City of Alameda
Sanitary Sewer System
Hydraulic Model Analysis

Final Report

Prepared by:

RMC
Water and Environment

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Abbreviations and Definitions

BWF	Base wastewater flow: sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system.
CIP	Capital Improvement Program
City	City of Alameda
d/D	Depth to diameter ratio: the depth of flow in a pipe compared to the pipe diameter.
Design Storm	Rainfall event that defines the peak wet weather flows for which required sewer system capacity is determined. For Alameda, the design storm is a specific historical rainfall event developed for the East Bay I/I Study in the 1980s and known as the "EBMUD Design Storm".
Diurnal Profile	Change in base wastewater flow over a typical 24-hour period.
DOF	California Department of Finance
DWF	Dry weather flow: the flow during non-rainfall periods, composed of base wastewater flow plus any dry season groundwater infiltration.
E2	E2 Consulting Engineers, Inc.
EBMUD	East Bay Municipal Utility District
ENR-CCI	Engineering News Record Construction Cost Index
FAR	Floor area ratio: ratio of building square footage to parcel area.
GIS	Geographic Information System: a computerized system in which geographical features (e.g., sewer facilities, parcels, land use) are linked to an attribute database to facilitate analysis and presentation of information.
gpd	Gallons per day
GWI	Groundwater infiltration: extraneous water that infiltrates into a sewer system from the ground through defective pipes and manholes. Groundwater infiltration is considered to be a relatively constant daily flow that varies seasonally and depends on location of sewers with respect to the groundwater table.
I/I	Infiltration/inflow: extraneous groundwater and/or storm water that enter a sanitary sewer system.
MGD	Million gallons per day

PDWF	Peak dry weather flow: the peak flow during a non-rainfall period.
PS	Pump Station
PWWF	Peak wet weather flow: the peak flow during a given storm event from dry weather flow plus infiltration and inflow.
RDI/I	Rainfall-dependent infiltration/inflow: the infiltration and inflow into a sewer system directly related to a rainfall event. RDI/I may cause rapid, short-term peak flows in the sewer system that recede after the rainfall has ended.
RMC	RMC Water and Environment
RTK	Parameters that define the RDI/I flow response to rainfall as a percentage of rainfall volume (R), time to peak flow (T), and coefficient of hydrograph recession (K)
Sewershed (or subcatchment)	An area tributary to a modeled manhole, used for estimating a flow load to the model.
SSES	Sewer System Evaluation Survey
SSMP	Sewer System Management Plan
Surcharge	The hydraulic condition in a sewer pipeline in which the elevation of the hydraulic gradeline (water level) is above the crown (top) of the pipe. Under such a condition, the water in the pipe rises into the manholes and could overflow onto the ground if the hydraulic gradeline exceeds the elevation of the manhole rims.
TDH	Total dynamic head: the total of pump station static lift plus friction headlosses in downstream force main.
WSMP	EBMUD Water Supply Management Program 2040
WWF	Wet weather flow: the flow during rainfall periods, composed on base wastewater flow, wet season groundwater infiltration, and rainfall-dependent I/I.

Executive Summary

This report summarizes the results and recommendations of the Sanitary Sewer System Hydraulic Model Analysis for the City of Alameda (City). The Hydraulic Analysis Report was prepared by RMC Water and Environment (RMC) in close coordination with City staff. The hydraulic model and the recommendations included herein will be used to guide improvements to the City's sanitary sewer system to accommodate current and future development and to ensure that the City continues to provide a high level of service to its customers.

This Executive Summary is presented in three parts:

- ***Background and Purpose of Hydraulic Analysis*** introduces the Alameda sewer system and presents the context of this analysis.
- ***How the Hydraulic Analysis Report was Prepared*** describes the scope and methodologies of the planning effort, including key planning and technical assumptions incorporated into the sewer system capacity analysis.
- ***Recommended Capacity Improvement Program*** presents the recommended Capital Improvement Program (CIP), including capacity improvement projects, priorities, and estimated costs. In addition, recommendations are presented for implementing the proposed capacity improvement program.

ES-1 Background and Purpose of Hydraulic Analysis

Alameda's sanitary sewer system includes 34 City-owned pump stations and about 140 miles of 6-inch through 27-inch diameter sewers that discharge into a network of large diameter interceptor pipelines and pump stations owned and operated by the East Bay Municipal Utility District (EBMUD). Wastewater collected by these interceptors flows to the Alameda siphons and ultimately to EBMUD's wastewater treatment plant in Oakland. The existing sewer system on Alameda Point (portion of system located northwest of Main Street on the former Alameda Naval Air Station site) is not included in this study. Although the City maintains the Alameda Point sewers under contract, the City does not own these sewers. Flows from Alameda Point are conveyed via an EBMUD pump station and force main to the Alameda siphon inlet structure, and therefore do not impact any portion of the City's sewer system.

The capacity of Alameda's sewer system was last evaluated in the 1980s as part of the East Bay Infiltration/Inflow Study Sewer System Evaluation Survey (SSES). Since that time, flows in the system have changed due to new development and redevelopment, as well as sewer system rehabilitation conducted by the City based on the results of the SSES. Additional growth is projected in the future, which will further increase wastewater flows.

This hydraulic analysis will help the City meet the requirements to complete a capacity evaluation and capacity assurance plan as part of preparing its Sewer System Management Plan (SSMP), as well as provide information to update projected sewer improvement project needs in the City's Capital Improvement Program (CIP). The SSMP addresses the overall management, operation, and maintenance of the sanitary sewer system and is required for all sewer system agencies by the San Francisco Bay Regional Water Quality Control Board, as well as under the Statewide General Waste Discharge Requirements adopted in 2006 by the State Water Resources Control Board.

ES-2 How the Hydraulic Analysis Report was Prepared

A model of the City's sewer system was developed using InfoWorks CS™, a GIS-based hydraulic modeling software package. In addition to the City of Alameda sewer system, EBMUD's interceptor system facilities in Alameda were also included in the model; however, EBMUD's infrastructure was not evaluated as part of this hydraulic analysis. The modeled sewer system is shown in **Figure ES-1**.The project team used a systematic process that incorporated GIS sewer and parcel data, land use planning information, flow monitoring data, and design criteria for estimating wastewater flows in a computer hydraulic model of the sewer system. The model was used to assess how the system would perform under existing and future dry and wet weather flow scenarios, and to identify gravity pipes, pressure force mains, and pumps stations that may not have sufficient capacity to convey the predicted flows.

Capacity Assessment Considers Existing and Future Planning Scenarios

Two planning scenarios were evaluated for this study. The existing scenario examined the current capacity of the sewer system based on existing development and flow monitoring data collected in the winters of 2005/2006 and 2007/2008. The future scenario was based on potential future developments and redevelopments as identified by City Planning staff and documented in EBMUD's Water Supply Management Program (WSMP) 2040, and the assumption that currently vacant or underutilized parcels will be developed in the future. These future developments will result in an approximate 27 percent increase in base wastewater flows compared to existing flows.

Hydraulic Model Identifies Potential Capacity Deficiencies

For both of the planning scenarios examined, projected dry and wet weather flows were simulated in the hydraulic model. The model was calibrated to actual flow monitoring data to ensure that it represents an accurate depiction of system conditions during both dry and wet weather conditions. The model integrates various dry and wet weather flow parameters to determine system capacity under different flow and planning scenarios. Key flow components incorporated into the model include base (dry weather) wastewater flow (BWF); groundwater infiltration (GWI), which occurs when water seeps into pipes under the ground through cracks and pipe joints; and rainfall-dependent infiltration and inflow (RDI/I) during storm events. For this analysis, a 5-year recurrence frequency rainfall event, assumed to fall under saturated soil conditions (i.e., maximum GWI and RDI/I response), was selected as the "design storm". This design storm was originally established as the basis for wet weather planning for EBMUD and its tributary collection systems (including Alameda) in the 1980s.

Proposed Improvement Projects Address Potential Capacity Deficiencies

Model results for both existing and future flow conditions under dry and wet weather flow scenarios were examined to determine where improvement projects would be needed to alleviate capacity deficiencies. When assessing pipe deficiencies, it was assumed that all pumps would be retrofitted, if required, to provide sufficient capacity to handle predicted peak flow conditions. This ensures that the model is accurately predicting peak flows in downstream pipes because sewage is not being held-up at the pump stations, and is a better representation of future flow conditions, assuming the City will upgrade its pump stations where needed to provide required capacity. Pipe deficiencies were identified based on the level of pipe surcharge, specifically, if surcharge exceeded one foot and reached to less than six feet of the ground surface during wet weather flows. Additionally, any pipe that surcharged during peak dry weather flows was considered deficient (even if surcharge was less than 1-foot). While surcharging should generally be avoided, these surcharge criteria allow the City to focus capital spending on areas with the greatest risk of causing sewer overflows. Where capacity improvement projects were identified, new pipes were sized to avoid surcharge under design peak flow conditions.

Figure ES-1a: Modeled Sewer System

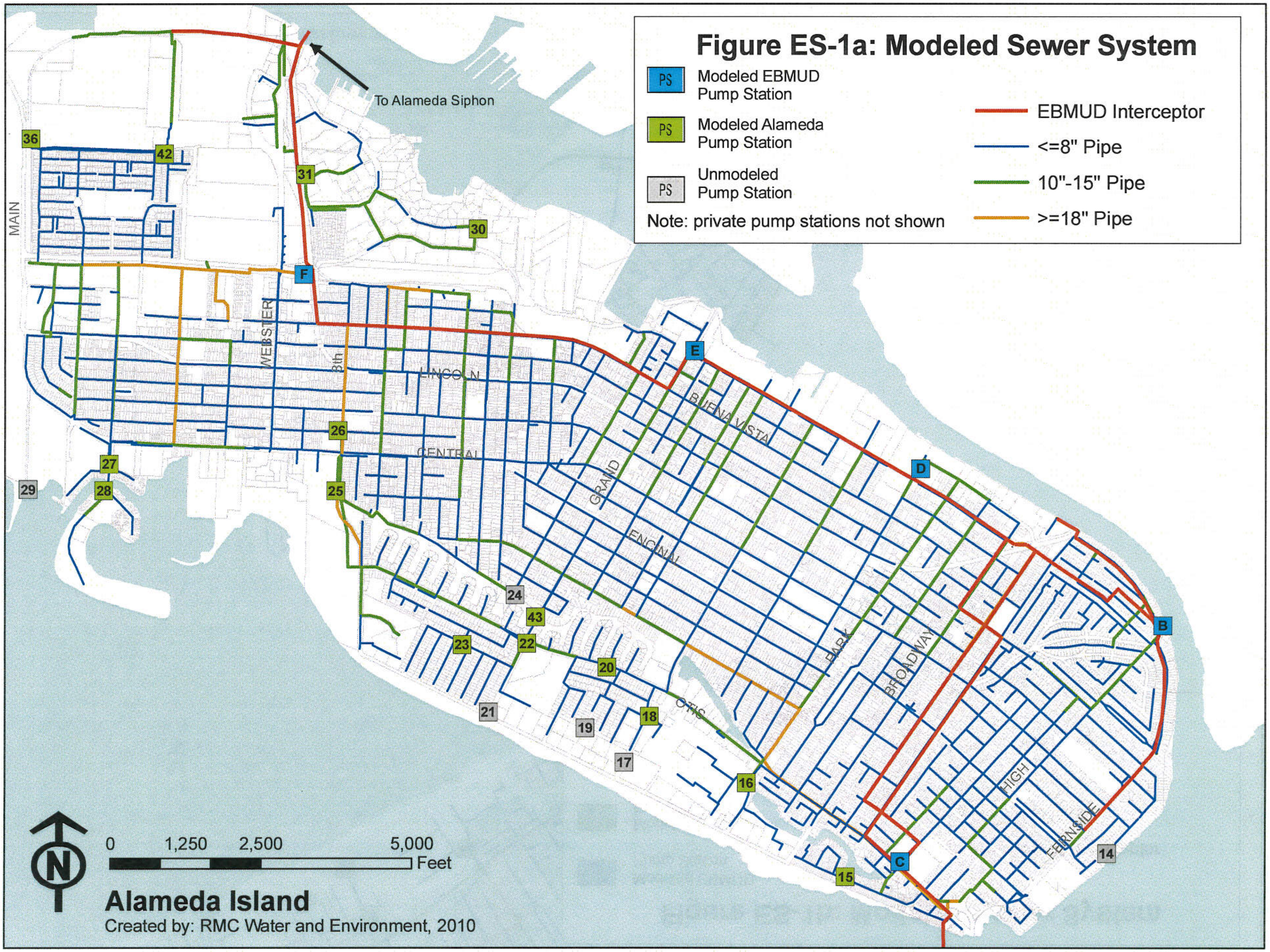
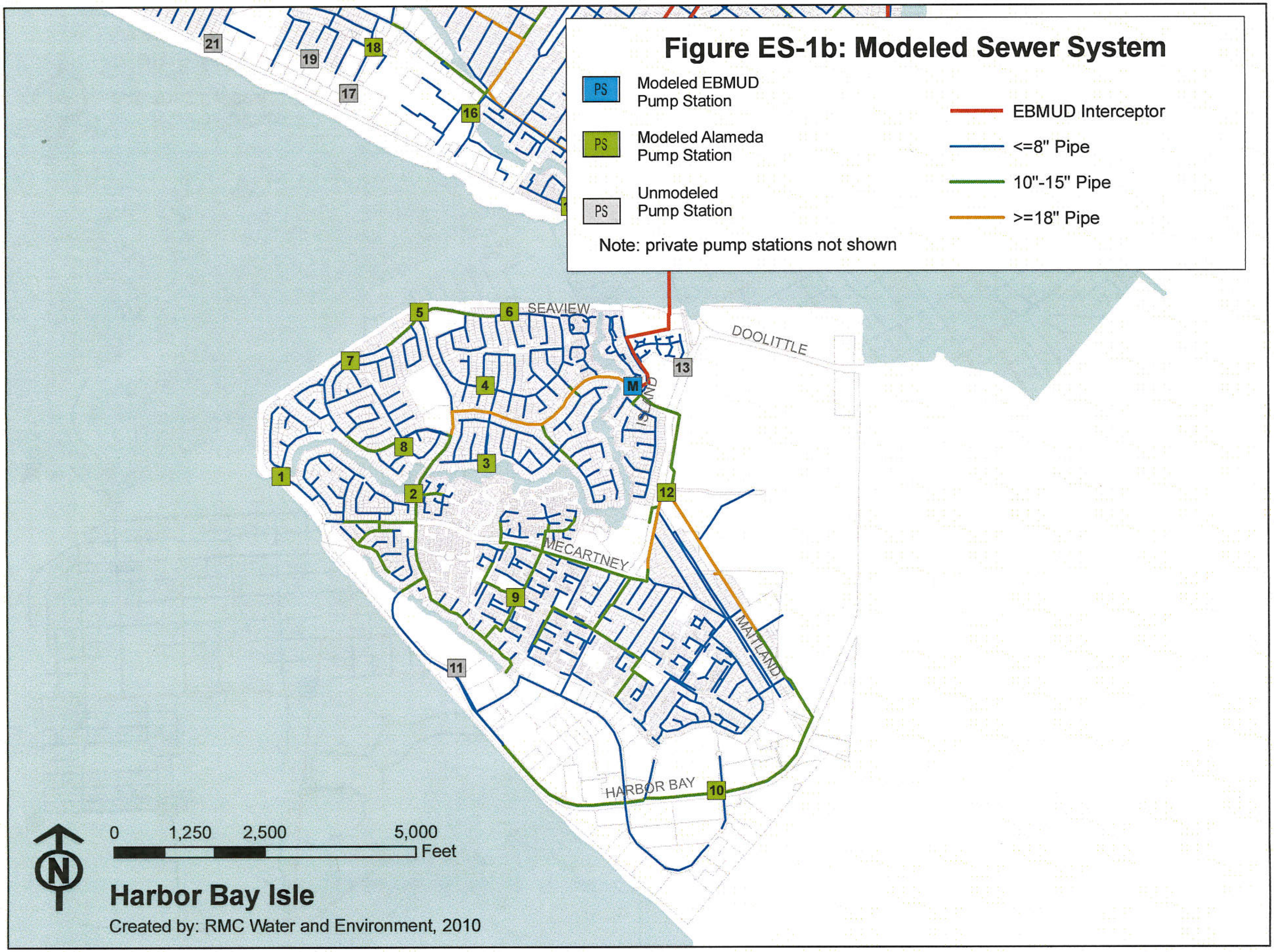


Figure ES-1b: Modeled Sewer System



Harbor Bay Isle

Created by: RMC Water and Environment, 2010

Pump stations were evaluated based on their performance under future dry weather flow conditions and under design storm wet weather flows. In addition to comparing pump capacities to peak inflows, other factors were considered when assessing pump capacity deficiencies such as, for example, the capacity of upstream pipe to "store" the flow or the ability of a high-level bypass to reroute flow around the pump station when water level reaches a certain height. Based on the results of this comprehensive analysis, three different deficiencies were identified: 1) pump stations with insufficient capacity, 2) pump stations with acceptable but less than optimal capacity, and 3) pump stations that need standby or redundant capacity.

ES-3 Recommended Capacity Improvement Program

The Capital Improvement Program (CIP) recommended in this study is designed to provide adequate sewer system capacity for the City's existing and anticipated future development. For deficient gravity pipes, improvement projects were developed, including the following information about each gravity pipe project:

- Description and location of the project
- Planning-level capital cost estimates
- Relative priority rating

Pump station capacity improvements have been identified and prioritized, but specific projects and cost estimates have not been developed as part of this study. Pump stations projects were handled separately from gravity pipe projects because the City has conducted a comprehensive pump station condition assessment under a separate project. Based on integrating the findings contained in this report with the results of the condition assessment work, the City has developed detailed pump station improvement projects (and costs) as part of the condition assessment work.

Gravity Pipe Improvement Projects

Three gravity sewer capacity improvement projects were identified based on the results of the hydraulic analysis. **Figure ES-2** shows the project locations. The total estimated capital cost of these projects is approximately $2.8 million, as shown in **Table ES-1**. These cost estimates include baseline construction costs for gravity sewers using trenchless methods, lower lateral replacement costs, and cost allowances for project mobilization and demobilization and traffic control. The total estimated capital costs also include a 30 percent allowance for contingencies for unknown conditions and an allowance of 25 percent of construction cost for engineering, administration, and legal costs. The estimated costs are considered planning or conceptual level estimates and are considered to have an estimated accuracy range of -30 to +50 percent, suitable for budget planning purposes.

Gravity pipe improvement projects were prioritized based on whether the deficiency was caused by existing or future conditions. Project C-1 is needed to resolve an existing capacity deficiency whereas Projects C-2, C-3, C-4, and C-5 are triggered by future developments or redevelopments. Therefore, Project C-1 is considered priority 1 and all other projects are considered priority 2. Note that the location of and need for priority 2 projects should be verified prior to implementation based on the final land uses and proposed sewerage plans for these future developments.

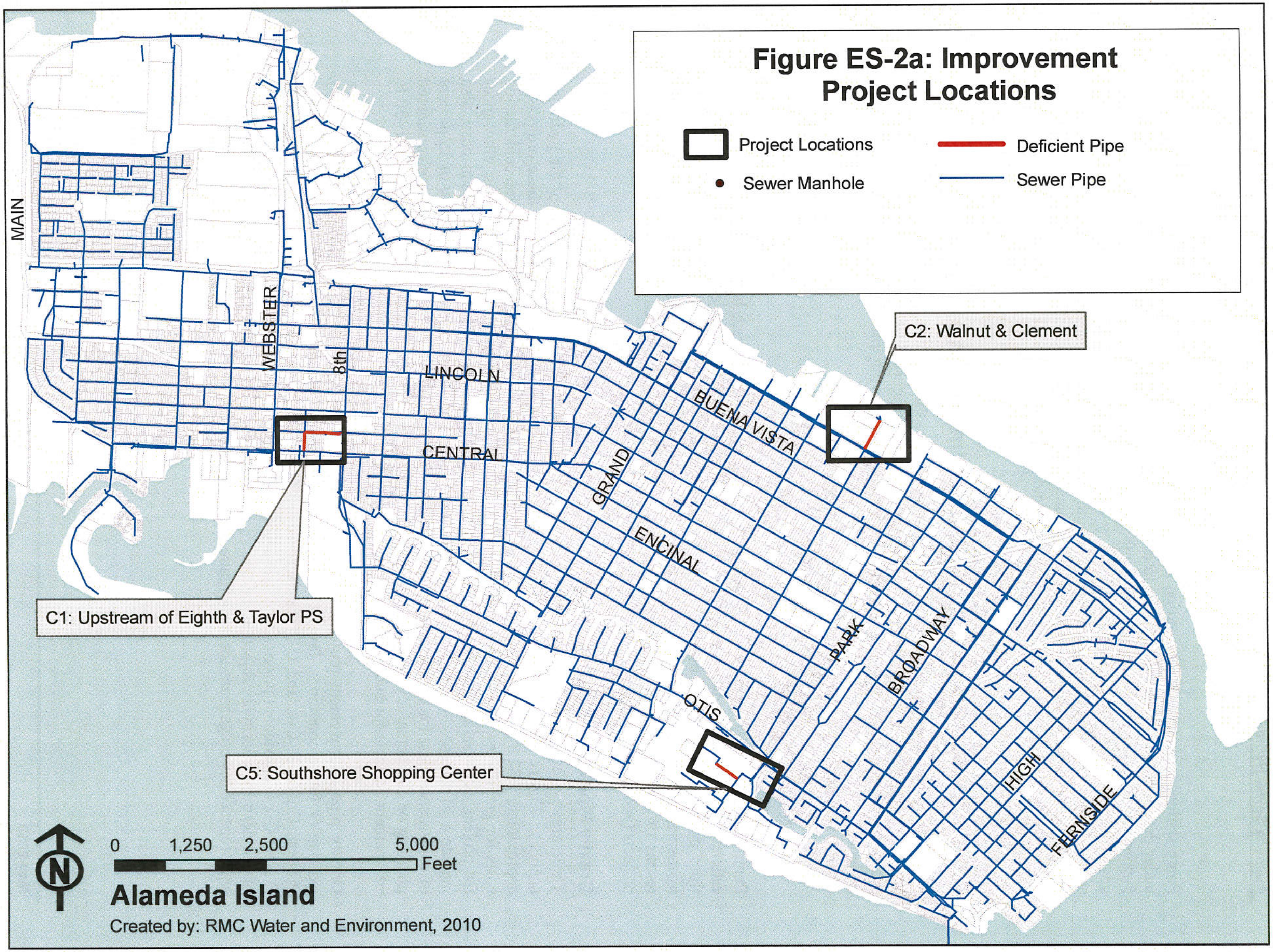


Figure ES-2a: Improvement Project Locations
Project Locations
Sewer Manhole
Deficient Pipe
Sewer Pipe
C2: Walnut & Clement
C1: Upstream of Eighth & Taylor PS
C5: Southshore Shopping Center
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
PARK
BROADWAY
OTIS
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

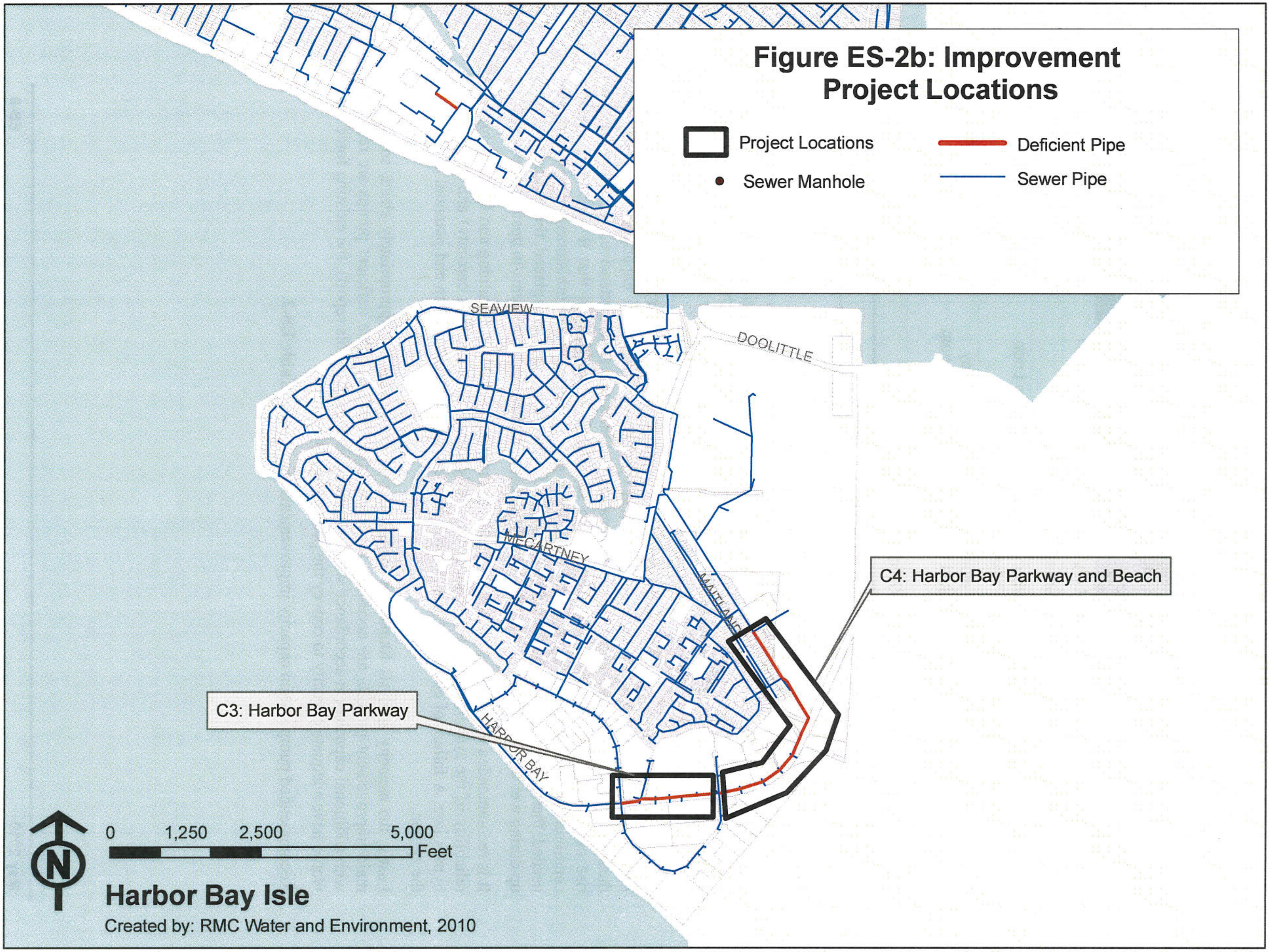


Figure ES-2b: Improvement Project Locations
Project Locations
Sewer Manhole
Deficient Pipe
Sewer Pipe
SEAVIEW
DOOLITTLE
MCCARTNEY
MAITLAND
HARBOR BAY
C4: Harbor Bay Parkway and Beach
C3: Harbor Bay Parkway
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

Table: ES- 1: CIP Project Summary

CIP No.	Project Description & Location	Priority	Estimated Capital Cost (2010 Dollars)
C-1	Upsize 6" pipe to 8" upstream of Eighth and Taylor Pump Station	1	$322,000
C-2	Upsize 6" pipe to 8" at Walnut Street and Clement Ave	2	$186,000
C-3	Upsize 10" pipe to 15" along Harbor Bay Parkway between Loop Road and Harbor Bay Parkway 1 pump station	2	$599,000
C-4	Upsize 12" pipes to 15" and 15" pipes to 21", along Harbor Bay Parkway and Beach Road from just downstream of the HB1 pump station to Seminary Ave	2	$1,501,000
C-5	Upsize 8" pipe to 10" pipe near Southshore Shopping Center	2	$155,000
	Total Estimated Capital Investment		**$2,763,000**

Pump Station Improvement Projects

The pump station capacity analysis revealed that, at a minimum, seven pump stations should be retrofitted with additional capacity (Grand Otis, Aughinbaugh, BFI, Park/Otis, Eight/Portola, and Harbor Bay Parkway 1, and Tideway). Based on this analysis, these seven pump stations have the potential to severely back up under the design storm event and in some cases during dry weather flow conditions, which may cause water levels in manholes to rise close to ground elevation.

