Attachments

1. 2008 Storm Drain Master Plan
2. 2009 Addendum to the Storm Drain Master Plan
3. 2015 Memorandum
4. 2017 Memorandum
5. 2011 Storm Drain Pump Station Assessment
6. Wood Rodgers Technical Memorandum
7. 2013 Storm Drain Outfall Assessment
8. 2019 Climate Action and Resiliency Plan
9. Response of the Shallow Groundwater Layer and Contaminants to Sea Level Rise.

FINAL Report
Storm Drain Master Plan
Alameda, California

August, 2008

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

CITY OF ALAMEDA
STORM DRAIN MASTER PLAN

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CHAPTER 1
MASTER PLAN OVERVIEW

Master planning has been undertaken to help guide the City of Alameda (City) establish a prioritized capital improvement program to mitigate the impacts of stormwater runoff. There is no indication in the City's records that a comprehensive, City-wide storm drain master plan has been conducted previously.

STUDY OBJECTIVES

The basic objective of this master plan document is to provide an examination of local flood risks within Alameda, and list those recommended projects necessary to mitigate those risks to an appropriate level. Specifically, this study identifies capital improvements needed to provide a level of flood protection consistent with the policies established by the City through this master planning process. Several objectives have been accomplished:

1. A geographical information system (GIS) based storm drain system model for the entire City has been built; allowing City staff, other engineers, and developers to easily locate relevant data on a computer screen.
2. Storm drainage criteria for various system elements and storm events are presented. These criteria will govern future infrastructure design; and are used to evaluate the performance of existing facilities, and plan remedial improvements.
3. The ability of existing storm drain facilities throughout Alameda to meet these criteria has been evaluated. System deficiencies are categorized in terms of the risk to public safety.
4. Projects that will improve storm drain operations are identified.
5. A prioritized Capital Improvement Program (CIP) is outlined.
6. Projected capital improvement costs are summarized.

BACKGROUND

Detailed study background including hydrologic and environmental settings, flood protection facilities, historic flooding and regulatory floodplain mapping efforts within the City are described in Chapter 2 of this report. A brief synopsis of the history of flooding analysis conducted prior to this master plan is provided below.

Drainage Study for the City of Alameda, 1977

Conducted by the Alameda County Flood Control District (ACFCD), this Study included the North Side Storm Drain System, Webster Street lateral lines north of Atlantic Avenue, and Main Street runoff north of Atlantic Avenue. Various improvement alternatives were proposed, although none were constructed.

Drainage Study for Northside, Webster Street & Main Street Drainage Basins, 1983

This study formed the basis for the construction of the Northside Basin at Marina Village. The Northside Basin is a weir diversion structure near the intersection of Constitution Way and Atlantic Avenue that diverts surcharge from the Arbor Street pump station to the Marina Village pump station. The City has on file the construction plans for the storm drain lines and pump station, but not the backup calculations.

Hydrology Analysis Calculations for the Main Street Drainage Improvement Area, 1997

This analysis formed the basis for the construction of the Main Street Pump Station. The tributary area studied includes Main Street from north of Ralph Appezzato Memorial Parkway (formerly Atlantic Avenue) to the vicinity of the ferry terminal at the northern end of Main Street.

Storm Drainage Facilities Rehabilitation and Repair Report, 1998

Thirty-five sites throughout the City (mostly culverts) identified by the City's maintenance department as having deficiencies were evaluated and solutions recommended. None of these recommendations were constructed.

Alameda Point Preliminary Master Storm Drain Plan, 2003

This study proposes a future drainage master plan for Alameda Point.

Basis of Design for Bayport Stormwater Pump Station, 2004

This study forms the basis for the construction of the Bayport Pump Station. The tributary area studied is located north of Ralph Appezzato Memorial Parkway (RAMP), between Main Street and Fifth Street and Mariner Loop to the estuary.

Hydrology & Hydraulic Report for Tinker Avenue – Webster Street Improvements, 2007

This report includes hydrology calculations for storm drain improvements on Tinker Avenue east of Webster Street. Improvements are scheduled for construction in 2007-2008.

FEMA Flood Insurance Study

The Federal Emergency Management Agency (FEMA) prepared a Flood Insurance Study (FIS) for the City of Alameda in 1991 and for Alameda County in 2000. The FIS concentrated on 100-year flooding from rainfall runoff and from the shoreline of Alameda Island, including San Francisco Bay, Oakland Inner Harbor, and Alameda Harbor. The study identified the Webster Street and Bay Farm Island drainages as subject to flooding due to 100-year storm events, and Main Street near Oakland Inner Harbor as subject to flooding from 100-year tide events.

SOURCES OF FLOODING

Local runoff is the major source of flooding that Alameda faces, complicated by tidal influences at each outlet point. This master plan focuses on how that runoff is conveyed by major conveyance facilities. The City's Public Works Department desires to work closely with local land use planning agencies, regulating agencies and property owners to develop a regional system of major conveyance facilities which will contain storm flows to prevent damage to property and threats to public safety.

Local Drainage

Runoff generated within the City's boundary is conveyed through the City owned storm drain system that outfalls to San Francisco Bay and associated estuaries. Conveyance and capacity deficiencies within the City's storm drain system can contribute to flooding within the City. The primary objective of the Storm Drain Master Plan is to address this risk. Because the City of Alameda is located on an island setting, the capacity of these drainage systems is linked to the tides and influence of the surrounding waters.

WORK PRODUCTS

This master plan is intended to function at several levels. City planners and engineers responsible for capital improvements should find that this document contains sufficient background information and data to serve as a basis for CIP implementation and/or modification. For those City staff and other parties interested in a more in-depth examination of storm drain facilities within Alameda, the companion ARCMAP GIS-based MOUSE model is available. MOUSE is a program designed by the Danish Hydraulic Institute (DHI) to model hydrology, hydraulics, water quality and sediment transport in urban drainage and sewer systems. As discussed in supporting reports and documents, the following information is available via the GIS:

1. ***Inventory of Drainage Facilities.*** City-owned drainage pipes at least 12 inches in diameter in the study area have been input into the storm drain model. Information pertaining to each system component may be accessed graphically or through database spreadsheets which have been provided on CD.

2. ***Tributary Drainage Areas.*** Land areas used to generate local runoff are also available graphically in the storm drain model, which catalogs tributary area, factors related to land use and soil conditions and other basin morphology.

3. ***Storm Drain Capacities and Street Flow Evaluation.*** Storm drain capacities are documented in the model. For each drainage system component, peak discharge, full pipe capacity and discharge as a percentage of capacity, and maximum hydraulic grade line are

computed. Based on hydraulic grade calculations, the degree of surcharge and depth of water in the street are also determined. This determination is then used to assign priorities for system remediation.

4. ***Drainage System Profiles.*** The main purpose of a GIS system is to eliminate the need for large quantities of paper documents. Those interested in viewing drainage system profiles may do so graphically using software features specifically designed for this purpose. Real-time animations of water surface profiles and corresponding street flood depths for design storm events are also available.

STUDY FINDINGS

Several conclusions have been reached regarding Alameda's storm drainage systems. From these conclusions, improvements are suggested to improve the system's performance so as to reduce the risk of flooding. While there are many areas within the City of Alameda that provide adequate stormwater conveyance, there are also known areas within each subsection of the City where flooding occurs. Based on both SDMP modeling results and the *Storm Drain Facilities Rehabilitation and Repair Report* (Harris & Associates, 1997) areas on Alameda Island that have notable flooding risk or past occurrences include: the intersection of Page and Taylor Streets, along both Washington and Mound Streets in the vicinity of their intersection, along much of the northern half of High Street, and in the area bounded by Pacific Avenue, Main Street, Atlantic Avenue and Fifth Street. On Bay Farm Island, the only area of known flooding is Veterans Court near the Bay Farm Island Bridge; however this issue is due to seepage in the seawall during high tides and is not related to stormwater runoff. Improvement alternatives and construction documents were generated previous to this storm drain master plan as mentioned in the Background section of this report. The improvements in this Master Plan should be considered a comprehensive Capital Improvement Program within the study area, superceding those previous improvement alternatives.

MASTER PLAN COSTS AND BENEFITS

Capital projects are needed to provide the benefits of reduced flood risk and relief from economic impacts during heavy stormwater runoff events. Failure to provide capital improvements or maintain the storm drain systems could interrupt daily commerce throughout the City, so all residents receive a benefit from a functional storm drain system regardless of whether their property is directly affected by said improvements and maintenance.

Table 1-1 summarizes all of the recommended storm drain capital improvement cost programs for storm drains per City drainage basin subareas, including extending existing storm drain pipelines.

Please refer to Chapters 5 and 7 for figures detailing the storm drain deficiencies and recommended improvements.

Table 1-1: Summary of Master Plan Costs

Master Plan Improvements, Alameda Island	**Eastside**	**North Central**	**Northside**	**South**	**Total**
Projects to Meet 10-Year Standard	$8,470,000	$9,686,000	$24,761,000	$11,999,000	$54,416,000
Projects to Meet 25-Year Standard	$11,940,000	$10,796,000	$37,811,000	$13,149,000	$73,196,000
Master Plan Improvements, Bay Farm Island	**East**	**North**	**Central**	**South**	**Total**
Projects to Meet 10-Year Standard	$2,550,000	$2,600,000	$4,590,000	$1,960,000	$11,700,000
Projects to Meet 25-Year Standard	$2,700,000	$3,210,000	$6,340,000	$6,570,000	$18,820,000

RECOMMENDATIONS

Reducing local flood risks by improving the City's storm drainage systems is a worthy goal that justifies the costs of said improvements presented in this report. This Master Plan provides a tool for Alameda citizens and officials to use in their efforts to reduce the risk of serious local flood hazards — whether nuisance flooding or real hazards to property — by completing the identified capital improvement projects.

ACKNOWLEDGMENTS

Several individuals have provided invaluable assistance in the collection of data for and review of the master plan documents. In particular, the assistance of Ed Sommeraur, Max Arbios, and Greg Stoia was paramount to completing this study.

CHAPTER 2
BACKGROUND

This chapter provides a general background of flood management issues currently affecting the City of Alameda. Hydrologic and environmental settings are described, along with flood protection and storm drain facilities. Historic flooding, a summary timeline of regulatory floodplain mapping efforts within the City, and Master Plan objectives are discussed herein.

HYDROLOGIC AND ENVIRONMENTAL SETTINGS

The City of Alameda encompasses most of Alameda Island and Bay Farm Island, which is adjacent to the Oakland Airport. The City is located in western Alameda County directly east of San Francisco. It is bordered by the San Francisco Bay to the west, the Oakland/Alameda Estuary to the east, and the Oakland Airport to the south. Cities that surround Alameda include Oakland and San Leandro to the east. Figure 2-1 places Alameda in its regional context.

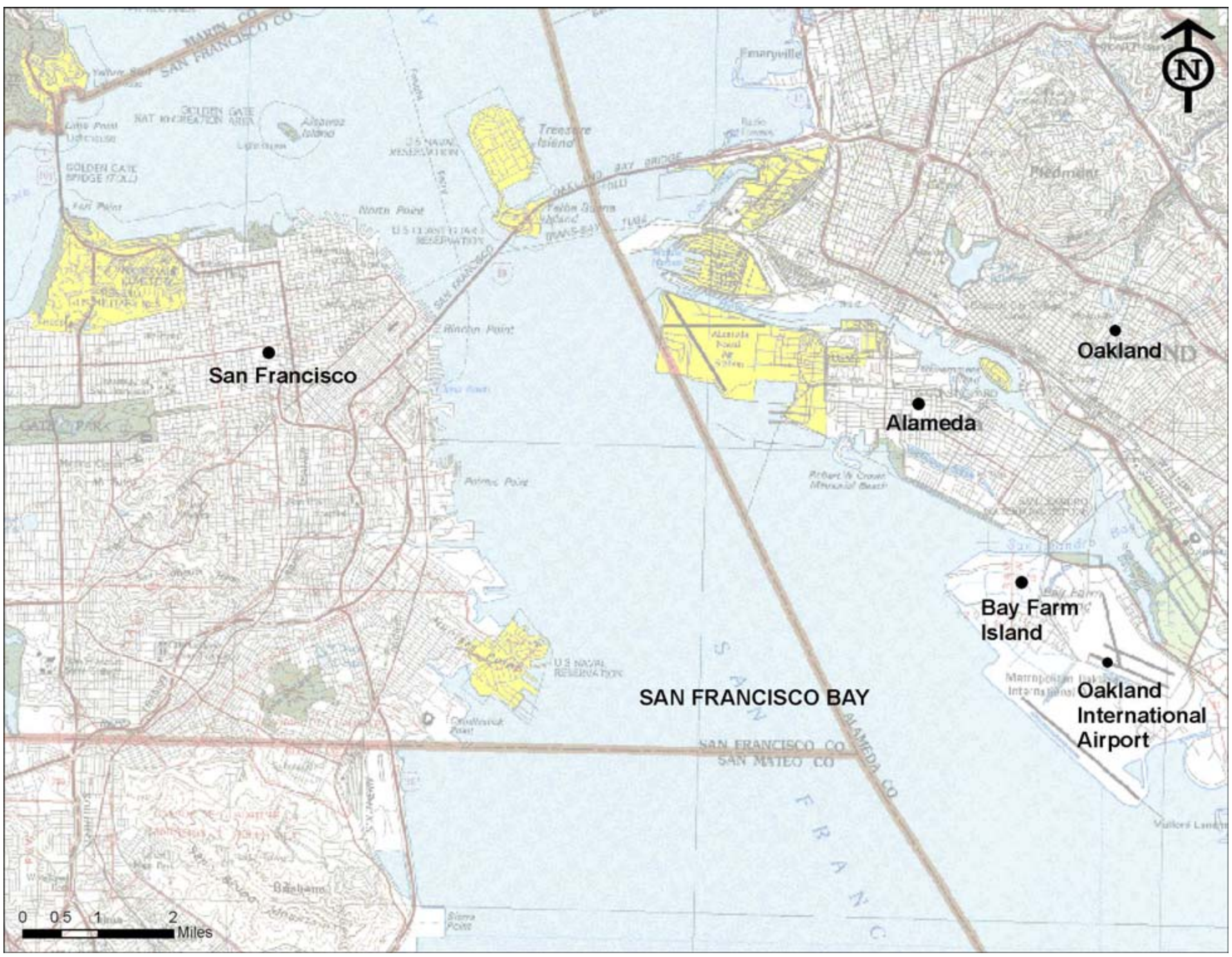


Figure 2-1: Vicinity Map

Alameda Island is relatively flat, with elevations ranging from negative 1 foot National Geodetic Vertical Datum (NGVD), just below mean sea level, to about 40 feet NGVD.

Figure 2-2 delineates the City's eight major drainage areas, all of which drain either by gravity or pump discharge into the waters surrounding Alameda Island and Bay Farm Island. There are four drainage sub-areas identified on Alameda Island, and four on Bay Farm Island. The study area is defined as the existing pipe network within the City of Alameda (excluding the Alameda Point Area) and each network's tributary area. Refer to Appendix A for labeled catchments within each drainage area.

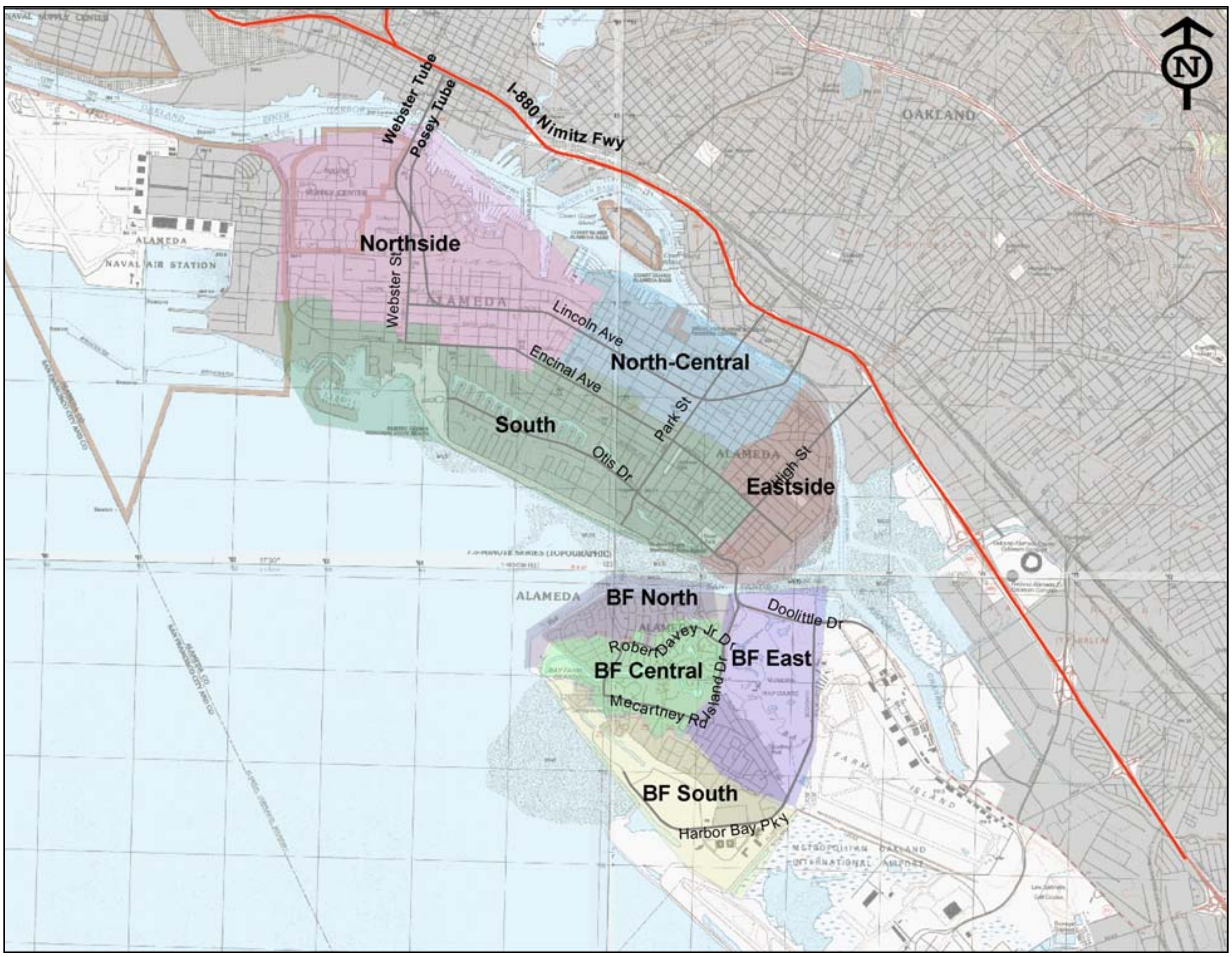


Figure 2-2: Drainage Sub Areas

Climate

Alameda's climate is marine-influenced with an average summertime high temperature of 73°F, dropping to an average winter nighttime low temperature of 45°F. Mean annual precipitation is roughly 19 inches, with the majority of that precipitation falling from November through March. Precipitation occurs entirely as rainfall. Snowmelt is not a hydrologic process that significantly affects runoff in the City.

Soils

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A,B,C, and D) according to their infiltration rate, which correlates to its ability to absorb and transmit water; this aids in the determination of total runoff. NRCS has classified all soils within the City of Alameda as group D, which have very slow infiltration rates and will increase the amount of runoff, affecting the magnitude of flood risk experienced throughout the City. A map of the City of Alameda along with the soil groups is shown in Figure 2-3.

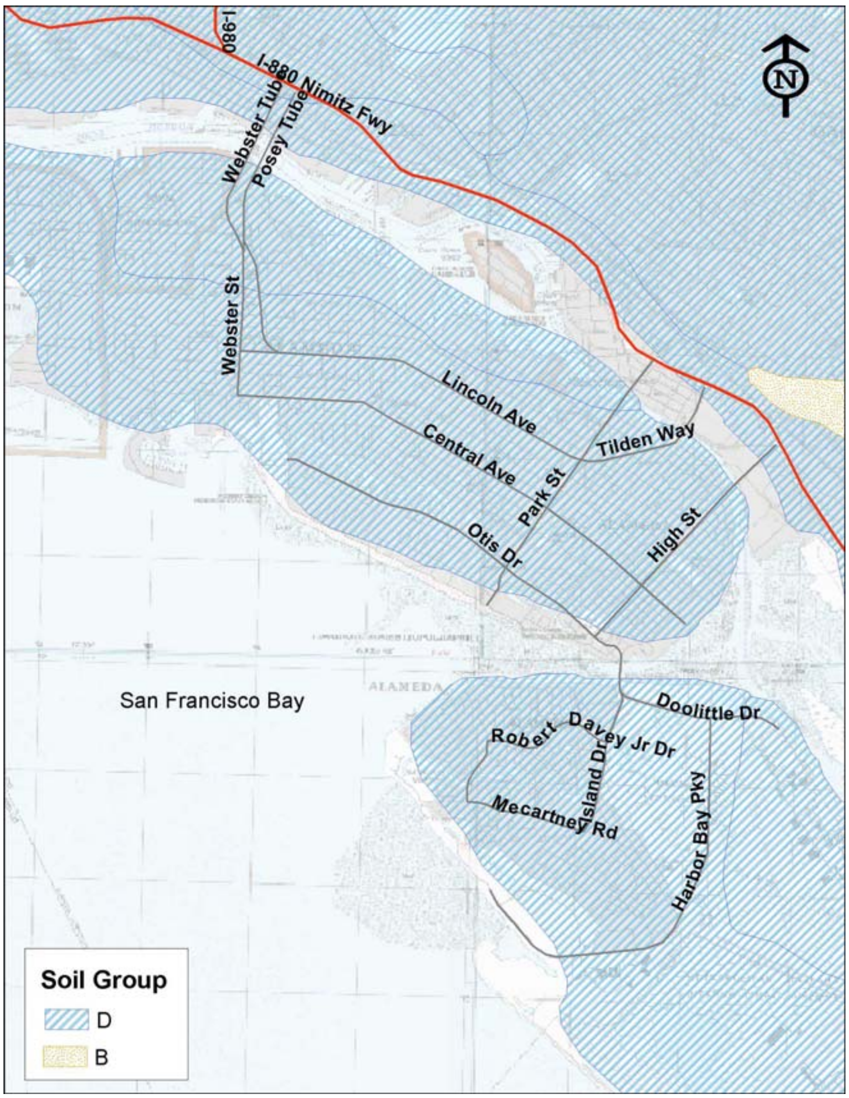


Figure 2-3: NRCS Hydrologic Soil Groups

The west end of Alameda Island is the former Naval Air Station, now known as 'Alameda Point', and is excluded from this study. The 'Southshore' area of Alameda is the area located south of the largest lagoon on Alameda Island, which is also generally south of Otis Drive. Both Alameda Point and the Southshore areas are built largely on fill.

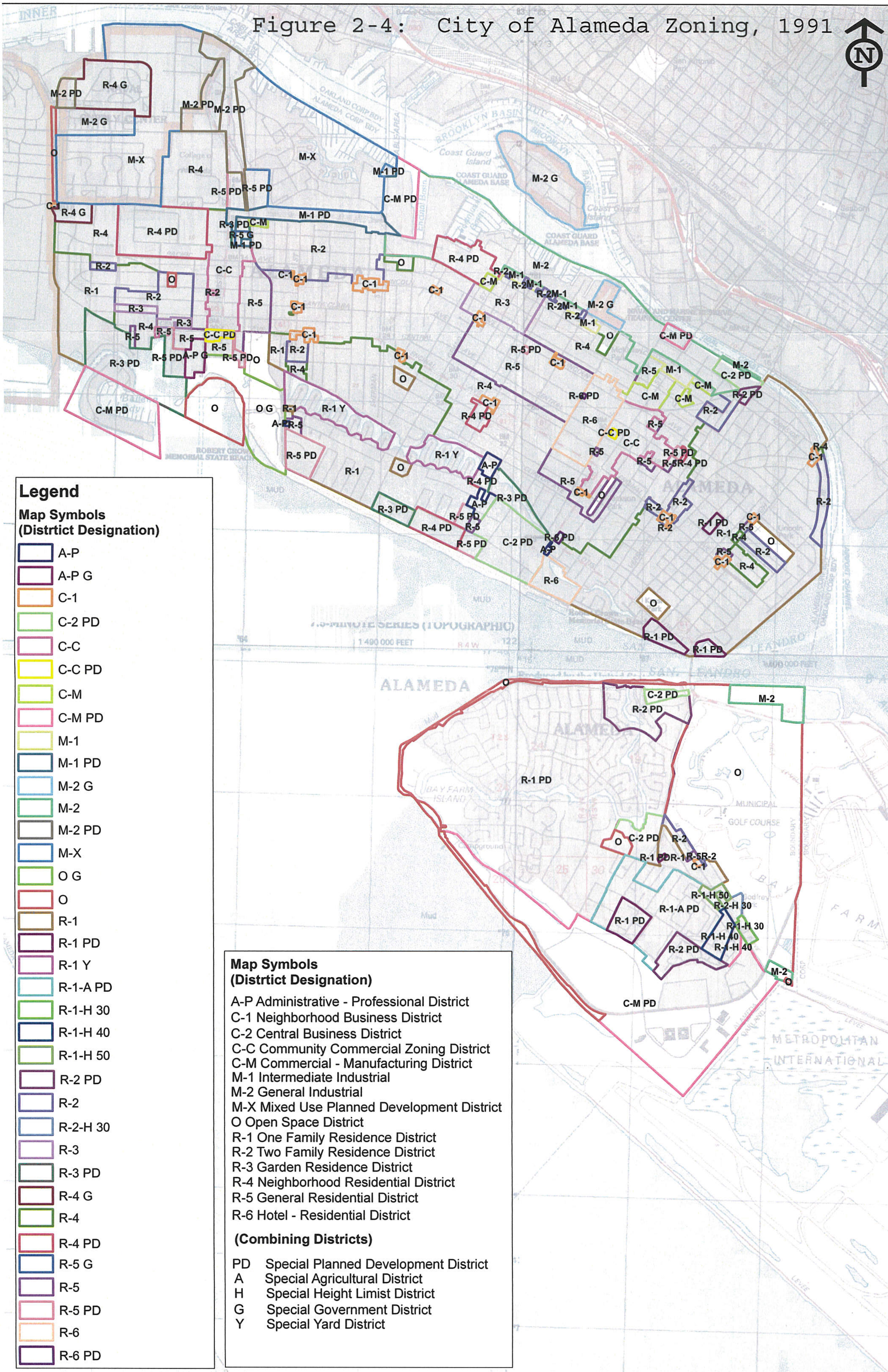
Land Use

Although open space is scattered throughout the City, the vast majority of Alameda has been urbanized. Because of its island setting, the City does not have a developing outer edge in the traditional sense. Those areas of the City that are currently being developed, particularly the Northern Waterfront area, are generally land uses changing from under-utilized industrial to mixed commercial and residential land use. As such, because these changes in land use tend to not increase the impervious surface in an area, there is expected to be no significant changes to storm water runoff due to future land use conditions. The 'Alameda Point' area, formerly a Naval Air Station, was decommissioned in 1997 and is currently a potential development area. The most current plan for the area proposes a golf course, National Wildlife Refuge area, and a development area. As in the case of the Northern Waterfront area, none of these developments are expected to create a significant net increase of impervious area due to the nature of the existing land coverage.

The *1991 City of Alameda General Plan* sets the City's development policies for the period 1990-2010. The General Plan presents land use classifications, which are broad categories including residential (low, medium, and 'measure A exception' - a development specific residential allowance), neighborhood business, community commercial, office, business park, general industry, commercial recreation, open space, public (e.g. schools and City facilities), federal facilities, and various specified mixed use designations. A more detailed delineation of City land uses is shown on the City-wide Zoning Map adopted in 1958 and updated to reflect up to1991 revisions.

This map uses 21 different categories to describe the land use within the City limits. Zoning within the City appears to be stable, and as noted in the 1991 General Plan, it is unlikely that existing residential land will be developed into commercial or industrial land. In general development consists of redevelopment of underutilized industrial land to residential uses (General Plan, p. 8). Schaaf & Wheeler rectified and delineated the City zoning map so that the land use for each parcel within the study area is known (see Figure 2-4). Each land use area is assigned an initial and constant loss rate that varies with land use, as set forth in the Alameda County Hydrology Manual and explained in more detail in Chapter 3.

Figure 2-4: City of Alameda Zoning, 1991



Moderately wide mixes of land uses characterize Alameda, but through intentional planning by the City it retains a 'small town' feeling. In 1973 an initiative known as Measure A was passed which prohibits residential structures having more than 2 units. The 'Measure A Exception' land use category is the result of a City Council Settlement Agreement which allowed the Alameda Housing Authority to replace 325 low cost housing units with multi family housing at the same density.

Most residential areas retain some open space in the form of lawns and gardens, and public parks are scattered throughout the City. Because of Measure A, residential areas within the City tend to be lower density than in other San Francisco Bay Area cities.

FLOOD PROTECTION FACILITIES

In addition to storm drains, flood protection is provided to the City of Alameda by a series of lagoons and pump stations that convey storm-generated runoff to the San Francisco Bay, the Alameda/Oakland Canal, or the San Leandro Channel. Figure 2-5 shows these facilities.

Figure 2-5: Drainage Facilities

Precipitation that falls on land within the City of Alameda generates stormwater runoff. This runoff is conveyed in a number of mostly manmade flood protection systems to discharge to the tidally influenced Bay or Canal. These systems interact with one another, and potential improvements to one system may impact the performance of other systems, either positively or negatively. The City of Alameda watershed, due to its island and peninsular setting, is entirely contained within the City itself. It is assumed (and updated topography seems to support the assumption) that no runoff from the Oakland Airport travels toward Bay Farm Island. Thus the total area of the watershed is equivalent to the study area area, which is roughly 9 square miles (5,900 acres).

A ridgeline runs generally through the middle of Alameda Island in the northwest-southeast direction, forming the most noticeable watershed feature on the Island. Bay Farm Island drains toward the shoreline or inward towards one of the four (4) lagoons. Rainfall flows overland via street gutters to storm drain inlets. The storm drain inlet types range from older, arched curb inlets to more modern gutter grates. The City standard plans currently include seven types of catch basin inlets. There are few inlets or pipes near the ‘peak’ of the Island.

Storm Drain Network

Once flow enters a storm drain, it travels through storm drain pipes until discharging to a lagoon, surrounding waters (i.e. San Francisco Bay, Oakland Canal, etc.) or reaching a pump station. The majority of pipes that discharge directly to the Bay do not have flap gates. Lagoons in the City drain eventually to surrounding waters through a system of storm drain pipes and weirs. Although generally not fitted with flap gates, weir structures and slide gates moderate backflow into the Lagoons from the surrounding waters. The tributary areas for each drainage sub-area in Alameda and the total length of associated storm drain pipes (12 inches and larger) and pump stations are shown in Table 2-1.

	Area (square miles)	Pipe (miles)	Pump Stations
Alameda Eastside	0.72	4.9	1
Alameda North-Central	1.0	6.2	0
Alameda Northside	2.3	24.9	6
Alameda South	2.4	11.2	0
Bayfarm East	0.93	5.1	1
Bayfarm North	0.38	3.7	1 (manual)
Bayfarm Central	0.58	11.1	0
Bayfarm South	0.85	8.2	1 (manual)
TOTAL	9.2	75.3	8 Automated 2 Manual

Table 2-1: Watershed Areas, Pump Stations and Length of Storm Drain Pipe

Seven pump station systems provide vital flood protection for the Island of Alameda. Bay Farm Island, however, relies almost entirely on gravity flow outlets and storage in lagoons for flood protection, with three pumps (one automated and two manually controlled) serving to empty and control water elevation in the lagoons. Pump stations studied in this master plan on Alameda Island include the Main Street, Third Street, Bayport, Webster Street, Northside (Marina Village), Arbor (aka Northshore), and the Central/Eastshore pump stations. Bay Farm Island includes the Golf Course pump station and two pumps that are manually operated to manage the Harbor Bay lagoon water surface elevations both for flood protection and seasonal recreational activities. The pump station locations are noted in Figure 2-5, and further pump station descriptions are provided in Chapter 6.

The Storm Drain Master Plan provides a numeric model of the City's local storm drains and ties them into the major flood protection facilities. This effort represents a comprehensive storm drain planning study for the City of Alameda.

Major flood protection facility improvements (i.e. to lagoons or pump stations) are not analyzed in detail in this master plan. General recommendations for increasing pump station capacity or new pump station locations have been identified; however subsequent detailed design for pump station improvements are beyond the scope of this work.

A coincident 10- or 25-year tide cycle is used as a boundary condition for all storm drain outlets. The development of the design tide cycles and their effects on modeled improvements are discussed in Chapter 3. Generally Schaaf & Wheeler has found that tides have a somewhat limited effect on flooding within the City. After finalizing improvements to meet the standard of protection set forth by the City of Alameda, two sea-level rise (global warming) scenarios have been analyzed to determine potential impacts to the improved flood protection network and assess its robustness.

HISTORY OF FLOODING WITHIN ALAMEDA

Heavy rainfalls in the winter months produce flood situations in the City of Alameda. Historical flooding information can be valuable in highlighting areas of recurring problems, and prioritizing future improvements. Areas with known flooding problems have been identified by City employees in previous studies, as well as in discussions during the course of this report. The most common local flooding occurs as a result of leaf litter in the system, which can plug inlets and significantly reduce the effectiveness of pump stations, obstructed outlets due to either vegetation or vandalism, and tree roots interfering with gutter or culvert flow. Flooding due to capacity limitations has been witnessed during extreme events, and claims for flooding homes have occurred in the City as recently as 1997. Based on the *Storm Drain Facilities Rehabilitation and Repair Report* (Harris & Associates, 1997) areas of known flooding issues include the Oak and Lincoln Streets intersection, the Taylor Street at Page and Eighth Streets, Johnson Avenue at Mound Street, Second Street at Brush, and along several blocks of Central between Pearl and High Streets. These areas are highlighted on the map in Figure 2-7. The numbers on Figure 2-7 correspond with descriptions of the storm drain deficiencies from the 1997 Harris report (see Table No. 1, pages 3 – 6). It should be noted that although this report was used to identify historic flooding areas, when assessing improvement needs and priorities this report was not utilized, instead model results and information from City staff was used to determine the extents of improvements and their relative priority ranking.

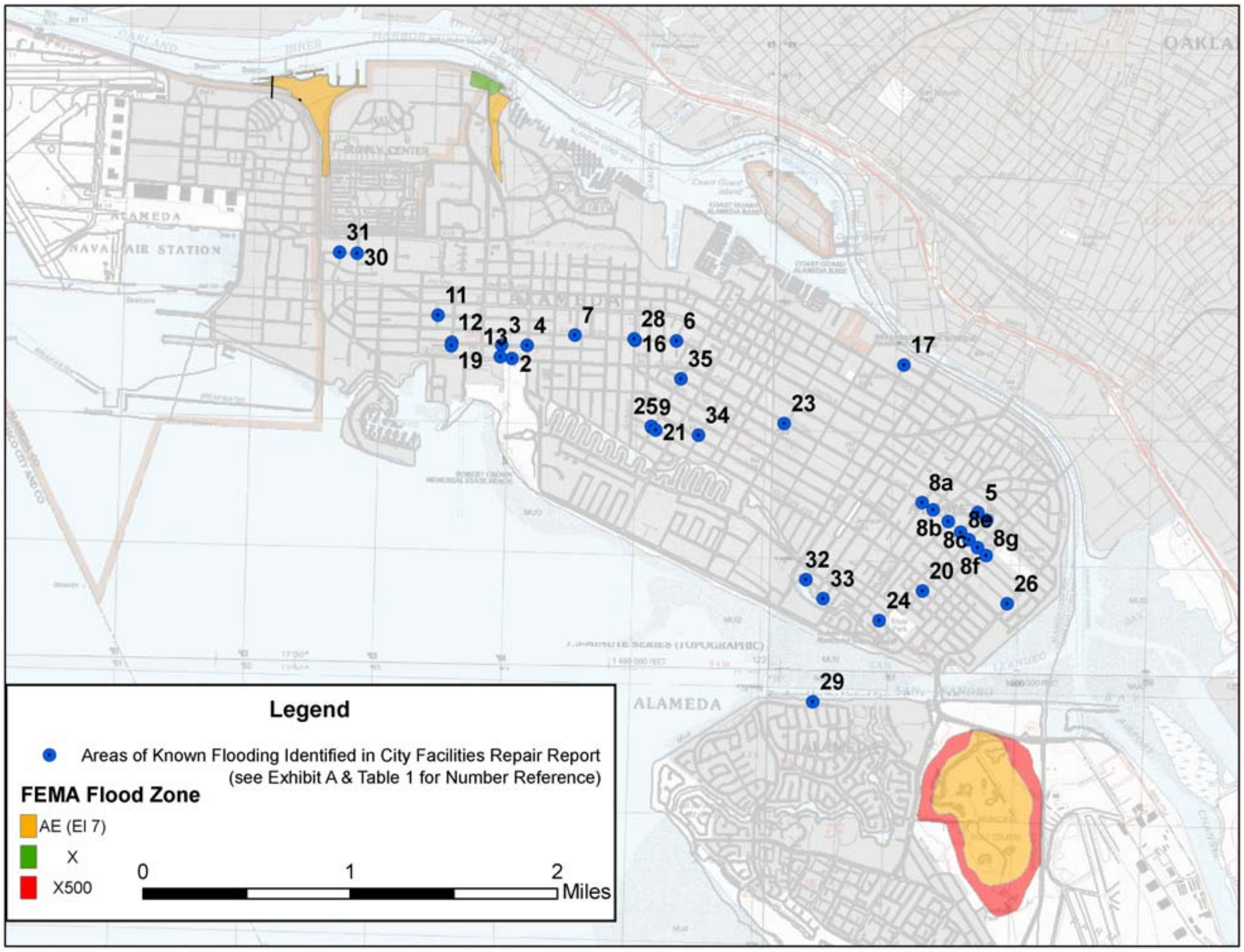


Figure 2-6: Known Flooding Locations

Flooding locations during a 100-year storm event were identified within the *1991 FEMA Flood Insurance Study* (FIS) for the City of Alameda, and updated in several subsequent letter of map revisions (LOMRs). The only area shown in the 100-year FEMA floodplain on Alameda Island is along and between the Southern Pacific Railway/Constitution Way and Main Street. This flooding is primarily Zone AE (El 7) which is flooding from the San Francisco Bay or Alameda Canal/Harbor. Zone X is defined by FEMA as areas of 100-year sheet flow flooding where average depths are less than 1 foot, and zone X500 is an area that falls between the 100 and 500-year flood zone. It appears as though the Webster Street pump station is within or immediately adjacent to the 100-year AE 7 flood zone. Approaches to removing the 100-year flooding should focus on raising the land at or above the base flood elevation. Storm drain improvements will have no impact to the FEMA 100-year floodplain.

Subsequent to the FIS there have been at least three LOMRs, each of which slightly changes the flood zone area in this vicinity. The first two LOMRs added an A flood zone to a detention basin located between Fifth and Main Streets, just south of the Naval Air Station (NAS) Alameda Supply Annex area. The most recent LOMR connected the two Zone AE (El 7) areas generally along Mitchell Avenue. An additional Zone A within a detention basin was also added.

Although significant areas of Alameda have experienced flooding during severe storm events in the past, this historically has not translated to significant structural damage in Alameda. Damage due to flooding is generally limited to landscaping losses and expenses on behalf of the City for staffing needs to prepare for, handle, and recover from flood events, although there have been a small number of claims for past structural damage due to flooding events.

Recent Flood Protection Measures Taken

The City of Alameda recognizes inadequacies in the existing storm drain system. In an effort to alleviate this problem, they have completed some pump and pipeline improvements. Recent City activity has focused on:

1. Construction of the Main Street pump station to alleviate flooding along Main Street (1998),
2. Construction of the Bayport Pump Station (2005),
3. Tinker Avenue street improvements west of Webster Street (2008)
4. Ongoing maintenance activities to keep storm drain inlets and outlets clear of vegetation and debris,
5. Ongoing categorizing and evaluation of problem areas within the storm drain network

MASTER PLAN OBJECTIVES

The basic objectives of this master plan are to evaluate existing storm drainage conveyance, storage and pumping facilities and identify capital improvements needed to provide a level of flood protection consistent with the policies of the Federal Emergency Management Agency (FEMA) as administered through the National Flood Insurance Program (NFIP) and City policies.

NFIP regulations define the “base flood” as a flood magnitude having a one percent chance of being equaled or exceeded in any given year. Often this is referred to as a “one-percent” or “100-year” flood. This level of risk, however, should not be confused with a flood that will occur once every one hundred years, but one that might occur once every one hundred years on the average over a very long period of time. In fact, over the life of a 30-year mortgage, there is a 26 percent chance of experiencing a flood equal to or greater in magnitude than the base flood, and a 96 percent chance of experience a 10-year or lesser storm event. This is demonstrated by Table 2-4, which provides an interesting perspective on flood risk.

Table 2-4: Relative Risk of Various Flood Events

	10-year	25-year	100-year
Annual risk of event	10%	4%	1%
Risk of at least one event in 5 years	41%	18%	5%
Risk of at least one event in 10 years	65%	34%	10%
Risk of at least one event in 30 years	96%	71%	26%
Risk of at least one event in 50 years	99%	87%	39%
Risk of at least one event in 100 years	99.997%	98%	63%

Based on the statistics presented above, this Master Plan establishes level-of-service criteria for the design of new drainage systems and the evaluation of existing systems. The Master Plan seeks to:

- Assess the performance of storm drainage systems against those criteria;
- Identify capital improvements to reduce flood risk and meet those criteria; and
- Prioritize said capital improvements based on risk reduction.

The 10-year storm event is used as the basis of design for all improvements in this storm drain master plan. The City of Alameda is also interested in additional or upsized improvements required to apply the same standard of protection to the 25-year storm event. The results of that analysis are included in Appendix C.

CHAPTER 3
METHODOLOGIES

The criteria used to evaluate storm drain system performance must be defensible yet simple to understand and apply. Ideally, the same criteria used to analyze system performance will also continue to be used for future infrastructure design. As discussed in this chapter and the next, storm drain evaluation criteria have developed with input from the City of Alameda and are also based on engineering judgment.

GIS BASED MODELING

The MIKE-URBAN (MOUSE) model has been selected to model the City of Alameda storm drains and pumps because it is tested and reliable software with a GIS interface. MOUSE is a package of software programs designed by the Danish Hydraulic Institute (DHI) for the analysis, design and management of urban drainage systems, including storm water sewers and sanitary sewers. The MOUSE model works within ArcView GIS and can simulate runoff, open channel flow, pipe flow, water quality and sediment transport. The program has been chosen to model the Alameda storm drain system because of its capabilities with overland flow, pumps, and storage areas; the incorporation of the Alameda County hydrology method; and the overall stability of the model. The City's modeling package consists of three interrelated products:

1. MOUSE is a group of hydrologic, hydraulic, water quality and sediment transport modeling modules which can be used together or independently. The modules used in the Alameda Storm Drain model include the Surface Runoff Module, which computes surface runoff using one of five computational methods; and the Hydrodynamic Pipe Flow Module, which calculates an implicit finite-difference numerical solution of the St. Venant flow equations for the modeled pipe network.

2. MIKE-URBAN (MU) is an ArcView based program which includes tools specifically designed to develop urban drainage models. MU provides a graphical user interface for data input and editing and serves as a bridge between ArcView GIS and the MOUSE modeling program. Capabilities of MU include import and export of model data, network editing and gap-filling, catchment delineation, network simplification, and importation and presentation of model results.

3. MIKEVIEW is a graphical tool used for viewing and presentation of MOUSE results. Capabilities include plan, longitudinal, and cross-section views; animation of results; presentation of flooding including water depth and pressure; and overlay of results on background graphics such as maps or aerial photos.

Data Sources

Some of the data used in this master plan has been obtained from AutoCAD data provided by the City. New development and street improvement plans have been consulted to fill in missing or conflicting information. Most data elevations are based on the City's vertical datum, which matches the AutoCAD records obtained from the City of Alameda, as well as most construction and record drawings for the storm drains. As a part of this study, the City obtained aerial LiDAR topography data, which provides topographic information for the City to within a half foot accuracy (plus or minus 0.5 foot) based on a NGVD29 datum. The data is provided in the State Plane (California Zone III) coordinate system, and covers the entire study area, but does not cover the former Alameda Naval Air Station (now Alameda Point).

Conversion from NGVD29 to City datum can be achieved using the following equation:

NGVD29 – 3.41 feet = City of Alameda Datum

It should be noted that generally Bay Farm Island data is on its own datum, which is the City of Alameda Datum plus 100 feet. In formula format, this is:

NGVD29 + 96.59 feet = Bay Farm Island Datum

Information regarding pump station operation has been obtained from conversations with City operations and maintenance staff, a tour of the facilities with maintenance staff, and available records. The Zoning Map previously described (Figure 2-4) is used to define the land use within Alameda.

Data Inadequacies

The City provided Schaaf & Wheeler with AutoCAD files for the storm drain system. Schaaf & Wheeler converted this data to GIS shapefiles for use in the modeling. The initial AutoCAD (and therefore GIS) data was missing a large quantity of information critical to accurately modeling the storm drain system. Routinely encountered examples include:

- missing pipe sizes
- no manhole indicated where two pipes join
- catch-basins represented as manholes
- sections of the system not drawn into the plans
- rim and/or invert elevations missing from manholes and catch-basins (nodes).

When AutoCAD is converted to GIS data, all of the attributed values in CAD are brought into GIS. However un-attributed data (for example text layers that are not linked to the pipe layer) are not brought into GIS. Schaaf & Wheeler found that much of the storm drain data in AutoCAD was either missing or un-attributed. Once in GIS, out of a total of 1,222 manholes in the original system provided by the City, 296 were missing both rim and invert elevations. Of 2,418 inlet nodes, 370 were missing rim and invert elevations; and of 3,837 pipe links, 1,123 were missing diameter information. Of all the nodes (both manholes and inlets, 3,640 total), 963 had an assigned invert value of zero and 1,460 had an assigned rim value of zero, which in some cases reflected an actual elevation, but in many cases was equivalent to a 'null' value (i.e. missing information). Numerous steps were taken to collect accurate missing data.

First, the AutoCAD file was compared to the GIS file and any information that was in AutoCAD but not attributed, was manually entered into GIS. The next steps included gathering and reviewing record drawings, and extensive field research to verify pipe sizes, layouts, and to measure invert depths. The previously described LiDAR data has been used to assign rim elevations to all nodes on a consistent known datum. Invert data from AutoCAD has been used wherever possible, and the field measured invert depths are used to assign missing invert elevation data. In cases where AutoCAD, record drawings, or field data is not available, Schaaf & Wheeler has interpolated invert data or pipe sizes based on available information.

MIKE-URBAN MOUSE MODEL

The City of Alameda storm drain system is modeled as eight independent urban drainage systems based on outlet points and major drainage for each area. On Alameda Island these sub-areas are: South, Eastside, Northside and North-Central. On Bay Farm Island, the four areas are Central, North, East and South. In order to keep track of the separate Bay Farm and Alameda Island sub-areas, all Bay Farm Island sub-area nomenclature is preceded with a "BF" denotation. Each drainage system model is composed of a pipe network (pipes, manholes, catch basins, etc.), and the urban catchments drained by the pipe network.

Operation

Two separate calculations are performed by MOUSE for the Alameda model: a stormwater runoff calculation that determines the amount of water entering the storm drain system from a specific rainfall event; and a pipe flow calculation that replicates how the storm drain system, including pumps, will convey those flows to outlets. Flows resulting from the runoff calculation are used as inflows for the subsequent pipe flow calculation.

MOUSE has five runoff routing descriptions: Time-Area, Kinematic Wave/Non-Linear Reservoir, Model C1, Model C2, and the Unit Hydrograph Method (UHM). The Alameda storm drain model uses the UHM model with the Alameda County synthetic unit hydrograph (SUH) method to calculate surface runoff. The runoff simulation duration is set equal to the design storm duration or some lesser duration depending on the period of interest; a 24-hour storm is used in Alameda. The model can be started at any point during the chosen design storm to assess surface runoff for any period of the design storm, with computations made based on a user-specified constant time step.

The MOUSE pipe flow model offers a choice of three flow description approximations: Dynamic Wave, Diffusive Wave, and Kinematic Wave; distinguished based on the set of forces that each takes into account. The Alameda storm drain model uses the most comprehensive flow description, Dynamic Wave, which incorporates the effects of gravitational, friction, pressure gradient and inertial forces. Because it accounts for all forces affecting flow conditions, this equation allows the model to accurately simulate fast transients and backwater profiles. As the calculated Froude number increases from 0 to 1, a reduction factor (decreasing from 0 to 1) is used on the calculated inertial forces. The simulation of flooding at a node is accommodated by the insertion of an artificial basin above the node which will store water when the water level rises above the ground level. The surface area of the basin gradually increases (up to a maximum of 1000 times the node surface area) with rising water levels at the node; replicating the effects of flooding. Water stored in the basin begins to re-enter the system when the outflow from the node becomes greater than the inflow. The pipe flow simulation can be executed using either a constant or variable time step, and can be run for any portion of the time interval specified by the input rainfall time series and corresponding calculated runoff hydrograph. A variable time step range of 1 to 60 seconds is used for most models within Alameda.

Input and Output

MOUSE surface runoff calculations require two types of input data: boundary data and urban catchment data. Boundary data for the run-off computation consists of an input rainfall time series representing the design storm event for the model. Urban catchment data includes the boundaries of each drainage catchment, along with relevant physical and hydrologic parameters including surface area and parameters used to calculate basin lag time. Drainage catchments for the study area are shown and labeled in Appendix A, and input data corresponding to the catchments are provided within the digital data in Appendix E. The runoff calculation output is a runoff hydrograph that corresponds to the input rainfall time series.

MOUSE pipe flow calculations require network data, operational data, and boundary data as input. Network data consists of the pipe network elements including nodes (manholes, outlets and storage nodes) and links (pipes, culverts, and roads modeled as open channels). Parameters required to describe nodes include the x and y coordinates of the node, a unique name, node type, diameter for manholes, geometry for storage areas, ground and invert levels, and water levels in outlets. For the Alameda storm drain model, a coincident tide cycle corresponding to the storm event is input as the boundary condition water level at San Francisco and Channel outlets. (A full description of the development of coincident tide cycles closes this chapter.)

Parameters required to describe links include name of upstream and downstream nodes, shape and dimensions, material, and upstream and downstream inverts. Streets are input as links with the cross section of the streets based on the City standard plans. Structural system elements including gates, weirs, pumps and orifices are all modeled as functional relationships connecting two nodes in the system, or associated with one node in the case of free flow out of the system. Operational data consists of parameters which describe how these elements function in the network. Boundary data for the pipe flow computation can include any external loading, inflow discharges, water levels at interaction points with receiving waters; as well as the results of a run-off calculation. Figure 3-1 displays several of these input parameters.

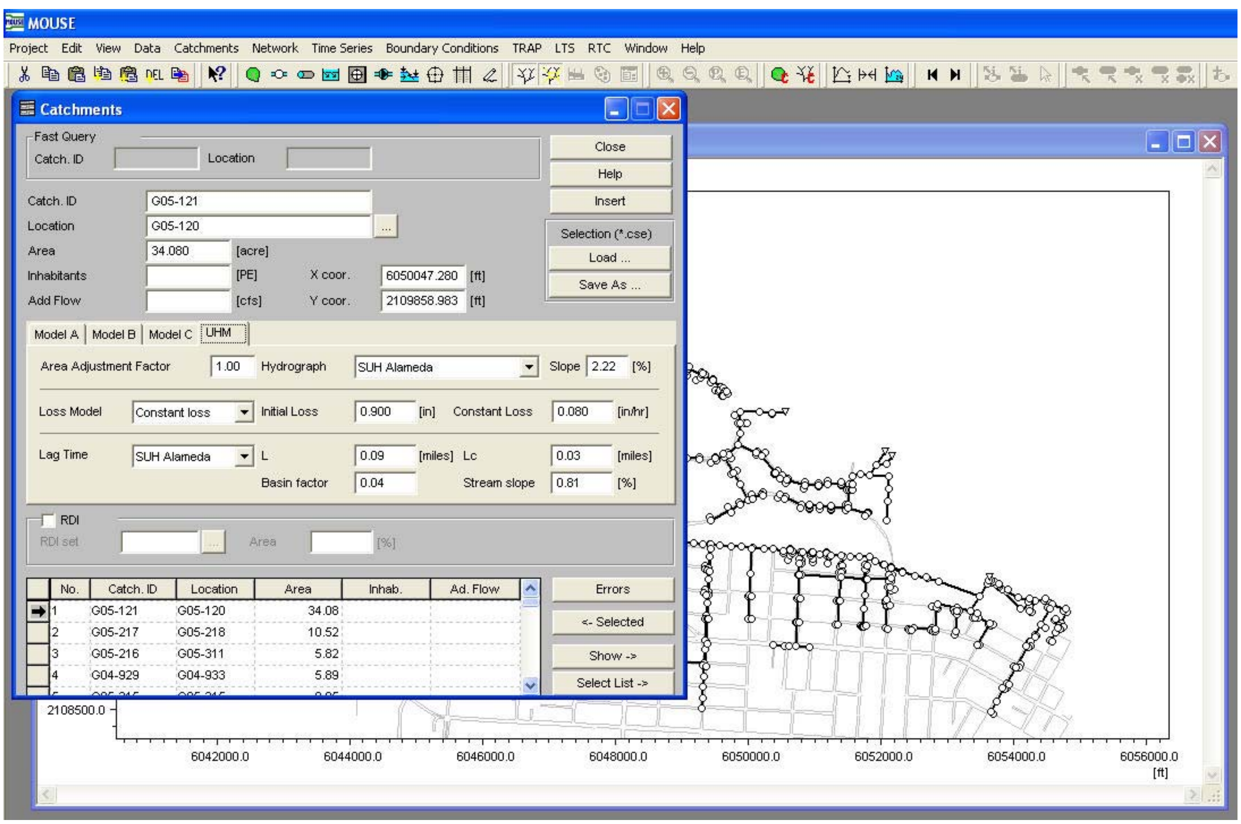


Figure 3-1: MOUSE Input

Output from the pipe flow computation includes the calculated water level at each node, pump discharges, weir discharges, water level in network branches, discharge in network branches, water velocity in network branches, water volume in the system, and time step data. Output is viewed using the MIKE View program. Results may be displayed in plan view or as a profile for a selected network section, and may be viewed as a temporal animation or at maximum or minimum values. Additional outputs which can be derived from MOUSE pipe flow results using MIKE View include water depth, flooding, pressure in closed conduits, percentage pipe filling, the flow (Q) calculated from Manning's equation for each link, and model instability.

RUNOFF ESTIMATION

Methods used in this master plan to estimate peak storm water flow rates and volumes require the input of precipitation data. Since it is impossible to anticipate the effect of every conceivable storm, precipitation frequency analyses are often used to design facilities that control storm runoff. A common practice is to construct a design storm, which is a rainfall pattern used in hydrologic models to estimate surface runoff.

A design storm is used in lieu of a historic storm event to ensure that local rainfall statistics (i.e. depth, duration and frequency) are preserved. When combined with regional specific data for land use and loss rates, the model should produce runoff estimates that are consistent with frequency analyses of gauged streamflows in the Alameda County area. In other words, the ten-year design storm pattern used for MOUSE modeling is consistent with a ten-year storm runoff event.

Precipitation frequency analyses are based on concepts of probability and statistics. Engineers generally assume that the frequency (probability) of a rainfall event is coincident with the frequency of direct storm water runoff, although runoff is determined by a number of factors (particularly land use conditions in the basin) not necessarily dependent upon the precipitation event. For the purpose of evaluating storm drain performance for this master plan, relevant frequency of occurrence for precipitation (and by assumption, runoff) studied were both ten and twenty five years. For readability and conciseness this report presents the results of the 10-year analysis. Results for the 25-year analysis can be found in Appendix C.

Unit Hydrograph and Design Storm

The synthetic unit hydrograph is a numerical representation of the time response of catchment runoff caused by one inch of excess rainfall applied uniformly over a unit of time. Many different techniques are available to estimate unit hydrographs. Alameda County has adopted a modified Snyder Unit Hydrograph method to transform hypothetical rainfall distribution and design rainfall depth into a runoff hydrograph. The rainfall distribution patterns for the Alameda Storm Drainage Master Plan is obtained from the Alameda County Hydrology and Hydraulics Manual (June 2003). The County's rainfall pattern is distributed in 15-minute time increments with a fraction of the total rainfall apportioned to each 15-minute increment. The resulting 24-hour rainfall pattern with 15-minute time steps is then balanced using HEC-1 such that the total rainfall resulting from the pattern matches the total rainfall depths for the 15-min, 30-min, 1-hour, 2-hour and 24-hour storm durations obtained using the following equation from Chapter 3 of the County manual:

$$\mathbf{P_{ij} = (0.33 + 0.091144 * MAP) * (0.249 + 0.1006 * K_i) * T_i^{0.43747}}$$

Where P_{ij} = Design rainfall depth (inches) for recurrence interval, MAP = Mean Annual Precipitation (inches), T_i = Storm duration (hours), and K_j = Frequency factor (1.339 for 10-year, 2.108 for 25-year). A Mean Annual Precipitation (MAP) value of 19-inches for the Alameda area is obtained from Attachment A-6 in the County manual.

For the purposes of the SDMP, the County pattern is broken into 5-minute time increments by assuming that the fraction of rainfall for each 5-minute period is equal to one-third of the 15-minute fraction, with the exception of the peak 15-minutes of each storm event, which is proportionally divided using the 5-, 10-, and 15-minute peak intensities from the County's intensity-duration-frequency (IDF) tables. Each fractional rainfall is multiplied by the total rainfall depth for the storm event, and then converted to a 'per hour' unit for input to the MOUSE model.

The 10-year balanced storm intensity graph is shown in Figure 3-2, with the resulting 10-year design rainfall shown in tabular values in Table 3-1. The same information for the 25-year storm event is included in Appendix C.

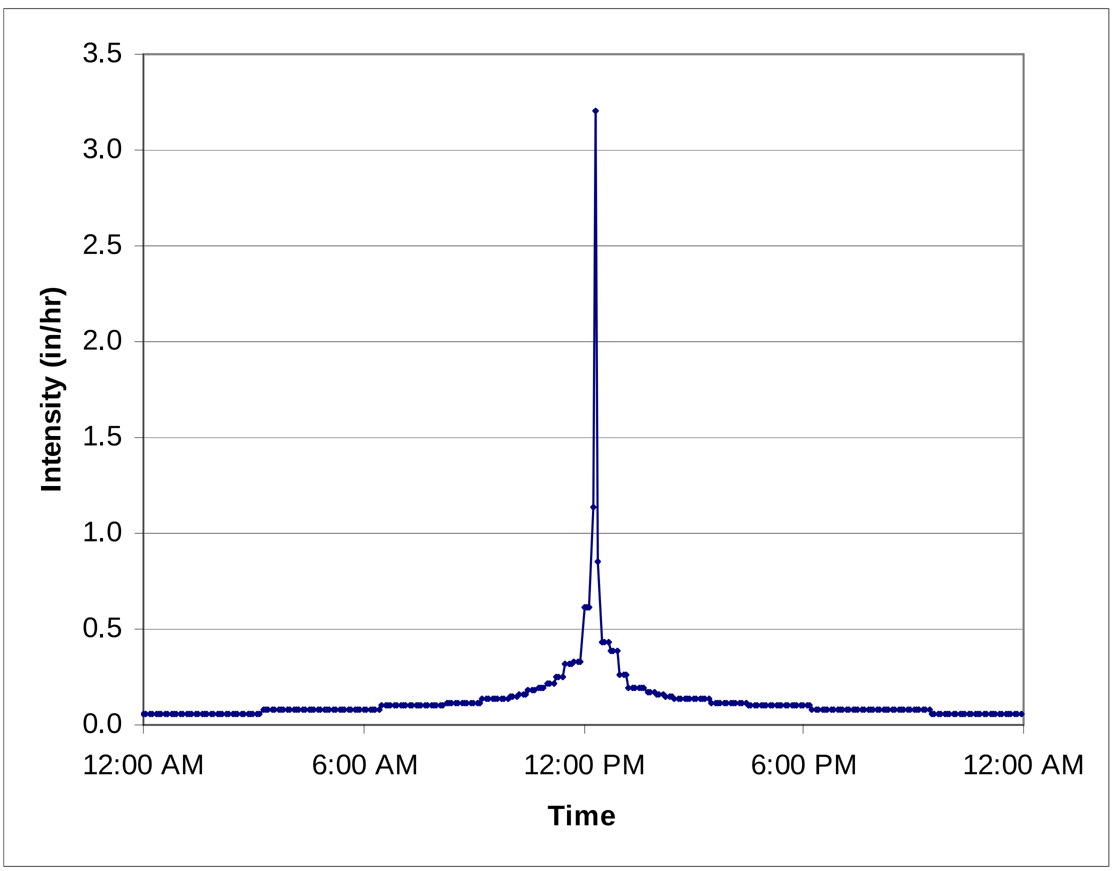


Figure 3-2*: Balanced 10-Year Storm Intensity Graph*

Table 3-1: 10yr-24hr Design Storm Values

Time (hr)	Unitless Rainfall (U/5 min)	Hyd Intensity (in/hr)	Act Rain (in)	Time (hr)	Unitless Rainfall (U/5 min)	Hyd Intensity (in/hr)	Act Rain (in)	Time (hr)	Unitless Rainfall (U/5 min)	Hyd Intensity (in/hr)	Act Rain (in)	Time (hr)	Unitless Rainfall (U/5 min)	Hyd Intensity (in/hr)	Act Rain (in)
12:00 AM	0.0015	0.0581	0.0048	6:00 AM	0.0020	0.0775	0.0065	12:00 PM	0.0160	0.6101	0.0508	6:00 PM	0.0025	0.0969	0.0081
12:05 AM	0.0015	0.0581	0.0048	6:05 AM	0.0020	0.0775	0.0065	12:05 PM	0.0160	0.6101	0.0508	6:05 PM	0.0025	0.0969	0.0081
12:10 AM	0.0015	0.0581	0.0048	6:10 AM	0.0020	0.0775	0.0065	12:10 PM	0.0160	0.6101	0.0508	6:10 PM	0.0025	0.0969	0.0081
12:15 AM	0.0016	0.0594	0.0050	6:15 AM	0.0021	0.0787	0.0066	12:15 PM	0.0299	1.1400	0.0950	6:15 PM	0.0021	0.0787	0.0066
12:20 AM	0.0016	0.0594	0.0050	6:20 AM	0.0021	0.0787	0.0066	12:20 PM	0.0839	3.2000	0.2667	6:20 PM	0.0021	0.0787	0.0066
12:25 AM	0.0016	0.0594	0.0050	6:25 AM	0.0021	0.0787	0.0066	12:25 PM	0.0223	0.8500	0.0708	6:25 PM	0.0021	0.0787	0.0066
12:30 AM	0.0015	0.0581	0.0048	6:30 AM	0.0025	0.0969	0.0081	12:30 PM	0.0113	0.4302	0.0358	6:30 PM	0.0020	0.0775	0.0065
12:35 AM	0.0015	0.0581	0.0048	6:35 AM	0.0025	0.0969	0.0081	12:35 PM	0.0113	0.4302	0.0358	6:35 PM	0.0020	0.0775	0.0065
12:40 AM	0.0015	0.0581	0.0048	6:40 AM	0.0025	0.0969	0.0081	12:40 PM	0.0113	0.4302	0.0358	6:40 PM	0.0020	0.0775	0.0065
12:45 AM	0.0016	0.0594	0.0050	6:45 AM	0.0026	0.0982	0.0082	12:45 PM	0.0102	0.3899	0.0325	6:45 PM	0.0021	0.0787	0.0066
12:50 AM	0.0016	0.0594	0.0050	6:50 AM	0.0026	0.0982	0.0082	12:50 PM	0.0102	0.3899	0.0325	6:50 PM	0.0021	0.0787	0.0066
12:55 AM	0.0016	0.0594	0.0050	6:55 AM	0.0026	0.0982	0.0082	12:55 PM	0.0102	0.3899	0.0325	6:55 PM	0.0021	0.0787	0.0066
1:00 AM	0.0015	0.0581	0.0048	7:00 AM	0.0026	0.0982	0.0082	1:00 PM	0.0069	0.2628	0.0219	7:00 PM	0.0021	0.0787	0.0066
1:05 AM	0.0015	0.0581	0.0048	7:05 AM	0.0026	0.0982	0.0082	1:05 PM	0.0069	0.2628	0.0219	7:05 PM	0.0021	0.0787	0.0066
1:10 AM	0.0015	0.0581	0.0048	7:10 AM	0.0026	0.0982	0.0082	1:10 PM	0.0069	0.2628	0.0219	7:10 PM	0.0021	0.0787	0.0066
1:15 AM	0.0015	0.0581	0.0048	7:15 AM	0.0025	0.0969	0.0081	1:15 PM	0.0051	0.1936	0.0161	7:15 PM	0.0020	0.0775	0.0065
1:20 AM	0.0015	0.0581	0.0048	7:20 AM	0.0025	0.0969	0.0081	1:20 PM	0.0051	0.1936	0.0161	7:20 PM	0.0020	0.0775	0.0065
1:25 AM	0.0015	0.0581	0.0048	7:25 AM	0.0025	0.0969	0.0081	1:25 PM	0.0051	0.1936	0.0161	7:25 PM	0.0020	0.0775	0.0065
1:30 AM	0.0016	0.0594	0.0050	7:30 AM	0.0026	0.0982	0.0082	1:30 PM	0.0051	0.1963	0.0164	7:30 PM	0.0021	0.0787	0.0066
1:35 AM	0.0016	0.0594	0.0050	7:35 AM	0.0026	0.0982	0.0082	1:35 PM	0.0051	0.1963	0.0164	7:35 PM	0.0021	0.0787	0.0066
1:40 AM	0.0016	0.0594	0.0050	7:40 AM	0.0026	0.0982	0.0082	1:40 PM	0.0051	0.1963	0.0164	7:40 PM	0.0021	0.0787	0.0066
1:45 AM	0.0015	0.0581	0.0048	7:45 AM	0.0026	0.0982	0.0082	1:45 PM	0.0046	0.1757	0.0146	7:45 PM	0.0020	0.0775	0.0065
1:50 AM	0.0015	0.0581	0.0048	7:50 AM	0.0026	0.0982	0.0082	1:50 PM	0.0046	0.1757	0.0146	7:50 PM	0.0020	0.0775	0.0065
1:55 AM	0.0015	0.0581	0.0048	7:55 AM	0.0026	0.0982	0.0082	1:55 PM	0.0046	0.1757	0.0146	7:55 PM	0.0020	0.0775	0.0065
2:00 AM	0.0015	0.0581	0.0048	8:00 AM	0.0025	0.0969	0.0081	2:00 PM	0.0041	0.1563	0.0130	8:00 PM	0.0021	0.0787	0.0066
2:05 AM	0.0015	0.0581	0.0048	8:05 AM	0.0025	0.0969	0.0081	2:05 PM	0.0041	0.1563	0.0130	8:05 PM	0.0021	0.0787	0.0066
2:10 AM	0.0015	0.0581	0.0048	8:10 AM	0.0025	0.0969	0.0081	2:10 PM	0.0041	0.1563	0.0130	8:10 PM	0.0021	0.0787	0.0066
2:15 AM	0.0016	0.0594	0.0050	8:15 AM	0.0031	0.1175	0.0098	2:15 PM	0.0038	0.1459	0.0122	8:15 PM	0.0020	0.0775	0.0065
2:20 AM	0.0016	0.0594	0.0050	8:20 AM	0.0031	0.1175	0.0098	2:20 PM	0.0038	0.1459	0.0122	8:20 PM	0.0020	0.0775	0.0065
2:25 AM	0.0016	0.0594	0.0050	8:25 AM	0.0031	0.1175	0.0098	2:25 PM	0.0038	0.1459	0.0122	8:25 PM	0.0020	0.0775	0.0065
2:30 AM	0.0015	0.0581	0.0048	8:30 AM	0.0031	0.1175	0.0098	2:30 PM	0.0036	0.1369	0.0114	8:30 PM	0.0021	0.0787	0.0066
2:35 AM	0.0015	0.0581	0.0048	8:35 AM	0.0031	0.1175	0.0098	2:35 PM	0.0036	0.1369	0.0114	8:35 PM	0.0021	0.0787	0.0066
2:40 AM	0.0015	0.0581	0.0048	8:40 AM	0.0031	0.1175	0.0098	2:40 PM	0.0036	0.1369	0.0114	8:40 PM	0.0021	0.0787	0.0066
2:45 AM	0.0016	0.0594	0.0050	8:45 AM	0.0031	0.1175	0.0098	2:45 PM	0.0036	0.1369	0.0114	8:45 PM	0.0020	0.0775	0.0065
2:50 AM	0.0016	0.0594	0.0050	8:50 AM	0.0031	0.1175	0.0098	2:50 PM	0.0036	0.1369	0.0114	8:50 PM	0.0020	0.0775	0.0065
2:55 AM	0.0016	0.0594	0.0050	8:55 AM	0.0031	0.1175	0.0098	2:55 PM	0.0036	0.1369	0.0114	8:55 PM	0.0020	0.0775	0.0065
3:00 AM	0.0015	0.0581	0.0048	9:00 AM	0.0030	0.1163	0.0097	3:00 PM	0.0036	0.1369	0.0114	9:00 PM	0.0021	0.0787	0.0066
3:05 AM	0.0015	0.0581	0.0048	9:05 AM	0.0030	0.1163	0.0097	3:05 PM	0.0036	0.1369	0.0114	9:05 PM	0.0021	0.0787	0.0066
3:10 AM	0.0015	0.0581	0.0048	9:10 AM	0.0030	0.1163	0.0097	3:10 PM	0.0036	0.1369	0.0114	9:10 PM	0.0021	0.0787	0.0066
3:15 AM	0.0021	0.0787	0.0066	9:15 AM	0.0036	0.1369	0.0114	3:15 PM	0.0036	0.1369	0.0114	9:15 PM	0.0020	0.0775	0.0065
3:20 AM	0.0021	0.0787	0.0066	9:20 AM	0.0036	0.1369	0.0114	3:20 PM	0.0036	0.1369	0.0114	9:20 PM	0.0020	0.0775	0.0065
3:25 AM	0.0021	0.0787	0.0066	9:25 AM	0.0036	0.1369	0.0114	3:25 PM	0.0036	0.1369	0.0114	9:25 PM	0.0020	0.0775	0.0065
3:30 AM	0.0020	0.0775	0.0065	9:30 AM	0.0036	0.1369	0.0114	3:30 PM	0.0031	0.1175	0.0098	9:30 PM	0.0016	0.0594	0.0050
3:35 AM	0.0020	0.0775	0.0065	9:35 AM	0.0036	0.1369	0.0114	3:35 PM	0.0031	0.1175	0.0098	9:35 PM	0.0016	0.0594	0.0050
3:40 AM	0.0020	0.0775	0.0065	9:40 AM	0.0036	0.1369	0.0114	3:40 PM	0.0031	0.1175	0.0098	9:40 PM	0.0016	0.0594	0.0050
3:45 AM	0.0021	0.0787	0.0066	9:45 AM	0.0036	0.1369	0.0114	3:45 PM	0.0030	0.1163	0.0097	9:45 PM	0.0015	0.0581	0.0048
3:50 AM	0.0021	0.0787	0.0066	9:50 AM	0.0036	0.1369	0.0114	3:50 PM	0.0030	0.1163	0.0097	9:50 PM	0.0015	0.0581	0.0048
3:55 AM	0.0021	0.0787	0.0066	9:55 AM	0.0036	0.1369	0.0114	3:55 PM	0.0030	0.1163	0.0097	9:55 PM	0.0015	0.0581	0.0048
4:00 AM	0.0020	0.0775	0.0065	10:00 AM	0.0038	0.1459	0.0122	4:00 PM	0.0031	0.1175	0.0098	10:00 PM	0.0015	0.0581	0.0048
4:05 AM	0.0020	0.0775	0.0065	10:05 AM	0.0038	0.1459	0.0122	4:05 PM	0.0031	0.1175	0.0098	10:05 PM	0.0015	0.0581	0.0048
4:10 AM	0.0020	0.0775	0.0065	10:10 AM	0.0038	0.1459	0.0122	4:10 PM	0.0031	0.1175	0.0098	10:10 PM	0.0015	0.0581	0.0048
4:15 AM	0.0021	0.0787	0.0066	10:15 AM	0.0041	0.1563	0.0130	4:15 PM	0.0031	0.1175	0.0098	10:15 PM	0.0016	0.0594	0.0050
4:20 AM	0.0021	0.0787	0.0066	10:20 AM	0.0041	0.1563	0.0130	4:20 PM	0.0031	0.1175	0.0098	10:20 PM	0.0016	0.0594	0.0050
4:25 AM	0.0021	0.0787	0.0066	10:25 AM	0.0041	0.1563	0.0130	4:25 PM	0.0031	0.1175	0.0098	10:25 PM	0.0016	0.0594	0.0050
4:30 AM	0.0020	0.0775	0.0065	10:30 AM	0.0046	0.1769	0.0147	4:30 PM	0.0026	0.0982	0.0082	10:30 PM	0.0015	0.0581	0.0048
4:35 AM	0.0020	0.0775	0.0065	10:35 AM	0.0046	0.1769	0.0147	4:35 PM	0.0026	0.0982	0.0082	10:35 PM	0.0015	0.0581	0.0048
4:40 AM	0.0020	0.0775	0.0065	10:40 AM	0.0046	0.1769	0.0147	4:40 PM	0.0026	0.0982	0.0082	10:40 PM	0.0015	0.0581	0.0048
4:45 AM	0.0021	0.0787	0.0066	10:45 AM	0.0051	0.1950	0.0162	4:45 PM	0.0025	0.0969	0.0081	10:45 PM	0.0015	0.0581	0.0048
4:50 AM	0.0021	0.0787	0.0066	10:50 AM	0.0051	0.1950	0.0162	4:50 PM	0.0025	0.0969	0.0081	10:50 PM	0.0015	0.0581	0.0048
4:55 AM	0.0021	0.0787	0.0066	10:55 AM	0.0051	0.1950	0.0162	4:55 PM	0.0025	0.0969	0.0081	10:55 PM	0.0015	0.0581	0.0048
5:00 AM	0.0020	0.0775	0.0065	11:00 AM	0.0056	0.2145	0.0179	5:00 PM	0.0026	0.0982	0.0082	11:00 PM	0.0016	0.0594	0.0050
5:05 AM	0.0020	0.0775	0.0065	11:05 AM	0.0056	0.2145	0.0179	5:05 PM	0.0026	0.0982	0.0082	11:05 PM	0.0016	0.0594	0.0050
5:10 AM	0.0020	0.0775	0.0065	11:10 AM	0.0056	0.2145	0.0179	5:10 PM	0.0026	0.0982	0.0082	11:10 PM	0.0016	0.0594	0.0050
5:15 AM	0.0021	0.0787	0.0066	11:15 AM	0.0066	0.2526	0.0211	5:15 PM	0.0026	0.0982	0.0082	11:15 PM	0.0015	0.0581	0.0048
5:20 AM	0.0021	0.0787	0.0066	11:20 AM	0.0066	0.2526	0.0211	5:20 PM	0.0026	0.0982	0.0082	11:20 PM	0.0015	0.0581	0.0048
5:25 AM	0.0021	0.0787	0.0066	11:25 AM	0.0066	0.2526	0.0211	5:25 PM	0.0026	0.0982	0.0082	11:25 PM	0.0015	0.0581	0.0048
5:30 AM	0.0020	0.0775	0.0065	11:30 AM	0.0083	0.3148	0.0262	5:30 PM	0.0025	0.0969	0.0081	11:30 PM	0.0016	0.0594	0.0050
5:35 AM	0.0020	0.0775	0.0065	11:35 AM	0.0083	0.3148	0.0262	5:35 PM	0.0025	0.0969	0.0081	11:35 PM	0.0016	0.0594	0.0050
5:40 AM	0.0020	0.0775	0.0065	11:40 AM	0.0083	0.3148	0.0262	5:40 PM	0.0025	0.0969	0.0081	11:40 PM	0.0016	0.0594	0.0050
5:45 AM	0.0021	0.0787	0.0066	11:45 AM	0.0086	0.3298	0.0275	5:45 PM	0.0026	0.0982	0.0082	11:45 PM	0.0015	0.0581	0.0048
5:50 AM	0.0021	0.0787	0.0066	11:50 AM	0.0086	0.3298	0.0275	5:50 PM	0.0026	0.0982	0.0082	11:50 PM	0.0015	0.0581	0.0048
5:55 AM	0.0021	0.0787	0.0066	11:55 AM	0.0086	0.3298	0.0275	5:55 PM	0.0026	0.0982	0.0082	11:55 PM	0.0015	0.0581	0.0048

Basin Runoff and Loss Parameters

As part of a watershed study program completed in 1994, Alameda County developed site-specific equations to be used in conjunction with the Snyder unit hydrograph (SUH). The SUH is balanced as described above; however, Alameda County-specific equations are used to translate that hydrograph to runoff hydrographs. MOUSE includes the modified Alameda County SUH method which is used for this SDMP analysis. The site specific equations include those used to determine basin lag time, initial and constant loss rates, and peaking factor.

Basin lag, or lag time, is defined as the time elapsed between rain fall occurring within a basin and runoff occurring at an outlet point. The Alameda County Hydrology Manual has adopted the following equation for determining basin lag:

$$Lag = K \cdot N \left[\frac{L \cdot Lc}{\sqrt{S}} \right]^{0.38}$$

where Lag is lag time in hours, L is the length of the basin's longest watercourse in miles, K is a unitless factor which is a function of L, Lc is the length along the basin's longest water course measured from the outlet to a point opposite the watershed area's centroid in miles, S is the average stream slope (ft/mile), and N is a basin roughness factor. It should be noted that the basin roughness factor is <u>not</u> the same as Manning's roughness coefficient (*n*). For urban watersheds, the relationship between Manning's *n*-value and the basin N-factor is:

$$N = 0.3318\, n^{0.6328}.$$

The peaking factor is calculated using:

$$Cp = 0.6\, e^{[0.06\,(So/A)]}$$

where Cp is the peaking factor, S_o is the basin slope in percent, and A is the drainage basin area in square miles, with a minimum value of 5 square miles. The minimum peaking factor, assigned for basins with a slope of less than 5%, is 0.6. Although the County Hydrology Manual provides a generalized map of basin slopes, Schaaf & Wheeler has calculated basin slope using the LiDAR data and found that a total of twelve basins had slopes greater than 5%, with a peak slope of about 8% calculated.

The MOUSE Alameda County SUH method calculates both the peaking factor and the lag time internally – L, Lc, S, N, A and S_o are parameters the only input directly into the model.

Direct runoff is estimated by subtracting soil infiltration and other losses from the rate of rainfall. The method described in the County Hydrology Manual for estimating losses is used for this analysis. The method assumes that an initial amount of rainfall is absorbed by tree cover, stored in depressions, and infiltrates soil before any direct overland runoff will occur. This initial loss is given in the County Hydrology Manual for any storm greater than a 5-year recurrence interval as 1.0 inches for a 24-hour storm event.

Uniform loss, which accounts for constant infiltration of rainfall into the soil, is a function of both soil type and ground cover (i.e. vegetation type or land use). The County methodology uses the uniform loss rate to account for the various potential land uses and soil types within a basin. As described previously, all of the soils within the City have been classified as Type D. ACPWA has found that there is less infiltration for new urban coverage compared to established urban landscaping. As such, the County has established constant loss values for three categories of soil coverage:

- Rural Coverage: Consisting of all rural areas with undisturbed soil cover and natural rural vegetation growth (uniform loss for soil D is 0.05 inches per hour),
- New Urban Coverage: Consisting of pervious areas of newly developed urban areas with less than 5 years vegetation growth (eg. lawn, golf course, landscape areas, uniform loss for soil D is 0.07 inches per hour); and
- Existing Urban Coverage: Consisting of areas of existing urban development with more than 5 years of vegetation growth (uniform loss for soil D is 0.09 inches per hour).

The overwhelming majority of the City soil coverage is Existing Urban Coverage. The exception to this is the Bay Point development area in the Northside sub-area, which is assigned New Urban Coverage. Much of Alameda County is highly urbanized, and the above loss factors account for this urbanization. The County manual also includes an equation to adjust the initial uniform loss rate for land uses with a high percentage of non-directly connecting impervious areas, which is calculated using the following equation:

$$\overline{\mathrm{L}} = \mathrm{L}(1\text{-}0.8A_i)$$

where L(bar) is the adjusted loss rate, L is the given loss rate described above, and A_i is the decimal fraction of non-directly connected impervious area. This equation is rarely necessary, but is used in some sections of Alameda where zoning suggest lot sizes at least 5,000 square feet in size. The initial and constant loss rates calculated for use in this SDMP are presented in Table 3-2.

Table 3-2: Land Use, Initial Loss and Constant Loss Values for Alameda

Zoning Land Use Designation	Initial Loss, Inches	Constant Loss, Inches / Hour
Existing Urban Coverage	1.0	0.09
Older Residential (Lots at least 5,000 SF)	0.79	0.07

DRAINAGE SYSTEM ANALYSES

Detailed analyses of peak stormwater discharge are performed with the MOUSE program, which also determines the flow condition in each drainage system element. The MOUSE technical manual is should be referenced for a more detailed description.

Intersection Culverts

The many intersection culverts throughout the City were not modeled in MOUSE. The purpose of these culverts is to keep intersections from ponding during storm events. Extending the system to blocks without closed conduit system was studied.

Closed conduits

Pipes are modeled as one-dimensional closed conduit links which connect two nodes in the model. The conduit link is described by a constant cross-section along its length, constant bottom slope, and straight alignment. The unsteady flow in closed conduits is calculated using conservation of continuity and momentum equations, distinguishing between pipes flowing partially full (free surface flow), and those flowing full (pressurized flow). MOUSE deals with pressurized flow conditions by introducing a fictitious slot in the top of the conduit cross section, essentially replacing the closed conduit with an open channel. The cross section of the slot is shaped so that flow in the channel will approximate the hydraulic behavior of the pressurized pipe. All pipes within the Alameda model are modeled as reinforced concrete pipe (n = 0.012).

Storage Facilities

Throughout Alameda some storm drain collection systems terminate in a storage facility (i.e. lagoon) where runoff is pumped into waters surrounding the City, or metered out to downstream conveyance facilities which eventually outlet to the Bay/Canals around the City. MOUSE models storage areas according to the volume of the basin. The model requires a basin bottom elevation and a spilling water surface elevation. Between the two elevation constraints, multiple basin surface areas and corresponding elevations are used by the model to create a basin storage volume. These characteristics are entered into a node representing the basin which is connected to the piping network with at least one upstream link and downstream link. Existing City parks were not studied as storage facilities; however, these open areas may provide potential storm water storage benefits.

Pumping Facilities

Pumps are modeled in MOUSE as a functional relation between the water level of two nodes. Pumps are characterized by starting and stopping water levels, an offset, and a capacity curve of differential head vs. flow data for the pump.

Outlet Boundary Conditions

Pipe network outlets require a water surface elevation for modeling any effects due to receiving water levels. In areas that outlet to a lagoon, the initial water surface elevation was assigned based on input from Public Works operations and maintenance personnel. Lagoon tide gates and pumps are operated manually to preserve storage during the winter, and the models attempt to capture that actual operation.

Where storm drain collection systems discharge to uncontrolled receiving waters such as San Francisco Bay, the Oakland or Alameda Harbors, or the San Leandro Channel, variable tide elevations provide the boundary condition. A *19-year mean tide cycle* is established for San Francisco Bay and other geographical locations on the West Coast. This cycle represents average tide heights over a specific period known as the tidal epoch, which spans the 19 years it takes for every possible combination of relative positions for the sun, moon and earth to occur. A mixed tide cycle predominates on the West Coast of the United States. This cycle consists of two high tides (one higher than the other) and two low tides (one lower than the other) each lunar day.

Based on calculations for these relative celestial positions, it is possible to predict tides for any day of the year at any time of the day. *Astronomic tides*, created by the gravitational forces of the moon and sun acting on earth's oceans, are provided in tide prediction calendars. The mean tide cycle is simply the long-term average of astronomic tides. *Observed tides*, on the other hand, are actual tidal elevations recorded by National Oceanic and Atmospheric Administration (NOAA) gaging stations located throughout coastal areas. Table 3-3 provides the extreme points of the 19-year metonic cycle for the current tidal epoch (1983-2001) and the relevant datum conversion, which is to subtract 2.92 feet from the MLLW datum to obtain tide elevations on the NGVD29 datum.

Table 3-3: Tide Cycle at Alameda Naval Air Station (9414750)

Tide	**19-year Mean (MLLW)**	**19-year Mean (NGVD29)**
Higher High (MHHW)	6.60	3.68
High (MHW)	5.97	3.05
Mean Sea Level (MSL)	3.45	0.53
NGVD29 Datum	2.92	0.00
Low (MLW)	1.13	-1.79
Lower Low (MLLW)	0.00	-2.92

Traditionally, Mean Higher High Water (MHHW) has been used as the backwater condition where riverine (freshwater) runoff meets an estuarine (saltwater) body. However, evidence shows that mean tide elevations are not an appropriate boundary condition during storm events and tide elevations in San Francisco Bay are elevated (relative to predicted tides) during periods of heavy rainfall. Furthermore, the relationship between coincident tides and maximum annual runoff can be quantified and used in the model, providing for a more statistically correct solution than an arbitrarily selected tide condition.

The El Niño storm of February 2-3, 1998 provided an ideal event for examining potential correlations between runoff events and tide action. While stream runoff as measured by local gages often approached historic recorded levels, observed tides in San Francisco Bay were substantially higher than predicted. Figure 3-3 shows predicted and recorded tides in early February 1998 at NOAA's Alameda NAS. Recorded tides during the week of this runoff event were consistently higher (on the order of 2 feet) than the astronomic (predicted) tide heights due to storm surge. (As a control, observed tide heights are compared to predicted tides six months later at the same station, using the same sets of data. Figure 3-4 shows tide elevations during early August 1998, when there is very close agreement between the predicted and the actual tides and no rainfall. Both figures present tides on the local Mean Lower Low Water (MLLW) datum.

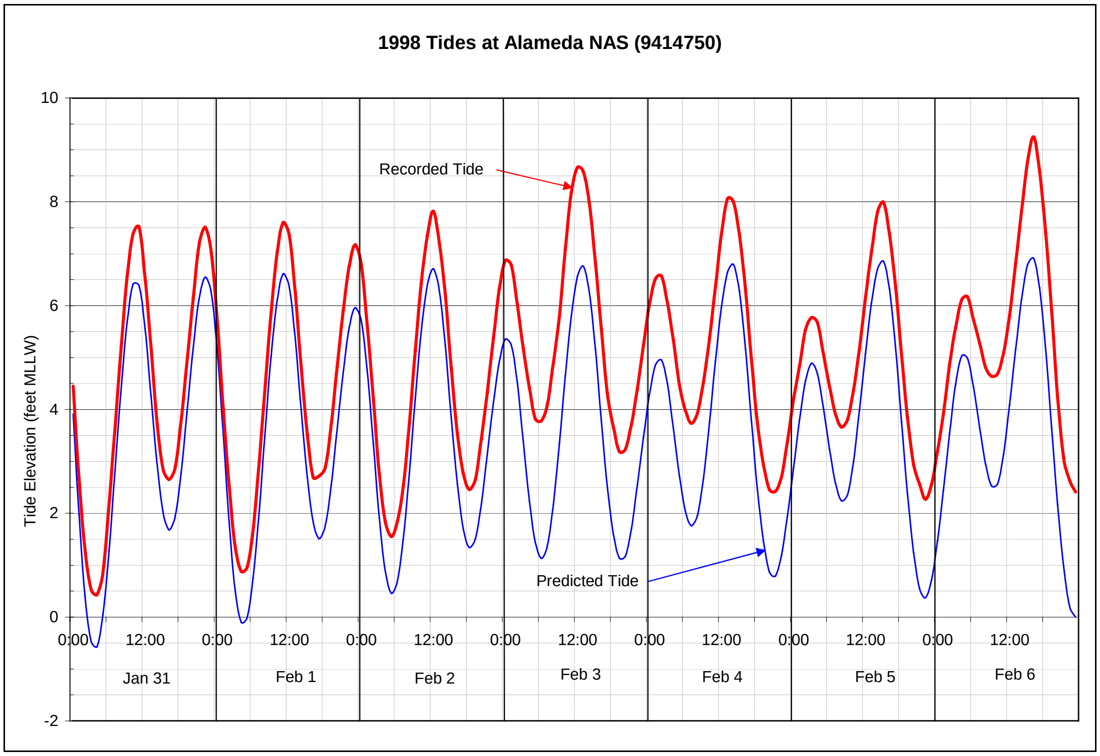


Figure 3-3: Impact of Storm Surge on San Francisco Bay Tides

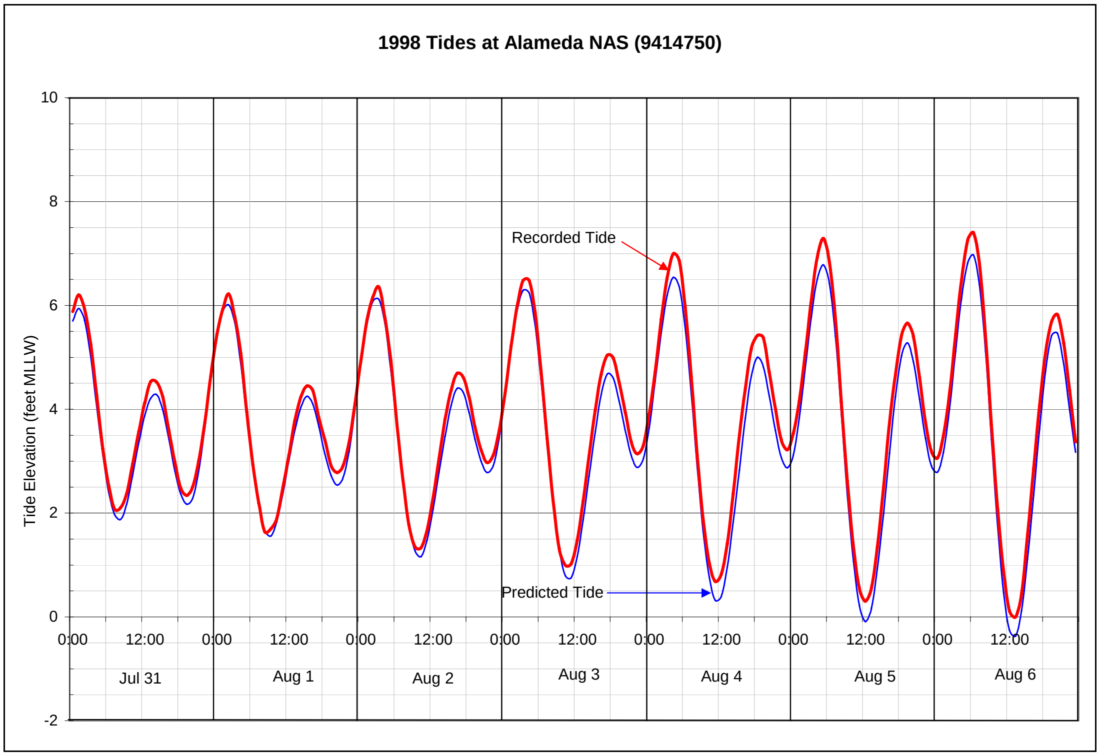


Figure 3-4: Lack of Storm Surge Effect during Summer Months

Historic tide records have been examined to see whether the phenomenon demonstrated in February 1998 at Alameda occurred elsewhere in the Bay Area and during other heavy runoff events in the past. Results of this investigation presented in Table 3-4 indicate that during the 1998 runoff event, similar rises in tide elevations (over astronomic) were experienced at other recording tide stations in the Bay.

Table 3-4: Storm Surge During February 1998 Event

Location	Maximum Difference Between Predicted and Recorded Tides in feet	
	Higher High	Lower Low
Golden Gate	2.0	2.9
Alameda	2.0	2.7
Redwood City	2.0	2.7
Monterey Harbor	1.7	1.8

The observed phenomenon presented in Table 3-4 is not strongly dependent upon tide gage location, particularly within San Francisco Bay, and is exhibited during many historic storm events. Data indicate that higher tides as observed during the February 1998 event are not an isolated incident; rather, higher than predicted tides can be expected during storm events that generate significant runoff. Increases in the data set between observed tides over predicted tides range from 0.3 foot to 2.0 feet for the higher high tide, and from 0.9 foot to 3.0 feet for the lower low tide.

From observed historical data, it appears that storm-related forces induce higher tides during rainfall events, and by extension, runoff events. This phenomenon may be due to a number of meteorological or hydrologic factors. NOAA refers to the term “inverse barometer effect”, and defines it as higher tides that are caused by lower barometric pressures associated with winter storm systems. References to “storm surges”, the meteorological effects of low barometric pressures and/or strong southerly winds, are also found in the literature.

The exact nature and cause of this phenomenon, however, are not as important as potential impacts to backwater conditions for Alameda storm drains. Desired system reliability governs the selection of an appropriate tidal cycle for storm drain system analysis. To model an appropriate San Francisco Bay tidal cycle during a storm event of particular return period, elevations for each critical point in the tide cycle are adjusted based on the one-percent conditional probability of coincident occurrence with the annual maximum discharge of Dry Creek at Union City, which represents the closest USGS streamflow gaging location with sufficient length of record for analysis. This procedure is as described by Dixon (1986), whose hypothesis was that high tide events tend to occur the same day as flood flow events using conditional probability:

$$P(x,y) = P(x|y)\ P(y)$$

where $P(x,y)$ is the probability of occurrence of x and y; $P(x|y)$ is the probability of occurrence of x given y; $P(y)$ is the probability of occurrence of y; x is tide elevation; and y is maximum annual peak discharge. Since we are interested only in annual maximum discharges, $P(y)$ is one and the probability of joint occurrence, $P(x,y)$, is equal to the probability of x given y.

Tide cycle points (Lower Low, Low, High, and Higher High) are taken from fitted probability curves using the median plotting position for every recorded tide extreme that occurred within 24 hours of the recorded maximum annual discharge. Figure 3-5 shows each probability distribution, Table 3-5 provides the values, and Figure 3-6 shows the 10-year and 25-year coincident tide cycles used in modeling along with selected Alameda tide cycles and values of interest.

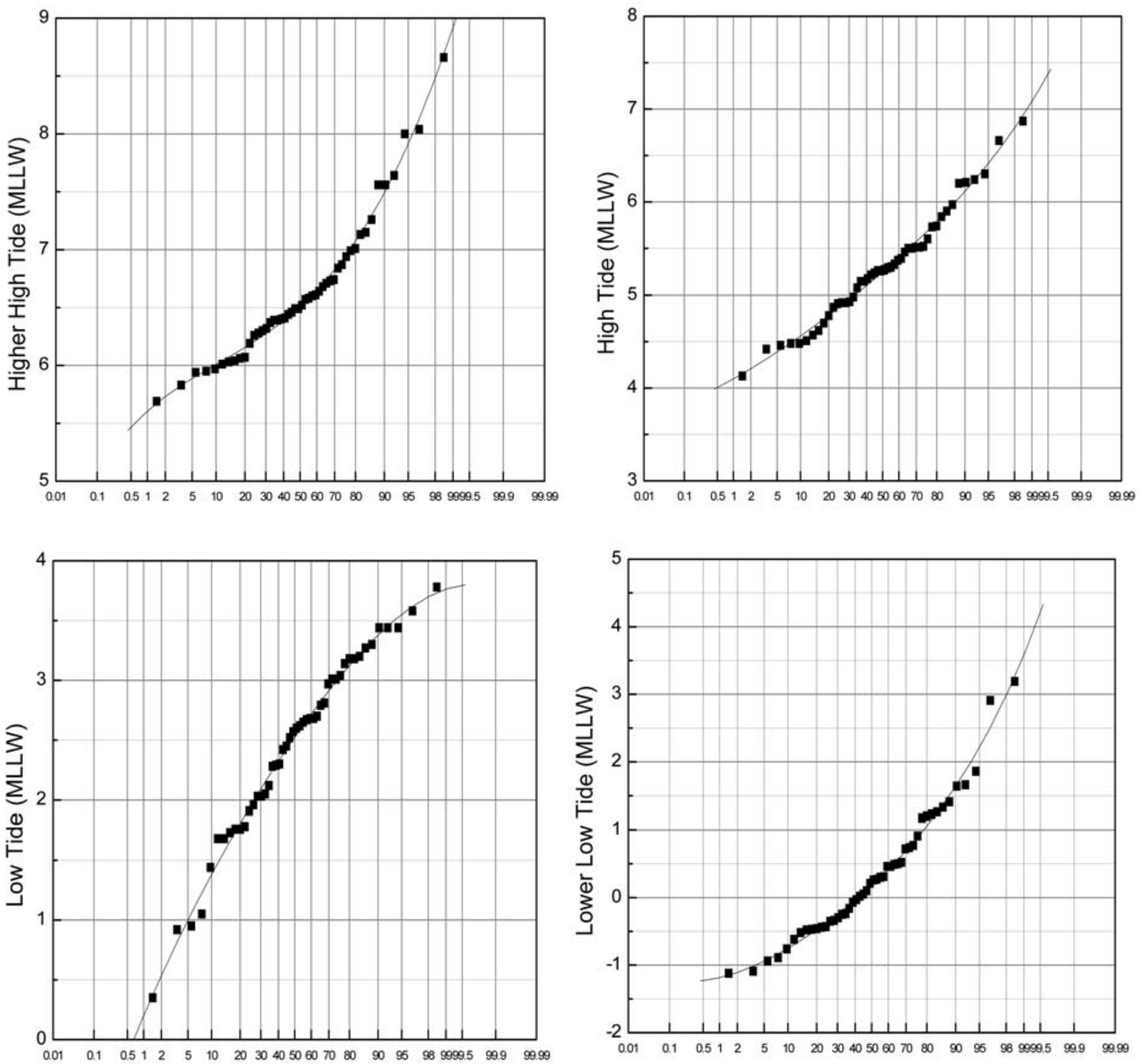


Figure 3-5: Annual Maxima of Tide Cycles Coincident with Peak Annual Runoff

Table 3-5: San Francisco Bay Boundary Conditions

Tide	**19-year Mean (feet NGVD)**	**10-year Coincident (feet NGVD)**	**25-year Coincident (feet NGVD)**	**100-year Coincident (feet NGVD)**
Higher High	3.68	4.56	5.13	6.00
High	3.05	3.18	3.62	4.16
Low	-1.79	0.44	0.68	0.88
Lower Low	-2.92	-1.29	-0.47	0.68

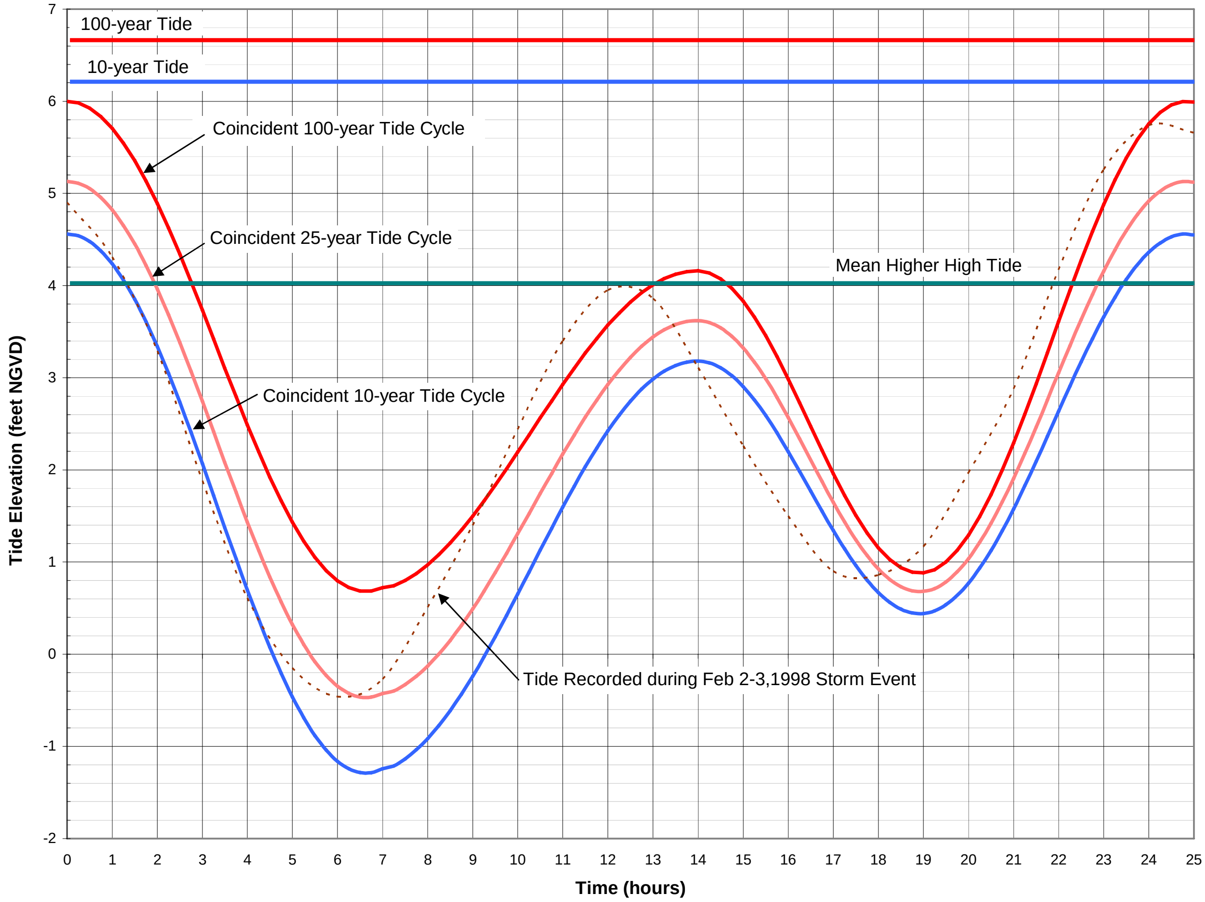
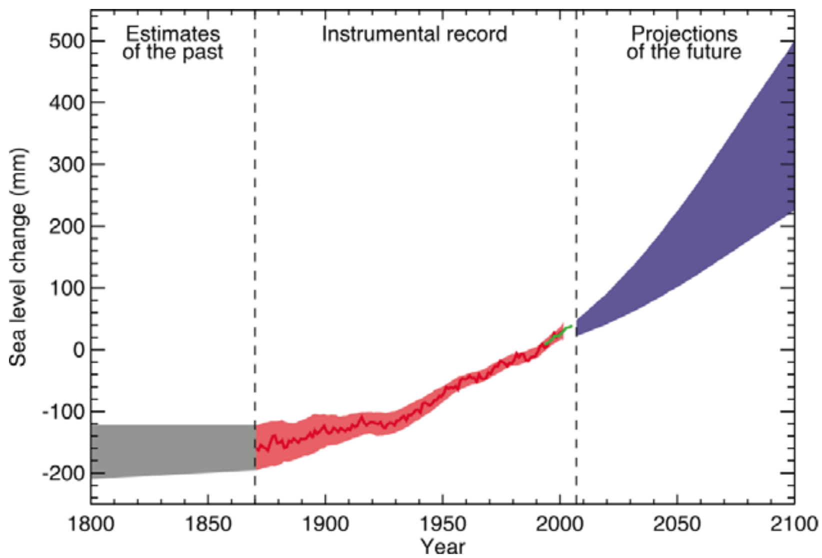


Figure 3-6: Design Boundary Conditions for Storm Drain Analysis

The timing of coincident tide elevations with peak rainfall/runoff is also a random process. Since there are not sufficient data to statistically analyze the impact of tide timing, a sensitivity analysis has been conducted to assess the impact of the scenario wherein the peak of local runoff roughly coincides with the peak tide (i.e. stage) at the collection system outfall. This analysis showed that this coincident peak scenario results in flooding almost identical to any other randomly selected tide /runoff timing relationship and storm drain system performance is not particularly sensitive to San Francisco Bay tides.

Sea Level Rise

Global temperatures have increased by about 1° F over the past century, and sea level has risen by approximately 0.5 foot.[1] An historic rate of sea level rise of 1.3 mm per year (0.4 foot per century, has been estimated for San Francisco.[2] Although quantitative consensus regarding future sea level rise is difficult to obtain, most credible scientific organizations agree that sea level will most likely continue to rise, perhaps at an accelerated rate. Figure 3-7 shows a range of potential future sea levels based on IPCC climate change scenarios.[3]



FAQ 5.1, Figure 1

Figure 3-7: Projections of Future Sea Level Rise

A 50-year planning horizon is used for the Alameda Storm Drain Master Plan to be consistent with Bay Conservation and Development Commission (BCDC) practices.[4] The mid-range projection of sea level change by 2058 from Figure 3-8 is approximately 160mm, or about six inches.

The performance of Alameda's storm drainage system after the completion of recommended improvements has been examined with an additional 0.5 foot added to the 10- and 100-year coincident tidal boundary conditions described herein. The system's performance is not found to be adversely impacted by this projection of future sea level rise.

[1] Intergovernmental Panel on Climate Change (IPPC), 1996.
[2] National Oceanographic and Atmospheric Administration (NOAA), 2001.
[3] IPPC AR4, WG1.
[4] ASCE San Francisco Section Symposium on Climate Change and Coastal Systems, September 28, 2007.

CHAPTER 4
DRAINAGE STANDARDS

The City of Alameda has established guidelines for improvement recommendations and new systems. Criteria used throughout the Master Plan to evaluate how well individual storm drainage systems are functioning, and how best to improve that function, are expanded from storm drain criteria in the most current edition of Associated General Contractors of California Joint Cooperative Committee's (APWA-AGC) "Standard Specifications for Public Works Construction". Other guidance is provided by the *City of Alameda's Standard Details and Specifications*.

NEW SYSTEM DESIGN

Any proposed storm drainage system should be designed in conformance with the following standards:

With 25-year Design Discharge	Hydraulic grade shall be no higher than 0.5 feet above the gutter elevation at any manhole or inlet such that the maximum hydraulic grade is the top of curb elevation.

Parts of Alameda's existing collection system do not strictly meet these criteria; so when new systems are tied into existing systems, it may not be possible to provide a design that meets the desired standard. The design and evaluation of new systems, particularly extensions of existing systems, must be done on a case-by-case basis and these exceptions to the listed criteria for new systems are suggested where new collection systems discharge to existing systems:

With 10-year Design Discharge	Pipes shall be sized to carry the 10-year discharge without surcharging the pipe. When downstream surcharge effects are included, upstream hydraulic grades shall be no higher than the top of curb elevation at any manhole or inlet.
With 25-year Design Discharge	Hydraulic grade shall not exceed the top of curb elevation at any location.

Manholes should be no farther than 500 feet apart, and catch basins are to be spaced so that the maximum width of gutter flow does not exceed eight feet from the face of curb during a ten-year design storm; or 600 feet, whichever is less.

Evaluation of Existing Systems

Improvement recommendations are developed with the goal of reducing 10-year flooding to the established standard of a hydraulic grade line no greater than the top-of-curb elevation. The 10-year tide developed for this analysis (Figure 3-6) was used for the boundary condition for pipe outlets, while the developed storm pattern (Figure 3-2) was used to simulate a 10-year rainfall event. A second analysis has been conducted to establish further improvements that might be necessary if this same standard is applied to the 25-year storm event by replacing these boundary conditions with the 25-year tide and storm pattern (see Figure 3-6 and Appendix C). Historic plans and City records were used to set Lagoon water levels for those outlet boundary conditions. For the main island, the Lagoon starts at a water surface elevation of 2.2 feet (NGVD), while the Lagoons on Bay Farm Island have varying started water surface elevations depending on the storm event. These levels are presented in more detail in Chapter 5.

This master plan recognizes that it may not be cost effective to replace facilities simply so that all areas within the City meet standards set for new systems. The goal of all recommended improvements is to meet these criteria; however prioritization of these improvements has been established to balance system performance and public safety against limited capital improvement funds. As such, collection system improvements are prioritized per Table 4-1. In addition to the factors described in Table 4, for Alameda Island the duration of flooding at each node was also calculated and used to establish improvement priories. This map is included in Appendix D.

It should be noted that in some isolated areas retrofitting the existing storm drain network to reach these standards may not be feasible. In that case, improvements were given low priority levels due to their infeasibility. In a few limited areas there are no feasible improvements that will entirely remove flooding. These areas are described in more detail in Chapters 5 and 7.

Table 4-1
Storm System Improvement Priorities

High Priority	Projects under this category have a large area of flooding where the 10-year flow depth in the street is more than one foot over the top-of-curb. These projects improve locations with the deepest and longest flooding situations in each of the five sections of the City. These projects may also be located at the downstream end of many projects, as they would logically be constructed first. Areas of significant historical flooding fall into this category.
Moderate Priority	This category has conditions similar to high priority, but has a smaller area affected by flooding. A 10-year design discharge still overtops the top-of-curb; however, the length and depth of flooding is less than that of a high priority improvement.
Low Priority	Low priority improvements are generally smaller projects that consist of placing a few pipe segments. Existing flooding is not necessarily contained within the roadway (top-of-curb); however, the area of flooding is much smaller and/or briefer in duration than that of moderate and high priority projects.

Outfalls

For the purposes of this SDMP, it is assumed that all outfalls are free of debris and vegetation, and are subject to a tide sequence as described previously in Chapter 3. It is further assumed that no outfalls, with the exception of those tied directly to a lagoon or pump station, are fitted with functioning flap gates. Any newly constructed outfalls should not be fitted with flap gates per City maintenance staff recommendations.

STORAGE FACILITIES

There are two basic categories of stormwater storage: detention and retention. Some facilities in fact blur the distinction, but detention generally refers to the temporary storage of incoming runoff that exceeds the permissible release. After the storm event, the facility empties and returns to its natural function; such as a water feature, parking lot, rooftop, or park. Retention facilities, on the other hand, hold on to the excess runoff for an indefinite period. Most storage facilities in Alameda are lagoons which serve a dual role for both stormwater detention and retention. For instance, pumps or weir structures are used to move attenuated flood waves through the facility, but a pool of water remains behind for aesthetic (or perhaps recreational) purposes. Parks within the City were not modeled as storage basins.

Design Reliability

Properly designed, constructed, and maintained, stormwater storage facilities can reduce peak flows, thereby better utilizing the capacity of downstream conveyance facilities. Such facilities can also potentially mitigate the need for system upgrades. Although large scale storage facilities within Alameda are unlikely given space restraints, some onsite storage (for stormwater quantity and/or quality applications) may be an aspect of new or re-development within the City. The efficacy of any detention facility, as well as ancillary improvements in the quality of storm runoff to receiving waters, needs to be evaluated on a case-by-case basis. However, some general design criteria should be applied to every basin:

1. Basins should be sized so that their output does not exceed the design capacity of downstream facilities.

2. There must be an emergency overflow section capable of safely discharging the 100-year peak inflow (should outlet works become clogged), without causing property damage.

3. At least one foot of freeboard over the maximum 100-year water surface elevation should be provided for excavated basins. Three feet of freeboard (minimum) must be provided where basins are created by berms or levees.

4. Infiltration capacity shall not be considered when designing basins, unless percolation rates are determined by on-site soils testing certified by a Civil or Geotechnical Engineer.

5. Debris and sediment loading must be considered in design (see below).

6. Open facilities need to be designed with shallow side slopes (3:1 minimum) so that people

and animals may extricate themselves from the water should the need arise. A safety shelf may also be considered. Facilities that pose an inordinate risk to the public should be fenced off. Inlet and outlet openings larger than six inches in diameter must be screened to protect children and animals.

7. A mechanism for draining the basin should be provided. If the basin also serves as a pumping forebay, the pumping facilities must be capable of fully dewatering the basin. Vehicle access to the basin should also be provided.

8. Facilities designed for the permanent (or semi-permanent) retention of water should be deep enough to avoid eutrophication and breeding insects. Pond surface areas should be at least one-half acre, with a minimum depth of ten feet over at least a quarter of the area. The average depth over the rest of the pond needs to be at least five feet. Basin outlets should be positioned opposite from the inlet to promote circulation. Stocking permanent ponds with fish also promotes good water quality. Drainage facilities must comply with the ACCWP Regional Water Quality Board Permit, Division of Dam Safety, and Mosquito Abatement.

9. Underdrain systems to minimize wetness should be considered for detention facilities not intended as permanent water features. This helps to prevent the facility from encouraging insect populations, and also provides for a quicker return to its dry weather function.

10. Basin bottoms and sides should be stabilized with vegetation to withstand periodic flooding and prevent erosion. Basin outlets and inlets (i.e. storm drain outfalls draining to the basin) need to be provided with scour and erosion protection such as riprap.

Debris & Sediment Loading

Detention and retention basins may eventually fill up with sediment and other debris, reducing their storage capacity to the point where they will not operate as designed. Therefore, some consideration of debris loading should be made for each basin. Based on work by Schaaf & Wheeler for the Santa Clara Valley Water District, the following empirical relationships are provided as a guideline (debris load per unit drainage area) for use to evaluate debris loading:

Highly urban areas	0.1acre-foot/mi^2/year
Open space areas	0.4 acre-foot/mi^2/year

Depending upon the desired frequency of maintenance, some allowance for dead storage should be made to handle sediment and debris using the loading rates given above. Basin sizing should meet ACCWP and City of Alameda design guidelines for stormwater quality detention and retention basins.

PUMPING FACILITIES

Without a safe gravity release for runoff, stormwater pumping facilities shall be designed to discharge the one-percent (100-year) design flow without endangering property. Associated storage facilities may be used to meet this criterion. Chapter 6 provides additional general pump station design and operating guidelines.

Reliability

Pump stations shall be designed to provide reliable, automatic service. Provisions must be made in facility design to promote the maintenance of pumping equipment and mechanical appurtenances (Chapter 8). The City should provide pursue redundant standby pumps for stormwater facilities.

Standby Power

Currently throughout Alameda the primary source of pump power is electric motors, and it is expected that any new pump stations constructed in the future will also be electric powered. Provisions for generating power for these motors during PG&E service outages shall be provided. The manual transfer of power to emergency generators is only acceptable if the pump station is configured so maintenance crews can safely connect a portable generator power plug to the switchgear. Otherwise, and for critical installations, a standby generator (or generators) shall be permanently installed on-site, capable of starting the largest pump motor with all other motors and ancillary demand already under load.

Stations with permanent generators shall be provided with automatic transfer switches that sense the loss of PG&E power, switch pump station control to the engine-generator, sense normal phase balance from the power utility, and provide a time-delayed retransfer to normal utility power. Provisions to maintain continuous power to all control, alarm, and telemetry systems through battery backup or other means shall also be made.

In the event that a new pump station proposes gas power, diesel is the fuel of choice due to its non flammability, availability, and ease of transportation. Natural gas engines may be considered with City approval, but natural gas is susceptible to interruption during earthquakes or other disasters. Propane and gasoline engines shall not be used. Solar and wind power cannot produce enough energy to operate the large pump motors; however, they may provide power for lighting and communications.

Tailwater Conditions

Pumps shall be designed for peak discharge to receiving waters assuming a one-percent (100-year) coincident tailwater (tide).

CHAPTER 5
STORM DRAIN COLLECTION SYSTEMS

Analyzing Alameda's storm drain collection system performance forms the essential core of this master plan. For each sub-basin area, this chapter describes major storm drain facilities, any historic problem areas, pumping or storage facilities (if applicable), and other known flood hazards. Within each basin, areas requiring system improvements are identified and prioritized. For the purposes of conciseness and readability, this Chapter presents only the 10-Year MOUSE predicted flooding depths and those projects required to alleviate or minimize flooding based on the 10-Year standard previously described in Chapter 4. The City of Alameda is interested in also understanding what projects would be required to apply this same standard to the 25-year storm event and resulting flooding. Those results and recommended improvements have been summarized in Appendix C.

Node-labeling within the model match the names in the files received from the City of Alameda. Pipes, culverts, and other system components can be identified by the nodes which they link. Conversations and meetings with City staff as well as past reports, most notably the *1998 Storm Drainage Facilities Rehabilitation and Repair Report* form the basis of the 'Historic Problem Areas' sections of this chapter.

EVALUATION OF STORM DRAIN CAPACITY

Criteria

Each collection system has been analyzed for existing land use based on the City Zoning Map (updated as part of the 1991 General Plan) to determine its runoff condition during the design ten-year storm. As described previously, future land use changes within the City are not expected to worsen flooding conditions, as the existing land uses which are slated for development are currently industrial or transportation/commercial based. Areas of significant flooding are recognized herein and necessary improvements to restore system performance in accordance with criteria outlined in Chapter 4 are proscribed.

Additional flow capacity requirements are determined by upsizing existing pipes in the MOUSE model until flooding is reduced to acceptable levels, increasing the capacity of existing pump stations, or some combination thereof. It is impossible to entirely remove predicted flooding throughout the City, either due to local topography (for example, at low 'bathtub' areas), or infeasibility of improvements, but the majority of model-predicted flooding can be mitigated to the previously described criteria with the capital improvements proposed herein.

In order to identify areas where the storm drain lines should be extended, street capacities immediately upstream of the storm drain lines were calculated and compared to the flow delivered to the manhole representing the upstream limit of the storm drain line. If the street upstream of this point could not convey the flow while allowing for a 10 foot wide dry emergency access corridor on the crown of the road, a new storm drain was recommended.

Prioritizing Deficiencies and Needed Improvements

Alameda's storm drain system is broken into four drainage areas for both Alameda and Bay Farm Islands, forming a total of eight drainage sub-areas. Each sub-area contains some combination of pipes, pumps, culverts, outlets and lagoons. These facilities all eventually discharge into the waters surrounding Alameda and Bay Farm Islands. Figure 2-2 delineates these major drainage areas. The basins are organized around natural topographic boundaries (i.e. the ridge in the middle of Alameda Island) and drainage facility boundaries or watersheds. It should be noted that neither private drainage systems nor site-specific drainage characteristics have been analyzed. Recently installed storm drain systems may have been designed to site-specific drainage characteristics established by the developer and/or City staff. These systems are not analyzed in detail, but are generally prioritized to low priority. Future refinement of the model could more precisely account for these site-specific drainage characteristics and more accurately represent the local drainage conditions.

Each basin analysis contains a schematic representation of the local stormwater collection systems, showing problem areas and recommended master plan improvements. The following color code is used to highlight project prioritization within each drainage area:

Red	High Priority
Yellow	Moderate Priority
Green	Low Priority

This section outlines the ultimate improvements needed to achieve the stated level of service criteria by alleviating or minimizing predicted flooding within each of the eight sub-areas. Each improvement was grouped with nearby improvements that would be undertaken simultaneously and named using a street within the improved system. This naming convention is used to identify the improvements in maps and tables. A complete CIP with figures depicting storm drain network improvement pipes including pipe location, size requirements and costs for each improvement is available in Chapter 7.

ALAMEDA ISLAND SYSTEMS

Eastside

Overview

The Alameda Island Eastside drainage area is approximately 0.7 square mile, and is bounded by water on the eastern half and by the North Central and South drainage sub-areas on the western half. The trunk lines of the Eastside collection system consist of 160 nodes, 14 outlets and one pump station. The Eastside area has a total (including lateral lines) of 26,000 linear feet (4.9 miles) of connecting storm drain pipes equal or greater than one foot in diameter. In general, the Eastside area drains eastward, with almost a third of the storm drains leading to the Central/Eastshore pump station.

Historic Problem Areas

According to the City of Alameda there have been historical flooding problems at a particular property on Fernside Blvd (between Briggs and Encinal Avenues) due to debris build-up at the storm drain outlet. Cleaning the outlet seems to have fixed flooding at this location. Additionally, historic flooding has occurred at the Johnson Avenue at both the Mound and Court Street intersections due to undersized culverts. Central Avenue at the intersections with Grove, Mount, Court, Fountain and High Streets have been identified as having flooding problems due to either undersized culverts and/or tree roots impacting culvert or gutter alignment.

Identified Deficiencies

MOUSE analysis of the Eastside systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 79 of the 160 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 21 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 15 of nodes, with the remaining 43 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-1. In addition to these improvements, additional capacity at the Central/Eastshore pump station is also recommended.

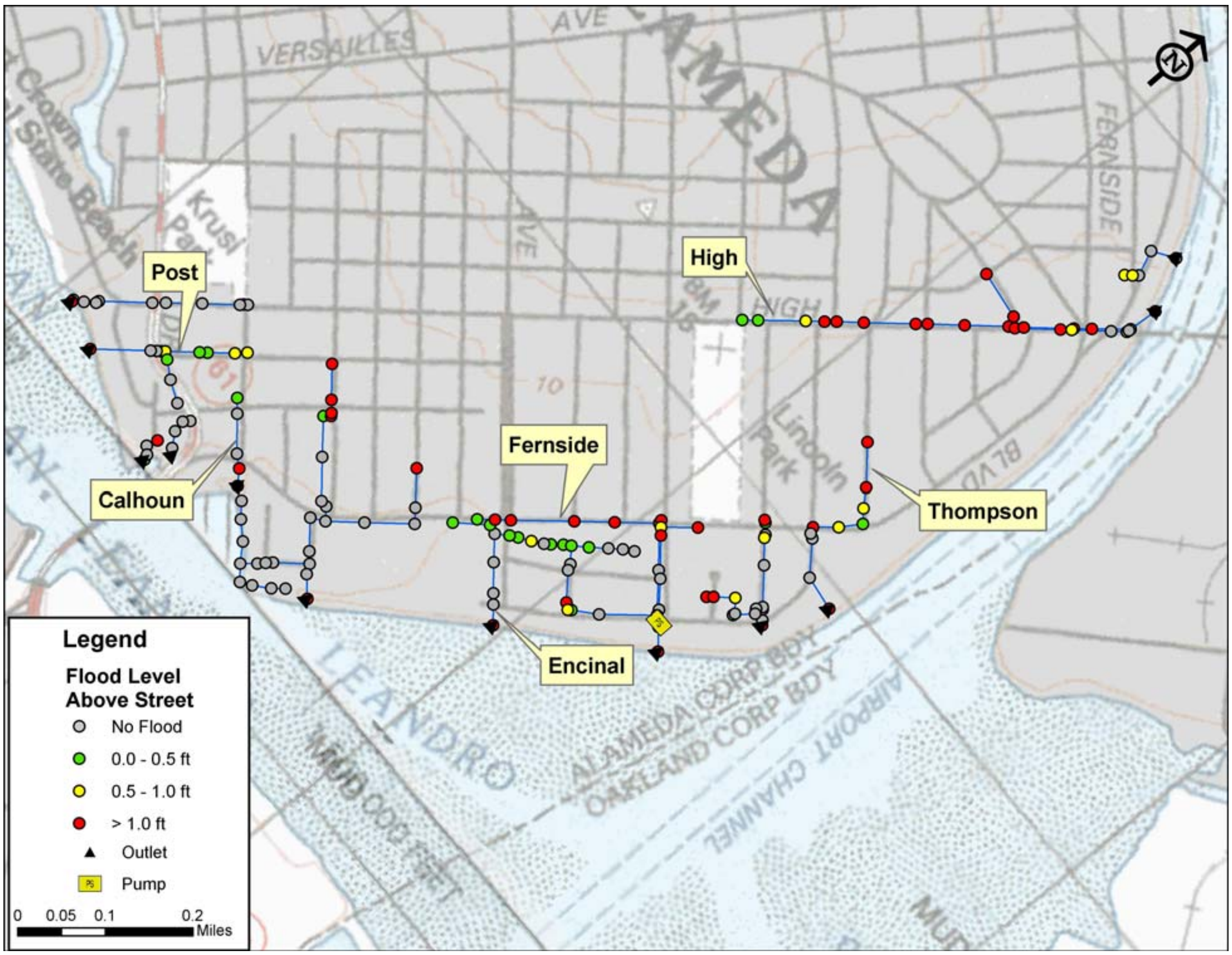


Figure 5-1: Alameda Eastside Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Alameda Eastside area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-2, which include storm drainage piping capacity improvements, new storm drains, and pump station capacity improvements.

There are some locations in the western parts of the Eastside area where the topography creates a 'bathtub' affect, requiring large sized pipes to reduce water surfaces below these low lying areas. This affect has an impact on the Washington, Post, and Calhoun improvements. High Street experiences significant flooding during the modeled 10-year event requiring a new pipe along Gibbons Street was to intercept some of the flow before it reaches High Street. Flooding along High Street is affected by tide and backwater conditions, and the City has not reported significant flooding in this area. Increasing the capacity of the Central/Eastshore pump station is required to mitigate the flooding throughout the network that drains to the pump station, and is a moderate priority improvement.

