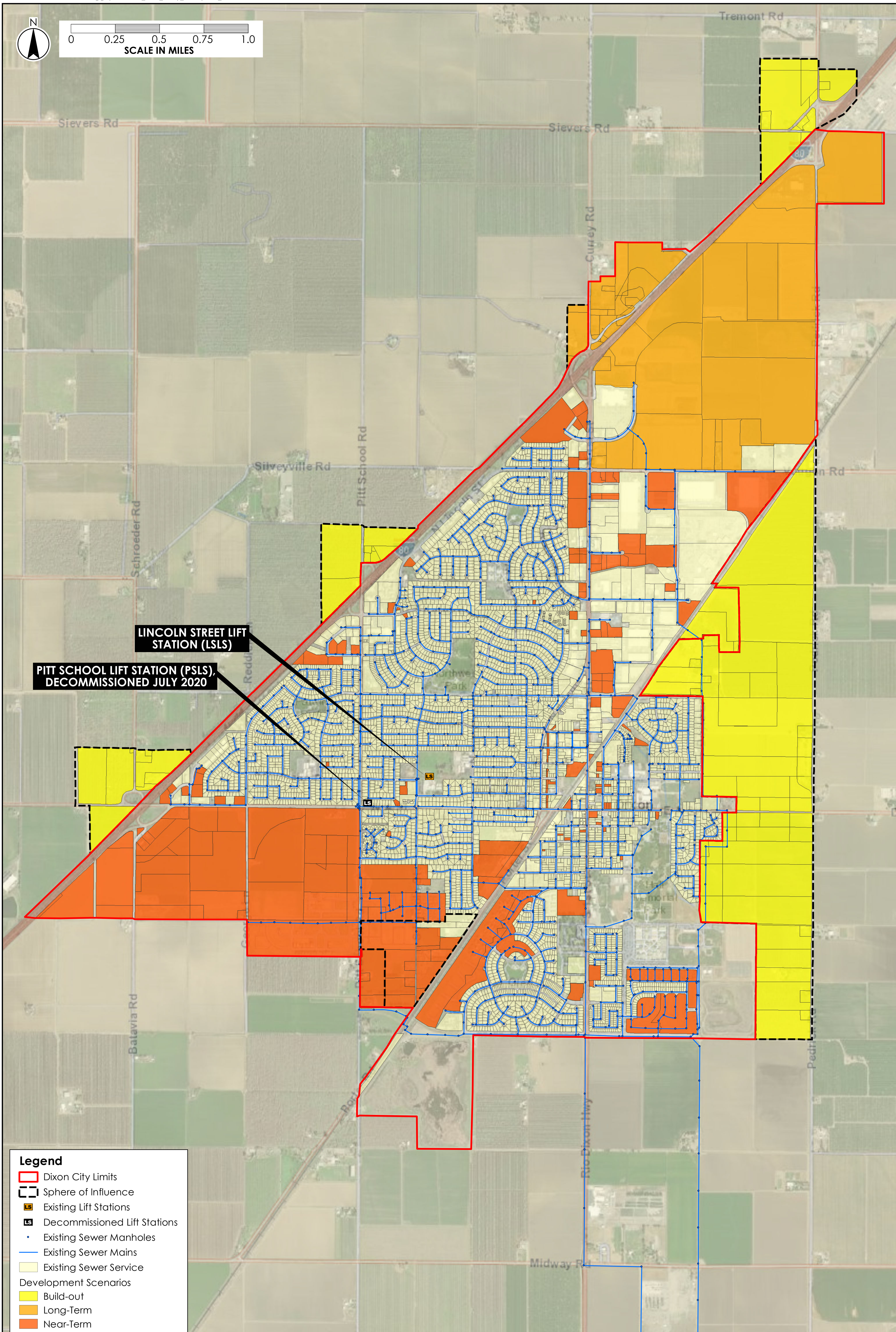

Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 5-1
City of Dixon Development Scenarios

6.0 HYDRAULIC MODEL DEVELOPMENT

The purpose of this chapter is to outline the details of the sewer collection system model construction and approach. A review of the data provided for use by the City in developing a model of the sewer collection system was performed through a GAP analysis to identify the data “gaps” within the City’s existing database and records. Data provided by the City included their existing GIS database, record drawings, and study reports which were used to complete quality enhancement to the City’s internal sanitary sewer GIS database.

The data review includes two elements: gap analysis, and data confidence for model application. The GAP analysis inventories the data and parameters associated with the physical network of the City’s wastewater collection system. This physical network becomes the underlying framework of hydraulic model development, and it is crucial that infrastructure data is reviewed for completeness and connectivity.

This chapter is divided into the following sections:

- GIS Data Development
- Hydraulic Model Approach
- Hydraulic Model Calibration
- Model Assumptions & Design Criteria

6.1 GIS DATA DEVELOPMENT

The review of the existing data was completed prior to building the collection system model to enhance the quality of the existing GIS database by identifying critical gaps and missing information. Missing and inconsistent data identified in the physical network was summarized for City review. Physical data required for model development includes manhole inverts and rim elevations, and sewer diameters, slopes, and inverts. The review also considered the hydraulic continuity and suitability of physical sewer data in terms of profile connectivity. A data verification program was developed to obtain missing or inconsistent data in an efficient manner, through the use of drawing review, field verification, and inference. As a final resort, data was inferred based on the surrounding network.

The City provided the existing GIS database and existing collection system record drawings for use in this assessment. The GIS database was provided in the form of individual shapefiles.

The following physical network data was received from the City:

- Sewer Manholes: Sewer_Manholes_Project.shp
- Sewer Mains: Sewer_Mains.shp
- Sewer Cleanouts: Sewer_Cleanouts.shp

Unique pipe and manhole identifiers (IDs) are critical in modeling software to provide identification for both nodes and conduits. Therefore, IDs were assigned for the active manholes and pipelines with blank entries for this field prior to importing data into the hydraulic model. IDs in the City's GIS database for manholes and mains follow patterns of SSXXXX for manholes and CXXXX for mains. For manholes and mains with duplicate IDs, one of the duplicates was given a "D" at the end of the ID. For manholes and mains without an ID, an ID was assigned in the format of SSXXXXN or CXXXXN. Manholes and mains added to the GIS database by Stantec were also assigned an ID in the format of SSXXXN or CXXXXN.

Data validation is the process of confirming the hydraulic continuity and suitability of physical sewer data in terms of profile connectivity. Most modern modeling software packages include routines and queries to help perform validations. Pipelines were assigned upstream and downstream manholes based on invert elevation and position within the geodatabase. Network modifications were made to account for changes within the collection system.

The City's GIS database contained geospatial location and diameter data for cleanouts, manholes and mains. The City's manhole shapefile contains few attribute fields used to define physical parameters and provide additional relevant information. Based on the data provided, there are 1,190 manholes, and 21 cleanouts in the City's sanitary geodatabase. Upon initial review of record drawing sets provided by the City, required attribute information for 486 manholes and 452 mains was missing. By reviewing the record drawing sets provided by the City, information for these attributes was found for 63.5% of the manholes and 67.8% of the sewer mains.

This includes 126 manholes and 139 mains that were found to be missing from the City's GIS database and were subsequently added by Stantec. The following major collection system improvement projects were reviewed and incorporated into the geodatabase: Parklane Subdivision, Valley Glen Subdivision, & E-W Trunk Sewer Connector. These added manholes and mains were given the letter 'N' at the end of their ID number. Survey data of manholes in the collection system was collected to obtain elevations at 80 critical points in the system. If record drawings were still unavailable for the remaining manholes and mains, elevations were inferred by analyzing typical slopes of sewer mains in the City.

One challenge in the GAP analysis effort is ensuring that the elevation datums used for each record drawing set are adjusted appropriately to ensure the accuracy of elevations input to the model. An outfall node was added to represent the discharge location of the influent sewer at the WWTF. Lift station information was incorporated into the model using record drawings and represented by storage nodes and pump links.

6.2 HYDRAULIC MODEL APPROACH

The following describes the development of the hydraulic model of the City's wastewater collection system. Prior to model development, the physical network of sewers, manholes, and pump stations was established in the City's GIS database, as discussed in the preceding section. Specifically, the physical information contained therein for the existing collection system. This physical network becomes the underlying framework for the model. Therefore, it is crucial that the infrastructure is reviewed for completeness and proper connectivity. In addition to the GIS data enhancement of the physical network of the existing collection system, review of data associated with parcel and land use information in the GIS database was also completed prior to model development. The data review was undertaken in GIS in both ESRI ArcMap and hydraulic modeling software utilizing built-in PCSWMM analysis tools.

6.2.1 Model Approach & Software

The City of Dixon's hydraulic modeling needs were assessed to help define the software and approach needed in the development of the hydraulic model. The preferred approach was to develop a detailed dynamic system model that includes all of the sewer pipes within the existing collection system.

Based on hydraulic needs and the level of detail and analysis required for this SCSMP, PCSWMM software, developed by Computational Hydraulics Inc., was selected for use in developing a collection system computer model for the City. This software package has been developed using the EPA SWMM 5.0 engine as its basis.

This software was selected for its ability to meet the following objectives:

- To determine the existing hydraulic capacity of the City's collection system and components.
- To identify system limitations such as bottlenecks and infrastructure incapable of accommodating future growth.
- To provide preliminary estimates of the infrastructure required to serve future development of the General Plan 2040.
- To evaluate future phasing strategies for the construction of future infrastructure.

6.2.2 System Elements

The model is comprised of a network of data elements called nodes (junctions) and links (conduits). The nodes and links represent the components of a typical wastewater system. A node or junction is a point in the network having an X and Y coordinate. Nodes can represent manholes, wet-wells, chambers, or outfalls. A link or conduit conveys flow between nodes, they are connected at one end to a start node and at the other end to an end node. Links can represent gravity sewers, force mains, or pumps.

6.3 HYDRAULIC MODEL CALIBRATION

The calibration process is required to verify the accuracy of the hydraulic model at predicting the collection system performance under varying flow conditions. The flow monitoring data from the 2020 flow monitoring study was used to calibrate the model under observed dry weather conditions. Due to lower than average rainfall conditions experienced during the 2020 flow monitoring period, historical WWTF influent data from significant storm events was used to calibrate the wet-weather system parameters. The calibrated model was used to assess system performance under design storm conditions, by simulating a 10-year, 24-hour design storm and the associated PWWF in the system.

The hydraulic model was calibrated for dry weather flow using flow monitoring data provided as part of the 2020 flow monitoring study conducted by V&A Consulting Engineers, Inc (discussed in **Section 4.3**). The study measured flow at three different locations within the collection system over a period of approximately 84 days from February 2, 2020, to April 26, 2020. The following summarizes the impacts to the quality of flow monitoring data used to calibrate the hydraulic model.

- Unideal flow conditions during the flow monitoring period – FOG blockage
- Changes in daily flow pattern during the flow monitoring period – SIP order
- Insignificant wet-weather flow response during the flow monitoring period – Lower than average rainfall
- Major system routing changes that have occurred since the flow monitoring period – East West Trunk Sewer/ PSLS decommissioned.
- Data collected during the flow monitoring study was impacted by a grease clog obstructing flow in the 42-inch Main Trunk.

A summary of dry weather and wet-weather flow calibration data is provided in **Appendix C**.

6.3.1 Dry Weather Calibration

The model was calibrated to the monitored DWF, considering weekday and weekend flow patterns. Dry weather flow calibration was completed by running model simulations for two observed “dry weather” periods. The primary seven-day period selected for calibration was between March 7 and March 14, 2020.

Lower than average rainfall was experienced throughout Northern California before and throughout the flow monitoring period and as a result, the data was only suitable for dry weather calibration. Typically, ADWF is calculated in the months of July – September, which is considered to be the dry season in the region. The average wastewater flow observed at the WWTF influent flow meter from July through September of 2019 (2019 ADWF) was 1.14 MGD. The average over the past five years (2014-2019) equates to 1.13 MGD.

The hydraulic model was calibrated to an ADWF flow value of 1.09 MGD. The ADWF was distributed within each flow monitoring shed based on a contributing area weighted distribution, with consideration for weekday and weekend flow patterns. The model results at each monitoring site were compared to the “observed” monitored flow for the dry weather flow period. The parameters were varied in a systematic manner within a reasonable range until an acceptable fit to the observed flow was obtained. Comparisons were made between modeled vs. observed flow, depth, volume and velocity, with a target level of accuracy of +/- 15 percent. Parameters for velocity and depth typically indicate significant differences between modeled and observed data, as field conditions such as sediment depth, minor defects and obstructions, and actual pipe slope in the vicinity of the flow monitor may vary from modeled conditions.

Without flow monitoring data collected during high groundwater conditions, in which flow exceeds baseline ADWF levels, accurate predictions of the distribution of groundwater infiltration within the collection system could not be made. The GWI component of the ADWF distributed within the hydraulic model is assumed to be zero.

It is recommended that the City monitor flow to capture GWI and recalibrate the hydraulic model to consider peak conditions. Flow monitors should be installed in portions of the system that exist at significant depth or very low invert elevations, in the vicinity of the 27-inch trunk sewer and other regions with low depth to groundwater, and near water ways and stormwater basins. It is recommended that the City monitor the depth to groundwater in the area regularly to determine an ideal time flow monitors should be deployed based on rising groundwater elevation.

6.3.2 Wet Weather Calibration

After dry weather flow calibration, the hydraulic model was calibrated for PWWF. Historical influent flow data was used for wet-weather calibration because data collected during the flow monitoring period showed little to no wet-weather flow response with limited rainfall. The high-resolution influent flow data and hourly rainfall data from CIMIS Station 121 used in the PHF analysis was also used to identify historical rainfall events and flow data within the following parameters for model calibration:

(1) Time Period: Post - 2017, to be within the reliable data window for high resolution influent flow data and to be representative of existing conditions (post WWTF Improvements Project, 2017).

(2) Storm Intensity: Correspond to peak influent flow conditions and total rainfall. Storm events were selected that matched the PHF design criteria used in the WWTF Facilities Report associated with WWTF Improvements Project (Stantec 2017).

(3) Water Year Frequency: Occur during a water year with average or greater total rainfall in the region. To be representative of average or greater water conditions, groundwater levels, river stages, etc.

The hydraulic model was calibrated using the “RTK Unit Hydrograph” method to determine the wet-weather response in the collection system under peak rainfall conditions. This method utilizes a set of three triangular unit hydrographs (UH) to represent the fast-response, medium-response, and slow-response to the rainfall dependent inflow and infiltration (RDII). Each UH is represented by three parameters (R, T, and K), which are used to calculate the intensity, duration, and rate of recession of the hydrograph.

Once initial unit hydrographs for each flow monitoring shed were developed, calibration was performed.

RDII parameters were input into the hydraulic model on a trial basis and the routed flow hydrograph produced by the model at the WWTF outfall location was compared to the recorded flow. The parameters were varied in a systematic manner within a reasonable range until an acceptable fit to the observed flow data was obtained. Comparisons were made between modeled versus monitored flow with a target level of accuracy of +/- 15 percent.

The calibration prioritized representing PWWF and total flow volume recorded at the WWTF during PWWF calibration events, as these parameters are more indicative of potential capacity restrictions. Calibration included modification of parameters such as wastewater generation rate, manning's n, and RTK values.

6.3.3 On-going Calibration

On-going calibration is recommended to ensure the model is up-to-date for potential future uses. Updates are continuously being made to the physical infrastructure geodatabases and should be reviewed and incorporated into the model as appropriate over time. In addition, dry weather flow and wet weather flows should be reviewed and the loading data adjusted as necessary based on future flow monitoring data. As part of the City's flow monitoring efforts, locations should be strategically selected to focus on areas of interest to ensure the model is accurate for more localized areas. In this way, the model will be a "living" City tool, expanding to include new infrastructure and improvements as they occur and reflecting new flow data as it is collected over time.

6.4 MODEL ASSUMPTIONS & DESIGN CRITERIA

After performing calibration of the existing system model, unit flow factors described in **Chapter 5.0** were used to estimate wastewater flow from future development areas within the City's General Plan 2040. The projected wastewater flow contributions were added to the model of each corresponding level of development scenario.

Each future level of development scenario extends the existing collection system to further expand the service area to reach build-out of the City's General Plan 2040. Each level of development requires construction of new collection system infrastructure, including new pump stations and trunk sewers. Design criteria and assumptions used to simulate future infrastructure within the modeling software is presented in **Table 6-1**.

Table 6-1 Hydraulic Model Assumptions & Design Criteria Summary

Design Parameter	Criteria Value
Manhole Spacing	400 – 500 feet
Manhole Drops	0.1 ft at bends > 45 degrees for sewers larger than 18-inches
Manhole Depth	6 (min) – 25 (max) feet below grade
Manning's "n"	Manning's formula shall be used to determine the relation of slope, design flow, velocity, and diameter. The "n" value shall not be less than 0.013 for all new pipes.
Minimum Pipe Size	8-inches
Design Flow Velocity (when flowing full)	• Minimum: 2.0 • Target: 2.5 - 7.0 fps • Maximum: 10 fps
Pipe Size/Hydraulic Loading Ratio (HLR)	• New Sewers: 70% • Existing System Improvements: 100%
Rim Elevation/Grade	3-foot LiDAR data from Solano County Elevation Products Download Application.
Dry Weather Flows	• Existing Service Area: Sewage generation is based on ADWF measured and calibrated for each sewer-shed. • Future Service Area: Future sewage generation was determined by the land uses and wastewater generation rates described in **Chapter 5.0**. Flow was distributed to nodes within each sewer-shed proportionally to the total area of parcels assumed to be contributing to that manhole.
Peaking Factors	• Existing Service Area: Diurnal patterns were derived from flow monitoring data for weekdays and weekends to capture differences in sewage generation trends for each sewer-shed. • Future Service Area: Diurnal patterns derived for the existing system were assigned to the future service area corresponding to the point of connection of new infrastructure to the existing system.
Design Storm	10-year, 24-hour design storm with a Huff distribution was used to model and evaluate the system under PWWF conditions. • 24-hour Total Rainfall = 3.32-inches
RDII Method	Horton infiltration model with RTK unit hydrograph parameters calibrated to flow data recorded at the WWTF.

7.0 EXISTING COLLECTION SYSTEM EVALUATION

The purpose of this chapter is to present an evaluation of the City's existing sewer collection system and its ability to meet the City's recommended LOS performance and planning criteria under existing development conditions. The existing level of development scenarios and LOS criteria are described in detail in **Chapter 5.0** of this SCSMP. This existing collection system evaluation includes both facility capacity and hydraulic performance evaluations. The system facility capacity evaluation assesses the existing sewer flow and pumping capacity. The hydraulic performance evaluation assesses the existing system's ability to meet the recommended LOS performance standards under PWWF conditions.

The results of each scenario simulated in the hydraulic model of the existing system were used to conduct the hydraulic evaluation of the City sewer system and identify any existing sewer collection system deficiencies. Recommendations have been made to address any existing system capacity deficiencies, and these recommendations were then used to develop a prioritized capital improvement program (CIP), including an estimate of probable construction costs. The CIP also includes improvement projects identified as a result of the condition assessment presented in **Chapter 9.0**. The recommended improvements and CIP are presented in **Chapter 9.4** of this SCSMP. Infrastructure improvements identified in this chapter to address existing system capacity are shown on **Figure 7-1** along with the existing primary trunk sewer network.

This chapter is divided into the following sections:

- Existing Sewer System Scenarios & Flows
- Existing Sewer System Capacity Evaluation
- Existing Sewer System Performance Evaluation
- Summary of Existing System Evaluation & Recommendations

7.1 EXISTING SEWER SYSTEM SCENARIOS & FLOWS

The hydraulic model of the existing collection system was assessed to determine the capacity of the existing trunk network, identify hydraulic deficiencies, and to determine capacity improvement projects for the sewer system, if necessary to accommodate flow under existing PWWF conditions.

7.1.1 Scenario 1 – Existing Dry Weather Flow

The existing collection system was modeled and calibrated to typical dry weather conditions using flow data corresponding to a period of "dry weather" which occurred during the flow monitoring study described in **Section 4.3**. The final calibrated "dry weather" model was used to evaluate the LOS of sewers within the collection system under typical dry weather conditions. The ADWF observed during the flow monitoring period and distributed in the hydraulic model, was 1.09 MGD. It should be noted that the dry weather calibration period occurred during a lower than average water year in terms of total rainfall.

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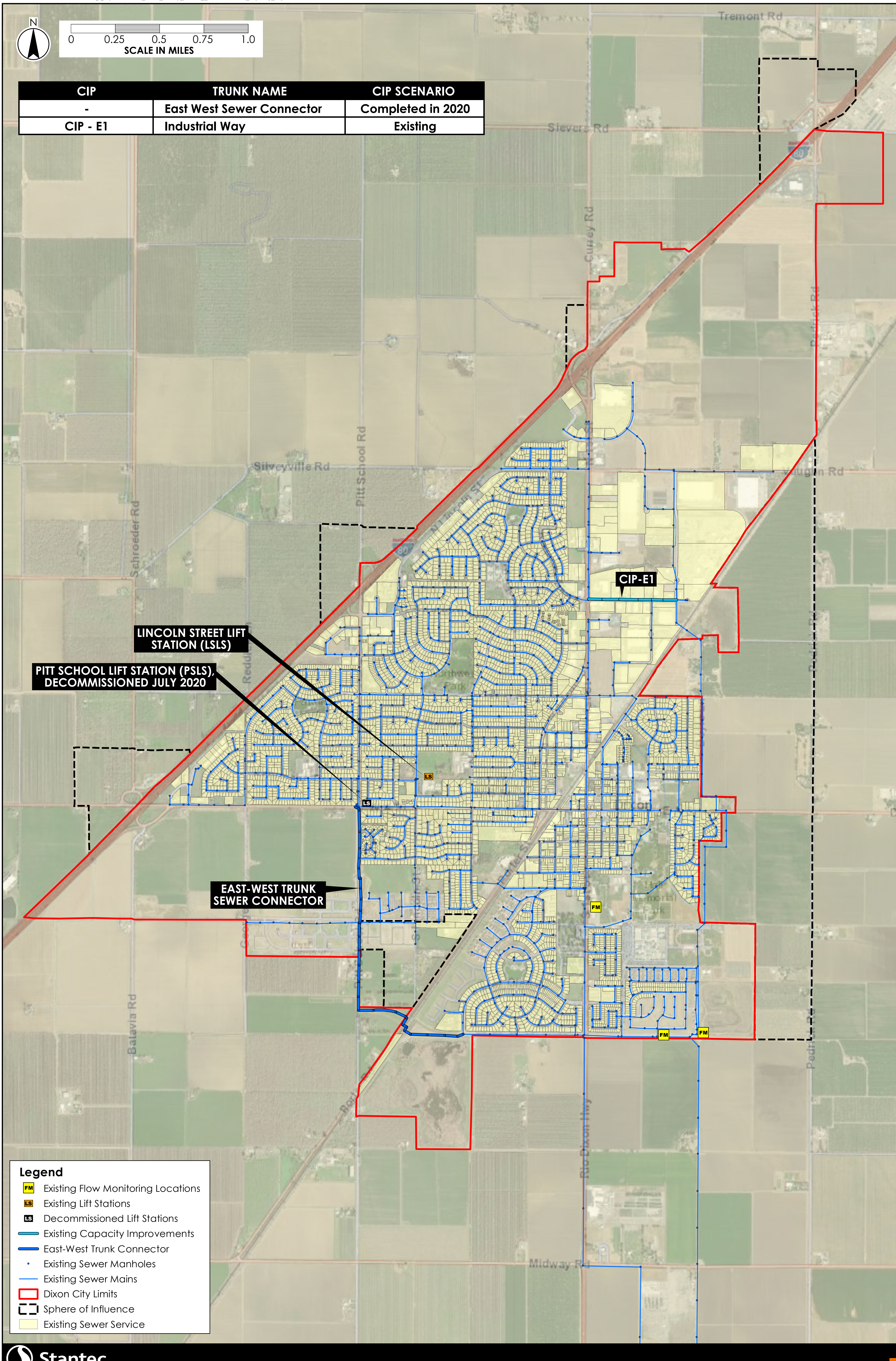

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Figure 7-1
Existing Collection System – Recommended Capacity Improvements

Based on rainfall conditions preceding and throughout the duration the flow monitoring period, groundwater levels in the region were also assumed to be lower than average. Although the model simulates "dry weather" conditions, flows could be larger under elevated regional groundwater conditions without rainfall.

When the local groundwater elevation rises above the invert elevations of the existing collection system, it could become submerged and take on a significant amount of GWI based on historical information provided by the City discussed in **Chapter 4.0**. This SCSMP assumes that GWI occurring during high regional or local groundwater conditions is not accurately represented by the hydraulic model. Peak GWI should be determined and considered on top of the results for PWWF presented in this SCSMP in the appropriate areas of the collection system.

The existing service area is approximately 2,500 acres and has an ADWF of 1.09 MGD. The existing system model assumes that approximately 1,444 acres of the service area contribute to RDII entering the collection system, this excludes parks, open space, and large public facilities located within the service area that do not contribute flow at same rate per acre as the remaining service area. The peak dry weather or peak diurnal flow predicted to occur at the WWTF in this scenario is 1.76 MGD. A summary of the dry weather flow model parameters is provided as **Table 7-1**.

Table 7-1 Scenario 1 – Model Flow Summary

Sewer-shed	Collection Area (Acres)	ADWF (MGD)	Peak ADWF (MGD)
1	427	0.30	0.52
2 (1)	304	0.22	0.35
3	713	0.57	1.03
Total/WWTF (2)	1,444	1.09	1.76

1) Flow monitoring site 3 was located upstream of site 2, and therefore the individual flows for sewer-shed 2 are calculated as the difference of the flow at site 2 and that recorded at site 3.
2) The total flow measured at the WWTF equates to the sum of flow from flow monitoring sites 1 and 2.

7.1.2 Scenario 2 – Existing Wet Weather Flow

A 10-year, 24-hour design storm was applied to the calibrated wet-weather model to simulate PWWF conditions used for design. The results of this simulation were used to evaluate the existing collection system under PWWF conditions. The City's 10-year, 24-hour design storm uses a Huff Distribution to simulate hourly rainfall conditions equating to a 24-hour rainfall total of 3.32 inches.

It should be noted that the wet-weather flow model was calibrated to historical conditions observed at the City's WWTF due to lack of rainfall experienced during the flow monitoring period. Therefore, the model does not represent localized RDII conditions within the collection system. In reality, some portions of the system may experience higher rates of I/I while others experience less, equating to the overall average represented by the model. It is recommended that the City collect additional flow monitoring data with the intent of collecting additional wet-weather flow responses throughout the collection system to determine the localized RDII response and further refine the model.

The wet-weather flow model has the same service area and ADWF component described for the dry-weather flow model in Scenario 1. Under existing conditions, the design storm is predicted to generate a PWWF of 4.29 MGD at the WWTF. A summary of the wet weather model flow is provided as **Table 7-2**.

Table 7-2 Scenario 2 – Model Flow Summary

Sewer-shed	Collection Area (Acres)	ADWF (MGD)	PWWF (MGD)
1	427	0.30	1.17
2 (1)	304	0.22	0.86
3	713	0.57	2.36
Total/WWTF (2)	1,444	1.09	4.29

1) Flow monitoring site 3 was located upstream of site 2, and therefore the individual flows for sewer-shed 2 are calculated as the difference of the flow at site 2 and that recorded at site 3.
2) The total flow measured at the WWTF equates to the sum of flow from flow monitoring sites 1 and 2.

7.1.3 Scenario 3 – Existing Wet-Weather Flow (without PSLS)

This third existing system scenario considers recent collection system improvements the City has implemented since the model was initially developed and the flow monitoring data used for calibration was collected. This includes bringing the East West Trunk Sewer Connector into service and decommissioning the PSLS. The model flows and sewer shed area parameters remain unchanged from those used in Scenario 2, but the physical elements associated with the East West Trunk Sewer collector were incorporated into the collection system model and the PSLS was removed.

Approximately 0.13 MGD of the total ADWF contributes to the PSLS, located in sewer-shed 3. In this scenario, this portion of sewer-shed 3's flow is allocated to the East West Trunk Sewer Connector where it would be measured by flow monitoring flow monitoring site 2. Re-routing this flow results in a reduction in the PWWF predicted to occur in West Cherry Street and at flow monitoring location 3. The PWWF predicted at site 3 is reduced by approximately 0.50 MGD. The PWWF predicted to occur at the WWTF is approximately unchanged from the previous scenario.

Table 7-3 Scenario 3 – Flow Routing Adjustment Summary

Flow Monitoring Site	Scenario 2 PWWF (MGD)	Scenario 3 PWWF (MGD)	Difference (MGD)
1	1.17	1.17	0.00
2 (1)	0.86	1.36	-0.50
3	2.36	1.86	0.50

1) Individual flows for sewer-shed 2 are calculated as the difference of the flow at site 2 and that for site 3. The total PWWF predicted in both scenarios at site 2 is 3.2 MGD.

7.2 EXISTING SEWER SYSTEM CAPACITY EVALUATION

To evaluate the existing sewer collection system, analyses addressing system facilities were conducted:

- Existing Trunk Sewer Flow Capacity
- Lift Station Flow Capacity Evaluation

The results of the existing sewer system facility analyses are discussed below.

7.2.1 Existing Trunk Sewer Flow Capacity

The City's existing trunk sewer network, presented in **Section 2.4** of this SCSMP, was evaluated to determine its existing capacity and identify hydraulic capacity constraints that may exist within the system. The City's primary trunk sewers are shown in **Figure 7-2**.

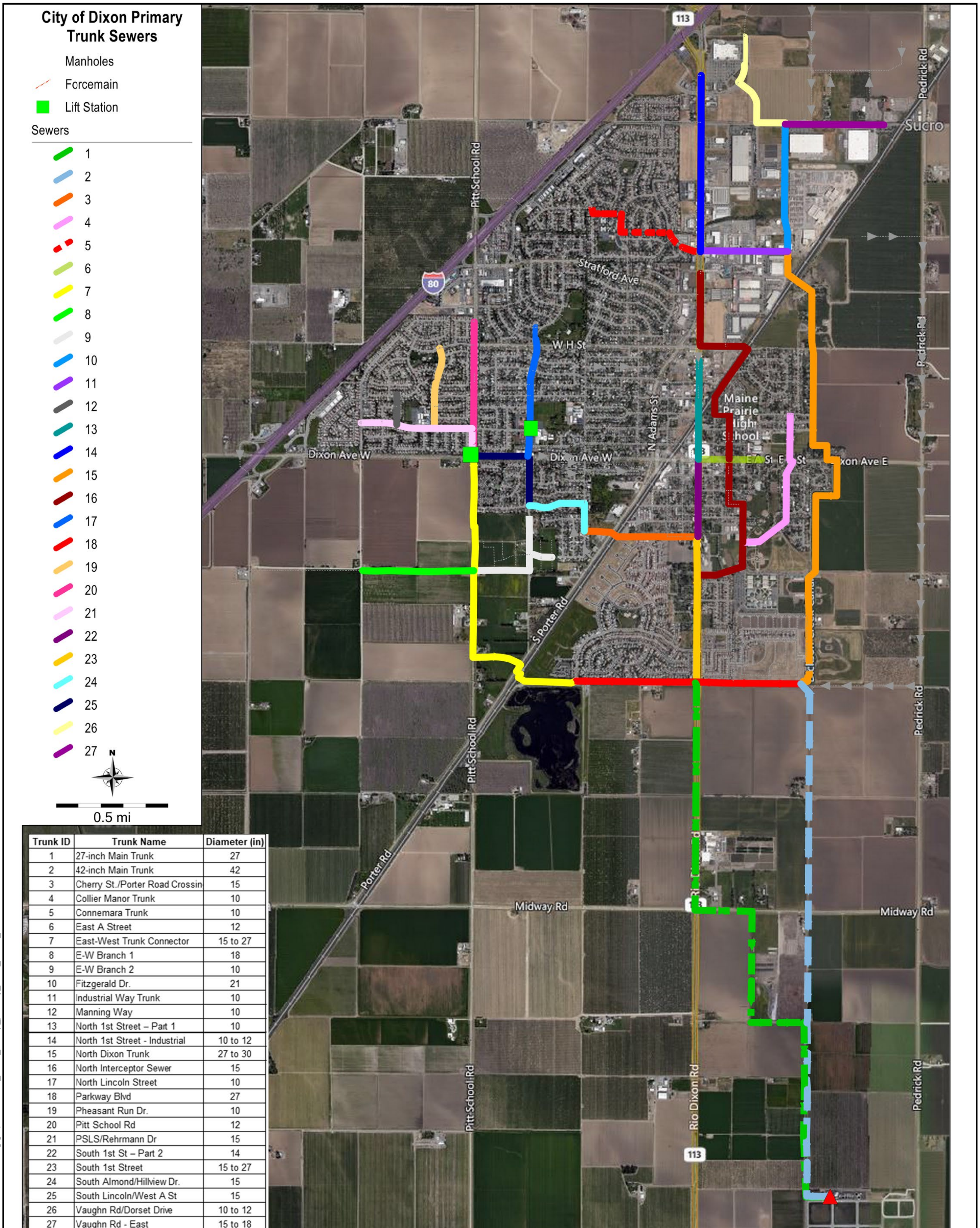
The approximate full pipe flow of the sewer, or the capacity of the sewer derived from manning's equation and the calibrated roughness coefficients within the existing system, is used within the model to determine the HLR of each sewer in the collection system. The capacity of the existing collection system is described below for each sewer-shed considered in this SCSMP.

Sewer-Shed 1

Sewer-Shed 1 collects from the northern portion of the City, i.e., flow is conveyed from north to south via the North Dixon trunk which converges with the Parkway Blvd. trunk at the 48-inch Main trunk sewer. From the North Dixon trunk, the sewer-shed splits into two primary branches. The general branching of the primary trunk sewers within sewer-shed 1 are listed from downstream to upstream below:

1) North Dixon Trunk
   a) Fitzgerald Dr.
      i) Vaughn Rd/Dorset Drive
      ii) Vaughn Rd - East
   b) Industrial Way
      i) North 1st Street
      ii) Connemara

The Fitzgerald Dr. and Industrial Way trunks converge at the upstream end of the North Dixon trunk. The Fitzgerald Dr. trunk primarily serves industrial users, collecting flow from the Vaughn Rd/Dorset Drive and Vaughn Rd – East trunks. The upstream end of the Fitzgerald trunk has a capacity of approximately 2.4 MGD, which is less than the sum of the downstream capacity of the two upstream trunks in Vaughn Rd. Therefore, additional flow could be conveyed through the Fitzgerald trunk if the upstream portion is upsized.



Trunk ID	Trunk Name	Diameter (in)
1	27-inch Main Trunk	27
2	42-inch Main Trunk	42
3	Cherry St./Porter Road Crossin	15
4	Collier Manor Trunk	10
5	Connemara Trunk	10
6	East A Street	12
7	East-West Trunk Connector	15 to 27
8	E-W Branch 1	18
9	E-W Branch 2	10
10	Fitzgerald Dr.	21
11	Industrial Way Trunk	10
12	Manning Way	10
13	North 1st Street – Part 1	10
14	North 1st Street - Industrial	10 to 12
15	North Dixon Trunk	27 to 30
16	North Interceptor Sewer	15
17	North Lincoln Street	10
18	Parkway Blvd	27
19	Pheasant Run Dr.	10
20	Pitt School Rd	12
21	PSLS/Rehrmann Dr	15
22	South 1st St – Part 2	14
23	South 1st Street	15 to 27
24	South Almond/Hillview Dr.	15
25	South Lincoln/West A St	15
26	Vaughn Rd/Dorset Drive	10 to 12
27	Vaughn Rd - East	15 to 18

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Figure 7-2
Primary Trunk Sewer Network

The Industrial Way trunk collects flow from industrial users and conveys flow from the North 1st Street trunk and the Connemara trunk which converge upstream. The Industrial Way trunk appears to be undersized at 10-inches in diameter. It is smaller than the upstream 12-inch North 1st Street trunk. The Industrial Way trunk has limited flow capacity ranging between 0.4 and 0.5 MGD, equating to approximately half of the capacity of the North 1st Street trunk alone. The Connemara trunk is 10-inches in diameter and conveys flow collected from the Connemara Subdivision. The North 1st Street trunk collects flow from the western edge of the industrial area and conveys a small amount of flow from the Connemara subdivision.

The approximate full pipe flow at the upstream and downstream end of each trunk sewer is presented in **Table 7-4**.

Table 7-4 Sewer-shed 1 Existing Trunk Sewer Capacity

Primary Trunk Sewer	Pipe Size (in)	Length (LF)	Upstream Capacity (MGD)	Downstream Capacity (MGD)
1) North Dixon Trunk	27 to 30	11,800	5.0	6.1
a) Fitzgerald Dr.	21 to 27	3,000	2.4	3.1
i) Vaughn Rd/Dorset Drive	10 to 12	3,000	0.7	1.2
ii) Vaughn Rd - East	15 to 18	2,300	1.5	2.5
b) Industrial Way Trunk	10	2,100	0.4	0.5
i) North 1st Street - Industrial	10 to 12	4,200	0.6	0.9
ii) Connemara Trunk	10	3,300	0.6	0.8

Industrial wastewater discharges typically do not have a consistent diurnal flow pattern. This is due to the variability of different industrial processes. For example, the rate and timing of wastewater discharged from a winery is much different than that of an industry which does not use water at all in its process. The inherent variability of industrial processes between industries, seasons, and periods of production makes generalizing flow patterns and characteristics difficult.

Therefore, it is recommended that the City collect localized flow monitoring data within Sewer-Shed 1 to validate the results presented herein and to provide further refinement of the model and resulting capacity requirements associated with future development in the area.

Sewer-Shed 2

The flow within Sewer-Shed 2 was determined by taking the difference in flow recorded at site 2 and site 3. The primary trunk in Sewer-Shed 2 extends from flow monitoring site 2 in Parkway Blvd. to South 1st Street, before turning east near Silveyville Cemetery to convey flow collected along North Interceptor sewer.

The trunk sewers of Sewer-Shed 2 extend from the primary trunk sewer branches, which include Parkway Blvd., South 1st Street, and the North Interceptor Sewer. The East-West Trunk Connector connects to the Parkway Blvd. trunk in existing system Scenario 3.

The general branching of the primary trunk sewers within sewer-shed 2 are listed from downstream to upstream below:

1) Parkway Blvd
   a) South 1st Street – Part 1
      i) North Interceptor Sewer
         (1) Collier Manor Trunk
   b) East-West Trunk Connector
      i) E-W Branch 1
      ii) E-W Branch 2
      iii) PSLS/Rehrmann Dr (now contributes to the East-West Trunk Connector)
         (1) Pheasant Run Dr.
         (2) Manning Way

The Parkway Blvd trunk has a capacity of approximately 12.2 MGD at its downstream end, where it meets the 42-inch Main Trunk, and a capacity of approximately 6.4 MGD at the upstream end where it meets the South 1st Street – Part 1 trunk. The South 1st Street trunk has an approximate capacity of 6.0 MGD between the North Interceptor Sewer and Parkway Blvd. The remaining portion of this trunk, upstream of the North Interceptor Sewer, is considered to exist in Sewer-Shed 3 and the capacity is approximately 2.2 MGD.

The North Interceptor Sewer ranges from 27-inches in diameter at the downstream end, where it meets South 1st Street – Part 1, to 15-inches at its most upstream segment. The capacity ranges from approximately 5.5 MGD at the downstream end to approximately 1.2 MGD at the upstream end. Although it should be noted that the majority of the trunk has a capacity of less than 2.0 MGD, averaging approximately 1.8 MGD along the entire length. Only five of the downstream segments of the sewer are greater than 15-inches in diameter.

The Collier Manor trunk collects flow from the Collier Manor subdivision and meets the North Interceptor sewer at Hall Park, south of E Chestnut St. The Collier Manor trunk ranges from 12 to 10-inches in diameter and its capacity ranges from 1.2 to 0.5 MGD, moving from downstream to upstream along the trunkline. The upstream portion of the North Interceptor sewer, north of the point of connection of the Collier Manor trunk, collects and conveys flow from the Creekside subdivision, the Dixon Business Park, and portions of the Stratford Manor and Watson Ranch subdivisions.

The Parkway Blvd trunk extends west, beyond South 1st Street to collect flow from the existing portion of the Valley Glenn Subdivision. The East-West Trunk Connector ties into the far west end of the Parkway Blvd trunk, at the far end of the Valley Glenn Development. The East-West Trunk Connector is 27-inches in diameter at the downstream end and 15-inches in diameter at the upstream end, where it meets the PSLS. It has two main trunk branches that will convey flow from the Homestead Development area in future level of development scenarios. The capacity of the East-West Trunk Connector ranges from 6.0 MGD at the downstream end to 1.8 MGD at the upstream end.

The 18-inch E-W Branch 1 trunk has a capacity of approximately 2.2 MGD and extends west from the main East-West Trunk Connector. The 10-inch E-W Branch 2 trunk has a capacity of approximately 0.7 MGD and extends east from the main East-West Trunk Connector. Trunk sewers within the PSLS service area are discussed in the following evaluation of sewer-shed 3, as that is where flow was routed during the flow monitoring study. The approximate full pipe flow at the upstream and downstream end of each trunk sewer is presented in **Table 7-5**.

Table 7-5 Sewer-Shed 2 Existing Trunk Sewer Capacity

Primary Trunk Sewer	Pipe Size (in)	Length (LF)	Upstream Capacity (MGD)	Downstream Capacity (MGD)
1) Parkway Blvd	27	5,300	6.4	12.2
a) South 1st Street – Part 1 (1)	15 to 27	3,400	2.2	6.0 (1)
i) North Interceptor Sewer	15 to 27	10,000	1.2	5.5
(1) Collier Manor Trunk	10 to 12	3,800	0.5	1.2
b) East-West Trunk Connector	15 to 27	7,800	1.8	6.0
i) E-W Branch 1	18	2,600	2.2	2.2
ii) E-W Branch 2	10	3,100	0.7	0.7
iii) PSLS/Rehrmann Dr (2)	*10 to 15*	*3,300*	*0.8*	*1.6*
(1) Pheasant Run Dr.	*10*	*1,900*	*0.7*	*0.8*
(2) Manning Way	*10*	*700*	*0.8*	*0.8*

1) The South 1st St – Part 1 trunk is included in both Sewer-Shed 2 and Sewer-Shed 3 trunk sewer capacity tables. Only the downstream portion of this trunk, south of the North Interceptor Sewer, is within Sewer-Shed 2, this portion of the trunk has a capacity of approximately 6.0 MGD.
2) The PSLS was decommissioned after flow monitoring was conducted and sewer-shed boundaries were delineated. The PSLS service area was re-routed to flow by gravity through the East-West Trunk Sewer Connector where it now connects to Sewer-Shed 2. The assessment of these trunks presented under Sewer-Shed 3, corresponding to its original routing.

Sewer-Shed 3

Sewer-Shed 3 exists upstream of Sewer-Shed 2, it starts at the upstream end of the South 1st Street – Part 1 trunk sewer, downstream of the intersection of W. Cherry St. and South 1st Street. This portion of the South 1st Street – Part 1 trunk has a capacity of approximately 2.2 MGD. The South 1st Street - Part 2 and the Cherry St./Porter Road Crossing trunks converge at the upstream end of the South 1st Street – Part 1 trunk.

Following the 14-inch South 1st Street - Part 2 trunk north, upstream leads to the confluence of the 12-inch East A Street and the 10-inch North 1st Street trunks. The maximum capacity of the South 1st Street – Part 2 trunk is approximately 1.6 MGD. The East A Street and North 1st Street trunks have a capacity of approximately 1.3 MGD and 0.9 MGD.

The branching of the primary trunk sewers in sewer-shed 3 are listed below from downstream to upstream:

1) South 1st St – Part 1
    a) South 1st St – Part 2
        i) North 1st Street
        ii) East A Street
    b) Cherry St./Porter Road Crossing
        i) South Almond/Hillview Dr.
            (1) South Lincoln/West A St
                (a) North Lincoln St.
                (b) Pitt School Rd.
                (c) PSLS/Rehrmann Dr (previously contributed to the South Lincoln/West A St.)
                    (i) Pheasant Run Dr.
                    (ii) Manning Way

The remaining trunks within Sewer-Shed 3 collect flow from the western portion of the City, including the lift station service areas, although the PSLS was recently re-routed to connect to the East-West Trunk Sewer Connector in Sewer-Shed 2. It is described here as it was routed to flow through Sewer-Shed 3 during the flow monitoring period used to develop this SCSMP.

The 15-inch Cherry St./Porter Road Crossing trunk continues west to cross Porter Road and has an approximate capacity of 1.9 MGD. The 15-inch South Almond/Hillview Dr trunk is immediately upstream and has an approximate capacity of 1.6 MGD and is followed by the 15-inch South Lincoln/West A St. trunk, which has a capacity of approximately 1.8 MGD.

The 10-inch North Lincoln Street Trunk connects to the South Lincoln/West A St. trunk at its midpoint in the intersection of North and South Lincoln St. This trunk has a capacity of approximately 1.1 MGD and conveys flow from the LSLS. The 12-inch Pitt School Rd trunk connects to the upstream end of the South Lincoln/West A St. trunk at Pitt School Rd and has an approximate capacity of 0.8 MGD. The PSLS/Rehrmann Dr trunk was also connected to the upstream end of the South Lincoln/West A St. trunk at Pitt School Rd before being re-routed south to the East-West Trunk Connector.

Pitt School Lift Station Service Area

The main branch of the trunk network within the PSLS service area is the PSLS/Rehrmann Dr trunk, which ranges from 15-inches at the downstream end (PSLS location) to 10-inches at it's upstream end at the intersection of Rehrmann Dr. and Evans Rd. PSLS/Rehrmann Dr trunk capacity ranges from approximately 1.6 MGD at the downstream end to 0.8 MGD at its upstream end. The two main branches extending from this trunk include the Pheasant Run Dr. trunk and Manning Way trunk, both being 10-inches in diameter and having a flow capacity of approximately 0.8 MGD.

The approximate full pipe flow at the upstream and downstream end of each trunk sewer is presented in **Table 7-6**.

Table 7-6 Sewer-shed 3 Existing Trunk Sewer Capacity

Primary Trunk Sewer	Pipe Size (in)	Length (LF)	Upstream Capacity (MGD)	Downstream Capacity (MGD)
1) South 1st St – Part 1 (1)	15 to 27	3,400	2.2 (1)	6.0
a) South 1st St – Part 2	14	1,800	1.5	1.6
i) North 1st Street	10	2,300	0.6	0.9
ii) East A Street	12	1,500	0.6	1.3
b) Cherry St./Porter Road Crossing	15	2,700	1.6	1.9
i) South Almond/Hillview Dr.	15	2,000	1.2	1.6
(1) South Lincoln/West A St	15	2,500	1.3	1.8
(a) North Lincoln St.	10	3,100	0.7	1.1
(b) Pitt School Rd.	12	3,200	0.8	0.8
(c) PSLS/Rehrmann Dr (2)	10 to 15	3,300	0.8	1.6
(i) Pheasant Run Dr.	10	1,900	0.7	0.8
(ii) Manning Way	10	700	0.8	0.8

1) The South 1st St – Part 1 trunk is included in both Sewer-Shed 2 and Sewer-Shed 3 trunk sewer capacity tables. Only the upstream portion of this trunk, north of the North Interceptor Sewer, is within Sewer-Shed 3, this portion of the trunk has a capacity of approximately 2.2 MGD.
2) The PSLS was decommissioned after flow monitoring was conducted and Sewer-Shed boundaries were delineated. The PSLS service area was re-routed to flow by gravity through the East-West Trunk Sewer Connector where it now connects to Sewer-Shed 2. The assessment of these trunks presented under Sewer-Shed 3, corresponding to its original routing.

Main Trunk Sewers (42-inch & 27-inch)

The approximate full pipe flow at the upstream and downstream end of each main trunk sewer conveying flow from the collection system in the City to the WWTF is presented in **Table 7-7**.

Table 7-7 Existing Main Trunk Sewer Capacity

Primary Trunk Sewer	Pipe Size (in)	Length (LF)	Sewer-shed	Upstream Capacity (MGD)	Downstream Capacity (MGD)
27-inch Main Trunk	27	15,200	Main	5.6	5.8
42-inch Main Trunk	42	12,700	Main	15.2	15.6

7.2.2 Lift Station Capacity Evaluation

As previously discussed, the City's sewer system initially included two sewer lift stations, the Pitt School Lift Station (PSLS) and the Lincoln Street Lift Station (LSLS). The PSLS was recently decommissioned with the construction and connection of the East West Trunk Sewer Connector. The impact of re-routing this flow on the collection system is the purpose of the Scenario 3 existing system evaluation.

The Pitt School Lift Station (PSLS)

Before being decommissioned the PSLS collected and conveyed an average dry weather flow of approximately 0.13 MGD, as simulated in the hydraulic model. Under PWWF conditions, a peak flow of approximately 0.55 MGD is projected to occur at the PSLS. This exceeds the lift station's reliable pumping capacity (0.50 MGD) by 0.05 MGD. Had this lift station not been decommissioned it would have been recommended that the City increase reliable pumping capacity to provide sufficient capacity for PWWF conditions at buildout of the lift station's service area.

Lincoln Street Lift Station (LSLS)

The LSLS collects and conveys an ADWF of approximately 0.12 MGD under existing conditions. Under PWWF conditions, a peak flow of approximately 0.49 MGD is projected from its service area. This is less than the reliable pumping capacity of the LSLS, which is approximately 0.80 MGD. Therefore, no improvements are recommended on the basis of pumping capacity under existing conditions. It should also be noted that the City has an existing improvement project, CIP 315, to completely reconstruct the Lincoln Street Lift Station (LSLS) including new piping, overflow controls, below grade pumps, and an emergency generator to address condition deficiencies.

7.3 EXISTING SEWER SYSTEM PERFORMANCE EVALUATION

The results of each scenario are evaluated based on the LOS performance criteria, including the predicted HLR, d/D, and level of surcharge within the system.

Scenario 1 – Existing Dry Weather Flow

The existing system model was used to review any sewers that exceed the HLR and d/D LOS criteria under PDWF conditions. A PDWF of 1.76 is predicted to occur at the WWTF, and most of the existing system maintains HLR and d/D ratios below 50% of its total flow capacity. Model results for existing system PDWF performance are shown in **Appendix D**.

There is no surcharging predicted to occur in the existing system under existing dry weather conditions, with only minor exceptions to the LOS criteria which include sections along the Industrial Way, North Lincoln Street, South Almond/Hillview Dr., and South Lincoln/West A Street trunks. It should also be noted that flow velocities in the Industrial Way trunk exceed 6 ft/s. These locations should be carefully evaluated for capacity deficiencies during peak WWF conditions.

Scenario 2 – Existing Wet Weather Flow

A 10-year, 24-hour design storm was simulated within the model and the results were used to evaluate the existing system under peak wet weather flow conditions prior to the decommissioning of the PSLS. Under existing conditions, the design storm is predicted to generate a PWWF of 4.3 MGD at the WWTF. This level of storm event is predicted to cause surcharging in the 10-inch Industrial Way Trunk. Surcharge depth is predicted to remain under 1-foot, but the HLR of this trunk exceeds 142%.

The primary trunks serving Sewer-Shed 3 downstream of the PSLS are near their flow capacity in this scenario, including portions of the Cherry St./Porter Road Crossing, South 1st Street, South Almond/Hillview Dr., and South Lincoln/West A St trunks. The HLR slightly exceeds 100% in flatter segments of these trunklines. The HLR of sewers along these trunks ranges between 70% and 111%, but the d/D ratios remain under 90% and no surcharging is predicted to occur. Additional capacity is made available in these trunk sewers in Scenario 3, when flow is re-routed from PSLS, therefore no improvements are recommended for these sewers in this scenario. Gravity flow velocity remains under 8 fps under these conditions.

The following capacity constraint within the existing system has been identified in Scenario 2:

HGL Profile 1: Industrial Way, Figure E-1
Location: Along Industrial Way, between N 1st Street and Fitzgerald Way, Sewer-shed 1
Surcharged Manholes: All along Industrial Way Trunk
Proposed Improvements: CIP-E1
Problem Description: The 10-inch Industrial Way Trunk is undersized. Larger sewers exist upstream. Increasing the pipe size from 10 to 12-inches would provide sufficient capacity to convey the predicted existing PWWF at the existing pipe slopes.

To help identify the extent of the predicted surcharging, hydraulic grade line (HGL) profiles have been included in **Appendix E** for areas of concern. It should be noted that the profiles also include the results of other growth scenarios, to be discussed in the following sections. The specific flows and HGLs listed at the bottom of the HGL profiles correspond to the build-out scenario.

Scenario 3 – Existing Wet Weather Flow (without PSLS)

The only differences between the results of Scenario 2 and Scenario 3 are associated with the flow routing of the PSLS service area. Flow from the PSLS was originally routed through sewer-shed 3 before reaching sewer-shed 2, but in this scenario, flow was redirected to the East West Trunk Sewer Connector, which bypasses sewer-shed 3 and connects directly to the end of the Parkway Blvd. Trunk.

There are no changes to Sewer-Shed 1 under this scenario, and the capacity constraint in Industrial Way identified in Scenario 2 remains under this these conditions. This evaluation only presents capacity impacts associated with re-routing the PSLS that occur within Sewer-Sheds 2 and 3.

Directing flow from the PSLS to the East West Trunk Sewer Connector reduces the predicted HLR along the primary trunklines in Sewer-Shed 3. The HLR in the Cherry St./Porter Road Crossing, South

Almond/Hillview Dr., and South Lincoln/West A Street trunks is predicted to remain under 70% and d/D ratio is predicted to be less than 60%. The HLR of flatter segments of these trunks is no longer predicted to exceed 100%. The upstream end of the South 1st Street Trunk, within Sewer-Shed 3 is predicted to have an HLR of approximately 80% and a d/D ratio of 67%.

Re-routing flow from the PSLS creates approximately 0.5 MGD of residual capacity within Sewer-Shed 3 trunk sewers between the PSLS and the intersection of Parkway Blvd and South 1st St. The impacts associated with the conditions in this scenario relieve existing system constraints and therefore there are no additional capacity restrictions or recommendations identified for this scenario.

7.4 SUMMARY OF EXISTING SYSTEM EVALUATION & RECOMMENDATIONS

The only capacity constraint identified based on the evaluation of the existing system model results and the LOS evaluation was in the Industrial Way Trunk sewer where a section of the trunk is 10-inch diameter downstream of larger upstream sewers. Surcharging under existing conditions is predicted to be less than 1-foot above the pipe crown and the available freeboard (depth between the rim elevation and the pipe crown) is greater than 15-feet along the length of the trunkline, reducing potential for a sewer system overflow (SSO). It is recommended that the City address this capacity constraint before adding additional flow or allowing new development to occur in the upstream service area.

This CIP-E1 should be implemented if it hasn't already been completed. This CIP is sized to accommodate future flow in the recommended servicing strategy but should be monitored/studied/planned as development occurs, with more refined flow projections for the expanded service area a more detailed and specific improvement project can be designed for build-out conditions.

8.0 FUTURE COLLECTION SYSTEM EVALUATION

The purpose of this chapter is to present the evaluation of the City's future sewer collection system and its ability to meet the meet the City's recommended LOS performance and planning criteria under future level of development conditions. The future level of development scenarios and LOS criteria are described in detail in **Chapter 5.0** of this SCSMP. The future collection system evaluation includes both facility capacity and hydraulic performance evaluations. The system facility capacity evaluation assesses the sewer flow and pumping capacity needs under future development conditions to provide preliminary sizing and recommendations of **new infrastructure needed to expand the service area**. The hydraulic performance evaluation assesses the existing system's ability to meet the recommended LOS performance standards under future PWWF conditions, to identify capacity limitations within the **existing system and recommend improvements**.

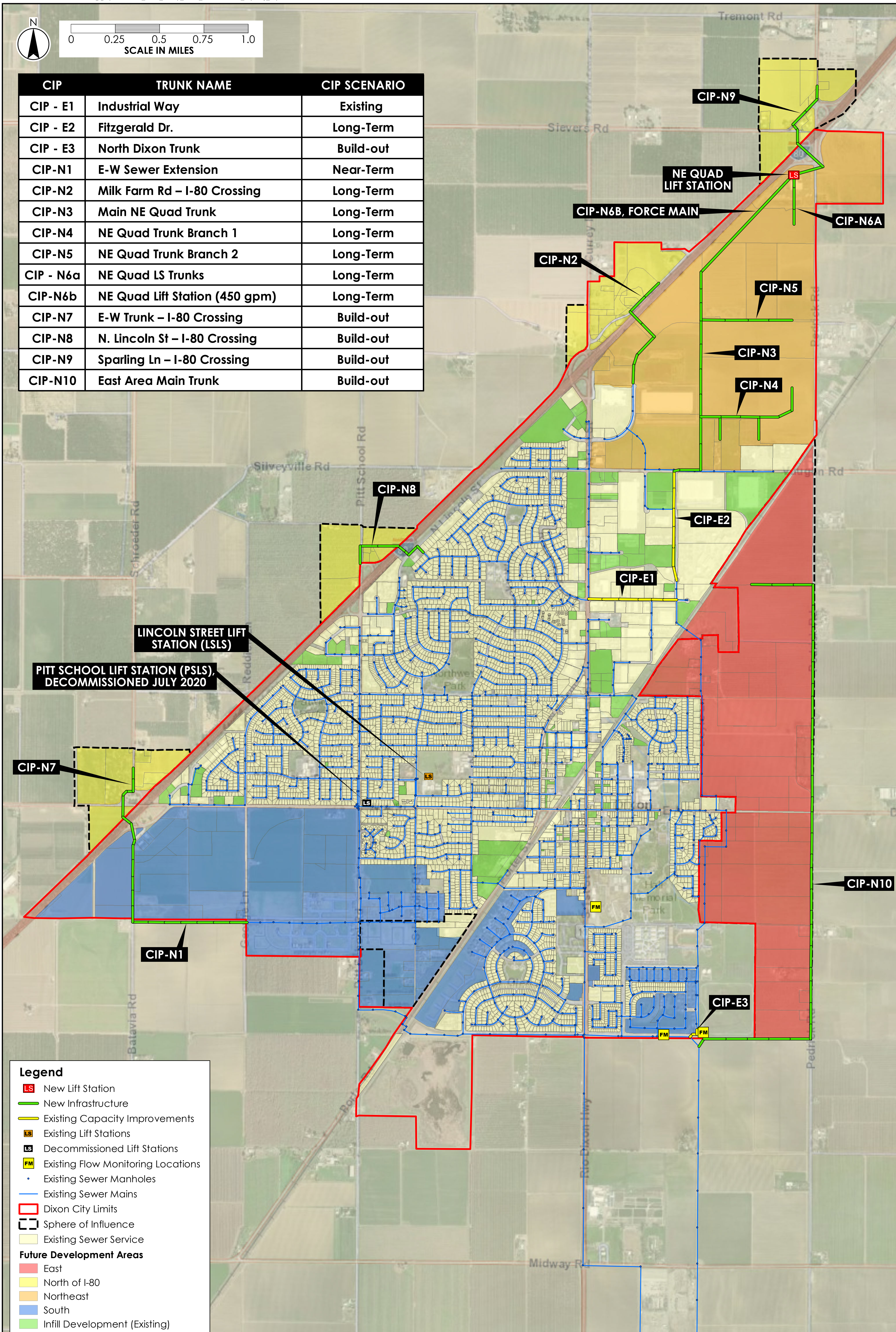
The existing system model evaluated in the previous chapter of this SCSMP was expanded to include flow estimates from future development areas and infill wastewater connections. The results of each future system scenario were evaluated to assess the impact of additional flow on the existing collection system and to determine the infrastructure needed to serve near-term, long-term, and build-out levels of development. This chapter identifies any existing sewer collection system deficiencies and provides recommendations for expanding the system to serve future development areas in each future level of development scenario.

These recommendations were used to develop a prioritized capital improvement program (CIP), including an estimate of probable construction costs. The CIP also includes improvement projects identified as a result of the condition assessment presented in **Chapter 9.0**. The recommended improvements and CIP are presented in **Chapter 9.4** of this SCSMP. Plan view figures of LOS criteria results for each modeled scenario are presented in **Appendix D**. Profile views of the future trunk network are available within the City's hydraulic model of the proposed system. Infrastructure improvements identified in this chapter, including new trunk sewers, lift stations, and existing system improvements needed to serve the future service area are shown on **Figure 8-1**.

This chapter is divided into the following sections:

- Future System Scenarios
- Future System Flow Projections
- Future Sewer System Capacity Evaluation
- Future Sewer System Performance Evaluation
- Main Trunk Capacity Evaluation
- Summary of Future System Evaluation & Recommendations

CIP	TRUNK NAME	CIP SCENARIO
CIP - E1	Industrial Way	Existing
CIP - E2	Fitzgerald Dr.	Long-Term
CIP - E3	North Dixon Trunk	Build-out
CIP-N1	E-W Sewer Extension	Near-Term
CIP-N2	Milk Farm Rd – I-80 Crossing	Long-Term
CIP-N3	Main NE Quad Trunk	Long-Term
CIP-N4	NE Quad Trunk Branch 1	Long-Term
CIP-N5	NE Quad Trunk Branch 2	Long-Term
CIP - N6a	NE Quad LS Trunks	Long-Term
CIP-N6b	NE Quad Lift Station (450 gpm)	Long-Term
CIP-N7	E-W Trunk – I-80 Crossing	Build-out
CIP-N8	N. Lincoln St – I-80 Crossing	Build-out
CIP-N9	Sparling Ln – I-80 Crossing	Build-out
CIP-N10	East Area Main Trunk	Build-out



Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 8-1
Future Collection System – Recommended Capacity Improvements

8.1 FUTURE SYSTEM SCENARIOS

This SCSMP evaluated three future scenarios including near-term, long-term, and build-out levels of development. The wastewater flow projections for each scenario are based on the City's General Plan 2040 land use information. Specific plans with more detailed development planning information were used where available for on-going development areas. A summary of the assumptions and planning parameters associated with each level of development scenario is presented in **Table 8-1.** Additional land use and planning information is presented in **Chapter 3.0**.

Table 8-1 Summary of Level of Development Scenarios

Level of Development	Model Scenario	Development Areas Added	Assumptions
Existing	Scenarios 1-3	• Existing Services	Current state of development and sewer service area.
Near-Term	Scenario 4	• Service Area Infill • On-going/South	Full development of the existing service area and on-going development areas
Long-Term	Scenario 5	• Northeast • North of I-80 (portion)	Full build-out of the City's existing city limits boundary, and on-going development areas, including the Northeast area and the adjacent portion of the SOI west of I-80.
Build-Out	Scenario 6	• East • North of I-80 (remaining)	Full buildout development of the City's SOI boundary.

8.2 FUTURE SYSTEM FLOW PROJECTIONS

Future wastewater flow projections used in the hydraulic model evaluation are based on the City's projected General Plan land uses, estimated population densities, and proposed unit flows as described in **Section 3.0**. The projected wastewater flow methodology and assumptions are discussed in detail in **Chapter 5.0**. The projected wastewater flow at buildout totaled 3.70 MGD under ADWF conditions. A summary of each scenario is shown as **Table 8-2**.

Table 8-2 Summary of Wastewater Flow Projections

Level of Development	Model Scenario	Total Area Serviced (Acres)	ADWF Added (MGD)	Modeled ADWF (MGD)	Modeled PWWF (MGD)
Existing	Scenarios 2 & 3	2,398	1.09	1.09	4.30
Near-Term	Scenario 4	3,247	0.98	2.07	7.53
Long-Term	Scenario 5	3,922	1.06	3.13	10.34
Build-Out	Scenario 6	4,772	0.57	3.70	12.69

8.3 FUTURE SEWER SYSTEM CAPACITY EVALUATION

The system facility capacity evaluation assesses the sewer flow and pumping capacity needs under future development conditions to provide preliminary sizing and recommendations for new infrastructure.

Future wastewater flow projections were distributed in the hydraulic model to develop preliminary infrastructure planning needed to extend the existing service area to serve future development areas.

The General Plan land use of each parcel within the future service area was used to develop its future wastewater flow projection. Future infrastructure conceptual routing is used within the model to connect each parcel to the existing system. The design parameters used to size the future system are summarized in **Section 6.4** of this SCSMP.

8.3.1 Scenario 4 – Near-Term Development

The near-term scenario proposes future infrastructure required to service the on-going development areas, specifically the Homestead Development. This scenario adds 0.98 MGD of ADWF to the model from infill development within the existing service area and undeveloped portions of on-going development areas. The only portion of these future development areas requiring extension of the existing collection system is the western portion of the Homestead Development. This new trunk extension will serve an ADWF of approximately 0.19 MGD, equating to approximately 40% of the total flow projected to be contributed by the 1,880 future EDUs associated with the Homestead Development. The remaining flow will contribute to the East West Trunk Sewer Connector and its two E-W Branches.

The 18-inch E-W Branch 1 trunk was extended west to Batavia Rd., where the new trunk turns north until it meets I-80. The new trunk sewer, referred to as the E-W Sewer Extension, ranges in diameter from 15-inch at its downstream point of connection to the existing system, to 10-inches in diameter at its upstream end where it meets I-80. The capacity of the proposed trunk extension ranges from 1.84 MGD at its downstream end to 0.76 at its upstream end. The parameters associated with the proposed E-W Sewer Extension are summarized in **Table 8-3**.

Table 8-3 Near-Term Development New Infrastructure: E-W Sewer Extension

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	15	1,145	0.0020	21.7	1.84
2	12	2,250	0.0025	18.1	1.16
3	10	1,190	0.0030	16.0	0.76
	Total:	4,585			

New infrastructure is laid out using the following parameters an HLR of approximately 70% under build-out conditions, a minimum full flow velocity of 2.0-2.5 fps, and a minimum depth of approximately 6.0-feet below the ground surface. Average pipeline depth was determined from the average depth of the upstream and downstream manholes for each sewer segment. Future manhole rim elevations were set using LiDAR elevation data available on the Solano County website.

The existing collection system has an overall average depth of approximately 9.6 feet and the existing trunk sewer network (sewers > 8-inches) has an average of approximately 14.7-feet. The depth of the proposed E-W Sewer Extension is 15.5 feet at its upstream end where it meets I-80 and 21.7 feet at its downstream end where it connects to the existing system. The sewer depth was maintained to allow it to be further extended to serve the immediately adjacent development area north of I-80.

8.3.2 Scenario 5 – Long-Term Development

The long-term development scenario extends the existing collection system to the remaining undeveloped portion of city limits, adding approximately 1.06 MGD of ADWF to the model. The proposed collection system improvements extend from the existing trunk sewers in sewer-shed 1 to reach parcels within the Northeast development area and the portion of the north of I-80 development area that lies within the current extent of city limits. The infrastructure needed to serve the area and the associated flow projection were developed and incorporated into the model.

There are three primary regions of the expanded service area that will require new collection system infrastructure:

- North of I-80
- Gravity Service (Southern Northeast Quad)
- Lift Station Service (Northern Northeast Quad)

The three regions are proposed to connect to the existing system at two points of connection. The Northeast Quad will connect to the upstream end of the Fitzgerald trunk and the area North of I-80 will tie into the existing system at the upstream end of the Vaughn Rd/Dorset Drive trunk.

The Northeast Quad is divided by the portion that can be served by gravity sewers and the portion requiring a lift station. A new lift station will be needed to serve the most north-east parcels in the overall planning area. The gravity and lift station service areas were delineated using LiDAR data to identify low lying areas and maximize gravity service potential. The lift station force main is proposed to tie into the collection system at the downstream gravity service region of the expanded service area.

North of I-80

The first area of new infrastructure, serving the western portion of the expanded service area, extends the Vaughn Road/Dorset Drive trunk to cross I-80 and provide service to the area across I-80 that exists within city limits. The new Milk Farm Rd. – I-80 Crossing trunk will need to be 10-inches at its downstream end and 8-inches at the upstream end, with a capacity of 0.45 MGD upstream and 0.78 MGD downstream. The physical parameters of this trunk are summarized in **Table 8-4**.

Table 8-4 Long-term Development New Infrastructure: Milk Farm Rd, I-80 Crossing

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	10	665	0.0030	11.30	0.78
2	8	2,350	0.0035 (1)	7.73	0.45
	Total:	3,015			

1) An end pipe slope of 0.0052 was assumed to ensure sufficient pipe flow velocity in accordance with the City's design standards.

Northeast-Quad – Gravity Service Area

The main trunk running through the gravity service area is referred to as the Main NE Quad trunk and ranges from 21-inches to 12-inches in diameter. It connects to the existing system at the intersection of Vaughn Road and Fitzgerald Drive. The physical parameters of this trunk are summarized in **Table 8-5**.

Table 8-5 Long-term Development New Infrastructure: Main NE Quad Trunk

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	21	1,854	0.0010	18.95	3.30
2	18	2,251	0.0015	17.32	2.64
3	12	1,110	0.002 (1)	18.36	1.03
	Total:	5,215			

1) The force main discharge location for the pump station is proposed at the upstream end of this sewer.

There are two main trunk branches proposed to extend from the Main NE Quad trunk to the east. The southern branch, Branch 1 ranges from 12 to 10 inches in diameter, with two 8-inch collector sewers, which were included to ensure sufficient depth would be maintained. Some of the flow in the southern portion of this service area may alternatively be served through the existing trunk in East Vaughn Road, as it appears there is available capacity.

Flow from these areas was routed to new infrastructure to provide a conservative approach to planning and an alternative in the event that topography doesn't allow for these areas to be connected to the existing Vaughn Road – East trunk. The physical parameters of this trunk are summarized in **Table 8-6**.

Table 8-6 Long-term Development New Infrastructure: NE Quad Trunk Branch 1

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	12	1,359	0.0020	15.77	1.03
2	10	1,330	0.0035	10.14	0.84
3	8	1,040	0.0052	11.88	0.57
	Total:	3,730			

The northern branch, branch 2 extends from the NE Quad Trunk connecting at its transition from 18 to 12-inches in diameter. The physical parameters of this trunk are summarized in **Table 8-7**.

Table 8-7 Long-term Development New Infrastructure: NE Quad Trunk Branch 2

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	12	1,084	0.0020	15.56	1.03
2	10	523	0.0035	11.88	0.84
3	8	523	0.0052	8.32	0.57
	Total:	2,130			

Northeast-Quad – Lift Service Area

The proposed lift station will service approximately 115 acres of the Northeast Quad under long-term development conditions. In Scenario 6, full buildout conditions, it is proposed to also serve the adjacent area north of I-80 for full build-out service area of approximately 195 acres. The reliable capacity of the lift station under long-term development conditions will need to be approximately 0.45 MGD, to be expanded to 0.65 MGD at buildout. The proposed lift station discharges at the upstream end of the Main NE Quad trunk. It is recommended that the City install dual 4-inch force mains to accommodate the phasing of development in the area. The physical parameters of this trunk are summarized in **Table 8-8**.

Table 8-8 Long-term Development New Infrastructure: NE Quad Lift Station

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	10	700	0.0025	15.56	0.71
2	8	1,150	0.0052	11.88	0.57
3	4, Dual	3,140	NA (1)	-	-
	Total:	4,990			

1) This is the lift station force main. The lift station itself will be needed in addition to the conduits listed in the table. It will require a reliable pumping capacity of 0.45 MGD under long-term conditions and 0.65 MGD at build-out.

The proposed collection system tributary to the new lift station includes an 8-inch sewer extending south and a 10-inch sewer is proposed to cross Pedrick Rd. to the east and will also convey flow from the area north of I-80 at build-out.

8.3.3 Scenario 6 – Build-Out Development

The build-out development scenario extends the existing collection system to the remaining undeveloped portion of the SOI planning area, adding approximately 0.57 MGD of ADWF to the model. The proposed collection system improvements extend from the existing trunk sewers to reach parcels within the remaining north of I-80 development areas and east development area. The infrastructure needed to serve the area and the associated flow projection were developed and incorporated into the model.

The proposed buildout infrastructure improvements include the following:

- E-W Trunk – I-80 Crossing
- N. Lincoln St. – I-80 Crossing
- Sparling Ln. – I-80 Crossing
- East Area Main Trunk

The first segment of new infrastructure further extends the proposed E-W Trunk extension, required for near-term development (Scenario 4) further north to cross I-80. It is proposed to be an 8-inch sewer at the minimum slope required to ensure a minimum full pipe flow velocity 2.5 fps. The physical parameters of this trunk are summarized in **Table 8-9.**

Table 8-9 Build-out Development New Infrastructure: E-W Trunk, I-80 Crossing

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	8	1,982	0.0052	11.72	0.57

The second proposed trunk extends the existing collection system in North Lincoln Street to serve the adjacent area North of I-80. It is also proposed to be an 8-inch sewer at the minimum slope required to ensure a minimum full pipe flow velocity 2.5 fps. The physical parameters of this trunk are summarized in **Table 8-10**.

Table 8-10 Build-out Development New Infrastructure: N. Lincoln St, I-80 Crossing

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	8	2,100	0.0052	9.53	0.57

The third proposed trunk is also needed to serve one of the development areas north of I-80, extending from the lift station service area proposed in the long-term level of development scenario in the Northeast Quad. This trunk is proposed to be a 10-inch sewer at the downstream end and reduce to an 8-inch sewer at the upstream end after crossing I-80. The expansion of the LS service area will also require that the reliable pumping capacity of the lift station be expanded from 0.45 MGD to 0.65 MGD. The physical parameters of this trunk are summarized in **Table 8-11**.

Table 8-11 Build-out Development New Infrastructure: Sparling Ln, I-80 Crossing

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	10	805	0.0025	14.57	0.71
2	8	1,610	0.0040 (1)	12.03	0.50
	Total:	2,415			

1) An end pipe slope of 0.0052 was assumed to ensure sufficient pipe flow velocity in accordance with the City's design standards.

The largest of the proposed buildout improvements is the 2.8-mile trunk needed to provide service to the East development area, which is approximately 645 acres along the eastern border of the SOI boundary. The proposed East Area Main trunk will require a PWWF capacity of 3.25 MGD at its downstream end where it connects at the confluence of the North Dixon trunk and the 48-inch Main Trunk. The new trunk alignment is proposed to run along the SOI border, parallel to the Dickson Creek Canal before turning north on Pedrick Rd. and will require sewers section ranging from 8-inches to 21-inches in diameter at minimum slopes. The physical parameters of this trunk are summarized in **Table 8-12**.

Table 8-12 Build-out Development New Infrastructure: East Area Main Trunk

Parts	Diameter (in)	Length (LF)	Slope (ft/ft)	Average Depth (ft)	Full Pipe Capacity (MGD)
1	21	2,708	0.0010	16.46	3.25
2	18	3,967	0.0012	16.12	2.36
3	15	3,984	0.0015	16.22	1.62
4	12	2,618	0.0022	10.35	1.08
5	8	1,441	0.0052	12.93	0.57
	Total:	14,718			

8.4 FUTURE SEWER SYSTEM PERFORMANCE EVALUATION

The proposed future system infrastructure and flow projections described in the previous sections were incorporated into the hydraulic model in order to assess the impacts of future growth on the hydraulic performance of the existing collection system. The hydraulic performance evaluation assesses the existing system's ability to meet the recommended LOS performance standards under future PWWF conditions to identify capacity limitations within the existing system and recommend improvements to provide sufficient capacity.

To help identify the extent of the predicted surcharging, hydraulic grade line (HGL) profiles have been included in **Appendix E** for areas where CIPs were identified. The capacity improvements identified under each future develop of development scenario are further described below.

8.4.1 Scenario 4 – Near-Term Development

The near-term scenario adds flow from infill development within the existing service area and on-going development areas. The PWWF predicted to occur at the WWTF in this scenario is approximately 7.5 MGD. The addition of this flow is not predicted to impose any new capacity constraints on the existing system but is expected to exacerbate the existing capacity constraint identified in Industrial Way.

The following portions of the system are identified as capacity concerns under near-term development:

- The shallow sloped sewer in the Parkway Blvd trunk immediately downstream of the intersection of Parkway Blvd and South 1st Street
- The upstream portion of the South 1st Street trunk, consisting of the two sewer segments between W. Cherry St. and the North Interceptor Sewer.

The HLR in these sewers is predicted to approach 100%, but they are still considered to be within the LOS criteria. LOS criterion requires that the HLR does not exceed 100% and there is no surcharging in the collection system at the specified level of development. Therefore, capacity improvements are not recommended for these sewer segments, but it is recommended that these sewers be closely monitored by the City.

The following improvements are recommended to relieve the capacity constraint within the existing system identified in Scenarios 2, 3, and 4:

HGL Profile 1: Industrial Way, Figure E-1
Location: Along Industrial Way, between N 1st Street and Fitzgerald Way, Sewer-shed 1
Surcharged Manholes: All along Industrial Way Trunk
Proposed Improvements: CIP-E1
Problem Description: The 10-inch Industrial Way Trunk is undersized. Larger sewers exist upstream. Increasing the pipe size from 10 to 12-inches would provide sufficient capacity to convey the predicted PWWF at the existing pipe slopes.

8.4.2 Scenario 5 – Long-Term Development

The long-term scenario adds flow from the remaining undeveloped portions of city limits, consisting primarily of the North East Quad. The PWWF expected to occur at the WWTF is approximately 10.3 MGD under long-term development conditions. The addition of flow upstream of sewer-shed 1 causes minor surcharging in the 21-inch portion of the Fitzgerald Dr. trunk sewer. The first two segments of the existing Fitzgerald Dr. trunk sewer are 27-inches in diameter, and to eliminate this capacity constraint it is recommended that the remaining 21-inch portion be upsized to 27-inches.

The 21-inch portion of the Fitzgerald trunk is projected to flow above its full flow capacity with a HLR of between 118% and 133% under long-term conditions. Surcharging along the trunk is predicted to be minor and flow depth remains below the pipe crown. The d/D ratio for this portion of the trunk remains within LOS criteria, ranging between 90 and 99%. The minimum freeboard, or depth between the manhole rim elevation and the HGL exceeds 15-feet under long-term development conditions.

The downstream end of the 30-inch North Dixon Trunk, at the location of flow monitoring Site 1, also fails to meet LOS criteria under long-term conditions depending on the actual slope of the sewer. Two segments in the 30-inch trunk, making a 90-degree turn at Parkway Blvd. have been reconstructed several times and information from record drawings was incorporated into the model, but the actual slope of these sewers should be verified to confirm the need for improvements in this portion of the system. Segments of this sewer at slopes of less than 0.0003 ft/ft fail to meet LOS criteria under long-term development conditions.

If the slopes of sewers are found to be less than 0.0003 ft/ft, they will exceed 100% HLR. The sewer segment in the model at 0.00025 ft/ft are predicted to have an HLR of 107% and the sewer at a slope of 0.00019 ft/ft is predicted to have an HLR of 122%.

Although these sewers fail to meet LOS criteria for HLR, no surcharging is predicted to occur in these sewers. The d/D ratio of these segments is predicted to be 82%. Therefore, it’s recommended that the City confirm the slopes of these sewers and monitor the flow in this area of the system as development occurs to further refine the need for improvements in this area of the system. Under long-term conditions, improvements are not recommended at the downstream end of the North Dixon Trunk and the Fitzgerald Trunk and they are only identified as a capacity concern. The capacity concerns identified under Scenario 4 remain under long-term conditions. These concerns exist outside of sewer-shed 1 and are not impacted by flow added in this scenario.

The following improvements are recommended to relieve the capacity constraint within the existing system identified in Scenario 5:

HGL Profile 2: Fitzgerald Dr, Figure E-2
Location: Along Fitzgerald Dr., between Industrial Way and Vaughn Rd, Sewer-shed 1
Surcharged Manholes: 21-inch portion of Fitzgerald Dr. Trunk
Proposed Improvements: CIP-E2
Problem Description: The 21-inch portion of the Fitzgerald Dr. trunk exceeds its full flow capacity with the addition of flow in the upstream system. Increasing the pipe size from 21 to 27-inches would provide sufficient capacity to convey the predicted PWWF at the existing pipe slopes.

8.4.3 Scenario 6 – Build-Out Development

The build-out level of development scenario considers the impacts of flow from the City's entire General Plan 2040 development area. The build-out model simulates an ADWF of approximately 3.70 MGD. The PWWF expected to occur at the WWTF is approximately 12.7 MGD. The long-term service area is expanded to include the East Development Area and the remaining areas North of I-80. The addition of flow from these areas does not appear to induce any additional capacity constraints in the existing system but does exacerbate those previously identified. Specifically, increasing the need to address capacity limitations and concerns identified in Scenario 5.

The predicted level of surcharge in the 21-inch portion of Fitzgerald trunk rises to approximately 5-inches above the pipe crown in the upstream segments. The HLR is predicted to range from 127% to 142% in the 21-inch portion of the trunk. Increasing this sewer from 21-inch to 27-inch as recommended in Scenario 5, will provide sufficient capacity for build-out flows. Given the minor surcharging and deep sewers, the recommendation to monitor this reach of sewer over time is still recommended over proposed replacements. If future flow monitoring efforts along this trunk indicate higher flows than projected in this SCSMP, additional capacity improvements may be required.

The capacity constraint identified at the downstream end of the North Dixon Trunk at Parkway Blvd. is further exacerbated by the addition of build-out flow. The HLR of the two sewer segments making a 90-degree turn at Parkway Blvd. (location of FM Site 1) is predicted to be 130% under build-out conditions and is further exacerbated by relieving capacity constraints upstream in the Industrial Way and Fitzgerald sewer trunks.

The sewer is not predicted to surcharge above the pipe crown and has adequate depth. The City could elect to implement an extensive FOG reduction program as an alternative to upsizing the sewer to 36-inches to relieve the hydraulic constraints at the main confluence in the system, which is known to have maintenance issues associated with grease build-up and blockages.

In addition to those identified in preceding scenarios, the following capacity concerns were identified under build-out conditions:

- After relieving the upstream capacity limitations in sewer-shed 1 and flow is unattenuated, a new capacity concern emerges in the upstream end of the North Dixon Trunk, along Doyle Ln from East A St. to East H St.
- The addition of flow from the Milk Farm Road – I-80 Crossing causes the 10-inch upstream end of Vaughn Rd/Dorset Drive trunk to near its full flow capacity, having an HLR of approximately 96%.

These sewers are predicted to have HLR near or at 100%, therefore at this level of planning they are only considered capacity concerns, and it is recommended that these sewers be monitored for indicators of capacity issues as development occurs and the required flow capacity can be further refined.

The following improvement is recommended to relieve the capacity constraint within the existing system identified in Scenario 6, an alternative to this improvement includes an extensive FOG reduction and monitoring program:

HGL Profile 3: North Dixon Trunk, Figure E-3
Location: E Parkway Blvd., the three segments immediately upstream of the confluence with the Parkway Blvd and 42-inch Main Trunk, sewer-shed 1
Surcharged Manholes: NA
Proposed Improvements: CIP-E3
Problem Description: The 21-inch portion of the Fitzgerald Dr. trunk exceeds its full flow capacity with the addition of flow in the upstream system. Increasing the pipe size from 21 to 27-inches would provide sufficient capacity to convey the predicted PWWF at the existing pipe slopes.

8.5 MAIN TRUNK CAPACITY EVALUATION

Although no capacity constraints along the 42-inch Main Trunk were identified in future development scenarios, a redundancy analysis of the main trunk sewers in the City's collection system was performed to identify residual capacity and conditions under which improvements may be necessary.

The 42-inch Main Trunk is currently the only conduit conveying flow from the City's collection system to the WWTF and is considered a critical piece of collection system infrastructure. As previously discussed, the City intends on repairing the 27-inch Main Trunk to bring it back into service. This would provide another path from the collection system to the WWTF and provide some level of redundancy in the event that the 42-inch sewer needed to be taken offline.

However, repairing the trunk at its current slope, diameter, and alignment would not allow sufficient wet weather flow capacity to provide full redundancy for the 42-inch Main Trunk beyond existing level of development conditions. The capacity of the 27-inch Main Trunk is approximately 5.7 MGD (assuming a slope of 0.0008 ft/ft) and the PWWF projected in future development scenarios exceeds this capacity. In other words, beyond existing conditions, the 27-inch Main Trunk will not have the capacity to convey PWWF from the entire service area.

In addition to this capacity limitation associated with the 27-inch Main Trunk, the alignment is such that bypass pumping would be required to convey flow from sewer-shed 1 and the Parklane development area to the 27-inch Main Trunk. Pumping would be required to divert flow that does not originate upstream the intersection of Parkway Blvd. and South 1st Street where the 27-inch sewer connects to the existing system.

If the City intended on bypassing all flow through the 27-inch Main Trunk to temporarily take the 42-inch Main Trunk out of service, it would need to be done under dry weather flow conditions with either a storage or flow equalization component, or by allowing temporary surcharging to occur along the 27-inch Main Trunk.

The full flow pipe capacity of the 42-inch Main Trunk is approximately 15.4 MGD (assuming a slope of 0.00055 ft/ft) which is sufficient to convey the projected PWWF from the entire collection system under build-out conditions. The hydraulic model projects a PWWF of approximately 12.7 MGD to occur in the 42-inch Main Trunk under buildout development conditions. The limiting slope required to convey this flow without surcharging is approximately 0.0004 ft/ft.

Therefore, if the City finds that the actual slope of the 42-inch sewer is less than 0.0004 ft/ft, the need to repair the 27-inch sewer increases. This could also be impacted by aging infrastructure with a higher rate of I/I, actual build-out connections and flows beyond what has been assumed in this SCSMP, or other parameters that may ultimately impact the projected PWWF under future conditions. Currently, the 42-inch Main Trunk is projected to have approximately 2.5 MGD of residual capacity at build-out.

Bringing the 27-inch Main Trunk back into service would provide additional capacity in the downstream end of the Parkway Blvd Trunk and the 42-inch Main Trunk. It would reduce capacity concerns identified in the shallow sloped sewer in the Parkway Blvd trunk immediately downstream of the intersection of

Parkway Blvd and South 1st Street and reduce the HGL at the confluence of the North Dixon Trunk, the 42-inch Main Trunk, and the Parkway Blvd Trunk. Reducing the HGL at this confluence could ultimately reduce the risk for SSOs in the 90-degree bend in the North Dixon Trunk identified as a capacity concern under long-term conditions and identified as CIP 3 under build-out conditions.

When it comes to the need to repair the 27-inch Main Trunk, the City should consider at the results of this SCSMP, historical and projected groundwater trends in the area, and the condition of both main trunklines in order to determine if and when it may need to be brought online.

It is recommended that the City evaluate the need, cost, and feasibility of repairing and eliminating excessive GWI in the 27-inch Main Trunk in comparison to the cost of removing/abandoning the line entirely and constructing a new main trunk at the required size for bypassing all flow, with an alignment that minimizes GWI and pumping, and that could be implemented at the time when additional capacity may be required or bypass is needed. If nothing is done to rectify GWI in this sewer, with the intention of maintaining it as a temporary backup to the 42-inch sewer, it is likely that the City will continually have to dewater and treat GWI in the trunk line every time ground water reaches this threshold elevation.

Groundwater elevations have likely been lower than normal conditions due to drought conditions, which is why the City has not had to dewater the trunk since 2011. It should be noted that high ground water conditions sustained over a long period of time could significantly impact the system. For this reason, the historical and projected return frequency of high groundwater elevations and groundwater mapping should be of high interest to the City as existing infrastructure ages and becomes more susceptible to GWI.

The City should consider conducting an cost-benefit analysis considering the following

(1) Do nothing: the cost of dewatering and treating the excess GWI.

(2) Repair and utilize the trunk line: the cost/feasibility to repair the line and eliminate GWI

(3) Cost of Constructing a new line: cost of abandoning the existing 27-inch line and finding a new alignment.

8.6 SUMMARY OF FUTURE SYSTEM EVALUATION & RECOMMENDATIONS

The proposed new trunk sewer infrastructure needed to expand the service area through buildout development is summarized in **Table 8-13**. The capital improvement projects proposed to address capacity constraints in the existing system are summarized in **Table 8-14**. The areas of the system considered to be capacity concerns do not require improvements but should be closely monitored by the City for capacity issues, as they are predicted to be just under the LOS threshold for requiring improvements. Areas identified as capacity concerns in each scenario are summarized in **Table 8-15**.

Table 8-13 Summary of Proposed New Infrastructure

ID	Scenario	Name	Diameter (in)	Length (LF)	Downstream Capacity (MGD)	Upstream Capacity (MGD)
N1	Near-term	E-W Sewer Extension	10 to 15	4,590	1.84	0.76
N2	Long-term	Milk Farm Rd., I-80 Crossing	8 to 10	3,010	0.78	0.45
N3	Long-term	Main NE Quad Trunk	12 to 21	5,220	3.30	1.00
N4	Long-term	Main NE Quad Trunk - Branch 1	8 to 12	3,730	1.03	0.57
N5	Long-term	Main NE Quad Trunk - Branch 2	8 to 12	2,130	1.03	0.57
N6a	Long-term	NE Quad LS Sewer-shed	8 to 10	1,850	1.03	0.57
N6b	Long-term	NE Quad LS (0.65 MGD) (1)	Dual 4-in	3,140	0.71	0.57
N7	Build-out	E-W Trunk, I-80 Crossing	8	1,980	0.57	0.57
N8	Build-out	N Lincoln St., I-80 Crossing	8	2,100	0.57	0.57
N9	Build-out	Sparling Ln., I-80 Crossing	8 to 10	2,410	0.71	0.50
N10	Build-out	East Area Main Trunk	8 to 21	14,720	3.25	0.57

1) Reliable pump station capacity of 0.65 MGD at build-out and 0.45 MGD under long-term development conditions.

Table 8-14 Summary of Capacity Related Improvements

CIP	Trunk Sewer	Scenario Identified	Length (LF)	Current Size (in)	Proposed Size (in)	Notes
E1	Industrial Way	Existing PWWF	2,100	10	15	A 12-inch sewer is required under existing conditions, but a 15-inch will be needed for future scenarios.
E2	Fitzgerald Dr.	Long-Term	2,550	21	27	Surcharging is not expected to exceed the pipe crown until build-out.
E3	North Dixon Trunk	Build-out	300	30	36	Surcharging is not expected to exceed the pipe crown.

Table 8-15 Summary of Capacity Concerns (95 – 100% HLR)

Area of Concern	Trunk Sewer	Scenario Identified	Length (LF)	Current Size (in)	Proposed Size (in)	Notes
1	Parkway Blvd.	Near-term	800	27	NA	Shallow sloped segment, s = 0.0007 ft/ft. Can be eliminated by bringing the 27-inch Main Trunk into service
2	South 1st Street	Near-term	800	15	NA	W Cherry St to Silveyville Cemetery/North Interceptor Sewer
-	North Dixon Trunk	Long-term	300	30	36	Recommended as improvement 3 under build-out conditions. Only a concern under long-term conditions
3	North Dixon Trunk	Build-out	3,100	27	NA	Doyle Ln from E A Street to E H Street.
4	Dorset Drive	Build-out	770	10	NA	E Dorset Drive, two segments flowing south
5	42-inch Main Trunk	Build-out	NA	42	NA	Records indicate a slope of slope = 0.00055 ft/ft. Capacity concern only exists at build-out if slopes are found to actually be < 0.0004 ft/ft

Infrastructure improvements identified in this chapter, including new trunk sewers, lift stations, and existing system improvements needed to serve the future service area are shown on **Figure 8-1**.

9.0 CONDITION ASSESSMENT

The purpose of this chapter is to present a detailed inventory of the City's existing wastewater collection system and its current condition. The information presented in this chapter was prepared by NexGEN. Stantec summarized and presented the information provided by NexGen in this chapter for consistency with the overall document. The original information provided by NexGen is presented in **Appendix F**.

This chapter is divided into the following sections:

- Wastewater System Asset Overview
- Sewer Planning Criteria
- CCTV & Condition Inspections
- Repair & Replacement Program Improvements

9.1 WASTEWATER SYSTEM ASSET OVERVIEW

A detailed inventory of the City's existing sewer collection system and its condition was developed from the City's GIS database, City improvement plans, interviews with City Staff, and City inspection records.

The City's wastewater system asset inventory resides within a layer on the City's GIS database. The GIS database include manhole numbers locations, rim elevations, pipe invert elevations, sewer numbers, sewer diameter, and pipe material. This information was then used to develop other information used in the analysis including pipeline depths and slopes.

Other information developed and organized as part of this analysis include:

(1) **Asset install dates and assumed useful life**. Installation dates were taken from improvement plans and, in some cases, approximated after discussions with City Staff. Assumed pipeline life, termed the asset's useful life, were developed for each pipe material. Sewer pump station life was based on typical industry values and observed station condition.

(2) **Asset replacement costs** were based on recent construction bids for pipeline and replacement, and new pump station construction.

9.1.1 Sewer Assets

A sewer map generated by the City's GIS has been shown as **Figure 2-1** in **Chapter 2.0**. The sewers and lift station are described below.

Sewers

The City has sewer lines as old as from the 1950s. The City has also performed several sewer improvement projects including replacing pipe along Vaughn Road and Lincoln Hwy in 2018.

The City's sewers are comprised of both reinforced concrete pipe (RCP) and vitrified clay pipe (VCP). The larger 27-inch and 42-inch trunk lines to the wastewater treatment plant are VCP. The City of Dixon Standard Specifications require new sewers to be VCP.

The industry recommendation for the useful life of a sewer line depends on the pipe material and environmental factors. VCP is typically a more corrosion-resistant material than RCP and generally has a longer useful life. For RCP, the industry standard useful life is estimated at 60 years. For VCP, the standard useful life ranges from 60-120 years depending on manufacturer, quality of installation, pipe depth, and flow velocities. For this analysis, the useful life of VCP was assumed at 90 years. City staff routinely perform CCTV inspections to better understand the pipe's actual condition.

Sewer replacement costs are based on recent construction data in the Sacramento area and are tabulated in **Table 9-1**. The costs are shown for different pipe sizes and pipe depths and include construction and allowances for design, construction management, and contingencies. The total asset replacement cost was calculated using the cost per foot and the pipe length and depth GIS attribute.

Table 9-1 Sewer Line Replacement Costs (1) (2) (3)

Pipe Diameter (in)	Depth (ft)										
	8	10	12	14	16	18	20	22	24	26	28
6	$94	$102	$108	$116	$125	$138	$155	$176	$204		
8	$113	$126	$133	$143	$154	$170	$191	$217	$250		
10	$137	$156	$165	$176	$190	$210	$235	$268	$310		
12	$164	$183	$193	$206	$223	$247	$277	$315	$366	$422	$501
15	$204	$227	$238	$252	$270	$295	$326	$363	$412	$470	$553
18		$275	$287	$302	$321	$349	$380	$420	$469	$528	$611
21		$332	$345	$361	$381	$409	$443	$485	$535	$595	$680
24		$391	$411	$428	$450	$480	$516	$559	$609	$670	$758
27		$414	$430	$451	$480	$516	$559	$611	$673	$738	
30		$464	$481	$505	$535	$571	$615	$670	$734	$802	
36		$508	$528	$553	$584	$622	$669	$727	$795	$870	
42		$675	$701	$732	$770	$816	$880	$956	$1,044		
48		$742	$742	$803	$846	$900	$966	$1,050	$1,144		
54		$860	$860	$925	$969	$1,024	$1,092	$1,179	$1,282		

1) This table represents the cost per foot for sewer pipe replacement. The cost includes trenching, removal of existing pipe, new pipe, subgrade prep, pavement restoration, and labor.
2) These costs include a 20% contingency and an additional 15% for admin and design costs.
3) VCP (24" or smaller), RCP (greater than 24")

Sewer Lift Station

At the time of data collection, the City operated the Lincoln Street Lift Station (LSLS) and Pitt School Lift Station (PSLS); however, the PSLS was recently decommissioned and has been omitted from this analysis. The LSLS is a duplex station with separate wet-well and dry-pit enclosed in a steel can. The lift station was originally manufactured as a package system by the Smith and Loveless Company and was installed in 1985. In 2004 the City installed the Smith and Loveless “Xpeller” to reduce the extent of pump plugging.

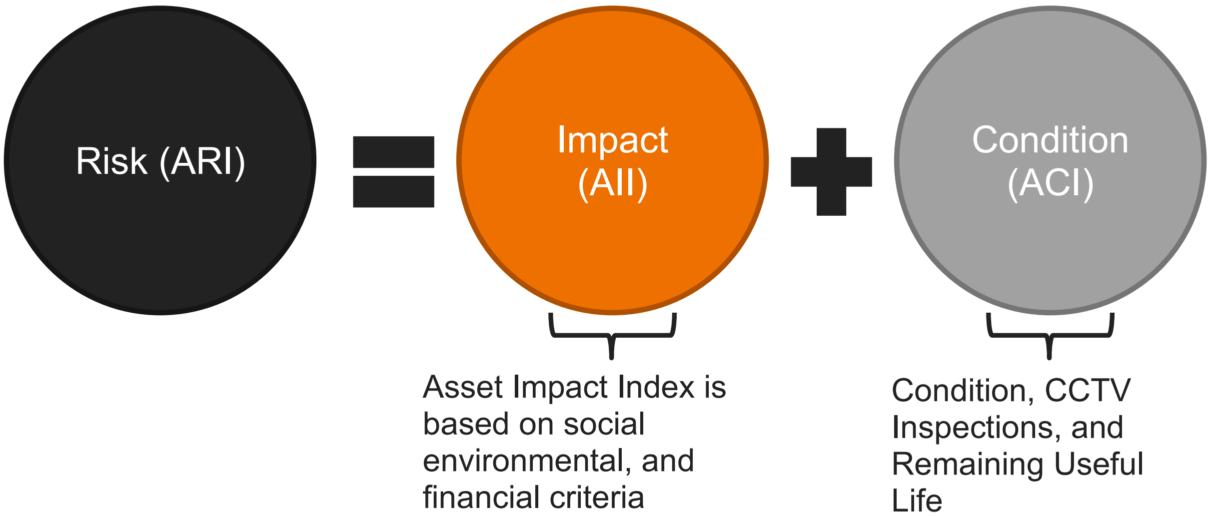
The station has a rated pumping capacity of 550 gallons per minute (gpm). The station consists of two pumps that alternate as lead or lag, isolation valves, and an above-ground control panel. The site does not have a dedicated backup generator during power outages but has a manual transfer switch for connection to a portable generator. The station lacks remote monitoring capabilities aside from a 4-channel autodialer system that calls out during pump failure.

The lift station replacement cost was calculated based on recent (Summer of 2020) construction bids for a rehabilitation and upgrades for similar size and type of sewer lift stations at the City of West Sacramento. That project scope included a new wet well, new submersible pumps, new control panels, new SCADA system, a valve vault, and a backup generator. The lift station replacement cost is estimated at $1.4 million and includes allowances for design and administration and contingencies.

Additional details of the lift station cost estimate are presented in **Appendix F**. Costs presented in Appendix F correspond to the January 2021 ENRCCI of 11,628. Costs presented herein have been scaled to reflect the July 2022 ENRCCI of 13,168.

9.2 SEWER PLANNING CRITERIA

As part of this sewer system master planning effort, the City is interested in developing a prioritized capital improvement plan (CIP) that takes into account the asset’s age, condition, as well as the consequence of its eventual failure. The consequence of failure, or “impact”, recognizes that more critical assets should have a higher priority in the CIP. This criticality can be defined by environmental criteria (for instance, the proximity of the sewer to a creek), financial criteria (for instance, if a specific pipe is more expensive to repair), and social criteria (for instance, if failure results in impacts to local businesses). The Asset’s Risk Index (ARI) is used to connect the impact of failure, termed the “Asset Impact Index” or AII, and probability of failure based on the assets condition, termed “Asset Condition Index” or (ACI). As part of this analysis, every asset was assigned a unique impact score and probability score.



9.2.1 Asset Impact Index

The Asset Impact Index (AII) describes the impact or consequence of asset failure. The AII ranges from 1 (low consequence) to 10 (extremely high consequence). Consequences include fines, property damage, traffic delays, public reputation, health safety, etc. **Table 9-2** displays a detailed description of each value on the AII range.

Every asset was assigned an AII based on the detailed definitions in **Table 9-2**.

- Lift Stations: 10
- Sewer Trunk Lines: 10
- Residential Sewer Lines: 5
- Commercial/Downtown Area Sewer Lines: 7
- Interceptor Sewer Lines: 8

Table 9-2 Asset Impact Index Definitions

All	Detailed Definitions
1	No operation and service interruptions. No impact on environment and regulatory compliance. No economic or financial impact.
2	Minimal operation interruptions but no service interruptions to customers. Minimal impact on environment and regulatory compliance. Trivial loss of economic and financial revenue from productions or service interruptions.
3	Minor operation interruptions but no service interruptions to customers. Minor impact on environment and no violation of regulatory compliance. Minor loss of economic and financial revenue from productions or service interruptions.
4	Limited operation interruptions with possible service interruptions to customers. Limited impact on environment and possible violation of regulatory compliance. Limited loss of economic and financial revenue from productions or service interruptions.
5	Moderate operation interruptions and minor service interruptions to customers. Moderate impact on environment and minor violation of regulatory compliance. Moderate loss of economic and financial revenue from productions or service interruptions.
6	Notable operation interruptions and service interruptions to limited customers. Notable impact on environment and minor violation of regulatory compliance. Notable loss of economic and financial revenue from productions or service interruptions.
7	Considerable operation interruptions, service interruptions to limited customers and potential public safety. Considerable impact on environment and violation of regulatory compliance with fines. Considerable loss of economic and financial revenue from productions or service interruptions.
8	Major operation interruptions, service interruptions to medium customers and inherent public safety. Major impact on environment and unavoidable violation of regulatory compliance with large fines. Major loss of economic and financial revenue from production or service interruptions.
9	Significant operations interruptions, service interruptions to large number of customers and imminent public safety. Significant environmental impact and imminent violation of regulatory compliance with significant fines. Significant loss of revenue from productions or service interruptions.
10	Extended operation interruptions, service interruptions to large number of customers and service public safety. Catastrophic environmental impact and monumental violation of regulatory compliance with extensive fines. Extreme loss of revenue from productions or service interruptions.

9.2.2 Asset Condition Index (ACI)

The Asset Condition Index (ACI) describes the likelihood of asset failure based on the condition of the asset. The ACI ranges from 1 (not likely) to 10 (extremely likely). The useful life remaining and the CCTV inspection scores were used to calculate most of the ACI values. **Table 9-3** displays an overview definition of each value on the ACI range.

Table 9-3 Asset Impact Index Definitions

Index	Definitions
1	Extremely low probability of failure
2	Very low probability of failure
3	Low probability of failure
4	Low-intermediate probability of failure
5	Intermediate probability of failure
6	Moderate probability of failure
7	Moderate-high probability of failure
8	High probability of failure
9	Very high probability of failure
10	Extremely high probability of failure

9.2.3 Asset Risk Index (ARI)

The Asset Risk Index (ARI) is the risk score describing the danger or loss associated with the failure of each asset. The ARI ranges from 1 (no risk) to 100 (extremely high risk). ARI is the product of the condition (ACI) and the impact of failure (AII). **Table 9-4** displays an overview definition of the ARI values.

Table 9-4 Asset Risk Index Definitions

Risk			Definition		
1	< ARI <	10	Extremely Low Risk	→	No Activity
11	< ARI <	20	Very Low Risk	→	No Activity
21	< ARI <	30	Low Risk	→	Sample Monitoring
31	< ARI <	40	Low Intermediate Risk	→	Routine Monitoring
41	< ARI <	50	Intermediate Risk	→	Routine Monitoring
51	< ARI <	60	Moderate Risk	→	Aggressive Monitoring
61	< ARI <	70	Moderate High Risk	→	Aggressive Monitoring
71	< ARI <	80	High Risk	→	Plan Work
81	< ARI <	90	Very High Risk	→	Intermediate Work
91	< ARI <	100	Extremely High Risk	→	Intermediate Work

9.3 CCTV & CONDITION INSPECTIONS

Closed circuit television (CCTV) technology is used for internal inspection of sewer lines to identify defects in the sewer line. A structural and service condition report is created as the live footage is being viewed, and a score is given for the overall pipe condition using the PACP (Pipeline Assessment and Certification Program) standards for coding defects. The PACP scoring gives each pipe a value from 0 (great condition) to 5 (major defects).

9.3.1 CCTV Inspection Data

Approximately four years of the City's historical CCTV inspection data (Jan 2017-Aug 2020) was reviewed for assessment of the overall sewer condition. **Table 9-5** displays a summary of the CCTV data.

Table 9-5 Summary of CCTV Data

CCTV Data	Total Sewer Line Assets	Assets with Non-Zero PACP Scores (1)	Assets with Critical PACP Scores (2)
Number of Assets	1,464	102	39
Percent of System	100%	9.5%	2.7%

1) If an asset's CCTV Inspection showed any non-zero number (1-5)
2) Critical score was defined as a PACP score of 4 or greater.

Assets with PACP scores below 4 are considered non-critical and should continue to be inspected over the upcoming years to monitor if additional defects arise. The assets with critical PACP scores were further analyzed to determine the types of defects, their severity, and whether the defects could be remedied with increased maintenance or structural repairs or replacement.

The 39 assets (pipes) with critical PACP scores were classified into two groups: structural defects or O&M defects. If an asset had both O&M and structural defects, it was put in the structural defect category. Fourteen of the 39 assets had O&M defects, the remaining 25 critical assets were structural defects. **Table 9-6** below includes the full list of assets with critical PACP scores, the PACP score, and the field notes that explain the reason for the score.

Structural Defects

Structural defects include cracks, breaks in the pipe or joints, voids visible, soil visible, or any combination of these issues. Examples of some structural defects found in the City's CCTV Data are shown in **Figure 9-1**.

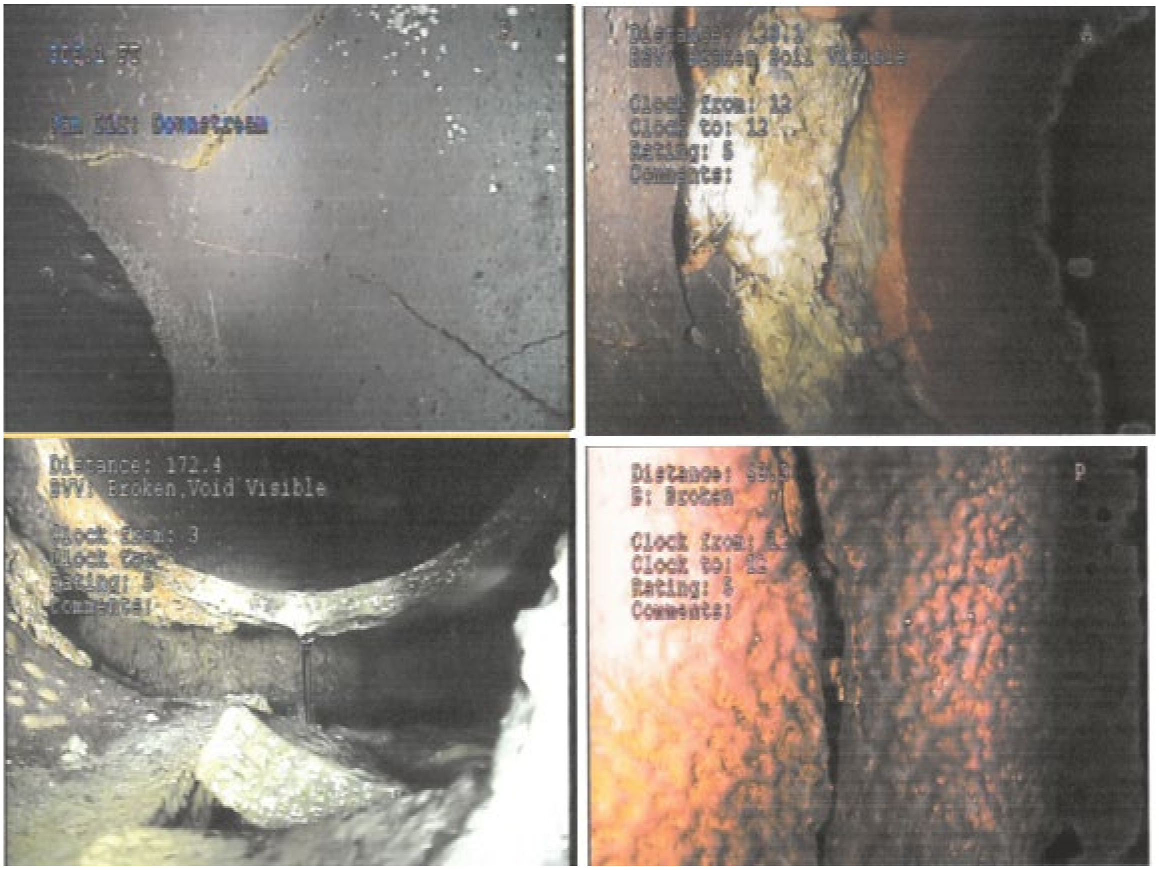


Figure 9-1 Typical Structural Defects from City's CCTV Inspections

The severity of these structural defects varies depending on the size of the defect and how frequent defects occur on the same asset. For example, a pipe with minor cracks in one location would not be as severe and may just need monitoring for increased severity, whereas a pipe with breaks and voids/soils visible all throughout the pipe would require full pipe replacement.

O&M Defects

O&M defects include roots in the pipe or joints, grease buildup, debris buildup, or any combination of these issues. Examples of some O&M defects found in the City's CCTV Data are shown in **Figure 9-2**.

Figure 9-2 Typical O&M Defects from City's Inspections

The majority of the City's O&M defects are caused by roots and/or grease. The City has already developed a list of assets with root issues and hotspot grease areas where they perform more frequent maintenance to reduce risk of asset failure. For example, the grease hotspot areas are jetted every 6 weeks to remove grease buildup.

Based on the list of 14 assets with critical O&M defects, it is recommended that the City continue to perform their current maintenance for the assets on the hotspot grease area list. It is also recommended that the City expand their list of assets that receive more frequent root control to include the assets highlighted in orange in **Table 9-6**.

Table 9-6 Assets with Critical PACP Scores

Asset ID	PACP Score	Type of Defect	Defect Comments	Recommendation
SL-0584-0583	4	O&M	Roots in joints throughout pipe	Add to root issue list
SL-0541-0538	5	Structural	Broken pipe with voids visible	Replace pipe
SL-0500-0499	4	O&M	Roots	Already on City's root issue list
SL-0501-0500	4	O&M	Roots	Already on City's root issue list
SL-0611-0610	4	O&M	Roots	Already on City's root issue list
SL-0610-0609	4	O&M	Roots	Already on City's root issue list
SL-0608-0607	5	Structural	Soil visible	Replace pipe
SL-0504-0500	4	O&M	Roots in joint	Add to root issue list
SL-0488-0487	4	O&M	Roots in joint	Add to root issue list
SL-0515-0514	5	Structural	Cracks and broken pipe; soil visible	Replace pipe
SL-0461-0460	4	O&M	Roots in joint	Add to root issue list
SL-0267-0265	4.5	Structural	Break voids visible	Replace pipe within 3-5 years
SL-0958-0956	4	O&M	Grease buildup	Already on City's grease hotspot
SL-0960-0956	4	O&M	Grease buildup	Already on City's grease hotspot
SL-1007-1006	5	Structural	Broken pipe; soil and voids visible	Replace pipe
SL-1004-1002	5	Structural	Voids visible	Replace pipe
SL-1003-1002	4.5	Structural	Broken pipe, multiple cracks	Replace pipe within 3-5 years
SL-0694-0693	4	O&M	Multiple joints with roots	Add to root issue list
SL-1059-1058	5	Structural	Broken Pipe	Replace pipe
SL-1072-1070	5	Structural	Soil visible, broken pipe	Replace pipe
SL-1034-1028	4.5	Structural	Multiple longitudinal cracks	Replace pipe within 3-5 years
SL-1035-1034	5	Structural	Broken voids visible	Replace pipe
SL-1038-1037	5	Structural	Soils visible	Replace pipe
SL-1033-1031	5	Structural	Voids and soil visible, broken pipe, multiple cracks	Replace pipe
SL-1032-1031	5	Structural	Broken pipe, soils and voids visible	Replace pipe
SL-1031-1028	5	Structural	Broken pipe	Replace pipe
SL-1001-1000	5	Structural	Multiple cracks, broken pipe	Replace pipe
SL-0926-0919A	4.5	Structural	Circumferential cracks	Replace pipe within 3-5 years
SL-0961-0960	5	Structural	Broken pipe	Replace pipe
SL-1023-1201	4.428	Structural	Breaks in pipe, grease build up	Replace pipe within 3-5 years
SL-1022-1021	4	Structural	Breaks in pipe, grease build up	Replace pipe within 3-5 years
SL-1021-1020	4	O&M	Grease buildup	Already on City's grease hotspot
SL-0947-0943	5	Structural	Multiple breaks caused by roots	Replace pipe within 3-5 years
SL-1006-1004	5	Structural	Breaks in pipe	Replace pipe
SL-1035-1035B	4	O&M	Factory tap intrusion, roots	Accelerated monitoring for future structural defects
SL-0716-0715	4.5	O&M	Roots breaking through pipe, multiple cracks	Replace pipe within 3-5 years
SL-1006-1001	4	Structural	Cracks and a broken spot of pipe	Replace pipe within 3-5 years
SL-1010-1006	5	Structural	Roots at joints, broken pipe, multiple circumferential cracks	Replace pipe
SL-1015-1010	4	Structural	Multiple cracks	Replace pipe within 3-5 years

9.3.2 Sewer Condition Scores

Asset condition scores were assigned to each sewer based on the CCTV PACP scores and the useful life remaining for each asset. The Asset Condition Index (ACI) ranges from 1 (as good as brand new) to 10 (needs to be replaced immediately). For the 39 critical assets described in **Table 9-6**, the PACP scores were multiplied by 2 to obtain an equivalent ACI.

The condition score of the City's sewers primarily consists of remaining useful life and CCTV inspection data. For VCP, published useful life ranges from 60-120 years (90 is used as a typical value in this study); due to this wide range, the CCTV assessment data was used to determine a more accurate condition score. Based on the CCTV data, the majority of the VCP sewer pipelines (97%) have an acceptable condition score below a 6 which indicates a low to intermediate risk of failure.

The 27 inch and 42-inch trunk sewers are both constructed of VCP. The standard useful life of VCP is 60 to 120 years; after which VCP tends to crack and leak. The 42-inch VCP trunk line was built in the 1990s and still has over half its useful life remaining and appears to be operating under good condition. As a result, it has been assigned an ACI of 5. The 27-inch VCP trunk was constructed in the 1950s and is approaching the end of its useful life. The CCTV inspections of this trunk were conducted by the City over the past few years. While not reviewed as part of this analysis, City staff have reported that the pipe showed signed of widespread deterioration. As a result, the 27-inch trunk was assigned an ACI of 10. It is important to note that for these larger VCP trunks, the higher ACI score would not necessarily trigger the pipeline replacement. Lining of sewer trunks with plastic liner system is a common method to extend the life of the pipe and is less expensive and impactful than replacement.

9.3.3 Lincoln Street Lift Station Condition Score

The Lincoln Street Lift Station is a duplex station with separate wet-well and dry-pit enclosed in a steel can. The lift station was originally manufactured as a package system by the Smith and Loveless Company (S&L). The station was originally constructed in 1985. Figure 4 in **Appendix F** provides recent images of the dry well. As shown, the can is showing signs of deterioration due to corrosive conditions. The integrity of the steel can cannot easily be ascertained but these stations were common 30- 40 years ago and are generally considered to be at the end of their useful life. For instance, this year the City of West Sacramento replaced two of their S&L stations due to observed corrosion, excessive maintenance, and concerns over station reliability.

For these reasons, the Lincoln Street Lift Station has been assigned an ACI of 10.

A heat map of the assets with ACI values between 7-10 are shown in **Figure 9-3** (green represents assets with 7-8 ACI and red represents assets with 9-10 ACI).

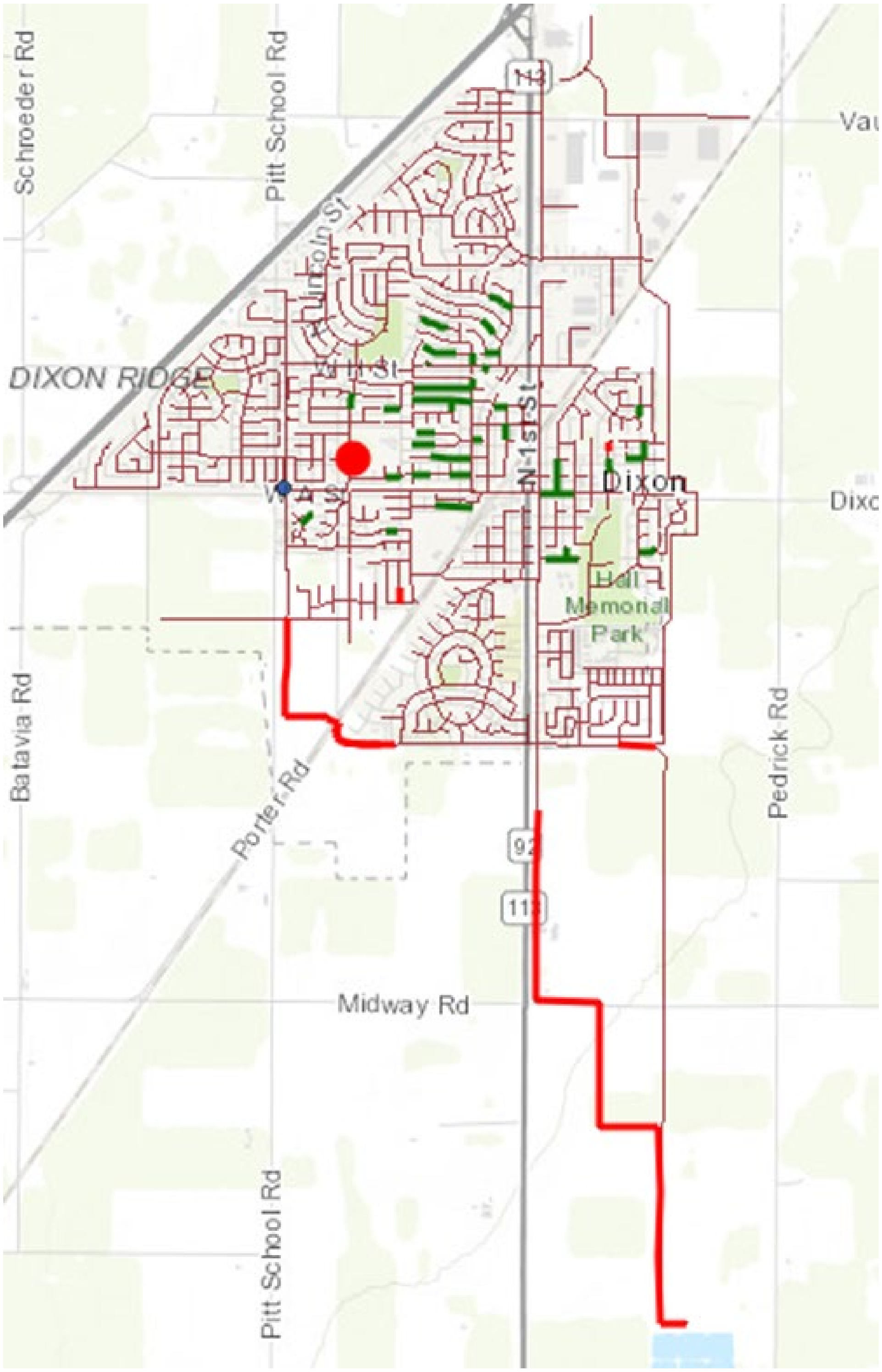


Figure 9-3 Asset Condition Heat Map

9.4 REPAIR & REPLACEMENT PROGRAM IMPROVEMENTS

As discussed in **Chapter 9.0**, all the City's sewer assets are ranked by their specific Asset Risk Index (ARI) score. ARI is calculated by multiplying the probability of failure based on the asset's condition (ACI) by the impact of failure (AII). Both ACI and AII are on a 1-10 scale, so an ARI of 100 would be prioritized for immediate replacement.

9.4.1 5-year Period Repair & Replacement Plan Projects & Costs

The Lincoln Street Sewer Lift Station, sewers with structural defects identified with CCTV inspections, and the 27-inch VCP trunk have been identified to have the highest ARI scores.

It is recommended to phase the above projects over the next five years. A possible means to complete these projects is shown in **Figure 9-4** below. Some projects will require more than one year to design and construct.

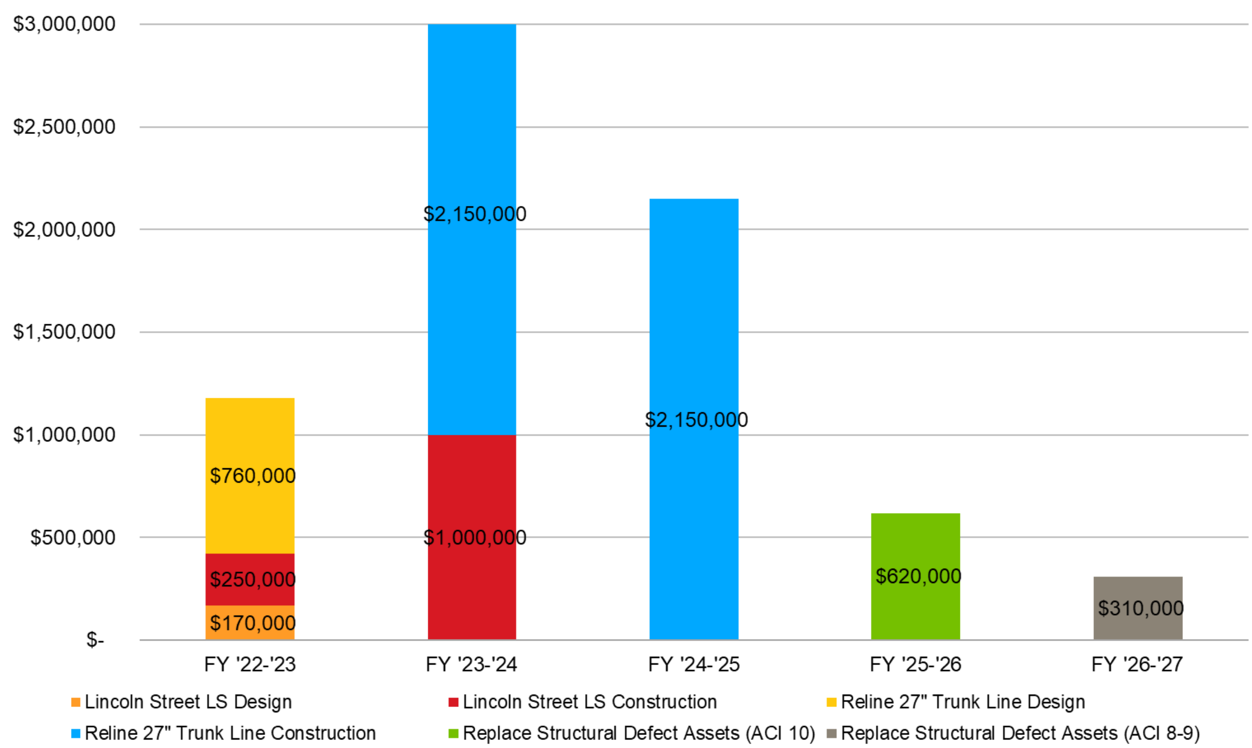


Figure 9-4 5-Year Repair & Replacement Program Cost

Table 9-7 Highest Risk Sewer Assets

Asset No.	Description	All	ACI	ARI	Approx. Depth (ft)	Length (LF)	Cost
Lincoln Street LS	550 gpm Capacity Lift Station	10	10	100	N/A	N/A	$1,420,000
Sewers with Structural Defects							
SL-0541-0538	8-inch VCP Sewer	5	10	50	5	345	$39,000
SL-0608-0607	8-inch VCP Sewer	5	10	50	5	241.	$27,000
SL-0515-0514	6-inch VCP Sewer	5	10	50	4	206.	$19,000
SL-0267-0265	8-inch VCP Sewer	5	9	45	5	220	$25,000
SL-1007-1006	6-inch VCP Sewer	5	10	50	8	623	$59,000
SL-1004-1002	6-inch VCP Sewer	5	10	50	6	358	$34,000
SL-1003-1002	6-inch VCP Sewer	5	9	45	6	363	$37,000
SL-1059-1058	6-inch VCP Sewer	5	10	50	6	175	$16,000
SL-1072-1070	6-inch VCP Sewer	5	10	50	6	266	$25,000
SL-1034-1028	8-inch VCP Sewer	5	9	45	7	359	$30,000
SL-1035-1034	6-inch VCP Sewer	5	10	50	8	375	$35,000
SL-1038-1037	8-inch VCP Sewer	5	10	50	7	360	$41,000
SL-1033-1031	6-inch VCP Sewer	5	10	50	4	179	$17,000
SL-1032-1031	6-inch VCP Sewer	5	10	50	4	200	$19,000
SL-1031-1028	6-inch VCP Sewer	5	10	50	5	358	$34,000
SL-1001-1000	12-inch VCP Sewer	7	10	70	13	364	$63,000
SL-0926-0919A	10-inch VCP Sewer	7	9	63	10	198	$31,000
SL-0961-0960	8-inch VCP Sewer	5	10	50	6	360	$27,000
SL-1023-1201	6-inch VCP Sewer	5	9	45	4	151	$14,000
SL-1022-1021	6-inch VCP Sewer	5	8	40	4	186	$18,000
SL-0947-0943	10-inch VCP Sewer	7	10	70	10	317	$49,000
SL-1006-1004	6-inch VCP Sewer	5	10	50	6	235	$22,000
SL-1035-1035B	6-inch VCP Sewer	5	8	40	6	187	$18,000
SL-0716-0715	8-inch VCP Sewer	5	9	45	6	322	$37,000
SL-1006-1001	12-inch VCP Sewer	7	8	56	10	359	$64,000
SL-1010-1006	12-inch VCP Sewer	7	10	70	9	359	$66,000
SL-1015-1010	12-inch VCP Sewer	7	8	56	7	358	$59,000
27 Inch Trunk Sewer	Line 27-inch VCP	10	10	100	18	22,350	$5,060,000
						Total	**$7,405,000**

9.4.2 25-year Period Repair & Replacement Funding Needs

As mentioned above, a majority of the City's 1,400 VCP sewers are in good condition and do not need to be immediately included in the City's CIP. VCP is a strong and corrosion resistant material and has a long-expected life. However, as the sewers age, the City should expect to see additional replacement projects and the pipes with operational defects (currently 14 pipes) can become structural defects in the future. In addition, the City's 42-inch VCP trunk is about halfway through its expected life before a significant rehabilitation would be needed. These pipes represent a majority of the value of City's sewer assets. As a result, it is important to continue and formalize the City's cleaning and inspection practices.