The pump station analysis also identified several other pump stations (Pond/Otis, Willow/Whitehall, Sand Beach, Verdemar, and Dublin) as having less than optimal, yet acceptable, capacity. More specifically these pump stations backed up during some scenarios, but backup was not extensive, nor did water levels rise close to ground elevation. For these pump stations, it is recommend that if the City's ongoing condition assessment work finds these pumps to be in poor condition and refurbishment or replacement is needed, then new, larger pumps should be installed. If these pumps are not replaced, the system may back up during wet weather flows which, over time, may cause maintenance problems in upstream sewers.

It is recommended that standby pumps be installed at two pump stations, Channing and Haile, to improve reliability of the system. These two stations have only one pump (zero firm capacity) and no high-level bypass line. A failure of either of these pumps could cause significant backup and potential overflows in the system.

Lastly, the capacity analysis found that force mains in the system are appropriately sized, so no force main improvement projects are necessary for the existing system. If the deficient pumps are retrofitted with additional capacity (in accordance with the recommendations in this report), the existing force mains would have adequate capacity to convey the peak design flows.

Recommended pump station capacity improvements are listed in **Table ES-2.**

Table ES- 2: Pump Station Capacity Improvement Recommendations

Pump No.	Pump Station Name	Recommendation	Recommended Firm Capacity (MGD)
22	Grand Otis	Install more capacity	0.89
5	Aughinbaugh	Install more capacity	0.41
12	BFI	Install more capacity	3.10
25	Eighth/Portola	Install more capacity	1.60
16	Park/Otis	Install more capacity	1.20
10	Harbor Bay Parkway I	Install more capacity	1.17
8	Tideway	Install more capacity	0.35
20	Pond/Otis	Install more capacity if pump is in poor condition	0.21
9	Willow/Whitehall	Install more capacity if pump is in poor condition	0.23
18	Verdemar	Install more capacity if pump is in poor condition	0.17
27	Dublin	Install more capacity if pump is in poor condition	0.34
23	Sand Beach	Install more capacity if pump is in poor condition	0.14
3	Channing	Install standby capacity	Same Capacity
42	Haile	Install standby capacity	Same Capacity

Project Implementation Recommendations

The City should begin implementation of the Capital Improvement Program recommended in this Report, starting with the highest priority projects. This plan does not specify an implementation schedule, as the City will need to balance sewer improvements with the need for other capital projects (specifically, pump improvements). The following items should be considered in project scheduling and design, and in future updates of the Sanitary Sewer System Hydraulic Analysis.

- Move forward with further planning and design of the Priority 1 projects.
- All pipe improvement projects detailed in this report are based on pipe replacement. The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.
- The hydraulic model has been developed to assist the City in performing capacity analyses and updating the Sewer System Management Plan (SSMP) in the future. The model should be kept up-to-date with any changes to existing sewer connections, development plans, and sewer system facilities.
- The City should continue with the current sewer inspection and condition assessment program, identifying sewers that should be replaced due to poor condition. To the extent possible, these improvements should be coordinated with the recommended capacity-related improvements.
- The City should assess its sewer rates and connection fees as needed to ensure adequate funding for the recommended capacity improvement CIP.

In addition to the project implementation recommendations listed above, the City should continue to address I/I through continued inspection and rehabilitation of sewer mains and lower laterals. The findings in this Report should also be updated whenever there are major changes in planning assumptions or significant additional rehabilitation of the sewer system.

Chapter 1 Introduction

This report presents the results and recommendations of the Sanitary Sewer System Hydraulic Model Analysis for the City of Alameda (City). The Hydraulic Analysis Report was prepared by RMC Water and Environment (RMC). This introductory chapter provides background information on the scope and objectives of the modeling work, the City's sewer system and service area, and the contents and organization of this study report.

1.1 Background and Study Objectives

In April 2008, the City retained RMC to develop a hydraulic model of the City's sanitary sewer system and use the model to identify capacity deficiencies and recommend improvements projects.

The capacity of Alameda's sewer system was last evaluated in the 1980s as part of the East Bay Infiltration/Inflow Study Sewer System Evaluation Survey (SSES). Since that time, flows in the system have changed due to new development and redevelopment, as well as sewer system rehabilitation conducted by the City based on the results of the SSES. Furthermore, additional growth is projected in the future, which will further increase wastewater flows.

This hydraulic analysis will help the City meet the requirements to complete a capacity evaluation and capacity assurance plan as part of preparing its Sewer System Management Plan (SSMP), as well as provide information to update projected sewer improvement project needs in the City's Capital Improvement Program (CIP). The SSMP addresses the overall management, operation, and maintenance of the sanitary sewer system and is required for all sewer system agencies by the San Francisco Bay Regional Water Quality Control Board, as well as under the Statewide General Waste Discharge Requirements adopted in 2006 by the State Water Resources Control Board.

The overall objectives of this study are to:

- develop wastewater flow projections for the City's sewer service area using up-to-date land use information and flow monitoring data;
- develop a hydraulic model of the sewer system;
- use the model to identify existing capacity deficiencies and future capacity requirements; and
- develop solutions to capacity deficiencies, including budget estimates, for implementing the needed capacity improvements to the wastewater collection system.

1.2 Service Area and Sewer System

The City of Alameda, shown in **Figure 1-1,** is located in Alameda County on the edge of the eastern side of San Francisco Bay, west of the City of Oakland. A major portion of the City lies on Alameda Island, which is physically separated from Oakland by a 300 to 1,000-foot wide channel, known as the Oakland Inner Harbor or Oakland Estuary. The southernmost portion of the City lies on a peninsula that is shared with Oakland International Airport. This portion of the City goes by the name of Harbor Bay Isle (formerly known as Bay Farm Island). The northwestern end of Alameda Island comprises the former Alameda Naval Air Station, now known as Alameda Point.

Alameda is characterized by well-established single family residential neighborhoods, with pockets of medium density residential development. Several recreation areas such as parks and golf courses are spread throughout the city. The city also includes areas of commercial and light industrial land uses, particularly along Park Street, Webster Street, and Marina Village Parkway, along the harbor side of the main island, and on Harbor Bay Isle, where a substantial portion of the area's developments are commercial business parks. The Alameda Point area is currently undergoing redevelopment, and several new adjacent residential and commercial areas have developed in recent years.

Alameda's sanitary sewer system includes 34 City-owned pump stations and about 140 miles 6-inch through 27-inch diameter sewers that discharge into several pump stations and large diameter interceptor pipelines owned and operated by the East Bay Municipal Utility District (EBMUD). Wastewater collected by these interceptors flows to the Alameda siphons, located at the northernmost point of Alameda island at the end of Webster Street, through which the flow is conveyed across the estuary to EBMUD's South Interceptor and eventually to EBMUD's wastewater treatment plant in Oakland near the entrance of the San Francisco-Oakland Bay Bridge. The existing sewer system on Alameda Point (portion of system located northwest of Main Street) is not owned by the City (although the City does maintain the facilities under contract), and is therefore not included in this study. Flows from Alameda Point are conveyed via an EBMUD pump station and force main to the Alameda siphon inlet structure, and therefore do not impact the City's sewer system.

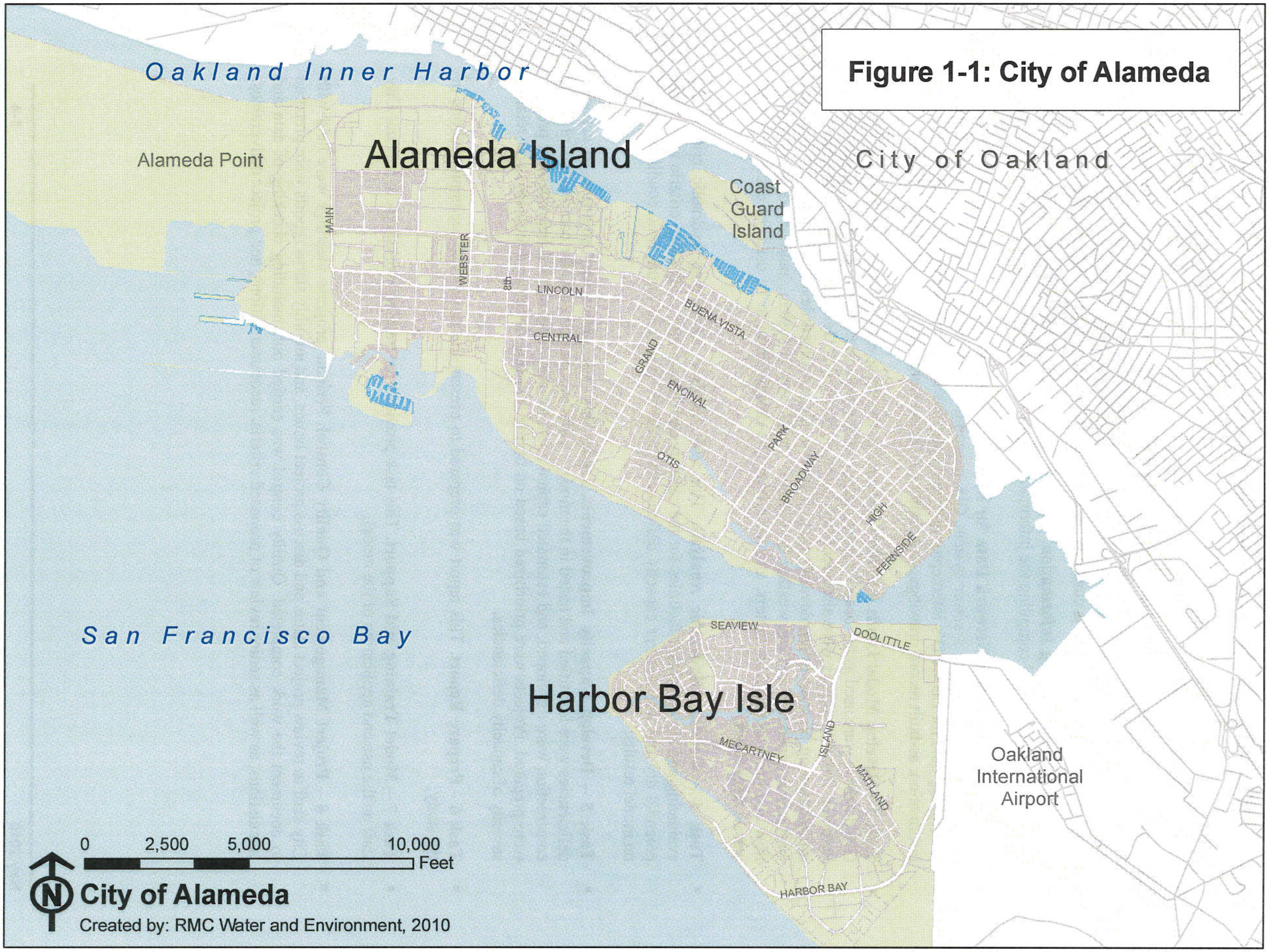


Figure 1-1: City of Alameda
Oakland Inner Harbor
Alameda Point
Alameda Island
City of Oakland
Coast Guard Island
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
San Francisco Bay
SEAVIEW
DOOLITTLE
Harbor Bay Isle
MECARTNEY
ISLAND
MAITLAND
HARBOR BAY
Oakland International Airport
0
2,500
5,000
10,000
Feet
N
City of Alameda
Created by: RMC Water and Environment, 2010

1.3 Scope of Study

The scope of this Sanitary Sewer System Hydraulic Model Analysis, as well as a brief discussion of work conducted under each task, is described below. Capacity assessment was the primary focus of this work; the study did not include inspections or assessments of the structural condition or maintenance issues of sewers or pump stations. The City is currently undertaking that work under separate projects.

- **Task 1 – Review Existing Information.** This task involved assembling, organizing, and reviewing maps and documents, and compiling the information in a format that was useful for subsequent tasks.
- **Task 2 – Develop Sewer Network Data for Model.** This task involved compilation and review of data contained in the City's sewer system GIS mapping and inventory database. It also included developing data for EBMUD's interceptor and pump stations to be included in the model. Block book information, as-built record drawings, and digital CAD data were used to corroborate GIS data.
- **Task 3 – Develop Model Loads and Flow Factors.** In this task, existing data from the City's parcel GIS database were used as the basis for computing existing wastewater flows for the hydraulic model. Potential future land uses were also identified based on the City's General Plan and other sources of information on planned future development. This task also involved developing criteria to estimate wastewater generation and wet weather flows in the sanitary sewer system, including base wastewater flow and infiltration/inflow factors, and identifying the design storm to be used for estimating peak wet weather flows.
- **Task 4 – Conduct Hydraulic Analysis.** This task involved developing hydraulic criteria for evaluating and sizing system facilities for the capacity assessment and using those criteria to identify potential dry and wet weather capacity deficiencies under existing and future flow conditions based on model results.
- **Task 5 – Develop Capacity Improvements.** Under this task, preliminary solutions to capacity deficiencies were identified and tested in the model. Unit construction costs for components of sewer construction were developed, and estimated capital costs for the recommended improvement projects were prepared. Projects were prioritized based on timing (existing or future deficiency) and relative severity of capacity deficiencies.
- **Task 6 - Prepare Report.** This report was prepared to present the results and recommendations of the study.
- **Task 7 – Model Training and Support.** This task provides for training workshops to instruct City staff on the use and application of the model.
- **Task 8 – Project Management and Quality Control.** Periodic progress meetings were held with City staff to review project status and discuss project issues, and monthly status reports were prepared to document the work completed. Quality control was carried out, including checking of data and calculations, as well as senior review of proposed criteria, recommendations, and project deliverables.

Chapter 2 Land Use Planning Scenarios

The first step in assessing sewer capacity is to define the spatial distribution and magnitude of residential, commercial, institutional, and industrial land uses that generate sewer flows. The second step is to make a determination about how these characteristics may change over time. This section describes current land use as well as the future changes in residential and commercial developments that will impact future wastewater flows. This Hydraulic Analysis Report considers two planning scenarios: Existing and Future. For the purposes of this analysis, the Existing Scenario is considered to be 2008. The Future Scenario represents (approximately) 2040 conditions and incorporates new residential and commercial projects as well as redevelopment projects that may occur in the next 30 years.

2.1 Land Use

Alameda consists of a range of development types ranging from low density, single family homes to high density, multi-family developments, as well as parks, commercial and industrial uses. Data about existing land uses by parcel was available from the City's Geographic Information System (GIS) database.

In terms of sewer loads, there are two main categories of land uses that are important to identify – residential and non-residential. These two categories have different wastewater flow patterns and loading rates. Residential land-uses include single and multi-family dwelling units. Non-residential uses include a variety of types of business and public facilities including:

- Retail and service establishments
- Restaurants
- Car washes
- Medical facilities
- Hotels
- Offices
- Warehouses
- Schools
- Parks

Figure 2-1 shows the distribution of residential and non-residential land uses throughout the City. The location and characteristics of existing developed areas were delineated based on parcel use data contained in the City's parcel GIS. The data identify the current parcel land use, square footage of building floor space, and number of dwelling units for residential parcels. This parcel data was the basis for estimating sewer flows.

In addition to the information contained in the parcel database, specific information on school populations was obtained by contacting the Alameda Unified School District and College of Alameda. This enabled development of more accurate estimates of wastewater flows from schools.

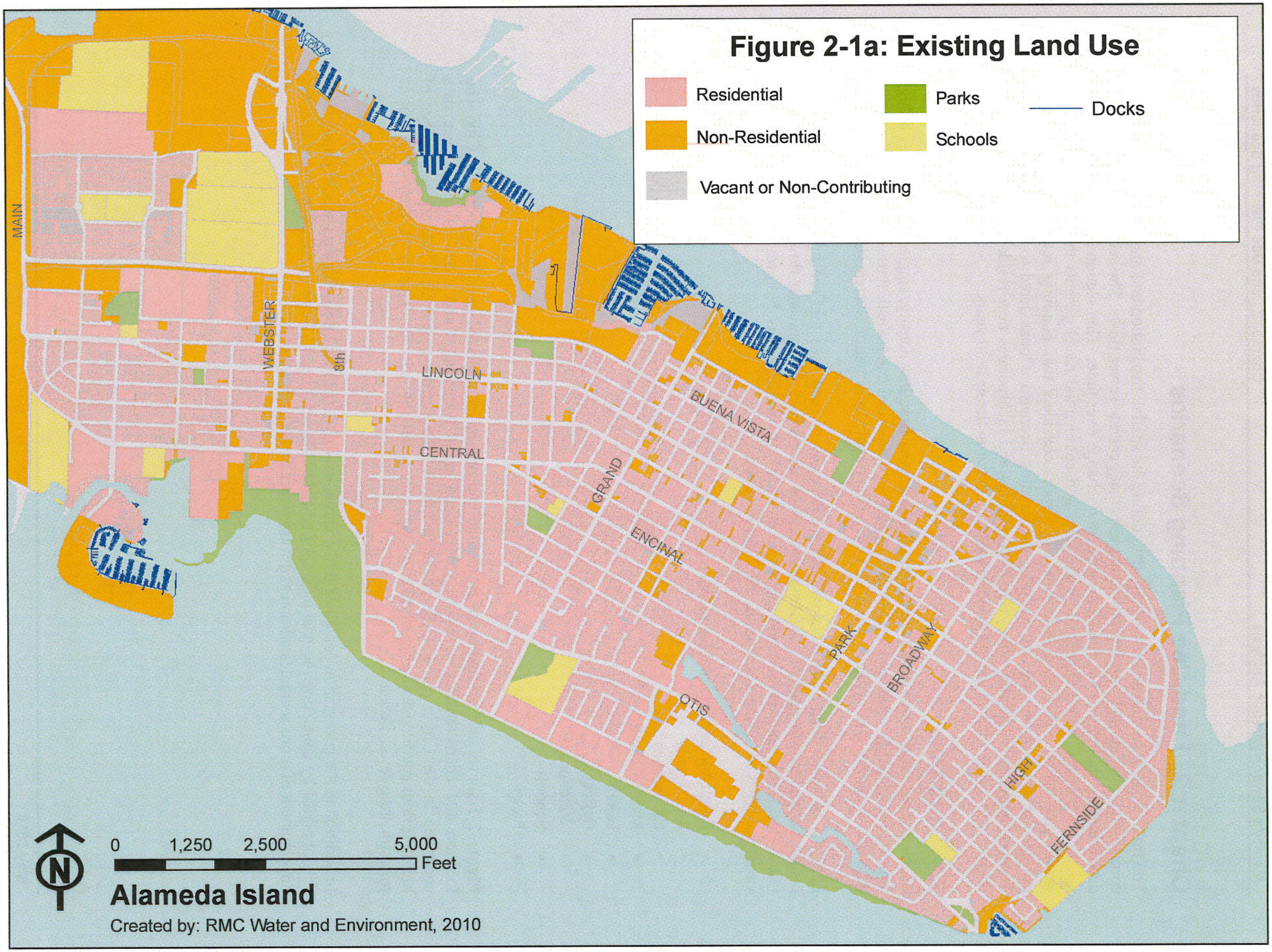


Figure 2-1a: Existing Land Use
Residential
Non-Residential
Vacant or Non-Contributing
Parks
Schools
Docks
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

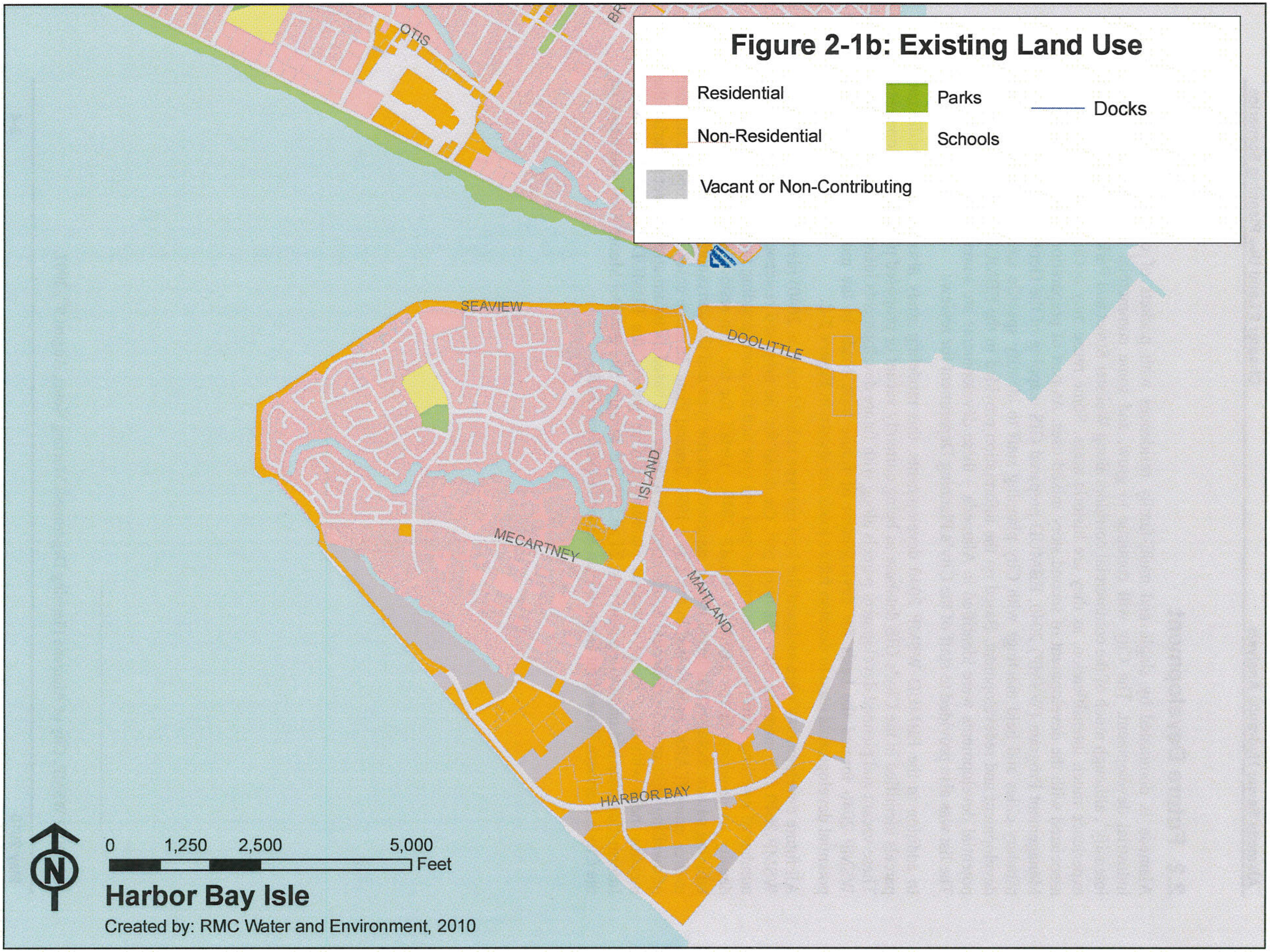


Figure 2-1b: Existing Land Use
Residential
Non-Residential
Vacant or Non-Contributing
Parks
Schools
Docks
OTIS
SEAVIEW
DOOLITTLE
ISLAND
MECARTNEY
MAITLAND
HARBOR BAY
N
0
1,250
2,500
5,000
Feet
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

2.2 Future Development

Alameda is dominated by single and multi-family residences with pockets of commercial and light industrial development. The City will continue to grow, and some commercial uses are expected to intensify[1], although much of the commercial corridors along Webster Street and Park Street will likely not experience much intensification, as they are historic areas. Some reuse may occur throughout the City associated with the development of second story mixed uses. As part of preparation of its Water Supply Management Program (WSMP) 2040, EBMUD prepared GIS mapping of the General Plans of all of its tributary cities and held meetings with City planning staff to identify specific areas of potential future development and redevelopment. Based on the information provided in the EBMUD documents, several potential developments were identified in Alameda – these developments are summarized in **Table 2-1.** The list was also provided to staff of the City's Planning Department for review.

