FINAL REPORT | OCTOBER 2025

Mare Island Infrastructure Assessment

PREPARED FOR

City of Vallejo
and
Vallejo Flood & Wastewater District

PREPARED BY

WEST YOST
Water. Engineered.

Mare Island Infrastructure Assessment

Prepared for

City of Vallejo
and
Vallejo Flood & Wastewater District

Project No. 174-60-22-06



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Appendix 8D. VIPFiber System

- 2024 Contact Information
- Existing and Proposed VIPFiber Utilities

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LIST OF ACRONYMS AND ABBREVIATIONS

2015 WMP	2015 City of Vallejo Master Water Plan
AC	Asphalt Concrete
AC/AB	Asphalt Concrete/Aggregate Base
ACCMP	Asphalt Coated Corrugated Metal Pipe
ADWF	Average Dry Weather Flow
AWWA	American Water Works Association
BEM	Broadband Electromagnetic
BFE	Base Flood Elevation
bgs	Below Ground Surface
CCTV	Close Circuit Television
cfs	Cubic feet per second
CI	Cast Iron
CIP	Concrete Iron Pipe
CIPP	Cured In Place Pipe
CIR	Cold-in-Place Recycling
City	City of Vallejo
CIWEM	Chartered Institution of Water and Environmental Management
CL	Lean Clay
CMP	Corrugated Metal Pipe
CPT	Cone Penetration Testing
CTM	California Test Method
DCP	Dynamic Cone Penetrometer
DIP	Ductile Iron Pipe
District	Vallejo Flood and Wastewater District
DM	Design Manual
DMI	Distance-Measuring Instrument
DTSC	Department of Toxic Services
DU	Dwelling Unit
FDR	Full-Depth Reclamation
FEMA	Federal Emergency Management Agency
FWD	Falling Weight Deflectometer
GHG	Greenhouse Gas
GIS	Geographic Information System
GIS	Geographic Information System
GPR	Ground Penetrating Radar
GWI	Groundwater Infiltration
GWR	Ground Water Rise
HDM	Highway Design Manual
HDPE	High-Density Polyethylene
HGL	Hydraulic Grade Line
HMA	Hot Mix Asphalt

IE	Island Energy
Island	Mare Island
LFR	Levine Fricke
LS	Lift Station
MACP	Manhole Assessment Certification Program
MCC	Motor Control Center
mgd	Million Gallons per Day
MI	Mare Island
MIC	Mare Island Company
MIIA	Mare Island Infrastructure Assessment
MISP	Mare Island Specific Plan
MLLW	Mean Lower Low Water
MLW	Mean Low Water
Mmax	Maximum Moment Magnitude Earthquake
MRP 3.0	San Francisco Bay Municipal Regional Stormwater Permit
MTC	Metropolitan Transportation Commission
MVA	Mega Volt-Ampere
MW	Million Watts
NASSCO	National Association of Sewer System Companies
NAVD	North American Vertical Datum
NAVFAC	Naval Facilities Engineering Command
NCE	Nichols Consulting Engineers
NEHRP	National Earthquake Hazards Reduction Program
NPDES	Non-Point Discharge Elimination System
O&M	Operation & Maintenance
OBC	Old Bay Clay
OBM	Old Bay Mud
OCC	Olivia Chen Consultants
PACP	Pipeline Assessment Certification Program
PCC	Portland Cement Concrete
PCI	Pavement Condition Index
PDWF	Peak Day Weather Flow
PE	Polyethylene
PGA	Peak Ground Acceleration
PI	Plasticity Index
PPC	Pittsburg Power Company
psi	Pounds Per Square Inch
PVC	Polyvinyl Chloride
PVC	Polyvinyl Chloride
PWWF	Peak Wet Weather Flow
RCP	Reinforced Concrete Pipe
RDII	Rainfall-Dependent Inflow and Infiltration

RUL	Remaining Useful Life
SCADA	Supervisory Control and Data Acquisition
SLR	Sea Level Rise
SLRG	Sea Level Rise Guide
SLS	Sea-Level Scenario
SSO	Sanitary Sewer Overflow
TC	Terra Cotta
TI	Traffic Index
UL	Useful Life
UPS	Uninterrupted Power Supply
UT	Ultrasonic Thickness
UWMP	Urban Water Management Plan
UXO	Unexploded Ordnance
VCP	Vitrified Clay Pipe
VFD	Variable Frequency Drive
VFWD	Vallejo Flood and Wastewater District
WSE	Water Surface-Elevations
WWTP	Wastewater Treatment Plant
YBM	Young Bay Mud

MARE ISLAND INFRASTRUCTURE ASSESSMENT REPORT

The Mare Island Infrastructure Assessment (MIIA) was completed for the City of Vallejo (City) and the Vallejo Flood and Wastewater District (District) to support infrastructure planning and coordinated redevelopment across Mare Island (Island). The assessment provides a comprehensive evaluation of the existing condition and capacity of Mare Island's core infrastructure systems including water, sanitary sewer, storm drainage and flood control, roads, dry utilities, and geotechnical conditions and identifies the improvements needed to support both near-term and long-term development of the island.

The project was initiated in January 2023 and completed in November 2025, with work conducted in close coordination with the City, District, and Mare Island Company.

INTRODUCTION (CHAPTER 1)

Mare Island has evolved from a historic U.S. naval shipyard into a growing residential, commercial, institutional, and industrial community within the City. Following the Navy's decommissioning of the shipyard in 1996, the City and District assumed responsibility over time for the Island's water, sanitary sewer, storm drainage, flood control, and road systems. In 2004, Mare Island was opened for public access and a decades-long process of redevelopment was initiated. Today, the City partners with Mare Island Company to guide the Island's redevelopment.

The MIIA builds upon prior studies completed since the adoption of the Mare Island Reuse Plan in 1994. At project outset, the team compiled and reviewed available data including prior infrastructure reports, geotechnical studies, as-built drawings, and GIS mapping to inform current analyses and avoid duplication of efforts.

The MIIA evaluates infrastructure systems under two planning scenarios:

- **Existing/Near-Term Scenario** (through 2027) – Represents known development and infrastructure needs based on current land use and projects in progress.
- **Buildout Scenario** (through 2063) – Represents full redevelopment of the island based on long-range planning assumptions and growth forecasts.

Each infrastructure system was evaluated using a consistent methodology; where applicable, field investigations, condition assessments, flow monitoring, and modeling were conducted to support analyses. Recommendations were developed for each system and land use scenario, with a focus on coordination between systems to reduce rework, minimize disruptions, and support resilient, phased implementation.

The MIIA findings are presented in nine chapters, as listed below:

- Chapter 1 – Introduction
- Chapter 2 – Land Use
- Chapter 3 – Water System
- Chapter 4 – Storm Drainage and Flood Control System
- Chapter 5 – Sanitary Sewer System
- Chapter 6 – Geotechnical Evaluation (including the Seawall)

- Chapter 7 – Roads System
- Chapter 8 – Dry Utility Systems
- Chapter 9 – Integrated Recommendations

For each land use scenario (existing/near-term and buildout), the MIIA identifies current system limitations, evaluates future service needs, and provides planning-level recommendations to ensure that infrastructure is not a constraint to achieving the Island's full development potential. The report enables the City, District, and their partners to plan proactively, align capital investments, and support phased, coordinated redevelopment.

The remaining sections of this Executive Summary summarize the key tasks, findings, and recommendations presented in each chapter.

LAND USE (CHAPTER 2)

Future growth for Mare Island was established in the context of two land use scenarios: an existing/near-term scenario, which represents development on the island through 2027, and a buildout scenario, which represents expected development on the island through 2063. Land use projections were developed in coordination with the City, District, and Mare Island Company, with input from key stakeholders and large utility users including Touro University, Mare Island Dry Docks, and Lind Marine.

In the existing/near-term scenario, major development includes the Connolly Street Corridor project, activation of Dry Dock 1 at Mare Island Dry Docks, and a 15-percent operational expansion at Lind Marine.

The buildout scenario was based on a preliminary draft plan of the Mare Island buildout land use (Mare Island Company, February 2023), a 60-percent increase in enrollment at Touro University, and an additional 15-percent growth at Mare Island Dry Docks beyond near-term levels.

This land use framework was used to estimate utility demands, evaluate infrastructure capacity, and size recommended improvements across the water, sewer, and storm drainage systems.

WATER SYSTEM (CHAPTER 3)

The existing Mare Island water system consists of 35.4 miles of transmission and distribution pipelines (ranging from 2-inch to 24-inch) and one water tank. Water service is provided to the island through three transmission pipelines crossing Mare Island Strait – two in the north and one in the south.

The water system evaluation included: collecting and reviewing existing documents and mapping; identifying proposed design criteria and preferences to be used in development of infrastructure improvements; developing a hydraulic model to evaluate the system capacity, performance, and proposed infrastructure improvements; performing condition assessment of select water system pipelines; performing a seismic and condition assessment of the water tank; and developing an integrated set of recommendations for the existing/near-term and buildout scenarios.

Based on available age information, approximately 73 percent of the pipelines are more than 75 years old, and 60 percent of the pipelines are more than 100 years old. Condition information for the pipelines is largely unavailable, so age and industry standard useful life values were used to estimate pipeline condition. Physical condition testing to estimate wall thickness loss was performed in the field for eight metallic pipelines. Three of the eight sites had calculated thickness loss that is considered negligible

(within manufacturing tolerance). The remaining five sites exhibited moderate to significant wall loss (ranging from 13 to 26 percent). Due to the lack of record information (resulting in unknown pipe age and material), in addition to varying soil and groundwater conditions, it is likely that the conditions of pipe will differ in other locations of the system. Thus, the age-based condition assessment results were not modified based on field testing results.

The water tank condition assessment included visual inspection of the tank exterior and diving of the tank interior. A desktop seismic assessment was performed. Based on the assessments, the tank is in overall good condition, with the isolated areas of moderate surface damage (spalling and staining) on tank walls, and minor to moderate corrosion on interior pipe fittings. The seismic assessment concluded that the maximum allowable water level in the tank should be limited to 7 feet below the roof soffit to maintain structural integrity during and after a seismic event. Due to current operating conditions, the tank water level already remains more than 7 feet below the roof soffit.

The hydraulic capacity and performance evaluation included development of water demands for the existing, near-term, and buildout land use scenarios and evaluation of pressure and fire flow availability for each of those scenarios. The City's existing performance and operational criteria were used to identify deficiencies and recommended improvements:

- The existing condition and near-term scenarios showed pressures below City criteria around the Mare Island tank. These pressure issues are known to the City.
- The near-term scenario identified fire flow deficiencies and associated pipeline improvements around the Connolly Street Corridor development.
- The buildout scenario identified pressures below City criteria around the Mare Island tank and along the south end of the island near Ribeiro Road. The buildout scenario identified fire flow deficiencies across the island and was used to develop associated pipeline improvements.

A hydraulic evaluation was performed on the three Mare Island Strait crossing pipelines for the near-term scenario. Assuming the two old crossings are not in service, operation with only one crossing does not affect the system ability to meet demands under normal operating conditions. Under fire flow conditions, available fire flow is reduced and some hydrants fall below minimum requirements. Replacement of the south crossing would provide a long term, reliable, redundant water connection for Mare Island.

Water System Recommendations

Based on the assessments and evaluations performed on the water system, an integrated set of recommendations for each land use scenario was developed. All new infrastructure should meet City of Vallejo design standards.

Existing/Near-term (by 2027) recommendations include:

- Replace 20,826 feet of pipeline, including the south transmission pipeline crossing to address capacity and condition deficiencies. Replace associated valves and hydrants as pipelines are replaced.
- Review existing hydrant spacing and locations to meet Fire Marshal specifications.
- Review existing valve spacing to provide operational flexibility.

- For the tank, repair delamination and corrosion of exterior, replace inlet pipe flap valve flange and overflow pipe flange, replace all interior piping fittings with stainless steel fittings and recoat/re-touch up interior piping coating.

Buildout (by 2063) recommendations include:

- Install or replace 126,936 feet of pipeline to address capacity and condition deficiencies. Replacements should be prioritized based on leak/break history, and pipelines critical to the system (transmission mains and pipelines providing looping and redundancy).
- For the tank, perform a condition assessment every 10 years and implement recommended improvements.

STORM DRAINAGE AND FLOOD CONTROL SYSTEM (CHAPTER 4)

Mare Island's storm drainage and flood protection is provided through a network of 34.6 miles of gravity pipelines and one lift station (Lift Station 15) which convey storm water generally from the west/central part of the Island (interior) to the east, discharging via storm drain outlets to the Mare Island Strait. A seawall exists on much of the eastern edge of the Island bordering Mare Island Strait. The purpose of the seawall is to separate land and water areas, limiting damage due to wave action and storm surge, while also providing locations for ships to berth.

The storm drainage and flood protection system evaluation included: collecting and reviewing existing documents and mapping; identifying proposed design criteria and preferences to be used in development of infrastructure improvements; developing a hydraulic and hydrodynamic model to evaluate the capacity needs of the proposed infrastructure improvements; performing condition assessment of Lift Station 15 and select structures (maintenance holes and inlets); and developing an integrated set of recommendations for the existing/near-term and buildout scenarios.

The majority of existing storm drainage pipelines do not meet current District engineering standards for pipe coverage, slope, or minimum pipe size. Previous studies performed by the District concluded that existing infrastructure mostly lacked adequate capacity and recommended nearly full replacement; additionally, the previous studies did not address sea level rise, which further necessitates upgrades to the storm drain and flood control infrastructure. For these reasons, detailed condition assessment (age-based or other methods) of the existing storm drain and flood control system was not performed as part of this project.

Existing District planning and design documents, along with local, state, and national best-available guidance documents on sea level rise were reviewed to determine future sea level rise (SLR) projections for Mare Island. Three industry standard projections for SLR were established:

- Current: Sea level at 10 feet (based on current FEMA Flood Insurance Rate Maps, and considering the 100-year tide, storm surge, and riverine flow)
- Year 2070: Sea level at 13.5 feet (3.5 feet of increase)
- Year 2100: Sea level at 17.0 feet (7.0 feet of increase)

Some areas of Mare Island (mostly undeveloped areas in the north and west) are currently below 10 feet elevation. The infrastructure needed for SLR does not need to be constructed immediately, but current and future infrastructure planning and designs do need to address and accommodate the required future SLR infrastructure.

The hydraulic capacity evaluation included development of a hydraulic and hydrodynamic model to simulate storm and drainage conditions and develop recommended infrastructure to serve the Island for future stormwater and sea level rise conditions (to Year 2100, or 17.0 feet sea level elevation). The sland was divided into 12 Improvement Areas (IAs) based on similar features and infrastructure needs. For each area, infrastructure improvements were developed which included a combination of new storm drains, lift stations, soil fill, detention storage, and seawalls.

Cursory condition assessment included inspection of 23 storm drain maintenance holes and drain inlets, which indicated that 16 out of 24 would require replacement to meet current standards. Visual assessment of Lift Station 15 indicates the facility is in moderate to severely compromised condition and is not reliable. LS-15 currently provides drainage and flood protection to the Highway 37 interchange, the surrounding undeveloped, low-lying areas of North Mare Island, and portions of the Alco and EPS properties. All other buildings and structures in areas of North Mare Island previously serviced by LS-15 have been demolished or decommissioned.

Storm Drainage and Flood Control System Recommendations

Based on the assessments and evaluations performed on the storm drainage and flood control system, an integrated set of recommendations was developed for each land use scenario and sea level rise projections. All new infrastructure should meet District design standards.

Existing/Near-term (by 2027) recommendations include:

- The primary use of LS-15 is to serve the Highway 37 interchange. LS-15 will not be required to serve Mare Island in the future. Therefore, it is recommended that coordination with Caltrans should be implemented to advise them that 1) LS-15 is moderately to severely compromised and not reliable, 2) The primary purpose of LS-15 to serve the Highway 37 interchange and LS-15 will not be required to serve Mare Island in the future, and (3) Caltrans should design and construct their own lift station in their right-of-way.
- Evaluate the seawall and develop a strategy to provide protection for the 2100 water level of 17.0 feet.
- Perform visual inspections of all maintenance holes and inlets. Replace those constructed of non-standard materials (e.g., brick, corrugated metal) in system areas not scheduled for demolition or rerouting.
- Perform visual inspection (via maintenance holes or CCTV) to identify corrugated metal pipelines. Prioritize replacement of corrugated metal pipelines.
- Implement infrastructure improvements to developed areas below an elevation of 10 feet in Infrastructure Areas 3, 4, 5, 7, 8, 9, 10, and 11.

Buildout (by 2063) recommendations include:

- Implement infrastructure improvements in all Infrastructure Areas as development occurs (if currently undeveloped) and to meet 2100 sea levels (17.0 feet).

It should be noted that the design criteria used to develop the infrastructure improvements for both scenarios are based on guidelines that are regularly changing (e.g., the State of California Sea Level Rise Guidance document and District Engineering Standards including rainfall design storm data). As the scientific understanding of the impacts of climate change are further refined in the future, it is expected that international, federal, State, and local agencies will continue to provide updates on climate and sea level rise trends, which will in turn be used to update guidance documents. Development of future solutions to address storm drainage and sea level rise on Mare Island should be based on the most current and best available science at the time any specific project is designed and constructed. While this report emphasizes capital improvements, solutions should be integrated with site-specific and community-level adaptation planning, regional and cross-jurisdictional collaboration, and consider the site-specific land uses and unique characteristics of Mare Island. Also, at the time any specific project is designed and constructed, the design should include vulnerability assessments and adaptation strategies based on the proposed land use.

SANITARY SEWER SYSTEM (CHAPTER 5)

Mare Island's sanitary sewer system consists of network of 22.6 miles of gravity pipelines, maintenance holes, nine lift stations, and force mains which convey wastewater generally from the south and north ends of the Island to the DOM4 (domestic) lift station. Wastewater is pumped off the island by the DOM4 lift station through a force main that runs below the Mare Island Strait/north of the Napa River Causeway and connects to the mainland trunk sewer system. On the mainland, the District's north interceptor trunk sewer connects to the Ryder Street wastewater treatment plant (WWTP).

The sewer system evaluation included: collecting and reviewing existing documents and mapping; condition assessment of select sanitary sewer system pipelines, manholes, and lift stations; developing a hydraulic model to evaluate the capacity needs of the proposed infrastructure improvements; and developing an integrated set of recommendations for the existing/near-term and buildout scenarios.

Closed-circuit television (CCTV) inspection was performed on 70 sewer segments which indicate that 11 segments have severe structural defects (1,463 feet). Visual condition assessment was performed on 120 maintenance holes which indicate that 102 maintenance holes have at least one component that is defective. Smoke testing was performed on all areas of the island with known sanitary sewer systems (active or otherwise), which identified 14 severe and 44 other defects in the system. Defects in pipelines and maintenance holes, and connections to other parts of the system contribute to inflow and infiltration (I&I), that reduces capacity available for the intended sanitary sewer flows.

Visual condition assessment was performed in the field for the nine active domestic sewer lift stations on Mare Island: DOM2, DOM3, DOM4, DOM6, DOM7, DOM8, DOM11, DOM13, and the L Street lift station. Improvement recommendations were developed based on observed condition and defects and were grouped into three categories: Priority 1 which are critical to current operation; Priority 2 for continued operation beyond 2027; and Buildout which are expected between 2027 and 2063. Six of the DOMs require improvements which are critical to current operation. It is expected that all DOMs will require full replacement or rehabilitation before 2063. All facilities have space available on-site for future expansion.

Flow monitoring was performed for a total of ten weeks, which was used to establish a baseline for sanitary sewer flows at each monitoring site, measure peak flow characteristics, isolate and analyze I&I, and monitor the level and pumping cycles at DOM4 as shown:

- I&I analysis estimated existing I&I rates ranging from 6,300 gallons per day (gpd) per acre to 17,700 gpd per acre for the three flow monitoring basins. For reference, the District I&I allowance for pipes built pre-1970 is 4,000 gpd per acre. For pipelines built 1970 and later, the I&I allowance is 600 gpd per acre. I&I was estimated to represent up to 85 percent of peak flows.

The hydraulic capacity evaluation included development of sewer flows for the existing, near-term, and buildout land use scenarios. Data from flow monitoring and historical DOM pumping records were used to calibrate the model. The calibrated hydraulic model was used to simulate the 5-year, 24-hour peak wet weather flow (PWWF) conditions in the existing, near-term (2027), and buildout (2063) systems. The District's existing design and performance criteria were used as a basis to identify deficiencies and recommended improvements as explained below:

- The DOM4 lift station is flow-limited to 3.9 mgd due to downstream capacity constraints in the collection system and at the District's WWTP. Expansion of DOM4 capacity is not viable in the near term due to the WWTP's physical and regulatory constraints, including the need for a new NPDES permit, major process upgrades, and limited available land.
- For these reasons, approaches to address capacity deficiencies focused primarily on reducing I&I to reduce unnecessary wet weather loading and preserve available capacity for future development. This I&I reduction strategy is also consistent with State Water Resources Control Board guidance, which discourages the conveyance of non-wastewater flows to treatment facilities.
- Although expansion of the WWTP is not feasible at this time, it may need to be reconsidered as part of long-term planning to address future capacity needs.

Sanitary Sewer System Recommendations

Based on the assessments and evaluations performed on the sanitary sewer system, an integrated set of recommendations for each land use scenario was developed. All new infrastructure should meet District design standards.

Existing/Near-term (by 2027) recommendations include:

- Eliminate severe inflow defects and cross connections to the sanitary sewer identified in smoke testing (14 severe defects and 4 cross connections). Addressing these defects and cross connections will provide the greatest reduction in I&I at the lowest cost.
- Replace or rehabilitate pipelines that were identified to have Grade 4 or 5 structural defects (11 pipelines; 1,463 feet). Rehabilitate maintenance holes in-line or adjacent to pipelines that are replaced or rehabilitated.
- Replace all brick pipelines (437 feet identified) and maintenance holes (18 identified).
- Rehabilitate maintenance holes with broken, cracked, or defective components (27 identified).

- Implement lift station Priority 1 improvement recommendations (DOM 2, 4, 6, 7, 8, L-Street).
- Complete condition assessment on all pipelines greater than 60 years old that were not inspected as part of this project – approximately 13.7 miles. Visually assess associated maintenance holes concurrently. Address severe defects as they are identified.
- Regarding capacity, is expected that the implementation of pipeline rehabilitation, defect elimination, and cross-connection removals will result in measurable I&I reduction, helping to alleviate capacity constraints in the system.
- According to anecdotal information provided by the District and Mare Island Company, no above-ground overflows have occurred on Mare Island in recent years. As development progresses, reductions in I&I from near-term improvements are anticipated to offset flow increases from new connections, thereby minimizing the risk of overflows.
- Once near-term rehabilitation and inflow elimination projects are complete, system capacity should be re-evaluated using updated flow monitoring data and development-based flow projections. This will establish a meaningful post-rehabilitation baseline and inform whether further improvements are necessary.

Buildout (by 2063) recommendations include:

- Replace or rehabilitate pipelines that were identified to have Grade 3 structural defects (14 pipelines, 3,134 feet) as part of this project (recommended within 5 years – by 2029 or earlier), and pipes that were inspected after this project was concluded.
- Implement a regular preventative maintenance program, including pipeline cleaning and condition assessment. Perform condition assessment of all pipes every 7 years (minimum) to identify structural and maintenance deficiencies.
- For continued lift station operation beyond 2027, implement lift station Priority 2 improvement recommendations (DOM 2, 3, 4, 6, 7, 8). Future upgrades or replacement may be needed by 2063 depending on performance, regulatory changes, or capacity needs. Preliminary evaluation indicates adequate space exists at all lift station sites for expansion.
- Regarding capacity, it is expected that continued pipeline rehabilitation and replacement throughout the buildout period will result in significant reductions in I&I, further relieving capacity constraints in the system and supporting long-term development.
- System capacity should be re-evaluated following completion of major rehabilitation efforts using updated flow monitoring and development-based flow projections. This re-evaluation will provide a critical baseline for determining whether additional capacity improvements – such as increasing the DOM4 flow limit – are warranted.
- If a formal capacity re-evaluation is not conducted, one of the alternative improvement approaches outlined in Section 5.7.10 should be implemented. These range from continued I&I reduction efforts to increasing the DOM4 flow limit.
- The following ongoing actions are recommended and will support future decision-making regarding capacity upgrades or targeted system rehabilitation:
  - Monitor surcharging at key maintenance holes to identify persistent or worsening capacity deficiencies. These observations can help prioritize additional targeted rehabilitation or system improvements.

- Evaluate downstream impacts if DOM4 capacity is increased. Upgrades to the wastewater WWTP may be necessary, including revised permitting, physical expansion of treatment processes, and compliance with applicable regulatory and effluent quality requirements.

GEOTECHNICAL EVALUATION (CHAPTER 6)

A study of available information was performed to evaluate the existing geotechnical conditions and considerations on Mare Island. Available information collected and reviewed included: existing geotechnical reports; subsurface data; historical aerial imagery; and published geotechnical documents from government agencies, including but not limited to historical topographic, soil and geologic, liquefaction susceptibility and landslide maps. Collected information is presented in Appendix 6A.

Based on available information and conclusions from previous geotechnical reports, the following summary statements can be made:

- The majority of previous subsurface investigations and studies are located in the area north of G Street and east of Azuar Drive.
- Consolidation settlement is one of the major issues for the area north of G Street and east of Azuar Drive, and west of Tisdale Avenue along San Pablo Bay due to a thick layer of Young Bay Mud (YBM).
- Based on estimates developed as part of this study, consolidation settlement due to compression of the YBM under placement of fill can range from 0.8 feet to 10.8 feet depending on the thickness of fill placement (5 to 10 feet) and YBM thickness (10 feet to 80 feet). The settlement is projected to take place over 10 months to 600 years depending on the thickness of YBM. The consolidation settlement timing can be reduced by implementing a surcharge program with wick drains. If a surcharge program alone is not feasible to mitigate the anticipated settlement, the existing fill can be removed and replaced with lightweight materials.
- Although shoreline areas along San Pablo Bay and the Mare Island Strait have moderate to very high liquefaction susceptibility, site-specific analysis by ENGEO (in letter to Lennar Mare Island, LLC in January 2016) concluded that the risk of liquefaction in planned development areas is generally low. However, lateral spreading remains a potential hazard in susceptible zones.

An overview of the existing seawalls and discussion of the options and limitations associated with improving existing seawalls to meet future sea level rise was presented and the following defines the seawall:

- The seawall was originally built by the Navy and has been rehabilitated or repaired several times since its original construction (the oldest repair as-built drawings date back to 1922).
- The existing seawalls, if retained, would need to be raised by between 3.5 to 5.5 feet to meet year 2070 SLR projections, and between 7 to 9 feet for Year 2100 projections.
- There are two scenarios for protection from sea level rise. One is to raise the ground level to provide a nominal level of safety at the expense of initiating long-term consolidation settlement. Second is to install a barrier seawall, which would need to be supported by a pumping system to pump stormwater and seepage flows from behind the land side of the barrier.

- Two conceptual approaches to retrofitting the wall are presented. It is acknowledged that additional condition, geotechnical, and hydrogeological investigations will need to be performed to determine design requirements of retrofit solution.

As a result of the initial findings of this study, an above-water reconnaissance-level field assessment of the Mare Island seawall (bulkhead) was conducted in April 2025 to document visible signs of deterioration, screen for structural concerns, and inform future inspection planning. The assessment included a visual, non-destructive inspection of the bulkhead wall and associated waterfront structures from both land and water, spanning Berths 1 through 24 and grouped into 11 structural zones (Zones A through K). Conditions are explained below:

- Condition ratings for the seawall elements, based on ASCE Manual of Practice 130, ranged from Satisfactory (2 to 3 zones), to Fair (4 zones), to Poor (3 zones), with observed defects including broken sheet piles, severe cracking, spalling, delamination, missing tie rods, and sinkholes behind the wall near Dry Dock 2.
- Two sinkholes were observed behind the bulkhead wall near Dry dock 2 and Building 340. A probable cause assessment and recommended repair approach are presented.

The Bulkhead Wall Reconnaissance Level Condition Assessment Report is included as Appendix 6B.

Geotechnical Evaluation Recommendations

Because many of the development areas lack site-specific data, future project designs should not rely solely on regional or historical data without confirming subsurface conditions through targeted investigations. The type of subsurface investigations and quantity of investigation will depend on the design requirements and may include:

- Borings to characterize ground and groundwater conditions and to collect soil and rock samples for laboratory testing.
- Laboratory testing to obtain design parameters for soil and rock underlying the project areas.
- Soil testing to characterize ground conditions (e.g., depth to groundwater, liquefiable soil, lateral spreading potential, average shear wave velocity).
- Seismic refraction survey to evaluate soil/rock interface profile on the southern side of the island for proposed improvements and new seawalls.
- In-situ permeability testing to assess implications of water seepage through/under the seawall resulting from an imbalance of water pressures between the water side and land side of the seawall to inform designs to intercept and convey seepage water that could otherwise cause elevated groundwater levels within the interior of the island.
- All infrastructure improvements should consider site-specific geotechnical data and the applicable building codes in effect at the time of design.

Regarding the seawall, two distinct sets of recommendations are provided: one to address the physical condition and structural integrity of the existing wall, and another to support long-term planning for SLR adaptation as defined below:

- Condition and Capacity of Existing Seawall Structure (building on the 2025 seawall assessment) to support evaluation of retrofit feasibility and future design. In the short term (0 to 2 years), perform the following inspections in order of priority:
  - Inspection to ascertain the root cause of the sinkholes near DD2 within Zone G. Repair sinkholes as appropriate (repair approaches provided in assessment report).
  - Level I above and below-water inspection of the steel sheet piles within Zones D, H & I.
  - Detailed inspection of the open timber wharf within Zone A (Berth 1).
  - Detailed inspection of the concrete sheet pile bulkhead within Zone F.

In the medium to longer term, the scope of future structural assessments should first be based on the planned development and land use. Development plans will inform on reaches of the waterfront that should be assessed and for which a load rating should be performed. Detail on other inspection methods is provided in the 2025 assessment report.

- Sea Level Rise Adaptation Planning Recommendations support long-term shoreline protection, adapt to projected SLR, and evaluate retrofit feasibility:
  - Conduct hydrological, geotechnical, and hydrogeological investigations to assess subsurface conditions, seismic risks, slope stability, settlement potential, and seepage.
  - Assess future groundwater rise and its interaction with shoreline infrastructure.
  - Develop a long-term shoreline strategy including a future facilities plan for wharfage integrated with coordinated land use planning.
  - Consider state-of-the-art SLR adaptation strategies, including collaboration with property owners, developers, and specialist marine engineers, as well as alignment with national and global best practices for resilient waterfront redevelopment.

ROADS SYSTEM (CHAPTER 7)

The Mare Island roads system consists of approximately 28.3 centerline-miles of streets (86 street segments), with surfaces ranging from asphalt concrete (AC) to unpaved gravel (mostly in the south end of the island). Road conditions are measured using the Pavement Condition Index (PCI), which scores streets on a scale from 0 (failed) to 100 (newly paved). At the start of this project, the City's StreetSaver database covered 21.1 centerline-miles with an average PCI of 76. During the project, an additional 5.4 centerline-miles of streets were added to the StreetSaver database in the south end of the island, which have an average PCI of 49 (not including gravel roads). For comparison, the overall PCI for the City of Vallejo's street network, updated in 2022, is 48.

The roads system evaluations were performed to understand the current condition and develop a proposed treatment plan. Evaluations included: vehicle and walking pavement condition survey to verify the StreetSaver database and pavement condition; automated survey vehicle capture of imagery of the entire street network; field testing on critical streets and road materials to evaluate various parameters (strength, thickness, layer composition, etc.); and developing appropriate pavement preservation,

rehabilitation, or reconstruction treatments for arterial and major collector streets based on field testing, analysis, and applicable standards.

The field testing included falling weight deflectometer (FWD) testing, ground penetrating radar (GPR) testing, dynamic cone penetrometer (DCP) testing, coring/boring, and laboratory testing of collected subgrade soil samples. Soil sampling defined:

- FWD testing was performed on 25.3 lane-miles of streets. The average FWD deflection was 16.0 mils (1 mil = 1/1000 of an inch), indicating medium structural capacity.
- GPR testing was conducted on 40.0 lane-miles of streets. The streets average AC layer thickness was estimated 5.4 ± 2.5 inches.
- A total of 60 cores and 3 bores were sampled and 43 DCP tests were conducted to estimate the existing pavement structural layer thicknesses and other properties. Coring data showed that AC layer thicknesses ranged between approximately 1.3 and 16.1 inches, with an average thickness of 6.9 ± 3.2 inches. The AC layers were thicker on major streets (average 7.3 ± 3.0 inches) than on residential streets (average 5.7 ± 3.5 inches). At several locations Portland cement concrete (PCC) or large aggregate size macadam bituminous base layers were identified under the AC layer.

The California Department of Transportation (Caltrans) Highway Design Manual (2022), the City roadway design standard, and engineering judgement was used to evaluate the pavement structure and identify appropriate pavement preservation, rehabilitation, or reconstruction treatments.

Roads System Recommendations

The following treatments were recommended for the selected design sections based on existing pavement condition and estimated traffic volume:

- Do Nothing (12 design sections): Sections recommended for no treatment are recently paved or in very good condition; preservation may be required in 5+ years
- Slurry Seal (17 design sections plus half of one design section)
- Rubberized Cape Seal (7 design sections plus half of one design section)
- Cold-in Place Recycling (CIR) with hot mix asphalt (HMA) overlay (7 design sections)
- Mill-and-overlay HMA with base repairs (12 design sections plus half of one design section)
- Remove existing asphalt concrete/aggregate base (AC/AB) and replace with HMA overlay (four design sections plus half of one design section)

The proposed treatments are based on the existing section conditions (late 2023). It is recommended the treatments are completed according to the following guidelines:

- Complete the surface seal treatment repairs in the next 2 to 3 years
- Complete all other rehabilitation treatment repairs in the next 3 to 5 years
- Some roads, such as those in the Connolly Street Corridor, should be rehabilitated in coordination with redevelopment and utility improvements

The recommended timing is to optimize treatment costs and slow the pavement deterioration to avoid further higher cost repairs. That said, a multi-year pavement treatment plan always is subject to change should it be related to planned utility work that needs to be completed ahead of paving, unexpected scheduling delays, funding shortfalls, and planned development/construction. The final treatment and construction schedule needs to be re-evaluated and verified by the City based on existing pavement condition, construction budget, nearby development plan and anticipated uses.

DRY UTILITY SYSTEMS (CHAPTER 8)

Existing dry utility providers on Mare Island include:

- Island Energy (electric and natural gas)
- Comcast/Xfinity (cable communications)
- AT&T (telephone and cable communications)
- VIPFiber (City-owned broadband infrastructure)

The dry utility system evaluations included identifying and contacting the existing dry utility providers on the island; obtaining and compiling existing mapping, plans, schematics, and reports which detail the extents of the dry utility systems; and discussing with each of the dry utility providers their current coverage, planned expansions, and concerns for future development.

Technical evaluations, mapping/field verification, or engineering of dry utility system capacity or condition was not performed. A log of information received during this investigation is provided in Appendix 8A through 8D, organized by utility company. No field verification of the information received was performed. Facility sizes and locations presented in this report should not be relied upon without verification.

Dry Utility Systems Conclusions

The island's dry utilities face similar issues to the wet utilities, but on a smaller scale. Many dry utilities have been installed or reconstructed after the Navy left, so the systems are generally newer than the water and sewer systems.

Dry utility infrastructure south of G Street is both functional and easily expandable to meet the needs of new development. Dry utility systems on North Mare Island are either non-existent or rudimentary and will need to be completely designed and constructed to serve development on the north island.

In general, the dry utilities will have to accommodate sea level rise, although several of the dry utilities are not adversely impacted by wet or submerged conditions. Although electric and natural gas infrastructure may be vulnerable to water intrusion, communication systems (e.g., VIPFiber, AT&T) are generally resilient to submerged conditions, with vault waterproofing required for maintenance access rather than operational integrity. Additionally, since the dry utilities typically utilize smaller transmission systems, relocating these are less intrusive and disruptive than water or sewer upgrades or relocations.

Similar to the wet utilities, the dry utility companies are aware that any subsurface work on the island must be prepared to deal with the unexpected, such as abandoned-in-place buried infrastructure or contaminated soil layers encountered during excavation.

The City and Mare Island Company should continue to communicate with dry utility companies to provide updates on future development and dry utility needs.

INTEGRATED RECOMMENDATIONS (CHAPTER 9)

Chapter 9 presents general considerations and an integrated set of infrastructure improvements covering the water, wastewater, storm drainage, roadway, geotechnical, and dry utility systems across Mare Island.

General considerations emphasize coordinated corridor planning, development flexibility, and climate resilience, with a focus on construction, adapting to evolving land use and regulatory conditions, and proactively addressing sea-level-rise and other natural hazard risks.

An interactive WebMap (https://arcg.is/0jyaei3) displays recommended improvements by utility system and land-use scenario, allowing users to explore recommendations in a geospatial context. Table 9-1 summarizes both system-specific and systemwide improvements.

The findings and recommendations in this report form the foundation for development of a coordinated, resilient, and cost-effective strategy to support the long-term redevelopment of Mare Island.

CHAPTER 1
Introduction

This Mare Island Infrastructure Assessment (MIIA) Report was completed for the City of Vallejo (City) and the Vallejo Flood and Wastewater District (District) to support infrastructure planning and redevelopment efforts on Mare Island (Island). The MIIA provides a comprehensive evaluation of the existing condition and capacity of the Island's key infrastructure systems, including water, sanitary sewer, storm drainage and flood control, and roads. These systems were assessed based on serving two land use scenarios, an existing/near-term scenario (through year 2027) and a build-out scenario (though year 2063) based on anticipated future land use.

Recommendations for infrastructure improvements were developed to support the needs of each land use scenario. In addition to these key infrastructure systems, the MIIA includes a summary of existing dry utility systems (electric, gas, and telecom) and a geotechnical evaluation of Mare Island's underlying soil and seismic conditions to inform future development strategies.

The MIIA project was initiated in January 2023 and concluded in October 2025. It represents a coordinated effort between City and District staff, Mare Island Company, and a multidisciplinary Consultant Team, and is intended to guide infrastructure investment and implementation over the coming decades.

1.1 BACKGROUND

Mare Island has undergone a significant transformation, evolving from the first United States naval base on the West Coast into a growing community and economic hub for the City of Vallejo. From 1854 to 1996, the Island was home to the Mare Island Naval Shipyard. Following its decommissioning in 1996, the City and District assumed responsibility over time for the Island's core infrastructure systems, including water, sanitary sewer, storm drainage and flood control, and roads.

In 2004, Mare Island was opened for public access and a decades-long process of redevelopment was initiated. Today, the City partners with The Nimitz Group, LLC dba Mare Island Company to guide the planning, redevelopment, and management of Mare Island.

Mare Island is part of the City of Vallejo and is located at the confluence of the Napa River (Mare Island Strait) and San Pablo Bay. Its regional location is presented in Figure 1-1.

1.2 PURPOSE

The purpose of the MIIA is to provide the City of Vallejo and Vallejo Flood and Wastewater District with a comprehensive understanding of the infrastructure improvements required to support the successful redevelopment and long-term growth of Mare Island. Reliable water, sanitary sewer, storm drainage and flood control, roads, and dry utility systems are essential for enabling new development, protecting public health and safety, and maintaining operational resilience. The MIIA identifies current system limitations, evaluates future service needs, and develops planning-level recommendations to ensure that infrastructure is not a constraint to achieving the Island's full development potential.

The assessment evaluates each system based on two land use scenarios:

- Existing/Near-Term Scenario (through year 2027) – Represents known development and infrastructure needs based on current land use and projects in progress.
- Build-Out Scenario (through year 2063) – Represents full redevelopment of the Island based on long-range planning assumptions and growth forecasts.

For each scenario, recommendations are provided to address both existing deficiencies and future capacity needs. By integrating condition assessments, capacity analysis, and land use projections, the report equips the City, District, and their partners to plan proactively, align capital investments, and support coordinated, long-term redevelopment.

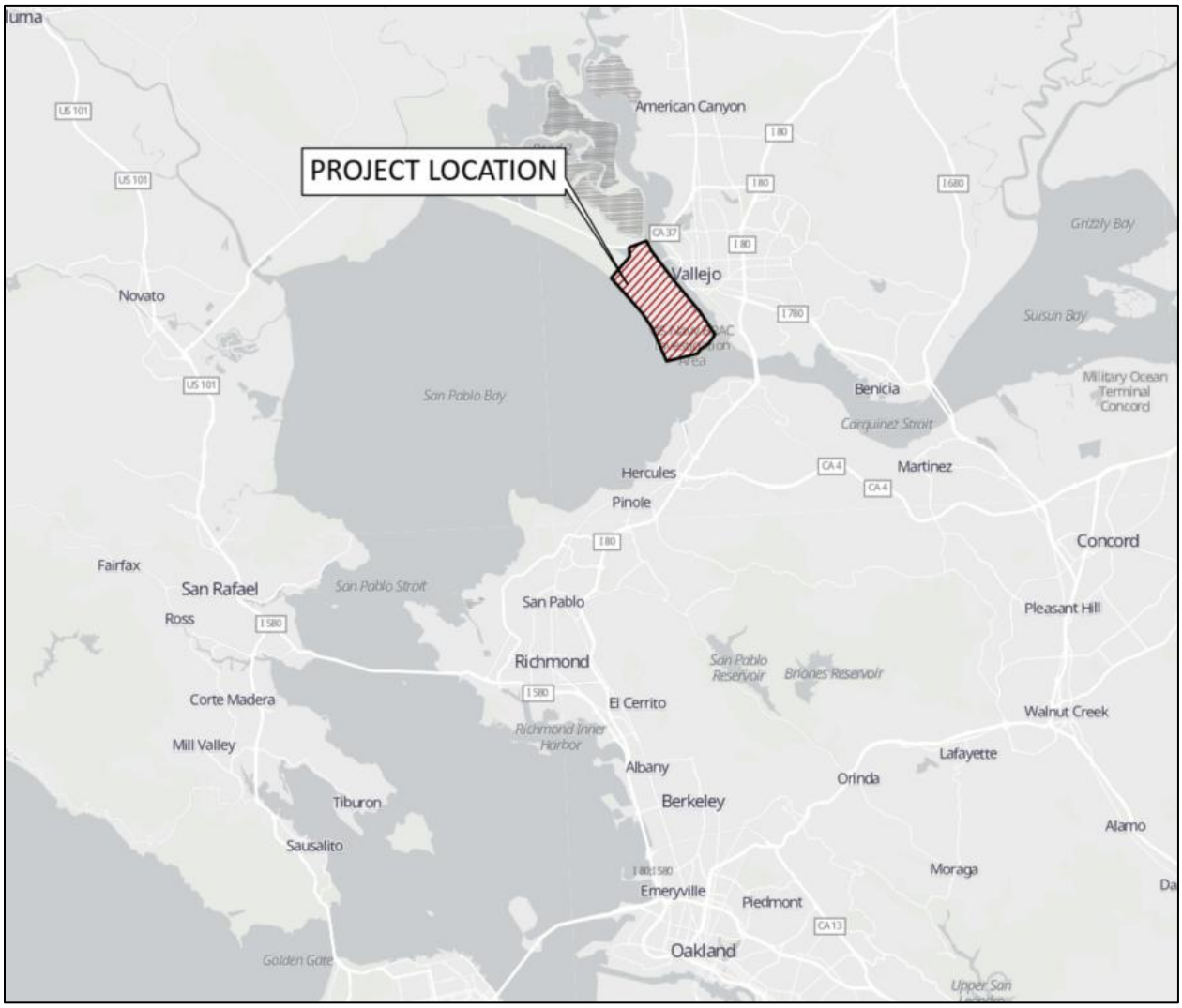


Figure 1-1. Mare Island Regional Location

1.3 EXISTING INFORMATION SOURCES

Over the course of Mare Island's redevelopment, a wide range of technical studies have been completed to evaluate whether existing infrastructure can support proposed land uses. Since the adoption of the *Mare Island Reuse Plan* in 1994, the City and its partners have initiated numerous site-specific and system-wide evaluations related to infrastructure, environmental conditions, and geotechnical constraints.

One of the goals of the MIIA project was to incorporate and build upon this prior work to avoid duplication of effort and ensure consistency with existing data. At the outset of the MIIA effort, a structured data collection process was conducted to obtain and review available documentation. In addition to studies, data such as as-built drawings, geographic information system (GIS) mapping, topographic mapping, and other information was obtained. Each chapter of this report identifies the specific data and references used to support its findings and conclusions.

1.4 PROJECT TEAM

The MIIA was developed through a collaborative effort between the Client Team – comprised of the City of Vallejo, Vallejo Flood and Wastewater District, and Mare Island Company – and the Consultant Team, led by West Yost:

MIIA Client Team:

City	District	Mare Island Company
Gillian Haen	William Ash	Dustin Rieger
Ivette Iraheta	Mark Tomko	Lee Rosenblatt
Michael Nimon	Tracy Rideout	Megan Alferness
Erin Hanford		
Dakari Barksdale		
Melissa Cansdale		
Beth Schoenberger		
Dan Sequeira		
Oscar Alcantar		
Fernando G. Bravo		

MIIA Consultant Team – The Consultant Team was led by **West Yost Associates**, with support from the following subconsultants:

- **Delve Underground** – geotechnical and seismic evaluations
- **V&A Consulting Engineers** – flow monitoring, smoke testing, and condition assessment support for the water tank and pipelines
- **Moffatt & Nichol** – sea wall bulkhead assessment
- **Mercurio Engineering** – dry utility coordination, data collection, historical infrastructure insight
- **Carlson, Barbee & Gibson** – surveying

1.5 ORGANZATION OF THE REPORT

The remaining chapters of this report include:

- Chapter 2. Land Use
- Chapter 3. Water System
- Chapter 4. Storm Drainage and Flood Control System
- Chapter 5. Sanitary Sewer System
- Chapter 6. Geotechnical Evaluation, including the Sea Wall
- Chapter 7. Roads System
- Chapter 8. Dry Utility Systems
- Chapter 9. Integrated Recommendations

Each chapter follows a consistent structure, beginning with a Summary and Conclusions section that highlights the key tasks, findings, and outcomes of the analysis. Chapters addressing physical infrastructure systems (Chapters 3 through 8) conclude with planning-level recommendations for improvements to serve both the existing/near-term and build-out land use scenarios. Where applicable, technical appendices are included to provide supporting detail, such as hydraulic modeling results, field investigation reports, and condition assessment data.

Chapter 9 consolidates recommendations across all systems and provides an integrated planning perspective, including:

- General Considerations – A summary of system-wide issues that influence infrastructure planning and delivery, including utility corridor coordination, climate resilience, regulatory constraints, flexible implementation, interagency collaboration, and funding partnerships.
- Integrated Recommendations – Presented in Table 9-1 and Figure 9-1 (a web-based map), this section outlines system-specific improvements to support cross-utility coordination and alignment with development phasing.

CHAPTER 2
Land Use

2.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

The tasks completed for the land use evaluation included:

- Developing land use estimates for an existing/near-term and build-out development scenario.
- Preparing GIS maps presenting the land use for the two development scenarios.
- Consulting with large utility users on the island to obtain information on current and expected growth within the two land use scenarios.

A summary of the findings and conclusions for the land use evaluation are listed below.

- An Existing/Near-Term Scenario was established which represents development on the island through 2027. The primary development in the Existing/Near-Term scenario is the Connolly Street Corridor Development Project.
- A Build-Out Scenario was established which represents expected conditions when the island is fully developed, estimated through 2063 (40 years).
- Several large utility users including Mare Island Dry Docks and Lind Marine expect minimal to moderate growth in the Existing/Near-Term Scenario timeline. Touro University and Mare Island Dry Docks expect moderate growth in the in the Build-Out Scenario timeline. Expected growth is quantified in terms of number of dry docks in operation or percent growth in students.
- Each development scenario is summarized in tabular and figure form.
- The information in this chapter will be used to estimate utility demands for establishing future infrastructure sizes.

2.2 PURPOSE

This chapter summarizes the future growth for Mare Island in the context of two land use scenarios. These land use scenarios are the basis for the infrastructure assessments and define the number of dwelling units and expected land use so that assumptions for estimating water demands, sewer and storm drain flows, and roads usage projections can be made. The evaluation process and assumptions were unique to each scenario based on available information.

The two scenarios include:

- **Existing/Near-Term Scenario** which represents existing conditions, plus any near-term development (through 2027).
- **Build-Out Scenario** which represents expected conditions when the island is fully developed (through 2063).

The following sections describe the expected growth on Mare Island, and then the process that was used to define the two land use scenarios.

2.3 EXPECTED GROWTH

The following sections describe the expected growth on Mare Island from known developments and existing large utility users. The large utility users were identified in consultation with the Client Team as having a potentially significant impact on utility demands.

2.3.1 Known Developments

The only known development is the Connolly Street Corridor Development Project, which will be redeveloped within the existing/near-term scenario.

2.3.1.1 Connolly Street Corridor Development Project

The Connolly Street Corridor is a portion of the historic core area of Mare Island. Redevelopment plans include approximately 29,000 square feet of office space, 373 multi-family residential units, a 200-room hotel, approximately 45,000 square feet of ancillary uses such as an event space and food and beverage kiosks, and approximately 194,000 square feet of open space[1]. The Connolly Street Corridor Project will be submitted to the City for approval in stages and at the drafting of this report has not been formally submitted or approved. The current Connolly Street Corridor Project boundary is shown on Figure 2-1.

2.3.2 Existing Large Utility Users

This section describes several of the large utility users and their planned growth within the Existing/Near-Term Scenario and the Build-Out Scenario.

2.3.2.1 Touro University

Touro University is a private health professions graduate school located at 1310 Club Drive. The following information was provided by the Provost of Touro University:

- Currently there are 260 full time or part time employees that spend some time on campus.
- Currently there are about 1,250 students (2022), but prior to COVID, there were about 1,500 students. About half of the students are doing clinical rotations at off-site locations.
- Over the next 3 years, student populations should return to 1,500 students, but about half of those students will be off campus for clinical rotations (expect about 500 to 750 students on campus).
- By 2030, there should be about 1,800 to 2,000 students, and about half or more will not be campus-based (on clinical rotations).
- Currently there is no on-campus student housing.

[1] North Mare Island Disposition and Development Agreement (DDA) and Implementation Project; CEQA Addendum to the Mare Island Specific Plan EIS/EIR (1999) and the Mare Island Amended and Restated Specific Plan Project Final Subsequent EIR (2005), Stantec, February 2022

- Future student housing is in early planning stages, but there may be some student housing in 2 to 3 years (2025 to 2026); the number of student dwelling units is not known at this time.
- There are no other campus developments planned at this time that would affect utility planning.

Based on the information provided from Touro University, and uncertain development status, it is assumed student population will return to pre-COVID levels, but there will be no significant additional growth in the existing/near-term. Significant growth in the Build-Out Scenario is expected. Specific utility demands are discussed in the respective utility chapters.

2.3.2.2 Mare Island Dry Docks

Mare Island Dry Docks (MIDD) operates Dry Docks (DD) 2 and 3 and are trying to secure operation of DD 1. The following information was provided by the MIDD Chief Engineer in February 2023:

2.3.2.2.1 Development Scenarios

- MIDD currently operate DD 2 and 3 and would like to secure operation of DD 1 within 3 to 5 years.
  - If DD 1 is not secured, work force and utility demands could increase by about 20 percent by 2027 through increased shift work.
  - If DD 1 is secured, work force and utility demands would increase by about 40 to 50 percent by 2027.
- The long-term growth (5 to 20 years) would not increase significantly over the near-term growth, but could be an additional 5 to 15 percent (with or without DD 1).
- Current work force is 61 people, which can increase to about 1,200 people with contract awards. With DD 1, this could increase to 1,500 to 1,700 people.
  - Current ship crew housing is about 400 to 500 people. With DD 1, this could increase by 40 to 50 percent.

2.3.2.2.2 Water Uses

- MIDD is probably the largest water user on Mare Island.
- Near-term water demands could increase by 20 percent (without DD 1) or up to 40 to 50 percent (with DD 1).
- Build-out water demands will be about the same as near-term demands.
- At one time, when water main repairs were occurring, the MIDD lost its water supply. Increasing the water system looping and valves could prevent this in the future.

2.3.2.2.3 Sewer Flows

- The industrial/operations wastewater is pumped to Baker Tanks. When the water quality is allowable, the tanks are drained to the sewer (about 20,000 gallons in 1.5 hours per tank). Up to six tanks are drained per day. Up to three tanks could be drained at one time into the three sewer receptacles.
- Near-term sewer flows could increase by 40 to 50 percent (with DD 1).
- Build-out would be the same as near-term.

2.3.2.2.4 Storm Drainage and Flood Control

- The storm drains work fine. The pipes are aging and may exceed useful life soon.
- Near-term and long-term storm drainage will not change significantly.
- The drain at the north side of the DD 2 was previously removed, and now the electrical vaults do not drain well.
- Sea level rise is an issue. Currently, the Mare Island Straight water level gets within 1 foot of the dry dock top during high tides with storm events. Sea level has risen by about 15 inches since the docks were constructed. Continued sea level rise will impact the ability to operate the MIDD in the future and ultimately may make the MIDD infeasible to operate unless improvements to the dry docks are implemented.

2.3.2.2.5 Roads

- Traffic demand increases should be proportioned similar to other utilities, especially for shift start and stop.

2.3.2.2.6 Dry Utilities

- Electric power and gas demand increases should be proportioned similar to other utilities.

Based on the information provided by MIDD, it is assumed that DD 1 will be operational within the existing/near-term scenario. The Build-Out Scenario is assumed to be a 15 percent increase in growth from that of the existing/near-term scenario. Specific utility demands are discussed in the respective utility chapters.

2.3.2.3 Lind Marine

Lind Marine operates DD 4. The following information was provided by the Vice President - Environment, Social & Governance Programs of Lind Marine in February 2023. The provided data are estimates based on the current business model and could change in the future. The growth presented below is expected in 3 to 5 years (2025 to 2027).

- Water demand comes from vessel repair/maintenance and employee sanitation needs (not major demands).
- Major wastewater flows to the sewer system include dry dock caisson leakage and dry dock stormwater, discharge of excess water from sand operation, and employee sanitation in the office/shops. The estimated water demand / wastewater flow should increase about 10 to 15 percent over the next 3 to 5 years.
- Currently there are about 72 employees based at Mare Island, including 32 in the office and shop at 1175 Nimitz and 40 in the shipyard areas at 1250 Nimitz.
- Anticipated future growth over the next 3 to 5 years is on the order of 20 to 30-percent, or 15 to 20 additional employees.
- None of the growth plans would affect current stormwater drainage infrastructure.
- The sizes of vehicles and trucks accessing Lind Marine are not expected to change in the future; however, the number of trips could be expected to increase by 10 to 20 percent.
- Lind Marine does not use natural gas.

- Electrical demands are anticipated to rise to accommodate powering the vessels that are docked and future drydock operation improvements. Electrical demands are estimated to increase by 20 to 30 percent.
- Communications (internet/data) infrastructure is a problem on the island. Frequent outages occur, and this will need improvement for any future growth anticipated on the island.

Based on the information provided from Lind Marine, it is assumed that utility demands will increase 15 percent in the Existing/Near-Term scenario. It is assumed there will be no additional growth in the Build-Out Scenario. Specific utility demands are discussed in the respective utility chapters.

2.3.2.4 Other Significant Users

There are other significant utility users on the island including Alco Iron & Metal Co. (Alco), Earthquake Protection System (EPS), Factory OS, etc. These users were not consulted directly as part of this study; utility projections for the Existing/Near-Term Scenario will be based on existing utility data (i.e., water meter data, sanitary sewer flow meter data) and for the Build-Out Scenario will be based on a preliminary draft plan of the Mare Island build-out land use [see Section 2.5]).

2.4 EXISTING / NEAR-TERM SCENARIO

The existing/near-term scenario was developed using existing GIS mapping of buildings, known development plans (specifically Connolly Street Corridor), and input from the MIIA Client Team. The following sections describe the agreed-upon assumptions used in development of the scenario, provide an overview of the scenario development, and present a figure and table summary of the scenario.

2.4.1 Assumptions

The assumptions used in the Existing/Near-term scenario were discussed during project meetings and agreed upon with the Client Team. Assumptions included the following:

- The GIS served as the database of record for the scenario.
- The Connolly Corridor Project will be the only new residential development.
- Residential buildings will be assigned either a Single-Family Residential (SFR) or a High-Density Residential (HDR) land use type.
- All non-residential buildings will be assigned a general Commercial/Industrial (C/I) land use type.

2.4.2 Scenario Development Steps

Development of the Existing/Near-Term Scenario included the following steps:

1. The City provided building status GIS information (February 2023) within a layer named STRUCTURES, which provided information on building spatial location, dimensions/square footage, demolition status (demolished, to be demolished, not demolished), and other building attributes.

2. The Client Team reviewed building demolition status information (via a map of the STRUCTURES layer with building number and an associated export of the GIS attributes in an Excel spreadsheet) and provided feedback on discrepancies between GIS information and known status. The GIS STRUCTURES layer was updated based on the Client Team input.
3. MIC provided a draft version of their 'Overall Island Base Map' (June 2020) which provided information on existing buildings and likely demolition status. This map was also compared with the City's GIS. Several buildings shown as existing in the City's GIS were not shown on the MIC Base Map. The City's GIS information was updated to align with the Overall Island Base Map for buildings that were shown as demolished or to not exist on MIC's Base Map.
4. West Yost used the City's and MIC's GIS mapping, aerial imagery, and input from the Client Team to identify residential buildings and assigned each SFR or HDR land use type. Existing land use is summarized in Table 2-1.
5. West Yost added the Connolly Corridor Project (buildings, land use types) to the GIS map.

Table 2-1. Summary of Existing Land Use

Land Use Type/Area	Count of Occupied Buildings or Lots	Other Users
Single-Family Residential	327 Lots[(a)]	-
High-Density Residential	5 Buildings[(b)]	-
Commercial/Industrial	503 Buildings[(c)]	-
Mare Island Dry Docks	-	Operation of DD 2 and DD 3
Lind Marine	-	Operation of DD 4

(a) Includes 10 Quarter Apartment Buildings along Azuar Avenue, assumed 1 building is equivalent to 1 lot.
(b) Includes buildings no. M37, 84, 84A, 1, 411.
(c) Sum of the building area (square foot) data from GIS STRUCTURES layer of the 503 occupied buildings.

2.4.3 Summary of Scenario

The resulting Existing/Near-Term Scenario is presented on Figure 2-1, showing buildings and land use type.

The Existing/Near-Term Scenario is summarized in Table 2-2. The large utility owners that expect growth in the Existing/Near-Term Scenario are listed individually and will be considered separately during development of utility demands.

Table 2-2. Summary of Existing/Near-Term Land Use Scenario

Land Use Type/Area	Count of Occupied Buildings or Lots	Square Feet (SF) of Occupied Buildings	Other Users/Development
Single-Family Residential	327 Lots[a]	-	-
High-Density Residential	5 Buildings[b]	-	-
Commercial/Industrial	503 Buildings	4,615,411 SF[c]	-
Connolly Street Corridor[d]	373 HDR Units 200-room hotel	29,000 SF Office 46,000 SF Ancillary Use	194,000 SF of Open Space
Mare Island Dry Docks	-	-	Operation of DD 1, 2, and 3
Lind Marine	-	-	Operation of DD 4 plus 15 percent growth

(a) Includes 10 Quarter Apartment Buildings along Azuar Avenue, assumed 1 building is equivalent to 1 lot.
(b) Includes buildings no. M37, 84, 84A, 1, 411.
(c) Sum of the building area (square foot) data from GIS STRUCTURES layer of the 503 occupied buildings.
(d) Based on North Mare Island DDA (February 2022).

Mare Island Strait

0 1,500 3,000 Feet

WEST YOST - N:\Client\174 City of Vallejo\60-22-06 MIA Project\GIS\MXD\Task 2 Land Uses\Fig_2-1_EX_NT_LandUse.mxd - aoerding - 7/2/2025

Prepared by:

WEST YOST

Legend

City Boundary

Structure Status

XX Existing Building

Land Use Type

- Single Family Residential
- High Density Residential
- Connolly Street Corridor
- Commercial or Mixed-Use
- Vacant - Undevelopable (Common Area, Cemetary, Taxable Below Min.)
- Vacant - Developable
- Marsh, Range, Watershed
- Governmental and Miscellaneous

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

City of VALLEJO California

Existing/Near-Term Land Use Scenario

Figure 2-1

2.5 BUILD-OUT SCENARIO

The Build-Out Scenario was developed using a preliminary draft plan of the Mare Island build-out land use (Mare Island Company, February 2023). The following sections describe the agreed-upon assumptions used in development of the scenario, provide an overview of the scenario development, and present a figure and table summary of the scenario.

2.5.1 Assumptions

The assumptions used in the Build-Out Scenario included the following:

- Full implementation of the build-out land use draft plan.
- Build-out is expected to occur within 40 years, year 2063.
- For all areas outside of the Mare Island Company build-out land use mapping (such as Western Magazine Area), buildings and land uses are to remain the same as the Existing/Near-Term Scenario.
- The scenario and development projections are based on available information at the time of this study. The final development plans will be determined upon City approval of a new Specific Plan.

2.5.2 Scenario Development Steps

Development of the Build-Out Scenario included:

- Mare Island Company provided a preliminary draft build-out land use plan, dated February 2023. This plan identified build-out land uses for all areas of the island, excluding Navy and State lands.

2.5.3 Summary of Scenario

The Build-Out Scenario is presented on Figure 2-2, showing land use type.

The Build-Out Scenario is summarized in Table 2-3. The large utility owners that expect growth in the Build-Out Scenario are listed individually and will be considered separately during development of utility demands.

Table 2-3. Summary of Build-Out Land Use Scenario

Land Use Type/Area	Dwelling Units	SF	Other Development[(a)]
Residential Uses	9,271	-	-
Commercial Uses[(b)]	-	8,130,380	-
Touro University	-	-	60 percent growth in students from existing/near-term
Mare Island Dry Docks	-	-	15 percent growth from existing/near-term

(a) Compared to Existing/Near-Term Scenario
(b) Includes 18,000 jobs

DRAFT FOR DISCUSSION PURPOSES ONLY.
FINAL PLAN WILL BE DETERMINED UPON CITY APPROVAL OF NEW SPECIFIC PLAN.

NOT TO SCALE

Legend

NEW DEVELOPMENT

- SINGLE FAMILY 0-9 DU/ACRE
- TOWNHOMES 10-29 DU/ACRE
- GARDEN STYLE 30-49 DU/ACRE
- COURTYARD STYLE 50-69 DU/ACRE
- MIXED-USE 70-89 DU/ACRE
- MIXED-USE TOWER 90+ DU/ACRE
- RECREATION
- OFFICE FAR: 0.70
- FLEX FAR: 0.40
- INDUSTRIAL FAR: 0.40
- HOSPITALITY / CULTURAL FAR: 0.60
- CIVIC FAR: 0.30
- RETAIL FAR: 0.50
- OPEN SPACE

EXISTING BUILDINGS/REUSE

- RETAIL
- OFFICE
- FLEX / R&D
- INDUSTRIAL
- CIVIC
- HOSPITALITY / CULTURAL
- RESIDENTIAL
- PARKING
- EXISTING TO REMAIN

Prepared by:
WEST YOST

Prepared for:
City of Vallejo
Mare Island
Infrastructure Assessment

Build-Out
Land Use Scenario

Figure 2-2

CHAPTER 3
Water System

3.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

The tasks completed for the water system evaluation included:

- Collecting and reviewing existing documents and mapping.
- Developing proposed design criteria and preferences to be used in development of infrastructure improvements.
- Developing a hydraulic model to evaluate the capacity needs of the proposed water system infrastructure improvements.
- Performing condition assessment of select water system pipelines and performing a seismic and condition assessment of the water tank.
- Developing an integrated set of recommendations for the existing/near-term (through 2027) and build-out (through 2063) development scenarios.

A summary of the findings and conclusions for the water system evaluations are listed below:

- The existing water system consists of approximately 35.4 miles of transmission and distribution pipelines and one water tank. Water service is provided from the mainland to Mare Island through three transmission pipelines crossing Mare Island Strait.
- Existing data was received including mapping, as-builts, and existing reports. Existing data sources were used to develop a comprehensive geospatial map of the existing water system, including asset attributes (material, installation date, diameter, etc.). Data was validated with City staff.
- Existing system pipelines were summarized by attribute, including diameter, material, and age.
  - Approximately 60 percent of the system by length is greater than 100 years old.
  - Approximately 25 percent of the system by length is less than 25 years old.
  - For pipes less than 8 inches diameter, approximately 28 percent of pipes are greater than 100 years old.
  - For pipes 8 inches diameter and larger, approximately 72 percent of pipes are greater than 100 years old.
- An age-based condition assessment was performed on water system pipelines, valves, hydrants, and meters. The assessment utilized an estimated useful life specific to each asset type and each asset's installation date to calculate an estimated remaining useful life.
  - Based on this assessment, it is expected that approximately 73 percent of the pipelines by length are currently in poor condition (pipes with less than 19 percent of estimated remaining useful life).
- Physical condition assessment was performed in the field for eight pipelines, including visual assessment, broadband electromagnetic (BEM) testing, and ultrasonic thickness (UT) testing. The assessment results were used to calculate estimated wall thickness loss.
  - Three of the sites had calculated thickness loss that is considered negligible. At the other five sites, the calculated wall thickness loss ranged from 13 percent to 26 percent, which is considered moderate. One site has a calculated thickness loss of 22 percent, which

should be considered significant because of the apparent very low-pressure rating (86 psi) of the originally installed pipe.

- Due to the lack of record information (resulting in unknown pipe age and material) in addition to varying soil and groundwater conditions, it is likely that the conditions of pipe will differ in other locations of the system.

- Age-based replacement projections for the existing/near-term and build-out scenario were developed for pipelines, valves, and hydrants. These projections were not justified to be modified based on field condition assessment information (i.e., the pipes field tested for condition assessment were already identified for age-based replacement).
- A water tank condition assessment was performed including visual inspection of the tank exterior and tank interior (using divers). Also, a seismic assessment was performed. Based on the assessments, the tank is in overall good condition, with the following exceptions:
  - Isolated areas of moderate surface damage (spalling and staining) on tank walls.
  - Interior piping has minor to moderate corrosion on fittings.
  - Tank improvement recommendations for the existing/near-term scenario include:
    - Investigation of exterior wall stains, and repair as required for delamination or corrosion.
    - Replacement of inlet pipe flap valve flange and overflow pipe flange.
    - Replacement of all interior piping fittings with stainless steel fittings.
    - Recoat/re-touch up interior piping coating.
  - The seismic assessment concluded that the maximum allowable water level in the tank should be limited to seven (7) feet below the roof soffit to maintain structural integrity during and after a seismic event. Due to current operating conditions, the reduction in available tank capacity associated with a lower water level will not have any immediate impact of system capacity or performance.
- A hydraulic capacity and performance evaluation was performed which included development of water demands for the existing (baseline), near-term, and build-out land use scenarios and evaluation of pressure and fire flow availability within each of those scenarios. The City's existing performance and operational criteria were used to identify deficiencies and recommended improvements.
  - The existing condition and near-term scenarios showed pressures below City criteria around the Mare Island Tank. These pressure issues are known to the City.
  - The near-term scenario identified fire flow deficiencies and associated pipeline improvements around the Connolly Street Corridor development.
  - The build-out scenario identified pressures below City criteria around the Mare Island Tank and also along the south end of the Island near Ribeiro Road.
  - The build-out scenario identified fire flow deficiencies across the Island and associated pipeline improvements. Depending on final build-out plans, a new water tank may be required. Pressures across the Island improve with implementation of the improvements.
- A hydraulic evaluation was performed on the three transmission pipelines crossing Mare Island Strait for the near-term development scenario. Assuming the two old crossings are

not in service, operation with only the one, new crossing does not affect the system's ability to meet demands under normal operating conditions. Under fire flow conditions, available fire flow is reduced and some hydrants fall below minimum requirements. Replacement of the south crossing would provide a long-term, reliable, redundant water connection for Mare Island (Island).

- Recommendations from the condition assessment, field testing results, and hydraulic capacity evaluation were integrated into a set of recommendations for each land use scenario.
  - Existing/near-term improvement recommendations include 20,826 feet of pipeline replacement and tank improvements.
  - Build-out improvement recommendations include 126,936 feet of new or pipeline replacement.

3.2 PURPOSE

This chapter describes the infrastructure assessment that was performed of the Mare Island potable water system (hereafter referred to as the water system). The assessment included several steps, as listed below and further described in the following sections:

- Existing Data – collection and review of existing data sources.
- Existing System Mapping – development of a consolidated water system map.
- Existing System Description – description of the existing water system configuration and components.
- Condition and Useful Life Assessment – an evaluation of existing condition-based on asset attributes and select testing; development of useful life (UL) and remaining useful life (RUL) estimates; and development of condition-based improvement recommendations.
- Capacity Evaluation – hydraulic capacity and performance evaluation of the water system using the City's hydraulic model to determine capacity-related improvement recommendations.
- Recommendations – integration of condition and capacity recommendations in terms of serving the two land use scenarios (existing/near-term and build-out).

3.3 EXISTING DATA

Table 3-1 summarizes the data received from the Client Team to support the water system assessment.

Table 3-1. Existing Data Summary

Data Source	Details	
City of Vallejo water system GIS, provided March and April 2023	Relevant feature classes and attributes included: • Distribution Mains and Transmission Pipelines including the following attributes: diameter, length, material, install date, lifecycle status, water category (e.g., potable, private, fire water, service water, etc.) • System Valves • Service Laterals • Hydrants • Water Zones	
Water system improvements from City of Vallejo's utility as-builts	• Mare Island Sewer and Water Main Replacement Project (2020) • Coal Sheds – Phase 2 (2017) • Walnut Avenue and "E" Street (2007) • Farragut Village – Unit No. 3 (2005) • Farragut Village – Unit No. 4 (2005) • Club Drive (2005) • Coral Sea South Village – Unit No. 1 (2005)	• Flagship Drive – Phase 1B & 1C (2005) • Azuar Drive and Flagship Drive (2004) • Kansas Street (2004) • Azuar Drive from Connolly Street to Residential Parkway (2004) • Mare Island Reservoir and Pipeline Project: Phase Two (1999)
Navy Fresh Water Quad Maps	• Created circa 1980s • Shows alignment, material, and diameter of existing and abandoned water system pipelines within Mare Island	
Other Documents	• Transmission pipeline crossings facility maps • Mare Island Specific Plan, Amended August 2014 • 2005 Mare Island Water Model prepared by Korve Engineering • 2015 City of Vallejo Master Water Plan (2015 MWP) prepared by Kennedy/Jenks Consultants	

3.4 EXISTING SYSTEM MAPPING

Several data sources were provided and consolidated to produce the existing system mapping and attributes. Throughout the consolidation process, the City was consulted to confirm status of certain assets, GIS attribute field preferences, and hierarchy of available data. The process undertaken is described below:

1. A copy of the City's water system GIS was created for verifying and editing water system features.

3. The Distribution Mains feature class was organized to only include Mare Island's distribution and transmission pipelines.
    a. All hydrant laterals and service connection laterals that were previously within the Distribution Mains feature class in GIS were removed and placed in the Service Laterals feature class.
    b. Transmission pipelines were previously located under a Transmission Main feature class. All Transmission Main features for Mare Island's water system were merged into the Distribution Mains feature class.
    c. Pipelines with GIS attributes 'Owned By' and 'Maintained By' listed as "Private" were not included in this assessment. This included approximately 1.8 miles of pipelines within Touro University as well as some fire protection pipelines for industrial buildings.
    d. In addition to the pipelines identified in GIS as privately owned and maintained, the City identified other pipelines that are considered private (i.e., not owned or maintained by the City). These pipelines are indicated on Figure 3-1 for information purposes and are included in the assessments herein to reflect the system-wide nature of this study.
4. Recent as-builts for Mare Island's water system were compiled and used to digitize new distribution and transmission pipelines in GIS.
    a. All available pipeline attributes were input into GIS attribute fields. Asset ID and Global ID fields were left Null and are expected to be populated by City staff.
    b. Pipelines that were abandoned or removed per the as-builts were updated using the Lifecycle Status and Water Category attribute fields.
5. Navy Quad Maps were used to verify water system features and attributes that did not have recent as-builts available.
    a. The water system pipeline material and diameter shown on the Navy Quad Maps were assumed to be more accurate compared to the existing GIS attributes per City staff.
    b. Distribution Main material and diameter attributes were corrected in GIS as needed based on a thorough review of the Navy Quad Maps.
    c. Pipeline alignment in GIS was verified and corrected as needed based on the Navy Quad Maps.

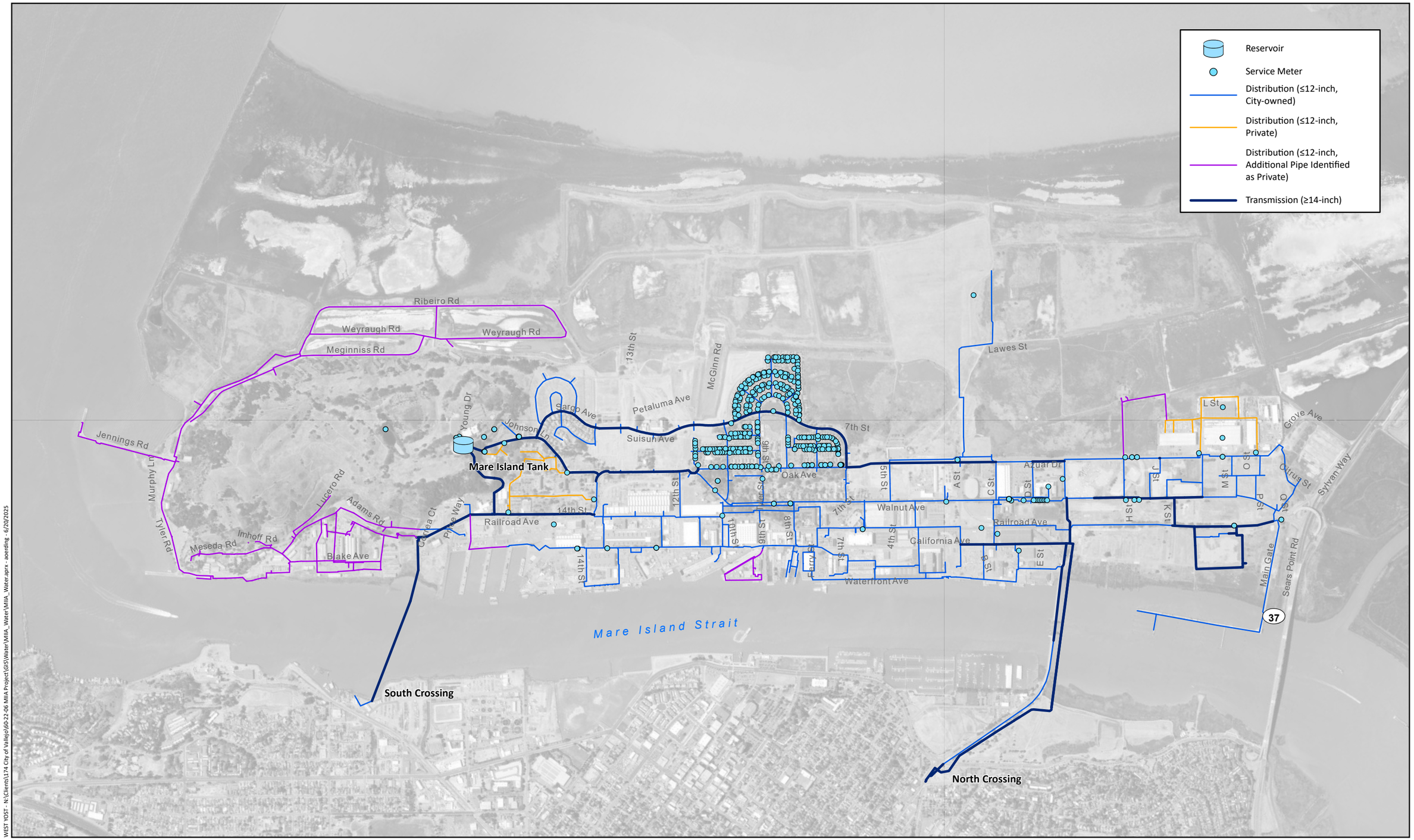
The resulting existing water system map is shown on Figure 3-1.