The assumed useful life of VCP (90 years) and RCP (60 years) was used to calculate a general cost for replacement of the sewers not included in the five-year plan. A 25-year horizon was selected to capture a reasonable grouping of some of the older VCP sewers and includes rehabilitation of the 42-inch trunk. Total costs over the 25-year period are projected at approximately $23 million (expressed in current dollars) and are shown over time in **Figure 9-5**. The $23 million is comprised of the five-year plan depicted in **Figure 9-4**, replacement the older VCP sewers that are beyond the 90-year life and assumed lining of the 42-inch trunk prior to 2047. Additional CCTV inspections will be needed in subsequent years to validate these projections.

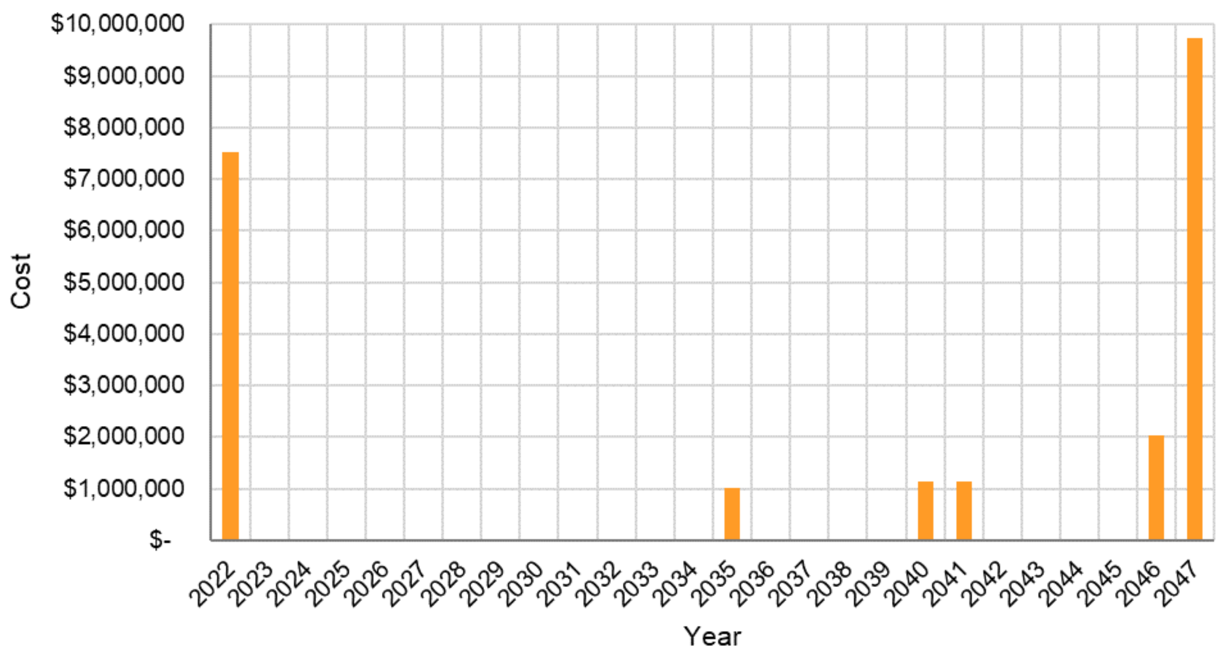


Figure 9-5 25-year Plan Repair & Replacement Program Cost

10.0 CAPITAL IMPROVEMENT PROGRAM

The purpose of this chapter is to present the recommended CIP for the City's existing and future sewer collection system. Recommendations for improvements to the existing and future sewer system were described previously in **Chapter 7.0** through **Chapter 9.0**. This chapter provides a summary of the recommended improvement projects, along with the estimates of probable construction costs. It should be noted that the recommended CIP only identifies improvements at a master plan level and does not constitute a design of such improvements. Subsequent detailed design is required to further determine the sizes, alignment, and scope of these proposed improvements.

This chapter is divided into the following sections:

- Recommended Existing Sewer System Improvements
- Recommended Future Sewer System Improvements
- Repair and Replacement Programs Improvements
- Capital Improvement Program Implementation

Costs are presented in July 2022 dollars based on an Engineering News Record Construction Cost Index (ENR CCI) of 13,168 (20-Cities). Total CIP costs include the following design and construction contingency and propose cost allowances:

- 20% of total estimated capital cost for Design and Construction Contingency
- 15% of total estimated capital costs for Engineering, Administrative, and Legal

A description of the assumptions used in developing the estimates of probable construction cost is provided in **Chapter 9.0** and **Appendix F**.

10.1 RECOMMENDED EXISTING SEWER SYSTEM IMPROVEMENTS

Chapter 7.0 provided a summary of the evaluation of the City's existing sewer collection system and its ability to meet the recommended level of service criteria described in **Chapter 5.0** and **Chapter 6.0**. Based on these results, the existing sewer collection system in sewer-shed 1, specifically the Industrial Way Trunk, is deficient in sewer capacity. Therefore, it is recommended that the City begin preliminary design of improvements to upsize this trunk sewer from 10-inches in diameter to 15-inches in diameter. To meet existing PWWF conditions, it would require a 12-inch sewer, but under future development conditions a 15-inch sewer needed. Therefore, it is recommended that 15-inch sewer be installed to avoid additional improvements in the near-term future.

10.1.1 Recommended Existing Sewer System CIP Costs

The recommended existing system projects are presented in **Table 10-1**, along with their probable construction costs. As shown, the existing system CIP cost is estimated to be approximately $617,000.

Table 10-1 Summary of Probable Construction Costs: Existing Sewer System

CIP ID	Reason for Improvement	Improvement Description	Capital Costs (1) (2)
CIP-E1	PWWF Capacity Deficiency	Upsize Industrial Way Trunk, *from 10 to 15-inch*	$617,000
		Subtotal	$617,000
		Total Existing System CIP	**$617,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.

The recommended CIP projects for the existing sewer collection system include:

Existing System Improvement

- Industrial Way Trunk (CIP – E1)
  Upsize the 10-inch trunk sewer in Industrial Way to 15-inches to provide sufficient capacity for existing and future PWWF conditions. Review of actual as-built conditions is recommended to confirm the available slope and existing pipe size.
  15-inch sewer, 2,100 LF

The location of the recommended existing sewer system improvements is shown on **Figure 10-1**.

10.2 RECOMMENDED FUTURE SEWER SYSTEM IMPROVEMENTS

Chapter 8.0 provided a summary of the evaluation of the City's future sewer collection system for near-term, long-term, and build-out levels of development. It also presented an evaluation of the collection systems ability to meet the recommended level of service criteria presented described in **Chapter 5.0** and **Chapter 6.0**.

The proposed E-W Sewer Extension extends the western branch (E-W Branch 1) of the existing East-West Trunk Sewer Connector to provide service to the western portion of the Homestead Development area. To expand the existing sewer trunk network to serve the near-term service area the following CIP is recommended:

New Trunk Sewer: Homestead Development Area

- E-W Sewer Extension (CIP – N1)
  10-inch sewer, 1,190 LF
  12-inch sewer, 2,250 LF
  15-inch sewer, 1,150 LF

The results of the long-term development scenario identified a capacity limitation in the 21-inch portion of the Fitzgerald Dr. trunk sewer. An improvement project is recommended to upsize the 21-inch portion of the sewer Fitzgerald Dr to provide sufficient capacity to convey PWWF under long-term development conditions. In addition to this existing system improvement, several new trunk sewers and a new lift station are proposed to serve the long-term service area, including the Northeast Quad.

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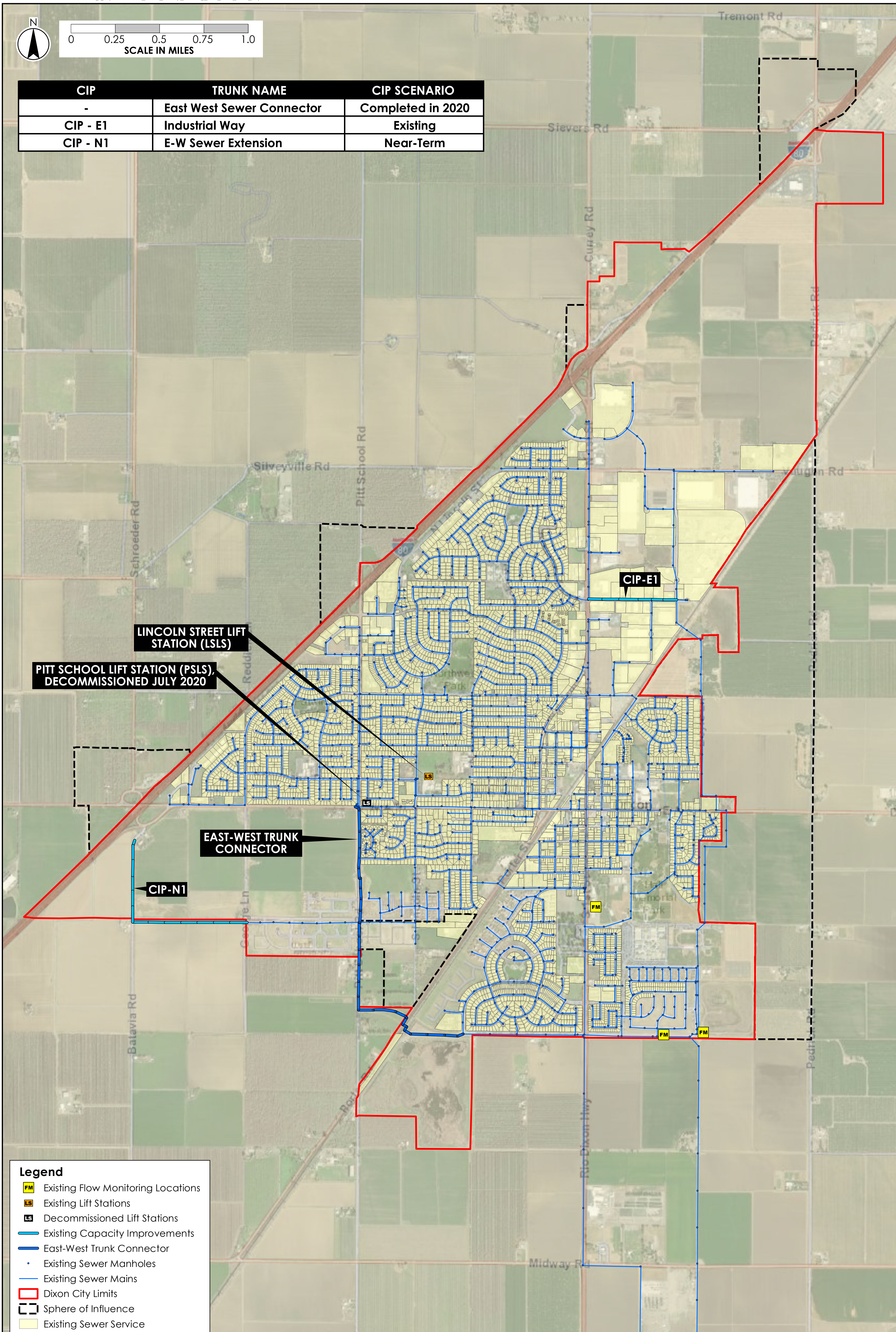

CIP	TRUNK NAME	CIP SCENARIO
-	East West Sewer Connector	Completed in 2020
CIP - E1	Industrial Way	Existing
CIP - N1	E-W Sewer Extension	Near-Term

Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 10-1
Existing & Near-term Sewer Collection System Improvements

The recommended long-term sewer collection system improvements have been grouped into several recommended CIP projects including the following:

Existing System Improvement

- Fitzgerald Drive Trunk (CIP – E2)
  Upsize the 21-inch trunk sewer in Fitzgerald Dr. to 27-inches to provide sufficient capacity for long-term PWWF conditions. Review of actual as-built conditions is recommended to confirm the available slope and existing pipe size.
  27-inch sewer, 2,550 LF

New Trunk Sewer: North of I-80

- Milk Farm Road, I-80 Crossing (CIP – N2)
  8-inch sewer, 2,350 LF
  10-inch sewer, 660 LF

New Trunk Sewers: Northeast Quad (Gravity Service Area)

- Main NE Quad Trunk (CIP – N3)
  12-inch sewer, 1,120 LF
  18-inch sewer, 2,250 LF
  21-inch sewer, 1,850 LF

- Main NE Quad Trunk – Branch 1 (CIP – N4)
  8-inch sewer, 1,040 LF
  10-inch sewer, 1,330 LF
  12-inch sewer, 1,360 LF

- Main NE Quad Trunk – Branch 2 (CIP – N5)
  8-inch sewer, 525 LF
  10-inch sewer, 525 LF
  12-inch sewer, 1,080 LF

New Trunk Sewers: Northeast Quad (LS Service Area)

- NE Quad LS - Sewer-shed (CIP – N6a)
  8-inch sewer, 1,150 LF
  10-inch sewer, 700 LF

New Lift Station: Northeast Quad (LS)

- NE Quad LS – Lift Station (CIP – N6b)
  Dual 4-inch force-main, 3,140 LF
  0.45 MGD needed under long-term conditions and 0.65 MGD at build-out.
  Build-out reliable capacity of 450 gpm.

The locations of the recommended long-term sewer system improvements are shown on **Figure 10-2**.

Build-Out Development

The results of the build-out development scenario identified a capacity limitation in the 30-inch sewers making a 90-degree bend in the North Dixon Trunk. An improvement project is recommended to upsize the 30-inch portion of the sewer to 36-inches to provide sufficient capacity to convey PWWF under build-out development conditions. In addition to this existing system improvement, several new trunk sewers are proposed to serve the build-out service area.

The build-out system improvements have been grouped into several recommended CIP projects, and including the following:

Existing System Improvement:

- Downstream end of the North Dixon Trunk (CIP – E3)
  Upsize three sewer segments immediately upstream of the confluence with the Parkway Blvd and 42-inch Main Trunks from 30-inches to 36-inches to provide sufficient capacity for build-out PWWF conditions.
  36-inch sewer, 300 LF

New Trunk Sewers: North of I-80

- E-W Trunk, I-80 Crossing (CIP – N7)
  8-inch sewer, 1,980 LF

- N. Lincoln St., I-80 Crossing (CIP – N8)
  8-inch sewer, 2,100 LF

- Sparling Ln., I-80 Crossing (CIP – N9)
  8-inch sewer, 1,610 LF
  10-inch sewer, 800 LF

New Trunk Sewer: East Development Area

- East Area Main Trunk (CIP – N10)
  8-inch sewer, 1,440 LF
  10-inch sewer, 2,620 LF
  12-inch sewer, 3,980 LF
  18-inch sewer, 3,970 LF
  21-inch sewer, 2,710 LF

The locations of the recommended build-out sewer system improvements are shown on **Figure 10-2**.

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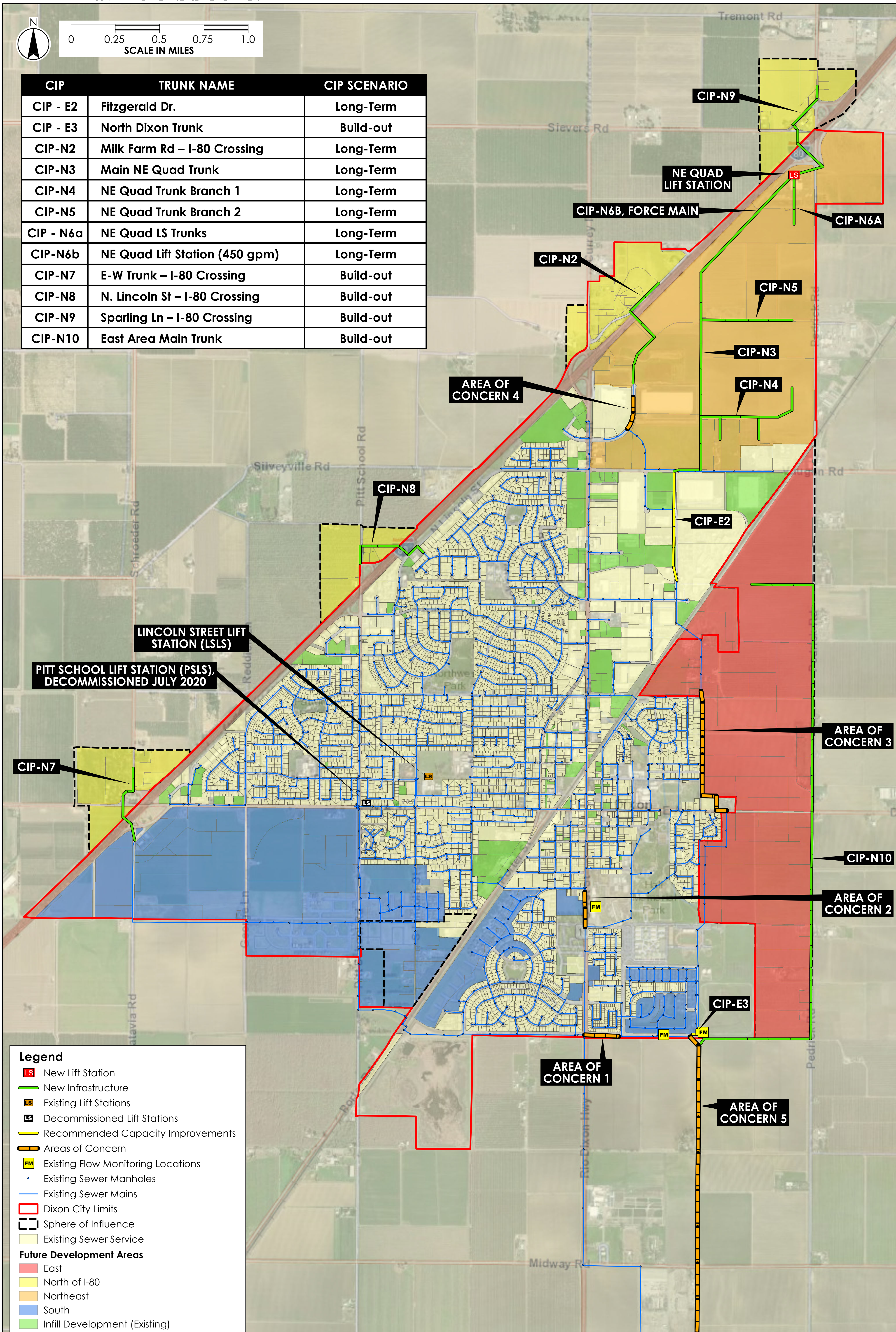

CIP	TRUNK NAME	CIP SCENARIO
CIP - E2	Fitzgerald Dr.	Long-Term
CIP - E3	North Dixon Trunk	Build-out
CIP-N2	Milk Farm Rd – I-80 Crossing	Long-Term
CIP-N3	Main NE Quad Trunk	Long-Term
CIP-N4	NE Quad Trunk Branch 1	Long-Term
CIP-N5	NE Quad Trunk Branch 2	Long-Term
CIP - N6a	NE Quad LS Trunks	Long-Term
CIP-N6b	NE Quad Lift Station (450 gpm)	Long-Term
CIP-N7	E-W Trunk – I-80 Crossing	Build-out
CIP-N8	N. Lincoln St – I-80 Crossing	Build-out
CIP-N9	Sparling Ln – I-80 Crossing	Build-out
CIP-N10	East Area Main Trunk	Build-out

Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure 10-2
Long-Term & Build-Out Sewer Collection System Improvements

10.2.1 Recommended Future Sewer System CIP Costs

The recommended near-term, long-term, and build-out sewer system projects are presented in **Table 10-2**, **Table 10-3**, and **Table 10-4** along with their probable construction costs. As shown, the CIP costs are estimated to be approximately $1.2 million for the near-term sewer system improvements, $6.8 million for the long-term system improvements, and $6.3 million for the build-out system improvements.

Recommended Near-Term Sewer System CIP Costs

The recommended near-term system projects are presented in **Table 10-2**, along with their probable construction costs. As shown, the near-term system CIP cost is estimated to be approximately $1,202,000.

Table 10-2 Summary of Probable Construction Costs: Near-Term Sewer System

CIP ID	Reason for Improvement	Improvement Description	Capital Costs (1) (2)
CIP-N1	Expand Service Area	E-W Sewer Extension, *10 to 15-inch trunk*	$1,202,000
		Subtotal	$1,202,000
		Total Near-Term System CIP	**$1,202,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.

Recommended Long-Term Sewer System CIP Costs

The recommended long-term system projects are presented in **Table 10-3**, along with their probable construction costs. As shown, the long-term system CIP cost is estimated to be approximately $6,759,000.

Table 10-3 Summary of Probable Construction Costs: Long-Term Sewer System

CIP ID	Reason for Improvement	Improvement Description	Capital Costs (1) (2)
• Existing System Improvement			
CIP-E2	PWWF Capacity Deficiency	Upsize Fitzgerald Dr., *from 21 to 27-inch*	$1,350,000
		Subtotal	$1,350,000
• New Trunk Sewer: North of I-80 (4)			
CIP-N2	Expand Service Area	Milk Farm Rd., I-80 Crossing, *8 to 10-inch*	$745,000
		Subtotal	$745,000
• New Trunk Sewers: Northeast Quad (Gravity Service Area)			
CIP-N3	Expand Service Area	Main NE Quad Trunk, *12 to 21-inch*	$1,819,000
CIP-N4	Expand Service Area	Main NE Quad Trunk - Branch 1, *8 to 12-inch*	$647,000
CIP-N5	Expand Service Area	Main NE Quad Trunk - Branch 2, *8 to 12-inch*	$391,000
		Subtotal	$2,857,000
• New Trunk Sewers & Lift Station: Northeast Quad (LS Service Area)			
CIP-N6a	Expand Service Area	NE Quad LS Sewer-shed, *8 to 10-inch*	$517,000
CIP-N6b	Expand Service Area	NE Quad LS & Force-main (0.65 MGD) (3)	$1,290,000
		Subtotal	$1,807,000
		Total Long-Term System CIP	**$6,759,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.
3) Includes total build-out lift station capacity and dual 4-inch force-main, force-main cost equates to approximately $170,000.
4) A unit rate of $1,000/LF was assumed for pipe segments crossing I-80 to provide an allowance for working in a CalTrans ROW.

Recommended Build-Out Sewer System CIP Costs

The recommended build-out system projects are presented in **Table 10-4**, along with their probable construction costs. As shown, the build-out system CIP cost is estimated to be approximately $6,289,000.

Table 10-4 Summary of Probable Construction Costs: Build-Out Sewer System

CIP ID	Reason for Improvement	Improvement Description	Capital Costs (1) (2)
• Existing System Improvement			
CIP-E3	PWWF Capacity Deficiency	Upsize North Dixon Trunk, *from 30 to 36-inch*	$227,000
		Subtotal	$227,000
• New Trunk Sewers: North of I-80 (3)			
CIP-N7	Expand Service Area	E-W Trunk, I-80 Crossing, *8-inch*	$478,000
CIP-N8	Expand Service Area	N Lincoln St., I-80 Crossing, *8-inch*	$462,000
CIP-N9	Expand Service Area	Sparling Ln., I-80 Crossing, *8 to 10-inch*	$1,020,000
		Subtotal	$1,960,000
• New Trunk Sewer: East Development Area			
CIP-N10	Expand Service Area	NE Quad LS Sewer-shed, *8 to 21-inch*	$4,102,000
		Subtotal	$4,102,000
		Total Build-Out System CIP	**$6,289,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.
3) A unit rate of $1,000/LF was assumed for pipe segments crossing I-80 to provide an allowance for working in a CalTrans ROW.

10.3 REPAIR AND REPLACEMENT PROGRAM IMPROVEMENTS

The condition assessment and repair and replacement program presented in **Chapter 9.0**, reviewed the condition of existing sewer system facilities and recommended a repair and replacement program based on the existing condition and life expectancy of the City's sewer collection system facilities. Costs for the repair and replacement program are presented for a 5-year period and a 25-year period.

The total costs over the 5-year period are projected at approximately $7.4 million and include specific projects to improve assets that have been identified as needing immediate to near-term replacement. These projects have been generally grouped into sewer replacements ($925,000), installation of a lining system in the 27-inch trunk ($5.1 million), and replacement of the Lincoln Street Sewer Lift Station ($1.4 million). It is recommended to complete these projects over the next five years.

Total costs over the 25-year period are projected at approximately $23 million (expressed in current dollars) and is compromised of the five-year plan described above, replacement of the older VCP sewers that are beyond the 90-year service life, and the assumed lining of the 42-inch trunk prior to 2047.

The projects included in the 5-year period should be included in the City's existing or near-term CIP in addition to those presented in this chapter. The costs and details regarding the 5-year period CIP can be found in **Section 9.4** and **Appendix F** of this SCSMP.

10.4 CAPITAL IMPROVEMENT PROGRAM IMPLEMENTATION

As shown in **Table 10-5**, several improvement projects have been recommended for the existing and future sewer collection system. The construction of the improvements for the future sewer collection system should be coordinated with the proposed schedules of future development to ensure that the required infrastructure will be in place to serve future users.

Table 10-5 Summary of Recommended Sewer Collection System CIP Cost (1) (2)

Improvement Cost Estimate	Existing System	Near-Term System	Long-Term System	Build-out System	Total Capital Costs
Repair & Replacement Program (3)	$5,750,000	$5,750,000	$5,750,000	$5,750,000	$23,000,000
Existing System Improvement (4)	$617,000	$0	$1,350,000	$227,000	$2,194,000
New Infrastructure	$0	$1,202,000	$5,409,000	$6,062,000	$12,673,000
Total:	**$6,367,000**	**$6,952,000**	**$12,509,000**	**$12,039,000**	**$37,867,000**

1) Costs shown are based on the July 2022, 20-Cities ENR CCI of 13,168.
2) Costs are rounded to the nearest $1,000. Costs include based construction costs plus 20 percent construction contingency, and 15 percent for administration and design costs.
3) Repair and replacement program projects are identified based on the existing assets physical condition. Repair and replacement plans are developed for a 5-year and 25-year period. The 5-year plan has a total cost of approximately $7.4 million and prioritizes improvements needed within the next 5-years. The 25-year plan has a total cost of $23 million and identifies projects needed to replace critical assets, including those in the 5-year plan, and those that meet the end of their useful life between now and 2047.
4) Existing system improvements are needed to address capacity deficiencies in the existing sewer system that occur under PWWF conditions at the specified level of development. The repair and replacement program costs are also ultimately for existing system improvements. As opposed to new infrastructure, which serves the expanded service area.
5) A unit rate of $1,000/LF was assumed for pipe segments crossing I-80 to provide an allowance for working in a CalTrans ROW.

APPENDICES

Appendix A   V&A FLOW MONITORING REPORT

City of Dixon

Sewer Flow Monitoring and Inflow/Infiltration Study

Prepared for: Stantec
3875 Atherton Road
Rocklin, CA 95765

Report Date: Draft: May 2020
Final: November 2020

Prepared by: V&A consulting engineers

V&A Project No. 19-0158

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Tables

Figures

Photo Log

Abbreviations and Acronyms

Abbreviations/Acronyms	Definition
ADWF	Average Dry Weather Flow
AVG.	Average
CCTV	Closed-Circuit Television
CDEC	California Data Exchange Center
CIP	Capital Improvement Plan
CO	Carbon Monoxide
CWOP	Citizen Weather Observing Program
DIA.	Diameter
d/D	Depth/Diameter Ratio
FT.	Feet
FM	Flow Monitor
GPD	Gallons per Day
GPM	Gallons per Minute
GWI	Groundwater Infiltration
H2S	Hydrogen Sulfide
IN.	Inch
I/I	Inflow and Infiltration
IDM	Inch-Diameter Mile
IDW	Inverse Distance Weighting
LEL	Lower Explosive Limit
MAX.	Maximum
MGD	Million Gallons per Day
MIN.	Minimum
NOAA	National Oceanic and Atmospheric Administration
N/A	Not applicable
PF	Peaking Factor
PWS	Private Weather Station
Q	Flow Rate
RDI	Rainfall-Dependent Infiltration
RG	Rain Gauge
V&A	V&A Consulting Engineers, Inc.
WEF	Water Environment Federation
WRCC	Western Regional Climate Center

Terms and Definitions

Term	Definition
Average dry weather flow (ADWF)	The average flow rate or pattern from days without noticeable inflow or infiltration response. ADWF usage patterns for weekdays and weekends differ and must be computed separately. ADWF is expressed as a numeric average and may include the influence of normal groundwater infiltration (not related to a rain event).
Basin	Sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. Also refers to the ground surface area near and enclosed by pipelines. A basin may refer to the entire collection system upstream from a flow meter or exclude separately monitored basins upstream.
Depth/diameter (*d*/*D*) ratio	Depth of water in a pipe as a fraction of the pipe's diameter. A measure of the fullness of the pipe used in the capacity analysis.
Infiltration and inflow	**Infiltration and inflow (I/I)** rates are calculated by subtracting the ADWF flow curve from the instantaneous flow measurements taken during and after a storm event. Flow in excess of the baseline consists of inflow, rainfall-responsive infiltration, and rainfall-dependent infiltration. **Total I/I** is the total sum in gallons of additional flow attributable to a storm event.
Infiltration, groundwater	**Groundwater infiltration (GWI)** is groundwater that enters the collection system through pipe defects. GWI depends on the depth of the groundwater table above the pipelines as well as the percentage of the system that is submerged. The variation of groundwater levels and subsequent groundwater infiltration rates are seasonal by nature. On a day-to-day basis, groundwater infiltration rates are relatively steady and will not fluctuate greatly.
Infiltration, rainfall-dependent	**Rainfall-dependent infiltration (RDI)** is similar to groundwater infiltration but occurs as a result of storm water. The storm water percolates into the soil, submerges more of the pipe system, and enters through pipe defects. RDI is the slowest component of storm-related infiltration and inflow, beginning gradually and often lasting 24 hours or longer. The response time depends on the soil permeability and saturation levels.
Inflow	Inflow is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard, and area drains, holes in manhole covers, cross-connections from storm drains, or catch basins. Inflow creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Overflows are often attributable to high inflow rates.
Peak Wet Weather Flow	The highest daily flow during and immediately after a significant storm event. Includes sanitary flow, infiltration, and inflow.
Peaking factor (PF)	PF is the ratio of peak measured flow to average dry weather flow. This ratio expresses the degree of fluctuation in flow rate over the monitoring period and is used in the capacity analysis.
Surcharge	When the flow level is higher than the crown of the pipe, then the pipeline is said to be in a **surcharged** condition. The pipeline is surcharged when the *d*/*D* ratio is greater than 1.0.

Executive Summary

Scope and Purpose

V&A Consulting Engineers (V&A) was retained by Stantec to perform sanitary sewer flow monitoring for the City of Dixon (City). Open-channel flow monitoring was performed at three (3) sites for approximately 11 weeks February 7, 2020 to April 22, 2020. There were three general purposes for this study.

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Establish the peak flow condition during the rainfall events and estimate available sewer capacity.
3. Quantify inflow/ infiltration (I/I) at the applicable flow monitoring sites.

Monitoring Sites

The flow monitoring site locations were selected and approved by Stantec and the City and are listed in Table ES-1, and shown in Figure ES-2.

Table ES-1. List of Flow Monitoring Sites

Monitoring Site	Monitored Pipe	Diameter (in)	Location
FM 1	North inlet	30	Yale Dr., 100 feet north of Parkway Blvd.
FM 2	West Inlet	27	Parkway Blvd., 900 feet west of Yale Dr.
FM 3	North Inlet	15	South 1st St., 525 feet south of W. Cherry St.

Rainfall Monitoring

There was approximately 2.79 inches of rainfall observed during the duration of the flow monitoring study, concentrated across two rainfall events in March and April (refer to Figure ES-1). The April 5 event was large enough to elicit a measurable I/I response and was used for I/I analysis.

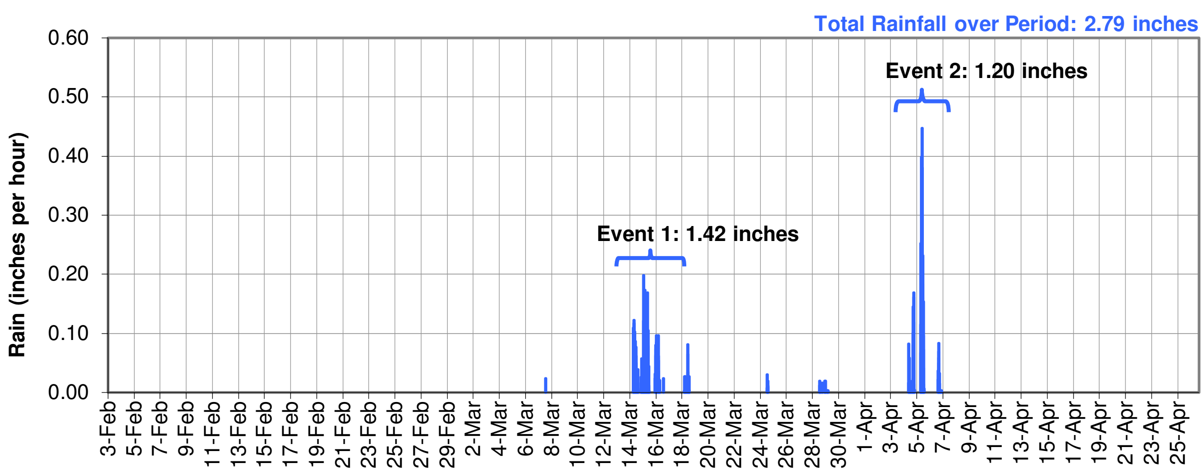

Figure ES-1. Rainfall during Flow Monitoring Period

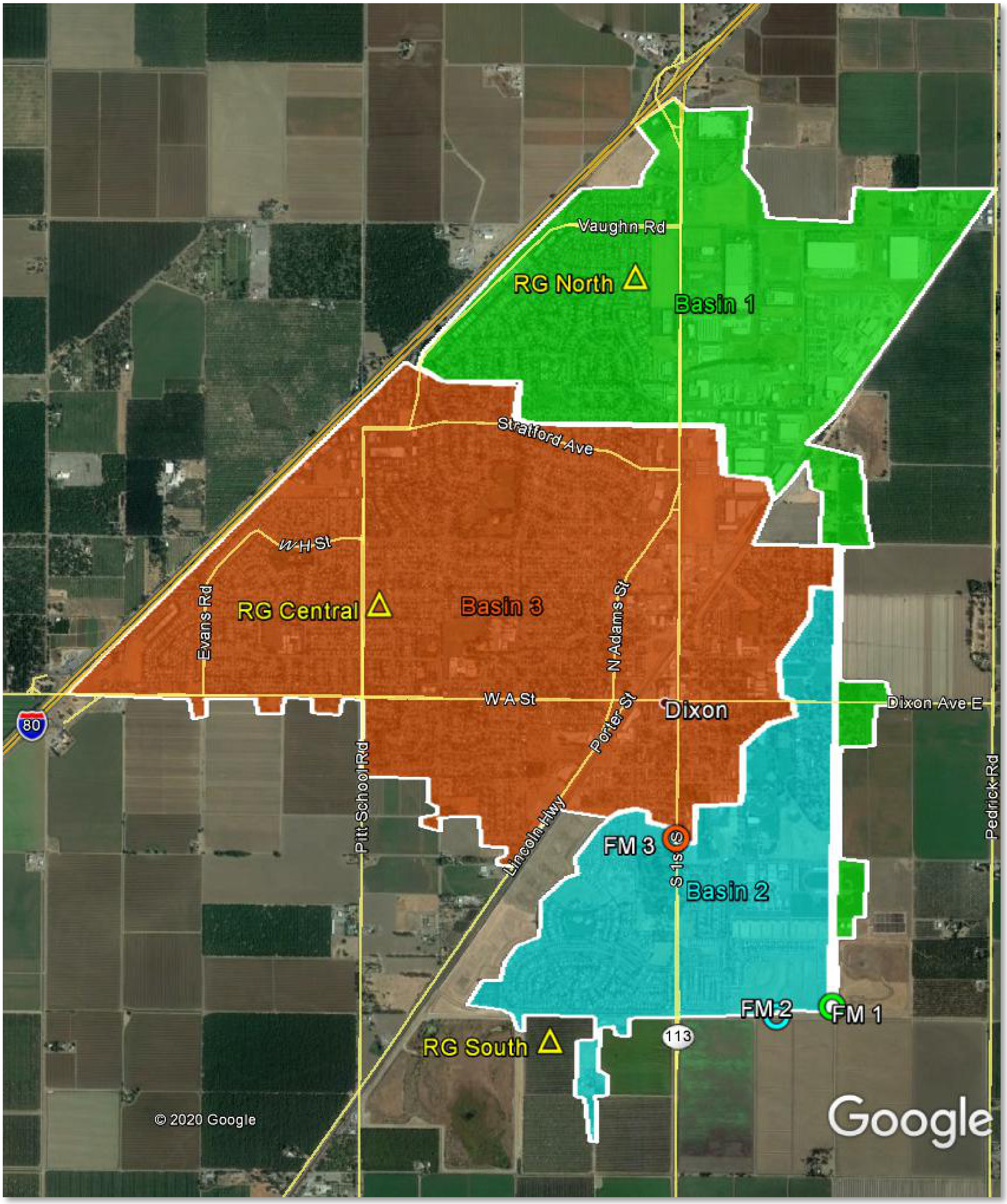

Figure ES-2. Map of Flow Monitoring Sites – Overall

Average Flow Analysis

For this study, two sets of average dry weather flow (ADWF) curves were established due to the advent of "shelter-in-place" (SIP) order for Covid-19. Pre-SIP ADWF can be used for modelling purposes. Post-SIP ADWF was used for I/I isolation and I/I analysis of the main rainfall event of this flow monitoring period. Table ES-2 summarizes the ADWF values per site during the flow monitoring period.

Table ES-2. Dry Weather Flow

Site	Pre-SIP ADWF (mgd)					Post-SIP ADWF (mgd)					SIP Delta
	Mon-Thu	Fri	Sat	Sun	Overall	Mon-Thu	Fri	Sat	Sun	Overall	
FM 1	0.289	0.275	0.250	0.227	0.273	0.282	0.288	0.251	0.242	0.273	0%
FM 2	0.763	0.740	0.795	0.853	0.777	0.838	0.835	0.862	0.833	0.840	+8%
FM 3	0.562	0.557	0.606	0.628	0.577	0.557	0.551	0.589	0.593	0.566	-2%

Peak Measured Flows and Pipeline Capacity Analysis

Peak measured flows and hydraulic grade line data are important to understanding the capacity limitations of a collection system. Relevant capacity analysis terms are defined as follows:

- **Peaking Factor**: Peaking factor is defined as the peak measured flow divided by the average dry weather flow (ADWF). Peaking factors are influenced by many factors including size/topography of the tributary area, flow attenuation, flow restrictions, and characteristics of I/I entering the collection system. Municipal standards for peaking factor vary agency by agency; The City should refer to jurisdictional standards when evaluating peaking factors[1].
- **d/D Ratio**: The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). The d/D ratio for each site was computed based on the maximum depth of flow for the study. Standards for d/D ratio vary from agency to agency, but typically range between d/D ≤ 0.5 and d/D ≤ 0.75. The City should refer to jurisdictional standards when evaluating d/D ratios, to be used at the discretion of City engineers.

Table ES-3 summarizes the peak recorded flows, levels, d/D ratios, and peaking factors per site during the flow monitoring period. Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the site locations; hydraulic conditions in other areas of the collection system will differ.

Table ES-3. Capacity Analysis Summary

Monitored Site	ADWF [A] (MGD)	Peak Flow (MGD)	Peaking Factor	Diameter, *D* (IN)	Max Depth, *d* (IN)	*Max d/D* Ratio
Site 1	0.273	0.57	2.1	30	10.9	0.36
Site 2	0.777	1.56	2.0	27	6.6	0.24
Site 3	0.577	2.07	3.6	15	10.2	0.68

[A] Pre-SIP ADWF was used for this analysis

[1] WEF Manual of Practice FD-6 and ASCE Manual No. 62 suggests typical peaking factor ratios range between 3 and 4, with higher values possibly indicative of pronounced I/I flows.

The following capacity analysis results are noted:

- **Peak flows**: The peak measured flows were taken from the whole monitoring study, including during the rainfall, and pre- and post-SIP.
  - Site 1: The peak flow occurred on March 11, not during a rainfall event.
  - Site 2: The peak flow occurred on April 5, corresponding to the April 4/5 rainfall event.
  - Site 3: The peak flow occurred on March 2, not during a rainfall event. The peak flow appeared to be artificial and the result of a "hold-and-release" event[3] (flows were held back, then released through the pipeline).
    - The peak flow value for Site 3 is greater than Site 2 due to attenuation (refer to
- **Peaking Factors**: All three sites had peaking factors lower than 4. The highest peaking factor was at Site 3 and was PF = 3.6; this peaking factor was primarily caused be the aforementioned hold-and-release event.
- **d/D Ratio**: Site 3 was the only site with a d/D ratio greater than 0.5. None of the sites surcharged during the study.

Figure ES-3 shows a schematic diagram of the peak measured flows with peak flow levels.

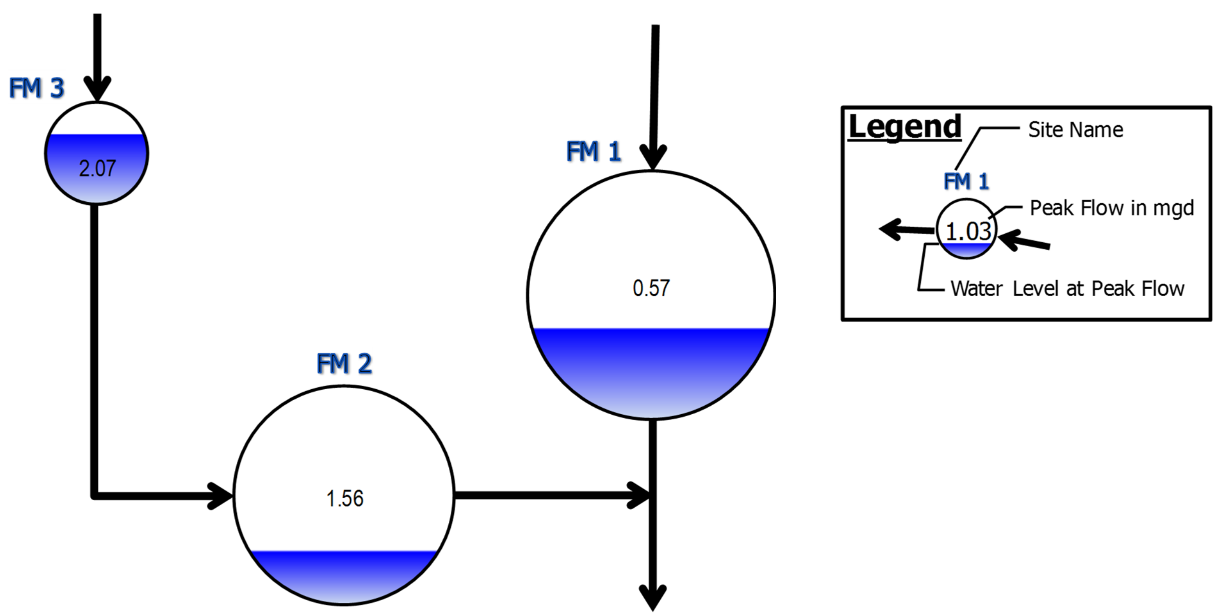

Figure ES-3. Peak Measured Flow (Flow Schematic)

[3] This March 2 "hold-and-release" event was also observed downstream through Site 2 and accounted for the second highest peak for Site 2.

Infiltration and Inflow

Flow monitoring basins are localized areas of a sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. The basin refers to the ground surface area near and enclosed by the pipelines. A basin may refer to the entire collection system upstream from a flow meter or may exclude separately monitored basins upstream. I/I analysis in this report will be conducted on a basin-by-basin basis. It is noted that Basin 3 flows into Basin 2, requiring a subtraction of flows (see Section 2.4 for more information).

I/I results were taken from the April 4/5, 2020 rainfall event. Table ES-4 summarizes the I/I results for this study. In all three I/I metrics, Basin 2 ranked the highest. Please refer to the I/I Methods section for more information on inflow and infiltration analysis methods and ranking methods.

Table ES-4. I/I Analysis Summary

Metering Basin	Basin ADWF [A] (MGD)	Peak Inflow Rate (mgd)	Total I/I (gallons)	Overall Inflow Rank	Total I/I Rank	Evidence of GWI?
Basin 1	0.273	0.174	23,400	2	2	Maybe
Basin 2	0.200	0.197	38,700	1	1	No
Basin 3	0.577	0.152	33,900	3	3	No

[A] Pre-SIP ADWF was used for this analysis

The following items are noted:

- Basin 2 had the highest weighted, normalized inflow and total I/I rates.
- Basin 1 had evidence of slightly elevated levels of groundwater infiltration, though it is noted that this basins may collect sanitary waste from industrial usages which could elevate the low-to-ADWF ratios.

Recommendations

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

1. **Determine I/I Reduction Program**: The City should examine its I/I reduction needs to determine their needs and goals for a future I/I reduction program.
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow occurs in Basin 2.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. The highest combined I/I occurs in Basin 2.
2. **I/I Reduction Cost Effective Analysis**: The City should conduct a study to determine which is more cost-effective: (1) locating the sources of I/I and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional rainfall dependent I/I flow.

1 Introduction

1.1 Scope and Purpose

V&A Consulting Engineers (V&A) was retained by Stantec to perform sanitary sewer flow monitoring for the City of Dixon (City). Open-channel flow monitoring was performed at three (3) sites for approximately 11 weeks February 7, 2020 to April 22, 2020. There were three general purposes for this study.

1. Establish the baseline sanitary sewer flows at the flow monitoring sites.
2. Establish the peak flow condition during the rainfall events and estimate available sewer capacity.
3. Quantify inflow/ infiltration (I/I) at the applicable flow monitoring sites.

1.2 Flow Monitoring Sites and Basins

Flow monitoring sites are identified as the manholes where the flow monitors were secured and the pipelines in which the flow sensors were placed. Capacity analysis and flow rate information is presented on a site-by-site basis. The flow monitoring site locations were selected and approved by Stantec. Information regarding the flow monitoring locations is listed in Table 1-1.

Table 1-1. List of Monitoring Locations

Monitoring Site	Monitored Pipe	Diameter (in)	Location
FM 1	North inlet	30	Yale Dr., 100 feet north of Parkway Blvd.
FM 2	West Inlet	27	Parkway Blvd., 900 feet west of Yale Dr.
FM 3	North Inlet	15	South 1st St., 525 feet south of W. Cherry St.

Flow monitoring site data may include the flows of one or many drainage basins. Flow monitoring basins are localized areas of a sanitary sewer collection system upstream of a given location (often a flow meter), including all pipelines, inlets, and appurtenances. The basin refers to the ground surface area near and enclosed by the pipelines. A basin may refer to the entire collection system upstream from a flow meter or may exclude separately monitored basins upstream, requiring basin isolation (subtraction of upstream flows). It is noted that Basin 3 flows into Basin 2, requiring a subtraction of flows (see Section 2.4 for more information).

The approximate basins were drawn from the overall system map and should be confirmed by the reviewing Engineer. The I/I analysis performed for this project was analyzed on a basin-by-basin basis.

Figure 1-1 illustrates the flow monitoring locations, basins, and rain gauge locations. Detailed descriptions of the individual flow monitoring sites, including photographs, are included in *Appendix A*.

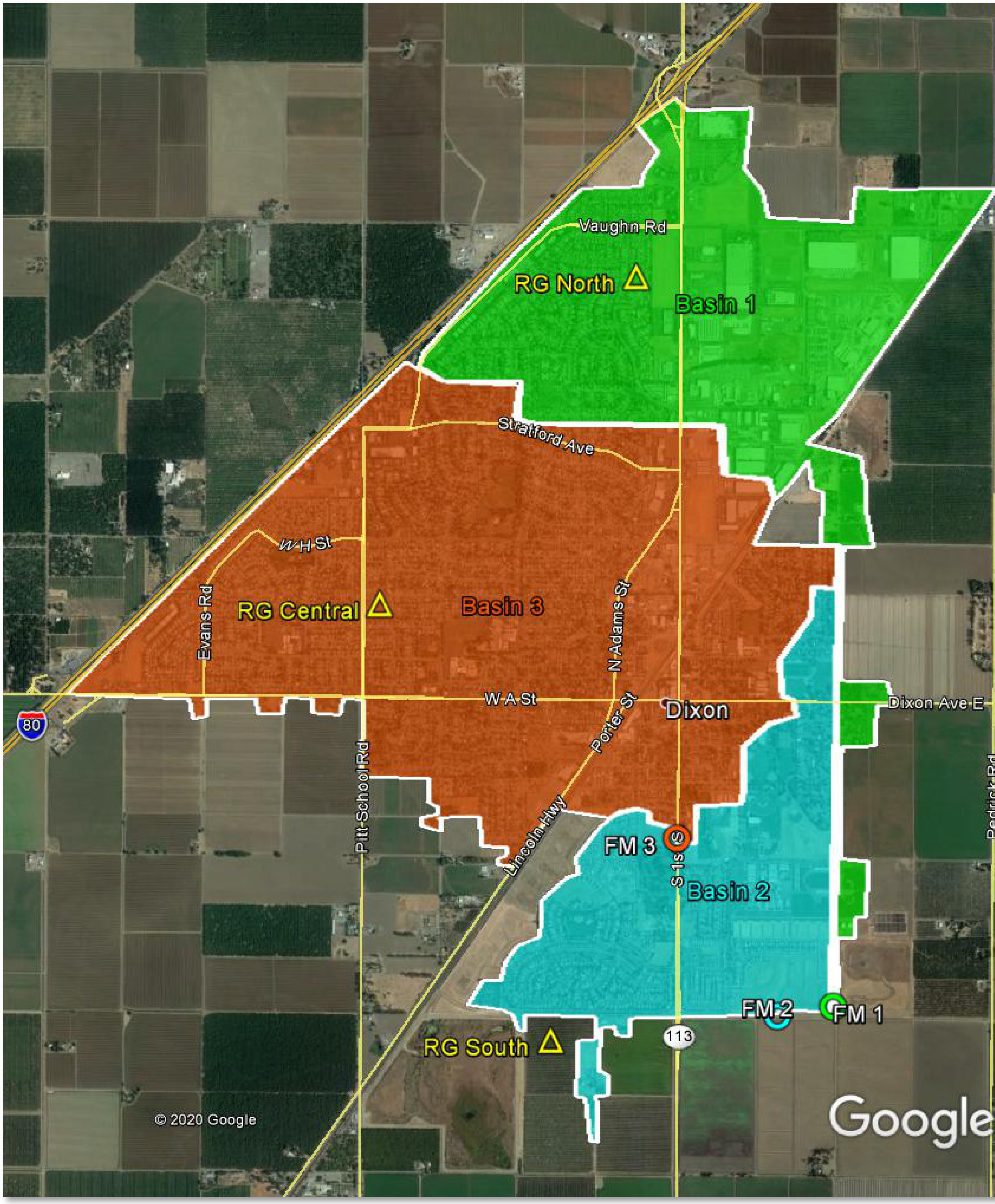

Figure 1-1. Map of Flow Monitoring Sites & Rain Gauge – Overall

2 Methods and Procedures

2.1 Confined Space Entry

A confined space (Photo 2-1) is defined as any space that is large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit and is not designed for continuous employee occupancy. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5% to 23.5%), and the presence of hydrogen sulfide (H_2S) gas, carbon monoxide (CO) gas, and lower explosive limit (LEL) levels. A typical confined space entry crew has members with OSHA-defined responsibilities of Entrant, Attendant, and Supervisor. The Entrant is the individual performing the work. He or she is equipped with the necessary personal protective equipment needed to perform the job safely, including a personal four-gas monitor (Photo 2-2). If it is not possible to maintain line-of-sight with the Entrant, then more Entrants are required until line-of-sight can be maintained. The Attendant is responsible for maintaining contact with the Entrants to monitor the atmosphere using another four-gas monitor and maintaining records of all Entrants if there is more than one. The Supervisor is responsible for developing the safe work plan for the job at hand prior to entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

V&A installed three (3) ISCO 2150 flow meters for temporary monitoring within the collection system. ISCO 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during the installation of the flow meters and again when they were removed and compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

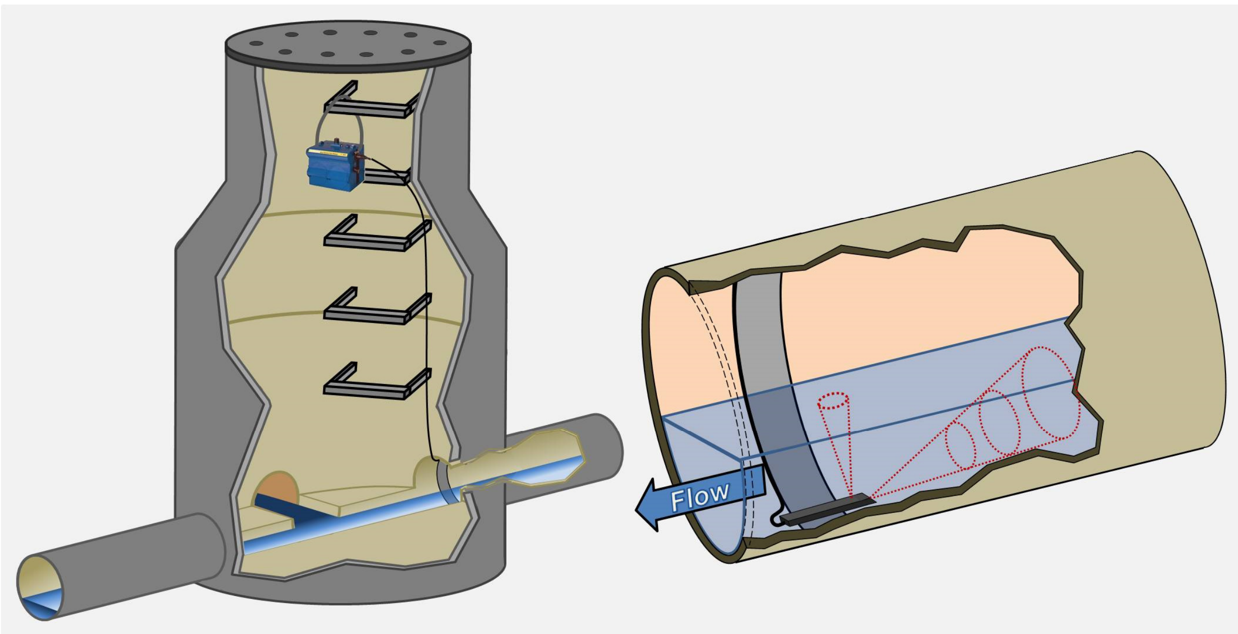

Figure 2-1. Typical Installation for ISCO 2150 Flow Meter with Submerged Sensor

2.3 Flow Calculation

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

$$Q = v \cdot A = v \cdot (A_T - A_S)$$

where Q: volume flow rate

v: average velocity as determined by the ultrasonic sensor

A: cross-sectional area available to carry the flow

A_T: total cross-sectional area with both wastewater and sediment

A_S: cross-sectional area of sediment.

For circular pipe,

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

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

where d_W: distance between wastewater level and pipe invert

d_S: depth of sediment

D: pipe diameter

2.4 Measurement Error and Uncertainty

For traditional engineering applications, measurement “error” is explained as a difference between a computed, estimated, or measured value and the generally accepted true or theoretically correct value. It can also be thought of as a difference between the desired and the actual performance of equipment. For equipment, error is usually expressed as a percentage relative to accuracy (i.e., “…the velocity sensor has an accuracy of ±2% of the reading…”).

However, for this study and flow monitoring applications, the cause of the measurement difference is important and a distinction will be made between the equipment not performing to industry standards (“error”) and expected inaccuracies (“uncertainty”) associated with monitoring technology limitations.

Gauging **“error”** occurs when the equipment is not performing to industry standards. This can occur as a result of the following common categories of conditions that can be encountered at a wastewater monitoring site.

- Malfunctioning equipment (i.e. a sensor is damaged, battery life ends, or a desiccant canister becomes saturated)
- Improper equipment choice or maintenance (i.e. the selected gauging equipment technologies are incompatible with hydraulic conditions within the sewer, or excessive gravel deposits are allowed to accumulate around the sensors without being removed)
- Improper equipment calibration (i.e. depth and/or velocity measurements are incorrectly taken within the sewer, or equipment is allowed to drift out of calibration)
- Field conditions within the sewer, (i.e. foaming at the water surface that “blinds” an ultrasonic depth sensor, or toilet paper catching and accumulating on a combination sensor, blinding the acoustic Doppler velocity meter)

For flow monitoring applications, gauging **“uncertainty”** is used to describe and quantify the expected inaccuracies that result from the limitations of the technologies that utilize indirect measurements to quantify wastewater flow.

It is important to try and install flow meters in “ideal” flow conditions. Ideal flow conditions are generally defined by as laminar flow in a straight-through, constant-slope pipeline with no disturbances (elbows, tees, hydraulic shifts, etc.) 10 diameters upstream and 5 diameters downstream from the flow monitoring location. If ideal flow conditions are met, then an expected uncertainty of final flow calculation from an open-channel flow meter may be approximately ±5%. For many situations, ideal flow conditions cannot be met and uncertainties increase.

2.4.1 Flow Addition versus Flow Subtraction

Due to the uncertainties involved in subtracting flows of similar magnitudes, the addition of flows at multiple monitoring sites is usually preferred over subtraction of flows. Subtraction becomes an issue especially when the flow difference from the subtraction falls within the measurement uncertainty range of the two larger flow data sets (i.e. subtracting a large flow from another large flow to obtain a small difference).

This concept is best demonstrated per the following example:

1. Meter A measures 2.00 MGD of flow and has an expected uncertainty of ±5%, thus the uncertainty range of the flow measurement is ±0.10 MGD.
2. Meter B measures 2.50 MGD of flow and has an expected uncertainty of ±6%, thus the uncertainty range of the flow measurement is ±0.15 MGD.

3. Meter C measures 0.50 MGD of flow and has an expected uncertainty of ±8%, thus the uncertainty range of the flow measurement is ±0.04 MGD.

<u>Scenario 1 – Flow Addition</u>

- Meter A + Meter B = 2.00 MGD (±0.10) + 2.50 MGD (±0.15) = 4.50 MGD (±0.25)
- Overall uncertainty = ±0.25 / 4.50 = ±5.6%
- For flow addition, the final uncertainty is essentially a weighted average of the component uncertainties.

<u>Scenario 2 – Flow Subtraction, Large Flow less Small Flow</u>

- Meter B - Meter C = 2.50 MGD (±0.15) - 0.50 MGD (±0.04) = 2.00 MGD (±0.19)
- Overall uncertainty = ±0.19 / 2.00 = ±9.5%
- For flow subtraction, the final uncertainty will always be greater than the component uncertainties.
- When subtracting a small flow from a large flow, the resulting uncertainties can still be manageable.

<u>Scenario 3 – Flow Subtraction, Large Flow less a similarly Large Flow</u>

- Meter B - Meter A = 2.50 MGD (±0.15) – 2.00 MGD (±0.10) = 0.50 MGD (±0.25)
- Overall uncertainty = ±0.25 / 0.50 = ±50%
- When subtracting a similarly sized flow rates, the resulting uncertainties may not be manageable. In this example, an uncertainty of ±50% may be considered unacceptable for confident analyses.

Scenario 3 is a very "real-world" situation. The uncertainties for Meter A and Meter B are extremely reasonable (indeed, most flow monitoring service providers would be extremely pleased with true meter uncertainties of ±5% to ±6%). However, the reality of the math is clear and the above example demonstrates the concept of flow subtraction and compounding or inflating uncertainty ranges.

The following points are emphasized in relation to the items of this section:

- For subtraction of flows, the overall uncertainty can be an inflated value that far exceeds the component uncertainties.
- The smaller the resultant flow from the subtraction equation, the larger the percentage uncertainty.
- Whenever possible, basins flows should be directly measured, rather than calculated as a subtraction of two or more flow meters.
- If flow subtraction cannot be avoided, it is better to have the magnitudes of the component flows be as dissimilar as possible.

2.5 Average Dry Weather Flow Determination

For this study, four distinct average dry weather flow curves were established for each site location:

- Mondays – Thursdays
- Fridays
- Saturdays
- Sundays

Flows for many sites differ on Friday evenings compared to Mondays through Thursdays. Starting around 7 pm, the flows are often decreased (compared to Monday through Thursday). Similarly, flow patterns for Saturday and Sunday were also separated due to their unique evening flow pattern. This type of differentiation can be important when determining I/I response, especially if a rain event occurs on a Friday, Saturday, or Sunday evening.

illustrates a sample of varying flow patterns within a typical dry week.

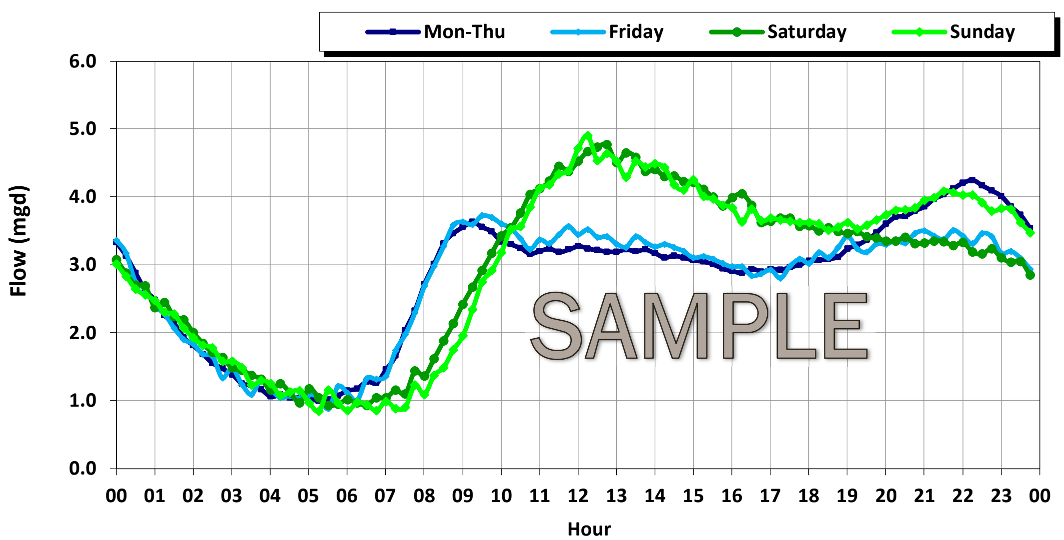

Figure 2-2. Sample ADWF Diurnal Flow Patterns

ADWF curves are taken from "Dry Days" when RDI had the least impact on the baseline flow. The overall average dry weather flow (ADWF) was calculated per the following equation:

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

2.6 Flow Attenuation

Flow attenuation in a sewer collection system is the natural process of the reduction of the peak flow rate through redistribution of the same volume of flow over a longer period of time. This occurs as a result of friction (resistance), internal storage and diffusion along the sewer pipes. Fluids are constantly working towards equilibrium. For example, a volume of fluid poured into a static vessel with no outside turbulence will eventually stabilize to a static state, with a smooth fluid surface without peaks and valleys. Attenuation within a sanitary sewer collection system is based upon this concept. A flow profile with a strong peak will tend to stabilize towards equilibrium, as shown in Figure 2-3.

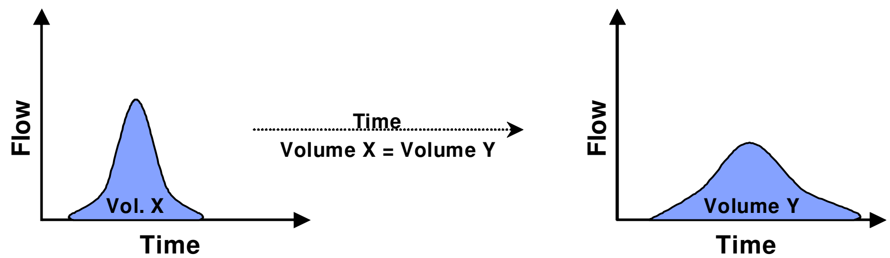

Figure 2-3. Attenuation Illustration

Within a sanitary sewer collection system, each individual basin will have a specific flow profile. As the flows from the basins combine within the trunk sewer lines, the peaks from each basin will (a) not necessarily coincide at the same time, and (b) due to the length and time of travel through the trunk sewers, peak flows will attenuate prior to reaching the treatment facility. The sum of the peak flows of the individual basins within a collection system will usually be greater than the peak flows observed at the treatment facility.

2.7 Inflow / Infiltration Analysis: Definitions and Identification

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

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

Figure 2-4 illustrates the possible sources and components of I/I.

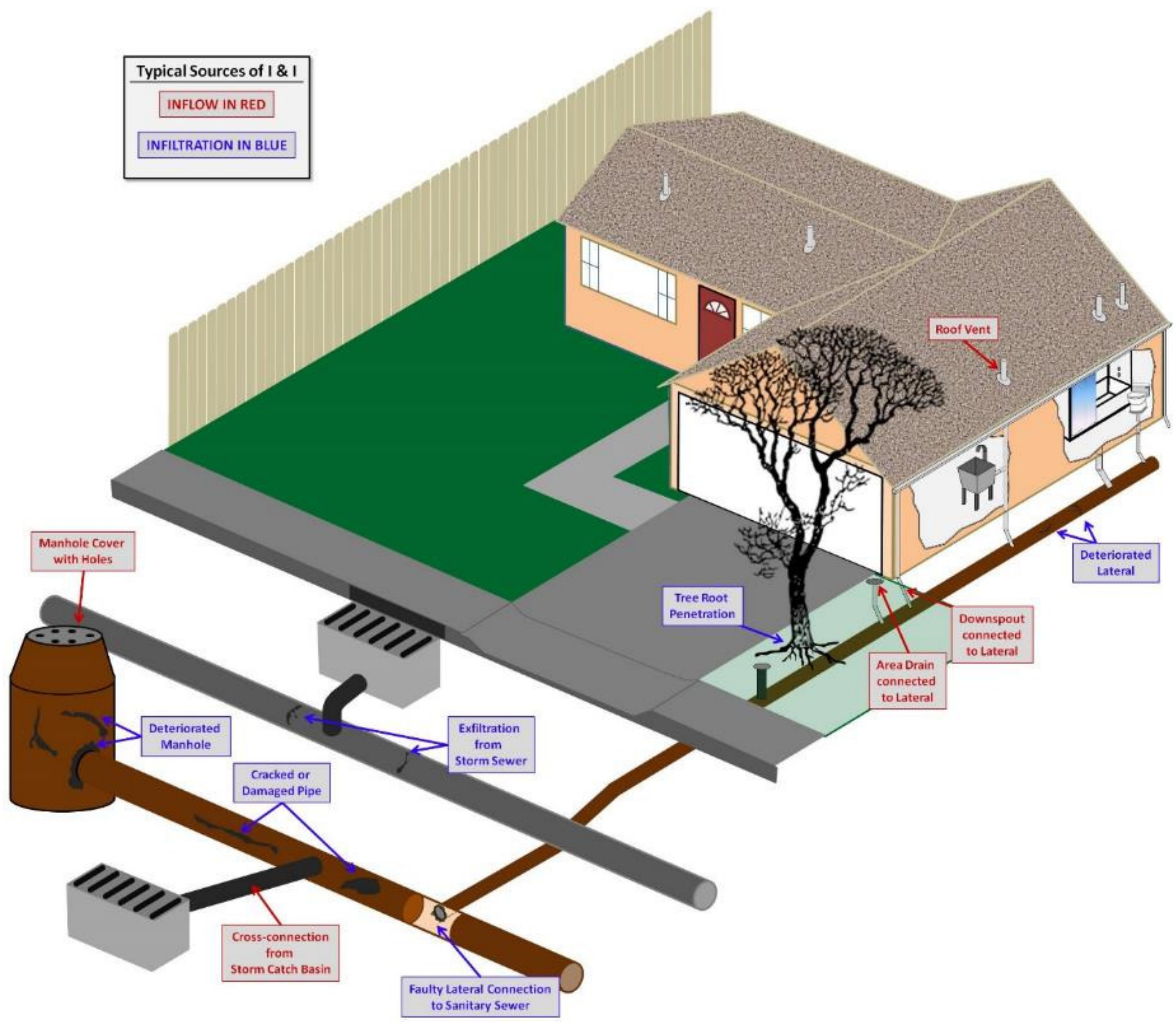

Figure 2-4. Typical Sources of Infiltration and Inflow

2.7.1 Infiltration Components

Infiltration can be further subdivided into components as follows:

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

2.7.2 Impact and Cost of Source Detection and Removal

- **Inflow:**
  - Impact: Inflow creates a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows. Because the response and magnitude of inflow are tied closely to the intensity of the storm event, the short-term peak instantaneous flows may result in surcharging and overflows within a collection system. Severe inflow may result in sewage dilution, resulting in upsetting the biological treatment (secondary treatment) at the treatment facility.
  - Cost of Source Identification and Removal: Inflow locations are usually less difficult to find and less expensive to correct. These sources include direct and indirect cross-connections with storm drainage systems, roof downspouts, and various types of surface drains. Generally, the costs to identify and remove sources of inflow are low compared to potential benefits to public health and safety or the costs of building new facilities to convey and treat the resulting peak flows.
- **Infiltration:**
  - Impact: Infiltration typically creates long-term annual volumetric problems. The major impact is the cost of pumping and treating the additional volume of water, and of paying for treatment (for municipalities that are billed strictly on flow volume).
  - Cost of Source Detection and Removal: Infiltration sources are usually harder to find and more expensive to correct than inflow sources. Infiltration sources include defects in deteriorated sewer pipes or manholes that may be widespread throughout a sanitary sewer system.

2.7.3 Graphical Identification of I/I

Inflow is usually recognized graphically by large-magnitude, short-duration spikes immediately following a rain event. Infiltration is often recognized graphically by a gradual increase in flow after a wet-weather event. The increased flow typically sustains for a period after rainfall has stopped and then gradually drops off as soils become less saturated and as groundwater levels recede to normal levels. Real-time flows are plotted against ADWF to analyze the I/I response to rainfall events. Figure 2-5 illustrates a sample of how this analysis is conducted and some of the measurements that are used to distinguish infiltration and inflow. Similar graphs have been generated for the individual flow monitoring sites and can be found in Appendix A.

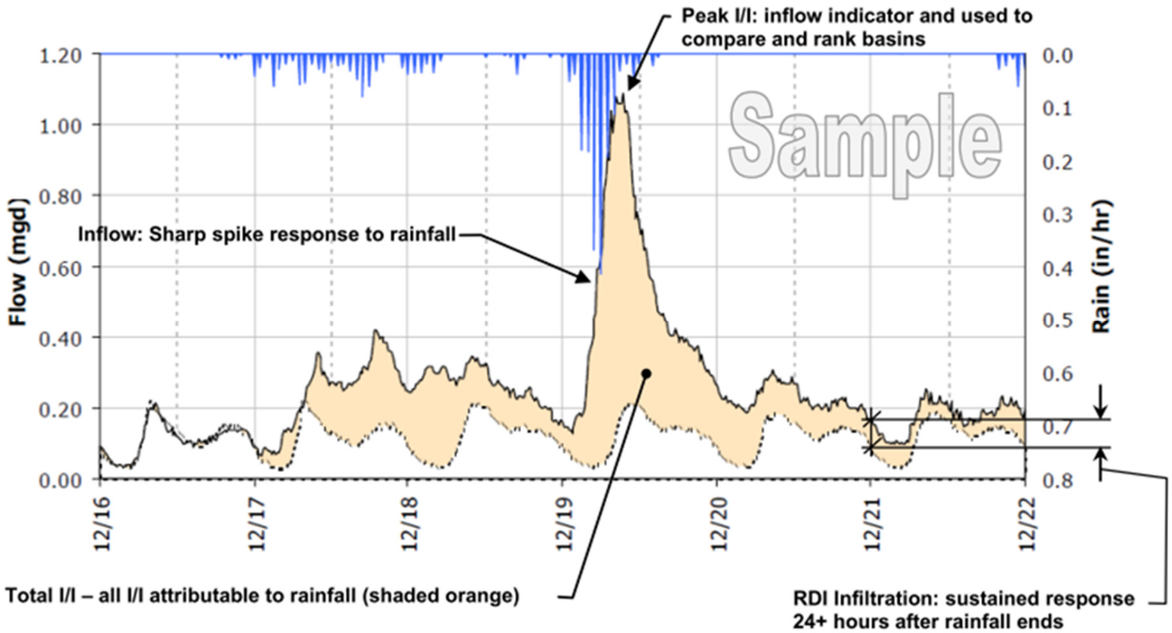

Figure 2-5. Sample Infiltration and Inflow Isolation Graph

2.7.4 Analysis Metrics

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

- **Inflow – Peak I/I Flow Rate**: Inflow is characterized by sharp, direct spikes occurring during a rainfall event. Peak I/I rates are used for inflow analysis. [4]
- **Groundwater Infiltration (GWI)**: GWI analysis is conducted by looking at minimum dry weather flow to average dry weather flow ratios and comparing them to established standards to quantify the rate of excess groundwater infiltration.
- **Rainfall-Dependent Infiltration (RDI)**: RDI Analysis is conducted by looking at the infiltration rates at set periods after the conclusion of a storm event. Depending on the particular collection system and the time required for flows to return to ADWF levels, different periods may be

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

examined to determine the basins with the greatest or most sustained rainfall-dependent infiltration rates.

- **Combined I/I**: The combined inflow and infiltration is measured in gallons per site and per storm event. Because it is based on combined I/I volume, it is used to identify the overall volumetric influence of I/I within the monitoring basin.

2.7.5 Normalization Methods

There are three ways to *normalize* the I/I analysis metrics for an "apples-to-apples" comparison among the different drainage basins:

- **per-ADWF**: The metric is divided by the established average dry weather flow rate and typically expressed as a ratio. Peaking Factors are examples of using ADWF to normalize data from different sites.
- **per-IDM**: The metric is divided by the length of pipe (IDM [inch-diameter mile]) contained within the upstream basin. Final units typically are gallons per day (gpd) per IDM.
- **per-ACRE**: The metric is divided by the acreage of the upstream basin. Final units typically are gallons per day (gpd) per ACRE.

The infiltration and inflow indicators were normalized by the per-ADWF and the per-ACRE methods in this report. V&A did not have IDM information for this study.

3 Results and Analysis

3.1 Rainfall

V&A captured rainfall data from publicly available private weather stations (PWS[5]), allowing for good coverage over the flow monitoring area (refer to Figure 1-1).

Graphs containing rainfall data overlaid with the flow data are included in Appendix A. Figure 3-1 shows the rainfall during the flow monitoring period averaged between the three rain gauges and Figure 3-2 shows the rain accumulation plot of the period rainfall, as well as the historical average rainfall[6] during this project duration. Table 3-1 summarizes the rainfall that fell between the three rain gauges. The cumulative precipitation for the rain gauges triangulated to between the two historical rain gauge stations was between 28% and 39% of historical precipitation totals for the duration of the flow monitoring.

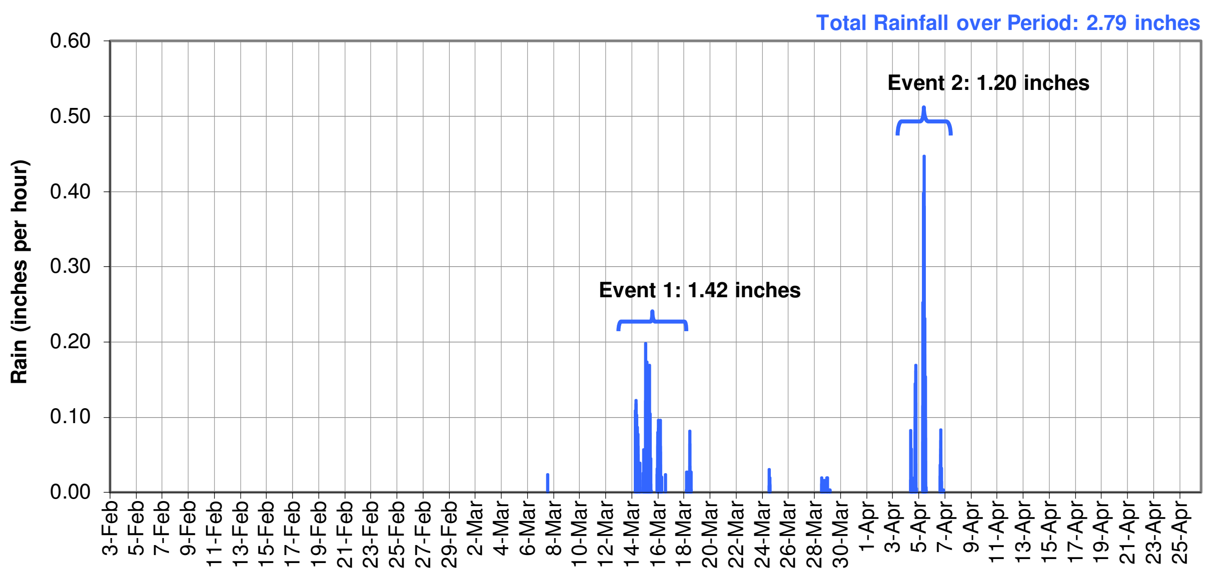

Figure 3-1. Rainfall during Flow Monitoring Period – Average of 3 Rain Gauges

[5] National Oceanic and Atmospheric Administration (NOAA) Citizen Weather Observer Program (CWOP) members send data from their PWS to the NOAA MADIS server; the data undergoes quality checking and then is distributed. While V&A has no direct control over the rain gauges, V&A performs additional QA/QC on the data to ensure its suitability for use.

[6] Historical data taken from the WRCC (average of Station 49200 in Vacaville and Station 42294 in Davis): http://www.wrcc.dri.edu/summary/climsmnca.html

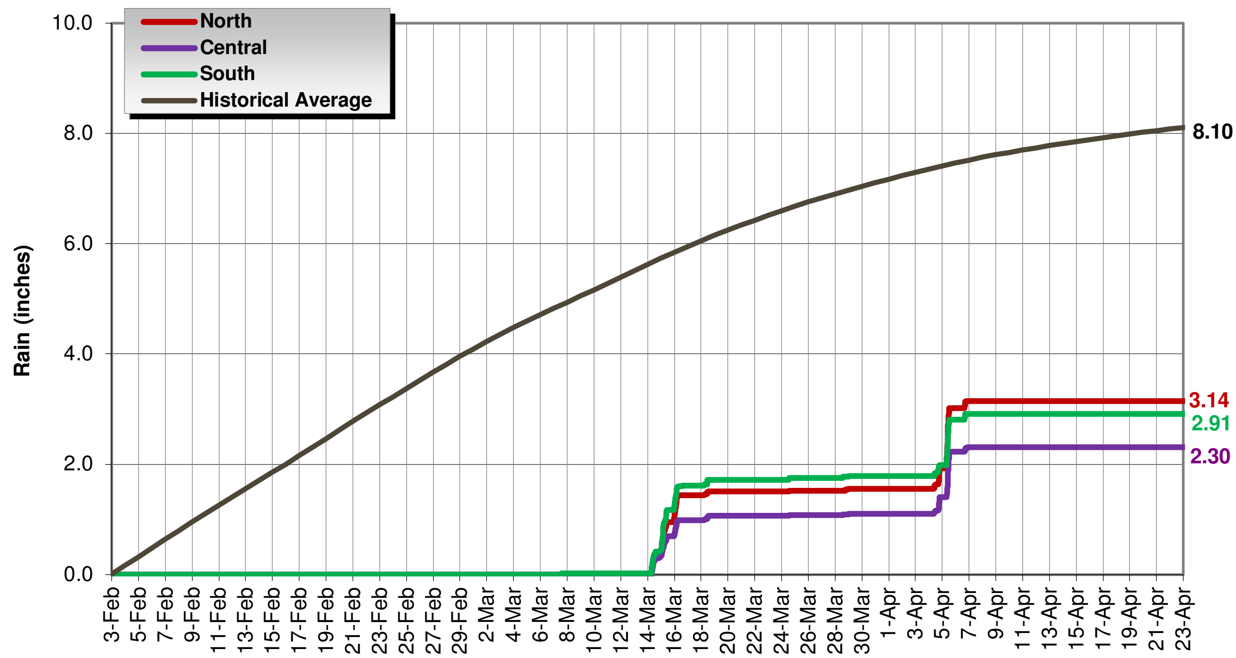

Figure 3-2. Rainfall Accumulation Plot

Table 3-1. Summary of Rainfall

Rain Gauge	Storm Event 1 – March 14 – 18, 2020 (inches)	Storm Event 2 – April 4 – 8, 2020 (inches)	Season Total (inches)
North	1.51	1.59	3.14
Central	1.06	1.20	2.30
South	1.70	1.13	2.91

3.1.1 Regional Rainfall Event Classification

It is important to classify the relative size of a major storm event that occurs over the course of a flow monitoring period[7]. Rainfall events are classified by intensity and duration. Based on historical data, frequency contour maps for storm events of given intensity and duration have been developed by the NOAA for all areas within the continental United States (Figure 3-3).

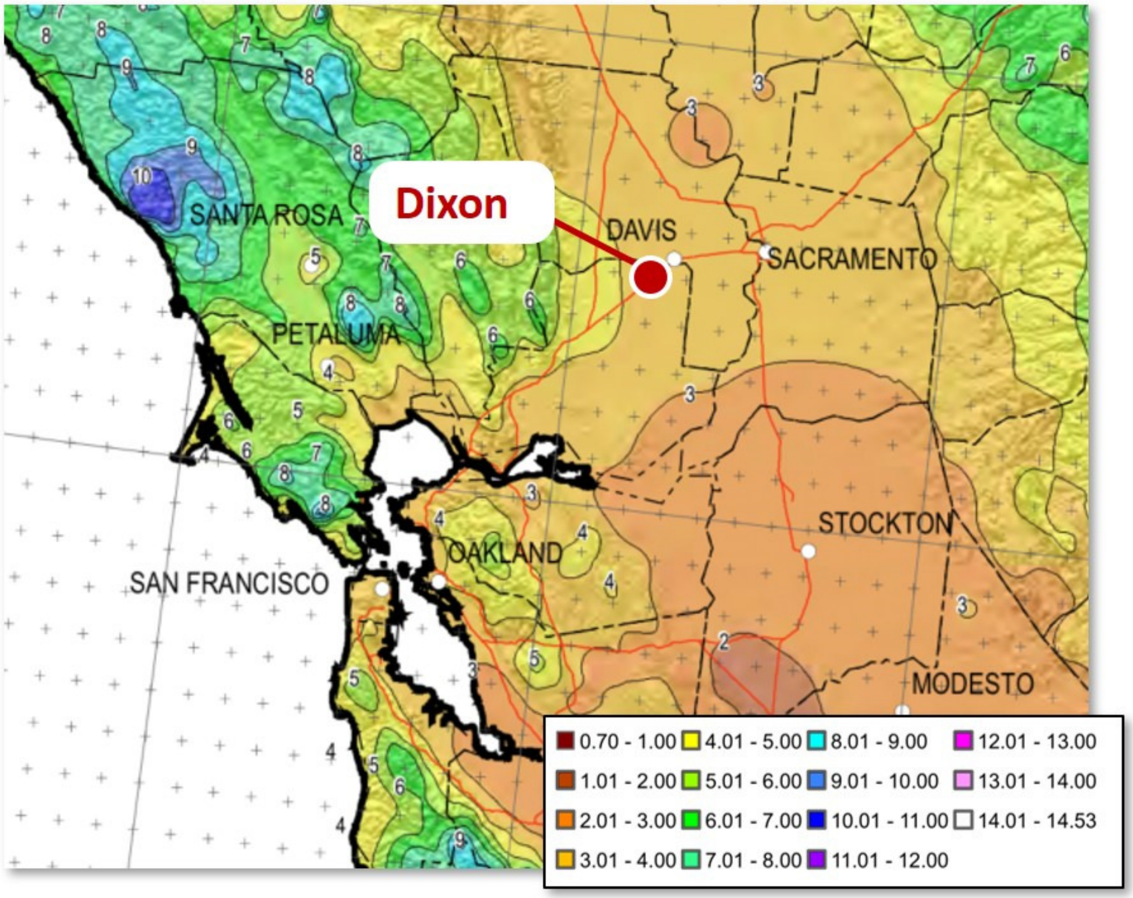

Figure 3-3. NOAA Northern California Rainfall Frequency Map

For example, the NOAA Rainfall Frequency Atlas[8] classifies a 10-year, 24-hour storm event at the South rain gauge location as **3.76** inches. This means that in any given year, at this specific location, there is a 10% chance that 3.76 inches of rain will fall in any 24-hour period.

From the NOAA frequency maps, for a specific latitude and longitude, the rainfall densities for period durations ranging from 1 hour to 20 days are known for rain events ranging from 1-year to 10-year intensities. These are plotted to develop a rain event frequency map specific to each rainfall monitoring site. Superimposing the peak measured densities for the rainfall events on the rain event frequency plot determines the classification of the rainfall event.

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

[8] NOAA Western U.S. Precipitation Frequency Maps Atlas 14, Volume 6, 2011: ftp://hdsc.nws.noaa.gov/pub/hdsc/data/sw/ca10y24h.pdf

The April 4/5 rainfall event at the North Rain Gauge was classified as a 1-year, 6-hour event. All other rainfall events at the three rain gauge locations were classified as less than 1 year, 24-hour events. Figure 3-4 illustrates the rain event classification plots per the rain gauge location.

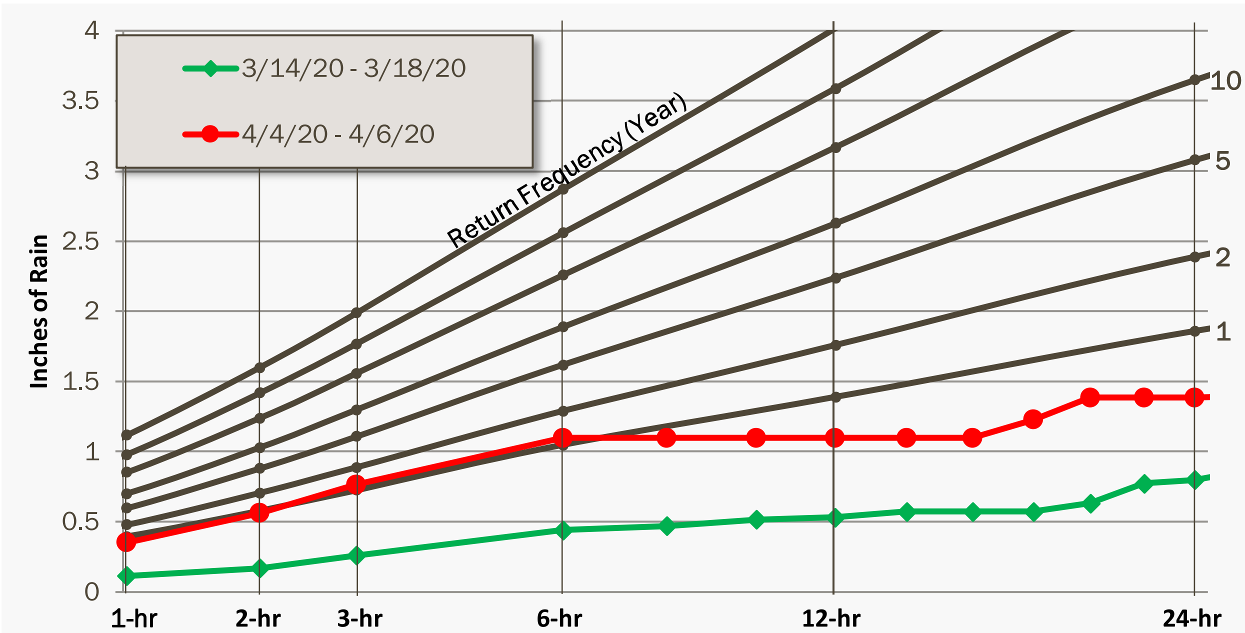

Figure 3-4. Rainfall Event Classification - North Rain Gauge

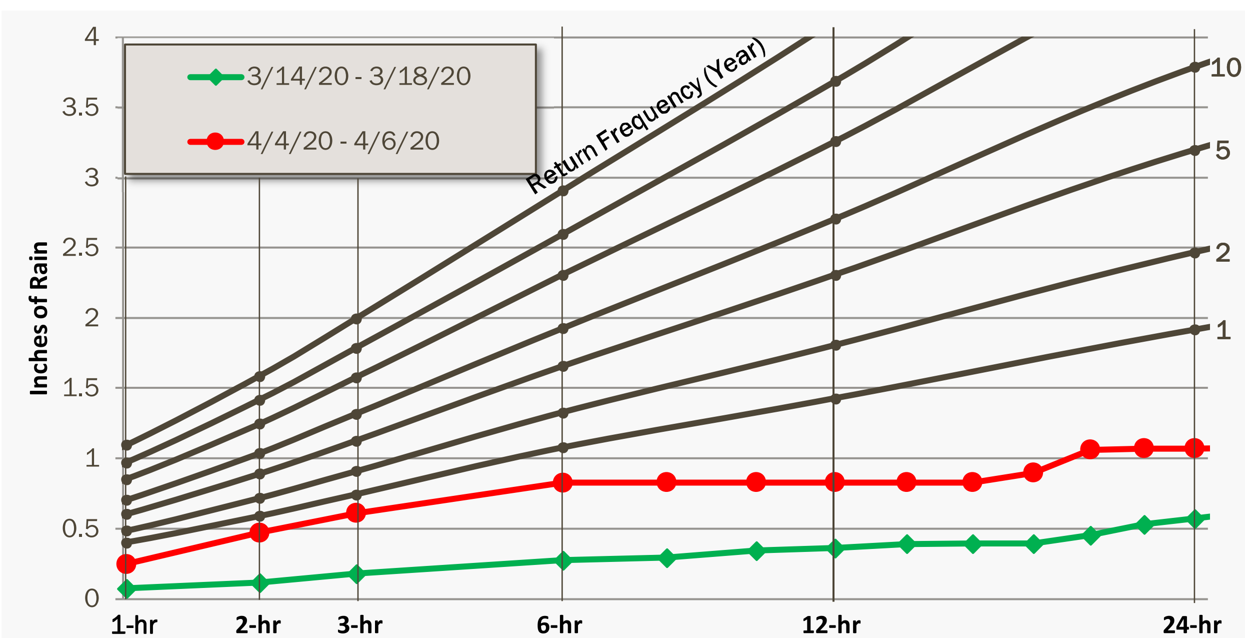

Figure 3-5. Rainfall Event Classification - Central Rain Gauge

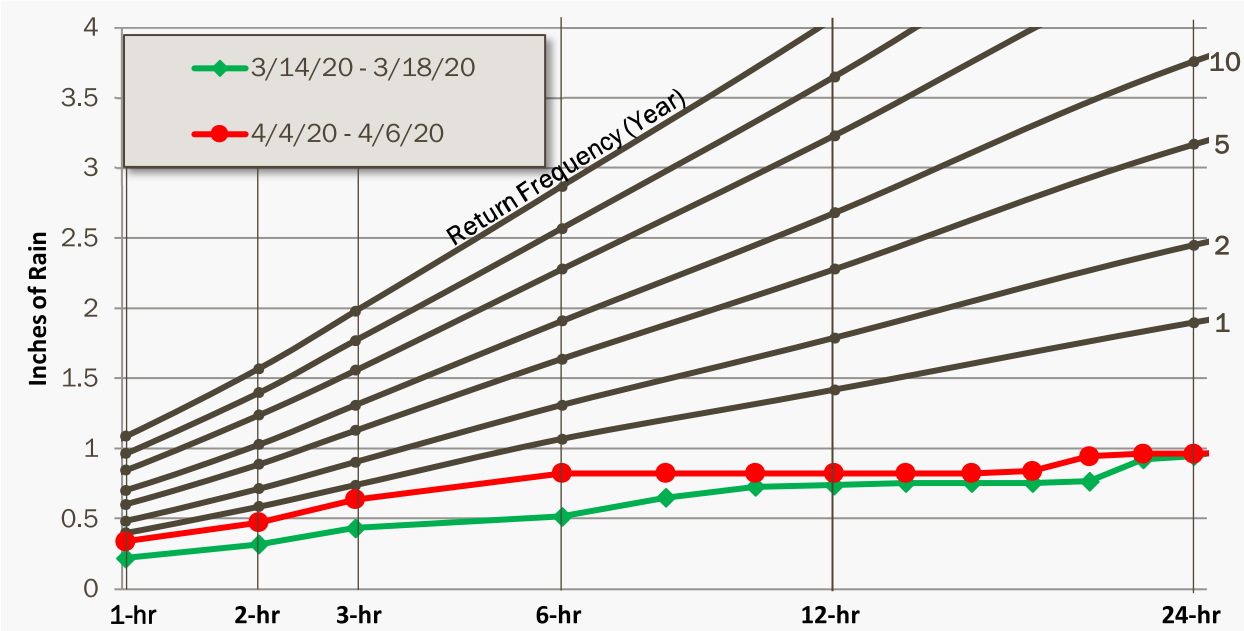

Figure 3-6. Rainfall Event Classification - South Rain Gauge

3.2 Flow Monitoring

3.2.1 Average Flow Analysis

For this study, two sets of average dry weather flow (ADWF) curves were established due to the advent of "shelter-in-place" (SIP) order for Covid-19. There were generally three time periods during this study, as detailed below and shown in Figure 3-7.

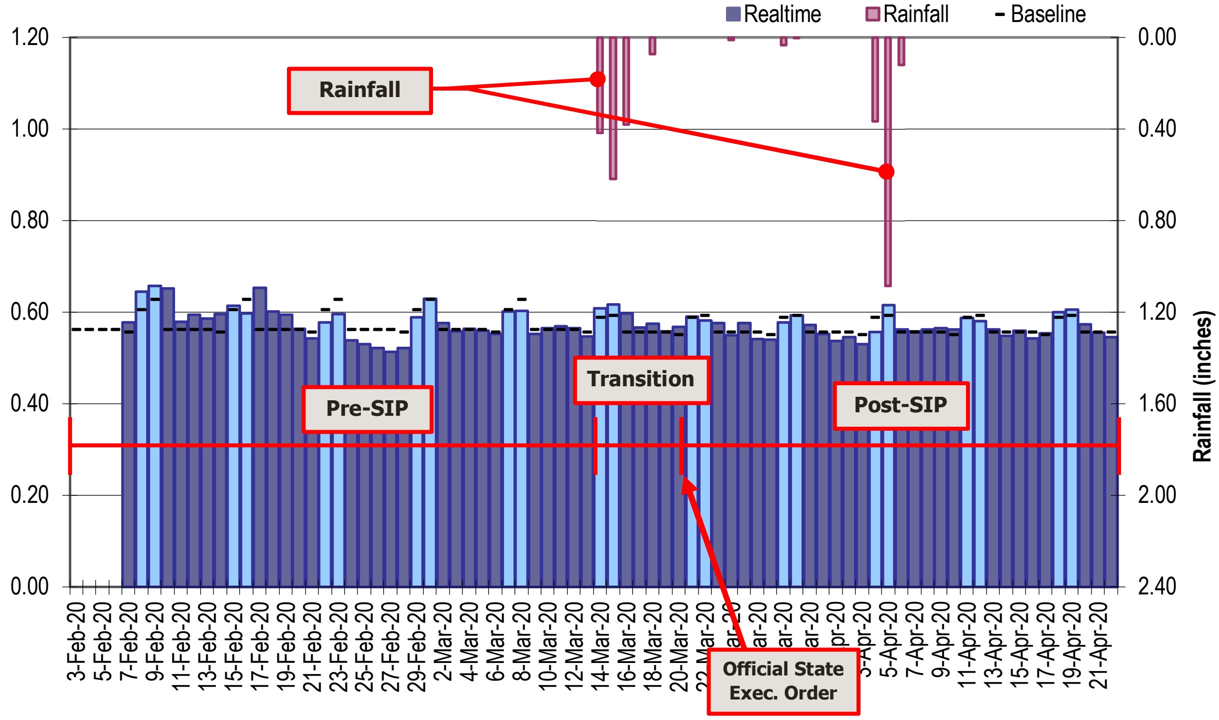

Figure 3-7. Period Daily Flow Totals - FM 3

1. **Pre-SIP:** One set of ADWF was established during "typical" dry days prior to SIP and can be used for modelling purposes. Due to the lack in rainfall in Northern California, the dry days that could be used pre-SIP were February 7 – February 15 and February 19 – March 6, 2020.
   - **Sediment:** Site FM-11 had sediment levels within the pipeline measured at approximately 2 inches during this study.
2. **Transition period:** On February 27, 2020, Solano County declared a local emergency. On March 16, schools were closed per order of Solano County Department of Public Health, the lead local agency for this public health matter in this area. Although "Shelter-in-Place" was officially announced on March 20, 2020, it is clear from the flow monitoring data that people were already starting to behave (in terms of sewage) as in SIP since March 6. By Sunday, March 15, people were mostly in SIP mode.
3. **Post-SIP:** The second set of ADWF was established during "typical" dry days post SIP. The dry days that could be used post-SIP were March 19-30, April 1 – 3, 7 - 11, and 13 – 22, 2020. The post-SIP ADWFs were used for I/I analysis of rainfall post-SIP (April 4 and 5 rainfall).

There were differences in ADWF patterns and volumes before and after the 'shelter-in-place' order. Daily peaks were delayed by several hours and had a dampened early-morning peak. For example, Figure 3-8 shows the sets of pre-SIP ADWF, post-SIP curves for Site 3, and a direct comparison of the pre- and post-SIP curves for weekday diurnal patterns.

Table 3-2 summarizes the dry weather flow data measured for this study. ADWF curves for each site pre-SIP and post- SIP can be found in *Appendix A*. Figure 3-9 illustrates a flow schematic of the ADWF values (pre-SIP) for the flow monitoring sites of this study.

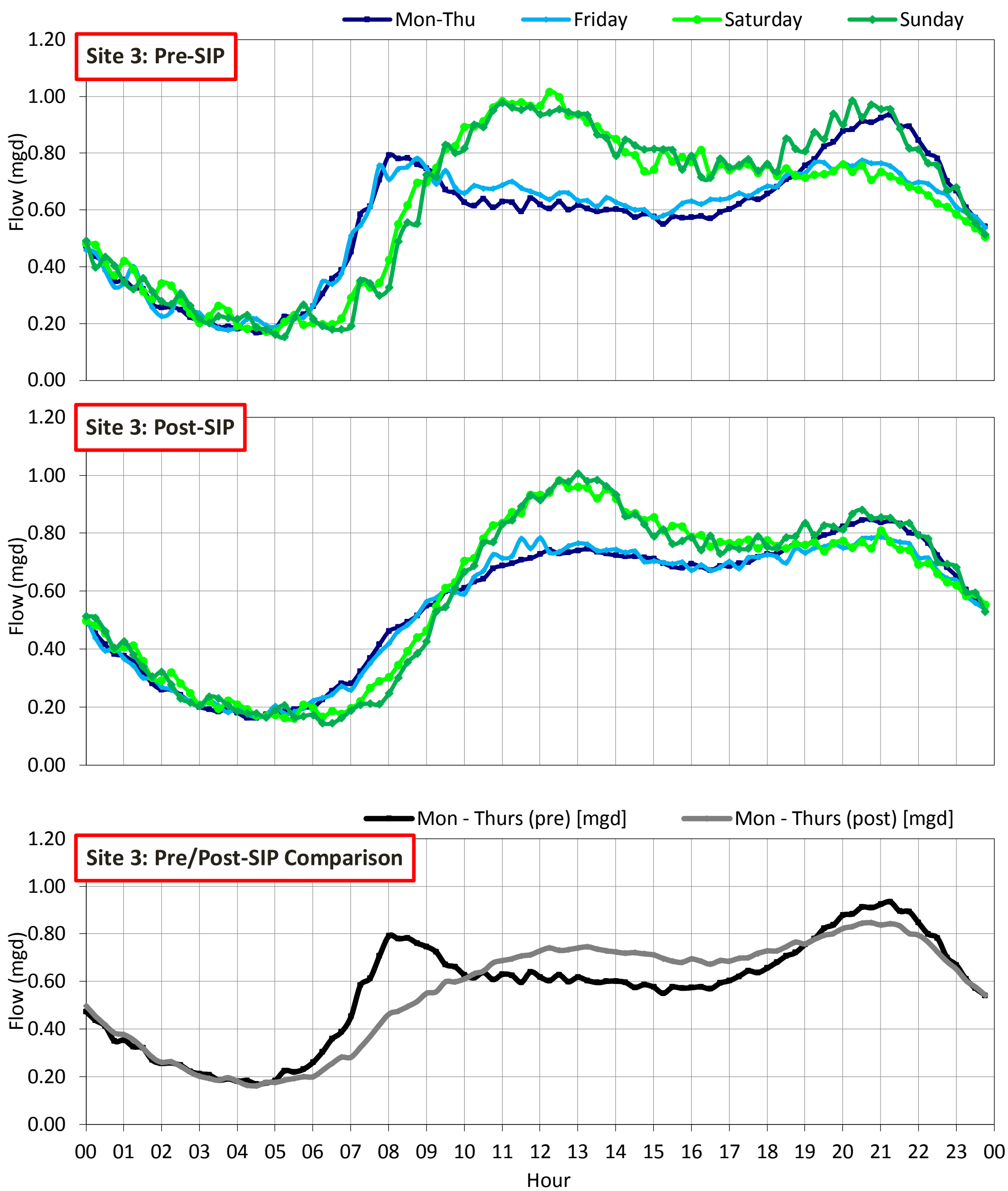

Figure 3-8. Site 3 pre-SIP and post-SIP Average Dry Weather Flow Curves with Comparison

Table 3-2. Dry Weather Flow

Site	Pre-SIP ADWF (mgd)					Post-SIP ADWF (mgd)					SIP Delta
	Mon-Thu	Fri	Sat	Sun	Overall	Mon-Thu	Fri	Sat	Sun	Overall	
FM 1	0.289	0.275	0.250	0.227	0.273	0.282	0.288	0.251	0.242	0.273	0%
FM 2	0.763	0.740	0.795	0.853	0.777	0.838	0.835	0.862	0.833	0.840	+8%
FM 3	0.562	0.557	0.606	0.628	0.577	0.557	0.551	0.589	0.593	0.566	-2%

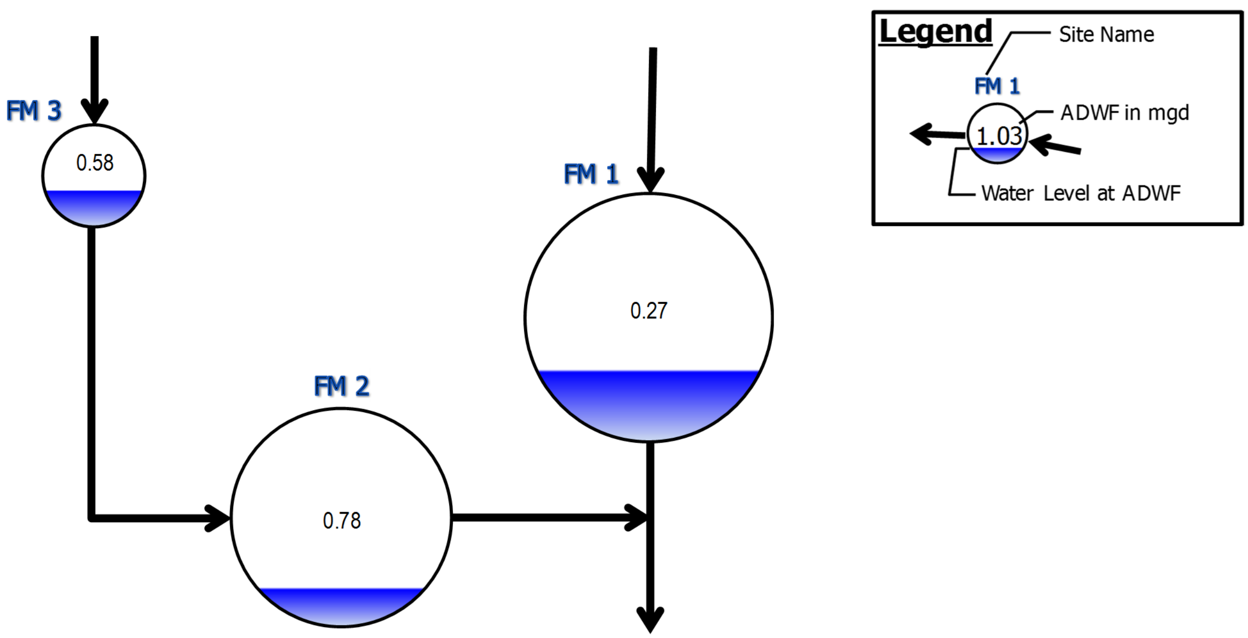

Figure 3-9. Average Dry Weather Flow (Flow Schematic)

3.2.2 Peak Measured Flows and Pipeline Capacity Analysis

Peak measured flows and the hydraulic grade line data (flow depths) are important to understanding the capacity limitations. The capacity analysis terms used in the text below are defined as follows:

- **Peaking Factor**: Peaking factor is defined as the peak measured flow divided by the average dry weather flow (ADWF). Peaking factors are influenced by many factors, including size and topography of the tributary area, flow attenuation, flow restrictions, and characteristics of I/I entering the collection system. Municipal standards for peaking factor vary agency by agency; the District should refer to jurisdictional standards when evaluating peaking factors[9].
- **d/D Ratio**: The d/D ratio is the peak measured depth of flow (d) divided by the pipe diameter (D). The d/D ratio for each site was computed based on the maximum depth of flow for the study. Standards for d/D ratio vary from agency to agency, but typically range between d/D ≤ 0.5 and d/D ≤ 0.75. The District should refer to jurisdictional standards when evaluating d/D ratios.

Table 3-3 summarizes the peak flows, levels, d/D ratios, and peaking factors during the flow monitoring period. Capacity analysis data are presented on a site-by-site basis and represents the hydraulic conditions only at the site nodes; hydraulic conditions in other areas of the collection system will differ.

Table 3-3. Capacity Analysis Summary

Monitored Site	ADWF [A] (MGD)	Peak Measured Flow (MGD)	Peaking Factor	Pipe Diameter, *D* (IN)	Max Depth, *d* (IN)	*Max d/D* Ratio
Site 1	0.273	0.57	2.1	30	10.9	0.36
Site 2	0.777	1.56	2.0	27	6.6	0.24
Site 3	0.577	2.07	3.6	15	10.2	0.68

[A] Pre-SIP ADWF was used for this analysis

The following capacity analysis results are noted:

- **Peak flows**: The peak measured flows were taken from the whole monitoring study, including during the rainfall, and pre- and post-SIP.
  - Site 1: The peak flow occurred on March 11, not during a rainfall event.
  - Site 2: The peak flow occurred on April 5, corresponding to the April 4/5 rainfall event.
  - Site 3: The peak flow occurred on March 2, not during a rainfall event. The peak flow appeared to be artificial and the result of a “hold-and-release” event[10] (flows were held back, then released through the pipeline).
    - The peak flow value for Site 3 is greater than Site 2 due to attenuation (refer to
- **Peaking Factors**: All three sites had peaking factors lower than 4. The highest peaking factor was at Site 3 and was PF = 3.6; this peaking factor was primarily caused be the aforementioned hold-and-release event.

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[9] WEF Manual of Practice FD-6 and ASCE Manual No. 62 suggests typical peaking factor ratios range between 3 and 4, with higher values possibly indicative of pronounced I/I flows.

[10] This March 2 “hold-and-release” event was also observed downstream through Site 2 and accounted for the second highest peak for Site 2.

- **d/D Ratio**: Site 3 was the only site with a d/D ratio greater than 0.5. None of the sites surcharged during the study.

Figure 3-10 shows the schematic diagram of the peak measured flows with peak flow levels of the whole monitoring period. Figure 3-11 shows bar graph summaries of the peaking factors and d/D ratios.

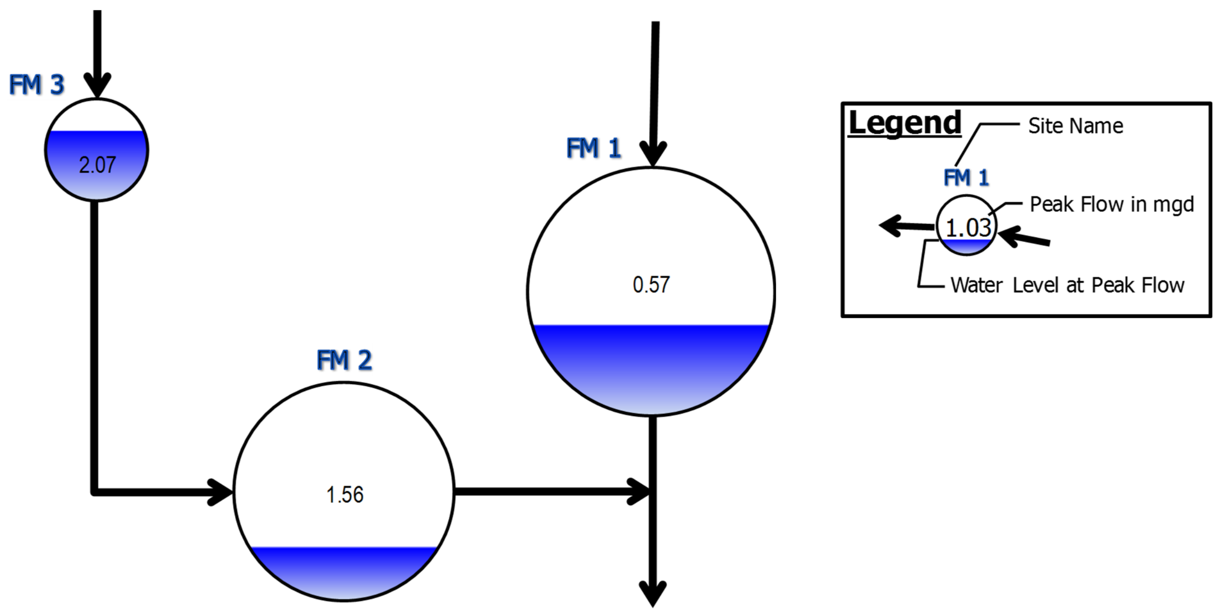


Figure 3-10. Peak Measured Flow (Flow Schematic)

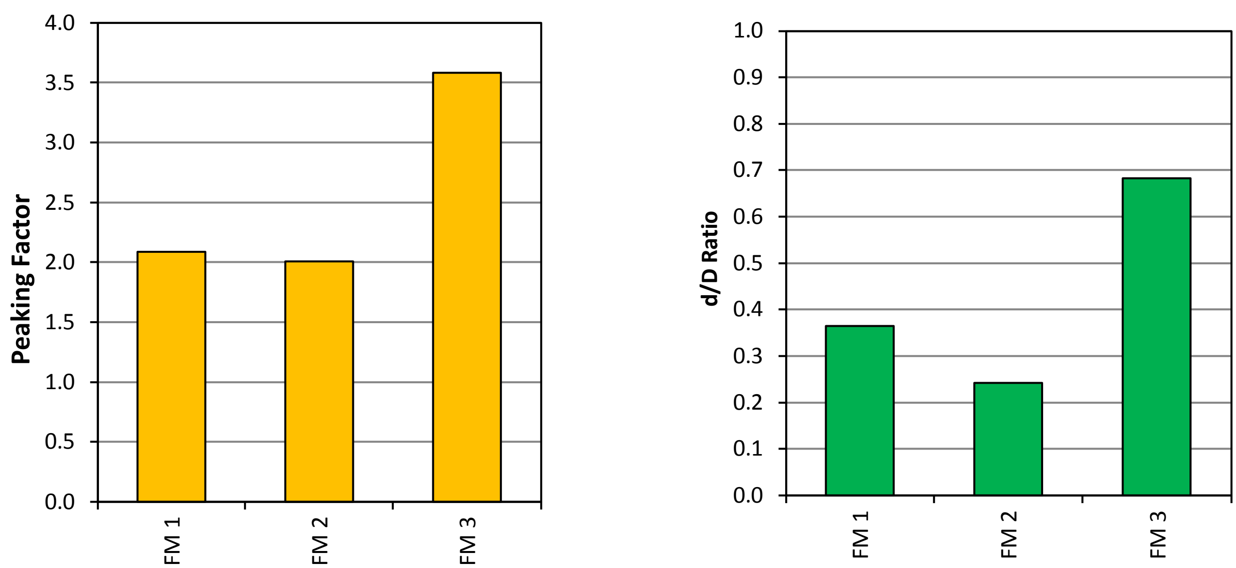


Figure 3-11. Peaking Factors

3.3 Inflow and Infiltration: Results

3.3.1 Preface

Inflow and infiltration (I/I) analyses are presented on a basin-by-basin basis. The following I/I terms are defined and may be useful for the full capacity analysis with proposed development flows:

- **I/I Isolation**: The I/I flow rate is the real-time flow less the estimated average dry weather flow rate (shown in Figure 3-12 as the RED line).
- **Inflow**: Storm water inflow is defined as water discharged into the sewer system, including private sewer laterals, from direct connections such as downspouts, yard, area drains, holes in manhole covers, cross-connections from storm drains, and/or catch basins.
- **Rain-Dependent Infiltration (RDI)**: Infiltration is defined as water entering the sanitary sewer system through defects in pipes, pipe joints, and manhole walls, which may include cracks, offset joints, root intrusion points, and broken pipes.
- **Total I/I**: the totalized volume (in gallons) of both inflow and RDI over the course of a rainfall event (shown below as the orange area).

During the April 5, 2020 rainfall event, the flow monitoring sites exhibited minor levels of inflow, and negligible rain-dependent infiltration (RDI). RDI will not be analyzed as a part of this study.

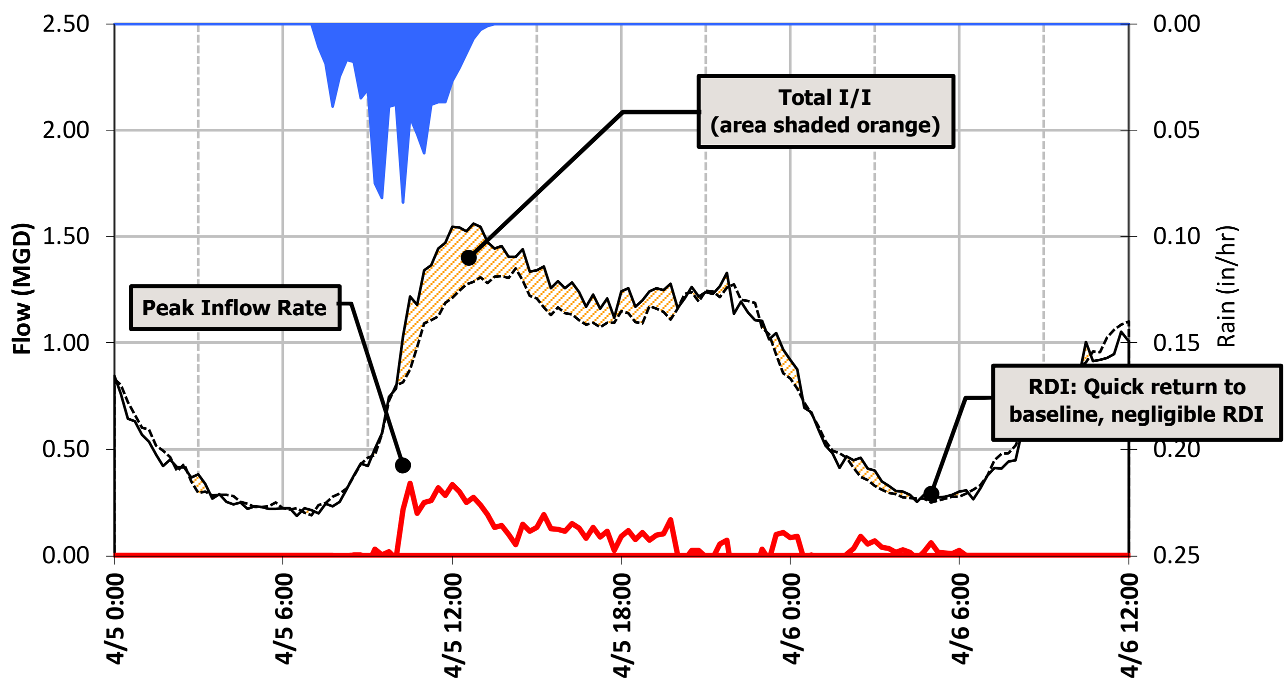


Figure 3-12. I/I during Rain Event (Site 2)

3.3.2 Inflow Results Summary

Inflow is storm water discharged into the sewer system through direct connections such as downspouts, area drains, cross-connections to catch basins, etc. These sources transport rainwater directly into the sewer system and the corresponding flow rates are tied closely to the intensity of the storm. This component of I/I often causes a peak flow problem in the sewer system and often dictates the required capacity of downstream pipes and transport facilities to carry these peak instantaneous flows.

Inflow results were taken from the April 4/5 rainfall event. Table 3-4 and Figure 3-13 summarize the peak measured inflow analysis results by basin. The following items are noted:

- Basin 2 had the highest weighted, normalized peak I/I rates, an indicator of high inflow upstream from the flow monitoring basin.

Table 3-4. Results and Rankings of Inflow Analysis

Metering Basin	ADWF [A] (MGD)	Basin Acreage	Inflow Rate (mgd)	Inflow per ADWF	Inflow per ACRE (gpd/ACRE)	Overall Ranking
Basin 1	0.273	770	0.174	0.64	225	2
Basin 2	0.200	476	0.197	0.99	414	1
Basin 3	0.577	1,361	0.152	0.26	112	3

[A] Pre-SIP ADWF was used for this analysis

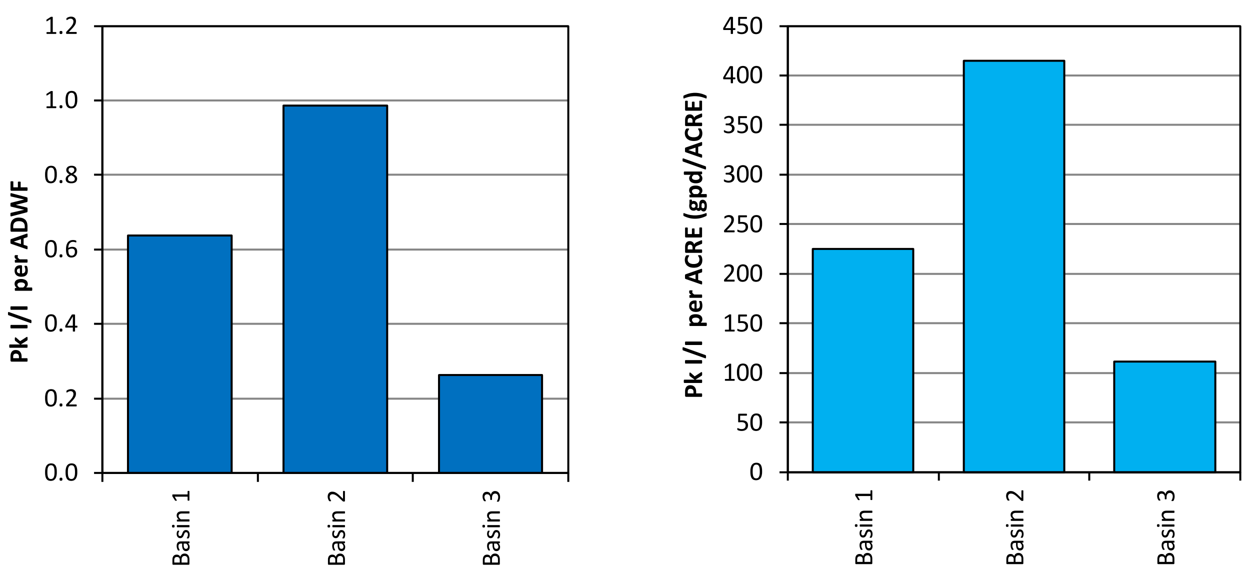


Figure 3-13. Bar Graphs: Inflow Analysis Summary

3.3.3 Total I/I Results

Total I/I analysis considers the totalized volume (in gallons) of both inflow and rainfall-dependent infiltration over the course of a storm event. Total I/I results were taken from the April 4/5 rainfall event. Table 3-5 and Figure 3-14 summarize the total I/I analysis results by basin. The following items are noted:

- Basin 2 had the highest weighted, normalized total I/I rates, an indicator of total I/I upstream from the flow monitoring basin.

Table 3-5. Combined I/I Analysis Summary

Metering Basin	ADWF [A] (MGD)	Basin Acreage	Total I/I (gallons)	Total I/I per ADWF (MGal per in-Rain per MGD)	Total I/I per ACRE (R-value)	Combined I/I Ranking
Basin 1	0.273	770	23,400	0.055	0.07%	2
Basin 2	0.200	476	38,700	0.162	0.25%	1
Basin 3	0.577	1,361	33,900	0.048	0.07%	3

[A] Pre-SIP ADWF was used for this analysis

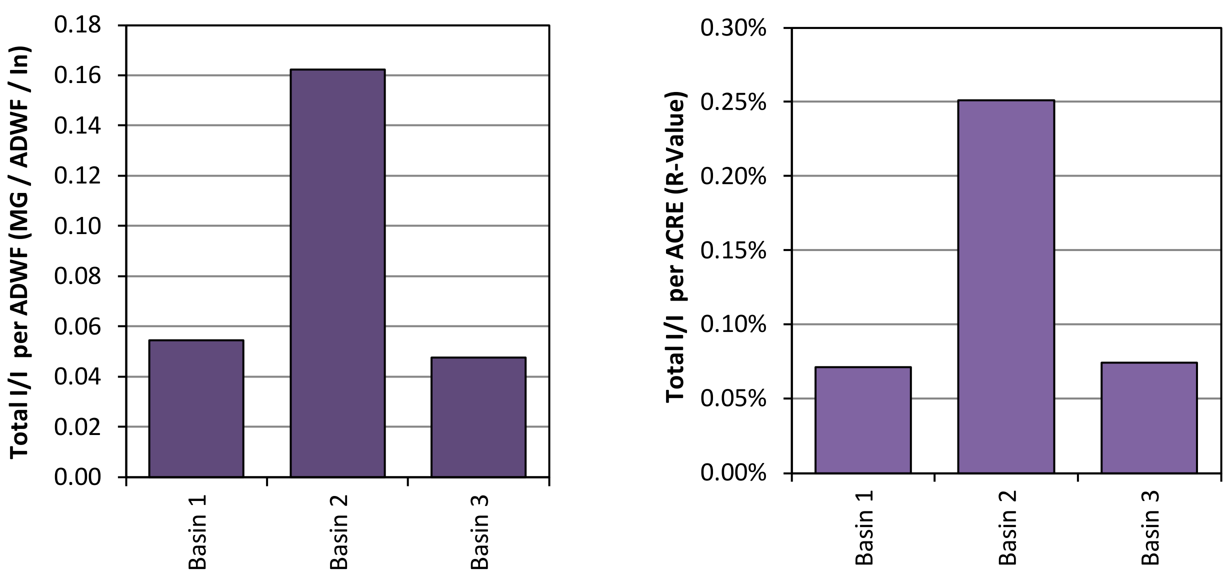


Figure 3-14. Bar Graphs: Combined I/I Analysis Summary

3.3.4 Groundwater Infiltration Results Summary

Dry weather (ADWF) flow can be expected to have a predictable diurnal flow pattern. While each site is unique, experience has shown that, given a reasonable volume of flow and typical loading conditions, the daily flows fall into a predictable range when compared to the daily average flow. If a site has a large percentage of groundwater infiltration occurring during the periods of dry weather flow measurement, the amplitudes of the peak and low flows will be dampened[11]. Figure 3-15 shows a sample of two flow monitoring sites, both with nearly the same average daily flow, but with considerably different peak and low flows. In this *sample* case, Site B1 may have a considerable volume of groundwater infiltration.

