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.

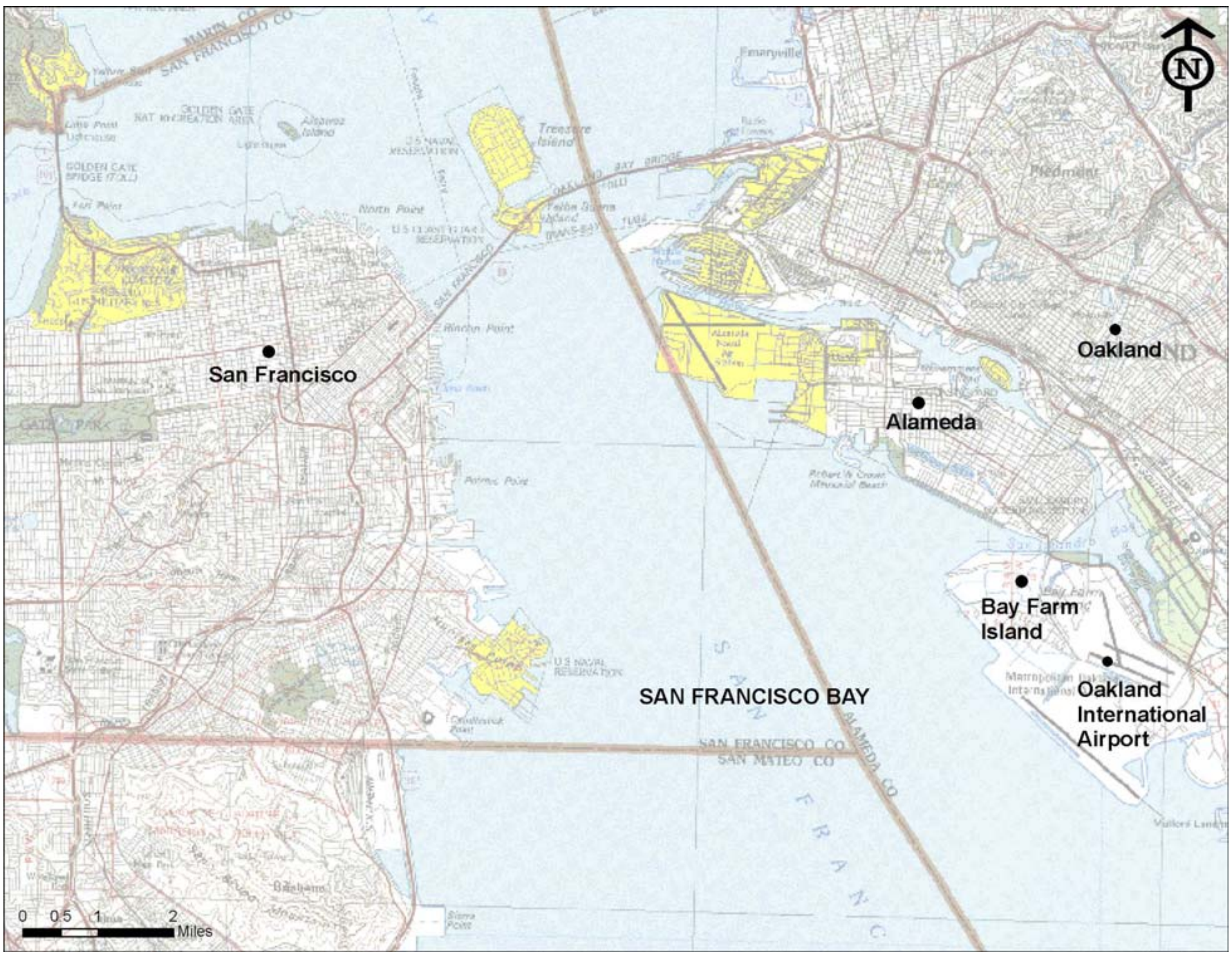


Figure 2-1: Vicinity Map

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.

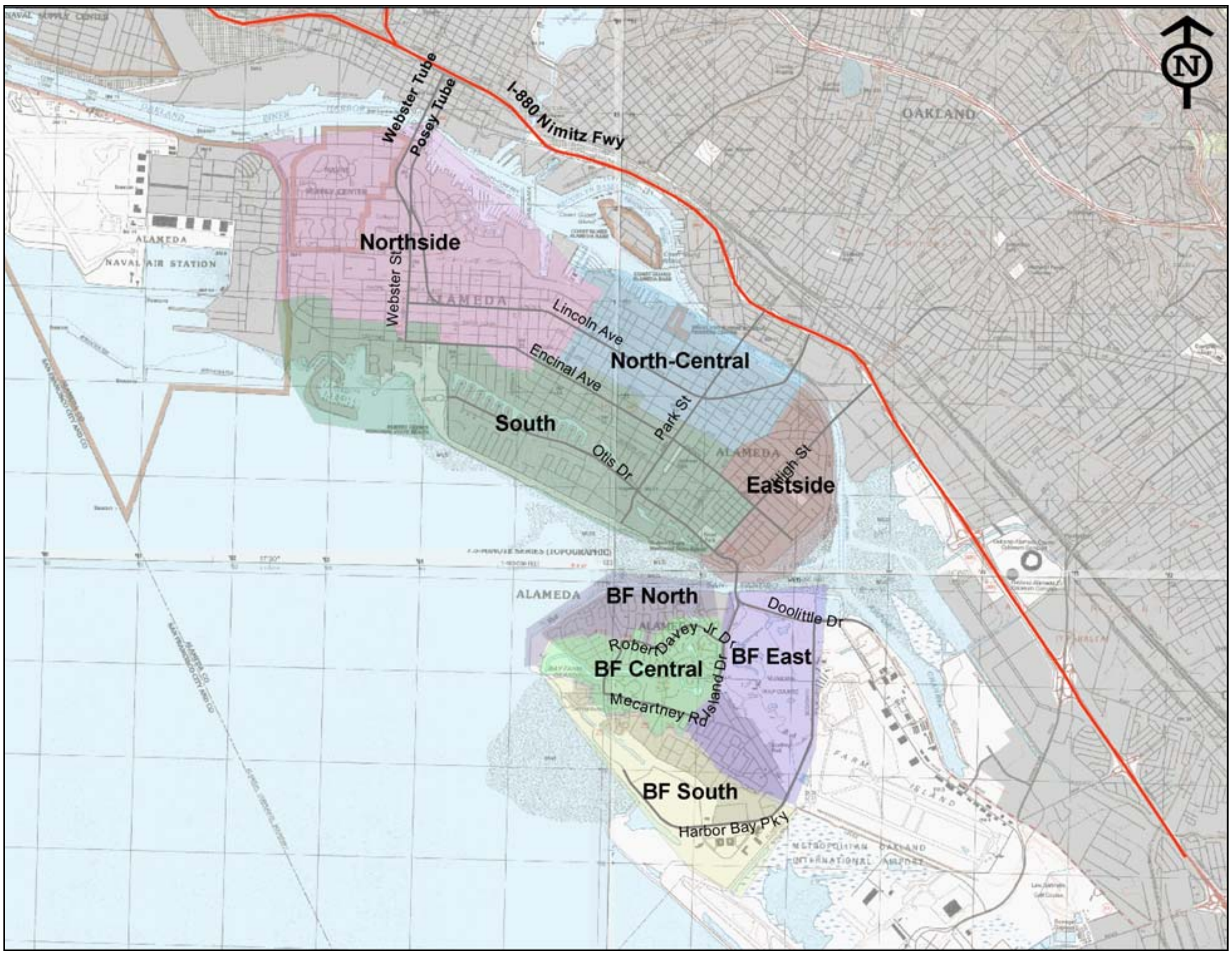


Figure 2-2: Drainage Sub Areas

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.

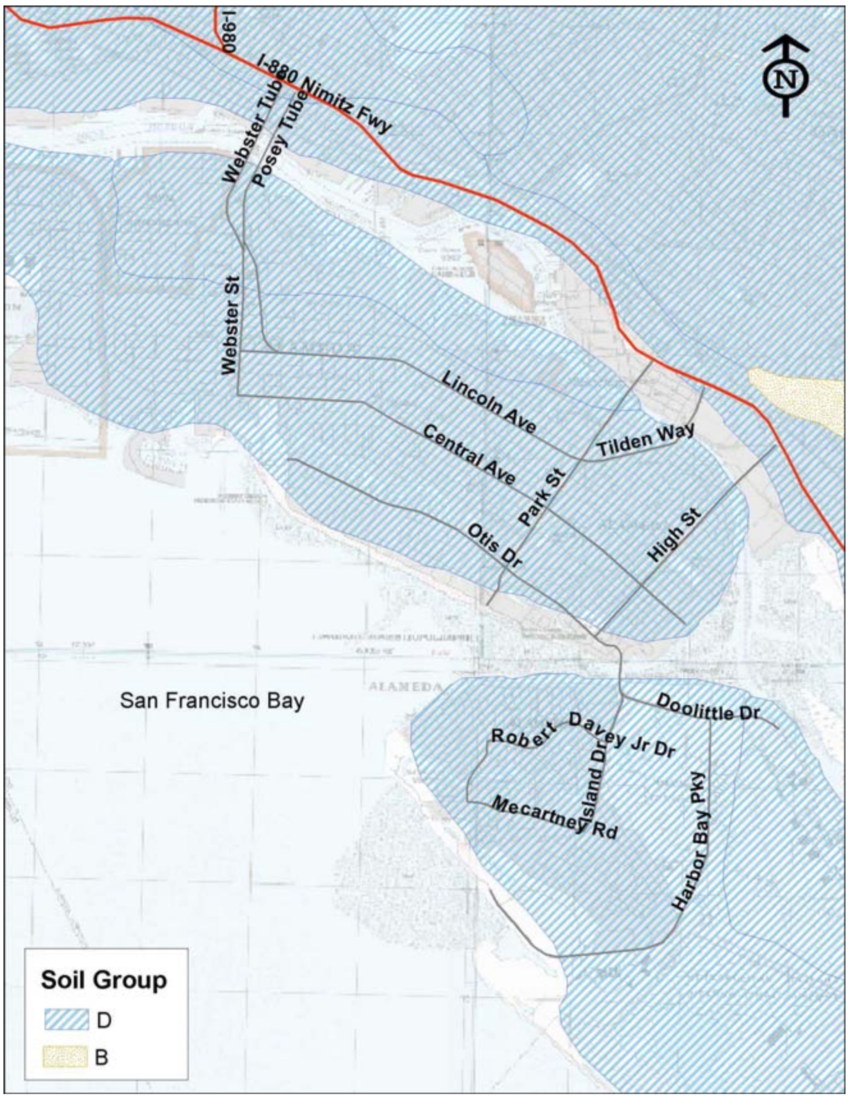


Figure 2-3: NRCS Hydrologic Soil Groups

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.

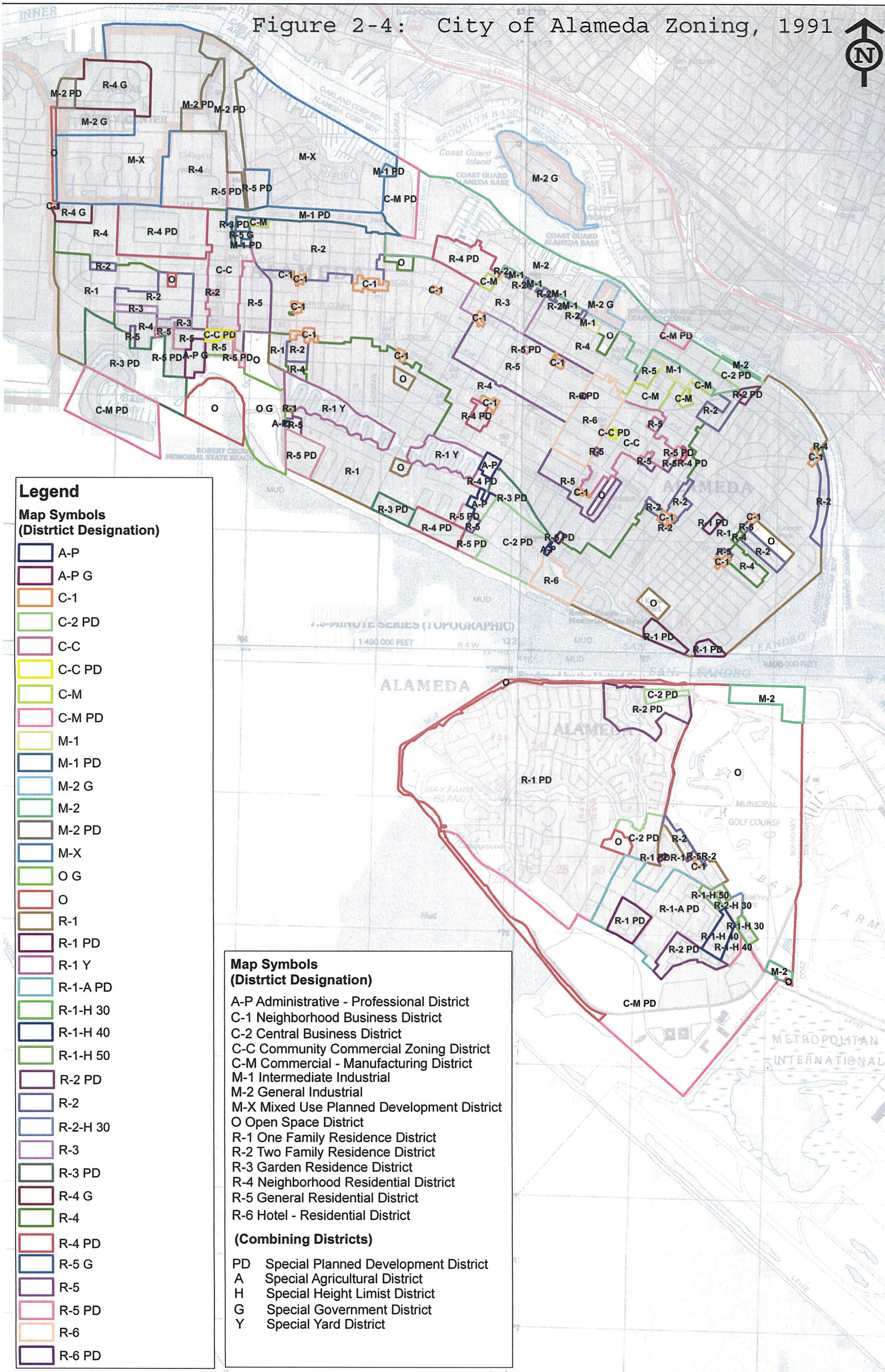
Land Use

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



Legend

Map Symbols (Distrtict Designation)

- A-P
- A-P G
- C-1
- C-2 PD
- C-C
- C-C PD
- C-M
- C-M PD
- M-1
- M-1 PD
- M-2 G
- M-2
- M-2 PD
- M-X
- O G
- O
- R-1
- R-1 PD
- R-1 Y
- R-1-A PD
- R-1-H 30
- R-1-H 40
- R-1-H 50
- R-2 PD
- R-2
- R-2-H 30
- R-3
- R-3 PD
- R-4 G
- R-4
- R-4 PD
- R-5 G
- R-5
- R-5 PD
- R-6
- R-6 PD

Map Symbols (Distrtict Designation)

A-P Administrative - Professional District
C-1 Neighborhood Business District
C-2 Central Business District
C-C Community Commercial Zoning District
C-M Commercial - Manufacturing District
M-1 Intermediate Industrial
M-2 General Industrial
M-X Mixed Use Planned Development District
O Open Space District
R-1 One Family Residence District
R-2 Two Family Residence District
R-3 Garden Residence District
R-4 Neighborhood Residential District
R-5 General Residential District
R-6 Hotel - Residential District

(Combining Districts)

PD Special Planned Development District
A Special Agricultural District
H Special Height Limist District
G Special Government District
Y Special Yard District

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.

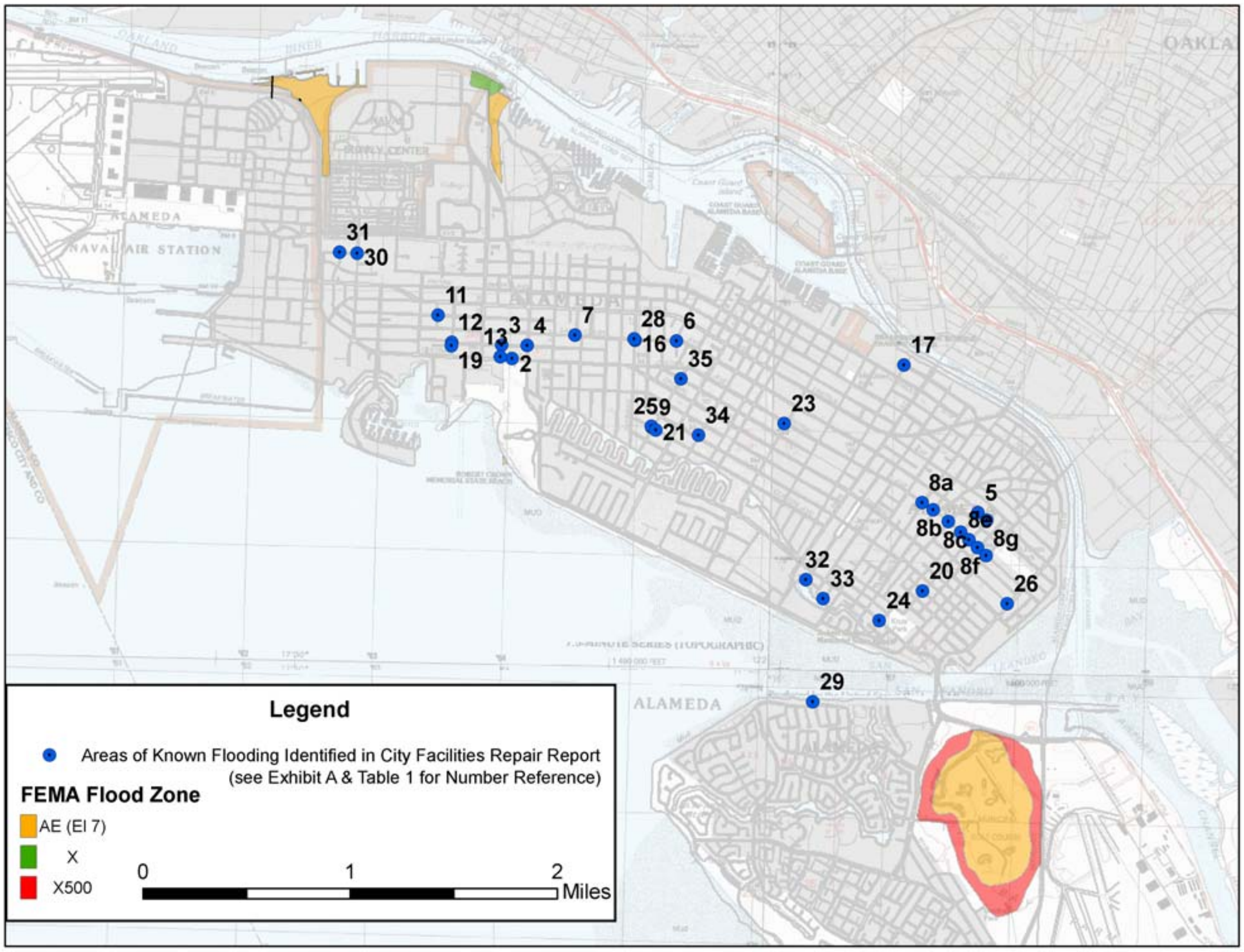


Figure 2-6: Known Flooding Locations

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:

$$\text{NGVD29} - 3.41 \text{ feet} = \text{City of Alameda Datum}$$

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:

$$\text{NGVD29} + 96.59 \text{ feet} = \text{Bay Farm Island Datum}$$

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.

