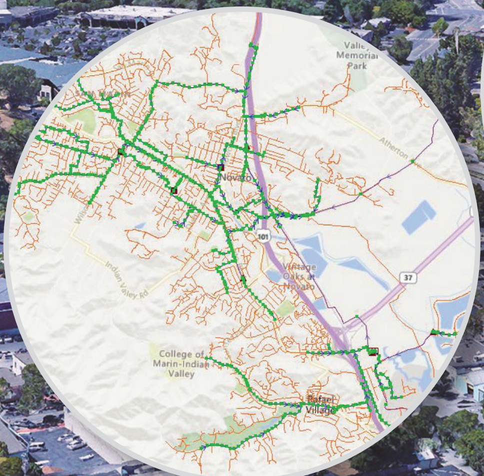
NOVATO SANITARY DISTRICT

FINAL REPORT

COLLECTION SYSTEM MASTER PLAN

October 2019



Prepared by

WOODARD & CURRAN

Novato Sanitary District

Wastewater Collection System Master Plan

Final Report

Prepared by:

WOODARD & CURRAN

October 2019

Table of Contents

Executive Summary 1
Background and Purpose of Master Plan 1
How the Capacity Assessment Was Prepared 2
How the Condition Assessment Was Prepared 9
Recommended Capital Improvement Program 13

Chapter 1 Introduction 1-1
1.1 Background and Study Objectives 1-1
1.2 Sanitary Sewer System 1-2
1.3 Scope of Study 1-6
1.4 Report Organization 1-7
Chapter 2 Basis of Flow Estimates 2-1
2.1 Wastewater Flow Components 2-1
2.2 Base Wastewater Flow 2-2
2.2.1 Existing Base Wastewater Flow 2-2
2.2.2 Future Base Wastewater Flow 2-2
2.2.3 Base Wastewater Flow Projections 2-4
2.2.4 Diurnal Curves 2-5
2.3 Groundwater Infiltration 2-6
2.4 Rainfall-Dependent I/I 2-6
Chapter 3 Hydraulic Model Development 3-1
3.1 Modeling Terminology 3-1
3.2 Delineation of Modeled System 3-1
3.3 Model Network Construction and Validation 3-2
3.4 Ignacio Transfer Pump Station Operation 3-4
3.5 Load Manholes and Sewer Subcatchments 3-5
3.6 Flow Monitoring Program 3-6
3.7 Development of Model Loads 3-10
3.8 Model Calibration 3-11
3.8.1 Dry Weather Calibration 3-11
3.8.2 Wet Weather Calibration 3-12
Chapter 4 Capacity Analysis 4-1
4.1 Design Event Criteria 4-1
4.1.1 Return Period 4-1
4.1.2 Duration 4-1
4.1.3 Rainfall Amount and Spatial Distribution 4-1
4.1.4 Rainfall Temporal Distribution 4-2
4.1.5 Storm Timing 4-3
4.1.6 Antecedent Conditions 4-3
4.1.7 Recommended Design Event for NSD 4-4
4.1.8 Future Conditions 4-4
4.2 Hydraulic Capacity Criteria 4-4
4.2.1 Capacity Deficiency Criteria 4-4
4.2.2 Design Criteria for New Sewer Facilities 4-5
4.3 Capacity Analysis Results and Preliminary Solutions 4-5
4.4 Recommended Capacity Improvement Projects 4-9
4.4.1 RDI/I Reduction Effects on Identified Capacity Deficiencies 4-12

Chapter 5 Condition Assessment 5-1
5.1 Sewer Inspection 5-1
5.2 Condition Rating 5-1
5.3 Risk Assessment 5-2
5.3.1 Likelihood of Failure Categories 5-2
5.3.2 Consequence of Failure Categories........ 5-5
5.3.3 Risk Score Calculations........ 5-5
5.4 Rehabilitation Decision Process 5-12
Chapter 6 Recommended Capital Improvement Program 6-1
6.1 Project Cost Estimates 6-1
6.2 Project Priorities 6-12
6.3 Implementation Considerations for Capacity Improvement Projects 6-12
6.3.1 Project 1 – Vineyard Road 6-12
6.3.2 Project 2 - Del Mar Avenue 6-13
6.3.3 Project 3 - Vallejo Avenue 6-13
6.3.4 Project 4 – Atherton Avenue 6-13
6.3.5 Project 5 – Arthur Street 6-14
6.3.6 Project 6 – Simmons Lane 6-14
6.3.7 Project 7 – San Marin Drive 6-14
6.3.8 Project 8 – Novato Boulevard 6-14
6.4 General Implementation Recommendations 6-15

Appendices

Appendix A - Future Development Assumptions by Parcel
Appendix B - Plots of Flow Monitoring Data
Appendix C - Dry Weather Model Calibration Graphs
Appendix D - Wet Weather Model Calibration Graphs
Appendix E - Condition/Risk Assessment Results and Rehabilitation Recommendations
Appendix F - Capacity Improvement Project Descriptions and Cost Estimates

List of Tables

Table ES-1: Collection System Flow Estimates 5
Table ES-2: Model-Predicted Capacity Deficiencies 8
Table ES-3: Recommended Capital Improvement Projects 15

Table 1-1: Collection System Inventory 1-2
Table 1-2: Gravity Pipe Materials 1-5
Table 1-3: Gravity Sewer Age 1-5
Table 2-1: Large Industrial and Institutional Users 2-2
Table 2-2: Future Growth Scenarios 2-4
Table 2-3: Base Wastewater Flow Projections 2-4
Table 3-1: Modeled Pump Stations 3-2
Table 3-2: Flow Meter and Rain Gauge Locations 3-6
Table 3-3: Rain Events During Flow Monitoring Period 3-9
Table 4-1: NOAA Atlas 14 Rainfall for NSD 4-2
Table 4-2: Summary of Capacity Analysis Results 4-6
Table 4-3: Summary of Modeled Pump Station Flows 4-6
Table 4-4: Capacity Deficiencies and Preliminary Solutions 4-8
Table 4-5: Recommended Capacity Improvement Projects 4-10
Table 4-6: Capacity Deficiencies Eliminated with Modeled RDI/I Reduction 4-12
Table 5-1: Likelihood of Failure Scoring 5-7
Table 5-2: Consequence of Failure Scoring 5-8
Table 5-3: Summary of Preliminary Rehabilitation Decisions 5-12
Table 5-4: Sewer Rehabilitation Decision Diagram Explanations 5-14
Table 6-1: Proposed Capital Improvement Projects 6-2
Table 6-2: Capital Improvement Program Summary 6-12

List of Figures

Figure ES-1: Sewer Service Area and Collection System 3
Figure ES-2: Modeled Trunk System 4
Figure ES-3: Model Results - Future Design Storm PWWF 7
Figure ES-4: Peak Structural Defect Scores 10
Figure ES-5: Total Structural Defect Scores 11
Figure ES-6: Risk Score Matrix 13
Figure ES-7: Overall Risk Priority 14
Figure ES-8: Recommended Capital Improvement Projects 16

Figure 1-1: Study Area 1-3
Figure 1-2: Existing Wastewater Collection System 1-4
Figure 2-1: Wastewater Flow Components 2-1
Figure 2-2: Potential Future Connections 2-3
Figure 2-3: NSD Diurnal Curves 2-5
Figure 2-4: Conceptual RDI/I Hydrograph Components 2-7
Figure 3-1: Modeled Trunk System 3-3
Figure 3-2: Ignacio Transfer Pump Station Operation 3-4
Figure 3-3: Example Sewer Subcatchments 3-5
Figure 3-4: Flow Meter and Rain Gauge Locations 3-7
Figure 3-5: NSD Flow Meter and System Schematic 3-8
Figure 3-6: Example Flow Monitoring Data (Meter FM1) 3-10
Figure 3-7: Example Dry Weather Calibration Graph (Meter FM1) 3-11
Figure 3-8: Example Wet Weather Calibration Graph (Meter FM1) 3-13
Figure 4-1: Nested vs SCS Type IA 5-Year, 24-Hour Design Storm 4-3
Figure 4-2: Model Results – Future Design Storm PWWF 4-7
Figure 4-3: Recommended Capacity Improvement Projects 4-11
Figure 5-1: Peak Structural Defect Scores 5-3
Figure 5-2: Total Structural Defect Scores 5-4
Figure 5-3: Risk Score Matrix 5-6
Figure 5-4: Likelihood of Failure Scores 5-9
Figure 5-5: Consequence of Failure Scores 5-10
Figure 5-6: Overall Risk Priority 5-11
Figure 5-7: Rehabilitation Decision Process 5-13
Figure 5-8: Preliminary Rehabilitation Decisions 5-15
Figure 6-1: Recommended Capital Improvement Projects 6-3
Figure 6-2: Project 1 – Vineyard Road 6-4
Figure 6-3: Project 2: Del Mar Avenue 6-5
Figure 6-4: Project 3: Vallejo Avenue 6-6
Figure 6-5: Project 4: Atherton Avenue 6-7
Figure 6-6: Project 5: Arthur Street 6-8
Figure 6-7: Project 6: Simmons Lane 6-9
Figure 6-8: Project 7: San Marin Drive 6-10
Figure 6-9: Project 8: Novato Boulevard 6-11

Acknowledgements

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List of Abbreviations

ABWF	Average base wastewater flow
ADWF	Average dry weather flow
APN	Assessor parcel number
BWF	Base wastewater flow
CCTV	Closed-circuit television
CIP	Capital Improvement Program
City	City of Novato
COF	Consequence of Failure
County	Marin County
CSMP	Collection System Master Plan
d/D	Ratio of flow depth to pipe diameter
DDF	Depth-duration-frequency
District	Novato Sanitary District
DSI	Damage Severity Index
DU	Dwelling unit
DWF	Dry weather flow
ENR-CCI	Engineering News Record Construction Cost Index
EQ	Equalization
FM	Flow meter
fps	Feet per second
GIS	Geographic Information System
gpad or gpd/ac	Gallons per day per acre
gpd	Gallons per day
gpm	Gallons per minute
GWI	Groundwater infiltration
HGL	Hydraulic gradeline
IDF	Intensity-duration-frequency
I/I	Infiltration/inflow
ITPS	Ignacio Transfer Pump Station
lf	Linear feet
LOF	Likelihood of Failure
MG	Million gallons
mgd	Million gallons per day
NMWD	North Marin Water District
NSD	Novato Sanitary District
O&M	Operation and Maintenance
PDWF	Peak dry weather flow
PS	Pump Station
PWWF	Peak wet weather flow
R value	Ratio of RDI/I volume to rainfall volume
RDI/I	Rainfall-dependent infiltration/inflow
RG	Rain gauge
SECAP	System Evaluation and Capacity Assurance Plan
SSMP	Sewer System Management Plan
SSO	Sanitary sewer overflow
WWTP	Wastewater Treatment Plant

EXECUTIVE SUMMARY

Novato Sanitary District's Collection System Master Plan

Executive Summary

This report summarizes the results and recommendations of the Wastewater Collection System Master Plan (Master Plan) for the Novato Sanitary District (NSD or District). The Master Plan was prepared by Woodard & Curran (formerly RMC Water and Environment) in close coordination with District staff. The Master Plan will be used to guide improvements to the NSD sanitary sewer system to accommodate current and future development and ensure that the District's customers continue to receive a high level of service.

Background and Purpose of Master Plan

The Novato Sanitary District is located in northern Marin County, and encompasses the City of Novato and some adjacent unincorporated areas. The District provides wastewater collection, conveyance, and treatment to an existing population of approximately 60,000. The collection system includes approximately 230 miles of sewer pipelines and conveys wastewater to the NSD Wastewater Treatment Plant, (WWTP), located east of Highway 101 and north of Novato Creek at the end of Davidson Street in Novato. Treated wastewater is discharged via a 5-mile outfall pipeline to San Pablo Bay. The NSD sewer system and service area are shown in **Figure ES-1**.

The District last initiated the preparation of a master plan for the collection system about 15 years ago. The plan included development of a hydraulic model of the trunk sewer system (primarily 10-inch and larger pipes) using data collected from flow monitoring in the system conducted in the 2003/04 and 2004/05 wet weather seasons. The model was used to assess the capacity of the trunk sewer system and identify capacity deficiencies and capacity improvements needed to handle peak flows from a 5-year return period design storm through the year 2025. The project scope also included a "criticality assessment" to prioritize areas of the system for sewer inspection and rehabilitation. Although a draft report was completed in 2008, that document was not finalized or formally adopted by the District, as the Wastewater Treatment Plant Upgrade projects (WWTP Upgrade) under construction at that time negated major conclusions and recommendations of the report. Primarily, this came about from a decision late in the design process of the WWTP Upgrade to lower the main sewer between the influent junction box and the influent pump station wet well under most influent flow conditions to optimize pump station operation. This had the added benefit of lowering the hydraulic profile (backwater curve) through the sewer system, which provided for a lowered risk profile for system overflows.

Since that time, the District has completed the construction of a new wastewater treatment plant at the site of the previous Novato treatment plant, and decommissioned the previous Ignacio treatment plant by construction of a new pump station and force main to convey the flow previously treated at the Ignacio plant to the new plant. The District has also rehabilitated or replaced about 7 miles of sewer pipelines, as well as completed a number of other sewer repairs and pump station upgrades. A substantial amount of the collection system has also been inspected by closed-circuit television (CCTV) as part of the District's ongoing sewer condition assessment activities. Due to the significant changes in the system over the past ten years and the additional information now available, the District recognized that it was time to prepare a new Master Plan for the wastewater collection system.

Under the Statewide General Waste Discharge Requirements for Sanitary Sewer Systems, adopted in 2006 by the State Water Resources Control Board, the District is required to prepare a Sewer System Management Plan (SSMP). The SSMP addresses the overall management, operation, and maintenance of the sanitary sewer system and is required for all sewer system agencies. The District last updated its SSMP in 2018. One of the elements of the SSMP is a System Evaluation and Capacity Assurance Plan (SECAP). This Master Plan provides the information needed to update the SECAP element of the District's SSMP.

The overall objectives of this Master Plan are to develop wastewater flow projections for the District's sewer service area using up-to-date development and land use information and flow monitoring data;

develop a new hydraulic model of the trunk sewer system; use the model to identify existing capacity deficiencies and future capacity requirements; analyze CCTV inspection data for the sewers to assess their condition and identify sewer rehabilitation and replacement needs; and develop a phased Capital Improvement Program (CIP), including budget estimates, for implementing the required improvements to the sewer system.

This Executive Summary is presented in three parts:

- ***How the Capacity Assessment Was Prepared*** describes the scope and methodologies of the capacity planning effort, including the key planning and technical assumptions incorporated into the hydraulic model and sewer system capacity analysis.
- ***How the Condition Assessment Was Prepared*** describes the scope and methodologies used in the condition assessment, including methodology for analysis of the CCTV data and developing condition scores and recommended rehabilitation methods and priorities.
- ***Recommended Capital Improvement Program*** presents the recommended Capital Improvement Program (CIP), including sewer rehabilitation and capacity improvement projects, priorities, and estimated costs. In addition, recommendations are presented for implementing the proposed capital improvement program.

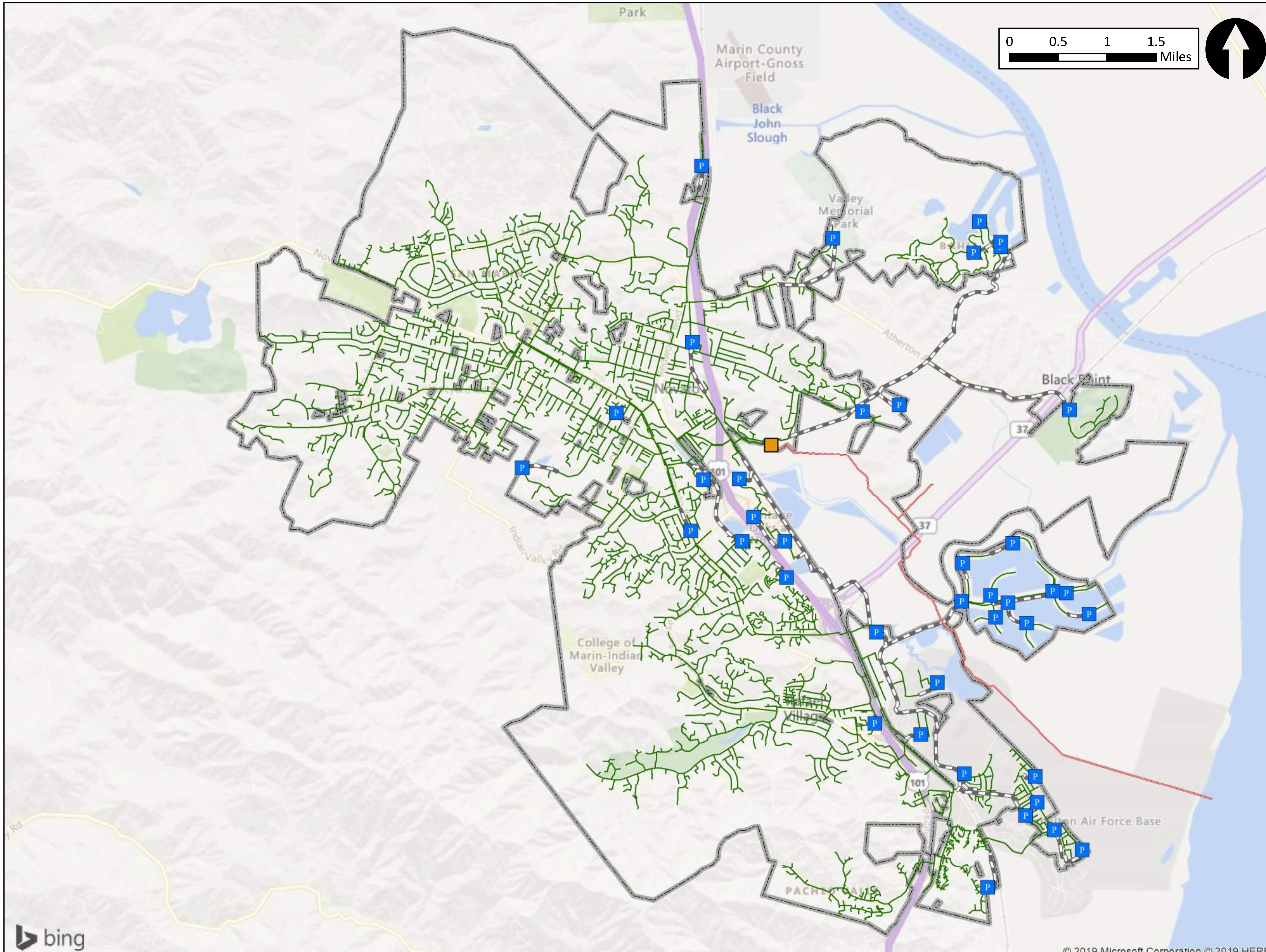
How the Capacity Assessment Was Prepared

The NSD sewer collection system includes pipelines ranging in size from 4 to 54 inches in diameter. The larger pipes, primarily the 10-inch and larger sewers and a portion of the smaller diameter pipes, comprise the trunk sewer system, which is the primary network for conveying flows generated in the service area to the NSD WWTP. The trunk sewer system was the focus of the capacity assessment in this study.

The project team used a systematic process that incorporated land use planning information, flow monitoring data, and design criteria for estimating wastewater flows, and applied the flows in a computer hydraulic model (InfoWorks™ ICM) of the trunk sewer system. The modeled system is shown in **Figure ES-2.** The model was used to assess how the system would perform under various planning and flow scenarios and to identify pipes that may not have sufficient capacity to convey the predicted peak flows under existing or future conditions. Improvement projects were developed to provide the required capacity, the capital costs of the required projects were estimated, and the projects were prioritized based on the model results and the potential impacts of predicted model overflows.

Capacity Assessment Considers Existing and Future Planning Scenarios

Planning scenarios representing existing and potential future development were evaluated for this study. The existing scenario examined the current capacity of the sewer system based on existing development, with flows defined based on the District's records of customer connections and calibrated to actual flow monitoring data. The future scenario assumed increases in residential dwelling units and commercial/industrial development based on information about planned development projects provided by the City of Novato, other potential development of currently vacant land, and potential connections of existing developed parcels currently served by on-site septic systems. For this study, the future scenario represents a 20-year planning horizon. The hydraulic model was used to examine the impact on the system of existing and potential future wastewater flows and determine the required sewer system capacity needed in the NSD system.



Novato Sanitary District

Collection System Master Plan

Figure ES-1

Sewer Service Area and Collection System

Sources: MarinMap, NSD

Map Created: 7/18/2019

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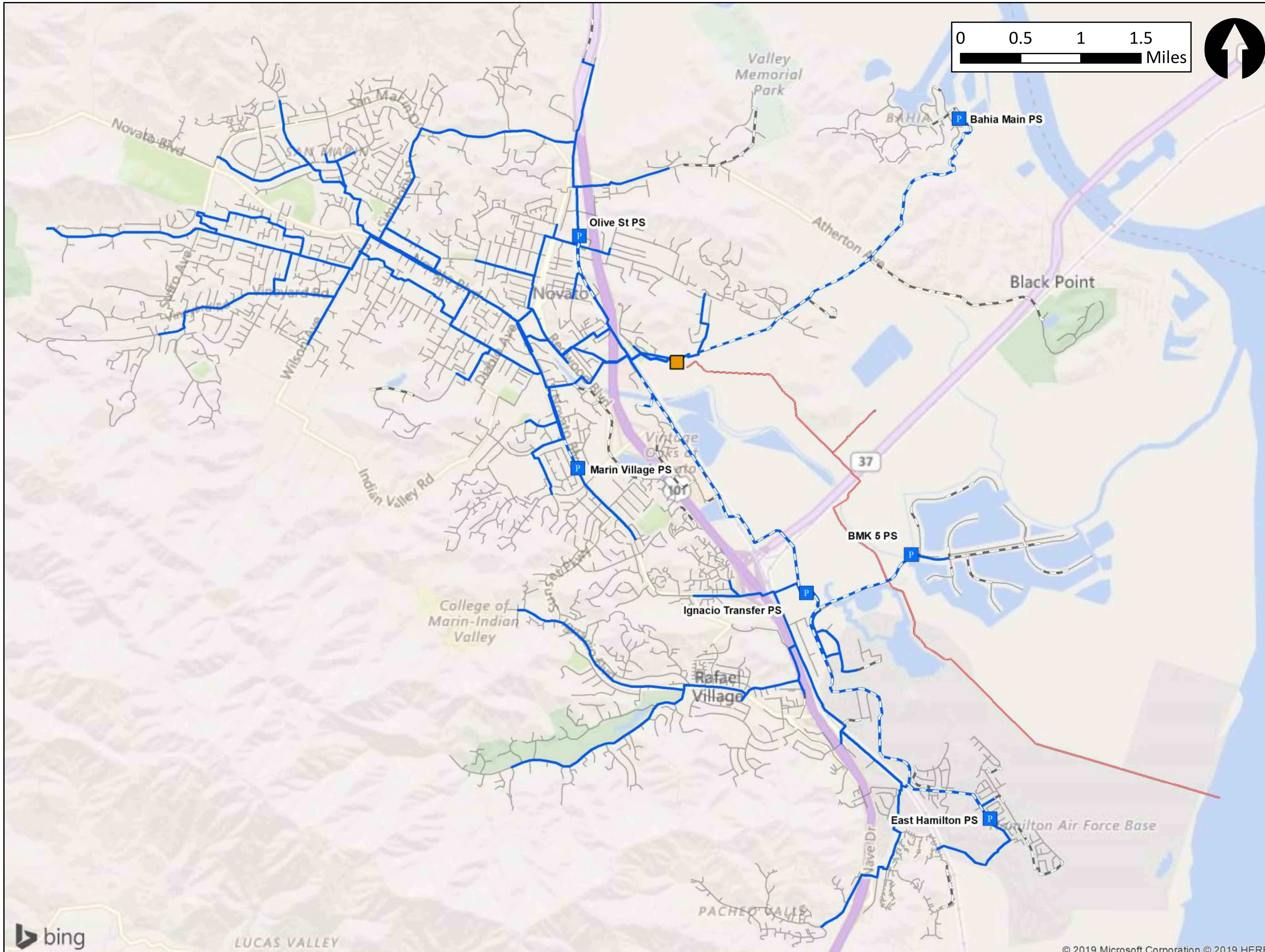

Legend

- Novato WWTP
- Modeled Pump Station
- Modeled Force Main
- Modeled Trunk Sewer
- Unmodeled Force Main
- Unmodeled Sewer
- WWTP Outfall

Novato Sanitary District

Collection System Master Plan

Figure ES-2

Modeled Trunk System

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Sources: Novato Sanitary District, Bing Maps

Map Created: 1/30/2019

Potential Capacity Deficiencies Under Existing and Future Flow Conditions

For each of the planning scenarios examined, projected dry and wet weather flows were simulated in the hydraulic model. The model was calibrated to flow monitoring data to ensure that it represents a reasonably accurate depiction of system conditions. For this study, a wet weather flow monitoring program consisting of 16 temporary flow meters installed in the system during the 2016/17 wet weather season, supplemented with data from the District's pump stations, was conducted to provide data to calibrate the hydraulic model and verify existing system flows.

The model integrates various dry and wet weather flow parameters to determine system capacity under different flow and planning scenarios. Key flow components incorporated into the model include: base (dry weather) wastewater flow, representing the sanitary and process flow contributions from existing and future customers; groundwater infiltration, which occurs when water seeps into pipes under the ground through cracks and pipe joints; and rainfall-dependent infiltration and inflow (RDI/I) during storm events. For this Master Plan, a 24-hour duration, 5-year return period storm event based on historical rainfall statistics was selected as the design event for evaluating system capacity and sizing required system improvements.

Table ES-1 presents the estimated existing and future average base wastewater flow (BWF), peak flow on a non-rainfall (dry) day, and peak wet weather flow (PWWF) for the selected design storm for the NSD sewer system developed based on the modeling conducted for this study. As noted previously, the future scenario represents a 20-year planning horizon.

Table ES-1: Collection System Flow Estimates

Flow Component	Existing (mgd)	Future (mgd)
Average Base Wastewater Flow		
Residential	3.10	3.34
Non-Residential	0.67	0.80
Total	3.77	4.14
Peak Flow on Non-Rainfall Day[a]	8.3	8.9
Peak Wet Weather Flow[b]	32.3	32.4 / 34.4[c]

a. Includes groundwater infiltration for a typical wintertime period.
b. For 5-year return period design storm.
c. Lower value represents conditions in existing sewer system, in which downstream peak flows are limited by upstream capacity deficiencies; higher value assumes that trunk sewer capacity deficiencies are relieved.

Model results were examined to determine trunk system capacity needs. Capacity deficiencies were identified where the modeled flow in the pipes exceeded pipe capacity and caused surcharge conditions (water levels higher than the crowns of the pipes) under peak dry weather flow (PDWF), or caused surcharging to within two (2) feet of manhole rims under design storm PWWF conditions. **Figure ES-3** shows the model results for future PWWF conditions, indicating existing trunk sewers that were predicted by the model to be surcharged due to "throttle" conditions (peak flow exceeding full pipe capacity) or due to backwater from a downstream throttle condition, and locations of model-predicted overflows. There were no areas of the trunk sewer system predicted to be surcharged under PDWF, either under existing or future development conditions.

Note that the locations of criteria violations (surcharge to within 2 feet of manhole rims under PWWF) are not necessarily the locations of the actual capacity-deficient pipes, but are typically located further upstream due to backwater from downstream deficiencies. The specific pipe reaches with capacity deficiencies that result in criteria violations are listed in **Table ES-2**. All of these locations were also predicted to be capacity issues under existing PWWF.

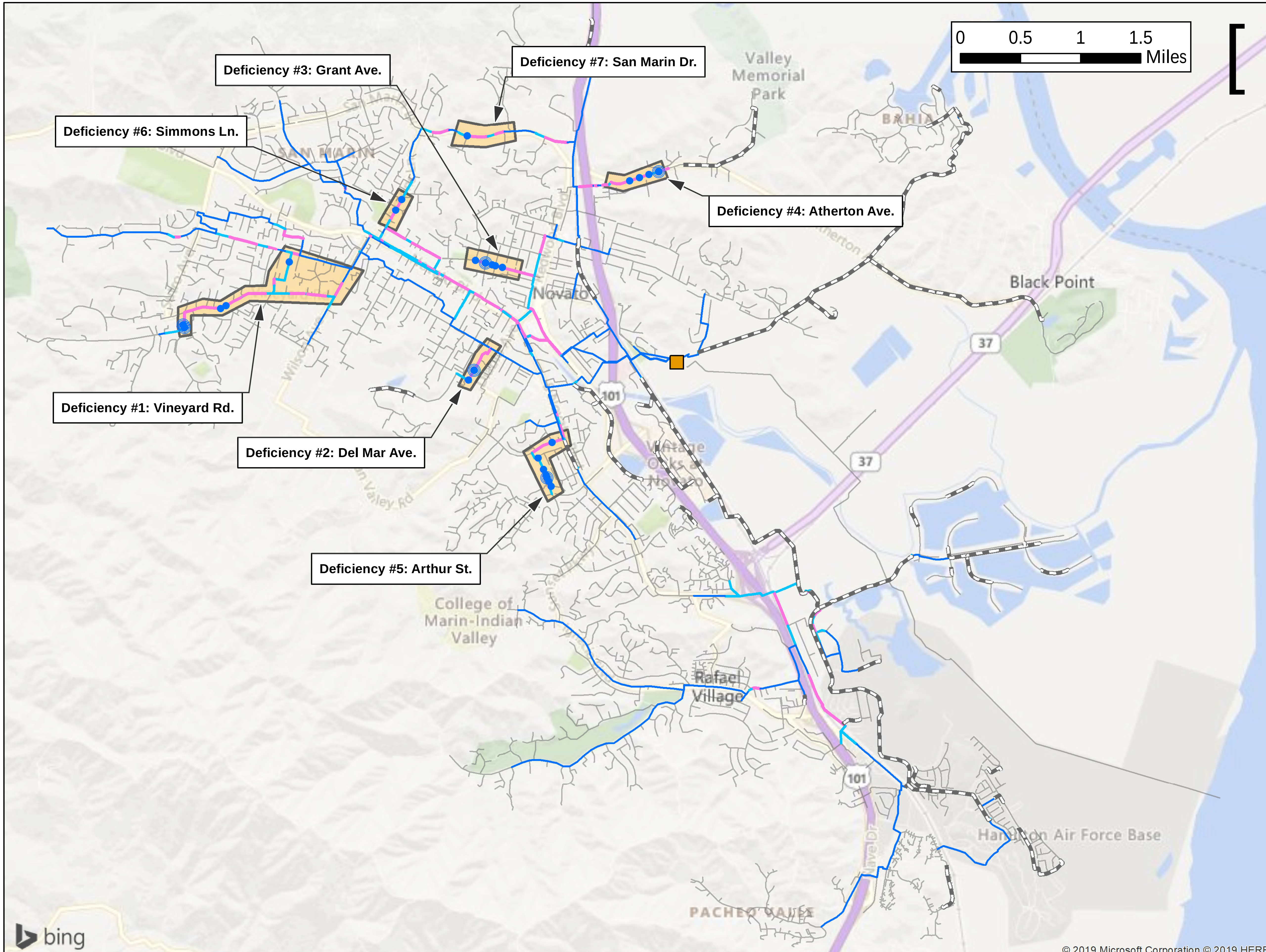
It should also be noted that the District has not documented overflows or significant surcharging in the system at all of these locations, and therefore this Master Plan recommends that additional verification monitoring be conducted to confirm predicted surcharge conditions in areas where such conditions have not previously been observed.

Solutions to Potential Capacity Deficiencies

To address the capacity deficiencies identified through the modeling effort, three types of capacity relief alternatives were explored:

- Upsizing pipes (i.e., replacing existing pipes with larger ones)
- Construction of parallel pipes
- Flow diversions to other sewers with available capacity or to new sewer pipelines

Potential flow routing and capacity improvement alternatives were developed and tested, and proposed improvements were verified using the hydraulic model. (Note that in some cases, improvements to relieve existing capacity deficiencies resulted in higher peak flows being conveyed downstream, with the resulting need for additional improvements. In the case of NSD, the need for an additional capacity improvement project in Novato Boulevard was triggered by relieving the capacity deficiencies shown in Table ES-2.) Each proposed project site was reviewed in the field and/or on aerial maps to identify site constraints and assess potential construction conditions, methods, and issues. Based on these analyses, recommended capacity improvement projects were developed and incorporated into the recommended CIP.



Legend

- Novato WWTP

Modeled Sewer

- Not surcharged
- Backwater surcharge
- Throttle surcharge
- Force Main
- Model Freeboard < 2 ft
- Model Predicted Overflow
- Unmodeled Sewer
- Capacity Deficiency Areas

Novato Sanitary District

Collection System Master Plan

Figure ES-3

Model Results - Future Design Storm PWWF

NOVATO SANITARY DISTRICT NSD

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Sources: Novato Sanitary District, Bing Maps

Map Created: 10/22/2019

Table ES-2: Model-Predicted Capacity Deficiencies

Deficiency ID	Deficiency Location[a]	Upstream MHID	Downstream MHID	Length (ft.)	Exist. Diam. (in.)	Predicted Freeboard (ft.)[b]	Predicted Overflow Volume[b] (gal)
1	Angelica Ct., Vineyard Rd., Wilson Ave. to Mary Jane Ln.; Sun Ln., easement, Kendon Ln., Center Rd. to Western Dr.	C16008 D16005	E15066 D15002	8,565	8 - 12	Overflow	140,000
2	Del Mar Ave. from Hill Rd. to Center Rd.	F17035[c]	G17139	1,413	8	Overflow	13,000
3	Grant Ave. from Sixth St. to First St.	G15046	G15042	1,742	10 - 12	Overflow	1,100
4	Atherton Ave. east of Oak Shade Ln. to east of Binford Rd.	I13001	H14056	1,722	8	Overflow	14,000
5	Arthur St. from Buchanan St. easement to S. Novato Blvd.	G19007	G19085	1,182	10	Overflow	150,000
6	Simmons Ln. from Feliz Rd. to Kristin Ln.[d]	F14005	F14004	335	8	1.5	N/A
7	San Marin Dr. west of Spinosa Way	G13013	G13014	319	6	1.5	N/A

a. Deficiency location is location of throttled pipes causing upstream criteria violation under future PWWF conditions.
b. Under future PWWF conditions in existing sewer system.
c. Manhole is F17111 on District maps (renumbered for model due to duplicate ID)
d. Assumes that temporary plug on Simmons Lane that is currently used during wet weather periods to back up and divert flow to San Marin Drive sewer would not be installed.