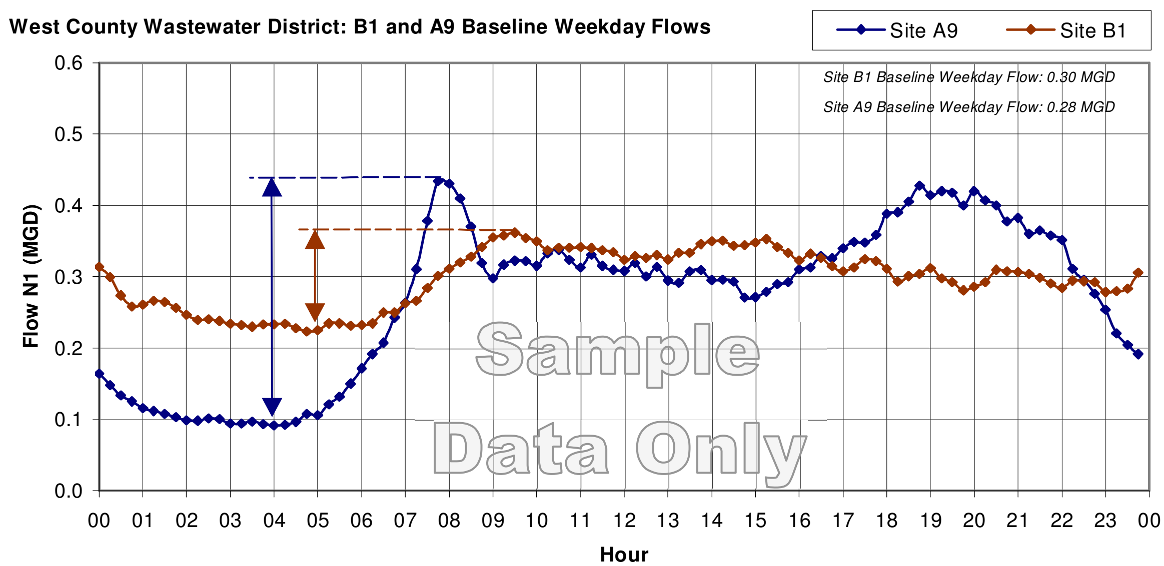


Figure 3-15. Groundwater Infiltration Sample Figure

It can be useful to compare the low-to-ADWF flow ratios for the flow metering sites. A site with abnormal ratios, and with no other reasons to suspect abnormal flow patterns (such as proximity to a pump station, treatment facilities, etc.), has a possibility of higher levels of groundwater infiltration in comparison to the rest of the collection system.

Figure 3-16 plots the low-to-ADWF flow ratios[12] against the ADWF flows for the relevant flow monitoring sites. The brown dashed line shows “typical” low-to-ADWF ratios per the Water Environment Federation (WEF).

FM 1 had evidence of slightly elevated levels of groundwater infiltration, though it is noted that FM 1 may collect sanitary waste from industrial usages which could elevate the low-to-ADWF ratios.

[11] In an extreme case, perhaps 0.2 mgd of ADWF flow and 2.0 mgd of groundwater infiltration, the peaks and lows would be barely recognizable; the ADWF flow would be nearly a straight line.

[12] The Minimum to Average flow ratio is calculated by taking the minimum flow and dividing by the ADWF value (using the Mon-Thu ADWF curve).

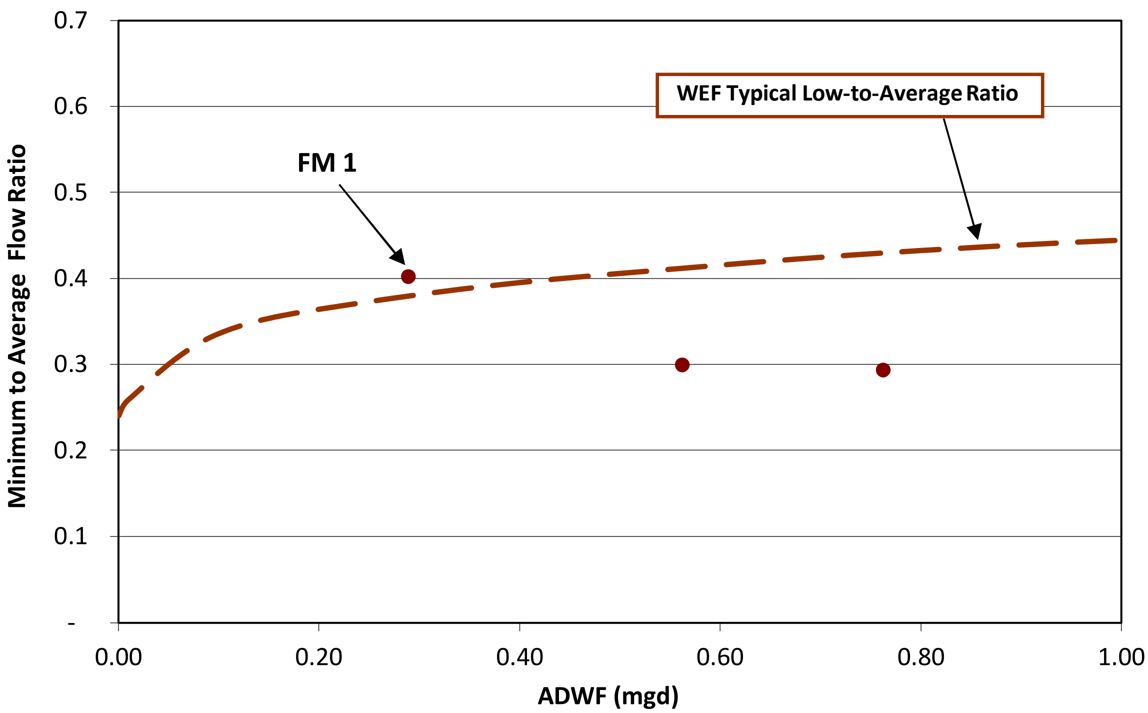


Figure 3-16. Minimum Flow Ratios vs ADWF[13]

[13] 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. This is why the WEF typical trend line's slope is closer to 1.0 as the ADWF increases, as shown in the figure.

4 Recommendations

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

1. **Determine I/I Reduction Program**: The City should examine its I/I reduction needs to determine their needs and goals for a future I/I reduction program.
    a. If peak flows, sanitary sewer overflows, and pipeline capacity issues are of greater concern, then priority can be given to investigate and reduce sources of inflow within the basins with the greatest inflow problems. The highest inflow occurs in Basin 2.
    b. If total infiltration and general pipeline deterioration are of greater concern, then the program can be weighted to investigate and reduce sources of infiltration within the basins with the greatest infiltration problems. The highest combined I/I occurs in Basin 2.
2. **I/I Reduction Cost Effective Analysis**: The City should conduct a study to determine which is more cost-effective: (1) locating the sources of inflow/infiltration and systematically rehabilitating or replacing the faulty pipelines; or (2) continued treatment of the additional rainfall dependent I/I flow.

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

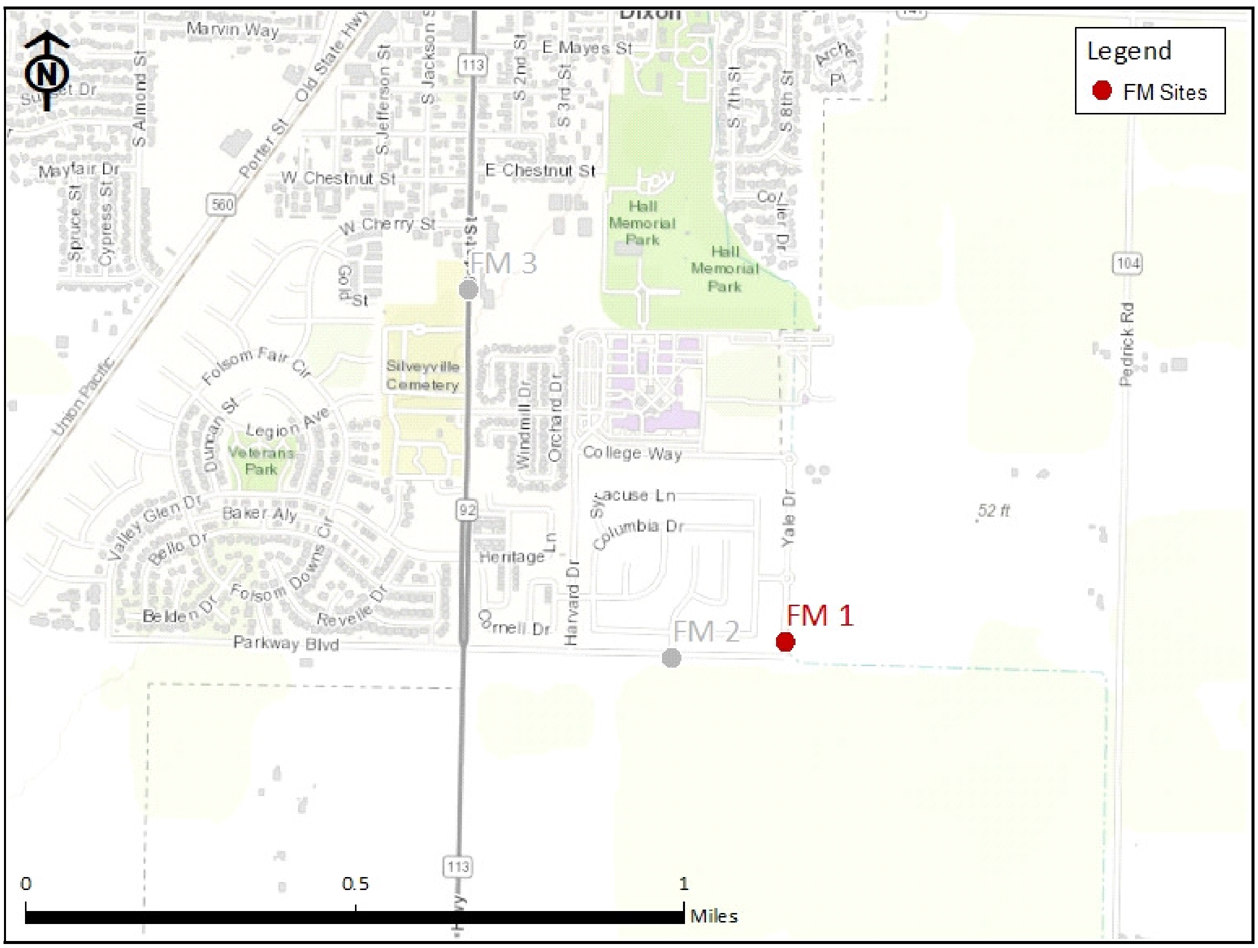
City of Dixon

Sanitary Sewer Flow Monitoring
February 06 - April 23, 2020

Monitoring Site: FM 1

Location: Yale Drive, 100' north of Parkway Blvd

Data Summary Report



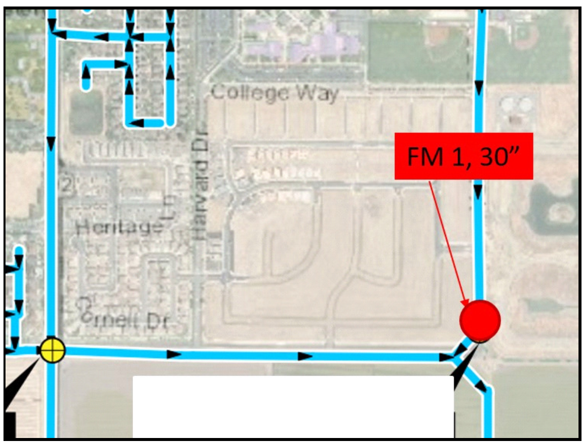
Vicinity Map: FM 1

FM 1

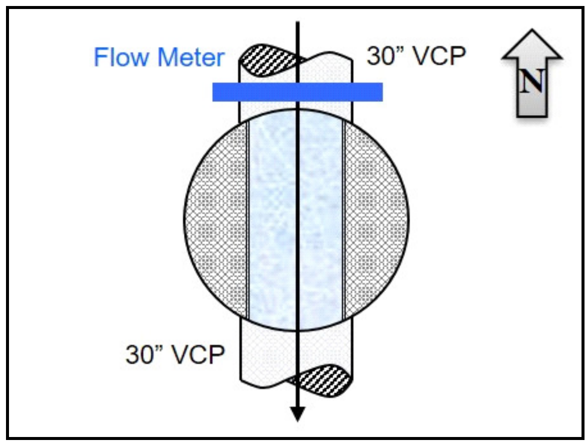
Site Information

Location:	Yale Drive, 100' north of Parkway Blvd
City Manhole:	FM 1
Coordinates:	121.8136° W, 38.4315° N
Rim Elevation (Earth):	50 feet
Pipe Diameter:	30 inches
ADWF:	0.273 mgd
Peak Measured Flow:	0.569 mgd

Satellite Map

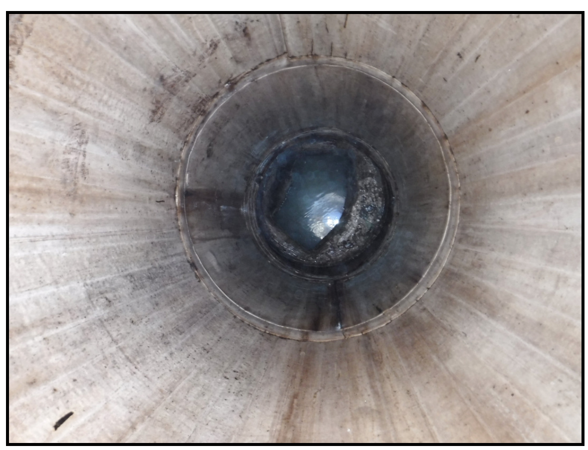


Sanitary Map



Flow Sketch

Street View


Plan View

FM 1

Additional Site Photos

Effluent Pipe

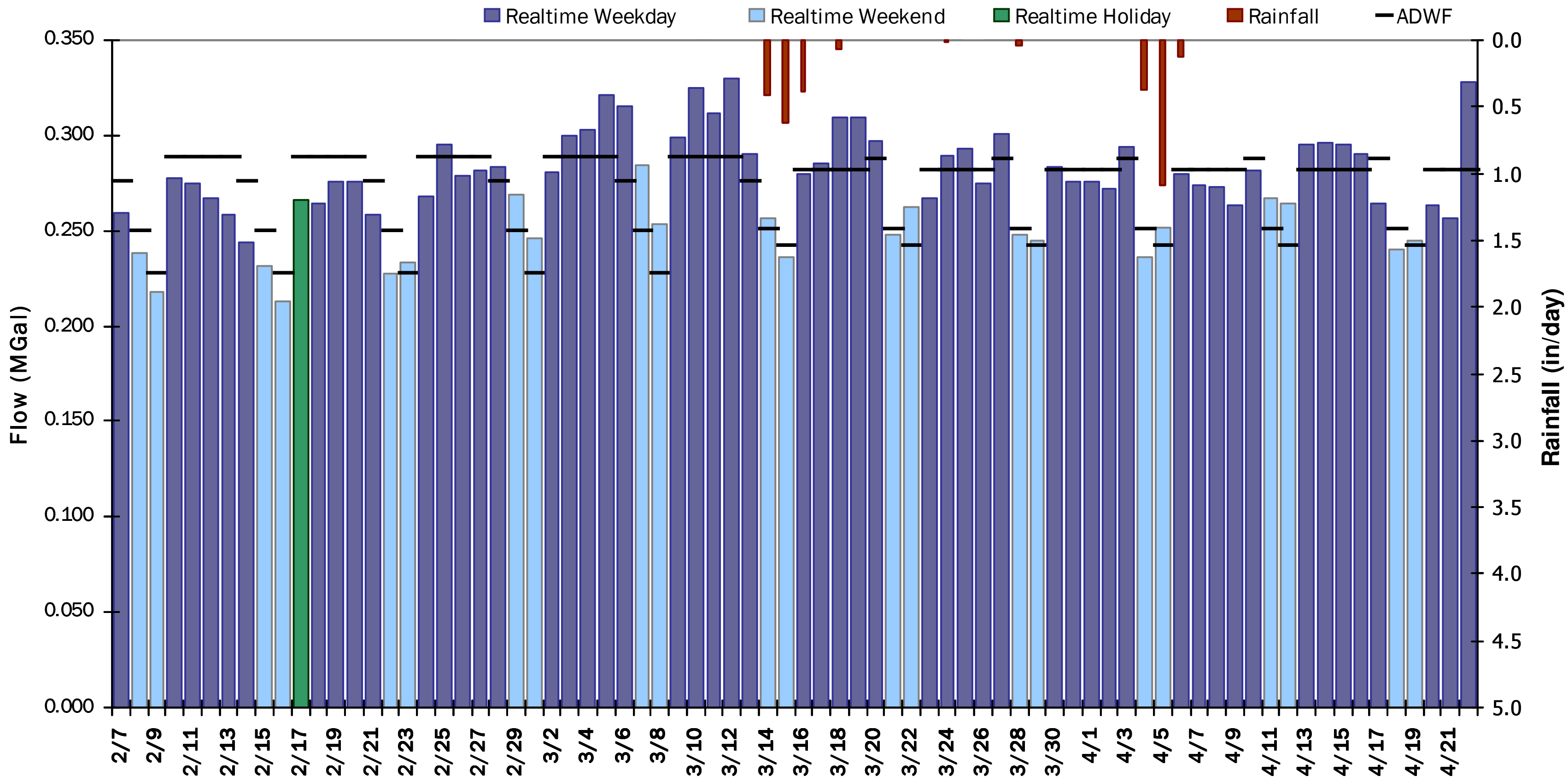
Monitored Influent Pipe

FM 1

Period Flow Summary: Daily Flow Totals

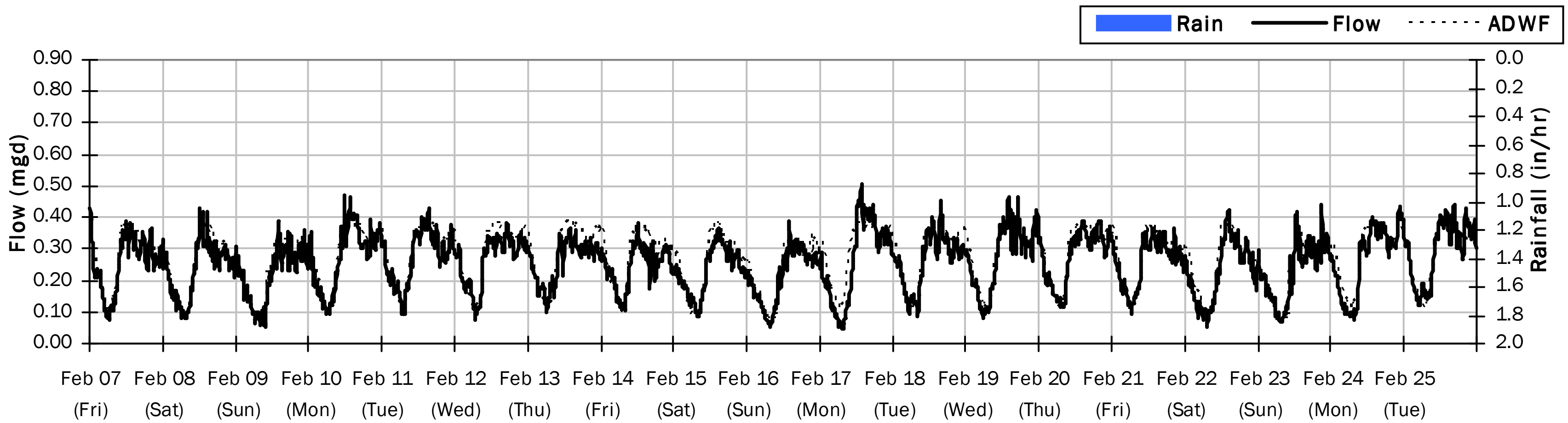
Avg Period Flow: 0.273 MGal **Peak Daily Flow: 0.330 MGal** **Min Daily Flow: 0.213 MGal**

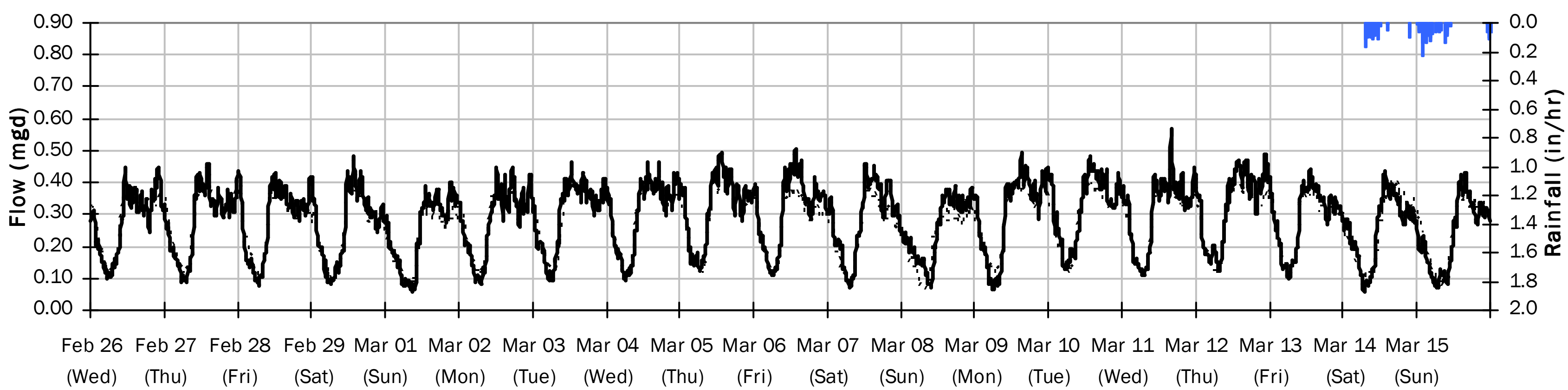
Total Period Rainfall: 3.11 inches



FM 1
Flow Summary: 2/7/2020 to 3/15/2020

Total Period Rainfall: 1.04 inches   **Avg Flow: 0.271 mgd**   **Peak Flow: 0.569 mgd**   **Min Flow: 0.044 mgd**

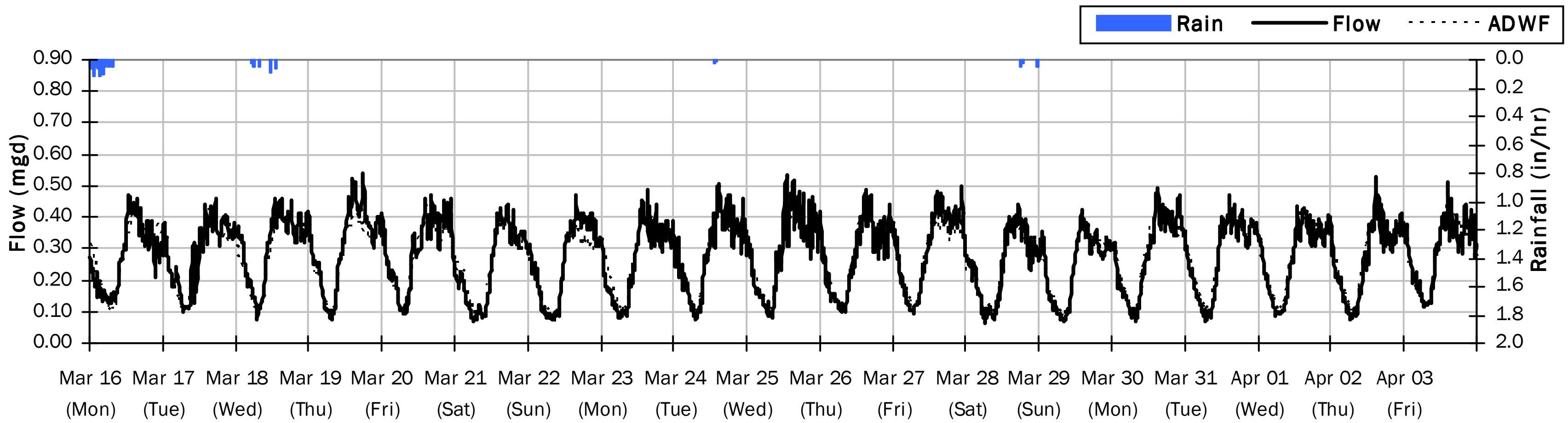


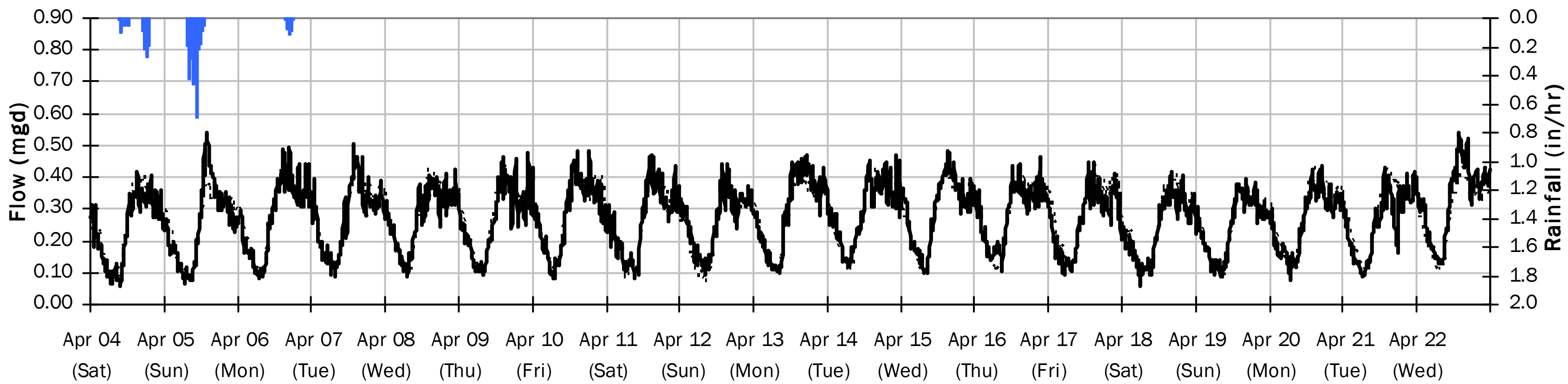


FM 1

Flow Summary: 3/16/2020 to 4/22/2020

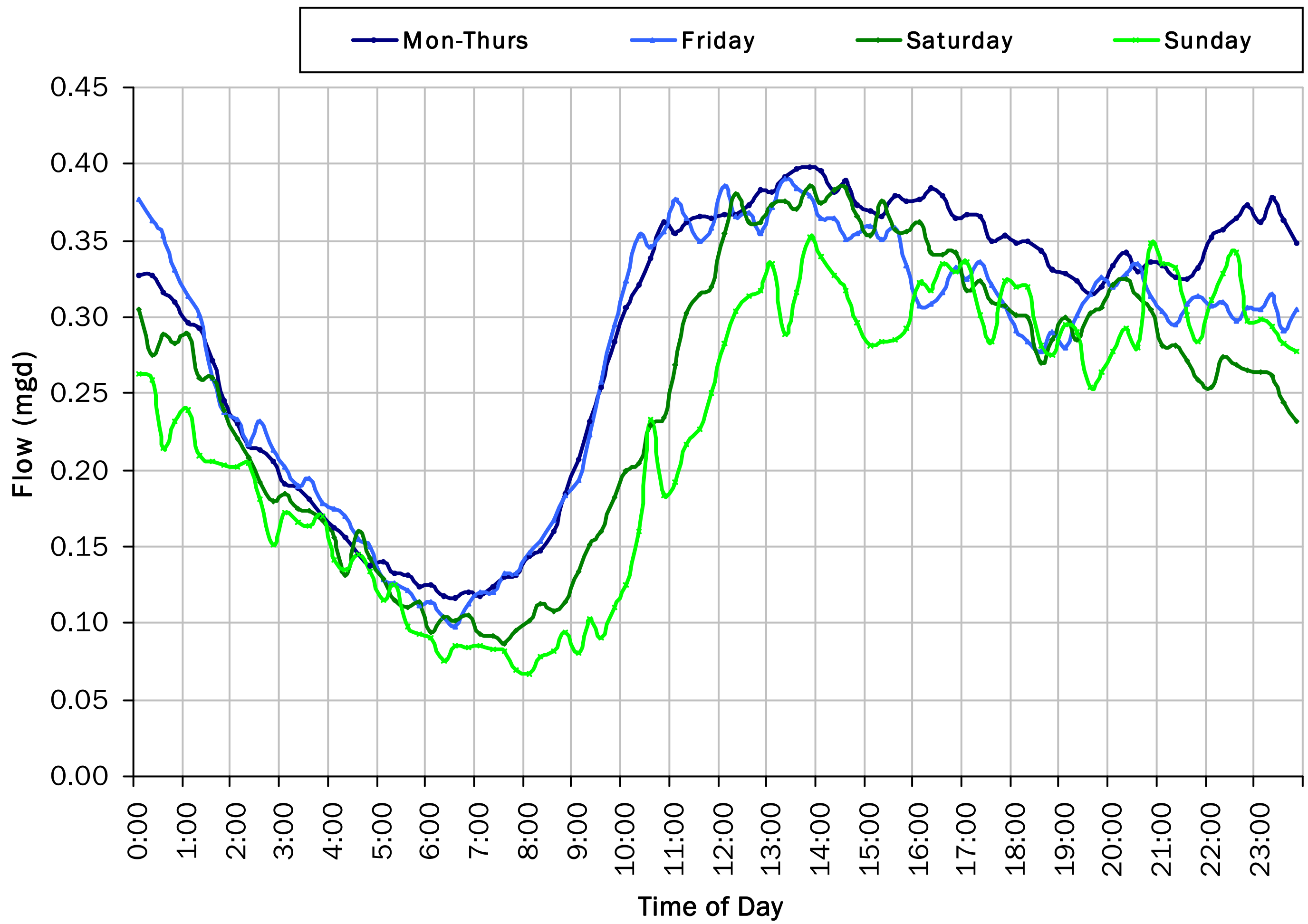
Total Period Rainfall: 2.08 inches   **Avg Flow: 0.276 mgd**   **Peak Flow: 0.541 mgd**   **Min Flow: 0.057 mgd**





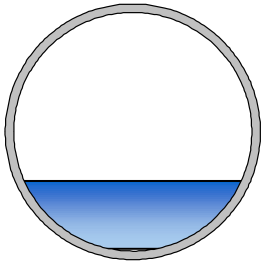
FM 1

Average Dry Weather Flow Hydrographs - Pre Shelter-In-Place (< 3/14/20)



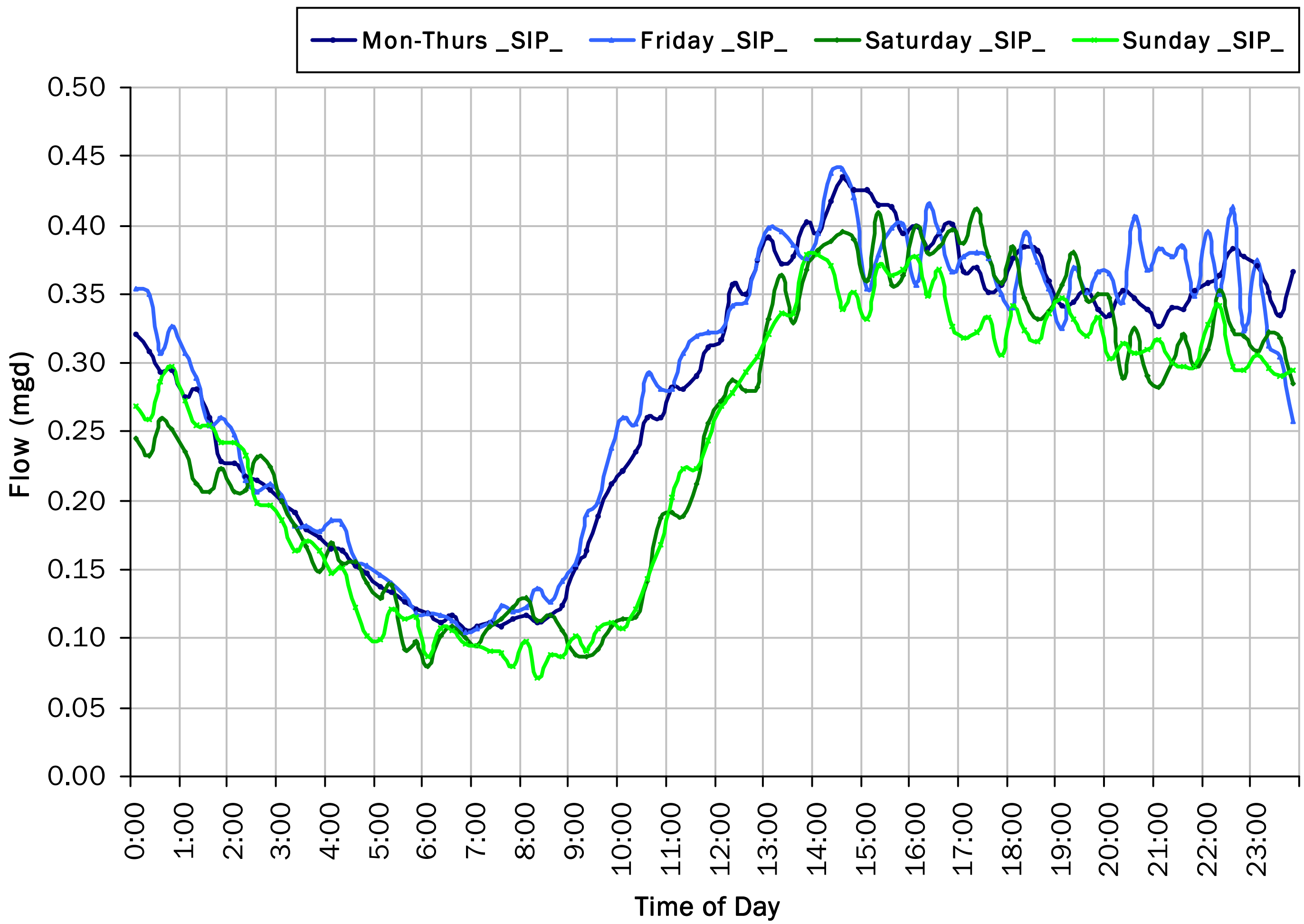
ADWF:

0.273 mgd



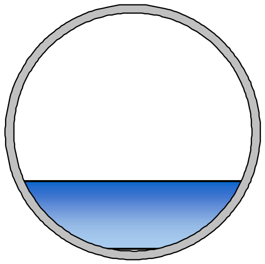
FM 1

Average Dry Weather Flow Hydrographs - Shelter In Place (> 3/14/20)



ADWF:

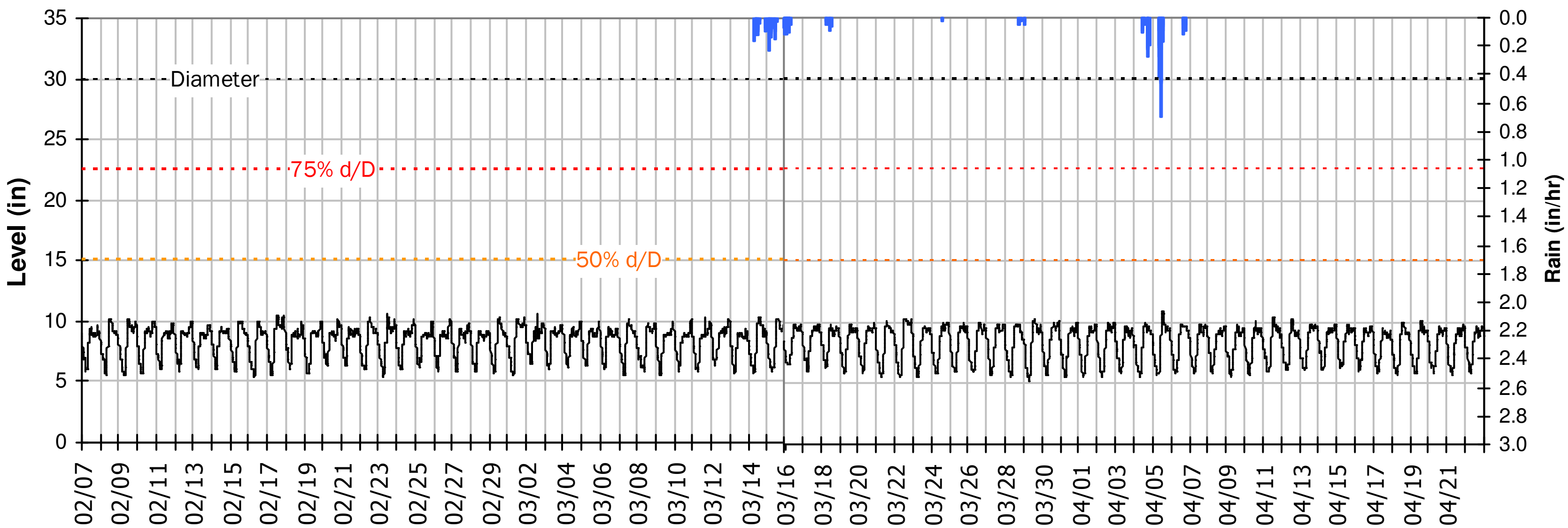
0.273 mgd

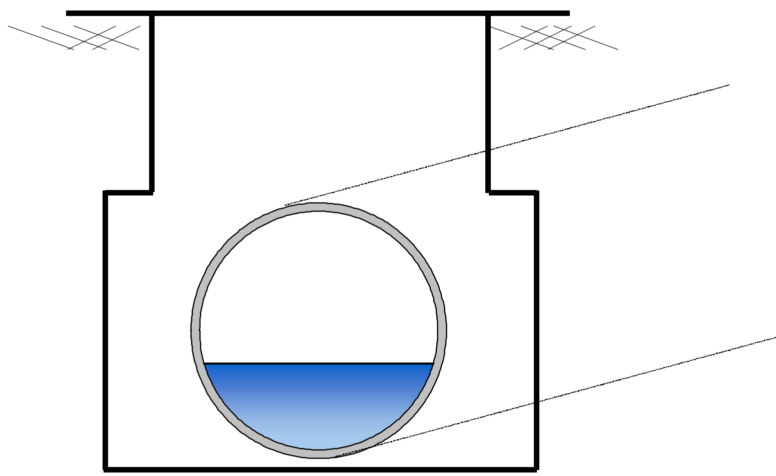


FM 1

Site Capacity and Surcharge Summary

Realtime Flow Levels with Rainfall Data over Monitoring Period

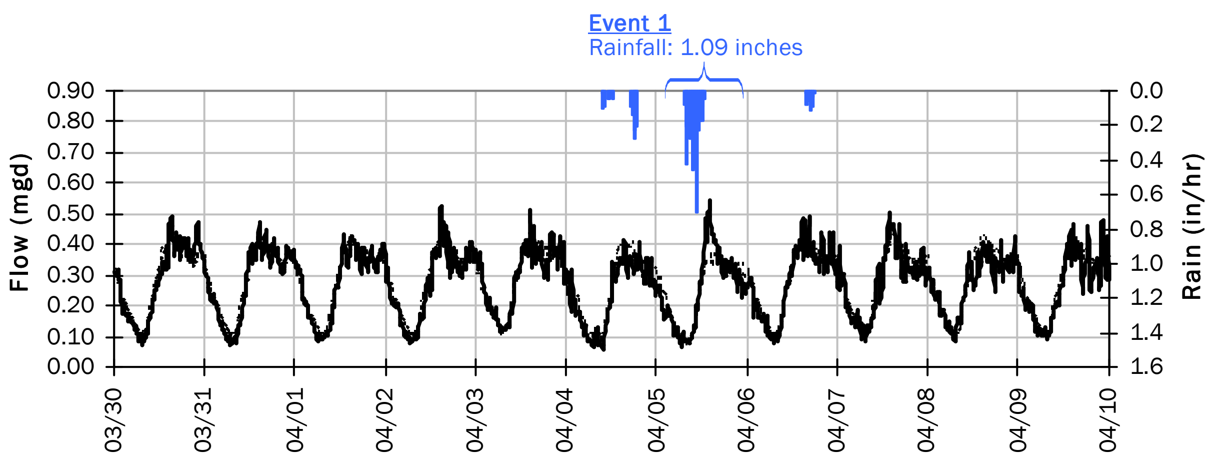




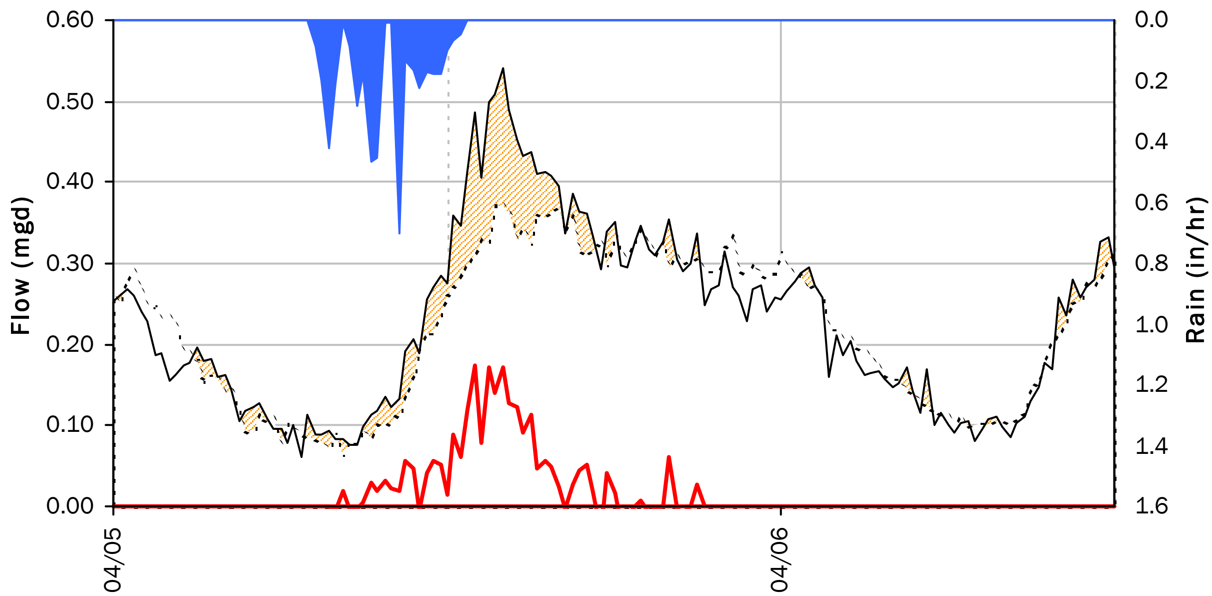
Pipe Diameter:	30	*inches*
Peak Measured Level:	10.9	*inches*
Peak d/D Ratio:	0.36	

FM 1
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



Event 1 Detail Graph



Storm Event I/I Analysis (Rain = 1.09 inches)

Capacity

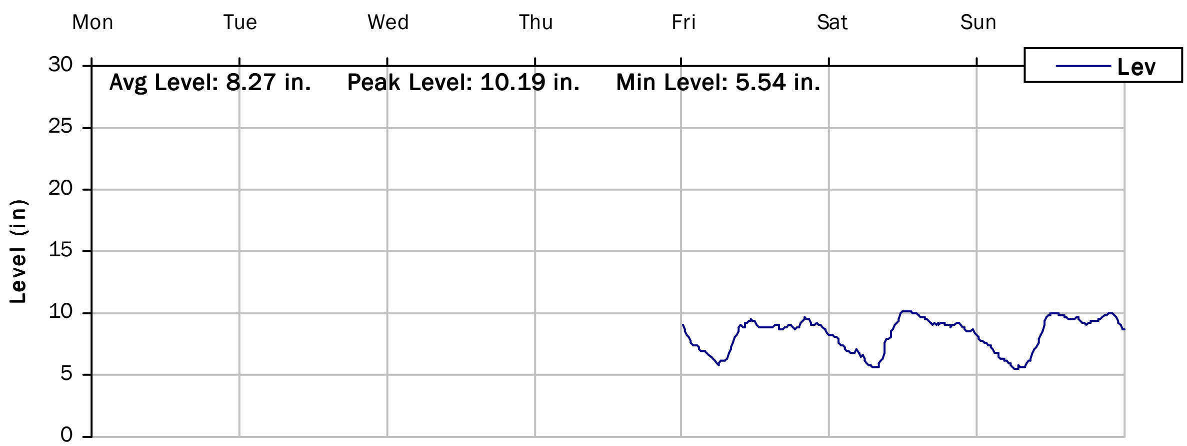
Peak Flow: 0.54 *mgd*
PF: 1.99

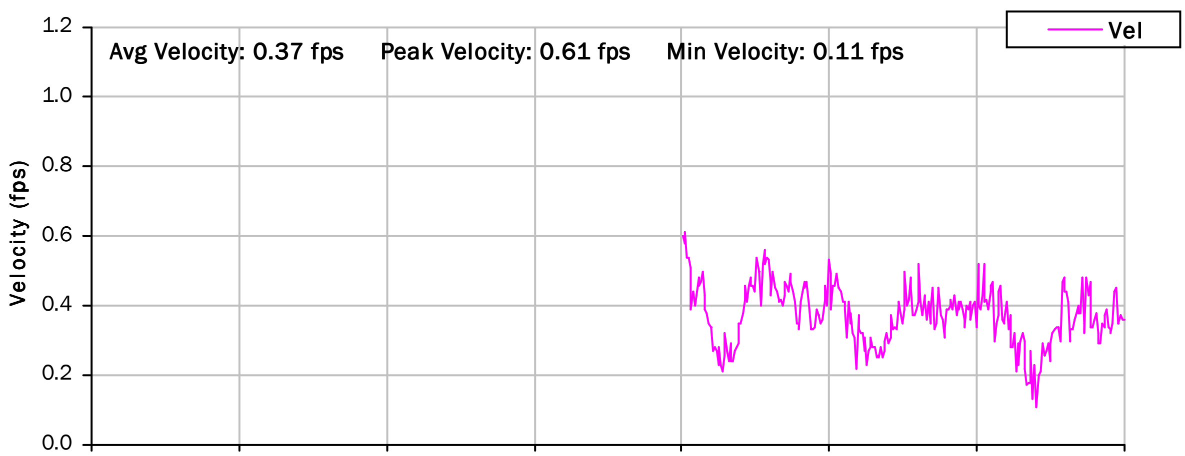
Peak Level: 10.93 *in*
d/D Ratio: 0.36

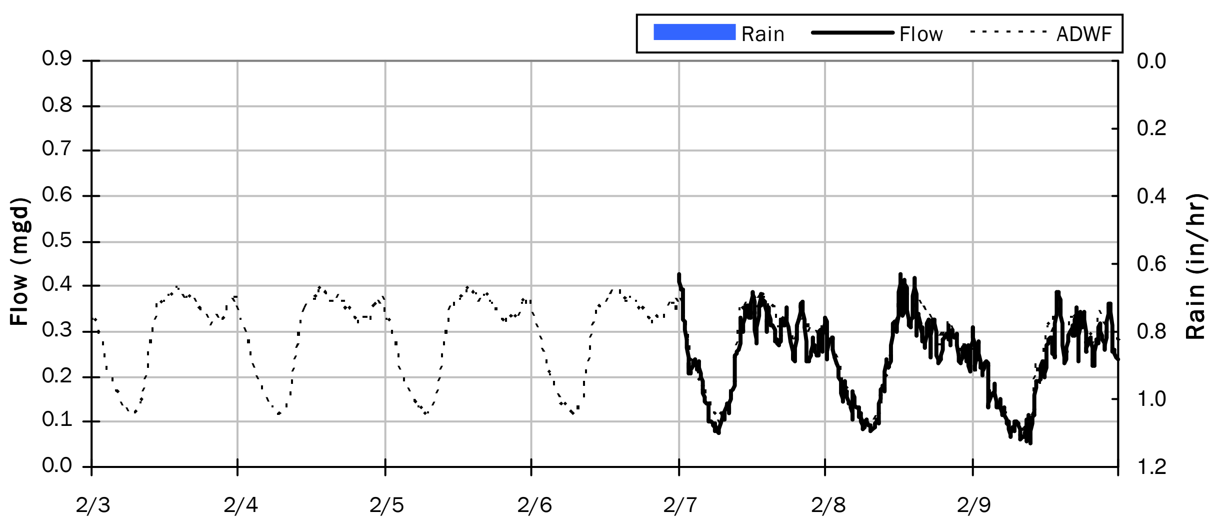
Inflow / Infiltration

Peak I/I Rate: 0.174 *mgd*
Total I/I: 23,000 *gallons*

FM 1
Weekly Level, Velocity and Flow Hydrographs
2/3/2020 to 2/10/2020

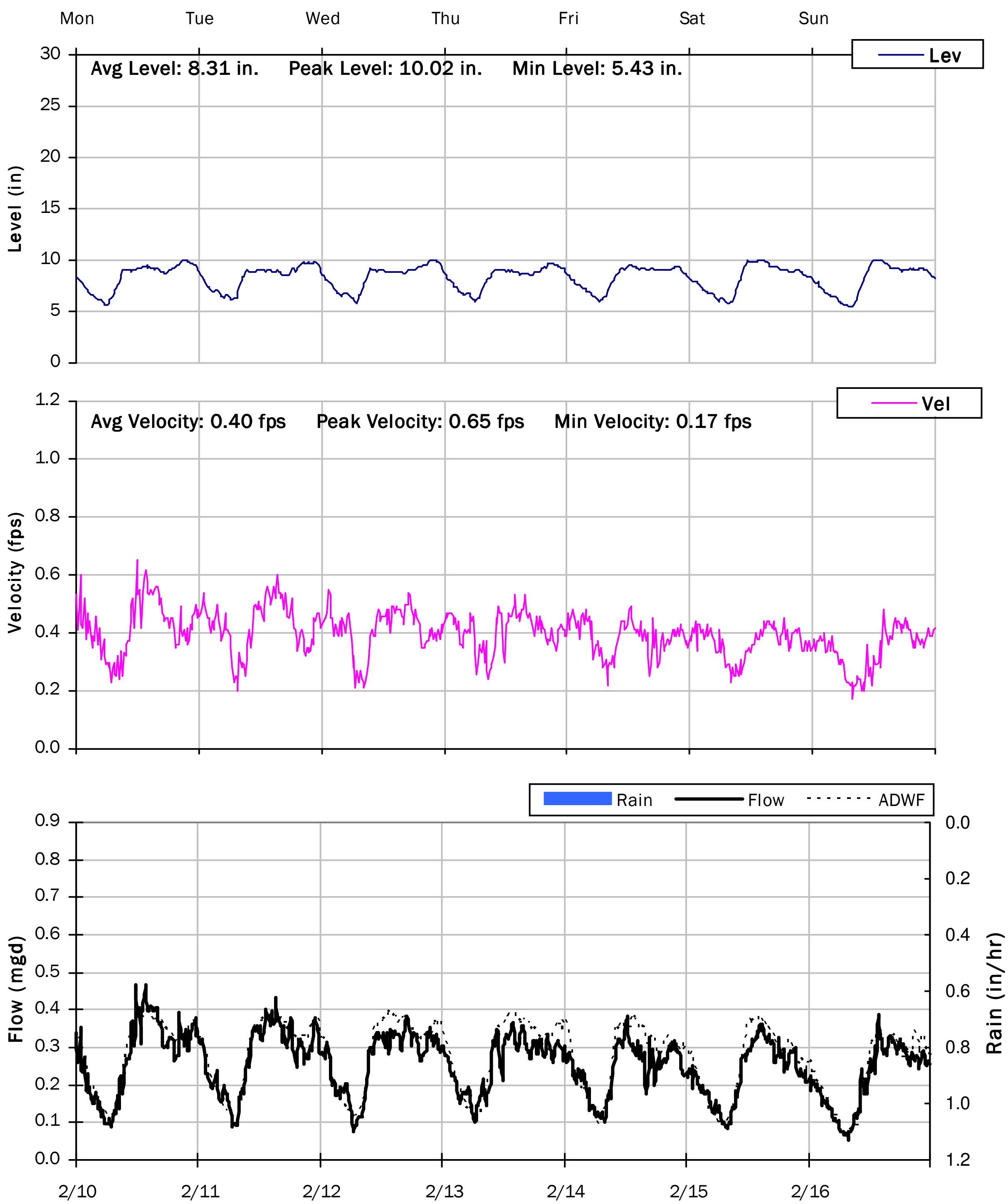






Avg Flow: 0.238 mgd Peak Flow: 0.428 mgd Min Flow: 0.054 mgd

FM 1
Weekly Level, Velocity and Flow Hydrographs
2/10/2020 to 2/17/2020

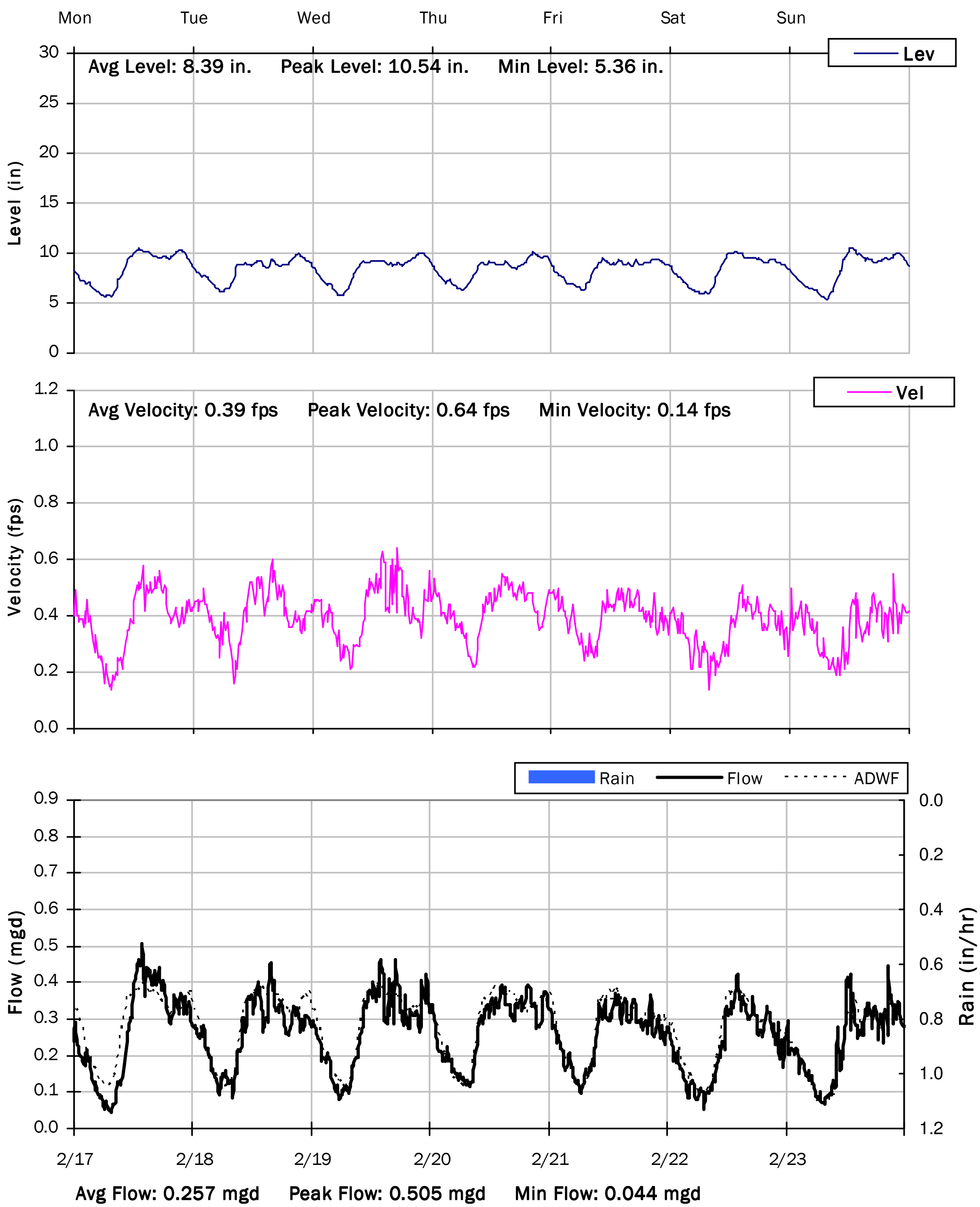


Avg Flow: 0.252 mgd **Peak Flow: 0.468 mgd** **Min Flow: 0.054 mgd**

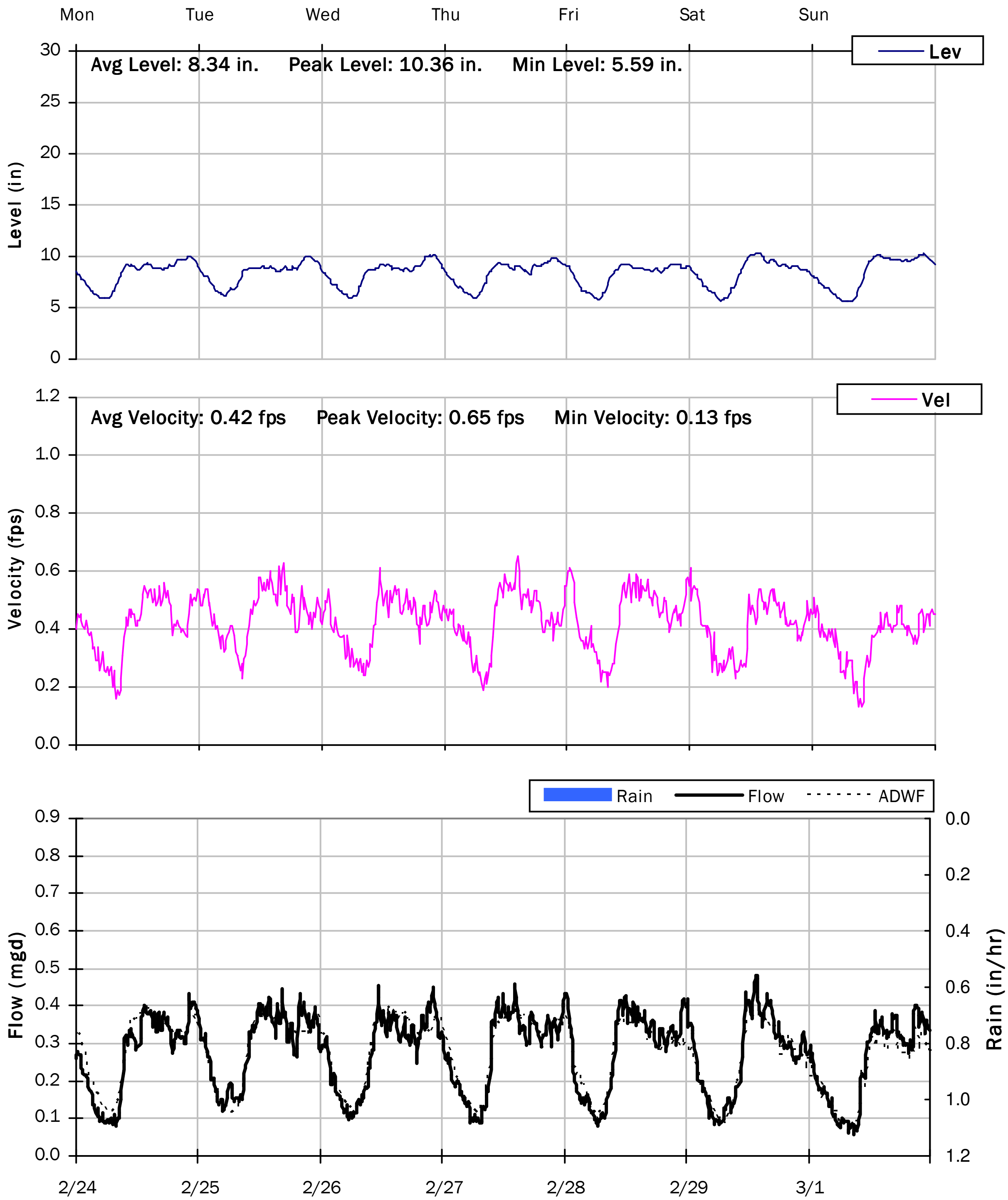
FM 1

Weekly Level, Velocity and Flow Hydrographs

2/17/2020 to 2/24/2020

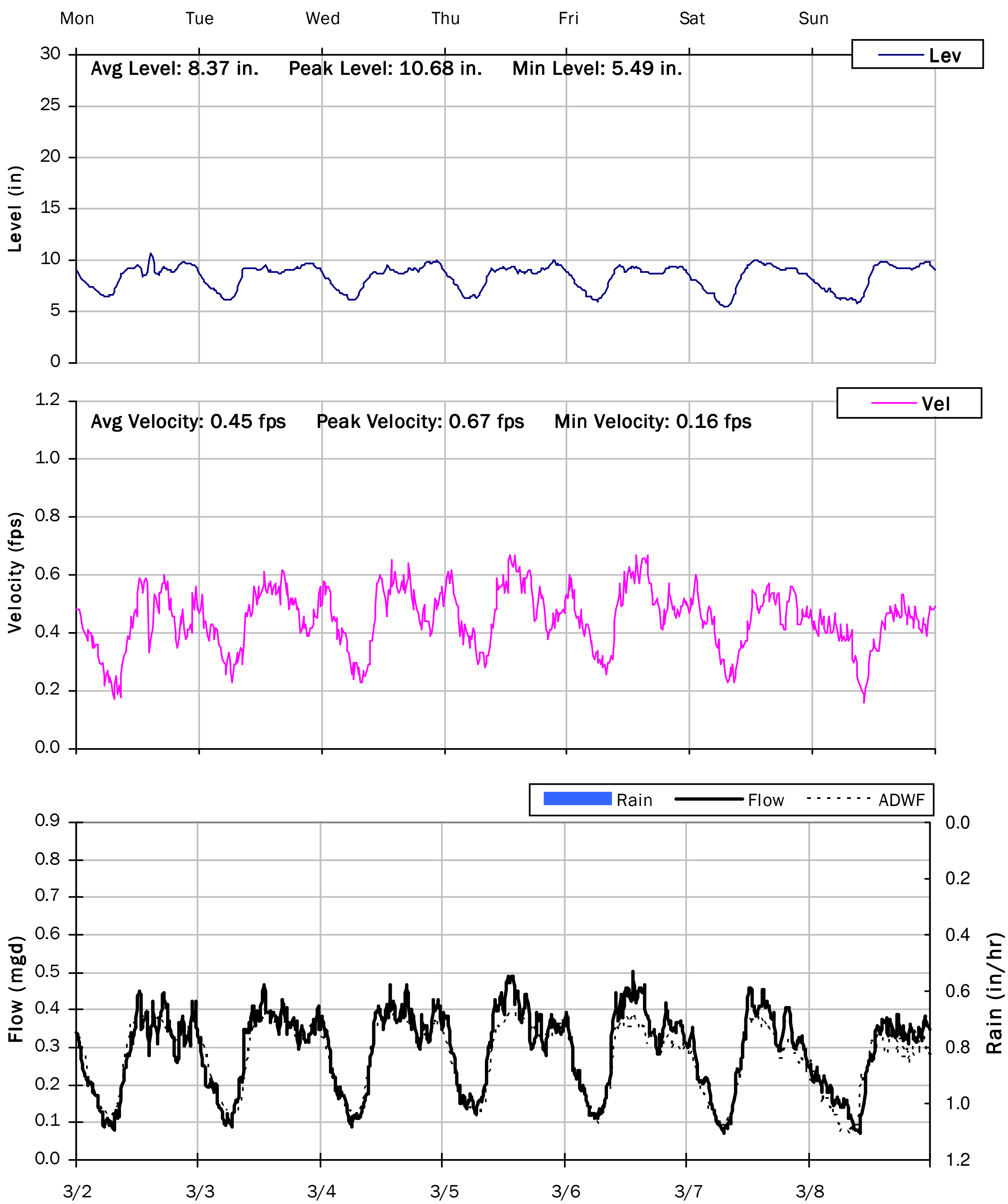


FM 1
Weekly Level, Velocity and Flow Hydrographs
2/24/2020 to 3/2/2020



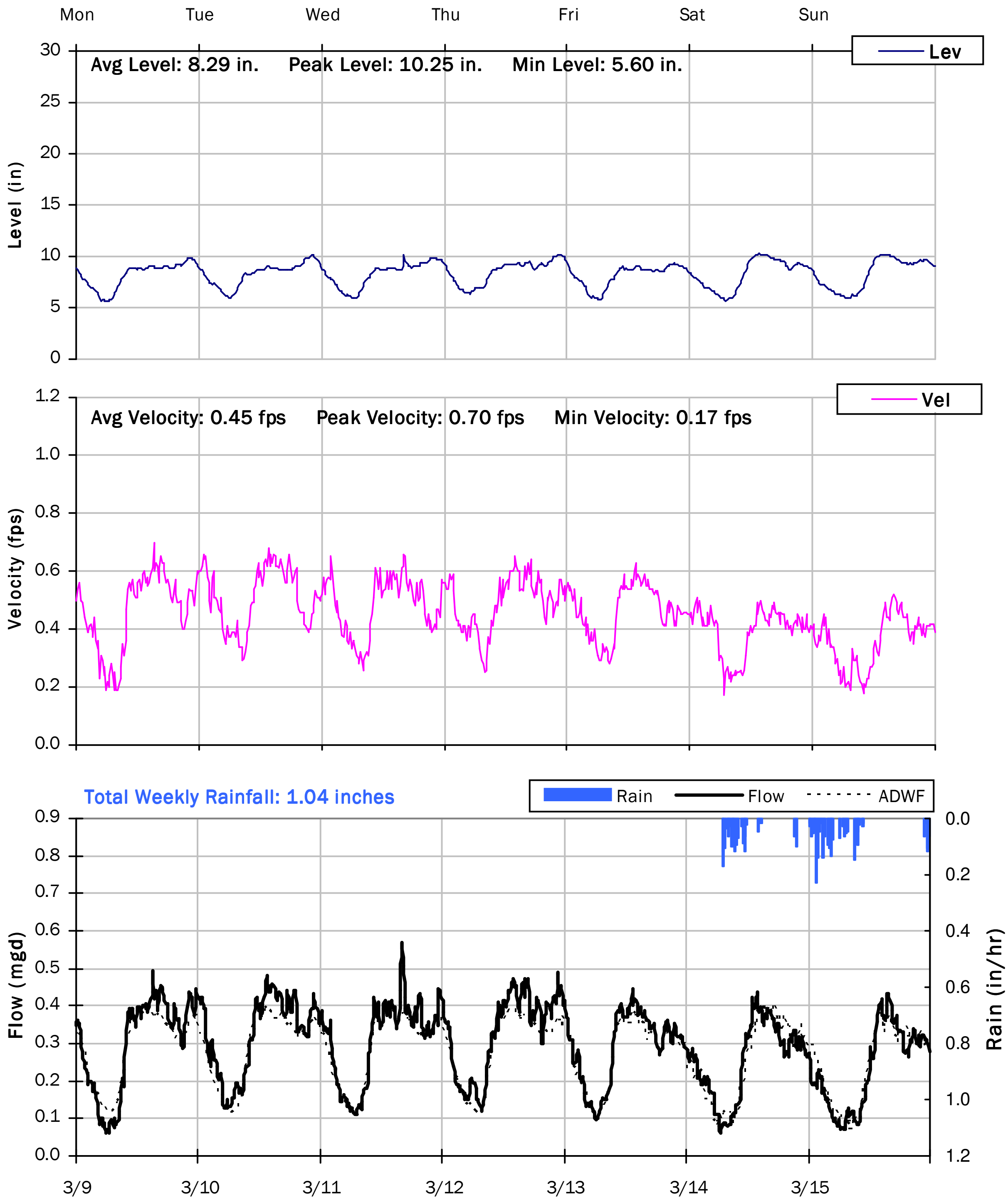
Avg Flow: 0.275 mgd **Peak Flow: 0.480 mgd** **Min Flow: 0.059 mgd**

FM 1
Weekly Level, Velocity and Flow Hydrographs
3/2/2020 to 3/9/2020



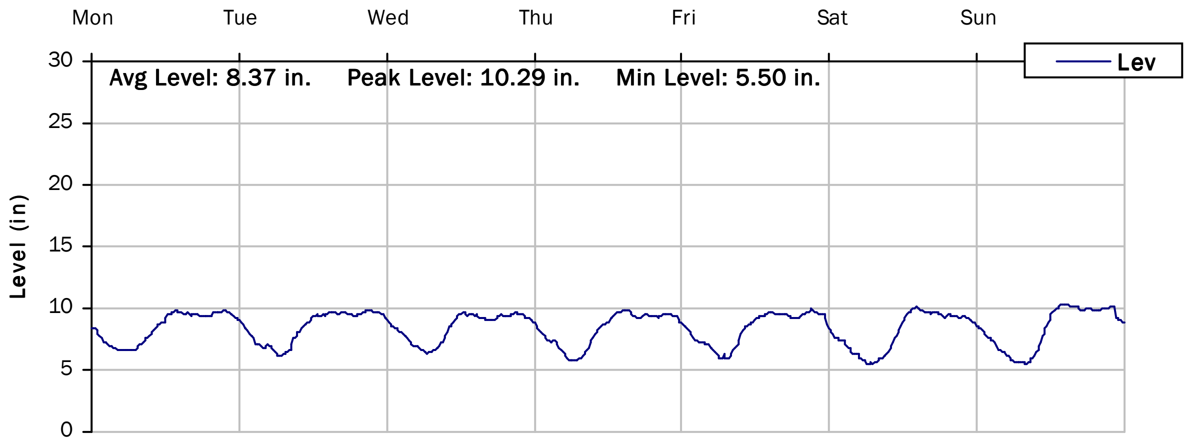
Avg Flow: 0.294 mgd **Peak Flow: 0.503 mgd** **Min Flow: 0.070 mgd**

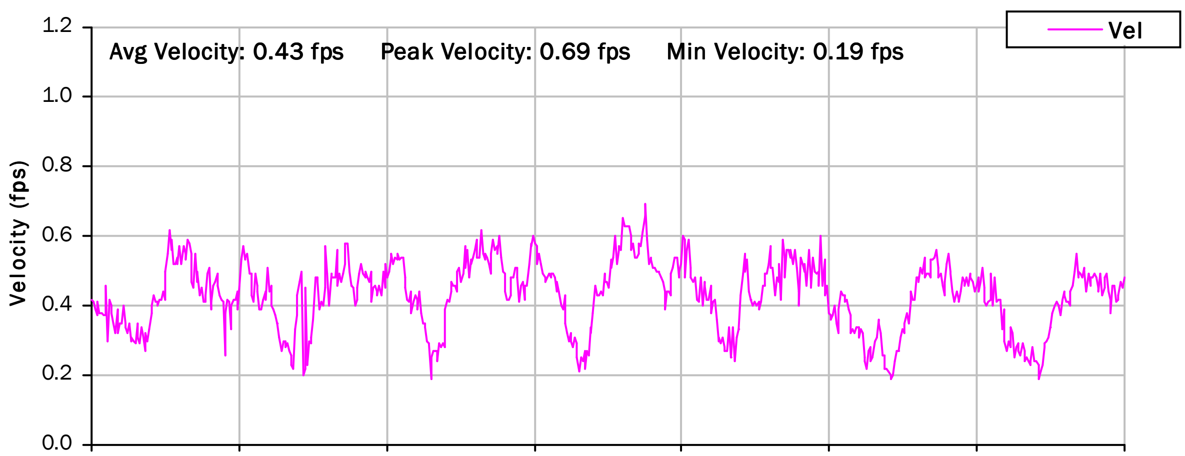
FM 1
Weekly Level, Velocity and Flow Hydrographs
3/9/2020 to 3/16/2020

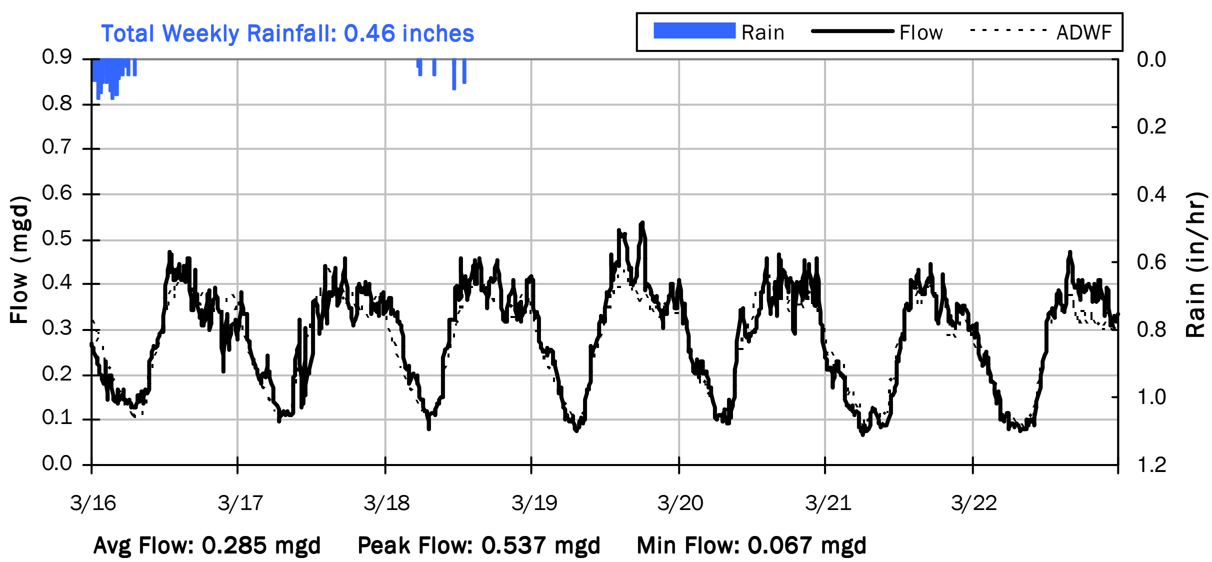


Avg Flow: 0.293 mgd Peak Flow: 0.569 mgd Min Flow: 0.060 mgd

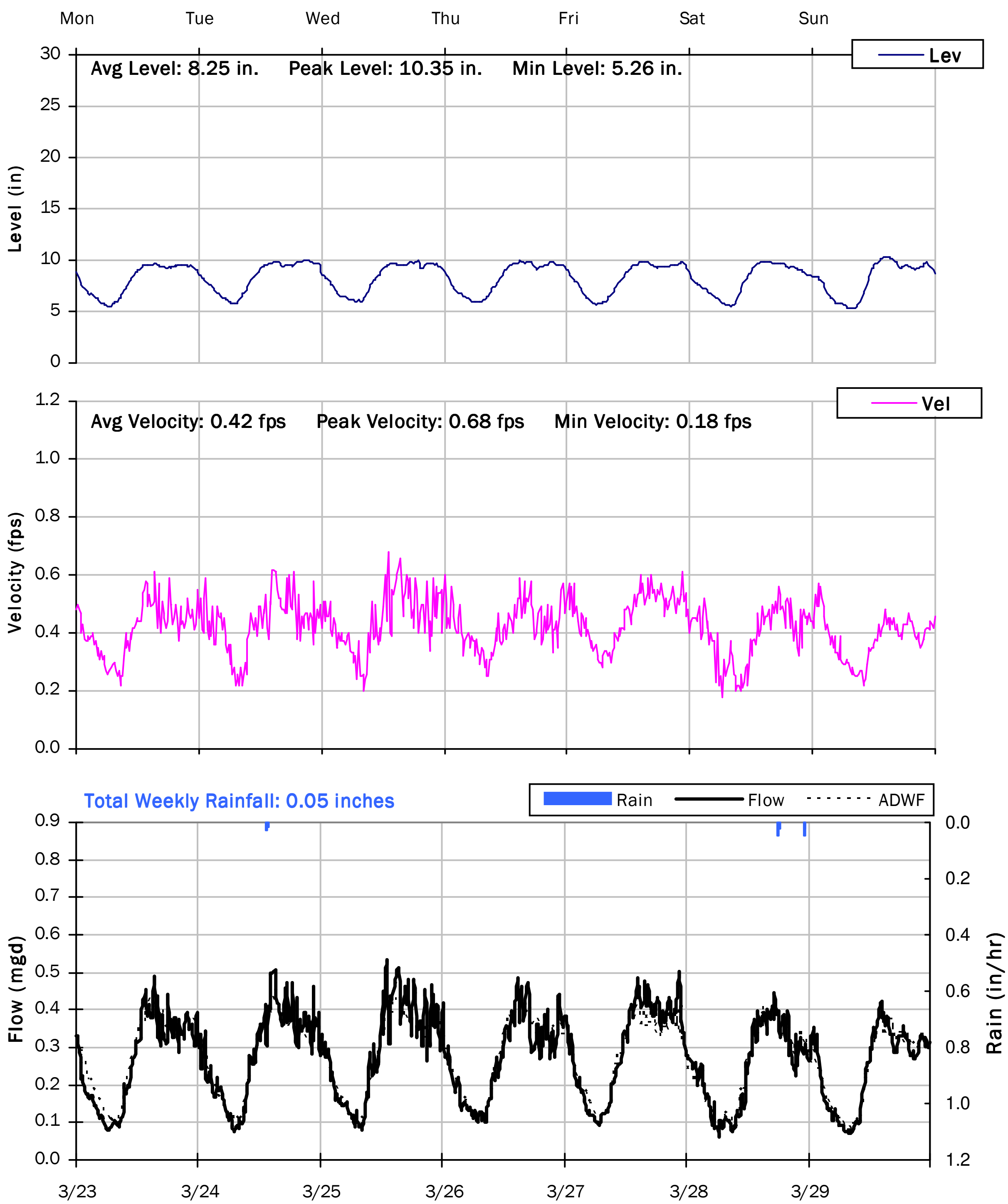
FM 1
Weekly Level, Velocity and Flow Hydrographs
3/16/2020 to 3/23/2020







FM 1
Weekly Level, Velocity and Flow Hydrographs
3/23/2020 to 3/30/2020

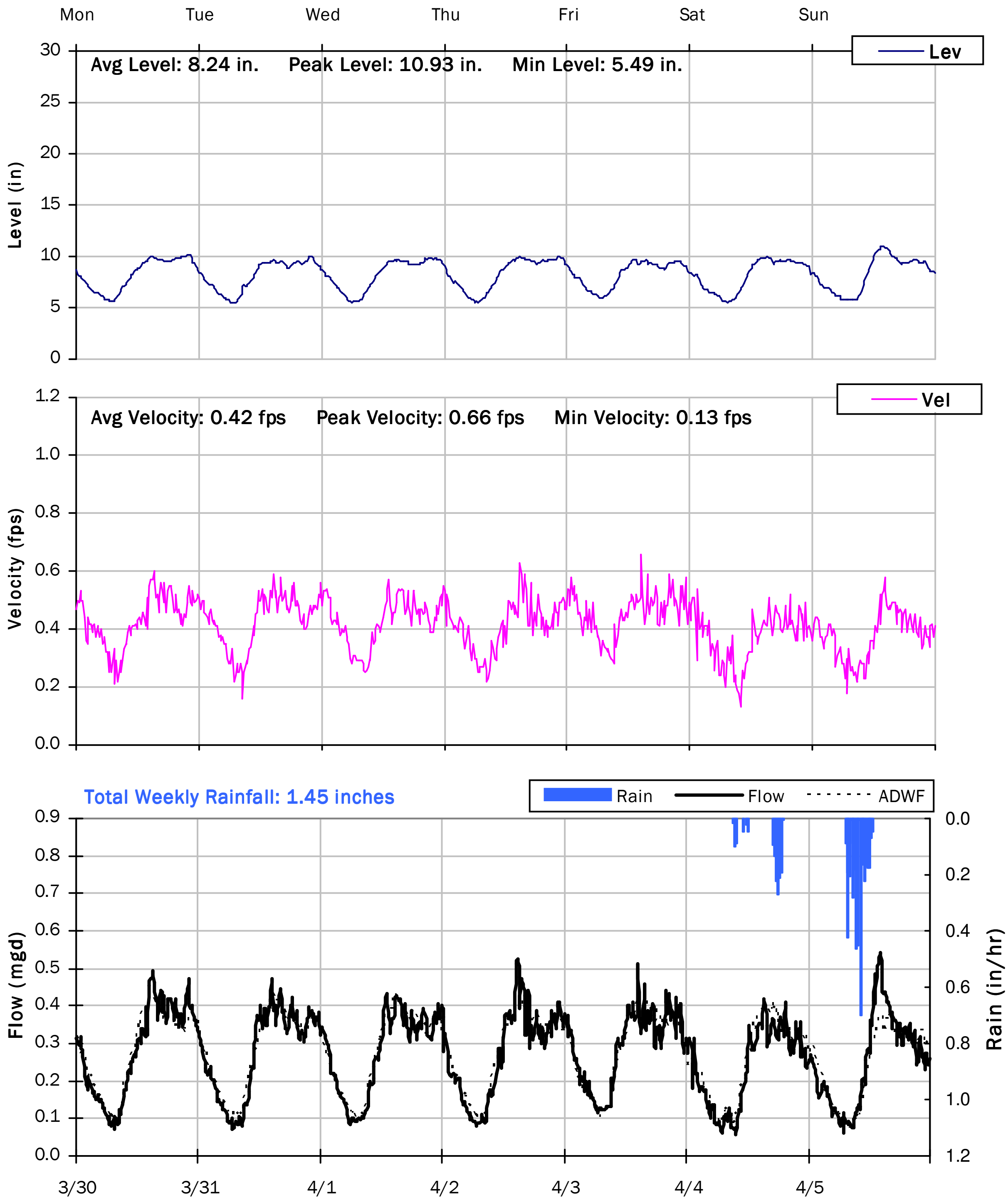


Avg Flow: 0.274 mgd **Peak Flow: 0.528 mgd** **Min Flow: 0.063 mgd**

FM 1

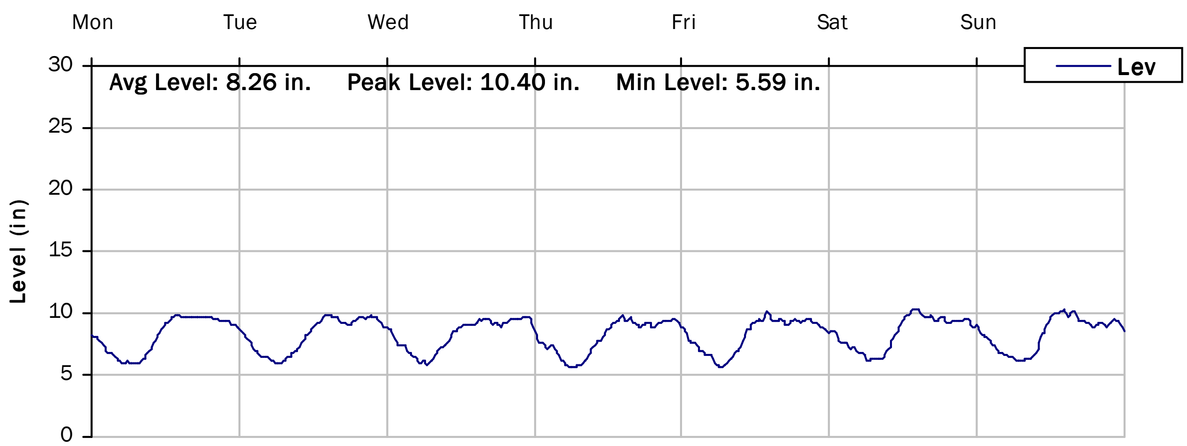
Weekly Level, Velocity and Flow Hydrographs

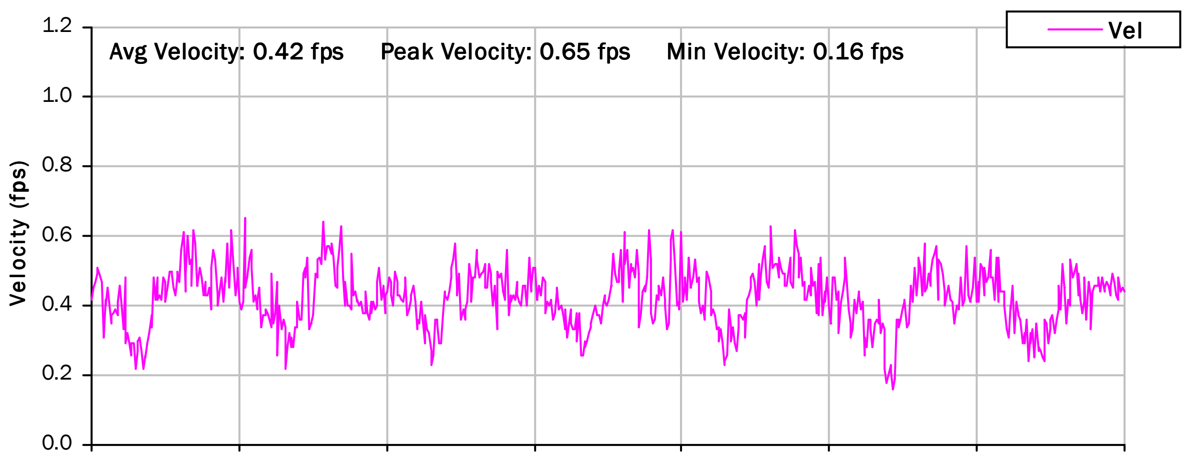
3/30/2020 to 4/6/2020

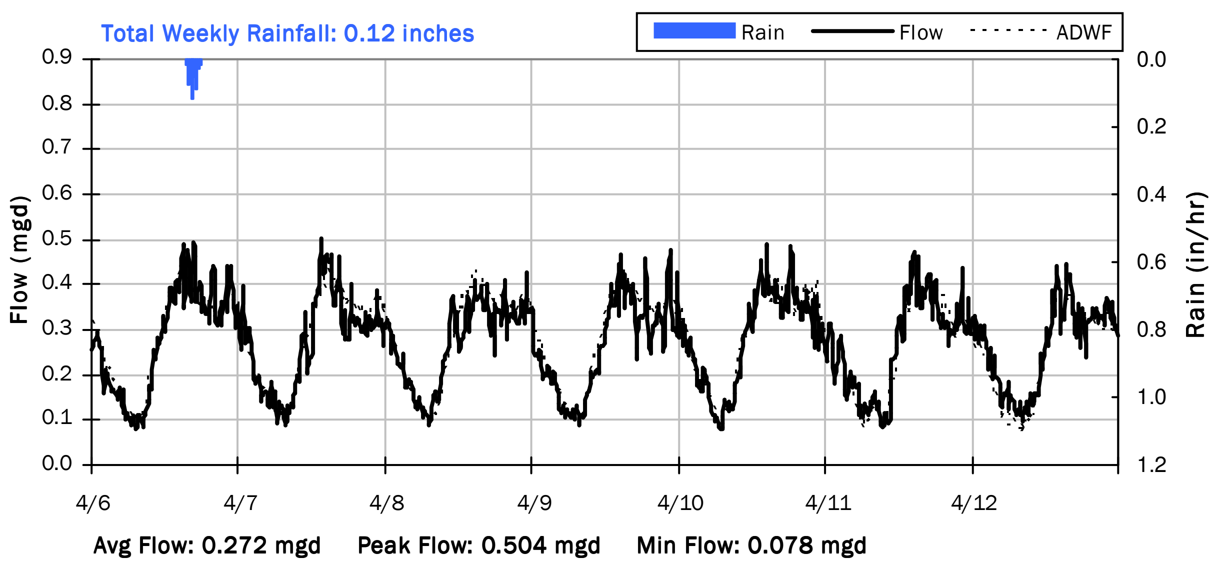


Avg Flow: 0.270 mgd **Peak Flow: 0.541 mgd** **Min Flow: 0.058 mgd**

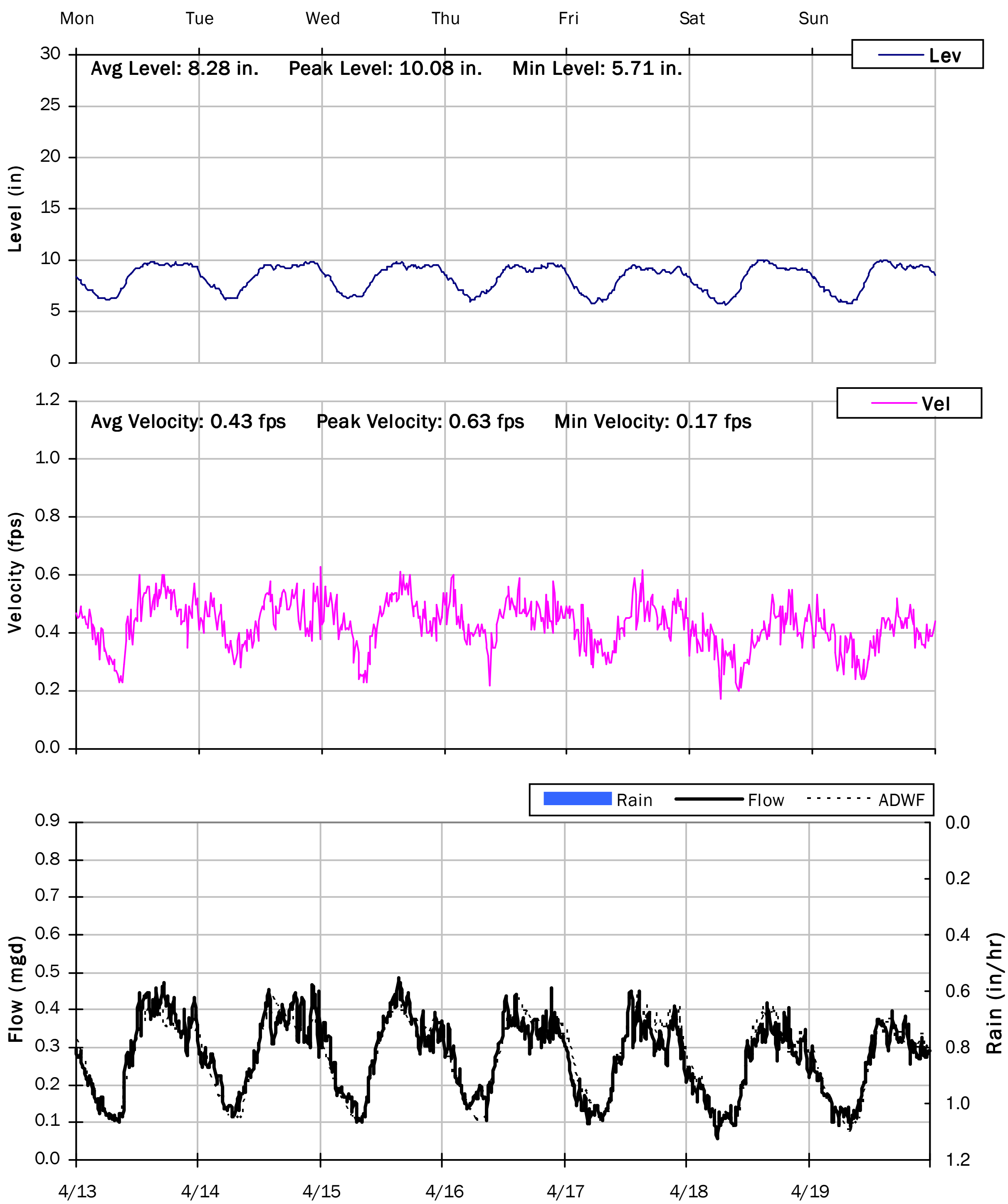
FM 1
Weekly Level, Velocity and Flow Hydrographs
4/6/2020 to 4/13/2020





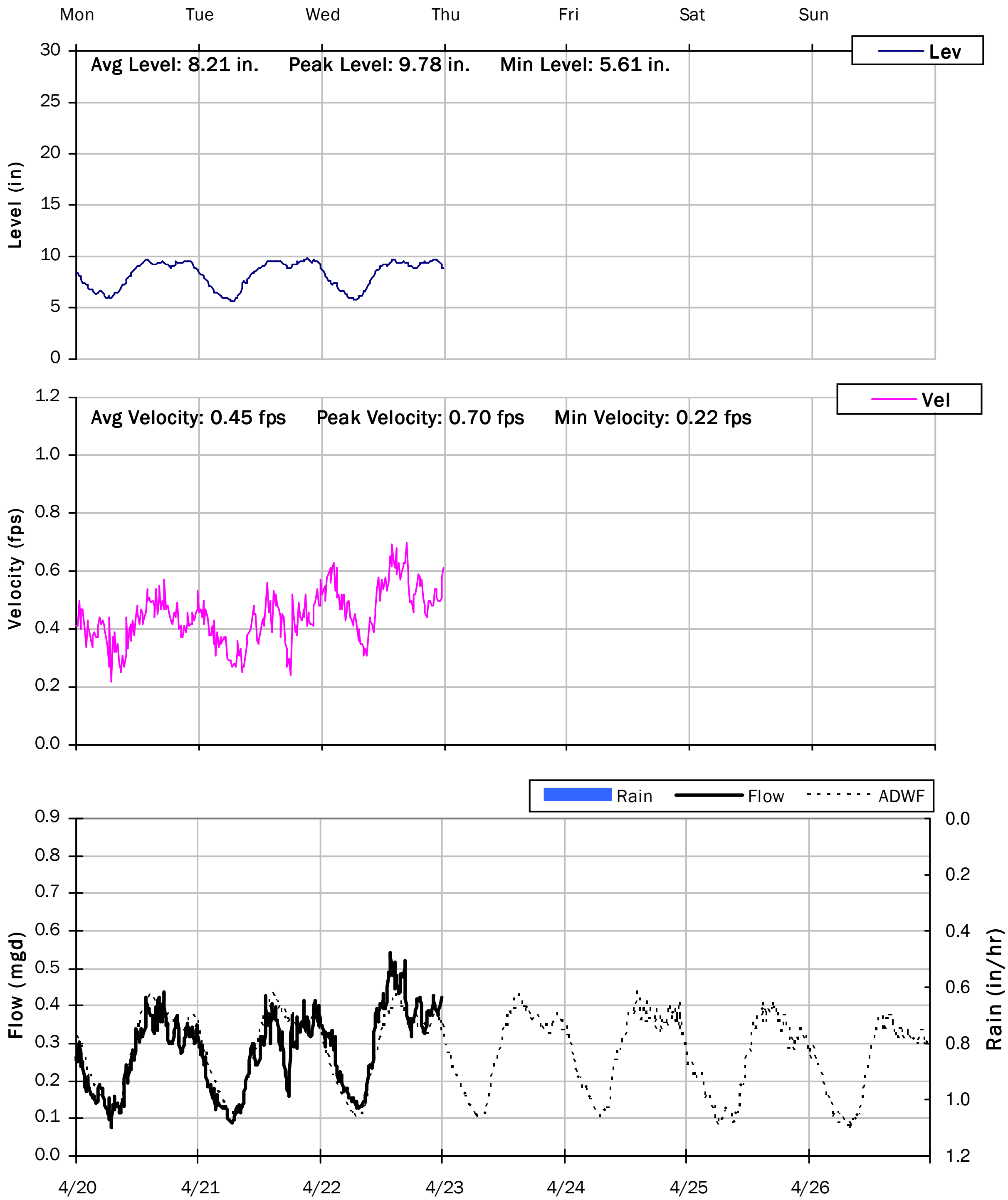


FM 1
Weekly Level, Velocity and Flow Hydrographs
4/13/2020 to 4/20/2020



Avg Flow: 0.275 mgd Peak Flow: 0.485 mgd Min Flow: 0.057 mgd

FM 1
Weekly Level, Velocity and Flow Hydrographs
4/20/2020 to 4/27/2020



Avg Flow: 0.282 mgd **Peak Flow: 0.541 mgd** **Min Flow: 0.077 mgd**

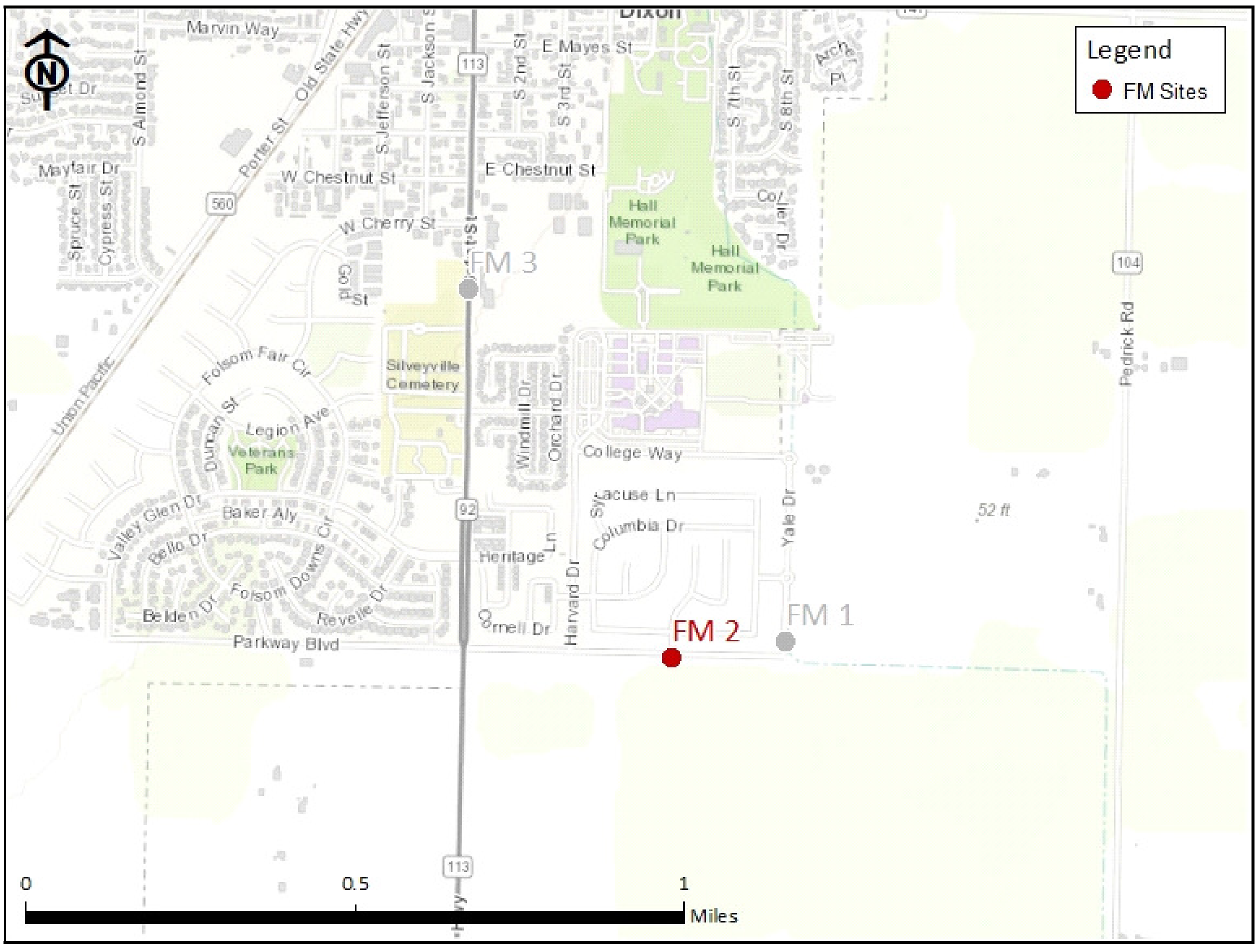
City of Dixon

Sanitary Sewer Flow Monitoring
February 06 - April 23, 2020

Monitoring Site: FM 2

Location: Parkway Blvd, 900' west of Yale Drive

Data Summary Report

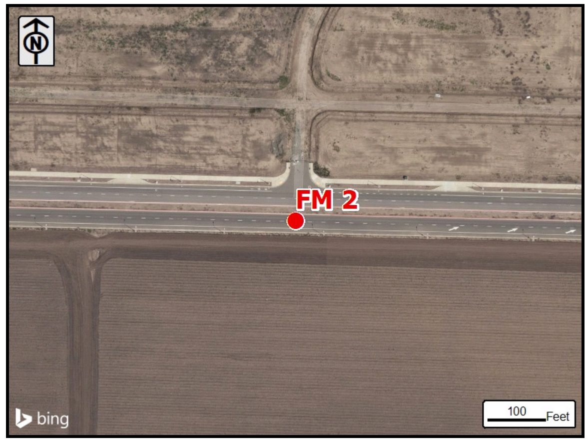


Vicinity Map: FM 2

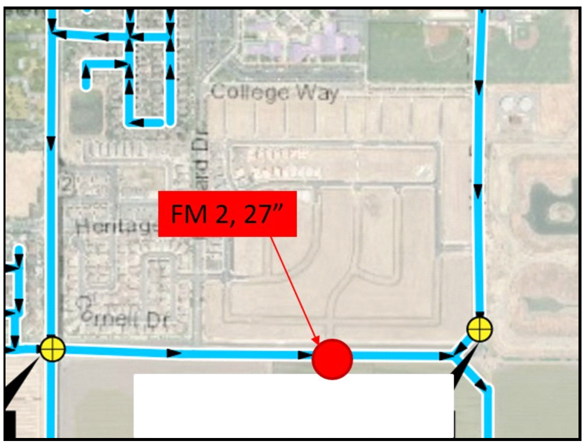
FM 2

Site Information

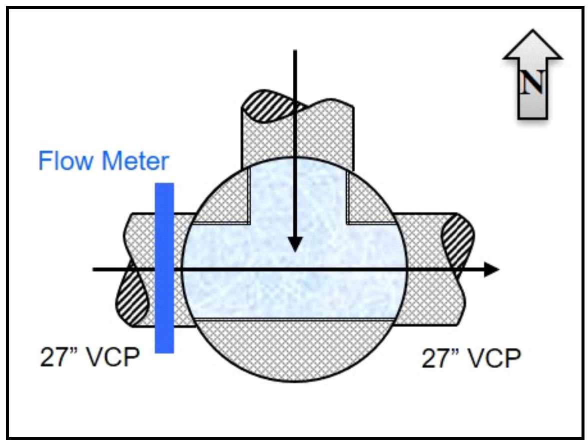
Location:	Parkway Blvd, 900' west of Yale Drive
City Manhole:	FM 2
Coordinates:	121.8167° W, 38.4311° N
Rim Elevation (Earth):	51 feet
Pipe Diameter:	27 inches
ADWF:	0.810 mgd
Peak Measured Flow:	1.560 mgd



Satellite Map



Sanitary Map



Flow Sketch

Street View

Plan View

FM 2

Additional Site Photos

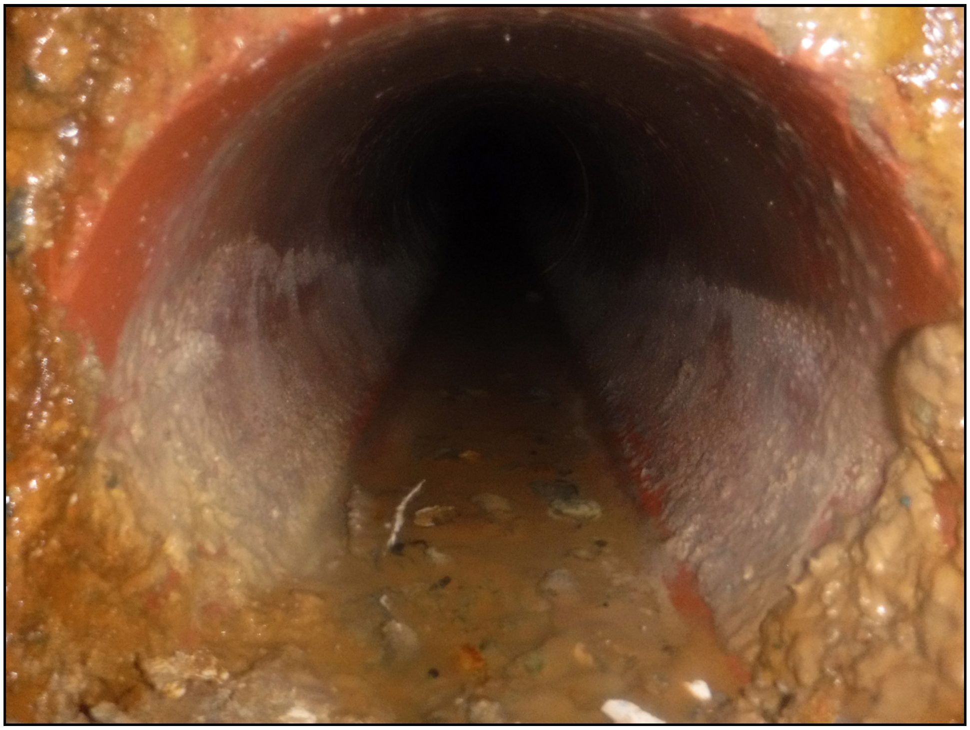
Effluent Pipe

Monitored Influent Pipe

FM 2

Additional Site Photos

North Lateral Pipe

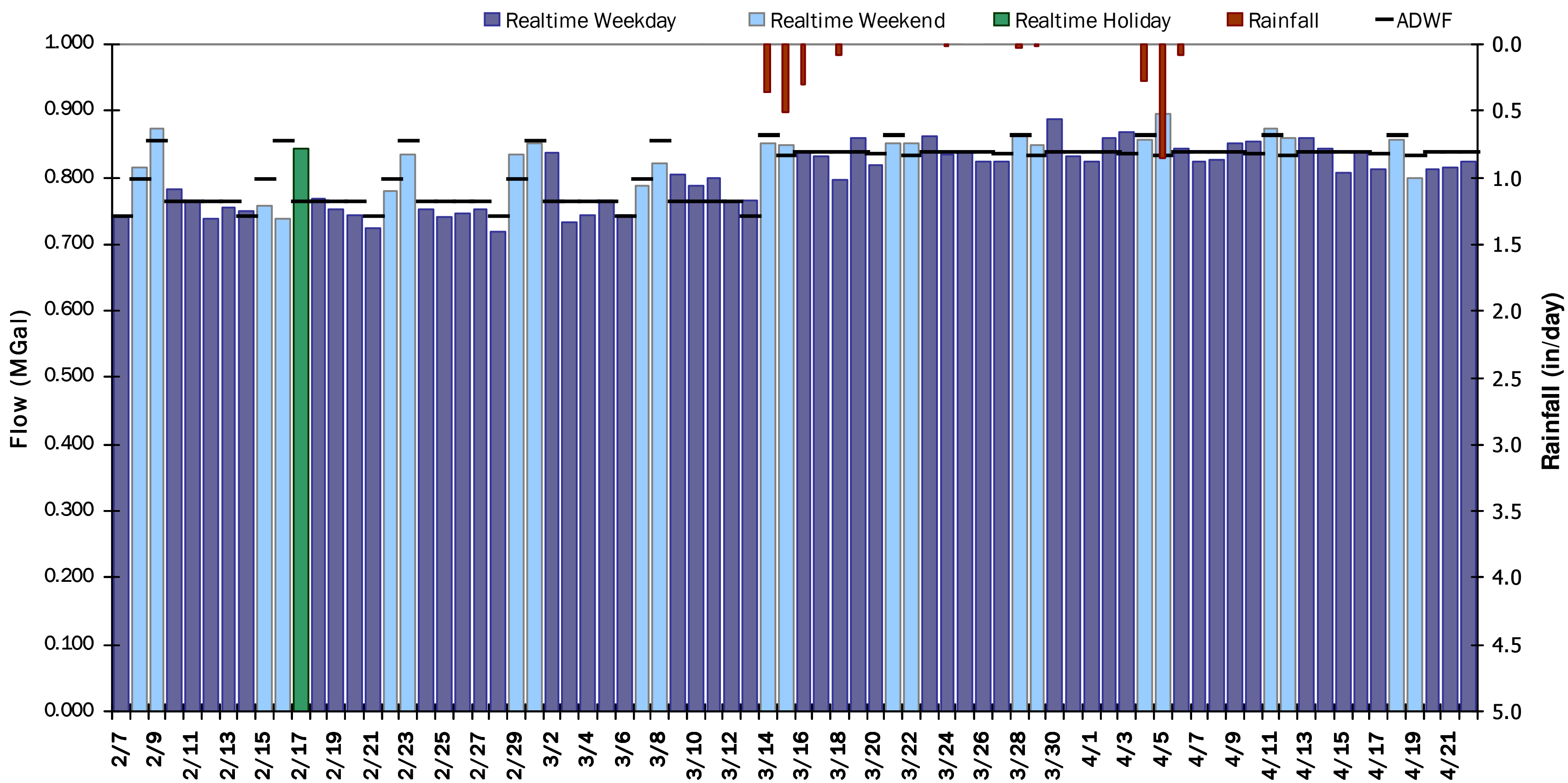


FM 2

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.810 MGal **Peak Daily Flow: 0.895 MGal** **Min Daily Flow: 0.719 MGal**

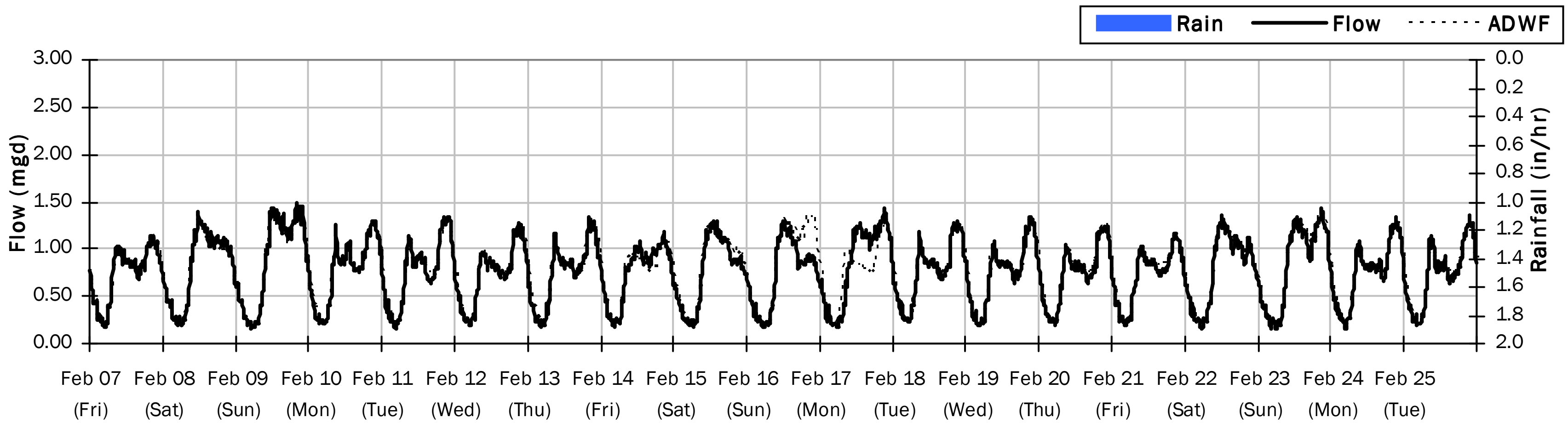
Total Period Rainfall: 2.52 inches

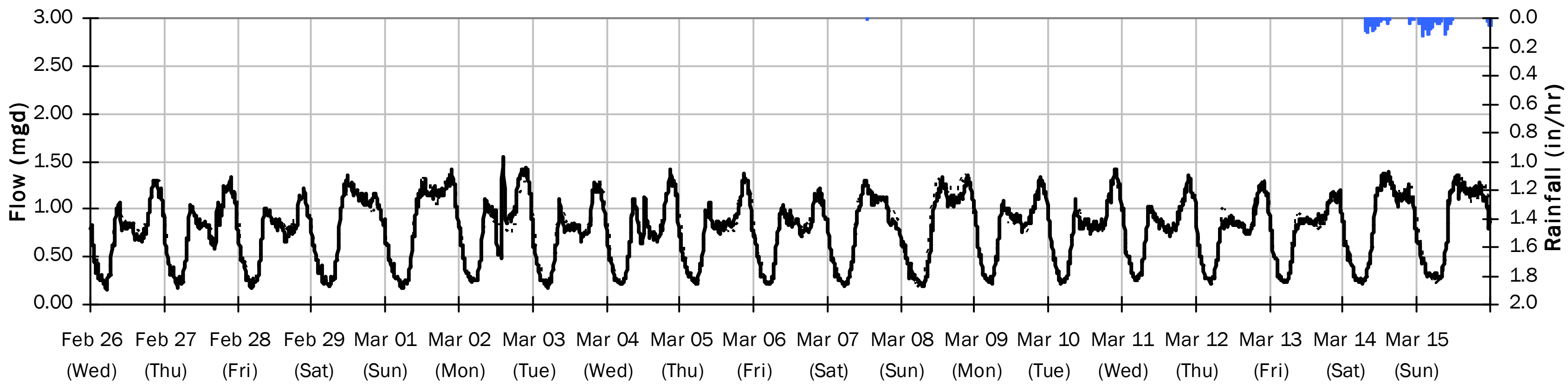


FM 2

Flow Summary: 2/7/2020 to 3/15/2020

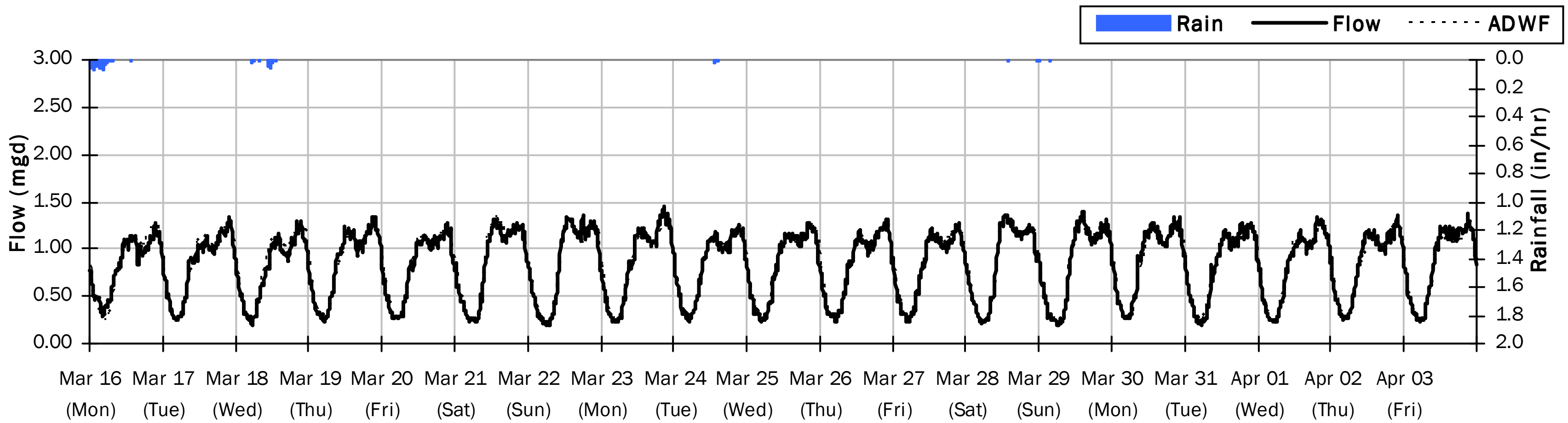
Total Period Rainfall: 0.87 inches   Avg Flow: 0.779 mgd   Peak Flow: 1.550 mgd   Min Flow: 0.156 mgd

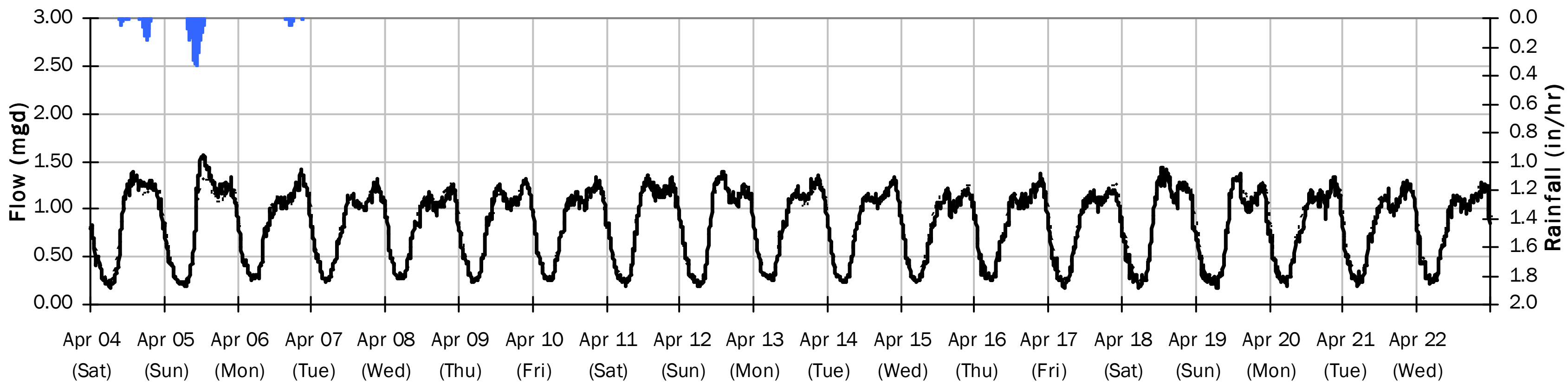




FM 2
Flow Summary: 3/16/2020 to 4/22/2020

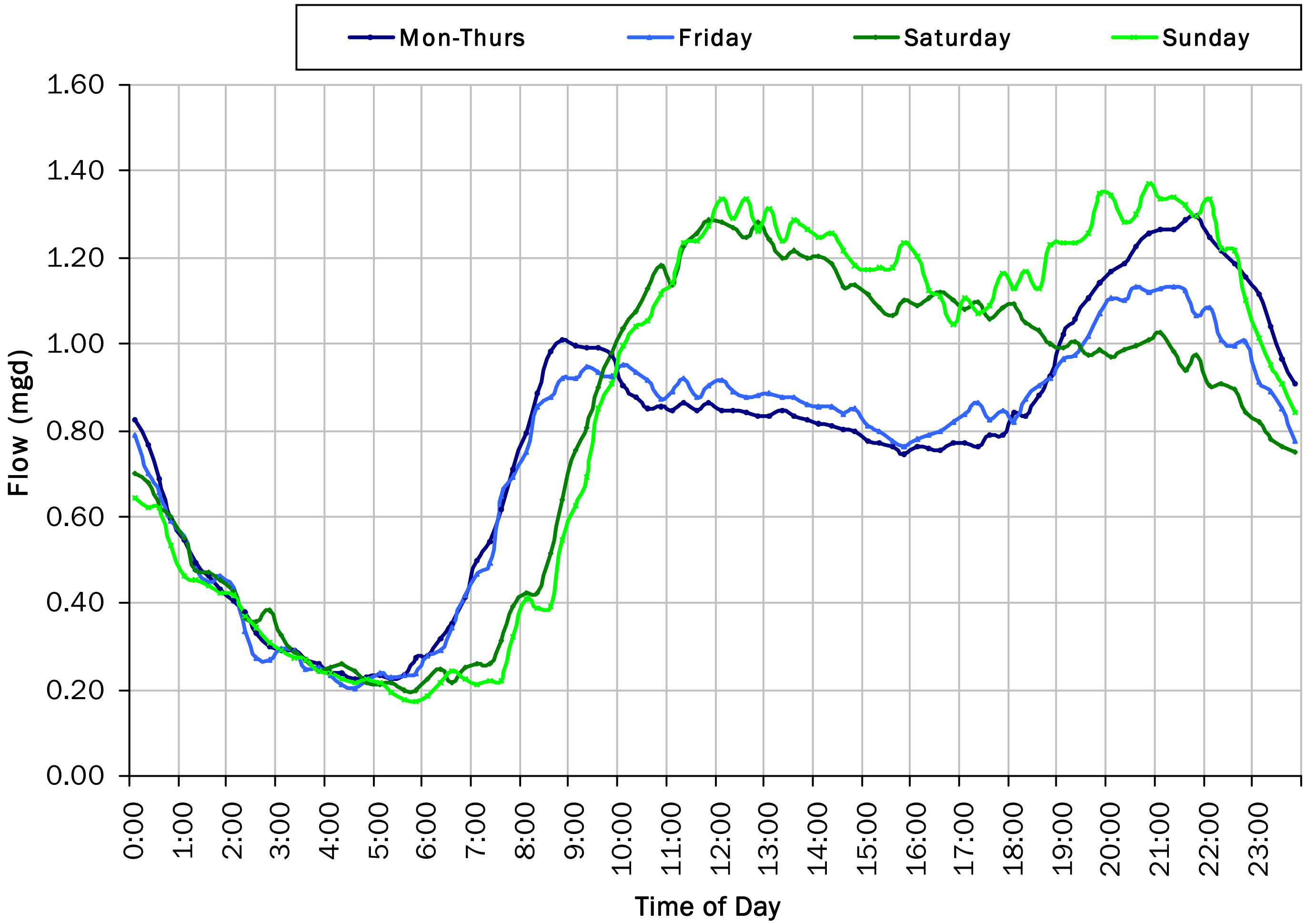
Total Period Rainfall: 1.65 inches   **Avg Flow: 0.841 mgd**   **Peak Flow: 1.560 mgd**   **Min Flow: 0.167 mgd**





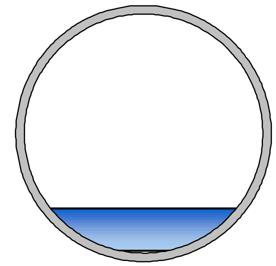
FM 2

Average Dry Weather Flow Hydrographs - Pre Shelter-In-Place (< 3/14/20)