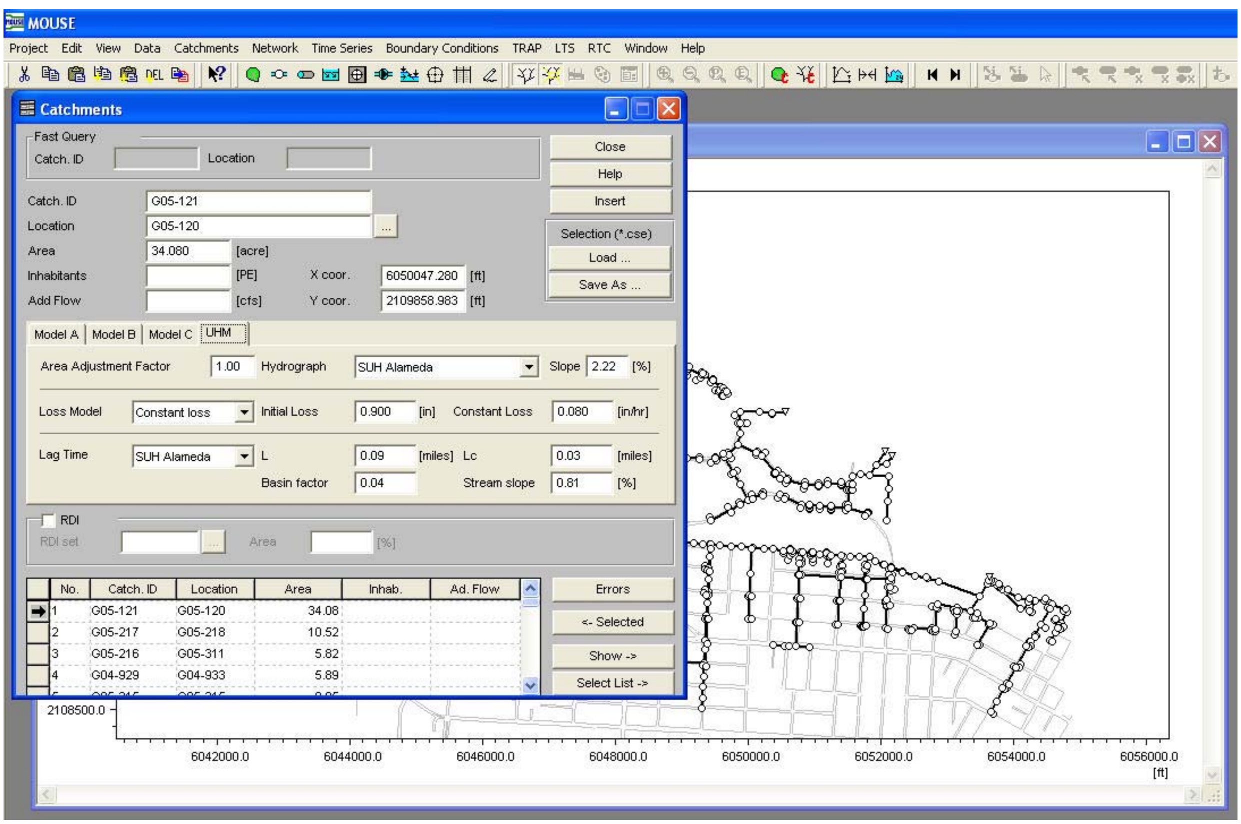


Figure 3-1: MOUSE Input

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.

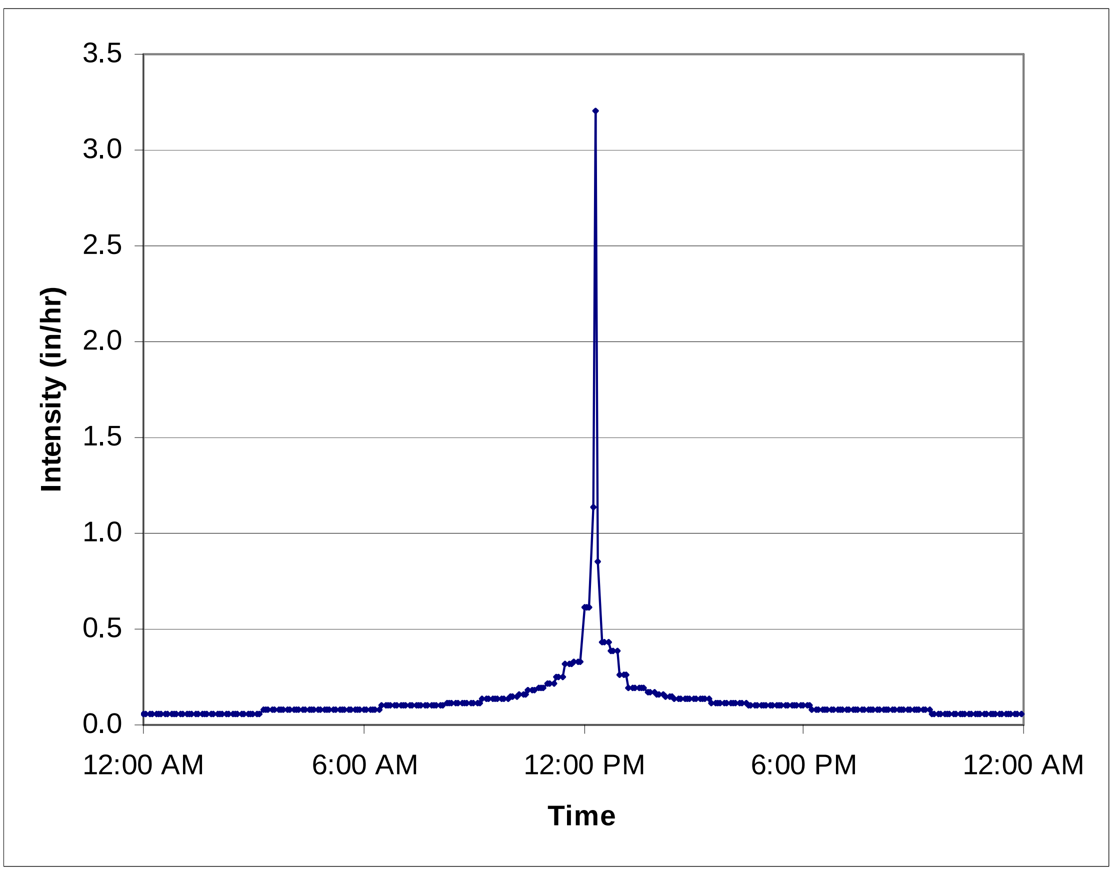


Figure 3-2*: Balanced 10-Year Storm Intensity Graph*

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.

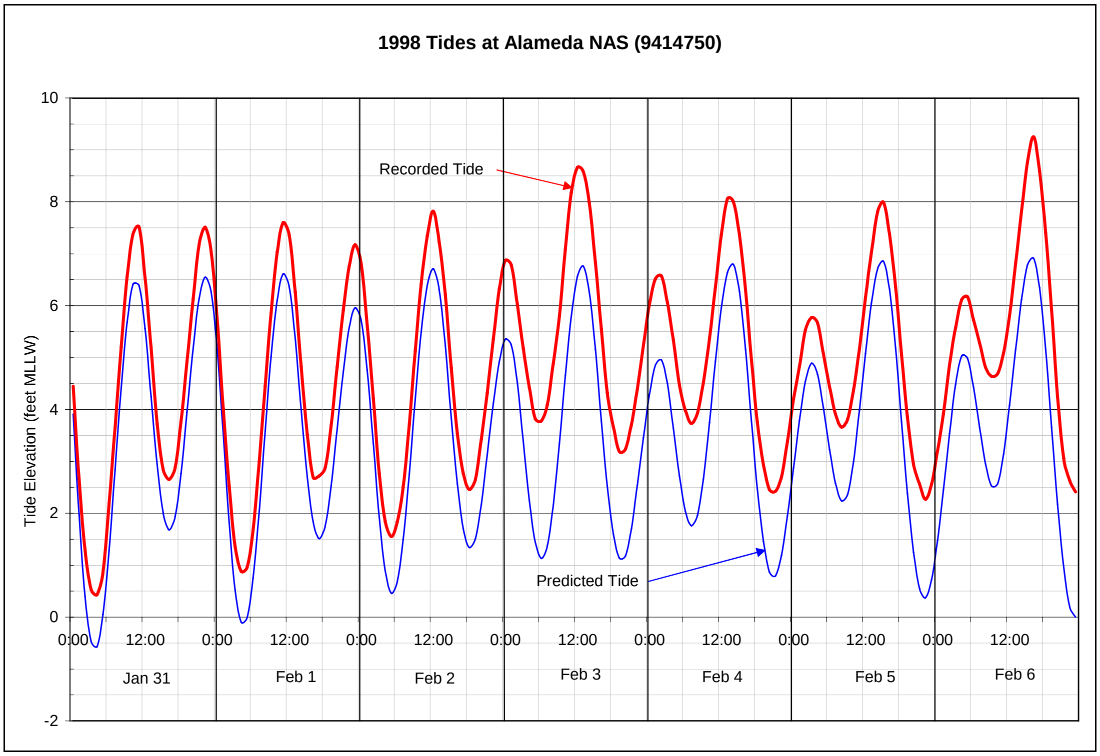


Figure 3-3: Impact of Storm Surge on San Francisco Bay Tides

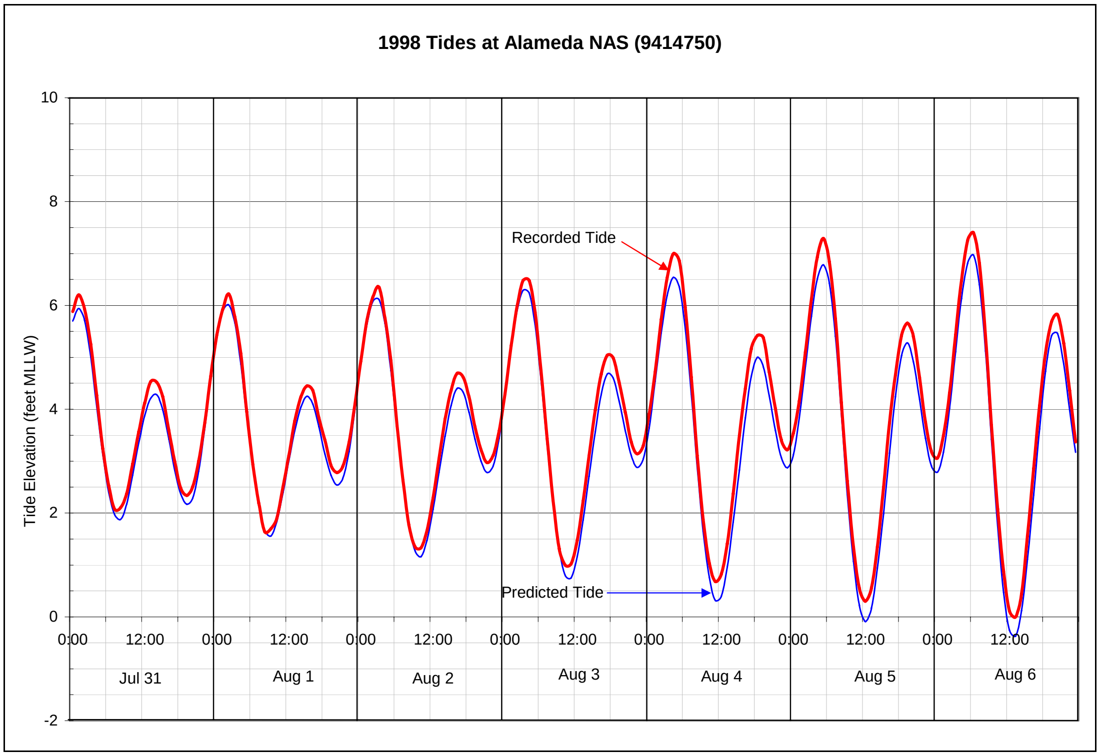


Figure 3-4: Lack of Storm Surge Effect during Summer Months

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.