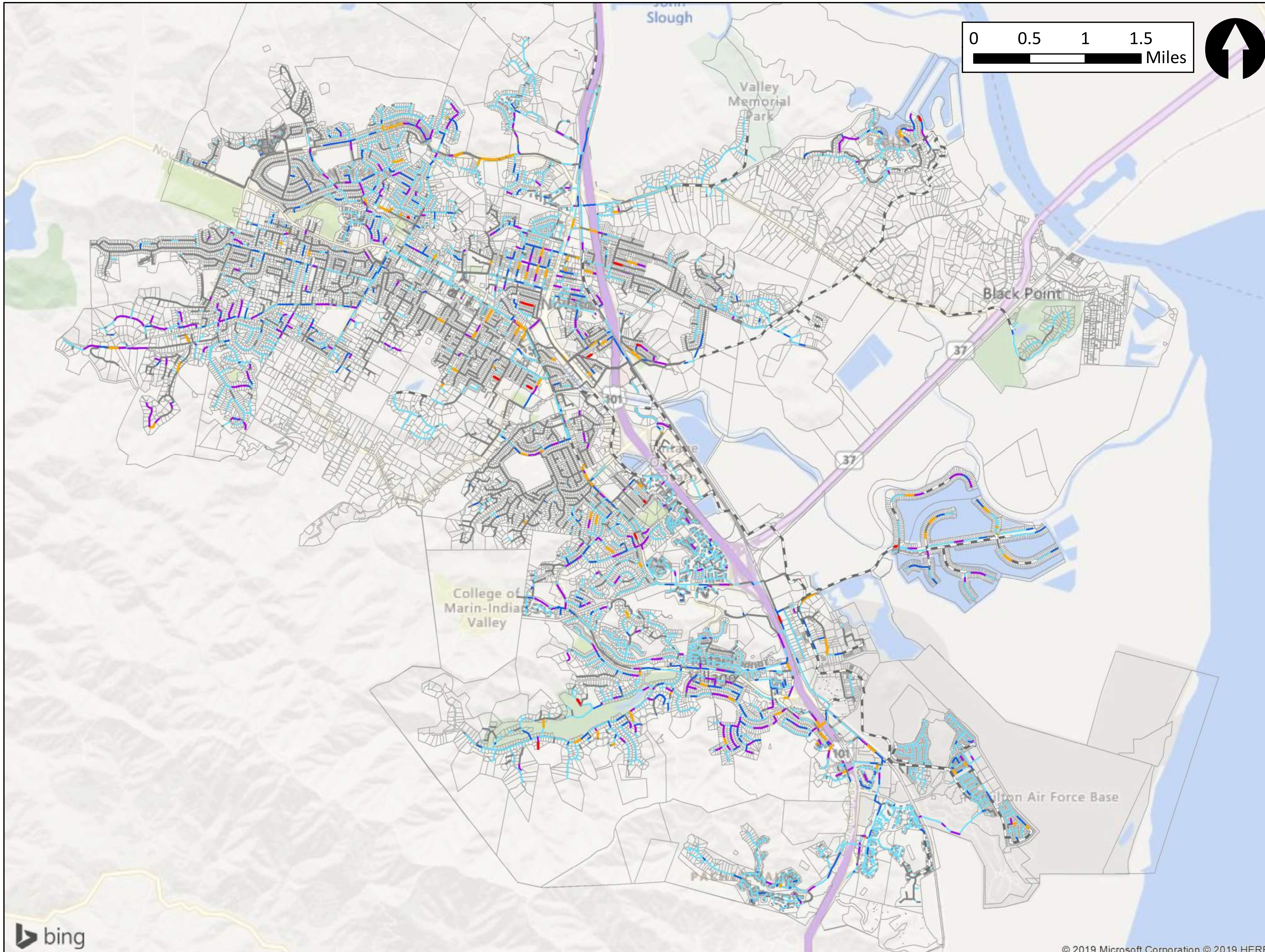
How the Condition Assessment Was Prepared

The District maintains a database of closed-circuit television (CCTV) inspection data for sewers dating back to 2009. To date, the District has inspected most of the sewers in the system, with the goal of inspecting the entire system on a five-year cycle. However, review of the CCTV data indicated that some inspections may have been conducted for reasons other than structural condition assessment (e.g., to verify effectiveness of root treatment) and not all defects had been recorded. Therefore, not all of the data could be used for the condition assessment. In total, 3,848 pipes (about 60 percent of the pipes in the gravity collection system) had usable data for the condition assessment.

Condition Rating

The condition assessment was conducted using InfoMaster™ software from Innovyze. InfoMaster is a pipeline asset planning program that incorporates condition scoring, risk assessment, and rehabilitation decision analysis in a GIS environment. For NSD, CCTV data was imported into InfoMaster and linked to the District's sewer pipeline GIS. The program computes condition scores based on a standard rating system specified by the Pipeline Assessment Certification Program (PACP) developed by the National Association of Sewer Service Companies (NASSCO). In the PACP system, defects are categorized as structural or maintenance defects (the latter also referred to as "service" defects) and graded on a scale of 1 to 5 with 5 representing the most severe. Of particular interest for this master plan were the peak structural defect scores (representing the number of the highest grade structural defect)[1] and the total structural defect score (sum of grades for all structural defects) for each pipe, with pipes with high peak and total scores indicating those sewers with the highest likelihood of failure due to structural condition. **Figure ES-4** and **Figure ES-5**, respectively, show the results of the peak and total structural condition ratings of the gravity sewers in the NSD system. Pipes indicated in red on the figures are those with two or more Grade 5 structural defects (18 pipes, as shown on Figure ES-4) or a total defect score greater than 50 (12 pipes, as shown on Figure ES-5). Over 70 percent of the inspected pipes had no recorded structural defects.

[1] See footnote a. in Table 5-1 for definition of peak defect score.



Legend

Peak Structural Defect Score

- 1 (No Defects)
- 1.1 - 3.1
- 3.2 - 4.1
- 4.2 - 5.1
- 5.2 - 6
- No CCTV Data
- Force Main

See footnote a in Table 5-1 for a definition of peak defect scores.

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Figure ES-4

Peak Structural Defect Score

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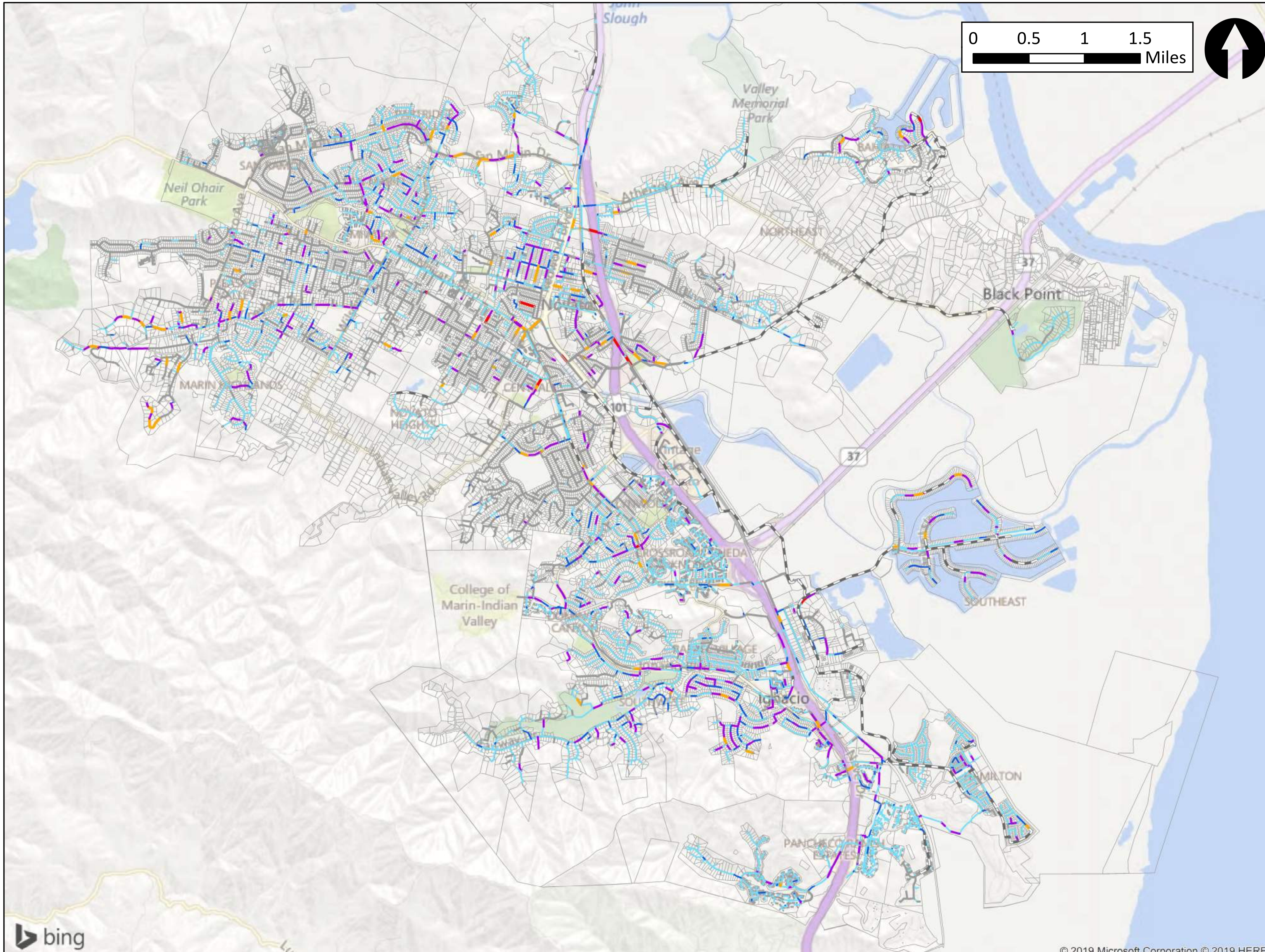
Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 7/17/2019



Legend

Total Structural Defect Score

- 0 (No Defects)
- 1 - 5
- 6 - 20
- 21 - 50
- > 50
- No CCTV Data
- Force Main

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Figure ES-5

Total Structural Defect Score

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Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/24/2019

Risk Assessment

A risk analysis approach ("risk model") was used to prioritize sewers for potential rehabilitation. The methodology involves quantifying and assessing the risks posed by the failure or inability of the sewer system to provide the level of service needed to meet the District's sewer system management goals. Using this approach, risk scores are calculated for each sewer pipe, and individual pipe scores are then used to prioritize the pipes for repair, rehabilitation, or replacement.

The risk of asset failure is calculated by quantifying the likelihood of failure (LOF) and the consequence of failure (COF) of a sewer asset. The likelihood of failure is the possibility of asset failure based on its condition. The consequence of failure is defined as the impact on the level of service resulting from asset failure. The combination of LOF and COF scores is therefore an indicator of potential risk.

Four LOF indicators were used in the risk model, as follows:

- **Structural Condition**, based on the peak and total structural defect scores. If CCTV inspection data was not available for a pipe, then the structural condition score was estimated solely based on pipe segment age and material (see Structural Vulnerability below). Structural condition is a strong indicator of likelihood of failure and was heavily weighted.
- **Structural Vulnerability**, based on pipe age and material. Older clay, asbestos cement, and concrete pipes, which are more susceptible to cracking and corrosion, were given higher scores than newer plastic pipe materials.
- **Operation and Maintenance (O&M) Condition**, based on a pipe's maintenance requirements as reflected by its current scheduled frequency of sewer cleaning. Scheduled cleaning frequency is an indicator of the likelihood of asset failure due to a maintenance-related issue that could lead to a sanitary sewer overflow (SSO). This LOF factor was given a lower weighting than structural condition since preventive maintenance cleaning can be used to alleviate this risk.
- **I/I Contribution**, based on the peak I/I rate of the flow meter area in which the pipe was located.

COF is assessed by examining the impact on economic, social and environmental factors. Four indicators were used to assess COF:

- **Potential Spill Volume**, based on pipe diameter, reflecting the potential size of a sewer spill, number of customers affected, and cost of repair. This COF factor was most heavily weighted.
- **Environmental Impact**, based on distance to surface waters.
- **Traffic/Response Impacts**, based on pipe location (type of road or easement).
- **Public Impact**, based on proximity to commercial areas or public facilities.

The risk model, also implemented in InfoMaster, utilized data from GIS and CCTV inspections to compute LOF and COF scores, ranging from 1 (very low) to 5 (very high) for each factor. The combination of the LOF and COF scores were then used to group the pipes into four risk categories (very low, low, medium, and high), as shown on the matrix in **Figure ES-6**. The results indicated that less than 5 percent of the pipes in the system (about 10 miles) were categorized as high or medium risk (with less than 1-1/2 miles as high risk), and almost 90 percent as very low risk. **Figure ES-7** shows the risk priority for the pipes in the system. Note that pipes with a "very high" LOF were considered high risk priority, regardless of their potential consequence of failure, as a failed pipe always has the potential for an SSO and adverse impact on District customers.

Figure ES-6: Risk Score Matrix

Consequence of Failure (COF)	Likelihood of Failure (LOF): Very Low LOF	Low LOF	Medium LOF	High LOF	Very High LOF
Very High COF	LOF Score ≤ 1.5, COF Score > 4	LOF Score ≤ 2, COF Score > 4	LOF Score ≤ 3, COF Score > 4	LOF Score ≤ 4, COF Score > 4	LOF Score > 4, COF Score > 4
High COF	LOF Score ≤ 1.5, COF Score ≤ 4	LOF Score ≤ 2, COF Score ≤ 4	LOF Score ≤ 3, COF Score ≤ 4	LOF Score ≤ 4, COF Score ≤ 4	LOF Score > 4, COF Score ≤ 4
Medium COF	LOF Score ≤ 1.5, COF Score ≤ 3	LOF Score ≤ 2, COF Score ≤ 3	LOF Score ≤ 3, COF Score ≤ 3	LOF Score ≤ 4, COF Score ≤ 3	LOF Score > 4, COF Score ≤ 3
Low COF	LOF Score ≤ 1.5, COF Score ≤ 2	LOF Score ≤ 2, COF Score ≤ 2	LOF Score ≤ 3, COF Score ≤ 2	LOF Score ≤ 4, COF Score ≤ 2	LOF Score > 4, COF Score ≤ 2
Very Low COF	LOF Score ≤ 1.5, COF Score ≤ 1.5	LOF Score ≤ 2, COF Score ≤ 1.5	LOF Score ≤ 3, COF Score ≤ 1.5	LOF Score ≤ 4, COF Score ≤ 1.5	LOF Score > 4, COF Score ≤ 1.5

Priority >>	Very Low	Low	Medium	High

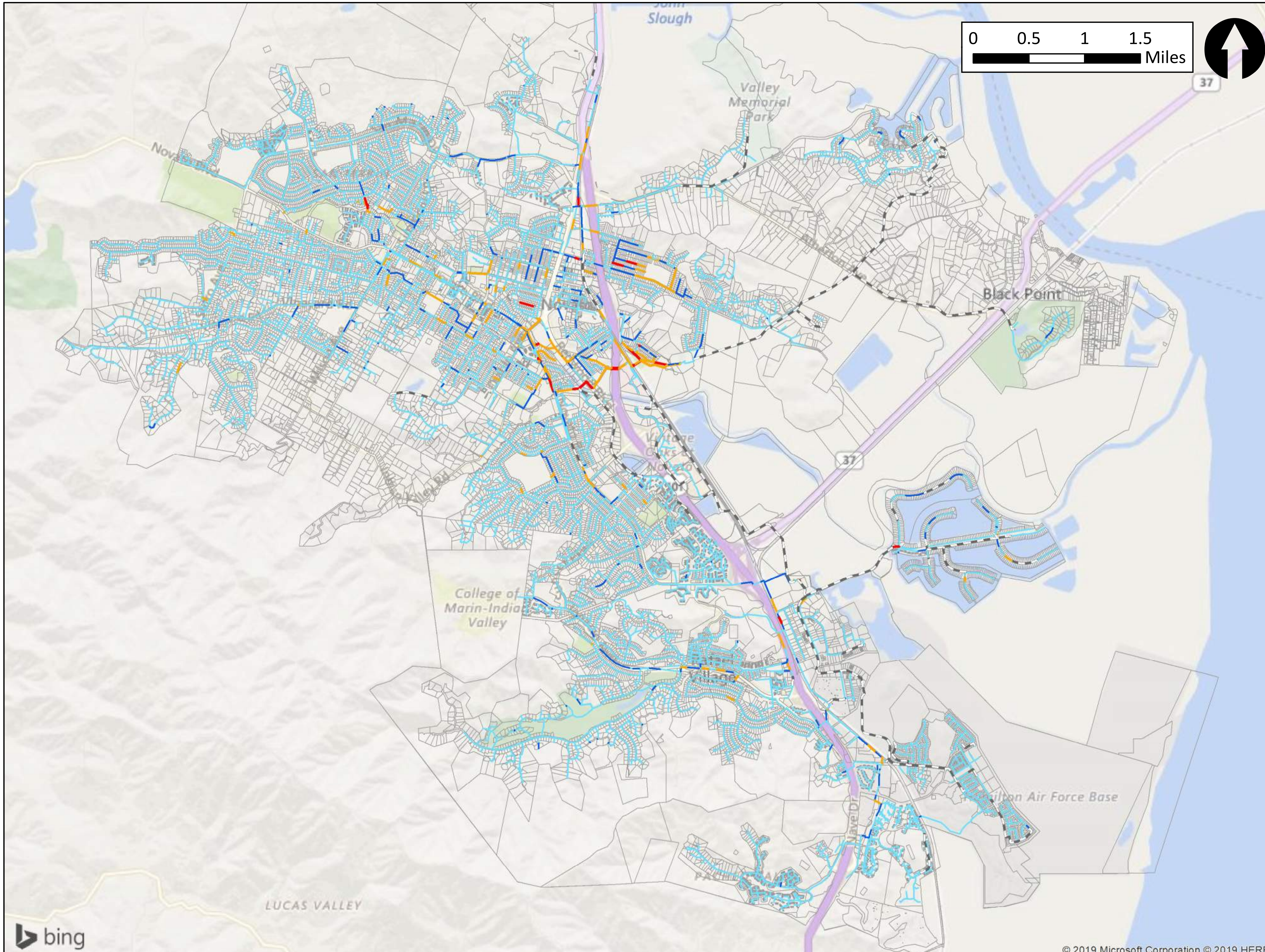
Rehabilitation Decision Process

The InfoMaster program was also used to generate preliminary rehabilitation recommendations for each pipe based on its condition from CCTV inspection. A decision process was developed to identify the preliminary "best method" of rehabilitation based on the number and type of defects in the pipe. Only pipes with "major" defects were identified for rehabilitation, with major defects being defined as NASSCO PACP Grade 4 or 5 structural defects. Examples of major defects include broken or collapsed pipe, significant fractures, large offset or separated joints, severe sags, or significant corrosion. Based on the number and type of defects and the pipe diameter, each sewer with major defects was recommended for either point repair, lining, or replacement (or for further evaluation if the pipe had sags, bends, or corrosion that would require more detailed review of CCTV video). In addition, pipes without CCTV data but with a high or medium risk priority rating were identified as priorities for CCTV inspection.

Of the 94 high and medium risk priority pipes with CCTV data, about 50 were recommended for replacement or lining and 11 for point repair of major defects. The remainder (mostly medium risk priority) did not have any major defects and are recommended for continued preventive maintenance and inspection. About 80 other pipes in the system also had one or more major (Grade 4 or 5) defects that may warrant further evaluation, but these pipes were considered low or very low risk priority. About 150 high and medium risk priority pipes did not have CCTV data and are recommended for near-term inspection.

Recommended Capital Improvement Program

The Capital Improvement Program (CIP) recommended in this study is designed as guidance for the District to provide for adequate sewer system capacity for the District's existing and anticipated future development and repair or replacement of sewers in need of rehabilitation. **Table ES-3** and **Figure ES-8** present the recommended capital improvement projects.



Legend

Risk Priority

- Very Low
- Low
- Medium
- High
- Force Main

Novato Sanitary District

Collection System Master Plan

Figure ES-7

Overall Risk Priority

NOVATO SANITARY DISTRICT NSD

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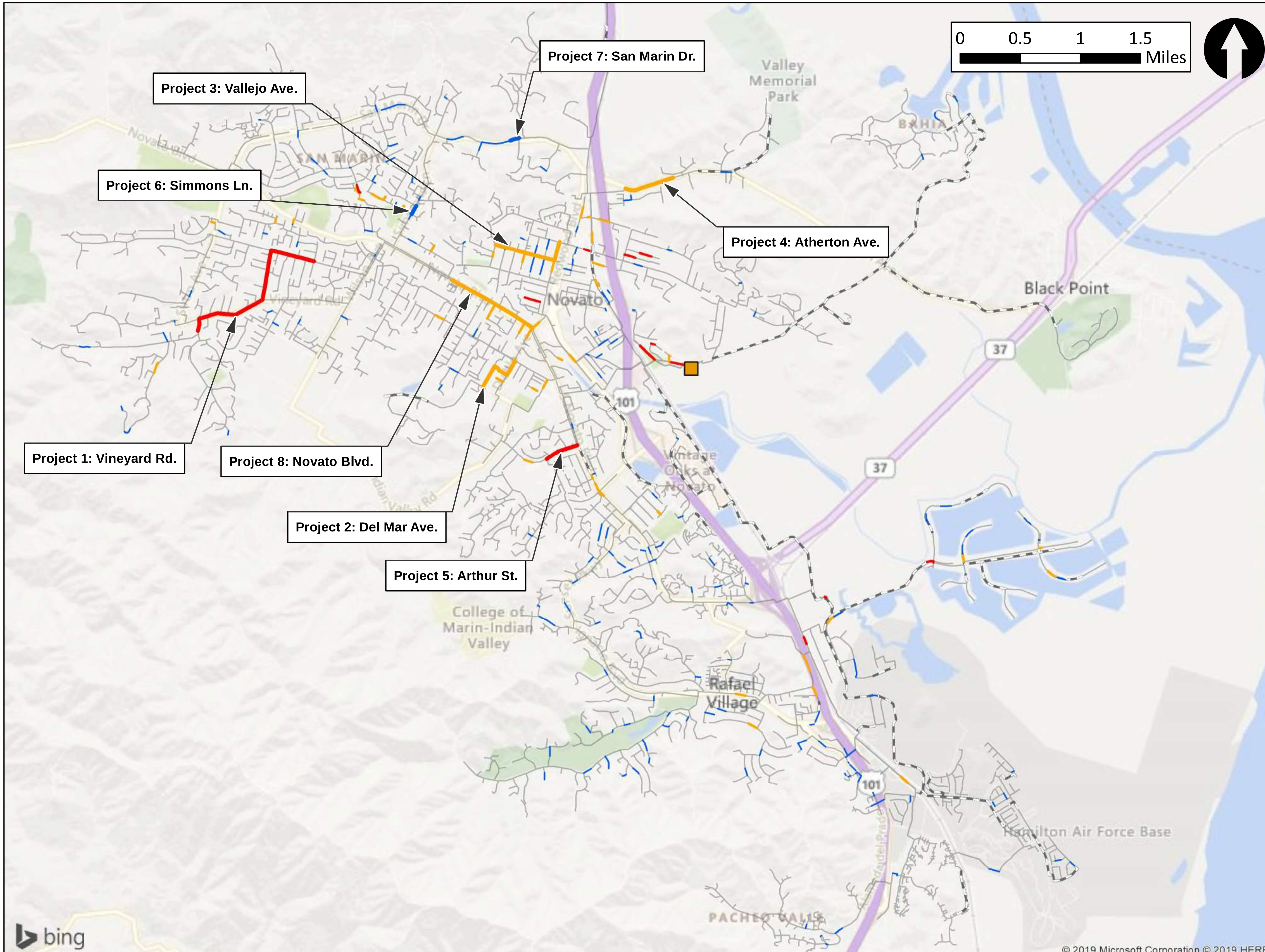
Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/28/2019

Table ES-3: Recommended Capital Improvement Projects

Project ID[a]	Priority	Project Name	Description	Est. Capital Cost[b] ($)
Capacity Improvement Projects				
1	1	Vineyard Road	5,600 lf of 12" to 15" pipe on Angelica Ct., Vineyard Dr., Eucalyptus Ave. and Center Rd. to Western Dr.	5,230,000
2	2	Del Mar Avenue	2,015 lf of 10" pipe in Del Mar Ave., Hotchkin Dr., and Diablo Ave. from Hill Rd. to Center Rd.	1,770,000
3	2	Vallejo Avenue	2,775 lf of 8" to 12" pipe in Vallejo Avenue and 1st Street from 7th St. to Olive Ave.	1,860,000
4	2	Atherton Avenue	1,722 lf of 10" to 12" pipe in Atherton Ave. from east of Oak Shade Ln. to east of Binford Rd.	980,000
5	1	Arthur Street	1,182 lf of 15" pipe in Arthur Street from west of Hayes St. to S. Novato Blvd.	1,210,000
6	3	Simmons Lane	335 lf of 10" pipe in Simmons Ln. from Feliz Rd. to Kristin Ln.	92,000
7	3	San Marin Drive	319 lf of 8" pipe in San Marin Dr. west of Spinosa Way	135,000
8	2	Novato Boulevard	3,240 of 24" pipe in Novato Blvd. from east of Grant Ave. to Diablo Ave.	3,890,000
			Subtotal – Capacity Improvements	**15,170,000**
Structural Rehabilitation				
	1	High Risk	Major defect repair, rehabilitation, and replacement	2,500,000
	2	Medium Risk	Major defect repair, rehabilitation, and replacement	7,900,000
	3	Low/Very Low Risk	Major defect repair, rehabilitation, and replacement	18,700,000
			Subtotal – Structural Rehabilitation	**29,100,000**
			Priority 1 Total	**8,900,000**
			Priority 2 Total	**16,400,000**
			Priority 3 Total	**19,000,000**
			Total Estimated Capital Cost:	**44,300,000**

a. Corresponds to Deficiency ID in Table ES-2, except Project 8, which was triggered by increased flows after relief of other capacity deficiencies. Priorities are based on magnitude of predicted overflow volume (Priorities 1 and 2) or surcharge resulting in freeboard violations (Priority 3).
b. Estimated capital costs based on late 2018 dollars; include allowances of 30% of estimated construction cost for contingencies for unknown conditions and 25% for engineering, administration, construction management, and legal costs.



Legend

- Novato WWTP

Capacity Projects

- Priority 1
- Priority 2
- Priority 3

Structural Rehabilitation

- Priority 1 (High Risk)
- Priority 2 (Medium Risk)
- Priority 3 (Low/Very Low Risk)
- Force Main
- Gravity Main

Novato Sanitary District

Collection System Master Plan

Figure ES-8

Recommended Capital Improvement Projects

Sources: Novato Sanitary District, Bing Maps

Map Created: 7/17/2019

\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure_ES-8_Recommended_Capital_improv_Projects.mxd

As shown in Table ES-3, the total estimated capital cost of the capacity improvement CIP is approximately $15.2 million. These costs, presented in late 2018 dollars, include baseline construction costs for gravity trunk sewers primarily using trenchless (e.g., pipe bursting) or open-cut methods; costs for new sewer structures, lateral reconnections, and lower lateral replacement (where sewer main would be replaced); and costs allowances for project mobilization/demobilization, traffic control, and bypass pumping. The total estimated construction costs also include a 30 percent allowance for contingencies for unknown conditions, and the total estimated capital cost includes an allowance of 25 percent of construction cost for engineering, administration, construction management, and legal costs. The estimated costs are planning or conceptual level estimates to be used for budgeting purposes only, and are considered to have an estimated accuracy range of -30 to +50 percent.

The estimated cost for repair, rehabilitation, and replacement of pipes with major structural defects, based on the results of the condition assessment, is approximately $29 million (about $10 million for high and medium risk pipes).

The recommended capacity improvement projects were prioritized based on the relative magnitude of the predicted deficiencies (e.g., extent of model-predicted surcharge or potential overflow). The project priorities are also shown in Table ES-3. The priorities generally represent decreasing magnitude of predicted overflow volume, as indicated in Table ES-2. Sewers identified for repair, rehabilitation, or replacement due to occurrence of major structural defects were grouped into three priority groups based on their risk priority rating from the risk model.

Implementation Recommendations

The District should begin implementation of the capacity improvement projects recommended in this Master Plan, starting with the highest priority projects. The first step in implementation would be the development of project validation plans to confirm the need for specific projects (e.g., based on flow monitoring, as discussed above) and further refine project sizing and alignments, proposed construction methods, and coordination between needed capacity and structural rehabilitation improvements. This information will allow the District to develop a detailed implementation plan for the CIP, in conjunction with associated financial studies, which would include updated cost estimates and proposed project schedules consistent with the District's financial capabilities.

The following items should be considered in project scheduling and design, and in future updates of the Master Plan.

- The District should consider conducting additional focused flow monitoring or observation of flow levels during large storm events to confirm the need for and refine project sizing if necessary.
- Smoke testing and other field investigations should be considered for areas where more intensive flow monitoring indicates particularly high I/I rates, especially where potential elimination of needed capacity improvements could be accomplished through modest I/I reductions.
- The alignments and sizes of all recommended projects should be verified with detailed predesign analyses, including topographic surveys, geotechnical investigations, utility research, and constructability reviews.
- The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.
- The hydraulic model has been developed to assist the District in performing capacity analyses and updating the Master Plan in the future. The model should be kept up-to-date with changes to existing sewer connections, development plans, and sewer system facilities.

- The District should continue to keep its sewer GIS mapping and data up-to-date and review the updated GIS database developed for this study to confirm and/or correct sewer attribute data as needed.
- The District should accelerate CCTV inspection of high and medium risk priority pipes that do not have CCTV inspection data, and should ensure that all future inspections rigorously follow NASSCO PACP observation coding standards.
- The District should undertake studies to evaluate if some of its smaller lift stations, such as Enfrente and Cypress, could be eliminated and replaced with gravity bypass sewers in order to reduce potential risk from pump station outages or failures.

This Master Plan report is intended to be a working document to be refined and updated as additional data and new planning information become available. The capacity assessment should be updated whenever there are major changes in planning assumptions or, at a minimum, every five to ten years.

1

CHAPTER 1

INTRODUCTION

Novato Sanitary District's Collection System Master Plan

Chapter 1 Introduction

This report presents the results and recommendations of the Novato Sanitary District Wastewater Collection System Master Plan (Master Plan) prepared by Woodard & Curran (formerly RMC Water and Environment). The Master Plan scope was developed based on the recommendations of the Collection System Master Plan Scoping Study prepared by RMC in April 2016.

1.1 Background and Study Objectives

The Novato Sanitary District (NSD or District) is located in northern Marin County, and encompasses the City of Novato and some adjacent unincorporated areas. The District provides wastewater collection, conveyance, and treatment to an existing population of approximately 60,000. **Figure 1-1** shows the boundary of the sewer service area, which is the study area for this Master Plan.

The District last initiated the preparation of a master plan for the collection system about 15 years ago, retaining RMA Engineering and Management (RMA) to prepare the plan. The plan included development of a hydraulic model of the trunk sewer system (primarily 10-inch and larger pipes) using data collected from flow monitoring in the system conducted in the 2003/04 and 2004/05 wet weather seasons. The model was used to assess the capacity of the trunk sewer system and identify capacity deficiencies and capacity improvements needed to handle peak flows from a 5-year return period design storm through the year 2025. The project scope also included a “criticality assessment” to prioritize areas of the system for sewer inspection and rehabilitation. RMA prepared a draft report in 2008; however, that document was never finalized or formally adopted by the District.

Since that time, the District has completed the construction of a new wastewater treatment plant at the site of the previous Novato treatment plant, and decommissioned the previous Ignacio treatment plant by construction of a new pump station and force main to convey the flow previously treated at the Ignacio plant to the new plant. The District has also rehabilitated or replaced about 7 miles of sewer pipelines, as well as completed a number of other sewer repairs and pump station upgrades. A substantial amount of the collection system has also been inspected by closed-circuit television (CCTV) as part of the District’s ongoing sewer condition assessment activities. Due to the significant changes in the system over the past ten years and the additional information now available, the District recognized that it was time to prepare a new Master Plan for the wastewater collection system.

Under the Statewide General Waste Discharge Requirements for Sanitary Sewer Systems, adopted in 2006 by the State Water Resources Control Board, the District is required to prepare a Sewer System Management Plan (SSMP). The SSMP addresses the overall management, operation, and maintenance of the sanitary sewer system and is required for all sewer system agencies. The District last updated its SSMP in 2018. One of the elements of the SSMP is a System Evaluation and Capacity Assurance Plan (SECAP). This Master Plan provides the information needed to update the SECAP element of the District’s SSMP.

The overall objectives of this Master Plan are to develop wastewater flow projections for the District’s sewer service area using up-to-date development and land use information and flow monitoring data; develop a new hydraulic model of the trunk sewer system; use the model to identify existing capacity deficiencies and future capacity requirements; analyze closed-circuit television (CCTV) inspection data for the sewers to assess their condition and identify sewer rehabilitation and replacement needs; and develop a phased Capital Improvement Program (CIP), including budget estimates, for implementing the required capacity improvements to the sewer system.