3.5 EXISTING SYSTEM DESCRIPTION

The City of Vallejo Water Department provides water service to Mare Island through three transmission pipelines crossing Mare Island Strait. Two of the crossing pipelines are at the north end of the Island, just north of the Mare Island Bridge, and include an 18-inch diameter HDPE (DR11, with internal diameter 14.53 inches) transmission pipeline constructed in 2020, and a 14-inch diameter cast iron transmission pipeline of unknown installation date. The third crossing pipeline is at the south end of the Island, which crosses just south of the finger piers and connects to the mainland just north of Lemon Street. This south crossing is a 20-inch diameter cast iron transmission pipeline constructed around 1941.

The Mare Island system is part of the City's 212 pressure zone (212 Zone), also referred to as the Grid Zone. The 212 Zone is supplied by the City's Fleming Hill Water Treatment Plant via the East Grid and West Grid pressure reducing valves. The 212 Zone serves the majority of the City including Mare Island.

Mare Island has one 6.4-million-gallon (6.2-million-gallon usable capacity) at-grade gravity storage tank (also referred to as a reservoir). Figure 3-1 shows Mare Island's existing key water facilities including transmission and distribution pipelines, the storage tank, and most of the water meters (those available in GIS).



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Prepared by:

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Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.

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Existing Water System

Figure 3-1

3.5.1 Pipelines

The existing water system consists of approximately 35.4 miles of transmission and distribution pipelines, with diameters ranging from 2 to 24 inches. Service laterals (i.e., laterals for fire hydrants and service connections) were not included in this assessment.

A small portion of 2- and 3-inch diameter galvanized steel distribution pipelines are shown in the GIS at the southern end of the water system . The south end is still active but there are no known services even though there appears to be a nightly demand. There may be an unmetered irrigation connection on a timer in the old officer housing area.

By length, approximately 72 percent of the Mare Island's pipelines are between 6- and 12-inch diameter. Cast iron (CI) and polyvinyl chloride (PVC) are Mare Island's two most common pipe materials, making up 55 and 19 percent of the water system, respectively. Approximately 73 percent of the pipelines are greater than 75 years old, and 60 percent of pipelines are greater than 100 years old.

The diameter, material, and age distributions of Mare Island's water system pipelines are summarized in Table 3-2, Table 3-3, Table 3-4, and Table 3-5. Additionally, the diameters, material, and age distributions are presented on Figure 3-2, Figure 3-3, and Figure 3-4.

Table 3-2. Water Pipelines by Diameter

Pipe Diameter, inches	Length of Pipelines, ft	Length of Pipelines, miles	Percent of System, %
Less than 4	3,502	0.7	1.9
4	9,095	1.7	4.9
6	21,821	4.1	11.7
8	50,615	9.6	27.1
10	23,743	4.5	12.7
12	38,648	7.3	20.7
14	7,526	1.4	4.0
16	580	0.1	0.3
18	16,009	3.0	8.6
20	10,473	2.0	5.6
24	4,908	0.9	2.6
Total	**186,920**	**35.4**	**100%**

Source: City water GIS, March 2023 updated with best available information.

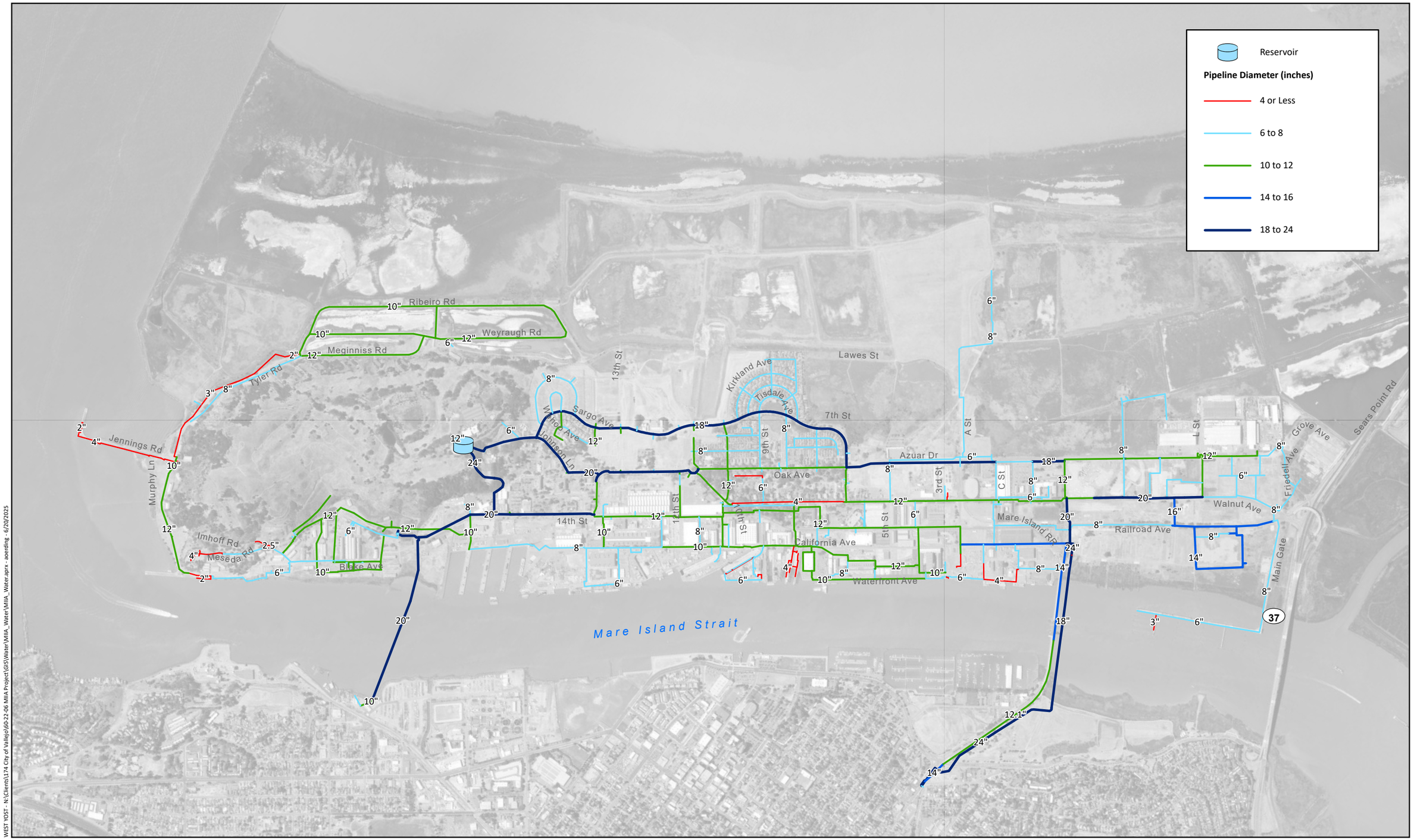
Table 3-3. Water Pipelines by Material

Pipe Material	Length of Pipelines, ft	Length of Pipelines, miles	Percent of System, %
Cast Iron	102,504	19.4	54.8
Polyvinyl Chloride	36,154	6.8	19.3
Asbestos Cement	20,457	3.9	10.9

Table 3-3. Water Pipelines by Material			
Pipe Material	Length of Pipelines, ft	Length of Pipelines, miles	Percent of System, %
High Density Polyethylene	9,431	1.8	5.0
Steel	8,791	1.7	4.7
Galvanized Steel	7,657	1.5	4.1
Ductile Iron	1,926	0.4	1.0
Total	**186,920**	**35.4**	**100%**
Source: City water GIS, March 2023 updated with best available information.			

Table 3-4. Water Pipelines by Age			
Pipe Age	Length of Pipelines, ft	Length of Pipelines, miles	Percent of System, %
Less than 25 years old (1998-Present)	47,284	9.0	25.3
25 to 75 years old (1948-1998)	1,601	0.3	0.9
75 to 100 years old (1923-1948)	24,309	4.6	13.0
Greater than 100 years old (Before 1923)	111,422	21.1	59.6
Unknown	2,304	0.4	1.2
Total	**186,920**	**35.4**	**100%**
Source: City water GIS, March 2023 updated with best available information.			

Table 3-5. Water Pipelines by Age and Diameter				
	Length of Pipelines, ft			
Pipe Age	Less than 8-inch Diameter	8- to 12-inch Diameter	Greater than 12-inch Diameter	Total Length, ft
Less than 25 years old (1998-Present)	480	27,628	19,175	47,284
25 to 75 years old (1948-1998)	-	1,601	-	1,601
75 to 100 years old (1923-1948)	2,725	15,326	6,259	24,309
Greater than 100 years old (Before 1923)	30,898	67,079	13,444	111,422
Unknown	314	1,373	617	2,304
			Total	**186,920**
Source: City water GIS, March 2023 updated with best available information.				



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Prepared by:

WEST YOST

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Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Distribution mains and water system facilities outside of Mare Island are not shown. Transmission mains are displayed to show point of connection to Mare Island water system.
3. Pipeline diameter is based on best available data.

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Existing Water System Pipeline Diameters

Figure 3-2



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Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Only Mare Island water system facilities are displayed for clarity.
3. Pipeline material is based on best available data.

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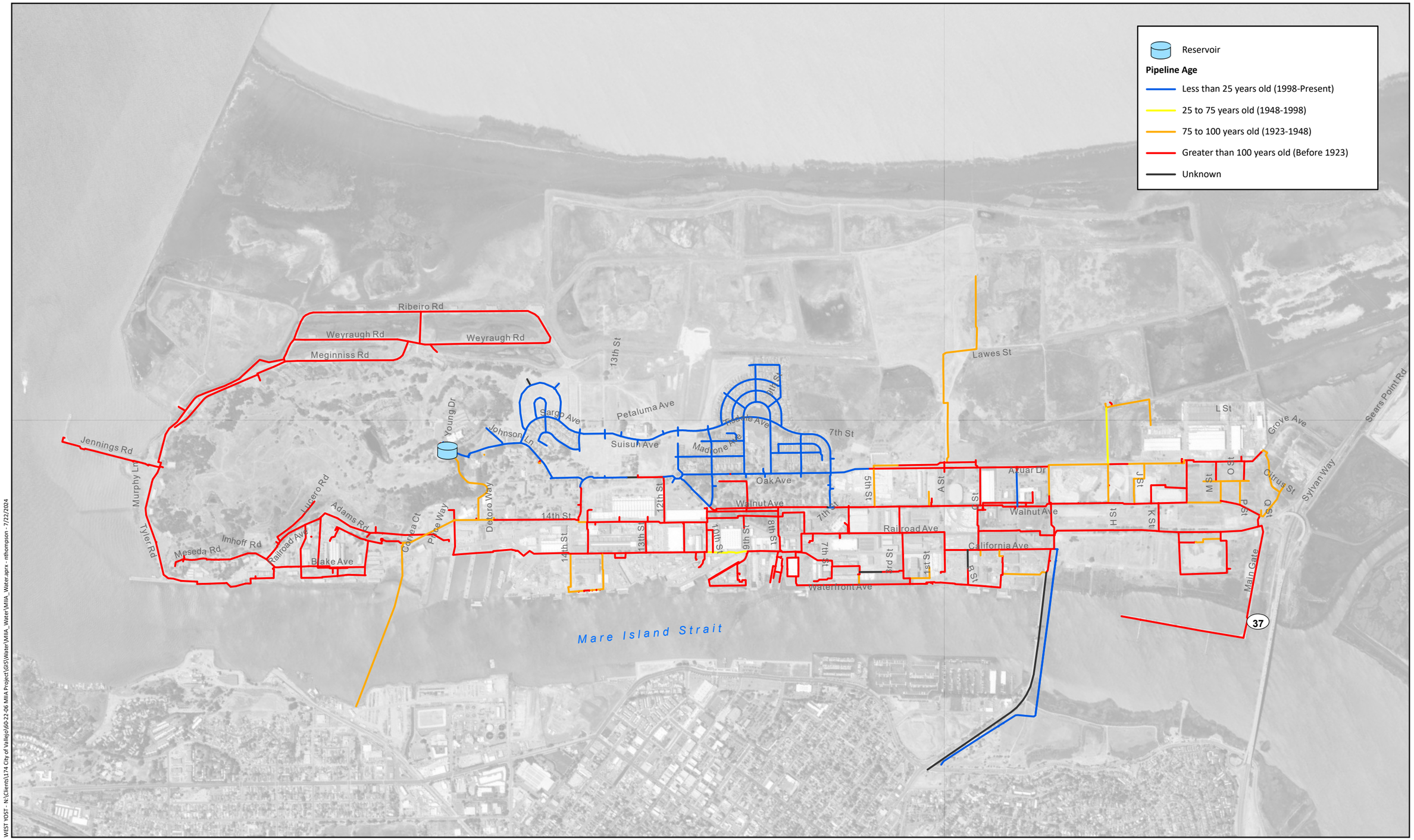
Existing Water System Pipeline Materials

Figure 3-3



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Prepared by:

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Notes:
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2. Only Mare Island water system facilities are displayed for clarity.
3. Pipeline age is based on best available data.

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Existing Water System
Pipeline Ages

Figure 3-4

3.5.2 Hydrants, Meters, and Valves

According to the City's GIS data, the Mare Island water system has approximately 271 hydrants. Of these, 98 are identified in the GIS or by the City as privately owned and maintained (including 62 on the south part of the Island). For purposes of this study, all 271 hydrants were included in the assessment.

The water system is partially metered. City staff provided anecdotal information that the majority of active connections are metered, but some connections have been found in the past that were not metered. The City is currently investigating the water usage. Figure 3-1 shows the location of most of the water meters throughout the water system. There are approximately 510 water meters with most of the meters located in residential areas and newer areas of the water system.

The water system does not have pressure reducing valves as it is part of a single pressure zone. The distribution pipelines have in-line isolation valves which are assumed to be the same age as its connected pipeline. Per the City's GIS, Mare Island's water system has approximately 971 City-owned valves.

3.5.3 Water Tank

The Zone 212 has one at-grade gravity storage tank with a design capacity of 6.2-million gallons (MG) [1]. The location of the storage tank is shown on Figure 3-1. The at-grade circular concrete tank was constructed in 1999 and effectively replaced several smaller water storage tanks previously used by the Navy. The tank is located near the southern end of Club Drive with a base elevation of approximately 176 feet. The tank has an inside diameter of 181.5 feet and a wall height of 35 feet (varies across tank section). The storage tank has an overflow inlet at an elevation of 210 feet, or approximately 34 feet above the tank's base.

3.6 CONDITION AND USEFUL LIFE ASSESSMENT

The water system assessment included an age-based condition assessment of the pipelines, hydrants, meters, and valves to assign an expected condition and estimated RUL. Broadband electromagnetic testing and UT testing were also performed in the field on select water pipelines to support the age-based assessment conclusions.

The water tank assessment included field investigation to assess the current condition of the infrastructure. Assessment methods included visual observation from the exterior of the tank and visual observations from the interior of the tank using divers. A seismic assessment of the storage tank was also performed.

3.6.1 Assessment Criteria

Water system assets were assessed initially for their condition by calculating each asset's estimated RUL. RUL is calculated by subtracting an asset's age (how long the asset has been installed) from its estimated UL:

$$RUL = UL - age$$

[1] The Mare Island Reservoir and Piping Project: Phase Two Record Drawings (Montgomery Watson, 1999) were referenced for tank evaluations and are shown in the U.S. Geological Survey's mean sea level datum.

In the absence of condition or performance data, this approach provides an initial determination of assumed condition and can be used to project estimated renewal needs. Installation dates were not available for hydrants, meters, and valves, so age assumptions were made based on available information (details in subsequent sections).

Based on their estimated RUL, assets were assigned an assumed condition from *Excellent* to *Poor*, as presented in Table 3-6. Newer assets with 50 percent or more of their UL remaining are expected to be in good to excellent condition. Older assets with less than 50 percent of their UL remaining are expected to be in fair to Poor condition. The following sections detail the assumptions and criteria used to estimate UL, RUL, and assumed condition for each asset type.

Table 3-6. RUL to Condition Translation

Percent RUL, %	Assumed Condition
100 to 80	Excellent
79 to 50	Good
49 to 20	Fair
19 and under	Poor

3.6.2 Pipelines

UL for pipelines is estimated by pipe material type, as presented in Table 3-7. Each pipeline material is assigned an estimated UL value. This represents the typical service duration under normal operating conditions before significant deterioration or failure is expected to occur.

Table 3-7. Water Pipeline Estimated Useful Life

Pipe Material	Estimated Useful Life Value, years
Cast Iron	90
Polyvinyl Chloride	100
Asbestos Cement	80
High Density Polyethylene	100
Steel	40
Galvanized Steel	60
Ductile Iron[(a)]	95

Source: Industry standard organizations including AWWA and BACWA

(a) The City has experienced severe corrosion on ductile iron pipeline in other areas of the City. Metallic pipes in corrosive soils may experience a shorter UL. Including assessments performed as part of this study (see Section 3.6.2.2), there is no direct evidence of corrosive soil on Mare Island, although extensive investigations have not been performed.

3.6.2.1 Pipeline Remaining Useful Life Summary

Table 3-8 summarizes the assumed condition of water pipelines based on each asset's estimated RUL. Note that the *Poor* condition category was split between pipes with greater than zero percent RUL and less than or equal to zero RUL. Approximately 69 percent of Mare Island pipelines have zero or less UL remaining. This is due to the large amount of very old pipelines that were installed by the Navy starting in 1868 (pipes up to 155 years old). Approximately 24 percent of pipelines have greater than 80 percent of their UL remaining; this is predominantly the PVC pipelines in the recently developed areas of the Island between Azuar Drive and Flagship Drive.

Table 3-8. Water Pipeline Condition Summary

Condition, %	Length of Pipelines, miles	Percent of System, %
Excellent (100 to 80 RUL)	8.6	24
Good (79 to 50 RUL)	0.4	1
Fair (49 to 20 RUL)	0.1	<1
Poor (greater than 0 to 19 RUL)	1.3	4
Poor (0 or less RUL)	24.6	69
No Data Available	0.4	1
Total	**35.4**	**100%**

3.6.2.2 Age-Based Replacement Projections

Based on the estimated RUL of each pipeline, an expected replacement year can be projected, providing insight into anticipated long-term replacement needs. These age-based projections provide a reasonable basis for long-term planning, and where possible, should be refined using direct condition data. Condition-based refinements are addressed in the remaining sections of this chapter to develop a more targeted understanding of infrastructure rehabilitation needs.

Using the estimated RUL of the pipelines, a projection of replacement needs was developed to support the two land use scenarios. Table 3-9 presents the age-based replacement projections for the existing/near-term (by 2027) and build-out scenarios (by 2063). These projections assume that pipelines are replaced at the end of their estimated UL (i.e., when RUL reaches zero).

As shown in Table 3-9, 24.6 miles of pipelines, or approximately 69 percent of the Mare Island system is expected to require replacement by 2027. An additional 1.4 miles of pipeline are expected to require replacement by 2063. Figures 3-5 and 3-6 present the pipeline replacement projections spatially for each scenario. Figure 3-6 (build-out scenario) assumes that existing/near-term pipeline replacements have been performed on or around 2027. It is expected that some pipelines that have reached the end of their UL will not be replaced as they may have served areas of Mare Island that do not have a future need for water service. Such pipelines may instead be abandoned in place or removed as needed.

Table 3-9. Age-Based Water Pipeline Replacement Projections

Replacement Year	Length of Pipelines, miles	Percent of System, %
By 2027 (Existing/Near-Term)	24.6	69
By 2063[(a)] (Build-Out)	1.4	4

(a) Assumes all pipes identified for replacement in the Existing/Near-Term scenario were replaced on or around 2027.



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Prepared by:

WEST YOST

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Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Only Mare Island water system facilities are displayed for clarity.
3. Pipeline replacement projections are determined using pipeline age and assumed pipeline useful life per industry standards. Pipeline age and material are based on best available data.
4. It is expected that some pipelines that have reached the end of their useful life will not be replaced as they may have served areas of Mare Island that do not have a future need for water service. Such pipelines may instead be abandoned in place or removed as needed.

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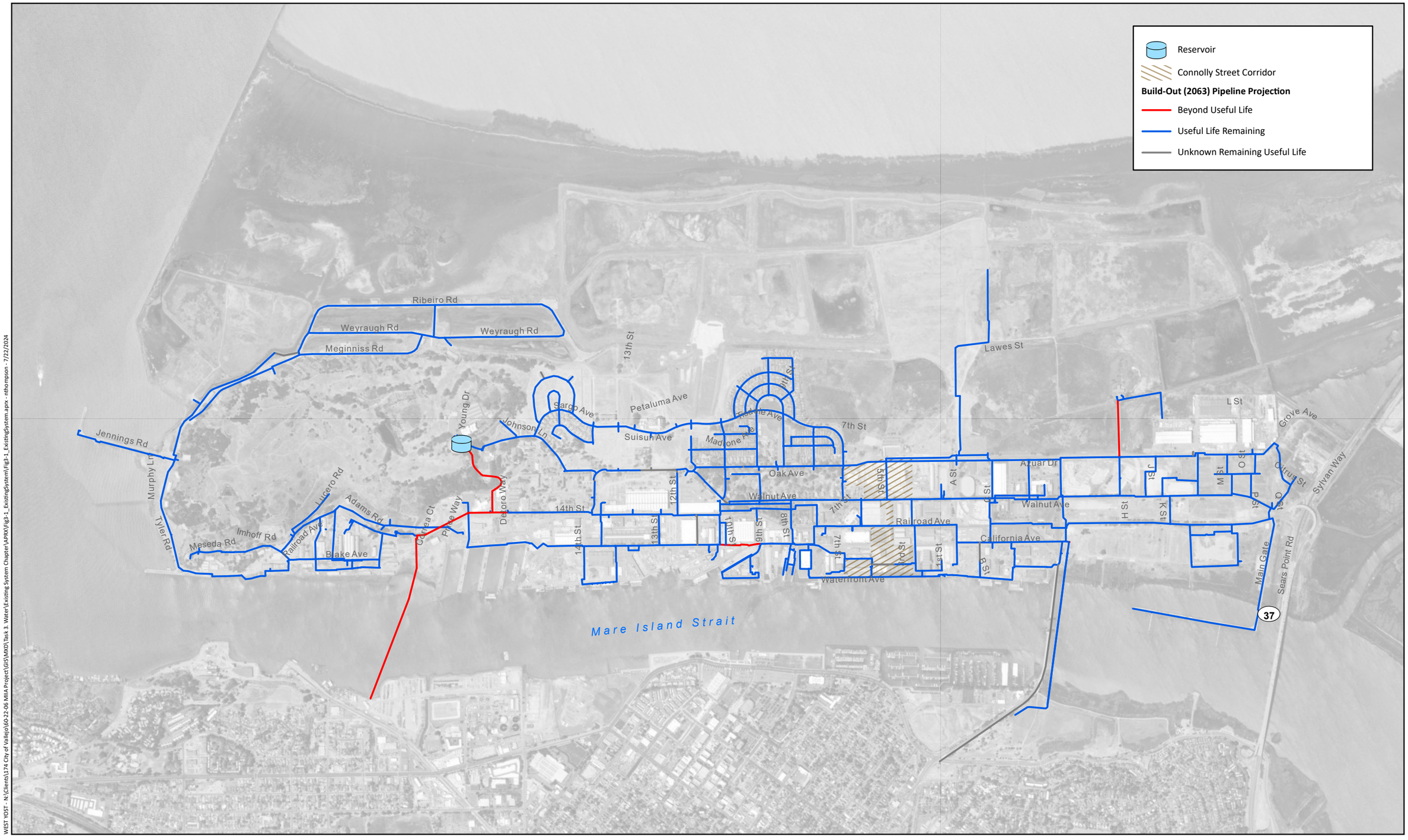
Age-Based
2027 Pipeline Replacement Projections

Figure 3-5



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Prepared by:

WEST YOST

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2. Only Mare Island water system facilities are displayed for clarity.
3. Pipeline replacement projections are determined using pipeline age and assumed pipeline useful life per industry standards. Pipeline age and material are based on best available data.
4. Build-out pipeline replacement projection assumes that existing/ near-term pipeline replacements have been performed.

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Age-Based
2063 Pipeline Replacement Projections

Figure 3-6

3.6.2.3 Pipeline Field Testing and Evaluation

The purpose of the field testing and evaluation was to determine the extent of metal degradation on select metallic pipelines and to validate the age-based replacement projections.

Field testing included visual assessment, broadband electromagnetic (BEM) testing, and UT testing on select CI and ductile iron water pipelines. Evaluation and conclusions were based on results from the three testing methods and review of available records and existing documentation.

Locations for testing were selected in June 2023 based on the desktop condition and UL assessment and City feedback. Testing was performed by V&A Consulting Engineers at seven excavated sites and one vault on September 27, November 7, and November 9, 2023. The City provided access to each site, including excavation, shoring, and backfilling of the excavations. The testing locations are presented on Figure 3-7.

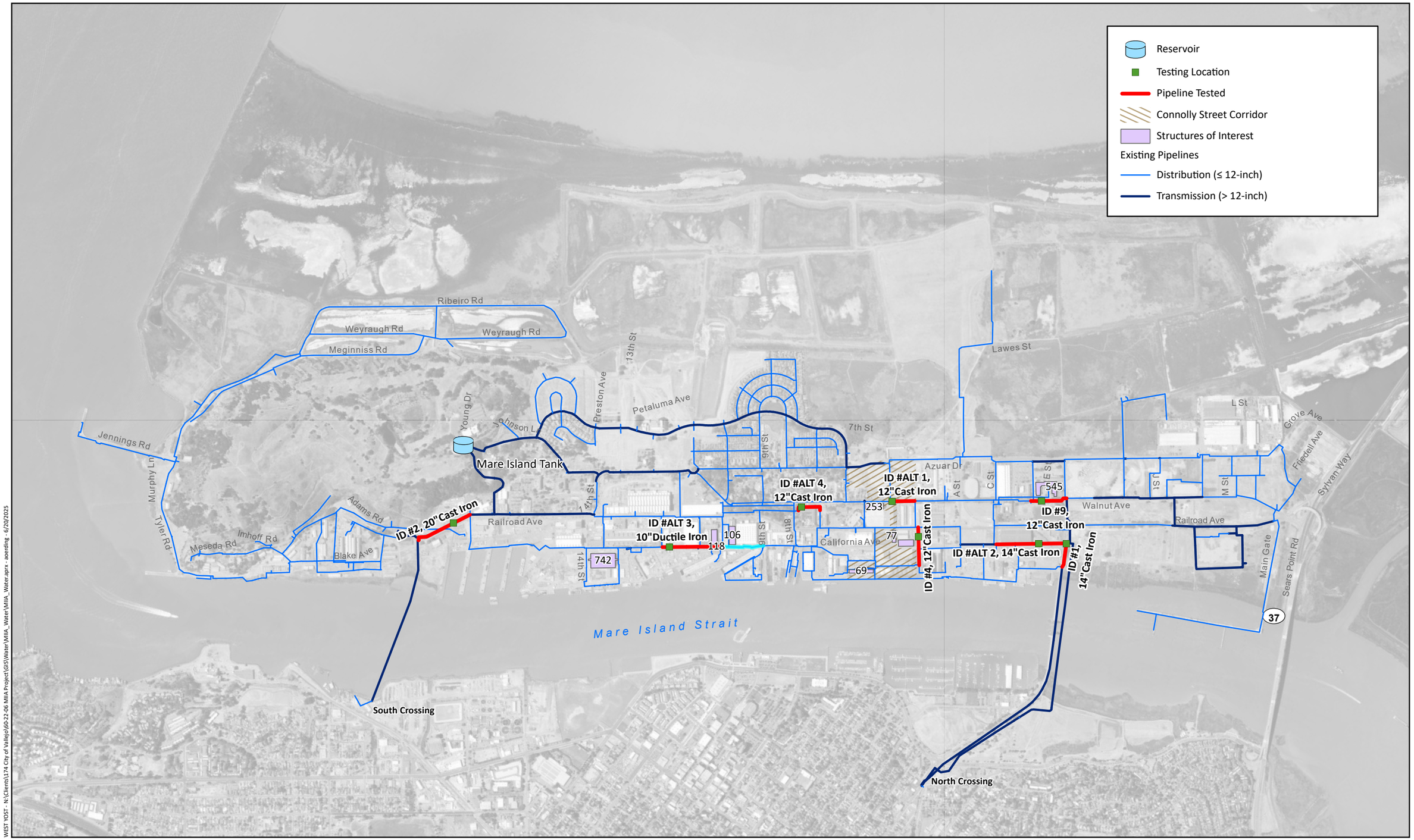
3.6.2.3.1 Testing Methodologies

BEM testing measures average wall thickness and provides an indication of wall thickness reduction. BEM can also identify graphitic corrosion of a pipe (the deterioration of CI pipes, in which the metallic constituents are leached or turned into corrosion products, leaving the graphite intact and exposed). A contour map of the apparent structural wall thickness around the circumference is also created for each location using the testing scan.

UT testing uses high-frequency sound waves transmitted through the pipe wall to measure wall thickness. Measurement locations generally were referenced to the coordinates used in the BEM scan grid. In areas of pitting, or extremely localized corrosion, a pit depth gauge was used since UT does not provide acceptable readings due to the rough surface.

Qualitative visual evaluations focused on the condition of the visible piping. Pipeline diameter and depth of cover was measured. It should be noted that much of the visual assessment data is subjective and is based upon V&A's extensive experience evaluating metallic structures and components in the water and wastewater industries.

Further details on testing and evaluation methods are provided in the V&A Mare Island Water Main Condition Assessment Report, included as Appendix 3A.



WEST YOST - N:\Clients\174 City of Vallejo\60-22-06 MIIA Project\GIS\Water\MIIA_Water\MIIA_Water.aprx - aoerding - 6/20/2025

Prepared by:

WEST YOST

0 1,500 Feet

Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Only Mare Island water system facilities are displayed for clarity.

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

City of VALLEJO California

Water System
Pipeline Testing Locations

Figure 3-7

3.6.2.3.2 Basis of Testing Evaluation

The pipe evaluations were based on comparison to an industry standard which specifies parameters including pipe diameter, pressure rating, wall thickness, casting tolerance, and other manufacturing specifications. Due to lack of record information, the applicable industry standards were not readily available for the pipelines. Therefore, because of the age of the piping, the possibility of undocumented pipe replacement, and the various standards that applied to CIP and DIP pipe, a review of the possible applicable standards was conducted to select the most appropriate standard for each pipeline.

The manufacturing casting tolerance gives an indication of how much variation in wall thickness could be expected for the pipe when it was new. It is common to measure remaining wall thickness that is less than the nominal thickness (i.e., thickness that can vary, plus or minus by a specified manufactured tolerance) but within the casting tolerance. When the remaining wall thickness is within the tolerance, it is more difficult to determine if corrosion has occurred because the pipe could have been manufactured at that wall thickness. On the other hand, even if the pipe has conclusively experienced corrosion, the corrosion can be considered negligible if it is within the casting tolerance because it would still meet the requirements for new pipe. In this assessment, wall thickness loss is determined by subtracting the minimum measured wall thickness from the nominal thickness, even if the actual as-manufactured thickness may have differed.

It should be noted that selecting the applicable industry standard for each pipe was based on the balance of probability rather than certainty. The actual pipe standard and class is not known for any of the sites, and the age of the pipe only serves as a rough guide since there is some evidence of later, undocumented pipe replacement (e.g., Site Alt 3, which was thought to be CIP, but found to be DIP when excavated).

3.6.2.3.3 Summary of Testing Results

Table 3-10 presents the measured external dimensions, selected industry standard, and results of the testing and visual evaluations for each pipeline.

Three of the eight sites had calculated thickness loss that is still within the manufacturing tolerances for the assumed pipe specification, so the corrosion at these sites can be considered negligible. At four sites, the calculated wall thickness loss ranged from 13 percent to 26 percent, which is considered moderate. Site Alt 4 had a calculated thickness loss of 22 percent, which should be considered significant because this site appears to have pipe originally rated at only 86 psi, as compared with the maximum allowable service pressure of 80 psi (see details in Conclusions below).

Table 3-10. Summary of Water Pipeline Testing Results

Site	Diameter and Material	Exterior Condition	Assumed Pipe Specifications				Maximum Thickness Loss			Interior Corrosion Level
			Class (Year of Standard)	Pressure Rating, psi	Nominal Thickness, inches	Casting Tolerance, inches	inches	Percent	Within casting tolerance?	
ID #2 Azuar Ave east of Bldg. 1296	20" CIP	Good	250 (1931)	250	0.88	0.08	0.160	18%	**No**	Moderate
ID Alt 2 West side of Nimitz Ave south of Causeway near Bldg. 599	14" CIP	Good	D (1908)	173	0.82	0.08	0.217	26%	**No**	Moderate
ID #1 Vault on the north side of the intersection of Nimitz Ave and Causeway	14" CIP	Fair	D (1908)	173	0.82	0.08	0.214	15%	**No**	Moderate
ID #4 West side of Nimitz Ave between Bldgs. 98 and 207	12" CIP	Fair	D (1908)	173	0.75	0.08	0.100	13%	**No**	Moderate
ID #9 East side of Walnut Ave opposite driveways for Bldg. 545	12" CIP	Good	B (1908)	86	0.62	0.08	0.041	7%	Yes	Negligible
ID Alt 1 West side of Walnut Ave just north of Connolly St	12" CIP	Fair	B (1908)	86	0.62	0.08	0.048	8%	Yes	Negligible
ID Alt 4 East of Walnut Ave between parking stalls and 521	12" CIP	Good	B (1908)	86	0.62	0.08	0.139	22%	**No**	Significant
ID Alt 3 West side of Nimitz Ave north of Building 680	10" DIP	Fair	50 (1965)	250	0.29	0.06	0.029	10%	Yes	Negligible

3.6.2.3.4 Testing Conclusions and Recommendations

Based on the results of the testing and evaluations, the following conclusions can be made:

1. Qualitatively, and as compared with other buried pipelines of similar vintage, the exterior condition of the pipelines at the excavation locations ranged from fair to good. The interior corrosion level based on wall thickness loss ranged from negligible to significant. Given the expected age of the pipelines and the thickness losses measured, the corrosion rate has been relatively low. This probably means that the failure rate for the pipes will be relatively stable in the near and medium future, although it will increase long-term.
2. On the other hand, although the measured wall thickness losses are considered minor to moderate in general terms, at many locations the original design pressure rating of the pipes appears to have been only 86 psi (Class B pit-cast pipes per the 1908 standard). This pressure rating is just above the maximum allowable service pressure for the City system of 80 psi under normal operating conditions (note that Mare Island operates at a higher pressure, approximately 90-100 psi; see Section 3.7.2). This is in contrast to more modern materials, which generally have higher pressure ratings that provide a safety factor against moderate corrosion (for example, with present-day DIP, the lowest pressure class for smaller pipe sizes is generally 250 or 350 psi, so a wall thickness loss of around 25 percent would be less of a concern at the anticipated working pressure.)

   If the pipe was installed before 1908 at some locations, as has been reported based on record information, the pressure ratings of the original pipe could have been even less than have been assumed in this report. Before the 1908 AWWA standard was introduced, there was less standardization, and sometimes lower-quality iron and manufacturing processes were employed. Given this and contemporary discussions in the literature surrounding the 1939 and 1952 AWWA standards, the pressure ratings attached to the CIP sites in this report should be regarded with caution.
3. Considering that other sites (pipelines not identified as the Class B standard) had comparable or greater wall thickness loss (in inches), it appears likely that a significant fraction of the apparent Class B portions of the network would exhibit similar or greater wall thickness loss. This is important because the Class B pipeline has the thinnest wall and least tolerance for wall loss of the CIP pipes under the 1908 standard.
4. It is possible (perhaps likely) that the condition of the pipes will differ in other locations. Local soil conditions, groundwater conditions, etc., could result in concentrated corrosion activity in localized areas, which would not be reflected by this assessment.
5. Because the corrosion rate appears to be relatively low, the performance and failure rate of the system will probably remain approximately constant in the short term if the operating pressure remains the same. As presented in Section 3.7, the pressures of the system are not expected to increase above 80 psi, but are expected to increase to improve system performance. Much of the system has existing pressures below 80 psi. Therefore, when recommended improvements are implemented, it is expected that these increased pressures may cause breaks and leaks, especially within the CIP Class B portions of the network.

   The City water staff provided anecdotal information that in the past, changing pressures in the system, even as little as five (5) psi have caused breaks and leaks. This supports the conclusions above that a portion of the system may indeed be the Class B, 86 psi pipe, which will continue to be susceptible to pressure changes.

6. Regarding further investigations:
    a. Further wall thickness testing, given the difficulties encountered in finding the pipelines and dealing with spoils from excavations (including inability to use vacuum excavation), would probably not be cost-effective and is not strongly recommended. This assessment has captured a range of wall thickness loss throughout the system, and this could probably be taken as typical except at local "hot spots" as may exist. Wall thickness testing does not address the pressure rating of the original pipe, which appears to be at least as significant a factor in determining the remaining strength of the pipe.
    b. Consider collecting pipe samples for inspection and testing when pipe sections are excavated and accessible, for example, when failures occur or pipe segments are replaced.

3.6.2.4 Condition and Useful Life Assessment Conclusions

Key conclusions from the age-based and field condition assessments include:

- The results of the field testing indicate that exterior condition of pipelines tested ranged from fair to good compared with buried pipelines of similar age. Interior corrosion ranged from negligible to significant.
- There is evidence that the pressure ratings of many of the existing pipelines may be as low as 86 psi (compared to a maximum allowable service pressure for the system of 80 psi under normal operating conditions).
- Due to the lack of record information (resulting in unknown pipe age and material) in addition to varying soil and groundwater conditions, it is likely that the conditions of pipe will differ in other locations of the system.

For these reasons, the age-based replacement projections presented in Table 3-9 and Figures 3-5 and 3-6 are reasonable and appropriate for the Mare Island water system. The pipeline testing results support the age-based evaluation. The age-based and field-testing results are integrated with the hydraulic capacity recommendations at the end of this chapter to provide a comprehensive set of recommendations for each land use scenario (existing/near-term and build-out).

3.6.3 Hydrants, Meters, and Valves

Installation dates were not available for hydrants, meters, or valves. Per City staff, there have been minimal replacements of hydrants and valves. Therefore, hydrants and valves were assumed to be the same age as their adjacent/connecting pipeline.

The City began an advanced metering infrastructure (AMI) project in 2023 which includes replacing all existing meters within the Mare Island water system. The City's GIS shows approximately 510 existing water meters and was the basis for this evaluation.

It is expected that some hydrants, meters, and valves will not be replaced as they may have served areas of Mare Island that do not have a future need for water service. Such hydrants, meters, and valves may instead be abandoned in place or removed as needed.

Table 3-11 presents the estimated UL values for hydrants, meters, and valves.

Table 3-11. Water System Asset Estimated Useful Life Values

Pipe Material	Estimated Useful Life
Hydrants	25
Valves	40
Meters	20

Source: Industry standard organizations including AWWA (Effective Useful Life Tool) and BACWA (Asset Cost & Lifecycle Tables)

3.6.3.1 Hydrant Remaining Useful Life Summary

Table 3-12 summarizes the assumed condition of hydrants based on estimated RUL. Approximately 67 percent of Mare Island hydrants have zero or less UL remaining.

Hydrants are assumed to require replacement when they reach zero RUL (end of their UL). Table 3-13 presents the age-based replacement projections.

Table 3-12. Hydrant Condition Summary

Condition, %	Number of Hydrants	Percent of System, %
Excellent (100 to 80 RUL)	0	0
Good (79 to 50 RUL)	0	0
Fair (49 to 20 RUL)	82	30
Poor (greater than 0 to 19 RUL)	6	2
Poor (0 or less RUL)	182	67
No Data Available	1	<1
Total	**271**	**100%**

Table 3-13. Age-Based Hydrant Replacement Projections

Replacement Year	Number of Hydrants	Percent of System, %
By 2027 (Existing/Near-Term)	188	64
By 2063 (Build-Out)[(a)]	271	100

(a) All hydrants to be replaced by 2027 must be replaced again by 2063 (due to the 25-year estimated UL of hydrants).

3.6.3.2 Valve Remaining Useful Life Summary

Table 3-14 summarizes the assumed condition of valves based on estimated RUL. Approximately 70 percent of Mare Island valves have zero or less UL remaining.

Valves are assumed to require replacement when they reach zero RUL (end of their UL). Table 3-15 presents the age-based replacement projections.

Table 3-14. Water Valve Condition Summary

Condition, %	Number of Valves	Percent of System, %
Excellent (100 to 80 RUL)	3	<1
Good (79 to 50 RUL)	275	28
Fair (49 to 20 RUL)	9	1
Poor (greater than 0 to 19 RUL)	0	0
Poor (0 or less RUL)	681	70
No Data Available	3	<1
Total	**971**	**100%**

Table 3-15. Age-Based Water Valve Replacement Projections

Replacement Year	Number of Valves	Percent of System, %
By 2027 (Existing/Near-Term)	681	70
By 2063 (Build-Out)[(a)]	290	30

(a) All valves replaced by 2027 must be replaced again by 2067 (four (4) years after the 2063 Build-Out date) – due to the 40-year estimated UL of valves.

3.6.3.3 Meter Remaining Useful Life Summary

The City began an AMI project in 2023 which includes replacing all existing meters within the Mare Island water system. There are approximately 510 existing water meters with most of the meters located in residential areas and newer areas of the water system. Table 3-16 summarizes the age-based replacement projections of meters based on a 20-year estimated UL. Assuming that all meters are replaced around 2024, all meters would have to be replaced in 2044 and again in 2064. Note the estimated 40-year replacement of meters (2064) is one year past the build-out scenario (2063).

Table 3-16. Age-Based Water Meter Replacement Projections

Replacement Year	Number of Meters[(a)]	Percent of System, %
Near-Term AMI Project (2024/2025)	510	100
2044 (20-Year Replacement)	510	100
2064 (40-Year Replacement)[(b)]	510	100
Total	**1,530**	**-**

(a) Based on existing meters in GIS.
(b) 40-year replacement of meters (2064) is one year past the Build-Out scenario date of 2063.

3.6.4 Water Tank

The water tank condition assessment included visual observations of the tank exterior and valve vault, and visual observations from the interior of the tank using divers. A desktop seismic assessment was also performed.

3.6.4.1 Condition Assessment

A summary of the condition assessment and recommendations is presented in Table 3-17. Overall, the exterior of the tank is in good condition with isolated areas of moderate surface damage (spalling and staining) on tank walls. The interior concrete and structural components are in good condition. The interior piping has minor to moderate corrosion on fittings. The detailed Assessment Report (V&A, May 2023) is provided as Appendix 3B.

Table 3-17. Water Tank Assessment – Condition and Recommendation Summary

Tank Feature	Structure	Condition Summary	Recommendations
Exterior	Roof	• Concrete in good condition	• Reassess in 10 years
	Walls	• Spalled concrete and staining along roof interface	• Investigate stains for possible delamination and corrosion of reinforcement; repair spalls as required in 0 to 2 years
	Roof Ladder	• Good condition	• Reassess in 10 years
	Access Hatch	• Good condition with minor surface corrosion	• Reassess in 10 years
Interior	Ceiling	• Concrete in good condition	• Reassess in 10 years
	Walls	• Concrete in good condition	• Reassess in 10 years
	Support Columns	• Good condition	• Reassess in 10 years
	Floor	• Good condition with thin layer of sediment	• Reassess in 10 years
	Inlet Pipe	• Moderate corrosion on flap valve	• Replace flap valve and pipe couplings in 2 to 5 years • Recoat/touch-up pipe coating in 2 to 5 years • Replace fittings with stainless steel fittings in 2 to 5 years. Electrically bond the fittings to the pipes and install magnesium anodes to protect the immersed portions of the pipes and fittings from corrosion.
	Overflow Pipe	• Minor corrosion on flange • Sections of coating failure and minor corrosion on pipe and support brackets	• Replace flange in 2 to 5 years • Recoat/touch-up pipe coating in 2 to 5 years • Replace fittings with stainless steel fittings in 2 to 5 years. Electrically bond the fittings to the pipes and install magnesium anodes to protect the immersed portions of the pipes and fittings from corrosion.
	Access Ladder	• Good condition with minor corrosion at welds	• Reassess in 10 years
Valve Vault	Concrete	• Concrete in good condition	• Reassess in 10 years
	Piping	• Coating in good condition	• Reassess in 10 years

3.6.4.2 Seismic Assessment

The seismic assessment evaluated the tank under two scenarios based on estimated water level in the tank. The first scenario simulated water level near the roof of the tank, which results in inadequate freeboard and a potential uplift force on the roof during an earthquake. The second scenario simulated the water level seven feet from the roof which allows for 'unconstrained sloshing' and no uplift forces on the roof. A software program (Strand7) was used to perform structural analysis. A 3-D finite element model of the reservoir tank was used to analyze the equivalent static loads.

Based on the above analysis, it was concluded that if the maximum allowable water level in the tank can be limited to seven feet from the roof soffit, the structural integrity of the tank would be maintained during and after a seismic event. The record drawings show an overflow pipe about nine inches below the roof slab soffit. It is recommended that the overflow pipe be adjusted down. If the overflow pipe elevation is seven feet below the roof soffit, it is unlikely that the sloshing water will impact the roof under the seismic loads. Due to current operating conditions, the reduction in available tank capacity associated with a lower water level will not have any immediate impact of system capacity or performance; the tank water level currently does not reach more than seven feet below the roof soffit.

Additionally, the potential use of stored water for fire suppression designates the tank as a Risk Category IV structure, which is the highest risk category possible for the assessment. If future planning includes a separate fire suppression source and the reservoir tank contents were designated only for potable water (i.e., not needed for fire suppression), the tank would qualify for Risk Category III, resulting in a higher allowable water level.

The detailed Seismic Assessment Report (Delve, July 2023) is provided as Appendix 3C.

3.7 CAPACITY AND PERFORMANCE EVALUATION

The water system assessment included a hydraulic capacity and performance evaluation of the pipelines.

3.7.1 Water Demands

This section describes Mare Island's existing and future water demands and the methodology used to develop the demands. Proposed future land use was reviewed for Mare Island, including possible re-development areas and anticipated increases to the water use of existing customers. Existing water use factors were refined and applied to future land uses to calculate future demands for Mare Island. These demands will be used to evaluate the capacity and performance of the City's water system to deliver water reliably to customers.

3.7.1.1 Existing and Future Land Use

As described in Chapter 2, future growth on Mare Island is evaluated under two future timeframes: (1) the existing/near-term timeframe, which is estimated to occur by 2027; and (2) the build-out timeframe, which is estimated to occur by 2063. Future water demands are based on the current development plan, as previously described in Chapter 2 and presented on Figure 2-2.

Near-term developments include the Connolly Street Corridor project, and expansion of Lind Marine and Mare Island Dry Docks. Build-out developments include new single family residential, multi-family residential, commercial, and hospitality/cultural land use areas, as well as two special case projects: (1) Adaptive Reuse; and (2) Touro University. These developments are shown on Figure 3-8.



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Prepared by:

WEST YOST

0 1,500 Feet

Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Land uses are categorized by water use factors presented in Table 3-18.

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Build-Out Land Use for Water Demand Development

Figure 3-8

3.7.1.2 Unit Water Demand Factors

Unit water demand factors were based on the 2015 City of Vallejo Water Master Plan (2015 WMP) which were calculated from 2009-2011 water billing records. Since the 2015 WMP did not include an industrial or hospitality/cultural unit water demand factor, the industrial land use demand factor was calculated as the average of industrial use factors found in Water Master Plans for surrounding cities (e.g., City of Stockton, City of Tracy) and the hospitality/cultural demand factor was estimated using existing meter data and area hotel information to find average unit demand. These factors are presented in Table 3-18.

Table 3-18: Summary of Unit Water Use Factors

Land Use Designation		Unit Water Use Factor, gallons per day
Recommended Generalized Land Use	February 2023 Mare Island Land Use Plan	
Single Family Residential[(a)]	Single Family Residential	234 gpd/ dwelling unit
Multi-Family Residential[(a)]	Townhomes	141 gpd / dwelling unit
	Garden Style	
	Courtyard Style	
Mixed-Use[(b)]	Mixed-Use	155.1 gpd /dwelling unit
Commercial[(a)]	Office	806 gpd /acre
	Flex	
	Retail	
Industrial[(c)]	Industrial	1300 gpd /acre
Hospitality / Cultural[(d)]	Hospitality	155 gpd /room

(a) Unit use factor based on factors from the 2015 WMP.
(b) Mixed-Use use factor calculated as a 10 percent increase in the multi-family residential demand presented in the 2015 WMP.
(c) Industrial use factor calculated as average of industrial use factors used in similar Water Master Plans (e.g., City of Stockton, City of Tracy, City of Mountain House, City of Fairfield and Suisun-Solano Water Authority).
(d) Hospitality/Cultural use factor estimated using existing meter data and area hotel information to calculate an average demand per room.

3.7.1.3 Existing and Future Water Demand

Mare Island's baseline existing water production was based on historical production (meter) data, which were reviewed during calibration and update of the City's hydraulic model. The baseline water production included 20 percent non-revenue water based on the City's UWMP (Table 4-3). Non-revenue water accounts for losses in the system (i.e., hydrant flushing, leaks, etc.). The methodology used to refine and spatially locate existing water demands is described in the Model Update and Calibration Technical Memorandum (TM) (West Yost, January 2024), included as Appendix 3D, and summarized in this chapter.

To calculate average daily water demands for existing/near-term (2027) and build-out (2063) conditions for Mare Island, the City's refined water use factors were applied to the proposed land uses and development described above. Future water production requirements were estimated by adding water demand projections and future non-revenue water to the existing baseline production. Non-revenue water is estimated as nine-percent of the production requirement based on the assumption that new infrastructure will reduce the amount of water loss in the distribution system. This rate is consistent with

California Water Efficiency Partnership's goal to improve water efficiency throughout California and meet the Urban Water Use Objectives for water loss budgets.

Table 3-19 summarizes the demands for existing, near-term, and build-out timeframes. Baseline demand is currently 373,824 gpd. The incremental near-term demand, including non-revenue water, is 103,408 gpd, resulting in 477,232 gpd (0.48 mgd) total near-term demand. Incremental build-out demand, including non-revenue water, is 1,395,915 gpd (1.4 mgd), totaling 1,873,146 gpd (1.87 mgd) for total future build-out demand.

Table 3-19: Summary of Mare Island Existing and Projected Future Demands

Demand Type	Assumptions	Total Average Day Demand, gpd	Total Maximum Day Demand[a], gpd
Existing			
Existing Baseline Production	Based on 2021 meter demand allocation - includes 20% non-revenue water	373,824	613,071
Near-Term			
Connolly Street Corridor [b]			
Multi-Family Residential	373 DUs	52,593	86,253
Commercial	46,000 sqft retail, 29,000 sqft office	2,434	3,992
Hotel (35 gpm)	200 rooms	31,000	50,840
Special Case Water Users [d]			
Lind Marine	15% increase	397	652
Mare Island Dry Docks	50% increase	7,677	12,590
Non-Revenue Water	9%	9,307	15,263
	Near-Term Subtotal	*103,408*	*169,589*
	Existing + Near-Term Subtotal	**477,232**	**782,660**
Buildout[c]			
Land Use Types			
Single Family Residential	206 units	48,204	79,055
Multi-Family Residential	Townhomes (1,565 units) + Garden Style (2,262 units) + Courtyard Style (1,589 units)	763,656	1,252,396
Mixed-Use	2013 units	312,216	512,035
Commercial	111 acres	89,877	147,398
Special Case Water Users [d]			
Adaptive Reuse[e]			
Industrial (3,605 gpd)		49,176	80,649
Touro University	60% increase	7,153	11,730
Non-Revenue Water	9%	125,632	206,037
	Build-Out Subtotal	*1,395,915*	*2,289,300*
	Incremental Build-Out Subtotal	*1,499,322*	*2,458,889*
	TOTAL - Build-Out (Existing + Near-Term + Build-out)	**1,873,146**	**3,071,960**

(a) Maximum day demand equals 1.64 time average day demand (2015 WMP Table 2-10).
(b) Connolly Corridor information is based on the North Mare Island Disposition and Development Agreement and Implementation Project provided by the Mare Island Co.
(c) Buildout land use information is based on the February Land Use Plan for Mare Island provided by the Mare Island Co.
(d) Refer to Chapter 2 for details about the special case water users and the assumptions used to estimate their future demands.
(e) Adaptive Reuse refers to existing Nimitz Controlled areas that are scoped to be redeveloped in the future. West Yost r eviewed these areas and determined that only certain industrial portions of the Adaptive Reuse areas would result in a material increase in demand in the future. Thus, only these areas were included in the demand projection.

gpd = gallons per day

3.7.2 Capacity Evaluation Criteria

The planning and modeling criteria used to evaluate the Mare Island water system is based on the City's system performance and operational criteria developed in the 2015 WMP. The criteria used to evaluate for existing, near-term, and build-out conditions consist of the following:

- Minimum allowable service pressure is 30 pounds per square inch (psi) under normal operating conditions.
- Maximum allowable service pressure is 80 psi under normal operating conditions.
- Residual pressure at the flowing hydrant (during maximum day demand plus fire flow condition) at service locations must be equal to or greater than 25 psi.
- Maximum allowable pipeline velocity is five (5) feet per second (ft/s) during a non-fire demand condition.
- Maximum allowable pipeline velocity is 10 ft/s during a simulated fire demand condition.
- New pipelines are modeled with a roughness coefficient (C-factor) of 130.
- Near-term and build-out fire flow requirements assume new development will be sprinklered.

3.7.3 Capacity Evaluation Findings and Conclusions

The City's existing water distribution system hydraulic model from the 2015 Water WMP was used as the basis for the evaluation. The model update and calibration is described in Appendix 3D. Several items to note related to the model conditions include:

- Modeled pipeline diameters are based on the GIS and are internal diameters.
- The City currently has difficulty filling the Mare Island water tank due to hydraulic constraints in the 212 Zone and is evaluating the use of a booster pump station to aid in filling the tank. The tank settings in the model were based off SCADA and reflect the current operating conditions (12-14 feet maximum depth).
- Valves were closed at the intersection of Azuar Drive and Pintado Street and along Waterfront Avenue in the model to create the pressure conditions observed in the field.

The updated hydraulic model was used to evaluate existing, near-term, and build-out systems under normal operations and maximum day plus fire flow demand conditions. These conditions were selected to evaluate whether the system meets the specific City performance and operational criteria. Results from the evaluations are discussed below.

3.7.4 Existing System Assessment

The existing system evaluation is used to establish baseline conditions under each operating scenario (normal operations and maximum day plus fire flow).

3.7.4.1 Normal Operations

The normal operations scenario evaluates the hydraulics of the system during a non-fire, 24-hour simulation period under average day demand conditions. Average day demand is 260 gpm (373,824 gpd, or 0.37 mgd; see Table 3-19).

Boundary conditions in the hydraulic model were established for the North and South crossing locations serving Mare Island. The boundary conditions capture the maximum and minimum hydraulic grade line over a 24-hour window between June 8th and 9th, 2023. The boundary conditions are based on pressure data gathered through hydrant pressure recorders located in the field at the intersection of Tennessee Street and Butte Street and Wilson Avenue (Trimble Unity name V212-Tennessee15) for the north crossing and at 401 Lemon Street (Trimble Unity name V212-Lemon402) for the south crossing. The diurnal curve pattern assigned to Mare Island junctions was developed based on the SCADA data for the Grid Zone (212) from June 8, 2023. Details on boundary conditions and diurnal curve development are provided in Appendix 3D.

Results from the 24-hour average day simulation indicate that the existing system can meet the City's minimum pressure criterion of 30 psi at all customer services on Mare Island except for locations shown in red on Figure 3-9. Locations with a minimum pressure less than 30 psi are located around Mare Island Tank which is an area known by the City for having low-pressure due to the high elevation associated with the services. Since the Mare Island system is older, designed to lower pressure standards, and is known to experience breaks with pressure fluctuations, improvements to increase pressure are not recommended.

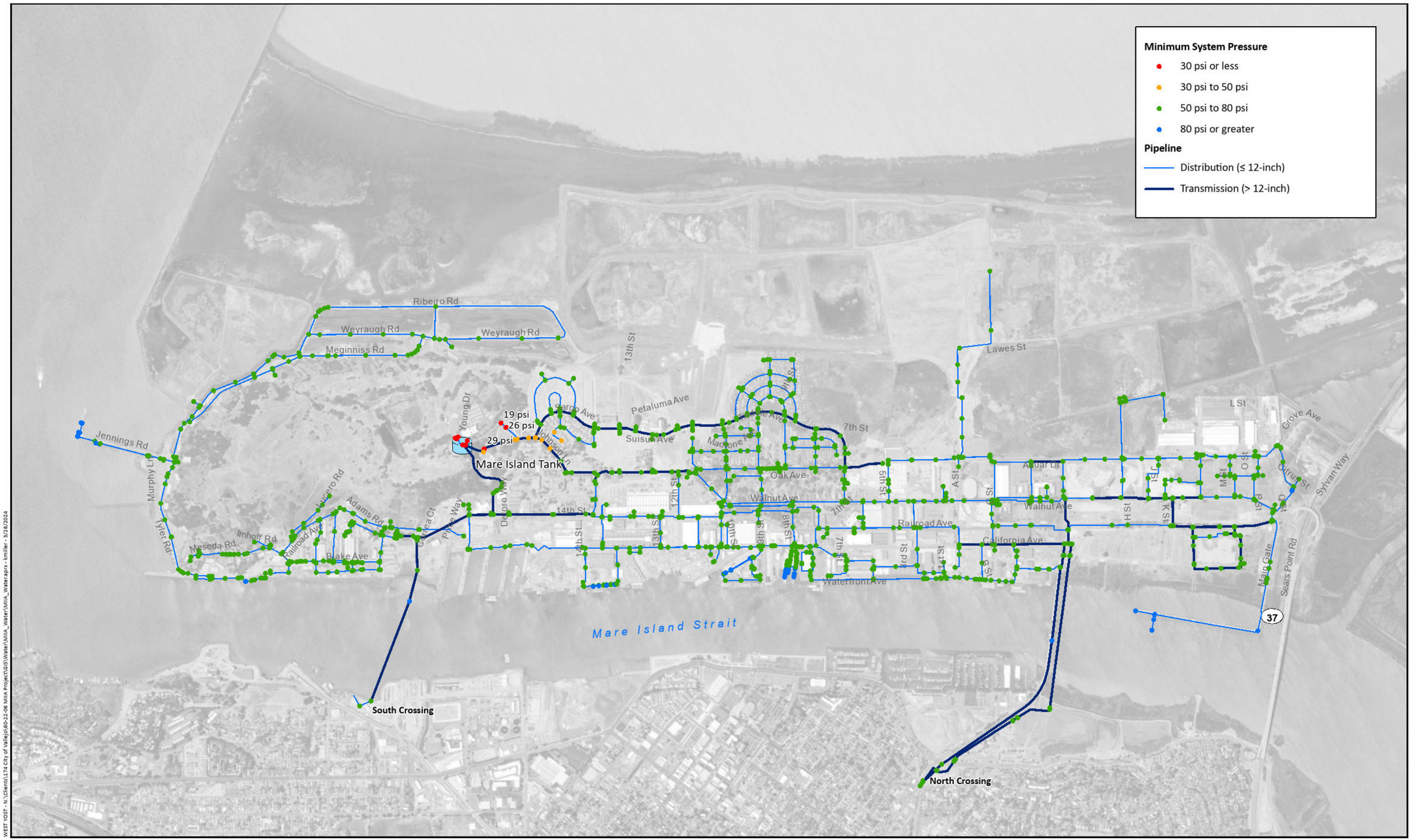
3.7.4.2 Maximum Day Plus Fire Flow

The maximum day demand plus fire flow scenario evaluates the existing fire flow availability under a maximum day demand condition. The maximum day demand for Mare Island is calculated as 1.64 times average day demand, or 426 gpm (613,071 gpd or 0.61 mgd; see Table 3-19).

To evaluate the existing fire flow availability, InfoWater Pro's *Available Fire Flow Analysis* tool was used to determine the available fire flow at a minimum residual pressure of 25 psi, per the City's criteria. For the existing system fire flow analysis, junctions that represent hydrant locations were evaluated to determine the available fire flow that can be provided during a maximum day demand condition. A maximum allowable pipeline velocity of 10 ft/s was applied to all pipelines, per the City's criteria.

Figure 3-10 summarizes the available fire flow at each tested hydrant location while meeting the minimum residual pressure criterion of 25 psi and limiting maximum pipeline velocity to 10 ft/s. Results presented are representative of the system capacity and do not represent available flow from a specific hydrant as the losses through appurtenances are not included. Areas with an available fire flow of 1,500 gpm or less, represented by the red dots on the figure, are located along the southern loop of the Island as well as in areas of single feed pipelines or where there are small diameter pipelines that constrain flow supplying an area. Areas with an available fire flow of 4,500 gpm or greater, represented as the purple dots on the figure, are located along the transmission mains north of the Mare Island Tank. Some locations appear to have different flows available at similar locations. However, the difference in flow is based on the distribution pipeline the hydrant is tied into. For example, hydrants along Flagship Drive show flows between 1,500 to 2,500 gpm on the west side and 3,500 to 4,500 gpm on the east side. The difference in available flow is due to the hydrants on the west side tying into an 8-inch diameter pipeline while hydrants on the east side tie into a 12-inch diameter pipeline.

Since the Mare Island system is older and designed to less stringent fire standards which were in place at the time pipelines were installed, the existing system fire flow evaluation presents the available fire flow only and does not recommend improvements to increase fire flow capacity to current standards.



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Notes:
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2. Only Mare Island water system facilities are displayed for clarity.
3. Minimum allowable service pressure is 30 pounds per square inch (psi) under normal operating conditions.

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Mare Island
Infrastructure Assessment

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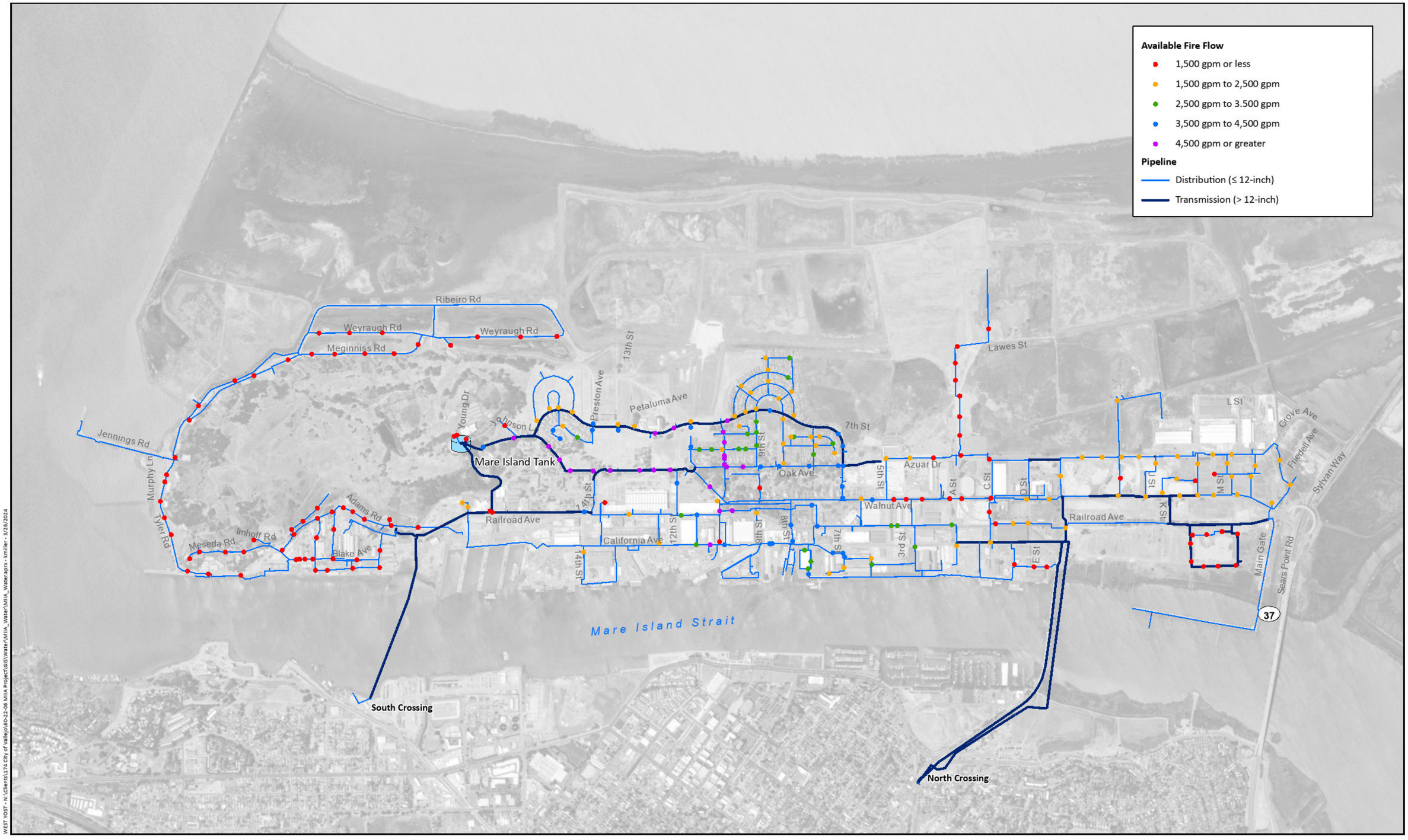
Water System - Existing System
Average Day: Minimum System Pressure

Figure 3-9



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3. Results based on maintaining a minimum residual pressure of 25 psi at customer service connections and limiting maximum pipeline velocities to 10 ft/s.
4. Minimum required fire flow is 1,500 gpm based on recommendations presented in the 2015 WMP.

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Infrastructure Assessment

City of VALLEJO California

Water System - Existing System
Max Day + Fire Flow: Available Fire Flow

Figure 3-10

3.7.5 Near-Term System Assessment

The near-term system evaluates the system's ability to serve near-term development under each operating scenario. Near-term development includes the Connolly Street Corridor project, and expansion of Lind Marine and Mare Island Dry Docks.

3.7.5.1 Normal Operations

The normal operations scenario evaluates the hydraulics of the system during a non-fire 24-hour simulation period under near-term average day demand conditions. Near-Term average day demand is 331 gpm (477,232 gpd or 0.48 mgd; see Table 3-19) with an increase in demand around Connolly Street Corridor project, Lind Marine, and Mare Island Dry Docks.

Figure 3-11 shows the minimum pressure results from the 24-hour day simulation. Results are consistent with the existing system as locations with a minimum pressure less than 30 psi are located around Mare Island Tank, and improvements are not recommended.

3.7.5.2 Maximum Day Plus Fire Flow

Since the Connolly Street Corridor development is the only development within the near-term scenario that involves changes to land use, fire flow was only evaluated for the hydrants serving that development. Fire flow requirements are based on land use, so increases in demand at Lind Marine and Mare Island Dry Docks would not affect fire flow requirements. The maximum day demand plus fire flow scenario evaluates the near-term fire flow availability under a maximum day demand condition around the Connolly Street Corridor. The maximum day demand is calculated as 1.64 times average day demand, or 543 gpm (782,660 gpd or 0.78 mgd).

To evaluate the near-term fire flow availability, InfoWater Pro's *Available Fire Flow Analysis* tool was used to determine the available fire flow at a minimum residual pressure of 25 psi. For the near-term system fire flow analysis, hydrant locations around the Connolly Street Corridor were evaluated to determine the available fire flow that can be provided, in addition to meeting maximum day demand. A maximum allowable pipeline velocity of 10 ft/s was applied to all pipelines. Fire flow requirements were assigned to each hydrant based on land uses of the proposed development and criteria established in the 2015 WMP. Table 3-20 summarizes the fire flow requirements by land use type. Proposed near-term and long-term development land uses for Mare Island were matched with fire flow categories established in the 2015 WMP.

Table 3-20. Fire Flow Requirements by Land Use		
Land Use	Fire Flow Category	Fire Flow Requirement, gpm[(a)]
Single Family	Low Density Residential	1,500
Townhomes	Medium Density Residential	2,500
Garden Style	Medium Density Residential	2,500
Courtyard Style	High Density Residential	3,000
Civic	Public & Quasi-Public	3,500
Flex	Planned Development Commercial	4,000
Hospitality/Cultural	Planned Development Commercial	4,000
Industrial	Planned Development Commercial	4,000
Office	Planned Development Commercial	4,000
Retail	Planned Development Commercial	4,000
Mixed-Use	Mixed-Use Planned Development	4,000

(a) Fire Flow requirements are based on the recommendations presented in the 2015 WMP and are presented only as general planning guidelines to evaluate the capacity of the Mare Island system.

Figure 3-12 summarizes the available fire flow at each tested hydrant location while meeting the minimum residual pressure criterion of 25 psi and limiting maximum pipeline velocity to 10 ft/s. Results presented are representative of the system capacity and do not represent available flow from a specific hydrant as the losses through appurtenances are not included. Locations that meet fire flow requirements are shown in green while locations that do not meet fire flow requirements are shown in red. Deficient locations are confined to areas where there are small diameter pipelines that constrain flow.

To meet fire flow requirements, it is recommended existing pipelines serving the Connolly Street Corridor be upsized to 12-inch diameter, and additional 12-inch diameter pipelines be installed, as presented in Figure 3-13. It is also recommended that several pipes adjacent to the Connolly Street Corridor development be replaced due to age (yellow pipes on Figure 3-13); although these pipes do not need to be upsized for hydraulic considerations, replacement of these pipes concurrent with development would improve looping and the overall condition and performance of the network in the area. The extent of the age-based replacements could be expanded if resources allow or if development requires construction in other streets/areas. Also note that if there are any significant changes to the land use/land use distribution within the Connolly Street Corridor development, the fire flow capacity upsizing needs may change.

3.7.5.3 Crossing Elimination Evaluation

There are three transmission pipelines crossing Mare Island Strait that provide service to Mare Island. These include two older pipelines: a 20-inch diameter south crossing pipeline (installed around 1941) and a 14-inch diameter north crossing pipeline (installation date unknown); and a newer 18-inch diameter north crossing pipeline which was installed in 2020.

The 20-inch diameter south crossing was identified during the field condition assessment as having moderate pipe wall thickness loss. Since these pipelines are assumed to be critical for reliable service to the Island, an analysis was performed to evaluate system performance impacts under a worst-case scenario in which both older crossings are eliminated in a near-term demand condition (i.e., only the new

18-inch diameter north crossing remains in service, with no near-term improvements implemented). Key findings from this evaluation include:

- Normal operations will not be impacted. Demands will be met while maintaining pressures consistent with near-term pressures.
- During the maximum day + fire flow scenario, available fire flow would be reduced. Some examples of this reduction include:
  - Fire flow availability at hydrant locations along Railroad Avenue and Walnut Avenue will drop by upwards of 1,000 gpm. The most significant reduction in fire flow at a hydrant in this area would be from 3,996 gpm to 2,942 gpm (a reduction of 1,054 gpm). In this case, the fire flow decreases significantly, but still meets the minimum flow requirement for single family residential land use in the area of 1,500 gpm.
  - For other hydrants in this area that serve commercial and mixed-use buildings, the minimum flow requirement is 4,000 gpm. Elimination of the two older crossings causes available fire flow to drop from 4,480 gpm to 3,702 gpm, which would no longer meet fire flow requirements.

The model was then used to evaluate conditions in two additional scenarios: 1) the Island supplied by two north crossings (i.e., the newer 18-inch diameter north crossing and the older 14-inch diameter north crossing); and 2) the Island supplied by a north crossing and the south crossing (i.e., the newer 18-inch diameter north crossing and the older 20-inch diameter south crossing).

For each scenario, a summary of the key findings is provided below, which is in comparison to conditions with only the 18-inch diameter north crossing serving the Island.

Scenario 1: Two North Crossings

With the two north crossings, there is some fire flow improvement in areas on the north side of the Island as expected (available fire flow along Azuar Drive increases by approximately 1,000 gpm).

- The most significant reduction in fire flow is at a hydrant near Railroad Avenue and Walnut Avenue which will drop by 1,054 gpm but still be able to meet the minimum single family residential fire flow requirement.
- There are ten (10) hydrants with a minimum commercial/mixed-use fire flow requirement of 4,000 gpm that will be deficient due to the reduction in available fire flow (reductions ranging from 84 gpm to 537 gpm).

Scenario 2: North Crossing + South Crossing

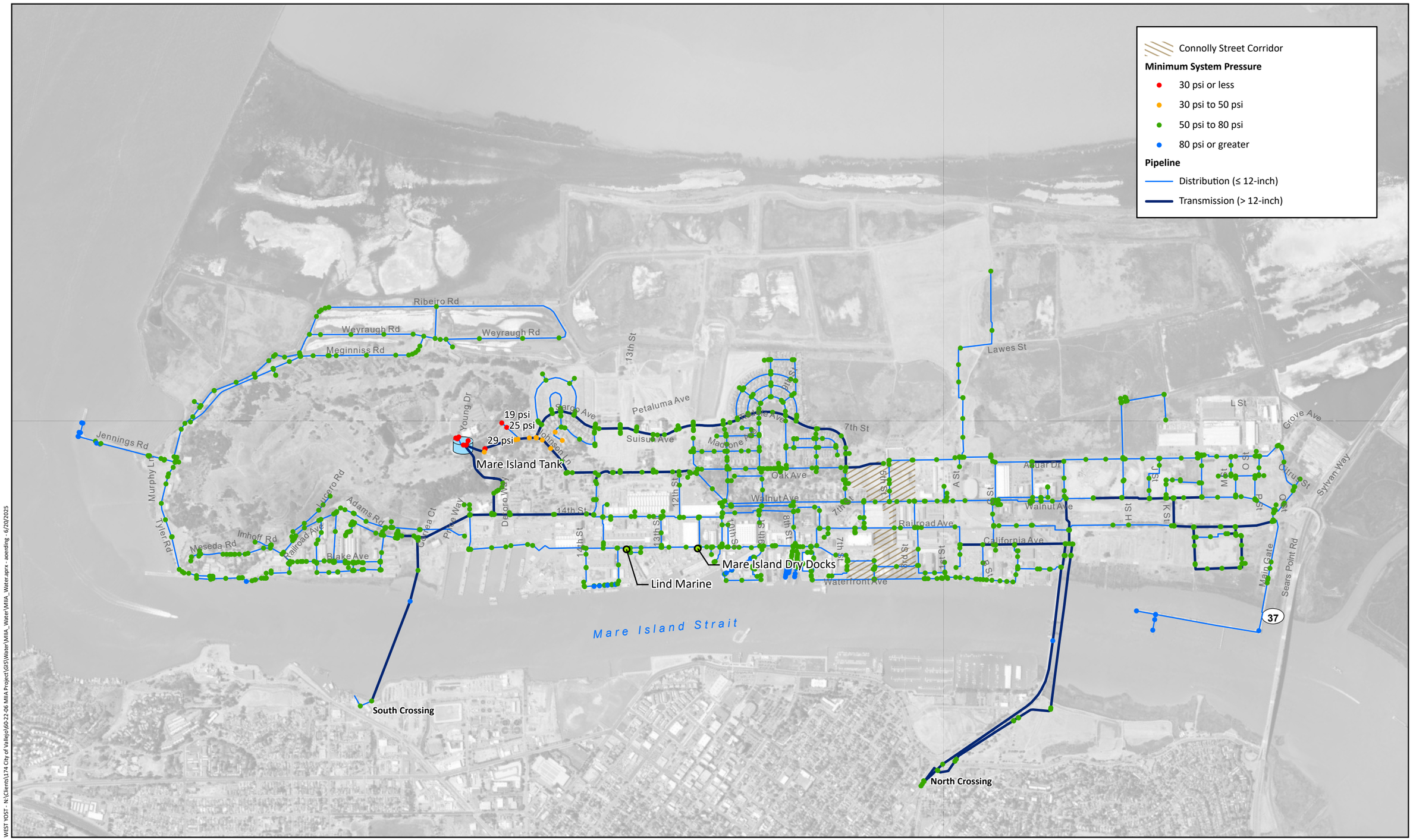
With the 18-inch diameter north crossing and the 20-inch diameter south crossing, results show that fire flow generally increases at most locations. Hydrants along Azuar Drive in the north side also increase by approximately 1,000 gpm as well as hydrants in the middle part of the Island.