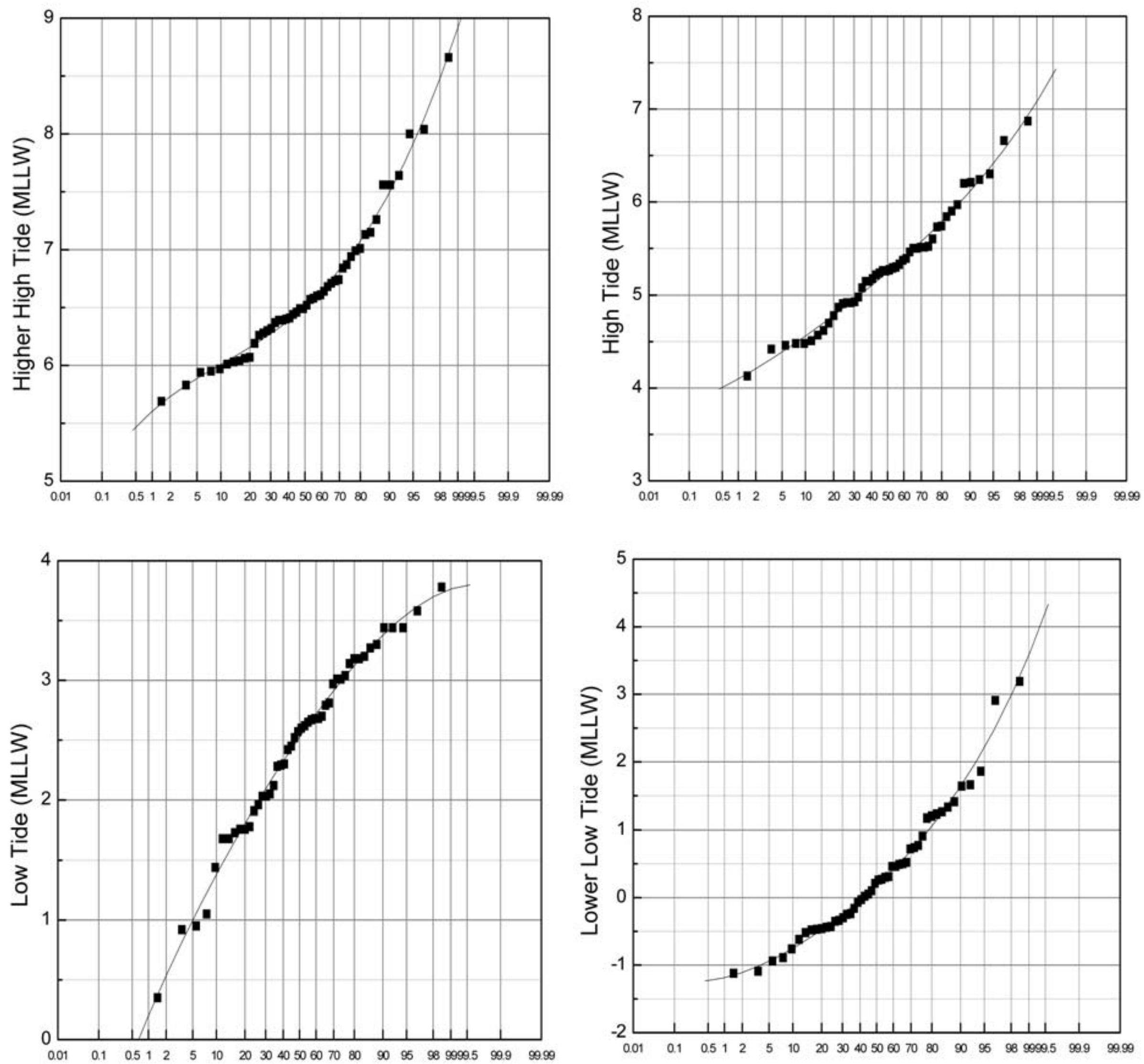


Figure 3-5: Annual Maxima of Tide Cycles Coincident with Peak Annual Runoff

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

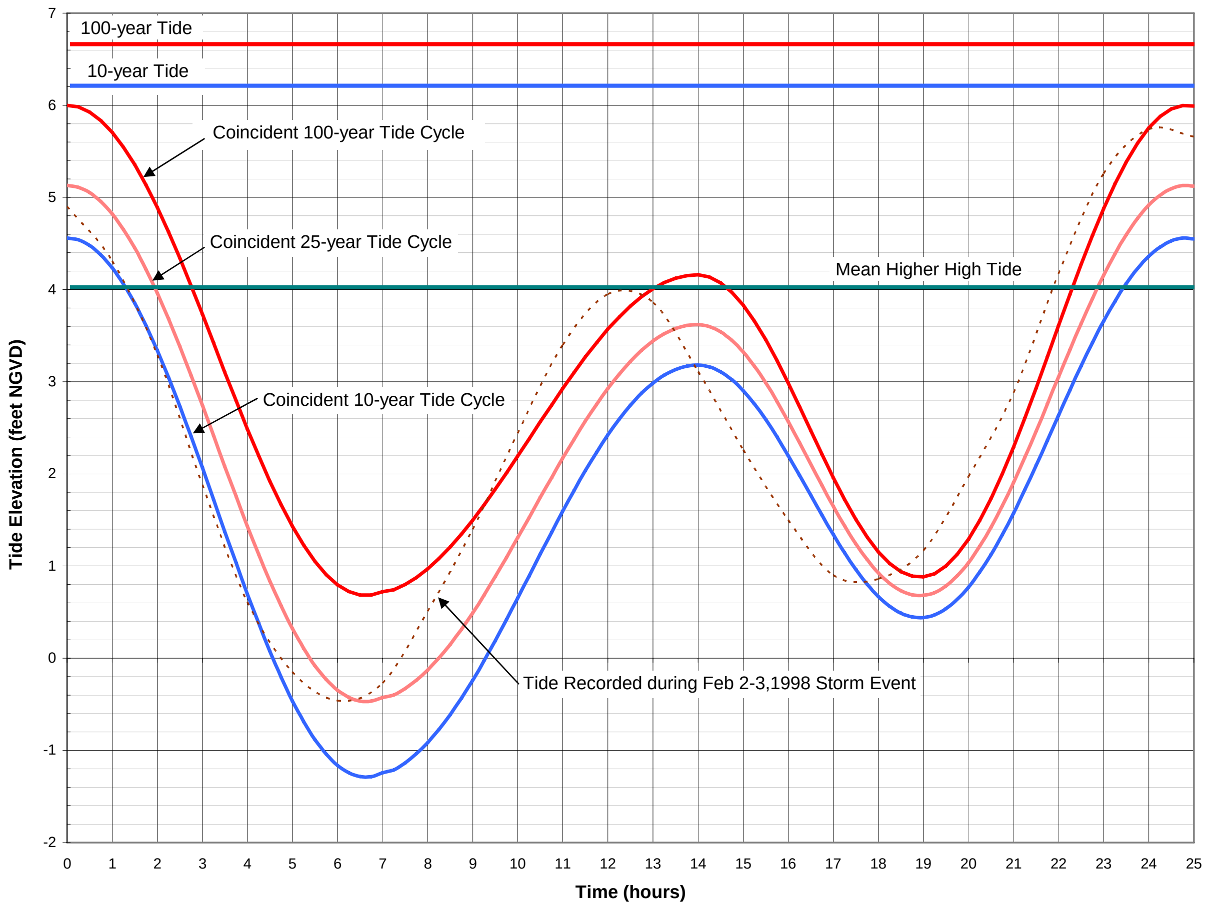
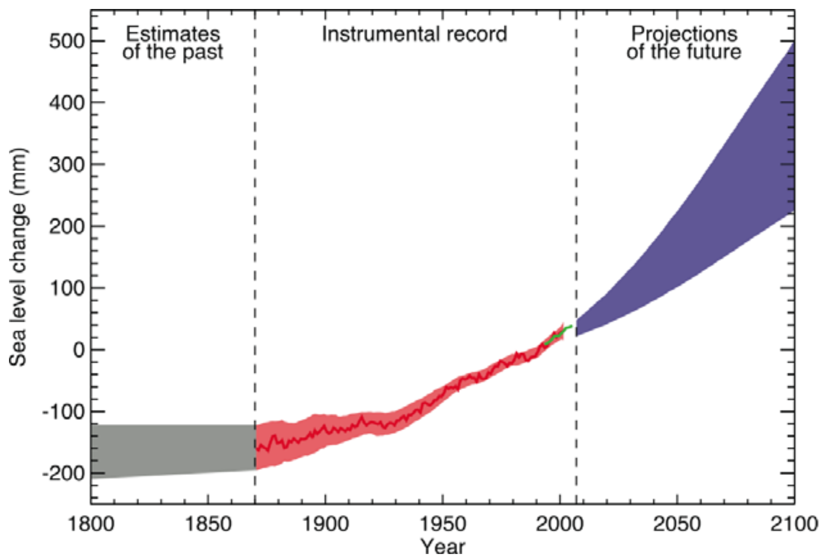


Figure 3-6: Design Boundary Conditions for Storm Drain Analysis

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.

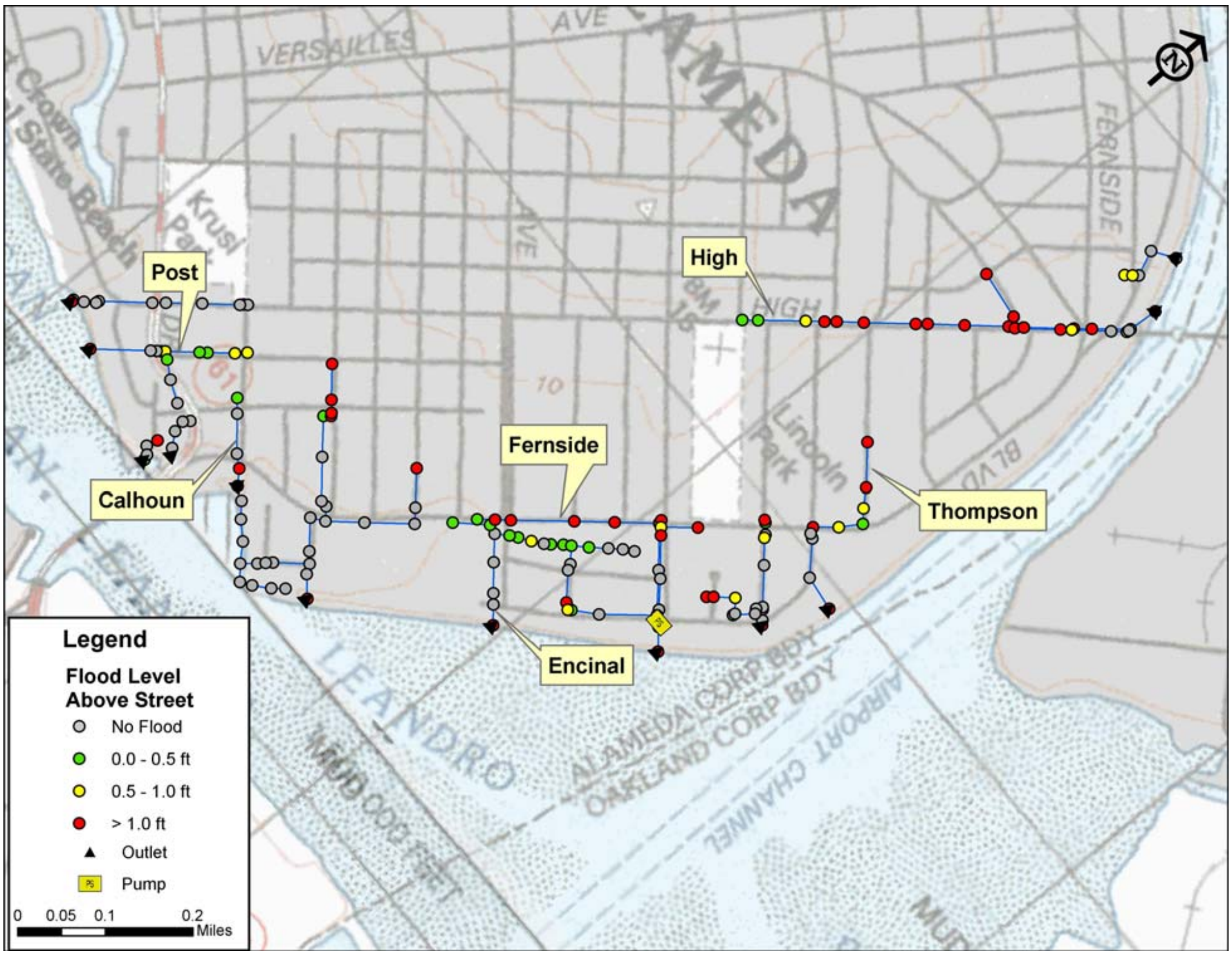


Figure 5-1: Alameda Eastside Area Existing 10-Year Flooding Depths

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.

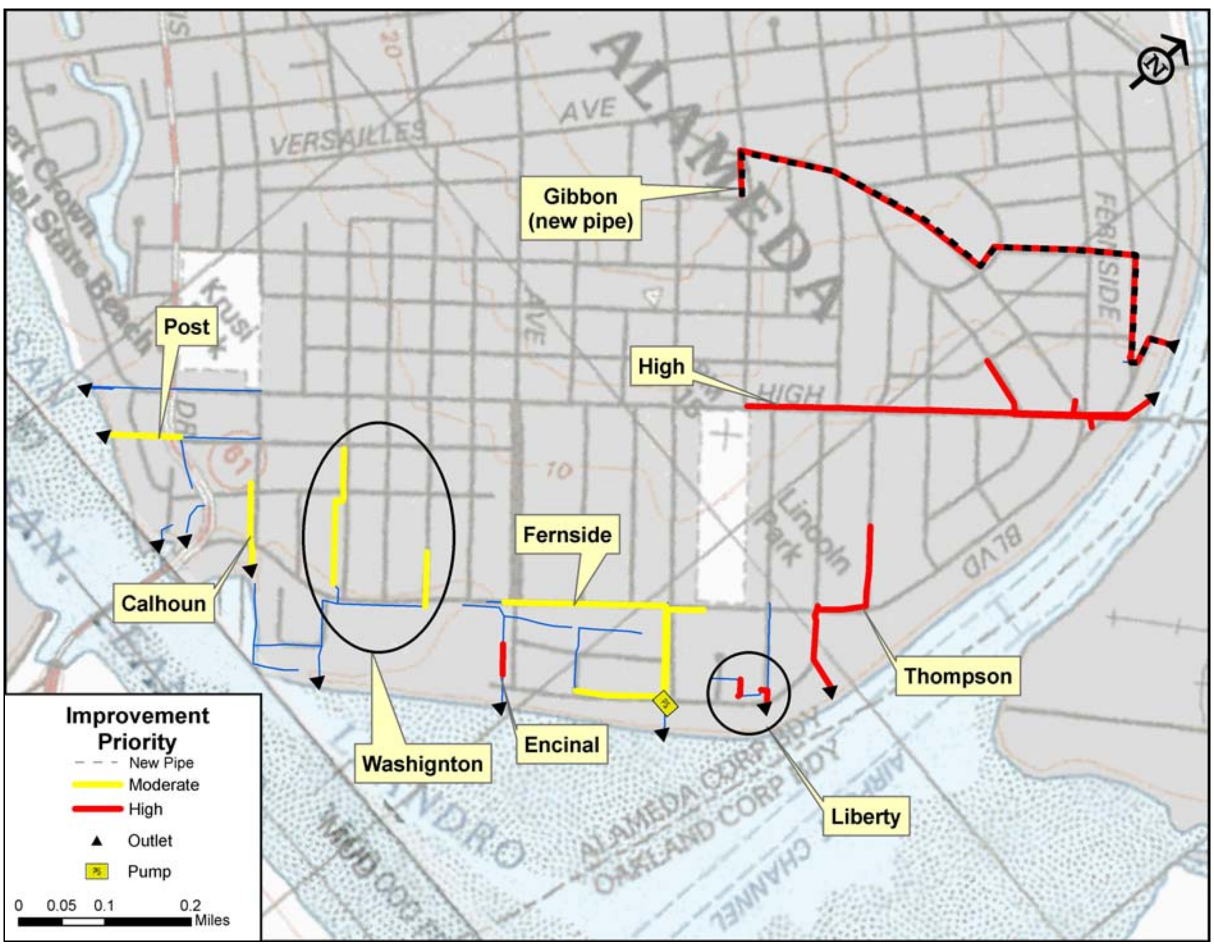


Figure 5-2: Alameda Eastside Area Prioritized 10-Year Improvements

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.

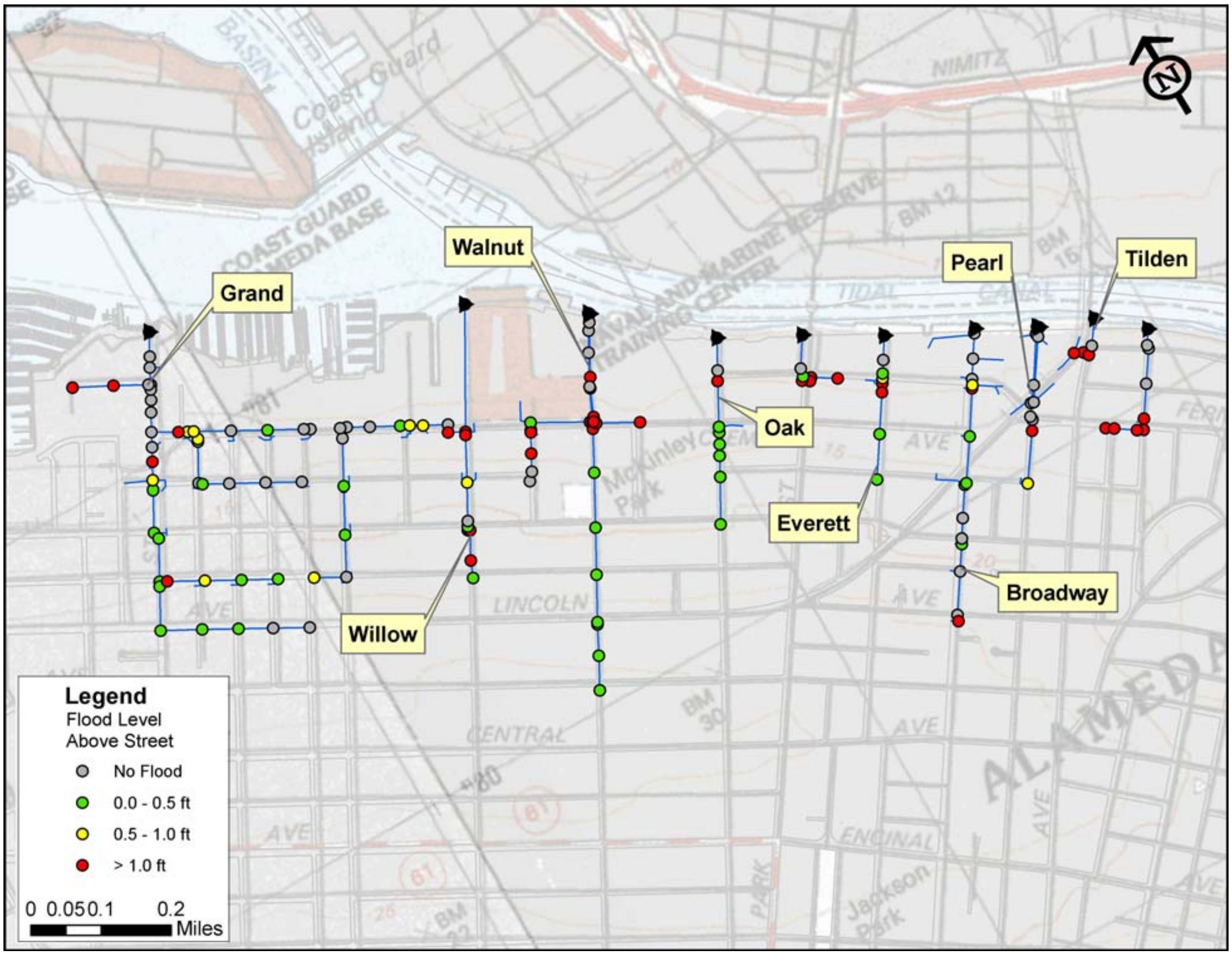


Figure 5-3: Alameda North Central Area Existing 10-Year Flooding Depths

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.