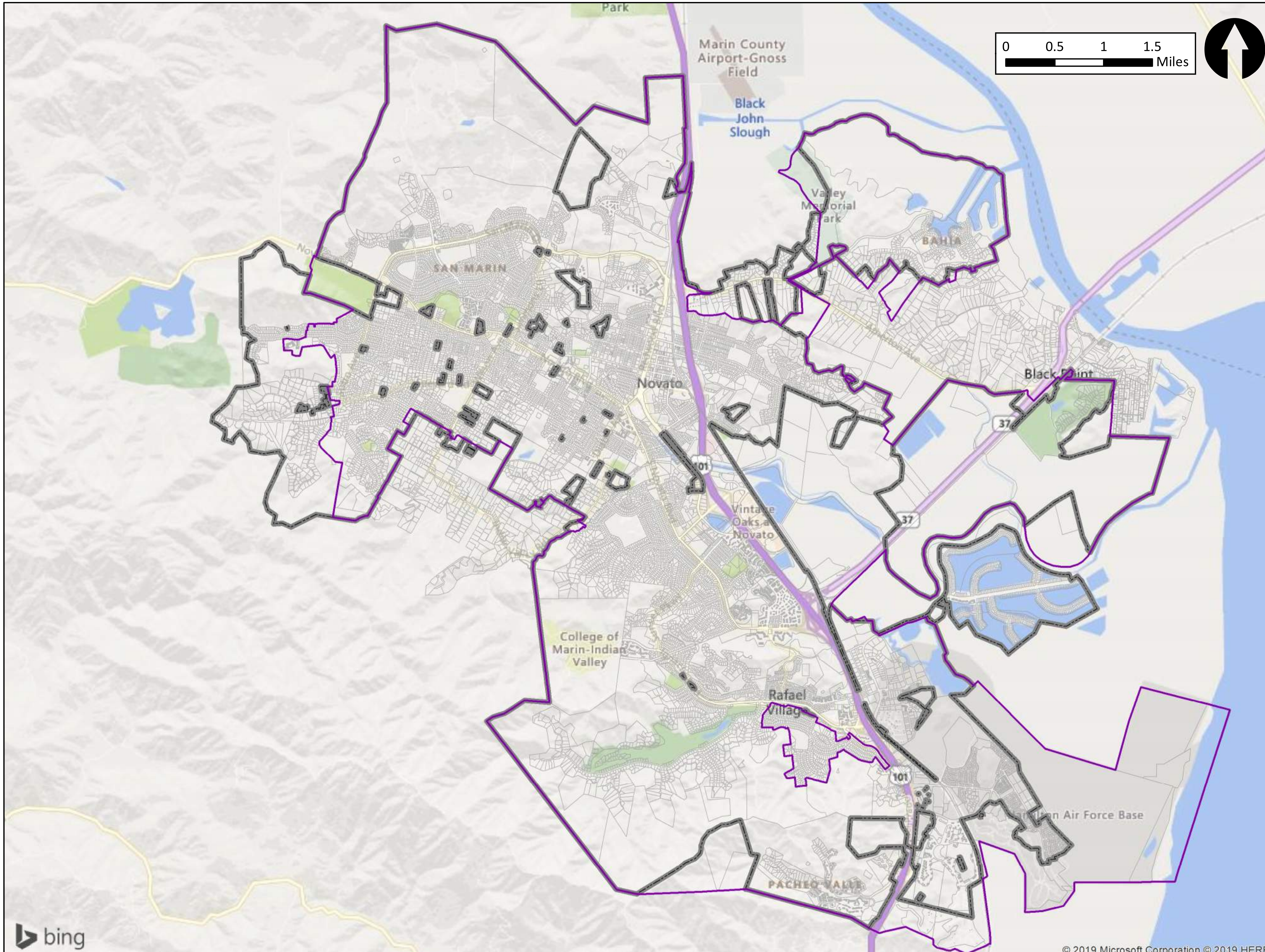
1.2 Sanitary Sewer System

The NSD sanitary sewer collection system includes 230 miles of sewer pipelines, including approximately 211 miles of gravity sewers ranging from 4 to 54 inches in diameter, about 19 miles of force mains, five main pump stations and 33 sewage lift stations. The system conveys wastewater to the NSD Wastewater Treatment Plant (WWTP), located east of Highway 101 and north of Novato Creek at the end of Davidson Street in Novato. Treated wastewater is discharged via a 5-mile outfall pipeline to San Pablo Bay. The sewer system is shown in **Figure 1-2**. **Table 1-1** provides a tabulation of the footage of existing pipe by diameter.

Table 1-1: Collection System Inventory

Pipe Diameter (in.)	Gravity Mains		Force Mains	
	Length (ft.)	Percent	Length (ft.)	Percent
4	264	0.0%	21,508	21.8%
6	716,258	64.2%	17,166	17.4%
8	224,432	20.1%	10,274	10.4%
10	59,690	5.3%	2,173	2.2%
12	35,167	3.2%	31,766	32.2%
14-16	28,134	2.5%	10,675	10.8%
18	15,817	1.4%	0	0.0%
21	4,146	0.4%	0	0.0%
24	13,678	1.2%	0	0.0%
27	2,607	0.2%	2,226	2.3%
30	7,707	0.7%	2,235	2.3%
33-36	3,480	0.3%	0	0.0%
39-42	2,633	0.2%	0	0.0%
48-54[a]	2,175	0.2%	0	0.0%
Unknown	35	0.0%	733	0.7%
Total	**1,116,223**		**98,756**	

a. Does not include WWTP outfall



Legend

Novato City Limits*

District Boundary*

*Downloaded from MarinMap June 2019

Novato Sanitary District

Collection System Master Plan

Figure 1-1

Study Area

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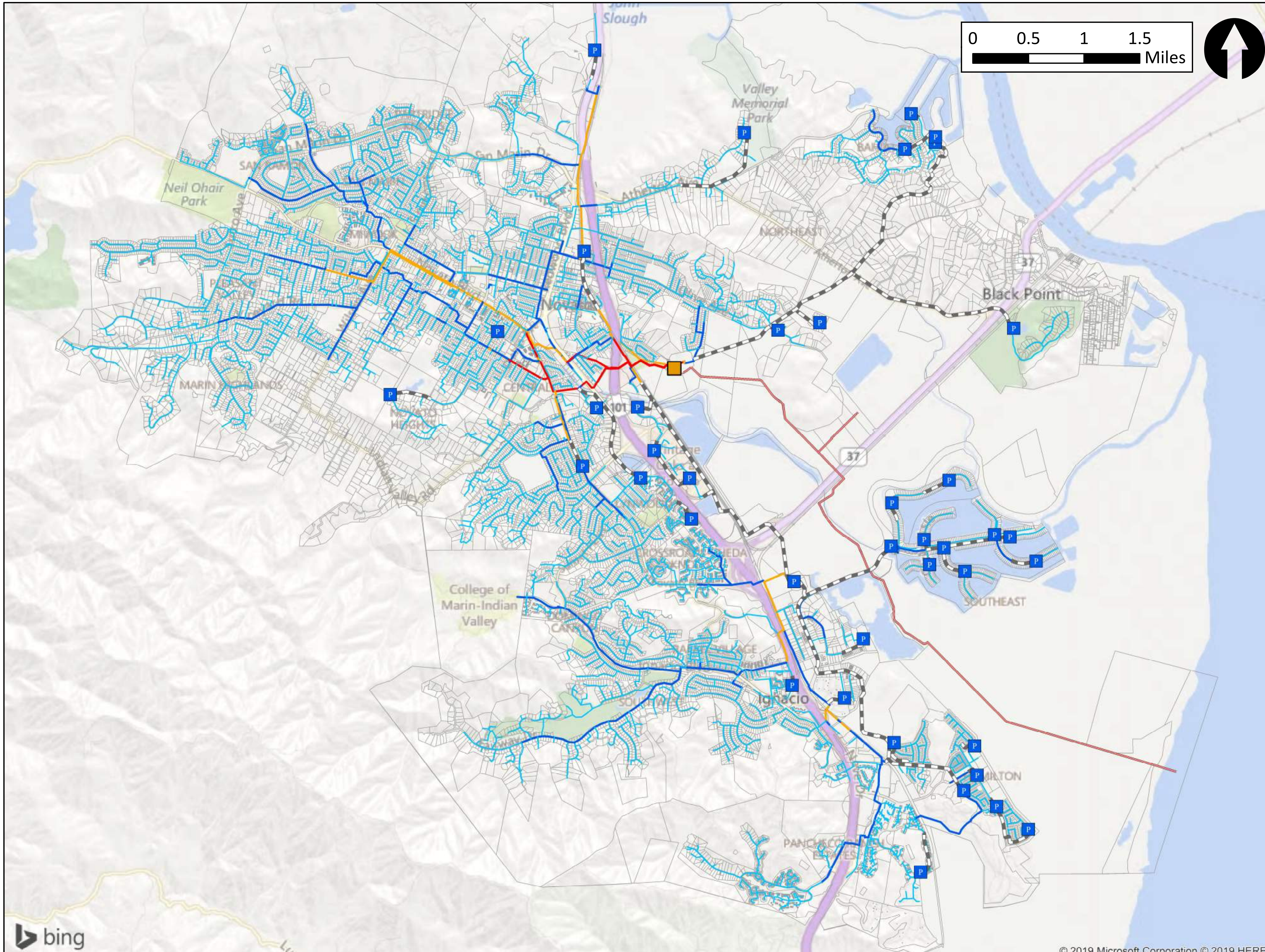
Sources: MarinMap

Map Created: 7/18/2019



Legend

- Novato WWTP
- Pump Station

Gravity Main Diameter

- <10
- 10 - 16
- 18 - 27
- 30+
- NTP Outfall
- Force Main

Novato Sanitary District

Collection System Master Plan

Figure 1-2

Existing Wastewater Collection System

NOVATO SANITARY DISTRICT NSD

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\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure_1-2_Existing_WW_Collection_System.mxd

Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/24/2019

The primary gravity pipe materials in the system are vitrified clay pipe (VCP), polyvinyl chloride (PVC) pipe, and asbestos cement pipe (ACP) with smaller percentages of other materials. Most force mains are plastic materials: PVC or high density polyethylene (HDPE). The oldest sewers in the system are about 70 years old, and over 100 miles (almost 50 percent) are over 50 years old. The oldest gravity sewers are primarily VCP, with ACP used predominantly from the mid-1960s to early 1980s, and PVC since then. **Table 1-2** and **Table 1-3** summarize the gravity pipe materials and age.

Table 1-2: Gravity Pipe Materials

Pipe Material	Pipe Material Code	Length (ft.)	Percent
Acrylonitrile butadiene styrene	ABS	9,617	0.9%
Asbestos cement	ACP	231,665	20.8%
Cast iron	CIP	9,109	0.8%
High-density polyethylene	HDPE	10,859	1.0%
Polyvinyl chloride	PVC	304,343	27.3%
Reinforced concrete	RCP	36,240	3.2%
Vitrified clay	VCP	510,188	45.7%
Other		4,201	0.4%

Table 1-3: Gravity Sewer Age

Install Date	Length (ft.)	Percent
Pre-1950	45,102	4.0%
1950s	187,338	16.8%
1960s	315,442	28.3%
1970s	194,407	17.4%
1980s	139,564	12.5%
1990s	96,873	8.7%
2000s	118,294	10.6%
2010s	12,922	1.2%
Unknown	6,282	0.6%

1.3 Scope of Study

The scope of the Master Plan, as well as a brief discussion of work conducted under each task, are described below.

- **Task 1 – Project Management and Coordination.** Periodic progress meetings and teleconferences were held with the District staff to review project status and discuss project issues, and monthly status reports were prepared to document the work completed. Presentations were also made to the District's Board of Directors to keep them updated on the progress, findings, and recommendations of the study.
- **Task 2 – Data Collection and Review.** Data, maps, GIS files, and previous reports related to the sanitary sewer system were reviewed to identify the information available for completing the Master Plan. This task also included developing updated sewer system GIS layers based on the District's latest AutoCAD sewer map. Sewer features that were not reflected in the District's current GIS were added or updated, and manhole rim and pipe invert elevations were populated based on the data in AutoCAD. The updated GIS was used for the capacity and condition assessments in this Master Plan.
- **Task 3 – Flow Monitoring.** A flow monitoring program, consisting of the 16 meters and 4 rain gauges installed in the sewer system from mid-December 2016 through mid-February 2017 was conducted to obtain data to characterize flows in the system and calibrate the hydraulic model. Flow data were also obtained for the District's five largest pump stations and the WWTP from the District's SCADA system.
- **Task 4 – Hydraulic Model Development and Calibration.** A hydraulic model of the NSD trunk sewer system was developed using InfoWorks™ ICM modeling software. The model was verified for system connectivity, and pipe and manhole data were populated based information from the updated GIS, record drawings, and field verification conducted by District staff. Subcatchments (small areas of unmodeled sewers that contribute flow to the modeled system) were delineated to define areas loading to the model, and flow loads to the model were compiled using parcel billing and land use data and flow factors representing unit base wastewater flow (BWF) rates, diurnal BWF patterns, and infiltration/inflow (I/I). Information was obtained from the City of Novato to identify potential growth or redevelopment areas within the service area. The model was calibrated for dry weather conditions, and then calibrated for wet weather conditions using actual storm events from the flow monitoring program.
- **Task 5 – System Performance Evaluation and Improvement Needs.** The model was used to determine sewer system capacity requirements and identify capacity deficiencies in the trunk sewer system under peak dry and wet weather flow conditions, defined based on a design storm and system performance criteria. Potential solutions to capacity deficiencies were identified and tested in the model, and potential capacity improvement project alternatives were evaluated.
- **Task 6 – Condition Assessment and Rehabilitation/Replacement (R/R) Recommendations.** This task involved using the District's closed-circuit television (CCTV) inspection data in order to assist the District in developing a systematic approach for assessing the condition of its sewer system assets and making appropriate decisions with respect to prioritizing inspection and sewer rehabilitation and replacement. The condition assessment was implemented using InfoMaster™ software.
- **Task 7 – Develop Methodology for Prioritizing Sewer Improvements.** A risk assessment methodology based on Likelihood of Failure (LOF) and Consequence of Failure (COF) factors was developed and used to prioritize sewers for inspection, repair, rehabilitation or replacement. The risk assessment was also implemented using InfoMaster.
- **Task 8 – Capital Improvement Program (CIP) Development.** Recommended capacity improvements and rehabilitation needs were prioritized in a phased CIP, including estimating costs.

- **Task 9 – Prepare Master Plan.** This report was prepared to summarize and present the results and recommendation of the study.

1.4 Report Organization

This report includes six sections, which are described below.

- **Chapter 1, Introduction,** presents the background, objectives, and scope of the Wastewater Collection System Master Plan and provides a description of the existing collection system facilities.
- **Chapter 2, Basis of Flow Estimates,** discusses the service area existing customers and projected land uses, the basis for developing estimates for each component of wastewater flows, and the base wastewater flow projections for the service area.
- **Chapter 3, Hydraulic Model Development,** describes the modeled sewer system, development of the model network and model loads, flow monitoring program, and model calibration.
- **Chapter 4, Capacity Assessment,** describes the capacity analysis and design criteria, model results, and recommended capacity improvement projects.
- **Chapter 5, Condition Assessment,** describes the condition and risk assessment methodology and results and the basis for sewer rehabilitation recommendations.
- **Chapter 6, Recommended Capital Improvement Program,** presents the recommended CIP, including project costs, prioritization, and implementation recommendations.

The Appendices to this report include a tabulation of future development assumptions by parcel; plots of the flow monitoring data; model calibration graphs; condition and risk assessment results; and documentation for the recommended CIP, including project descriptions and cost estimates.

2

CHAPTER 2

BASIS OF FLOW ESTIMATES

Novato Sanitary District's Collection System Master Plan

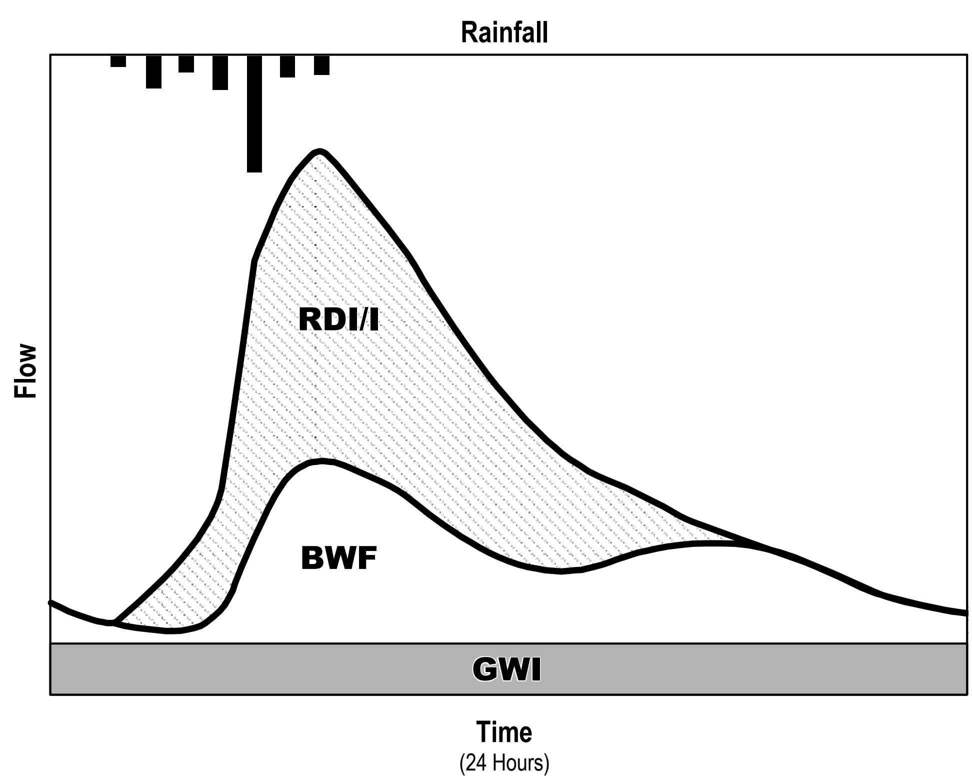
Chapter 2 Basis of Flow Estimates

This chapter presents the basis of wastewater flow estimates for the NSD sewer system. The section describes the wastewater flow components used in the hydraulic model and the existing and projected future land uses for the service area, which form the basis for generating base wastewater flows. Design flow estimates were developed based on criteria developed for each flow component: base wastewater flow, groundwater infiltration, and rainfall-dependent infiltration and inflow, and confirmed through model calibration, as described in Chapter 3 of this report.

2.1 Wastewater Flow Components

Wastewater flows include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I), as illustrated conceptually in **Figure 2-1**. BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system. GWI is groundwater that infiltrates into defects in sewer pipes and manholes, particularly in winter and springtime in low-lying areas. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year. RDI/I is storm water inflow and infiltration that enter the system in direct response to rainfall events, typically through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or, more commonly, through defects in sewer pipes, manholes, and service laterals. RDI/I typically results in short term peak flows that recede quickly after the rainfall ends.

Figure 2-1: Wastewater Flow Components
(Not to scale)



2.2 Base Wastewater Flow

BWF loads were developed for both existing and future development conditions. Existing loads were developed based on the District's customer billing database and water use data, and future loads were based on development and redevelopment projects anticipated by the City of Novato. The loading methodology for existing conditions and for projecting future BWF are discussed in the following subsections.

2.2.1 Existing Base Wastewater Flow

Existing residential and non-residential BWF was estimated based on customer sewer billing data provided by the District. The District provided winter water use data for 2014 through 2016 from the North Marin Water District (NMWD), although only 2016 data was used for this study for consistency with the flow monitoring period. Metered water use during the winter months most closely approximates wastewater generation, since outdoor water use is at a minimum.

Base wastewater flows for specific large industrial and institutional users were determined based on water use records. **Table 2-1** lists the large users and their current average wastewater flows and typical use patterns.

Table 2-1: Large Industrial and Institutional Users

Name	Load Manhole	Existing Average BWF (gpd)	Use Pattern
BioMarin	K23071	89,000[a]	See Figure 2-3
NMWD Treatment Plant	B14025	150,000/40,000[b]	Constant (24/7)

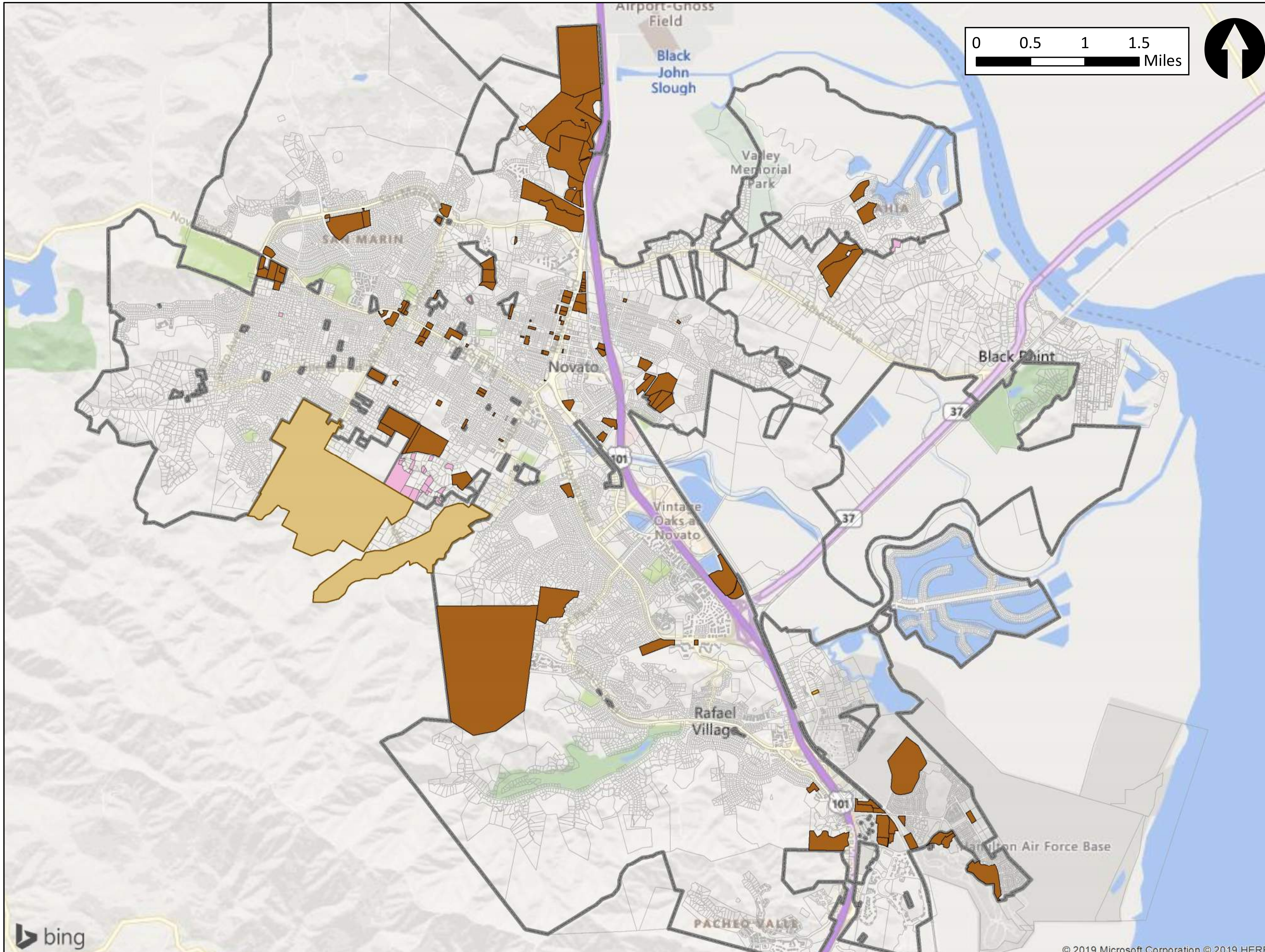
a. Based on 2016/17 water use.
b. Dry weather/wet weather; based on maximum allowable discharge

2.2.2 Future Base Wastewater Flow

Future BWF was estimated by applying unit flow factors to the anticipated increase in the number of dwelling units and non-residential building square footage. The City of Novato Planning staff provided a listing of proposed projects and development areas in the City as well as in adjacent unincorporated areas, and the District identified additional parcels that may also have the potential to develop. The potential development sites identified by the City and District include approximately 1,100 residential units and over 920,000 square feet of non-residential building floor space.

In addition to new development and redevelopment, various portions of the service area that are currently served by on-site septic systems may be connected to the sewer system in the future. These areas were also included in the future BWF estimates and include existing septic units plus about 450 existing and future parcels in the Indian Valley area. However, the currently unsewered Black Point and Green Point areas located to the east of the District boundary were not included in the future scenario, since if they were to connect, they would require separate pump stations and force mains to convey flow directly to the WWTP and therefore would not impact the existing collection system. Future connection of these areas would require area-specific studies to develop sewerage plans and identify required conveyance facilities.

The information on future development and potential future connections to the system was tabulated by assessor parcel number (APN) and processed in GIS by overlaying them on the sewer subcatchment layers. These areas are shown in **Figure 2-2.** A detailed tabulation of the future development and sewer connection assumptions by parcel is included in **Appendix A**.



Legend

- Potential Future Development
- Indian Valley
- Potential Septic Conversion
- District Boundary*

*Downloaded from MarinMap June 2019

Novato Sanitary District

Collection System Master Plan

Figure 2-2

Potential Future Development

NOVATO SANITARY DISTRICT NSD

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\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure_2-2_FutureDevelopment.mxd

Sources: MarinMap, City of Novato, NSD

Map Created: 7/18/2019

Three future growth scenarios were defined for the Master Plan, as described in **Table 2-2.**

Table 2-2: Future Growth Scenarios

Scenario (Timeframe)	Development Assumptions
5 Year	Near-term development projects identified by City
10 - 15 Year	Other developments identified by City and District; septic system conversions on Canyon Road
20 Year	Other vacant developable sites; currently unconnected parcels within District, out-of-District "islands", Indian Valley

Based on the District's guidelines, a flow factor of 150 gpd per dwelling unit was used for all future residential development and septic system conversion areas. A typical of value of 0.1 gpd/square foot of building floor space was assumed for most non-residential development, except hotels (100 gpd/room), industrial parks (0.15 gpd/square foot), and warehouses (0.05 gpd/square foot). An estimated 25 percent future increase in flow (~23,000 gpd) for BioMarin was assumed for this study based on the best information available at this time.

The resulting BWF from future development and septic conversions was added to the existing BWF and imported into the model to represent the future scenarios.

2.2.3 Base Wastewater Flow Projections

Table 2-3 summarizes the existing and future BWF for residential and non-residential land use categories. Based on these estimates, BWF in the NSD could increase by about 3 percent due to potential future development and redevelopment.

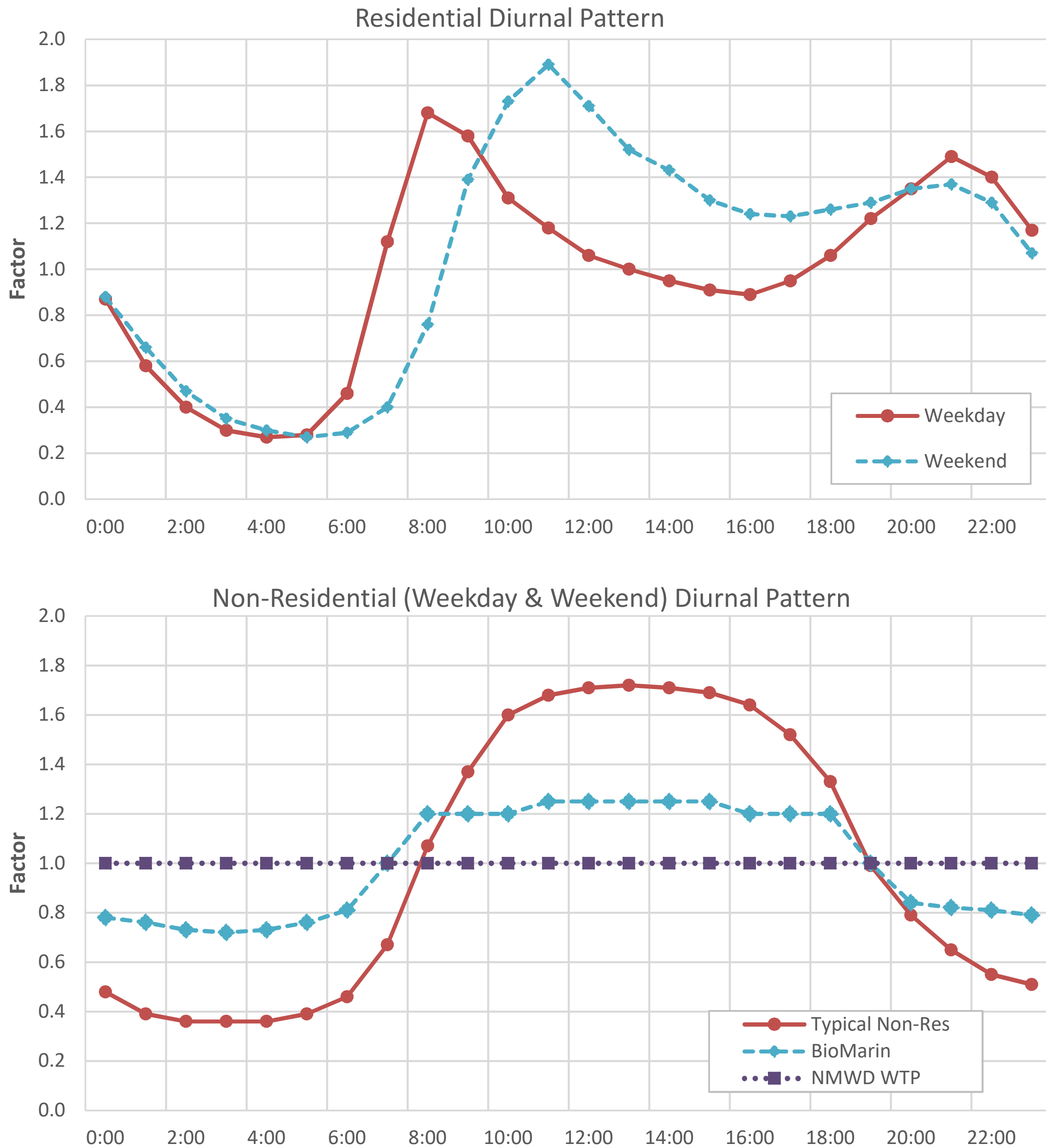
Table 2-3: Base Wastewater Flow Projections

Type of Development	Estimated BWF (mgd)			
	Existing	Future 5 Year	Future 10-15 Year	Future 20 Year
Residential	3.10	3.15	3.23	3.34
Non-Residential	0.67	0.73	0.80	0.80
Total	**3.77**	**3.88**	**4.03**	**4.14**

2.2.4 Diurnal Curves

In most sewer systems, BWF exhibits typical diurnal patterns depending on the type of land use. The most common ones are residential and commercial (non-residential). The diurnal curves developed for NSD are shown in **Figure 2-3.** For modeling purposes, the same diurnal curve for both weekdays and weekends was assumed for non-residential flows (insufficient data was available to estimate separate weekend patterns). As noted in Table 2-1, specific diurnal curves were also applied for large users based on their use patterns.

Figure 2-3: NSD Diurnal Curves



2.3 Groundwater Infiltration

Groundwater infiltration is generally quantified based on actual flow monitoring data, since it is difficult to predict GWI rates based on physical system data alone. In the context of design flow criteria, GWI represents the incremental groundwater infiltration that occurs during the wet weather season above the "baseline" infiltration level during the driest months of the year.

GWI can be estimated based on minimum flows during non-rainfall periods within a wet weather flow monitoring period. Minimum flows typically occur during the nighttime or early morning hours when base wastewater flows are at a low. Alternatively, GWI can be estimated as the difference between average metered flow during non-rainfall periods and computed average BWF. In either case, the resulting GWI, is expressed on a unit basis (gpd/acre or gpad) by dividing by the sewered acreage of the monitored area. Typical GWI rates may range from less than 100 to over 1,000 gpad.

GWI flows were estimated through the model calibration process (described in Chapter 3) by comparing model-simulated BWF to actual flow measurements from the temporary flow monitoring program. Cases where model-predicted BWF was noticeably lower than monitored flow indicated the possible occurrence of GWI.