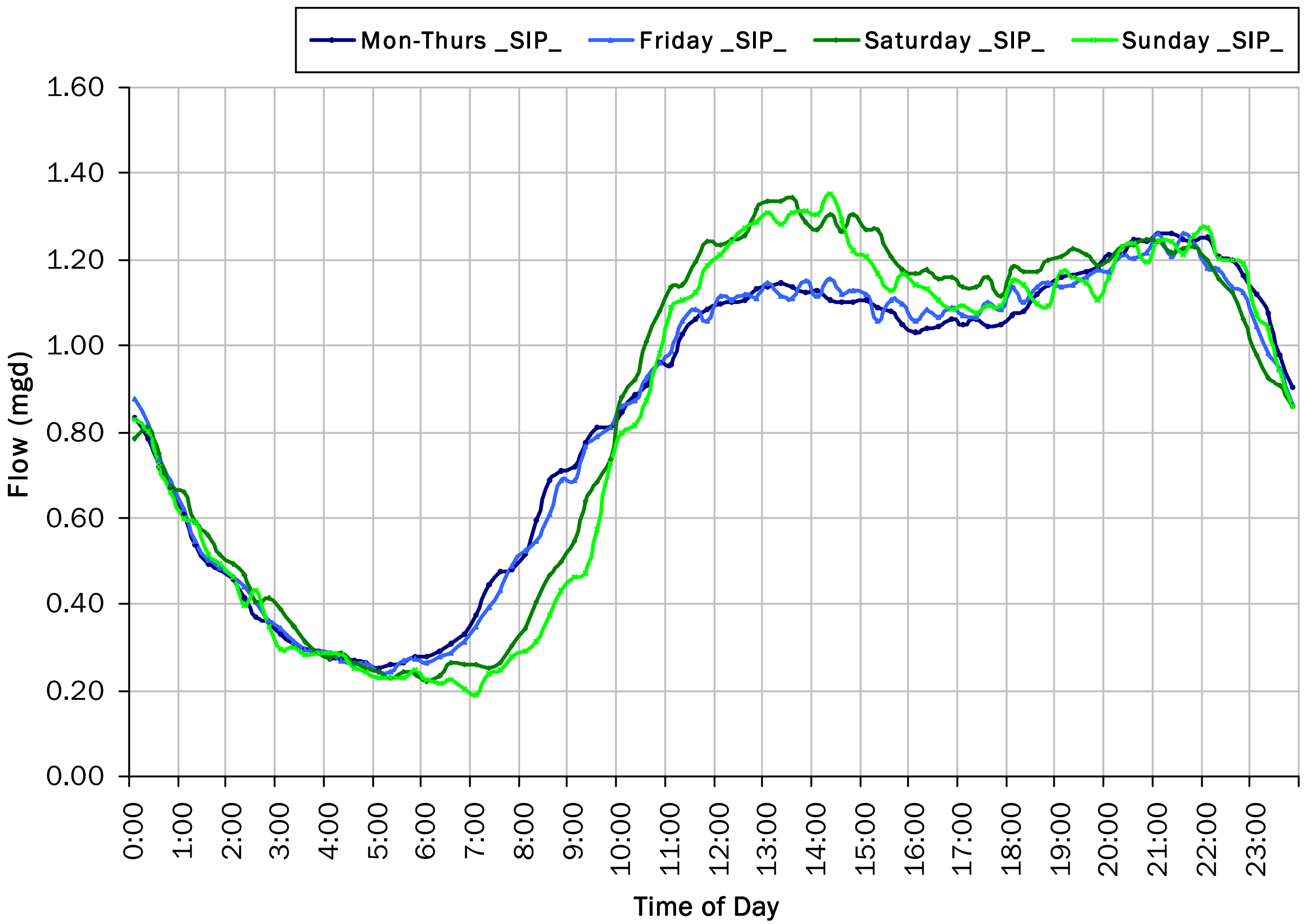


ADWF:

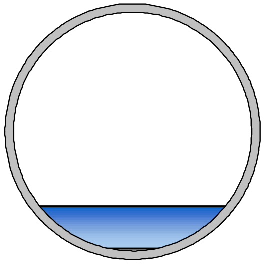
0.777 mgd



FM 2
Average Dry Weather Flow Hydrographs - Shelter In Place (> 3/14/20)

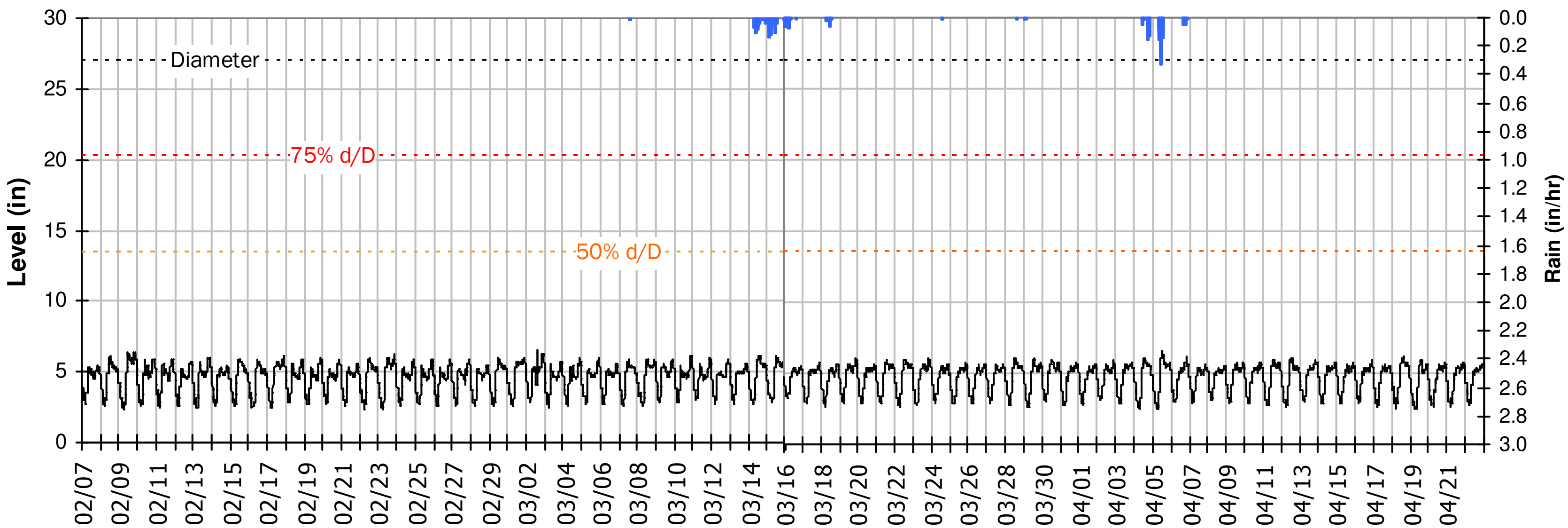


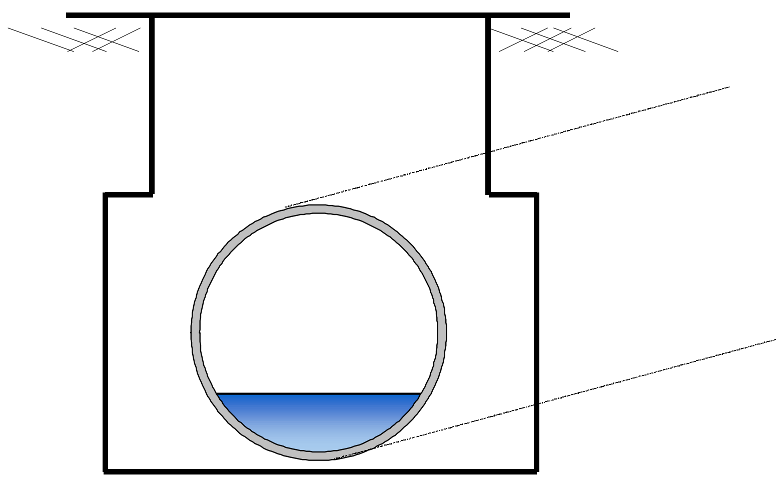
ADWF:
0.840 mgd



FM 2
Site Capacity and Surcharge Summary

<u>Realtime Flow Levels with Rainfall Data over Monitoring Period</u>

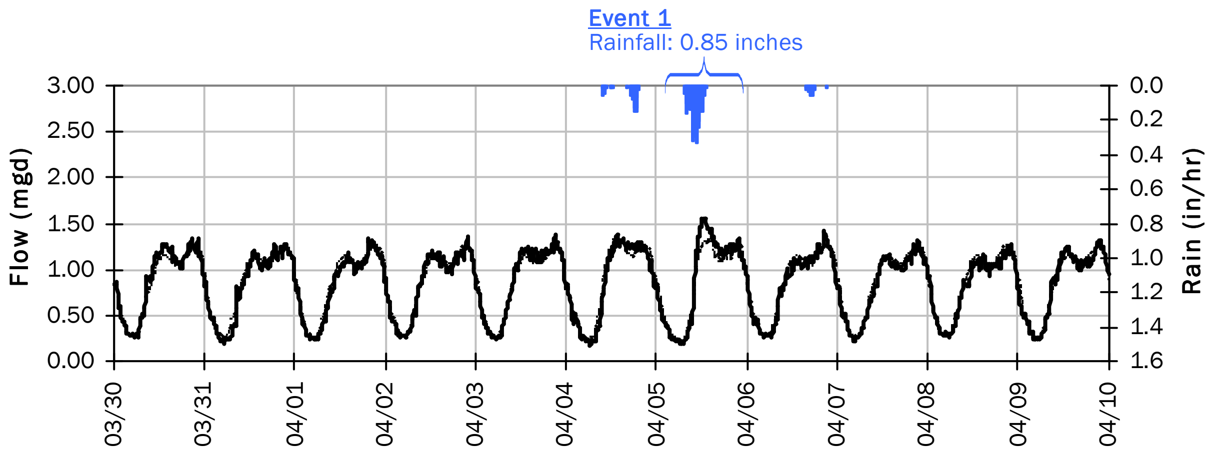




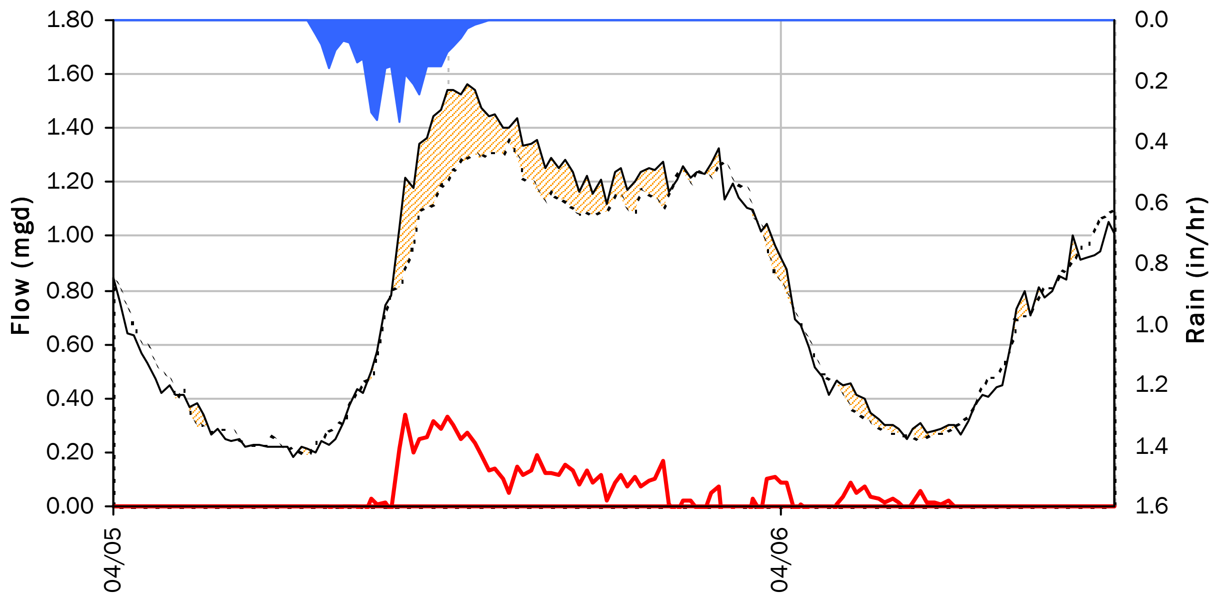
Pipe Diameter:	27	*inches*
Peak Measured Level:	6.55	*inches*
Peak d/D Ratio:	0.24	

FM 2
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



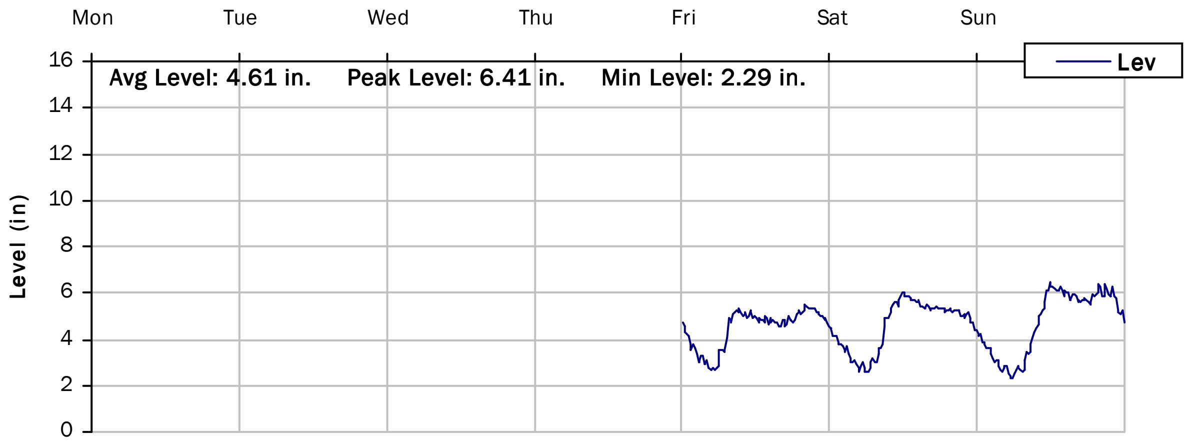
Event 1 Detail Graph

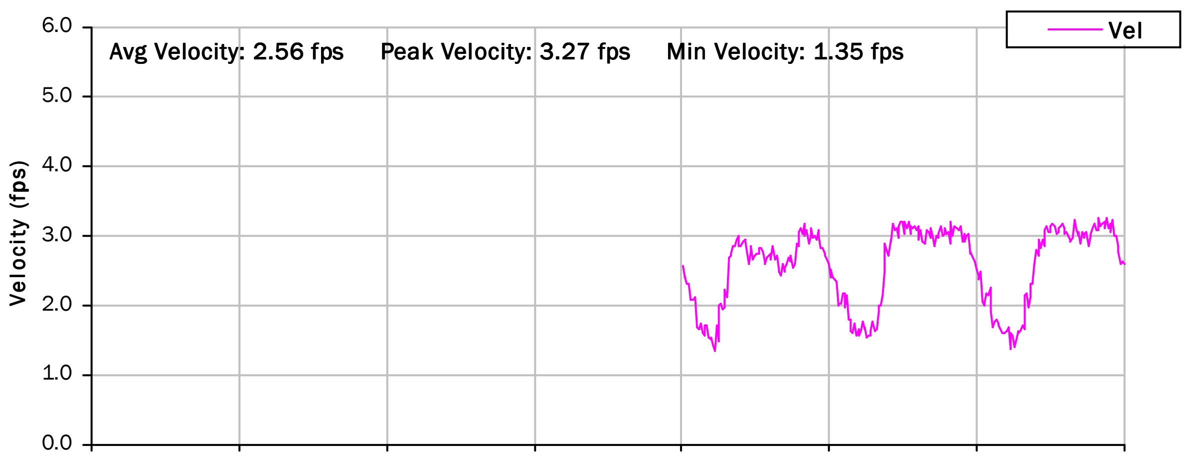


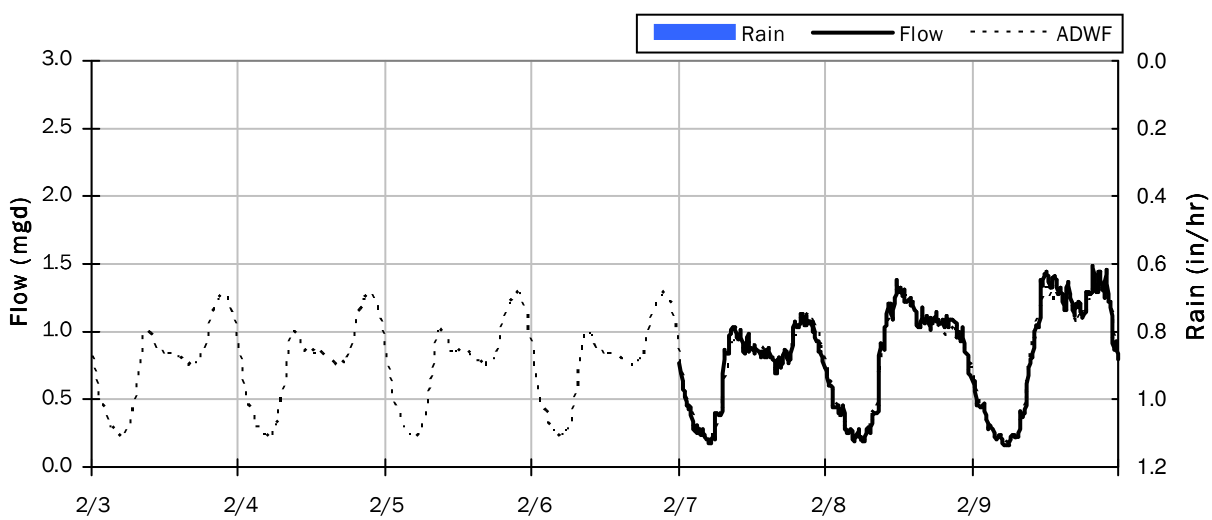
Storm Event I/I Analysis (Rain = 0.85 inches)

Capacity		Inflow / Infiltration	
Peak Flow:	1.56 *mgd*	**Peak I/I Rate:**	0.342 *mgd*
PF:	1.93	**Total I/I:**	73,000 *gallons*
Peak Level:	6.55 *in*		
d/D Ratio:	0.24		

FM 2
Weekly Level, Velocity and Flow Hydrographs
2/3/2020 to 2/10/2020

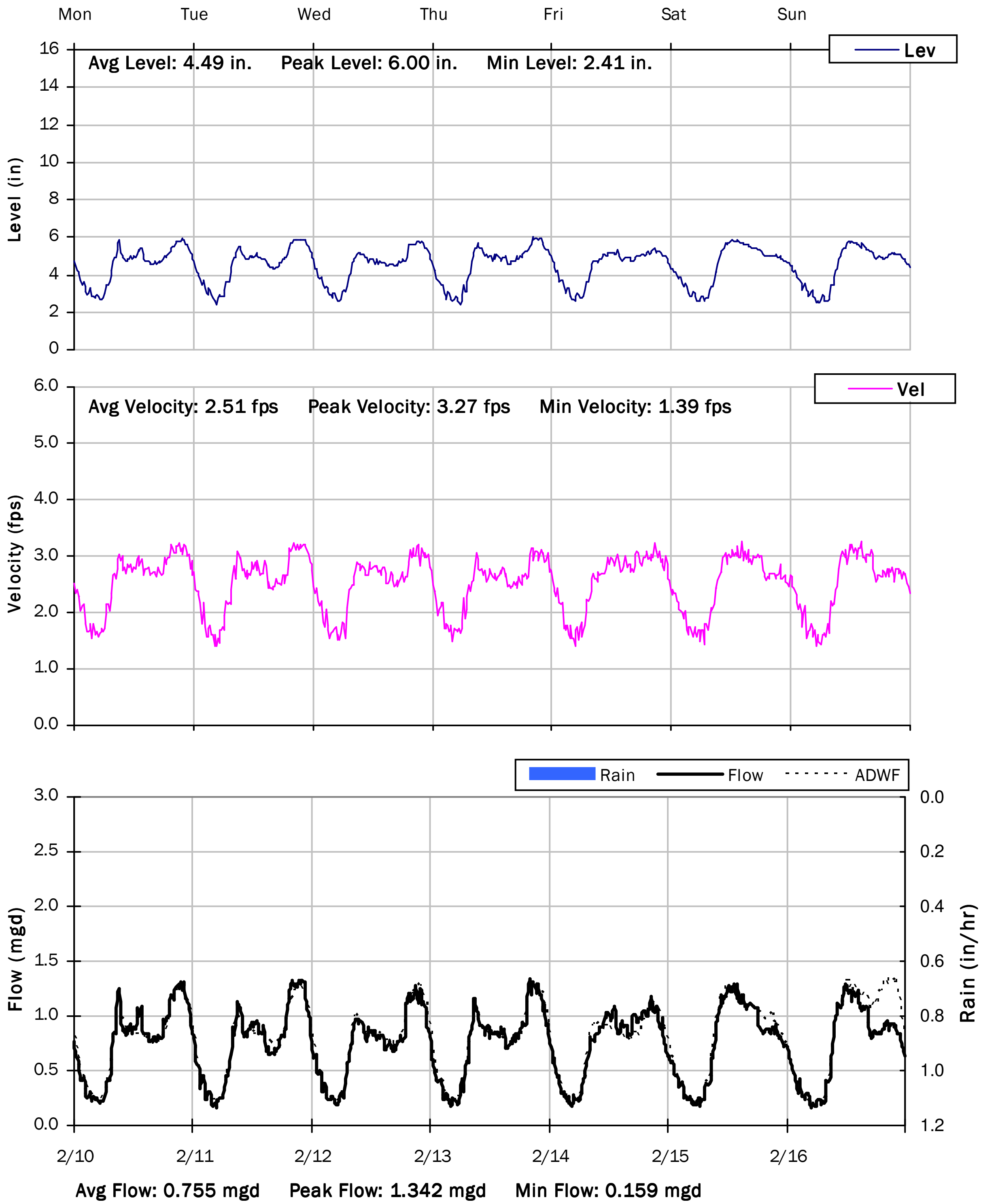




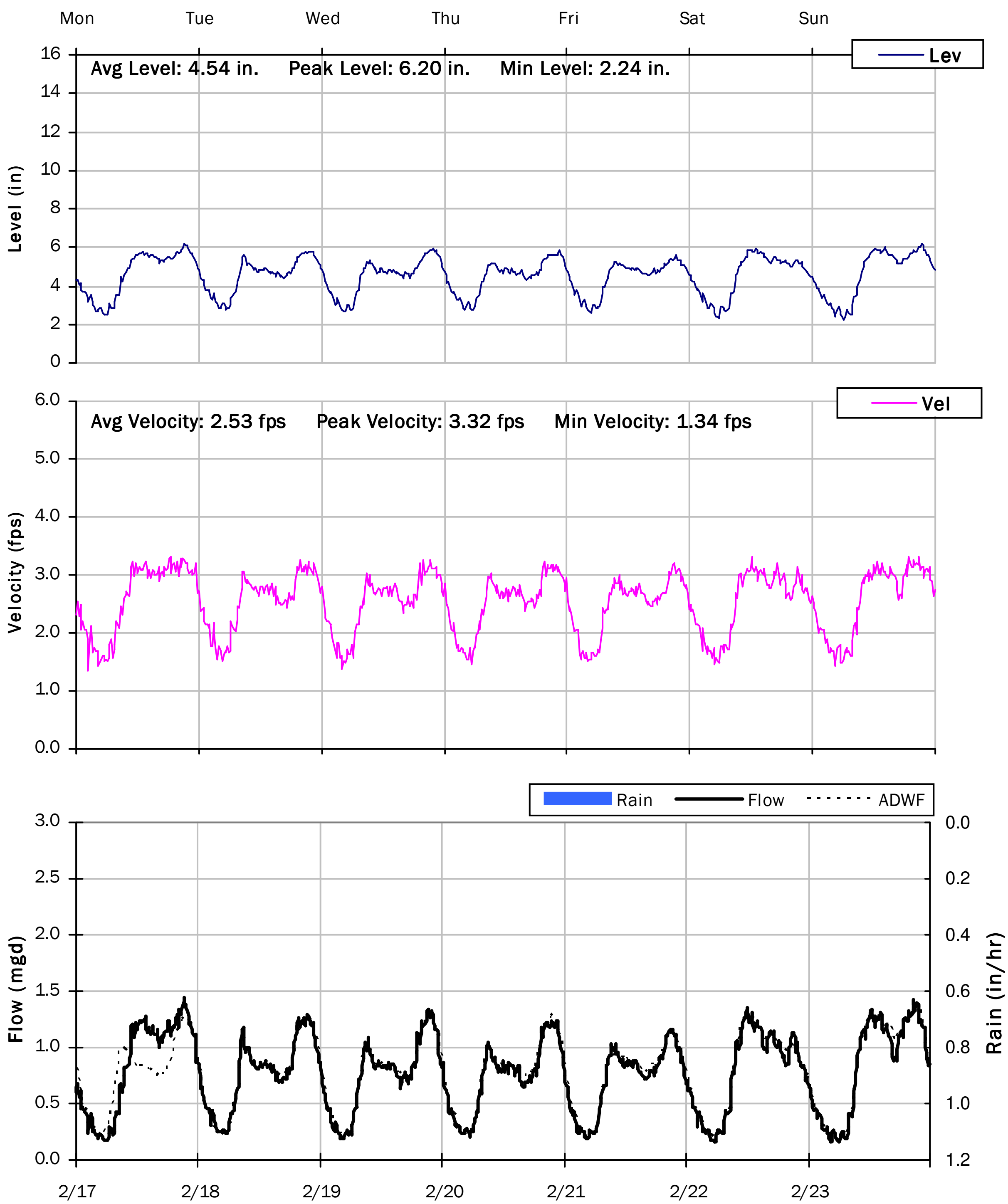


Avg Flow: 0.810 mgd **Peak Flow: 1.491 mgd** **Min Flow: 0.157 mgd**

FM 2
Weekly Level, Velocity and Flow Hydrographs
2/10/2020 to 2/17/2020

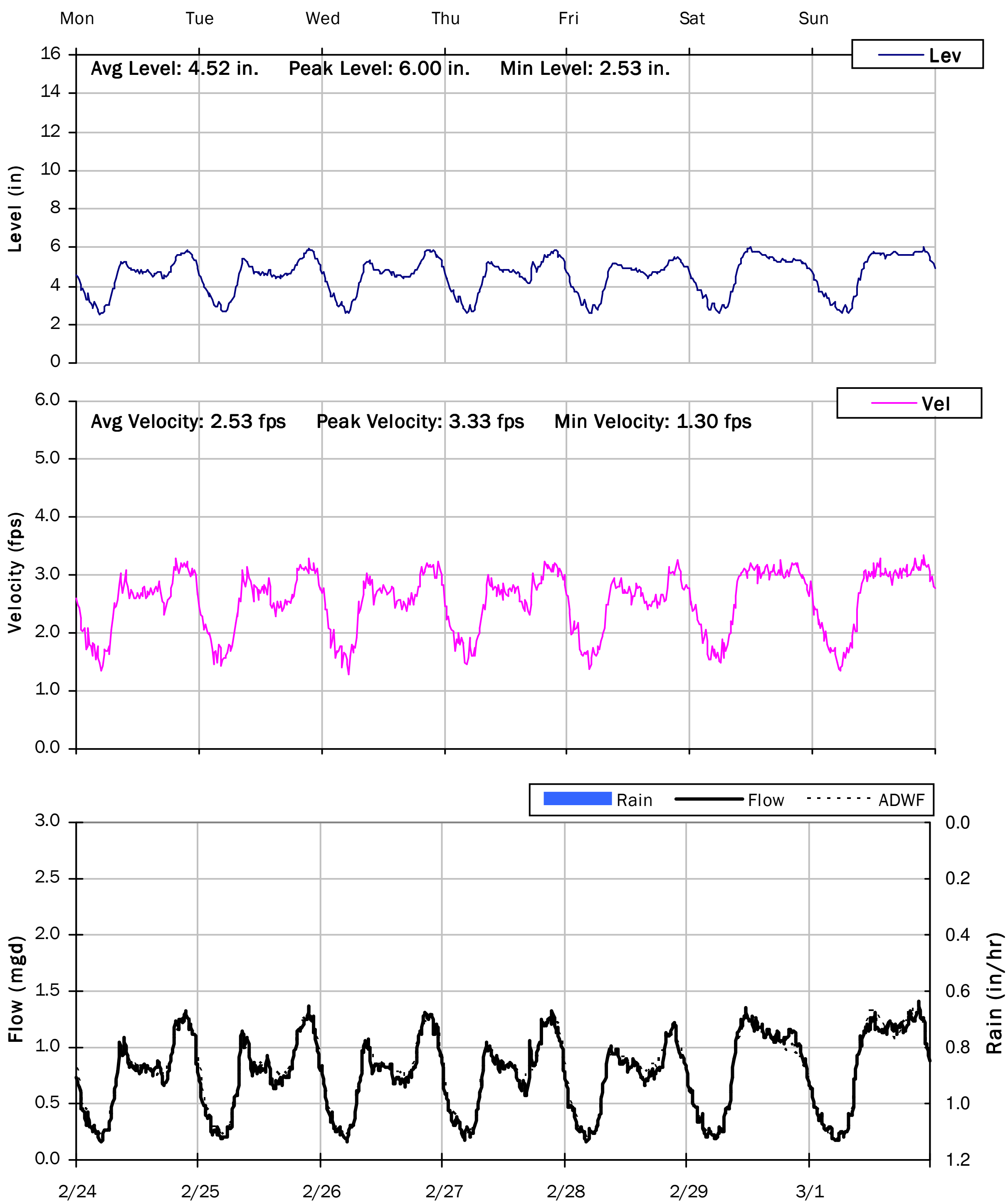


FM 2
Weekly Level, Velocity and Flow Hydrographs
2/17/2020 to 2/24/2020



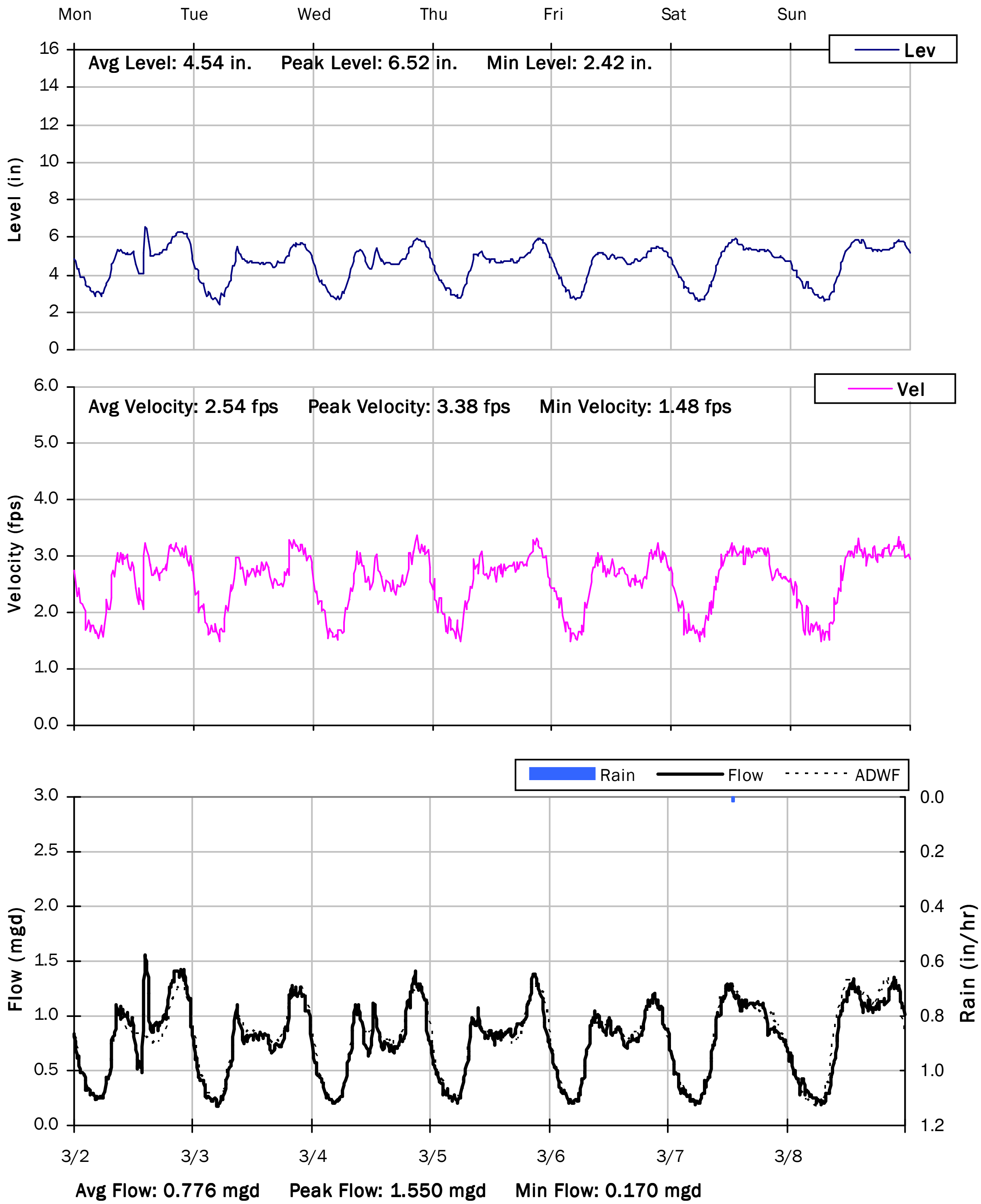
Avg Flow: 0.778 mgd **Peak Flow: 1.436 mgd** **Min Flow: 0.156 mgd**

FM 2
Weekly Level, Velocity and Flow Hydrographs
2/24/2020 to 3/2/2020

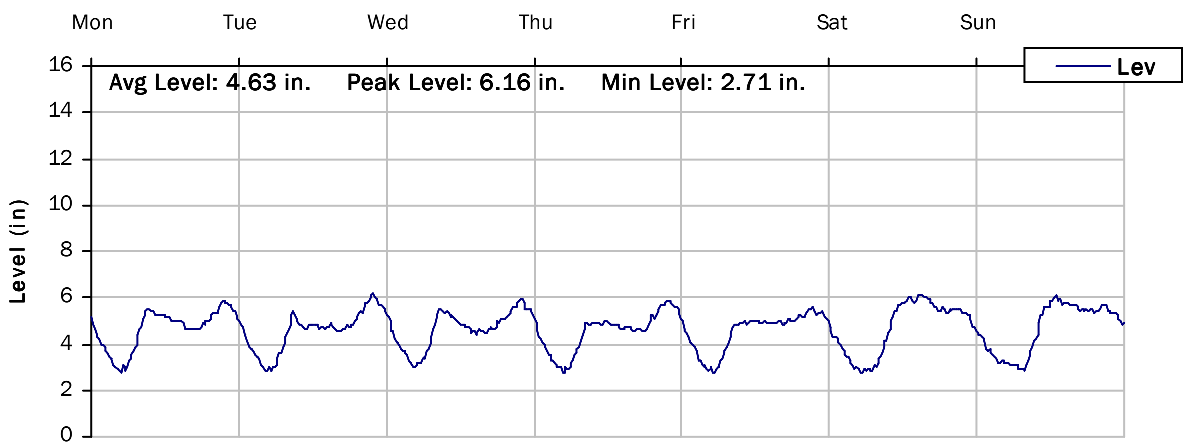


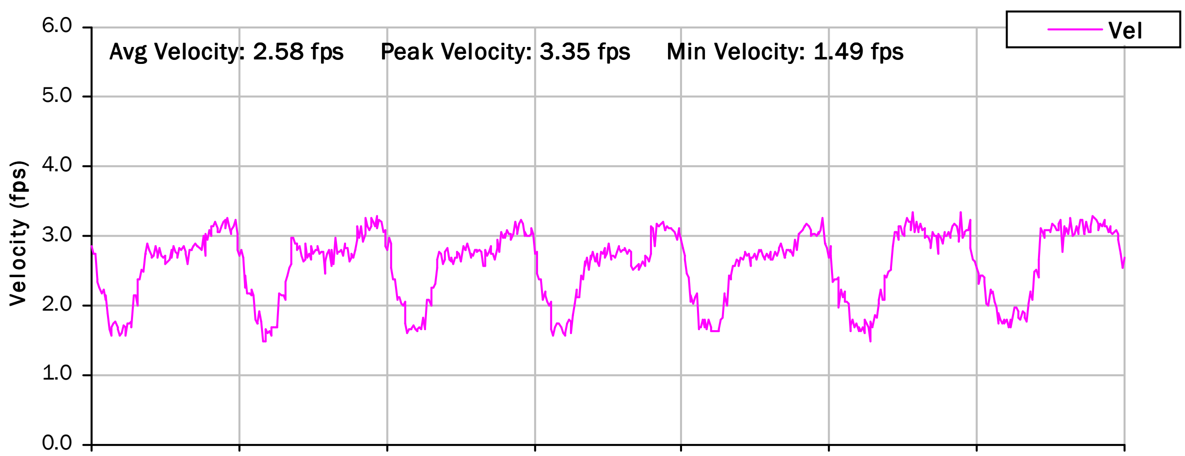
Avg Flow: 0.771 mgd **Peak Flow: 1.417 mgd** **Min Flow: 0.162 mgd**

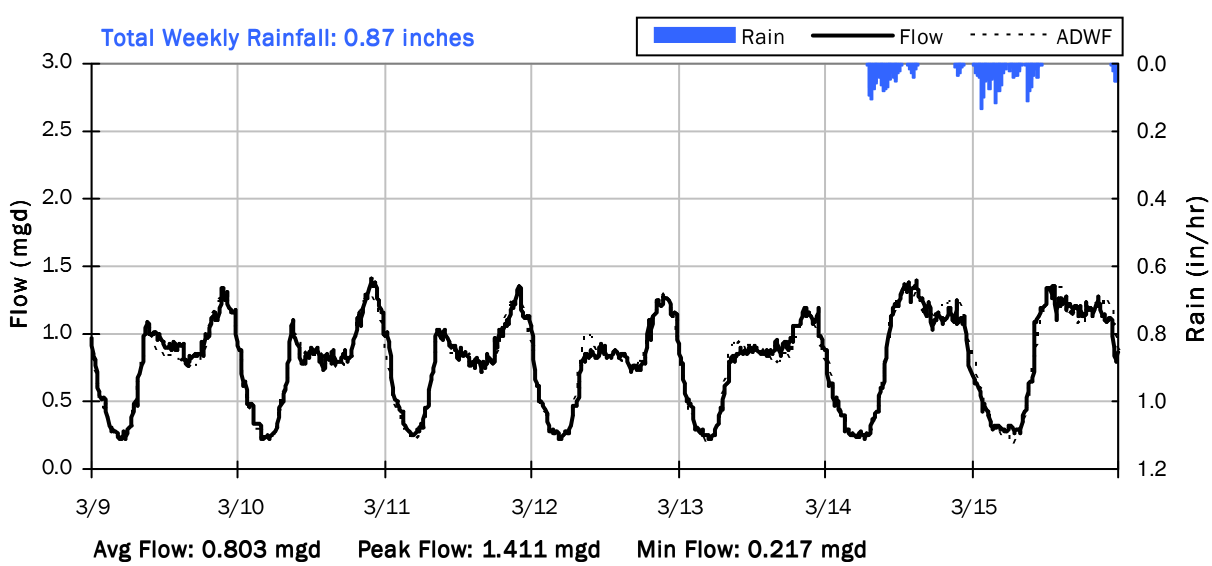
FM 2
Weekly Level, Velocity and Flow Hydrographs
3/2/2020 to 3/9/2020



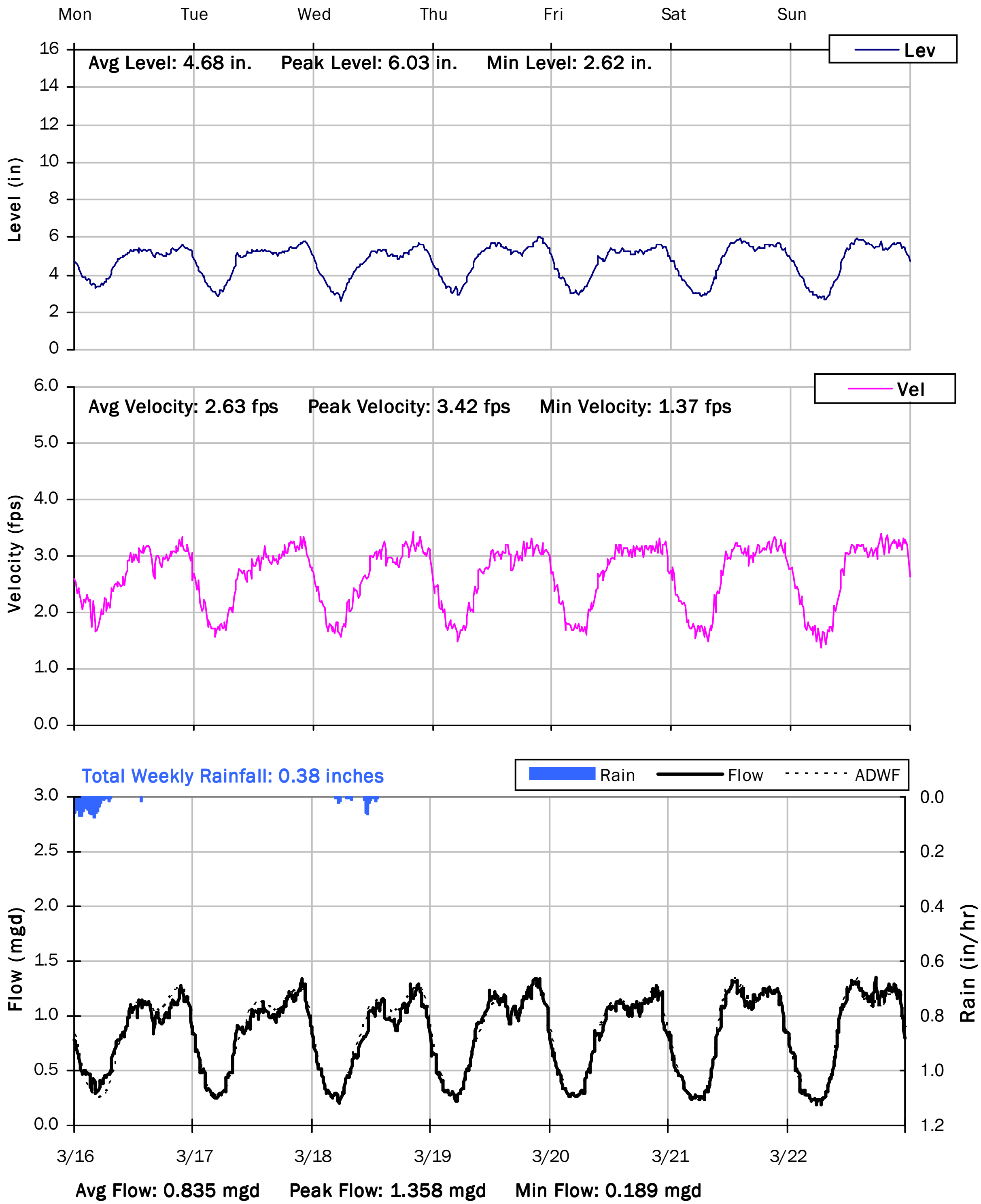
FM 2
Weekly Level, Velocity and Flow Hydrographs
3/9/2020 to 3/16/2020



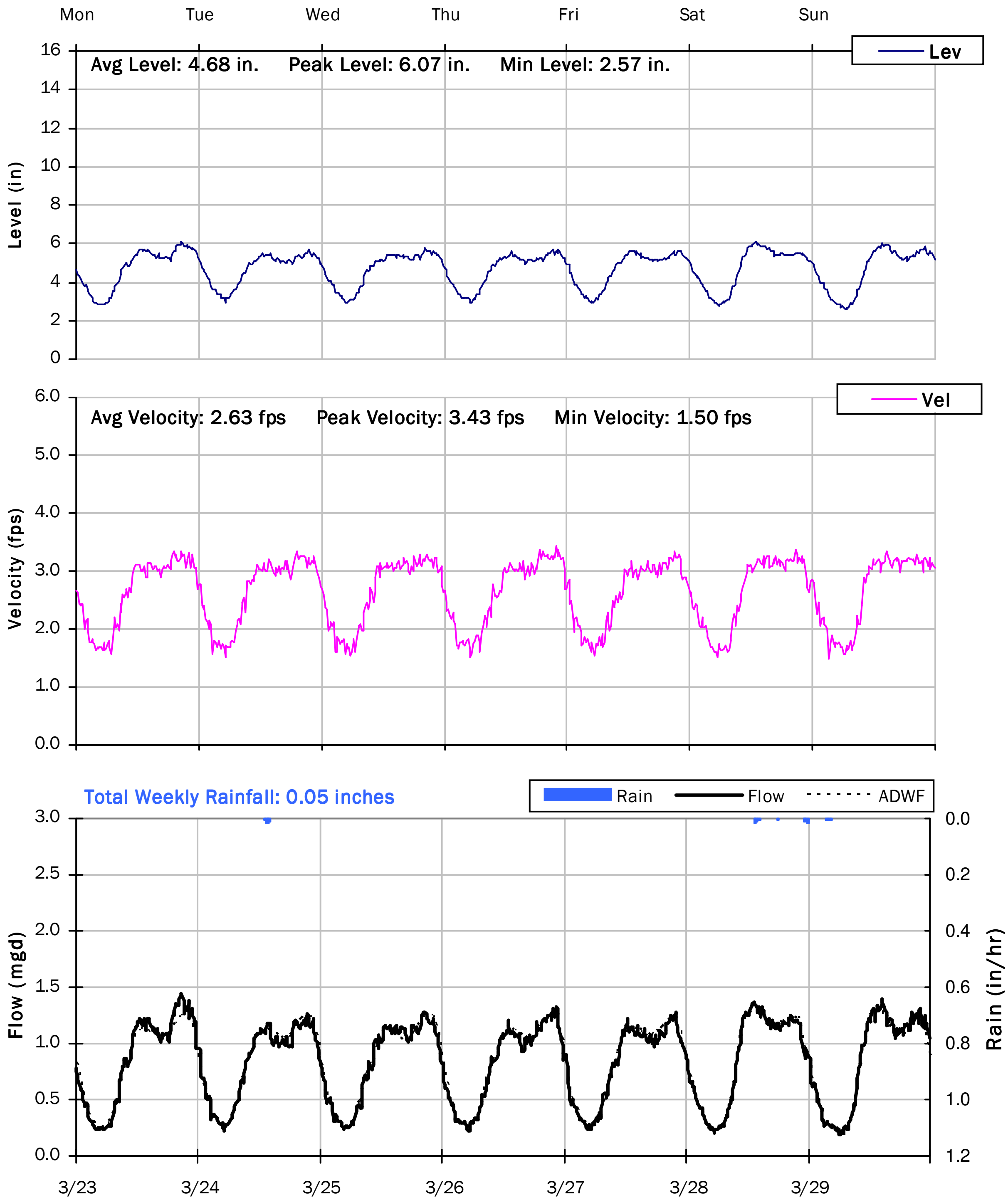




FM 2
Weekly Level, Velocity and Flow Hydrographs
3/16/2020 to 3/23/2020



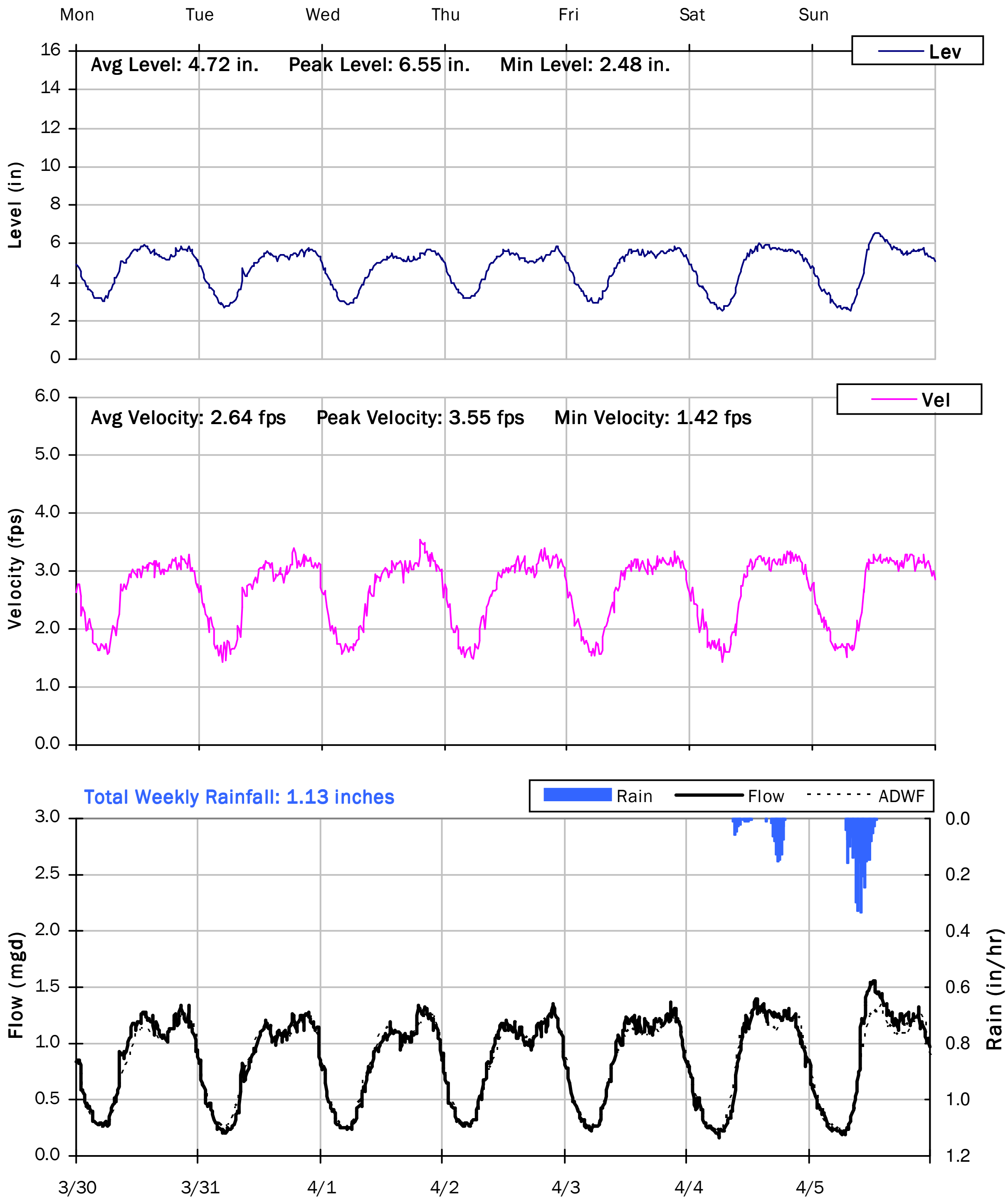
FM 2
Weekly Level, Velocity and Flow Hydrographs
3/23/2020 to 3/30/2020



Avg Flow: 0.842 mgd Peak Flow: 1.448 mgd Min Flow: 0.195 mgd

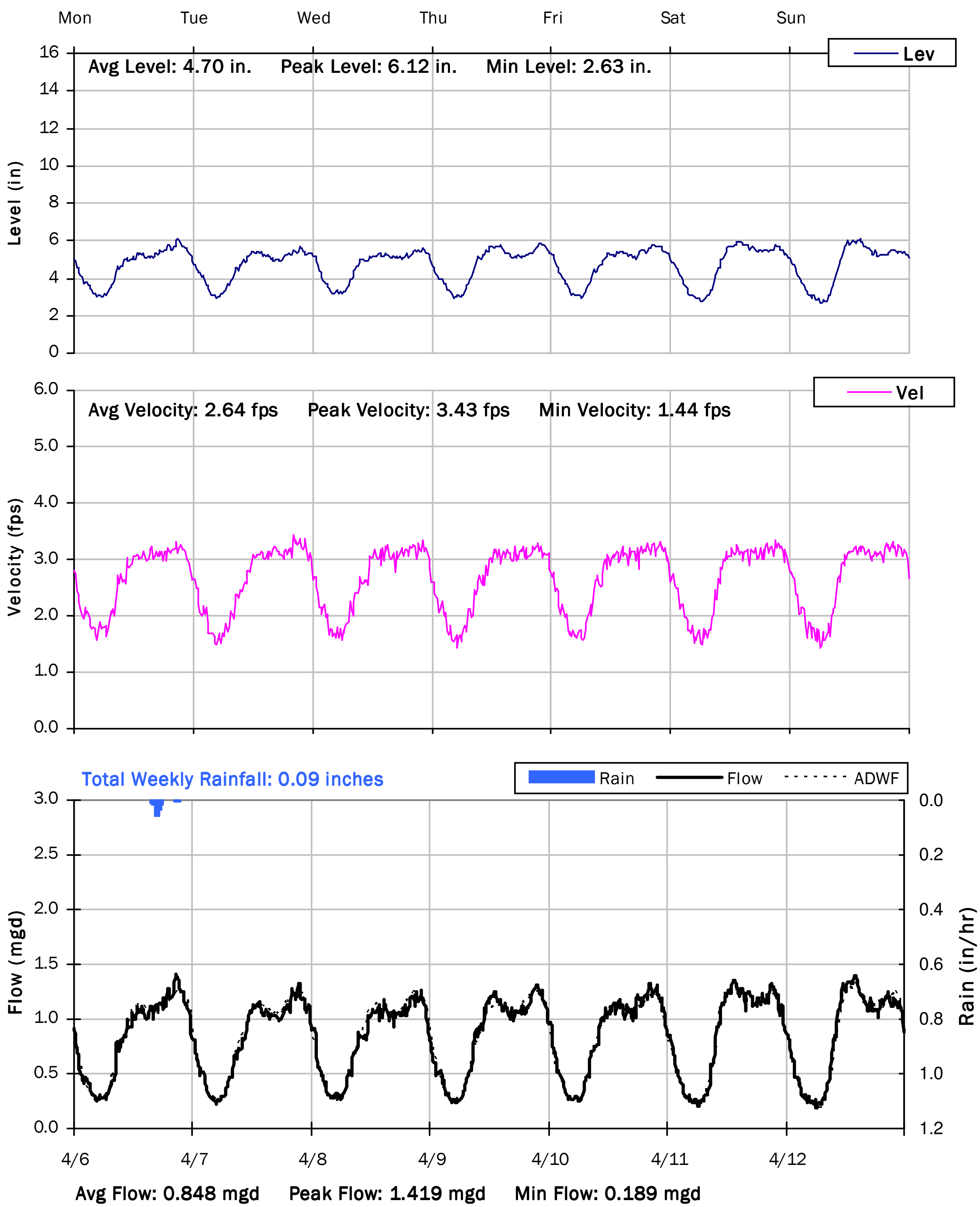
FM 2

Weekly Level, Velocity and Flow Hydrographs
3/30/2020 to 4/6/2020

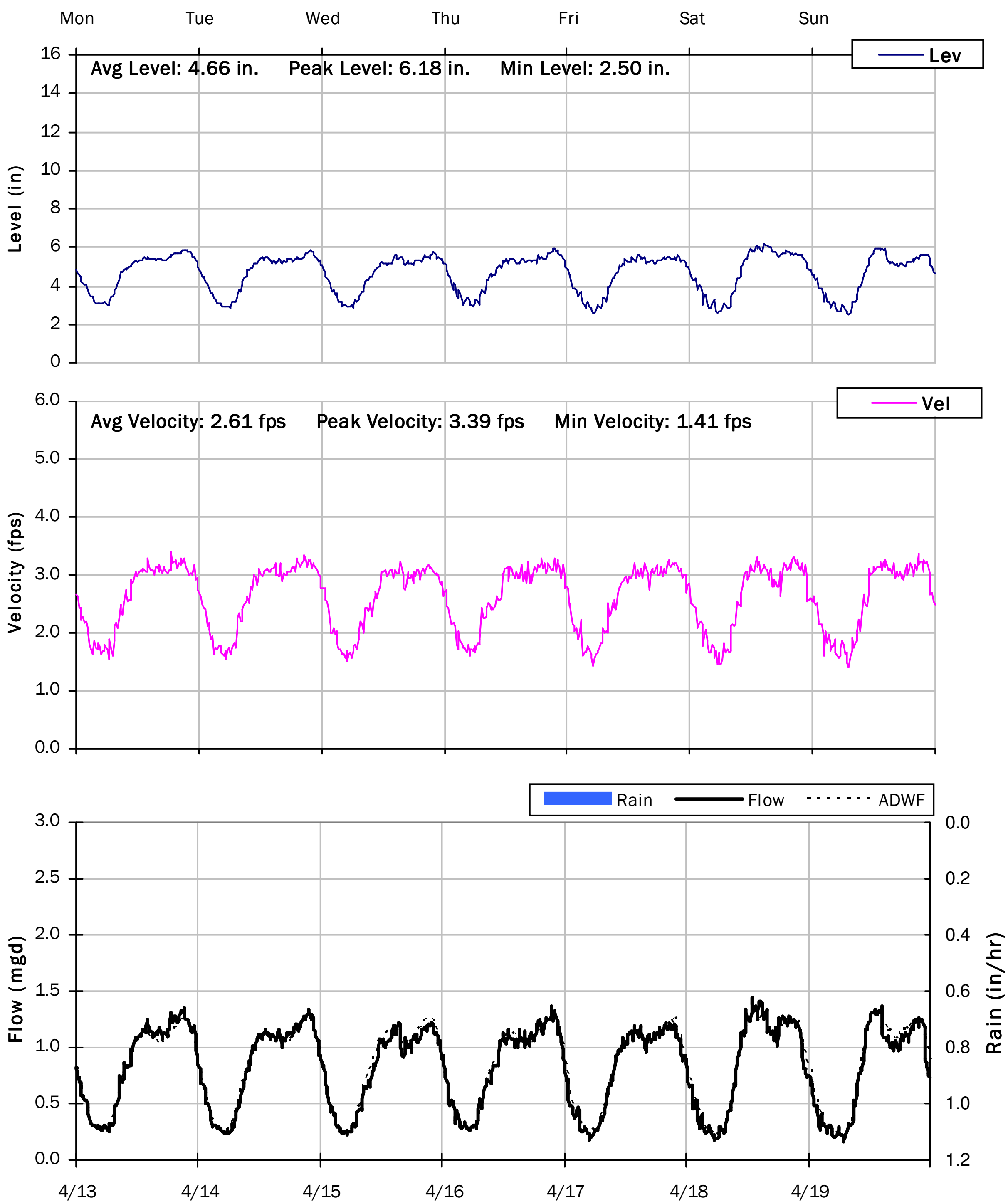


Avg Flow: 0.860 mgd **Peak Flow: 1.560 mgd** **Min Flow: 0.167 mgd**

FM 2
Weekly Level, Velocity and Flow Hydrographs
4/6/2020 to 4/13/2020

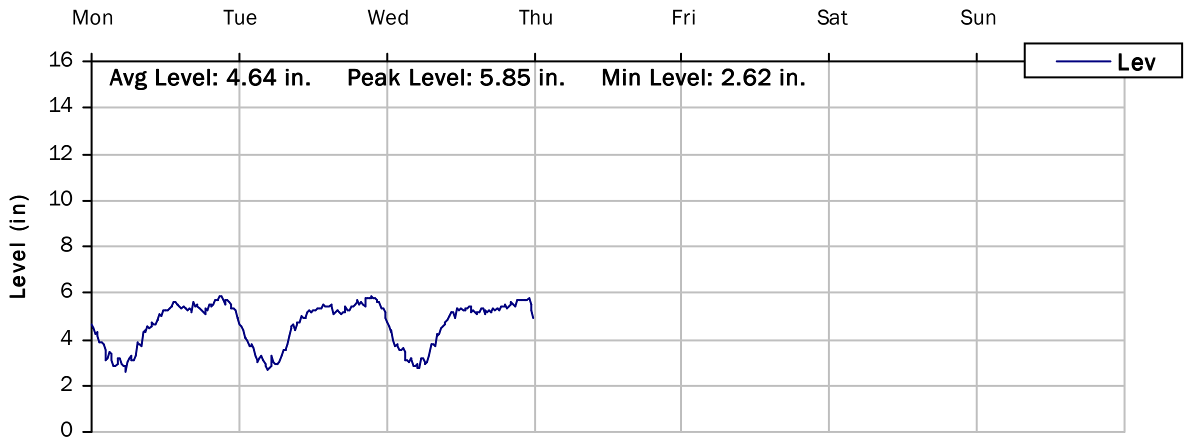


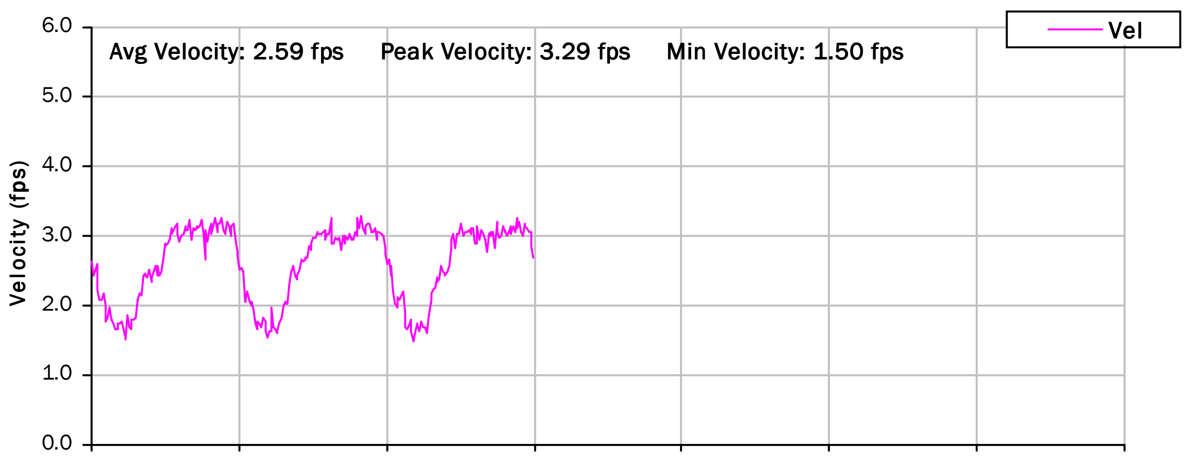
FM 2
Weekly Level, Velocity and Flow Hydrographs
4/13/2020 to 4/20/2020

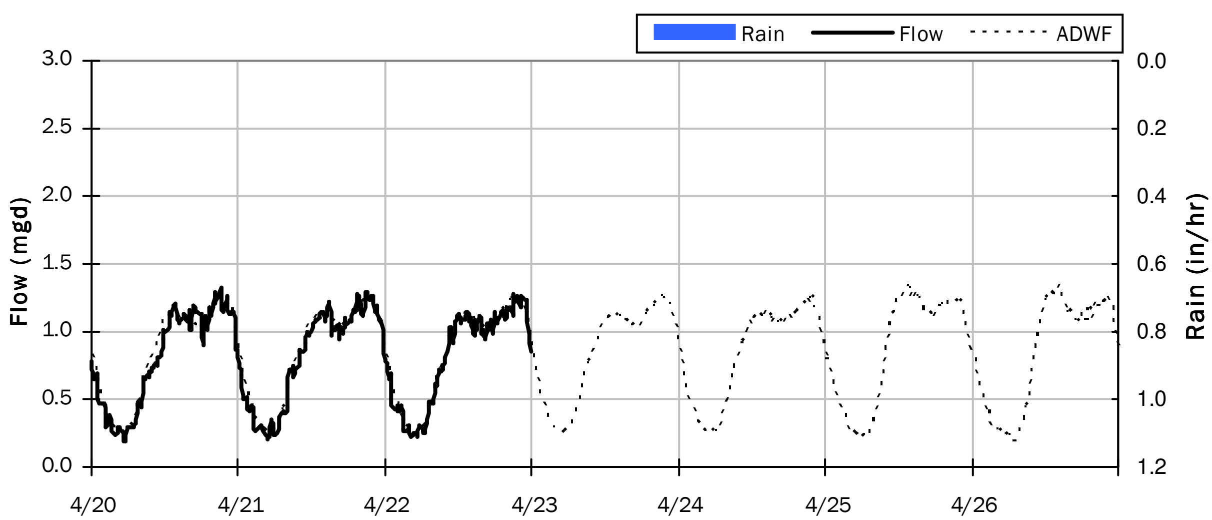


Avg Flow: 0.832 mgd **Peak Flow: 1.439 mgd** **Min Flow: 0.168 mgd**

FM 2
Weekly Level, Velocity and Flow Hydrographs
4/20/2020 to 4/27/2020







Avg Flow: 0.817 mgd **Peak Flow: 1.329 mgd** **Min Flow: 0.195 mgd**

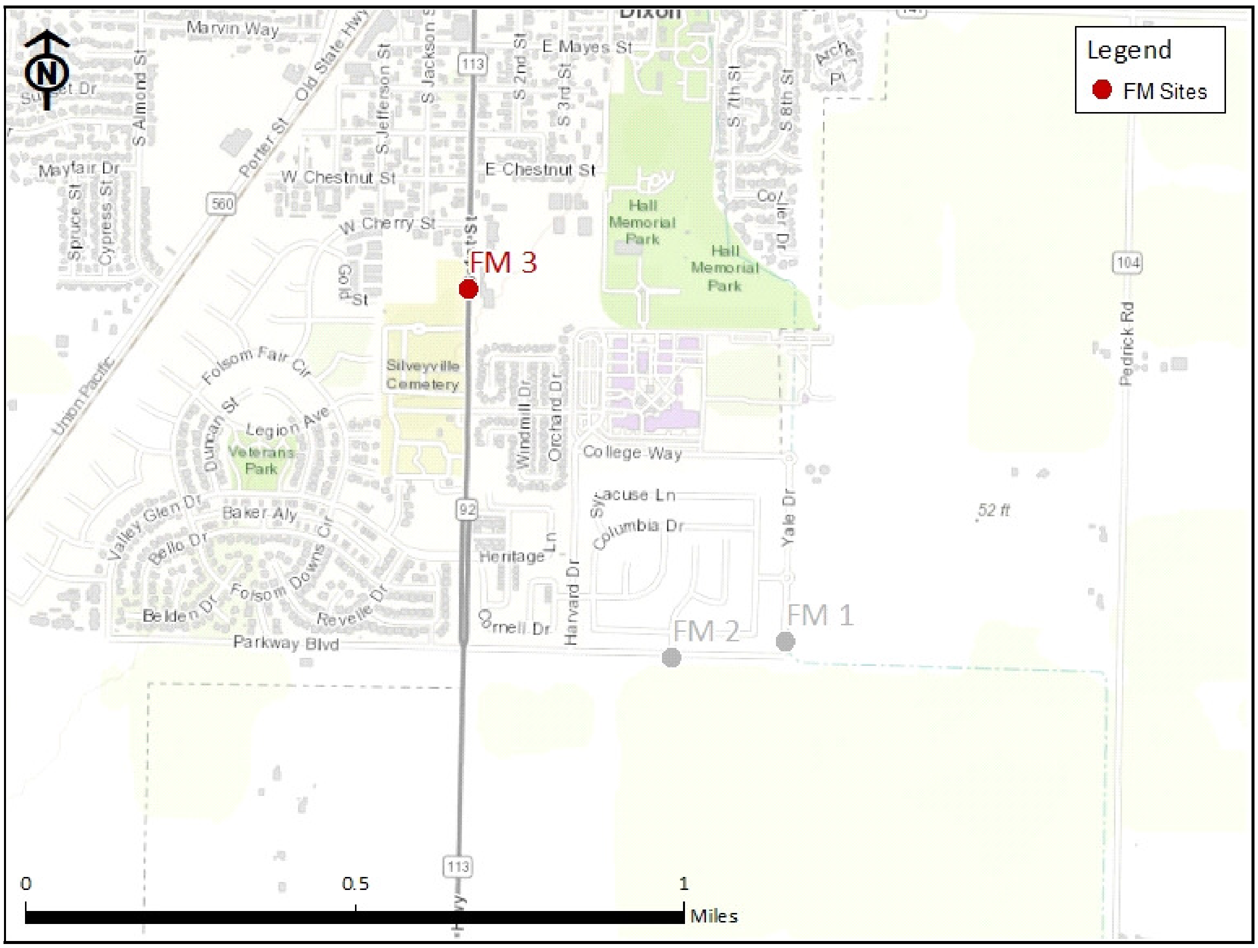
City of Dixon

Sanitary Sewer Flow Monitoring
February 06 - April 23, 2020

Monitoring Site: FM 3

Location: S 1st Street, 525' south of W Cherry Street

Data Summary Report



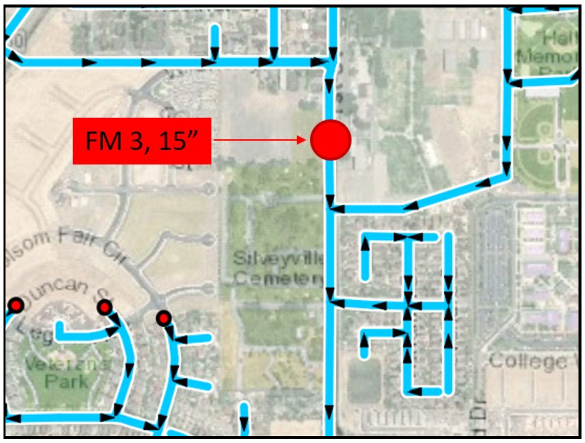
Vicinity Map: FM 3

FM 3

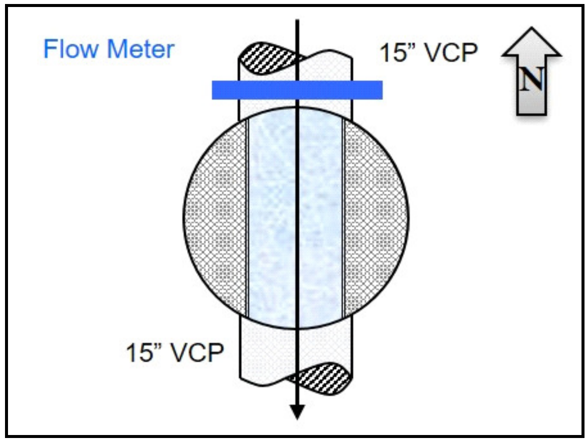
Site Information

Location:	S 1st Street, 525' south of W Cherry Street
City Manhole:	FM 3
Coordinates:	121.8226° W, 38.4391° N
Rim Elevation (Earth):	64 feet
Pipe Diameter:	15 inches
ADWF:	0.571 mgd
Peak Measured Flow:	2.067 mgd

Satellite Map



Sanitary Map



Flow Sketch

Street View

Plan View

FM 3

Additional Site Photos

Effluent Pipe

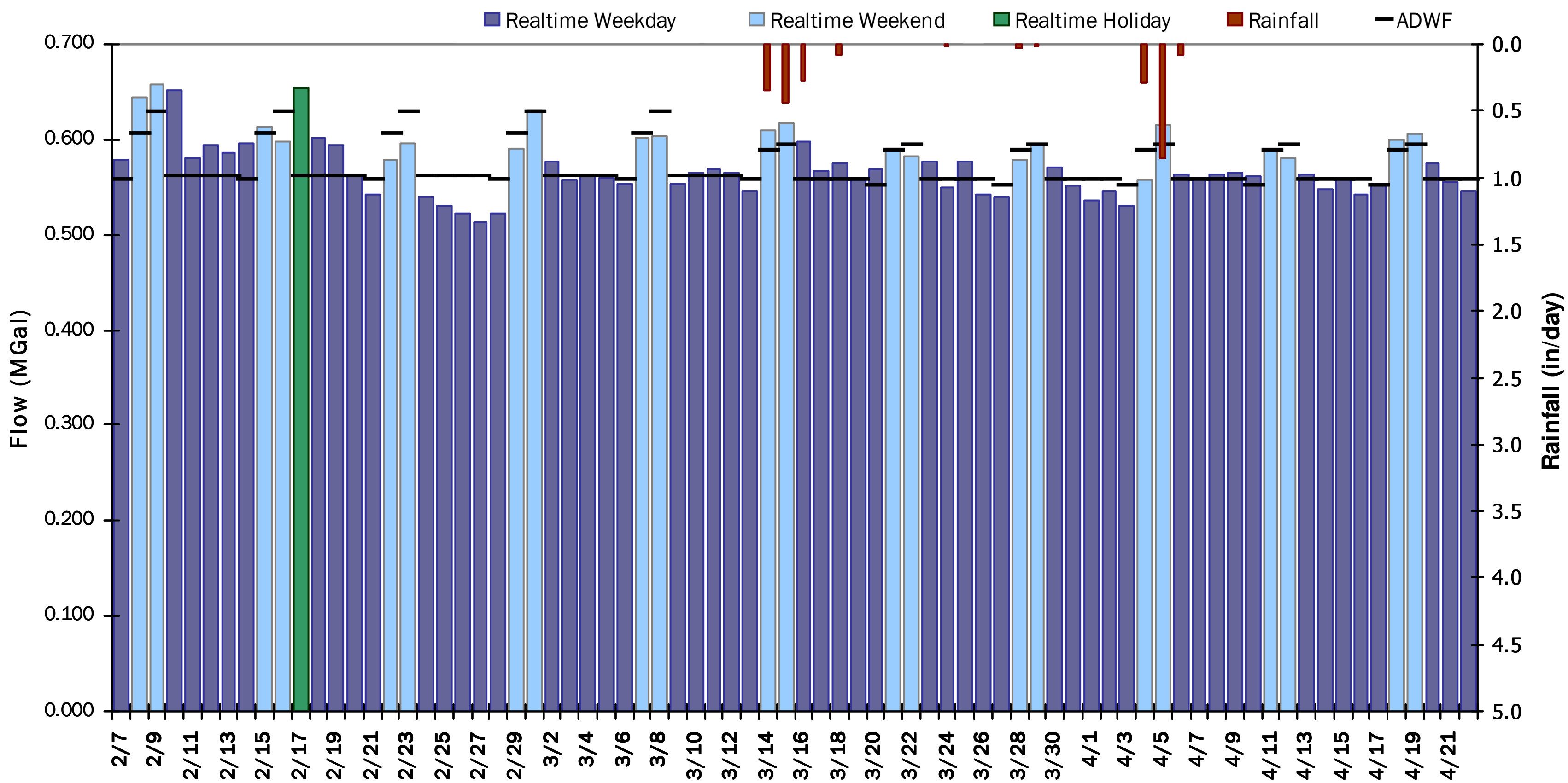
Monitored Influent Pipe

FM 3

Period Flow Summary: Daily Flow Totals

Avg Period Flow: 0.574 MGal **Peak Daily Flow: 0.658 MGal** **Min Daily Flow: 0.513 MGal**

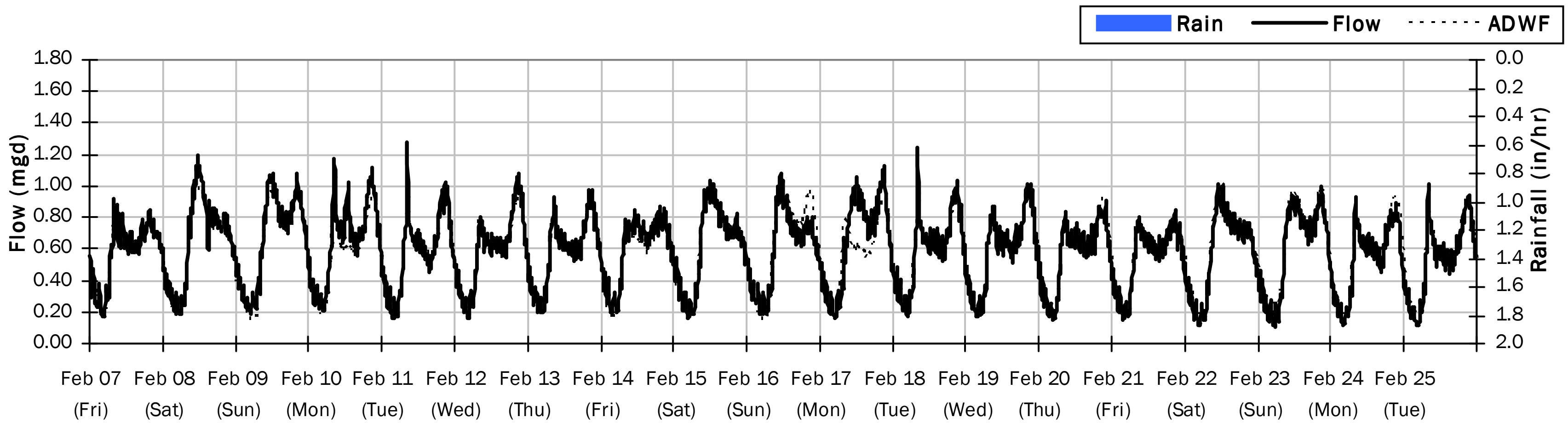
Total Period Rainfall: 2.42 inches

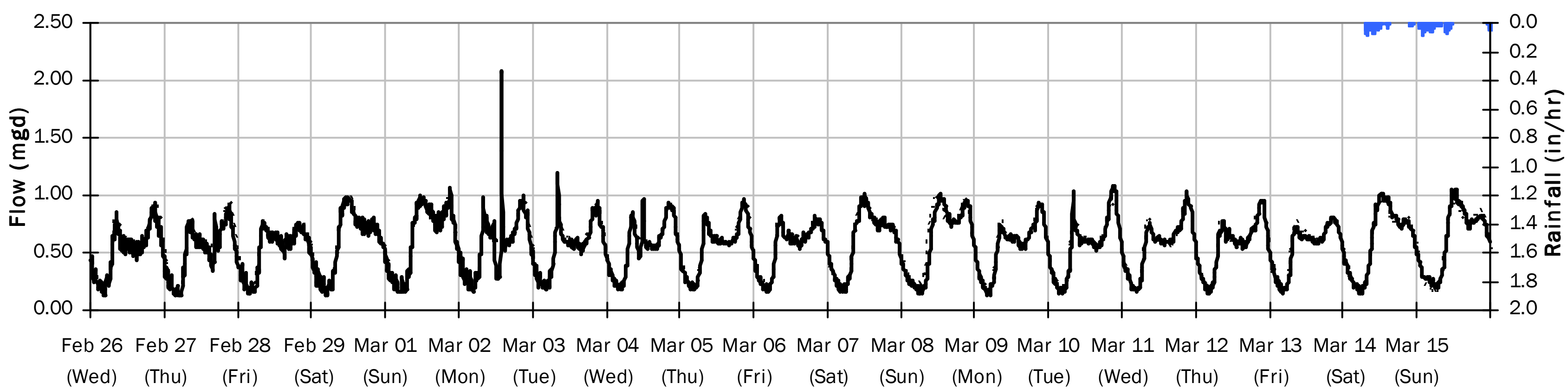


FM 3

Flow Summary: 2/7/2020 to 3/15/2020

Total Period Rainfall: 0.79 inches   **Avg Flow: 0.582 mgd**   **Peak Flow: 2.067 mgd**   **Min Flow: 0.110 mgd**

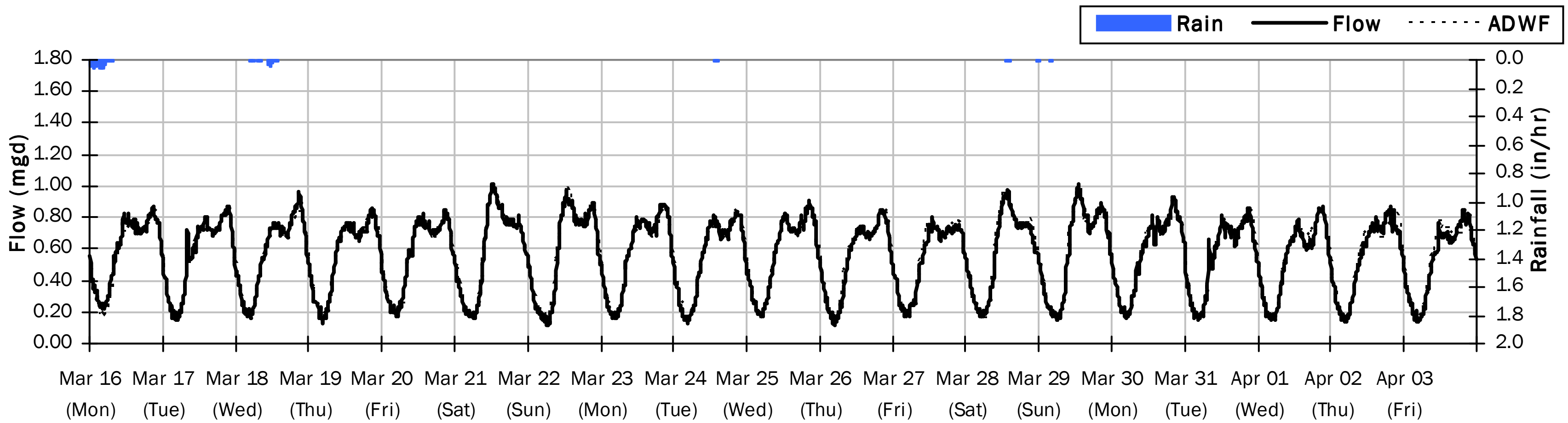


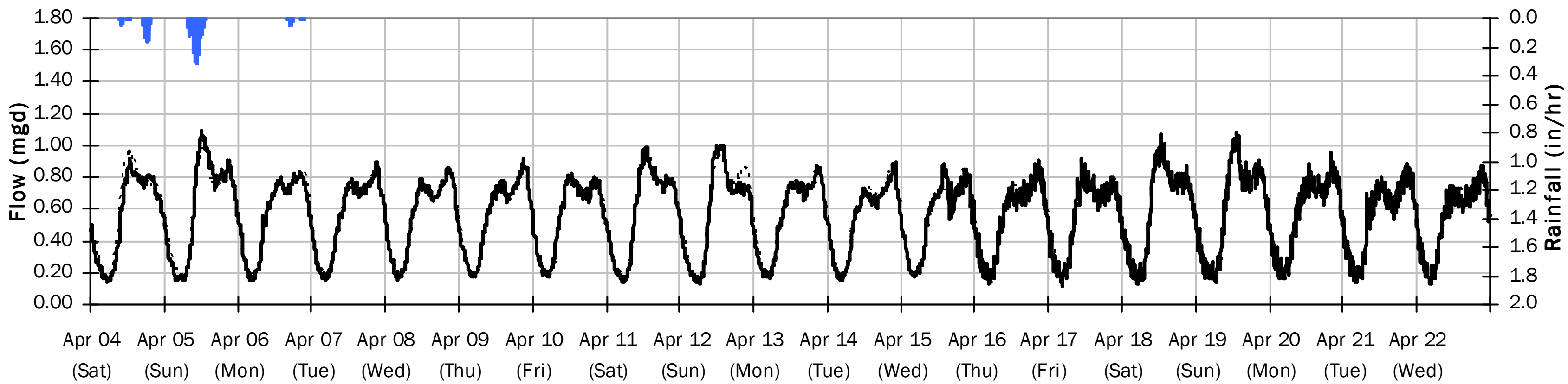


FM 3

Flow Summary: 3/16/2020 to 4/22/2020

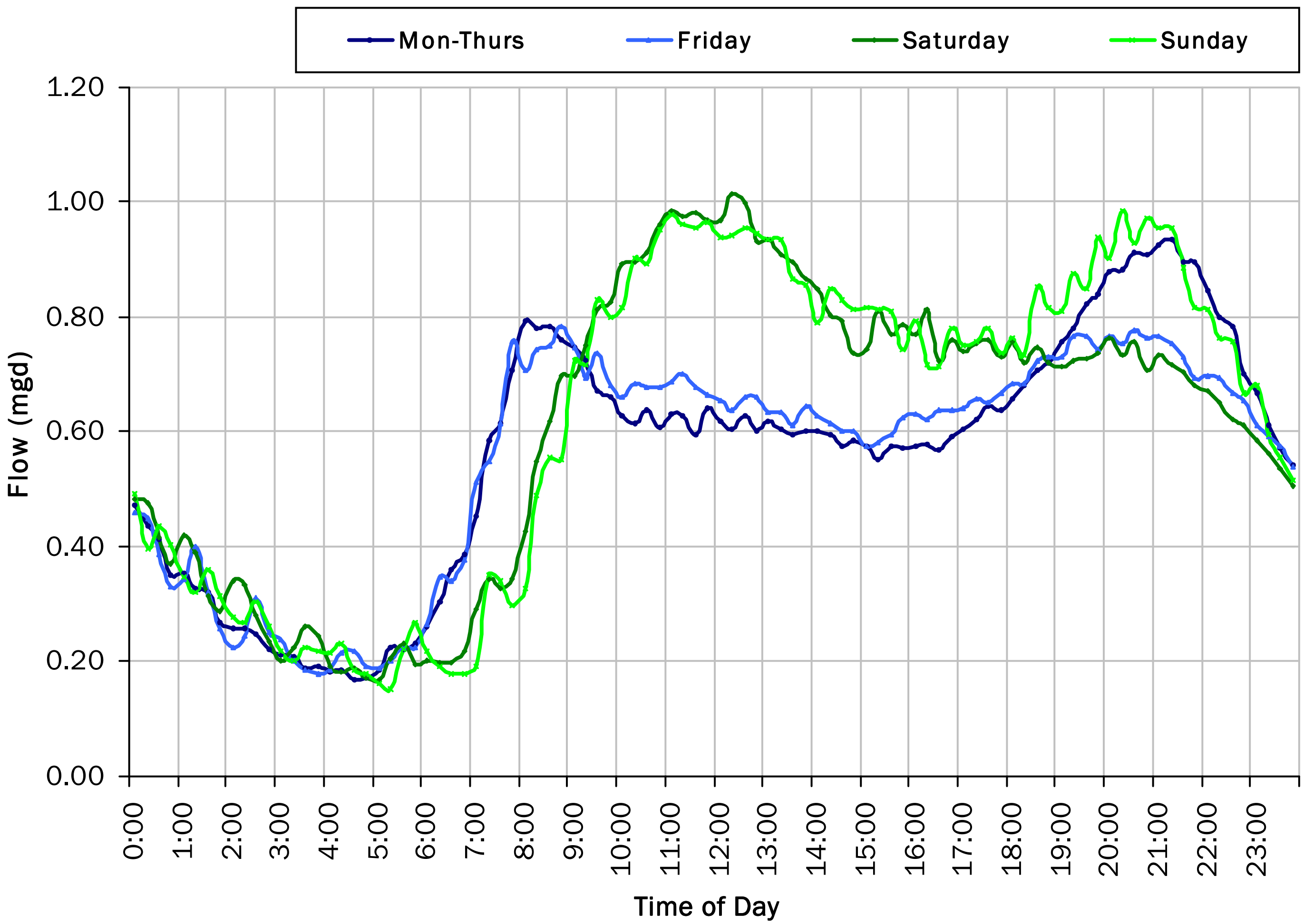
Total Period Rainfall: 1.63 inches   **Avg Flow: 0.566 mgd**   **Peak Flow: 1.087 mgd**   **Min Flow: 0.119 mgd**





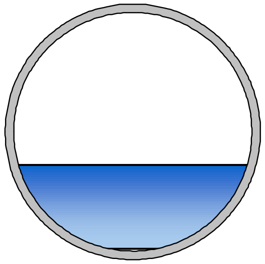
FM 3

Average Dry Weather Flow Hydrographs - Pre Shelter-In-Place (< 3/14/20)

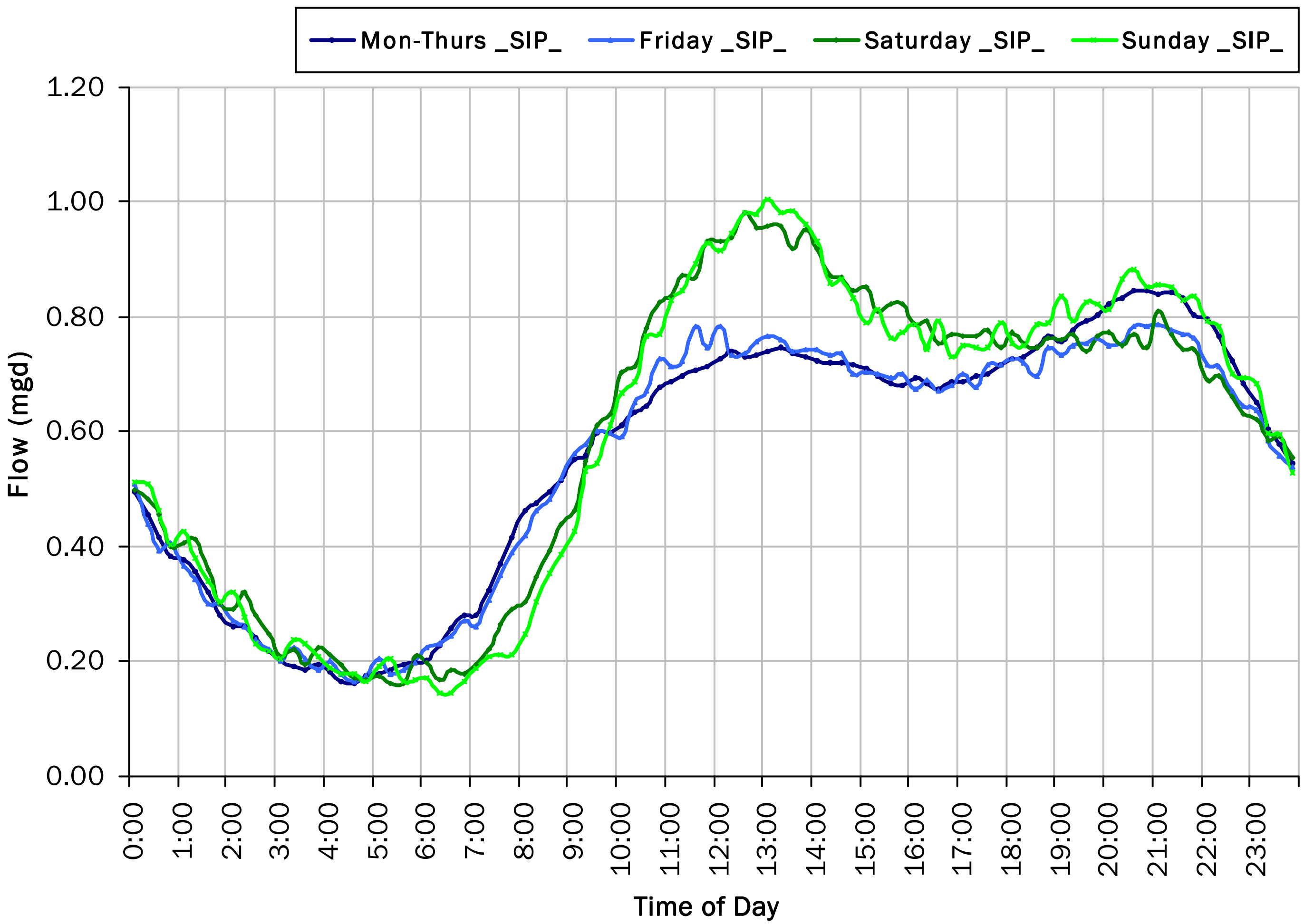


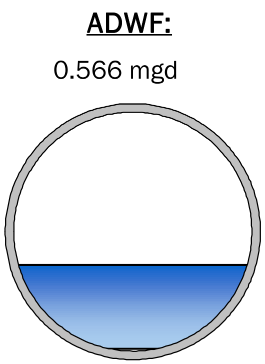
ADWF:

0.577 mgd



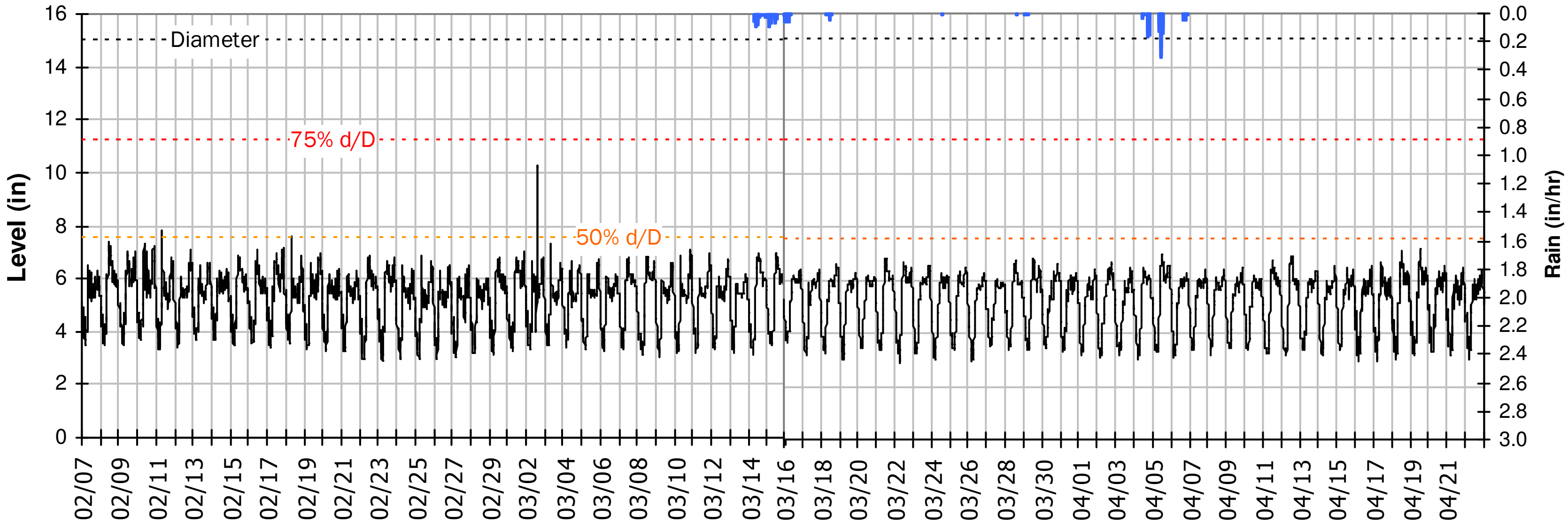
FM 3
Average Dry Weather Flow Hydrographs - Shelter In Place (> 3/14/20)

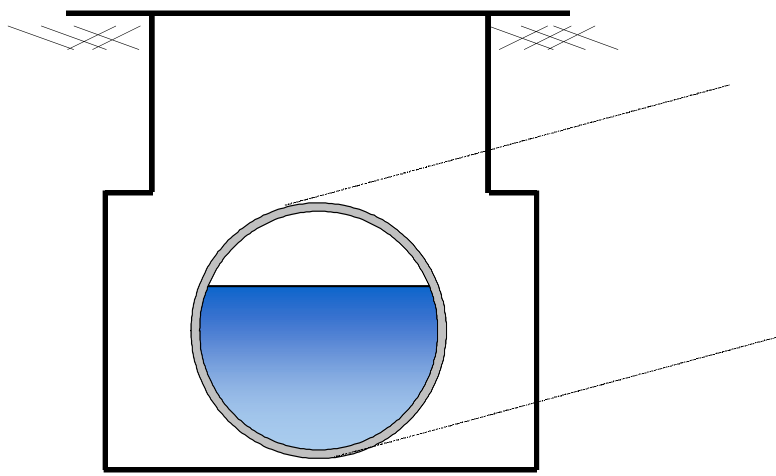




FM 3
Site Capacity and Surcharge Summary

<u>Realtime Flow Levels with Rainfall Data over Monitoring Period</u>

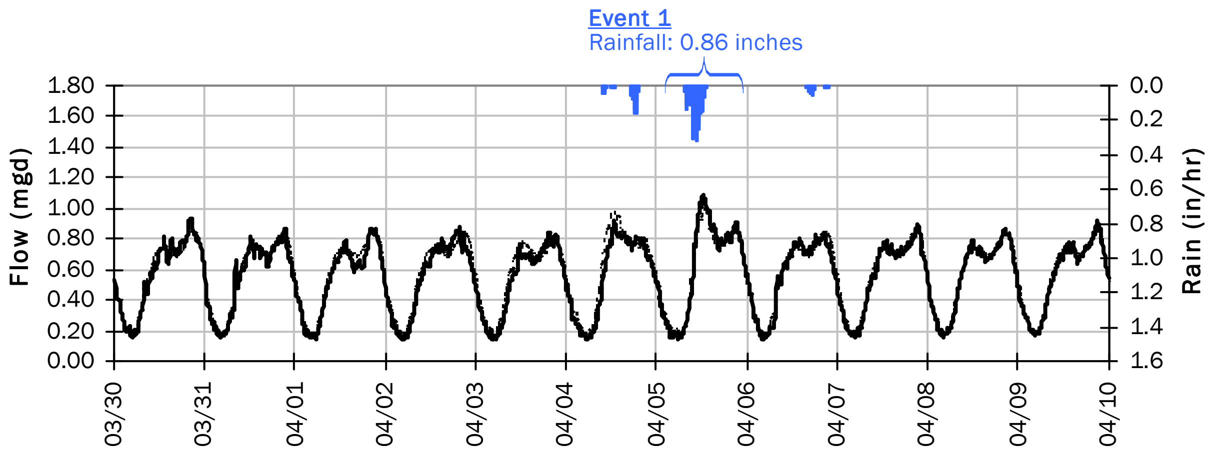




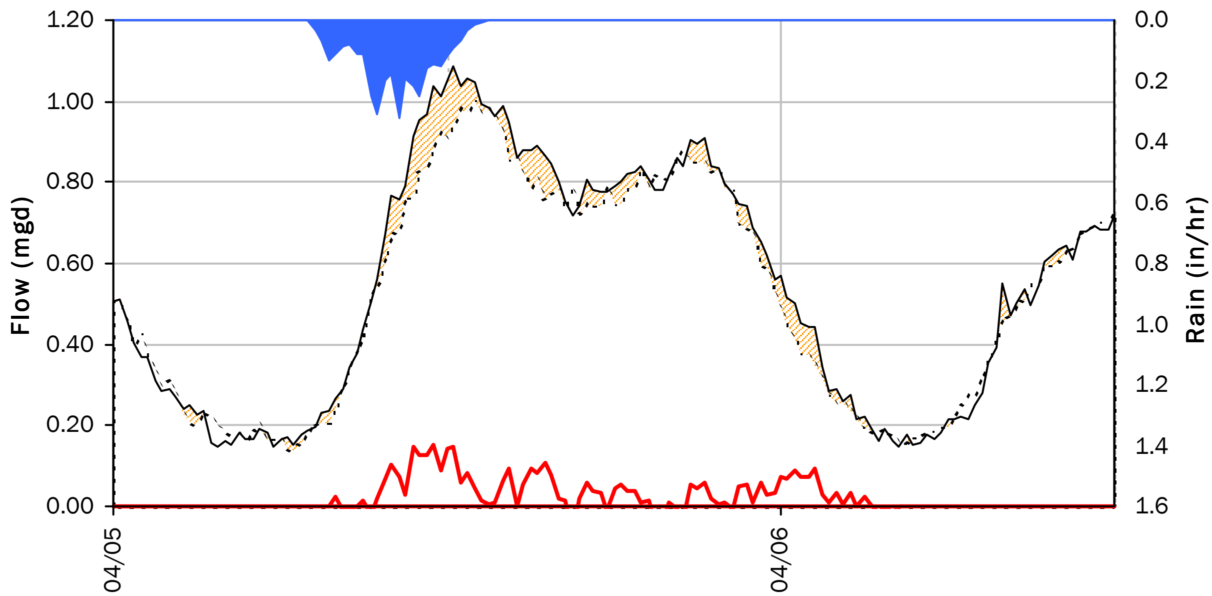
Pipe Diameter:	15	*inches*
Peak Measured Level:	10.2	*inches*
Peak d/D Ratio:	0.68	

FM 3
I/I Summary: Event 1

Baseline and Realtime Flows with Rainfall Data over Monitoring Period



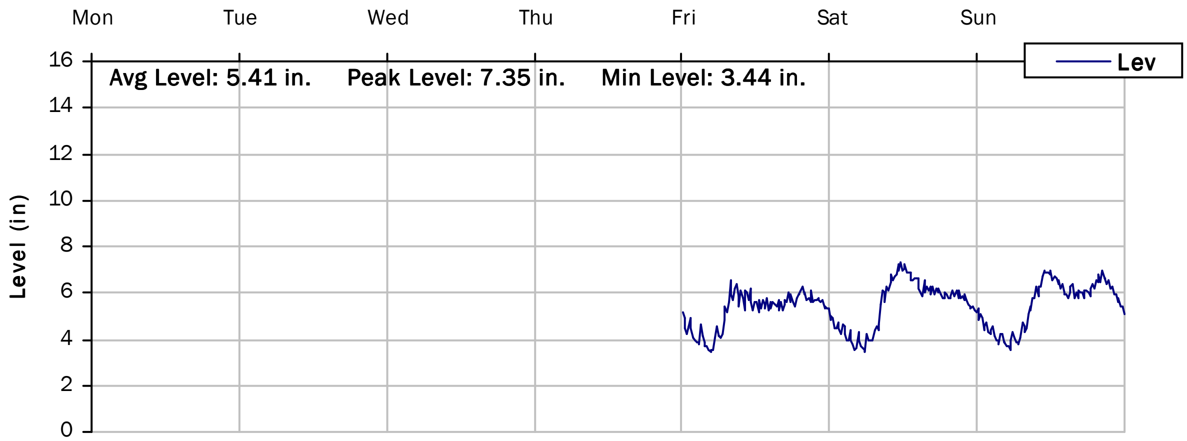
Event 1 Detail Graph

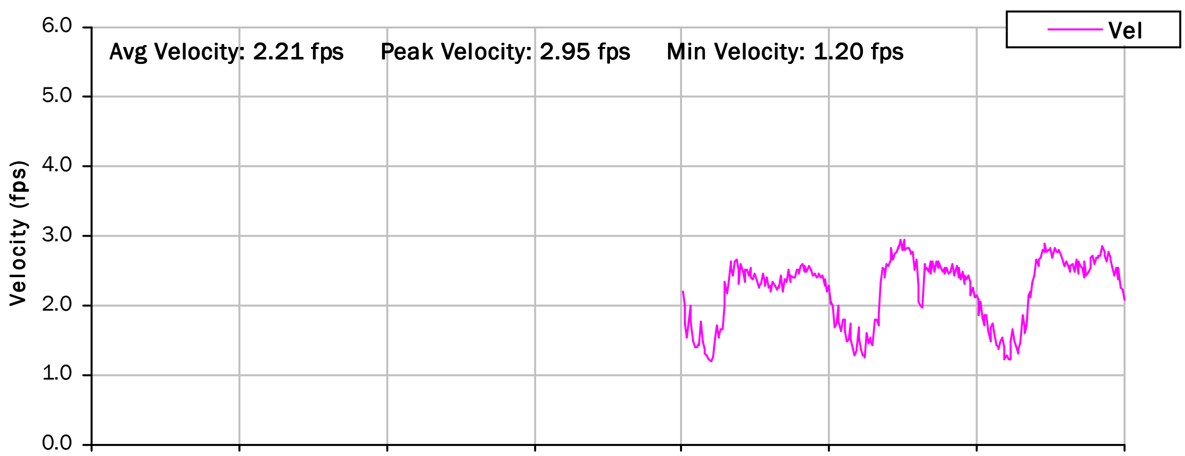


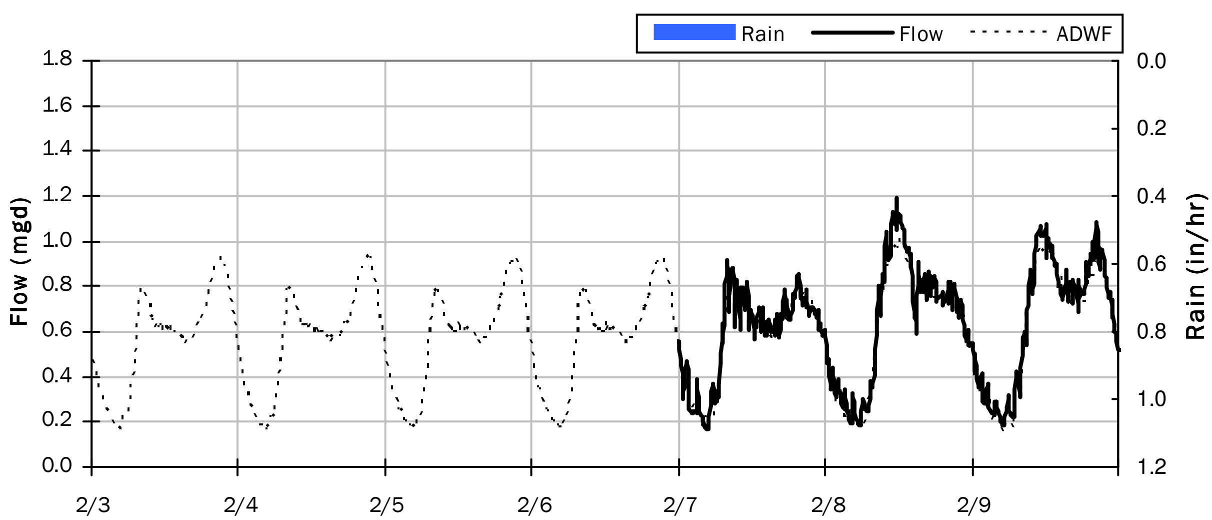
Storm Event I/I Analysis (Rain = 0.86 inches)

Capacity		**Inflow / Infiltration**	
Peak Flow:	1.09 *mgd*	**Peak I/I Rate:**	0.152 *mgd*
PF:	1.90	**Total I/I:**	34,000 *gallons*
Peak Level:	6.96 *in*		
d/D Ratio:	0.46		

FM 3
Weekly Level, Velocity and Flow Hydrographs
2/3/2020 to 2/10/2020

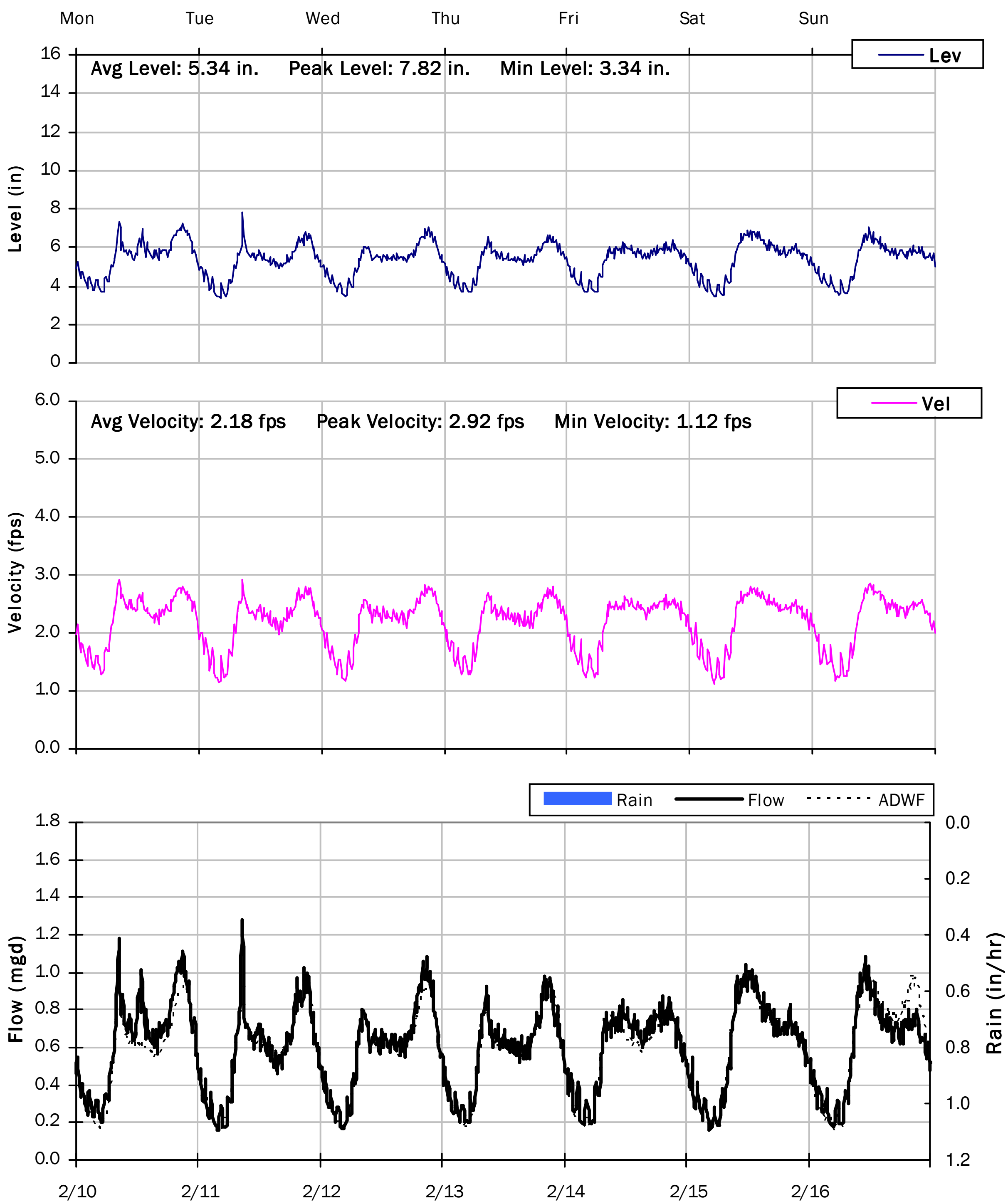






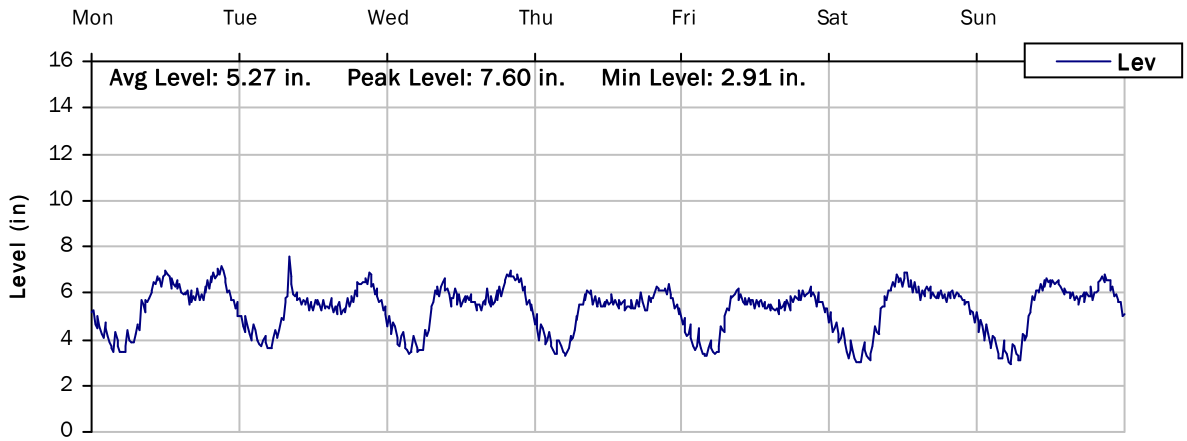
Avg Flow: 0.627 mgd Peak Flow: 1.191 mgd Min Flow: 0.172 mgd

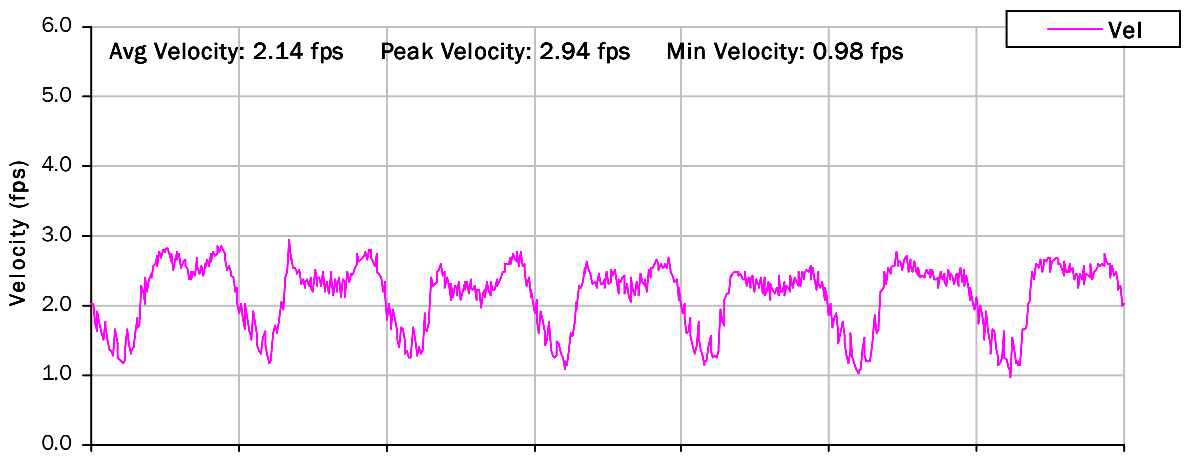
FM 3
Weekly Level, Velocity and Flow Hydrographs
2/10/2020 to 2/17/2020

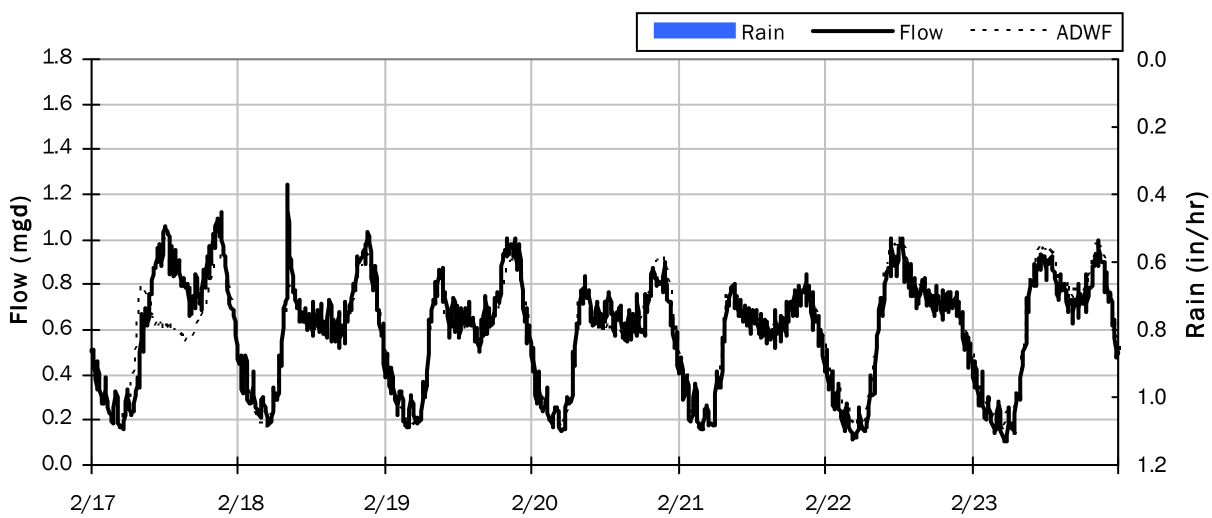


Avg Flow: 0.603 mgd Peak Flow: 1.278 mgd Min Flow: 0.159 mgd

FM 3
Weekly Level, Velocity and Flow Hydrographs
2/17/2020 to 2/24/2020

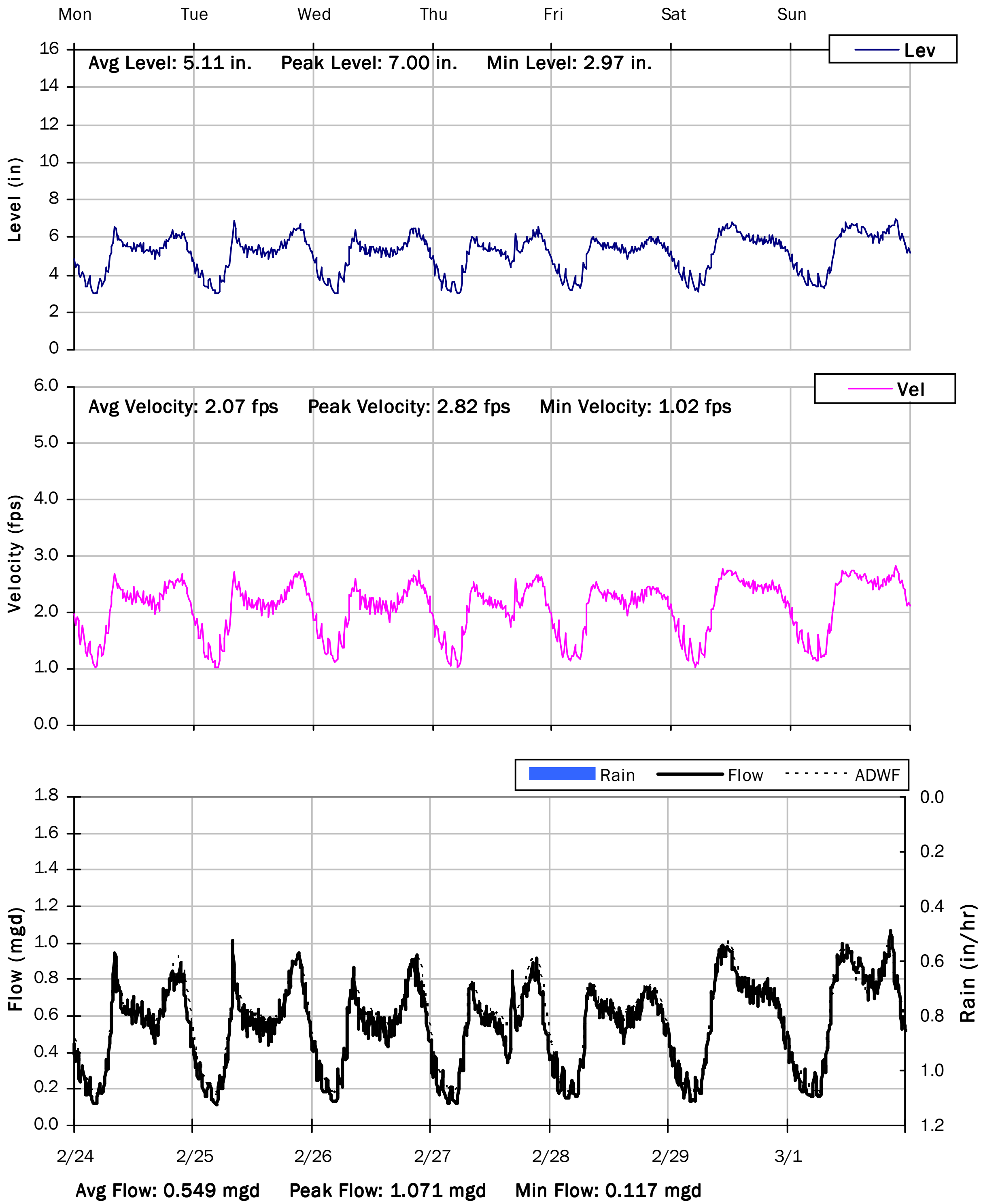




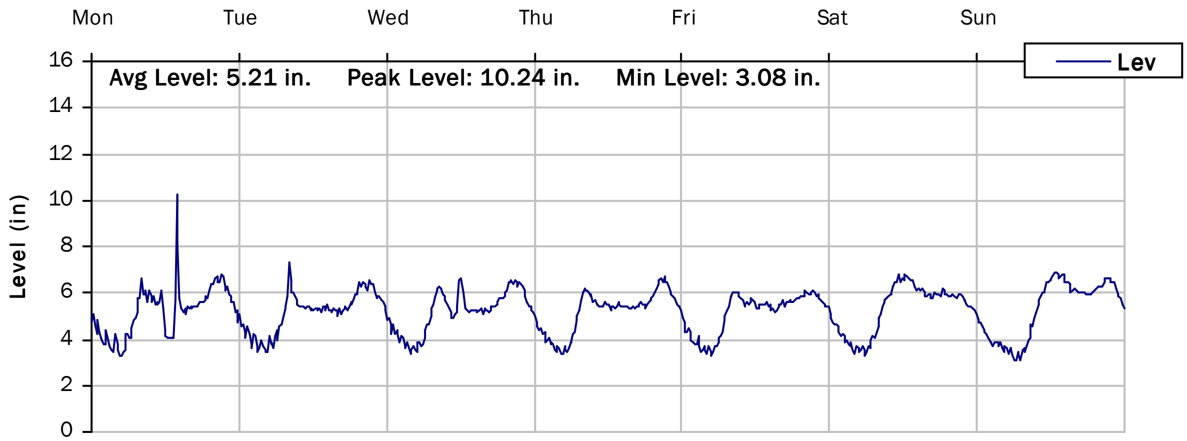


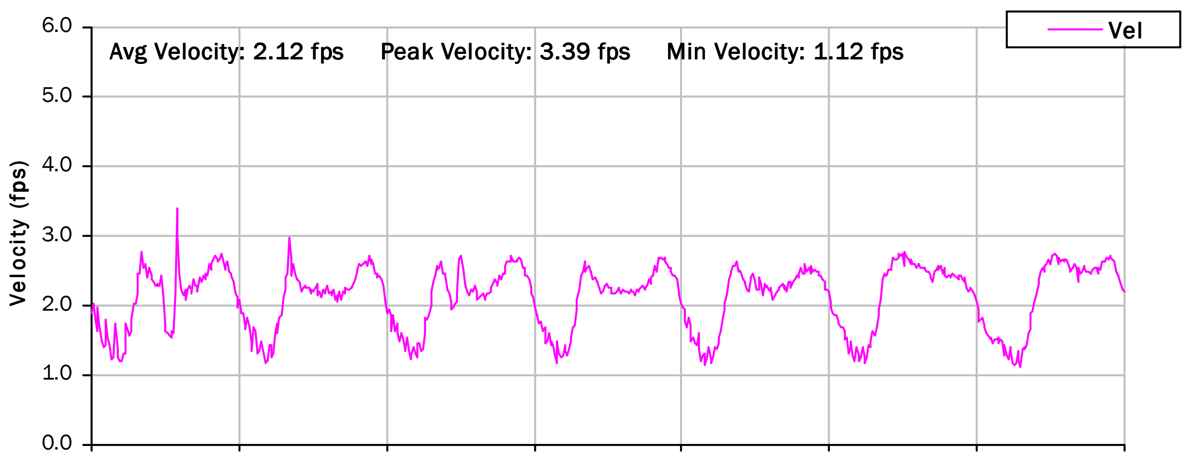
Avg Flow: 0.590 mgd **Peak Flow: 1.241 mgd** **Min Flow: 0.110 mgd**

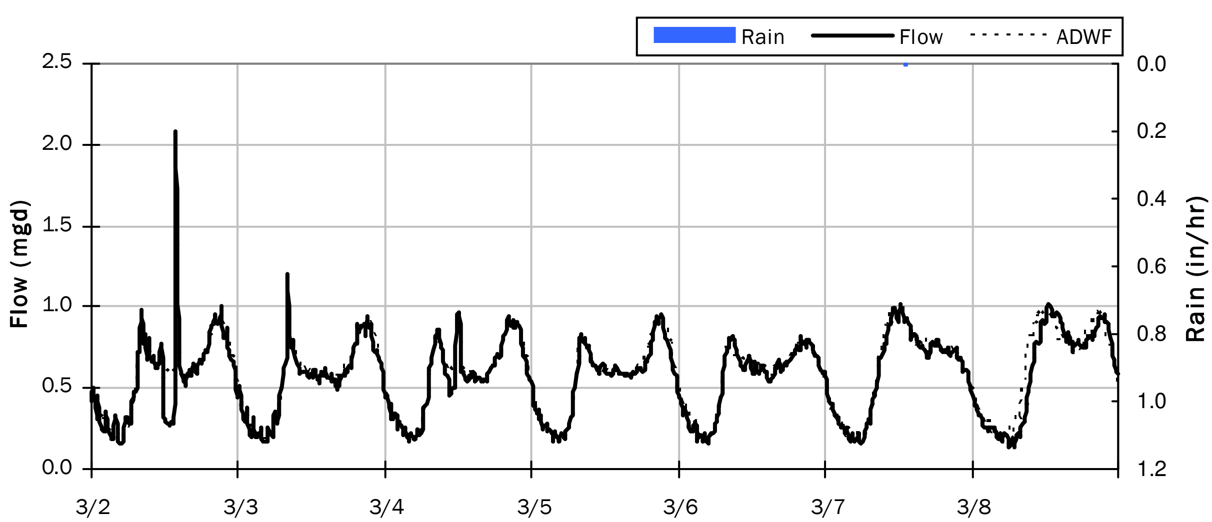
FM 3
Weekly Level, Velocity and Flow Hydrographs
2/24/2020 to 3/2/2020