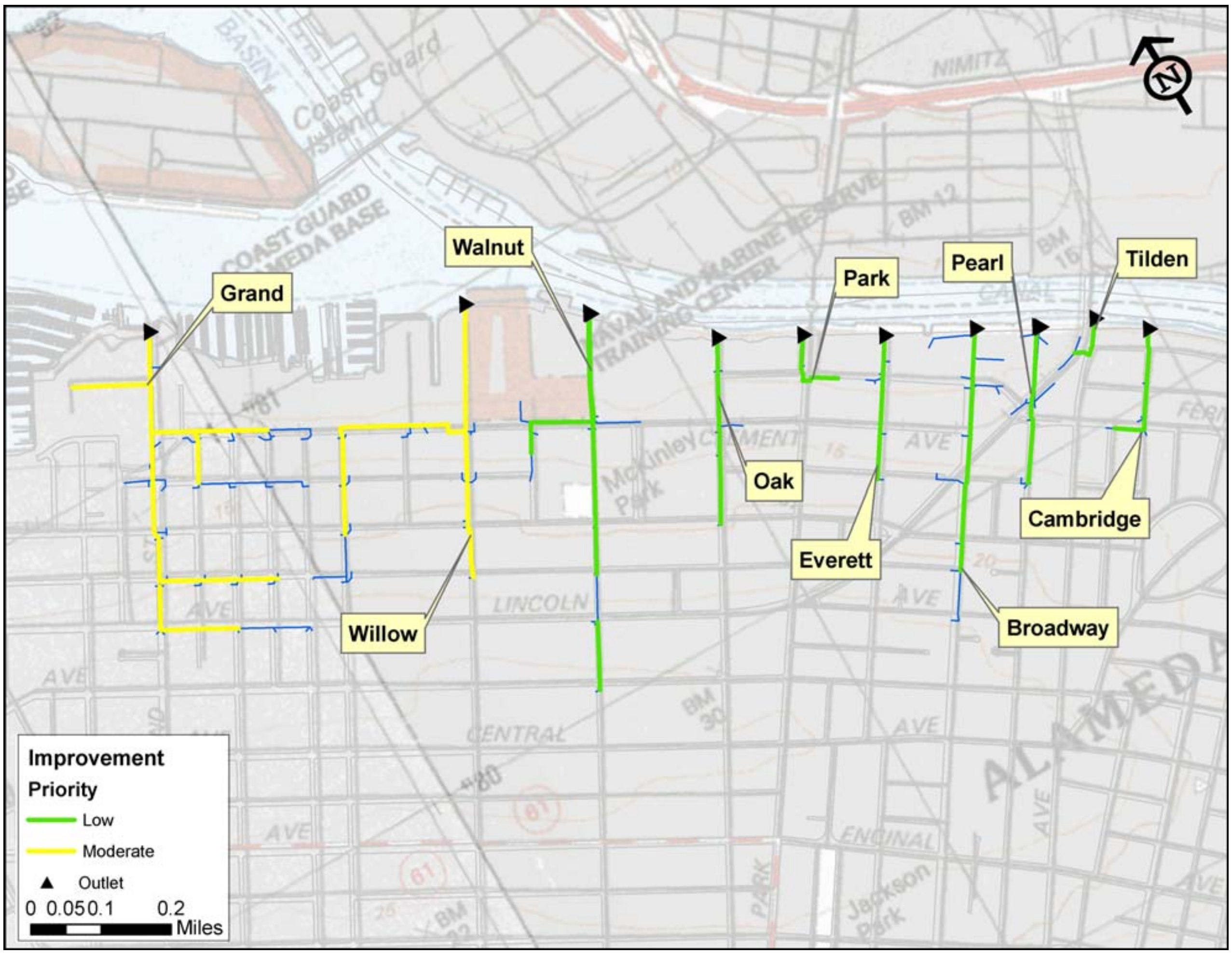


Figure 5-4: Alameda North Central Area Prioritized 10-Year Improvements

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.

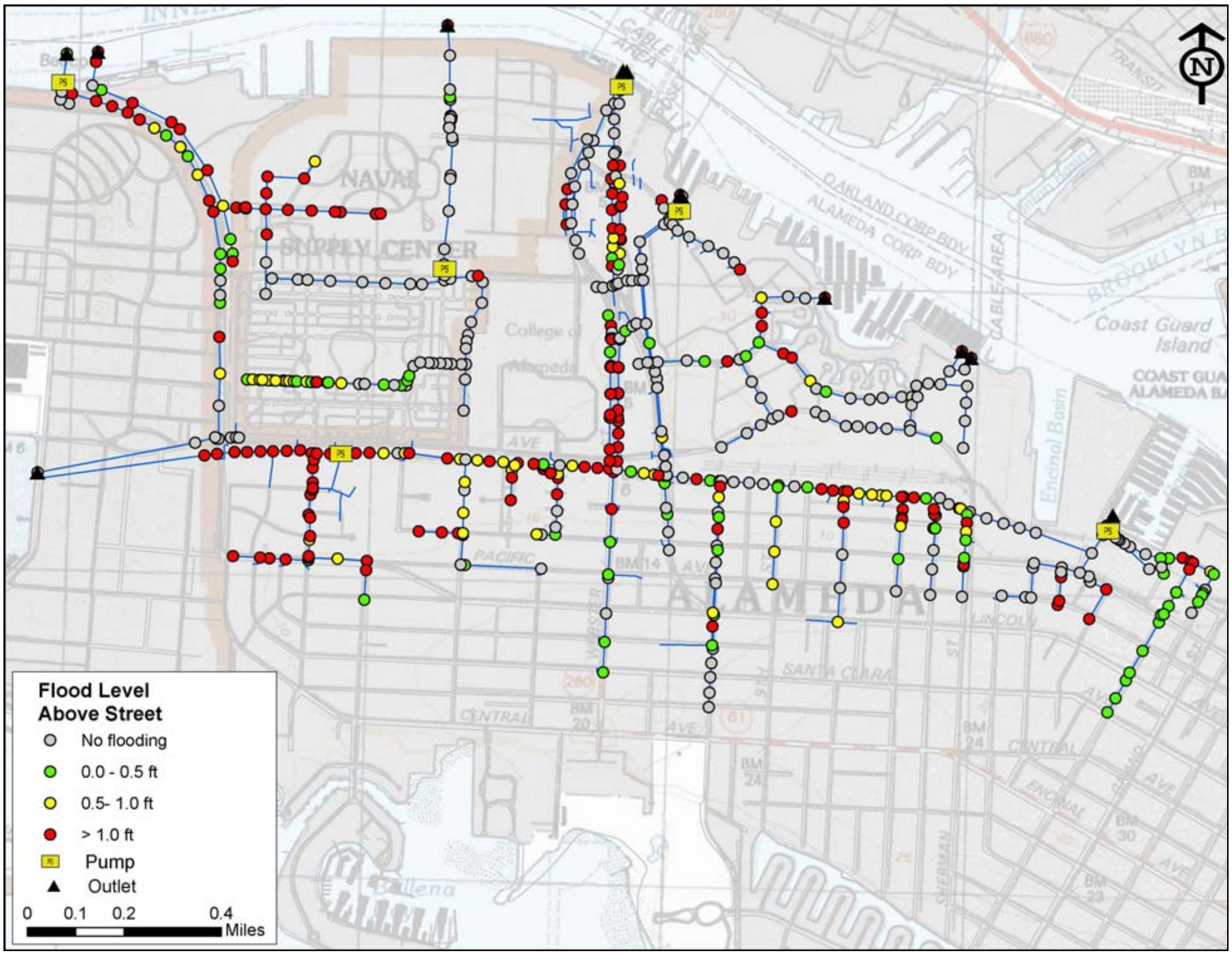


Figure 5-5: Alameda Northside Area Existing 10-Year Flooding Depths

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.

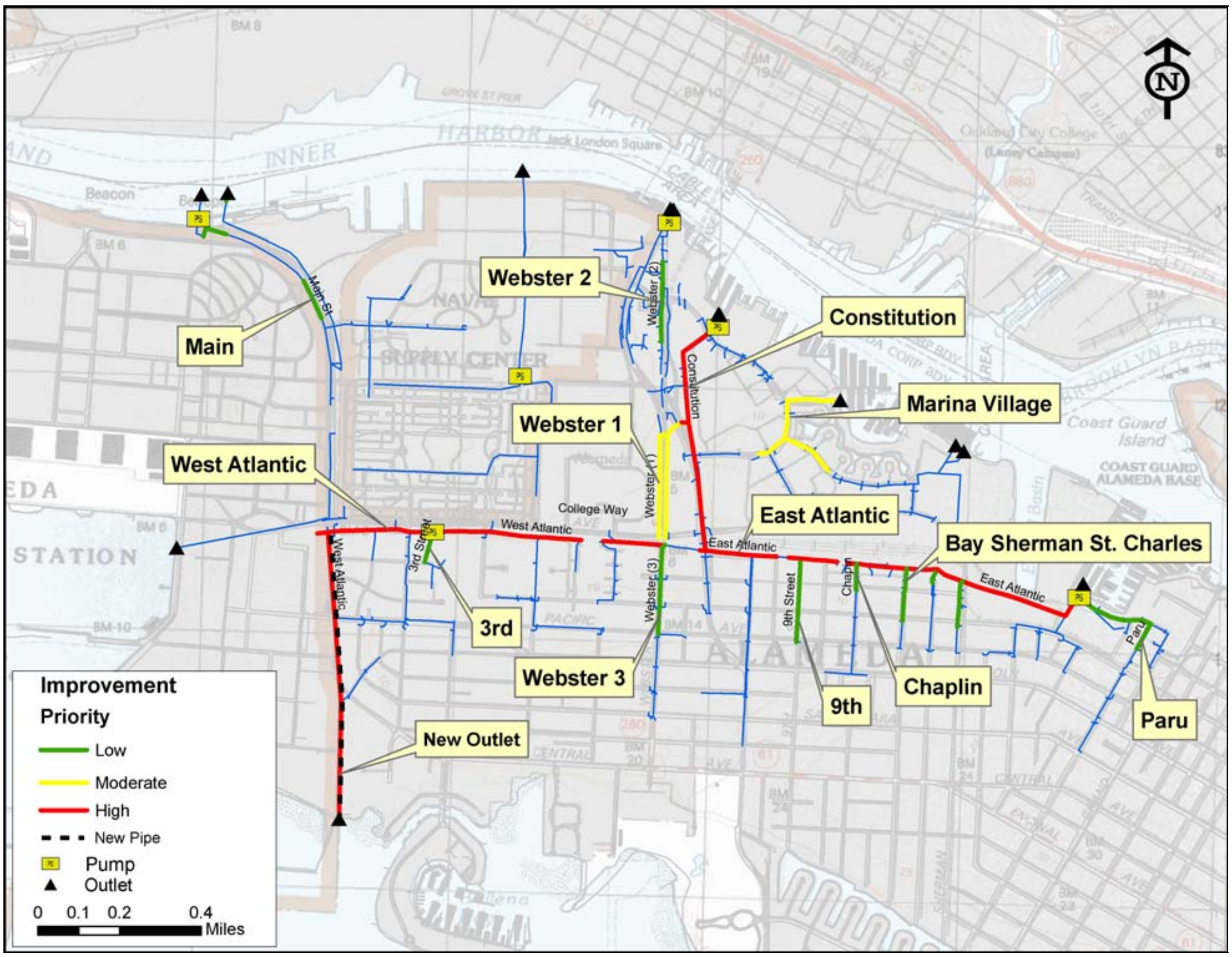


Figure 5-6: Alameda North West Area Prioritized 10-Year Improvements

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.

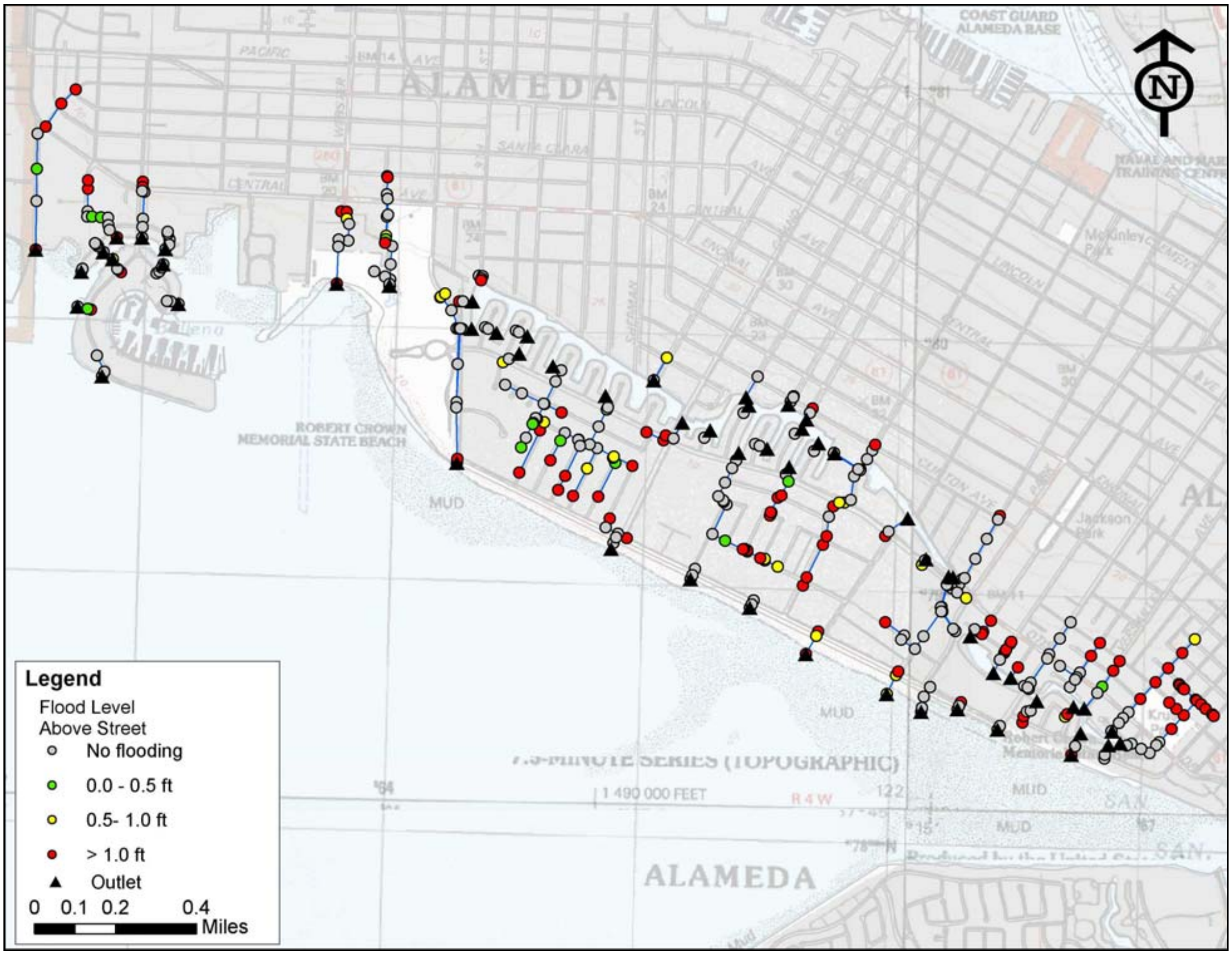


Figure 5-7: Alameda South Area Existing 10-Year Flooding Depths

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.

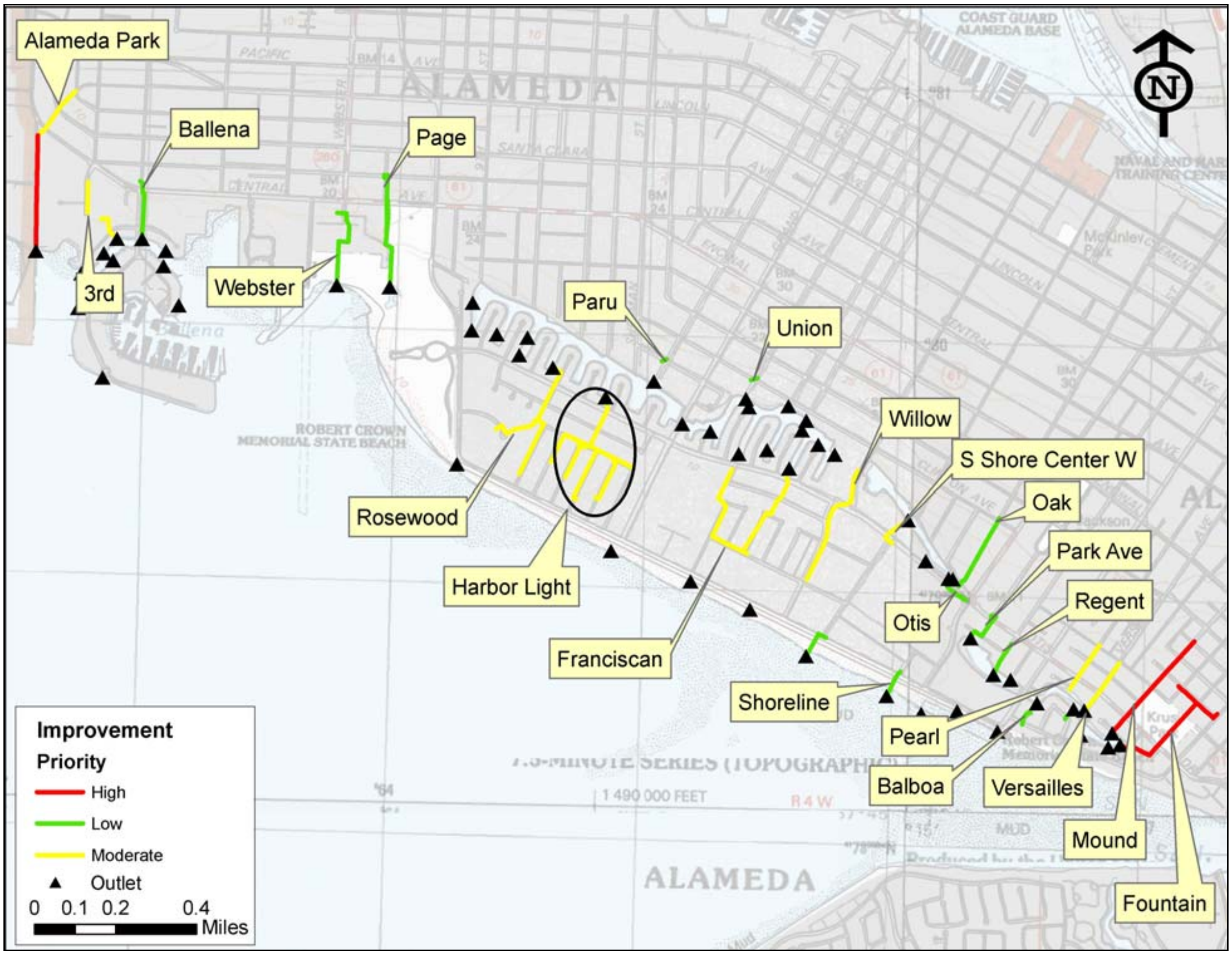


Figure 5-8: Alameda South Area Prioritized 10-Year Improvements

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.