- The most significant reduction in fire flow is 580 gpm (4,053 gpm to 3,473 gpm) along Kansas Street near the southwest corner of Connolly Street Corridor. The fire flow requirement of this hydrant is 3,000 gpm for high density residential so it will still be able to meet the fire flow requirement with the reduction.

- There is only one hydrant with a 4,000 gpm requirement that will be deficient due to the reduction in available fire flow (569 gpm). This hydrant is located just east of the Kansas Street hydrant mentioned above.

A North and South crossing combination (Scenario 2) provides improved available fire flow across the Island; additionally, it reduces the likelihood of both crossings being impacted in a localized damage event, which improves overall system reliability.

Because the crossings do not require upsizing for capacity (i.e., the distribution system is limiting the fire flow, not the crossing pipeline diameter), the south crossing will be proposed as a condition-based improvement due to wall thickness loss.



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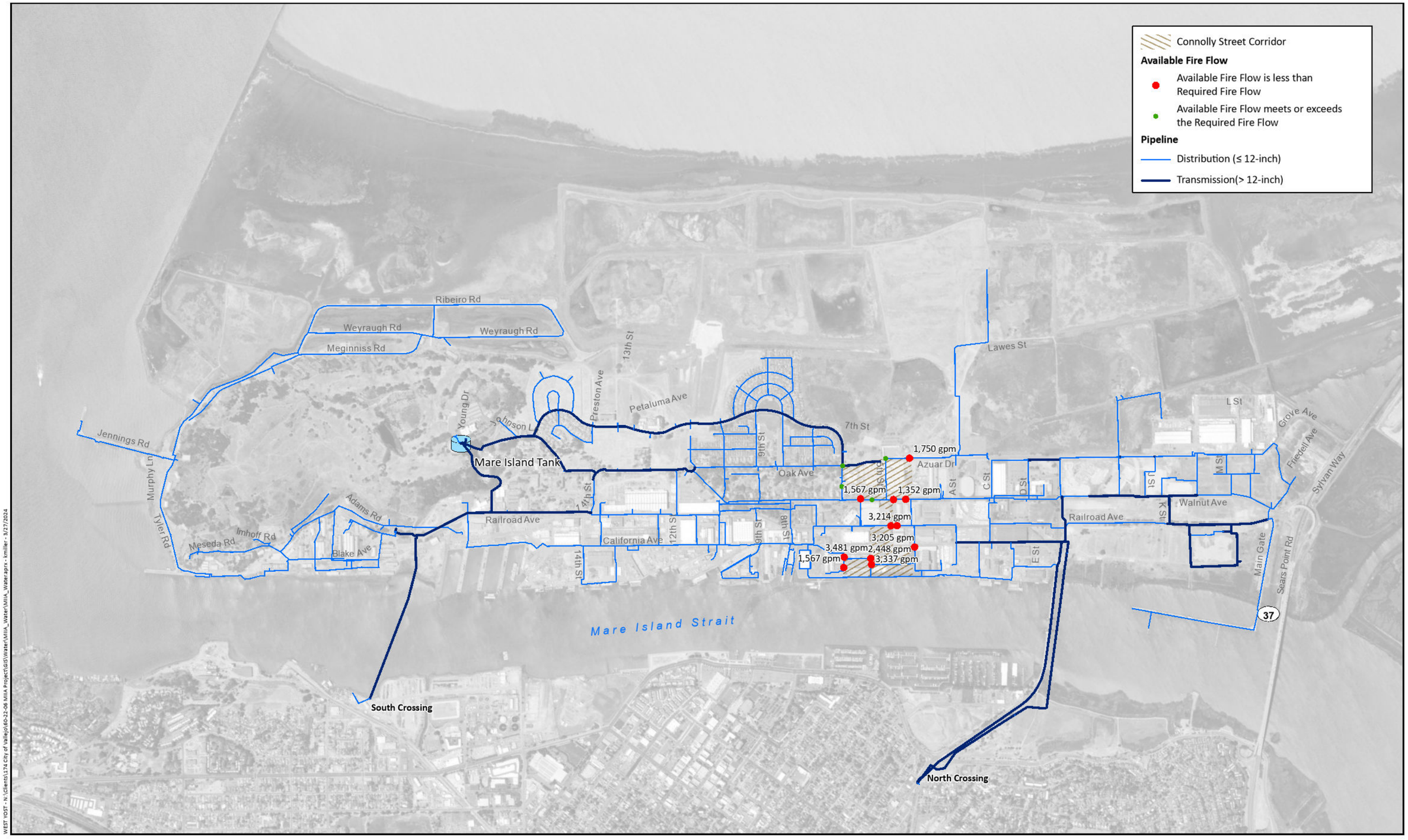
Mare Island
Infrastructure Assessment

City of VALLEJO California

Water System
Near-Term System
Average Day: Minimum System Pressure

Figure 3-11



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3. Results based on maintaining a minimum residual pressure of 25 psi at customer service connections and limiting maximum pipeline velocities to 10 ft/s.
4. Numbers in red represent the available fire flow at junctions that do not meet fire flow requirements.

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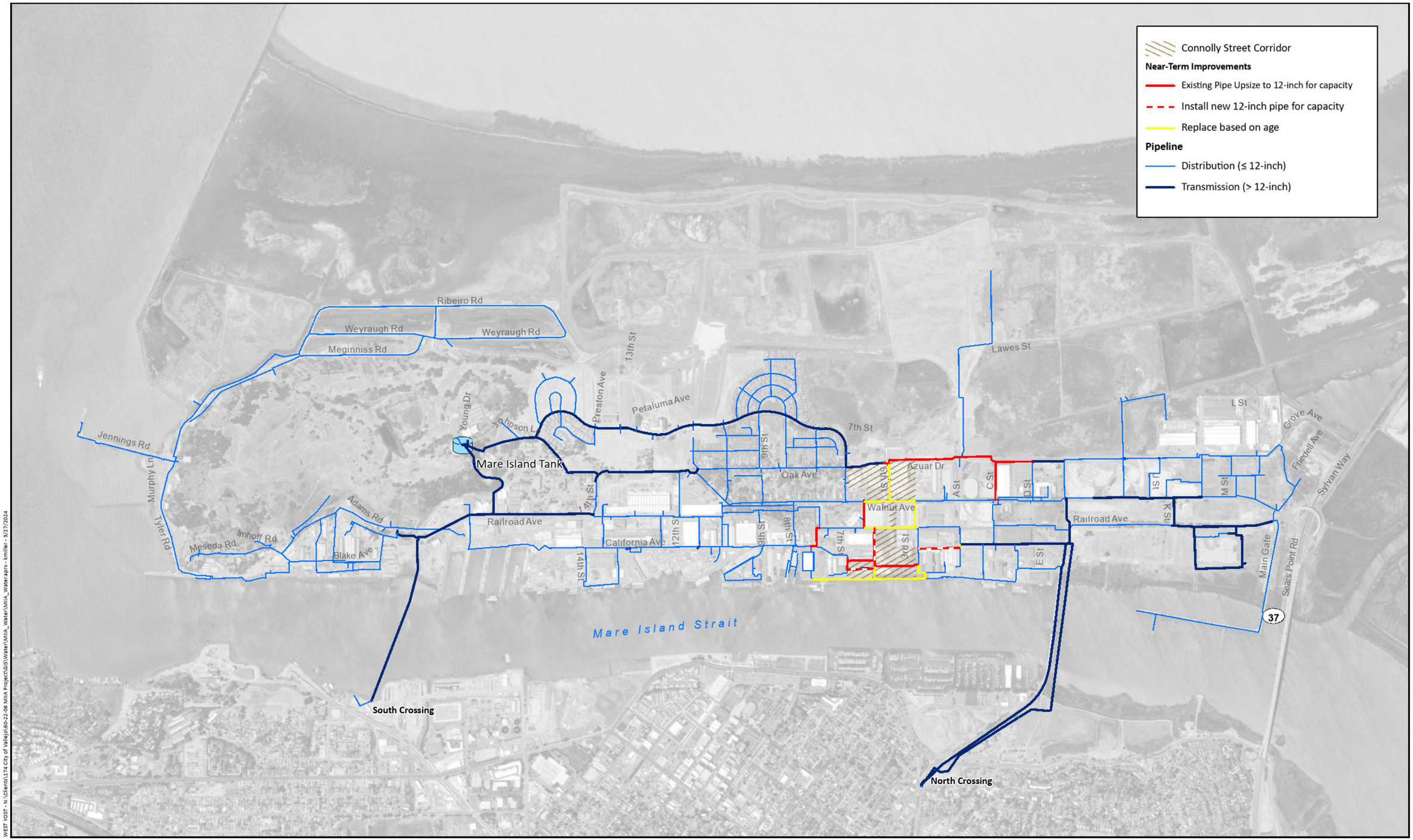
Mare Island
Infrastructure Assessment

Water System
Near-Term System
Max Day + Fire Flow: Available Fire Flow

Figure 3-12



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Infrastructure Assessment

Water System
Near-Term System
Hydraulic Capacity Pipeline Improvements

Figure 3-13

3.7.6 Build-Out System Assessment

The build-out system evaluates the system's ability to serve build-out development plans (year 2063) under each operating scenario. The build-out development plan is described in Chapter 2.

3.7.6.1 Normal Operations

The normal operations scenario evaluates the hydraulics of the system during a non-fire 24-hour simulation period under build-out average day demand conditions. Build-out average day demand is 1,300 gpm (1,873,146 gpd or 1.87 mgd; see Table 3-19).

Figure 3-14 shows the minimum pressure results from the 24-hour day simulation. Results are generally consistent with the existing system as locations with a minimum pressure less than 30 psi are located around Mare Island Tank. Pressures along the southside of the Island near Ribeiro Road drop drastically due to the increase in demand, higher elevations, and existing diameter restrictions on the single-feed pipeline.

3.7.6.2 Maximum Day Plus Fire Flow

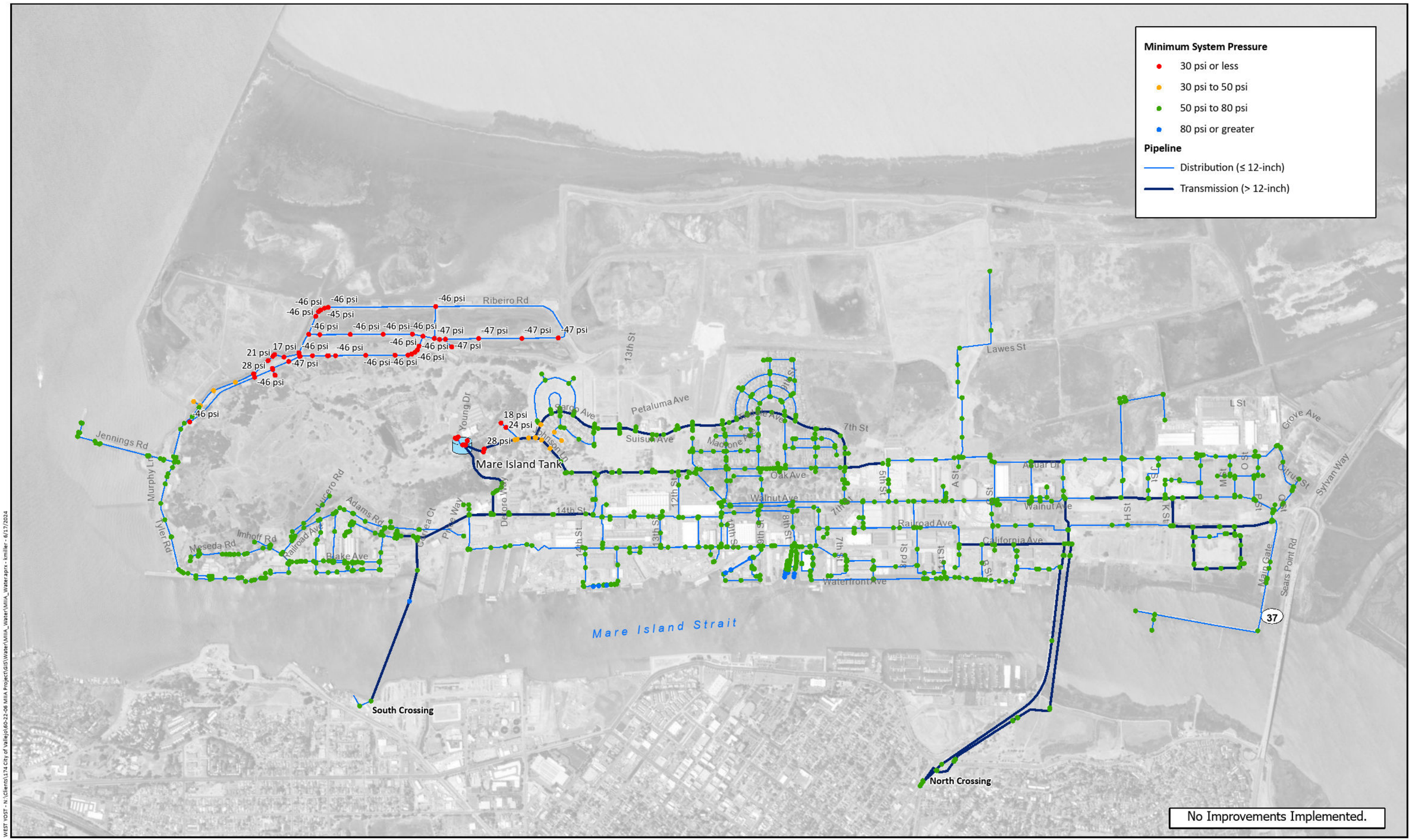
The maximum day demand plus fire flow scenario evaluated the build-out fire flow availability under a maximum day demand condition. The maximum day demand is calculated as 1.64 times average day demand, or 2,133 gpm (3,071,960 gpd or 3.07 mgd; see Table 3-19).

To evaluate the build-out fire flow availability, InfoWater Pro's *Available Fire Flow Analysis* tool was used to determine the available fire flow at a minimum residual pressure of 25 psi. For the build-out system fire flow analysis, hydrant locations were evaluated to determine the available fire flow that can be provided, in addition to meeting maximum day demand. A maximum allowable pipeline velocity of 10 ft/s was applied to all pipelines. Fire flow requirements were assigned to each hydrant based on proposed build-out land uses.

Figure 3-15 summarizes the available fire flow at each tested hydrant location while meeting the minimum residual pressure criterion of 25 psi and limiting maximum pipeline velocity to 10 ft/s. Results presented are representative of the system capacity and do not represent available flow from a specific hydrant as the losses through appurtenances are not included. Locations that meet fire flow requirements are shown in green while locations that do not meet fire flow requirements are shown in red. Deficient locations are where there are single feeds to an area or where there are small diameter pipelines that constrain flow. Hydrants located at the end of the single feed line on the south part of the Island have zero available flow due to low static pressure and a flow constraint at Ribeiro Road with an existing 2-inch diameter pipeline.

To meet fire flow requirements, it is recommended pipelines throughout the Island be upsized as shown on Figure 3-16. To support development along the southern side of the Island, a 12-inch diameter pipeline loop with a new tie-in connection on the west side is recommended. This will allow for the development in this area to be served from a looped system which improves the redundancy and reliability to serve this area. It is important to note that additional hydraulic evaluations are recommended for development in the preserve as it may require its own boosted pressure zone to meet pressure requirements due to the elevation.

Figure 3-17 shows the minimum pressure results from the 24-hour day simulation when the pipeline improvements are implemented. Note this assumes pipeline improvements recommended for the near-term scenario were implemented (grey pipes in Figure 3-17). While pressures around Mare Island Tank are still below 30 psi, pressures throughout the rest of the system are 50 psi or greater.



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Mare Island
Infrastructure Assessment

Water System
Buildout System
Average Day: Minimum System Pressure

Figure 3-14

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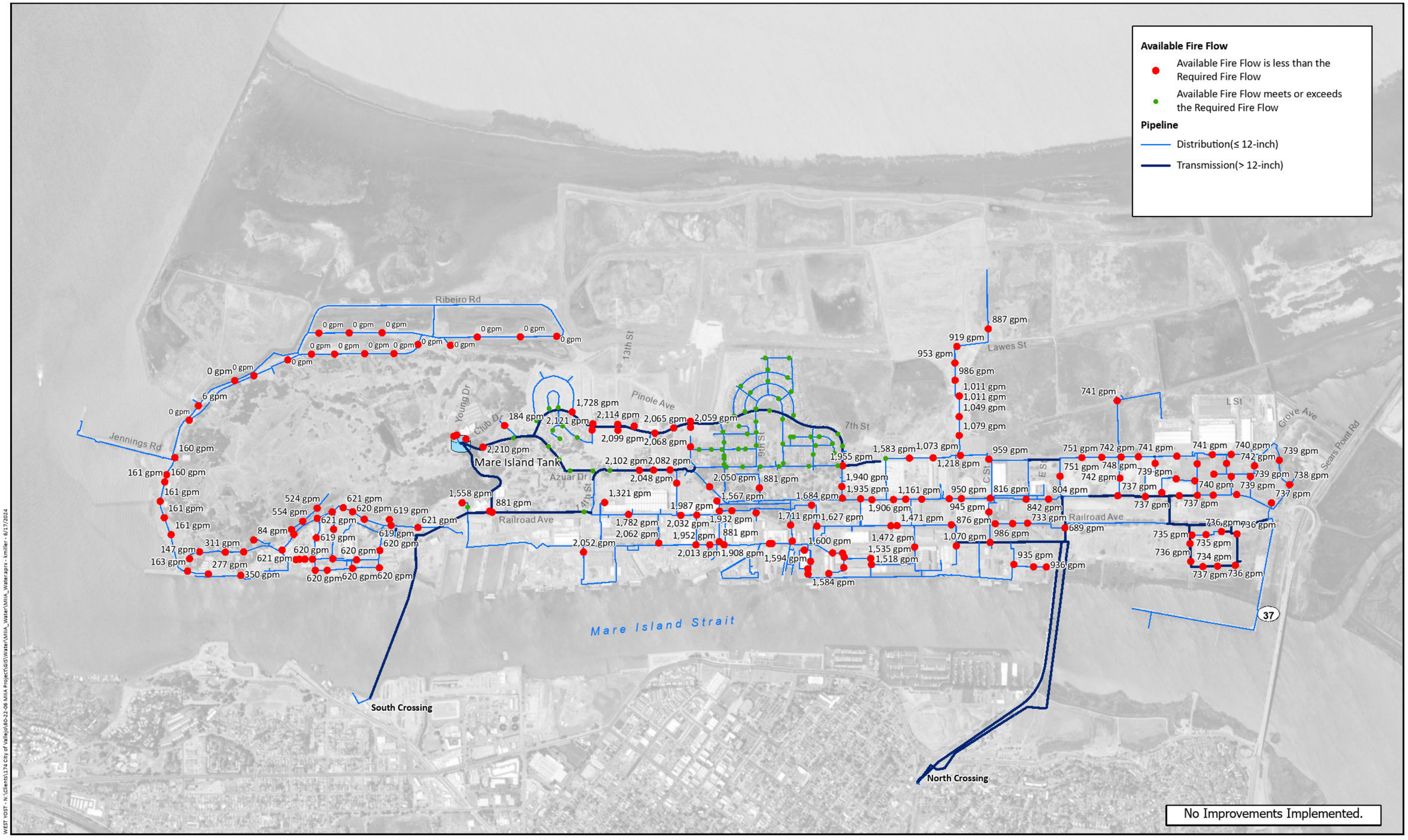

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Mare Island
Infrastructure Assessment

Water System
Buildout System
Max Day + Fire Flow: Available Fire Flow

Figure 3-15

Exact pipeline sizing and location for this area should be verified based on final development/land use.

Additional hydraulic evaluations are recommended based on final development/land use to confirm whether a new booster pump station and storage tank will be required to meet pressure

Buildout Improvements
- Upsize to 10-inch
- Upsize to 12-inch
- Upsize to 14-inch
- Upsize to 16-inch
- Upsize to 20-inch
- Install new 8-inch pipe
- Install new 12-inch pipe
- Install new 20-inch pipe

Existing/Near-Term Pipelines
- Distribution(≤ 12-inch)
- Transmission (> 12-inch)
- Near-Term Improvements

Ribeiro Rd
Weyraugh Rd
Weyraugh Rd
Meginnjss Rd
Jennings Rd
Murphy Ln
Tyler Rd
Meseda Rd
Imhoff Rd
Adams Rd
Blake Ave
Young Dr
Johnson Ln
Mare Island Tank
Railroad Ave
13th St
Petaluma Ave
14th St
California Ave
12th St
9th St
8th St
7th St
Oak Ave
7th St
Lawes St
5th St
Azuar Dr
Walnut Ave
3rd St
A St
C St
D St
E St
L St
K St
M St
Railroad Ave
Grove Ave
Linden St
Citrus St
Main Gate
Sears Point Rd
37
Mare Island Strait
South Crossing
North Crossing

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Mare Island
Infrastructure Assessment

Water System
Build-out System
Hydraulic Capacity Fire Flow Improvements

Figure 3-16

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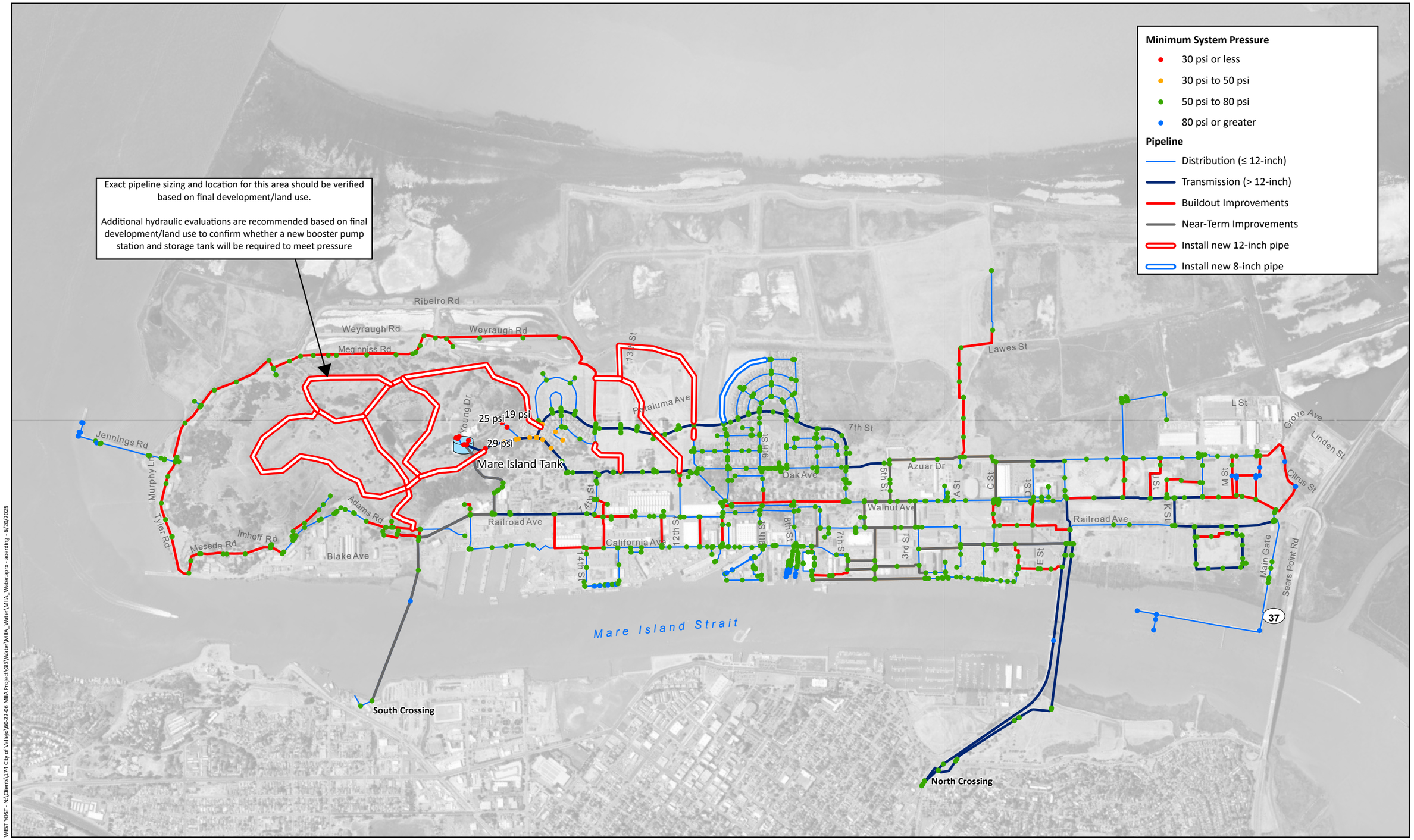

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Mare Island
Infrastructure Assessment

City of VALLEJO California

Water System Build-out System Minimum System Pressure with Pipeline Improvements Implemented

Figure 3-17

3.8 INTEGRATION OF CONDITION AND CAPACITY RECOMMENDATIONS

This section incorporates recommendations from age-based condition assessment, field testing, and hydraulic capacity evaluation into an integrated set of recommendations for each land use scenario (existing/near-term and build-out).

3.8.1 Existing/Near-Term

3.8.1.1 Pipelines

Recommendations for existing/near-term pipeline replacements are described below, summarized in Table 3-21, and presented on Figure 3-18:

- Pipelines that require upsizing for hydraulic capacity/fire flow. Pipelines are shown red on Figure 3-18.
- Pipelines which were identified to have significant wall thickness loss that are either critical to the water system network or serve the Connolly Street Corridor development. Pipelines are shown in orange on Figure 3-18 and include:
  - The 20-24-inch diameter CIP south crossing, from Mare Island Tank to its termination on the mainland at Lemon Street. This is one of three existing water crossings to the Island, and the only crossing on the south end of the Island. The pipeline was tested at Azuar Drive near Price Way and had an estimated wall loss of 18 percent. Portions of this pipe near the tank are located in private rights-of-way; it is recommended it be relocated in public/City rights-of-way for future access.
  - The 14-inch diameter CIP pipeline on California Avenue, from the Mare Island Causeway to A Street. This pipeline was tested at two locations and had estimated wall loss of 15 percent and 22 percent. Note that this is the same 14-inch diameter pipeline that crosses Mare Island Strait and thus it can be assumed other parts of this pipeline may be in poor condition and should not be relied upon for continued reliable service.
  - The 12-inch diameter CIP pipeline on 2nd Street between Railroad Avenue and 4th Street. This pipeline had an estimated wall loss of 13 percent.
- Pipelines that are adjacent to the Connolly Street Corridor, beyond their UL, and will improve the performance of the system in the area if replaced. Pipelines are shown in yellow on Figure 3-18 (approximately one (1) mile of pipeline).
  - It should be noted that the age-based assessment indicated approximately 69 percent of the system (24.6 miles) is expected to require replacement by 2027. There are several reasons why only a portion is included for replacement in these final recommendations. The final infrastructure plan for areas of the Island outside of the Connolly Street Corridor are not complete and may change in the future; it is expected that underground utilities, including the water system, will be replaced as development occurs. Therefore, the pipelines within the known development areas (Connolly Street Corridor) were prioritized for replacement.
- The City should continue to replace and repair pipelines that fail due to age (leaks, breaks) and prioritize replacements of pipelines with repeat failures or areas of known issues.

3.8.1.2 Hydrants, Valves, and Meters

Recommendations for existing/near-term hydrant, meter, and valves replacements are summarized in Table 3-21 and include:

- Hydrants and valves should be replaced with adjacent/connecting pipelines recommended in Figure 3-18. This includes approximately 72 valves and 23 hydrants based on existing mapping. The City has indicated that more valves are required in the Mare Island system so the final number of valves may increase.
- Existing hydrants should be reviewed to confirm they meet the City of Vallejo and Fire Marshal specifications including location and maximum spacing.
- Meters are expected to be installed as part of the City's AMI project. New meter location and count will be based on final development.
- All new infrastructure should meet City of Vallejo Standard Specifications.

Table 3-21. Existing/Near-Term Integrated Pipeline, Valve, and Hydrant Improvement Recommendations

Pipeline Replacement Type[a]	Length of Pipelines, feet	Count of Existing Valves	Count of Existing Hydrants
Capacity Improvements			
Upsize to 12-inch	7,356	33	12
Upsize to 16-inch	20	1	1
Replace based on Wall Thickness Loss			
Replace Existing 12-inch	632	2	1
Replace Existing 14-inch	1,745	5	1
Replace Existing 20-inch	4,205	7	2
Replace Existing 24-inch	1,624	1	0
Replace based on Age/Improved Performance			
Replace Existing 6-inch	439	1	0
Replace Existing 8-inch	889	5	0
Replace Existing 10-inch	1,903	6	0
Replace Existing 12-inch	2,013	11	6
Total	**20,826**	**72**	**23**

(a) Pipelines should be replaced with the size (diameter) specified, or the next largest standard size available.

3.8.1.3 Water Tank

Recommendations for the Mare Island water tank within the existing/near-term scenario include:

Table 3-22. Existing/Near-Term Water Tank Recommendations

Tank Feature	Structure	Condition to Address	Recommendations
Exterior	Walls	• Spalled concrete and staining along roof interface	• Investigate stains for possible delamination and corrosion of reinforcement; repair spalls as required in 0 to 2 years
Interior	Inlet Pipe	• Moderate corrosion on flap valve	• Replace flap valve and pipe couplings in 2 to 5 years • Recoat/touch-up pipe coating in 2 to 5 years • Replace fittings with stainless steel fittings in 2 to 5 years. Electrically bond the fittings to the pipes and install magnesium anodes to protect the immersed portions of the pipes and fittings from corrosion
	Overflow Pipe	• Impact of sloshing to roof under seismic loads	• Move the overflow pipe down to 7-feet below the roof soffit, within one (1) year
		• Minor corrosion on flange • Sections of coating failure and minor corrosion on pipe and support brackets	• Replace flange in 2 to 5 years • Recoat/touch-up pipe coating in 2 to 5 years • Replace fittings with stainless steel fittings in 2 to 5 years. Electrically bond the fittings to the pipes and install magnesium anodes to protect the immersed portions of the pipes and fittings from corrosion

3.8.2 Build-Out

The build-out scenario recommendations are highly dependent upon final development land use and timing. If there are any significant changes to land use, land use distribution, or development timing, project-level evaluations should be performed to confirm capacity upsizing needs. Transmission pipelines have been included for new development areas, but final distribution piping system will depend on development layout and is not included.

As presented in this chapter, much of the existing water distribution system is old and likely in need of full replacement. Although build-out recommendations are expected to be implemented as development occurs, the existing system is likely to continue experiencing leaks and breaks until it is replaced.

3.8.2.1 Pipelines

Recommendations for build-out pipeline replacements are described below, summarized in Table 3-23, and presented on Figure 3-19:

- Pipelines required for hydraulic capacity/fire flow. This includes upsizing of existing pipelines and installation of new pipelines, as shown on Figure 3-19. This assumes pipelines recommended for capacity improvements in the near-term scenario were constructed (pipelines are shown in grey on Figure 3-19).

- Pipelines beyond their UL that were not replaced as part of the existing/near-term scenario. Pipelines are shown in yellow on Figure 3-19. It is assumed that pipelines will be replaced as development plans are finalized and implemented. Otherwise, pipelines should be prioritized by:
  - Pipelines with the most severe leak/break history.
  - Pipelines that are critical to the system and provide looping redundancy, typically transmission mains.

3.8.2.2 Hydrants, Valves, and Meters

Recommendations for build-out hydrant, meter, and valves replacements are summarized in Table 3-23 and include:

- Hydrants and valves should be replaced with adjacent/connecting pipelines recommended on Figure 3-19. Valve count and location will depend on final development plans, pipeline configuration, and maximum spacing requirements.
  - Hydrant counts in Table 3-23 were estimated using an average spacing of 750 feet, which is an average of the City's recommended fire hydrant spacing of a maximum of 500 feet on streets or 1,000 feet on parkways.
  - Valve counts in Table 3-23 were estimated based on an average valve spacing of 500 feet.
- Meters are expected to be installed as part of the City's AMI project around 2024. Meters have an estimated UL of approximately 20 years, so all meters are expected to require replacement at least once within the build-out scenario. New meter location and count will be based on final development.
- All new infrastructure should meet City of Vallejo Standard Specifications.

3.8.2.3 Water Tank

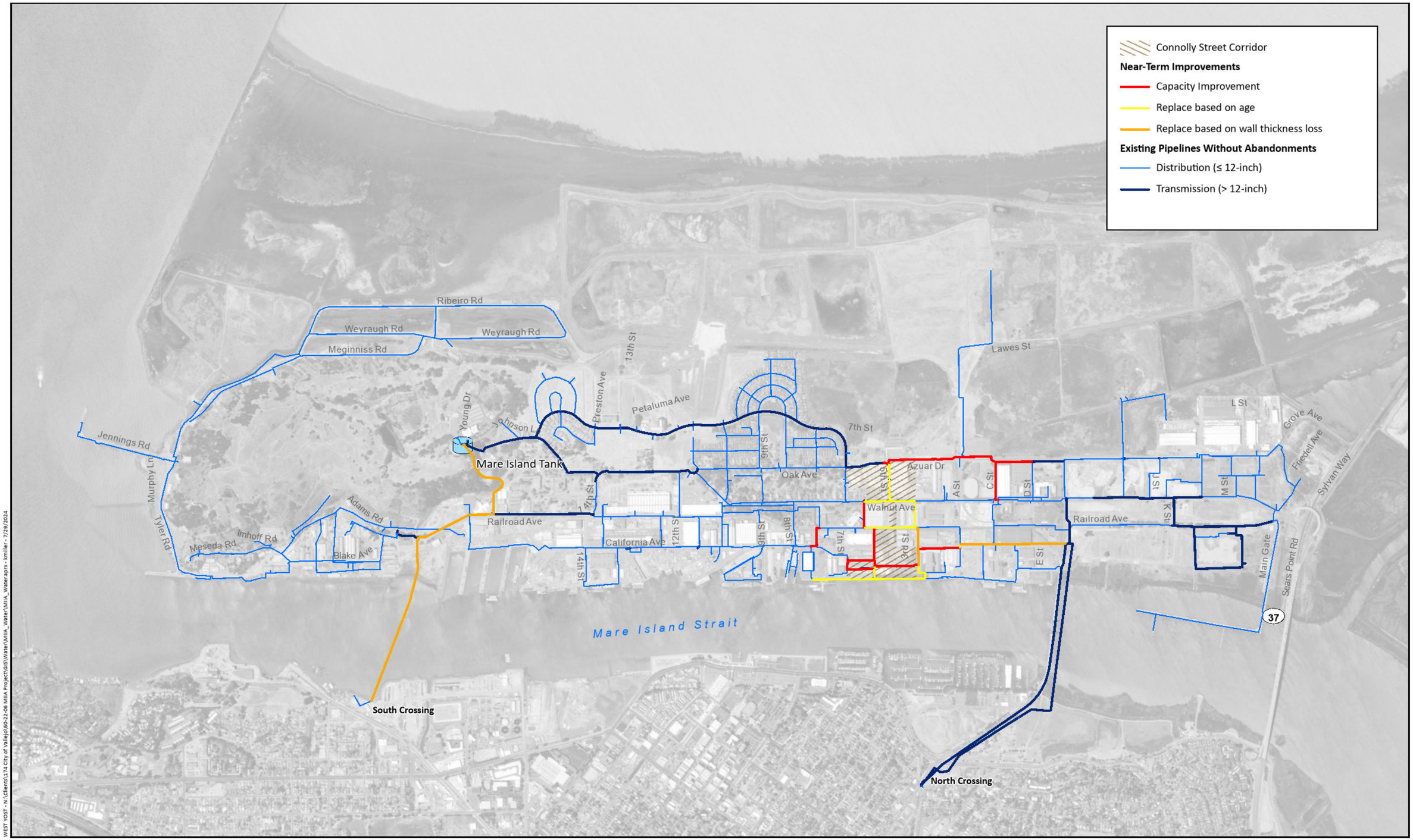
Recommendations for the Mare Island water tank within the build-out scenario include:

- Perform an updated condition assessment on the tank and vault every 10 years (2034, 2044, 2054).
- Perform recommended improvements based on assessment.

Table 3-23. Build-Out Integrated Pipeline, Valve, and Hydrant Improvement Recommendations

Pipeline Replacement Type[a]	Length of Pipelines, feet	Count of Valves	Count of Hydrants
Capacity Improvements			
Upsize to 10-inch	2,711	5	4
Upsize to 12-inch	35,019	70	47
Upsize to 14-inch	624	1	1
Upsize to 16-inch	1,726	3	2
Upsize to 20-inch	963	2	1
New Pipelines			
New 8-inch	1,392	3	2
New 12-inch	23,317	47	31
Replace based on Age (Beyond Useful Life)			
Replace Existing 3-inch[b]	284	1	0
Replace Existing 4-inch[b]	6,032	12	8
Replace Existing 6-inch	8,708	17	12
Replace Existing 8-inch	13,533	27	18
Replace Existing 10-inch	5,995	12	8
Replace Existing 12-inch	17,012	34	23
Replace Existing 14-inch	4,549	9	6
Replace Existing 16-inch	496	1	1
Replace Existing 18-inch	535	1	1
Replace Existing 20-inch	4,040	8	5
Total	**126,936**	**254**	**169**

(a) Pipelines should be replaced with the size (diameter) specified, or the next largest standard size available.
(b) Pipelines smaller than 6-inch diameter should be replaced with 6-inch diameter or greater to meet City minimum pipeline diameter standards.



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Prepared by:

WEST YOST

0 1,500 Feet

Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Only Mare Island water system facilities are displayed for clarity.

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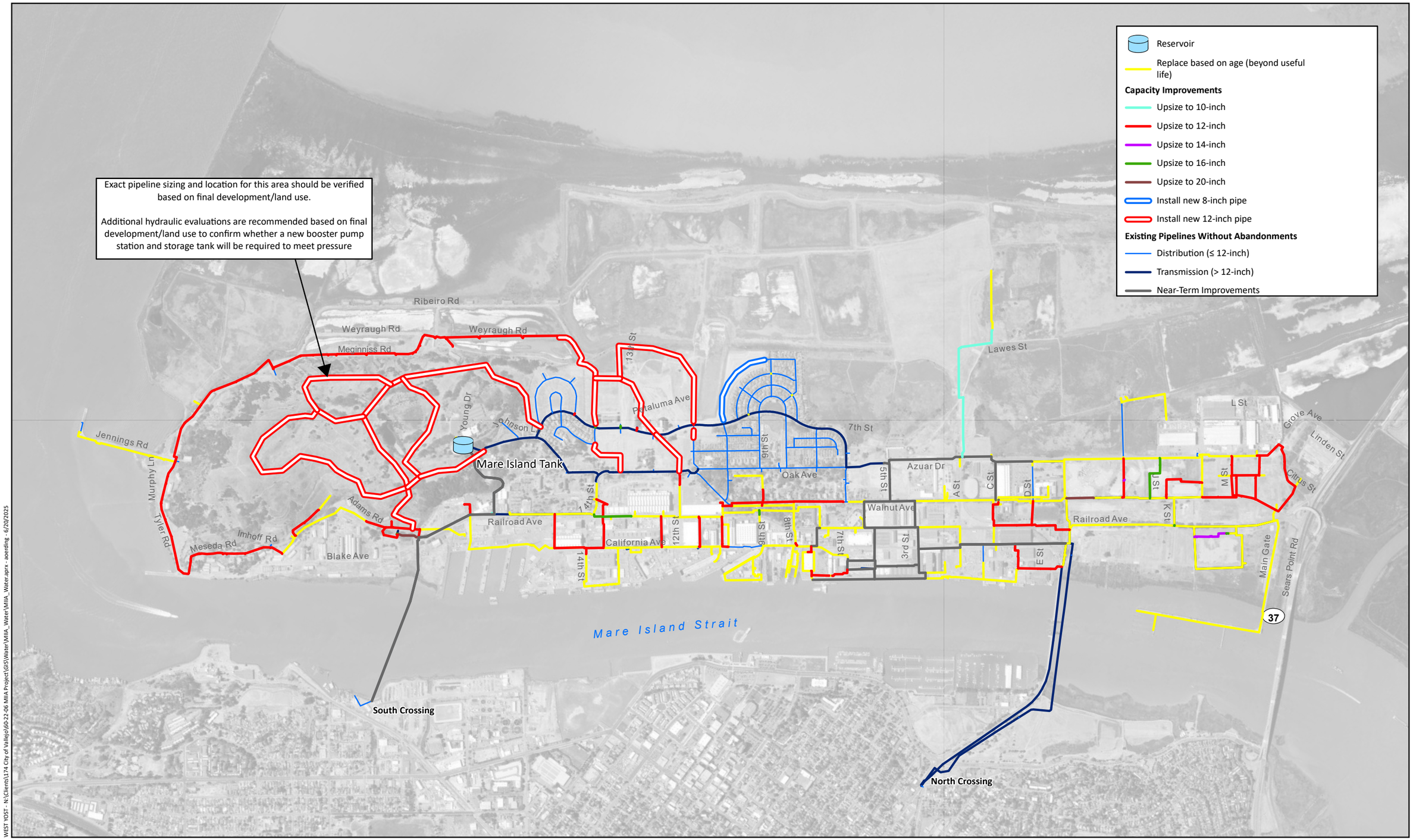
Mare Island
Infrastructure Assessment

Water System
Existing/Near-Term System
Integrated Final Recommendations

Figure 3-18



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Prepared by:

WEST YOST

0 1,500 Feet

Notes:
1. Drawing is based on City-wide water system geodatabase provided by the City of Vallejo on February 6, 2023 and updated by West Yost in July 2023.
2. Only Mare Island water system facilities are displayed for clarity.

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Mare Island
Infrastructure Assessment

City of VALLEJO California

Water System
Build-Out System
Integrated Final Recommendations

Figure 3-19

CHAPTER 4
Storm Drainage and Flood Control System

4.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

The tasks completed for the storm drainage and flood control system evaluation included:

- Collecting and reviewing existing documents and mapping
- Developing proposed design criteria and preferences to be used in development of infrastructure improvements.
- Developing a hydraulic/hydrodynamic model to evaluate the capacity needs of the proposed storm drainage and flood control system infrastructure improvements.
- Performing condition assessment of storm drainage and flood control system assets, including Lift Station 15 and select maintenance holes and drain inlets.
- Developing an integrated set of recommendations for the existing/near-term and build-out development scenarios.

A summary of the findings and conclusions for the storm drainage and flood control system evaluations are listed below.

- Key documents and maps used in the stormwater and flood control evaluation included:
  - Navy Quad maps, which show the storm drain system.
  - The FEMA Flood Insurance Rate Maps, which were used to establish the current Mare Island Strait Base Flood Tide Elevation of 10.0 feet NAVD88.
  - State of California Sea-Level Rise Guidance (2018), which was used to establish/verify the Mare Island Strait design tide of 13.5 feet NAVD88 for the year 2070 and the year 2100 design tide elevation of 17.0 feet NAVD 88. As of April 2024, the next edition of this guidance has been published but not yet adopted by the State.
  - Topographic mapping, which was used to delineate Mare Island into elevation zones based on the various sea level rise (SLR) elevations.
  - The Vallejo Flood and Wastewater District Engineering Standards (July 2020), which were used to evaluate existing infrastructure adequacy and to size the proposed future storm drain infrastructure.
  - The Mare Island Preliminary Storm Drain Report – Draft (BKF, May 2022), from which it was concluded that most of the existing storm infrastructure does not currently meet the District Engineering Standards and will not meet the Standards with SLR.
- Storm drainage and flood protection service for Mare Island are currently provided through a network of gravity pipelines which convey storm water generally from the west/central part of the island (inland), to the east, discharging via storm drain outlets to the Mare Island Strait.
- The District's Engineering Standards along with the State of California Sea-Level Rise Guidance are appropriate and were used for sizing the storm drainage and flood control infrastructure for Mare Island, including: (1) storm drains are sized for the 15-year, storm peak flow, (2) pump stations are sized for the 100-year storm peak flow, and (3) buildings are protected for the 100-year, design storm with SLR (fill or sea walls). See Chapter 6 for

additional information on the existing sea wall. The infrastructure needed for SLR does not need to be constructed immediately, but current and future infrastructure planning and designs do need to address and accommodate the required, future SLR infrastructure.

- Because the storm drain and flood control infrastructure is not adequate for future conditions, it was concluded that nearly the entire existing storm drainage system would be replaced, and therefore limited condition assessment of the existing infrastructure was performed.
- The condition assessment that was performed included:
  - Inspection of 23 storm drain maintenance holes and drain inlets, which indicated that 16 out of 23 would require replacement to meet current standards.
    - Assessment of storm water Lift Station 15 (LS-15). LS-15 is in moderately to severely compromised condition and is unreliable. It is currently understood that the station's primary use is to serve the Highway 37 interchange. Some surrounding areas of North Mare Island contribute runoff to LS-15, which would need to be rerouted to gravity drain to avoid potential ponding and vector control. Also, LS-15 will not be needed in the future to serve Mare Island. It is recommended that coordination with Caltrans be initiated to advise them that: 1) LS-15 is moderately to severely compromised and not reliable. 2) The primary purpose of LS-15 to serve the Highway 37 interchange. LS-15 is not required to serve Mare Island in the future; and 3) Caltrans should construct a new lift station located in the Caltrans right-of-way to serve the Highway 37 interchange. In the meantime, minimize improvements or upgrades to the facility.
- In order to demonstrate a tangible infrastructure solution for the island, the design criteria were applied in a prescriptive approach using 0.1-foot accuracy, which was carried through in development of the proposed infrastructure solutions.
  - Mare Island was divided into 12 stormwater Improvement Areas based on the preferred approach to providing storm drainage service and flood protection.
  - For each area, trunk storm drains were delineated and sized. For areas that have no existing buildings, flood control was recommended based on filling the area to have a ground surface at or above elevation 18.0 feet. For areas with existing buildings that are to remain in the future (precluding the option to fill the area), use of a floodwall or levee was recommended to achieve flood protection for SLR to elevation 17.0 feet (to the year 2100).

4.2 PURPOSE

The purpose of the MIIA storm drainage and flood control evaluation was to assess the condition and capacity of the existing infrastructure and identify the needed future infrastructure. This chapter describes the infrastructure assessment that was performed of the Mare Island storm drainage and flood control system. The assessment included several steps, as listed below and further described in the following sections:

- Existing Data – collection and review of existing data sources.
- Existing System Description – description of the existing storm drainage and flood control system configuration and components.

- Design Criteria and Design Preferences – establishes criteria to be used in development of the proposed storm drainage and flood control systems.
- Condition Assessment – field condition assessment of storm drain inlets, storm drain maintenance holes, and the stormwater lift station.
- Capacity Assessment and Recommended Improvements – evaluation of the island using a hydrologic and hydraulic (H&H) drainage model to develop recommended storm water and SLR infrastructure solutions.
- Recommendations – integration of condition and capacity recommendations in terms of serving the two land-use scenarios (existing/near-term and build-out).

4.3 EXISTING DATA

Table 4-1 summarizes the data received from the Client Team to support the assessment. The existing data was used to the maximum extent possible to minimize duplication of previous work efforts.

Table 4-1. Existing Data Summary

Data Source	Details
Stormwater System Geographic Information System (GIS) – Mare Island "MI" Layers, provided March 2023 from District	Included the Mare Island stormwater system. Relevant feature classes and attributes included: • Stormwater pipes • Stormwater nodes (maintenance holes and drain inlets)
Stormwater System GIS – "VFWD District-wide" Stormwater System, provided March 2023 from District	Included the entire District-wide stormwater system, and newer developed areas of Mare Island only. Relevant feature classes and attributes included: • Stormwater pipes (attributes: pipe ID, upstream and downstream node, length, diameter, material, installation date) • Stormwater nodes (attributes: node ID, type (e.g., maintenance hole, catch basin), rim elevation, depth, installation date)
Stormwater system improvements from City of Vallejo's utility as-builts	• Mare Island Sewer and Water Main Replacement Project (2018) • Coal Sheds – Phase 2 (2017) • Walnut Avenue and "E" Street (2007) • Railroad Avenue Phase 2 (2006) • Farragut Village – Unit No. 3 (2005) • Farragut Village – Unit No. 4 (2005) • Club Drive (2005) • Coral Sea South Village – Unit No. 1 (2005) • Flagship Drive – Phase 1B & 1C (2005) • Kansas Street (2004) • Azuar Drive from Connolly Street to Residential Parkway (2004)
Navy Storm Quad Maps	• Shows alignment, material, and diameter of stormwater system pipelines within Mare Island (December 1976)

Table 4-1. Existing Data Summary

Data Source	Details
Other City or District Documents	• Mare Island Preliminary Storm Drain Report – Draft (BKF, May 2022) • Mare Island [Pump Station] Condition and Rehab Pricing report (2022, VFWD) • VFWD Engineering Standards (July 2020) • CCTV inspections from four outfalls (December 2021) • Chapter 7.98 – Flood Management Regulations, City of Vallejo Municipal Code (Supp. 6-22) • Northern Waterfront Sea Level Rise Requirements – Pump Stations (VFWD, January 2020) • Mariners Cove Preliminary Drainage Report (RJA, 2019) • Flood Design Considerations for Sea Level Rise at Mariners Cove memo (RJA and S&W, November 2019) • Northern Waterfront Sea Level Rise Review (City, October 2019)
Other Reference Documents	• San Francisco Bay Tidal Datums and Extreme Tides Study (February 2016, AECOM) • State of California Sea Level Rise Guidance (2018 and 2024 Updates) • FEMA Flood Insurance Study and Flood Insurance Rate Maps for MI (Attachment 4-1) • Sea Level Rise and Flood Control solutions implemented by Other Bay Area Cities and Districts (City of Burlingame, Oro Lomo Sanitary District, Foster City, Port of San Francisco)

4.4 EXISTING SYSTEM DESCRIPTION

Figure 4-1 presents Mare Island's existing storm drain and flood control system, including pipelines, maintenance holes, drain inlets, outfalls, one pump station, and a sea wall.

The existing system provides storm drainage and flood protection service to Mare Island through a network of gravity pipelines which convey storm water generally from the inland, or west/central part of the island, to the east, discharging via storm drain outfalls to the Mare Island Strait.

The District currently operates and maintains the stormwater system infrastructure.

The existing piped storm drainage system consists of approximately 870 maintenance holes and 34.6 miles of gravity pipelines, with diameters ranging from 5- to 72-inches. Figure 4-1 shows the storm drain mapping to the extent that GIS mapping files were available. As shown in Table 4-1, two GIS mapping files were received, and pdf copies of the Navy Quad maps were received. An effort was made to consolidate the various mapping sources to develop a comprehensive GIS map for the existing storm drain system. The "MI" GIS layers covered more of the island, but were older than the "VFWD District-wide" GIS. For example, in the newer Farragut and Coral Sea development areas, the "VFWD District-wide" GIS had been

populated and therefore superseded the "MI" GIS files. Based on the available GIS mapping, pipe materials include corrugated metal pipe (12 percent of pipes), reinforced concrete pipe (11 percent of pipes), and vitrified clay pipes (9 percent of pipes). The remaining pipe materials are not documented in the GIS mapping.

The majority of the existing pipes do not meet current District engineering standards for pipe coverage, slope, or minimum pipe size. A recent District study, Mare Island Preliminary Storm Drain Report (BKF, 2022) concluded that the existing infrastructure mostly lacked adequate capacity and recommended nearly full replacement. The Mare Island Preliminary Storm Drain Report also did not address sea level rise, which further necessitates upgrades to the storm drain and flood control infrastructure. For these reasons, detailed evaluation and condition assessment of the existing storm drain and flood control system was not performed.

The existing storm drain system includes one pump station, called Lift Station 15 (LS-15). The District currently operates and maintains LS-15. LS-15 is located in North Mare Island on Independence Street (see Figure 4-1), adjacent to the Highway 37 eastbound off-ramp. LS-15 currently provides drainage and flood protection to the Highway 37 interchange, surrounding low-lying areas of North Mare Island, and portions of the Alco and EPS properties. The other building and structures in areas of North Mare Island previously served by LS-15 have been demolished or decommissioned.

A sea wall exists on much of the eastern edge of the island bordering Mare Island Strait (see Figure 4-1). The purpose of the sea wall is to separate land and water areas, limiting damage due to wave action and storm surge, while also providing locations for ships to berth. The sea wall was originally built by the Navy and has been rehabilitated or repaired several times since its original construction (the oldest repair as-built drawings date back to 1922). Thus, the original sea wall is older than 1922. See Chapter 6 for more information on the sea wall.



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Prepared by:

WEST YOST

0 1,800 Feet

Notes:
1. VFWD sewer system geodatabase provided by VFWD in 2023. Modified based on data available in Navy Quad maps, and record drawings.
2. Only Mare Island sewer system facilities are displayed for clarity.

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Infrastructure Assessment

City of VALLEJO California

Existing Storm Drain and Flood Control System

Figure 4-1

4.5 DESIGN CRITERIA AND DESIGN PREFERENCES

Existing planning and design criteria documents were reviewed with the purpose of establishing criteria to be used in development of the proposed storm drainage and flood control systems for Mare Island. As presented in Table 4-1, key documents included:

- VFWD Engineering Standards (July 2020)
- City of Vallejo Municipal Code, Chapter 7.98 – Flood Management Regulations (Supp. 6-22)
- Various memos related to sea level rise from the District and City

In addition, the following local or national documents/data sources were reviewed:

- San Francisco Bay Tidal Datums and Extreme Tides Study (February 2016, AECOM)
- State of California Sea Level Rise Guidance (2018 and 2024 Draft[1] Updates)
- FEMA Flood Insurance Study and Flood Insurance Rate Maps for Mare Island
- Sea Level Rise and Flood Control solutions implemented by Other Bay Area Cities and Districts (City of Burlingame, Oro Lomo Sanitary District, Foster City, Port of San Francisco)

These documents were reviewed, and with input from the Client Team, the storm drainage and flood control design criteria for use in the MIIA Project were established, including criteria related to SLR.

The following sections of this report utilize the established design criteria to develop proposed infrastructure improvements for the Mare Island storm drainage and flood control systems. In order to demonstrate a tangible infrastructure solution for the island, the design criteria were applied in a prescriptive approach using 0.1-foot accuracy, which is carried through in development of the proposed infrastructure solutions.

This report acknowledges that the design criteria used to develop the infrastructure solutions are based on guidelines that are regularly changing (e.g., the State of California Sea Level Rise Guidance document and the Solano County Water Agency Hydrology Manual, upon which the VFWD rainfall design storms are based). As the scientific understanding of the impacts of climate change are further refined in the future, it is expected that international, federal, State, and local agencies will continue to provide updates on climate and sea level rise trends, which will in turn be used to update guidance documents (including freeboard requirements and design storm rainfall depths, intensities). Development of future solutions to address storm drainage and sea level rise on Mare Island should be based on use of the most current and best available science at the time any specific project is designed and constructed. While this report emphasizes capital improvements, solutions should be integrated with site-specific and community-level adaptation planning, regional and cross-jurisdictional collaboration, and consider the site-specific land uses and unique characteristics of Mare Island. Also, at the time any specific project is designed and constructed, the design should include vulnerability assessments and adaptation strategies based on the proposed land use.

[1] The State of California Sea Level Rise Guidance: 2024 Update document draft was released during preparation of this report. Therefore, the 2018 version was used for development of infrastructure solutions.

4.5.1 Sea Level Rise Design Criteria

The SLR design criteria establish the Mare Island Strait design water surface elevations (WSE) for the years 2024, 2070, and 2100 are summarized below. All elevations are in the NAVD 88 vertical datum.

- The year 2024 Base Flood Elevation (BFE) is 10.0 feet. The Federal Emergency Management Agency (FEMA) Base Flood Elevation on Mare Island Strait is 10 feet based on the current FEMA Flood Insurance Rate Maps. This BFE is based on a still water WSE of 9.9 feet (includes tide, storm surge, and riverine flow, but not waves). Mare Island Strait has minimal wave hazard, so the FEMA BFE was established at 10 feet.
- The year 2070 and 2100 Water Surface Elevations (WSEs) are based on the *State of California Sea Level Rise Guidance* (SLRG), 2018 Update, which projects SLR to and beyond the year 2100 for various conditions, including:
  - Low and high greenhouse gas (GHG) emissions scenarios. Currently and through 2050, emissions are forecast based only on the high emissions scenario. Beyond 2050, the low and high emission forecasts are provided. Higher GHG emissions result in larger depths of SLR.
  - Low, medium-high, and extreme risk aversion requirements. These risk aversion projections are shown on Figure 4-2. The low risk aversion forecast is for facilities such as coastal trails that can easily be relocated or can withstand some flooding. Medium-High risk aversion is for facilities such as coastal housing. Extreme risk aversion is for critical facilities such as coastal power plants.
  - Based on the SLRG (Figure 4-2), the recommended year 2070 SLR WSE is 13.5 feet (3.5 feet of increase[2]) to be consistent with the SLRG recommendation for High Emissions and Medium-High Risk Aversion (coastal housing). This WSE is also consistent with the District design criteria SLR WSE for 2070.
  - Based on the SLRG (Figure 4-2), the recommended year 2100 SLR WSE is 17.0 feet (7.0 feet of increase[3]) to be consistent with the SLRG recommendation for High Emissions and Medium-High Risk Aversion (coastal housing).
  - For future open space areas, use of low risk aversion WSE should be considered at the time of design of the seal level protection improvements.

The Mare Island topography (elevation ranges) are shown on Figure 4-3 by color shading as discussed below:

- Areas below the current FEMA Mare Island Strait Base Flood Elevation of 10.0 feet are shown in red and orange. These areas currently have a 1 percent or greater annual risk of tidal flooding.
- Areas at 1 percent annual risk of tidal flooding by 2070 for SLR to 13.5 feet are shown in yellow.

[2] The State of California Sea Level Rise Guidance: 2024 Update recommendation for 2070 SLR WSE is 3.0 feet of increase, associated with the High Sea Level Scenario (SLS).

[3] The State of California Sea Level Rise Guidance: 2024 Update recommendation for 2100 SLR WSE is 6.6 feet of increase, associated with the High SLS.

- Areas at 1 percent annual risk of tidal flooding by 2100 from SLR to 17.0 feet are shown in green and purple.
- Areas with no colored shading are already above elevation 17.0 feet.

Based on a year 2100 SLR elevation of 17.0 feet, Figure 4-3 shows (with a red hatch) the areas that require future protection from SLR. Some of these areas are privately owned and/or outside of the Mare Island Company redevelopment area. Nevertheless, flood protection facilities for these areas were developed and are presented in Section 4.7.

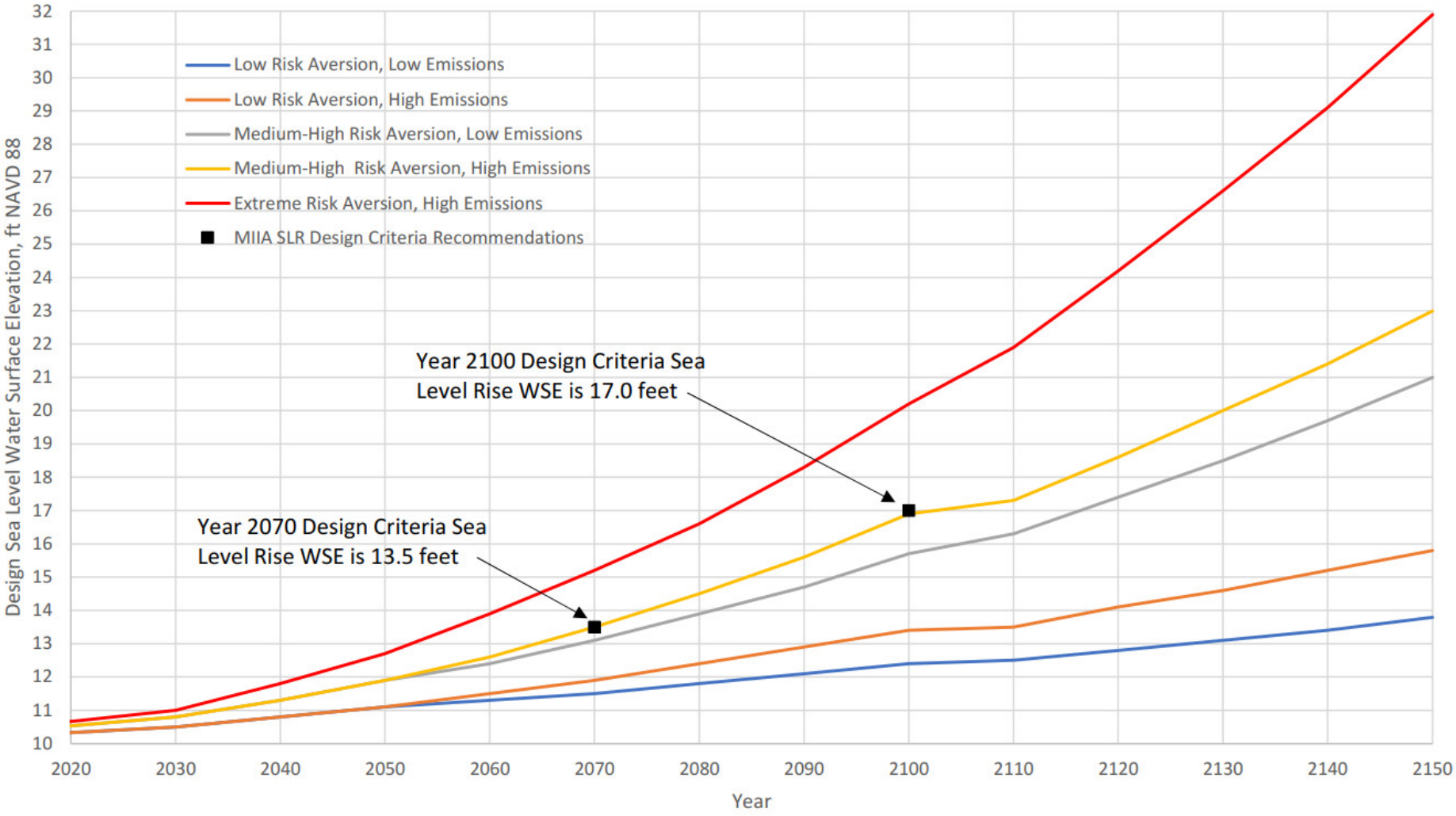
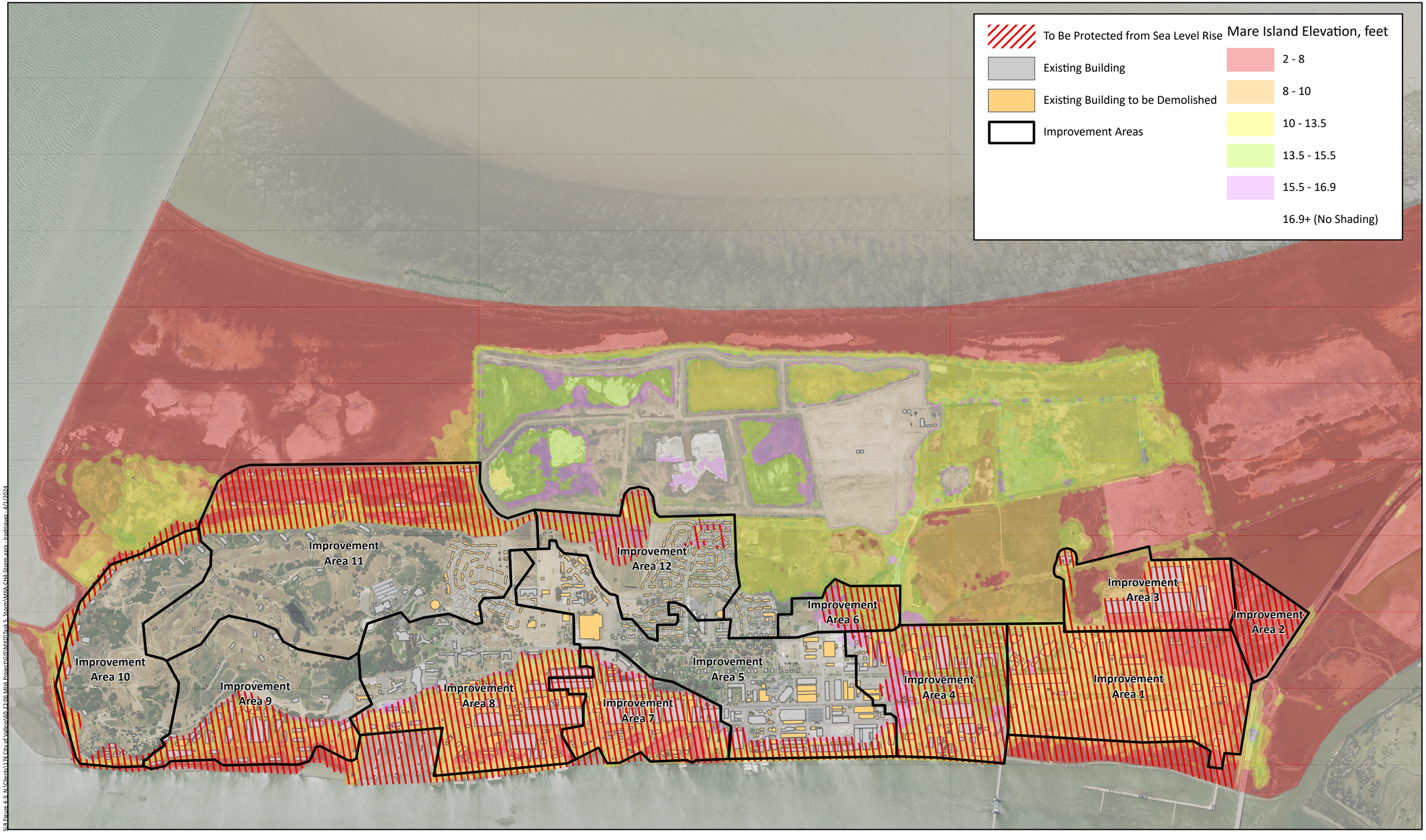


Figure 4-2. Forecast Design Sea Level Elevations Based on the State of California Sea-Level Rise Guidance, 2018 Update



SLR Figure 4-3 N:\Clients\174 City of Vallejo\GIS\2206 MIIA Project\GIS\MXD\Task 5. Storm\MIIA_Ch4_Storm.aprx - Ironhorse - 4/1/2024

Prepared by:

WEST YOST

0 875 1,750 Feet

At the time of design of these Sea Level Rise (SLR) facilities, the following should be verified:
1. The current and project future SLR design water surface elevations.
2. This approach for SLR as an individual SLR facility and in coordination with other SLR facilities previously constructed and proposed for Mare Island.
3. The sizes and capacities of these facilities.

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Mare Island Elevations and Sea Level Rise Protection Areas

Figure 4-3

4.5.2 City and District Storm Drainage and Flood Control Design Criteria

The City's relevant storm drainage and flood control requirements are summarized below from City Ordinances, Chapter 7.98 Flood Management Regulations, Section V. Provision for Flood Hazard Reduction:

- Residential Elevation and Floodproofing. Residential construction, new or substantial improvement, shall have the lowest floor, including basement protected from the FEMA Base Flood Elevation. Around the perimeter of Mare Island, the Base Flood Elevation is mapped as AE[4], with an elevation of 10.0 feet. For the AE zone, the ordinance requires one-half foot or more of freeboard above the base flood elevation, resulting in a minimum elevation for flood protection of 10.5 feet for Mare Island.
- All new and replacement water supply and sanitary sewage systems shall be designed to minimize or eliminate infiltration of floodwaters into the systems and discharge from the systems into floodwaters.
- All subdivision proposals shall have public utilities and facilities such as sewer, gas, electrical and water systems located and constructed to minimize flood damage.

The City ordinance is based on current FEMA base flood elevation and mapping and does not address sea level rise.

The District's current (July 2020) Engineering Standards are summarized below:

- Storm drains shall be designed for the 15-year, 24-hour peak flow with the hydraulic grade line (HGL, basically the water level) at least 1.0 foot below the gutter flow line (based on Mean Annual Precipitation of 20-inches per year for Mare Island).
- Building pads shall be at least 1.0 foot above the 100-year WSE, or the 100-year WSE must be at least 1.0 foot below the building pads.
- Storm drain O&M efforts and costs shall be minimized.
- Soil infiltration shall be based on the soil hydrologic soil group, which for Mare Island is mostly group D, group C/D, or undefined (Mare Island fill). There is a very small area of group C, which is in the industrial area, so to be conservative, for this area the soils are assumed to be compacted resulting in low infiltration rates like hydrologic soil group D soils. Thus, the entire island is considered to be hydrologic soil group D.
- Drainage studies shall also include an evaluation of 3.5 feet of SLR for the next 50-years (for the year 2070). The facilities needed for SLR need not be constructed now but should be accommodated during current planning and design.
- Detention basins shall be designed to safely store the flow from the 100-year, 10-day storm event. Minimum freeboard during the 100-year design storm shall be one foot if the design water level is below the surrounding ground surface. Leveed detention basins are not allowed. However, recently, District staff required that a basin be sized for the 100-year,

[4] FEMA AE Zone is defined as an area that will be inundated by the flood event having a 1-percent chance of being equaled or exceeded in any given year. The 1-percent annual chance flood is also referred to as the base flood or 100-year flood.

1-day storm event (versus the 100-year 10-day storm event), and the 100-year, 1-day storm event is use for sizing detention basins on Mare Island.

- Pump stations shall be designed to pump the 100-year, 1-day peak flow reaching the pump station using the firm capacity of the pump station. Pump stations shall include one redundant pump and a stand-by generator capable of powering all the firm capacity pumps.

District staff indicated a design preference that pump station components critical to operation and maintenance, including but not limited to electrical components and certain mechanical components, shall be placed at or above elevation 18.0 feet, which includes the FEMA 100-year storm surge elevation of 10 feet, and the SLR increase associated with the medium-high risk aversion for 2100 and 1 foot of freeboard. Additionally, pump station pad elevations and unimpeded access corridors shall be placed at or above elevation 13.5 feet, which includes the FEMA 100-year storm surge elevation and the SLR increase associated with the medium-high risk aversion for 2070. City staff indicated a design preference that for mixed use commercial/residential projects, 7.0 feet of SLR for the year 2100 would be appropriate. Thus, for this MIIA, all infrastructure and all development shall be based on a year 2100 WSE of 17.0 feet.

4.5.3 Recommended Drainage and Flood Control Design Criteria

Based on the review of documents and feedback from the Client Team, the following criteria were used for Mare Island storm drainage, flood control, and SLR design criteria. These criteria are primarily based on the VFWD Engineering Standards (July 2020). The VFWD Standards allow the Director of Technical Services to waive or modify the standards if unusual conditions exist. The written request to waive or modify the standards must explain why the standards cannot be achieved. Mare Island presents many unusual conditions; and consequently, waiver or modification of the VFWD standards may be justified on a case-by-case basis. The District may update their standards in the future, and future changes to the VFWD standards should be integrated in improvement designs when the improvements are designed.

1. Storm drains shall be sized for the 15-year, 24-hour peak flow with the HGL at least 1.0 foot below the gutter flow line (based on Mean Annual Precipitation of 20 inches per year).
2. Building pads shall be at least 1.0 foot above the 100-year WSE, or the 100-year WSE shall be at least 1 foot below the building pads.
3. Storm drain O&M efforts and costs shall be minimized.
4. Infiltration shall be based on the soil hydrologic group D properties.
5. SLR should be addressed as follows:
    a. The Year 2070 SLR elevation of 13.5 feet shall be assumed (see Figure 4-3 of the VFWD Engineering Standards).
    b. The Year 2100 SLR elevation of 17.0 feet shall be assumed (see Figure 4-2). It is noted that there are other reliable sources that forecast other SLR elevations for 2100. The best available SLR forecasts should be used at the time individual SLR protection facilities are designed. This may result in higher or lower SLR elevations than identified above.
    c. Areas above elevation 17 feet should gravity drain (Criteria 1 and 2 shall also be achieved).
    d. Areas from 13.5 feet to 17.0 feet can initially gravity drain but must include a plan to protect these areas for a SLR to 17.0 feet. Land shall be reserved for the required future

facilities and the proposed drainage facilities must be designed to facilitate the future flood protection improvements.

e. Areas below elevation 13.5 feet shall be protected as soon as feasible. The initial protection should provide for a SLR elevation of 13.5 feet and allow for the protection to be increased for a SLR elevation 17.0 feet.

6. Pump stations shall be designed to pump the 100-year, 1-day peak flow reaching the pump station using the firm capacity of the pump station. Pump stations shall include one redundant pump and a stand-by generator capable of powering all the firm capacity pumps.
7. Detention basins may be difficult to construct, operate, and maintain successfully and should not be used. Rising sea and ground water levels may also result in the basins being continuously full of water. Nevertheless, there may be unique situations where use of detention basins could be considered.
8. Flood proofing individual buildings could be considered for some specific buildings.
9. Stormwater treatment facilities including Best Management Practice devices and Low Impact Development strategies will be designed as part of the redevelopment process. All stormwater treatment measures shall meet City and District requirements.

4.6 CONDITION ASSESSMENT

The storm drainage and flood control infrastructure condition assessment included field inspection of storm drain inlets, storm drain maintenance holes, and LS-15.

4.6.1 Drain Inlets and Maintenance Holes

Visual inspection of 23 drain inlets and maintenance holes was performed. Inspections focused around the outfalls to Mare Island Straight and were performed in September 2023 by Carlson, Barbee & Gibson, Inc. as a subconsultant to West Yost. The structures were also surveyed to obtain topographic elevation data in support of the H&H model and capacity assessment. The previous storm drain evaluation (*Mare Island Preliminary Storm Drain Report – Draf*t, BKF, May 2022) concluded that most of the existing storm infrastructure does not currently meet the District Engineering Standards. Consequently, the infrastructure will not meet the Standards with SLR. Therefore, only 23 drain inlets and maintenance holes were inspected, and a thorough inspection of most of the storm water infrastructure was not performed.

Table 4-2 summarizes the condition of the structures as good (in good structural condition and generally meet existing design and construction standards); acceptable (in good or acceptable structural condition – potentially with minor defects but does not meet current design or construction standards); or poor (structure has defects which compromise its structural integrity and/or are an immediate concern to effective operation and maintenance of the system, and do not meet current design or construction standards).