2.4 Rainfall-Dependent I/I

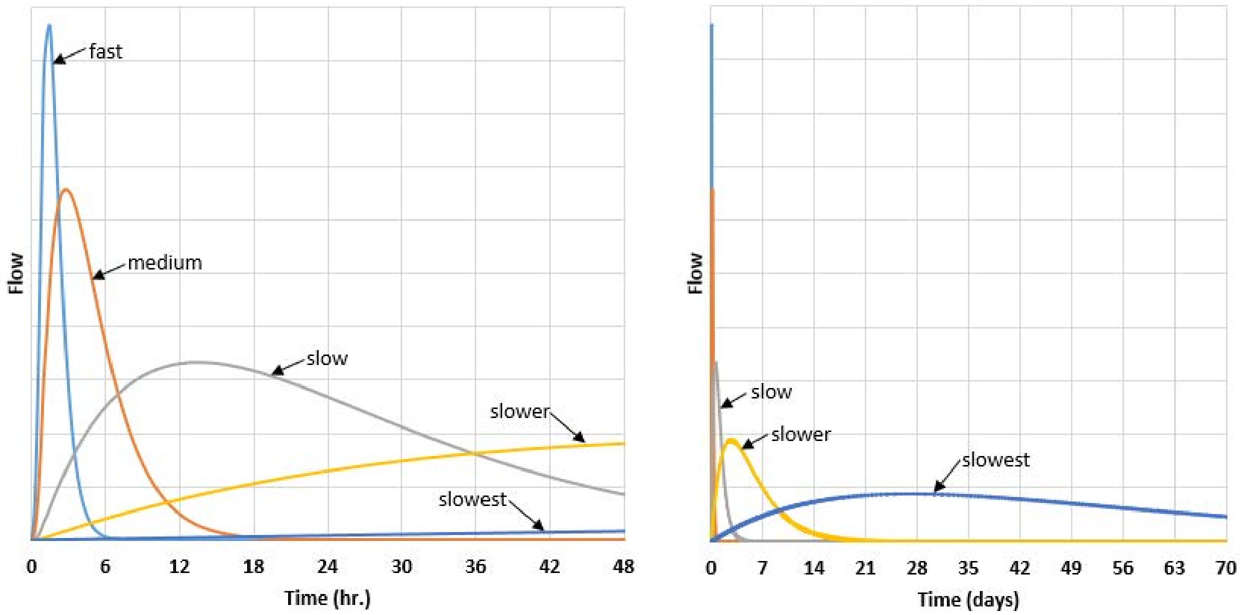
RDI/I flows result from rainfall events that produce infiltration and inflow of storm water runoff into the sewer system. RDI/I can be quantified as the difference between the total flow during and immediately following a storm event and the non-rainfall "base flow" (BWF plus GWI) that is estimated to have occurred during the storm period. The magnitude of the resulting RDI/I response is typically described by the percentage of the rainfall volume (called the "R value") represented by the volume of the RDI/I hydrograph. The R value can vary from storm to storm, depending on such factors as the degree of soil saturation (due to antecedent rainfall) prior to the storm event.

The shape of the RDI/I hydrograph is also important in determining the peak RDI/I response. The RDI/I hydrograph shape is often defined by separating the total RDI/I hydrograph volume into components, representing different response times to rainfall**.** Up to three or more response patterns may be used, as illustrated in **Figure 2-4**. The slowest component may result in a wet weather response several weeks or even months after the rainfall. Alternately, this component could be considered to be a gradual increase in GWI as a result of increased soil saturation and higher groundwater levels after storm events.

Summing all of the component hydrographs for the duration of the rainfall events results in the total RDI/I hydrograph for that area. In most sewer systems, the "fast" component of the hydrograph usually has the biggest impact on the magnitude of the peak wet weather flow response (although this was not always the case in the NSD system), while the slower components can contribute significantly to the total volume of the RDI/I response. These parameters, when applied to a different rainfall pattern, can be used to estimate the RDI/I response to that particular rainfall event.

As described in Chapter 3, R values and hydrograph parameters are determined through the process of model calibration, in which actual observed rainfall events are simulated in the hydraulic model, and the resulting model hydrographs are compared to the measured flows at flow meters locations. The RDI/I parameters are adjusted as needed to achieve the best match of modeled to monitored flows. Once calibrated, the model RDI/I parameters can be applied to a design storm to simulate wet weather flows for a design event.

Figure 2-4: Conceptual RDI/I Hydrograph Components

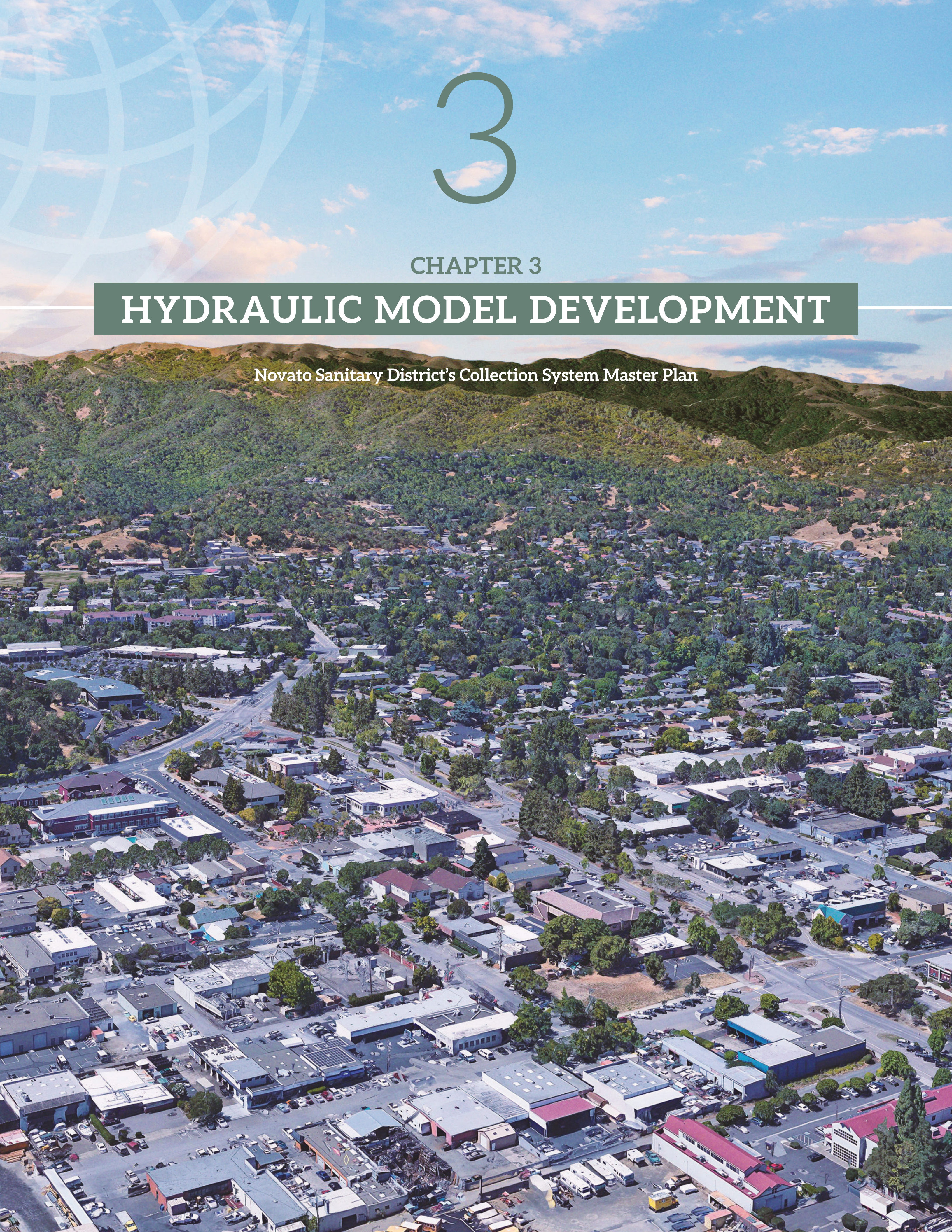


3

CHAPTER 3

HYDRAULIC MODEL DEVELOPMENT

Novato Sanitary District's Collection System Master Plan



Chapter 3 Hydraulic Model Development

This chapter of the report describes the process of developing and calibrating the hydraulic model for use in the capacity assessment, including the flow monitoring program that was conducted for this study.

3.1 Modeling Terminology

Network refers to the representation of the physical facilities being modeled. Components of the modeled network include nodes, links, and subcatchments, as discussed below.

Nodes are primarily manholes, but also include pump station wet wells, storage (such as the Ignacio Transfer PS equalization (EQ) basin), outfalls (discharge points from the modeled system) and breaks (changes in slope or diameter without a structure). Rim elevations are the key data associated with manholes, while wet well data also include the cross-sectional area and base elevation.

Links are connections between nodes, including conduits (pipes), pumps, and other control structures such as gates and weirs.

- **Conduits** are pipes, including gravity sewers, force mains, and siphons. Key data associated with conduits include upstream and downstream node IDs, pipe length, diameter, roughness factor, and upstream and downstream invert elevations.
- **Pumps** are represented by links connecting the wet well node(s) and the discharge manhole or node at the upstream end of the force main. Pumps are controlled by a number of user-defined parameters, including the switch-on and switch-off levels, types of pumps, and head-discharge curves.
- **Weirs** are walls within manholes that allow flow to overflow into adjacent pipes (e.g., relief lines). Weirs are represented by links in the model connecting the manhole in which the weir wall is located to a "dummy node" that represents the upstream end of the pipe into which the flow is diverted over the weir.

Subcatchments define areas that contribute flow to the modeled sewer network and represent the un-modeled sewers in the collection system. Data associated with subcatchments include base wastewater (sanitary) flow (computed based on population, water use, or other available data), type of land use (which defines the diurnal curves associated with the base wastewater flow), infiltration/inflow (I/I) parameters, and the node at which the flow from the subcatchment enters the modeled system.

Model loads are the flows input to the modeled sewers. Components of model loads are residential and commercial base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent I/I (RDI/I). As a sum, they represent the total wastewater flow applied to the model.

Models are the combination of a modeled network, its associated subcatchments and loads, and other data files (e.g., rainfall, diurnal curves, inflows from other areas, etc.) that comprise a specific model scenario.

3.2 Delineation of Modeled System

The hydraulic model developed for the NSD Master Plan represents the trunk sewer system, the network of larger diameter pipes that comprises the "backbone" of the system.

As described in Chapter 1, the NSD sanitary sewer collection system includes approximately 230 miles of sewers. Of the total sewer pipelines, about 20 percent or 47 miles is included in the modeled system. The modeled network includes almost all of the 10-inch and larger pipes, about 10 percent of the 8-inch pipes, and some 6-inch pipes. The model also includes six of the system pump stations, as listed in **Table 3-1**. The modeled network is shown in **Figure 3-1**.

Table 3-1: Modeled Pump Stations

Pump Station	Node ID	Type	No. of Pumps	Est. Firm Capacity (mgd)	Est. Total Capacity (mgd)
Ignacio Transfer PS	J22053P	All variable speed	3[a]	5.5[b]	
Olive Street PS	H15080P	All fixed speed	4	10.4	13.0
Bahia Main PS	L13050 P	All fixed speed	3	2.7[c]	2.9[c]
BMK #5 Main PS	K21010P	All variable speed	3	2.0	2.0[d]
East Hamilton PS	L26051P	2 fixed speed (dry weather), 2 variable speed (wet weather)	4	2.8[c]	2.9[c]
Marin Village PS	H19073P	All fixed speed	3	1.4	2.3

a. Not including pump dedicated to EQ Basin flow.
b. 5.5 mgd is the flow with 2 pumps at 100%. Flows greater than 5.5 mgd are diverted to storage.
c. Capacity calculated based on pump curves inferred from operating points (950 gpm at 160 ft. for Bahia Main and 1900 gpm at 157 ft. for the East Hamilton wet weather pumps).
d. BMK #5 is not currently capable of running all 3 pumps simultaneously, so total capacity is equal to firm capacity.

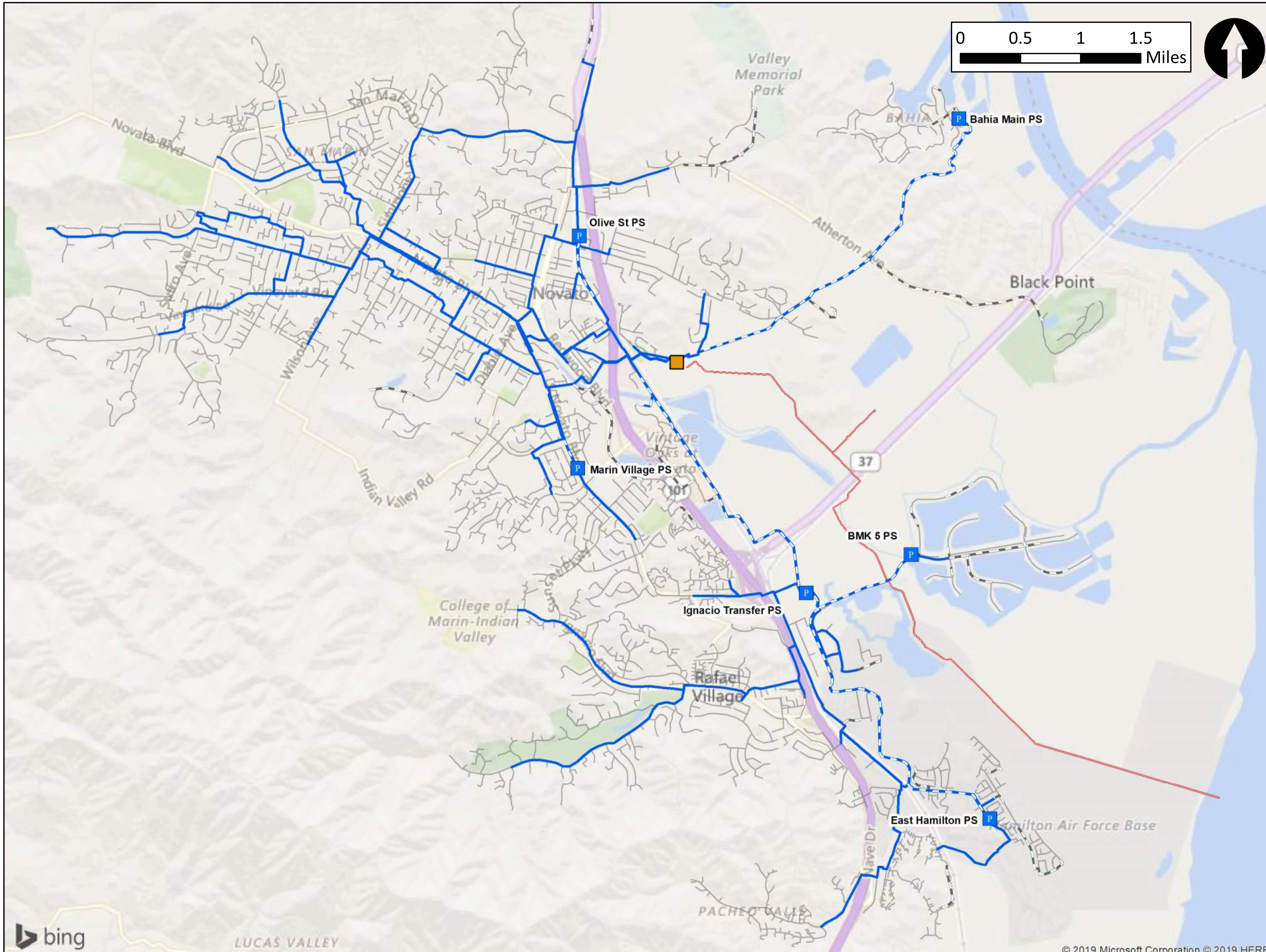
3.3 Model Network Construction and Validation

The model was developed using InfoWorks™ ICM (InfoWorks) software by Innovyze. InfoWorks provides a fully-dynamic solution for modeling sanitary sewer systems using the St. Venant equations, allowing accurate simulation of such hydraulic features and conditions as flow diversions, pump stations, sewer surcharge and backwater, and overflows. This type of model represents a significant improvement over the modeling software used in the District's previous master planning efforts.

The model was developed using the updated GIS files developed as part of this project with manhole and pipe attribute data (manhole coordinates and rim elevations; pipe diameters, lengths, and invert elevations). The GIS data were imported into InfoWorks, and the pipes and manholes in the trunk system were extracted to create the model network.

Model construction and validation includes the following:

- **Connectivity checks.** The modeled network was checked for connectivity, which includes verifying that correct upstream/downstream manholes were identified for each pipe, with no missing links or nodes in the network. A connected network means that all pipes and manholes will be selected when the network is traced from downstream to upstream and vice-versa.
- **Missing data checks.** Key data required for modeling were reviewed to identify missing values. Missing data were inferred where reasonable (e.g., where one or two invert elevations were missing between populated values, the data could be interpolated). The remaining missing data were referred back to the District, and much of it was populated based on available record drawings.
- **Profile review.** Profiles were plotted for each series of pipe segments in the modeled network to visually check for suspect data. Examples of suspect data include negative pipe slopes, abrupt steps up or down in pipe inverts, and pipe diameters that conflict with surrounding pipes. Where appropriate, suspect data were inferred or adjusted based on estimated datum differences. Otherwise, verification in the form of more detailed review of AutoCAD mapping or available record drawings provided by the District was conducted.



Legend

- Novato WWTP
- Modeled Pump Station
- Modeled Force Main
- Modeled Trunk Sewer
- Unmodeled Force Main
- Unmodeled Sewer
- WWTP Outfall

Novato Sanitary District

Collection System Master Plan

Figure 3-1

Modeled Trunk System

\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure3-1_Modeled_Trunk_System.mxd

Sources: Novato Sanitary District, Bing Maps

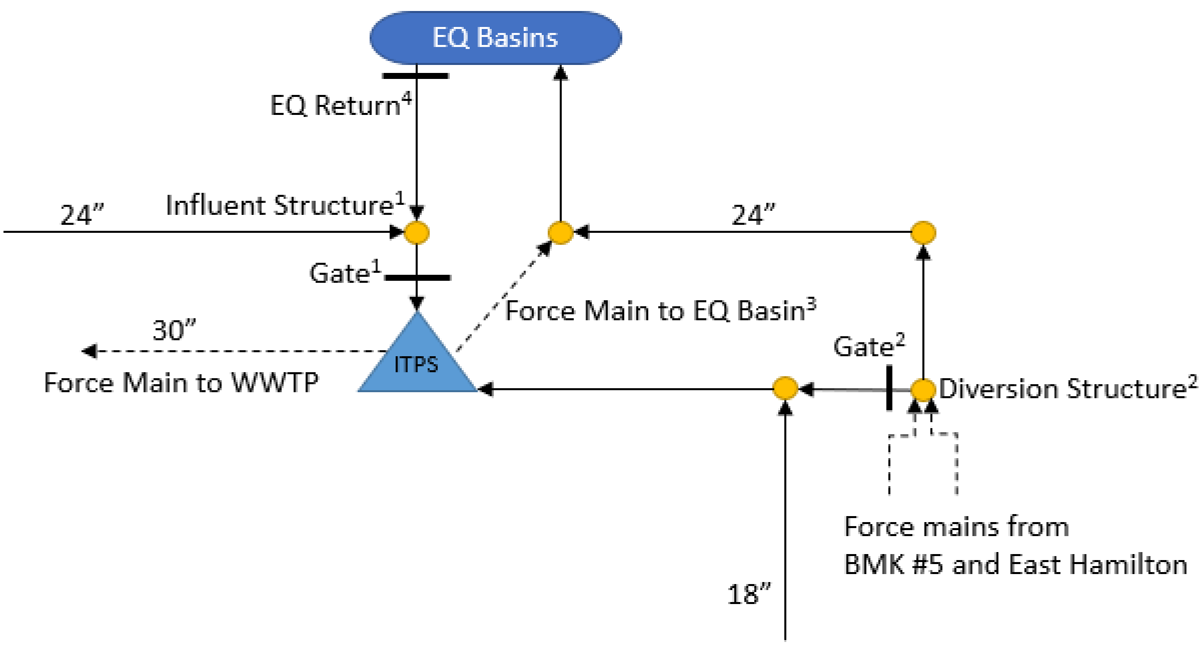
Map Created: 1/30/2019

- **Special structures.** Flow splits (manholes with more than one outlet pipe) were identified for further verification of outlet pipe elevations and/or the existence of weir overflows or other control structures. Field verification and/or record drawings were requested from the District as needed.

3.4 Ignacio Transfer Pump Station Operation

The Ignacio Transfer Pump Station (ITPS) was constructed in 2006 at the site of the former Ignacio treatment facility to convey wastewater previously treated at that facility to the new main WWTP. The station receives flow from several trunk sewer pipelines, including a 30-diameter trunk sewer from the west, which serves much of the southern portion of the service area; an 18-diameter gravity sewer from the Bel Marin Keys business park area; and force mains from the East Hamilton and BMK#5 Pump Stations. Due to pumping limitations, the ITPS utilizes existing storage basins on the site to store wastewater during peak wet weather flow periods. The typical process and the components involved are depicted in **Figure 3-2**.

Figure 3-2: Ignacio Transfer Pump Station Operation



Steps typically performed as flow increases up to and above 5.5 mgd.

1. Influent structure gate closes to transition PS to manual operation. Influent gate is then slowly throttled back open to relieve upstream surcharge
2. Diversion structure gate is closed to divert force main flows to EQ Basin
3. Flows above 5.5 mgd are pumped by the EQ Pump to the EQ Basin.
4. When flows recede, steps above are reversed, then flows are returned from EQ Basin.

3.5 Load Manholes and Sewer Subcatchments

Flows are loaded into the model at "load manholes," which are typically the manhole where flows from unmodeled sewers discharge into the modeled network. "Subcatchments" are the polygon areas delineated to show the areas contributing flows to the load manholes. The NSD model includes approximately 670 subcatchments. An example zoomed-in area showing sewer subcatchments and load manholes is shown in **Figure 3-3.**

Figure 3-3: Example Sewer Subcatchments

3.6 Flow Monitoring Program

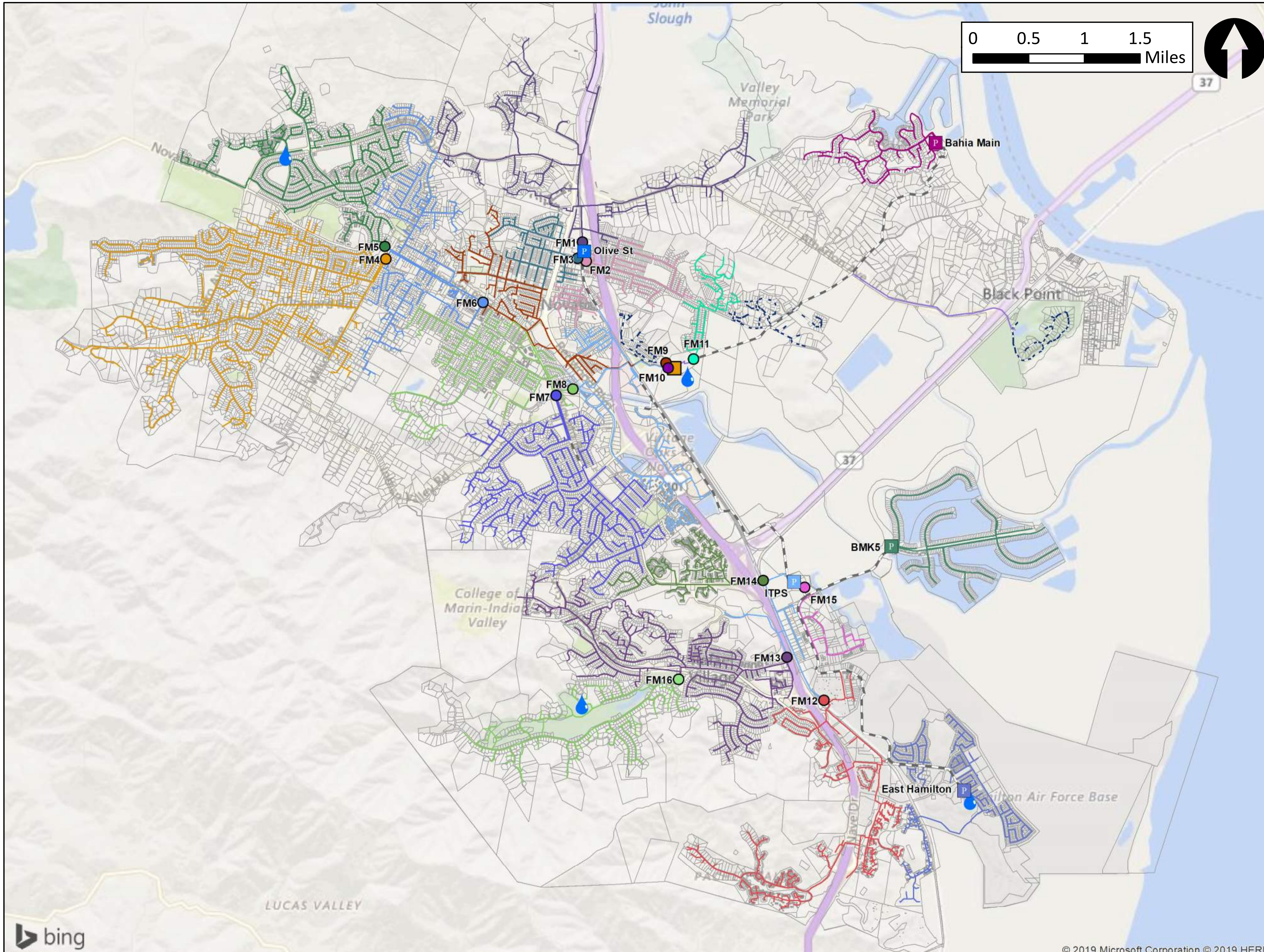
A temporary flow monitoring program consisting of 16 flow meters and 4 rain gauges installed in the NSD system during the period December 9, 2016 through February 14, 2017 provided data for calibration of the hydraulic model. The monitoring was conducted by ADS Environmental Services as subconsultant to Woodard & Curran.

The locations of the flow meters and rain gauges are listed in **Table 3-2** and shown in **Figure 3-4**, and a flow meter schematic is shown in **Figure 3-5**. Note that the District installs temporary plugs in the system at strategic locations during wet weather periods. The plugs back up flow in order to divert flow to other sewers, thus preventing overflows. Two such plugs were installed during the monitoring period, as indicated on Figure 3-5. In addition to the temporary flow meters, the District provided flow data from its SCADA system for the Ignacio, Olive Street, Bahia Main, BMK #5, and East Hamilton Pump Stations, as well as the WWTP. Figure 3-3 shows the sewers tributary to each meter and pump station.

Table 3-2: Flow Meter and Rain Gauge Locations

Meter/ Rain Gauge ID	Meter Manhole	Upstream Manhole	Diam. (in.)	Location
Flow Meters				
FM1	H14030	H14031	24	Plum Ave. north of Olive Ave.
FM2	H15085	H15087	24	Plum Ave. at Olive Ave.
FM3	H15004	G15001	15	Olive Ave. west of Plum Ave.
FM4	E15099	E15144	24	Wilson Ave. at Norma Ct.
FM5	E14082	E14081	15	Reservoir Ln. at Novato Blvd.
FM6	F16001	G16009	24	Novato Blvd. near Twin Creek Ct.
FM7	G18070	G18071	18	S Novato Blvd. south of Lauren Ave.
FM8	G17120	G17069	36	Easement behind Lauren Ave.
FM9	I17003	I17004	30	west of WWTP
FM10	I17013	I17014	48	west of WWTP
FM11	I17018	I17019	15	east of WWTP
FM12	J24004	K24001	15	Roblar Dr.
FM13	J23005	J23006	18	Entrada Dr.
FM14	J22015	J22039	15[a]	Bike path west of Ignacio PS
FM15	J22026	J22027	10	Bel Marin Keys Blvd. at Frosty Ln.
FM16	I24017	I24018	10	Fairway Dr. south of Ignacio Blvd.
Rain Gauges				
RG1				Marin Country Club
RG2				East Hamilton PS
RG3				Fire Station No. 3 (San Marin Dr.)
RG4				Wastewater Treatment Plant

a. Pipe measured as 13" high by 14" wide



Legend

- Flow Meter
- Novato WWTP
- Rain Gauge
- Metered Pump Station
- Force Main
- Meter Tributary Sewer
- Unmetered Sewer

Novato Sanitary District

Collection System Master Plan

Figure 3-4

Flow Meter and Rain Gauge Locations

NOVATO SANITARY DISTRICT NSD

WOODARD & CURRAN

\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure3-4_FM_Rain_Gauge.mxd

Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/30/2019

Figure 3-5: NSD Flow Meter and System Schematic

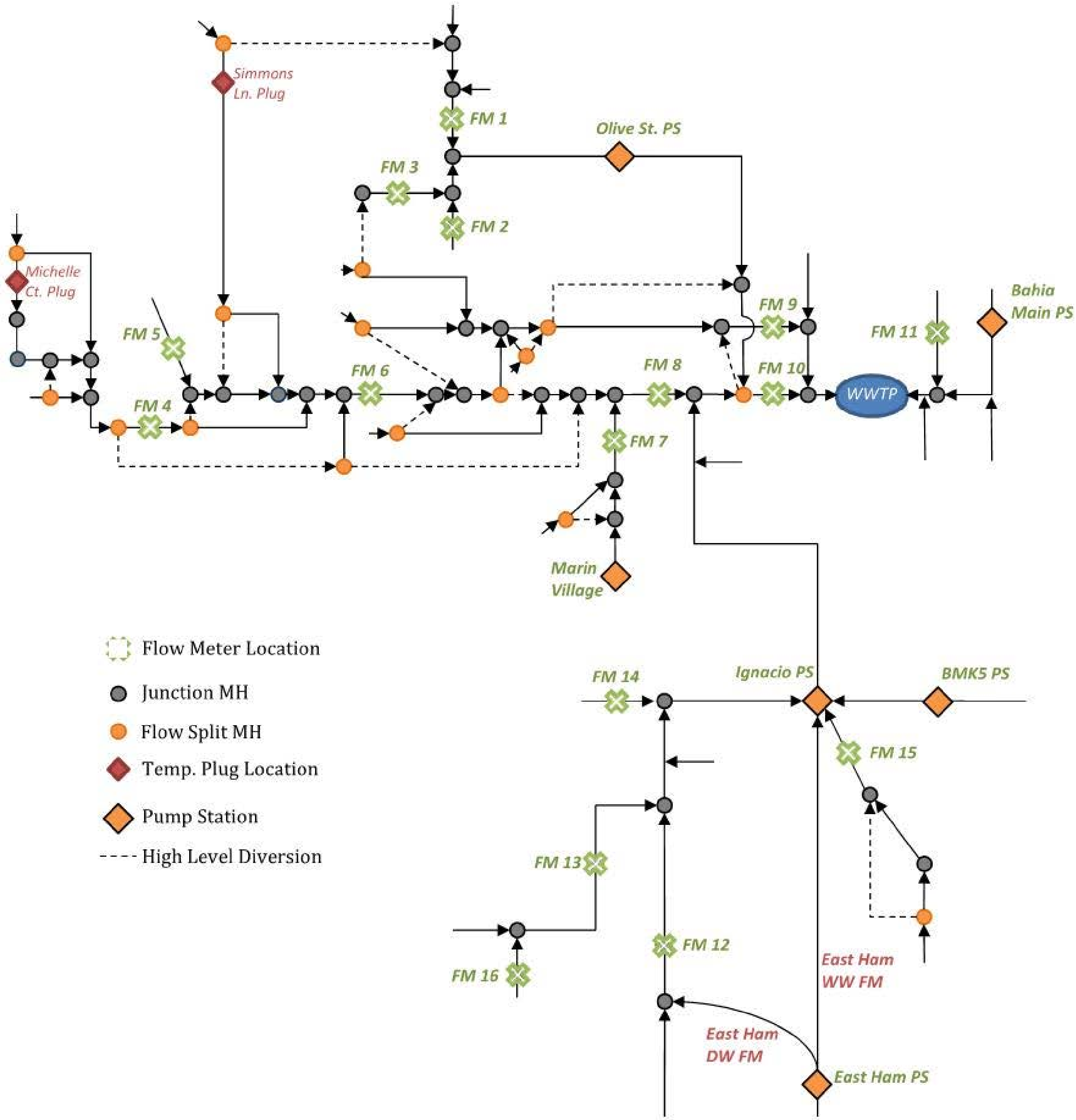


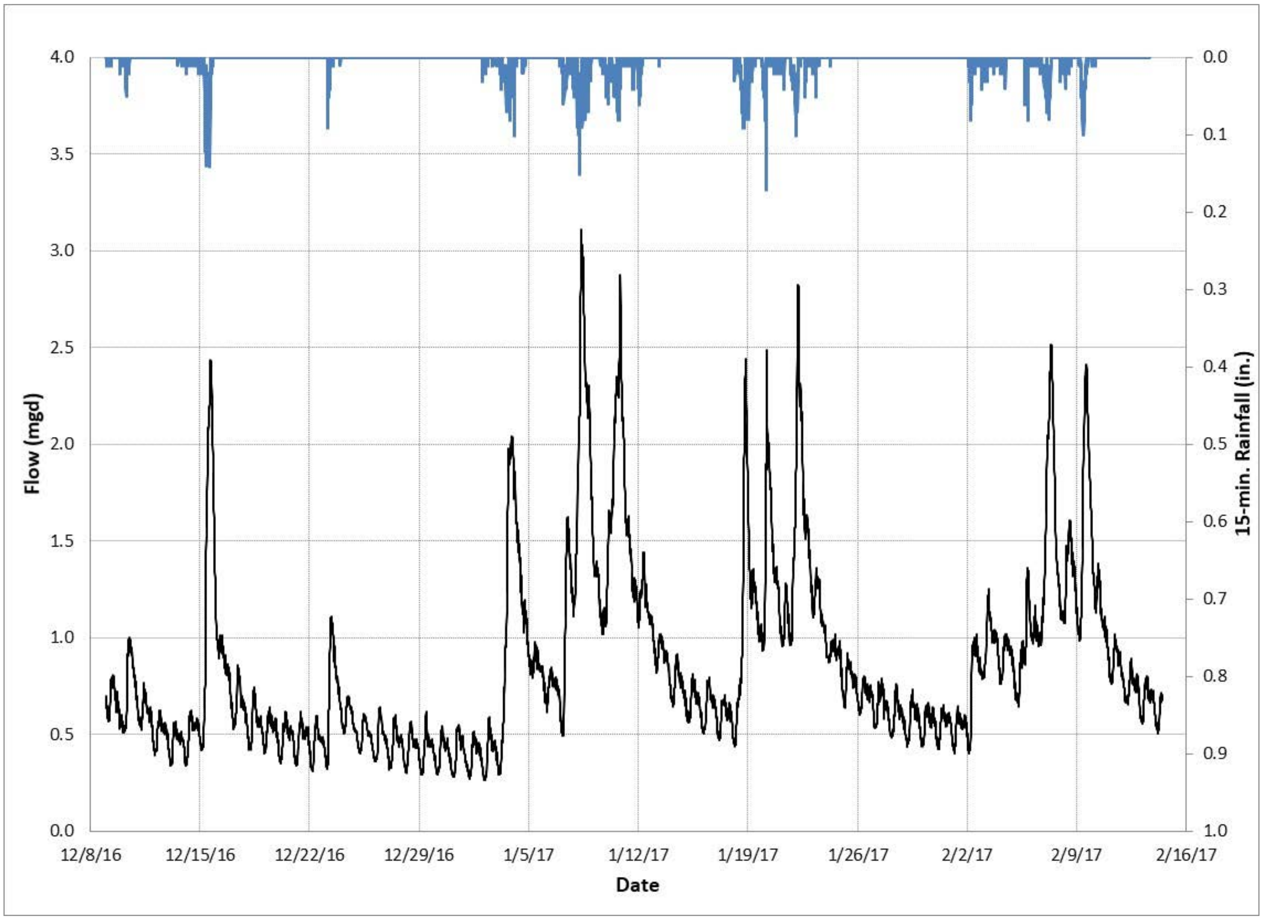
Table 3-3 lists the rainfall events that occurred during the flow monitoring period, and **Figure 3-6** shows a graph of the flow recorded at a typical meter. The graph shows the flow response to the large storm events in mid-December and to the later storm periods in January and early February. Plots of the flow data for all of the monitoring sites are included in **Appendix B** of this report.