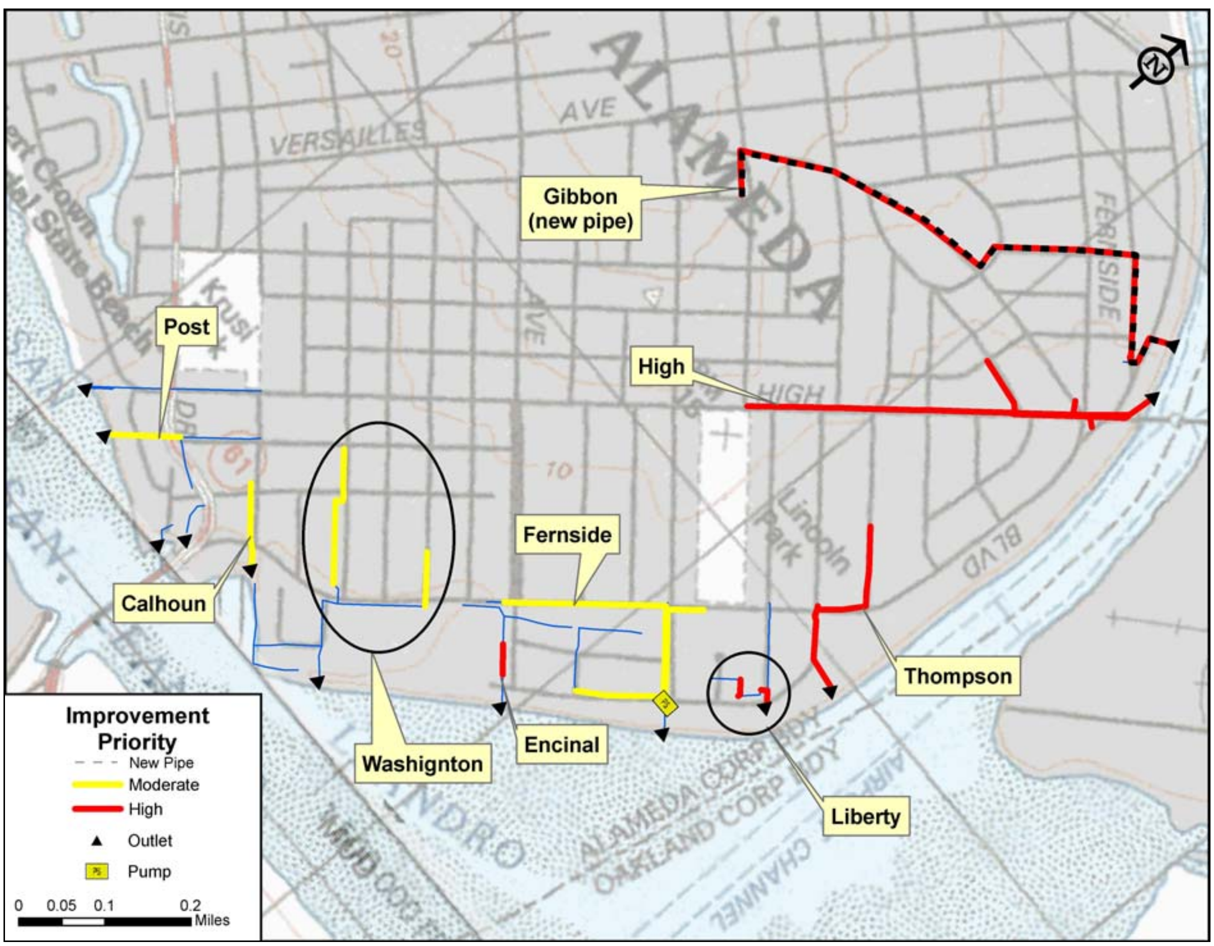


Figure 5-2: Alameda Eastside Area Prioritized 10-Year Improvements

North Central

Overview

The Alameda Island North Central drainage area is approximately 1.1 square miles, and is bounded by the Oakland Canal to the northeast and by Northside, South, and Eastside drainage areas to the northwest, southwest and south east respectively. The trunk lines of the North Central collection system consist of 159 nodes and 11 outlets. The North Central area has a total (including lateral lines) of 32,500 linear feet (6.2 miles) of connecting storm drain pipes equal or greater than one foot in diameter. The North Central area is the simplest drainage network on Alameda Island, with no lagoons or pump stations.

Historic Problem Areas

Within the North Central area, Central Avenue at the intersections with Pearl Street and Versailles Avenue have been identified as having flooding problems due to either undersized culverts and/or tree roots impacting culvert or gutter alignment. A culvert runs along Oak Street to discharge to the Oakland Canal. Based on the 1998 Storm Drain Facilities report, this culvert may be crushed where it passes beneath the abandoned railroad tracks near Blanding Avenue, and seasonally heavy vegetation where this culvert discharges to the Canal may obstruct flows.

Identified Deficiencies

MOUSE analysis of the North Central systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 106 of the 159 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 41 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 14 nodes, with the remaining 51 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-3.

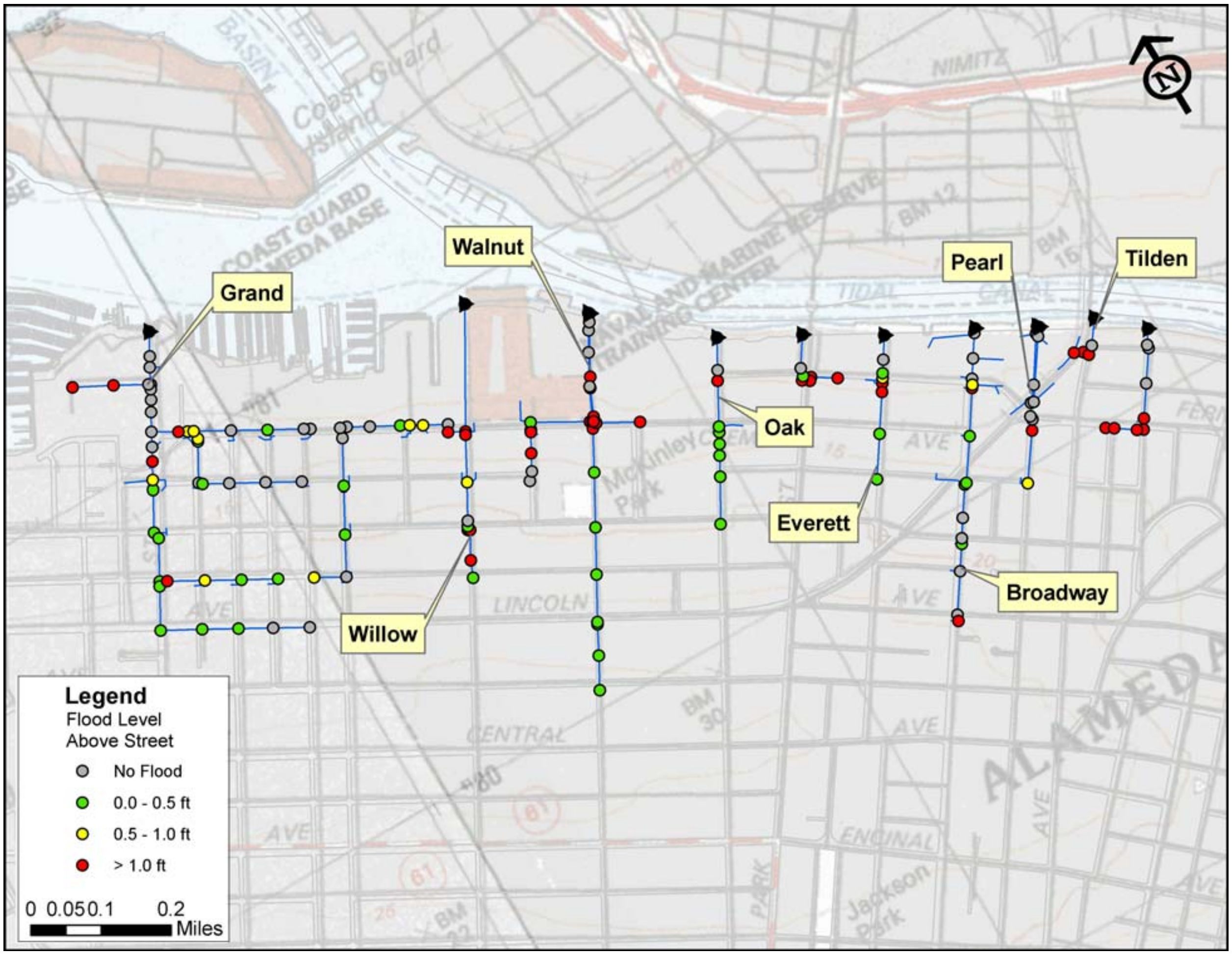


Figure 5-3: Alameda North Central Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Alameda North Central area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-4, which is limited to storm drainage pipe capacity improvements.

The North Central area experiences the least severe flooding during a 10-year event, and although significant projects are required to bring to system to a 10-year standard, none of these projects are categorized as a high priority.

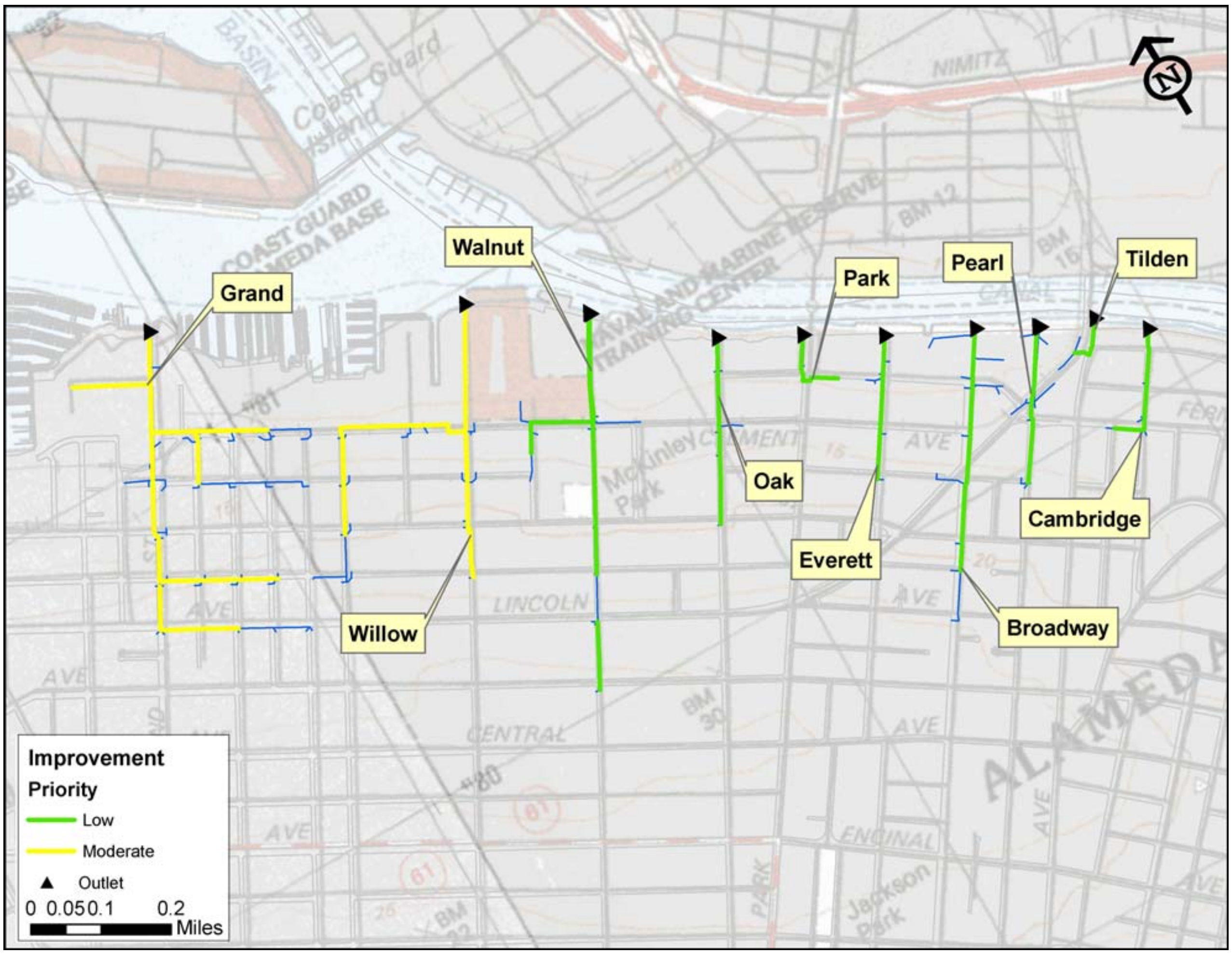


Figure 5-4: Alameda North Central Area Prioritized 10-Year Improvements

Northside

Overview

The Alameda Island Northside drainage area the City's largest (approximately 2.3 square miles) and most complex. It is bounded by Oakland Inner Harbor and Canal to the north, the North Central area and south area to the east and south, and the Naval Air Station to the west. The trunk lines of the North West collection system consists of 563 nodes, 12 outlets and six pump stations. The North West area has a total (including lateral lines) of 131,500 linear feet (24.9 miles) of connecting storm drain pipes equal to or greater than one foot in diameter.

The most notable features of the Northside storm drain network are the large diameter pipes which run along Atlantic and Constitution and drain to the Northside (Marina Village) and Arbor Pump stations. A low flow weir diversion structure at the intersection of Constitution Way and Atlantic Avenue regulates low flow (less than one foot of depth in the pipe) between these pump stations. Because the island is quit flat, it is difficult to reduce the hydraulic grade line (i.e. the water surface elevation) far from the pumping stations.

Historic Problem Areas

Historic local flooding has been noted at several locations within the Northside area, including the Santa Clara Avenue intersection with Stanton, Mozart and Shermon Streets. Past flooding has occurred at the southwest and southeast corners of the Eighth Street and Taylor Avenue intersection. Culvert capacity inadequacies have been noted along Sixth Street at the intersections with Taylor and Palace Avenues, and on Haight Ave at Linden Street. Street flooding has occurred over the entire Second and Brush Street intersection, as well as at the Third and Brush Streets intersection. The street area along Mariner Square Drive east of Webster, south of Marina Village Parkway, often floods due to catch basin inlets being higher than the street low points.

Identified Deficiencies

MOUSE analysis of the Northside systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 318 of the 563 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 81 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 65 of nodes, with the remaining 172 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-5.

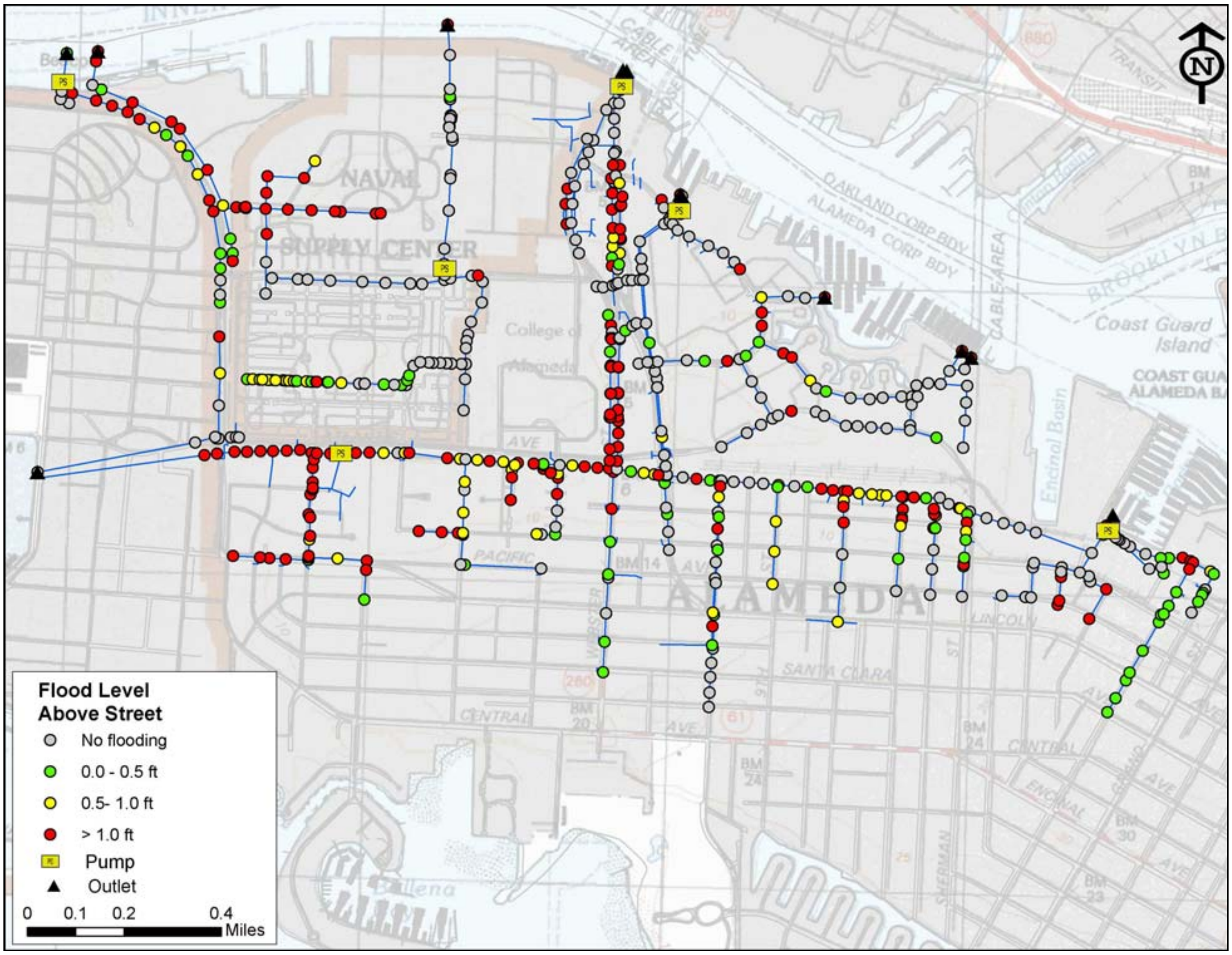


Figure 5-5: Alameda Northside Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Alameda Northside area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-6, which includes pipe and pump station capacity improvements (at both Arbor and Northside (Marina Village) Pump Stations).

A key component of improving this system is a new 72-inch tying into an existing outfall to the San Francisco Bay. This replaced outfall will reduce the demand on the Marina Village Pump Station. The existing system along Ralph Appezzato Parkway should be disconnected near College Avenue; this will prevent reverse flows in the system. Another disconnect in the system should occur along the railroad easement near Chapin Street; this will isolate the area draining to the Arbor Pump Station. These disconnects will allow the system to operate more effectively and will minimize the need for pump station improvements. The system along Singleton Avenue is not clearly shown in the CAD data, and field visits were unable to clarify the drainage situation along this street. At this time no improvements to this system are recommended at this location.

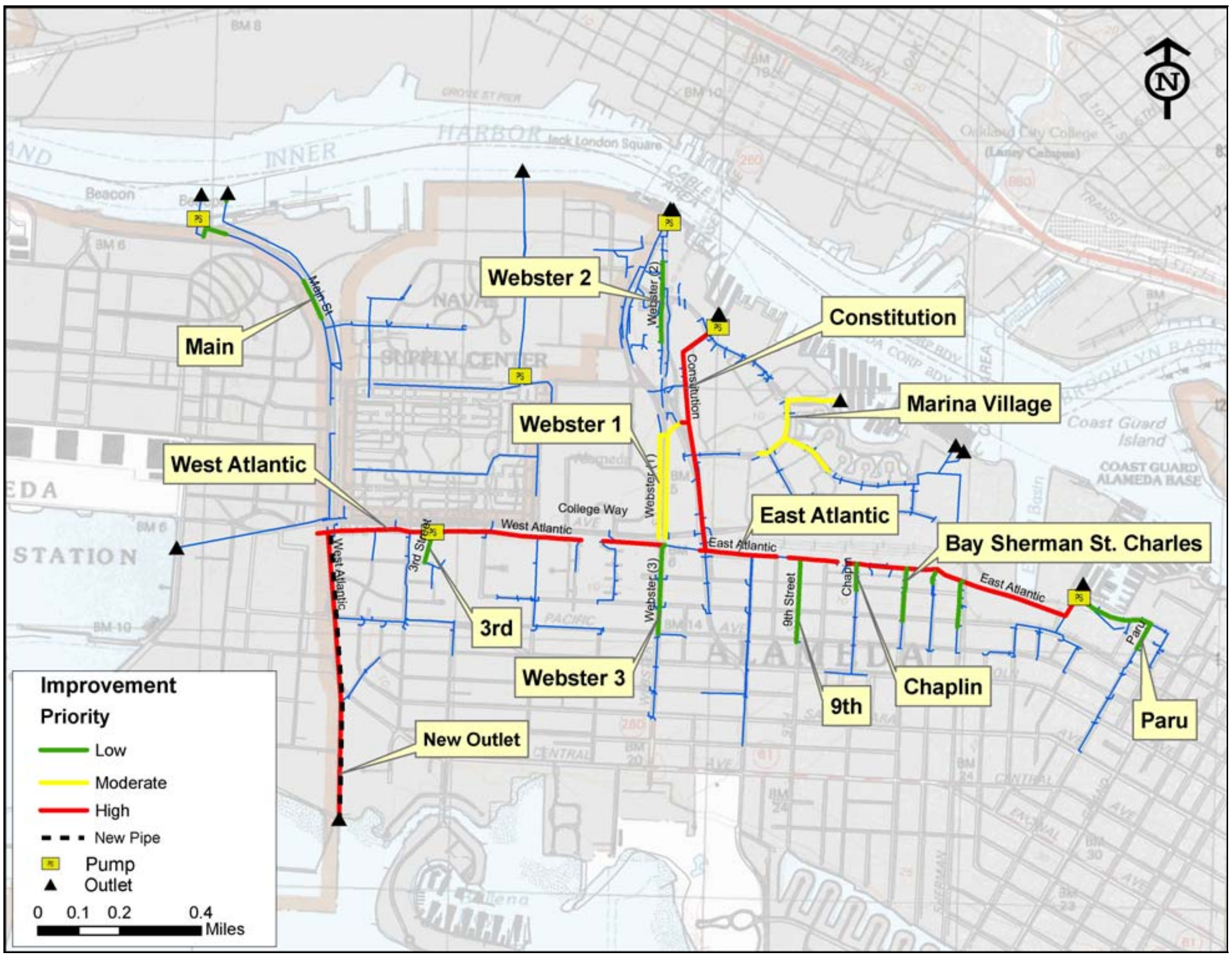


Figure 5-6: Alameda North West Area Prioritized 10-Year Improvements

South

Overview

The Alameda Island South drainage area is approximately 2.3 square miles, and is bounded by the San Francisco Bay to the south and by Eastside, North Central, and Northside drainage areas to the east, north, and north-west, respectively. The trunk lines of the South collection system consist of 322 nodes, 23 outlets and three interconnected lagoon storage areas. The South area has a total (including lateral lines) of 59,400 linear feet (11.2 miles) of connecting storm drain pipes equal or greater than one foot in diameter.

Historic Problem Areas

Past flooding has occurred within the Alameda Island South drainage subarea in several locations. Page Street at Central and Taylor has experienced flooding, as well as nearby Central Avenue between Page and Eighth Streets. Paru Street at its intersection with Clinton Avenue has previously been identified as undersized in the 1997 Storm Drain Rehabilitation Report, and tree roots have impacted the gutter flow along Clinton Avenue in this vicinity. It is not clear how much of the ponding is from tree roots and how much is from the system. Ponding during storm events has occurred at both east and west sides of Delmar Avenue just south of Otis Drive. Court Street at Adams Street has been previously noted for insufficient storm drain capacity.

Identified Deficiencies

MOUSE analysis of the South systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 138 of the 322 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 17 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 23 of nodes, with the remaining 98 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-7.

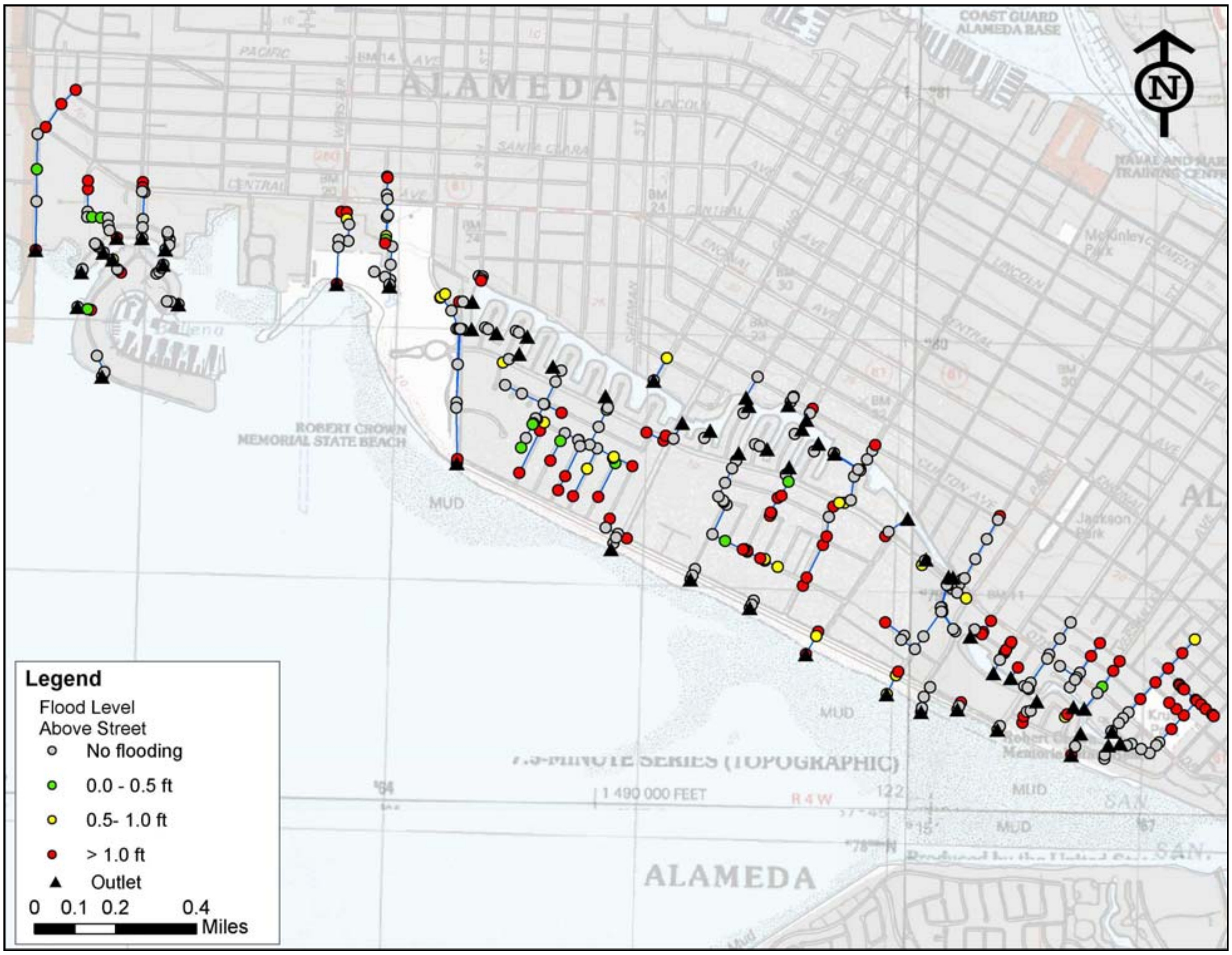


Figure 5-7: Alameda South Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Alameda South area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-8.

Like the Eastside system, the South area has some 'bathtub' areas formed by the topography, which both increases flooding depths and the improvements needed to mitigate those depths. This is the case for both the Fountain and Mound Street improvements. The Lagoon levels in the South area do not have a significant impact on flooding depths and recommended improvements.

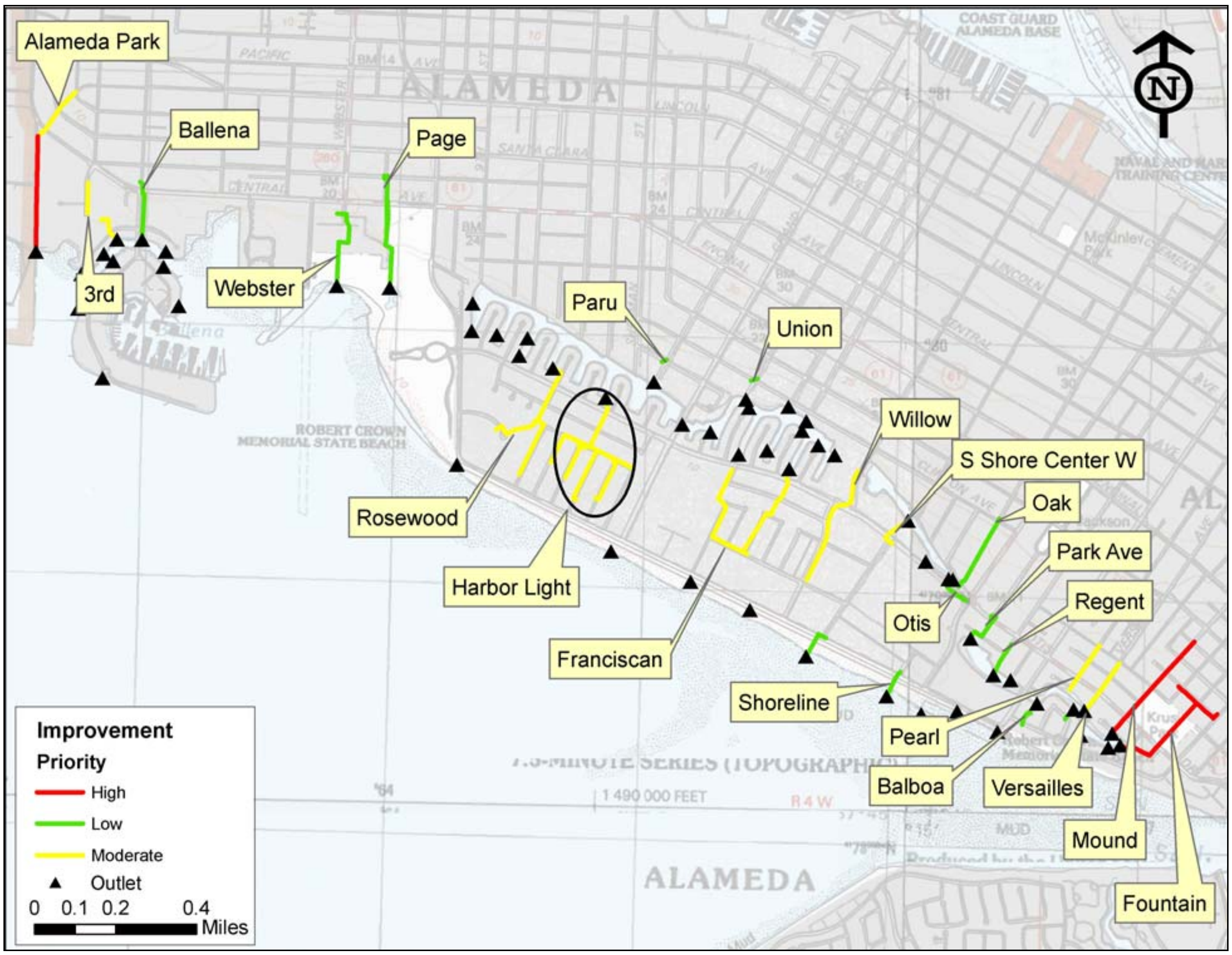


Figure 5-8: Alameda South Area Prioritized 10-Year Improvements

The 'Alameda Park' improvement identified in Figure 5-8 is a high priority improvement to address 10-year flooding in the Alameda South area. An improvement in the Northside Area ('New Outlet') calls for replacing this pipe and outlet to address flooding in the Northside area. The size recommended for the Northside improvement includes consideration of the South flooding, and so, if constructed, should supersede the ‘Alameda Park’ recommended improvement.

Extension of Storm Drain Pipes

The MOUSE model predicts flooding depths at nodes on the storm drain system, but does not include flooding depths along the roads to enter the storm drain system. In order to determine if flooding along roads occurred before water is able to enter the storm drain system, the street capacity upstream of each storm drain trunk line was determined. Street capacity is defined as the flow that would result in a dry path at least eight feet wide at the crown of the road. This definition was developed with City staff input on the width needed for emergency access. Using City standard street cross sections a rating curve comparing street slope to capacity was developed for each standard street cross section. In any location where the flow reaching the most upstream node was greater than the street capacity an extension of the pipe was recommended and the analysis repeated in the upstream direction. Figure 5-9 shows these locations.

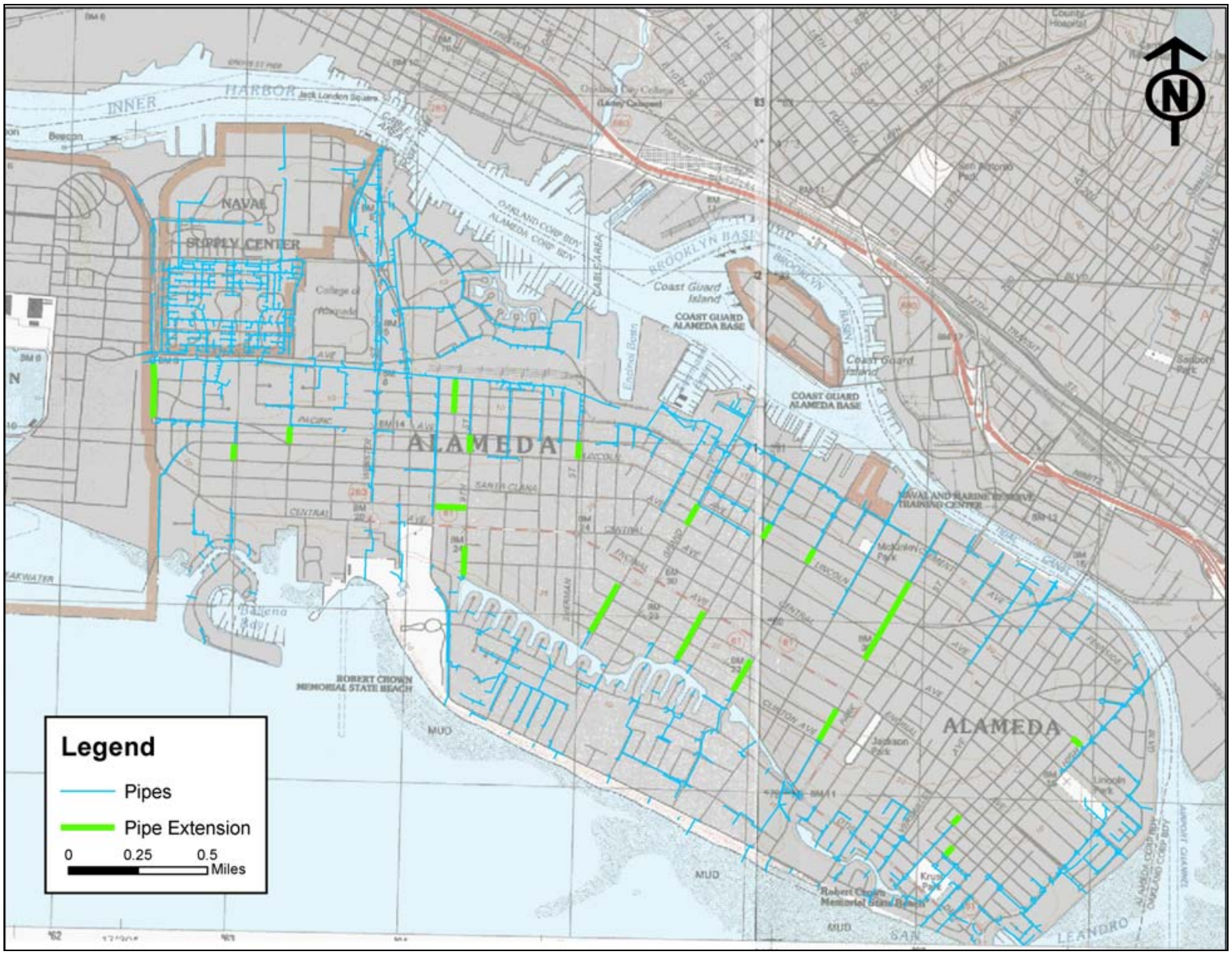


Figure 5-9: Alameda Main Island Recommended Pipe Extensions

With the exception of the Oak Street extension, which will reduce existing flooding at the City Police Station, these extensions are low priority. Because these improvements are not based on modeling results, they have not been included in the cost estimates. In general Schaaf & Wheeler recommends that all of these improvements meet the recommended City standard of a minimum of 18-inches in diameter.

BAY FARM ISLAND SYSTEMS

The majority of Bay Farm Island was not included in the 1998 Storm Drainage Facilities Rehabilitation and Repair Report, which formed the basis for many of the historic problem areas described for the Alameda Island areas. Much of Bay Farm Island is relatively recently developed, and relies heavily on storage both in lagoons and the golf course area. As such, the historic problem areas presented herein are few.

Bay Farm East

Overview

The Bay Farm Island East drainage area is approximately 0.9 square miles, and is bounded by the Oakland International Airport to the east, the San Leandro Channel to the north, and the Bayfront central and south sub-areas to the west and south respectively. The trunk lines of the East sub-area collection system consist of 197 nodes, 2 outlets and an interconnected lagoon storage area and pump station on the golf course. The East area has a total (including lateral lines) of 27,100 linear feet (5.1 miles) of connecting storm drain pipes equal or greater than one foot in diameter.

Historic Problem Areas

There are no known areas of historic flooding in the Bay Farm East area.

Identified Deficiencies

MOUSE analysis of the East systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 82 of the 197 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 29 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 24 of nodes, with the remaining 29 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-10.

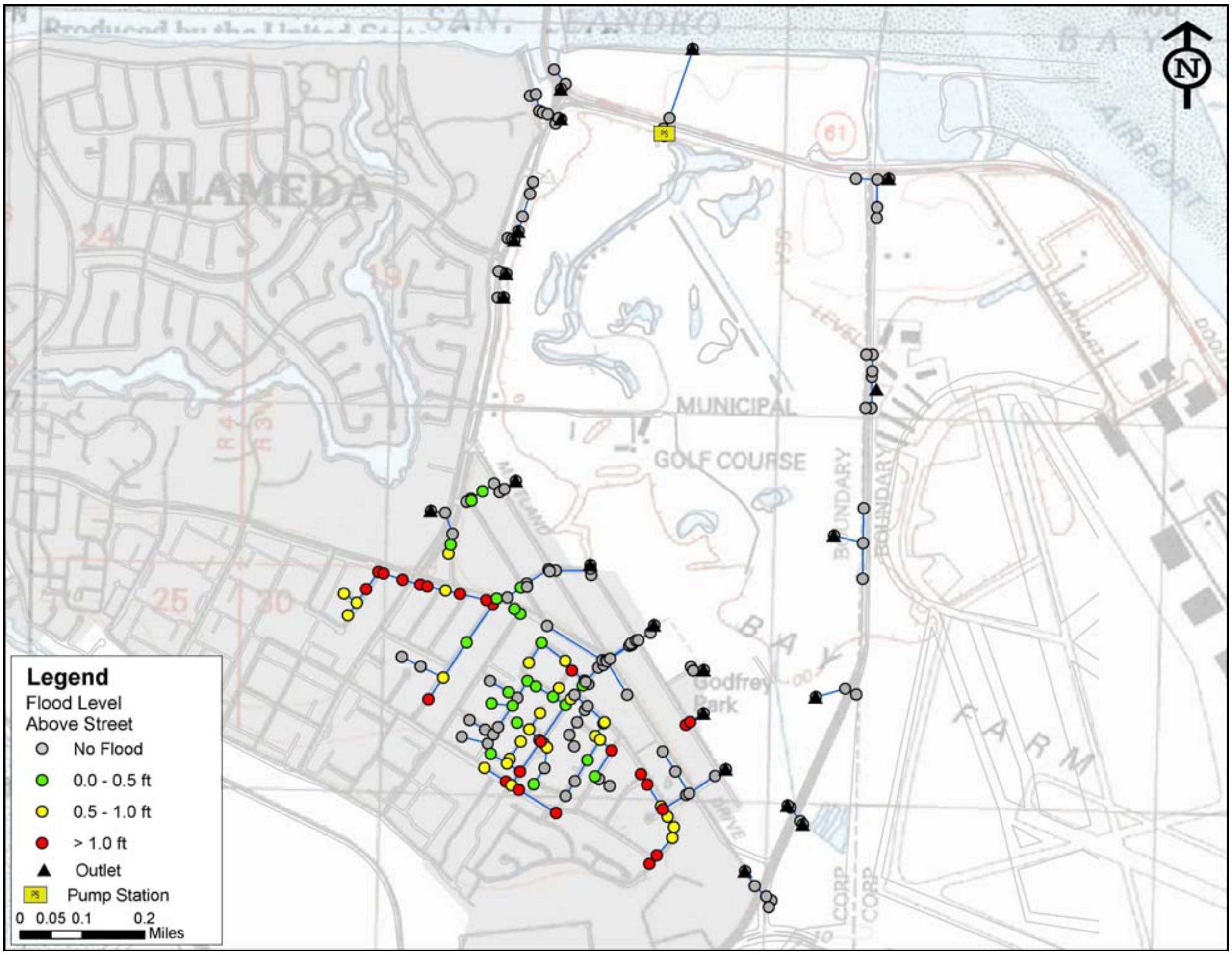


Figure 5-10: Bay Farm East Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Bay Farm East area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-11. In general flooding in the Bay Farm East area is relatively minor, mostly due to the large storage volume provided by the Golf Course. Figure 5-11 does not include the recommended upgrades to the Golf Course pump stations, which is the only high priority improvement in the area.

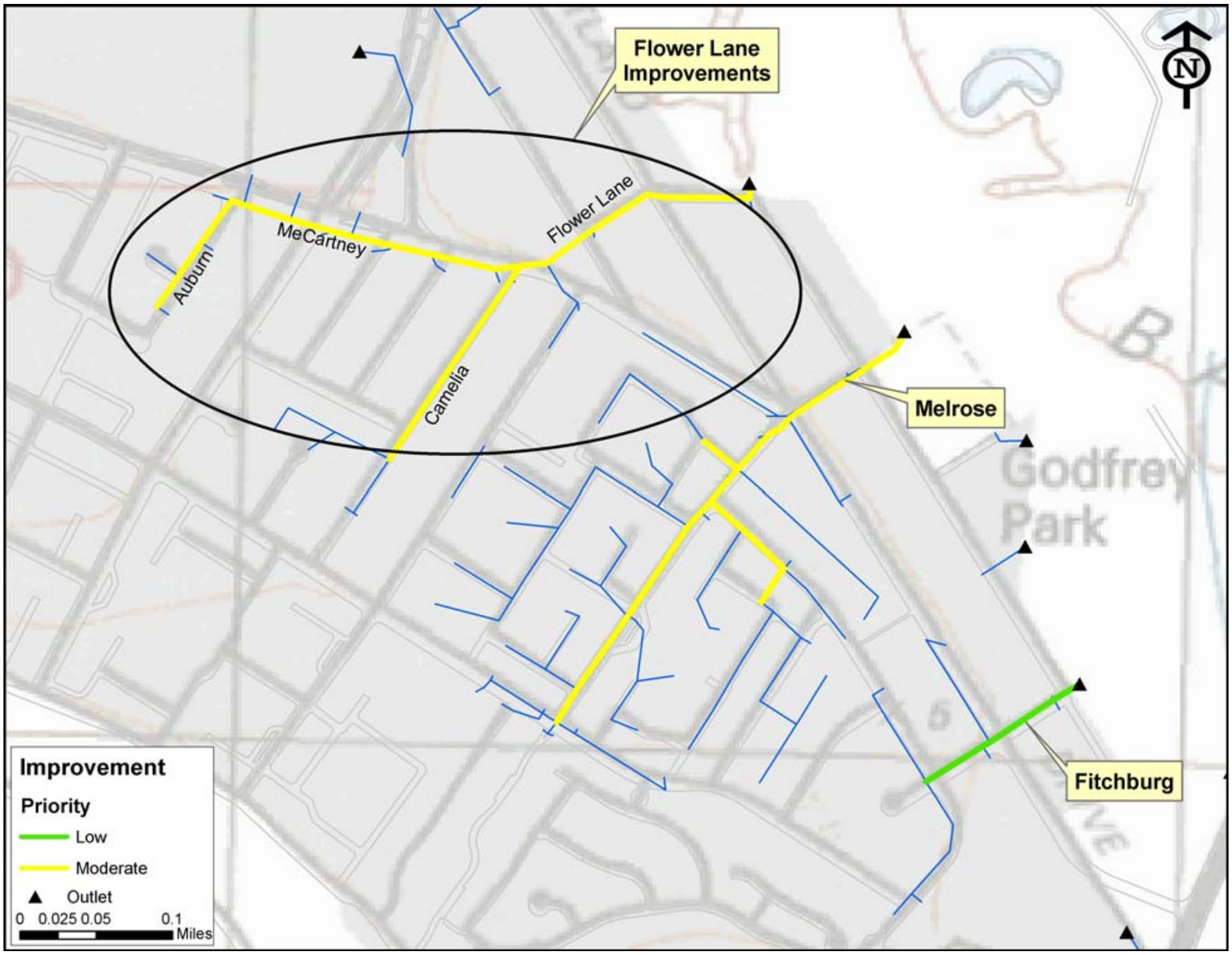


Figure 5-11: Bay Farm East Area Prioritized 10-Year Improvements

Bay Farm North

Overview

The Bay Farm Island North drainage area is approximately 0.38 square miles, and is bounded by the San Leandro Channel to the north, the San Francisco Bay to the west, and Bay Farm Island sub-areas to the south and east. The trunk lines of the North collection system consist of 121 nodes and 4 outlets. The North area has a total (including lateral lines) of 19,400 linear feet (3.7 miles) of connecting storm drain pipes equal to or greater than one foot in diameter.

Historic Problem Areas

There are no known areas of historic flooding in the Bay Farm North area.

Identified Deficiencies

MOUSE analysis of the North systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 41 of the 121 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 15 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 14 of nodes, with the remaining 12 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-12.

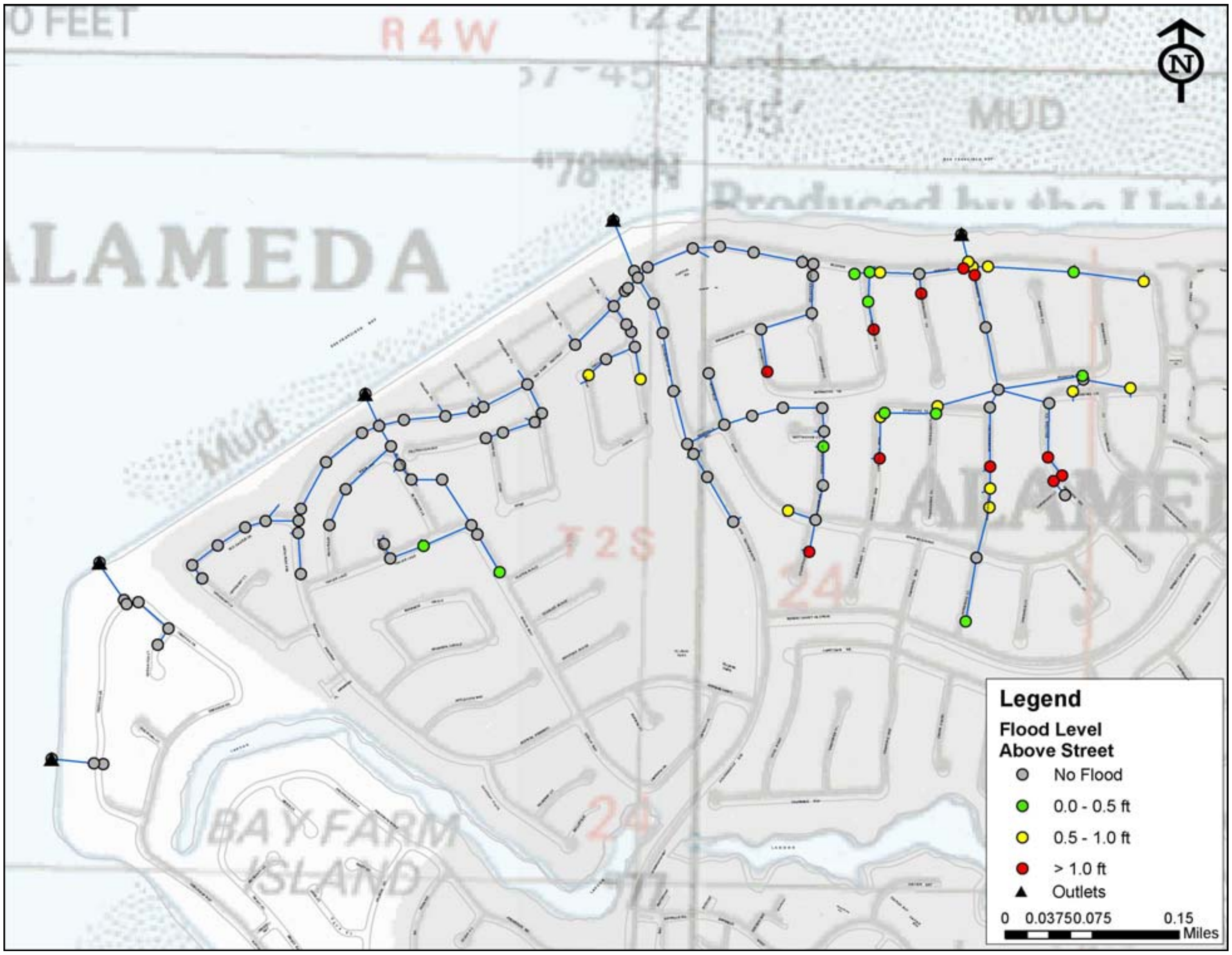


Figure 5-12: Bay Farm North Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Bay Farm East area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-13.

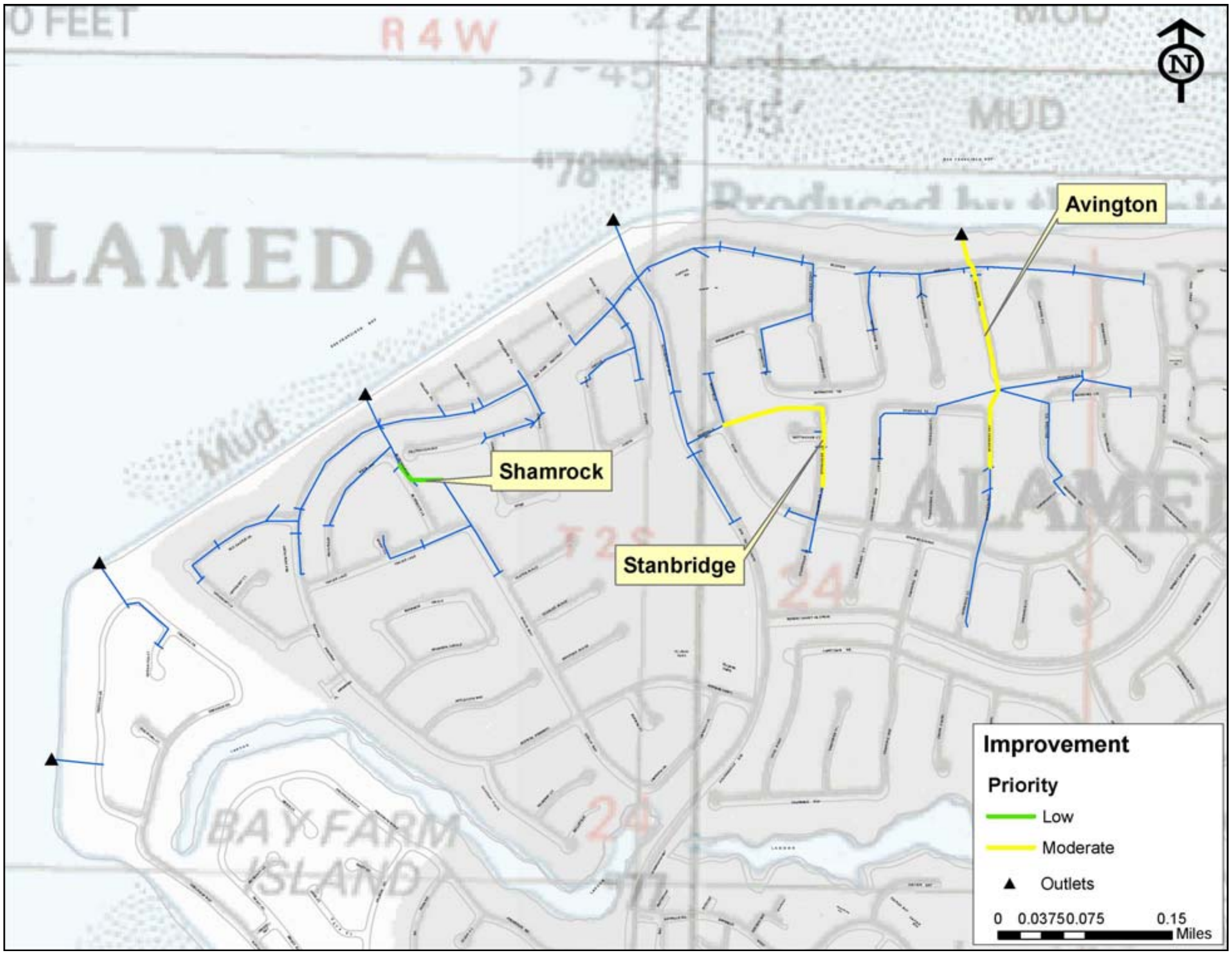


Figure 5-13: Bay Farm North Area Prioritized 10-Year Improvements

Bay Farm Central

Overview

The Bay Farm Island Central drainage area is approximately 0.58 square miles, and is bounded by the San Francisco Bay to the west and by Bay Farm Island sub-areas to the south, east and north. The trunk lines of the Bay Farm Central collection system consist of 422 nodes, 33 outlets and four interconnected lagoon storage areas. The Central area has a total (including lateral lines) of 58,900 linear feet (11.1 miles) of connecting storm drain pipes equal or greater than one foot in diameter. All of the outlets in the Central area discharge to a lagoon except for one. The Lagoons water levels are controlled via the manually operated System I pump station, which is described in more detail in Chapter 6, and is the one non-lagoon outfall in the Central area. Because this pump is manually operated, it was not included in the storm drain model.

Historic Problem Areas

There are no known areas of historic flooding in the Bay Farm Central area.

Identified Deficiencies

MOUSE analysis of the Central systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 126 of the 422 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 43 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 47 of nodes, with the remaining 36 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-14.

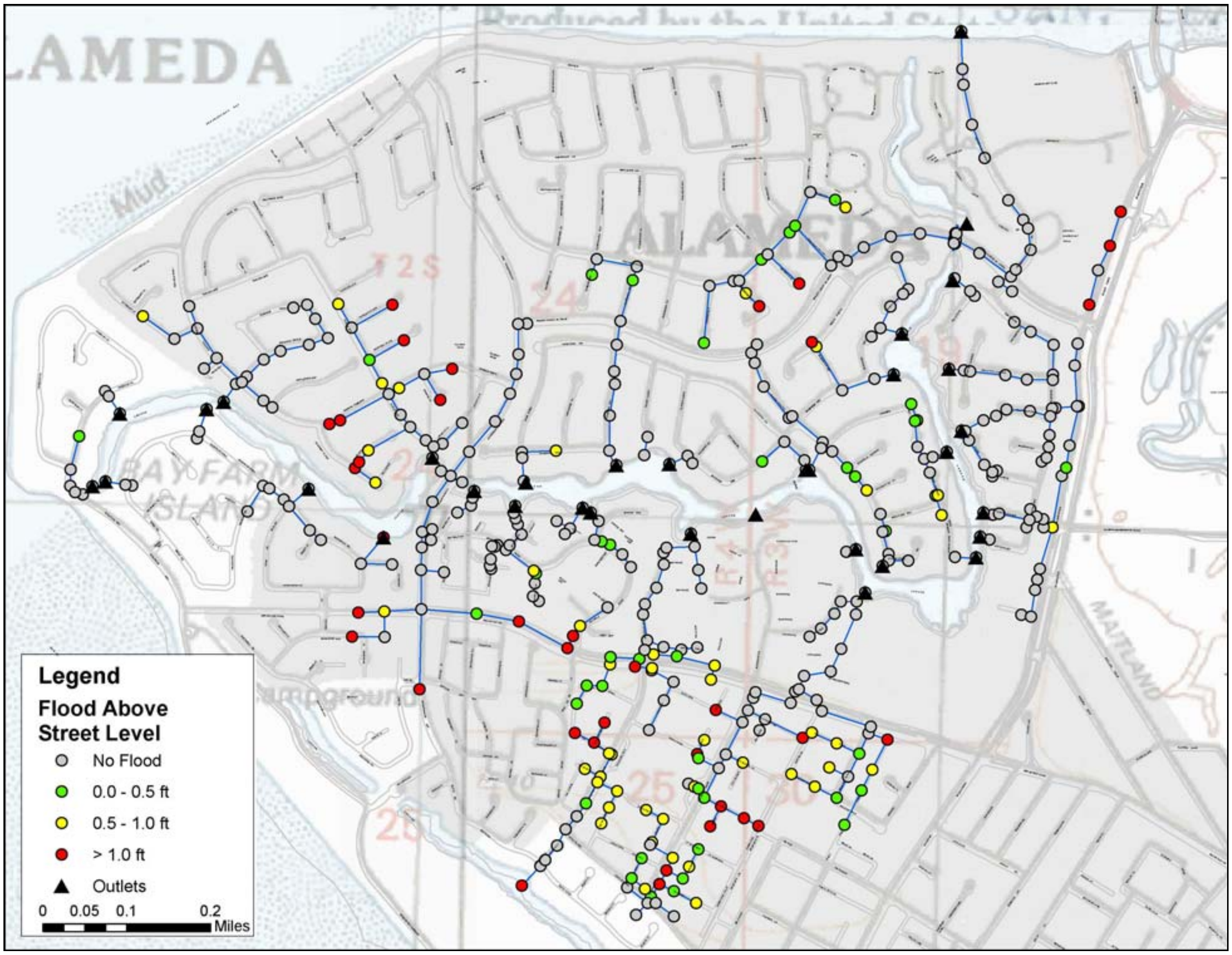


Figure 5-14: Bay Farm Central Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Bay Farm Central area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-15. Figure 5-15 does not include the recommended upgrades to the Lagoon system pump station, which are categorized as moderate priority improvements.

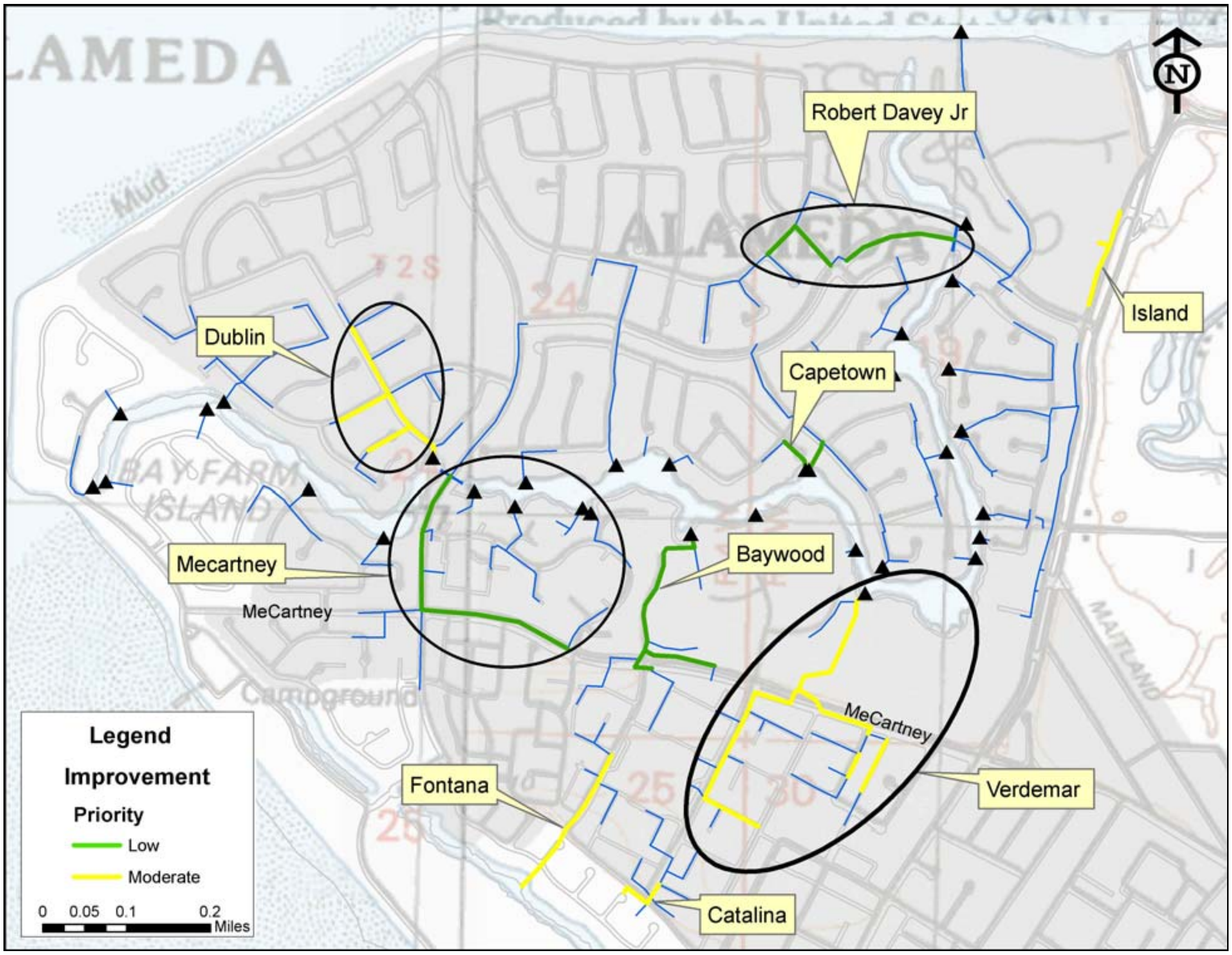


Figure 5-15: Bay Farm Central Area Prioritized 10-Year Improvements

Bay Farm South

Overview

The Bay Farm Island South drainage area is approximately 0.85 square miles, and is bounded by the San Francisco Bay to the west, the Oakland International Airport to the south and southeast, and by Bay Farm Island sub-areas to the northeast and north. The trunk lines of the Bay Farm South collection system consist of 204 nodes, 5 outlets and 2 interconnected lagoon storage areas. The South area has a total (including lateral lines) of 43,300 linear feet (8.2 miles) of connecting storm drain pipes equal to or greater than one foot in diameter. The Lagoons water levels are controlled via the manually operated System II pump station, which is described in more detail in Chapter 6. Because this pump is manually operated, it was not included in the storm drain model.

Historic Problem Areas

City staff report that during the 1997-1998 winter severe storms caused Lagoon waters to overtop banks in several locations. It is unknown if this flooding caused structural property damages.

Identified Deficiencies

MOUSE analysis of the South systems for the 10-year storm event showed some flooding (HGL above the rim elevation of the node) occurring at 64 of the 204 trunk line nodes. Of these, MOUSE predicts a flooding depth of less than 0.5 foot at 33 nodes. Depths of between 0.5 and 1.0 feet above the street occurred at 17 of nodes, with the remaining 14 nodes experiencing flooding depths greater than one foot. A map of the 10-year flooding depths predicted by MOUSE with no improvements is presented in Figure 5-16.

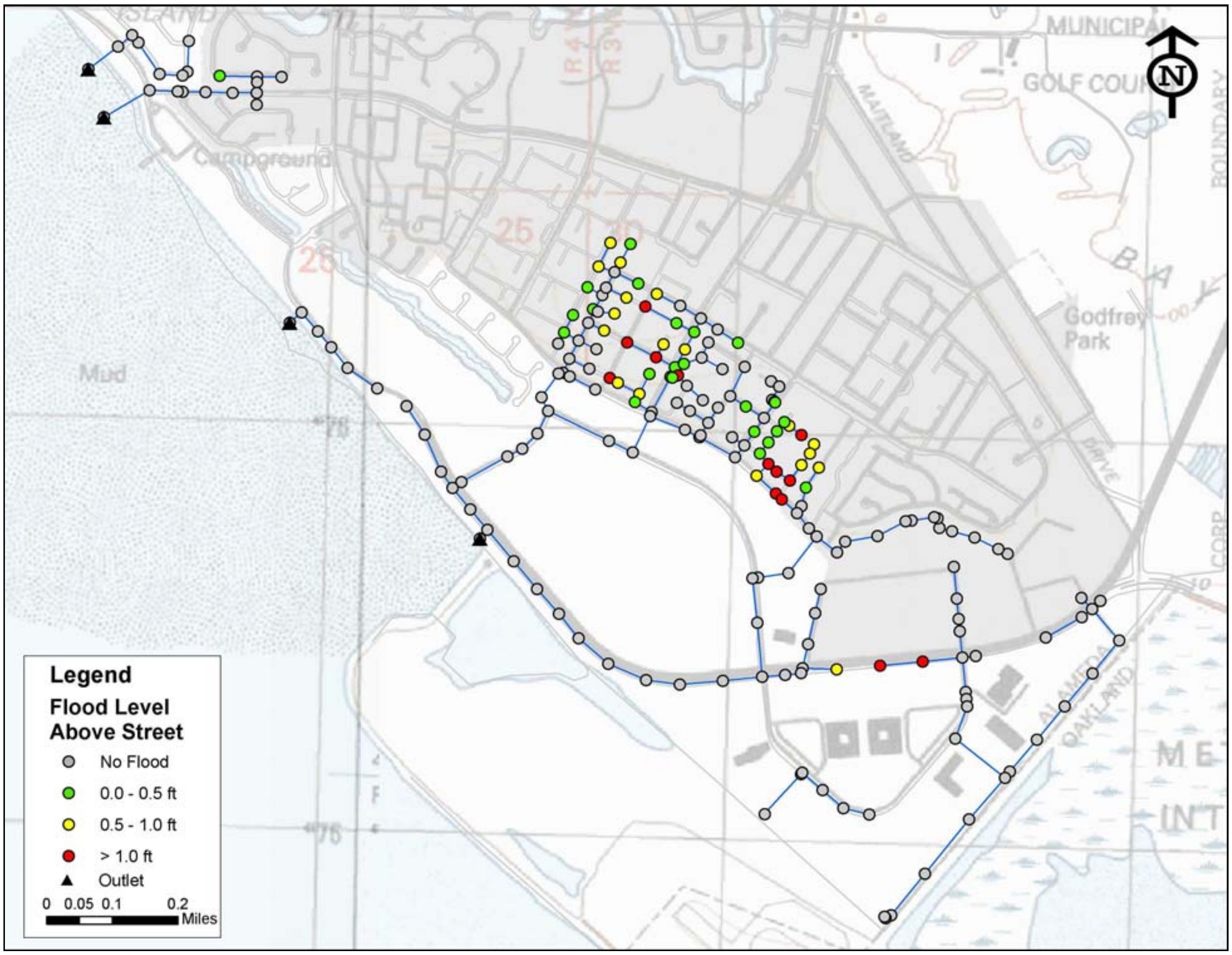


Figure 5-16: Bay Farm South Area Existing 10-Year Flooding Depths

Prioritized Improvements

The Bay Farm South area prioritized improvements that are required to alleviate or minimize flooding during a ten-year storm event are shown in Figure 5-17. Figure 5-17 does not include the recommended upgrades to the Lagoon system pump station, which are categorized as low priority improvements.

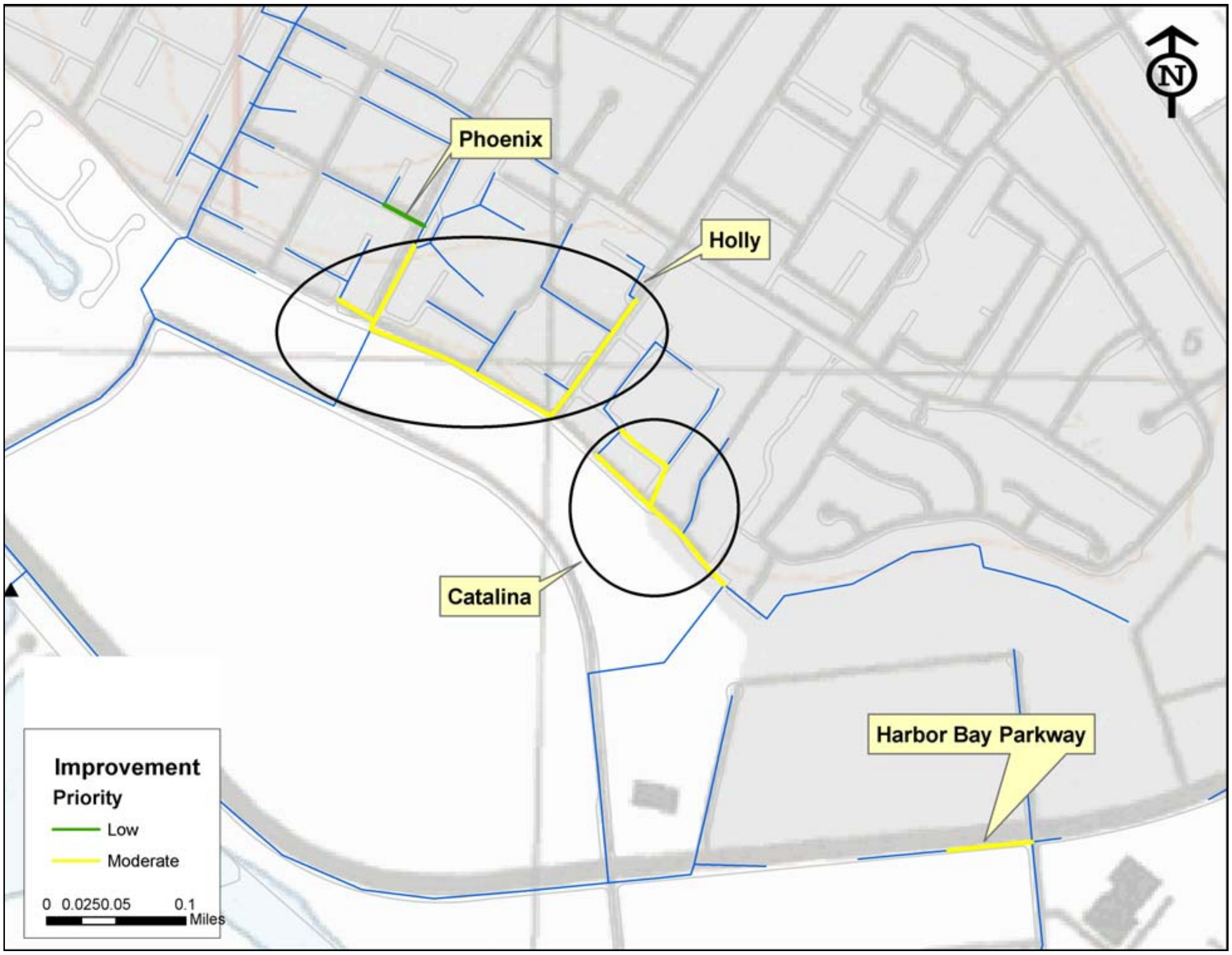


Figure 5-17: Bay Farm South Area Prioritized 10-Year Improvements

CHAPTER 6
PUMP STATIONS

Alameda currently operates eight automatic stormwater pumping facilities and two manually operated facilities on Bay Farm Island. Locations of the stormwater pump stations are shown in Figure 6-1. There are also smaller, privately operated pumps within the City that are not included in the master plan. This chapter evaluates pump station adequacy in the context of the stormwater master plan, recommending rehabilitation as necessary.

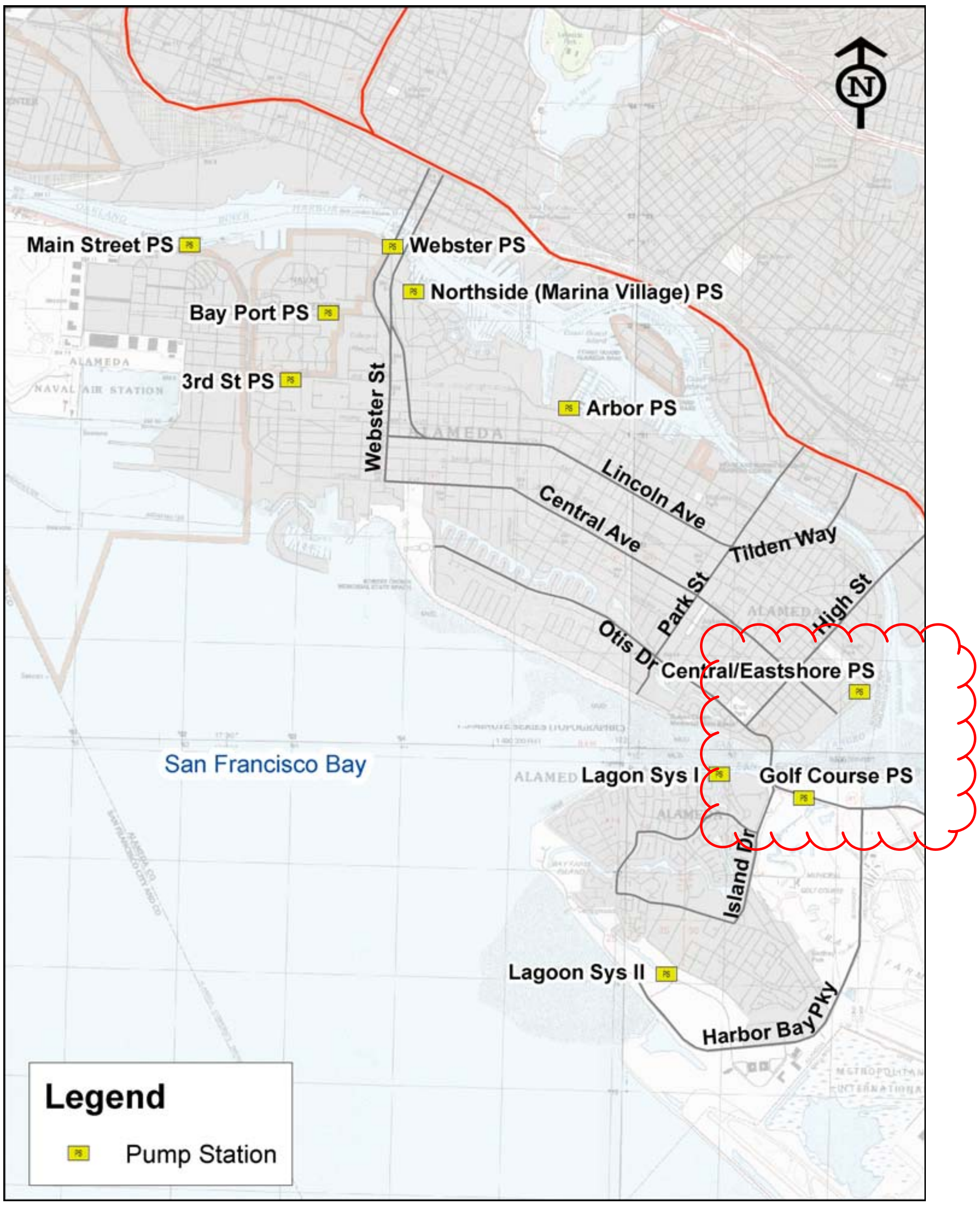


Figure 6-1: Pump Station Locations

GENERAL PUMP STATION CRITERIA

If City staff is able to operate and maintain a station without undue hardship, the station has adequate flow capacity, and provides for stationary or backup mobile pump in the event of mechanical failure, there is no need for master plan improvement. General pump station design criteria are listed below.

Capacity. Pump stations have been evaluated for adequate capacity within the MOUSE model. Pump stations are generally considered adequate if there is sufficient pump capacity to discharge design runoff into the receiving waters or if excess flows can be stored without causing property damage. The pump stations have been analyzed using the 10-year storm event with all available pumps running. (Pumping conditions with a 25-year storm event are included in Appendix C.) Table 6-1 (page 6-5) lists pump station design inflows and capacities.

Ideally at least two identical pumps would be installed in every storm water pump station for some redundancy and ease of maintenance. Other than 3rd Street and Harboy Bay Systems I and II all Alameda pump stations have more than one pump on site and/or in operation. It is not usual industry practice to include standby pumps in a stormwater station, because providing excess capacity is expensive and generally not justified by the relatively small risk of having a major storm event coincide with mechanical failure. All things considered, however, installing a larger number of smaller pumps is generally better than a lesser number of large pumps for the same capacity. When individual pumps comprise a smaller percentage of overall pump station capacity, having one pump fail is less detrimental. In terms of redundancy and ease of maintenance, all of the pumping units within one particular station should be identical.

Pumps and Drivers. Pump types differ from station to station in Alameda, although most are axial flow pumps and all are electric motor driven. A general trend in current pump station design is to use electric motors for prime power rather than direct-drive engines due to noise, ventilation and air quality considerations. Submersible pumps are also widely used for stormwater applications to reduce the complexity of lift station components. Alameda has a mix of submersible and more conventional shafted axial flow pumps driven by vertical electric motors. New pumps should be submersible, unless matching an existing pump or other site constraints dictate a more conventional pump.

Operation. Lead and lag pumps should be automatically alternated on every start to minimize pump cycling, equalize the number of operating hours among pumps as practicable, and extend the operating life of the equipment. Sufficient wet well storage must also be available in order to prevent excessive pump cycling for proposed operating levels.

The maximum number of pump starts per hour should be held below the maximum criterion established by pump, motor, and/or engine manufacturers. In the absence of specific data, pump starts should be limited to six per hour. This criterion is based on general limits set by large electric motor manufacturers; diesel engine suppliers also recommend that engines should run at least five to ten minutes at full operating temperatures each time they are started.

Pumping equipment must be specified so that motor or engine nameplate ratings are not exceeded at any point on the pump characteristic curve. Pump performance under different hydraulic conditions should be analyzed to ensure that pumps operate within manufacturers' recommended limits.

Excessive pump wear, vibration, noise, or cavitation could be indicative of more serious hydraulic problems associated with the sump and intake geometries.

Standby Power. Generators should be present on-site and connected to the power supply with an automatic transfer switch to be considered as available in an emergency under FEMA flood hazard mapping requirements. The use of portable generators, or even permanently parked generators with manual transfer switches, is only feasible where crews may respond to high water alarms during power outages, physically reach the pump station with a generator, and manually restore power before property damage has occurred. Small lift or pumping stations that generally handle nuisance flows (flows for which significant property damage would not occur should the pump station fail) do not necessarily require a standby power source. Currently, none of the Alameda pump stations have standby power, and the City has experienced periodic problems with power failure.

An emergency generator receptacle for portable standby power through manual transfer has been installed at Harbor Bay System No. 1.

Controls. Pump starts and stops may be controlled in a number of ways depending upon the age and condition of the equipment at any individual pump station. Newer pump stations often use a programmable logic controller (PLC) or a simpler programmable pump controller. Pump station controls and level monitoring systems should be coordinated with City operations and maintenance staff regarding function, standardization and ease of use. Control systems should also be provided with standby power to ensure that the station can function even during prolonged power outages. The preferred mechanism for providing standby power to control systems is rechargeable batteries, so that engines or engine-generators do not need to start during a power outage where pumping is not required. Alameda has a mix of programmable controllers and level controls that operate using compressed air (“bubblers”); the latter being simple and generally reliable.

Equipment Housing. All electrical equipment in or open to the wet well should be explosion-proof. Submersible motors should also be explosion-proof. Control panels must be located so that they are not subject to possible flooding. All equipment must be housed in NEMA-rated weatherproof enclosures or in buildings. Sufficient lighting (including back-up battery power) should be provided so that crews may work on equipment during the night. Also, access must be provided that will allow for the removal and reinstallation of all equipment. Noise abatement, visual impacts, and other aesthetics should also be considered. This is particularly important where pump stations are located near residential areas, which is the case for virtually all stations in Alameda.

Ventilation. Good ventilation is important to maintaining a dry, benign environment for mechanical and electrical equipment within a pump station. Proper ventilation helps reduce the deterioration of equipment due to condensation, and provides better working conditions for City crews. Without adequate ventilation, enclosures below grade may be classified as confined spaces, requiring special permits and rescue equipment for anyone entering them. Explosive gases from illegally dumped flammable liquids may also accumulate in wet wells and ancillary spaces. Many deaths and illnesses have been attributed to poor ventilation at pump stations.

PUMP STATION EVALUATION

Alameda's stormwater pumping facilities comprise both new, updated stations and older systems which have been partially updated or are in their original configurations.

Required pump station capacities are calculated assuming that proposed 10-year CIP improvements are complete. Table 6-1 provides a summary of current and required pump station capacities throughout Alameda. Pump station locations within the City are shown in Figure 6-1.

Table 6-1: Pumping Station Summary with 10-Year Storm Drain Improvements

Station Name	Location (Watershed)	Year Built or Updated	Design Capacity of Existing Station (GPM)	Actual Station Peak Discharge from Model (GPM)	Additional Req'd Station Discharge (GPM)
Main Street	Alameda Northside	1998	13,500 GPM	11,900 GPM	0 GPM
Third Street	Alameda Northside	1993	1,650 GPM	2,000 GPM	0 GPM
Webster Street	Alameda Northside	1947	5,250 GPM	4,600 GPM	0 GPM
Northside (Marina Village)	Alameda Northside	1984	72,000 GPM	83,300 GPM	0 GPM
Arbor	Alameda Northside	1994	31,600 GPM	38,200 GPM	50,400 GPM
Central/ Eastshore	Alameda Eastside	1967	8,600 GPM	11,300 GPM	7,000 GPM
Bayport	Alameda Northside	2004	42,600 GPM	44,000 GPM	0 GPM
Golf Course	Bayfarm East	1986	19,200 GPM*	22,000 GPM	0 GPM

* Pump design capacity data based on bid documents

It should be noted that the 'Actual Station Peak Discharge' column is the peak outflow from the pump stations with the existing pipe network. In some locations, most notably at Northisde (Marina Village) Pump Station, recommended pipe network improvements act to improve pump station operating capacity, even though additional capacity is not added via new pumps. For Central/Eastshore and Arbor pump stations, the additional capacity must be achieved via new pumps at the stations.

Main Street Pump Station

Main Street near Ferry Terminal

Constructed 1998

Tributary Area: 30 acre (from plans)

Outfall: Oakland Inner Harbor

Existing Equipment:

(3) Prime Pump 10” AV axial flow
Note: Only 2 of the 3 pumps working
4,500 gpm @ 13’ TDH (1,200 rpm)

25 hp submersible electric motors

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Add on-site standby power generator and automatic transfer switch.

Medium Priority: Repair 3rd Pump

Low-Priority (next major replacement):
None

Main Street Pump Station, located in the north western corner of the Northside sub-area, was constructed in 1998. The pump station consists of three pumps controlled by a programmable logic controller (PLC), although only two of the pumps are operational at this time. The pumps are driven by submersible electric motors with no backup power supply. There is a trash rack (4-inch spacing) that O&M staff indicate is sufficient for the station.

A certified pump curve (for input into the MOUSE model) is not available for these pumps; however a Prime Pump characteristic curve meeting the performance criteria outlined in the pump station plans (Model M12) has been used to model the pump station performance during storm events.

Flapgates protect the pump station and inlet storm drain system from backflow.

Third Street Pump Station

Appezzato Parkway (aka Atlantic) at 3rd Street

Constructed 1993

Tributary Area: 116.4 acres, 0.18 sq. miles

Outfall: Storm Drain system Leading to Arbor and Marina Village Pump Stations

Existing Equipment:

(1) Prime Pump P10 axial flow
1,650 gpm @ 8' TDH (1,150 rpm)

5 hp submersible electric motor

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Add standby power and automatic transfer switch. Determine condition of inlet pipes

Medium Priority: Add an additional backup pump

Low-Priority (next major replacement): Replace with new pump station that includes trash rack & dual pumps

Third Street Pump Station, located in the mid-east of the Northside sub-area, was constructed in 1993. The pump station consists of one pump controlled by a programmable logic controller (PLC). The pumps are driven by electric motors with no backup power supply. There is no trash rack at this pump station.

Although there is a flapgate that protects the pump station from backflow, it is currently frozen in the open position.

In the past, the area served by this pump station has flooded at 3rd and Brush, and in the Woodstock neighborhood. The storm drains leading to this pump station have not been inspected, and are old enough that some may be crushed.

Webster Street Pump Station

North end of Mariner Square Drive

Constructed 1947

Tributary Area: 123.3 acres, 0.19 sq. miles

Outfall: Oakland Inner Harbor

Existing Equipment:

(3) Prime Pump P10 axial flow
Note: Only 2 pumps in service
1,750 gpm @ 10’ TDH (1,150 rpm)

7.5 hp vertical electric motor drivers

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Add on-site standby power generator with automatic transfer switch

Medium Priority: Install self-cleaning trash rack if feasible

Low-Priority (next major replacement):
None

Webster Street Pump Station, located at the northern limint of the Northside sub-area, was constructed in 1947. The pump station consists of three pumps which are controlled by simple pump level controls (bubblers), although only two of the pumps are currently in service. The pumps are run by electric motors with no backup power supply. There is no trash rack at this pump station. Given the layout of the pump station and surrounding area, construction of a trash rack at this pump station may be unfeasible. The pump station is equipped with a flap gate to prevent backflow into the station.

Although located very near the Northside (Marina Village) pump station, Webster Street pump station is not connected by storm drain lines to any other pump stations. At the time of the pump station inspection, there was a noticeable amount of sediment/silt build up in the wet well.

Northside (Marina Village) Pump Station

Northern end of Marina Village Parkway

Constructed 1984

Tributary Area: 450.3 acres (0.7 sq. mi) between both Marina Village and Arbor pump stations

Outfall: Oakland Inner Harbor

Existing Equipment:

(3) Johnston Pumps 30” axial flow
24,000 gpm @ 10’ TDH (500 rpm) design, however maintenance crews estimate 13,000 gpm per pump capacity

75 hp vertical electric motor drivers

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Install on-site standby power generator with automatic transfer switch

Medium Priority: Replace the grating above the vault, which is corroded and deteriorating

Low-Priority (next major replacement):
None

The Northside (Marina Village) pump station is located at the northern end of the Northside sub-area, just southeast of the Webster Street pump station. The station was constructed in 1984. The pump station consists of three pumps which are controlled by simple pump level controls (bubblers). The pumps are run by electric motors with no backup power supply. However, the original station design allows space for a future generator in its own room. (Since 1984, tightening emissions standards have tended to increase the size of engine-generator sets.) This pump station is equipped with a self-cleaning inlet trash rack and a flapgate that protects the pump station from backflow.

The No. 2 pump motor has been recently re-worked, and the No. 3 motor is due for re-working.

Arbor Pump Station

Arbor Street at Clement Avenue

Constructed 1948, Additional Pumps 1994

Tributary Area: 450.3 acres (0.7 sq. mi) between both Marina Village and Arbor pump stations

Outfall: Oakland Inner Harbor

Existing Equipment:

(4) Prime Pump P16A axial flow
Note: Only 3 in service
7,900 gpm @ 15' TDH (1,150 rpm)

40 hp submersible electric motors

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Add on-site standby power generator with automatic transfer switch. Remove bar screen and replace with self-cleaning trash rack, replace corroded pump tubes. Increase pump station capacity (see Table 6-1).

Medium Priority: None

Low-Priority (next major replacement):
Replace with new pump station

Arbor Street Pump Station, located in the north eastern corner of the Northside sub-area, was originally constructed in 1948 with two pumps. In 1994 the two original pump station pumps were replaced with four submersible pumps and the roof was modified for the new pump installations. The pumps are controlled by simple pump level controls (bubblers). Of these, one pump is currently out of service due to corrosion of the pump enclosure tube. The pumps are driven by submersible electric motors with no backup power supply. A flapgate protects the pump station from backflow. During the summer, the pump station is opened to allow tidal waters to flush the system.

Currently, the pump station has a bar screen which acts as a trash rack. The pump station receives considerable debris, much of which is leaf litter. Cleaning the bar screen is a labor intensive process, which involves confined space access and requires the monitoring of oxygen levels (low levels have been detected, requiring trash rack cleaning to cease). The process to clean the trash rack can take an entire day. The leaf litter in particular make the bar screen act as a barrier to flows, and a hydraulic gradient develop behind (i.e. upstream) of the bar screen. This leads to excessive cycling of the pumps.

One area of known flooding that is served by this pump station is the northern end of 9th Street at the Railroad property.

Central / Eastshore Pump Station

Eastshore Drive at Central Avenue

Constructed 1967

Tributary Area: 137.5 acres, 0.21 sq. miles

Outfall: San Leandro Canal

Existing Equipment:
(2) Prime Pump M12A axial flow
4,300 gpm @ 15' TDH (1,180 rpm)

25 hp submersible electric motors

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority:
Add on-site standby power generator with automatic transfer switch. Install self-cleaning mechanism to trash rack (or replace w/ self cleaning rack)

Medium Priority:
Increase Station Capacity (see Table 6-1).

Low-Priority (next major replacement):
None

Central/Eastshore Avenue Pump Station, located at the eastern end of the Eastside sub-area, was constructed in 1967. The pump station consists of two pumps which are controlled by simple pump level controls (bubblers). The pumps are run by electric motors with no backup power supply. There is a gravity bypass in the case of power failure which would allow floodwaters to exit the pump station during low tides; however the bypass gates are currently frozen shut. There is a bar screen that acts as a trash rack at this pump station as well as a flapgate that protects the pump station from backflow.

In general, the maintenance department does not have nuisance flooding issues relating to this pump station, except in the case of power outages.

<table>
<tr><td colspan="2">Bayport Pump Station</td></tr>
<tr><td>5th Street at Tinker Avenue</td><td rowspan="6">Bayport Street Pump Station, located on the eastern side of the Northside sub-area between the Webster and 3rd Street pump stations, is the most recent pump station constructed in Alameda (2004). The pump station was constructed as an element of the Bayport housing development, and consists of four pumps which are controlled by a programmable logic controller (PLC), as well as a small submersible sump pump. The pumps are run by electric motors and the pump station does have automatic standby power. There is a self cleaning trash rack at this pump station, and a flapgate that protects the pump station from backflow.

The City does not experience nuisance flooding in areas protected by this pump station.</td></tr>
<tr><td>Constructed 2004</td></tr>
<tr><td></td></tr>
<tr><td>Tributary Area: 123.6 acres, 0.2 sq. miles</td></tr>
<tr><td>Outfall: Oakland Inner Harbor via constructed pond</td></tr>
<tr><td>Existing Equipment:
(4) Flygt PL7061 Axial Flow Pumps
10,650 gpm @ 17’ TDH (590 rpm)

(1) submersible sump pump
25 hp, 3,300 gpm @ 18’ TDH

70 hp submersible motors

Standby Power: Yes

<u>Master Plan Recommendations</u>
High Priority:
None

Medium Priority:
None

Low-Priority (next major replacement):
None</td></tr>
</table>

Golf Course Pump Station

Golf Course Slough at Doolittle Drive

Constructed 1985

Tributary Area: 478.1 acres, 0.75 sq. miles

Outfall: San Leandro Canal / Estuary

Existing Equipment:
(2) Prime Pumps 10” AV
Assumed: 9,600 gpm @ 5’ TDH (1,165 rpm)

50 hp submersible electric motor drivers

Standby Power:
None

<u>Master Plan Recommendations</u>

High Priority: Add on-site standby power generator with automatic transfer switch. Add an additional pump for capacity redundancy.

Medium Priority: Add Bypass Pumping Capability

Low-Priority (next major replacement):
None

The Golf Course Pump Station, located in the north eastern corner of Bay Farm Island (in the Bay Farm East subarea), was constructed in 1985. The pump station consists of 2 pumps which are controlled by five electronic probes. The pumps are run by electric motors with no backup power supply. A fence in the approach slough serves as the only ‘trash rack’ in this location. Backflow protection is provided by swing checks in the outlet pipes.

During the 1998 winter storms, the entire golf course area flooded and maintenance staff approximate that it took 3 weeks of this pump station running 24-hours a day to empty the accumulated flood waters.

No plans or pump curves were available for this pump station. A bid for providing 2 pumps was provided from both Worthingon and Aurora pump suppliers; however the actual pump station controls are labeled as Prime Pumps. A sensitivity analysis was conducted on the pump capacity, and it was found that due to the large amount of storage provided by the golf course itself, the capacity of the pumps (within the range indicated by received bids and field data) did not have an impact on anticipated flooding on Bay Farm Island. This finding was confirmed by City maintenance staff, who indicated that the capacity of the existing pump station is sufficient, and unrelated to local flooding outside of the golf course.

Harbor Bay Systems I and II Pump Stations

Bay Farm Island, at the Northern and Southern Limits of Harbor Bay Lagoon

Constructed dates unknown

System I Pump Station:

System II Pump Station:

Existing Equipment:

System I: Unknown
System II: Prime Pump No. 20P16A-11.5, 60 hp, 1170 RPM

Standby Power: Portable generator for Sys I.

The Harbor Bay pump stations are operated manually, and for this reason were not included in the storm drain model. Additionally, only limited research into the specifics of these pump stations was conducted. The purpose of these pump stations is to drawn down the Harbor Bay Lagoons, which were purportedly built to withstand a 100-year event.

Similar to the Golf Course pump station analysis, Schaaf & Wheeler conducted a sensitivity analysis on the operation of these pump stations via an analysis on the impacts of Lagoon water levels to flooding on Bay Farm island, and found that as long as the Lagoon water levels are kept within their target range, those levels do not impact flooding on Bay Farm Island. If a pump station was to fail completely, however, and Lagoon water levels exceed their intended range, this could result in significant flooding along the Lagoons and tributary areas. In general, City staff aim to preemptively drawn down Lagoon levels when storms are expected.

System I: The Lagoon System I pump station is located on the northern end of Bay Farm Island, within the Bay Farm North sub-area. The pump station discharges to the San Leandro Canal, and is the original pump; however the pump has been reworked within the past 10 months. There is an off-site

<u>Master Plan Recommendations</u> **High Priority:** System I: New gate operators at inlet, on-site backup power System II: Replace/Repair Bayside Leak (in process), backup power **Medium Priority:** Systems I & II: Automate the pump operation System II: Investigate and repair possible sag in outlet pipe **Low-Priority (next major replacement):** System I: New Inlet Gates	generator that is intended to provide back up power to this pump station. <u>System II:</u> The pump station for Lagoon System II is located at the end of Souza Court near its intersection with Ratto Road and discharges to the San Francisco Bay. Currently, the pump station is inoperable due to a leak in the bayside gate which causes circular pumping. A new gate is in the process of being installed. Recent investigations by maintenance staff to determine the condition of the outlet pipe from this pump station has indicated that there may be a sag in the outlet pipe (i.e. adverse grade). During the 1998 winter storms (nearly a 100-year storm event in some Bay Area locations), flood waters overtopped the Lagoon banks by up to four feet.

CHAPTER 7
CAPITAL IMPROVEMENTS

Chapters 5 and 6 evaluate Alameda's storm drain collection and pumping systems, and recommend prioritized capital improvements to address deficiencies. This chapter provides a Capital Improvement Program (CIP) that recognizes these priorities. The CIP provides an overall guideline for the City to use in preparing annual budgets. Exigent circumstances and future in-field experiences may necessitate deviations from the Storm Drain CIP. A master plan is intended to be just that; a tool for planning. Capital improvement priorities are not intended to be hard and fast.

The CIP does not include the cost of new facilities related to new development (e.g., pipeline extensions to serve areas that are currently undeveloped and not served by an existing City pipeline). These new facilities would be constructed as part of the new developments, and are not included in the CIP.

CAPITAL IMPROVEMENT PRIORITIES

The proposed CIP for storm drainage in Alameda is broken into three priority levels for funding and implementation, as shown in Table 4-1. The total costs summary for the 10-year CIP projects along with the required lengths are shown for each priority level in Table 7-1. Each subarea includes the recommended capacity improvements, including pump station capacity improvements. Also included in the table are recommended pump station upgrades such as self cleaning trash racks and on site backup power. As noted previously, the CIP information for the 25-year storm has been included in Appendix C, and summarized in Table 1-1 in the first chapter of this report.

Table 7-1 Summary of 10-Year Storm Protection CIP Costs

Alameda Island						
	High		Medium		Low	
	Length	Cost	Length	Cost	Length	Cost
Northside	17,000	$19,740,000	2,300	$1,460,000	7,700	$2,940,000
North Central	0	$0	9,400	$3,930,000	11,500	$5,290,000
Eastside	9,600	$5,350,000	5,100	$3,070,000	0	$0
South	3,600	$2,000,000	15,800	$6,340,000	7,100	$2,870,000
Total Alameda Island	30,200	$26,590,000	32,600	$14,800,000	26,300	$11,100,000
Bay Farm Island						
	High		Medium		Low	
	Length	Cost	Length	Cost	Length	Cost
North	0	$600,000	1,900	$1,320,000	200	$680,000
South	0	$0	3,200	$1,290,000	200	$670,000
East	0	$0	5,700	$2,300,000	600	$250,000
Central	0	$0	7,000	$2,530,000	5,200	$2,060,000
Total Bayfarm Island	0	$600,000	17,800	$7,440,000	6,200	$3,660,000
TOTAL:	30,200	$27,690,000	50,400	$22,240,000	32,500	$14,760,000

Table 7-1 costs include a 40% increase in construction cost estimates to include design, administration, and contingency costs. Also included in the above summary table are the costs to complete improvements that are not directly related to system capacity but to system safety, operations, or redundancy (see Table 7-11). Not included in Table 7-1 are costs to extend the storm drain system, as presented in Chapter 5, Figure 5-9. Those costs are included in Table 7-12.

ALTERNATIVE IMPROVEMENT PROJECTS

In general, to increase storm drain system capacity, two essential types of projects are available: installing a new relief storm drain parallel to the system lacking capacity; or replacing the overloaded pipe with larger diameter pipe in the same alignment. The two alternatives can be made equivalent to one another using the following formula, assuming that pipe material and length are equal:

$$D_R = (D_e^{2.63} + D_p^{2.63})^{0.38}$$

where D_R = diameter of replacement pipe;
D_e = diameter of overloaded pipe; and
D_p = diameter of parallel relief drain.

The selection of a capacity improvement strategy will vary from project to project; and be governed by field constraints such as conflicting utilities, rights-of-way, and traffic control. Based on discussions with City staff, the Storm Drain Capital Improvement Program for Alameda generally utilizes replacing existing pipes with larger pipes where improvements are needed. Occasionally new pipes in locations where there are currently no storm drain pipes have been recommended when increasing the size of existing pipes was not able to mitigate the flooding.

Traditional cut and cover methods of construction will be employed for most storm drain construction. However the utilization of bore and jack, trenchless (e.g. directional drilling), and other methods may find application in special circumstances such as railroad crossings. Discussions with industry representatives indicate that some other special techniques such as sliplining and pipe bursting are only applicable to smaller (i.e. 24-inch and less) pipe sizes.

COST OF IMPROVEMENTS

Costs have been estimated using information from other projects, cost estimating guides (*2004 Current Construction Costs*, Saylor Publications, Inc.), and engineering judgment. The cost per linear foot of improvement used for the cost estimates are given in Table 7-2 (note that these costs do not include the 40% increase for design, administration, and contingency included in all other tables). Connection (i.e. manhole) replacement cost estimates ranged from $9,000 to $12,000 depending on diameters. All estimates are based on the ENR January 2008 index of 7797. Costs include open trenching in roadway from up to ten feet in depth. Costs do not include permitting or any environmental documentation. Most of these projects are expected to qualify for negative declarations from permitting agencies.

Table 7-2: Storm Drain Cost Per Linear Foot

Diameter (inches)	Dollar per Linear foot of Pipe	Dollar per Connection
15	$110	$8,621
18	$122	$9,015
21	$142	$9,170
24	$163	$9,324
27	$184	$9,479
30	$205	$9,634
33	$229	$9,800
36	$253	$9,967
42	$284	$10,311
48	$318	$10,666
54	$350	$11,033
60	$392	$11,431
72	$476	$12,226

In addition to increased pipe capacity, some increased pump station capacity recommendations were also made. The pump station capacity improvement costs are for complete rehabilitation of the pump stations, which includes the installation of on-site backup power and self-cleaning trash racks, are estimated to be $20,000 - $25,000 per station cfs. This amount is based on the total build out capacity of the pump station, since for those pump stations were additional capacity is recommended there is not room to install a new pump in the existing station. New outfall costs were estimated to be $25,000 per new outfall, although it should be noted that wide variations in actual outfall costs is expected.

CAPITAL IMPROVEMENT PROGRAM

A proposed Storm Drain Capital Improvement Program which summarizes the CIP cost allowances by project name and watershed is presented in Tables 7-3 thru 7-11. All cost estimates include an additional 25% for design and administration and 15% percent contingency. Maps of the improvement priorities are shown in Chapter 5 and Figures 7-1 through 7-8 show the recommended improved pipe diameters. In summary (including an additional 25% for design and administration and a 15% contingency):

High Priority Capital Improvements $27,700,000

Moderate Priority Capital Improvements	$22,200,000
Low Priority Capital Improvements	$14,800,000
Total Capital Improvement Program	$64,700,000

Table 7-3: Alameda Island, Eastside Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Gibbons (new pipe)	High	4000	13	1	$1,151,000	$1,611,000
Liberty	High	243	6	1	$115,000	$161,000
Encinal	High	187	2	0	$58,000	$81,000
Thompson	High	1344	11	1	$388,000	$543,000
High	High	3776	26	1	$1,284,000	$1,798,000
Fernside	Moderate	2930	16	0	$738,000	$1,033,000
Washington	Moderate	1204	9	0	$346,000	$484,000
Post	Moderate	454	5	1	$135,000	$189,000
Calhoun	Moderate	534	5	1	$154,000	$216,000

Table 7-4: Alameda Island, North Central Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Grand	Moderate	5553	34	1	$1,597,000	$2,236,000
Willow	Moderate	3873	21	1	$1,210,000	$1,694,000
Walnut	Low	3999	23	1	$1,221,000	$1,709,000
Oak	Low	1399	9	1	$469,000	$657,000
Park	Low	637	7	1	$235,000	$329,000
Everett	Low	1086	8	1	$385,000	$539,000
Broadway	Low	1830	14	1	$582,000	$815,000
Pearl	Low	1189	8	1	$347,000	$486,000
Tilden	Low	395	5	1	$136,000	$190,000
Cambridge	Low	986	8	1	$402,000	$563,000

Table 7-5: Alameda Island, Northside Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Constitution	High	3300	12	1	$1,446,000	$2,024,000
West Atlantic	High	3400	26	1	$1,627,000	$2,278,000
East Atlantic (1)	High	2900	22	0	$1,454,000	$2,036,000
East Atlantic (2)	High	3300	24	1	$1,787,000	$2,502,000
New Outfall	High	4100	11	1	$2,320,500	$3,249,000
Marina Village Parkway	Moderate	2300	12	1	$686,000	$960,000
Main St	Low	900	6	0	$246,000	$344,000
Webster (2)	Low	1000	7	0	$251,000	$351,000
3rd Street	Low	400	2	0	$81,000	$113,000
Webster (3)	Low	1200	5	0	$260,000	$364,000
9th Street	Low	1100	5	0	$290,000	$406,000
Chapin	Low	300	4	0	$109,000	$153,000
Paru	Low	1300	13	0	$419,000	$587,000
Bay Sherman	Low	1500	16	0	$447,000	$626,000

Table 7-6: Alameda Island, South Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Fountain	High	2025	20	1	$911,000	$1,275,000
Mound	High	1616	9	1	$517,000	$724,000
Franciscan	Moderate	2719	16	0	$732,000	$1,025,000
Harbor Light	Moderate	4085	22	1	$1,111,000	$1,555,000
Rosewood	Moderate	2331	18	1	$548,000	$767,000
Versailles	Moderate	769	5	0	$245,000	$343,000
Pearl	Moderate	696	6	0	$269,000	$377,000
Alameda Park	Moderate	2277	7	0	$616,000	$862,000
3rd	Moderate	794	7	1	$252,000	$353,000
Willow	Moderate	1670	10	1	$627,000	$878,000
S Shore Center W	Moderate	484	4	0	$127,000	$178,000
Regent	Low	462	7	1	$202,000	$283,000
Park	Low	1020	8	0	$210,000	$294,000
Page	Low	2146	17	1	$564,000	$790,000
Webster	Low	1211	9	1	$337,000	$472,000
Ballena	Low	795	1	1	$260,000	$364,000
Paru	Low	71	2	0	$25,000	$35,000
Union	Low	90	2	0	$27,000	$38,000
Shoreline	Low	817	7	2	$243,000	$340,000
Balboa	Low	207	4	0	$64,000	$90,000
Otis/Oak	Low	292	4	0	$114,000	$160,000

Table 7-7: Bay Farm Island, Central Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Dublin Way	Moderate	1642	11	1	$395,000	$553,000
Island Drive	Moderate	692	5	0	$129,000	$180,600
Catalina Ave	Moderate	339	5	0	$97,000	$135,800
Fontana Drive	Moderate	1007	10	1	$262,000	$366,800
Verdemar Drive	Moderate	3367	26	1	$927,000	$1,297,800
Robert Davey Jr Dr	Low	1308	8	0	$312,000	$436,800
Capetown Court	Low	430	5	1	$139,000	$194,600
Baywood Road	Low	1633	16	1	$524,000	$733,600
Mecartney Road	Low	1855	9	0	$493,000	$690,200

Table 7-8: Bay Farm Island, North Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Stanbridge	Moderate	810	7	0	$193,000	$270,200.0
Avington	Moderate	1089	8	1	$318,000	$445,200.0
Shamrock	Low	223	3	0	$58,000	$81,200.0

Table 7-9: Bay Farm Island, East Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Flower Lane	Moderate	3212	23	0	$863,000	$1,208,200
Melrose	Moderate	2479	23	0	$782,000	$1,094,800
Fitchburg	Low	632	5	0	$178,000	$249,200

Table 7-10: Bay Farm Island, South Area 10-Year Storm Protection CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Harbor Bay	Moderate	319	2	0	$101,000	$141,400
Catalina	Moderate	1075	9	0	$309,000	$432,600
Holly	Moderate	1823	11	0	$509,000	$712,600
Phoenix	Low	173	2	0	$47,000	$65,800

The improvements recommended in the above tables are all capacity related improvements – projects that will decrease flooding on the streets of Alameda. In addition to these pipe capacity improvements, there are several recommended pump station upgrades which are recommended to increase capacity and/or to add water quality, reliability and redundancy, and/or maintenance improvements. These projects include installation of on-site backup power and trash racks at all pump stations. Based on past projects and engineering judgment, the estimated cost for adding back up is $500,000 per pump station. This cost includes the evaluation and installation of standby power, an automated transfer switch, and electrical panel modifications to accommodate the backup power. The estimated cost for the installation of a new, self cleaning trash rack is $100,000 per pump station. This cost includes the design and installation of new trash racks but does not include significant structural work that may be necessary to accommodate new trash racks. Table 7-11 presents the total allowance recommendations for these improvements, which includes contingencies.

Table 7-11: Pump Station Recommendations for Improvements

Pump Station	Trash Rack	Backup Power	Pump Station Rehabilitation Allowance	Capacity Improvements	Priority Level
Arbor		included		$4,000,000	High
Central		included		$800,000	Moderate
Main Street	☑	$500,000	~		High
Third Street	n/a	~	$250,000 (10 cfs)		High
Marina Village (Northside)	☑	included	$1,000,000		High
Webster Street	n/a	~	$300,000 (12 cfs)		High
Bayport	☑	$500,000	~		Moderate
Golf Course	$100,000	$500,000	~		High
Harbor Bay Sys. I	$100,000	$500,000	~		Low
Harbor Bay Sys. II	$100,000	$500,000	~		Moderate
TOTAL	$300,000	$3,000,000	$1,100,000	$4,800,000	$9,200,000

At Webster and 3rd Street Pump Stations, although not recommended for a capacity upgrades, given the small size of the pump stations it is more reasonable to estimate a cost allowance for trash rack and back up power installation as station rehabilitation projects. The cost for capacity and rehabilitation improvements for these pump stations includes the installation of on-site backup power and self cleaning trash racks. The same may be true for the Harbor Bay pump stations, but since the capacity of those pumps is unknown the more conservative cost estimate was used. These improvement projects have been ranked as high priority by City staff, although they do not all directly impact storm drain capacities.

In addition to the pump station upgrades, extension of some existing storm drain lines is recommended (see Chapter 5, Figure 5-9). Similar to some of the pump station upgrades, these improvements do not impact the capacity of the existing system, although they are expected to lessen street flooding in the locations recommended. The estimated costs for these extensions, which with the exception of the Oak Street extension are considered low priority, are presented in Table 7-12. These costs were included in the summary of CIP costs in Chapter 1, Table 1-1.

Table 7-12: Storm Drain Pipe Extension Recommendations Costs

Area	Pipe Length	Number of Connections	Number of Inlets	Total Cost
Northside	3500	17	18	$621,000
North Central	2800	11	14	$466,000
South	4500	21	28	$789,000
Eastside	200	2	2	$50,000

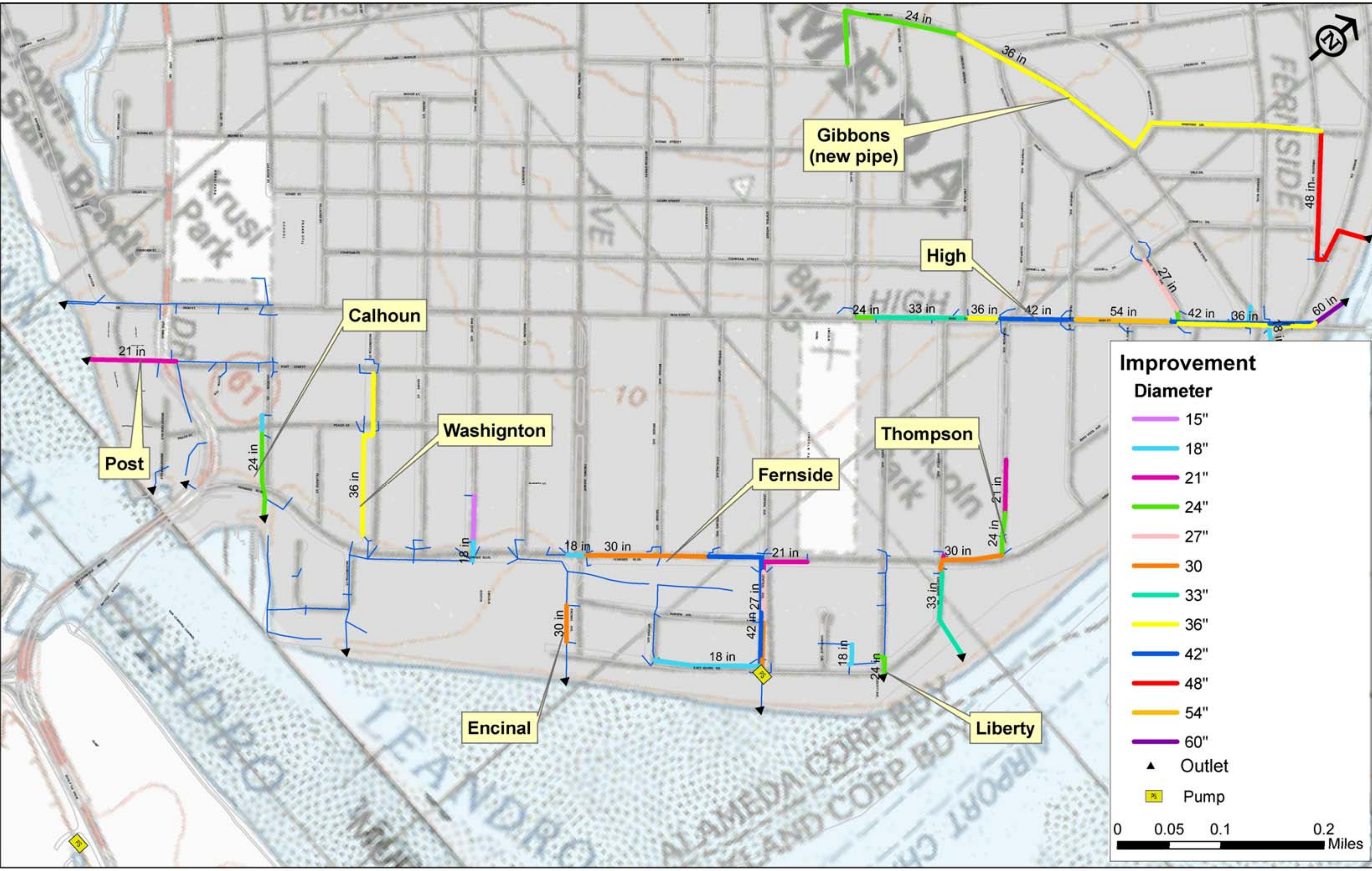


Figure 7-1: Alameda Eastside Area 10-Year Improvement Recommended Diameters

Fig 7-1

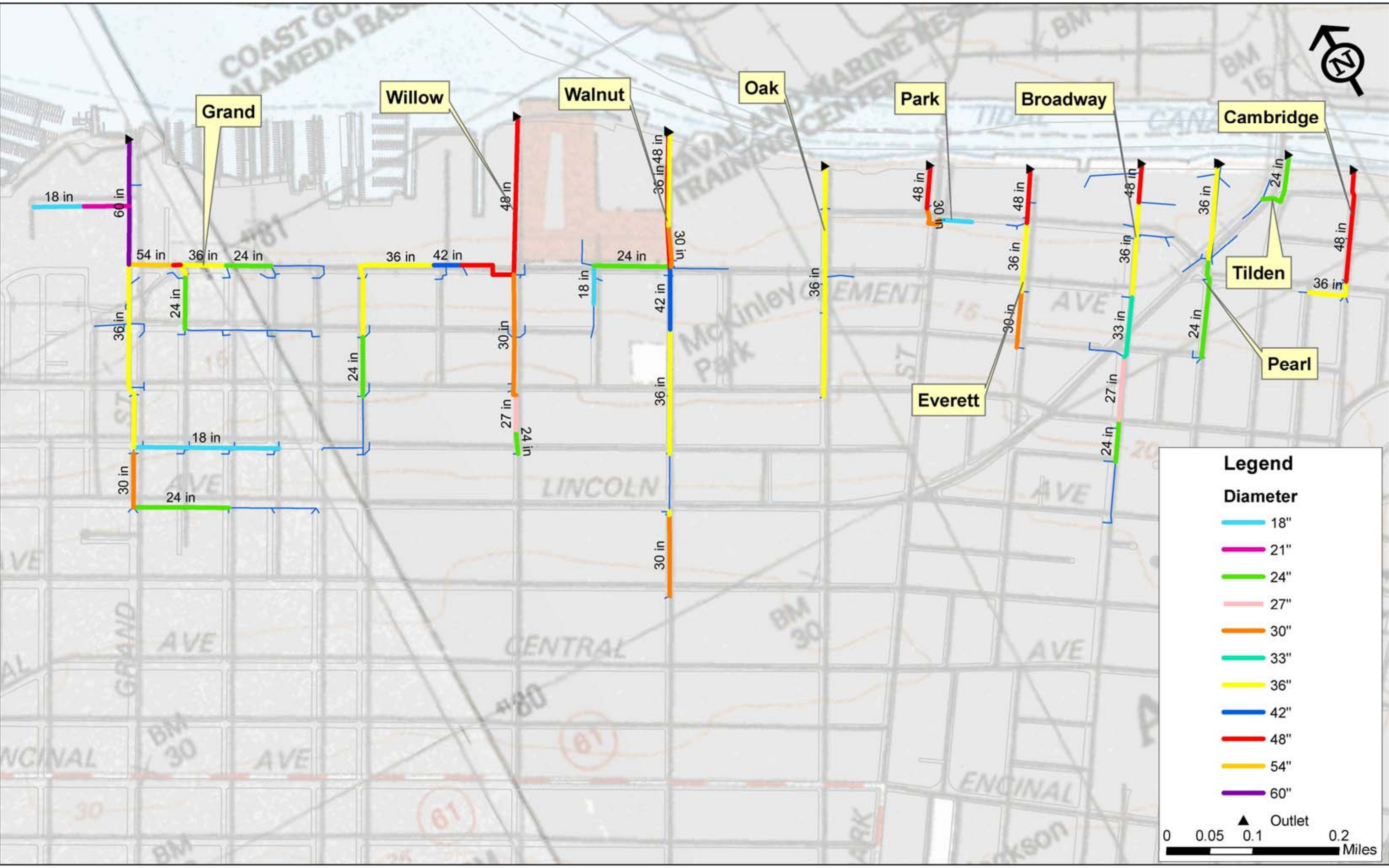


Figure 7-2: Alameda North Central Area 10-Year Improvement Recommended Diameters

Fig 7-2

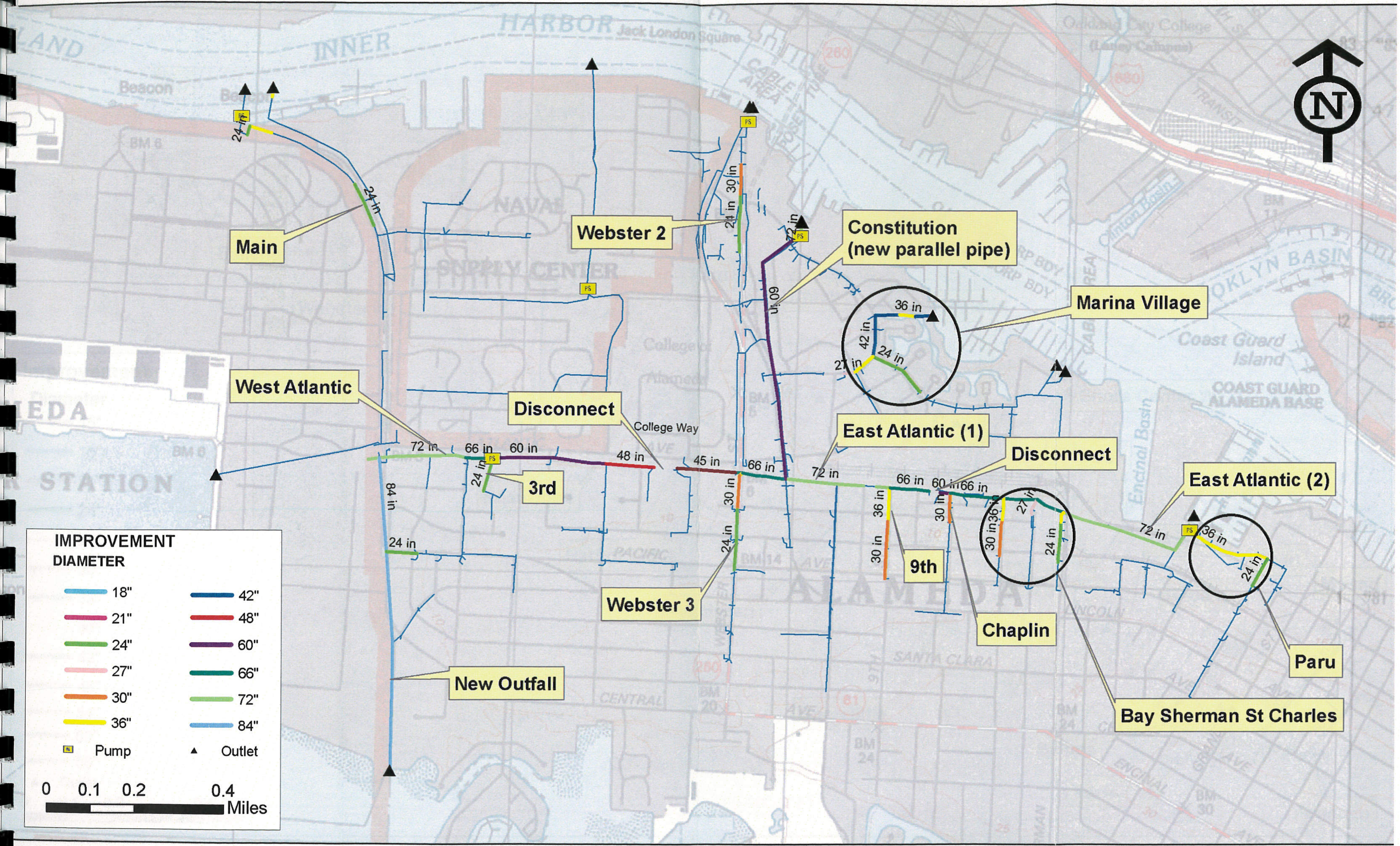


Figure 7-3: Alameda Northside Area 10 Year Improvement Recommended Diameters

Fig 7-3

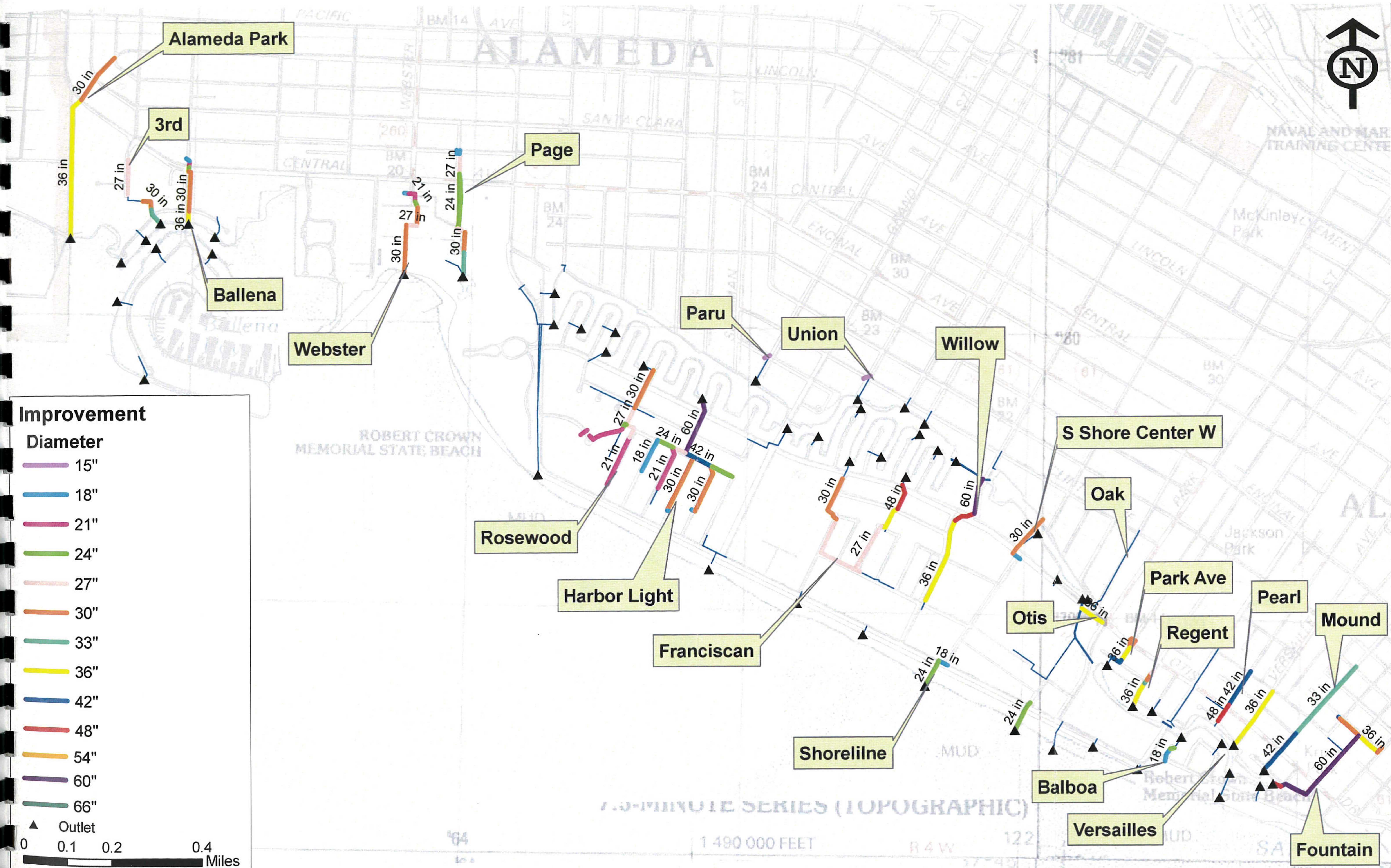


Figure 7-4: Alameda South Area 10-Year Improvement Recommended Diameters

Fig 7-4

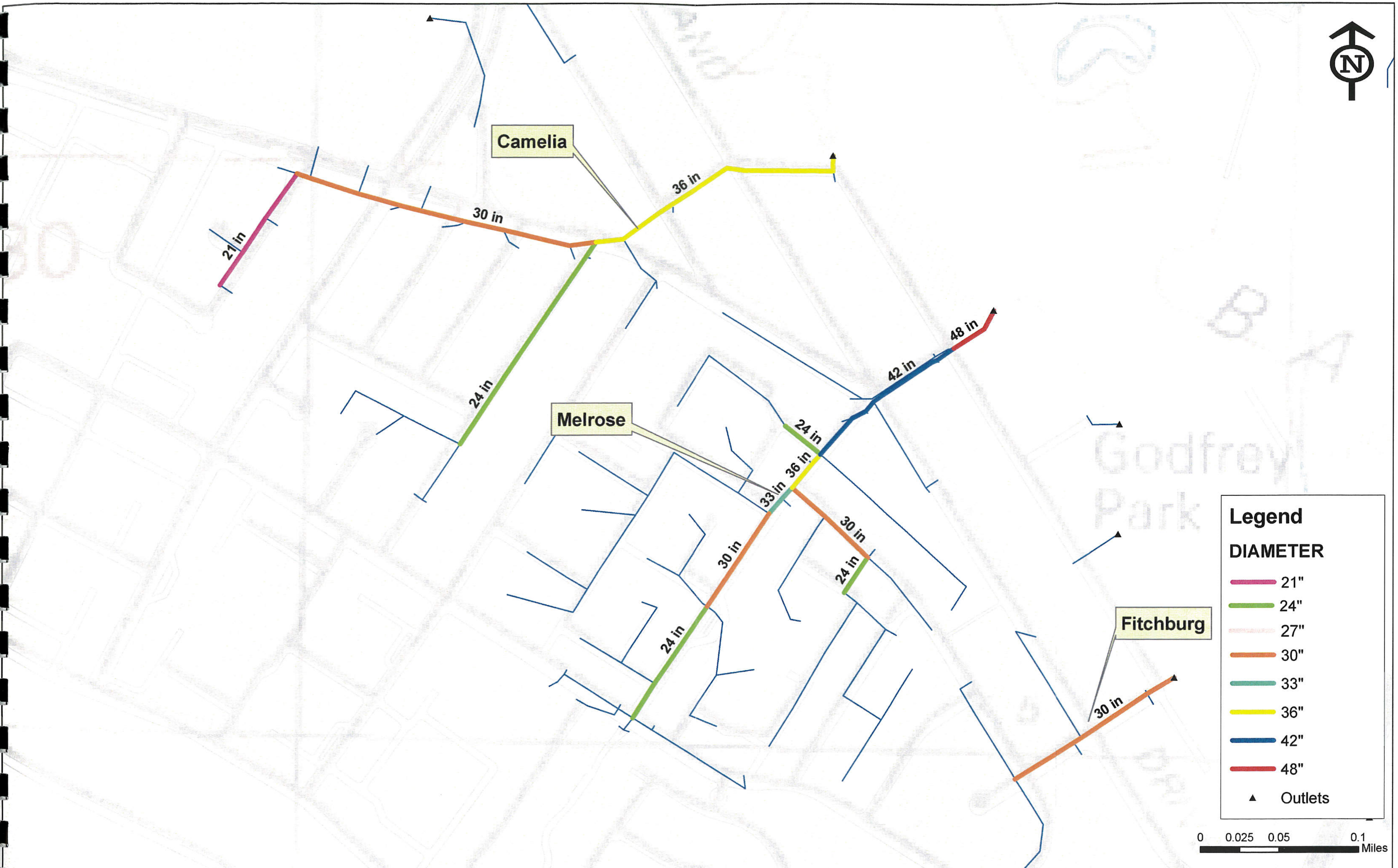


Figure 7-5: Bay Farm East Area 10-Year Improvement Recommended Diameters

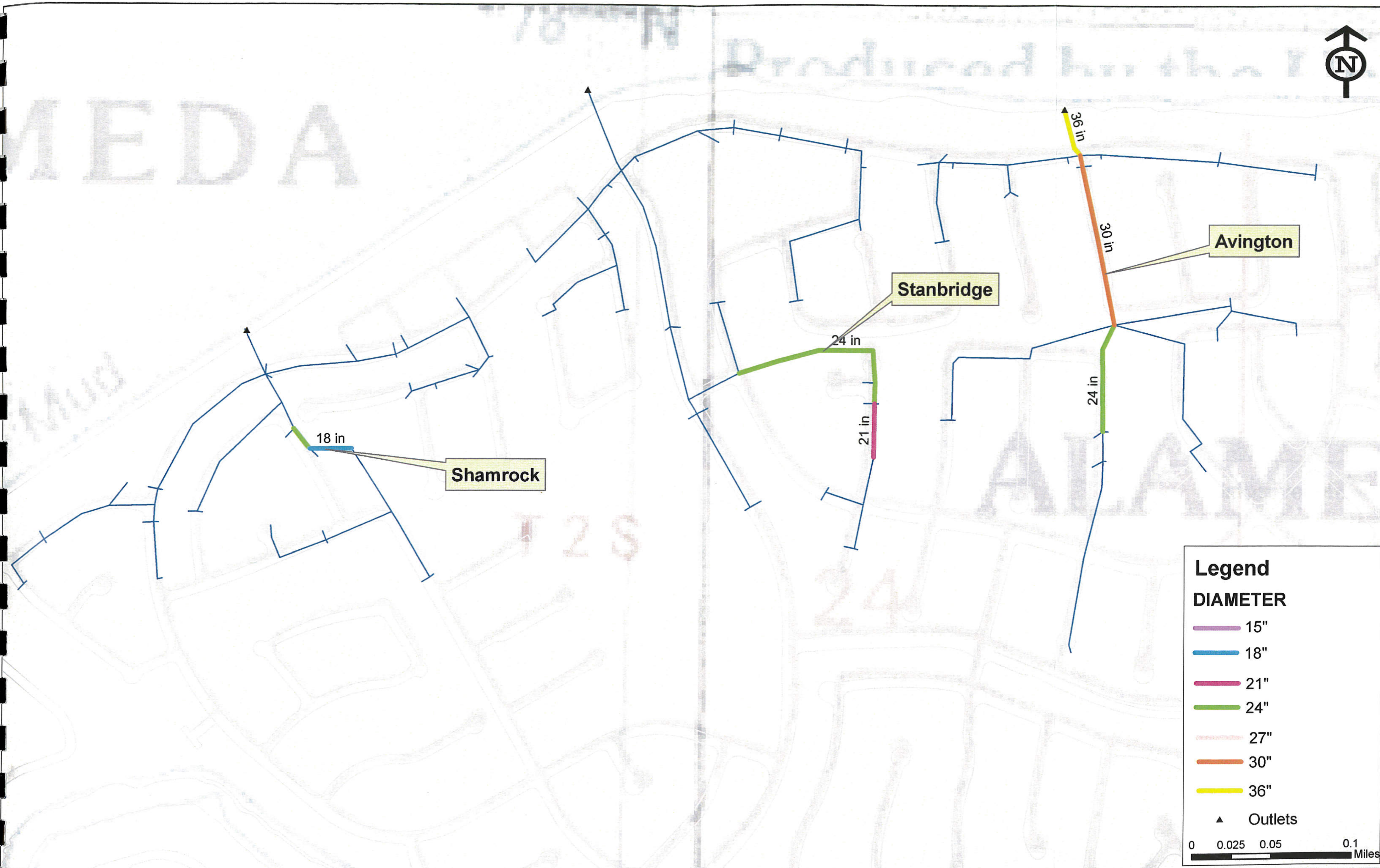


Figure 7-6: Bay Farm North Area 10-Year Imporovement Recommended Diameter

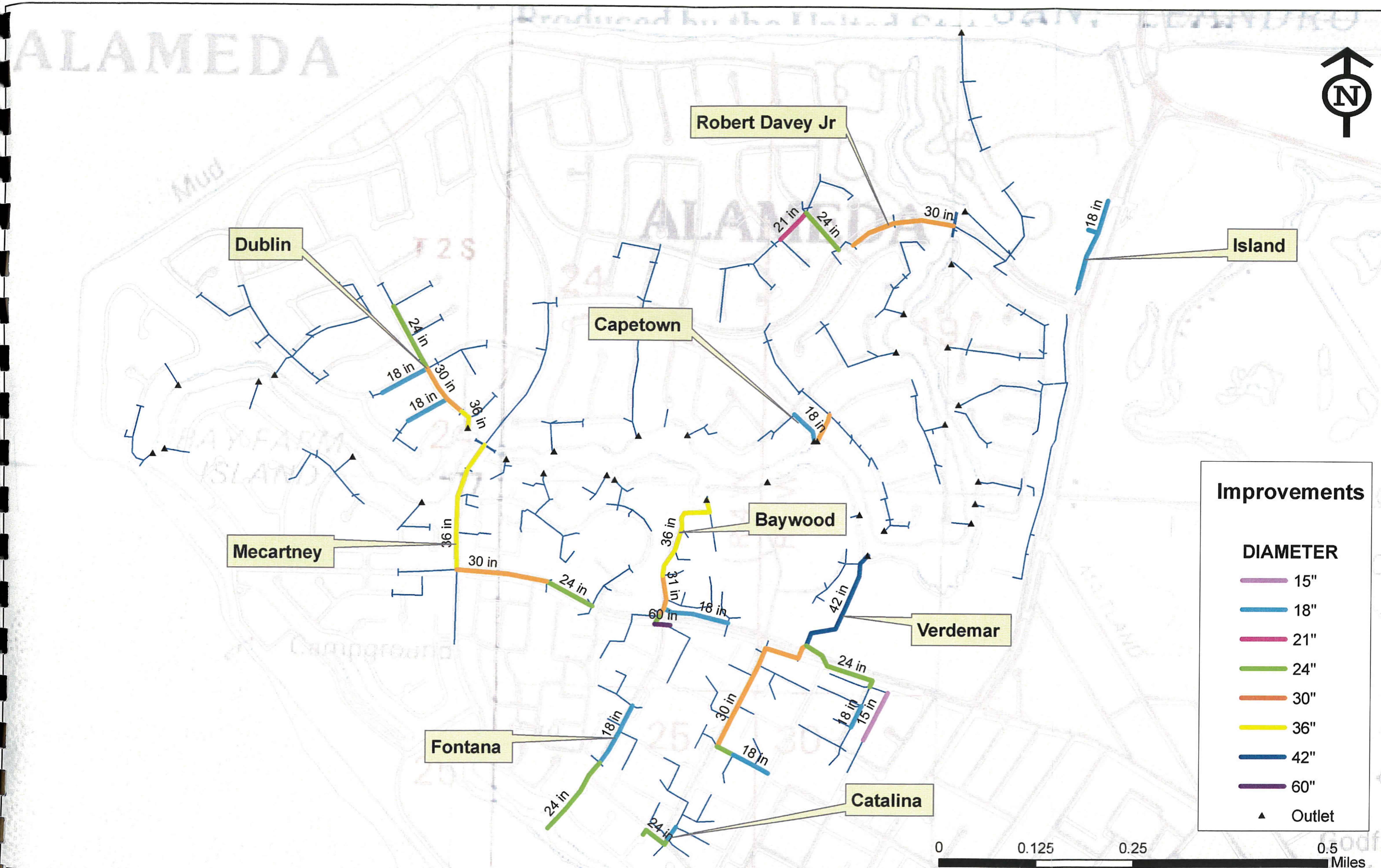


Figure 7-7: Bay Farm Central Area 10-Year Improvement Recommed Diameters

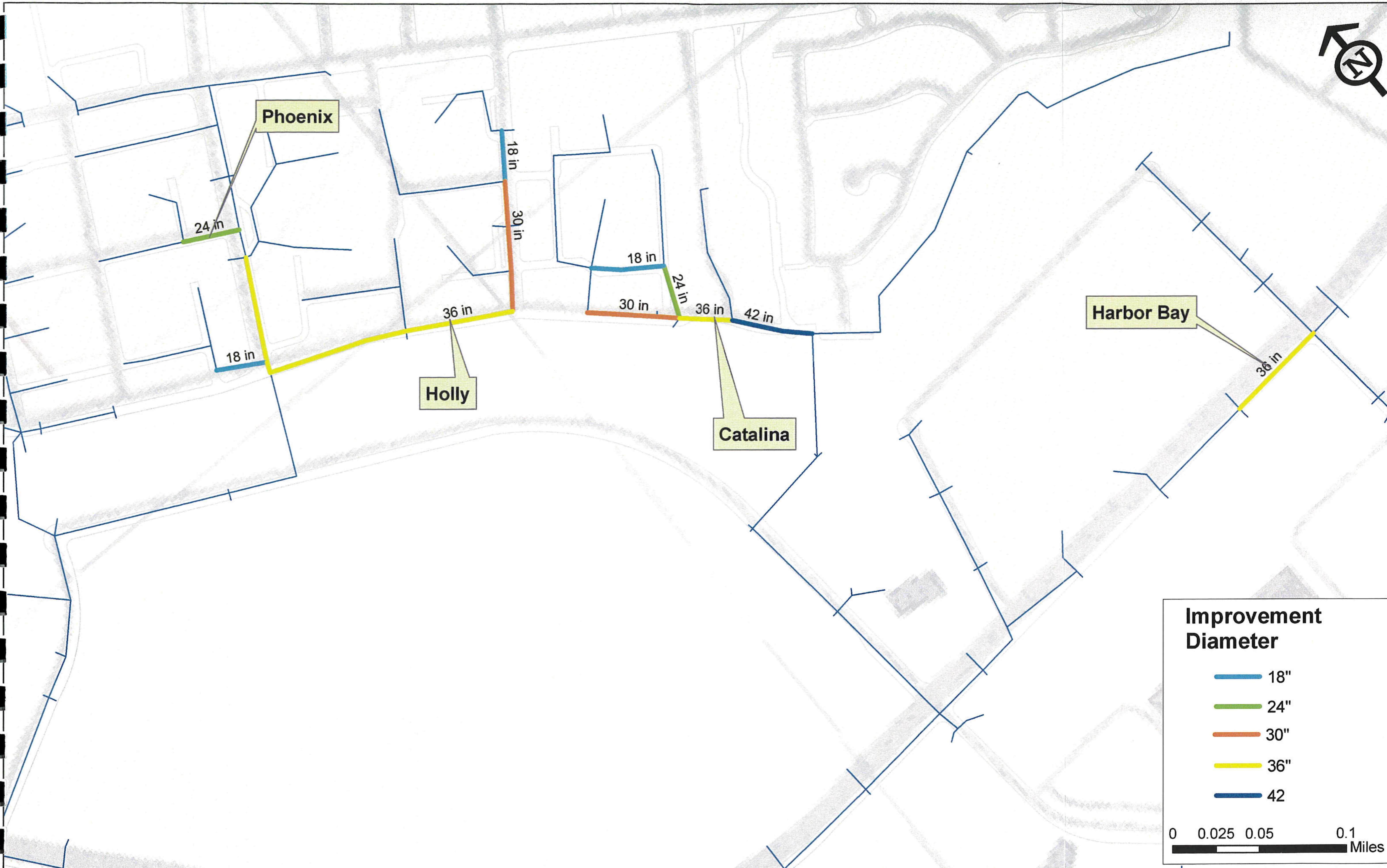


Figure 7-8: Bay Farm South Area 10-Year Improvement Recommended Diameters

CHAPTER 8
MAINTENANCE AND REPLACEMENT

The Master Plan document is not intended as a treatise on operations and maintenance requirements or techniques. (City operations and maintenance staff are the foremost authorities on this subject.) Rather, some foresight is provided into anticipated ongoing maintenance schedules, which include periodic replacement of major storm system components. The age and type of existing storm drain pipe are not analyzed in this study. All modeling assumed that pipes are in good condition and flow is unobstructed; therefore, older storm drain mains should be inspected for functionality.