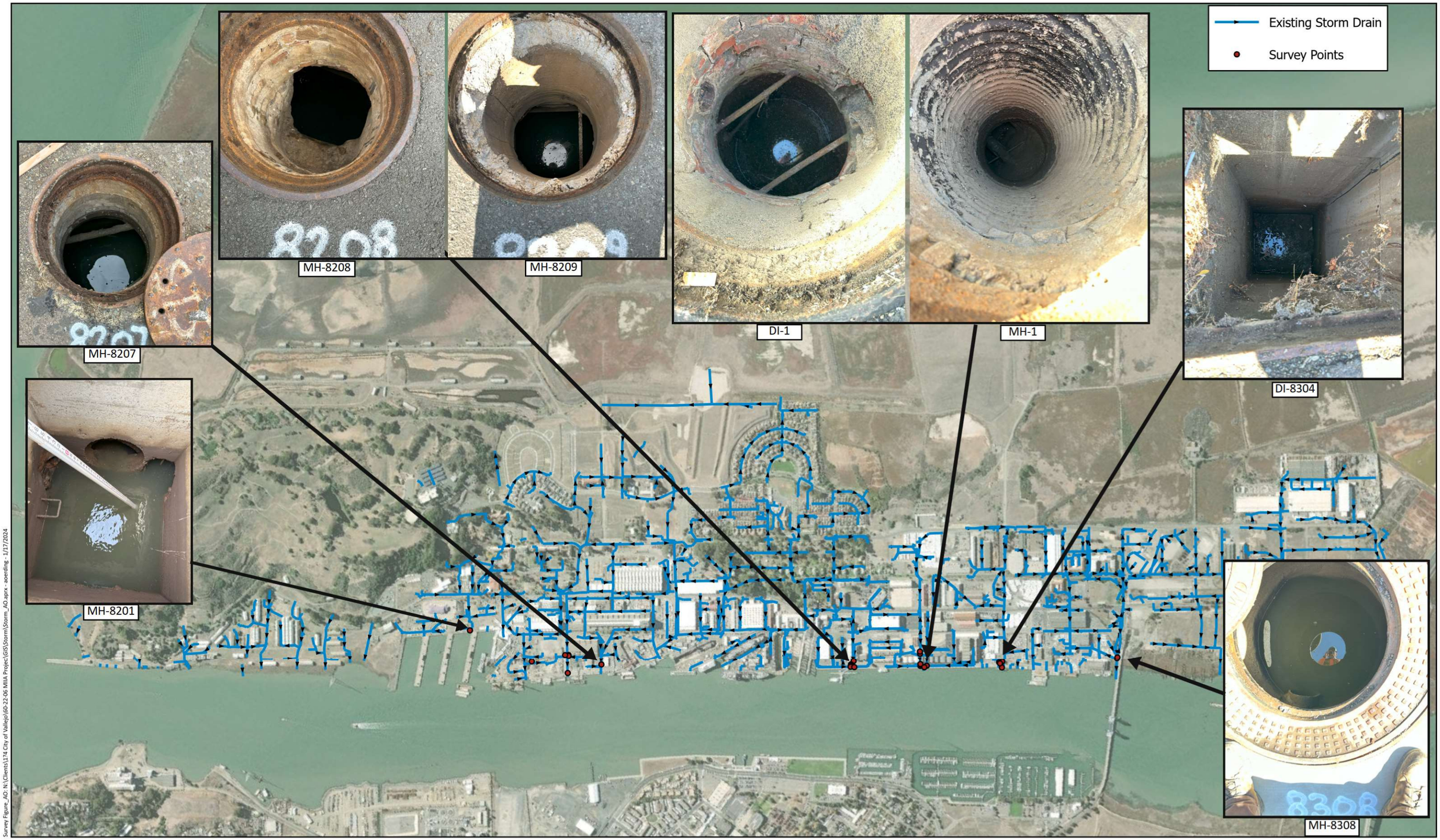
Table 4-2. Storm Drainage Structure Condition Summary

Rating	Good	Acceptable	Poor	**Total**
Drain Inlets	1	3	3	**7**
Maintenance Holes	6	7	3	**16**
Total	**7**	**10**	**6**	**23**

Figure 4-4 presents the location of the inspected structures and photos from several structures. Observations from the inspections include:

- Most structures had water in the bottom. Sometimes, this extended up to several feet below ground surface elevation. Higher water levels were observed in areas closer to Mare Island Strait (see MH-8308 on Figure 4-4). Water in the structures is not necessarily indicative of poor condition or operation but does not allow visibility or access to the structure and pipe connections.
- Many structures are made of brick (see MH-8207, MH-8208, DI-1) and other non-standard materials (e.g., corrugated metal – see MH-1). Some of the brick maintenance holes appear to have since been lined with mortar during some previous rehabilitation. Individual bricks have been observed in pipes during sewer CCTV inspections, so it is likely bricks have also become loose and fallen into the storm drainage system. Bricks can obstruct the pipeline, reducing capacity and leading to further debris buildup and potential blockage. The District Engineering Standards do not allow brick maintenance holes and require them to be replaced.
- Many structures have what appear to be utility bore-throughs. Bore-throughs are pipes or conduits of other utilities (potentially gas, electrical, etc.) that have been constructed through the maintenance hole or drain inlet structure (see MH-8207, MH-8209, D-1, MH-1). Bore-throughs compromise the structural integrity of the host structure, can reduce the conveyance capacity, can increase debris accumulation, and restrict access for rehabilitation or maintenance.
- Most metallic elements (frame, cover) were corroded, and some structural issues were observed in the barrels of maintenance holes.
- Several new structures were in good condition (see MH-8308, which is in good condition but has high water level).
- Most structures need sediment and debris to be cleared.

Based on the structures inspected, approximately 70 percent (16 out of 23 inspected) would require replacement to meet current District standards.



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Prepared by:

WEST YOST

0 1,000 2,000 Feet

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

Existing Storm Drain and Flood Control System Inspection and Survey Results

Figure 4-4

4.6.2 Lift Station 15

As shown on Figure 4-1, LS-15 is located in North Mare Island at the north end of Independence Street. Based on information provided by the District and Mare Island Company, it is understood that the station currently provides drainage and flood protection to the Highway 37 interchange, the surrounding undeveloped, low-lying areas of North Mare Island, and portions of the Alco and EPS properties. All other buildings and structures in areas of North Mare Island previously serviced by LS-15 have been demolished or decommissioned. LS-15 consists of two pumps (one submersible pump in a wet well and one vertical turbine pump), a wet well, valve vault, and electrical kiosk (see Photo 4-1).



Photo 4-1
Lift Station 15

The lift station condition assessment included visual inspection of LS-15 and its major components. Inspections were performed by two West Yost civil/mechanical engineers and two District staff on February 23, 2023.

Major components were rated for their physical condition/integrity on a scale of 1 to 5, as defined in Table 4-3.

Table 4-3. Condition Rating Index	
Rating	Condition Rating Index
1	Excellent or like new
2	Slight visible degradation
3	Visible degradation
4	Integrity of component moderately compromised
5	Integrity of component severely compromised

In 2022, the District conducted a condition assessment of lift stations currently in service on Mare Island (including sewer lift stations and storm water LS-15). This assessment included identification of major interior components that required rehabilitation or replacement to bring to District standards. The assessment included an estimation of costs for the required improvements. The 2022 assessment was used to validate the February 2023 inspection and recommendations herein.

A summary of the condition assessment results for LS-15 is presented in Table 4-4 by major components. Ratings presented in Table 4-4 were generated by grouping components as detailed in the headings; for example, the Pumps and Motors grouping for LS-15 includes two pumps and two motors, which have an average condition rating of 5.

Table 4-4. Summary of Lift Station 15 Condition				
Condition Rating of Component(s) and Comments				
Pumps and Motors	Suction and Discharge Piping and Valves	Wet Well and Vaults	Electrical	Site
5	3	4	4	3
• P1 (submersible) has performance issues; rebuilt in 2022 due to power issue • P2 (vertical) has bad bearing but continues to run; rebuilt in 2018 • District report (2022) documents performance issues on both pumps • No backup pump	• Corroded check valves	• Concrete old and worn • Corrosion on grating, bar screens	• Defective 3-phase power makes P1 motor smoke (likely only single-phase incoming to motor) • Lighting panel old • Site lighting poor/inoperable	• Gate location not good for truck access

LS-15 is in moderately to severely compromised condition and is unreliable. It is currently understood that the station's primary use is to serve the Highway 37 interchange. Some surrounding areas of North Mare Island contribute runoff to LS-15, which would need to be rerouted to gravity drain to avoid potential ponding and vector control. Also, as discussed in Section 4.7, LS-15 will not be needed in the future. Consequently, it is recommended that the City and District:

- Coordinate with Caltrans and advise them that:
  - LS-15 is moderately to severely compromised and not reliable.
  - The primary purpose of LS-15 is to serve the Highway 37 interchange. LS-15 will not required to serve Mare Island in the future.
  - Caltrans should construct a new lift station located in the Caltrans right-of-way to serve the Highway 37 interchange.
- Until the Caltrans lift station is constructed, LS-15 should continue to be operated and maintained. In the meantime, minimize improvements or upgrades to the facility.
- Once the Caltrans lift station is constructed, LS-15 can then be demolished and removed to facilitate the redevelopment of North Mare Island.

4.6.2.1 Lift Station 15 Condition Improvement Recommendations

If LS-15 is not replaced with a new lift station facility, improvements required to maintain current operations are summarized in Table 4-5.

Table 4-5. Summary of Lift Station 15 Improvement Recommendations

Pumps and Motors	Suction and Discharge Piping and Valves	Wet Well and Vaults	Electrical	Site
• Replace all pumps and motors (2) • Consider adding backup (3rd) pump	• Replace all piping and valves	• Perform non-destructive testing to confirm condition for future use. • Likely rehab with patch and repair; alternatively, replace with new structure. • Replace metal components	• Resolve electrical service to 3-phase (appears to be single-phase currently) • Replace, upgrade all site lighting • Connect station to SCADA for monitoring, install uninterrupted power supply (UPS) • Add portable generator	• Relocate gate for access improvement

4.7 CAPACITY ASSESSMENT AND RECOMMENDED IMPROVEMENTS

The capacity assessment and recommended improvements are discussed below.

4.7.1 Need for Capacity Assessment

Based on the *Mare Island Preliminary Storm Drain Report – Draf*t (BKF, May 2022 the existing storm infrastructure mostly does not meet the District Engineering Standards capacity requirements and will not meet the capacity requirements with SLR. Figure 4-3 shows the areas of Mare Island that would be inundated with a WSE of 10.0 feet.

The existing sea walls are likely between 10.5 and 12.7 feet, not considering possible settlement since construction. As such, it currently provides inadequate flood protection freeboard for the current Base Flood WSE of 10.0 feet. With SLR, the current sea wall top will be below the 2070 WSE of 13.5 feet by as much as 3 feet and will be below the 2100 WSE of 17.0 feet by as much as 6.5 feet. Figure 4-3 shows the areas of Mare Island that would be inundated with WSEs of 13.5 and 17.0 feet. Thus, the existing sea wall lacks adequate elevation to provide current and future flood protection for Mare Island. See Chapter 6 for additional evaluation of the sea wall.

A significant portion of current or future developed areas on Mare Island are below an elevation of 17.0 feet and will require a variety of methods to provide effective storm drainage and flood protection. To aid in this evaluation, an H&H drainage model was developed, and used to evaluate the island under different storm and SLR conditions.

4.7.2 Hydrologic and Hydraulic Drainage Model

An H&H model was developed for Mare Island to address sea level rise using recommended criteria for design storm performance. The West Yost team met with the Client Team to identify areas of Mare Island that require protection from sea level rise. Areas of Mare Island outside these protection areas are not expected to be developed and will remain subject to sea level influence.

The H&H model was developed in CHIWater's PCSWMM, which is fully dynamic H&H modeling software ideal for incorporating the intricacies found in the Mare Island drainage collection system including flow splits, surcharging, and tidal influenced water levels.

The H&H model was developed using the previously calibrated rainfall, infiltration, and catchment behavior values from another District PCSWMM H&H Model. The project area was assessed using the 15-year, 24-hour storm for storm drain sizing, and the 100-year, 24-hour storm for flood projection, based on current VFWD standards. To simulate worst-case tidal conditions, outfalls were modeled with a fixed tailwater stage of 17.0 feet, the year 2100 SLR. The final model included 101 catchment areas that delineate 12 Improvement Areas (IA) identified for sea level rise protection. Catchments were delineated based on the overall watershed limits, topography, and feasible mainline trunk drainage paths. Each catchment was assigned an inlet to model surface drainage, which was subsequently routed through conduits to an outfall exiting the island. Conduit pathways were identified by reviewing existing storm drainage alignments and re-using existing storm drainage segments or alignments where feasible. In areas where re-development or new development are anticipated, conduit pathways that reduce overall pipe lengths were selected.

4.7.3 Development of Recommended Improvements

Results from the H&H model were used to develop recommended infrastructure to achieve the storm drainage and flood control design criteria presented in Section 4.5.

There are several options to developing infrastructure solutions, based on the elevation and other site conditions. The lower areas of Mare Island can flood from the current and future anticipated increased tide levels. The higher levels of Mare Island could still drain by gravity. The increased SLR levels can be addressed by:

- Gravity draining the high areas
- Filling the low tributary areas to be above the SLR elevation
- Constructing a levee or sea wall around the low areas to block the water from the Mare Island Strait from flowing onto Mare Island
- Flood proofing individual buildings

There are advantages and disadvantages to each approach. Other Bay Area cities and flood control districts including the City of Burlingame, Oro Loma Sanitary District, Foster City, and the Port of San Francisco have developed projects to address future SLR. Because addressing SLR is such a complicated issue, the efforts and experiences of these and other Bay Area communities should continue to be reviewed and considered as the efforts to protect Mare Island are further planned, designed, and constructed.

The recommended improvements developed in this study include use of fill to raise the ground surface to be above the future SLR WSEs (along with the needed freeboard), use of retrofit or new sea walls to hold the Mare Island Strait sea water back, storm drain pipe system to convey the runoff to Mare Island Strait, and pump stations to lift runoff into Mare Island Straight. Location and sizing of infrastructure is based on the assumption that a sea wall or levee will provide protection to the Island from Mare Island Strait. Without the assumed sea wall, flood protection for much of the current development on Mare Island will not be possible.

The improvements have been grouped into twelve geographic areas (Improvement Areas) based on location and similar protection needs. H&H model results for each IA are discussed in further detail in the following sections. Itemized summaries of recommendations for each IA are provided in Tables 4-6 through 4-17. The recommended improvements are grouped as shown on Figures 4-5 through 4-11 (at the end of this chapter). An overview of all IAs is shown on Figure 4-3.

4.7.4 Improvement Areas

The following sections describe each of the IAs, recommended improvements, and associated timing based on land elevations and expected sea level rise.

4.7.4.1 Improvement Area 1: North Mare Island East of Azuar Drive

IA 1 (shown on Figure 4-5) is located in North Mare Island east of Azuar Drive and consists of mostly open space, paved areas, and demolished buildings. Because of the lack of existing buildings in this area, SLR protection could be achieved by filling the entire area to a minimum elevation of approximately 18.0 feet

(1-foot above the 2100 SLR elevation of 17.0 feet). Areas along the shoreline at lower elevations could remain at the current elevation to protect the natural shoreline resources.

Most, if not all, of the existing storm drain system could be abandoned and replaced with the proposed storm drain system shown on Figure 4-5. Depending on the SLR protection measures chosen for Highway 37, LS-15 will either no longer be needed or require significant rehabilitation or replacement (see Section 4.6.2). For the purpose of this evaluation, it was assumed that LS-15 will be abandoned in the future as SLR protection for Highway 37 is further refined. Table 4-6 presents an itemized summary of the recommended improvements for IA 1.

Ground surface elevations within IA 1 vary from as low as 8 feet, up to 16 feet. The lowest areas within this IA are expected to be impacted by SLR in current conditions, based on SLR forecasts. However, since the majority of the area is undeveloped, timing of recommendations for IA 1 will be based on future land use and development timing. The identified fill should be completed several years in advance of the future development and the storm drains could be installed as development occurs.

Table 4-6. Itemized Summary for Improvement Area 1

Item Description	Unit	Quantity
Fill Material	CY	1,985,000
15-inch diameter storm drain	LF	90
18-inch diameter storm drain	LF	100
24-inch diameter storm drain	LF	2,600
30-inch diameter storm drain	LF	2,400
36-inch diameter storm drain	LF	600
42-inch diameter storm drain	LF	450
48-inch diameter storm drain	LF	3,000
Maintenance Holes or Inlet Structures	EA	70
Outfalls	EA	2

4.7.4.2 Improvement Area 2: North Mare Island North of Independence Street

IA 2 (shown on Figure 4-5) is located in North Mare Island south of Highway 37 and north of Independence Street and consists of open, undeveloped space at elevations below 6 feet. Because of the lack of existing buildings in this area, SLR protection could be achieved by filling the entire area to a minimum elevation of approximately 18.0 feet (1-foot above the 2100 SLR elevation of 17.0 feet). Areas along the shoreline at lower elevations could remain at the current elevation to protect the natural shoreline resources.

Table 4-7 presents an itemized summary of the recommended improvements for IA 2.

Ground surface elevations within IA 2 are as low as 6 feet. Timing of recommendations for IA 2 will be based on future land use and development plans. If Improvement Area 2 remains as open space or preserved land, then no improvements may be needed before year 2100.

Table 4-7. Itemized Summary for Improvement Area 2		
Item Description	Unit	Quantity
Fill Material	CY	540,000
21-inch diameter storm drain	LF	90
24-inch diameter storm drain	LF	620
30-inch diameter storm drain	LF	250
Maintenance Holes or Inlet Structures	EA	10
Outfalls	EA	1

4.7.4.3 Improvement Area 3: North Mare Island West of Azuar Drive

IA 3 (shown on Figure 4-5) is located in North Mare Island west of Azuar Drive and consists of active industrial buildings and open space. Ground elevations vary from 6 feet up to 12 feet. SLR protection could be provided to IA 3 with sheet pile flood walls that extend around the west and south sides of the developed area and active buildings. After IAs 1 and 2 have been filled to 18.0 feet, additional protection would be provided to the north and east sides of IA 3. Some of the open space that is below elevation 8-feet could be excavated to an elevation of 2.0 feet and used as detention storage. A single 30 cubic feet per second (cfs) pump station would pump storm drainage from low lying developed areas and the detention basin. Table 4-8 presents an itemized summary of the recommended improvements for IA 3.

Since the majority of the developed area and active buildings within IA 3 are between the elevations of 8 and 10 feet, SLR protection should be implemented as soon as possible. Depending on the timing of SLR improvements in IAs 1 and 2, additional, temporary protections may be required to protect IA 3 from the north and east until permanent improvements have been made.

Table 4-8. Itemized Summary for Improvement Area 3		
Item Description	Unit	Quantity
Excavation, Hauling, and Grading	CY	125,000
Road Fill	CY	25,000
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight	LF	4,800
12-inch diameter storm drain	LF	830
24-inch diameter storm drain	LF	130
36-inch diameter storm drain	LF	2,950
Maintenance Holes or Inlet Structures	EA	8
30 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	1

4.7.4.4 *Improvement Area 4: Central Mare Island North of Connolly Street*

IA 4 (shown on Figure 4-6) is located in Central Mare Island east of Azuar Drive, south of G Street, and north of Connolly Street. The area consists mostly of active industrial buildings, with some residential development towards the southwest. Most of the existing developed area is projected to remain in use.

4.7.4.4.1 Alternative 1

Ground surface elevations vary from as high as 20 feet, down to 8 feet. SLR protection for IA 4 could be provided by raising the existing sea wall to at least 18.0 feet and extending a new sea wall segment under the G Street Bridge to connect to the North Mare Island fill. Most of IA 4 is situated at elevations below 17.0 feet and will require rerouting of existing multiple outfalls to a single 100 cfs pump station. Table 4-9 presents an itemized summary of the recommended improvements for IA 4.

Timing of recommendations for IA 4 is based on the lowest ground surface elevations of 8 feet, concentrated in the eastern boundary of the IA. Due to the density of development and low ground elevations, SLR protection is recommended as soon as possible for elevations below 10 feet. Improvements to elevations above 13.5 feet could be delayed until as late as year 2070.

Table 4-9. Itemized Summary for Improvement Area 4

Item Description	Unit	Quantity
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	4,900
30-inch diameter storm drain	LF	1,250
36-inch diameter storm drain	LF	1,500
42-inch diameter storm drain	LF	1,500
48-inch diameter storm drain	LF	2,100
Maintenance Holes or Inlet Structures	EA	60
100 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	1

4.7.4.4.2 Alternative 2

Because the existing buildings east of Railroad Avenue are scheduled for demolition in the future, there is an alternative to the approach discussed above for this IA. Most of the area east of Railroad Avenue and potentially some of the area between Walnut Avenue and Railroad Avenue could be filled to elevation 18.0 feet, and the filled area could then gravity drain to Mare Island Strait. The area west of Walnut Avenue and some of the area between Walnut and Railroad Avenue that is not filled could drain to a pump station that discharges to Mare Island Strait. This fill could be tied in with the fill for IA 1. This could reduce the pump station capacity from 100 cfs to about 50 cfs. The feasibility of this fill versus a tall sea wall will be dependent on the results of the full evaluation of the quay/sea wall (see Chapter 6).

4.7.4.5 *Improvement Area 5: Central Mare Island East of Azuar Drive*

IA 5 is shown on Figure 4-7. The area consists mostly of active industrial buildings, with some residential development towards the southwest. Most of the existing buildings are projected to remain in use.

A portion of existing developed area is below elevation 17.0 feet and will require rerouting of existing multiple outfalls to a single 30 cfs pump station. The remaining portions of IA 5 with higher ground surface elevations will require gravity storm drains to convey the flow to two additional outfalls. IA 5 could be protected from SLR by raising the existing sea wall to 17.0 feet and extending new sea wall segments west to areas of ground above the 2100 SLR elevation. Table 4-10 presents an itemized summary of the recommended improvements for IA 5.

IA 5 includes the Connolly Street Corridor redevelopment, which is part of the existing/near-term land use scenario (development through 2027). Timing of the recommendations for IA 5 is based on the lowest ground surface elevations of approximately 11 to 12 feet near the existing sea wall. This low area includes the eastern portion of Connolly Corridor. Thus, the lowest portion of the Connolly Street Corridor currently has only 1 to 2 feet of freeboard above the current design tide elevation of 10.0 feet. By 2034,

the design tide elevation will be about 11 feet, meaning the eastern portion of the Connolly Street Corridor will have no freeboard. Thus, the current redevelopment planning of the Connolly Street Corridor should:

- Include construction of flood control improvements for a Mare Island Straight water elevation of 11 to 12 feet (minimum). By 2034, without this minimum flood protection, the eastern portion of the Connolly Street Corridor will have no flood protection freeboard. It may be that it is more practical to initially construct the flood protection improvements for a water elevation of 13.5 feet than for 11 to 12 feet.
- Identify how flood protection will be achieved through the next 46 years for a water elevation of 13.5 feet by 2070.
- Identify how flood protection will be achieved for a 2100 water elevation of 17.0 feet.
- Allocate space for the future flood improvements in the current Connolly Street Corridor redevelopment land use planning. Improvements protecting these lower elevations should be constructed before the year 2030 based on the SLR forecast. The remaining improvements within IA 5 should be pursued by year 2080.

The flood protection for the Connolly Street Corridor is dependent on the existing conditions and ultimate plans for raising the sea wall (see Chapter 6). The urgency of flood protection for the Connolly Street Corridor means the evaluations of the sea wall and planning the sea wall improvements are also urgent. The sea wall improvement planning and construction will be complex and time consuming. Thus, it is important to begin the sea wall evaluations along with the Connolly Street Corridor land use planning.

Although not recommended as a permanent solution, the existing buildings withing the Connolly Street Corridor along the sea wall (above elevation 10 feet) could be evaluated for protection through retrofitting the individual buildings to achieve the needed freeboard while the full SLR protection plan for IA 5 and the sea wall is identified and implemented.

Table 4-10. Itemized Summary for Improvement Area 5

Item Description	Unit	Quantity
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	3,100
24-inch diameter storm drain	LF	460
30-inch diameter storm drain	LF	1,600
36-inch diameter storm drain	LF	4,900
42-inch diameter storm drain	LF	970
48-inch diameter storm drain	LF	2,400
Maintenance Holes or Inlet Structures	EA	90
30 cfs Pump Station	EA	1
Outfalls	EA	3

4.7.4.6 *Improvement Area 6: Central Mare Island West of Azuar Drive*

IA 6 (shown on Figure 4-6) consists of mostly undeveloped land west of Azuar Drive. The lowest ground surface elevations are 13.5 feet. Because of the lack of existing development in much of this area, SLR protection could be achieved by filling the entire area to a minimum elevation of approximately 18 feet (1-foot above the 2100 SLR elevation of 17.0 feet). A new storm drainage outfall could be installed, as shown on Figure 4-6. Table 4-11 presents an itemized summary of the recommended improvements for IA 6.

Timing of recommendations for IA 6 will be based on future land use and planned development schedule. The existing buildings are as low as 15.5 feet, so these buildings will need flood protection by the year 2080. Flood protection for the undeveloped areas will not be necessary until development plans are in place.

Table 4-11. Itemized Summary for Improvement Area 6

Item Description	Unit	Quantity
Fill Material	CY	201,000
15-inch diameter storm drain	LF	300
36-inch diameter storm drain	LF	1,640
Maintenance Holes or Inlet Structures	EA	12
Outfalls	EA	1

4.7.4.7 *Improvement Area 7: Central Mare Island South of 8th Street*

IA 7 (shown on Figure 4-8) is located in Central Mare Island east of Azuar Drive, and south of 8th Street. The area consists mostly of active industrial development, including the Mare Island Dry Docks and Lind Marine, which are anticipated to remain operational. Ground surface elevations within IA 7 are primarily between 10 and 12 feet.

SLR protection for IA 7 is complicated by the anticipated need for the dry docks to remain operational. The existing sea wall could be raised to 17.0 feet. The dry docks could be retrofit with new cassions at the 2100 SLR elevation and a surrounding sea or flood wall that would allow for dry dock access at low tide. Due to the overall low elevations of the area, the existing storm drain system would need to be rerouted to a single 75 cfs pump station. Table 4-12 presents an itemized summary of the recommended improvements for IA 7. Additional infrastructure planning for this area will require close coordination with Mare Island Dry Docks and Lind Marine.

Portions of IA 7 that are at 10 feet in elevation require SLR protection as soon as possible, with the last improvements installed by the year 2040.

Table 4-12. Itemized Summary for Improvement Area 7		
Item Description	Unit	Quantity
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	7,800
30-inch diameter storm drain	LF	820
36-inch diameter storm drain	LF	1,400
42-inch diameter storm drain	LF	1,200
48-inch diameter storm drain	LF	940
Maintenance Holes or Inlet Structures	EA	40
75 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	1

4.7.4.8 Improvement Area 8: Central Mare Island South of Bagley Street

IA 8 (shown on Figure 4-8) is located in Central Mare Island south of Bagley Street and north of Price Way. The area consists mostly of active industrial development and much of the Touro University campus. Ground elevations vary from as low as 8 feet along the Strait to over 50 feet within inland areas.

IA 8 could be protected from SLR by raising the existing sea wall to a minimum elevation of 18.0 feet and extending the sea wall south to areas above the 2100 SLR elevation. Due to their current lack of use, the existing wet docks to the south could be abandoned, reducing the amount of required sea wall extensions. Areas below a ground surface elevation of 18.0 feet would require the rerouting of existing storm drains to a single 100 cfs pump station. Storm drainage for upland areas could similarly be rerouted to two gravity outfalls. Table 4-13 presents an itemized summary of the recommended improvements for IA 8.

Timing of recommendations for IA 8 is based on the lowest ground-surface elevations of 8 to 10 feet, located adjacent to the Strait. Areas within these elevations should be protected as soon as possible from SLR. In addition, a significant remaining portion of IA 8 consists of elevations at or below 14 feet and should be protected from SLR by year 2034.

Table 4-13. Itemized Summary for Improvement Area 8		
Item Description	Unit	Quantity
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	5,200
24-inch diameter storm drain	LF	300
30-inch diameter storm drain	LF	70
36-inch diameter storm drain	LF	1,500
42-inch diameter storm drain	LF	3,500
48-inch diameter storm drain	LF	1,100
54-inch diameter storm drain	LF	1,350
Maintenance Holes or Inlet Structures	EA	70
100 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	3

4.7.4.9 Improvement Area 9: Central Mare Island East of Railroad Ave and South of Price Way

Central Mare Island east of Railroad Avenue and South of Price Way (shown as IA 9 on Figure 4-9) consists of steep, undeveloped hills which are planned for residential development. The lower area (elevations 8 to 10 feet) has mostly industrial buildings which are expected to remain. The low area does not have an existing sea wall. Should the existing buildings remain in service, SLR protection could be provided by the installation of a new sea wall or levee to a minimum elevation of 18.0 feet. Areas with a ground surface elevation below 18.0 feet would require rerouting of existing storm drains to a single 30 cubic feet per second (cfs) pump station. New gravity flow piping gravity convey storm drainage from the upland, undeveloped areas could be installed and routed to two gravity outfalls. Table 4-14 presents an itemized summary of the recommended improvements for IA 9.

Timing of recommendations for IA 9 is based on the lowest ground surface elevations of 8 to 10 feet, which require SLR protection under current conditions. The low-lying elevations are concentrated along the strait. Due to potential for redevelopment of low-lying areas, and new development in upland areas, full SLR protection for year 2100 can be implemented over time and incorporated with future development plans. Temporary protection up to elevations of 12 feet would be adequate until the year 2040.

Table 4-14. Itemized Summary for Improvement Area 9

Item Description	Unit	Quantity
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	4,400
21-inch diameter storm drain	LF	80
24-inch diameter storm drain	LF	2,900
30-inch diameter storm drain	LF	2,200
36-inch diameter storm drain	LF	3,500
42-inch diameter storm drain	LF	1,000
48-inch diameter storm drain	LF	510
Maintenance Holes or Inlet Structures	EA	80
30 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	3

4.7.4.10 Improvement Area 10: Tyler Road

IA 10 includes Tyler Road, which loops around the southern perimeter of Mare Island at elevations of 10 to 14 feet, with small pockets of existing development in between the existing road and upland areas. Tylor Road could be partially filled and relocated above elevation 18.0 feet (1-foot above the 2100 SLR elevation of 17.0 feet) to provide SLR protection to much of the southern portion of the Island. Figure 4-9 shows the existing Tyler Road alignment with a proposed re-alignment. The east end of Tyler Road would tie into the proposed sea wall protecting IA 9. The west end of Tyler Road would tie into the proposed sea wall protecting IA 11.

Due to existing buildings that are to remain on the landside of Tyler Road, there are several low areas that will require two 5 cfs pump stations to prevent flooding during heavy rainfall events, which are shown on Figure 4-9. The Tyler Road realignment will also require approximately 10 culverts to drain runoff underneath Tyler Road and into the Mare Island Strait. To minimize pump station sizing, additional storm drains and catch basins will be needed to convey runoff from higher elevations away from the low areas and to one of the culverts. Table 4-15 presents an itemized summary of the recommended improvements for IA 10.

Timing of recommendations for IA 10 is based on the lowest elevations of Tyler Road between 8 to 10 feet. Segments of Tyler Road within this range of elevation should be planned for relocation to higher elevations as soon as possible to provide protection from SLR under current conditions. Improvements to roadway sections up to elevations of 12 feet, and the adjacent interior existing buildings, should be also protected from SLR as soon as possible. Remaining sections of Tyler Road and the buildings above 12 feet can be protected after the year 2034 to ensure they have adequate freeboard.

Table 4-15. Itemized Summary for Improvement Area 10

Item Description	Unit	Quantity
Road Fill	CY	71,000
12-inch diameter storm drain	LF	50
15-inch diameter storm drain	LF	60
18-inch diameter storm drain	LF	540
24-inch diameter storm drain	LF	1,600
30-inch diameter storm drain	LF	700
36-inch diameter storm drain	LF	50
Maintenance Holes or Inlet Structures	EA	30
5 cfs Pump Station (firm capacity)	EA	2
Outfalls	EA	12

4.7.4.11 Improvement Area 11: Southern Mare Island

Southwestern Mare Island, between Riberio Road and Meginniss Road (shown as IA 11 on Figure 4-10), consists of multiple low-lying buildings surrounded by undeveloped areas at elevations of 8 to 10 feet, and does not have an existing sea wall. Should the existing buildings remain in service, SLR protection could be provided by the installation of a new sea wall or levee to at least elevation 18.0 feet. New storm drainage would need to be installed and directed to a single 15 cfs pump station. The undeveloped low-lying areas between the existing buildings could be filled to a minimum elevation of 18.0 feet and have gravity storm drains that also serve from the remaining upland, undeveloped areas. Table 4-16 presents an itemized summary of the recommended improvements for IA 11.

Timing of recommendations for IA 11 is based on the lowest ground surface elevations of 8 to 10 feet, paired with the potential current and future land use of the area. The existing buildings within the lowest elevations are currently unused, but have potential for listing as historical buildings, requiring protection and complicating the redevelopment of the area. Should these building require preservation as historical buildings, protection would be required for SLR up to 12 feet as soon as possible, with future increases in

SLR protection installed over time. If preservation of the buildings is not required, and the area is slated for redevelopment, SLR protection could be achieved with only fill, and the sea wall could be eliminated. In this case, the timing of the fill could be paired with redevelopment plans. The first step for achieving SLR protection for this area is to determine if the existing magazine buildings below elevation 17.0 feet can be removed or must remain. The magazine buildings above elevation 17.0 feet are already protected from SLR and would not need additional protection nor do they need to be removed.

Table 4-16. Itemized Summary for Improvement Area 11

Item Description	Unit	Quantity
Fill Material	CY	1,130,000
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	6,650
15-inch diameter storm drain	LF	1,800
18-inch diameter storm drain	LF	1,450
24-inch diameter storm drain	LF	8,600
30-inch diameter storm drain	LF	4,100
36-inch diameter storm drain	LF	2,300
42-inch diameter storm drain	LF	1,350
48-inch diameter storm drain	LF	760
54-inch diameter storm drain	LF	1,850
Maintenance Holes or Inlet Structures	EA	130
15 cfs Pump Station (firm capacity)	EA	1
Outfalls	EA	4

4.7.4.12 Improvement Area 12: Crisp Avenue and Tisdale Avenue Residential Area

The Crisp Avenue and Tisdale Avenue Residential Areas (shown as IA 12 on Figure 4-11), consists of a recent neighborhood development and adjacent undeveloped Marine Corp Firing Range. The residential developments are predominately above the year 2100 SLR, with the lowest building pad at an elevation just about 17.0 feet and minimal portions of residential roadway as low as 14.6 feet. To provide adequate freeboard from SLR, a low flood berm or wall could be installed along the western perimeter of the development, and the outfalls reconstructed with flap gates. The adjacent undeveloped Marine Corp Firing Range is predominately at elevations of 14 to 16 feet and could be filled to provide SLR protection. Table 4-17 presents an itemized summary of the recommended improvements for IA 12.

Timing of recommendations for IA 12 is based on the relatively high existing elevations of the developed area. With the lowest ground surface elevation of approximately 14.6 feet, SLR protections would not be required until the year 2075.

Table 4-17. Itemized Summary for Improvement Area 12		
Item Description	Unit	Quantity
Fill Material	CY	67,000
Sheet Pile Flood Wall (or other approach for flood protection from Mare Island Straight)	LF	1,350
18-inch diameter storm drain	LF	90
24-inch diameter storm drain	LF	1,700
30-inch diameter storm drain	LF	510
42-inch diameter storm drain	LF	260
Maintenance Holes or Inlet Structures	EA	5
Outfalls	EA	3

4.8 INTEGRATION OF CONDITION AND CAPACITY ASSESSMENTS AND RECOMMENDED IMPROVEMENTS

This section incorporates recommendations from the condition and capacity assessments into an integrated set of recommendations for the existing/near-term and build-out scenarios. Because the existing infrastructure was concluded to lack adequate capacity and was recommended for full replacement (BKF, 2022), the drivers for improvements to the storm drainage and flood control system are sea level rise, existing elevations on the island (relative to the current (10.0 feet), 2070 (13.5 feet), and 2100 (17.0 feet) SLR WSEs), existing development status, and future development activity. As mentioned previously, the basis for development of recommendations and associated timing was based on current design standards and best available guidance documents. Development of final solutions to address storm drainage and sea level rise on Mare Island should be based on use of the most current guidance documents.

Table 4-18 summarizes the existing/near-term recommendations and associated justification for timing of the improvements. For all Improvement Areas, protection of the lowest elevations drives the improvements, but that does not mean protection of the entire area must occur at that time. Protection of higher elevations could be implemented over a longer time frame, aligned with SLR. The recommendations provided in Table 4-18 are the earliest improvements required for each IA. The long-term protection needs to be planned in coordination with the near-term protection, and space needs to be allocated for the long-term protection.

As presented in this chapter, much of the existing storm drain and flood control system appears to be in poor condition, does not meet current design and SLR guidance standards, and is recommended for full replacement. Although improvement recommendations are expected to be implemented as development occurs, the existing system is likely to continue to require reactive replacements as failures/issues occur, until it is fully replaced.

Table 4-18. Existing/Near-Term Storm Drainage and Flood Control Recommendations

Recommendation	Timing and Justification
LS-15 Reconstruction	• Coordinate with Caltrans to construct their own lift station for Highway 37. • Demolish and remove LS-15 after the Caltrans lift station is constructed to facilitate the redevelopment of North Mare Island. In the meantime, minimize improvements or upgrades to the facility.
Continue evaluation of the sea wall and develop a strategy to provide protection for the 2100 water level of 17.0 feet.	• Development of a long-term sea level rise adaptation strategy should begin as soon as possible. This process should be collaborative, involving affected communities, regulatory agencies, and stakeholders to evaluate feasible protection and adaptation options for the projected 2100 water level of 17.0 feet. • A visual assessment of the existing seawall was completed in April 2025 as part of this project. Recommendations from that assessment should be advanced to support long-term protection for the island (see Chapter 6). • Given the scale, complexity, and cost of conventional sea wall improvements, early planning is critical to secure funding, coordinate across agencies, and implement phased adaptation strategies that adequately protect Mare Island from sea level rise.
Perform visual inspections of all maintenance holes and inlets. Replace maintenance holes and inlets constructed of non-standard materials (e.g., brick, corrugated metal) in system areas not scheduled for demolition or rerouting.	• Complete as soon as possible and before 2027. • Non-standard materials such as brick and corrugated metal do not meet design standards and may cause issues such as early failure and other negative impacts on the system. Individual bricks been observed in pipes during sewer CCTV inspections, so it is likely bricks have also become loose and fallen into the storm drainage system.
Perform visual inspection (via maintenance holes or CCTV) to identify corrugated metal pipelines. Prioritize replacement of corrugated metal pipelines.	• Non-standard materials such as corrugated metal pipe are prone to corrosion and failure.
IA 3 Improvements	• Developed area; existing elevations as low as 6 feet. Implement as soon as possible. Coordinate improvements with the existing businesses and with IA 1 improvements.
IA 4 Improvements	• Developed area; existing elevations as low as 8 feet. Implement as soon as possible for areas below 12 feet.
IA 5 Improvements (Connolly Street Corridor)	• Developed area; implement lowest areas (11 to 12 feet) before Connolly Street Corridor is developed. Start evaluations for sea wall along with Connolly Street Corridor land use planning.

Table 4-18. Existing/Near-Term Storm Drainage and Flood Control Recommendations	
Recommendation	Timing and Justification
IA 7 Improvements	• Developed area; implement as soon as possible for areas below 12 feet. Coordinate improvements with Mare Island Dry Docks and Lind Marine. Existing elevations as low as 10 feet.
IA 8 Improvements	• Developed area; existing elevations as low as 8 feet. Implement as soon as possible for areas below 12 feet. Coordinate improvements with Lind Marine.
IA 9 Improvements	• Developed area; existing elevations as low as 8 feet. Implement as soon as possible for areas below 12 feet.
IA 10 Improvements	• Developed area; existing elevations as low as 10 feet. Implement as soon as possible for areas below 12 feet if Tyler Road is expected to be kept in service.
IA 11 Improvements	• Developed area; existing elevations as low as 8 feet. Determine if the magazine buildings in the low areas must be retained as historical structures or if they can be removed. • Implement as soon as possible for areas below 12 feet if buildings are expected to be retained/kept in service. Coordinate SLR protection areas and elevations with future land use.

Table 4-19 summarizes the improvement recommendations for the build-out scenario (which represents through year 2063) and beyond, to cover all areas of the island.

Table 4-19. Build-Out and Beyond Storm Drainage and Flood Control Recommendations	
Recommendation	Timing and Justification
IA 1 Improvements	• Currently mostly undeveloped area; implement as development occurs (existing elevations as low as 8 feet).
IA 2 Improvements	• Currently undeveloped area; implement as development occurs and as required for future land use (timing and elevation of protection) (existing elevations as low as 6 feet).
IA 6 Improvements	• Currently undeveloped area; implement as development occurs and as required for future land use (timing and elevation of protection) (existing elevations as low as 13.5 feet).
IA 12 Improvements	• Developed area; implement by 2075 (existing lowest elevations of 14.6 feet)

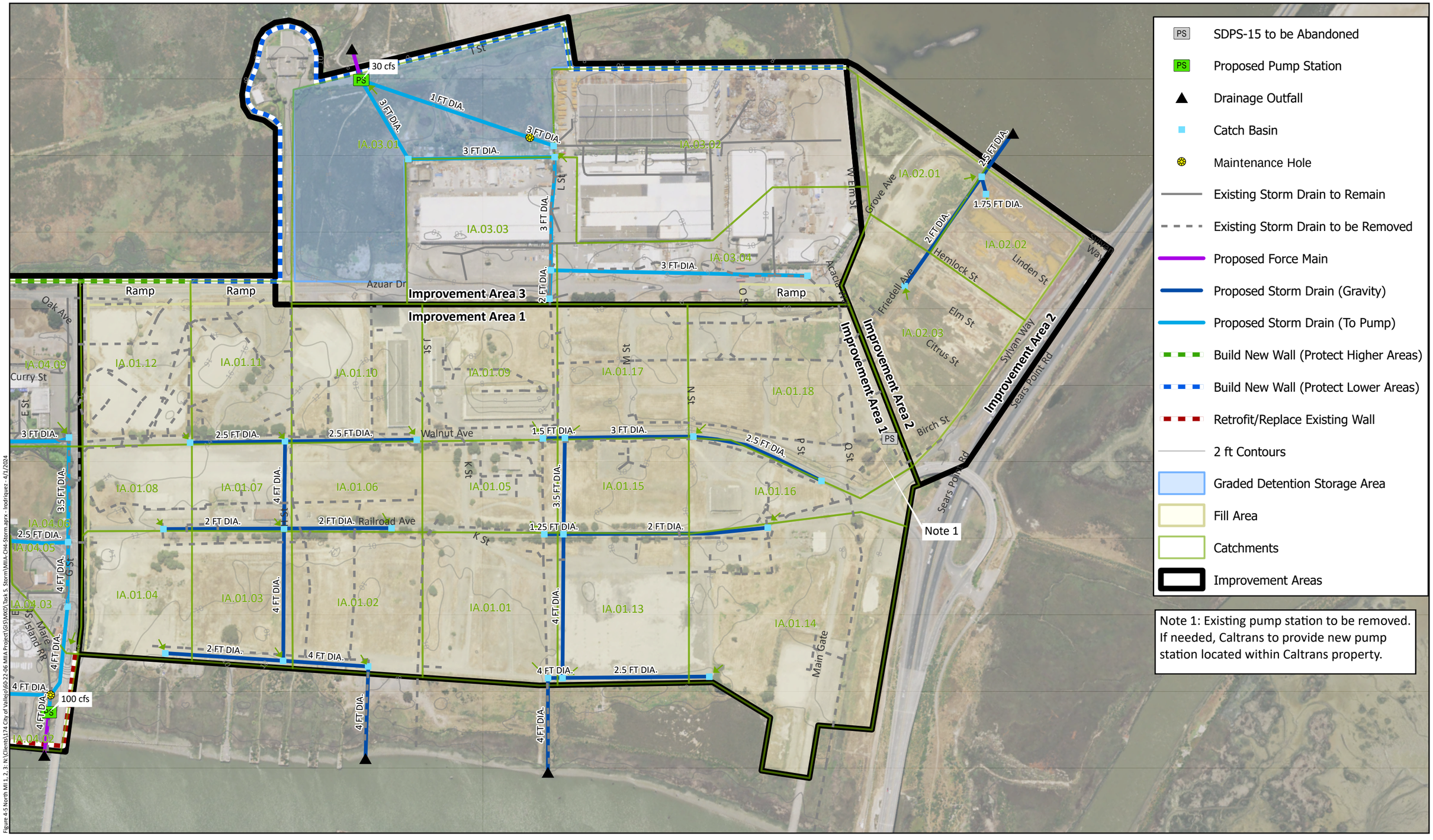

Figure 4-5 North M1 1, 2, 3 - N:\Client\174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - lrodriquez - 4/1/2024

Prepared by:

WEST YOST

0 350 700 Feet

At the time of design of these Sea Level Rise (SLR) facilities, the following should be verified:
1. The current and project future SLR design water surface elevations.
2. This approach for SLR as an individual SLR facility and in coordination with other SLR facilities previously constructed and proposed for Mare Island.
3. The sizes and capacities of these facilities.

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Drainage Improvements for North Mare Island
Improvement Areas 1, 2, and 3

Figure 4-5

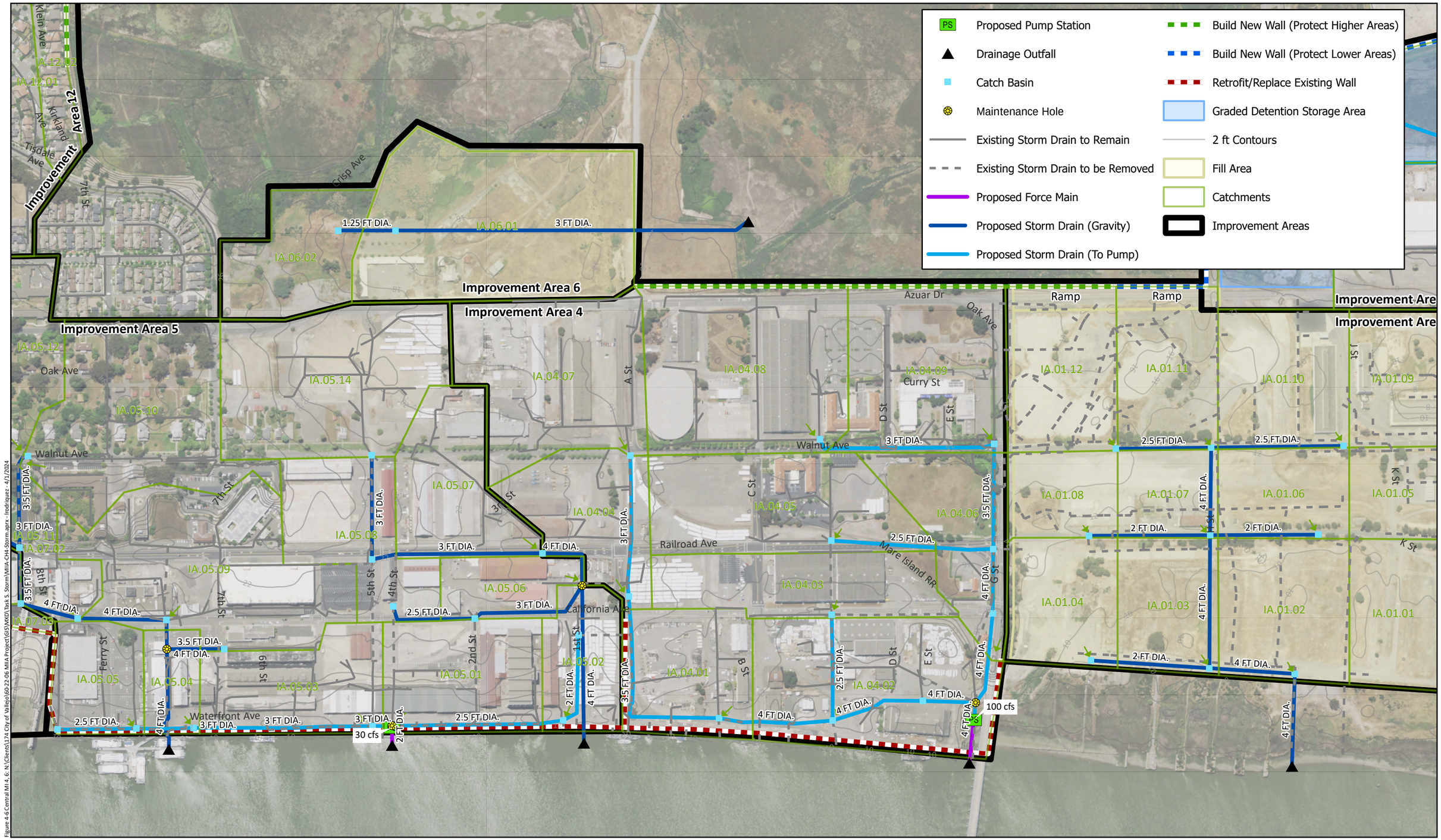

Figure 4-6 Central M14_6: N:\Client\1174 City of Vallejo\062-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - todriquez - 4/1/2024

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3. The sizes and capacities of these facilities.

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Drainage Improvements for Central Mare Island Improvement Areas 4 and 6

Figure 4-6

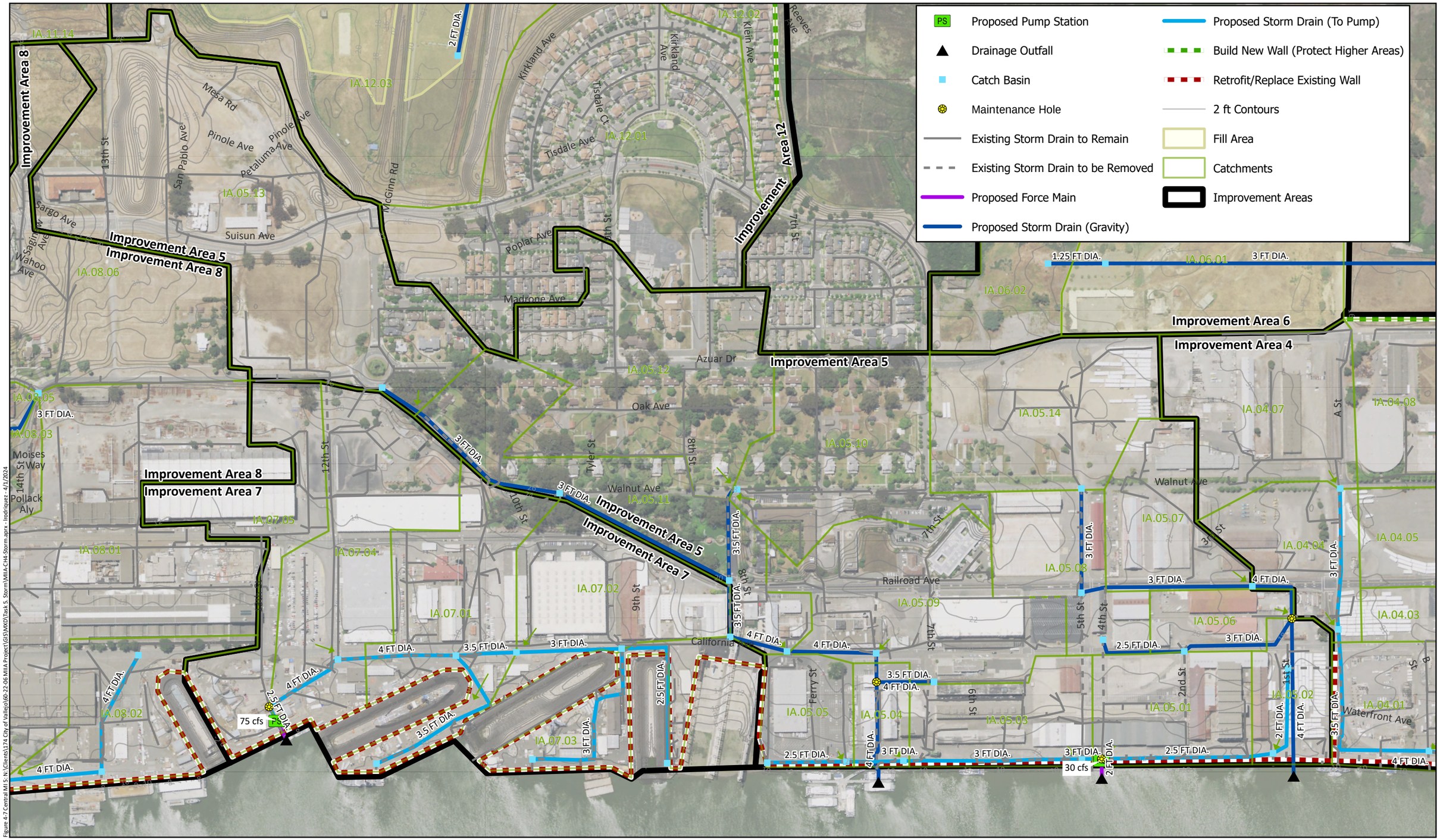

Figure 4-7 Central MI S:\Clients\174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - lrodriguez - 4/1/2024

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0 350 700 Feet

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3. The sizes and capacities of these facilities.

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Drainage Improvements for Central Mare Island Improvement Area 5

Figure 4-7

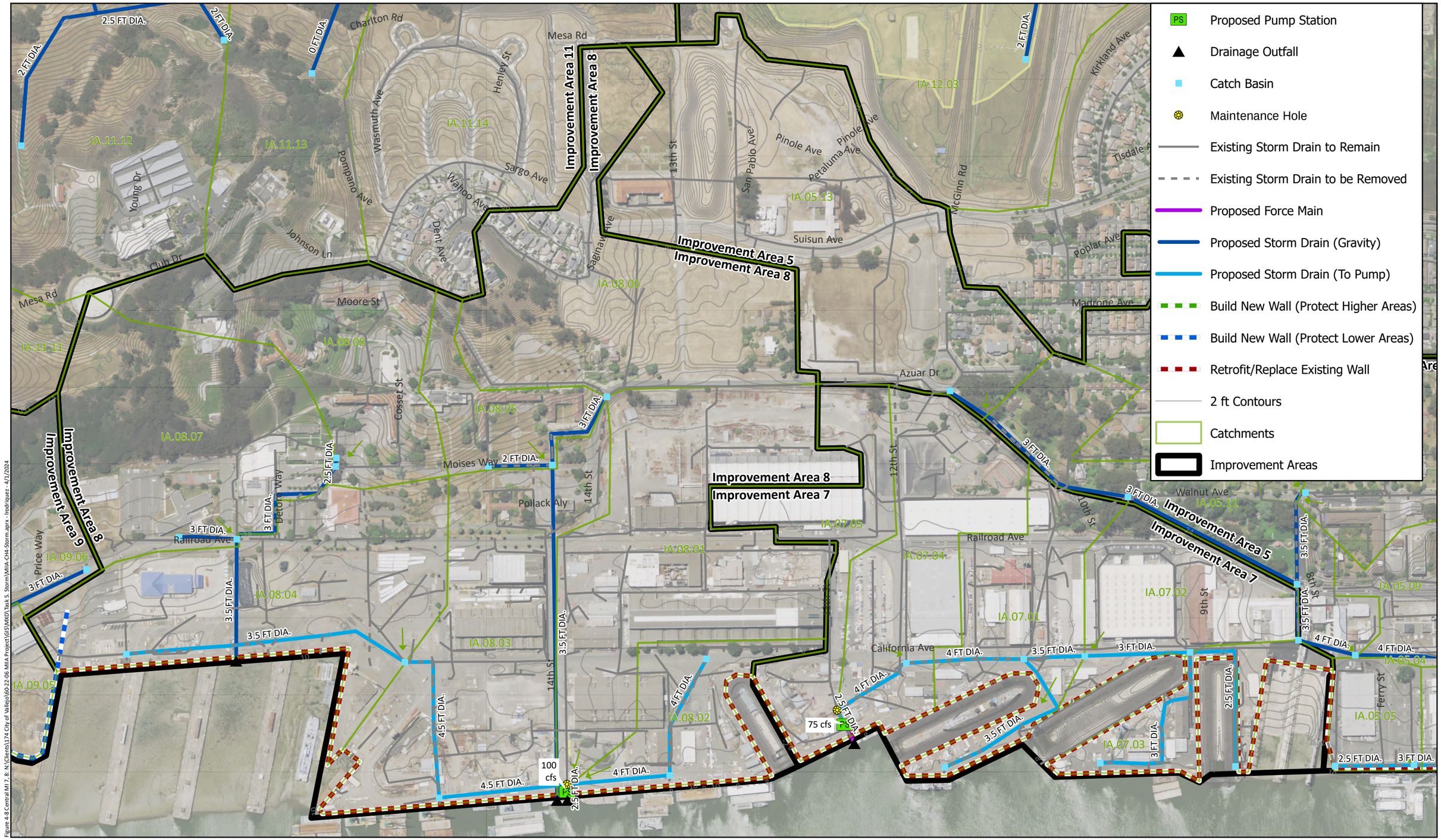

Figure 4-8 Central M17_8: N:\Clients\174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - lrodriquez - 4/1/2024

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WEST YOST

0 350 700 Feet

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3. The sizes and capacities of these facilities.

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Drainage Improvements for Central Mare Island Improvement Areas 7 and 8

Figure 4-8

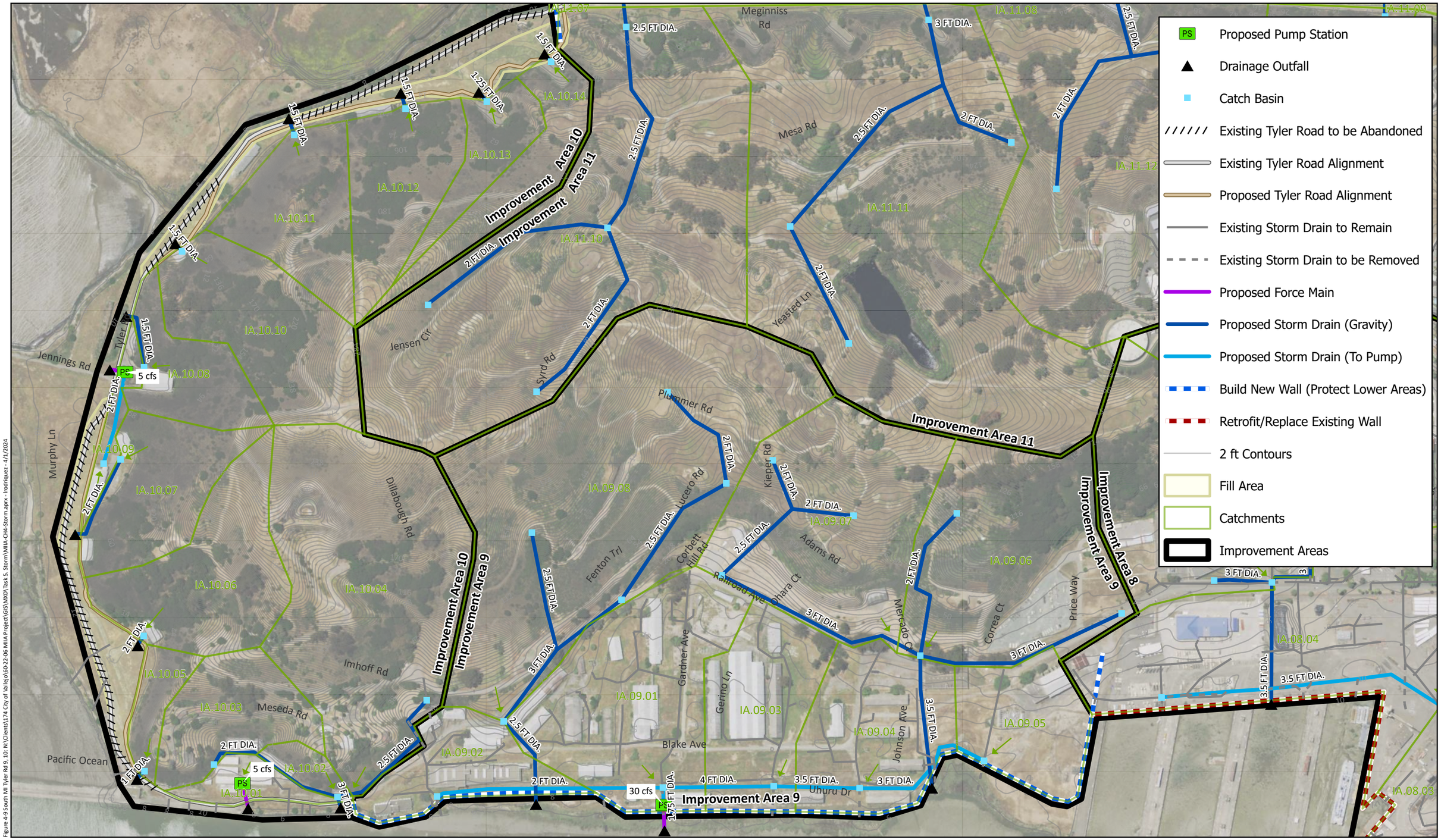

Figure 4-9 South Mt Tyler Rd 9, 10: N:\Clients\174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - lrodriquez - 4/1/2024

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At the time of design of these Sea Level Rise (SLR) facilities, the following should be verified:
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2. This approach for SLR as an individual SLR facility and in coordination with other SLR facilities previously constructed and proposed for Mare Island.
3. The sizes and capacities of these facilities.

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Drainage Improvements for South Mare Island and Tyler Road Improvement Areas 9 and 10

Figure 4-9

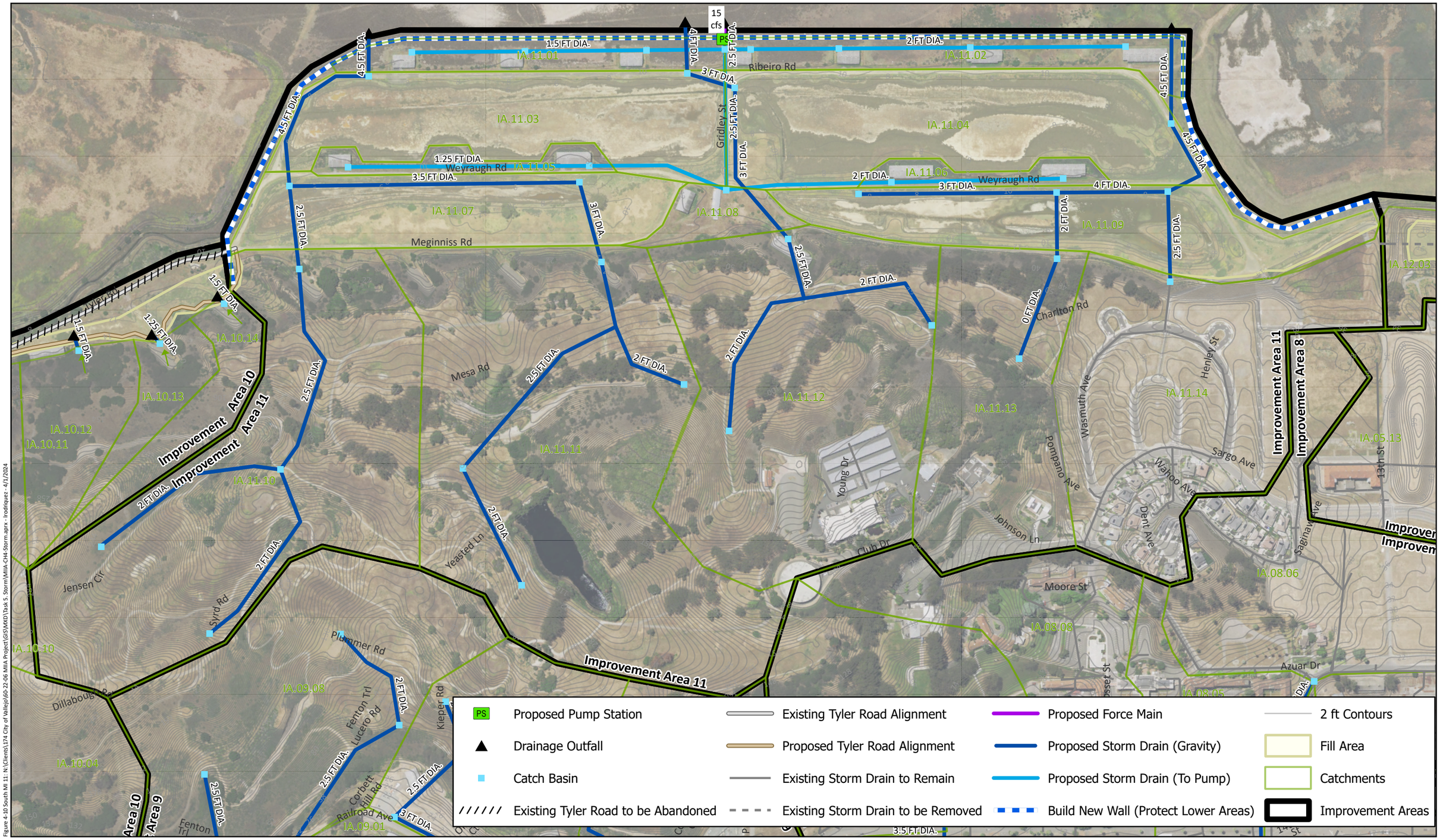

Figure 4-10 South MI 11: N:\Clients\174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5 Storm\MIIA-CH4-Storm.aprx - lrodriquez - 4/1/2024

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0 350 700 Feet

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3. The sizes and capacities of these facilities.

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Mare Island
Infrastructure Assessment

Drainage Improvements for South Mare Island
Improvement Area 11
Figure 4-10

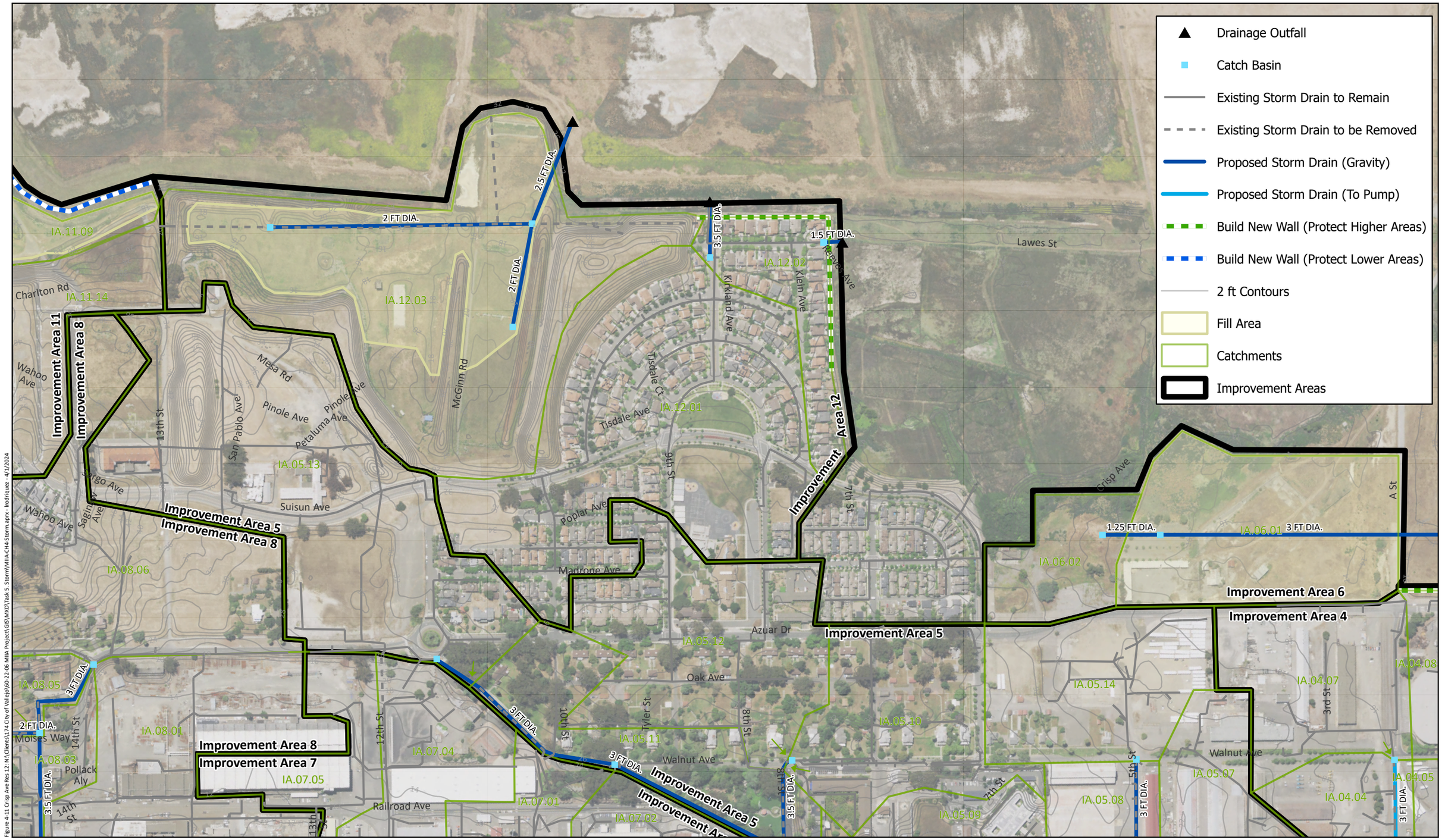

Figure 4-11 Crisp Ave Res 12 - N:\Client\L174 City of Vallejo\60-22-06 MIIA Project\GIS\MXD\Task 5. Storm\MIIA-CH4-Storm.aprx - lrodriguez - 4/1/2024

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Mare Island
Infrastructure Assessment

Drainage Improvements for South Mare Island
Improvement Area 12

Figure 4-11

CHAPTER 5
Sanitary Sewer System

5.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

The tasks completed for the sanitary sewer system evaluation included:

- Collecting and reviewing existing documents and mapping
- Performing condition assessment of select sanitary sewer system pipelines, maintenance holes, and lift stations.
- Developing proposed design criteria and preferences to be used in development of infrastructure improvements.
- Developing a hydraulic model to evaluate the capacity needs of the proposed sanitary sewer system infrastructure improvements.
- Developing an integrated set of recommendations for the existing/near-term (through 2027) and build-out (through 2063) development scenarios.

A summary of the findings and conclusions for the sanitary sewer system evaluations are listed below.

- The existing sanitary sewer system consists of a network of 22.6 miles of gravity pipelines, maintenance holes, nine lift stations, and force mains which convey wastewater generally from the south and north ends of the island to the DOM4 lift station. Wastewater is pumped off the island by DOM4 through a force main that runs under the Mare Island Strait/Napa River and connects to the mainland trunk sewer system. On the mainland, the District's north interceptor connects to the Ryder Street wastewater treatment plant (WWTP).
  - Although the capacity of the DOM4 lift station is 10.2 mgd, it is currently limited to a flow of 3.9 mgd based on current capacity limitations of the downstream collection system and the District's WWTP.
- Existing data was received including mapping, as-builts, and existing reports. Existing data sources were used to develop a comprehensive geospatial map of the existing sanitary sewer system, including asset attributes (material, installation date, diameter, etc.). Field verification of mapping was performed through visual location and geo-location of maintenance holes.
- Existing system pipelines were summarized by attribute, including diameter, material, and age.
  - Approximately 28 percent of the system by length is vitrified clay pipe (VCP); the next highest material by length is PVC at approximately 20 percent. Material is unknown for 11 percent of the system.
  - The age of installation was not directly available for approximately 71 percent of system by length, but it was assumed that pipes are at least as old as the island's lift stations, most of which were installed around 1959.
- An age-based condition assessment was performed on sewer pipelines and maintenance holes. The assessment utilized an estimated useful life specific to each asset type and each asset's installation date to calculate an estimated remaining useful life.
  - Based on this assessment, it is expected that approximately 17 percent (3.9 miles) of the pipelines by length are currently in poor condition.

- Closed-circuit television (CCTV) inspection was performed on 70 sewer segments (12,975 feet) which indicate that 11 segments have severe structural defects (1,463 feet).
- Visual condition assessment was performed on 120 maintenance holes (16 percent of all known manholes) which indicates that 102 maintenance holes have at least one component that is defective.
- Smoke testing was performed on all areas of the island with known sanitary sewer systems (active or otherwise), which identified 14 severe and 44 other defects in the sanitary sewer system, which contribute to inflow and infiltration (I&I).
- Visual condition assessment was performed in the field for the nine active domestic lift stations on Mare Island: DOM2, DOM3, DOM4, DOM6, DOM7, DOM8, DOM11, DOM13, and the L Street lift station. Improvement recommendations were developed based on observed condition and defects and were grouped into three categories: Priority 1 which are critical to current operation and Priority 2 for continued operation beyond 2027. Seven of the DOMs require improvements which are critical to current operation, which includes replacement of pumps and motors, rehabilitation of wet wells, and replacement of electrical and instrumentation components. It is expected that all DOMs will require full replacement or rehabilitation before build-out (2063). All facilities have space available on-site for future expansion.
- Flow monitoring was performed for a total of ten weeks, which was used to establish a baseline for sanitary sewer flows at each monitoring site, measure peak flow characteristics, isolate and analyze I&I, and monitor the level and pumping cycles at DOM4.
  - I&I analysis estimated I&I rates ranging from 6,300 gallons per day (gpd) per acre to 17,700 gpd per acre for the three flow monitoring basins. For reference, the District I&I allowances for pipes built before 1970 is 4,000 gpd per acre.
- A hydraulic capacity and performance evaluation was performed which included development of sewer flows for the existing, near-term, and build-out land use scenarios. Data from flow monitoring and historical DOM pumping records were used to calibrate the model. The calibrated hydraulic model was used to simulate the 5-year, 24-hour peak wet weather flow (PWWF) conditions in the existing, near-term (2027), and build-out (2063) systems. The District's existing design and performance criteria were used as a basis to identify deficiencies and recommended improvements.
  - In the existing condition and near-term scenario, model results indicate peak flows reaching DOM4 are 4.9 mgd, which causes local surcharging and backflow that propagates into other areas of the system, causing modeled sanitary sewer overflows (or SSOs, defined as water surface rising above the ground surface).
    - Based on information provided by the District and Mare Island Company, there have been no known SSOs on the island. It is possible that abandoned parts of the system, particularly on North Mare Island, may be acting as storage during surcharging periods, which has prevented above-ground surcharging.
  - The build-out scenario indicates capacity issues in similar locations to the existing and near-term system results, but with a higher severity magnitude. Peak flows reaching DOM4 at build-out are 5.0 mgd.

- The key factors contributing to the capacity deficiencies in the Mare Island collection system are the high I&I rates across the island and the DOM4 flow limit of 3.9 mgd. Although the DOM4 lift station has a capacity of 10.2 mgd, the DOM4 lift station is currently limited to a flow of 3.9 mgd and cannot be increased in the near-term based on limitations downstream in the collection system and at the District WWTP. For this reason, addressing capacity issues in the existing and near-term scenarios focused on reduction of I&I as opposed to increasing the DOM4 pumping rate. Reducing I&I is also in alignment with State Water Board guidance, which discourages conveying stormwater or groundwater infiltration to treatment facilities.
  - The District's WWTP faces several constraints related to capacity and the feasibility of upgrades. Any expansion would require a new NPDES permit, a process made more complex and time-consuming due to the plant's operation as a blending facility. Increasing treatment capacity would require substantial upgrades to critical systems, including headworks, primary treatment, and secondary treatment. Furthermore, compliance with evolving California regulatory requirements for nutrient removal (e.g., nitrogen and phosphorus limits) would necessitate advanced tertiary treatment processes. These upgrades could exceed the physical footprint of the existing site, further limiting the viability of capacity expansion.
  - Although capacity expansion is not feasible in the near term, potential WWTP upgrades may need to be revisited in the long term, particularly under the build-out (2063) scenario, depending on the success of island-wide I&I reduction efforts and actual development progression. Planning for such upgrades should be informed by ongoing flow monitoring, regulatory developments, and system performance tracking.
  - Capacity improvement recommendations developed based on three distinct approaches each involving different combinations of I&I reduction and adjustments to the DOM4 lift station flow capacity.