In addition to the EBMUD WSMP 2040 developments, this modeling study also identified additional parcels identified in the City's GIS database as being currently vacant or proposed planned developments. These vacant and planned development parcels along with future developments identified by the EBMUD WSMP 2040 formed the basis for development of future base loads in the model. The location of potential developments used to develop future loads are shown in **Figure 2-2.**

All future developments were assigned the land use type at build out (i.e. 2040) identified in the EBMUD WSMP 2040 GIS mapping. For the most part, the detail for the proposed residential developments (i.e. number of dwelling units) was based on the land use type and the corresponding maximum dwelling unit density as provided in the EBMUD WSMP 2040. Detail for the non-residential developments (i.e. building square footage) was based on the assigned land use type and the corresponding maximum floor-to-area ratios (FARs) as indicated in the City's 1991 General Plan. In some cases, the proposed number of dwelling units or non-residential building square footage was based on information collected during the meeting held with the City of Alameda planning staff as part of the EBMUD WSMP 2040 work. Information obtained on development nos. 24 and 25 (i.e. Storage Site and Del Monte Site) was based on the City's Northern Waterfront General Plan Amendment Draft EIR.

[1] EBMUD WSMP, City of Alameda Planning Department Meeting Notes, August 9th, 2007.

Table 2-1: City of Alameda Potential Future Developments

No.	Name	Description
1	Alameda Gateway	Located south of Rosenblum Cellars. Redevelopment to single-family residential uses with 15 dwelling units (DUs) per acre by 2015. Size of redevelopment area is approximately 12 acres containing up to 180 multi-family DUs.
2	Alameda Landing	Located north of Mitchell Avenue. Current land use type is light intensity industrial. Proposed redevelopment to high density office uses with approximately 400,000 square feet (SF) of building space.
3	Alameda Landing	Located north of Mitchell Avenue. Proposed redevelopment to retail uses with approximately 100,000 SF of building space.
4	Alameda Landing	Same as #3 abovewith approximately 200,000 SF of building space.
5	Alameda Landing	Located south of Mitchell Avenue and west of Fifth Street. Proposed redevelopment to single-family housing with approximately 250 DUs.
6	Alameda Landing	Located north of Mitchell Avenue and currently vacant. This site is proposed for single-family housing with approximately 50 DUs.
7	Alameda Landing	This site is located north of Tinker Avenue and is proposed for redevelopment to multi-family residential with approximately 39 condos.
8	Coast Guard Housing	This parcel is the site of the former Coast Guard housing development. It is proposed for redevelopment to single-family residential with approximately 15 DUs per acre. The size of the development area is approximately 50 acres resulting in 750 proposed DUs.
9	Park and Ride	Located east of Webster Street. Currently vacant. Proposed development to a Park & Ride by 2015. Size of development area is 3 acres.
10	Town Center	Located in the former South Shore Shopping Center. Proposed intensification of current commercial uses with an additional 100,000 SF of commercial space.
11	Northern Waterfront (Grand Marina)	Located north of Fortman. Currently used as retail and industrial uses. Proposed redevelopment as high density residential uses with 40 multi-family DUs by 2010. Size of redevelopment area is approximately 5 acres.
12	Northern Waterfront (Grand Marina)	Located south of Fortman. Currently used as retail and industrial uses. Proposed redevelopment as high density residential uses with 10 to 20 DUs per acre by 2015. Size of redevelopment area is approximately 6 acres resulting in up to 119 multi-family DUs.
13	Northern Waterfront	Located north of Clement between Park and Blanding. Currently used as low intensity industrial uses. Proposed redevelopment as mixed use residential with 10 to 20 DUs per acre by 2015. Size of development area is approximately 23 acres resulting in up to 1,087 multi-family DUs.
14	Northern Waterfront (Pennzoil Site)	Located at Fortman Way and Grand St. Currently used as low intensity industrial uses. Proposed redevelopment as retail and industrial uses by 2020. Size of redevelopment area is approximately 2 acres. Total proposed square footage of commercial/retail uses is 27,000 SF (existing Alaska Packers Bldg) plus 2,500 SF for marina retail.

No.	Name	Description
15	Northern Waterfront (Encinal Terminal)	Located north of Entrance. Currently used as low intensity industrial uses. Proposed redevelopment as mixed use residential uses by 2025. Size of redevelopment area is approximately 12 acres. Total Encinal terminal proposed residential units is 165 DUs (likely multi-family).
16	Northern Waterfront (Encinal Terminal)	Same as #15 except proposed redevelopment to commercial/industrial land uses. Total Encinal terminal proposed commercial/industrial land use is 200,000 SF.
17	Northern Waterfront (Encinal Terminal)	Located north of Entrance. Currently used as low intensity industrial uses. Proposed redevelopment as a school by 2035. Size of redevelopment area is approximately 4 acres.
18	Northern Waterfront	Located north of Buena Vista between Entrance and Nautilus. Currently used as low intensity industrial uses. Proposed redevelopment as high density residential uses with 10 to 20 DUs per acre by 2015. Size of redevelopment area is 8 acres resulting in up to 159 multi-family DUs.
19	NA	Located west of McKay. Currently used as office and industrial uses. Proposed redevelopment as high density residential uses with 10 to 20 DUs per acre by 2020. Size of development area is approximately 10 acres resulting in approximately 198 multi-family DUs.
20	Theater	Located at Park and Central. Currently vacant. Proposed development to office, retail, services, and industrial uses by 2010 (FAR of 0.5). Size of development area is approximately 2 acres resulting in 34,000 SF of building space.
21	Harbor Bay Business Park	Located on the south-eastern portion of Harbor Bay Isle. Proposed intensification of office uses. Total proposed new square footage is approximately 1,034,000 SF (assumed FAR of 1.0).
22	NA	Located West of Harbor Bay between Maitland and Garden. Currently vacant. Proposed development to office, retail, services, and industrial uses by 2010. Size of development area is 5 acres (assumed FAR of 0.5 with a resulting building square footage of approximately 113,000 SF).
23	Shipway	Located east of Invincible. Proposed development of a 145,000 square foot of office space or 11 dwelling units by 2025.
24	Storage Site	29 single-family dwelling units. Land use classification is medium density residential (source: Northern Waterfront General Plan Amendment Draft EIR).
25	Del Monte Site	235,000 SF building of mixed uses (this site is currently used as warehouse). The development would include 29 multi-family dwelling units. Land use classification is medium density residential (source: Northern Waterfront General Plan Amendment Draft EIR).

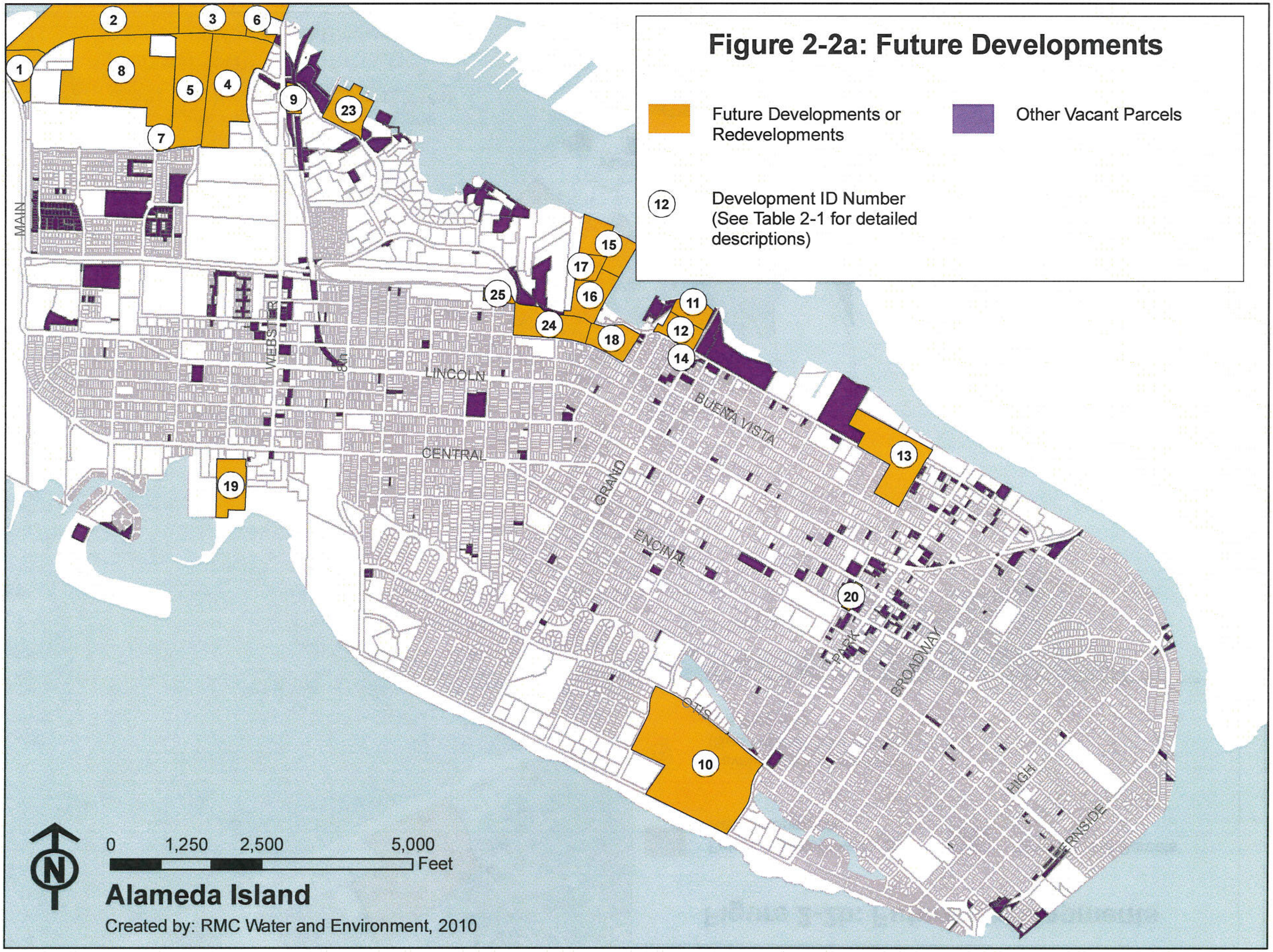


Figure 2-2a: Future Developments
Future Developments or Redevelopments
Other Vacant Parcels
12
Development ID Number (See Table 2-1 for detailed descriptions)
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
23
24
25
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
ENCINAL
BUENA VISTA
OTIS
PARK
BROADWAY
HIGH
N
0
1,250
2,500
5,000
Feet
Alameda Island
Created by: RMC Water and Environment, 2010

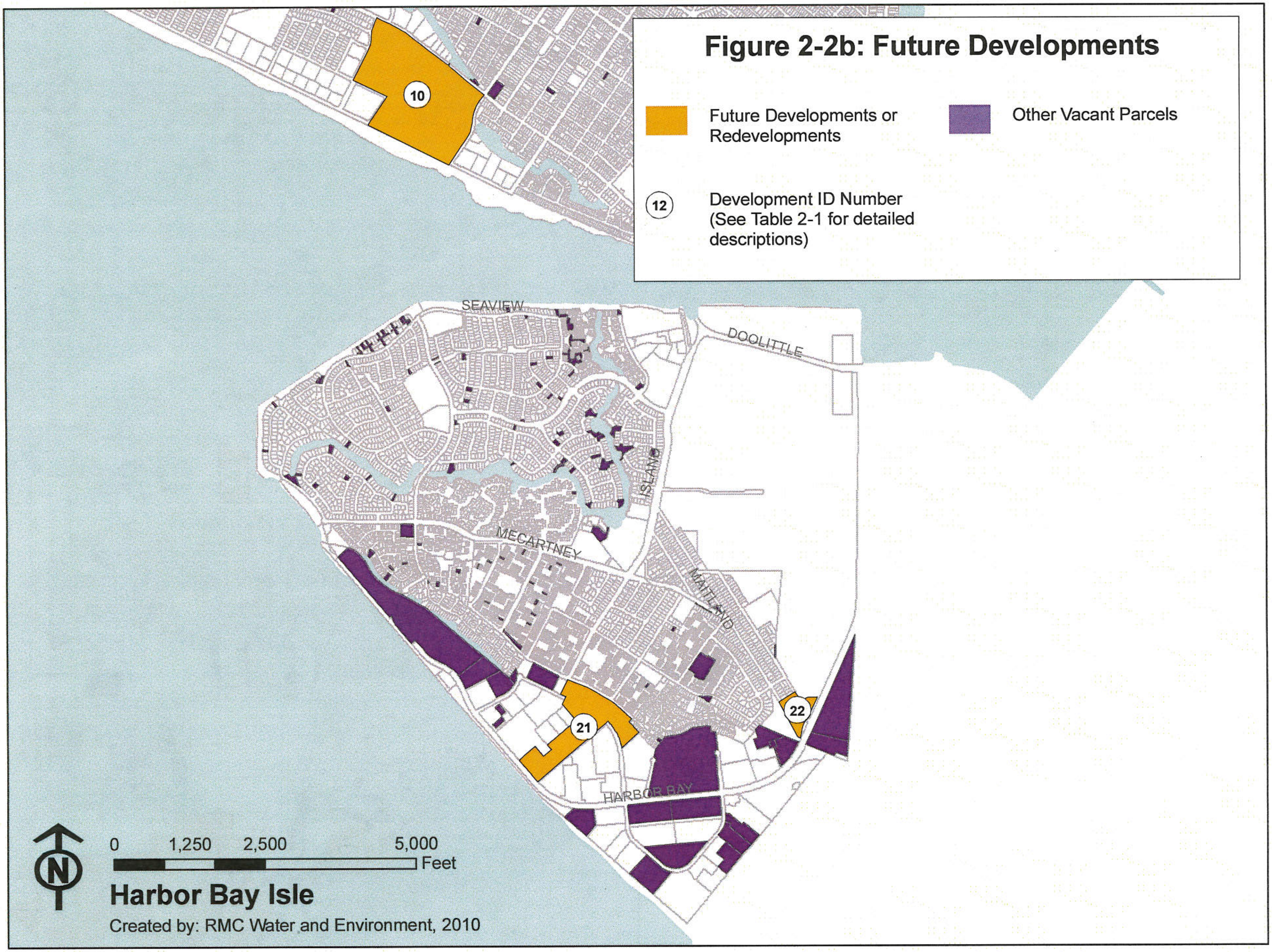


Figure 2-2b: Future Developments
Future Developments or Redevelopments
Other Vacant Parcels
12
Development ID Number (See Table 2-1 for detailed descriptions)
10
21
22
SEAVIEW
DOOLITTLE
ISLAND
MECARTNEY
MAITLAND
HARBOR BAY
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

Chapter 3 Hydraulic Model Development

A hydraulic model of the City's sewer system was developed to estimate flows and assess sewer and pump station capacities. This section describes the modeled system and modeling methodologies used. Topics include the development and validation of the sewer network and delineation of model sewersheds to generate wastewater loads.

3.1 Model Development Overview

RMC utilized InfoWorks CS™ modeling software for the sewer system hydraulic analysis. Several steps are involved in the model building and application process in order to ensure that the model will accurately predict existing and future flows and capacity limitations. This methodology is summarized below. Additional details are presented in subsequent sections of this chapter.

- Define the model network, extract the data on each modeled pipe segment and manhole required for modeling (spatial coordinates; manhole identifiers, rim and invert elevations; pipe invert elevations, diameters, and lengths), validate all data (i.e., check for and correct missing or erroneous data values), and further develop data as necessary based on the validation.
- Compile data on pump stations to be included in the model, including wet well elevations and dimensions, pump curves, on/off levels, and discharge capacities.
- Divide the service area into sewersheds, consisting of areas tributary to modeled manholes.
- Compile information on land use and development for both existing and future conditions using parcel data and other available planning information.
- Develop unit flow factors and diurnal patterns to estimate base wastewater flow (BWF) loads into the modeled system from each sewershed.
- Select representative dry weather days from available flow monitoring data and compare model results to metered flows. Calibrate the model by making reasonable adjustments to unit flow factors, 24-hour diurnal flow patterns, and dry weather groundwater infiltration (GWI) rates as needed to match the observed flow volumes and peaks at each meter.
- Select wet weather events for use in wet weather flow calibration. For the selected events, compare model results to meter data to develop and calibrate model wet weather parameters governing the volume and response pattern of rainfall-dependent infiltration/inflow (RDI/I) into the modeled system.
- Identify an appropriate design storm to be used for analyzing and determining the required capacity of sewer system facilities.
- Determine appropriate hydraulic design and capacity analysis criteria.
- Run the calibrated model to identify sewer reaches and/or pump stations having capacity deficiencies under existing and future design flows.
- Develop and test potential solutions to capacity deficiencies, such as flow diversions, parallel pipes, larger replacement pipes, and pump station upgrades.

3.2 Modeled Sewer System

The modeled sewer network, shown in **Figure 3-1**, consists of the entire sewer system owned by the City of Alameda (with a few exceptions as described below), plus the EBMUD interceptors located within the City, but not including the system serving Alameda Point, which is not owned by the City. Several areas with private sewer systems owned by homeowner associations (mostly on Harbor Bay Isle) are also not included in the model, although the flows from those developments discharging into the Alameda sewer system are accounted for. The modeled system includes over 30 pump stations and over 140 miles of 6- through 60-inch diameter pipe. Approximately 75 percent of the modeled pipes are 8 inches in diameter and smaller.

It should be noted that while the EBMUD interceptor facilities have been included in Alameda's hydraulic model, the capacities of those facilities were not assessed for this study. The purpose of including the EBMUD facilities in the model is to provide a single, hydraulically connected system (essentially, the interceptor serves as the "backbone" of the entire Alameda sewer system) and ensure that the potential impact of flow levels in the interceptor are accounted for in the modeling of the City's sewers. Furthermore, the flow meters used for model calibration, as discussed later in this report, were located on the EBMUD interceptors.

The modeled collection system consists of links and nodes, which represent the major pipes, manholes, pumps, and lift station wet wells. The service area is divided into subareas (sewersheds), each of which defines the tributary area to a node on the modeled system. The following subsections discuss the modeled system and sewersheds.

3.2.1 Sewer Manholes and Pipes

The modeled sewer network was developed primarily from existing GIS files provided by the City, which had been created by another consultant by conversion of the City's AutoCAD sewer maps. This geo-referenced dataset included information about pipe diameters and lengths, invert elevations, and manhole rim elevations. The GIS data was supplemented by information from the City's CAD maps, record drawings, and other data sources. Data for the EBMUD interceptor facilities were confirmed through review of EBMUD's interceptor hydraulic model and as-built drawings of the interceptor facilities. Elevations for EBMUD facilities were adjusted to match the City of Alameda's datum.

The manholes in the system are identified by ID numbers that were originally assigned as part of the East Bay Infiltration/Inflow Study Sewer System Evaluation Survey (SSES) conducted in the 1980s. The manhole numbers were originally based on a sewer basin/subbasin numbering system, but there have been deviations from that numbering methodology since that time.

The final hydraulic modeled system represents almost the entire Alameda sewer system along with EBMUD's interceptor facilities in Alameda. However, several short 6-inch pipes, less than 100-feet long, and the network upstream of unmodeled pump stations (in addition to the unmodeled force main) have been excluded from the model to improve model performance and reduce the time required to construct the network.