GENERAL CRITERIA

Table 8-1 presents very general criteria that may be useful in establishing maintenance regimens. Again, City staff will have the best feel for the necessary frequency and extent of ongoing maintenance on a system-by-system basis. Also, maintenance needs will fluctuate depending upon seasonal and annual factors, particularly the amount of precipitation; and to a lesser extent, the general climate.

It is vitally important that all collection, storage, and pumping systems be in working order prior to the start of Alameda's wet season near the end of October. Realizing the limited number of maintenance staff, and the limited number of hours in a year, it is a given that certain items will have higher priorities than others.

Table 8-1: Storm System Maintenance Guidelines

Category	Schedule
Inlet Inspection	annually (summer-fall)
Inlet Cleaning	as required (ongoing)
Storm Drain Pipe Cleaning	annually (ongoing)
Channel Cleaning	annually (fall)
Detention Basin Dredging	every ten years
Pump Exercising	monthly (year round)
Engine Exercising	monthly at full load (year round)
Equipment Lubrication	per manufacturers' recommendations
Drain and fill diesel fuel tank (generators)	every six months
Motor / Engine Control Testing	annually (fall)

COLLECTION SYSTEM MAINTENANCE

The storm drain and channel system cannot function if one of its components is plugged, and whether or not hydraulic analyses say criteria are met, blocked inlets or pipes will cause flooding; with potentially serious consequences. Although even the most rigorous maintenance programs cannot prevent all problems during every storm event, it is important that problems do not accumulate.

Actual maintenance techniques may include grate cleaning, inlet flushing, pipe flushing (hydrojetting), balls and mandrels for cleaning, vactoring, and physically entering storm pipes to remove accumulated debris by hand. The City is responsible for approximately 400,000 lineal feet (75 miles) of underground pipe.

LAGOON MAINTENANCE

Routine removal of mud and debris within open lagoons and ancillary channels maintained by the City of Alameda is necessary to preserve design capacities and function. Visual inspection should be conducted annually for any build-up of mud or debris within channel reaches or underneath any bridge or culvert crossings. Any significant build-up of mud or debris should be removed with a Bobcat or other mechanized means, or manually removed by shovels.

Prior to every flood season in October, City crews should remove any bank vegetation that encroaches beyond each toe of the excavated channel. Emergent wetland vegetation and even dense weeds can be allowed to remain along channel banks where they naturally occur. However, any woody brush or other vegetation that grows below the top of bank should be removed by City personnel during their annual maintenance. The City of Alameda must obtain and keep current any necessary permits from governing jurisdictional agencies.

Lagoons and other storage facilities should be monitored for sediment accumulation and cleaned out as frequently as possible to avoid the emergence of wetlands vegetation that may render future cleaning impossible. Basins are recommended to be cleaned out at least every ten years or whenever sediment deposition approaches minimum operating levels. Avoid accumulation of standing water for extended periods of time to eliminate mosquito breeding concerns.

PUMPING FACILITY MAINTENANCE

Stormwater pump stations are critical to maintain since mechanical or electrical failure can jeopardize system operation. Each pump station should have a bound copy of its site-specific operations and maintenance manual on site; and all personnel need to be familiar with its content. Proper equipment lubrication and maintenance following manufacturers' recommendations (which must be included in the operations and maintenance manual) is essential to efficient operation and longevity, particularly when one considers how infrequent pump operation may be. Pump station control systems should also automatically alternate lead and lag pump status so that each pump within a station operates roughly the same number of hours each year.

Pumps

Stormwater pumps are exposed to harsh pumping conditions and require routine maintenance. Shafts and bearings need to be periodically balanced and/or replaced. The frequency of inspection will vary depending upon the “L-10” bearing life rating of the pump in question. Average bearing life is defined as the operating hours at which half of the group of bearings fails and the rest continue to operate, generally three to five times the L-10 life. Grease is the most maintenance-free bearing lubricant. Other pumps may have drip fee oil systems, which ensure the lowest bearing operating temperatures. The oiling reservoir needs to be checked on a routine basis and topped off as necessary. Submersible pumps should be inspected by a manufacturer’s representative annual to insure cable seal integrity and proper lubrication.

Engines

Manufacturers’ maintenance instructions should be strictly followed, particularly when engines are still under warranty. Maintenance schedules depend somewhat on whether an engine is used as the prime pump driver or is on standby (for power generation). A typical schedule of maintenance based on references provided by Cummins/Onan (Sanks, 1989) is provided as Table 8-2; giving both operating hours and calendar time.

Table 8-2: Typical Maintenance Frequency for Engines and EG-Sets

Maintenance Task	Operating Time	Calendar Time
Inspect fuel, oil level, coolant	8 hours	1 month
Inspect air cleaner, battery	50 hours	1 year
Clean governor linkage, breather, air cleaner	100 hours	1 year
Clean fuel filter, replace oil filter, change crankcase oil, check switchgear	200 hours	1 year
Clean commutator, collector rings, relays, cooling system; inspect brushes, valve clearances, starting and stopping systems, water pump	500 hours	1 year
Check injectors, grind valves (if required), remove carbon, clean oil passages, replace secondary fuel filter, clean generator, grease bearings	1000 hours	----

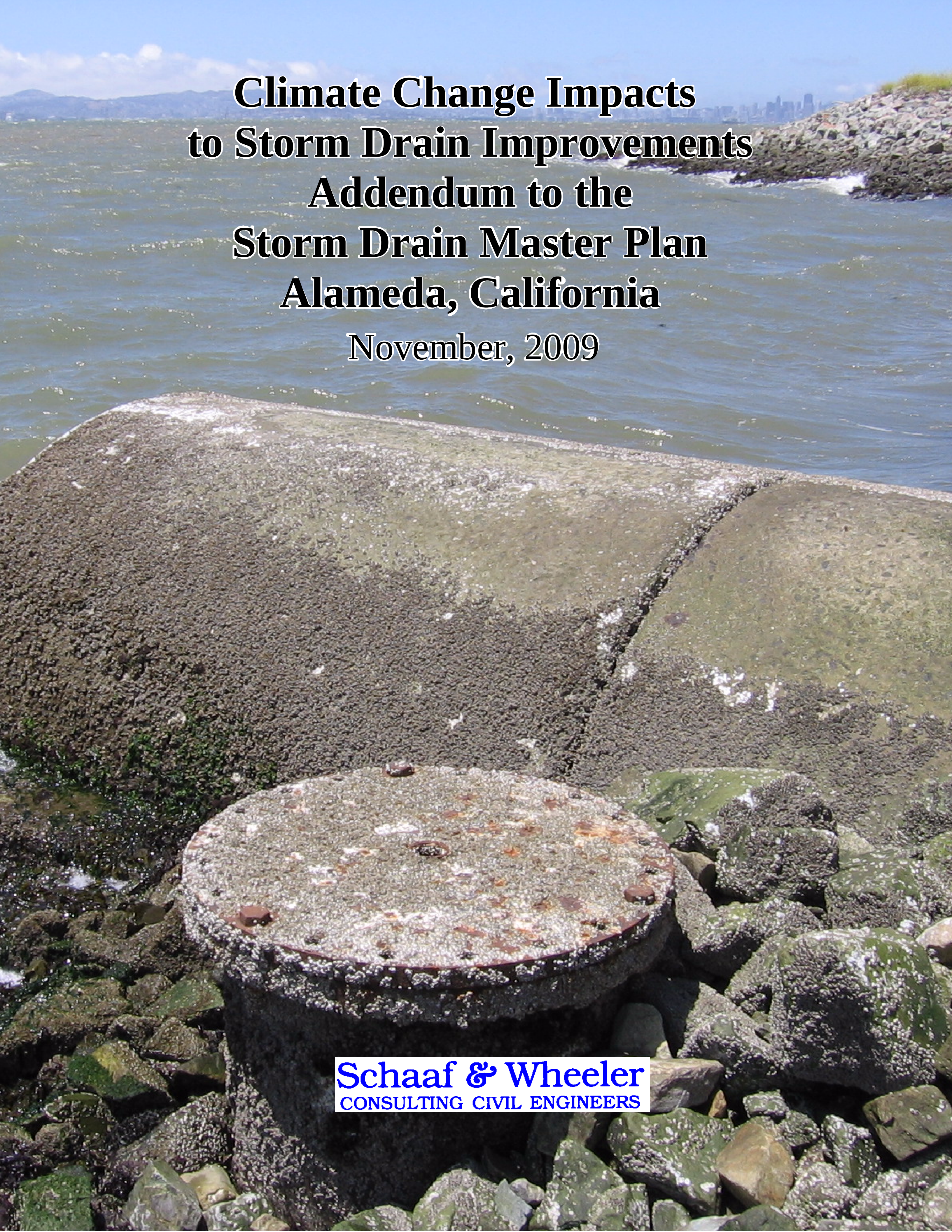
Diesel engines should be operated at full power for at least 15 to 30 minutes after reaching operating temperatures once a month to eliminate carbon deposits where source water makes this possible. Diesel oil is safer to store than most fuels and is easy to obtain and transport, but diesel deteriorates in storage and must be turned over every six months to one year. All maintenance work must comply with water quality standards such as containing lubricants, and other fluids so they do not enter storm drains and the Bay.

SYSTEM REPLACEMENT

With predominantly reinforced concrete pipe, the collection system can be expected to last almost indefinitely. System breaks, joint misalignment, and other problems do occur, of course, so part of the annual maintenance budget should be reserved for periodic pipe repair and replacement. Pump facilities, on the other hand, rely heavily on mechanical and electrical equipment that will wear out and become obsolete over time. On average, pumping equipment can be expected to last anywhere from 30 to 40 years or more with proper maintenance. Structural facilities should last much longer although metal, wood, and even concrete surfaces all require regular care. City maintenance crews need to monitor the condition of these facilities and prepare for system replacement several years in advance. Equipment replacement schedules should be staggered to avoid a large number of simultaneous projects.

Climate Change Impacts to Storm Drain Improvements Addendum to the Storm Drain Master Plan Alameda, California

November, 2009



Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Introduction

Schaaf & Wheeler completed the City of Alameda (City) Storm Drain Master Plan (SDMP) in August, 2008. That report included a brief analysis and discussion of the impacts of sea level rise to City stormwater facilities. In that analysis, a 50-year planning horizon and corresponding 0.5 foot of sea level rise was used. The conclusion was that a half foot of sea level rise applied to the tidal cycle had no significant impacts to the operation of the existing storm drain system.

A more in-depth analysis of sea level rise scenarios was desired by the City. Specifically, the City wished to understand the impacts of a more severe sea level rise scenario on the 10-year improved storm drain system as well as potential inundation of rising sea levels within City limits. This addendum to the City of Alameda SDMP presents the results of these analyses. First, a general background on climate change and sea level rise projections is provided. The current understanding of other potential climate change impacts relevant to the City's flood risks and water resources is also summarized. Next, the impacts of sea level rise assuming an 18 inch rise relative to the City's coastline are presented, including both the risk of inundation within City limits by surrounding water and impacts to the storm drain capacity and operation under this specific sea level rise scenario. Additional improvements to the 10-year improved system are recommended to mitigate these impacts, and cost estimates for the improvements provided. Finally, current regulations, policies, and actions related to climate change from state and local organizations are summarized.

It should be noted that this report does not attempt to detail the specific causes of climate change, nor the distribution between anthropogenic (i.e. human induced) versus natural sources of carbon dioxide in the atmosphere. The purpose of this report is to detail the potential impacts of a specific climate change scenario to Alameda flooding risks, both in magnitude and uncertainty, and discuss conceptual and master planning level mitigation activities. Mitigation activities discussed herein focus on mitigating the impacts of global warming to flood risk within the City rather than mitigating carbon emissions.

Current Status of Climate Change Understanding and Research

It is well understood that carbon dioxide and other anthropogenic green house emissions act as heat trapping greenhouse gasses, which increase troposphere temperatures. Throughout the 1980s, scientists began to note increases in these emissions and postulated that the increases in atmospheric carbon dioxide may cause a range of impacts, some of which may be adverse. Climate change refers to an identifiable change in the state of the climate that persists for an extended period of time. The use of the phrase

‘climate change’ does not necessarily distinguish whether changes are due to natural processes versus human activity. Climate variability, however, refers to natural climate cycles or changes that are not caused by human activities. Many of the impacts of climate change occur quite slowly. Thus, even if carbon emissions are stabilized or greatly reduced in coming years, some impacts such as sea level rise will continue to occur, albeit potentially at a slower pace than predicted by most global climate change models.

As awareness of climate change spreads, an increasing number of analyses are conducted and reports published every year. Tasked with gathering, reviewing, and synthesizing the multitude of published studies is the Intergovenmental Panel on Climate Change. In addition to this international organization, this report summarizes the current understanding reflected in reports produced by the United States Army Corps of Engineers and by departments within the state of California.

Intergovernmental Panel on Climate Change

The Intergovernmental Panel on Climate Change (IPCC) was established in 1988 to provide an objective source of information about climate change. The IPCC does not independently conduct research or gather data. Instead it acts as a comprehensive assessor of the latest scientific, technical, and socio-economic literature produced worldwide relevant to the understanding of human-induced climate change, its impacts, and mitigation strategies. The IPCC was set up by the World Meteorological Organization and by the United Nations Environment Programme.

The First Assessment Report was released by the IPCC in 1990, the Second in 1995, the Third in 2001, and the Fourth in 2007. The conclusion that human induced climate change is occurring has been progressively more certain in each Assessment Report, with the 2007 Assessment Report stating that there is *very high confidence* (at least 9 out of 10 chance of being correct) that the global average net effect of human activities since 1750 has been one of warming, and that human induced warming over the last three decades has *likely* (greater than 66% probability) had a discernible influence at the global scale. Global warming refers to the general warming of the climate system, and the fact that global warming is occurring is unequivocal, based on IPCC findings. The next IPCC Assessment Report is scheduled for publication in 2012.

Uncertainty and Scale

IPCC uses a system of self-explanatory terms to convey qualitative and quantitative uncertainty. Three approaches are used to describe uncertainty. Where uncertainty is

assessed qualitatively, a relative sense of the amount and quality of evidence to support a statement is provided through use of terms such as: high agreement, much evidence; high agreement, medium evidence; medium agreement, medium evidence; etc. Where uncertainty is assessed quantitatively using expert judgment of the correctness of underlying data or analyses, a scale of confidence levels is used to express the assessed change of a finding being correct: very high confidence (at least 9 out of 10); high confidence (about 8 out of 10); medium confidence (about 5 out of 10); low confidence (about 2 out of 10); and very low confidence (less than 1 out of 10). Finally, where uncertainty in specific quantitative outcomes is assessed using expert judgment and statistical analysis, then likelihood ranges are used to express the probability of occurrence: virtually certain (>99%); extremely likely (>95%); very likely (>90%); likely (>66%); more likely than not (>50%); about as likely as not (33%-66%); unlikely (<33%); very unlikely (<10%); extremely unlikely (<5%); and exceptionally unlikely (<1%) (IPCC, 2007). Throughout this report, when these phrases are used based on IPCC findings they have been italicized as a visual reminder of this paragraph.

There are several global climate models that have been developed to estimate future impacts of climate change and global warming. Within each model there are various future condition scenarios representing the range of potential future carbon dioxide and other greenhouse gas emission levels. The more conservative approach is to assume that these emissions increase at a rate equal to or greater than recent trends. Generally the emissions and global warming predictions and impacts are directly proportional – the greater the emissions, the more severe the warming trend.

The vast majority of climate models are global in scale, and although general trends and impact estimates may be concluded from these models, there are multiple issues encountered when trying to downscale either results or models to determine trends or impacts in a localized area. The IPCC has produced a Special Report on the Regional Impacts of Climate Change which analyzes impacts at a continental or sub-continental scale; however this report focuses on impacts due to regional vulnerabilities as opposed to regional differences in physical impacts. Efforts to downscale from the global climate model to the catchment scale for hydrologic analyses and to utilize regional climate models to drive hydrologic models have shown that different ways of creating regional scenarios from the same source can lead to substantial differences in the estimated regional effect of climate change and that errors in the modeling procedure or differences in climate models are greater than hydrologic model uncertainty (Kundzewicz, 2007).

There is no single agreed upon methodology for downscaling climate change results for use in regional hydrology, and results may differ substantially depending on the source model and method used. The process of downscaling does not resolve any of the

uncertainty inherent in global climate models, and introduces new sources of uncertainty such that overall trends are less well defined compared to global models. For example, depending on the global climate model and scaling methodology used the estimated range of impact to mean annual precipitation in California varies in both magnitude and sign by at least 10% (Dettinger, 2004). What this means is that while global climate change trends are relatively well known and documented, regional and local trends, particularly hydrologic parameters such as rainfall and runoff, are less well known.

<u>California Climate Action Team</u>

The California Climate Action Team (CAT) was established by Governor Schwarzenegger under an Executive Order on June 1, 2005. The purpose of the CAT is to coordinate state-level actions relating to Climate Change. The Team is led by the Secretary of the California Environmental Protection Agency and includes the Secretary of the Business, Transportation and Housing Agency, Secretary of the Department of Food and Agriculture, Secretary of the Resources Agency, Chairperson of the Air Resources Board, Chairperson of the Energy Commission and President of the Public Utilities Commission. The Climate Action Team is charged with implementing global warming emission reduction programs and reporting on the progress made toward meeting the statewide greenhouse gas targets that were established in the Assembly Bill 32 (described in more detail later in this report). The first report was sent to the Governor and the Legislature in 2006, and should be updated bi-annually thereafter.

<u>California Climate Change Center</u>

The California Energy Commission's Public Interest Energy Research (PIER) Program conducts public interest research, development, and demonstration projects to benefit California's electricity and natural gas ratepayers. In 2003, the California Energy Commission's PIER Program established the California Climate Change Center (CCCC) to document climate chance research relevant to the states. The CCCC Report Series details ongoing center-sponsored research on climate change predictions and impact analyses. All of the final CCCC reports include a preface which clarifies that the findings presented are interim project results, and information contained within the reports is subject to change.

Global Warming Impacts

The IPCC range of best estimate *likely* temperature increases by the year 2099 is 0.6 – 4.0 degrees Celsius (1 – 7 degrees Fahrenheit), depending on the global climate model utilized (IPCC, 2007). Regionally, scaled down climate models for northern California

estimate global temperature increases up to 4.5 degrees Celsius (9 degrees Fahrenheit) by 2100 (Cayan, 2007). An increase in global temperatures in the IPCC range may have multiple impacts on the water resources of the City of Alameda, even if the changes in local and regional temperature are not yet known.

Sea Level Rise

One of the most publicized impacts of global warming, and the impact with the most direct consequences to the City of Alameda, is sea level rise. Sea level rise can be defined as global or relative. Global sea level rise is defined as the increase of global average sea level. Throughout the world, land may be uplifting or subsiding. This will impact the relative change in depth of water at any given location, depending on the rate of movement compared to the rate of global sea level rise. In addition, coastal bays such as the San Francisco Bay may not experience sea level rise at the same rate as the global average. Relative sea level rise refers to the rise of sea levels accounting for local hydraulics, land uplifting or subsidence.

An example of the importance of global vs. relative sea level rise can be seen when examining the historic sea level trends in San Francisco Bay at the National Ocean and Atmospheric Administration (NOAA) gages for San Francisco (at the Presidio) and Alameda (Pier 3 at the Naval Air Station). The Alameda gage shows a long term average mean sea level rise of 0.82 millimeters per year (NOAA, Alameda Mean Sea Level Trend), while the San Francisco gage long term average mean sea level rise is 2.01 millimeters per year (NOAA, San Francisco Mean Sea Level Trend). Although the San Francisco gage period of record is longer, essentially the same rate of sea level rise is found if it is truncated to match the Alameda gage period of record. The reasons for this difference are unknown, and likely due to a combination of factors, but it serves to exemplify the complexity between local trends, global predictions, and site specific hydraulics.

IPCC Sea Level Rise Estimates

Depending on the emission scenario used, the predicted *likely* global sea level rise ranges from 0.18 – 0.59 meters (IPCC 4^{th} Assessment Report), or 0.6 – 1.9 feet by the year 2099. IPCC reports do not provide mid-range estimates; e.g. sea level rise by 2050. The upper limit of this range is lower than the upper range stated in previous IPCC reports. The two primary factors affecting global sea level rise are thermal expansion of ocean waters due to increased atmospheric temperature, and melting ice. The IPCC estimates that of the global sea level rise that has occurred since 1993, thermal expansion of the ocean has contributed 57% of the total rise, decreases in the extent of glaciers and ice caps have

contributed 28%, and the remaining 15% is due to losses from the polar ice sheets. It must be noted that this range does not include uncertainties in climate-carbon cycle feedbacks or the full effect of changes to ice sheet flow, because a basis in published literature is lacking. Thus these values do not represent an upper bound to projected sea level rise. Long term projections show that global warming sufficient to eliminate the Greenland Ice Sheet (one millennium exposed to an average temperature rise in excess of 1.9 – 4.6 degrees Celsius) results in an additional seven meters (23 feet) of global sea level rise. The IPCC does not offer any uncertainty scale for this possibility.

United States Army Corps of Engineers Sea Level Rise Estimates

The United States Army Corps of Engineers (USACE) published an engineering circular (USACE, 2009) to direct the consideration of sea level rise estimates in project planning and design. While this methodology is required only for USACE civil work activities, it offers a valuable guidance for any planning effort. In summary, the USACE report recommends that the planning, engineering and designing for projects within the tidal zone or with downstream tidal boundary conditions consider how sensitive and adaptable the project is to a range of sea level rise estimates (low, intermediate and high). Specifically, the USACE directs determination of “how sensitive alternative plans and designs are to these rates for future local mean sea-level change, how this sensitivity affects calculated risk, and what design of operations and maintenance measures should be implemented to minimize adverse consequences while maximizing beneficial effects”.

The “low” sea level rise estimate recommended by the USACE report is based on local historic tide gauges. In San Francisco, the Presidio tide gauge has the longest period of record and is consistently used for historic sea level trends in San Francisco Bay. For consistency with regional documents the Presidio gauge is used for calculations herein, although the Alameda gauge records described above may be more appropriate for the City. The long term average sea level rise at the Presidio gauge is 2.01 millimeters per year (mm/yr), with a 95% confidence limit of plus or minus 0.21 mm/yr (NOAA, Station 9414290). “Intermediate” and “high” sea level rise estimates are based on the National Resource Council (NRC) curves and equations developed for a 1987 Report (*Responding to Changes in Sea Level: Engineering Implications*), modified to account for the updated annual estimate of sea level rise made in the 2007 IPCC report, and manipulated to include consideration of the date of the equation development. The “intermediate” sea level rise projection is based on the modified NRC Curve I, and the “high” sea level rise projection on the modified NRC Curve III. This equation is:

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)$$

where:

t_1 = time between construction date and 1986;

t_2 = time between date at which sea level rise projection is desired and 1986;

$E(t)$ = eustatic sea-level rise, in meters, as a function of (t);

b = Variable, 2.36E-5 for modified NRC Curve I, 1.005E-4 for modified NRC Curve III.

Table 1 presents the range of sea level rise projects for the City of Alameda using this methodology, assuming adoption of the Presidio gauge for the local historic sea level trend, and construction of any given project in 2010.

Table 1: Range of Sea Level Rise Projections Using USACE Methodology with Presidio Gage and 2010 Construction Year

USACE Methodology Sea Level Rise Projection Range (feet)			
Year	**Low**	**Intermediate**	**High**
2025	0.1	0.2	0.4
2050	0.3	0.5	1.4
2075	0.4	0.9	2.8
2100	0.6	1.5	4.6

California Climate Change Center Sea Level Rise Estimates

A draft version of the *Impacts of Sea-Level Rise on the California Coast*, developed by The Pacific Institute for the CCCC was released in March, 2009, with much publicity of the new 2100 sea level rise estimate of "5 feet" (Chronicle article, March 12, 2009). The development of this sea level rise estimate is presented in somewhat more detail, however, in the *Climate Change Scenarios and Sea Level Rise Estimates for the California 2009 Climate Change Scenarios Assessment* Report (Cayan, 2009), also produced for the CCCC. In short, the sea level rise estimates adopted by the CCCC are based on an empirical formula developed by Rahmstorf (2007) which relates global mean sea level rise to global mean surface air temperature. The report states (and shows graphically) that the Rahmstorf predicted values are then manipulated to include the impact of reservoirs and dams, but exactly what this manipulation entails, and its justification, is unclear. The supporting article cited as the basis of this manipulation, *Impact of Artificial Reservoir Water Impoundment on Global Sea Level* (Chao, 2008),

appears to focus on the impact of reservoir and dam storage to historic sea level trends, and Schaaf & Wheeler was unable to locate any published article which details a modified Rahmstorf method.

Using the above methodology, the 2009 Assessment Report gives a range of sea level rise of 30-45 cm (12 – 18 inches) by 2050 (relative to 2000 levels). Although other CCCC reports, as well as the San Francisco Bay Conservation and Development District, have adopted a 2100 sea level rise projection of 1.4 meters (4.6 feet), this projection is not explicitly stated in the text of the 2009 Assessment Report (it can only be deduced from included graphs). It should be noted that the range of sea level rise estimates produced from this methodology is about 0.6 m – 1.45 m (2.0 – 4.8 feet). The 4.6 feet of rise by 2100 predicted at the upper end of this range is similar to the USACE methodology high range for 2100 for San Francisco Bay, as shown in Table 1.

Sea Level Rise Estimates Summary

In summary, significant uncertainties remain in sea level rise projections, particularly as one forecast's farther into the future. <u>The most current available estimates for sea level rise by 2050 range from 0.3 foot to 1.5 feet, and by 2100 from 0.6 foot – 4.8 feet.</u> Confidence in any sea level rise prediction decreases the further into the future that analysis is projected, due to unknowns about future emission scenarios, potential climate feedback loops and the severity of melting ice. It is important to note that emphasis should not be placed on a particular specific value for sea level rise. Not only is a consensus on a particular value unlikely, but the selection of the year 2100 as a reporting point for sea level rise projections is arbitrary. Even with drastic reductions in carbon emissions sea levels are expected to continue to rise beyond 2100 due at least to continued thermal expansion of ocean waters. Thus, any planning for sea level rise impacts should recognize the inherent uncertainty and long term ongoing nature of these projections.

Rising sea levels have two potential impacts to the City: inundation of Bay water onto City lands and impacts to the operation and performance of City storm drain facilities. Each of these impacts is discussed in more detail below.

<u>Other Climate Change Impacts</u>

Climate change has many predicted impacts in addition to sea level rise. Below, other climate change impacts which may adversely affect flooding risk of the City of Alameda are described. These impacts are: storm surge, wave runup, and precipitation.

Storm Surge

During storm events, ocean water increases in elevation due to low barometric surface pressure. This phenomenon is called storm surge. The FEMA 1% storm surge for San Francisco Bay at Alameda is 7 feet NGVD, compared to a mean high-high tide of 3.7 feet NGVD (NOAA, Alameda Datums). This represents a 1% surge of 3.3 feet. It is *likely* that the incidence of extreme high sea level has increased at a broad range of sites wordwide since 1975. Extreme high sea level is defined as the highest 1% of hourly values of observed sea level at a station for a given reference period (IPCC, 2007).

Pronounced multi-year fluctuations of San Francisco non-tidal residuals (NTR; total water elevations above tidal elevations – for San Francisco Bay NTRs are primarily storm surge and wind driven waves) are evidenced in historical records and no significant changes in the mean monthly positive NTRs exist between 1858 and 2000. However when considering only the highest 2% of extreme winter NTRs there has been a significant increasing trend since about 1950 (Bromirski, 2003). This increased 'storminess' may be part of a larger cycle, but it suggests a relationship between global climate warming and overall storminess on the west coast.

The occurrence of hourly observed high sea levels (above the 99.99th percentile thresholds) in San Francisco Bay has increased sharply since 1969. The maximum observed sea level has also increased since that time, although the period of 1987-2004 had a slightly lower peak sea level than 1969-1987. Recent studies have concluded that if sea level rise is on the lower end of the current predicted ranges, the occurrence of extremely high sea level events will increase, but the increase in extremes would be not so different from the increasing trend that has been seen in California for the past several decades. If, however, sea level increases reach the higher end of the range, extreme events would increase not only in their frequency but also their duration, substantially beyond the historic trend seen in the 19th and 20th centuries (Cayan, 2007).

In short, it is expected that as sea levels rise, not only will the occurrence of high sea level, or surge, events increase, but so may the amount of surge itself (currently about 3.3 feet above mean high high water in Alameda). This increased storm surge elevation may impact flood risk, backwater conditions and storm water pump station operation; however quantitative estimates for the increased storm surge have not been made, and are unlikely to be determined in the near future.

Wave Runup

Wave runup is the elevation wind-driven waves will reach as waves break on land and may be affected by global warming. However, these impacts are not particularly well understood at this time. A review of recently published literature finds that different published studies come to different, and at times directly opposing, conclusions regarding likely climate change impacts to wave energy. Wave heights are greatly influenced by local conditions, likely a major cause for the differing results found in the available literature. Some general trends are well understood, such as that extreme wave heights and surge fluctuations tend to increase from the south to the north along California Coast, as a result of increasing storm intensities along the northern coast (Cayan, 2007).

Wave runup is a function of water depth, wind speed and direction, and the features of the land on which the wave is breaking (slope, roughness, etc.). In some parts of San Francisco Bay, rising sea levels will inundate low lying marshes, creating broad, but shallow, flooded areas. In this scenario, wave runup will likely decrease, as the shallow water will dampen wave heights. In Alameda however, which is generally protected by high land, rising sea levels will create deeper water surrounding the City, potentially resulting in increased wave heights and runup.

Published literature has found that when short term sea level is highest (i.e. during storm surge events), wave energy has an increased likelihood of reaching very high levels. The peak likely significant wave height (the average height of the one third highest waves) increases by 2.5 meters in one scenario where the surge value increased from 4 centimeters (cm) to 30 cm (Cayan, 2007). Thus in that particular scenario, as the storm surge increases, so does wave energy and height, which in turn may increase wave runup. That said, recent downscaled models have also indicated that the incidence of large coastal storms will lessen as part of the overall drying trend (discussed in more detail in the precipitation section below), resulting in a marginal decrease in the wind wave energy reaching California's coast as well as a decreasing trend for significant wave heights (Cayan, 2009). In short, although climate change is expected to impact storm surge and wave runup, these impacts (or even the trend of impacts) is not well understood at this time, and in any event, these impacts are expected to be dwarfed by the impact of increasing mean sea level.

The Bay floor near Alameda is largely composed of Bay mud, a thick deposit of soft, unconsolidated silty clay, which is saturated with water. One potential mitigation action against increased wave height due to deepening water would be to fill to maintain existing water depths. In addition to the multitude of permitting and environment issues with this activity, however, Bay Mud has a very high compressibility. In other words,

Bay Mud will continue to compress even when large volumes or weights are set on it. Thus filling on top of Bay Mud is ineffectual, and when additional environmental impacts are considered with the uncertainty of wave height and runup impacts, not a feasible mitigation alternative for Alameda to offset increased wave heights and runup.

Precipitation

It is *likely* that the frequency of heavy precipitation events (or proportion of total rainfall from heavy storms) has increased over most areas (IPCC, 2007). Global analyses of precipitation from 1901-2005 do not show statistically significant trends due to many discrepancies between data sets and the variability of precipitation in both space and time (Bates, 2008). Likewise, there is no consensus among regional climate models as to how mean annual precipitation totals might change in the United States (Dettinger, 2004), although most recent global and regional models predict that total mean precipitation will modestly decrease (5-20%) in the latter half of the next century (Hayhoe, 2004; Cayan, 2007, Draft 2009). Long term historic analyses of precipitation in the state of California show that there is no statistically significant change in total annual mean precipitation from 1890 through 2000, although the variability of total rainfall in any given year appears to have an increasing trend (DWR, 2006).

While the total mean annual precipitation is not predicted to change significantly, the timing and intensity of storm events is expected to change, with a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events Climate models predict (and historic records reflect) that proportionally less rainfall will fall during spring and summer months (April – July) and more in winter months (November – March) in northern California due to global climate change (Dettinger, 2004; Cayan 2007; DWR 2006). These shifts in precipitation timing and intensity may have impacts on flooding and water supply.

The most updated *Climate Change Scenarios* report (Cayan, 2009) states that the occurrence of significant storms declines at least marginally and that the occurrence of high daily precipitation events generally remains about the same through 2100 as it does in the historical projections. It should be noted that this conclusion is markedly different from previous conclusions by the same authors, which predicted a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events (Bates, 2008; Cayan 2007). Several CCCC reports reviewed for this analysis repeat the earlier, and presumably outdated, conclusion.

In summary, while a small decrease in annual precipitation is forecast, the trend in number and magnitude of large precipitation events is unknown. The most current

studies reviewed for this analysis both conflict previous conclusions and other updated studies, further exemplifying that there is no consensus regarding the potential impacts of climate change on the frequency or magnitude of large storm events.

Sea Level Rise Impacts to the City of Alameda

The effects of climate change described above have potential impacts to virtually all water resources within the City of Alameda, including not only local flood control and risk but also regional impacts to sectors such as agricultural and water supply. This report focuses on how rising sea levels may impact the risk of flood inundation of the City from its surrounding waters, and the impact of rising sea levels to the 10-year improvements previously made in the City SDMP. For this analysis, 18 inches (1.5 foot) of sea level rise was assumed. This represents the upper bound of the range of the most recently published sea level rise projections by year 2050 (Cayan, 2009).

When discussing projects to mitigate the impacts of sea level rise, there are several important points to keep in mind. As described above, there is not currently and unlikely to ever be a true consensus in the prediction of sea level rise, particularly a consensus on a projection 100 years into the future. A planning horizon of 100 years is not only far beyond most planning timelines typical to public agencies, but it is also beyond the typical useful life of structural flood protection elements. In other words, even if it were financially feasible to construct a project today to protect for a sea level rise scenario in 2100, it may not be advisable to do so, since that project could be structurally unsound by the time it was needed. Finally, it should be noted that although currently the year 2100 is the most common projection date, sea levels are expected to continue to rise beyond the year 2100.

<u>Inundation due to Rising Waters</u>

As an island community, Alameda is uniquely vulnerable to rising water levels in San Francisco Bay. Currently, Alameda is protected from inundation from its surrounding waters primarily by high ground, as opposed to floodwalls or levees. Interior lagoons are hydraulically connected to the surrounding waters via weir inlets, pumps, or gated outlets.

Figures 1 through 6 reflect City-wide recent topographic data adjusted to show three elevations of interest: existing mean sea level, mean sea level with 18" increase, and the highest tide elevation for various storm events with 18" of sea level rise added. The storm specific tide cycles were developed for the SDMP and the methodology and results of that process are described in detail in that report. It should be noted that these figures

do not take into account potential flood protection of naturally occurring high ground or existing flood control facilities. In other words, a shaded area represents an elevation range only, and does not necessarily mean that surrounding water will be able to reach and pond in all of those locations. One good example of this is shown in Figure 2, which reflects the fact that much of the golf course is below mean sea level. This does not mean, however that the golf course is always inundated with surrounding waters, due to existing high ground and storm drain facilities. That said, the lack of flap gates on many storm drain outlets may allow for backwater due to high tides to reach interior locations of the City. Figures 7 and 8 translate the water surface elevation into depth of water for the most severe (100-year event) scenario. Again, these figures represent potential risk areas without consideration of existing natural or man made protection measures.

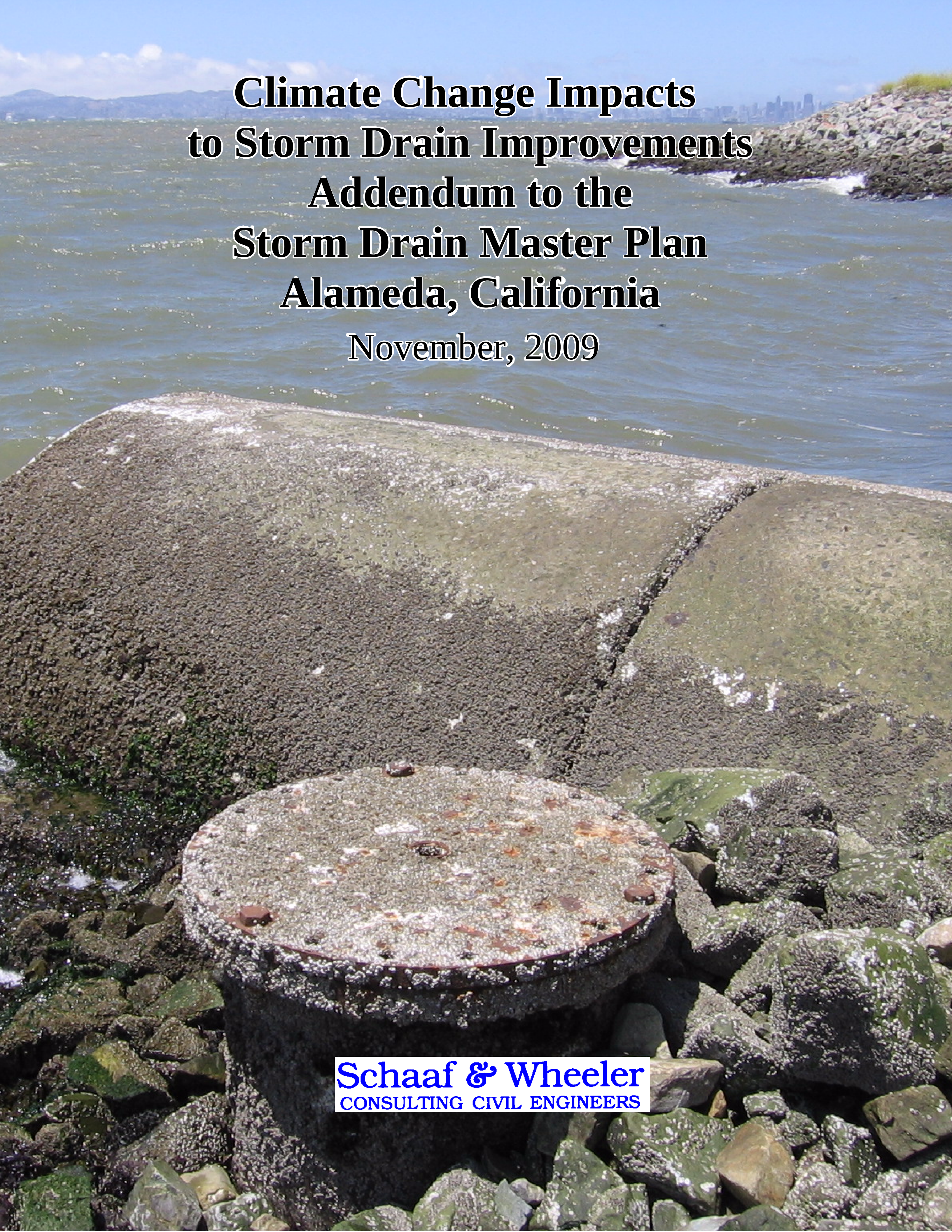
Table 2 summarizes the existing and sea level rise scenario mean and high tide levels reflected in Figures 1 through 8. Storm specific tide cycles were developed for the SDMP, and a more complete description of the methodology for that process can be found in the SDMP, Chapter 3.

Table 2: Mean and High Tide Elevations for Existing and Sea Level Rise Scenario

	Existing (NGVD)	Sea Level Rise (18") Scenario (NGVD)
Mean Sea Level	0.5'	2.0'
10-Year High Tide	5.1'	6.6'
25-Year High Tide	5.4'	6.9'
100-Year High Tide	6.2'	7.7'

Climate Change Impacts to Storm Drain Improvements Addendum to the Storm Drain Master Plan Alameda, California

November, 2009



Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Introduction

Schaaf & Wheeler completed the City of Alameda (City) Storm Drain Master Plan (SDMP) in August, 2008. That report included a brief analysis and discussion of the impacts of sea level rise to City stormwater facilities. In that analysis, a 50-year planning horizon and corresponding 0.5 foot of sea level rise was used. The conclusion was that a half foot of sea level rise applied to the tidal cycle had no significant impacts to the operation of the existing storm drain system.

A more in-depth analysis of sea level rise scenarios was desired by the City. Specifically, the City wished to understand the impacts of a more severe sea level rise scenario on the 10-year improved storm drain system as well as potential inundation of rising sea levels within City limits. This addendum to the City of Alameda SDMP presents the results of these analyses. First, a general background on climate change and sea level rise projections is provided. The current understanding of other potential climate change impacts relevant to the City's flood risks and water resources is also summarized. Next, the impacts of sea level rise assuming an 18 inch rise relative to the City's coastline are presented, including both the risk of inundation within City limits by surrounding water and impacts to the storm drain capacity and operation under this specific sea level rise scenario. Additional improvements to the 10-year improved system are recommended to mitigate these impacts, and cost estimates for the improvements provided. Finally, current regulations, policies, and actions related to climate change from state and local organizations are summarized.

It should be noted that this report does not attempt to detail the specific causes of climate change, nor the distribution between anthropogenic (i.e. human induced) versus natural sources of carbon dioxide in the atmosphere. The purpose of this report is to detail the potential impacts of a specific climate change scenario to Alameda flooding risks, both in magnitude and uncertainty, and discuss conceptual and master planning level mitigation activities. Mitigation activities discussed herein focus on mitigating the impacts of global warming to flood risk within the City rather than mitigating carbon emissions.

Current Status of Climate Change Understanding and Research

It is well understood that carbon dioxide and other anthropogenic green house emissions act as heat trapping greenhouse gasses, which increase troposphere temperatures. Throughout the 1980s, scientists began to note increases in these emissions and postulated that the increases in atmospheric carbon dioxide may cause a range of impacts, some of which may be adverse. Climate change refers to an identifiable change in the state of the climate that persists for an extended period of time. The use of the phrase

‘climate change’ does not necessarily distinguish whether changes are due to natural processes versus human activity. Climate variability, however, refers to natural climate cycles or changes that are not caused by human activities. Many of the impacts of climate change occur quite slowly. Thus, even if carbon emissions are stabilized or greatly reduced in coming years, some impacts such as sea level rise will continue to occur, albeit potentially at a slower pace than predicted by most global climate change models.

As awareness of climate change spreads, an increasing number of analyses are conducted and reports published every year. Tasked with gathering, reviewing, and synthesizing the multitude of published studies is the Intergovenmental Panel on Climate Change. In addition to this international organization, this report summarizes the current understanding reflected in reports produced by the United States Army Corps of Engineers and by departments within the state of California.

Intergovernmental Panel on Climate Change

The Intergovernmental Panel on Climate Change (IPCC) was established in 1988 to provide an objective source of information about climate change. The IPCC does not independently conduct research or gather data. Instead it acts as a comprehensive assessor of the latest scientific, technical, and socio-economic literature produced worldwide relevant to the understanding of human-induced climate change, its impacts, and mitigation strategies. The IPCC was set up by the World Meteorological Organization and by the United Nations Environment Programme.

The First Assessment Report was released by the IPCC in 1990, the Second in 1995, the Third in 2001, and the Fourth in 2007. The conclusion that human induced climate change is occurring has been progressively more certain in each Assessment Report, with the 2007 Assessment Report stating that there is *very high confidence* (at least 9 out of 10 chance of being correct) that the global average net effect of human activities since 1750 has been one of warming, and that human induced warming over the last three decades has *likely* (greater than 66% probability) had a discernible influence at the global scale. Global warming refers to the general warming of the climate system, and the fact that global warming is occurring is unequivocal, based on IPCC findings. The next IPCC Assessment Report is scheduled for publication in 2012.

Uncertainty and Scale

IPCC uses a system of self-explanatory terms to convey qualitative and quantitative uncertainty. Three approaches are used to describe uncertainty. Where uncertainty is

assessed qualitatively, a relative sense of the amount and quality of evidence to support a statement is provided through use of terms such as: high agreement, much evidence; high agreement, medium evidence; medium agreement, medium evidence; etc. Where uncertainty is assessed quantitatively using expert judgment of the correctness of underlying data or analyses, a scale of confidence levels is used to express the assessed change of a finding being correct: very high confidence (at least 9 out of 10); high confidence (about 8 out of 10); medium confidence (about 5 out of 10); low confidence (about 2 out of 10); and very low confidence (less than 1 out of 10). Finally, where uncertainty in specific quantitative outcomes is assessed using expert judgment and statistical analysis, then likelihood ranges are used to express the probability of occurrence: virtually certain (>99%); extremely likely (>95%); very likely (>90%); likely (>66%); more likely than not (>50%); about as likely as not (33%-66%); unlikely (<33%); very unlikely (<10%); extremely unlikely (<5%); and exceptionally unlikely (<1%) (IPCC, 2007). Throughout this report, when these phrases are used based on IPCC findings they have been italicized as a visual reminder of this paragraph.

There are several global climate models that have been developed to estimate future impacts of climate change and global warming. Within each model there are various future condition scenarios representing the range of potential future carbon dioxide and other greenhouse gas emission levels. The more conservative approach is to assume that these emissions increase at a rate equal to or greater than recent trends. Generally the emissions and global warming predictions and impacts are directly proportional – the greater the emissions, the more severe the warming trend.

The vast majority of climate models are global in scale, and although general trends and impact estimates may be concluded from these models, there are multiple issues encountered when trying to downscale either results or models to determine trends or impacts in a localized area. The IPCC has produced a Special Report on the Regional Impacts of Climate Change which analyzes impacts at a continental or sub-continental scale; however this report focuses on impacts due to regional vulnerabilities as opposed to regional differences in physical impacts. Efforts to downscale from the global climate model to the catchment scale for hydrologic analyses and to utilize regional climate models to drive hydrologic models have shown that different ways of creating regional scenarios from the same source can lead to substantial differences in the estimated regional effect of climate change and that errors in the modeling procedure or differences in climate models are greater than hydrologic model uncertainty (Kundzewicz, 2007).

There is no single agreed upon methodology for downscaling climate change results for use in regional hydrology, and results may differ substantially depending on the source model and method used. The process of downscaling does not resolve any of the

uncertainty inherent in global climate models, and introduces new sources of uncertainty such that overall trends are less well defined compared to global models. For example, depending on the global climate model and scaling methodology used the estimated range of impact to mean annual precipitation in California varies in both magnitude and sign by at least 10% (Dettinger, 2004). What this means is that while global climate change trends are relatively well known and documented, regional and local trends, particularly hydrologic parameters such as rainfall and runoff, are less well known.

California Climate Action Team

The California Climate Action Team (CAT) was established by Governor Schwarzenegger under an Executive Order on June 1, 2005. The purpose of the CAT is to coordinate state-level actions relating to Climate Change. The Team is led by the Secretary of the California Environmental Protection Agency and includes the Secretary of the Business, Transportation and Housing Agency, Secretary of the Department of Food and Agriculture, Secretary of the Resources Agency, Chairperson of the Air Resources Board, Chairperson of the Energy Commission and President of the Public Utilities Commission. The Climate Action Team is charged with implementing global warming emission reduction programs and reporting on the progress made toward meeting the statewide greenhouse gas targets that were established in the Assembly Bill 32 (described in more detail later in this report). The first report was sent to the Governor and the Legislature in 2006, and should be updated bi-annually thereafter.

California Climate Change Center

The California Energy Commission's Public Interest Energy Research (PIER) Program conducts public interest research, development, and demonstration projects to benefit California's electricity and natural gas ratepayers. In 2003, the California Energy Commission's PIER Program established the California Climate Change Center (CCCC) to document climate chance research relevant to the states. The CCCC Report Series details ongoing center-sponsored research on climate change predictions and impact analyses. All of the final CCCC reports include a preface which clarifies that the findings presented are interim project results, and information contained within the reports is subject to change.

Global Warming Impacts

The IPCC range of best estimate *likely* temperature increases by the year 2099 is 0.6 – 4.0 degrees Celsius (1 – 7 degrees Fahrenheit), depending on the global climate model utilized (IPCC, 2007). Regionally, scaled down climate models for northern California

estimate global temperature increases up to 4.5 degrees Celsius (9 degrees Fahrenheit) by 2100 (Cayan, 2007). An increase in global temperatures in the IPCC range may have multiple impacts on the water resources of the City of Alameda, even if the changes in local and regional temperature are not yet known.

Sea Level Rise

One of the most publicized impacts of global warming, and the impact with the most direct consequences to the City of Alameda, is sea level rise. Sea level rise can be defined as global or relative. Global sea level rise is defined as the increase of global average sea level. Throughout the world, land may be uplifting or subsiding. This will impact the relative change in depth of water at any given location, depending on the rate of movement compared to the rate of global sea level rise. In addition, coastal bays such as the San Francisco Bay may not experience sea level rise at the same rate as the global average. Relative sea level rise refers to the rise of sea levels accounting for local hydraulics, land uplifting or subsidence.

An example of the importance of global vs. relative sea level rise can be seen when examining the historic sea level trends in San Francisco Bay at the National Ocean and Atmospheric Administration (NOAA) gages for San Francisco (at the Presidio) and Alameda (Pier 3 at the Naval Air Station). The Alameda gage shows a long term average mean sea level rise of 0.82 millimeters per year (NOAA, Alameda Mean Sea Level Trend), while the San Francisco gage long term average mean sea level rise is 2.01 millimeters per year (NOAA, San Francisco Mean Sea Level Trend). Although the San Francisco gage period of record is longer, essentially the same rate of sea level rise is found if it is truncated to match the Alameda gage period of record. The reasons for this difference are unknown, and likely due to a combination of factors, but it serves to exemplify the complexity between local trends, global predictions, and site specific hydraulics.

IPCC Sea Level Rise Estimates

Depending on the emission scenario used, the predicted *likely* global sea level rise ranges from 0.18 – 0.59 meters (IPCC 4th Assessment Report), or 0.6 – 1.9 feet by the year 2099. IPCC reports do not provide mid-range estimates; e.g. sea level rise by 2050. The upper limit of this range is lower than the upper range stated in previous IPCC reports. The two primary factors affecting global sea level rise are thermal expansion of ocean waters due to increased atmospheric temperature, and melting ice. The IPCC estimates that of the global sea level rise that has occurred since 1993, thermal expansion of the ocean has contributed 57% of the total rise, decreases in the extent of glaciers and ice caps have

contributed 28%, and the remaining 15% is due to losses from the polar ice sheets. It must be noted that this range does not include uncertainties in climate-carbon cycle feedbacks or the full effect of changes to ice sheet flow, because a basis in published literature is lacking. Thus these values do not represent an upper bound to projected sea level rise. Long term projections show that global warming sufficient to eliminate the Greenland Ice Sheet (one millennium exposed to an average temperature rise in excess of 1.9 – 4.6 degrees Celsius) results in an additional seven meters (23 feet) of global sea level rise. The IPCC does not offer any uncertainty scale for this possibility.

United States Army Corps of Engineers Sea Level Rise Estimates

The United States Army Corps of Engineers (USACE) published an engineering circular (USACE, 2009) to direct the consideration of sea level rise estimates in project planning and design. While this methodology is required only for USACE civil work activities, it offers a valuable guidance for any planning effort. In summary, the USACE report recommends that the planning, engineering and designing for projects within the tidal zone or with downstream tidal boundary conditions consider how sensitive and adaptable the project is to a range of sea level rise estimates (low, intermediate and high). Specifically, the USACE directs determination of "how sensitive alternative plans and designs are to these rates for future local mean sea-level change, how this sensitivity affects calculated risk, and what design of operations and maintenance measures should be implemented to minimize adverse consequences while maximizing beneficial effects".

The "low" sea level rise estimate recommended by the USACE report is based on local historic tide gauges. In San Francisco, the Presidio tide gauge has the longest period of record and is consistently used for historic sea level trends in San Francisco Bay. For consistency with regional documents the Presidio gauge is used for calculations herein, although the Alameda gauge records described above may be more appropriate for the City. The long term average sea level rise at the Presidio gauge is 2.01 millimeters per year (mm/yr), with a 95% confidence limit of plus or minus 0.21 mm/yr (NOAA, Station 9414290). "Intermediate" and "high" sea level rise estimates are based on the National Resource Council (NRC) curves and equations developed for a 1987 Report (*Responding to Changes in Sea Level: Engineering Implications*), modified to account for the updated annual estimate of sea level rise made in the 2007 IPCC report, and manipulated to include consideration of the date of the equation development. The "intermediate" sea level rise projection is based on the modified NRC Curve I, and the "high" sea level rise projection on the modified NRC Curve III. This equation is:

$$E(t_2) - E(t_1) = 0.0017\left(t_2 - t_1\right) + b(t_2^2 - t_1^2)$$

where:

t_1 = time between construction date and 1986;

t_2 = time between date at which sea level rise projection is desired and 1986;

$E(t)$ = eustatic sea-level rise, in meters, as a function of (t);

b = Variable, 2.36E-5 for modified NRC Curve I, 1.005E-4 for modified NRC Curve III.

Table 1 presents the range of sea level rise projects for the City of Alameda using this methodology, assuming adoption of the Presidio gauge for the local historic sea level trend, and construction of any given project in 2010.

Table 1: Range of Sea Level Rise Projections Using USACE Methodology with Presidio Gage and 2010 Construction Year

USACE Methodology Sea Level Rise Projection Range (feet)			
Year	**Low**	**Intermediate**	**High**
2025	0.1	0.2	0.4
2050	0.3	0.5	1.4
2075	0.4	0.9	2.8
2100	0.6	1.5	4.6

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In summary, significant uncertainties remain in sea level rise projections, particularly as one forecast's farther into the future. <u>The most current available estimates for sea level rise by 2050 range from 0.3 foot to 1.5 feet, and by 2100 from 0.6 foot – 4.8 feet.</u> Confidence in any sea level rise prediction decreases the further into the future that analysis is projected, due to unknowns about future emission scenarios, potential climate feedback loops and the severity of melting ice. It is important to note that emphasis should not be placed on a particular specific value for sea level rise. Not only is a consensus on a particular value unlikely, but the selection of the year 2100 as a reporting point for sea level rise projections is arbitrary. Even with drastic reductions in carbon emissions sea levels are expected to continue to rise beyond 2100 due at least to continued thermal expansion of ocean waters. Thus, any planning for sea level rise impacts should recognize the inherent uncertainty and long term ongoing nature of these projections.

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Storm Surge

During storm events, ocean water increases in elevation due to low barometric surface pressure. This phenomenon is called storm surge. The FEMA 1% storm surge for San Francisco Bay at Alameda is 7 feet NGVD, compared to a mean high-high tide of 3.7 feet NGVD (NOAA, Alameda Datums). This represents a 1% surge of 3.3 feet. It is *likely* that the incidence of extreme high sea level has increased at a broad range of sites wordwide since 1975. Extreme high sea level is defined as the highest 1% of hourly values of observed sea level at a station for a given reference period (IPCC, 2007).

Pronounced multi-year fluctuations of San Francisco non-tidal residuals (NTR; total water elevations above tidal elevations – for San Francisco Bay NTRs are primarily storm surge and wind driven waves) are evidenced in historical records and no significant changes in the mean monthly positive NTRs exist between 1858 and 2000. However when considering only the highest 2% of extreme winter NTRs there has been a significant increasing trend since about 1950 (Bromirski, 2003). This increased 'storminess' may be part of a larger cycle, but it suggests a relationship between global climate warming and overall storminess on the west coast.

The occurrence of hourly observed high sea levels (above the 99.99th percentile thresholds) in San Francisco Bay has increased sharply since 1969. The maximum observed sea level has also increased since that time, although the period of 1987-2004 had a slightly lower peak sea level than 1969-1987. Recent studies have concluded that if sea level rise is on the lower end of the current predicted ranges, the occurrence of extremely high sea level events will increase, but the increase in extremes would be not so different from the increasing trend that has been seen in California for the past several decades. If, however, sea level increases reach the higher end of the range, extreme events would increase not only in their frequency but also their duration, substantially beyond the historic trend seen in the 19th and 20th centuries (Cayan, 2007).

In short, it is expected that as sea levels rise, not only will the occurrence of high sea level, or surge, events increase, but so may the amount of surge itself (currently about 3.3 feet above mean high high water in Alameda). This increased storm surge elevation may impact flood risk, backwater conditions and storm water pump station operation; however quantitative estimates for the increased storm surge have not been made, and are unlikely to be determined in the near future.

Wave Runup

Wave runup is the elevation wind-driven waves will reach as waves break on land and may be affected by global warming. However, these impacts are not particularly well understood at this time. A review of recently published literature finds that different published studies come to different, and at times directly opposing, conclusions regarding likely climate change impacts to wave energy. Wave heights are greatly influenced by local conditions, likely a major cause for the differing results found in the available literature. Some general trends are well understood, such as that extreme wave heights and surge fluctuations tend to increase from the south to the north along California Coast, as a result of increasing storm intensities along the northern coast (Cayan, 2007).

Wave runup is a function of water depth, wind speed and direction, and the features of the land on which the wave is breaking (slope, roughness, etc.). In some parts of San Francisco Bay, rising sea levels will inundate low lying marshes, creating broad, but shallow, flooded areas. In this scenario, wave runup will likely decrease, as the shallow water will dampen wave heights. In Alameda however, which is generally protected by high land, rising sea levels will create deeper water surrounding the City, potentially resulting in increased wave heights and runup.

Published literature has found that when short term sea level is highest (i.e. during storm surge events), wave energy has an increased likelihood of reaching very high levels. The peak likely significant wave height (the average height of the one third highest waves) increases by 2.5 meters in one scenario where the surge value increased from 4 centimeters (cm) to 30 cm (Cayan, 2007). Thus in that particular scenario, as the storm surge increases, so does wave energy and height, which in turn may increase wave runup. That said, recent downscaled models have also indicated that the incidence of large coastal storms will lessen as part of the overall drying trend (discussed in more detail in the precipitation section below), resulting in a marginal decrease in the wind wave energy reaching California's coast as well as a decreasing trend for significant wave heights (Cayan, 2009). In short, although climate change is expected to impact storm surge and wave runup, these impacts (or even the trend of impacts) is not well understood at this time, and in any event, these impacts are expected to be dwarfed by the impact of increasing mean sea level.

The Bay floor near Alameda is largely composed of Bay mud, a thick deposit of soft, unconsolidated silty clay, which is saturated with water. One potential mitigation action against increased wave height due to deepening water would be to fill to maintain existing water depths. In addition to the multitude of permitting and environment issues with this activity, however, Bay Mud has a very high compressibility. In other words,

Bay Mud will continue to compress even when large volumes or weights are set on it. Thus filling on top of Bay Mud is ineffectual, and when additional environmental impacts are considered with the uncertainty of wave height and runup impacts, not a feasible mitigation alternative for Alameda to offset increased wave heights and runup.

Precipitation

It is *likely* that the frequency of heavy precipitation events (or proportion of total rainfall from heavy storms) has increased over most areas (IPCC, 2007). Global analyses of precipitation from 1901-2005 do not show statistically significant trends due to many discrepancies between data sets and the variability of precipitation in both space and time (Bates, 2008). Likewise, there is no consensus among regional climate models as to how mean annual precipitation totals might change in the United States (Dettinger, 2004), although most recent global and regional models predict that total mean precipitation will modestly decrease (5-20%) in the latter half of the next century (Hayhoe, 2004; Cayan, 2007, Draft 2009). Long term historic analyses of precipitation in the state of California show that there is no statistically significant change in total annual mean precipitation from 1890 through 2000, although the variability of total rainfall in any given year appears to have an increasing trend (DWR, 2006).

While the total mean annual precipitation is not predicted to change significantly, the timing and intensity of storm events is expected to change, with a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events Climate models predict (and historic records reflect) that proportionally less rainfall will fall during spring and summer months (April – July) and more in winter months (November – March) in northern California due to global climate change (Dettinger, 2004; Cayan 2007; DWR 2006). These shifts in precipitation timing and intensity may have impacts on flooding and water supply.

The most updated *Climate Change Scenarios* report (Cayan, 2009) states that the occurrence of significant storms declines at least marginally and that the occurrence of high daily precipitation events generally remains about the same through 2100 as it does in the historical projections. It should be noted that this conclusion is markedly different from previous conclusions by the same authors, which predicted a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events (Bates, 2008; Cayan 2007). Several CCCC reports reviewed for this analysis repeat the earlier, and presumably outdated, conclusion.

In summary, while a small decrease in annual precipitation is forecast, the trend in number and magnitude of large precipitation events is unknown. The most current

studies reviewed for this analysis both conflict previous conclusions and other updated studies, further exemplifying that there is no consensus regarding the potential impacts of climate change on the frequency or magnitude of large storm events.

Sea Level Rise Impacts to the City of Alameda

The effects of climate change described above have potential impacts to virtually all water resources within the City of Alameda, including not only local flood control and risk but also regional impacts to sectors such as agricultural and water supply. This report focuses on how rising sea levels may impact the risk of flood inundation of the City from its surrounding waters, and the impact of rising sea levels to the 10-year improvements previously made in the City SDMP. For this analysis, 18 inches (1.5 foot) of sea level rise was assumed. This represents the upper bound of the range of the most recently published sea level rise projections by year 2050 (Cayan, 2009).

When discussing projects to mitigate the impacts of sea level rise, there are several important points to keep in mind. As described above, there is not currently and unlikely to ever be a true consensus in the prediction of sea level rise, particularly a consensus on a projection 100 years into the future. A planning horizon of 100 years is not only far beyond most planning timelines typical to public agencies, but it is also beyond the typical useful life of structural flood protection elements. In other words, even if it were financially feasible to construct a project today to protect for a sea level rise scenario in 2100, it may not be advisable to do so, since that project could be structurally unsound by the time it was needed. Finally, it should be noted that although currently the year 2100 is the most common projection date, sea levels are expected to continue to rise beyond the year 2100.

<u>Inundation due to Rising Waters</u>

As an island community, Alameda is uniquely vulnerable to rising water levels in San Francisco Bay. Currently, Alameda is protected from inundation from its surrounding waters primarily by high ground, as opposed to floodwalls or levees. Interior lagoons are hydraulically connected to the surrounding waters via weir inlets, pumps, or gated outlets.

Figures 1 through 6 reflect City-wide recent topographic data adjusted to show three elevations of interest: existing mean sea level, mean sea level with 18” increase, and the highest tide elevation for various storm events with 18” of sea level rise added. The storm specific tide cycles were developed for the SDMP and the methodology and results of that process are described in detail in that report. It should be noted that these figures

do not take into account potential flood protection of naturally occurring high ground or existing flood control facilities. In other words, a shaded area represents an elevation range only, and does not necessarily mean that surrounding water will be able to reach and pond in all of those locations. One good example of this is shown in Figure 2, which reflects the fact that much of the golf course is below mean sea level. This does not mean, however that the golf course is always inundated with surrounding waters, due to existing high ground and storm drain facilities. That said, the lack of flap gates on many storm drain outlets may allow for backwater due to high tides to reach interior locations of the City. Figures 7 and 8 translate the water surface elevation into depth of water for the most severe (100-year event) scenario. Again, these figures represent potential risk areas without consideration of existing natural or man made protection measures.

Table 2 summarizes the existing and sea level rise scenario mean and high tide levels reflected in Figures 1 through 8. Storm specific tide cycles were developed for the SDMP, and a more complete description of the methodology for that process can be found in the SDMP, Chapter 3.

Table 2: Mean and High Tide Elevations for Existing and Sea Level Rise Scenario

	Existing (NGVD)	Sea Level Rise (18") Scenario (NGVD)
Mean Sea Level	0.5'	2.0'
10-Year High Tide	5.1'	6.6'
25-Year High Tide	5.4'	6.9'
100-Year High Tide	6.2'	7.7'

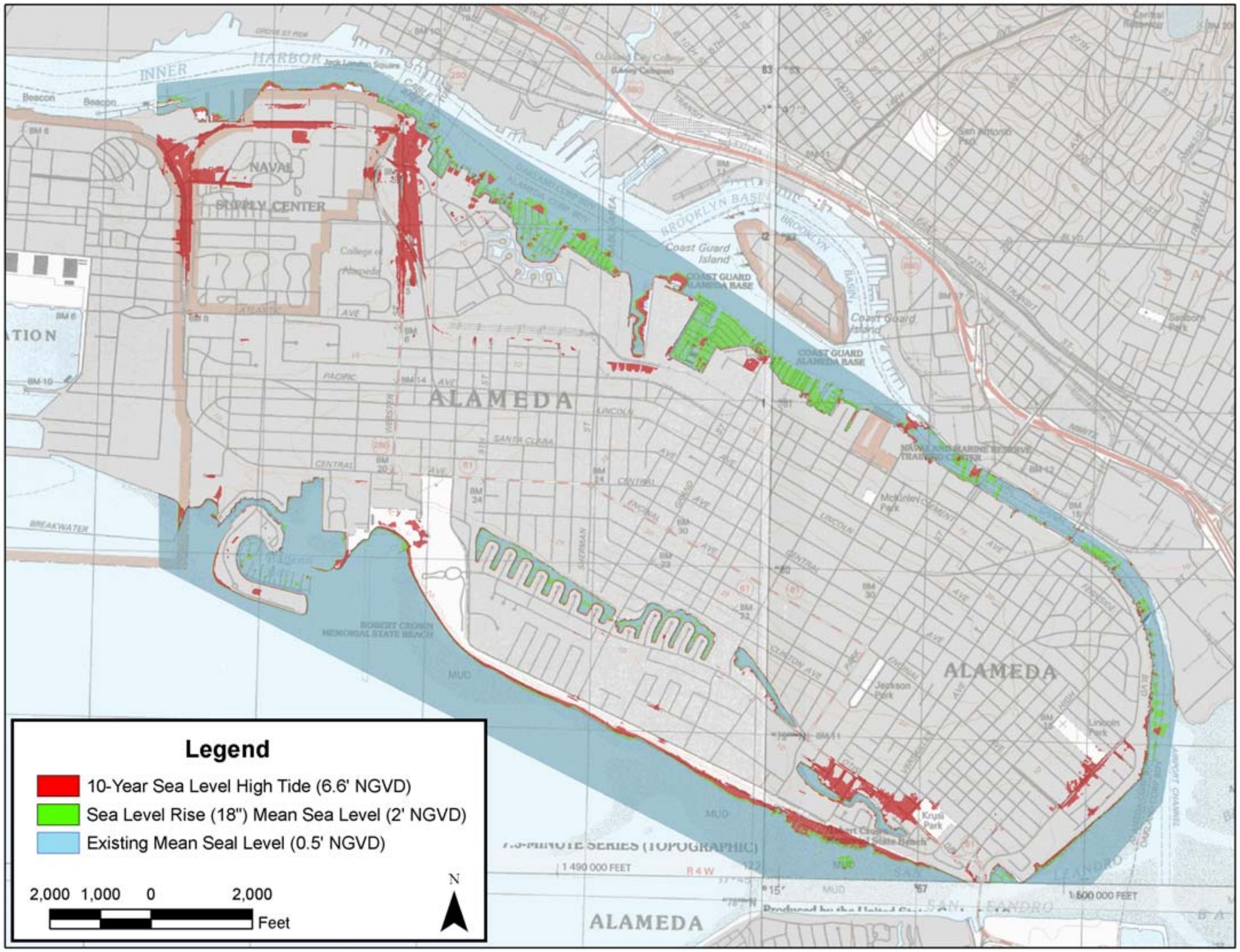

Figure 1: Areas below the 10-year High Tide with 18" of Sea Level Rise, Main Island

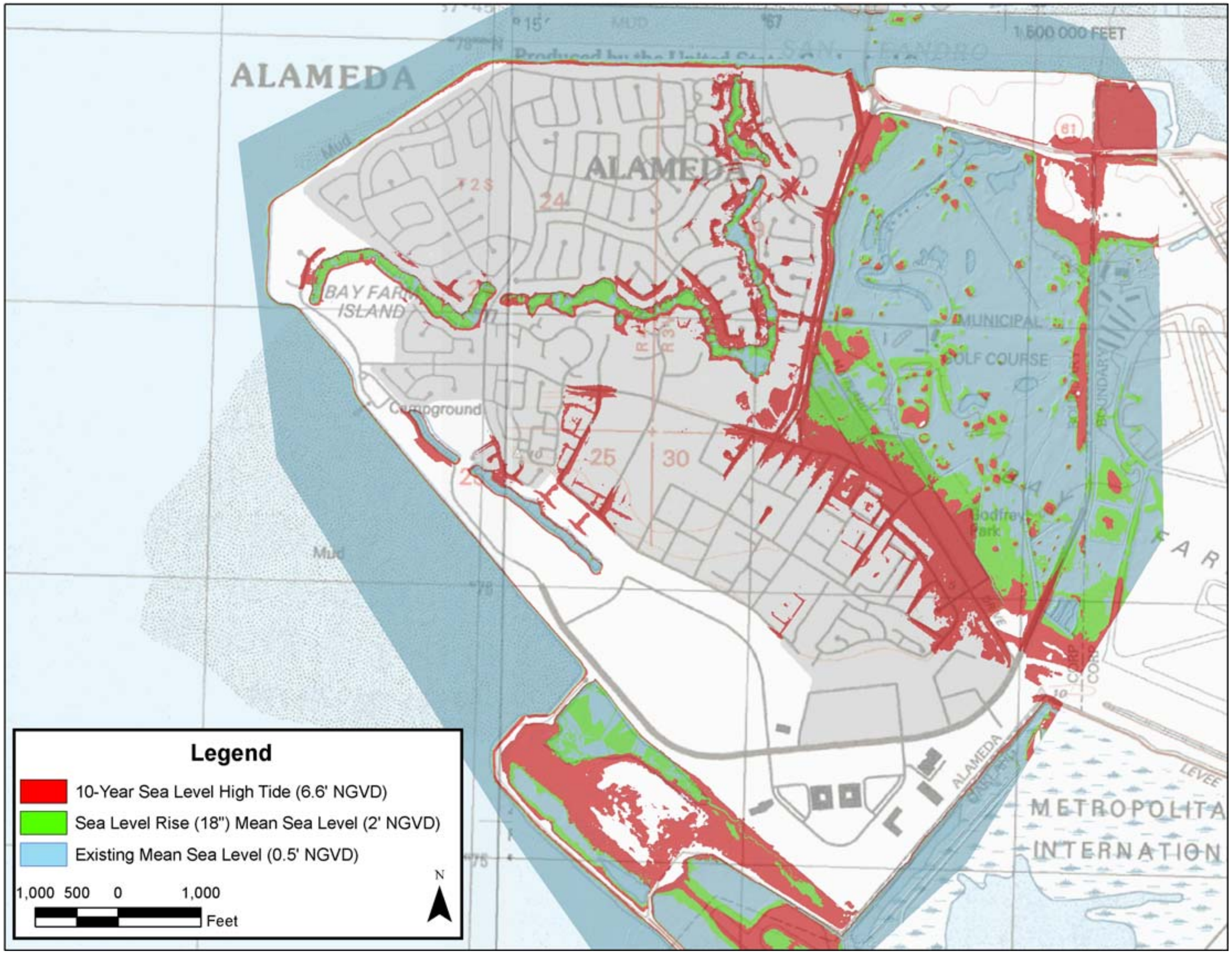

Figure 2: Areas below the 10-year High Tide with 18" of Sea Level Rise, Bay Farm Island

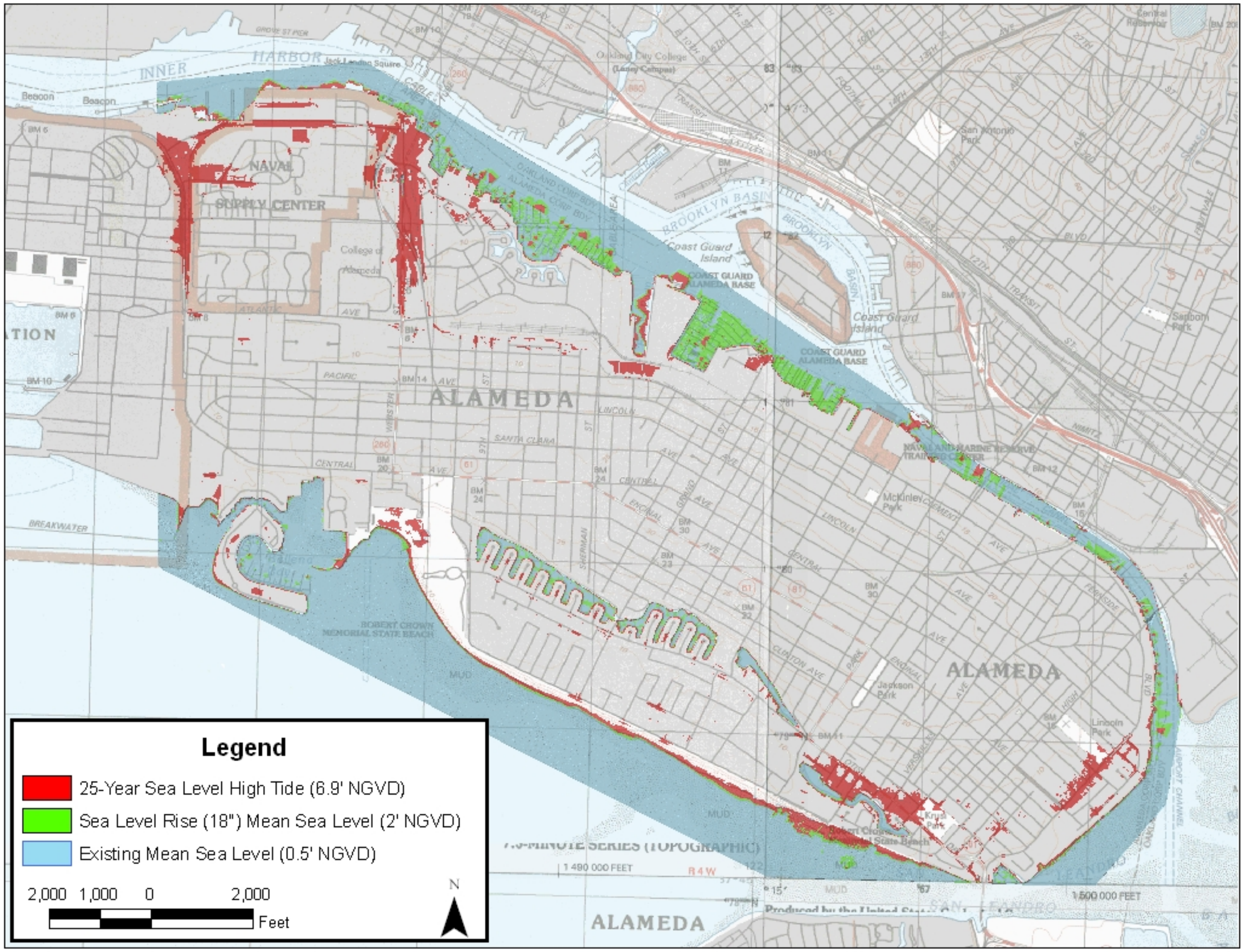

Figure 3: Areas below the 25-year High Tide with 18" of Sea Level Rise, Main Island

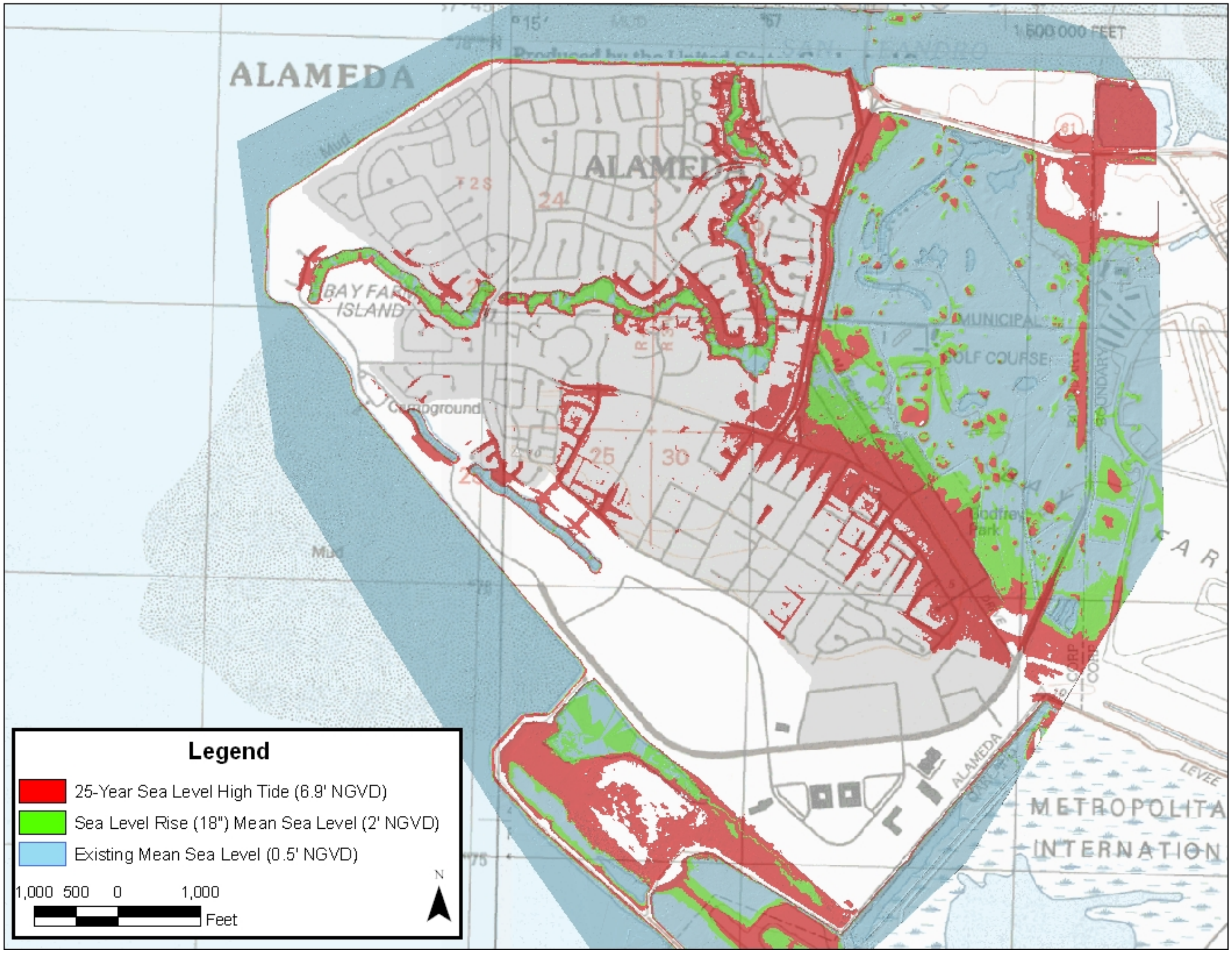

Figure 4: Areas below the 25-year High Tide with 18" of Sea Level Rise, Bay Farm Island

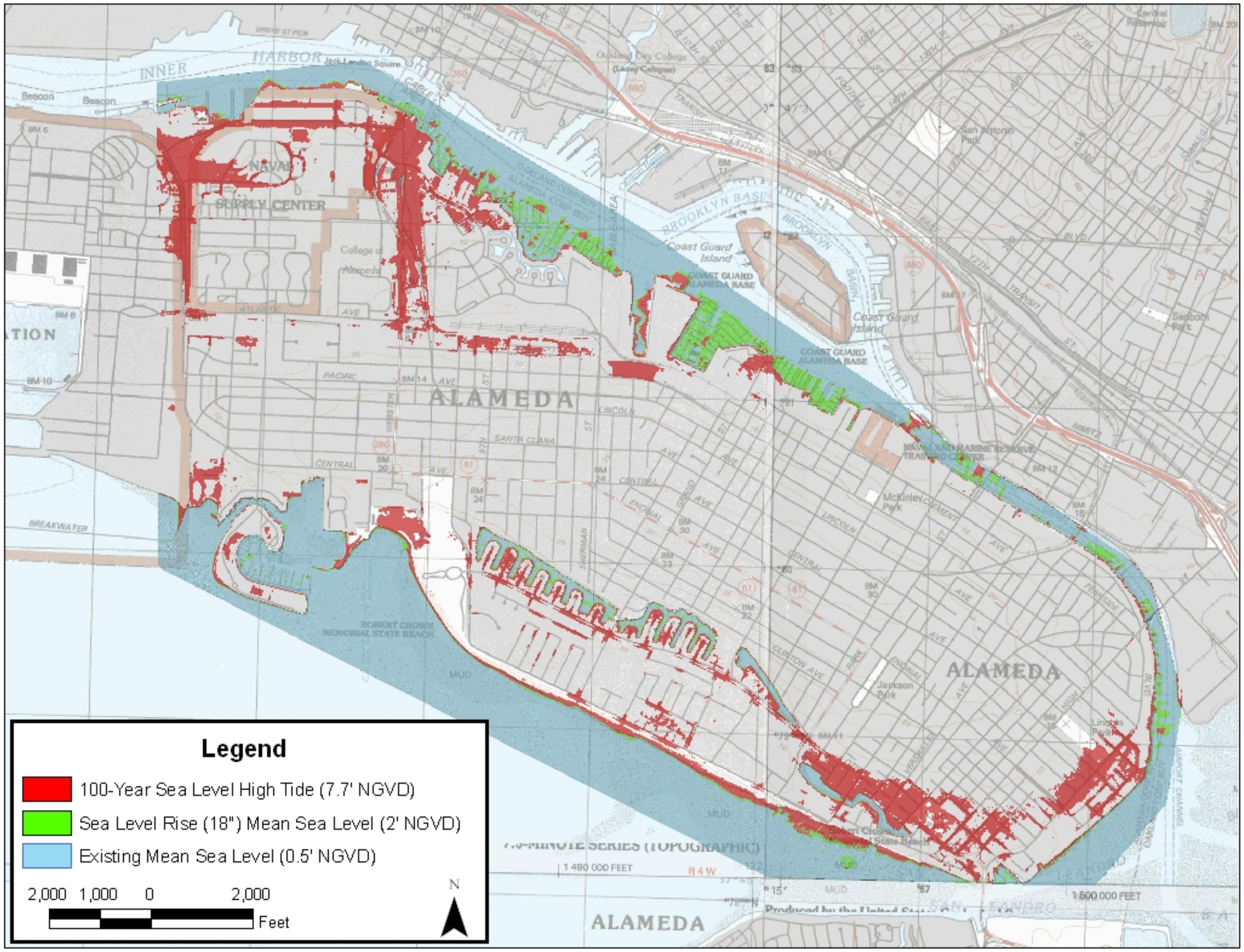

Figure 5: Areas below the 100-year High Tide with 18" of Sea Level Rise, Main Island

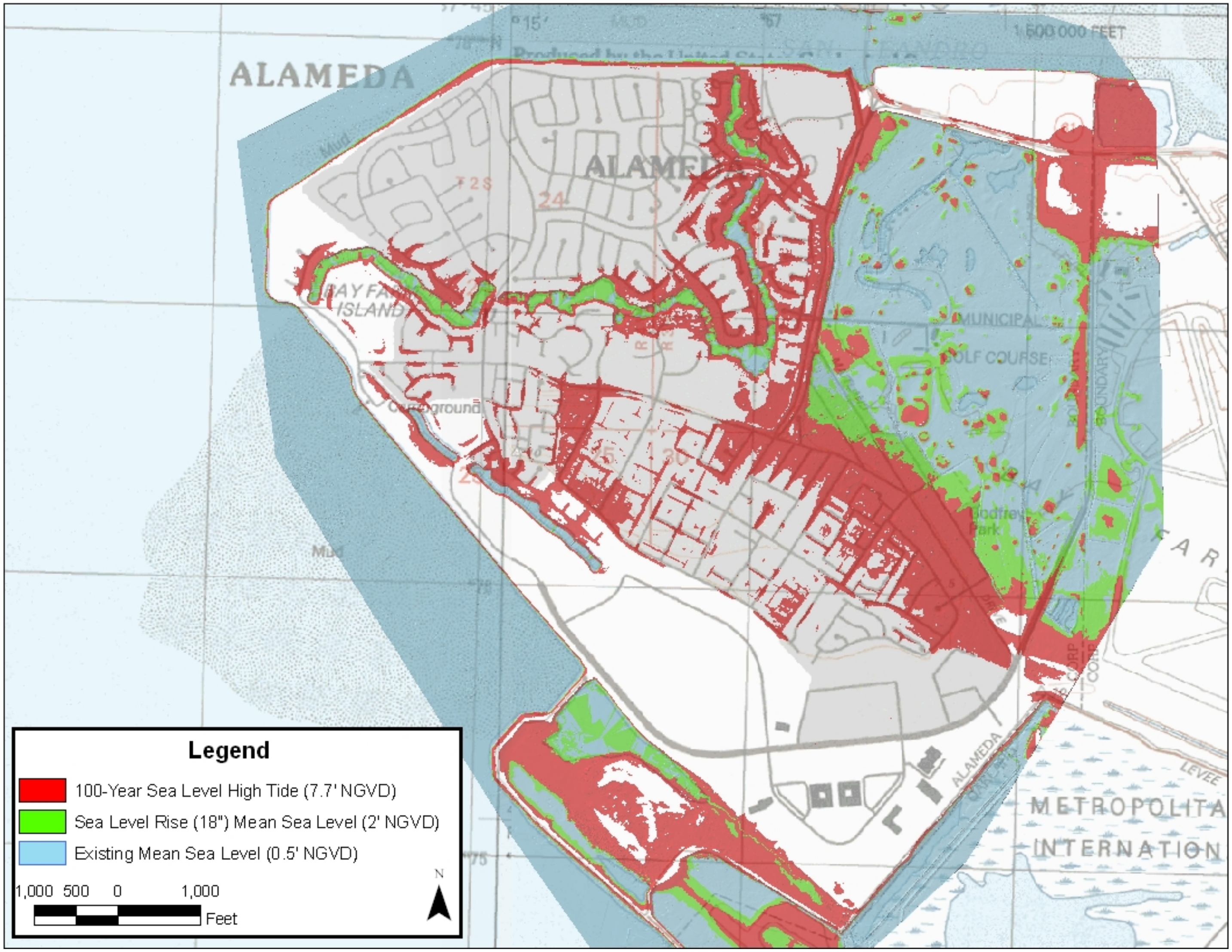

Figure 6: Areas below the 100-year High Tide with 18" of Sea Level Rise, Bay Farm Island

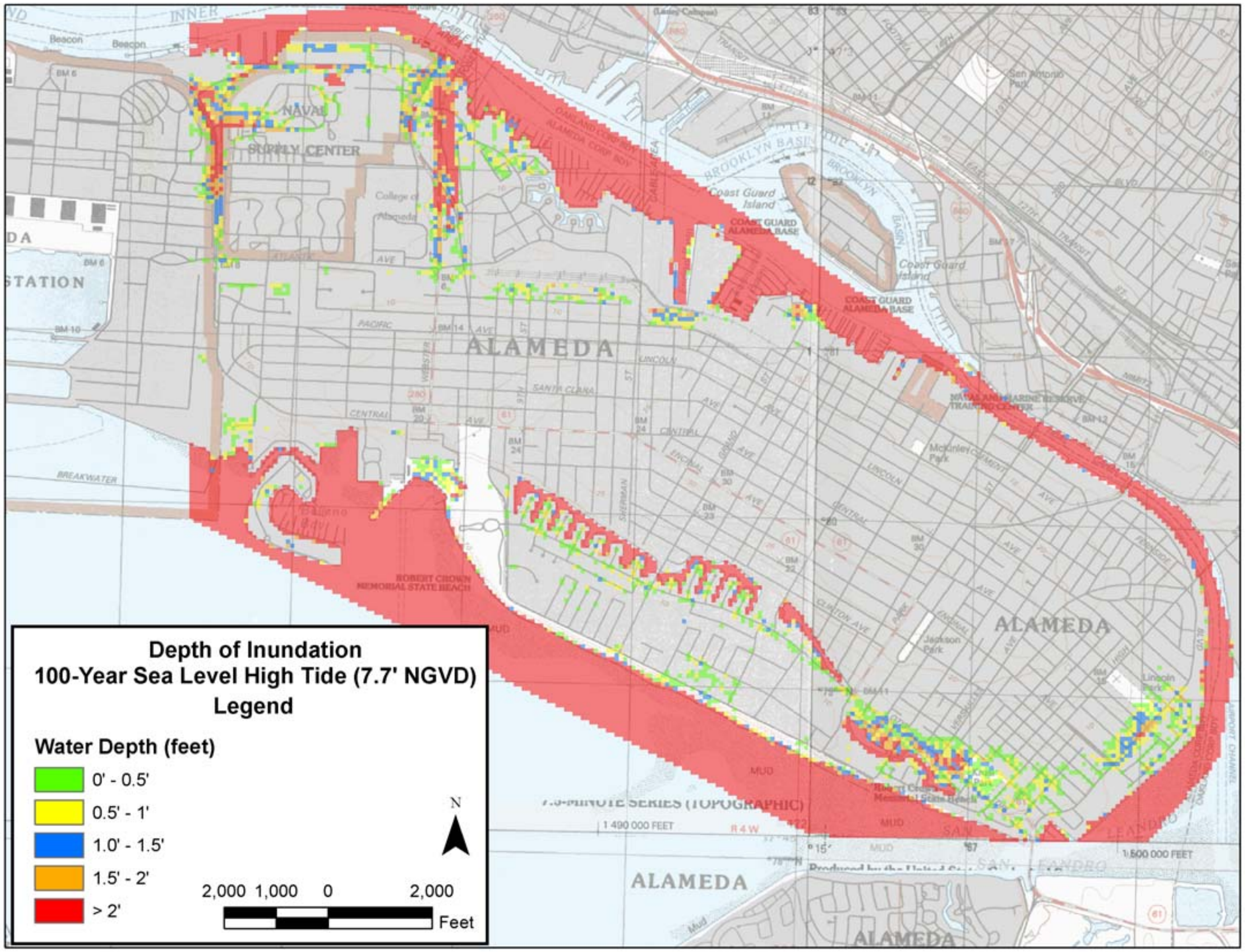

Figure 7: Depth of Water below the 100-year High Tide with 18" of Sea Level Rise, Main Island

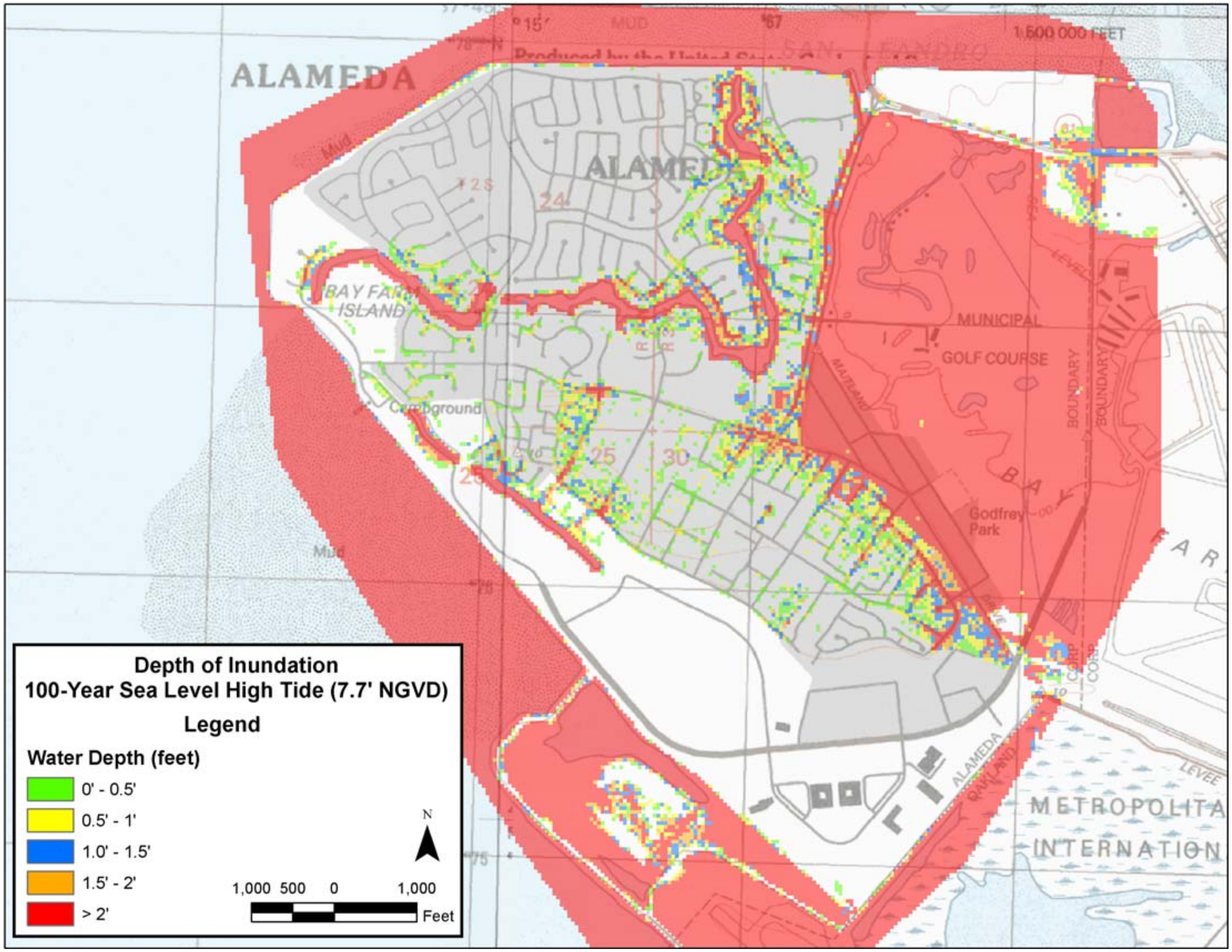

Figure 8: Depth of Water below the 100-year High Tide with 18" of Sea Level Rise, Bay Farm Island

As shown in the figures above, the primary inundation area on the Main Island is the low lying ground in the south eastern portion of the island, as well as the area immediately south of the tunnel (Webster / Posey Tube) between the Cities of Alameda and Oakland. This second area is largely included in the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) 100-year floodplain delineation. On Bay Farm Island, the primary areas of inundation are those areas adjacent to interior lagoons or the low-lying areas of the golf course.

On both Alameda Island and Bay Farm Island, the most significant impact is seen during the peak storm conditions. That is, the increase in mean sea level from 0.5 feet NGVD to 2.0 feet NGVD is relatively small, whereas the area in shaded red, representing the elevation of the peak storm tide with an additional 18 inches of sea level rise, is much larger. The water surface elevation represented by the red area is a peak elevation and is not sustained over long periods of time even during a storm event.

Projects to Mitigate Sea Level Rise Inundation

There are several projects which may partially mitigate the inundation areas shown in Figures 1 through 8 above. As mentioned previously, however, it should be noted that structural improvements are not necessarily a recommended long term strategy to mitigate sea level rise impacts, particularly inundation. Any projects undertaken should include flexibility to adjust or adapt the project for continued sea level rise beyond the 18 inches used for this analysis.

Bay Farm Island lagoon water levels are controlled via pump station outlets, and an intake weir. The operation of the pump stations is currently manual. The operation of the pump stations and the configuration of the intake weirs may need to be adjusted to maintain existing lagoon water surface elevations in the event of sea level rise. While inundation via the golf course appears more significant on the above figures, the peak tide condition is not expected to be maintained long enough to cause the widespread inundation shown. If inundation does occur, a floodwall along Island and Doolittle Drives is one potential mitigation project, however at that stage, the Oakland Airport would also be experiencing flooding due to higher sea levels and coordination with the Airport on flood protection measures is advisable. Alternative mitigation options may include increased pumping capacity at the golf course pump station, or raising the streets bordering the Golf Course to act as flood barriers.

The Main Island inundation map shows the low lying areas of land in south eastern as well as north central Alameda below extreme water surface elevations in the sea level rise scenario. In this case, although only a slim strip of bayfront land is below the high tide with sea level rise elevation, water may also reach many of these areas via storm drain pipes which are currently without flap gates. Projections of sea level rise predict that not only will extreme sea levels occur more often, they may also occur for longer durations (Cayan, 2009). Thus, interior ponding due to backwater from long periods of extreme tides is a possibility. The installation of flap gates at storm drain outfalls would protect City streets from this backwater condition. Due to the relatively short duration of high waters expected, an adaptive management approach which prioritizes projects based on actual backwater experienced is recommended for outfall flap gate installation. As such, flap gates have not been included in this analysis of sea level rise impacts to the CIP.

Storm Drain Capacity

In the 2008 Storm Drain Master Plan, Schaaf & Wheeler presented a Capitol Improvement Program (CIP) to achieve a 10-year level of service for the storm drain network throughout the City of Alameda. The CIP included upsizing existing pipes, additional capacity at several storm drain pump stations, new pipes to provide storm drain capacity to areas currently underserved by the existing system, and several non-capacity related improvements such as trash racks at pump stations. The City directed Schaaf & Wheeler to determine how sea level rise would affect the proposed CIP. In other words, if all CIP projects were completed to meet a 10-year level of service, what additional projects would be necessary to achieve this same level of service assuming that 18” of sea level rise occurs.

For this analysis, Schaaf & Wheeler assumed that sea level rise affects the tide cycle uniformly, that is, both peak and ebb tides are increased by 18 inches. Global warming may in fact impact the tide cycle itself during storm events, particularly storm surge as described above; however numerical projections of these impacts do not currently exist. Figures 9 through 15 show the impact of this sea level rise scenario on the 10-year improved storm drain network.

In addition to the impact scenario described above, Schaaf & Wheeler analyzed the improved 10-year storm drain system operation during a 2-year storm event, but with a 100-year sea level rise scenario tide. This serves to exemplify how the improved system will operate under a relatively minor storm but severe tide. The result of this analysis is essentially identical to the areas shows in Figures 1-6. In this scenario the rainfall is inconsequential, and backwater from the tide cycle determines peak water surface elevation. As described previously, the time period when water surface elevations exceed rim (i.e. ground) elevations is relatively short, generally on the order of 15 minutes or less, although the duration of flooding increases closer to the outfalls due to lower ground elevations at the boundaries of the City.

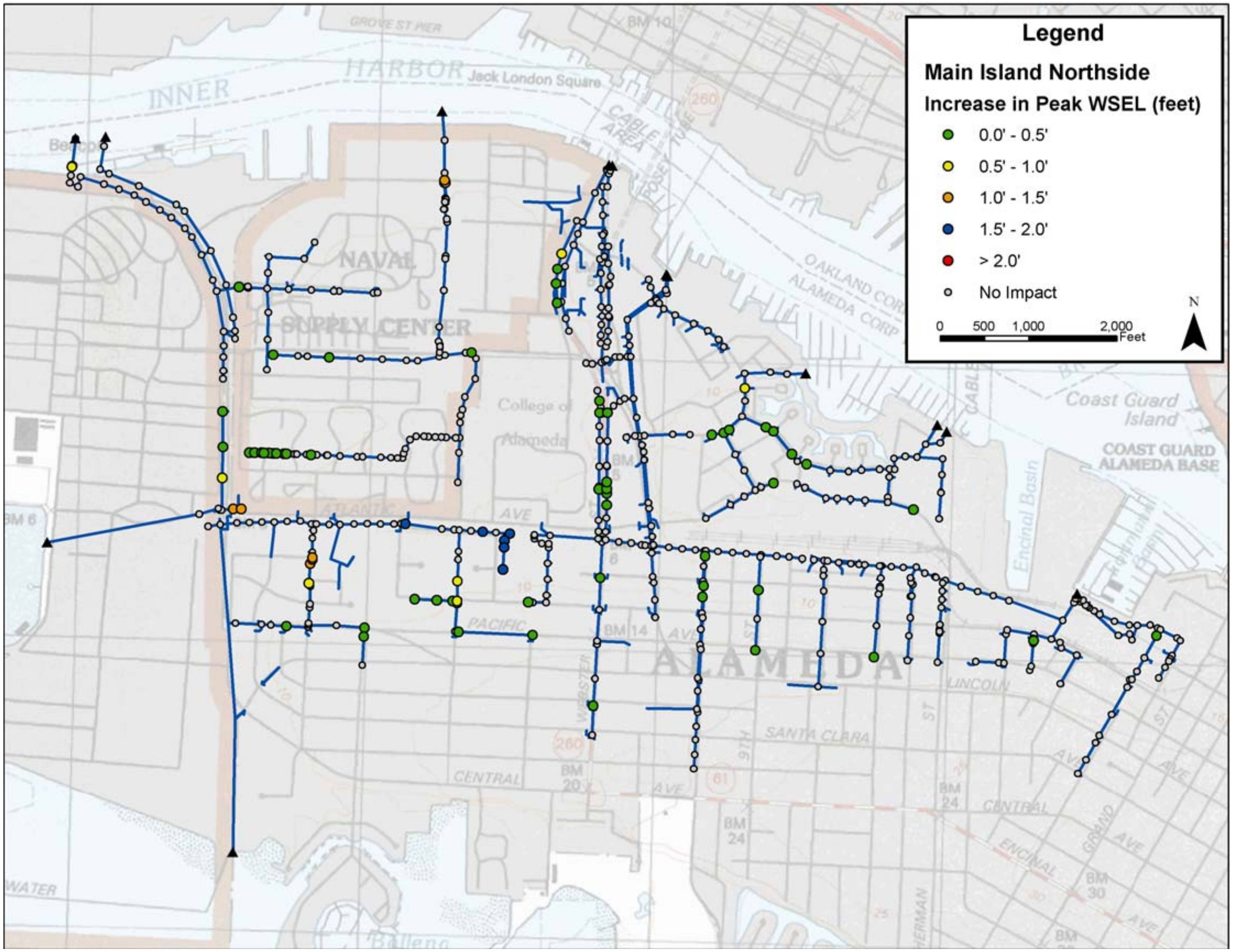

Figure 9: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island Northside

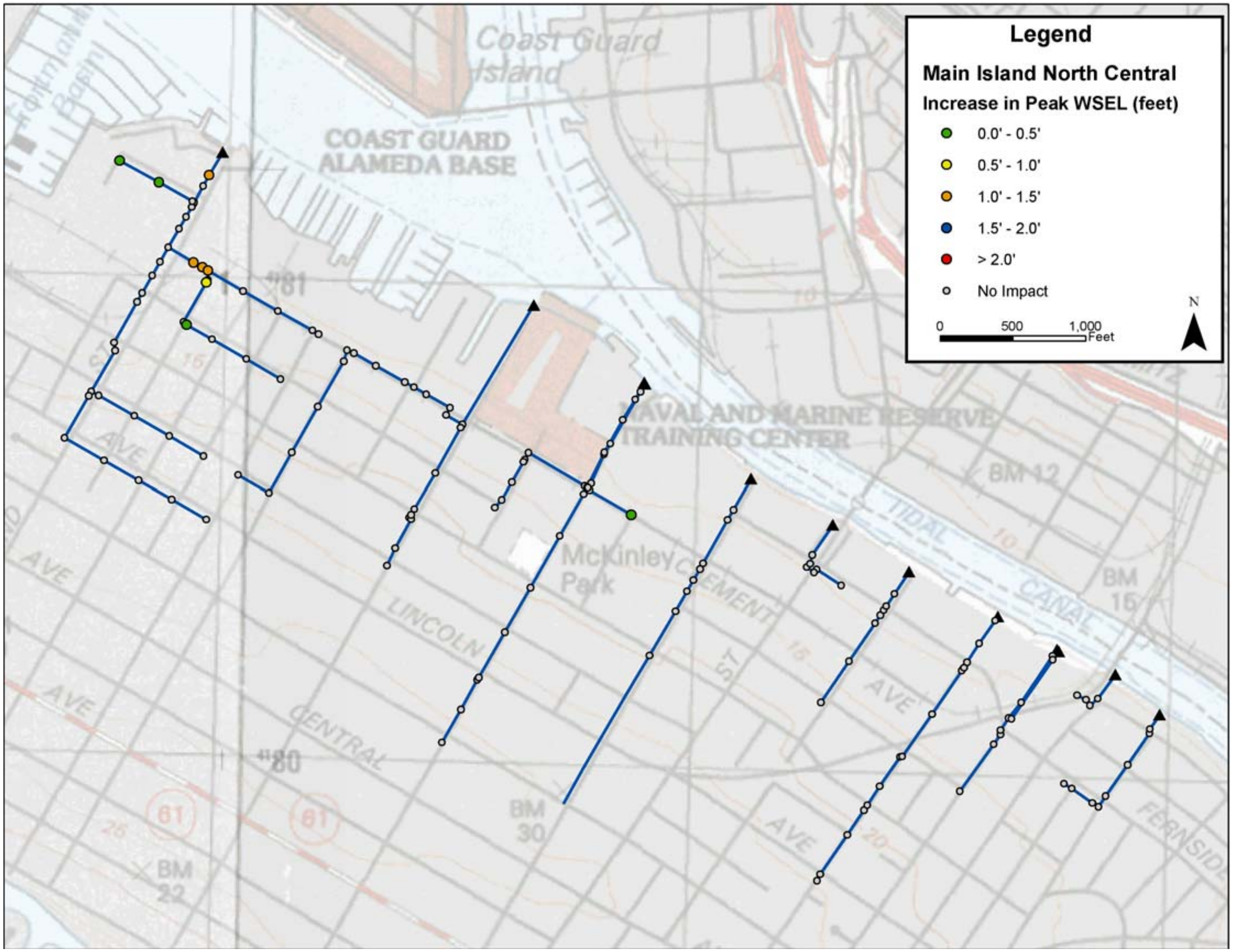

Figure 10: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island North Central

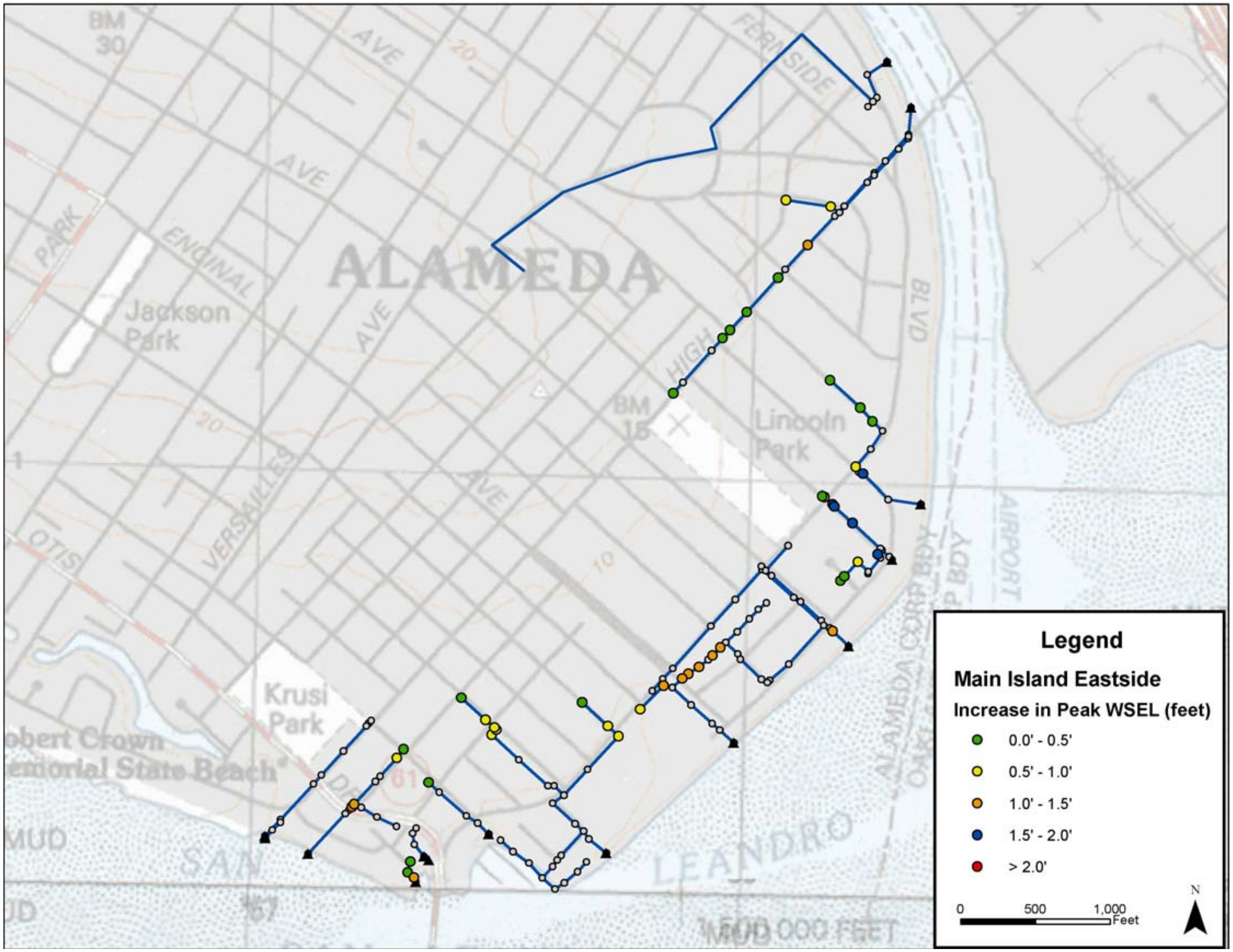

Figure 11: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island Eastside

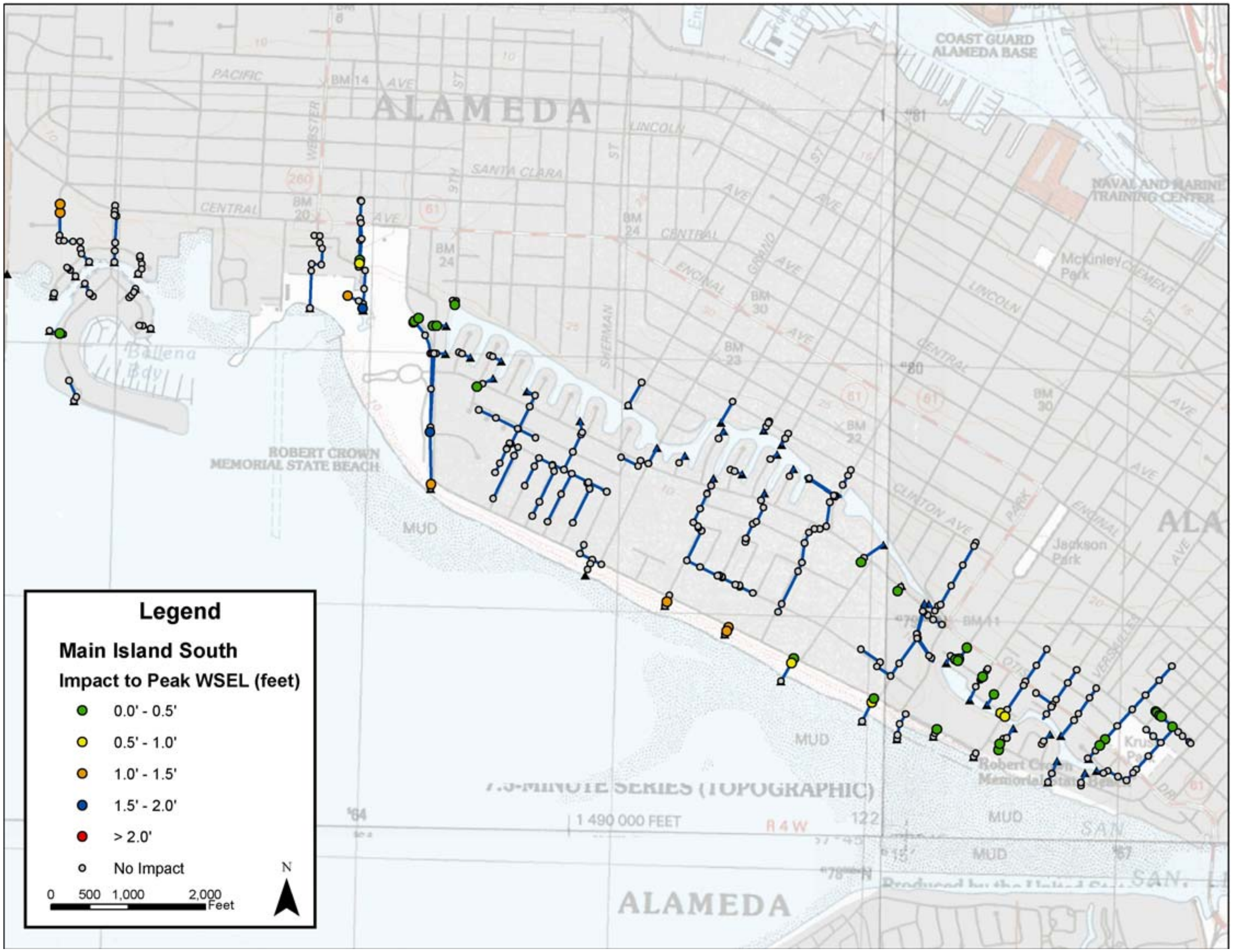

Figure 12: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island South

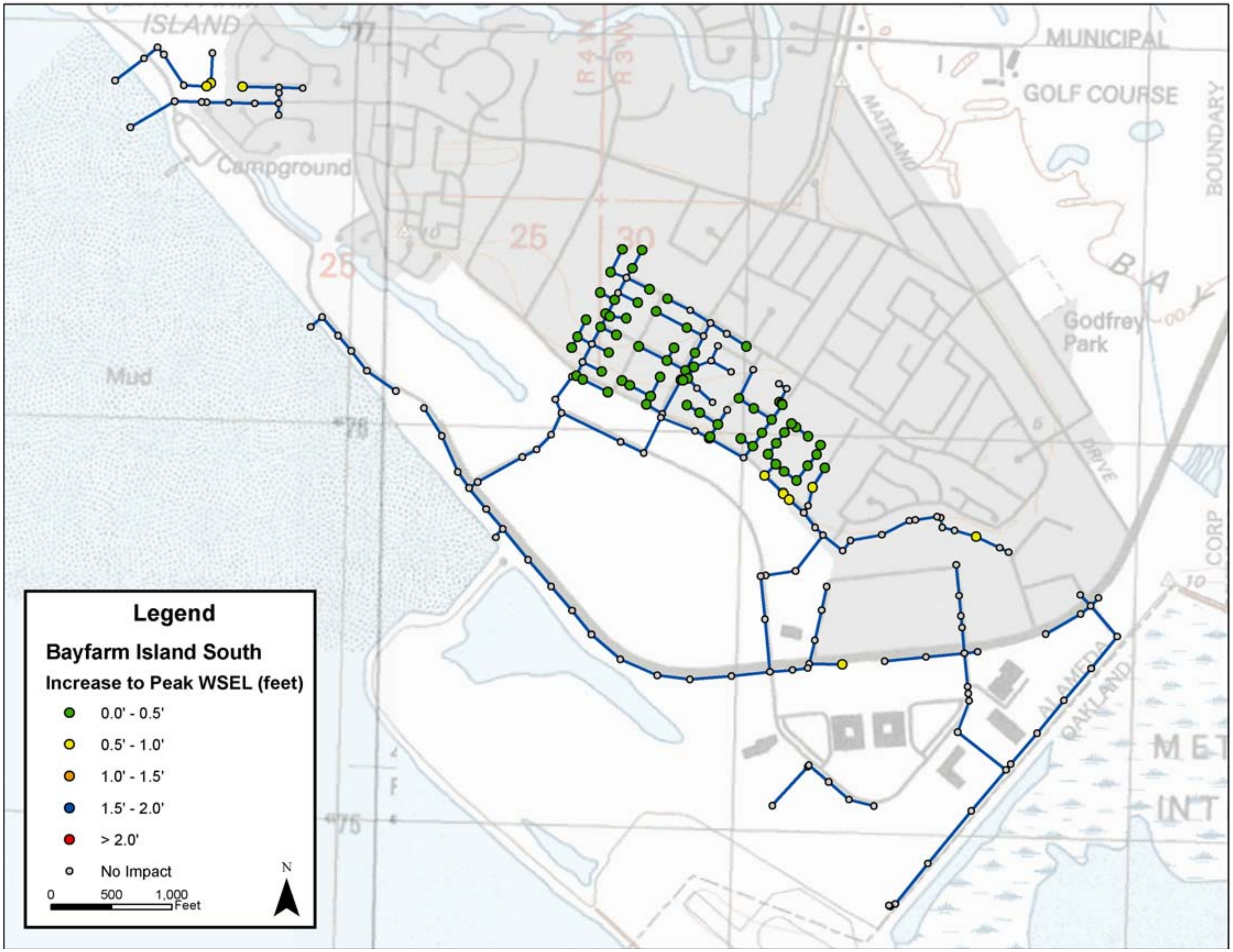

Figure 13: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island South

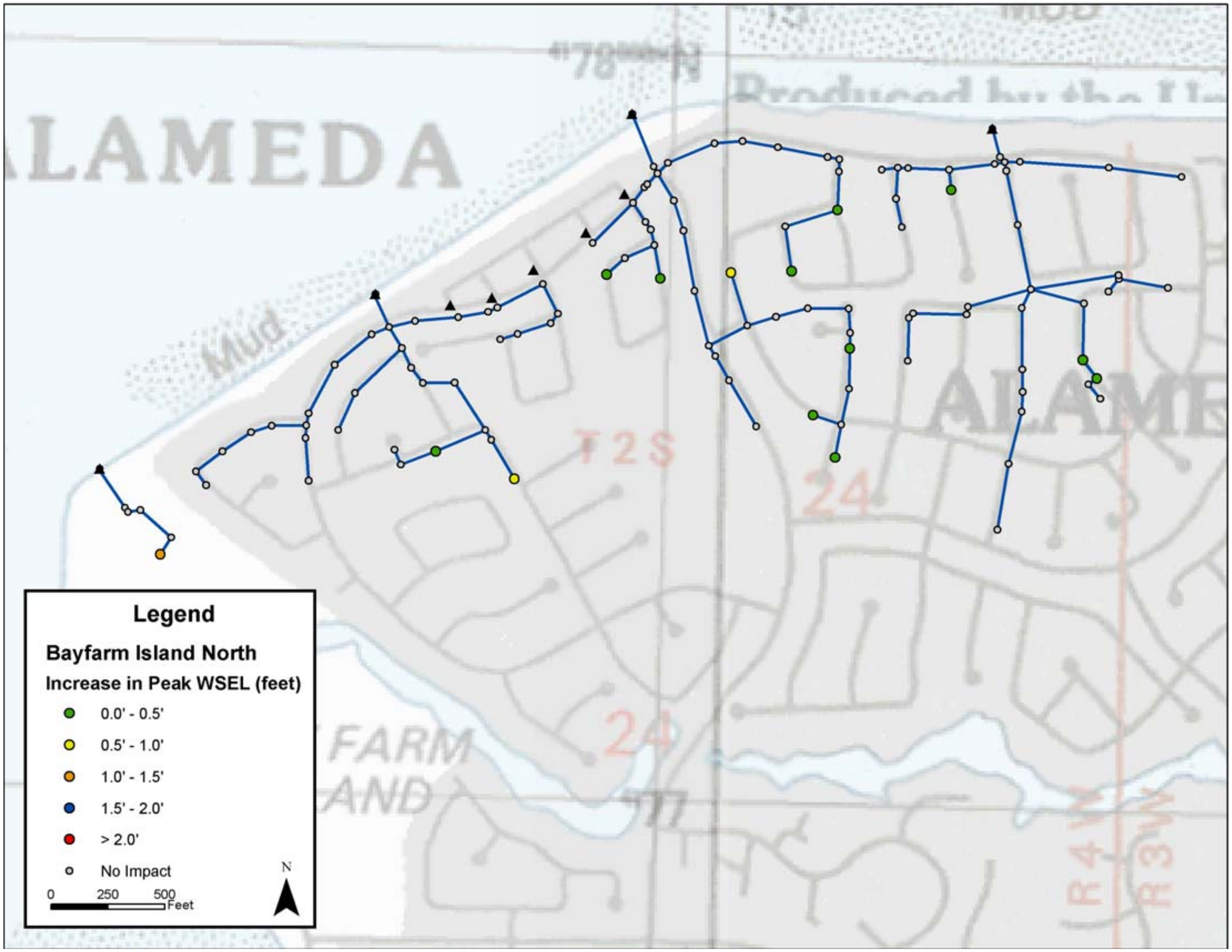

Figure 14: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island North

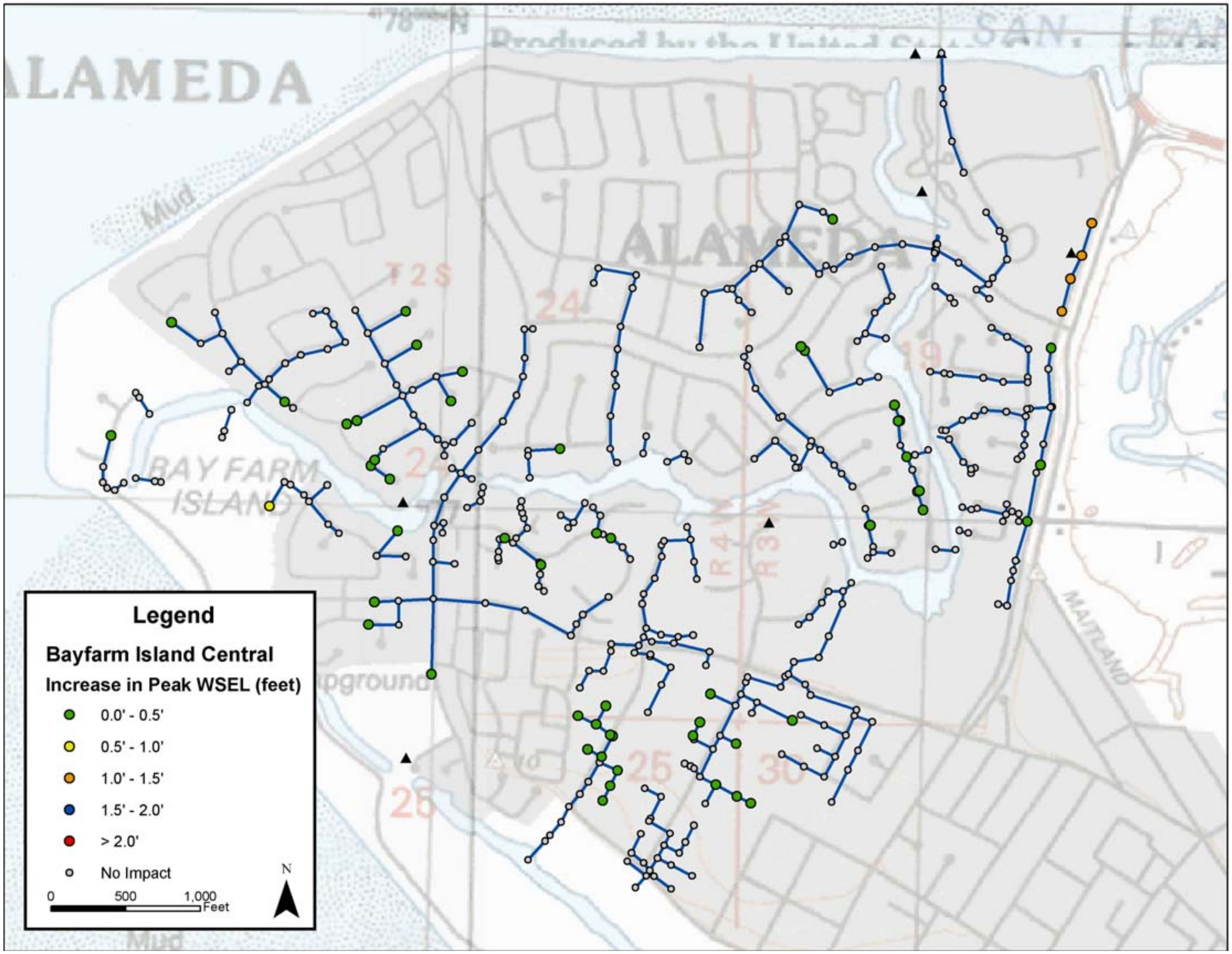

Figure 15: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island Central

No figure is included for Bay Farm Island East because there are no impacts in that area. As shown above, in general these impacts are relatively small (most commonly less than a half foot) as expected given the existing conditions 10-year level of service of the system in this scenario.