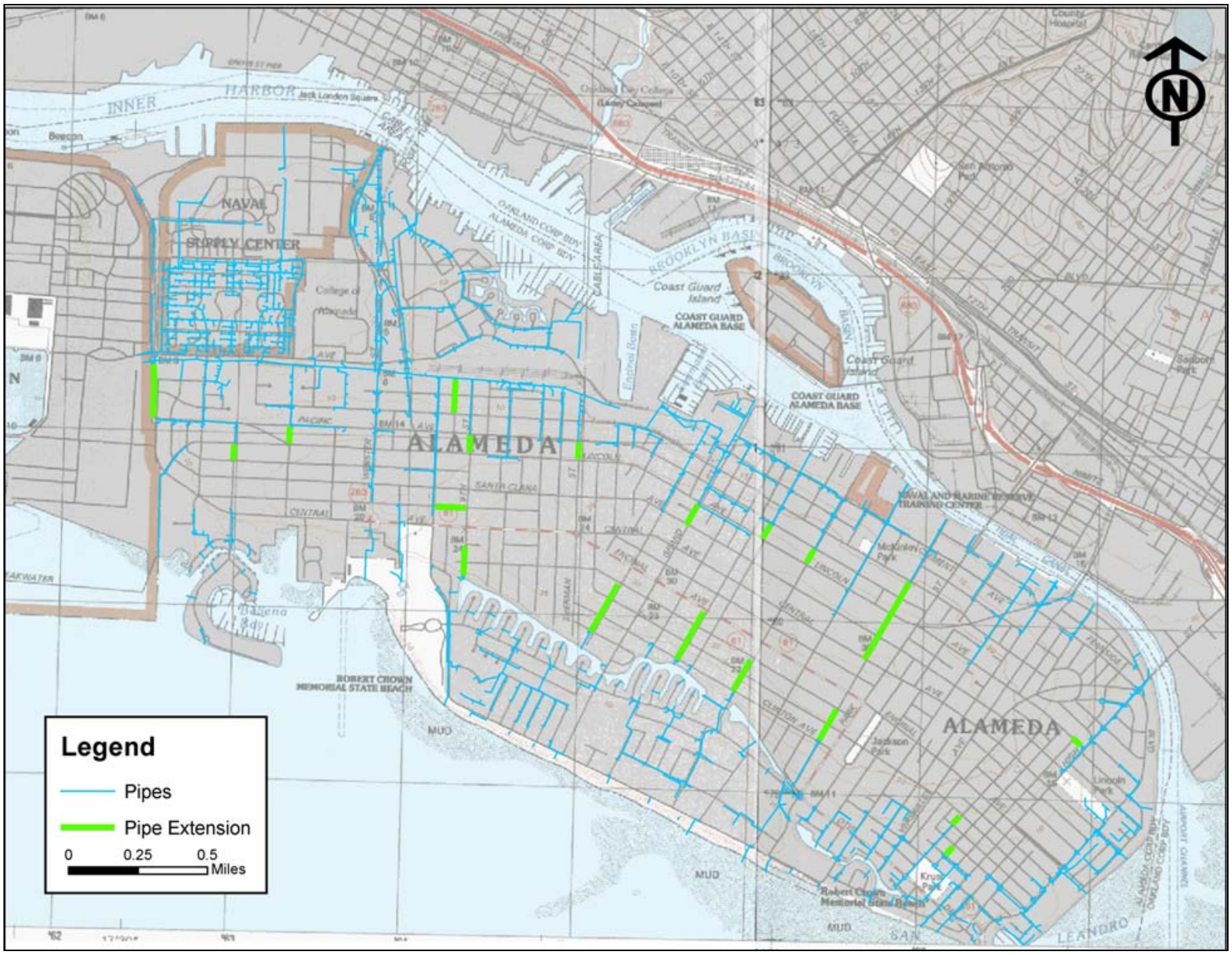


Figure 5-9: Alameda Main Island Recommended Pipe Extensions

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.

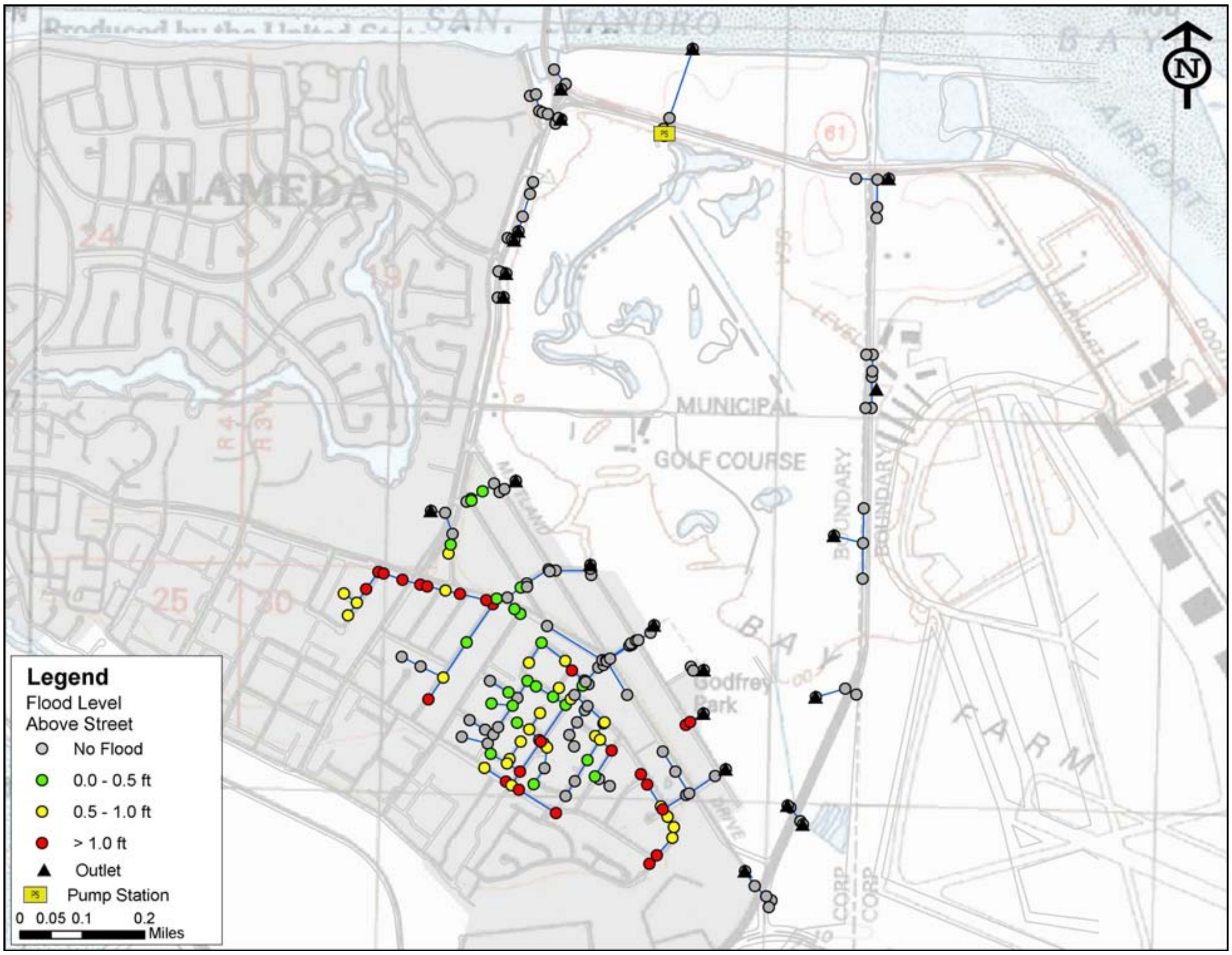


Figure 5-10: Bay Farm East Area Existing 10-Year Flooding Depths

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.

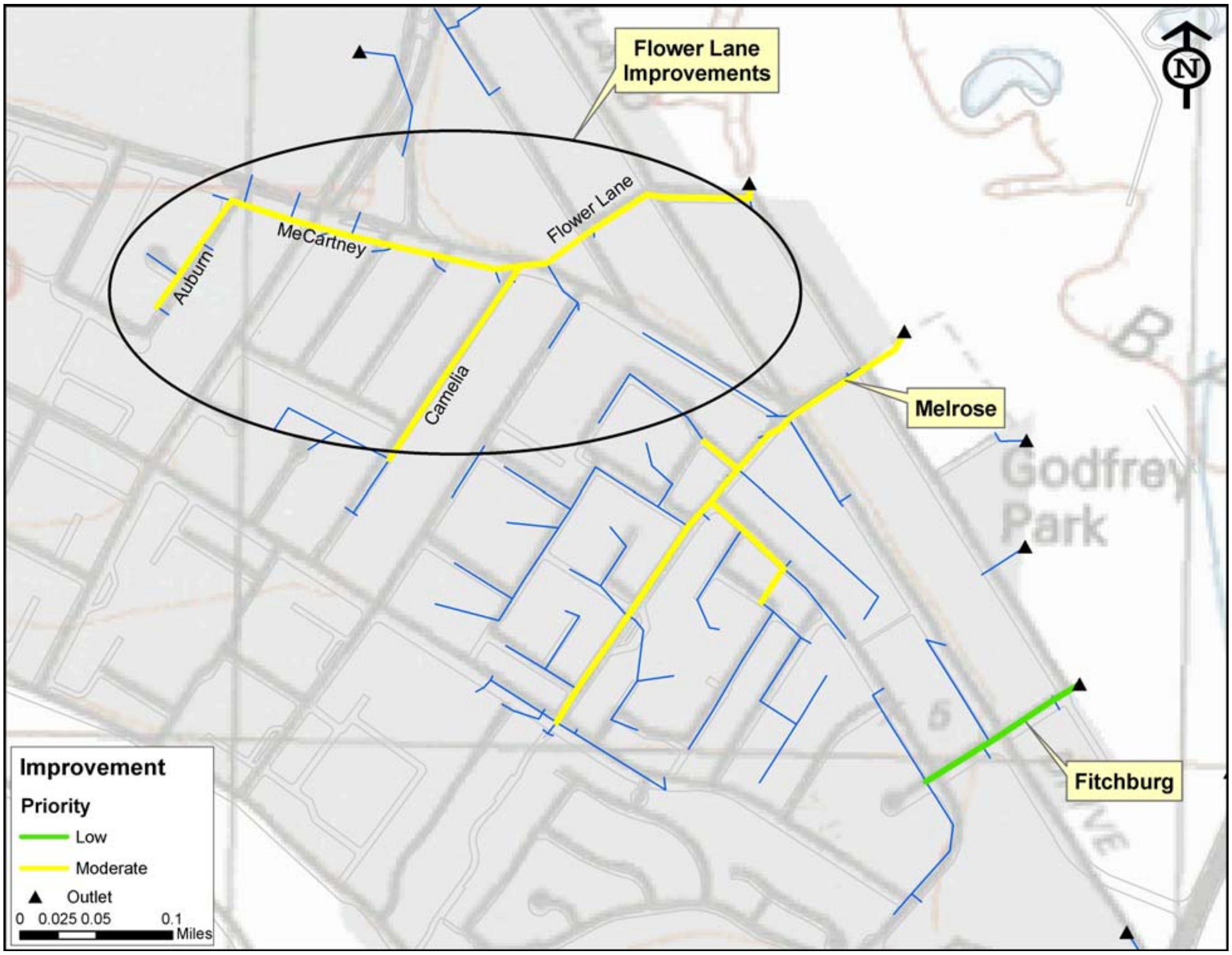


Figure 5-11: Bay Farm East Area Prioritized 10-Year Improvements

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.

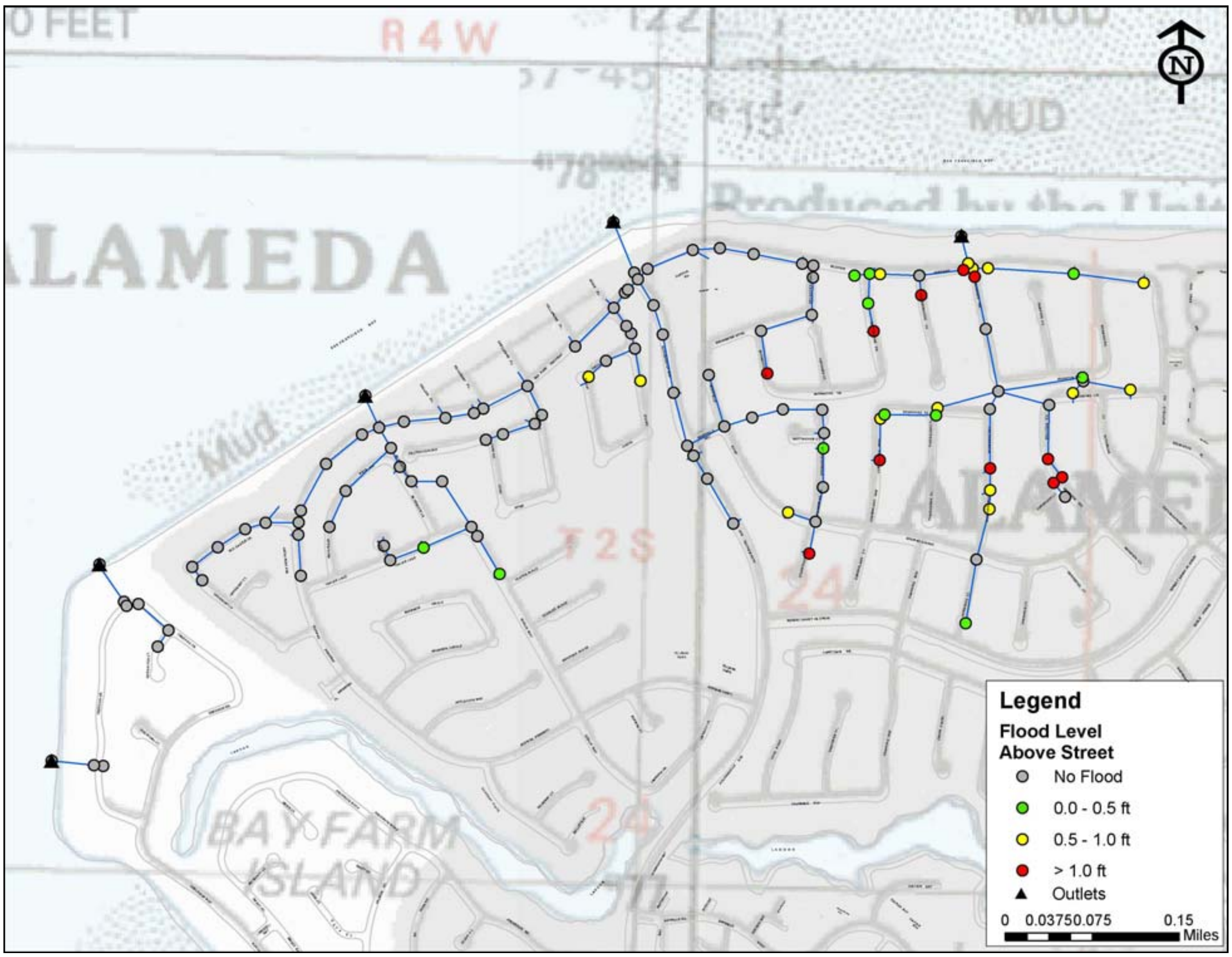


Figure 5-12: Bay Farm North Area Existing 10-Year Flooding Depths

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.

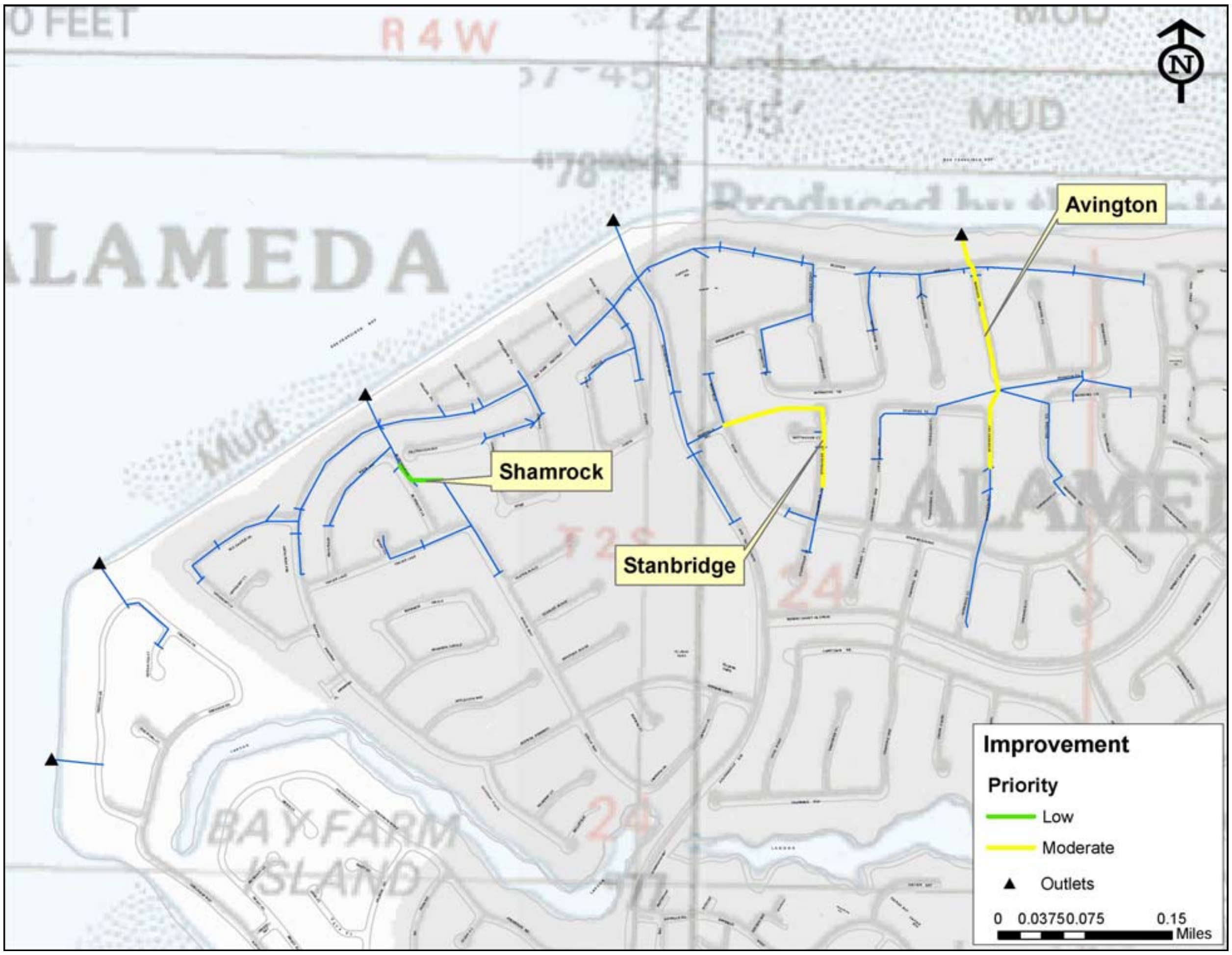


Figure 5-13: Bay Farm North Area Prioritized 10-Year Improvements

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.