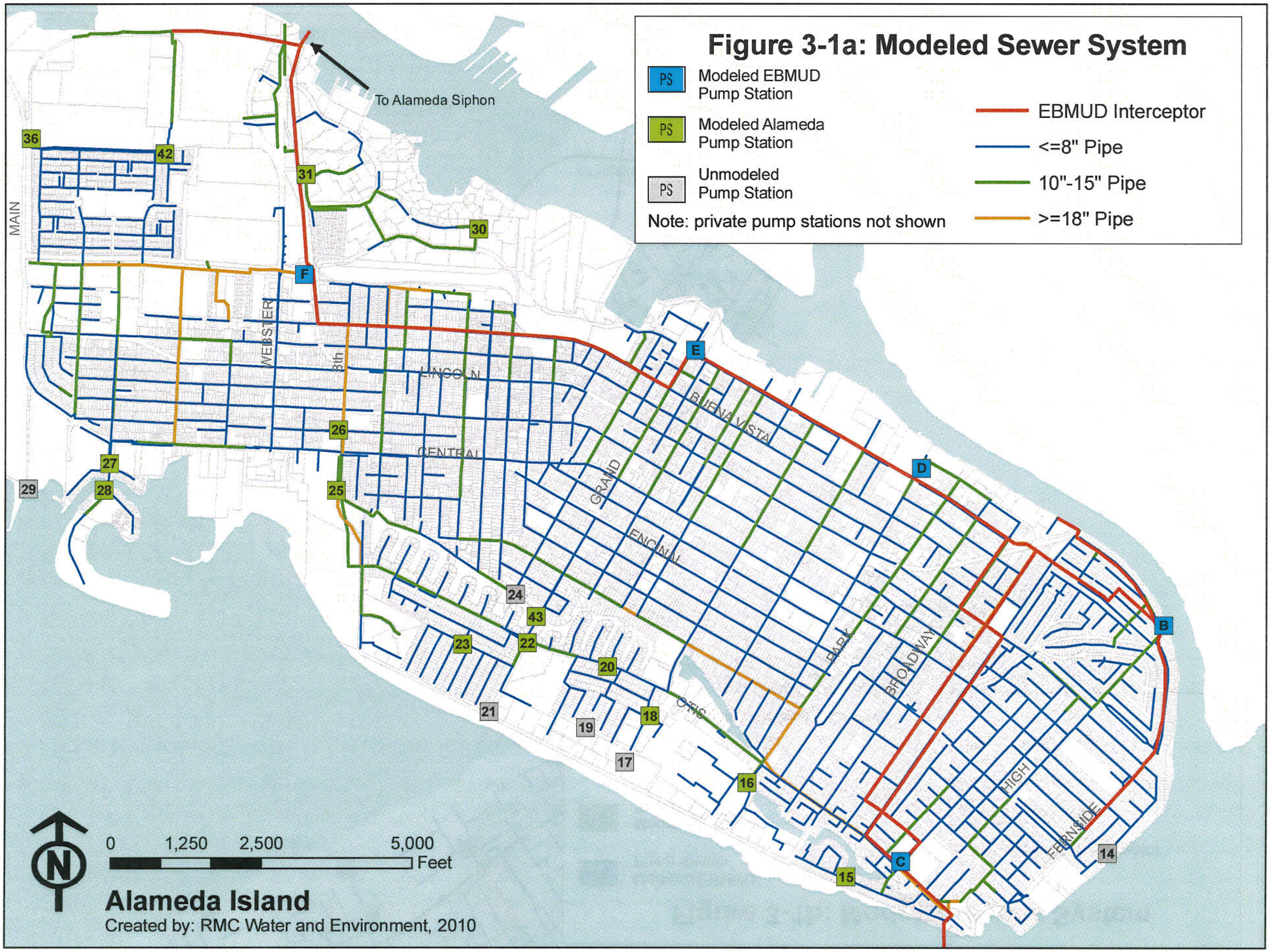


Figure 3-1a: Modeled Sewer System
PS
Modeled EBMUD Pump Station
PS
Modeled Alameda Pump Station
PS
Unmodeled Pump Station
Note: private pump stations not shown
EBMUD Interceptor
<=8" Pipe
10"-15" Pipe
>=18" Pipe
To Alameda Siphon
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
ENCINAL
BUENA VISTA
PARK
BROADWAY
OTIS
HIGH
FERNSIDE
A
B
C
D
E
F
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
36
42
43
N
0
1,250
2,500
5,000
Feet
Alameda Island
Created by: RMC Water and Environment, 2010

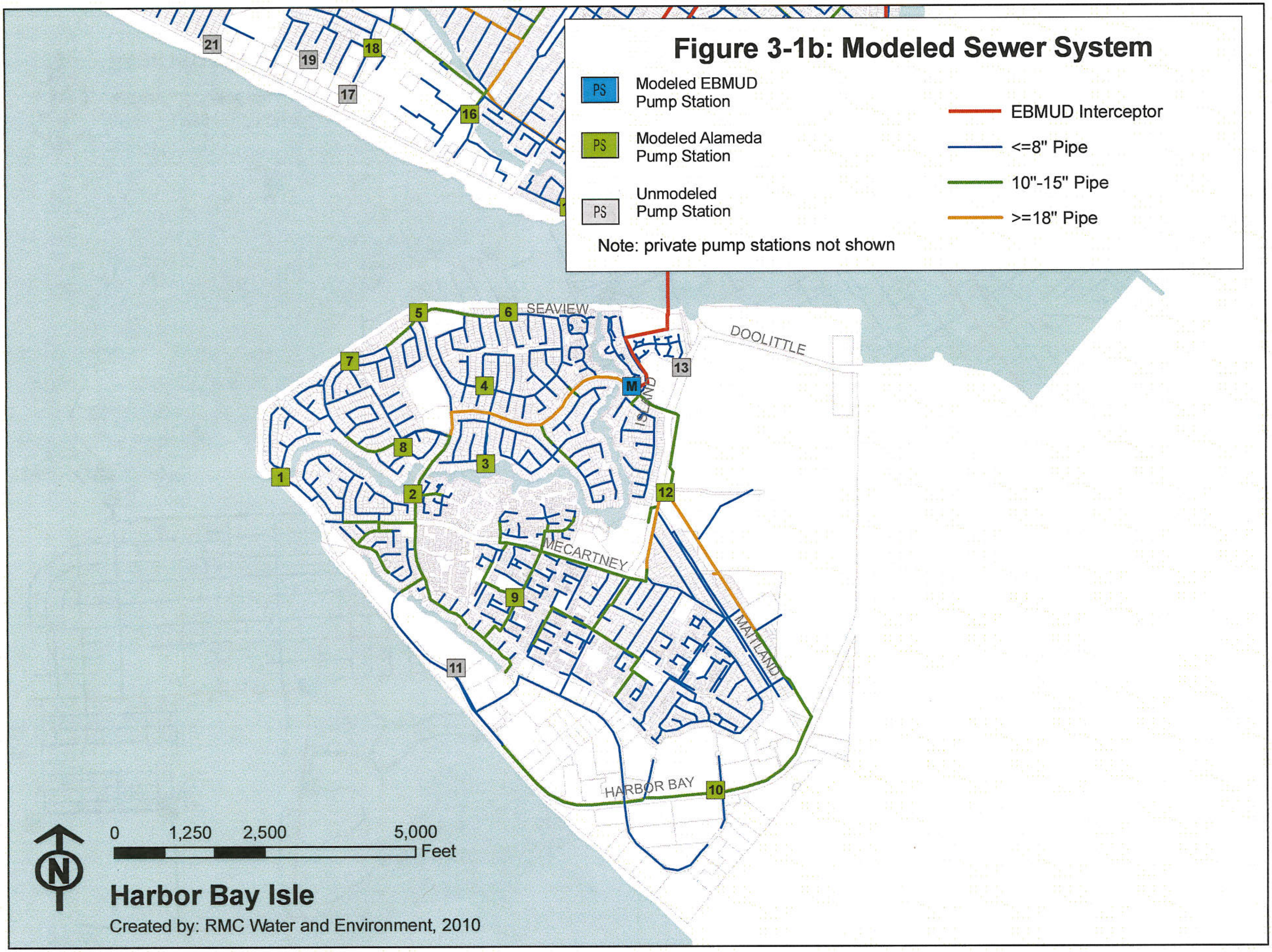


Figure 3-1b: Modeled Sewer System
PS
Modeled EBMUD Pump Station
PS
Modeled Alameda Pump Station
PS
Unmodeled Pump Station
EBMUD Interceptor
<=8" Pipe
10"-15" Pipe
>=18" Pipe
Note: private pump stations not shown
SEAVIEW
DOOLITTLE
ISLAND
MECARTNEY
MAITLAND
HARBOR BAY
M
1
2
3
4
5
6
7
8
9
10
11
12
13
16
17
18
19
21
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

As mentioned above, GIS files were the source of most data used to construct the majority of the existing sewer system, but for parts of the network, this information was supplemented with record drawings, the District's CAD maps, and survey data. The sources of all data in the model, whether obtained from the GIS, as-built drawings, or interpolation, are documented in the model using data "flags". The most common situations that required use of supplemental information were 1) incomplete or erroneous data and 2) portions of the system not yet included in GIS. These two data issues and how they were identified and mitigated is discussed below.

Incomplete or Erroneous Data

During data validation, some data were found to be missing or were suspected to be erroneous. These issues were identified using the following steps:

- The modeled network connectivity was checked (i.e., it was verified that correct upstream/downstream manholes were identified for each pipe and there were no missing links or isolated manholes in the network).
- Manholes and pipes with missing data (diameter, inverts, or rim elevations) were identified.
- Profiles for each series of pipe segments in the modeled network were reviewed to visually check for suspect data. Examples of suspect data include downstream pipes smaller than upstream pipes, pipe crowns above ground level, negative pipe slopes, or abrupt steps up or down in invert elevations.

Where these validation steps revealed invalid data, the City's block book records or digital CAD maps were used to populate or modify information in the database. If either of these supplemental sources of information were incomplete, the data were inferred using engineering judgment. Initially, about 30 percent of all pipes were associated with questionable or missing upstream invert elevation, downstream invert elevation, or pipe diameter (or some combination thereof). Of that 30 percent, about 65 percent of these issues were solved by referencing supplemental sources of information, and 35 percent had to be inferred.

Approximately 10 percent of all manholes were missing rim elevations. Missing rim elevations were also obtained from as-built drawings, block books, or CAD maps, where available. Where not available, rim elevations were inferred based on surrounding ground elevations.

GIS Data Updates

Several modifications to parts of the system (e.g., relief or replacement sewers) have been constructed since the GIS data were compiled. These areas were specifically identified by the City, and RMC updated the GIS database as necessary based on CAD maps or plans provided by the City.

There were also several new developments that had been incorporated into the City's CAD maps but not GIS. These developments were digitized in the model from CAD drawings, and relevant information such as pipe diameters, lengths, invert and manhole rim elevations were manually entered into the database. These areas included the Bayport (Catellus) development located adjacent to the College of Alameda, and three developments on Harbor Bay Isle.

3.2.2 Pump Stations

Out of the 40 public sewer pump stations located in Alameda, 32 were modeled based on information provided by the City, EBMUD, and by two consultants. Six of these pump stations are owned and operated by EBMUD and 26 are owned by the City. Seven of the 34 City-owned pump stations were not included in the model because their tributary areas are very small (the upstream pipe network totaled less than 500 feet).

Information about EBMUD pump stations was developed based on data from EBMUD, including wet well dimensions and pump curves. Information about Alameda's pump stations was based on data provided by the City, from field notes, and pump testing by Schaaf & Wheeler (under a separate contract to the City) and E2 Consulting Engineers (RMC's subconsultant for pump flow monitoring and testing). Wet well dimensions and pump set-points were read from as-built drawings and, when available, corroborated with observations made during field visits.

City-Owned Pump Stations

Most of the City-owned pump stations were modeled as fixed-discharge pumps. This is a simplified approach, but in order to more dynamically model these pumps, head-discharge curves (also referred to as "pump curves") are necessary. However, pump curves were available for only one, relatively new Alameda pump station (Grand Street). Instead, the discharge capacity of each pump station was estimated based on draw down testing. Draw down testing is a method used to approximate pump flow rate by measuring the time rate of change in wet well water surface elevation while the pump is operating. The drop in water level is converted to a volume based on wet-well dimensions, and flow rate through the pump is calculated as the change in volume divided by the time elapsed during the level measurement. Flow rates determined by this method are approximate – it is difficult to accurately measure the change in volume because the dimensions of many of the pump station wet wells vary with height. Nonetheless, draw down testing gives a good approximation of pump flows.

Draw down tests coupled with calculations of total dynamic head (static head plus friction losses) determine a pump's "operating point". This point represents one possible combination of flow and operating pressure with a single pump operating. The capacity of the pump station with two or more pumps operating was determined by using a calculated system curve and a theoretical, parallel pump curve – the combined flow rate is where the two curves intersect[2]. A graphical representation of this is shown in **Figure 3-2.**

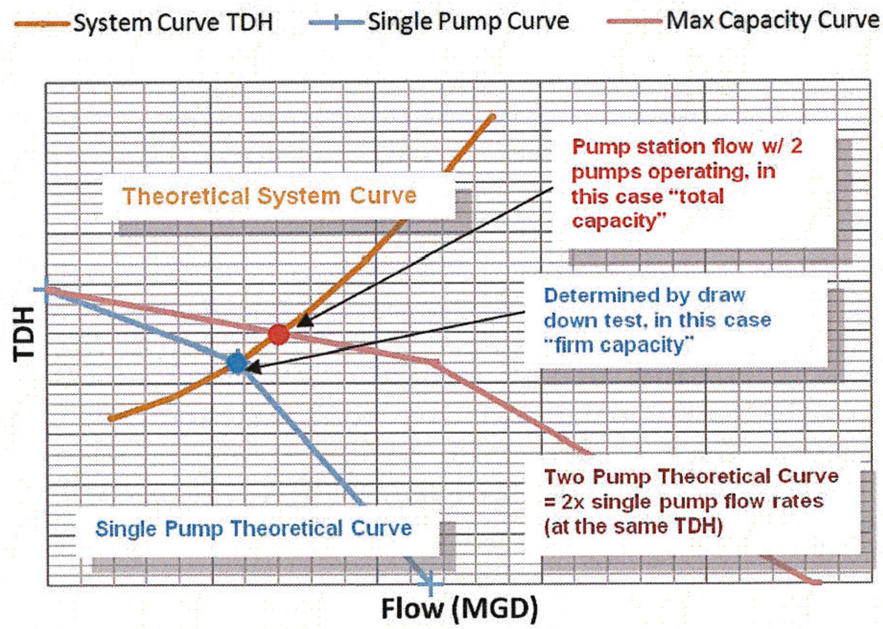


Figure 3-2: Development of Pump Curves

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[2] BFI and Park Otis are exceptions – pump station flow rates with more than one pump operating were determined directly using draw down tests (i.e. flow rates were not calculated for these pumps).

The firm and total capacities of Alameda's pump stations are show in **Table 3-1.** Note that "firm" capacity refers to the capacity of the station with the largest pump out of service and "total" capacity is the capacity of the station with all pumps in operation.

Table 3-1: Existing Firm and Total Pump Station Capacities (As Modeled)

Pump Station No.	Pump Station Name	# Duty Pumps	# Standby Pumps	Firm Capacity (MGD)	Total Capacity (MGD)
1	Adelphian	1	1	0.15	0.26
2	Catalina	1	1	1.16	1.40
3	Channing	1	0	--	0.09
4	Sheffield/Cumberland	1	0	--	0.06
5	Aughinbaugh	1	1	0.09	0.14
6	Seaview I	1	1	0.34	0.40
7	Seaview II	1	0	--	0.14
8	Dublin	1	1	0.18	0.26
9	Verdemar	1	1	0.07	0.14
10	Harbor Bay Parkway I	2	1	0.75	1.00
12	BFI	1	1	1.85	2.00
15	Bayview	1	0	--	0.78
16	Park/Otis	1	1	0.61	0.89
18	Willow/Whitehall	1	1	0.07	0.13
20	Pond/Otis	1	0	--	0.10
22	Grand/Otis	1	1	0.27	0.46
23	Sand Beach	1	0	--	0.06
25	Eighth/Portola[7]	1	1	0.67	1.35
26	Eighth/Taylor	1	1	0.41	0.50
27	Tideway	1	1	0.23	0.44
28	Cola Ballena	1	1	0.19	0.32
30	Triumph/Independence	1	1	0.32	0.65
31	Marina Village	3	1	2.00	2.20
36	LS6	1	1	0.48	0.55
42	Haile	1	0	--	0.11
43	Grand Street	1	1	0.89	1.00
13	Bay Fairway Hall	Capacity Not Assessed			
11	Harbor Bay Parkway II				
14	Eastshore/Myers				
17	Willow				
19	Yorkshire/Franciscan				
21	Grand/Shoreline				
24	Paru				
29	Encinal Boat Ramp				

Note: "Firm" capacity refers to the capacity of the station with the largest pump not in operation and "Total" capacity is the capacity of the station with all pumps in operation. Firm and Total capacities do not account for pumps that may have operational or mechanical issues which prevent them from functioning properly. For pump stations with only one pump, firm capacity is zero.

EBMUD Pump Stations

Most EBMUD pumps were modeled as rotodynamic pumps (i.e., based on pump curves), with the exception of Pump Station C ("PSC") which was modeled as a screw pump. Pump station PSC utilizes variable frequency drives, while other EBMUD pumps are constant speed. Although variable frequency pumps can be modeled very accurately using "real time control" rules in InfoWorks, modeling them as screw pumps provides a somewhat easier yet suitable approach. Screw pumps, which gradually pump out more flow as wet well levels increase (up to the estimated capacity of the pump station) provide a reasonable representation of the operation of a variable speed pump station for capacity planning purposes. In effect, the screw pump mimics a variable frequency motor by increasing the pump rate as inflow increases. This also dampens fluctuations in wet well levels.

3.2.3 Sewersheds

Sewersheds are used in the model to define loads to the modeled system. A sewershed represent flow from individual parcels or groups or parcels tributary to a modeled manhole, called a "load manhole". The Alameda model consists of over 2,600 sewersheds (also called subcatchments). Typically, sewersheds are developed as an aggregate of unmolded portions of the sewer system, but because the entire Alameda network is modeled, these sewersheds are small, typically less than one acre in size. Thiessen polygons were used to delineate the contributing area around a load manhole and to assign each parcel (based on the location of its centroid) to a sewershed. The end result of this process was a link between the modeled sewer network and wastewater flows from individual parcels. In the model, each sewershed or subcatchment is assigned a wastewater load to the modeled network. Calculation of sewershed loads is discussed in Chapter 4.

3.2.4 Sewer Basins

Sewer basins are larger areas of the system that represent areas tributary to the EBMUD interceptor. Sewer basins for Alameda were originally delineated as part of the 1980s SSES. In the Alameda model, the basins are primarily used to group sewersheds into larger areas that are assumed to have similar infiltration/inflow (I/I) characteristics (discussed in more detail in Chapter 4). As part of this study, some modifications were made to EBMUD's basin boundaries to reflect the most up-to-date system configuration, and some basins were assigned new ID numbers to minimize confusion that had resulted from use of different sewer basin IDs in various documents in previous years. The Alameda sewer basins are shown in **Figure 3-3** and discussed further in Chapter 4.

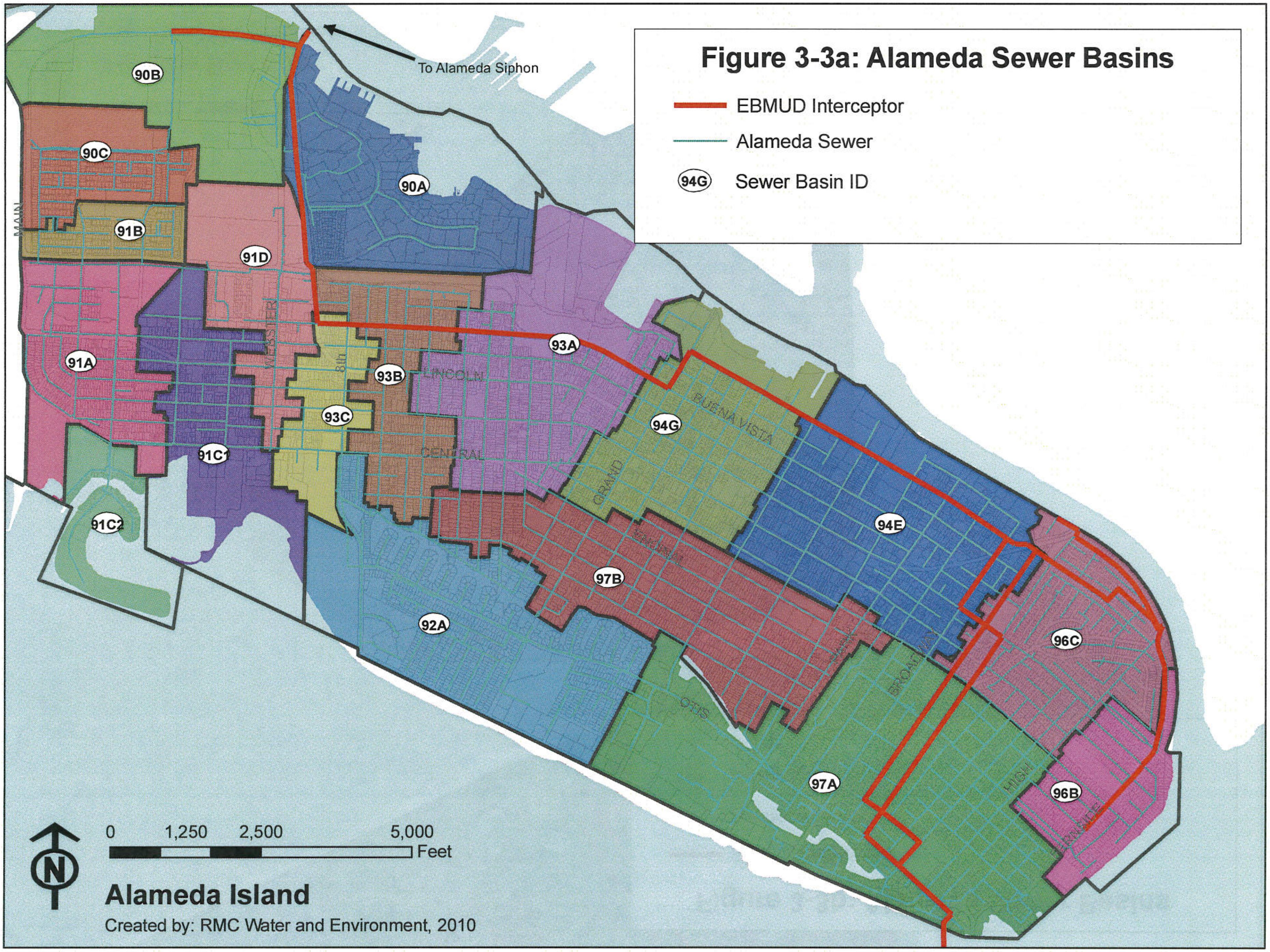
Figure 3-3a: Alameda Sewer Basins
EBMUD Interceptor
Alameda Sewer
94G
Sewer Basin ID
To Alameda Siphon
90A
90B
90C
91A
91B
91C1
91C2
91D
92A
93A
93B
93C
94E
94G
96B
96C
97A
97B
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
BUENA VISTA
OTIS
BROADWAY
HIGH
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

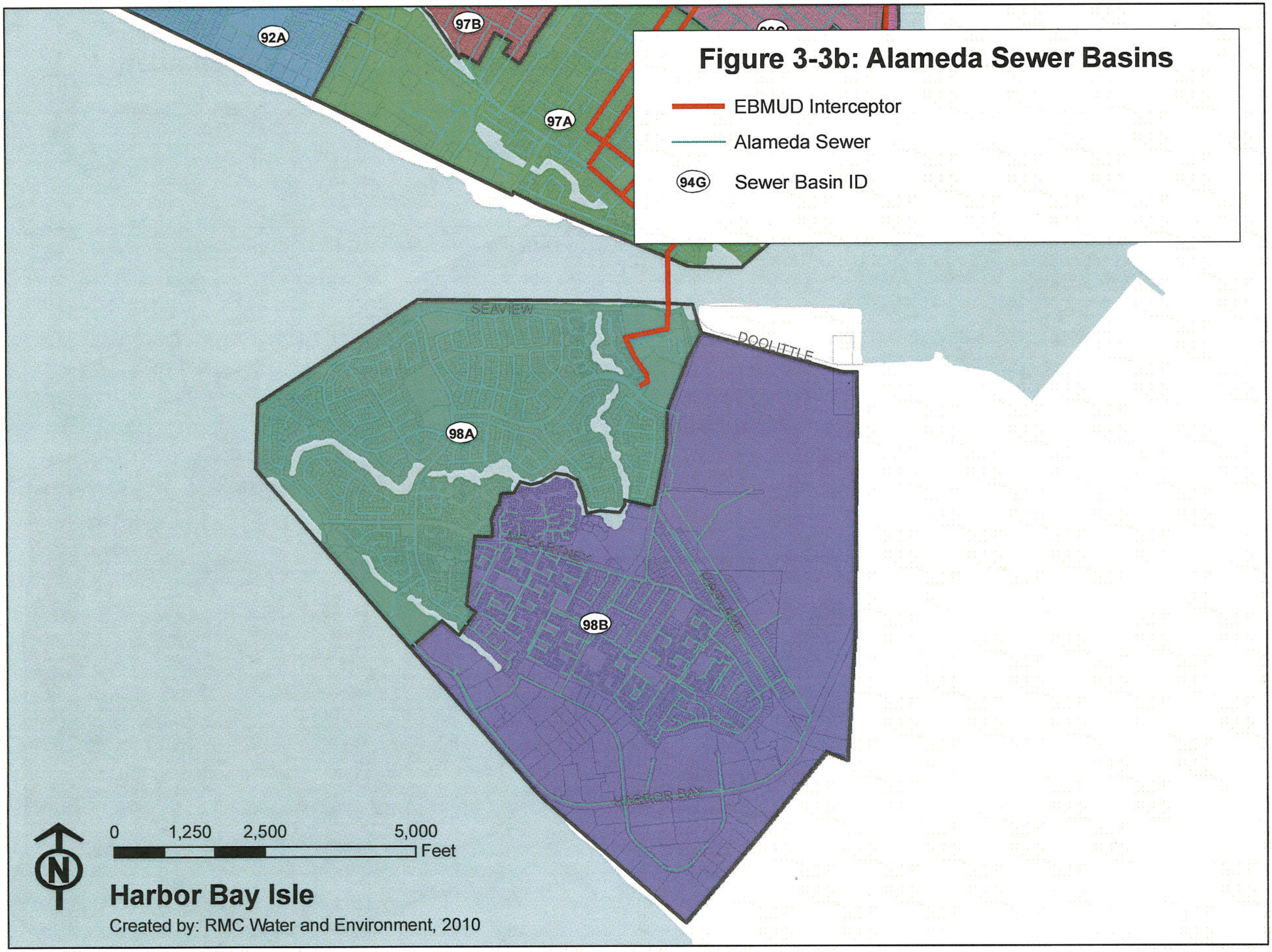

Figure 3-3b: Alameda Sewer Basins
EBMUD Interceptor
Alameda Sewer
94G
Sewer Basin ID
92A
97B
97A
98A
98B
SEAVIEW
DOOLITTLE
HARBOR BAY
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

Chapter 4 Existing and Future Wastewater Flows

This chapter describes different wastewater flow components and the development of wastewater flow estimates. Land use data provided the basis for estimating average base wastewater flows. Flow monitoring and rainfall data were then used to calibrate the sewer model for both dry and wet weather flow conditions.

4.1 Wastewater Flow Components

Wastewater flows typically include three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration/inflow (RDI/I). BWF represents the sanitary and process flow contributions from residential, commercial, institutional, and industrial users of the system. GWI is groundwater that infiltrates into the sewer through defects in pipes and manholes. GWI is typically seasonal in nature and remains relatively constant during specific periods of the year, although may not vary much on a seasonal basis in low-lying areas such as Alameda . RDI/I is storm water inflow and infiltration that enter the system in direct response to rainfall events. RDI/I can occur through direct connections such as holes in manhole covers or illegally connected roof leaders or area drains, or through defects in sewer pipes, manholes, and service laterals. RDI/I typically results in short term peak flows that recede quickly after the rainfall ends. Dry weather flow (DWF) consists of BWF plus GWI, while wet weather flow (WWF) adds the RDI/I component.