Projects to Mitigate Impacts of Sea Level Rise on Storm Drain Capacity

Additional projects are required to maintain a 10-year level of service if 18” of sea level rise is applied to the tide cycle. Figures 16 through 22 show additional projects necessary to reach a 10-year level of service under the assumed sea level rise scenario. Pipes which have a white highlighted background are pipes that have been improved from the existing condition to meet existing 10-year service levels. In other words, these pipes are already recommended for improvement in the SDMP CIP, but their recommended size must be adjusted to meet the sea level rise scenario. Pipes which are not highlighted represent new projects where no previous work was recommended to meet 10-year service levels under existing sea levels.

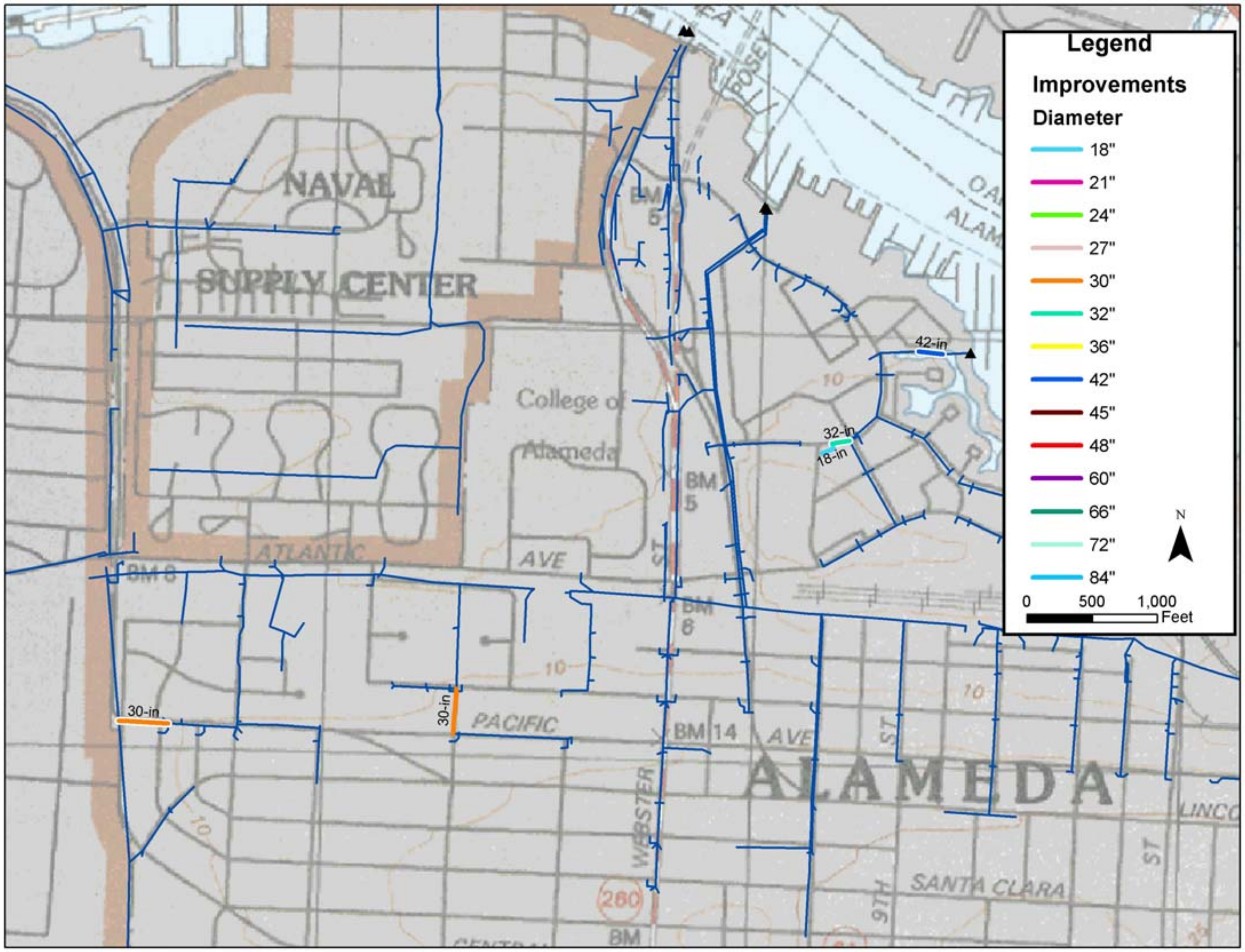

Figure 16: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island Northside

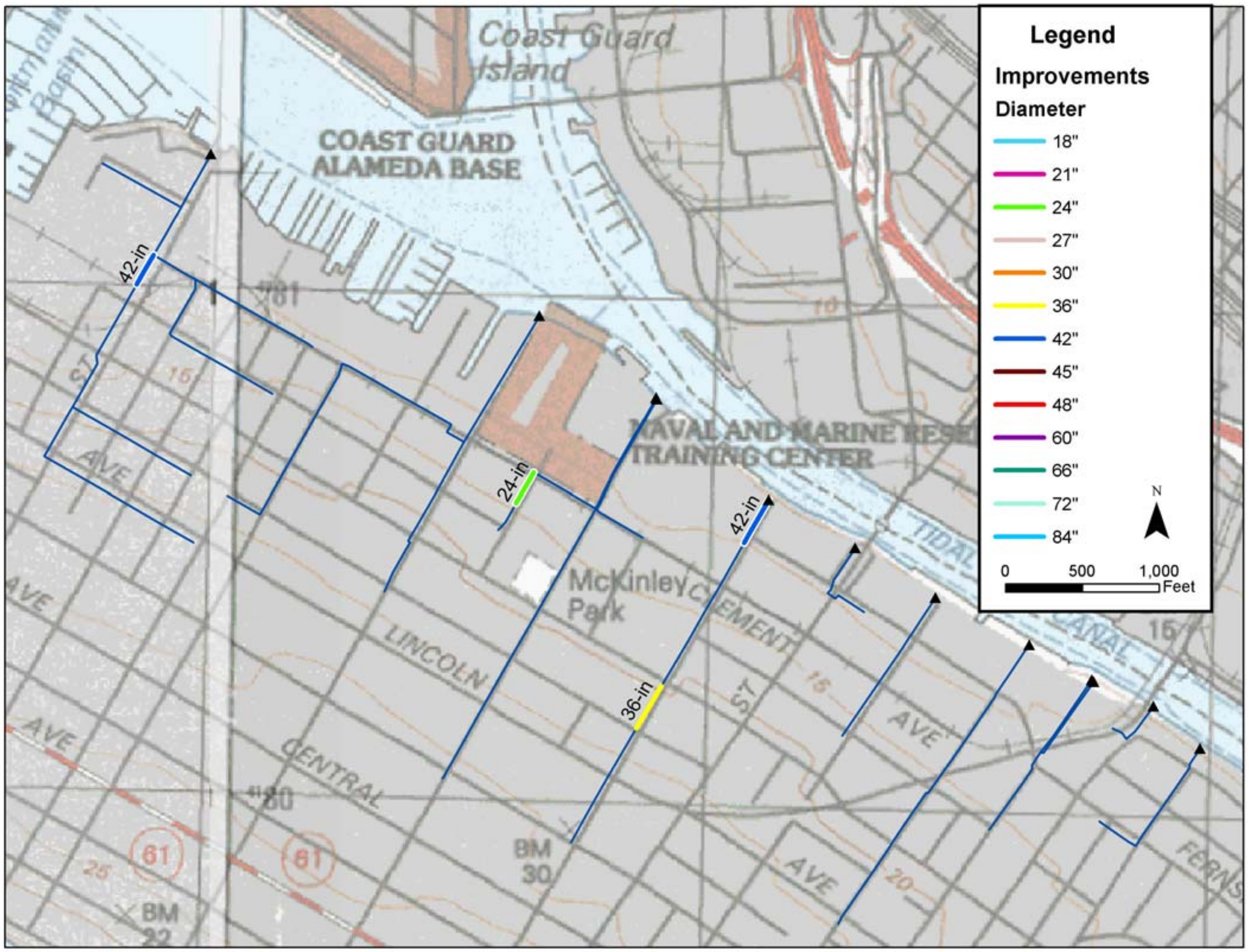

Figure 17: Improvements to Maintain 10-Year Level of Service with 18” of Sea Level Rise, Main Island North Central

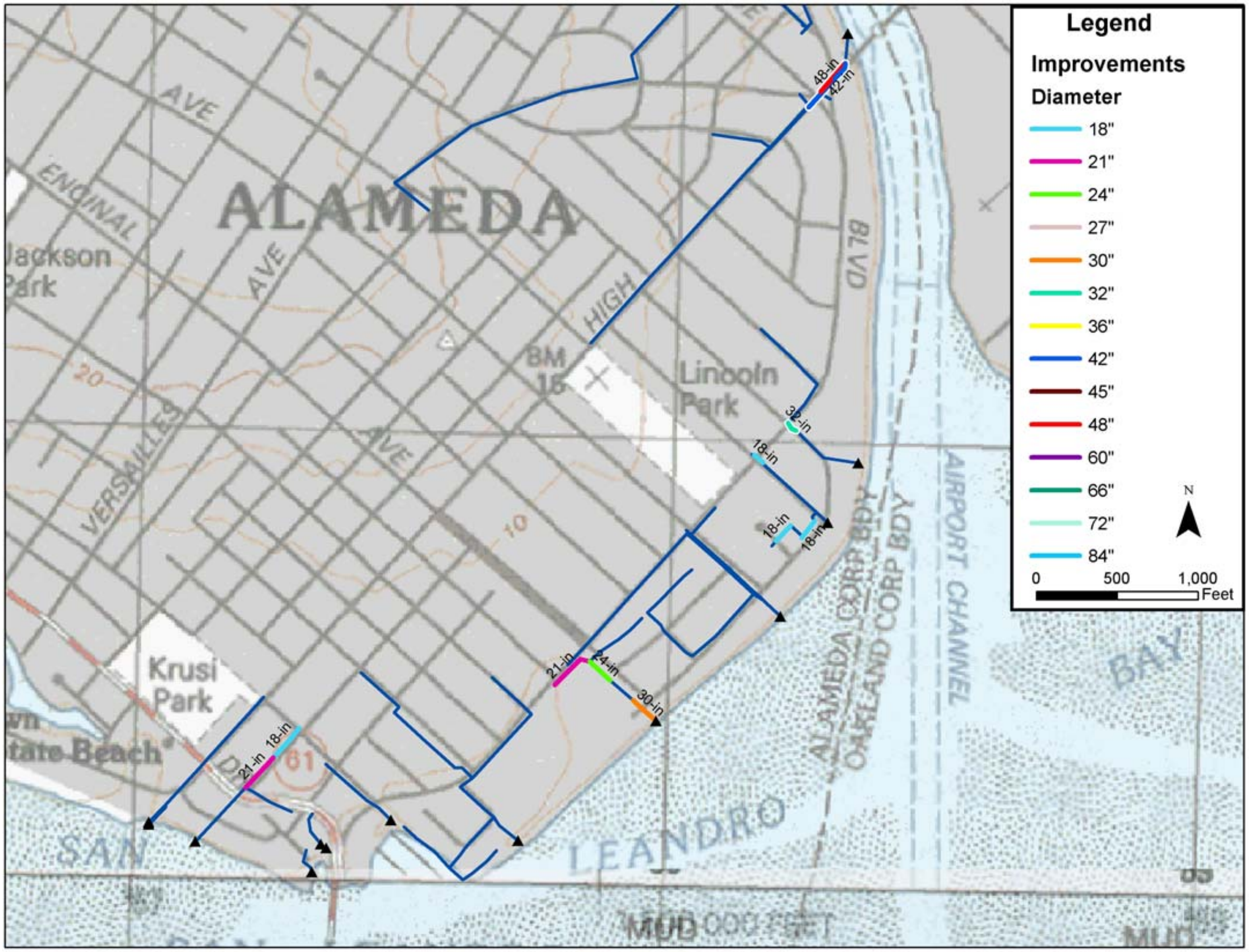

Figure 18: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island Eastside

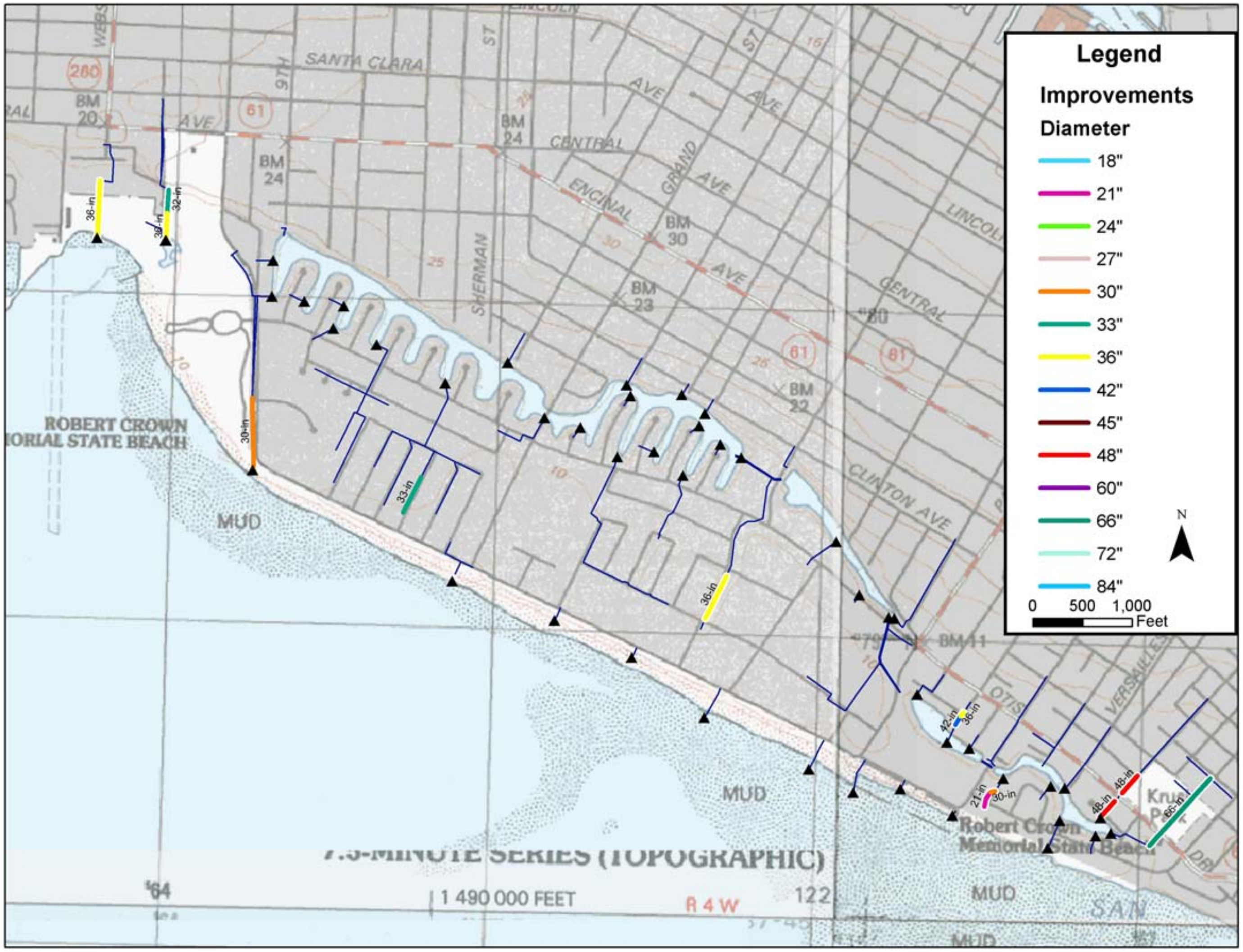

Figure 19: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island South

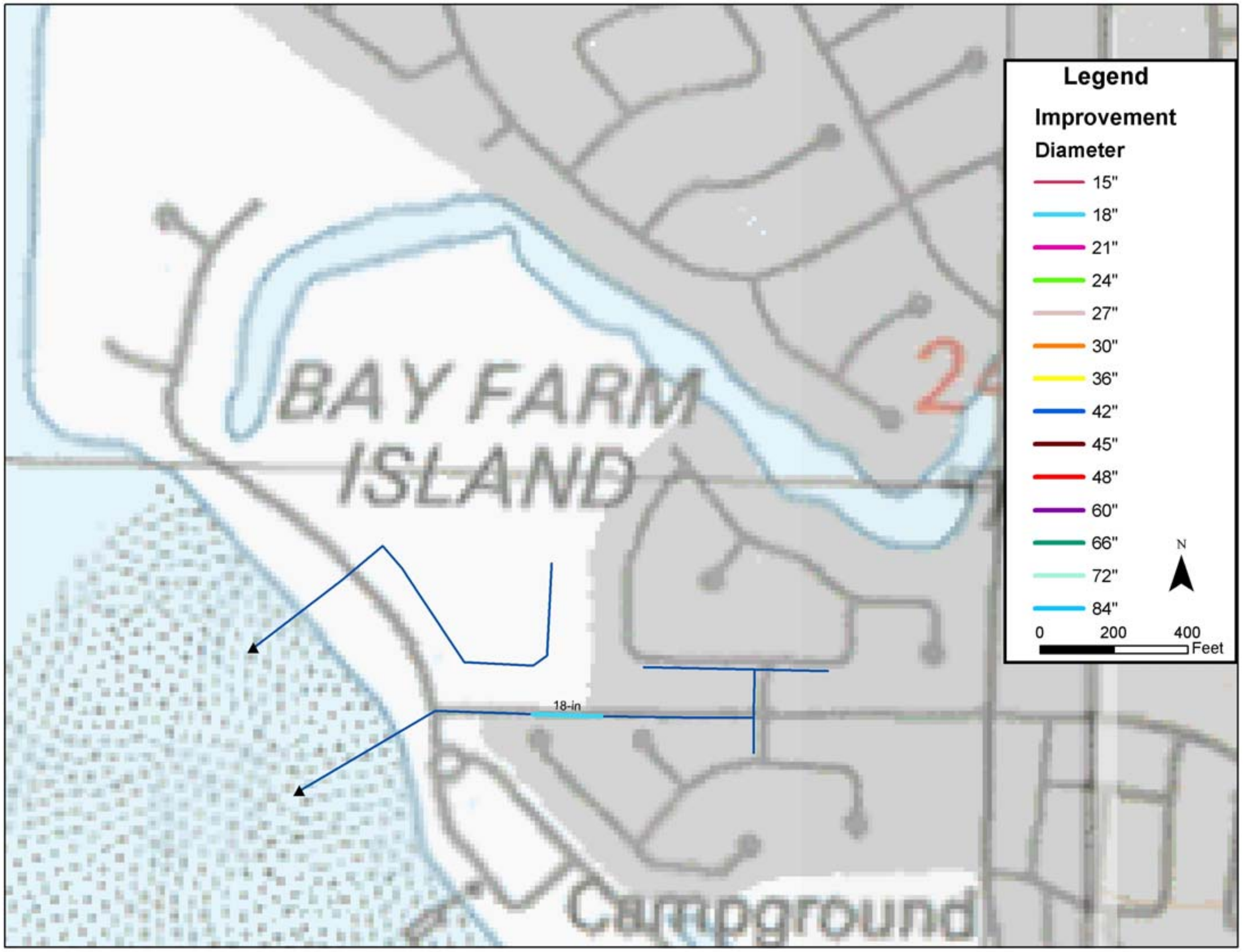

Figure 20: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Bay Farm Island South

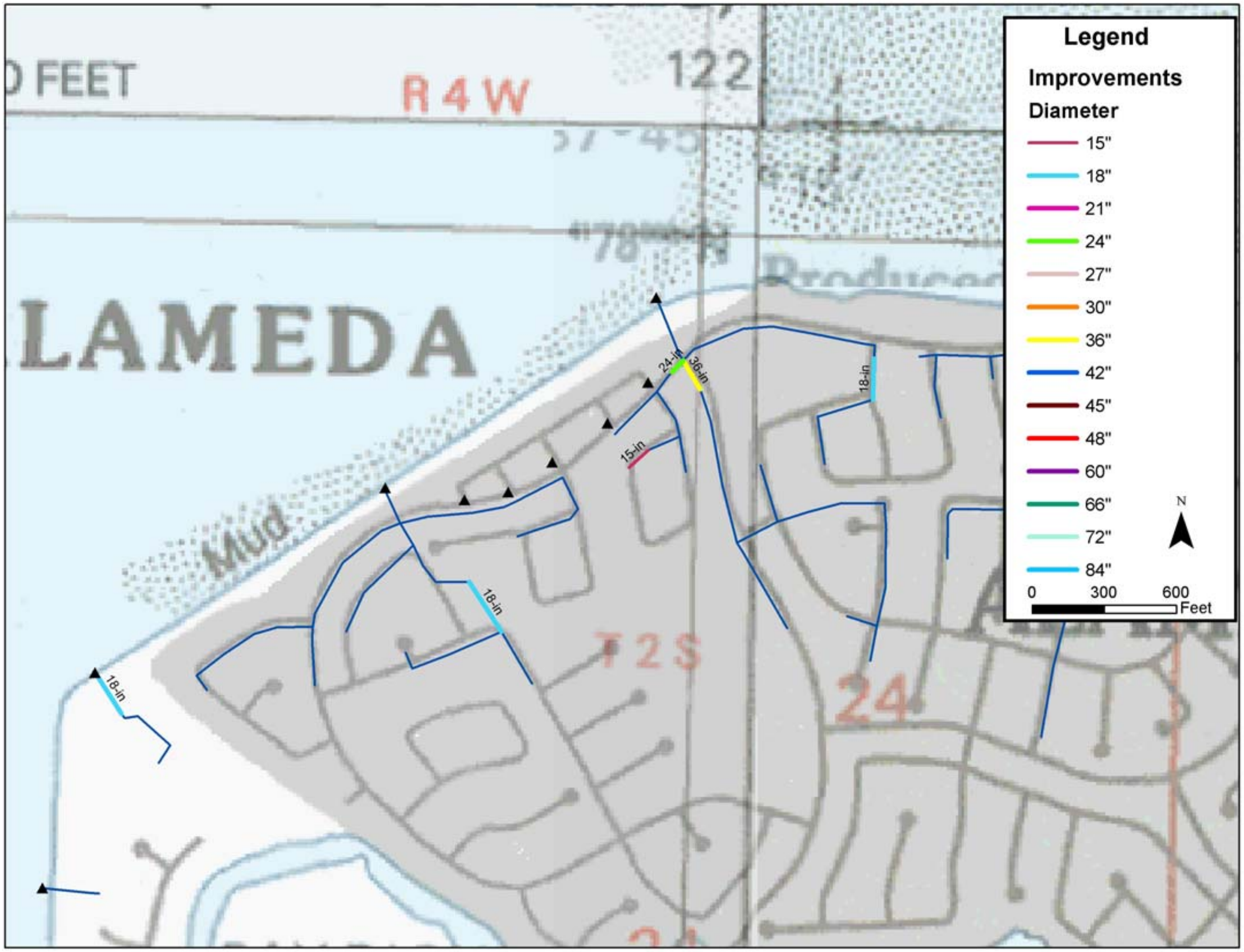

Figure 21: Improvements to Maintain 10-Year Level of Service with 18” of Sea Level Rise, Bay Farm Island North

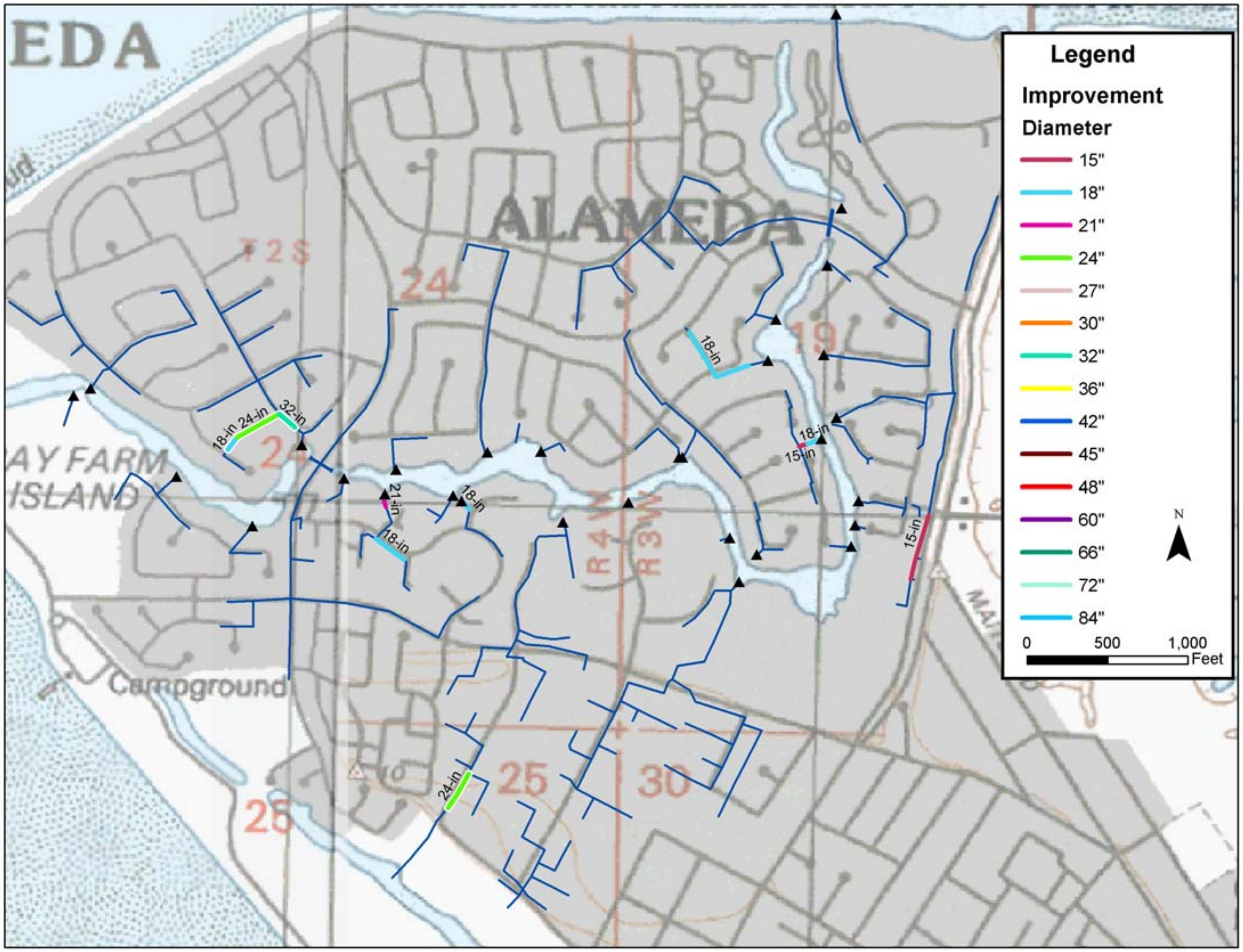

Figure 22: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Bay Farm Island Central

Impact of Sea Level Rise Improvements to Storm Drain Master Plan CIP

As shown in Figures 16 through 22 above, new pipe replacement projects, or increased pipe diameters compared to the SDMP CIP are required to maintain a 10-year level of service to the system in the event of 18” of sea level rise. Costs have been estimated using information from other projects, cost estimating guides (2009 Current Construction Costs, Saylor Publications, Inc.), and engineering judgment. These costs are summarized in Table 3.

Table 3: Storm Drain Cost Per Linear Foot

Diameter (inches)	Dollar per Linear foot of Pipe	Dollar per Connection
15	$116	$9,089
18	$128	$9,504
21	$150	$9,668
24	$172	$9,830
27	$194	$9,993
30	$216	$10,157
33	$241	$10,332
36	$267	$10,508
42	$300	$10,870
48	$335	$11,245
54	$369	$11,632
60	$413	$12,051
72	$502	$12,890
96	$679	$14,567

Table 4 summarizes the cost impact of these additional improvements. For pipes which are recommended for improvements in the SDMP (i.e. the highlighted pipes in Figures 14 through 20), the cost included in this table is the difference in costs between the SDMP recommended improvement and the size needed to provide the same level of protection for this sea level rise scenario. Note that costs presented in Table 3 do not include the 40% increase for design, administration, and contingency included in Table 4.

Table 4: Increase in Storm Drain Master Plan Capitol Improvement Program to Maintain 10-Year Level of Service with 18" of Sea Level Rise

City of Alameda Areas	Additional Costs to SDMP CIP
Main Island Eastside	$711,000
Main Island North Central	$190,000
Main Island South	$652,000
Main Island Northside	$234,000
MAIN ISLAND TOTAL	**$1,800,000**
Bay Farm Island North	$325,000
Bay Farm Island Central	$542,000
Bay Farm Island South	$70,000
BAY FARM ISLAND TOTAL	**$937,000**
CITY OF ALAMEDA TOTAL	**$2,700,000**

More detailed cost summary tables are included in Appendix A.

In addition to the costs of structural improvements (i.e. increased storm drain capacity requirements), there are also indirect costs to the City due to the sea level rise scenario. Due to the change in boundary conditions, storm drain pumps may run for longer periods of time, resulting in increased energy usage, maintenance and replacement costs. If the golf course is more often rendered unusable by flood waters, this could also indirectly impact City economics. While these costs are expected to be small compared to the improvement costs, they will be experienced regardless of projects undertaken to mitigate storm drain performance.

Current Status of Regulations Pertaining to Climate Change

The current status of potential regulations pertaining to climate change is explored below. Research and regulations regarding climate change are regularly, and sometimes rapidly, updated and modified; thus this section should be considered representative, and may not represent a complete list of current or pending regulations.

<u>Federal</u>

At a Federal level there are currently very few recommendations or guidelines for incorporating the risks of sea level rise into project planning, and virtually no required measures. It should be noted, however, that with the administration change of 2009, based on President Obama's statements that global warming is a priority of the new administration, relatively rapid changes in the Federal government's involvement in global warming analyses and impacts may be forthcoming. Thus far it appears that those changes will be focused on emission standards as opposed to impact mitigation.

Flood Programs - Federal Emergency Management Agency

Although the Federal Emergency Management Agency (FEMA) has issued several statements in the last decade pertaining to climate change and the risks of global warming, at this time FEMA policy has not changed to reflect these risks or impacts. Sea level rise is not directly considered in the National Flood Insurance Program (NFIP). In 2001 FEMA published a report on the projected impact of relative sea level rise on the NFIP, which concluded that the NFIP would not be significantly impacted by sea level rises under one foot by the year 2100, and the gradual timeframe of sea level rise provides ample opportunity for the NFIP to consider alternatives and implement them. The report recommended that FEMA should continue to monitor analyses and predictions of sea level rise and strengthen the Community Rating System (CRS) by encouraging measures that would mitigate the impacts of sea level rise (FEMA, 1991).

In March 2007 the United States Government Accountability Office published a report on the financial risks to federal and private insurers as a result of climate change, and recommended that the NFIP analyze the potential long-term fiscal implications of climate change and report these findings to Congress (GAO-07-285, March 2007). It is foreseeable that when this analysis takes place, changes to the NFIP will be made to lessen the financial risk to the insurers. Potential policy changes may include increased freeboard requirements for Bay or Riverfront levees and/or some consideration or discussion of sea level change in floodplain analyses, but when or if any policy changes will occur is unknown.

On March 17, 2009, the National Association of Insurance Commissioners (NAIC) adopted a mandatory requirement that insurance companies disclose to regulators the financial risks they face from climate change, and actions that the companies are taking to respond to those risks. This requirement impacts all insurance companies with annual premiums of $500 million or more. Those companies must complete an “Insurer Climate Risk Disclosure Survey” every year, with the first report due on May 1, 2010.

Research on Climate Change - National Oceanic and Atmospheric Administration

The National Oceanic and Atmospheric Administration (NOAA) is the federal agency that appears to have taken the lead in analyses of the impacts of global warming to the United States of America. NOAA is primarily a scientific research and reporting agency, with little regulatory power. From the NOAA webpage:

> "NOAA is charged with helping society understand, plan for, and respond to climate variability and change. This is achieved through the development and delivery of climate information services, the implementation of a global observing system, and focused research and modeling to understand key climate processes. The NOAA climate mission is an end-to-end endeavor focused on providing a predictive understanding of the global climate system so the public can incorporate the information and products into their decisions."

Recent budget proposals from President Obama suggest that this responsibility may shift from NOAA to NASA in the future.

<u>State</u>

California has been on the leading edge of creating legislation to mitigate both greenhouse gas emissions and the impacts of climate change. At this time, several concrete steps have been taken to reduce greenhouse gas emissions in the state, while specific impact mitigation strategies have been recommended but not fully developed. The California Climate Action Team, described in detail earlier in this report, is responsible for coordinating state-level actions relating to climate change.

Assembly Bill 32

The California Global Warming Solution Act, also known at Assembly Bill 32 (AB32), was signed into law by Governor Schwarzenegger in 2006. AB32 requires the California Air Resources Board (CARB) to:

- Establish a statewide greenhouse gas emissions cap for 2020, based on 1990 emissions by January 1, 2008.
- Adopt mandatory reporting rules for significant sources of greenhouse gases by January 1, 2009.
- Adopt a plan by January 1, 2009 indicating how emission reductions will be achieved from significant greenhouse gas sources via regulations, market mechanisms and other actions.
- Adopt regulations by January 1, 2011 to achieve the maximum technologically feasible and cost-effective reductions in greenhouse gas, including provisions for using both market mechanisms and alternative compliance mechanisms.
- Convene an Environmental Justice Advisory Committee and an Economic and Technology Advancement Advisory Committee to advise CARB.
- Ensure public notice and opportunity for comment for all CARB actions.

- Prior to imposing any mandates or authorizing market mechanisms, CARB must evaluate several factors, including but not limited to impacts on California's economy, the environment and public health; equity between regulated entities; electricity reliability, conformance with other environmental laws and ensure that the rules do not disproportionately impact low-income communities.

In September, 2008, Governor Schwarzenegger signed Senate Bill 375, which builds on AB32 by requiring the CARB to develop regional greenhouse gas emission reduction targets to be achieved from the automobile and light truck sectors for 2020 and 2035. Both AB32 and Senate Bill 375 focus on reducing greenhouse gas emissions, as opposed to predicting or mitigating climate change impacts in California.

AB 32 Scoping Plan

The AB 32 Scoping Plan contains the main strategies California will use to reduce the greenhouse gases (GHG) that cause climate change. The Scoping Plan has a range of GHG reduction actions which include direct regulations, alternative compliance mechanisms, monetary and non-monetary incentives, voluntary actions, market-based mechanisms such as a cap-and-trade system, and an administrative fee to fund the program. The AB 32 Scoping Plan was approved at the Air Resources Board hearing on December 11, 2008.

Six greenhouse gas emission reduction measures are proposed for the Water sector in the Scoping Plan. They address water use efficiency, water recycling, water system energy efficiency, reuse of urban runoff, increased renewable energy production and public goods charges for funding investments that improve water and energy efficiency (CARB, 2008).

California Environmental Quality Act

The California Governor's Office of Planning and Research is expected to certify and adopt amendments to the CEQA Guidelines which incorporate analyses and mitigation of Greenhouse Gas Emissions (GHG) on or before January 1, 2010 (CA Governor's Office, 2008). In the interim, the Office of Planning and Research has created a technical advisory which includes the recommended approach for incorporating climate change impacts to the CEQA process. The recommended approach includes recommendations for approaches to identifying project GHG emissions, determining significance, and mitigating the impacts.

California Adaptation Strategy

In November, 2008, Governor Schwarzenegger signed Executive Order S-13-08 (EO), which calls for the development of California's first statewide climate change adaptation strategy to assess the state's expected climate change impacts, vulnerabilities, and recommend climate adaptation policies, to be completed by 2009. This is the first legislative action to initiate active planning for the impacts of global warming in the state of California. In addition to the climate change adaptation strategy, the EO also requests that the National Academy of Science establishes an expert panel to report on sea level rise impacts in California, issues interim guidance to state agencies for how to plan for sea level rise in designated coastal and floodplain areas for new projects, and initiates a report on critical infrastructure (planned and existing) vulnerable to sea level rise. In the interim, all state agencies planning construction projects are directed to consider a range of sea level rise scenarios for the years 2050 and 2100 in order to assess project vulnerability and, to the extent feasible, reduce expected risks and increase resiliency to sea level rise (CA Governor Press Release, 2008).

California Water Plan

Following the passage of AB 32 in 2006 which called for a reduction in greenhouse gas emissions, DWR voluntarily joined the California Climate Action Registry. DWR addresses climate change in its California Water Plan, updated every five years, that provides a framework for water managers, legislators, and the public to consider options and make decisions regarding California's water future. In July, 2008, DWR published a technical memorandum report on the progress of incorporating climate change into the management of California's water resources. The focus of this report was the impact of global warming to California's water supply, although increased flood risks were presented in brief. In October 2008, the Department released a climate change white paper that proposes a series of adaptation strategies for state and local water managers to improve their capacity to handle change. On a regional level these strategies include integrated water management and increased water use efficiency.

Local

City of Alameda

The City of Alameda completed a Local Action Plan for Climate Protection in February 2008. This Plan identified initiatives to reduce City-wide greenhouse gas emissions by 25% of the 2005 levels by 2020. Initiatives are divided into four categories: transportation and land use, energy, waste and recycled, and community outreach and education. Initiatives that are particularly relevant to new or re-development include (City of Alameda, 2008):

- Requirement that all new major developments' short and long-term transportation emissions impacts are reduced by 10%;
- Require that all recommended City Council actions include an analysis or evaluation of whether the action supports or is consisten with Alameda's Local Action Plan Initiatives and futhers progress toward the Greenhouse Gases Reduction Target;
- Ammend the Alameda Municipal Code to include sustainable design and green building standards for all new, substaintially expanded, and remodeled buildings; and
- Develop a wood-burning prohibition ordiancne to reduce air pollution for new residential construction.

East Bay Municipal Utility District

The East Bay Municipal Utility District (EBMUD) provides domestic water to the City of Alameda. EBMUD's primary source of water is the Mokelumne River watershed, which is fed by snowpack in the Sierra Mountains. In addition to the current drought, climate change is expected to decrease snow pack, and thus snow melt and water supply, in coming years. In 2008, EBMUD incorporated climate change into its strategic plan, and is currently pursuing water conservation, water recycling, and seeking out additional water sources for future use (Wallis, 2008). The City of Alameda already has several water conservation programs, but additional reductions may eventually be required by EBMUD to address decreasing water supply as a result of climate change.

San Francisco Bay Conservation and Development District

The San Francisco Bay Conservation and Development District (BCDC) is a state agency created in 1965 to regulate development in the Bay and along its shoreline for the purpose of limiting and controlling the amount of fill placed in the Bay. In October 2007 BCDC released an eight year regional strategy for controlling greenhouse gases and preparing for the impacts of sea level rise of San Francisco Bay. BCDC does not have the authority or responsibility to initiate many of the identified strategies. In September 2008 BCDC released a revised strategy which considers the regulatory limitations of the agency.

In May 2009, BCDC submitted preliminary recommendations for amendments to the Bay Plan to incorporate climate change. This proposal adopts sea level rise estimates of 16 inches (1.3 feet) by 2050 and 55 inches (4.6 feet) by 2100. Proposed changes to the Bay Plan which may be relevant to the City include the following (Travis, 2009):

- "Addressing the impacts of sea level rise and shoreline flooding may require large-scale flood protection projects, including some that extend across jurisdictional or property boundary. Coordination with adjacent property owners or jurisdictions to create contiguous, effective shoreline protection is critical when planning and constructing flood protection projects. Failure to coordinate may result in inadequate shoreline protection (e.g., a protection system with gaps or one that causes accelerated erosion in adjacent areas)"

- "New shoreline protection projects and the maintenance or reconstruction of existing projects should be authorized if: (a) the project is necessary to proect the shoreline from erosion or to protect shoreline development from flooding; (b) the type of the protective structure is appropriate for the project site, the uses to be protected, and the erosion and flooding conditions at the site, (c) the project is properly engineering to provide erosion control and flood protection for the expected life of the project based on a 100-year flood event that takes future sea level rise into account; (d) the project is properly designed and constructed to prevent signiciant impediments to physical and visual public access; and (e) the protection is integraded with adjacent shoreline protection measures."

- "…the Commission should…encourage new projects on the shoreline to be set back from the edge of the shore above a 100-year flood level that takes future sea level rise into account for the expected life of the project, or otherwise be specifically designed to tolerate sea level rise and stroms and to minimize environmental impacts; discourage new projects that will require new structureal shoreline protection during the expected life of the projects, especially where no shoreline protection currently exits [*sic*]; determine whether alternative measures that would involve less fill or impacts to the Bay are feasible; require an assessment of risks from a 100-year flood that takes future sea level rise into account for the expected life of the project; and require that where shoreline protection is necessary, ecosystem impacts are minimized.".

- "The Commission may approve fill that is needed to provide flood protection for existing projects. New projects on fill or near the shoreline should either be set back from the edge of the shore so that the project will not be subject to dynamic wave energy, be built so the bottom floor level of structures will be above a 100-year flood elevation that takes future sea level rise into account for the expected life of the project, be specifically designed to tolerate periodic flooding, or employ other effective means of addressing the impacts of future sea level rise and storm activity. Right-of-way for levees or other structures protecting inland areas from tidal flooding should be sufficiently wide on the upland side to allow for future levee widening to support additional levee height so that no fill for levee widening is placed in the Bay."

- “Design and evaluation (of any ecosystem restoration project) should include an analysis of: (a) how the system’s adaptive capacity can be enhanced so that it is resilient to sea level rise an climate change…(h) an appropriate buffer, where feasible, between shoreline development and habitats to protect wildlife and provide space for marsh migration as sea level rises…”.

- “Public access should be sited, designed, managed, and maintained to avoid significant adverse impacts from sea level rise and shoreline flooding.”

These changes, if approved, may have significant impacts on the City’s approach to development, planning, and design of both flood control projects and new or re-development within portions of the City.

Other Storm Drain Master Plan Updates

During the preparation of this report, the Northside (Marina Village) pump station experienced failure during an approximately 10-year storm event. The impact to the City’s storm drainage system operation during the storm due to this failure was significant. The Northside (Marina Village) pump station is connected via storm drains to the Arbor pump station. In the SDMP, standby power is identified as high priority improvements for both the Arbor and Northside (Marina Village) pump stations. Capacity improvements are also recommended at the Arbor pump station. In addition, the operation of the Main Island lagoon system has experienced elevation water levels during storm events.

Due to the recent pump station failure and its consequences, the Northside (Marina Village) pump station is currently a high priority storm drain pump station for potential improvement. The pump stations’ generators, panels, pumps, motors and layout need to be optimized to provide the highest level of service possible. It may be feasible that increasing the capacity of the Northside (Marina Village) pump station while this work occurs would reduce or remove the need for increased capacity at Arbor pump station. A more detailed analysis of how this connected system operates and whether improvements at Northside (Marina Village) may offset recommended improvements at Arbor pump station is needed. Additional analysis and eventual improvements of the Main Island Lagoon system are also needed.

Summary

As an island community, the City of Alameda is uniquely exposed to climate change impacts to the San Francisco Bay region, particularly rising sea levels. Rising sea levels may impact the City via both inundation of City lands by higher mean sea levels and tide cycles, and also may impact the capacity and operation of its storm drain system. At this time, structural projects to mitigate inundation from surrounding waters, such as floodwalls, levees, elevating structures, etc. are not recommended due to the inherent uncertainty and long time scale of sea level rise projections. If structural solutions are sought in the long term, coordination with the Oakland Airport adjacent to Bay Farm Island will be essential to protect that portion of the City.

Projects have been identified to maintain a 10-year level of service for the storm drain network in the event of 18" of sea level rise. Many of these storm drain system improvements are increased pipe diameters in areas already identified for capacity improvements in the SDMP. The impact to the SDMP CIP is $2,700,000. The majority of the additional required improvements are on the Main Island, and of those, the majority are located in the South and Eastside areas. Given the relatively low cost to install a slightly larger pipe if a pipe replacement project is already planned, Schaaf & Wheeler recommends using the pipe sizes herein for pipe replacement projects undertaken in the future. Replacing pipes for the sole purpose of meeting the sea level rise scenario 10-year level of service should be considered low priority.

Regulations regarding climate change are currently in a state of rapid development and fluctuation. At this time, the most significant existing regulations potentially affecting the City are those contained in the City Local Action Plan. Based on our findings, Schaaf & Wheeler concludes that it is likely that significant development of the former naval base in the future will be required to study and mitigate for not only greenhouse gas emissions, but also future sea level rise scenarios.

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Appendix A: Detailed Cost Calculations

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island Eastside						**$687,000**	**$179,000**	**$508,000**
SL141	1.25	15	1	75		$17,324	-	
SL142	1.50	18	1	166		$30,263	-	
SL146	1.75	21	2	206		$49,165	-	
SL195	1.75	21	1	225		$42,823	-	
SL2_2	1.75	21	1	48		$16,395	-	
SL206	1.75	21	1	60		$18,095	-	
SL207	2.00	24	2	172		$48,181	-	
SL275	1.50	18	2	133		$35,018	-	
SL280	1.50	18	2	134		$35,143	-	
SL300	2.50	30	1	126		$36,996	-	
SL325	2.50	30	1	67		$24,126	-	
SL97	1.50	18	2	72		$27,313	-	
SLIMP_104	2.75	33	1	41	21	$19,797	$15,369	
SLIMP_105	2.75	33	2	31		$27,152	-	
SLIMP10_44	3.50	42	1	237	36	$81,345	$73,267	
SLIMP10_49	4.00	48	2	234	42	$99,696	$90,781	
SLIMP10_50	3.50	42	1	96		$38,957	-	
SLIMP10_51	3.50	42	1	15		$14,794	-	
SLIMP10_52	3.50	42	2	13		$24,565	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island North Central						**$407,000**	**$271,000**	**$136,000**
SLimp03l1	3.0	36	2	319		$105,049	-	
SLimp184	2.0	24	2	55	18	$28,064	$25,058	
SLimp230	2.0	24	1	159	18	$36,566	$29,350	
SLimp232	2.0	24	1	18	18	$12,416	$11,322	
SLimp276	3.5	42	1	78	36	$33,828	$30,924	
SLimp279	3.5	42	2	245	36	$93,976	$85,302	
SLimp430	3.5	42	1	108	36	$42,660	$38,794	
SLimp431	3.5	42	2	113	36	$54,634	$50,243	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island South						**$1,586,000**	**$1,165,000**	**$421,000**
SLimp449l1	2.75	33	2	399		$116,008	-	
SLimp178	3.00	36	1	28	30	$17,579	$15,800	
SLimp178_1	3.50	42	2	103	33	$51,582	$44,526	
SLimp178_2	3.00	36	1	21	24	$15,653	$12,978	
SLimp220	1.75	21	2	79	18	$30,218	$28,199	
SLimp223	1.75	21	1	81	18	$21,263	$19,368	
SLimp224	2.50	30	1	47	24	$19,843	$17,423	
SLimp376l1	4.00	48	2	233	42	$99,429	$90,542	
SLimp378l1	4.00	48	2	84	42	$49,583	$45,914	
SLimp379l1	4.00	48	1	142	42	$58,368	$53,018	
SLimp408l3	5.50	66	2	256	60	$128,660	$128,660	
SLimp415l2	5.50	66	1	235	60	$108,573	$108,573	
SLimp425l1	5.50	66	1	236	60	$108,931	$108,931	
SLimp426l1	5.50	66	1	55	60	$34,214	$34,214	
SLimp427l1	5.50	66	1	132	60	$66,129	$66,129	
SLimp99	3.00	36	2	476	24	$147,034	$100,315	
SLimpF06-511l1	3.00	36	1	91	30	$34,333	$29,370	
SLimpF06-512l1	3.00	36	1	487	30	$140,026	$114,976	
SLimpF06-612l1	2.75	33	1	234	30	$66,233	$60,226	
SLimpF06-615l1	3.00	36	1	198	33	$62,813	$57,545	
SLimpF06-619l1	3.00	36	1	84	30	$32,482	$27,870	
SLintake3l1	2.50	30	1	55		$21,494	-	
SLintakel1	2.50	30	1	673		$155,192	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island Northside						**$375,000**	**$208,000**	**$167,000**
SLD381EC0A14C702DA	1.50	18	1	39		$14,026	-	
SLD381EC0A14C702F2	1.50	18	1	95		$21,162	-	
SLD381EC0A14C71C52	2.50	30	2	21		$23,817	-	
SLD381EC0A14C71C5B	2.50	30	1	324		$79,735	-	
SLimpD381EC0A14C70200	3.50	42	2	188	36	$76,902	$70,086	
SLimpD381EC0A14C702A3	2.75	32	2	134	27	$48,214	$44,901	
SLimpD381EC0A14C7173B	2.50	30	1	295	24	$73,497	$59,986	
SLimpE05-111I1	2.50	30	2	85	24	$37,712	$33,280	
Bay Farm Island North						**$232,000**	**$0**	**$232,000**
SL176	1.50	18	2	248		$49,866	-	
SL208	1.50	18	1	204		$35,108	-	
SL32	1.25	15	2	108		$29,766	-	
SL48	3.00	36	1	139		$47,043	-	
SL53	2.00	24	2	66		$29,958	-	
SL79	1.50	18	2	171		$39,931	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Bay Farm Island Central						**$541,000**	**$154,000**	**$387,000**
SL105	1.50	18	1	94		$21,118	-	
SL166	1.75	21	1	37		$14,674	-	
SL167	1.75	21	2	44		$24,936	-	
SL176	1.50	18	2	72		$27,278	-	
SL241	1.50	18	2	323		$59,394	-	
SL242	1.50	18	1	216		$36,739	-	
SL290	1.25	15	1	56		$15,110	-	
SL291	1.50	18	2	94		$30,059	-	
SL420	1.50	18	2	219		$46,069	-	
SL620	1.25	15	2	275		$49,156	-	
SL621	1.25	15	1	69		$16,697	-	
SL623	1.25	15	1	67		$16,401	-	
SLimp106	2.00	24	1	303	18	$61,315	$47,825	
SLimp93	2.75	33	2	130	30	$50,886	$47,316	
SLimps53	2.00	24	2	151	18	$44,577	$37,384	
SLimps57	2.00	24	1	100	18	$26,463	$21,809	
Bay Farm Island South						**$50,000**	**$0**	**$50,000**
SL25	2.00	24	2	185		$50,424	$0	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Appendicies
Storm Drain Master Plan
Alameda, California

August, 2008

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

LIST OF APPENDICES

APPENDIX C

The City of Alameda wished to know the expected flooding depth of the 25-year storm event, and the required projects to apply the same standard of allowable flooding to the City infrastructure for the 25-year storm. This Appendix presents the results of that analysis. Table 1-1 in Chapter 1 is repeated here to show the summary of the required projects to apply the improvement standard to both the 10- and 25-year storm events.

Table 1-1: Summary of Master Plan Costs

Master Plan Improvements, Alameda Island	**Eastside**	**North Central**	**Northside**	**South**	**Total**
Projects to Meet 10-Year Standard	$8,470,000	$9,686,000	$24,261,000	$11,999,000	$54,416,000
Projects to Meet 25-Year Standard	$11,940,000	$10,796,000	$37,311,000	$13,149,000	$73,196,000
Master Plan Improvements, Bay Farm Island	**East**	**North**	**Central**	**South**	**Total**
Projects to Meet 10-Year Standard	$2,550,000	$2,600,000	$4,590,000	$1,960,000	$11,700,000
Projects to Meet 25-Year Standard	$2,700,000	$3,210,000	$6,340,000	$6,570,000	$18,820,000

The total costs summary for the 25-year CIP projects along with the required lengths are shown for each priority level in Table A-1. Each subarea includes the recommended capacity improvements, including pump station capacity improvements. Also included in the table are recommended pump station upgrades such as self cleaning trash racks and on site backup power (as shown in Table 7-11). These costs include a 40% increase in construction cost estimates to include design, administration, and contingency costs.

Table A-1 Summary of 25-Year CIP Costs

Alameda Island						
	High		Medium		Low	
	Length	Cost	Length	Cost	Length	Cost
Northside	17,000	$29,780,000	2,300	$1,550,000	13,700	$5,360,000
North Central	0	$0	11,000	$4,540,000	12,300	$5,790,000
Eastside	10,000	$9,570,000	6,000	$2,320,000	0	$0
South	3,600	$2,060,000	15,900	$6,740,000	8,300	$3,560,000
Total Alameda Island	30,600	$41,410,000	35,200	$15,150,000	34,300	$14,710,000
Bay Farm Island						
	High		Medium		Low	
	Length	Cost	Length	Cost	Length	Cost
North	0	$600,000	2,900	$1,650,000	1,200	$960,000
South	0	$0	7,100	$3,410,000	3,800	$3,160,000
East	0	$0	5,700	$2,450,000	600	$250,000
Central	0	$0	7,800	$2,840,000	8,900	$3,500,000
Total Bayfarm Island	0	$600,000	23,500	$10,350,000	14,500	$7,870,000
TOTAL:	30,600	$42,010,000	58,700	$25,500,000	48,800	$22,580,000

Not included in Table A-1 are the costs to extend storm drain lines on Alameda Island, as described in the text of the report. This accounts for the discrepancy between the total values between Tables 1-1 and A-1.

Pump stations may operate differently in a 25-year storm event due to changes in the flow delivered to the pump station and in the tide cycle. Table A-2 presents the model-generated results for pump station operation in a 25-Year storm event, and the additional required capacity to meet the 25-year improvement standards.

Table A-2: Pumping Station Summary with 25-Year Storm Drain Improvements

Station Name	Location (Watershed)	Year Built or Updated	Design Capacity of Existing Station (GPM)	Actual Station Peak Discharge from Model (GPM)	Additional Req'd Station Discharge (GPM)
Main Street	Alameda Northside	1998	13,500 GPM	11,900 GPM	0 GPM
Third Street	Alameda Northside	1993	1,650 GPM	2,000 GPM	2,000 GPM
Webster Street	Alameda Northside	1947	5,250 GPM	2,400 GPM	0 GPM
Northside (Marina Village)	Alameda Northside	1984	72,000 GPM	89,800 GPM	73,300 GPM
Arbor	Alameda Northside	1994	31,600 GPM	38,200 GPM	57,000 GPM
Central / Eastshore	Alameda Eastside	1967	8,600 GPM	9,100 GPM	13,500 GPM
Bayport	Alameda Northside	2004	42,600 GPM	44,000 GPM	0 GPM
Golf Course	Bay Farm East	1986	19,200 GPM*	GPM	GPM

* Pump design capacity data based on bid documents

It should be noted that the 'Actual Station Peak Discharge' column is the peak outflow from the pump stations with the existing pipe network. In some locations, most notably at Marina Village & Webster Street Pump Stations, recommended pipe network improvements act to improve pump station operating capacity, even though additional capacity is not added via new pumps. For Eastshore, Arbor, Northside and 3rd Street Pump Stations, the additional capacity must be achieved via new pumps at the stations.

For each sub area, first the existing depths for the 25-year storm event are presented, followed by a table describing the improvements required to meet the standard set for by the City. The same methodology for determining improvements and assigning priority levels was used for the 25-year scenario as described within the report for the 10-year scenario. A figure showing these improvements with their priority level highlighted is next, followed finally by a large scale figure showing the recommended improvement pipe sizing, extent, and location.

Figure A-1: Alameda Eastside Area Existing 25-Year Flooding Depths

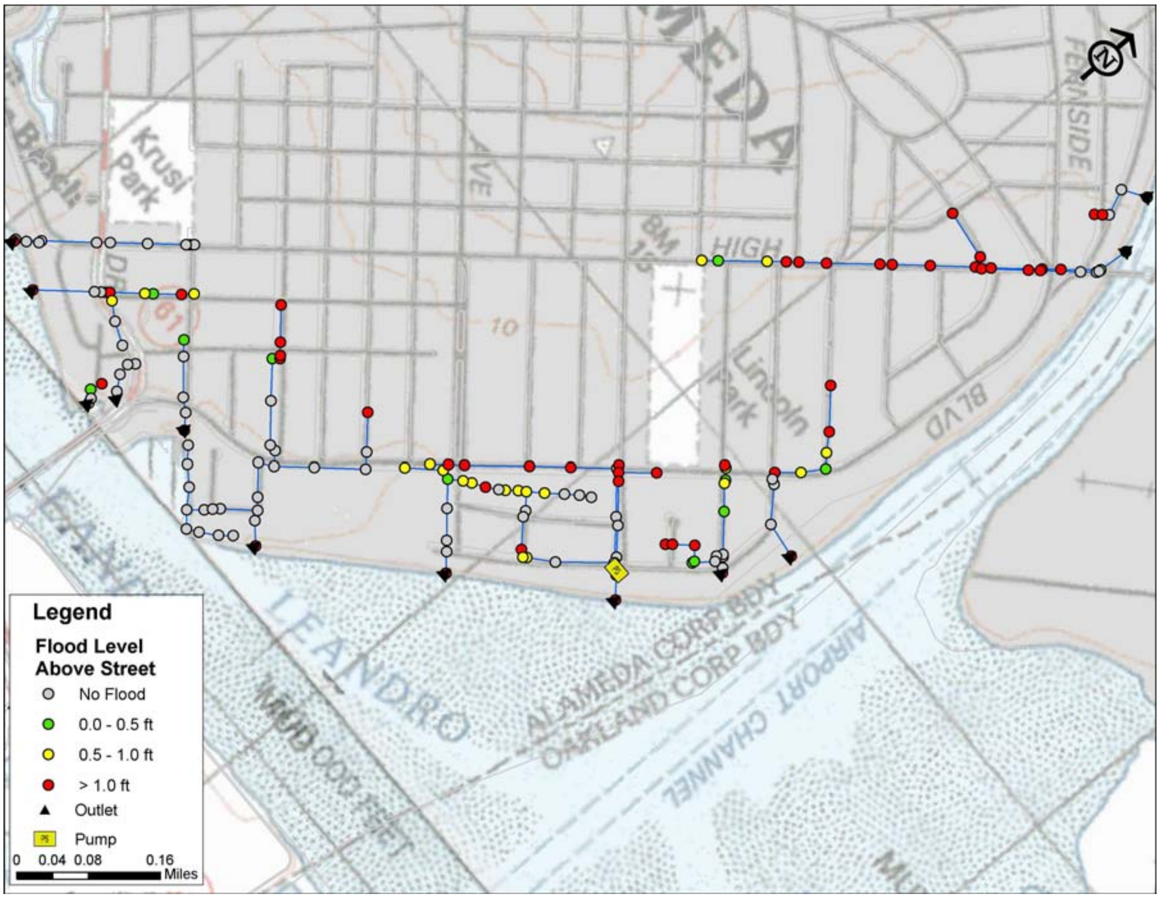


Table A-3: Alameda Island, Eastside Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Thompson	High	1344	11	1	$404,000	$566,000
Gibbons (new pipe)	High	4000	13	1	$1,121,000	$1,569,000
Liberty	High	509	8	1	$172,000	$241,000
Encinal	High	359	3	0	$121,000	$169,000
High	High	3776	26	1	$1,380,000	$1,932,000
Fernside	Moderate	2930	16	0	$754,000	$1,056,000
Washington	Moderate	1849	13	0	$575,000	$805,000
Post	Moderate	660	6	1	$175,000	$245,000
Calhoun	Moderate	534	5	1	$154,000	$216,000

Figure A-2: Alameda Eastside Area Prioritized 25-Year Improvements

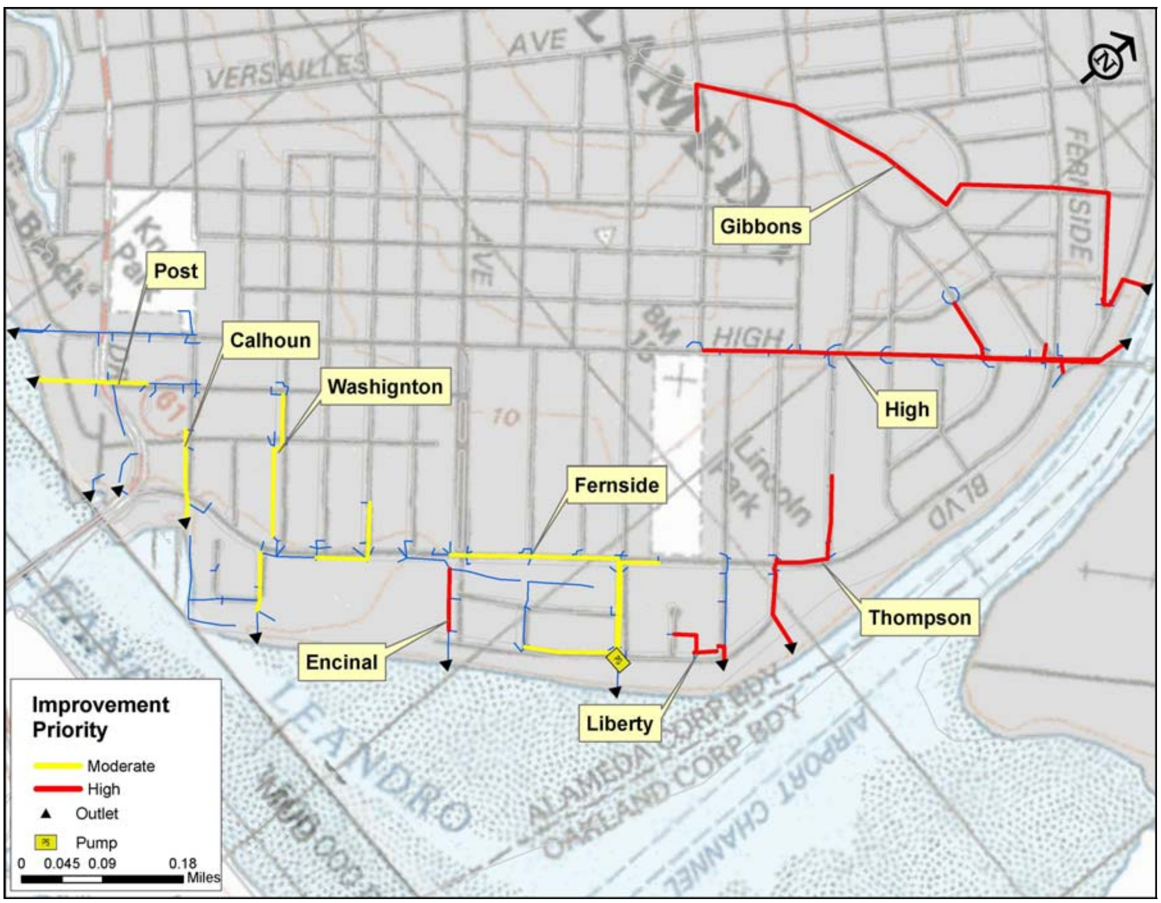


Figure A-3: Alameda North Central Area Existing 25-Year Flooding Depths

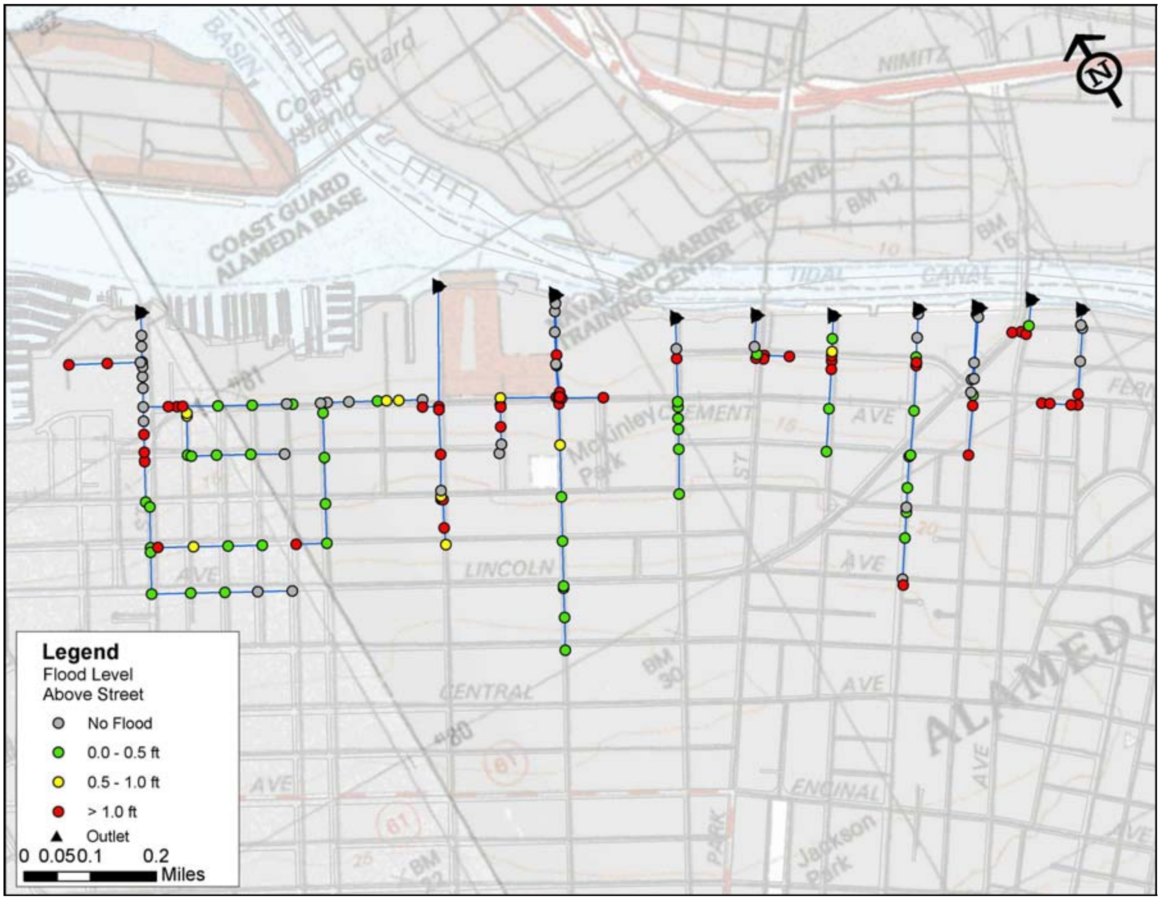


Table A-4: Alameda Island, North Central Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Grand	Moderate	6356	39	1	$1,898,000	$2,657,000
Clement	Moderate	4611	23	1	$1,343,000	$1,880,000
Walnut	Low	4357	24	1	$1,382,000	$1,935,000
Oak	Low	1399	9	1	$469,000	$657,000
Park	Low	740	8	1	$261,000	$365,000
Everett	Low	1086	8	1	$390,000	$546,000
Broadway	Low	2159	15	1	$700,000	$980,000
Pearl	Low	1189	8	1	$375,000	$525,000
Tilden	Low	395	5	1	$136,000	$190,000
Cambridge	Low	986	8	1	$424,000	$594,000

Figure A-4: Alameda North Central Area Prioritized 25-Year Improvements

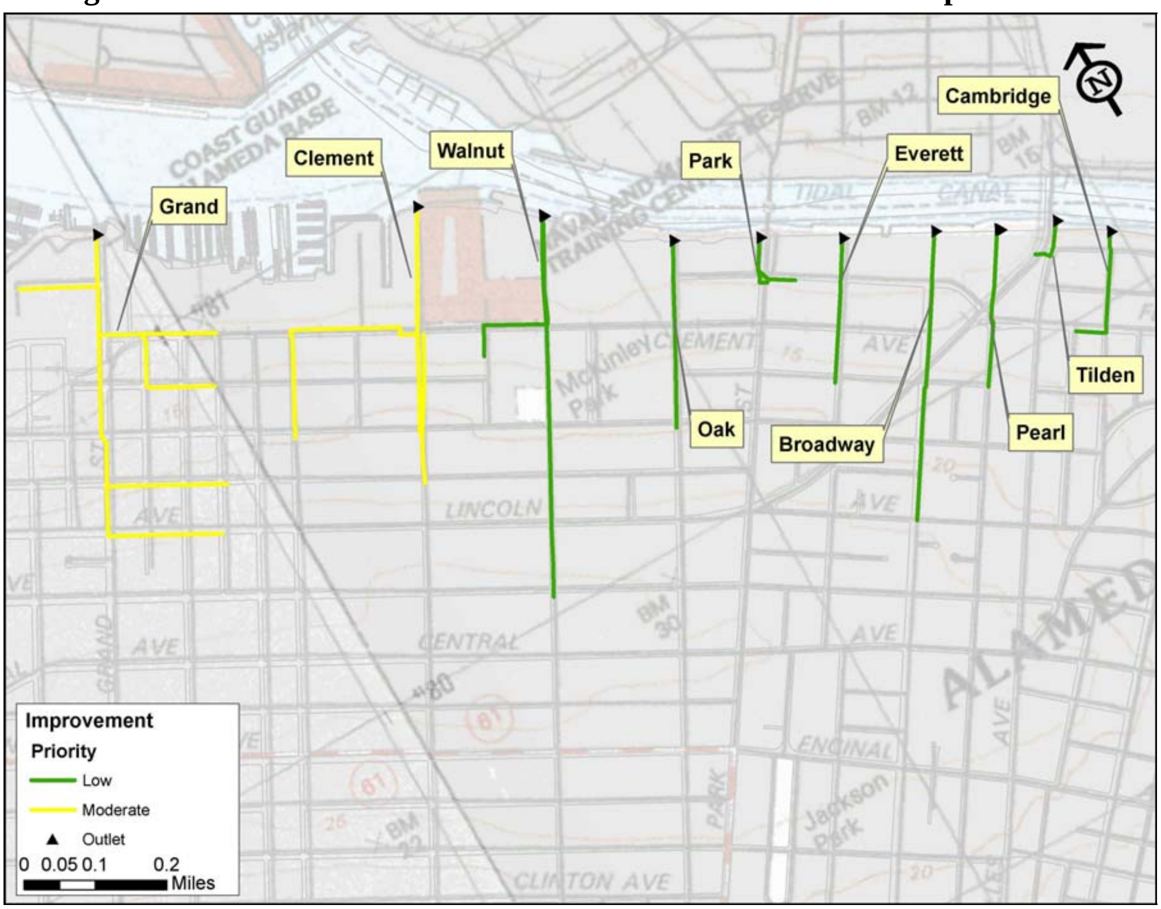


Figure A-5: Alameda Northside Area Existing 25-Year Flooding Depths

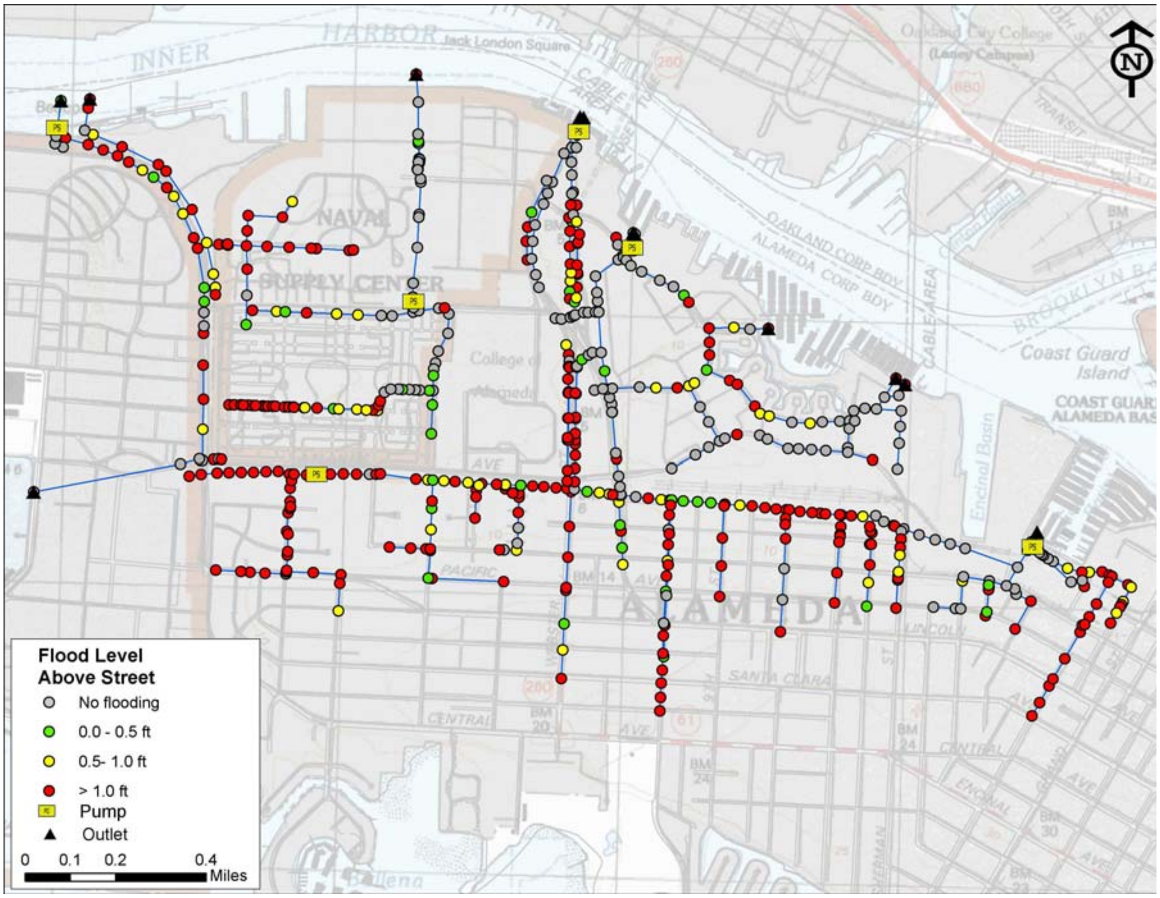


Table A-5: Alameda Island, Northside Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Constitution	High	3300	12	1	$2,324,000	$3,254,000
West Atlantic	High	3400	26	1	$2,800,000	$3,920,000
East Atlantic (1)	High	2900	22	0	$1,454,000	$2,036,000
East Atlantic (2)	High	3300	24	1	$1,787,000	$2,502,000
New Outfall	High	4100	8	1	$2,281,500	$3,194,000
Marina Village Parkway	Med	2300	12	1	$749,000	$1,049,000
Main St	Low	2300	11	0	$549,000	$769,000
Webster (2)	Low	2400	19	1	$690,000	$966,000
3rd Street	Low	400	2	0	$81,000	$113,000
Webster (3)	Low	1900	7	0	$480,000	$672,000
9th Street	Low	1100	5	0	$337,000	$472,000
Chapin	Low	300	4	0	$109,000	$153,000
Bay Sherman St. Charles	Low	1500	16	0	$447,000	$626,000
Paru	Low	1300	13	0	$419,000	$587,000
5th	Low	800	5	0	$280,000	$392,000
8th	Low	1700	6	0	$440,000	$616,000

Figure A-6: Alameda Northside Area Prioritized 25-Year Improvements

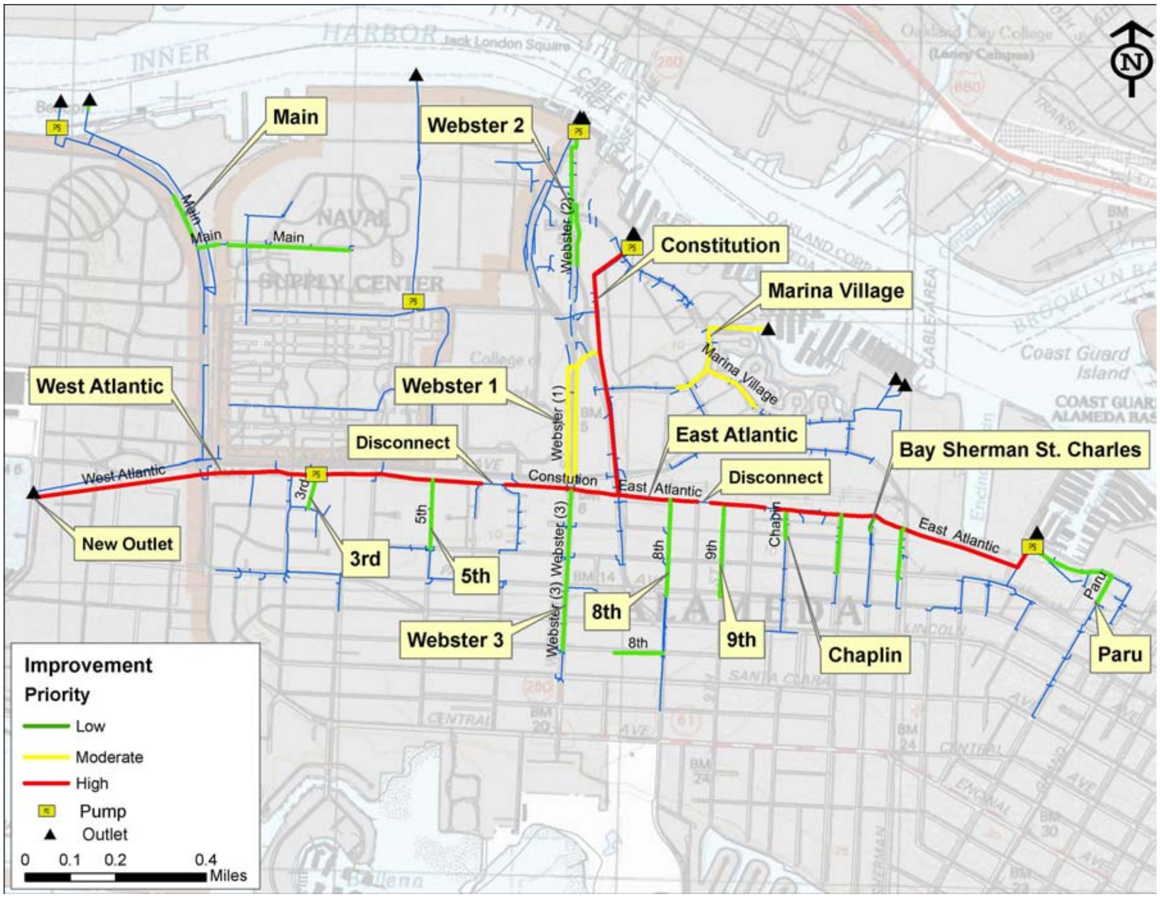


Figure A-7: Alameda South Area Existing 25-Year Flooding Depths

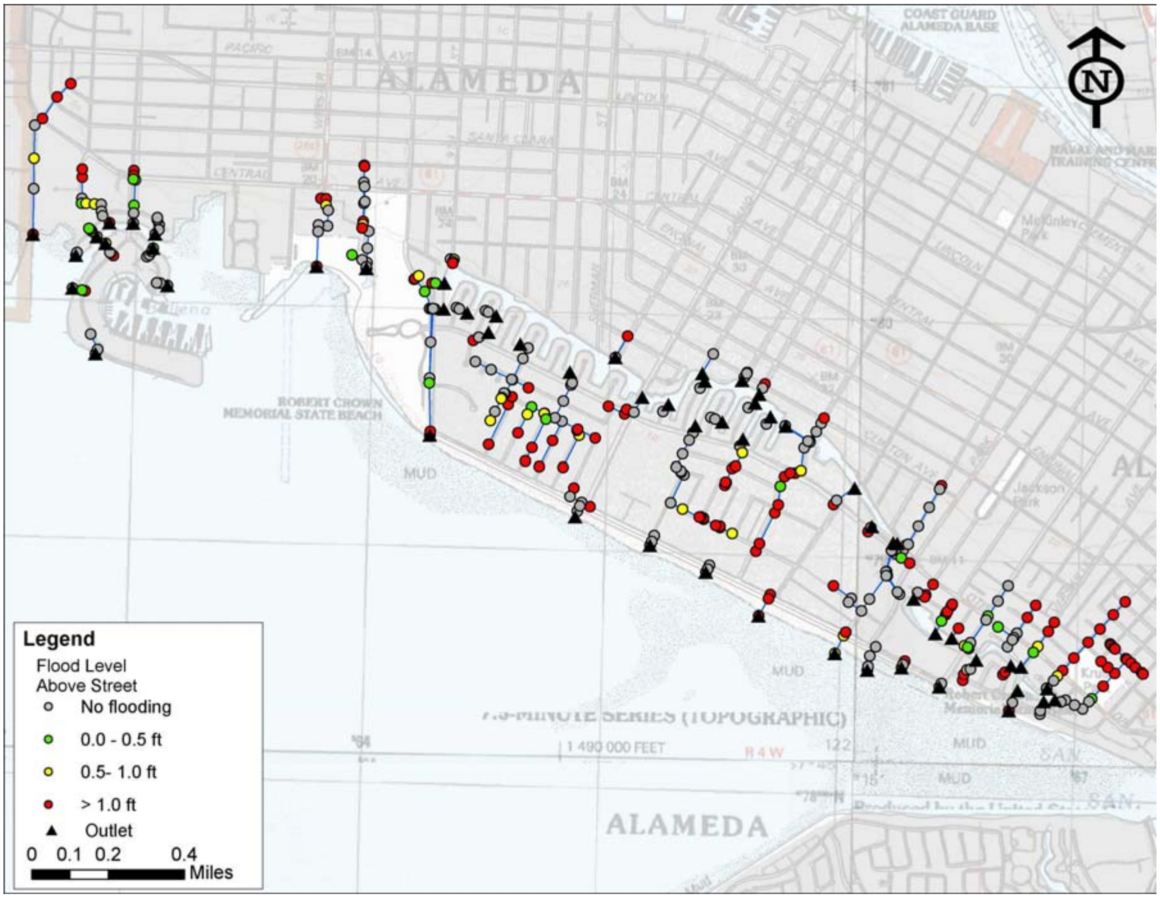


Table A-6: Alameda Island, South Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Fountain	High	2000	20	1	$919,000	$1,287,000
Mound	High	1600	10	1	$553,000	$774,000
Franciscan	Moderate	2700	16	0	$732,000	$1,025,000
Heather	Moderate	4100	23	1	$1,196,000	$1,674,000
Shell Gate	Moderate	2300	20	1	$641,000	$897,000
School	Moderate	800	5	1	$282,000	$395,000
Pearl	Moderate	700	6	0	$294,000	$412,000
12th	Moderate	2300	7	1	$641,000	$897,000
3rd	Moderate	800	7	1	$251,000	$351,000
Willow	Moderate	1700	10	0	$602,000	$843,000
S Shore Center W	Moderate	500	4	1	$176,000	$246,000
Regent	Low	500	7	1	$212,000	$297,000
Park	Low	1000	9	1	$401,000	$561,000
Page	Low	2100	17	1	$564,000	$790,000
Webster	Low	1200	9	1	$337,000	$472,000
Ballena	Low	800	8	1	$260,000	$364,000
Dayton (NEW)	Low	100	2	0	$25,000	$35,000
Union	Low	100	2	0	$27,000	$38,000
Balboa	Low	200	4	0	$71,000	$99,000
Laguna Vista	Low	46	2	0	$24,000	$34,000
Oak	Low	1200	8	1	$318,000	$445,000
Shoreline	Low	800	7	1	$193,000	$270,000
Otis	Low	300	4	0	$114,000	$160,000

Figure A-8: Alameda South Area Prioritized 25-Year Improvements

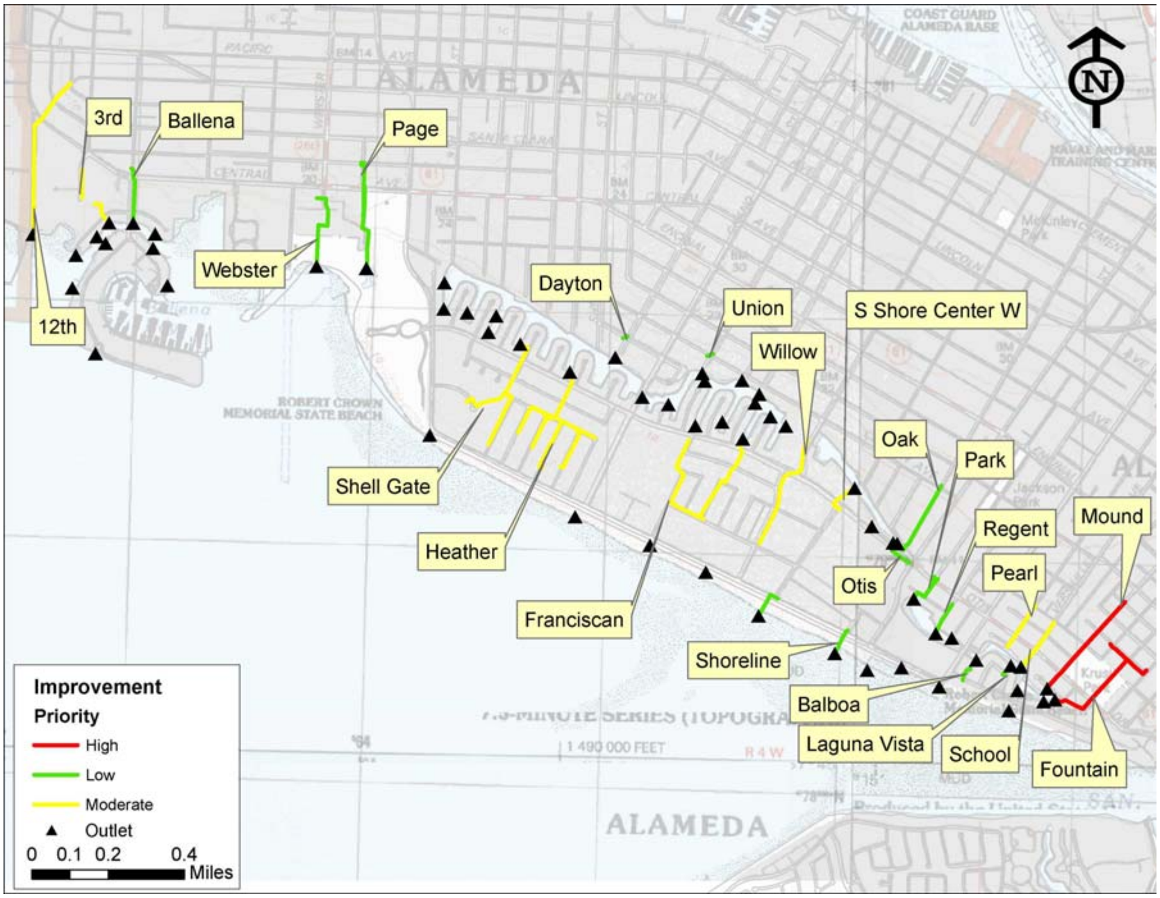


Figure A-9: Bay Farm East Area Existing 25-Year Flooding Depths

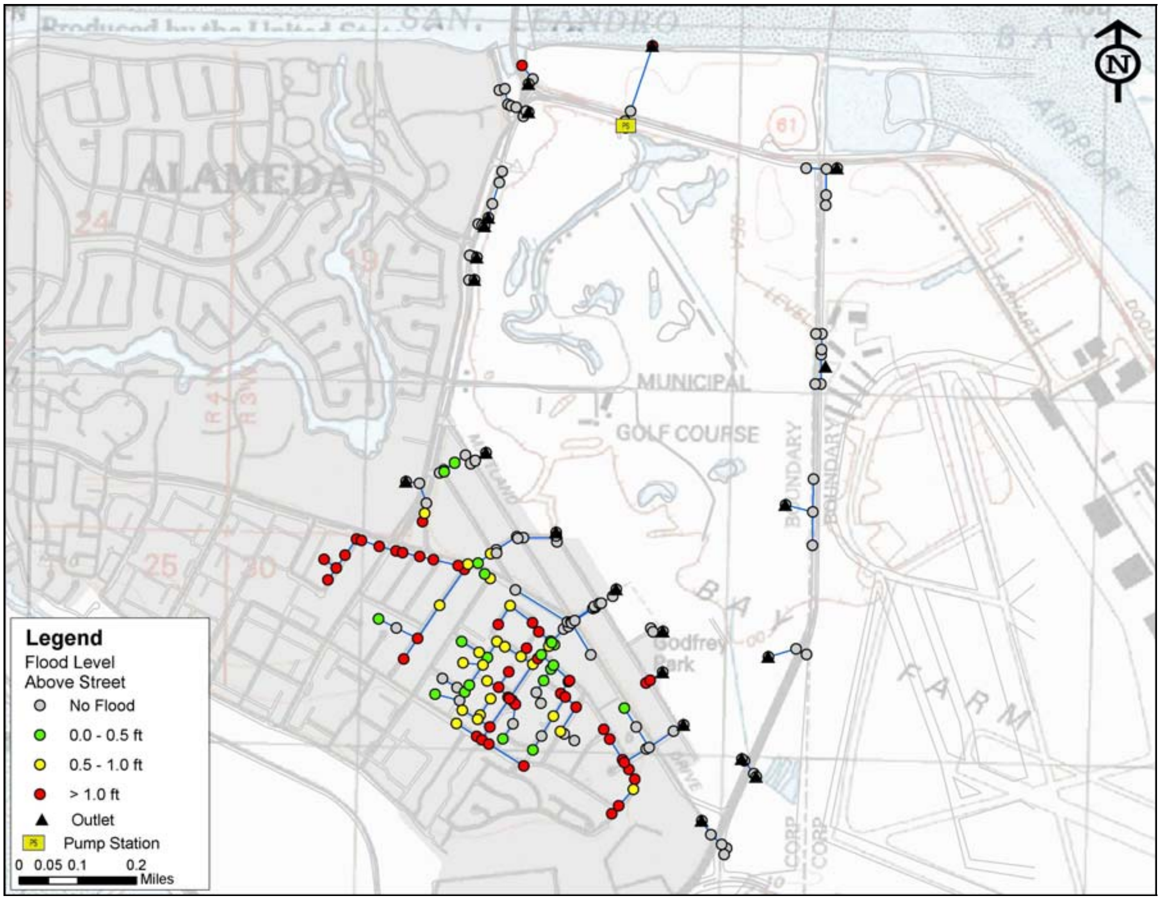


Table A-7: Bay Farm Island, East Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Camelia	Moderate	3212	23	0	$897,000	$1,255,800
Melrose	Moderate	2479	23	0	$854,000	$1,195,600
Fitchburg	Low	632	5	0	$178,000	$249,200

Figure A-10: Bay Farm East Area Prioritized 25-Year Improvements

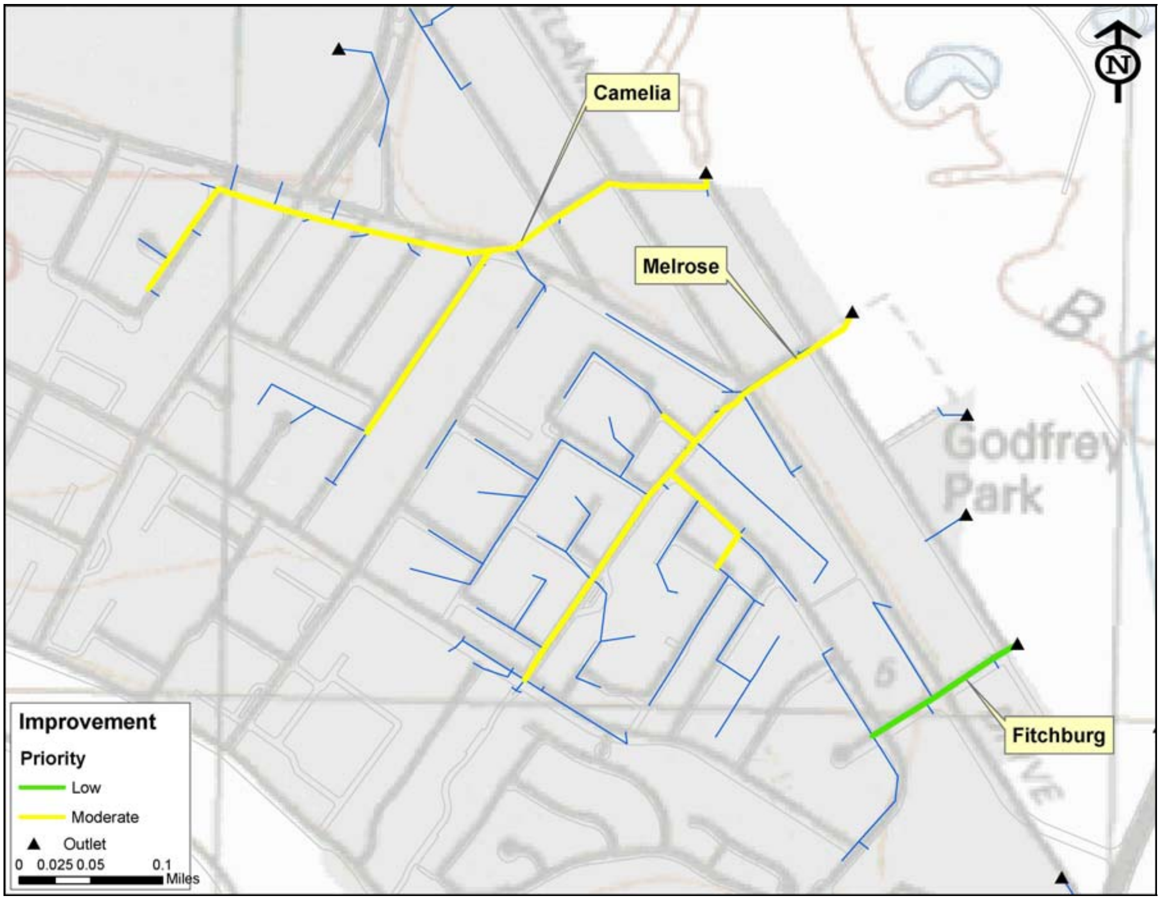


Figure A-11: Bay Farm North Area Existing 25-Year Flooding Depths

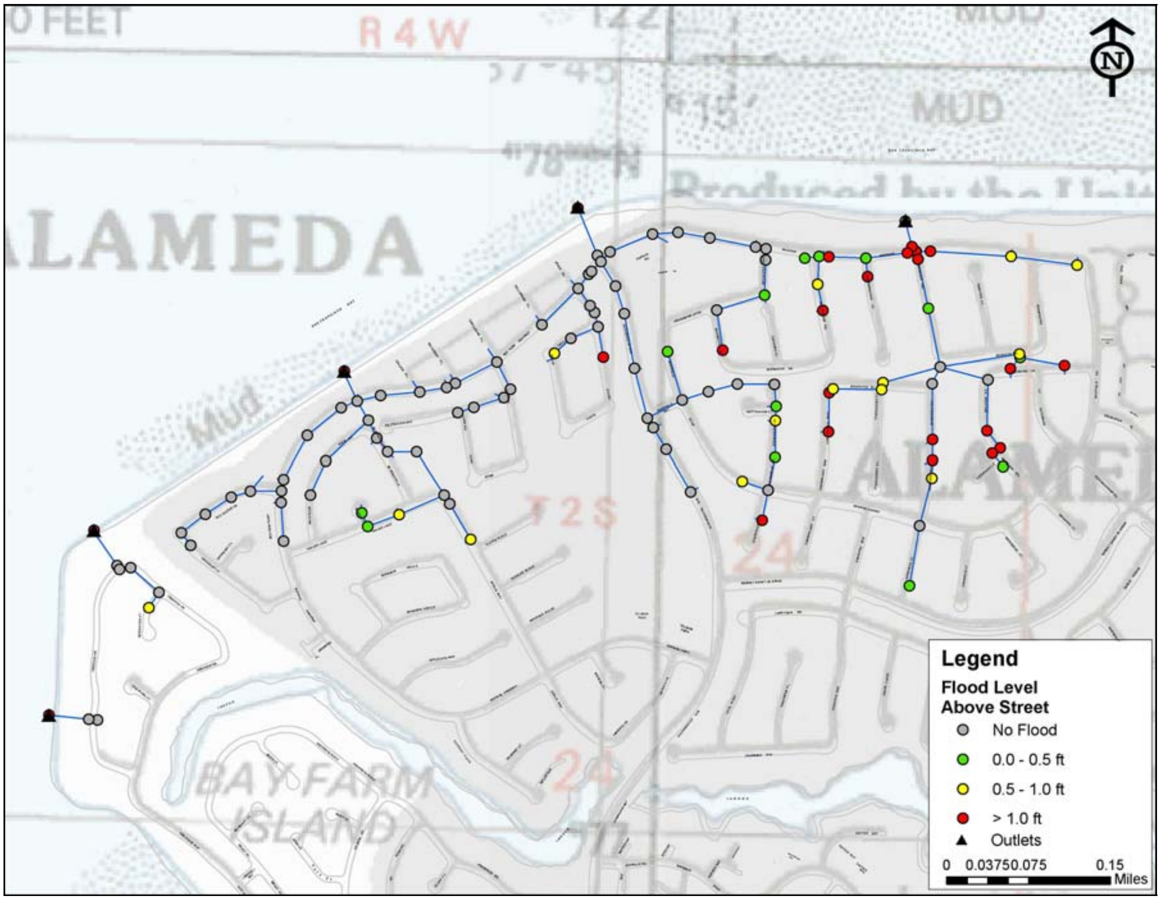


Table A-8: Bay Farm Island, North Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Stanbridge	Moderate	1000	5	0	$265,000	$371,000.0
Avington	Moderate	1851	8	1	$487,000	$681,800.0
Shamrock	Low	710	11	0	$135,000	$189,000.0
Justin	Low	319	5	0	$80,000	$112,000.0
Brunswick	Low	171	2	0	$39,000	$54,600.0

Figure A-12: Bay Farm North Area Prioritized 25-Year Improvements

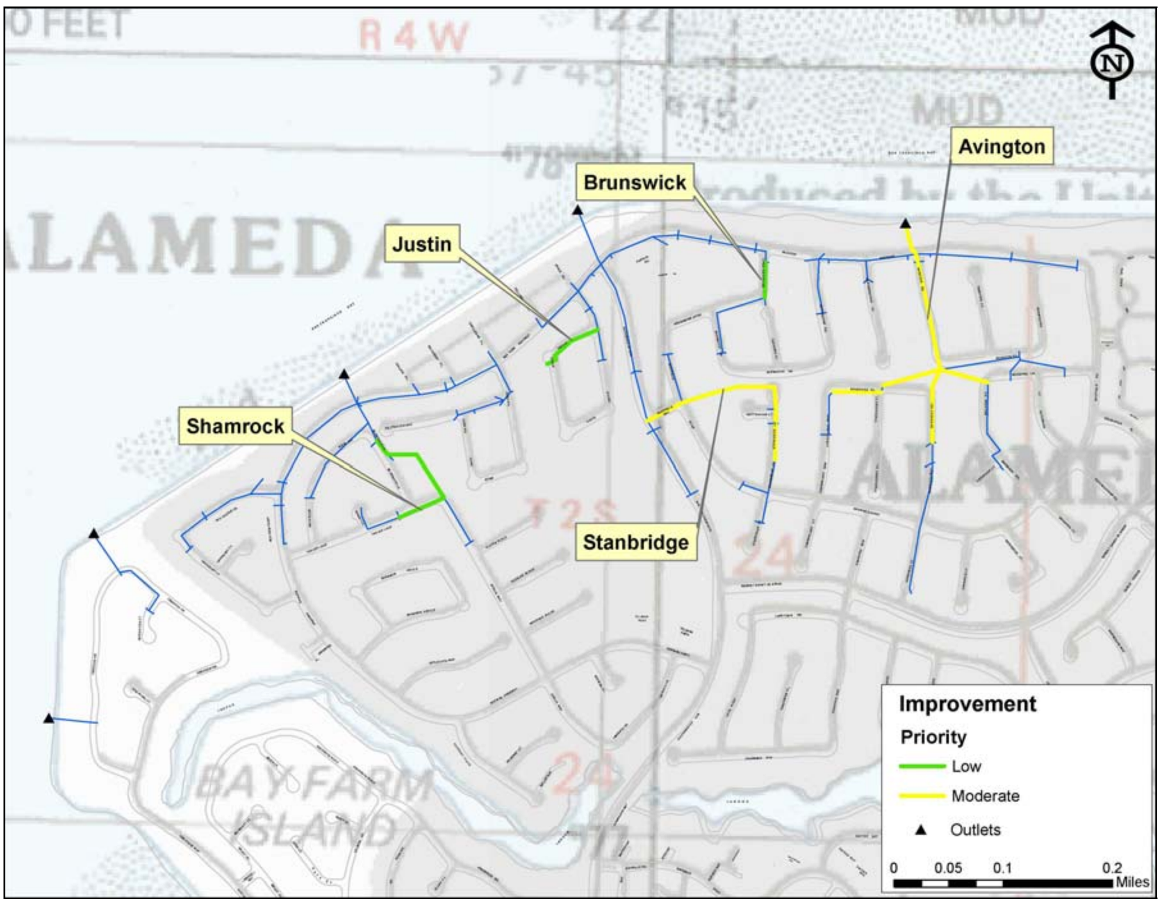


Figure A-13: Bay Farm Central Area Existing 25-Year Flooding Depths

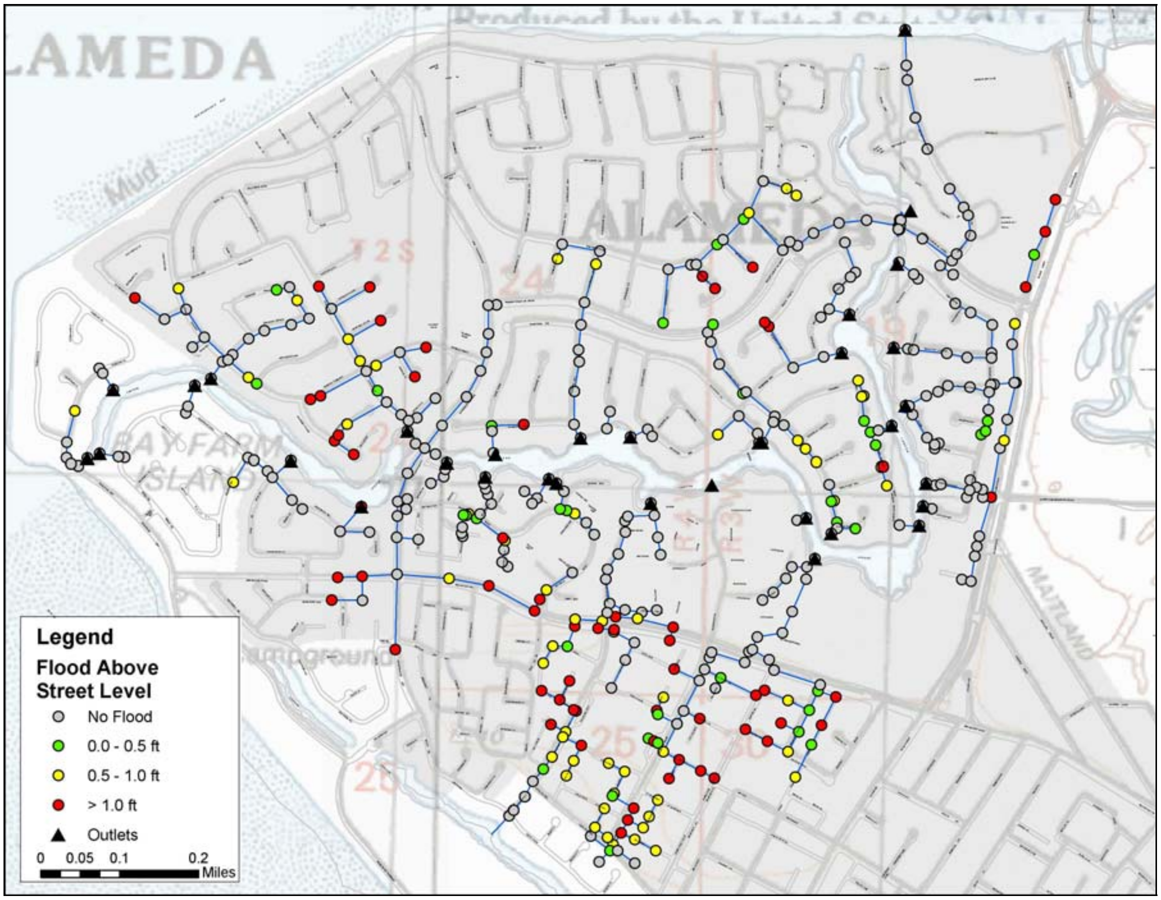


Table A-9: Bay Farm Island, Central Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Dublin Way	Moderate	1900	13	1	$475,000	$665,000
Island Dr	Moderate	692	5	0	$159,000	$222,600
Catalina Ave	Moderate	525	7	0	$151,000	$211,400
Fontana Drive	Moderate	1276	13	0	$317,000	$443,800
Verdemar Drive	Moderate	3367	26	1	$930,000	$1,302,000
Robert Davey Jr Dr	Low	1437	10	0	$417,000	$583,800
Capetown Court	Low	616	6	2	$197,000	$275,800
Baywood	Low	1633	16	1	$530,000	$742,000
Mecartney Road	Low	1855	9	0	$497,000	$695,800
Channing Way	Low	670	5	0	$156,000	$218,400
Oyster Shls	Low	446	6	1	$150,000	$210,000
Island 2	Low	2220	19	1	$553,000	$774,200

Figure A-14: Bay Farm Central Area Prioritized 25-Year Improvements

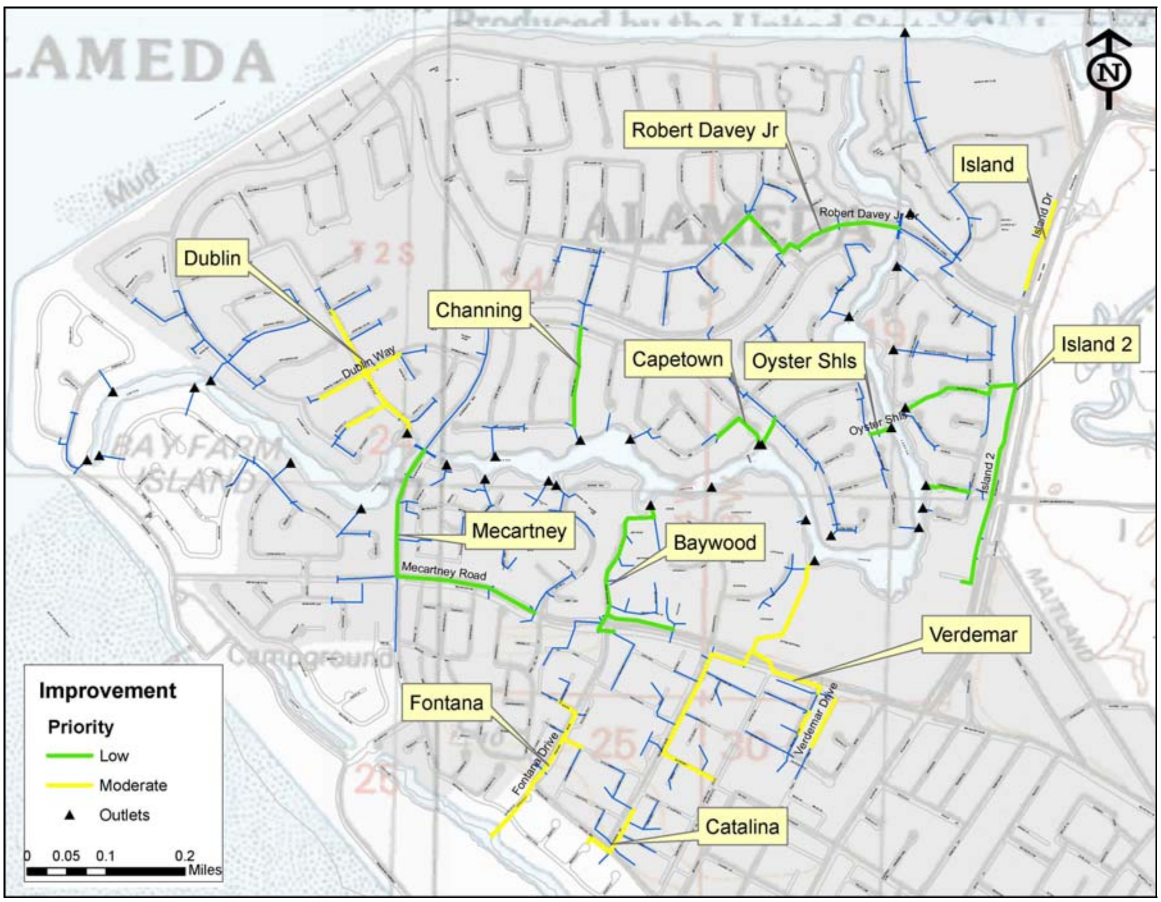


Figure A-15: Bay Farm South Area Existing 25-Year Flooding Depths

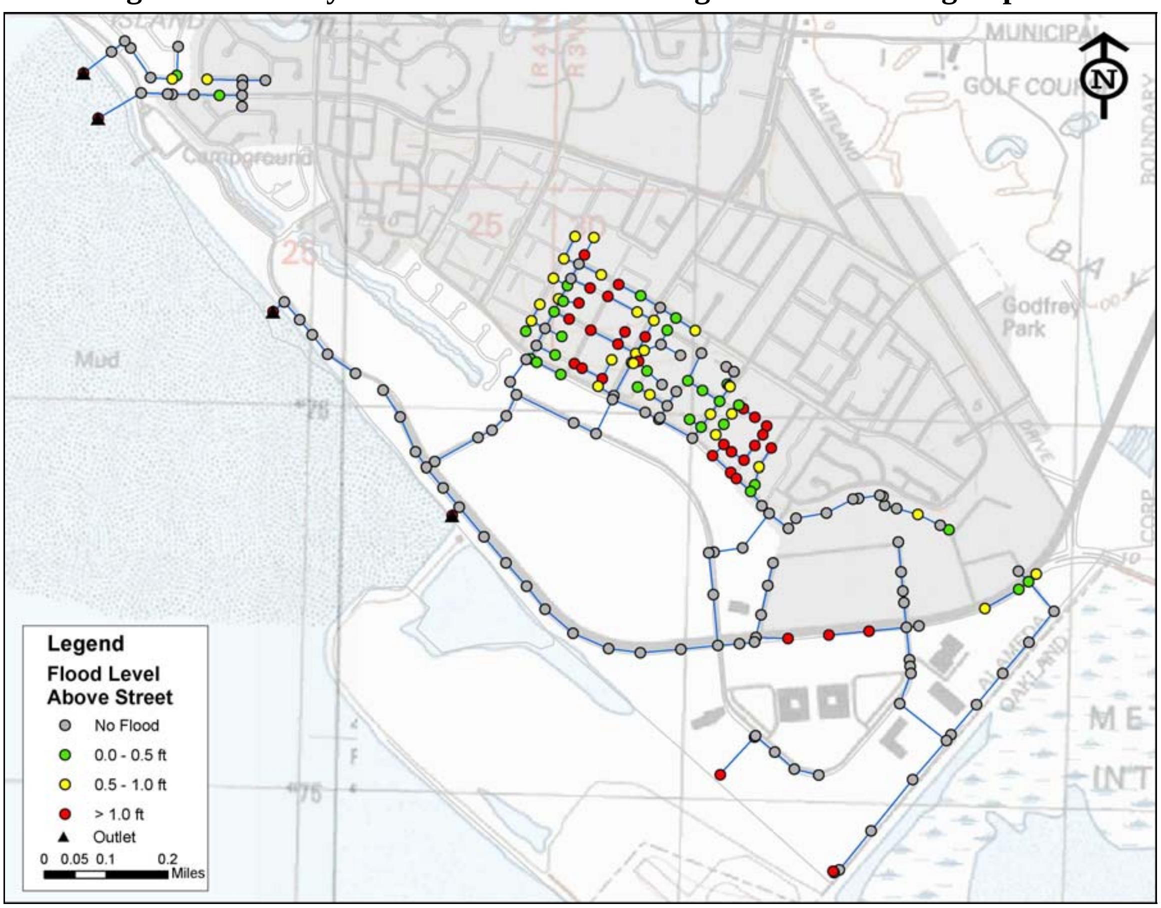
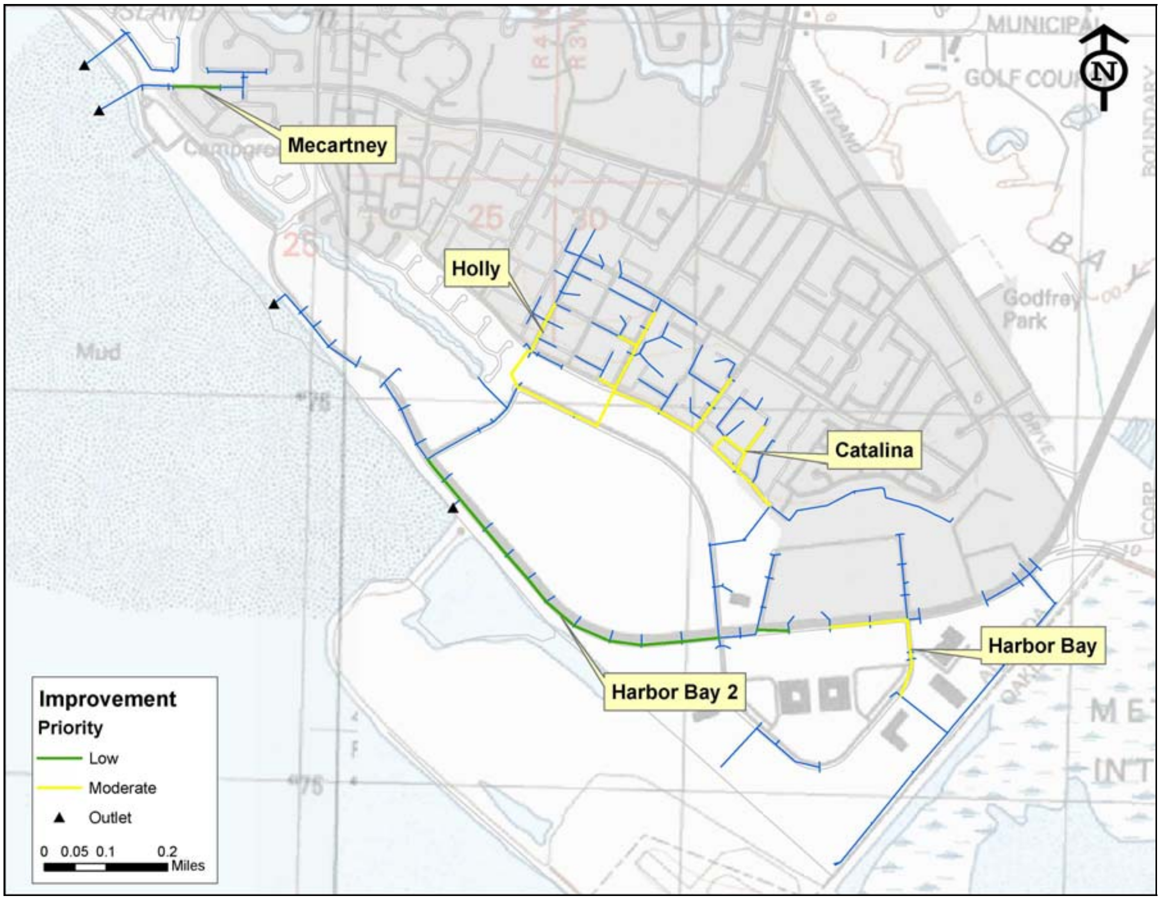
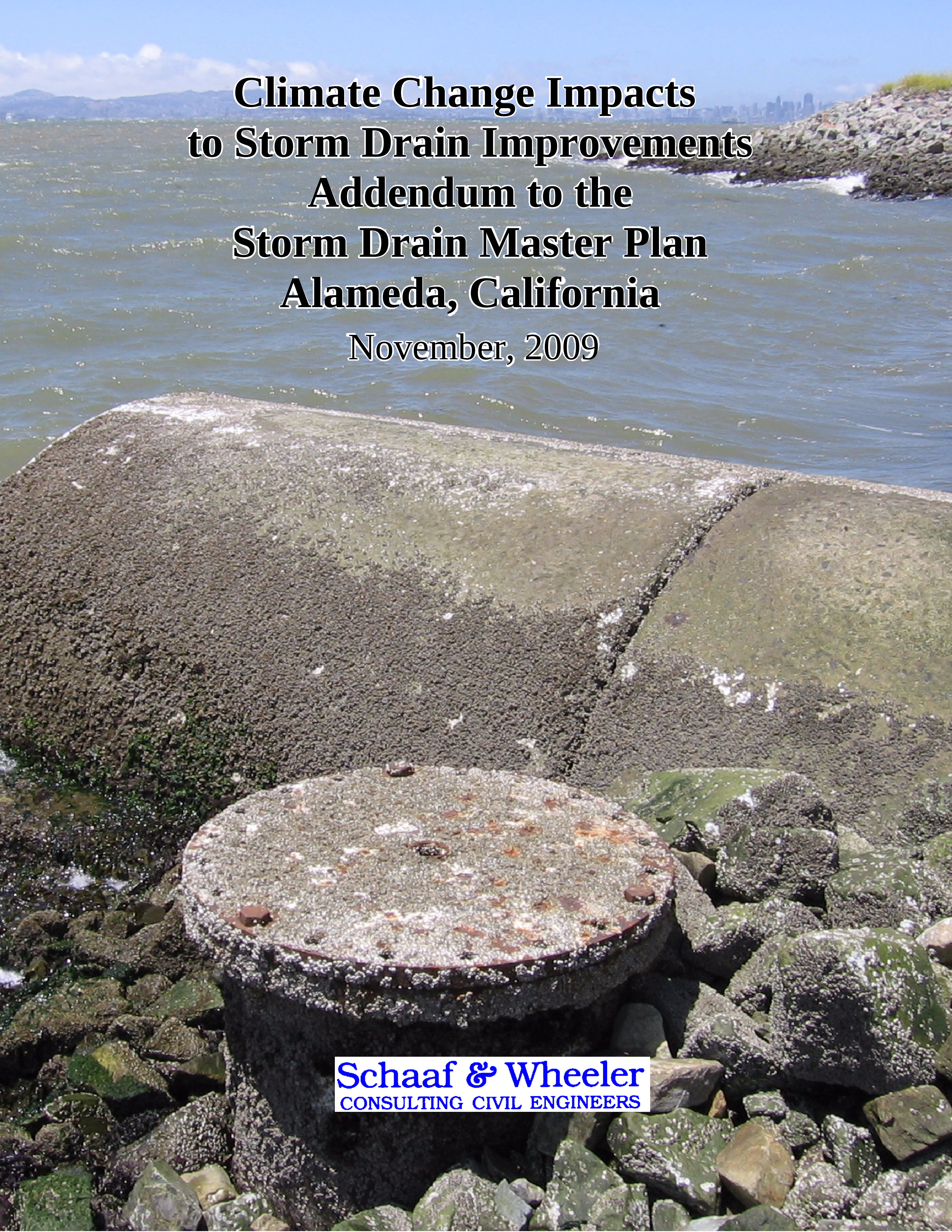


Table A-10: Bay Farm Island, South Area 25-Year CIP

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Construction Allowance	Total Allowance w/ Contingencies
Harbor Bay	Moderate	1335	7	0	$487,000	$681,800
Catalina	Moderate	1482	12	0	$448,000	$627,200
Holly	Moderate	4333	24	0	$1,500,000	$2,100,000
Harbor Bay 2	Low	3440	13	0	$1,733,000	$2,426,200
Mecartney	Low	403	3	0	$94,000	$131,600

Figure A-16: Bay Farm South Area Prioritized 25-Year Improvements


Mecartney
Holly
Catalina
Harbor Bay 2
Harbor Bay
Mud
Godfrey Park
MUNICIPAL
GOLF COU
BOUNDARY
MAITLAND
DRIVE
CORP
Improvement
Priority
Low
Moderate
Outlet
0 0.05 0.1 0.2
Miles



Climate Change Impacts to Storm Drain Improvements Addendum to the Storm Drain Master Plan Alameda, California

November, 2009

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

Introduction

Schaaf & Wheeler completed the City of Alameda (City) Storm Drain Master Plan (SDMP) in August, 2008. That report included a brief analysis and discussion of the impacts of sea level rise to City stormwater facilities. In that analysis, a 50-year planning horizon and corresponding 0.5 foot of sea level rise was used. The conclusion was that a half foot of sea level rise applied to the tidal cycle had no significant impacts to the operation of the existing storm drain system.