FM 3
Weekly Level, Velocity and Flow Hydrographs
3/2/2020 to 3/9/2020

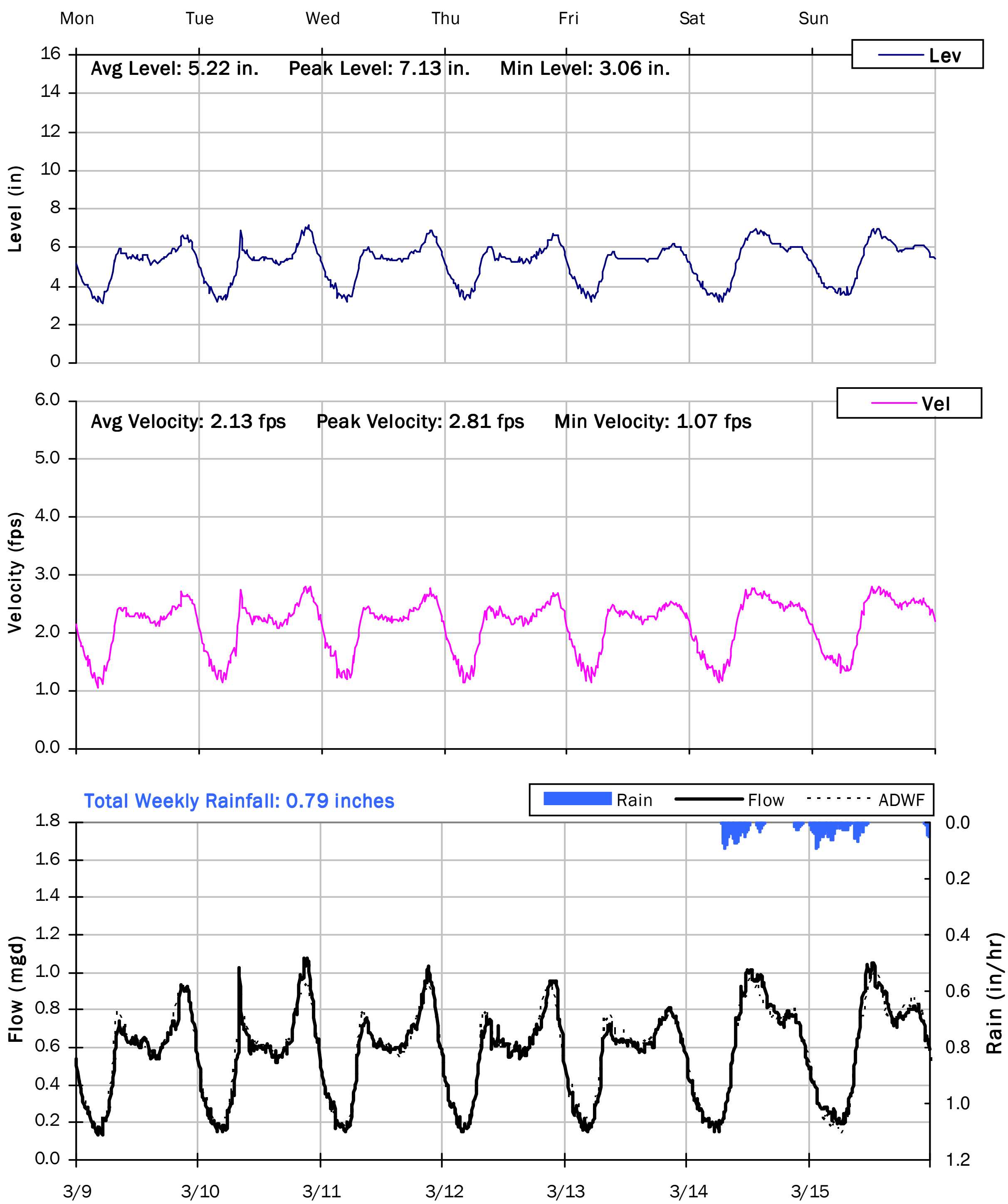






Avg Flow: 0.574 mgd **Peak Flow: 2.067 mgd** **Min Flow: 0.138 mgd**

FM 3
Weekly Level, Velocity and Flow Hydrographs
3/9/2020 to 3/16/2020

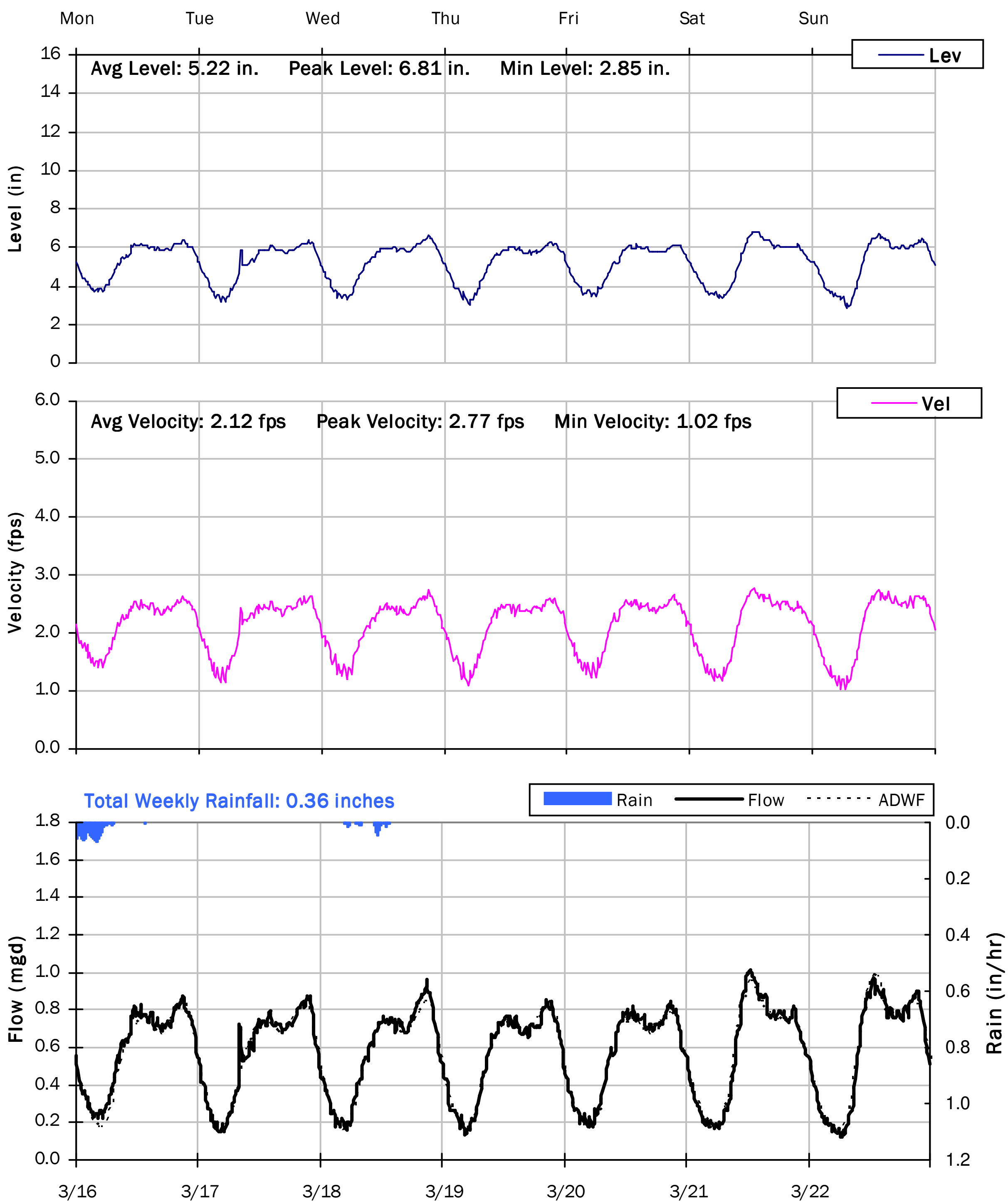


Avg Flow: 0.575 mgd **Peak Flow: 1.080 mgd** **Min Flow: 0.134 mgd**

FM 3

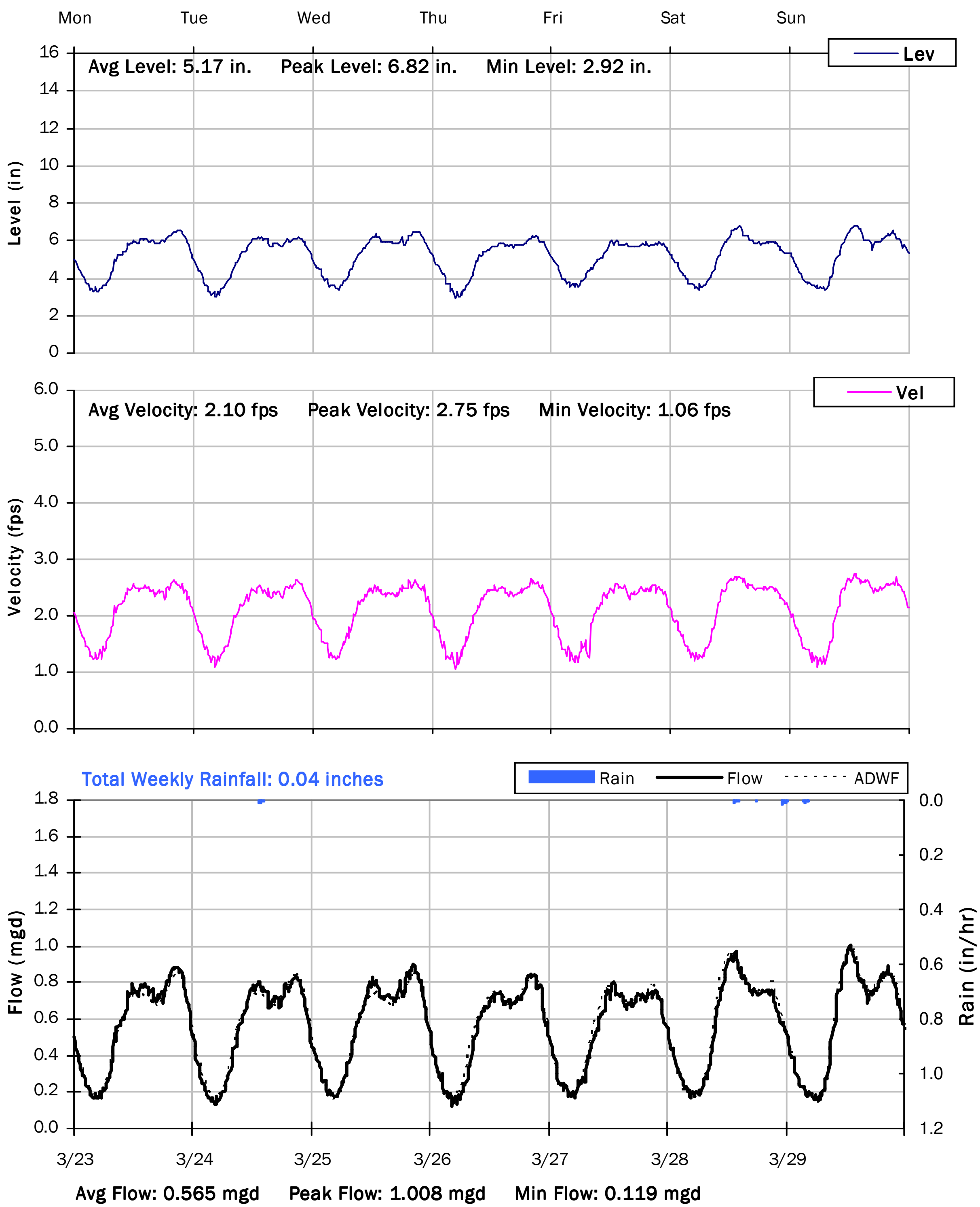
Weekly Level, Velocity and Flow Hydrographs

3/16/2020 to 3/23/2020

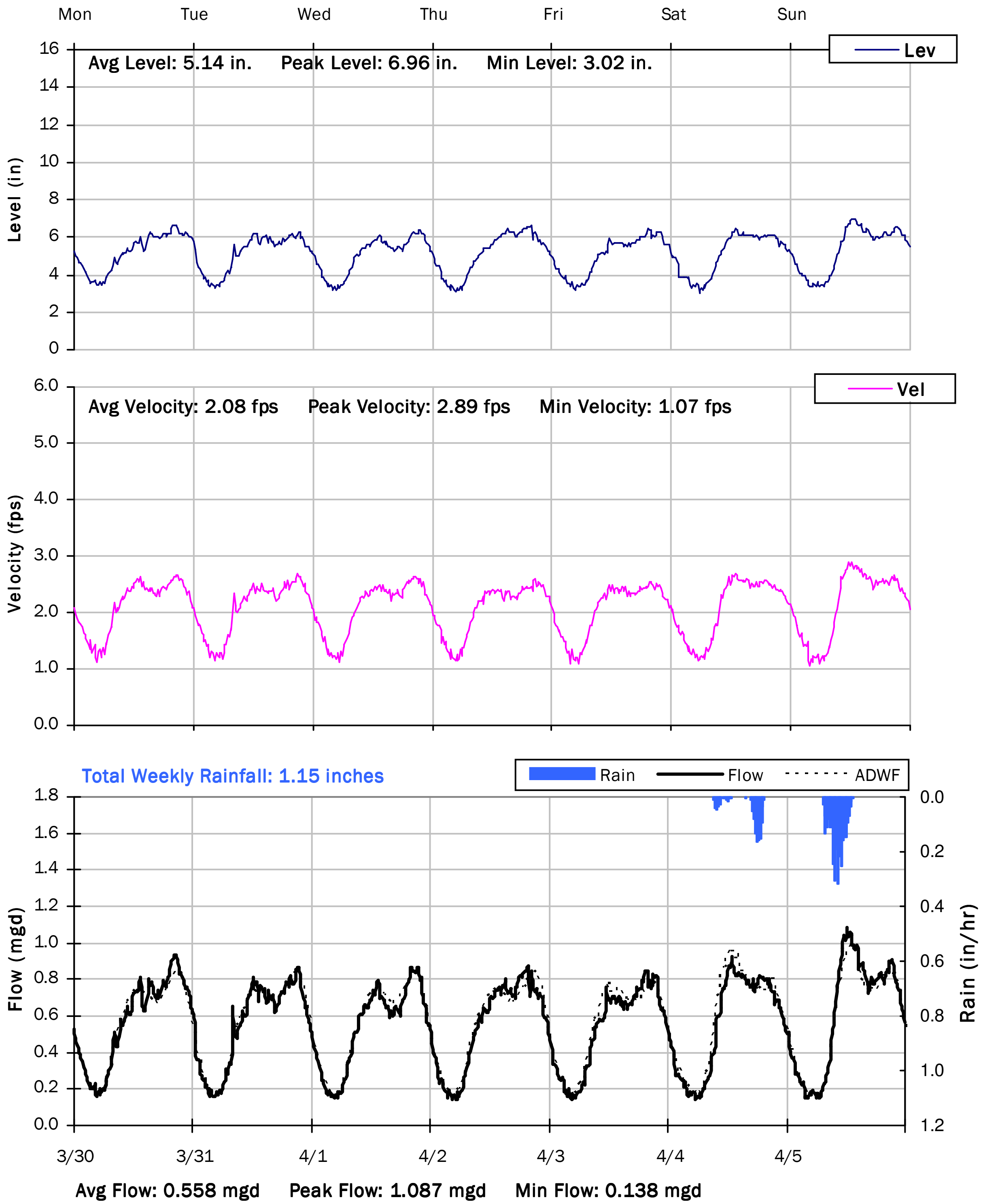


Avg Flow: 0.577 mgd **Peak Flow: 1.012 mgd** **Min Flow: 0.120 mgd**

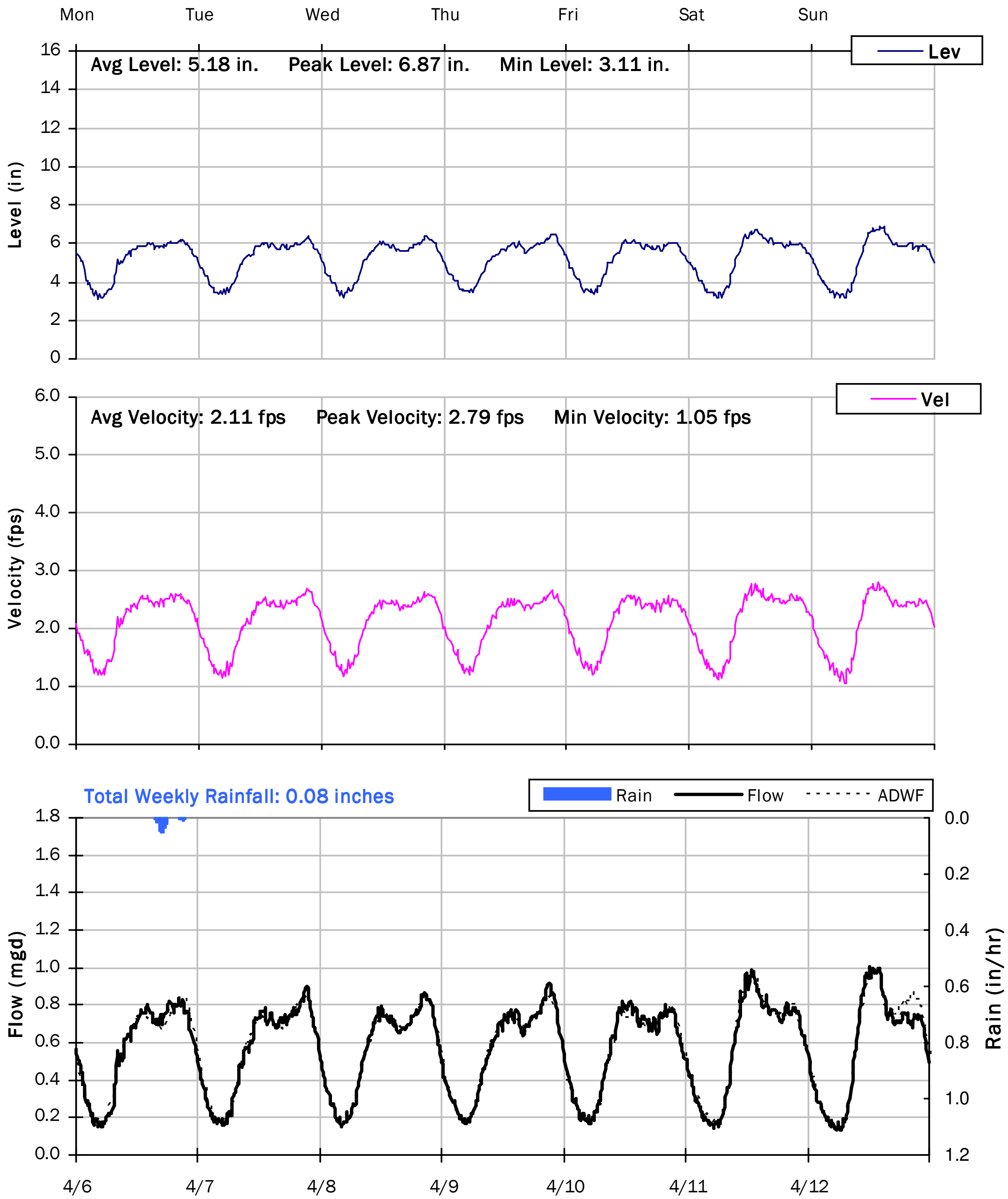
FM 3
Weekly Level, Velocity and Flow Hydrographs
3/23/2020 to 3/30/2020



FM 3
Weekly Level, Velocity and Flow Hydrographs
3/30/2020 to 4/6/2020

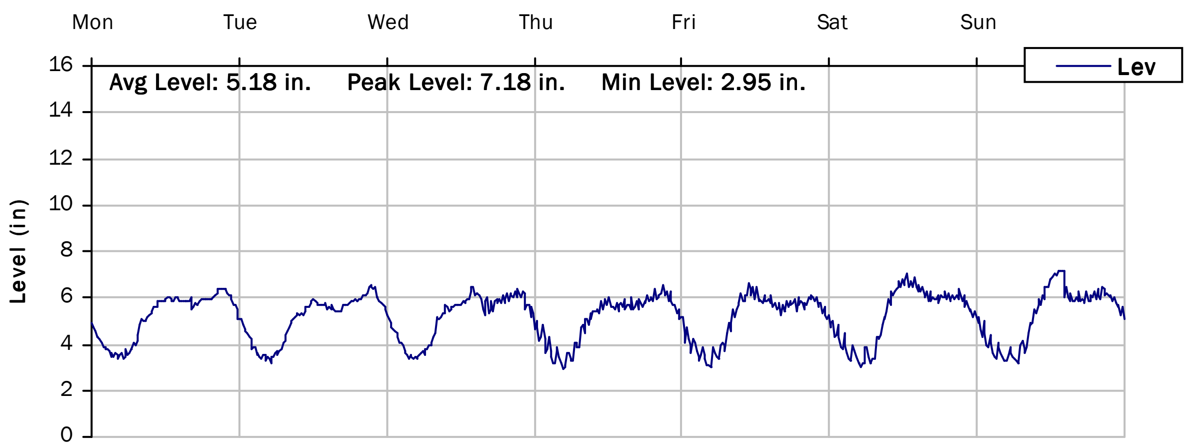


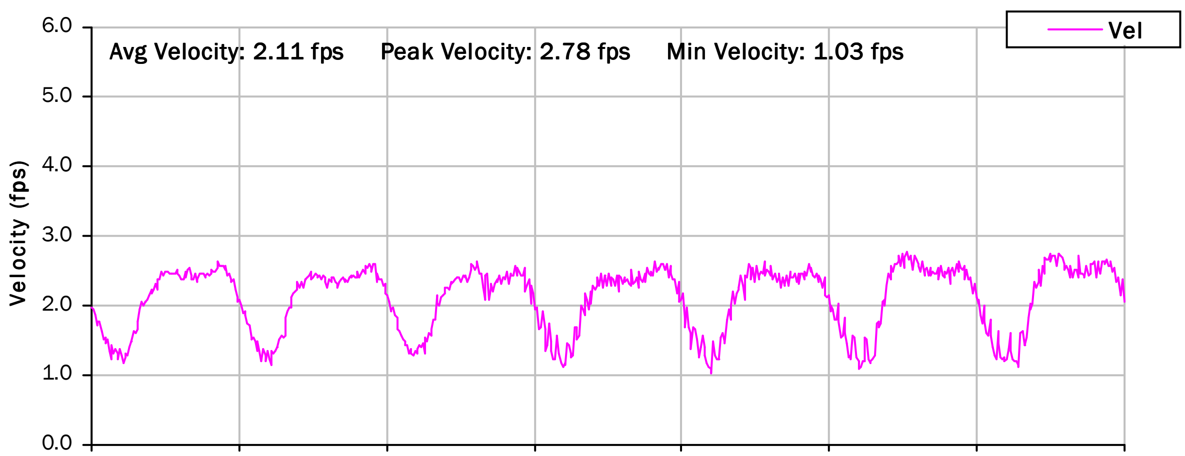
FM 3
Weekly Level, Velocity and Flow Hydrographs
4/6/2020 to 4/13/2020

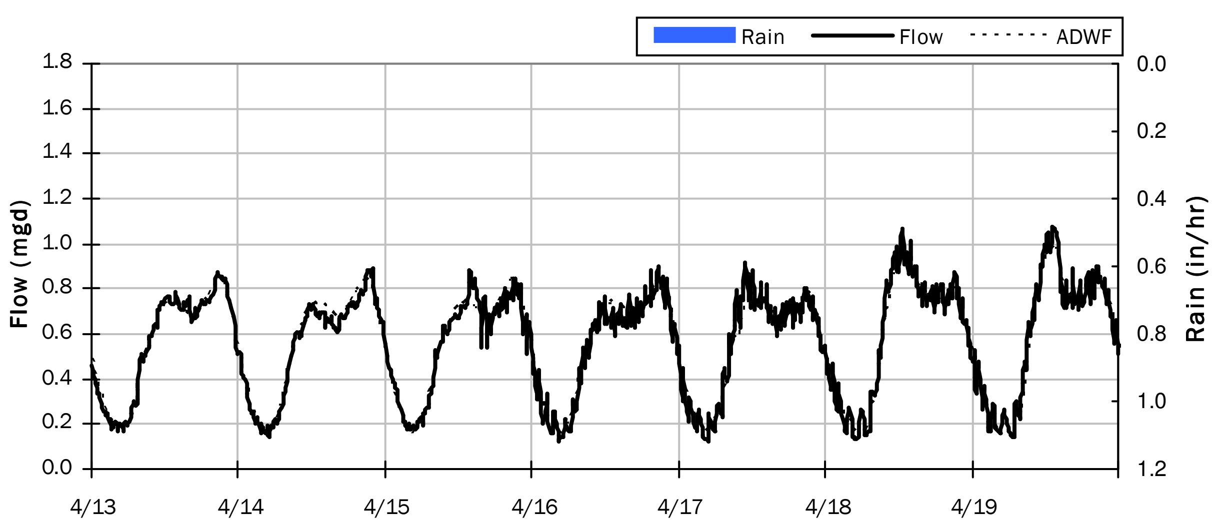


Avg Flow: 0.568 mgd   Peak Flow: 1.003 mgd   Min Flow: 0.131 mgd

FM 3
Weekly Level, Velocity and Flow Hydrographs
4/13/2020 to 4/20/2020

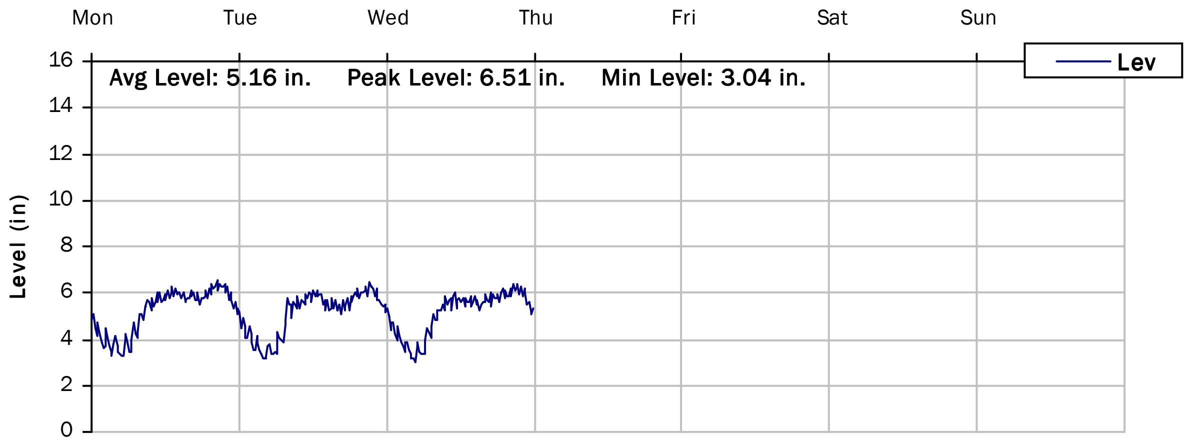


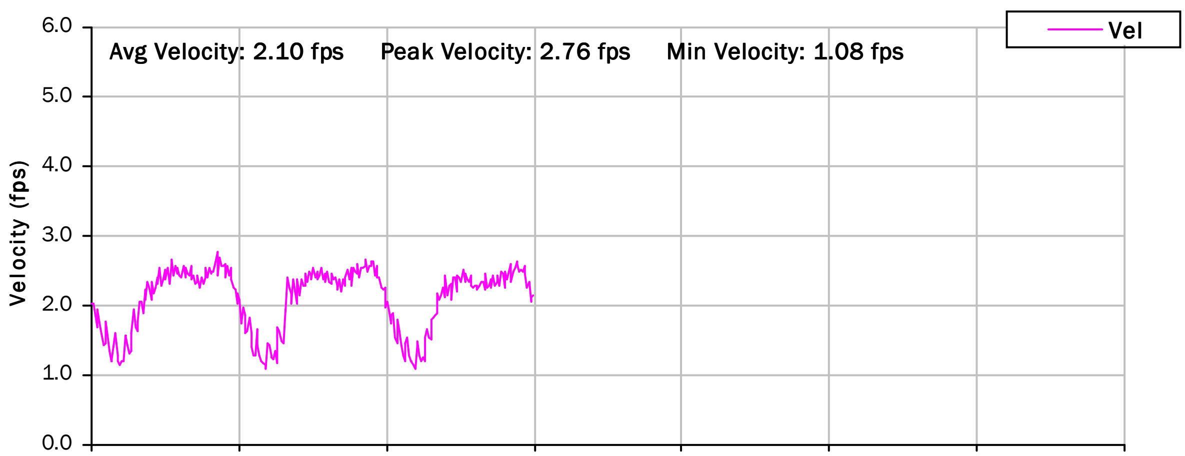


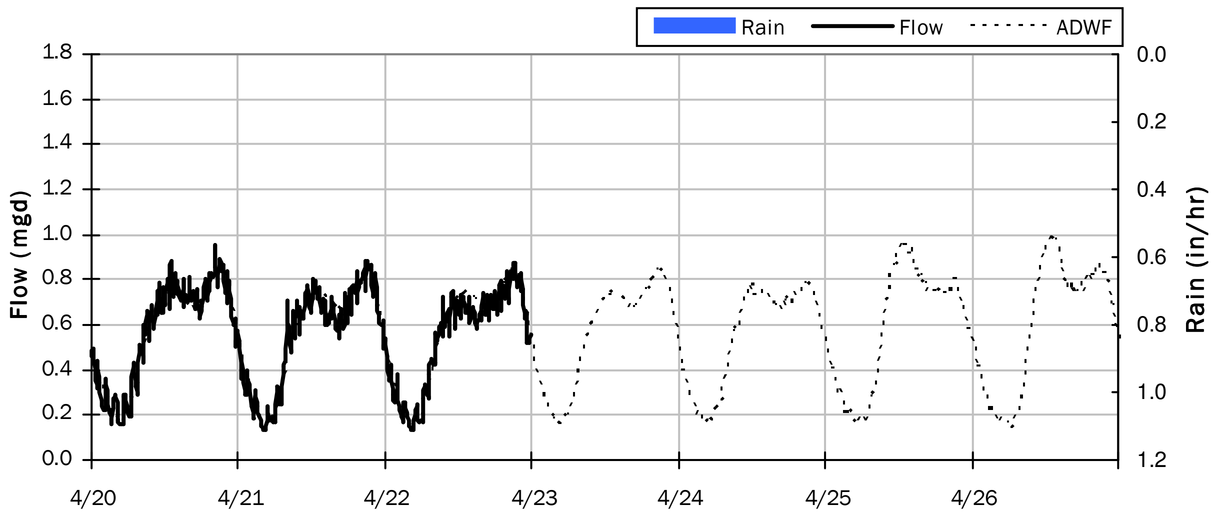


Avg Flow: 0.567 mgd **Peak Flow: 1.080 mgd** **Min Flow: 0.120 mgd**

FM 3
Weekly Level, Velocity and Flow Hydrographs
4/20/2020 to 4/27/2020







Avg Flow: 0.558 mgd **Peak Flow: 0.951 mgd** **Min Flow: 0.129 mgd**

Appendix B PREVIOUS CITY OF DIXON SEWER STUDIES

SANITARY SEWER MASTER PLAN REPORT
ADDENDUM

SOUTHWEST DIXON SPECIFIC PLAN

Prepared for
Southwest Dixon Builder Group

Prepared by
Carlson, Barbee & Gibson, Inc.
6111 Bollinger Canyon Road, Suite 150
San Ramon, CA 94583

February 2007

cbg
Carlson, Barbee & Gibson, Inc.
CIVIL ENGINEERS • SURVEYORS • PLANNERS

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

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LIST OF FIGURES

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

1 – Introduction & Purpose

This report is an addendum to the Sanitary Sewer Master Plan Report prepared by Nolte Associates, Inc. dated August 2005. This report is consistent with the underlying principles and sanitary sewer system design concept presented in the August 2005 report, which was reviewed by the City Engineering Department and was a supporting document to the approved Southwest Dixon Specific Plan (SWDSP).

The proposed sanitary sewer system provides capacity for the flows generated by the various land uses within the SWDSP areas along with off-site flows for surrounding future and existing development areas. The proposed land uses within the approved SWDSP areas include commercial, employment, residential and public uses such as parks and a fire station. The off-site flows collected and conveyed by the proposed system include future flows from GP1, GP5 and the interception of existing flows currently conveyed through the Pitt School Road Lift Station.

The purpose of this addendum is to provide the technical background and verification for the proposed sanitary sewer system supporting the SWDSP Area. This addendum is built upon the previous August 2005 Nolte Associates, Inc report and focuses on the updates to the August 2005 report. The necessary updates to the sanitary sewer system include addressing current phasing requirements and advancements to support and align with current land planning within the SWDSP areas.

Please refer to the August 2005 report attached in Appendix 2 for detailed descriptions of background reports, existing sewer facilities of surrounding areas and design criteria.

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

2 – Sanitary Sewer System Updates

This addendum identifies the updates to the August 2005 report and verifies that the proposed system adequately conveys the flows generated by on-site and off-site land uses.

The proposed sanitary sewer system pipe network has been modified to address the Southwest Dixon Builder Group's desired infrastructure phasing. The Southwest Dixon Builder Group consists of six projects within the SWDSP areas. Please see Figure 1 depicting the six projects within SWDSP. These six projects have been granted residential housing allocations through a Development Agreement with the City of Dixon. The initial phasing of the backbone infrastructure is defined by providing the necessary improvements to support these six projects and the future development of the remaining areas within SWDSP .

The August 2005 report proposed a trunk sanitary sewer main aligned in South Parkway from Evans Road to Pitt School Road. This portion of South Parkway is outside of the six projects. Therefore, the trunk sanitary sewer main has been updated to be aligned within North Parkway which is to be constructed as part of the Phase 1 Backbone Infrastructure. The proposed sanitary sewer pipe network is depicted on Figure 2.

The six projects within SWDSP are currently processing site plans and tentative maps based upon the approved Specific Plan and Supplemental Design Guidelines. The flow generation calculations of the sanitary sewer backbone system have been updated to align with the current site plans.

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

3 – Flow Generation Analysis

The following Table 1 summarizes the flows generated by the various land uses within the SWDSP area. These flow generations are based upon the City of Dixon's current Design Criteria for sanitary sewer systems and assume a peaking factor of 2.5 for all land uses.

TABLE 1
SOUTHWEST DIXON SPECIFIC PLAN ON-SITE FLOWS

Land Use	Acres	Dwelling Units	ADWF (mgd)	PWWF (mgd)
SF (LD & MDL)	340.8	1,239	0.434	1.255
MF (MDH)	9.4	176	0.047	0.122
COM	30.7	-	0.046	0.130
E	61.0	-	0.122	0.335
PUB	10.5	-	0.016	0.045
Park	23.8	-	0.036	0.101
Fire Station	0.7	-	0.001	0.003
TOTAL	**476.9**	**1,415**	**0.701**	**1.992**

The following Table 2 summarizes the flows generated by either future or existing development areas surrounding SWDSP that are conveyed by the SWDSP sanitary sewer system. These off-site flows are consistent with the flows provided by the City of Dixon and attached in Appendix B of the August 2005 report.

TABLE 2
OFF-SITE FLOWS

	Land Use	Acres	Dwelling Units	ADWF (mgd)	PWWF (mgd)
GP1	COM	71.0	-	0.107	0.302
Pitt School Lift Station	SCHOOL	9.8	-	0.049	0.128
	COM	16.2		0.024	0.069
	SF	238.4	928	0.325	0.931
GP5	COM	16.0		0.024	0.068
	SF	59.0	236	0.083	0.236
Valley Glen	MIX	110.0		0.235	0.642
TOTAL		**520.4**	**1,164**	**0.846**	**2.376**

Figure 2 depicts the key collection nodes and the cumulative flows at these nodes. Additional information, including projected flow and invert elevations are also shown at these key nodes. Each segment of pipe is labeled for reference to the hydraulic calculations. The hydraulic calculations verifying that the proposed pipe network adequately conveys the flow generated by the on-site and off-site land uses are attached in Appendix 1.

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

4 – Flow Contribution

The following Table 3 summarizes the flow contributions for the pipe segments that convey on and off site flows. This table is intended to identify the required funding participation from each area for these pipe segments. The flows used to identify the contributions all assume a peaking factor of 2.5.

TABLE 3
SANITARY SEWER FLOW CONTRIBUTION

			GP 1		SWDSP		City (Pitt School Lift Station)		GP 5		Total	
Sewer Pipeline	Diameter (inches)	Length (feet)	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%
SS-01	27	1470	0.302	8%	1.993	53%	1.128	30%	0.304	8%	3.726	100%
SS-02	27	624	0.302	8%	1.993	53%	1.128	30%	0.304	8%	3.726	100%
SS-03	24	1984	0.302	9%	1.993	58%	1.128	33%	0.000	0%	3.422	100%
SS-04	24	1348	0.302	9%	1.818	56%	1.128	35%	0.000	0%	3.248	100%
SS-05	24	1026	0.302	10%	1.596	53%	1.128	37%	0.000	0%	3.025	100%
SS-06	15	1547	0.000	0%	0.000	0%	1.128	100%	0.000	0%	1.128	100%
SS-07	21	511	0.302	16%	1.581	84%	0.000	0%	0.000	0%	1.883	100%
SS-08	21	355	0.302	17%	1.435	83%	0.000	0%	0.000	0%	1.737	100%
SS-09	21	513	0.302	17%	1.426	83%	0.000	0%	0.000	0%	1.728	100%
SS-10	18	355	0.302	18%	1.379	82%	0.000	0%	0.000	0%	1.680	100%
SS-11	18	490	0.302	20%	1.191	80%	0.000	0%	0.000	0%	1.493	100%
SS-12	18	476	0.302	21%	1.106	79%	0.000	0%	0.000	0%	1.408	100%
SS-13	15	1194	0.302	23%	1.002	77%	0.000	0%	0.000	0%	1.304	100%
SS-14	15	659	0.302	30%	0.697	70%	0.000	0%	0.000	0%	0.999	100%
SS-15	10	1050	0.302	76%	0.096	24%	0.000	0%	0.000	0%	0.398	100%

SANITARY SEWER MASTER PLAN REPORT ADDENDUM
SOUTHWEST DIXON SPECIFIC PLAN

5 – Conclusion

This addendum verifies that the proposed SWDSP sanitary sewer system will provide a level of service consistent with the City of Dixon requirements. This addendum along with the August 2005 report generally conforms to the recommendations identified within the City of Dixon South Sewer Trunk Report by Morton & Pitalo, Inc.

The primary components of the sanitary sewer system verified in this addendum include the backbone collection system that conveys on-site and off-site flows from SWDSP, GP1, GP5 and the existing Pitt School Road Lift Station.

FIGURES

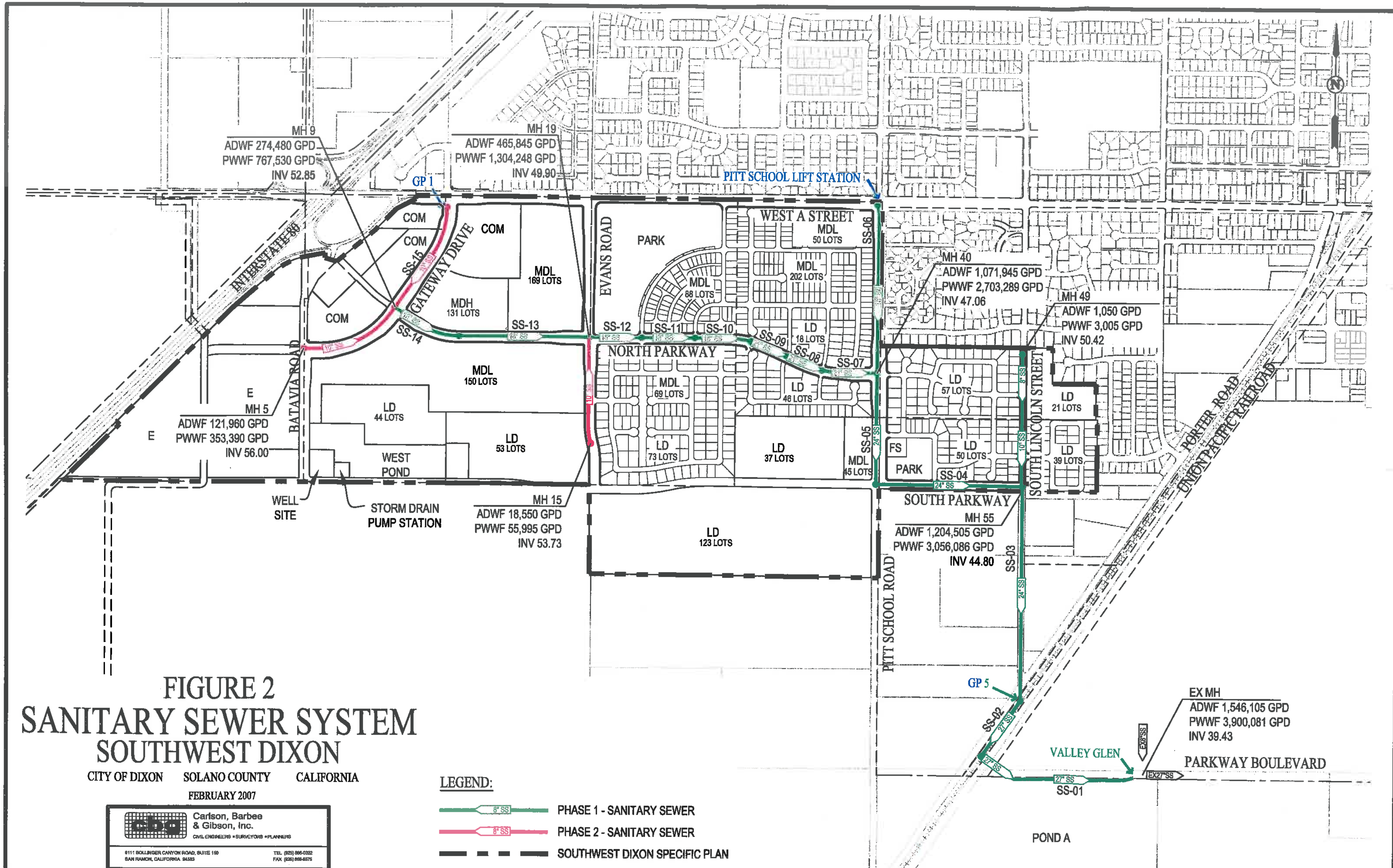

FIGURE 2
SANITARY SEWER SYSTEM
SOUTHWEST DIXON

CITY OF DIXON SOLANO COUNTY CALIFORNIA

FEBRUARY 2007

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APPENDIX 1

HYDRAULIC ANALYSIS

cbg

SOUTHWEST DIXON SPECIFIC PLAN
SANITARY SEWER DESIGN & FLOW CALCULATIONS

15-Feb-07
Job: 1311-00

	From MH	Invert (Feet)	To MH	Invert (Feet)	Area (Acres)	Gummulative Area	Land Use	Pipe Diameter	Length	Peaking Factor	Average Flow (gpd)	Qp Peak Flow	I Infiltration Flow	Qd Design Flow	Qd Total Flow (gpd)	Slope	Full Velocity	Capacity Full (gpd)	Actual Velocity (fps)	Percent Capacity
GP1	1	56.95	2	56.01	71		COM			2.5	106,500	266,250	35,500	301,750	301,750	0.0039				
					3.19		COM	10	242	2.5	4,785	11,963	1,595	13,558	315,308	0.0039	2.52	886,722	2.30	35.6
	2	56.01	3	55.06				10	242	2.5					315,308	0.0039	2.52	886,722	2.30	35.6
	3	55.06	4	54.03	9.45		COM			2.5	14,175	35,438	4,725	40,163	355,470	0.0039				
					10		COM	10	265	2.5	15,000	37,500	5,000	42,500	397,970	0.0039	2.52	886,722	2.44	44.9
	4	54.03	9	52.85				10	302	2.5					397,970	0.0039	2.52	886,722	2.44	44.9
	5	56.00	6	55.41	60.98		E	10	150	2.5	121,960	304,900	30,490	335,390	335,390	0.0039	2.52	886,722	2.34	37.8
	6	55.41	7	53.95				10	376	2.5					335,390	0.0039	2.52	886,722	2.34	37.8
	7	53.95	8	53.01	8.04		COM	10	376	2.5	12,060	30,150	4,020	34,170	369,560	0.0025	2.01	709,945	2.03	52.1
	8	53.01	9	52.85				10	63	2.5					369,560	0.0025	2.01	709,945	2.03	52.1
MH 9					162.66	162.66				2.5	274,480		81,330		767,530					
	9	52.43	10	51.89				15	237	2.5					767,530	0.0023	2.53	2,007,685	2.36	38.2
	10	51.89	11	51.25				15	275	2.5					767,530	0.0023	2.53	2,007,685	2.36	38.2
	11	51.25	12	51.03	6		MF			2.5	30,000	75,000	3,000	78,000	845,530	0.0015				
					31.32		SF			2.5	39,200	98,000	15,660	113,660	959,190	0.0015				
					9.4		PUB	15	148	2.5	14,100	35,250	4,700	39,950	999,140	0.0015	2.04	1,621,352	2.16	61.6
	12	51.03	13	50.43				15	398	2.5					999,140	0.0015	2.04	1,621,352	2.16	61.6
	13	50.18	14	49.71	49.54		SF	18	400	2.5	87,850	219,625	24,770	244,395	1,243,535	0.0012	2.06	2,358,155	2.09	52.7
	14	49.71	19	49.23				18	397	2.5					1,243,535	0.0012	2.06	2,358,155	2.09	52.7
	15	53.73	16	53.04	19.24		SF			2.5	18,550	46,375	9,620	55,995	55,995					
					1.11		PUB	10	175	2.5	1,665	4,163	555	4,718	60,713	0.0039	2.52	886,722	1.44	6.8
	16	53.04	17	52.00				10	268	2.5					60,713	0.0039	2.52	886,722	1.44	6.8
	17	52.00	18	50.95				10	270	2.5					60,713	0.0039	2.52	886,722	1.44	6.8
	18	50.95	19	49.90				10	269	2.5					60,713	0.0039	2.52	886,722	1.44	6.8
MH 19					116.61	279.27				2.5	465,845		139,635		1,304,248					
	19	49.23	20	48.99				18	203	2.5					1,304,248	0.0012	2.06	2,358,155	2.11	55.3
	20	48.99	22	48.66	20.8		PARK	18	273	2.5	31,200	78,000	10,400	88,400	1,392,648	0.0012	2.06	2,358,155	2.15	59.1
	21	49.56	22	49.49	5.68		SF	8	14	2.5	4,900	12,250	2,840	15,090	15,090	0.0052	2.50	564,716	1.08	2.7
MH 22					26.48	305.75				2.5	501,945		152,875		1,407,738					
	22	48.66	23	48.36	16.44		SF	18	245	2.5	17,500	43,750	8,220	51,970	1,459,708	0.0012	2.06	2,358,155	2.17	61.9
	23	48.36	24	48.07	5.55		SF	18	245	2.5	12,250	30,625	2,775	33,400	1,493,108	0.0012	2.06	2,358,155	2.18	63.3
	24	48.07	25	47.73	7.4		SF	18	278	2.5	12,250	30,625	3,700	34,325	1,527,433	0.0012	2.06	2,358,155	2.19	64.8
	25	47.73	26	47.45	24.61		SF	18	235	2.5	32,200	80,500	12,305	92,805	1,620,238	0.0012	2.06	2,358,155	2.22	68.7
	26	47.20	27	47.00	11.86		SF			2.5	21,700	54,250	5,930	60,180	1,680,418					
					4.78		SF	21	209	2.5	5,950	14,875	2,390	17,265	1,697,683	0.0010	2.09	3,247,181	2.11	52.3
	27	47.00	28	46.85				21	146	2.5					1,697,683	0.0010	2.09	3,247,181	2.11	52.3
	28	46.85	29	46.71	7.14		SF			2.5	10,500	26,250	3,570	29,820	1,727,503					
					3.08		SF	21	143	2.5	3,150	7,875	1,540	9,415	1,736,918	0.0010	2.09	3,247,181	2.12	53.5
	29	46.71	30	46.49				21	212	2.5					1,736,918	0.0010	2.09	3,247,181	2.12	53.5
	30	46.49	31	46.27	11.96		SF			2.5	22,050	55,125	5,980	61,105	1,798,023					
					2.22		SF	21	220	2.5	2,100	5,250	1,110	6,360	1,804,383	0.0010	2.09	3,247,181	2.14	55.6
	31	46.27	32	46.19				21	80	2.5					1,804,383	0.0010	2.09	3,247,181	2.14	55.6
	32	46.19	40	45.98	14.23		SF			2.5	22,400	56,000	7,115	63,115	1,867,498					
					5.07		SF	21	214	2.5	5,250	13,125	2,535	15,660	1,883,158	0.0010	2.09	3,247,181	2.16	58.0
Pitt School	33	50.08	34	49.86	9.81		SCHOOL			2.5	49,050	122,625	4,905	127,530	127,530					
Road					16.2		COM			2.5	24,300	60,750	8,100	68,850	196,380					
Lift Station					238.4		SF	15	88	2.5	324,800	812,000	119,200	931,200	1,127,580	0.0025	2.64	2,093,156	2.68	53.9
	34	49.86	35	49.37				15	200	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2
	35	49.37	36	48.79				15	250	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2

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SOUTHWEST DIXON SPECIFIC PLAN
SANITARY SEWER DESIGN & FLOW CALCULATIONS

15-Feb-07
Job: 1311-00

	From MH	Invert (Feet)	To MH	Invert (Feet)	Area (Acres)	Cummulative Area	Land Use	Pipe Diameter	Length	Peaking Factor	Average Flow (gpd)	Qp Peak Flow	I Infiltration Flow	Qd Design Flow	Qd Total Flow (gpd)	Slope	Full Velocity	Capacity Full (gpd)	Actual Velocity (fps)	Percent Capacity
	36	48.79	37	48.21				15	250	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2
	37	48.21	38	47.64				15	250	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2
	38	47.64	39	47.06				15	250	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2
	39	47.06	40	46.48				15	254	2.5					1,127,580	0.0023	2.53	2,007,685	2.60	56.2
	69	47.31	40	47.06	5.52		SF	8	48	2.5	4,550	11,375	2,760	14,135	14,135	0.0052	2.50	564,716	1.06	2.5
MH 40					384.27	690.02				2.2	1,071,945		345,010		2,703,289					
	40	45.73	41	45.52				24	269	2.2					2,703,289	0.0008	2.04	4,146,643	2.17	65.2
	41	45.52	42	45.30				24	266	2.2					2,703,289	0.0008	2.04	4,146,643	2.17	65.2
	42	45.30	43	45.11				24	245	2.2					2,703,289	0.0008	2.04	4,146,643	2.17	65.2
	43	45.11	44	44.91				24	247	2.2					2,703,289	0.0008	2.04	4,146,643	2.17	65.2
	44	44.71	45	44.49	3.41		MF			2.2	17,050	37,510	1,705	39,215	2,742,504					
					16.69		SF			2.2	12,950	28,490	8,345	36,835	2,779,339					
					50.32		SF	24	267	2.2	43,050	94,710	25,160	119,870	2,899,209	0.0008	2.04	4,146,643	2.20	69.9
	45	44.49	46	44.28				24	268	2.2					2,899,209	0.0008	2.04	4,146,643	2.20	69.9
	46	44.28	47	44.06				24	270	2.2					2,899,209	0.0008	2.04	4,146,643	2.20	69.9
	47	44.06	48	43.85				24	270	2.2					2,899,209	0.0008	2.04	4,146,643	2.20	69.9
	48	43.85	55	43.63				24	270	2.2					2,899,209	0.0008	2.04	4,146,643	2.20	69.9
	49	50.42	50	49.47	0.76		SF	8	184	2.5	1,050	2,625	380	3,005	3,005	0.0052	2.50	564,716	0.66	0.5
	50	49.47	51	48.51	0.53		SF	8	184	2.5	700	1,750	265	2,015	5,020	0.0052	2.50	564,716	0.78	0.9
	51	48.34	52	47.23	13.19		SF			2.5	13,650	34,125	6,595	40,720	45,740					
					7.03		SF	10	285	2.5	7,350	18,375	3,515	21,890	67,630	0.0039	2.52	886,722	1.48	7.6
	52	47.23	53	46.13				10	283	2.5					67,630	0.0039	2.52	886,722	1.48	7.6
	53	46.13	54	45.10	16.4		SF			2.5	17,500	43,750	8,200	51,950	119,580					
					3		PARK			2.5	4,500	11,250	1,500	12,750	132,330					
					0.74		FS			2.5	1,110	2,775	370	3,145	135,475					
					10.26		SF	10	264	2.5	13,650	34,125	5,130	39,255	174,730	0.0039	2.52	886,722	1.95	19.7
	54	44.90	55	44.80				10	26	2.5					174,730	0.0039	2.52	886,722	1.95	19.7
MH 55					122.33	812.35				2.2	1,204,505		406,175		3,056,086					
	55	43.43	56	43.08				24	384	2.2					3,056,086	0.0009	2.17	4,398,179	2.34	69.5
	56	43.08	57	42.72				24	400	2.2					3,056,086	0.0009	2.17	4,398,179	2.34	69.5
	57	42.72	58	42.36				24	400	2.2					3,056,086	0.0009	2.17	4,398,179	2.34	69.5
	58	42.36	59	42.00				24	400	2.2					3,056,086	0.0009	2.17	4,398,179	2.34	69.5
	59	42.00	60	41.64				24	400	2.2					3,056,086	0.0009	2.17	4,398,179	2.34	69.5
	60	41.39	61	41.19				27	284	2.2					3,056,086	0.0007	2.07	5,310,157	2.13	57.6
	61	41.19	62	40.95				27	340	2.2					3,056,086	0.0007	2.07	5,310,157	2.13	57.6
GP5	62	40.75	63	40.50	59		SF			2.2	82,600	181,720	29,500	211,220	3,267,306					
					16		COM	27	361	2.2	24,000	52,800	8,000	60,800	3,328,106	0.0007	2.07	5,310,157	2.18	62.7
	63	40.30	64	40.06				27	343	2.2					3,328,106	0.0007	2.07	5,310,157	2.18	62.7
	64	40.06	65	39.83				27	330	2.2					3,328,106	0.0007	2.07	5,310,157	2.18	62.7
	65	39.83	66	39.58				27	354	2.2					3,328,106	0.0007	2.07	5,310,157	2.18	62.7
	66	39.58	EX STUB	39.52				27	82	2.2					3,328,106	0.0007	2.07	5,310,157	2.18	62.7
Valley Glen	EX STUB	39.52	EX MH	39.43	109.95		MIX	27	86	2.2	235,000	517,000	54,975	571,975	3,900,081	0.0010	2.47	6,346,852	2.59	61.4
TOTAL					184.95	997.30				2.2	1,546,105		498,650		3,900,081					

APPENDIX 2

SANITARY SEWER MASTER PLAN REPORT
NOLTE ASSOCIATES, INC.
AUGUST 2005

DRAFT

Sanitary Sewer Master Plan Report

for

The Southwest Dixon Specific Plan Area

Prepared for:
The Southwest Dixon Landowners Association

Prepared by:
Nolte Associates, Inc.
1750 Creekside Oaks Drive, Suite 200
Sacramento, CA 95833

NOLTE
BEYOND ENGINEERING

August 2005

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Executive Summary

Introduction

The Southwest Dixon Specific Plan Area (SWDSP) incorporates about 477 acres located within the City of Dixon General Plan. The site is bordered by Interstate 80 to the west, South Lincoln Street to the east, West 'A' Street to the north, and the General Plan boundary to the south.

The current land uses are primarily agricultural. The proposed land uses include low, medium, and high-density residential, commercial, industrial (E), and public uses such as parks.

Purpose

The purpose of this Sanitary Sewer Master Plan Report is to:

- Identify existing sewer facilities and predict generated flows.
- Determine an efficient and cost effective sewer system that supports the proposed development within the SWDSP Area and integrates with existing systems outside of the Project area.

Objectives

The objectives of this Sanitary Sewer Master Plan Report are:

- To provide a plan for a gravity sewer system to convey sewer flows generated by the SWDSP Area.
- To provide a plan for infrastructure to accept offsite sewer where required.
- To provide a plan for infrastructure to convey sewer flows offsite to regional facilities.

Background

The Pre-Design Report for the City of Dixon South Sanitary Sewer Trunk by Morton & Pitalo (M&P), dated August 3, 1999, provided the background for the existing sewer infrastructure in the vicinity of the SWDSP Area. This Report identified problems with the current Pitt School Road Lift Station and the downstream gravity line and proposed improvement alternatives. It recommended placing the East-West Connector south along Pitt School Road, south of the Lift Station, then east along the south border of the Valley Glen Project to connect to the South Dixon Sewer Trunk. The South Dixon Sewer Trunk would convey the flows to the City of Dixon Wastewater Treatment Plant.

City has requested that the SWDSP to provide for sewer service to General Plan Area 1 (GP1) which is comprised of 71± acres zoned for commercial located north of A Street and west of Interstate 80. Flows form GP1 will enter SWDSP at Gateway Dr. and West A Street and will flow through the internal backbone system. Capacity to serve General Plan Area 5 (GP5) is also provided for in the flow calculations for SWDSP. GP5 is comprised of 75± acres of residential and commercial and will enter the system just before it crosses the railroad track at Porter Road.

Proposed Infrastructure

The proposed sewer infrastructure to support the SWDSP Area primarily consists of a backbone collection system within the project and infrastructure to convey offsite flows, thus allowing abandonment of the Pitt School Road Lift Station.

The East-West Connector will extend from the intersection of West 'A' Street and Pitt School Road, south along Pitt School Road to the intersection of Pitt School and Southwest Parkway. The proposed sewer system from lands within the SWDSP west of Pitt school will tie to the east-west connector at two locations, the intersection of North Parkway and Pitt School Road and Southwest Parkway and Pitt School Road. The Connector will then extend east within Southwest Parkway to South Lincoln Street, then turn south along South Lincoln to Porter Road, then turn southwest along Porter Road to a future intersection with Parkway (south border of Valley Glen project), then east along Parkway to connect to the recently constructed East/West connector at South First Street.

Total flow from the SWDSP Area, GP1, relief to the Pitt School Road Lift Station, and GP5 is approximately 1.3 million gallons per day (mgd) average dry-weather flow (ADWF) or 3.1 million gallons per day (mgd) peak wet-weather flow (PWWF). At manhole MH01 (see Figure 1), the Valley Glen project will contribute 0.646 mgd PWWF to the East-West Connector according to flow calculations provided by City staff which have been included in Appendix B.

Sanitary Sewer Master Plan

Southwest Dixon Specific Plan

1 - Introduction

This report provides the technical background for the Sanitary Sewer Master Plan in support of the Southwest Dixon Specific Plan (SWDSP). The SWDSP Area is located within the southwestern Dixon City Limits, west of I-80 and south of West 'A' Street (see Figure 1). The SWDSP Area incorporates about 477 acres of land that is currently being used for agricultural purposes. Proposed land uses within the SWDSP Area include commercial, industrial, residential, and public uses such as parks. Surrounding land uses are similar to proposed land use within the SWDSP Area. Land use to the south is primarily agricultural. To the north is commercial and residential. To the east is residential and to the west is highway and agricultural. Currently, there are a few residences with barns or equipment workshops and storage, and two businesses along Batavia Road.

There is currently no sewer infrastructure in place within the SWDSP Area.

Existing offsite sewer infrastructure in West 'A' Street flows east through the Pitt School Road lift station and force main and serves the West 'A' Street Assessment District (WASAD) north of the SWDSP Area.

Future offsite flows from GP1 will flow into the SWDSP at Gateway Drive and West A Street. Future flows from GP5 will enter to the East-West Connector at Porter Road and the future extension of Parkway across the Railroad tracks.

2 - Purpose

The purpose of this study is to develop a comprehensive Sanitary Sewer Master Plan for the SWDSP Area. This Study will be the basis of the design for the sanitary sewer systems that will be included within infrastructure improvement plans that will occur at a future date. This master plan identifies needed onsite sanitary sewer improvements as well as offsite improvements that are needed to tie with the City of Dixon sanitary sewer infrastructure.

3 - Scope

This Sanitary Sewer Master Plan for the SWDSP Area will accomplish the following:

- Identify existing sewer facilities and flows generated from proposed developments in the SWDSP Area.
- Analyze previous sewer studies and master plans and incorporate the findings that impact the SWDSP Area.

- Integrate recent City determinations on service shed areas.
- Determine a sanitary sewer system that provides a level of service that meets the requirements of the City of Dixon and integrates with existing systems.

This report does not address the issue of Wastewater Treatment Plant (WWTP) expansion, which is currently being planned by the City. This Plan area will contribute through City development fees to provide funding for the WWTP expansion.

4 - Master Plan Objectives

The primary objectives of this master plan are:

- To provide a plan for a gravity sewer system to convey sewer flows generated by the SWDSP Area.
- To provide a plan for infrastructure to accept offsite sewer where required.
- To provide a plan for infrastructure to convey sewer flows offsite to regional facilities.

5 - Background

5.1 General

Sanitary sewer service to the SWDSP Area was originally considered during the 1995 Specific Plan document for the area. Conceptual on-site sanitary sewer collection facilities were proposed in that document. The EIR prepared and certified for that document discussed sanitary sewer service on a programmatic level. Mitigation Measure 5.2-1.a as written in that EIR, required a "...design-level Wastewater System Master Plan." This Sanitary Sewer Master Plan Report expands on the previous analysis and provides significantly greater detail. A new environmental impact report is being prepared to identify impacts and define mitigation measures resulting from the facilities identified in this report.

The SWDSP Area sewer was considered by Morton & Pitalo (M&P) in 1999. The M&P study was conducted in support of the City of Dixon South Sewer Trunk Project. The East-West Connector was recommended to provide sewer service to the SWDSP Area and relieve the Pitt School Road Lift Station.

5.2 Existing Sewer Facilities

5.2.1 On-site Facilities

There are no on-site sewer facilities as the current primary land use is agricultural.

5.2.2 Off-site Facilities

The Pitt School Road Lift Station (at the northwest corner of Pitt School Road and West 'A' Street) is located just off-site of the northeast corner of the project. The Lift Station serves the WASAD by pumping the flow from the area to the northwest to a 15" gravity line. The 15" gravity line conveys this flow to the existing South Dixon Sewer Trunk at the Dixon May Fair. According to the M&P study, this line "would operate under pressure conditions during peak flows" if the West 'A' Street Area continues to use it.

6 - Flow Generation Analysis

6.1 Land Use Assumptions

To calculate generated flows, it was assumed that the on-site developments would be at proposed ultimate condition land use.

The Southwest Dixon Specific Plan document proposes land use areas and projected base line densities for those areas. Base line density is defined as maximum normal gross density, net of arterial and major collector streets. The Plan area target maximum number of dwelling units identified in previous plan approvals was 1,221 (1,121 single-family plus 100 multi-family). This dwelling unit yield is less than that generated by extending the various land use areas by the baseline density. A total of 1,219 single-family and 107 multi-family units result from extension of the areas and base line densities. The City has also required that an additional 144 multi-family dwelling units above the 1221 total units be provided for. Thus, for purposes of the infrastructure master plan studies, the larger number resulting from either the extension of the land use areas by the baseline density factors or the addition of 1221 plus 144 units has been used. This was done to assure adequate capacity of infrastructure should future additional dwelling units be approved by the City.

The on-site land uses are consistent with the Southwest Dixon Specific Plan Draft by Nolte Associates, Inc., dated August 2002. The Equivalent Dwelling Units per acre (EDU/acre) are based on this plan.

Additional flows from offsite areas (i.e., GP1, Pitt School Road Lift Station, and GP5) were provided by City Staff and included in the SWDSP flow calculations, see Appendix A.

6.2 Design Criteria and Constraint

The criteria used to design proposed sanitary sewer infrastructure are consistent with current City of Dixon Standards. Summarized below are the key criteria that were used.

- Manning's Equation, using n=0.013 (DS6-03.B).
- Shed area < 500 Ac., peaking factor = 2.5. 500 Ac. < Shed area < 1500 Ac., peaking factor =2.2.(DS6-03.B)
- Maximum flow at design conditions in any sewer main (10" diameter or less) shall be shall be 70% of pipe capacity. Lines 12" or larger may be designed to flow full unless direct service sewer connections are planned, in which case the 70% of pipe capacity maximum flow shall govern (DS6-03.B, DS6-08.G).
- Based on the SWDSP baseline densities, Medium Density High zoned properties were assigned 11 units per acre, Medium Density Low zoned properties were assigned 5.5 units per acre, Low Density zoned properties were assigned 3.25 units per acre.

Invert elevations in our calculations are based on the Improvement Plans for the South Dixon Sewer Trunk by Morton & Pitalo dated May 23, 2001. The benchmark on these is based on NGVD'29 which is consistent with the SWDSP master plans.

The average daily flow rates and infiltration (I+I) factors are taken from the City of Dixon Standards and are summarized in the following table.

Land Use	Average Daily Flow	I+I Factor
Single-Family	350 gpd per unit	500 gpd per Ac.
Multi-Family	5000 gpd per Ac.	500 gpd per Ac.
Commercial/Public	1500 gpd per Ac.	500 gpd per Ac.
Industrial	2000 gpd per Ac.	500 gpd per Ac.
Schools	5000 gpd per Ac.	500 gpd per Ac.

6.3 *Results*

The following table is a summary of flow generation by land use.

Land Use	Acres	Dwelling Units	ADWF (mgd)	PWWF (mgd)
LD	185.5	603	0.21	0.62
MDL	112.0	616	0.22	0.59
MDH	9.7	146	0.051	0.13
GC	20.9	-	0.03	0.09
HC	11.3	-	0.02	0.06
E	41.8	-	0.08	0.22
Park	22.5	-	0.03	0.10
Fire Station	0.5	-	0.0005	0.0013
TOTAL FLOW			0.64	1.81

The entire project can be serviced by a single backbone system flowing west to east. The project generates approximately 1.81 mgd of on-site sewage peak wet-weather flow. A gravity collection system has been designed to provide adequate service to all areas of the Specific Plan. (See Figure 1) The system depicted shows only lines within collector or larger roadways, and is sufficient to show how service is provided to each individually owned property within the Specific Plan area. Subsequent subdivision of individual properties will require planning of internal collection systems.

The new 15-inch gravity line will run from the Pitt School Road Lift Station, south in Pitt School Road to MH08. From there, an 18-inch line will convey the flow from the properties within SWDSP on either side of Pitt School Road and the Lift Station through MH07 to MH06(see Figure 1). From this point, a 24-inch line will convey the combined flow from the westerly properties of SWDSP, GP1, and the Lift Station east to South Lincoln Street (see cross section, Figure 2) where flows from the remaining easterly properties of SWDSP will be collected. The 24-inch line will continue south to Porter Road (see cross section, Fig. 3), south on Porter Road to a future intersection with Parkway Blvd. where flow from GP5 will be collected, and then along the southern border of the Valley Glen Project. A 27-inch line is being constructed by Valley Glen from Valley Glen Drive to Highway 113. A 27-inch line has been constructed to convey this flow from Highway 113 to the South Dixon Trunk Line.

An alternate alignment is proposed for the East-West Connector than that set forth by the M&P report, which runs south in Pitt School Road until it turns east to run in Parkway Blvd. on the south border of Valley Glen (see Figure 1). The new alignment extends east to South Lincoln Street as described previously. We propose this alignment for two reasons. First, the existing 84" storm drain down Pitt School Road south of SWDSP would be in jeopardy due to deep excavation for the new sewer line. Second, there are two houses to the south of SWDSP on either side of the alignment proposed by M&P. There is not enough room to fit the new infrastructure between them.

Figure 1 shows key collection nodes and the cumulative flows at those nodes. Additional summary information, including projected flow, and invert elevation are shown for those key nodes. Each manhole is numbered for reference to the hydraulic calculations. The hydraulic calculations for that pipe system are attached as Appendix A.

6.4 Flow Contribution

The new 15" gravity line extending south along Pitt School Road will carry flows from the existing service area north of West A Street (see Figure 1). Originally, as stated in the 1995 Southwest Dixon Specific Plan document by WPM Planning

Team, this extension was designed to serve the land within the WASAD. This document also recommended that this line be upsized to accommodate additional demand required by the SWDSP. One hundred percent of the total flow can be attributed to the Pitt School Road Lift Station from manhole No. 9 to manhole No. 8. From manhole No. 8 to manhole No. 7, the contribution decreases to approximately 76% of the total flow. Contribution decreases to approximately 75% from No. 7 to No. 6. From No. 4 to No. 1 the contribution decreases to 39%. From No. 1 to the City of Dixon Wastewater Treatment Plant, with the additional flow of 646,000 gpd PWWF, from the Valley Glen Project (M&P report) the contribution decreases to approximately 29%. This contribution from Valley Glen accounts for approximately 17% of the total flow. Please see the table below for a summary.

Southwest Dixon Specific Plan Area
SANITARY SEWER IMPROVEMENTS

			SWDSP		City		GP5		Valley Glen		Total	
Project Ref. No.	Diameter (in.)	Length (ft)	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%
SS-01	27	1580	1.80	46%	1.133	29%	0.304	8%	0.646	17%	3.88	100%
SS-02	24	2685	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-03	24	458	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-04	24	1869	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-06	24	1324	1.65	59%	1.133	41%	0	0%	0	0%	2.78	100%
SS-07	18	616	0.39	25%	1.133	75%	0	0%	0	0%	1.52	100%
SS-08	18	394	0.35	24%	1.133	76%	0	0%	0	0%	1.48	100%
SS-09	15	1593	0	0%	1.133	100%	0	0%	0	0%	1.13	100%

The flow used to calculate these percentages is 1,133,000 gallons per day PWWF, based on projected full build-out of the WASAD. This value was provided to Nolte by City Staff (Appendix B). A recent application submitted to the City to rezone 31.4 acres of the commercial and industrial lands not yet developed to residential, (Pheasant Run #7) was submitted and approved. All of the existing residential zoned lands in the WASAD have been developed. Due to the fact that the generation rates for low density residential and commercial/industrial zones are not significantly different, there would be no significant impact to the flow to the Pitt School Road Lift Station. The flow generated from Valley Glen was also provided by the City and is assumed to be correct based on full build out of the project. For more information on this subject, please see the Pheasant Run #7, Draft Environmental Impact Report, by LSA Associates, Inc., dated December 2000.

7 - Phasing Considerations

Phasing of sanitary sewer infrastructure normally has few options. General approaches require construction of adequate down stream infrastructure sufficient to convey increased sewer flows from developing areas. Thus, under normal considerations, the out-fall line proposed in this document would need to be constructed to Highway 113. The City has constructed the Dixon South Trunk and a portion of the east-west collector has been constructed between the railroad and Highway 113 by the Valley Glen Development.

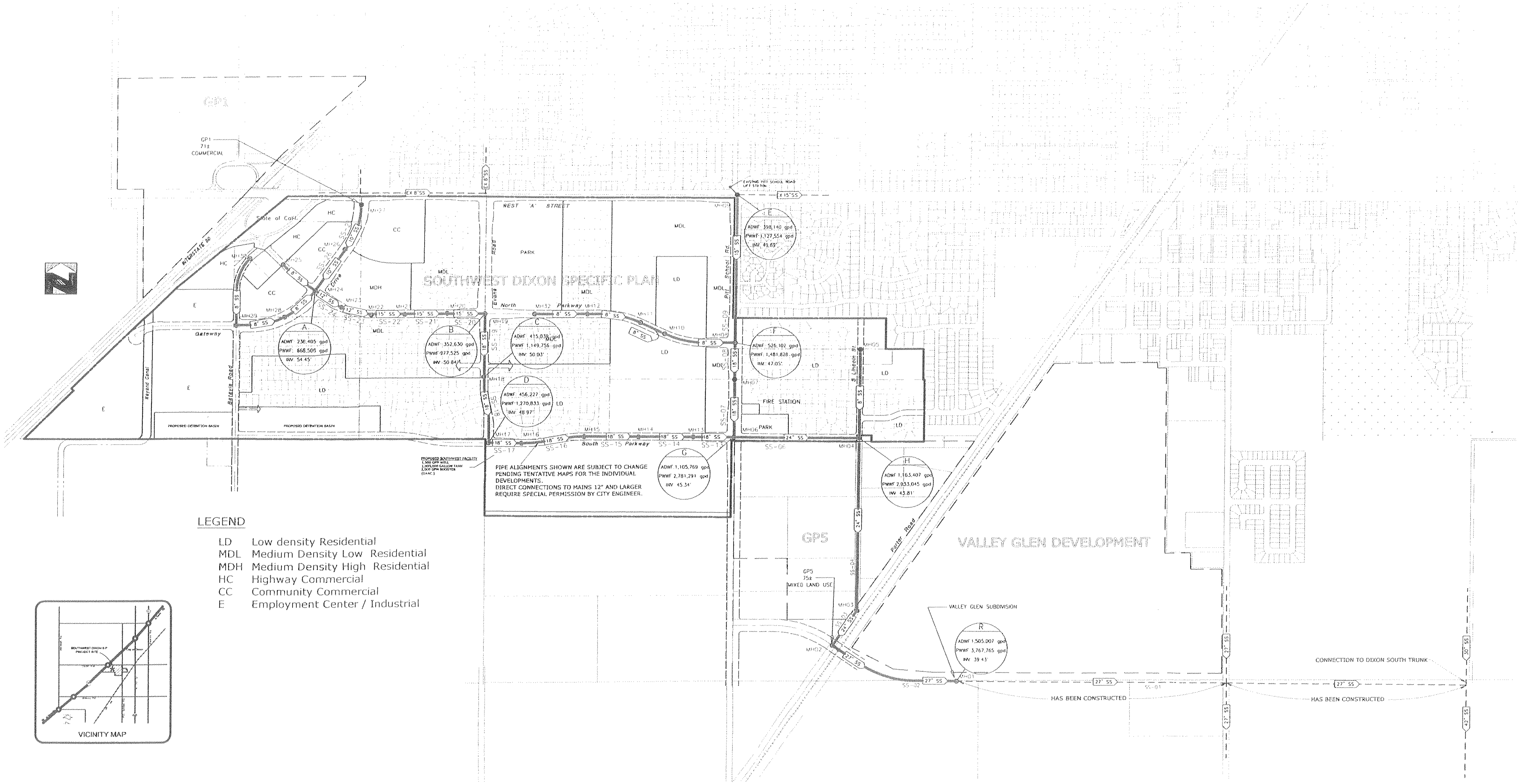
The system described under normal phasing conditions has significant construction costs. While construction of the ultimate system is desirable, initial costs may be prohibitive because of the limitations on building permits per year. A Financing Plan will be prepared which will identify the feasibility of financing full infrastructure costs based on the potential build-out rates. It may be necessary to identify creative phasing solutions to accommodate initial growth. The existing Pitt School Road Pump Station is an example of functional system phasing that allowed development to proceed without incurring prohibitive initial costs.

8 - Conclusions

This report, SWDSP Sanitary Sewer Master Plan Report, outlines a sewer system that will provide a level of service consistent with the City of Dixon Standards. The plan generally conforms to the recommendations made within the City of Dixon South Sewer Trunk Report by Morton & Pitalo, Inc.

The primary components of the plan include: a backbone collection system to convey on-site flows and infrastructure to convey flow from the GP1, Pitt School Road Lift Station, GP5, and the on-site flows to the City of Dixon Wastewater Treatment Plant.

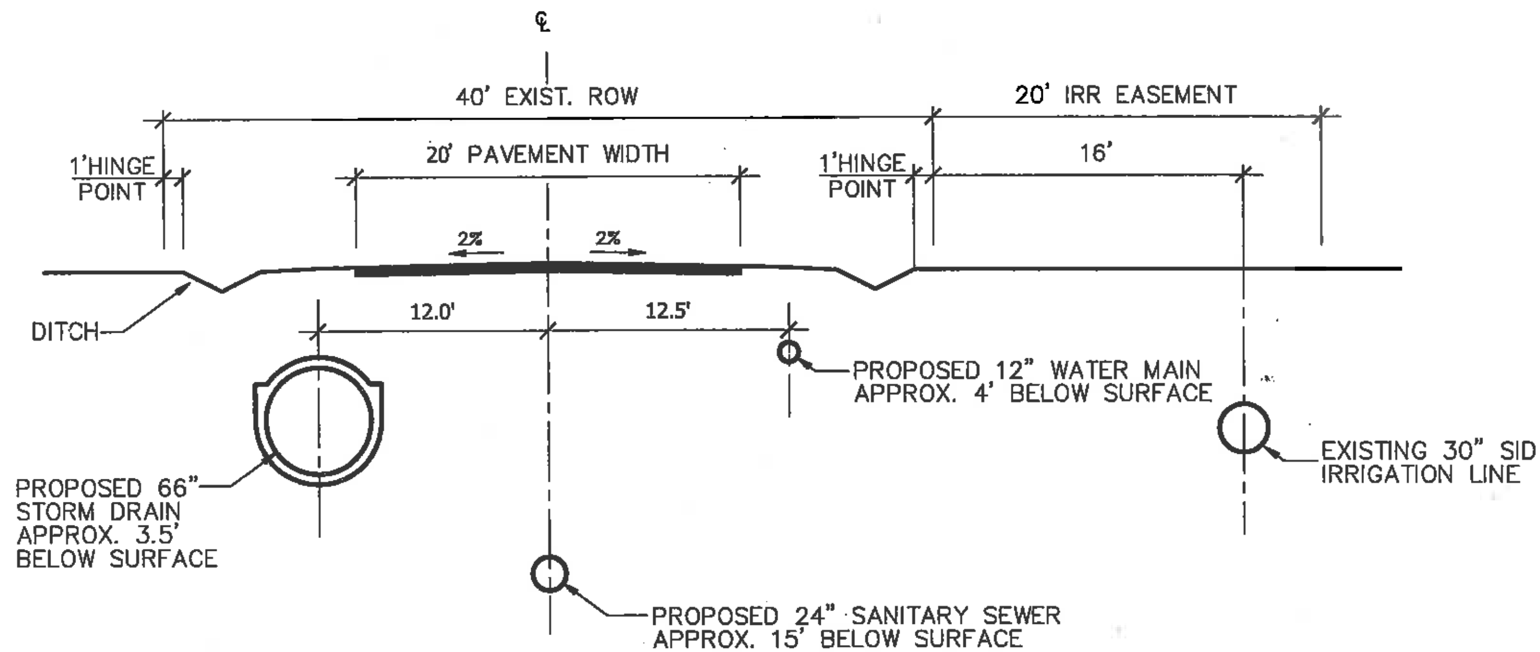
One item will require further discussion. Cost distribution for construction of the off-site infrastructure as well as infrastructure required to convey off-site flows shall be identified in the Capital Improvement Plan.



NOTE:
PIPELINE DIAMETERS AND ALIGNMENTS SUBJECT TO CHANGE.

May 2004

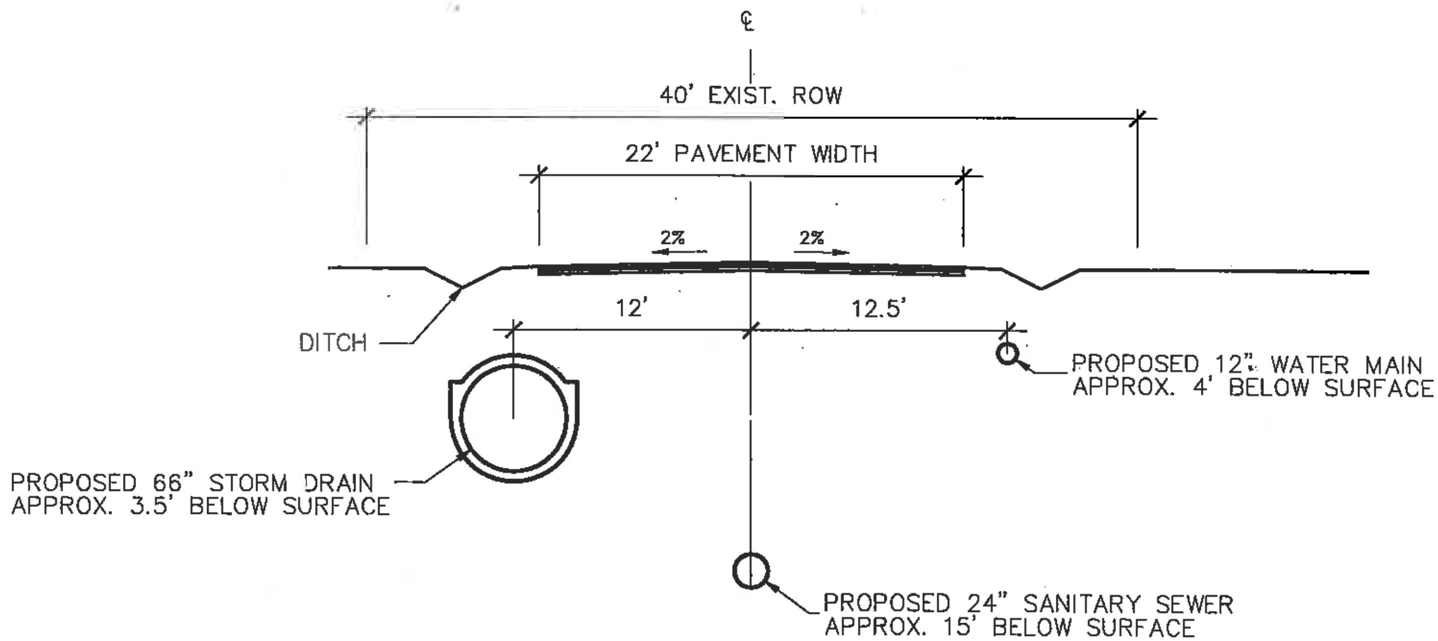
Figure 1 - Preliminary Sanitary Sewer Collection ▪ Master Plan



March 2004

Figure 2 - Preliminary South Lincoln Street Typical Section

not to scale

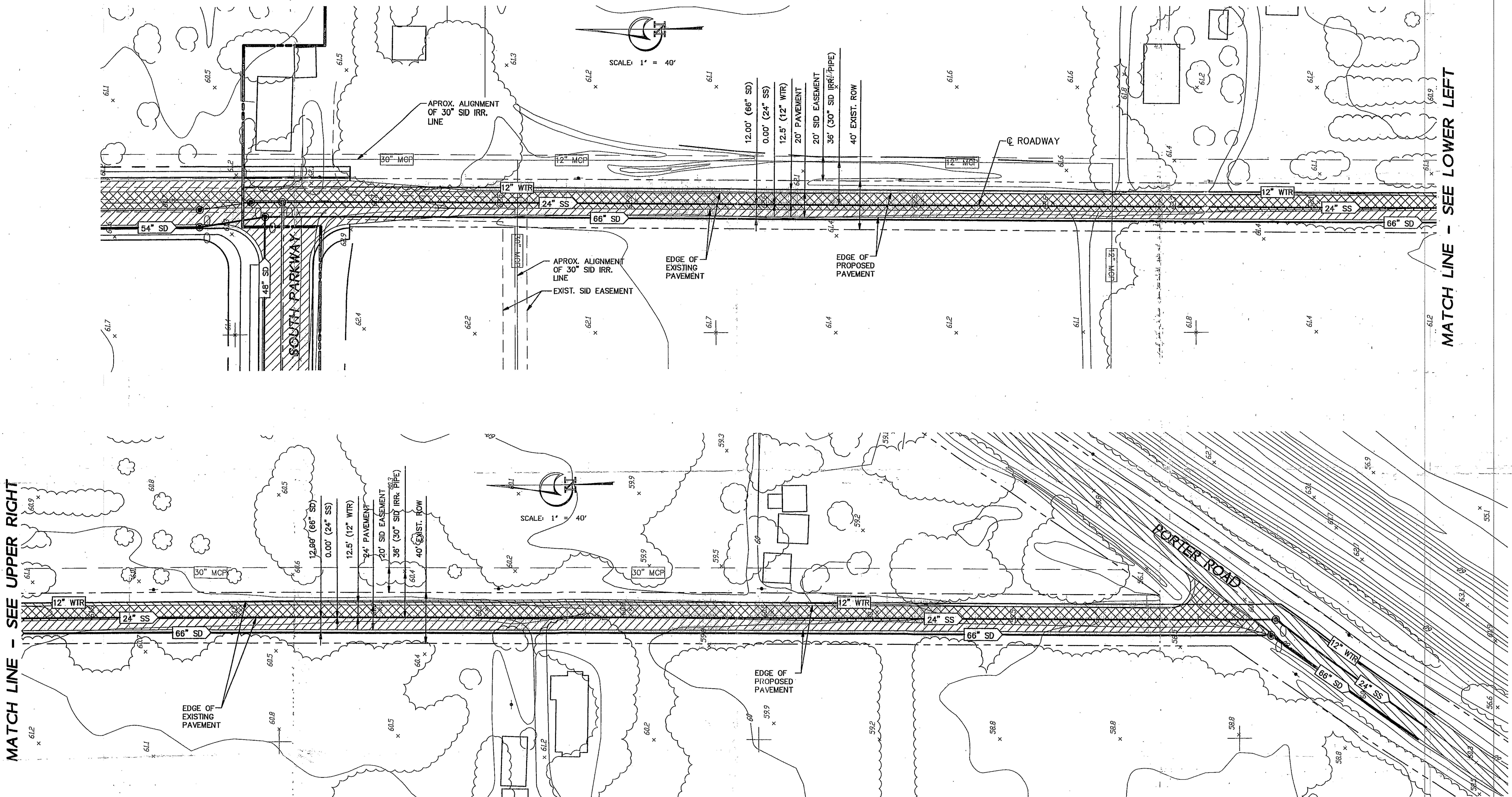


March 2004

Figure 3 - Preliminary Porter Road Typical Section

not to scale

NOLTE
BEYOND ENGINEERING



EXISTING PAVEMENT

EXISTING PAVEMENT TO BE REMOVE AND REPLACE

PROPOSED PAVEMENT

SOUTH LINCOLN STREET

Figure 4

DATE: ______ TIME: ______	NO.	BY	DATE	REVISIONS:
SERVER: ______ SERVICE: ______				
PATH: ______				
DRAWING NAME: ______				
PLOTTING VIEW: ______				
DESIGNER: ______ PROJ. MGR: ______				

CAUTION: The engineer preparing these plans will not be responsible for, or liable for, unauthorized changes to or uses of these

NOLTE
BEYOND ENGINEERING

1750 CREEKSIDE OAKS DR. SUITE 200, SACRAMENTO, CA. 95833
916.641.1500 TEL 916.641.9222 FAX www.NOLTE.COM

SOUTH LINCOLN STREET
PROPOSED UTILITY LAYOUT
SOUTHWEST DIXON SPECIFIC PLAN AREA

PREPARED FOR: SOUTHWEST DIXON LANDOWNERS

DATE SUBMITTED: 3/16/04

SHEET NUMBER 1 OF 1 SHEETS

SCALE
VERTICAL: 1"=4'
HORIZONTAL: 1"=40'

JOB NUMBER [illegible]

APPENDIX A

SWDSP

SANITARY SEWER DESIGN & FLOW CALCULATIONS

Flow per dwelling	350	Gal/Day
Infiltration (trunk line)	500	Gal/Acre/Day
Peaking factor	2.5	

DATE: 2/20/2004
BY: SMC

	FROM	INV.	TO	INV.		Cum.	Land	PIPE DIA	LENGTH	PEAK.	AVER. FLOW	Qp PEAK FLOW	I INFIL. FLOW	Qd ACCU. FLOW	Qd TOTAL FLOW	SLOPE	FULL VEL.	CAPA-CITY FULL	ACTUAL VEL	%
	MH	ELEV	MH	ELEV	Acres	Acres	Use	(IN.)	(FT.)	FACT.	(GPD)	(GPD)	(GPD)	(GPD)	(GPD)		(FPS)	(GPD)	(FPS)	CAP.
	30	62.12	29	58.16	6.36		HC	8	762	2.5	9,540	23,850	3180	27,030	27,030	0.0052	2.50	564,719	1.54	0.05
	29	58.16	28	56.30	41.84		E	8	530	2.5	83,680	209,200	20920	230,120	257,150	0.0035	2.05	463,302	2.20	0.56
	28	56.30	24	54.79	5.85		CC	8	432	2.5	8,775	21,938	2925	24,863	282,013	0.0035	2.05	463,302	2.20	0.61
71± Ac. Comm.																				
from offsite	27	57.26	26	56.03	83.31		CC	10	492	2.5	124,965	312,413	41655	354,068	354,068	0.0025	2.01	709,949	2.01	0.50
	26	56.03	24	54.62	5.04		CC	10	565	2.5	7,560	18,900	2520	21,420	375,488	0.0025	2.01	709,949	2.16	0.53
	25	57.08	24	54.79	2.59		HC	8	440	2.5	3,885	9,713	1295	11,008	11,008	0.0052	2.50	564,719	1.00	0.02
MH24					144.99	144.99				2.5	238405		72495		668,508					
	24	54.45	23	53.80	0.00			12	328	2.5	0	0	0	0	668,508	0.0020	2.03	1,032,576	2.18	0.65
	23	53.80	22	53.15	0.00			12	324	2.5	0	0	0	0	668,508	0.0020	2.03	1,032,576	2.18	0.65
	22	52.90	21	52.37	9.70		MDH	15	357	2.5	51,100	127,750	4850	132,600	801,108	0.0015	2.04	1,621,360	2.04	0.49
	21	52.37	20	51.68	17.20		MDL	15	455	2.5	33,110	82,775	8600	91,375	892,483	0.0015	2.04	1,621,360	2.19	0.55
	20	51.68	19	51.09	20.01		P	15	395	2.5	30,015	75,038	10005	85,043	977,525	0.0015	2.04	1,621,360	2.19	0.60
MH19					46.91	191.90				2.5	352630		95950		977,525					
	19	50.84	18	50.03	32.42		MDL	18	676	2.5	62,409	156,021	16210	172,231	1,149,756	0.0012	2.06	2,358,166	2.06	0.49
MH18					32.42	224.32				2.5	415039		112160		1,149,756					
	18	50.03	17	49.17	36.21		LD	18	719	2.5	41,189	102,972	18105	121,077	1,270,833	0.0012	2.06	2,358,166	2.21	0.54
MH17					36.21	260.53				2.5	456227		130265		1,270,833					
	17	48.97	16	48.54	0.00			18	353	2.5	0	0	0	0	1,270,833	0.0012	2.06	2,358,166	2.21	0.54
	16	48.54	15	47.73	0.00			18	677	2.5	0	0	0	0	1,270,833	0.0012	2.06	2,358,166	2.21	0.54
	15	47.73	14	47.04	0.00			18	572	2.5	0	0	0	0	1,270,833	0.0012	2.06	2,358,166	2.21	0.54
	14	47.04	13	46.35	0.00			18	581	2.5	0	0	0	0	1,270,833	0.0012	2.06	2,358,166	2.21	0.54
	12	55.49	11	52.41	36.70		MDL	8	592	2.5	70,648	176,619	18350	194,969	194,969	0.0052	2.50	564,719	2.50	0.35
	11	52.41	10	51.26	26.50		MDL	8	288	2.5	51,013	127,531	13250	140,781	335,750	0.0040	2.20	495,291	2.46	0.68
	10	51.26	8	47.88	5.54		LD	8	750	2.5	6,302	15,754	2770	18,524	354,274	0.0045	2.33	525,335	2.61	0.67
					238.40		LD			2.5	324,800	812,000	119200	931,200	931,200					
Pitt School					9.81		SCH			2.5	49,040	122,600	4904	127,504	1,058,704					
Road Lift	9	49.69	8	47.30	16.20		CC	15	1593	2.5	24,300	60,750	8100	68,850	1,127,554	0.0015	2.04	1,621,360	2.29	0.70
MH08					333.15	333.15				2.5	526102		166574		1,481,828					
	8	47.05	7	46.58	3.61		MDL	18	394	2.5	6,949	17,373	1805	19,178	1,501,007	0.0012	2.06	2,358,166	2.21	0.64
	7	46.58	6	45.84	3.61		MDL	18	616	2.5	6,949	17,373	1805	19,178	1,520,185	0.0012	2.06	2,358,166	2.21	0.64
	13	46.35	6	45.84	96.30		LD	18	424	2.5	109,541	273,853	48150	322,003	1,592,837	0.0012	2.06	2,358,166	2.31	0.68

SWDSP

SANITARY SEWER DESIGN & FLOW CALCULATIONS

Flow per dwelling	350	Gal/Day
Infiltration (trunk line)	500	Gal/Acre/Day
Peaking factor	2.5	

DATE: 2/20/2004
BY: SMC

	FROM MH	INV. ELEV	TO MH	INV. ELEV	Acres	Cum. Acres	Land Use	PIPE DIA (IN.)	LENGTH (FT.)	PEAK. FACT.	AVER. FLOW (GPD)	Qp PEAK FLOW (GPD)	I INFIL. FLOW (GPD)	Qd ACCU. FLOW (GPD)	Qd TOTAL FLOW (GPD)	SLOPE	FULL VEL. (FPS)	CAPA-CITY FULL (GPD)	ACTUAL VEL (FPS)	% CAP.
MH06					103.52	697.20				2.2	1105769		348599		2,781,291					
	6	45.34	4	44.01	2.42		P	24	1324	2.5	3,630	9,075	1210	10,285	2,791,576	0.00100	2.28	4,636,110	2.45	0.60
	5	50.09	4	45.15	47.48		LD	8	950	2.5	54,009	135,021	23740	158,761	158,761	0.0052	2.50	564,719	2.26	0.28
MH04					49.90	747.10				2.2	1163407		373549		2,933,045					
	4	43.81	3	41.95	0.00			24	1869	2.5	0	0	0	0	2,933,045	0.00100	2.28	4,636,110	2.45	0.63
75± Ac. Mixed	3	41.75	2	41.29	59.00		LD	24	458	2.5	82,600	206,500	29500	236,000	3,169,045	0.00100	2.28	4,636,110	2.56	0.68
from offsite	2	41.09	1	39.68	16.00		CC	24	1407	2.5	24,000	60,000	8000	68,000	3,237,045	0.00100	2.28	4,636,110	2.56	0.70
Valley Glen	1	39.43	HWY 113		109.95		Mixed	27	2858	2.5	235,000	587,500	45700	633,200	3,870,245	0.00100	2.47	6,346,882	2.65	0.61
MH01					184.95	932.04				2.2	1505007		456749		3,767,765					

Notes: 1. (9.70 Ac.)*(11DU/Ac.)=106.7 DU. To model flows for the required 146 Multi-Family units, 39.3 DUs were added to the Aver. Flow column from MH22 to MH21. 146-106.7=39.3.

2. The starting elevation of 39.43 at MH01 was taken from the Improvement Plans for Valley Glen Phase 1 - Unit 2 by Wood Rodgers dated February 2003.

3. Flow values for Valley Glen were taken from the South Master Sewer Study for Valley Glen by Wood Rodgers dated August 2003.

APPENDIX B

(28)

FACSIMILE TRANSMITTAL COVER SHEET

CITY OF DIXON
DEPT. OF PUBLIC WORKS (707) 678-7031
600 EAST A STREET, DIXON, CA 95620

DATE: June 11, 2003

TO: Nolte

ATTN: Stephen Campbell

FAX: 916-641-9222

FROM: Jim Ingram

NUMBERS OF PAGES INCLUDING COVER SHEET: 5

COMMENTS: Tables 1 through 10 are the individual and flows; Table 11 shows cumulative flow as you progress from the Pitt School Rd lift station southerly and easterly to SR 113 connection

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SENT FROM FAX NUMBER (707) 678-7039

OPERATOR: Jim

formal faxtrans pw

Table 1

Pitt School Road Lift Station: Based on Actual Lots and Areas										
SS11 E	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
SF	238.4	928	350		324,800					
Mult	0		5,000		0					
School	8.008		5,000		40,040					
Comml	18.2		1,500		27,300					
Ind	0		2,000		0					
Park	0		1,500		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	266.208				400,140	2.5	1,000,350			
I&I				500				133,104		
Total									1,133,454	1.133

Table 2

SWDSP Based on Proposed Lots and Areas										
SS10 (F)	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
SF	51	230	350		80,500					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	0		1,500		0					
Ind	0		2,000		0					
Park/Pub	17.74		1,500		26,610					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	68.74				107,110	2.5	267,775			
I&I				500				34,370		
Total									302,145	0.302

Table 3

GP 1 Based on Projected Lots and Areas										
C-Line A/Gateway	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
SF	0	0	350		0					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	71		1,500		106,500					
Ind	0		2,000		0					
Park	0		1,500		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	71				106,500	2.5	266,250			
I&I				500				35,500		
Total									301,750	0.302

Table 4

SWDSP Based on Proposed Lots and Areas										
SS9	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
SF	0	0	350		0					
Mult	3.21	32	5,000		16,050					
School	0		5,000		0					
Comml	0		1,500		0					
Ind	0		2,000		0					
Park	0		1,500		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	3.21				16,050	2.5	40,125			
I&I				500				1,605		
Total									41,730	0.042

Table 5

SWDSP Based on Proposed Lots and Areas										
SS8 G	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
SF	126.4	580	350		203,000					
Mult	51.1	128	5,000		255,500					
School	0		5,000		0					
Comml	44.4		1,500		66,600					
Ind	41.84		2,000		83,680					
Park/Pub	20.01		1,500		30,015					
Fire Sta	0		1,000		0					
Det Pond	9.3		0		0					
Sub Total	293.05				608,780	2.5	1,521,950			
I&I				500				146,525		
Total									1,668,475	1.668

Table 6

SWDSP Based on Proposed Lots and Areas										
SS7 H	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
SF	54.8	176	350		61,460					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	0		1,500		0					
Ind	0		2,000		0					
Park	2.47		1,500		3,705					
Fire Sta	0.47		1,000		470					
Det Pond	0		0		0					
Sub Total	57.74				65,635	2.5	164,088			
I&I				500				28,870	192,958	0.193
Total										

Table 7

GP 5 Based on Projected Lots and Areas										
SS8 OR SS9	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
SF	59	236	350		82,600					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	16		1,500		24,000					
Ind	0		2,000		0					
Park	0		1,500		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	75				106,600	2.5	266,500			
I&I				500				37,500	304,000	0.304
Total										

Table 8

Valley Glen Based on Approved Lots and Areas										
SS4 (A)	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
SF	12	80	350		28,000					
Mult	4.74	40	6,000		23,700					
School	0		5,000		0					
Comml	0		1,500		0					
Ind	0		2,000		0					
Park	0		1,500		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	16.74				51,700	2.5	129,250			
I&I				500				8,370	137,620	0.138
Total										

Table 9

Valley Glen Based on Approved Lots and Areas										
SS3 (B)	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
SF	69.5	410	350		143,500					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	0		1,500		0					
Ind	0		2,000		0					
Park	5.106		2,000		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	74.606				143,500	2.5	358,750			
I&I				500				37,303	396,053	0.396
Total										

Table 10

Valley Glen Based on Approved Lots and Areas										
SS2 (C)	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
SF	15.3	107	350		35,350					
Mult	0		5,000		0					
School	0		5,000		0					
Comml	3.905		1,500		5,858					
Ind	0		2,000		0					
Park	0		2,000		0					
Fire Sta	0		1,000		0					
Det Pond	0		0		0					
Sub Total	19.205				41,208	2.5	103,019			
I&I				500				9,603	112,621	0.113
Total										

Table 11

Flow at Pitt School Road Lift Station-SS11 (E)										
Existing City-Pt E	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
Res	238.40	928	350		324,800					
School	9.608		5,000		48,040					
Comml	18.20		1,500		27,300					
Sub Total	266.208				400,140	2.5	1,000,350			
I&I				500				133,104		
Total Flow @ SS11 (E) (15" Stub from Pitt School Rd Lift Station)									1,133,454	1.133
SS10 (F)										
Point F	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
Res	289.40	1158	350		405,300					
School	9.608		5,000		48,040					
Comml	18.20		1,500		27,300					
Park	17.74		2,000		35,480					
Sub Total	334.948				516,120	2.5	1,290,300			
I&I				500				167,474		
Total Flow at SS10 (F)									1,457,774	1.458
SS9 (*)										
	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
Res	289.40	1158	350		405,300					
School	9.608		5,000		48,040					
Mult	3.61		5,000		18,050					
Comml	18.20		1,500		27,300					
Park	17.74		2,000		35,480					
Sub Total	338.558				534,170	2.5	1,335,425			
I&I				500				169,279		
Total Flow @ SS9 (*)									1,504,704	1.505
SS8 (G)										
Point G	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
Res	415.80	1559	350		545,650					
Mult	54.71	126	5,000		273,550					
School	9.608		5,000		48,040					
Comml	133.60		1,500		200,400					
Ind	41.84		2,000		83,680					
Park	37.75		2,000		75,500					
Det Pond	9.3		0		0					
Sub Total	702.608				1,226,820	2.2	2,699,004			
I&I				500				351,304		
Total Flow @ SS8 (G)									3,050,308	3.050
SS7 (H)										
Point H	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow gpd	Pk Flow mgd
Res	470.60	1735	350		607,250					
Mult	54.71	126	5,000		273,550					
School	9.608		5,000		48,040					
Comml	133.60		1,500		200,400					
Ind	41.84		2,000		83,680					
Park	40.22		2,000		80,440					
Det Pond	9.3		0		0					
Fire Sta	0.47		1,500		705					
Sub Total	760.348				1,293,360	2.2	2,845,392			
I&I				500				380,174	3,225,566	3.226

Total Flow @ SS7 (H)										
GP 5 SS6 or SS5										
SS6 or SS5	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
Res	529.60	1971	350		689,850					
Mult	54.71	126	5,000		273,550					
School	9.608		5,000		48,040					
Comml	149.60		1,500		224,400					
Ind	41.84		2,000		83,680					
Park	40.22		2,000		0					
Det Pond	9.3		0		0					
Fire Sta	0.47									
Sub Total	835.348				1,319,520	2.2	2,902,944			
I&I				500				417,674		
Total Flow @ SS6 or SS5									3,320,618	3.321

SS4 [A]										
SS4 [A]	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
Res	541.60	2051	350		717,850					
Mult	59.45	126	5,000		297,250					
School	9.608		5,000		48,040					
Comml	149.60		1,500		224,400					
Ind	41.84		2,000		83,680					
Park	40.22		2,000		0					
Det Pond	9.3		0		0					
Fire Sta	0.47									
Sub Total	852.088				1,371,220	2.2	3,016,684			
I&I				500				426,044		
Total Flow at SS4 [A]									3,442,728	3.443

SS3 [B]										
SS3 [B]	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
Res	611.10	2461	350		861,350					
Mult	59.45		5,000		297,250					
School	9.608		5,000		48,040					
Comml	149.60		1,500		224,400					
Ind	41.84		2,000		83,680					
Park	45.326		2,000		0					
Det Pond	9.3		0		0					
Fire Sta	0.47									
Sub Total	926.69				1,514,720	2.2	3,332,384			
I&I				500				463,347		
Total Flow at Control Point SS3 (B)									3,795,731	3.796

SS2 [C]										
SS2 [C]	Ac	DU by count	Flow	I&I Fact	ADWF	Pk Factor	Flow	I&I	Pk Flow	
									gpd	mgd
Res	626.40	2562	350		896,700					
Mult	59.45		5,000		297,250					
School	9.608		5,000		48,040					
Comml	149.60		1,500		224,400					
Ind	41.84		2,000		83,680					
Park	45.326		2,000		0					
Det Pond	9.3		0		0					
Fire Sta	0.47									
Sub Total	941.994				1,550,070	2.2	3,410,154			
I&I				500				470,997		
Total Flow at SS2 [C]									3,881,151	3.881
Total Flow at SS1 (27" Stub at SR113)										

DRAFT

Sanitary Sewer Master Plan Report

for

The Southwest Dixon Specific Plan Area

Prepared for:
The Southwest Dixon Landowners Association

Prepared by:
Nolte Associates, Inc.
1750 Creekside Oaks Drive, Suite 200
Sacramento, CA 95833

NOLTE

BEYOND ENGINEERING

August 2005

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Executive Summary

Introduction

The Southwest Dixon Specific Plan Area (SWDSP) incorporates about 477 acres located within the City of Dixon General Plan. The site is bordered by Interstate 80 to the west, South Lincoln Street to the east, West 'A' Street to the north, and the General Plan boundary to the south.

The current land uses are primarily agricultural. The proposed land uses include low, medium, and high-density residential, commercial, industrial (E), and public uses such as parks.

Purpose

The purpose of this Sanitary Sewer Master Plan Report is to:

- Identify existing sewer facilities and predict generated flows.
- Determine an efficient and cost effective sewer system that supports the proposed development within the SWDSP Area and integrates with existing systems outside of the Project area.

Objectives

The objectives of this Sanitary Sewer Master Plan Report are:

- To provide a plan for a gravity sewer system to convey sewer flows generated by the SWDSP Area.
- To provide a plan for infrastructure to accept offsite sewer where required.
- To provide a plan for infrastructure to convey sewer flows offsite to regional facilities.

Background

The Pre-Design Report for the City of Dixon South Sanitary Sewer Trunk by Morton & Pitalo (M&P), dated August 3, 1999, provided the background for the existing sewer infrastructure in the vicinity of the SWDSP Area. This Report identified problems with the current Pitt School Road Lift Station and the downstream gravity line and proposed improvement alternatives. It recommended placing the East-West Connector south along Pitt School Road, south of the Lift Station, then east along the south border of the Valley Glen Project to connect to the South Dixon Sewer Trunk. The South Dixon Sewer Trunk would convey the flows to the City of Dixon Wastewater Treatment Plant.

City has requested that the SWDSP to provide for sewer service to General Plan Area 1 (GP1) which is comprised of 71± acres zoned for commercial located north of A Street and west of Interstate 80. Flows form GP1 will enter SWDSP at Gateway Dr. and West A Street and will flow through the internal backbone system. Capacity to serve General Plan Area 5 (GP5) is also provided for in the flow calculations for SWDSP. GP5 is comprised of 75± acres of residential and commercial and will enter the system just before it crosses the railroad track at Porter Road.

Proposed Infrastructure

The proposed sewer infrastructure to support the SWDSP Area primarily consists of a backbone collection system within the project and infrastructure to convey offsite flows, thus allowing abandonment of the Pitt School Road Lift Station.

The East-West Connector will extend from the intersection of West 'A' Street and Pitt School Road, south along Pitt School Road to the intersection of Pitt School and Southwest Parkway. The proposed sewer system from lands within the SWDSP west of Pitt school will tie to the east-west connector at two locations, the intersection of North Parkway and Pitt School Road and Southwest Parkway and Pitt School Road. The Connector will then extend east within Southwest Parkway to South Lincoln Street, then turn south along South Lincoln to Porter Road, then turn southwest along Porter Road to a future intersection with Parkway (south border of Valley Glen project), then east along Parkway to connect to the recently constructed East/West connector at South First Street.

Total flow from the SWDSP Area, GP1, relief to the Pitt School Road Lift Station, and GP5 is approximately 1.3 million gallons per day (mgd) average dry-weather flow (ADWF) or 3.1 million gallons per day (mgd) peak wet-weather flow (PWWF). At manhole MH01 (see Figure 1), the Valley Glen project will contribute 0.646 mgd PWWF to the East-West Connector according to flow calculations provided by City staff which have been included in Appendix B.

Sanitary Sewer Master Plan

Southwest Dixon Specific Plan

1 - Introduction

This report provides the technical background for the Sanitary Sewer Master Plan in support of the Southwest Dixon Specific Plan (SWDSP). The SWDSP Area is located within the southwestern Dixon City Limits, west of I-80 and south of West 'A' Street (see Figure 1). The SWDSP Area incorporates about 477 acres of land that is currently being used for agricultural purposes. Proposed land uses within the SWDSP Area include commercial, industrial, residential, and public uses such as parks. Surrounding land uses are similar to proposed land use within the SWDSP Area. Land use to the south is primarily agricultural. To the north is commercial and residential. To the east is residential and to the west is highway and agricultural. Currently, there are a few residences with barns or equipment workshops and storage, and two businesses along Batavia Road.

There is currently no sewer infrastructure in place within the SWDSP Area.

Existing offsite sewer infrastructure in West 'A' Street flows east through the Pitt School Road lift station and force main and serves the West 'A' Street Assessment District (WASAD) north of the SWDSP Area.

Future offsite flows from GP1 will flow into the SWDSP at Gateway Drive and West A Street. Future flows from GP5 will enter to the East-West Connector at Porter Road and the future extension of Parkway across the Railroad tracks.

2 - Purpose

The purpose of this study is to develop a comprehensive Sanitary Sewer Master Plan for the SWDSP Area. This Study will be the basis of the design for the sanitary sewer systems that will be included within infrastructure improvement plans that will occur at a future date. This master plan identifies needed onsite sanitary sewer improvements as well as offsite improvements that are needed to tie with the City of Dixon sanitary sewer infrastructure.

3 - Scope

This Sanitary Sewer Master Plan for the SWDSP Area will accomplish the following:

- Identify existing sewer facilities and flows generated from proposed developments in the SWDSP Area.
- Analyze previous sewer studies and master plans and incorporate the findings that impact the SWDSP Area.

- Integrate recent City determinations on service shed areas.
- Determine a sanitary sewer system that provides a level of service that meets the requirements of the City of Dixon and integrates with existing systems.

This report does not address the issue of Wastewater Treatment Plant (WWTP) expansion, which is currently being planned by the City. This Plan area will contribute through City development fees to provide funding for the WWTP expansion.

4 - Master Plan Objectives

The primary objectives of this master plan are:

- To provide a plan for a gravity sewer system to convey sewer flows generated by the SWDSP Area.
- To provide a plan for infrastructure to accept offsite sewer where required.
- To provide a plan for infrastructure to convey sewer flows offsite to regional facilities.

5 - Background

5.1 General

Sanitary sewer service to the SWDSP Area was originally considered during the 1995 Specific Plan document for the area. Conceptual on-site sanitary sewer collection facilities were proposed in that document. The EIR prepared and certified for that document discussed sanitary sewer service on a programmatic level. Mitigation Measure 5.2-1.a as written in that EIR, required a "…design-level Wastewater System Master Plan." This Sanitary Sewer Master Plan Report expands on the previous analysis and provides significantly greater detail. A new environmental impact report is being prepared to identify impacts and define mitigation measures resulting from the facilities identified in this report.

The SWDSP Area sewer was considered by Morton & Pitalo (M&P) in 1999. The M&P study was conducted in support of the City of Dixon South Sewer Trunk Project. The East-West Connector was recommended to provide sewer service to the SWDSP Area and relieve the Pitt School Road Lift Station.

5.2 Existing Sewer Facilities

5.2.1 On-site Facilities

There are no on-site sewer facilities as the current primary land use is agricultural.

5.2.2 Off-site Facilities

The Pitt School Road Lift Station (at the northwest corner of Pitt School Road and West 'A' Street) is located just off-site of the northeast corner of the project. The Lift Station serves the WASAD by pumping the flow from the area to the northwest to a 15" gravity line. The 15" gravity line conveys this flow to the existing South Dixon Sewer Trunk at the Dixon May Fair. According to the M&P study, this line "would operate under pressure conditions during peak flows" if the West 'A' Street Area continues to use it.

6 - Flow Generation Analysis

6.1 Land Use Assumptions

To calculate generated flows, it was assumed that the on-site developments would be at proposed ultimate condition land use.

The Southwest Dixon Specific Plan document proposes land use areas and projected base line densities for those areas. Base line density is defined as maximum normal gross density, net of arterial and major collector streets. The Plan area target maximum number of dwelling units identified in previous plan approvals was 1,221 (1,121 single-family plus 100 multi-family). This dwelling unit yield is less than that generated by extending the various land use areas by the baseline density. A total of 1,219 single-family and 107 multi-family units result from extension of the areas and base line densities. The City has also required that an additional 144 multi-family dwelling units above the 1221 total units be provided for. Thus, for purposes of the infrastructure master plan studies, the larger number resulting from either the extension of the land use areas by the baseline density factors or the addition of 1221 plus 144 units has been used. This was done to assure adequate capacity of infrastructure should future additional dwelling units be approved by the City.

The on-site land uses are consistent with the Southwest Dixon Specific Plan Draft by Nolte Associates, Inc., dated August 2002. The Equivalent Dwelling Units per acre (EDU/acre) are based on this plan.

Additional flows from offsite areas (i.e., GP1, Pitt School Road Lift Station, and GP5) were provided by City Staff and included in the SWDSP flow calculations, see Appendix A.

6.2 Design Criteria and Constraint

The criteria used to design proposed sanitary sewer infrastructure are consistent with current City of Dixon Standards. Summarized below are the key criteria that were used.

- Manning's Equation, using n=0.013 (DS6-03.B).
- Shed area < 500 Ac., peaking factor = 2.5. 500 Ac. < Shed area < 1500 Ac., peaking factor =2.2.(DS6-03.B)
- Maximum flow at design conditions in any sewer main (10" diameter or less) shall be shall be 70% of pipe capacity. Lines 12" or larger may be designed to flow full unless direct service sewer connections are planned, in which case the 70% of pipe capacity maximum flow shall govern (DS6-03.B, DS6-08.G).
- Based on the SWDSP baseline densities, Medium Density High zoned properties were assigned 11 units per acre, Medium Density Low zoned properties were assigned 5.5 units per acre, Low Density zoned properties were assigned 3.25 units per acre.

Invert elevations in our calculations are based on the Improvement Plans for the South Dixon Sewer Trunk by Morton & Pitalo dated May 23, 2001. The benchmark on these is based on NGVD'29 which is consistent with the SWDSP master plans.

The average daily flow rates and infiltration (I+I) factors are taken from the City of Dixon Standards and are summarized in the following table.

Land Use	Average Daily Flow	I+I Factor
Single-Family	350 gpd per unit	500 gpd per Ac.
Multi-Family	5000 gpd per Ac.	500 gpd per Ac.
Commercial/Public	1500 gpd per Ac.	500 gpd per Ac.
Industrial	2000 gpd per Ac.	500 gpd per Ac.
Schools	5000 gpd per Ac.	500 gpd per Ac.

6.3 *Results*

The following table is a summary of flow generation by land use.

Land Use	Acres	Dwelling Units	ADWF (mgd)	PWWF (mgd)
LD	185.5	603	0.21	0.62
MDL	112.0	616	0.22	0.59
MDH	9.7	146	0.051	0.13
GC	20.9	-	0.03	0.09
HC	11.3	-	0.02	0.06
E	41.8	-	0.08	0.22
Park	22.5	-	0.03	0.10
Fire Station	0.5	-	0.0005	0.0013
TOTAL FLOW			0.64	1.81

The entire project can be serviced by a single backbone system flowing west to east. The project generates approximately 1.81 mgd of on-site sewage peak wet-weather flow. A gravity collection system has been designed to provide adequate service to all areas of the Specific Plan. (See Figure 1) The system depicted shows only lines within collector or larger roadways, and is sufficient to show how service is provided to each individually owned property within the Specific Plan area. Subsequent subdivision of individual properties will require planning of internal collection systems.

The new 15-inch gravity line will run from the Pitt School Road Lift Station, south in Pitt School Road to MH08. From there, an 18-inch line will convey the flow from the properties within SWDSP on either side of Pitt School Road and the Lift Station through MH07 to MH06(see Figure 1). From this point, a 24-inch line will convey the combined flow from the westerly properties of SWDSP, GP1, and the Lift Station east to South Lincoln Street (see cross section, Figure 2) where flows from the remaining easterly properties of SWDSP will be collected. The 24-inch line will continue south to Porter Road (see cross section, Fig. 3), south on Porter Road to a future intersection with Parkway Blvd. where flow from GP5 will be collected, and then along the southern border of the Valley Glen Project. A 27-inch line is being constructed by Valley Glen from Valley Glen Drive to Highway 113. A 27-inch line has been constructed to convey this flow from Highway 113 to the South Dixon Trunk Line.

An alternate alignment is proposed for the East-West Connector than that set forth by the M&P report, which runs south in Pitt School Road until it turns east to run in Parkway Blvd. on the south border of Valley Glen (see Figure 1). The new alignment extends east to South Lincoln Street as described previously. We propose this alignment for two reasons. First, the existing 84" storm drain down Pitt School Road south of SWDSP would be in jeopardy due to deep excavation for the new sewer line. Second, there are two houses to the south of SWDSP on either side of the alignment proposed by M&P. There is not enough room to fit the new infrastructure between them.

Figure 1 shows key collection nodes and the cumulative flows at those nodes. Additional summary information, including projected flow, and invert elevation are shown for those key nodes. Each manhole is numbered for reference to the hydraulic calculations. The hydraulic calculations for that pipe system are attached as Appendix A.

6.4 *Flow Contribution*

The new 15" gravity line extending south along Pitt School Road will carry flows from the existing service area north of West A Street (see Figure 1). Originally, as stated in the 1995 Southwest Dixon Specific Plan document by WPM Planning

Team, this extension was designed to serve the land within the WASAD. This document also recommended that this line be upsized to accommodate additional demand required by the SWDSP. One hundred percent of the total flow can be attributed to the Pitt School Road Lift Station from manhole No. 9 to manhole No. 8. From manhole No. 8 to manhole No. 7, the contribution decreases to approximately 76% of the total flow. Contribution decreases to approximately 75% from No. 7 to No. 6. From No. 4 to No. 1 the contribution decreases to 39%. From No. 1 to the City of Dixon Wastewater Treatment Plant, with the additional flow of 646,000 gpd PWWF, from the Valley Glen Project (M&P report) the contribution decreases to approximately 29%. This contribution from Valley Glen accounts for approximately 17% of the total flow. Please see the table below for a summary.

Southwest Dixon Specific Plan Area

SANITARY SEWER IMPROVEMENTS

			SWDSP		City		GP5		Valley Glen		Total	
Project Ref. No.	Diameter (in.)	Length (ft)	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%	Flow (mgd)	%
SS-01	27	1580	1.80	46%	1.133	29%	0.304	8%	0.646	17%	3.88	100%
SS-02	24	2685	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-03	24	458	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-04	24	1869	1.80	61%	1.133	39%	0	0%	0	0%	2.93	100%
SS-06	24	1324	1.65	59%	1.133	41%	0	0%	0	0%	2.78	100%
SS-07	18	616	0.39	25%	1.133	75%	0	0%	0	0%	1.52	100%
SS-08	18	394	0.35	24%	1.133	76%	0	0%	0	0%	1.48	100%
SS-09	15	1593	0	0%	1.133	100%	0	0%	0	0%	1.13	100%

The flow used to calculate these percentages is 1,133,000 gallons per day PWWF, based on projected full build-out of the WASAD. This value was provided to Nolte by City Staff (Appendix B). A recent application submitted to the City to rezone 31.4 acres of the commercial and industrial lands not yet developed to residential, (Pheasant Run #7) was submitted and approved. All of the existing residential zoned lands in the WASAD have been developed. Due to the fact that the generation rates for low density residential and commercial/industrial zones are not significantly different, there would be no significant impact to the flow to the Pitt School Road Lift Station. The flow generated from Valley Glen was also provided by the City and is assumed to be correct based on full build out of the project. For more information on this subject, please see the Pheasant Run #7, Draft Environmental Impact Report, by LSA Associates, Inc., dated December 2000.

7 - Phasing Considerations

Phasing of sanitary sewer infrastructure normally has few options. General approaches require construction of adequate down stream infrastructure sufficient to convey increased sewer flows from developing areas. Thus, under normal considerations, the out-fall line proposed in this document would need to be constructed to Highway 113. The City has constructed the Dixon South Trunk and a portion of the east-west collector has been constructed between the railroad and Highway 113 by the Valley Glen Development.

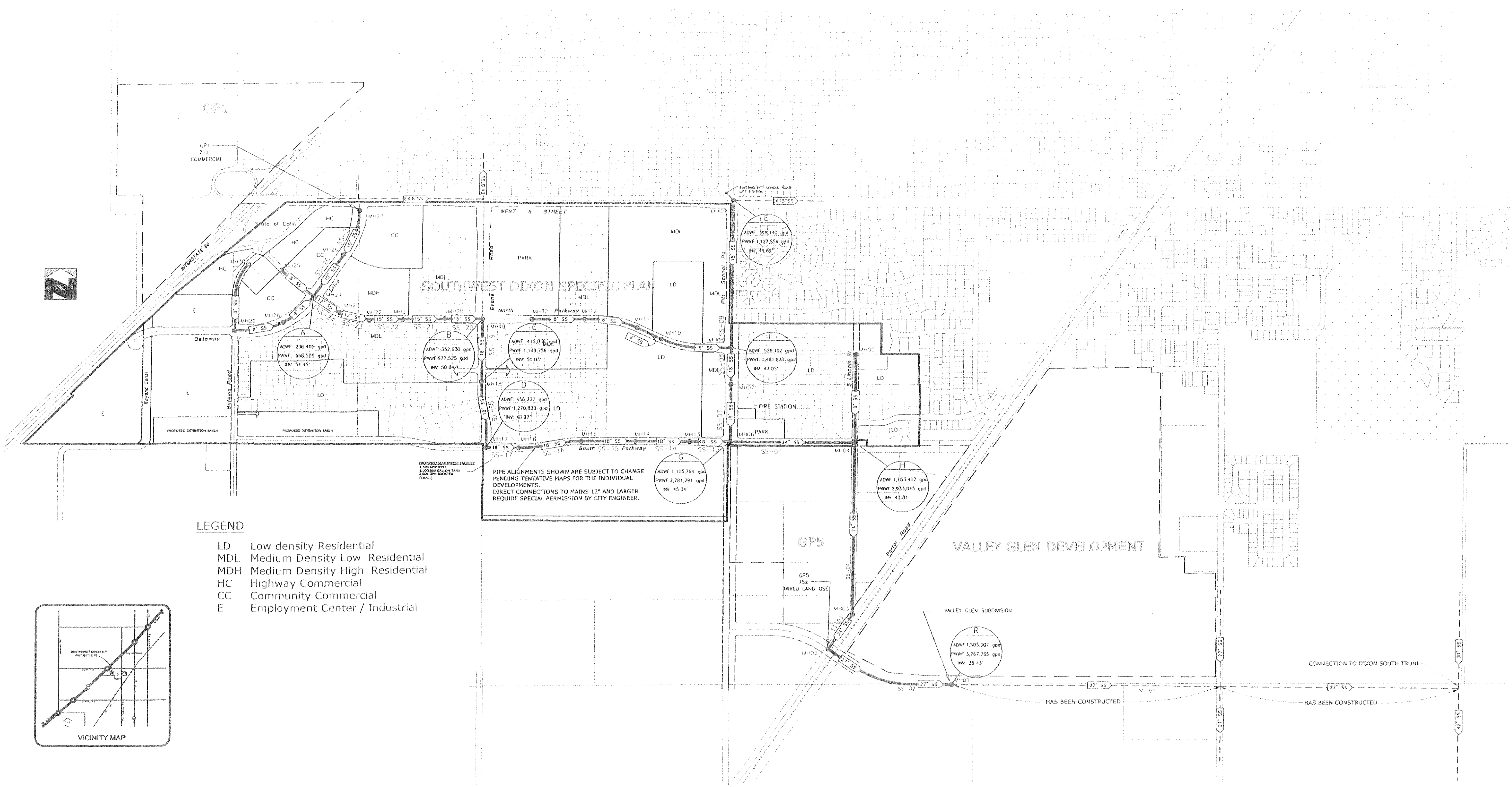
The system described under normal phasing conditions has significant construction costs. While construction of the ultimate system is desirable, initial costs may be prohibitive because of the limitations on building permits per year. A Financing Plan will be prepared which will identify the feasibility of financing full infrastructure costs based on the potential build-out rates. It may be necessary to identify creative phasing solutions to accommodate initial growth. The existing Pitt School Road Pump Station is an example of functional system phasing that allowed development to proceed without incurring prohibitive initial costs.

8 - Conclusions

This report, SWDSP Sanitary Sewer Master Plan Report, outlines a sewer system that will provide a level of service consistent with the City of Dixon Standards. The plan generally conforms to the recommendations made within the City of Dixon South Sewer Trunk Report by Morton & Pitalo, Inc.

The primary components of the plan include: a backbone collection system to convey on-site flows and infrastructure to convey flow from the GP1, Pitt School Road Lift Station, GP5, and the on-site flows to the City of Dixon Wastewater Treatment Plant.

One item will require further discussion. Cost distribution for construction of the off-site infrastructure as well as infrastructure required to convey off-site flows shall be identified in the Capital Improvement Plan.