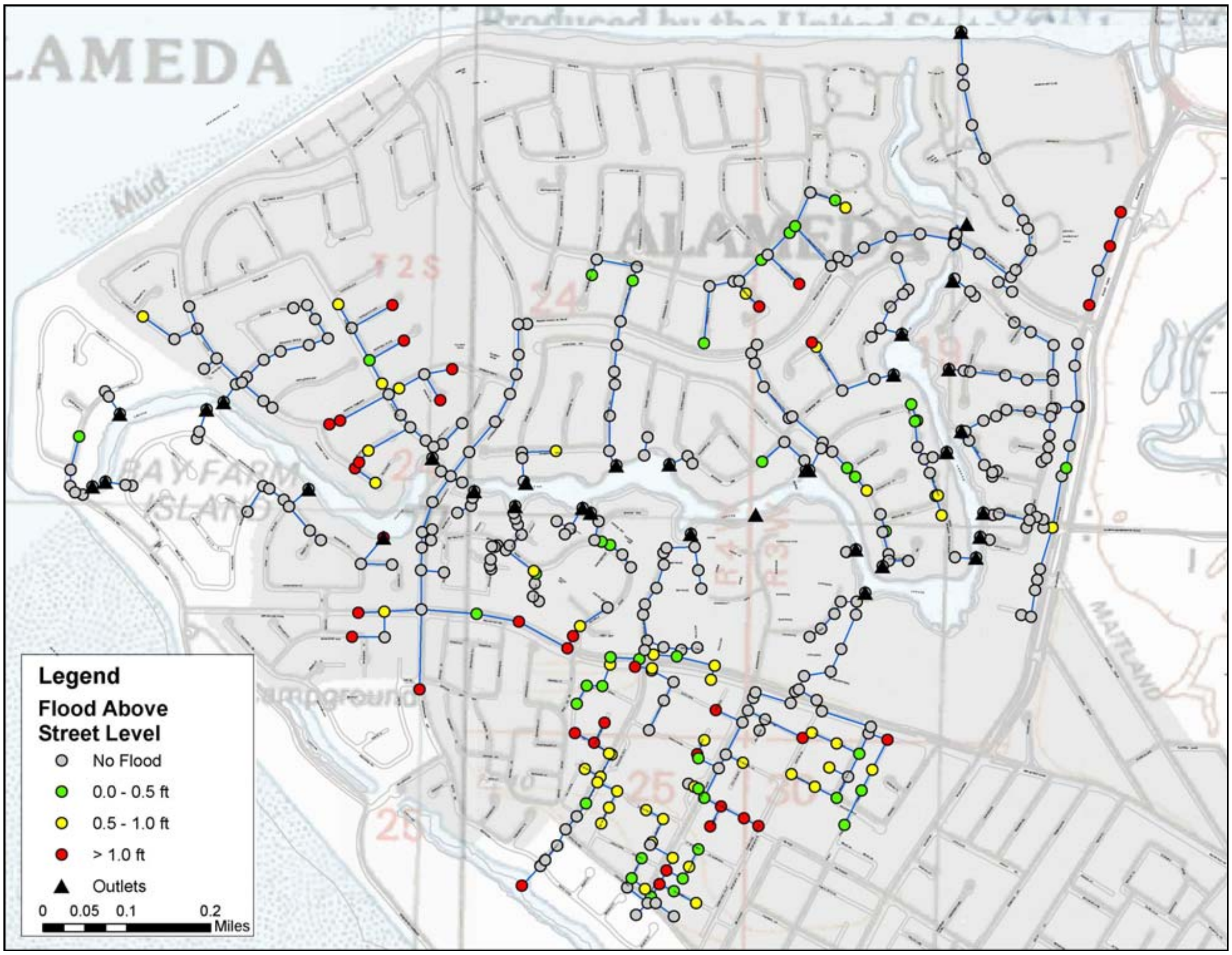


Figure 5-14: Bay Farm Central Area Existing 10-Year Flooding Depths

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.

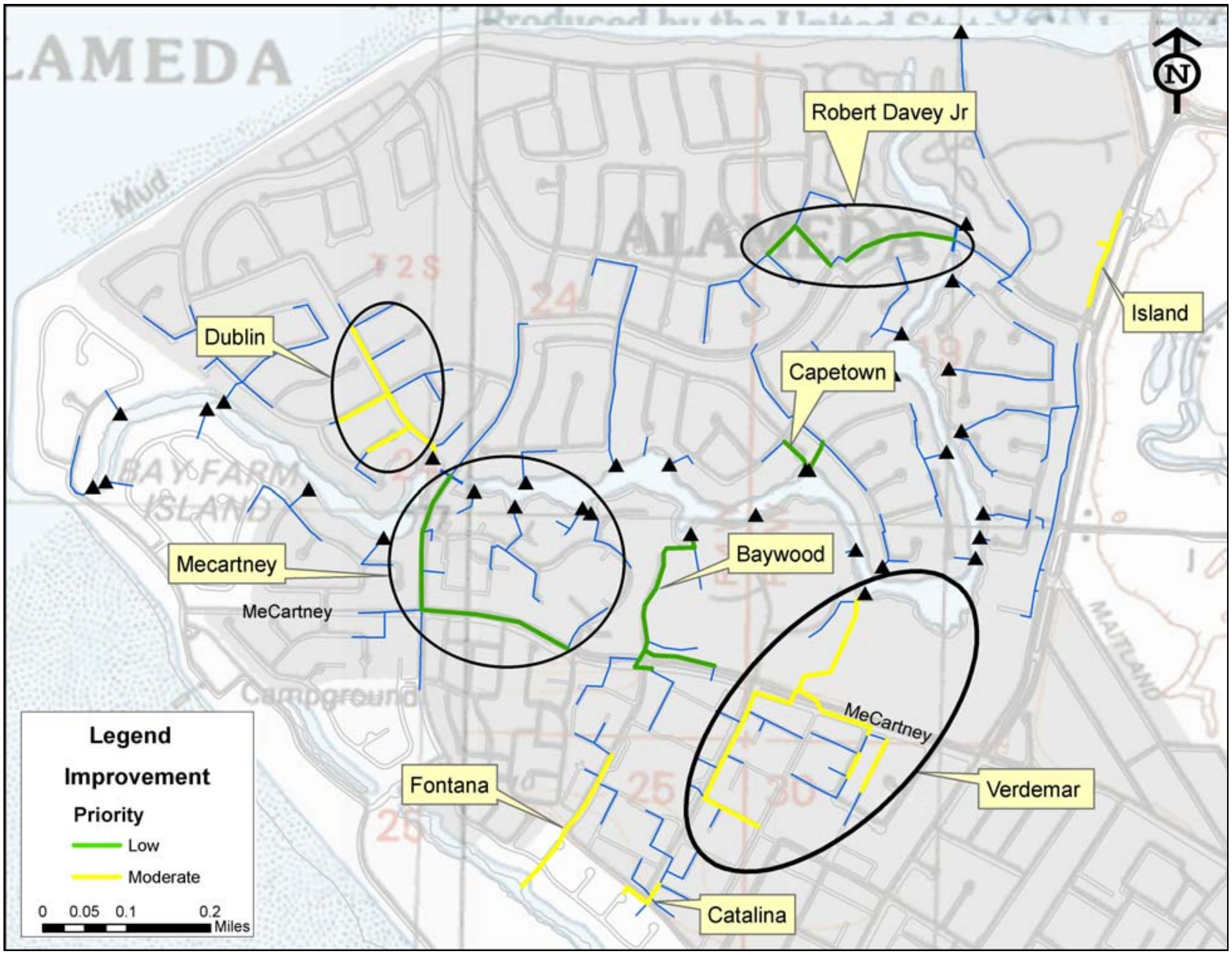


Figure 5-15: Bay Farm Central Area Prioritized 10-Year Improvements

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.

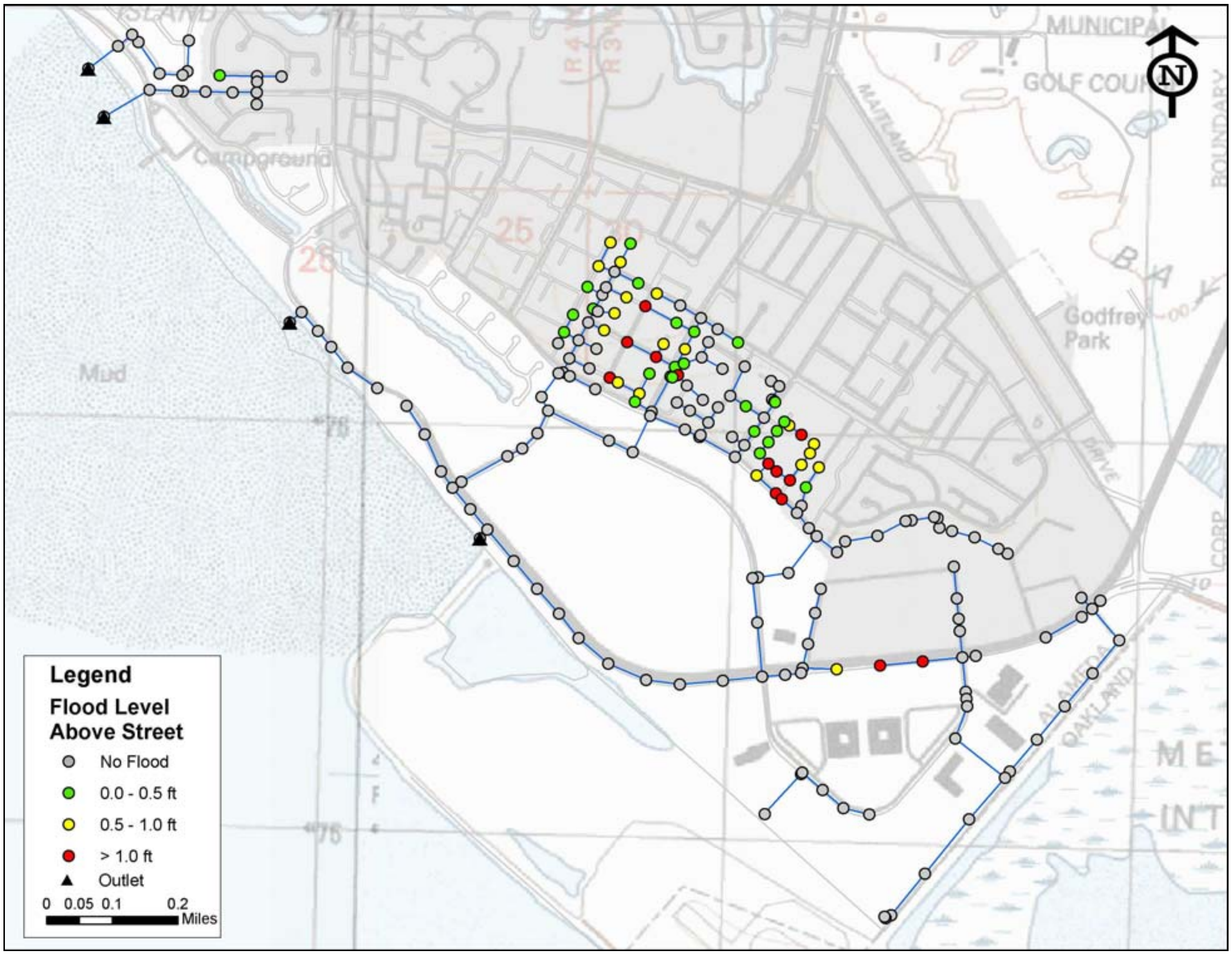


Figure 5-16: Bay Farm South Area Existing 10-Year Flooding Depths

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.

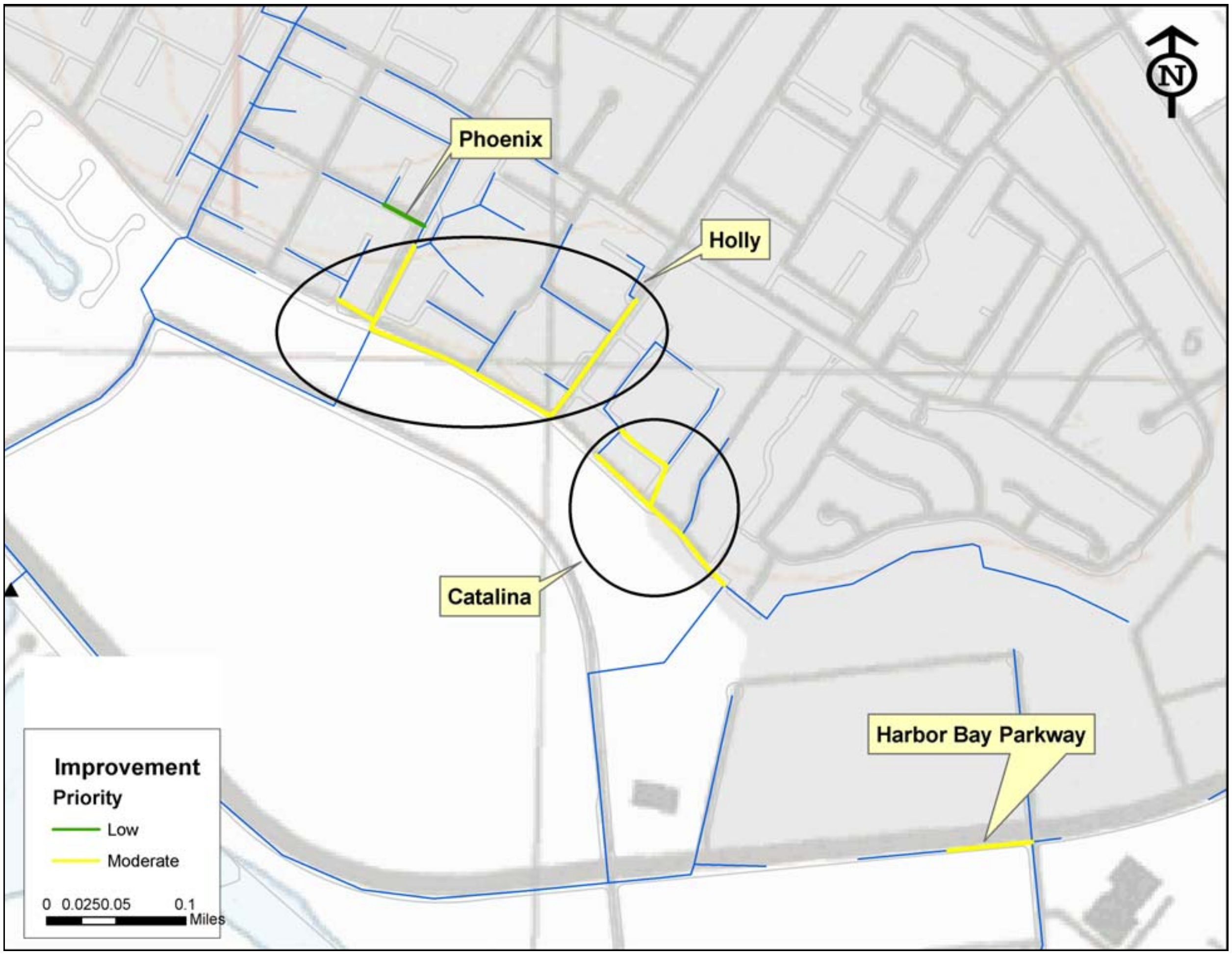


Figure 5-17: Bay Farm South Area Prioritized 10-Year Improvements

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.