These three flow components are illustrated conceptually in **Figure 4-1**.

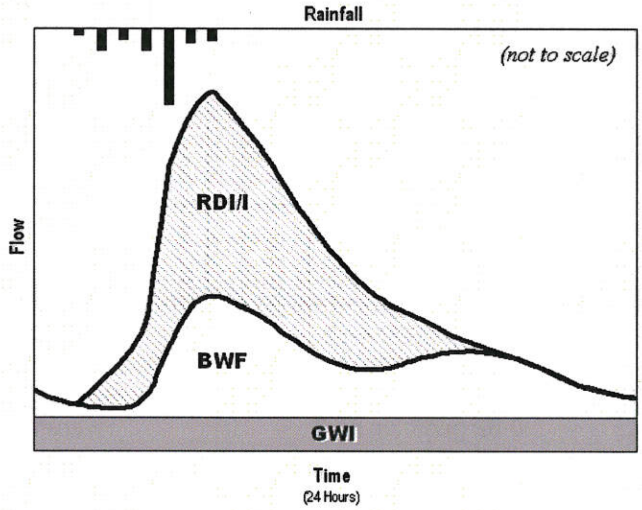

Figure 4-1: Wastewater Flow Components

4.2 Flow Estimating Methodology

As discussed in Section 3.2, sewersheds represent flow from a group of parcels tributary to a modeled manhole. All model loads, including BWF, GWI, and RDI/I are estimated at the sewershed level for input into the model. Flow monitoring data collected throughout the system are then compared to model flows at certain points throughout the system to refine the magnitude and timing of model loads. The following sub-sections describe the development and assumptions for each load component. Subsequent sections of this chapter describe the flow monitoring and model calibration, which contributed to the development and refinement of each flow component described below.

4.2.1 Base Wastewater Flows

Existing BWF entering the existing modeled system were estimated based on information obtained from the City's assessor parcel data including the number of dwelling units for residential parcels and square feet of building space for non-residential parcels.

Existing Residential Base Wastewater Flows

Both existing and future residential loads were determined by applying per-unit flow factors to the existing and future residential developments based on the number and type of dwelling units. "Units" are defined as single or multi-family dwelling units. The number of units per residential parcel was determined from parcel data provided by the City.

Per-unit flow rates used in the model are as follows:

- Single family = 240 gpd/unit
- Multi-family = 170 gpd/unit

These flow factors are similar to factors used for other Bay Area cities and were verified by calibration of the model to actual flow monitoring data. It should be noted that the unit flow rates likely include some amount of dry weather GWI that cannot easily be separated from the actual base wastewater (sanitary) flow component of dry weather flow.

Existing Non-Residential Base Wastewater Flows

Non-residential base wastewater flows were determined based on typical flow factors per building square footage for various types of business and other non-residential establishments (flows from schools were determined based on the number of students). These flow factors are given in **Table 4-1**.

Table 4-1: Non-Residential Flow Factors

Water Use Type	Flow Factor	Typical Building Type
High use	0.5 gpd/bldg sq ft	restaurants/car washes
Medium use	0.25 gpd/bldg sq ft	medical facilities/hotels/mixed uses
Low use	0.1 gpd/bldg sq ft	offices/general commercial
Very low use	0.02 gpd/bldg sq ft	warehouses/distribution
Schools	15 gpd/student	elementary/middle/high/college

Future Base Wastewater Flows

The future loads were determined similarly to existing loads, that is, by applying per-unit flow factors (i.e. gallons per day per dwelling unit) to the number of dwelling units for residential developments and by applying areal flow factors (i.e. gallons per day per building area) to non-residential buildings.

The location of specific future developments identified in EBMUD's WSMP along with the number of dwellings and the size and type of non-residential buildings were presented in Chapter 2. **Table 4-2** summarizes the estimated existing and future base wastewater flows and the percentage increase by basin due to anticipated new development and redevelopment.

Table 4-2: Estimated Base Wastewater Flows

Basin	Existing BWF (MGD)	Future BWF (MGD)	BWF Increase (MGD)	BWF Increase (%)
90A	0.23	0.35	0.12	50%
90B	0.09	0.45	0.36	397%
90C	0.10	0.13	0.04	40%
91A	0.25	0.27	0.02	8%
91B	0.05	0.09	0.03	62%
91C1	0.37	0.42	0.05	13%
91C2	0.11	0.12	0.01	5%
91D	0.35	0.37	0.03	8%
92A	0.46	0.46	0.00	0%
93A	0.40	0.60	0.20	49%
93B	0.28	0.28	0.00	1%
93C	0.14	0.14	0.01	4%
94E	0.55	0.87	0.32	58%
94G	0.40	0.45	0.05	12%
96B	0.19	0.19	0.00	2%
96C	0.30	0.31	0.01	2%
97A	0.97	1.01	0.03	4%
97B	0.70	0.72	0.02	3%
98A	0.68	0.84	0.16	24%
98B	0.66	1.14	0.49	74%
Total	**7.28**	**9.22**	**1.94**	**27%**

4.2.2 Diurnal Wastewater Profiles

The diurnal profile, or change in the base wastewater flow throughout the day, must also be defined in order to simulate time-varying flows in the model.

Different diurnal profiles were developed to represent residential areas, non-residential areas, mixed development, and business parks. For each of these types, two diurnal profiles were developed, one for weekdays and another for weekends. **Figure 4-2** through **Figure 4-5** show the various diurnal profiles used in the model. As can be seen from these graphs, residential areas show high flows in the morning and evening with weekends having a later and often slightly higher morning peak than weekdays. A typical non-residential profile was applied to retail and service establishments that tend to have steady flows throughout normal business hours, including weekends. Mixed developments consist of both non-residential and residential units and so the diurnal curve is a hybrid of those two profiles. Lastly, business parks have a similar flow pattern as non-residential, except that they have very low flows during the weekends. This profile was used primarily on Harbor Bay Isle for the Harbor Bay Business Park.

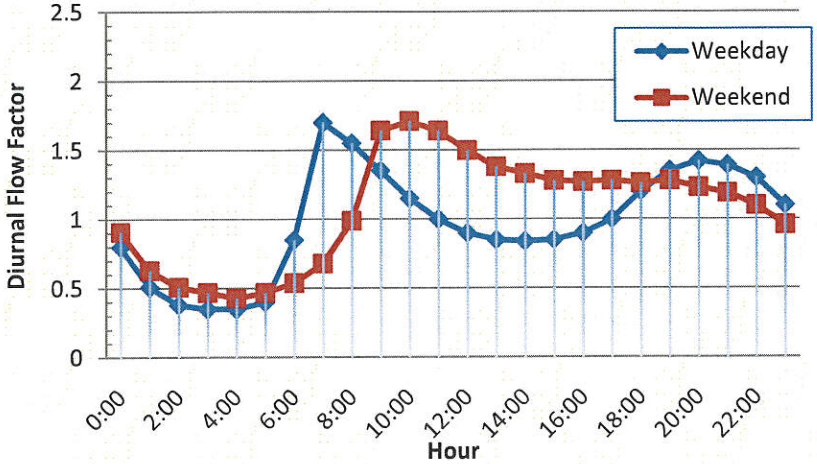

Figure 4-2: Residential Diurnal Profile

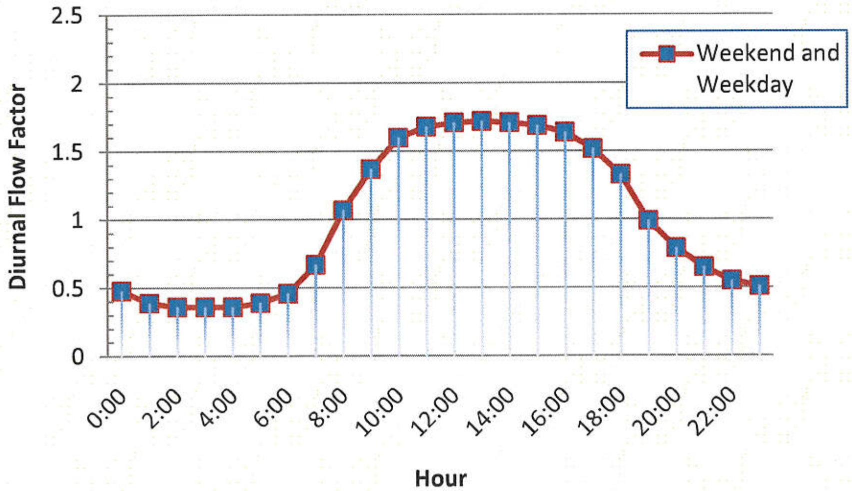

Figure 4-3: Non-Residential Diurnal Profile

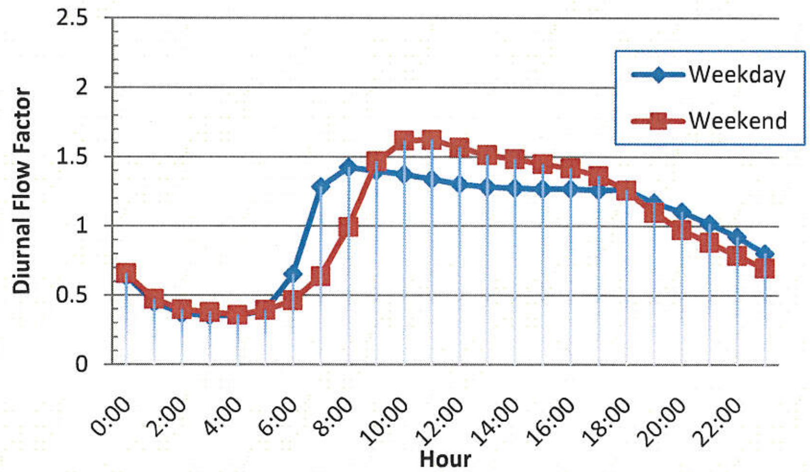

Figure 4-4: Mixed-Use Diurnal Profile

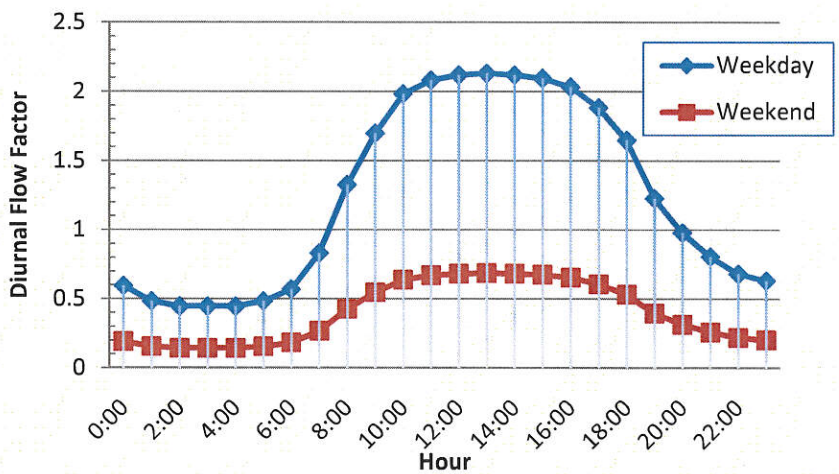

Figure 4-5: Business Park Diurnal Profile

4.2.3 Infiltration/Inflow

As discussed previously, infiltration and inflow (I/I) are terms used to describe extraneous groundwater and stormwater that enter into dedicated sanitary sewer systems. Note that in the context of this modeling analysis, groundwater infiltration (GWI) represents the incremental amount of infiltration (above the nominal amount assumed to be included in dry weather BWF), typically expressed on a unit areal basis (gpd per sewered acre) that would be expected during the wettest times of the year (i.e., late winter or early spring, after winter storms have saturated the ground).

Rainfall dependent infiltration and inflows (RDI/I) are defined by the magnitude, shape, and timing of the RDI/I response. The magnitude of the RDI/I response is typically described by the percentage of the rainfall volume (the "R value") that enters the sewer system within a specified drainage area. The RDI/I hydrograph shape is defined by separating the total RDI/I hydrograph volume into components, representing different response times to rainfall (fast, medium, and slow). Each of the components has a specific time to peak (T) and recession coefficient (K). The R component percentages and T and K values are applied to each hour of rainfall to generate a "synthetic hydrograph" that approximates the volume and shape of the hydrograph from an actual observed event. These parameters, when applied to a different rainfall pattern (e.g., a "design storm", as discussed later), can be used to estimate the RDI/I response to that particular rainfall event. **Figure 4-6** illustrates the three RDI/I components and the resulting total RDI/I hydrograph.

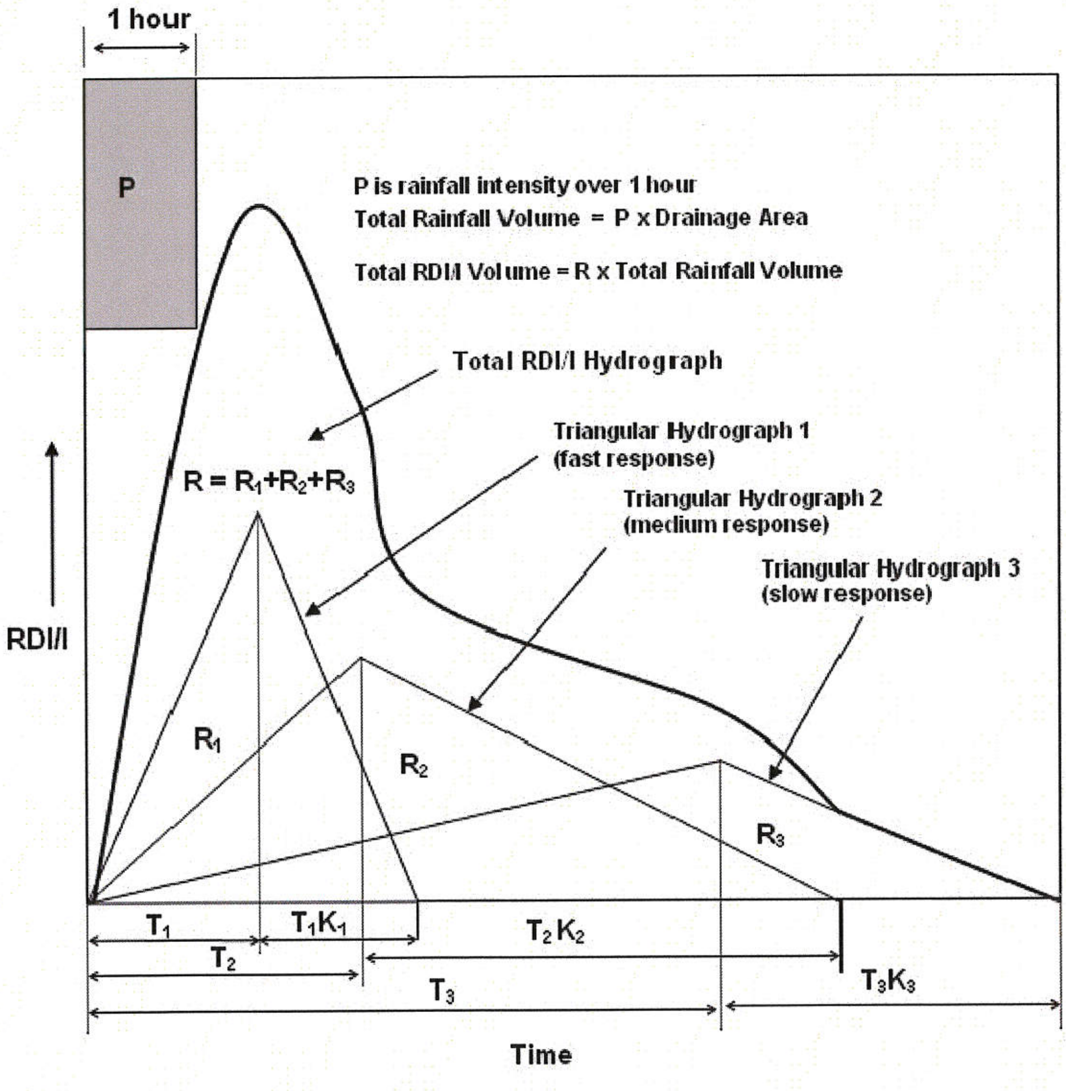

Figure 4-6: RDI/I Hydrograph Components

Existing Infiltration/Inflow

I/I flows for the existing system cannot be estimated by using "standard" factors as with BWF, since they are highly dependent on local soil and groundwater conditions and the physical condition of the sewer pipelines, manholes, and service laterals. Therefore, I/I estimates for the existing system must be developed based on actual flow monitoring data.

Preliminary GWI and RDI/I parameters were defined for the Alameda system based on analyses conducted by EBMUD as part of calibration of their interceptor system hydraulic model. These GWI and RDI/I rates were developed based on extensive analysis of flow monitoring and radar rainfall data collected during the 2005/06 wet weather season for EBMUD's Wet Weather Infrastructure Improvements Studies. The EBMUD model defines GWI and RDI/I parameters for each sewer basin. The parameters were modified for Alameda based on calibration of the Alameda system model and to reflect the much smaller loading areas (sewersheds versus basins) to the model, and also supplemented by information from other studies conducted specifically for the Alameda Landing area. The resulting GWI and RDI/I parameters are presented later in this chapter under the discussion of model calibration.

Future Infiltration/Inflow

Because I/I is dependent on the sewered or "contributing" area, future developments will result in some increase in I/I into the system. However, new development areas will likely have lower I/I responses than older areas of development because they will be served by sewers that are newer and better constructed. To account for potential increases in I/I while also considering lower I/I characteristics of new developments, the incremental increase in contributing area due to future developments was calculated based on 50 percent of current basin I/I rates. This assumption strikes a balance between accounting for future increases in I/I while not overstating the I/I contribution from sewers in new development areas.

4.3 Flow Monitoring

Flow monitoring data provided the basis for comparing and refining the model loads to better match actual flows measured in the field. Flow monitoring was not directly conducted for this study; however, flow data were available from previous monitoring efforts by EBMUD at several points along the interceptors and at EBMUD Pump Station C. Since the interceptor forms the 'backbone' of the Alameda sewer system, these monitoring locations provided information about tributary flows at several points throughout the system.

EBMUD conducted flow monitoring during the 2005/2006 and 2007/2008 wet weather seasons. The duration of monitoring and specific duration for these two monitoring studies are given in **Table 4-3**, and the locations are shown in **Figure 4-7**. A schematic of the flow monitoring locations is shown in **Figure 4-8.**

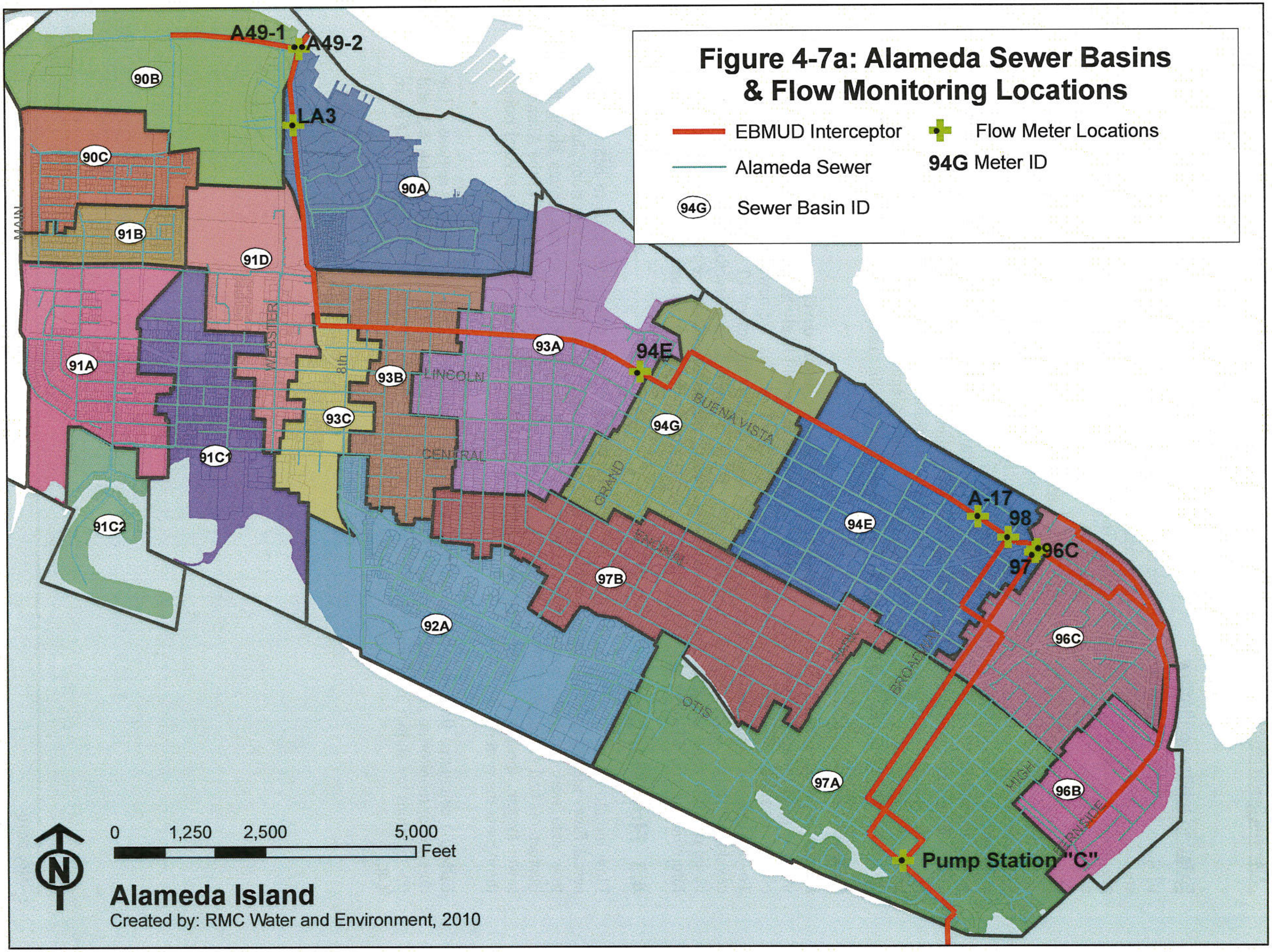

Figure 4-7a: Alameda Sewer Basins & Flow Monitoring Locations
EBMUD Interceptor
Flow Meter Locations
Alameda Sewer
94G Meter ID
94G
Sewer Basin ID
A49-1
A49-2
LA3
94E
A-17
98
96C
97
Pump Station "C"
90A
90B
90C
91A
91B
91C1
91C2
91D
92A
93A
93B
93C
94E
94G
96B
96C
97A
97B
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
BUENA VISTA
OTIS
BROADWAY
HIGH
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

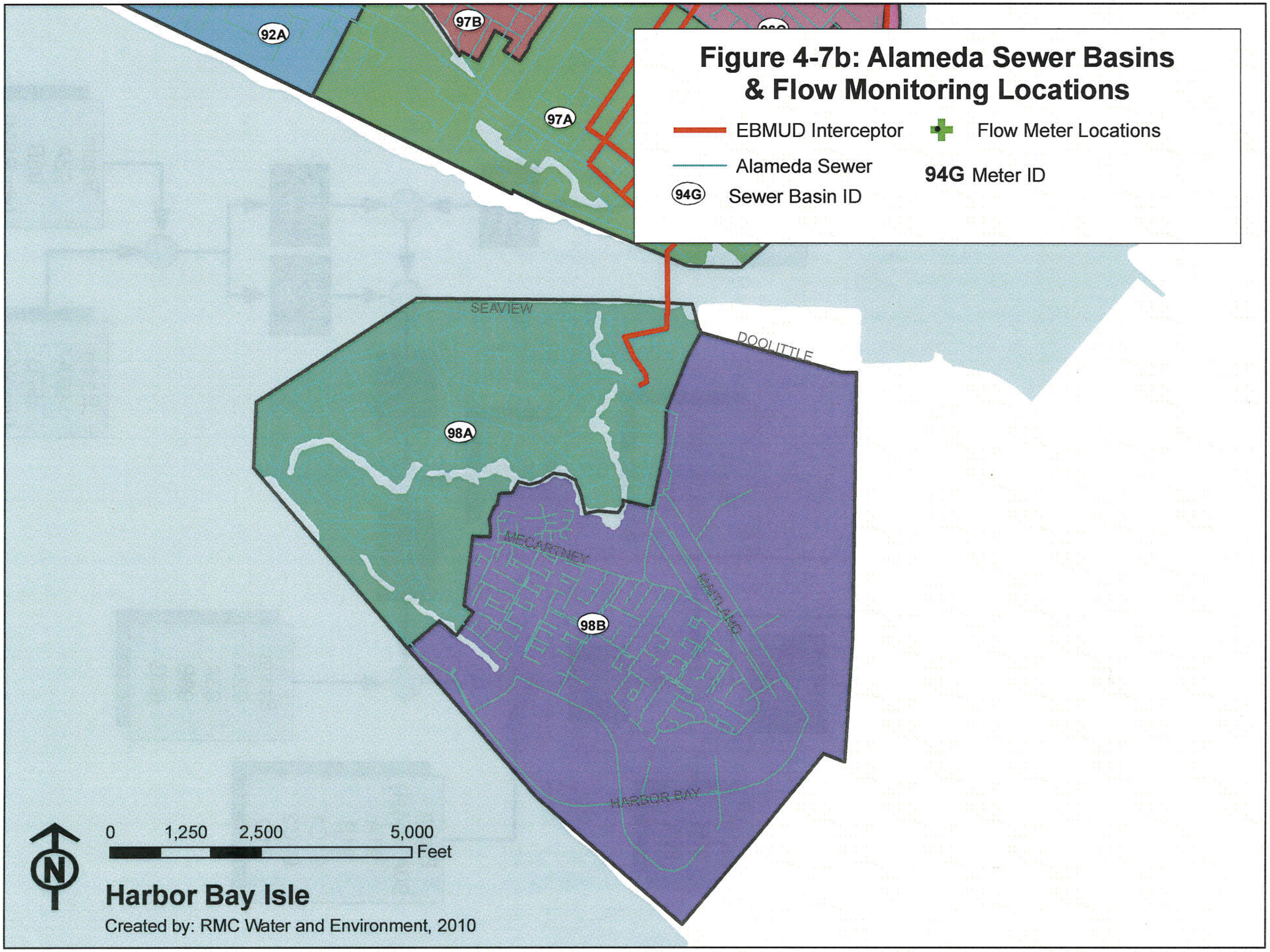
Figure 4-7b: Alameda Sewer Basins
& Flow Monitoring Locations
EBMUD Interceptor
Flow Meter Locations
Alameda Sewer
94G Meter ID
94G
Sewer Basin ID
92A
97B
97A
98A
98B
SEAVIEW
DOOLITTLE
MECARTNEY
MAITLAND
HARBOR BAY
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