Table 3-3: Rain Events During Flow Monitoring Period

Start Date/Time	Duration (hrs)[a]	Storm Rainfall (in.)				Max. Hour (in.)			
		RG1	RG2	RG3	RG4	RG1	RG2	RG3	RG4
12/8/16 15:00[b]	47	1.03	0.66	0.78	0.41	0.22	0.15	0.13	0.09
12/13/16 20:00	50	3.89	3.15	1.96	2.73	0.53	0.55	0.26	0.36
12/23/16 4:00	14	0.78	0.87	0.75	0.68	0.23	0.28	0.21	0.21
1/2/17 17:00	52	3.21	2.88	1.92	2.61	0.24	0.30	0.16	0.22
1/6/17 23:00	50	5.38	4.56	4.10	4.53	0.37	0.37	0.49	0.46
1/9/17 16:00	63	4.75	4.19	3.72	3.51	0.32	0.43	0.30	0.25
1/18/17 4:00	35	2.75	2.95	1.94	2.09	0.39	0.46	0.37	0.31
1/19/17 21:00	37	1.82	1.85	1.63	1.56	0.37	0.46	0.40	0.43
1/21/17 17:00	15	2.07	2.71	2.10	1.77	0.47	0.61	0.36	0.32
1/22/17 15:00	23	0.48	0.42	0.48	0.38	0.20	0.17	0.15	0.12
2/2/17 2:00	58	ND	2.15	1.77	1.61	ND	0.33	0.14	0.20
2/5/17 16:00	44	ND	3.33	2.69	2.64	ND	0.32	0.28	0.27
2/7/17 23:00	18	ND	0.65	0.69	0.58	ND	0.08	0.14	0.08
2/9/17 4:00	21	ND	1.79	1.61	1.59	ND	0.32	0.29	0.29
Total Monitoring Period		26.3	32.3	26.4	26.9	0.53	0.61	0.49	0.46

a. Storm durations reflect range of all gauges; storm durations for individual gauges may be slightly shorter or longer. Rain events generally defined by separation of at least 8 to 12 hours of no rainfall.
b. Storm event rainfall started on 12/7 prior to rain gauge installation.
RG1 – Marin Country Club
RG2 – East Hamilton PS
RG3 – Fire Station No. 3 (San Marin Dr.)
RG4 – Wastewater Treatment Plant
ND – no data due to clogged gauge.

Figure 3-6: Example Flow Monitoring Data (Meter FM1)



3.7 Development of Model Loads

As noted previously, flows are loaded into the model at "load manholes," each of which represents the point where flows from the unmodeled sewers discharge into the modeled network. The unmodeled sewers were grouped into sewer subcatchments, each with a unique load manhole in the network.

Chapter 2 described how BWF model loads were developed from water use and land use and growth projections. GWI and RDI/I flows were also loaded to the model by subcatchment by associating each subcatchment with a flow meter area. For each subcatchment, the total sewered area (i.e., area that potentially contributes I/I) was determined by summing the acreage of non-open space land uses (sum of parcel sizes up to maximum of 2 acres per parcel) in the subcatchment. Non-open space land uses include residential, commercial, industrial, and institutional uses, but do not include parks, drainage channels, and large roadways such as freeways. I/I flows for each subcatchment were computed in the model by applying the appropriate meter area GWI and RDI/I parameters (determined during the model calibration process described below) to the sewered area of the subcatchment.

3.8 Model Calibration

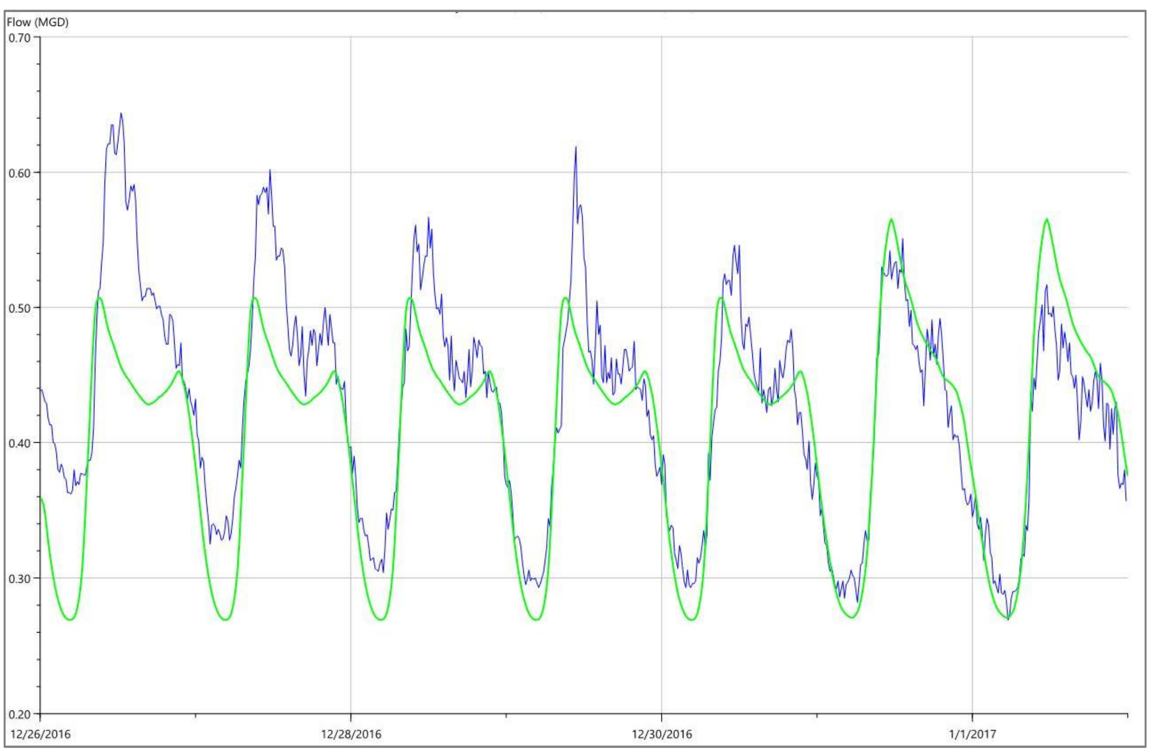
Model calibration is the process of comparing model-computed flows to observed (monitored) flows to verify that the model is accurately simulating flows in the sewer system. As described above, a temporary flow monitoring program was conducted in the NSD system during the December 2016 through February 2017 wet weather period. The data collected during the flow monitoring program were used for model calibration. During calibration, it is not expected that every meter match flow data at all times, but most meters should reasonably match the flow volumes and peak flows. Model calibration is achieved first through comparison of modeled versus metered flows during a dry weather (non-rainfall) period to achieve an accurate prediction of BWF and GWI, then during the wet weather period to estimate RDI/I response.

3.8.1 Dry Weather Calibration

Dry weather calibration for this study was challenging due to the large amount of rainfall that fell during the flow monitoring period. The longest "dry" period occurred in late December, just prior to the large storm event of January 2. Therefore the 7-day period from December 26 through January 1 was used as the dry weather calibration period for the model. While not ideal because flow patterns during the holiday season are often different than normal patterns, this was the only period that was not heavily impacted by prior rainfall. However, for many of the meters, the flow was still receding from the mid-December storms during the first few days of the calibration period. As noted above, a series of large storms hit the region during January and early February. In most cases, the flows during the days and weeks after these storm events never receded to pre-storm dry flow levels.

Figure 3-7 shows an example plot of model vs. metered flow for one meter location (Meter FM1). In this graph, the blue line represents the monitored (observed) flow, and the green line is the model-simulated flow. As noted above, the flow on the first few days of this period was still elevated due to the prior rainfall, but the modeled flow matches the observed flow fairly well for the last few days.

Figure 3-7: Example Dry Weather Calibration Graph (Meter FM1)



The primary focus of the dry weather calibration was to confirm that the calculated average BWF based on winter water consumption was consistent with the measured flows at the meter locations. The other objectives of the dry weather calibration were to confirm the flow routing in the system, particularly in areas where flow can be diverted in more than one direction (flow splits), as well as to confirm the diurnal profiles used to represent the hourly variations in BWF.

The dry weather calibration process was also used to quantify GWI (as indicated by monitored flows that were higher than estimated BWF). GWI was added when the observed (metered) dry weather hydrographs were greater than the model-simulated hydrographs by a relatively constant value throughout the day. GWI was applied in most of the flow meter areas with rates ranging from less than 100 gpd/acre to about 1,600 gpd/acre, with an overall, systemwide average of 290 gpd/acre or 1.39 mgd total. Note this value represents a wet season GWI following initial winter season rains. It should also be noted that it may be difficult to assess the actual amount of GWI, as the relative accuracy of the flow monitoring data, water consumption data, and other model assumptions will affect the amount of flow attributed to GWI. However, this methodology is considered adequate for modeling purposes.

The model calibration resulted in a reasonably good match of modeled to metered flow at most locations, but some differences at others. These differences may be due to inaccuracies in the meter data or in the configuration of the system (e.g., upstream flow splits). Every effort was made to resolve as many of the calibration differences as possible. However, the overall match between modeled and observed flows at the WWTP was very good (within 8 percent). **Appendix C** contains the dry weather calibration graphs for all of the meters.

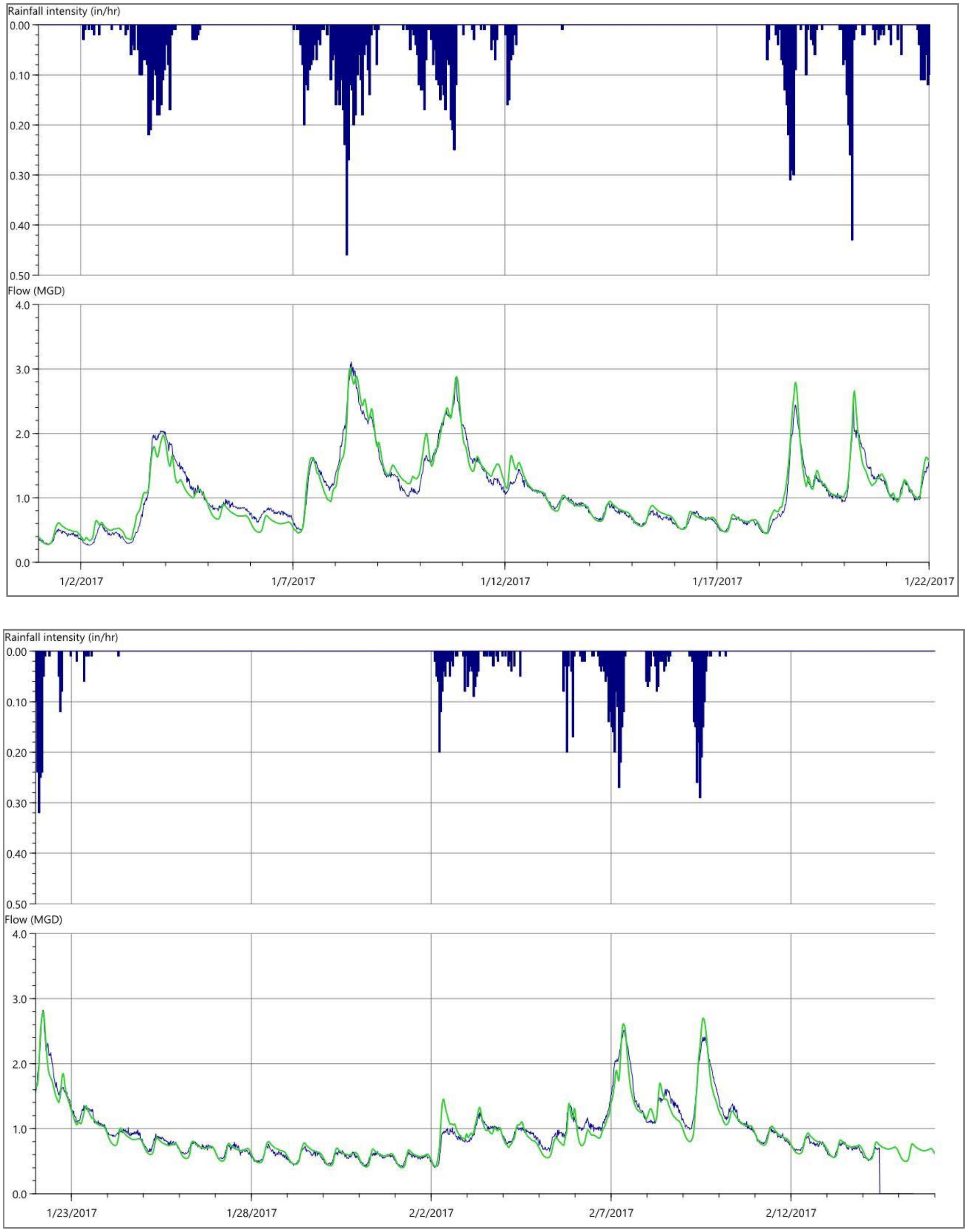
3.8.2 Wet Weather Calibration

The January and February 2017 storm periods were used for model wet weather calibration. During wet weather calibration, the percentage volume of each of five RDI/I component parameters, as illustrated previously in Figure 2-4, were adjusted to simulate the volume and timing of RDI/I for monitored storm events in order to best match the overall wet weather hydrograph shape and magnitude of peak flows. Rainfall was assigned to subcatchments using data from the closest of four rain gauges maintained by ADS during the monitoring period. Note that the slowest RDI/I hydrograph component generally resulted in wet weather response up to several weeks after the rainfall. Alternately, this component could be considered to be a gradual increase in GWI as a result of increased soil saturation and higher groundwater levels after storm events.

Figure 3-8 shows an example set of wet weather flow calibration graphs for Meter FM1 for the January/February 2017 wet weather period. **Appendix D** contains calibration graphs for all of the meters. Based on the wet weather calibration, RDI/I parameters (R values for each hydrograph component) were determined for each flow meter area. These parameters were applied to a design storm event (as discussed in Chapter 4) to simulate the flows in the system under design storm conditions.

Storage at ITPS was utilized during the larger rainfall events of the flow monitoring period, resulting in some backup surcharge into the upstream flowmeters during the transition period (i.e. before the storage pump was started). A simplified version of these operating procedures was applied to the model to represent this backup surcharge; however, since the operation is performed manually it is not expected that the model will accurately predict depths at all times. Reviewing the model predicted versus observed results, the model reasonably predicts the depth and duration of surcharge during most of the large events, in addition to flows.

Figure 3-8: Example Wet Weather Calibration Graph (Meter FM1)



4

CHAPTER 4

CAPACITY ANALYSIS

Novato Sanitary District's Collection System Master Plan

Chapter 4 Capacity Analysis

This chapter describes the hydraulic analysis and design criteria used to evaluate system performance and size capacity relief projects, the capacity deficiencies based on the results of model runs, and recommended solutions to identified capacity deficiencies.

4.1 Design Event Criteria

Peak design flows for sewer systems consist of dry weather base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I). Criteria for computing existing BWF, GWI, and RDI/I (developed as part of model calibration), and unit flow assumptions for future development were discussed in the previous chapters. However, the peak design flow criteria must also specify the set of conditions (e.g., design storm rainfall and timing with respect to seasonal GWI and diurnal BWF) that will generate the highest peak flows that the sewer system must be capable of hydraulically conveying.

The following subsections discuss the design event used in this Master Plan. Key factors needed to define a design event include the storm return period and duration, rainfall amount and distribution, storm timing, and antecedent conditions. These factors are discussed below.

4.1.1 Return Period

The return period defines the probability that the design rainfall will be exceeded in any given year. For example, a storm with a 5-year return period means there is a 1 in 5 chance, or 20 percent probability, of exceeding the design rainfall in a given year. The chosen return period reflects the degree of risk an agency will tolerate of experiencing sanitary sewer overflows (SSOs) due to future storm events. However, choosing a design storm with a very high return period (reflecting a very low risk tolerance) could lead to the identification of so many system capacity deficiencies that the cost of improvements is prohibitive. Additionally, sizing a system for a very rare event could mean that the system does not function well under typical conditions during much lower flows (due to slow velocities in oversized pipes, or oversized pump stations).

Although there is no regulatory standard for design return periods for wastewater collection systems, the majority of San Francisco Bay Area agencies that have adopted a specific return period have selected return periods of 5 or 10 years. Some of the storms chosen by these agencies are historical rainfall events; others are synthetic storms. The District's previous master planning analyses were based a 5-year return period design storm. As discussed further below, depending on the type of rainfall distribution chosen, although the total storm rainfall will be the same for storms of the same stated return period and overall duration, shorter durations within each storm may or may not be an equivalent return period.

4.1.2 Duration

A storm duration must be specified along with the return period. Most Bay Area agencies use a 24-hour storm, though shorter or longer durations may sometimes be appropriate, e.g., a shorter duration in a very small system with fast response to rainfall or in an area where storm events are typically very brief in duration; a longer duration in a very large system or one with a very slow response to rainfall.

4.1.3 Rainfall Amount and Spatial Distribution

A design storm may be a synthetic storm or an actual historical event. A historical event is most beneficial as a design storm when a long enough record of both sewer flow data (for instance at a wastewater treatment plant) and hourly rainfall data exist to identify the types of rainfall events that tend to generate the highest flows in the sewer system. In general, the NSD WWTP does not have sufficient historical hourly data available to identify a potential historical rainfall event that could be used as a design event, as flows to the

WWTP prior to the construction of the new facility were heavily impacted by upstream conveyance system limitations.

Synthetic design storms are typically based on rainfall depth-duration-frequency (DDF) or intensity-duration-frequency (IDF) statistics that have been compiled for a local area. These statistics give the rainfall depths or intensities for various return periods (e.g., 2-year, 5-year, etc.) and durations (e.g., 1-hour, 2-hour, etc.). A standard source of IDF statistics is NOAA Atlas 14, Volume 6[2], which provides IDF statistics for any location in the U.S. based on latitude/longitude coordinates. **Table 4-1** summarizes the 1-, 6-, and 24-hour rainfall for the NSD service area (approximate downtown Novato location) for return periods ranging from 1 to 25 years, based on NOAA Atlas 14 data. Note that the exact rainfall amounts may vary spatially based on location within the District, with generally higher amounts in the west and lower in the east. For example, the NOAA Atlas 14 24-hour rainfall for a 5-year return period varies from about 3.5 to 4.6 inches over the service area.

Table 4-1: NOAA Atlas 14 Rainfall for NSD

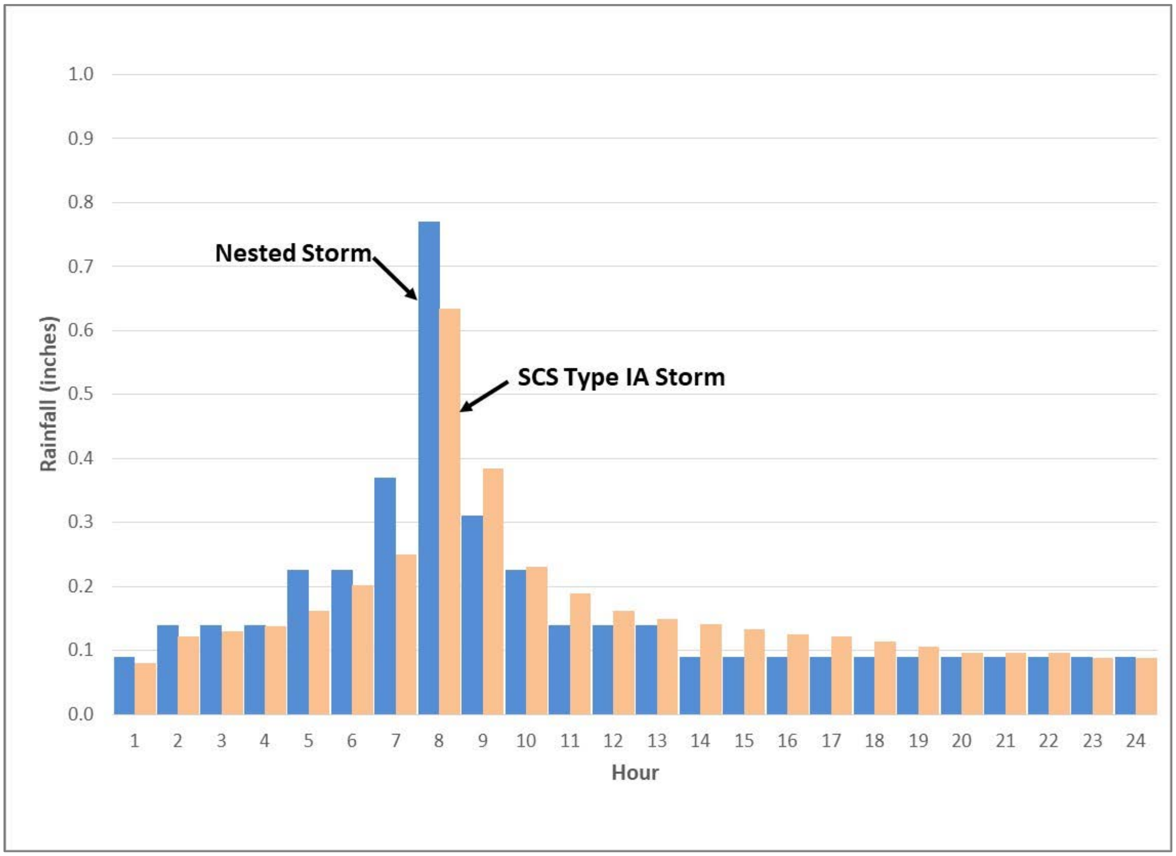
Return Period	Design Rainfall (in.)		
	1-hour	6-hour	24-hour
1-year	0.50	1.41	2.52
2-year	0.62	1.72	3.18
5-year	0.77	2.13	4.04
10-year	0.90	2.46	4.74
25-year	1.10	2.92	5.68

4.1.4 Rainfall Temporal Distribution

Commonly used synthetic storm temporal distributions include nested storms (which include the rainfall amounts for the storm return period for all durations within the total storm duration) or the 24-hour “SCS Type IA” storm distribution, a general distribution characteristic of northern California coastal areas, as defined by the USDA guidance document Urban Hydrology for Small Watersheds TR-55 (June 1986). Nested storms may be considered conservative, as the peak intensities for all durations may not necessarily occur in the same storm event. The peak intensity for a nested synthetic storm tends to be higher than the peak intensity in the SCS Type IA distribution, as illustrated in **Figure 4-1** below for a 5-year return period storm for NSD. Note that for either distribution, the timing of the peak rainfall can be adjusted to occur at any hour of the day, depending on the desired timing of the storm with respect to the diurnal base wastewater flow (see discussion of Storm Timing below). The peak hour of the nested storm could also occur at any hour within the 24-hour period.

[2] U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, NOAA Atlas 14, Volume 6 Version 2.3: California, 2011, Revised 2014

Figure 4-1: Nested vs SCS Type IA 5-Year, 24-Hour Design Storm



4.1.5 Storm Timing

The timing of the rainfall distribution compared to the sewer flow dry weather diurnal profile should also be considered. For example, rainfall can be timed to generate RDI/I that peaks roughly at the same time as the dry weather flow diurnal peak (referred to as "peak-on-peak" timing). This consideration is most important in systems where flow due to RDI/I is relatively small compared to BWF and in systems where the response to rainfall occurs relatively quickly (over hours instead of days). Alternately, a less conservative approach would be to time the rainfall such that the peak RDI/I occurs under average BWF conditions, based on the assumption that the design storm could occur at any time of day. This approach can be applied when using a synthetic design storm distribution (nested or SCS).

4.1.6 Antecedent Conditions

As discussed in Chapter 3, flow response to rainfall in the NSD service area is characterized by prolonged elevated flows extending for several days or weeks after storm events. Thus, a storm falling later in the season after several large events have occurred will produce higher peak flows because the ground is saturated (meaning less rainfall can be "absorbed" by the soil) and flows are already elevated due to preceding events. This antecedent condition can be modeled as a prolonged RDI/I response (when the model simulation includes multiple events) or as an elevated, antecedent GWI condition at the beginning of the event (when modeling a single event). Although it is not unusual for large storm events to occur after periods of preceding rainfall, assuming maximum GWI conditions for the design event would be considered conservative (i.e., would tend to make the peak *flow* return period less frequent than the rainfall return period), especially for a system such as NSD that already experiences high RDI/I response. Assuming moderately wet soil conditions as were observed at the beginning of the flow monitoring period would be a less conservative approach.

4.1.7 Recommended Design Event for NSD

Based on the above discussions, the following design event parameters have been adopted for this Master Plan:

- Storm return period — 5 years
- Storm duration — 24 hours
- Rainfall amount — Per NOAA Atlas 14, including spatial variation
- Rainfall temporal distribution — SCS Type IA
- Storm timing — "peak-on-peak"
- Antecedent conditions — average winter soil conditions (similar to early Jan. 2017 period)

4.1.8 Future Conditions

It is not known if I/I will increase over time due to further deterioration of the sewer system, and/or whether the District's current and future sewer rehabilitation efforts will be sufficient to prevent such increases. However, studies to assess such changes in other systems throughout the country have not produced conclusive results that such increases will occur. Furthermore, new sewer construction and replacement will primarily use more watertight pipe materials that will minimize any new sources of I/I. Therefore, this Master Plan assumes that I/I in the existing system will not increase in the future and that any increases due to new sewer construction to serve future development will be minimal.

4.2 Hydraulic Capacity Criteria

Hydraulic capacity criteria determine which sewer pipes should be relieved or replaced due to inadequate capacity to convey existing or future peak flows, and how large the new sewers should be. The criteria adopted by the District should ideally be stringent enough to ensure that sewer overflows caused by capacity limitations (as distinguished from other causes such as obstructions or structural failures) are very rare occurrences, but not so conservative that they cause the District to spend capital improvement funds unnecessarily or result in pipes that are so large that cleaning velocities cannot be achieved under normal flow conditions, causing solids build up and related problems.

Capacity deficiency criteria identify the need to replace an existing facility, while design criteria determine the sizing of new facilities. These criteria are discussed in the paragraphs below.

4.2.1 Capacity Deficiency Criteria

Capacity deficiency criteria are used to determine if the capacity of an existing sewer facility is exceeded to the extent that a capacity relief project is needed. These criteria are sometimes called "trigger" criteria, in that they trigger the need for a capacity relief project, unlike design criteria that are applied to determine the size of a new facility. The difference between deficiency criteria and design criteria reflect the fact that some existing facilities can continue to provide adequate, if not optimal, conveyance capacity, but new facilities should be designed to a higher standard.

It is important that the capacity deficiency criteria be coordinated with the peak design flow criteria. For example, if the peak design flow were to be based on only peak dry weather flow (PDWF), the deficiency criteria would need to be conservative (e.g., require pipes to flow less than full to allow capacity for I/I). On the other hand, if the peak design flow includes I/I from an infrequent storm event, it may be appropriate to allow the sewers to flow surcharged to some extent, since the peak flows will be infrequent and relatively brief in duration.

For this Master Plan, a capacity deficiency was identified under the following conditions:

- Any modeled surcharging under PDWF
- Any modeled overflows or surcharge reaching within 2 feet of manhole rims under design storm PWWF

Note: A Manning's 'n' of 0.013 is assumed for the capacity evaluation of existing sewers.

For master planning, pump stations typically are considered capacity deficient if the peak design flow exceeds the station's estimated firm capacity (capacity with largest pump out of service). Force mains are considered to be deficient if velocity under peak design flow exceeds 8 to 10 feet per second (fps).

4.2.2 Design Criteria for New Sewer Facilities

The District's design standards specify criteria for hydraulic design of sewer mains and associated facilities. The following criteria have been adopted for design and sizing of gravity trunk sewer capacity projects for this Master Plan:

- Maximum allowable depth-to-diameter ratio (d/D) of 1.0 (full pipe) under modeled future PWWF (design storm conditions), assuming no downstream flow restrictions (i.e., pipe will be sized to convey the modeled peak flow, not to store backwater from downstream restrictions)
- Manning's 'n' of 0.013
- Minimum pipe size of 8 inches
- For new pipes (new alignment)
    - Minimum slope to achieve velocity of 2 fps at peak dry weather flow, if feasible
    - Minimum cover of 4.5 feet in streets or 3.5 feet in easements
    - Maximum manhole spacing of 350 feet
- For existing alignments (upsizing pipe), the existing pipe slope, invert elevations, and manhole locations are assumed
- New pipe material PVC (HDPE for pipe bursting)

4.3 Capacity Analysis Results and Preliminary Solutions

Based on the criteria described above, the hydraulic model was run for conditions representing combinations of existing or future development and dry weather flow or design storm wet weather flow. **Table 4-2** summarizes the results of these model runs, indicating the resulting peak flow to the WWTP and the volume of model-predicted overflows, and **Table 4-3** summarizes the flows to the modeled pump stations. **Figure 4-2** shows the model results for future PWWF conditions (results for existing PWWF conditions are very similar), indicating pipes that are predicted to be surcharged due to "throttle" (peak flow exceeding full pipe capacity) or due to backwater from a downstream throttle condition, plus any model-predicted overflows. Under future PDWF conditions, no sewers were found to be surcharged. The areas of capacity deficiencies (throttle conditions resulting in model-predicted overflows or freeboard less than 2 feet) are identified by a capacity deficiency ID on Figure 4-2 and described in **Table 4-4.**

Preliminary solutions for the identified capacity deficiencies (upsizing of capacity-deficient pipes) were developed in order to assess the potential extent of required capacity improvements and to estimate the resulting peak flow to the WWTP once capacity deficiencies are relieved. Table 4-2 also indicates the resulting peak flow for the preliminary "solutions" model run for future wet weather conditions, and Table 4-4 describes the preliminary solutions (diameter and length of required upsized pipe). (Note that in some cases, providing relief for existing capacity deficiencies resulted in higher peak flows being conveyed downstream, resulting in need for additional improvements. In the case of NSD, the need for an additional capacity improvement project in Novato Boulevard was triggered by relieving the capacity deficiencies shown in Table 4-4.) As discussed below, more detailed analyses of potential alternative solutions were conducted on a project-specific basis in order to identify the most feasible projects from a constructability standpoint.