- Recommendations from the condition assessment and hydraulic capacity evaluation were integrated into a set of recommendations for each land use scenario.
  - Existing and near-term improvement recommendations include:
    - Eliminate severe inflow defects and cross connections to sanitary sewer identified in smoke testing (14 severe locations and 4 cross connections). Addressing these defects and cross connections will provide the greatest reduction in I&I.
    - Replace or rehabilitate pipelines that were identified to have PACP Grade 4 or 5 structural defects (11 pipelines; 1,463 feet). Rehabilitate maintenance holes in-line or adjacent to pipelines that are replaced or rehabilitated.
    - Replace all brick pipelines (437 feet known) and maintenance holes (18 known).
    - Implement lift station Priority 1 improvement recommendations (DOM 2, 3, 4, 6, 7, 8, L Street).
    - Complete condition assessment on all pipelines greater than 60 years old (see Figure 5-4) that were not inspected as part of this project. Visually assess associated maintenance holes concurrently. Address severe defects as they are identified.
    - Regarding capacity, implementation of pipeline rehabilitation, defect elimination, and cross-connection removals is expected to result in a measurable I&I reduction, helping to alleviate capacity constraints in the system.

- Once near-term rehabilitation and inflow elimination projects are complete, system capacity should be re-evaluated using updated flow monitoring data and development-based flow projections. This will establish a meaningful post-rehabilitation baseline and inform whether further improvements are necessary.

— Build-out improvement recommendations are required to support future development and include:

- Replace or rehabilitate pipelines that were identified to have Grade 3 structural defects (14 pipelines, 3,134 feet) as part of this project (recommended within 5 years – by 2029 or earlier), and pipes that were inspected after this project was concluded.
- Implement a regular preventative maintenance program, including pipeline cleaning and condition assessment. Perform condition assessment of all pipes every 7 years (minimum) to identify structural and maintenance deficiencies.
- For continued lift station operation beyond 2027, implement lift station Priority 2 improvement recommendations (DOM 2, 3, 4, 6, 7, 8). Full replacement or upgrades of stations may be required by 2063, the extents of which should be determined during project-level design evaluations. Preliminary evaluation indicates adequate space exists at all lift station sites for expansion.
- Regarding capacity, continued pipeline rehabilitation and replacement throughout the build-out period is expected to result in significant reductions in I&I, further relieving capacity constraints in the system and supporting long-term development.
- System capacity should be re-evaluated following completion of major rehabilitation efforts using updated flow monitoring and development-based flow projections. This re-evaluation will provide a critical baseline for determining whether additional capacity improvements – such as increasing the DOM4 flow limit – are warranted.
- If a formal capacity re-evaluation is not conducted, one of the alternative improvement approaches outlined in Section 5.7.10 should be implemented. These range from continued I&I reduction efforts to increasing the DOM4 flow limit. Increases to DOM4 may require upgrades to the District WWTP.

— Future system capacity needs will be influenced by uncertainties such as land use changes, development pace, I&I reduction success, climate change, and regulatory shifts. A proactive, adaptive management approach – including regular flow monitoring, updated projections, and coordination with planning – will be essential to ensure the system meets service needs as conditions evolve.

5.2 PURPOSE

This chapter describes the infrastructure assessment of the Mare Island sanitary sewer system. The assessment included several steps, as listed below and further described in the following sections:

- Existing Data – collection and review of existing data sources.
- Existing System Mapping – development of a consolidated sewer system GIS map.
- Existing System Description – description of the existing sanitary sewer system configuration and components.
- Condition and Useful Life Assessment – evaluation of existing condition based on asset attributes and select testing including development of useful life (UL) and remaining useful life (RUL) estimates; development of age-based replacements; and condition assessment (CCTV and smoke testing).
- Capacity and Performance Evaluation – expansion of the District's hydraulic model to include the Mare Island system and evaluation of the system using the expanded hydraulic model to identify capacity-related deficiencies and determine proposed improvement recommendations.
- Recommendations – integration of condition and capacity recommendations in terms of serving the two land-use scenarios (near-term and build-out).

5.3 EXISTING DATA

Table 5-1 summaries the data received from the Client Team to support the sanitary sewer system assessment. The existing data was used to the maximum extent possible to minimize duplication of previous work efforts.

Table 5-1. Existing Data Summary

Data Source	Details
Sewer System GIS – Mare Island "MI" Layers, provided March 2023 by the District	Included the Mare Island sewer system. Relevant feature classes and attributes included: • Sewer pipes (lateral or backbone) • Sewer nodes (maintenance hole or clean out)
Sewer System GIS – "VFWD District-wide" Sewer System, provided March 2023 by the District	Included the entire District-wide sewer system, and newer developed areas of Mare Island only. Relevant feature classes and attributes included: • Sewer pipes (attributes: pipe ID, upstream and downstream node, length, diameter, material, installation date) • Sewer nodes (attributes: node ID, type (e.g., maintenance hole, clean out, or pump station), rim elevation, depth, material, installation date)
Navy Sanitary Sewer Quad Maps from Mare Island Company (MIC)	• Created circa 1980s • Shows alignment, material, and diameter of existing and abandoned sewer system pipelines within Mare Island.
Sewer system improvements from District as-builts	• Mare Island Sewer and Water Main Replacement Project (2018) • Coal Sheds – Phase 2 (2017) • Mare Island Way Sewer Rehab (2016) • North Bay Operations and Maintenance Facility Landside Phase 1 – DOM6 Force main Re-route (Water Emergency Transportation Authority, 2014) • DOM4 Force Main Repair Project (2014) • DOM11 and DOM13 record drawings (2008 and 2017 respectively) • Walnut Avenue and "E" Street (2007) • Railroad Avenue Phase 2 (2006) • Farragut Village – Unit No. 3 (2005) • Farragut Village – Unit No. 4 (2005) • Club Drive (2005) • Coral Sea South Village – Unit No. 1 (2005) • Flagship Drive – Phase 1B & 1C (2005) • Azuar Drive and Flagship Drive (2004) • Kansas Street (2004) • Azuar Drive from Connolly Street to Residential Parkway (2004) • DOM2, DOM3, DOM4, DOM6, DOM7, and DOM8 record drawings (1959)
Other Reports or Data from the District	• VFWD Sanitary Sewer Collection System Master Plan (2023, Carollo) • VFWD Engineering Standards (July 2020) • Mare Island Condition and Rehab Pricing report (2022, VFWD) • DOM2, DOM3, and DOM8 Field Investigation reports (2022, VFWD) • DOM4 Replacement Assessment TM (2022, HydroScience) • Mare Island Sanitary Sewer Pump Stations report (2020, VFWD) • Ship-to-Shore Listing (VFWD) • Ship Wastewater Collection Schematic (no date, Kennedy Engineers) • Ship Wastewater Collection Ashore Pump Station O drawings and report (no date, Kennedy Engineers) • Lift Station flow data for March 8 to May 9, 2023 & April 16 to May 2, 2024 • Lift Station draw down test results from March 2022

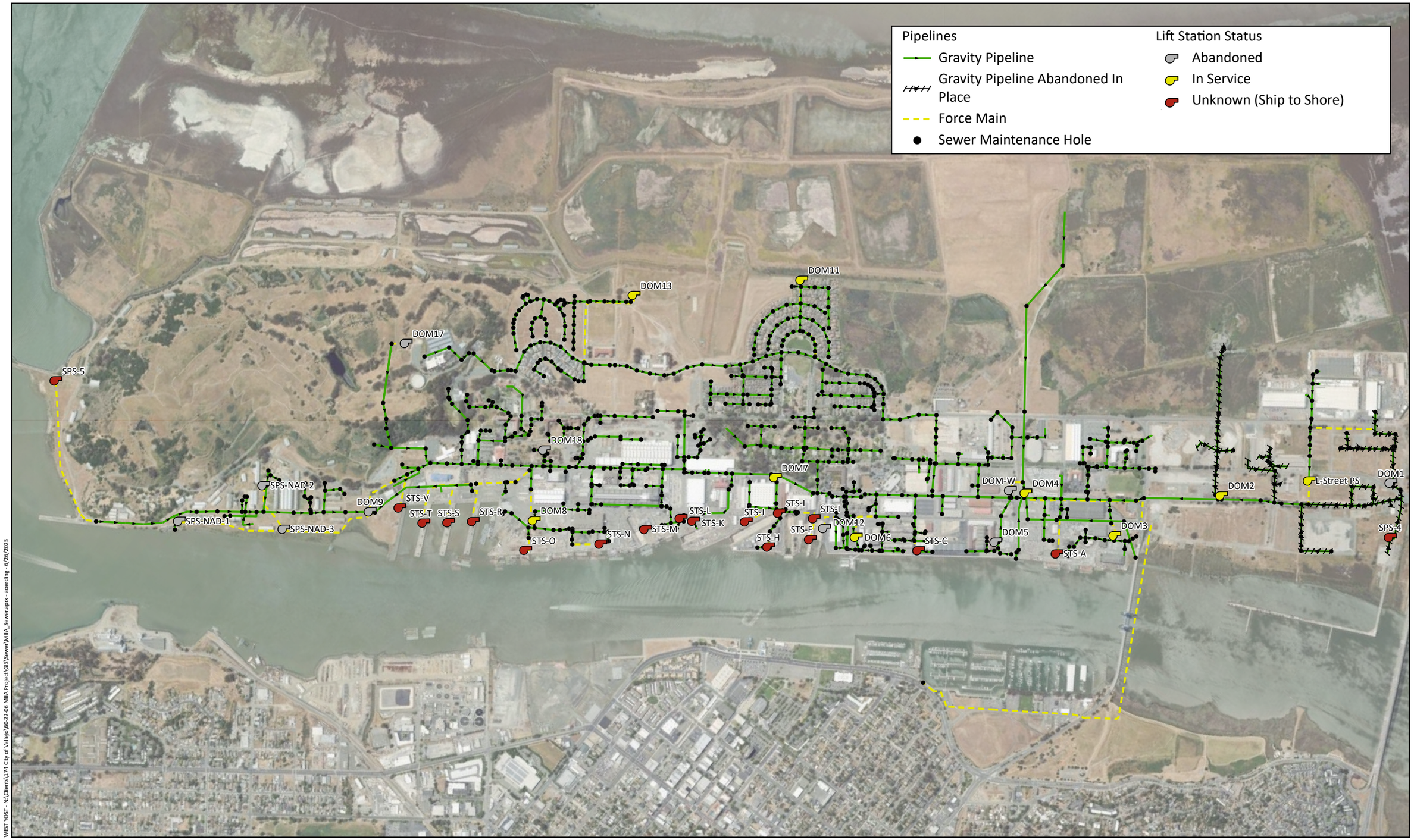
5.4 EXISTING SYSTEM MAPPING

Several data sources were reviewed and consolidated to produce the existing system mapping and attributes. Throughout the consolidation process, the District was consulted to confirm status of certain assets, GIS attribute field preferences, and hierarchy of available data. The process undertaken is described below:

1. A copy of the District's sanitary sewer system GIS database was created for verifying and editing sewer system features.
    a. The attributes in the GIS database copy were maintained from the "VFWD District-wide" GIS.
2. "VFWD District-wide" and "MI" GIS files were combined based on the following logic:
    a. The "VFWD District-wide" GIS files included the entire mainland VFWD system, and only certain areas of the Mare Island system – newer residential areas and the Railroad Avenue trunk (from approximately Azure Avenue to the Mare Island Causeway).
    b. The MI files were more comprehensive (covered more of the island) but were older than the VFWD District-wide GIS (i.e., the newer residential areas had not been updated).
    c. The MI GIS files were used as the basis, and were superseded by the VFWD District-wide GIS files in areas of overlap.
3. Navy Quad Maps were reviewed to confirm geometry and other attribute data from the combined GIS data. Navy Quad Map data was used to populate GIS data for manhole rim, pipe inverts, pipe diameter, and pipe material. County topographic data was used to populate missing manhole rim elevations.
4. Recent as-builts for Mare Island's sewer system were compiled and used to confirm the combined and quad-map verified GIS, and used to populate any missing attributes.
    a. The pipe and manhole attributes shown on the Navy Quad Maps were concluded to be more accurate compared to the "VFWD District-wide" or "MI" GIS attributes, except in the newer residential areas.
    b. Pipeline alignment (geometry) in GIS was verified and corrected as needed based on the Navy Quad Maps and as-builts.

5.4.1 Field Verification

After the mapping data consolidation was complete, field verification of the system was performed. Maintenance holes were first visually field located, then geo-located using centimeter-level accuracy GPS and logged via ESRI's ArcGIS Collector software into an online database. Using this field verified data, a dynamic, real-time GIS map of the District's collection system was created. A total of 139 maintenance holes were geo-located and their locations updated in the GIS database. Of the 139 maintenance holes surveyed, 120 were inspected and photographed to document condition. The resulting existing sewer system map is shown on Figure 5-1.



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Notes:
1. VFWD sewer system geodatabase provided by VFWD in 2023. Modified based on data available in Navy Quad maps, record drawings, and field verification.
2. Only Mare Island sewer system facilities are displayed for clarity.

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Existing Sewer System

Figure 5-1

5.5 EXISTING SYSTEM DESCRIPTION

The District provides sanitary sewer service to Mare Island through a network of gravity pipelines, lift stations, and force mains. Wastewater generally flows from the south end of the island to the north, where it is pumped off the island via a force main that crosses beneath the Mare Island Strait/Napa River north of the Causeway alignment. Once on the mainland, the flow enters the District's north interceptor, which conveys it to the Ryder Street WWTP. Figure 5-1 shows Mare Island's existing key sanitary sewer facilities including pipelines, maintenance holes, lift stations, and force mains.

The District currently operates and maintains the sewer system infrastructure.

5.5.1 Gravity Pipelines

The existing sewer system consists of approximately 22.6 miles of gravity pipelines, with diameters ranging from 4- to 39-inches. Pipelines that were plugged and abandoned in place were not included in this assessment (abandoned pipes include approximately 10,300 feet of pipeline located upstream of DOM2 lift station; see Figure 5-1). By length, approximately 75 percent of the Mare Island's pipelines are between 6- and 12-inch diameter. Vitrified Clay Pipe (VCP), and Polyvinyl Chloride (PVC) are Mare Island's two most common pipe materials, making up 28 and 20 percent of the sewer system, respectively.

The diameter and material distributions of Mare Island's sanitary sewer system gravity pipelines are summarized in Table 5-2 and Table 5-3, respectively. The diameter and material distributions are presented on Figures 5-2 and Figure 5-3, respectively.

Table 5-2. Sanitary Sewer Gravity Pipelines by Diameter

Pipe Diameter, inches	Length of Pipelines, ft	Length of Pipelines, miles (Rounded)	Percent of System, %
4-6	12,457	2.4	10.4
8-10	55,919	10.6	46.8
12-15	27,889	5.3	23.3
18-27	14,764	2.8	12.4
39	437	0.1	0.4
Unknown[(a)]	7,957	1.5	6.7
Total	**119,424**	**22.6**	**100%**

Source: VFWD Sewer GIS, March 2023 with best available information.

(a) Pipes with unknown diameter are located on the south end of the island and are likely 6-inch diameter or smaller (see Figure 5-2).

Table 5-3. Sanitary Sewer Gravity Pipelines by Material

Pipe Material	Length of Pipelines, ft	Length of Pipelines, miles (Rounded)	Percent of System, %
Asphalt Coated Corrugated Metal Pipe (ACCMP)	18,175	3.4	15.2
Brick	437	0.1	0.4
Cast Iron (CI)	2,516	0.5	2.1
Cured-in-Place Pipe (CIPP)	9,032	1.7	7.6
Corrugated Metal Pipe (CMP)	1,009	0.2	0.8
Concrete	87	0.0	0.1
Polyethylene (PE)	342	0.1	0.3
Polyvinyl chloride (PVC)	24,136	4.5	20.2
Reinforced Concrete Pipe (RCP)	8,832	1.7	7.4
Steel	3,948	0.7	3.3
Terra Cotta (TC)	3,433	0.7	2.9
Vitrified Clay Pipe (VCP)	33,579	6.4	28.1
Unknown	13,898	2.7	11.6
Total	**119,424**	**22.6**	**100%**

Source: VFWD Sewer GIS, March 2023 updated with best available information.

Date of installation data was not directly available for the majority of the gravity pipelines (84,398 feet, or approximately 71 percent of the system by length). Installation dates were available for newer residential areas (Flagship Drive residential areas of Coral Sea Circle and Crescent Park) and the Railroad Avenue trunk main (Azure Avenue to the Mare Island Causeway). The following assumptions were made regarding pipe age for gravity pipelines without installation dates:

- Pipes are at least as old as the Navy Quad Maps, dated 1982.
- Pipes are at least as old as the Mare Island lift stations. DOM2, DOM3, DOM4, DOM6, DOM7, and DOM8 were installed in 1959. Much of system requires lift stations to operate and the pipelines would not have been installed unless the lift stations existed.

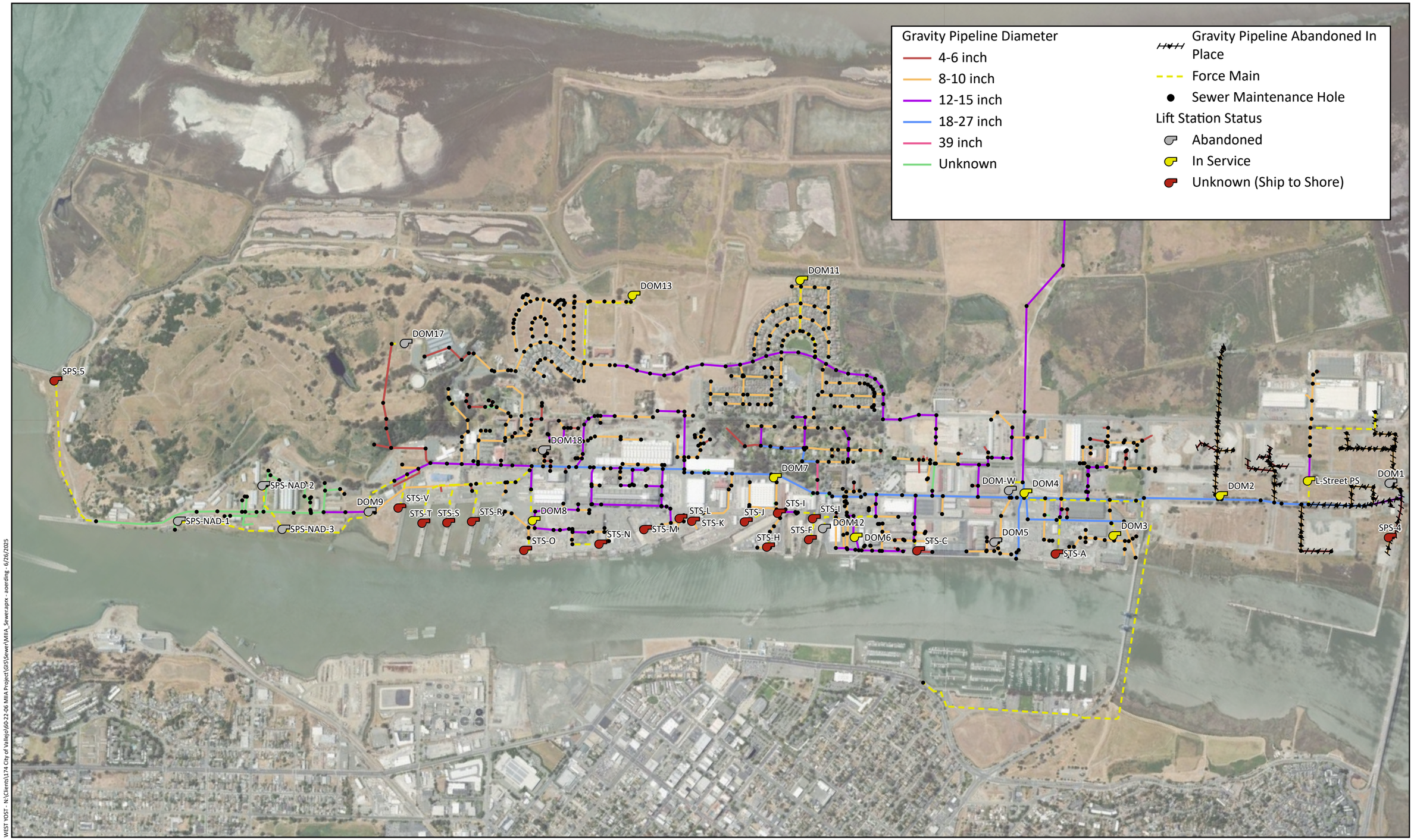
Based on these assumptions, pipes without installation dates were assigned an installation date of 1959. The age distribution of Mare Island's sanitary sewer system gravity pipelines is summarized in Table 5-4 and Figure 5-4. Note there are assumed to be no pipes between the ages of 25 and 60 years old.

Table 5-4. Sanitary Sewer Gravity Pipelines by Age

Pipe Age	Length of Pipelines, ft	Length of Pipelines, miles (Rounded)	Percent of System, %
Less than 25 years old (1998-Present)	34,058	6.5	28.5
Greater than 60 years old (1959 or older)[(a)]	85,366	16.2	71.5
Total	**119,424**	**22.6**	**100%**

Source: VFWD Sewer GIS, March 2023 updated with best available information.

(a) Includes 83,583 feet of pipeline with an assumed installation date of 1959.



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Notes:
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2. Only Mare Island sewer system facilities are displayed for clarity.

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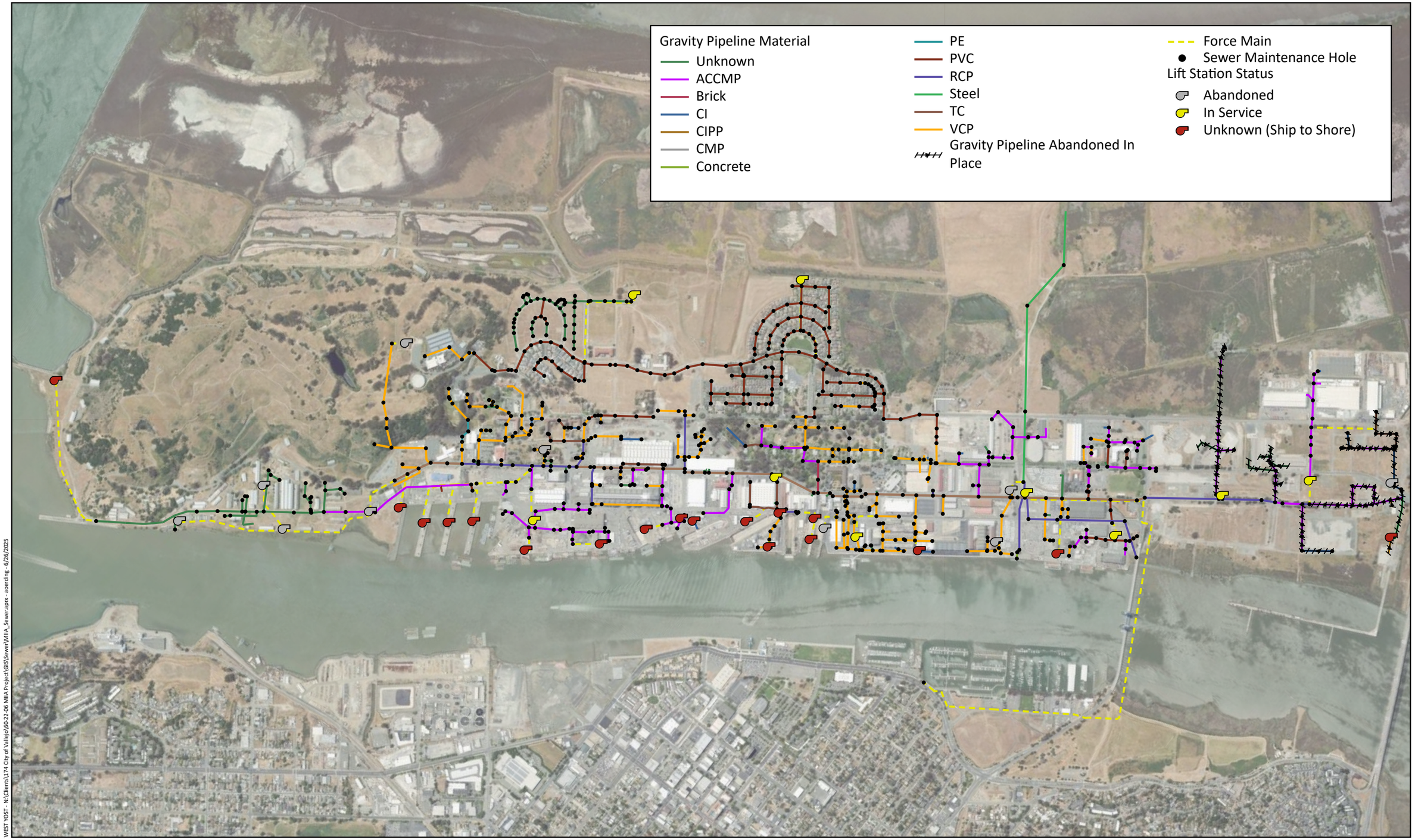
Existing Sewer System Pipeline Diameter

Figure 5-2



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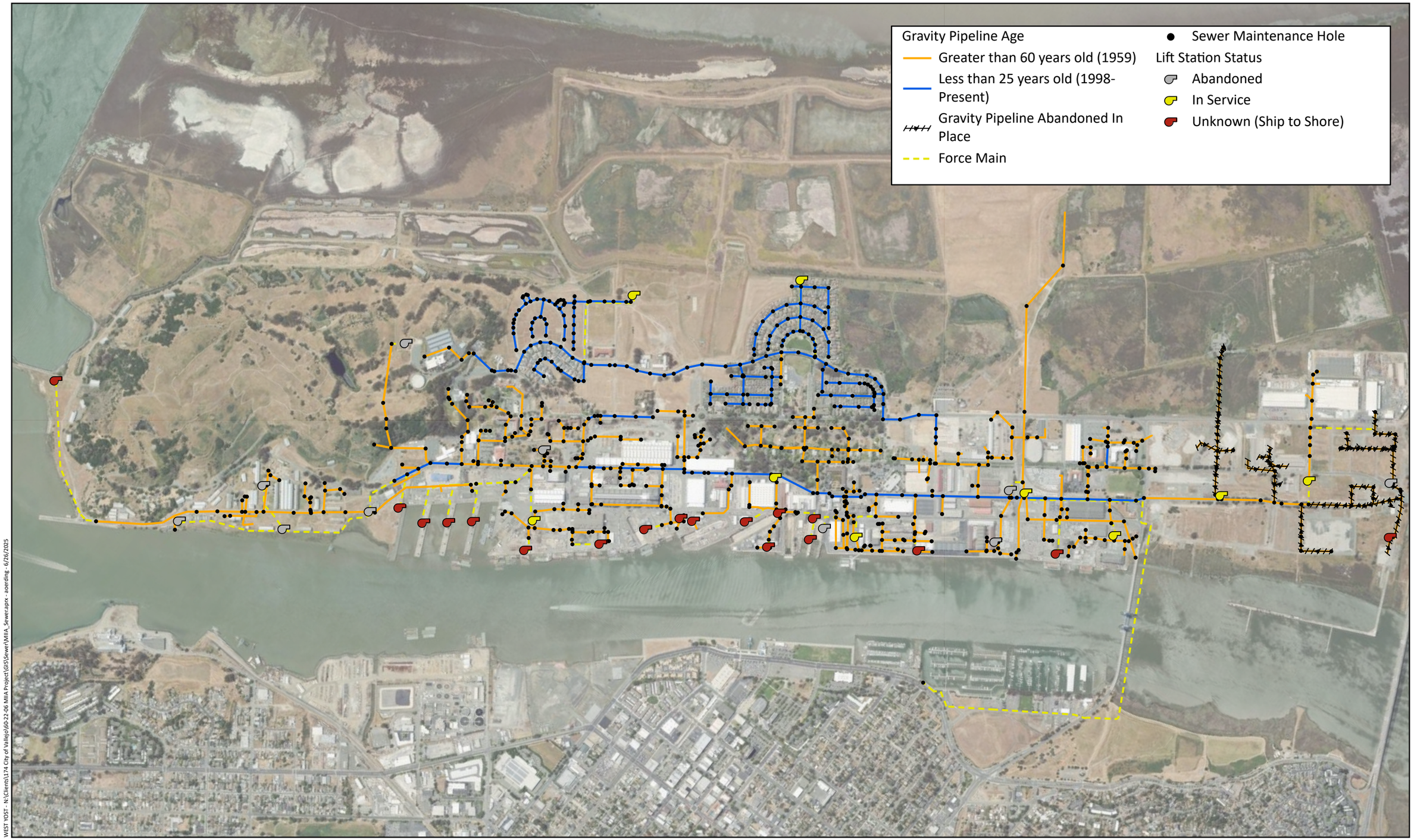
Existing Sewer System Pipeline Material

Figure 5-3



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Existing Sewer System Pipeline Age

Figure 5-4

5.5.2 Maintenance Holes

Based on available mapping, Mare Island's sewer system has approximately 754 maintenance holes. Maintenance hole materials include brick, concrete, and pre-cast concrete. Some maintenance holes have been rehabilitated with linings since their original installation. Table 5-5 and Figure 5-5 present Mare Island's sanitary sewer system maintenance holes by material.

Table 5-5. Sanitary Sewer Maintenance Holes by Material

Material	Count	Percent of Total Maintenance Holes, %
Brick	18	2.4
Pre-Cast	157	20.8
Lined	43	5.7
Unknown	536	71.1
Total	**754**	**100%**

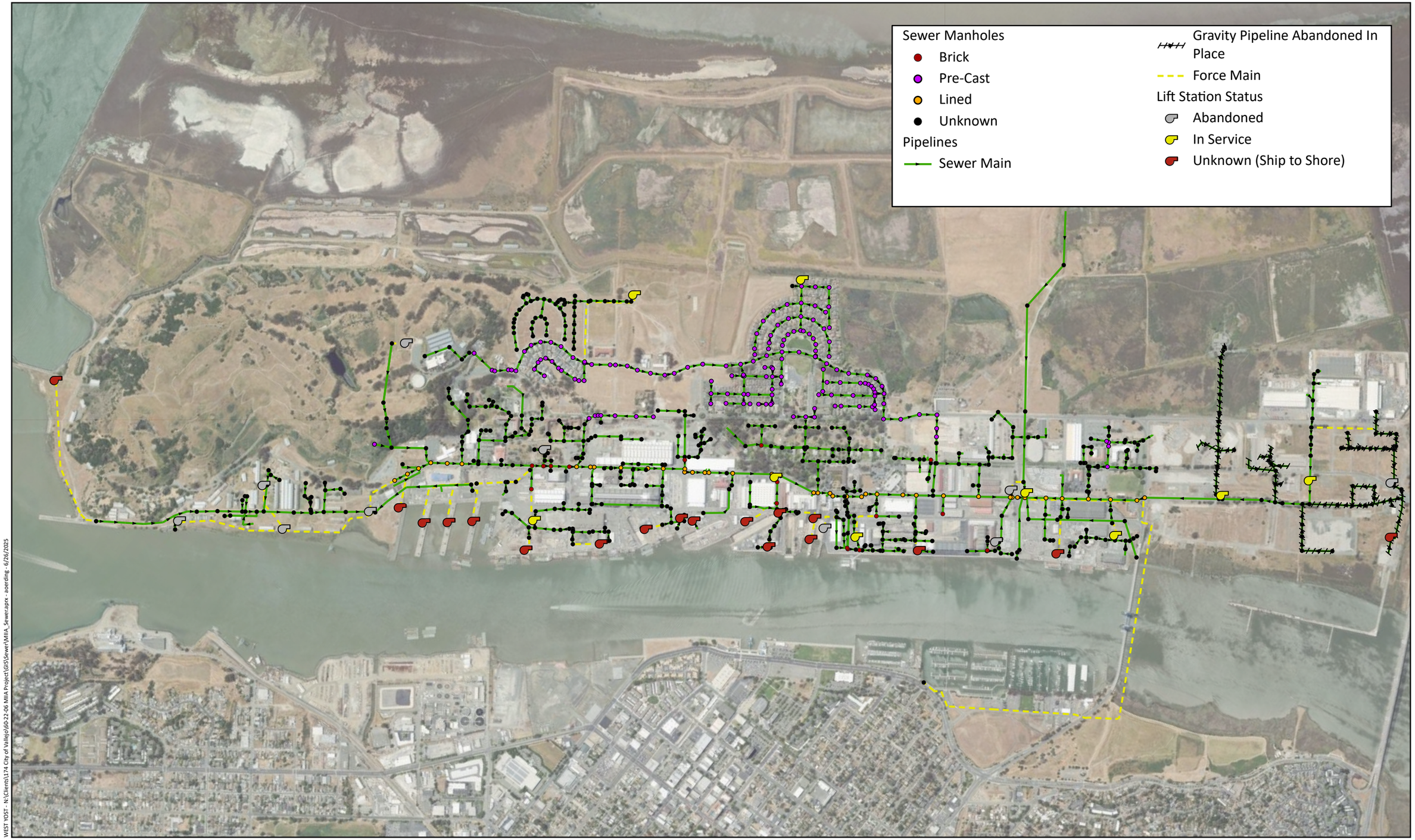
Source: VFWD Sewer GIS, March 2023 updated with best available information.

Age data was unavailable for most maintenance holes (497 maintenance holes, or approximately 66 percent of total maintenance holes). Maintenance holes with no age data are assumed to be the same age as connecting pipelines. As a result, the age distribution of maintenance holes is assumed to closely mirror that of the pipeline system. Table 5-6 summarizes Mare Island's sanitary sewer system maintenance holes by age.

Table 5-6. Sanitary Sewer Maintenance Holes by Age

Age	Count	Percent of Total Maintenance Holes, %
Less than 25 years old (1998-Present)	233	30.9
Greater than 60 years old (1959 or older)	521	69.1
Total	**754**	**100%**

Source: VFWD Sewer GIS, March 2023 updated with best available information.



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Existing Sewer System
Maintenance Hole Material

Figure 5-5

5.5.3 Lift Stations

The District currently operates nine active domestic lift stations (DOMs) on Mare Island: DOM2, DOM3, DOM4, DOM6, DOM7, DOM8, DOM11, DOM13, and the L Street lift station. Four additional DOMs are not in service and are considered abandoned (DOM1, DOM5, DOM9, DOM-W). In addition to the DOMs, the island also has several smaller, privately owned and operated lift stations called ship-to-shore (STS) stations. Based on available information, there are currently 22 STS stations, many of which are inactive or abandoned. The District does not currently operate or maintain any of the STS stations.

DOM4 is the largest station on the island and collects flow from the entire island. DOM4 pumps via a force main which runs under the Mare Island Strait/Napa River on the north side of the Causeway and connects to the mainland gravity trunk sewer system around Mare Island Way and Kentucky Street.

Table 5-7 presents an inventory of Mare Island domestic lift stations and their current status. Figure 5-1 presents the location, type, and status of the lift stations.

Table 5-7. Sanitary Sewer Lift Stations

Lift Station	Status	Original Installation Date
DOM1	Abandoned • Flow from Alco and EQS buildings still in use was re-routed via a private lift station at Alco through the L Street lift station.	1959
DOM2	Active	1959
DOM3	Active	1959
DOM4	Active	1959
DOM5	Abandoned • Flow from Coal Sheds was re-routed through a private lift station.	1959
DOM6	Active	1959
DOM7	Active	1959
DOM8	Active	1959
DOM9	Abandoned • Service area included vacant or demolished buildings along the waterfront and around the north point of Mare Island.	1959
DOM11 (Farragut)	Active	2008
DOM13 (Coral Sea)	Active (not started up as of 2024)	2017
DOM W	Abandoned • Lift station pumped industrial wastewater to the Navy's former treatment plant at the end of Dump Road.	1959
L Street	Active • Converted maintenance hole with submersible pumps.	1997

5.5.4 Force Mains

Each of the sewer lift stations has an associated force main, which conveys the pumped wastewater to the gravity collection system. Force mains associated with active lift stations are summarized in Table 5-8 and presented on Figure 5-1. Force mains are assumed to be the same age as their associated lift station, unless otherwise specified.

Table 5-8. Sanitary Sewer Force Mains

Associated Lift Station	Force Main Length, feet	Force Main Diameter, inches	Force Main Material	Force Main Installation Date
DOM2	76	15	ACCMP	Assumed 1959
DOM3	192	6	CI	Assumed 1959
DOM4	4,832	18	RCP/HDPE	Assumed 1959; portion from G Street to Mainland replaced in 2018
DOM6	541	8	Unknown	Assumed 1959
DOM7	85	14	CI	Assumed 1959
DOM8	452	12	ACCMP	Assumed 1959
DOM11 (Farragut)	1,204	4	PVC	Assumed 2008
DOM13 (Coral Sea)	1,564	4	HDPE	Assumed 2017
L Street	324	2.5	Unknown	Assumed 1997
Total	**9,270**			

Source: VFWD Sewer GIS, March 2023 updated with best available information.

5.6 CONDITION AND USEFUL LIFE ASSESSMENT

The sanitary sewer system condition assessment included a desktop evaluation of the gravity pipelines, maintenance holes, and force mains to assign an estimated remaining useful life and expected condition.

Field inspection and testing included CCTV of gravity pipelines, visual condition assessment of maintenance holes, and smoke testing to verify and support desktop assessment conclusions. The active DOMs were field inspected to assess current condition based on visual observations.

5.6.1 Assessment Criteria

Gravity pipeline and maintenance hole assets were assessed initially for their condition by calculating each asset's estimated RUL. RUL is calculated by subtracting an asset's age (how long the asset has been installed) from its estimated UL:

$$RUL = UL - age$$

This approach provides an initial determination of assumed condition and can be used to project estimated renewal needs. Installation dates were not available for many pipelines and maintenance holes, so age assumptions were made based on available information, as discussed in Section 5.5.

Based on their estimated RUL, assets were assigned an assumed condition from Excellent to Poor, as presented in Table 5-9. Newer assets with 50 percent or more of their useful life remaining are expected to be in good to excellent condition. Older assets with less than 50 percent of their useful life remaining are expected to be in fair to poor condition. The following sections detail the assumptions and criteria used to estimate useful life, remaining useful life, and assumed condition for each asset type.

Table 5-9. RUL to Condition Translation

Percent RUL, %	Assumed Condition
100 to 80	Excellent
79 to 50	Good
49 to 20	Fair
19 and under	Poor

5.6.2 Gravity Pipelines

Useful life for pipelines is estimated by pipe material, as presented in Table 5-10. Useful life values were developed based on a combination of industry standards and local guidance, including the District's *Collection System Asset Management Plan (2019)*, Table 4. Each pipeline is assigned an estimated useful life value.

Table 5-10. Sanitary Sewer Gravity Pipeline Estimated Useful Life

Pipe Material	Estimated Useful Life, Years
Asphalt Coated Corrugated Metal Pipe (ACCMP)	85
Brick	90
Cast Iron (CI)	70
Cured-in-Place Pipe (CIPP)	50
Corrugated Metal Pipe (CMP)	75
Concrete	100
Polyethylene (PE, HDPE)	70
Polyvinyl Chloride (PVC)	90
Reinforced Concrete Pipe (RCP)	90
Steel	70
Terra Cotta (TC)	70
Vitrified Clay Pipe (VCP)	90
Unknown	70

5.6.2.1 Gravity Pipeline Remaining Useful Life Summary

Table 5-11 summarizes the assumed condition of sanitary sewer gravity pipelines based on each asset's estimated RUL. Approximately 15 percent of Mare Island pipelines have greater than 80 percent of their useful life remaining and are expected to be in excellent condition. These are mostly pipelines in the newer residential areas and rehabilitated (CIPP) pipelines along Railroad Avenue. Approximately 17 percent of Mare Island pipelines have less than 19 percent of their useful life remaining and are expected to be in poor condition.

Table 5-11. Gravity Pipeline Condition Summary

Condition, %	Length of Pipelines, miles (Rounded)	Percent of System, %
Excellent (100 to 80 RUL)	3.4	15
Good (79 to 50 RUL)	3.4	15
Fair (49 to 20 RUL)	11.9	53
Poor (1 to 19 RUL)	3.9	17
Total	**22.6**	**100%**

5.6.2.2 Force Mains Remaining Useful Life Summary

Table 5-12 summarizes the assumed condition of sanitary sewer force mains based on estimated RUL.

Table 5-12. Force Main Condition Summary

Associated Lift Station	Material	Installation Date and (Age)	RUL, %	Assumed Condition
DOM2	ACCMP	Assumed 1959 (65 years)	24	Fair
DOM3	CI	Assumed 1959 (65 years)	7	Poor
DOM4	RCP/HDPE	Assumed 1959 (65 years); portion from G Street to Mainland replaced in 2018 (6 years)	28; 91	Fair; Excellent
DOM6	Unknown	Assumed 1959 (65 years)	7	Poor
DOM7	CI	Assumed 1959 (65 years)	7	Poor
DOM8	ACCMP	Assumed 1959 (65 years)	24	Fair
DOM11 (Farragut)	PVC	Assumed 2008 (16 years)	82	Excellent
DOM13 (Coral Sea)	HDPE	Assumed 2017 (7 years)	90	Excellent

5.6.2.3 Age-Based Replacement Projections

Based on the estimated RUL of each pipeline, an expected replacement year can be projected, providing insight into anticipated long-term replacement needs. These age-based projections provide a reasonable basis for long-term planning, and where possible, should be refined using direct condition data. Condition-based refinements are addressed in the remaining sections of this chapter, where information from CCTV inspection, smoke testing, and visual assessments are incorporated to develop a more targeted understanding of infrastructure rehabilitation needs.

Using the estimated RUL of the pipelines, a projection of replacement needs was developed to support the two land-use scenarios. Table 5-13 presents the age-based replacement projections for the near-term (by 2027) and build-out scenarios (by 2063). These projections assume that pipelines are replaced at the end of their estimated useful life (i.e., when RUL reaches zero).

As shown in Table 5-13, no replacements are projected in the near-term scenario, meaning no pipelines are expected to reach the end of their useful life. For the build-out scenario (by 2063), 17 miles of pipeline are expected to require replacement. Figure 5-6 presents the pipeline replacement projections spatially for the build-out scenario. It is expected that some pipelines that have reached the end of their useful life will not be replaced as they may have served areas of Mare Island that do not have a future need for sewer service. Such pipelines may instead be abandoned in place or removed as needed.

Table 5-13. Age-Based Sanitary Sewer Pipeline Replacement Projections

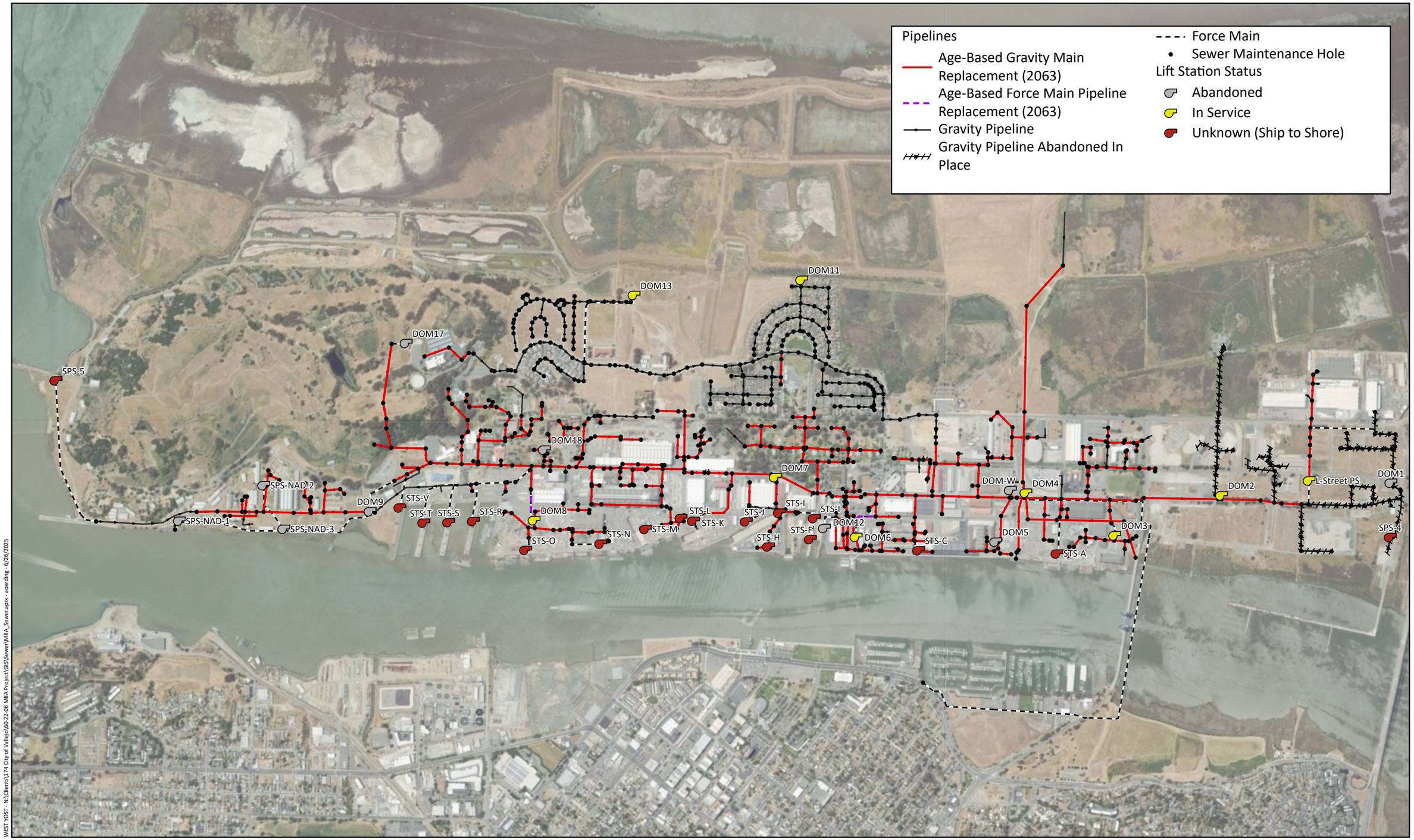
Scenario	Length of Pipelines, feet (miles) (Rounded)	Percent of System, %[(a)]
By 2027 (Near-Term)	No Replacements based on age	0
By 2063 (Build-Out)	92,486 (17.5 miles) Gravity Pipelines 1,345 (0.3 miles) Force Mains	77

(a) Does not include force main lengths.

5.6.2.1 CCTV Inspection

Closed-circuit television inspection of gravity pipelines was performed on 70 pipelines, or approximately 2.5 miles to identify location and severity of existing defects and validate the age-based replacement projections. Pipelines were selected based on importance within the collection system, available age and condition information, and Client Team feedback.

CCTV was performed using the National Association of Sewer System Companies (NASSCO) Pipeline Assessment Certification Program (PACP) assessment standards. PACP classifies pipe defects as either structural or operation and maintenance (O&M) related. Structural defects provide an indication of the condition and structural degradation of the pipe, and include defects such as holes, cracks, breaks, and fractures. O&M defects provide an indication of the operational condition of the pipe and are related to maintenance issues such as deposits (grease, sediment), roots, or inflow and infiltration (I&I). All identified defects are assigned a corresponding defect grade based on their risk of further deterioration or failure (from least severe – Grade 1, to most severe – Grade 5). Defect Grade severity definitions for structural and O&M defects are presented in Table 5-14. Further, each pipe is assigned an overall structural and O&M Grade between 1 and 5 based on the maximum Grade defect observed.



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Notes:
1. VFWD sewer system geodatabase provided by VFWD in 2023. Modified based on data available in Navy Quad maps, record drawings, and field verification.
2. Only Mare Island sewer system facilities are displayed for clarity.

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2063 Age-Based Pipeline Replacement Projections Sewer System

Figure 5-6

Table 5-14. PACP Defect Grade Definitions[1]	
Defect Grade	Definition
Grade 5	Pipe segment has failed or will likely fail within the next five years - requires immediate attention. Pipe segment has more than thirty percent (by pipe area) of flow restricted.
Grade 4	Pipe segment has severe defects - risk of failure within the next five to ten years. Pipe segment has between twenty to thirty percent of flow restricted.
Grade 3	Pipe segment has moderate defects - deterioration may continue, at a ten to twenty-year timeframe. Pipe segment has between ten and twenty percent of flow restricted.
Grade 2	Pipe segment has minor defects - pipe unlikely to fail for at least 20 years. Pipe segment has less than ten percent of flow restricted.
Grade 1	Pipe segment has minor defects - failure unlikely in the foreseeable future.

Tables 5-15 and 5-16 present a summary of the pipeline inspection results by PACP structural or O&M defect Grade. About 62 percent of inspected pipelines have no structural defects. Approximately 35 percent of assessed pipelines have Grade 3 or higher structural defects. The most prevalent structural defects are offset joints, broken pipes, and cracks. Many CIPP-lined pipelines (Railroad Avenue) have liner wrinkling and creasing, which are classified as Grade 3 defects. Figure 5-7 presents two instances of this type of defect for reference. Figure 5-8 presents the CCTV status and structural condition of the pipelines, with each pipe symbolized by its maximum structural defect grade.

Table 5-15. Pipeline CCTV – Structural Defects			
Maximum Structural Defect Grade	Count of Pipelines	Length of Pipelines, feet	Percent by Length, %
Grade 5	8	833	6
Grade 4	3	630	5
Grade 3	14	3,134	24
Grade 2	2	319	3
Grade 1	0	0	0
No Structural Defects	43	8,059	62
Total	**70**	**12,975**	**100%**

About 57 percent of inspected pipelines have no O&M defects. The most prevalent O&M defects are deposits or debris (in some cases bricks which are likely dislodged from brick maintenance holes).

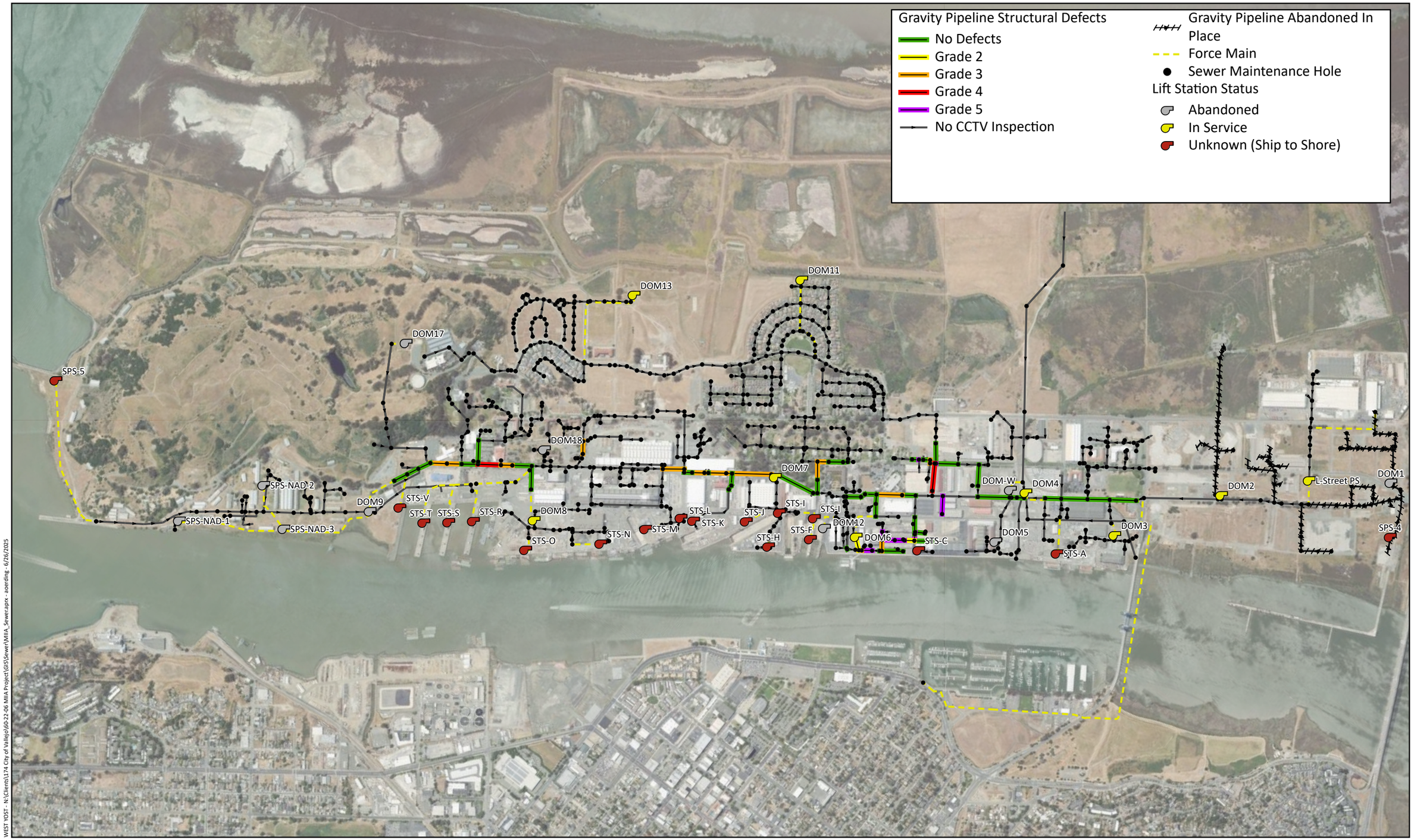
[1] Condition Assessment of Underground Pipes, U.S. EPA Water Infrastructure Outreach, 2015

Table 5-16. Pipeline CCTV – O&M Defects			
Maximum O&M Defect Grade	Count of Pipelines	Length of Pipelines, feet	Percent by Length, %
Grade 5	2	313	2
Grade 4	4	850	7
Grade 3	7	1,040	8
Grade 2	12	2,836	22
Grade 1	4	552	4
No O&M Defects	41	7,384	57
Total	**70**	**12,975**	**100%**





Figure 5-7. Photos of Grade 3 Defects – CIPP Liner Wrinkling and Creasing



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Gravity Pipeline Maximum PACP Structural Defect Grade

Figure 5-8

5.6.3 Maintenance Holes

Table 5-17 presents the estimated useful life values for maintenance holes, by material.

Installation dates were not available for 497 maintenance holes (66 percent of total maintenance holes). Therefore, maintenance holes with unknown installation dates were assumed to be the same age as their connecting pipeline.

Table 5-17. Maintenance Hole Estimated Useful Life Values

Maintenance Hole Material	Estimated Useful Life, Years
Brick	50
Concrete	100
Pre-Cast	100
Unknown	50[(a)]

Source: BACWA Asset Cost and Lifecycle data, adjusted based on field observations.

(a) 50 years was used as a conservative value for manholes of unknown material.

5.6.3.1 Maintenance Hole Remaining Useful Life Summary

Table 5-18 summarizes the assumed condition of maintenance holes based on estimated remaining useful life. Approximately 22 percent of Mare Island maintenance holes are assumed to be in excellent condition while 67 percent are assumed to be in poor condition and beyond their useful life.

Table 5-19 summarizes the replacement projections of maintenance holes. In the near-term scenario, 502 maintenance holes are expected to require replacement. This includes 18 known brick maintenance holes and 484 maintenance holes of unknown material. By the build-out year of 2063, an additional 13 maintenance holes are anticipated to reach the end of their useful life.

These projections assume that replacement is required when RUL reaches zero. However, it is important to note that for many maintenance holes, rehabilitation methods – such as lining, patching, or replacement of defective components – can be employed instead of full replacement, depending on the severity and nature of observed defects.

Table 5-18. Maintenance Hole Condition Summary

Condition, %	Count of Maintenance Holes	Percent of Total Maintenance Holes, %
Excellent (100 to 80 RUL)	166	22
Good (79 to 50 RUL)	73	9.7
Fair (49 to 20 RUL)	13	1.7
Poor (0 to 19 RUL)	0	0
Poor (Less than 0 RUL)	502	67
Total	**754**	**100%**

Table 5-19. Age-Based Maintenance Hole Replacement Projections		
Scenario	Count of Maintenance Holes	Percent of Total Maintenance Holes, %
By 2027 (Near-Term)	502	66.6
By 2063 (Build-Out)	13	1.7

5.6.3.2 Maintenance Hole Inspection

Maintenance holes were selected for inspection based on importance within the collection system and Client Team feedback. Maintenance hole inspection was performed according to the NASSCO Manhole Assessment Certification Program (MACP) Level 1 assessment standards which consists of a visual assessment from the exterior of the manhole frame. This includes observation notations, and photographs of the maintenance hole barrel, cover, and interior. Figure 5-9 presents the results of the maintenance hole inspections. A total of 120 maintenance holes were inspected. Seven additional maintenance holes planned for inspection could not be inspected due to surcharge or being inaccessible but were assessed visually from the exterior.

5.6.3.3 Maintenance Hole Condition Assessment Results

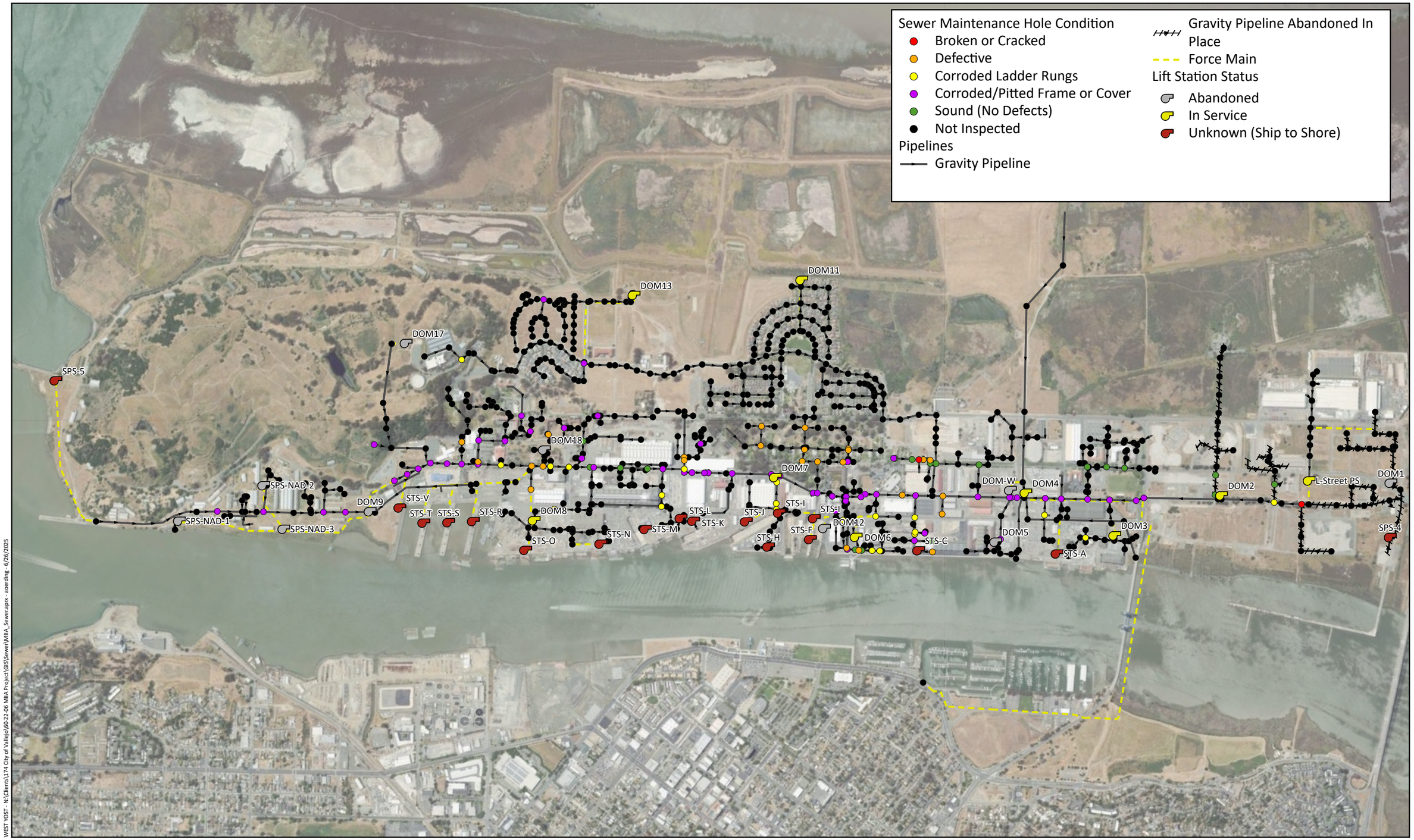
Table 5-20 summarizes the results of the MACP Level 1 inspections, with each maintenance hole categorized by its most severe observed defect. The MACP Level 1 inspection rates the condition of each manhole component as: Sound (no defects); Corroded/Pitted; Defective (e.g., structural [corrosion] or operational deficiencies); or Broken (e.g., cracked or missing pieces).

Where multiple defects were present, the maintenance hole was categorized based on the most severe condition observed to avoid double-counting. Details on maintenance hole defects are provided in Appendix 5A. Of the 120 maintenance holes inspected, 102 (85 percent) exhibited at least one defective component. The most common issues included corrosion or pitting of the frame and cover (54 instances) and defective components (25 instances). 18 maintenance holes were found to be in sound condition.

Table 5-20. Maintenance Hole MACP Level 1 Inspection Results	
Component Condition (from most to least severe)	Count of Maintenance Holes
Broken or Cracked	2
Defective	25
Corroded Ladder Rungs	21
Corroded/Pitted Frame or Cover	54[(a)]
Sound (No Defects)	18
Total Inspected	**120[(b)]**

(a) Includes three maintenance holes that were unable to be opened (surface inspection only).
(b) Out of a total of 754 maintenance holes.

These findings generally support the conservative assumptions used in the age-based replacement analysis, which identified approximately 67 percent of the system in poor condition. While the MACP Level 1 inspections suggest that deterioration is widespread, the defects observed are often localized and may be addressable through targeted rehabilitation measures rather than full replacement.



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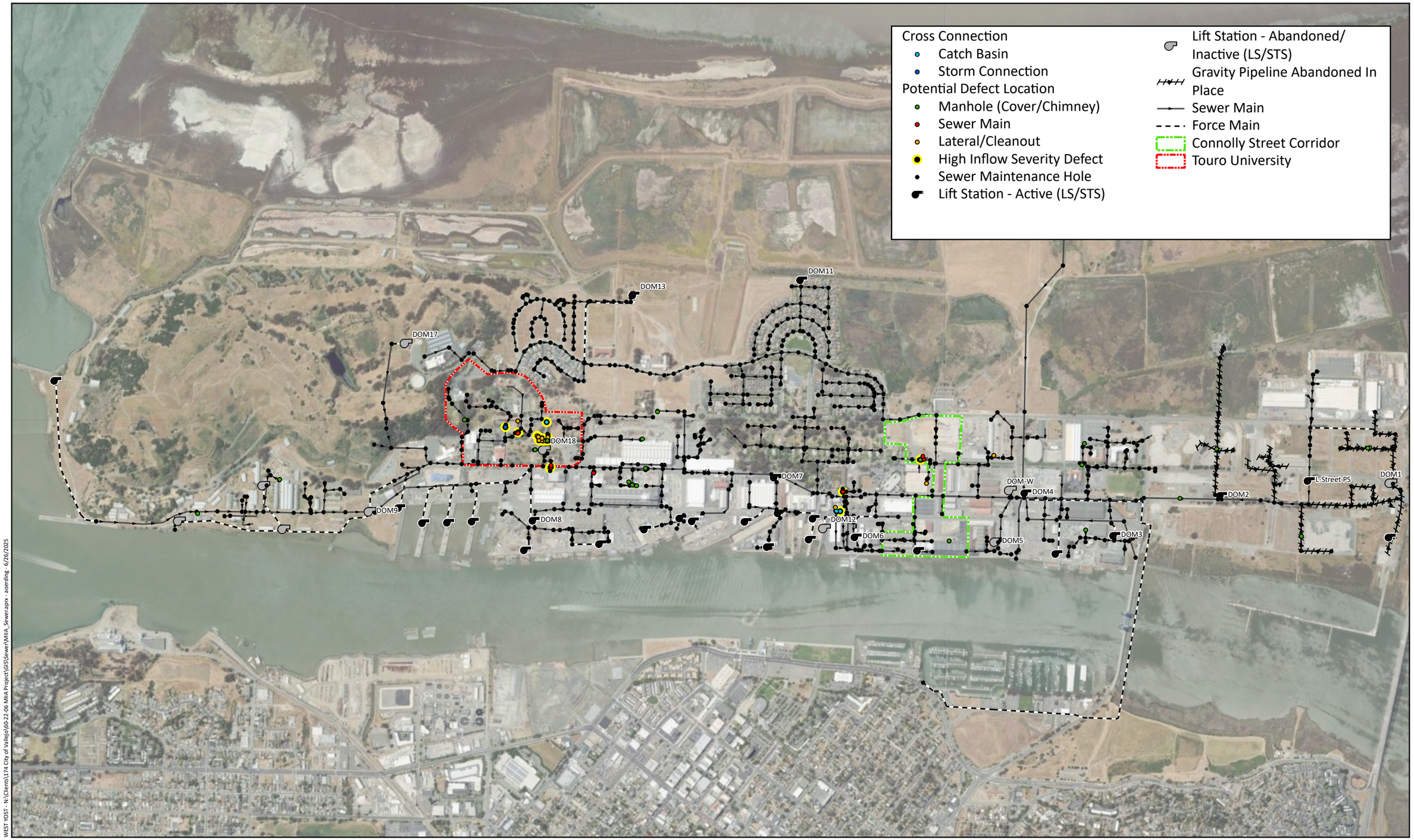
Maintenance Hole Condition

Figure 5-9

5.6.4 Smoke Testing

Smoke testing involves forcing non-toxic, smoke-filled air into the sanitary sewer system through existing maintenance holes. The smoke travels through the system and exits through openings, such as other maintenance holes and other defects – identified where smoke is observed at the surface. Defects include cross-connections from storm sewers; roof downspouts; and basement, yard, and foundation drains, all of which may contribute to stormwater inflow. Smoke may also locate defects in the sewer pipelines, maintenance holes, or laterals, which may contribute to infiltration.

All areas of the island with a known sanitary sewer system (active or otherwise) were smoke tested. Identified defects were grouped into two categories: cross connection or other defects. There were 4 cross connection defects, consisting of catch basin or storm connections. There were 54 other defects, consisting of maintenance holes, sewer mains, or laterals. Figure 5-10 presents all defects; the most severe defects (14) are highlighted in yellow, based on estimated inflow severity. The full index of defects is provided in Appendix 5B.



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Smoke Testing Results

Figure 5-10

5.6.5 Lift Stations

The lift station condition assessment included visual observation of the nine active DOMs and major interior components. Inspections were conducted on February 22 and 23, 2023, by two West Yost civil/mechanical engineers and two District staff members. Major components were rated for physical condition and structural integrity on a scale of 1 to 5, as defined in Table 5-21.

Table 5-21. Condition Rating Index

Rating	Condition Rating Index
1	Excellent or like new
2	Slight visible degradation
3	Visible degradation
4	Integrity of component moderately compromised
5	Integrity of component severely compromised

In 2022, the District conducted a condition assessment of seven lift stations currently in service on Mare Island (DOM2, DOM 3, DOM 4, DOM 5, DOM 6, DOM 7, DOM 8). This included identification of major interior components that required rehabilitation or replacement to bring to District standards. The assessment included an estimation of costs for the required improvements. The 2022 assessment was used to validate the February 2023 inspection and recommendations herein.

Abandoned DOMs were not inspected; ship-to-shore stations were not inspected.

5.6.5.1 Condition Assessment Summary

A summary of the condition assessment results for the nine active DOMs is presented in Table 5-22 by major components. Ratings were generated by grouping components as detailed in the headings; for example, the Pumps and Motors grouping for DOM2 includes three pumps and three motors, which have an average condition rating of 4.

Table 5-22. Summary of Lift Station Condition

Station	Condition Rating of Component(s) and Comments				
	Pumps and Motors	Suction and Discharge Piping and Valves	Wet Well and Vaults	Electrical	Building
DOM2	4 Severe to heavy corrosion; Pump 1 (of 3) inoperable due to faulty variable frequency drive (VFD)	3 Heavy corrosion especially on suction piping; valves do not fully close, difficult to operate	3 Inlet, bypass and overflow gates inoperable/ corroded	4 Original equipment (obsolete); corrosion	3 Windows damaged/ boarded
	Most of station is original construction (1956) except a few components (e.g., suction plug valves).				

Table 5-22. Summary of Lift Station Condition

Station	Condition Rating of Component(s) and Comments				
	Pumps and Motors	Suction and Discharge Piping and Valves	Wet Well and Vaults	Electrical	Building
DOM3	1 Pumps replaced July 2025	1 Suction valves replaced July 2025	3 Inlet gate seized open/corroded	3 A few components replaced once since original construction; now obsolete	3 General degradation due to age
	Most of station is original construction (1956). Generator installed in 2010 in good condition.				
DOM4	4 During inspection, 2 of 3 pumps were out of service; in October 2024, District noted that all 3 pumps were back in service	3 Minor corrosion	3 Deterioration of coatings, minor corrosion; Inlet and bypass gates inoperable/ corroded	4 All electrical components old and obsolete	3 General degradation due to age
	Largest station on island; last rehabilitated 2002. Generator installed in 2005 and in acceptable condition. Most field instruments (pressure element, sampler) out of service or do not work; MCC and controls are planned for replacement.				
DOM6	4 Pump volutes corroded, signs of leakage (2 of 2)	3 Suction piping severely corrode; valves do not fully close, difficult to operate	4 Concrete degraded (spalling, pitting); Inlet gate seized open/corroded	3 A few components replaced once since original construction; now obsolete	3 General degradation due to age
	Most of station is original construction (1956). Generator installed in 2010 and in good condition.				
DOM7	2 Pump 3 (of 3) not in service due to faulty seal; Pump 2 regularly experiences air entrainment	2 Valves and piping replaced in 2015	3 Inlet gate seized open/corroded; Corrosion on wet well hatch; wet well concrete coating deteriorated	3 Most components now obsolete	3 General degradation due to age
	Rehabilitated in 2015. Generator installed in 2010 and in good condition.				

Table 5-22. Summary of Lift Station Condition

Station	Condition Rating of Component(s) and Comments				
	Pumps and Motors	Suction and Discharge Piping and Valves	Wet Well and Vaults	Electrical	Building
DOM8	4 Heavy corrosion	4 Valves do not close fully, difficult to operate	4 Concrete deterioration of wet well; corrosion on hatch and vault walls; Inlet gates (2) seized open/corroded	2 Acceptable condition but most components now obsolete	3 Windows damaged/ boarded
	Generator installed in 2010 in good condition. Influent and discharge maintenance holes unable to be located. High tide pumps do not shut down.				
DOM11	2	2	2	2	1
	Station in overall good condition. Generator installed at original construction (2008) and in good condition.				
DOM13	1	1	1	1	1
	Station is essentially brand new but inactive (not started up). Generator installed at original construction (2018) in like-new condition.				
L Street	2	2	2	3	NA
	L Street lift station is a converted maintenance hole with two submersible pumps in the maintenance hole channel; no telemetry or monitoring. This was intended to be a temporary solution for Alco and EPS buildings connected to the L Street sewer.				

5.6.5.2 Condition Assessment Recommendations

Table 5-23 summarizes the lift station improvement recommendations, which are grouped into two categories:

- Priority 1 - Improvements that are critical to maintaining current operation
- Priority 2 - Improvements that are recommended for operation beyond 2027 (near-term scenario)

For the build-out scenario, all lift stations are expected to require replacement due to age before year 2063; the extent of those improvements, specifically whether the capacity of the station will need to be increased, should be determined during future project-level design evaluations. Based on current conditions, all stations have space available on-site for future expansion, if needed. All improvements should meet District design standards.

Table 5-23. Lift Station Improvement Recommendations

Station	Improvement Recommendations
DOM2	**Priority 1 – Critical to Current Operation** • Replace all pumps and motors (3) • Replace all pump suction and discharge plug valves (6), check valves (3) and associated piping • Replace/repair faulty VFD • Replace all electrical components
	Priority 2 – Continued operation beyond 2027 • Replace building windows • Replace wet well inlet and bypass gates • Replace piping and valves inside the valve pit • Rehabilitate wet well (re-coat; refurbish metal components and penetrations) • Install new generator or quick-connect • Decommission and abandon overflow/bypass pit to remove potential for discharge to Waters of U.S.
DOM3	**Priority 2 – Continued operation beyond 2027** • Replace wet well inlet gate • Rehabilitate wet well (re-coat; refurbish metal components and penetrations) • Install new motor control center (MCC) • Replace all electrical components; improve lighting
DOM4	**Priority 1 – Critical to Current Operation** • Replace VFDs on all Pumps (3) • Replace Pump 2 (seal leak) and motor • Replace all electrical components (control panels planned for replacement) and instrumentation; improve lighting
	Priority 2 – Continued operation beyond 2027 • Replace other pumps and motors (2) • Replace all pump suction and discharge plug valves (6), check valves (3) and associated piping • Replace wet well inlet and bypass gates • Replace MCC (Planned for replacement) • Rehabilitate building exterior • Install emergency bypass • Rehabilitate wet well (re-coat; refurbish metal components and penetrations)
DOM6	**Priority 1 – Critical to Current Operation** • Replace all pumps and motors (2) • Replace all pump suction and discharge plug valves (4), check valves (2) and associated piping • Replace all electrical components; improve lighting
	Priority 2 – Continued operation beyond 2027 • Replace wet well inlet gate • Install VFDs • Rehabilitate wet well (re-coat; refurbish metal components and penetrations)

Table 5-23. Lift Station Improvement Recommendations	
Station	Improvement Recommendations
DOM7	**Priority 1 – Critical to Current Operation** • Replace mechanical seals (Pump 3)
	Priority 2 – Continued operation beyond 2027 • Replace wet well inlet gate • Replace all electrical components; improve lighting • Assess all pumps and motors (3) and rebuild/replace as necessary • Reconfigure suction piping in wet well to eliminate/reduce air entrainment on Pump 2 • Install VFDs • Replace MCC • Rehabilitate wet well (re-coat; refurbish metal components and penetrations)
DOM8	**Priority 1 – Critical to Current Operation** • Replace all pumps and motors (2) • Rehabilitate suction and discharge plug valves (4) and check valves (2)
	Priority 2 – Continued operation beyond 2027 • Replace building windows • Install VFDs • Replace all electrical components; improve lighting • Repair high tide pumps • Rehabilitate wet well (re-coat; refurbish metal components and penetrations)
DOM11 (Farragut)	**None**
DOM13 (Coral Sea)	**None**
L Street	**Convert to dedicated off-line pump station if intended to be used permanently**

5.7 CAPACITY AND PERFORMANCE EVALUATION

The sewer system assessment included a hydraulic capacity and performance evaluation of the Mare Island sanitary sewer collection system. Major tasks completed in the capacity and performance include:

- Model Development
- Flow Monitoring
- Flow Development
- Model Calibration
- Review of Design and Performance Criteria
- Capacity Evaluation
- Capacity Issues and Considerations
- Approach to Recommended Improvements

Each of these tasks are further detailed in the following sections.