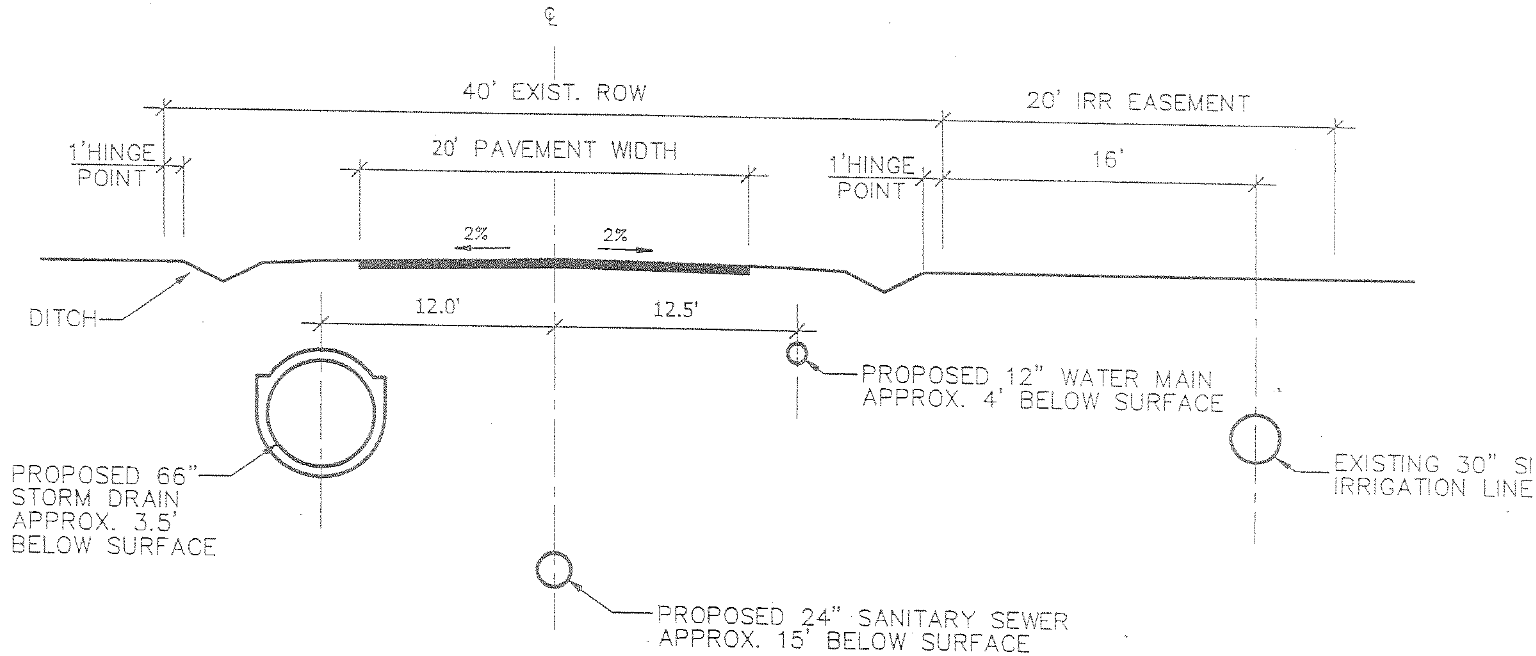


NOTE:
PIPELINE DIAMETERS AND ALIGNMENTS SUBJECT TO CHANGE.

May 2004

Figure 1 - Preliminary Sanitary Sewer Collection ▪ Master Plan

NOLTE
BEYOND ENGINEERING

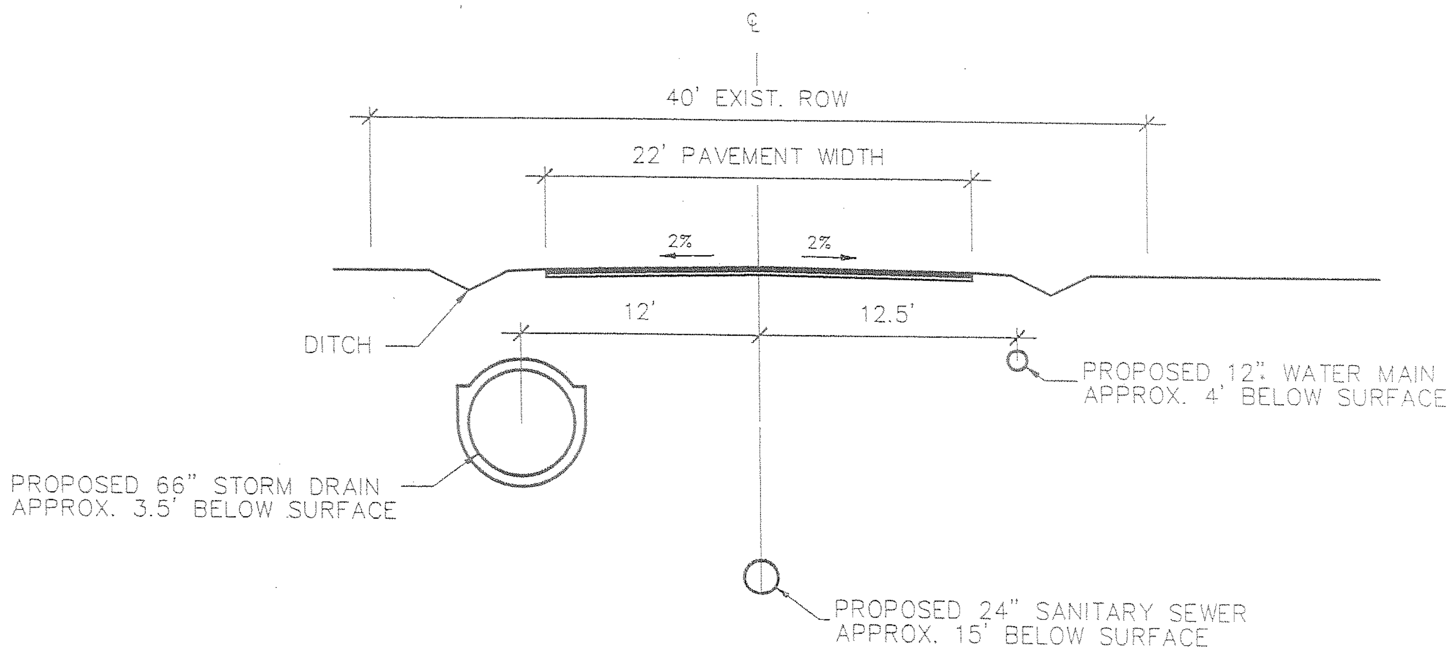


March 2004

Figure 2 - Preliminary South Lincoln Street Typical Section

not to scale

NOLTE

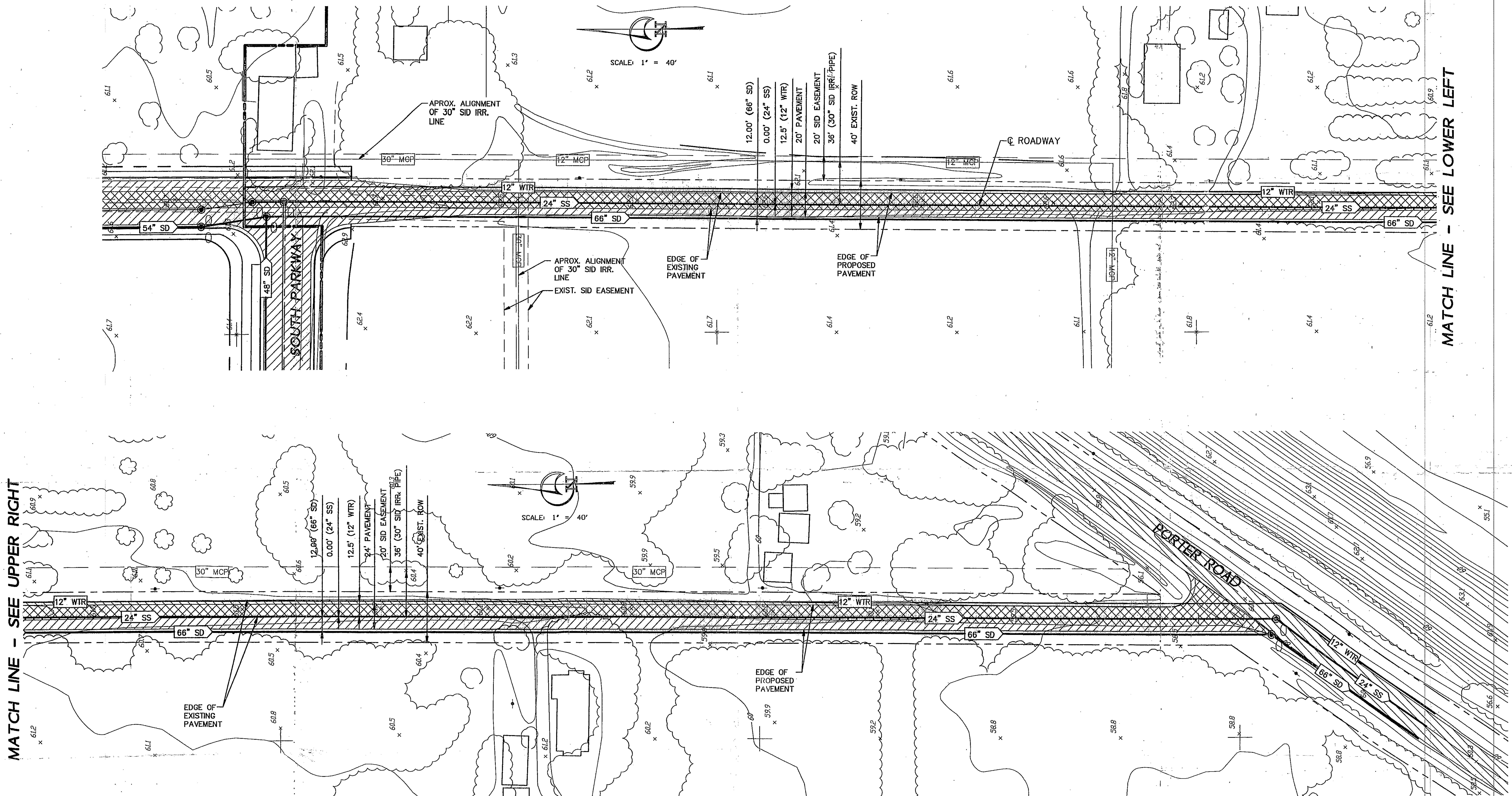


March 2004

not to scale

Figure 3 - Preliminary Porter Road Typical Section

NOLTE
BEYOND ENGINEERING



EXISTING PAVEMENT

EXISTING PAVEMENT TO BE REMOVE AND REPLACE

PROPOSED PAVEMENT

SOUTH LINCOLN STREET

Figure 4

DATE: ________ TIME: ________	NO.	BY	DATE	REVISIONS:
SERVER: ________ SERVICE: ________				
PATH: ________				
DRAWING NAME: ________				
PLOTTING VIEW: ________				
DESIGNER: ________ PROJ. MGR: ________				

CAUTION: The engineer preparing these plans will not be responsible for, or liable for, unauthorized changes to or uses of these

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SOUTH LINCOLN STREET PROPOSED UTILITY LAYOUT SOUTHWEST DIXON SPECIFIC PLAN AREA

PREPARED FOR: SOUTHWEST DIXON LANDOWNERS

DATE SUBMITTED: 3/16/04

SHEET NUMBER 1 OF 1 SHEETS

SCALE
VERTICAL: 1"=4'
HORIZONTAL: 1"=40'

JOB NUMBER

Sanitary Sewer Report

Homestead

CITY OF DIXON, CALIFORNIA

July 2019

Prepared For:

Jen California 6, LLC
508 Gibson Drive, Suite 260
Roseville, California 95678

Prepared By:

CARLSON, BARBEE & GIBSON, Inc.
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SAN RAMON

TABLE OF CONTENTS

APPENDIX

I. INTRODUCTION

A. Purpose

This study is an addendum to the Sanitary Sewer Master Plan Report prepared by Nolte Associates, Inc. dated August 2005. This report is consistent with the underlying principles and sanitary sewer system design concept presented in the August 2005 report, which was reviewed by the City Engineering Department and was a supporting document to the approved Southwest Dixon Specific Plan (SWDSP).

The proposed sanitary sewer system provides capacity for the flows generated by the various land uses within the SWDSP areas along with off-site flows for surrounding future and existing development areas. The proposed land uses within the approved SWDSP areas include commercial, employment, residential and parks. The off-site flows collected and conveyed by the proposed system include future flows from GP1, GP5 and the interception of existing flows currently conveyed through the Pitt School Road Lift Station.

B. Scope of Work

This addendum identifies the updates to the August 2005 report and verifies that the proposed system adequately conveys the flows generated by on-site and off-site land uses.

The proposed sanitary sewer pipe network has been modified to address the Southwest Dixon Builder Group's desired infrastructure phasing. The Southwest Dixon Builder Group consists of five phases within the SWDSP areas *(See Appendix A)*. These five phases include single and multi-family residential housing and various different commercial developments. The proposed sanitary sewer system has been analyzed and designed to support all the aforementioned developments throughout the entirety of the project. Additionally, this report accounts for an additional 60 single family units planned for future development within the specific plan located south of Village 2 and just west of South Lincoln Street.

C. Proposed Conditions

The proposed SWDSP areas include 1,369 single family homes, 10.4 acres of multi-family dwelling units, 69.5 acres of Commercial/Public areas and 49.9 acres of Industrial areas. The average daily flow and I&I factors for the aforementioned different land uses are taken from the City of Dixon Sanitary Sewer design standards *(See Appendix C)*.

The following Table 2 summarizes the flows generated by either future or existing development areas surrounding SWDSP that are conveyed by the SWDSP sanitary sewer system. These off-site flows are consistent with the flows provided by the City of Dixon and attached in Appendix B of the August 2005 report.

TABLE 1
OFF-SITE FLOWS

	Land Use	Acres	Dwelling Units	ADF (mgd)	PWF (mgd)
GP1	COM	71.0	-	0.085	0.213
Pitt School Lift Station	SCHOOL	9.8	-	0.039	0.098
	COM	16.2		0.019	0.048
	SF	238.4	928	0.325	0.813
GP5	COM	16.0		0.019	0.048
	SF	59.0	236	0.083	0.208
TOTAL		**410.4**	**1,164**	**0.57**	**1.428**

II. DESIGN

A. Methodology

The analysis used for these calculations is in accordance with Section 6 of the City of Dixon Engineering Design Standards dated August 2014 *(See Appendix C)*. This guideline was used to establish the average daily flow values and the I/I factor for the various proposed land uses.

Since the SWDSP is less than 500 acres, a peaking factor of 2.5 is assumed to calculate peak flows. Manning's Circular Pipe Calculator was used for the analysis and design of all pipes in the system. Manning's circular pipe calculator was used for sizing all pipes in the system. An "n" value of 0.013 was used per the City of Dixon Engineering Design Standards *(See Appendix B)*.

Table 3 summarizes the minimum slope for different pipe sizes needed to yield a velocity of 2.0ft/sec and while not exceeding 70% capacity.

TABLE 2
SLOPE PARAMETERS

Pipe Diameter (in)	Minimum Slope (ft/ft)	Full Velocity @ Min Slope (ft/sec)	Full Capacity (cfs)	70% of Capacity (cfs)
8	0.0034	2.02	0.714	0.4998
10	0.0026	2.04	1.105	0.7735
12	0.002	2.03	1.593	1.1151
15	0.0015	2.04	2.501	1.7507
18	0.0012	2.06	3.638	2.5466
21	0.001	2.08	5.009	3.5063
24	0.0008	2.04	6.397	4.4779
27	0.0007	2.06	8.192	5.7344

B. Sewer Sheds

The study area limits are delineated on the Sanitary Sewer Systems Map in this report *(See Appendix A)*. The SWDSP sewer shed conveys a total of 1,189,080 gpd (Average Daily Flow) to the existing 24" sanitary sewer main at Valley Glen Drive.

III. RESULTS

1. This study verifies that the proposed SWDSP sanitary sewer system will provide sufficient capacity to convey the design flows at full build out scenario. At the most downstream pipe (SSMH 139 – EX SSMH), the proposed 27” pipe will be at 65.5% capacity which is less than the maximum allowed 70% pipe capacity *(See Appendix B)*.

2. All pipes have been designed to have the velocity range between 2.0 ft/sec to 10.0 ft/sec when flowing full. The minimum calculated velocity is 2.0 ft/sec at pipe (J-10 to SSMH 124) and the maximum calculated velocity is 2.56 ft/sec at pipe (SSMH 119 to SSMH 124).

3. The sanitary sewer pipe in South Parkway east of Pitt School Road has been up-sized to be 1 pipe size larger than necessary (10” vs 8”) to achieve the desired flows and lessen the pipe slopes to avoid conflict with the storm drain stub for future connection @ Camelia Drive.

Appendix A
Sanitary Sewer Map

SANITARY SEWER MAP

CITY OF DIXON SOLANO COUNTY CALIFORNIA

0' 300' 900' 1,200'

SCALE: 1" = 300' DATE: JULY 2019

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<u>Appendix B</u>

Hydraulic Analysis

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SANITARY SEWER FLOW CALCULATIONS
SOUTHWEST DIXON SPECIFIC PLAN
CITY OF DIXON, CALIFORNIA

Downstream Node	From Upstream Node	Upstream Node Description	Land Use	No. of Single Family Units	Gross Area (Acres)	Net Area (Acres)	Average Daily Flow (gpd)	Peaking Factor	I &I[7] (gpd)	Design Flow, Q_d (gpd)	Design Flow, Q_d (cfs)	Pipe Diameter (in)	Slope (ft/ft)	Velocity (fps)	Velocity Full (fps)	Capacity Full (cfs)	Percent Capacity
	SSMH 119	Pitt School Rd Lift Station	SF	928	238.4	190.7	324,800	2.5	119,200	931,200	1.441						
	SSMH 119	Pitt School Rd Lift Station	COM		16.2	13.0	19,440	2.5	8,100	56,700	0.088						
	SSMH 119	Pitt School Rd Lift Station	SCHOOL		9.8	7.8	39,200	2.5	4,900	102,900	0.159						
	SSMH 119	Pitt School Right of Way			4.4	3.5	0	2.5	2,200	2,200	0.003						
	SSMH 119	Subtotal from SSMH 119			268.8	215.0	383,440		134,400	1,093,000	1.691	15	0.0023	2.56	2.52	3.097	54.6%
	East stub	Phase 1A, Village 4	SF	53	16.8	13.4	18,550	2.5	8,400	54,775	0.085	8	0.0034	1.30	2.02	0.714	11.9%
	J-10	Phases 3&4 (North Pkwy)	SF	291	70.4	56.3	101,850	2.5	35,200	289,825	0.449						
	J-10	Phases 4 Park Area	COM		34.3	27.4	41,160	2.5	17,150	120,050	0.186						
	J-10	Subtotal from J-10			104.7	83.8	143,010		52,350	409,875	0.634	12	0.0020	1.89	2.03	1.593	39.8%
SSMH 124				1,272	390.3	312.2	545,000		195,150	1,557,650	2.411						
	SSMH 124	Upstream of SSMH 124		1,272	390.3	312.2	545,000		195,150	1,557,650	2.411	18	0.0012	2.17	2.06	3.638	66.3%
	West Stub	Phase 1, Village 3	SF	48	18.0	14.4	16,800	2.5	9,000	51,000	0.079	8	0.0034	1.32	2.02	0.714	11.1%
SSMH 126				1,320	408.3	326.6	561,800		204,150	1,608,650	2.489	21	0.0010	2.07	2.08	5.009	49.7%
	SSMH 111	Phase 1, Village 1 Phase 1A, Village 4	SF	65	18.0	14.4	22,750	2.5	9,000	65,875	0.102	10	0.0034	1.36	2.33	1.263	8.1%
	SSMH 112	Phase 1, Village 2	SF	96	34.8	27.8	33,600	2.5	17,400	101,400	0.157	8	0.0034	1.54	2.02	0.714	22.0%
SSMH 114				161	52.8	42.2	56,350		26,400	167,275	0.259						
	SSMH 114	Upstream of SSMH 114		161	52.8	42.2	56,350		26,400	167,275	0.259	10	0.0034	1.78	2.02	0.714	36.3%
	SSMH 114	S. Parkway Right of Way			2.4	1.9	0	2.5	1,200	1,200	0.002						
	North stub	Phase 1 Village 1 & Park	SF	27	11.5	9.2	9,450	2.5	5,750	29,375	0.045	8	0.0034	1.05	2.02	0.714	6.4%
SSMH 117				188	66.7	53.4	65,800		33,350	197,850	0.306	10	0.0034	1.90	2.02	0.714	42.9%
	GP1	General Plan Area 1	COM		71.0	56.8	85,200	2.5	35,500	248,500	0.385						
	J-1	Highway Commercial	COM		4.3	3.4	5,160	2.5	2,150	15,050	0.023						
	J-1	Employment Center	IND		7.1	5.7	11,360	2.5	3,550	31,950	0.049						
J-2				0	82.4	65.9	101,720		41,200	295,500	0.457						

cbg

SANITARY SEWER FLOW CALCULATIONS
SOUTHWEST DIXON SPECIFIC PLAN
CITY OF DIXON, CALIFORNIA

Downstream Node	From Upstream Node	Upstream Node Description	Land Use	No. of Single Family Units	Gross Area (Acres)	Net Area (Acres)	Average Daily Flow (gpd)	Peaking Factor	I &I[7] (gpd)	Design Flow, Q_d (gpd)	Design Flow, Q_d (cfs)	Pipe Diameter (in)	Slope (ft/ft)	Velocity (fps)	Velocity Full (fps)	Capacity Full (cfs)	Percent Capacity
	J-2	Upstream of J-2 to J-1		0	82.4	65.9	101,720		41,200	295,500	0.457						
	J-2	Employment Centers	IND		42.8	34.2	68,480	2.5	21,400	192,600	0.298						
	J-2	Evans Ranch ASB Property	SF	125	27.5	22.0	43,750	2.5	13,750	123,125	0.191						
J-2 to J-5				125	152.7	122.2	213,950		76,350	611,225	0.946						
	J-2	J-2 to J-5		125	152.7	122.2	213,950	0	76,350	611,225	0.946						
	J-3	Highway Commercial	COM		2.9	2.3	3,480	2.5	1,450	10,150	0.016						
	J-3	Park	COM		13.5	10.8	16,200	2.5	6,750	47,250	0.073						
	J-3	Subtotal from J-3			16.4	13.1	19,680		8,200	57,400	0.089						
	J-4	Various Commercial	COM		14.5				7,250	7,250	0.011						
J-5				125	183.6	135.3	233,630		91,800	675,875	1.046						
	J-5	Upstream of J-5		125	183.6	135.3	233,630		91,800	675,875	1.046						
	J-5	Phase 2 Evans Ranch MF	MF		10.4	8.3	41,600	2.5	5,200	109,200	0.169						
	J-5	Phase 2 Evans Ranch SF	SF	87	24.0	19.2	30,450	2.5	12,000	88,125	0.136						
J-6				212	218.0	162.8	305,680		109,000	873,200	1.351						
	J-6	Upstream of J-6		212	218.0	162.8	305,680		109,000	873,200	1.351						
	J-6	Phases 2 & 3	SF	231	52.4	41.9	80,850	2.5	26,200	228,325	0.353						
SSMH 101				443	270.4	204.7	386,530		135,200	1,101,525	1.705						
	SSMH 101	Upstream of SSMH 101		443	270.4	204.7	386,530		135,200	1,101,525	1.705						
	SSMH 101	S. Parkway Right of Way			6.4	5.1	0	2.5	3,200	3,200	0.005						
	SSMH 101	Subtotal from SSMH 101		443	276.8	209.8	386,530		138,400	1,104,725	1.710	18	0.0012	2.00	2.06	3.638	47.0%
	North Stub	Phase 1, Village 3	SF	23	5.6	4.5	8,050	2.5	2,800	22,925	0.035	8	0.0034	0.92	2.02	0.714	5.0%
	South Stub	Phase 1, Village 3	SF	6	1.6	1.3	2,100	2.5	800	6,050	0.009	8	0.0034	0.57	2.02	0.714	1.3%
SSMH 106				472	284.0	215.6	396,680		142,000	1,133,700	1.754	18	0.0012	2.01	2.06	3.638	48.2%
	SSMH 129	Combined Flows at SSMH 129		1,980	759.0	595.6	1,024,280		379,500	2,940,200	4.550	27	0.0007	2.08	2.06	8.192	55.5%
	SSMH 129	Pitt School Right of Way			1.6	1.3	0	2.5	800	800	0.001						
	East stub	Phase 1 Village 2	SF	47	17.2	13.8	16,450	2.5	8,600	49,725	0.077						
	West stub	Phase 1 Village 3	SF	129	45.9	36.7	45,150	2.5	22,950	135,825	0.210						
	Ag District	Future Development	SF	17	17.0	13.6	5,950	2.5	8,500	23,375	0.036						
SSMH 130				2,173	840.7	661	1,091,830		420,350	3,149,925	4.875						

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SANITARY SEWER FLOW CALCULATIONS
SOUTHWEST DIXON SPECIFIC PLAN
CITY OF DIXON, CALIFORNIA

Downstream Node	From Upstream Node	Upstream Node Description	Land Use	No. of Single Family Units	Gross Area (Acres)	Net Area (Acres)	Average Daily Flow (gpd)	Peaking Factor	I &I[7] (gpd)	Design Flow, Q_d (gpd)	Design Flow, Q_d (cfs)	Pipe Diameter (in)	Slope (ft/ft)	Velocity (fps)	Velocity Full (fps)	Capacity Full (cfs)	Percent Capacity
	SSMH 130	Flow at SSMH 130		2,173	840.7	661	1,091,830		420,350	3,149,925	4.875	27	0.0007	2.14	2.06	8.192	59.5%
	SSMH 130	I&I Area (Assume 60' width)			4.3	3.4	0	2.5	2,150	2,150	0.003						
	GP5	General Plan Area 5	SF	236	59.0	47.2	82,600	2.5	29,500	236,000	0.365						
	GP5	General Plan Area 5	COM		16.0	12.8	19,200	2.5	8,000	56,000	0.087						
SSMH 139				2,409	920.0	724.4	1,193,630		460,000	3,444,075	5.330						
	SSMH 139	Flow at SSMH 139		2,409	920.0	724.4	1,193,630		460,000	3,444,075	5.330	27	0.0007	2.19	2.06	8.192	65.1%
	SSMH 139	I&I Area (Assume 60' width)			2.1	1.7	0	2.5	1,050	1,050	0.002						
EX SSMH	EX SSMH	Connect to Existing		2,409	922.1	726.1	1,193,630		461,050	3,445,125	5.331	27	0.0007	2.19	2.06	8.192	65.1%
TOTAL				**2,409**	**922.1**	**726.1**	**1,193,630**		**461,050**	**3,445,125**	**5.331**						

Notes:

1. Minimum Sewer main size is 10"

2. Manning's Coefficient of Roughness, n = 0.013 per City of Dixon design standards

3. Cumulative Gross Area (for calculating flow Infiltration and Inflow, I&I)

4. Cumulative Net Area for various land uses is taken as 80% of the gross area per City of Dixon design standards.

5. Average daily flow rate is based on 300 gallons per single family unit per day (gpd) and 1,500 gpd per net acre for commercial/public parcels per City of Dixon design standards

6. Assumes a peak to average flow factor of 2.5 per City of Dixon design standards

7. Assumes an infiltration rate of 500 GPD per gross acre per City of Dixon design standards

8. Actual flow velocity using design flow (Determined using Manning's Circular Pipe Calculator)

9. Flow velocity assuming pipe is full (Determined using Manning's Circular Pipe Calculator)

Appendix C
City of Dixon Sanitary Sewer Design Standards

ENGINEERING DESIGN STANDARDS

SECTION 6 - SANITARY SEWER DESIGN

DS6-01. GENERAL: Sanitary sewer improvements shall be designed to serve the ultimate level of development as defined in the City General Plan. All improvements shall conform to the requirements of the Solano County Health Department, the Uniform Plumbing Code, and the City of Dixon Engineering Design Standards and Construction Specifications.

DS6-02. PLAN REQUIREMENTS: Sanitary sewer improvement plans shall show geometric designs in both plan and profile views. Required information shall be main and lateral sizes and slopes, utility crossings, manholes, cleanouts, invert elevations, and any calculation used in the design of the system.

DS6-03. DESIGN

A. Flow - The design sanitary sewer flow in gallons per day (gpd) shall be calculated using the following formula:

$$Q_d = Q_p + I \,\&\, I \text{ , where}$$

Q_d = Design flow
Q_p = Peak flow = Average Daily Flow x Peaking Factor
I+ I = Infiltration & Inflow Factor

The average daily flow rates and the I+I (Infiltration & Inflow) factors for various land uses are shown in the following table:

DESIGN FLOWS		
LAND USE	**AVERAGE DAILY FLOW**	**I+I FACTOR**
Single-Family	350 gpd per unit	500 gpd per gross acre
Multi-Family	5000 gpd per net acre	500 gpd per gross acre
Commercial/Public	1500 gpd per net acre	500 gpd per gross acre
Industrial	2000 gpd per net acre	500 gpd per gross acre
Schools	5000 gpd per net acre	500 gpd per gross acre

*Note: Net Acres is assumed as 80% of Gross Acres.

The peaking factors to be used to calculate the peak flow are shown in the following table:

PEAKING FACTORS	
SHED AREA	**PEAKING FACTOR**
Shed area less than 500 acres	2.5
500 acres ≤ Shed Area ≤1,500 acres	2.2
Shed area greater than 1,500 acres	2.1

B. Pipe Capacity - Typically sewer mains shall be sized based upon the sewer flowing at 70% of pipe capacity using the following formula:

Manning's Formula: $Q = A(1.49/n)(R^{2/3})(S^{1/2})$, where

Q = Flow, in cubic feet per second (cfs)
A = Area of Pipe in square feet (sf)
R = Hydraulic Radius (Area/ Wetted Perimeter)
S = Slope of Pipe
n = Roughness of 0.013 or as recommended by the pipe manufacturer, whichever is greater

Pipe capacity, in all cases, shall be adequate to carry the design flow from the entire tributary area, even though said tributary area is not located within the project boundaries. Sewer trunk line design criteria shall be done on a case by case basis, as approved by the City Engineer.

C. Velocity - Sewer velocity shall be equal to or greater than 2 feet per second for all sewers when flowing full with a maximum velocity of 10 feet per second. Sewers which will exceed 50% capacity at ultimate development shall have their minimum design slope determined using a minimum velocity flowing full of 2 feet per second. Sewers which will not exceed 50% capacity at full development shall have a minimum design velocity flowing full of 2.5 feet per second.

D. Main Size - Minimum size sewer main shall be 8 inches.

E. Sewer Pipe Type - Typical sanitary sewers shall be constructed of extra-strength vitrified clay pipe (ESVCP). SDR35 polyvinyl chloride pipe (PVC) material may be used in residential areas, on a case by case basis, upon approval by the City Engineer. PVC pipe consideration will require a design and construction analysis using ASTM Specifications for the pipe material. A report will be submitted identifying all design and construction criteria per ASTM Specifications.

F. STUDY MAP - A study map may be required prior to review of the sewer design if there is a possibility that upstream or adjacent areas might require service through the subject property. The map should show the entire service area including upstream tributary and adjacent areas, and all other data necessary to determine anticipated service area, including pipe sizes and slopes, shall be shown to the extent necessary to determine the requirements within the subject property. Any required study map shall be paid for by the project developer; however, said study map may be waived by the City Engineer if previously preformed.

DS6-04. VERTICAL ALIGNMENT

A. At all manholes where a change of direction of more than 20 degrees occurs, the flow line of the upstream main shall be 0.20 ft. above the flow line of the downstream main.
B. Where a change in size of mains occurs, the crowns shall be matched.
C. No vertical curves shall be allowed.
D. Where minor mains connect to trunk mains, the crowns shall match if feasible. Under no circumstances shall the invert of the minor main enter the trunk main below springline.

DS6-05. HORIZONTAL ALIGNMENT

A. All sanitary sewers shall be installed in the pavement area of the street. Generally the location should be 6 feet from the center line of the street, on the opposite side of the centerline from the water line.
B. Under special circumstances, if approved in advance of plan submittal, exception may be granted by the City Engineer which will allow a sanitary sewer line to be placed in an easement. In such cases, a minimum 15 foot wide easement shall be given, and the easement shall cross not more than one lot. Deeper lines shall require a wider easement to the satisfaction of the City Engineer.
C. Location in existing streets - The following shall be considered: location of curbs, gutters, and sidewalks; traffic lane configurations; future street improvement plans; and existing utilities.

DS6-06. SEWER MAIN CLEARANCES: Clearances between sanitary sewer mains and other facilities shall conform to state law, but shall not be less than:

Horizontal: 10 feet minimum from any water line
5 feet minimum from all other facilities
Vertical: 1 foot minimum from all facilities for main lines
6" minimum from all facilities for service laterals

DS6-07. APPURTENANCES

A. Manholes - Normal maximum spacing for manholes shall be 400 feet. Where the location of two manholes is determined by intersecting lines, the distances between intervening manholes shall be approximately equal. Sewers on curved alignments with a radius of less than 400 feet shall have manholes spaced at a maximum of 300 feet on the beginning and ending of the curve and adjusted to fit the individual case. The spacing of manholes on trunk sewer lines 12 inches and larger in diameter shall be proposed for each individual case and shall be approved by the City Engineer. All manhole connections of trunk lines 12-inch and larger shall be epoxy-coated to reduce inflow & infiltration. Manholes shall also be located at all change in pipe sizes and slopes, and at angles of 20° or more in alignment. Manholes shall also be placed at the termination of all sewer mains including those lines which may be extended in the future and cul-de-sacs. Services to adjacent properties within the cul-de-sac should be connected to this manhole.

B. Drop manholes will be allowed upon approval of the City Engineer. Change in sewer pipe invert through a manhole is not to exceed 2 feet on an 8 or 10 inch sewer main.

C. Cleanouts - Cleanouts on sewer main lines shall not be used. Cleanout spacing on sewer laterals shall not exceed 100 feet within the City right-of-way. Cleanouts shall be placed at all changes of size, slope, or angle points greater than 20 degrees; at intersections of mains; and at service connections where service lines are 6-inch and larger.

DS6-08. SERVICE LATERAL

A. GRADIENT: Four inch (4") sewer services shall have a minimum slope of 2%.

B. LOCATION AND ALIGNMENT: Sewer services shall be at right angles or radial to street right-of-way. The location shall be stationed on the plans. Services shall be located near the center of each parcel, however not located within driveways, and shall be not less than 10 feet from water services, fire hydrants, street lights, etc. In cul-de-sac bulbs services should enter manholes.

C. SIZE: Minimum size for single family dwellings is 4-inch. Minimum size for commercial, apartments and industrial developments shall be 6 inches.

D. DEPTH: Sewer services shall have 5-foot to 5-foot, 6-inches of cover at the right-of-way line, and 12 inches at any buildable location within the properties to be served.

E. CLEANOUTS: Cleanouts shall be installed on the service lateral at the back of sidewalk as shown on the Construction Details.

F. IDENTIFICATION: Sewer laterals shall be identified with an "S" stamped or etched on the top of curb.

G. CONNECTIONS TO LARGE MAINS: Sewer service may be directly connected to sewer mains smaller than 12 inches in diameter. For trunk sewer lines 12 inches and larger in diameter, or more than 15 feet in depth, the service sewer may be directly connected only with the approval of the City Engineer.

H. TYPE OF PIPE: Same as sewer mains. Cleanout assemblies and service to site from the cleanout may be ABS per Construction Detail 6020.

I. ONSITE CONNECTIONS: Storm runoff shall not be designed to enter the sanitary sewer system.

J. Each parcel within commercial and industrial districts, including multi-family development service laterals, shall connect to a sewer main manhole unless approved otherwise by the City Engineer.

DS6-09. TRENCH LOADING: For sanitary sewer lines over 10 feet deep, Marston's formula shall be used to determine the load placed on the pipe by backfill. The procedure for rigid pipe is described in the ASCE Manual of Engineering Practice No. 60, the Clay Pipe Engineering Manual, and in similar handbooks. The Design Engineer shall determine the factor of safety. Only the three edge bearing strength of the pipe shall be used in the computations for rigid pipe. The minimum trench width shall be O.D. plus 12 inches.

DS6-10. BEDDING AND INITIAL BACKFILL: Bedding types and factors for V.C.P. shall be as per Construction Details 3280 and 3290. For other materials, the trench width, bedding, and initial backfill shall be consistent with the pipe manufacturer's requirements. Bedding and initial backfill type shall be as necessitated by depth of cover over the pipe, trench width, pipe strength, and other factors used to determine safe pipe loading. Any special backfill requirements shall be noted on the plans.

DS6-11. LIFT STATIONS: Lift stations shall not be permitted unless specifically approved by the City Engineer in advance of plan submittal.

DS6-12. UNUSUAL DESIGN: Special designs of sanitary sewer facilities or other unusual features or structures will require individual study and approval by the City Engineer.

cbg

CIVIL ENGINEERS ▪ SURVEYORS ▪ PLANNERS

June 20, 2019
Job No.: 1311-080

MEMORANDUM

TO: Joe Leach – City of Dixon
Andrew MacDonald – City of Dixon

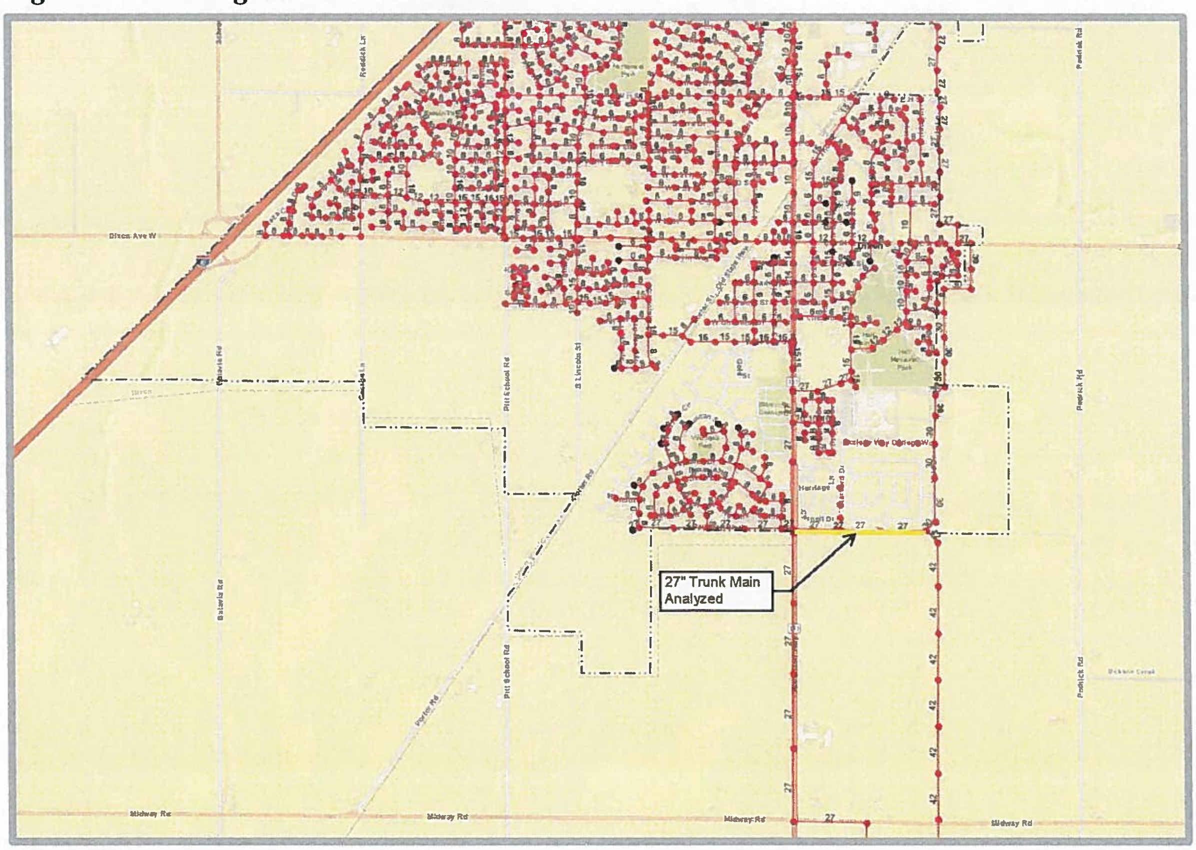
FROM: Angelo J. Obertello, P.E., LEED AP, QSD, Principal

SUBJECT: Downstream Sanitary Sewer Truck Main Capacity Analysis
Homestead
Dixon, California

REGISTERED PROFESSIONAL ENGINEER
ANGELO OBERTELLO
No. 64345
Exp. 6-30-21
CIVIL
STATE OF CALIFORNIA

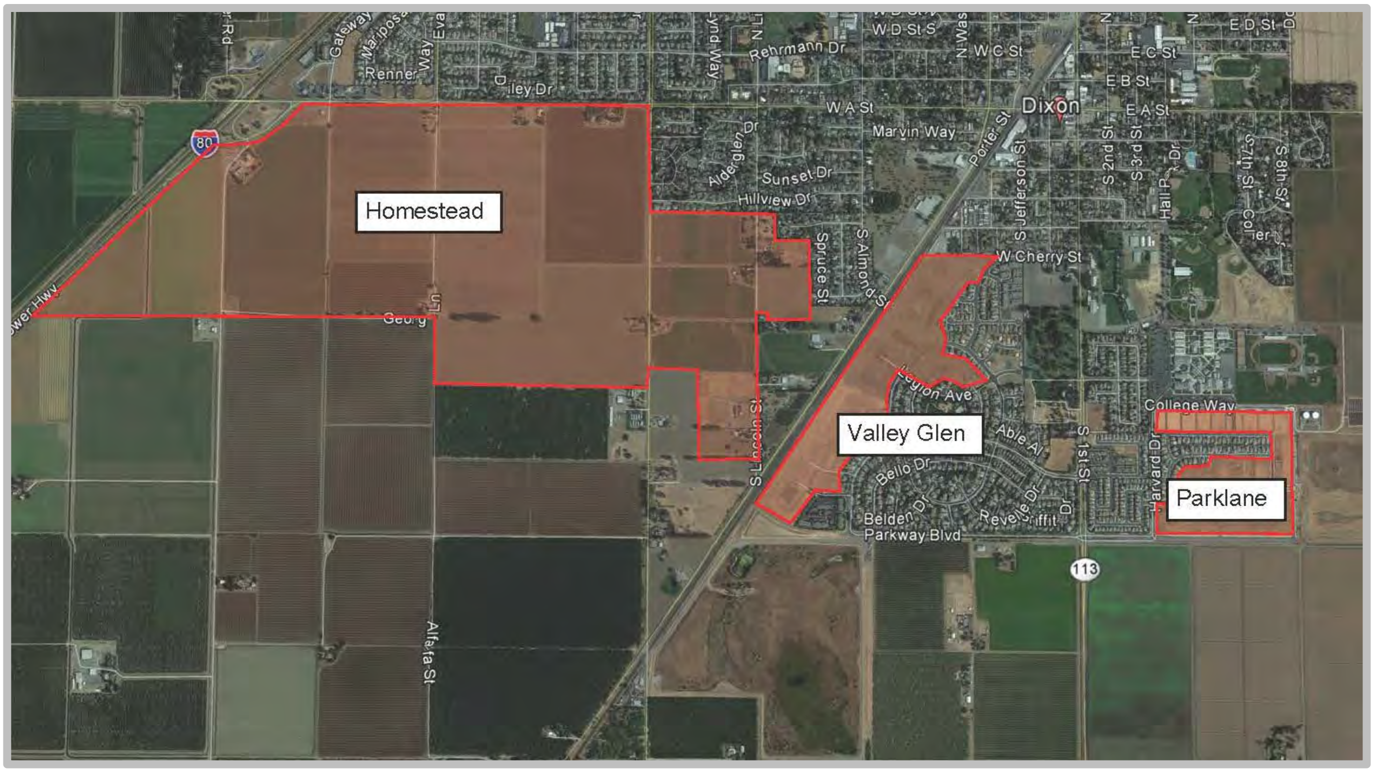
The following provides an analysis of the capacity of the existing 27-inch diameter sanitary sewer trunk main downstream of the future development anticipated in the southern portion of the City of Dixon. The existing trunk main evaluated in this analysis is located at the southern limit of the City of Dixon and conveys wastewater flows from a majority of the City to the wastewater treatment plant. See Figure 1 depicting the location of the trunk main analyzed in this memorandum.

Figure 1 – Existing 27-Inch Trunk Main



There are three active development projects within the southern portions of the City of Dixon. These projects include Parklane, Valley Glen and Homestead. The locations of these projects are depicted on Figure 2. Parklane and Valley Glen are partially completed. The purpose of this analysis is to document the existing capacity of the sanitary sewer trunk main and evaluate the estimated capacity after the three development projects are completed.

Figure 2 – Future Development Projects



EXISTING TRUNK MAIN CAPACITY

The existing trunk main capacity has been established through actual flow monitoring conducted at two sites along the trunk main. See the Southwest Dixon Flow Monitoring Study enclosed as Attachment 1 providing a summary and the results of this flow monitoring effort.

The results of this flow monitoring indicate that existing measured peak flow within the trunk main is 2.271 MGD. The City of Dixon Engineering Design Standards indicate the pipeline capacity shall not exceed 70% of the full-pipe capacity, which is 7.82 MGD for this trunk main. Accordingly, the trunk main was measured to be flowing 29% full, with an available capacity of 5.5 MGD for future flows while not exceeding 70% of the full-pipe capacity.

FUTURE DEVELOPMENT FLOWS

The estimated wastewater design flows generated from the future development of the three projects noted above are calculated based upon the City of Dixon Engineering Design Standards. Conservatively, a peaking factor of 2.5 has been utilized because each project separately is less than the 500 acres. Two future development scenarios have been evaluated. Future Development Scenario 1 includes the build-out of the Parkland and Valley Glen projects, along with the Phase 1 of the Homestead Project. Future Development Scenario 2 includes the build-out of all three project. The estimated flows for these future development scenarios are summarized below in Table 1 & 2.

Table 1 – Future Development – Scenario 1

	Single Family	Multi-Family Units	Multi-Family Area (acres)	Commercial Area (acres)	Employment Area (acres)	Area (acres)	Peak Flow (gdp)	I&I (gdp)	Design Flow (gdp)
Valley Glen Remaining Lots	259	-	-	-	-	40	22,6625	20,000	246,625
Parkland Remaining Lots	210	-	-	-	-	55	182,750	27,500	211,250
Homestead Phase 1	403	-	-	-	-	137	352,625	68,500	421,125
								Total Future Development – Scenario 1 Design Flow (gdp)	**879,000**

Table 2 - Future Development – Scenario 2

	Single Family	Multi-Family Units	Multi-Family Area (acres)	Commercial Area (acres)	Employment Area (acres)	Area (acres)	Peak Flow (gdp)	I&I (gdp)	Design Flow (gdp)
Valley Glen Remaining Lots	259	-	-	-	-	40	226,625	20,000	246,625
Parkland Remaining Lots	210	-	-	-	-	55	182,750	27,500	211,250
Homestead Build-Out	1,234	131	10	29.2	41.8	480	1,358,500	240,000	1,598,500
								Total Future Development – Scenario 2 Design Flow (gdp)	**2,056,375**

<u>TRUNK MAIN CAPACITY WITH FUTURE DEVELOPMENT</u>

The future development estimated design flows are added to the existing measured peak flows within the trunk main to confirm the 70% capacity threshold is not exceeded.

Future Development Scenario 1

Total Flow = Existing Measured Peak Flow + Estimated Future Design Flow

= 2.271 MGD + 0.879 MGD = 3.15 MGD

3.15 MGD < 27" Truck Main Capacity (70%) 7.82 MCD

∴ **Adequate Capacity**

Future Development Scenario 2

Total Flow = Existing Measured Peak Flow + Estimated Future Design Flow

= 2.271 MGD + 2.06 MGD = 4.33 MGD

4.33 MGD < 27" Truck Main Capacity (70%) 7.82 MGD

∴ **Adequate Capacity**

This evaluation confirms that the existing 27-inch diameter sanitary trunk main has adequate capacity for the build-out of the three development projects, Parklane, Valley Glen and Homestead.

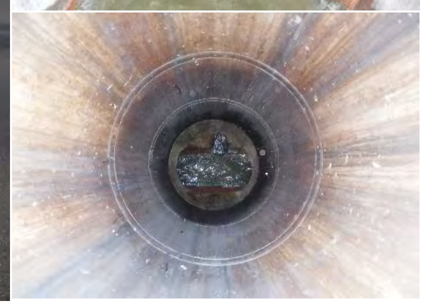
<u>Attachment:</u>

1. Southwest Dixon Flow Monitoring Study

Attachment 1

City of Dixon

Southwest Dixon Flow Monitoring Study



Prepared for: Carson, Barbee & Gibson, Inc.
2633 Camino Ramon, Suite 350
San Ramon, CA 94583

Date: May 2019

Prepared by: V&A consulting engineers

V&A Project No. 19-0071

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Photo Log

1 Introduction

V&A Consulting Engineers, Inc. (V&A) was retained by Carson, Barbee & Gibson, Inc. (CBG) to perform sanitary sewer flow monitoring and capacity analysis within the City of Dixon, California (City). Open-channel flow monitoring was performed at two manholes for two weeks between April 15, 2019 and May 16, 2019. Due to an equipment problem, the two monitoring periods differed. Site 1 was monitored from April 15, 2019 to May 3, 2019 and Site 2 was monitored from May 3, 2019 to May 16, 2019. The purpose of this study was to identify the average and peak flows and to determine the available capacity of the subject pipes.

Flow monitoring sites are identified as the manholes where the flow monitors were secured and the pipelines wherein the flow sensors were placed.

The flow monitoring sites were selected and approved by CBG and the City. Information regarding the flow monitoring locations is listed in Table 1-1. Figure 1-1 shows the location of the flow monitoring sites. A detailed description of each of the flow monitoring sites, including photographs, is included in Appendix A.

Table 1-1. List of Flow Monitoring Locations

Name	Location	Manhole Number	Pipe Diameter	Pipe Material	Monitored Inlet
Site 1	E Park Boulevard at Harvard Drive (38° 25.866'N, 121° 49.178'W)	21	27 inch	VCP	West
Site 2	E Park Boulevard east of Harvard Drive (38° 25.865'N, 121° 49.005'W)	49	27 inch	VCP	West

Figure 1-1. Location of Future Development and Flow Monitoring Sites

2 Methods and Procedures

2.1 Confined Space Entry

A confined space (Photo 2-1) is defined as any space that is large enough and so configured that a person can bodily enter and perform assigned work, has limited or restricted means for entry or exit and is not designed for continuous employee occupancy. In general, the atmosphere must be constantly monitored for sufficient levels of oxygen (19.5% to 23.5%), and the presence of hydrogen sulfide (H_2S) gas, carbon monoxide (CO) gas, and lower explosive limit (LEL) levels. A typical confined space entry crew has members with OSHA-defined responsibilities of Entrant, Attendant, and Supervisor. The Entrant is the individual performing the work. He or she is equipped with the necessary personal protective equipment needed to perform the job safely, including a personal four-gas monitor (Photo 2-2). If it is not possible to maintain line-of-sight with the Entrant, then more Entrants are required until line-of-sight can be maintained. The Attendant is responsible for maintaining contact with the Entrants to monitor the atmosphere using another four-gas monitor and maintaining records of all Entrants, if there is more than one. The Supervisor is responsible for developing the safe work plan for the job at hand prior to entering.

Photo 2-1. Confined Space Entry

Photo 2-2. Typical Personal Four-Gas Monitor

2.2 Flow Meter Installation

V&A installed two Isco 2150 area-velocity flow meters for temporary metering within the collection system. Isco 2150 meters use submerged sensors with a pressure transducer to collect depth readings and an ultrasonic Doppler sensor to determine the average fluid velocity. The ultrasonic sensor emits high-frequency (500 kHz) sound waves, which are reflected by air bubbles and suspended particles in the flow. The sensor receives the reflected signal and determines the Doppler frequency shift, which indicates the estimated average flow velocity. The sensor is typically mounted at a manhole inlet to take advantage of smoother upstream flow conditions. The sensor may be offset to one side to lessen the chances of fouling and sedimentation where these problems are expected to occur. Manual level and velocity measurements were taken during the installation of the flow meters and again when they were removed and compared to simultaneous level and velocity readings from the flow meters to ensure proper calibration and accuracy. Figure 2-1 shows a typical installation for a flow meter with a submerged sensor.

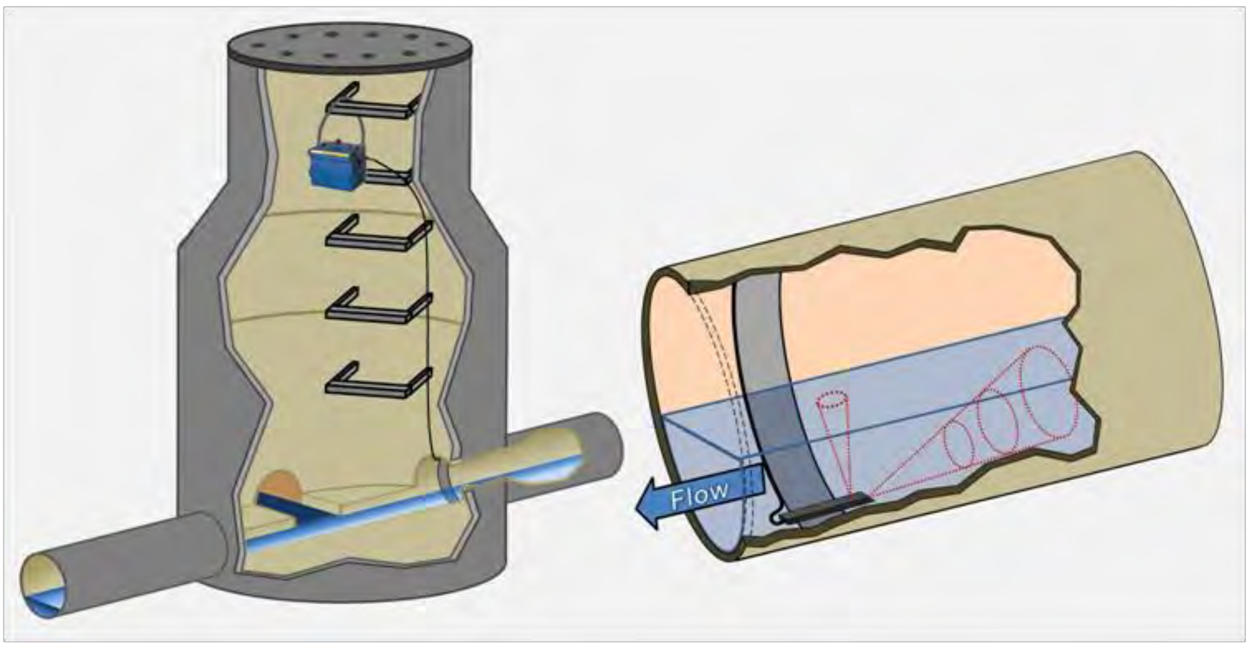


Figure 2-1. Typical Installation for Flow Meter with Submerged Sensor

2.3 Flow Calculation

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

$$Q = v \cdot A = v \cdot (A_T - A_S)$$

where Q: volume flow rate

v: average velocity as determined by the ultrasonic sensor

A: cross-sectional area available to carry flow

A_T: total cross-sectional area with both wastewater and sediment

A_S: cross-sectional area of sediment.

For circular pipe,

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

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

where d_W: distance between wastewater level and pipe invert

d_S: depth of sediment

D: pipe diameter

Weekday and weekend flow patterns differ and are separated when determining average dry weather flows (ADWF). The Overall ADWF was determined from:

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

3 Results and Analysis

3.1 Flow Monitoring Results

Table 3-1 lists the ADWF, peak measured flow and other calculated factors used to determine the pipeline capacity. Detailed graphs of the flow monitoring data are included in *Appendix A*.

Table 3-1. Dry Weather Flow Monitoring Summary

Item	Site 1 SSMH21	Site 2 SSMH48
Pipe Diameter (in):	27	27
Weekday (Monday-Thursday) ADWF (mgd):	1.005	1.107
Friday ADWF (mgd):	1.001	1.176
Saturday ADWF (mgd):	1.057	1.236
Sunday ADWF (mgd):	1.105	1.240
Overall ADWF (mgd):	1.026	1.154
Peak Measured Flow (mgd):	1.906	2.271
Peak Level (in):	5.687	8.047
d/D Ratio:	0.211	0.298
Peaking Factor:	1.857	1.968

3.2 Pipeline Capacity

The pipeline capacity was estimated by using the Manning equation:

$$Q = \frac{669 \times R^{2/3} \times S^{1/2} \times A}{n}$$

where

A: Cross-sectional area of flow (ft^2)
R: hydraulic radius (ft), calculated from flow level *d* and pipe diameter *D*
S: Pipeline slope (ft/ft)
n: Roughness coefficient (unitless)
Q: Flow rate (ft^3/s)

Table 3-2 shows the results of the pipeline capacity calculations. The following factors were selected to determine the pipeline capacity.

- **Roughness coefficients:** 0.013 for VCP and concrete pipes is a widely accepted number for sanitary sewer design.
- **Pipeline Slopes:** The pipeline slopes were derived from maps provided by CBG.

Table 3-2. Pipeline Capacity

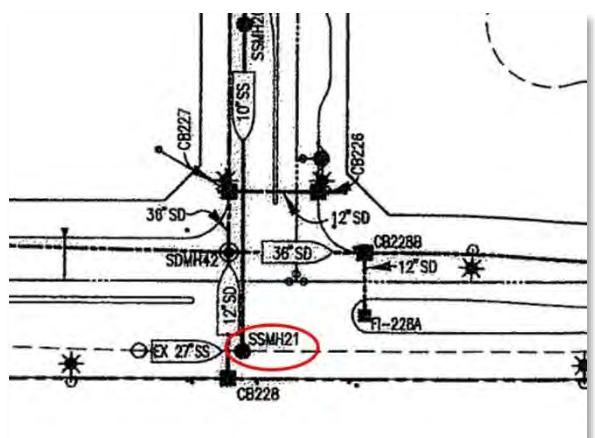
Site	Site 1	Site 2
Pipe Diameter (in):	27	27
Slope:	0.0035	0.0031
Roughness Coefficient:	0.013	0.013
Peak Measured Flow (mgd):	1.906	2.271
Full-Pipe Capacity (mgd):	11.84	11.18

Capacity analysis data is presented on a site-by-site basis and represents the hydraulic conditions only at the site locations; hydraulic conditions in other areas of the collection system will differ. The City is encouraged to consult the hydraulic model and model assumptions.

For pipeline capacity analysis based on the actual development please follow the City of Dixon Sanitary Sewer Design guidelines which have been included as Appendix B.

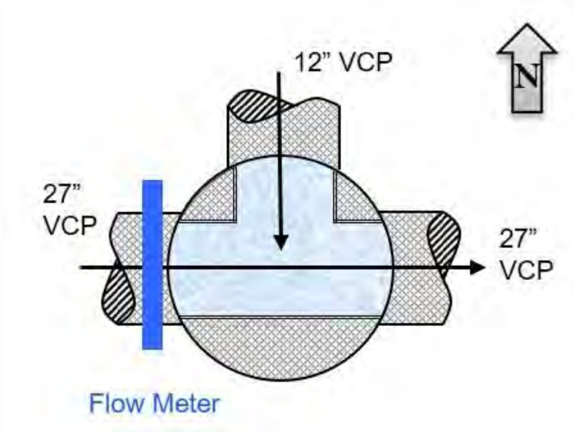
Appendix A
Flow Monitoring Site Report: Data, Graphs, Information

Satellite View



Street View

Sanitary Sewer Map



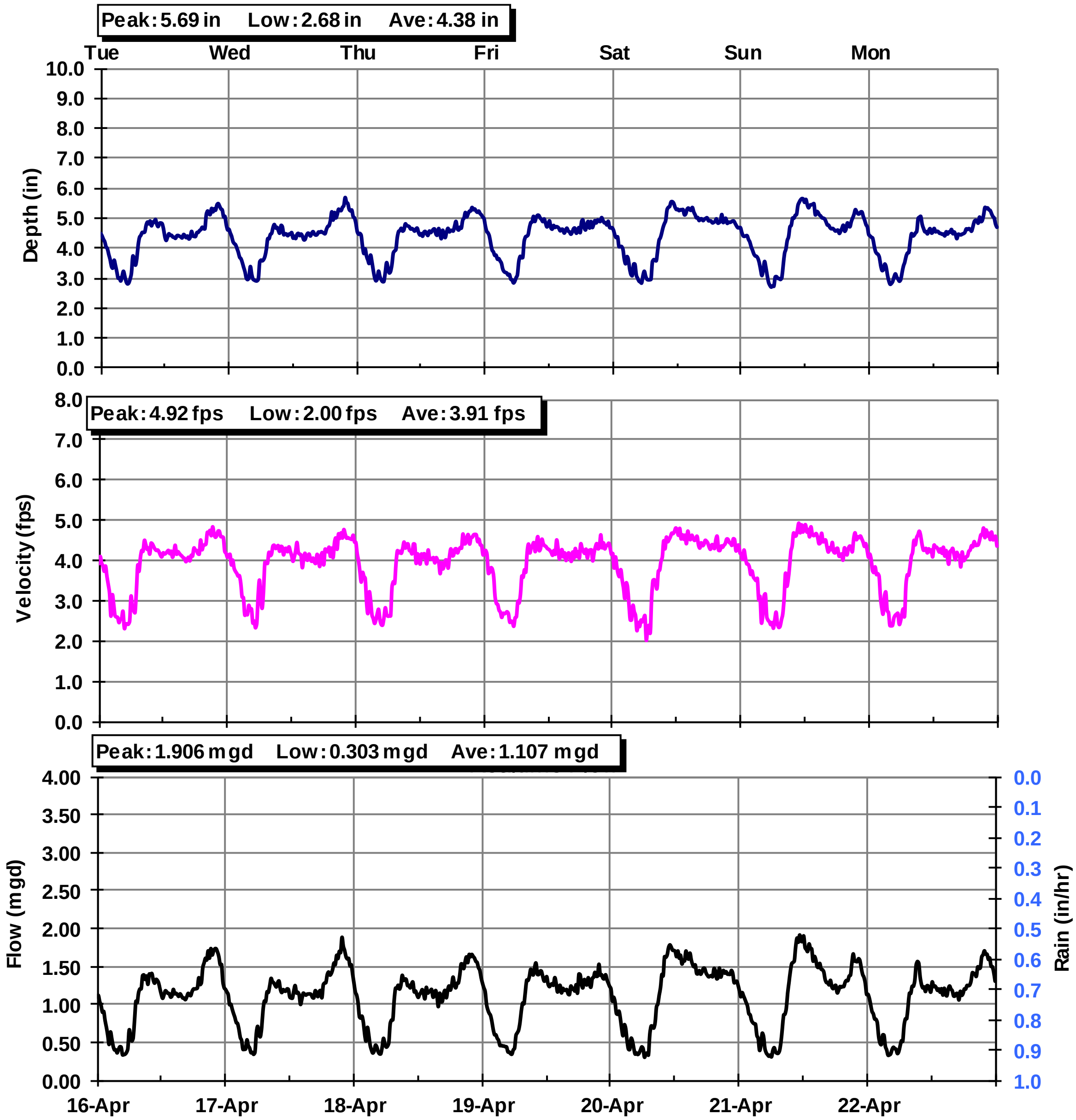
Flow Diagram

Plan View

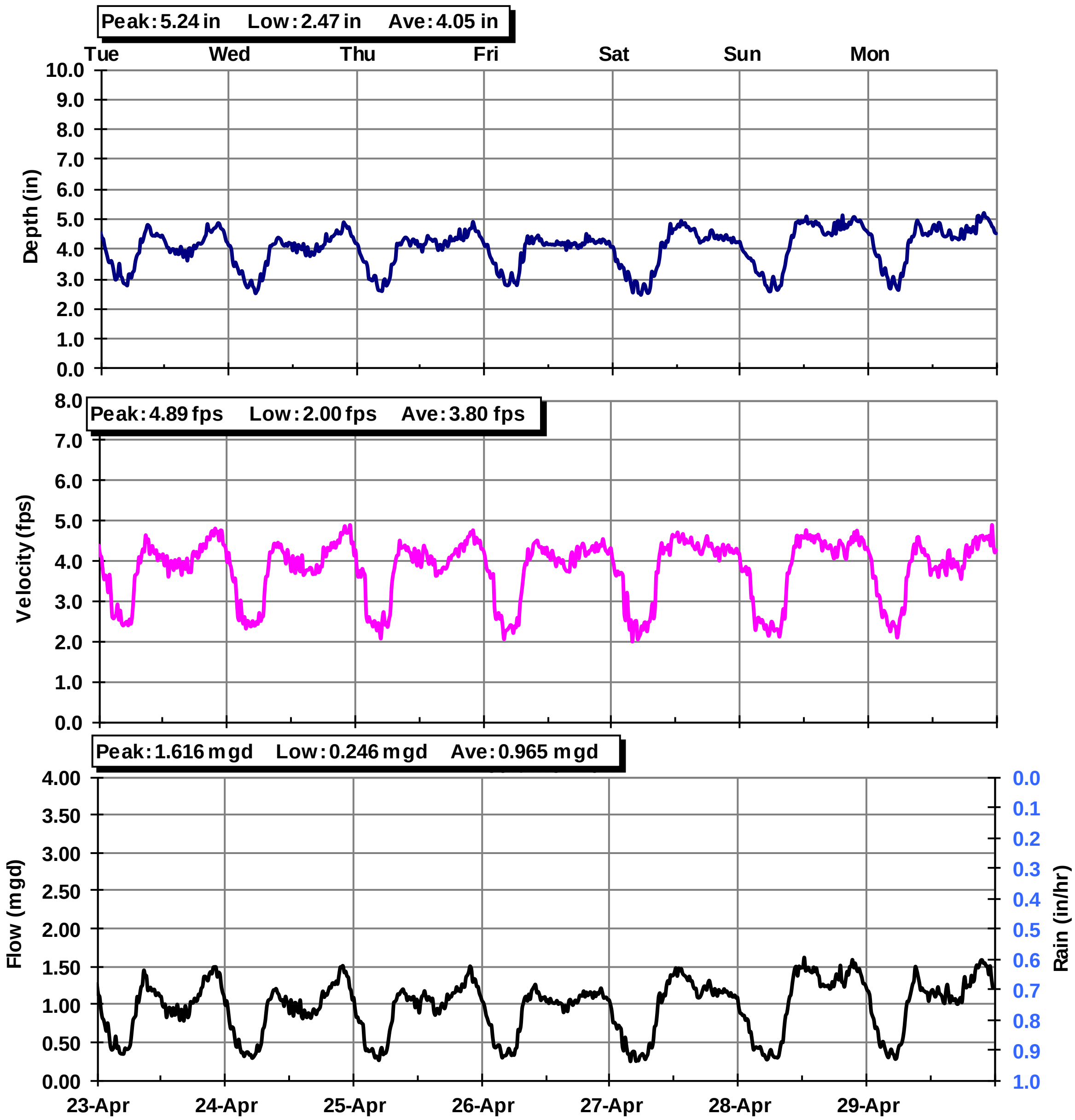
Effluent Pipe

Monitored West Influent Pipe

North Influent Pipe



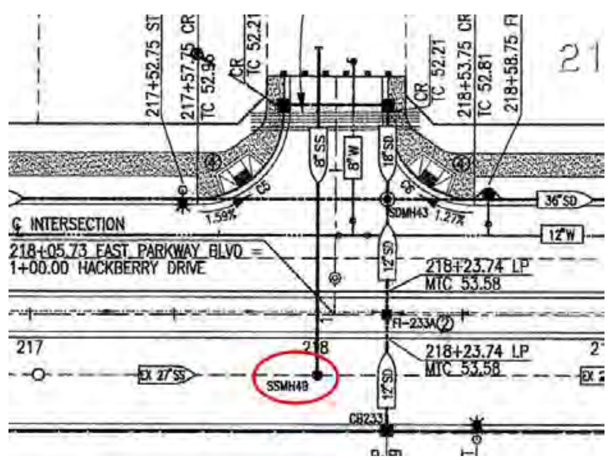
Site 1: Flow Monitoring Details (4/16/19 to 4/22/19)



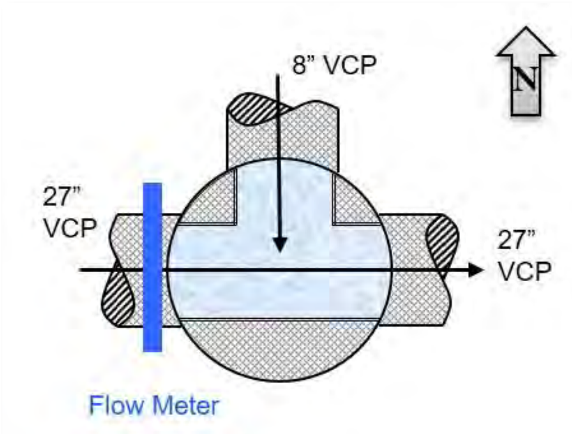
Site 1: Flow Monitoring Details (4/23/19 to 5/3/19)

Satellite View

Street View



Sanitary Sewer Map

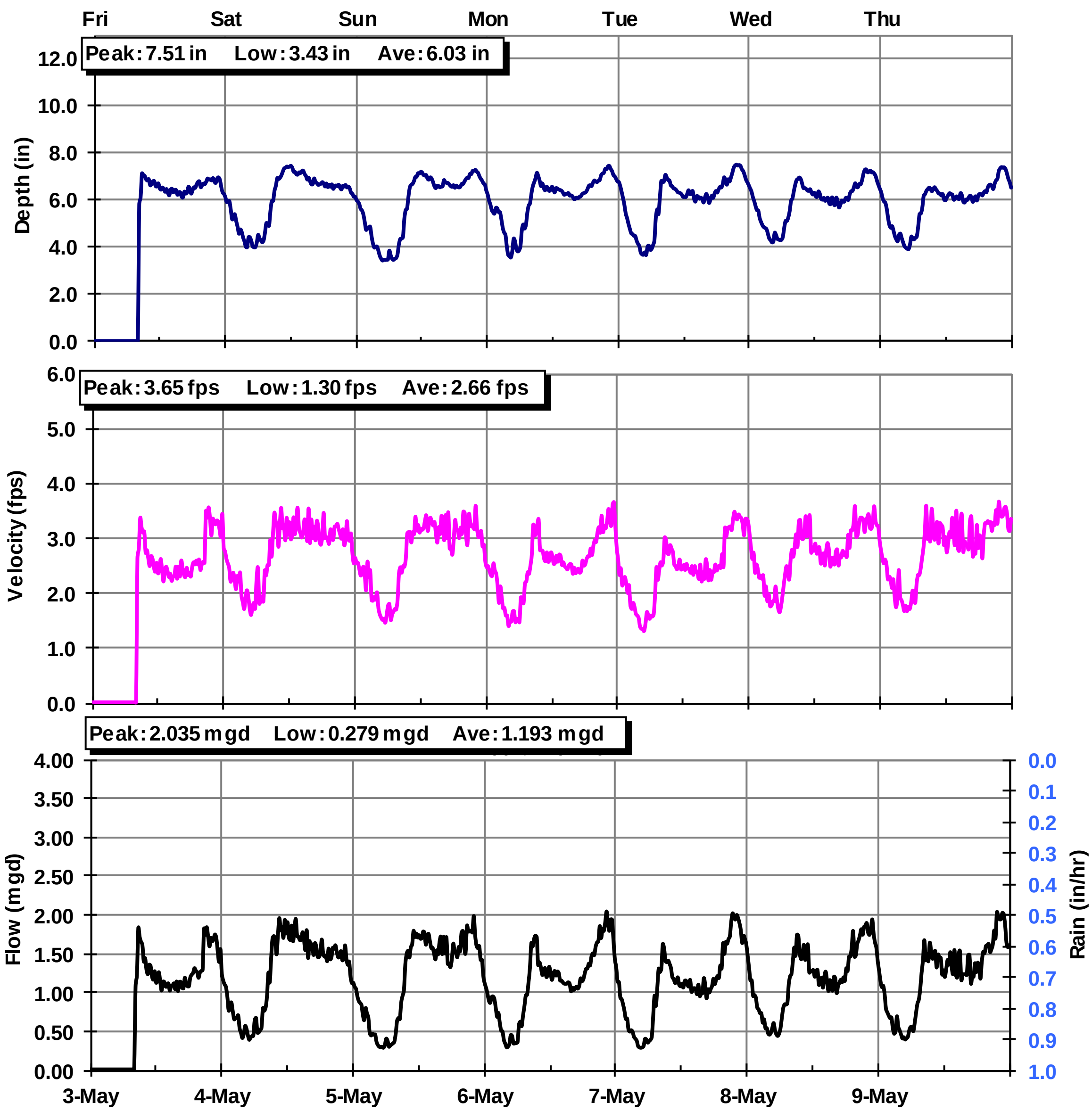


Flow Diagram

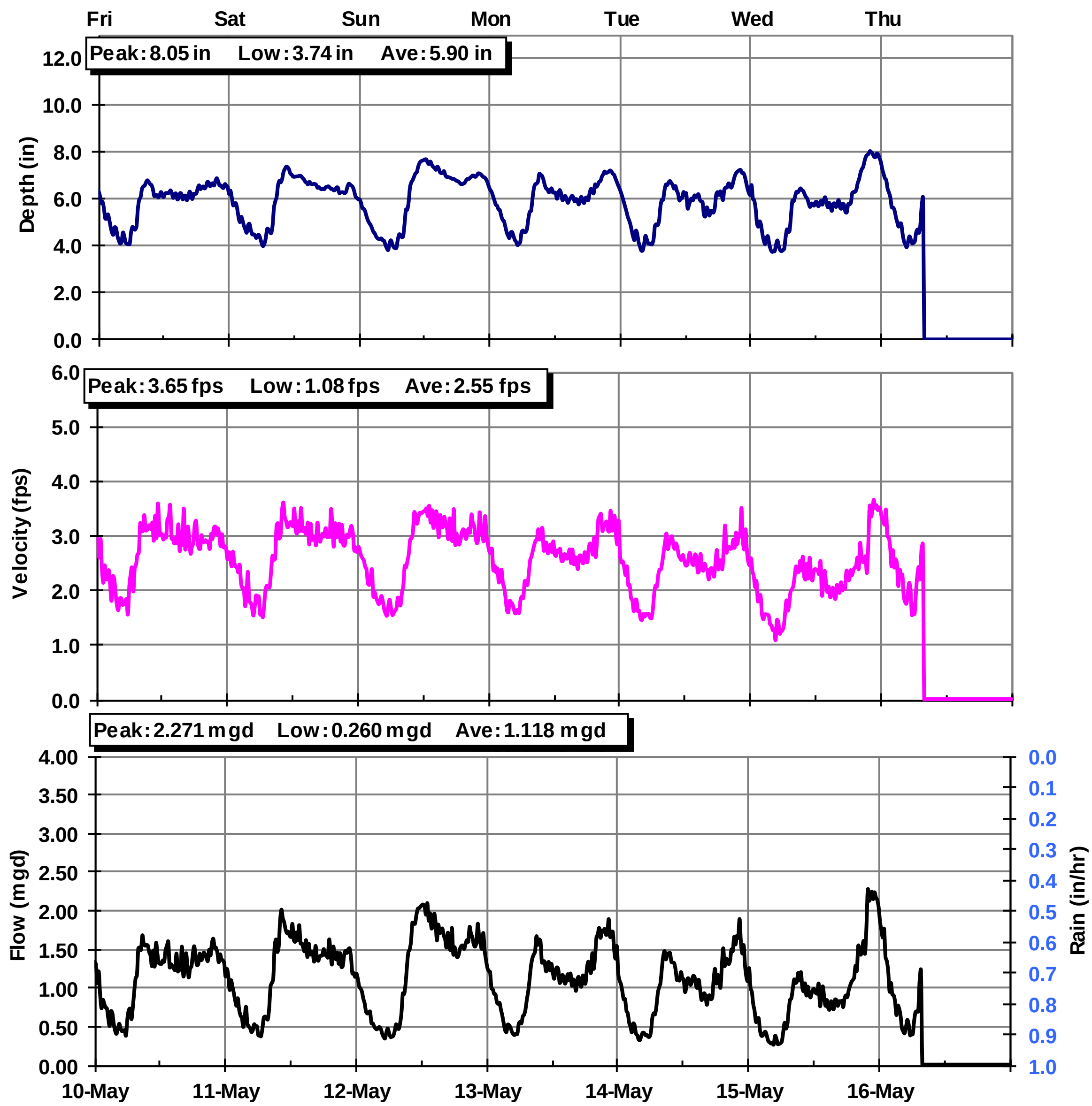
Effluent Pipe

Monitored West Influent Pipe

North Influent Pipe



Site 2: Flow Monitoring Details (5/3/19 to 5/9/19)



Site 2: Flow Monitoring Details (5/10/19 to 5/16/19)

Appendix B
City of Dixon Sanitary Sewer Design Standards

ENGINEERING DESIGN STANDARDS

SECTION 6 - SANITARY SEWER DESIGN

DS6-01. GENERAL: Sanitary sewer improvements shall be designed to serve the ultimate level of development as defined in the City General Plan. All improvements shall conform to the requirements of the Solano County Health Department, the Uniform Plumbing Code, and the City of Dixon Engineering Design Standards and Construction Specifications.

DS6-02. PLAN REQUIREMENTS: Sanitary sewer improvement plans shall show geometric designs in both plan and profile views. Required information shall be main and lateral sizes and slopes, utility crossings, manholes, cleanouts, invert elevations, and any calculation used in the design of the system.

DS6-03. DESIGN

A. Flow - The design sanitary sewer flow in gallons per day (gpd) shall be calculated using the following formula:

$$Q_d = Q_p + I \,\&\, I \text{ , where}$$

Q_d = Design flow
Q_p = Peak flow = Average Daily Flow x Peaking Factor
I+ I = Infiltration & Inflow Factor

The average daily flow rates and the I+I (Infiltration & Inflow) factors for various land uses are shown in the following table:

DESIGN FLOWS		
LAND USE	**AVERAGE DAILY FLOW**	**I+I FACTOR**
Single-Family	350 gpd per unit	500 gpd per gross acre
Multi-Family	5000 gpd per net acre	500 gpd per gross acre
Commercial/Public	1500 gpd per net acre	500 gpd per gross acre
Industrial	2000 gpd per net acre	500 gpd per gross acre
Schools	5000 gpd per net acre	500 gpd per gross acre

*Note: Net Acres is assumed as 80% of Gross Acres.

The peaking factors to be used to calculate the peak flow are shown in the following table:

PEAKING FACTORS	
SHED AREA	**PEAKING FACTOR**
Shed area less than 500 acres	2.5
500 acres ≤ Shed Area ≤1,500 acres	2.2
Shed area greater than 1,500 acres	2.1

B. Pipe Capacity - Typically sewer mains shall be sized based upon the sewer flowing at 70% of pipe capacity using the following formula:

Manning's Formula: $Q = A(1.49/n)(R^{2/3})(S^{1/2})$, where

Q = Flow, in cubic feet per second (cfs)
A = Area of Pipe in square feet (sf)
R = Hydraulic Radius (Area/ Wetted Perimeter)
S = Slope of Pipe
n = Roughness of 0.013 or as recommended by the pipe manufacturer, whichever is greater

Pipe capacity, in all cases, shall be adequate to carry the design flow from the entire tributary area, even though said tributary area is not located within the project boundaries. Sewer trunk line design criteria shall be done on a case by case basis, as approved by the City Engineer.

C. Velocity - Sewer velocity shall be equal to or greater than 2 feet per second for all sewers when flowing full with a maximum velocity of 10 feet per second. Sewers which will exceed 50% capacity at ultimate development shall have their minimum design slope determined using a minimum velocity flowing full of 2 feet per second. Sewers which will not exceed 50% capacity at full development shall have a minimum design velocity flowing full of 2.5 feet per second.

D. Main Size - Minimum size sewer main shall be 8 inches.

E. Sewer Pipe Type - Typical sanitary sewers shall be constructed of extra-strength vitrified clay pipe (ESVCP). SDR35 polyvinyl chloride pipe (PVC) material may be used in residential areas, on a case by case basis, upon approval by the City Engineer. PVC pipe consideration will require a design and construction analysis using ASTM Specifications for the pipe material. A report will be submitted identifying all design and construction criteria per ASTM Specifications.

F. STUDY MAP - A study map may be required prior to review of the sewer design if there is a possibility that upstream or adjacent areas might require service through the subject property. The map should show the entire service area including upstream tributary and adjacent areas, and all other data necessary to determine anticipated service area, including pipe sizes and slopes, shall be shown to the extent necessary to determine the requirements within the subject property. Any required study map shall be paid for by the project developer; however, said study map may be waived by the City Engineer if previously preformed.

DS6-04. VERTICAL ALIGNMENT

A. At all manholes where a change of direction of more than 20 degrees occurs, the flow line of the upstream main shall be 0.20 ft. above the flow line of the downstream main.

B. Where a change in size of mains occurs, the crowns shall be matched.

C. No vertical curves shall be allowed.

D. Where minor mains connect to trunk mains, the crowns shall match if feasible. Under no circumstances shall the invert of the minor main enter the trunk main below springline.

DS6-05. HORIZONTAL ALIGNMENT

A. All sanitary sewers shall be installed in the pavement area of the street. Generally the location should be 6 feet from the center line of the street, on the opposite side of the centerline from the water line.

B. Under special circumstances, if approved in advance of plan submittal, exception may be granted by the City Engineer which will allow a sanitary sewer line to be placed in an easement. In such cases, a minimum 15 foot wide easement shall be given, and the easement shall cross not more than one lot. Deeper lines shall require a wider easement to the satisfaction of the City Engineer.

C. Location in existing streets - The following shall be considered: location of curbs, gutters, and sidewalks; traffic lane configurations; future street improvement plans; and existing utilities.

DS6-06. SEWER MAIN CLEARANCES: Clearances between sanitary sewer mains and other facilities shall conform to state law, but shall not be less than:

Horizontal: 10 feet minimum from any water line
5 feet minimum from all other facilities

Vertical: 1 foot minimum from all facilities for main lines
6" minimum from all facilities for service laterals

DS6-07. APPURTENANCES

A. Manholes - Normal maximum spacing for manholes shall be 400 feet. Where the location of two manholes is determined by intersecting lines, the distances between intervening manholes shall be approximately equal. Sewers on curved alignments with a radius of less than 400 feet shall have manholes spaced at a maximum of 300 feet on the beginning and ending of the curve and adjusted to fit the individual case. The spacing of manholes on trunk sewer lines 12 inches and larger in diameter shall be proposed for each individual case and shall be approved by the City Engineer. All manhole connections of trunk lines 12-inch and larger shall be epoxy-coated to reduce inflow & infiltration. Manholes shall also be located at all change in pipe sizes and slopes, and at angles of 20° or more in alignment. Manholes shall also be placed at the termination of all sewer mains including those lines which may be extended in the future and cul-de-sacs. Services to adjacent properties within the cul-de-sac should be connected to this manhole.

B. Drop manholes will be allowed upon approval of the City Engineer. Change in sewer pipe invert through a manhole is not to exceed 2 feet on an 8 or 10 inch sewer main.

C. Cleanouts - Cleanouts on sewer main lines shall not be used. Cleanout spacing on sewer laterals shall not exceed 100 feet within the City right-of-way. Cleanouts shall be placed at all changes of size, slope, or angle points greater than 20 degrees; at intersections of mains; and at service connections where service lines are 6-inch and larger.

DS6-08. SERVICE LATERAL

A. GRADIENT: Four inch (4") sewer services shall have a minimum slope of 2%.

B. LOCATION AND ALIGNMENT: Sewer services shall be at right angles or radial to street right-of-way. The location shall be stationed on the plans. Services shall be located near the center of each parcel, however not located within driveways, and shall be not less than 10 feet from water services, fire hydrants, street lights, etc. In cul-de-sac bulbs services should enter manholes.

C. SIZE: Minimum size for single family dwellings is 4-inch. Minimum size for commercial, apartments and industrial developments shall be 6 inches.

D. DEPTH: Sewer services shall have 5-foot to 5-foot, 6-inches of cover at the right-of-way line, and 12 inches at any buildable location within the properties to be served.

E. CLEANOUTS: Cleanouts shall be installed on the service lateral at the back of sidewalk as shown on the Construction Details.

F. IDENTIFICATION: Sewer laterals shall be identified with an "S" stamped or etched on the top of curb.

G. CONNECTIONS TO LARGE MAINS: Sewer service may be directly connected to sewer mains smaller than 12 inches in diameter. For trunk sewer lines 12 inches and larger in diameter, or more than 15 feet in depth, the service sewer may be directly connected only with the approval of the City Engineer.

H. TYPE OF PIPE: Same as sewer mains. Cleanout assemblies and service to site from the cleanout may be ABS per Construction Detail 6020.

I. ONSITE CONNECTIONS: Storm runoff shall not be designed to enter the sanitary sewer system.

J. Each parcel within commercial and industrial districts, including multi-family development service laterals, shall connect to a sewer main manhole unless approved otherwise by the City Engineer.

DS6-09. TRENCH LOADING: For sanitary sewer lines over 10 feet deep, Marston's formula shall be used to determine the load placed on the pipe by backfill. The procedure for rigid pipe is described in the ASCE Manual of Engineering Practice No. 60, the Clay Pipe Engineering Manual, and in similar handbooks. The Design Engineer shall determine the factor of safety. Only the three edge bearing strength of the pipe shall be used in the computations for rigid pipe. The minimum trench width shall be O.D. plus 12 inches.

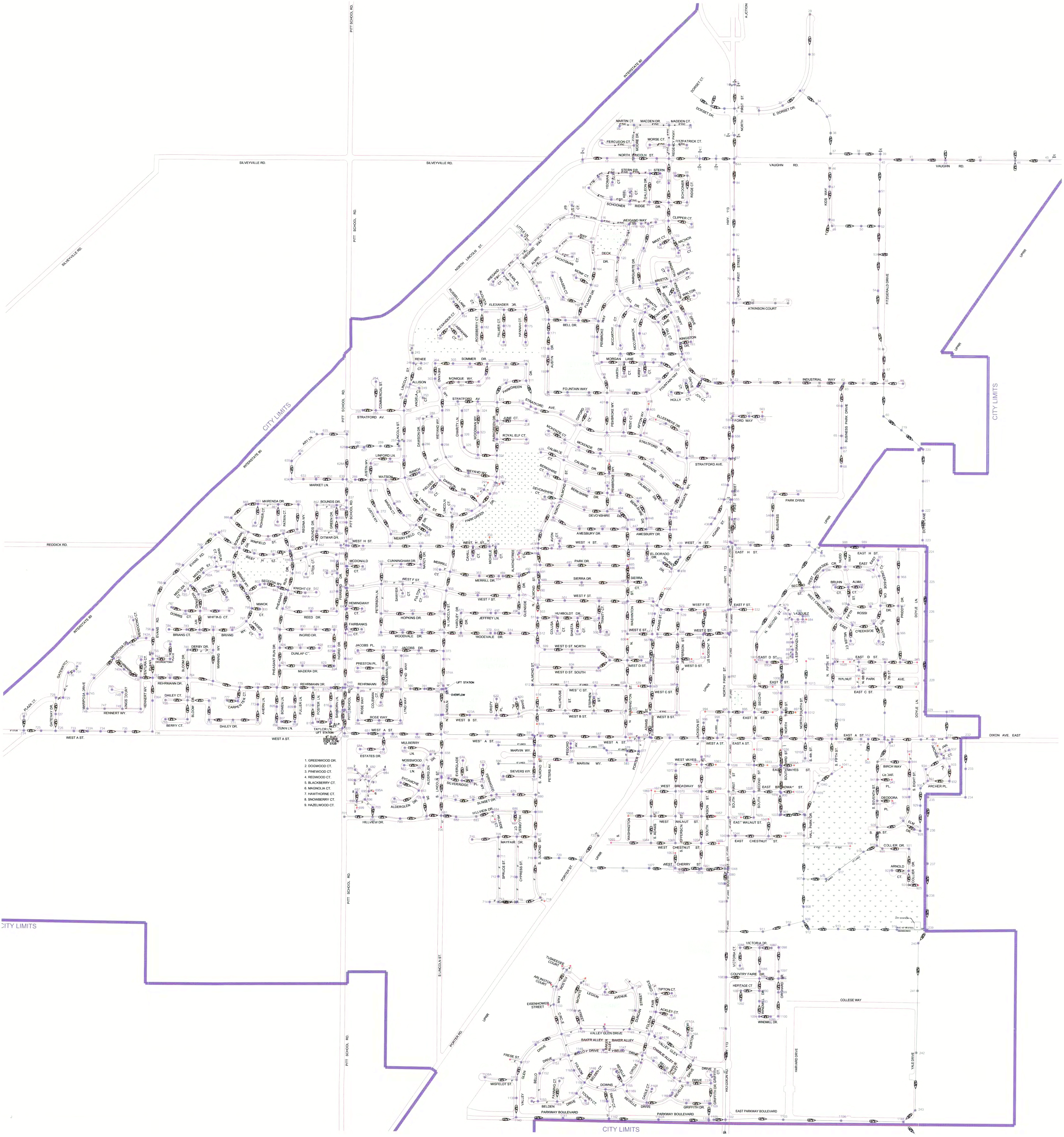
DS6-10. BEDDING AND INITIAL BACKFILL: Bedding types and factors for V.C.P. shall be as per Construction Details 3280 and 3290. For other materials, the trench width, bedding, and initial backfill shall be consistent with the pipe manufacturer's requirements. Bedding and initial backfill type shall be as necessitated by depth of cover over the pipe, trench width, pipe strength, and other factors used to determine safe pipe loading. Any special backfill requirements shall be noted on the plans.

DS6-11. LIFT STATIONS: Lift stations shall not be permitted unless specifically approved by the City Engineer in advance of plan submittal.

DS6-12. UNUSUAL DESIGN: Special designs of sanitary sewer facilities or other unusual features or structures will require individual study and approval by the City Engineer.

V&A Project No. 19-0071

consulting engineers
1000 Broadway, Suite 320
Oakland, CA 94607
510.903.6600
510.903.6601 Fax


CITY LIMITS
INTERSTATE 80
SILVEYVILLE RD.
PITT SCHOOL RD.
VAUGHN RD.
NORTH FIRST ST.
NORTH LINCOLN ST.
REDDICK RD.
WEST A ST.
DIXON AVE EAST
INDUSTRIAL WAY
STRATFORD AVE.
PARK DRIVE
BUSINESS PARK DRIVE
FITZGERALD DRIVE
DOYLE LN.
WEST H ST.
EAST H ST.
UPRR
S. LINCOLN ST.
PORTER RD.
HWY 113
COLLEGE WAY
HARVARD DRIVE
YALE DRIVE
VALLEY GLEN DRIVE
EAST PARKWAY BOULEVARD
PARKWAY BOULEVARD
LIFT STATION
1. GREENWOOD DR.
2. DOGWOOD CT.
3. PINEWOOD CT.
4. REDWOOD CT.
5. BLACKBERRY CT.
6. MAGNOLIA CT.
7. HAWTHORNE CT.
8. SNOWBERRY CT.
9. HAZELWOOD CT.

MASTER SEWER STUDY

VALLEY GLEN SOUTH AREA

City of Dixon, California

Prepared for:

Richland Communities

April 2015

WOOD RODGERS

DEVELOPING INNOVATIVE DESIGN SOLUTIONS

MASTER SANITARY SEWER STUDY

FOR

VALLEY GLEN SOUTH AREA

City of Dixon, California

April 2015

Prepared for:

Richland Communities

Prepared by:

REGISTERED PROFESSIONAL ENGINEER
PHILIP J. TABOR
No. 71802
Exp. 12-31-15
CIVIL
STATE OF CALIFORNIA

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS
3301 C Street, Bldg. 100-B Tel: 916.341.7760
Sacramento, CA 95816 Fax: 916.341.7767

Accepted By:

City of Dixon

TABLE OF CONTENTS

APPENDICES

Project Description

The 210-acre Valley Glen residential development is located in South Dixon, located adjacent to South First Street (State Route 113) on the west side.

The proposed 210-acres project is located within the City of Dixon and is represented by the “Valley Glen Tentative Map” as included in **Appendix A**. The project is bound to the north by existing residential homes along West Cherry Street, to the east by Highway 113, south by rural farmland adjacent Parkway Boulevard, and west by the Union Pacific Rail Road. See **Figure 1: Vicinity Map** locating the project.

The project consists of seven single-family residential villages containing 676 single family dwelling units, a 3.75-acre commercial site, a 4.4-acre condominium site, a 4.7-acre apartment site, and a 5.0-acre park.

Due to the relatively flat topography across the site in and in the surrounding areas, the sewerage serviceability for the site is provided from both the north and south, in West Cherry Street and Parkway Boulevard, respectively.

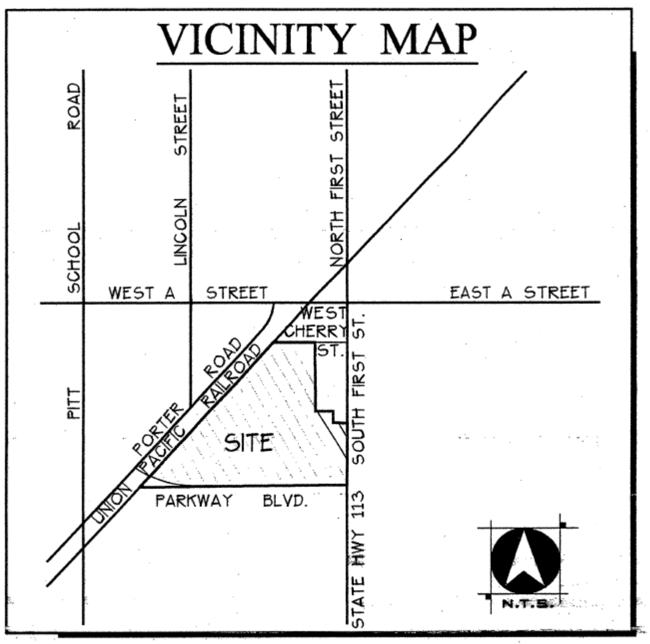


Figure 1: Vicinity Map

Purpose & Background

This study has been prepared at the request of the City of Dixon to summarize sewer infrastructure requirements for the Valley Glen South Sewer Shed at full build out of the community. Several iterations of sewer studies have been prepared in the past for the entire project site, identifying infrastructure needs and connection points in support splitting site service into a north and south shed area. Details of current City design requirements are also provided.

<u>North Area</u>

Wood Rodgers Inc. previously prepared and received City approval on the “Master Sewer Study- North Area (August 2007)” (**Appendix B**), whereby the trunk sewer connection was identified as an offsite facility anticipated to traverse east of Lot E (Condominium Site) across the adjacent 36th District Agricultural Association vacant lot and connect to the existing trunk facility in highway 113.

Due to complications in obtaining a sewer easement across the Ag Association’s vacant lot, supplemental analysis was provided by LJ Consultants Inc., evaluating routing north area sewer flows through the existing collector pipe in West Cherry Street. As it relates to the South Area study, the result of this supplemental evaluation allowed for a shed shift to occur, where 27 single family dwelling units, near the intersection of Iowa Ct/ Presidio Street and West Cherry, were re-routed to the south service area (pictorially identified in Appendix D). The related North Area shed map as provided in the "Valley Glen – Sewer Study – North Area (April 2014)" by LJ Consultants is additionally shown in **Appendix C**.

<u>South Area</u>

Wood Rodgers Inc. previously prepared the "Master Sewer Study- South Area (August 2003)" shed map (**Appendix D**), delineating the shed boundary and identifying pipe capacities and depths. This report provides updates to all previous work and validates that sufficient pipe capacity exists to allow the shed shift (27 units north to south).

City Design Criteria

The criteria used in the development of this study are consistent with the City's Engineering Standards and Specifications, dated 2009. The methodology for estimating sanitary sewer flows from Section 6 are summarized in the excerpt below:

Flow - The design sanitary sewer flow in gallons per day (gpd) shall be calculated using the following formula:

$$Q_d = Q_p + I \,\&\, I \text{ , where}$$

Q_d = Design flow
Q_p = Peak flow = Average Daily Flow x Peaking Factor
I+ I = Infiltration & Inflow Factor

The average daily flow rates and the I+I (Infiltration & Inflow) factors for various land uses are shown in the following table:

DESIGN FLOWS		
LAND USE	**AVERAGE DAILY FLOW**	**I+I FACTOR**
Single-Family	350 gpd per unit	500 gpd per gross acre
Multi-Family	5000 gpd per net acre	500 gpd per gross acre
Commercial/Public	1500 gpd per net acre	500 gpd per gross acre
Industrial	2000 gpd per net acre	500 gpd per gross acre
Schools	5000 gpd per net acre	500 gpd per gross acre

*Note: Net Acres is assumed as 80% of Gross Acres.

The peaking factors to be used to calculate the peak flow are shown in the following table:

PEAKING FACTORS	
SHED AREA	**PEAKING FACTOR**
Shed area less than 500 acres	2.5
500 acres ≤ Shed Area ≤1,500 acres	2.2
Shed area greater than 1,500 acres	2.1

Pipe Capacity - Typically sewer mains shall be sized based upon the sewer flowing at 70% of pipe capacity using the following formula:

Manning's Formula: $Q = A(1.49/n)(R^{2/3})(S^{1/2})$, where

Q = Flow, in cubic feet per second (cfs)
A = Area of Pipe in square feet (sf)
R = Hydraulic Radius (Area/ Wetted Perimeter)
S = Slope of Pipe
n = Roughness of 0.013 or as recommended by the pipe manufacturer, whichever is greater

Pipe capacity, in all cases, shall be adequate to carry the design flow from the entire tributary area, even though said tributary area is not located within the project boundaries. Sewer trunk line design criteria shall be done on a case by case basis, as approved by the City Engineer.

Velocity - Sewer velocity shall be equal to or greater than 2 feet per second for all sewers when flowing full with a maximum velocity of 10 feet per second. Sewers which will exceed 50% capacity at ultimate development shall have their minimum design slope determined using a minimum velocity flowing full of 2 feet per second. Sewers which will not exceed 50% capacity at full development shall have a minimum design velocity flowing full of 2.5 feet per second.

Main Size - Minimum size sewer main shall be 8 inches.

Using Manning's Formula with a friction constant of n=0.013 (clay pipe), the following pipe capacity table was developed for 8" pipe.

Pipe Size	Min. Pipe Slope	"n" Value	100% Full	70% Full	50% Full
8"	0.0035	0.013	0.47 mgd	0.38 mgd	0.23 mgd

Collector Pipe Slope Design Exception

In the previous design standard excerpt, the City requires that when a collector pipe does not achieve 50% capacity at full build out, the minimum design velocity shall be 2.5 feet per second.

Using Manning's equation, this would require an 8" collector pipe to have a minimum slope increased to S=0.0052.

This development will have multiple occasions where 50% capacity will not be achieved, primarily on the northern and western edges of the South Sewer Shed. Due to the relatively flat topography of the project site and in combination with the fact that the site is served by a shallow drain system, implementation of the steeper minimum sewer slope of S=0.0052 would render some areas of the site not serviceable by the gravity system.

Under previous discussion with the City regarding pursuit of a design variance, and as documented in **Appendix B**, it was agreed that a minimum slope of S=0.0035 would be allowed for all segments EXCEPT the upper shed "dead end" run. Dead end runs would be designed for a minimum slope of 0.0052 (or greater), in accordance with the City's minimum velocity requirements.

House Service Design Exception

The vertical constraints imposed on the sewer system due to the flat terrain and shallow drain system, pose cover problems for a select number of individual house services even with implementation of the collector pipe slope design variance detail above. Meeting the City's vertical burial clearance requirements, as detailed in standard detail 6030, is not possible.

Under previous discussion between Wood Rodgers and the City, an alternate shallow service detail was agreed upon, where the minimum cover over the service lateral, as measured from finish grade at back of sidewalk, was allowed to be reduced to 36-inches. This detail is illustrated on the South Area shed map in **Appendix E**.

Summary & Results

This study expands upon a prior approved Valley Glen South Area Master Sewer Study (August 2003). The necessity of updating this study arose from Richland Communities desire to re-route trunk sewer facilities in the Valley Glen (VG) North Sewer Service Area.

In the North Sewer Service Area, the planned sewer connection traversing the undeveloped 36th District Ag Association Parcel, east of the VG Multi-Family Site and north of the existing Silveyville Cemetery, proved to be more challenging than anticipated due to difficulty in obtaining a required easement. As a result of this difficulty, other options needed to be explored. Additional analysis was provided by LJ Consultants demonstrating flows could be re-routed through existing pipe within West Cherry Street, in-lieu of the offsite connection across the Ag Association Property. The results of this analysis are contained in the Final Valley Glen- Sewer Study- North Area, shown in **Appendix C**.

In an effort to reduce the peak flow impact to the existing 8" sewer main within West Cherry Street, between the north eastern project limits and Hwy 113, Wood Rodgers has shifted as many single family lots as possible to receive gravity service from the South Service Area (27 lots total).

Through the application of design exceptions, as allowed by the City, all areas in the South Sewer Shed are estimated to be serviceable by gravity service. The resultant shed shift of 27 dwelling units from the North Area Shed to the South Area Shed had negligible impact on previous flow and pipe calculations, and had an acceptable level of flow increase in the existing project pipe network installed with the Phase 1 villages. All existing and future pipe segments within the South Sewer Shed area will not exceed currently published City design capacities. A brief project summary is provided below with the complete tabulated results shown in **Appendix E**.

Shed	Area (ac.)	Q(total) mgd	Amount Shifted from North to South (mgd)
North	22.2	0.124	-.026
South	172.8	3.766	+.026
Total	**195.0**	**3.890**	-

Sewer Study Pipe Segment	Pipe Segment Diameter	Downstream Pipe Capacity prior to 27 unit shift	Downstream Pipe Capacity following 27 unit shift
Node B11	8"	(0.200 mgd) (51% full)	(0.226 mgd) (55% full)
Node B2	10"	(0.386 mgd) (54% full)	(0.412 mgd) (56% full)

APPENDIX A

Revised Tentative Subdivision Map Substantial Conformance Exhibit

(Dated July 9, 2002)

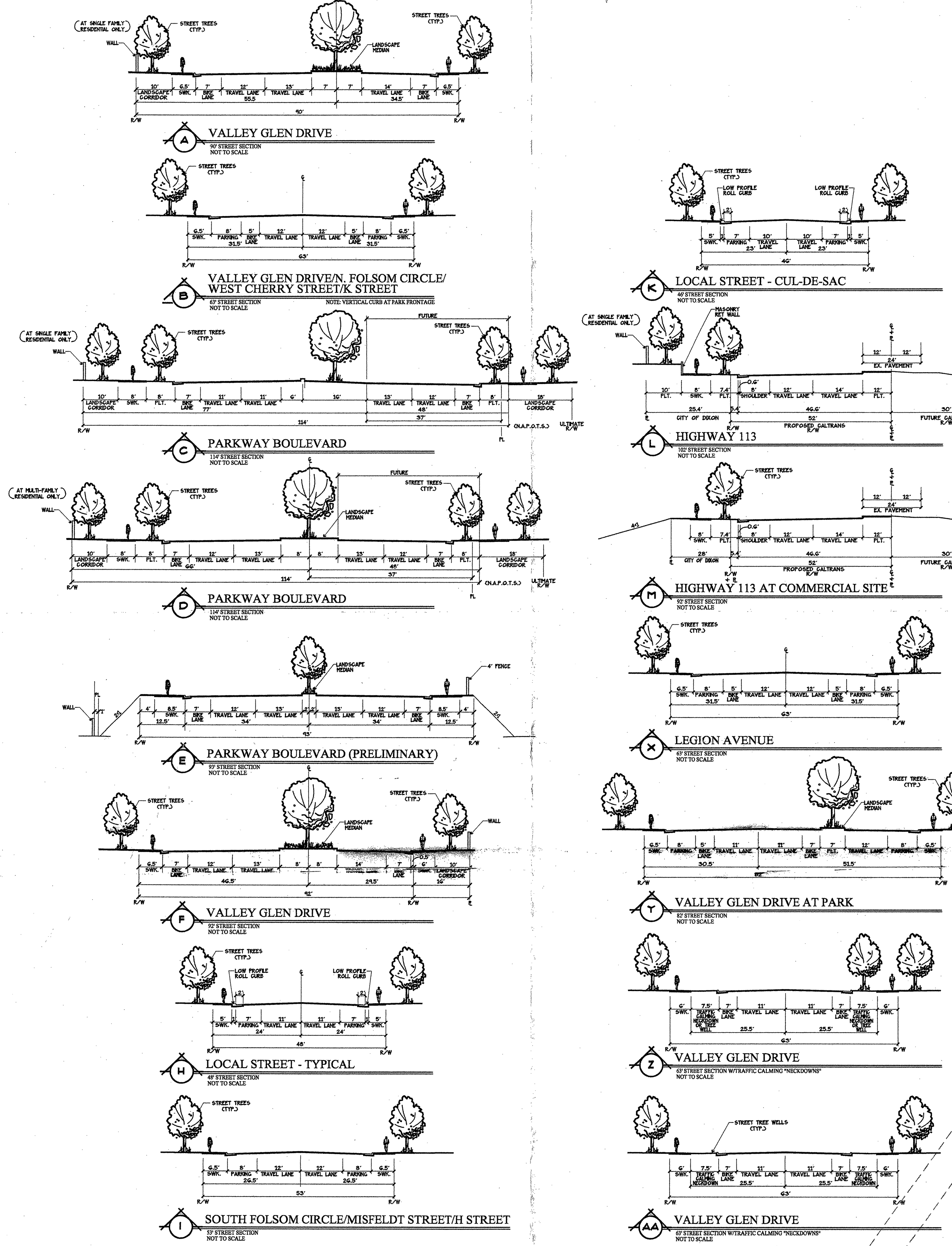
REVISED TENTATIVE SUBDIVISION MAP-SUBSTANTIAL CONFORMANCE EXHIBIT

RESOLUTION NO. 02-123 AS AMENDED JULY 9, 2002

VALLEY GLEN

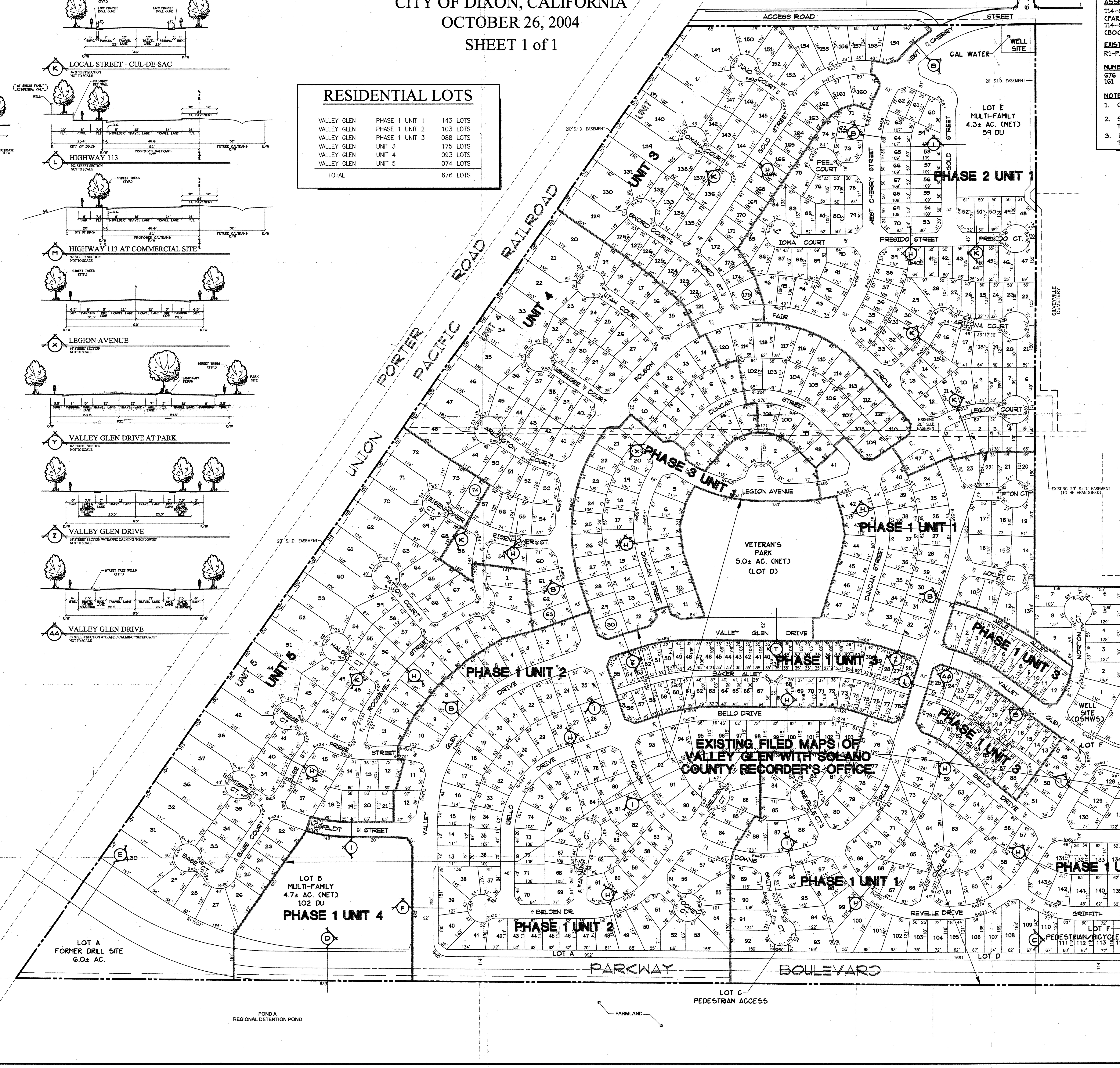
(FORMERLY SOUTHPARK)

CITY OF DIXON, CALIFORNIA
OCTOBER 26, 2004
SHEET 1 of 1



RESIDENTIAL LOTS

VALLEY GLEN	PHASE 1 UNIT 1	143 LOTS
VALLEY GLEN	PHASE 1 UNIT 2	103 LOTS
VALLEY GLEN	PHASE 1 UNIT 3	088 LOTS
VALLEY GLEN	UNIT 3	175 LOTS
VALLEY GLEN	UNIT 4	093 LOTS
VALLEY GLEN	UNIT 5	074 LOTS
TOTAL		676 LOTS



PROJECT NOTES

DEVELOPER
PULTE HOME CORPORATION
7031 KOLL CENTER PARKWAY, SUITE 150
PLEASANTON, CA 94566
CONTACT: JOHN JOHNSON
(925) 249-3215

ENGINEER
WOOD RODGERS, INC.
3301 'C' STREET, BLDG. 100-B
SACRAMENTO, CA. 95814
CONTACT: DENNIS PHILIPS/DONNA FRAGOSO
PHONE: (916) 341-7760

AREA
210.604 AC.

ASSESSOR'S PARCEL NO.
114-033-011 + -014 (JULY 9, 2002)
(PARCEL NUMBERS)
114-021 THRU -026
(BOOK/PAGE ONLY)

EXISTING ZONING
R1-PD, PMR-PD, RM-2-PD, CN-PD

NUMBER OF UNITS
676 SINGLE FAMILY DETACHED UNITS
161 MULTI-FAMILY UNITS

PRESENT USE
VACANT

STORM DRAINAGE
SANITARY SEWER
PARKS AND RECREATION
FIRE PROTECTION
POLICE PROTECTION
CITY OF DIXON

WATER
DSMWS AND
CAL. WATER

ELECTRICITY
PACIFIC GAS AND
ELECTRIC COMPANY

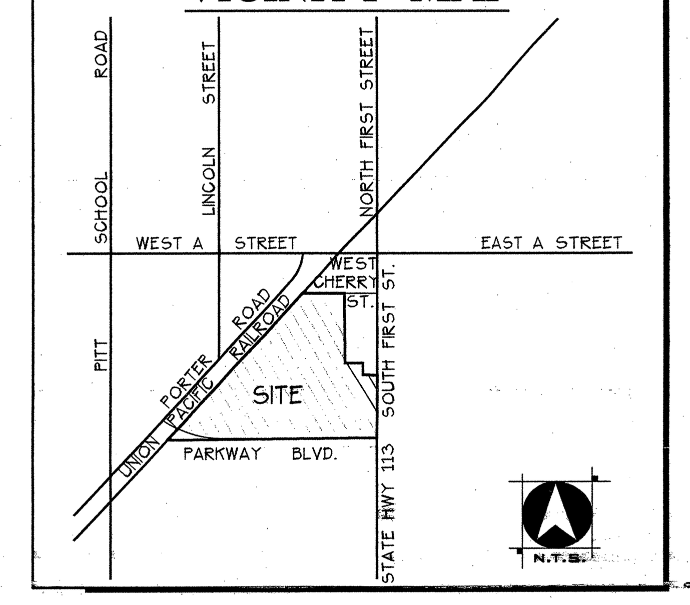
GAS
PACIFIC GAS AND ELECTRIC
COMPANY

TELEPHONE
SBC

SCHOOL DISTRICT
DIXON UNIFIED

NOTES:
1. GROSS ACREAGES INCLUDE ALL STREETS.
2. SUBDIVIDER RESERVES THE RIGHT TO FILE MULTIPLE FINAL MAPS PURSUANT TO SECTION 6463.1(a) OF THE SUBDIVISION MAP ACT.
3. UNITS 1 + 2 AND LOTS A-F ARE CONSISTENT WITH PREVIOUSLY APPROVED TENTATIVE MAP AND ARE SHOWN FOR ILLUSTRATIVE PURPOSES ONLY.

VICINITY MAP



APPROVAL

REVISED TENTATIVE MAP
SUBSTANTIAL CONFORMANCE EXHIBIT
APPROVED BY:

Rebecca Van Buren 12/8/04
REBECCA VAN BUREN — DATE
COMMUNITY DEVELOPMENT DIRECTOR, CITY OF DIXON

12/8/2004
RONALD J. TRIBBETT — DATE
CITY ENGINEER

LICENSED LAND SURVEYOR
No. 6046
Exp. 6-30-05

0 150 300 600
NORTH

WOOD RODGERS
ENGINEERING - MAPPING - PLANNING - SURVEYING
3301 C St, Bldg. 100-B Tel 916.341.7760
Sacramento, CA 95816 Fax 916.341.7767

REVISED TENTATIVE SUBDIVISION MAP-SUBSTANTIAL CONFORMANCE EXHIBIT

APPENDIX B

Previous Valley Glen North Area Master Sewer Study

(Prepared by Wood Rodgers; Approved August 2007)

FINAL

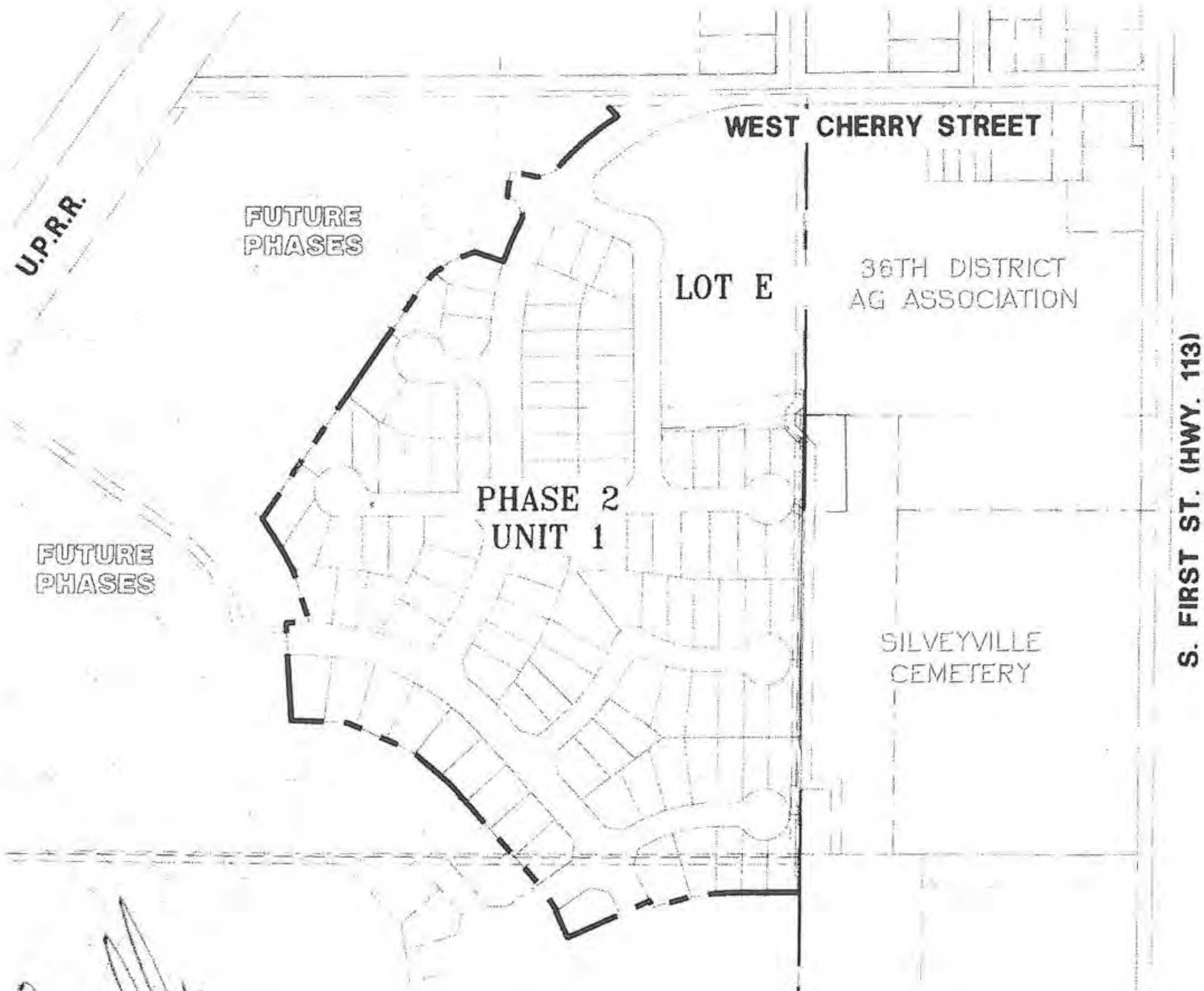
MASTER SEWER STUDY - NORTH AREA

VALLEY GLEN

PULTE HOMES

CITY OF DIXON **CALIFORNIA**

AUGUST 2007



REGISTERED PROFESSIONAL ENGINEER
DENNIS L. PHILLIPS
No. 50398
Exp. 6-30-09
CIVIL
STATE OF CALIFORNIA

ACCEPTED BY CITY OF DIXON

9-11-07

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

3301 C St. Bldg. 100-B
Sacramento, CA 95816

Tel 916.341.7760
Fax 916.341.7767

TECHNICAL MEMORANDUM

August 23, 2007
Mr. Chris Gioia
Associate Engineer
City of Dixon Engineering Department
Dixon, CA 95620

SUBJECT: Valley Glen ~ Master Sewer Study : North Area

1.1 SITE DESCRIPTION

The 210-acre Valley Glen residential development is located in south Dixon adjacent to South First Street (State Route 113). Existing infrastructure adjacent to the project are West Cherry Street to the north, Union Pacific Railroad on the West, and Parkway Blvd to the South.

The Valley Glen development includes 676 single-family lots, two 4.5-acre multi-family housing sites, a 3.75-acre commercial site and a 5.0-acre park.

1.2 EXISTING SEWER FACILITIES:

There is an existing 15" trunk line in West Cherry Street that routes sewer flows from the west side of the UPRR easterly to the intersection of West Cherry Street and HWY 113. There are also 8" and 15" West Cherry Street trunk sewer mains that connect into an existing lined 15" sewer main within HWY 113 and then flow south to the existing 27" trunk sewer main. There is an existing 27" trunk sewer main within Parkway Boulevard that was constructed with phase 1 to serve those improvements.

Currently there are 4 subdivisions within the project that have been constructed. These are: Phase 1~Units 1, 2, 3 and Phase 3 Unit 1. Portions of the project from the north area will obtain gravity sewer service through these existing subdivisions via existing sewer stubs as identified by the sewer shed boundaries as shown on the sewer study map. (See Appendix 'A')

1.3 PIPE SIZING CRITERIA AND MINIMUM SLOPE ANALYSIS:

In accordance with the City of Dixon Improvement Standards, the following average daily flow rates were used in generating the peak design flows for the shed areas shown on the study map.

LAND USE	AVERAGE DAILY FLOW	I+I FACTOR
Single Family	350 gpd/unit	500 gpd per acre
Multi-Family	5000 gpd per net acre	500 gpd per acre
Commercial	1500 gpd per net acre	500 gpd per acre
Industrial	2000 gpd per net acre	500 gpd per acre
Schools	5000 gpd per net acre	500 gpd per acre

Using Manning's formula with a friction constant of N=0.013, the following pipe capacity table was developed for 8" pipe.

PIPE SIZE	N VALUE	FULL FLOW	70% FULL	50% FULL
8"	0.013	0.47 MGD	0.33 MGD	0.24 MGD

Per the City of Dixon Improvement Standards, the minimum allowable sewer pipe size is 8" and all sewer pipes that will not exceed 50% of the full flow capacity shall have a minimum design velocity flowing full of 2.5 fps. Under a full flow Manning's calculation, the minimum slope for an 8" pipe to maintain a velocity of 2.5fps is S=0.0052.

Since none of the pipes within the northern sewer shed limits of Valley Glen will ever exceed 50% of their capacity at full development – the minimum velocity requirement of 2.5fps essentially would require all 8" pipes to be constructed at a slope of S=0.0052 for compliance with the Dixon Improvement Standards. In order to maximize the depth of our 8" collector sewer mains, improve sewer serviceability, and to therefore maximize the number of units that can serve through the existing phase 1 sewer stubs – we are proposing to lower the minimum slope requirement from S=0.0052 to S=0.0035 for all 8" sewer collector mains. The upper shed "dead end" 8" sewer mains which are typically located within cul-de-sacs would still be designed at the minimum S=0.0052 slope(or greater) in accordance with the minimum velocity requirement of the Dixon Improvement Standards.

The request in this study for a design standard variance of the minimum slope requirement is in accordance with the approved standard design variance for our phase 1 sewer studies and improvement plans that have already been accepted and approved by the City.

1.4 HOUSE SERVICE DESIGN

Even with the design velocity variance that would allow our 8" collector sewer pipes to be constructed at a grade of S=0.0035 in order to maximize both depth and serviceability, there still would be a remaining number of individual lots that would require a shallow sewer service design deviation from the City of Dixon typical house service standard detail 610. These lots are identified on the sewer study map included with this report. Our proposal for these lots is to construct house services at a minimum cover of 48" to finished grade.

1.5 FLOW MONITORING AND EXISTING CAPACITY SUMMARY:

In September 2003, Pulte Homes contracted ASC Controls Inc. to monitor the flows in the 15" trunk line in West Cherry Street and First Street (HWY 113) to determine the feasibility of routing the flows from Valley Glen Phase 2 Unit 1 and the multi-family development to either West Cherry Street and/or HW 113. (See attached flow results, Appendix 'B'). Since the data was sampled during this relatively dry month of September, the measured flows are considered to be dry weather flows, with any unknown but likely insignificant wet weather component.

Based on the results of the flow monitoring, the average dry peak flow in West Cherry Street is 430 gpm or 0.62 MGD. The average dry peak flow in HWY 113 is 1079 gpm or 1.55 MGD.

Since the 15" trunk main within HW113 was previously lined with a polyethylene material, the resultant capacity increase of this pipe must be considered.

Holding all other Manning's variables constant, the following equation represents the increase in pipe capacity:

$$\% \text{ Capacity Increase} = (n_{existing} / n_{new}) \times (\text{Dia}_{new} / \text{Dia}_{existing})^{8/3} \times 100$$

For the existing 15" trunk main we are using an N value of 0.014 for in-service clay pipe and a value of N=0.010 for Insituform CIPP lined pipe. A conservative estimate of ¼ inch was assumed for the liner thickness to determine the reduced pipe diameter.

% Capacity Increase for 15" Clay Sewer Pipe = $(0.014/0.010)\text{x}(14.5/15)^{8/3}$ x 100 = 1.278 **(28% increase)**

For the existing 15" sewer trunk main within SR113, the full flow capacity based on the existing pipe grades(S=0.002) is approximately 1.87MGD. Applying the pipe capacity increase due to the reduced 'N' value of the liner, the full flow capacity then becomes 1.28 x 1.87MGD = 2.39 MGD.

The resultant maximum design capacity for a 15" lined clay pipe flowing at 70% of capacity at the existing pipe grades(S=0.002) then becomes, 0.7 x 2.39 = **1.67 MGD**. This is the maximum design capacity for the HW 113 15" HDPE lined sewer pipe at a grade of S=0.002.