A more in-depth analysis of sea level rise scenarios was desired by the City. Specifically, the City wished to understand the impacts of a more severe sea level rise scenario on the 10-year improved storm drain system as well as potential inundation of rising sea levels within City limits. This addendum to the City of Alameda SDMP presents the results of these analyses. First, a general background on climate change and sea level rise projections is provided. The current understanding of other potential climate change impacts relevant to the City's flood risks and water resources is also summarized. Next, the impacts of sea level rise assuming an 18 inch rise relative to the City's coastline are presented, including both the risk of inundation within City limits by surrounding water and impacts to the storm drain capacity and operation under this specific sea level rise scenario. Additional improvements to the 10-year improved system are recommended to mitigate these impacts, and cost estimates for the improvements provided. Finally, current regulations, policies, and actions related to climate change from state and local organizations are summarized.

It should be noted that this report does not attempt to detail the specific causes of climate change, nor the distribution between anthropogenic (i.e. human induced) versus natural sources of carbon dioxide in the atmosphere. The purpose of this report is to detail the potential impacts of a specific climate change scenario to Alameda flooding risks, both in magnitude and uncertainty, and discuss conceptual and master planning level mitigation activities. Mitigation activities discussed herein focus on mitigating the impacts of global warming to flood risk within the City rather than mitigating carbon emissions.

Current Status of Climate Change Understanding and Research

It is well understood that carbon dioxide and other anthropogenic green house emissions act as heat trapping greenhouse gasses, which increase troposphere temperatures. Throughout the 1980s, scientists began to note increases in these emissions and postulated that the increases in atmospheric carbon dioxide may cause a range of impacts, some of which may be adverse. Climate change refers to an identifiable change in the state of the climate that persists for an extended period of time. The use of the phrase

‘climate change’ does not necessarily distinguish whether changes are due to natural processes versus human activity. Climate variability, however, refers to natural climate cycles or changes that are not caused by human activities. Many of the impacts of climate change occur quite slowly. Thus, even if carbon emissions are stabilized or greatly reduced in coming years, some impacts such as sea level rise will continue to occur, albeit potentially at a slower pace than predicted by most global climate change models.

As awareness of climate change spreads, an increasing number of analyses are conducted and reports published every year. Tasked with gathering, reviewing, and synthesizing the multitude of published studies is the Intergovenmental Panel on Climate Change. In addition to this international organization, this report summarizes the current understanding reflected in reports produced by the United States Army Corps of Engineers and by departments within the state of California.

<u>Intergovernmental Panel on Climate Change</u>

The Intergovernmental Panel on Climate Change (IPCC) was established in 1988 to provide an objective source of information about climate change. The IPCC does not independently conduct research or gather data. Instead it acts as a comprehensive assessor of the latest scientific, technical, and socio-economic literature produced worldwide relevant to the understanding of human-induced climate change, its impacts, and mitigation strategies. The IPCC was set up by the World Meteorological Organization and by the United Nations Environment Programme.

The First Assessment Report was released by the IPCC in 1990, the Second in 1995, the Third in 2001, and the Fourth in 2007. The conclusion that human induced climate change is occurring has been progressively more certain in each Assessment Report, with the 2007 Assessment Report stating that there is *very high confidence* (at least 9 out of 10 chance of being correct) that the global average net effect of human activities since 1750 has been one of warming, and that human induced warming over the last three decades has *likely* (greater than 66% probability) had a discernible influence at the global scale. Global warming refers to the general warming of the climate system, and the fact that global warming is occurring is unequivocal, based on IPCC findings. The next IPCC Assessment Report is scheduled for publication in 2012.

<u>Uncertainty and Scale</u>

IPCC uses a system of self-explanatory terms to convey qualitative and quantitative uncertainty. Three approaches are used to describe uncertainty. Where uncertainty is

assessed qualitatively, a relative sense of the amount and quality of evidence to support a statement is provided through use of terms such as: high agreement, much evidence; high agreement, medium evidence; medium agreement, medium evidence; etc. Where uncertainty is assessed quantitatively using expert judgment of the correctness of underlying data or analyses, a scale of confidence levels is used to express the assessed change of a finding being correct: very high confidence (at least 9 out of 10); high confidence (about 8 out of 10); medium confidence (about 5 out of 10); low confidence (about 2 out of 10); and very low confidence (less than 1 out of 10). Finally, where uncertainty in specific quantitative outcomes is assessed using expert judgment and statistical analysis, then likelihood ranges are used to express the probability of occurrence: virtually certain (>99%); extremely likely (>95%); very likely (>90%); likely (>66%); more likely than not (>50%); about as likely as not (33%-66%); unlikely (<33%); very unlikely (<10%); extremely unlikely (<5%); and exceptionally unlikely (<1%) (IPCC, 2007). Throughout this report, when these phrases are used based on IPCC findings they have been italicized as a visual reminder of this paragraph.

There are several global climate models that have been developed to estimate future impacts of climate change and global warming. Within each model there are various future condition scenarios representing the range of potential future carbon dioxide and other greenhouse gas emission levels. The more conservative approach is to assume that these emissions increase at a rate equal to or greater than recent trends. Generally the emissions and global warming predictions and impacts are directly proportional – the greater the emissions, the more severe the warming trend.

The vast majority of climate models are global in scale, and although general trends and impact estimates may be concluded from these models, there are multiple issues encountered when trying to downscale either results or models to determine trends or impacts in a localized area. The IPCC has produced a Special Report on the Regional Impacts of Climate Change which analyzes impacts at a continental or sub-continental scale; however this report focuses on impacts due to regional vulnerabilities as opposed to regional differences in physical impacts. Efforts to downscale from the global climate model to the catchment scale for hydrologic analyses and to utilize regional climate models to drive hydrologic models have shown that different ways of creating regional scenarios from the same source can lead to substantial differences in the estimated regional effect of climate change and that errors in the modeling procedure or differences in climate models are greater than hydrologic model uncertainty (Kundzewicz, 2007).

There is no single agreed upon methodology for downscaling climate change results for use in regional hydrology, and results may differ substantially depending on the source model and method used. The process of downscaling does not resolve any of the

uncertainty inherent in global climate models, and introduces new sources of uncertainty such that overall trends are less well defined compared to global models. For example, depending on the global climate model and scaling methodology used the estimated range of impact to mean annual precipitation in California varies in both magnitude and sign by at least 10% (Dettinger, 2004). What this means is that while global climate change trends are relatively well known and documented, regional and local trends, particularly hydrologic parameters such as rainfall and runoff, are less well known.

<u>California Climate Action Team</u>

The California Climate Action Team (CAT) was established by Governor Schwarzenegger under an Executive Order on June 1, 2005. The purpose of the CAT is to coordinate state-level actions relating to Climate Change. The Team is led by the Secretary of the California Environmental Protection Agency and includes the Secretary of the Business, Transportation and Housing Agency, Secretary of the Department of Food and Agriculture, Secretary of the Resources Agency, Chairperson of the Air Resources Board, Chairperson of the Energy Commission and President of the Public Utilities Commission. The Climate Action Team is charged with implementing global warming emission reduction programs and reporting on the progress made toward meeting the statewide greenhouse gas targets that were established in the Assembly Bill 32 (described in more detail later in this report). The first report was sent to the Governor and the Legislature in 2006, and should be updated bi-annually thereafter.

<u>California Climate Change Center</u>

The California Energy Commission's Public Interest Energy Research (PIER) Program conducts public interest research, development, and demonstration projects to benefit California's electricity and natural gas ratepayers. In 2003, the California Energy Commission's PIER Program established the California Climate Change Center (CCCC) to document climate chance research relevant to the states. The CCCC Report Series details ongoing center-sponsored research on climate change predictions and impact analyses. All of the final CCCC reports include a preface which clarifies that the findings presented are interim project results, and information contained within the reports is subject to change.

Global Warming Impacts

The IPCC range of best estimate *likely* temperature increases by the year 2099 is 0.6 – 4.0 degrees Celsius (1 – 7 degrees Fahrenheit), depending on the global climate model utilized (IPCC, 2007). Regionally, scaled down climate models for northern California

estimate global temperature increases up to 4.5 degrees Celsius (9 degrees Fahrenheit) by 2100 (Cayan, 2007). An increase in global temperatures in the IPCC range may have multiple impacts on the water resources of the City of Alameda, even if the changes in local and regional temperature are not yet known.

Sea Level Rise

One of the most publicized impacts of global warming, and the impact with the most direct consequences to the City of Alameda, is sea level rise. Sea level rise can be defined as global or relative. Global sea level rise is defined as the increase of global average sea level. Throughout the world, land may be uplifting or subsiding. This will impact the relative change in depth of water at any given location, depending on the rate of movement compared to the rate of global sea level rise. In addition, coastal bays such as the San Francisco Bay may not experience sea level rise at the same rate as the global average. Relative sea level rise refers to the rise of sea levels accounting for local hydraulics, land uplifting or subsidence.

An example of the importance of global vs. relative sea level rise can be seen when examining the historic sea level trends in San Francisco Bay at the National Ocean and Atmospheric Administration (NOAA) gages for San Francisco (at the Presidio) and Alameda (Pier 3 at the Naval Air Station). The Alameda gage shows a long term average mean sea level rise of 0.82 millimeters per year (NOAA, Alameda Mean Sea Level Trend), while the San Francisco gage long term average mean sea level rise is 2.01 millimeters per year (NOAA, San Francisco Mean Sea Level Trend). Although the San Francisco gage period of record is longer, essentially the same rate of sea level rise is found if it is truncated to match the Alameda gage period of record. The reasons for this difference are unknown, and likely due to a combination of factors, but it serves to exemplify the complexity between local trends, global predictions, and site specific hydraulics.

IPCC Sea Level Rise Estimates

Depending on the emission scenario used, the predicted *likely* global sea level rise ranges from 0.18 – 0.59 meters (IPCC 4th Assessment Report), or 0.6 – 1.9 feet by the year 2099. IPCC reports do not provide mid-range estimates; e.g. sea level rise by 2050. The upper limit of this range is lower than the upper range stated in previous IPCC reports. The two primary factors affecting global sea level rise are thermal expansion of ocean waters due to increased atmospheric temperature, and melting ice. The IPCC estimates that of the global sea level rise that has occurred since 1993, thermal expansion of the ocean has contributed 57% of the total rise, decreases in the extent of glaciers and ice caps have

contributed 28%, and the remaining 15% is due to losses from the polar ice sheets. It must be noted that this range does not include uncertainties in climate-carbon cycle feedbacks or the full effect of changes to ice sheet flow, because a basis in published literature is lacking. Thus these values do not represent an upper bound to projected sea level rise. Long term projections show that global warming sufficient to eliminate the Greenland Ice Sheet (one millennium exposed to an average temperature rise in excess of 1.9 – 4.6 degrees Celsius) results in an additional seven meters (23 feet) of global sea level rise. The IPCC does not offer any uncertainty scale for this possibility.

United States Army Corps of Engineers Sea Level Rise Estimates

The United States Army Corps of Engineers (USACE) published an engineering circular (USACE, 2009) to direct the consideration of sea level rise estimates in project planning and design. While this methodology is required only for USACE civil work activities, it offers a valuable guidance for any planning effort. In summary, the USACE report recommends that the planning, engineering and designing for projects within the tidal zone or with downstream tidal boundary conditions consider how sensitive and adaptable the project is to a range of sea level rise estimates (low, intermediate and high). Specifically, the USACE directs determination of "how sensitive alternative plans and designs are to these rates for future local mean sea-level change, how this sensitivity affects calculated risk, and what design of operations and maintenance measures should be implemented to minimize adverse consequences while maximizing beneficial effects".

The "low" sea level rise estimate recommended by the USACE report is based on local historic tide gauges. In San Francisco, the Presidio tide gauge has the longest period of record and is consistently used for historic sea level trends in San Francisco Bay. For consistency with regional documents the Presidio gauge is used for calculations herein, although the Alameda gauge records described above may be more appropriate for the City. The long term average sea level rise at the Presidio gauge is 2.01 millimeters per year (mm/yr), with a 95% confidence limit of plus or minus 0.21 mm/yr (NOAA, Station 9414290). "Intermediate" and "high" sea level rise estimates are based on the National Resource Council (NRC) curves and equations developed for a 1987 Report (*Responding to Changes in Sea Level: Engineering Implications*), modified to account for the updated annual estimate of sea level rise made in the 2007 IPCC report, and manipulated to include consideration of the date of the equation development. The "intermediate" sea level rise projection is based on the modified NRC Curve I, and the "high" sea level rise projection on the modified NRC Curve III. This equation is:

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)$$

where:

t_1 = time between construction date and 1986;

t_2 = time between date at which sea level rise projection is desired and 1986;

$E(t)$ = eustatic sea-level rise, in meters, as a function of (t);

b = Variable, 2.36E-5 for modified NRC Curve I, 1.005E-4 for modified NRC Curve III.

Table 1 presents the range of sea level rise projects for the City of Alameda using this methodology, assuming adoption of the Presidio gauge for the local historic sea level trend, and construction of any given project in 2010.

Table 1: Range of Sea Level Rise Projections Using USACE Methodology with Presidio Gage and 2010 Construction Year

USACE Methodology Sea Level Rise Projection Range (feet)			
Year	**Low**	**Intermediate**	**High**
2025	0.1	0.2	0.4
2050	0.3	0.5	1.4
2075	0.4	0.9	2.8
2100	0.6	1.5	4.6

California Climate Change Center Sea Level Rise Estimates

A draft version of the *Impacts of Sea-Level Rise on the California Coast*, developed by The Pacific Institute for the CCCC was released in March, 2009, with much publicity of the new 2100 sea level rise estimate of “5 feet” (Chronicle article, March 12, 2009). The development of this sea level rise estimate is presented in somewhat more detail, however, in the *Climate Change Scenarios and Sea Level Rise Estimates for the California 2009 Climate Change Scenarios Assessment* Report (Cayan, 2009), also produced for the CCCC. In short, the sea level rise estimates adopted by the CCCC are based on an empirical formula developed by Rahmstorf (2007) which relates global mean sea level rise to global mean surface air temperature. The report states (and shows graphically) that the Rahmstorf predicted values are then manipulated to include the impact of reservoirs and dams, but exactly what this manipulation entails, and its justification, is unclear. The supporting article cited as the basis of this manipulation, *Impact of Artificial Reservoir Water Impoundment on Global Sea Level* (Chao, 2008),

appears to focus on the impact of reservoir and dam storage to historic sea level trends, and Schaaf & Wheeler was unable to locate any published article which details a modified Rahmstorf method.

Using the above methodology, the 2009 Assessment Report gives a range of sea level rise of 30-45 cm (12 – 18 inches) by 2050 (relative to 2000 levels). Although other CCCC reports, as well as the San Francisco Bay Conservation and Development District, have adopted a 2100 sea level rise projection of 1.4 meters (4.6 feet), this projection is not explicitly stated in the text of the 2009 Assessment Report (it can only be deduced from included graphs). It should be noted that the range of sea level rise estimates produced from this methodology is about 0.6 m – 1.45 m (2.0 – 4.8 feet). The 4.6 feet of rise by 2100 predicted at the upper end of this range is similar to the USACE methodology high range for 2100 for San Francisco Bay, as shown in Table 1.

Sea Level Rise Estimates Summary

In summary, significant uncertainties remain in sea level rise projections, particularly as one forecast's farther into the future. <u>The most current available estimates for sea level rise by 2050 range from 0.3 foot to 1.5 feet, and by 2100 from 0.6 foot – 4.8 feet.</u> Confidence in any sea level rise prediction decreases the further into the future that analysis is projected, due to unknowns about future emission scenarios, potential climate feedback loops and the severity of melting ice. It is important to note that emphasis should not be placed on a particular specific value for sea level rise. Not only is a consensus on a particular value unlikely, but the selection of the year 2100 as a reporting point for sea level rise projections is arbitrary. Even with drastic reductions in carbon emissions sea levels are expected to continue to rise beyond 2100 due at least to continued thermal expansion of ocean waters. Thus, any planning for sea level rise impacts should recognize the inherent uncertainty and long term ongoing nature of these projections.

Rising sea levels have two potential impacts to the City: inundation of Bay water onto City lands and impacts to the operation and performance of City storm drain facilities. Each of these impacts is discussed in more detail below.

<u>Other Climate Change Impacts</u>

Climate change has many predicted impacts in addition to sea level rise. Below, other climate change impacts which may adversely affect flooding risk of the City of Alameda are described. These impacts are: storm surge, wave runup, and precipitation.

Storm Surge

During storm events, ocean water increases in elevation due to low barometric surface pressure. This phenomenon is called storm surge. The FEMA 1% storm surge for San Francisco Bay at Alameda is 7 feet NGVD, compared to a mean high-high tide of 3.7 feet NGVD (NOAA, Alameda Datums). This represents a 1% surge of 3.3 feet. It is *likely* that the incidence of extreme high sea level has increased at a broad range of sites wordwide since 1975. Extreme high sea level is defined as the highest 1% of hourly values of observed sea level at a station for a given reference period (IPCC, 2007).

Pronounced multi-year fluctuations of San Francisco non-tidal residuals (NTR; total water elevations above tidal elevations – for San Francisco Bay NTRs are primarily storm surge and wind driven waves) are evidenced in historical records and no significant changes in the mean monthly positive NTRs exist between 1858 and 2000. However when considering only the highest 2% of extreme winter NTRs there has been a significant increasing trend since about 1950 (Bromirski, 2003). This increased 'storminess' may be part of a larger cycle, but it suggests a relationship between global climate warming and overall storminess on the west coast.

The occurrence of hourly observed high sea levels (above the 99.99th percentile thresholds) in San Francisco Bay has increased sharply since 1969. The maximum observed sea level has also increased since that time, although the period of 1987-2004 had a slightly lower peak sea level than 1969-1987. Recent studies have concluded that if sea level rise is on the lower end of the current predicted ranges, the occurrence of extremely high sea level events will increase, but the increase in extremes would be not so different from the increasing trend that has been seen in California for the past several decades. If, however, sea level increases reach the higher end of the range, extreme events would increase not only in their frequency but also their duration, substantially beyond the historic trend seen in the 19th and 20th centuries (Cayan, 2007).

In short, it is expected that as sea levels rise, not only will the occurrence of high sea level, or surge, events increase, but so may the amount of surge itself (currently about 3.3 feet above mean high high water in Alameda). This increased storm surge elevation may impact flood risk, backwater conditions and storm water pump station operation; however quantitative estimates for the increased storm surge have not been made, and are unlikely to be determined in the near future.

Wave Runup

Wave runup is the elevation wind-driven waves will reach as waves break on land and may be affected by global warming. However, these impacts are not particularly well understood at this time. A review of recently published literature finds that different published studies come to different, and at times directly opposing, conclusions regarding likely climate change impacts to wave energy. Wave heights are greatly influenced by local conditions, likely a major cause for the differing results found in the available literature. Some general trends are well understood, such as that extreme wave heights and surge fluctuations tend to increase from the south to the north along California Coast, as a result of increasing storm intensities along the northern coast (Cayan, 2007).

Wave runup is a function of water depth, wind speed and direction, and the features of the land on which the wave is breaking (slope, roughness, etc.). In some parts of San Francisco Bay, rising sea levels will inundate low lying marshes, creating broad, but shallow, flooded areas. In this scenario, wave runup will likely decrease, as the shallow water will dampen wave heights. In Alameda however, which is generally protected by high land, rising sea levels will create deeper water surrounding the City, potentially resulting in increased wave heights and runup.

Published literature has found that when short term sea level is highest (i.e. during storm surge events), wave energy has an increased likelihood of reaching very high levels. The peak likely significant wave height (the average height of the one third highest waves) increases by 2.5 meters in one scenario where the surge value increased from 4 centimeters (cm) to 30 cm (Cayan, 2007). Thus in that particular scenario, as the storm surge increases, so does wave energy and height, which in turn may increase wave runup. That said, recent downscaled models have also indicated that the incidence of large coastal storms will lessen as part of the overall drying trend (discussed in more detail in the precipitation section below), resulting in a marginal decrease in the wind wave energy reaching California's coast as well as a decreasing trend for significant wave heights (Cayan, 2009). In short, although climate change is expected to impact storm surge and wave runup, these impacts (or even the trend of impacts) is not well understood at this time, and in any event, these impacts are expected to be dwarfed by the impact of increasing mean sea level.

The Bay floor near Alameda is largely composed of Bay mud, a thick deposit of soft, unconsolidated silty clay, which is saturated with water. One potential mitigation action against increased wave height due to deepening water would be to fill to maintain existing water depths. In addition to the multitude of permitting and environment issues with this activity, however, Bay Mud has a very high compressibility. In other words,

Bay Mud will continue to compress even when large volumes or weights are set on it. Thus filling on top of Bay Mud is ineffectual, and when additional environmental impacts are considered with the uncertainty of wave height and runup impacts, not a feasible mitigation alternative for Alameda to offset increased wave heights and runup.

Precipitation

It is *likely* that the frequency of heavy precipitation events (or proportion of total rainfall from heavy storms) has increased over most areas (IPCC, 2007). Global analyses of precipitation from 1901-2005 do not show statistically significant trends due to many discrepancies between data sets and the variability of precipitation in both space and time (Bates, 2008). Likewise, there is no consensus among regional climate models as to how mean annual precipitation totals might change in the United States (Dettinger, 2004), although most recent global and regional models predict that total mean precipitation will modestly decrease (5-20%) in the latter half of the next century (Hayhoe, 2004; Cayan, 2007, Draft 2009). Long term historic analyses of precipitation in the state of California show that there is no statistically significant change in total annual mean precipitation from 1890 through 2000, although the variability of total rainfall in any given year appears to have an increasing trend (DWR, 2006).

While the total mean annual precipitation is not predicted to change significantly, the timing and intensity of storm events is expected to change, with a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events Climate models predict (and historic records reflect) that proportionally less rainfall will fall during spring and summer months (April – July) and more in winter months (November – March) in northern California due to global climate change (Dettinger, 2004; Cayan 2007; DWR 2006). These shifts in precipitation timing and intensity may have impacts on flooding and water supply.

The most updated *Climate Change Scenarios* report (Cayan, 2009) states that the occurrence of significant storms declines at least marginally and that the occurrence of high daily precipitation events generally remains about the same through 2100 as it does in the historical projections. It should be noted that this conclusion is markedly different from previous conclusions by the same authors, which predicted a tendency in California for a modest increase in the number and magnitude of large precipitation events, with longer dry periods between events (Bates, 2008; Cayan 2007). Several CCCC reports reviewed for this analysis repeat the earlier, and presumably outdated, conclusion.

In summary, while a small decrease in annual precipitation is forecast, the trend in number and magnitude of large precipitation events is unknown. The most current

studies reviewed for this analysis both conflict previous conclusions and other updated studies, further exemplifying that there is no consensus regarding the potential impacts of climate change on the frequency or magnitude of large storm events.

Sea Level Rise Impacts to the City of Alameda

The effects of climate change described above have potential impacts to virtually all water resources within the City of Alameda, including not only local flood control and risk but also regional impacts to sectors such as agricultural and water supply. This report focuses on how rising sea levels may impact the risk of flood inundation of the City from its surrounding waters, and the impact of rising sea levels to the 10-year improvements previously made in the City SDMP. For this analysis, 18 inches (1.5 foot) of sea level rise was assumed. This represents the upper bound of the range of the most recently published sea level rise projections by year 2050 (Cayan, 2009).

When discussing projects to mitigate the impacts of sea level rise, there are several important points to keep in mind. As described above, there is not currently and unlikely to ever be a true consensus in the prediction of sea level rise, particularly a consensus on a projection 100 years into the future. A planning horizon of 100 years is not only far beyond most planning timelines typical to public agencies, but it is also beyond the typical useful life of structural flood protection elements. In other words, even if it were financially feasible to construct a project today to protect for a sea level rise scenario in 2100, it may not be advisable to do so, since that project could be structurally unsound by the time it was needed. Finally, it should be noted that although currently the year 2100 is the most common projection date, sea levels are expected to continue to rise beyond the year 2100.

<u>Inundation due to Rising Waters</u>

As an island community, Alameda is uniquely vulnerable to rising water levels in San Francisco Bay. Currently, Alameda is protected from inundation from its surrounding waters primarily by high ground, as opposed to floodwalls or levees. Interior lagoons are hydraulically connected to the surrounding waters via weir inlets, pumps, or gated outlets.

Figures 1 through 6 reflect City-wide recent topographic data adjusted to show three elevations of interest: existing mean sea level, mean sea level with 18” increase, and the highest tide elevation for various storm events with 18” of sea level rise added. The storm specific tide cycles were developed for the SDMP and the methodology and results of that process are described in detail in that report. It should be noted that these figures

do not take into account potential flood protection of naturally occurring high ground or existing flood control facilities. In other words, a shaded area represents an elevation range only, and does not necessarily mean that surrounding water will be able to reach and pond in all of those locations. One good example of this is shown in Figure 2, which reflects the fact that much of the golf course is below mean sea level. This does not mean, however that the golf course is always inundated with surrounding waters, due to existing high ground and storm drain facilities. That said, the lack of flap gates on many storm drain outlets may allow for backwater due to high tides to reach interior locations of the City. Figures 7 and 8 translate the water surface elevation into depth of water for the most severe (100-year event) scenario. Again, these figures represent potential risk areas without consideration of existing natural or man made protection measures.

Table 2 summarizes the existing and sea level rise scenario mean and high tide levels reflected in Figures 1 through 8. Storm specific tide cycles were developed for the SDMP, and a more complete description of the methodology for that process can be found in the SDMP, Chapter 3.

Table 2: Mean and High Tide Elevations for Existing and Sea Level Rise Scenario

	Existing (NGVD)	Sea Level Rise (18") Scenario (NGVD)
Mean Sea Level	0.5'	2.0'
10-Year High Tide	5.1'	6.6'
25-Year High Tide	5.4'	6.9'
100-Year High Tide	6.2'	7.7'

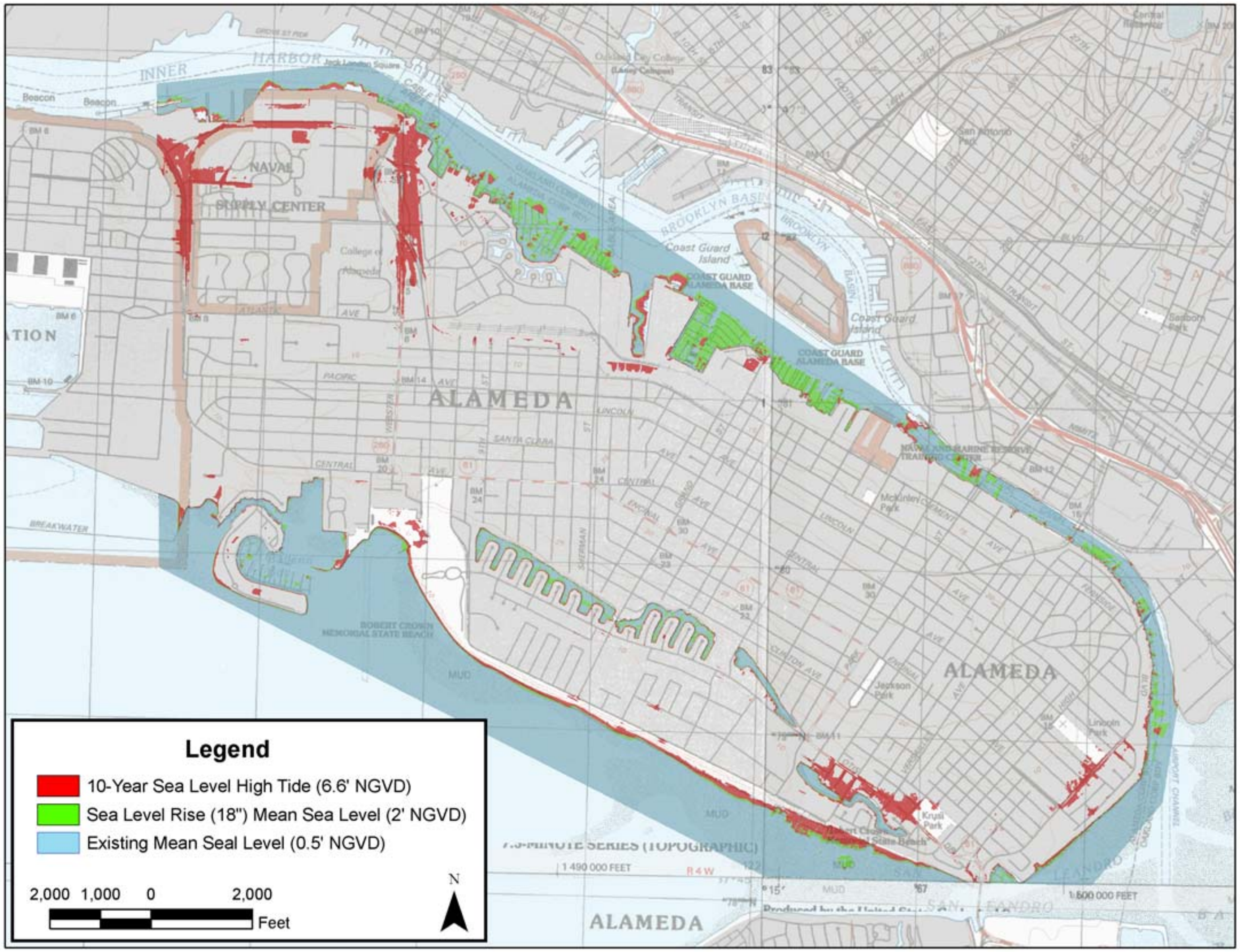


Figure 1: Areas below the 10-year High Tide with 18" of Sea Level Rise, Main Island

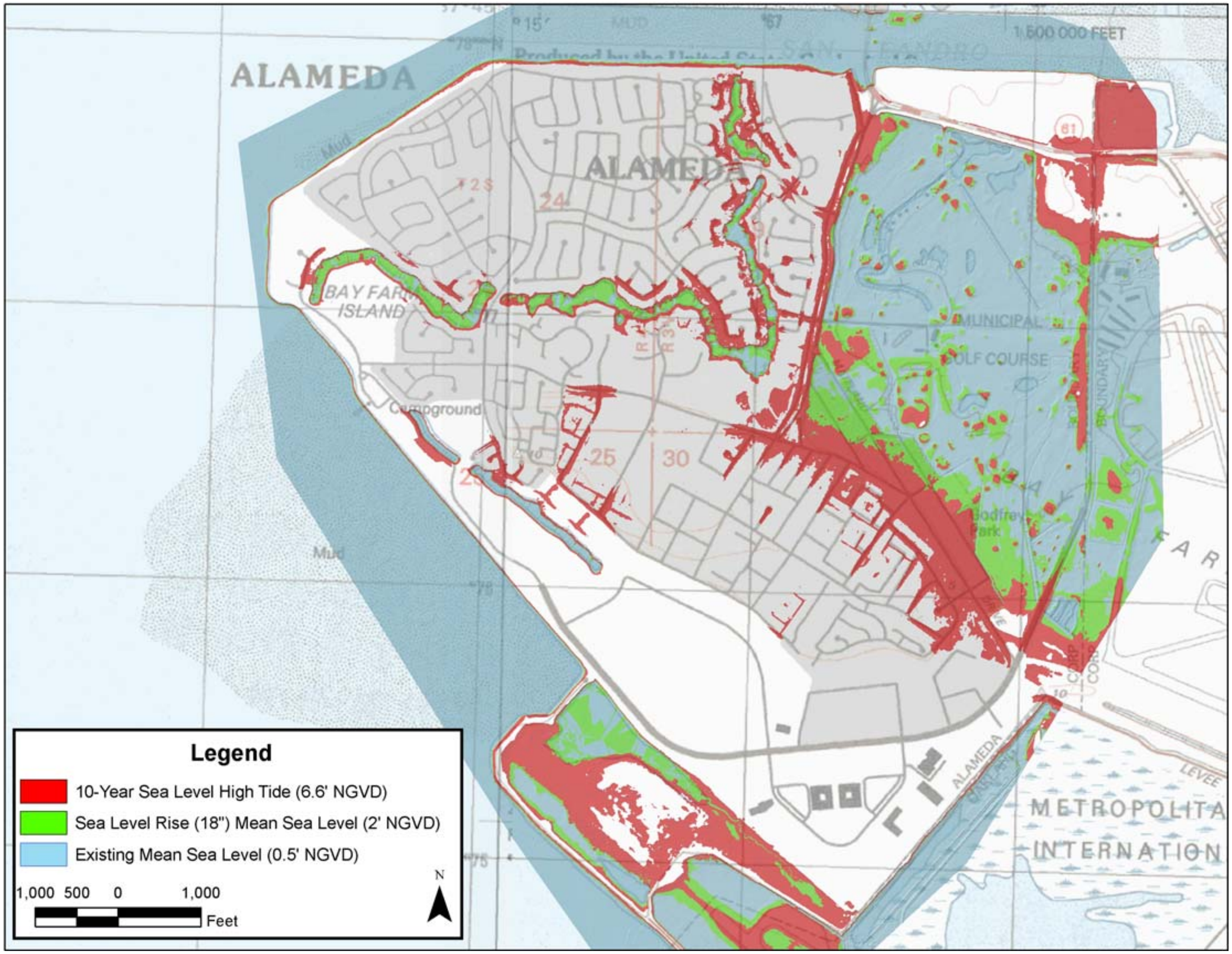


Figure 2: Areas below the 10-year High Tide with 18" of Sea Level Rise, Bay Farm Island

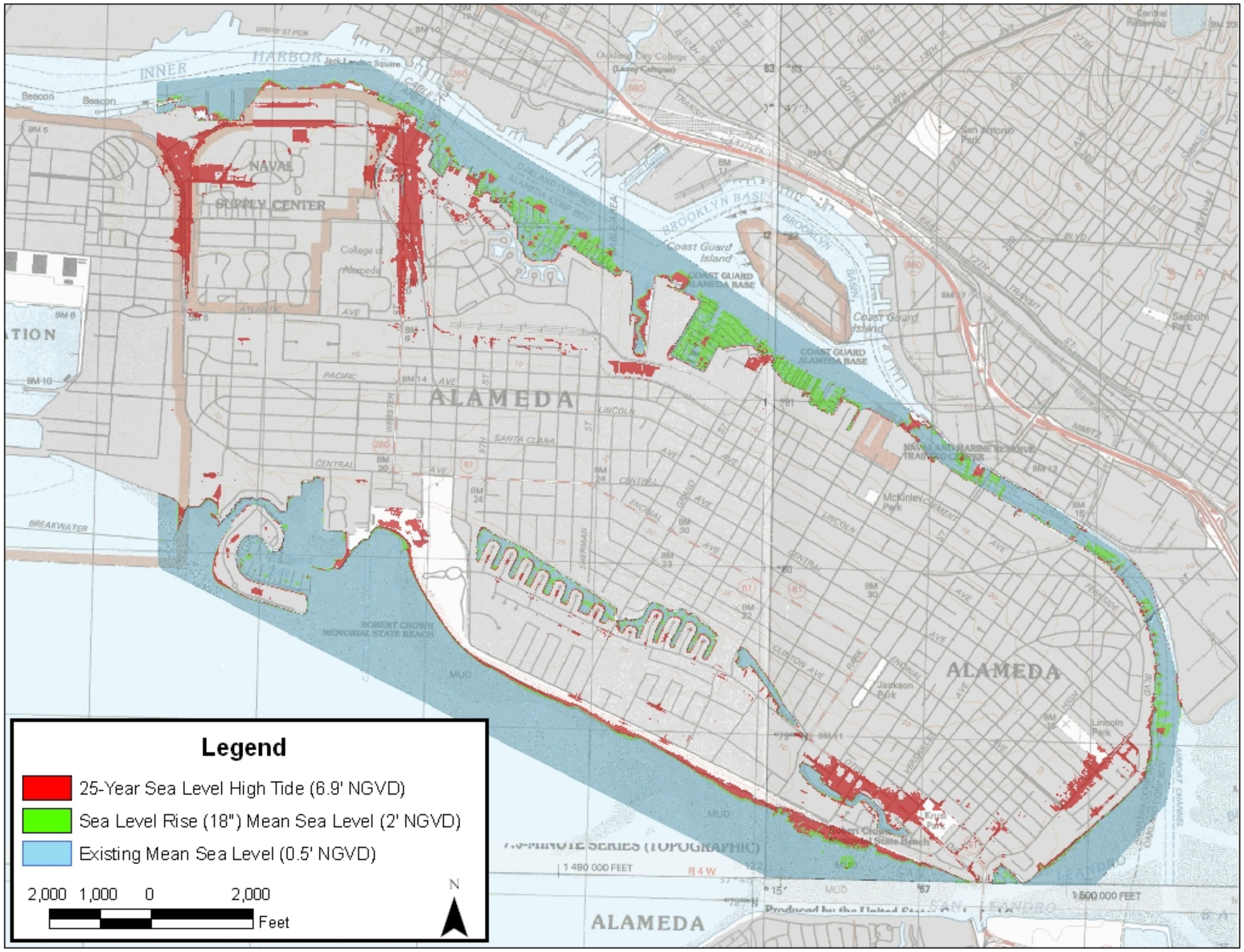


Figure 3: Areas below the 25-year High Tide with 18" of Sea Level Rise, Main Island

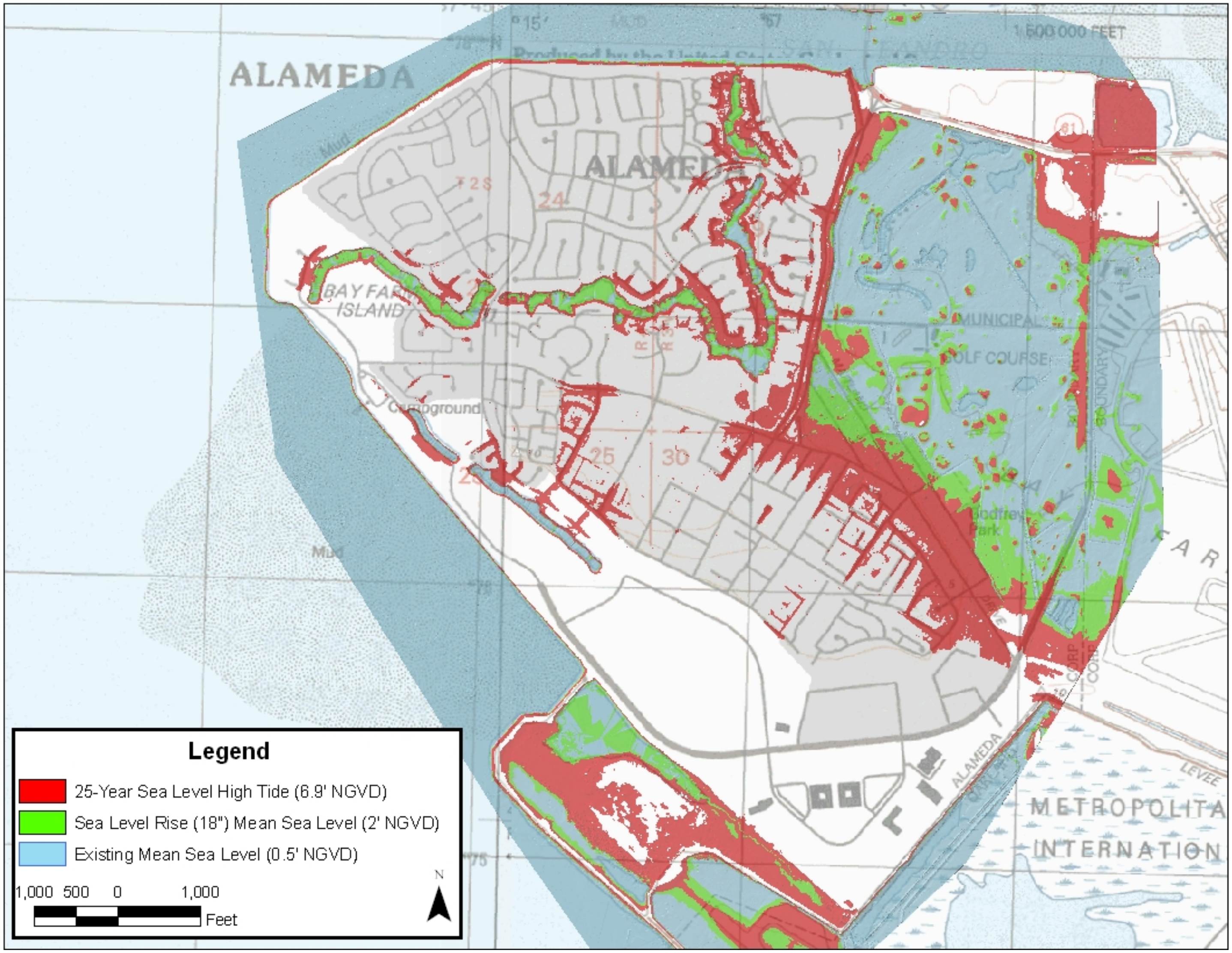


Figure 4: Areas below the 25-year High Tide with 18" of Sea Level Rise, Bay Farm Island

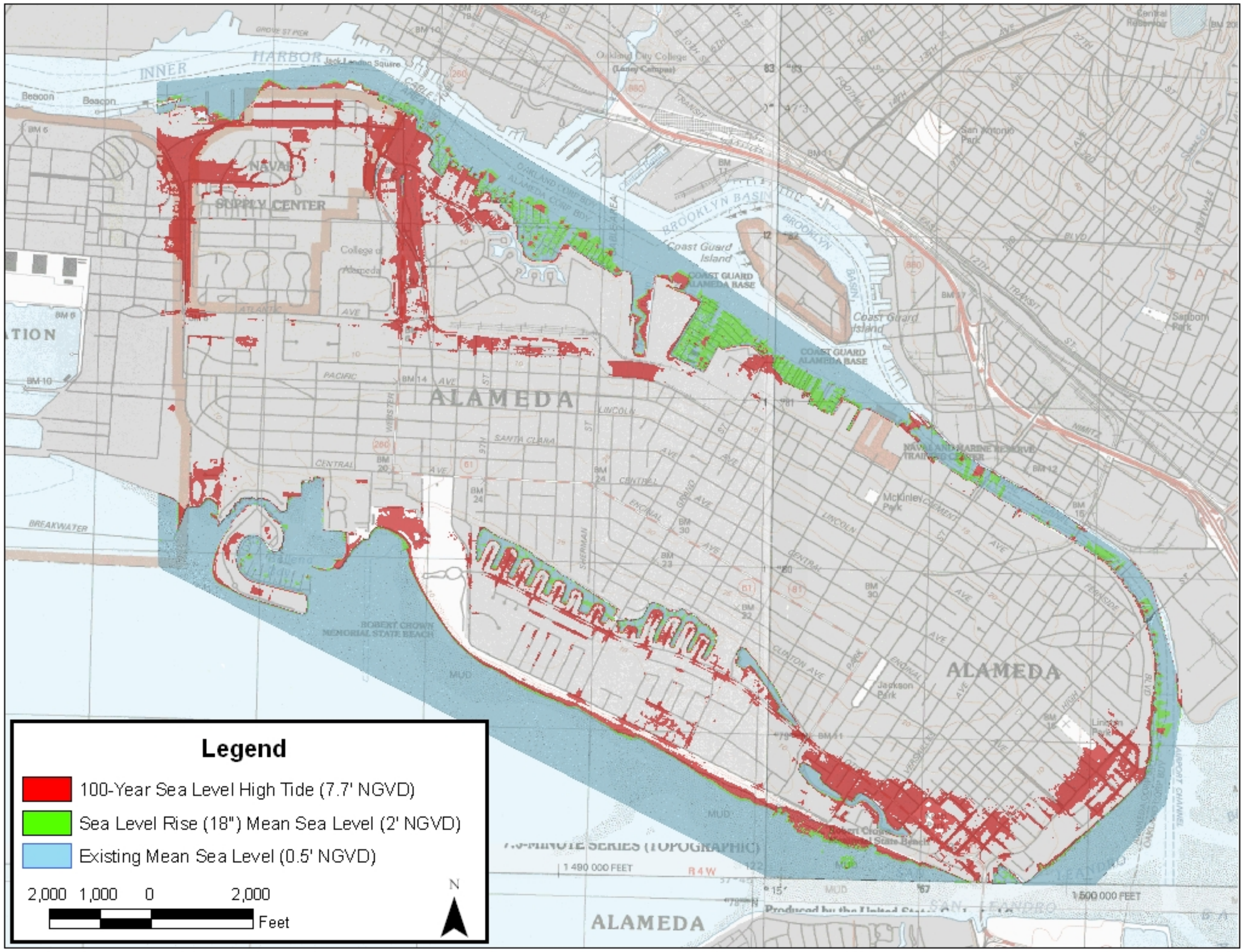


Figure 5: Areas below the 100-year High Tide with 18" of Sea Level Rise, Main Island

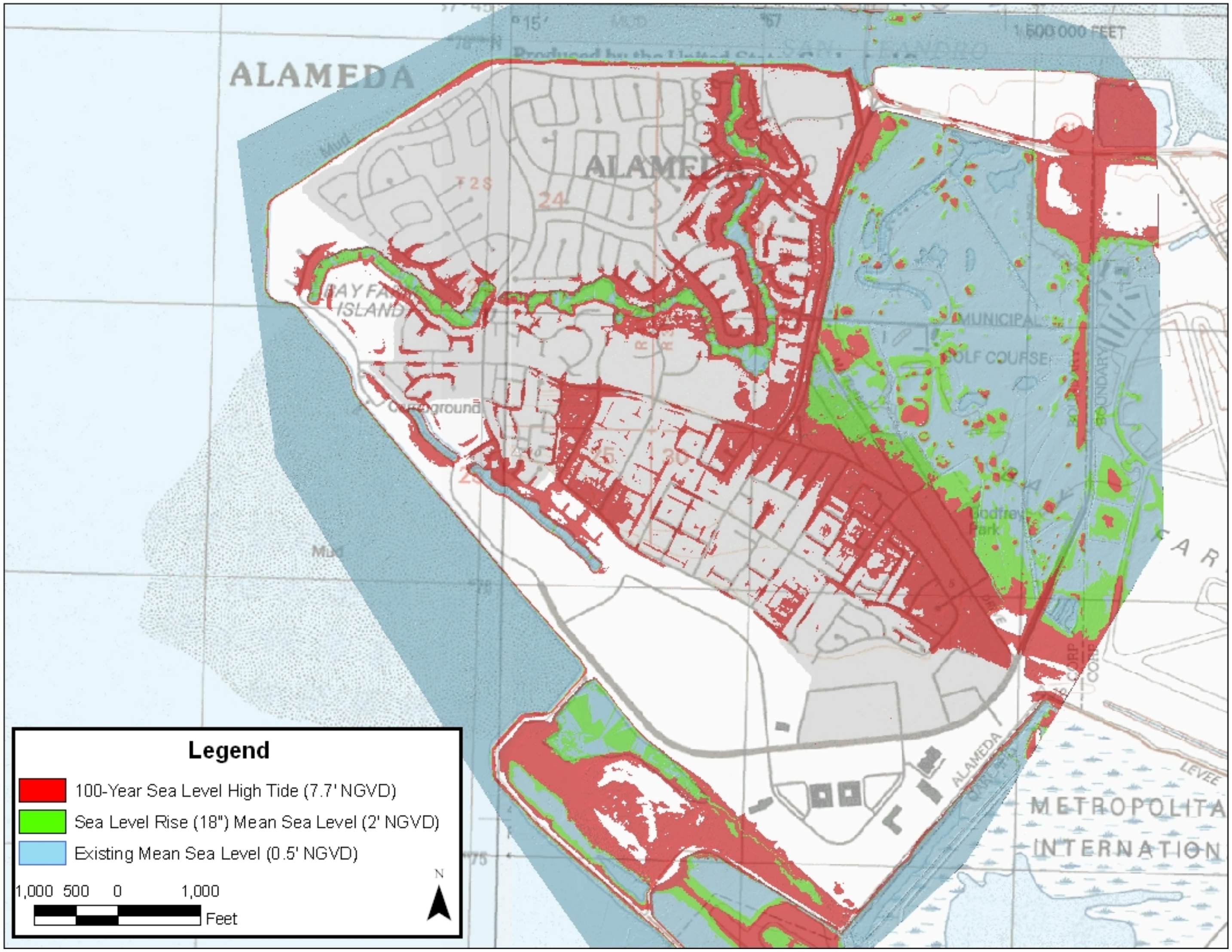


Figure 6: Areas below the 100-year High Tide with 18" of Sea Level Rise, Bay Farm Island

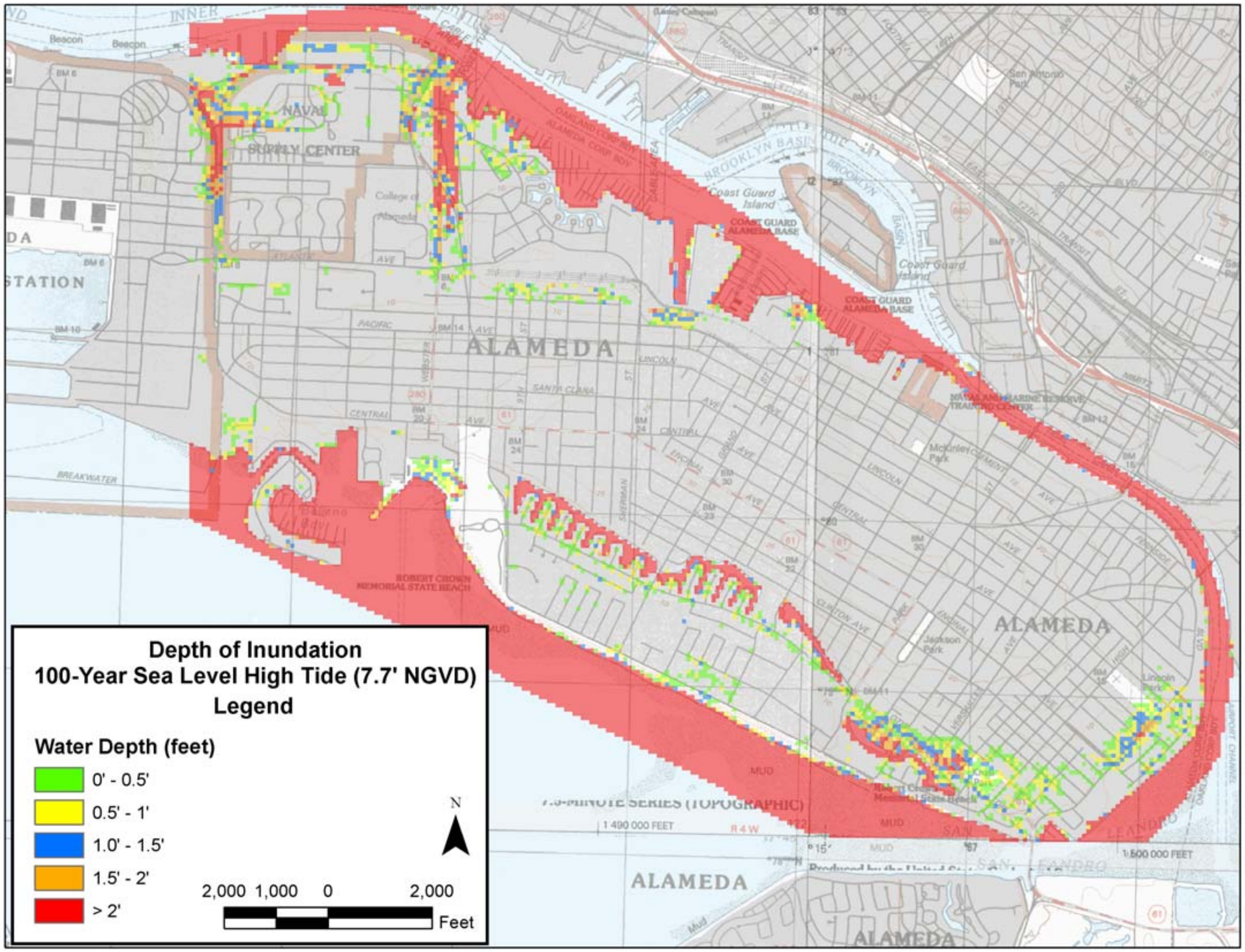


Figure 7: Depth of Water below the 100-year High Tide with 18" of Sea Level Rise, Main Island

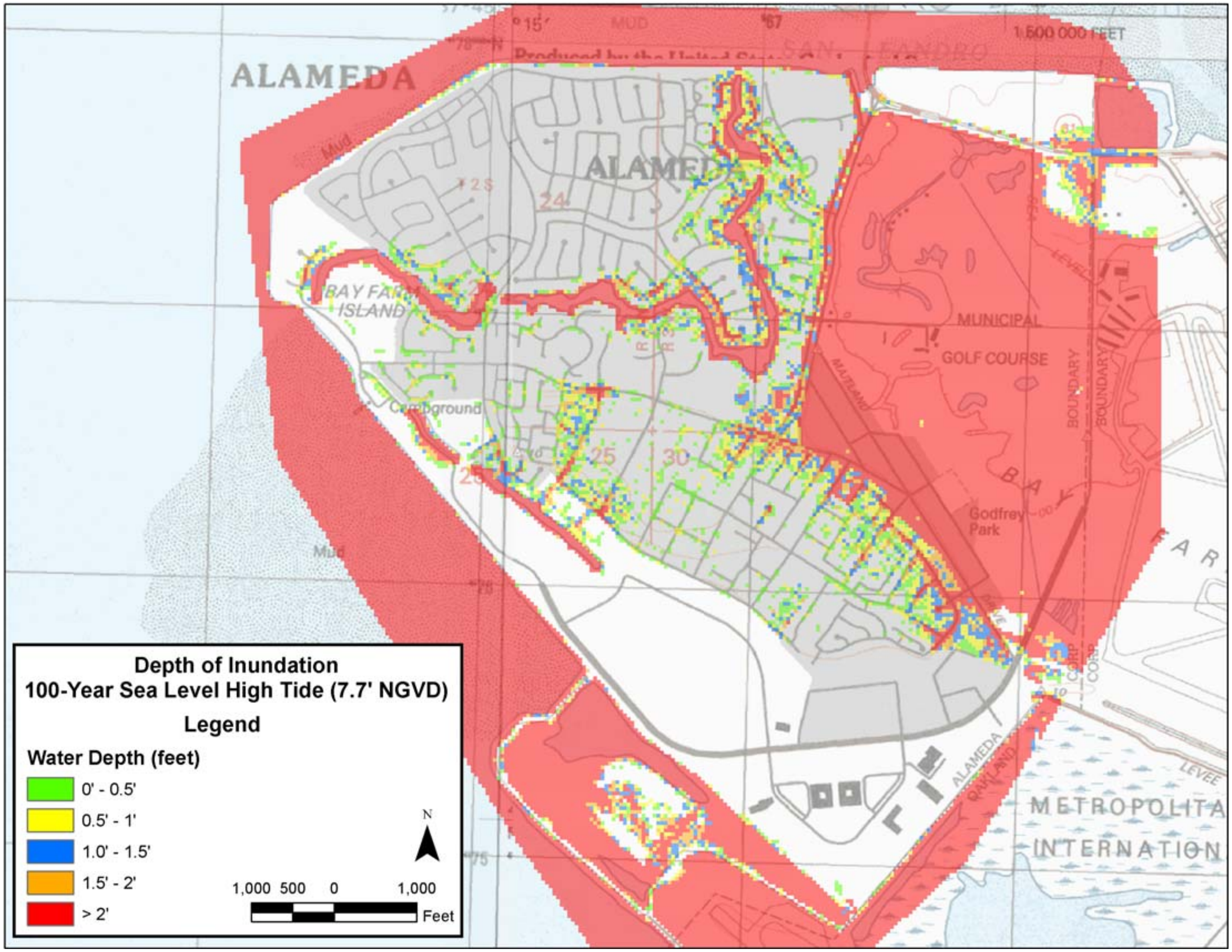


Figure 8: Depth of Water below the 100-year High Tide with 18" of Sea Level Rise, Bay Farm Island

As shown in the figures above, the primary inundation area on the Main Island is the low lying ground in the south eastern portion of the island, as well as the area immediately south of the tunnel (Webster / Posey Tube) between the Cities of Alameda and Oakland. This second area is largely included in the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) 100-year floodplain delineation. On Bay Farm Island, the primary areas of inundation are those areas adjacent to interior lagoons or the low-lying areas of the golf course.

On both Alameda Island and Bay Farm Island, the most significant impact is seen during the peak storm conditions. That is, the increase in mean sea level from 0.5 feet NGVD to 2.0 feet NGVD is relatively small, whereas the area in shaded red, representing the elevation of the peak storm tide with an additional 18 inches of sea level rise, is much larger. The water surface elevation represented by the red area is a peak elevation and is not sustained over long periods of time even during a storm event.

Projects to Mitigate Sea Level Rise Inundation

There are several projects which may partially mitigate the inundation areas shown in Figures 1 through 8 above. As mentioned previously, however, it should be noted that structural improvements are not necessarily a recommended long term strategy to mitigate sea level rise impacts, particularly inundation. Any projects undertaken should include flexibility to adjust or adapt the project for continued sea level rise beyond the 18 inches used for this analysis.

Bay Farm Island lagoon water levels are controlled via pump station outlets, and an intake weir. The operation of the pump stations is currently manual. The operation of the pump stations and the configuration of the intake weirs may need to be adjusted to maintain existing lagoon water surface elevations in the event of sea level rise. While inundation via the golf course appears more significant on the above figures, the peak tide condition is not expected to be maintained long enough to cause the widespread inundation shown. If inundation does occur, a floodwall along Island and Doolittle Drives is one potential mitigation project, however at that stage, the Oakland Airport would also be experiencing flooding due to higher sea levels and coordination with the Airport on flood protection measures is advisable. Alternative mitigation options may include increased pumping capacity at the golf course pump station, or raising the streets bordering the Golf Course to act as flood barriers.

The Main Island inundation map shows the low lying areas of land in south eastern as well as north central Alameda below extreme water surface elevations in the sea level rise scenario. In this case, although only a slim strip of bayfront land is below the high tide with sea level rise elevation, water may also reach many of these areas via storm drain pipes which are currently without flap gates. Projections of sea level rise predict that not only will extreme sea levels occur more often, they may also occur for longer durations (Cayan, 2009). Thus, interior ponding due to backwater from long periods of extreme tides is a possibility. The installation of flap gates at storm drain outfalls would protect City streets from this backwater condition. Due to the relatively short duration of high waters expected, an adaptive management approach which prioritizes projects based on actual backwater experienced is recommended for outfall flap gate installation. As such, flap gates have not been included in this analysis of sea level rise impacts to the CIP.

<u>Storm Drain Capacity</u>

In the 2008 Storm Drain Master Plan, Schaaf & Wheeler presented a Capitol Improvement Program (CIP) to achieve a 10-year level of service for the storm drain network throughout the City of Alameda. The CIP included upsizing existing pipes, additional capacity at several storm drain pump stations, new pipes to provide storm drain capacity to areas currently underserved by the existing system, and several non-capacity related improvements such as trash racks at pump stations. The City directed Schaaf & Wheeler to determine how sea level rise would affect the proposed CIP. In other words, if all CIP projects were completed to meet a 10-year level of service, what additional projects would be necessary to achieve this same level of service assuming that 18" of sea level rise occurs.

For this analysis, Schaaf & Wheeler assumed that sea level rise affects the tide cycle uniformly, that is, both peak and ebb tides are increased by 18 inches. Global warming may in fact impact the tide cycle itself during storm events, particularly storm surge as described above; however numerical projections of these impacts do not currently exist. Figures 9 through 15 show the <u>impact</u> of this sea level rise scenario on the 10-year improved storm drain network.

In addition to the impact scenario described above, Schaaf & Wheeler analyzed the improved 10-year storm drain system operation during a 2-year storm event, but with a 100-year sea level rise scenario tide. This serves to exemplify how the improved system will operate under a relatively minor storm but severe tide. The result of this analysis is essentially identical to the areas shows in Figures 1-6. In this scenario the rainfall is inconsequential, and backwater from the tide cycle determines peak water surface elevation. As described previously, the time period when water surface elevations exceed rim (i.e. ground) elevations is relatively short, generally on the order of 15 minutes or less, although the duration of flooding increases closer to the outfalls due to lower ground elevations at the boundaries of the City.

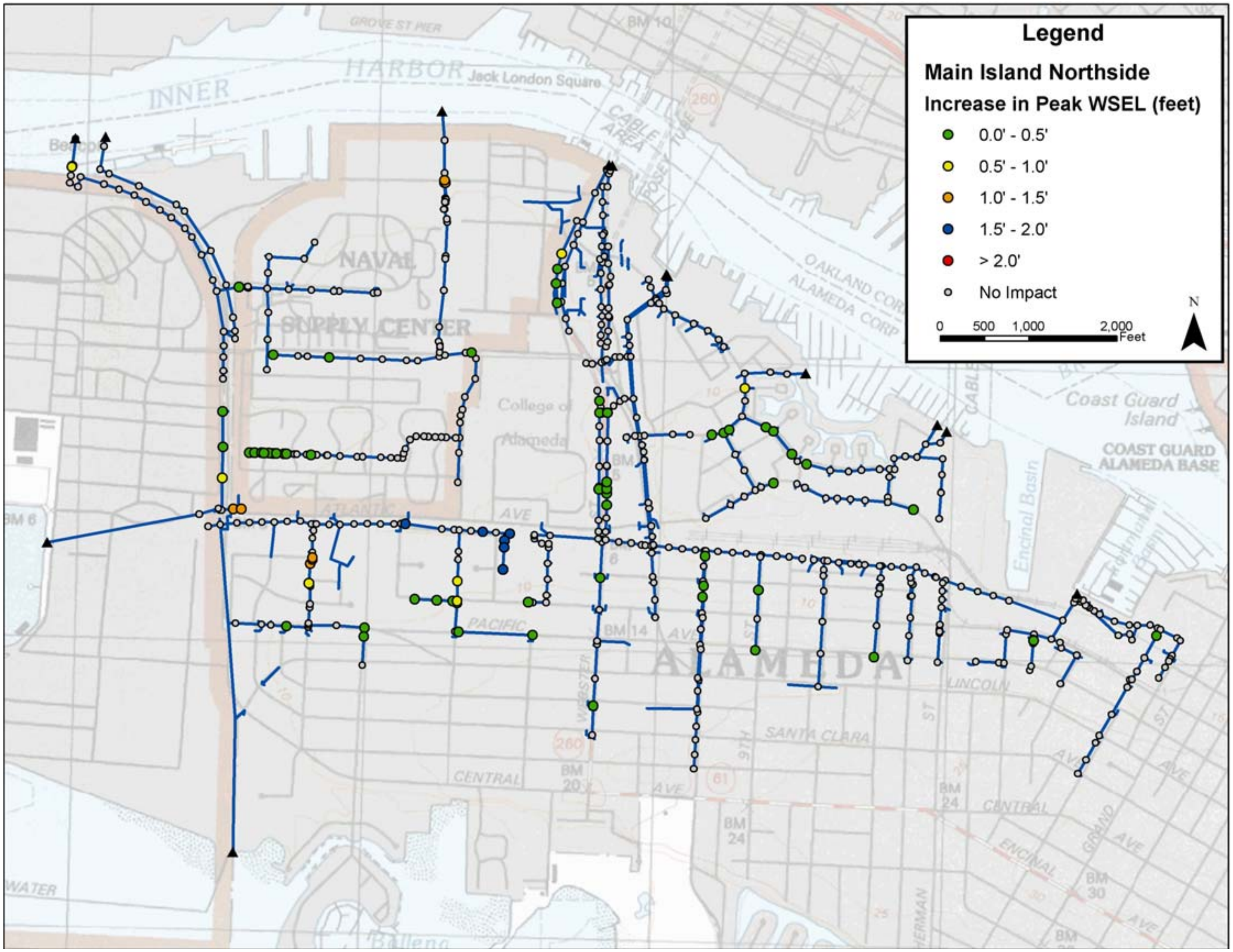


Figure 9: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island Northside

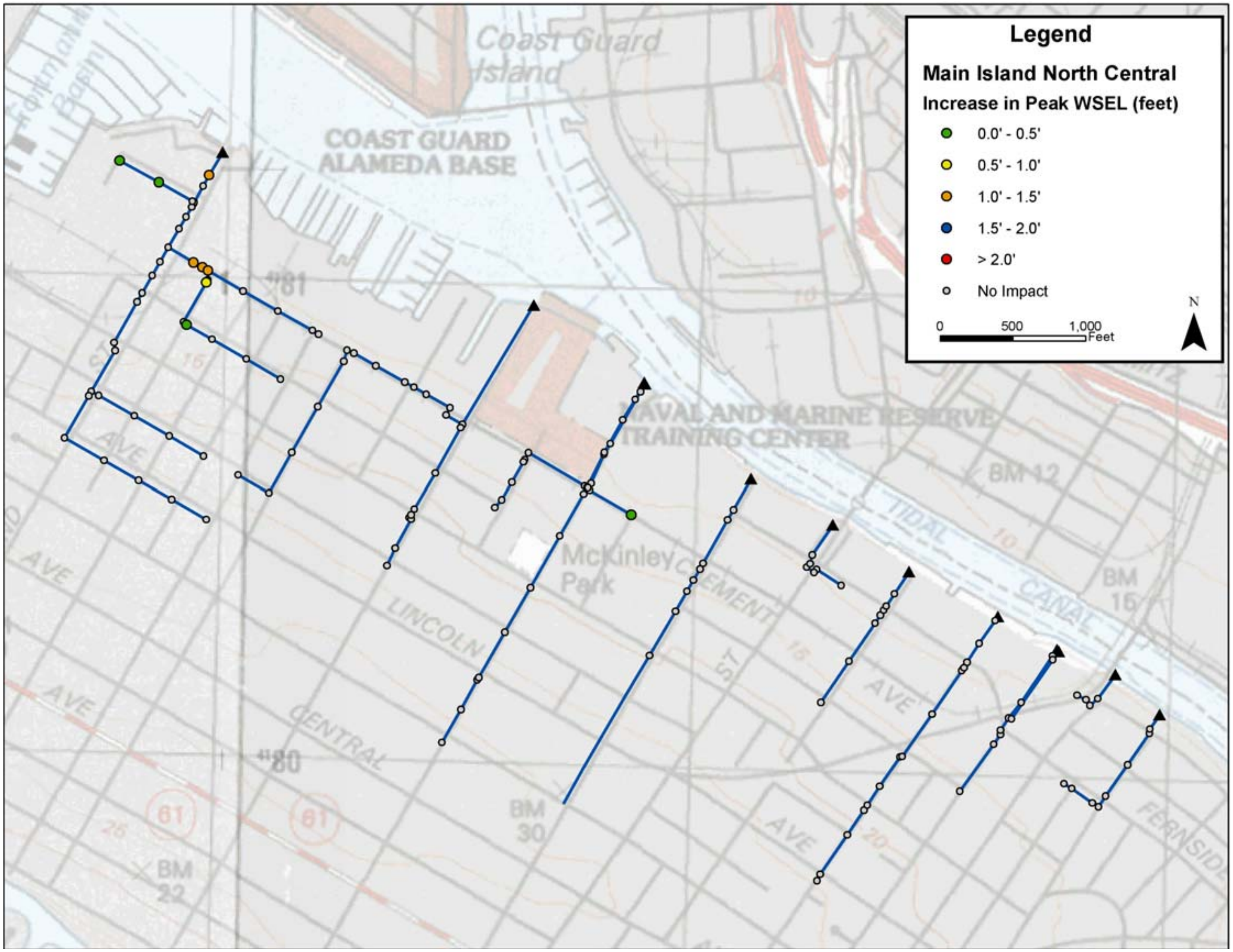


Figure 10: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island North Central

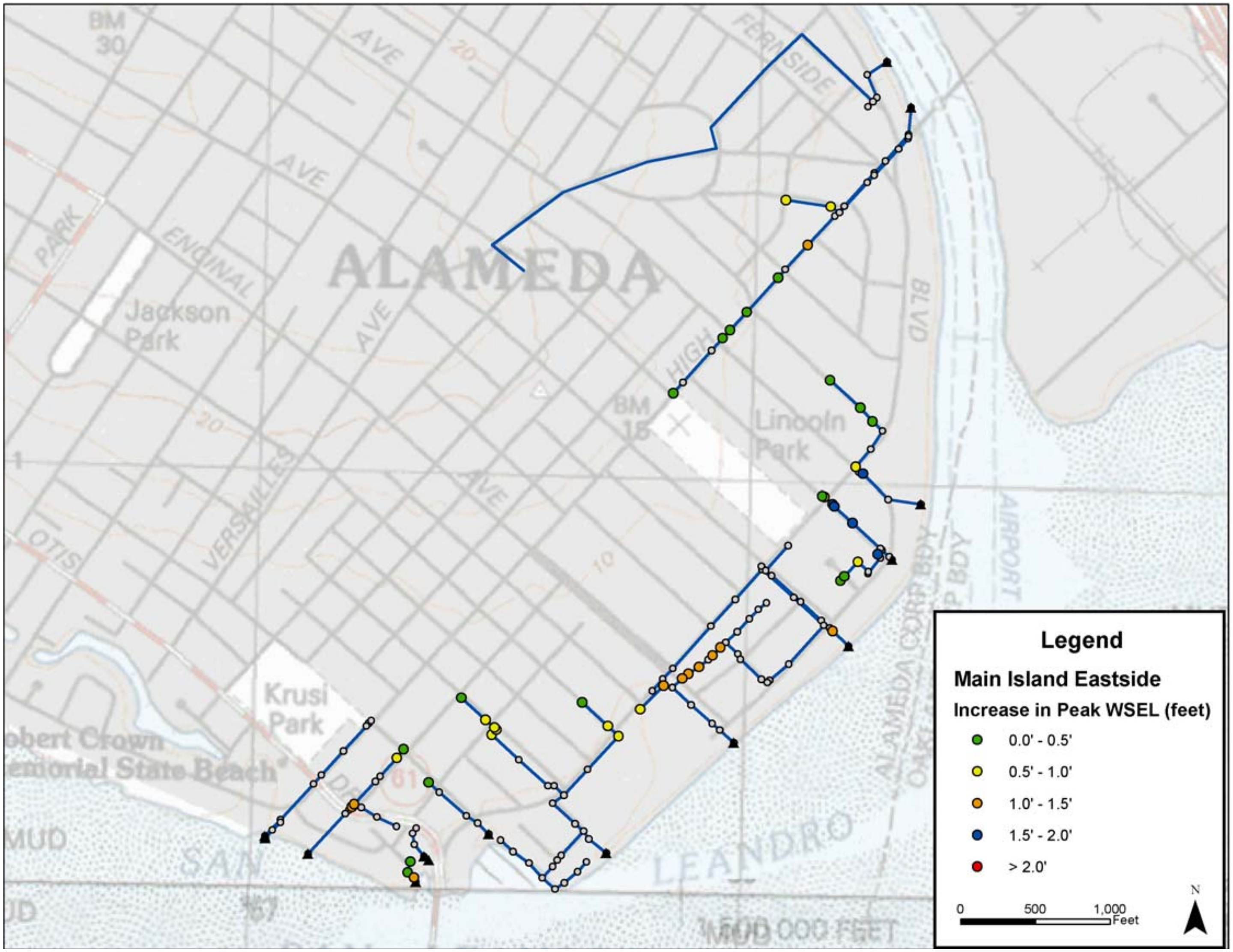


Figure 11: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island Eastside

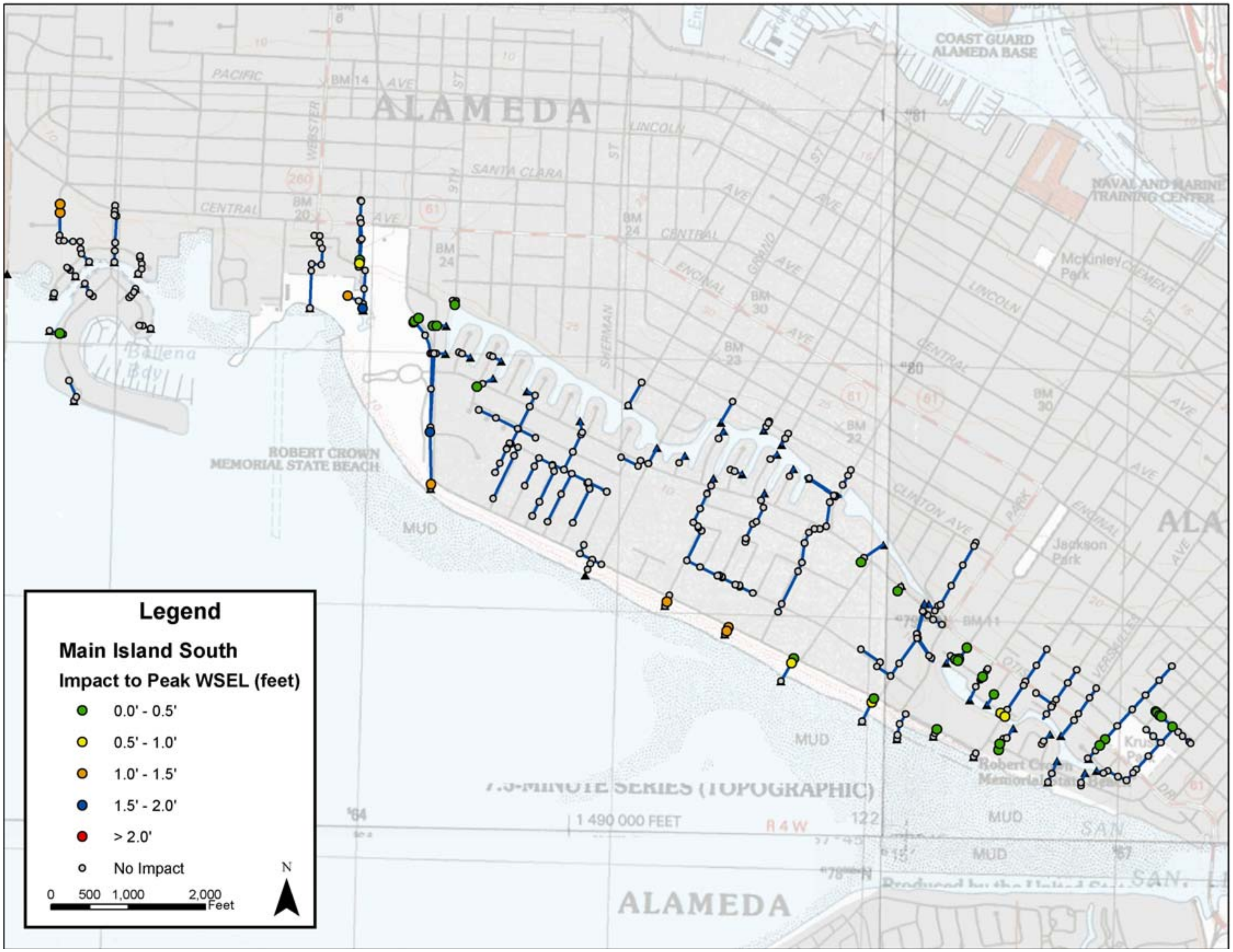


Figure 12: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Main Island South

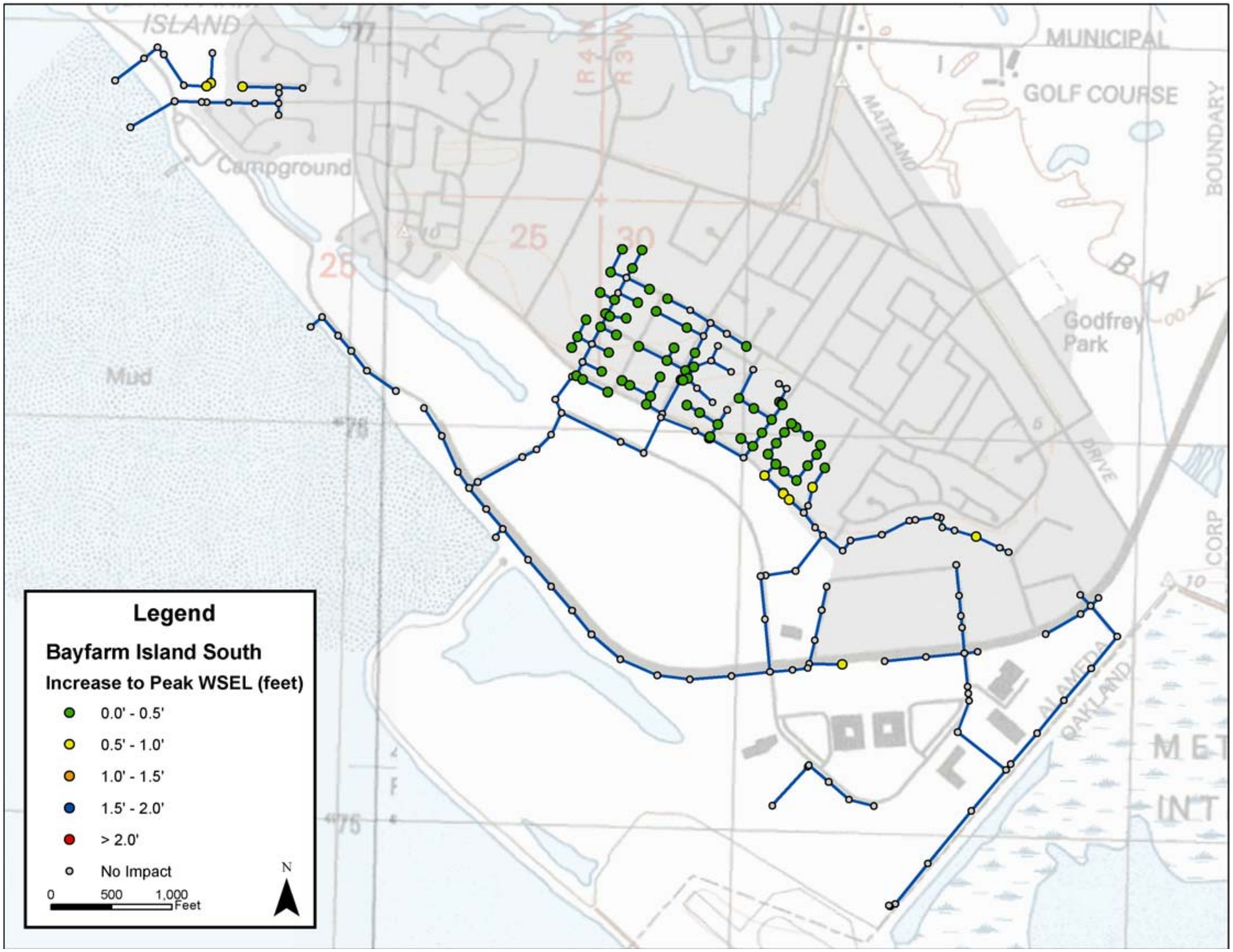


Figure 13: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island South

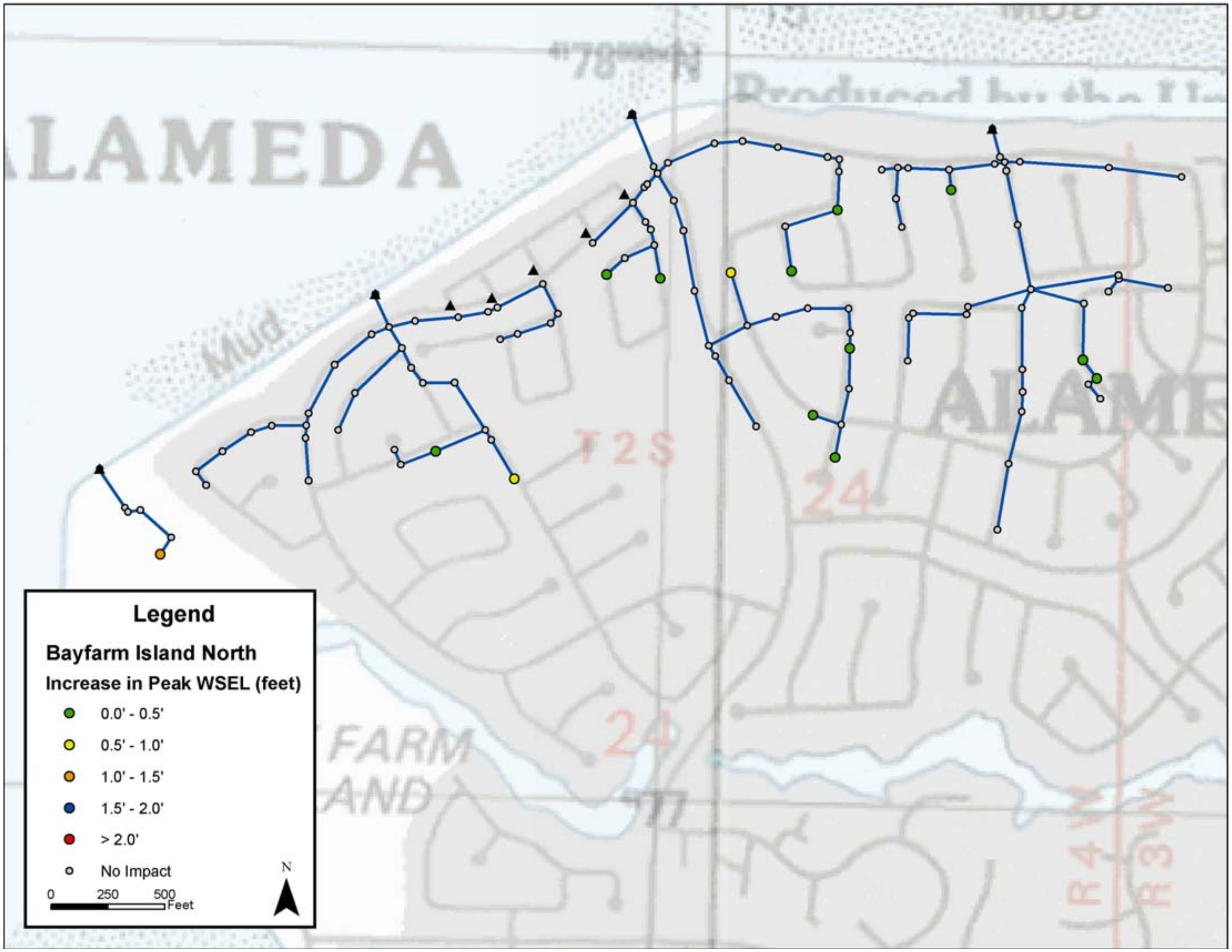


Figure 14: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island North

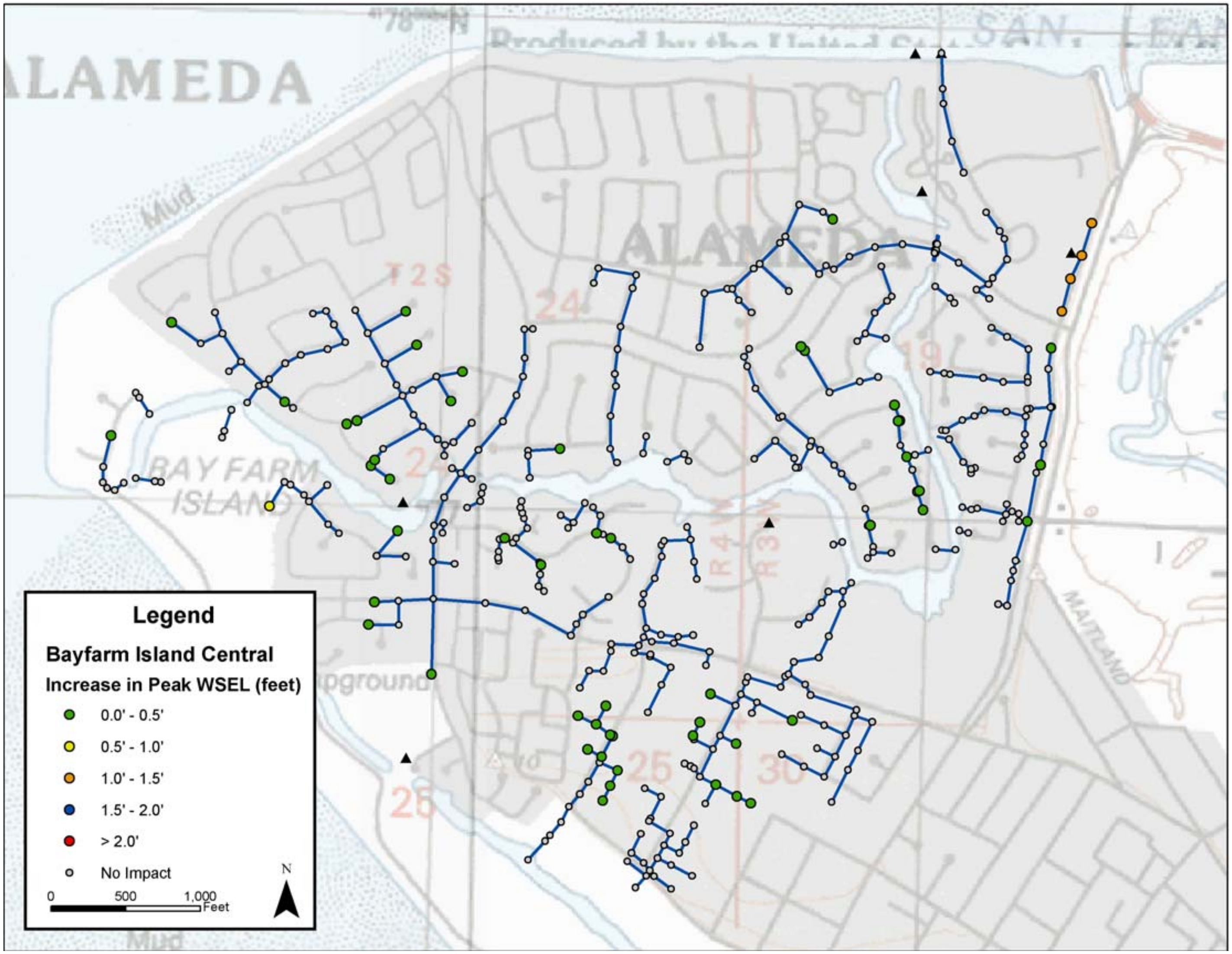


Figure 15: Impact of 18" Sea Level Rise on 10-Year Improved System, 10-Year Storm, Bay Farm Island Central

No figure is included for Bay Farm Island East because there are no impacts in that area. As shown above, in general these impacts are relatively small (most commonly less than a half foot) as expected given the existing conditions 10-year level of service of the system in this scenario.

Projects to Mitigate Impacts of Sea Level Rise on Storm Drain Capacity

Additional projects are required to maintain a 10-year level of service if 18" of sea level rise is applied to the tide cycle. Figures 16 through 22 show additional projects necessary to reach a 10-year level of service under the assumed sea level rise scenario. Pipes which have a white highlighted background are pipes that have been improved from the existing condition to meet existing 10-year service levels. In other words, these pipes are already recommended for improvement in the SDMP CIP, but their recommended size must be adjusted to meet the sea level rise scenario. Pipes which are not highlighted represent new projects where no previous work was recommended to meet 10-year service levels under existing sea levels.

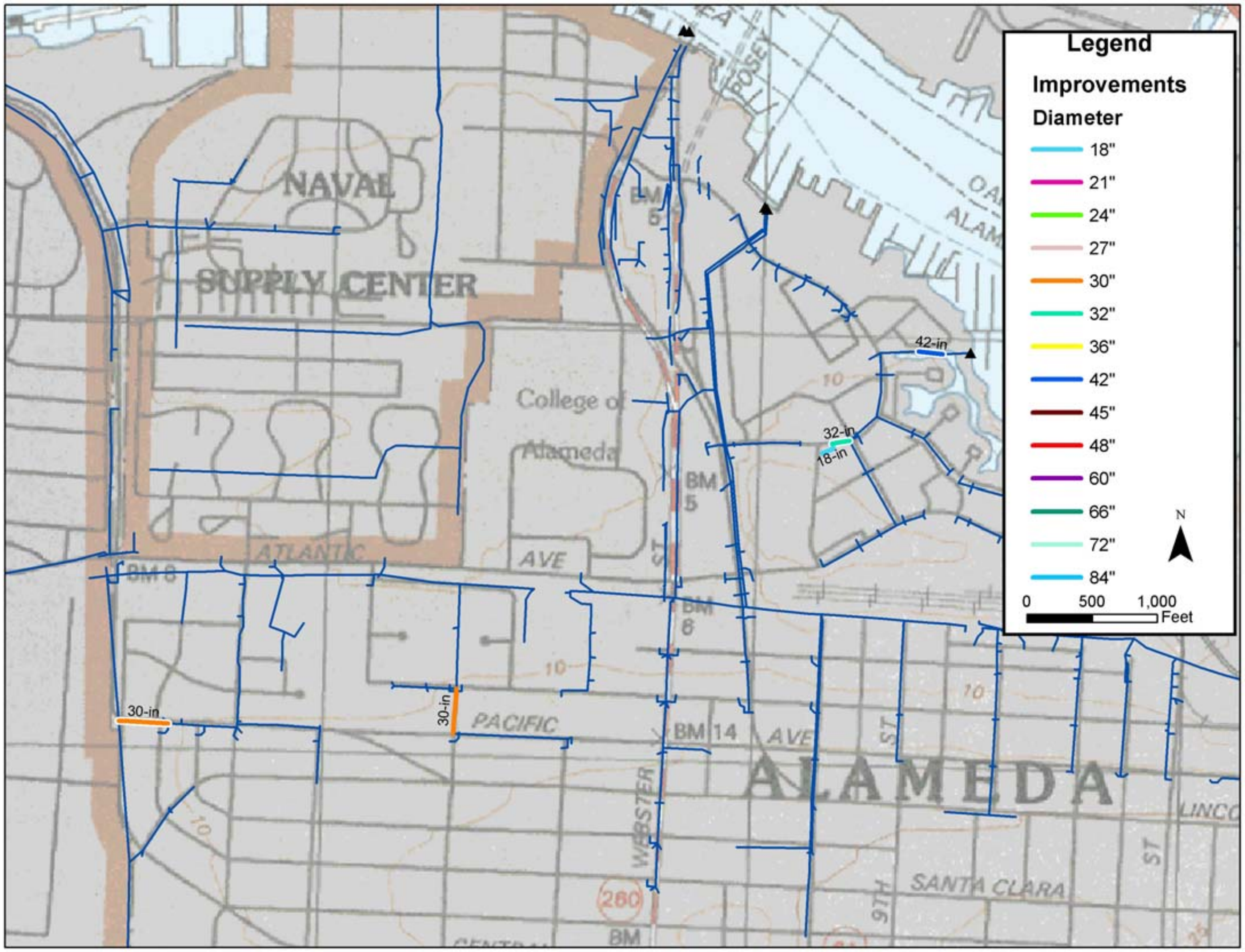


Figure 16: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island Northside

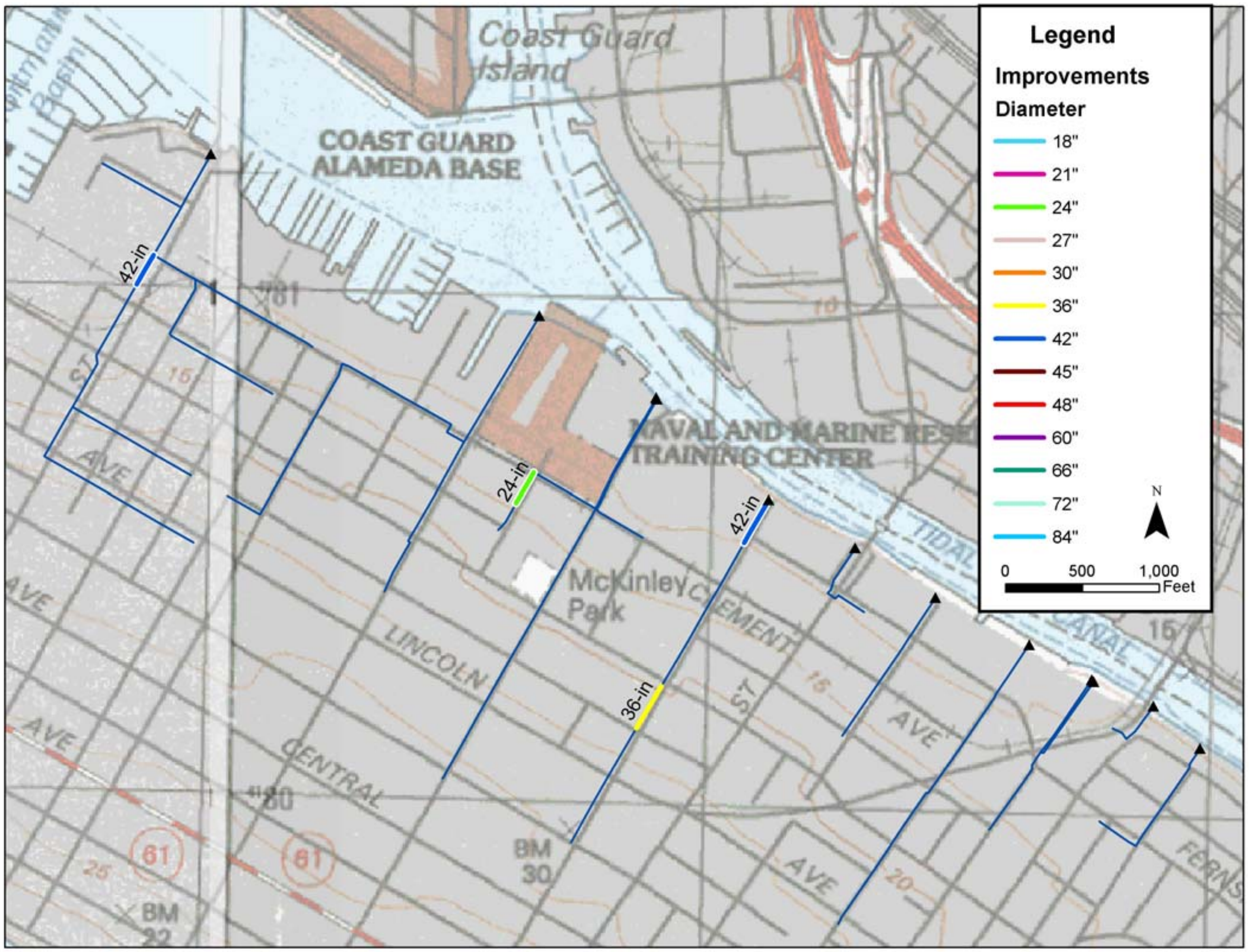


Figure 17: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island North Central

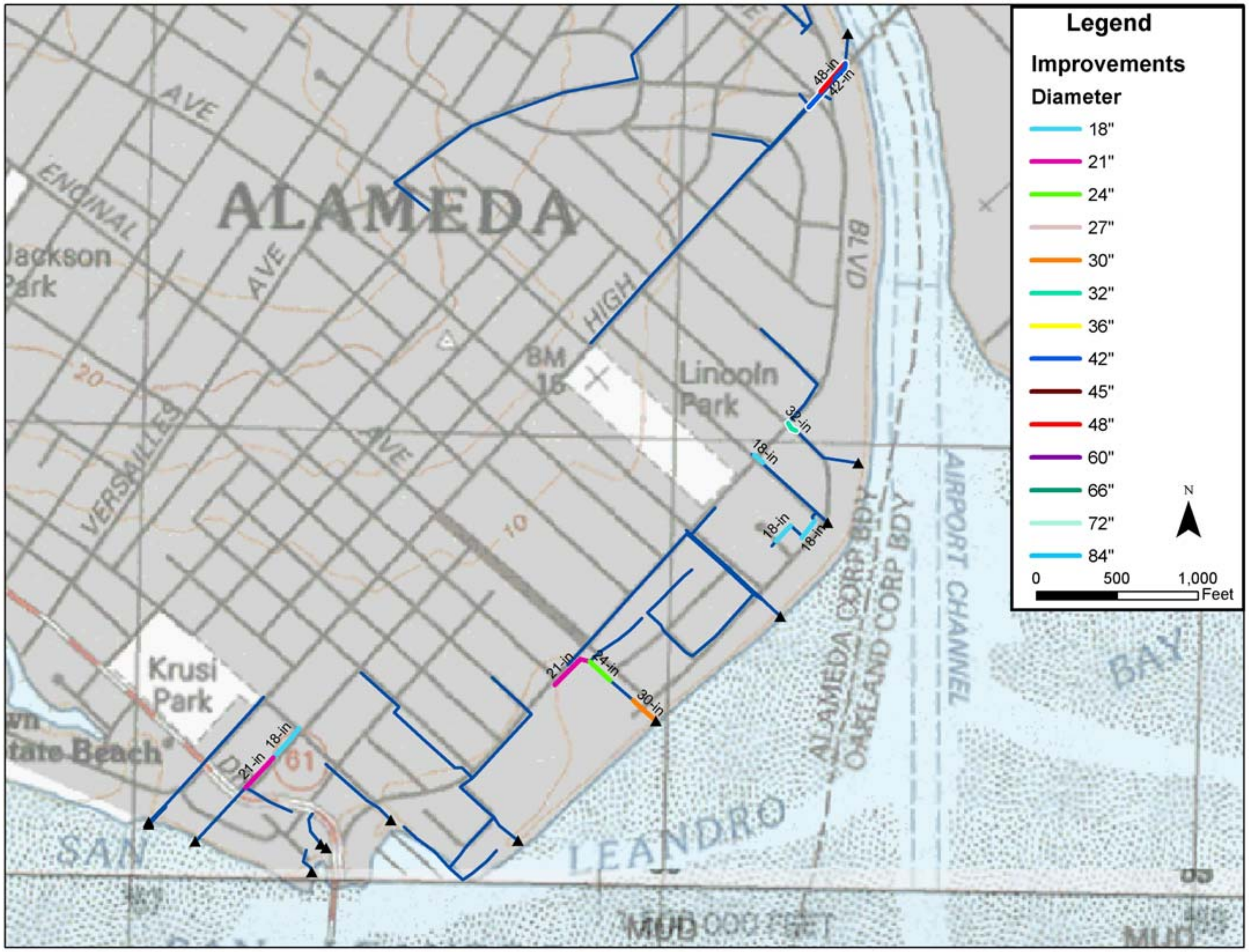


Figure 18: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island Eastside

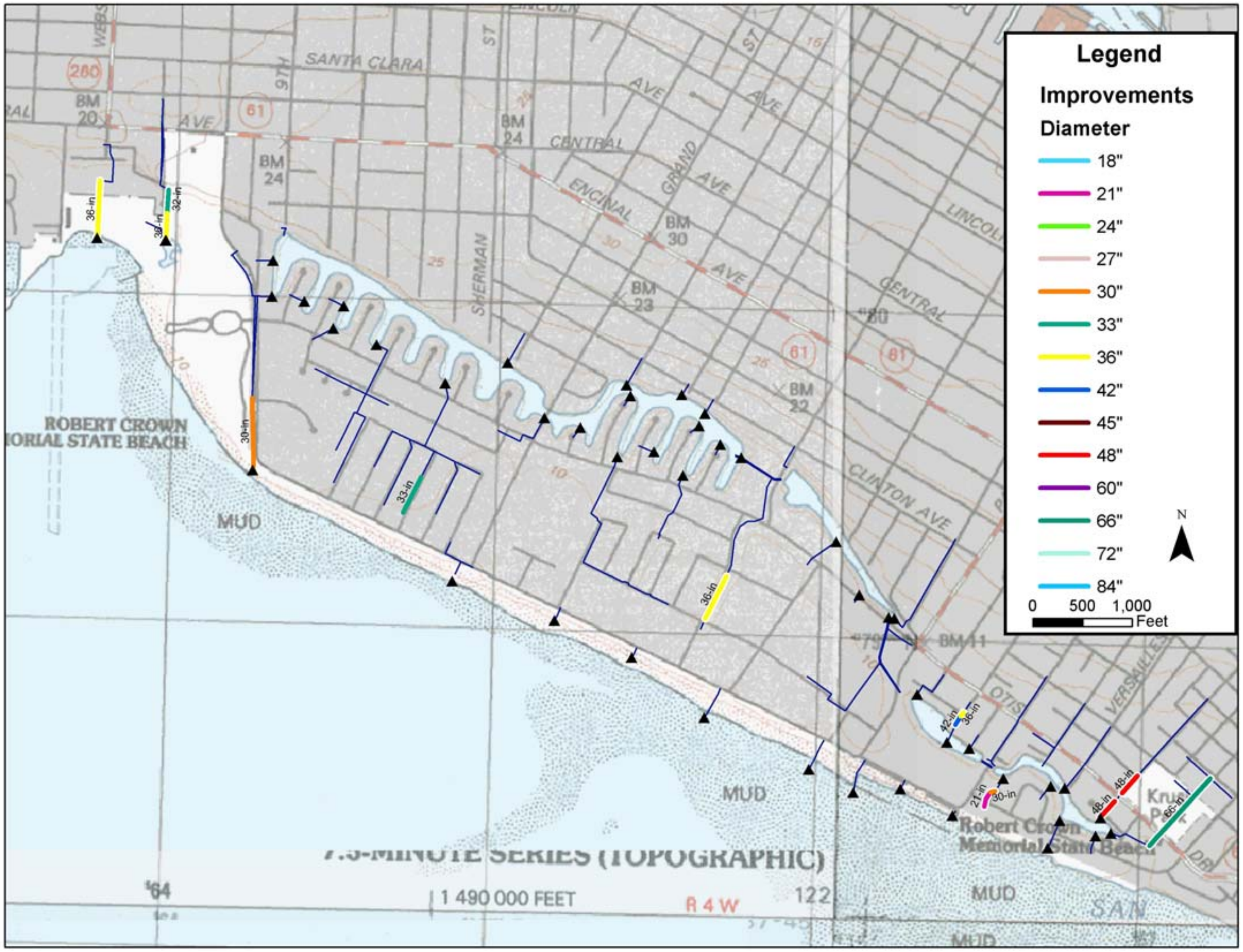


Figure 19: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Main Island South

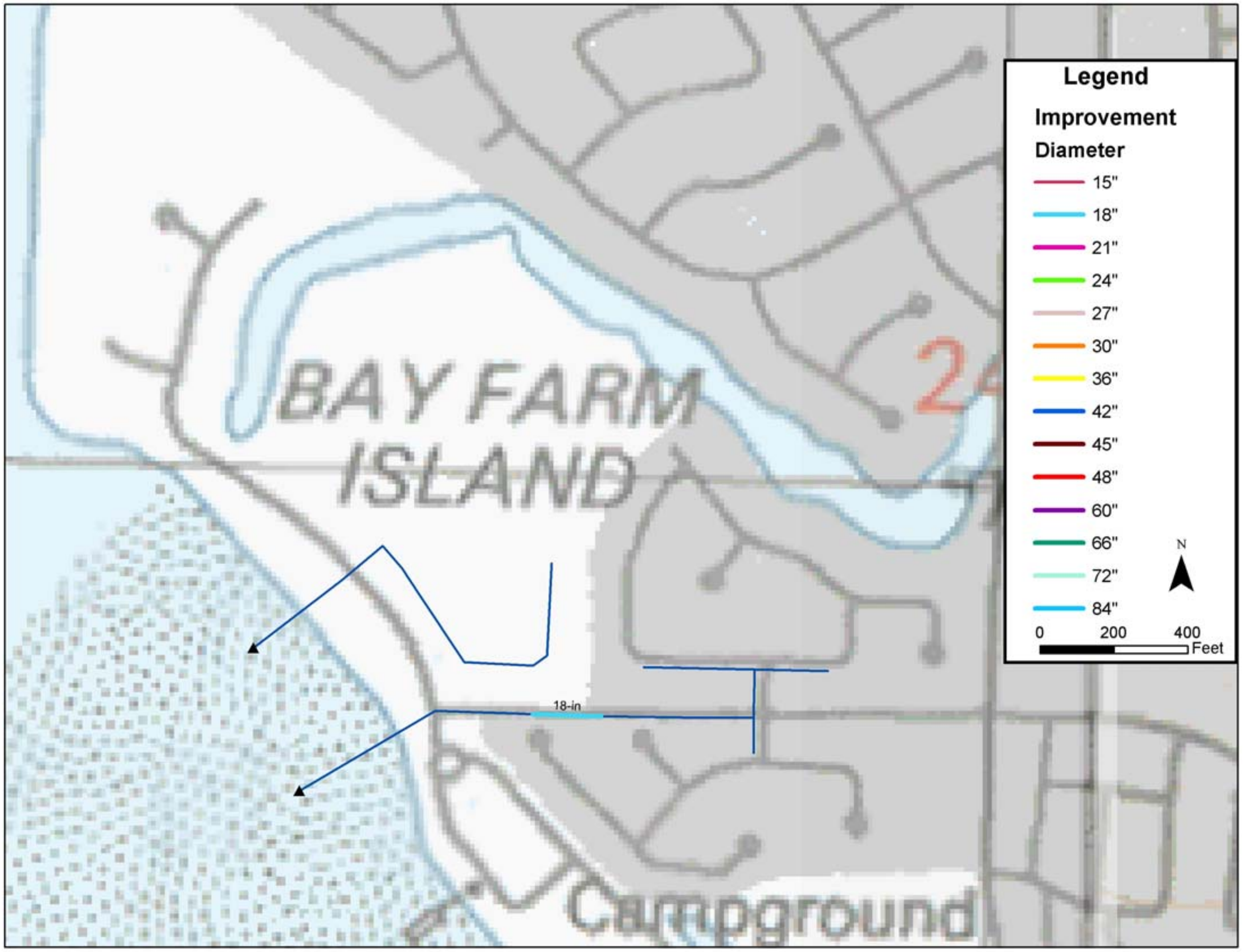


Figure 20: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Bay Farm Island South

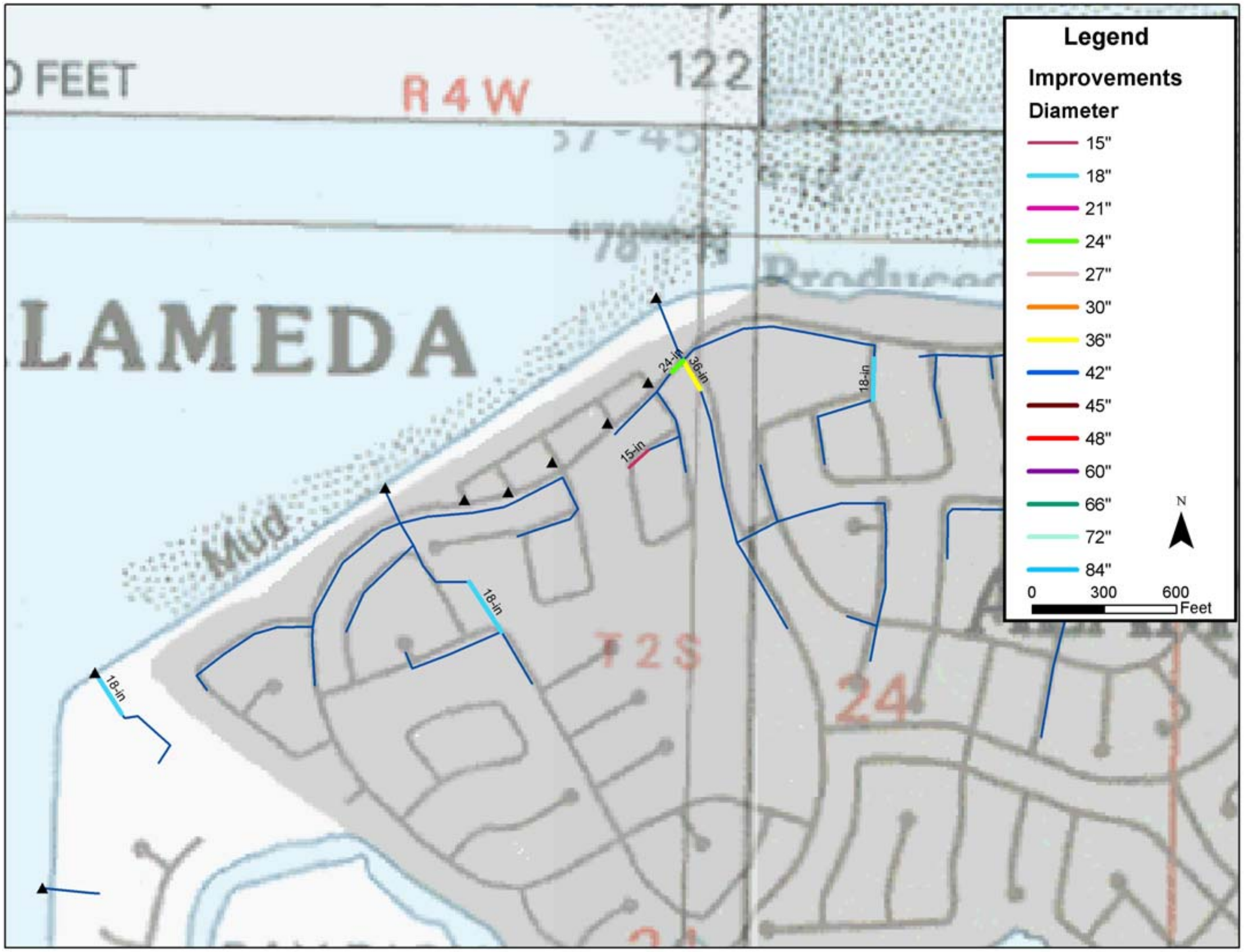


Figure 21: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Bay Farm Island North

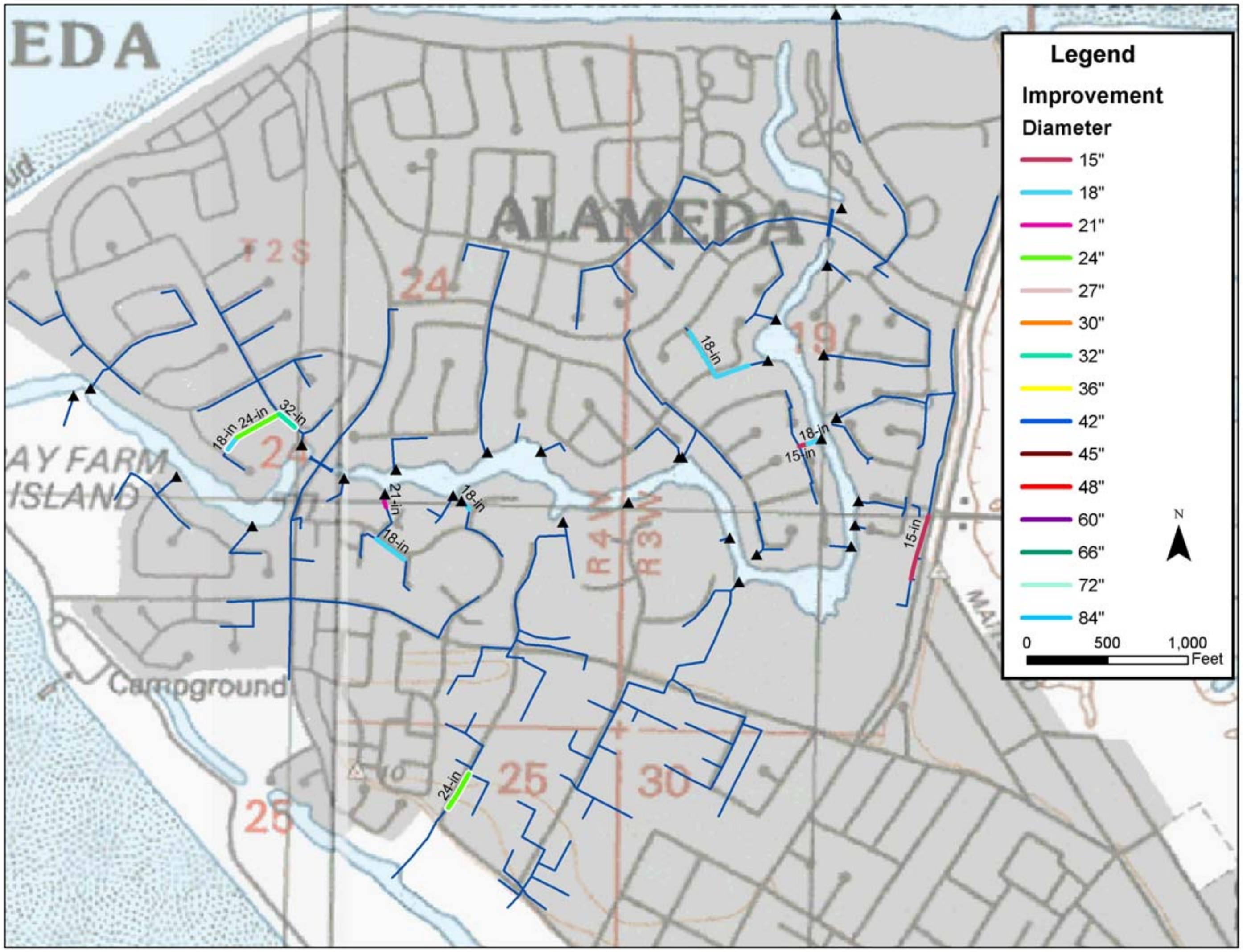


Figure 22: Improvements to Maintain 10-Year Level of Service with 18" of Sea Level Rise, Bay Farm Island Central

Impact of Sea Level Rise Improvements to Storm Drain Master Plan CIP

As shown in Figures 16 through 22 above, new pipe replacement projects, or increased pipe diameters compared to the SDMP CIP are required to maintain a 10-year level of service to the system in the event of 18” of sea level rise. Costs have been estimated using information from other projects, cost estimating guides (2009 Current Construction Costs, Saylor Publications, Inc.), and engineering judgment. These costs are summarized in Table 3.

Table 3: Storm Drain Cost Per Linear Foot

Diameter (inches)	Dollar per Linear foot of Pipe	Dollar per Connection
15	$116	$9,089
18	$128	$9,504
21	$150	$9,668
24	$172	$9,830
27	$194	$9,993
30	$216	$10,157
33	$241	$10,332
36	$267	$10,508
42	$300	$10,870
48	$335	$11,245
54	$369	$11,632
60	$413	$12,051
72	$502	$12,890
96	$679	$14,567

Table 4 summarizes the cost impact of these additional improvements. For pipes which are recommended for improvements in the SDMP (i.e. the highlighted pipes in Figures 14 through 20), the cost included in this table is the difference in costs between the SDMP recommended improvement and the size needed to provide the same level of protection for this sea level rise scenario. Note that costs presented in Table 3 do not include the 40% increase for design, administration, and contingency included in Table 4.

Table 4: Increase in Storm Drain Master Plan Capitol Improvement Program to Maintain 10-Year Level of Service with 18" of Sea Level Rise

City of Alameda Areas	Additional Costs to SDMP CIP
Main Island Eastside	$711,000
Main Island North Central	$190,000
Main Island South	$652,000
Main Island Northside	$234,000
MAIN ISLAND TOTAL	**$1,800,000**
Bay Farm Island North	$325,000
Bay Farm Island Central	$542,000
Bay Farm Island South	$70,000
BAY FARM ISLAND TOTAL	**$937,000**
CITY OF ALAMEDA TOTAL	**$2,700,000**

More detailed cost summary tables are included in Appendix A.

In addition to the costs of structural improvements (i.e. increased storm drain capacity requirements), there are also indirect costs to the City due to the sea level rise scenario. Due to the change in boundary conditions, storm drain pumps may run for longer periods of time, resulting in increased energy usage, maintenance and replacement costs. If the golf course is more often rendered unusable by flood waters, this could also indirectly impact City economics. While these costs are expected to be small compared to the improvement costs, they will be experienced regardless of projects undertaken to mitigate storm drain performance.

Current Status of Regulations Pertaining to Climate Change

The current status of potential regulations pertaining to climate change is explored below. Research and regulations regarding climate change are regularly, and sometimes rapidly, updated and modified; thus this section should be considered representative, and may not represent a complete list of current or pending regulations.

<u>Federal</u>

At a Federal level there are currently very few recommendations or guidelines for incorporating the risks of sea level rise into project planning, and virtually no required measures. It should be noted, however, that with the administration change of 2009, based on President Obama's statements that global warming is a priority of the new administration, relatively rapid changes in the Federal government's involvement in global warming analyses and impacts may be forthcoming. Thus far it appears that those changes will be focused on emission standards as opposed to impact mitigation.

Flood Programs - Federal Emergency Management Agency

Although the Federal Emergency Management Agency (FEMA) has issued several statements in the last decade pertaining to climate change and the risks of global warming, at this time FEMA policy has not changed to reflect these risks or impacts. Sea level rise is not directly considered in the National Flood Insurance Program (NFIP). In 2001 FEMA published a report on the projected impact of relative sea level rise on the NFIP, which concluded that the NFIP would not be significantly impacted by sea level rises under one foot by the year 2100, and the gradual timeframe of sea level rise provides ample opportunity for the NFIP to consider alternatives and implement them. The report recommended that FEMA should continue to monitor analyses and predictions of sea level rise and strengthen the Community Rating System (CRS) by encouraging measures that would mitigate the impacts of sea level rise (FEMA, 1991).

In March 2007 the United States Government Accountability Office published a report on the financial risks to federal and private insurers as a result of climate change, and recommended that the NFIP analyze the potential long-term fiscal implications of climate change and report these findings to Congress (GAO-07-285, March 2007). It is foreseeable that when this analysis takes place, changes to the NFIP will be made to lessen the financial risk to the insurers. Potential policy changes may include increased freeboard requirements for Bay or Riverfront levees and/or some consideration or discussion of sea level change in floodplain analyses, but when or if any policy changes will occur is unknown.

On March 17, 2009, the National Association of Insurance Commissioners (NAIC) adopted a mandatory requirement that insurance companies disclose to regulators the financial risks they face from climate change, and actions that the companies are taking to respond to those risks. This requirement impacts all insurance companies with annual premiums of $500 million or more. Those companies must complete an “Insurer Climate Risk Disclosure Survey” every year, with the first report due on May 1, 2010.

Research on Climate Change - National Oceanic and Atmospheric Administration

The National Oceanic and Atmospheric Administration (NOAA) is the federal agency that appears to have taken the lead in analyses of the impacts of global warming to the United States of America. NOAA is primarily a scientific research and reporting agency, with little regulatory power. From the NOAA webpage:

> "NOAA is charged with helping society understand, plan for, and respond to climate variability and change. This is achieved through the development and delivery of climate information services, the implementation of a global observing system, and focused research and modeling to understand key climate processes. The NOAA climate mission is an end-to-end endeavor focused on providing a predictive understanding of the global climate system so the public can incorporate the information and products into their decisions."