5.7.1 Model Development

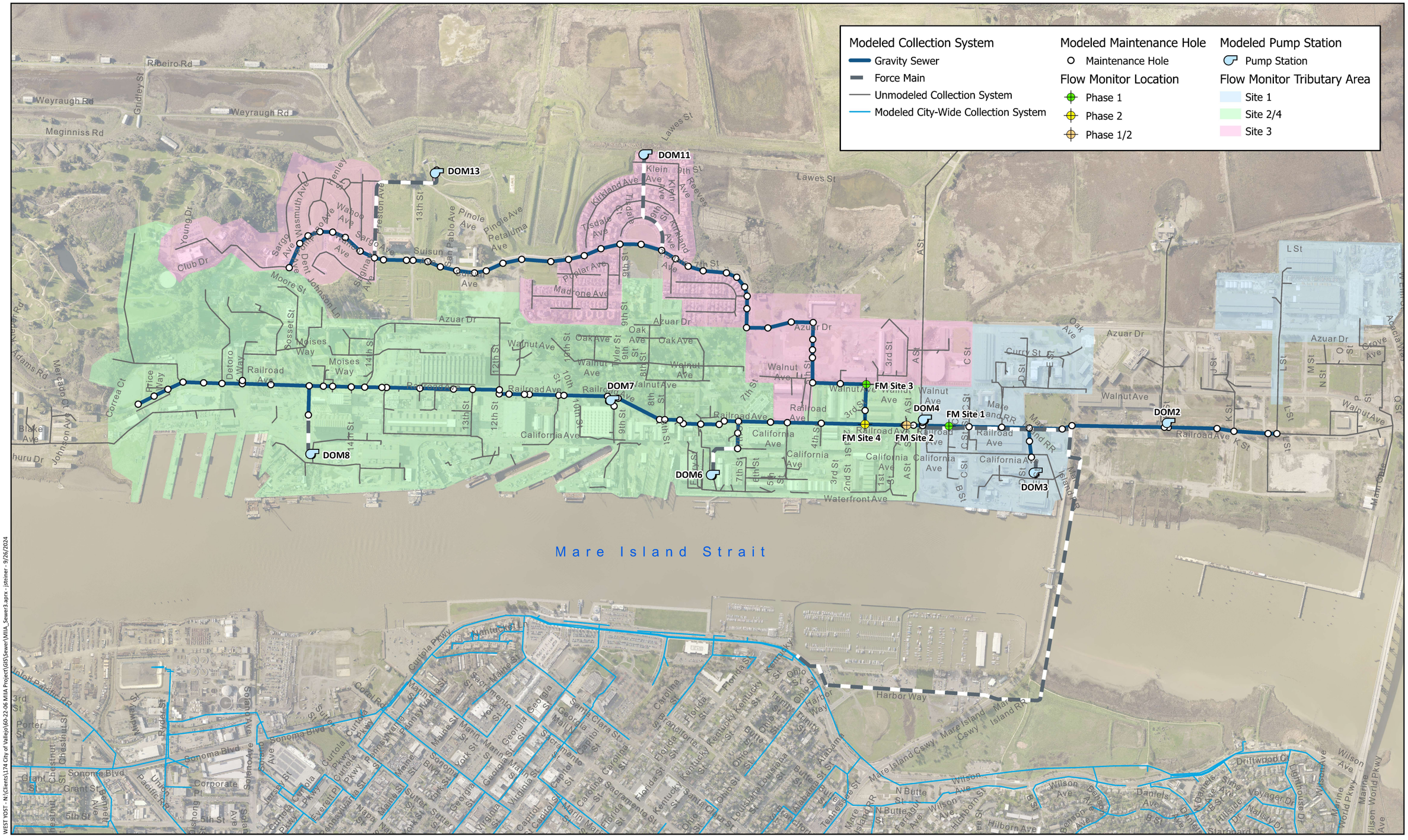
The hydraulic model used to perform the capacity and performance evaluation was developed in InfoWorks ICM software, which is the software the District has utilized for development of their mainland (City-wide) hydraulic model. The Mare Island portion of the hydraulic model was added to the District's City-wide hydraulic model as part of the MIIA project. The Mare Island model includes 138 model nodes, associated pipes, and 8 lift stations. The network is based on the existing system mapping described in Section 5.4 and is presented on Figure 5-11. The model was developed for existing conditions plus two future scenarios. As described in Chapter 2, future growth on Mare Island is evaluated under two timeframes: (1) the existing/near-term timeframe, which is estimated to occur by 2027; and (2) the build-out timeframe, which is estimated to occur by 2063 and is based on a preliminary draft plan of the Mare Island build-out land use (MIC, February 2023).

The wastewater flows discussed throughout the following sections were spatially allocated to one of 50 modeled sewersheds. Sewersheds were then assigned a discharge node location representing the area in the Mare Island collection system where sewer discharges occur as well as the contributing sewer area at each of those locations. For buildout conditions, an additional five sewersheds were created to represent the expansion of development on Mare Island in currently undeveloped areas.

5.7.2 Flow Monitoring

The purpose of flow monitoring is to measure real-world sanitary sewer flows, which can then be used to calibrate, or adjust, the hydraulic model to match real-world conditions. V&A Consulting Engineers (V&A) was retained by West Yost to perform temporary sanitary sewer flow and rainfall monitoring. The study was used to establish a baseline for sanitary sewer flows at each chosen monitoring site, measure peak flow characteristics, isolate and analyze I&I, and monitor the level and pumping cycles at DOM4.

Flows were monitored from March 8 to May 8, 2023 (8 weeks) at three temporary locations and at DOM4. The three locations monitored, and their tributary areas, are shown on Figure 5-11 as Phase 1 flow monitors. A second phase of flow monitoring was performed from April 16 to May 2, 2024 (2.5 weeks) at two temporary locations – shown as Phase 2 flow monitors on Figure 5-11. The purpose of the Phase 2 flow monitoring was to check results from the Phase 1, Site 2 flow monitoring, which was confirmed to be reporting incorrect flow measurements. The *Mare Island Infrastructure Assessment, Sanitary Sewer Flow Monitoring Technical Memo* (West Yost, October 2023) describes the flow monitoring in detail and is included as Appendix 5C.



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Infrastructure Assessment

City of VALLEJO California

Mare Island
Modeled Sanitary Sewer System

Figure 5-11

5.7.3 Flow Development

The following sections discuss the key components of wastewater collection system flows, as shown on Figure 5-12, and the methodology used for developing wastewater flows for use in the Mare Island hydraulic model.

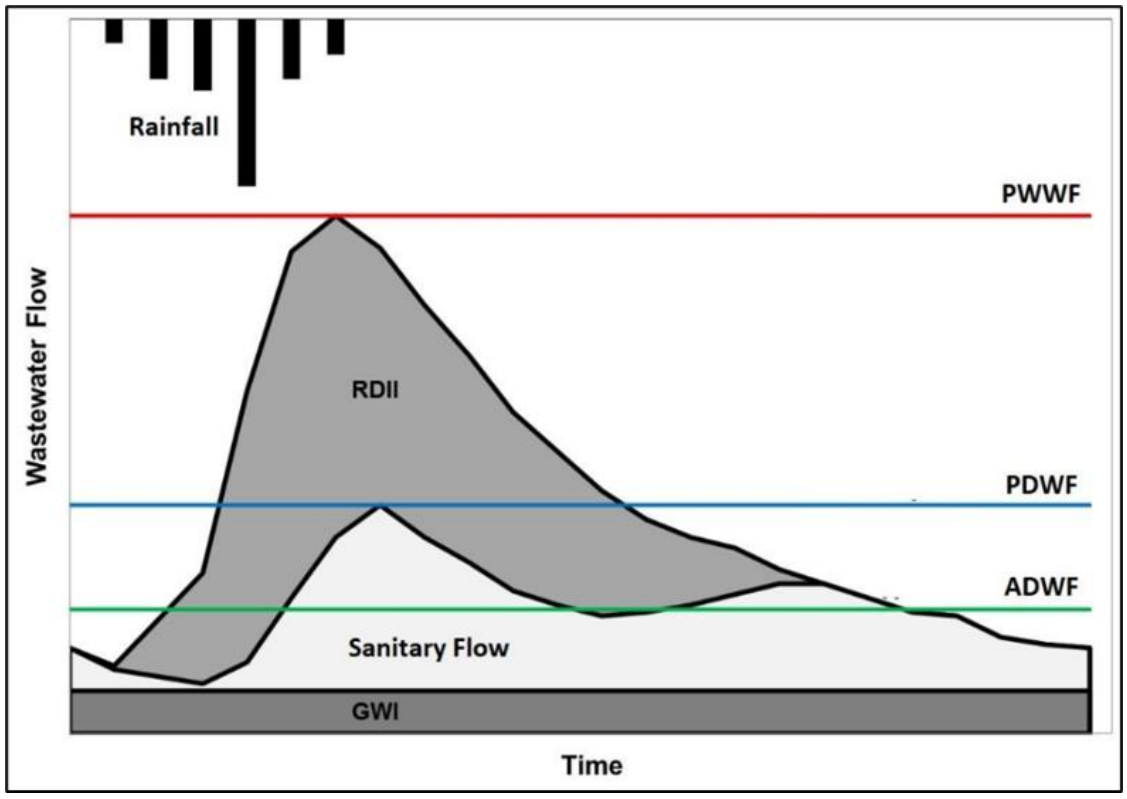


Figure 5-12. Typical 24-hour Cycle of Wastewater Components for PWWF Conditions

5.7.3.1 Average Dry Weather Flow

Average dry weather flow (ADWF) is a term that describes average wastewater flow conditions that typically occur later in the dry season when groundwater infiltration into the collection system is at a seasonal minimum and prior to the onset of significant rainfall events. Depending on groundwater elevations and system porosity, the ADWF may have a component of groundwater infiltration (GWI) that is present year-round. Otherwise, the ADWF represents sanitary flow contributions from residential, commercial, institutional, and industrial dischargers to the collection system.

For reasons described in Appendix 5C, the methodology used for developing existing ADWFs within flow monitoring basin 2 differed from flow monitoring basins 1 and 3. ADWFs developed for flow monitoring basins 1 and 3 were based on 2021 winter (January through March) water consumption data whereas ADWFs developed for flow monitoring basin 2 were based on a back calculation method using flow rates from flow monitoring basin 1 and SCADA data from DOM4. On Mare Island, base ADWFs generally exceed winter water consumption, indicating a high amount of groundwater infiltration and/or (likely to a lesser extent) unaccounted for dischargers. Due to known tidal influence, high groundwater (or in this case, sea water) influence is expected. To calibrate the hydraulic model to the flow monitoring results, pump station SCADA data was used to spatially allocate excess ADWFs throughout the hydraulic model.

For near-term and buildout conditions, the wastewater generation factors shown in Table 5-24 were used in combination with the estimated contributing area for each planning horizon. The wastewater generation factors in Table 5-24 are consistent with the water generation factors discussed in Chapter 3 – Water System[2]. The single-family residential wastewater generation factor (216 gallons per day per dwelling unit) was calculated using 2.7 persons per dwelling unit (DU) and 80 gallons per capita per day (gpcd) per the 2020 District Engineering Design Standards, which equates to a return to sewer ratio of 0.92 when compared to the water generation factors. The remaining wastewater generation factors assume a return to sewer ratio of 1.00, which conservatively assumes that all planned water usage (excluding single-family residential) will be discharged to the Mare Island collection system.

Table 5-24. Near-Term and Build-out Wastewater Generation Factors

Land Use	Water Generation Factor	Wastewater Generation Factor	Units	Return to Sewer Ratio
Singly-Family Residential	234	216	gpd/DU	0.92
Multi-Family Residential	141	141	gpd/DU	1.00
Mixed-Use	155.1	155.1	gpd/DU	1.00
Commercial	806	806	gpd/DU	1.00
Civic	686	686	gpd/DU	1.00
Hospitality/Cultural	155	155	gpd/Room	1.00

Table 5-25 summarizes the existing, near-term, and buildout ADWFs for the Mare Island sanitary sewer system based on the methodology discussed above.

Table 5-25. Existing, Near-Term, and Build-out ADWFs (mgd)

Basin and Tributary Area	Existing	Near-Term (2027)	Build-Out (2063)
Basin 1 (North Mare Island)	0.084	0.084	0.522
Basin 2 (South Mare Island and Site/Basin 3)	0.779	0.870	1.693
Basin 3[(a)] (Redeveloped residential area)	0.259	0.338	0.563
Total arriving at DOM4 (Flows to be pumped off the island)	**0.864**	**0.954**	**2.215**

(a) Basin 3 is tributary to Basin 2.

[2] Water generation factors were based on the 2015 City of Vallejo Water Master Plan, see Chapter 3 – Water System

5.7.3.2 Peak Dry Weather Flow

Peak dry weather flow (PDWF) is defined as the diurnal (daily) peak flow during baseline dry weather flow conditions. The timing of the PDWF typically depends on the contributing land use types. Residential dischargers tend to produce higher flows in the morning hours and in the evening hours, while commercial dischargers tend to have steady discharge during business hours but very low discharge outside of business hours. Industrial dischargers have flow patterns that depend upon their individual processes.

Diurnal curves are typically created for wastewater hydraulic models to represent a fluctuation of sanitary flows throughout the day. Diurnal curves were generated for areas tributary to lift stations using lift station SCADA data while diurnal curves for the remaining areas not directly tributary to a lift station were generated using the flow monitoring data. For each area where diurnal curves were created, weekday and weekend curves were included to capture the differences in sanitary flow based on day of the week. For build-out sewersheds with multiple planned land uses, hybrid diurnals were created assuming the DOM6 diurnal curve represented a typical commercial or industrial land use while the DOM11 diurnal curve represented a typical residential land use. Diurnal curves used throughout the Mare Island hydraulic model are provided in Appendix 5D.

Table 5-26 summarizes the existing, near-term, and build-out PDWFs for Mare Island based on the methodology discussed above.

Table 5-26. Existing, Near-Term, and Build-out PDWFs (mgd)

Basin and Tributary Area	Existing	Near-Term (2027)	Build-Out (2063)
Basin 1 (North Mare Island)	0.352	0.373	0.816
Basin 2 (South Mare Island and Basin 3)	1.410	1.560	2.676
Basin 3[(a)] (Redeveloped residential area)	0.289	0.385	0.682
Total arriving at DOM4[(b)]	**1.652**	**1.801**	**3.373**

(a) Basin 3 is tributary to Basin 2.

(b) The total PDWF arriving at DOM4 is less than the sum of flows from Basins 1 and 2 because there are differences in peak flow timing between flow monitoring basins, and flows reaching DOM4 are impacted by the water surface elevation in the DOM4 wet well, which causes minor upstream surcharging.

5.7.3.3 Inflow and Infiltration

The term infiltration and inflow (I&I) refers to the portion of wastewater flow that does not come from indoor sanitary flow generating activities, but instead enters the collection system in the form of elevated groundwater infiltration (over and above the year-round groundwater infiltration component that may be present in the ADWF), and/or flows entering the collection system during and immediately after rainfall events. This latter component is referred to as rainfall-dependent I&I (RDII) and is defined as storm-induced flow that enters the collection system through porous and/or defective maintenance holes, sewer mains, and service laterals, or illicit connections such as roof leaders that should be connected to storm drain facilities. The magnitude of RDII flows is related to the intensity and duration of the rainfall and the degree of soil saturation and increased groundwater levels arising from rainfall conditions.

The quantity of I&I is dependent on storm intensity and duration. To be consistent with the 2023 District Sanitary Sewer Collection System Master Plan, the 5-year, 24-hour design storm was used to quantify RDII and evaluate wet weather capacity. Table 5-27 shows the existing conditions I&I rates – gallons per day per acre, for each flow monitoring basin. For Basin 1 (North Mare Island), the I&I value calculated was notably high. This is most likely due to a combination of factors:

- High actual high I&I rates associated with old or poor condition infrastructure
- Uncertainty in the defined basin area, particularly regarding which portions of the system are active and contributing flow versus those that are inactive or isolated

Many pipelines in Basin 1 have been intentionally plugged to separate non-serviced areas from the active sewer system. However, field observations indicate that several of these plugs may not be fully sealed, allowing continued inflow. For example, active inflow was observed at a plug in manhole MH376, confirming leakage into the system. As a result, the actual contributing area of Basin 1 is likely much larger than the area only served by active pipelines. To address this uncertainty, a "low scenario" for Basin 1 RDII was calculated based on a tributary area of all pipelines, regardless of their service status (plugged, active, etc.). The true RDII rate for Basin 1 is likely somewhere in between the "low" and "high" scenarios.

For reference, the District I&I allowance (Engineering Standards, Table 3-2) for pipes built before 1970 is 4,000 gpd per acre. For new development (pipes built from 1970), the allowance is 600 gpd per acre.

Table 5-27. Existing Conditions 5-year, 24-hour Rainfall-Dependent Inflow & Infiltration

Basin	Contributing area (acres)	Peak RDII Rate (mgd)	Percent of Peak Flows, %	RDII per Area (gpd per acre)
Basin 1 – High Scenario[(a)] (North Mare Island)	115.5	2.05	85	17,723
Basin 1 – Low Scenario[(a)] (North Mare Island)	249.8			8,195
Basin 2[(b)] (South Mare Island and Basin 3)	386.3	5.94	81	15,382
Basin 3[(b)] (Redeveloped residential area)	158.3	1.01	78	6,374

(a) Peak flows when allowed to flow freely with no influence from backflow or surcharging.

(b) The peak RDII rates modeled at each flow monitoring location do not account for upstream flow losses due to above ground surcharging (SSOs). The effect of this is that the actual peak RDII rates may be slightly higher than what was modeled and reported in this table.

5.7.3.4 Peak Wet Weather Flow

The peak wet weather flow (PWWF) is composed of sanitary flow plus a combination of groundwater infiltration and RDII and is generally defined as the highest flow that occurs during a given storm or wet season. The magnitude of the PWWF is thus dependent on storm intensity and duration, antecedent rainfall conditions, and the time of day at which the peak I&I occurs. The highest PWWF values are typically associated with large, intense storm events, and tend to be greater following a series of other storm events after soils become saturated. Moreover, a large storm occurring during the day will have a higher PWWF than a similar storm occurring at night due to the elevated sanitary flows that occur during the day.

The same 5-year, 24-hour design storm discussed in the previous section was used to determine the PWWF for each flow monitoring basin. Table 5-28 summarizes the existing, near-term, and buildout PWWFs for Mare Island based on the methodology discussed throughout this section.

For Basin 1, two PWWF values are presented to reflect different hydraulic assumptions. Because North Mare Island sits at a lower elevation than the rest of the island, it effectively acts as a relief point for surcharging flows from other areas of the system. According model simulations, during backwater conditions, flow is hydraulically pushed into the North Mare Island sewer system, allowing it to temporarily store excess volume and thereby reduce overflows elsewhere on the island. The North Mare Island system remains largely empty, providing available storage capacity. The actual volume of effective storage provided by the North Mare Island sewer system has not been quantified.

- The first Basin 1 scenario in Table 5-28 assumes backflow is allowed, which most accurately reflects actual system behavior during peak wet weather conditions. This results in negative flow values, indicating reverse flow into Basin 1 caused by backwater effects and system-wide surcharging.
- The second scenario assumes no backflow is allowed, representing a hydraulically isolated Basin 1. In this case, positive flow values reflect flow leaving the basin, as there is no surcharging from other parts of the island to generate backwater conditions in Basin 1.

Table 5-28. Existing, Near-Term, and Build-out PWWFs (mgd)

Basin and Tributary Area	Existing	Near-Term (2027)	Build-Out (2063)
Basin 1 (North Mare Island), backflow allowed[(a)]	-3.405	-3.439	-2.472
Basin 1 (North Mare Island), no backflow allowed[(b)]	2.420	2.430	3.143
Basin 2 (includes South Mare Island and Basin 3)	7.352	7.405	7.477
Basin 3 (Redeveloped residential area)	1.298	1.372	1.778
Total arriving at DOM4	**4.884**	**4.740**	**4.952**

(a) Negative values indicate reversal of flow direction due to backwater conditions and storage for other areas of the system. The reversal of flow direction in Basin 1 is discussed in further detail in Section 5.7.6.

(b) PWWF from Basin 1 when allowed to flow freely with no influence from backflow or surcharging.

5.7.4 Model Calibration

Calibration of Mare Island hydraulic models can be broken down into a dry weather calibration and a wet weather calibration, the results of which can be seen in Appendix 5E for each flow monitoring basin. Depth, velocity, and flow measurements collected from March 15, 2023 until March 25, 2023 were used to calibrate the hydraulic model for both dry and wet weather conditions.

5.7.4.1 Dry Weather Calibration

Dry weather calibration is essentially performed through the flow development process, where ADWFs are established and applied a diurnal curve based on results and trends found in the flow monitoring data. The criteria used for assessing the accuracy of the dry weather flow calibration are taken from the

Chartered Institution of Water and Environmental Management (CIWEM) guidelines, which are summarized as follows:

- The volume of modeled versus metered flow should be in the range of ±10 percent.
- The modeled versus metered peak flow rate should be in the range of ±10 percent.
- The modeled versus metered peak flow depth should be in the range of ±0.5 feet.

In general, the dry weather flows for each flow monitoring basin shown in Appendix 5E are in conformance with the dry weather calibration criteria.

5.7.4.2 Wet Weather Calibration

Wet weather calibration is performed through adjustments of unit hydrographs that characterize an area's susceptibility to RDII. The components of these unit hydrographs, also known as RTK unit hydrographs, are defined as follows:

- **R:** The portion of total rainfall within the tributary area that enters the collection system in the form of I&I.
- T: The time from the onset of rainfall to the peak of the I&I response.
- K: The ratio of the time-to-recession to the time-to-peak of the hydrograph, where the time-to-recession is defined as the time from the peak to the point in time where there is no longer any appreciable I&I response.

The RTK parameters are derived for each flow monitoring basin included in the Phase 1 flow monitoring period. The resultant RTK parameters established through calibration of the March 2023 storm event are then applied to the collection system model using the 5-year, 24-hour design storm for existing, near-term, and build-out conditions. RTK values for near-term and build-out development areas were assumed to have the same RTK values as flow monitoring basin 3, as this area contains the newest collection system and is less susceptible to RDII. It is assumed that portions of the collection system constructed for near-term and buildout development areas would likely have the same RDII susceptibility as flow monitoring basin 3.

- The criteria used for assessing the accuracy of wet weather flow calibration are taken from CIWEM guidelines noted previously, and are summarized as follows:
- The modeled versus metered peak flow rate should be in the range of –15 percent to +25 percent.
- The volume of modeled versus metered flow during and following the calibration storm should be in the range of – 10 percent to +20 percent.
- The modeled versus metered non-surcharged flow depth should be in the range of ±0.5 feet.
- The modeled versus metered depth of surcharge should be in the range of –0.5 feet to +1.6 feet.

In general, the wet weather flows for each flow monitoring basin shown in Appendix 5E are in conformance with the wet weather calibration criteria.

5.7.5 Review of Design and Performance Criteria

The current District Engineering Standards and the 2023 Sanitary Sewer Collection System Master Plan were used to establish the Mare Island sewer collection system performance criteria discussed in the following sections.

5.7.5.1 District Performance Criteria

The following is a summary of the relevant District performance and design criteria that were reviewed when establishing the recommended performance criteria for Mare Island.

July 2020 Engineering Standards

- The average family unit is 2.7 persons per dwelling unit. As per Table 3-1, the average daily sewer discharge for residential and commercial land uses is 80 gpcd and 20 gpcd, respectively. As per Table 3-1, the infiltration allowances for sewers built prior to 1970 is 4,000 gpd/acre and 600 gpd/acre for sewers built after 1970 (Section 3.3.1).
- The collection system shall be evaluated under the PWWF condition. The methodology used for calculating the PWWF is the same used in the August 2023 Sanitary Sewer Collection System Master Plan (Section 3.3.2).
- Sewers shall be designed to flow at peak wet weather flows with a ratio of flow depth to pipe diameter (d/D) no greater than 0.7 for new pipe and 0.9 for existing pipe. Under no condition should a new gravity sanitary sewer be designed to flow under pressure (Section 3.4.5).

August 2023 Sanitary Sewer Collection System Master Plan

- The design storm event is a 5-year, 24-hour storm (Master Plan Section 3.3).
- As part of the Master Plan, a comparison was completed for three design storms and peak water surface elevation/free board combinations. Through this exercise, it was recommended that a minimum allowable free board of 5 feet should be used for identifying capacity deficiencies and to size improvements (see Figure 5-13 for definition of freeboard).

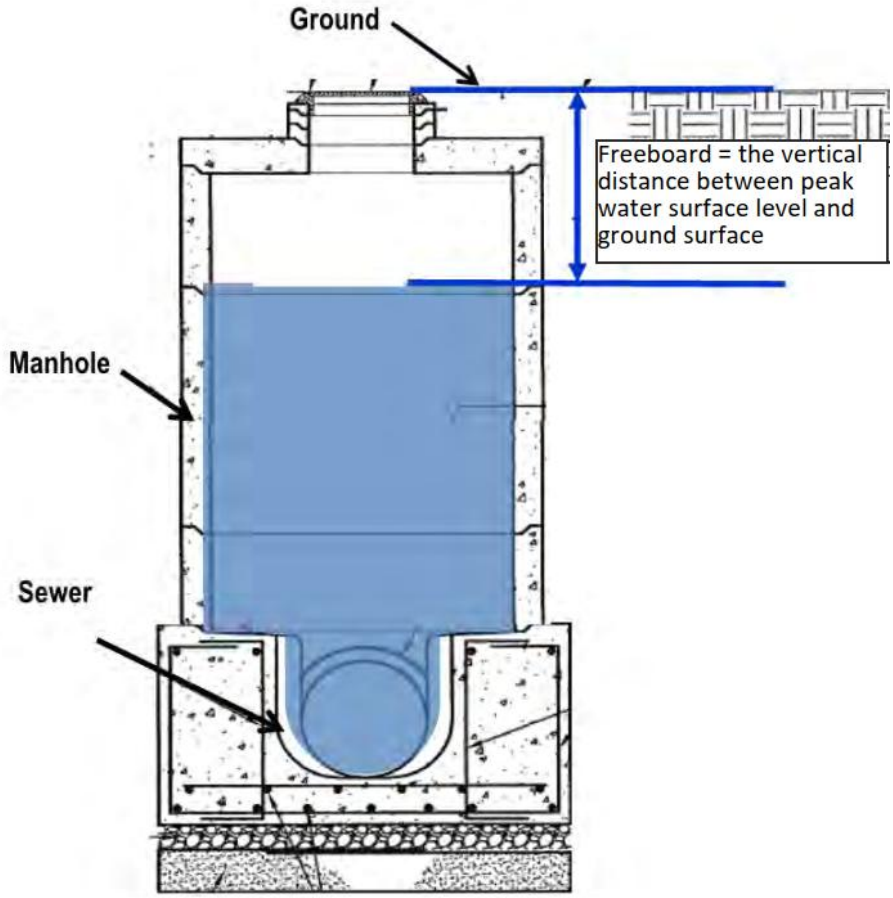


Figure 5-13. Freeboard in Manhole Section
(adapted from Carollo Master Plan Appendix A)

5.7.5.2 Mare Island Performance Criteria

In review of the performance criteria in the previous section and in collaboration with the District, the following performance criteria were recommended for identifying existing, near-term, and build-out capacity issues within the Mare Island sanitary sewer collection system.

1. Priority 1 – above-ground surcharging (no freeboard). Also known as a sanitary sewer overflow (SSO). Depicted in red on the following results figures.
2. Priority 2 – surcharging reaches up to 1 foot below the ground surface elevation, depicted in orange on the results figures.
3. Priority 3 – surcharging reaches up to 3 feet below the ground surface elevation, depicted in yellow on the results figures.
4. Priority 4 – surcharging is greater than 0.9 d/D (water level depth-to-pipeline diameter ratio); in other words, d/D > 0.9. Depicted in green on the results figures.
5. Priority 5 – surcharging is less than or equal to 0.9 d/D (water level depth-to-pipeline diameter ratio); in other words, d/D ≤ 0.9, which meets District allowable d/D design requirements. Depicted in blue on the results figures.

5.7.5.3 Lift Station Performance Criteria

In absence of specific District pump station design criteria and in recognition of the aging lift station infrastructure on Mare Island, it is important to also evaluate the lift station performance in addition to the capacity of the gravity collection system. In general, lift stations are designed with firm capacities (largest pump out of service) exceeding the influent flow rate for a chosen design storm interval. For this evaluation, the firm capacities for lift stations on Mare Island shall meet or exceed the peak influent 5-year, 24-hour flow rate.

There are nine active lift stations on Mare Island, eight of which are included in the hydraulic model. Lift station operations were modeled based on settings observed during field visits, SCADA data provided by the District, and other information provided by the District including pump station level settings, pump curves, VFD settings, and other operation information. A summary of the modeled Mare Island lift stations and their firm capacities is provided in Table 5-29.

Table 5-29. Summary of DOM Operational Setting Details

Lift Station	Modeled?	Pump ON/OFF	Pump Curve Source	Firm Capacity, mgd	Notes
DOM2	Yes	Based on site visit	1956 as-builts	3.08	--
DOM3	Yes	Based on site visit	1956 as-builts	0.40	--
DOM4	Yes	SCADA and site visit	2006 pump station data	10.21	Flow limit set to 3.9 mgd consistent with current settings.
DOM6	Yes	SCADA and site visit	1956 as-builts	0.94	--
DOM7	Yes	SCADA	2001 pump station data	5.04	Electrical issue that prevents all three pumps from coming on at the same time without tripping the breaker. Only two pumps modeled.
DOM8	Yes	SCADA and site visit	1956 as-builts	0.72	Missing updated pump curve data. New pumps installed in 2006.
DOM11	Yes	SCADA and site visit	--	0.15	Missing pump curve data. Modeled as fixed discharge pumps.
DOM13	Yes	Received separately from VFWD	--	0.38	Missing pump curve data. Modeled as fixed discharge pumps.
L Street	No	--	--	--	--

5.7.6 Capacity Evaluation

The calibrated hydraulic model was used to simulate 5-year, 24-hour PWWF conditions in the existing, near-term (2027), and build-out (2063) systems. Model results were compared with the recommended Mare Island performance criteria to identify capacity deficient areas of the collection system. The following sections discuss the existing, near-term, and build-out capacity issues.

5.7.6.1 Existing System Evaluation

The existing system evaluation is used to establish baseline conditions. The existing system capacity issues are presented on Figure 5-14 and summarized as follows:

- The PWWF reaching DOM4 is 4.88 mgd, which causes local surcharging and backflow that propagates into other areas of the system.
- Due to the low elevations and shallow piping of North Mare Island, backflow from DOM4 is projected to cause surcharging that reaches above the ground surface elevation. Backflow is also expected to spill over into the DOM2 wet well which causes an exceedance in the DOM2 firm capacity.
- The DOM3 downstream gravity pipelines are shallow and are expected to experience surcharging above the ground surface elevation due to the surcharging and backflow from DOM4.
- The DOM7 force main discharge maintenance hole is very shallow and surcharging from DOM4 is expected to cause surcharging above the ground surface elevation in this location.
- The DOM11 firm capacity is slightly exceeded, but adequate freeboard exists in the wet well for this period.

Based on anecdotal information provided by the District and Mare Island Company, there have been no known above ground surface surcharging (or SSOs) on the island. It is possible that abandoned parts of the system, particularly on North Mare Island, may be acting as storage during surcharging periods, which has prevented above-ground surcharging.

5.7.6.2 Near-Term System Evaluation

The near-term scenario evaluates the system's ability to serve near-term development, including the Connolly Street Corridor project and expansion of Lind Marine and Mare Island Dry Docks. Locations where near-term sewer flow has been added are shown in purple hexagons on Figure 5-15. The near-term system capacity issues are presented on Figure 5-15 and summarized as follows:

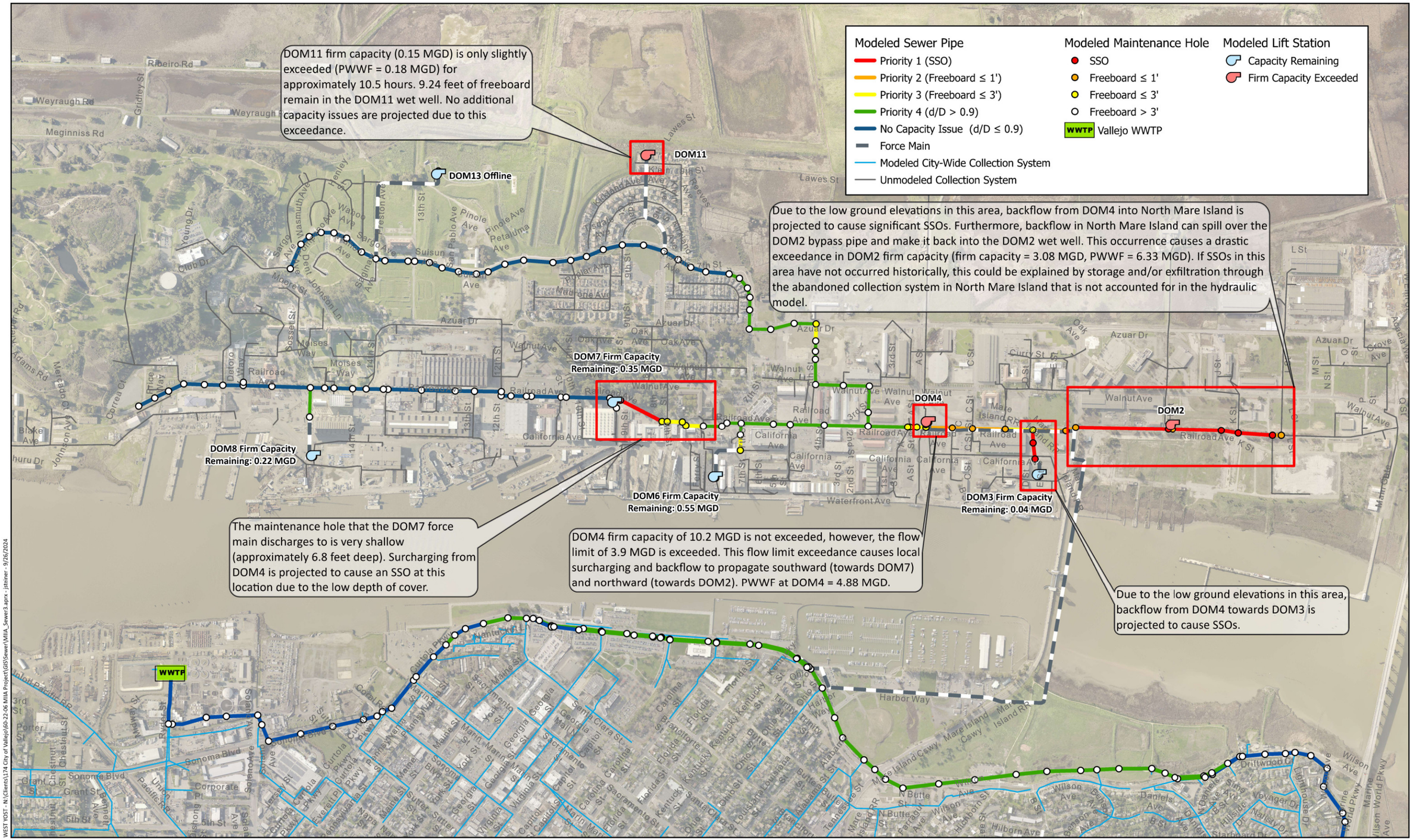
- Capacity issues are similar to the existing system results. The PWWF reaching DOM4 is 4.74 mgd, which is slightly less than the PWWF reaching DOM4 for existing conditions (4.88 mgd). This is caused by increases in SSO volumes elsewhere in the system, which reduced the overall volume reaching DOM4.
- All locations with above-ground surcharging in the existing condition also experience above-ground surcharging in the near-term condition, with slight increases in SSO volumes (e.g., the SSO volume at DOM7 increases from 3,600 cubic feet in the existing scenario to 4,700 cubic feet in the near-term scenario).

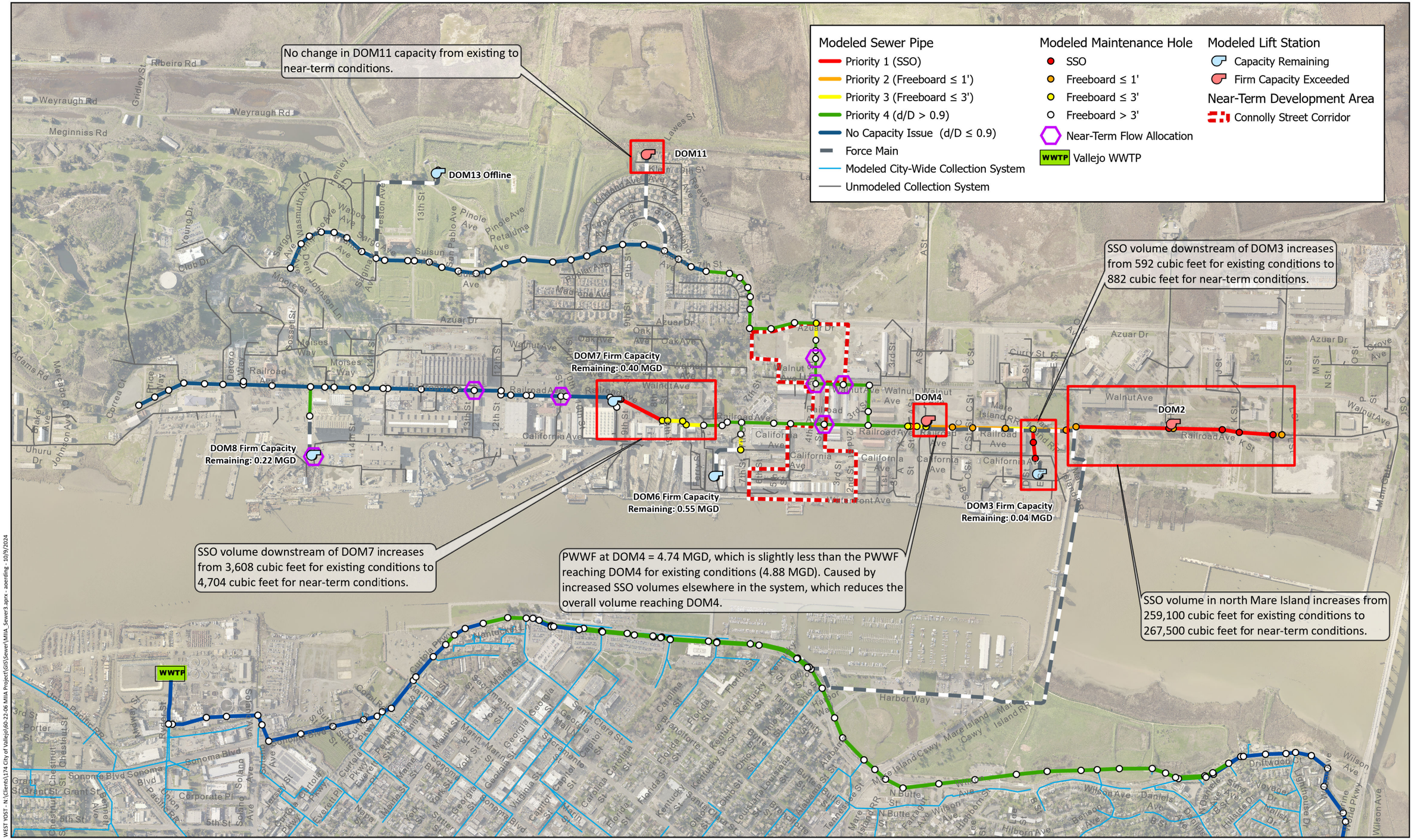
5.7.6.3 Build-Out System Evaluation

The build-out scenario evaluates the system's ability to serve buildout development plans (year 2063). The build-out scenario evaluations and recommendations are highly dependent upon final development land use and timing. If there are any significant changes to land use, land use distribution, or development timing, project-level evaluations should be performed to confirm flows and infrastructure needs. Trunk pipelines have been included for new development areas, but final collection system piping system will depend on development layout and is not included.

The buildout pipelines and system capacity issues are presented on Figure 5-16 and summarized as follows:

- Capacity issues are similar to the existing and near-term system results, but with a higher severity magnitude. The PWWF reaching DOM4 is 5.0 mgd.
- The surcharging in North Mare Island is eliminated under the assumption that this area will be filled to an elevation of 18 feet with new development and new pipelines. These higher elevations allow for gravity flow to DOM4, and DOM2 to be abandoned.
- The DOM13 firm capacity is slightly exceeded, but adequate freeboard exists in the wet well for this period. The District should confirm the planned buildout tributary area for DOM13.
- In the Flagship Drive area just downstream of DOM11, there is above-ground surcharging in two pipelines with a total SSO volume of 5,100 cubic feet.
- All other locations with above-ground surcharging in the existing and near-term scenarios also experience above-ground surcharging, with significant increases in sewage spilled (e.g., the SSO volume at DOM7 increases from 3,600 cubic feet in the existing scenario to 4,700 cubic feet in the near-term scenario to 88,000 cubic feet in the build-out scenario).


DOM11 firm capacity (0.15 MGD) is only slightly exceeded (PWWF = 0.18 MGD) for approximately 10.5 hours. 9.24 feet of freeboard remain in the DOM11 wet well. No additional capacity issues are projected due to this exceedance.
Modeled Sewer Pipe
Priority 1 (SSO)
Priority 2 (Freeboard ≤ 1')
Priority 3 (Freeboard ≤ 3')
Priority 4 (d/D > 0.9)
No Capacity Issue (d/D ≤ 0.9)
Force Main
Modeled City-Wide Collection System
Unmodeled Collection System
Modeled Maintenance Hole
SSO
Freeboard ≤ 1'
Freeboard ≤ 3'
Freeboard > 3'
WWTP Vallejo WWTP
Modeled Lift Station
Capacity Remaining
Firm Capacity Exceeded
DOM11
DOM13 Offline
Due to the low ground elevations in this area, backflow from DOM4 into North Mare Island is projected to cause significant SSOs. Furthermore, backflow in North Mare Island can spill over the DOM2 bypass pipe and make it back into the DOM2 wet well. This occurrence causes a drastic exceedance in DOM2 firm capacity (firm capacity = 3.08 MGD, PWWF = 6.33 MGD). If SSOs in this area have not occurred historically, this could be explained by storage and/or exfiltration through the abandoned collection system in North Mare Island that is not accounted for in the hydraulic model.
DOM7 Firm Capacity Remaining: 0.35 MGD
DOM4
DOM2
DOM8 Firm Capacity Remaining: 0.22 MGD
DOM6 Firm Capacity Remaining: 0.55 MGD
DOM3 Firm Capacity Remaining: 0.04 MGD
The maintenance hole that the DOM7 force main discharges to is very shallow (approximately 6.8 feet deep). Surcharging from DOM4 is projected to cause an SSO at this location due to the low depth of cover.
DOM4 firm capacity of 10.2 MGD is not exceeded, however, the flow limit of 3.9 MGD is exceeded. This flow limit exceedance causes local surcharging and backflow to propagate southward (towards DOM7) and northward (towards DOM2). PWWF at DOM4 = 4.88 MGD.
Due to the low ground elevations in this area, backflow from DOM4 towards DOM3 is projected to cause SSOs.
WWTP
Ribeiro Rd
Weyraugh Rd
Meginniss Rd
Gridley St
Lawes St
Kiernan Ct
Kiernan Ave
Kidder Ct
Kidder Ave
Kidder Creek
Kidder Creek Ct
Kidder Creek Dr
Kidder Creek Pl
Kidder Creek Way
Kidder Creek Cir
Kidder Creek Ln
Kidder Creek Loop
Kidder Creek Terrace
Kidder Creek Trail
Kidder Creek Path
Kidder Creek Alley
Kidder Creek Boulevard
Kidder Creek Parkway
Kidder Creek Avenue
Kidder Creek Street
Kidder Creek Road
Kidder Creek Drive
Kidder Creek Court
Kidder Creek Place
Kidder Creek Way
Kidder Creek Circle
Kidder Creek Lane



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Prepared by:

WEST YOST

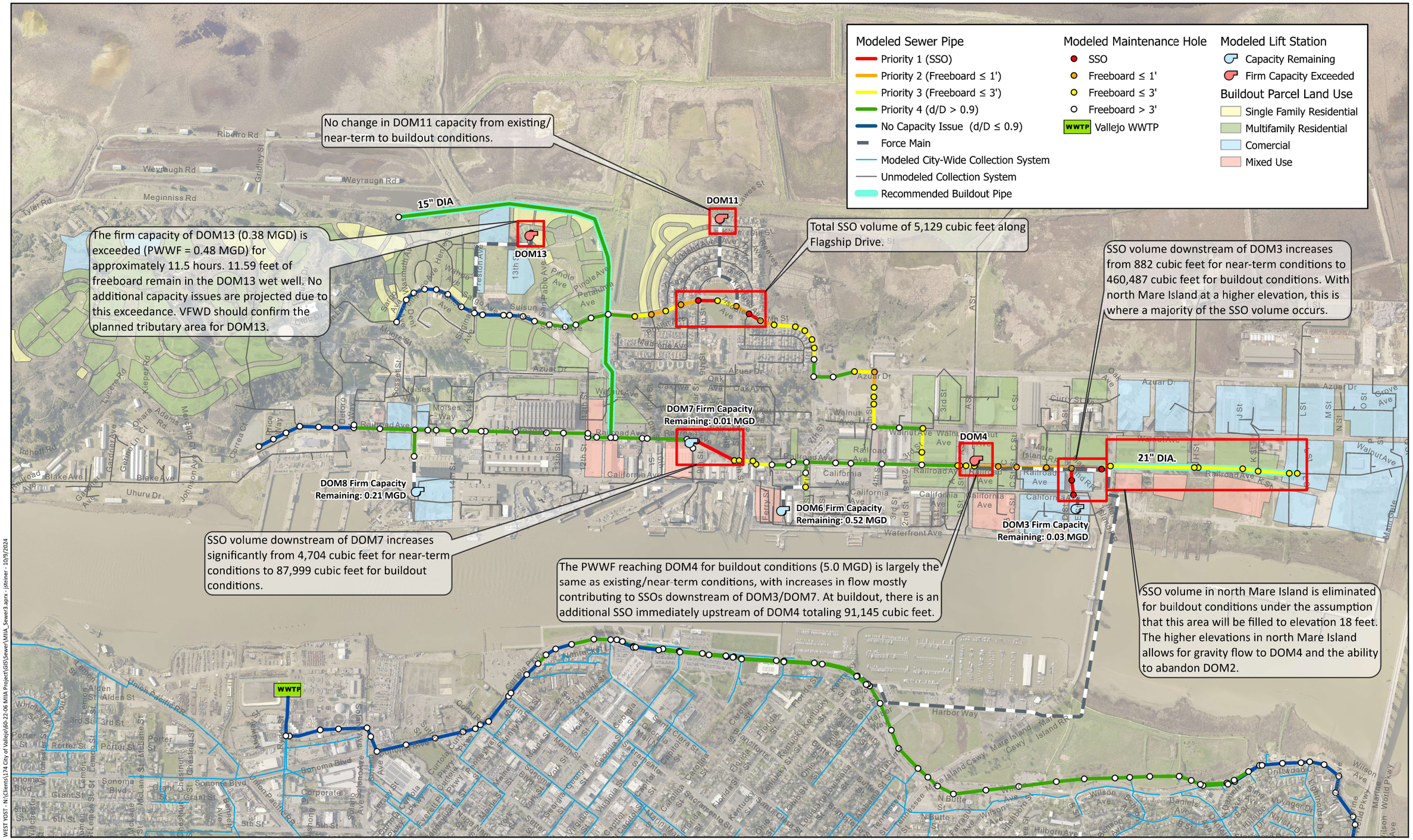
0 1,000 Feet

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

Sanitary Sewer System - Near-Term System
5-year, 24-hour Capacity Issues
DOM4 Flow Limit = 3.9MGD

Figure 5-15



Prepared by:

WEST YOST

0 1,100 Feet

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

City of VALLEJO California

Sanitary Sewer System - Buildout System
5-year, 24-hour Capacity Issues
DOM4 Flow Limit = 3.9MGD

Figure 5-16

5.7.7 Capacity Issues and Considerations

Capacity issues in the Mare Island collection system are primarily driven by high I&I rates and the DOM4 lift station flow limit of 3.9 mgd. The development of recommended capacity improvements considered the following key factors, each described below:

- DOM4 and WWTP Capacity
- I&I
- Future Uncertainties

5.7.7.1 DOM4 and WWTP Capacity

The DOM4 lift station is currently limited to 3.9 mgd due to downstream capacity constraints in the collection system and at the District's WWTP.

The District WWTP faces several constraints that limit the feasibility of near-term capacity expansion. Any increase in treatment capacity would require a new NPDES permit, a complex and lengthy process, particularly due to the plant's operation as a blending facility. Expanding treatment capacity would necessitate major upgrades to core systems, including headworks, primary treatment, and secondary treatment. Additionally, compliance with evolving California regulatory requirements for nutrient removal (e.g., nitrogen and phosphorus limits) would require advanced tertiary treatment processes. These processes are both capital- and space-intensive, and the current site has limited available land, further constraining expansion feasibility.

Due to these constraints, increasing the DOM4 pumping capacity is not considered viable in the near term. Although expansion of the WWTP is also not feasible at this time, it may need to be reconsidered as part of long-term planning to address future capacity needs driven by development and regulatory compliance.

5.7.7.2 I&I

I&I contributes a substantial portion of peak wet weather flows – up to 85 percent in some locations under existing conditions (see Table 5-27). These I&I levels significantly impact system capacity, particularly given the flow limitations at the DOM4 lift station and downstream in the system.

As development progresses, I&I levels are expected to decrease through pipeline replacement and rehabilitation. These efforts will not only restore structural integrity but also reduce unnecessary conveyance of stormwater and groundwater into the sewer system.

For the purposes of developing capacity improvement recommendations, it was assumed that peak I&I rates could be reasonably reduced by up to 35 percent, based on rehabilitation of approximately 50 percent of pipelines within a given basin. These assumptions are based on consultations with the Client Team and similar recent analyses performed by the District[3].

[3] VFWD Sears Point Sewershed Rehabilitation Nexus Study, West Yost (January 2024)

An I&I reduction strategy is also consistent with State Water Resources Control Board guidance, which discourages the conveyance of non-wastewater flows (such as stormwater and groundwater infiltration) to treatment facilities.

5.7.7.3 Future Uncertainties

A range of future uncertainties will influence system capacity needs over time, including changes in land use, the pace and type of development, the effectiveness of I&I reduction efforts, impacts from climate change, and evolving regulatory requirements.

Due to the difficulty of quantifying these variables, a proactive and adaptive approach is recommended. This includes regular monitoring of sewer flows and I&I, periodic updates to flow projections, and close coordination with planning and development activities. This approach will help ensure the system continues to meet service needs as conditions evolve.

5.7.8 Approach to Recommended Improvements

Based on the considerations discussed above, improvements were identified to address system performance for the existing, near-term, and build-out land-use conditions. Improvements focus on three different approaches, each involving different combinations of I&I reduction and adjustments to the DOM4 lift station flow capacity:

- **Approach 1: I&I Reduction with Existing DOM4 Capacity**
  - Maintain DOM4 capacity at 3.9 mgd.
  - Implement targeted I&I reduction to eliminate all remaining above-ground surcharging.
  - Applies to all land-use scenarios.
- **Approach 2: I&I Reduction with Hydraulic Separation of North Mare Island**
  - Maintain DOM4 capacity at 3.9 mgd.
  - Assume North Mare Island is hydraulically separated from the rest of the system. Hydraulic separation of North Mare Island would involve construction of a new lift station and force main to serve only the domestic flow of current customers. The gravity trunk pipeline from North Mare Island would be plugged to prevent any I&I from coming into the system and reaching DOM4.
  - Implement targeted I&I reduction to eliminate all remaining above-ground surcharging.
  - Applies to only the near-term scenario, based on the assumption that future development will reconstruct the gravity sewer system in North Mare Island to serve new development.
- **Approach 3: Increase DOM4 Flow Limit**
  - Increase DOM4 flow limit above 3.9 mgd.
  - Implement targeted I&I reduction to eliminate above ground surcharging.
  - Applies to only the build-out scenario, assuming that capacity upgrades to DOM4 are not feasible in the near term.

A summary of the conditions associated with each approach and land-use scenario (existing, near-term, build-out) is presented in Table 5-30 and discussed in the following sections.

Table 5-30. Summary of Approaches to Capacity Improvements

Land-Use Scenario	DOM4 Max Flow Rate	I&I Conditions	Notes
Approach 1: I&I Reduction with Existing DOM4 Capacity			
Existing/Near-Term Build-Out	3.9 mgd	35 percent reduction in peak I&I rates	Several capacity issues remain
		50 percent reduction in peak I&I rates	No remaining capacity issues
		35 percent reduction in peak I&I rates	Several capacity issues remain
		75 percent reduction in peak I&I rates	No remaining capacity issues
Approach 2: I&I Reduction with Hydraulic Separation of North Mare Island			
Existing/Near-Term	3.9 mgd	50 percent reduction in peak I&I rates	No remaining capacity issues
Approach 3: Increase DOM4 Flow Limit			
Build-Out	7.5 mgd	35 percent reduction in peak I&I rates	No remaining capacity issues

5.7.9 Existing and Near-Term Recommended Improvements

Through the sewer capacity analysis, it was determined that the recommended capacity improvements for the existing and near-term conditions are functionally equivalent, therefore, the improvements discussed in this section apply to both scenarios.

5.7.9.1 Approach 1: I&I Reduction with Existing DOM4 Capacity

This approach targets a 35 percent reduction in peak I&I rates through rehabilitation of approximately 50 percent of pipelines across the island (estimated 59,000 feet). This level of I&I reduction is considered feasible based on District experience. While this reduction resolves capacity issues in most areas, above-ground surcharging is still projected in North Mare Island due to its low elevation.

To fully resolve above-ground surcharging with pipeline rehabilitation alone would require a 50 percent reduction in peak I&I rates through rehabilitation of approximately 71 percent of pipelines across the island (estimated at 84,000 feet).

Removal of inflow defects and cross connections would further reduce peak I&I rates and will likely lower pipeline rehabilitation targets.

5.7.9.2 Approach 2: I&I Reduction with Hydraulic Separation of North Mare Island

This approach targets a 50 percent reduction in peak I&I rates through rehabilitation of approximately 71 percent of pipelines (estimated 75,000 feet), which is less than the estimated 84,000 feet required in Approach 1. The reduced rehabilitation length reflects the hydraulic isolation of North Mare Island, which is achieved by redirecting sewer flows from existing North Mare Island developments to DOM4 via a new lift station and force main. Under this configuration, the North Mare Island gravity sewer collection system and DOM2 pump station are abandoned and hydraulically isolated (disconnected) from the rest of the system.

By excluding some of North Mare Island from the rehabilitation scope, this approach achieves the same I&I reduction target with less pipeline rehabilitation. The tradeoff is the need for new pumping infrastructure to convey flows to DOM4. This approach allows DOM4 to continue operating at its current capacity of 3.9 mgd while fully resolving above-ground surcharging.

Removal of inflow defects and cross connections would further reduce peak I&I rates and will likely lower pipeline rehabilitation targets.

5.7.10 Build-Out Recommended Improvements

In all build-out scenarios, it is assumed that North Mare Island would be filled to an elevation of 18 feet which eliminates the surcharging risk currently associated with its low-lying topography.

5.7.10.1 Approach 1: I&I Reduction with Existing DOM4 Capacity

Consistent with the existing/near-term scenario, Approach 1 targets a 35 percent reduction in peak I&I rates through rehabilitation of approximately 50 percent of pipelines, but only in areas other than North Mare Island (estimated 51,000 feet). Surcharging is not projected in North Mare Island due to its now higher elevation, but capacity issues remain in other parts of the system.

To fully resolve above-ground surcharging would require a 75 percent reduction in peak I&I rates. This level of reduction is not considered achievable through pipeline rehabilitation alone. However, as discussed in the near-term approaches, continued removal of inflow defects and cross connections (both known and unknown) would further reduce peak I&I rates. A lateral renewal program would also target infiltration sources that are typically outside District responsibility or control. With these additional measures, it is feasible that peak I&I rates could be reduced to levels necessary to fully eliminate surcharging under build-out conditions.

5.7.10.2 Approach 3: Increase DOM4 Flow Limit

This approach involves increasing the DOM4 lift station flow capacity to approximately 7.5 mgd, paired with targeted I&I reduction (again estimated at 35 percent). This combination is projected to eliminate all above-ground surcharging across the system under build-out conditions.

5.7.10.3 Summary of Approach Feasibility

Near-Term

In the near-term, both Approach 1 and Approach 2 require I&I reduction to completely eliminate above-ground surcharging, with less pipeline rehabilitation needed for Approach 2 (75,000 feet vs. 84,000 feet in Approach 1) with the tradeoff of an interim pump station for existing development on North Mare Island. Removal of inflow defects and cross connections would further reduce peak I&I rates and will likely lower pipeline rehabilitation needs.

Build-Out

Under build-out conditions, with North Mare Island elevated, surcharging risk shifts to other parts of the system. Approach 1 remains aligned with District practices and industry norms, but it is unlikely to fully resolve systemwide capacity issues without requiring impractically high levels of I&I reduction. Approach 3, which increases DOM4 capacity to 7.5 mgd, is the only approach that eliminates surcharging across the entire system, though it would require major infrastructure investment and long-term coordination.

It is important to note that all I&I reduction rates and PWWF projections presented are planning-level estimates. Actual I&I contributions can vary significantly by defect type and location; for example, a single severe defect may contribute disproportionately to I&I within a basin. For these reasons, a phased and adaptive implementation strategy is recommended, prioritizing:

- Identification and removal of high-contributing defects,
- Targeted pipeline rehabilitation, and
- Ongoing monitoring and reassessment to refine projections and support future development needs.

5.8 INTEGRATION OF CONDITION AND CAPACITY ASSESSMENT RECOMMENDATIONS

This section incorporates recommendations from the condition assessment and hydraulic capacity evaluation into an integrated set of recommendations for each land-use scenario (existing/near-term and build-out).

5.8.1 Existing and Near-Term

Through the sewer capacity analysis, it was determined that there is no discernable difference between the recommended capacity improvements for the existing and near-term scenarios; the same assumption can be made for recommendations from the condition assessment. Therefore, the improvements discussed in this section apply to both existing and near-term scenarios.

Recommendations for the existing and near-term land-use scenario are presented on Figure 5-17 and include:

5.8.1.1 Condition Recommendations

- Eliminate severe inflow defects and cross connections identified in smoke testing (14 severe defects and 4 cross connections). Addressing these defects and cross connections will provide the greatest reduction in I&I.
- Replace or rehabilitate pipelines that were identified to have Grade 4 or 5 structural defects (11 pipelines; 1,463 feet). Rehabilitate maintenance holes in-line or adjacent to pipelines that are replaced or rehabilitated.
- Replace all brick pipelines (437 feet identified) and maintenance holes (18 identified).
- Rehabilitate maintenance holes with broken, cracked, or defective components (27 identified).

- Implement lift station Priority 1 improvement recommendations (DOM 2, 4, 6, 7, 8, L Street) (see Table 5-22).
- Complete condition assessment on all pipelines greater than 60 years old (see Figure 5-4) that were not inspected as part of this project – approximately 13.7 miles of pipeline. Visually assess associated maintenance holes concurrently. Address severe defects as they are identified

5.8.1.2 Capacity Conclusions and Recommendations

- Implementation of pipeline rehabilitation, defect elimination, and cross-connection removals is expected to result in a measurable I&I reduction, helping to alleviate capacity constraints in the system.
- According to anecdotal information provided by the District and Mare Island Company, no SSOs have occurred on Mare Island in recent years. As development progresses, reductions in I&I from near-term improvements are anticipated to offset flow increases from new connections, thereby minimizing the risk of SSOs.
- Once near-term rehabilitation and inflow elimination projects are complete, system capacity should be re-evaluated using updated flow monitoring data and development-based flow projections. This will establish a meaningful post-rehabilitation baseline and inform whether further improvements are necessary.

5.8.2 Build-Out

The build-out scenario recommendations are highly dependent upon final development land use and timing. If there are any significant changes to land use, land use distribution, or development timing, evaluations should be revised to confirm capacity upsizing needs.

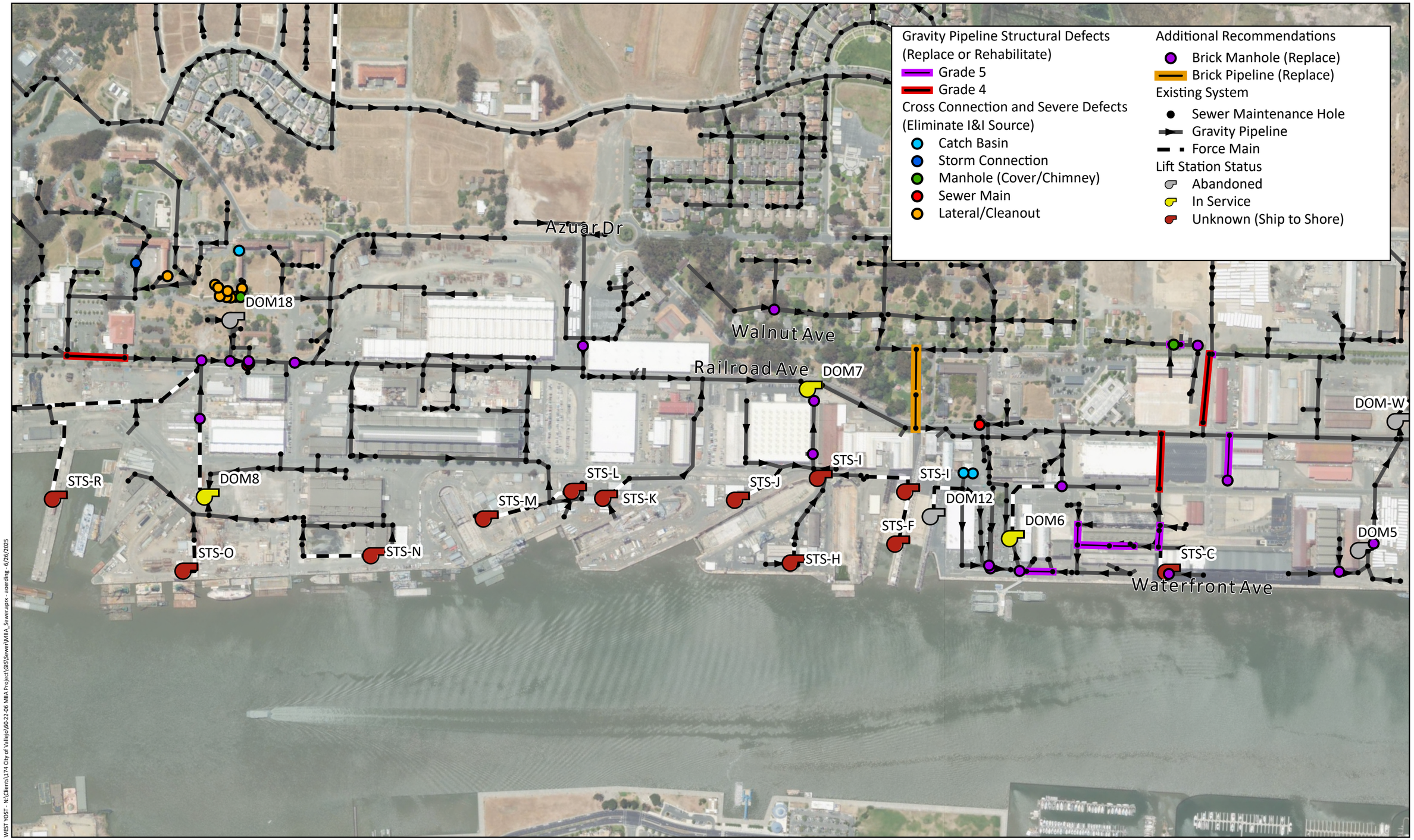
Recommendations for the build-out land-use scenario are as follows:

5.8.2.1 Condition Recommendations

- Replace or rehabilitate pipelines that were identified to have Grade 3 structural defects (14 pipelines, 3,134 feet) as part of this project (recommended within 5 years – by 2029 or earlier), and pipes that were inspected after this project was concluded (see Figure 5-8).
- Implement a regular preventative maintenance program, including pipeline cleaning and condition assessment. Perform condition assessment of all pipes every 7 years (minimum) to identify structural and maintenance deficiencies.
- For continued lift station operation beyond 2027, implement lift station Priority 2 improvement recommendations (DOM 2, 3, 4, 6, 7, 8). Full replacement or upgrades of stations may be required by 2063, the extents of which should be determined during project-level design evaluations. Preliminary evaluation indicates adequate space exists at all lift station sites for expansion.

5.8.2.2 Capacity Conclusions and Recommendations

- Continued pipeline rehabilitation and replacement throughout the build-out period (2028–2063) is expected to result in significant reductions in I&I, further relieving capacity constraints in the system and supporting long-term development.
- System capacity should be re-evaluated following completion of major rehabilitation efforts using updated flow monitoring and development-based flow projections. This re-evaluation will provide a critical baseline for determining whether additional capacity improvements – such as increasing the DOM4 flow limit – are warranted.
- If a formal capacity re-evaluation is not conducted, one of the alternative improvement approaches outlined in Section 5.7.10 should be implemented. These range from continued I&I reduction efforts to increasing the DOM4 flow limit.
- The following ongoing actions are recommended and will support future decision-making regarding capacity upgrades or targeted system rehabilitation:
  - Monitor surcharging at key maintenance holes to identify persistent or worsening capacity deficiencies. These observations can help prioritize additional targeted rehabilitation or system improvements.
  - Evaluate downstream impacts if DOM4 capacity is increased. Upgrades to the wastewater WWTP may be necessary, including revised permitting, physical expansion of treatment processes, and compliance with applicable regulatory and effluent quality requirements.



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Prepared by:

WEST YOST

0 500 Feet

Notes:
1. VFWD sewer system geodatabase provided by VFWD in 2023. Modified based on data available in Navy Quad maps, record drawings, and field verification.
2. Only Mare Island sewer system facilities are displayed for clarity.

Prepared for:

City of Vallejo
Mare Island
Infrastructure Assessment

City of VALLEJO California

Existing and Near-Term Integrated Final Recommendations

Figure 5-17

CHAPTER 6
Geotechnical Evaluation

6.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

Delve Underground (Delve) was subcontracted by West Yost Associates for the Mare Island Infrastructure Assessment Project to perform a desktop study of available information and summarize the known geotechnical conditions on Mare Island.

A summary of the findings and conclusions for the geotechnical evaluation are listed below.

- Existing geotechnical information, subsurface data, and recommendations from public and private work projects near or within the area of project elements (Plate 1) were collected and reviewed. Collected reference documents are presented in Appendix 6A.
  - The majority of reference borings are located in North Mare Island, the area north of G Street and east of Azuar Drive (Plate 2).
  - Recommendations and conclusions from the previous geotechnical reports include:
    - Shallow spread and/or strip footings for the reservoir tank located in the hills of the south side of Mare Island.
    - Consolidation settlement is one of the major issues for North Mare Island, and the area west of Tisdale Avenue along San Pablo Bay due to a thick layer of Young Bay Mud (YBM).
    - Recommended mitigation measures for consolidation settlement include (1) deep foundations founded in competent materials, (2) placement of surcharge fill to pre-consolidate the ground and reduce the settlement time by installing wick drains to accelerate the settlement, (3) use of lightweight fill materials to reduce magnitude of settlement, and/or (4) ground improvement, potentially including vibratory replacement, soil-cement mixing, and drilled displacement columns.
- Historical aerial imagery and published geotechnical documents from government agencies, including but not limited to historical topographic, soil and geologic, liquefaction susceptibility, and landslide maps were collected and reviewed. Collected information are presented in Plates 3.1 through 14.
  - Geologic and soil maps indicate that the flat low-lying areas on the northern and southern sides of Mare Island, along San Pablo Bay and along Mare Island Strait are covered with artificial fill underlain by YBM. The central low-lying areas of Mare Island are covered with late Pleistocene-age alluvial fan deposits. The hilly southern area of Mare Island is composed of late Cretaceous-age sandstones and shales of the Great Valley Complex.
  - Based on the information from those previous geotechnical reports, thickness of YBM ranges from less than 1 foot to 53 feet.
  - Portions of the project area underlain by Great Valley sandstone and shale, and alluvial fans overlying the sandstone and shale are considered to have a very low to low liquefaction susceptibility. Portions of the project area underlain by YBM, and artificial fill are considered to have a moderate to very high liquefaction susceptibility. Based on the

ENGEO letter provided to Lennar Mare Island, LLC in January 2016, the risk of liquefaction within planned development areas at Mare Island is generally considered low[1].

  - Areas along the shorelines of San Pablo Bay and the Mare Island Strait that have moderate to very high liquefaction susceptibility pose a potential hazard for lateral spreading.

- An example was provided on how to (1) determine the consolidation settlement due to compression of the YBM under placement of fill and the time it takes for 90% of consolidation to be completed; and (2) mitigate the impact of consolidation settlement on future improvements and accelerate the consolidation process.
  - Consolidation settlement due to compression of the YBM under placement of fill can range from 0.8 feet to 4 feet depending on the thickness of fill placement (5 to 10 feet) and YBM thickness (10 feet to 40 feet). The settlement is projected to take place over 0.8 year to 40 years for these examples depending on the thickness of YBM.
  - The amount of consolidation settlement can be reduced by implementing a surcharge program with wick drains. If a surcharge program alone is not feasible to mitigate the anticipated settlement, the existing fill can be removed and replaced with lightweight materials.
- An overview of the existing seawalls and discussion of the options and limitations associated with improving existing seawalls to meet future sea level rise was developed.
  - The seawalls support various marine infrastructure including quay walls, dry docks, and finger piers at the south end.
  - The existing seawalls, if retained, would need to be raised by between 3.5 feet and 5.5 feet for sea level rise (SLR) to year 2070, and between 7 feet and 9 feet for SLR to year 2100 based on present day guidance document projections.
  - There are two scenarios for protection from SLR. One is to raise the ground level to provide a nominal level of safety at the expense of initiating long term consolidation settlement. Second is to install a barrier seawall, which would need to be supported by a pumping system to lift stormwater and seepage flows from behind the barrier and be treated prior to discharge into the bay via an outfall.
  - Detailed geotechnical and hydrogeological investigations will need to be performed to determine design requirements of any new systems.
- An above-water, reconnaissance-level condition assessment of the Mare Island bulkhead walls and waterfront structures was completed in April 2025, covering 11 structural zones from Berth 1 to Berth 24. The assessment documented visible signs of deterioration such as corrosion, timber loss, and structural distress, using field observations and representative photographs. Condition ratings ranged from Satisfactory to Poor, with several zones requiring further evaluation due to limited access or visibility.
- Recommendations were developed to support future development and SLR planning on Mare Island, organized into three focus areas:

[1] Areas evaluated did not include North Mare Island.

- Geotechnical Investigations for Future Development: Site-specific subsurface investigations such as borings, CPT, laboratory testing, seismic refraction, and permeability testing are recommended to confirm ground conditions and support final design.
- Assessment of Existing Seawall Condition and Capacity: Building on the 2025 reconnaissance assessment, follow-up structural inspections and targeted repairs (e.g., sinkhole remediation) are recommended in the short term. In the medium to longer term, the scope of future structural assessments, should first be based on the planned development and land use.
- Sea Level Rise Adaptation Planning: Foundational studies (e.g., groundwater rise modeling, hydrogeologic investigations, seawall surveys) and strategic planning efforts (e.g., wharf facilities plan, land use coordination, marine engineering input) are needed to guide long-term shoreline adaptation.

6.2 PURPOSE

This chapter summarizes the known geotechnical conditions on Mare Island based on a desktop study of available information. No field investigations were performed as part of this project phase, but historical geotechnical borings were reviewed to the extent made available.

The geotechnical desktop study included several components, as listed below and further described in the following sections:

- Available Geotechnical Data – a list of geotechnical documents prepared for previous projects on Mare Island, which were utilized for this study.
- Geotechnical-related findings from Previous Reports – description of reference subsurface investigations and data (including borings, cone penetration tests, and laboratory tests) and associated analyses, findings, and recommendations from all documents.
- Geotechnical Description – description of the site condition, development of Mare Island, geologic, and seismic setting of the project area including geotechnical hazards.
- Consolidation Settlement – conceptual level estimates of settlement throughout the island based on YBM and hypothetical applied fill thicknesses.
- Seawall Overview – overview of the existing seawalls and discussion of the options and limitations associated with improving existing seawalls to meet future SLR.
- Recommendations – geotechnical and structural related recommendations to support future developments, and to address the SLR problem.

6.3 AVAILABLE GEOTECHNICAL INFORMATION

Table 6-1 provides a brief summary of the reports received from the Client Team to support the geotechnical desktop assessment. A detailed summary of the reports is provided in Section 6.4 and the reports are presented in Appendix 6A. A map of reference data is provided in Plate 2. All projects in Northern Mare Island are included in the area marked as “Reference Subsurface Data from ENGEO (2018)” on Plate 2 since ENGEO (2018) refers to the data from all projects in Northern Mare Island listed in Table 6-1.

Table 6-1. Existing Geotechnical Information Summary

Data Source	Details
Preliminary Geotechnical Study for 5.7 Million Gallon Water Reservoir, Mare Island (AGS, 1998)	• Geotechnical investigation and desktop study for the water reservoir on a hill in Southern Mare Island • Recommendations for the design of water reservoir including foundations, seismic design considerations, settlement, slope stability, etc.
Geotechnical Evaluation for Farragut Village on Mare Island in Vallejo, California (ENGEO, 2000)	• This report was not reviewed by Delve Underground.[1]
Preliminary Geotechnical Engineering Study and Consolidation Evaluation for North Mare Island Business Park (Levine Fricke, 2001)	• Geotechnical investigation and desktop study for the planning of new underground utilities and establishing preliminary building pad elevations in North Mare Island • Discussion regarding consolidation settlement of YBM due to fill placement and future developments • Trenching recommendations for new underground utilities
Geotechnical Exploration for Mare Island Infrastructure Improvements in Vallejo, California (ENGEO, 2003a)	• This report was not reviewed by Delve Underground.[1]
Geotechnical Exploration for Residential Developments 6B, 6C, 8A, 8B, 8C, and 8D on Mare Island in Vallejo, California (ENGEO, 2003b)	• This report was not reviewed by Delve Underground.[1]
Historic Site Topography and Geologic Characterization for Residential Development Areas 6B, 6C, and 8A through 8D on Mare Island in Vallejo, California (ENGEO, 2003c)	• This report was not reviewed by Delve Underground.[1]
Geotechnical Design & Materials Report for Mare Island/Route 37 Project (Parikh Consultants, 2003)	• Geotechnical investigation and desktop study for an addition to existing Route 37 bridge structure and realignment/widening of Railroad Ave and existing ramps • Addresses design of pavement sections, corrosion investigation, and bridge embankment recommendations
Supplemental Geotechnical Report for Mare Island Dredge Ponds Project (Olivia Chen Consultants [OCC], 2004)	• Soil parameters for Fill, YBM, Old Bay Mud, and Alluvium developed based on existing geotechnical data provided in Fugro West, Inc January 2003 report (not made available for review) • Analyses results for ground motion, consolidation settlement, stress distribution, and static and seismic stability
Preliminary Geotechnical Cost Summary for North Mare Island Development (ENGEO, 2006)	• Recent Bay Sediments thickness contours • Land use descriptions • Cost estimate for various mitigation and foundation alternatives for development of North Mare Island

Table 6-1. Existing Geotechnical Information Summary

Data Source	Details
Supplemental Exploration for Golf Basin Infrastructure Improvements on Mare Island in Vallejo, California (ENGEO, 2007a)	• This report was not reviewed by Delve Underground.[(a)]
Supplemental Exploration for Lima, Oscar, India, Foxtrot, Delta, and Charlie Drainage Systems Infrastructure Improvements on Mare Island in Vallejo, California (ENGEO, 2007b)	• This report was not reviewed by Delve Underground.[1]
Supplemental Geotechnical Exploration for Residential Development Areas 6W and 8W in Mare Island, California (ENGEO, 2010)	• This report was not reviewed by Delve Underground.[1]
Geotechnical Investigation Report for Project Zeus (Amec Foster Wheeler, 2017)	• Geotechnical investigations and desktop study for warehouse and manufacturing facilities in Northern Mare Island • Geologic and seismic hazards evaluation • Geotechnical recommendations including settlement, foundations, and geotechnical observation
Overview of Geologic Hazards and Geotechnical Engineering Land Development Considerations for North Mare Island (ENGEO, 2018a)	• Presents previous exploration data (including the data from some of the reports listed above) for the future mixed-use development of 157 acres on North Mare Island • Geologic and seismic hazards • Geotechnical engineering land development considerations • Preliminary recommendations for treatment of YBM and Fill, ground improvement, foundations, grading, and drainage • Thickness contours for Young Bay Mud Deposit
North Mare Island Land Development, Preliminary Geotechnical Recommendations (ENGEO, 2018b)	

(a) Included for completeness when made available after the May 2024 Draft Report review comment phase.