Figure 4-8: Schematic of Alameda Interceptor, Basins, and Flow Meters

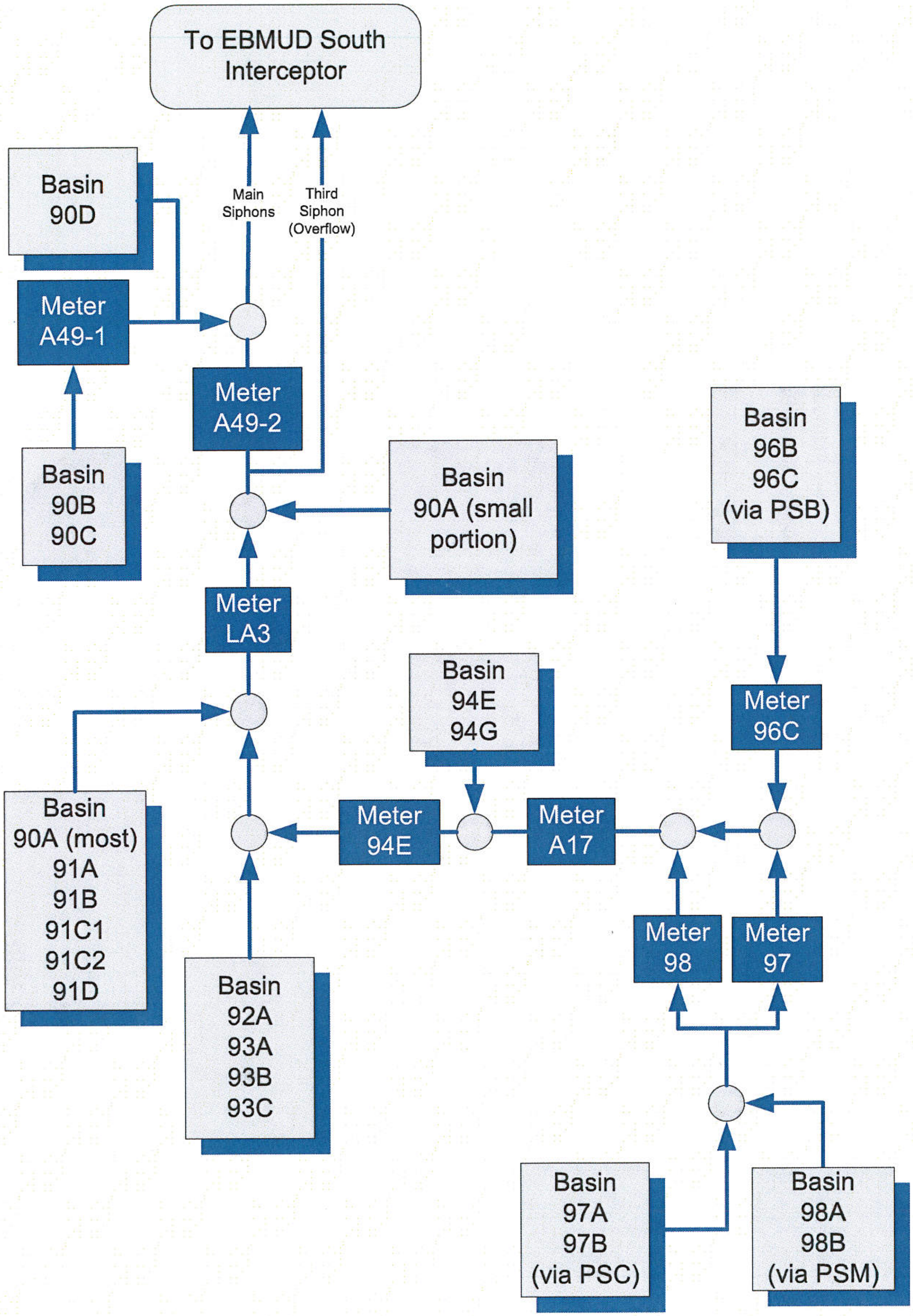

Table 4-3: Flow Monitoring Locations

Monitoring Period		Meter ID	Location/Description
Start Date	End Date		
January 21, 2008	April 22, 2008	A49-1 A49-2 A17	Mariner Square Dr. & Mitchell Ave. (24" Mitchell line) Mariner Square Dr. & Mitchell Ave. (60" interceptor) Clement Ave. at Everett St. (42" interceptor)
January 27, 2006	April 17, 2006	LA3 94E 96C 97 98	Mariner Square Dr. near Tynan Ave. (60" interceptor) Buena Vista Ave. at Paru St. (54" interceptor) Fernside Blvd. at Pearl St. (discharge of PS B Force Main) Pearl St. at Fernside Blvd. (21" interceptor) Broadway at Clement Ave. (24" interceptor)
January 2005	September 2006	PSC	EBMUD Pump Station C

To supplement the EBMUD monitoring data, a limited monitoring program was conducted at four Alameda pump stations (Harbor Bay Parkway 1, BFI, Marina, and Park Otis) by RMC's subconsultant, E2. The monitoring consisted of pump runtime recorders installed for several weeks during October and November 2009. The runtime data were converted to flow rates based on results of draw down testing. The data obtained from the pump recorders were used to confirm dry weather flow calibration for the areas tributary to these pump stations.

4.4 Model Flow Calibration

Dry and wet weather model flows were calibrated by comparing model results to metered flows during representative dry and wet periods. The following sub-sections describe the calibration of dry and wet weather flows.

4.4.1 Dry Weather Flow Calibration

Dry weather flow calibration involves verifying that the unit BWF rates, 24-hour diurnal flow profiles, and GWI rates in the model result in a reasonable match of modeled flows to monitored flows for a typical dry period. Dry weather flow calibration periods in February 2006 and April 2008 were selected from the EBMUD flow monitoring data, plus the October/November 2009 period for the four monitored pump stations. Calibration periods were also selected to have minimal preceding rainfall and to avoid major holidays.

Based on the dry weather calibration, the preliminary GWI rates from the EBMUD analyses were adjusted for the monitored basins. The resulting rates that yielded the best match to measured dry weather flows were:

- 300 gpd/acre for Basins 90A, 90C, 91B, 98A, and 98B,
- 1,000 gpd/acre for Basins 97A and 97B
- 2,600 gpd/acre for Basin 90B[3]
- 1,500 gpd/acre for all other areas

Note that the low GWI rates used for Basins 90A, 90C, and 91B were not determined directly based on flow monitoring but were chosen primarily because these are relatively new areas of development and

[3] Based on estimate of tidal component of flow as presented in Alameda Landing Wastewater Technical Memorandum No. 1.7, BKF Engineers, June 3, 2009.

would be expected to have lower rates than older areas of the City due to more watertight pipe materials and better quality construction.

Overall, the dry weather calibration yielded reasonably good results. **Figure 4-9** shows an example comparison between model results and meter data for EBMUD meter LA3. This particular flow meter accounts for most of the flow from the City's service area. In this figure, the blue line shows the meter data, and the red line shows the model results.

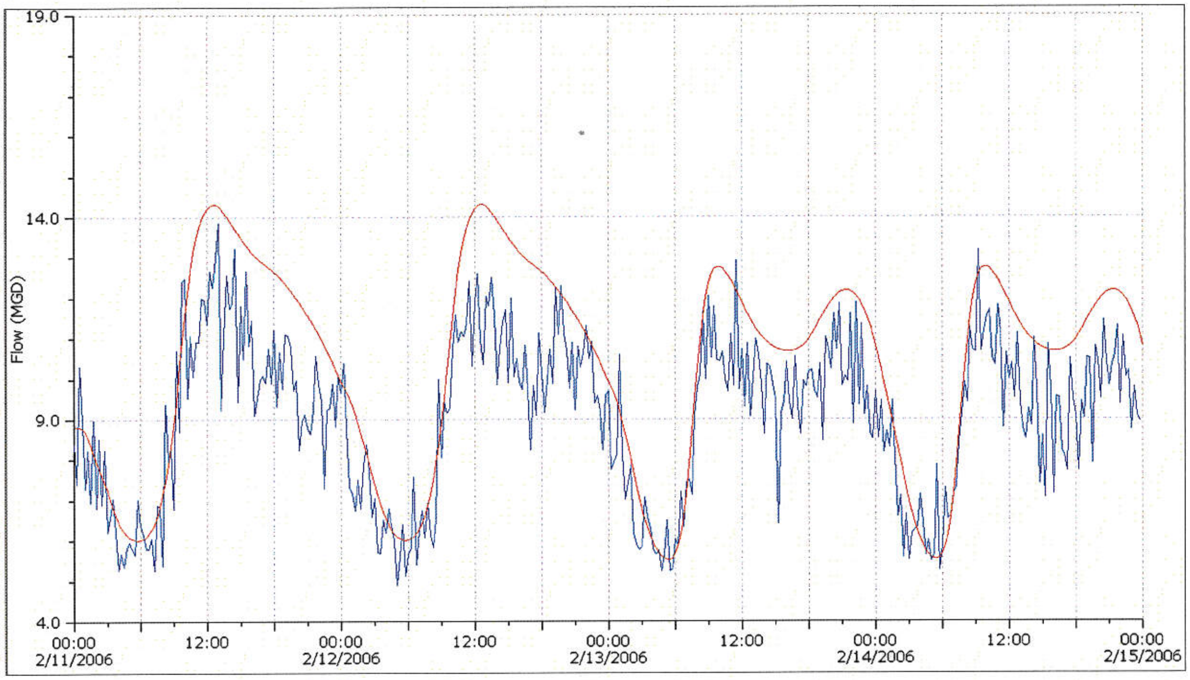

Figure 4-9: Comparison of Modeled to Measured Dry Weather Flows (Meter LA3)

4.5 Wet Weather Flow Calibration

As previously mentioned, in order to model wet weather flows, parameters (RTK values) must be defined to describe the RDI/I response to rainfall. These values were determined by first calibrating to flows from the EBMUD interceptor model for EBMUD's design storm event (the design storm is discussed further in Chapter 5). The objective of this approach is to ensure that the flows in the Alameda model will be consistent with EBMUD's interceptor system planning. Calibration was then confirmed by running the Alameda model for observed storm events from the 2006 monitoring period. Each of these steps is described in the following subsections.

4.5.1 Calibration to the EBMUD Model

The Alameda model was calibrated to have a similar response as the EBMUD model when run under the design storm. The design storm is an actual historical rainfall event that was originally selected for the EBMUD system during EBMUD's I/I and Wet Weather Studies in the 1980s. In the EBMUD model, each basin has a rainfall multiplier which is applied to the design storm rainfall at Oakland Airport. Since Alameda is in close proximity to the airport, the multipliers are close to 1.0, ranging from 0.95 to 0.99. For the current Alameda model, it was assumed that using different multipliers would yield little or no additional value to the model and so a constant, average rainfall multiplier of 0.97 was used for all of Alameda.

The EBMUD model has RTK parameters for each basin. The "sewered area" to which these rates are applied is the *gross* area of the basins from GIS, which does not necessarily match the sum of the subcatchment contributing areas in the Alameda model because the subcatchment areas represent the *net* acreage (lot sizes) of developed tributary parcels. Furthermore, as part of current Alameda modeling study, these basin boundaries have been refined. For these reasons, RTK parameters from the EBMUD model could not be directly used in the current model. Furthermore, initial wet weather calibration runs indicated that the response hydrographs based on the EBMUD model RTK parameters were not "peaky" enough. This is expected because the EBMUD model parameters are applied to larger basins and are not routed through City sewers before discharge to the EBMUD interceptor, while in the current model the parameters are applied to very small sewersheds. To account for this issue the following adjustments were made:

- The T1, K1, and T2 parameters were reduced to give the hydropgraphs a quicker, steeper response (i.e. more "peaky"). This resulted in a relatively good fit to the EBMUD model flow hydrograph shapes.
- R values were increased to account for the differences in EBMUD basin areas versus the contributing areas in the current model. Specifically, the R values were increased by 70 percent (multiplier of 1.7) to account for the fact that the contributing areas are about 60 percent of the EBMUD basin areas. This adjustment ensures that the resulting RDI/I *volumes* in both models are approximately the same.

When these modified RTK parameters were used in the current Alameda model, the result was a good agreement in response hydrographs for the design storm between the Alameda model and the EBMUD model. The final hydrograph parameters are shown in **Table 4-4.** Comparisons were made for predicted flows under the design storm for discharges from Pump Stations M, C, and B, and two additional locations further downstream along the main interceptor pipeline. An example comparison is shown in **Figure 4-10** (this graph compares flows along the EBMUD interceptor at the intersection of Clement and Oak Street). In this figure, the blue line shows the EBMUD model results and the red line shows the Alameda model results.

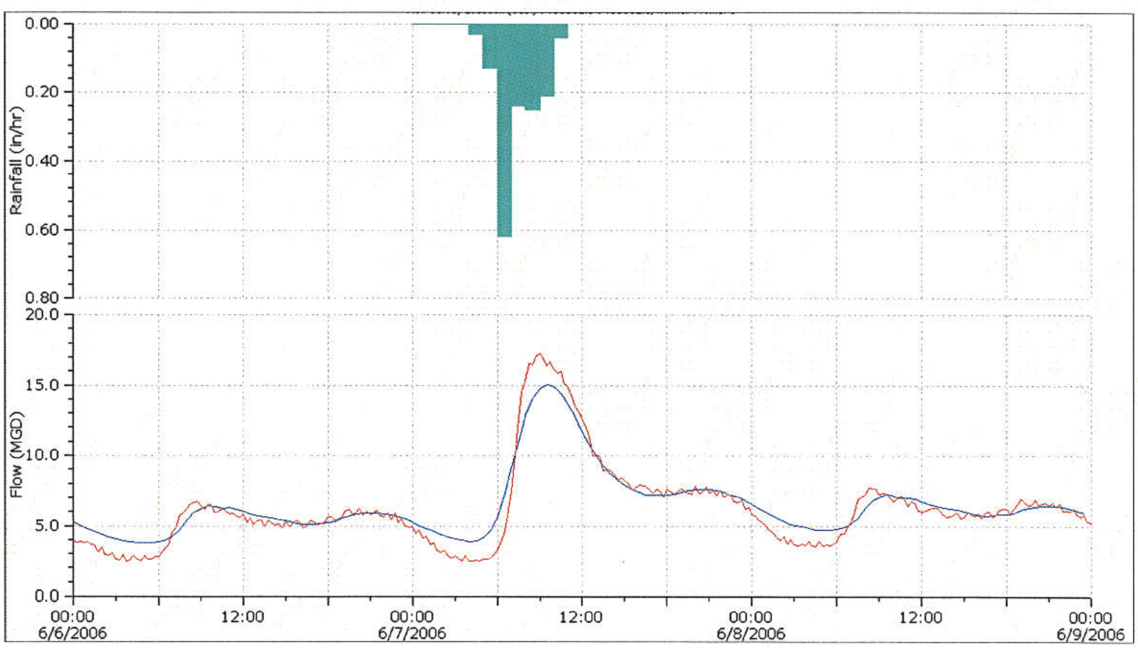

Figure 4-10: EBMUD Model Verses Alameda Model, Design Storm

Table 4-4: Calibrated RTK Parameters

Basin (RTK Hydrograph ID)	R1 (%)	T1 (hours)	K1	R2 (%)	T2 (hours)	K2	R3 (%)	T3 (hours)	K3
90A	0.3	1	2	0.3	3	3	0.3	12	3
90B[1]	1.2	2	2	2.0	3	4	0.0	12	3
90C	0.9	1	2	0.5	3	3	0.3	12	3
90D	0.9	1	2	0.5	3	3	0.3	12	3
91A	0.9	1	2	0.5	3	3	0.3	12	3
91B	0.9	1	2	0.5	3	3	0.3	12	3
91C1	0.9	1	2	0.5	3	3	0.3	12	3
91C2	0.9	1	2	0.5	3	3	0.3	12	3
91D	0.9	1	2	0.5	3	3	0.3	12	3
92A	0.9	1	2	0.5	3	3	0.3	12	3
93A	0.9	1	2	0.5	3	3	0.3	12	3
93B	0.9	1	2	0.5	3	3	0.3	12	3
93C	0.9	1	2	0.5	3	3	0.3	12	3
94E	0.9	1	2	0.5	3	3	0.3	12	3
94G	0.9	1	2	0.5	3	3	0.3	12	3
96B	7.2	1	2	6.7	3	3	12.0	15	3
96C	7.2	1	2	6.7	3	3	12.0	15	3
97A	2.5	1	2	1.4	3	3	2.0	15	3
97B	2.5	1	2	1.4	3	3	2.0	15	3
98A	1.7	1	2	0.0	3	3	0.0	12	3
98B	1.7	1	2	0.0	3	3	0.0	12	3

1) Parameters based on RDI/I analysis conducted by Talavera & Richardson of flow monitoring data collected in 2004 for sites on EBMUD Mitchell line (see Appendix C of Alameda Landing Wastewater Technical Memorandum No. 1.7, BKF Engineers, June 3, 2009).

4.5.2 Calibration Confirmation

Wet weather flow calibration was further confirmed by comparing flows predicted by the Alameda model to measured flows during the 2006 flow monitoring period. Specifically two storm events were used: 1) March 5-6 and March 24–25, 2006. **Figure 4-11** and **Figure 4-12** show examples of model results compared to metered flows at Meter LA3. In these figures, the blue line represents meter data, and the red line represents the model results.

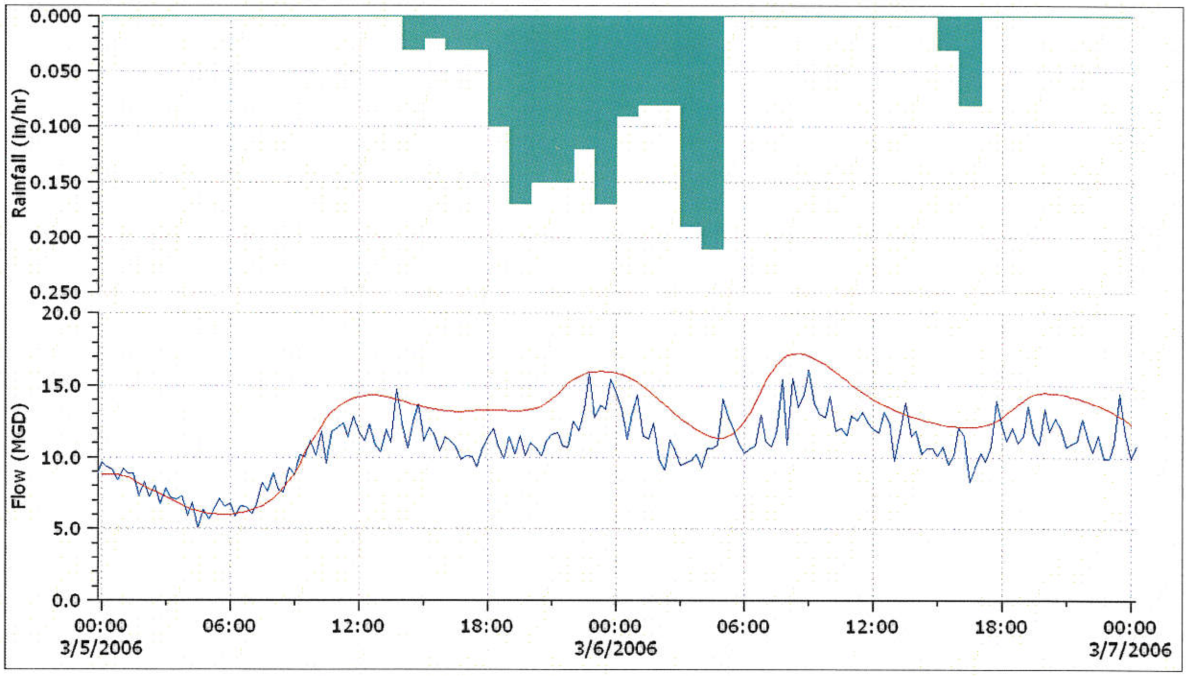

Figure 4-11: Modeled vs. Measured Flows, March 5 - 6, 2006 Storm (Meter LA3)

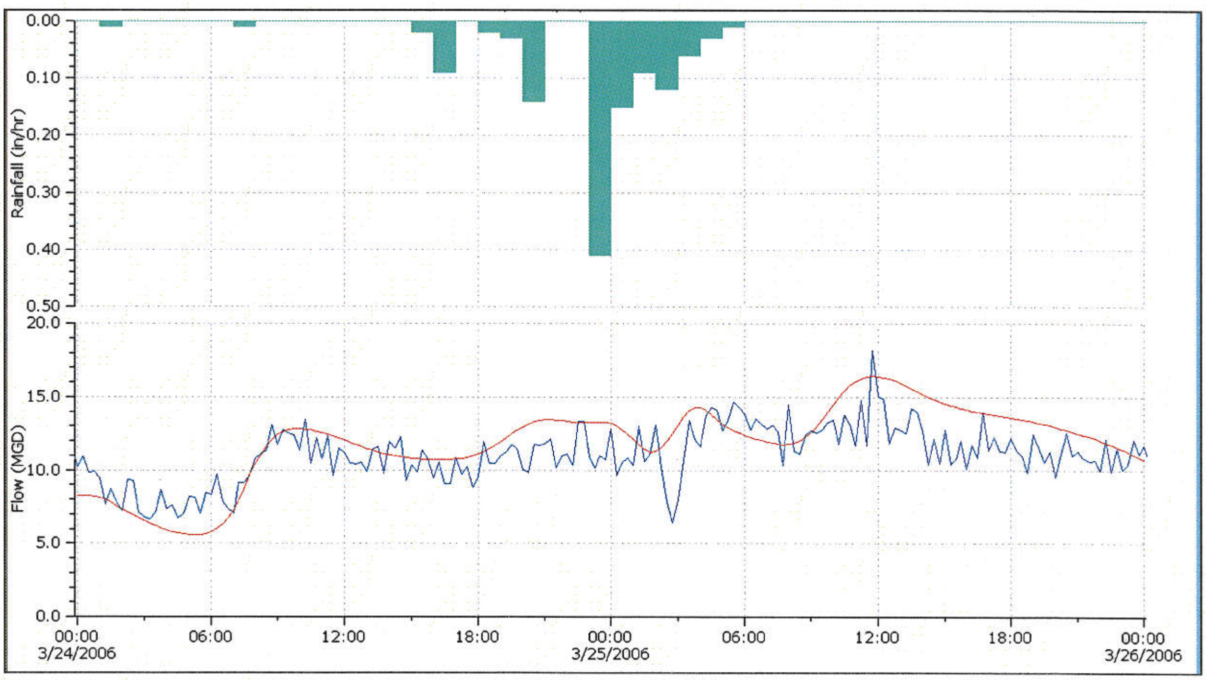

Figure 4-12: Modeled vs. Measured Flows, March 24 - 25, 2006 Storm (Meter LA3)

Chapter 5 Sewer System Capacity Analysis

The hydraulic capacity of the sewer system must be sufficient to convey peak wastewater flows. The calibrated hydraulic model was used to generate design flows and to identify capacity deficiencies. This chapter presents the criteria used to evaluate capacity and the results of the capacity analysis performed for both dry and wet weather flows under existing and future conditions.