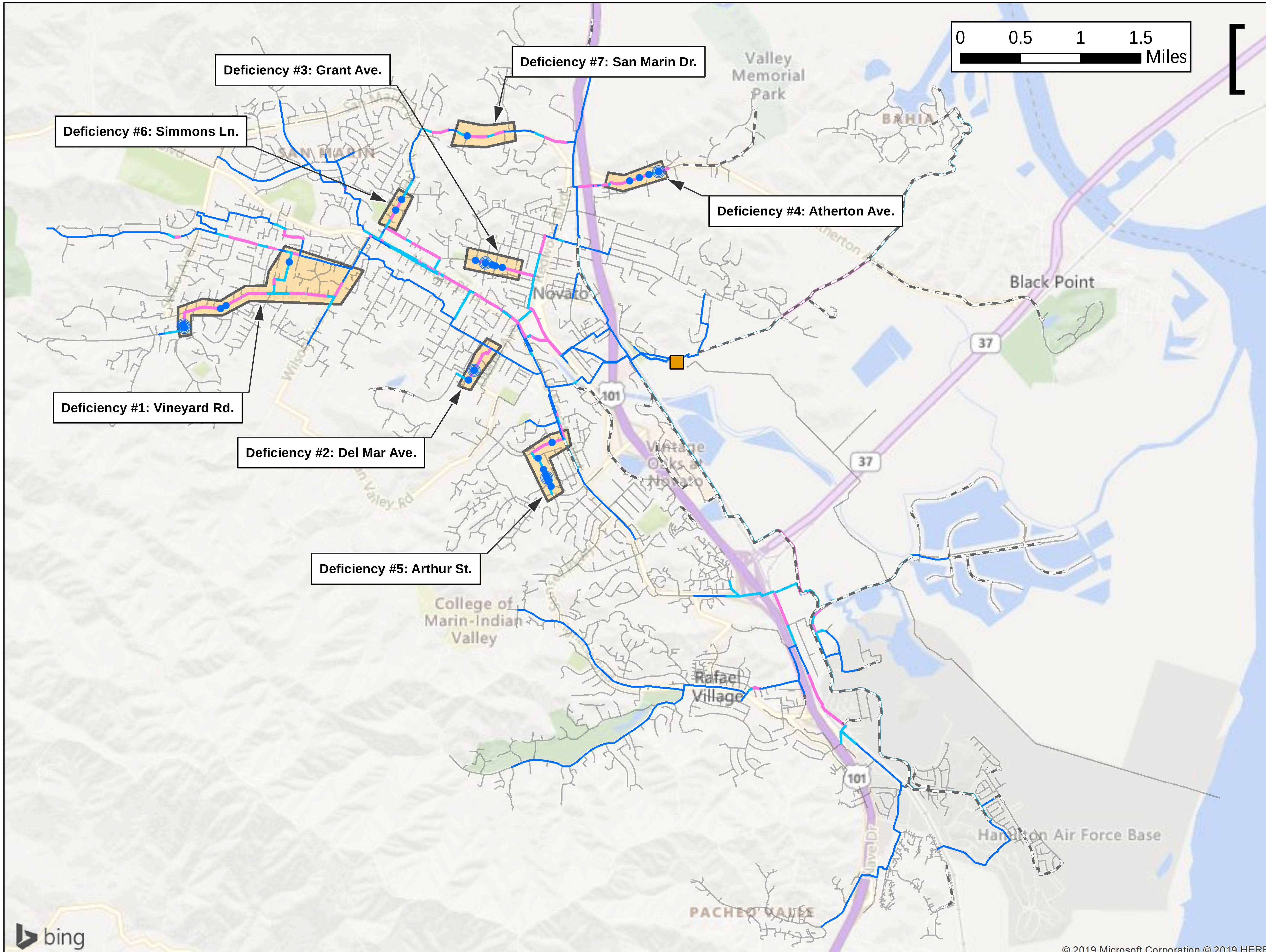
Table 4-2: Summary of Capacity Analysis Results

Model Scenario	Peak Flow to WWTP (mgd)	Volume of Model Overflows (MG)
Existing Trunk Sewer Network		
Existing Dry Weather Flow	8.3	--
Existing Wet Weather Flow	32.3	0.30
Future Dry Weather Flow	8.9	--
Future Wet Weather Flow	32.4	0.32
Trunk Sewer Network with Preliminary Solutions		
Future Wet Weather Flow	34.4	--

Table 4-3: Summary of Modeled Pump Station Flows

Pump Station	Est. Firm Capacity (mgd)	Est. Total Capacity (mgd)	Future Flow (20-year Growth Scenario)		
			ADWF (mgd)	PDWF (mgd)	PWWF (mgd)
Ignacio Transfer PS	5.5[a]		2.0	3.1	11.2
Olive Street PS	10.4	13.0	1.1	1.6	8.2
Bahia Main PS	2.7[b]	2.9[b]	0.08	0.13	1.2
BMK #5 Main PS	2.0	2.0[c]	0.11	0.20	1.9
East Hamilton PS	2.8[b]	2.9[b]	0.33	0.51	1.9
Marin Village PS	1.4	2.3	0.26	0.42	1.5

a. Capacity with 2 pumps running at 100%. Flows greater than 5.5 mgd are diverted to storage.
b. Capacity calculated based on pump curves inferred from operating points (950 gpm at 160 ft. for Bahia Main and 1900 gpm at 157 ft. for the East Hamilton wet weather pumps).
c. BMK #5 is not currently capable of running all 3 pumps simultaneously, so total capacity is equal to firm capacity.



Sources: Novato Sanitary District, Bing Maps

Legend

- Novato WWTP

Modeled Sewer

- Not surcharged
- Backwater surcharge
- Throttle surcharge
- Force Main
- ! Model Freeboard < 2 ft
- $ Model Predicted Overflow
- Unmodeled Sewer
- Capacity Deficiency Areas

Novato Sanitary District

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Figure 4-2

Model Results - Future Design Storm PWWF

Map Created: 10/22/2019

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Table 4-4: Capacity Deficiencies and Preliminary Solutions

Deficiency ID	Deficiency Location[a]	Upstream MHID	Downstream MHID	Length (ft.)	Exist. Diam. (in.)	Upsize Diam. (in.)	Predicted Freeboard (ft.)[b]	Predicted Overflow Volume (gal)[b]
1	Angelica Ct., Vineyard Rd., Wilson Ave. to Mary Jane Ln.; Sun Ln., easement, Kendon Ln., Center Rd. to Western Dr.	C16008 D16005	E15066 D15002	8,565	8 - 12	12 - 15	Overflow	140,000
2	Del Mar Ave. from Hill Rd. to Center Rd.	F17035[b]	G17139	1,413	8	10	Overflow	13,000
3	Grant Ave. from Sixth St. to First St.	G15046	G15042	1,742	10 - 12	12 - 15	Overflow	1,100
4	Atherton Ave. east of Oak Shade Ln. to east of Binford Rd.	I13001	H14056	1,722	8	10	Overflow	14,000
5	Arthur St. from Buchanan St. easement to S. Novato Blvd.	G19007	G19085	1,182	10	15	Overflow	150,000
6	Simmons Ln. from Feliz Rd. to Kristin Ln.[d]	F14005	F14004	335	8	10	1.5	N/A
7	San Marin Dr. west of Spinosa Way	G13013	G13014	319	6	8	1.5	N/A

a. Deficiency location is location of throttled pipes causing upstream criteria violation under future PWWF conditions.
b. Under future PWWF conditions in existing system.
c. Shown as F17111 on District maps. Manhole number was changed for the updated GIS developed for us in this Master Plan because it was found to be a duplicate ID.
d. Assumes that temporary plug on Simmons Lane that is currently used during wet weather periods to back up and divert flow to San Marin Drive sewer would not be installed.

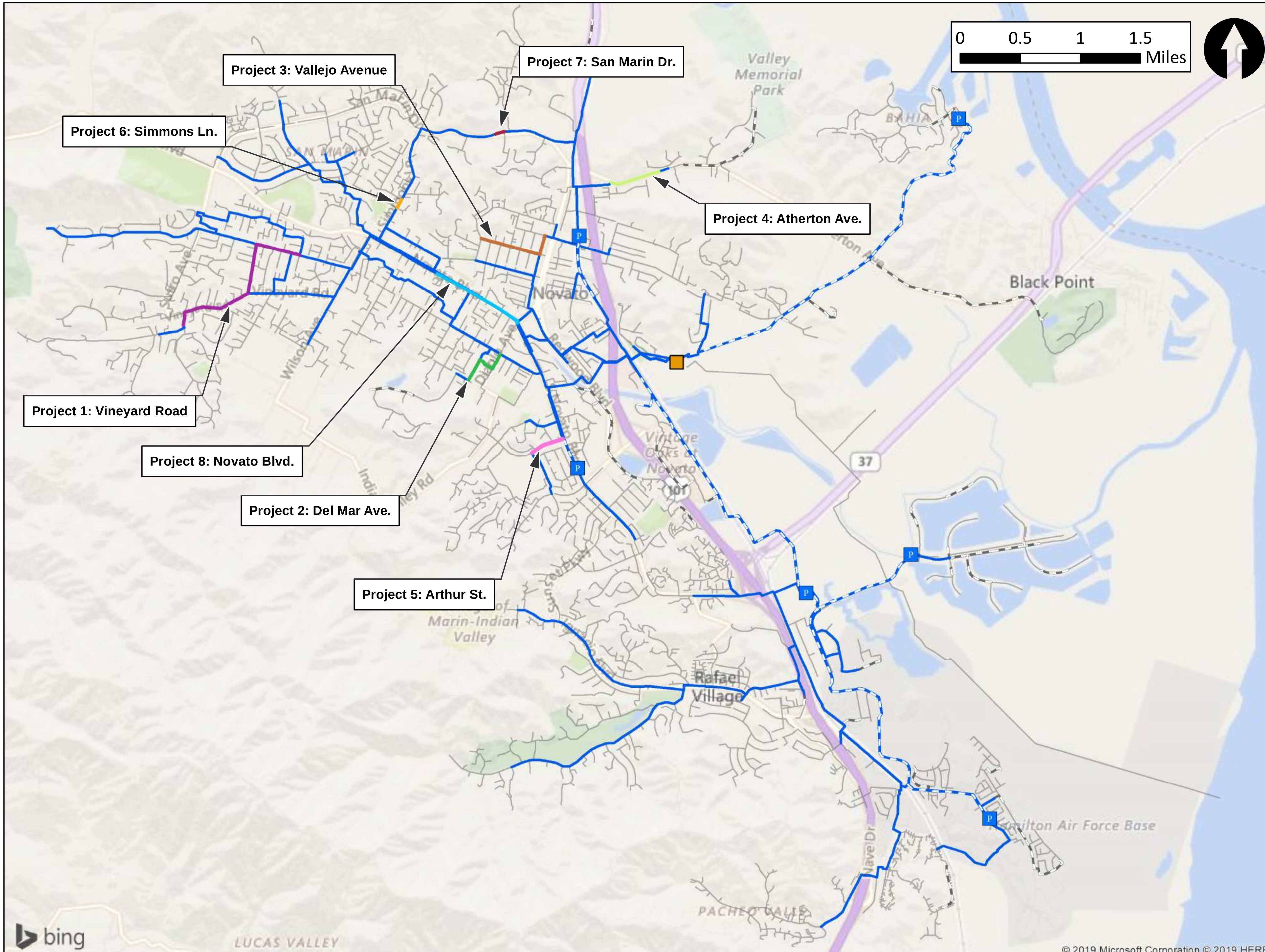
4.4 Recommended Capacity Improvement Projects

Additional analyses were conducted to refine the preliminary solutions described in Table 4-4. Each capacity deficiency area was reviewed to identify potential alternatives to simply upsizing or replacing the existing pipes, particularly for those sewers located in busy roads or commercial areas or difficult to access easements. For several of these projects, various alternatives involving diversions to other sewers or to potential new sewer pipelines were formulated and tested in the model. Finally, each potentially viable alternative was reviewed in the field and/or on aerial mapping, as well as discussed with District staff, to identify potential design, permitting, environmental, and constructability issues. Feasible construction methods were also identified for each project, and further refinements were made using the hydraulic model. **Table 4-5** presents the recommended capacity improvement projects associated with each of the identified capacity deficiencies. **Figure 4-3** shows the locations of the proposed improvements. Additional discussion and more detailed project descriptions and estimated costs for the recommended capacity improvement projects are provided in Chapter 6 and **Appendix F**.

Table 4-5: Recommended Capacity Improvement Projects

Project ID[a]	Project Name	Project Description[b]	Length (ft.)	Design Flow[c] (mgd)
1	Vineyard Road	Upsize exist. 8" and 10" pipes to 12" and 14" in Angelica Ct. and Vineyard Rd. to Eucalyptus Ave.; divert flow to new 12" to 15" sewer in Eucalyptus Ave. from Vineyard Rd. to Center Rd.; replace exist. 10" pipe with 15" in Center Rd. to Western Dr.	5,600	1.4 – 3.7
2	Del Mar Avenue	Upsize exist. 8" pipe to 10" in Del Mar Ave. from Hill Rd. to Hotchkin Dr.; divert portion of flow to new 10" sewer in Hotchkin Dr. from Del Mar Ave. to Diablo Ave.; upsize exist. 8" pipe to 10" in Diablo Ave. from Hotchkin Dr. to Center Rd.	2,015	0.72, 0.53
3	Vallejo Avenue (Grant Street Relief)	Divert flow at 7th St. to new 8" sewer in Vallejo Ave. to Jade Ct; upsize existing 8" pipe to 10" in Vallejo Ave. to 1st St.; upsize exist. 10" pipe to 12" in 1st St. from Vallejo Ave. to Olive Ave.	2,775	0.35 – 1.1
4	Atherton Avenue	Upsize exist. 8" pipe to 10" and 12" in Atherton Ave. from east of Oak Shade Ln. to east of Binford Rd.	1,722	0.98
5	Arthur Street	Replace exist. 10" pipe with 15" in Arthur St. from easement west of Hayes St. to S. Novato Blvd.	1,182	1.8
6	Simmons Lane	Upsize exist. 8" pipe to 10" in Simmons Ln. from Feliz Rd. to Kristin Ln.	335	0.86
7	San Marin Drive	Upsize exist. 6" pipe to 8" in San Marin Dr. west of Spinosa Way	319	0.45
8	Novato Boulevard	Construct new parallel 24" sewer in Novato Blvd. from east of Grant Ave. to Diablo Ave.	3,240	4.8 – 5.8[d]

a. Corresponds to Deficiency ID in Table 4-4. Project 8 was triggered by increased flows after relief of other capacity deficiencies.
b. See Appendix F for additional detail.
c. Under future PWWF conditions.
d. Design flow for parallel sewer to be refined during project design; total flow in both pipes ranges from 9.5 to 10.6 mgd.



Novato Sanitary District

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Figure 4-3

Recommended Capacity Improvement Projects

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Sources: Novato Sanitary District, Bing Maps

Map Created: 1/28/2019

4.4.1 RDI/I Reduction Effects on Identified Capacity Deficiencies

Rehabilitation of sewers (and also potentially service laterals) could reduce RDI/I in the sanitary sewer system, resulting in reduction in peak wet weather flows and the need for some capacity improvements. Therefore, the model was used to test the potential effects of RDI/I reduction in areas upstream of identified capacity deficiencies. Specifically, across-the-board reductions of 10, 30, and 50 percent in the R values for all subcatchments were tested in the model to determine if such reductions could reduce predicted surcharge to an acceptable level so as to eliminate the need for some of the capacity improvements. **Table 4-6** summarizes the findings from this analysis, in terms of which deficiencies could be eliminated with RDI/I reduction.

Table 4-6: Capacity Deficiencies Eliminated with Modeled RDI/I Reduction

Percent RDI/I Reduction	Deficiencies Eliminated[a]
10%	Deficiencies 6 and 7
30%	Deficiencies 3 and 4
50%	Deficiencies 1 and 2
More than 50%	Deficiency 5

a. See Table 4-4 and Figure 4-2 for description and location of deficiencies.

The modeling indicated that very few of the identified capacity deficiencies could be eliminated with realistically achievable reductions in RDI/I in the near term. It is not known to what extent these reductions can be achieved solely through rehabilitation or replacement of sewer mains alone, or if more extensive private service lateral rehabilitation and/or identification and correction of direct inflow sources (e.g., directly connected catch basins, area drains, and roof downspouts, if they exist) would be required. Reductions of more than 30 percent would almost certainly require rehabilitation of private laterals, unless some very large direct inflow sources are found. However, it may be in the District's interest to focus rehabilitation efforts in some of the areas where more modest RDI/I reduction could potentially achieve benefits before constructing capacity improvements.

5

CHAPTER 5

CONDITION ASSESSMENT

Novato Sanitary District's Collection System Master Plan

Chapter 5 Condition Assessment

This chapter describes the assessment of the physical condition of the collection system sewers, the approach for prioritizing the sewers for rehabilitation, and the sewers recommended for repair, rehabilitation, or replacement. Note that in the context of this discussion, the term "rehabilitation" is used interchangeably to mean upgrading the condition of a sewer pipeline through repair of individual defects or through manhole-to-manhole lining or replacement methods.

5.1 Sewer Inspection

The District maintains a database of closed-circuit television (CCTV) inspection data for sewers dating back to 2009. The CCTV inspections were originally documented using a set of observation codes developed specifically for the District for use with its PicAx™ CCTV inspection software, and used to compute a "damage severity index" (DSI) for each pipe. With the industry migration to a standard set of CCTV codes and rating system under the National Association of Sewer Services Companies (NASSCO) Pipeline Assessment Certification Program (PACP), the District switched to NASSCO PACP codes in 2016. In order to evaluate the condition of the District's sewers in a consistent format for this study and for future assessments, the older "legacy" CCTV codes were converted to roughly equivalent PACP codes for analysis. The code translation, however, was not always "apples to apples" due to differences in the two coding systems.

To date, the District has inspected most of the sewers in the system, with the goal of inspecting the entire system on a five-year cycle. However, review of the CCTV data indicated that some inspections may have been conducted for reasons other than structural condition assessment (e.g., to verify effectiveness of root treatment) and therefore, the most recent inspection for each pipe may not have been fully coded to document all defects and observations. For this reason, inspections with fewer than three observations (generally, a minimum of two observations are recorded for each pipe, for the starting and ending manholes) were eliminated from the assessment. In total, 3,848 pipes (about 60 percent of the pipes in the gravity collection system) had usable data for the condition assessment.

5.2 Condition Rating

The condition assessment was conducted using InfoMaster™ software from Innovyze. InfoMaster is a pipeline asset planning program that incorporates condition scoring, risk assessment, and rehabilitation decision analysis in a GIS environment. For NSD, CCTV data (converted to NASSCO PACP format) was imported into InfoMaster and linked to the District's sewer pipeline GIS (note, some inspections could not be linked because of differing manhole IDs in the CCTV database). In addition to visual display of the CCTV observations, the program computes condition scores based on the PACP rating system. In the PACP system, defects are categorized as structural or maintenance defects (the latter also referred to as 'service" defects) and graded on a scale of 1 to 5 with 5 representing the most severe. Of particular interest for this master plan were the peak structural defect scores (representing the number of the highest grade structural defect)[3] and the total structural defect score (sum of grades for all structural defects) for each pipe, with pipes with high peak and total scores indicating those sewers with the highest likelihood of failure due to structural condition. **Figure 5-1** and **Figure 5-2**, respectively, show the results of the peak and total structural condition ratings of the gravity sewers in the NSD system. Pipes indicated in red on the figures are those with two or more Grade 5 structural defects (18 pipes, as shown on Figure 5-1) or a total defect score greater than 50 (12 pipes, as shown on Figure 5-2). Over 70 percent of the inspected pipes had no

[3] See footnote a. in Table 5-1 for definition of peak defect score.

recorded structural defects. It should be noted that some pipes may have been repaired or replaced since their last CCTV inspection; therefore, the structural defect scores may not reflect their current condition.

5.3 Risk Assessment

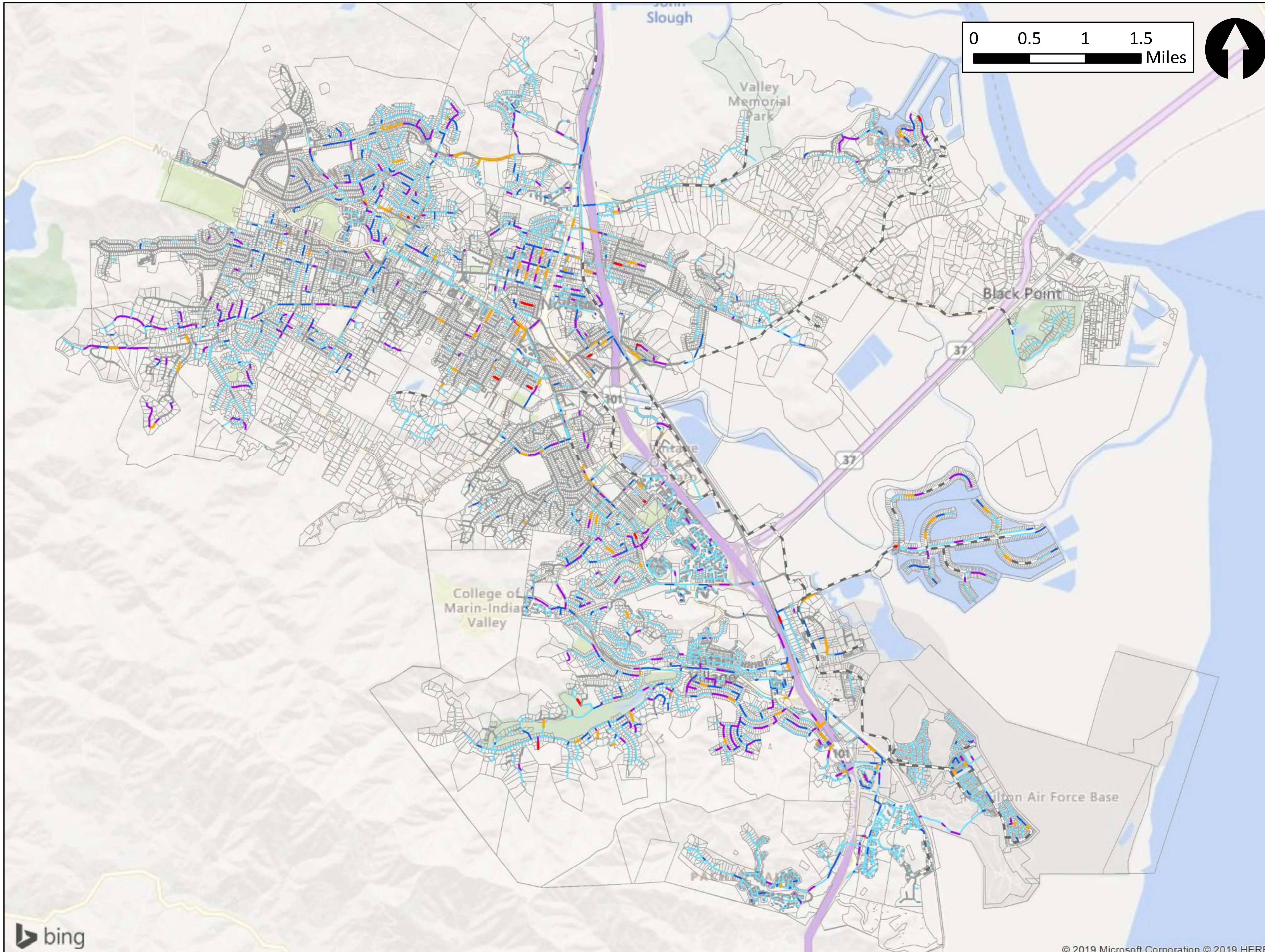
A risk analysis approach ("risk model") was used to prioritize sewers for potential rehabilitation. The methodology involves quantifying and assessing the risks posed by the failure or inability of the sewer system to provide the level of service needed to meet the District's sewer system management goals. Using this approach, risk scores are calculated for each sewer pipe, and individual pipe scores are then used to prioritize the pipes for repair, rehabilitation, or replacement.

The risk of asset failure is calculated by quantifying the likelihood of failure (LOF) and the consequence of failure (COF) of a sewer asset. The likelihood of failure is the possibility of asset failure based on its condition. The consequence of failure is defined as the impact on the level of service resulting from asset failure. The combination of LOF and COF scores is therefore an indicator of potential risk.

5.3.1 Likelihood of Failure Categories

Four LOF indicators were used in the risk model, as follows:

- **Structural Condition**: Structural defects can be a significant cause of sewer failures. CCTV inspection results are the best indicator of current pipe condition. Structural condition was rated, based on two calculations: peak and total structural defect scores. The peak structural defect score reflects the number and NASSCO PACP grade of the highest grade structural defect(s). The total structural defect score reflects the number and grade of all structural defects. If CCTV inspection data was not available for a pipe segment, then the structural condition score was estimated solely based on pipe segment age and material (see Structural Vulnerability below). Structural condition is a strong indicator of likelihood of failure and was heavily weighted.
- **Structural Vulnerability**: Even if a pipe's structural condition is not currently compromised, sewer failures are more likely to occur in older pipes and pipes of certain types of materials. Therefore, older clay, asbestos cement, and concrete pipes, which are more susceptible to cracking and corrosion, were given higher scores than newer plastic pipe materials. However, this factor was not weighted as high as documented structural condition from CCTV inspection.
- **O&M Condition**: Operation & maintenance (O&M) condition was rated based on a pipe's maintenance requirements based on its current scheduled frequency of sewer cleaning. Scheduled cleaning frequency is an indicator of the likelihood of asset failure due to a maintenance-related issue that could lead to a sanitary sewer overflow (SSO). This LOF factor was given a lower weighting than structural condition since preventive maintenance cleaning can be used to alleviate this risk.
- **I/I Contribution**: As described in previous chapters, I/I is a significant issue in the NSD collection system, resulting in areas of sewer surcharge and historical wet weather overflows. This LOF factor reflects the potential hydraulic impact of I/I in the City's system based on the peak I/I rate of the flow meter area in which the pipe is located.



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Peak Structural Defect Score

- 1 (No Defects)
- 1.1 - 3.1
- 3.2 - 4.1
- 4.2 - 5.1
- 5.2 - 6
- No CCTV Data
- Force Main

See footnote a in Table 5-1 for a definition of peak defect scores.

Novato Sanitary District

Collection System Master Plan

Figure 5-1

Peak Structural Defect Score

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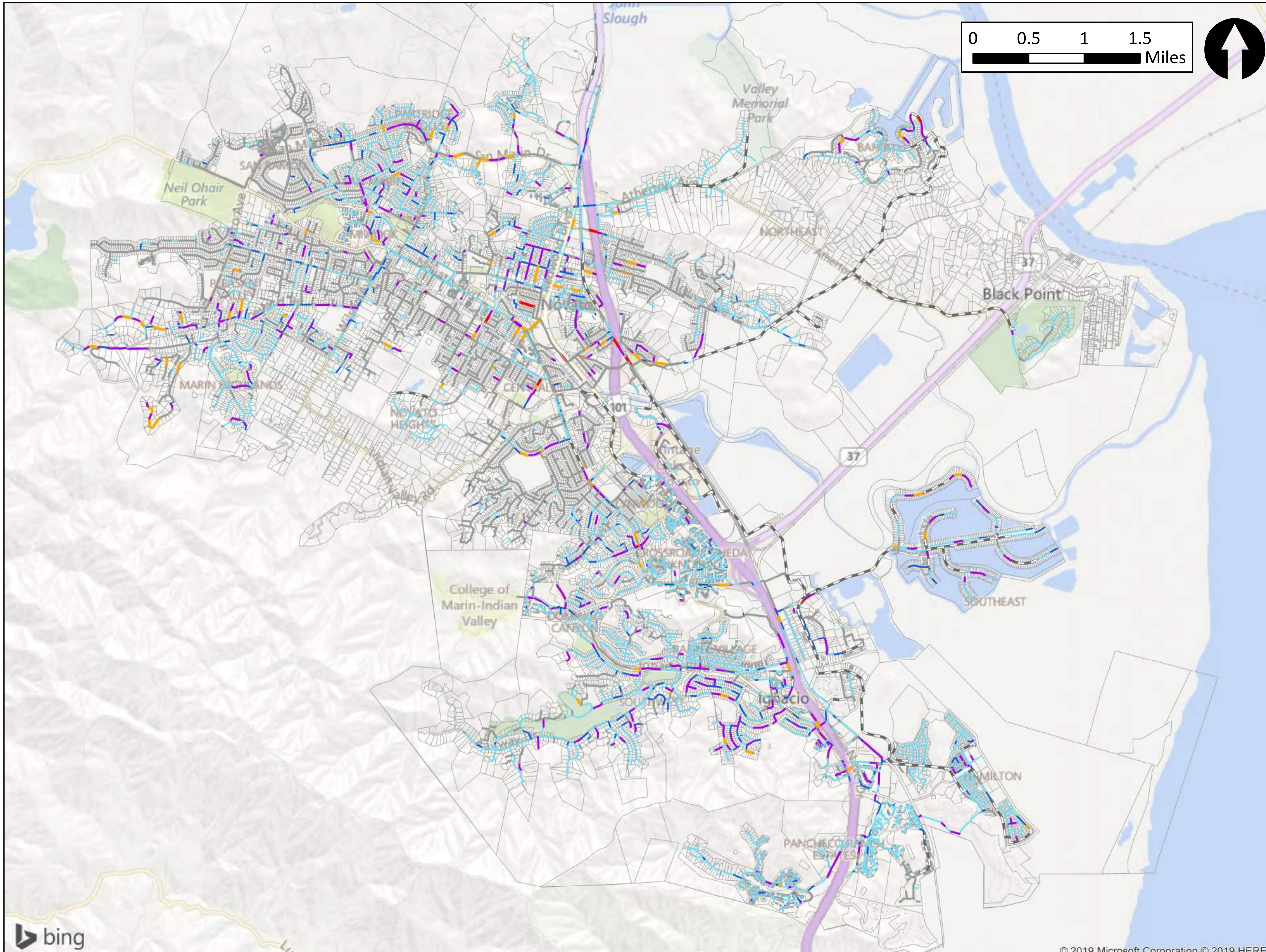
Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 7/17/2019



Legend

Total Structural Defect Score

- 0 (No Defects)
- 1 - 5
- 6 - 20
- 21 - 50
- > 50
- No CCTV Data
- Force Main

Novato Sanitary District

Collection System Master Plan

Figure 5-2

Total Structural Defect Score

Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/24/2019

\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\InfoMaster_Figures\012219_Fig\Figure_5-2_Total_Score.mxd

5.3.2 Consequence of Failure Categories

COF is assessed by examining the impact on economic, social and environmental factors. This approach, often referred to as the Triple-Bottom-Line approach, involves identifying and quantifying suitable indicators that represent these core categories. For this study, four indicators were used to assess COF. Four indicators were used to assess COF:

- **Potential Spill Volume**: Larger sewer spills or failure of a sewer asset serving a large tributary area can have a significant impact on the cost of fixing the pipe and restoring damaged property and the surrounding area. The size of the sewer was chosen as an indicator of the potential impact of large spills or failure of a major sewer asset. This COF factor was most heavily weighted.
- **Environmental Impact**: Sewer overflows that reach surface waters can adversely impact water quality and the environment. Distance to surface water was used as the indicator of the potential environmental impact of a sewer spill.
- **Traffic/Response Impacts:** Sewer failures and sewer construction can also significantly impact commuters and the public. Pipe location (type of road or easement) was used as indicator of potential traffic or response impact.
- **Public Impact:** Sewer failures can significantly impact public facilities and the community in general. Proximity to public facilities such as schools, parks, and commercial centers were used as indicators of potential public impact.

5.3.3 Risk Score Calculations

The risk model, also implemented in InfoMaster, utilized data from GIS and CCTV inspections to compute LOF and COF scores, ranging from 1 (very low) to 5 (very high) for each factor. **Table 5-1** and **Table 5-2** present the scoring criteria and weights for the LOF and COF categories, respectively. The combination of the LOF and COF scores were then used to group the pipes into four risk categories (very low, low, medium, and high), as shown on the matrix in **Figure 5-3**. The results indicated that less than 5 percent of the pipes in the system (about 10 miles) were categorized as high or medium risk (with less than 1-1/2 miles as high risk), and almost 90 percent as very low risk. **Figure 5-4**, **Figure 5-5**, and **Figure 5-6,** respectively, display the LOF and COF scores and overall risk priority for the pipes in the system. Note that pipes with a “very high” LOF were considered high risk priority, regardless of their potential consequence of failure, as a failed pipe always has the potential for an SSO and adverse impact on District customers. As noted previously, some pipes may have been repaired or replaced since their last inspection; therefore, their LOF score and resulting risk priority may not reflect their current condition.

A detailed tabulation of scores for each gravity sewer pipe in the NSD system is included in **Appendix E**.

Figure 5-3: Risk Score Matrix

Consequence of Failure (COF)	Very Low LOF	Low LOF	Medium LOF	High LOF	Very High LOF
Very High COF	LOF Score ≤ 1.5, COF Score > 4	LOF Score ≤ 2, COF Score > 4	LOF Score ≤ 3, COF Score > 4	LOF Score ≤ 4, COF Score > 4	LOF Score > 4, COF Score > 4
High COF	LOF Score ≤ 1.5, COF Score ≤ 4	LOF Score ≤ 2, COF Score ≤ 4	LOF Score ≤ 3, COF Score ≤ 4	LOF Score ≤ 4, COF Score ≤ 4	LOF Score > 4, COF Score ≤ 4
Medium COF	LOF Score ≤ 1.5, COF Score ≤ 3	LOF Score ≤ 2, COF Score ≤ 3	LOF Score ≤ 3, COF Score ≤ 3	LOF Score ≤ 4, COF Score ≤ 3	LOF Score > 4, COF Score ≤ 3
Low COF	LOF Score ≤ 1.5, COF Score ≤ 2	LOF Score ≤ 2, COF Score ≤ 2	LOF Score ≤ 3, COF Score ≤ 2	LOF Score ≤ 4, COF Score ≤ 2	LOF Score > 4, COF Score ≤ 2
Very Low COF	LOF Score ≤ 1.5, COF Score ≤ 1.5	LOF Score ≤ 2, COF Score ≤ 1.5	LOF Score ≤ 3, COF Score ≤ 1.5	LOF Score ≤ 4, COF Score ≤ 1.5	LOF Score > 4, COF Score ≤ 1.5

Likelihood of Failure (LOF) spans the columns.

Priority >>	Very Low	Low	Medium	High

Table 5-1: Likelihood of Failure Scoring

LOF Category	Indicator	Weight (%)	LOF Score				
			1 (low)	2	3	4	5 (high)
Structural Condition[c]	Peak Structural Defect Score (PACP risk method)[a]	40	1.0	1.1 – 3.1	3.2 – 4.1	4.2 – 5.1	5.2 – 6.0
	Total Structural Defect Score[b]	20	0	1 – 5	6 – 20	21 – 50	>50
Structural Vulnerability	Pipe Age[d] and/or Material[d]	10	Any pipe ≤ 20 years old	VCP/XXX >20-60 ACP/RCP >20-40 Plastic >20 or unknown	VCP/XXX >60-80 ACP/RCP >40-60	All CIP VCP/XXX >80-100 or unknown ACP/RCP >60-80 or unknown	VCP/XXX > 100 ACP/RCP > 80
O&M Condition	Scheduled Maintenance Frequency	10	36 mo. or unknown	24 mo.	12 mo.	6 mo.	≤3 mo.
I/I Contribution	peak I/I rate[e]	20	<20 gpd/ft	20 to <40 gpd/ft	40 to <60 gpd/ft	60 to <80 gpd/ft	≥80 gpd/ft

a. Based on number of highest grade structural defects as computed based on methodology in NASSCO PACP Reference Manual Version 7.0.1, Appendix D, Risk Management in Piping Systems: 1.0 = no defects, 1.1 = 1 grade 1 defect, 1.2 = 2 grade 1 defects, ……1.9 = 9 grade 1 defects, 2.0 = 10+ grade 1 defects, 2.x = x grade 2 defects…… 5.x = x grade 5 defects, 6.0 = 10+ grade 5 defects.

b. Reflects total number and grades of defects (sum of number of defects x defect grade).

c. If pipe has no CCTV data, Structural Condition score will be based on Structural Vulnerability score, but pipe will be flagged to indicate pipe has not been inspected or CCTV data cannot be located based on upstream/downstream manhole IDs.