6.4 GEOTECHNICAL-RELATED FINDINGS FROM PREVIOUS REPORTS

Documents detailing previous subsurface investigations on Mare Island are listed in Table 6-1 and provided in Appendix 6A. The following sections summarize the key findings of the geotechnical investigations, and recommendations. The extents of the investigations within Mare Island are shown on Plate 2.

6.4.1 AGS (1998)

AGS's 1998 draft preliminary geotechnical study was conducted for the 5.7-million-gallon water reservoir tank located in the hills of the south side of Mare Island. The exploration included a total of eight borings to depths of about 20 feet to 44 feet below ground surface (bgs). Selected rock and soil samples from these borings were used to perform laboratory testing to evaluate dry density, moisture content, Atterberg limits, and grain size.

Bedrock was encountered during drilling at depths between 0 feet and 4 feet bgs. Bedrock at the site generally consists of highly weathered, poorly cemented, and very weak to weak interlayered sandstone, shale, and siltstone. AGS reported that except for two specific locations: between 18 feet to 19 feet in Boring B-3 and below 42 feet in Boring B-8, where it is too hard to continue drilling with a soil bit, almost all rock was drilled easily using a soil bit. The bedding has widely varying thickness and typically dips very steeply (70-90 degrees) and is overturned in some areas. Groundwater was not encountered in any of the borings.

Recommendations and conclusions made by AGS include:

- Shallow spread and/or strip footings were recommended to support the proposed reservoir tank.
- The nearby Franklin Fault with the maximum moment magnitude earthquake (Mmax) of 6.5 governs the ground motions at the proposed project site. The Franklin Fault appears to have a low level of seismic activity; however, additional studies would be required to accurately assess the hazard posed by the Franklin Fault.
- Soil liquefaction is very unlikely to occur at the project site based on the information from subsurface exploration.
- Slope stability assessments may be necessary for the steeper slopes in the southern part of Mare Island, although slopes appear to be stable.
- The majority of bedrock encountered during the subsurface exploration is weak and poorly cemented, and can be excavated using conventional, heavy earth moving equipment. However, heavy rippers or a jack hammer or hoe-ram may be required to break down and remove the harder rocks encountered in Boring B-3 and B-8.

6.4.2 Levine Fricke (2001)

Levine Fricke's (LFR) 2001 preliminary geotechnical engineering study and consolidation evaluation was conducted for master planning of redevelopment activities in North Mare Island Business Park. The exploration included four borings drilled to depths between 78 feet and 100 feet bgs, eight cone penetration tests (CPT) pushed to depths between 89 and 120 feet bgs, and laboratory testing on selected soil samples to evaluate moisture content, dry density, Atterberg limits, grain size, unconfined compressive strength, and unconsolidated-undrained triaxial compressive strength, and consolidation.

LFR's descriptions of subsurface condition encountered in the borings and CPTs are summarized below:

- Fill soils: generally soft/loose to very stiff sandy clay/clayey sand and sandy gravel with silt. Fill is present from the ground surface to depths between 5 and 11 feet bgs. Fill thickness contours are provided in the report.
- Bay Mud: highly plastic, expansive, and very soft to stiff silty clay underlying the fill. The thickness ranges from 20 to 50 feet. Plasticity index test results of the Bay Mud range from 36 to 51. Consolidation test results indicated that Bay Mud was generally normally consolidated with an exception of the upper portion of the Bay Mud in B-3, which was slightly over consolidated.

- Alluvial deposits: predominantly stiff to very stiff silty and sandy clay with some layers of medium dense silty or clayey sand and sandy or clayey silt. Alluvial deposits were encountered beneath the Bay Mud to depths of 120 feet.
- Groundwater levels stabilized after drilling to depths of 2 to 6 feet bgs.

Consolidation settlement analyses performed by LFR are as follow:

- Consolidation settlement calculations were performed assuming new fill thicknesses of 1, 3 and 5 feet placed on 5 to 10 feet of existing fill overlying 20 to 50 feet of Bay Mud.
- Settlement results range from 0.2 to 2.6 feet depending on the thicknesses of new fill, existing fill, and Bay Mud.
- Depending on Bay Mud layer thickness, 95 percent of the ultimate consolidation settlement is expected to occur over a period of 20 to 90 years.

6.4.3 Parikh Consultants (2003)

Parikh Consultants' 2003 geotechnical design and materials report details the geotechnical investigation conducted for the Mare Island/Route 37 Project, which involved additions to the existing Vallejo-Mare Island Bridge and realignment and widening of Railroad Avenue and on- and off-ramps for Highway 37. The exploration included drilling nine shallow borings to a maximum depth of 5 feet and two deeper borings to a depth of 75 feet. Laboratory testing includes grain size, moisture, dry density, unconfined compression, consolidation, and R-value. Reference borings were also reviewed.

Very stiff to hard clays were generally encountered in the upper 2 feet to 5 feet and were underlain by Bay Mud consisting of soft clay with occasional peat layers. The thickness of Bay Mud ranged from 33 feet to 44 feet. Groundwater was encountered between 1.5 feet and 5 feet bgs.

The report includes discussion on settlement of Bay Mud under embankment fill. Parikh Consultants concluded that the estimated settlement magnitude and time period is not acceptable and recommended the installation of wick drains at the off-ramp and bridge embankment to accelerate settlement and the placement of lightweight fill adjacent to wick drains along off-ramp and bridge embankment to decrease settlement.

6.4.4 Olivia Chen Consultants (2004)

OCCs' 2004 geotechnical report supplements an earlier report prepared by Fugro West, Inc. for their geotechnical investigation of the Mare Island Dredge Ponds project on the west side of the island. The Fugro investigation included borings, CPTs, and laboratory testing. They developed recommendations for static and seismic design criteria of levees at the dredge ponds, which are predominantly underlain by YBM and analyzed levee stability and settlement. OCC reviewed and evaluated the data from existing geotechnical reports prepared by Peter Kaldveer and Associates, Geotechnical Consultants, Inc., Subsurface Consultants, Inc. and Fugro, and prepared the reference report which supplements the analysis and findings of the Fugro report, including:

- Soil parameters
- Ground motion analysis
- Settlement, stress distribution, and static and seismic stability
- Liquefaction potential, and seepage and piping through or beneath the levees

OCC provides the soil parameters of the following soil layers:

- Existing Fill is divided into the Upper Fill and the Lower Fill. The Upper Fill refers to the existing levee fill that was placed in 1989 and 1990 and generally consists of excavated high-plasticity silt and clay YBM that was placed as compacted fill. The thickness of Upper Fill varies from about 5 to 15 feet. The Lower Fill generally consists of high-plasticity silt and clay YBM fill and has lower strength than the Upper Fill.
- YBM is divided into an upper and lower layer. The upper YBM consists of high-plasticity, slightly over consolidated silt and clay with isolated zones of low-plasticity soil. The lower YBM consists of high-plasticity, normally consolidated silt and clay with isolated zones of lower-plasticity soil. The total thickness of upper and lower YBM ranges from 10 to 50 feet.
- Old Bay Mud (OBM) encountered below YBM at the project site consists of over consolidated Pleistocene very stiff to hard silty clay.
- Alluvial deposits were encountered above or embedded within the OBM and generally consist of dense to very dense fine sands with varying proportions of clay, silt, and fine gravel.
- Proposed Levee Fill will consist of moisture-conditioned, placed and compacted existing dredge materials from the dredge pond.
- Dredge Materials predominantly consist of high-plasticity silt and clay YBM placed using hydraulic fill methods.

The levee was planned to be filled in up to three stages to maintain the stability of the levees during construction. Based on OCC analyses, consolidation settlements after each stage of levee fill range between 4 to 35 inches and the majority of the settlement is expected to occur over about 2 to 10 years after each stage of levee fill. Instrumentation would be installed for each stage of levee construction to monitor the amount and rate of settlement. The potential for seismic liquefaction and related hazard is negligible.

6.4.5 ENGEO (2006)

ENGEO's 2006 preliminary geotechnical cost summary for North Mare Island Development provides the proposed land usage breakdown, and mitigation and foundation alternatives for North Mare Island Development. The land usage for North Mare Island Development includes Office/R&D, Light Industrial, Mixed-Use, Retail, Road, Recreation and Conservation. A total of 12 foundation alternatives together with information on advantages, disadvantages, and total cost estimates were provided for buildings (i.e., Mixed-Used, Office/R&D, Light Industrial and Retail). The foundation types include spread footing, structural mat, piles, and a combination of piles and spread footings. The following alternatives, among these 12 alternatives, were considered as preferred alternatives for each usage based on the cost and applicable foundations:

- Mixed-Use and Office/R&D Building:
  - Driven piles with fill rework within top 4 feet.
  - Driven piles with surcharge settlement program within 20 feet of the building footprint for design fill load. 10-foot surcharge fills over a 1 to 2 years' time frame.

- Light Industrial Building:
  - Spread footings with fill rework within top 4 feet and surcharge fill program within 20 feet of the building footprint for design fill and structural load. 10-foot surcharge fills over 2 years' time frame.
  - Spread footings with surcharge fill program within 20 feet of the building footprint for design fill and structural load. 10-foot surcharge fills over a 6 months' time frame. Wick drain to facilitate surcharge fill program with 7-foot spacing.
- Retail Building:
  - Spread footings with fill rework within top 4 feet.
  - Spread footings with surcharge fill program within 20 feet of the building footprint for design fill and structural load. 10-foot surcharge fills over 2 years' time frame.

A total of five alternatives together with information on advantages, disadvantages, and total cost estimates were provided for mitigation for roads and utilities. Among these five alternatives, the alternative with surcharge settlement program was considered as a preferred alternative based on the cost and applicable foundation. Surcharge settlement program includes surcharging road areas with 5 feet of surcharge fill for anticipated traffic load.

6.4.6 Amec Foster Wheeler (2017)

Amec Foster Wheeler's 2017 geotechnical investigation report provides the results of a geotechnical investigation conducted for Project Zeus, which aimed to redevelop 144 acres of North Mare Island. In addition to reviewing reference data and reports, the investigation included six soil borings to depths of up to 111.5 feet bgs and 27 CPTs to depths of up to 118 feet bgs. Laboratory testing includes moisture; density; Atterberg limits; modified Proctor; consolidation; minimum resistivity; pH, chloride, and sulfate tests; expansion index; R-value; direct shear; and unconsolidated-undrained triaxial compressive strength.

The report provides the following subsurface information based on the exploration and testing program in North Mare Island:

- The fill thickness ranges between 3 to 12 feet and is variable in composition and parameters.
- YBM, described as a very soft to soft fat clay, underlies the fill layer and varies in thickness from less than 1 foot to 53 feet. YBM is generally shallower to the west and south, and thicker and deeper to the north and east.
- Old Bay Clay (OBC), underlying the YBM in most locations, is described as a soft to stiff fat clay with interbedded sand and silt. The bottom of OBC was encountered at depths of approximately 90 to 95 feet to the south, and over 120 feet to the north. On the west side of the project site, a relatively continuous layer of loose sand and sandy silt was encountered at a depth of approximately 25 to 30 feet between YBM and OBC.
- Older alluvium underlying the fill, YBM and OBC generally consists of very stiff lean clay with some fine sand and silt.
- Bedrock varies in depth from approximately 95 feet on the south end of the site to 240 feet on the north side of the site at Highway 37. Bedrock at the site is generally composed of marine sedimentary rocks of the Great Valley Group.
- Groundwater was encountered at depths between 2 to 10 feet bgs.

Amec Foster Wheeler (2017) discusses settlement induced by liquefaction and compression of YBM and includes estimated settlements of new buildings constructed on YBM if mitigation measures are not taken. The liquefaction induced total settlement on the order of 2 inches and differential settlement on the order of 1 inch are possible at the project site. Consolidation settlement due to YBM can range from a few inches to greater than 7 feet depending on the thickness of YBM, applied foundation load and time after construction. Recommended mitigation measures for settlement include deep foundations founded in OBC, placement of surcharge fill with installation of wick drains, use of lightweight fill materials, and soil-cement mixing.

6.4.7 ENGEO (2018)

ENGEO's 2018 overview of geologic hazards and geotechnical engineering land development considerations report and preliminary geotechnical recommendations report were developed for the planned future redevelopment of 157-acre development area on North Mare Island. ENGEO reviewed reference subsurface data and previous investigations and reports for North Mare Island, including several of the previously summarized reports in Section 6.1. The findings presented in ENGEO (2018a and 2018b) are similar to that of Levine Fricke (2001), Parikh Consultants (2003), and Amec Foster Wheeler (2017). No additional field explorations were conducted for the report.

These reports include discussion on the compressibility of YBM, and the recommendation of various measures to mitigate settlement, including:

- A surcharge fill program with wick drains installed 5 feet on center, placed in a triangular pattern to accelerate settlement and complete the program within about 7 to 9 months. ENGEO recommended establishing a settlement-monitoring program as part of the surcharge fill program.
- Compensation loading by removing existing fill and replacing with lightweight fill where new fill is placed or a lightweight building is constructed.
- Ground improvement, potentially including vibratory replacement, soil-cement mixing, and drilled displacement columns.

6.5 GEOTECHNICAL DESCRIPTION

Project elements being considered in this geotechnical study are shown in Plate 1 and include SLR protection areas, proposed fill areas, proposed seawalls, and the new proposed Tyler Road alignment (see Chapter 7 – Storm Drainage and Flood Control). The following sections summarize geotechnical-related findings of the study.

6.5.1 Site Condition

Mare Island is a generally flat and low-lying peninsula bounded by San Pablo Bay to the west and the Napa River/Mare Island Strait to the east. The southern part of the island has steeper terrain. The site includes vacant land being used to dispose of dredged material from the Mare Island Strait, existing Naval buildings and foundation remnants, roads, residential and commercial properties, and underground utility corridors.

Based on SLR evaluation (see Chapter 7 – Storm Drainage and Flood Control), areas of the island below 10 feet elevation are within the current Federal Emergency Management Agency (FEMA) 100-year flood zone. Quay walls were constructed along the shoreline of Mare Island Strait before the 1920s. One of the

preliminary approaches for SLR protection is to raise the existing quay wall, however, structural, and geotechnical evaluations are required to assess the condition of these existing quay walls before they can be relied upon for SLR protection.

6.5.2 Development of Mare Island

The Mare Island Naval Shipyard was established on Mare Island in 1854 and was the first Navy base on the West Coast of the United States. It is reported that in 1854, Mare Island had only one house, a few buildings, and a floating dry dock constructed by a shipping company. Fills of undocumented thickness and composition were placed on Mare Island throughout the early and mid-1900s as the base became one of the most prominent ship- and submarine-building facilities in the world. The Mare Island Naval Shipyard was officially closed in 1996, and the Navy has transferred much of the land to the City of Vallejo and/or State lands over time for reuse and redevelopment, however some land remains in Navy ownership.(Arevalo, 2022).

Historical topographic maps and aerial photographs of the project area are provided in Plates 3.1 and 3.2 and Plates 4.1 and 4.2, respectively. The topographic maps date back to 1902 and the aerial photographs date back to 1947. Interpretation of these maps and photographs suggests the following development of Mare Island:

- Roadway development on Mare Island dates back to at least 1902. Roadway pavements of this era may have been constructed with Portland cement concrete (PCC) which have since been covered over and concealed with asphalt concrete (AC) overlays. Significant expansion of roadways and construction of buildings and other structures took place between 1902 and 1916 as the Naval Shipyard expanded.
- Levee roads were constructed on the western side of the island before 1947. This portion of the island was used as disposal ponds for material dredged by the U.S. Navy. The dredge ponds can be seen most clearly in the 1982 and 1993 aerial photos and are evaluated in OCC (2004). According to OCC (2004), several of the dredge ponds were proposed for continued use as an upland disposal facility by the City of Vallejo and the California State Lands Commission to dispose dredged material throughout the San Francisco Bay Area.
- The roadway and bridge currently known as CA-37/Sears Point Road and the Vallejo-Mare Island Bridge are mapped as early as 1916.
- The interchange at Walnut Avenue was constructed in the 1950s. Additions to the bridge structure and realignments of on- and off-ramps have occurred since the construction in the 1950s. One such project is addressed in Parikh Consultants (2003).
- The roadway or bridge near A Street was mapped in 1916 and it was replaced by the roadway or bridge currently known as Mare Island Causeway Bridge before 1942.
- The railroad tracks along northeast side of the island were mapped as early as 1916 and majority of the tracks are still visible in the present-day map. Railroad tracks along the west and south side of Mare Island can be seen in the 1942 map, however, they are not visible in the 1947 aerial photo or the 1949 map. Even though traces of abandoned tracks are visible along Tyler Road in the present-day map, it is not clear whether the rest of the tracks were fully removed or paved over.

- The placement of fill on Mare Island can be observed throughout the early and mid-1900s. Between 1902 and 1916, the land north and northwest of Mare Island was significantly increased through the placement of fill. Fill placement along the eastern shoreline of Mare Island also occurred between 1902 and 1916. By 1949, the northern, western, and southern portions of the island had mostly been built out to their present-day extents.
- Construction of piers at the shoreline of Mare Island Strait began prior to 1902, and several additional piers were constructed by 1916. The shoreline along the Strait began to resemble its present-day shape by 1949, likely due to the continued installation and rehabilitation of seawalls.
- Based on Naval Facilities Engineering Command (NAVFAC) Design Manual (DM) 29.3, the dry docks on the eastern shoreline of Mare Island were constructed between 1891 and 1942 using concrete and granite.
- Seawall structure along the east side of Mare Island extends from the Mare Island Causeway, south for approximately 2 miles. The seawalls support various marine infrastructure including quay walls, dry docks, and finger piers at the south end. The earliest engineering drawings of the seawalls are dated 1921 and indicate repairs performed at that time, therefore the original wall construction precedes 1921. Later drawings indicate that the walls were further rehabilitated and extended in the 1940's and 1980's.
- The western half of Mare Island remains largely undeveloped in the present day and covered by ponds used to dispose of material dredged from the Mare Island Strait, with the exception of few small structures and levee roads. Residential development has been underway in the central portion of the island between 1993 and 2009. By 2009, the central portion of the island had mostly been built out to its present-day extents.
- The U.S. Navy has supported a remediation program since the 1990s to remove unexploded ordinances (UXO) and allow for safe civilian use of Mare Island (Daily et al., 2000). Some environmental sites have yet to be transferred from the U.S. Navy as of this report writing and cleanup efforts, including removal of UXO, are ongoing. Areas identified by the Navy as still containing UXO material are primarily the dredge ponds in the western and southern portions of Mare Island, and the shoreline along the Mare Island Strait (Anderson, 2020). All new development areas and open spaces accessible to the public will have undergone cleanup efforts by the U.S. Navy before development begins and deemed safe for public use by the Department of Toxic Services (DTSC), the lead agency in the Mare Island environmental cleanup program.

6.5.3 Geology

Mare Island is located in the central portion of the Coast Range geomorphic province, which is characterized by subparallel northwest trending faults, mountain ranges, and valleys. The project area is within the Great Valley Complex. The Great Valley Complex is part of an amalgamated and highly deformed terrane and is composed of sandstone, conglomerate, and shale originally deposited on top of ophiolite. The rocks of the Great Valley Complex are accreted and deformed materials from Jurassic-age oceanic crust overlain with thickly bedded turbidites (Graymer et al., 2002).

Soil and geologic maps of the project area are provided in Plates 5.1, 5.2 and 6. These maps generally indicate that the flat low-lying areas on northern and southern sides of Mare Island, along San Pablo Bay and along Mare Island Strait are covered with artificial fill and YBM. The central low-lying areas of Mare

Island are covered with late Pleistocene-age alluvial fan deposits. Some of these low-lying areas are composed of naturally accumulated sediments due to the construction of dikes along the southern portion of the island in the late 1800's (Daily et al., 2000). The hilly southern area of Mare Island is composed of late Cretaceous-age sandstones and shales of the Great Valley Complex. This is consistent with (1) information provided in the reports summarized in Section 6.4 and provided in Appendices 6A, and (2) the thickness of YBM in the project area as mapped in Plate 7.1 (note, the contours shown in Plate 7.1 represent the thickness of YBM in feet).

Average seismic shear wave velocity for the upper 30 meters or 98 feet (Vs30) is a key parameter for evaluating the dynamic properties of soils. It is used in the National Earthquake Hazards Reduction Program (NEHRP) provisions and the building codes to separate sites into different site classes. The assumption of the Vs30 is that sites in the same Site Class will respond similarly to a given earthquake. A correlation of Vs30 map data provided in Plate 8 with Site Classification descriptions from ASCE 7-22 presented in Table 6-2 suggests the following:

- Areas mapped as artificial fill in Plate 6 are generally identified as Site Class D in Plate 8 and are generally consistent with loose sand or medium stiff clay to medium dense sand or stiff clay.
- Areas mapped as YBM in Plate 6 are generally identified as Site Class DE in Plate 8 and are generally consistent with soft to medium stiff clay.
- Areas mapped as alluvial fan deposits in Plate 6 are generally identified as Site Class CD in Plate 8 and are generally consistent with dense sand and very stiff clay.
- Areas mapped as Great Valley Complex sandstone and shale in Plate 6 are generally identified in Plate 8 as Site Class C (very dense sand or hard clay). This classification is consistent bedrock encountered in the reference boring logs and bedrock descriptions provided in AGS (1998).

Table 6-2. Site Classification (Adapted from ASCE 7-22 Table 20.2.1)

Site Class	Range of Vs_{30} (ft/sec)	Generic Description
A	> 5,000	Hard rock
B	> 3,000 to 5,000	Medium hard rock
BC	> 2,100 to 3,000	Soft rock
C	**> 1,450 to 2,100**	**Very dense sand or hard clay**
CD	**> 1,000 to 1,450**	**Dense sand or very stiff clay**
D	**> 700 to 1,000**	**Medium dense sand or stiff clay**
DE	**> 500 to 700**	**Loose sand or medium stiff clay**
E	≤ 500	Very loose sand or soft clay

Bold signifies site classes for the different Mare Island ground conditions

6.5.4 Young Bay Mud

Most of the project elements are underlain by YBM (see Plates 6, 7.1, 7.2, and 7.3). YBM typically consists of soft, compressible, organic-rich silt and clay that is prone to consolidation settlement. Portions of the

project that are underlain by thicker sections of YBM will experience higher levels of consolidation settlement with any new added loading.

Some of the reference reports listed previously provide YBM thickness information based on the subsurface exploration. Based on the information from those previous geotechnical reports, thickness of YBM ranges from 10 to 50 feet within the dredge ponds area (areas within yellow polygon in Plate 2) and from less than 1 foot to 53 feet in north side of Mare Island (areas within green polygon in Plate 2). Plates 7.1 and 7.2 also mapped the range of YBM thicknesses underlying project elements and depth to bottom of YBM, respectively. However, information provided in Plates 7.1 and 7.2 maps are very general and provide indicative information as to what could be encountered along bay margins being the maps were developed for the whole San Francisco Bay Region using the available published data in the area (Goldman, 1969). Plate 7.3 provides updated information on the approximate thickness of YBM deposits contours based on site-specific studies completed by ENGEO for the City of Vallejo-Mare Island Specific Plan. Review of site-specific boring data and Plate 7.3 suggest the YBM thicknesses and depths in Plates 7.1 and 7.2 maps do not correlate very well with the ENGEO -developed site-specific data, especially within the redevelopment areas in the eastern half of the island which indicates an average thickness of about 20 to 30 feet. Therefore, this information should only be used for the preliminary planning of the project and site-specific subsurface exploration should be performed to obtain data for the areas where there is no site-specific subsurface information.

6.5.5 Faulting and Seismicity

No Holocene-active faults[2] cross the project area. According to Plates 6 and 9, inferred traces of the Franklin Fault cross the western portion of Mare Island and east of Mare Island along the Mare Island Strait. The Franklin Fault is not classified as an active fault; however, the fault may be part of a fault system that could experience movement in the future. The nearest active faults to the project area are the West Napa, Concord-Green Valley, and Hayward Faults (Plate 9). At their nearest points, the West Napa and Concord-Green Valley Faults are located approximately 4.5 miles and 10 miles east of the project area, respectively, and the Hayward Fault is located approximately 8 miles west of the project area. The project area will be subjected to ground shaking during earthquakes on the West Napa, Concord-Green Valley, and Hayward Faults and on other active seismogenic sources in the region (Plate 10). Plate 11 depicts three earthquake scenarios that may generate relatively high seismic shaking in the project area. The anticipated peak ground acceleration (PGA) in the planned improvement area calculated using U.S. Seismic Design Maps (ASCE 7-22) ranges between 0.65g to 0.87g based on the Site Class mapped in Plate 8. The anticipated PGA provided herein are for preliminary planning purposes and are based on the current building code. For future design-level studies, the PGA should be determined based on the site-specific site condition information using the applicable building codes at the time of the design. The actual ground acceleration that will be experienced in the project area during earthquakes will be a function of variables such as magnitude, epicenter distance, mode and direction of seismic wave propagation (e.g., directivity), soil amplification or attenuation, and near source factors. Modified Mercalli Intensity descriptions for ground shaking with average peak accelerations exceeding 0.5g include damage to well-designed structures and broken underground pipes (Plate 12).

[2] Where a Holocene-active fault is defined by the State of California as a fault with known surface displacement within the last 11,000 years (CGS 2018).

6.5.6 Liquefaction

Liquefaction is a phenomenon in which soils lose internal strength because of increased pore pressure generated by cyclic loading. Cyclic loading is commonly induced by ground shaking during earthquakes. Soils prone to liquefaction are saturated, noncohesive, relatively clay-free silt and sand layers of very loose to loose density. Liquefaction hazards for the project area were evaluated using the liquefaction susceptibility map provided in Plate 13 for preliminary planning purposes and information from reports summarized previously and provided in Appendix 6A.

According to the liquefaction susceptibility map provided in Plate 13, portions of the project area underlain by Great Valley sandstone and shale are considered to have a very low liquefaction susceptibility. Areas where alluvial fans overlay the sandstone and shale are considered to have a low liquefaction susceptibility. Portions of the project area underlain by YBM are considered to have a moderate liquefaction susceptibility. Portions of the project area underlain by artificial fill have very high liquefaction susceptibility. However, this liquefaction susceptibility map only provides a general categorization over the whole central San Francisco Bay Area region based on a number of risk factors and no bore log data was used in the characterization of site-specific soils, or performance of soils (ENGEO, 2016). Based on the ENGEO letter provided to Lennar Mare Island, LLC in January 2016, ENGEO and other geotechnical engineering companies have performed site-specific, design-level geotechnical studies at Mare Island. They have concluded that the risk of liquefaction within planned development areas at Mare Island[1] is generally considered low, especially for areas where ground modification is being considered to mitigate the effects of YBM settlement (e.g., preloading and wick drains).

Minor vertical settlement associated with liquefaction is generally not a significant hazard to buried pipeline. Lateral spreading (e.g., landslides that commonly form on gentle slopes that have rapid fluid-like flow movement caused by earthquake-induced liquefaction) on gently sloping ground represents the most pervasive and damaging type of liquefaction-related effect on pipelines due to large earthquakes (Bartlett and Youd, 1995; CGS, 2008). Lateral spreading processes usually occur in liquefiable soil near vertical free-faces (e.g., creek banks and along the shoreline) that are not topographically confined. Based on the liquefaction susceptibility map provided in Plate 13, areas along the shorelines of San Pablo Bay and the Mare Island Strait that have moderate to very high liquefaction susceptibility pose a potential hazard for lateral spreading. However, according to the ENGEO letter provided to Lennar Mare Island, LLC in January 2016, no signs of liquefaction including lateral spreading were observed during a geotechnical engineering reconnaissance of Mare Island hours after the 2014 South Napa Earthquake approximately 9 miles north of Mare Island that generated a ground acceleration of 0.38 g at the seismic station on Mare Island. Therefore, lateral spreading potential at planned development areas at Mare Island is considered low.

The existing quay walls may provide limited protection against lateral spreading but would not have been designed for liquefaction and would almost certainly be overloaded and unstable in a design event aligned with current codes and guidance documents. The potential for lateral spreading adjacent to the quay walls will need to be evaluated with site-specific studies to assess potential free face conditions resulting from historical channel deepening.

6.5.7 Landslides

The best available predictor of where landslides might occur in the future is the distribution of past landslides. The landslide map prepared by Wentworth et al. (1997) provided in Plate 14 illustrates the distribution of previous slope instabilities in the project area. Based on this map, the southern area of Mare Island mapped as Great Valley sandstone and shale is located within a “few landslides” zone, which

consists of a few large landslides and scattered small landslides. Two localized areas are defined as a "mostly landslide" zone, which consists of the largest and most concentrated landslides.

6.6 CONSOLIDATION SETTLEMENT

For conceptual design purposes, the consolidation settlement due to compression of the YBM under placement of fill and the time it takes for 90% of consolidation to be completed can be calculated using the YBM thickness provided in the previous investigation reports or Plate 7.3 and the charts provided in Plate 7.4. The estimated consolidation settlement and the time it takes to complete 90% of consolidation for a few scenarios are provided in Table 6-3 as examples. In these cases, thickness of YBM was estimated using Plate 7.3. The thickness of YBM provided in the previous geotechnical report can also be used in lieu of the thickness provided in Plate 7.3 to estimate the settlement of YBM for areas where there is reference subsurface information.

Table 6-3. Conceptual Consolidation Settlement Estimates

Young Bay Mud Thickness, (ft)	Artificial Fill Thickness, ft	Estimated Consolidation Settlement, ft[(a)]	Time for 90% Consolidation, yr[(a), (b)]
10	5	0.8	0.8
	10	1.6	
20	5	1.4	5
	10	2.6	
40	5	2.1	40
	10	4.0	

(a) See charts on Plate 7.4.
(b) Time estimates assume there are no additional measures taken to accelerate the consolidation process and pore water pressures dissipate through a single drainage boundary.

A surcharge program can be implemented to mitigate the impact of consolidation settlement on future improvements. The program includes placing surcharge fills above design grades in areas of the project where pre-consolidation is required to reduce the amount of settlement. The surcharge fills will remain in place for a period long enough to allow the required degree of consolidation to be achieved, such that the impact of settlement is significantly reduced on future improvements. The time it takes to complete 90% consolidation settlement provided in Table 6-3 were estimated assuming no additional measures were taken to accelerate the consolidation process. Vertical wick drains, similar to what was proposed in ENGEO (2018), could be installed to accelerate the consolidation settlement process (see Figure 6-1). Generally, the owner and the design engineers determine the time duration required to complete the consolidation settlement process, and the spacing and layout of the wick drains is designed to achieve consolidation settlement within the determined time duration. In areas where surcharge program alone is not feasible to reduce the settlement, the existing fill can be removed and replaced with a lightweight fill or lightweight cellular concrete to compensate for the loading being added such that there is no net increase in load imposed on the YBM.

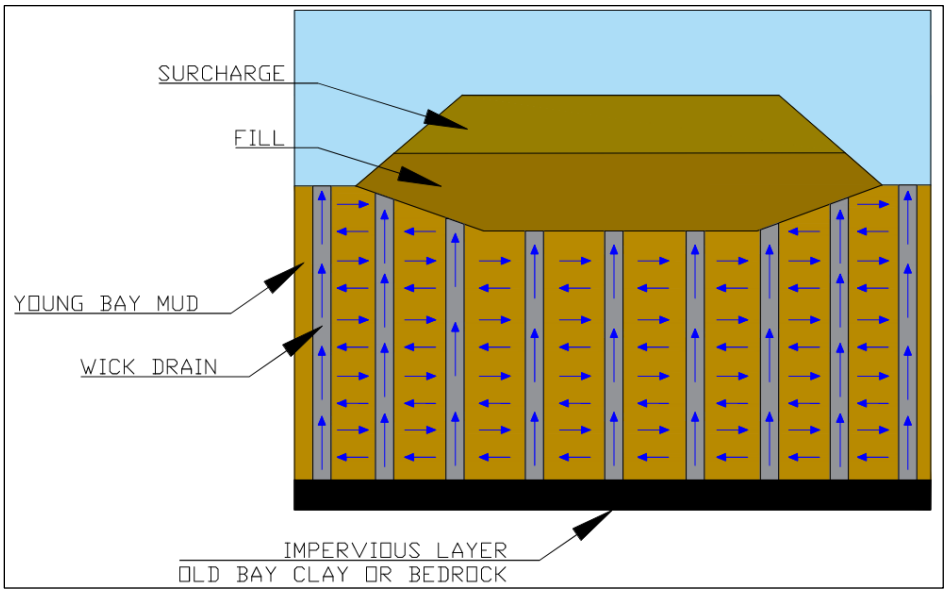

Figure 6-1. Schematic Figure of a Surcharge Program with Wick Drains

6.7 SEAWALL OVERVIEW

This section provides an overview of the proposed reuse of existing seawalls that will be needed to address SLR on Mare Island. A summary of the reconnaissance-level assessment of the seawall bulkhead is also included based on the field inspections conducted in April 2025.

6.7.1 Existing Seawalls

The existing seawall is located on the east side of the Island at functioning wharves, jetties, and dry docks. The seawalls support various marine infrastructure including quay walls, dry docks, and finger piers at the south end.

The existing seawalls have been built and improved at different times over the last century. The structural drawings that are available indicate that some walls were repaired in 1921 and, therefore, must have been built even earlier. The drawings indicate several layers of rehabilitation from the 1920s through the 1940s and the latest drawings for improvements are from the 1980s. Rehabilitation included widening of the wharf platforms, raising the elevation of land-based infrastructure, deepening and rebuilding sheet pile walls to accommodate dredging.

The existing seawalls use several structural materials and methods. An example of the 1923 design for a section of wharf rehabilitation is shown in Figure 6-2. Timber sheet piles and bearing piles support the original timber platform. New concrete sheet piles, new timber bearing piles and a concrete retaining wall and relieving platform were used to improve the structure. The structural drawings indicate that over the years new concrete sheet piles, new concrete bearing piles, and steel sheet piles were commonly used to improve the wharf structures.

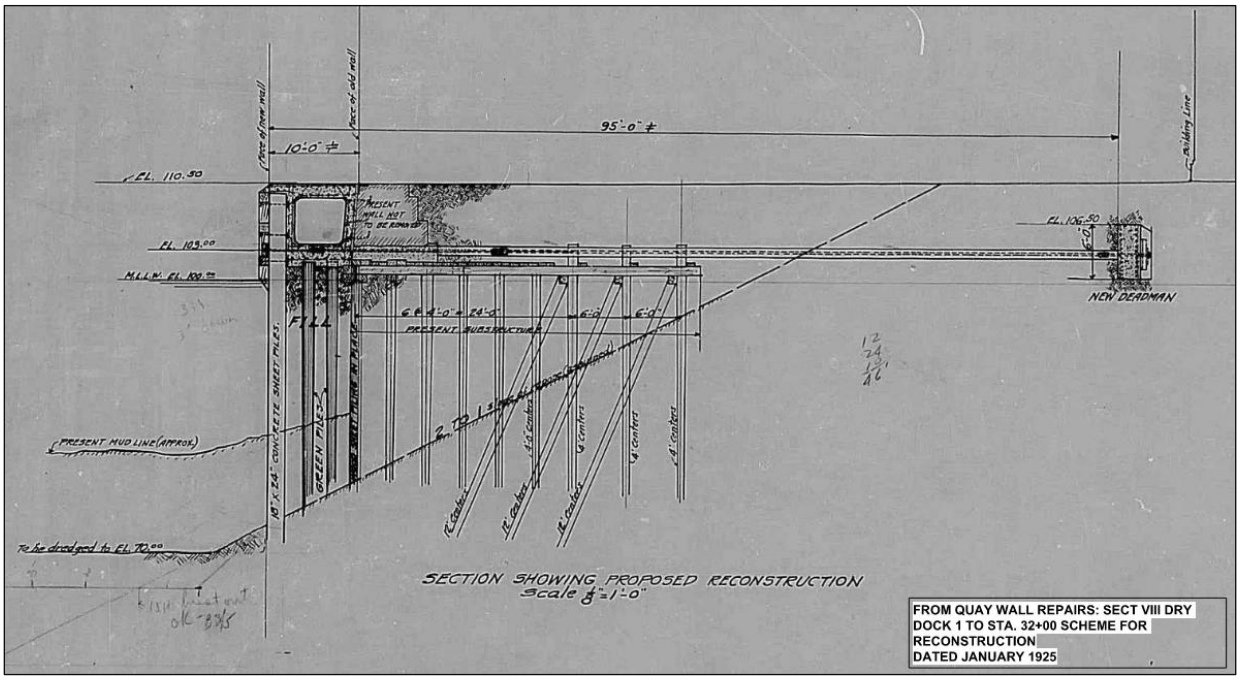

Figure 6-2. Typical Original Wharf Structural Repairs in 1925

6.7.2 Design Basis for Proposed Seawall Solutions

The following sections of this report utilize currently established design criteria to develop proposed infrastructure improvements, specifically the quay walls, to support the Mare Island storm drainage and flood control systems. To demonstrate a tangible infrastructure solution for existing island quay walls, present day design criteria from guidance documents were used for the preliminary assessment of repurposing existing quay walls as seawalls for flood protection.

This report acknowledges that the design criteria used to assess the quay walls are based on guidelines in guidance documents that are continually evolving as the knowledge base expands (e.g., the State of California Sea Level Rise Guidance document and the Solano County Water Agency Hydrology Manual, upon which the VFWD rainfall design storms are based). As the scientific understanding of the impacts of climate change are further refined, it is expected that international, federal, State, and local agencies will continue to refine/validate prior projection models on climate and SLR trends to further inform updates to the guidance documents. Development of future seawall solutions to address storm drainage and SLR on Mare Island should be based on use of the most current and best available science at the time any specific project elements are designed and constructed. While this report emphasizes capital improvements, solutions should be integrated with site-specific and community-level adaptation planning, regional and cross-jurisdictional collaboration, and consider the site-specific land uses and unique characteristics of Mare Island.

6.7.3 Seawall Elevations Required for Future SLR

As discussed in the Chapter 7, the design SLR elevations (including tide and design storm surge effects) are El. 13.5 feet NAVD88 and El. 17.0 feet NAVD88 for years 2070 and 2100 respectively. If 2.5 feet of freeboard is added to these elevations, the design elevations of the top of the seawalls would be El. 16.0 feet NAVD88 and 19.5 feet NAVD88 for years 2070 and 2100 respectively.

The actual design of seawall elevations will depend on many factors including design freeboard, air draft of mooring vessels (where applicable), final design storm surge effects, and near term/long term land use. However, the elevations above are considered a conceptual guide.

The existing seawall elevations are indicated on existing structural design drawings as recompiled by ZFA Structural Engineers (ZFA, 2019) between 110.5 feet and 112.7 feet, where 100 feet is indicated as mean lower low water (MLLW) level (and in one instance mean low water (MLW) level). These drawings are dated between the 1920s and the 1980s. However, their depicted tidal water levels have changed over time and therefore cannot be converted directly to reference current datum NAVD88 necessitating a detailed survey of the seawall elevations to determine accurate existing wall elevations referenced to current datum. In addition, the walls and coastal infrastructure will have settled over time. Nevertheless, based on a rudimentary assessment, we believe that conversion to NAVD88 datum is approximately minus 100 ft. Therefore, the existing seawalls are likely between El. 10.5 feet NAVD88 and EL 12.7 feet NAVD88, not considering possible settlement since construction.

Based on the above, the existing seawalls, if retained, would need to be raised by between 3.5 feet and 5.5 feet for SLR to 2070, and between 7.0 feet and 9.0 feet for SLR to year 2100.

6.7.4 Seismic Loading

The location of Mare Island dictates that the structure could experience high seismic demands, in addition to a moderate to high potential for liquefaction and potential global slip/rotational ground deformations. Liquefaction will result in higher load demand from the ground on the seawalls as well as causing destabilization of bearing pile and sheet pile toes. The existing structure would not be expected to have been designed for seismic loading, nor detailed for ductility demands required by current codes and standards. Even if the condition of the structure was reasonable, it may not be feasible to improve the existing seawall structures to withstand the seismic demands required by current codes, due to the age of the structures and the methods used to detail the previous designs. An exemption or relaxation of the seismic design event and acceptance of a lower hazard to any newly constructed infrastructure and buildings would be needed to repurpose the existing walls.

6.7.5 Seawall Foundation Conditions

The geology of Mare Island is described in detail earlier in this chapter. There is limited geotechnical information available at the locations of the existing seawalls along the east side of the Island. However, based on available geotechnical information, it is expected the upper 40 feet to 50 feet of ground is comprised of variable fill over YBM, and this to be underlain by a more robust material, variously described as stiff to very stiff/dense, and comprising silt, sand and clay, and OBC. Bedrock is expected to be 95 feet to 240 feet below the ground surface.

The existing sheet piles at the existing seawalls extend to between 55 feet and 85 feet below the ground surface. The bearing pile toes are deeper than the sheet piles and extend as deep as 115 feet at some wharves. This indicates that all piles key into the stiffer, denser material, as would be expected for toe and bearing support.

There are two scenarios for protection from SLR. One is to raise the ground level to provide a nominal level of safety at the expense of initiating long term consolidation settlement. Otherwise, a barrier seawall option is possible, but this will need to be supported by a pumping system to lift stormwater and seepage flows from behind the barrier into the bay via an outfall.

If the future, retrofitted seawalls or the new seawalls are required to protect the island from higher ground water levels during storm events or even due to SLR alone, then it is expected that the toes of the new sheet piles will need to key deeper into the ground to cut off or substantially limit seepage under the wall toe, that is acting as a plug/barrier. However, limiting seepage alone is not sufficient to prevent ground water levels from rising in the long term. As the SLRs, the local ground water table will rise along with it since there will be opportunities for water to move around the ends of the structure and also well upward through the soil and rock structure. Even for a completely enclosed sheet pile wall system, one would expect to have to draw the water down on the island side to prevent the ground water level from rising over time as a result of SLR.

To summarize, the ground would need to be raised to a level above the water level or a pumping/dewatering system would be required, even if the rate of seepage can be somewhat reduced by deeper sheet pile wall toes. These new sheet piles may need to extend deeper than 100 feet below the existing ground level.

Detailed geotechnical and hydrogeological investigations will need to be performed to determine hydraulic conductivity and estimate seepage rates for different geotechnical conditions and toe wall depths. These studies will partially inform the feasibility and design of future pumping systems in long term low-lying areas.

6.7.6 Categories of Seawall

Three main categories of seawalls are considered for the redevelopment of Mare Island, each required to meet specific criteria. These are presented in Figure 6-3 and described below:

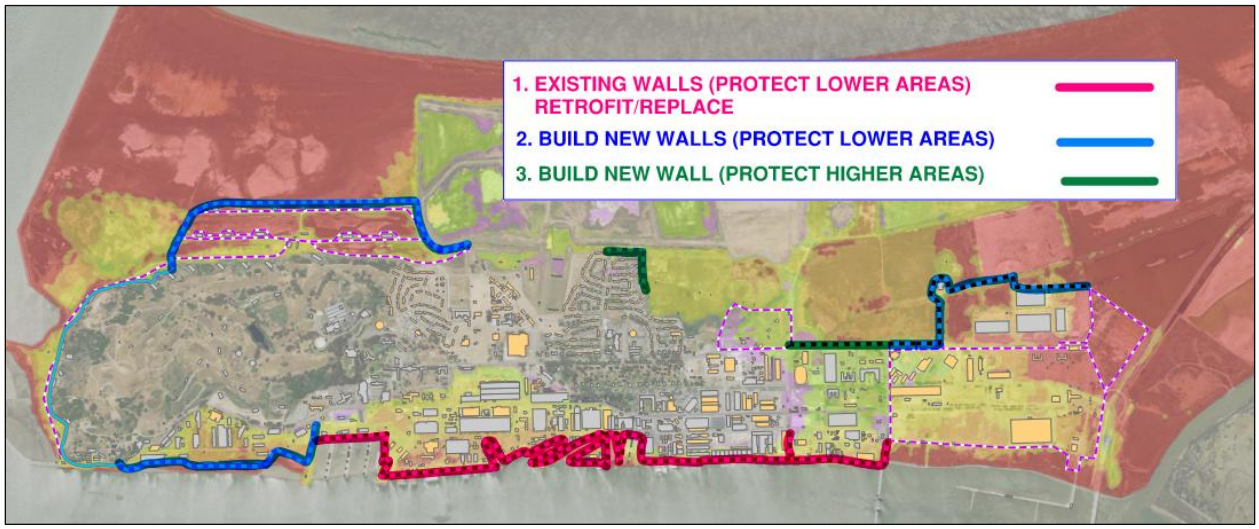

Figure 6-3. Mare Island Redevelopment – Three Main Categories of Seawalls

6.7.6.1 Existing Seawalls

These walls are along the east side of the Island at functioning wharves, jetties, and dry docks. They would need to be retrofitted, or demolished and rebuilt, to meet the following requirements:

1. Raise existing walls by between 3.5 feet and 5.5 feet for SLR to year 2070 and by between 7 feet and 9 feet for SLR to year 2100.
2. Protect low-lying marine infrastructure from inundation.
3. Deepen toe of wall to reduce seepage and maintain low water levels landside.
4. Design stormwater and seepage drainage network with pumped outfall discharge (or discharges – distributed network option).
5. Withstand earthquake loading (including liquefaction.)
6. Maintain vessel access (berthing, mooring, dry dock function, where needed for future land uses.)
7. Maintain man/vehicle/crane access to mooring vessels from low infrastructure behind.

6.7.6.2 New Seawalls (Low Lying Areas)

These new seawalls would protect low lying areas that have existing buildings that are to be preserved. Requirements include:

1. Provide top of wall elevation to prevent SLR inundation (Include minimum 2-3 feet freeboard.)
2. Toe depth to mitigate seepage.
3. Withstand earthquake loading (including liquefaction.)

6.7.6.3 New Seawalls (Higher Areas)

These new seawalls would protect higher elevation areas, but areas that are still low enough to require protection from future SLR that have existing buildings that are to be preserved. Requirements include:

1. Provide top of wall elevation to prevent SLR inundation (Include minimum 2-3 feet freeboard.)
2. Withstand earthquake loading (including liquefaction.)

6.7.7 Additional Seawalls

If the ground water level of Mare Island rises with SLR, additional walls may be required at the following locations and are indicated on Figure 6-4:

1. Boundaries of new fill and protected low-lying areas.
2. Boundaries of higher ground and protected low-lying areas.

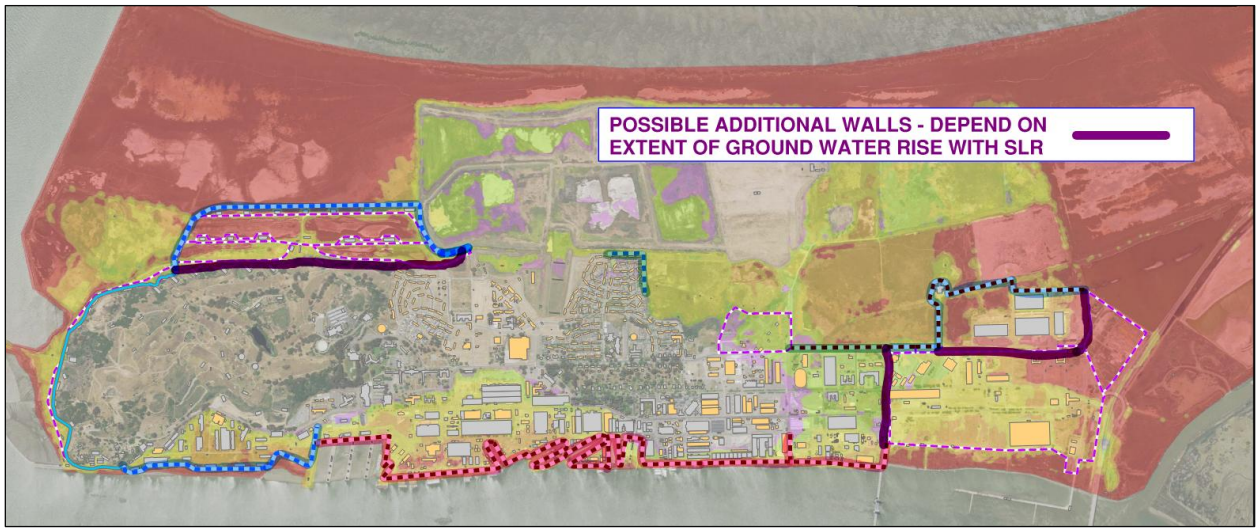

Figure 6-4. Mare Island Redevelopment – Seawalls if Ground Water Rises with SLR

6.7.8 Retrofitting 100-Year-Old Seawalls

It is common for seawalls to see many cycles of rehabilitation throughout their operational lives, apart from consideration of SLR. Constantly increasing vessel sizes require deeper dredging and therefore deeper and stronger walls, as well as increasing berthing, mooring and crane load demands on the wharf structure. The feasibility of retrofitting the existing seawalls to protect Mare Island from SLR will largely depend on:

1. Balancing the benefits of land aggregation with the requirements for long term pumping, treating and discharge outfall systems and/or land fill.
2. The extent and criteria of the retrofit, which must be informed by the intended future use and purpose of the improved structure.
3. The condition of the existing structure.

A condition assessment of the existing structures is recommended and is discussed in more detail in the next section.

Figure 6-5 presents a simplified sketch of what a retrofitted wharf at Mare Island might look like. The key added structure is in red and includes the following:

- New steel sheet pile wall in front of existing wall. New wall will need to be deeper than the existing wall in order to:
  - Cut off/mitigate seepage under the sheet pile wall to maintain low ground water on landside
  - Ensure wall and overall structural stability during seismic event and liquefaction condition
  - Allow required dredge depth for vessel access
- New sheet piles will have design life corrosion protection, likely passive cathodic protection
- New Stormwater and Seepage pumping, treatment and discharge outfall system

- New concrete capping structure:
  - Is higher than future storm surge water levels, including SLR
  - Ties into existing structure
  - May be required to clear existing utility corridor
- Height difference between existing wall and new wall is approximately up to 5.5 feet for SLR 2070 and up to 9 feet for SLR 2100 protection. This could restrict access to the top of the wall which may not be practicable for landside operations.

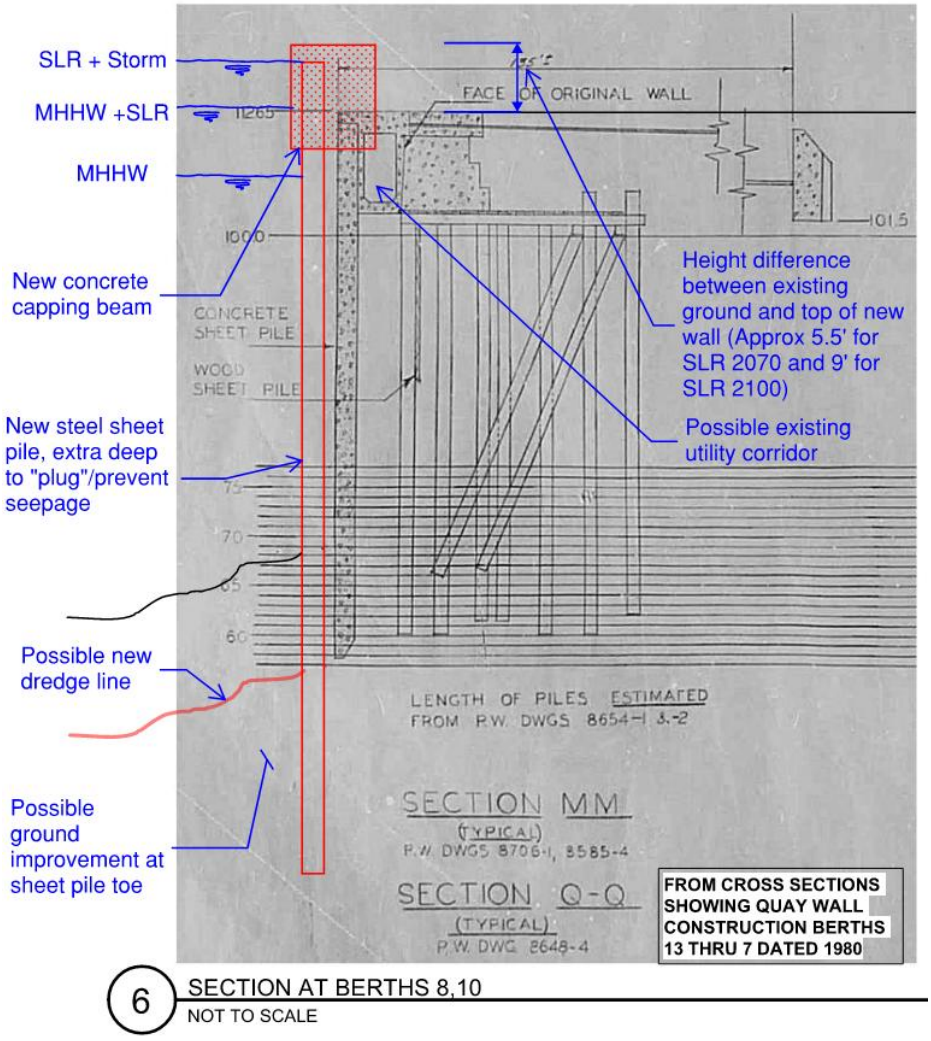

Figure 6-5. Example of Improvement to Existing Seawall

Figure 6-6 shows an alternative retrofit option that does not restrict landside access, but requires landside fill and a more extensive structural improvements. In addition to those shown in Figure 6-6 the following are added:

- Fill to top of wall level
- New deadman and anchor system

- May need existing reclaimed ground improvement (need consolidation and settlement assessment and mitigation for additional fill)
- New concrete platform/surface infrastructure
- New additional bearing piles if required (not shown)
- No height difference between top of wall and landside ground/infrastructure

This option provides a top of wall at the same level as the landside infrasture, but requires fill and rebuilding or repurposing the infrastructure on the landside.

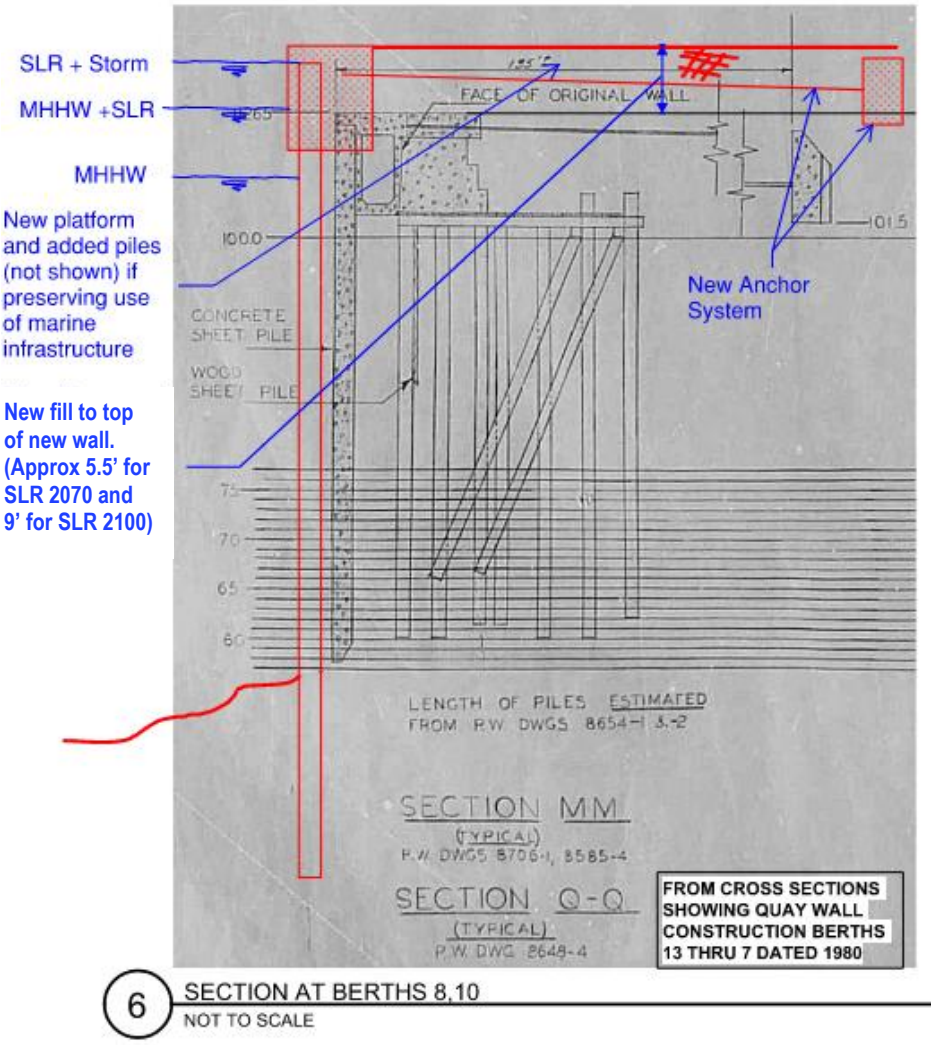

Figure 6-6. Example of Improvement to Existing Seawall – With Fill

6.7.9 Reconnaissance-Level Seawall Condition Assessment (2025)

To support long-term SLR planning and infrastructure improvements, an above-water, reconnaissance-level condition assessment of the Mare Island bulkhead wall and associated waterfront structures was conducted in April 2025 by Moffatt & Nichol (M&N). The objective of the assessment was to document visible signs of deterioration, identify areas requiring further investigation, and inform future structural

evaluation and retrofit planning. The full assessment report is provided in Appendix 6B – Mare Island Bulkhead Wall – Reconnaissance Level Condition Assessment (M&N, October 2025).

6.7.9.1 Scope and Methodology

The assessment was conducted to evaluate the above-water condition of Mare Island's waterfront structures, including bulkhead walls, timber and concrete wharves, and associated elements along the eastern shoreline. The assessment included visual, non-destructive inspections of all accessible structural components. Observations were made from both the landward and bayward sides, with bayward inspections conducted via small skiff under varying tidal conditions.

The assessment area was divided into 11 structural zones (Zones A–K), encompassing Berths 1 through 24, with each zone representing a distinct structural type or construction era.

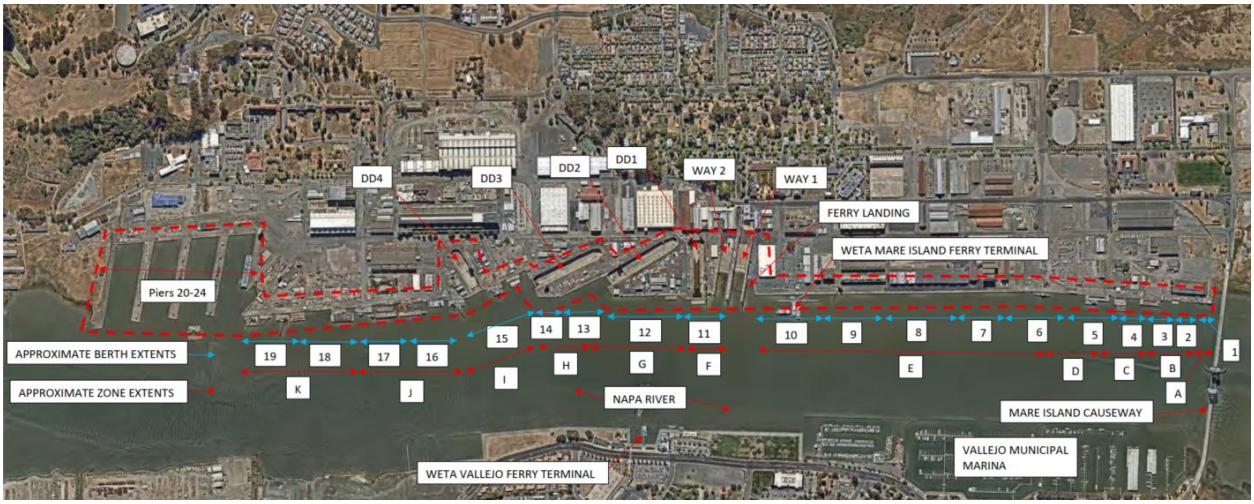

Figure 6-6. Extents of Inspection Scope (Figure 2, Appendix 6B)

Each structure was evaluated for signs of distress or deterioration, including cracking, spalling, section loss, exposed reinforcement, corrosion, joint separation, pile misalignment, marine borer damage, and failure of grout encasement at tiebacks or walers.

Findings were documented through field notes and representative photographs, and condition ratings were assigned in accordance with the ASCE Manual of Practice 130: Waterfront Facilities Inspection and Assessment. Ratings reflect the observed severity of defects in structural components but are considered preliminary due to the limited access to submerged and interior elements. Several areas, particularly beneath wharves and in basins between piers, could not be fully assessed due to siltation, vessel mooring, or operational restrictions.

This reconnaissance-level evaluation was not intended to serve as a basis for load rating or structural certification but rather to identify visibly deteriorated elements, prioritize zones for further study, and inform the planning of detailed structural inspections and repair strategies.

6.7.9.2 Summary of Findings

The condition of bulkhead walls and wharf structures across Mare Island varied significantly by location and structural type. The assessment identified a broad range of deterioration, from minor surface defects

to severe structural damage, including extensive cracking, spalling, corrosion, and section loss. Several zones exhibited signs of advanced degradation, while others appeared in fair to satisfactory condition. In some areas, access limitations prevented full evaluation, and further inspection is warranted. Table 6-4 presents a summary of findings by zone including condition rating.

Table 6-4. Bulkhead Assessment – Summary of Zone Conditions

Zone	Berths/structures included	Structural system	Zone Length (ft)	Relieving plateform	Wall tiebacks	Observed Conditions	Condition Assessment Rating of Structures within Zone (*)
A	Berth 1	Open Wharf. Timber piles, caps, stringers and deck	30	N	N	Severe damage to braces. Major to severe damage to piles.	Poor
B	Berths 2 & 3	Concrete Sheet Pile Bulkhead	610	Y	Y	Only Minor damage noted. Very little of the concrete sheets could be observed.	Satisfactory
C	Berths 4 & 5	Open Wharf. Concrete piles, caps and slab	600	Y	N	Moderate to Major cracks and spalls to beams and piles.	Fair
D	Berths 5 & 6	Bulkhead Wall. Wall material below water is unknown.	620	?	?	Sheet piles were not observed. Relevant drawings for this zone where not available. Bulkhead Structural system not known	Condition rating for overall structure could not be assessed (**). Assumed to be Poor, more inspection effort needed Satisfactory for cap
E	Berths 7, 8, 9 & 10	Concrete Sheet Pile Bulkhead	2300	Y	Y	Approximately 25% of concrete sheets had signs of severe damage.	Poor to Serious
F	Berth 11	Open Concrete Wharf with Concrete sheet Pile Bulkhead Beyond. Concrete piles, caps and slab	320	Y	?	Severe spalls in wharf soffit. Moderate cracks in wharf piles. No visible damage to bulkhead wall.	Poor
G	Berth 12 & 13	Concrete Sheet Pile Bulkhead	840	Y	Y	Failure of waler encasement. Only minor to moderate defects to sheets noted. Severe cracks noted in concrete cap Pesence of sinkhole behind bulkhead wall	Fair to Poor
H	Berth 13 & 14	Steel Sheet Pile Bulkhead	520	Y	?	Steel sheet piles not observed	Condition rating for overall structure could not be assessed (**). Assumed to be Poor, more inspection effort needed Satisfactory (for cap)
I	Berth 15	Steel Sheet Pile Bulkhead	570	Y	?	Moderate cracks in the bulkhead concrete cap. Steel sheet piles not observed	Condition rating for overall structure could not be assessed (**). Assumed to be Poor, more inspection effort needed Fair (for cap)
J	Berth 16 & 17	Open Wharf. Concrete piles, caps an slab. Concrete cutoff wall at the back of the open wharf.	1000	N	N	Minor to Moderate cracks in piles. Moderate to svere spalls in beam and slab soffits	Satisfactory to Fair
K	Berth 18 & 19	Concrete sheet pile wall	1250	Y	?	Concrete sheet piles could not be observed. Severe cracks present in concrete cap	Satisfactory to Fair

* Condition Assessment Rating is based Solely on structural components observed. Conditions could be worse.
** Large portions of this structure are unknown

Source: Moffatt & Nichol, April 2025. Mare Island Bulkhead Wall–Reconnaissance Level Condition Assessment.

6.8 RECOMMENDATIONS

This section presents geotechnical and structural recommendations to support future development on Mare Island, with a focus on the seawall.

6.8.1 Recommendations for Geotechnical Investigation

As presented in Plate 2, many of the future development areas do not have site-specific investigation information. Information from published maps can be used for conceptual design purposes, however, site-specific subsurface investigations will need to be performed for final designs. The type of subsurface investigations that can be performed for the design and construction of future improvements include:

- Borings: to characterize ground and groundwater conditions; and to collect soil and rock samples for laboratory testing.
- Laboratory Testing: to obtain design parameters for soil and rock underlying the project areas.
- CPT: to characterize ground conditions (e.g., depth to groundwater, liquefiable soil, lateral spreading potential, average shear wave velocity).
- Seismic Refraction Survey: to evaluate soil/rock interface profile on the southern side of the island for proposed improvements and new seawalls.
- In-situ permeability testing to assess implications of water seepage through/under the seawall resulting from an imbalance of water pressures between the seawalls and landside and to inform designs to intercept and convey seepage water that could otherwise cause elevated groundwater levels within the interior of the island.

The type and quantity of investigation will depend on the design requirements. For example, for areas where raising by placing fill is planned, both borings and CPTs can be performed to determine the bottom of YBM and YBM samples should be collected to determine consolidation properties of the YBM. All infrastructure improvements should consider site-specific geotechnical data and the applicable building codes in effect at the time of design.

6.8.2 Recommendations for Future Assessments for Seawalls

Further investigation is required to advance the seawall component of Mare Island's redevelopment. These efforts fall into two categories: (1) evaluations focused on the physical condition and structural capacity of the existing wall, and (2) assessments to support long-term SLR adaptation planning.

6.8.2.1 Condition and Capacity of Existing Seawall Structure

A structural study and detailed inspection of the key structural elements comprising the seawalls is needed to confirm consistency with original design drawings and to inform future retrofit designs. As an initial step, an above-water, reconnaissance-level condition assessment of the Mare Island bulkhead walls and waterfront structures was completed in April 2025 (see Section 6.7.9 and Appendix 6B).

Building on the 2025 assessment, the following additional studies, follow-up investigations, and remediations are recommended which will support evaluation of retrofit feasibility and future design:

- In the short term (0 to 2 years), perform the following inspections in order of priority:
  - Inspection to ascertain the root cause of the sinkholes near DD2 within Zone G. Repair sinkholes as appropriate (repair approaches provided in assessment report).
  - Level I above and below-water inspection of the steel sheet piles within Zones D, H & I. The intent is to first assess the overall condition of the steel sheets that could not be observed. Given the age of these structures, it is anticipated that the steel sheets will be in poor condition within the intertidal zone. This Level I assessment may be followed by a more detailed assessment if required.
  - Detailed inspection of the open timber wharf within Zone A (Berth 1).
  - Detailed inspection of the concrete sheet pile bulkhead within Zone F.
- In the medium to longer term, the scope of future structural assessments should first be based on the planned development and land use. Development plans will inform on reaches (zones) of the waterfront that should be assessed and for which a load rating should be performed. Detail on other inspection methods is provided in the assessment report.

Because many of the recommended assessments and inspections are extensive and time-consuming, they should be incorporated into a comprehensive seawall management strategy. This strategy should manage cost, schedule, and resources, and be integrated with Mare Island's long-term shoreline planning and broader SLR adaptation efforts.

6.8.2.2 Sea Level Rise Adaptation Planning Recommendations

To support long-term shoreline protection, adapt to projected SLR, and evaluate retrofit feasibility, the following planning-focused efforts are recommended:

1. Conduct foundational site assessments to assess the geotechnical, hydrogeological, and physical data necessary to understand existing conditions:

- Conduct hydrological, geotechnical, and hydrogeological investigations to evaluate subsurface and hydrogeological conditions at seawall locations and to inform:
  - Liquefaction and seismic risks
  - Stability of existing and proposed sheet piles
  - Ground improvement options at seawall toes
  - Global slope stability of adjacent fill and shoreline areas
  - Consolidation and settlement impacts due to fill placement or reclaim operations
  - Stormwater management and pumping requirements including seepage conditions beneath seawalls
- Assess future ground water rise (GWR): Model and evaluate long-term GWR across Mare Island to SLR to understand interactions with shoreline structures and potential inland impacts such as elevated water tables and seepage-related instability.

2. Define a long-term shoreline strategy to guide future seawall design approaches:

- Develop a future facilities plan for wharfage: Define future vessel operations, berthage needs, and service requirements. This plan should consider access constraints, dredge

depth, berthing loads, ongoing capital dredging requirements, and long-term operational costs for wharf and shoreline facility maintenance.

- Coordinate future land use planning: Collaboration with the owners and operators of existing marine infrastructure will be essential to determine how best to integrate seawall redevelopment with future land use plans. Key considerations include the viability of maintaining existing wharf and dry dock operations if seawalls are raised, and whether adjacent landside areas would also need to be elevated to match. In areas where new walls are proposed to protect low-lying development, wall heights may exceed eye level, potentially affecting views, access, or compatibility with surrounding uses. Alternative land uses or repurposing of existing infrastructure should be evaluated as part of a comprehensive redevelopment strategy.

3. Incorporate Innovation and Expertise

- Consider state-of-the-art SLR adaptation approaches: SLR is a global problem. New and innovative solutions to prevent or mitigate inundation and rehabilitate, improve, and rebuild affected costal infrastructure are constantly being developed both locally and globally. The following plans have been recently developed locally in the San Francisco area and in Los Angeles:
  - The U.S. Army Corps of Engineers unveiled a plan in January 2024 to raise the Embarcadero by as much as 7 feet for rising sea levels. This is the order of magnitude that Mare Island seawalls will need to be raised for year 2100 SLR.
  - In January 2024 the Port of San Francisco recently released a Waterfront Resilience Program refers to adapting 7.5 miles of the San Francisco Waterfront to be resilient to earthquakes, coastal storms and SLR, which includes the preservation and fortification of the 100-year-old Embarcadero Seawall.
  - In 2018 the Port of Los Angeles published the “Port of Los Angeles Sea Level Rise Adaptation Study” which included an overview of vulnerabilities and a suite of adaptation strategies to both adapt over time and stay resilient to sea level rise.

These and other case studies provide insight into innovative engineering, policy frameworks, and collaborative strategies that may be applicable to Mare Island

- Engage specialist marine engineering expertise: Retain marine engineering consultants experienced in coastal resilience and waterfront rehabilitation. Their insight will be critical to developing feasible, cost-effective, and innovative retrofit strategies tailored to Mare Island’s unique geometry and historical infrastructure.

6.9 LIMITATIONS

This geotechnical and seawall chapter has been prepared for the exclusive use of West Yost, the City of Vallejo, and VFWD for the subject project as described herein. The scope of the study was limited to project details and data cited herein and is subject to revision when new pertinent details and data become available. A geotechnical field exploration program was not conducted for this study, and the findings summarized in this chapter were limited to the review of the reference materials cited herein. Delve and West Yost make no warranty as to the accuracy and completeness of the reference materials. A geotechnical investigation specifically scoped and completed for and along the selected project elements (i.e., fill areas, seawalls, and new road alignments) should be performed for project design.

Environmental-related (e.g., hazardous materials) and corrosion-related engineering are outside of Delve's expertise and are not part of their services. The limited environmental and corrosion-related data that is provided in this chapter was obtained incidentally by and during Delve's geotechnical desktop research and is provided solely as a value-added courtesy.

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CHAPTER 7
Roads System

7.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

Nichols Consulting Engineers (NCE) was subcontracted by West Yost for the Mare Island Infrastructure Assessment Project to perform an assessment of the existing road network and develop a pavement rehabilitation plan for the streets on Mare Island.

The tasks completed for the roads system evaluation included:

- Performing a review of the City's StreetSaver database and road centerline shapefiles.
- Performing a vehicle and walking pavement condition survey to verify the StreetSaver database pavement condition.
- Completing an automated survey vehicle capture of imagery of the entire street network.
- Performing pavement deflection and GPR testing on arterial and major collector streets to obtain information on pavement condition and layer characteristics.
- Performing dynamic cone penetrometer testing and pavement coring based on the deflection and GPR testing to support the pavement design process.
- Developing appropriate pavement preservation, rehabilitation, or reconstruction treatments for arterial and major collector streets based on field testing, analysis, and applicable standards.

A summary of the findings and conclusions for the roads system evaluations are listed below.

- There are 114 street sections totaling approximately 21.4 centerline miles of streets (or 42.2 lane-miles) on Mare Island in the City's current StreetSaver database. Most of the streets are AC pavement. On Mare Island, the City has rehabilitated 4.2 centerline miles of streets in 2012 and 2013, including surface reconstruction and wedge grinding and overlay, and 2.2 centerline miles of streets received slurry seal in 2017.
- According to the current City database, the average Pavement Condition Index (PCI) for Mare Island streets is 76. PCI is a numerical index between 0 and 100, which is used to indicate the general condition of a pavement section; 0 represents the worst possible condition and 100 represents the best possible condition. The Mare Island PCI of 76 does not include gravel roads and is an area-weighted calculation from StreetSaver based on condition surveys performed in 2021. For reference, the City conducted pavement condition updates on the entire City of Vallejo street network in 2022, and the overall City network PCI is 48.
- A field investigation was performed to identify Mare Island candidate streets to evaluate existing pavement conditions and develop a street rehabilitation plan. The field investigation included a pavement condition survey, falling weight deflectometer (FWD) testing, GPR testing, dynamic cone penetrometer (DCP) testing, coring/boring, and laboratory testing of collected subgrade soil samples. A total of 60 cores and three bores were sampled and 43 DCP tests were conducted to estimate the existing pavement structural layers' thicknesses and other properties.
  - FWD testing was performed on 25.3 lane-miles of streets. Their average FWD deflection was 16.0 mils (1 mil = 1/1000 of an inch), indicating medium structural capacity.