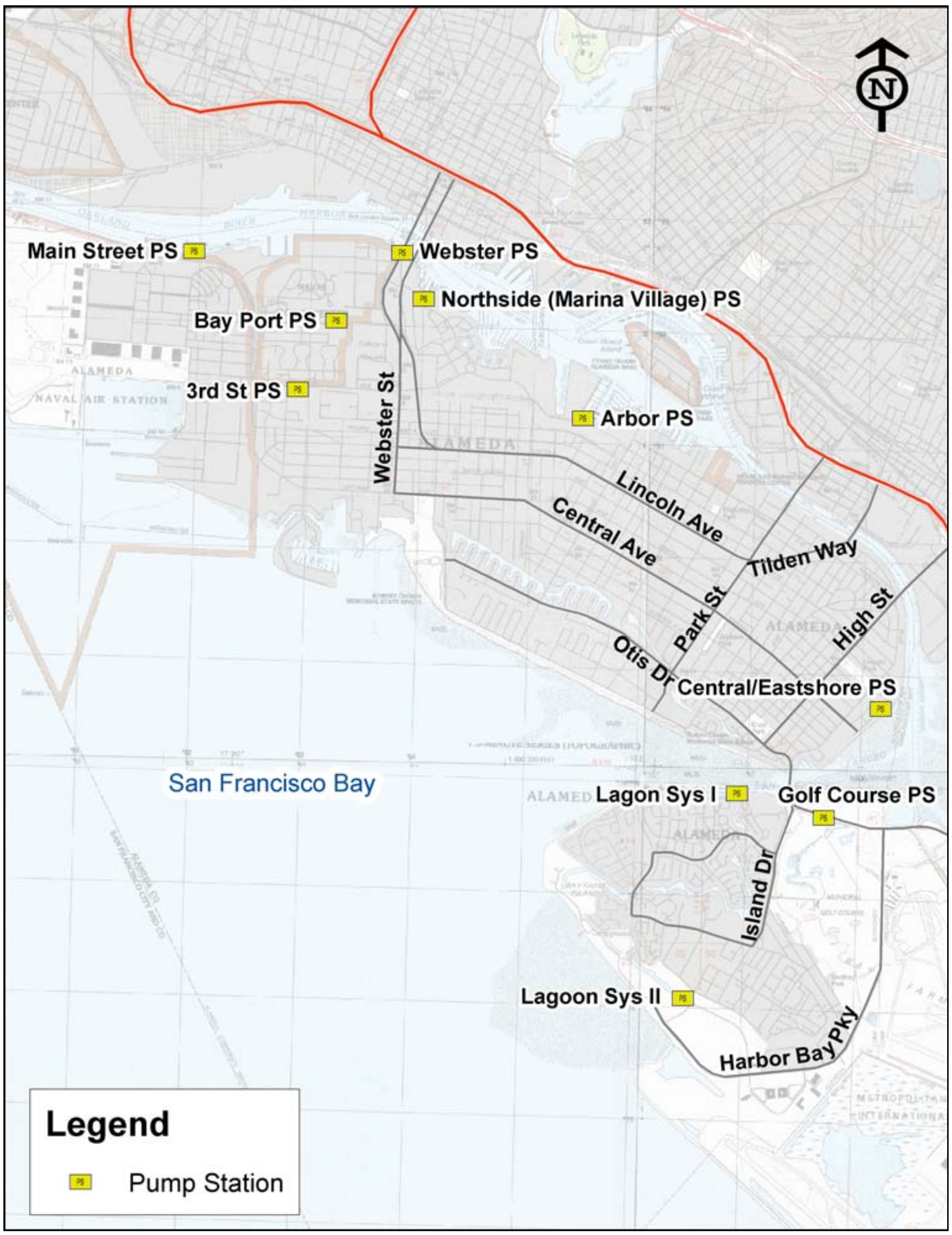


Figure 6-1: Pump Station Locations

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.

<table>
<tr><td colspan="2">Third Street Pump Station</td></tr>
<tr><td>Appezzato Parkway (aka Atlantic) at 3rd Street</td><td rowspan="7">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.</td></tr>
<tr><td>Constructed 1993</td></tr>
<tr><td></td></tr>
<tr><td>Tributary Area: 116.4 acres, 0.18 sq. miles</td></tr>
<tr><td>Outfall: Storm Drain system Leading to Arbor and Marina Village Pump Stations</td></tr>
<tr><td>Existing Equipment:
(1) Prime Pump P10 axial flow
1,650 gpm @ 8’ TDH (1,150 rpm)

5 hp submersible electric motor

Standby Power:
None</td></tr>
<tr><td><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</td></tr>
</table>

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.

<table>
<tr><td></td><td></td></tr>
<tr><td colspan="2">Central / Eastshore Pump Station</td></tr>
<tr><td>Eastshore Drive at Central Avenue</td><td rowspan="7">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.</td></tr>
<tr><td>Constructed 1967</td></tr>
<tr><td></td></tr>
<tr><td>Tributary Area: 137.5 acres, 0.21 sq. miles</td></tr>
<tr><td>Outfall: San Leandro Canal</td></tr>
<tr><td>Existing Equipment:
(2) Prime Pump M12A axial flow
4,300 gpm @ 15’ TDH (1,180 rpm)

25 hp submersible electric motors

Standby Power:
None</td></tr>
<tr><td><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</td></tr>
</table>

<table>
<tr><td colspan="2">Bayport Pump Station</td></tr>
<tr><td>5th Street at Tinker Avenue</td><td rowspan="8">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</td></tr>
<tr><td><u>Master Plan Recommendations</u>
High Priority:
None

Medium Priority:
None

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

<table>
<tr><td colspan="2">Golf Course Pump Station</td></tr>
<tr><td>Golf Course Slough at Doolittle Drive</td><td rowspan="8">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.</td></tr>
<tr><td>Constructed 1985</td></tr>
<tr><td></td></tr>
<tr><td>Tributary Area: 478.1 acres, 0.75 sq. miles</td></tr>
<tr><td>Outfall: San Leandro Canal / Estuary</td></tr>
<tr><td>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</td></tr>
<tr><td><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</td></tr>
</table>

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.

<u>System I:</u> 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	Outfall s	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