Subtracting the average measured peak flow of 1.55 MGD from the 70% max design flow of 1.67 MGD results in an existing capacity of **0.12 MGD** within the existing 15" lined trunk main within HW 113. Under full flow conditions for the 15" lined trunk main, the available capacity within the existing pipe is 2.39 MGD-1.55 MGD = **0.84 MGD**.

1.6 VALLEY GLEN SEWER FLOW SUMMARY:

Sewer Lateral 'A':

There are a total of 106 single family lots within the Phase 2 Unit 1 subdivision. Of the 106 unit total, 46 units will obtain sewer service through an existing phase 1 sewer stub and the 60 remaining lots will serve through the proposed sewer lateral 'A' as shown on the study map. In addition to the 60 units from phase 2 Unit 1, there will be 36 units from a portion of a future phase that will also serve through the proposed sewer lateral 'A' shed area as shown on the study map. Combining the 36 future phase units with the 60 units from Phase 2 Unit results in a total of 96 lots that will obtain sewer service through the proposed sewer lateral 'A' connection across the Dixon May Fair Lot. The flow calculation summary for sewer lateral 'A' is as follows:

Remainder of Phase 2 Unit 1: 60 Units ; Qa=60x350 gpd/unit = 0.021 MGD

Remainder of Future Phase Area: 36 Units ; Qa=36x350gpd/unit = 0.013 MGD

Single Family Total = 0.012+0.013=0.034 MGD

Multi Family Parcel: 4.21 acres @ 5000gpd/acre = 0.021 MGD

I&I = 40.2 acres x 500gpd/acre = 0.020 MGD

Total Remaining North Area Qpwwf = (Sum of Qa x PF) + I&I = 0.055 x 2.5 + 0.021 = **0.16 MGD**

Future Phases:

The remaining units of future Valley Glen phases within the north area will all obtain sewer service through the existing 8" sewer stubs that were constructed with the Phase 1 Unit 1 and Phase 3 Unit 1 subdivisions as shown on the sewer study map.(See Appendicex 'A') With the future improvement plans for these remainder phase areas, the existing 8" mains will be extended within the streets into the remaining areas at the minimum pipe size of 8" in accordance with the Dixon Improvement Standards. As with Phase 1 and the proposed alternative minimum pipe slopes identified within this study, these future extensions of 8" sewer pipe will be proposed to continue at the minimum design slope variance of S=0.0035. At the "dead end" runs within future cul-de-sacs, the minimum slope will increase to S=0.0052 or greater.

HW 113:

Since the City has previously stated that any proposed sewer solution for the north area must replace and/or preserve the sewer capacity within existing trunk mains, we are proposing to construct a new manhole at the sewer lateral 'A' tie in point at HW 113 and accordingly replace the existing 15" main with a new 18" trunk main that will terminate at the location where the existing 27" trunk main begins.(See sewer study map, Appendix 'A'). With this trunk main replacement, the north area of the Valley Glen Project will produce no net capacity decrease within the existing system. The sewer capacity information for the new 18" trunk main based on a Manning's calculation for circular pipe is identified in the table below:

PIPE SIZE	N VALUE	SLOPE	CAPACITY(FULL)	CAPACITY(70%)
18"	0.013	S=0.002	3.04 MGD	2.13 MGD

As previously identified in section 1.5, the measured peak flow within the existing HW 113 15" trunk main is approximately 1.55 MGD. Combining the proposed peak sewer flow from sewer lateral 'A'(0.16 MGD) with the existing measured peak flow within HW 113(1.55 MGD) results in a combined peak flow of 1.71 MGD for the proposed 18" trunk main. Approximately 0.42 MGD of usable capacity would remain in the proposed 18" trunk main under the 70% capacity requirement per the City Standards. During the winter months and under the full flow conditions of the proposed 18" trunk main, approximately 1.32 MGD of capacity would remain in the proposed 18" trunk main for both future development as well as peak wet weather infiltration.

1.7 PREVIOUSLY IDENTIFIED NORTH AREA SEWER STUDY ALTERNATIVE:

West Cherry St:

As identified in previous submittals of sewer study alternatives to the City, the West Cherry St sewer connection point was an option that was explored in an effort to provide sewer service for the north area of the Valley Glen Project. Since the time of those studies, the City has stated the Valley Glen Project would be prohibited in using any of the existing available capacity within the West Cherry St sewer systems since that capacity has already been allocated to future development located elsewhere within the City.

In addition to the issue of existing capacity reserved for future development, City maintenance staff has also indicated that there is a large(>1 MGD) wet weather infiltration flow that is currently present within the 15" West Cherry St trunk main during heavy rain periods. Since these wet weather flow contributions have already exceeded the capacity of the existing 15" system to the point where, according to City maintenance staff, the system is surcharged at times – a sewer alternative involving the use of the West Cherry Street trunk sewer facilities would be required to upsize those facilities several pipe sizes in order to provide a viable sewer connection for the north area of the Valley Glen Project. In addition to the upsizing of the 15" West Cherry St trunk main – this alternative would also require that the existing 15" trunk main that flows southerly within HW 113 to be upsized as well.

In considering the magnitude of sewer pipe replacement both in West Cherry St. and HW 113, the interruption to existing water and sewer service laterals to existing residences along West Cherry and HW 113, the interruption to existing traffic flows within the excavation limits of trunk sewer main replacement areas, and the lengthy sewer flow bypass operation that would be required prior to the new trunk mains becoming operational, the West Cherry St. design alternative was abandoned in favor of the Dixon May Fair sewer lateral 'A' alternative.

Silveyville Cemetery Sewer Lateral:

This sewer alternative involved a sewer lateral that proceeded through the internal cemetery roads in an easterly direction eventually connecting into the HW 113 trunk sewer via a new manhole. Due to the problematic issues related to the depth of excavation through a very narrow and sensitive construction corridor, this alternative was appropriately abandoned.

Service Through Phase 1:

The depth of the phase 1 sewer systems, while built at minimum grade, would not provide adequate serviceability to the upper reaches of the sewer shed to the north. Since this alternative could not provide a sewer solution that addressed the needs of the entire remaining Valley Glen project area, this alternative was also abandoned.

Southwest Dixon Sewer Trunk Main:

The City brought to our attention that once the 27" sewer trunk main is extended up to 'A' street per the Southwest Dixon Plan, that there would be additional available capacity within the 15" trunk sewer of West Cherry St due to the upstream flows being rerouted. While initially thought that this offsite improvement could make the West Cherry St connection a more viable alternative, the timing of these offsite improvement by others would be difficult to estimate as it related to providing service to Valley Glen.

1.8 SUMMARY

The following table summarizes the capacity increases for the 15" to 18" upsize of trunk sewer main within HW 113:

Pipe Size/Material	N Value	Slope	Full Capacity (MGD)	70% Capacity (MGD)
Existing lined 15"	0.010	0.002	2.39	1.67
18" VCP	0.013	0.002	3.04	2.13
		Capacity Increase:	0.65 MGD	0.46 MGD

The Phase 2 Unit 1 peak design flow and the existing dry weather peak flow within HW 113 total to 1.71 MGD. In subtracting this flow from the full flow capacity of the new 18" trunk main, the total remaining flow capacity within this section of HW 113 is 1.33 MGD. (3.04 MGD-1.71 MGD=1.33 MGD)

The primary advantages of sewer lateral 'A' is that it bypasses the twin issues of available capacity and wet weather flow within West Cherry St. by providing a new 8" connection across the Dixon May Fair Lot. This alternative accomplishes the primary objective of providing a point of connection for the north area of the Valley Glen Project as well as minimizes the impact to existing available sewer capacity, roadway infrastructure, and traffic control measures during construction.

If you have any comments or questions about the information presented in this study, I can be reached at 916-341-7712. Thank you.

Sincerely,

Ron Conn

Wood-Rodgers, Inc.

APPENDIX

- Exhibit 'A' : Valley Glen Phase 2 Unit 1 Sewer Study Map
- Exhibit 'B' : ASC Sewer Flow Monitoring Data

APPENDIX 'A'

APPENDIX 'B'

NORTH MASTER SEWER STUDY

VALLEY GLEN

PULTE HOMES

CITY OF DIXON **CALIFORNIA**

MAY 2007

VALLEY GLEN

City of Dixon

Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.76

Note:

Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

ASSUMPTIONS:

Area = 40.2 Acres
Single family residences = 96 Units
Multifamily Area = 4.21 Acres
Peaking Factor = PF = 2.5
Peak Wet weather peak flows = Q_{PWWF}
Average Flows = Q_A
Peak flows = Q_P

FLOW CALCULATIONS: SEWER LATERAL 'A'

$Q_P = Q_A \times PF$
$Q_{PWWF} = \sum Q_P + I\&I$

Q_A (Single family) = 350gpd/unit x 96 units = 0.034 mgd

Q_A (Multifamily) = 5000 gpd per acre x 4.21 acres = 0.021 mgd
$\sum Q_A = 0.034 + 0.021 = 0.055$ mgd

I&I = 500 gpd per acre x 40.2 Acres = 0.02 mgd

$Q_{PWWF} = (\sum QA \,.\, PF) + I \& I$

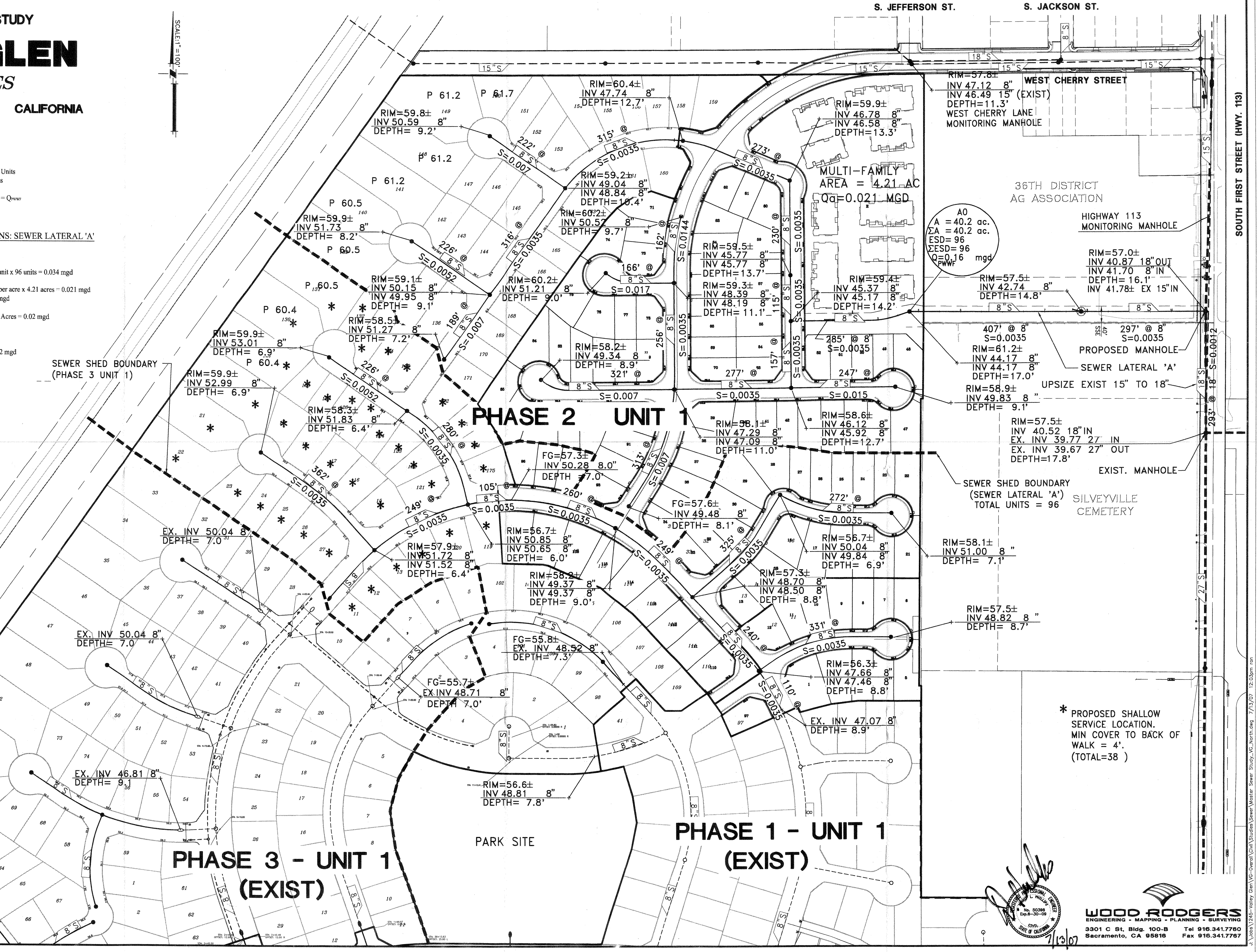
$Q_{PWWF} = (0.055\text{mgd} \,.\, 2.5) + 0.02$ mgd

$Q_{PWWF} = 0.14$ mgd + 0.02 mgd

$Q_{PWWF} = 0.16$ mgd

LEGEND

8"S ← PROPOSED SEWER PIPE (INCHES)
PROPOSED SEWER MANHOLE
PROJECT BOUNDARY
NODE NUMBER
POINT ACREAGE
CUMULATIVE ACREAGE
POINT ESD
CUMULATIVE ESD
PEAK WET WEATHER FLOW (MGD)



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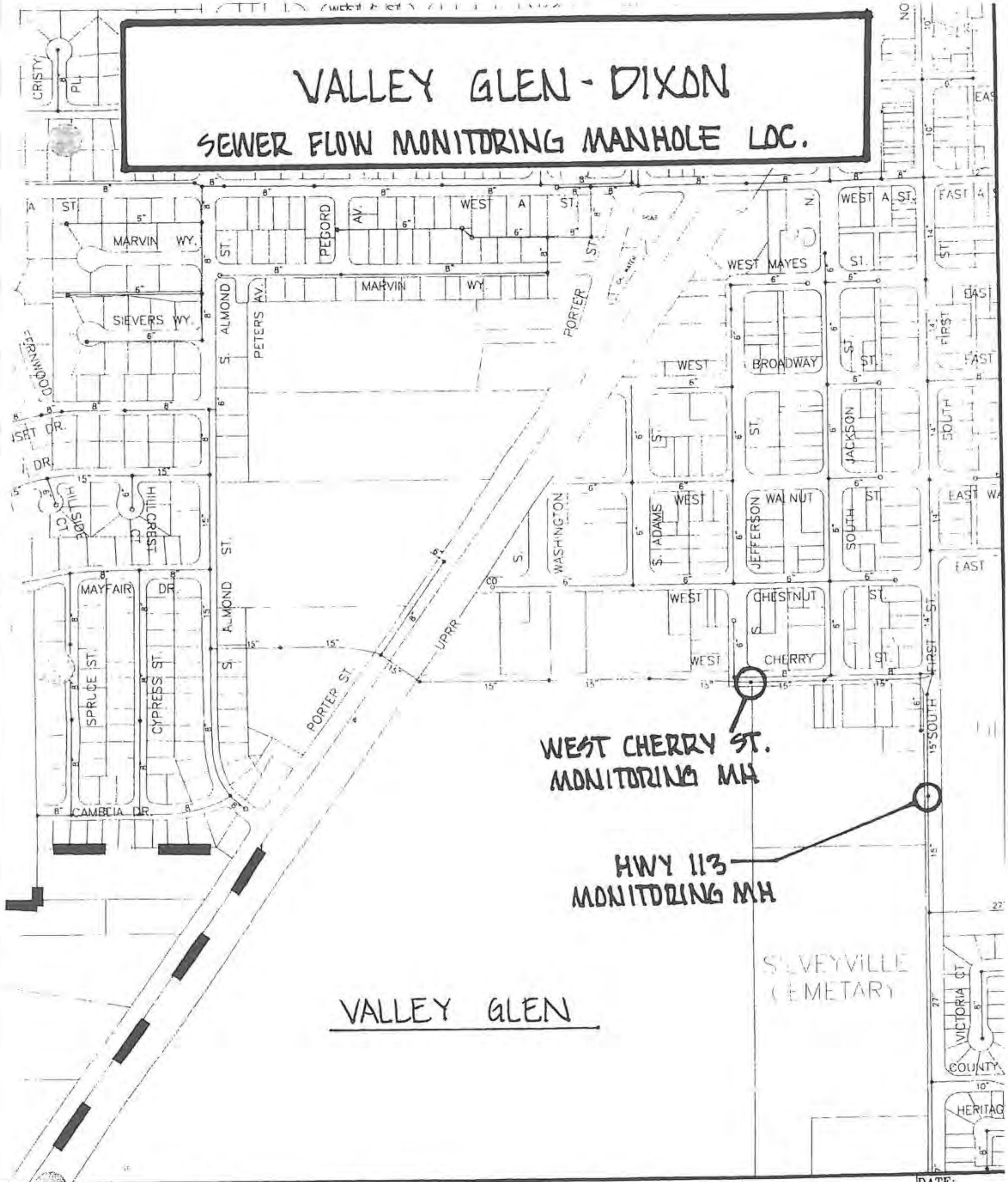
ENGINEERING • MAPPING • PLANNING • SURVEYING
3301 C St, Bldg. 100-B Tel 916.341.7760
Sacramento, CA 95816 Fax 916.341.7767

7/13/07

APPENDIX 'B'



CITY OF DIXON
DEPARTMENT OF PUBLIC WORKS
600 EAST A STREET DIXON, CA 95620
PHONE: 707/678-7030 FAX: 707/678-7039

CITY OF DIXON
SEWER SYSTEM MASTER PLAN

DATE: DEC. 2002
SHEET: 10
OF: 13

ASC Controls, Inc.

1650 Industrial Drive
Auburn, CA 95603
(800) 649-4287
(530) 823-3241
Fax: (530) 823-3475
www.aquasierra.com

October 2, 2003

Dennis Phillips, P.E.
Wood Rodgers
3301 C Street, Bldg. 100-B
Sacramento, Ca. 95816

Subject: Flow Reports
Re: City of Dixon, Hwy 113 and Cherry Ave Manholes

Dear Dennis

Please review the following reports submitted for your review. The reports reflect the flow study performed on the 15-inch sewer mains at the City of Dixon. Two ISCO 2150 Area velocity flow data Loggers were installed for this test. Meters were configured for one concrete lined pipe on hwy 113 manhole and a non-lined 15-inch main on Cherry Ave. Test period seven days continuos with removal of the meters on the eight day. Rep[orts include a 7-day flow graph and a flow graph at 15-minute intervals for each location, Plus day/time/flow printed reports. Let me know if you have any questions or comments. Thank you for using our firm as your technical consultant.

Les Watson
Applications Engineering/Sales

MONITORED FLOW SUMMARY (See Referenced Sewer Exhibit for Monitored Manhole Exhibit)

West Cherry Street

	FLOW RATE (GPM)		
DAY	AVE	HIGH	LOW
TUES SEPT. 23	190	440 @ 8:30 AM (24TH)	50 @ 3:45 AM (24TH)
WED SEPT. 24	180	420 @ 8:30 AM (25TH)	30 @ 3:30 AM (25TH)
THURS SEPT. 25	180	580 @ 7:30 PM (25TH)	30 @ 3:45 AM (26TH)
FRI SEPT. 26	200	400 @ 10:00 AM (27TH)	60 @ 6:15 AM (27TH)
SAT SEPT. 27	220	400 @ 11:45 AM (28TH)	40 @ 5:00 AM (28TH)
SUN SEPT. 28	220	390 @ 8:30 PM (28TH) & 8:30 AM (29TH)	30 @ 4:00 AM (29TH)
MON SEPT. 29	170	380 @ 8:00 AM (30TH)	30 @ 4:45 AM (30TH)
7 DAY AVERAGE	194	430	39

HWY 113

	FLOW RATE (GPM)/ TIME OF OCCURRENCE		
DAY	AVE	HIGH	LOW
TUES SEPT. 23	580	1100 @ 8:15 AM (24TH)	170 @ 3:45 AM (24TH)
WED SEPT. 24	600	1090 @ 8:30 AM (25TH)	200 @ 3:30 AM (25TH)
THURS SEPT. 25	570	1240 @ 7:45 PM (25TH)	170 @ 3:45 AM (26TH)
FRI SEPT. 26	540	1010 @ 10:30 AM (27TH)	150 @ 6:15 AM (27TH)
SAT SEPT. 27	610	1150 @ 11:30 AM (28TH)	170 @ 5:00 AM (28TH)
SUN SEPT. 28	640	1010 @ 12:45 PM (28TH)	180 @ 4:00 AM (29TH)
MON SEPT. 29	600	950 @ 9:15 PM (29TH) & 8:45 AM (30TH)	180 @ 4:45 AM (30TH)
7 DAY AVERAGE	591	1079	179

APPENDIX C

Valley Glen North Area Master Sewer Study

(By LJ Consultants Inc., dated April 2014.)

April 21, 2014

Final Valley Glen – Sewer Study – North Area

SITE DESCRIPTION:

Valley Glen is a 210-acre residential development located in south Dixon adjacent to South First Street, State Highway 113. The project as approved consists of 676 single family lots, two multi-family sites, a commercial site and a neighborhood park. The existing sewer facilities surrounding the project are as follows: There is a 27-inch sewer main in Parkway Boulevard that serves the majority of the Valley Glen site. In West Cherry Street there is a 15-inch trunk sewer main that serves areas west of the UPRR. This sewer main ties into a lined 15-inch trunk main that runs north –south in South First Street (HWY 113) and ties into the 27-inch main in South First Street.

BACKGROUND:

The Valley Glen development was originally approved to sewer entirely southerly into the 27-inch sewer main in Parkway Boulevard with the original project approval in 2002. In July 2007 Wood Rodgers prepared a Sewer Master Study, entitled "Master Sewer Study – North Area". This study requested the approval of 94 single family lots located in the northern section of Valley Glen in Units 2 and 3 and the 4.42 acre multi-family site to sewer through an 8-inch connection running easterly through the State of California AG Property to the existing trunk main in South First Street. The report was approved by the City of Dixon on September 11, 2007. Most recently in March 2013, the City of Dixon approved the 4.42 acre multi-family site to sewer northerly into the existing 15-inch sewer main in West Cherry Street.

This study will demonstrate that the sewer flow from 69 single family lots located in Units 2 and 3 can sewer north to the existing 15-inch sewer in West Cherry Street. As a part of this study, upstream vacant parcels that are tributary to this wastewater shed will be included in the analysis to ensure that the future parcels will have capacity in the existing lines. The remaining 25 lots will sewer southerly into the existing sewer system located within Valley Glen. These 25 lots are addressed in the Wood Rodgers "South Master Sewer Study" dated August 2003 and updated in March 2014. The March 2014 update is a draft version as it is being reviewed by the City. A draft exhibit is included in this study as Exhibit 'J'.

The northern areas of Valley Glen that requires sewer into the West Cherry Street are as shown on the attached sewer shed boundaries map, see exhibit 'A'. The vacant parcels that are tributary to the wastewater shed are shown on exhibit 'B'.

ACCEPTED
CITY OF DIXON

REGISTERED PROFESSIONAL ENGINEER
LISA ANNE SALAZAR
C53460
6/30/17
CIVIL
STATE OF CALIFORNIA

LJConsultants
www.LJ-Consultants.com
250 Cherry Lane
Suite 109
Manteca, CA 95337

CITY OF DIXON STANDARDS:

In accordance with the City of Dixon Engineering Design Standards, the following daily flow rates were used in generating the peak design flows the shed areas as shown on the study map.

LAND USE	AVERAGE DAILY FLOW	I+I FACTOR
Single Family	350 gpd/unit	500 gpd per gross acre
Multi-Family	5000 gpd per net acre	500 gpd per gross acre
Commercial / Public	1500 gpd per net acre	500 gpd per gross acre
Industrial	2000 gpd per net acre	500 gpd per gross acre
Schools	5000 gpd per net acre	5000 gpd per gross acre

*note: net acreage is assumed as 80% of gross acreage

Using Manning's formula with a friction constant of N=0.013, the following pipe capacity table was developed for the 8" sewer main.

PIPE SIZE	N VALUE	FULL FLOW	70% FULL	50% FULL
8"	0.013	0.56 MGD	0.47 MGD	0.28 MGD

As stated in the City of Dixon Engineering Design Standards, the minimum allowable sewer pipe size is 8-inch and all sewer pipes that will not exceed 50% of the full flow shall have a minimum design velocity flowing full of 2.5 fps. Under a full flow Manning's calculations, the minimum slope for an 8" pipe is S=0.0052 in order to maintain the 2.5 fps velocity. With the sewer flows being minimal in the area none of the pipes will ever exceed the 50% capacity at build-out; therefore the minimum slope requirement per the design standards would need to be S=0.0052. This study is requesting for a design standard variance of the minimum slope requirements to be S=0.0035 in-lieu of S=0.0052. The variance is being requested to limit the number of shallow sewer services at the individual homes. Per the Wood Rodgers "North Master Sewer Study" dated November 2013 there are 4 lots that will require shallow sewer services.

It is noted that a variance in slope requirements is acceptable per the City of Dixon as per the approved sewer study shown in the "Valley Glen Master Sewer Study – North" dated July 2007 as prepared by Wood Rodgers.

With the approval of this study, the previously approved sewer located in Gold Street adjacent to the multi-family site will need to be removed and replaced per the revised plans as prepared by Wood Rodgers entitled "Valley Glen Off-Site Improvements Phase 2 Lot E" Sheets 8 and 9, dated December 2012, revision No. 2. Along with the removal and replacement of the sewer in Gold Street there are some additional changes to the design of the remaining improvements in Phase 2, unit 1. Per Wood Rodgers the following design changes were necessary to accommodate the sewer flowing northerly to the existing 15-inch line in West Cherry Street. The storm drain lines in West Cherry Street are proposed to be on the non-standard side of the street along with the water

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and sewer mains as well. This allows for the sewer services to maintain the standard cover requirement along the east side of West Cherry Street. At select locations the clearance between utilities will be reduced to six inches.

FLOW MONITORING AND EXISTING CAPACITY IN 15-INCH MAIN IN WEST CHERRY STREET AND SOUTH FIRST STREET:

WEST CHERRY STREET:
In February and March of 2013, Veolia Water conducted a flow monitoring study to determine the normal flow rate in the West Cherry Street sewer main in addition to the I/I rate. The monitoring was documented and is attached as Exhibit 'C'. After review and comment by the City of Dixon it was determined to do additional flow monitoring of the 15-inch main in South First Street. This monitoring was done for 7 days in July; the results are attached as Exhibit 'D'. The sampling was done in the manhole south of West Cherry Street in front of the State of California AG Property.

The monitoring performed for West Cherry Street measured a peak flow of 0.64 MGD which includes an I/I factor of 0.11 MGD. It should be noted that the peak flow was solely dependent on the release from the pump station and was experienced every 12 hours when the pump station released into the 15-inch trunk main.

PIPE SIZE	N VALUE	SLOPE	FULL FLOW	70% CAPACITY
Ex. 15" VCP	0.013	0.002	1.87 MGD	1.56 MGD

SOUTH FIRST STREET:

The monitoring performed for South First Street measured the depth of flow in the main; the peak depth was 8.11 inches. The 15-inch trunk main in South First Street was lined with a PVC liner, per the City of Dixon Engineering Department. The peak flow was calculated using the following existing parameters:

Diameter = 14.5 inches
Manning's Coefficient N=0.010
Slope=0.002

The peak flow for the 15-inch trunk main on South First Street in front of the State of California AG Property was determined to be 902 GPM or 1.33 MGD. See attached calculations, Exhibit 'F'.

Since this study was done during dry conditions and per the City of Dixon Engineering Design Standards an I/I factor is required to determine the total flow in the 15-inch sewer main. This study is taking a conservative approach and applying the same I/I factor that was experienced during the wet weather flow monitoring of the 15-inch main in West Cherry Street. The peak flow for West Cherry Street was determined to be 0.64 MGD including an I/I factor of 0.11 MGD, see below. Therefore the percentage of I/I experienced was calculated to be 21%. This correlates to an I/I flow of 0.28MGD. The total peak flow plus I/I factor for the existing 15-inch trunk main on South First Street is **1.61 MGD**.

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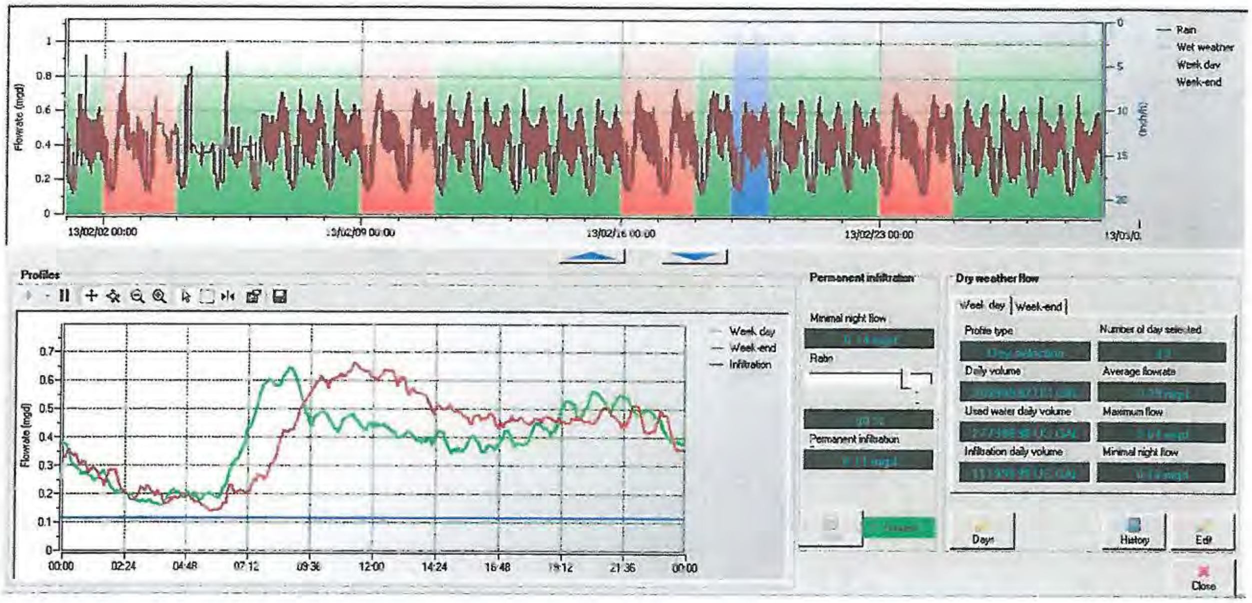

SEWER FLOW SUMMARY:

VACANT PARCELS TRIBUTARY TO THE WASTEWATER SHED:

As a part of this study, the vacant parcels that are tributary to this wastewater shed will be included in the analysis to ensure that the future parcels will have capacity in the existing lines. The City of Dixon provided a vacant parcel map that was used to determine the location of the vacant parcels as well as the zoning to determine the sewer flow rates for each parcel; see Exhibit 'B'. The acreage for each parcel was obtained from the Solano County Assessor's Office. The parcels are shown in Exhibit 'E'.

It should be noted that from the location and current uses of some of these parcels that they are generating sewer flow into the system as it is installed today. This study assumes that all these parcels are additional flows to the system. Additionally this study uses the highest use for the flow generation rates for each vacant parcel for example; if the parcel has a zoning of CS-R1 this study used the Commercial flow generation rate of 1500 GPD per Acre in calculating the flow from the parcels into the 15-inch trunk main in South First Street.

The total flow from the vacant parcels is **0.16 MGD.**

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MULTI-FAMILY SITE AT VALLEY GLEN:

It should be noted that in February of 2013, the 4.42 Acre multi-family site was approved to sewer north to the existing 15-inch sewer main in West Cherry Street. The flow for the multi-family site is as follows:

Multi Family Parcel (4.42 Acres) = 3.53 acres * 5000 gpd/acre = 0.017 MGD
I&I = 4.42 acres * 500 gpd/acre = 0.0022 MGD

Qpwwf= (sum Qa * PF) + I&I = (0.017 * 2.5) + 0.0022 = **0.044 MGD**

69 SINGLE-FAMILY LOTS AT VALLEY GLEN:

There are 69 single family units that are proposed to be serviced through West Cherry Street.

The flow calculations summaries are as follows:

Single Family Lots (69 lots) = Qa=69 units *350 gpd/unit = 0.024 MGD

I&I = 18.0 acres * 500 gpd/acre = 0.009 MGD

Qpwwf= (sum Qa * PF) + I&I = (0.024 * 2.5) + 0.009 = **0.069 MGD**

RESULTS AND CONCLUSIONS:

CURRENT CONDITIONS, FUTURE VACANT PARCELS & VALLEY GLEN

LOCATIONS	TOTAL FLOW (MGD)
Existing 15-inch South First Street (includes flow from West Cherry Street)	1.33
Existing 15-inch South First Street I/I flow	0.28
Vacant Parcels	0.16
Valley Glen Multi-Family Site	0.044
Valley Glen 69 Single Family Lots	0.069
TOTAL	**1.88 MGD**

PIPE SIZE	N VALUE	SLOPE	FULL FLOW	70% CAPACITY
Ex. 15" PVC Lined	0.010	0.002	2.21 MGD	1.85 MGD

Per the City of Dixon, the existing 15-inch sewer main in South First Street was lined with PVC, this is reflected in the table above. The existing slope of 0.002 in 15-inch trunk main in South First Street was taken from the approved Wood Rodgers "Valley Glen – Master Sewer Study : North" dated July 2007, page 6 and existing as-built construction plans dated 1967.

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Per discussions with the City of Dixon the pump station that operates upstream on the 15-inch trunk main in West Cherry Street is planned to be decommissioned and the flows re-routed southerly to tie into the 27-inch trunk main in Parkway Boulevard once the South West Property develops therefore removing the majority of the 0.64 MGD from West Cherry Street and South First Street 15-inch trunk mains. This study will assume that removal of the pump station will divert 50% of the 0.64 MGD peak flow in the West Cherry Street sewer main.

The chart below shows the flows once the pump station is removed from the system and the flows are diverted southerly.

LOCATIONS	TOTAL FLOW (MGD)
Existing 15-inch South First Street (includes flow from West Cherry Street)	1.33
Existing 15-inch South First Street I/I flow	0.28
Removal of Pump Station Flows	-0.32
Vacant Parcels	0.16
Valley Glen Multi-Family Site	0.044
Valley Glen 69 Single Family Lots	0.069
TOTAL	**1.56 MGD**

During discussions with the City of Dixon it was suggested that the flow generation rates specified in the City of Dixon Engineering Design Standards were overstated given the current UBC water conservation requirements. We looked into surrounding jurisdictions and determined that the flow generation rates per the City of Dixon Engineering Design Standards were extremely conservative. For example the City of Vacaville "Sanitary Sewer System Design Standards" Table DS 6-1 state that a Residential Unit will generate 240 gpd/unit vs. 350 gpd/unit, as shown in Exhibit 'G'. That correlates to 31% difference of flow generation rates between the two standards.

Additionally the City of Roseville "Design Standards, Sanitary Sewer Design" Table 1 state Residential Unit will generate 190 gpd/unit vs. 350 gpd/unit, as shown in Exhibit 'H'. This correlates to 46% difference of flow generation rates between the two standards.

Furthermore, it has been determined through actual water meter readings at the Bristol Apartments that the City of Dixon Engineering Design Standards for multi-family sites are conservative in regards to flow generation rates. The meter readings are included in Exhibit 'I'. As provided by the City of Dixon the actual flow rates per acre are 3,050 GPD per acre versus the Engineering Design Standards of 5,000 GPD per acre.

Given the scope of the request to re-route the approved sewer system northerly into West Cherry Street we have taken all steps to ensure conservatism throughout the calculations and this analysis. To summarize the layers of conservatism this report incorporates the following:

- Use of higher than expected flow generation rates

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- With regards to the vacant parcels, use the highest flow generation rates on a mix-use parcel
- With regards to the vacant parcels, assumed all current parcels will contribute flow into the line in the future when it appears that some are already contributing to the system
- Use of an assumed I/I rate on South First Street of 21%. If this study used the Engineering Design Standards for the acreage contributing to the wastewater shed the I/I rate would be lower

With these layers of conservatism and the knowledge that the pump station will be de-commissioned in the future we are requesting the City Engineer approve the 69 single family lots from Valley Glen to sewer northerly into the West Cherry Street 15-inch trunk main, in spite of the projected flows for current conditions being higher than the 70% capacity design requirement.

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EXHIBIT 'A'

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NORTH MASTER SEWER STUDY

VALLEY GLEN

RICHLAND COMMUNITIES

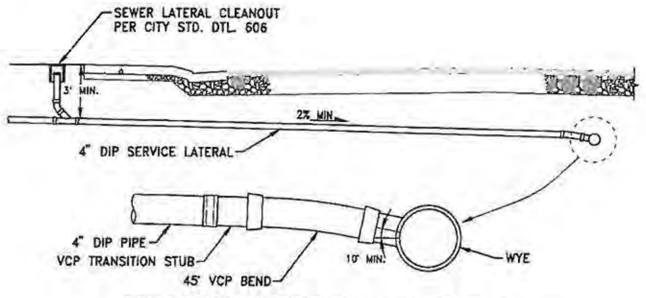
CITY OF DIXON CALIFORNIA

MARCH 2014

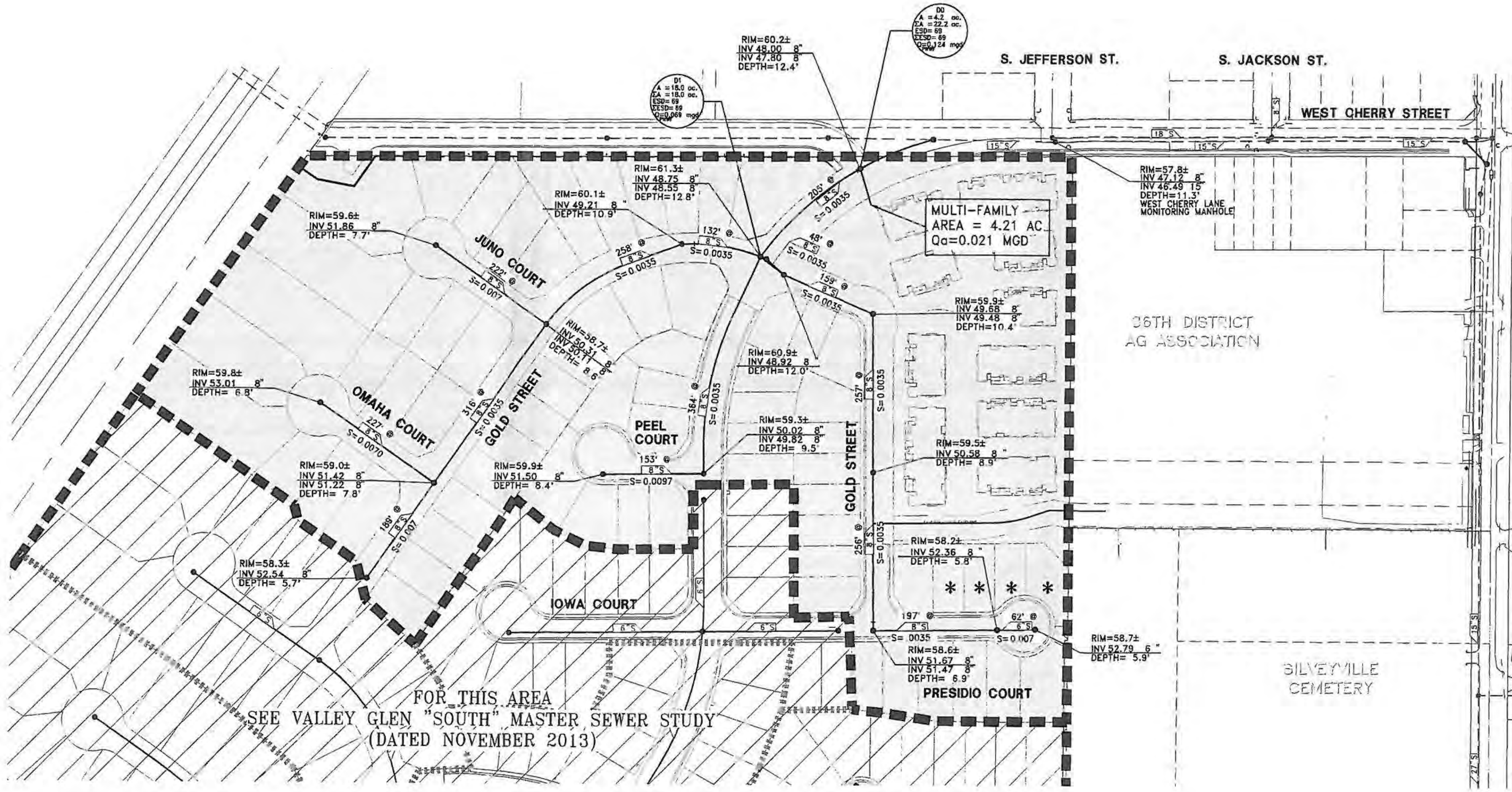
EXHIBIT "A"

SCALE: 1"=100'

* NOTE:
INDICATES LOTS POTENTIALLY REQUIRING SHALLOW SEWER SERVICE (SEE DETAIL BELOW)



SHALLOW SEWER SERVICE DETAIL



VALLEY GLEN

City of Dixon

Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.76

Note:

Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

NODE		D_U	ΣD_U	Q_{AVE}	Multi Fam [Q_{AVE}]	ΣQ_{AVE}	Q_P	I&I					Q_{TOT}
US	DS			$=\Sigma D_U * 350$	$=\Sigma A * 5000$ gpd		$=\Sigma Q_{AVE} * 2.5$	Area	ΣArea	I	I	ΣI	$= Q_P + \Sigma I$
				(mgd)	(mgd)	(mgd)	(mgd)	(acres)	(acres)	(gpd)	(gpd)	(mgd)	(mgd)
D1	D0	69	69	0.0242		0.024	0.060	18.0	18.0	9000	0.009	0.01	0.069
D0	W. CHERRY	0	69	0.0000	0.021	0.045	0.113	4.2	22.2	2100	0.002	0.01	0.124

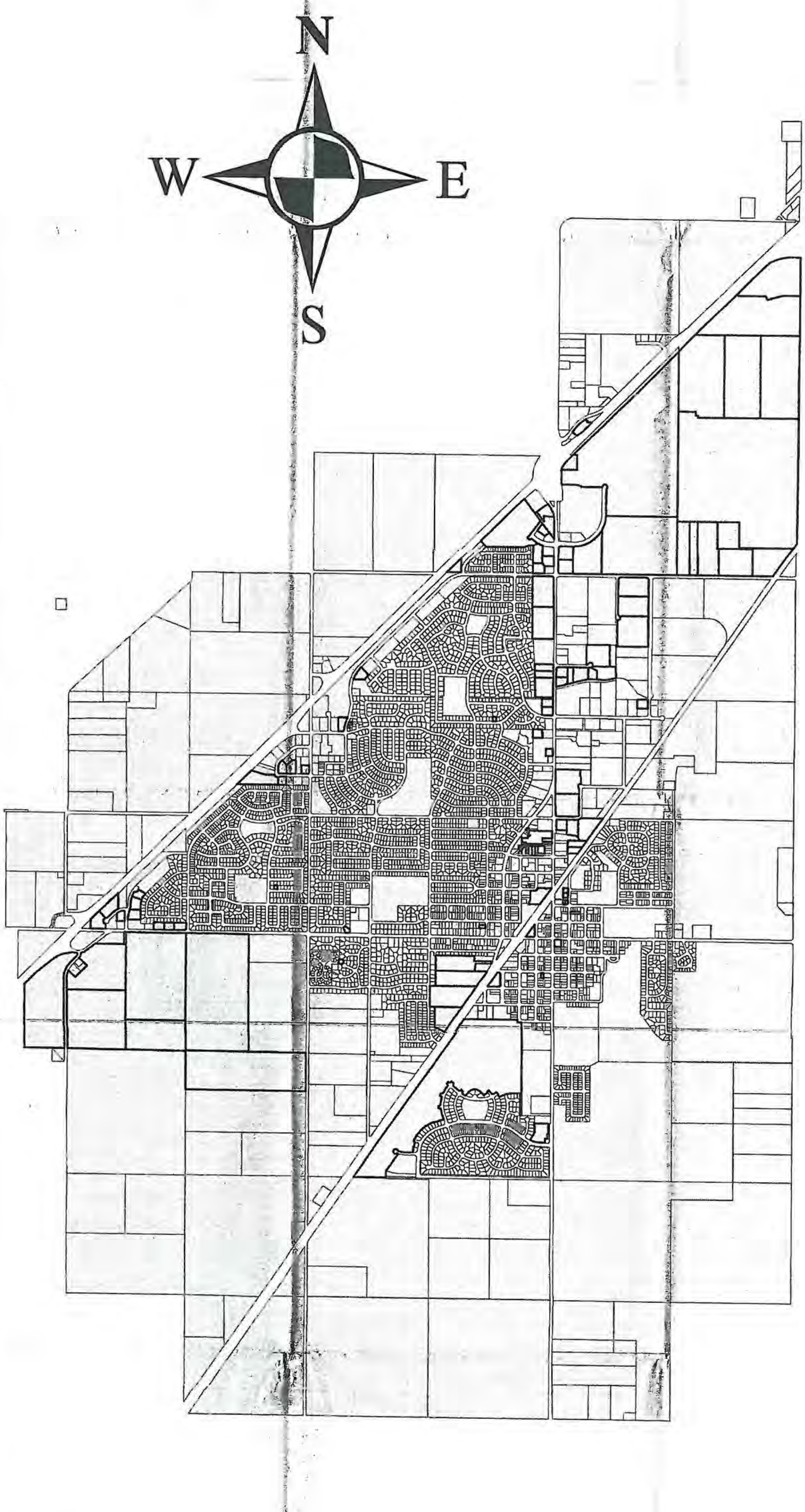
WOOD RODGERS
ENGINEERING - MAPPING - PLANNING - SURVEYING
3301 C St, Bldg. 100-B Tel 916.341.7760
Sacramento, CA 95816 Fax 916.341.7767

EXHIBIT 'B'

LJConsultants
www.LJ-Consultants.com
250 Cherry Lane
Suite 109
Manteca, CA 95337

EXBIT "B"

City of Dixon Vacant Parcel Map



Parcel_Num	Parcel_ac	Parcel_sf	VACANT	Redev_	Zoning	SS	WS	SD
011349012	1.32	57499	YES		CH-CN-PD	Y	Y	Y
011349011	1.70	74052	YES		CH-CN-PD	Y	Y	Y
011349015	0.63	27372	YES		CH-CN	Y	Y	Y
011349003	1.49	64904	YES		CH	6"	6"	24"
011349006	0.77	33541	YES		CH-CN	6"	Y	15"
011302011	1.02	44431	YES		PAO-ML-CS	8"	Y	15"
011302044	0.89	38689	YES		PAO-ML-CS	8"	Y	Y
011349013	0.81	35415	YES		CH-CN-PD	Y	Y	Y
011302022	1.21	52707	YES		PAO-ML-CS	8"	Y	Y
011401104	24.03	1046746	YES		CN-CH	N	N	N
011401101	58.29	2539112	YES		CN	N	N	N
011401204	78.70	3428172	YES		PMR-PD-R	N	N	N
011401103	38.35	1670523	YES		PMR-PD	N	N	N
011401102	19.64	855518	YES		R1-PD	N	N	N
010903009	37.93	1652230	YES		CS-PAO-ML	N	N	N
011344017	0.73	31857	YES		CH-PAO	6"	8"	15"
011344023	1.23	53578	YES		CH-PAO	6"	8"	15"
011344012	1.40	60984	YES		CH-PAO	6"	8"	15"
011344022	0.93	40609	YES		CH-PAO	6"	8"	15"
011344030	4.19	182516	YES		CH-PAO	6"	8"	15"
011345210	0.17	7250	YES		R1-PD	Y	Y	Y
011606114	1.38	60056	YES	YES	CC	Y	Y	N
011402001	49.43	2153170	YES		R1-PD	N	N	N
011403315	94.36	4110321	YES		R1-PD	Y	Y	Y
010829132	2.08	90604	YES		CH-PAO-PD	Y	Y	Y
010829134	0.96	41948	YES		CH-PAO-PD	Y	Y	Y
010829136	1.46	63597	YES		CH-PAO-PD			
011108019	9.85	429066	YES		CN	Y	Y	Y
011108017	2.21	96268	YES		CN	Y	Y	Y
011108018	2.19	95396	YES		CN	Y	Y	Y
011410001	10.08	439085	YES	YES	CS-R1	N	Y	N
011409003	7.60	331056	YES	YES	CS-R1	N	Y	N
011411020	4.84	210830	YES	YES	CS	N	N	N
011406101	1.00	43560	YES	YES	ML	Y	Y	N
011406711	0.13	5662	YES	YES	RM-2 PD	Y	Y	N
011513035	1.01	43995	YES	YES	MH	6"	6"	18"
011501028	6.19	269637	YES	YES	CN	Y	Y	Y
011501028	6.19	269637	YES	YES	CN	8"	10"	24"
011505027	3.87	168577	YES	YES	CS	6"	10"	Y
011505026	2.03	88426	YES	YES	CS	Y	Y	18"
011505025	1.21	52707	YES	YES	CS	6"	10"	24"
011505008	1.24	54014	YES	YES	ML	Y	Y	N
011505010	0.98	42800	YES	YES	ML	Y	Y	N
011306613	2.56	111513	YES	YES	PMU-PD	Y	Y	N
011305408	1.21	52708	YES	YES	CC	Y	Y	N
011508313	0.17	7500	YES	YES	PMU	Y	Y	N
011507002	0.81	35384	YES	YES	PMU-PD	Y	Y	Y
011507007	0.11	5000	YES	YES	PMU-PD	Y	Y	Y
011507005	0.12	5358	YES	YES	CC	Y	Y	Y
011507004	0.10	4500	YES	YES	CC	Y	Y	N
011108020	47.22	2056903	YES		CS-PAO-ML	Y	Y	N
011108023	7.50	326700	YES		ML	Y	N	N
011108022	4.56	198633	YES	YES	ML	Y	N	N
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011108008	6.55	285318	YES		ML	Y	N	N
011108009	10.00	435600	YES		ML	Y	N	N
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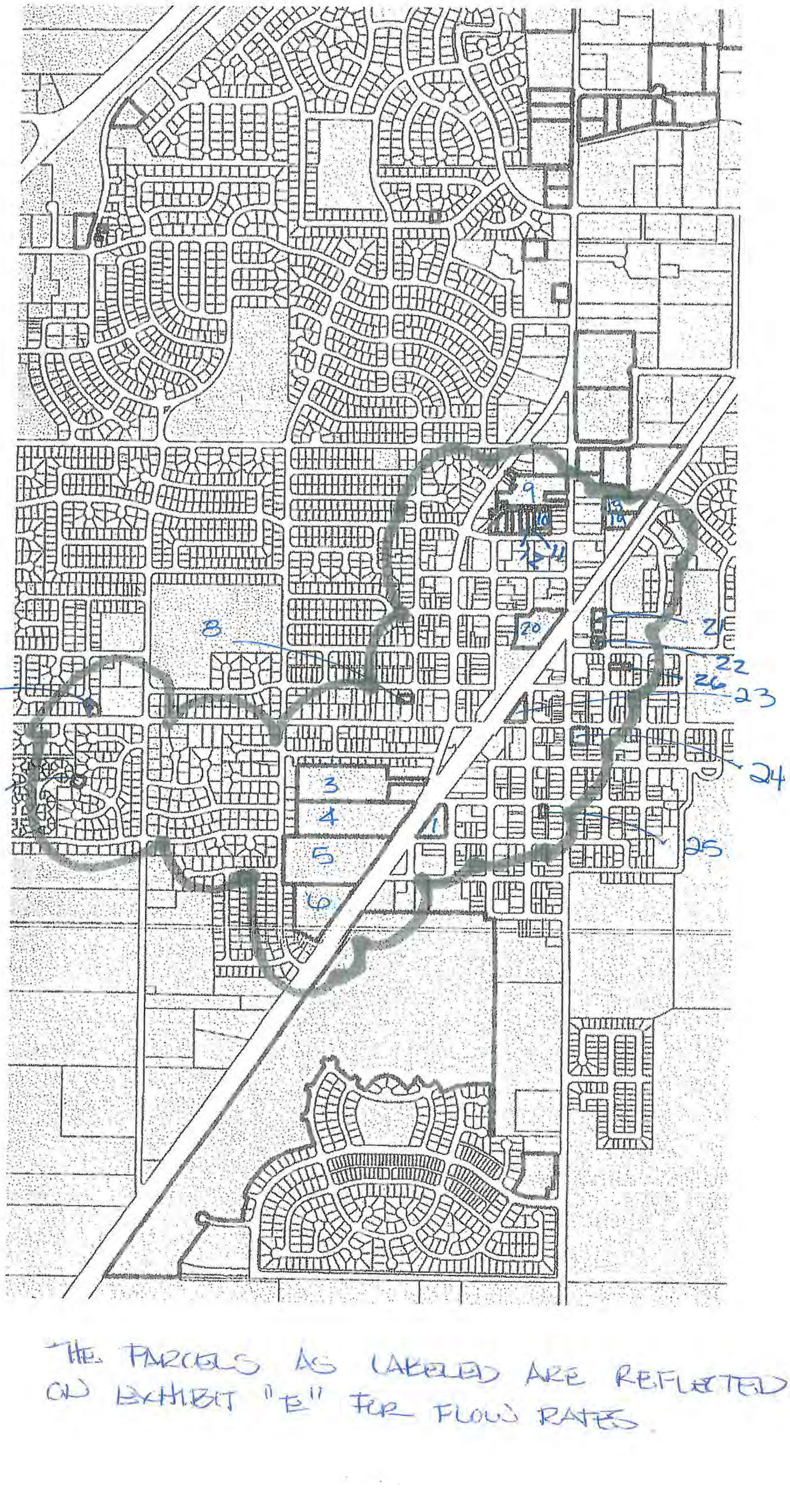
011104002	31.68	1379981	YES		ML	Y	N	N
011104003	32.31	1407424	YES		ML	Y	N	N
011108005	101.33	4413934	YES		ML	Y	N	N
011119001	67.61	2945092	YES		CS-PAO-ML	N	N	N
011101007	21.58	940024	YES		CH-PD	N	N	N
011101008	37.57	1636549	YES		CH-ML-PD	N	N	N
011119004	2.16	94089	YES		CH	Y	N	N
010810014	1.61	70132	YES		CH	6"	6"	15"
010810008	1.59	69260	YES		CH	Y	Y	Y
010810029	0.80	35022	YES		CH-PAO	Y	Y	15"
010810024	16.64	724838	YES		CH-PAO	8"	12"	18"
010811045	5.83	253954	YES	YES	ML-PAO	Y	Y	Y
011339508	0.79	34282	YES	YES	CS	Y	Y	Y
011339510	0.73	31711	YES	YES	CS	Y	Y	Y
010811046	7.43	323650	YES	YES	ML-PAO	Y	Y	Y
010830017	0.78	33772	YES	YES	ML-PAO	Y	Y	Y
010830019	0.78	34143	YES	YES	ML-PAO	Y	Y	Y
010830007	1.80	78408	YES	YES	ML-PAO	Y	Y	Y
011109090	0.97	42253	YES	YES	CS	Y	Y	Y
011109091	1.18	51400	YES	YES	CS	Y	Y	Y
011109092	0.94	40816	YES		CS	Y	Y	Y
011109093	2.10	91476	YES		CS	Y	Y	Y
011109094	1.00	43560	YES		CS	Y	Y	Y
011109095	2.77	120661	YES		CS	Y	Y	Y
011109088	5.07	220849	YES		ML-MH-PD	Y	Y	Y
011109096	9.51	414256	YES		ML-MH-PD	Y	Y	Y
011109078	6.43	280090	YES		ML-MH-PD	Y	Y	Y
011109079	6.43	280090	YES		ML-MH-PD	Y	Y	Y
011409012	7.53	328006	YES	YES	CN-R1	N	N	N
011417105	0.25	10890	YES		PUD	Y	Y	Y
011308216	3.66	159429	YES	YES	ML-CS	Y	Y	Y
011307305	0.12	5205	YES	YES	ML	Y	Y	N
011307328	0.31	13684	YES	YES	ML			
011307312	0.55	24000	YES	YES	ML	Y	Y	N
011307311	0.31	13350	YES	YES	ML	Y	Y	N
011307310	0.38	16688	YES	YES	ML	Y	Y	N
011307309	0.38	16688	YES	YES	ML	Y	Y	N
011307308	0.26	11125	YES	YES	ML	Y	Y	N
011307306	0.45	19602	YES	YES	ML	Y	Y	N
011307328	0.31	13684	YES	YES	ML			
011311201	0.85	37053	YES	YES	CS	Y	Y	Y
011311210	0.51	22086	YES	YES	CS	Y	Y	Y
011327014	0.33	14247	YES		R1-7 PD	Y	Y	Y
010828102	0.77	33650	YES		CH-PAO-P	Y	Y	Y
011336404	0.99	43263	YES		CH	Y	Y	Y
011318432	0.21	9008	YES	YES	R1-S	Y	Y	N
010811056	0.85	36982	YES		CH-PAO	Y	Y	Y
011341112	0.09	3824	YES		PAO-PD	Y	Y	Y
011341113	0.08	3380	YES		PAO-PD	Y	Y	Y
011341111	0.08	3278	YES		PAO-PD	Y	Y	Y
011348201	0.16	7000	YES		R1-PD	Y	Y	Y
011103010	1.31	57063	YES			N	N	N
011421119	2.84	12371	YES		CN-PD	Y	Y	Y

EXHIBIT "B"



THE PARCELS AS LABELED ARE REFLECTED ON EXHIBIT "E" FOR FLOW RATES

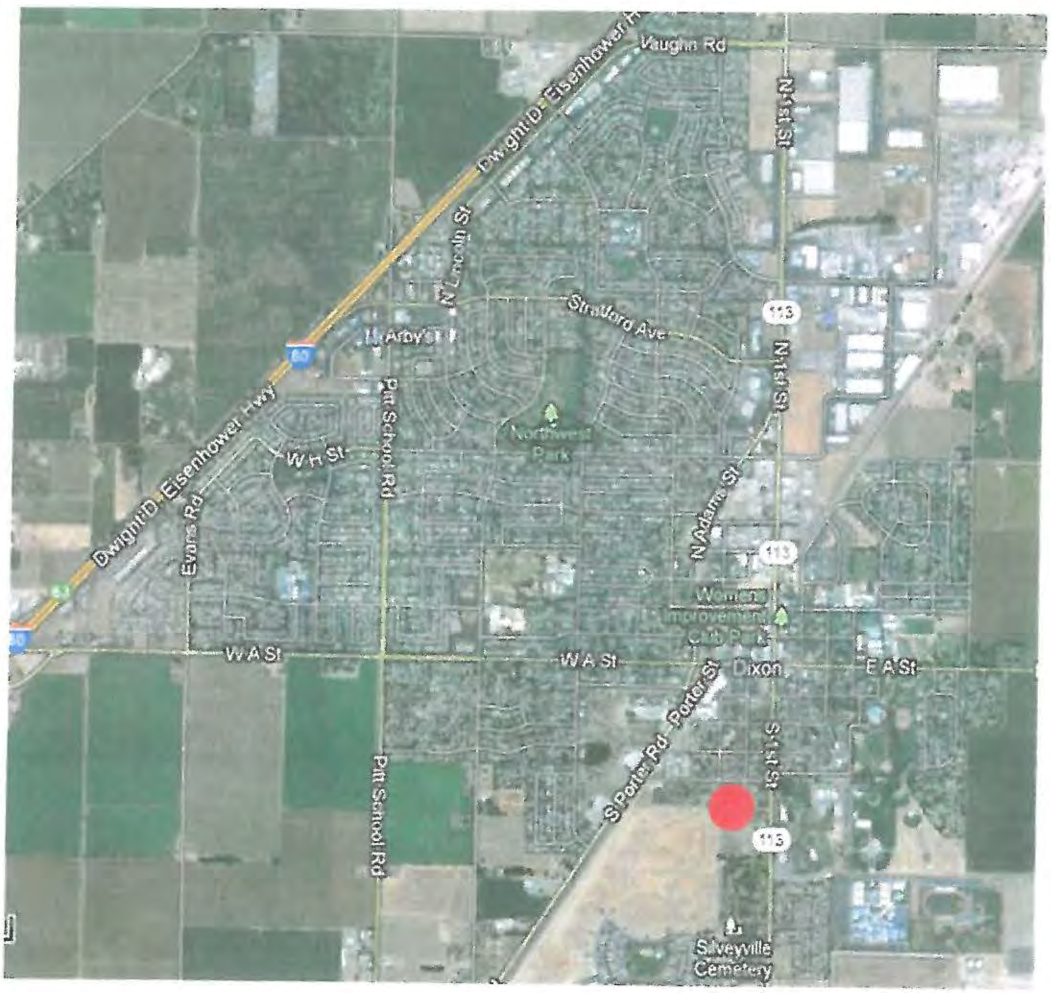
EXHIBIT 'C'

LJConsultants
www.LJ-Consultants.com
250 Cherry Lane
Suite 109
Manteca, CA 95337

Infiltration and Inflow Study

West Cherry Street

City of Dixon, California



Wes Pierce – Maurin Lovera - April 2013

TDG UGAM and Network Management Group

VEOLIA WATER

INTRODUCTION

Flows introduced into a sewer system as daily infiltration or direct inflow from water generated from storm events can be difficult to manage from an operational perspective. These flows generate extra volumes of water that are not always designed to be treated by the waste water treatment plants (WWTP), and can cause overflows from the system, impacting the surrounding environment.

PURPOSE

This report is based on a two-month monitoring period (from February 1st to March 31st 2013) focused on one catchment area upstream of West Cherry Street in Dixon CA, to evaluate the potential influence of volumes of water originating from inflow and infiltration (I&I) on the capacity of the existing 15 inch pipe.

Brief description of the methodology used by Veolia Water North America for I&I study:

- Select and deploy the monitoring points based on known critical areas or system hot spots;
- Collect and validate data from the monitoring points in the sewer system (flow depth data / flow data / pumping station data), and rain gauge data;
- Veolia applied the GesCIRA™ software:
    - Isolate dry and wet weather events (night and day);
    - Extract and adjust a dry flow profile using the collected data;
    - Based on the dry flow profile plus the rain data, estimate the amount of Inflow and Infiltration for each monitoring point;
- Identify potential I&I issues found from the above investigations;
- Propose adapted solutions if any

METERING LOCATION

Figure 1: Flow Meter Location

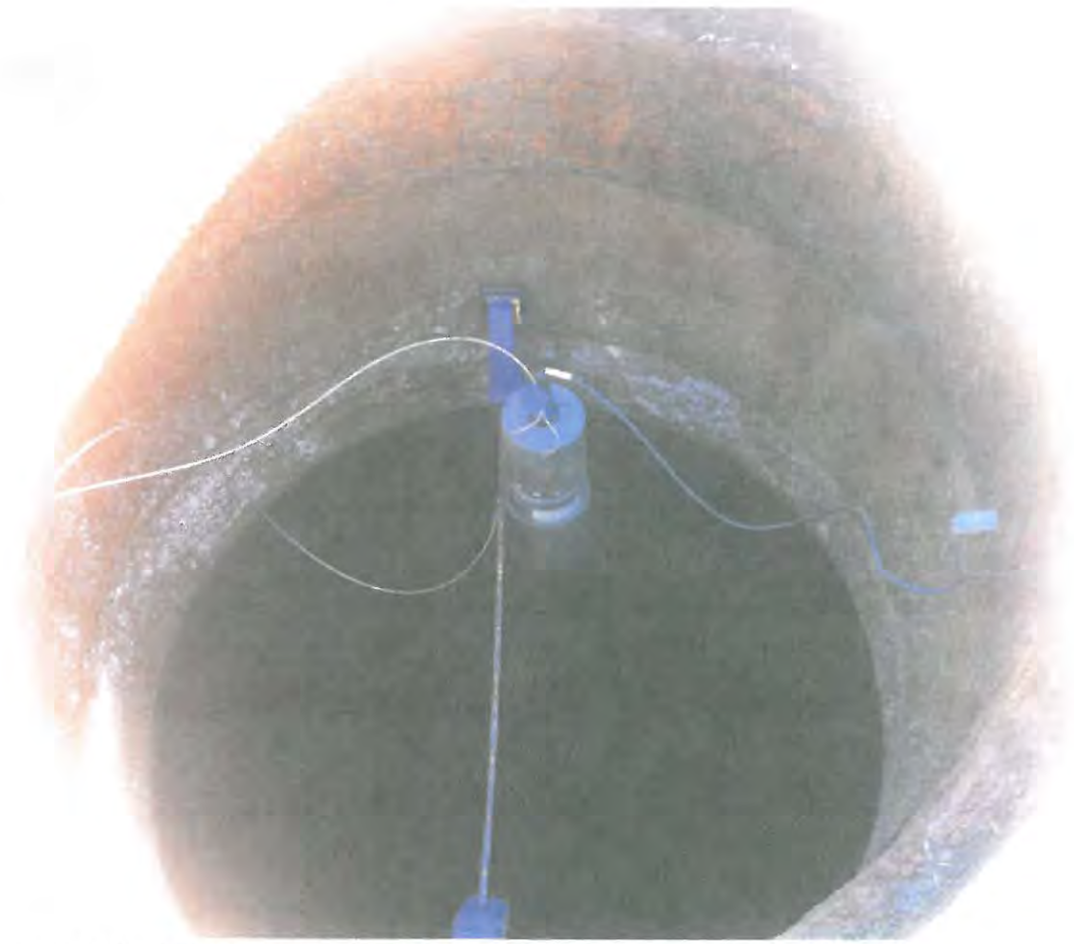


Figure 2: Flow Metering Sensor Installed in Manhole

ANALYSIS USING GesCIRA™

Monitored height data and calculated flow value

The value of the flow is calculated in the GesCIRA program by using the monitored height data collected by the Ijinus Sensors and by applying the Manning-Strickler equation. The slope of the pipe and the Manning number are also needed to complete the calculation of the flow (both inputs were based on data provided to VW). Since no surcharging occurred during the monitoring period, a straight-forward depth to flow conversion was applicable and appropriate methodology.

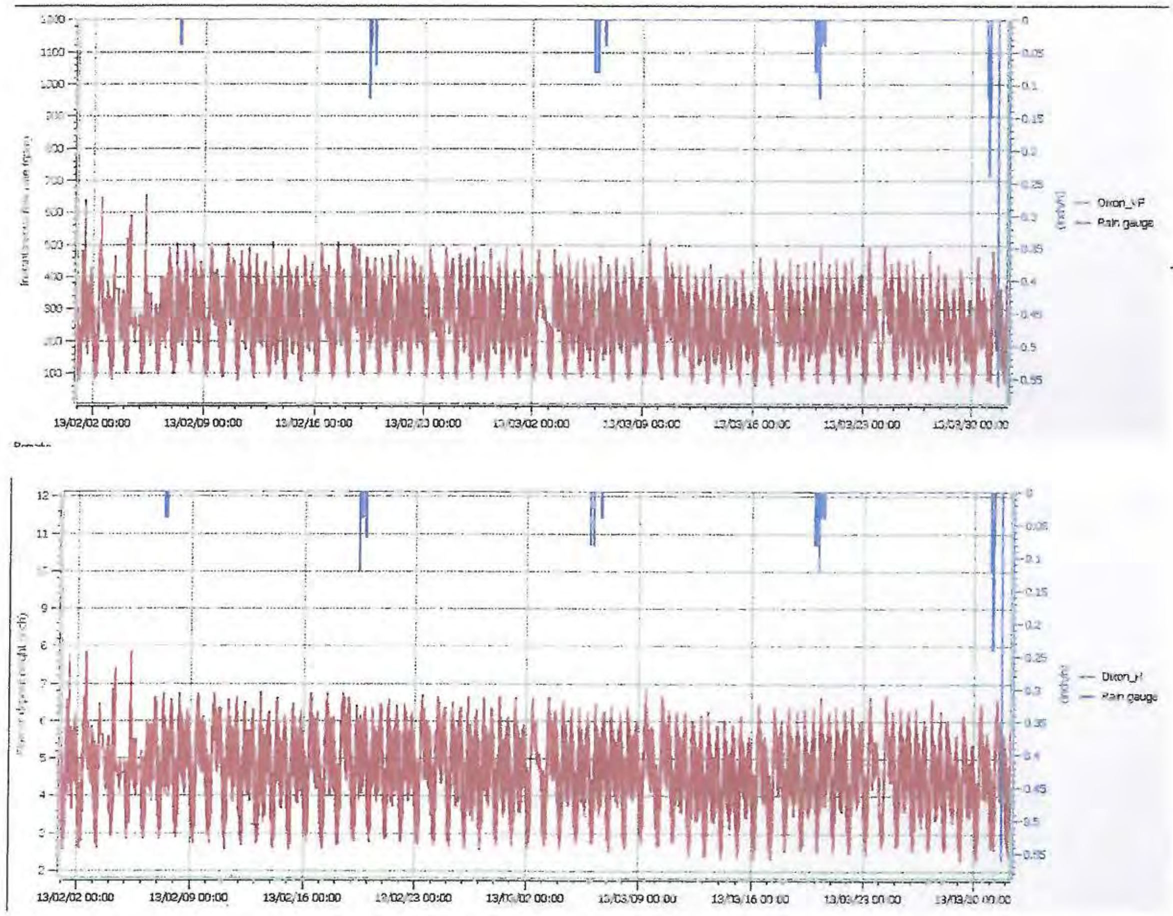


Figure 3: Raw height data and converted flow values through GesCIRA, for the period of the study (February 2013 and March 2013).

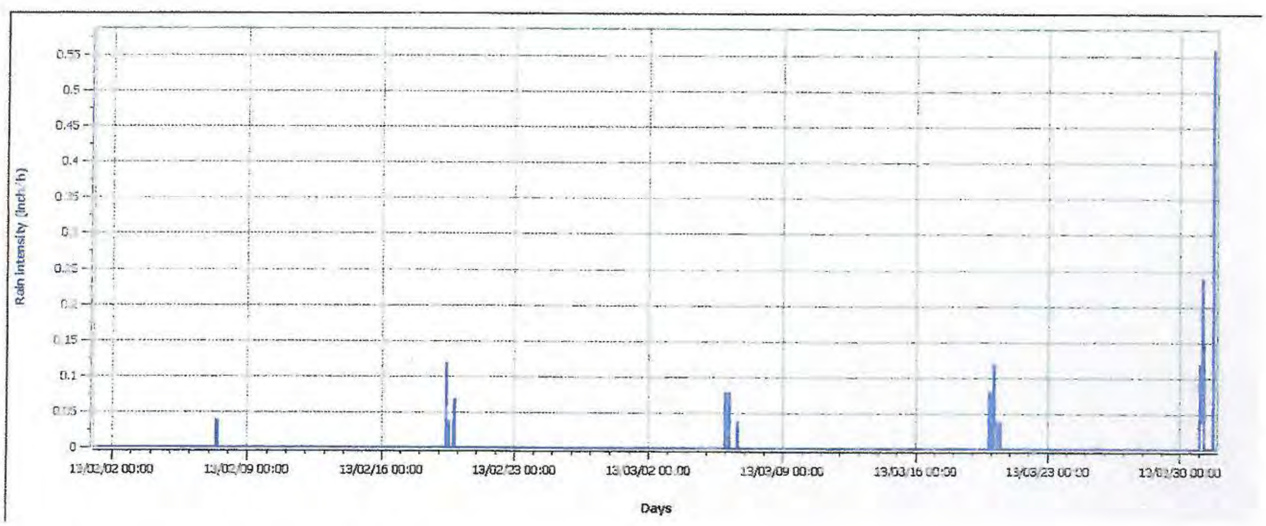
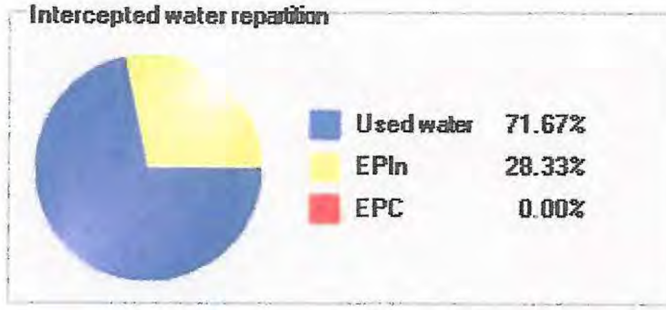


Figure 4: Rainfall data during study period

The rainfall accumulated from Februarys 2013 to March 2013 is 1.89 inches. The largest rain event was 1.27 inches over a 7 hour period.

Profiles

GesCIRA™ automatically sorts dry and wet weather days, and uses the collected flow data during dry days to calculate the average diurnal patterns. Next, GesCIRA refines and splits these into week days and week end patterns for each month. The baseline infiltration rate is then estimated by the user based on the early morning diurnal flow patterns (estimated at 80% of lowest flow for this investigation).



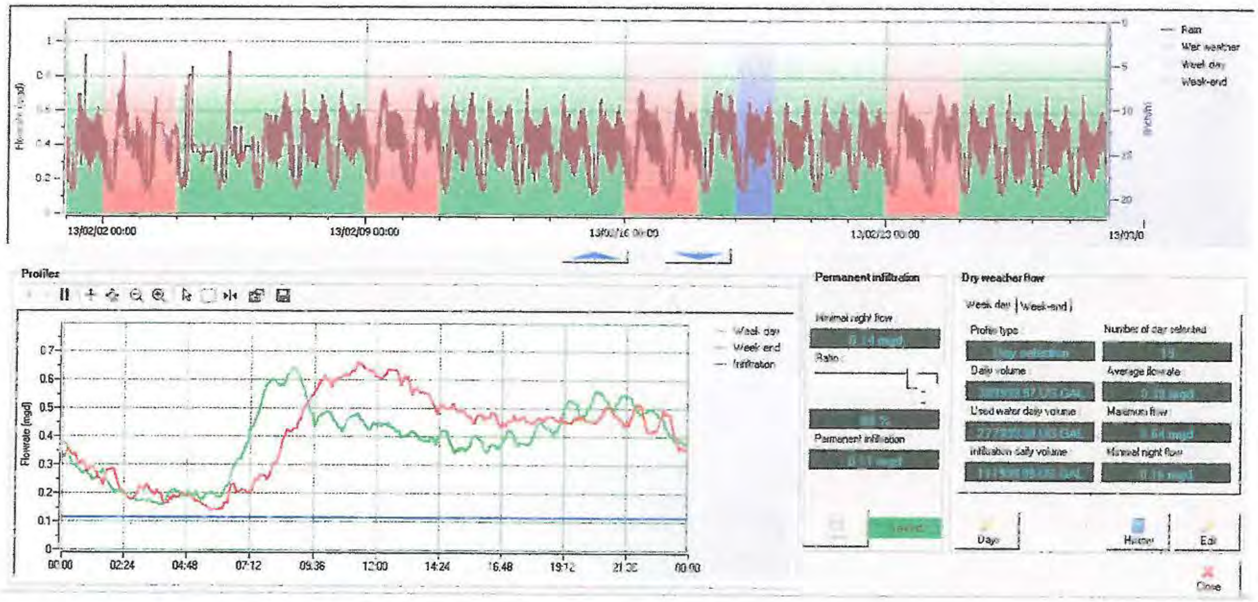


Figure 5: Diurnal Patterns

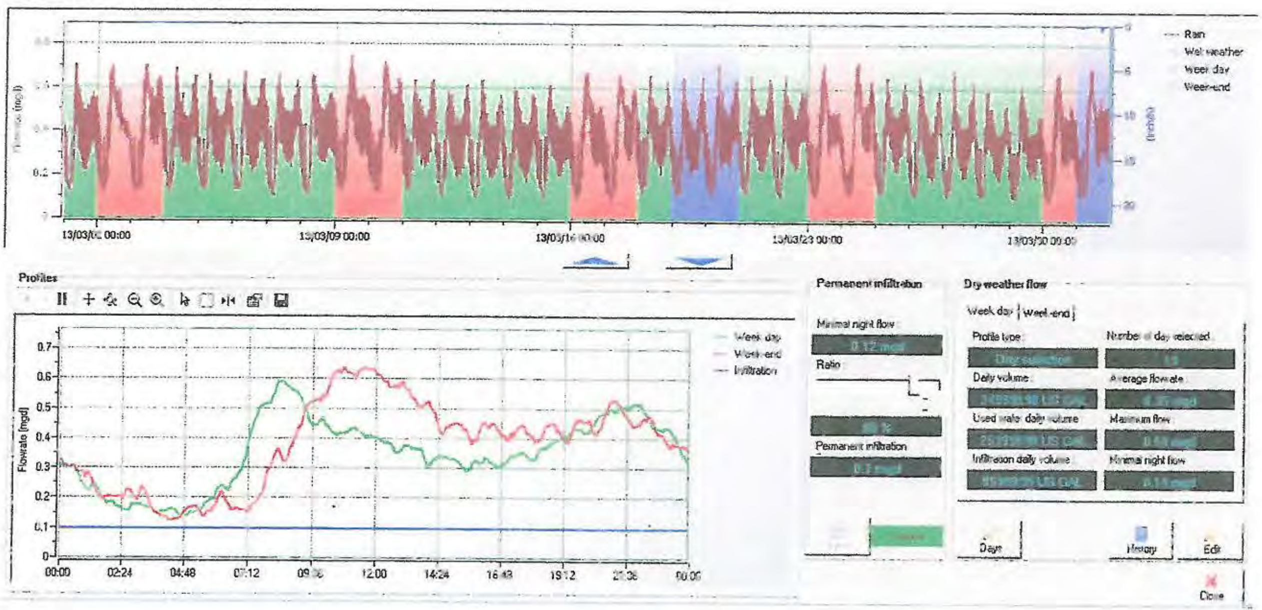


Figure 6: Average DWF

The graphs above show the average daily calculated dry flow pattern for any given week day and weekend for the months of February and March 2013 at the monitoring location. The horizontal blue line represents the limit of 80% baseline infiltration noted previously.

RESULTS

GesCIRA™ uses algorithms which basically reproduce the I&I assessment methodology to process the data. GesCIRA is then able to produce visual and numerical reports of the data imported into the database. Based on the initial parameterization of the software, the three categories of volumes can be split into daily Waste Water, Inflow and Infiltration volumes.

- **Daily Waste Water volume** is the normal sewage flow contribution into the system each day by serviced population and industries;
- **Daily Inflow volume** can be defined as the volume of rainwater that penetrates into the system each day by direct run-off and the influence of rain events;
- **Daily Infiltration volume** can be defined as the volume of water that penetrates into the system each day through the groundwater influence.

During the flow monitoring period there was no evidence of Inflow due to rain upstream of West Cherry Street Sub-catchment

Below is a table giving the total volume of water passing under the sensor and a pie chart of the corresponding percentages during the period of the study. They are distributed into the three categories (sewer volume – "used water", infiltration volume – "EPIn", inflow volume – "EPC"):

	Volume (MG)
Total Volume	22.2
Sewer Volume	17.1
Infiltration Volume	5.1
Inflow Volume	0

The results in Dixon computed by GesCIRA indicated an average volume of infiltration with **no evidence of Inflow from rain events** for the flow monitoring period.

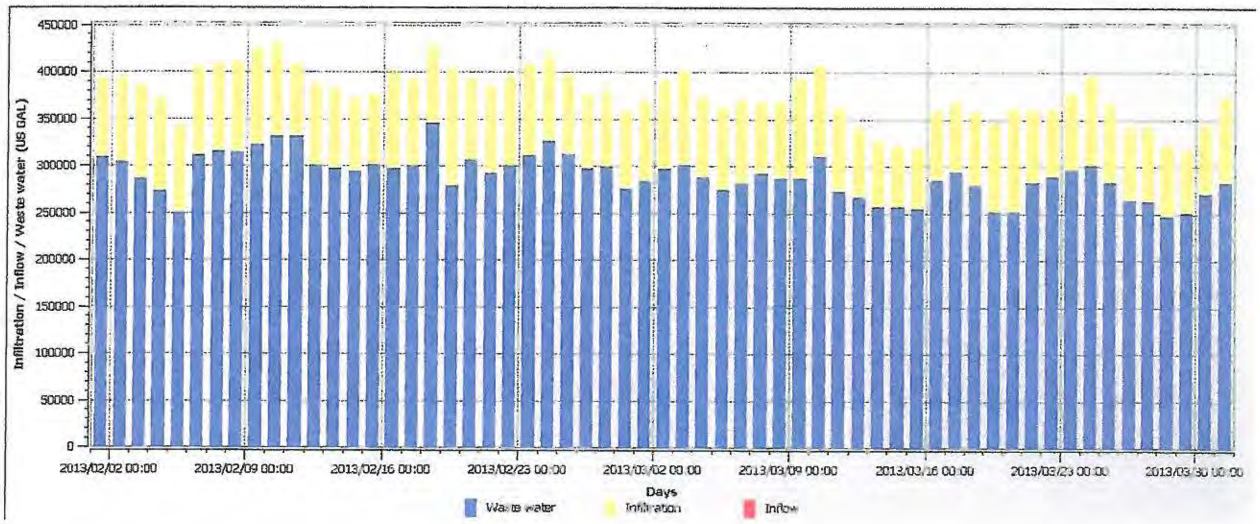


Figure 7: Distribution of the total volume of water at West Cherry

Based on the height data recorded in the two month period, there appears to be around a 50% remaining capacity in the pipe during the peak flow conditions. The pipe is a 15 inch circular VCP pipe with PVC liner. The maximum water height is less than 8 inches and is less than 7 inches for 90% of the study period.

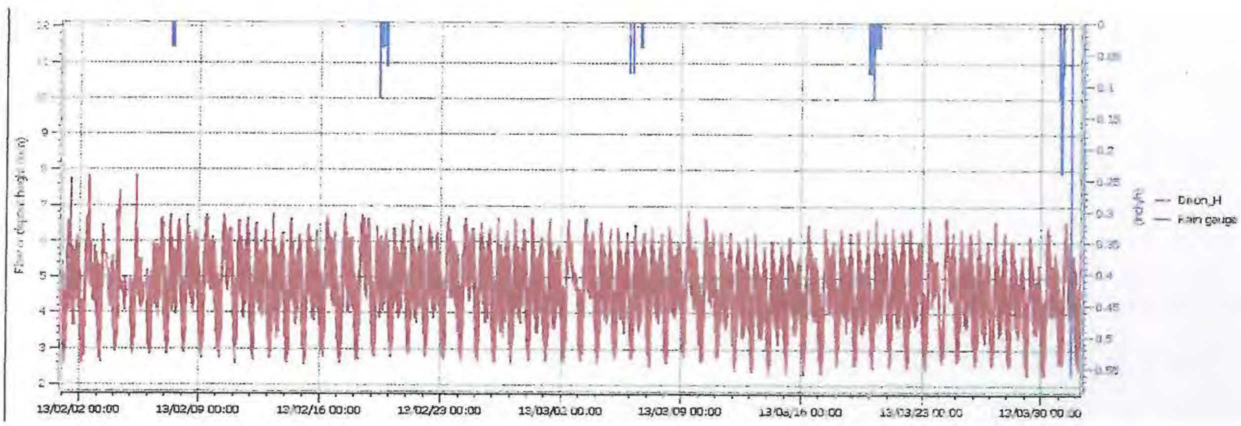


Figure 8: Height data vs rainfall

Some of this observed height is influenced by a pumping station that is upstream of this monitoring point. The graph below shows the reflected flow pattern from pump starts/stops regardless of the other conditions that might influence the flows.

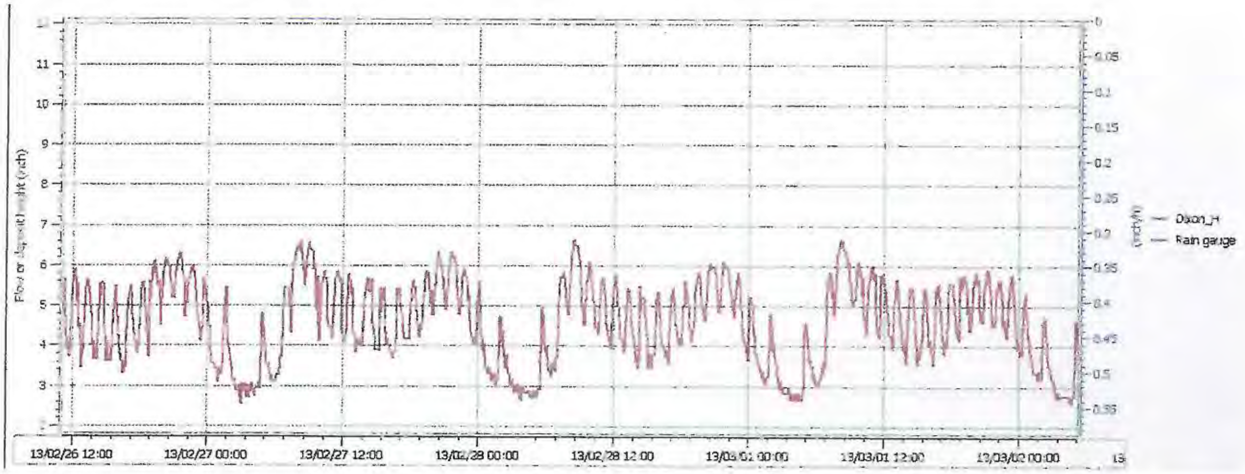


Figure 9: Pump station influence on flow patterns.

CONCLUSION

Based on the results of the study performed between February 1st and March 31st 2013 in Dixon on Cherry Street:

- Rain events did not have any observable influence on the volumes of water during the study;
- There is a remaining capacity of around 50% of the 15 inch pipe;
- The flow at the location is directly influenced by an upstream pump station.

EXHIBIT 'D'

LJConsultants
www.LJ-Consultants.com
250 Cherry Lane
Suite 109
Manteca, CA 95337

EXHIBIT 'D'

SOUTH FIRST STREET SEWER MONITORING LOCATION

EXHIBIT 'D'

LJConsultants
www.LJ-Consultants.com
250 Cherry Lane
Suite 109
Manteca, CA 95337

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch
Dixon_CA	IJA0101-00000073	7/11/2013 11:24	6.50
Dixon_CA	IJA0101-00000073	7/11/2013 15:00	6.50
Dixon_CA	IJA0101-00000073	7/11/2013 16:00	6.14
Dixon_CA	IJA0101-00000073	7/11/2013 17:00	5.59
Dixon_CA	IJA0101-00000073	7/11/2013 18:00	5.59
Dixon_CA	IJA0101-00000073	7/11/2013 19:00	5.94
Dixon_CA	IJA0101-00000073	7/11/2013 20:00	6.14
Dixon_CA	IJA0101-00000073	7/11/2013 21:00	6.61
Dixon_CA	IJA0101-00000073	7/11/2013 22:00	7.52
Dixon_CA	IJA0101-00000073	7/11/2013 23:00	7.52
Dixon_CA	IJA0101-00000073	7/12/2013 0:00	7.05
Dixon_CA	IJA0101-00000073	7/12/2013 1:00	6.54
Dixon_CA	IJA0101-00000073	7/12/2013 2:00	5.31
Dixon_CA	IJA0101-00000073	7/12/2013 3:00	4.80
Dixon_CA	IJA0101-00000073	7/12/2013 4:00	4.80
Dixon_CA	IJA0101-00000073	7/12/2013 5:00	4.80
Dixon_CA	IJA0101-00000073	7/12/2013 6:00	5.43
Dixon_CA	IJA0101-00000073	7/12/2013 7:00	5.87
Dixon_CA	IJA0101-00000073	7/12/2013 8:00	6.85
Dixon_CA	IJA0101-00000073	7/12/2013 9:00	7.20
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Dixon_CA	IJA0101-00000073	7/12/2013 12:00	7.36
Dixon_CA	IJA0101-00000073	7/12/2013 13:00	6.89
Dixon_CA	IJA0101-00000073	7/12/2013 14:00	6.69
Dixon_CA	IJA0101-00000073	7/12/2013 15:00	6.69
Dixon_CA	IJA0101-00000073	7/12/2013 16:00	6.85
Dixon_CA	IJA0101-00000073	7/12/2013 17:00	6.93
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Dixon_CA	IJA0101-00000073	7/12/2013 19:00	7.05
Dixon_CA	IJA0101-00000073	7/12/2013 20:00	7.05

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch
Dixon_CA	IJA0101-00000073	7/12/2013 21:00	6.57
Dixon_CA	IJA0101-00000073	7/12/2013 22:00	6.54
Dixon_CA	IJA0101-00000073	7/12/2013 23:00	6.54
Dixon_CA	IJA0101-00000073	7/13/2013 0:00	6.26
Dixon_CA	IJA0101-00000073	7/13/2013 1:00	5.83
Dixon_CA	IJA0101-00000073	7/13/2013 2:00	5.75
Dixon_CA	IJA0101-00000073	7/13/2013 3:00	5.83
Dixon_CA	IJA0101-00000073	7/13/2013 4:00	4.69
Dixon_CA	IJA0101-00000073	7/13/2013 5:00	4.80
Dixon_CA	IJA0101-00000073	7/13/2013 6:00	4.80
Dixon_CA	IJA0101-00000073	7/13/2013 7:00	5.83
Dixon_CA	IJA0101-00000073	7/13/2013 8:00	6.22
Dixon_CA	IJA0101-00000073	7/13/2013 9:00	6.89
Dixon_CA	IJA0101-00000073	7/13/2013 10:00	7.36
Dixon_CA	IJA0101-00000073	7/13/2013 11:00	7.64
Dixon_CA	IJA0101-00000073	7/13/2013 13:00	7.64
Dixon_CA	IJA0101-00000073	7/13/2013 14:00	7.44
Dixon_CA	IJA0101-00000073	7/13/2013 15:00	6.26
Dixon_CA	IJA0101-00000073	7/13/2013 16:00	6.10
Dixon_CA	IJA0101-00000073	7/13/2013 17:00	6.10
Dixon_CA	IJA0101-00000073	7/13/2013 18:00	6.85
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Dixon_CA	IJA0101-00000073	7/13/2013 22:00	6.54
Dixon_CA	IJA0101-00000073	7/13/2013 23:00	6.54
Dixon_CA	IJA0101-00000073	7/14/2013 0:00	6.10
Dixon_CA	IJA0101-00000073	7/14/2013 1:00	6.10
Dixon_CA	IJA0101-00000073	7/14/2013 2:00	6.02
Dixon_CA	IJA0101-00000073	7/14/2013 3:00	6.06
Dixon_CA	IJA0101-00000073	7/14/2013 4:00	4.61

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch
Dixon_CA	IJA0101-00000073	7/14/2013 5:00	4.57
Dixon_CA	IJA0101-00000073	7/14/2013 6:00	4.57
Dixon_CA	IJA0101-00000073	7/14/2013 7:00	5.83
Dixon_CA	IJA0101-00000073	7/14/2013 8:00	6.02
Dixon_CA	IJA0101-00000073	7/14/2013 9:00	6.89
Dixon_CA	IJA0101-00000073	7/14/2013 10:00	7.48
Dixon_CA	IJA0101-00000073	7/14/2013 11:00	7.87
Dixon_CA	IJA0101-00000073	7/14/2013 12:00	7.87
Dixon_CA	IJA0101-00000073	7/14/2013 13:00	6.97
Dixon_CA	IJA0101-00000073	7/14/2013 14:00	6.97
Dixon_CA	IJA0101-00000073	7/14/2013 15:00	6.81
Dixon_CA	IJA0101-00000073	7/14/2013 16:00	6.69
Dixon_CA	IJA0101-00000073	7/14/2013 17:00	5.98
Dixon_CA	IJA0101-00000073	7/14/2013 18:00	5.83
Dixon_CA	IJA0101-00000073	7/14/2013 19:00	5.98
Dixon_CA	IJA0101-00000073	7/14/2013 20:00	6.10
Dixon_CA	IJA0101-00000073	7/14/2013 21:00	6.61
Dixon_CA	IJA0101-00000073	7/14/2013 22:00	6.85
Dixon_CA	IJA0101-00000073	7/14/2013 23:00	6.85
Dixon_CA	IJA0101-00000073	7/15/2013 0:00	6.61
Dixon_CA	IJA0101-00000073	7/15/2013 1:00	6.50
Dixon_CA	IJA0101-00000073	7/15/2013 2:00	4.92
Dixon_CA	IJA0101-00000073	7/15/2013 3:00	4.80
Dixon_CA	IJA0101-00000073	7/15/2013 4:00	4.61
Dixon_CA	IJA0101-00000073	7/15/2013 5:00	4.61
Dixon_CA	IJA0101-00000073	7/15/2013 6:00	5.28
Dixon_CA	IJA0101-00000073	7/15/2013 7:00	7.05
Dixon_CA	IJA0101-00000073	7/15/2013 8:00	7.64
Dixon_CA	IJA0101-00000073	7/15/2013 9:00	7.64
Dixon_CA	IJA0101-00000073	7/15/2013 10:00	7.60
Dixon_CA	IJA0101-00000073	7/15/2013 11:00	7.17

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch
Dixon_CA	IJA0101-00000073	7/15/2013 12:00	6.61
Dixon_CA	IJA0101-00000073	7/15/2013 13:00	6.34
Dixon_CA	IJA0101-00000073	7/15/2013 14:00	6.10
Dixon_CA	IJA0101-00000073	7/15/2013 15:00	6.10
Dixon_CA	IJA0101-00000073	7/15/2013 16:00	6.18
Dixon_CA	IJA0101-00000073	7/15/2013 17:00	6.73
Dixon_CA	IJA0101-00000073	7/15/2013 18:00	6.89
Dixon_CA	IJA0101-00000073	7/15/2013 19:00	6.89
Dixon_CA	IJA0101-00000073	7/15/2013 20:00	7.20
Dixon_CA	IJA0101-00000073	7/15/2013 21:00	7.44
Dixon_CA	IJA0101-00000073	7/15/2013 22:00	7.44
Dixon_CA	IJA0101-00000073	7/15/2013 23:00	7.17
Dixon_CA	IJA0101-00000073	7/16/2013 0:00	6.06
Dixon_CA	IJA0101-00000073	7/16/2013 1:00	6.06
Dixon_CA	IJA0101-00000073	7/16/2013 2:00	5.63
Dixon_CA	IJA0101-00000073	7/16/2013 3:00	4.96
Dixon_CA	IJA0101-00000073	7/16/2013 4:00	4.96
Dixon_CA	IJA0101-00000073	7/16/2013 5:00	4.84
Dixon_CA	IJA0101-00000073	7/16/2013 6:00	5.43
Dixon_CA	IJA0101-00000073	7/16/2013 7:00	6.81
Dixon_CA	IJA0101-00000073	7/16/2013 8:00	7.99
Dixon_CA	IJA0101-00000073	7/16/2013 9:00	8.03
Dixon_CA	IJA0101-00000073	7/16/2013 10:00	8.03
Dixon_CA	IJA0101-00000073	7/16/2013 11:00	7.64
Dixon_CA	IJA0101-00000073	7/16/2013 12:00	7.01
Dixon_CA	IJA0101-00000073	7/16/2013 13:00	6.42
Dixon_CA	IJA0101-00000073	7/16/2013 14:00	6.02
Dixon_CA	IJA0101-00000073	7/16/2013 15:00	6.02
Dixon_CA	IJA0101-00000073	7/16/2013 16:00	6.54
Dixon_CA	IJA0101-00000073	7/16/2013 17:00	7.28
Dixon_CA	IJA0101-00000073	7/16/2013 18:00	7.32

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch	
Dixon_CA	IJA0101-00000073	7/16/2013 19:00	7.32	
Dixon_CA	IJA0101-00000073	7/16/2013 20:00	7.32	
Dixon_CA	IJA0101-00000073	7/16/2013 21:00	7.36	
Dixon_CA	IJA0101-00000073	7/16/2013 22:00	6.93	
Dixon_CA	IJA0101-00000073	7/16/2013 23:00	6.93	
Dixon_CA	IJA0101-00000073	7/17/2013 0:00	6.22	
Dixon_CA	IJA0101-00000073	7/17/2013 1:00	6.22	
Dixon_CA	IJA0101-00000073	7/17/2013 2:00	5.47	
Dixon_CA	IJA0101-00000073	7/17/2013 3:00	4.72	
Dixon_CA	IJA0101-00000073	7/17/2013 4:00	4.72	
Dixon_CA	IJA0101-00000073	7/17/2013 5:00	4.84	
Dixon_CA	IJA0101-00000073	7/17/2013 6:00	5.04	
Dixon_CA	IJA0101-00000073	7/17/2013 7:00	6.46	
Dixon_CA	IJA0101-00000073	7/17/2013 8:00	7.83	
Dixon_CA	IJA0101-00000073	7/17/2013 9:00	7.95	
Dixon_CA	IJA0101-00000073	7/17/2013 10:00	7.95	
Dixon_CA	IJA0101-00000073	7/17/2013 11:00	8.11	PEAK FLOW
Dixon_CA	IJA0101-00000073	7/17/2013 12:00	7.72	
Dixon_CA	IJA0101-00000073	7/17/2013 13:00	7.24	
Dixon_CA	IJA0101-00000073	7/17/2013 14:00	6.81	
Dixon_CA	IJA0101-00000073	7/17/2013 15:00	5.71	
Dixon_CA	IJA0101-00000073	7/17/2013 16:00	5.71	
Dixon_CA	IJA0101-00000073	7/17/2013 17:00	6.18	
Dixon_CA	IJA0101-00000073	7/17/2013 18:00	6.22	
Dixon_CA	IJA0101-00000073	7/17/2013 20:00	7.09	
Dixon_CA	IJA0101-00000073	7/17/2013 21:00	7.56	
Dixon_CA	IJA0101-00000073	7/17/2013 22:00	7.91	
Dixon_CA	IJA0101-00000073	7/17/2013 23:00	7.91	
Dixon_CA	IJA0101-00000073	7/18/2013 0:00	6.65	
Dixon_CA	IJA0101-00000073	7/18/2013 1:00	6.14	
Dixon_CA	IJA0101-00000073	7/18/2013 2:00	4.84	

EXHIBIT 'D'
VALLEY GLEN
SOUTH FIRST STREET FLOW MONITORING
JULY 2013

site	sensor	date	water height : inch
Dixon_CA	IJA0101-00000073	7/18/2013 3:00	4.80
Dixon_CA	IJA0101-00000073	7/18/2013 4:00	4.84
Dixon_CA	IJA0101-00000073	7/18/2013 5:00	4.96
Dixon_CA	IJA0101-00000073	7/18/2013 6:00	5.51
Dixon_CA	IJA0101-00000073	7/18/2013 7:00	6.97
Dixon_CA	IJA0101-00000073	7/18/2013 8:00	7.95
Dixon_CA	IJA0101-00000073	7/18/2013 9:00	7.95
Dixon_CA	IJA0101-00000073	7/18/2013 10:00	7.72
			6.45

EXHIBIT 'E'

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EXHIBIT 'E'
CITY OF DIXON
VACANT PARCEL
SEWER FLOW CALCULATIONS FOR SOUTH FIRST STREET SEWER MAIN
OCTOBER 2013

Parcel Number	APN	Acreage	Zoning	Land Use	Average Daily Flow (gpd)	I + I Factor (gpd)	NOTES
1	0113270140	0.32	R1-7 PD	SF	350	160	In Exisiting Subd.
2	0114171050	0.25	PUD	SF	350	125	In Exisiting Subd.
3	0114090120	7.53	CN-R1	CC	9036	3765	
4	0114090030	7.6	CS-R1	CC	9120	3800	
5	0114100010	10.08	CS-R1	CC	12096	5040	
6	0114110200	4.84	CS	CC	5808	2420	
7	0113184320	0.2	R1-S	CC	240	100	
8	0114061010	1	ML	I	1600	500	storage lot
9	0113082160	1.575	ML-CS	I	2520	787.5	
		1.575		CC	1890	787.5	
10	0113073060	0.45	ML	I	720	225	
11	0113073080	0.26	ML	I	416	130	
12	0113073090	0.38	ML	I	608	190	
13	0113073100	0.38	ML	I	608	190	
14	0113073110	0.31	ML	I			Water Tank Site/Removed per City Comments dated 8/5/13
15	0113073120	0.55	ML	I	880	275	
16	0113073130	0.19	ML	I	304	95	
17	0115050310	1.06	ML	I	1696	530	
18	0115050100	0.98	ML	I	1568	490	
19	0113056130	2.56	PMU-PD	CC	3072	1280	Strip Mall
20	0115070170	0.25	CC	CC	300	125	
21	0115070180	0.25	CC	CC	300	125	
22	0115070050	0.12	CC	CC	144	60	
23	0115070040	0.1	CC	CC	120	50	
24	0115083130	0.17	PMU	CC	204	85	
25	0116061140	0.68	CC	CC	816	340	
26	0114067110	0.13	RM-2 PD	SF	350	65	In Exisiting Subd.
	TOTAL	43.79			0.06	0.02	
		Peaking Factor of 2.5*flow			0.14		

TOTAL FLOW INTO FIRST STREET SOUTH OF WEST CHERRY STREET **0.16 MGD**

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EXHIBIT 'F'

Free Online Manning Pipe Flow Calculator

List of Calculators Hydraulics Language

Manning Formula Uniform Pipe Flow at Given Slope and Depth

Can you help me translate this calculator to your language or host this calculator at your web site?

Set units: m mm ft inches

Pipe diameter, d_0	15 inches
Manning roughness, n ?	0.013
Pressure slope (possibly ? equal to pipe slope), S_0	.002 rise/run
Percent of (or ratio to) full depth (100% or 1 if flowing full)	70 %

Results:

Flow, q	1.5630	MGD
Velocity, v	2.6359	ft/sec
Velocity head, h_v	0.1080	ft
Flow area	0.9176	ft^2
Wetted perimeter	2.4779	ft
Hydraulic radius	0.3703	ft
Top width, T	1.1456	ft
Froude number, F	0.52	
Shear stress (tractive force), tau	0.1093	psf

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WEST CHERRY ST.
70% DESIGN FLOW

EXHIBIT 'F'

Free Online Manning Pipe Flow Calculator

List of Calculators Hydraulics Language

Manning Formula Uniform Pipe Flow at Given Slope and Depth

Can you help me translate this calculator to your language or host this calculator at your web site?

Set units: m mm ft inches

Pipe diameter, d_0	15	inches
Manning roughness, n ?	0.013	
Pressure slope (possibly ? equal to pipe slope), S_0	.002	rise/run
Percent of (or ratio to) full depth (100% or 1 if flowing full)	100	%

Results:

Flow, q	1.8669	MGD
Velocity, v	2.3539	ft/sec
Velocity head, h_v	0.0861	ft
Flow area	1.2272	ft^2
Wetted perimeter	3.9270	ft
Hydraulic radius	0.3125	ft
Top width, T	0.0000	ft
Froude number, F	0.00	
Shear stress (tractive force), tau	0.1561	psf

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WEST CHERRY ST.
100% DESIGN FLOW

EXHIBIT 'F'

List of Calculators Hydraulics Language

Manning Formula Uniform Pipe Flow at Given Slope and Depth

Can you help me translate this calculator to your language or host this calculator at your web site?

Pipe diameter, d_0	14.5	inches
Manning roughness, n ?	0.010	
Pressure slope (possibly ? equal to pipe slope), S_0	.002	rise/run
Percent of (or ratio to) full depth (100% or 1 if flowing full)	56	%

Results:

Flow, q	1.3369	MGD
Velocity, v	3.1306	ft/sec
Velocity head, h_v	0.1523	ft
Flow area	0.6608	ft^2
Wetted perimeter	2.0434	ft
Hydraulic radius	0.3234	ft
Top width, T	1.1996	ft
Froude number, F	0.74	
Shear stress (tractive force), tau	0.0845	psf

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SOUTH FIRST ST.
EX FLOW 15" PVC
LINED

EXHIBIT 'P'

List of Calculators Hydraulics Language

Manning Formula Uniform Pipe Flow at Given Slope and Depth

Can you help me translate this calculator to your language or host this calculator at your web site?

		Results:		
		Flow, q	2.2171	MGD
Pipe diameter, d_0	14.5 inches	Velocity, v	2.9917	ft/sec
		Velocity head, h_v	0.1391	ft
Manning roughness, n ?	.01	Flow area	1.1467	ft^2
Pressure slope (possibly ? equal to pipe slope), S_0	.002 rise/run	Wetted perimeter	3.7961	ft
		Hydraulic radius	0.3021	ft
Percent of (or ratio to) full depth (100% or 1 if flowing full)	100 %	Top width, T	0.0000	ft
		Froude number, F	0.00	
		Shear stress (tractive force), tau	0.1509	psf

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S. FIRST ST.

15" PVC, 100%

EXHIBIT 'F'

List of Calculators Hydraulics Language

Manning Formula Uniform Pipe Flow at Given Slope and Depth

Can you help me translate this calculator to your language or host this calculator at your web site?

Pipe diameter, d_0	14.5 inches	
Manning roughness, n ?	0.010	
Pressure slope (possibly ? equal to pipe slope), S_0	.002 rise/run	
Percent of (or ratio to) full depth (100% or 1 if flowing full)	70 %	

Results:		
Flow, q	1.8563	MGD
Velocity, v	3.3501	ft/sec
Velocity head, h_v	0.1744	ft
Flow area	0.8574	ft^2
Wetted perimeter	2.3953	ft
Hydraulic radius	0.3579	ft
Top width, T	1.1074	ft
Froude number, F	0.67	
Shear stress (tactive force), tau	0.1056	psf

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S. FIRST STREET
15" PVC, 70%
DESIGN FLOW

EXHIBIT 'G'

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EXHIBIT 'G'
CITY OF VACAVILLE

Table DS 6-1

Average Dry Weather Sanitary Sewer Flow Criteria (Qa)

Description	Residential, gpd/du [a]	Schools, gpd/student	Non-Residential, gpd/acre [a]	Non-Residential, gpd/ft^2
Residential – One Bedroom [b]	120	–	–	–
Residential – Two Bedroom [b]	160	–	–	–
Residential – Three Bedroom [b]	200	–	–	–
Residential – Four (or more) Bedroom	240	–	–	–
Residential [c]	240			
Office	–	–	1,500	0.115
Business Park	–	–	2,000	0.153
Industrial [d]	–	–	2,000	0.153
Retail Sales	–	–	1,900	0.145
Downtown Commercial	–	–	5,000	0.383
Highway Commercial	–	–	5,000	0.383
Service Commercial	–	–	1,900	0.145
Public - Low Water Use	–	–	0	0
Public - Medium Water Use	–	–	1,500	0.115
Public - High Water Use	–	–	1,500	0.115
Park	–	–	0	0
Private Recreation [e]	–	–	1,500	0.115
Elementary School	–	25	–	–
Secondary School	–	30	–	–
School Acreage	–	–	0	0
Open Space	–	–	0	0
Hospitals and Medical offices [f]	–	–	4,000	0.306
Places of Worship			1900	0.145
Agriculture	–	–	0	0

(a) gpd = gallons per day; du = dwelling unit

(b) Applicable only where the actual allowable dwelling unit and bedroom count is known and subject to no further changes by virtue of an executed development agreement or similar instrument. Where an executed development agreement or approved subdivision map applies to the sewer service area, the number of dwelling units shall be equal to the maximum allowed under such documents.

(c) This factor shall be applied where only an approximate dwelling unit count is available, and for predicted future growth in the service area.

(d) Applies only to dry industries. Design flows for industrial developments with the potential to produce above average flows must be computed on a case-by-case basis.

(e) Factor is not applied to golf course areas of play

(f) Qa shall be based upon project-specific flow projection with a minimum of 4,000 gpd/acre.

***NOTE: Table DS 6-1** is subject to periodic revisions based upon updated wastewater flow monitoring studies and master planning.*

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EXHIBIT 'H'
CITY OF ROSEVILLE

SECTION 9

SANITARY SEWER DESIGN

9-1 **DESIGN CRITERIA** – These criteria shall apply to the engineering design of any sanitary sewer system to be maintained by the City of Roseville, or with those exceptions as noted, within private multiple ownership residential or multi-parcel commercial and industrial developments.

9-2 **AVERAGE FLOW DETERMINATION** – The determination of average dry weather flows for design purposes shall be based upon the best available information concerning land use and density as determined by the Environmental Utilities Director. This information may include approved land use and density in accordance with current zoning in the absence of more specific information pertaining to expected development. Average dry weather flow factors are listed in Table 1.

Table 1 – Average Dry Weather Unit Flow Factors

Land Use Designation	**Units**	**Flow Factor (gpd/unit)**[1]
Commercial	gpd per acre	850
Heavy Industrial	gpd per acre	850
Light Industrial	gpd per acre	850
Mixed Use	gpd per acre	2,300
Public/Quasi-Public	gpd per acre	660
Schools	gpd per acre	170
Residential 1 DU	gpd per DU	190
Residential 2 DU	gpd per DU	190
Residential 3 DU	gpd per DU	190
Residential Multiple DU[2]	gpd per acre Or gpd per DU	2,040 Or 130
Open Space	gpd per acre	0
Parks> 10 Acres	gpd per acre	10
Vacant	gpd per acre	0

[1]Includes allowances for dry season groundwater infiltration (GWI)
[2]Future development projects should use the factor that results in the highest flow.

EXHIBIT 'I'

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EXHIBIT 'I'
WATER METER READINGS
BRISTOL APARTMENTS

Customer Name	Service Address	Read Date	Read	Consumption
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/30/2005	0	0
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/31/2006	391	391
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/31/2006	742	351
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/31/2006	1454	712
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/27/2006	2189	735
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/26/2006	2933	744
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/29/2006	3764	831
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/29/2007	4605	841
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/29/2007	5396	791
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/31/2007	6136	740
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/30/2007	6892	756
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/27/2007	7876	984
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	12/3/2007	9000	1124
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/31/2008	9773	773
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/31/2008	595	822
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	6/2/2008	1437	842
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/31/2008	2294	857
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/30/2008	3275	981
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/26/2008	4142	867
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/30/2009	5155	1013
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/31/2009	6065	910
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/29/2009	6910	845
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/31/2009	7746	836
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/30/2009	8651	905
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/30/2009	9591	940
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/29/2010	523	932
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/29/2010	1379	856
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	6/1/2010	2327	948
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/30/2010	3257	930
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/30/2010	4280	1023
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/24/2010	5167	887
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	2/1/2011	6323	1156
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/31/2011	7302	979
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/31/2011	8259	957
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/29/2011	9148	889
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/30/2011	44	896
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	11/30/2011	925	881
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/27/2012	1777	852
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/28/2012	2676	899
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/31/2012	3606	930
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	7/27/2012	4429	823
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	9/21/2012	5245	816
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	12/4/2012	6406	1161
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	1/31/2013	7322	916
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	3/28/2013	8162	840
FAIRFIELD BRISTOL LP	1550 VALLEY GLEN DRIVE	5/29/2013	9101	939

EXHIBIT 'J'

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EXHIBIT 'J'

SOUTH MASTER SEWER STUDY

VALLEY GLEN

PULTE HOMES

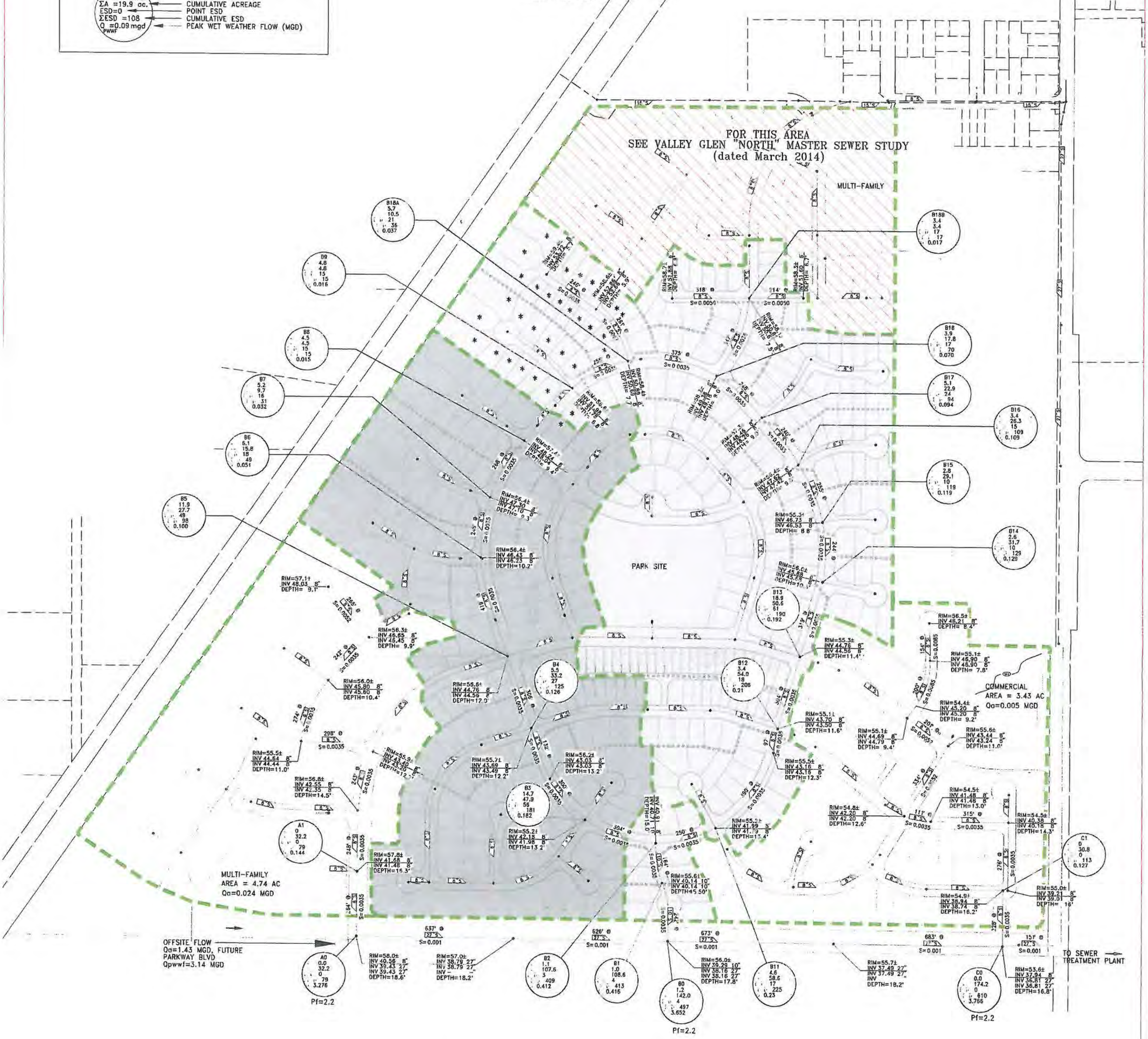
CITY OF DIXON, CALIFORNIA

REV. March, 2014

SCALE: 1"=200'

LEGEND

- 8"S — PROPOSED SEWER PIPE (INCHES)
- PROPOSED SEWER MANHOLE
- PROJECT BOUNDARY
- PROJECT BOUNDARY
- PROJECT BOUNDARY
- 1 — NODE NUMBER
- A =0.00 ac. — POINT ACREAGE
- ΣA =19.9 ac. — CUMULATIVE ACREAGE
- ESD=0 — POINT ESD
- ΣESD =108 — CUMULATIVE ESD
- Q =0.09 mgd PWWF — PEAK WET WEATHER FLOW (MGD)



VALLEY GLEN

City of Dixon

Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.75

Note:

Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

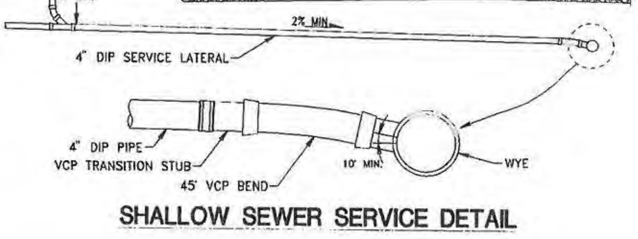
Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

NODE		D_U	ΣD_U	Q_{AVE}	Multi Fam [Q_{AVE}]	ΣQ_{AVE}	Q_P	I&I					Q_{TOT}
US	DS			$=\Sigma D_U$ * 350	$=\Sigma A$ * 5000 gpd		$=\Sigma Q_{AVE}$ * 2.5	Area	ΣArea	I	I	ΣI	$= Q_P + \Sigma I$
				(mgd)	(mgd)	(mgd)	(mgd)	(acres)	(acres)	(gpd)	(gpd)	(mgd)	(mgd)
B18B	B18	17	17	0.0060	-	0.006	0.015	3.4	3.4	1700	0.002	0.00	0.017
B18A	B18	21	36	0.0074	-	0.013	0.032	5.7	10.5	2850	0.003	0.01	0.037
B18	B17	17	70	0.0060	-	0.025	0.061	3.9	17.8	1950	0.002	0.01	0.070
B17	B16	24	94	0.0084	-	0.033	0.082	5.1	22.9	2550	0.003	0.01	0.094
B16	B15	15	109	0.0053	-	0.038	0.095	3.4	26.3	1700	0.002	0.01	0.109
B15	B14	10	119	0.0035	-	0.042	0.104	2.8	29.1	1400	0.001	0.01	0.119
B14	B13	10	129	0.0035	-	0.045	0.113	2.6	31.7	1300	0.001	0.02	0.129
B13	B12	61	190	0.0214	-	0.067	0.166	18.9	50.6	9450	0.009	0.03	0.192
B12	B11	18	208	0.0063	-	0.073	0.182	3.4	54	1700	0.002	0.03	0.209
B11	B2	17	225	0.0060	-	0.079	0.197	4.6	58.6	2300	0.002	0.03	0.226
B9	B18A	15	15	0.0053	-	0.005	0.013	4.8	4.8	2400	0.002	0.00	0.016
B8	B7	15	15	0.0053	-	0.005	0.013	4.5	4.5	2250	0.002	0.00	0.015
B7	B6	16	31	0.0056	-	0.011	0.027	5.2	9.7	2600	0.003	0.00	0.032
B6	B5	18	49	0.0063	-	0.017	0.043	6.1	15.8	3050	0.003	0.01	0.051
B5	B4	49	98	0.0172	-	0.034	0.086	11.9	27.7	5950	0.006	0.01	0.100
B4	B3	28	126	0.0098	-	0.044	0.110	5.5	33.2	2750	0.003	0.02	0.127
B3	B2	56	182	0.0196	-	0.064	0.159	14.7	47.9	7350	0.007	0.02	0.183
B2	B1	3	410	0.0011	-	0.144	0.359	1.1	107.6	550	0.001	0.05	0.413
B1	B0	4	414	0.0014	-	0.145	0.362	1.0	108.6	500	0.001	0.05	0.417
B0	C0	4	498	0.0014	-	1.628	3.582	1.2	121.7	600	0.001	0.06	3.642
A1	A0	0	79	0.0000	0.024	0.051	0.128	32.2	32.2	16100	0.016	0.02	0.144
A0	B0	0	80	0.0000	0.024	1.482	3.260	11.9	11.9	5950	0.006	0.01	3.266
C1	C0	0	113	0.0000	0.005	0.045	0.112	32.2	32.2	16100	0.016	0.02	0.128
C0	WWTP	0	611	0.0000	0.029	1.673	3.680	0	153.9	0	0.000	0.08	3.757

* NOTE:
INDICATES LOTS WITH POTENTIAL FOR SHALLOW SEWER SERVICE (SEE DETAIL BELOW)



SHALLOW SEWER SERVICE DETAIL

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NORTH MASTER SEWER STUDY

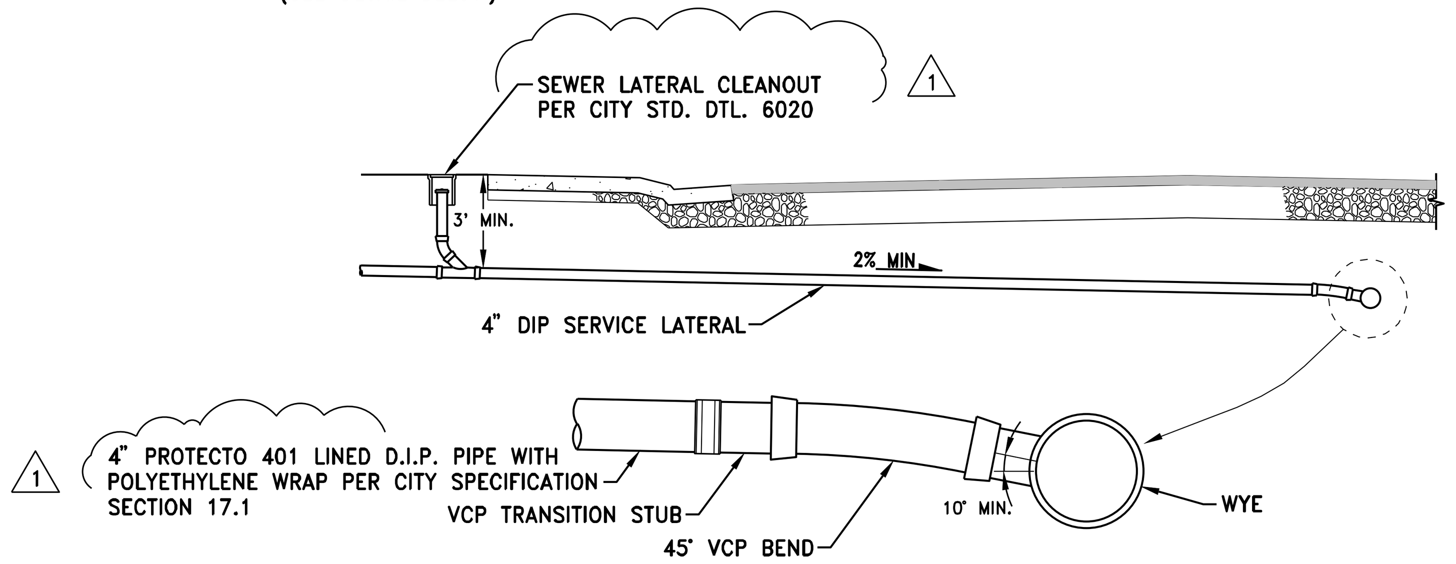
VALLEY GLEN

RICHLAND COMMUNITIES

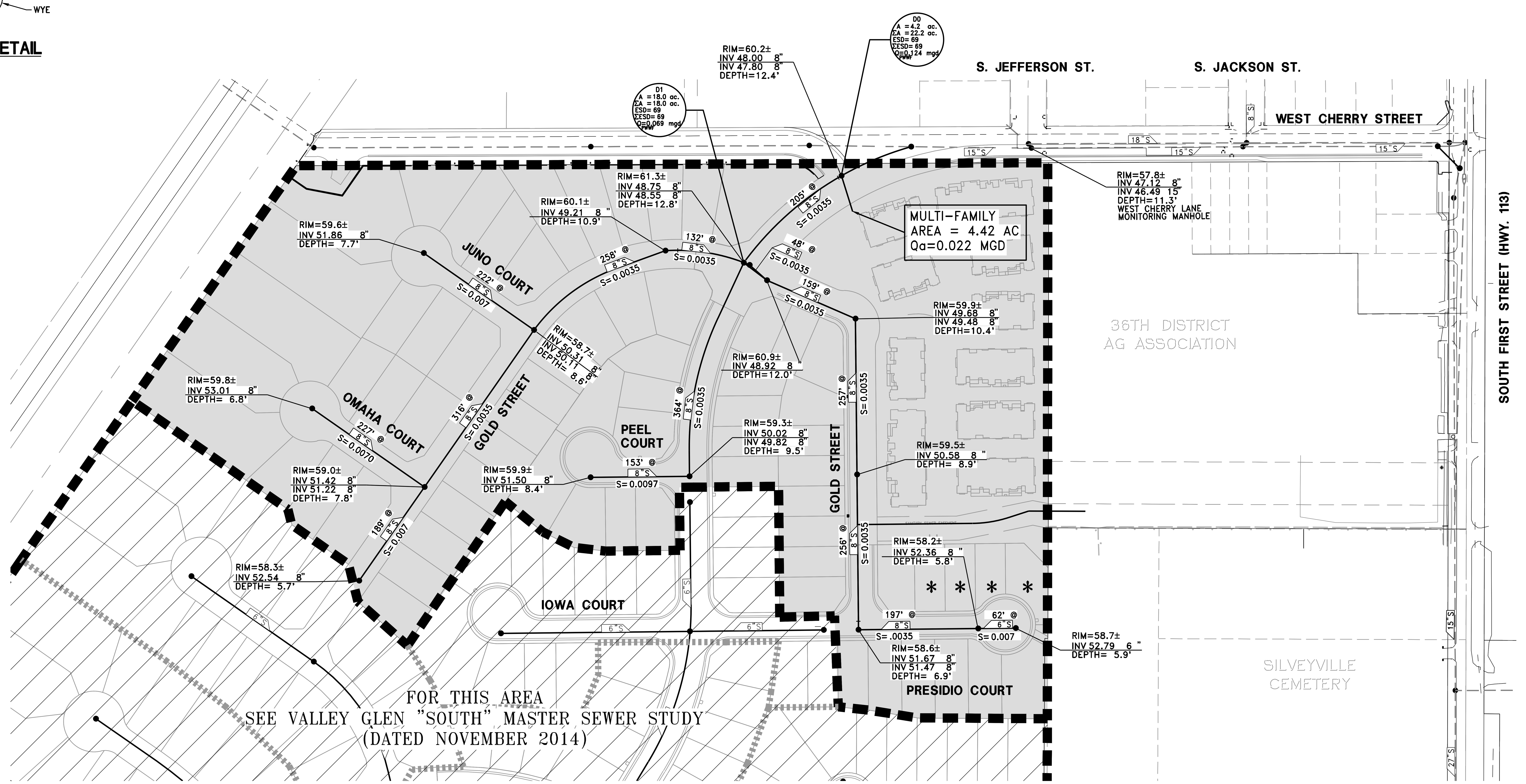
CITY OF DIXON CALIFORNIA

MARCH 2014
Rev. November 2014

* NOTE:
INDICATES LOTS POTENTIALLY REQUIRING SHALLOW SEWER SERVICE (SEE DETAIL BELOW)



SHALLOW SEWER SERVICE DETAIL



SCALE: 1"=100'

VALLEY GLEN
City of Dixon
Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.76

Note:
Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

NODE		D_U	ΣD_U	Q_{AVE}	Multi Fam [Q_{AVE}]	ΣQ_{AVE}	Q_P	I&I					Q_{TOT}
US	DS			$=\Sigma D_U * 350$	$=\Sigma A * 5000$ gpd		$=\Sigma Q_{AVE} * 2.5$	Area	ΣArea	I	I	ΣI	$= Q_P + \Sigma I$
				(mgd)	(mgd)	(mgd)	(mgd)	(acres)	(acres)	(gpd)	(gpd)	(mgd)	(mgd)
D1	D0	69	69	0.0242		0.024	0.060	18.0	18.0	9000	0.009	0.01	**0.069**
D0	W. CHERRY	0	69	0.0000	0.022	0.046	0.115	4.4	22.4	2200	0.002	0.01	**0.127**

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APPENDIX D

Previously Approved Valley Glen South Area Master Sewer Study Shed Map

(August 2003)

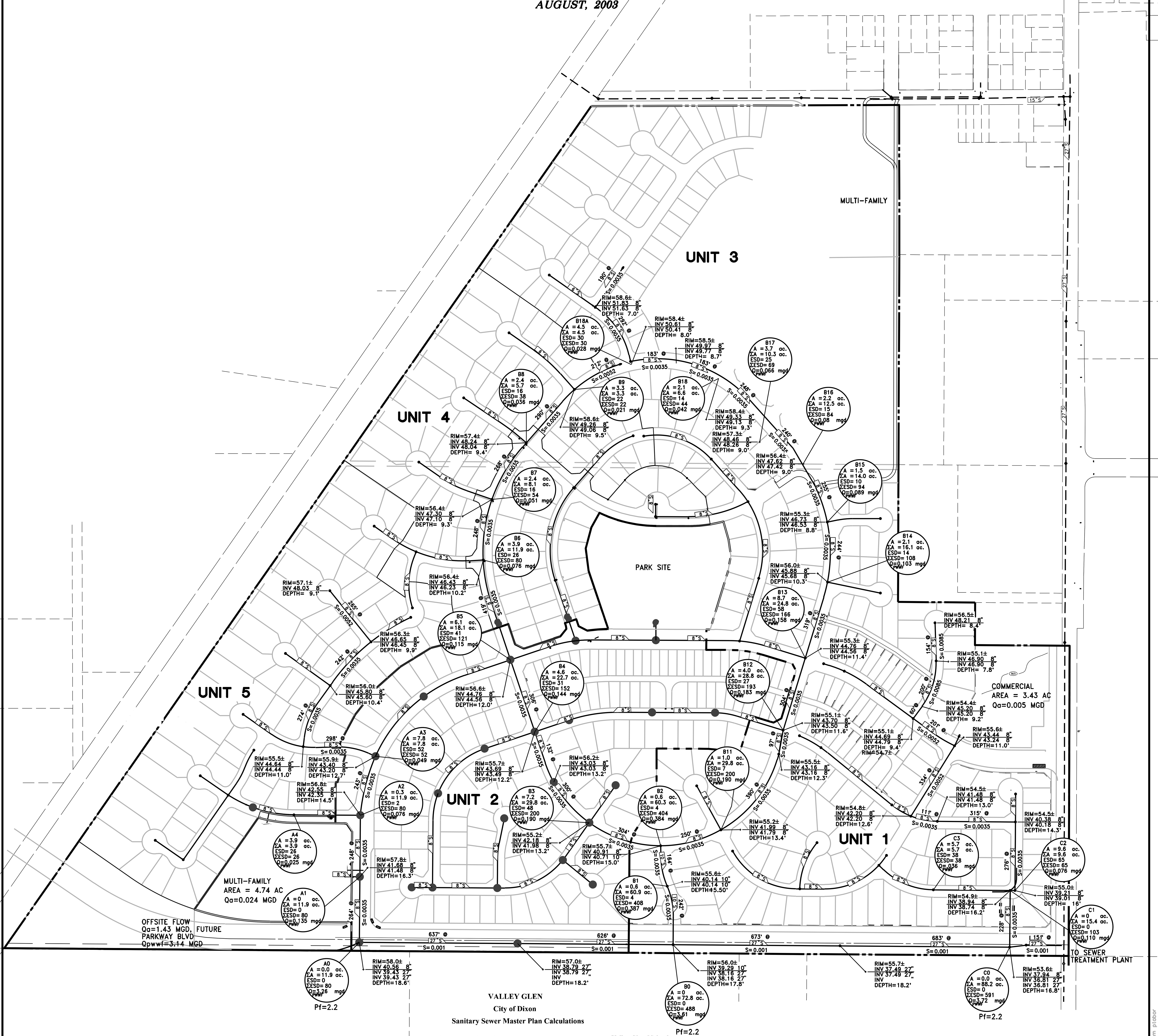
SOUTH MASTER SEWER STUDY

VALLEY GLEN

PULTE HOMES

CITY OF DIXON CALIFORNIA

AUGUST, 2003



VALLEY GLEN

City of Dixon

Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.76

Note:

Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

Valley Glen SS Study

NODE		D_U	Q	ΣQ_{AVE}	Q_P	I & I				Q
US	DS		$=\Sigma D_U * 350$		$=\Sigma Q_{AVE} * 2.5$	Area	I	I	ΣI	$=Q_P + \Sigma I$
			(mgd)	(mgd)	(mgd)	(acres)	(gpd)	(mgd)	(mgd)	(mgd)
A4	A2	26	0.0091	0.0091	0.023	3.9	1939.9	0.0019	0.0019	**0.025**
A3	A2	52	0.0182	0.0182	0.046	7.8	3879.7	0.0039	0.0039	**0.049**
A2	A1	2	0.0007	0.0280	0.070	0.3	149.2	0.0001	0.0060	**0.076**
A1[1]	A0	0	0.0237	0.0517	0.129	0.0	0.0	0.0000	0.0060	**0.135**
B18A	B18	30	0.0105	0.0105	0.026	4.5	2238.3	0.0022	0.0022	**0.028**
B18	B17	14	0.0049	0.0154	0.039	2.1	1044.5	0.0010	0.0033	**0.042**
B17	B16	25	0.0088	0.0242	0.060	3.7	1865.2	0.0019	0.0051	**0.066**
B16	B15	15	0.0053	0.0294	0.074	2.2	1119.1	0.0011	0.0063	**0.080**
B15	B14	10	0.0035	0.0329	0.082	1.5	746.1	0.0007	0.0070	**0.089**
B14	B13	14	0.0049	0.0378	0.095	2.1	1044.5	0.0010	0.0081	**0.103**
B13	B12	58	0.0203	0.0581	0.145	8.7	4327.4	0.0043	0.0124	**0.158**
B12	B11	27	0.0095	0.0676	0.169	4.0	2014.5	0.0020	0.0144	**0.183**
B11	B2	7	0.0025	0.0700	0.175	1.0	522.3	0.0005	0.0149	**0.190**
B9	B8	22	0.0077	0.0077	0.019	3.3	1641.4	0.0016	0.0016	**0.021**
B8	B7	16	0.0056	0.0133	0.033	2.4	1193.8	0.0012	0.0028	**0.036**
B7	B6	16	0.0056	0.0189	0.047	2.4	1193.8	0.0012	0.0040	**0.051**
B6	B5	26	0.0091	0.0280	0.070	3.9	1939.9	0.0019	0.0060	**0.076**
B5	B4	41	0.0144	0.0424	0.106	6.1	3059.0	0.0031	0.0090	**0.115**
B4	B3	31	0.0109	0.0532	0.133	4.6	2312.9	0.0023	0.0113	**0.144**
B3	B2	48	0.0168	0.0700	0.175	7.2	3581.3	0.0036	0.0149	**0.190**
B2	B1	4	0.0014	0.1414	0.354	0.6	298.4	0.0003	0.0301	**0.384**
B1	B0	4	0.0014	0.1428	0.357	0.6	298.4	0.0003	0.0304	**0.387**
C3	C1	38	0.0133	0.0133	0.033	5.7	2835.2	0.0028	0.0028	**0.036**
C2	C1	63	0.0221	0.0272	0.068	9.4	4700.4	0.0047	0.0064	**0.074**
C1	C0	0	0.0000	0.0405	0.101	0.0	0.0	0.0000	0.0093	**0.110**
A0[1]	B0[1]	0	0.0000	1.4300	3.260	0.0	0.0	0.0000	0.0000	**3.260**
B0	C0	0	0.0000	1.6400	3.610	0.0	0.0	0.0000	0.0000	**3.610**
C0	END	0	0.0000	1.6900	3.720	0.0	0.0	0.0000	0.0000	**3.720**

1) INCLUDES FUTURE OFFSITE FLOW-Qa=1.43MGD, FUTURE PARKWAY BLVD.