  - GPR testing was conducted on 40.0 lane-miles of streets. The streets average AC layer thickness was estimated to be 5.4 ± 2.5 inches.
  - Coring data showed that AC layer thicknesses ranged between approximately 1.3 and 16.1 inches, with an average thickness of 6.9 ± 3.2 inches. The AC layers were thicker on major streets (average 7.3 ± 3.0 inches) than on residential streets (average 5.7 ± 3.5 inches). At several locations PCC or large aggregate size macadam bituminous base layers were identified under the AC layer. The subgrade soil taken from the three bore locations had relatively high in-situ moisture content (>19 percent), and their stiffness ranged from low to high.

- The California Department of Transportation (Caltrans) Highway Design Manual (2022), the City roadway design standard, and engineering judgement was used to evaluate the pavement structure and identify appropriate pavement preservation, rehabilitation, or reconstruction treatments. Furthermore, images were captured over the Mare Island street network (sections with traffic access) using an automated survey vehicle. A design life of 2 years was assumed for sections with rehabilitation treatment. Traffic indices (TI) of 5 to 10 were used based on the section functional class identified in the City's StreetSaver database or traffic type/volume observed in the field. The following treatment types are proposed for the selected design sections based on existing pavement condition and estimated traffic volume; details are included in Section 7.5:
  - Do-Nothing (12 design sections)
  - Slurry Seal (17 design sections plus half of one design section)
  - Rubberized Cape Seal (7 design sections plus half of one design section)
  - Cold-in-Place Recycling (CIR) with hot mix asphalt (HMA) overlay (7 design sections)
  - Mill-and-Overlay HMA with base repairs (12 design sections plus half of one design section)
  - Remove existing asphalt concrete/aggregate base (AC/AB) and replace with HMA overlay (4 design sections plus half of one design section)
- The proposed treatments are based on the existing section conditions (late 2023). It is recommended the treatments are completed according to the following guidelines:
  - Complete the surface seal treatment repairs in the next 2 to 3 years.
  - Complete all other rehabilitation treatment repairs in the next 3 to 5 years.
- The recommended timing is to optimize treatment costs and slow the pavement deterioration to avoid further higher repairs costs. That said, a multi-year pavement treatment plan is always subject to change should it be related to planned utility work that needs to be completed ahead of paving, unexpected scheduling delays, funding shortfalls, and planned development/ construction. The final treatment and construction schedule needs to be re-evaluated and verified by the City based on existing pavement condition, construction budget, nearby development plan and anticipated uses.

7.2 METHODOLOGY

The City's StreetSaver database and the GIS shapefile received from the City (Figure 7-1) was used to identify candidate streets for pavement design and evaluation. Pavement testing or evaluation was only conducted on major streets or sections with high traffic volume.

The field investigation included a pavement condition survey, FWD testing, GPR testing, DCP testing, coring/boring, and laboratory testing of collected subgrade soil samples. The work performed and the design approach are discussed in detail in the following sections.

A walking pavement condition survey was also conducted on several sections located in the southern and southwest end of the Island (thick red lines on Figure 7-1) that were not in the existing StreetSaver database and are currently owned by the U.S. Navy. The inventory and condition survey data for these sections were added to the City's StreetSaver database, however, they were not included in the pavement design process since no other pavement testing was conducted in this area. Additionally, the entire Mare Island street network (sections with traffic access) was imaged using automated survey vehicles and recorded for pavement surface distresses. No condition data or PCI calculations were conducted as part of the automated survey in this project per the City's request (condition data was updated in 2021/2022 for the entire City). All collected images were submitted to the City as shapefile files.

The roadway analysis presented in this chapter focuses on pavement condition, structural assessment, and rehabilitation needs. Broader circulation planning elements are guided by the City of Vallejo's General Plan circulation element and will be evaluated at the project level.

7.3 EXISTING STREETSAVER DATABASE AND GIS SHAPEFILE

The City's StreetSaver database originally included 86 street sections totaling approximately 15.7 centerline miles of streets (or 31.4 lane-miles). All the streets are AC pavement. According to the current City database, the average PCI for Mare Island streets that are shown with thin green lines on Figure 7-1 is 76. PCI is a numerical index between 0 and 100, which is used to indicate the general condition of a pavement section; 0 represents the worst possible condition and 100 represents the best possible condition. The Mare Island PCI of 76 does not include gravel roads and is an area-weighted calculation from StreetSaver based on condition surveys performed in 2021. For reference, the City conducted pavement condition updates on entire City of Vallejo street network in 2022, and the overall City street network PCI is 48.

There were 28 street sections in the South Island added to the StreetSaver database, totaling approximately 5.4 centerline miles of streets (thick red lines on Figure 7-1). Of the 5.4 miles, 3.6 miles of streets were AC, and 1.4 miles of streets were gravel or unpaved. The average 2023 PCI of the added sections is 49 (does not include gravel roads). The newly added sections are not included in the analysis since the project pavement testing was completed before these sections were added to the database.

At the end of this project, there were a total of 114 street sections (21.1 centerline miles) of streets in the City's StreetSaver database, as summarized in Table 7-1.

Table 7-1. Mare Island StreetSaver Streets Inventory Summary (Including the Added Sections)				
Functional Class	Number of Sections	Centerline Miles	Lane-Miles	Network Area, %
Arterial	21	6.6	13.3	37.1
Collector	24	4.1	8.2	22.2
Residential	69	10.4	20.7	40.6
Total	**114**	**21.1**	**42.2**	**100**

The City's database shows that the City rehabilitated 4.2 centerline miles of streets on Mare Island in 2012 and 2013, including surface reconstruction and wedge grinding and overlay. The City also surface sealed 2.2 centerline miles of streets in 2017. Of the 6.3 centerline miles that were sealed or rehabilitated over the past few years, 4.9 centerline miles were arterial streets, and 1.4 centerline miles were collector or residential streets.

In addition to the StreetSaver database, the City provided a GIS shapefile. This shapefile included sections that were not accessible or in the current City's StreetSaver database, such as Azuar Drive (from Club Drive to south end), I Street, and North Pier Street. Some of the streets and sections were added to the final pavement analysis/design either by City request or due to their relatively high traffic volume. Table A-1 (Appendix 7A) shows the details of the City's Mare Island street database on Mare Island.

Table B-1 (Appendix 7B) shows the design sections that were selected from the City's database for pavement design and evaluation with pertinent street features and recommended designs.

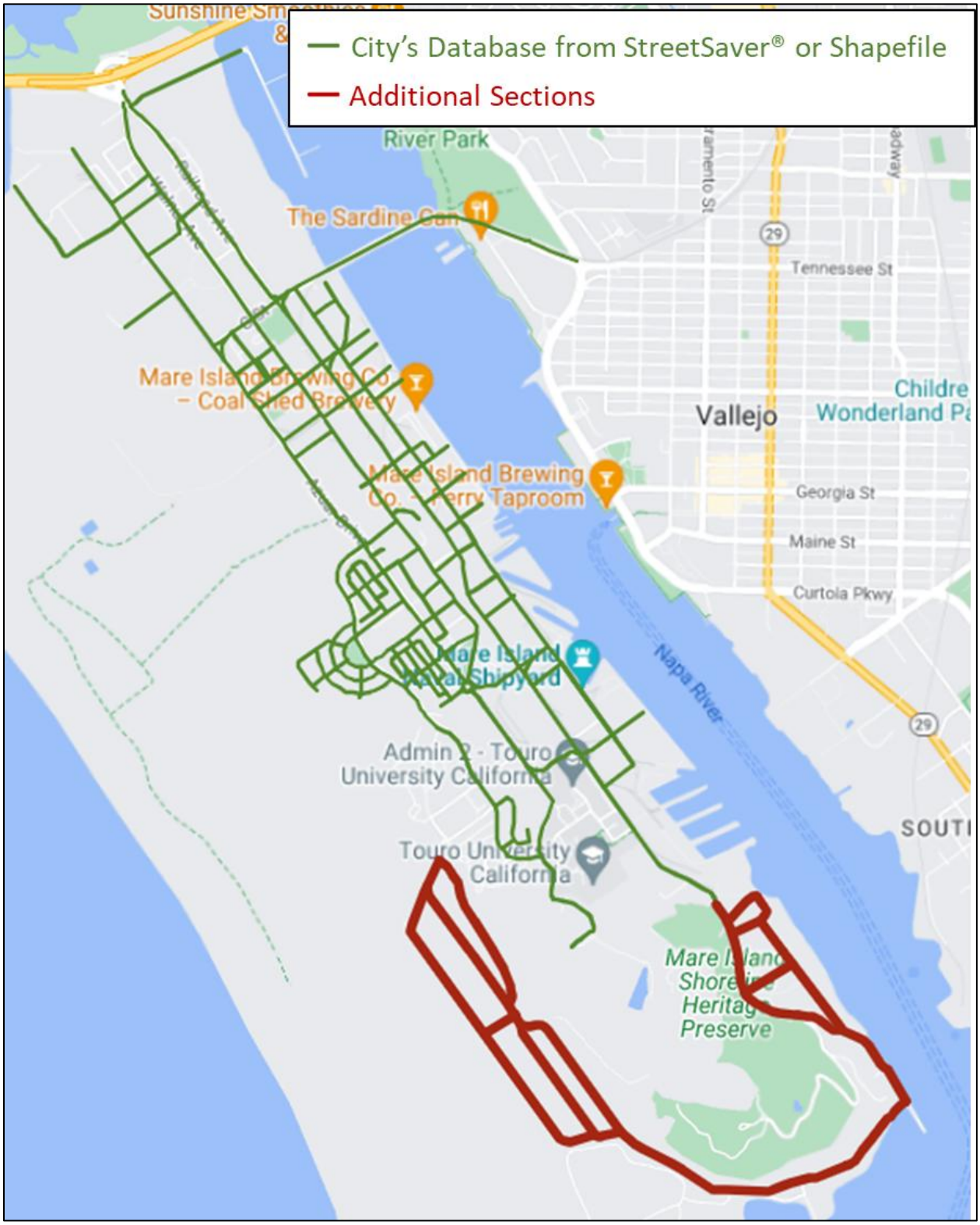

Figure 7-1. Map of Sections in City Database

7.4 FIELD DATA COLLECTION AND ANALYSIS

The field investigation included surveys of pavement condition/distresses, structural capacity, and layer thickness and subgrade properties. Data collection included automated pavement condition survey, FWD testing, GPR testing, and pavement coring. All data collected in the field were utilized for pavement analysis and design. It should be noted that the additional streets in the South Island (thick red lines on Figure 7-1) were not included in the evaluation or the design.

7.4.1 Automated Pavement Condition Survey

The entire Mare Island street network with traffic access was imaged using automated survey vehicles and collected pavement surface distresses at the same time. However, no PCI calculations were conducted in this project as part of the automated survey. The survey included the top-down pavement image collection on approximately 39 lane-miles of pavements to trace cracks, utility cuts, distortions, depressions, and other distresses on travel lanes only. Data were collected using the Metropolitan Transportation Commission's (MTC) modified[1] ASTM D6433[2] survey procedures. Figure 7-2 shows an example of the top-down photos that were used to identify and estimate the type, severity, and magnitude of the distresses. Environmental cracks, fatigue cracking, patches, and weathering are shown in this image. All images were converted to a shapefile and submitted to the City. Note that the images were only in sections in the City's original StreetSaver database with traffic access. Gated sections or newly added streets in the South Island were not included in the automated condition survey.

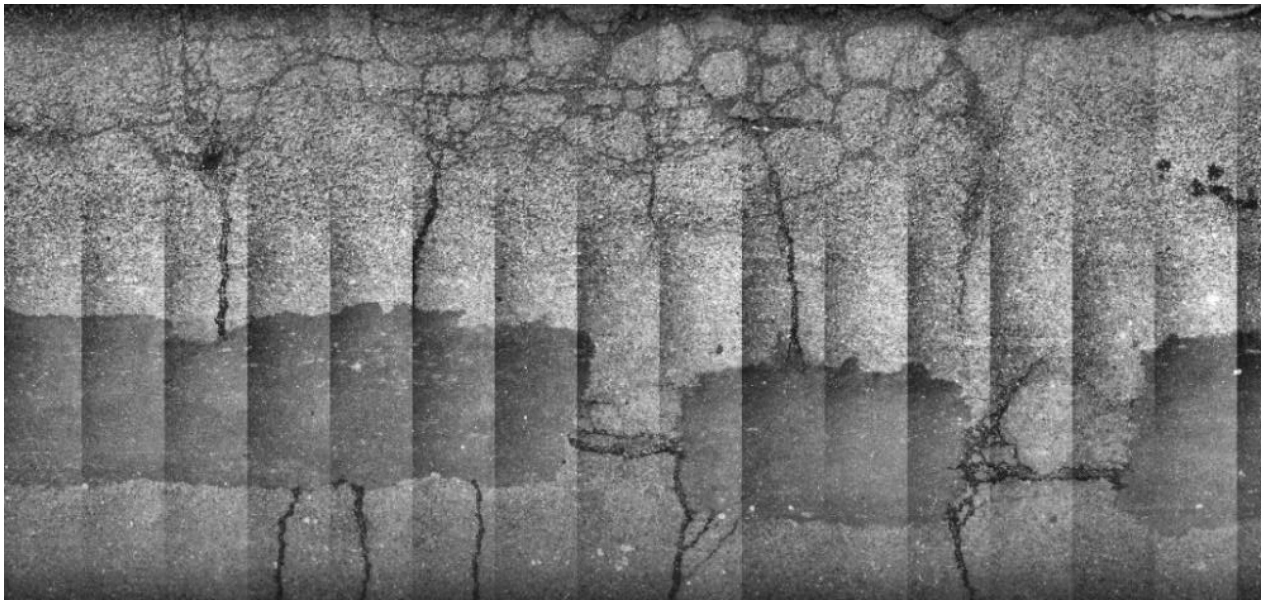

Figure 7-2. Example Automated Top-Down Pavement Image (G Street)

[1] PCI Distress Identification Manuals (AC 5th Edition, PCC 4th Edition), Metropolitan Transportation Commission, San Francisco, CA March 2022.

[2] ASTM D6433-18 Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys, ASTM International, West Conshohocken, PA, 2018, www.astm.org.

A walking condition survey was also performed on 5.4 centerline miles of streets in the South Island (thick red lines on Figure 7-1). The survey included recording the sections' lengths and widths, number of lanes, surface type, and condition using MTC-modified ASTM D6433 survey procedures. Appendix 7A, Table A-2 shows a summary of the data collected during the walking survey. The sections in the south of Mare Island had an average PCI of 49. Pavement inventory and distresses data for the additional sections were then entered into the City's StreetSaver database.

7.4.2 Pavement Deflection Testing

FWD testing was performed in accordance with California Test Method (CTM) 356[3]. As shown on Figure 7-3, a load pulse is produced by dropping a large weight (approximately 9,000 pounds) onto a "buffer", which shapes the pulse. The pulse is then transmitted to the pavement through a circular 12-inch loading plate. Data were collected from the sensor immediately beneath the buffer. Deflection testing was only conducted on arterial and collector streets with significant traffic volume, as identified either in the City's StreetSaver database or during field review. A total of 25.3 lane-miles of streets were tested and approximately 1,730 data points were collected. All travel lanes were tested at intervals of 100 feet or at shorter intervals to achieve a minimum of 21 deflection test points for each street based on the Caltrans design manual requirement. The intervals were staggered at approximately one-half the test interval length in adjacent lanes. Spacing was adjusted for both cost economy and for efficient coverage under varying pavement conditions and field constraints.

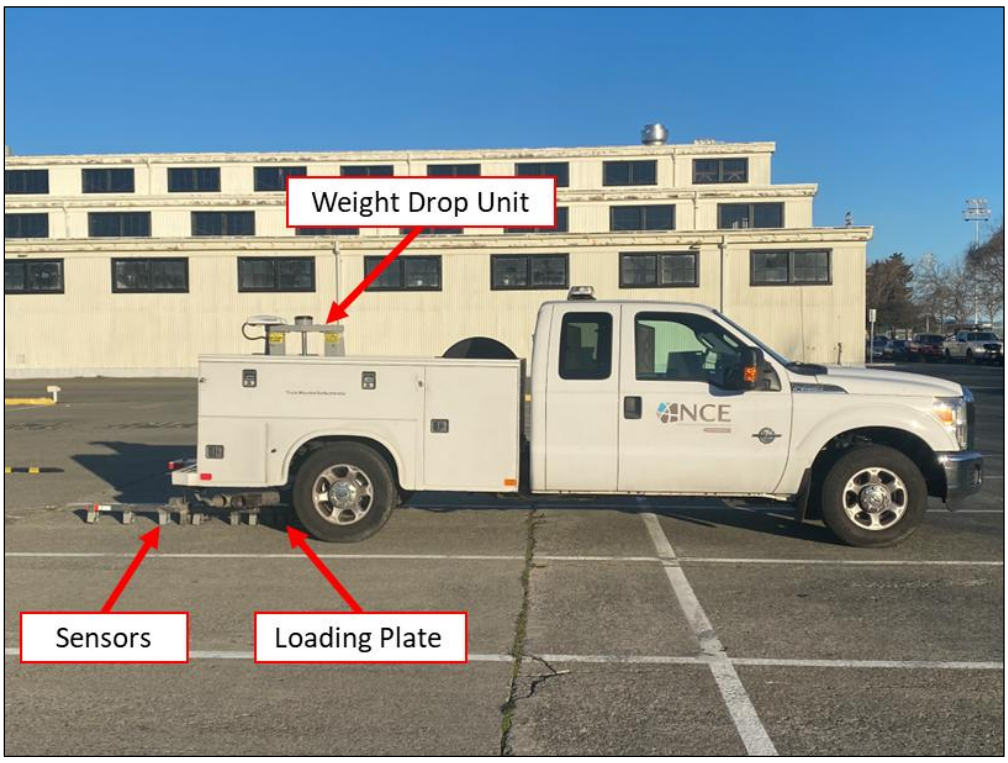

Figure 7-3. FWD Test Truck and Equipment

[3] Method of Test to Obtain Flexible Pavement Deflection Measurements for Determining Pavement Rehabilitation Requirements, California Department of Transportation, CA 2013, www.dot.ca.gov.

The average and standard deviation deflections for each design section are shown in Table B-1 (Appendix 7B). Figure 7-4 shows the FWD deflection ranges for all tested sections. Detailed FWD test results along each section for the sections with rehabilitation recommended treatment are shown in Appendix 7C.

Deflection values can reveal weak points and variability throughout pavement sections. The tested sections had an average deflection of 16.0 mils and standard deviation of 14.4 mils. Streets such as North Piers Street, G Street (south of Walnut Avenue), and Pintado Street (south of Walnut Avenue) had an average deflection of 30 mils, indicating low pavement stiffness or low structural capacity. Streets such as Flagship Drive and Rickover Street had much lower deflection (average 10 mils), indicating their high structural capacity.

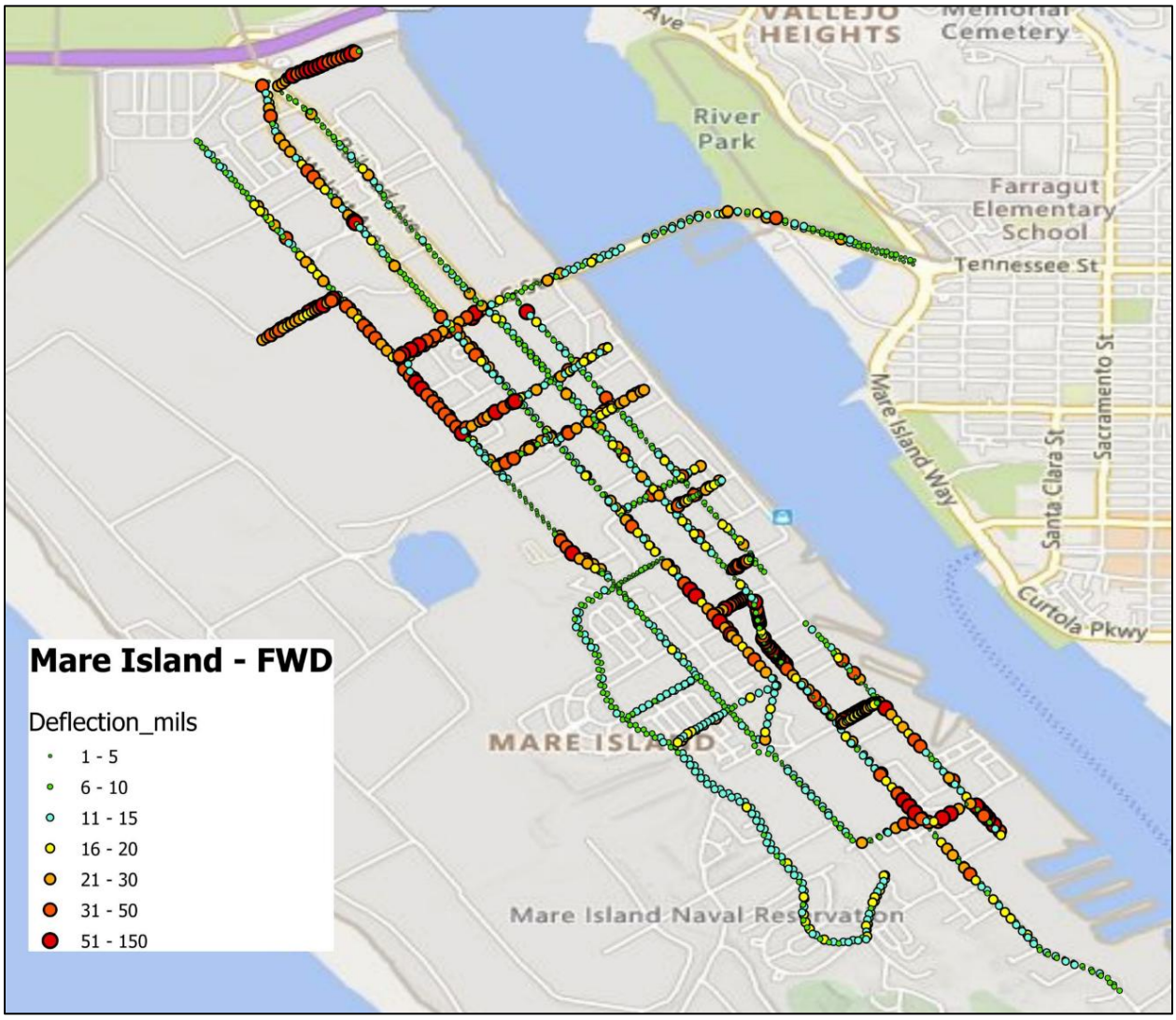

Figure 7-4. FWD Test Result Range for Tested Sections on Mare Island

7.4.3 Ground Penetrating Radar (GPR) Testing

GPR testing was performed on arterial, collector, and several residential streets with traffic access in accordance with ASTM D4748[4] to measure the thickness of the AC and underlying layers (when discernable). Data collection was carried out at normal driving speed using a 1-GHz horn antenna system manufactured by GSSI, Inc. of Nashua, NH (Figure 7-5). An electronic distance-measuring instrument (DMI) mounted to the rear wheel provided synchronous distance data. A Trimble GPS unit provided high resolution, differentially corrected geo-spatial information. Data collection and recording are controlled by an SIR-4000 GPR system operated from within the survey vehicle. The data were collected at a rate of 2 scans per foot of travel along the centerline of each prescribed lane.

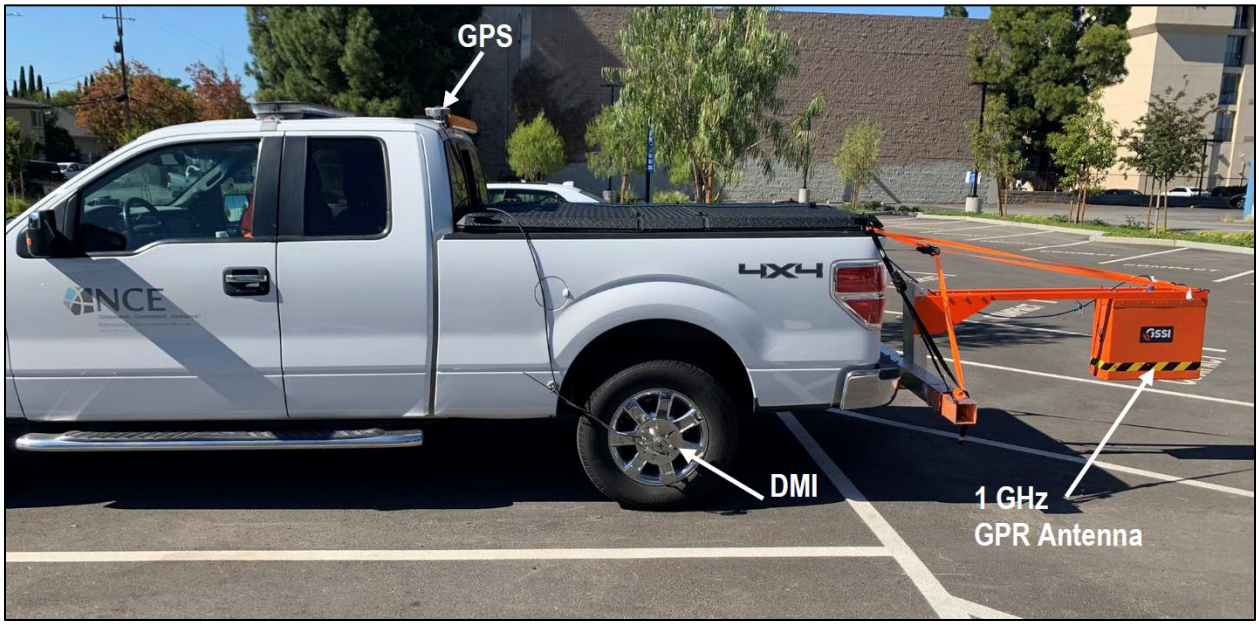

Figure 7-5. GPR Equipment Setup

Approximately 40 lane-miles of streets were tested (all travel lanes) and 7,063 data points were collected. GPR data was analyzed by Infrasense Inc., of Woburn, MA. The average and standard deviation deflection of each design section are shown in Table B-1. The details of the GPR test results are provided in Appendix 7C with the FWD test results. Overall, the tested sections had an average AC layer thickness of 5.4 ± 2.5 inches. Major streets such as Azuar Avenue and Walnut Avenue had thicker AC layers while residential streets such as Tisdale Avenue and Prospect Circle had thinner AC layers (Figure 7-6). For example, Walnut Avenue had an average AC layer thickness of 6.4 ± 3.0 inches and Tisdale Avenue had an average AC layer thickness of 3.3 ± 0.4 inches. The same pattern was observed in the underneath layers.

[4] Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar, ASTM International, West Conshohocken, PA, 2020, www.astm.org.

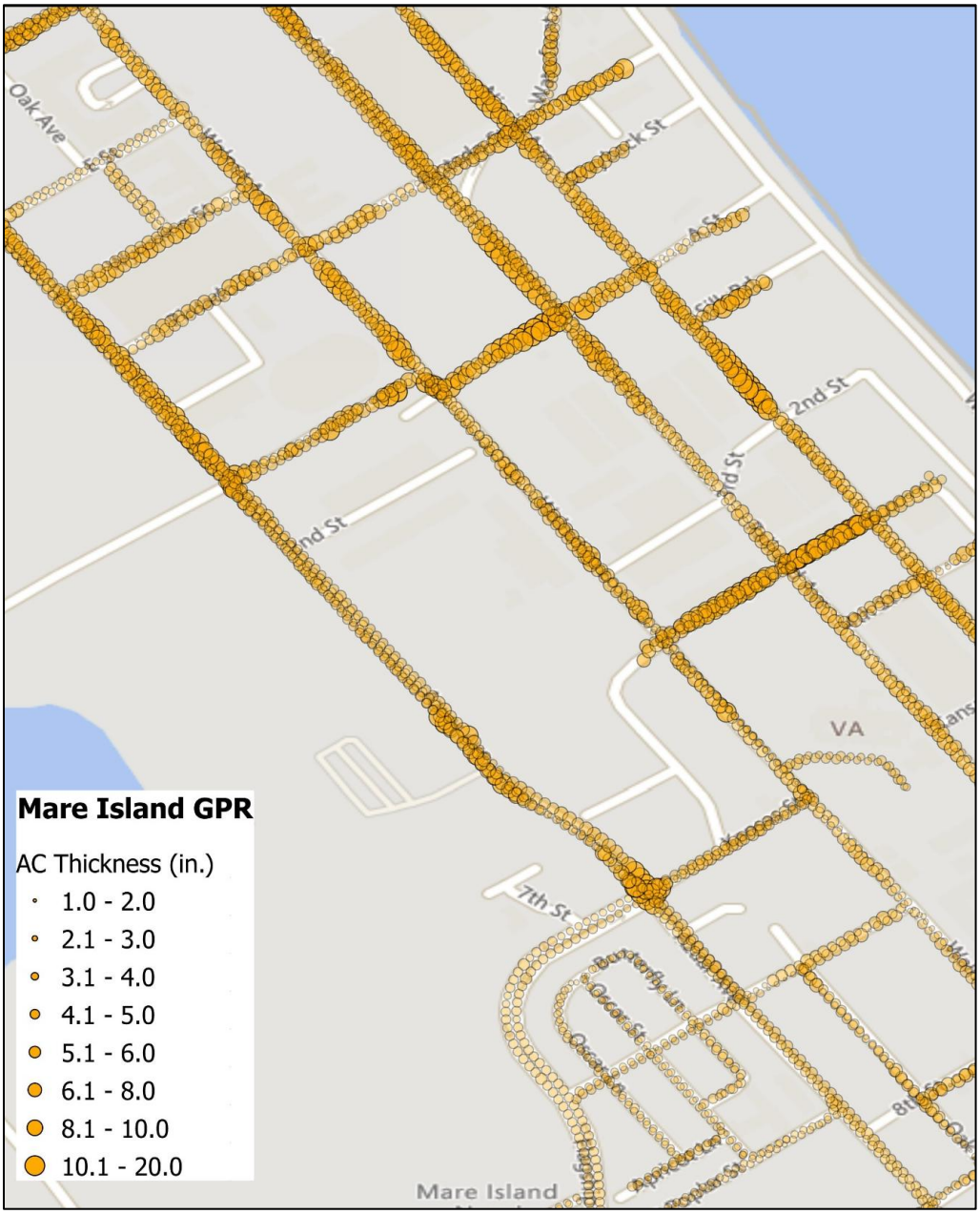

Figure 7-6. Examples of AC Thickness from GPR Test Results

7.4.4 Pavement Coring and Laboratory Testing

Sixty cores and three bores were sampled on selected streets and 43 DCP tests were conducted to estimate the existing pavement structural layer thicknesses and other properties. Figure 7-7 shows the locations of the core and bore samples.

Core locations were identified mainly based on pavement condition of the street while other factors such as length of the section, traffic volume, etc. were considered. The pavement cores were collected primarily from sections with poor and failed conditions. A few core samples were also collected from sections in fair to good pavement condition to recognize the typical AC thickness of Mare Island streets

and identify the base repair thickness. The coring included the AC thickness measurement by coring a 4-inch diameter AC sample. For each core sample, material thickness was measured and recorded. The presence and location of pavement fabric were also noted.



Figure 7-7. Core and Bore Sample Locations

DCP testing was initially planned to be performed on all the core holes to estimate AB thickness and underlying layer strength. However, due to some restrictions such as shallow utilities and exposure to toxic soil, the DCP was performed through 43 core holes. The DCP is a simple device (Figure 7-8) often used for *in-situ* characterization of shallow soils, especially for pavement design purposes. The number of repeated blows from a slide hammer along the cone correlates to an R-value which is a measure the mechanical strength of the soil. The R-values estimated from DCP testing confirm the existence of a stiff layer beneath the AC layer, which is either engineered fill or aggregate base on the top of the subgrade. The following summarizes the results of the field coring and DCP testing activities:

- AC layer thickness and fabric presence:
  - The existing AC layer thicknesses ranged between approximately 1.3 inches and 16.1 inches, with an average thickness of 6.9 ± 3.2 inches.
  - Fabric was observed in 11 core locations at Azuar Avenue, G Street, Nimitz Avenue, Railroad Avenue, and A Street.
- AB layer thickness:
  - The presence and thickness of the AB layers were investigated at three boring locations and 43 DCP test locations (total of 46 locations). AB was observed at all testing locations. The DCP test results show the presence of high stiffness layers under the AC layers for all tested sections.
  - There were several sections in which AC layers were built on top of PCC or large aggregate size macadam bituminous base. Figure 7-9 shows examples of PCC and macadam layers. PCC layers were found on Azuar Avenue, Railroad Avenue, Nimitz Avenue, Walnut Avenue, and A Street. Macadam layers were found on Connolly Street, Mare Island Causeway, Nimitz Avenue, Walnut Avenue, and A Street.
  - The AC layers were thicker on major streets (average 7.3 ± 3.0 inches) than on residential streets (average 5.7 ± 3.5 inches).

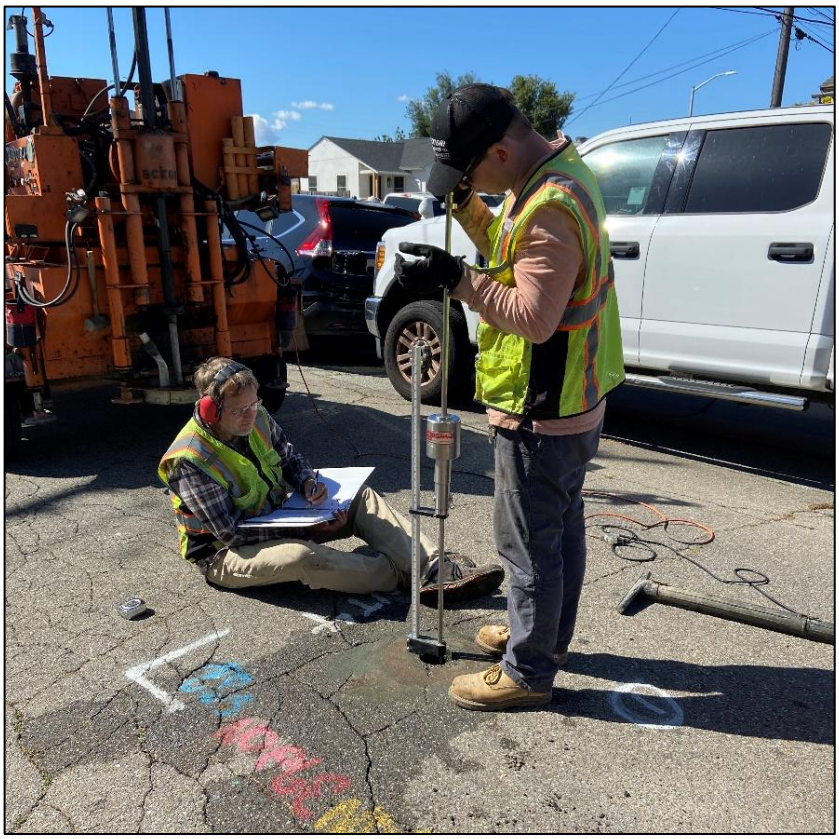

Figure 7-8. DCP Testing



Figure 7-9. Examples of Macadam (Left) and PCC (Right) Layers under AC Layers

Eight-inch diameter borings were collected at three sections with high traffic volume and poor pavement condition (Table 7-2). One boring each was collected on Mare Island Causeway, Nerus Street, and G Street to obtain bulk subgrade soil samples for laboratory testing and to measure AB thickness. BSK, Inc., in Livermore, California, performed laboratory testing on bulk samples to evaluate soil characteristics to support pavement design and construction preparation and compaction. The laboratory tests measured *in-situ* moisture content, Atterberg limits (liquid limit, plastic limit, and plasticity index (PI)[5]), and R-value[6]. These results provide robust indications of subgrade strength and drainage potential. A subgrade with high R-value and low moisture content is desired for pavement to provide more structural support. The PI gives a good indication of subgrade expansion/contraction potential. The greater the PI, the greater the soil expansion/contraction potential.

Table 7-2 shows the properties of the soils on each street. All subgrade soils were lean clay with relatively high moisture content, ranging from 19.3 to 29.3 percent. The PI was medium to high, varying from 17 to 25. Mare Island Causeway soil had an R-value of 20, indicating medium-to-high stiffness, while G Street and Nerus Street had low-to-medium stiffness soil. The laboratory test results are provided in Appendix 7D.

Table 7-2. Subgrade Soil Sample Test Results

Street Name	Functional Class	Subgrade Soil Type	Moisture Content, %	PI	R-Value
Mare Island Causeway	Arterial	Lean Clay (CL)	17.8	17	20
G Street	Collector	Lean Clay (CL)	19.3	25	15
Nerus Street	Collector	Lean Clay (CL)	29.3	23	12

The detailed core/bore summary is presented in Table B-2 (Appendix 7B). The results of the DCP test are presented in Appendix 7E. Core sample photos are presented in Appendix 7F.

7.5 PAVEMENT DESIGN AND RECOMMENDATIONS

Proposed treatments were developed using collected data and analysis, Caltrans Highway Design Manual (HDM) methods, the City roadway design standard, engineering judgement, and City input. Section 633.1 of the 2022 Caltrans *Highway Design* Manual details an empirical design method based on research and field experimentation performed by Caltrans. The method calculates the design thicknesses of the pavement structural layer using design life, traffic index, subgrade R-values, core thickness, and deflection data.

[5] Plasticity Index (PI): A measure of soil plasticity and volume-change potential of fine-grained soils. Typical PI values range from <10 (low plasticity, minimal volume change) to >20 (high plasticity, high shrink-swell potential), which can lead to pavement instability and cracking.

[6] R-Value: A measure of the resistive strength and stiffness of soil under load. Typical R-Values range from <10 (weak, compressible soils) to >50 (stiff, well-draining materials). Higher R-Values indicate greater structural support and improved pavement performance.

A design life of 20 years was assumed for sections with rehabilitation and shallow/surface reconstruction treatment. Twenty-year TI values of 5 to 10 were assumed depending on section functional class identified in the StreetSaver database and traffic type/volume observed in the field. The FWD deflection test data, laboratory test results, DCP test results, pavement core information, layer thicknesses from the GPR test results, and visual condition information was used to evaluate the pavement structure capacity and appropriate pavement preservation, rehabilitation, or reconstruction treatments.

As part of the design process, traditional engineering solutions were explored, such as mill-and-overlay and reconstruction. Value and sustainable engineering solutions were also explored, including rubberized cape seal, CIR, full-depth reclamation (FDR), and rubberized HMA and proposed those options as they were appropriate. In addition, the following were considered: budget constraints, constructability, existing conditions, land use (i.e., bikes, transit, residential, commercial, and industrial), grade constraints, pavement defects, AC layer lift thicknesses, and the City's construction requirements to produce site-appropriate and cost-effective recommendations. Some street sections were merged, or split based on pavement condition, structural layers, or other pavement properties to ensure consistent recommended treatments across each design section.

7.5.1 Proposed Treatments

Figure 7-10 shows a map of the proposed treatments for the 61 design sections. The following treatment types are proposed:

- Do-Nothing (12 design sections)
- Slurry Seal (17 design sections plus half of one design section)
- Rubberized Cape Seal (7 design sections plus half of one design section)
- CIR with HMA overlay (7 design sections)
- Mill-and-Overlay HMA with base repairs (12 design sections plus half of one design section)
- Remove existing asphalt concrete/aggregate base (AC/AB) and replace with HMA overlay (4 design sections plus half of one design section)

These treatments are summarized in the following sections and proposed treatment options for each section are listed in Table B-1 in Appendix 7B. Multiple treatment options with different expected pavement lives are presented for many sections. Base repairs evaluation was outside the scope of this project; however, depending on the condition of the pavement, necessity of the base repair was included in the recommendation.



Figure 7-10. Pavement Design Recommendation Map

7.5.2 Timing of Proposed Treatments and Other Considerations

The proposed treatments are based on the existing section conditions (late 2023). It is recommended the treatments be completed according to the following guidelines:

1. Complete the surface seal treatment repairs in the next 2 to 3 years.
2. Complete all other rehabilitation treatment repairs in the next 3 to 5 years.

Pavement sections with surface seal treatments typically begin in good to fair condition and therefore deteriorate more rapidly once distress initiates, particularly when the PCI ranges between 40 and 70. Pavements already in poor condition (PCI < 25) generally change little over a 3- to 5-year period. Preventive surface treatments are most effective when applied before pavements reach poor condition – delaying these treatments can allow deterioration to accelerate, resulting in more severe distresses that will require more extensive and costly rehabilitation (e.g., overlays or base repairs).

The timing of repairs should follow the guidelines above, or at the latest, coincide with construction in areas adjacent, or serving, areas that are redeveloped. For example, all roadways serving the proposed Connolly Street Corridor development (from Connolly Street, and all roads – Walnut Avenue, Railroad Avenue, and Nimitz Avenue – to the north up to G Street/Mare Island Causeway) should be repaired as part of the redevelopment. Additionally, some areas of the island that require fill for future development will require roadways to be fully reconstructed after fill and consolidation is complete (refer to Chapter 4 – Storm Drainage and Flood Control of this report for details on proposed fill locations). Repairs should also be coordinated with underground utility repairs and replacements (see Chapters 3, 4, and 5 for details on underground utility recommendations; and Chapter 9 for a discussion on integrating recommendations).

The recommended timing is to optimize treatment costs and slow the pavement deterioration to avoid further higher repair costs. That said, a multi-year pavement treatment plan is always subject to change should it be related to planned utility work that needs to be completed ahead of paving, unexpected scheduling delays, funding shortfalls, and planned development/construction. The final treatment and construction schedule needs to be re-evaluated and verified by the City based on existing pavement condition, construction budget, nearby development plan and anticipated uses.

7.5.2.1 Stormwater Treatment Requirements

During design, the City will also need to review and consider the recent third re-issuance of the San Francisco Bay Municipal Regional Stormwater Permit (MRP 3.0) with additional requirements for stormwater treatment. When pavement reconstruction or street widening projects exceed the contiguous 1-acre threshold, it requires the construction of green stormwater infrastructure.

7.5.3 Treatment Summary Descriptions

The following sections describe each of the proposed treatment types.

7.5.3.1 Do-Nothing

During the field condition survey, it was observed that several sections such as Flagship Drive, Tisdale Avenue, Rickover Street, and G Street from Railroad Avenue to the Mare Island Causeway Bridge (Photo 7-1) were recently paved or are in very good condition with low severity weathering. These streets

do not need any treatment in the next 2 to 3 years. Surface seal preservation such as slurry seal may be recommended over the next 5 years.



Photo 7-1
Pavement Condition of Do-Nothing Treatment (G Street from Railroad Avenue to the Mare Island Causeway Bridge)

7.5.3.2 Slurry Seal

Slurry seal is a homogenous mixture of emulsified asphalt, water, well-graded fine aggregate, and mineral filler that has a creamy appearance when applied. After applying the slurry to the pavement, the asphalt emulsion breaks onto the pavement surface. Then, when the mixture cures by loss of water, it will be bonded to the existing pavement to create a long-lasting wearing surface. Slurry Seal is used to fill existing pavement surface defects as either a preparatory treatment for other maintenance treatments or as a wearing course. It is used to correct minor defects in pavement such as weathering, raveling, tight cracks, or loss of matrix or skid resistance, and extend the life of the pavement. It provides skid resistance, restricts moisture intrusion, protects the structure from further oxidation and raveling, and restores a uniform black appearance.

Type II Slurry Seal is recommended as it should provide adequate seal coverage on surfaces in relatively good condition. Crack sealing is highly recommended prior to application of slurry seal surface seals to provide further protection against infiltration of water into the pavement section. In NCE's experience, crack sealing enhances the performance of surface seals. For the sections where surface seal treatment is recommended, it is advised to fill cracks wider than 2.0 inches with asphalt mastic or patch asphalt.

Photo 7-2 shows the A Street section from Railroad Avenue to Nimitz Avenue that was selected for slurry seal treatment. This section had an old slurry seal that was worn off, and some reflective cracks observed on the surface. The existing pavement has 5.5 inches of AC over a macadam layer.



Photo 7-2
Description Pavement Condition of Slurry Seal Treatment (A Street from Railroad Avenue to Nimitz Avenue)

7.5.3.3 Rubberized Cape Seal

Rubberized cape seal is a preventive maintenance treatment that combines the benefits of a chip seal with a slurry seal. Rubberized cape sealing is a 3-part process that involves the application of a rubberized asphalt emulsion, a chip seal aggregate (typically ⅜-inch aggregate size), and a slurry seal. Rubberized cape seal provides a durable, skid-resistant surface that seals out moisture and protects the underlying pavement from further damage.

Rubberized cape seals are typically proposed for streets with more prevalent longitudinal and transverse cracks and scattered low-to-medium load-related distresses throughout. Crack sealing is highly recommended before application of a chip seal layer to improve the performance of the cape seal. For the sections for which surface seal treatment is recommended, it is advised to fill cracks wider than 2.0 inches with asphalt mastic or asphalt patch.

Photo 7-3 shows a section of Connolly Street from Walnut Avenue to its east end that was selected for rubberized cape seal treatment. This section has a rough surface with severe weathering, moderate stripping, and raveling. Some reflective cracks and a few localized fatigue cracks were observed near the patch area. Rubber cape seal will restore surface smoothness and improve ride quality.



Photo 7-3
Pavement Condition of Rubberized Cape Seal Treatment (Connolly Street from Walnut Avenue to East End)

7.5.3.4 Cold-In-Place Recycling Asphalt Concrete with HMA Overlay

CIR is a process in which a portion of the asphalt pavement layers is pulverized, mixed with a recycling agent, repaved in place, and capped with a new HMA layer. CIR occurs within the roadway and uses 100 percent of the reclaimed asphalt pavement generated during the process. Typical treatment depths are 3 to 4 inches for emulsion methods and up to 5 inches for expanded foam methods. The CIR process commonly uses asphalt emulsion or hot asphalt cement, water, and a mixing unit, either a single vehicle as in foam methods or both milling and mixing vehicles for emulsion methods. Pavers and rollers are used to spread and compact the recycled material. The benefits of CIR include the following:

- Reduces public inconvenience and project duration by up to 30 percent.
- Utilizes the old surface to make a new interlayering layer.
- Removes existing crack patterns.
- Rejuvenates previous pavement.
- Preserves existing bridge clearances and curb heights.
- Minimizes excess/milled materials hauling.

CIR is generally used with high frequency and medium-to-high severity, non-load associated distresses (thermal cracking, block cracking, raveling, etc.). Cracked pavements that are structurally sound and have well-drained bases are the best candidates. For CIR to be effective in mitigating cracking, as much of the existing asphalt pavement layer should be treated as possible. The greater the depth of the crack that is removed, the less impact the remaining crack will have on pavement performance. CIR can also be used to address load-related distress when used in conjunction with an asphalt overlay to increase the pavement's structural capacity.

CIR treatment is proposed for seven design sections in this project. The AC layer thickness for these sections is between 4.9 inches and 13.1 inches. The proposed treatments include 3.0 inches pre-mill, 3.0 to 4.0 inches CIR with 2.5 inches HMA overlay. Photo 7-4 shows a Mare Island Causeway section with AC layer thickness of approximately 5.0 to 7.0 inches over 6.0 to 7.0 inches of macadam. The structural capacity of this section is approximately 8 mils, which is appropriate for CIR. Because traffic is heavy in this section, and the pavement exhibited moderate severity thermal and block cracks and localized fatigue cracks, NCE recommends 3.0 inches pre-mill, 3.0 inches CIR with 2.5 inches HMA overlay.



Photo 7-4
Pavement Condition of CIR Treatment (Mare Island Causeway)

7.5.3.5 Mill-and-Overlay

In a mill-and-overlay treatment, a portion of the existing AC surface is removed, base repairs are performed at select locations, and a new AC overlay is placed. Since the existing pavement varies in thickness, AC layer lift thickness was recorded at all core locations to determine what mill depth will leave at least 1.5 inches of the existing surface course intact. This protects the remaining material during milling. The mill-and-overlay treatment is typically proposed for pavements with both non-load-related and load-related distresses and for which high-severity distressed areas with base failure do not exceed approximately 25 percent of the total section area. This treatment is also appropriate for pavements with a small amount of load-related-distresses where milling of a thin layer of asphalt section, followed by applying a thin AC overlay, will provide adequate structural capacity. Areas with high-severity load-related distress are usually marked for base repair.

This treatment is proposed for 12 street sections, as well as the southbound lane of 7th between Nimitz Avenue and Railroad Avenue. AC layer thickness for these sections is between 2.8 inches and 16.1 inches, and 1.5 to 4.0 inches of mill-and-overlay was proposed for different sections. Photo 7-5 shows the G Street section from Azuar Street to Walnut Boulevard that was selected for 3.0 inches of mill with 3.0 inches of HMA overlay. This section is highly aged and raveled and has localized medium- to high-fatigue cracks along the wheel path and several longitudinal and transverse cracks. This section has 5.4 to 6.4 inches of

AC constructed over 11.6 inches of AB layer. FWD deflection at this section is approximately 23 mils, showing medium to poor structural capacity. Since this is a major collector street, a thick AC overlay is needed to mitigate the current distresses and restore the pavement to good condition.



Photo 7-5
Pavement Condition of Mill-and-Overlay Treatment (G Street from Azuar Street to Walnut Boulevard)

7.5.3.6 Remove and Replace

This treatment option requires removal of the existing AC/AB surface along with a few inches of subgrade, followed by placement of a new, thick HMA overlay over the AB. This treatment would allow regrading of the surface as needed. Since thick removal of AC/AB may leave only native soil or subgrade as an underlying layer for new pavement, a thick lift of HMA is typically proposed to increase the structural capacity of the sections. Typically, this treatment is recommended for pavement sections showing full width medium- to high-severity fatigue cracking, or high-severity distressed areas that exceed approximately 20 percent of the total section area or have very thin AC pavement thicknesses. This treatment is not typically recommended for sections with an AC layer atop a subgrade with inferior properties (high plasticity and moisture content) because compacting the subgrade after removing the existing AC layer will be problematic.

Removal and replacement of 3.5 to 4.0 inches of AC/AB is proposed for four street sections and the northbound lane of 7th Street between Nimitz Avenue and Railroad Avenue because these sections have medium- to high-severity full width fatigue cracking with some areas of pumping and depression. Two sections at Azuar Avenue and Railroad Avenue have less than 2.0 inches of AC over existing AB. Consequently, removing the existing thin, distressed AC layer and replacing it with a thicker AC section will increase structural capacity and service life. Three design sections at 7th Street, I Street, and N. Pier Street have approximately 3.5 inches of HMA over existing AB. Removing the whole AC layer and replacing it with the new HMA will restore the sections to good condition.

Photo 7-6 shows Railroad Avenue from Nerus Street to 500 feet north of Nerus Street. This section has an average existing AC thickness of only 1.6 inches over the AB layer and showed medium- to high-severity fatigue cracks, pumping, and localized depression. A full HMA replacement will restore the pavement to very good condition. Since toxic soils were identified at this section, shallow (2.0 inches) AC remove and replace is recommended. Otherwise, a 4.0-inch AC/AB remove and replace with HMA will provide improved long-term pavement condition.



Photo 7-6
Pavement Condition of Remove and Replace Treatment (Railroad Avenue from Nerus Street to 500 feet North of Nerus Street)

CHAPTER 8
Dry Utility Systems

8.1 SUMMARY AND CONCLUSIONS

This Summary and Conclusions section brings forward the key points from the chapter.

The tasks completed for the dry utility systems evaluation included:

- Identifying and contacting the existing dry utility providers on Mare Island.
- Obtaining and compiling existing mapping, plans, schematics, and reports which detail the extents of the dry utility systems.
- Discussing with each of the dry utility providers their current coverage, planned expansions, and concerns for the build-out development scenario.

A summary of the findings and conclusions for the dry utility systems evaluations are listed below:

- Dry utility providers on Mare Island include Island Energy (electric power and natural gas), Comcast/Xfinity (cable communications), AT&T (telephone and cable communications), and VIPFiber (broadband internet).
- Available infrastructure mapping, specifications, and current contact information was obtained from each dry utility provider and is included as the Chapter 8 Appendices. No field verification of the information received was performed. Facility sizes and locations presented in this chapter and associated appendices should not be relied upon without verification.
- The island's dry utilities face similar issues to the wet utilities, but on a smaller scale. Many dry utilities have been installed or reconstructed after the Navy left, so the systems are generally newer than the water and sewer systems.
- Dry utility infrastructure south of G Street is both functional and easily expandable to meet the needs of new development.
- Dry utility systems on North Mare Island are either non-existent or rudimentary and will need to be completely designed and constructed to meet the utility demands on the north island.
- In general, the dry utilities will have to accommodate sea level rise, although several of the dry utilities are not adversely impacted by wet or submerged conditions. Additionally, since the dry utilities typically utilize smaller transmission systems, relocating these are less intrusive and disruptive than water or sewer upgrades or relocations.
- Similar to the wet utilities, the dry utility companies are aware that any subsurface work on the island must be prepared to deal with the unexpected such as abandoned-in-place buried infrastructure or contaminated soil layers encountered during excavation.

8.2 PURPOSE

This chapter describes the data collection that was performed of the Mare Island dry utility systems, including electric, natural gas, cable, telephone, and broadband utilities. This included several steps, including collection and review of existing data sources, discussions with the planning and engineering staff at the various utilities, and site visits.

The chapter presents the existing facilities of the various dry utilities which operate on Mare Island, utility concerns, and current and future utility plans for island "build-out." The data collection did not include any technical evaluations, mapping/field verification, or engineering of dry utility system capacity or infrastructure. The following dry utility companies have infrastructure on Mare Island and were included in the data collection:

- Island Energy (electric and natural gas)
- Comcast/Xfinity (cable communications)
- AT&T (telephone and cable communications)
- VIPFiber (City-owned broadband infrastructure)

The sections below list each of the dry utility companies and then detail the following information:

- Current Coverage
- Planned Expansion
- Concerns for Build-out

A log of information received during this investigation is provided in Appendix 8A through 8D, organized by utility company. No field verification of the information received was performed. Facility sizes and locations presented in this chapter and associated appendices should not be relied upon without verification.

8.3 ISLAND ENERGY

Pittsburg Power Company ("PPC") is a California joint powers agency of the City of Pittsburg, California, and provides retail utility electric and natural gas distribution services on Mare Island, Vallejo in the name of "Island Energy". Island Energy is the sole provider of electric power and natural gas services to the island. A PG&E 115kv transmission crosses the Napa River and terminates at Island Energy's (IE) Substation H on the southern portion of the island. There are no other PG&E facilities on the island.

8.3.1 Electric Power

IE has a current electrical capacity based on the existing primary transformers of 20 MVA (with a conservative rating of 18MWA) island-wide, with its largest user being Alco Iron & Metal Co. on North Mare Island which utilizes approximately 10 percent of the current total capacity.

8.3.1.1 Current Coverage

The existing electric system coverage is shown in Appendix 8A.1 with electrical duct banks shown as magenta lines. Turquoise lines indicate wires suspended on utility poles. The yellow lines are gas pipelines which are also shown in Appendix 8A.1.

8.3.1.2 Planned Expansion

IE is currently working with the Mare Island Company regarding the development of the Connolly Street Corridor. IE recognizes the need for additional capacity and currently has near-term plans to increase production to 40 MVA with an additional transformer and adding a new circuit on Azuar Drive to the Alco Iron & Metal Co. for a new metals shredder. IE has not considered major infrastructure upgrades because there has not been any material increase in demand.

8.3.1.3 Concerns for Build-Out

IE believes that demands up to and beyond 100 MW are realistic if high demand users (e.g., data centers) are part of the new customer base in the long term. Data centers have the potential to draw 40 MW each, which far exceeds current capacity.

A System Impact Study to determine assess current capacity and projected needs would be critical to determine if PG&E could realistically get this amount of power to the island. PG&E provides power to the island through their Ygnacio substation in Novato, as well as multiple facilities in American Canyon and Napa. New transmission lines would need to be installed as the original transmission lines were removed during the base closure.

North Mare Island systems will need to be completely rebuilt since there are currently few customers. New infrastructure would be designed and constructed to provide power to any new north island customers. The systems south of G Street would need to be assessed depending on the demand, as the current system may not be adequate based on the current uses.

Much of the south island, including main electrical feeds from Station H, are vintage lead cable and can be prone to failure. IE performing annual testing on the system to identify areas that could be compromised (fail).

Sea level rise is a concern to IE. Continued communication with the City regarding sea level rise projections will help IE in their long-term planning of new facilities and improvements.

8.3.2 Natural Gas

IE is the sole provider of natural gas to Mare Island. Gas service is provided to the island via the Mare Island Causeway. A 10-inch diameter gas transmission line crosses on the south side of the Causeway, then drops down under the Napa River at the lift span.

8.3.2.1 Current Coverage

The main transmission line for the south island is along Railroad Avenue. Touro University has a small segment of pipeline they own and operate on the campus. North Mare Island has very minimal services for natural gas. See Appendix 8A.2 for the Gas System schematics.

8.3.2.2 Planned Expansion

There is no planned expansion for the system currently. Additional lines and service connections can be made from the existing infrastructure on the south portion of the island as needed. Gas service to North Mare Island will need to be completely rebuilt to meet any future demands.

8.3.2.3 Concerns for Build-Out

Access to gas facilities with sea level rise is a concern, although newly designed and constructed infrastructure on North Mare Island may be better suited to accommodate future sea level rise than the existing infrastructure south of G Street.

8.4 COMCAST/XFINITY

Comcast/Xfinity provides cable communications to Mare Island, primarily from G Street south along Walnut Avenue and adjacent properties through self-owned and operated infrastructure.

8.4.1 Current Coverage

The existing system is shown in Appendix 8B. The fiber backbone system enters the island via the Mare Island Causeway (shown in teal on the diagrams in the Appendix), turning south on Walnut Avenue to the traffic circle and then west to Flagship Drive where the system serves the residential communities to the north to Farragut Village and south to Club Drive and Azuar Avenue to serve Touro University. The thin green lines on the diagrams in the Appendix indicate underground cable facilities.

8.4.2 Planned Expansion

There is no planned expansion currently, but the utility can be expanded to meet any new development on north or south Mare Island.

8.4.3 Concerns for Build-Out

There are no current facilities on North Mare Island. Comcast/Xfinity staff stated they had no concerns about their ability to meet build-out conditions.

8.5 AT&T

AT&T provides telephone and cable communications to Mare Island through self-owned and operated infrastructure. AT&T has feeds to Mare Island from the north end of the island at State Route 37 and via the Mare Island Causeway at G Street. Facilities feeding the island are buried, underground, and submarine in nature. AT&T facilities north of G Street are primarily underground with a fair number of conduits. There are currently no functioning aerial AT&T lines on North Mare Island.

South of G Street, AT&T has “building” buried and underground infrastructure, with a higher density of conduits in the residential and industrial areas. The lines that are labeled as “buried” are most likely those installed by others (such as the Navy) and AT&T has no current information on them. These systems are most likely to be abandoned in place, so while they are shown on the schematic, they may or may not be functioning.

Lines labeled as “underground” are AT&T lines and are likely protected by conduit. Frequently the “underground” lines are adjacent to the “conduit” lines on the schematic, indicating that AT&T is more confident that those underground wires are placed within a conduit.

Lines marked “building” are wires that were privately owned and operated (by the Navy, in this case.) Generally, lines marked “building” indicate that these are the lines on private property from the AT&T service point to the building. As typical for large parcels, the original service provider would provide one main feed to the property. All the lines that feed individual privately-owned buildings or facilities on the property would be shown as “building” on the AT&T maps.

Given the general uncertainty of the current disposition of much of the on-site infrastructure, AT&T highly recommends reaching out to their staff when work is needed in the area and they would be able to assist.

Appendix 8C includes an Index Map and detail sheets showing AT&T's infrastructure, AT&T's commercial specifications for infrastructure installation, and a list of current contacts and general information about securing new services from AT&T.

8.5.1 Current Coverage

AT&T's existing coverage is quite comprehensive with primarily undergrounded infrastructure conduits and main lines. There are limited aerial systems primarily near the old officers' quarters and areas with heavy terrain. More than 99 percent of the AT&T system is below grade.

8.5.2 Planned Expansion

AT&T has no expansion plans for Mare Island at this time.

8.5.3 Concerns for Build-Out

AT&T has no significant concerns for future expansion and build out on Mare Island. Future development is likely to remove the existing copper infrastructure and utilize fiberoptic cable, which is not impacted by sea level rise. Given the age of the original maps and certainty that not all systems are accurately labeled on the detail sheets, AT&T requests close coordination with their staff during any future development.

8.6 VIPFIBER

VIPFiber is the City-owned broadband internet system. Awarded a five-year contract in late 2022, SmartFiber is the City's partner which is managing the system. VIPFiber is actively expanding throughout Vallejo – including on Mare Island.

8.6.1 Current Coverage

On Mare Island, VIPFiber currently has limited infrastructure serving its customers on Nimitz Avenue. There is also a new joint trench recently installed on Azuar Drive from State Route 37 along the west side of the island and approximately ¼ mile of conduit on Walnut Avenue.

The City is currently negotiating an agreement with IE to utilize IE conduits and communication vaults, similar to an agreement IE had with an earlier broadband provider. The pathways from conduits to the buildings will likely need to be newly constructed. There are also a handful of IE power poles that SmartFiber hopes to be able to access if needed. The City is also considering laying dedicated conduit for future use any time there is open trench work anywhere within the City limits.

VIPFiber existing and planned fiber location maps are included in Appendix 8D.

8.6.2 Planned Expansion

VIPFiber is eager to install fiber to suit whatever demands new customers may have. Once conduit is placed it is not difficult to augment fiber and conduit supplies and virtually any demand could be met. VIPFiber is an available option for residences and businesses on the island.

8.6.3 Concerns for Build-Out

Mare Island's old infrastructure and connections were presented as concerns for build-out. VIPFiber also mentioned the old steam system installed by the Navy that is still prevalent, and the inaccuracies of the historic maps can be problematic during site work. Open excavations with the unmarked and unknown subgrade issues can lead to additional work throughout the island. VIPFiber stated that it is not overly concerned with sea level rise as fiber is not subject to damage from water. Submerged vaults would need to be water tightened to provide maintenance access, but the infrastructure itself would not be at risk.

CHAPTER 9
Integrated Recommendations

9.1 PURPOSE

This chapter presents both general considerations and an integrated set of recommendations to guide implementation of infrastructure improvements on Mare Island. The recommendations are drawn from evaluations in Chapters 3 through 8, which address the water, sanitary sewer, storm drainage and flood control, roadway, geotechnical, and dry utility systems.

While these general and system-specific recommendations form the foundation for future implementation, detailed project phasing, timing, and design-level coordination will be developed through the Mare Island Specific Plan (MISP) Infrastructure Plan.

The remaining sections of this chapter include:

- **General Considerations** – Key planning themes and recommendations that will shape implementation across all utility systems.
- **Integrated Recommendations** – A summary table and interactive map presenting system-specific improvements within the two land use scenarios, and key dependencies between utility systems.

9.2 GENERAL CONSIDERATIONS

Throughout the MIIA project, several key themes emerged that will be critical to the successful implementation of infrastructure improvements. These general considerations, including utility coordination, development flexibility, climate resilience, and stakeholder coordination should guide planning, design, and construction across all utility systems.

9.2.1 Corridor Coordination

Most infrastructure improvements will occur within shared roadway corridors where multiple utility systems are co-located. Although the condition and capacity needs of each utility system vary, coordinating the timing of improvements across systems is essential to minimize disruption, avoid repeated trenching, reduce costs, and maintain public confidence in the redevelopment process.

To support efficient implementation, the following coordination principles should be applied:

- Roadways should not be resurfaced until all underlying utilities have been evaluated and, if needed, rehabilitated or replaced.
- Trenching should follow a logical sequence, with sanitary sewer improvements (typically the deepest) completed first, followed by water and storm drainage, then dry utilities, and finally roadway reconstruction.
- Coordinate utility improvements to avoid conflicts with recently resurfaced roadways and comply with City pavement moratoriums.

9.2.2 Development Timing and Flexibility

While the recommendations in this report reflect specific land use projections and anticipated development, actual development will evolve based on permitting, financing, and market conditions.

The City, District, and development partners should maintain flexibility by regularly updating assessments, project priorities, and implementation strategies to reflect evolving conditions, new data, and stakeholder input.

9.2.3 Climate Resilience and Adaptive Management

Mare Island's location presents long-term risks related to SLR, seismic activity, and land settlement. These conditions affect all utility systems and must be proactively addressed as redevelopment progresses.

While this report recommends specific capital improvements based on current regulatory guidance and scientific understanding, design criteria related to SLR, precipitation, and seismic hazards will continue to evolve. For example, the State of California's *Sea Level Rise Guidance* and the *Solano County Water Agency Hydrology Manual* are regularly updated based on new data and climate projections.

To ensure infrastructure remains resileient under future conditions:

- Future infrastructure assessment, design, and construction should use the most current, best available science and regulatory guidance at the time of implementation.
- Site-specific vulnerability assessments should be incorporated into the design of each capital project. These assessments should consider proposed land use, local drainage conditions, shoreline proximity, and exposure to natural hazards to tailor adaptation strategies.
- Adaptive management strategies should be integrated into project planning. While this report emphasizes physical infrastructure solutions, it is recommended that these be paired with community-level and cross-jurisdictional adaptation planning that reflects the unique characteristics of Mare Island.
- Long-term monitoring of shoreline elevations, groundwater levels, and infrastructure performance should be established and maintained. Monitoring results can inform the timing and design of future SLR adaptation measures, including seawall improvements, pump station upgrades, and drainage improvements.

Resilience planning on Mare Island must remain flexible, science-based, and adaptive to address the uncertainty of future climate impacts.

9.2.4 Capacity and Regulatory Constraints

The DOM4 sewer lift station is currently limited to 3.9 mgd due to downstream constraints in the collection system and at the District's WWTP. Increasing DOM4 flow capacity would require significant capital investment to expand the WWTP's treatment processes. Any increase would also trigger a new or modified NPDES permit, involving a complex and time-intensive regulatory process, making expansion infeasible in the near term. For these reasons, near-term sewer capacity strategies should focus on reducing I&I through pipeline rehabilitation and cross-connection elimination.

9.2.5 Agency and Stakeholder Coordination

Coordinated planning among the City, District, developers, dry utility providers, and other stakeholders will be essential throughout redevelopment. Tools such as shared trenching plans, joint utility mapping, and regular interagency coordination meetings should be used to streamline project delivery, reduce conflicts, and maintain communication among all project partners.

9.2.6 Funding and Partnerships for Major Capital Improvements

Many of the recommended improvements – including seawall adaptation, large-scale pipeline replacement, and future WWTP upgrades – may be eligible for regional, state, or federal funding. Grants, low-interest loans, and public-private partnerships should be pursued to deliver these improvements. Project packaging (e.g., bundling storm and roadwork) can enhance funding competitiveness and improve overall implementation efficiency.

9.3 INTEGRATED RECOMMENDATIONS

The recommended infrastructure improvements for each utility system are presented in a web-based, interactive map (WebMap), available at: https://arcg.is/0jyaei3. This tool allows users to explore recommended improvements across Mare Island in a geospatial context and toggle between utility systems and scenarios to view recommendations at any location on the island.

Table 9-1 provides a consolidated summary of all recommended improvements by utility system, including recommendations that are not location-specific and therefore not included in the WebMap. For additional detail on specific improvements, refer to the corresponding chapter for each utility system. Improvement timing is organized around the two land use scenarios: Existing/Near-Term (through 2027) and Build-Out (through 2063).

The findings and recommendations in this report form the foundation for development of a coordinated, resilient, and cost-effective strategy to support the long-term redevelopment of Mare Island.

Table 9-1. Integrated Recommendations by Utility Summary

Utility/System	Near-Term (by 2027)	Build-Out (by 2063)	Comments and Key Dependencies
Land Use & Growth (Chapter 2)	• Focus on Connolly Street Corridor development. Coordinate infrastructure phasing with developers to avoid rework and support initial growth zones.	• Support full island development consistent with MISP and utility user growth projections (Touro, MIDD, Connolly build-out). • Adjust infrastructure phasing based on actual development.	• Development phasing must align with utility capacity and infrastructure improvements. Close coordination with developer timelines is required.
Water System (Chapter 3)	• Replace ~20,800 feet of pipeline including the 20-inch south transmission crossing to address age, condition, and capacity deficiencies. Improvements focus on serving Connolly Corridor. • Improve valve and hydrant spacing as development progresses and pipelines are replaced. • Conduct tank repairs (valves, coatings, fittings).	• Replace/install ~127,000 feet of pipeline. Prioritize looping, redundancy, and fire protection. • Schedule tank inspections every 10 years. • System capacity should be re-verified as development progresses. • Exact layout of piping and facilities (tank, pump stations) subject to development progress.	South transmission crossing should be replaced before relying on single crossing for fire flow. • System capacity must be re-verified as development progresses. • Coordinate valve/hydrant spacing, pressure zones, and tank rehab schedules with development and emergency service needs.
Storm Drainage & Flood Control System (Chapter 4)	• Perform inspection of all maintenance holes and inlets; complete repairs and remove non-standard materials (brick, corrugated metal pipe used for maintenance holes and pipelines) • Coordinate with Caltrans to construct their own lift station for Highway 37 so LS-15 can be demolished. • Complete recommended improvements in Improvement Areas 3, 4, 5 (Connolly Corridor), 7, 8, 9, 10, and 11 for areas below 12 feet with existing development. • Begin seawall adaptation planning (covered further in Chapter 6).	• Expand drainage improvements as development progresses; this will include improvements in all Improvement Areas 1 through 12. By 2063, all areas below 13.5 feet should be protected. • Employ an adaptive planning approach, consistent with California's best available science and state guidance, to accommodate uncertainty in sea level rise projections. This includes phased implementation, community-based engagement, and coordination with evolving land use and infrastructure needs. • Finalize seawall adaptation measures (e.g., elevation, hybrid barriers) for 2070 (13.5 feet) and 2100 (17.0 feet) SLR projections.	• Storm system upgrades must be phased with development and seawall adaptation. • Coordinate stormwater upgrades with seawall planning, future SLR projections, groundwater rise, and pump station design to ensure long-term functionality of system.
Sanitary Sewer System (Chapter 5)	• Eliminate severe inflow defects (14) and cross connections (4) to sanitary sewer identified in smoke testing. • Replace or rehabilitate pipelines that were identified to have Grade 4 or 5 structural defects (11 pipelines; 1,463 feet). Rehabilitate maintenance holes in-line or adjacent to pipelines that are replaced or rehabilitated. • Replace all brick pipelines (437 feet identified) and maintenance holes (18 identified). • Rehabilitate maintenance holes with broken, cracked, or defective components (27 identified). • Complete lift station Priority 1 improvement recommendations (DOM 2, 4, 6, 7, 8, L Street). • Complete condition assessment on all pipelines greater than 60 years old. Visually assess associated maintenance holes concurrently. Address severe defects as they are identified. • Re-evaluate system capacity once near-term rehabilitation and direct connections establish a meaningful post-rehabilitation baseline and inform whether further improvements are necessary.	• Rehabilitate ~3,100 feet of Grade 3 defect pipes by 2029. • Continue pipeline inspection cycle every 7 years. • Implement DOM 2, 3, 4, 6, 7, 8 Priority 2 upgrades. • Evaluate full replacement of DOMs by 2063. • Re-evaluate system capacity following completion of major rehabilitation efforts using updated flow monitoring and development-based flow projections. Re-evaluation will provide a critical baseline for determining whether additional capacity improvements – such as increasing the DOM4 flow limit – are warranted.	• I&I reduction through repair of inflow sources and severe defects is expected to measurably improve system capacity. • Post-rehabilitation flow monitoring is essential to validate improvements. • Any increase in DOM4 pump station capacity may trigger WWTP upgrades and require new permitting.
Geotechnical (Chapter 6)	• Conduct site-specific subsurface investigations for the design and construction of future improvements. This may include borings, laboratory testing, and other geotechnical evaluations, with the scope and extent determined by the specific design requirements. • Evaluate seismic hazards, including liquefaction and landslide potential, using site-specific geotechnical data and the applicable building codes in effect at the time of design.	• Implement a surcharge program to address consolidation settlement impacts on future improvements. Settlement may be mitigated using wick drains, engineered fill, or other ground improvement measures, as needed. • Continue to conduct investigations and assessments, as needed, to address evolving site conditions and support the planning and design of future improvements.	• Many of the future development areas do not have site-specific investigation information. Information from published maps can be used for conceptual design purposes, however, site-specific subsurface investigations will need to be performed for final designs. • Coordinate geotechnical design (e.g., settlement mitigation, seismic resilience) with other infrastructure improvements.
Seawall (Chapter 6)	• Develop a comprehensive seawall management strategy to guide the planning, phasing, and implementation of recommended assessments, inspections, and improvements. This strategy should manage cost, schedule, and resources, and be integrated with Mare Island's long-term shoreline planning and broader SLR adaptation efforts. • Utilize findings from the 2025 bulkhead condition assessment to prioritize structural assessments and evaluate the feasibility of retrofitting existing walls. Initial recommended efforts (within 2 years) include: Inspection to ascertain the root cause of the sinkholes near DD2 within Zone G and repair of sinkholes as appropriate; Level I above and below-water inspection of the steel sheet piles within Zones D, H & I; Detailed inspection of the open timber wharf within Zone A (Berth 1); and Detailed inspection of the concrete sheet pile bulkhead within Zone F. • Initiate development of a long-term shoreline strategy, including a future facilities plan for wharfage, integrated with coordinated land use planning. • Evaluate retrofit feasibility based on initial assessments, SLR adaptation planning, and long-term shoreline needs. Identify additional assessments required.	• Update the seawall management strategy based on development, SLR adaptation planning, and the long-term shoreline strategy/needs. • Conduct recommended inspections and investigations to support the design and feasibility of seawall retrofits or new construction. Efforts may include: (1) Level I–III inspections of timber, concrete, and steel structural elements in accordance with ASCE MOP 130, including underwater dive inspections and material sampling; (2) Hydrological, geotechnical and hydrogeological investigations to assess subsurface conditions, seismic performance, slope stability, settlement potential, and seepage. • Construct or retrofit seawalls to meet 2070 (13.5 ft) and 2100 (17.0 ft) sea level rise projections based on SLR adaptation planning approach.	• Structural exploration and surveys should be performed prior to any other detailed inspection effort to efficiently guide subsequent inspections and focus resources where most needed. • Future designs must address settlement, seepage, groundwater, and seismic vulnerabilities; seawall elevations and footprints must align with adjacent utility corridors, long-term land use plans. • Maintain ongoing collaboration with property owners, developers, and specialist marine engineers to support retrofit feasibility, phased implementation, and shoreline resilience.
Roads System (Chapter 7)	• Complete surface sealing treatments (Slurry seal - 17 design sections; Rubberized Cape Seal - 7 design sections). • Coordinate with development and utility upgrades.	• Complete remaining treatments by 2028 (Cold-in-Place Recycling with HMA Overlay - 7 design sections; Mill-and-Overlay HMA with base repairs - 12 design sections; Remove existing asphalt base and replace with HMA Overlay - 4 design sections) • Update road condition assessments post-2027, recommended at least every 5 years. • Continue to coordinate with development and utility upgrades. • Consider road elevation adjustments in SLR-affected areas. • Implement long-term pavement maintenance strategy.	• Road improvements should follow utility upgrades to avoid rework. • Prioritize improvements in segments where development timing is known and conflicts can be minimized. Avoid improvements where future development or trenching is planned.
Dry Utilities Systems (Chapter 8)	• Coordinate utility installation/trenching across disciplines to reduce conflicts and accommodate shared corridors. • Incorporate sea level rise protection in design, such as conduit elevation and watertight manhole access covers.	• Design and construct new dry utility corridors and service connections to support full island development. North Mare Island will need to be fully redesigned and constructed. • Continue to coordinate utility installation/trenching across disciplines to reduce conflicts and accommodate shared corridors. • Continue to incorporate sea level rise protection in design of systems.	-