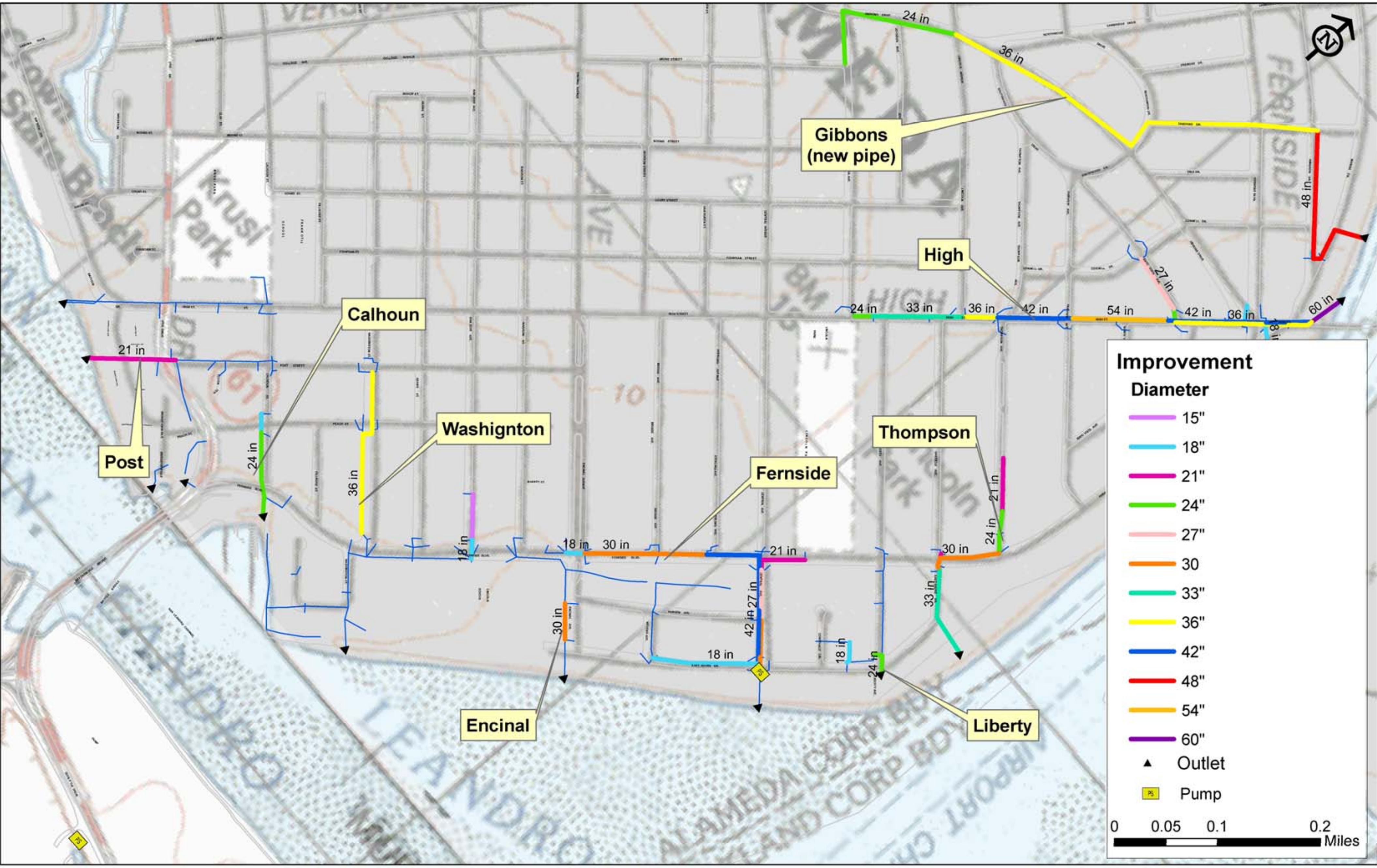


Figure 7-1: Alameda Eastside Area 10-Year Improvement Recommended Diameters

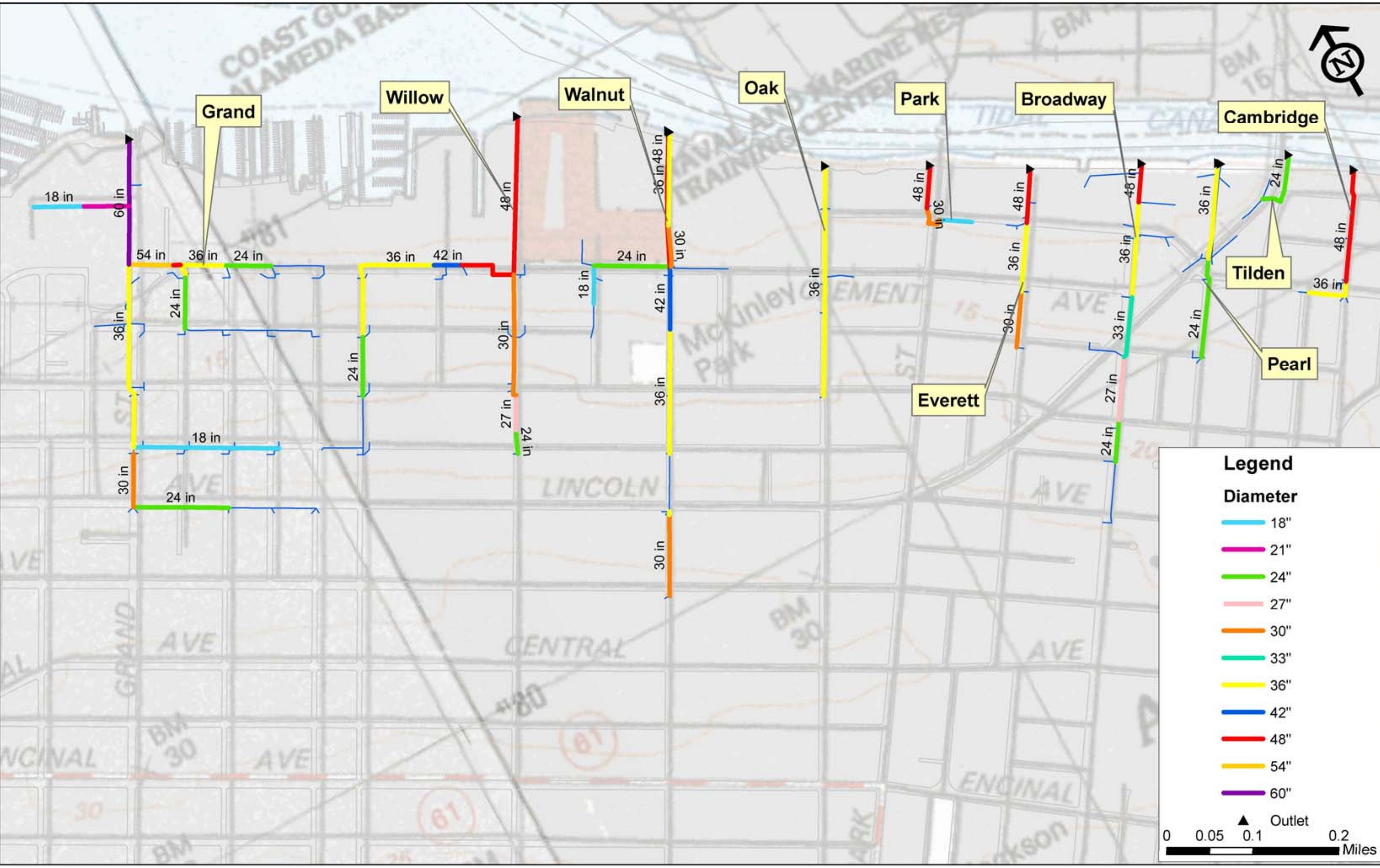


Figure 7-2: Alameda North Central Area 10-Year Improvement Recommended Diameters

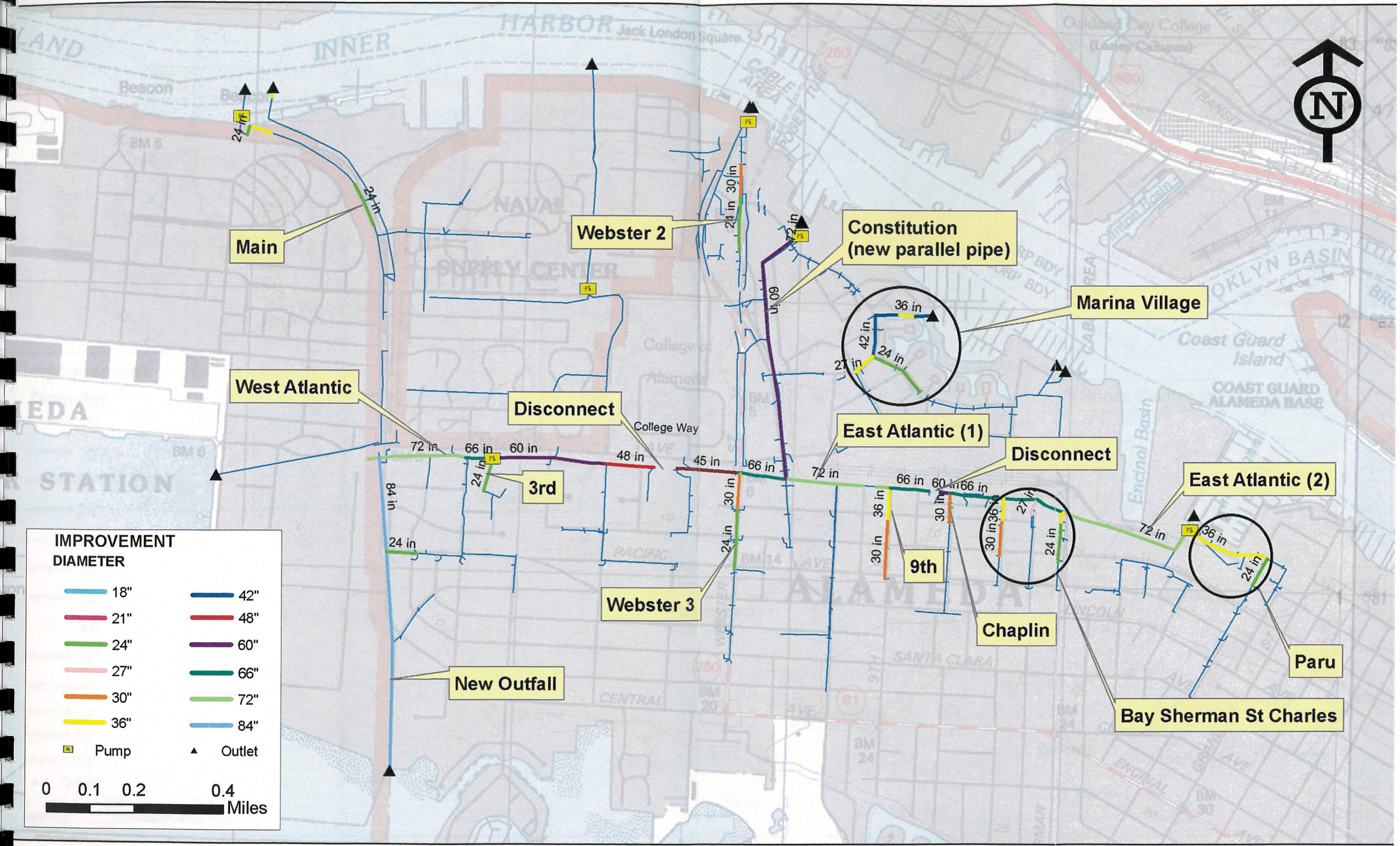


Figure 7-3: Alameda Northside Area 10 Year Improvement Recommended Diameters

Fig 7-3

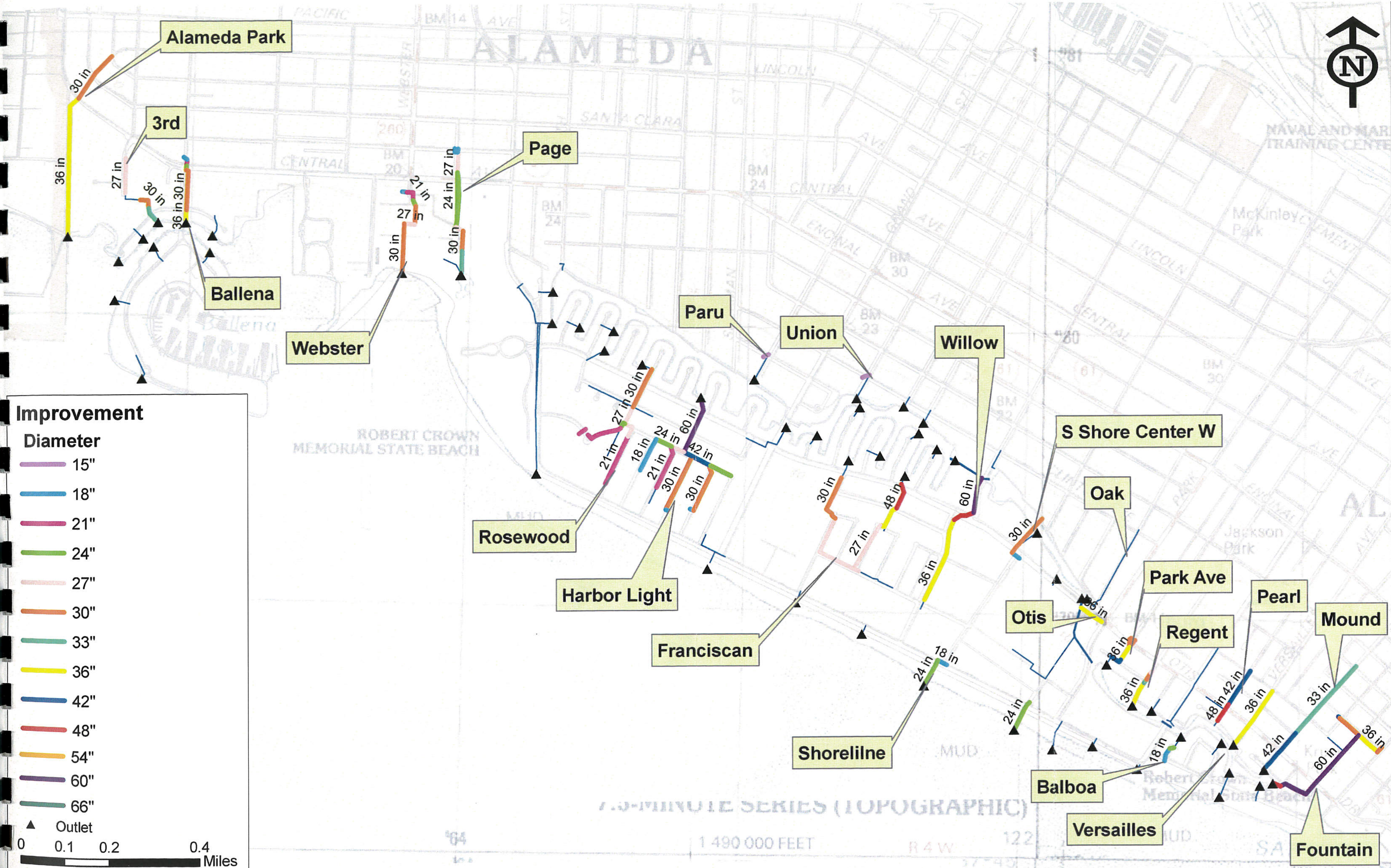


Figure 7-4: Alameda South Area 10-Year Improvement Recommended Diameters

Fig 7-4

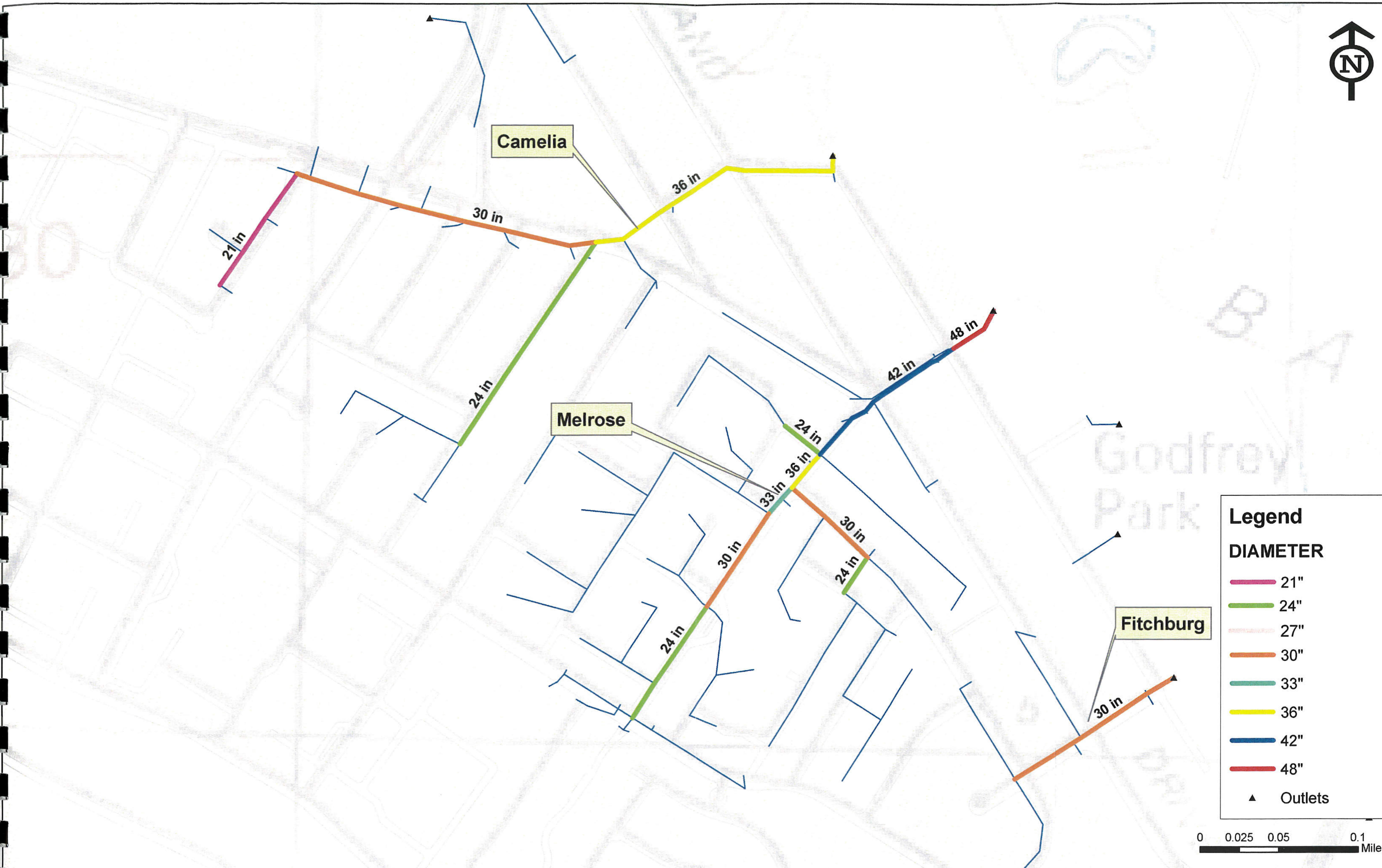


Figure 7-5: Bay Farm East Area 10-Year Improvement Recommended Diameters

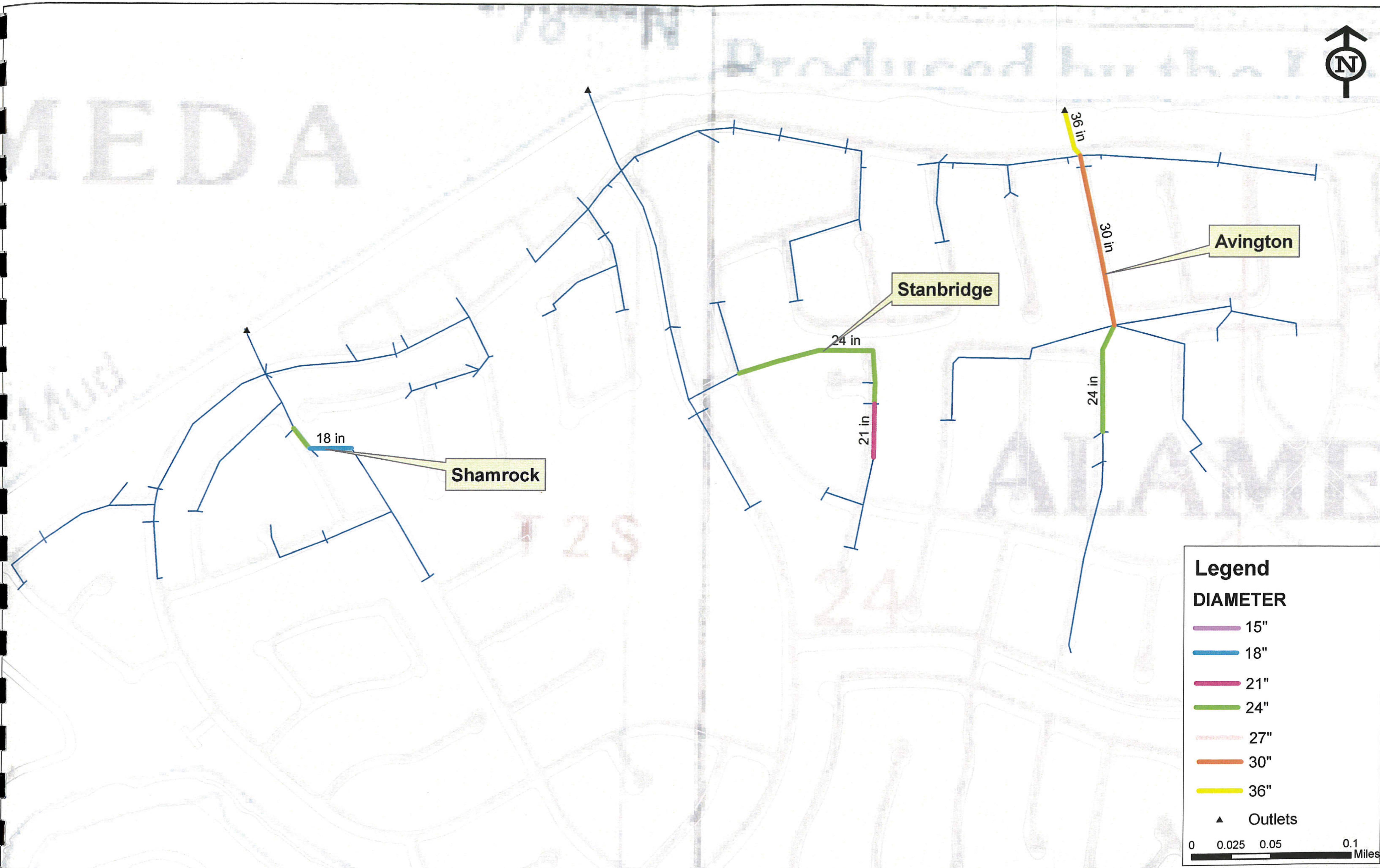


Figure 7-6: Bay Farm North Area 10-Year Imporovement Recommended Diameter

Fig 7-6

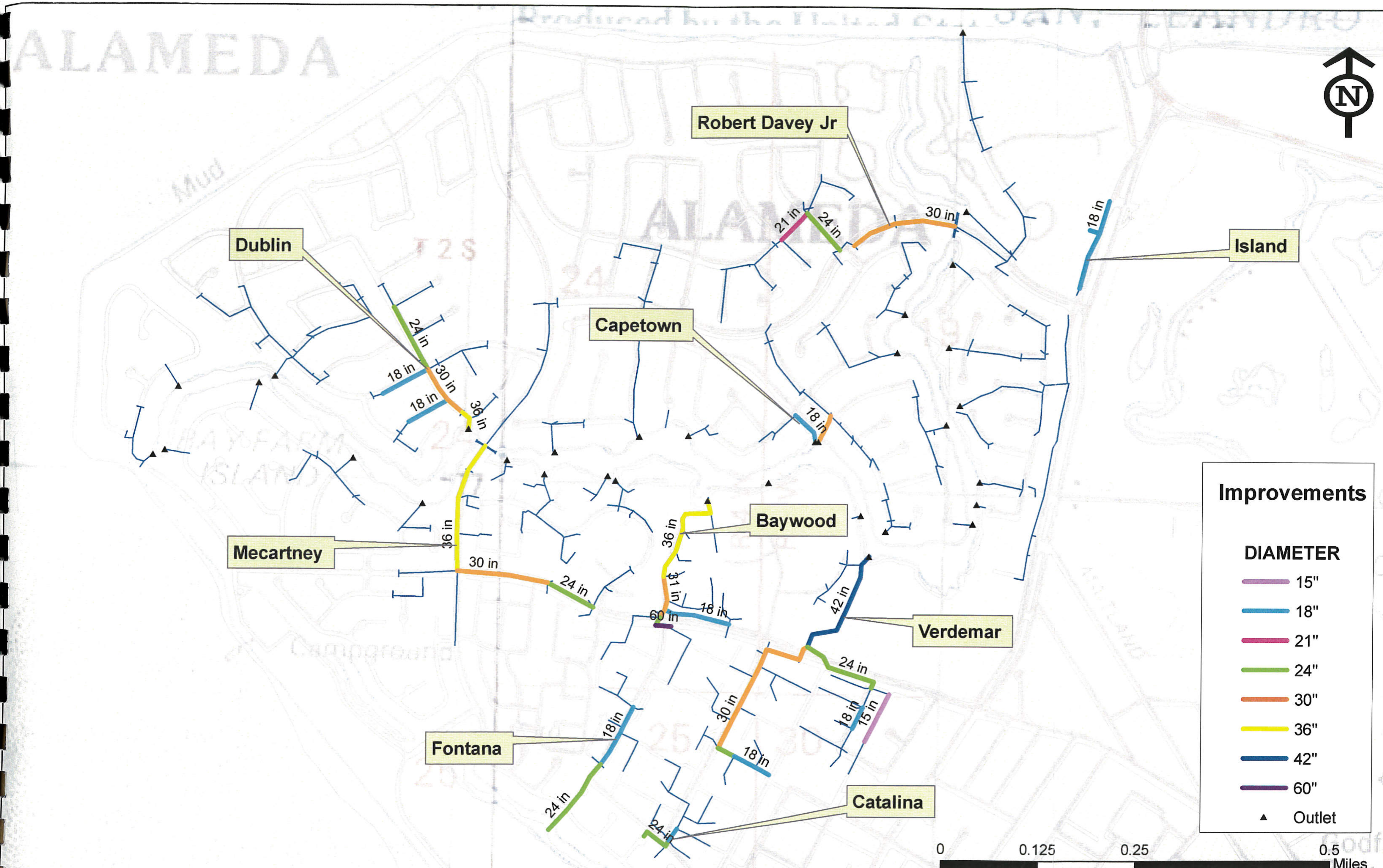


Figure 7-7: Bay Farm Central Area 10-Year Improvement Recommed Diameters

Fig 7-7

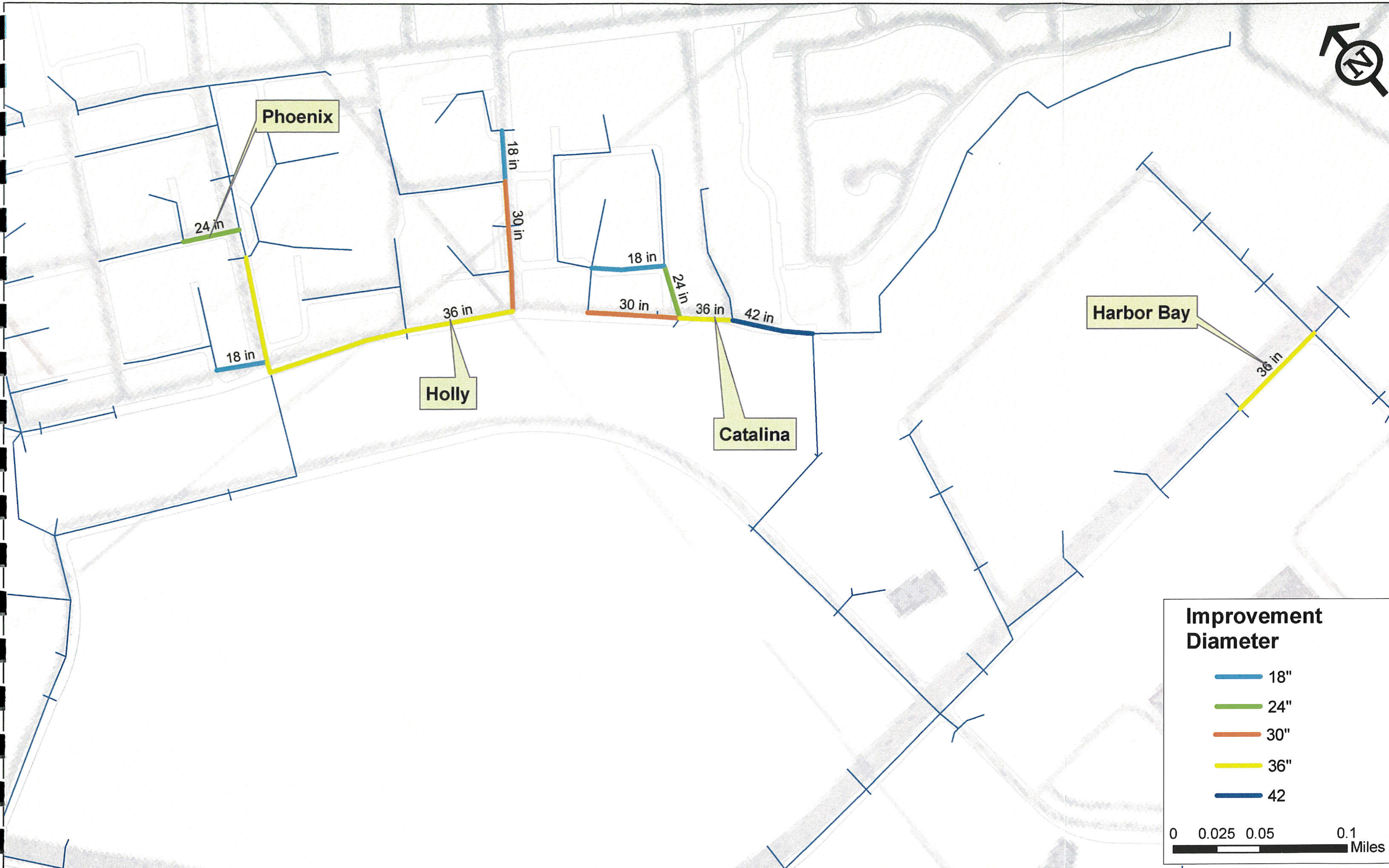


Figure 7-8: Bay Farm South Area 10-Year Improvement Recommended Diameters

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.