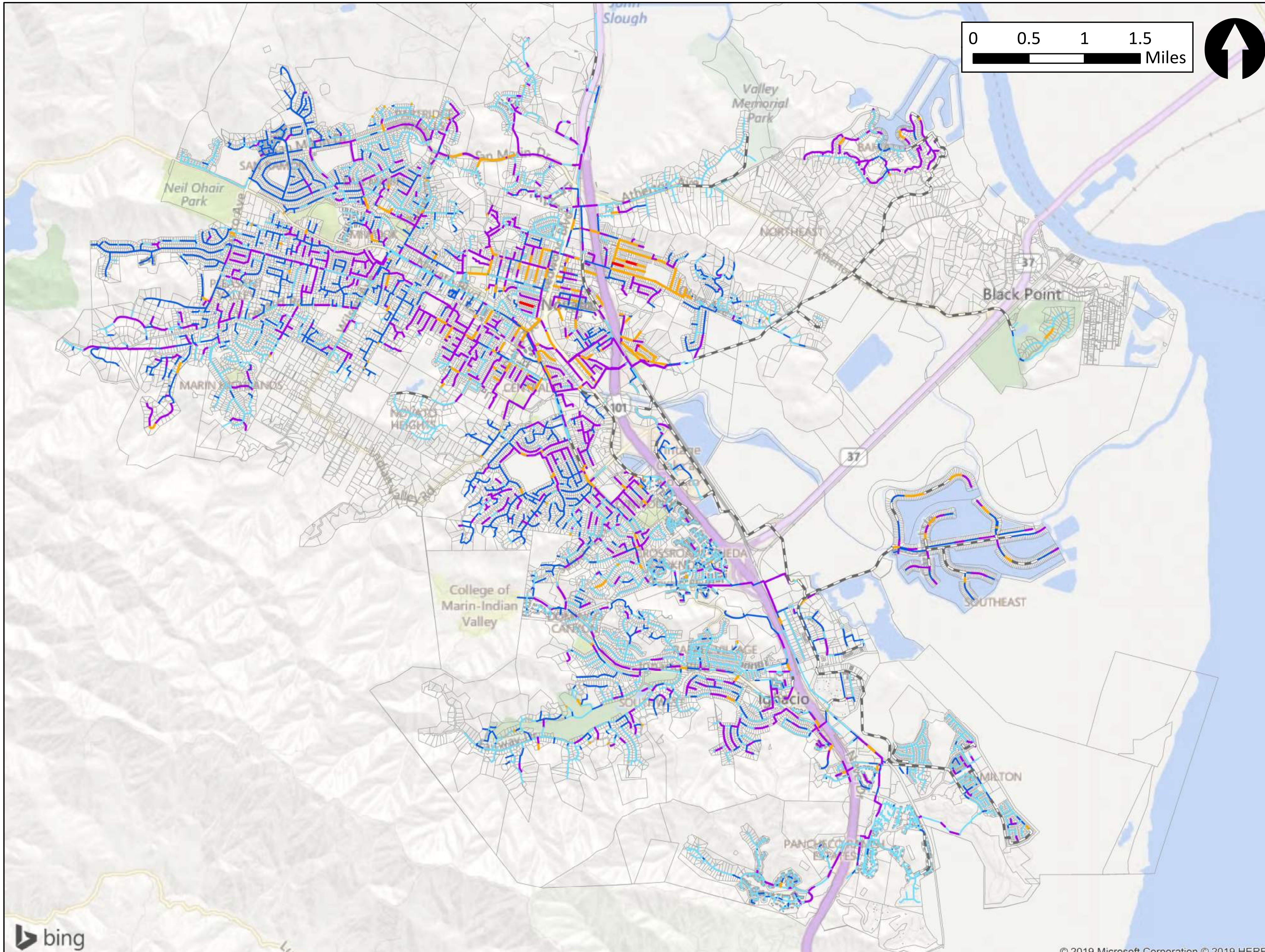
d. Based on new fields "Material_use" and "Year_use" in updated sewer GIS. "XXX" are pipe material codes in original GIS or AutoCAD that could not be specifically identified or categorized.

e. Based on sum of model subcatchment GWI plus peak RDI/I by flow meter area

Table 5-2: Consequence of Failure Scoring

COF Category	Indicator	Weight (%)	COF Score				
			1 (low)	2	3	4	5 (high)
Potential Spill Volume	Pipe diameter	50	<8”	8”	10”-12”	15”	> 15”
Environmental Impact	Proximity to waterways	20	>1000 ft.	500-1000 ft.	200-500 ft.	100-200 ft.	Within 100 ft. of waterway
Traffic / Response Impact	Road type or easement	10	Other	Collector street	Easement	Arterial street	Highway ramp or railroad
Public Impact	Proximity to commercial areas or public facilities[a]	20	>150 ft.	Within 100-150 ft.	Within 75-100 ft.	Within 75 ft.	Within or intersecting

a. Hospitals, schools, parks, retail centers



Legend

Likelihood of Failure

- Very Low
- Low
- Medium
- High
- Very High
- Force Main

Novato Sanitary District

Collection System Master Plan

Figure 5-4

Likelihood of Failure Score

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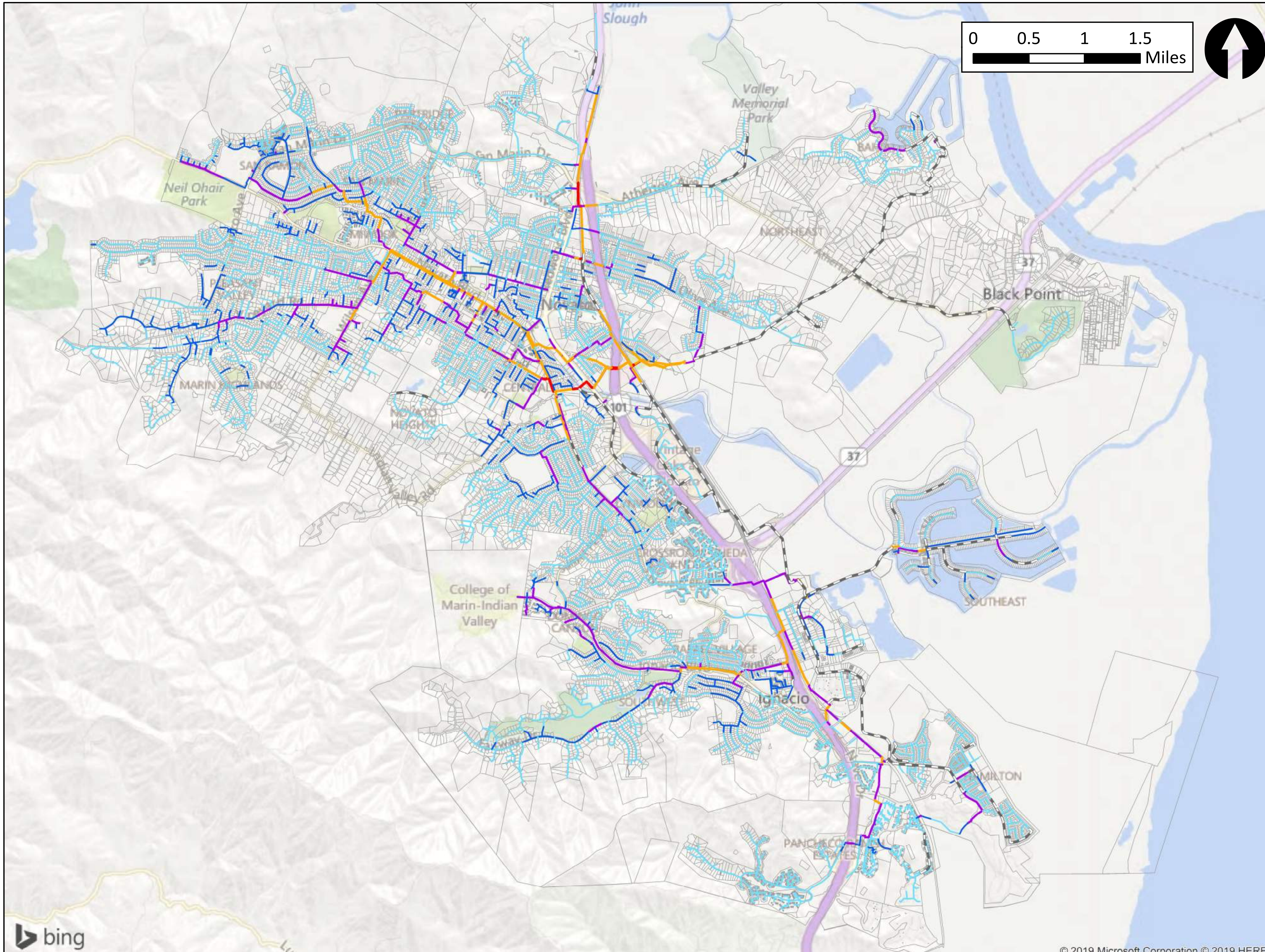
Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/28/2019



Legend

Consequence of Failure

- Very Low
- Low
- Medium
- High
- Very High
- Force Main

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Collection System Master Plan

Figure 5-5

Consequence of Failure Score

NSD NOVATO SANITARY DISTRICT

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Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/28/2019

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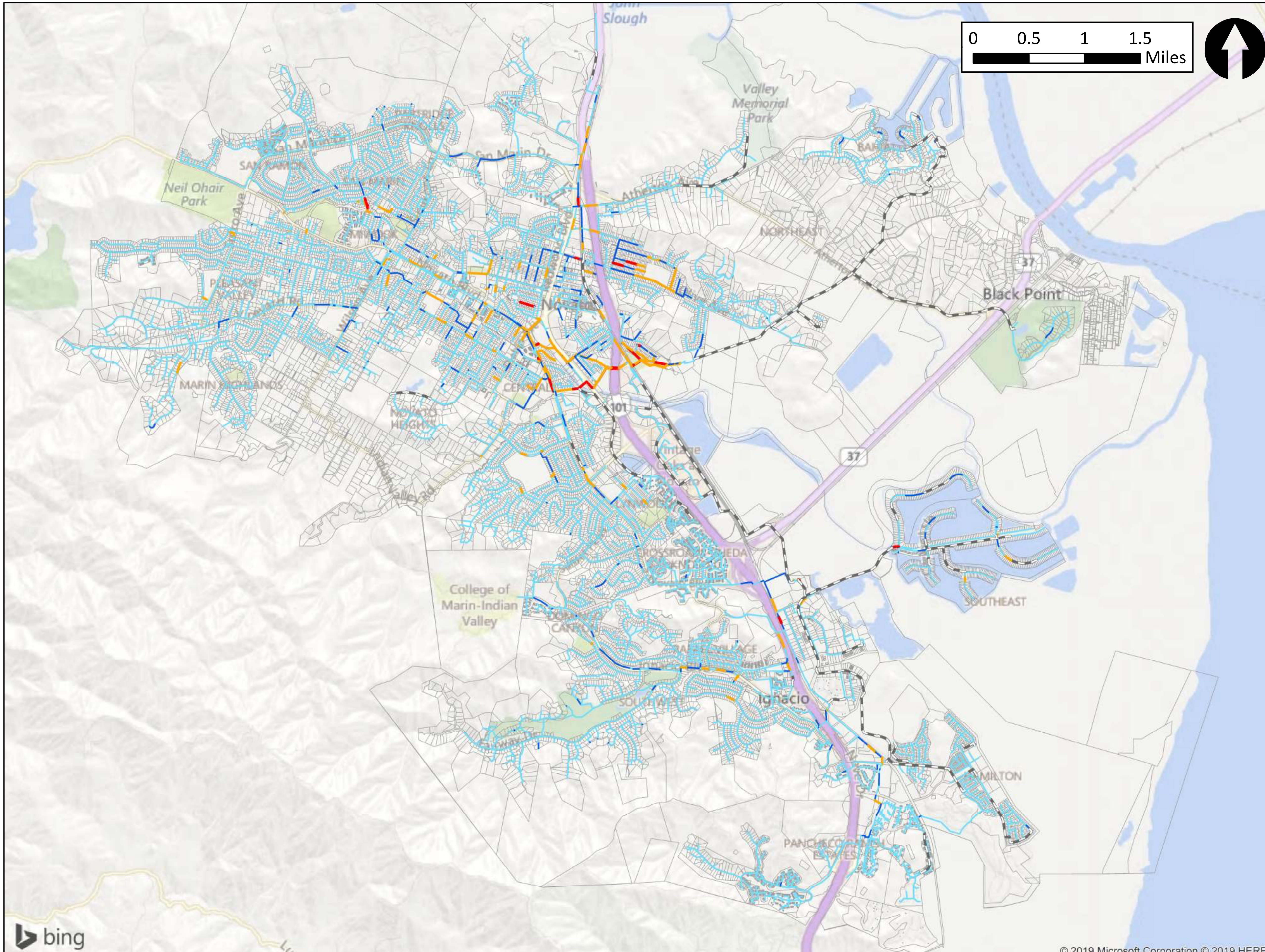

Legend

Risk Priority

- Very Low
- Low
- Medium
- High
- Force Main

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Figure 5-6

Overall Risk Priority

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Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/28/2019

5.4 Rehabilitation Decision Process

The InfoMaster program was also used to generate preliminary rehabilitation recommendations for each pipe based on its condition from CCTV inspection. A decision process was developed to identify the "best method" of rehabilitation based on the number and type of defects in the pipe. Only pipes with "major" defects were identified for rehabilitation, with major defects being defined as NASSCO PACP Grade 4 or 5 structural defects. Examples of major defects include broken or collapsed pipe, significant fractures, large offset or separated joints, severe sags, or significant corrosion. Based on the number and type of defects and the pipe diameter, each sewer with major defects was recommended for either point repair, lining, or replacement (or for further evaluation if the pipe had sags, bends, or corrosion that would require more detailed review of CCTV video). In addition, pipes without CCTV data but with a high or medium risk priority rating were identified as priorities for CCTV inspection.

Table 5-3 summarizes the results of the rehabilitation decision process in terms of the number and type of preliminary recommended rehabilitation actions for pipes in each risk priority category. **Figure 5-7** illustrates the decision process that was used to identify needed rehabilitation actions, and **Table 5-4** defines the terms used in the flowchart and provides explanations of the decision points and outcomes. Note that the process is designed specifically for developing preliminary recommendations for system-wide rehabilitation planning efforts rather than design decisions for individual projects.

Table 5-3: Summary of Preliminary Rehabilitation Decisions

Risk Priority	Number of Pipes						
	Replace	Line	Evaluate Replace vs. Lining	Point Repair	Prioritize CCTV	Normal PM	Total
High	8	2	4	1	18	1	**34**
Medium	33	3	4	10	138	28	**216**
Low	54	2	2	16	0	334	**408**
Very Low	52	4	1	58	0	5,656	**5,771**
Total	**147**	**11**	**11**	**85**	**156**	**6,019**	**6,429**

The results of the application of the decision process are presented in **Figure 5-8**. This figure identifies those pipes identified for point repairs, lining, or replacement to address major structural defects. Pipes without major defects are identified for continued maintenance and regular inspection.

Of the 94 high and medium risk priority pipes with CCTV data, 54 were recommended for replacement or lining and 11 for point repair of major defects. The remainder (mostly medium risk priority) did not have any major defects and are recommended for continued preventive maintenance and inspection. About 190 other pipes in the system also had one or more major (Grade 4 or 5) defects that may warrant further evaluation, but these pipes were considered low or very low risk priority. About 150 high and medium risk priority pipes did not have CCTV data and are recommended for near-term inspection. **Appendix E** includes lists of the preliminary rehabilitation recommendations for pipes with major structural defects and those recommended for near-term CCTV inspection. Note that since some of the CCTV inspection data is up to 10 years old, some of the identified defects may have been corrected through pipe repairs and replacement already completed by the District.

Figure 5-7: Rehabilitation Decision Process

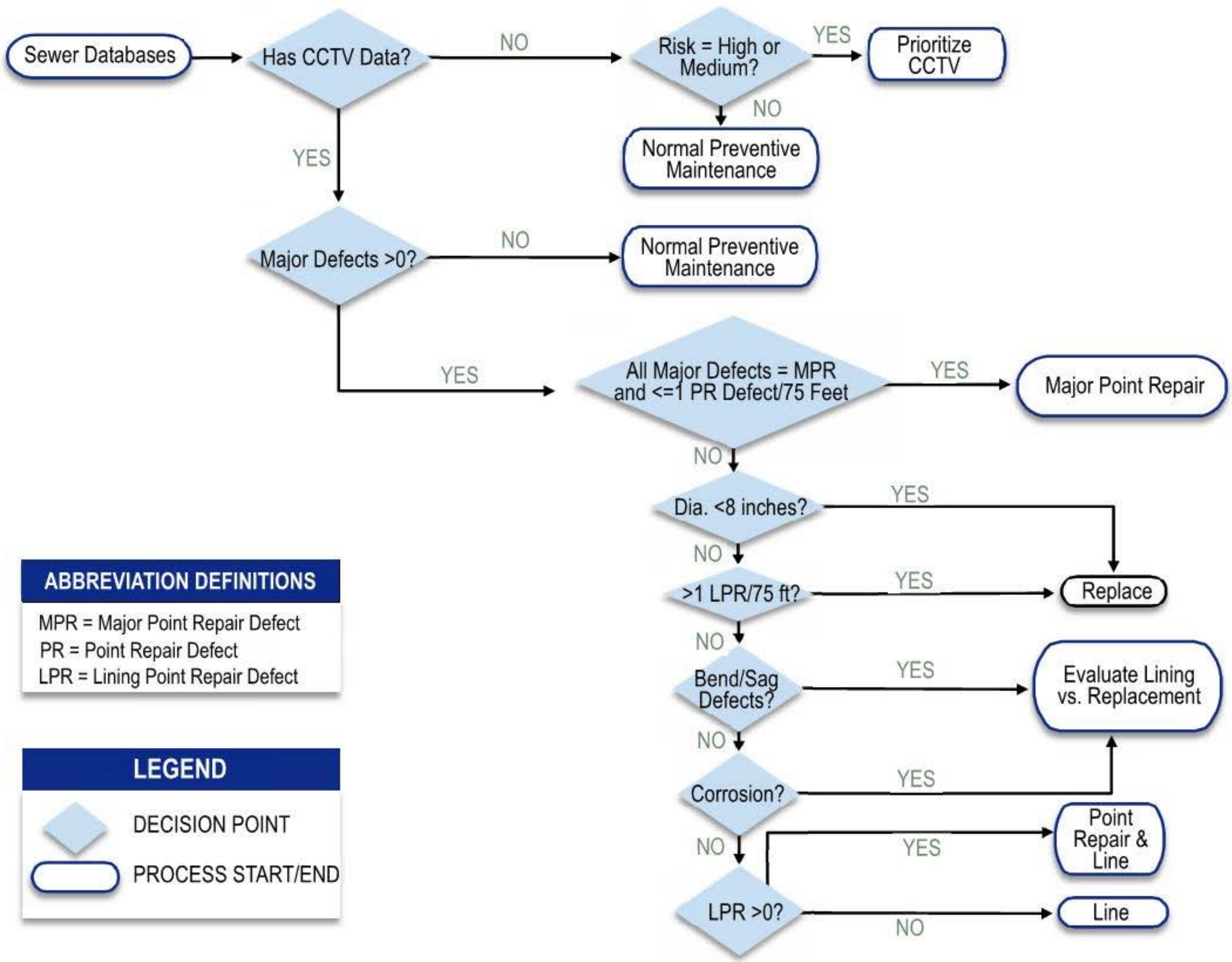

Table 5-4: Sewer Rehabilitation Decision Diagram Explanations

Type	Description	Explanation
Input	Sewer Databases	This refers to the data sets, including CCTV defects, defect code categorization, and pipe attribute data, used for pipe renewal decision-making.
Decision	Major Defects > 0?	Does the pipe segment have a major (Structural Grade 4 or 5) defects? If yes, then the pipe will continue through the decision process. If no, then a "Normal Preventive Maintenance" outcome results.
Decision	All Major Defects = MPR & <= 1 PR Defect/75 feet	Does it make sense to perform one or more point repairs to correct major defects? This is determined via two criteria: 1) All major defects must be able to be addressed using a point repair solution (Major Defects = MPR); 2) There can be no more 1 PR per 75 feet of pipe.
Decision	Dia. <8 inches?	Is the pipe diameter too small to line or smaller than current standards? Going through the process, the pipe in question has already been shown to have major defects that cannot be addressed by point repair. If yes, the decision will be to "Replace" the pipe. If the answer is no, then this pipe segment will continue through the process.
Decision	> 1 LPR/75 feet?	Is there more than 1 LPR per 75 feet? If this is the case, then the decision will be to "Replace" the pipe instead. If this is not the case, the pipe will continue through the process.
Decision	Bend/Sag Defects?	Does the pipe have any bend/sag defects? If bend/sag defects exist, then further evaluation will need to be performed to determine if the defect needs to be repaired, can be repaired, and if a lining project is feasible. If yes, then the decision will be "Evaluate for Lining or Replacement". If no, then the pipe will continue through the process.
Decision	LPR > 0?	Does the pipe have any LPRs that need to be addressed prior to lining? If yes, then the result will be to continue through the process. If no, the result will be a decision to line.
Decision	Corrosion?	Does the pipe have corrosion such that point repairs would not be feasible? If yes, then the result will be to "Replace" the pipe. If no, then the result will be to "Point Repair and Line" the pipe.
Outcome	Maintain	This is a decision to continue the ongoing maintenance of the pipe segment because, in its current condition, it does not warrant immediate additional action.
Outcome	Major Point Repair	This is a decision to perform one or more localized repairs on the pipe segment to address major defects.
Outcome	Replace	This is a decision to remove and replace the pipe because it failed one of the conditions necessary for the pipe to be point repaired or lined.
Outcome	Evaluate Lining vs. Replacement	This outcome indicates that staff must evaluate whether to line or replace the pipe segment based on a review of the pipe defects.
Outcome	Point Repair & Line	This is a decision to perform necessary point repairs and line the pipe.
Outcome	Line	This is a decision to line the pipe (point repairs not required).

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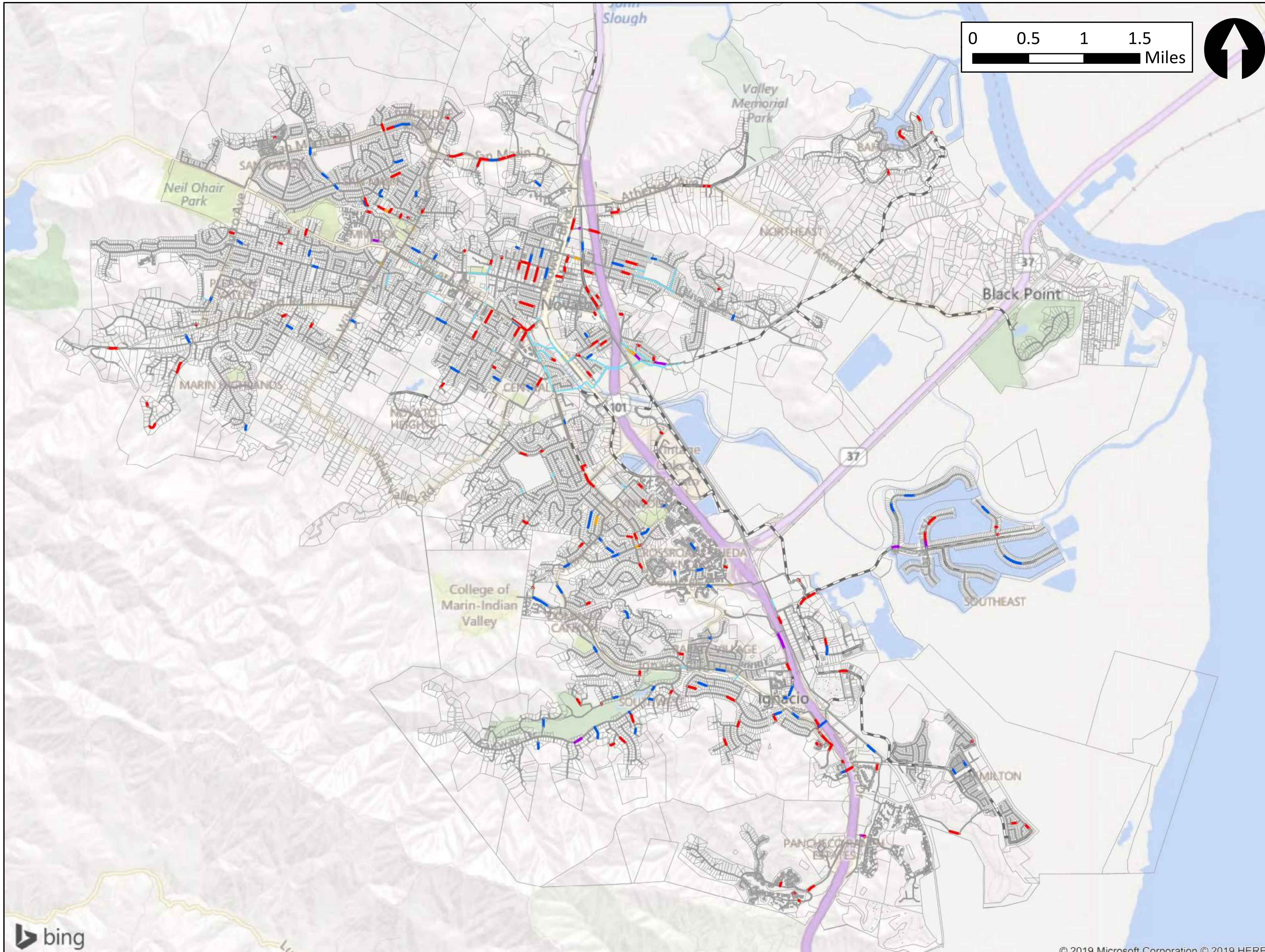

Legend

- Replace
- Line
- Point Repair+Full Lining
- Point Repair
- Evaluate Lining vs Replacement
- Prioritize CCTV
- Normal Preventive Maintenance
- Force Main

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Collection System Master Plan

Figure 5-8

Preliminary Rehabilitation Decisions

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Sources: ALCO Streets, ALCO Freeways, USD

Map Created: 1/24/2019

6

CHAPTER 6

RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

Novato Sanitary District's Collection System Master Plan

Chapter 6 Recommended Capital Improvement Program

This section summarizes the projects recommended for NSD's wastewater collection system capital improvement program (CIP). Recommended improvements were developed from the results of hydraulic modeling of the trunk sewer system, as described in Chapter 4, and the sewer condition and risk assessments described in Chapter 5. This section discusses the estimated capital costs and priorities for construction of the recommended improvements, and provides recommendations for moving forward with implementation of the CIP.

6.1 Project Cost Estimates

Estimated costs for capacity improvement projects include baseline construction costs for gravity sewers using open-cut or trenchless (pipe bursting) methods; costs for new sewer structures, lateral reconnections, and lower lateral replacement (where sewer main would be replaced); and cost allowances for project mobilization/demobilization, traffic control, and bypass pumping. Unit costs were derived from similar projects in the San Francisco Bay Area, including recent District projects. Factors considered in estimating pipe construction costs included depth of trench excavation, size of pipe being upsized or new pipe being installed, saw cut and excavation of trench, removal and handling of existing sewer pipe, trench bedding, pipe placement, trench backfill, reconnection of service laterals, and pavement restoration requirements. Estimated traffic control costs were based on type of road (e.g., busy street versus typical residential neighborhoods). Bypass pumping was considered necessary for all projects involving removal and replacement of existing sewers. Unit costs for manholes include existing manhole removal if necessary.

Estimated construction costs include a 30 percent allowance for contingencies for unknown conditions, and estimated capital costs include an allowance of 25 percent of the estimated construction cost for engineering, administration, construction management, and legal costs. The costs are conceptual level estimates, considered to have an estimated accuracy range of -30 to +50 percent, suitable for use for budget forecasting, capital improvement program development, and project evaluations, with the understanding that refinements to project details and costs would be necessary as projects proceed to design and construction. Material and labor price fluctuations are likely to affect project cost at the time of scheduled construction. All costs are presented in late 2018 dollars. Cost estimates for the capacity improvement projects are included in **Appendix F**.

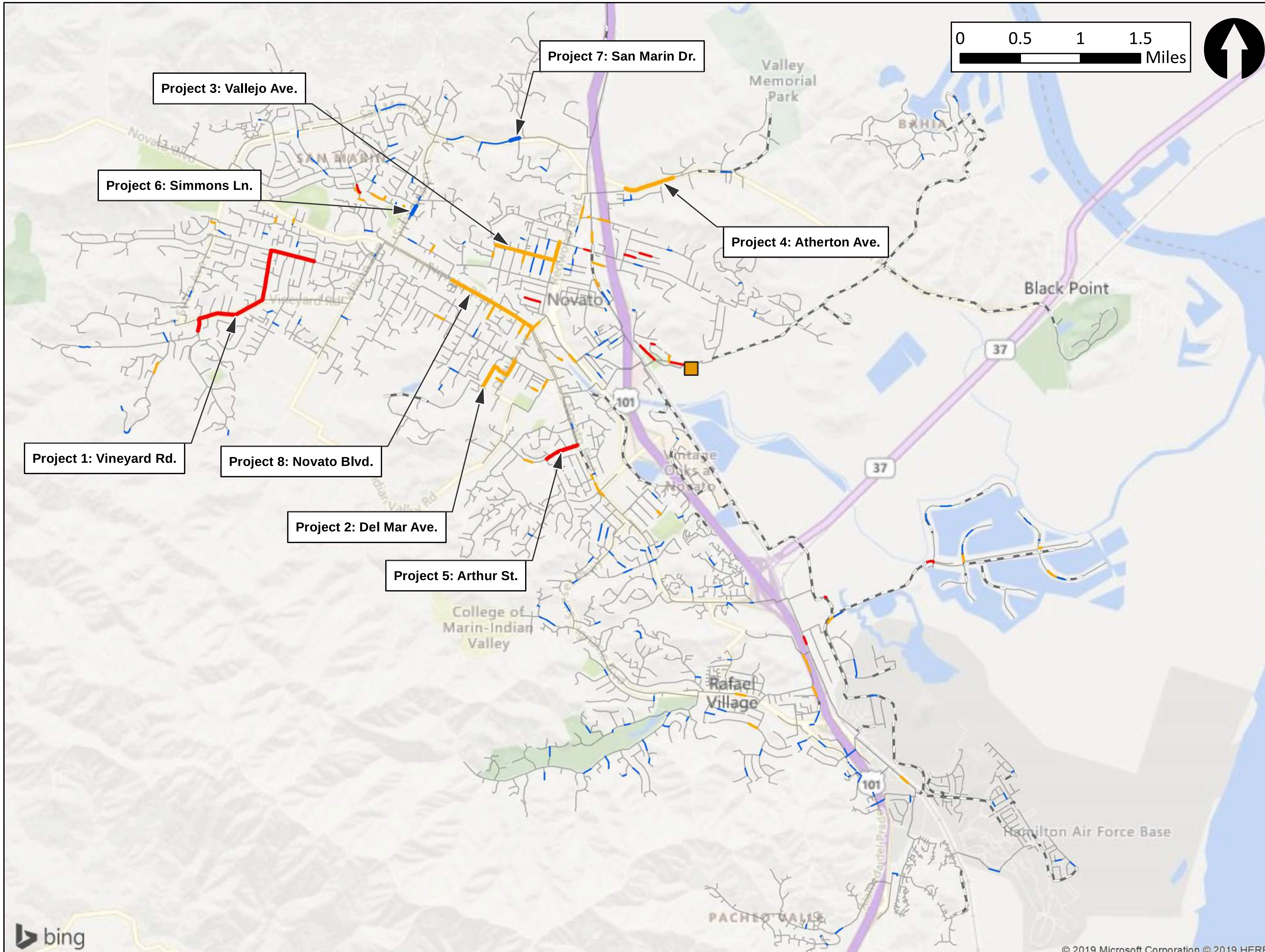
Costs for structural rehabilitation are based on the recommended method from the rehabilitation decision process described in Chapter 5. For costing purposes, pipe replacement was assumed for those pipes identified as needing further evaluation to determine if lining or replacement would be most appropriate method. Since pipe replacement could entail either trenchless (e.g., pipe bursting) or open-cut methods, costs were based on a rough average of the two methods; however, pipe replacement costs included cost for lower lateral replacement based on the number of lateral connections recorded in the CCTV inspection. As with capacity improvement projects, allowances of 30 percent for contingencies and 25 percent for engineering, administration, construction management, and legal costs were also added to the base construction costs. Estimated costs for the pipes identified for structural rehabilitation are included in **Appendix E**.

Table 6-1 presents the recommended capital improvement projects and estimated capital costs, and **Figure 6-1** shows the locations of the proposed improvements. **Figure 6-2 through Figure 6-9** provide more detailed maps showing the proposed capacity improvement projects.