LEGEND

8"S — PROPOSED SEWER PIPE (INCHES)

PROPOSED SEWER MANHOLE

PROJECT BOUNDARY

1 — NODE NUMBER

A =0.00 ac. — POINT ACREAGE

ΣA =19.9 ac. — CUMULATIVE ACREAGE

ESD=0 — POINT ESD

ΣESD =108 — CUMULATIVE ESD

Q_{PWWF} =0.09 mgd — PEAK WET WEATHER FLOW (MGD)

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APPENDIX E

Valley Glen South Area Master Sewer Study Shed Map

(February 2015)

LEGEND

PROPOSED SEWER PIPE (INCHES)
PROPOSED SEWER MANHOLE
PROJECT BOUNDARY
PROJECT BOUNDARY
PROJECT BOUNDARY
NODE NUMBER
POINT ACREAGE
CUMULATIVE ACREAGE
POINT ESD
CUMULATIVE ESD
PEAK WET WEATHER FLOW (MGD)

SOUTH AREA MASTER SEWER STUDY SHED MAP FOR:

VALLEY GLEN

PULTE HOMES

CITY OF DIXON CALIFORNIA

AUGUST, 2003
REV. January, 2014
REV. March, 2014
REV. November, 2014
FEBRUARY 2015

NOTE 1: DUE TO WATER MAIN DESIGN, "DEAD END" MIN. SLOPE OF 0.0052 CANNOT BE ACHIEVED.

VALLEY GLEN
City of Dixon
Sanitary Sewer Master Plan Calculations

Sewer Main Capacity:

Pipe Size (in)	Min. Slope (ft/ft)	Capacity 50% full (mgd)	Capacity 70% full (mgd)
8"	0.0035	0.23	0.32
10"	0.0030	0.39	0.54
12"	0.0022	0.54	0.76

Note:

Pipe capacity is based upon manning's n=0.013

Unit Flow Rates

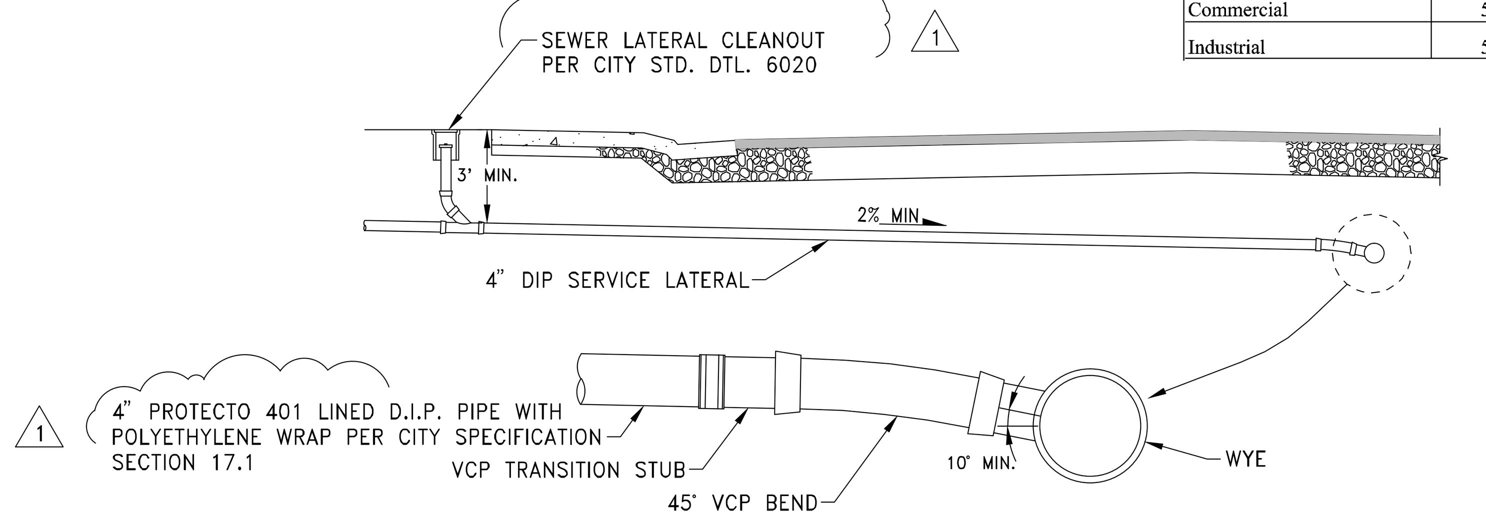
Single Family	350	gpd/unit
Multi-Family	5000	gpd/acre
Commercial	1500	gpd/acre
Industrial	2000	gpd/acre

Infiltration and Inflow

Single Family	500	gpd/acre
Multi-Family	500	gpd/acre
Commercial	500	gpd/acre
Industrial	500	gpd/acre

NODE		D_U	ΣD_U	Q_{AVE}	Multi Fam [Q_{AVE}]	ΣQ_{AVE}	Q_P	I&I					Q_{TOT}
US	DS			$=\Sigma D_U * 350$	$=\Sigma A * 5000$ gpd		$=\Sigma Q_{AVE} * 2.5$	Area	ΣArea	I	I	ΣI	$= Q_P + \Sigma I$
				(mgd)	(mgd)	(mgd)	(mgd)	(acres)	(acres)	(gpd)	(gpd)	(mgd)	(mgd)
B18B	B18	17	17	0.0060	-	0.006	0.015	3.4	3.4	1700	0.002	0.00	**0.017**
B9	B18A	15	15	0.0053	-	0.005	0.013	4.8	4.8	2400	0.002	0.00	**0.016**
B18A	B18	21	36	0.0074	-	0.013	0.032	5.7	10.5	2850	0.003	0.01	**0.037**
B18	B17	17	70	0.0060	-	0.025	0.061	3.9	17.8	1950	0.002	0.01	**0.070**
B17	B16	24	94	0.0084	-	0.033	0.082	5.1	22.9	2550	0.003	0.01	**0.094**
B16	B15	15	109	0.0053	-	0.038	0.095	3.4	26.3	1700	0.002	0.01	**0.109**
B15	B14	10	119	0.0035	-	0.042	0.104	2.8	29.1	1400	0.001	0.01	**0.119**
B14	B13	10	129	0.0035	-	0.045	0.113	2.6	31.7	1300	0.001	0.02	**0.129**
B13	B12	61	190	0.0214	-	0.067	0.166	18.9	50.6	9450	0.009	0.03	**0.192**
B12	B11	18	208	0.0063	-	0.073	0.182	3.4	54	1700	0.002	0.03	**0.209**
B11	B2	17	225	0.0060	-	0.079	0.197	4.6	58.6	2300	0.002	0.03	**0.226**
B8	B7	15	15	0.0053	-	0.005	0.013	4.5	4.5	2250	0.002	0.00	**0.015**
B7	B6	16	31	0.0056	-	0.011	0.027	5.2	9.7	2600	0.003	0.00	**0.032**
B6	B5	18	49	0.0063	-	0.017	0.043	6.1	15.8	3050	0.003	0.01	**0.051**
B5	B4	49	98	0.0172	-	0.034	0.086	11.9	27.7	5950	0.006	0.01	**0.100**
B4	B3	28	126	0.0098	-	0.044	0.110	5.5	33.2	2750	0.003	0.02	**0.127**
B3	B2	56	182	0.0196	-	0.064	0.159	14.7	47.9	7350	0.007	0.02	**0.183**
B2	B1	3	410	0.0011	-	0.144	0.358	1.1	107.6	550	0.001	0.05	**0.412**
B1	B0	4	414	0.0014	-	0.145	0.362	1.0	108.6	500	0.001	0.05	**0.417**
B0	C0	4	498	0.0014	-	1.628	3.582	1.2	142	600	0.001	0.07	**3.653**
A1	A0	0	79	0.0000	0.024	0.051	0.128	32.2	32.2	16100	0.016	0.02	**0.144**
A0	B0	0	80	0.0000	0.024	1.482	3.260	0	32.2	0	0.000	0.02	**3.276**
C1	C0	0	113	0.0000	0.005	0.045	0.112	30.8	30.8	15400	0.015	0.02	**0.127**
C0	WWTP	0	611	0.0000	0.029	1.673	3.680	0	172.8	0	0.000	0.09	**3.766**

* NOTE:
INDICATES LOTS POTENTIALLY REQUIRING SHALLOW SEWER SERVICE (SEE DETAIL BELOW)



SHALLOW SEWER SERVICE DETAIL

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Appendix C   MODEL CALIBRATION DETAILS

ADWF Calibration - Sewer Collection System Model

Dixon Sewer Collection System Master Plan

DATE:	2/11/2021
PREPARED BY:	BEW
PROJ NUMBER:	184031201
PAGE:	1 of 3

Site	MHID	UP PIPE ID	DWN PIPE ID	Size (in)
1	SS0098N	C0059N	C0058N	30
2	SS1104	C117	C0056N	27
3	SS1081	C284	C285	15

Upstream conduit results

Calibration Period: 3/7/2020 3/14/2020

Average Flow (MGD)			
Site	Modeled	Observed	Error
1	0.2996	0.2995	0.03%
2	0.7891	0.7911	-0.25%
3	0.572	0.5728	-0.14%

Average Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.4281	0.4654	-8.01%
2	2.65	2.57	3.11%
3	2.123	2.121	0.09%

Average Depth (ft)			
Site	Modeled	Observed	Error
1	0.6605	0.6887	-4.09%
2	0.3745	0.3824	-2.07%
3	0.4467	0.4338	2.97%

Maximum Flow (MGD)			
Site	Modeled	Observed	Error
1	0.5232	0.569	-8.05%
2	1.377	1.411	-2.41%
3	1.029	1.08	-4.72%

Maximum Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.6715	0.7	-4.07%
2	3.194	3.34	-4.37%
3	2.538	2.81	-9.68%

Maximum Depth (ft)			
Site	Modeled	Observed	Error
1	0.8201	0.8517	-3.71%
2	0.5047	0.5133	-1.68%
3	0.6297	0.5942	5.97%

Minimum Flow (MGD)			
Site	Modeled	Observed	Error
1	0.06418	0.063	1.87%
2	0.1867	0.185	0.92%
3	0.1273	0.134	-5.00%

Minimum Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.1641	0.16	2.56%
2	1.756	1.48	18.65%
3	1.363	1.07	27.38%

Minimum Depth (ft)			
Site	Modeled	Observed	Error
1	0.4179	0.4575	-8.66%
2	0.1921	0.2133	-9.94%
3	0.218	0.255	-14.51%

Total Flow (MG)			
Site	Modeled	Observed	Error
1	2.097	2.096	0.05%
2	5.524	5.529	-0.09%
3	4.004	4.009	-0.12%

ADWF Calibration - Sewer Collection System Model

Dixon Sewer Collection System Master Plan

DATE:	2/11/2021
PREPARED BY:	BEW
PROJ NUMBER:	184031201
PAGE:	2 of 3

Site	MHID	UP PIPE ID	DWN PIPE ID	Size (in)
1	SS0098N	C0059N	C0058N	30
2	SS1104	C117	C0056N	27
3	SS1081	C284	C285	15

Downstream conduit results

Calibration Period: 3/7/2020 3/14/2020

Average Flow (MGD)			
Site	Modeled	Observed	Error
1	0.2996	0.2995	0.03%
2	0.7891	0.7911	-0.25%
3	0.572	0.5728	-0.14%

Average Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.4019	0.4654	-13.64%
2	2.599	2.57	1.13%
3	2.355	2.121	11.03%

Average Depth (ft)			
Site	Modeled	Observed	Error
1	0.6914	0.6887	0.39%
2	0.3797	0.3824	-0.71%
3	0.4137	0.4338	-4.63%

Maximum Flow (MGD)			
Site	Modeled	Observed	Error
1	0.523	0.569	-8.08%
2	1.377	1.411	-2.41%
3	1.028	1.08	-4.81%

Maximum Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.6396	0.7	-8.63%
2	3.132	3.34	-6.23%
3	2.858	2.81	1.71%

Maximum Depth (ft)			
Site	Modeled	Observed	Error
1	0.8515	0.8517	-0.02%
2	0.5118	0.5133	-0.29%
3	0.5796	0.5942	-2.46%

Minimum Flow (MGD)			
Site	Modeled	Observed	Error
1	0.06279	0.063	-0.33%
2	0.1867	0.185	0.92%
3	0.1286	0.134	-4.03%

Minimum Velocity (ft/s)			
Site	Modeled	Observed	Error
1	0.1424	0.16	-11.00%
2	1.722	1.48	16.35%
3	1.555	1.07	45.33%

Minimum Depth (ft)			
Site	Modeled	Observed	Error
1	0.4512	0.4575	-1.38%
2	0.1947	0.2133	-8.72%
3	0.2013	0.255	-21.06%

Total Flow (MG)			
Site	Modeled	Observed	Error
1	2.097	2.096	0.05%
2	5.524	5.538	-0.25%
3	4.004	4.009	-0.12%

PWWF Calibration - Sewer Collection System Model
Dixon Sewer Collection System Master Plan
PWWF was calibrated using WWTF influent flow data

DATE:	2/11/2021
PREPARED BY:	BEW
PROJ NUMBER:	184031201
PAGE:	3 of 3

Calibration Events

		Start	End
Wet-Weather Event:		1/5/2019	1/8/2019
Total = 26.1 Time:		7:25 AM	9:00 AM
Wet-Weather Event 1			
Parameter	Modeled	Observed	Error
Mean	1.247	1.256	-0.72%
Max	2.73	2.724	0.22%
Min	0.2848	0.3727	-23.58%
Total	3.824	3.849	-0.65%

>10-year Rainfall Season (Water Year)
Total = 26.13 in

		Start	End
Wet-Weather Event:		1/14/2019	1/19/2019
Total = 26.13 Time:		12:30 PM	7:50 AM
Wet-Weather Event 2			
Parameter	Modeled	Observed	Error
Mean	1.341	1.298	3.31%
Max	3.207	3.179	0.88%
Min	0.3753	0.415	-9.57%
Total	5.102	4.941	3.26%

>10-year Rainfall Season (Water Year)
Total = 26.13 in

Validation Events

		Start	End
Wet-Weather Event:		2/25/2019	2/28/2019
Total = 26.1 Time:		11:10 AM	11:45 PM
Wet-Weather Event 3			
Parameter	Modeled	Observed	Error
Mean	1.473	1.41	4.47%
Max	2.677	2.452	9.18%
Min	0.4361	0.4844	-9.97%
Total	5.19	4.974	4.34%

>10-year Rainfall Season (Water Year)
Total = 26.13 in

		Start	End
Wet-Weather Event:		11/30/2019	12/3/2019
Total = 26.13 Time:		1:30 PM	6:50 AM
Wet-Weather Event 4			
Parameter	Modeled	Observed	Error
Mean	1.4	1.482	-5.53%
Max	2.097	2.284	-8.19%
Min	0.3866	0.3792	1.95%
Total	3.81	4.036	-5.60%

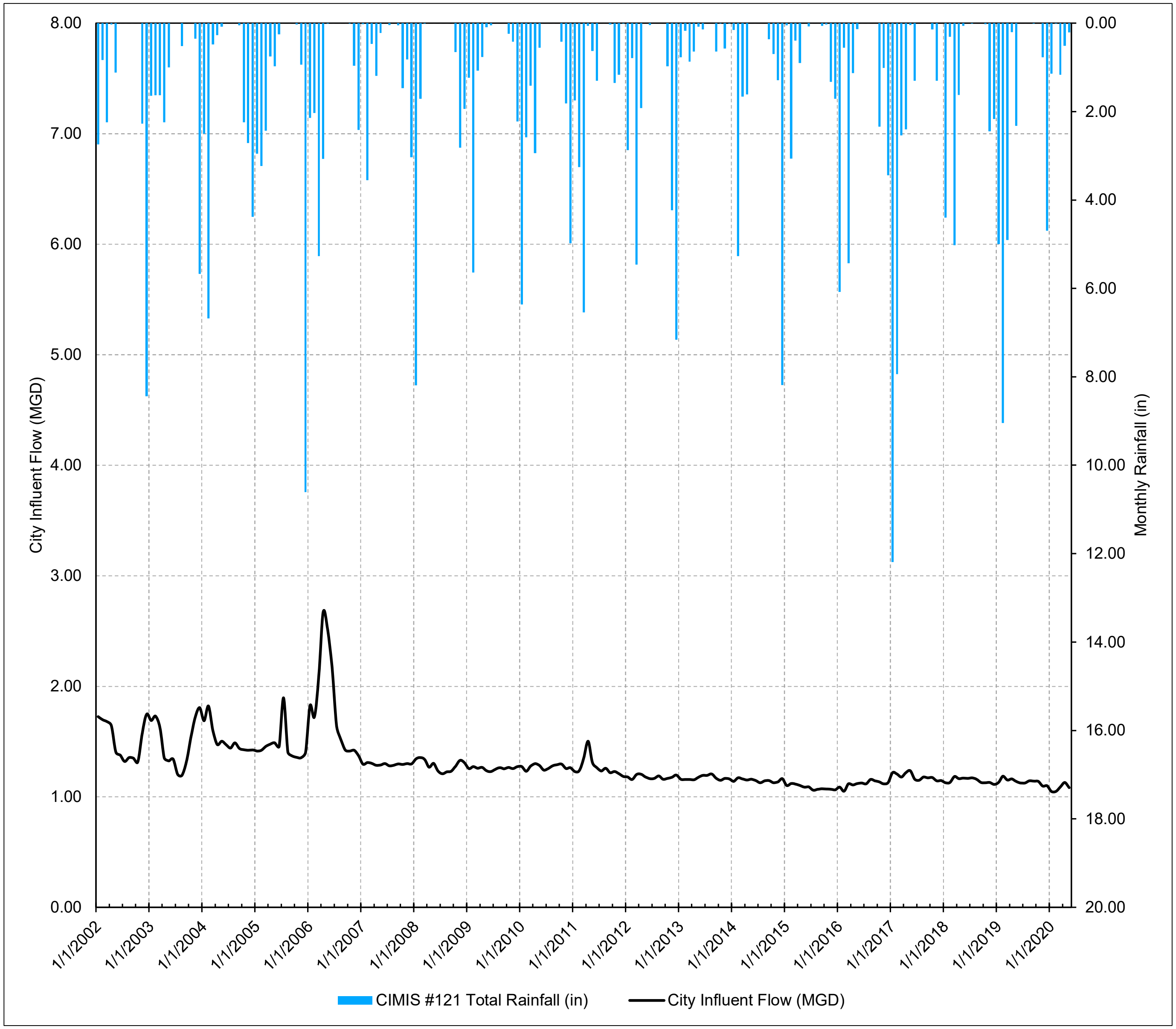
>10-year Rainfall Season (Water Year)
Total = 26.13 in

		Start	End
Wet-Weather Event:		1/8/2018	1/10/2018
Total = 12.89 Time:		1:30 AM	6:55 AM
Wet-Weather Event 4			
Parameter	Modeled	Observed	Error
Mean	1.525	1.298	17.49%
Max	2.819	2.202	28.02%
Min	0.3802	0.4254	-10.63%
Total	3.394	2.889	17.48%

<1-year Rainfall Season (Water Year)
Total = 12.89 in

PROJECT *DIXON SCSMP*
DESCRIPTION *RAINFALL DATA SUMMARY - MODEL CALIBRATION*
PROJ NUMBER: 184031201

DATE 2/11/2021
PREPARED BY: BEW



	Calendar Year	
Year	Rainfall (in)	AAF (MGD)
2002	17.7	1.52
2003	14.7	1.49
2004	19.4	1.52
2005	22.2	1.46
2006	15.9	1.82
2007	10.9	1.29
2008	15.3	1.29
2009	11.7	1.26
2010	21.1	1.28
2011	16.1	1.27
2012	23.4	1.18
2013	3.9	1.17
2014	19.2	1.15
2015	7.6	1.08
2016	20.1	1.12
2017	27.9	1.18
2018	16.1	1.15
2019	26.9	1.14

PROJECT *DIXON SCSMP*
HISTORICAL RAIN vs. Flow
CIMIS 121 Historical Rainfall **>>** 1994-2020

DATE 2/11/2021
PREPARED BY: BEW
PROJ NUMBER: 184031201
PAGE: 2 of 2

Month/Year	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	Mar	April	May	June	Water Year Rainfall (in)	Recurrance Interval Year(s)	Calendar Year Rainfall (in)
Average	0.01	0.02	0.21	0.72	2.07	3.44	4.06	3.67	2.90	1.09	0.49	0.15	18.82		
1977/1978	0.00	0.00	0.83	0.34	2.30	4.32	8.86	3.28	4.22	2.02	0.02	0.00	26.19	10	
1978/1979	0.00	0.00	0.54	0.00	2.27	0.68	5.66	3.82	1.89	0.98	0.16	0.00	16.00	<1	21.89
1979/1980	0.00	0.00	0.00	2.20	1.72	4.54	5.70	7.87	2.34	1.13	0.40	0.00	25.90	10	20.97
1980/1981	0.20	0.00	0.00	0.08	0.10	2.52	4.98	0.87	3.03	0.32	0.06	0.00	12.16	<1	20.34
1981/1982	0.00	0.00	0.27	1.40	4.83	3.40	6.94	3.19	6.38	5.16	0.00	0.03	31.60	50	19.16
1982/1983	0.00	0.00	1.11	2.74	5.64	3.13	5.26	5.37	7.49	2.81	0.42	0.00	33.97	100	34.32
1983/1984	0.00	0.00	0.00	0.99	6.16	6.89	0.35	1.29	1.09	0.41	0.00	0.03	17.21	1	35.39
1984/1985	0.00	0.15	0.04	1.24	6.69	1.12	0.92	2.15	3.21	0.00	0.03	0.03	15.58	<1	12.41
1985/1986	0.01	0.00	0.43	0.53	4.24	3.41	4.30	9.98	4.18	0.90	0.17	0.00	28.15	25	14.96
1986/1987	0.00	0.00	0.98	0.00	0.30	1.13	2.12	2.98	3.29	0.10	0.01	0.00	10.91	<1	21.94
1987/1988	0.00	0.00	0.00	0.52	2.23	4.64	5.00	0.64	0.09	1.74	0.65	0.32	15.83	<1	15.89
1988/1989	0.00	0.00	0.00	0.13	1.77	3.03	0.56	0.85	4.35	0.35	0.02	0.30	11.36	<1	13.37
1989/1990	0.00	0.09	2.46	1.59	1.26	0.00	4.29	3.64	0.69	0.00	2.44	0.00	16.46	<1	11.83
1990/1991	0.00	0.00	0.02	0.12	0.35	1.17	0.26	2.93	9.90	0.34	0.06	0.17	15.32	<1	12.72
1991/1992	0.00	0.03	0.00	0.60	0.09	1.61	2.07	7.90	3.19	0.53	0.00	0.28	16.30	<1	15.99
1992/1993	0.00	0.00	0.00	1.70	1.06	5.85	11.57	5.75	2.02	0.86	0.91	1.02	30.74	50	22.58
1993/1994	0.00	0.00	0.00	0.88	2.23	1.79	1.91	3.44	0.09	0.75	1.18	0.00	12.27	<1	27.03
1994/1995	0.00	0.00	0.16	0.30	3.81	2.58	11.57	0.11	8.69	1.29	0.61	0.65	29.77	25	14.22
1995/1996	0.00	0.00	0.00	0.03	0.02	6.93	5.41	6.04	1.73	1.53	2.11	0.00	23.80	1	29.90
1996/1997	0.00	0.00	0.00	1.08	1.73	7.85	9.62	0.31	0.59	0.06	0.70	0.12	22.06	1	27.48
1997/1998	0.00	0.14	0.05	0.43	3.91	1.91	5.15	10.18	1.83	0.80	2.46	0.12	26.98	10	17.84
1998/1999	0.00	0.00	0.43	0.55	2.76	0.72	2.12	4.60	1.74	1.03	0.00	0.00	13.95	<1	25.00
1999/2000	0.00	0.00	0.18	0.27	1.88	0.25	5.16	7.87	1.69	1.19	1.07	0.09	19.65	1	12.07
2000/2001	0.00	0.00	0.12	2.06	0.84	0.40	2.60	4.50	1.90	0.39	0.00	0.09	12.90	<1	20.49
2001/2002	0.00	0.00	0.20	0.40	3.52	6.97	2.74	0.83	2.24	0.03	1.12	0.00	18.05	1	20.57
2002/2003	0.00	0.00	0.00	0.00	2.27	8.44	1.64	1.63	1.63	2.24	1.00	0.00	18.85	1	17.67
2003/2004	0.00	0.52	0.00	0.00	0.35	5.67	2.50	6.68	0.48	0.27	0.08	0.00	16.55	<1	14.68
2004/2005	0.00	0.00	0.05	2.24	2.71	4.38	2.95	3.23	2.43	0.75	0.97	0.25	19.96	1	19.39
2005/2006	0.00	0.00	0.00	0.03	0.94	10.61	2.14	2.03	5.27	3.07	0.01	0.00	24.10	1	22.16
2006/2007	0.00	0.00	0.00	0.01	0.96	2.41	0.02	3.55	0.47	1.19	0.22	0.00	8.83	<1	15.90
2007/2008	0.04	0.00	0.05	1.47	0.82	3.03	8.19	1.71	0.01	0.00	0.00	0.00	15.32	<1	10.86
2008/2009	0.00	0.00	0.00	0.65	2.81	1.94	1.23	5.64	1.07	0.76	0.09	0.05	14.24	<1	15.31
2009/2010	0.00	0.00	0.00	0.24	0.42	2.22	6.36	2.58	1.41	2.94	0.55	0.00	16.72	<1	11.72
2010/2011	0.00	0.00	0.00	0.42	1.81	4.98	1.75	3.25	6.54	0.06	0.63	1.30	20.74	1	21.05
2011/2012	0.00	0.00	0.03	1.35	1.16	0.01	2.87	0.79	5.46	1.92	0.00	0.04	13.63	<1	16.08
2012/2013	0.00	0.00	0.00	0.97	4.23	7.16	0.77	0.17	0.87	0.64	0.08	0.14	15.03	<1	23.44
2013/2014	0.00	0.00	0.64	0.01	0.57	0.01	0.15	5.27	1.66	1.61	0.00	0.00	9.92	<1	3.90
2014/2015	0.00	0.00	0.36	0.70	1.29	8.18	0.05	3.06	0.39	0.90	0.00	0.07	15.00	<1	19.22
2015/2016	0.00	0.00	0.06	0.03	1.33	1.71	6.08	0.55	5.43	1.13	0.13	0.00	16.45	<1	7.60
2016/2017	0.00	0.00	0.00	2.34	1.01	3.44	12.19	7.94	2.54	2.40	0.04	1.30	33.20	100	20.11
2017/2018	0.00	0.00	0.00	0.14	1.30	0.00	4.40	0.31	5.02	1.62	0.06	0.00	12.85	<1	27.85
2018/2019	0.01	0.00	0.00	0.02	2.44	2.17	5.00	9.04	4.90	0.20	2.32	0.00	26.10	10	16.05
2019/2020	0.00	0.00	0.01	0.00	0.77	4.69	1.14	0.01	1.16	0.51	0.21	0.01	8.51	<1	26.93

PROJECT: Sewer Collection System Master Plan
JOB NUMBER: 184031201
CLIENT: City of Dixon
DESCRIPTION: *WWTF Flow Data*

DATE: 11/2/2020
PREPARED BY: BEW

WW FLOW PER CAPITA ASSESSMENT

Year	AAF (MGD)	ADWF (MGD)	Population	Per Capita Flow (gpcd)
2003	1.43	1.25		
2004	1.52	1.47		
2005	1.46	1.59	17,449	91
2006	1.82	1.78	17,914	99
2007	1.29	1.29	18,105	71
2008	1.29	1.22	18,148	67
2009	1.26	1.25	18,293	68
2010	1.27	1.26	18,441	68
2011	1.27	1.25	18,293	68
2012	1.18	1.17	18,388	64
2013	1.17	1.20	18,525	65
2014	1.15	1.14	18,986	60
2015	1.08	1.07	19,080	56
2016	1.12	1.13	19,229	59
2017	1.18	1.16	19,485	60
2018	1.15	1.17	19,686	59
2019	1.14	1.13	19,920	57

Wastewater Generation Rate Assessment, gpd per EDU

Average Per Capita Flow (gpcd):	68
Persons/EDU:	3.7
Average (gpd/EDU):	250
Design Standard (gpd/EDU):	350
Difference:	-100

Source: Facilities Plan Report Data, WWTF Facilities Plan Report (Stantec, 01/2014)

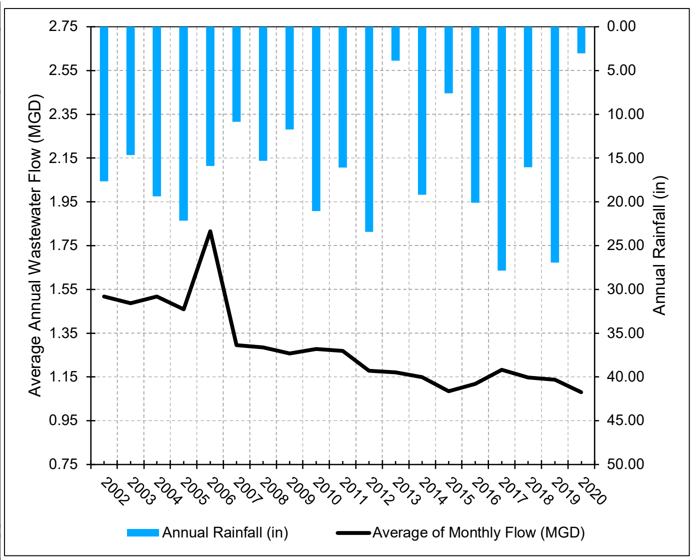
WWTF INFLUENT FLOW, PEAKING FACTORS

Year	Rainfall (in) CIMIS 121	AAF (MGD)	ADWF (MGD)	MMF (MGD)	MDF (MGD)	AAF/ADWF	MMF/AAF	MDF/AAF
2003		1.43	1.25	1.81	2.00	1.15	1.27	1.40
2004		1.52	1.47	1.82	2.26	1.03	1.20	1.49
2005		1.46	1.59	1.90	2.45	0.92	1.30	1.68
2006		1.82	1.78	2.67	3.16	1.02	1.47	1.74
2007		1.29	1.29	1.31	1.98	1.00	1.01	1.53
2008		1.29	1.22	1.36	1.97	1.05	1.05	1.53
2009		1.26	1.25	1.27	1.90	1.01	1.01	1.51
2010	21.05	1.27	1.26	1.30	1.71	1.01	1.02	1.35
2011	16.08	1.27	1.25	1.50	1.75	1.01	1.18	1.38
2012	23.44	1.18	1.17	1.20	1.51	1.00	1.02	1.28
2013	3.90	1.17	1.20	1.21	1.32	0.98	1.03	1.13
2014	19.22	1.15	1.14	1.17	1.69	1.01	1.02	1.48
2015	7.60	1.08	1.07	1.12	1.41	1.01	1.03	1.30
2016	20.11	1.12	1.13	1.16	1.45	0.99	1.04	1.30
2017	27.85	1.18	1.16	1.24	1.84	1.02	1.05	1.56
2018	16.04	1.15	1.17	1.18	1.40	0.98	1.03	1.22
2019	26.91	1.14	1.13	1.19	1.66	1.00	1.04	1.46
2020								
Average		1.28	1.27	1.44	1.85	1.01	1.11	1.43
Post 2006 Ave.		1.20	1.19	1.25	1.66	1.01	1.04	1.39
WWTF Design Rpt						*1.01*	*1.05*	*1.50*

Source: City of Dixon WWTF Improvements Project Design Report, TM 2: Influent Flows and Loads (11/07/13)

ANNUAL RAINFALL VS. AAF

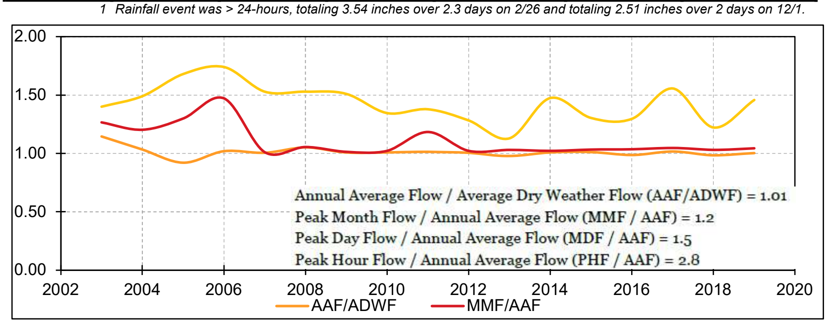
Year	Annual Rainfall (in)	Average of Monthly Flow (MGD)
2002	17.67	1.52
2003	14.68	1.49
2004	19.39	1.52
2005	22.16	1.46
2006	15.90	1.82
2007	10.86	1.29
2008	15.31	1.29
2009	11.72	1.26
2010	21.05	1.28
2011	16.08	1.27
2012	23.44	1.18
2013	3.90	1.17
2014	19.22	1.15
2015	7.60	1.08
2016	20.11	1.12
2017	27.85	1.18
2018	16.05	1.15
2019	26.93	1.14
2020	3.02	1.08
Average	16.47	1.29



PHF EVENT ANALYSIS

PHF Event	Date	AFF (MGD) 2019	Total Daily Flow (MGD)	PHF (MGD)	PHF/AAF	24-hr Rainfall Total (in) CIMIS #121
1	1/7/2019	1.14	1.42	2.67	2.35	1.84
2	1/16/2019	1.14	1.51	3.18	2.80	1.49
3 [(1)]	2/26/2019	1.14	1.53	2.45	2.16	2.17
4 [(1)]	12/1/2019	1.14	1.52	2.28	2.01	1.25
5	12/2/2019	1.14	1.67	2.28	2.00	0.97
6	1/8/2018	1.15	1.58	2.20	1.92	2.97

1 Rainfall event was > 24-hours, totaling 3.54 inches over 2.3 days on 2/26 and totaling 2.51 inches over 2 days on 12/1.



PROJECT: Sewer Collection System Master Plan
JOB NUMBER: 184031201
CLIENT: City of Dixon
DESCRIPTION: *WWTF Flow Data*

DATE: 11/2/2020
PREPARED BY: BEW
PAGE: 2 of 2

HISTORICAL WWTF FLOW

Month/Year	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT	OCT.	NOV.	DEC.	MAX (MMF)	AVE. (AAF)	July-Sept "ADWF"
1987	0.87	0.77	0.78	0.87	0.92	0.96	0.92	0.88	0.91	0.90	0.88	0.90	0.96	0.88	0.90
1988	0.92	0.89	0.88	0.91	0.93	0.93	0.92	0.91	0.91	0.91	0.92	0.90	0.93	0.91	0.91
1989	0.90	0.89	0.93	0.92	0.94	0.94	0.93	0.96	0.99	0.99	0.98	0.95	0.99	0.94	0.96
1990	0.98	0.89	0.93	0.92	0.94	0.94	0.93	0.96	0.99	0.99	0.98	0.95	0.99	0.95	0.96
1991	0.95	0.95	0.97	0.90	0.93	0.93	0.93	0.95	1.03	1.02	1.01	1.03	1.03	0.97	0.97
1992	1.03	1.08	1.05	1.04	1.04	1.06	1.03	1.04	1.06	1.08	1.07	1.14	1.14	1.06	1.04
1993	1.18	1.16	1.17	1.19	1.23	1.22	1.20	1.23	1.22	1.21	1.21	1.19	1.23	1.20	1.22
1994	1.19	1.24	1.22	1.22	1.28	1.26	1.18	1.19	1.24	1.14	1.14	1.15	1.28	1.20	1.20
1995	1.31	1.30	1.68	1.78	1.73	1.62	1.44	1.15	1.11	1.10	1.06	1.15	1.78	1.37	1.23
1996	1.19	1.98	2.28	2.13	2.02	1.75	1.36	1.19	1.16	1.12	1.16	1.22	2.28	1.55	1.24
1997	1.96	2.21	2.08	1.87	1.78	1.51	1.35	1.27	1.24	1.09	1.25	1.24	2.21	1.57	1.29
1998	1.33	2.53	2.77	2.41	2.44	2.44	1.93	1.48	1.40	1.34	1.35	1.31	2.77	1.89	1.60
1999	1.32	1.37	1.45	1.50	1.49	1.46	1.31	1.33	1.34	1.40	1.42	1.39	1.50	1.40	1.33
2000	1.44	1.54	1.92	1.91	2.01	1.62	1.39	1.34	1.44	1.58	1.57	1.58	2.01	1.61	1.39
2001	1.58	1.56	1.60	1.66	1.57	1.31	1.28	1.23	1.39	1.40	1.43	1.59	1.66	1.47	1.30
2002	1.73	1.70	1.68	1.65	1.40	1.38	1.32	1.35	1.35	1.31	1.58	1.75	1.75	1.52	1.34
2003	1.69	1.73	1.63	1.35	1.38	1.34	1.20	1.20	1.32	1.53	1.72	1.81	1.81	1.49	1.24
2004	1.69	1.86	1.60	1.47	1.50	1.47	1.44	1.49	1.44	1.43	1.42	1.42	1.86	1.52	1.46
2005	1.42	1.42	1.46	1.48	1.49	1.45	1.90	1.41	1.37	1.36	1.35	1.40	1.90	1.46	1.56
2006	1.82	1.72	2.09	2.67	2.51	2.17	1.65	1.52	1.42	1.41	1.42	1.37	2.67	1.82	1.53
2007	1.30	1.31	1.30	1.29	1.29	1.30	1.28	1.29	1.30	1.29	1.30	1.30	1.31	1.29	1.29
2008	1.34	1.36	1.34	1.27	1.30	1.23	1.21	1.23	1.23	1.28	1.33	1.31	1.36	1.29	1.22
2009	1.26	1.27	1.26	1.27	1.24	1.23	1.25	1.26	1.25	1.27	1.25	1.27	1.27	1.26	1.26
2010	1.27	1.23	1.28	1.30	1.28	1.24	1.26	1.29	1.29	1.30	1.26	1.26	1.30	1.27	1.28
2011	1.23	1.24	1.34	1.50	1.31	1.26	1.23	1.26	1.22	1.23	1.21	1.18	1.50	1.27	1.24
2012	1.18	1.16	1.20	1.20	1.18	1.17	1.17	1.19	1.16	1.17	1.18	1.20	1.20	1.18	1.17
2013	1.16	1.16	1.16	1.15	1.18	1.19	1.19	1.21	1.17	1.15	1.17	1.16	1.21	1.17	1.19
2014	1.14	1.17	1.16	1.15	1.16	1.15	1.13	1.14	1.15	1.13	1.13	1.16	1.17	1.15	1.14
2015	1.10	1.12	1.12	1.10	1.09	1.09	1.06	1.07	1.07	1.07	1.07	1.06	1.12	1.09	1.07
2016	1.09	1.05	1.12	1.11	1.12	1.13	1.12	1.16	1.15	1.13	1.12	1.13	1.16	1.12	1.14
2017	1.22	1.21	1.18	1.22	1.24	1.16	1.15	1.18	1.17	1.17	1.14	1.15	1.24	1.18	1.17
2018	1.13	1.13	1.18	1.16	1.17	1.17	1.17	1.16	1.13	1.13	1.13	1.11	1.18	1.15	1.15
2019	1.13	1.19	1.15	1.16	1.14	1.13	1.13	1.14	1.14	1.14	1.10	1.10	1.19	1.14	1.14
2020	1.05	1.05	1.09	1.13	1.08								1.13		

Appendix D MODEL RESULTS LOS PLAN VIEW FIGURES

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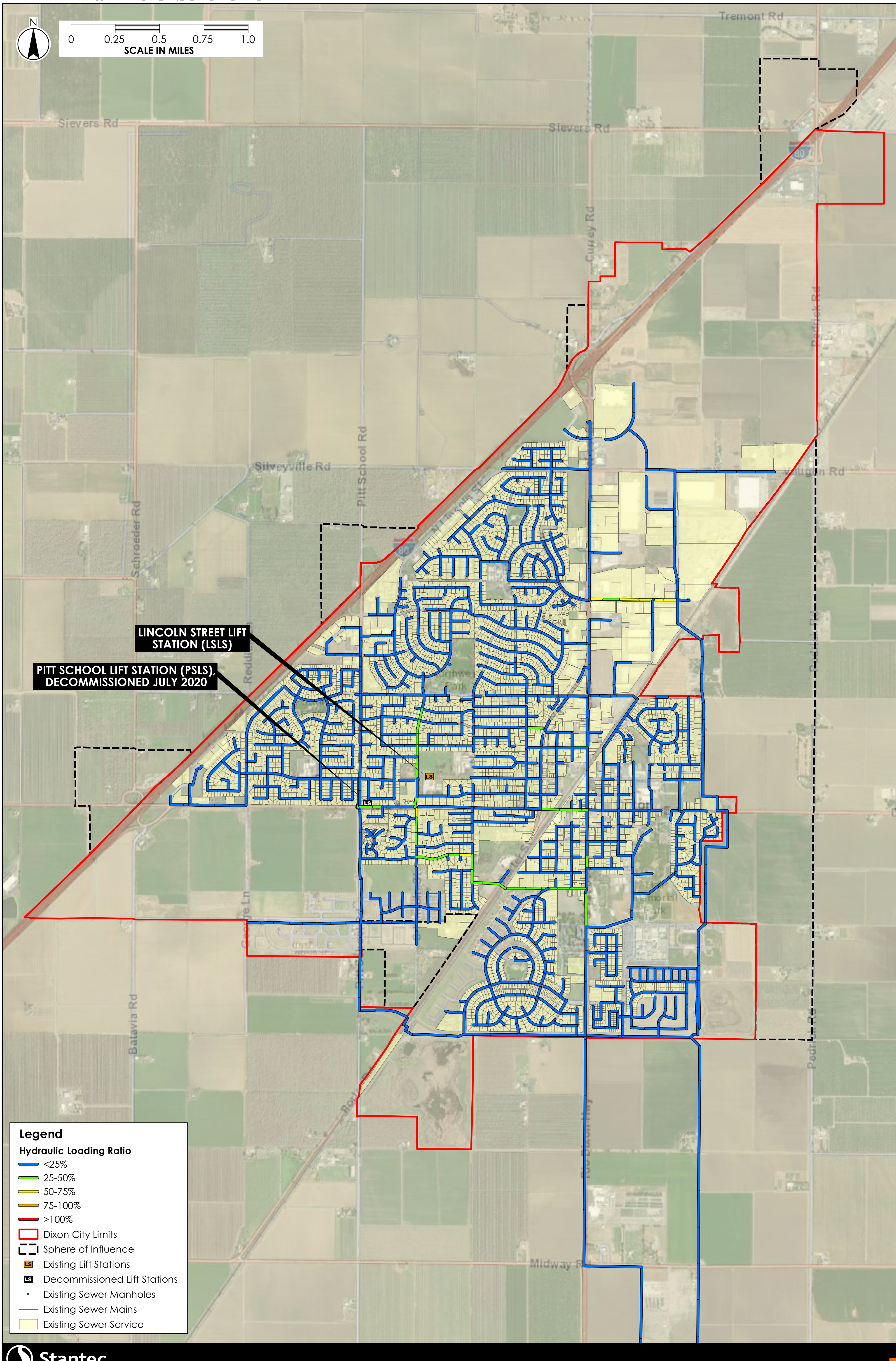
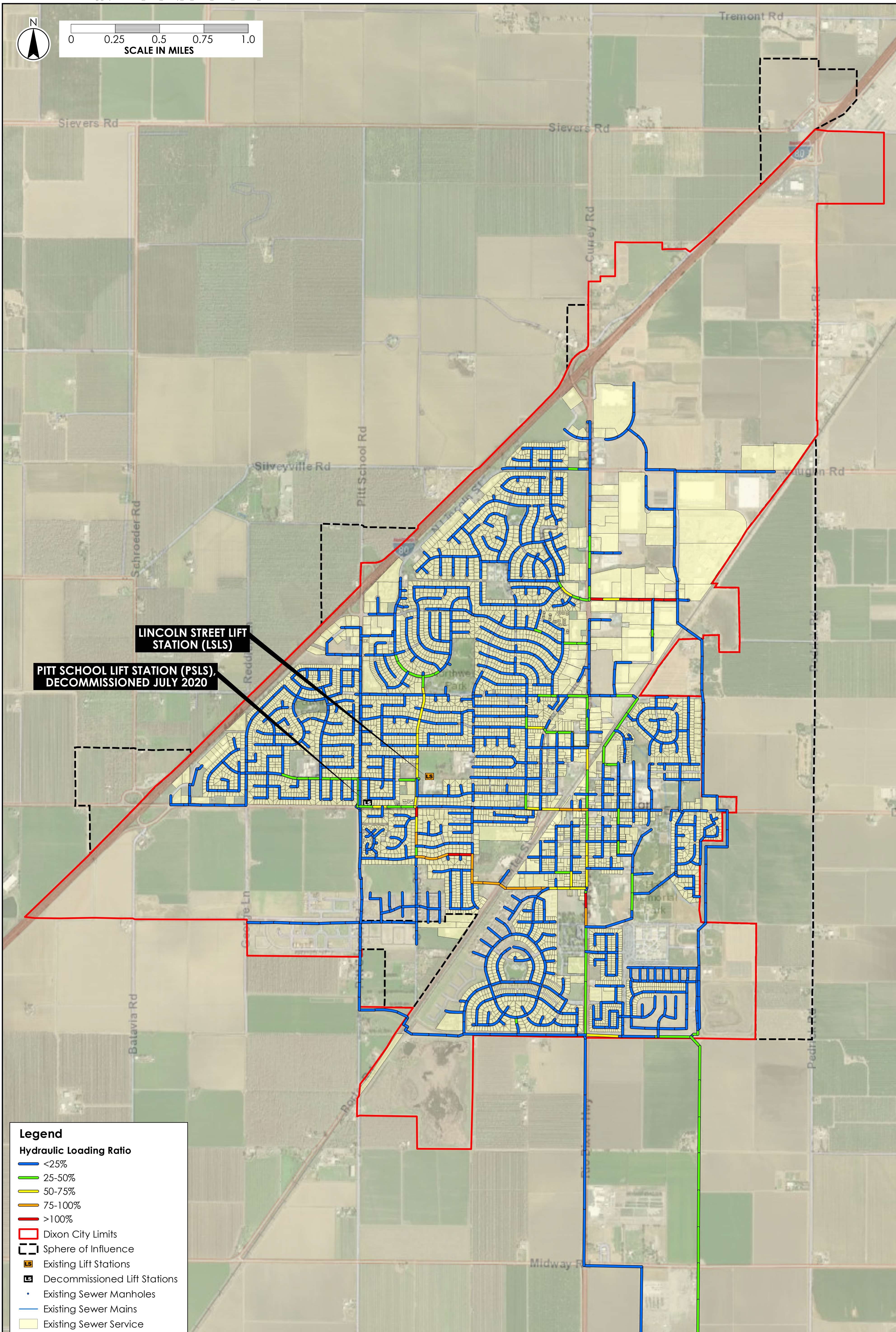


Stantec

City of Dixon
Sewer Collection System
Master Plan

Figure D-1
HLR Model Results: Scenario 1 – Existing Dry Weather Flow



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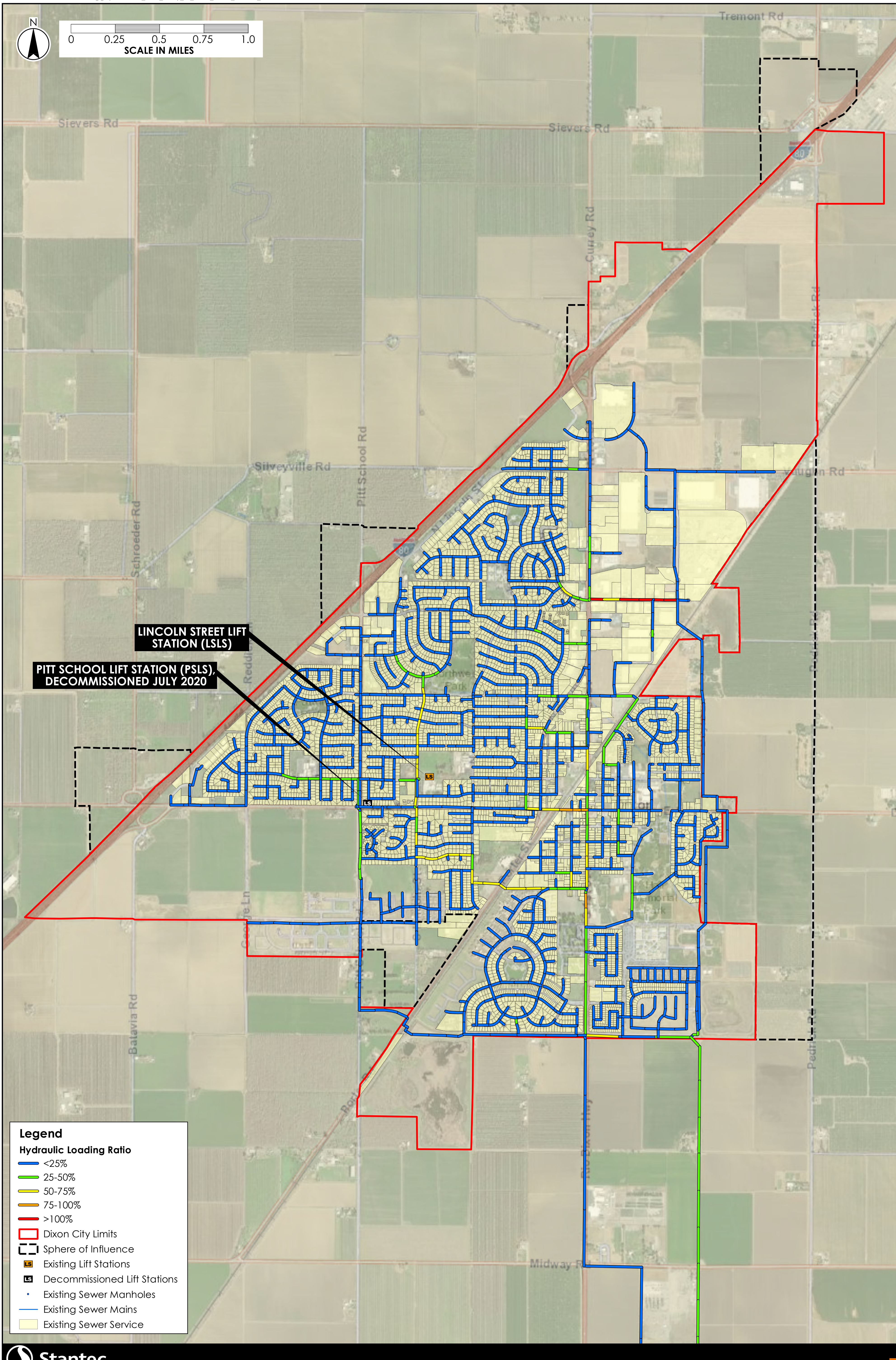
Sewer Collection System
Master Plan

Figure D-2
HLR Model Results: Scenario 2 – Existing Wet Weather Flow



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City of Dixon
Sewer Collection System
Master Plan

Figure D-3
HLR Model Results: Scenario 3 – Existing Wet Weather Flow (without PSLS)

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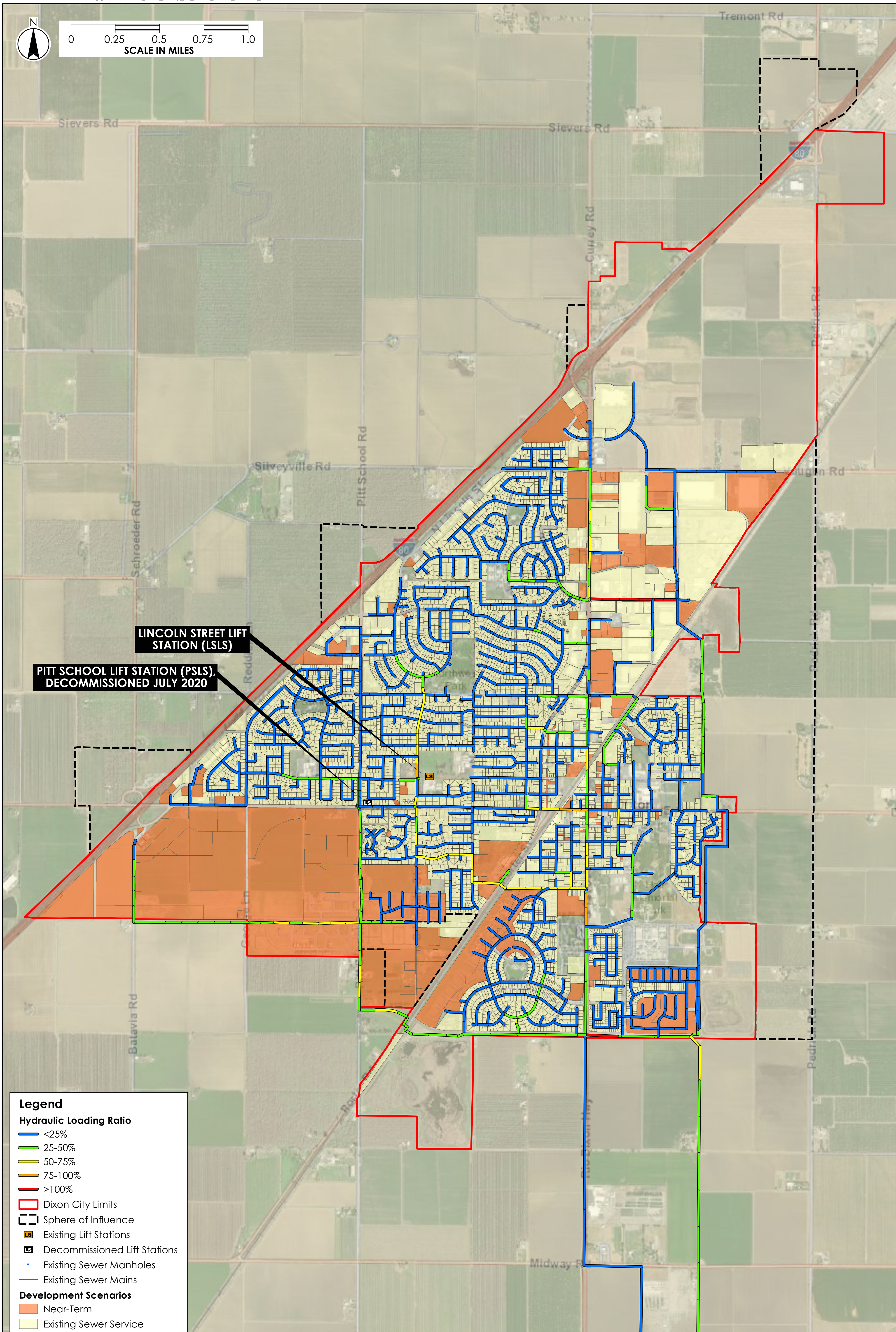

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Sewer Collection System
Master Plan

Figure D-4
HLR Model Results: Scenario 4 – Near-Term Development

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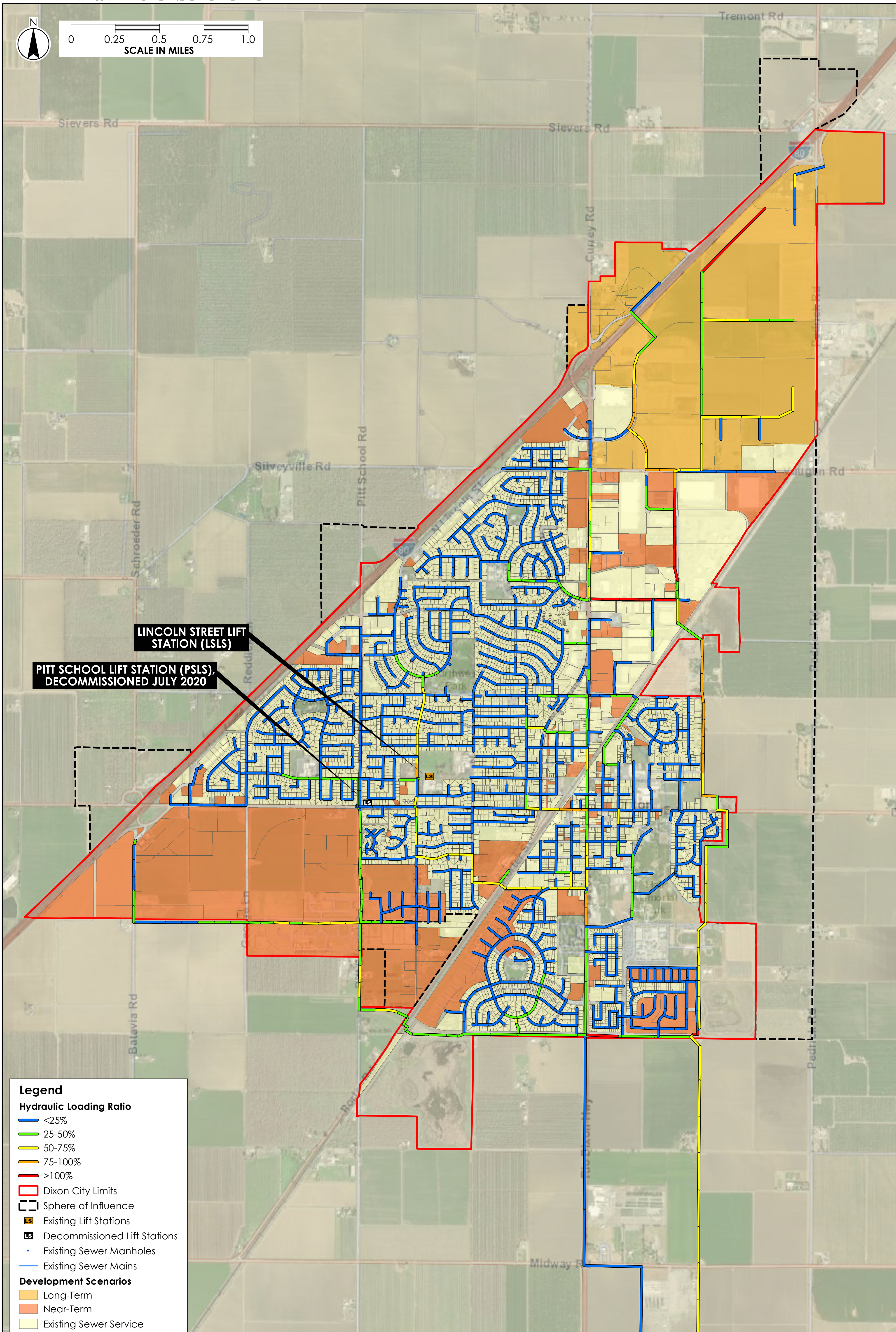

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City of Dixon
Sewer Collection System
Master Plan

Figure D-5
HLR Model Results: Scenario 5 – Long-Term Development

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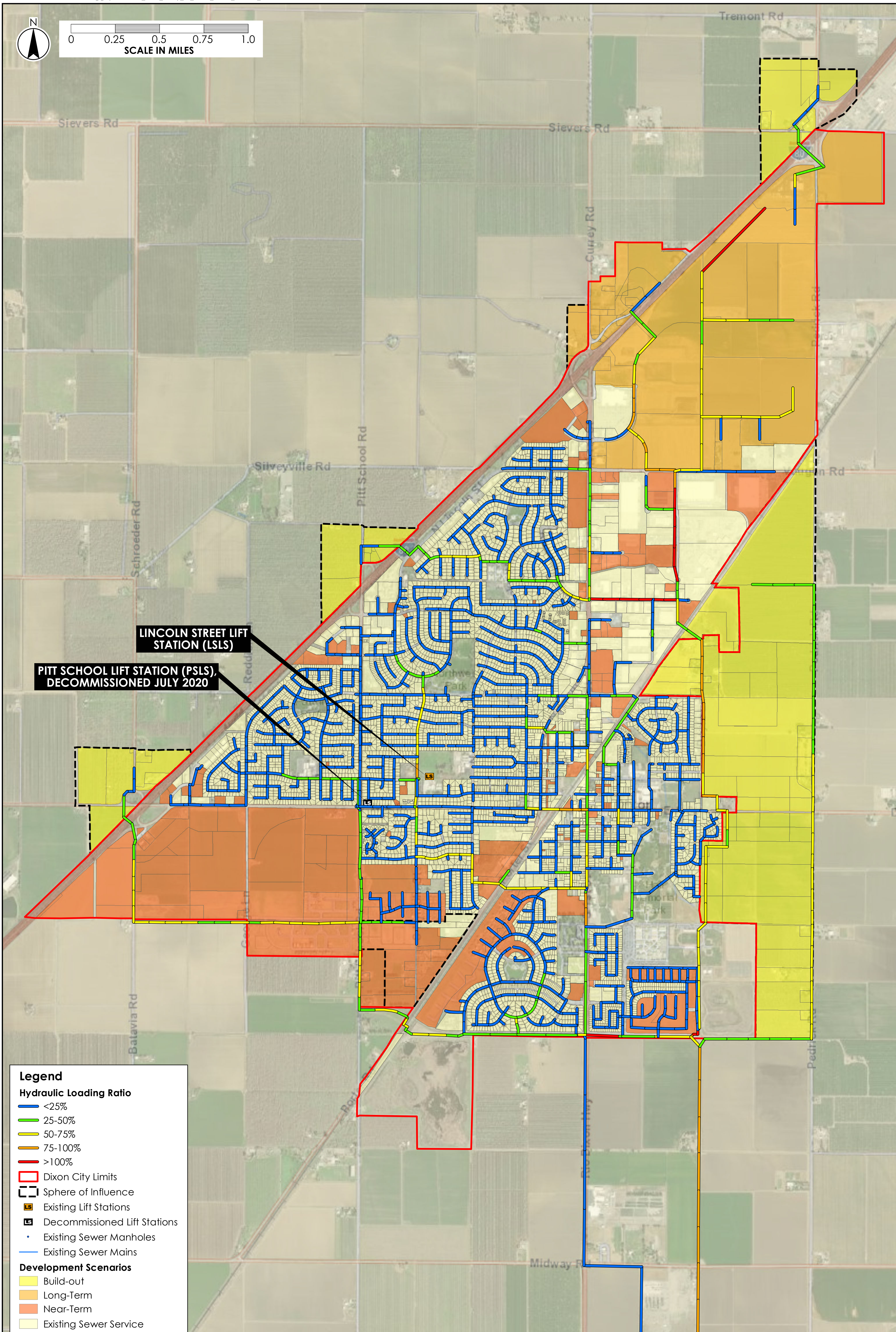

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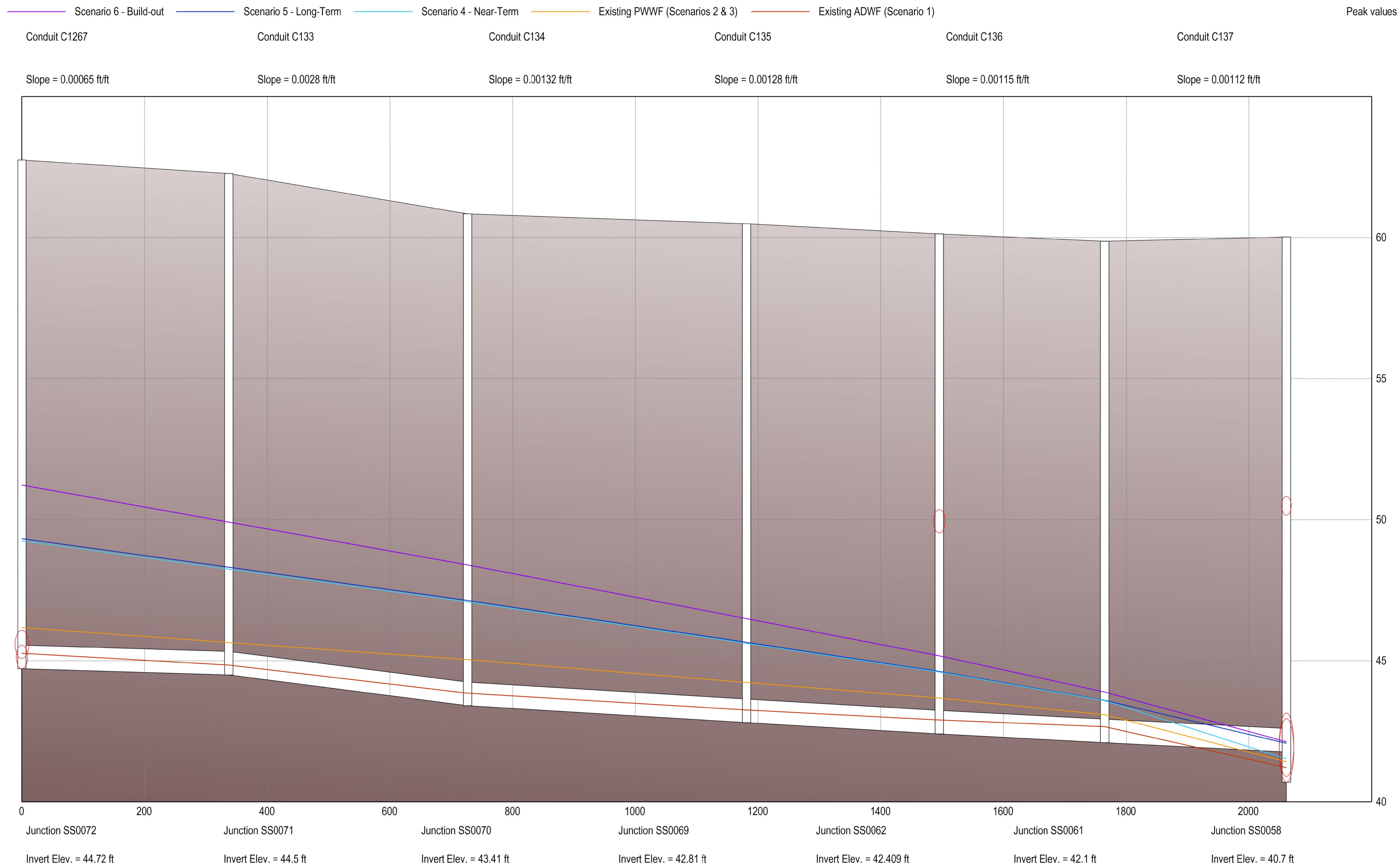
City of Dixon
Sewer Collection System
Master Plan

Figure D-6
HLR Model Results: Scenario 6 – Build-out Development

Appendix E MODEL RESULTS HGL PROFILE FIGURES



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City of Dixon
Sewer Collection System
Master Plan

Figure E-1

HGL PROFILE: CIP-E1, Industrial Way Trunk Sewer

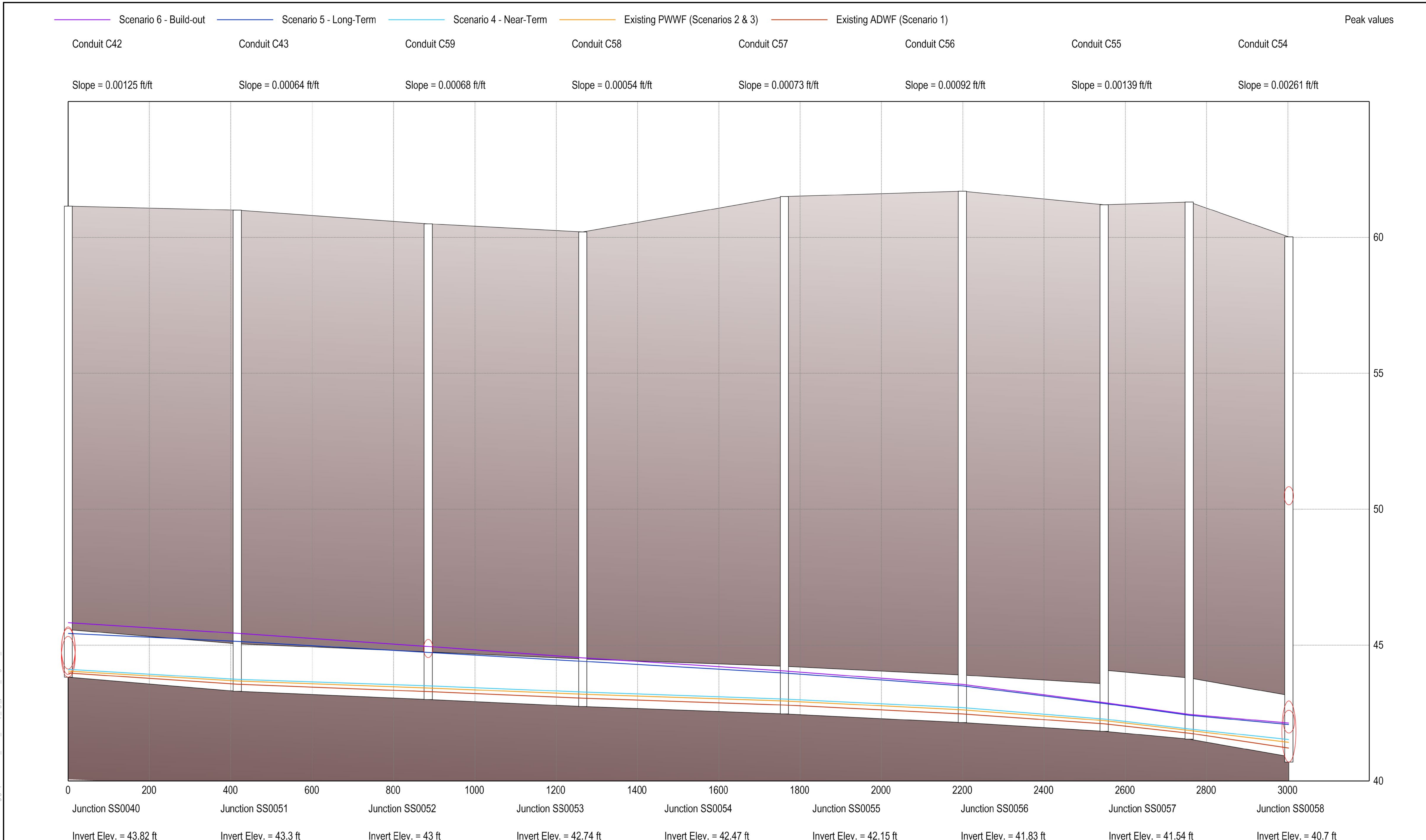
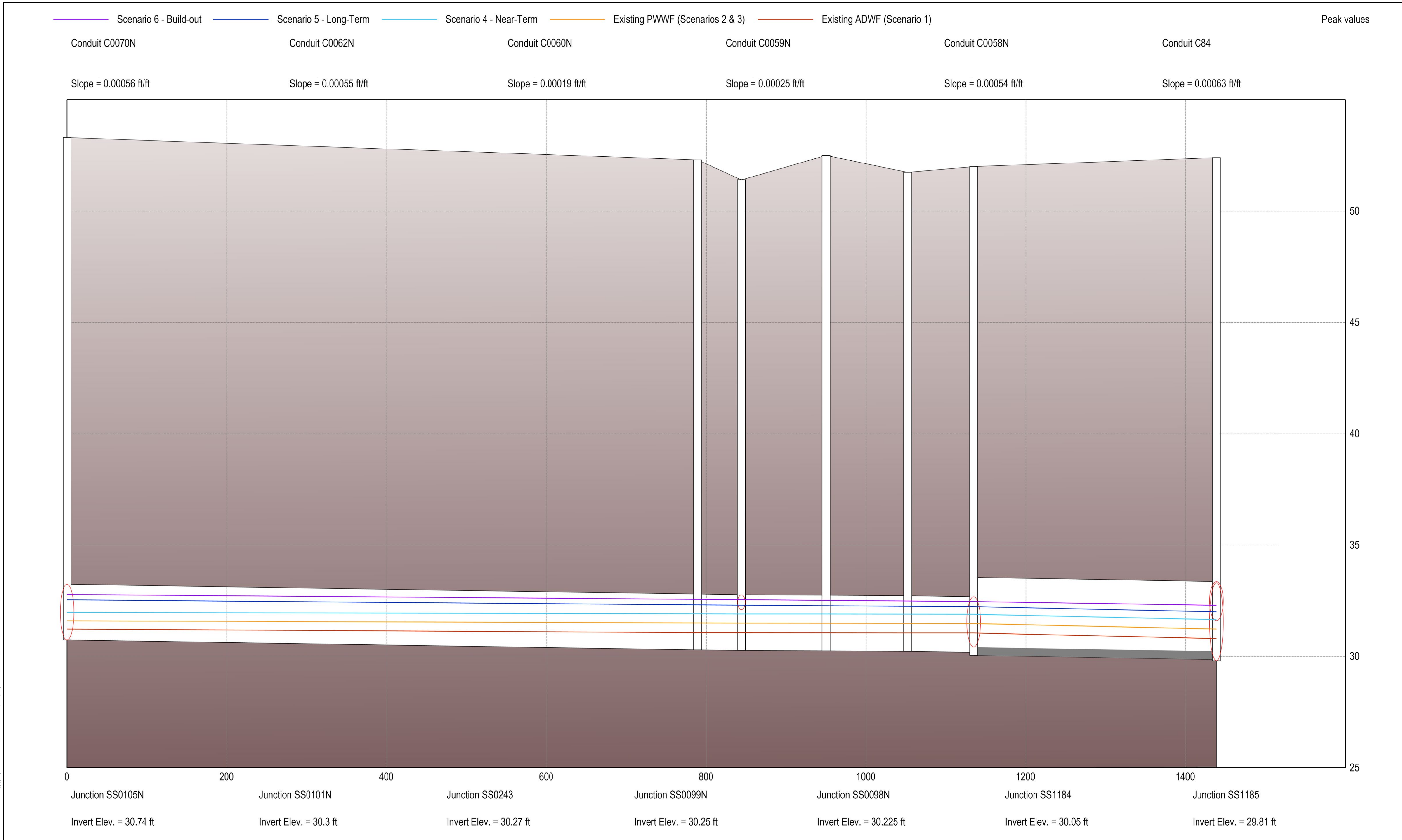


Figure E-2
HGL PROFILE: CIP-E2, Fitzgerald Dr. Trunk Sewer



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City of Dixon
Sewer Collection System
Master Plan

Figure E-3

HGL PROFILE: CIP-E3, North Dixon Trunk Sewer Bend

Appendix F NEXGEN ASSET MANAGEMENT TM

T E C H N I C A L   M E M O R A N D U M

City of Dixon
Topic: City of Dixon Sewer Master Plan – Condition Assessment

PREPARED FOR:	Steve Beck, P.E. / Stantec
PREPARED BY:	Rachel Schonwit, EIT/NEXGEN
REVIEWED BY:	Dan Rich, P.E./NEXGEN
DATE:	December 22, 2020

Section 1

Summary of Findings

A detailed inventory of the City's existing wastewater system and its condition was developed from the City's GIS database, City improvement plans, interviews with City Staff, and City inspection records.

The City's closed circuit television (CCTV) sewer inspection records were reviewed. The records indicate 39 of the City's 1,464 sewer pipeline assets have critical (poor) condition scores due to structural and O&M defects. The O&M defects (14 pipes) were caused by roots, grease buildup, or blockages; the assets with O&M defects should be added to the hotspot maintenance lists for more frequent maintenance and monitoring for further defects. The structural defects (25 pipes) were caused by cracks, breaks, offsets, or holes in the pipe and need to be replaced. For the 25 sewer assets with structural defects, the condition scores were used to prioritize those replacements.

The City has also conducted CCTV inspections of the 27-inch sewer trunk constructed in the 1950s. The sewer is near the typical useful life of reinforced concrete pipe. While not reviewed as part of this analysis, the CCTV inspections of the 27-inch trunk were reported to show widespread deterioration of the pipe due to corrosion. Installation of a plastic liner system is recommended to extend the RCP's life. This trunk system is a high priority restoration project.

The City's Lincoln Street Sewer Lift Station has been in operation for over 35 years, lacks alarms and automation expected with stations of this size, shows signs of corrosion, and requires excessive maintenance and oversight from City crews. The station is a package station meaning it is comprised of a steel "can" with equipment inside and is not easily rehabilitated. As a result, the station is a high priority replacement project and should be replaced with a new station consistent with other City wastewater facilities that have been designed for a 50+ year life.

Approximately $6.5 million of sewer and pump station replacement and rehabilitation projects have been identified as part this evaluation. The have been generally grouped into sewer replacements ($820,000), installation of a lining system in the 27-inch trunk ($4.5 million), and replacement of the Lincoln Street Sewer Lift Station ($1.3 million). It is recommended to complete these projects over the next five years.

An estimate of annual funding needs for sewer replacement was developed based on CCTV scores and the sewer's age and assumed life. The projection was completed over the next 25 years to capture larger projects such as the City's 42-inch trunk sewer. Those projects are total about $20 million (expressed in current dollars) over the 25 year period.

Section 2

Wastewater System Asset Overview

A detailed inventory of the City's existing wastewater system and its condition was developed from the City's GIS database, City improvement plans, interviews with City Staff, and City inspection records.

The City's wastewater system asset inventory resides in within a layer on the City's GIS database. The GIS database include manhole numbers locations, rim elevations, pipe invert elevations, sewer numbers, sewer diameter, and pipe material. This information was then used to develop other information used in the analysis including pipeline depths and slopes.

Other information developed and organized as part of this analysis include:

(1) **Asset install dates and assumed useful life**. Installation dates were taken from improvement plans and in some cases approximated after discussions with City Staff. Assumed pipeline life, termed the asset's useful life, were developed for each pipe material. Sewer pump station life was based on typical industry values and observed station condition.

(2) **Asset replacement costs** were based on recent construction bids for pipeline and replacement, and new pump station construction.

2.1 SEWER ASSETS

A sewer map generated by the City's GIS is shown in Figure 1. The sewers and lift station are described below.

2.1.1 Sewers

The City has sewer lines as old as from the 1950s. The City has also performed several sewer improvement projects including replacing pipe along Vaughn Rd and Lincoln Hwy in 2018

The City's sewers are comprised of both reinforced concrete pipe (RCP) and vitrified clay pipe (VCP). The larger 27-inch and 42-inch trunk lines to the wastewater treatment plant are RCP. The City of Dixon Standard Specifications require new sewers to be VCP.

The industry recommendation for the useful life of a sewer line depends on the pipe material and environmental factors. VCP is typically a more corrosion-resistant material than RCP and generally has a longer useful life. For RCP, the industry standard useful life is estimated at 60 years. For VCP, the standard useful life ranges from 60-120 years depending on manufacturer, quality of installation, pipe depth, and flow velocities. For this analysis, the useful life of VCP was assumed at 90 years. City staff routinely perform CCTV inspections to better understand the pipe's actual condition.

Sewer replacement costs are based on recent construction data in the Sacramento area and are tabulated in Table 1. The costs are shown for different pipe sizes and pipe depths and include construction and allowances for design, construction management, and contingencies. The total asset replacement cost was calculated using the cost per foot and the pipe length and depth GIS attribute

Table 1
Sewer Line Replacement Costs Used in this Analysis (a) (b) (c)

Pipe Diameter (in)	Depth (ft)										
	8	10	12	14	16	18	20	22	24	26	28
6	$83	$90	$95	$102	$110	$122	$137	$155	$180	---	---
8	$100	$111	$118	$126	$136	$150	$168	$191	$221	---	---
10	$121	$138	$146	$155	$167	$186	$208	$237	$274	---	---
12	$145	$161	$171	$182	$197	$218	$245	$278	$323	$373	$442
15	$180	$200	$210	$223	$239	$260	$288	$321	$363	$415	$488
18	---	$243	$253	$267	$284	$308	$335	$371	$414	$466	$540
21	---	$293	$305	$318	$337	$361	$391	$428	$473	$525	$601
24	---	$346	$363	$378	$397	$424	$456	$494	$538	$591	$669
27	---	$366	$380	$399	$423	$456	$494	$540	$594	$651	---
30	---	$410	$425	$446	$472	$504	$543	$592	$648	$708	---
36	---	$449	$466	$488	$516	$549	$591	$642	$702	$768	---
42	---	$596	$619	$646	$680	$721	$777	$844	$922	---	---
48	---	$655	$655	$709	$747	$795	$853	$927	$1,010	---	---
54	---	$759	$759	$817	$856	$904	$964	$1,041	$1,132	---	---

(a) This table represents the cost per foot for sewer pipe replacement. The cost includes trenching, removal of existing pipe, new pipe, subgrade prep, pavement restoration, and labor.
(b) These costs include a 20% contingency and an additional 15% for admin and design costs
(c) VCP (24" or smaller), RCP (greater than 24")

2.1.2 Sewer Lift Station

At the time of data collection, the City operated the Lincoln and Pitt School Sewer Lift Stations; however, the Pitt School Lift Station was recently decommissioned and has been omitted from this analysis. The Lincoln Street Lift Station is a duplex station with separate wet-well and dry-pit enclosed in a steel can. The lift station was originally manufactured as a package system by the Smith and Loveless Company and was installed in 1985. In 2004 the City installed the Smith and Loveless "Xpeller" to reduce the extent of pump plugging.

The station has a rated pumping capacity of 550 gallons per minute (gpm). The station consists of two pumps that alternate as lead or lag, isolation valves, and an above-ground control panel. The site does not have a dedicated backup generator during power outages, but has a manual transfer switch for connection to a portable generator. The station lacks remote monitoring capabilities aside from a 4-channel autodialer system that calls out during pump failure.

The lift station replacement cost was calculated based on recent (Summer of 2020) construction bids for a rehabilitation and upgrades for similar size and type of sewer lift stations at the City of West Sacramento. That project scope included a new wet well, new submersible pumps, new control panels, new SCADA system, a valve vault, and a backup generator. The lift station replacement cost is estimated at $1.3 million and includes allowances for design and administration and contingencies.

Table 2
Pump Station Replacement Cost Estimate

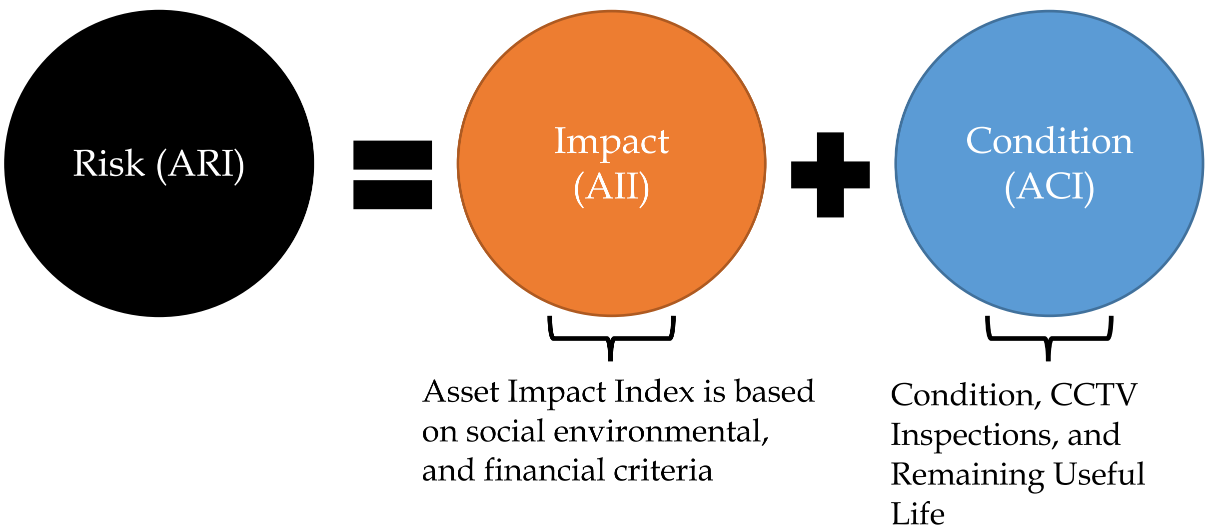
Item	*Description*	***Total***	
1	Mobilization and Demobilization:	$	30,000
2	Demolition:	$	25,000
3	Sitework and Site Grading:	$	70,000
4	Bypass Pumping:	$	50,000
5	Sheeting and Shoring:	$	15,000
6	Lift Station Construction:	$	320,000
7	Electrical and Controls:	$	340,000
8	Operator/Maintenance Staff Training and Testing:	$	10,000
Total Construction Costs		**$**	**860,000**
Design Services (15%)		$	129,000
Construction Management Services (10%)		$	86,000
Contingency (20%)		$	172,000
Total Project Costs (Rounded)		**$**	**1,300,000**

Figure 1
Wastewater System Map

Section 3

Sewer Planning Criteria

As part of this sewer system master planning effort, the City is interested in developing a prioritized capital improvement plan (CIP) that takes into account the asset's age, condition, as well as the consequence of its eventual failure. The consequence of failure, or "impact", recognizes that more critical assets should have a higher priority in the CIP. This criticality can be defined by environmental criteria (for instance, the proximity of the sewer to a creek), financial criteria (for instance, if a specific pipe is more expensive to repair), and social criteria (for instance, if failure results in impacts to local businesses). The Asset's Risk Index (ARI) is used to connect the impact of failure, termed the "Asset Impact Index" or AII, and probability of failure based on the assets condition, termed "Asset Condition Index" or (ACI). As part of this analysis, every asset was assigned a unique impact score and probability score.



3.1 Asset Impact Index (AII)

The Asset Impact Index (AII) describes the impact or consequence of asset failure. The AII ranges from 1 (low consequence) to 10 (extremely high consequence). Consequences include fines, property damage, traffic delays, public reputation, health safety, etc. Table 3 displays a detailed description of each value on the AII range.

Every asset was assigned an AII based on the detailed definitions in Table 3.

- Lift Stations: 10
- Sewer Trunk Lines: 10
- Residential Sewer Lines: 5

- Commercial/Downtown Area Sewer Lines: 7
- Interceptor Sewer Lines: 8

Table 3
Asset Impact Index Definitions

All	Detailed Definitions
1	No operation and service interruptions. No impact on environment and regulatory compliance. No economic and financial impact.
2	Minimal operation interruptions but no service interruptions to customers. Minimal impact on environment and regulatory compliance. Trivial loss of economic and financial revenue from productions or service interruptions.
3	Minor operation interruptions but no service interruptions to customers. Minor impact on environment and no violation of regulatory compliance. Minor loss of economic and financial revenue from productions or service interruptions.
4	Limited operation interruptions with possible service interruptions to customers. Limited impact on environment and possible violation of regulatory compliance. Limited loss of economic and financial revenue from productions or service interruptions.
5	Moderate operation interruptions and minor service interruptions to customers. Moderate impact on environment and minor violation of regulatory compliance. Moderate loss of economic and financial revenue from productions or service interruptions.
6	Notable operation interruptions and service interruptions to limited customers. Notable impact on environment and minor violation of regulatory compliance. Notable loss of economic and financial revenue from productions or service interruptions.
7	Considerable operation interruptions, service interruptions to limited customers and potential public safety. Considerable impact on environment and violation of regulatory compliance with fines. Considerable loss of economic and financial revenue from productions or service interruptions.
8	Major operations interruptions, service interruptions to medium customers and inherent public safety. Major impact on environment and unavoidable violation of regulatory compliance with large fines. Major loss of economic and financial revenue from productions or service interruptions.
9	Significant operations interruptions, service interruptions to large number of customers and imminent public safety. Significant environmental impact and imminent violation of regulatory compliance with significant fines. Significant loss of revenue from productions or service interruptions.
10	Extended operations interruptions, service interruptions to large number of customers and serious public safety. Catastrophic environmental impact and monumental violation of regulatory compliance with extensive fines. Extreme loss of revenue from productions or service interruptions.

3.2 ASSET CONDITION INDEX (ACI)

The Asset Condition Index (ACI), describes the likelihood of asset failure based on the condition of the asset. The ACI ranges from 1 (not likely) to 10 (extremely likely). The useful life remaining and the CCTV inspection scores were used to calculate most of the ACI values. Table 4 displays an overview definition of each value on the ACI range.

Table 4
Asset Condition Index Definitions

Index	Definition
1	Extremely low probability of failure
2	Very low probability of failure
3	Low probability of failure
4	Low-intermediate probability of failure
5	Intermediate probability of failure
6	Moderate probability of failure
7	Moderate-high probability of failure
8	High probability of failure
9	Very high probability of failure
10	Extremely high probability of failure

3.3 ASSET RISK INDEX (ARI)

The Asset Risk Index (ARI) is the risk score describing the danger or loss associated with the failure of each asset. The ARI ranges from 1 (no risk) to 100 (extremely high risk). ARI is the product of the condition (ACI) and the impact of failure (AII). Table 5 displays an overview definition of the ARI values.

Table 5
Asset Risk Index Definitions

Risk					Definition		
1	<	ARI	<	10	Extremely Low Risk	→	No Activity
11	<	ARI	<	20	Very Low Risk	→	No Activity
21	<	ARI	<	30	Low Risk	→	Sample monitoring
31	<	ARI	<	40	Low Intermediate Risk	→	Routine monitoring
41	<	ARI	<	50	Intermediate Risk	→	Routine monitoring
51	<	ARI	<	60	Moderate Risk	→	Aggressive monitoring
61	<	ARI	<	70	Moderate High Risk	→	Aggressive monitoring
71	<	ARI	<	80	High Risk	→	Plan Work
81	<	ARI	<	90	Very High Risk	→	Immediate Work
91	<	ARI	<	100	Extremely High Risk	→	Immediate Work

Section 4

CCTV and Condition Inspections

Closed circuit television (CCTV) technology is used for internal inspection of sewer lines to identify defects in the sewer line. A structural and service condition report is created as the live footage is being viewed, and a score is given for the overall pipe condition using the PACP (Pipeline Assessment and Certification Program) standards for coding defects. The PACP scoring gives each pipe a value from 0 (great condition) to 5 (major defects).

4.1 CCTV INSPECTION DATA

Approximately four years of the City's historical CCTV inspection data (Jan 2017-Aug 2020) was reviewed for assessment of the overall sewer condition. Table 6 displays a summary of the CCTV data.

Table 6
Summary of CCTV Data

	Total Sewer Line Assets	Assets with Non-Zero PACP Scores (a)	Assets with Critical PACP Scores (b)
Number of Assets	1,464	102	39
Percent of System	100%	9.5%	2.7%

(a) If an asset's CCTV Inspection showed any non-zero number (1-5)
(b) Critical score was defined as a PACP score of 4 or greater

Assets with PACP scores below 4 are considered non-critical and should continue to be inspected over the upcoming years to monitor if additional defects arise. The assets with critical PACP scores were further analyzed to determine the types of defects, their severity, and whether the defects could be remedied with increased maintenance or structural repairs or replacement.

The 39 assets (pipes) with critical PACP scores were classified into two groups: structural defects or O&M defects. If an asset had both O&M and structural defects, it was put in the structural defect category. **Fourteen of the 39 assets had O&M defects, the remaining 25 critical assets were structural defects.** Table 5 includes the full list of assets with critical PACP scores, the PACP score, and the field notes that explain the reason for the score.

4.1.1 Structural Defects

Structural defects include cracks, breaks in the pipe or joints, voids visible, soil visible, or any combination of these issues. Examples of some structural defects found in the City's CCTV Data are shown in Figure 2.

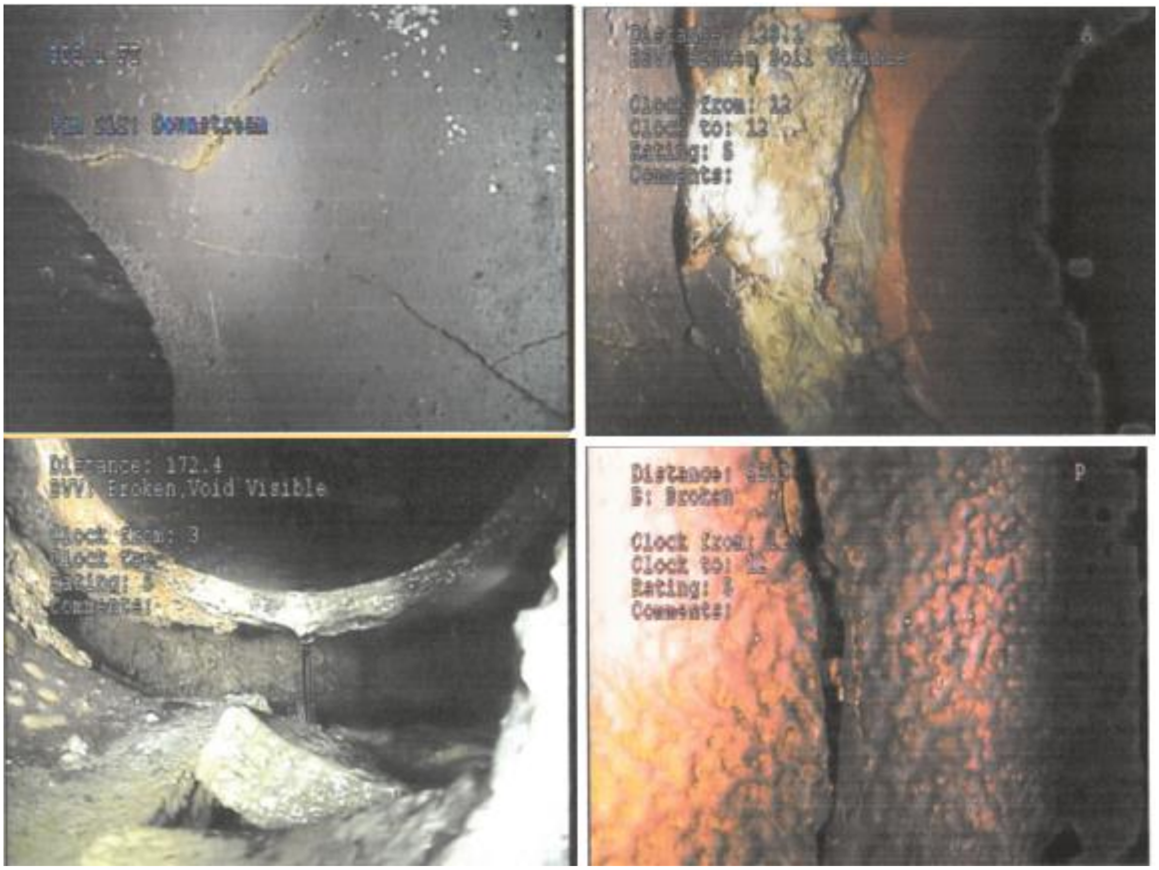


Figure 2
Typical Structural Defects from City's CCTV Inspections

The severity of these structural defects varies depending on the size of the defect and how frequent defects occur on the same asset. For example, a pipe with minor cracks in one location would not be as severe and may just need monitoring for increased severity; whereas a pipe with breaks and voids/soils visible all throughout the pipe would require full pipe replacement.

4.1.2 O&M Defects

O&M defects include roots in the pipe or joints, grease buildup, debris buildup, or any combination of these issues. Examples of some O&M defects found in the City's CCTV Data are shown in Figure 3.

Figure 3
Typical O&M Defects from City's CCTV Inspections

The majority of the City's O&M defects are caused by roots and/or grease. The City has already developed a list of assets with root issues and hotspot grease areas where they perform more frequent maintenance to reduce risk of asset failure. For example, the grease hotspot areas are jetted every 6 weeks to remove grease buildup.

Based on the list of 14 assets with critical O&M defects, it is recommended that the City continue to perform their current maintenance for the assets on the hotspot grease area list. It is also recommended that the City expand their list of assets that receive more frequent root control to include the assets highlighted in yellow in Table 7.

Table 7
Assets with Critical PACP Scores

Asset ID	PACP Score	Type of Defect	Defect Comments	Recommendation
SL-0584-0583	4	O&M	Roots in joints throughout pipe	Add to root issue list
SL-0541-0538	5	Structural	Broken pipe with voids visible	Replace pipe
SL-0500-0499	4	O&M	Roots	Already on City's root issue list
SL-0501-0500	4	O&M	Roots	Already on City's root issue list
SL-0611-0610	4	O&M	Roots	Already on City's root issue list
SL-0610-0609	4	O&M	Roots	Already on City's root issue list
SL-0608-0607	5	Structural	Soil visible	Replace pipe
SL-0504-0500	4	O&M	Roots in joint	Add to root issue list
SL-0488-0487	4	O&M	Roots in joint	Add to root issue list
SL-0515-0514	5	Structural	Cracks and broken pipe; soil visible	Replace pipe
SL-0461-0460	4	O&M	Roots in joint	Add to root issue list
SL-0267-0265	4.5	Structural	Break voids visible	Replace pipe within next 3-5 years
SL-0958-0956	4	O&M	Grease buildup	Already on City's grease hotspot
SL-0960-0956	4	O&M	Grease buildup	Already on City's grease hotspot
SL-1007-1006	5	Structural	Broken pipe; soil and voids visible	Replace pipe
SL-1004-1002	5	Structural	Voids visible	Replace pipe
SL-1003-1002	4.5	Structural	Broken pipe, multiple cracks	Replace pipe within next 3-5 years
SL-0694-0693	4	O&M	Multiple joints with roots	Add to root issue list
SL-1059-1058	5	Structural	Broken Pipe	Replace pipe
SL-1072-1070	5	Structural	Soil visible, broken pipe	Replace pipe
SL-1034-1028	4.5	Structural	Multiple longitudinal cracks	Replace pipe within next 3-5 years
SL-1035-1034	5	Structural	Broken voids visible	Replace pipe
SL-1038-1037	5	Structural	Soils visible	Replace pipe
SL-1033-1031	5	Structural	Voids and soil visible, broken pipe, multiple cracks	Replace pipe
SL-1032-1031	5	Structural	Broken pipe, soils and voids visible	Replace pipe
SL-1031-1028	5	Structural	Broken pipe	Replace pipe
SL-1001-1000	5	Structural	Multiple cracks, broken pipe	Replace pipe
SL-0926-0919A	4.5	Structural	Circumferential cracks	Replace pipe within next 3-5 years
SL-0961-0960	5	Structural	Broken pipe	Replace pipe
SL-1023-1201	4.428	Structural	Breaks in pipe, grease build up	Replace pipe within next 3-5 years
SL-1022-1021	4	Structural	Breaks in pipe, grease build up	Replace pipe within next 3-5 years
SL-1021-1020	4	O&M	Grease buildup	Already on City's grease hotspot
SL-0947-0943	5	Structural	Multiple breaks caused by roots	Replace pipe within next 3-5 years
SL-1006-1004	5	Structural	Breaks in pipe	Replace pipe
SL-1035-1035B	4	O&M	Factory tap intrusion, roots	Accelerated monitoring for future structural defects
SL-0716-0715	4.5	O&M	Roots breaking through pipe, multiple cracks	Replace pipe within next 3-5 years
SL-1006-1001	4	Structural	Cracks and a broken spot of pipe	Replace pipe within next 3-5 years
SL-1010-1006	5	Structural	Roots at joints, broken pipe, multiple circumferential cracks	Replace pipe
SL-1015-1010	4	Structural	Multiple cracks	Replace pipe within next 3-5 years

4.2 SEWER CONDITION SCORES

Asset condition scores were assigned to each sewer based on the CCTV PACP scores and the useful life remaining for each asset. The Asset Condition Index (ACI) ranges from 1 (as good as brand new) to 10 (needs to be replaced immediately). For the 39 critical assets described in Table 5, the PACP scores were changed from a 1-5 to a 1- 10 score, i.e. a PACP score of 4 is an 8 on the ACI.

The condition score of the City's sewers primarily consists of remaining useful life and CCTV inspection data. For VCP, published useful life ranges from 60-120 years (90 is used as a typcial value in this study); due to this wide range, the CCTV assessment data was used to determine a more accurate condition score. Based on the CCTV data, the majority of the VCP sewer pipelines (97%) have an acceptable condition score below a 6 which indicates a low to intermediate risk of failure.

The 27 inch and 44-inch trunk sewers are both constructed of RCP. The standard useful life of RCP is 65 years; after which RCP tends to corrode and leak. The 42-inch RCP trunk line was built in the 1990s and still has over half its useful life remaining and appears to be operating under good condition. As a result, it has been assigned an ACI of 5. The 27-inch RCP trunk was constructed in the 1950s and is technically beyond its useful life. The CCTV inspections of this trunk were conducted by the City over the past few years. While not reviewed as part of this analysis, City staff have reported that the concrete showed signed of widespread deterioration. As a result, the 27-inch trunk was assigned an ACI of 10. It is important to note that for these larger RCP trunks, the higher ACI score would not necessarily trigger the pipeline replacement. Lining of sewer trunks with plastic liner system is a common method to extend the life of the pipe and is less expensive and impactful than replacement.

4.3 LINCOLN STREET LIFT STATION CONDITION SCORE

The Lincoln Street Lift Station is a duplex station with separate wet-well and dry-pit enclosed in a steel can. The lift station was originally manufactured as a package system by the Smith and Loveless Company (S&L). The station was originally constructed in 1985. Figure 4 provides recent images of the dry well. As shown, the can is showing signs of deterioration due to corrosive conditions. The integrity of the steel can cannot easily be ascertained but these stations were common 30- 40 years ago and are generally considered to be at the end of their useful life. For instance, this year the City of West Sacramento replaced two of their S&L stations due to observed corrosion, excessive maintenance, and concerns over station reliability.

For these reasons, the Lincoln Street Lift Station has been assigned an ACI of 10.

A heat map of the assets with ACI values between 7-10 are shown in Figure 5 (green represents assets with 7-8 ACI and red represents assets with 9-10 ACI).

Figure 4
Lincoln Street Sewer Lift Station

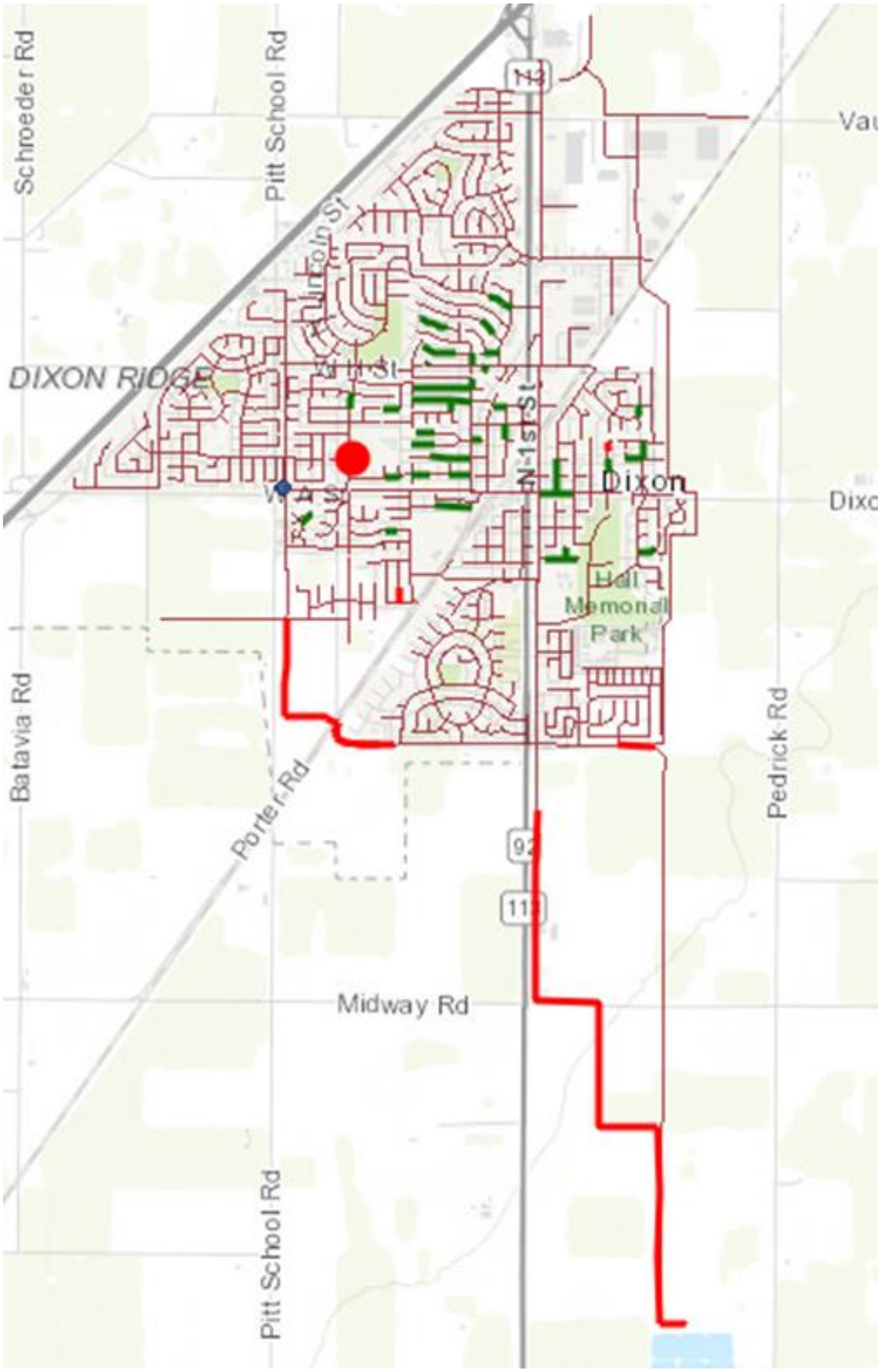


Figure 5
Asset Condition Heat Map

Section 5

Capital Improvement Plan

As mentioned previously, all the City's sewer assets are ranked by their specific Asset Risk Index (ARI) score. ARI is calculated by multiplying the probability of failure based on the asset's condition (ACI) by the impact of failure (AII). Both ACI and AII are on a 1-10 scale, so an ARI of 100 would be prioritized for immediate replacement.

5.1 Near-Term Improvement Projects

The Lincoln Street Sewer Lift Station, sewers with structural defects identified with CCTV inspections, and the 27-inch RCP trunk have been identified to have the highest ARI scores.

Table 8
Highest Risk Sewer Assets

Asset No.	Description	AII	ACI	ARI	Approx. Depth (feet)	Length (feet)	Cost
Lincoln Street Sewer Lift Station	550 gpm Capacity Sewer Lift Station	10	10	100	N/A	N/A	1,250,000
Sewers with Structural Defects							
SL-0541-0538	8-inch VCP Sewer	5	10	50	5	345	34,550
SL-0608-0607	8-inch VCP Sewer	5	10	50	5	241.	24,200
SL-0515-0514	6-inch VCP Sewer	5	10	50	4	206.	17,140
SL-0267-0265	8-inch VCP Sewer	5	9	45	5	220	22,089
SL-1007-1006	6-inch VCP Sewer	5	10	50	8	623	51,837
SL-1004-1002	6-inch VCP Sewer	5	10	50	6	358	29,823
SL-1003-1002	6-inch VCP Sewer	5	9	45	6	363	32,719
SL-1059-1058	6-inch VCP Sewer	5	10	50	6	175	14,561
SL-1072-1070	6-inch VCP Sewer	5	10	50	6	266	22,110
SL-1034-1028	8-inch VCP Sewer	5	9	45	7	359	26,421
SL-1035-1034	6-inch VCP Sewer	5	10	50	8	375	31,203
SL-1038-1037	8-inch VCP Sewer	5	10	50	7	360	36,119
SL-1033-1031	6-inch VCP Sewer	5	10	50	4	179	14,942

SL-1032-1031	6-inch VCP Sewer	5	10	50	4	200	16,655
SL-1031-1028	6-inch VCP Sewer	5	10	50	5	358	29,803
SL-1001-1000	12-inch VCP Sewer	7	10	70	13	364	55,596
SL-0926-0919A	10-inch VCP Sewer	7	9	63	10	198	27,301
SL-0961-0960	8-inch VCP Sewer	5	10	50	6	360	24,153
SL-1023-1201	6-inch VCP Sewer	5	9	45	4	151	12,612
SL-1022-1021	6-inch VCP Sewer	5	8	40	4	186	15,495
SL-0947-0943	10-inch VCP Sewer	7	10	70	10	317	43,618
SL-1006-1004	6-inch VCP Sewer	5	10	50	6	235	19,542
SL-1035-1035B	6-inch VCP Sewer	5	8	40	6	187	15,548
SL-0716-0715	8-inch VCP Sewer	5	9	45	6	322	32,279
SL-1006-1001	12-inch VCP Sewer	7	8	56	10	359	56,174
SL-1010-1006	12-inch VCP Sewer	7	10	70	9	359	58,021
SL-1015-1010	12-inch VCP Sewer	7	8	56	7	358	51,977
27 Inch Trunk Sewer	Line 27 inch RCP	10	10	100	18	22,350	4,470,000*
Total							6,546,500

*The cost to install a plastic lining system on the 27-inch RCP is estimated at $200/ft

It is recommended to phase the above projects over the next five years. A possible means to complete these projects is shown in Figure 6 below. Some projects will require more than one year to design and construct.

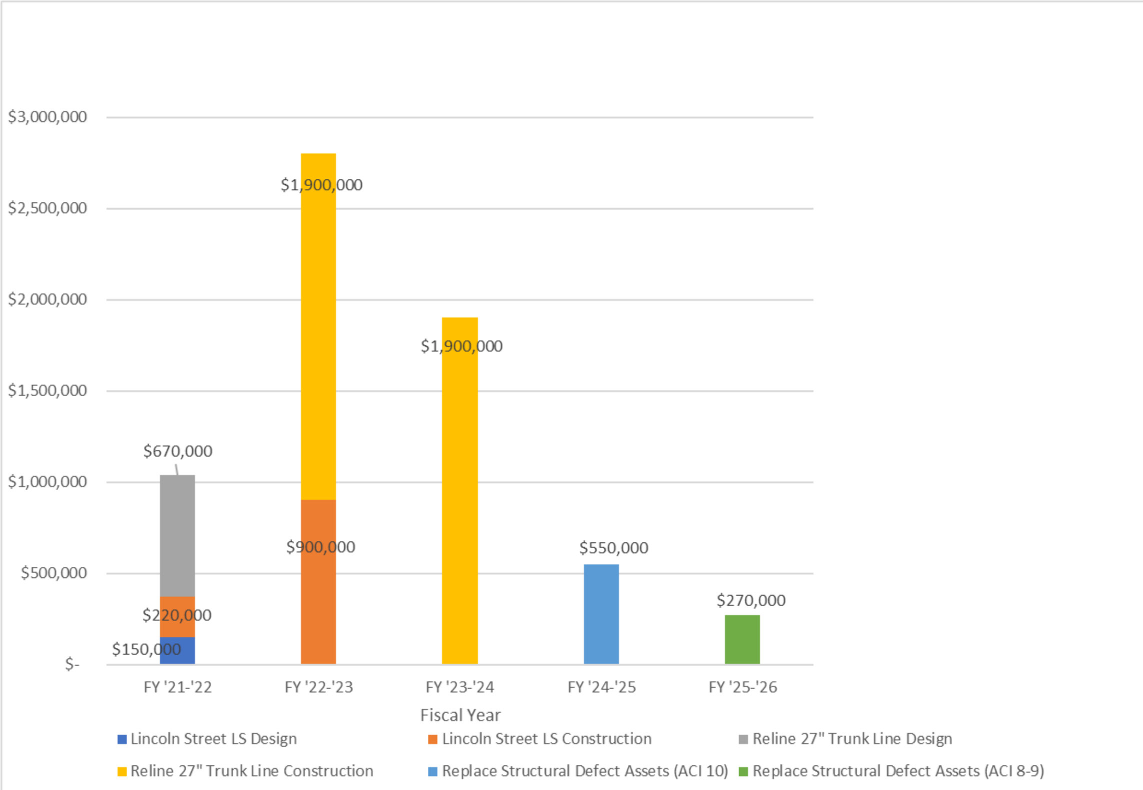


Figure 6
Five Year Capital Improvement Program

5.2 Long Term Replacement Funding Needs

As mentioned above, a majority of the City's 1,400 VCP sewers are in good condition and do not need to be included in CIP. VCP is a strong and corrosion resistant material and has a long expected life. However, as the sewers age, the City should expect to see additional replacement projects and the pipes with operational defects (currently 14 pipes) can become structural defects in the future. In addition, the City's 42-inch RCP trunk is about half way through its expected life before a significant rehabilitation would be needed. These pipes represent a majority of the value of City's sewer assets. As a result, it is important to continue and formalize the City's cleaning and inspection practices.

The assumed useful life of VCP (90 years) and RCP (60 years) was used to calculate a general cost for replacement of the sewers not included in the five-year plan. A 25-year horizon was selected to capture a reasonable grouping of some of the older VCP sewers and also includes rehabilitation of the 42-inch trunk. Total costs over the 25-year period are projected at approximately $20 million (expressed in current dollars) and are shown over time in Figure 7. The $20 million is compromised of the five-year plan depicted in Figure 6, replacement the older VCP sewers that are beyond the 90 year life, and assumed lining of the 42-inch trunk prior to 2045. Additional CCTV inspections will be needed in subsequent years to validate these projections.

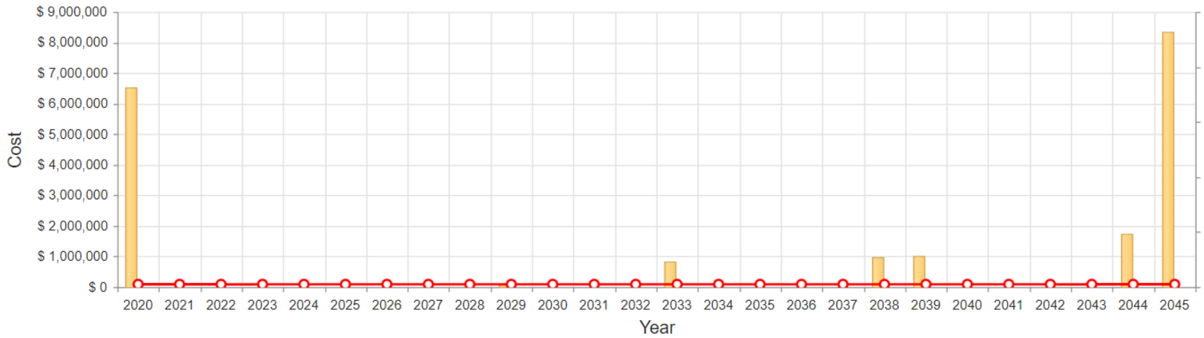


Figure 7
Long Term Funding Requirements for Sewer Replacement