5.1 Design Flow Criteria

The model calibration determined dry and wet weather flow parameters that represent existing flow conditions. These parameters were reviewed to determine their applicability for use in identifying existing and future capacity deficiencies and for sizing sewer improvements and future sewers. Based on this review, the following design flow criteria were adopted for use in the capacity analysis:

- The calibrated unit BWF rates were used for both existing and future conditions. This assumes that there will be no significant reductions (e.g., from water conservation) or increases (e.g., from more intense water use) in the rates in the future.
- The same calibrated GWI and RDI/I parameters for the existing sewered area were applied to generate flows throughout the system under both existing and future conditions[1]. This assumes that there will be no significant reductions (e.g., from rehabilitation or replacement of older sewers), or increases (e.g., from sewer deterioration) in I/I in the future. Note that this may be a conservative assumption for some areas, as the City plans to continue rehabilitation of the sewer system on an ongoing basis, which may result in reductions in I/I in some areas.
- A design rainfall event must be applied in the model to determine design peak wet weather flows. The design storm used for this analysis is the EBMUD design event, as discussed previously. This storm is equivalent to a 5-year rainfall event assumed to fall under saturated soil conditions, (i.e., maximum GWI and RDI/I response). The timing of the design storm also affects the resulting peak wastewater flows. If the design storm is timed to cause peak RDI/I at about the same time as peak base wastewater flow ("peak-on-peak"), the total peak wet weather flow will be higher than if the design storm occurs at another time of day. Timing the storm to produce peak-on-peak results is generally thought to create a return period in the peak wastewater flow that is greater than the return period of the design rainfall event itself (a 5-year rainfall event occurring at the same time as peak base wastewater flow would occur less often than once every five years).

  To be conservative, this analysis timed the design storm to occur just before the peak base wastewater flow period. The peak base wastewater flow occurs at different times for business parks and commercial areas than it does for residential areas. Business parks and commercial areas have a diurnal pattern that peaks at about mid-day (and for business parks, there is a significant difference between weekday and weekend flows), while residential and mixed-used areas tend to peak in the morning at about 7 a.m. on weekdays and, at a slightly higher level, at about 10 a.m. on weekends. Therefore, in order to model "peak on peak" conditions for these different diurnal patterns, the design storm was applied separately under two scenarios:

  1. Peak rainfall intensity occurring at 8 a.m. on a weekend day
  2. Peak rainfall intensity occurring at 11 a.m. on weekday

  The design storm rainfall, along with corresponding diurnal patterns, are shown in **Figure 5-1** and **Figure 5-2.** The largest resulting peak flow in a given sewer component (e.g., pipe or pump station) from either of these two scenarios was assumed to be the "design peak wet weather flow" and was

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[1] I/I rates from newly developed areas were assumed to be half of I/I rates in the surrounding older areas.

used to determine capacity requirements and identify capacity deficiencies and needed sewer improvements.

- When assessing pipe deficiencies, it was assumed that all pumps were retrofitted with sufficient capacity. This results in higher peak flows, especially at downstream pipes, because sewage is not being held-up at the pump stations. Although this assumption results in more conservative flow characteristics, it is a likely representation of future flow conditions, assuming the City plans on upgrading its pumps.

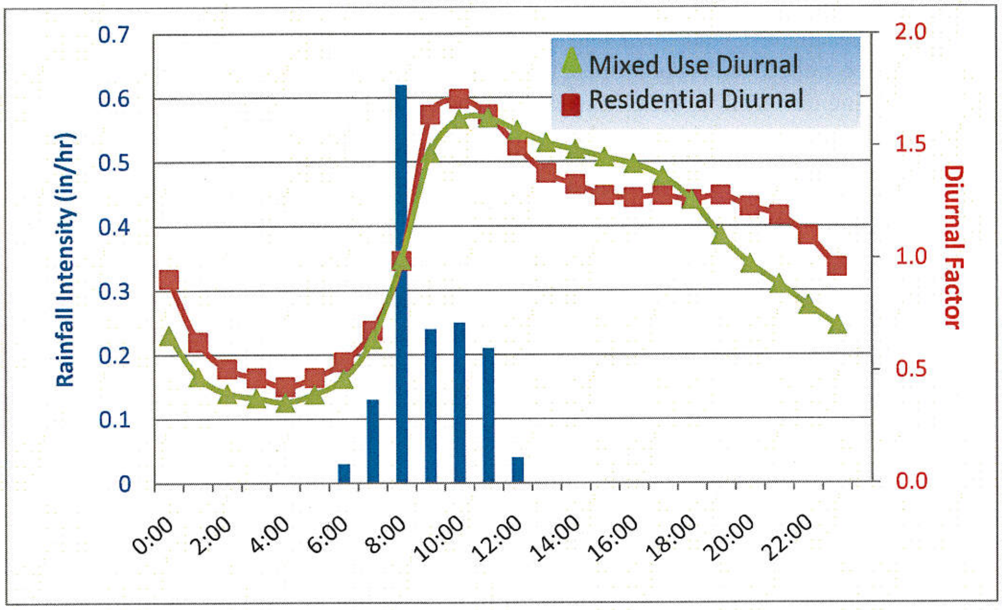


Figure 5-1: Design Storm Occurring on Weekend

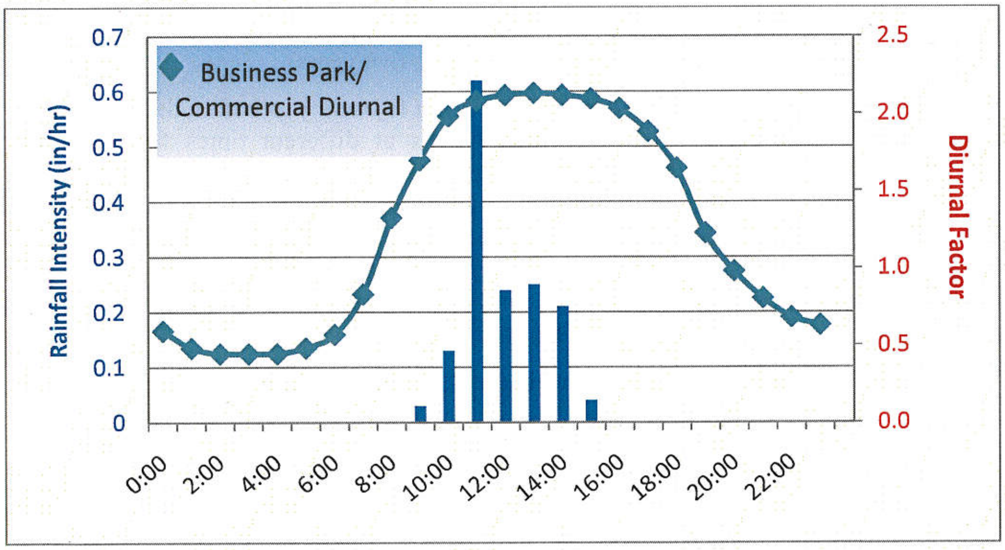


Figure 5-2: Design Storm Occurring on Weekday

5.2 Gravity Sewer Pipe Capacity Analysis

This section describes the criteria used to identify gravity pipe deficiencies and the results of the hydraulic modeling analysis.

5.2.1 Pipe Capacity Criteria

Capacity deficiencies requiring improvements projects were identified based on model-predicted pipe surcharge. Surcharge occurs when the flow depth-to-diameter ratio (d/D) of a given pipe is greater than one, indicating that the water surface (hydraulic grade line) is higher than the crown of the pipe. Note that surcharging does not necessarily indicate a capacity restriction at that particular location, as flows can back up due to a downstream capacity deficient area and cause upstream surcharging due to backwater.

The criteria used for identifying capacity deficiencies are as follows:

- undersized pipe causes more than a foot of surcharge during PWWF (or any amount of surcharge during PDWF); and
- surcharge causes water level to reach within 6 feet of the manhole rim

5.2.2 Pipe Capacity Results

The calibrated model was run for dry weather flows under both existing and future scenarios. **Figure 5-3** and **Figure 5-4** present the maximum d/D under existing and future peak dry weather flow (PDWF) conditions. It is important to note that backup due to insufficient pump station capacities are not shown in these figures. As can be seen in these figures, flow depths in most of the system are less than half-full under PDWF.

The calibrated model was also run with the design storm for both the existing and future scenarios. **Figure 5-5** and **Figure 5-6** present the d/D results for existing and future PWWF. As seen in the figures, the model indicates that surcharging would occur in some areas of the system under a design storm condition.

However, as noted previously, surcharging does not necessarily indicate a capacity limitation at that particular location, as flows can back up due to a downstream capacity deficient area and cause upstream surcharging due to backwater. These effects were considered during project development; therefore, projects were not defined for pipes that were surcharged only due to backwater conditions or when surcharge was minor (less than 1 foot), as was the case for many of the areas of potential surcharge indicated in the figures.

The main area of existing capacity deficiencies indicated by the modeling was in the pipes upstream of the Eighth & Taylor Pump Station. Areas of future capacity deficiencies included Walnut Street north of Clement Avenue, Southshore Shopping Center, and areas within and downstream of the Harbor Bay Business Park. Improvements to address these areas of capacity deficiencies are presented in Chapter 6.

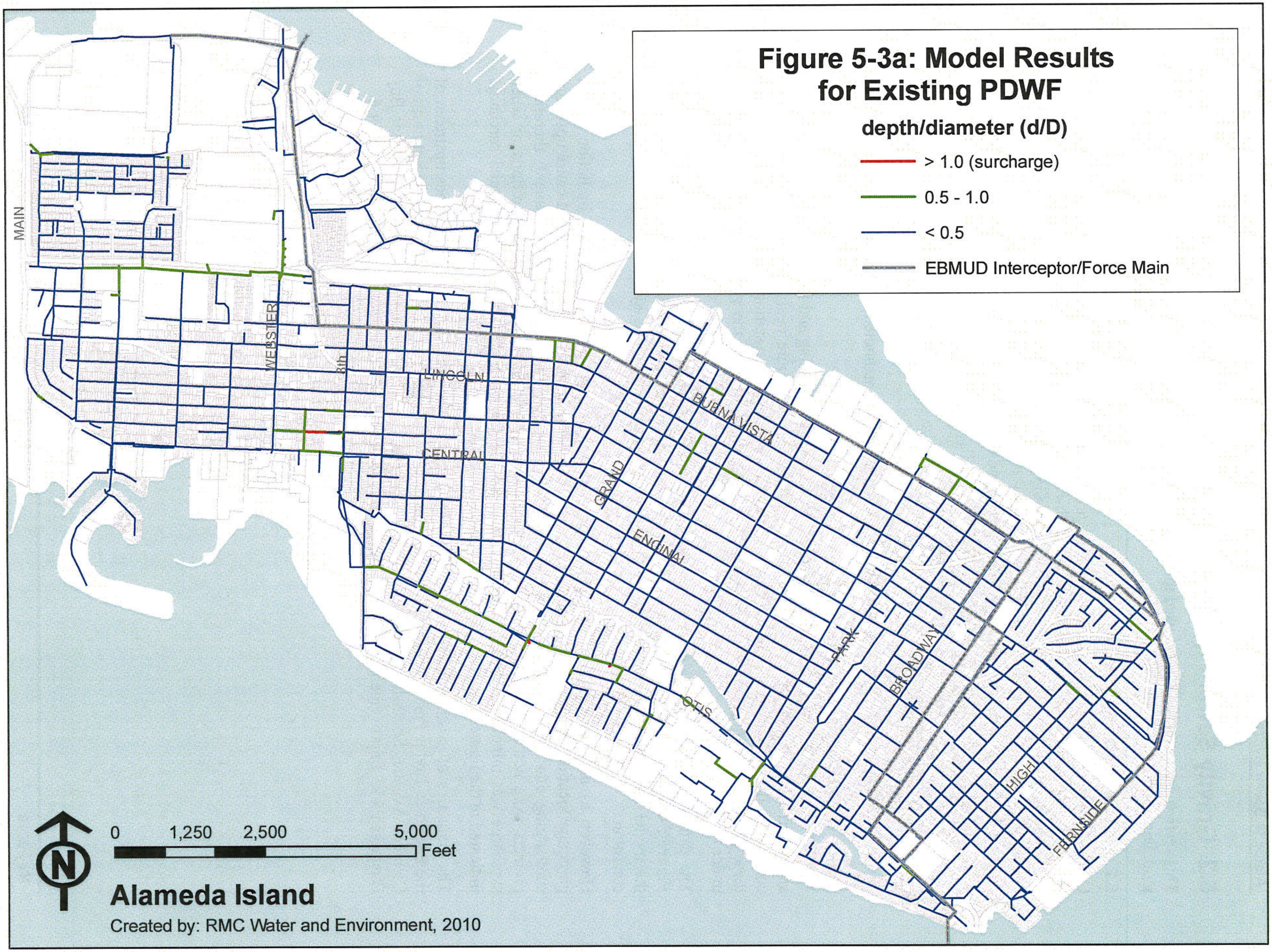


Figure 5-3a: Model Results
for Existing PDWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
ENCINAL
BUENA VISTA
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

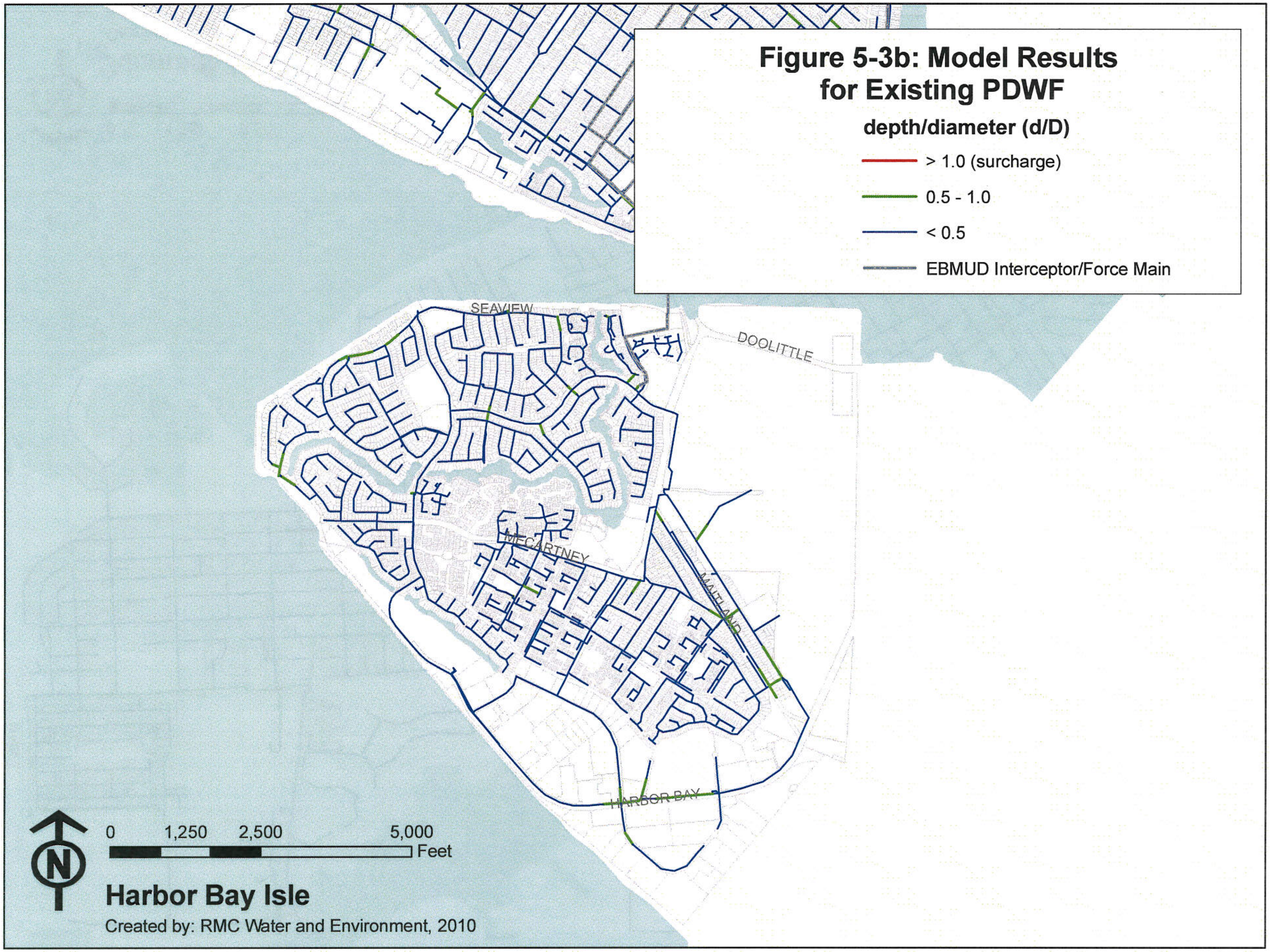


Figure 5-3b: Model Results
for Existing PDWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
SEAVIEW
DOOLITTLE
MCCARTNEY
MAITLAND
HARBOR BAY
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

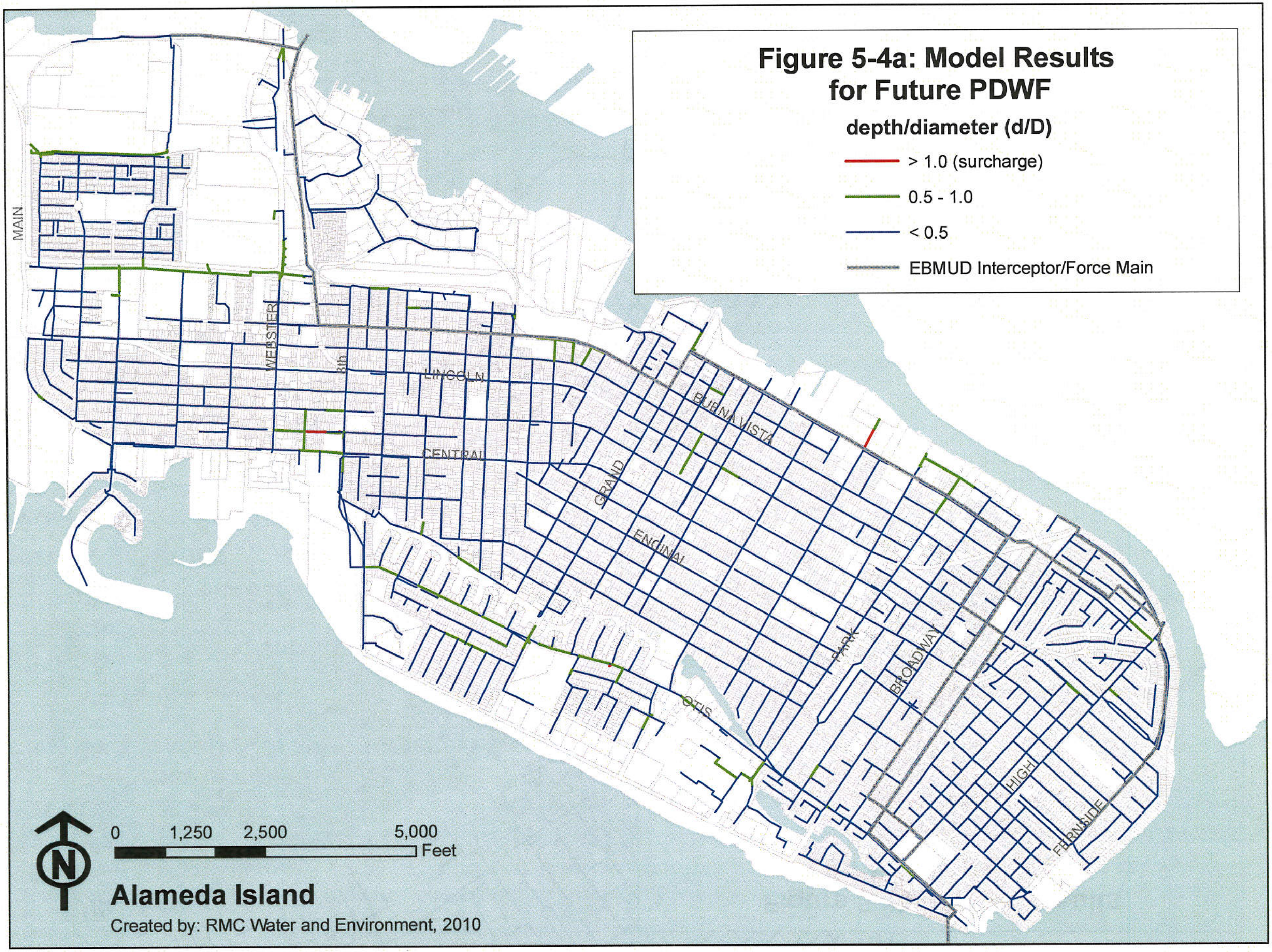


Figure 5-4a: Model Results for Future PDWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

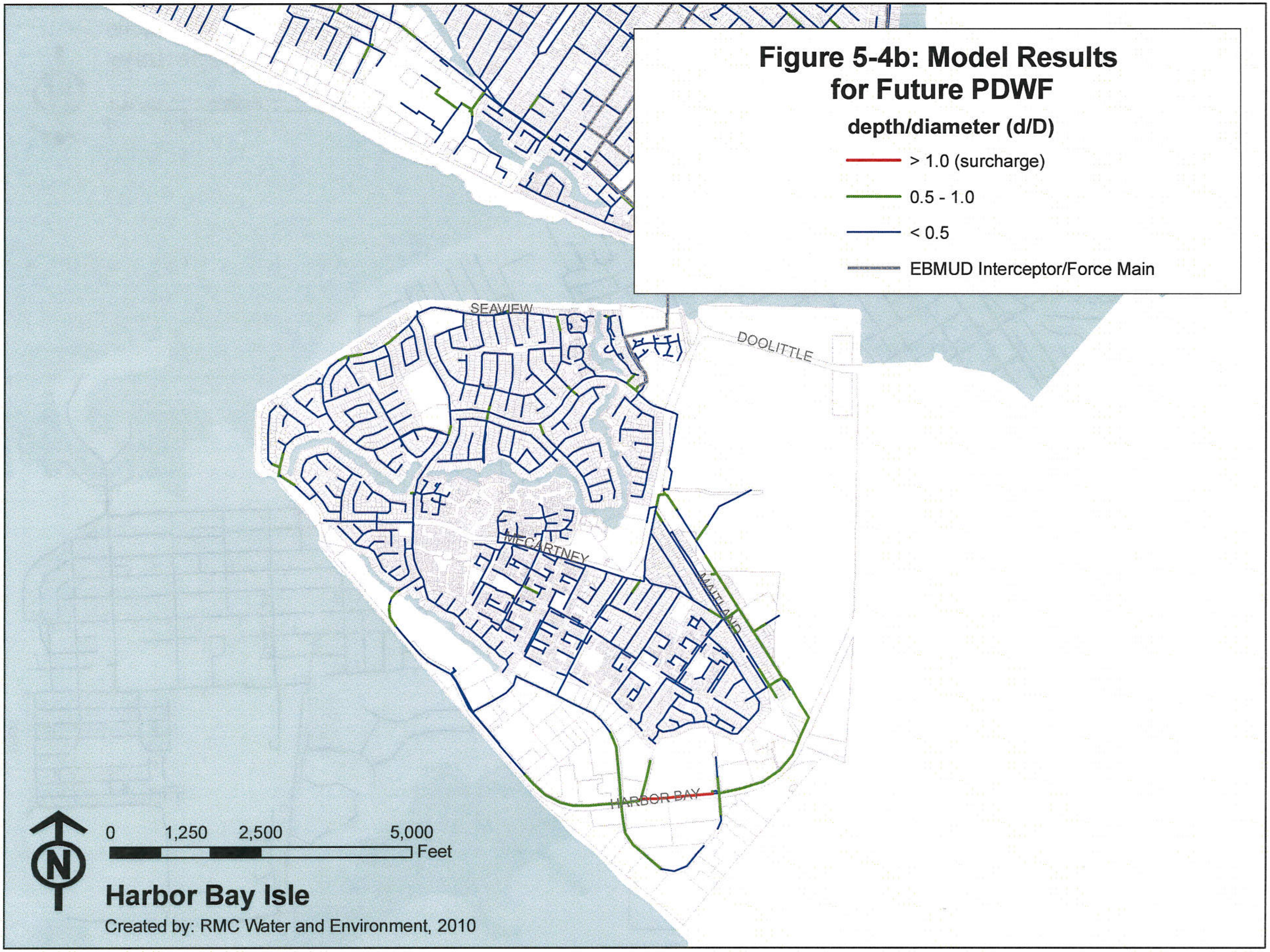


Figure 5-4b: Model Results
for Future PDWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
SEAVIEW
DOOLITTLE
MCCARTNEY
MAITLAND
HARBOR BAY
N
0
1,250
2,500
5,000
Feet
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