Table 6-1: Proposed Capital Improvement Projects

Project ID[a]	Priority	Project Name	Description	Est. Capital Cost[b] ($)
Capacity Improvement Projects				
1	1	Vineyard Road	5,600 lf of 12" to 15" pipe on Angelica Ct., Vineyard Dr., Eucalyptus Ave. and Center Rd. to Western Dr.	5,230,000
2	2	Del Mar Avenue	2,015 lf of 10" pipe in Del Mar Ave., Hotchkin Dr., and Diablo Ave. from Hill Rd. to Center Rd.	1,770,000
3	2	Vallejo Avenue	2,775 lf of 8" to 12" pipe in Vallejo Avenue and 1st Street from 7th St. to Olive Ave.	1,860,000
4	2	Atherton Avenue	1,722 lf of 10" to 12" pipe in Atherton Ave. from east of Oak Shade Ln. to east of Binford Rd.	980,000
5	1	Arthur Street	1,182 lf of 15" pipe in Arthur Street from west of Hayes St. to S. Novato Blvd.	1,210,000
6	3	Simmons Lane	335 lf of 10" pipe in Simmons Ln. from Feliz Rd. to Kristin Ln.	92,000
7	3	San Marin Drive	319 lf of 8" pipe in San Marin Dr. west of Spinosa Way	135,000
8	2	Novato Boulevard	3,240 of 24" pipe in Novato Blvd. from east of Grant Ave. to Diablo Ave.	3,890,000
			Subtotal – Capacity Improvements	**15,170,000**
Structural Rehabilitation				
	1	High Risk	Major defect repair, rehabilitation, and replacement	2,500,000
	2	Medium Risk	Major defect repair, rehabilitation, and replacement	7,900,000
	3	Low/Very Low Risk	Major defect repair, rehabilitation, and replacement	18,700,000
			Subtotal – Structural Rehabilitation	**29,100,000**
			Priority 1 Total	**8,900,000**
			Priority 2 Total	**16,400,000**
			Priority 3 Total	**19,000,000**
			Total Estimated Capital Cost:	**44,300,000**

a. Corresponds to Deficiency ID in Table 4-4 except Project 8, which was triggered by increased flows after relief of other capacity deficiencies. Priorities are based on magnitude of predicted overflow volume (Priorities 1 and 2) or surcharge resulting in freeboard violations (Priority 3).
b. Estimated capital costs based on late 2018 dollars; include allowances of 30% of estimated construction cost for contingencies for unknown conditions and 25% for engineering, administration, construction management, and legal costs.



Legend

- Novato WWTP

Capacity Projects

- Priority 1
- Priority 2
- Priority 3

Structural Rehabilitation

- Priority 1 (High Risk)
- Priority 2 (Medium Risk)
- Priority 3 (Low/Very Low Risk)
- Force Main
- Gravity Main

Novato Sanitary District

Collection System Master Plan

Figure 6-1

Recommended Capital Improvement Projects

NOVATO SANITARY DISTRICT NSD
WOODARD & CURRAN

Sources: Novato Sanitary District, Bing Maps

Map Created: 7/17/2019

\\woodardcurran.net\shared\Projects\RMC\WCR\0049 Novato Sanitary District\0049-040 Collection System Master Plan\GIS\MXDs\Figure6-1_Recommended_Capital_imporv_Projects.mxd

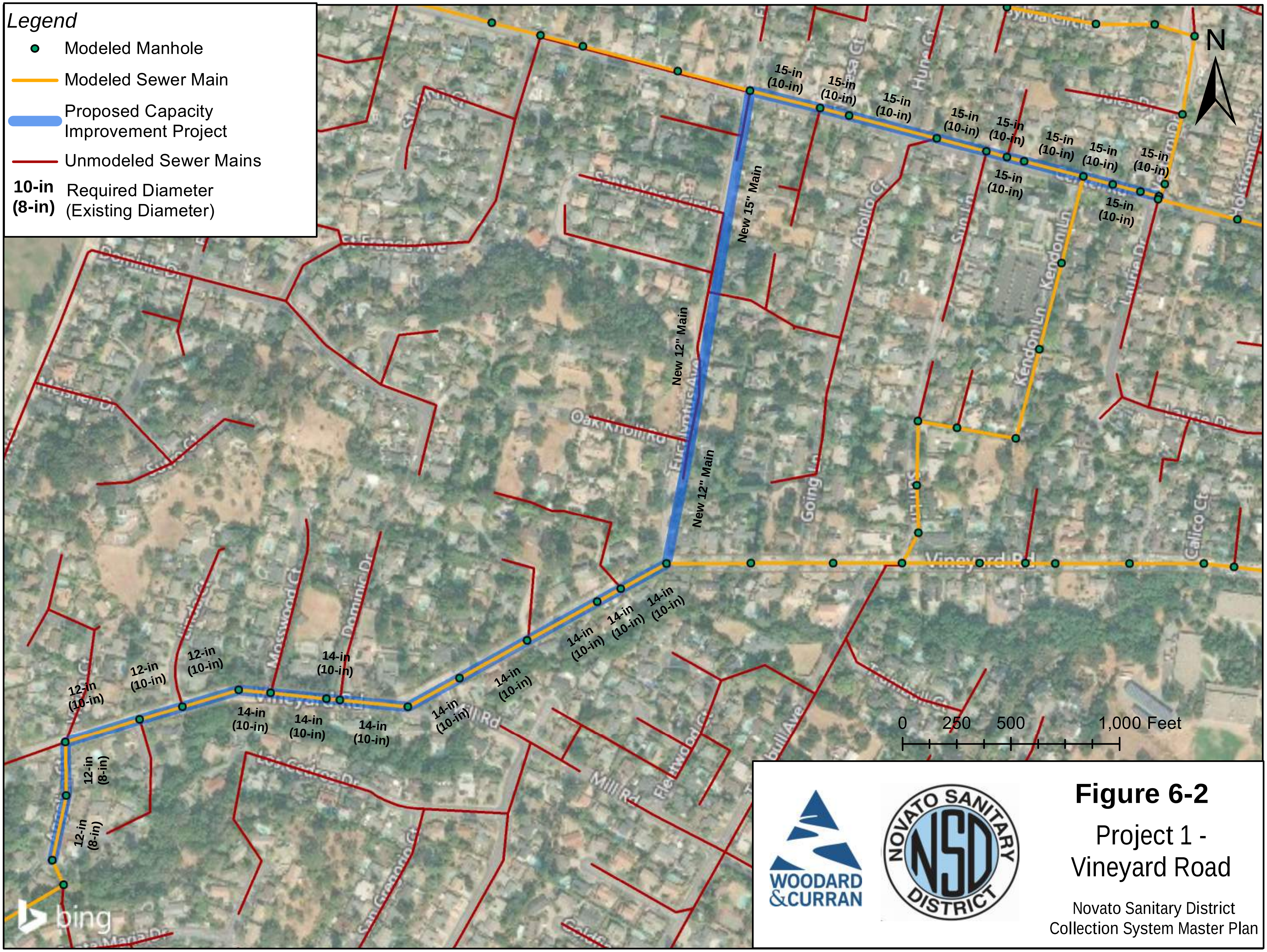


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Figure 6-3

Project 2 - Del Mar Avenue

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NOVATO SANITARY DISTRICT NSD

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Figure 6-4

Project 3 - Vallejo Avenue

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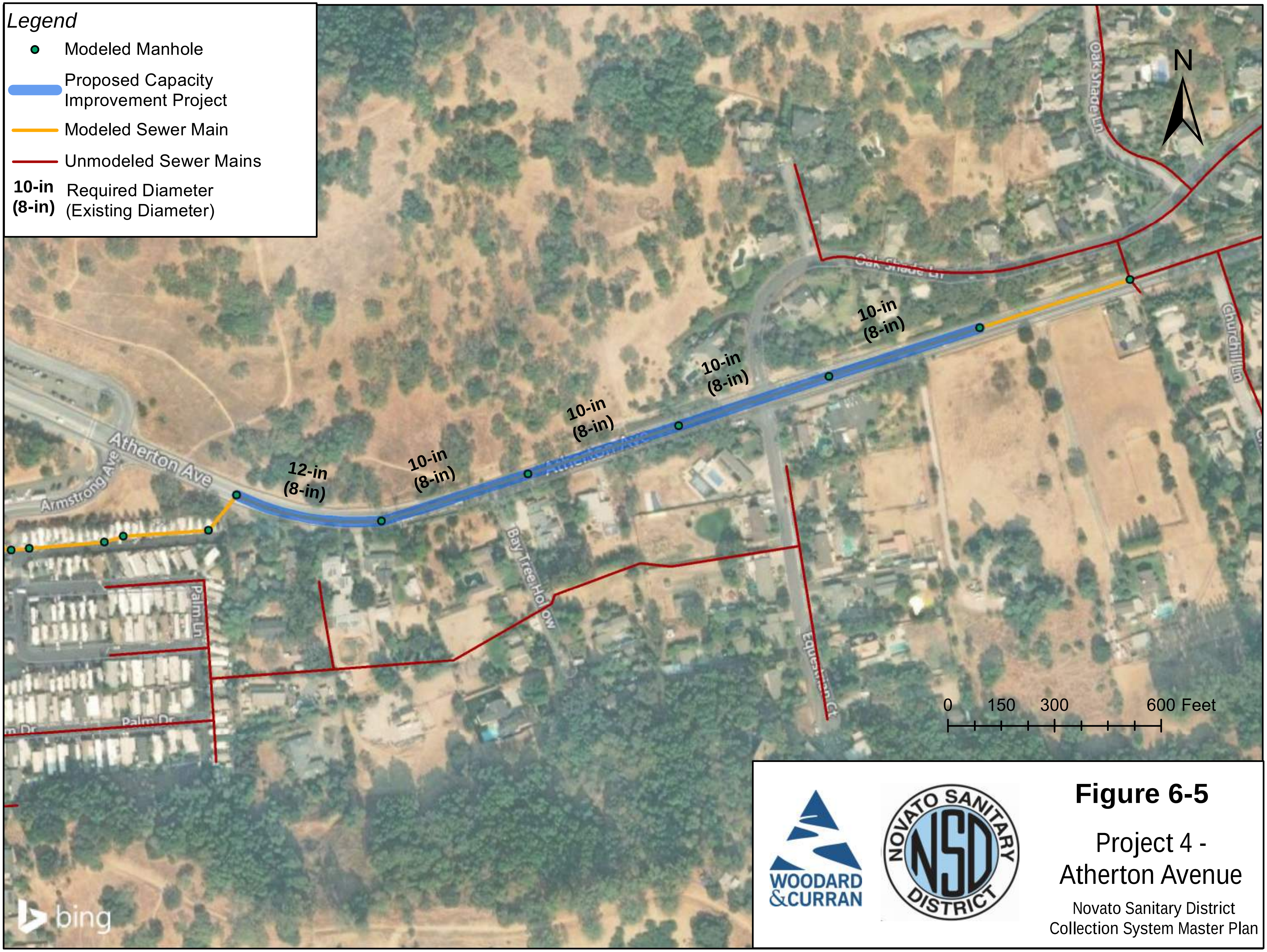

Figure 6-5

Project 4 - Atherton Avenue

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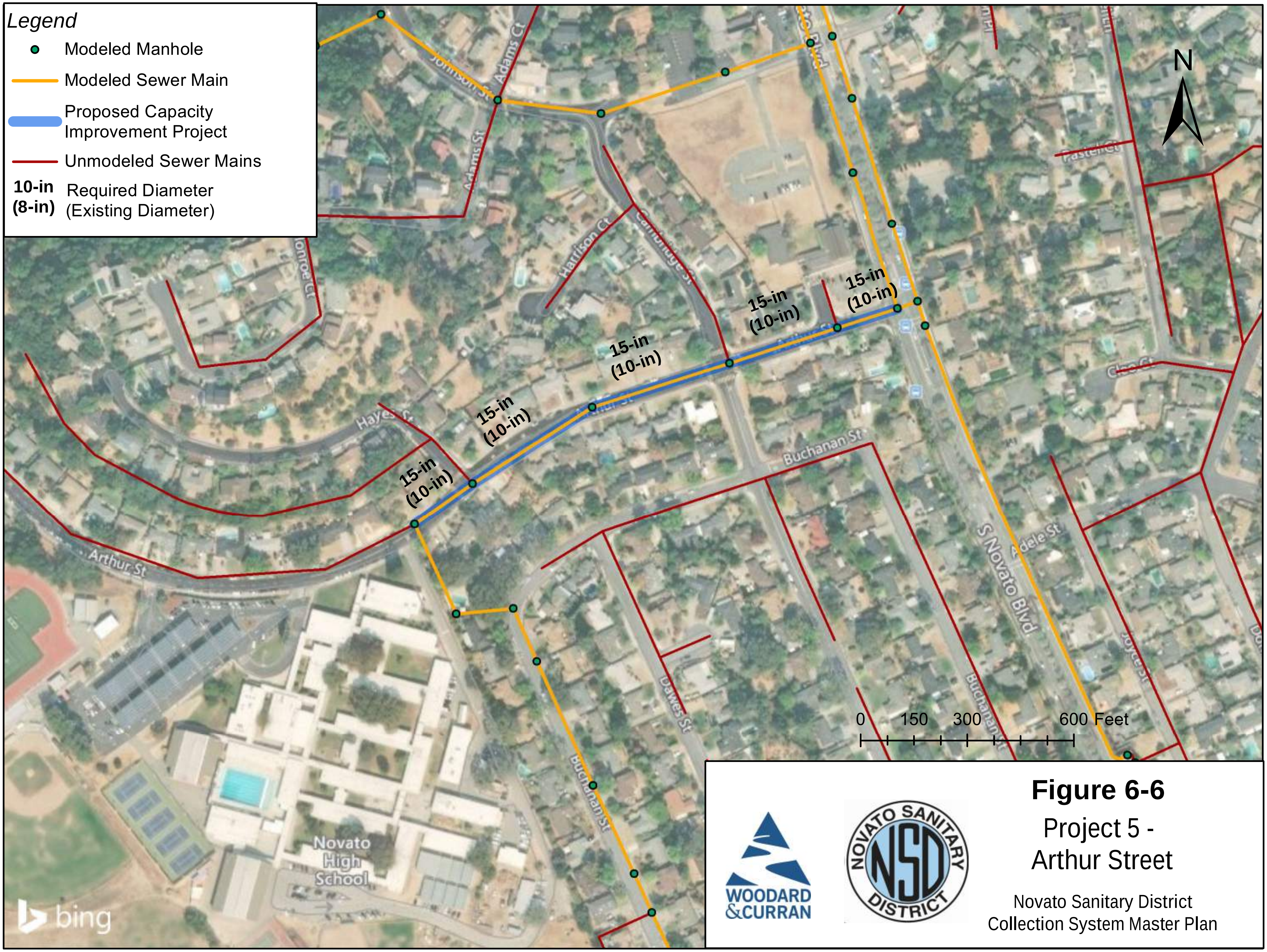

Figure 6-6

Project 5 - Arthur Street

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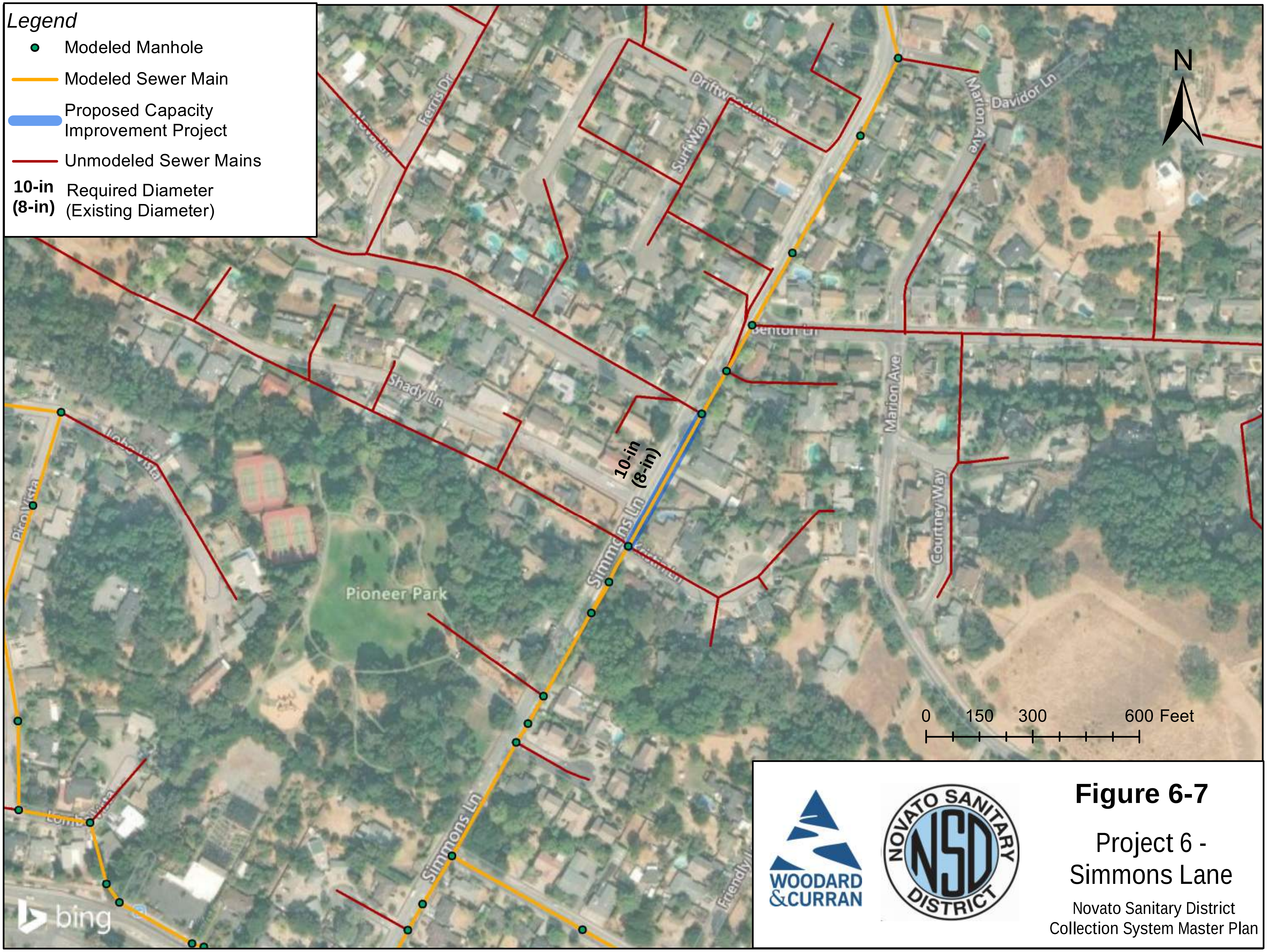

Figure 6-7

Project 6 - Simmons Lane

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Figure 6-8

Project 7 - San Marin Drive

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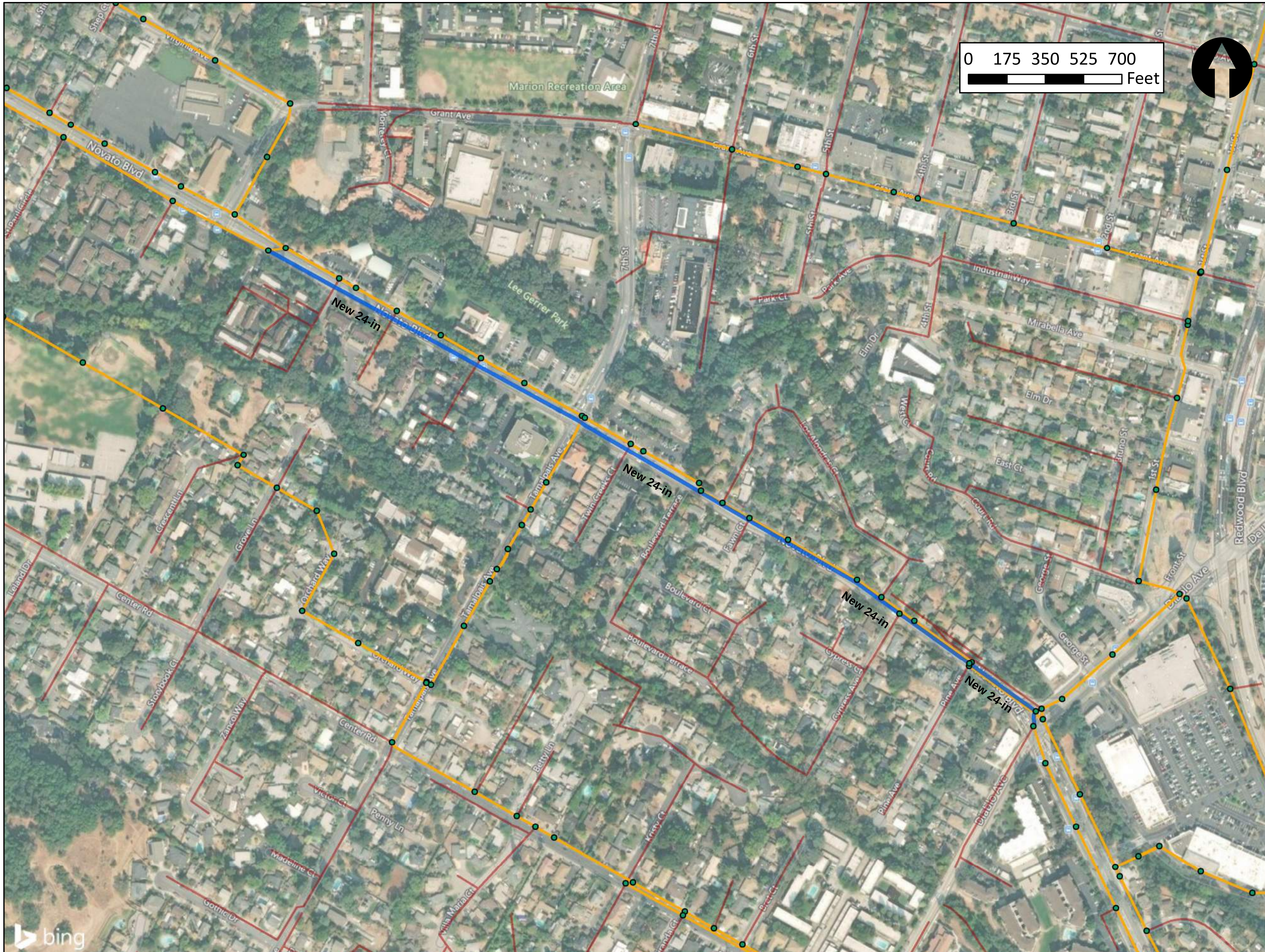
Map Created: 7/17/2019



Legend

- Modeled Manhole
- Proposed Capacity Improvement Project
- Modeled Sewer Main
- Unmodeled Sewer Mains
- **10-in (8-in)** Required Diameter (Existing Diameter)

Novato Sanitary District

Collection System Master Plan

Figure 6-9

Project 8 - Novato Boulevard

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Sources: Novato Sanitary District, Bing Maps

Map Created: 7/17/2019

6.2 Project Priorities

Relative priorities have been assigned to the projects based on the severity and extent of the deficiency. Capacity improvement projects have been prioritized based on the severity of the capacity deficiency as reflected in model-predicted overflow volume or freeboard under peak wet weather flow conditions, as indicated in Table 4-4. Although capacity improvement Project 8, Novato Boulevard, is not identified as an existing capacity deficiency in Table 4-4, the construction of other upstream capacity improvements will trigger the need for this project as higher peak flows can be conveyed downstream. The project will also provide relief for other upstream trunk sewers (for example, the 10-inch sewer in Virginia Street) that are also shown to be surcharged under peak wet weather conditions. Furthermore, the Novato Boulevard sewer improvements will need to be coordinated with the City of Novato’s planned road improvements, and the District must be able to move ahead with this project at any time based on the City’s plans. Therefore, this project was elevated to Priority 2 to reflect these considerations.

Sewers identified for repair, rehabilitation, or replacement due to occurrence of major structural defects were grouped into three priority groups based on their risk priority rating from the risk model. The capacity improvement projects and pipes recommended for structural rehabilitation are color-coded by priority in Figure 6-1.

Table 6-2 summarizes the recommended capital improvement CIP by priority.

Table 6-2: Capital Improvement Program Summary

Priority	Estimated Capital Cost ($)		
	Capacity Improvements	Structural Rehabilitation	Total
1	$ 6,440,000	$ 2,500,000	$ 8,900,000
2	$ 8,500,000	$ 7,900,000	$ 16,400,000
3	$ 230,000	$ 18,700,000	$ 19,000,000
Total	**$ 15,170,000**	**$29,100,000**	**$ 44,300,000**

6.3 Implementation Considerations for Capacity Improvement Projects

The following paragraphs discuss specific issues with respect to each of the proposed capacity improvement projects that will need to be considered in moving forward with implementing these projects.

6.3.1 Project 1 – Vineyard Road

As shown in Figure 6-2, this project would involve upsizing the existing sewers in Vineyard Road from Angelica Court to Eucalyptus Avenue, diverting flow into a new sewer in Eucalyptus Avenue to Center Road, and then replacing the existing 10-inch sewer in Center Road with a new 15-inch pipe downstream to Western Drive. This project is the longest and most expensive of the proposed capacity improvements, as well as the most critical with respect to predicted capacity deficiencies.

The area tributary to the Vineyard Road trunk sewer is a part of the larger overall tributary area for flow meter FM4 (see Figure 3-4), which is the basis for the I/I flow characteristics for the Vineyard Road tributary area. Therefore, before implementing the project, it is recommended that additional flow monitoring be conducted to further confirm the I/I rates specifically for the Vineyard Road tributary area to confirm the need for and required capacity of the improvements.

Various alternatives were evaluated for this project, including upsizing the entire existing sewer in Vineyard Road to Wilson Avenue and in Wilson Avenue to Center Road, as well as other alignments for a diversion pipeline between Vineyard Road and Center Road. While the Eucalyptus Avenue alignment was found to be the most viable alternative, more detailed evaluation of the potential alternative alignments should be conducted following the flow monitoring study.

District staff have identified the pipe in Center Road as needing replacement due to its age and condition. Therefore, the recommended project offers the opportunity to prioritize replacement of this pipe as part of the capacity improvement project. It may be advisable to phase the project such that the Center Road portion is constructed first, followed by the upstream segments of the project.

6.3.2 Project 2 - Del Mar Avenue

As shown in Figure 6-3, this project would upsize the sewer in Del Mar Avenue north of Hill Road, then divert the flow into a new sewer in Hotchkins Drive to Diablo Avenue. The existing sewer in Diablo Avenue would also need to be upsized to Center Road to accommodate the diverted flow. The proposed alignment would avoid the need for construction in the existing sewer easement between the end of Del Mar Avenue to Center Road.

Other alternatives for this project were considered, including diverting flow east along Hill Road to Diablo Avenue, or west from Canyon Road along Hill Road to Bradley Avenue. Neither of these alternatives were found to be favorable compared to the recommended alignment.

6.3.3 Project 3 - Vallejo Avenue

The Vallejo Avenue relief sewer, shown in Figure 6-4, is intended to provide relief for the predicted capacity deficiencies in the existing 10- and 12-inch sewers in Grant Avenue from Sixth Street to First Street. The project would divert a significant portion of the flow that now reaches Grant Avenue at the intersection of Vallejo Avenue and Seventh Street into a new sewer in Vallejo Avenue. The existing small diameter pipes in Vallejo Avenue from Jade Court to First Street would be upsized to handle the diverted flow. The pipe in First Street from Vallejo Avenue to Olive Avenue would also need to be upsized. The Vallejo Avenue alignment was selected to avoid construction in Grant Avenue, which is a busy, tree-lined commercial street. Furthermore, the sewer in Grant Avenue is relatively new (replaced about 15 years ago with new PVC pipe).

Additional flow monitoring is highly recommended to confirm the need for and sizing of this project, as the tributary area was not metered directly.

District staff also note that the 24-inch trunk sewer crossing the tracks to Railroad Avenue along Olive Avenue was recently replaced at a deeper elevation, intended to allow the sewers in the area west of this point to ultimately be replaced at steeper slopes to improve performance. While not directly related to the Vallejo Avenue capacity improvement project, it may be possible to coordinate sewer replacement in this area with the relief sewer project.

6.3.4 Project 4 – Atherton Avenue

This project, shown in Figure 6-5, would upsize five segments of existing 8-inch pipe to 10 and 12-inch diameter in Atherton Avenue east of Binford Road to alleviate model-predicted capacity deficiencies. District staff have not reported any issues in this line, and the tributary area to the Atherton Avenue sewer is just a part of the overall tributary area for flow meter FM1, which also includes substantial area on the west side of Highway 101. Therefore, before implementing the project, it is recommended that additional flow monitoring be conducted to further confirm the I/I rates specifically for the Atherton Avenue tributary area to confirm the need for and required capacity of the improvements.

6.3.5 Project 5 – Arthur Street

The Arthur Street sewer improvement project, shown in Figure 6-6, would replace the 10-inch sewer in Arthur Street west of S. Novato Boulevard with a 15-inch pipe constructed slightly deeper than the existing sewer. This would provide the additional capacity needed to convey the peak design flows, as well as minimize, to the extent possible, surcharge in the upstream sewer in Buchanan Street, which is relatively shallow.

As a potential alternative, a portion of the flow in Arthur Street could be diverted further upstream to an existing 8-inch sewer in Taft Court by constructing a short segment between two existing manholes at the intersection. This would reduce enough flow in the Arthur Street sewer to allow upsizing the pipe by pipe bursting rather than replacement. This alternative should be investigated further during the implementation phase of the project.

6.3.6 Project 6 – Simmons Lane

The Simmons Lane project, shown in Figure 6-7, would upsize one pipe segment between Feliz Road and Kristin Lane from 8 to 10 inches. This would alleviate predicted surcharge in the Simmons Lane sewer and would be consistent with the downstream segments in Simmons Lane which are 10-inch diameter.

It should be noted that further downstream in Simmons Lane at the intersection of Virginia Street, flow is conveyed east into the 10-inch sewer in Virginia Street, which is predicted to be surcharged (but not in violation of freeboard criteria) under design storm peak weather flow conditions. Constructing a new pipe in Simmons Lane from this intersection to Novato Boulevard and connecting into the southern (24-inch) trunk sewer in Novato Boulevard would help alleviate the surcharge in Virginia Street as well as upstream in Simmons Lane. Coordination of this improvement with other improvements in Simmons Lane could be considered during project implementation.

6.3.7 Project 7 – San Marin Drive

Project 7 would upsize one pipe segment in San Marin Drive immediately wet of Spinosa Way from 6-inch to 8-inch diameter (Figure 6-8) This segment is very flat (0.6 percent slope) with a drop manhole at the downstream end, possibly because of an elevation conflict where the sewer crosses a storm drain at the Spinosa Way intersection. As part of implementing this project, the potential to steepen the slope of the pipe should be investigated.

6.3.8 Project 8 – Novato Boulevard

The Novato Boulevard sewer improvement project, shown in Figure 6-9, would involve construction of a parallel 24-inch trunk sewer in Novato Boulevard from east of Grant Avenue to Diablo Avenue. The upstream portion of this trunk sewer extending west to Wilson Avenue was constructed over 30 years ago, and the need for this downstream section has been identified in previous planning studies but has been deferred until the time that the City of Novato implements its planned road improvements in this area. The project would provide capacity relief not only for the existing trunk sewer in this portion of Novato Boulevard but also for upstream sewers in Virginia Street, as discussed above. Note that in downstream section of this project, between Cypress Avenue and Diablo Avenue, there are multiple parallel small diameter sewers; therefore, consolidation of these pipes should be investigated as part of project implementation.

Novato Boulevard is a very busy thoroughfare, and construction will be challenging and expensive. While the cost estimate for this project is based on open cut construction, trenchless methods such as microtunneling should be evaluated to minimize disruption to businesses and traffic. Although identified as a Priority 2 project in this Master Plan, the District will need to be able to move forward with this project at any time, depending on the timing of the City's road improvements.

6.4 General Implementation Recommendations

The District should begin planning for the implementation of the Capital Improvement Program recommended in this Master Plan, starting with the highest priority projects. The first step in implementation would be the development of project validation plans to confirm the need for specific projects (e.g., based on flow monitoring, as discussed above) and further refine project sizing and alignments, proposed construction methods, and coordination between needed capacity and structural rehabilitation improvements. This information will allow the District to develop a detailed implementation plan for the CIP, in conjunction with associated financial studies, which would include updated cost estimates and proposed project schedules consistent with the District's financial capabilities.

The following items should also be considered in project scheduling and design, and in future updates of the Master Plan.

- The District should consider conducting additional flow monitoring or observation to document flow levels during large storm events at locations in the system where the model predicts significant surcharge. Flow levels during large storm events should be compared to the water levels simulated by the hydraulic model to verify if the modeling predictions for the design storm seem reasonable, and to confirm the need for and refine project sizing if necessary. As noted in the previous section, additional, more intensive flow monitoring to further confirm model flow predictions should be conducted to better quantify I/I rates and peak wet weather flows for the areas tributary to specific projects.

- Smoke testing and other field investigations should be considered for areas where more intensive flow monitoring indicates particularly high I/I rates, especially where potential elimination of needed capacity improvements could be accomplished through modest I/I reductions.

- The alignments and sizes of all recommended projects should be verified with detailed predesign analyses, including topographic surveys, geotechnical investigations, utility research, and constructability reviews.

- The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.

- The hydraulic model has been developed to assist the District in performing capacity analyses and updating the Master Plan in the future. The model should be kept up-to-date with any changes to existing sewer connections, development plans, and sewer system facilities. The District could also refine the model by incorporating more accurate survey data on pipe rim and invert elevations as it becomes available.

- The District should continue to keep its sewer GIS mapping and data up-to-date and review the updated GIS database developed for this study to confirm and/or correct sewer attribute data as needed.

- The District should accelerate CCTV inspection of high and medium risk priority pipes that do not have CCTV inspection data, and should ensure that all future inspections rigorously follow NASSCO PACP observation coding standards.

- The District should undertake studies to evaluate if some of its smaller lift stations, such as Enfrente and Cypress, could be eliminated and replaced with gravity bypass sewers in order to reduce potential risk from pump station outages or failures.

This Master Plan report is intended to be a working document to be refined and updated as additional data and new planning information become available. The capacity assessment should be updated whenever there are major changes in planning assumptions or, at a minimum, every five to ten years.