Recent budget proposals from President Obama suggest that this responsibility may shift from NOAA to NASA in the future.

<u>State</u>

California has been on the leading edge of creating legislation to mitigate both greenhouse gas emissions and the impacts of climate change. At this time, several concrete steps have been taken to reduce greenhouse gas emissions in the state, while specific impact mitigation strategies have been recommended but not fully developed. The California Climate Action Team, described in detail earlier in this report, is responsible for coordinating state-level actions relating to climate change.

Assembly Bill 32

The California Global Warming Solution Act, also known at Assembly Bill 32 (AB32), was signed into law by Governor Schwarzenegger in 2006. AB32 requires the California Air Resources Board (CARB) to:

- Establish a statewide greenhouse gas emissions cap for 2020, based on 1990 emissions by January 1, 2008.
- Adopt mandatory reporting rules for significant sources of greenhouse gases by January 1, 2009.
- Adopt a plan by January 1, 2009 indicating how emission reductions will be achieved from significant greenhouse gas sources via regulations, market mechanisms and other actions.
- Adopt regulations by January 1, 2011 to achieve the maximum technologically feasible and cost-effective reductions in greenhouse gas, including provisions for using both market mechanisms and alternative compliance mechanisms.
- Convene an Environmental Justice Advisory Committee and an Economic and Technology Advancement Advisory Committee to advise CARB.
- Ensure public notice and opportunity for comment for all CARB actions.

- Prior to imposing any mandates or authorizing market mechanisms, CARB must evaluate several factors, including but not limited to impacts on California's economy, the environment and public health; equity between regulated entities; electricity reliability, conformance with other environmental laws and ensure that the rules do not disproportionately impact low-income communities.

In September, 2008, Governor Schwarzenegger signed Senate Bill 375, which builds on AB32 by requiring the CARB to develop regional greenhouse gas emission reduction targets to be achieved from the automobile and light truck sectors for 2020 and 2035. Both AB32 and Senate Bill 375 focus on reducing greenhouse gas emissions, as opposed to predicting or mitigating climate change impacts in California.

AB 32 Scoping Plan

The AB 32 Scoping Plan contains the main strategies California will use to reduce the greenhouse gases (GHG) that cause climate change. The Scoping Plan has a range of GHG reduction actions which include direct regulations, alternative compliance mechanisms, monetary and non-monetary incentives, voluntary actions, market-based mechanisms such as a cap-and-trade system, and an administrative fee to fund the program. The AB 32 Scoping Plan was approved at the Air Resources Board hearing on December 11, 2008.

Six greenhouse gas emission reduction measures are proposed for the Water sector in the Scoping Plan. They address water use efficiency, water recycling, water system energy efficiency, reuse of urban runoff, increased renewable energy production and public goods charges for funding investments that improve water and energy efficiency (CARB, 2008).

California Environmental Quality Act

The California Governor’s Office of Planning and Research is expected to certify and adopt amendments to the CEQA Guidelines which incorporate analyses and mitigation of Greenhouse Gas Emissions (GHG) on or before January 1, 2010 (CA Governor’s Office, 2008). In the interim, the Office of Planning and Research has created a technical advisory which includes the recommended approach for incorporating climate change impacts to the CEQA process. The recommended approach includes recommendations for approaches to identifying project GHG emissions, determining significance, and mitigating the impacts.

California Adaptation Strategy

In November, 2008, Governor Schwarzenegger signed Executive Order S-13-08 (EO), which calls for the development of California's first statewide climate change adaptation strategy to assess the state's expected climate change impacts, vulnerabilities, and recommend climate adaptation policies, to be completed by 2009. This is the first legislative action to initiate active planning for the impacts of global warming in the state of California. In addition to the climate change adaptation strategy, the EO also requests that the National Academy of Science establishes an expert panel to report on sea level rise impacts in California, issues interim guidance to state agencies for how to plan for sea level rise in designated coastal and floodplain areas for new projects, and initiates a report on critical infrastructure (planned and existing) vulnerable to sea level rise. In the interim, all state agencies planning construction projects are directed to consider a range of sea level rise scenarios for the years 2050 and 2100 in order to assess project vulnerability and, to the extent feasible, reduce expected risks and increase resiliency to sea level rise (CA Governor Press Release, 2008).

California Water Plan

Following the passage of AB 32 in 2006 which called for a reduction in greenhouse gas emissions, DWR voluntarily joined the California Climate Action Registry. DWR addresses climate change in its California Water Plan, updated every five years, that provides a framework for water managers, legislators, and the public to consider options and make decisions regarding California's water future. In July, 2008, DWR published a technical memorandum report on the progress of incorporating climate change into the management of California's water resources. The focus of this report was the impact of global warming to California's water supply, although increased flood risks were presented in brief. In October 2008, the Department released a climate change white paper that proposes a series of adaptation strategies for state and local water managers to improve their capacity to handle change. On a regional level these strategies include integrated water management and increased water use efficiency.

<u>Local</u>

City of Alameda

The City of Alameda completed a Local Action Plan for Climate Protection in February 2008. This Plan identified initiatives to reduce City-wide greenhouse gas emissions by 25% of the 2005 levels by 2020. Initiatives are divided into four categories: transportation and land use, energy, waste and recycled, and community outreach and education. Initiatives that are particularly relevant to new or re-development include (City of Alameda, 2008):

- Requirement that all new major developments' short and long-term transportation emissions impacts are reduced by 10%;
- Require that all recommended City Council actions include an analysis or evaluation of whether the action supports or is consisten with Alameda's Local Action Plan Initiatives and futhers progress toward the Greenhouse Gases Reduction Target;
- Ammend the Alameda Municipal Code to include sustainable design and green building standards for all new, substaintially expanded, and remodeled buildings; and
- Develop a wood-burning prohibition ordiancne to reduce air pollution for new residential construction.

East Bay Municipal Utility District

The East Bay Municipal Utility District (EBMUD) provides domestic water to the City of Alameda. EBMUD's primary source of water is the Mokelumne River watershed, which is fed by snowpack in the Sierra Mountains. In addition to the current drought, climate change is expected to decrease snow pack, and thus snow melt and water supply, in coming years. In 2008, EBMUD incorporated climate change into its strategic plan, and is currently pursuing water conservation, water recycling, and seeking out additional water sources for future use (Wallis, 2008). The City of Alameda already has several water conservation programs, but additional reductions may eventually be required by EBMUD to address decreasing water supply as a result of climate change.

San Francisco Bay Conservation and Development District

The San Francisco Bay Conservation and Development District (BCDC) is a state agency created in 1965 to regulate development in the Bay and along its shoreline for the purpose of limiting and controlling the amount of fill placed in the Bay. In October 2007 BCDC released an eight year regional strategy for controlling greenhouse gases and preparing for the impacts of sea level rise of San Francisco Bay. BCDC does not have the authority or responsibility to initiate many of the identified strategies. In September 2008 BCDC released a revised strategy which considers the regulatory limitations of the agency.

In May 2009, BCDC submitted preliminary recommendations for amendments to the Bay Plan to incorporate climate change. This proposal adopts sea level rise estimates of 16 inches (1.3 feet) by 2050 and 55 inches (4.6 feet) by 2100. Proposed changes to the Bay Plan which may be relevant to the City include the following (Travis, 2009):

- “Addressing the impacts of sea level rise and shoreline flooding may require large-scale flood protection projects, including some that extend across jurisdictional or property boundary. Coordination with adjacent property owners or jurisdictions to create contiguous, effective shoreline protection is critical when planning and constructing flood protection projects. Failure to coordinate may result in inadequate shoreline protection (e.g., a protection system with gaps or one that causes accelerated erosion in adjacent areas)”

- “New shoreline protection projects and the maintenance or reconstruction of existing projects should be authorized if: (a) the project is necessary to proect the shoreline from erosion or to protect shoreline development from flooding; (b) the type of the protective structure is appropriate for the project site, the uses to be protected, and the erosion and flooding conditions at the site, (c) the project is properly engineering to provide erosion control and flood protection for the expected life of the project based on a 100-year flood event that takes future sea level rise into account; (d) the project is properly designed and constructed to prevent signiciant impediments to physical and visual public access; and (e) the protection is integraded with adjacent shoreline protection measures.”

- “…the Commission should…encourage new projects on the shoreline to be set back from the edge of the shore above a 100-year flood level that takes future sea level rise into account for the expected life of the project, or otherwise be specifically designed to tolerate sea level rise and stroms and to minimize environmental impacts; discourage new projects that will require new structureal shoreline protection during the expected life of the projects, especially where no shoreline protection currently exits [*sic*]; determine whether alternative measures that would involve less fill or impacts to the Bay are feasible; require an assessment of risks from a 100-year flood that takes future sea level rise into account for the expected life of the project; and require that where shoreline protection is necessary, ecosystem impacts are minimized.”.

- “The Commission may approve fill that is needed to provide flood protection for existing projects. New projects on fill or near the shoreline should either be set back from the edge of the shore so that the project will not be subject to dynamic wave energy, be built so the bottom floor level of structures will be above a 100-year flood elevation that takes future sea level rise into account for the expected life of the project, be specifically designed to tolerate periodic flooding, or employ other effective means of addressing the impacts of future sea level rise and storm activity. Right-of-way for levees or other structures protecting inland areas from tidal flooding should be sufficiently wide on the upland side to allow for future levee widening to support additional levee height so that no fill for levee widening is placed in the Bay.”

- "Design and evaluation (of any ecosystem restoration project) should include an analysis of: (a) how the system's adaptive capacity can be enhanced so that it is resilient to sea level rise an climate change…(h) an appropriate buffer, where feasible, between shoreline development and habitats to protect wildlife and provide space for marsh migration as sea level rises…".

- "Public access should be sited, designed, managed, and maintained to avoid significant adverse impacts from sea level rise and shoreline flooding."

These changes, if approved, may have significant impacts on the City's approach to development, planning, and design of both flood control projects and new or re-development within portions of the City.

Other Storm Drain Master Plan Updates

During the preparation of this report, the Northside (Marina Village) pump station experienced failure during an approximately 10-year storm event. The impact to the City's storm drainage system operation during the storm due to this failure was significant. The Northside (Marina Village) pump station is connected via storm drains to the Arbor pump station. In the SDMP, standby power is identified as high priority improvements for both the Arbor and Northside (Marina Village) pump stations. Capacity improvements are also recommended at the Arbor pump station. In addition, the operation of the Main Island lagoon system has experienced elevation water levels during storm events.

Due to the recent pump station failure and its consequences, the Northside (Marina Village) pump station is currently a high priority storm drain pump station for potential improvement. The pump stations' generators, panels, pumps, motors and layout need to be optimized to provide the highest level of service possible. It may be feasible that increasing the capacity of the Northside (Marina Village) pump station while this work occurs would reduce or remove the need for increased capacity at Arbor pump station. A more detailed analysis of how this connected system operates and whether improvements at Northside (Marina Village) may offset recommended improvements at Arbor pump station is needed. Additional analysis and eventual improvements of the Main Island Lagoon system are also needed.

Summary

As an island community, the City of Alameda is uniquely exposed to climate change impacts to the San Francisco Bay region, particularly rising sea levels. Rising sea levels may impact the City via both inundation of City lands by higher mean sea levels and tide cycles, and also may impact the capacity and operation of its storm drain system. At this time, structural projects to mitigate inundation from surrounding waters, such as floodwalls, levees, elevating structures, etc. are not recommended due to the inherent uncertainty and long time scale of sea level rise projections. If structural solutions are sought in the long term, coordination with the Oakland Airport adjacent to Bay Farm Island will be essential to protect that portion of the City.

Projects have been identified to maintain a 10-year level of service for the storm drain network in the event of 18" of sea level rise. Many of these storm drain system improvements are increased pipe diameters in areas already identified for capacity improvements in the SDMP. The impact to the SDMP CIP is $2,700,000. The majority of the additional required improvements are on the Main Island, and of those, the majority are located in the South and Eastside areas. Given the relatively low cost to install a slightly larger pipe if a pipe replacement project is already planned, Schaaf & Wheeler recommends using the pipe sizes herein for pipe replacement projects undertaken in the future. Replacing pipes for the sole purpose of meeting the sea level rise scenario 10-year level of service should be considered low priority.

Regulations regarding climate change are currently in a state of rapid development and fluctuation. At this time, the most significant existing regulations potentially affecting the City are those contained in the City Local Action Plan. Based on our findings, Schaaf & Wheeler concludes that it is likely that significant development of the former naval base in the future will be required to study and mitigate for not only greenhouse gas emissions, but also future sea level rise scenarios.

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Appendix A: Detailed Cost Calculations

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island Eastside						**$687,000**	**$179,000**	**$508,000**
SL141	1.25	15	1	75		$17,324	-	
SL142	1.50	18	1	166		$30,263	-	
SL146	1.75	21	2	206		$49,165	-	
SL195	1.75	21	1	225		$42,823	-	
SL2_2	1.75	21	1	48		$16,395	-	
SL206	1.75	21	1	60		$18,095	-	
SL207	2.00	24	2	172		$48,181	-	
SL275	1.50	18	2	133		$35,018	-	
SL280	1.50	18	2	134		$35,143	-	
SL300	2.50	30	1	126		$36,996	-	
SL325	2.50	30	1	67		$24,126	-	
SL97	1.50	18	2	72		$27,313	-	
SLIMP_104	2.75	33	1	41	21	$19,797	$15,369	
SLIMP_105	2.75	33	2	31		$27,152	-	
SLIMP10_44	3.50	42	1	237	36	$81,345	$73,267	
SLIMP10_49	4.00	48	2	234	42	$99,696	$90,781	
SLIMP10_50	3.50	42	1	96		$38,957	-	
SLIMP10_51	3.50	42	1	15		$14,794	-	
SLIMP10_52	3.50	42	2	13		$24,565	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island North Central						**$407,000**	**$271,000**	**$136,000**
SLimp03l1	3.0	36	2	319		$105,049	-	
SLimp184	2.0	24	2	55	18	$28,064	$25,058	
SLimp230	2.0	24	1	159	18	$36,566	$29,350	
SLimp232	2.0	24	1	18	18	$12,416	$11,322	
SLimp276	3.5	42	1	78	36	$33,828	$30,924	
SLimp279	3.5	42	2	245	36	$93,976	$85,302	
SLimp430	3.5	42	1	108	36	$42,660	$38,794	
SLimp431	3.5	42	2	113	36	$54,634	$50,243	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island South						**$1,586,000**	**$1,165,000**	**$421,000**
SLimp449l1	2.75	33	2	399		$116,008	-	
SLimp178	3.00	36	1	28	30	$17,579	$15,800	
SLimp178_1	3.50	42	2	103	33	$51,582	$44,526	
SLimp178_2	3.00	36	1	21	24	$15,653	$12,978	
SLimp220	1.75	21	2	79	18	$30,218	$28,199	
SLimp223	1.75	21	1	81	18	$21,263	$19,368	
SLimp224	2.50	30	1	47	24	$19,843	$17,423	
SLimp376l1	4.00	48	2	233	42	$99,429	$90,542	
SLimp378l1	4.00	48	2	84	42	$49,583	$45,914	
SLimp379l1	4.00	48	1	142	42	$58,368	$53,018	
SLimp408l3	5.50	66	2	256	60	$128,660	$128,660	
SLimp415l2	5.50	66	1	235	60	$108,573	$108,573	
SLimp425l1	5.50	66	1	236	60	$108,931	$108,931	
SLimp426l1	5.50	66	1	55	60	$34,214	$34,214	
SLimp427l1	5.50	66	1	132	60	$66,129	$66,129	
SLimp99	3.00	36	2	476	24	$147,034	$100,315	
SLimpF06-511l1	3.00	36	1	91	30	$34,333	$29,370	
SLimpF06-512l1	3.00	36	1	487	30	$140,026	$114,976	
SLimpF06-612l1	2.75	33	1	234	30	$66,233	$60,226	
SLimpF06-615l1	3.00	36	1	198	33	$62,813	$57,545	
SLimpF06-619l1	3.00	36	1	84	30	$32,482	$27,870	
SLintake3l1	2.50	30	1	55		$21,494	-	
SLintakel1	2.50	30	1	673		$155,192	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Main Island Northside						**$375,000**	**$208,000**	**$167,000**
SLD381EC0A14C702DA	1.50	18	1	39		$14,026	-	
SLD381EC0A14C702F2	1.50	18	1	95		$21,162	-	
SLD381EC0A14C71C52	2.50	30	2	21		$23,817	-	
SLD381EC0A14C71C5B	2.50	30	1	324		$79,735	-	
SLimpD381EC0A14C70200	3.50	42	2	188	36	$76,902	$70,086	
SLimpD381EC0A14C702A3	2.75	32	2	134	27	$48,214	$44,901	
SLimpD381EC0A14C7173B	2.50	30	1	295	24	$73,497	$59,986	
SLimpE05-111I1	2.50	30	2	85	24	$37,712	$33,280	
Bay Farm Island North						**$232,000**	**$0**	**$232,000**
SL176	1.50	18	2	248		$49,866	-	
SL208	1.50	18	1	204		$35,108	-	
SL32	1.25	15	2	108		$29,766	-	
SL48	3.00	36	1	139		$47,043	-	
SL53	2.00	24	2	66		$29,958	-	
SL79	1.50	18	2	171		$39,931	-	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

Model Pipe ID	Recommended Diameter (ft)	Recommended Diameter (in)	Manholes	Length (feet)	SDMP Improvement Diameter (in)	NEW COST	OLD COST	Actual Increased Cost
Bay Farm Island Central						**$541,000**	**$154,000**	**$387,000**
SL105	1.50	18	1	94		$21,118	-	
SL166	1.75	21	1	37		$14,674	-	
SL167	1.75	21	2	44		$24,936	-	
SL176	1.50	18	2	72		$27,278	-	
SL241	1.50	18	2	323		$59,394	-	
SL242	1.50	18	1	216		$36,739	-	
SL290	1.25	15	1	56		$15,110	-	
SL291	1.50	18	2	94		$30,059	-	
SL420	1.50	18	2	219		$46,069	-	
SL620	1.25	15	2	275		$49,156	-	
SL621	1.25	15	1	69		$16,697	-	
SL623	1.25	15	1	67		$16,401	-	
SLimp106	2.00	24	1	303	18	$61,315	$47,825	
SLimp93	2.75	33	2	130	30	$50,886	$47,316	
SLimps53	2.00	24	2	151	18	$44,577	$37,384	
SLimps57	2.00	24	1	100	18	$26,463	$21,809	
Bay Farm Island South						**$50,000**	**$0**	**$50,000**
SL25	2.00	24	2	185		$50,424	$0	

Note: Above Costs do not include 40% Contingency Applied to Summary Table in Report Text

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CONSULTING CIVIL ENGINEERS

MEMORANDUM

TO:	Laurie Kozisek	DATE:	4-30-2015
FROM:	Dan Schaaf	JOB#:	APWD.11.14
SUBJECT:	DRAFT 55-Inch Sea Level Rise Study		

Introduction

Schaaf & Wheeler completed the City of Alameda (City) Storm Drain Master Plan (SDMP) in 2008. The study identified improvements to the storm drain network required to provide a 10 and 25-year level of service for existing hydrologic and hydraulic conditions. Following the completion of the SDMP, Schaaf & Wheeler also completed a more detailed analysis assessing Sea Level Rise (SLR) impacts on the City infrastructure; the completed analysis was added to the original SDMP as the Climate Change Impacts (CCI) Addendum 2009. The CCI Addendum addressed existing climate change research and understanding, as well as the inherent uncertainty of climate change impacts. It also included an analysis of the storm drain system performance with the SDMP CIP during a 10-year design storm with the inclusion of 18-inches of SLR. The Addendum concluded with the improvements required to mitigate the impact of 18-inches of SLR on the CIP storm drain network, but also noted that additional SLR would necessitate further improvements.

This study builds on the existing SDMP (2008) and CCI Addendum (2009) by providing an assessment of the impacts resulting from 55-inches of SLR on the City infrastructure, and by identifying improvements to prevent flooding.

Background

The City of Alameda is uniquely susceptible to SLR due to its location in the San Francisco Bay and low ground elevation. The elevation throughout the City varies but in many areas it is lower than the projected tides when accounting for 55-inches of SLR. The low lying areas along the islands' perimeter are somewhat susceptible to flooding through direct overland flow during high tides. Increased water levels at the outfalls contribute to interior flooding during storm events. The potential inundation extent due to these mechanisms is discussed herein.

Sea Level Rise Impacts

Sea level rise results in two distinct problems for low lying areas: overland inundation resulting directly from higher tides, and a decrease in storm drain capacity from higher tides at outfalls.

Overland Inundation

The CCI Addendum identified the 18-inch SLR extent of the 10, 25, and 100-yr tides. This study builds on that by assessing the potential inundation resulting from three different scenarios: 10-Year High Tide with 55-inch SLR, 25-Year High Tide with 55-inch SLR, and 100-Year High Tide with 55-inch SLR. These resulting inundation extents for Bay Farm Island (BFI) and the Main Island (MI) are illustrated in Figures 1 through 6. It's unlikely that the tides will completely inundate the areas shown due to the short peak duration. All elevations are on National Geodetic Vertical Datum (NGVD) of 1929. The figures are also included in Appendix A on 11x17-inch sheet to provide more detail.

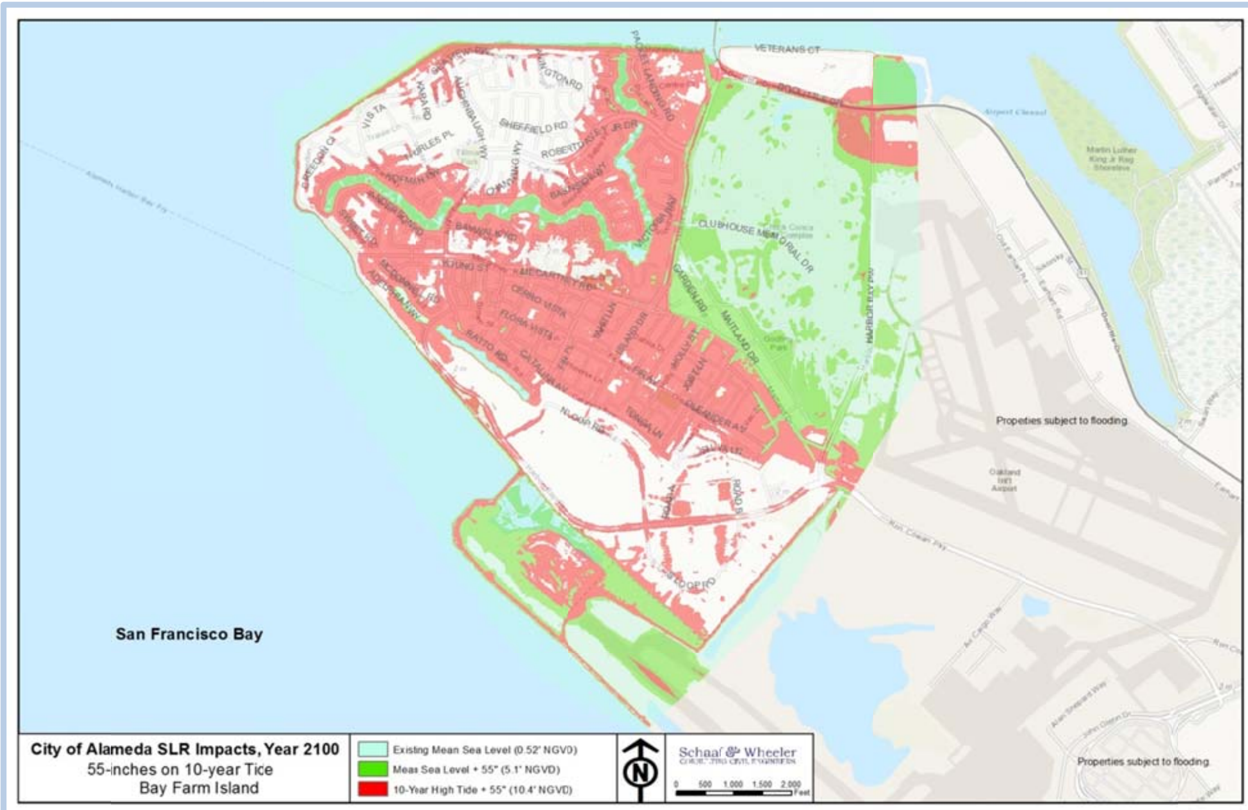

Figure 1: BFI 10-Yr Tide + 55" SLR Inundation

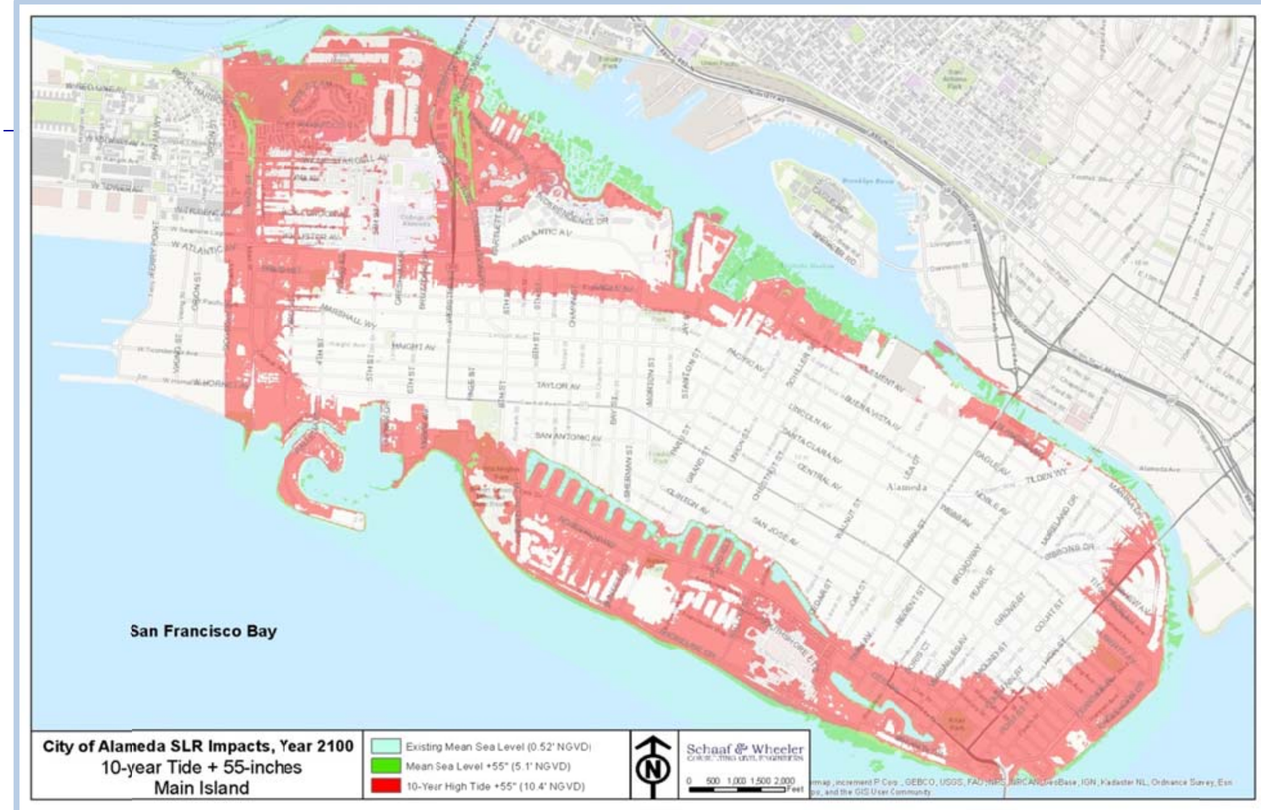

Figure 2: MI 10-Yr Tide + 55" SLR Inundation

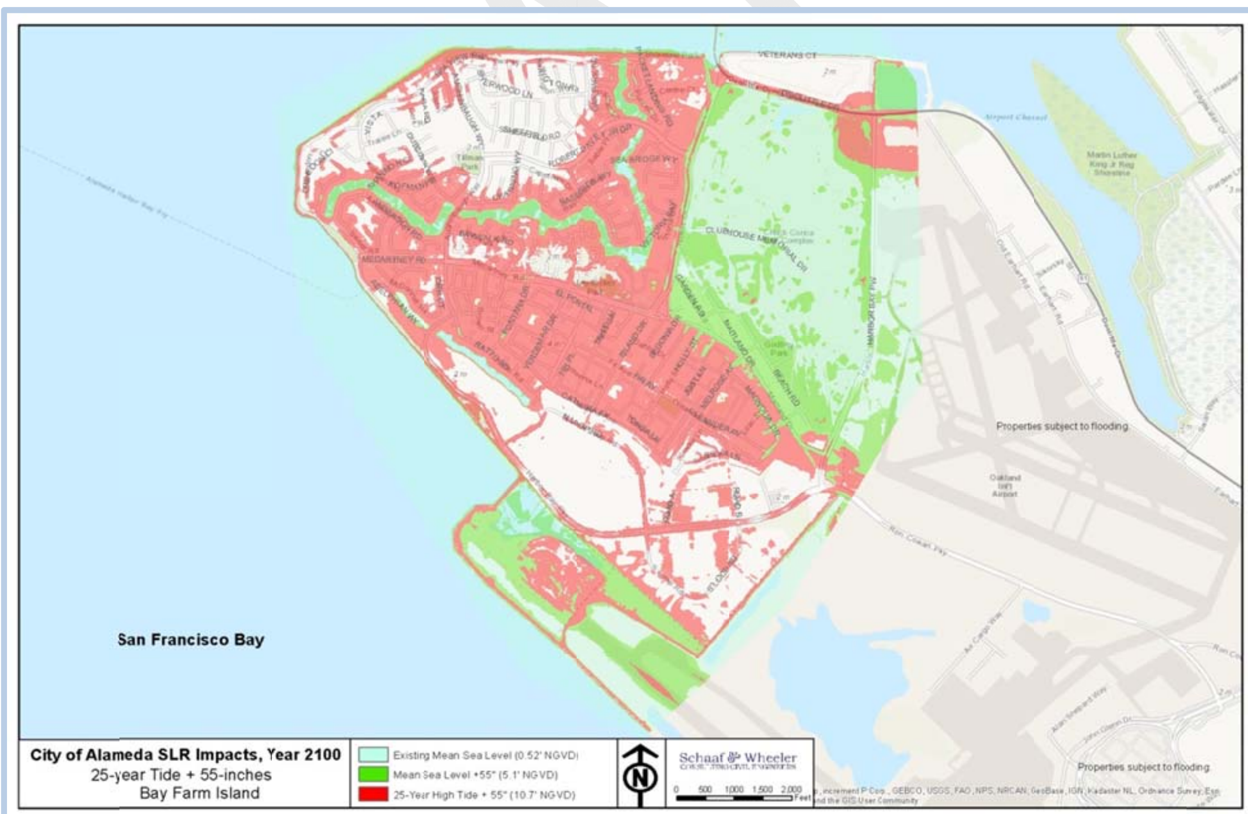

Figure 3: BFI 25-Yr Tide + 55" SLR Inundation

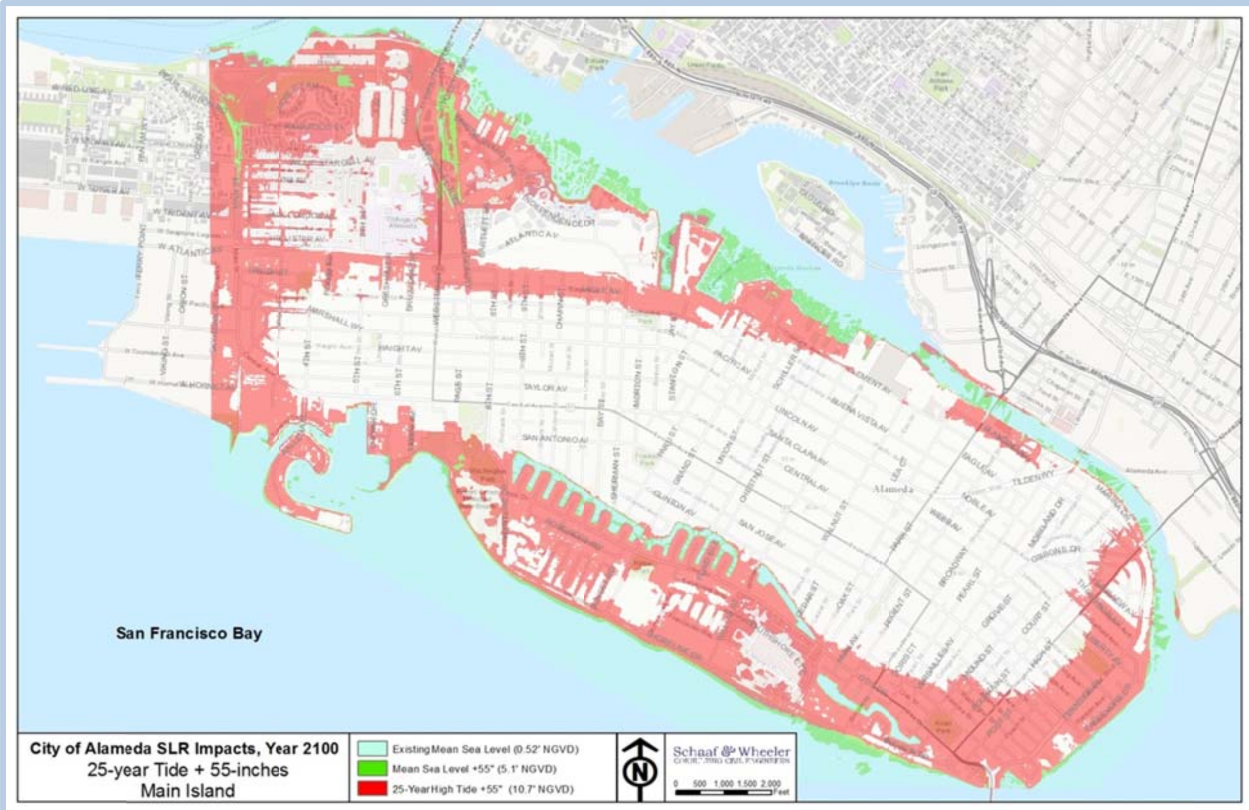

Figure 4: BFI 25-Yr Tide + 55" SLR Inundation

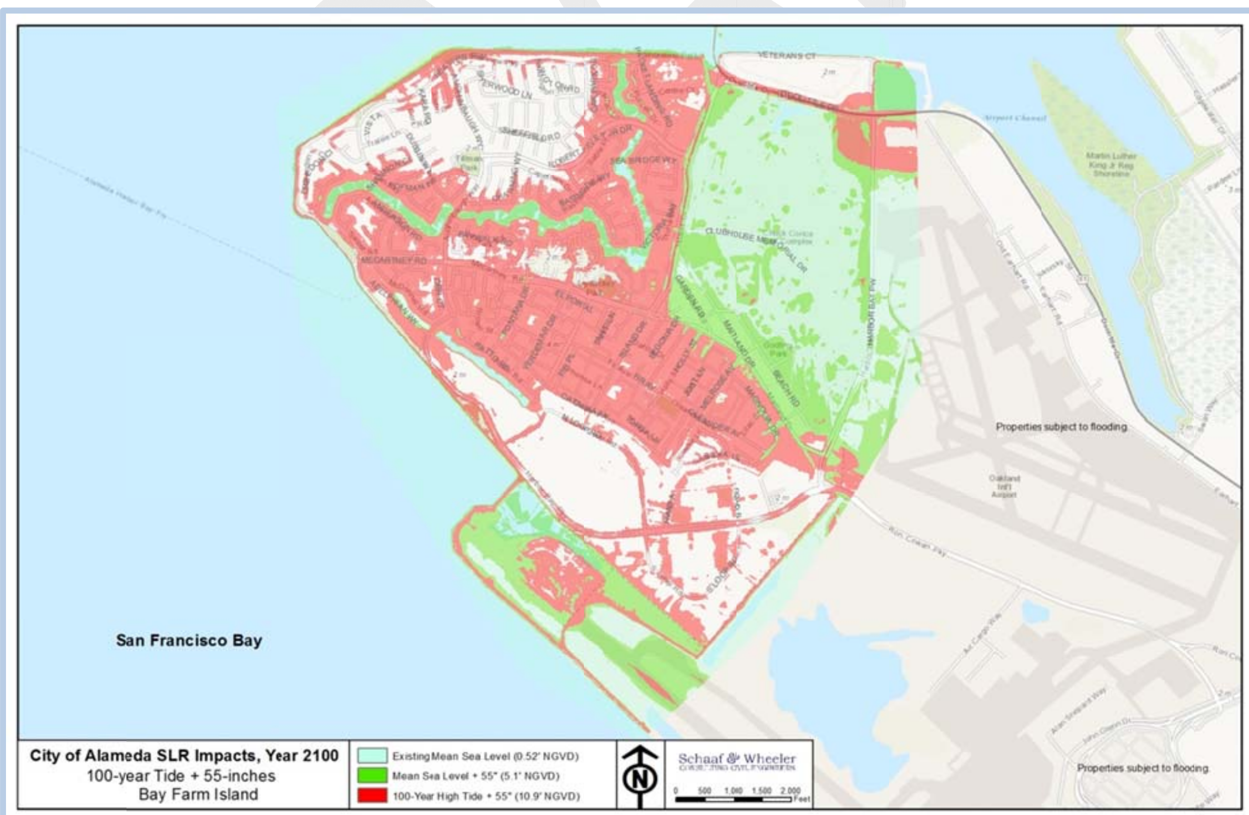

Figure 5: BFI 100-Yr Tide + 55" SLR Inundation

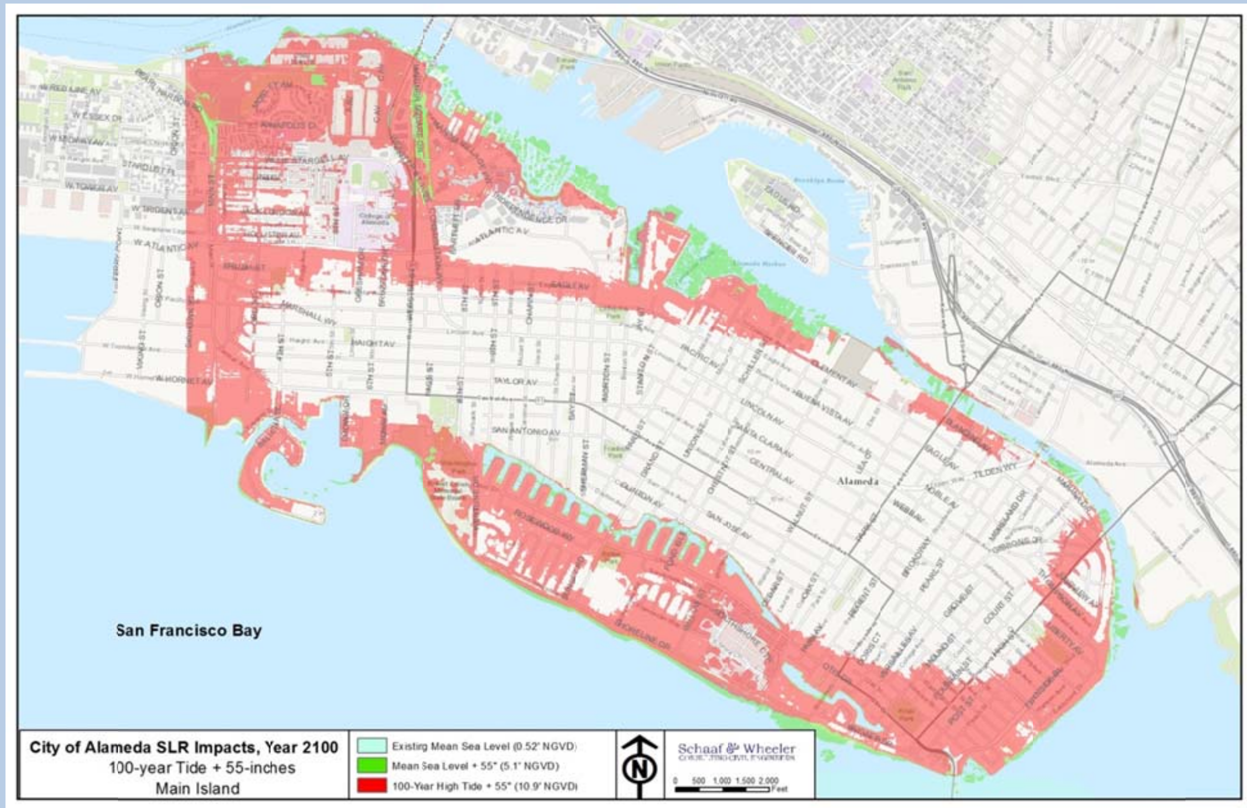

Figure 6: MI 100-Yr Tide + 55" SLR Inundation

Storm Drain System Capacity

Storm drain system capacity in tidally influenced areas is impacted by the elevation of receiving waters. Alameda's system capacity will decrease due to 55-inches of SLR and is anticipated to increase interior flooding. The impacts of SLR on system capacity were assessed using the updated MIKE URBAN (MU) 2012 software package from the Danish Hydraulic Institute (DHI). The modeled system included the improvements identified in the CIP from the original SDMP (2008) with additions/alterations made where information from new development was available. The CCI Addendum CIP improvements were not used as the base model; the majority of improvements recommended for the 18-inch SLR scenario are obsolete for the 55-inch SLR scenario. Storm drain improvements identified in this study supersede the CCI Improvements where they overlap. Improvements for the 55-inch SLR scenario that correlated to the same or similar improvement in the 18-inch scenario were identified as "High Priority".

Boundary Conditions

The change in sea level greatly influences the performance of the City's storm drain network due to the large number of existing gravity flow outfalls to the bay. There are over 200 outfalls in the existing storm drain system, 85 of which are 12-inches in diameter or larger and are included in the storm drain models. The City requested an assessment the future storm drain capacity for two tidal scenarios: the 10-year tide cycle with added 55-inches of SLR, and the 25-year tide cycle with 55-inches of SLR. A dynamic boundary condition was used to simulate tide elevations at the outfalls.

Identified Deficiencies

MIKE-Urban (MU) analysis of the systems for the 10-year design storm with 55-inches of SLR shows some flooding (HGL above the rim elevation of the node) occurring at 2,015 of the 3,828 nodes. The 25-

year results are similar with some flooding occurring at 2,228 of the 3,828 nodes; Table 1 summarizes the results for each scenario. The storm drain system fails to function properly due to the gravity outfall conditions; the resulting flood depths for the 10 and the 25-year storm events paired with their respective tide plus 55-inches of SLR are illustrated below in Figures 7-10. The figures are also included in Appendix A on larger 11"x17" sheets for more detail.

Table 1: Storm Drain Flooding Results with 55-inches of SLR

Sea Level Rise Network Results		
Flood Depth	**10-Year**	**25-Year**
0-6"	485	433
6"-1'	445	450
>1'	1,085	1,335
Total	**2,015**	**2,218**

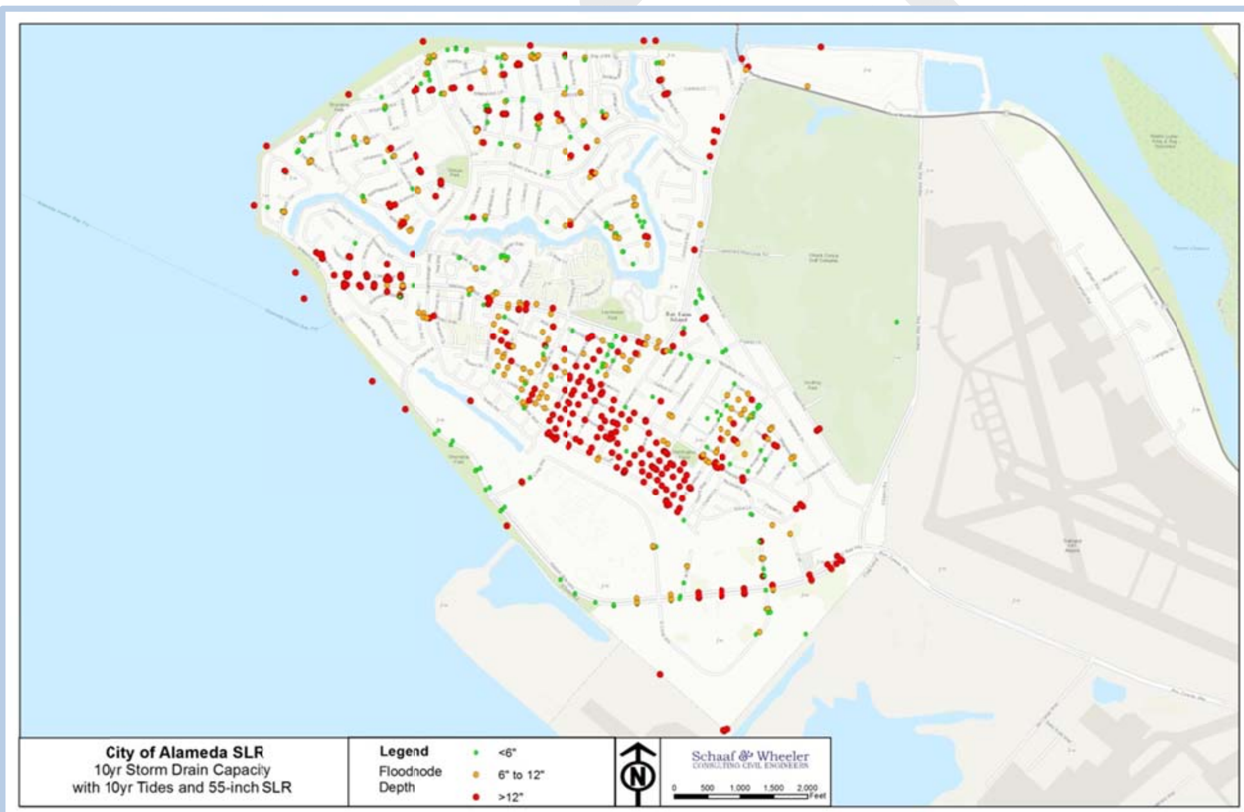

Figure 7: BFI 10-Year Existing System CIP Flood Depth w/55-inch SLR

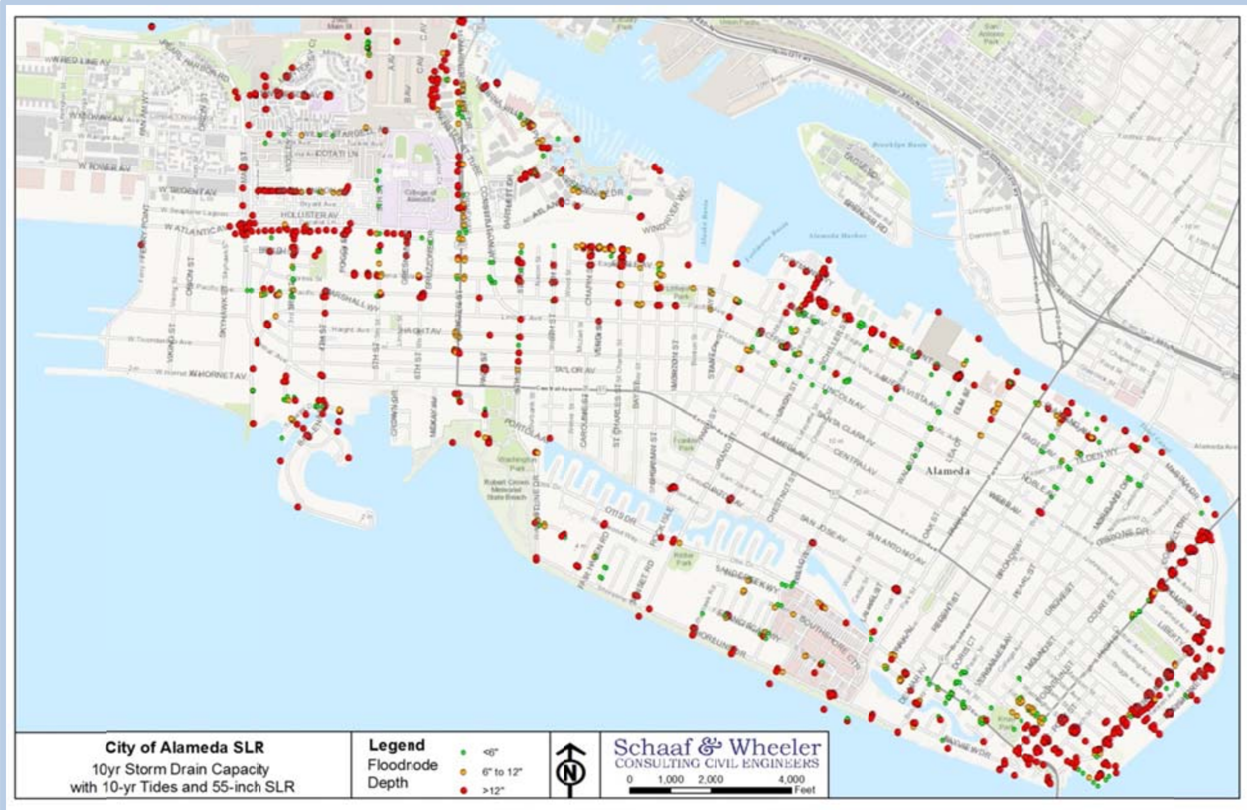

Figure 8: MI 10-Year Existing System CIP Flood Depth w/55-inch SLR

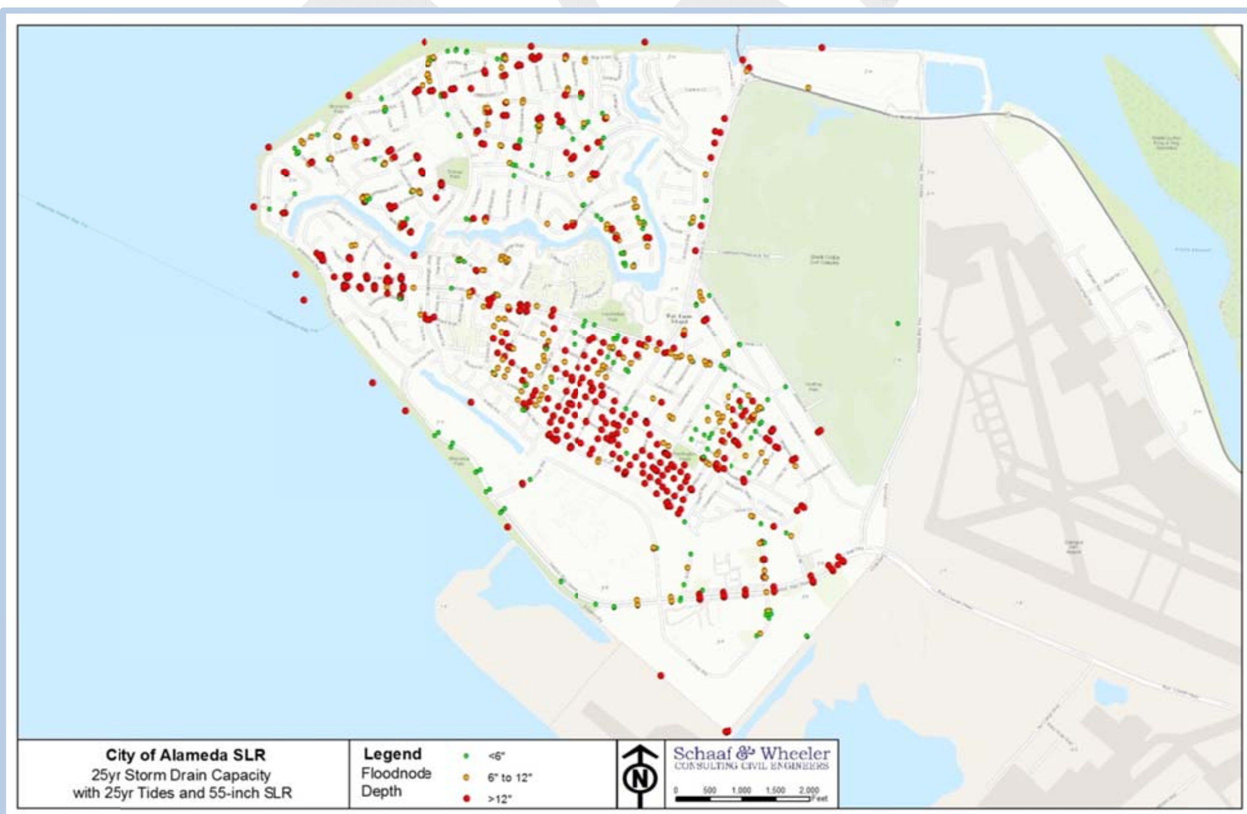

Figure 9: BFI 25-Year Existing System CIP Flood Depth w/55-inch SLR

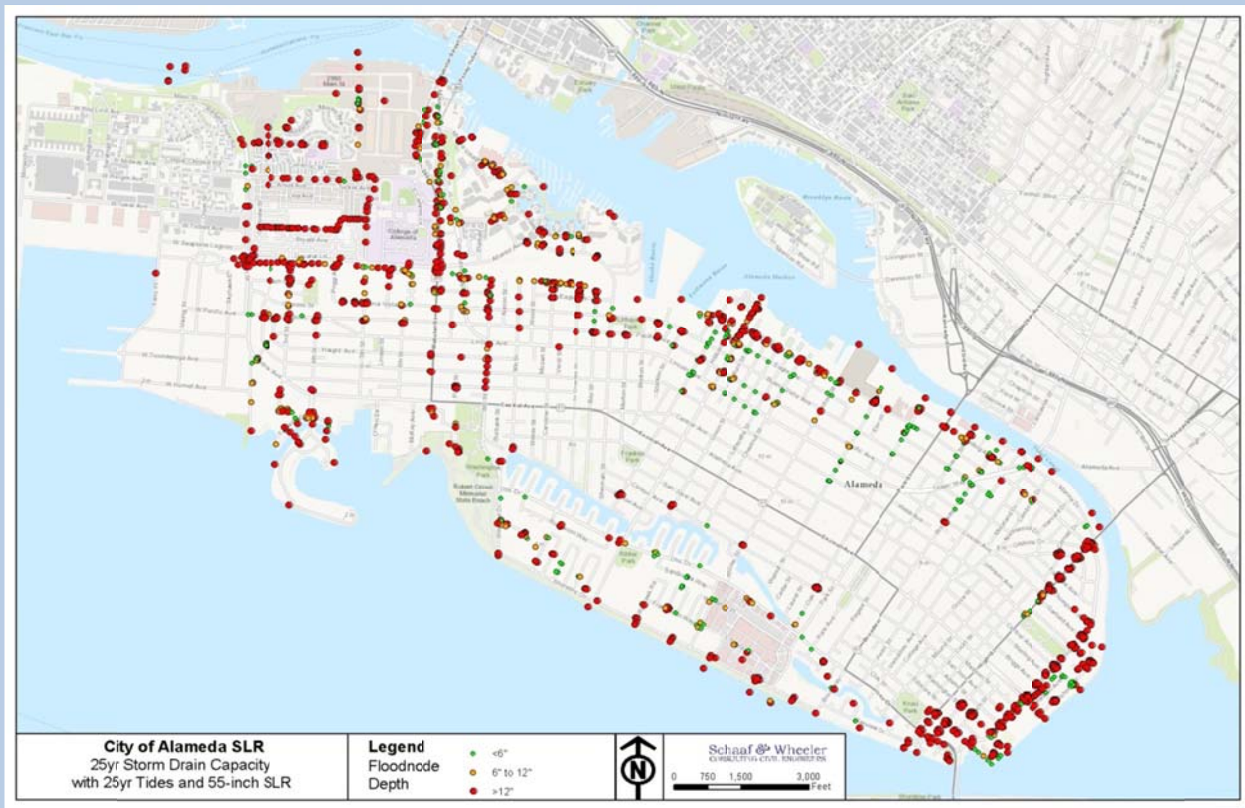

Figure 10: MI 25-Year Existing System CIP Flood Depth w/55-inch SLR

Capital Improvement Program

Schaaf & Wheeler has developed a Capital Improvement Program (CIP) to mitigate the projected 55-inches of SLR. Improvements have been split into two types: overland improvements required to prevent tidal inundation, such as levees and sea walls; and storm drain network improvements required to maintain system capacity, such as pipe upsizing and new pump stations. For the purposes of this study, it is assumed all outfalls have been equipped with tide gates to prevent surcharging in low lying areas.

Overland Improvement Alternatives

The City of Alameda's ground elevations are high enough tp prevent the need for levees or floodwalls with current mean sea level. SLR projections indicate coastline flood protection will be required to prevent overland inundation during high tides. Three basic alternatives are described in this study: earthen levees, lightweight fill levees, and floodwalls. It is also recommended that as much work as possible be conducted on the landward side of the existing roadways and bike paths to limit shoreline disturbance. This approach will not eliminate the need for regulatory approval, but may reduce permitting delays and compensatory mitigation requirements.

Earthen Levees

Earthen levees are commonly used as flood protection along creeks, rivers, and coastlines. Typical constraints of earthen levees include large width due to having maximum side slopes of 3:1 (H:V) + 5' minimum crest width, construction constraints when building on Bay Mud, difficulty with increasing height once built, and the need for slope protection. Figure 11 below illustrates a typical earthen levee cross-section. In Alameda, the main concern is settlement issues due to the possibility of Bay Mud. If new earthen fill is placed in an area underlain by Bay Mud, 6 to 8 inches of settlement per foot of new fill placed can be expected. It is typically recommended to place the new fill in one-to two-foot intervals with 3:1 (h:v) or flatter bank slopes since rapid loading from fill placement may cause instability. For new fills placed on top of existing roadways, settlement will generally be in the range of 3 to 5 inches per foot of new fill.

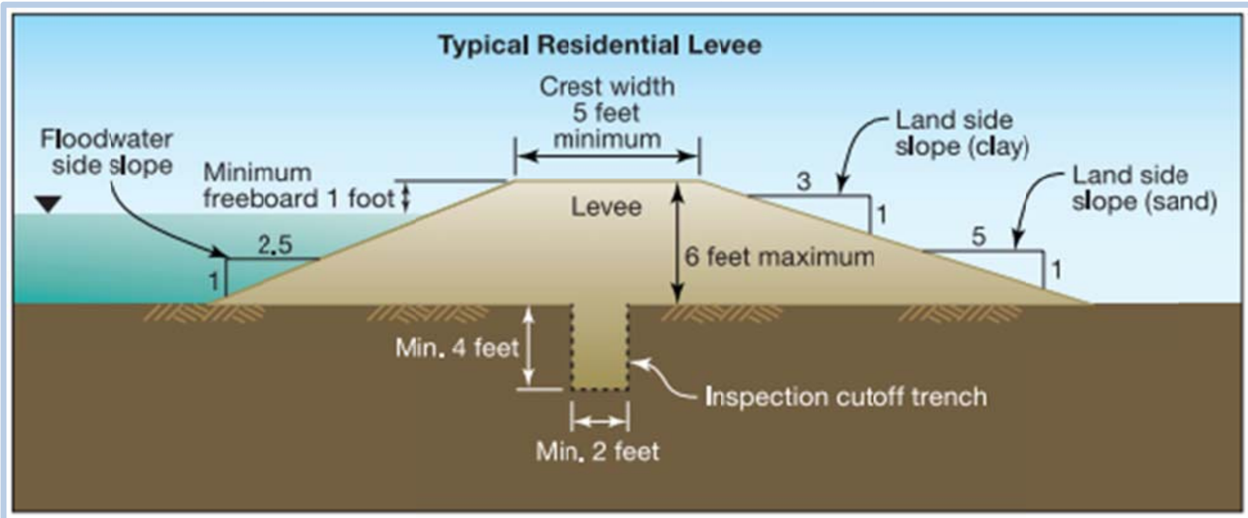

Figure 11. Typical Earthen Levee Cross Section (ref. FEMA)

Levee improvement profiles included with this report do not show a settlement allowance. The actual settlement allowance required is a function of levee location, the underlying stratigraphy and Bay Mud thickness. These parameters need to be identified with a thorough program of subsurface exploration, laboratory testing, and geotechnical engineering during more advanced planning and design phases. For preliminary cost estimating purposes it is assumed that the volume of earthen fill to be placed must be doubled to ultimately result in the levee profiles required for FEMA levee accreditation. This assumption reflects a settlement allowance of 6 inches for every foot of fill added as well as a remobilization cost since only so much fill can be placed at one time while maintaining soil stability.

Lightweight Levee Fill

The construction of levee improvements using lightweight fill are similar to that of an earthen levee. Lightweight fill consists of material from volcanic sources that has a unit weight on the order of 70 to 90 pounds per cubic foot, compared to a saturated unit weight of 125 pounds per cubic foot for conventional levee fill material. The use of lightweight fill material may reduce the total settlement of the levee to one-third or one-half that of a conventional raised levee.

Lightweight fill is more expensive due to the cost of the fill material itself and the ancillary seepage cutoff wall required (lightweight fill, even when blended with more conventional fill, is relatively porous), but may be cost effective due to the smaller volume required and faster completion of construction. San Mateo recently selected this alternative for their Bayfront Levee Improvement Project in 2011.

Structural Floodwalls

Floodwalls can be placed on the Bay side or the landward side of the trails or bike paths. If floodwalls are placed on the landward side, access ramps over the wall or closure devices are required for pedestrians and bikers to access the trail. However, if floodwalls are placed on the Bay side, permitting may be more complex and require additional coordination with agencies such as BCDC and the USACE. In addition, a floodwall on the Bay side may detract from the trails appeal as it may limit the view into the Bay.

Figure 12 shows a conceptual cross section of a typical floodwall that could be placed on the Bay or land side of the existing trail or roadway to meet freeboard requirements for FEMA accreditation. Floodwalls have a more narrow footprint than traditional levees, which lessen the land acquisition impact. Floodwalls may be easier to increase in height in the future (without a commensurate increase in footprint) and may be considered more readily adaptable to sea level rise than an earthen levee. While floodwalls will not experience as much settlement as earthen levees, a wall on a shallow spread footing may experience three to six inches of settlement that should be added to the height of the wall to compensate.

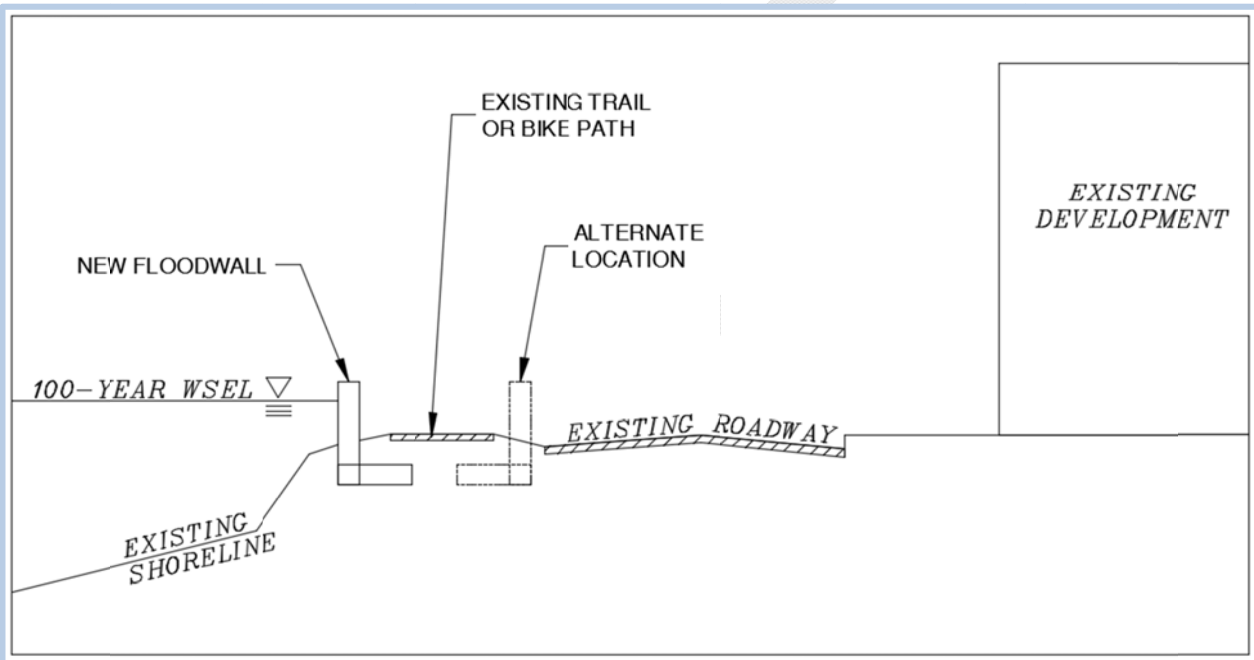

Figure 12: Typical Floodwall Cross Section (Not to Scale)

Recommended Improvements

Schaaf & Wheeler has developed improvement projects recommended to help the City maintain flood protection from coastal flooding resulting from SLR. These improvements identify the optimal locations for seawalls and levees, although some locations and/or type of selected improvement may be revised in the future as more data becomes available and more is known about the desires of impacted property owners. It should also be noted that this study does not include Alameda Point. Some projects may be altered or removed entirely based on future development in Alameda Point; for the purposes of this study it is not assumed to have flood protection.

Coastal flood protection improvements are relatively straight forward for Bay Farm Island, it is encompassed by a bike trail along the west side and by Harbor Bay Parkway along the east. Recommended improvements include elevating a 3-mile stretch of the bike path, and a two-mile stretch of Harbor Bay Parkway; the BFI improvements are illustrated in Figure 13. A portion of Harbor Bay

Parkway runs along the Oakland Airport; it is recommended that the City coordinate with the airport to provide flood protection for the east side of BFI. Similarly, bike paths, roadways, and floodwalls would be raised around the Main Island to provide flood protection. Some properties along the shore may require unique solutions for protection. This study assumes the flood protection is installed within the existing island limits; the identified improvements for the MI are illustrated in Figure 14.

The actual levee and seawall height will vary; existing ground elevations were determined by averaging existing surface elevation along the levee and seawall paths. FEMA California Coast Analysis and Mapping Project (CCAMP) Maps were used as a basis for determining future 100-year Water Surface Elevations (WSELs) around Alameda. The existing elevations vary between elevation 10' on NAVD88 (7.3' NGVD) datum to elevation 14' NAVD (12.3 NGVD). 55-inches of SLR plus an additional foot for freeboard were added to the average FEMA WSELs along to determine the required height of each levee or seawall. Table 2 below identifies the average height required for each levee and seawall to provide flood protection with 55-inches of SLR.

Table 2: Levee and Seawall

Improvement	Length (ft.)	Average FEMA (NGVD ft.)	Average Existing (NGVD ft.)	FEMA+55-inch+1' Freeboard (NGVD ft.)	Required Levee/Seawall height (ft.)
BFI East Levee	12,500	8.3	6	13.9	7.9
BFI West Levee	15,000	10.1	9.5	15.6	6.1
Clement Seawall	14,100	7.3	9.4	12.9	3.5
Fernside Levee	13,000	8.3	9.7	13.9	4.2
Hornet Levee	3,300	8.3	8	13.9	5.9
Main Street Levee	8,700	7.3	7.4	12.9	5.5
Northside Seawall	14,300	7.3	8.1	12.9	4.8
Shoreline Levee	13,500	9.8	10.1	15.4	5.3
Tideway Levee	1,000	9.3	8.9	14.9	6

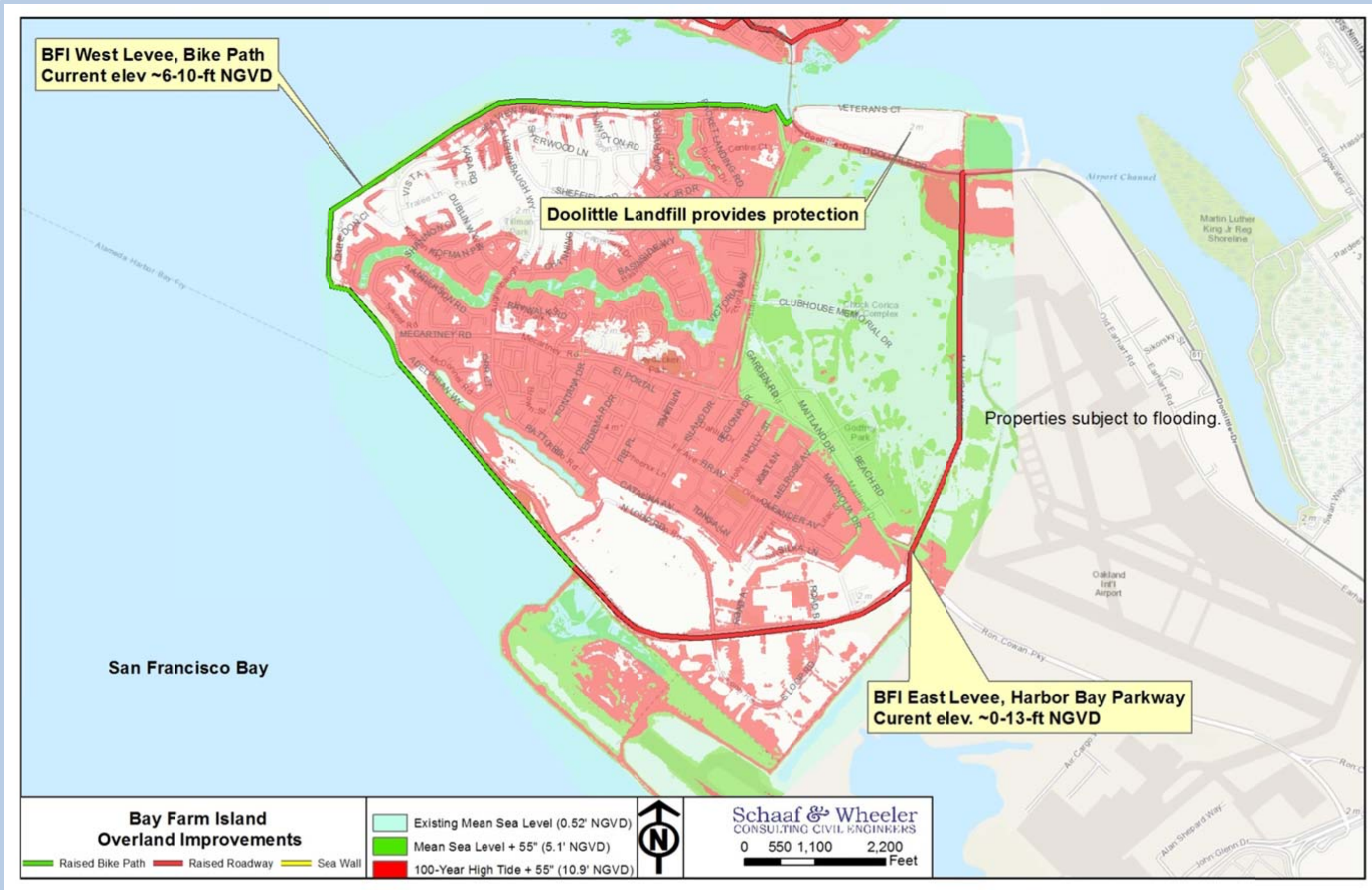

Figure 13: BFI Overland Improvements

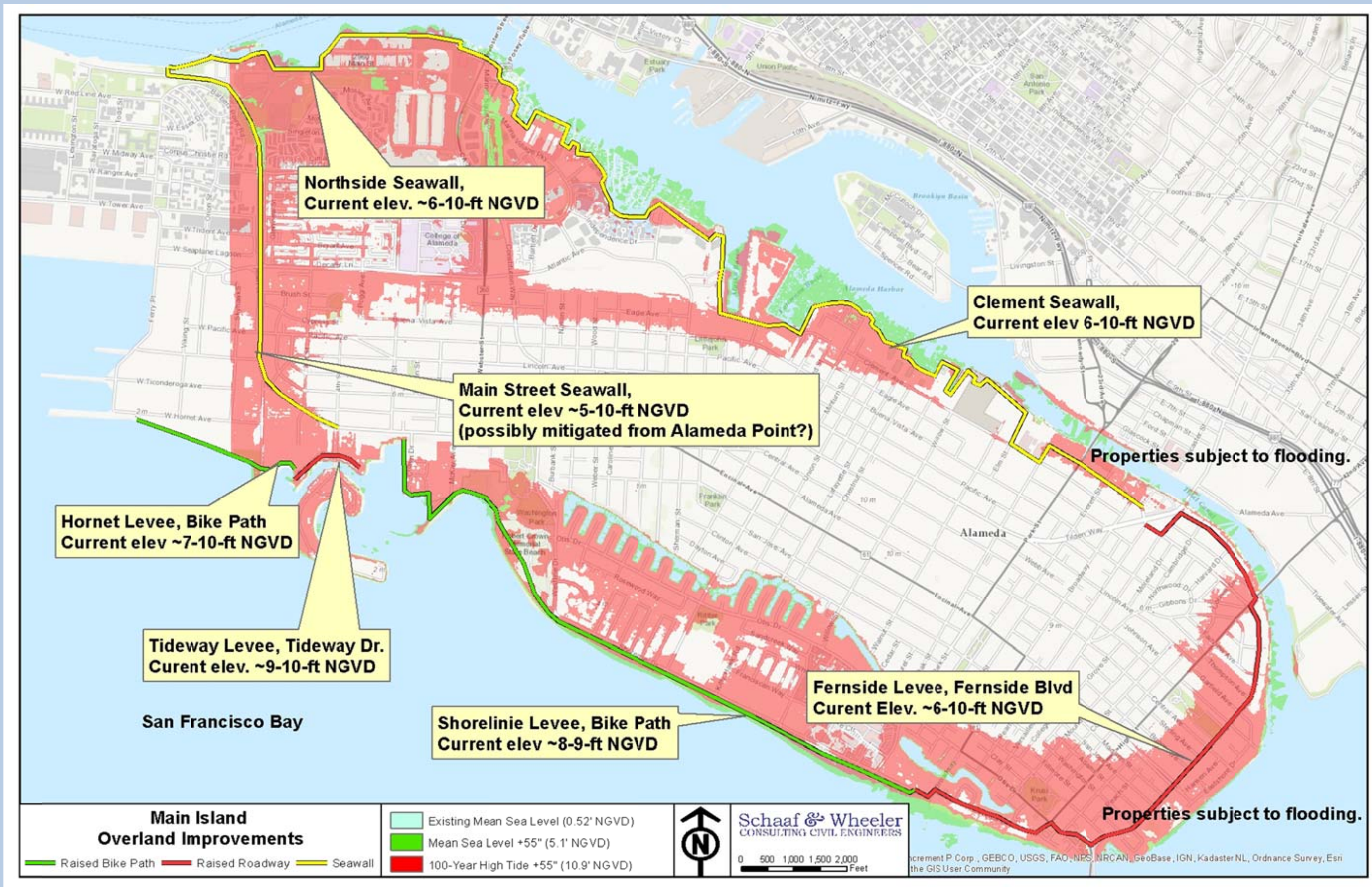

Figure 14: MI Overland Improvements

Adaptive Management Techniques for Sea Level Rise

If feasible, levee improvement planning and design should consider providing additional freeboard for future sea level rise projections. This will entail increasing the base width of levees and floodwall footings. In general, it is easier to raise the height of floodwalls than levees to provide additional freeboard. However, if earthen levees are constructed, it may be possible to build a short wall to meet sea level estimates. The anticipated lifespan of inert substances such as earth and concrete is approximately 100 years, so 100-year projected sea level rise should be planned for if feasible. Figure 15 illustrates how the developers of Treasure Island are building levees that are adaptable to future sea level rise.

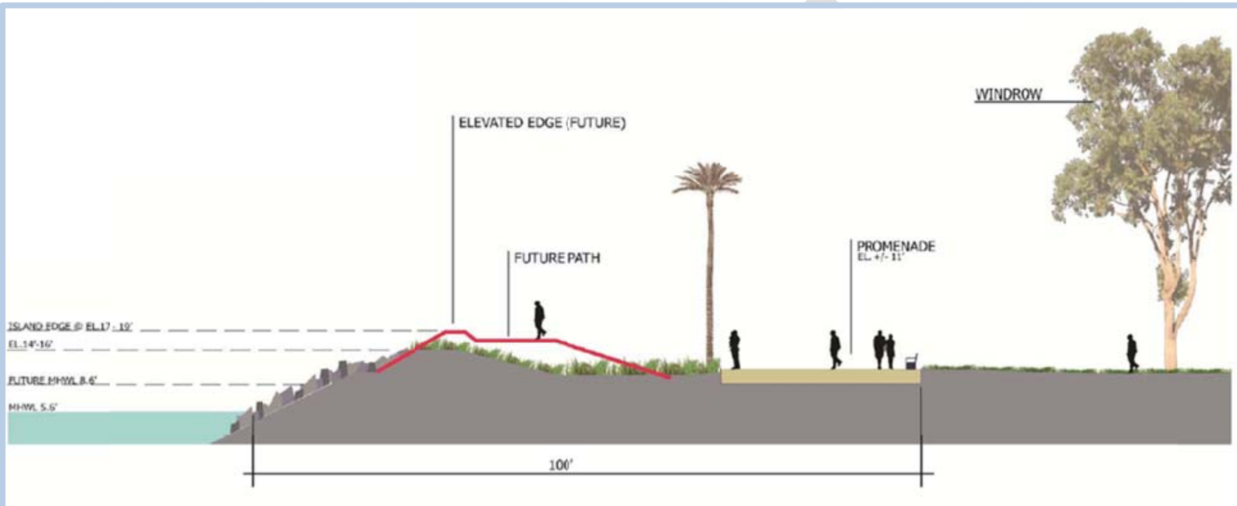

Figure 15: Adaptive Design on Treasure Island for Sea Level Rise (ref. Moffatt & Nichol)

Aesthetics

There are several trails, roadways, and private developments that run along the Alameda's shoreline. Part of the appeal of the City of Alameda is the ready access to its waterfront and bay views. Raising levees or constructing a structural floodwall may impact the ease of access and the visibility of the Bay from the shorefront and surrounding area. Providing flood protection and meeting levee accreditation

Figure 16: Glass Floodwall (ref. Flood Control International)

requirements are considered superior to aesthetic considerations; however, there are options with more aesthetically pleasing solutions that would help maintain the existing appeal of the waterfront. Glass and Plexiglas style floodwalls have already been implemented (see Figure 16) in many places around the globe as innovative solutions to combat water levels while maintaining waterfront views and could potentially be used in some areas of Alameda for flood protection.

Apart from seawalls that are permanent fixtures, there are also more passive forms of flood protection that are only raised when necessary. One passive product is the Floodbreak FreeView Levee Topper that rises as water levels rise. This type of barrier is FEMA approved, and would provide protection without obstructing views of the bay while the water levels are low, see Figure 17 below.

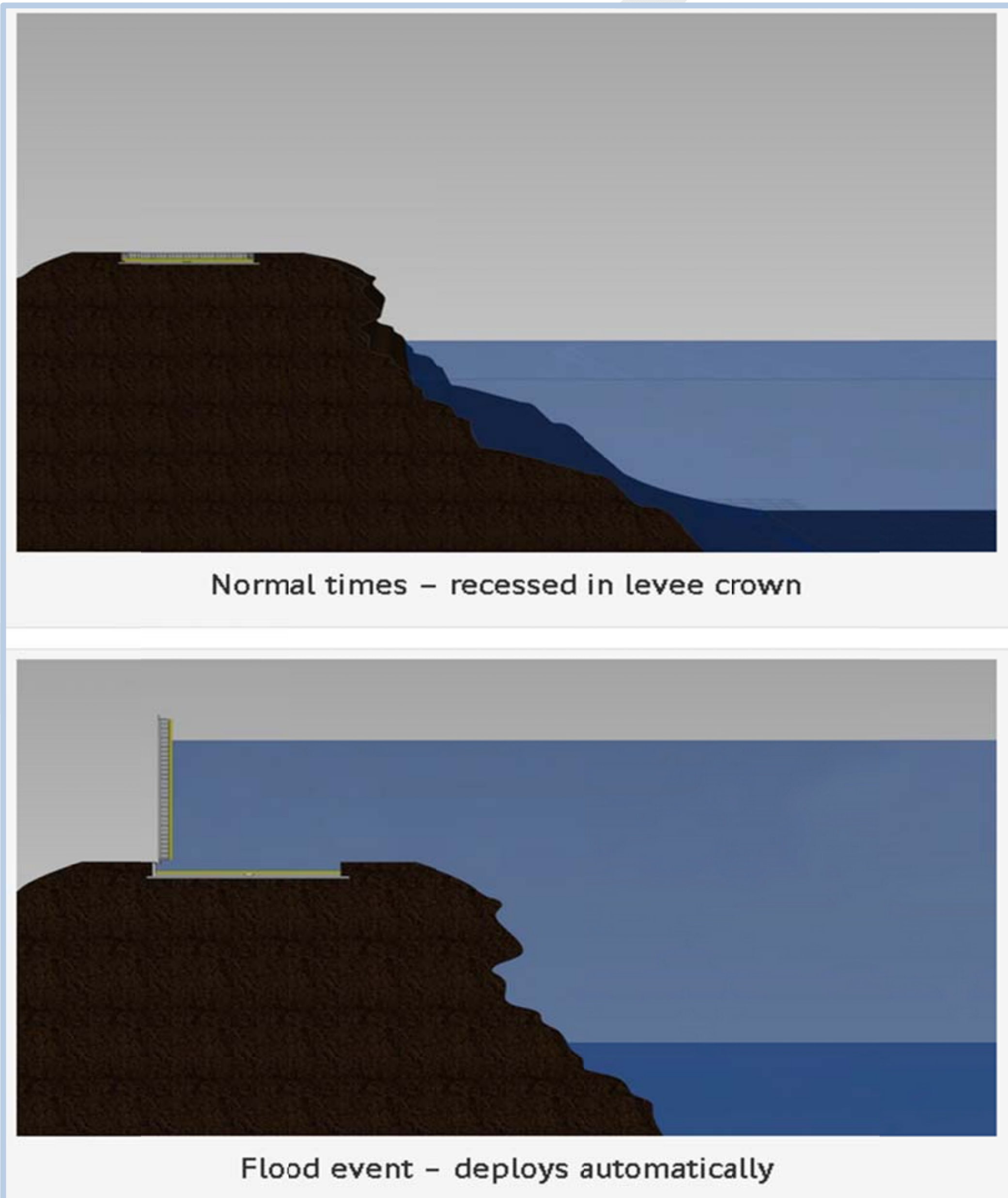

Figure 17: FreeView Levee Topper (ref. FloodBreak)

Permitting Challenges

The San Francisco Bay area is considered a sensitive habitat and may be home to endangered species. The California Department of Fish and Game BIOS website lists multiple endangered species known to be in the vicinity of Alameda. Prior to beginning construction activity, the California Environmental Quality Act (CEQA) requires documentation to identify sensitive habitats and species that might be impacted by construction and compensatory mitigation measures that render those impacts less than significant. Permits will also need to be obtained from entities such as the United States Army Corps of Engineers (USACE) and Bay Area Conservation and Development Commission (BCDC). The Joint Aquatic Resource Permit Application (JARPA) is used to apply for regulatory approval of projects that take place along the San Francisco Bay and the coastline. The San Francisco BCDC will be keenly interested in beach and trail access during construction. The following is an indication of the types of agencies that will require permits to improve the levee system:

Local

- San Francisco Bay Conservation and Development Commission (BCDC)
- San Francisco Estuary Partnership
- Regional Water Quality Control Board (RWQCB)

State

- California Environmental Quality Act (CEQA)
- California Department of Fish and Wildlife (CDFW)

Federal

- Corps of Engineers Section 10 permit (USACE)
- Corps of Engineers Section 404 permit (USACE)
- United States Fish and Wildlife (USFWS)
- National Marine Fisheries (NMFS)

Beach and Trail Access during Construction

Access to Robert W. Crown Memorial State Beach and associated bike paths and trails will be limited for the duration of the levee construction; a comprehensive detour plan will need to be addressed in the CEQA document and JARPA permit.

Storm Drain Improvements

CIPs described herein are required for the storm drain system to convey the 10 and 25-year storm runoff flows to the bay in the event of 55-inches of sea level rise. This study identifies necessary improvements to protect both public and private infrastructure from inundation. These improvements are based on the assumption that the CIPs identified in the SDMP are installed and operational prior to the implementation of these projects. It is also assumed that all outfalls have been equipped with flap gates or have been plugged prior to the 55-inch SLR impact.

Prioritized Improvements

Projects are prioritized as High, Moderate, or Low priority. The priority rating is assigned based on the level of benefit provided by a project and provides a suggested order of completion. It is possible that projects assigned as Moderate, or Low priority may be completed prior to High priority projects where an existing system is failing due to unforeseen conditions or if there is an opportunity to complete a CIP project concurrently with other construction.

Two projects are identified as high priority for both the 10 and 25-year scenarios. The High Street project is identified in the CCI Addendum CIP and is also required for the 55-inch SLR scenario. The Southshore Lagoon Pump is the other high priority project. The Southshore lagoon system was assessed as part of a weir operations study by Schaaf & Wheeler. It is recognized that the only solution to prevent property damage from high water levels in the lagoons is to install a pump to lower lagoon levels.

There are 24 Moderate priority improvements recommended to provide a 10-year level of service with 55-inches of SLR, and 33 Moderate priority improvements recommended to provide a 25-year level of service. Moderate priority projects reduce flooding significantly at multiple locations during the 55-inch SLR scenario. Apart from being necessary to provide an adequate level of service, the projects are also large in scale and will require a significant level of effort to be completed. All required pump station improvements are included as Moderate priority projects; large trunk lines that are downstream of other improvement projects are also ranked as moderate priority projects.

Low priority improvements are recommended at 17 locations for the 10-year level of service and at 15 locations to provide a 25-year level of service. These projects do not need to be completed for any other improvement projects to function properly. In general, low priority projects are of a much smaller scale and would be easier to construct.

A maintenance schedule is also recommended for the pump stations as well as all flap gated outlets. Performing regular maintenance on pumps stations and removing organic and inorganic debris from flap gates ensures proper functionality during regular operations and will be a key factor in preventing floods during large storm events.

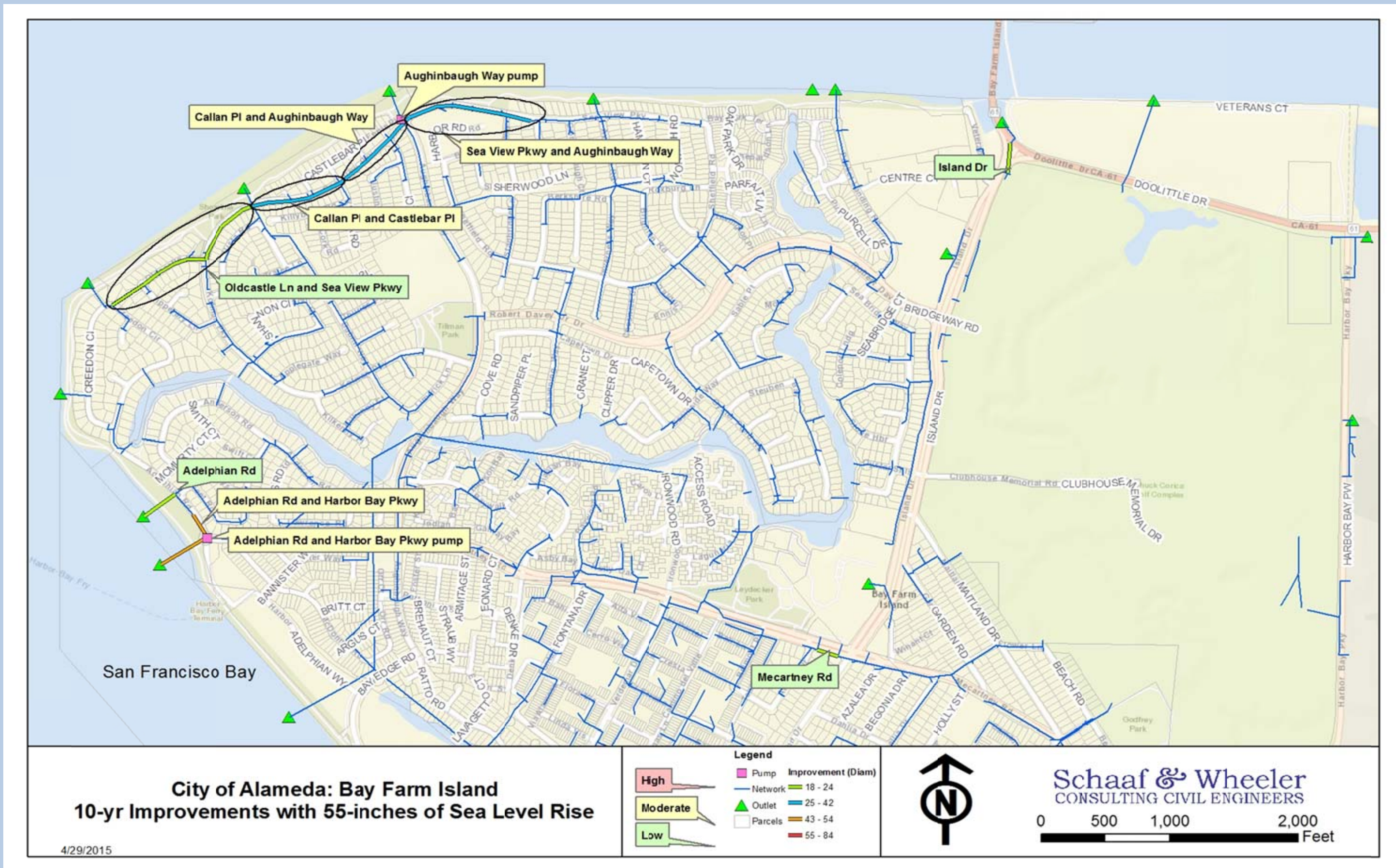

Figure 18: BFI North 10-Yr SLR Improvements

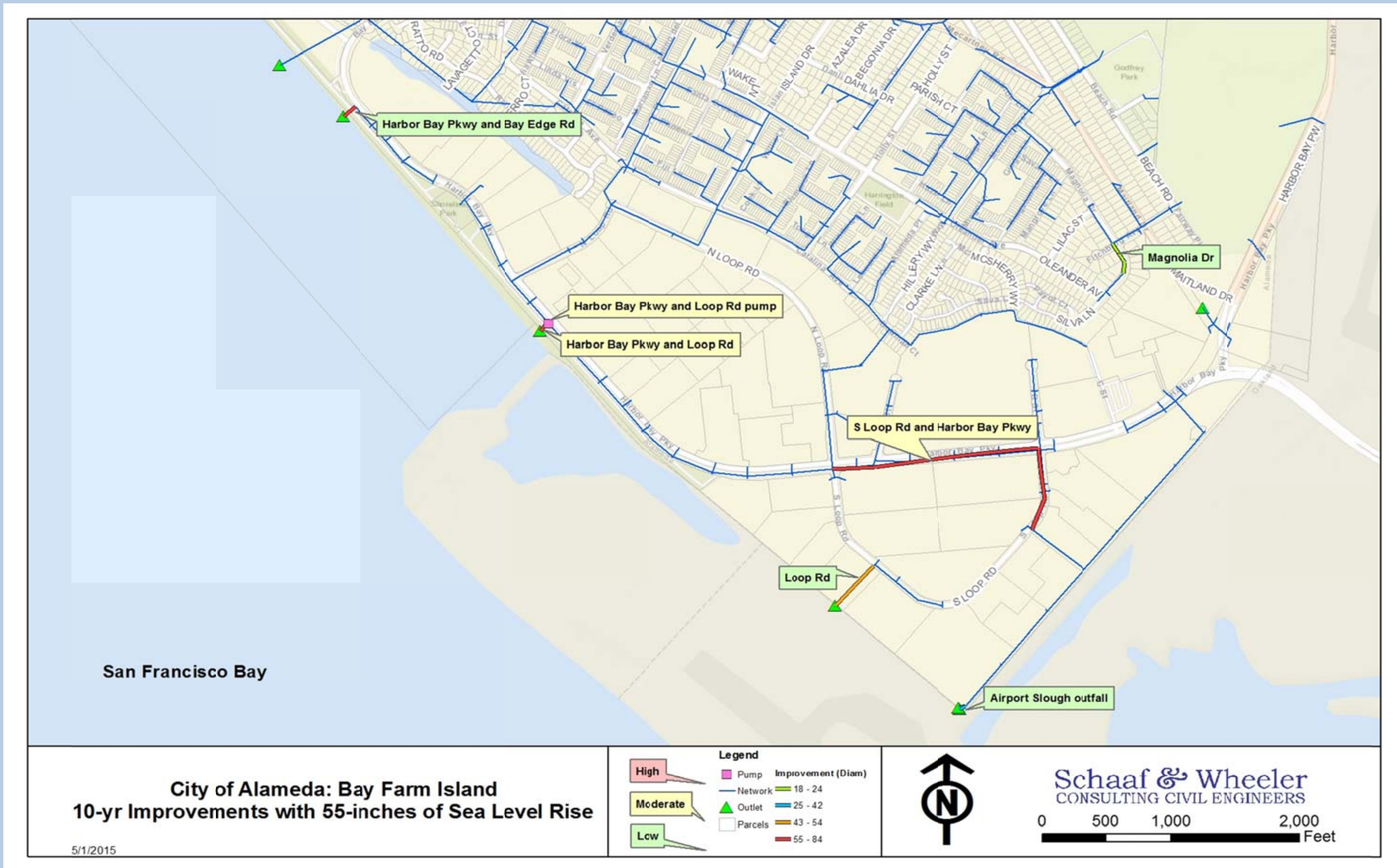

Figure 19: BFI South 10-Yr SLR Improvements

Figure 20: MI North 10-Yr SLR Improvements

Figure 21: MI South 10-Yr SLR Improvements

Figure 22: BFI North 25-Yr SLR Improvements

Figure 23: BFI South 25-Yr SLR Improvements

Figure 24: MI North 25-Yr SLR Improvements

Figure 25: MI South 25-Yr SLR Improvements

Cost Estimates

Cost estimates have been prepared for the overland improvements required to prevent tidal flooding as well as the necessary storm drain improvements to maintain the 10 and 25-year level of service. All costs have been calculated using the April 2015 ENR Index of San Francisco.

Overland Improvements

The cost associated with building levees and floodwalls is dynamic and varies based on the total height added. The costs were estimated using a method that adjusts for the added volumes as height increases. Table 3 below illustrates the variation in unit cost for each type of improvement. Actual costs are expected to vary based on site specific characteristics and the level of permitting effort required for individual alternatives.

Table 3: Levee and Seawall Costs

Height Increase	(Cost/linear ft)		
	Conventional Earthen Levee	Lightweight Fill Earthen Levee	Structural Floodwall
1 foot	$ 700	$ 1,200	$ 600
2 feet	$ 1,200	$ 1,700	$ 1,100
3 feet	$ 1,800	$ 2,100	$ 1,500
4 feet	$ 2,300	$ 2,500	$ 2,100
5 feet	$ 2,700	$ 2,900	$ 2,700

These costs do not include the 25% increase for design, administration, and contingency.

Levee Improvement Costs

Costs for Levee Improvements are based on the total fill required to achieve the design height. For cost estimating, the levees along bike paths are assumed to have a top width of 12 ft. which includes a 10 ft wide path with 1 ft shoulder on each side. Levees under roadways are assumed to be 44 ft wide with two 12 ft lanes and a 10 ft shoulder on either side. It is assumed that earthen levees are built on Bay Mud and would therefore require larger volumes than lightweight fill levees. Lightweight fill typically consists of permeable volcanic rock, requiring the installation of a cut off wall to prevent water seepage through the levee. Lightweight fill levees settle less than conventional earthen levees and therefore require less fill material; however, they are more costly in some cases due to the addition of the cut off wall

Floodwall Improvement Costs

The cost estimate for structural floodwalls is calculated assuming the use of a shallow spread footing. Since the exact soil conditions are not known, structural floodwall costs may increase if soil conditions require the use of deep foundations.

Levee and Seawall Project Costs

Each projects has been assumed to be a certain type. A summary of the project cost along with height, and total length is included in Table 4 below. It should be noted that floodwalls were assumed to be built only where levees are not feasible.

Table 4: Levee and Seawall Improvement Cost Summary

Project ID	Height	Length (ft)	Earthen Levee Cost
Shoreline Levee	5.3	13,500	$ 58,551,000
Fernside Levee	4.2	13,000	$ 58,551,000
Main Street Levee	5.5	8,700	$ 51,253,000
Hornet Levee	6	3,300	$ 8,383,000
Tideway Levee	6	1,000	$ 6,442,000
BFI West Levee	6.1	15,000	$ 39,539,000
BFI East Levee	8	12,500	$ 107,810,000
Northside Seawall	9.8	14,300	$ 26,793,000
Clement Seawall	8.5	14,100	$ 17,282,000
Total		**95,400**	**$374,604,000**

Note: These costs include a 60% increase for design, administration, and contingency included in all other tables.

Storm Drain Improvements

Costs have been estimated using information from other projects, cost estimating guides (*2015 RSMeans Heavy Construction Cost Data*), and engineering judgment. The cost per linear foot of improvement used for the pipe cost estimates are given in Table 5, and assume replacement pipe installed using the open trench method. Costs are likely to vary greatly depending on site specific circumstances; in some cases it may be more practical to use trenchless methods or a parallel pipe for construction.

As per our estimates, connection (manhole or catch basin) replacement cost estimates ranged from $11,770 to $22,000 depending on connecting pipe diameters. Pipe costs include open trenching in the roadway up to ten feet in depth. New outfall costs are estimated to be $40,000 per new outfall. It should be noted that wide variations in actual outfall costs are expected due to location of outfall, whether energy dissipation is required, environmental concerns, etc. Since most of these improvement projects are expected to qualify for negative declarations from permitting agencies, these costs do not include permitting or any environmental documentation.

Table 5: Storm Drain Replacement Unit Costs

Diameter (inches)	2015 Dollar per Linear foot of Pipe	2015 Dollar per Connection
15	$260	$12,150
18	$280	$12,230
21	$300	$12,310
24	$330	$12,380
27	$360	$12,460
30	$380	$12,540
33	$410	$12,610
36	$440	$12,690

42	$490	$12,840
48	$540	$13,000
54	$590	$13,150
60	$640	$13,290
66	$690	$13,450
72	$740	$13,600
78	$800	$13,690
84	$850	$13,910
Note: These costs do not include the 40% increase for design, administration, and contingency included in all other tables.		

All cost estimates prepared by Schaaf & Wheeler include an additional 40% for design, administration, construction management and contingency. Maps of the improvement priorities are shown in the CIP section above. New pump station costs were calculated assuming an estimated $40,000 per cfs of pumping capacity required. This assumption is based on previous project experience with building pump stations and is expected to vary based on individual site constraints. Land acquisition and permitting costs are not included in the individual pump station costs.

10-Year Improvements Plus 55-inches of SLR

The 10-Year improvements are required to route the runoff of the 10-year storm event paired with the 10-year tide and 55-inches of SLR. The improvement project costs and lengths are summarized below in Tables 5 and 6.

Table 5. Bay Farm Island 10-Year Plus 55-Inch SLR Improvements

Bay Farm Island 10-Year Improvements				
Project ID	Length (ft)	Outfalls	Total	Project Cost w/ 40% Contingency
Mecartney Rd	169	-	$68,000	$95,000
Magnolia Dr	268	-	$121,000	$169,000
Island Dr	244	-	$81,000	$113,000
Oldcastle Ln and Sea View Pkwy	1,432	-	$549,000	$768,000
Callan Pl and Castlebar Pl	711	-	$376,000	$527,000
Callan Pl and Aughinbaugh Way	701	-	$372,000	$521,000
Sea View Pkwy and Aughinbaugh Way	1,036	-	$558,000	$781,000
Aughinbaugh Way Pump	-	1	$4,000,000	$5,600,000
Adelphian Rd	301	-	$112,000	$157,000
Harbor Bay Pkwy and Loop Rd	86	1	$117,000	$164,000
Adelphian Rd and Harbor Bay Pkwy	628	-	$376,000	$527,000
Loop Rd	437	1	$289,000	$405,000
Airport Slough outfall	104	2	$157,000	$219,000
Harbor Bay Pkwy and Bay Edge Rd	126	1	$134,000	$188,000
S Loop Rd and Harbor Bay Pkwy	2,294	1	$1,588,000	$2,223,000
Harbor Bay Pky and Loop Rd pump	-	1	$12,000,000	$16,800,000
Adelphian Rd and Harbor Bay Pkwy	-	1	$1,120,000	$1,568,000

Bay Farm Island 10-Year Improvements				
Project ID	**Length (ft)**	**Outfalls**	**Total**	**Project Cost w/ 40% Contingency**
Total	**8,539**	**9**	**$22,018,000**	**$30,825,000**

Table 6: Main Island 10-Year Plus 55-Inch SLR Improvements

Main Island 10-Year Improvements				
Project ID	**Length (ft)**	**Outfalls**	**Total**	**Project Cost w/ 40% Contingency**
Tideway Dr	1,258	1	$584,000	$817,000
Central Ave and Encinal Boat Ramp	1,608	1	$973,000	$1,363,000
Crolis Garden Ct	191	-	$110,000	$155,000
Washington Park (2)	280	-	$119,000	$166,000
Washington Park (1)	243	-	$105,000	$147,000
McKay Ave and Washington Park	930	1	$521,000	$730,000
Shoreline Dr	4,789	1	$3,184,000	$4,458,000
Central Ave and Encinal Boat Ramp pump	-	1	$1,400,000	$1,960,000
Lagoon pump	-	1	$4,000,000	$5,600,000
Washington Park pump	-	1	$2,000,000	$2,800,000
Tideway Dr Pump	-	1	$2,400,000	$3,360,000
Shoreline Dr Pump	-	1	$2,000,000	$2,800,000
Marina Village Pkwy	951	-	$675,000	$945,000
Independence Dr	1,885	-	$1,052,000	$1,473,000
Main St	541	-	$231,000	$323,000
Singleton Ave	1,201	-	$714,000	$999,000
St Charles St and Sherman St	1,218	-	$650,000	$910,000
W Atlantic and Main St	165	-	$86,000	$120,000
W Hornet Ave and Endinal Boat Ramp Pump	-	1	$6,000,000	$8,400,000
Fernside Blvd	537	-	$262,000	$367,000
Eastshore Dr	3,038	1	$1,533,000	$2,147,000
High St	2,020	-	$1,488,000	$2,083,000
CA-61 and Fernside Blvd	2,024	1	$987,000	$1,382,000
High St pump	-	-	$4,000,000	$5,600,000
CA-61 and Fernside Blvd Pump	-	-	$2,000,000	$2,800,000
Clement Ave	7,644	2	$5,033,000	$7,046,000
Clement Ave Pump	-	1	$12,000,000	$16,800,000
Total	**30,524**	**15**	**$54,107,000**	**$75,751,000**

25-Year Improvements Plus 55-inches of SLR

The 25-Year improvements are required to route the runoff of the 25-year storm event paired with the 25-year tide and 55-inches of SLR. The improvement project costs and lengths are summarized below in Tables 7 and 8. The improvement costs are slightly higher due to increased storm event.

Table 7: Bay Farm Island 25-Year Plus 55-Inch SLR Improvements

Bay Farm Island 25-Year Improvements				
Project ID	**Length (ft)**	**Outfalls**	**Total**	**Project Cost w/ 40% Contingency**
Melrose	1,773	-	$1,010,000	$1,413,000
Savana	185	-	$86,000	$120,000
Flower and Mecartney	1,707	-	$961,000	$1,346,000
Island Dr	244	-	$81,000	$113,000
Packet Landing Rd Lift Station	-	-	$120,000	$168,000
Oldcastle Ln and Sea View Pkwy	1,432	-	$549,000	$768,000
Callan Pl and Castlebar Pl	711	-	$376,000	$527,000
Stanbridge Ln and Sheffield Way	823	-	$388,000	$543,000
Berkshire Rd and Avington Rd	1,097	-	$529,000	$740,000
Trajee Ln and Shamrock Ln	710	-	$296,000	$414,000
Justin Cir and Sea View Pwy	1,055	1	$596,000	$835,000
Sea View Pkwy and Aughinbaugh Way	1,036	-	$558,000	$781,000
Aughinbaugh Way Pump	-	1	$4,000,000	$5,600,000
Phoenix Ln and Island Dr	614	-	$356,000	$498,000
Harbor Bay Pkwy and N Loop Rd	4,029	-	$3,316,000	$4,643,000
Harbor Bay Pkwy and S Loop Rd	1,498	-	$1,071,000	$1,500,000
Adelphian Rd	443	-	$171,000	$239,000
S Loop Rd	928	-	$390,000	$546,000
Harbor Bay Pkwy	88	-	$119,000	$166,000
Harbor Bay Pkwy outfall	126	1	$134,000	$188,000
Harbor Bay Pkwy Pump	-	1	$16,000,000	$22,400,000
Adelphian Rd Pump	-	1	$1,280,000	$1,792,000
Total	**18498**	**5**	**$32,387,000**	**$45,340,000**

Table 8: Main Island Island 25-Year Plus 55-Inch SLR Improvements

Main Island 25-Year Improvements				
Project ID	**Length (ft)**	**Outfalls**	**Total**	**Project Cost w/ 40% Contingency**
Tideway Dr	1,246	1	$658,000	$921,000
Central Ave and Encinal Boat Ramp	1,565	1	$950,000	$1,330,000
Shoreline Dr	5,870	1	$4,392,000	$6,149,000
McKay Ave and Washington Park	922	1	$416,000	$582,000
Alameda Towne Center	111	0	$62,000	$86,000
Central Ave and Encinal Boat Ramp pump	-	1	$2,000,000	$2,800,000
Washington Park pump	-	1	$1,200,000	$1,680,000
Shoreline Dr Pump	-	1	$2,400,000	$3,360,000
Tideway Dr Pump	-	1	$2,000,000	$2,800,000
Constitution Way and Marina Village Pkwy	56	0	$40,000	$56,000
Independence Dr	490	0	$199,000	$278,000
Main St	2,339	1	$1,472,000	$2,061,000
Marina Village Dr	381	0	$212,000	$297,000
W Altantic and Main St	165	0	$85,000	$120,000
W Hornet Ave and Endinal Boat Ramp Pump	-	1	$6,400,000	$8,960,000
Main St Pump	-	1	$1,440,000	$2,016,000
Washington St	90	-	$62,000	$87,000
Eastshore Dr	2,907	1	$1,616,000	$2,262,000
Fernside Blvd	2,188	1	$1,339,000	$1,874,000
High St	3,210	1	$2,761,000	$3,865,000
High St pump	-	1	$4,800,000	$6,720,000
Fernside Blvd pump	-	1	$4,000,000	$5,600,000
Clement Ave	5,078	2	$3,123,000	$4,373,000
Grand St	185	1	$172,000	$240,000
Clement Ave pump	-	1	$8,000,000	$11,200,000
Grand St Pump *New Pump Station	-	1	$4,800,000	$6,720,000
Total	**26,803**	**21**	**$54,599,000**	**$76,437,000**

Conclusion

As a low lying island, the City of Alameda is highly susceptible to rising sea levels. Several types of improvements need to be made in order to protect the City from future flooding. The projects will require a substantial investment and several decades to complete in their entirety. Current estimates indicate 55-inches of SLR will occur around the year 2100; luckily this provides a long time period for planning to determine the best possible method of implementation. Some projects will become necessary while others maybe become easier to construct at different points in time; the order in which these projects are constructed will vary based on future Sea Level Rise projections, level of impact, and available funding.

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Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS

MEMORANDUM

TO:	Liam Garland, Public Works Director, City of Alameda Public Works Department	DATE:	December 5, 2017
FROM:	Dan Schaaf, P.E.	JOB#:	APWD.16.17
SUBJECT:	City of Alameda Storm Drain Master Plan CIP Update		

Introduction

The City of Alameda (City) has tasked Schaaf & Wheeler with updating existing Storm Drain Capital Improvement Plans (CIPs) identified as a part of previous studies on the existing storm drain system.

This memorandum is organized to show the costs for existing CIPs, newly identified CIPs, and the additional CIP projects necessary to accommodate for increases in the tide from Sea Level Rise. The City has provided Schaaf & Wheeler with various sources where existing CIPs have been identified. Sources of CIPs include:

- Storm Drain Master Plan, Schaaf & Wheeler
- The 2011 Pump Station Assessment Study, Psomas
- The Lagoon Operations Study, Schaaf & Wheeler
- Various Development Plans, Schaaf & Wheeler
- New CIPs identified by City staff
- 18-inch Sea Level Rise Study, Schaaf & Wheeler
- 55-inch Sea Level Rose Study, Schaaf & Wheeler

All of the proposed CIPs are broken into three priority levels for funding and implementation. Descriptions for each priority level are shown in Table 1. Priorities for all CIPs have been evaluated and updated based on input from the City and changes to existing infrastructure over time.

Total Costs, 2017 Costs

Category	Priority			Total
	High	Moderate	Low	
10-yr Storm Total w/o SLR	$ 18,400,000	$ 31,900,000	$ 50,500,000	**$ 100,800,000**
Total w/ 18" SLR Pipe CIPs	$ 18,400,000	$ 35,500,000	$ 50,500,000	**$ 104,400,000**
Total w/ 55" SLR Pipe CIPs	$ 18,400,000	$ 141,300,000	$ 50,500,000	**$ 210,200,000**
Total w/ 55" SLR Pipe CIPs + Inundation	$ 18,400,000	$ 584,900,000	$ 50,500,000	**$ 604,100,000**

Note: 18" SLR costs do not include costs for floodwall or levee improvements

Table 1 – Capital Improvement Priority Levels

Priority Level	Description
High	Projects under this category have either been specifically identified by City staff as high priority or have a large area of flooding where the 10-year flow depth in the street is more than one foot over the top-of-curb. These projects improve locations with the deepest and longest flooding situations in each of the five sections of the City. Areas of significant historical flooding fall into this category.
Moderate	This category has conditions similar to high priority, but has a smaller area affected by flooding. A 10-year design discharge still overtops the top-of-curb; however, the length and depth of flooding is less than that of a high priority improvement.
Low	Low priority improvements are generally smaller projects that consist of placing a few pipe segments. Existing flooding is not necessarily contained within the roadway (top-of-curb); however, the area of flooding is much smaller and/or briefer in duration than that of moderate and high priority projects.

All CIPs include a 50% contingency for design, engineering, and administration and have been updated to September 2017 dollars. Costs updates come from better pipe and connection cost data and adjustments for inflation tracked by the Engineering News Record Construction Cost Index. The updated pipe and connection costs can be found in Table 2.

Table 2 –2017 Pipe and Connection Costs

Diameter (inches)	Dollar per Linear Foot of Pipe	Dollar per Connection
15	$275	$12,800
18	$295	$13,895
21	$315	$12,975
24	$355	$13,050
27	$375	$13,135
30	$405	$13,215
33	$440	$13,575
36	$460	$13,715
42	$510	$13,995
48	$575	$14,275
54	$625	$15,445
60	$680	$16,135
66	$730	$17,160
72	$780	$17,515
78	$845	$18,760
84	$895	$20,000
96	$940	$21,985

Costs are updated from recent area Storm Drain Master Plans by Schaaf & Wheeler.

High Priority Capital Improvements

Several improvements were identified by the City as high priority projects. Descriptions of each project are summarized below.

Veterans Court

The top of the Seawall at Veterans Court (Figure 1A) is lower than FEMA's 100 year flood elevation. In order to protect the area, raising the roadway along Veterans Court or installation of a flood gate is recommended. For planning purposes we have assumed a top of roadway or gate elevation of 12.5 feet in order to account for an 18-inch raise in sea level and 1 foot of freeboard. Raising the road has a greater impact on the overall square footage disturbed as shown in Figure 1B. The preliminary cost estimate for installing a flood gate is slightly higher than raising the road but it would have a smaller square footage disturbed as shown in Figure 1C. The extent of utility relocation and landscaping features can greatly impact the construction costs. Both options assume repaving of Veterans Court.

This work is to add protection for sea level rise and in the event the exiting seawall becomes compromised. It does not include a solution for the flooding and sea level rise effects resulting from inundation along Doolittle Drive. The flooding from Doolittle Drive needs to be further analyzed.

Figure 1A – Veterans Court Location Map

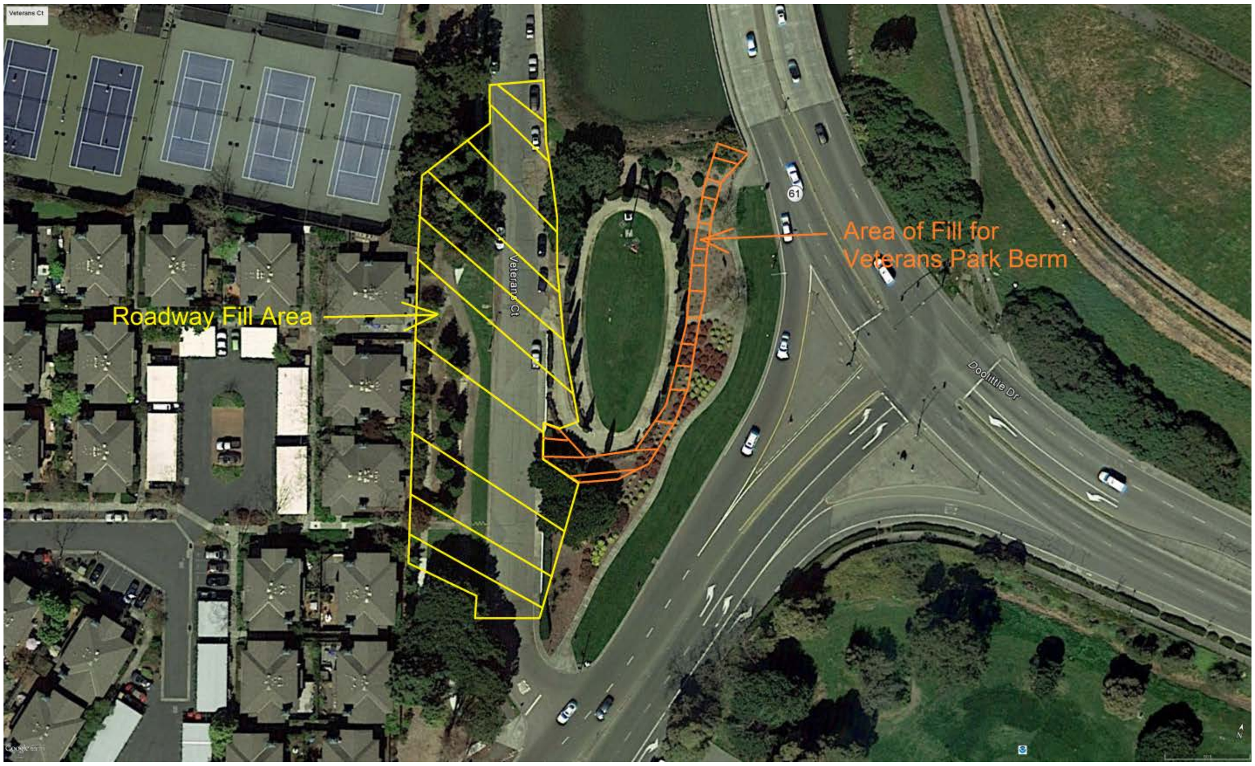

Figure 1B – Veterans Court: Raising Roadway

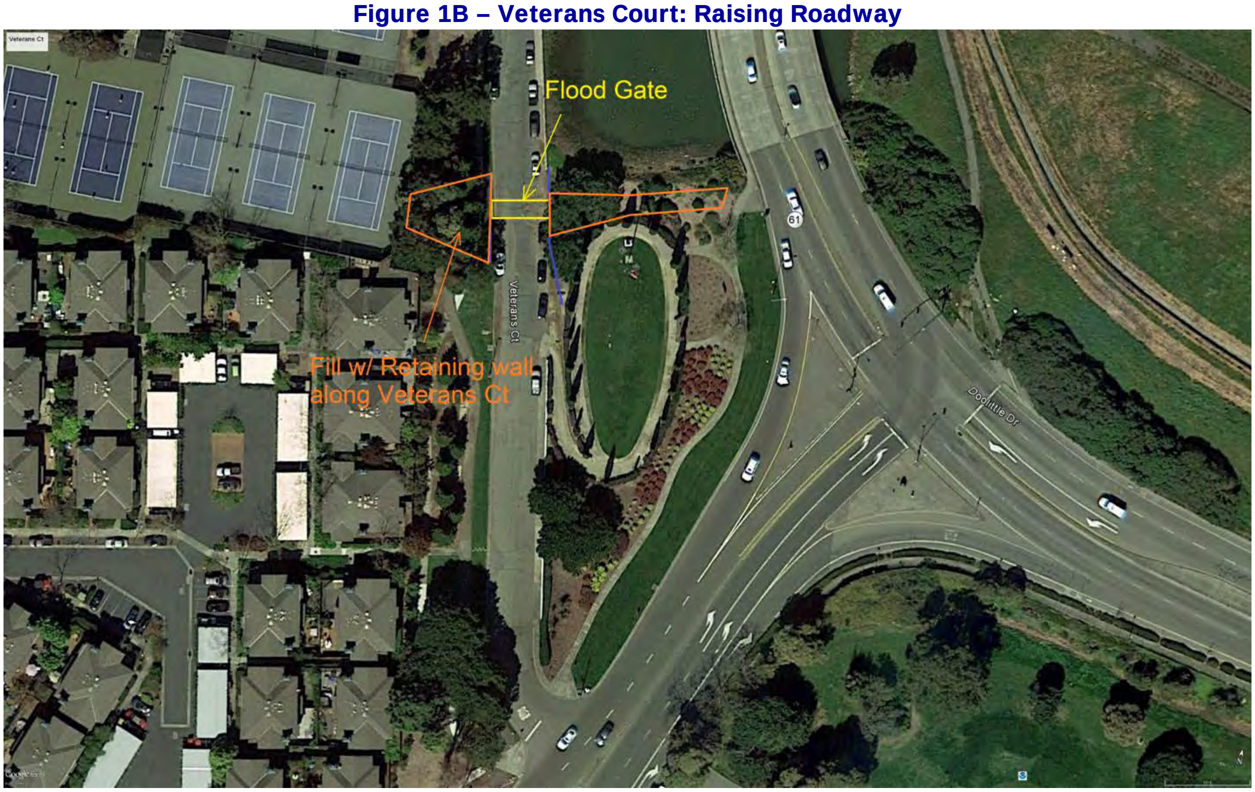

Figure 1C – Veterans Court: Flood Gate

Seawall at Bay Farm Island Gate Structure

The retaining wall at the Bay Farm Island Lagoon Gate Structure (Figure 2A) on the North Shore is lower than the surrounding levees and is vulnerable to flooding as shown on the 2015 Preliminary FEMA Flood Insurance Rate Maps (FIRMs). It is recommended to install a new retaining wall two feet higher than the existing retaining wall in order to be level with the gate structure platform and to remove the homes from the FEMA flood zone. To minimize permitting and costs the new retaining wall should be built behind the existing retaining wall. A taller retaining wall may be needed to account for future sea level raise; however a taller retaining wall currently does not make sense because the surrounding levees would be shorter than the retaining wall. The existing gate structure is shown in Figure 2B.

Figure 2A – Bay Farm Island Gate Location Map

Figure 2B – Bay Farm Island Gate Structure

Lagoon Intake Pipe

The existing 24 inch sliplined intake pipe at the intersection of Westline Drive and Shoreline Drive (Figure 3A) is potentially compromised. The intake pipe feeds the existing intake pump shown in Figure 3B. A new 24-inch intake pipe is proposed to replace the existing intake pipe. The new intake pipe should be HDPE and installed below the bottom of the sea floor with approximately 2 feet of cover using dredging or jet trenching construction methods.

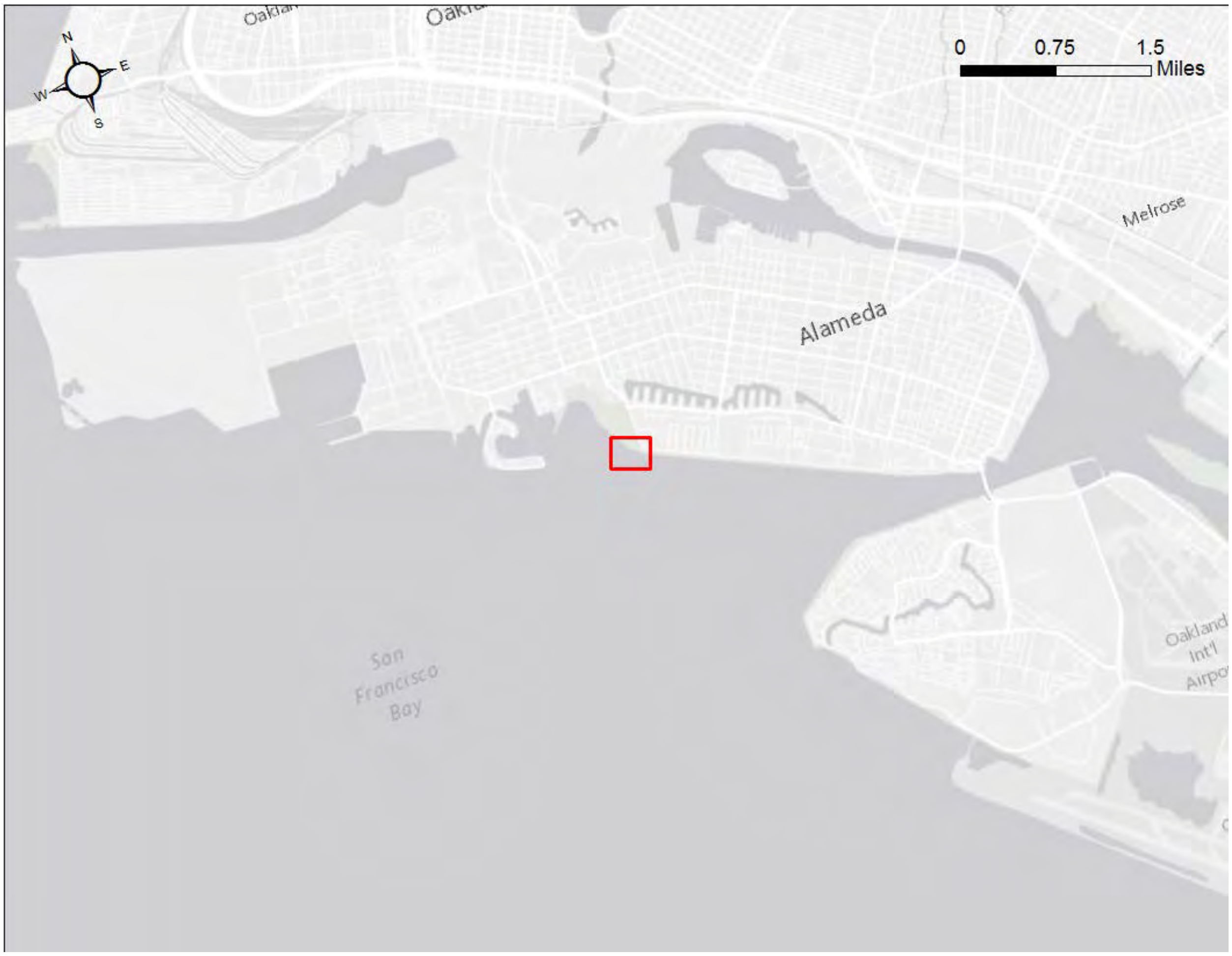

Figure 3A –Lagoon Intake Location Map

Figure 3B –Lagoon Intake Pump

Dredge Existing Sediment in Lagoon #3

The 2014 South Shore Lagoon Dredging Project called for approximately 2,385 cubic yards of sediment to be removed from Lagoon #3 (Figure 4). Lagoon #3 was ultimately excluded from the project due to high levels of contaminants at sampled locations. A portion of dredged material is expected to have contaminants and will need to be disposed of at a Class 2 landfill or a Class 1 landfill that accepts Class 2 contaminants in soil.

Figure 4 –Lagoon #3 Location Map

Interior Lagoon Outlet Works

The Lagoon outlet flap gate (Figure 5A) does not function properly and has built up sediment. Removal of the sediment and installation of a new flap gate will restore the functionality of the Lagoon outlet. Figure 5B shows the current condition of the gate structure.

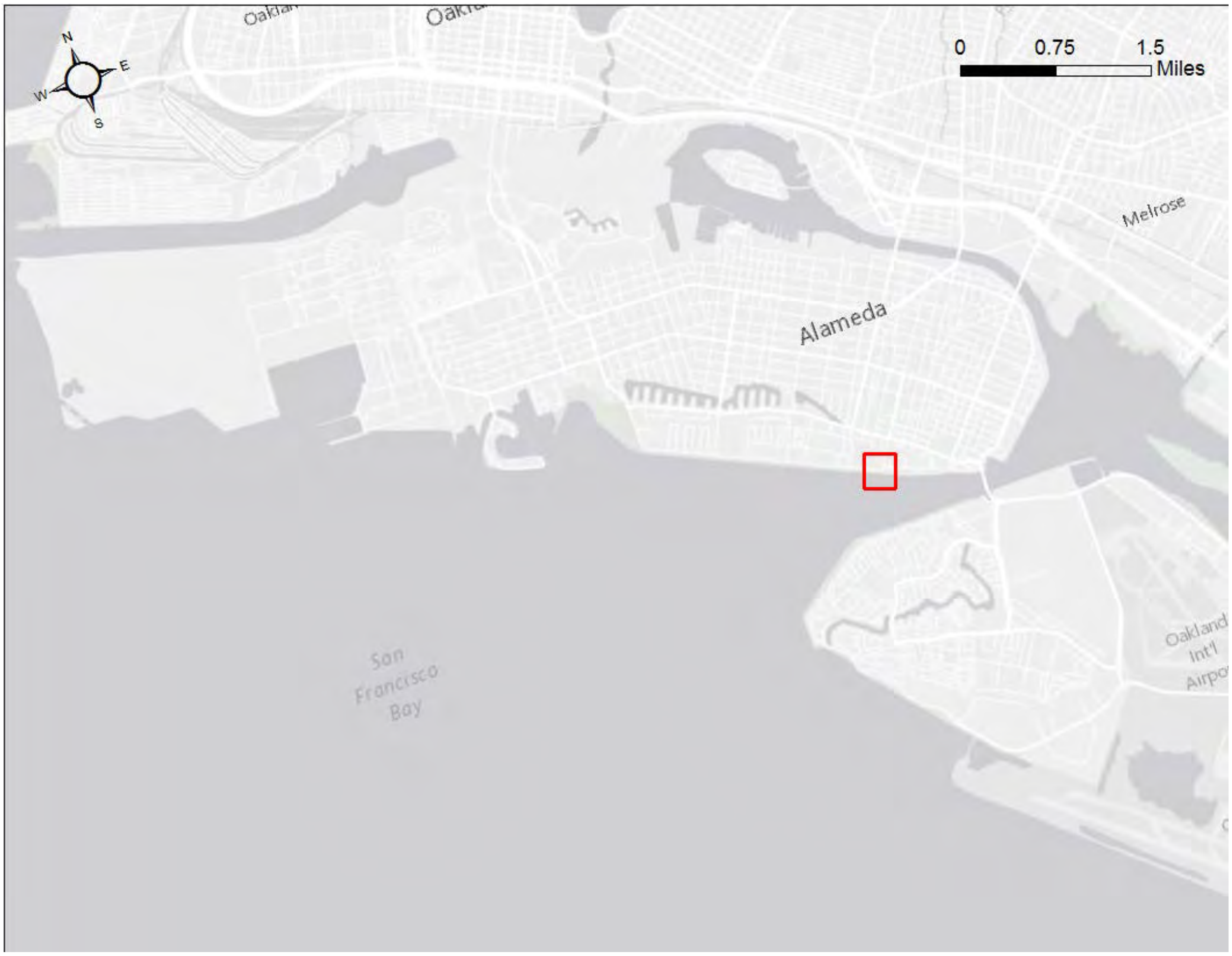

Figure 5A –Lagoon Outlet Location Map

Figure 5B –Lagoon Outlet Condition

Removing Shoreline Drive Outfalls

The storm drain outfalls along Shoreline Drive (Figure 6A) have become less effective in discharging storm runoff to the San Francisco Bay. In addition, some outfalls are damaged. The outfalls are subject to tidal effects, sedimentation and corrosion. This project would remove the existing outfalls and divert flows to the South Lagoon via an RCP pipeline (Figure 6B). This pipeline varies in diameter from 24-inches to 48-inches (Figure 6C) and should have a constant slope and adequate cover.

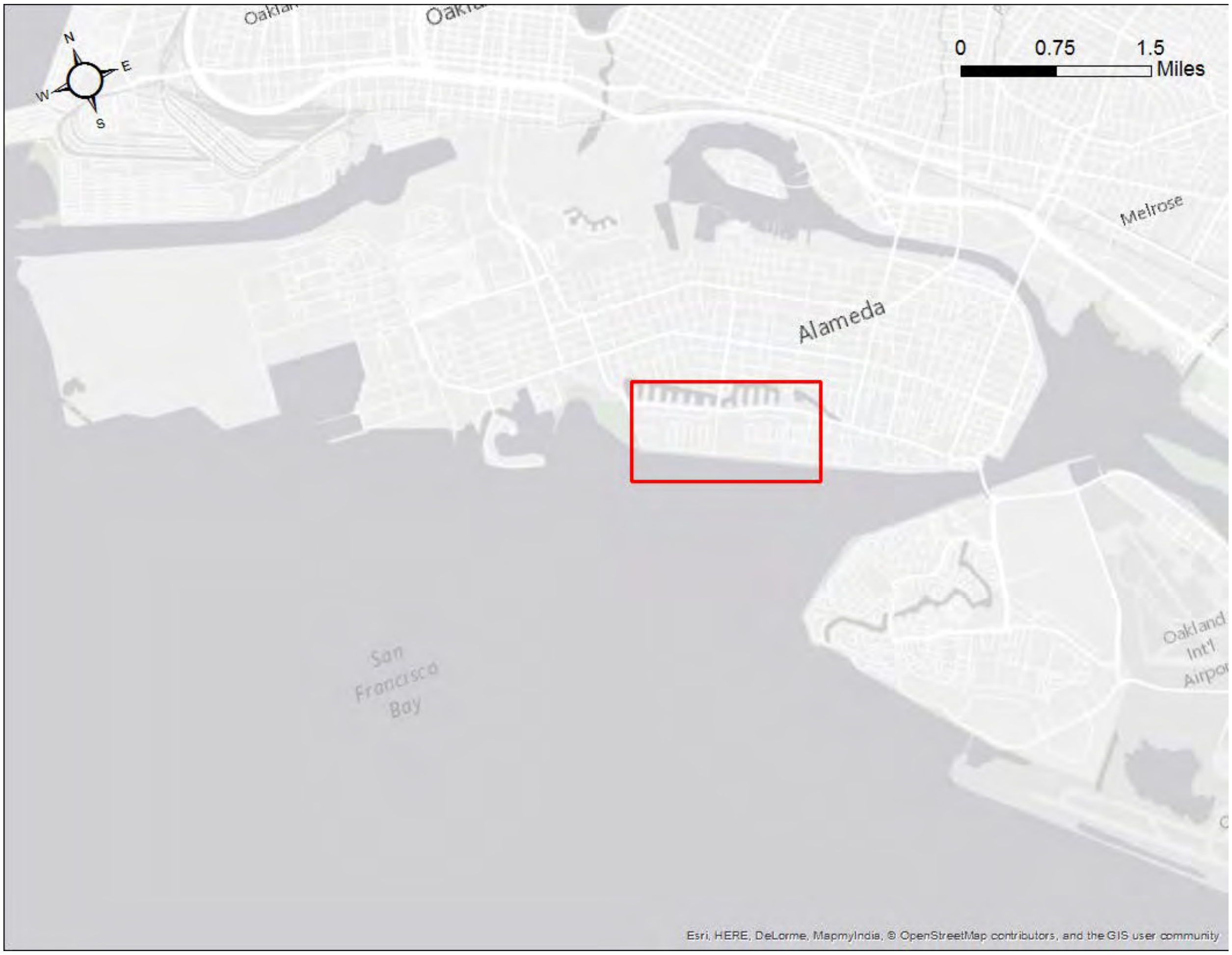

Figure 6A –Shoreline Drive Outfalls Location Map

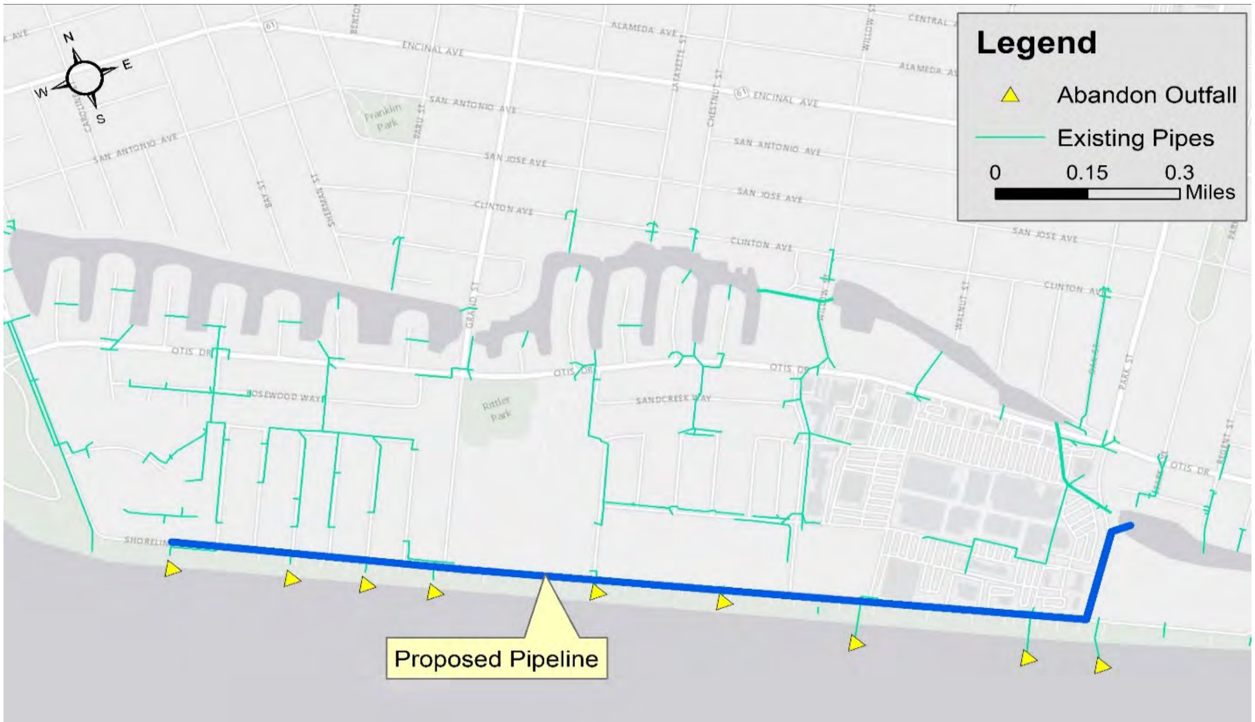

Figure 6B –Shoreline Drive Pipeline

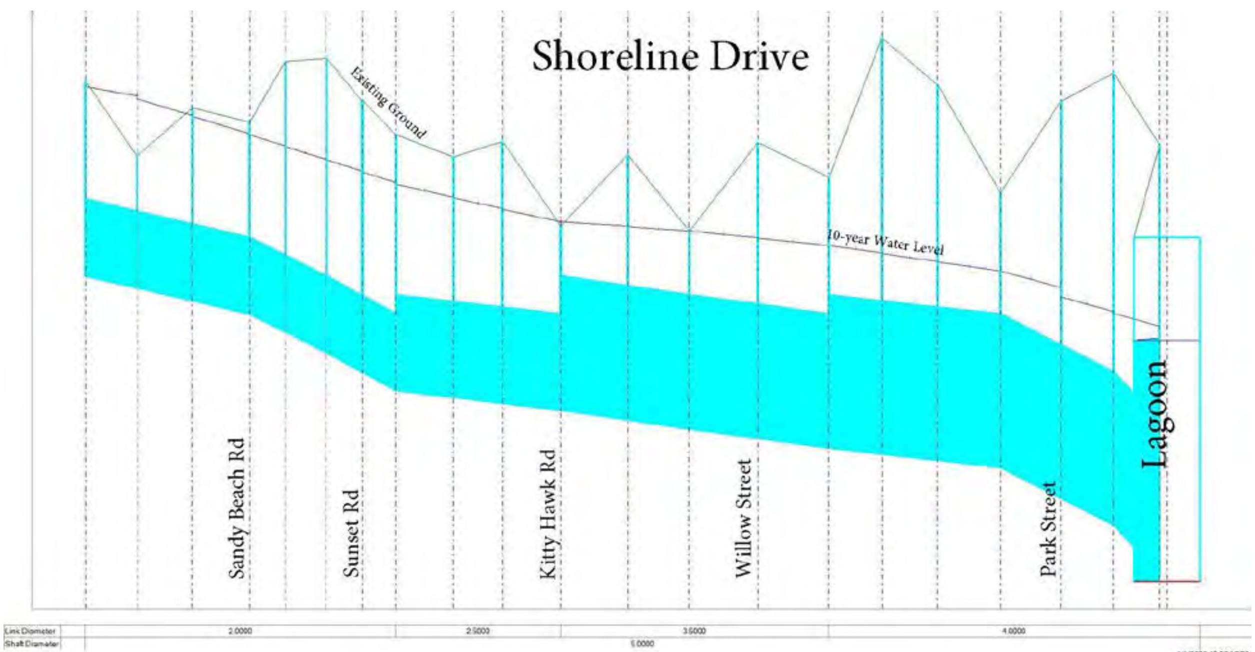

Figure 6C–Shoreline Drive 10-year Profile

CIP Costs

A cost summary for the near term capital improvements described above is shown in Table 3. A more detailed estimate of initial construction costs can be found in Appendix A.

Table 3 – High Priority CIP Priority and Cost

Improvement Name	Probable Construction Cost Subtotal	Total Cost w/ 50% Contingency
Veterans Court Option 1A - Raise Roadway	$1,100,000	$1,800,000
Veterans Court Option 1B - Floodgate	$1,100,000	$1,700,000
Seawall at Bay Farm Island Gate Structure	$ 200,000	$ 300,000
Lagoon Intake Pipe	$ 600,000	$ 900,000
Dredge Existing Sediment in Lagoon #3	$ 700,000	$1,100,000
Interior Lagoon Outlet Works	$ 40,000	$ 60,000
Remove Shoreline Drive Outfalls	$3,700,000	$5,600,000

Existing CIP Cost Updates

The Storm Drain Master Plan (SDMP) for the City of Alameda was originally completed by Schaaf & Wheeler in 2008 and was revised in 2011. The cost updates in this memorandum come from the 2011 version of the SDMP report and include the Pump Station improvements recommended in the Storm Drain Pump Station Assessment Report by Psomas in 2011.

The 2011 SDMP evaluates two design storm scenarios: the 10-year design discharge and the 25-year design discharge. Since it may not be possible to provide a design that meets the desired 25-year standard for the existing storm drain system, it is recommended that the existing CIPs be designed to follow the design criteria listed in Table 4. The design criteria for new CIPs must be evaluated on a case-by-case basis.

Table 4 – Storm Drain Master Plan Design Criteria for Existing Systems

Design Storm Discharge	Design Criteria
10-Year Design Discharge	Pipes shall be sized to carry the 10-yeardischarge without surcharging the pipe. When downstream surcharge effects are included, upstream hydraulic grades shall be no higher than the top of curb elevation at any manhole or inlet.
25-Year Design Discharge	Hydraulic grade shall not exceed the top of curb elevation at any location.

Additional design criteria in the SDMP are used to evaluate the distances between existing storm drain structures. Since City standards allow for distances less than those listed in the SDMP, additional evaluation may be needed to identify areas where existing pipe lengths exceed City standards. The following additional considerations for the existing storm drain are evaluated in the SDMP:

- Manholes shall be spaced no farther than 400 feet apart.
- Catch basins shall be spaced so the maximum width of gutter flow does not exceed eight (8) feet from the face of the curb during the 10-year event; or 400 feet, whichever is less.

The SDMP uses eight (8) drainage sub areas to organize the pipe CIPs. Figure 7 shows the location of each sub drainage area.

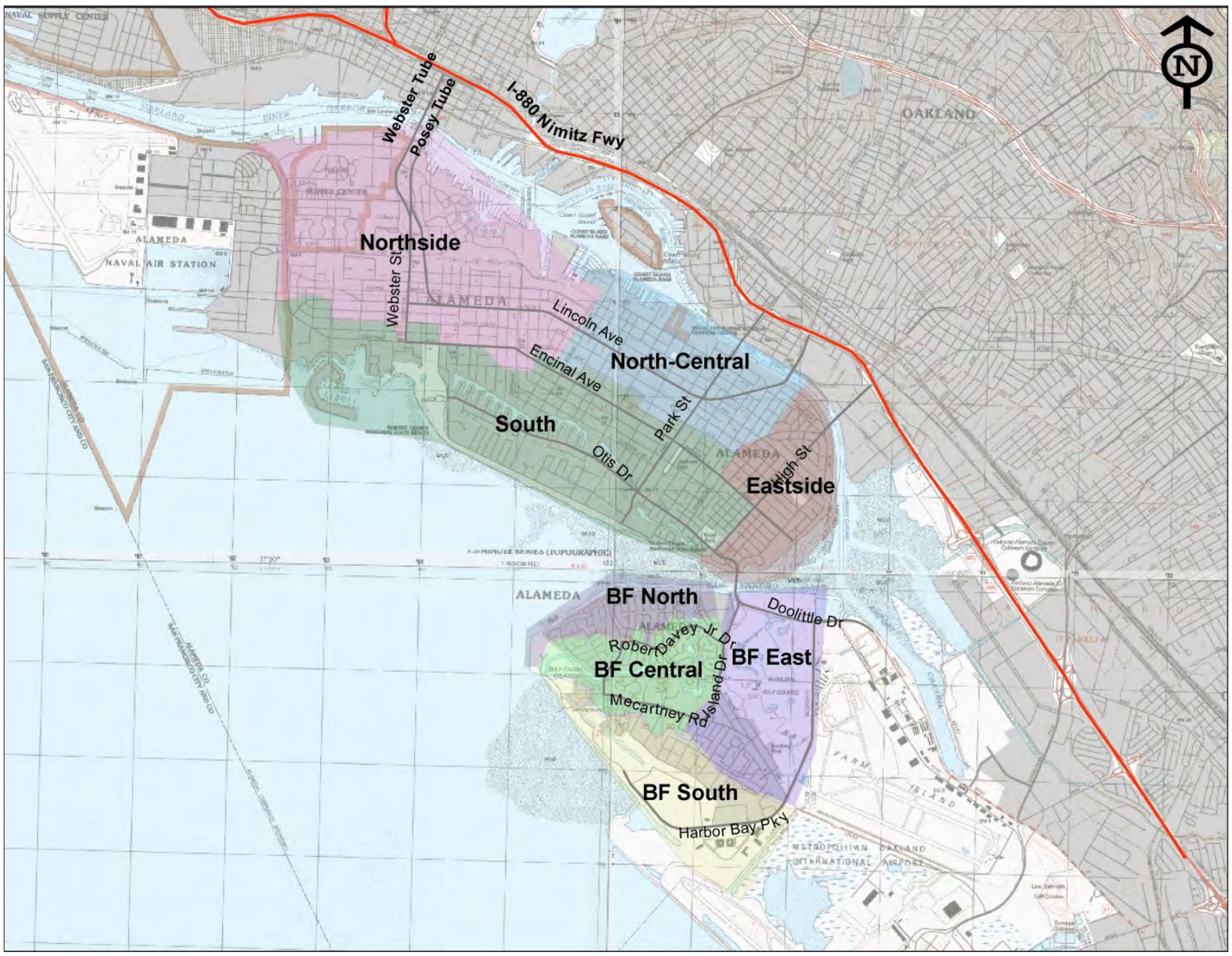

Figure 7 – City of Alameda Storm Drain Master Plan Drainage Sub Areas

Pipe Costs

The existing CIPs for pipe costs are evaluated at the 10-year design level. A figure showing the approximate location of each CIP can be found in Appendix B. The CIPs are organized by sub drainage area and are listed in Tables 5-12. Pipe extensions are considered to be separate projects from existing conditions CIPs and can be found in Table 13.

Table 5 – Alameda Island, Eastside Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Gibbons (new pipe)	Moderate	3968	11	1	$2,000,000	$3,000,000
Thompson	Low	1344	11	1	$700,000	$1,100,000
High	Moderate	3691	26	1	$2,100,000	$3,200,000
Fernside	Low	2411	14	0	$1,200,000	$1,800,000
Washington	Low	1161	8	0	$ 500,000	$ 800,000
Calhoun	Low	289	4	1	$ 200,000	$ 300,000

Table 6 – Alameda Island, North Central Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Grand	Med	4106	29	1	$2,200,000	$3,300,000
Willow	Med	3513	19	1	$1,900,000	$2,900,000
Walnut	Low	2763	20	1	$1,500,000	$2,300,000
Oak	Low	2573	13	1	$1,300,000	$2,000,000
Park	Low	637	7	1	$ 400,000	$ 600,000
Everett	Low	1086	8	1	$ 600,000	$ 900,000
Broadway	Low	449	7	1	$ 400,000	$ 600,000
Pearl	Low	790	7	1	$ 500,000	$ 800,000
Tilden	Low	395	5	1	$ 300,000	$ 500,000
Cambridge	Low	986	8	1	$ 600,000	$ 900,000

Table 7 – Alameda Island, Northside Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Constitution	Moderate	3300	12	1	$2,900,000	$4,400,000
West Atlantic	Low	3400	26	1	$2,600,000	$3,900,000
East Atlantic (1)	Low	700	3	0	$500,000	$800,000
East Atlantic (2)	Low	300	4	1	$400,000	$600,000
New Outfall	Moderate	3700	11	1	$3,100,000	$4,700,000
Main St	Low	500	4	0	$ 300,000	$ 500,000
Webster (2)	Low	100	2	0	$ 90,000	$ 140,000
3rd Street	Low	700	8	0	$ 500,000	$ 800,000
Webster (3)	Low	1500	7	0	$ 700,000	$1,100,000
Chapin	Low	300	4	0	$ 300,000	$ 300,000
Paru	Low	1600	16	0	$1,100,000	$1,700,000
Bay Sherman	Low	2200	21	0	$1,200,000	$1,800,000
Main St (2)	Low	1200	5	0	$ 500,000	$ 800,000
5th Street	Low	1700	13	0	$ 900,000	$1,400,000
Pacific St	Low	1400	7	0	$ 700,000	$1,100,000

Table 8 – Alameda Island, South Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Fountain	Low	1659	20	1	$1,000,000	$1,500,000
Mound	Low	524	3	1	$ 300,000	$ 500,000
Franciscan	Low	2063	15	0	$ 1,000,000	$1,500,000
Harbor Light	Moderate	3456	18	1	$1,500,000	$2,300,000
Rosewood	Moderate	1295	18	1	$ 700,000	$1,100,000
Pearl	Low	990	7	0	$ 600,000	$ 900,000
Alameda Park	Moderate	2277	7	0	$1,100,000	$1,700,000
3rd	Low	501	7	1	$ 300,000	$ 500,000
Willow	Low	1670	0	1	$ 30,000	$ 50,000
S Shore Center W	Low	1593	6	0	$ 700,000	$1,100,000
Regent	Low	275	6	1	$ 300,000	$ 500,000
Park	Low	320	5	0	$ 300,000	$ 500,000
Page	Low	1983	14	1	$1,000,000	$1,500,000
Webster	Low	1154	8	1	$ 600,000	$ 900,000
Ballena	Low	795	8	1	$ 500,000	$ 800,000
Paru	Low	74	3	0	$ 60,000	$ 90,000
Shoreline	Low	700	7	2	$ 400,000	$ 600,000

Table 9 – Bay Farm Island, Central Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Dublin Way	Low	1107	9	1	$ 600,000	$ 900,000
Island Drive	Low	69	2	0	$ 50,000	$ 80,000
Verdemar Drive	Low	1460	13	1	$ 700,000	$1,100,000
Robert Davey Jr Dr	Low	1308	3	0	$ 100,000	$ 200,000
Mecartney Road	Low	1855	9	0	$ 800,000	$1,200,000

Table 10 – Bay Farm Island, North Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Avington	Low	1052	7	1	$ 600,000	$ 900,000

Table 11 – Bay Farm Island, East Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Camelia	Low	2547	18	0	$1,300,000	$1,200,000
Fitchburg	Low	632	5	0	$ 400,000	$ 600,000

Table 12 – Bay Farm Island, South Area, 10-Year Storm Protection CIP, 2017 Costs

Improvement Name	Priority Level	Pipe Length	Connections	Outfalls	Subtotal	Total Cost w/ 50% Contingency
Holly	Low	1823	7	0	$ 700,000	$ 1,100,000

Table 13 – Pipe Extensions, 10-Year Storm Protection CIP, 2017 Costs

Improvement Area	Pipe Length	Connections	Inlets	Subtotal	Total Cost w/ 50% Contingency
Northside	2567	12	12	$ 900,000	$1,400,000
North Central	2772	11	14	$1,000,000	$1,500,000
South	3418	17	22	$1,200,000	$1,800,000
Eastside	224	2	2	$100,000	$200,000

Pump Station Costs

Pump Station Costs come from the SDMP and the Pump Station Assessment Report. Pump Station locations are shown in Figure 8. Costs were combined and updated to reflect 2017 dollars and are shown in Table 14.

Figure 8 – City of Alameda Pump Station Location Map

Table 14 – Pump Station Improvements Costs by Pump Station

Pump Station	Priority Level	Subtotal	Total Cost w/ 50% Contingency
Arbor Pump Station	High	$ 2,500,000	$ 3,800,000
Bayport Pump Station	Moderate	$ 700,000	$ 1,100,000
Central/ Eastshore Pump Station	Moderate	$ 1,800,000	$ 2,700,000
Golf Course Pump Station	High	$ 700,000	$ 1,100,000
Harbor Bay System I Pump Station	Low	$ 600,000	$ 900,000
Harbor Bay System II Pump Station	Moderate	$ 700,000	$ 1,100,000
Main Street Pump Station	High	$ 200,000	$ 300,000
Northside Pump Station	High	$ 1,500,000	$ 2,300,000
Third Street Pump Station	High	$ 400,000	$ 600,000
Webster Pump Station	High	$ 700,000	$ 1,100,000
		Total	**$15,000,000**

Sea Level Rise CIP Costs

The City of Alameda is susceptible to Sea Level Rise (SLR) due to its location in San Francisco Bay and its relatively low ground elevation. Two studies were completed by Schaaf & Wheeler to analyze the impacts of Seal Level Rise: Climate Change Impacts to Storm Drain Improvements: an addendum to the Storm Drain Master Plan (CCI) for 18-inches of SLR, completed in 2009 and a memorandum for the 55-inch Sea Level Rise completed in 2015.

Sea Level Rise Impacts

The impact from SLR on the City of Alameda comes from both overland inundation from the tide and a decrease in storm drain capacity resulting from increased water surface elevations at the outfalls. Both reports examine the effects of SLR on the existing storm drain system and provide recommended improvements; however, only the 55-inch SLR report provides recommendations for inundation for the 100-year-tide plus SLR. The potential CIPs and costs identified as a part of these studies have been updated for September 2017 dollars.

Updated Sea Level Rise Data

In April 2017, the Ocean Protection Science Advisory Team published updated SLR projections for the State of California. *The Rising Seas in California: an Update on Sea-Level Rise Science Report* provides updated SLR projections for San Francisco Bay for the years 2050, 2100, and 2150. Several different emission scenarios, known as representative concentration pathways (RCP) were analyzed in the report; however, RCP 8.5 is the accepted emission scenario for 2050. Since RCP 8.5 SLR represents the expected SLR if there are “no significant global efforts to limit or reduce emissions,” it is summarized for both 2050 and 2100 SLR in Table 15 below (Griggs, et al., 2017). Both of the existing SLR reports use elevations that fall between the 5% and 0.5% probability scenarios.

Table 15 – RCP 8.5 Sea Level Rise for San Francisco Bay

Feet above 1991-2009 mean	Median	Likely Range	1-in-20 Chance	1-in-200 Chance
Year	*50% probability SLR meets or exceeds...*	*67% probability SLR is between...*	*5% probability SLR meets or exceeds...*	*0.5% probability SLR meets or exceeds...*
2050	0.9	0.6 – 1.1	1.4	1.9
2100	2.5	1.6 – 3.4	4.4	6.9

Source: The Rising Seas in California: an Update on Sea-Level Rise Science Report (Griggs, et al., 2017).

18-Inch Sea Level Rise

The CCI Addendum addresses the impacts from capacity changes in the storm drain system for 18 inches of SLR. The report specifically assesses the 10-year storm with three different outfall water surface elevations:

- The 10-year tide plus 18 inches of SLR
- The 25-year tide plus 18 inches of SLR
- The 100-year tide plus 18 inches of SLR.

The cost summaries from the CCI Report analyze the increase to the CIP necessary to maintain a 10-year level of service. The Addendum uses the 2008 SDMP models updated for 18-inches of SLR. A summary of the overall increase to existing improvement costs are shown in Table 16.

Table 16 –10-Year Storm Protection CIP Plus 18 Inches of Sea Level Rise, 2017 Costs

Location	Additional Subtotal Cost	Total Additional Cost w/ 50% Contingency
Alameda Island	$ 1,900,000	$ 2,900,000
Bay Farm Island	$ 1,100,000	$ 1,700,000
Total	**$ 3,000,000**	**$ 4,600,000**

55-Inch Sea Level Rise

The 55-inch SLR study addresses the impacts from inundation on low-lying areas and capacity changes in the storm drain system for 55 inches of SLR.

Inundation from Rising Tides

Since the City has a ground surface that is very low, 55 inches of SLR has a significant effect on inundation from the tide. The 55-Inch SLR study assesses the inundation on the City of Alameda for three water surface elevation scenarios:

- The 10-year tide plus 55 inches of SLR
- The 25-year tide plus 55 inches of SLR
- The 100-year tide plus 55 inches of SLR.

Overland flooding from the tide plus 55 inches of SLR can be mitigated by constructing improvements around the low areas of the Main Island and Bay Farm Island. Figures 9 and 10 show a combination of raised bike paths, raised roadways, and seawalls that would provide flood protection for the 100-year tide plus the 55 inches of SLR. The projects analyzed as a part of the 55 inch SLR study are larger in both scale and cost. Smaller scale projects due to the inundation from SLR, such as a project that would benefit the area east of Fernside from rising tides, have not been studied and require more detailed analysis.

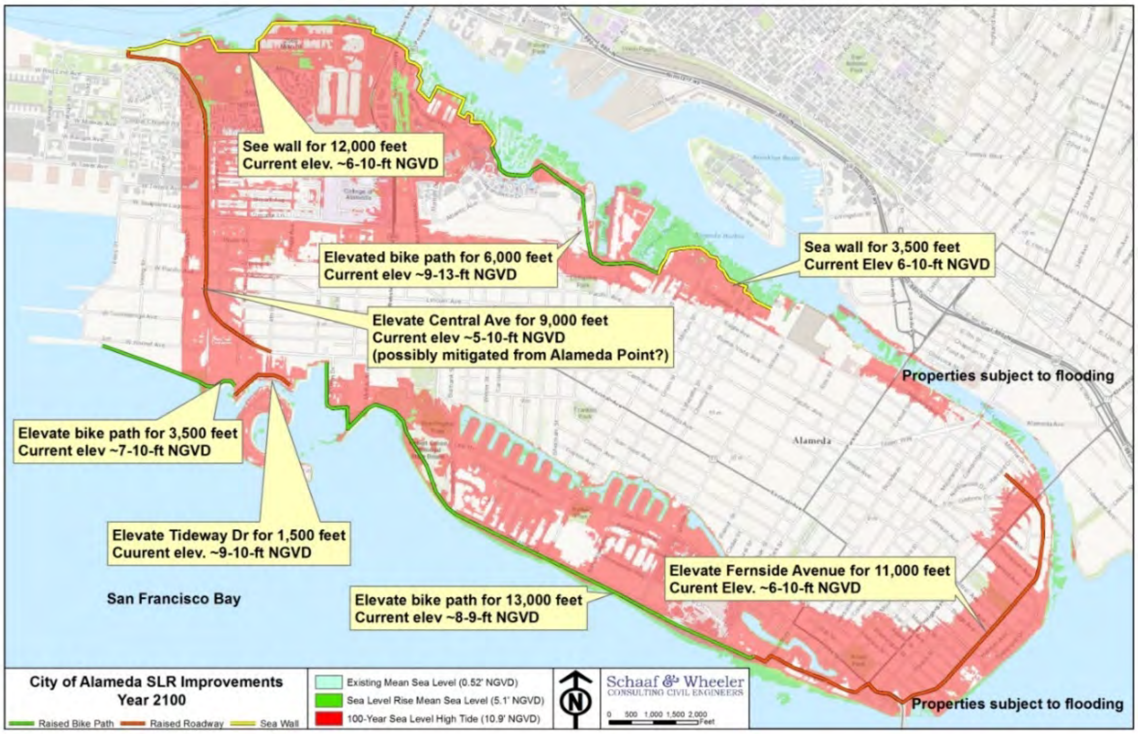

Figure 9 – Main Island Overland Improvements, 100-Year-Tide with 55 Inches of Sea Level Rise

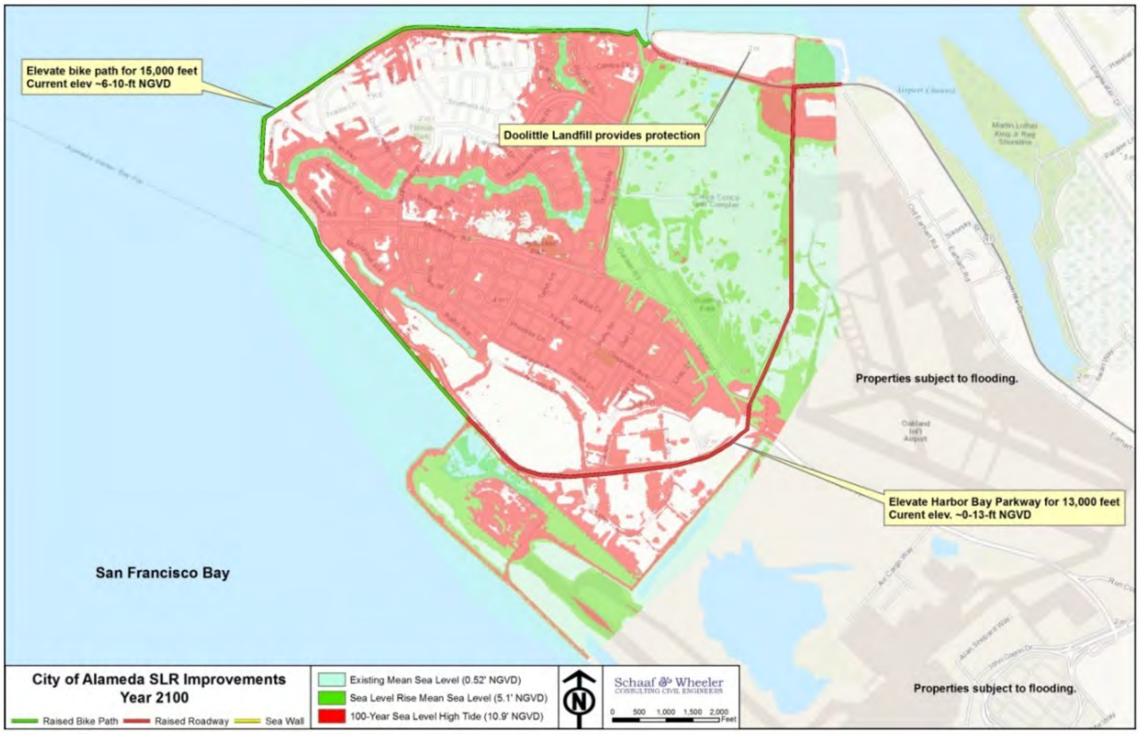

Figure 10 – Bay Farm Island Overland Improvements, 100-Year-Tide with 55 Inches of Sea Level Rise

Levee and Seawall improvement costs are dependent on the type of improvement, the existing soil conditions and the height that the improvements would need to be raised to provide 100-year flood protection with the addition of the 55 inches of SLR. Costs for the improvements for the Main Island and Bay Farm Island are shown in Table 17 below.

Table 17 –100-Year-Tide Overland Improvements Plus 55 Inches of SLR, 2017 Costs

Project ID	Type	Height (feet)	Length (feet)	Total Additional Cost w/ 50% Contingency
Shoreline	Levee	5.3	13,500	$ 63,100,000
Fernside	Levee	4.2	13,000	$ 63,100,000
Main Street	Levee	5.5	8,700	$ 55,200,000
Hornet	Levee	5.9	3,300	$ 9,000,000
Tideway	Levee	6.0	1,000	$ 6,900,000
BFI West	Levee	6.1	15,000	$ 42,600,000
BFI East	Levee	7.9	12,500	$116,200,000
Northside Seawall	Floodwall	9.8	14,300	$ 28,900,000
Clement Seawall	Floodwall	8.5	14,100	$ 18,600,000
Misc. Small Scale Projects	Misc.	n/a	n/a	$ 40,400,000
Total			**95,400**	**$ 444,000,000**

Note: Smaller scale projects assumed to be 10% of the total overland improvement costs. Small scale projects are not shown in Figures 9 and 10.

Storm Drain Capacity Changes Due to Rising Tides

The 55 inch SLR study builds on the 18-inch SLR report by modeling an increase in tidal elevation for three different scenarios:

- The 10-year storm drain model with outfall elevations set to a 10-year tide elevation plus 55 inches of SLR.
- The 25-year storm drain model with outfall elevations set to a 25-year tide elevation plus 55 inches of SLR.
- The 25-year storm drain model with outfall elevations set to a 100-year tide elevation plus 55 inches of SLR.

The cost summaries from the 55-inch SLR Study analyze the increase to the CIP necessary to maintain a 10-year level of service and to maintain a 25-year level of service. Summaries of the overall increase to existing CIP costs are shown in Tables 18 and 19.

Table 18 –10-Year Storm Protection CIP Plus 55 Inches of Sea Level Rise, 2017 Costs

Location	Additional Subtotal Cost	Total Additional Cost w/ 50% Contingency
Alameda Island	$ 58,300,000	$ 87,500,000
Bay Farm Island	$ 23,300,000	$ 35,000,000
Total	**$ 81,600,000**	**$ 122,500,000**

Table 19 –25-Year Storm Protection CIP Plus 55 Inches of Sea Level Rise, 2017 Costs

Location	Additional Subtotal Cost	Total Additional Cost w/ 50% Contingency
Alameda Island	$ 74,600,000	$ 111,900,000
Bay Farm Island	$ 69,500,000	$ 104,200,000
Total	**$ 144,100,000**	**$ 216,100,000**

Conclusion

Project priorities and costs for existing and new CIPs are evaluated within this memorandum. Project priorities are updated based on City staff input and the priority from previous reports. Initial costs for the new near-term CIPs identified have been identified and costs for the existing SDMP CIPs, the report for 18 inches of Sea Level Rise, and the report for 55 inches of Sea Level Rise are all updated to September 2017 dollars. Figure 11 shows the breakdown of total pipe CIP costs with and without the addition of sea-level-rise improvements. A detailed breakdown of the costs is shown in Table 20.

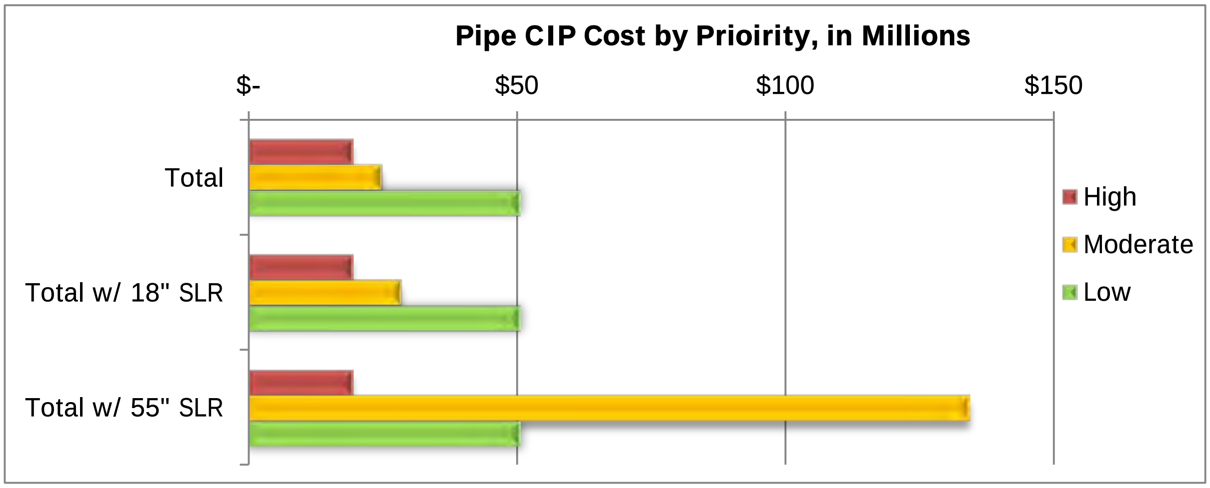

Figure 11 –Total Pipe CIP Costs by Priority, in Millions of Dollars

Table 20 – Total Costs, 2017 Costs

Category	Priority			Total
	High	Moderate	Low	
High Priority CIPs	$ 9,700,000	$ -	$ -	**$ 9,700,000**
SDMP Pipe CIPs	$ -	$ 27,100,000	$ 49,500,000	**$ 76,600,000**
SDMP Pump CIPs	$ 8,700,000	$ 4,800,000	$ 1,000,000	**$ 14,500,000**
18” SLR Pipe CIPs	$ -	$ 3,600,000	$ -	**$ 3,600,000**
55” SLR Pipe CIPs	$ -	$ 105,800,000	$ -	**$ 105,800,000**
55” SLR Inundation CIPs	$ -	$ 444,000,000	$ -	**$ 444,000,000**
10-yr Storm Total w/o SLR	$ 18,400,000	$ 31,900,000	$ 50,500,000	**$ 100,800,000**
Total w/ 18” SLR Pipe CIPs	$ 18,400,000	$ 35,500,000	$ 50,500,000	**$ 104,400,000**
Total w/ 55” SLR Pipe CIPs	$ 18,400,000	$ 141,300,000	$ 50,500,000	**$ 210,200,000**
Total w/ 55” SLR Pipe CIPs + Inundation	$ 18,400,000	$ 584,900,000	$ 50,500,000	**$ 604,100,000**

Note: 18” SLR costs do not include costs for floodwall or levee improvements

References

Climate Change Impacts to Storm Drain Improvements: Addendum to the Storm Drain Master Plan, Alameda, California. Schaaf & Wheeler, November 2009.

Draft 55-Inch Sea Level Rise Study. Schaaf & Wheeler, April 2015.

Draft South Shore Lagoon Operations Study. Schaaf & Wheeler, June 2015

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PSOMAS

Balancing the Natural and Built Environment

July 12, 2011

City of Alameda
City Hall West
950 W. Mall Square, Room 110
Alameda, Ca 94501
Attention: Barbara Hawkins, City Engineer

RE: Assessment of the City of Alameda Storm Drain Pump Stations

Dear Barbara:

We are pleased to submit four (4) bound double-sided copies of the report entitled, "Storm Drain Pump Station Assessment", dated July 12, 2011, for your review and use. We are also enclosing a disc containing an electronic copy of the report and photos taken during this study. This final report incorporates comments provided on the draft version previously submitted (dated February 28, 2011).

This report includes detailed recommendations to replace or rehabilitate the nine storm drain pump stations evaluated as part of this study. Three pump stations were given a Level 1 priority rating based on very insufficient pumping capacities (as tested and/or estimated) as compared to estimated inflows provided by Schaaf & Wheeler. These three storm drain pump stations are:

1. Arbor-recommend full pump station replacement.
2. Central/Eastshore-recommend full pump station replacement.
3. Northside-Upgrades per City of Alameda Project No. PW 02-01-06.

Please contact us should you have any questions on the data presented in this report. We are also prepared to assist the City in developing detailed design plans for each of these pump stations.

I look forward to continuing our work with you and your staff.

Sincerely,
PSOMAS

Gregory Ow, PE
Project Manager

Enclosures

165 Lennon Lane
Suite 105
Walnut Creek, CA 94598-2409

Tel 925.937.3440
Fax 925.937.3450
www.psomas.com

FINAL REPORT

STORM DRAIN PUMP STATION ASSESSMENT
CITY OF ALAMEDA

July 12, 2011

Prepared for:
CITY OF ALAMEDA, PUBLIC WORKS DEPARTMENT
950 West Mall Square, Room 110
Alameda, California 94501-7575

Prepared by:
PSOMAS
Gregory Ow
165 Lennon Lane
Suite 105
Walnut Creek, CA 94598
Project No. P6DOD08719

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1.0 Executive Summary

The City of Alameda's existing storm drain pump stations were evaluated to determine their ability to convey storm water flows from a 10-year storm event during the current 10-year high tide, as well as during the anticipated future 10-year high tide. The reliability and capacity of these stations is of the utmost importance due to the low elevations in the majority of Alameda and potential for flooding from wet weather events during high tides. Chapter 2 contains a summary of stations that were evaluated for this study, along with a summary of design criteria for each station.

Another goal of the assessment was to identify needed improvements to provide reliable, safe, and efficient pump station facilities. Psomas worked closely with Fard Engineers, and The Crosby Group to determine the electrical and structural inadequacies at each station and to identify where improvements are needed. Pump station assessment and recommended improvements are provided in Chapter 5. Photographs taken during field assessments are included in Appendix C.

Psomas worked with Pumping Efficiency Testing Services (PETS) to perform field pump testing at most of the pump stations to determine actual pump flow rates and pump head, as well as pump efficiency. PETS used the field test data to provide an analysis of where pump rehabilitation and/or replacement could result in energy cost savings for the City. The results of the pump testing are presented in Chapter 3. Pump testing data sheets are included in Appendix A.

1.1 Capital Improvement Program

Chapter 6 presents the organization of the recommended improvements developed in Chapters 3 and 5 into capital improvement program (CIP) projects. The CIP projects were prioritized into the following three categories:

- Level 1 - High Priority – defined as projects which are necessary to prevent a significant risk of flooding from heavy storm water runoff events.
- Level 2 - Necessary Projects – defined as projects that must be done to improve pump station capacity and/or reliability or safety.
- Level 3 - Discretionary Projects – defined as those that are needed in the long-term, but where the City has a significant level of control as to when they should be implemented.

1.2 Cost Evaluation

All project costs presented herein were estimated at a planning-level of accuracy (plus 50 percent to minus 25 percent). The project costs include an estimating contingency of 30 percent and an implementation cost of 55 percent. Components of the implementation cost are as follows:

Engineering Design	10 %
Construction Contingency	30 %
Project Administration / Legal & Permitting	15%
Total	55 %

For more information concerning the estimated costs for improvements at each station, refer to Appendix D.

1.3 Recommended Improvements

A summary of the recommended improvements and their associated level of pri
estimated costs are listed in Table 1-1.

do we want these numbies updated?

Table 1-1 Recommended Improvements

Station	Level of Priority	Summary of Improvements	Estimated Cost[1]
Arbor	1	Complete Pump Station Replacement. Outfall replacement. Install Standby Generator and Automatic Trash Rack.	$3,891,000
Central/Eastshore	1	Complete Pump Station Replacement. Outfall Replacement. Install Standby Generator and Automatic Trash Rack.	$2,805,000
Golf Course	2	Install Wiring in Conduit; Provide Anti-Slip Surfacing; Install Fire Extinguisher	$6,000
Golf Course	3	Install Paved Driveway, Standby Generator, and Fence Around Control Panel. Provide Site lighting and Wash Down Facilities. Coatings and Repairs.	$198,000
Harbor Bay System I	3	Provide Wash Down Facilities. Minor Structural Repair. Provide Control Panel Alarm and Fire Extinguisher.	$17,000
Harbor Bay System II	3	Replace Pump. Install Standby Generator. Replace hatches. Site Lighting and Alarms.	$257,000
Main Street	2	Install Ladder & Handrailing; Install Ventilation & Fire Extinguisher; Anchors for Control Panel & Pump Controller and Level Sensors	$84,000
Main Street	3	Install Standby Generator. Replace Hatches. Provide Station Wash Down Facilities.	$158,000
Northside	1	Replace Pumps. Other improvements per City of Alameda Northside Pump Station Upgrades Project No. PW 02-10-06.	$900,000[2]
Northside	2	Replace Pumps with New Pumps. Replace Checker Plate to Remove Trip Hazard.	$587,000
Third Street	2	Replace Hatch. Provide System for Wet Well Entry. Recoat piping. Replace Site Fence. Provide wash down facilities and Standby Generator.	$86,500

Station	Level of Priority	Summary of Improvements	Estimated Cost[1]
Webster	2	Complete Pump Station Replacement (Reuse Existing Pumps). Install Standby Generator and New Electrical Equipment.	$1,043,000

Notes:

1. Cost estimate in February 2011 Dollars.
2. Bid amount for Northside Pump Station Upgrades Project No. PW 02-10-06 per Schaaf & Wheeler. Does not include 55 % markup for related project costs.

2.0 Introduction

The City of Alameda's Storm Drain Pump Station Assessment includes nine of the City's ten storm water pump stations. The primary goal of the assessment is to ensure that the City's stations have the capacity to handle storm water flows from a 10-year storm event during the current 10-year high tide, as well as during the anticipated future 10-year high tide. The reliability and capacity of these stations is of the utmost importance due to the low elevations in the majority of Alameda and potential for flooding from wet weather events during high tides.

Another goal of the assessment is to identify needed improvements to provide reliable, safe, and efficient pump station facilities. Psomas worked closely with Fard Engineers, and The Crosby Group to determine the electrical and structural inadequacies at each station and to identify where improvements are needed. Psomas also worked with Pumping Efficiency Testing Services (PETS) to perform field pump testing at most of the pump stations to determine actual pump flow rates and pump head, as well as pump efficiency. PETS used the field test data to provide an analysis of where pump rehabilitation and/or replacement could result in energy cost savings for the City. The results of the pump testing are presented in Chapter 3. PETS pump testing data sheets are included in Appendix A.

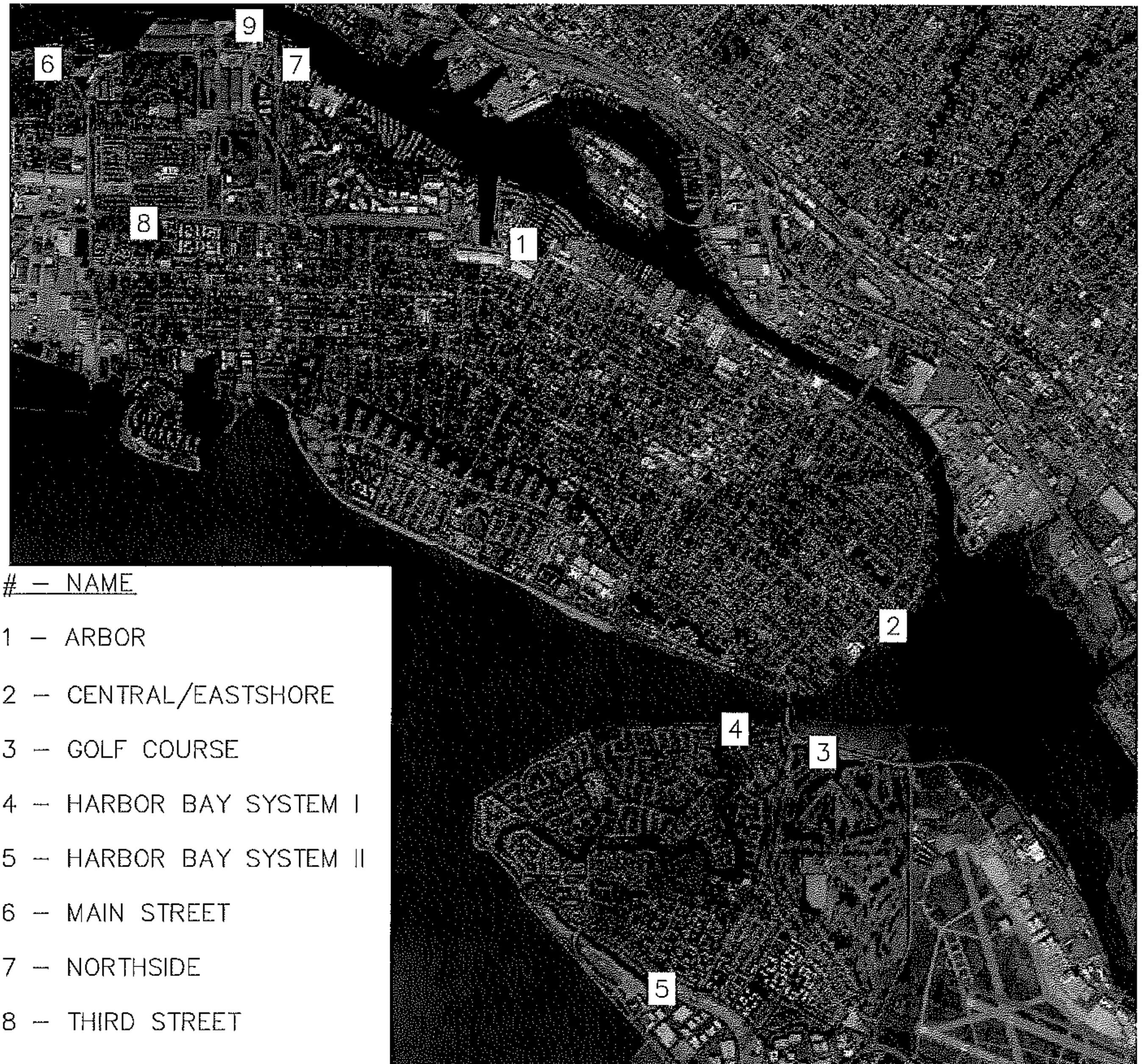
The City of Alameda's storm drain pump stations included in the assessment are:

- Arbor Pump Station
- Central/Eastshore Pump Station
- Golf Course Pump Station
- Harbor Bay System I Pump Station
- Harbor Bay System II Pump Station
- Main Street Pump Station
- Northside Pump Station
- Third Street Pump Station
- Webster Street Pump Station

The Bay Port Pump Station was not included in this assessment because it was recently renovated. According to the City, the Bay Port pump Station is in good working order. There are also privately operated pump stations located in the City that were not included in this assessment. Figure 2-1 shows the location of the pump stations included in this evaluation.

2.1 Summary of Existing Facilities

A brief summary of each pump station, including location, and station design criteria is presented herein.



— NAME

1 — ARBOR

2 — CENTRAL/EASTSHORE

3 — GOLF COURSE

4 — HARBOR BAY SYSTEM I

5 — HARBOR BAY SYSTEM II

6 — MAIN STREET

7 — NORTHSIDE

8 — THIRD STREET

9 — WEBSTER STREET

PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	PUMP STATION LOCATION MAP	FIGURE NO. 2–1 JOB NO. 6ALA010100

2.1.1 *Arbor Pump Station*

The Arbor Pump Station is located on the northern side of Alameda off of Buena Vista Avenue near the Alameda Yacht Club. The station was originally constructed in 1948 with two pumps located in a dry well. In 1994 the dry well was abandoned, and the wet well was modified for installation of four submersible pumps. Each pump discharges through individual 24-inch diameter pipes equipped with flaps gates into a discharge structure. Flow is conveyed from the discharge structure by gravity through a 54-inch outfall to the San Francisco Bay. Design criteria for the Arbor Pump Station is summarized in Table 2-1.

Table 2-1 Arbor Pump Station Design Criteria

Criteria	Value
Number of pumps	4
Type of pump	Submersible, propeller type
Manufacturer & Model	Paco Model SP16A
Pump barrel diameter (inches)	30
Pump discharge diameter (inches)	24
Pump capacity, each (gpm)	7,900(1)
Pump total dynamic head (feet)	15(1)
Total pump station capacity (gpm)	31,600(1)
Motor horsepower (hp)	40
Motor nameplate amperes (A)	53.3
Motor manufacturer	Reliance
Motor type	P
Motor frame	X320TY
Motor voltage & phase	460 volt, 3 phase
Motor revolutions per minute (rpm)	1170
Pump controls	Automatic; bubbler level control
Main Switchboard Electrical Criteria	
Enclosure type	NEMA 3R
Switchboard nameplate amperes (A)	400
Main breaker (A)	300
Power supply	480 volt, 3 phase, 4W
Inlet and Outlet Piping	
Inlet pipe diameter (inches)	54
Outfall pipe diameter (inches)	54

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.2 Central/Eastshore Pump Station

The Central/Eastshore Pump Station is located on the eastern side of Alameda in a residential area off Eastshore Drive. The pump station was originally constructed in 1967 and housed two vertical turbine type pumps. The pump station was later modified to house submersible pumps with each pump discharging through individual 16-inch diameter pipes equipped with flap gates into a discharge structure. Flow is conveyed from the discharge structure by gravity through a 21-inch outfall to the San Francisco Bay. A 24-inch pipe gate between the wet well and pump discharge structure allows water to flow by gravity from the wet well to the discharge structure in the event of pump failure or power outage. Electrical controls for the station are located on top of the structure. Design criteria for the Central/Eastshore Pump Station is summarized in Table 2-2.

Table 2-2 Central/Eastshore Pump Station Design Criteria

Criteria	Value
Number of pumps	2
Type of pump	Submersible, axial flow type
Manufacturer & Model	Paco; model unknown
Pump barrel diameter (inches)	16
Pump discharge diameter (inches)	16
Pump capacity, each (gpm)	4,300[(1)]
Pump total dynamic head (feet)	15[(1)]
Total pump station capacity (gpm)	8,600[(1)]
Motor horsepower (hp)	25
Motor nameplate amperes (A)	70.4/35.2
Motor manufacturer	Reliance
Motor enclosure	TENV
Motor type	P
Motor frame	X025QTY
Motor voltage & phase	230/460 volt 3 phase
Motor revolutions per minute (rpm)	1160
Pump controls	Automatic; bubbler level control
Inlet and Outlet Piping	
Inlet pipe diameter (inches)	(1) 27-inch and (1) 21-inch
Outfall pipe diameter (inches)	21

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.3 Golf Course Pump Station

The Golf Course Pump Station is located in the northeastern corner of Bay Farm Island at the Chuck Corica Municipal Golf Course off of Doolittle Drive. The station was originally constructed in 1985 with two vertical turbine type pumps designed to pump water as required from a collection pond at the golf course. The station was later modified to house two submersible pumps. The pump's discharges combine into a single forcemain (initially 24-inch increasing to 30-inch) prior to discharging to a storm drain manhole located across Doolittle Drive. From the manhole a 42-inch outfall conveys flow by gravity to the San Francisco Bay. Design criteria for the Golf Course Pump Station is summarized in Table 2-3.

Table 2-3 Golf Course Pump Station Design Criteria

Criteria	Value
Number of pumps	2
Type of pump	Submersible
Manufacturer & Model	Prime Pump Corporation Model 26SM14-14
Pump barrel diameter (inches)	18
Pump discharge pipe diameter (inches)	18
Forcemain diameter (inches)	24 increasing to 30
Pump capacity, each (gpm)	9,600[(1)]
Pump total dynamic head (feet)	5[(1)]
Total pump station capacity (gpm)	19,200[(1)]
Motor horsepower (hp)	50
Motor nameplate amperes (A)	67
Motor manufacturer	Reliance
Motor type	ID # P25G2712F
Motor voltage & phase	460 volt, 3 phase
Pump controls	Automatic; level control by float switches
Main Meter Panel	
Panel size (A)	600
Panel disconnect (A)	200
Power supply	277/480 volt, 3 phase, 4W

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.4 Harbor Bay System I Pump Station

The Harbor Bay System I Pump Station, constructed in 1978, is located on the north side of Bay Farm Island on Harbor Bay Lagoon. The station consists of a single vertical propeller type pump mounted on a concrete structure located adjacent to the lagoon's outfall. Normally, the lagoon level is controlled by an outfall gate structure with gates that open and close to allow water to pass between the lagoon and San Francisco Bay. The pump station is used when the lagoon level must be lowered and the tide level is equal to or greater than the lagoon level, preventing the use of the outfall gates. The pump discharges through a 24-inch forcemain to the San Francisco Bay. Design criteria for the Harbor Bay System I Pump Station is summarized in Table 2-4.

Table 2-4 Harbor Bay System I Pump Station Design Criteria

Criteria	Value
Number of pumps	1
Type of pump	Vertical Propeller
Manufacturer & Model	Prime Pump Corporation Model 20P16A-11.5
Pump discharge pipe diameter (inches)	20
Forcemain diameter (inches)	24
Pump capacity (gpm)	10,000
Motor horsepower (hp)	60
Motor manufacturer	General Electric
Motor type	K
Motor frame	C404TP16
Motor voltage & phase	460 volt, 3 phase
Motor revolutions per minute (rpm)	1170
Pump controls	Manual with low level cut-off by float switch
Main Meter Panel	
Panel size (A)	200
Power supply	277/480 volt, 3 phase, 4W

2.1.5 Harbor Bay System II Pump Station

The Harbor Bay System II Pump Station, constructed in 1991, is located on the west side of Bay Farm Island off the cul-de-sac at the end of Ratto Road. Similar to Harbor Bay System I, Harbor Bay System II is used to draw down Harbor Bay Lagoon in the event that the lagoon level cannot be lowered by the outfall gate structure because of high tides. The station is located adjacent to the lagoon's outfall gate structure and is connected via a manually operated isolation gate. The station consists of one pump that discharges through a short 16-inch forcemain into a manhole on the east side of the station. From the manhole, flow is conveyed by gravity through a 48-inch outfall to the San Francisco Bay. Design criteria for the Harbor Bay System II Pump Station is summarized in Table 2-5.

Table 2-5 Harbor Bay System II Pump Station Design Criteria

Criteria	Value
Number of pumps	1
Type of pump	Submersible Axial Flow Propeller Type
Manufacturer & Model	Prime Model M10 (w/ 12 degree impeller blades)
Pump discharge pipe diameter (inches)	16
Forcemain diameter (inches)	16
Pump capacity (gpm)	unknown
Motor horsepower (hp)	10
Motor voltage & phase	208 volt, 3 phase
Motor revolutions per minute (rpm)	1180
Pump controls	Manual with low level cut-off
Main Meter Panel	
Panel size (A)	200
Power supply	120/208 volt, 3 phase, 4W

2.1.6 Main Street Pump Station

The Main Street Pump Station is located on the northwest side of Alameda off Main Street. The station was constructed in 1998 and has three submersible pumps housed in a concrete structure that extends partially above grade. Each pump is installed in a 28-inch diameter barrel and discharges through individual 16-inch diameter pipes equipped with flap gates into a discharge structure. Flow is conveyed from the discharge structure by gravity through two parallel 28-inch diameter HDPE outfall pipes to the San Francisco Bay. Design criteria for the Main Street Pump Station is summarized in Table 2-6.

Table 2-6 Main Street Pump Station Design Criteria

Criteria	Value
Number of pumps	3
Type of pump	Submersible, axial flow type
Manufacturer & Model	Flygt; Model unknown
Pump barrel diameter (inches)	28
Pump discharge diameter (inches)	16
Pump capacity, each (gpm) (1)	4,500(1)
Pump total dynamic head (feet)	13(1)
Total pump station capacity (gpm)	13,500(1)
Motor horsepower (hp)	25
Pump controls	Automatic; PLC Controlled
Main Switchboard Electrical Criteria	
Switchboard nameplate amperes (A)	400
Power supply	277/480 volt, 3 phase, 4W
Inlet and Outlet Piping	
Inlet pipe diameter (inches)	36
Outfall pipe diameter (inches)	Two 30-inch
Automatic Trash Rack	
Manufacturer	Duperon
Model	Flexrake
Bar Spacing (inches)	2
Motor Horsepower (hp)	1/8
Year Installed	2009

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.7 Northside Pump Station

The Northside Pump Station is located on the northern side of Alameda off of Marina Village Parkway. The station was constructed in 1984 and has three vertical turbine pumps housed in a concrete structure that extends partially above grade. Each pump discharges into a common discharge box through 36-inch diameter discharge pipes equipped with flap valves. Flow is then conveyed by gravity through a 72-inch outfall pipe to the San Francisco Bay. Design criteria for the Northside Pump Station is summarized in Table 2-7.

Table 2-7 Northside Pump Station Design Criteria

Criteria	Value
Number of pumps	3
Type of pump	Vertical Axial Flow Type
Manufacturer & Model	Paco; Model unknown
Pump discharge diameter (inches)	36
Pump capacity, each (gpm) [1]	24,000[1]
Pump total dynamic head (feet)	10[1]
Total pump station capacity (gpm)	72,000[1]
Motor horsepower	75
Motor nameplate amperes (A)	137
Motor manufacturer	Reliance
Motor type	HU
Motor frame	Titan 5108-P
Motor voltage & phase	460 volt, 3 phase
Motor revolutions per minute (rpm)	505
Pump controls	Automatic; PLC Controlled (Pump Commander)
Main Switchboard Electrical Criteria	
Switchboard nameplate amperes (A)	400
Power supply	480 volt, 3 phase, 4W
Inlet and Outfall Pipe Diameter (inches)	72
Automatic Trash Rack	
Manufacturer	Duperon
Model	Flexrake
Bar Spacing (inches)	2
Motor Horsepower (hp)	1/8
Year Installed	2009

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.8 Third Street Pump Station

The Third Street Pump Station is located on the northern side of Alameda near the intersection of Third Street and Ralph Appezzato Memorial Parkway. The station was constructed in 1993 and consists of a concrete wet well with one submersible pump. The pump is located in a steel barrel and discharges through a 12-inch steel pipe with equipped with a flap gate into an adjacent storm drain manhole. Design criteria for the Third Street Pump Station is summarized in Table 2-8.

Table 2-8 Third Street Pump Station Design Criteria

Criteria	Value
Number of pumps	1
Type of pump	Submersible, axial flow type
Manufacturer & Model	Unknown
Pump discharge diameter (inches)	12
Pump capacity (gpm) (1)	1,650(1)
Pump total dynamic head (feet)	8(1)
Motor horsepower	5
Motor manufacturer	Reliance
Motor type	P
Motor frame	ID # P18G27093
Motor voltage & phase	230 volt, 3 phase
Motor revolutions per minute (rpm)	1150
Pump controls	Automatic; bubbler level control
Main Meter Panel	
Panel size (A)	100
Power supply	120/240 volt, 3 phase, 4W
Inlet pipe diameter (inches)	15

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

2.1.9 Webster Street Pump Station

The Webster Street Pump Station is located on the northern side of Alameda at the end of Mariner Square Drive. The station was constructed in 1947 and consists of a concrete structure that extends partially above grade containing a dry well on top of a wet well. The station has three submersible pumps that discharge through a 21-inch forcemain into the San Francisco Bay. To prevent backflow into the wet well under high tide conditions, each pump discharge is provided with a 10-inch Val-Matic swing-flex check valve. Design criteria for the Webster Street Pump Station is summarized in Table 2-9.

Table 2-9 Webster Street Pump Station Design Criteria

Criteria	Value
Number of pumps	3
Type of pump	Submersible, propeller type
Manufacturer & Model	Paco; Model unknown
Pump discharge diameter (inches)	10
Pump capacity, each (gpm) (1)	1,750(1)
Pump total dynamic head (feet)	10(1)
Total pump station capacity (gpm)	5,250(1)
Motor horsepower	7.5
Motor enclosure	TENV
Motor type	P
Motor frame	X0210TY
Motor nameplate amperes (A)	23
Motor manufacturer	Reliance
Motor voltage & phase	230 volt, 3 phase
Motor revolutions per minute (rpm)	1150
Pump controls	Automatic; PLC Controlled (Pump Commander)
Main Meter Panel	
Panel size (A)	200
Power supply	120/208 volt, 3 phase, 4W
Inlet and Outlet Piping	
Inlet pipe diameter (inches)	24
Forcemain diameter (inches)	21
Automatic Trash Rack	
Manufacturer	Duperon
Model	Flexrake
Bar Spacing (inches)	2
Motor Horsepower (hp)	1/8
Year Installed	2009

Notes:

1. Values for pump design capacity, total dynamic head (TDH), and station capacity provided by Schaaf and Wheeler.

3.0 Pump Station Hydraulic Capacity

Each storm water pump station's hydraulic capacity and ability to meet the design 10-year storm water flows at current 10-year high tide and future high tide was evaluated and is presented herein. Pump testing was conducted on some stations as well to compare actual pumping capacity against design pumping capacity. The method to determine the existing station's hydraulic capacity is presented first, followed by the results of the hydraulic analysis and pump testing for each station.

3.1 Storm Water Flow and High Tide Design Basis

The first step in assessing the City's storm water pump stations was to determine the required storm water volume the station must convey for the design 10-year storm event, and the ability of the pumps to convey that flow during the 10-year high tide and at the anticipated future 10-year high tide. The 10-year storm water flows were provided by Schaaf & Wheeler, and are summarized in Table 3-1. The 10-year high tide high tide level was determined from the 2008 Storm Drain Master Plan. The anticipated future high tide elevation was determined to be 18 inches above the current 10 year high tide, per "The Climate Change Impacts to Storm Drain Improvements Addendum to the Storm Drain Master Plan" prepared by Schaaf & Wheeler in 2009. The values for the 10-year high tide and anticipated future high tide were confirmed by communication with Schaaf and Wheeler and are as follows:

- The 10-year high tide elevation = 4.56 feet National Geodetic Vertical Datum (NGVD).
- The anticipated future high tide elevation = 6.06 feet NGVD (assumed to be 18 inches higher than current tide levels).

yeah, but this isn't done, right?

e 3-1 Design 10-year Storm Water Flow Influent to Each on

Pump Station	10-Year Storm Water Flow (gallons per minute)(1)
Arbor Pump Station	88,420(2)
Central/Eastshore Pump Station	19,750(2)
Golf Course Pump Station	5,390
Harbor Bay System I Pump Station	NA(3)
Harbor Bay System II Pump Station	NA(3)
Main Street Pump Station	2,560
Northside Pump Station	60,000(2)
Third Street Pump Station	2,065
Webster Street Pump Station	2,700

Notes:

1. Design 10-year storm water flows provided by Schaaf & Wheeler.
2. Design 10-year storm water flow into station following improvements to the storm water collection system and construction of a new outfall as recommended in Schaaf & Wheeler's storm drain master plan. Flow information provided by Schaaf & Wheeler.

3. NA = Not Applicable. Pumps are designed to maintain lagoon water surface levels and are not storm water pumps.

3.2 Hydraulic Analysis

3.2.1 Existing Pump Curves

A hydraulic analysis was performed for each pump station. Pump curves for all stations except Harbor Bay System I and II were created using data provided by Schaaf and Wheeler. Pump curves for Harbor Bay System I and II were developed based on record drawings and information provided from Prime Pump Corporation and the City.

3.2.2 System Curves

System curves were developed for pump discharges and forcemains (as applicable), and/or for determining outfall capacity (for gravity flow to the San Francisco Bay).

A hydraulic analysis of pump station gravity outfall piping was performed for the following stations:

- Arbor Pump Station
- Central/Eastshore Pump Station
- Golf Course Pump Station
- Main Street Pump Station
- Northside Pump Station

For the following stations, system curves were developed for pump forcemains only:

- Harbor Bay System I Pump Station
- Harbor Bay System II Pump Station
- Third Street Pump Station
- Webster Street Pump Station

The Harbor Bay System I Pump Station discharges into a small structure, which is located in the San Francisco Bay. It does not have an outfall. The Harbor Bay System II Pump Station discharges into a structure with an outfall. The outfall was not analyzed because it is a 48-inch diameter pipe, which is clearly more than adequate for the one 10 horsepower pump at the station. The Third Street Pump Station discharges into the storm drain collection system and the Webster Street Pump Station discharges through a forcemain into the San Francisco Bay.

In order to develop the system curves for each pump station, the following information was required:

- Pump static lift (determined from pump operating levels and the elevation at discharge)
- Outfall pipe length, diameter, and material
- Hazen-Williams Coefficient of Roughness (C-Value)

- Minor losses (K-Value)

3.2.2.1 Pump Static Lift

To develop system curves for the existing storm drain pump stations, the pump static lift must be determined. Static lift is the difference in suction and discharge elevation for the pump. To determine suction elevation, pump operating levels were added to the existing wet well floor elevations (obtained from record drawings) and adjusted to NGVD. Where possible, pump operating levels were determined from station pump control panels in the field. In some cases, actual operating levels were unknown, and record drawings were consulted for determining suction elevation.

The discharge elevation must be determined in one of two ways:

- For non-submerged discharges, the pump's discharge pipe centerline elevation is used for the discharge elevation.
- For submerged discharges, the discharge elevation is equal to the water surface elevation above the discharge.

For the pump stations with pump discharge structures, the water elevation at discharge is dependent upon outfall capacity. During a storm event and high tide, the water level in the pump discharge structure will rise to a level where there is sufficient head above the tide level to push incoming flow through the outfall. Water elevations in pump discharge structures were determined by calculating the head required to push incoming flows from a 10-year storm event through outfalls during the anticipated future 10-year high tide level. Table 3-2 summarizes suction and discharge elevations used for static lift. The minimum and maximum static lifts are used to determine pump and station operating flow ranges.

Table 3-2 Pump Suction and Discharge Elevations

Pump Station	Elevation Pumps On (ft NGVD)	Elevation Pumps Off (ft NGVD)	Discharge Elevation (ft NGVD)	Pump Discharge Pipe Centerline Elevation (ft NGVD)	Minimum Static Lift(1) (ft)	Maximum Static Lift(2) (ft)
Arbor	-0.4	-3.4	9.1(3)	3.8	4.2	12.5
Central / Eastshore	-1.7	-5.2	11.1(3)	4.1	5.8	16.3
Golf Course	-4.6(4)	-6.1(4)	6.2(3)	NA(5)	9.3(7)	12.3
Harbor Bay System I(8)	0.2	NA(5)	6.1(6)	NA(5)	4.4	5.9
Harbor Bay System II(8)	-0.1	NA(5)	6.1(6)	NA(5)	4.7	6.2
Main Street	-0.6	-3.1	6.7(3)	3.4	4.0	9.8

Pump Station	Elevation Pumps On (ft NGVD)	Elevation Pumps Off (ft NGVD)	Discharge Elevation (ft NGVD)	Pump Discharge Pipe Centerline Elevation (ft NGVD)	Minimum Static Lift(1) (ft)	Maximum Static Lift(2) (ft)
Northside	-0.3	-2.3	7.0(3)	4.5	4.8	9.3
Third Street	3.4(4)	-1.6(4)	3.7(9)	3.7	0.3	6.0
Webster	-3.0	-6.0	6.1(6)	NA(5)	7.2(7)	12.1

Notes:

1. Minimum static lift is equal to the pump centerline elevation minus the elevation when pumps turn on.
2. Maximum static lift is equal to the discharge elevation minus the elevation when pumps turn off.
3. Water elevation in pump discharge structure required to push incoming flows from a 10-year storm event through outfall during the anticipated future 10-year high tide level of 6.06 ft.
4. Data obtained from record drawings.
5. NA = Not applicable.
6. Discharge elevation equal to the anticipated future 10-year high tide level.
7. Because this is a forcemain, minimum static lift based on discharge water surface elevation rather than on pump discharge pipe centerline elevation.
8. Pump on elevation equal to average lagoon elevation from record drawings. Minimum static lift equal to 10-year high tide elevation minus pump on elevation. Maximum static lift equal to anticipated future high tide elevation minus pump on elevation.
9. Assumes free discharge into storm drain collection system. Discharge elevation equal to pump discharge pipe centerline elevation.

3.2.2.2 Pipe Length, Diameter, and Material

Pipe length, diameter, and material for pump discharges and forcemains/outfalls were determined at each station from the City's record drawings and field verified wherever possible. Outfall lengths could not be determined from record drawings for Arbor Pump Station and Golf Course Pump Station. For these stations, outfall lengths from Schaaf and Wheeler's model created for the 2008 "Storm Drain Master Plan" was used.

3.2.2.3 Hazen-Williams Coefficient of Roughness (C-Value)

To determine friction head loss in the system, a coefficient of roughness (C-value) must be determined for the piping. Higher C-values indicate less resistance to flow. New pipelines with cement mortar lining (CML) and cast iron pipe (CIP) with no lining are expected to have C-values in the vicinity of 120. C-values can be affected by corrosion, grease accumulation and air accumulation at high points. Pipelines in service for many years typically have C-values about 20 points below that of a new pipe. For the purposes of the hydraulic modeling, a C value of 100 was used. It was further assumed that outfall piping was clear and free of sediment accumulation for the entire length.

3.2.2.4 Minor Loss Coefficient (K-Value)

System curve development also requires that minor losses are taken into account. Minor losses occur in the system as a result of fittings in the line, changes in direction, valves, or pipe entrances or exits. The number and type of fittings and valves were determined from record drawings and verified in the field where possible.

3.3 Pump Station Hydraulic Capacity Summary

The results of the hydraulic analysis and pump testing for each station are presented in this section. The hydraulic analysis and pump testing results are compared to the design 10-year storm water flow and pump station design capacity in Table 3-3.

Table 3-3 Summary - Pump Station Hydraulic Capacity

Pump Station	10-Year Storm Water Flow (gpm)(1)	Pump Station Design Capacity (gpm)(2)	Hydraulic Model (gpm)	Pump Test Results (gpm)
Arbor	88,420	31,600	32,000	Confined space, unable to test
Central/Eastshore	19,750	8,600	8,000	5,000 gpm at 15 ft TDH
Golf Course(3)	5,390	19,200	13,000	8,000 gpm at 12 ft TDH
Harbor Bay System I	NA(3)	Unknown	12,500	13,600 gpm at 4 ft TDH
Harbor Bay System II	NA(3)	Unknown	1,500	740 gpm at 6 ft TDH
Main Street	2,560	13,500	15,750	Confined space, unable to test
Northside	60,000	72,000	59,000	51,400 gpm at 9 ft TDH
Third Street	2,065	1,650	1,800	1,600 gpm at 5 ft TDH
Webster	2,700	5,250	5,000	5,200 gpm at 10 ft TDH(4)

Notes:

1. Design 10-year storm water flows provided by Schaaf & Wheeler.
2. Station capacity obtained from the 2008 Schaaf and Wheeler Storm Drain Master Plan Report.
3. NA = Not Applicable. Pumps are designed to maintain lagoon water surface levels and are not storm water pumps.
4. One pump was out of service during testing. Pump test results assume pump that was not tested is refurbished and able to pump the average of the two tested pumps (1730 gpm at 10 ft TDH).

3.4 Arbor Pump Station

3.4.1 Pump Testing

The Arbor Pump Station pumps could not be safely tested because of limited access to discharge piping and confined space.

3.4.2 Pump Hydraulic Analysis

A hydraulic analysis of the Arbor Pump Station confirmed the design pump station capacity of approximately 32,000 gpm when all four pumps are operating (see Table 3-3). According to Schaaf and Wheeler (personal communication with Dan Schaaf), the design 10-year storm flow is estimated to be approximately 88,420 gpm (following completion of improvements to the storm water collection system leading into the pump station). Hydraulic limitations in the collection system upstream currently limit the influent flow to the station to approximately 64,600 gpm. The pump station is undersized to handle the design storm water flow of 88,420 gpm, as well as the current maximum flow of 64,600 gpm. Figure 3-1 (in Appendix B) shows required pump station capacity, pumping capacity when all 4 pumps are operational, and the system curve for the Arbor Pump Station.

3.4.3 Outfall Hydraulic Analysis

The existing 54-inch outfall and pump discharge structure were analyzed to verify capacity to handle the design storm water flow at the current 10-year high tide and future high tide.

Based on record drawings, the top of the pump discharge structure is at an elevation of approximately 6.4 feet NGVD, which would allow water to rise to an approximate elevation of 5.4 feet inside the structure. At the current 10-year high tide elevation of 4.56 feet, the existing pump discharge structure and outfall have approximately 40,000 gpm hydraulic capacity which exceeds the existing pumping capacity (32,000 gpm), but is insufficient to handle flows from the design 10-year storm (88,420 gpm). The anticipated future high tide elevation of 6.06 feet is higher than the ceiling elevation of the discharge structure (5.2 feet) which would lead to flooding in any storm event. Figure 3-2 (in Appendix B) shows the existing 54-inch outfall system curves and the water elevations within the discharge structure.

3.4.4 Recommendation

New pumps are required to convey the design 10-year storm water volume of 88,420 gpm. The outfall must be increased in size to 72-inches to convey this flow at a velocity of 7.0 feet per second or less. Figure 3-3 (in Appendix B) shows the system curves for the proposed 72-inch outfall and the elevation that the new pump discharge structure must exceed to avoid flooding. A completely new pump station and upsized outfall (72-inches) is recommended for the following reasons:

- Pumping capacity is inadequate to convey the design 10-year storm inflow.
- The existing pump barrel (30-inch) must be increased to 48-inch for the size pump required to convey the design storm water volume. The size of the station can not accommodate barrels of this size without compromising the integrity of the pump deck (only 6-inches of concrete would remain between pump barrels in the current configuration).
- The hydraulics of the pump station are not ideal. Influent enters the wet well perpendicular to the pumps and is not evenly distributed to the pumps. This can cause vortexing and/or other issues, which affects the efficiency of pump operation.
- A new discharge structure is required to accommodate the larger outfall pipe.
- Because the anticipated future high tide is higher than the top of the existing pump discharge structure, the station is subject to flooding during any storm event at the future high tide elevation.
- Additional operations and maintenance deficiencies are listed in Chapter 5.

A preliminary pump selection determined that four 90 hp pumps will be required to handle the 10-year storm water inflow. Information on the proposed 90 hp Flygt submersible propeller pumps is included in Appendix E.

3.5 Central/Eastshore Pump Station

3.5.1 Pump Testing

Results of the pump testing conducted at the Central/Eastshore Pump Station are summarized below in Table 3-4. Pump testing data sheets are included in Appendix A.

Table 3-4 Central/Eastshore Pump Testing

Pump	Flow (gpm)	Total Dynamic Head (TDH) (feet)
Pump No. 1	3,400	10
	3,000	12
	2,488	15
Pump No. 2	3,600	10
	3,051	13
	2,500	15

3.5.2 Pump Hydraulic Analysis

A hydraulic analysis of the Central/Eastshore Pump Station determined that the existing pumps should have a combined capacity of approximately 8,000 gpm which is close to the design capacity contained in the Schaaf and Wheeler 2008 Storm Drain Master Plan, but significantly less than the design 10-year storm flow of 19,750 gpm. Figure 3-4 (Appendix B) shows pump and system curves for the Central/Eastshore Pump Station when both pumps are running, as well as required station capacity.

The results of the pump testing indicate that the actual pumping rate when both pumps are operational is approximately 5,000 gpm which is significantly less than the station's design capacity. The lower pumping rate could be caused by inefficiencies in the installed system (pump and barrel configuration not compatible), or the pumps are in need of repair.

3.5.3 Outfall Hydraulic Analysis

The existing 21-inch outfall and pump discharge structure were analyzed to verify capacity to handle the incoming flow from a 10-year storm event during the current 10-year high tide and future high tide.

Based on record drawings, the top of the pump discharge structure is at an elevation of 9.6 ft NGVD, which would allow water to rise to an elevation of approximately 8.5 ft inside the structure. At the current 10-year high tide elevation of 4.56 feet, the existing pump discharge structure and outfall have approximately 10,000 gpm hydraulic capacity which is insufficient to handle the flows from the design 10-year storm (19,750 gpm). The anticipated future high tide elevation of 6.06 feet further limits the hydraulic capacity did we get new pumps?

…pm. Figure 3-5 (in Appendix B) shows the existing 21… …l the water elevations within the discharge structure.

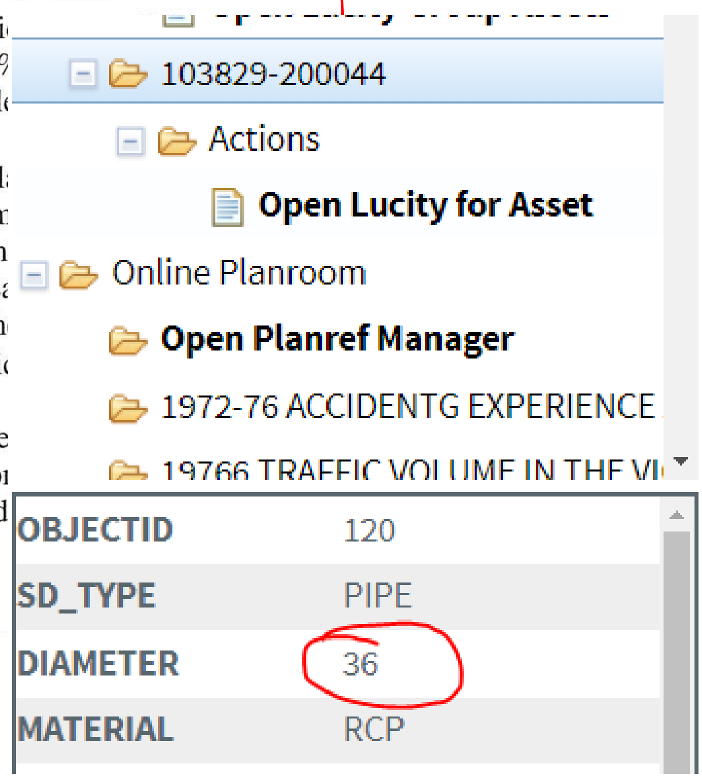
this appears to have been done

3.5.4 Recommendation

New pumps are required to convey the design 10-year storm water volume of 19,750 gpm. A new larger outfall is therefore required to convey the design storm water volume. The outfall must be increased in size to 36-inches to convey this flow at a velocity of 7.0 feet per second or less. Figure 3-6 (Appendix B) shows system curves for an upsized 36-inch diameter outfall. A completely new pump station and upsized outfall (36-inches) is recommended for the following reasons:

- Installing two new larger pumps would require major structural modifications including installing larger openings in the roof deck, pump deck (to accommodate larger pump cans), and pump station wall (to accommodate larger diameter pump discharges).
- Installing larger openings in the pump deck is not possible due to the close proximity of openings to the discharge structure and wet well below the floor slab. A complete slab removal and replacement would likely be required to accommodate the larger submersible pumps which would require removing the existing concrete stairs and manual bar rack as well.
- Installing two rather than three pumps provi… of a single pump failure (50 % versus 66.7 %… three pumps is recommended where possibl… well is too small to install three pumps.
- The existing station's manual bar screen is l… the drywell/wet well. City operations and m… desire to install an automatic trash rack at th… between the existing station structure and E… trash rack without major modifications to th…
- Additional operations and maintenance defi…

A preliminary pump selection determined that three… handle the 10-year storm water volume. Informatio… submersible propeller pumps is included in Append…



3.6 Golf Course Pump Station

3.6.1 Pump Testing

Results of the pump testing conducted at the Golf Course Pump Station are summarized below in Table 3-5. Pump testing data sheets are included in Appendix A.

Table 3-5 Golf Course Pump Testing

Pump	Flow (gpm)	Total Dynamic Head (TDH) (feet)
Pump No. 1	5,223	12
Pump No. 2	5,564	12

3.6.2 Pump Hydraulic Analysis

Hydraulic analysis of the Golf Course Pump Station pumps and forcemain system determined that the existing pumps should have a combined pumping capacity of over 13,000 gpm and can easily handle the required 5,390 gpm inflow from a 10-year storm event. Results of the pump testing are slightly lower than the flow predicted by the hydraulic model, however each pump operating alone is very near the design 10-year storm flow, and both pumps operating together provide more than enough capacity to convey the design storm water flow. Figure 3-7 (Appendix B) shows pump and system curves for the Golf Course Pump Station.

3.6.3 Outfall Hydraulic Analysis

The 42-inch outfall and discharge manhole have more than enough capacity to handle flows from a 10-year storm event during the current 10-year high tide and the anticipated future high tide. Hydraulic analysis of the outfall system shows that there will be nearly 10 ft of freeboard available in the existing discharge manhole during future high tides and the 10-year storm event. Figure 3-8 (Appendix B) shows the outfall system curves and the water level elevations required within the discharge manhole.

3.6.4 Recommendation

No improvements are necessary for pumping and/or outfall capacity at the Golf Course Pump Station. Pumps and outfall capacity are sufficient for the design 10-year storm water flow at current 10-year and future high tides.

3.7 Harbor Bay System I Pump Station

3.7.1 Pump Testing

Testing conducted for the 60 hp pump at the Harbor Bay System I Pump Station measured a pump flow rate of 13,600 gpm at a total dynamic head of 4 feet. This exceeds the flow rate predicted by the hydraulic model. Pump testing data sheets are included in Appendix A.

3.7.2 Pump Hydraulic Analysis

Hydraulic analysis determined that the existing pump has capacity for approximately 12,500 gpm during the anticipated future high tide. The actual pump flow rate of 13,600 gpm at 4 feet is close to that predicted by the model. Since the station is only used to lower lagoon levels if outfall gates cannot be used, it is not a storm drain pump station and does not need to meet capacity for a 10-year storm event. According to City operators, the capacity of the existing station is sufficient to lower lagoon levels as required. Figure 3-9 (Appendix B) shows pump and system curves for the Harbor Bay System I Pump Station.

3.7.3 Recommendation

No improvements are necessary to the pump or forcemain at the Harbor Bay System I Pump Station.

3.8 Harbor Bay System II Pump Station

3.8.1 Pump Testing

Testing conducted for the 10 hp pump at the Harbor Bay System II Pump Station measured a pump flow rate of 740 gpm at a total dynamic head of 6 feet. This is approximately half of the flow rate predicted by the hydraulic model. It was also observed that the 16-inch discharge pipe was not flowing full during the test, and therefore has excess capacity for the pump. Pump testing data sheets are included in Appendix A.

3.8.2 Pump Hydraulic Analysis

Limited information on the existing pump was available. The record drawings indicate that the pump is a 10 Hp submersible axial flow propeller pump model M10 as manufactured by Prime Corporation (see Table 2-5). Based on pump curves provided by Prime for this pump model, the pump should be capable of pumping approximately 1,500 gpm. This is significantly higher than the 740 gpm flow measured during pump testing. The difference in measured versus calculated flows could be due to the possibility that the actual pump installed differs from that indicated on the record drawings, the pump and barrel configuration are not compatible resulting in lowered efficiency, or the pump is in need of repair.

Because the pump controls the lagoon level and is not a storm drain pump, it does not need to meet capacity for a 10-year storm event. Figure 3-10 (Appendix B) shows pump and system curves for the Harbor Bay System II Pump Station.

3.8.3 Recommendation

Operators have noted the pump at this station takes a long time to affect the lagoon levels. A new pump is required to increase the pumping rate and lower the lagoon levels when necessary. A preliminary pump selection determined that a 27 hp Flygt submersible propeller pump could be used to pump approximately 6,000 gpm from the lagoon. The existing 16-inch discharge piping is adequately sized for this flow rate. Information on the proposed pump is included in Appendix E.

3.9 Main Street Pump Station

3.9.1 Pump Testing

The Main Street Station pumps could not be safely tested because of limited access to discharge piping and confined space.

3.9.2 Pump Hydraulic Analysis

Hydraulic analysis determined that the existing pumps have capacity for over 15,750 gpm and can easily handle the required 2,560 gpm incoming flow from a 10-year storm event. Figure 3-11 (Appendix B) shows pump and system curves for Main Street Pump Station.

3.9.3 Outfall Hydraulic Analysis

Hydraulic analysis was performed to verify that the two 30-inch outfalls and existing pump discharge structure have adequate capacity to handle incoming flow from a 10-year storm event during the current 10-year high tide and during the anticipated future high tide.

During the 10-year storm event and future high tide, the water level in the discharge structure would rise to an elevation only slightly higher than high tide (6.1 feet) which is well below the maximum allowable water surface elevation in the pump discharge structure of 8.8 feet (based on record drawings). Figure 3-12 (Appendix B) shows system curves for the outfalls and the water elevations required within the discharge structure to convey the 10-year storm water flow at current 10-year and future high tide.

3.9.4 Recommendation

No improvements are necessary for pumping and/or outfall capacity at the Main Street Pump Station. Pumps and outfall capacity are sufficient for the design 10-year storm water flow at current 10-year and future high tides.

3.10 Northside Pump Station

3.10.1 Pump Testing

Results of the pump testing conducted at the Northside Pump Station are summarized below in Table 3-6. Pump testing data sheets are included in Appendix A.

Table 3-6 Northside Pump Testing

Pump	Flow (gpm)	Total Dynamic Head (TDH) (feet)
Pump No. 1	19,433	6
	18,431	8
	18,825	9
Pump No. 2	18,571	6
	16,400	7
	16,200	8
Pump No. 3	18,978	7
	17,764	8
	16,578	9

3.10.2 Pump Hydraulic Analysis

Hydraulic analysis determined that the combined pumping capacity of the Northside pump station should be approximately 59,000 gpm which is less than the design station capacity of 72,000 gpm, and slightly less than the design60,000 gpm storm water flow into the station (following improvements to the storm water collection system per Schaaf & Wheeler's master plan report. Figure 3-13 (Appendix B) shows the pump and system curves when all three pumps are running, as well as the required station capacity at the Northside Pump Station. Pump testing results indicate the actual capacity of the pumps is insufficient to meet the design capacity of 60,000 gpm per Schaaf & Wheeler's storm drain master plan.

3.10.3 Outfall Hydraulic Analysis

A hydraulic analysis of the existing 72-inch outfall and pump discharge structure and verified that the existing outfall has sufficient capacity to handle flows from a 10-year storm event during current 10-year high tide and the anticipated future high tide. Figure 3-14 (Appendix B) shows the outfall system curves and the water elevations required within the pump discharge structure.

3.10.4 Recommendation

New pumps are required to bring the station capacity up to 60,000 gpm to meet the design 10-year storm water flow. No improvements are needed to the outfall or discharge structure. Preliminary pump selection provided by Schaaf and Wheeler for 75 hp Cascade pumps (Curve No. AP3612) is included in Appendix E.

3.11 Third Street Pump Station

3.11.1 Pump Testing

Results of the pump testing conducted at the Third Street Pump Station are summarized below in Table 3-7. Pump testing data sheets are included in Appendix A.

Table 3-7 Third Street Pump Testing

Flow (gpm)	Total Dynamic Head (TDH) (feet)
1,876	2
1,600	5
1,430	8

3.11.2 Pump Hydraulic Analysis

Hydraulic analysis of the station determined that the existing pump has capacity for 1,800 gpm to 2,000 gpm depending upon operating levels in the wet well. This capacity is slightly below the 2,065 gpm incoming flow from a 10-year storm event. Figure 3-15 (Appendix B) shows pump and system curves for the Third Street Pump Station. Pump testing results indicate the actual capacity of the pump is slightly less than the design capacity shown in Table 2-8 (1,650 gpm each at 8 feet TDH).

3.11.3 Recommendation

Both the pump testing and the hydraulic analysis determined that the existing pump capacity is very near the inflow from the 10-year storm event, though slightly insufficient. Because the pump nearly meets the capacity requirements, which are relatively low at this station, the replacement of the existing pump is currently a low priority.

3.12 Webster Street Pump Station

3.12.1 Pump Testing

Results of the pump testing conducted at the Webster Street Pump Station are summarized below in Table 3-8. Pump 1 was under repair at was not at the station at the time of testing. Pump testing data sheets are included in Appendix A.

Table 3-8 Webster Pump Testing

Pump	Flow (gpm)	Total Dynamic Head (TDH) (feet)
Pump No. 2	1,900	6
	1,874	8
	1,800	10
Pump No. 3	1,900	6
	1,779	8
	1,670	10

3.12.2 Pump Hydraulic Analysis

Hydraulic analysis of the station determined that the existing pumps have capacity to pump approximately 5,000 gpm at the current 10-year high tide level which exceeds the required 10-year storm water flow of 2,700 gpm. At the future high tide, the operating levels for the pumps may need to be adjusted to keep the pumps operating within the allowable range on the pump curve. Figure 3-16 (Appendix B) shows pump and system curves for the Webster Street Pump Station. Pump testing results indicate the actual combined pumping for the pump station should be close to the design capacity shown in Table 2-9 (5,250 gpm each at 10 feet TDH), assuming the pump under repair will pump the average of the two tested pumps following its repair (1730 gpm at 10 ft TDH).

3.12.3 Recommendation

Although the pumps and forcemain capacity are sufficient for the design 10-year storm water flow at current 10-year and future high tides, a completely new pump station is recommended due to operations and maintenance and safety deficiencies at the station (see Chapter 5).

The existing pumps could be reused with the new station. The new station should be designed to improve hydraulics. Currently, influent enters the wet well perpendicular to the pumps and is not evenly distributed to the pumps. Also, submersible propeller pumps are typically are not designed to combine flow into a common forcemain. The station in its current configuration may be causing vortexing that could reduce pump efficiency. The new station should be designed for hydraulics that better suit submersible propeller pumps.

4.0 Evaluation Criteria

Pump station evaluation criteria was developed by Psomas, Fard Engineers, The Crosby Group, and the City's operations and maintenance staff to address needed station improvements. Each pump station was evaluated in the following areas:

- Reliability and Redundancy
- Electrical and Instrumentation
- Operations and Maintenance
- Structural
- Site Security

4.1 Reliability and Redundancy

Designing the storm drain pump stations with enhanced reliability and pumping redundancy minimizes the probability of flooding resulting from equipment failure. Equipment reliability is required to reduce unscheduled maintenance. The following outlines design features that increase station reliability:

- Newly constructed storm drain pump stations should be provided with a minimum of three pumps, if possible, to minimize capacity lost from pump failure. Unlike sewer pump stations, storm water pump stations are not normally designed with redundant (or standby) pumping systems. Therefore, the more pumps provided to meet the design capacity, the less the impact on the station's overall capacity should any single pump fail to operate or happen to be off-line for maintenance.
- Sump pumps in dry wells (Only for dry well/wet well pump stations).
- Monitoring, SCADA, and alarms for equipment failures and out of range process conditions.
- Uninterruptible power supply for instrumentation, controls, and telemetry.
- Spare parts.
- Redundant critical auxiliary equipment (equipment essential to the operation of the pump station).
- Proven equipment from respected manufacturers.
- Standby power provided (either a permanent standby generator and/or a generator receptacle at the site). Permanently installed generators to be provided with weatherproof enclosure, a subbase diesel fuel tank sized for a minimum of 24 hours under full load, and an automatic transfer switch (ATS). An automatic transfer switch is an electrical switch that automatically reconnects electric power source from its primary source to a standby source. If the utility source fails, the ATS safely switches to the generator as a temporary source of electric power. The ATS will also command the backup generator to start.

4.2 Electrical and Instrumentation

Each pump station's electrical infrastructure and instrumentation systems were evaluated to determine if they meet the following criteria:

- Electrical design and installation based on the latest version of the National Electrical Code (NEC).
- Site provided with adequate lighting.
- Enclosures, components, anchors, and other hardware constructed of corrosion resistant material (Type 316 stainless steel) and/or provided with corrosion resistant coatings.
- Electrical service equipment housed in NEMA rated Type 316 stainless steel enclosures and/or provided with corrosion resistant coatings.
- Pump motors designed to match electrical service available to the pump station.
- Circuit breakers provided with lock-out mechanisms. Local equipment disconnects provided where needed.
- Motors provided with thermal sensors and where submersible also provided with moisture detection systems.
- Where corrosive or wet conditions prevail, conduit, fittings and boxes constructed of PVC coated rigid steel. Where possible, avoid installing junction boxes (or any type of connection or termination points) within wet wells or below high water levels.

  Pump stations provided with PLC control with remote status and alarm indication.
- Pump stations provided with high level alarms for remote indication, and low level cut-off to prevent pumps from running dry or at levels that may cause the pumps to cavitate.
- Pump level sensing accomplished by bubbler level monitoring systems.

4.3 Operations and Maintenance

Each pump station was evaluated from an operations and maintenance perspective (i.e., a user friendly pump station that can be safely, easily, and effectively operated and maintained). The following items were considered:

- Equipment access and clearances; ease of pump removal
- Lightweight and easy to open access hatches; ladders provided with ladder-up assistance.
- Site access (fuel trucks, cranes, parking, etc.).
- Automatic trash racks.
- Wash down facilities provided.
- Spare parts.
- Safety (tripping hazards).
- Ventilation; confined space entry.
- Handrails provided where needed (fall protection).
- Proper signage.
- Ladders meet OSHA requirements.
- Fire extinguishers provided.

4.4 Structural

Each pump station structural components were inspected with recommended improvements based on visual observation of areas where concrete has spalled, cracked, or where obvious signs of corrosion were observed.

4.5 Site Security

Each site was reviewed with respect to site security. Fencing and intrusion alarms were reviewed for each pump station including:

- Site fencing provided with barbed wire, slats, and pad lockable gates.
- Intrusion alarms provided on all access hatches and control panels.

5.0 Pump Station Evaluations & Recommendations

Each pump station was reviewed by Psomas, Fard Engineers, and The Crosby Group based on the evaluation criteria outlined in Chapter Four. Psomas and sub consultants visited each site and met with the City's operations & maintenance staff to discuss their concerns and inadequacies at each station. Results of the site evaluations and recommendations are presented herein.

5.1 Arbor Pump Station

A summary of the Arbor Pump Station evaluation and recommended improvements is presented in Table 5-1.

Table 5-1
Arbor Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	The existing pump capacity is insufficient to handle the flow from a 10-year storm.	Provide new pump station (see Chapter 3).
	The existing 54-inch outfall does not have capacity to handle the flow from a 10-year storm at the current 10-year high tide or anticipated future high tide.	Provide new 72-inch outfall pipe (see Chapter 3).
Reliability and Redundancy	The station does not have standby power capabilities.	Provide new standby power generator. A 400 kilowatt (KW) generator was assumed for estimating purposes.
	The 30-inch pump barrel for pump 1 was recently replaced. Barrels for the other three pumps are corroded and in poor condition.	New pump barrels provided with new pump station. Inspect and salvage 30-inch barrel for future City projects.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	An electrical junction box located between pump 3 and 4 makes access for maintenance purposes difficult. The box must be relocated in order to remove pump barrels.	Provide better access in design for new pump station.
	The existing pump controllers are old and it is hard to find spare parts.	Provide new pump "Vision" or pump commander controls with new pump station. Include electronic pressure transducers/ level transmitters to replace existing bubbler type level sensors.
	The main switchboard has inadequate clearance in front of it per NEC (< 42 inches).	Design new pump station's electrical switchgear to have the required minimum clearances per NEC.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
	There are no exterior light fixtures or site lighting.	Design new station with exterior lighting.
	Existing 400 amp electrical service is insufficient for the proposed new pumps.	Provide new electrical service for new pumps and equipment. A 600 amp 480 volt service was assumed for estimating purposes.
Operations and Maintenance	Access to piping and pump barrels within the wet well is difficult and considered a confined space. One of the two hatches is difficult to open, and only one ladder is provided to enter the wet well.	Design new pump station to eliminate the need for confined space entry to access pumps and associated piping/instrumentation.
	The galvanized steel access hatch above the trash rack is heavy and requires two operators to lift. The grating located over the pumps is constructed of steel and is also heavy and difficult to remove.	Design new pump station with aluminum hatches with lift assist that can be easily opened.
	There is currently a manual trash rack that requires confined space entry. The rack is very labor intensive to clean.	Design new pump station with an automatic trash rack that dumps trash into dumpsters located above grade.
	The pump station does not have wash down facilities.	Design new pump station with backflow preventer and hose bib for station wash down.
	Ladders into the pump discharge box and the wet well do not have ladder up assistance.	Design new pump station with ladder up assistance provided on all ladders.
	There is currently no ventilation within the wet well or at the trash rack. The existing fan is not functional.	Design new pump station to eliminate confined space entry and/or with adequate ventilation to eliminate confined space hazards.
	Missing a fire extinguisher.	Provide fire extinguisher in design for new pump station.
Structural	There is an existing concrete pad that is cracked at two locations, and is not wide enough.	Provide new pump station.
Site Security	Access to the station is shared with the Alameda Yacht Club through a gate in the 6-foot tall wooden fence. The gate on this fence is often left open and allows access to the station's control panel and the Alameda Yacht Club storage area.	Design new pump station to have controls located integrally within the pump station site. Provide a PVC lined chain link fence with slats and barbed wire on all sides of the pump station.
	There are no alarms on hatches at the station.	Design new station to have intrusion alarms provided on all access points to the station.

5.2 Central/Eastshore Pump Station

A summary of the Central/Eastshore Pump Station evaluation and recommended improvements is presented in Table 5-2.

Table 5-2 Central/Eastshore Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	The existing pump capacity is insufficient to handle the flow from a 10-year storm.	Provide new pump station (see Chapter 3).
	The existing 21-inch outfall does not have capacity to handle the flow from a 10-year storm at the current 10-year high tide or the anticipated future high tide.	Provide new 36-inch outfall pipe (see Chapter 3).
Reliability and Redundancy	The station does not have standby power capabilities.	Provide new standby power generator, capable of powering new pumps. A 250 kilowatt (KW) generator was assumed for estimating purposes.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	New electrical and instrumentation will be required for new pump station.	Provide new electrical and instrumentation as required for new pump station. Provide new pump "Vision" or pump commander controls with new pump station. Include electronic pressure transducers/ level transmitters to replace existing bubbler type level sensors.
	Existing 200 amp electrical service is insufficient for the proposed new pumps and equipment.	Provide new electrical service for new pumps and equipment. An 800 amp 3 phase service was assumed for estimating purposes.
Operations and Maintenance	The galvanized steel access hatches above the pumps and discharge box are heavy and difficult to open.	Design new pump station with aluminum hatches with lift assist that can be easily opened.
	The trash rack must be manually cleaned.	Design new pump station with an automatic trash rack.
	The pump station does not have wash down facilities.	Design new pump station with wash down facilities.
	There is currently no ventilation within the wet well. Ventilation is provided by an open grate near the dry well access hatch and from the entry hatch into the wet well.	Design new pump station to eliminate entry into the wet well (for access to trash rack) and/or with adequate ventilation to eliminate confined space hazards.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Structural	Handrail anchorage at the base of the stairs is causing the concrete to spall.	New pump station will correct this deficiency.
Site Security	There is currently no fencing around the site.	Provide a PVC coated chain link fence with slats, barbed wire, and pad lockable gates.
	Station lacks intrusion alarms.	Design new station to have intrusion alarms provided on all access points to the station.

5.3 Golf Course Pump Station

A summary of the Golf Course Pump Station evaluation and recommended improvements is presented in Table 5-3.

Table 5-3 Golf Course Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	No deficiencies.	None.
Reliability and Redundancy	Missing standby power capabilities.	Provide new standby power generator. A 125 kilowatt (KW) generator was assumed for estimating purposes.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Exposed wiring around pumps poses a safety hazard.	Reinstall wiring in rigid PVC coated steel conduit.
	Missing exterior site lighting.	Provide exterior pole lighting with either photocell and time clock control or manual controls from electrical panel.
	Metal cover over level sensor is severely corroded.	Replace cover with type 316 stainless steel cover.
Operations and Maintenance	Missing wash down facilities.	Install wash down facilities.
	Missing a driveway to access the station. Access currently requires staff to cross approximately 50 feet of grass which can not be driven on in the winter due to muddy conditions. Crane access to station for removing and/or installing pumps is hampered by the lack of a driveway.	Provide a paved driveway to access the station. Concrete encase the existing shallow water line as required to prevent damage from traffic from the new paved section.
	Existing exposed piping coating is chalked and cracking and piping is showing signs of corrosion. The 18-inch check valve on Pump No. 2's discharge piping is leaking.	Re-coat piping. Replace or repair check valve as required to eliminate leakage.
	The station floor is often slippery because of moss. This may be partly caused by the constant presence of moisture due to the leaking check valve.	Provide an anti-slip layer of grout or concrete coating. Fix the leaking check valve as noted above.
	Missing a fire extinguisher.	Install a fire extinguisher.
Structural	No deficiencies noted.	None.
Site Security	Missing fence around control panel.	Provide chain link fence with slats around the control panel.

5.4 Harbor Bay System I Pump Station

A summary of the Harbor Bay System I Pump Station evaluation and recommended improvements is presented in Table 5-4.

Table 5-4 Harbor Bay System I Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	No deficiencies.	None.
Reliability and Redundancy	A generator receptacle for a portable generator to power the outfall gates and the pump station was recently installed at this site.	None.
	Pump was recently reconditioned.	None.
	The pump at this station is only used to lower the lagoon level under high tide conditions. Normally level is controlled by opening and closing the outfall gates.	None. Because the pump station is a backup to the outfall gate system, a redundant pump is not needed. The City could use a portable pump in the event the station's pump failed and the lagoon level needed to be lowered.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Missing exterior site lighting.	Since telemetry was recently added, Operators control the station from the corporate yard and are rarely at the site after dark. Site lighting is not a high priority improvement.
	Telemetry was recently added to allow pump control from the City's corporation yard.	None.
Operations and Maintenance	Missing wash down facilities.	Install wash down facilities.
	Missing a fire extinguisher.	Install a fire extinguisher.
Structural	The outfall gate structure has minor deterioration in the structural system including minor cracks developing at the cantilever section of the slab.	Inject cracks with epoxy to prevent water intrusion.
	The concrete has spalled off at the corner of the slab at the fence post connection.	Patch corner as required with concrete and re-attach fence post.
Site Security	Missing intrusion alarms at the station.	An intrusion alarm should be added to the pump control panel.

5.5 Harbor Bay System II Pump Station

A summary of the Harbor Bay System II Pump Station evaluation and recommended improvements is presented in Table 5-5.

Table 5-5 Harbor Bay System II Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	Although the pump is used as a backup to the outfall gates, the pump station capacity is inadequate and can take several hours to noticeably lower the lagoon level when use of the pump is required.	This pump station is not a storm water station and does not need to be upgraded to handle storm water. However, the City should upgrade to improve the dewatering time on the lagoon.
Reliability and Redundancy	Missing standby power capabilities.	Provide new standby power generator. A 35 kilowatt (KW) generator capable of powering the proposed pump was assumed for estimating purposes.
	The pump at this station is only used to lower the lagoon level under high tide conditions. Normally level is controlled by opening and closing the outfall gates.	None. Because the pump station is a backup to the outfall gate system, a redundant pump is not needed. The City could use a portable pump in the event the station's pump failed and the lagoon level needed to be lowered.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Missing exterior site lighting.	Provide exterior pole lighting with either photocell and time clock control or manual controls from electrical panel.
Operations and Maintenance	Missing wash down facilities.	Install wash down facilities.
	The steel access hatch over the discharge vault is heavy and difficult to open.	Operators rarely need to access the discharge manhole, however an aluminum hatch with lift assist could be added to improve accessibility.
	The removable reinforced steel plates on the pump check valve vault are heavy and very difficult to remove. Also, the rectangular plates could fall into the vault and damage the piping or vault in the event of an operator error while removing/placing the plates.	Operators rarely are required to access the valve vault, but the steel plates could be replaced with an aluminum hatch with lift assist to improve accessibility. This would eliminate the possibility of dropping the plates into the vault.
Structural	There is rust developing on the pump barrel base plate.	Recoat barrel base plate.
Site Security	Missing intrusion alarms at the station.	An intrusion alarm should be added to the pump control panel.

5.6 Main Street Pump Station

A summary of the Main Street Pump Station evaluation and recommended improvements is presented in Table 5-6.

Table 5-6 Main Street Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	No deficiencies.	None.
Reliability and Redundancy	Missing standby power capabilities.	Provide new standby power generator. An 80 kilowatt (KW) generator was assumed for estimating purposes.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Clearance from the pump control panel to the pump discharge box wall is approximately 39 inches. A minimum of 42-inches is required per NEC.	Due to the cost that would be required to move the control panel a few inches, this is a low priority improvement.
	Pump controls are by Tesco. Controller is the only one of it's kind in the City, and staff are not sure how to operate it. Existing bubbler level controls are outdated and in need of replacement.	Provide new pump "Vision" or pump commander controls to be consistent with other stations in the City. Include electronic pressure transducers/ level transmitters to replace existing bubbler type level sensors.
Operations and Maintenance	Missing wash down facilities.	Install wash down facilities.
	Plates over pumps are large and heavy and must be removed with a crane.	Replace with aluminum hatches with lift assist. This is a low priority improvement as the plates are infrequently removed and only for pump removal/replacement.
	Missing steps and/or ladder access to the top of the pump structure. The top of the pump structure is approximately 42-inches above grade in areas.	Provide ladder or steps for access to the top of the structure.
	Hand railing around the automatic trash rack blocks access between the pump side and the trash rack side of the structure. Operators must climb down and then climb back up to go from one side to the other.	Provide ladder or steps to improve access around the trash rack.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
	The top of the station structure is approximately 42-inches above grade and lacks fall protection.	Provide aluminum handrail around top of structure.
	There are currently two vent pipes with "U" bends on top of the station to ventilate the wet well, but there are no ventilation fans.	Operators are rarely required to enter the wet well at this station, but a fan should be added to ventilate the wet well when access is required.
	Missing a fire extinguisher.	Install a fire extinguisher.
Structural	Control panel anchors appear to be missing.	Provide anchors as required per seismic design calculations.
Site Security	No deficiencies.	None.

5.7 Northside Pump Station

A summary of the Northside Pump Station evaluation and recommended improvements is presented in Table 5-7.

Table 5-7 Northside Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	The existing pumps are undersized to handle incoming flow from a 10-year storm event during the anticipated future high tide.	Replace pumps with new pumps with adequate capacity to handle incoming flow from a 10-year storm (see Chapter 3).
Reliability and Redundancy	Missing standby power capabilities.	The City is in the process of adding a 230 kW standby power generator as part of the Northside Pump Station Upgrades Project No. PW 02-10-06.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	This pump station is currently being renovated for new electrical service and standby generator.	New electrical equipment, conduits, and lighting are provided as part of the Northside Pump Station Upgrades Project No. PW 02-10-06.
Operations and Maintenance	The control room access hatch is difficult to close. If closed improperly, the intrusion alarm will remain on.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06.
	The top deck of the pump station is approximately 62 inches above grade and is missing handrail. Handrail is required per OSHA for surfaces 42-inches or higher above surrounding grade and where Operator access is required.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06..
	Lifting rings on checker plates at the top of the structure protrude above the surface of the plate and pose a tripping hazard.	Checker plate with lifting rings over electrical room removed as part of the Northside Pump Station Upgrades Project No. PW 02-10-06. Remaining checker plate over pumps should be replaced with new plate that has handles or rings that are flush with the surface to remove tripping hazard.
	Existing ventilation fan is not working.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06.
	There is a hose bib, but no rack or hose.	Install hose rack and hose.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Structural	The existing grating is corroded, and has a hole in it that has been covered up with a sheet of plywood.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06.
	The edge of the concrete deck is spalling at the back end of the pump station (above the steel grate).	Repair spalled concrete surfaces to prevent further deterioration of the structure. Should be included as part of the Northside Pump Station Upgrades Project No. PW 02-10-06 during installation of new FRP grating.
	The handrail and handrail baseplates are corroded, and there are several locations where the concrete has spalled at handrail anchorage locations.	Recoat handrail and baseplates, and repair spalled concrete surfaces to prevent further deterioration of the structure. Should be included as part of the Northside Pump Station Upgrades Project No. PW 02-10-06 during installation of new handrailing.
	There are several locations on the structure that exhibit rust staining that may be a result the steel reinforcement rusting within the concrete wall.	Repair cracks and seal at locations where rust staining is occurring.
Site Security	The existing fence is beginning to corrode and will need to be replaced in the near future.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06.
	An intrusion alarm is provided on the control room hatch, but not on pump entrance hatch or gates.	Addressed in the Northside Pump Station Upgrades Project No. PW 02-10-06.

5.8 Third Street Pump Station

A summary of the Third Street Pump Station evaluation and recommended improvements is presented in Table 5-8.

Table 5-8 Third Street Pump Station Evaluation and Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	The existing pump capacity is very near the inflow from the 10-year storm event, though slightly insufficient.	Because the pump nearly meets the capacity requirements, which are relatively low at this station, replacement of the existing pump is currently a low priority.
Reliability and Redundancy	Missing standby power capabilities.	Provide new standby power generator. A 10 kilowatt (KW) generator was assumed for estimating purposes. A Tier 4 rating is required due to the close proximity to the Wood Stock Education Center.
	No redundant pumping system. The existing manhole is not large enough to accommodate a second pump. In the event of pump failure, flooding may occur in the area. The station is located near the City's existing storm drain collection system which may limit the extent of the flooding in the event the station's pump failed.	The City could replace the existing station with a larger station capable of accommodating redundant pumping systems. However, with the low capacity requirements, a more economical option may be to have a portable pump that could handle capacity requirements in the event that the station's pump fails.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Clearance around main service meter meets code but is very tight.	None.
	The pump control panel contains control relays are old and fail often. The City has recently replaced the pump controls with the "Vision" or pump commander controls and has replaced the bubbler level controls with new electronic pressure transducers/ level transmitters.	None.
	The pump cables are exposed and are not installed in conduit or MC cable for proper protection.	Reinstall pump cables in conduit per NEC requirements.
Operations and Maintenance	There is no trash rack at the station. Floating debris must be manually removed from this station.	Due to the relatively small flows present at this station, the cost to install a new manual or automatic trash rack upstream of the station over periodic manual cleaning may not be economically justifiable.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
	The piping within the wet well is corroded.	Remove and recoat piping. Replace if necessary.
	The existing flap gate at the discharge into the manhole is stuck in the open position.	Replace flap gate on pump discharge.
	The hatch for access to the ladder is only 18 inches deep and does not meet OSHA clearance requirements for ladder access. There is also no ladder up assistance on the ladder.	Install a larger access hatch that provides 24 inches clearance behind ladder rungs to meet OSHA requirements. Hatch should include a ladder up safety post.
	Removal of the pump barrel requires confined space entry into the wet well. There is insufficient room inside the fenced area at the station for a portable tripod to lower someone into the wet well. This presents an additional safety hazard for confined space entry.	Install a stainless steel sleeve for mounting a portable davit arm next to the wet well access hatch. A permanent davit arm could also be installed but is not recommended as it would further limit space within the fenced area.
	Missing a fire extinguisher.	Install a fire extinguisher.
Structural	The electrical control panel appears to be inadequately supported to the concrete base.	Provide new connection for the electrical box to the concrete base.
Site Security	The existing fence located above the station is in poor condition and needs to be replaced.	Provide new PVC coated chain link fence with barbed wire and slats.

5.9 Webster Street Pump Station

A summary of the Webster Street Pump Station evaluation and recommended improvements is presented in Table 5-9.

Table 5-9 Webster Street Pump Station Evaluation & Recommendations

Evaluation Criteria	Evaluation Summary	Recommended Improvement
Pumping Systems and Capacity	Although the pumps meet capacity requirements, the hydraulics of the pump station are not ideal for submersible propeller pumps.	A completely new pump station is recommended due to operations and maintenance and safety deficiencies at the station. Provide new pump station and reuse existing pumps. New station should be designed for hydraulics that better suit submersible propeller pumps (see Chapter 3).
Reliability and Redundancy	Missing standby power capabilities.	Provide new standby power generator. A 25 kilowatt (KW) generator was assumed for estimating purposes.
SCADA System	SCADA system is in overall good condition.	Perform regular upgrades to the Wonderware system and radio communication upgrades to digital per Section 5.10.1.
Electrical and Instrumentation	Missing exterior site lighting. Pump station bubbler controls and pump controller are aging and in need of replacement.	Design new station with exterior lighting. Provide new pump "Vision" or pump commander controls with new pump station. Include electronic pressure transducers/ level transmitters to replace existing bubbler type level sensors.
Operations and Maintenance	The existing piping in the dry well is showing signs of corrosion.	New piping provided with new station.
	The existing fan located inside the dry well is no longer operational.	Design new pump station to eliminate entry into the wet well.
	The hatch for entry into the dry well is awkwardly positioned in the corner of the station structure. It is a double leaf hatch that opens to the sides of the ladder, making it difficult to access the ladder into the dry well. The hatch does not meet OSHA requirements for clearance behind a ladder, which must be at least 24 inches from the center of the ladder rungs.	Design new pump station without a dry well and eliminate the need for entry into the wet well.

Evaluation Criteria	Evaluation Summary	Recommended Improvement
	The ladder that provides access to the top of the structure from the sidewalk is located directly in front of the dry well access hatch. The hatch position makes access to the top of the structure dangerous when the hatch is open.	Design new pump station so that structure is only approximately 12 inches above grade, eliminating the need for a ladder.
	The ladder for access into the dry well is located directly above the opening to the wet well below. The opening is currently not covered with grating or treadplate presenting a safety hazard for staff entering the pump station.	Design new pump station without a dry well and eliminate the need for entry into the wet well.
	The clearance behind the center of the ladder rungs to the opening in the wet well below is less than the required 24 inches per OSHA.	Design new pump station without a dry well and eliminate the need for entry into the wet well.
	Missing a fire extinguisher.	Provide a fire extinguisher with new station.
Structural	Exposed rebar at the manhole entrance is showing signs of rust at a few locations.	New pump station will correct this deficiency.
	The existing ships ladder appears to be rusted.	New pump station will correct this deficiency.
Site Security	There is currently no fencing around the site.	Provide a PVC coated chain link fence with slats, barbed wire, and pad lockable gates. Design new station to have intrusion alarms provided on all access points to the station.

5.10 SCADA System

5.10.1 Existing SCADA System

The SCADA system which monitors the City's storm water pump stations is made up of four components.

1. The control center: The control center is a SCADA software product called Wonderware, which is running on a windows based personal computer. Wonderware sends and receives data from the pump station controllers. It then displays this information to the operation's personnel on a graphical interface.
2. The communication method: The control center SCADA computer communicates with the pump stations using a licensed 900MHz Microwave Data System (MDS) analog radio system. The radios are currently being replaced with dual mode (analog/digital) radios. All radios will continued to be run in analog mode until they are all replaced, at which time all the radios will be converted to digital mode.
3. The pump station controller: There are currently two controllers at each pump station. A Motorola Moscad which handles the communication with the radio, and a controller to monitor/control the pumps and associated instrumentation. The connection between these two controllers is through their inputs and outputs (I/O). Any piece of information, for example pump run confirm, requires a separate I/O pair (output of one controller feeds the input of the other) to be wired together in order to communicate that information between the controllers, and then back to the control center. The City is in the process of upgrading the controllers (the Non-Moscad controller) with the "Pump Vision" controller. It is a Unitronics V350 Controller which is prepackaged as a "ready to go" pump station controller supplied by California Motor Control (CMC). It comes with standard pump control software and operator interface screens.
4. Instrumentation: The primary data monitored at the pump station is the wet well level. The pump station controller operates the pumps based on this level. The City is replacing the "bubbler type" level sensors with "electronic pressure transducers/ level transmitters" level sensors. The bubbler systems are outdated and have significant parts that could fail, making their reliability less than desirable.

5.10.2 Assessment

The City's SCADA system is in overall good condition. The physical condition of the control equipment is good; such as the pump station controllers, control enclosures and instrumentation. The antennas are mounted well and are also in good condition.

The City is in the process of standardizing their pump station controllers and level transmitters which will help Operators become familiar with the station controls and make system operation more reliable.

The most important observation is the use of two controllers at each storm water pump station which minimizes flexibility and adds costs to the operation of the stations. The City is not using the features of the Moscad Controller. All the pump logic and operator interaction is done with the "Pump Vision" Controller. Having the controllers in this configuration prevents the control center SCADA computer from accessing or changing all the control information that is in the "Pump Vision" controller, without installing more wired connections. Information such as:

- Viewing and changing the pump start/stop level set points.
- Changing the pump status (ON/OFF/AUTO).
- Viewing all the alarms monitored or derived by the controller.

5.10.2.1 Recommended Improvements:

The recommended improvements for the City of Alameda's SCADA system are outlined below.

- The City's existing Wonderware system meets the City's needs as long as the software and hardware are upgraded at intervals to remain supported by the vendor.
- The City's radio replacements should be completed and converted to digital. The City should prepare to split the frequency now before it's required by the FCC (in a year or two). Having two frequencies will improve the SCADA system performance. Once the split is made, the City should use both frequencies in their system so as not to lose one since frequencies in the 952/928 MHz range for data communication are extremely difficult to obtain.
- The City's level sensor replacements should be completed and the remnants of the bubbler systems should be removed.
- The City's upgrade to the "Pump Vision" controllers should be completed, to eliminate the very outdated pump station controllers which are at the end of their useful life.
- The City should prepare a SCADA System Strategic Plan. The City has varied needs for all the sites the SCADA system monitors. The best way to address these needs is to have a strategic plan so when changes are made they move toward an efficient, integrated system. The Strategic Plan should address the issues of:
    - Future FCC requirements
    - Future operational needs
    - The added complexities to monitoring and control caused by having two controllers at each station.

6.0 Capital Improvements Program (CIP)

The overall goal of the storm drain pump station assessment was to develop a plan for the City to fund necessary improvements to the City's storm drain pump stations. This chapter presents the improvements that are recommended for incorporation in the City's Capital Improvements Program (CIP). A prioritization guidelines and summaries of proposed CIP projects are presented herein.

6.1 CIP Prioritization Guidelines

Prioritizing projects allows the City to determine which projects should be funded within the CIP planning time-frame, as well as the order in which projects should be pursued within that time frame. The prioritization was focused on identifying projects that should be funded in the short-term (high priority), those that should be implemented to improve pump station reliability, capacity, or safety (necessary projects), and those that should be implemented as funding becomes available (discretionary projects).

The projects are prioritized into three main categories (in order of high priority to low priority):

- Level 1 - High Priority – defined as projects which are necessary to prevent a significant risk of flooding from heavy storm water runoff events.
- Level 2 - Necessary Projects – defined as projects that must be done to improve pump station capacity and/or reliability or safety.
- Level 3 - Discretionary Projects – defined as those that are needed in the long-term, but where the City has a significant level of control as to when they should be implemented.

6.2 Level 1 - High Priority Projects

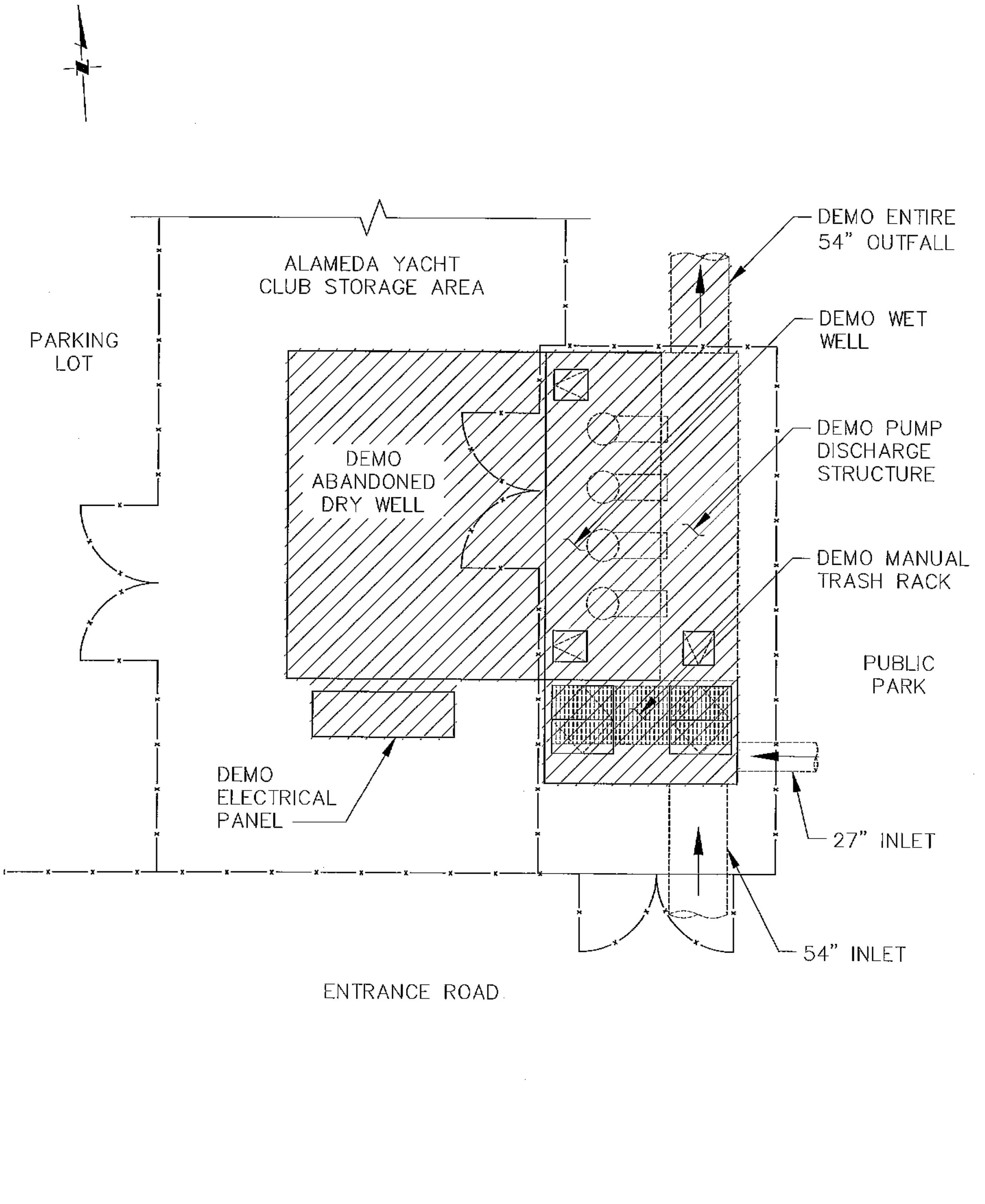
Level 1 or high priority projects are summarized in Table 6-1. For more information concerning the estimated costs for improvements at each station, refer to Appendix D.

Table 6-1 Summary of Level 1 High Priority Projects

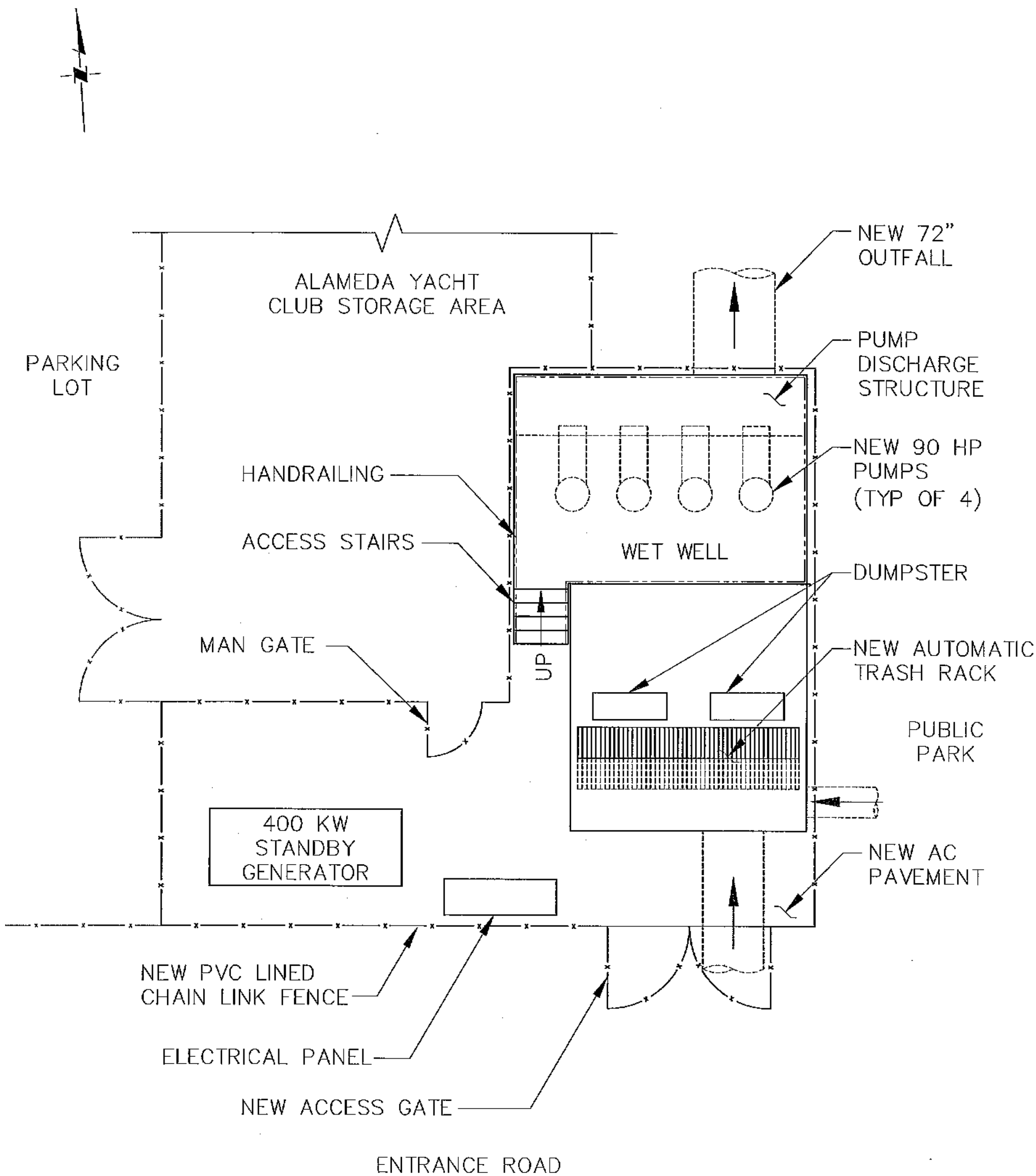
Pump Station	Summary of Recommended Improvements	Project Cost
Arbor	Complete Pump Station Replacement. Outfall replacement. Install Standby Generator and Automatic Trash Rack. The Existing Arbor Pump Station and Proposed Station are shown in Figures 6-1 and 6-2 respectively.	$3,891,000
Central/Eastshore	Complete Pump Station Replacement. Outfall Replacement. Install Standby Generator and Automatic Trash Rack. The Existing Central/ Eastshore Pump Station and New Proposed Pump Station are shown in Figures 6-3 and 6-4 respectively.	$2,805,000
Northside	Improvements to Northside constructed per the Northside Pump Station Upgrades Project No. PW 02-10-06.	$900,000[1]

Notes

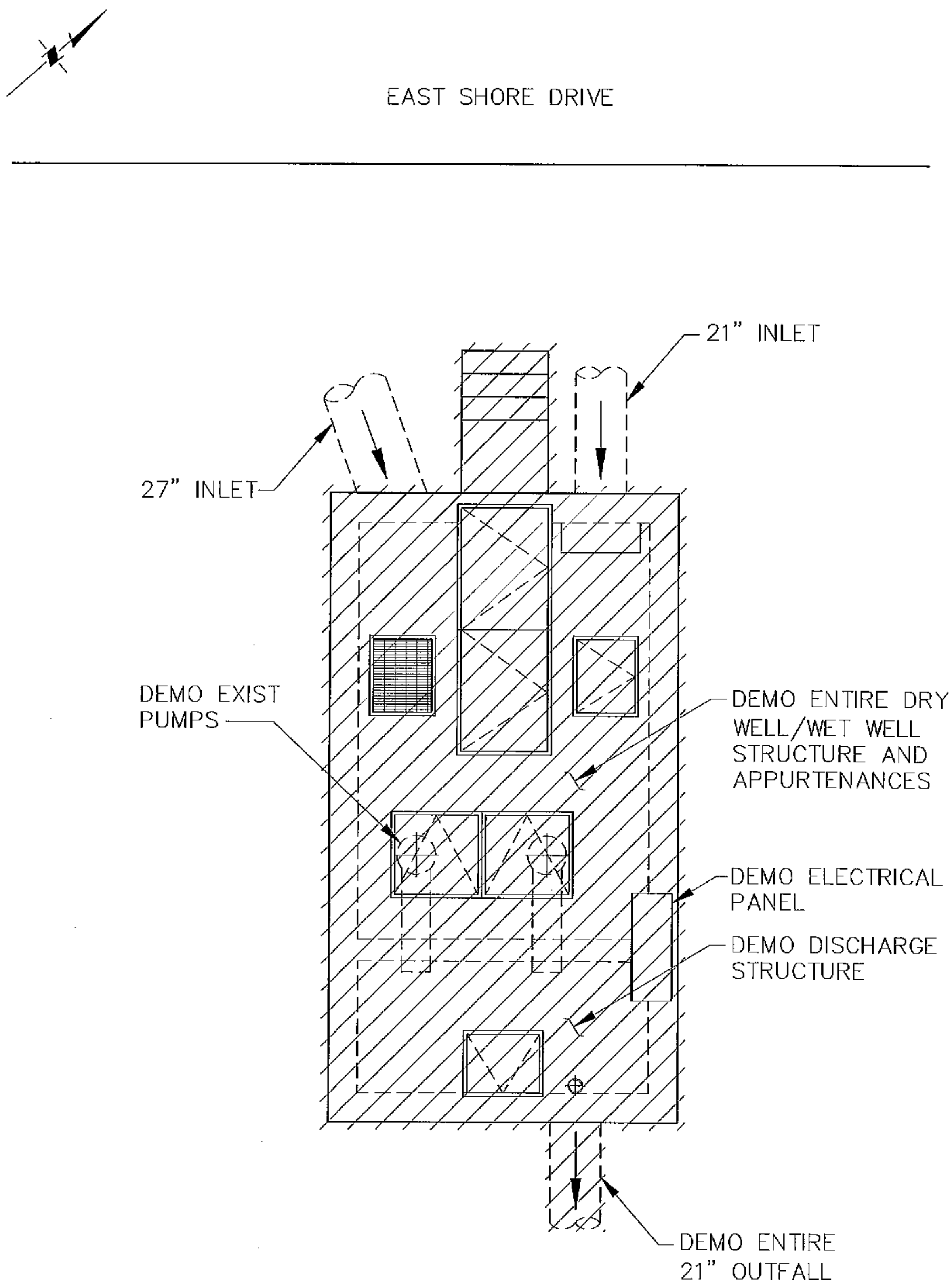
1. Bid amount for Northside Pump Station Upgrades Project No. PW 02-10-06 per Schaaf & Wheeler. Does not include 55 % markup for related project costs.



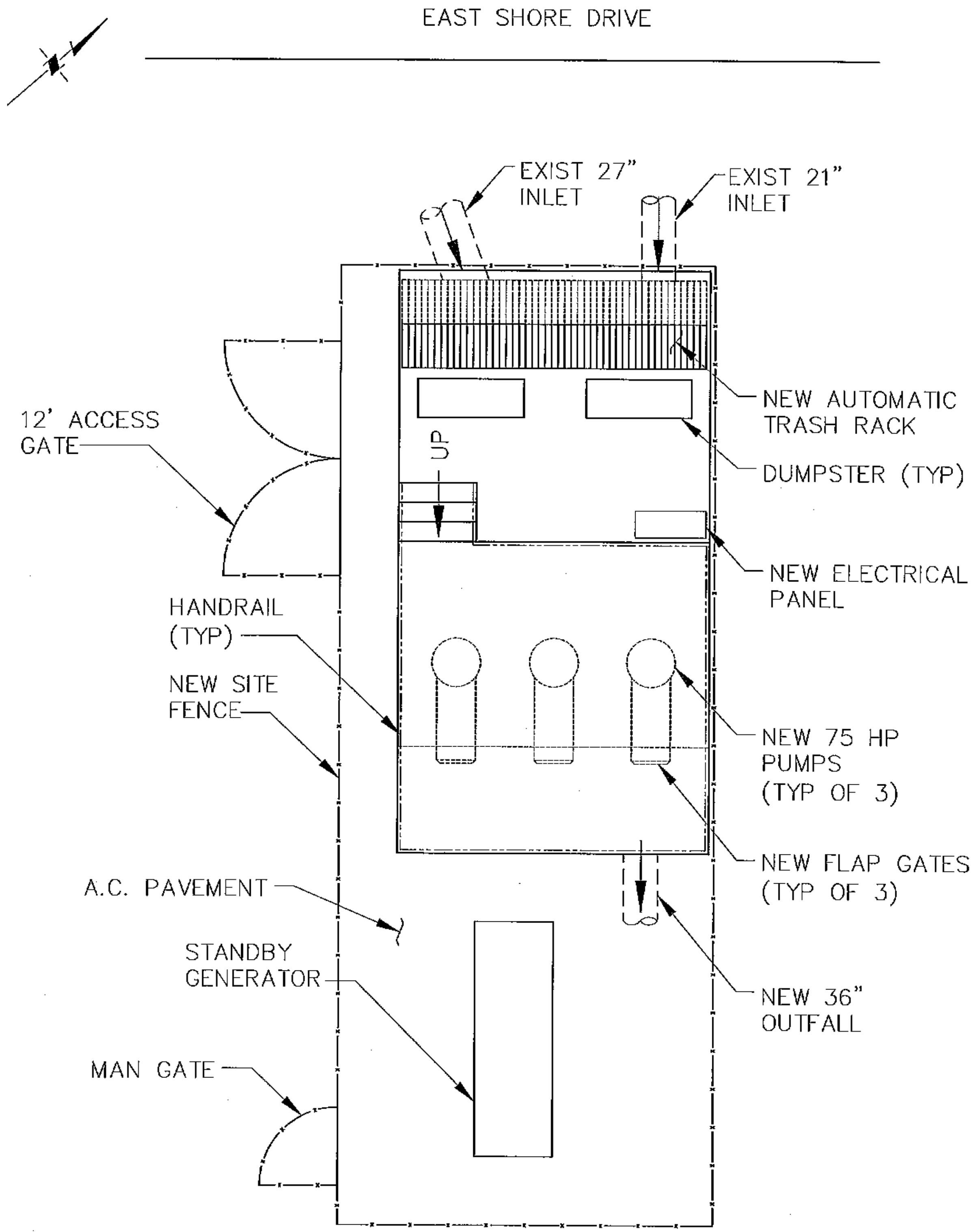
PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	EXISTING ARBOR PUMP STATION	FIGURE NO. 6–1 JOB NO. 6ALA010100



PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	PROPOSED ARBOR PUMP STATION	FIGURE NO. 6–2 JOB NO. 6ALA010100



PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	EXISTING CENTRAL/EASTSHORE PUMP STATION	FIGURE NO. 6-3 JOB NO. 6ALA010100



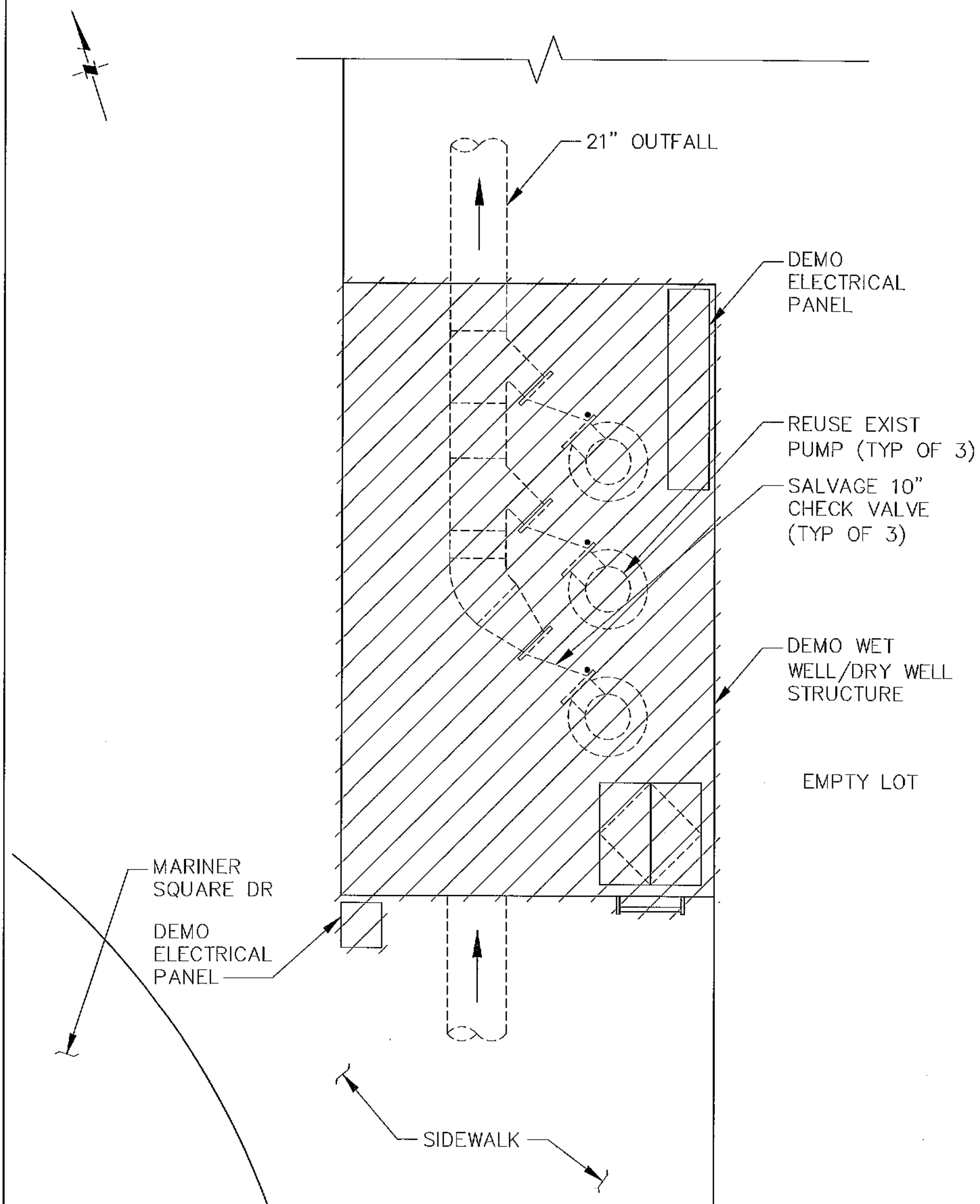
PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	PROPOSED CENTRAL/EASTSHORE PUMP STATION	FIGURE NO. 6–4 JOB NO. 6ALA010100

6.3 Level 2 - Necessary Projects

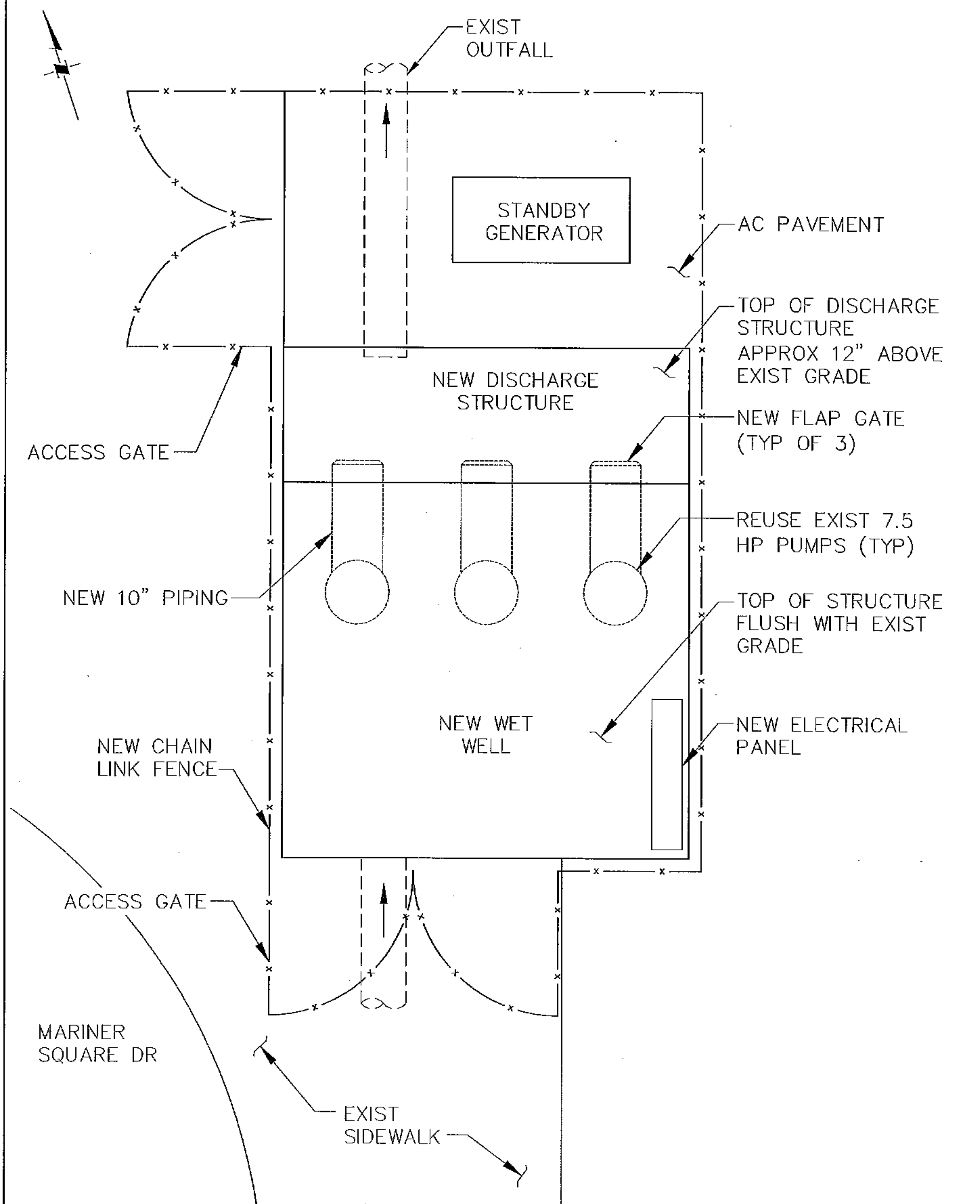
Level 2 or necessary projects are summarized in Table 6-2. For more information concerning the estimated costs for improvements at each station, refer to Appendix D.

Table 6-2 Summary of Level 2 Necessary Projects

Pump Station	Summary of Recommended Improvements	Project Cost
Golf Course	Install exposed wiring in rigid conduit. Provide anti-slip surfacing on existing station floor as required. Install fire extinguisher.	$6,000
Main Street	Install Ladder and Handrailing. Install Ventilation in Wet Well. Install Fire Extinguisher. Install Anchors for Control Panel. Replace Pump Controller with Pump Vision Controller. Replace Existing Bubbler Level Controls with Pressure Level Transmitters.	$84,000
Northside	Replace pumps with new pumps capable of handling design stormwater flows. Replace checker plate with new plate as required to remove tripping hazards.	$587,000
Webster Street	Complete Pump Station Replacement to address operations and maintenance and safety concerns (Reuse Existing Pumps). Install Standby Generator and New Electrical Equipment. The Existing Webster Street Pump Station and Proposed Station are shown in Figures 6-5 and 6-6 respectively.	$1,043,000
Third Street	Replace wet well hatch. Provide Davit Arm System for Wet Well Entry. Provide Standby Generator. Recoat piping (replace if required). Replace Flap Gate on Pump Discharge. Replace Site Fence. Provide wash down facilities. Install Fire Extinguisher. Install Pump Cables in Rigid Conduit. Improve Control Panel Anchorage to Slab.	$86,500



PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	EXISTING WEBSTER STREET PUMP STATION	FIGURE NO. 6–5 JOB NO. 6ALA010100



PSOMAS	CITY OF ALAMEDA STORM DRAIN PUMP STATION ASSESSMENT REPORT	PROPOSED WEBSTER STREET PUMP STATION	FIGURE NO. 6–6 JOB NO. 6ALA010100

6.4 Level 3 - Discretionary Projects

Level 3 or discretionary projects are summarized in Table 6-3. For more information concerning the estimated costs for improvements at each station, refer to Appendix D.

Table 6-3 Summary of Level 3 Discretionary Projects

Pump Station	Summary of Recommended Improvements	Project Cost
Golf Course	Install Paved Driveway, Standby Generator, and Fence Around Control Panel. Provide Site lighting and Wash Down Facilities. Coat Piping and Station Floor. Repair Check Valve. Replace Corroded Level Sensor Cover.	$198,000
Harbor Bay System I	Provide Wash Down Facilities. Inject grout into Cracks, Patch Concrete. Install Control Panel Intrusion Alarm and Fire Extinguisher.	$17,000
Harbor Bay System II	Replace Pump. Provide Wash Down Facilities. Install Standby Generator. Replace hatches. Install Site Lighting and Intrusion Alarms on Electrical Panel. Recoat Barrel Base Plate.	$257,000
Main Street	Install Standby Generator. Replace Hatches. Provide Station Wash Down Facilities.	$158,000

Appendix D

Capital Improvement

Cost Estimates

Arbor Pump Station

Capital Improvements Cost Estimate

Deficiency	Action Required	Cost Estimate
Insufficient Pump Capacity, Operations and Maintenance Issues, Safety Issues	Demo Existing Station and Abandoned Dry Well	$54,000
	Construct New Station Structure (wet well, discharge, and trash rack structure)	$367,000
	Install new pumps	$464,000
	Install New Pump Barrels, Piping, and Flap Gates. Install Backflow Preventer and Hose for Wash down	$111,000
	Install A.C Paving, Entire Site	$7,000
Insufficient Outfall Capacity	Install new 72-inch RCP outfall	$112,000
Lacks Backup Power	Install New Standby Generator and ATS	$155,000
Labor intensive manual trash rack	Install Automatic Trash Rack	$450,000
Lacks Private Fence Around Entire Station	Install New Site Fence	$20,000
Electrical, Instrumentation, SCADA required for New Equipment	Install new conduit, wiring, control panel, PLC, MCCs, site lighting (plus programming, testing and training)	$363,000
	Subtotal:	**$2,103,000**
	30% Estimating Contingency:	$631,000
	55% for Contingency, Admin., CM, and Engr:	$1,157,000
	Total w/ Contingency:	**$3,891,000**

1. Cost estimate in April 2011 dollars.

Central/Eastshore Pump Station

Capital Improvements Cost Estimate

Deficiency	Action Required	Cost Estimate
Insufficient Pump Capacity, Operations and Maintenance Issues, Safety Issues	Demo Existing Station and Abandoned Dry Well	$40,000
	Construct New Station Structure (wet well, discharge, and trash rack structure)	$307,000
	Install new pumps	$180,000
	Install New Pump Barrels, Piping, and Flap Gates. Install Backflow Preventer and Hose for Wash down	$79,000
	Install A.C Paving, Entire Site	$6,000
Insufficient Outfall Capacity	Install new 36-inch RCP outfall	$52,000
Lacks Backup Power	Install New Standby Generator and ATS	$119,000
Labor intensive manual trash rack	Install Automatic Trash Rack	$450,000
Lacks Fencing Around Station	Install Site Fence	$15,000
Electrical, Instrumentation, SCADA required for New Equipment	Install New Wiring, Motor Controls, New Transformer, Misc. Electrical Testing and Training	$268,000
	Subtotal:	**$1,516,000**
	30% Estimating Contingency:	$455,000
	55% for Contingency, Admin., CM, and Engr:	$834,000
	Total w/ Contingency:	**$2,805,000**

1. Cost estimate in April 2011 dollars.

Golf Course Pump Station

Capital Improvements Cost Estimate - Level 2 Projects

Deficiency	Action Required	Cost Estimate
Safety concerns (slippery floor, loose wires, lack of fire extinguisher)	Install Anti-Slip Coating to Floor, New Conduit for Loose Wires, Provide Fire Extinguisher	$3,000
	Subtotal:	**$3,000**
	30% Estimating Contingency:	$1,000
	55% for Contingency, Admin., CM, and Engr:	$2,000
	Total w/ Contingency:	**$6,000**

Capital Improvements Cost Estimate - Level 3 Projects

Deficiency	Action Required	Cost Estimate
Existing Coating on Piping is Chalked and Cracking	Recoat Piping	$2,000
Existing Check Valve is leaking	Repair Existing Check Valve	$2,000
Lacking Backup Power Supply	Install New Standby Generator and ATS	$75,000
Muddy site that cannot be driven on	Install Paved Driveway, Encase Existing Water Line	$15,000
Lacking Station wash down facilities	Install Backflow Preventer and Hose	$4,000
Lacking Fence Around Control Panel	Provide chain link fence with slats.	$6,000
Lacking Site Lighting	Install Site Lighting	$3,000
	Subtotal:	**$107,000**
	30% Estimating Contingency:	$32,000
	55% for Contingency, Admin., CM, and Engr:	$59,000
	Total w/ Contingency:	**$198,000**

1. Cost estimate in April 2011 dollars.

Harbor Bay System I Pump Station
Capital Improvements Cost Estimate

Deficiency	Action Required	Cost Estimate
Minor Cracks in Structure, Fence Post Spalled	Inject Grout Into Cracks, Patch Concrete Fence Post	$3,000
Lacks Station Wash Down Facilities	Provide Backflow Preventer and Hose bib	$4,000
Lacks Intrusion Alarm and Fire Extinguisher	Install Intrusion Alarm on Control Panel, Provide Fire Extinguisher	$2,000
	Subtotal:	**$9,000**
	30% Estimating Contingency	$3,000
	55% for Contingency, Admin., CM, and Engr:	$5,000
	Total w/ Contingency:	**$17,000**

1. Cost estimate in April 2011 dollars.

Harbor Bay System II Pump Station

Capital Improvements Cost Estimate

Deficiency	Action Required	Cost Estimate
Insufficient Pump Capacity	Replace Pump	$53,000
Lacks Backup Power	Install New Standby generator and ATS	$45,000
Existing Steel Plates on Valve Vault and Hatch on Discharge Structure are Heavy and Difficult to Open	Install Aluminum Hatches	$7,000
Lacks Wash Down Facilities	Install Backflow Preventer and Hose	$4,000
Lacks Site Lighting	Install Site Lighting	$3,000
Misc. Electrical Work Required For New Pumps and Generator	Install New Wiring, Motor Controls, Misc. Electrical Testing and Training	$25,000
Lacks Alarm on Control Panel	Install Alarm on Control Panel	$2,000
	Subtotal:	**$139,000**
	30% Estimating Contingency:	$42,000
	55% for Contingency, Admin., CM, and Engr:	$76,000
	Total w/ Contingency:	**$257,000**

1. Cost estimate in April 2011 dollars.

Main Street Pump Station

Capital Improvements Cost Estimate - Level 2 Projects

Deficiency	Action Required	Cost Estimate
No Ventilation in Wetwell	Install Ventilation in Wetwell	$2,500
Pump Controller Different from Standard Being Used by City. Existing Bubbler Level Controls Outdated.	Install New Pump Controller and Level Controls.	$15,000
No Ladder to Access Pump Side of Structure	Install Ladder	$2,000
Handrailing Required to Meet OSHA Requirements	Install Handrailing Around Structure	$25,000
No Fire Extinguisher	Install Fire Extinguisher	$250
Install Anchors for Control Panel	Provide Anchors for Control Panel	$500
	Subtotal:	**$45,000**
	30% Estimating Contingency:	$14,000
	55% for Contingency, Admin., CM, and Engr:	$25,000
	Total w/ Contingency:	**$84,000**

Capital Improvements Cost Estimate - Level 3 Projects

Deficiency	Action Required	Cost Estimate
Lacks Backup Power	Install New Standby generator and ATS	$70,000
Existing Plates above Pumps are Difficult to Remove	Install New Aluminum Hatches	$11,000
Lacks Wash Down Facilities and Fire Extinguisher	Install Backflow Preventer, Hose, and Fire Extinguisher	$3,750
	Subtotal:	**$85,000**
	30% Estimating Contingency:	$26,000
	55% for Contingency, Admin., CM, and Engr:	$47,000
	Total w/ Contingency:	**$158,000**

1. Cost estimate in April 2011 dollars.

Northside Pump Station

Capital Improvements Cost Estimate

Schaaf and Wheeler Designed Improvements	Cost
Schaaf and Wheeler Designed Improvements, under construction (Cost from Schaaf and Wheeler):	**$900,000**

Additional Improvements		
Deficiency	**Action Required**	**Cost Estimate**
Insufficient Pump Capacity	Install new pumps	$320,000
Checker Plate with Lifting Eyes Presenting Trip Hazard	Replace with Checker Plate with Recessed Lifting Eyes	$25,000
	Subtotal:	**$345,000**
	15% Estimating Contingency:	$52,000
	55% for Contingency, Admin., CM, and Engr:	$190,000
	Additional Improvements w/ Contingency:	**$587,000**

Total Cost (Including Schaaf and Wheeler Improvements)	$1,487,000

1. Cost estimate in April 2011 dollars.

Third Street Pump Station

Capital Improvements Cost Estimate - Level 2 Projects

Deficiency	Action Required	Cost Estimate
Lacks Backup Power	Install New Standby Generator and ATS	$13,000
Hatch Access Does Not Meet OSHA Requirements	Replace Hatch With Larger Hatch	$3,000
Unsafe Confined Space Entry in Wet Well	Install Sleeve for Removable Davit Arm. Provide Davit Arm For Fall Protection and Operator Removal	$8,000
Electrical Control Panel not Anchored Properly	Anchor Electrical Control Panel to Concrete Base	$1,000
Wet Well Piping is Corroded	Perform Pipe Corrosion Assessment. Recoat piping (replace if necessary)	$3,000
Existing Flap Gate Stuck Open	Replace Flap Gate	$4,500
Lacks Wash down Facilities and Fire Extinguisher	Install Backflow Preventer and Hose bid, and Fire Extinguisher	$4,000
Existing Fence is Falling Apart	Replace Fencing, Increasing Fenced Area to Include New Generator	$10,000
	Subtotal:	**$46,500**
	30% Estimating Contingency:	$14,000
	55% for Contingency, Admin., CM, and Engr:	$26,000
	Total w/ Contingency:	**$86,500**

1. Cost estimate in April 2011 dollars.

Webster Street Pump Station
Capital Improvements Cost Estimate

Deficiency	Action Required	Cost Estimate
Operations and Maintenance Issues, Safety Issues	Demo Existing Station and Abandoned Dry Well	$40,000
	Construct New Station Structure (wet well, discharge, and trash rack structure)	$239,000
	Install New Pump Barrels, Piping, and Flap Gates (Reuse existing pumps)	$69,000
	Install A.C Paving	$3,000
Lacks Backup Power	Install New Standby Generator and ATS	$45,000
Lacks Fencing Around Station	Install New Site Fence	$15,000
Electrical & Instrumentation	Install new conduit, wiring, ATS, control panel, site lighting	$153,000
	Subtotal:	**$564,000**
	30% Estimating Contingency:	$169,000
	55% for Contingency, Admin., CM, and Engr:	$310,000
	Total w/ Contingency:	**$1,043,000**

1. Cost estimate in April 2011 dollars.

To: **Erin Smith, PE**

From: **Andrew Augustine, PE**

CC: **Dan Matthies, PE; Cheng Soo, PE**

Date: **December 5, 2022**

Project Name: Chuck Corica Golf Course Drainage

Subject: Project Technical Memorandum

1 INTRODUCTION

The Chuck Corica Golf Course (Golf Course) is a municipally-owned 300 acre public golf course complex located in the City of Alameda on Bay Farm Island in the East Bay region of San Francisco Bay. It has been operating since 1927, with two 18-hole courses (North Course and South Course), a 9-hole par three course, and a driving range. See Figure 1.

The Golf Course's drainage is served by two sloughs, the West Slough and the East Slough. The sloughs collect untreated rainfall and irrigation runoff from the Golf Course, surrounding roads, residential neighborhoods, and shallow groundwater. The West Slough collects runoff from the western portion of the Golf Course, and from several storm sewers discharging runoff from residential neighborhoods adjacent to Fitchburg Avenue, Melrose Avenue, Flower Lane, and Maitland Drive. The East Slough collects runoff from the eastern portion of the Golf Course, and from Harbor Bay Parkway. No runoff from the Port of Oakland's property (east of Harbor Bay Parkway) enters the slough system. Both sloughs drain from south to north, ultimately discharging into a retention pond near State Highway 61 (Doolittle Drive), where it is pumped by two 60 horsepower pumps into the San Leandro Bay at the Golf Course Storm Drain Pump Station (Golf Course SDPS). See Figure 2 and Figure 3. Shallow groundwater keeps a constant depth of water in the downstream portions of the sloughs. This depth can range from between three to four feet in the northern (downstream) portion of the sloughs, to a few inches in the southern (upstream) portions. See Figure 4.

Over the past two years, reports of flooding in residential neighborhoods adjacent to the Golf Course have been brought to the attention of the City of Alameda (City) Public Works Department. Concerned citizens on Maitland Drive and Garden Road have observed a significant increase in the frequency of property flooding during rainfall events (Figure 5), while City crews have also observed frequent flooding on Harbor Bay Parkway (Figure 6), sometimes resulting in the closure of the street.

The Golf Course is currently leased to a third-party operator with defined maintenance responsibilities. This technical memorandum will not detail these maintenance responsibilities, but will instead focus on Wood Rodgers' (WR) investigation of the flooding and their proposed solutions.

Figure 1 – Vicinity Map

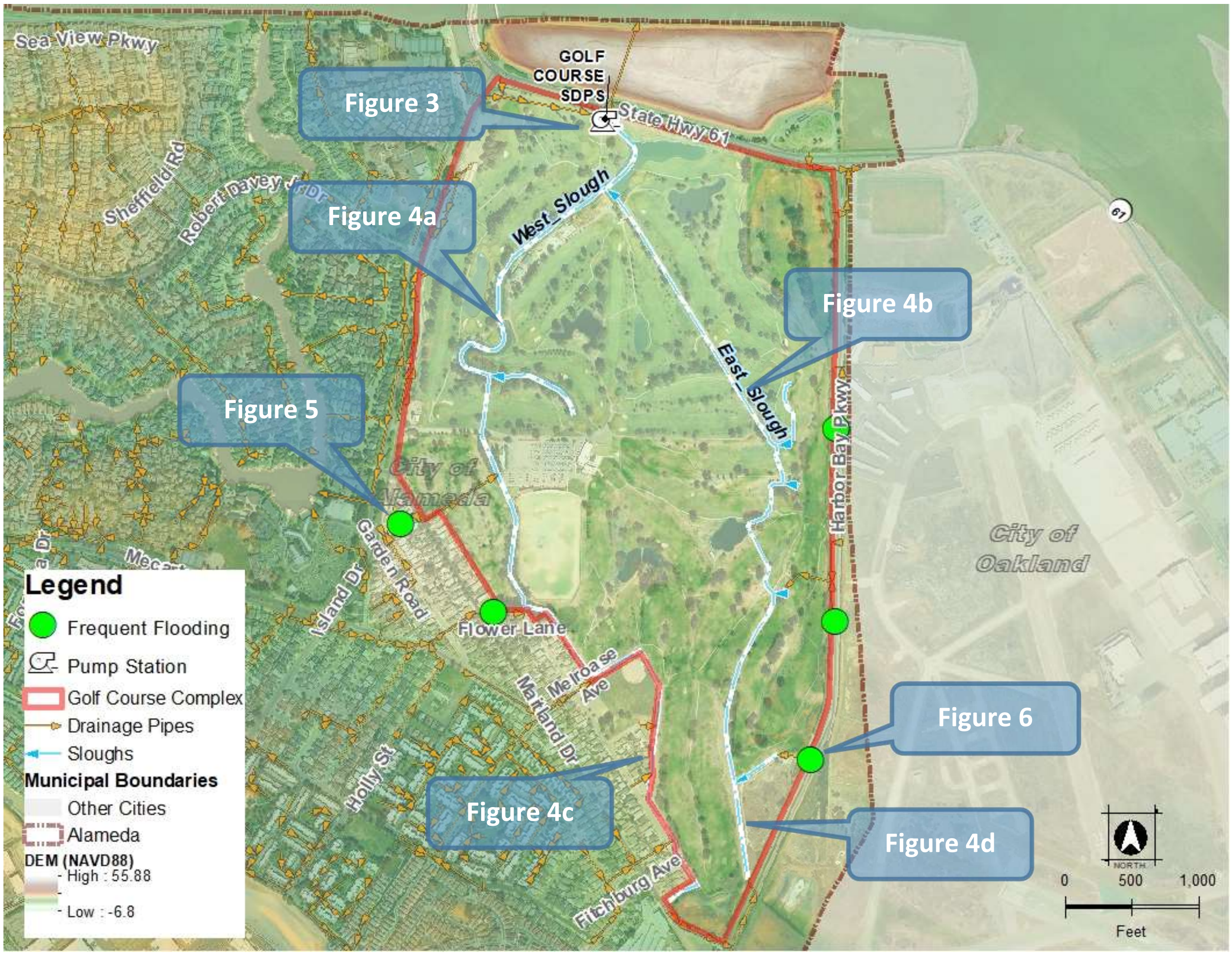
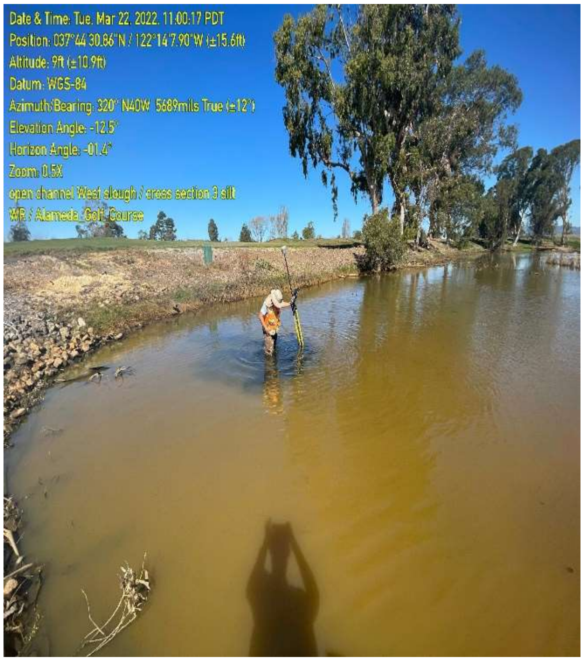


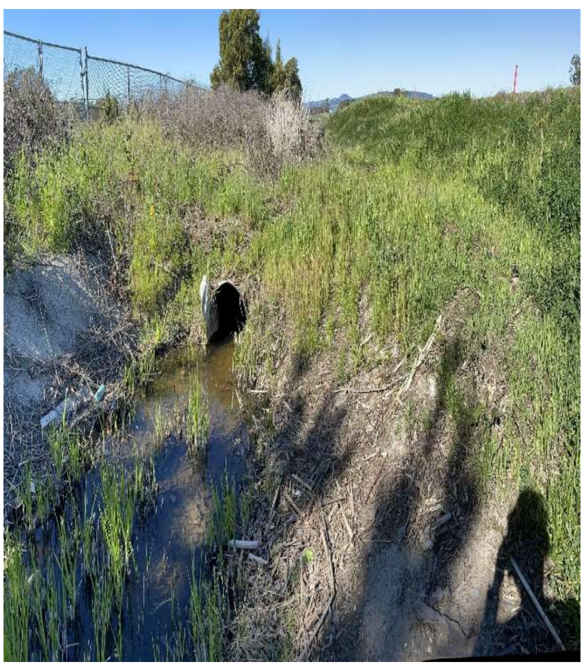
Figure 2 – Golf Course Drainage and System

Figure 3 – Golf Course SDPS



(a)

(b)



(c)

(d)

Figure 4 – Golf Course Drainage System Photos (a) Downstream West Slough (b) Downstream East Slough (c) Upstream West Slough (d) Upstream East Slough

Figure 5 – December 2021 Property Flooding at 3 Garden Road

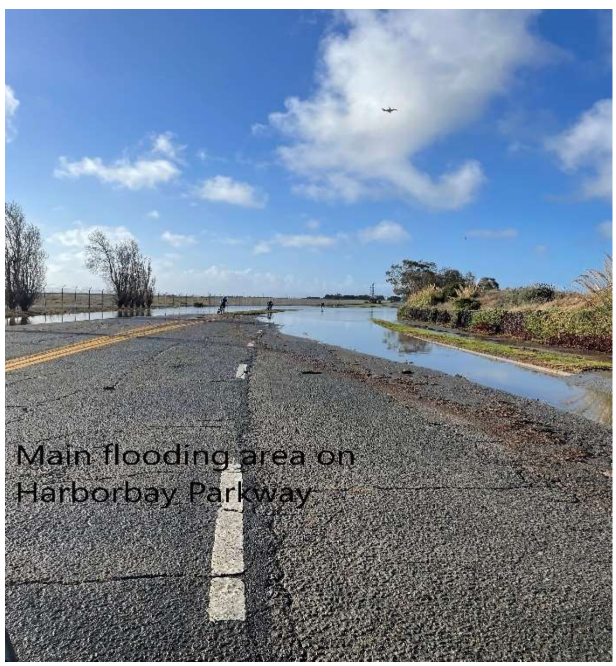


Figure 6 – December 2021 Flooding of Harbor Bay Parkway 1,500 feet North of Maitland Drive

2 PURPOSE

The purpose of this project is to:

1. Determine the probable cause of the frequent flooding reported by residents and City crews;
2. Propose maintenance activities and immediate action rehabilitation work to mitigate the frequent flooding;
3. Identify next steps to solidify flood mitigation.

3 BACKGROUND

Residents on Maitland Drive and Garden Road have stated they believe flooding on their properties during recent storms would not have occurred two or more years ago. This suggests a recent change to the storm sewer drainage system draining the area or to the downstream boundary conditions.

4 APPROACH

WR first requested information on the storm sewers and slough system to understand the unique drainage properties of the Golf Course and the surrounding residential neighborhoods.

Data describing the storm sewer infrastructure in the Golf Course, however, is limited. Therefore, information to fill these gaps was collected from three additional sources: staff interviews, field surveys, and condition assessments. To accomplish the purpose of this project the following approach was proposed:

1. Interview City and Golf Course staff. Each entity had firsthand knowledge of the flooding shown in Figure 2 - Figure 6, and can provide crucial information on the Golf Course drainage system. Desired information included field observations, operations and maintenance (O&M) logs, photos, and Golf Course construction history.
2. Conduct a field survey of missing drainage system assets. A field survey fills in the data gaps provided to WR by the City. A survey of drainage assets provides a complete picture of the Golf Course complex drainage. The survey collected information such as diameters, invert elevations, and material. Cross sections of the East and West Sloughs were also collected.
3. Conduct an in-depth, above ground, inspection of the Golf Course drainage system. An inspection was conducted to observe the structural and operational and maintenance conditions of the drainage assets (storm sewers, culverts, sloughs, Golf Course SDPS, etc.). Observations of structurally compromised assets, clogged assets, reduced slough capacity, or Golf Course SDPS reduced capacity were valuable information to help WR determine a probable cause of flooding.
4. Determine the probable cause of flooding with this information using engineering judgement.
5. Determine a solution to reduce flooding using engineering judgement.

5 PREVIOUS PROJECTS & STUDIES

5.1 1995 Channel Dredging

The City provided an asbuilt titled "*Drainage Improvements, Chuck Corica Municipal Golf Complex*" dated May 24, 1995. The asbuilt details a survey and proposed dredging to improve the West Slough, south of Clubhouse Memorial Road, and the northern half of the East Slough. See Figure 7.

The West Slough's survey included flow line and top of bank elevations. Typical sections were provided detailing the depth of dredging. See Figure 8. The East Slough's survey included top of bank elevations and typical dredging exhibits; no flow line elevations were provided. See Figure 9.

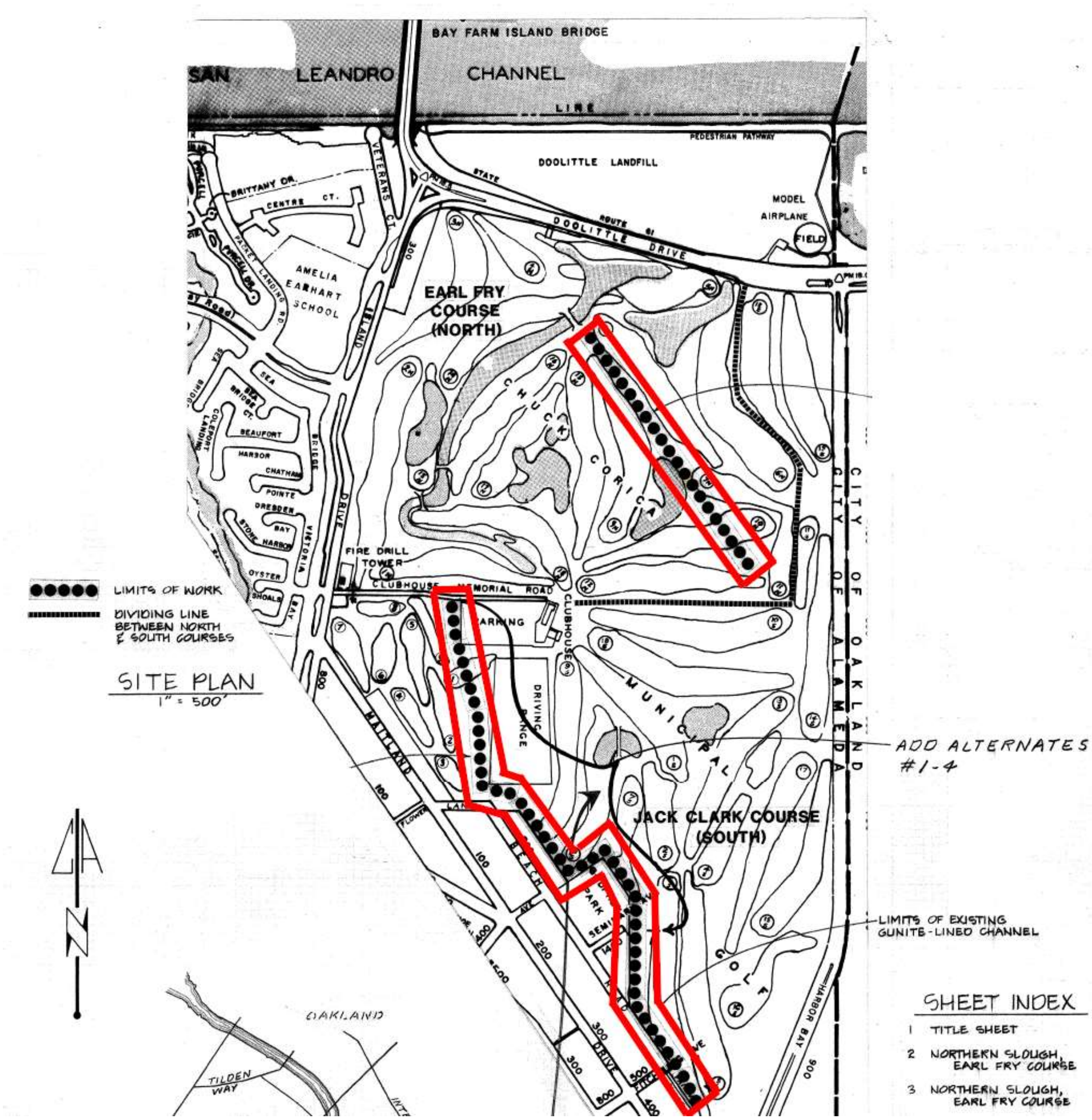


Figure 7 – Asbuilt "Drainage Improvements, Chuck Corica Municipal Golf Complex" Improvement Extents (red)

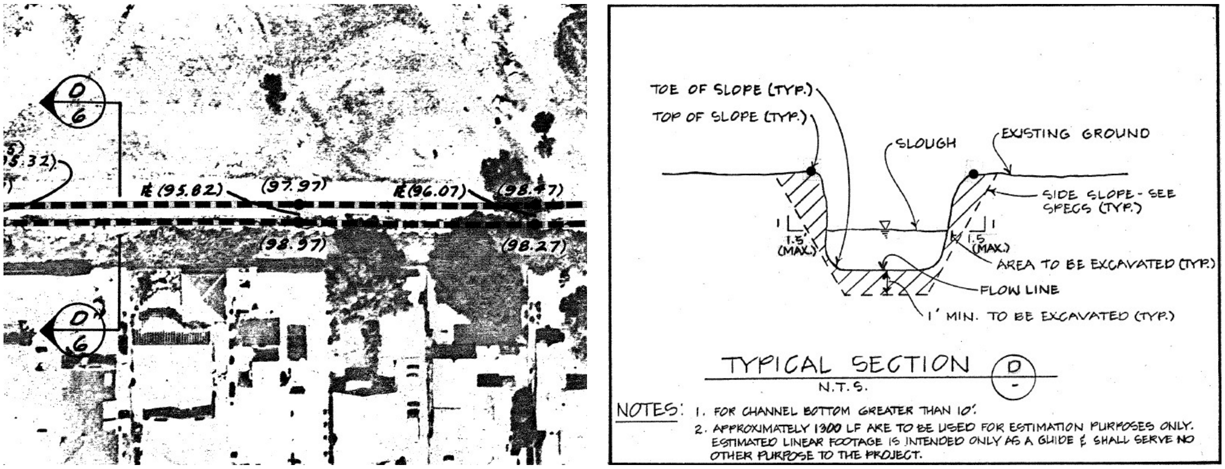


Figure 8 – Typical West Slough Bankline and Flow Line Elevation Callouts (Left) and Dredging Detail (Right)

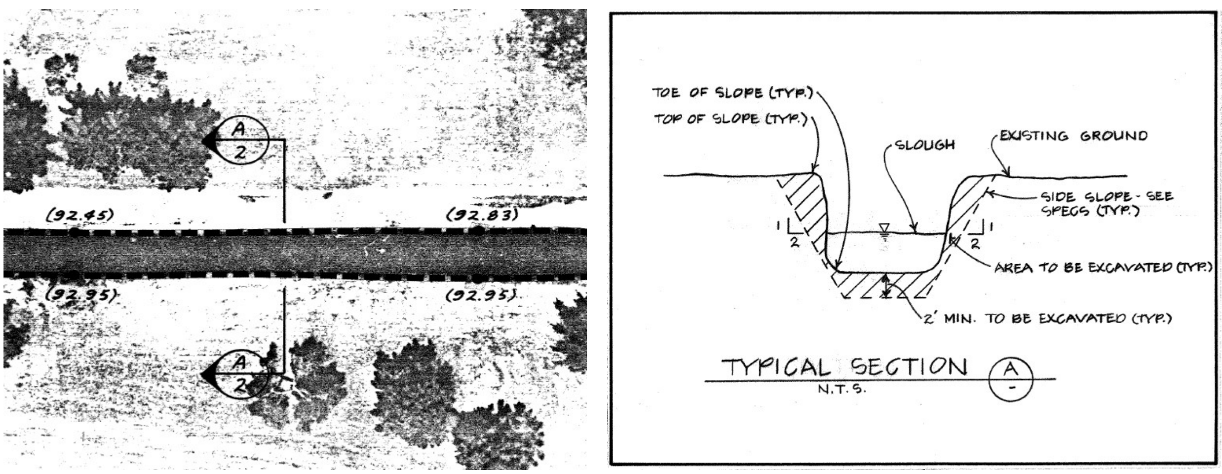
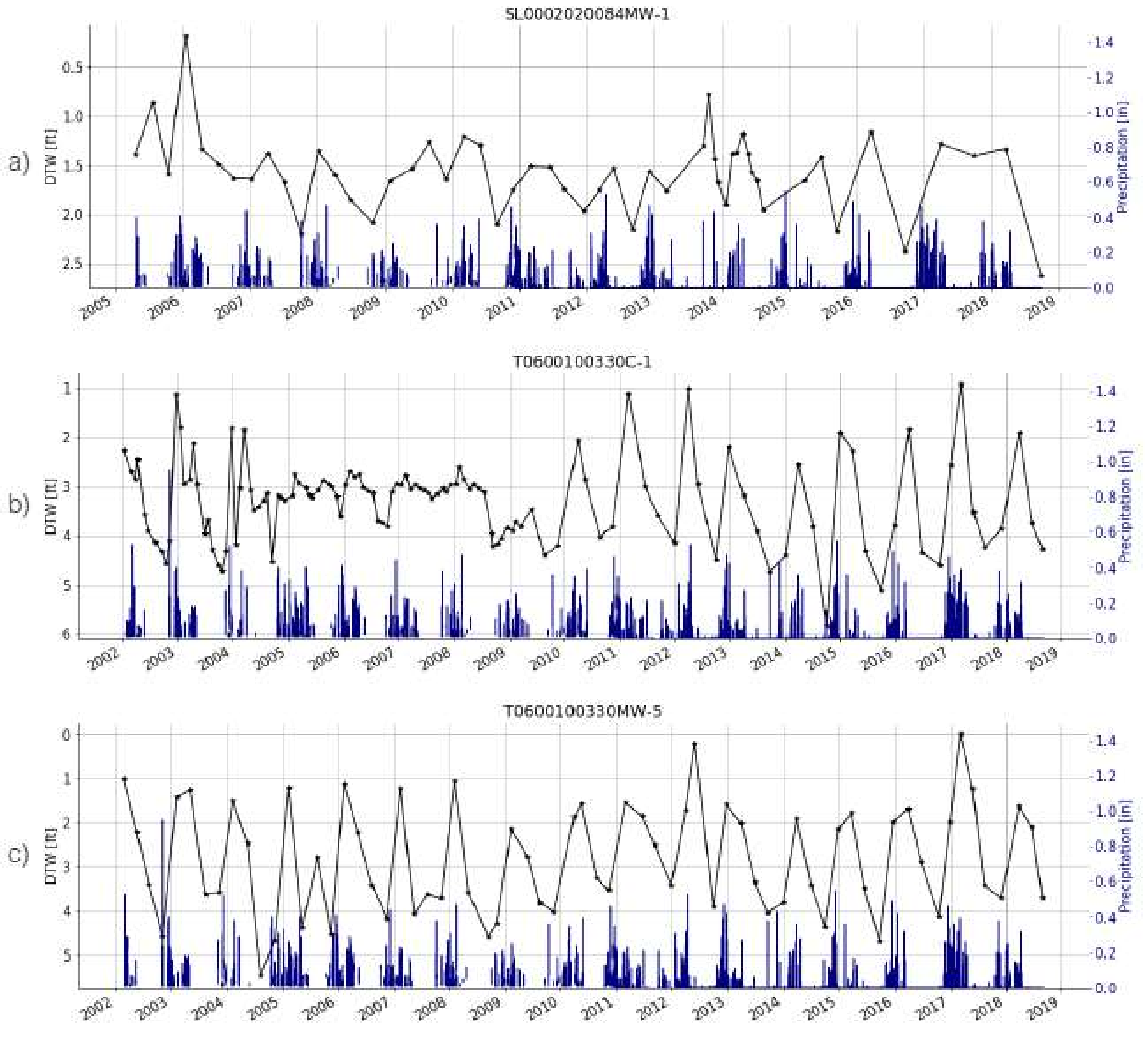


Figure 9 – Typical East Slough Bankline Elevation Callouts (Left) and Dredging Detail (Right)

5.2 City of Alameda Groundwater Study

In September 2020, Silvestrum Climate Associates detailed the effects of sea-level rise on the City's shallow groundwater and contaminants in a report titled "*The Response of the Shallow Groundwater Layer and Contaminants to Sea Level Rise*". Pertinent information from the report to this study include detailed graphs of depth to groundwater at various locations across the City observed over the past 17 years (Figure 10), and project the impact of sea-level rise on the shallow groundwater (Figure 11).



a) Doolittle Drive on Bay Farm Island, b) High Street and Gibbons Drive, c) High Street and Gibbons Drive

Figure 10 – Depths to Groundwater Over Time at Various Locations in the City (black) with Precipitation (blue)

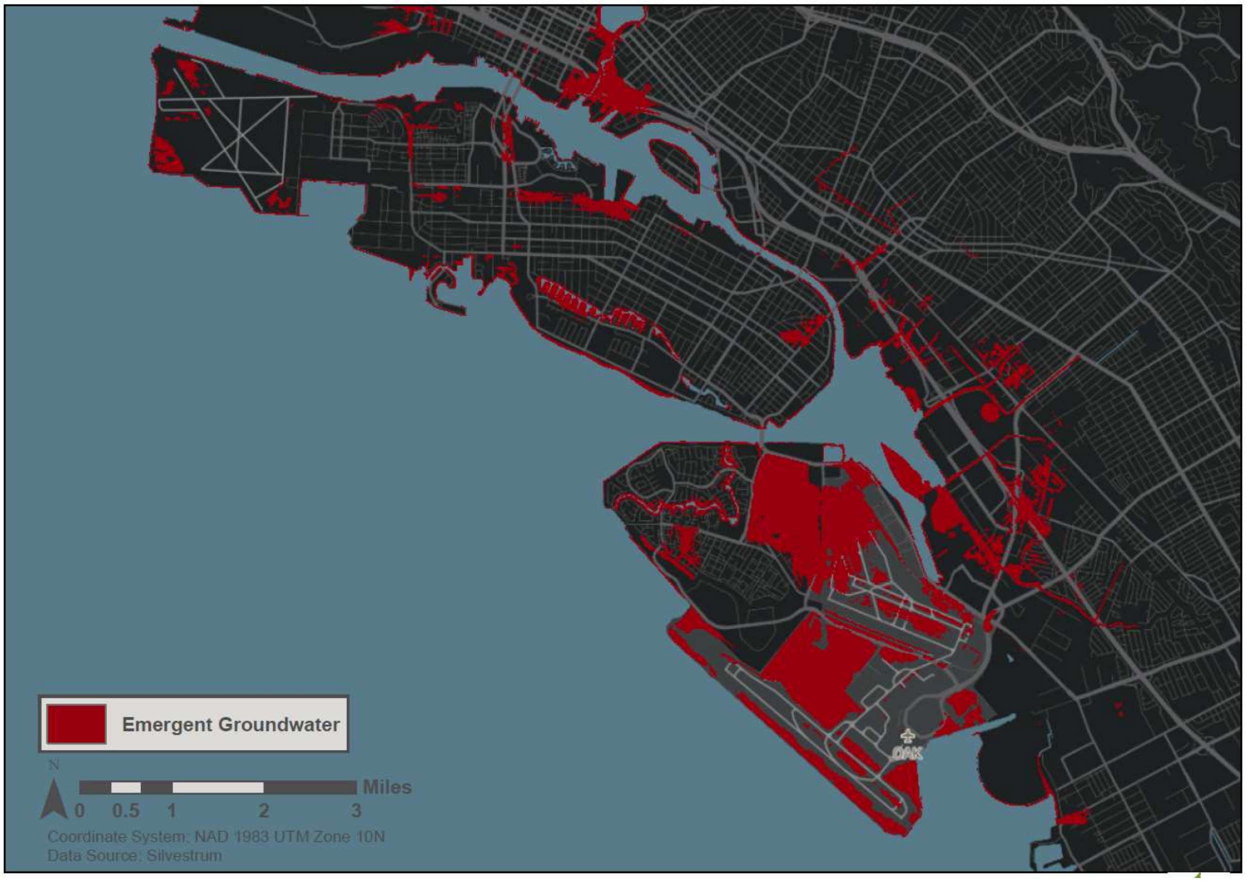


Figure 11 – Shallow Groundwater with 12" of Sea-Level Rise

6 DATA COLLECTION

Requested data included an inventory of the City's drainage system (storm drains, culverts, manholes, pump stations, etc.), logs of the City's maintenance activities, asbuilts, and plan sets. The City provided information they had, including an ArcGIS Geodatabase of their drainage system, asbuilts and plan sets pertinent to the Golf Course drainage, and a verbal discussion of their typical drainage system maintenance activities. These data were comprehensive in areas outside of the Golf Course complex, however, there was little to no data inside. Several locations along the sloughs had no information describing the drainage system. Furthermore, maintenance activities on drainage assets inside the Golf Course complex was unknown.

6.1 Staff Interviews

6.1.1 Erin Smith & Manny Rios

Erin Smith is the Public Works Director for the City of Alameda, and Manny Rios is the Public Works Supervisor for the City of Alameda. Both Erin and Manny were interviewed on February 14, 2022, and shared the following information:

1. Manny stated there are two pipes from the Port property that enter the Golf Course.
    a. Update – When Manny and his crew investigated the City's storm sewers in March of 2022 on Harbor Bay Parkway, he determined no storm sewers from the Port enter the Golf Course.
2. The North course is still under construction.
3. The Maitland storm sewer system is cleaned once a year prior to the wet season. It was inspected with CCTV about three years ago. Nothing wrong with the storm sewer was observed.
4. Manny's crew observed the Golf Course SDPS turning off during the December 2021 storm when Harbor Bay Parkway was still flooded.
5. Manny's crew observed a chokepoint in the slough system at the culvert draining into the pump station retention pond from the east slough. There is a noticeable sinkhole in the culvert. Crew observed water backing up on the upstream side, but water was barely trickling out of the culvert at the downstream side. Marc Logan, the Golf Course Maintenance Supervisor, has informed the City the sink hole will be fixed in May or June of 2022.
6. A picture of the Harbor Bay flooding was provided to WR. It was taken the day after the rainfall event in December 2021. Manny stated it took approximately 3 days to drain.
7. Resident complaints on Maitland Drive started approximately 3 – 5 years ago.
8. City would like the following big-picture questions answered: "Were the sloughs designed to have water in them at all times?", "Should the City drain the sloughs in the winter?" and "Do the sloughs have enough capacity?"
9. Manny's crew, when verifying a storm system, verifies the manholes, pop the manholes or inlets, and verifies flow directions.

10. WR requested Manny, when draining culverts/pipes, to observe sediment depths and water surface elevations.
11. Manny said there has been flooding on Flower Lane, but no property damage.

6.1.2 Marc Logan – Golf Course Maintenance Supervisor

Marc Logan worked at the Golf Course from September of 2012 to April of 2022 and has intimate knowledge of its drainage system. Marc was interviewed on March 1, 2022 by Andrew Augustine of WR, with Erin Smith and Emanuel Rios from the City in attendance. Marc shared the following information:

1. Marc observed flooding at Harbor Bay Parkway and Ron Cowan Parkway in the December 2014 event.
2. In general, a storm event less than 3 inches does not cause flooding issues.
3. In general, runoff takes 24 – 48 hours to move through the system.
4. Marc observed Maitland Drive flooding in winter of 2014, 2018 and 2021. Observed the sloughs were at capacity.
5. Marc has not observed a noticeable chokepoint within the sloughs.
6. In August of 2021, the City drew down the system using the Golf Course SDPS. Within 60 hours the system was back to its previous conditions. Marc estimates 12 acre-feet came back into the system after pumping.
7. The Golf Course SDPS is turning on during summer months.
8. Observes Golf Course SDPS pumping continuously, not intermittently during storms.
9. Renovations to the Par 3 course started September 2013, ended May 2014. Renovations to the south course started December 2014, ended June 2018. Renovations to the north course started mid July 2018 and are ongoing.

6.2 Field Survey

On March 22 and March 23 of 2022, staff from WR conducted a field survey of the Golf Course storm system. A Global Positioning System (GPS) survey and spot inspections with a three-person crew consisting of two experienced engineers and a licensed surveyor was performed. The process recorded spatial locations, elevations, and storm drainage facility types. RTK (Real-Time-Kinematic) GPS surveying was used, which uses a network of satellites that communicate with receivers on the ground to determine the horizontal coordinates (x, y) and elevations (z). The surveying method provides a horizontal and vertical accuracy up to 0.1 feet. The field inspector or engineer utilized several standard inspection tools to document pipe/structure information (diameter, shape, material, depth, etc.).

6.3 Condition Assessment

In conjunction with the field survey, a condition assessment was conducted. A condition assessment is a technical assessment of the data collected in Section 6.2. The assessment provides standard ratings of the structural and maintenance conditions of the inspected facilities and the corresponding rehabilitation and replacement recommendations. The Environmental Protection Agency's (EPA) "*Asset Management Handbook*" and the National Association of Sewer Service Companies (NASSCO) *Pipeline Assessment Certification Program (PACP)* condition grading systems guidelines were used to provide a standard condition rating system for each facility.

Experienced inspectors assessed the pipe/structure conditions, and record any observed performance issues (plugging, erosion, sedimentation, overtopping, etc.). The inspection tools include electronic devices (digital tablets, GPS enabled cameras, and manhole inspection cameras), measurement devices (sediment probes and steel or vinyl tape measures), and standard access tools (manhole picks, sledgehammers, ratchet and sockets, and bolt hole alignment tools). The digital tablet is loaded with the ArcGIS Survey123 application to aid the inspection. Survey123 allows the inspector to take geo-located photographs, and asses the structural and O&M deficiencies of the asset, as shown in Figure 12. A typical inspection setup is displayed in Figure 13.

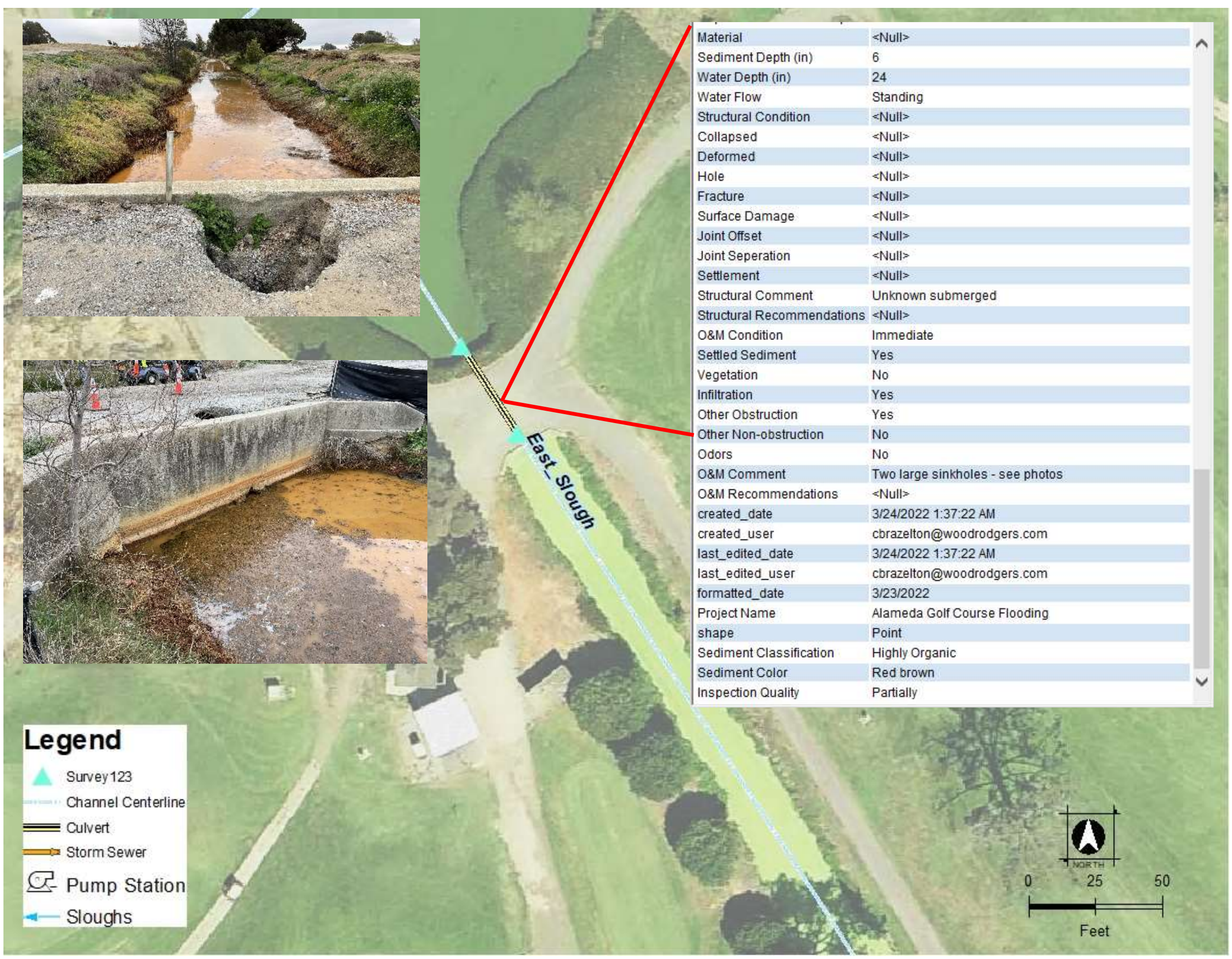


Figure 12 – Example of Survey123 Application for Structural and O&M Deficiencies

Figure 13 – Typical Inspection Setup

7 RESULTS

Information gathered in Section 6 was processed and described below.

7.1 Survey Results

Survey data of the Golf Course's storm sewer system was post-processed into a GIS GeoDatabase. Surveyed assets were built into a featrueclasses, such as storm sewers, culverts, and channel centerlines. Additional information such as diameter, invert, and material were populated into the asset's attribute table, as shown in Figure 14. A summary of all surveyed locations is shown in Figure 15.

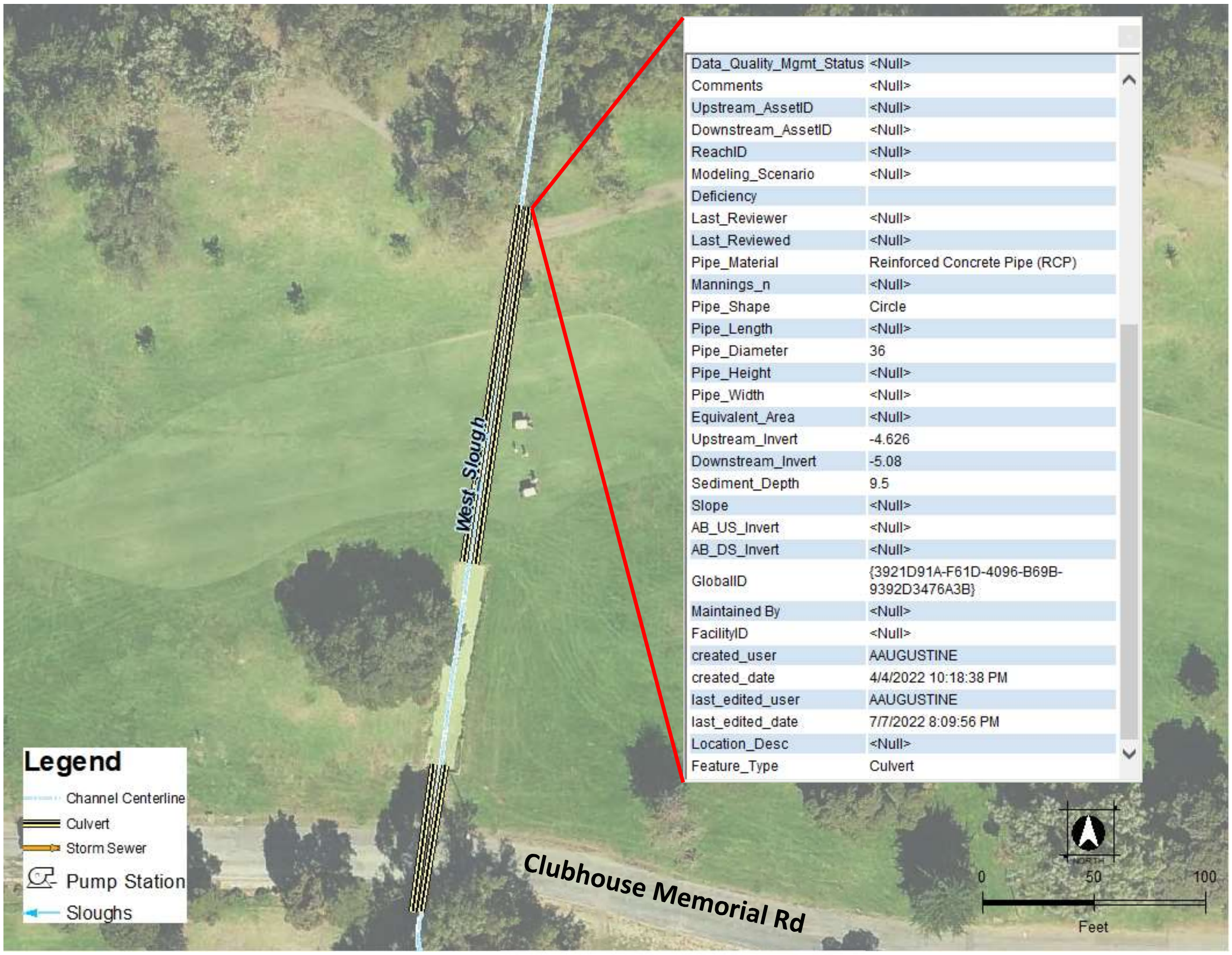


Figure 14 – Surveyed Culvert with Attributes

Figure 15 – Survey Locations

Profiles of the East and West Sloughs was created to display the existing slough centerline, culverts, storm sewers discharging into the sloughs, and the 1995 slough centerline as described Section 5.1. Profiles for the East and West Slough is in Appendix A.

7.2 Condition Assessment Results

Information gathered using the Survey123 application was used to inventory the structural and O&M deficiencies of the Golf Course storm sewer system. For each storm sewer asset, comments regarding the structural integrity and the O&M condition were provided. Geo-located pictures of the asset were also taken. All assets examined for structural deficiencies are shown in Figure 16. Those assets assigned a "poor" rating are called out in the figure. Similarly, all assets examined for O&M deficiencies are shown in Figure 17 and Figure 18. Those assets assigned an "immediate" rating are called out in the figures.

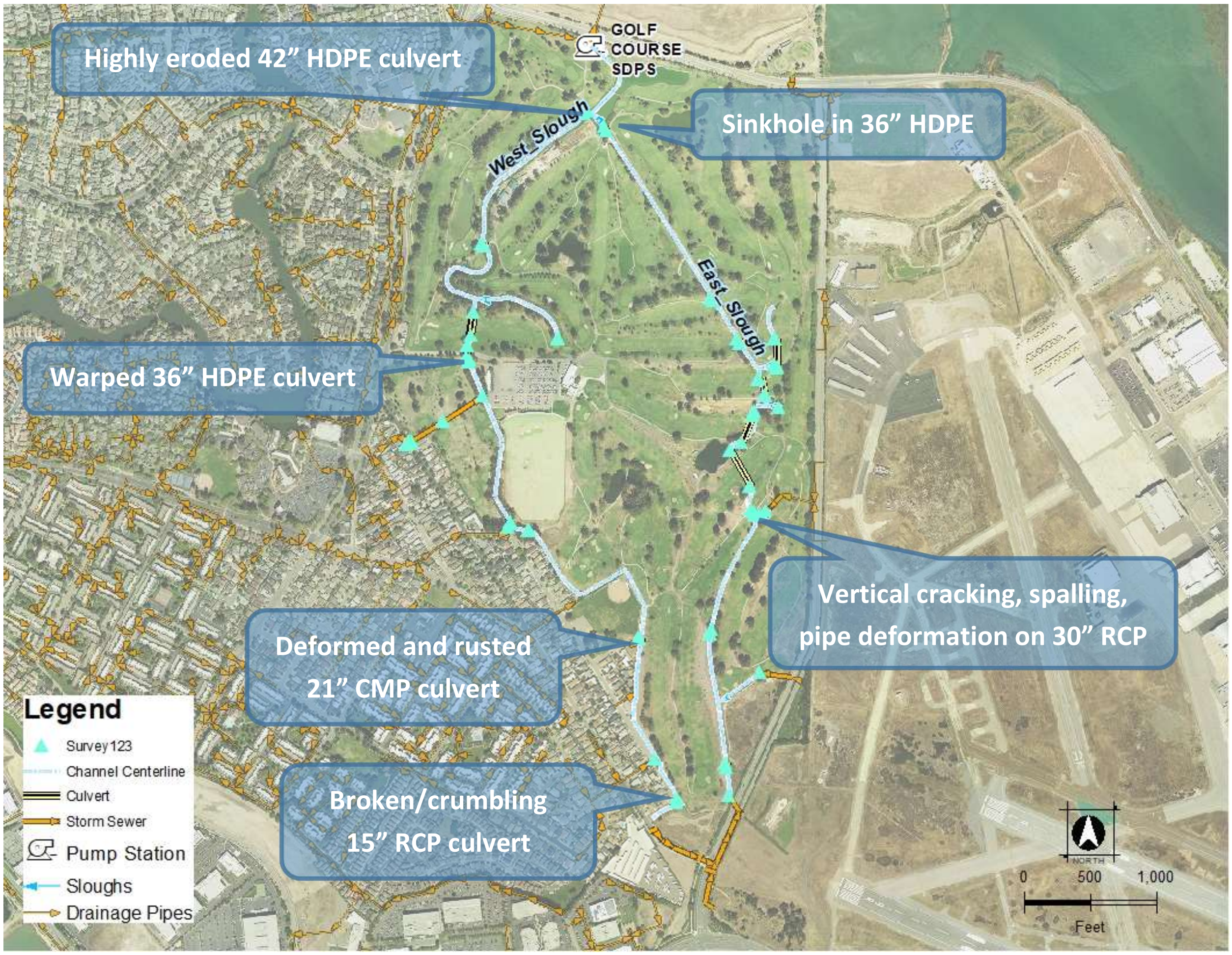


Figure 16 – Survey123 Structural Assessment Locations (Poor Condition Rating Called Out)

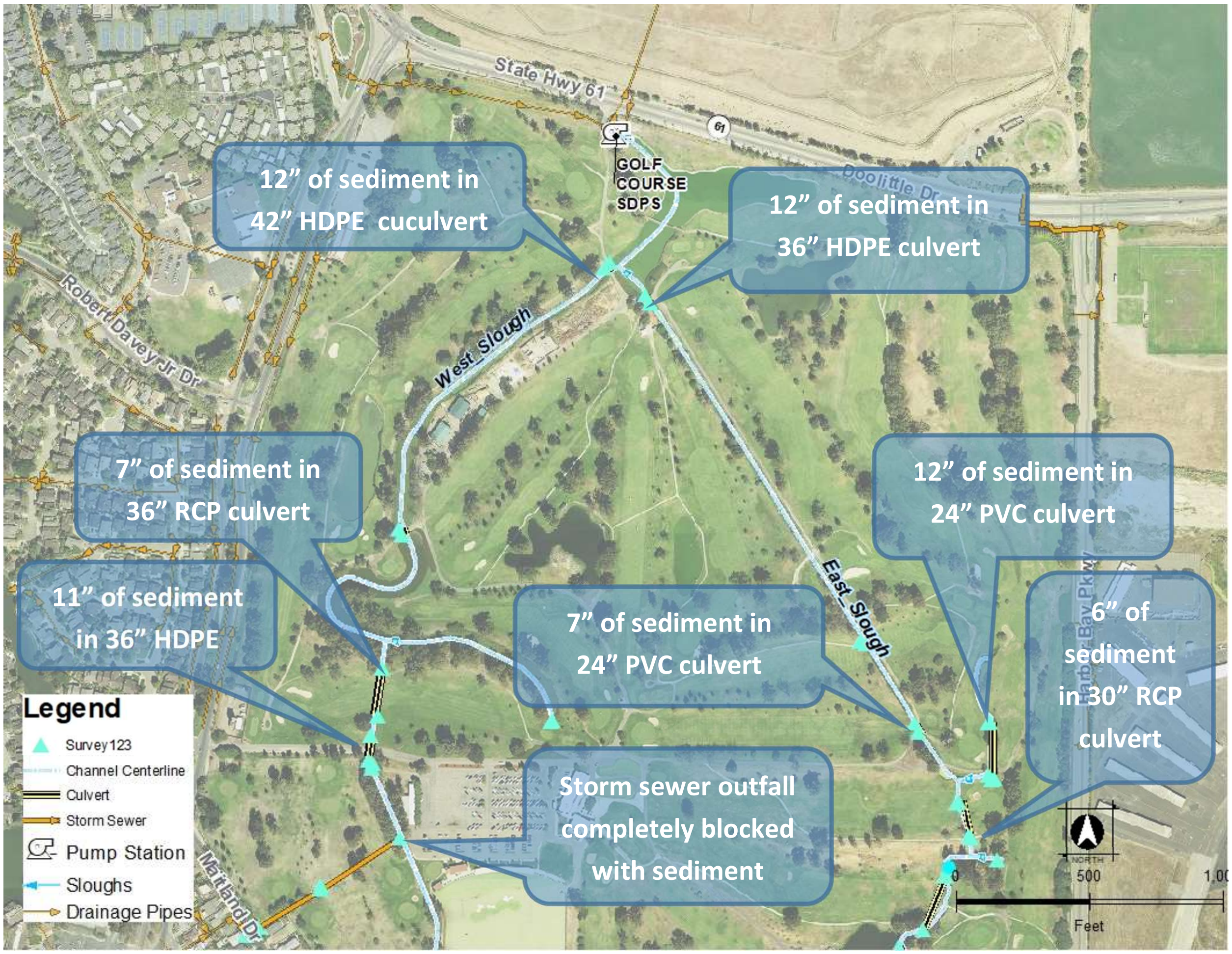


Figure 17 – Survey123 O&M Assessment Locations - North ("Immediate" O&M Condition Rating Called Out)

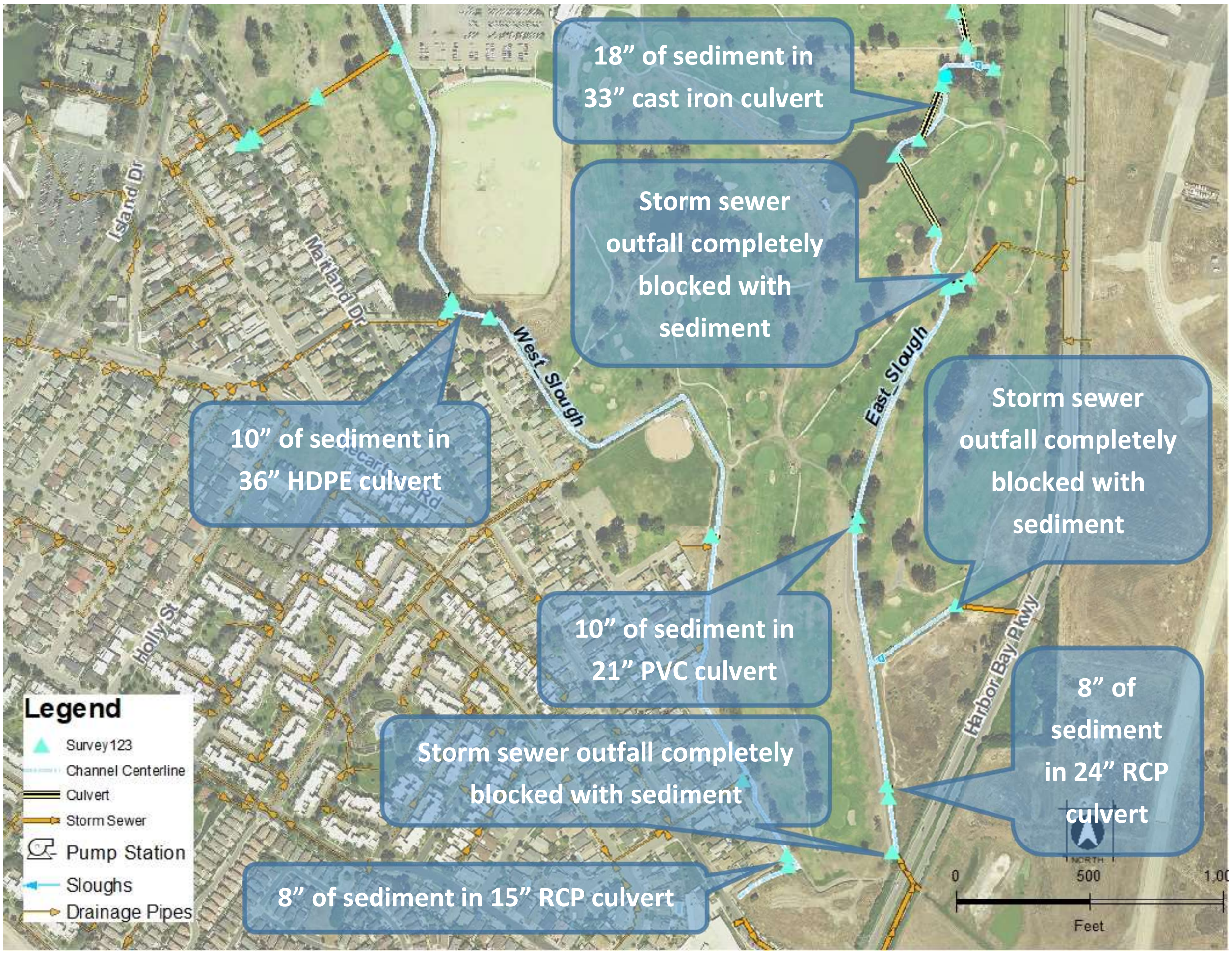


Figure 18 – Survey123 O&M Assessment Locations - South (“Immediate” O&M Condition Rating Called Out)

8 DISCUSSION

By analyzing the data collected in Section 6 and the results in Section 7, it appears likely that flooding witnessed on Maitland Drive and Harbor Bay Parkway was caused mostly by accumulated sediment in the system.

The sloughs are prone to sediment collecting at the bottom of the channel due to multiple low points in the slough's profile (Appendix A), and standing water caused by backwater from the Golf Course SDPS (Figure 4). Referencing the 1995 slough dredging plan (Section 5.1) and Appendix A, approximately 1.5 ft – 2 ft of sediment has accumulated in the sloughs over the past 27 years.

Sedimentation effects the ability of the Golf Course storm sewer system to drain runoff to the Golf Course SDPS in three ways: reducing the slough's hydraulic capacity, reducing the hydraulic capacity of the slough's culverts, and blocking storm sewer outfalls discharging into the sloughs.

Sediment accumulation in the slough decreases its hydraulic capacity, reducing the quantity of runoff it can safely drain to the Golf Course SDPS. The reduced capacity causes higher hydraulic grade lines (HGL), which can back up into storm sewer systems draining into the sloughs. Low-lying areas draining to the Golf Course sloughs are particularly susceptible. The storm sewer system draining the Maitland Drive neighborhood has approximately 2 feet of elevation difference to the slough's bank. The slough's higher HGLs can reduce Maitland Drive's ability to drain into the sloughs.

As shown in Figure 17 and Figure 18, sedimentation accumulated inside the slough's culverts, reducing the hydraulic capacity by 20% to 50%. This has a similar effect as the reduced slough capacity described above. HGLs upstream of the culvert will increase, potentially affecting low-lying areas adjacent to the culvert.

Storm sewers draining residential and street drainage from Maitland Drive and Harbor Bay Parkway were observed to be partially or completely blocked with sediment at their outfall to the Golf Course sloughs. Figure 19 shows the location of the outfalls, while Figure 20 show their images (Images in Figure 20 were taken after exploratory excavation by City staff in early March of 2022. The drawn red line shows the approximate level of sediment prior to excavation). The ability of the outfall to discharge the runoff collected by storm sewers upstream is severely reduced by the sediment. Observations by City staff of the ponding on Harbor Bay Parkway draining in 3+ days is explained by Figure 20 (b), (c), and (d). The runoff cannot get into the sloughs because the sediment blocking the outfall.

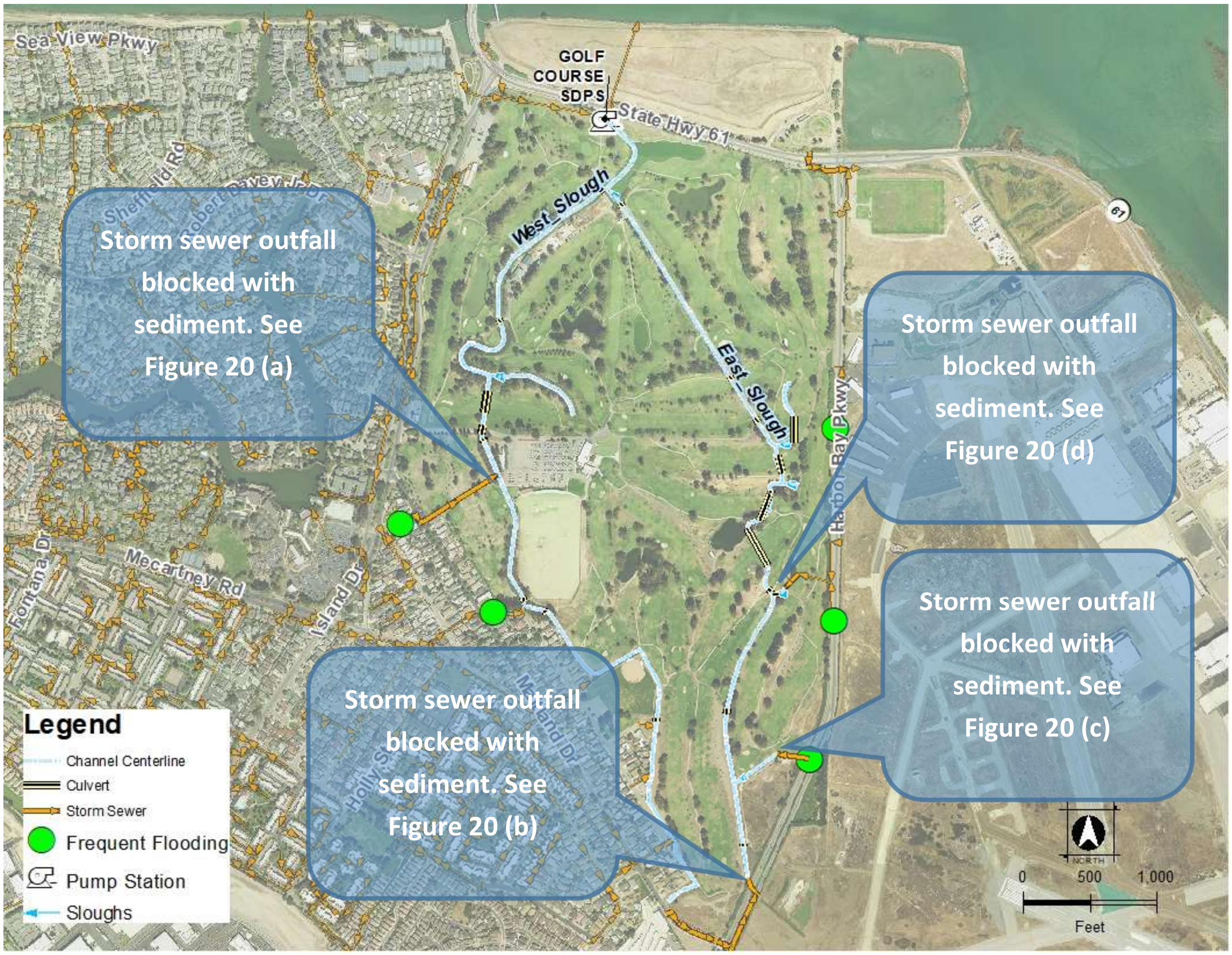


Figure 19 – Locations of Storm Sewers Outfalls Blocked with Sediment

(a) (b) (c) (d)

Figure 20 – Images of Storm Sewer Outfalls Blocked with Sediment (Red Line Shows Approximate Level of Sediment Prior to Excavation)

To a lesser extent, other factors such as groundwater and structurally compromised assets contribute to the Golf Course storm system's flooding. The 2020 groundwater report published by Silvestrum Climate Associates (Section 5.2) concluded the City is highly susceptible to shallow groundwater. Figure 21, taken from their report, shows the depth to shallow groundwater adjacent to Doolittle Drive on Bay Farm Island can range between 0.5 feet and 2.5 feet, depending on the time of year. Any location in the slough with a depth larger than 2.5 feet below its bank will most likely experience groundwater intrusion throughout the calendar year. In fact, water surface elevations measured in the field were shown to be within the depth tolerance of Figure 21, concluding the standing water observed in the northern portion of the sloughs is most likely from shallow groundwater intrusion. The consequence of groundwater draining into the sloughs is that the capacity is further reduced. Groundwater is occupying hydraulic capacity and storage which, if not present, could be used to convey and store storm runoff.

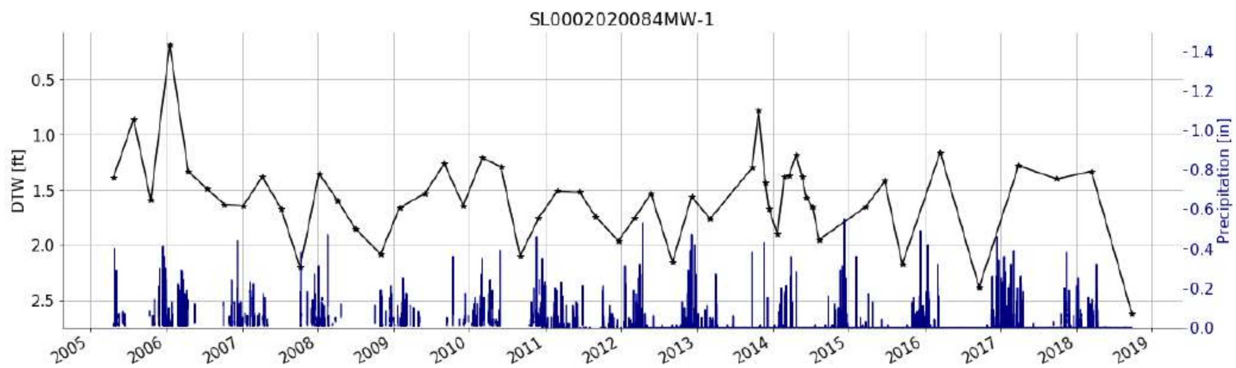


Figure 21 – Depth to Groundwater (DTW) at Doolittle Drive on Bay Farm Island[1]

Lastly, several slough culverts were observed to have structural deficiencies (Section 7.2). Sinkholes and deformations reduce a culverts hydraulic capacity resulting in increased HGLs upstream. See Figure 22. Spalling concrete and rusting corrugated metal pipes can result in a complete failure of the culvert, resulting in sinkholes and increased upstream HGLs.

[1] City of Alameda, The Response of the Shallow Groundwater Layer and Contaminants to Sea Level Rise, September 2020, Silvestrum Climate Associates

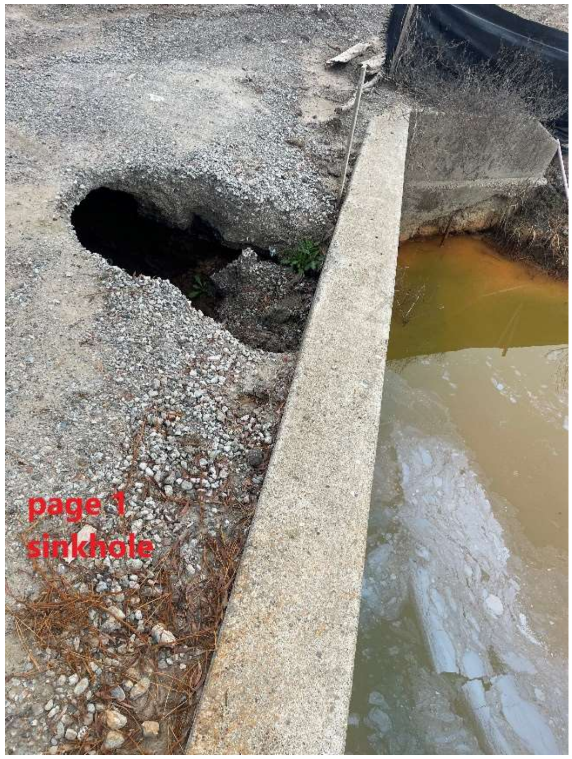


Figure 22 – Sinkhole in 36" PVC Culvert Just Upstream of the Golf Course SDPS Retention Pond on the East Slough

9 RECOMMENDATIONS

The following recommendations were made to reduce the flooding observed on Maitland Drive and Harbor Bay Parkway.

Immediate actions will help alleviate flooding, but the reduction cannot be quantified. Next step actions include the construction of a hydrologic and hydraulic (H&H) model, which can be used to quantify the near-term recommendations and other potential solutions.

Immediate actions (in order of importance):

1. Remove sediment blocking the outfalls of storm sewers discharging into the sloughs. In March of 2022, City crews have already completed this task, but it is stated in this report to signify its importance.
2. Remove sediment from slough culverts. As shown in Figure 17 and Figure 18, several slough culverts have significant sediment accumulation, reducing its conveyance capacity from 20% to 50% less than design capacity. Removing the sediment will allow runoff to drain through the culverts in a more hydraulicly efficient manner, thus reducing the HGL upstream.
3. Lower the Golf Course SDPS "pump-on" elevation. As stated in Section 8, groundwater drains into the sloughs, reducing its hydraulic capacity. Lowering the pump-on elevation will continuously pump the groundwater out of the sloughs, leaving more conveyance and storage for runoff during storm events. This new pump station operation can be seasonally implemented in the winter, or before known rainstorms. During summer months, the pump station can be operated as it is now to keep standing water in the sloughs for aesthetic purposes.
4. Repair structurally compromised culverts. Like sediment-filled culverts, structurally compromised culverts have a reduced hydraulic capacity. It is recommended culverts called out in Figure 16 be replaced or repaired.
5. Install duck-billed flap gates at outfalls of storm sewers discharging into the sloughs. The flap gates will prevent sediment from collecting inside City-owned storm sewers, reducing the annual maintenance of the asset.

Next step actions:

6. Develop a H&H model of the Golf Course storm system. An H&H model is beneficial because it can be used to:
    a. Quantify the benefits of the immediate action recommendations (1-5). Immediate action recommendations will alleviate the existing flooding extent but won't be able to quantify the benefit or reduction without an H&H model.
    b. Verify slough dredging. The H&H model would quantify the amount of dredging required to keep HGLs low enough to not effect adjacent low-lying areas and storm sewer systems.
    c. Develop a long-term maintenance plan. An H&H model can simulate the reduction of slough and culvert conveyance by sedimentation and recommend a maintenance plan to avoid flooding.

   d. Develop Golf Course storm system improvements to reduce maintenance activities and increase flood protection. Examples of improvements include, but not limited to: larger pump station, paved sloughs, wider sloughs, bypass pipe system to the pump station, and identify locations to elevated the outfall of storm sewers discharging into the sloughs.

7. Determine the impacts of sea-level rise. According to the September 2020 groundwater report by Silvestrum Climate Associates, sea-level rise will have a direct impact on the City's shallow ground water. Sea level rise would increase the amount of ground water flowing to the Golf Course SDPS and reduce its efficiency due to increased water surface elevations in San Leandro Bay. Predicted groundwater elevations can be coupled with the H&H model to assess the impacts and the necessary mitigation.

City of Alameda
Chuck Corica Golf Course

Slough Stationing

Alameda County, California
July 2022

Legend

Channel Centerline
Culvert
Storm Sewer
Pump Station

NORTH

0 500 1,000
Feet

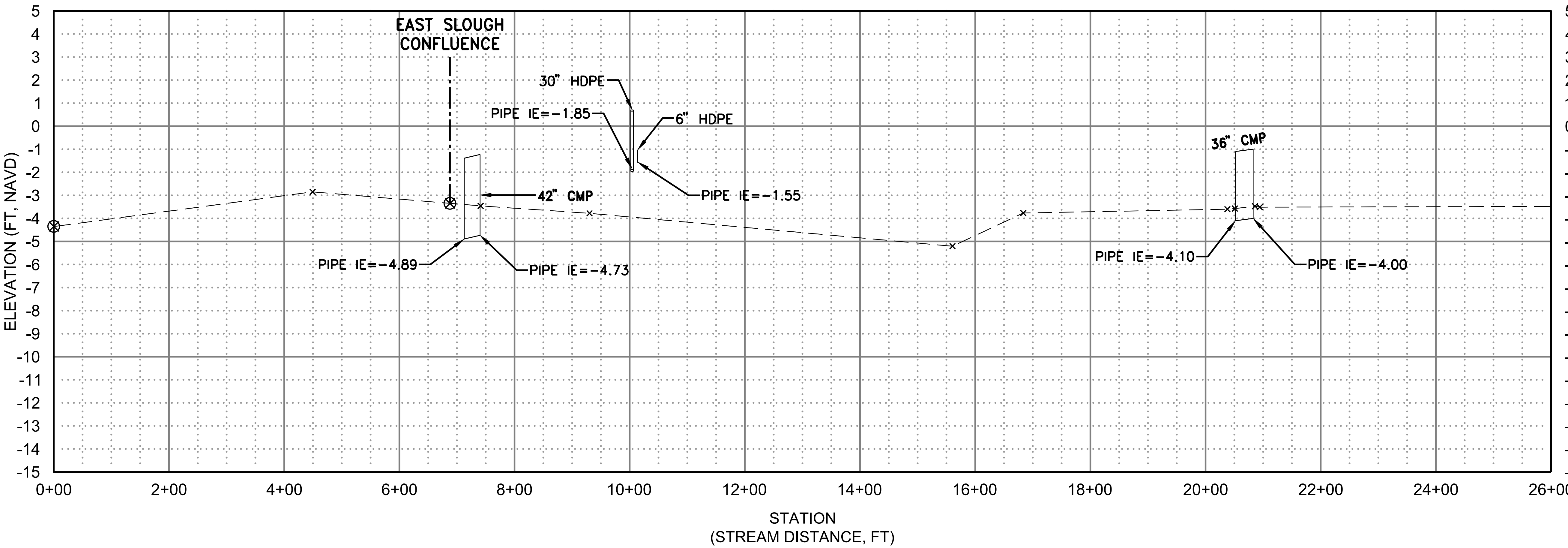
Vicinity Map

PRELIMINARY

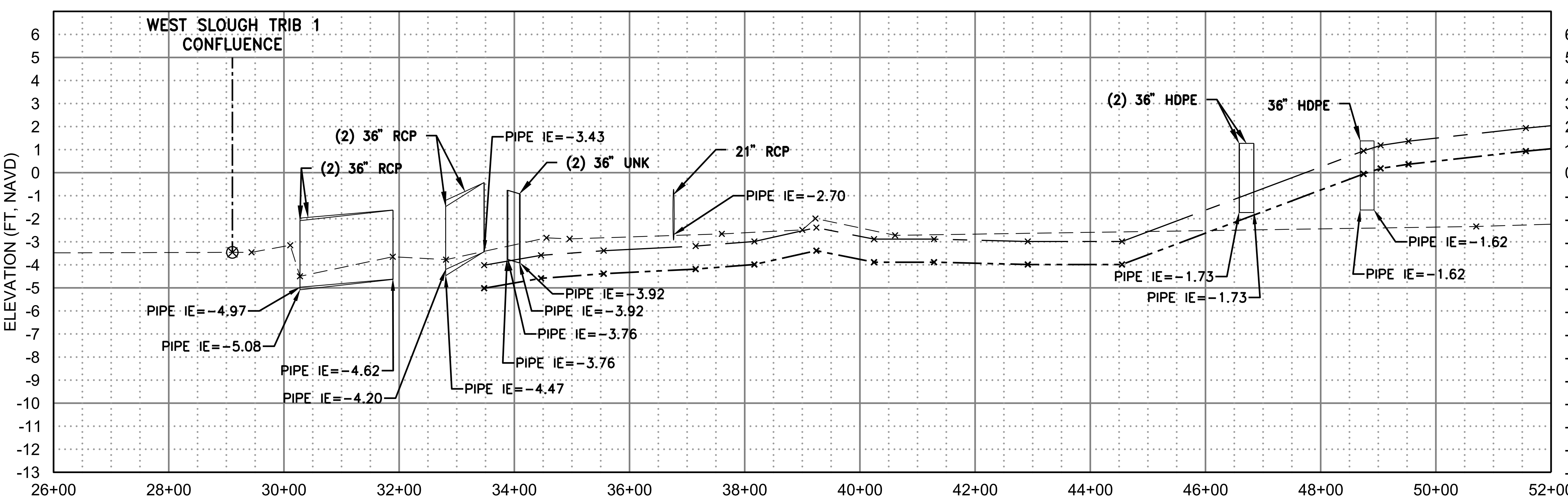
Appendix A

WOOD RODGERS

WEST SLOUGH STA 0+00 TO STA 26+00



WEST SLOUGH STA 26+00 TO STA 52+00



STATION
(STREAM DISTANCE, FT)

LEGEND

LOW FLOW CHANNEL EG	LOW FLOW CHANNEL EG 1995
LOW FLOW CHANNEL FG	LOW FLOW CHANNEL FG 1995
× SURVEY POINT LOCATION	

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME
3301 C ST, BLDG. 100-B TEL 916.341.7760
SACRAMENTO, CA 95816 FAX 916.341.7767

CALIFORNIA

CHUCK CORICA GOLF COURSE

WEST SLOUGH PROFILE

ALAMEDA COUNTY

DATE: 05/31/2022

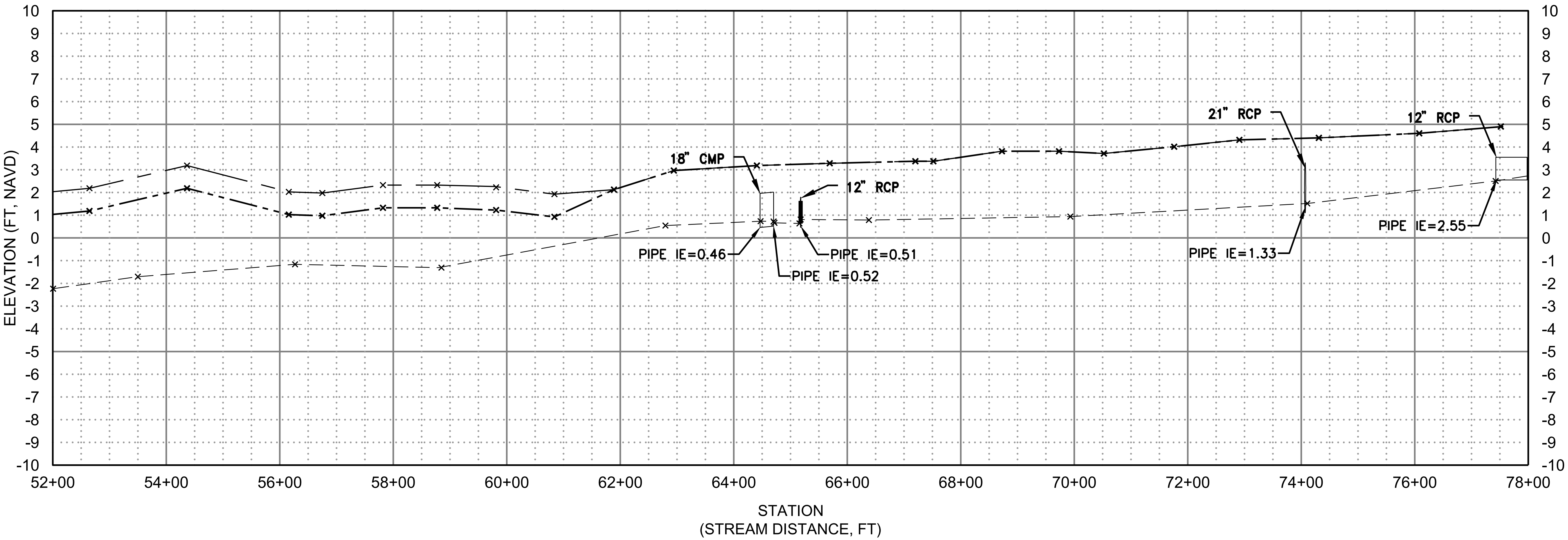
SCALE:
VERT: 1" = 5'
HORIZ: 1" = 200'

PROJECT NO: ----

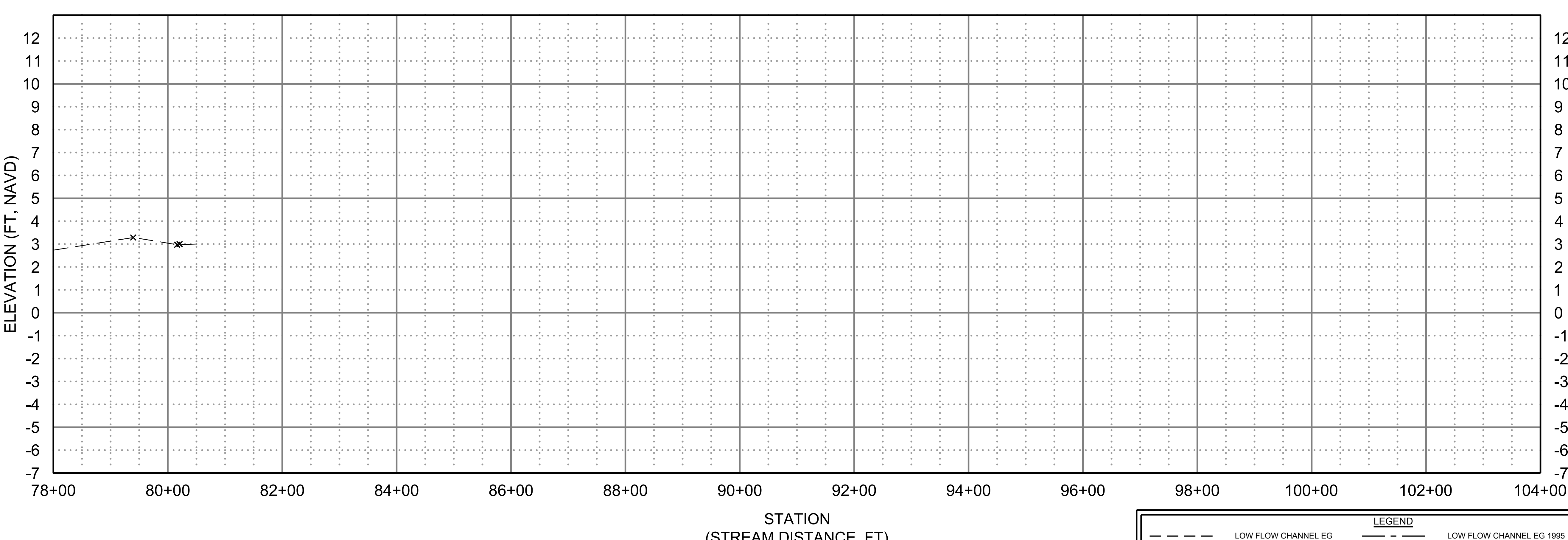
DRAWING: ----

SHT 1 OF 4

WEST SLOUGH STA 52+00 TO STA 78+00



WEST SLOUGH STA 78+00 TO STA 104+00



LEGEND

LOW FLOW CHANNEL EG	LOW FLOW CHANNEL EG 1995	× SURVEY POINT LOCATION	
LOW FLOW CHANNEL FG	LOW FLOW CHANNEL FG 1995		

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME
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SACRAMENTO, CA 95816 FAX 916.341.7767

CHUCK CORICA GOLF COURSE

WEST SLOUGH PROFILE

ALAMEDA COUNTY CALIFORNIA

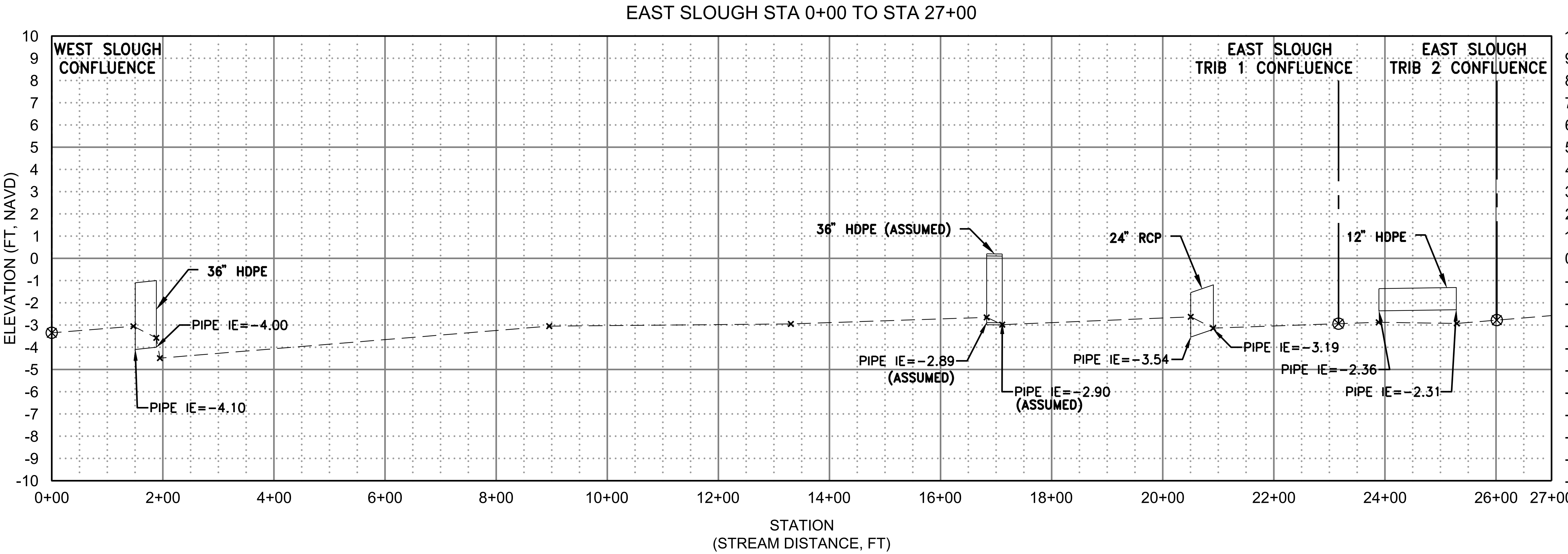
DATE: 05/31/2022

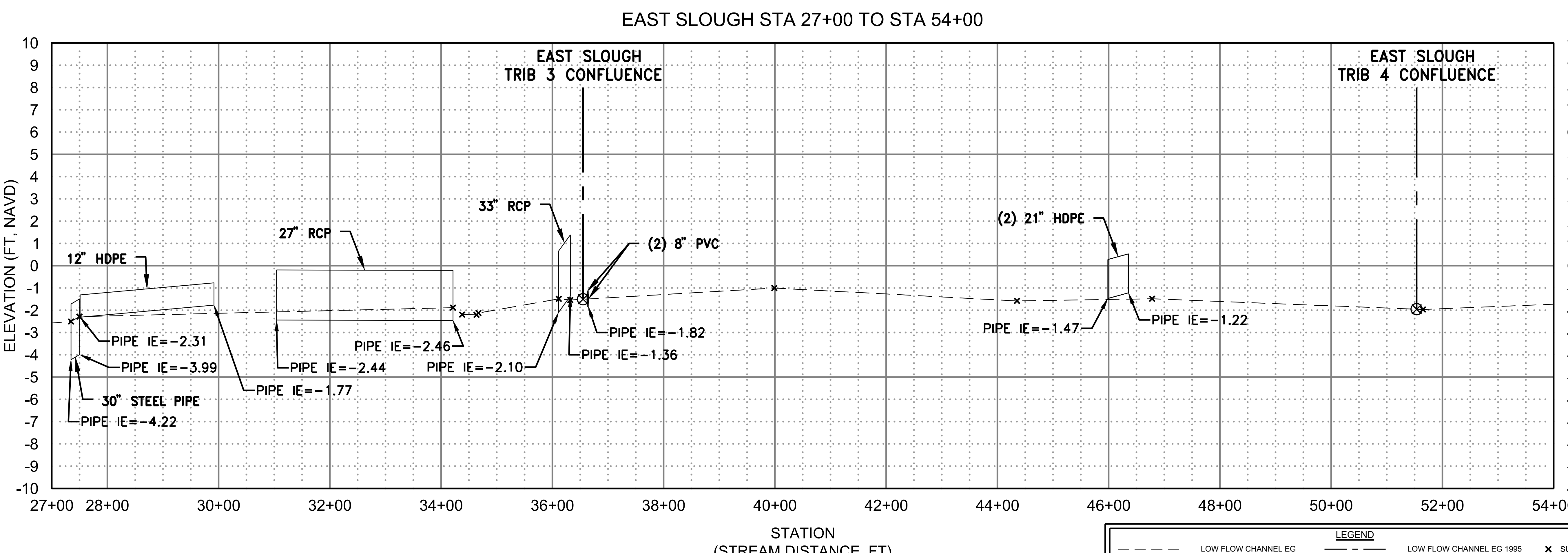
SCALE: VERT: 1" = 5' HORIZ: 1" = 200'

PROJECT NO: ----

DRAWING: ----

SHT 2 OF 4





LEGEND

LOW FLOW CHANNEL EG	LOW FLOW CHANNEL EG 1995	× SURVEY POINT LOCATION
LOW FLOW CHANNEL FG	LOW FLOW CHANNEL FG 1995	

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SACRAMENTO, CA 95816 FAX 916.341.7767

CHUCK CORICA GOLF COURSE
EAST SLOUGH PROFILE

ALAMEDA COUNTY CALIFORNIA

DATE: 05/31/2022

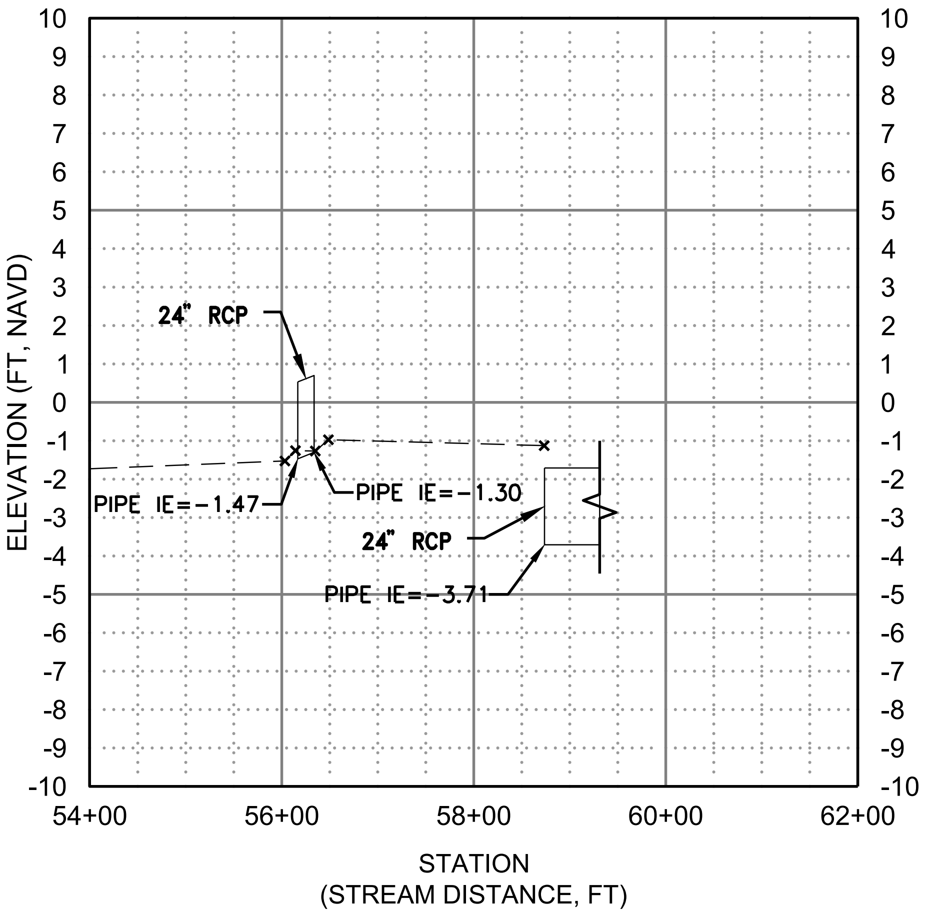
SCALE: VERT: 1" = 5' HORIZ: 1" = 200'

PROJECT NO: ----

DRAWING: ----

SHT 3 OF 4

EAST SLOUGH STA 54+00 TO STA 62+00



LEGEND

LOW FLOW CHANNEL EG	LOW FLOW CHANNEL EG 1995	× SURVEY POINT LOCATION
LOW FLOW CHANNEL FG	LOW FLOW CHANNEL FG 1995	

WOOD RODGERS
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SACRAMENTO, CA 95816 FAX 916.341.7767

CHUCK CORICA GOLF COURSE

EAST SLOUGH PROFILE

ALAMEDA COUNTY CALIFORNIA

DATE: 05/31/2022

SCALE:
VERT: 1" = 5'
HORIZ: 1" = 200'

PROJECT NO: ----

DRAWING: ----

SHT 4 OF 4

J:\Jobs\8426_CityAlameda_OnCall\ChuckCorica_GolfCourse\Civil\Dwg\DLV\4 EAST SLOUGH PROFILE.dwg 7/21/2022 11:35 AM Caela Brazelton