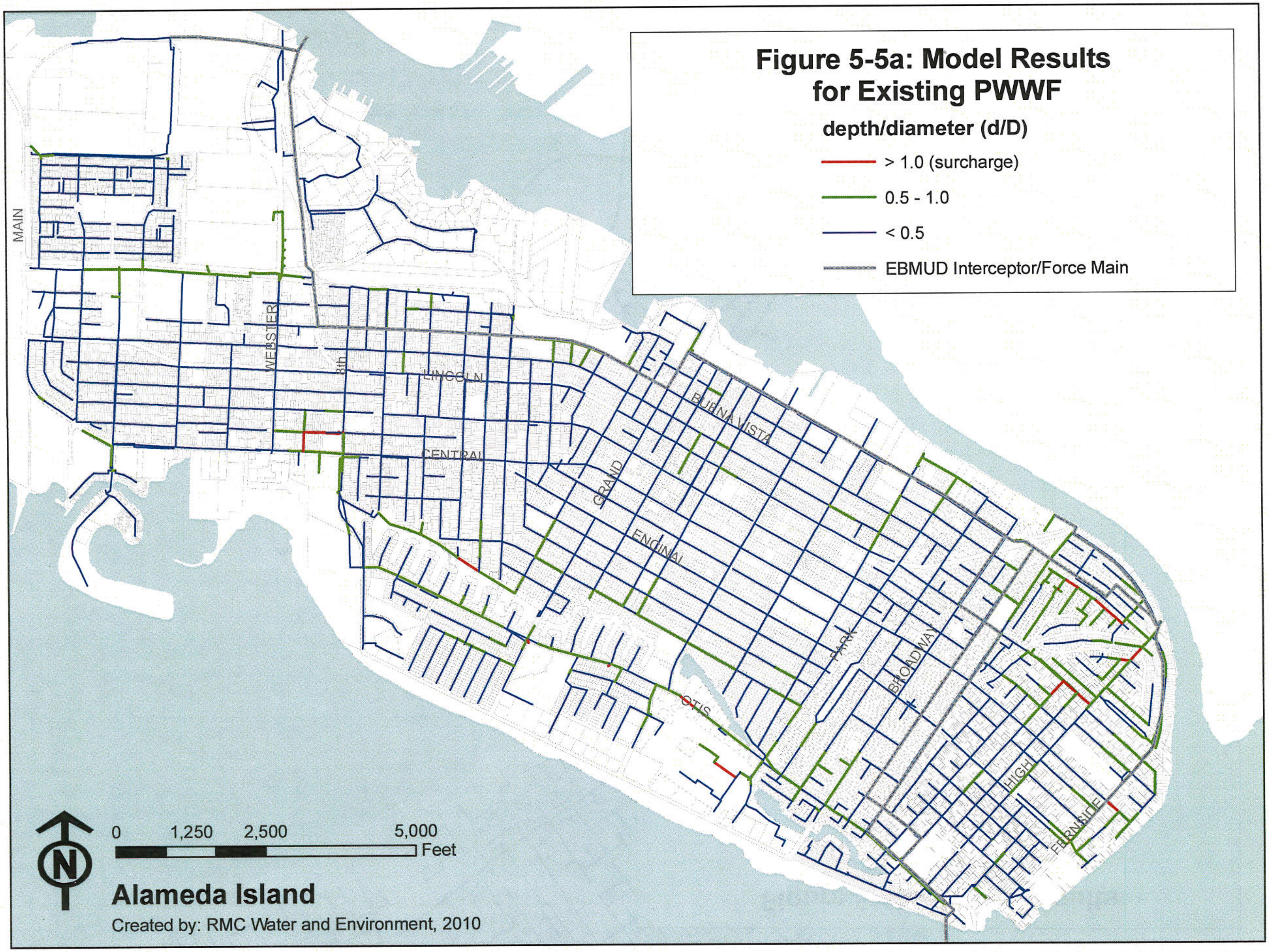


Figure 5-5a: Model Results for Existing PWWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
GRAND
BUENA VISTA
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

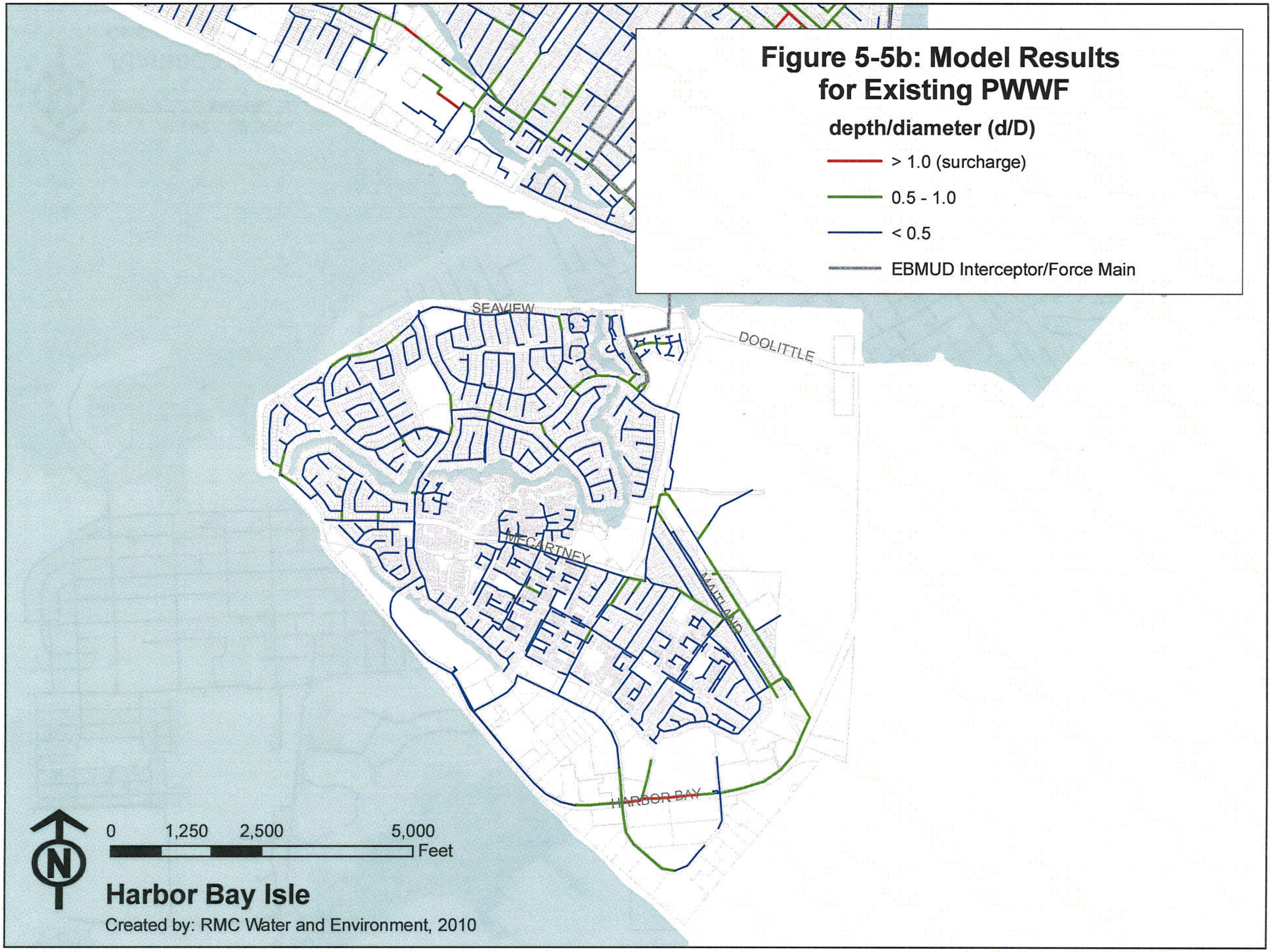


Figure 5-5b: Model Results
for Existing PWWF
depth/diameter (d/D)
> 1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
SEAVIEW
DOOLITTLE
MCCARTNEY
MAITLAND
HARBOR BAY
0
1,250
2,500
5,000
Feet
N
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

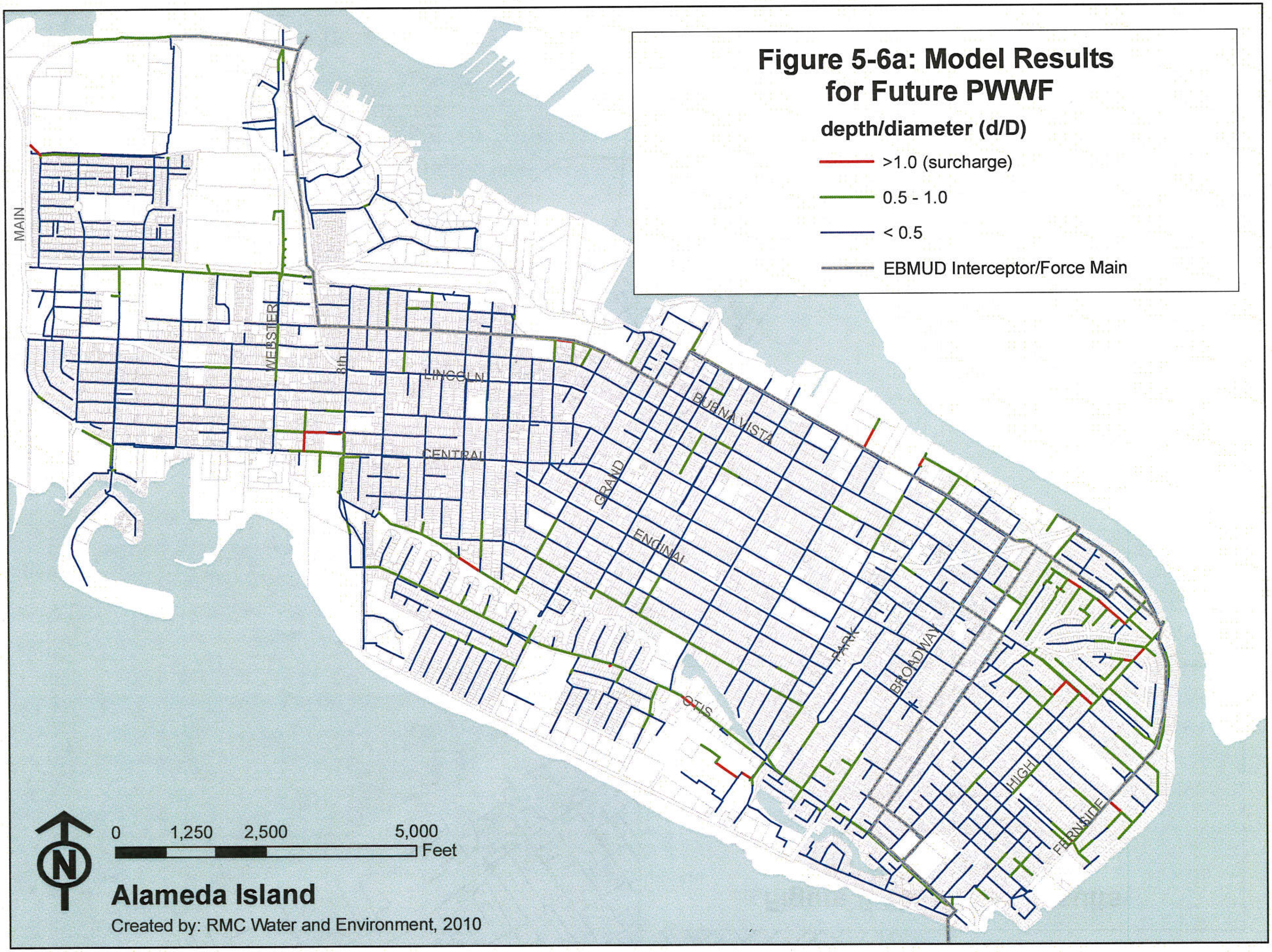


Figure 5-6a: Model Results for Future PWWF
depth/diameter (d/D)
>1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

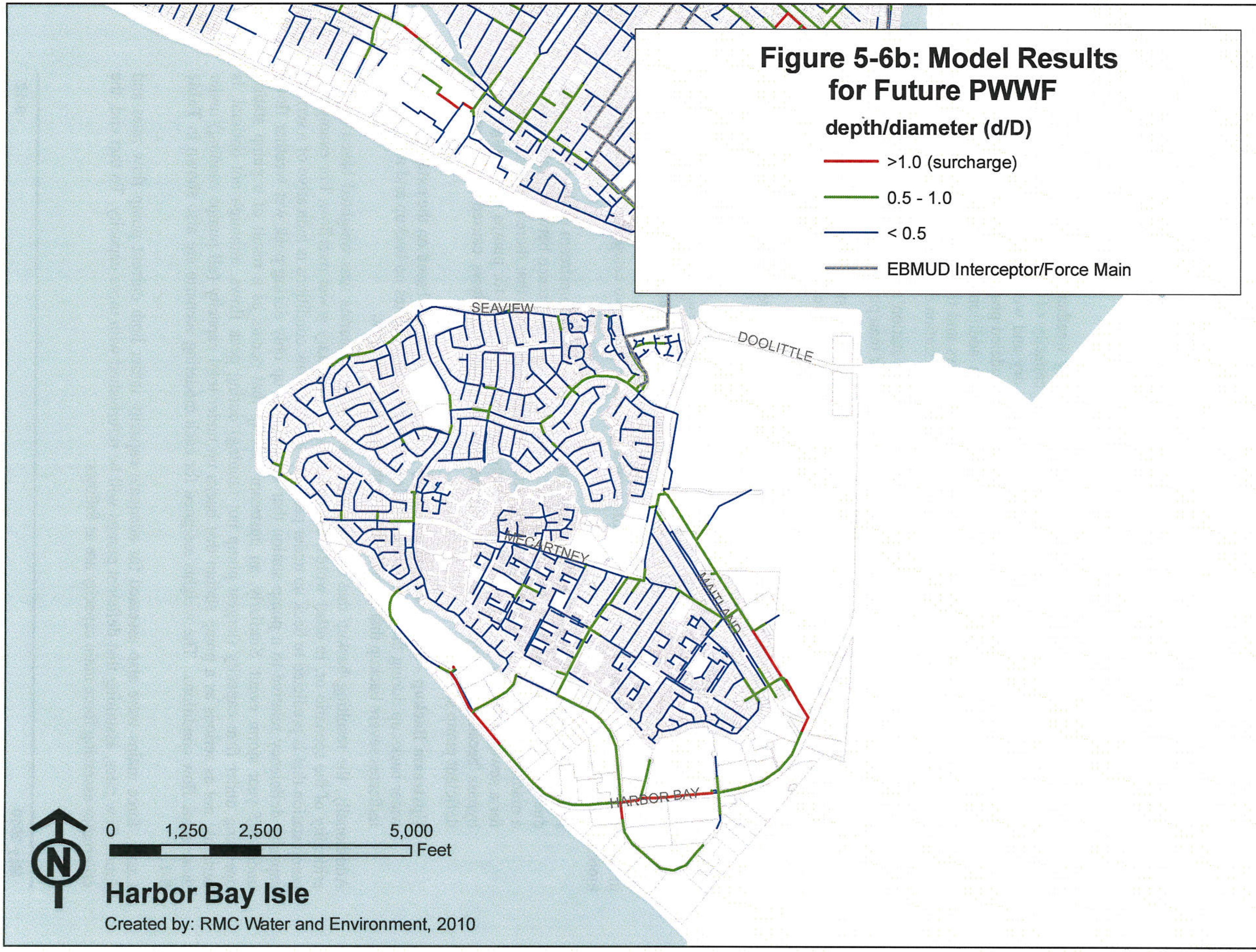


Figure 5-6b: Model Results
for Future PWWF
depth/diameter (d/D)
>1.0 (surcharge)
0.5 - 1.0
< 0.5
EBMUD Interceptor/Force Main
SEAVIEW
DOOLITTLE
MECARTNEY
MAITLAND
HARBOR BAY
N
0
1,250
2,500
5,000
Feet
Harbor Bay Isle
Created by: RMC Water and Environment, 2010

5.3 Pump Station Capacity Analysis

This section describes the criteria used to identify pump station and force main deficiencies and the results of this analysis.

5.3.1 Pump Station Capacity Criteria

Pump stations were evaluated based on their performance under dry weather flow conditions and under design-storm wet weather flows. Generally, firm capacity (the capacity of the pump station with one pump not in service) should be adequate to convey future peak dry and wet weather flows. Several of Alameda's pump stations have firm capacities that are less than modeled peak inflows, but this does not automatically mean the pump station needs to be retrofitted with additional capacity. To distinguish between capacity issues that could potentially cause sewer overflows and those that can be considered acceptable, a comprehensive approach was taken to identifying pump station deficiencies.

Alameda's sewer system has two key features that can assist a pump station's ability to adequately move sewage through the system and mitigate risks when inflows are higher than pump capacity:

- Sufficient pipe capacity upstream of the pump station, which can "store" the flow when the wet well backs up. Although upstream pipe can act as additional wet-well capacity, long-term back-up into the upstream network can lead to maintenance concerns.
- An upstream "high-level bypass" that allows flow to be rerouted around the pump station when water level reaches a relatively high stage.

Pump station deficiency scores were developed with these features in mind. Scores were assigned based on the performance of each pump station under several combinations of "testing" scenarios, including firm and total capacities and wet and dry weather flows. These scenarios are described in **Table 5-1.** To account for "adequate upstream pipe capacity" and "high-level bypass" lines, pumps were assigned scores in two categories: 1) "freeboard" and 2) extent of upstream backup. These two categories are described in more detail, below.

> **Freeboard** - Points were assigned based on the minimum freeboard upstream of the pump station due to pump station backup. Freeboard refers to the minimum difference between water level in a manhole and the ground elevation. Very small freeboard indicates that there is a high potential for a sewer overflow. Freeboard "scores" were assumed to be zero for pumps with a high-level bypass because the bypass will mitigate the risk of sewer overflows caused by a capacity-deficient pump station.
>
> **Upstream Backup** - Scores were qualitatively assigned from 0 to 3 based on the extent to which water backs up during the peak flow condition. A score of 0 indicates no backup and a score of 3 indicates severe backup that extends far into the upstream network.

Additionally, the resulting backup that occurs under the various scenarios was weighted based on the criticality of the scenario or, in other words, the potential for a sanitary sewer overflow. For example, a pump station that is predicted to back up under peak dry weather flow conditions is indicative of a serious capacity deficiency relative to a pump station that would back up only during peak wet weather flows under a 5-year storm event. Similarly, an upstream backup that occurs as a result of a pump station operating under firm capacity (largest pump not running) during peak flows, although not optimal, is certainly not as serious as a pump station that backs up under total capacity (all pumps running) under those same flow conditions. The weights assigned for each modeling scenario are also shown in **Table 5-1.**

Lastly, force mains were also assessed for adequate capacity under both current pump flow rates and future flow rates, assuming that deficient pumps will be refurbished with more capacity. To that end, the criteria for assessing force main capacity are as follows:

- Under peak wet weather flows, when the pump station is likely operating at total capacity (e.g. more than one pump is running which results in high flowrates), velocity should not exceed 10 feet per second. Exceptions were made for very short force mains, less than 100' feet.
- Under peak dry weather flows, when the pump station is likely operating at firm capacity (e.g. when only one pump is running), velocity should not exceed 5 feet per second. Exceptions were made for very short force mains, less than 100' feet.

Table 5-1: Pump Station "Testing" Scenarios and Weighting Factors

Capacity Test Scenario	Weighting Factor[1]
Pump operating with **firm capacity** only during **wet weather** flow	1
Pump operating with **firm capacity** only during **dry weather** flow	2
Pump operating with **total capacity** during **wet weather** flow	3
Pump operating with **total capacity** during **dry weather** flow	4

[1] Higher weight indicates that pump station capacity deficiency under that scenario is more critical.

5.3.2 Pump Station Capacity Analysis Results

The results of the pump station capacity analysis are explained in the following sub-sections along with the results of the pump deficiency scores and how these scores were used. These sub-sections also discuss where there is a need for additional standby pump capacity and force main capacity.

Pump Capacity Deficiencies

Pump station capacity deficiency scores were used in three ways. First, the overall score (the sum of each "freeboard" and "backup" score in each scenario) was used to identify which pump stations have less than optimal capacity. Second, and most importantly, the score within each scenario was examined to determine if a particular pump station should be retrofitted with additional capacity. More specifically, if a pump station's "freeboard" and "back-up" score within any scenario was equal to or greater than 4, then it was assumed that the pump should be given more capacity. If this was not the case but the pump station had an overall score greater than zero, then it was considered to have less than optimal, yet acceptable, capacity. Lastly, the overall score can be used by the City for prioritization of pump station capacity improvements.

The pump station capacity analysis revealed that, at a minimum, seven pump stations should be retrofitted with additional capacity (Grand Otis, Aughinbaugh, BFI, Park/Otis, Eight/Portola, and Harbor Bay Parkway 1, and Tideway). Based on this analysis, these seven pumps stations have the potential to severely backup during the design storm event and in some cases during dry weather flow, which may cause water levels in manholes to rise close to ground elevation.

The pump station analysis also identified several other pump stations (Pond/Otis, Willow/Whitehall, Sand Beach, Verdemar, and Dublin) as having less than optimal, yet acceptable, capacity. For these pump stations, it is recommend that if the City's ongoing condition assessment work finds these pumps to be in poor condition and refurbishment or replacement is needed, then new, larger pumps should be installed. If these pumps are not replaced, the system may back up during wet weather flows which, over time, may cause maintenance problems in upstream sewers.

Although the pump scores are assigned based on future flow conditions (i.e. build-out conditions), most pump stations did not see a significant increase in peak dry or wet weather inflows. In fact, only three pump stations (BFI, Harbor Bay Parkway 1, and Park/Otis) showed appreciable increase in flows. A comparison of existing and future peak inflows for pump stations is shown in **Table 5-2**.

Table 5-2: Existing vs. Future Inflows for Undersized Pump Stations (MGD)

Pump Station No.	Pump Station	Peak DWF (Existing)	Peak DWF (Future)	Peak WWF (Existing)	Peak WWF (Future)
22	Grand/Otis	0.62	0.62	0.89	0.89
5	Aughinbaugh	0.20	0.20	0.41	0.41
12	BFI	**1.03**	**2.00**	**2.31**	**3.10**
25	Eighth/Portola	1.06	1.06	1.60	1.60
16	Park/Otis	**0.49**	**0.54**	**1.15**	**1.20**
10	Harbor Bay Parkway 1	**0.30**	**0.90**	**0.69**	**1.17**
20	Pond/Otis	0.15	0.15	0.21	0.21
27	Tideway	0.25	0.25	0.35	0.35
8	Dublin	0.13	0.13	0.34	0.34
9	Verdemar	0.10	0.10	0.17	0.17
18	Willow/Whitehall	0.13	0.13	0.23	0.23
23	Sand Beach	0.09	0.09	0.14	0.14

The capacity analysis results for all pump stations under each of the testing scenarios are shown in **Table 5-3.**

Standby Pump Capacity

It is recommended that standby pumps be installed at two pump stations, Channing and Haile, to improve reliability of the system. These two stations have only one pump (zero firm capacity) and no high-level bypass line. A failure of either of these pumps could cause significant backup in the system.

Force Main Capacity

The capacity analysis found that for the most part, force mains in the system are appropriately sized, so no force main improvement projects are necessary for the existing system. If deficient pump stations are retrofitted with additional capacity to meet peak wet weather inflows, the existing force mains would have adequate capacity.

Table 5-3: Pump Station Capacity Scores

No.	Pump Station Name	Additional Capacity Needed?[1]	High-Level Bypass?[2]	Notes	PWWF, Firm Capacity (1)		PDWF, Firm Capacity (2)		PWWF, Total Capacity (3)		PDWF, Total Capacity (4)		Pump Deficiency Score (with weighing factors)[3]
					Free-board[4,2]	Extent of Backup[5]	Free-board[4,2]	Extent of Backup[5]	Free-board[4,2]	Extent of Backup[5]	Free-board[4,2]	Extent of Backup[5]	
				Maximum Possible Score-->	*4*	*3*	*4*	*3*	*4*	*3*	*4*	*3*	*70 (max)*
22	Grand Otis	Yes	No	Insufficient firm capacity (PWWF & PDWF), Insufficient total capacity (PWWF)	4	3	4	3	2	3	-	2	**44**
5	Aughinbaugh	Yes	No	Insufficient firm capacity (PWWF & PDWF), Insufficient total capacity (PWWF)	4	3	3	3	2	2	-	1	**35**
12	BFI	Yes	No	Insufficient firm capacity (PWWF), Insufficient total capacity (PWWF)	4	3	-	2	4	3	-	-	**32**
25	Eighth/Portola	Yes	No[6]	Insufficient firm capacity (PWWF & PDWF)	4	3	4	3	-	2	-	-	**27**
16	Park/Otis	Yes	No	Insufficient firm capacity (PWWF), Insufficient total capacity (PWWF)	4	2	-	-	4	2	-	-	**24**
10	Harbor Bay Parkway I	Yes	Yes	Insufficient firm capacity (PWWF & PDWF), Insufficient total capacity (PWWF)	-	3	-	2	-	2	-	-	**13**
20	Pond/Otis	Maybe	Yes	Sewer backup during firm and total capacity (there is only one pump)	-	-	-	-	-	2	-	1	**10**
27	Tideway	Yes	No	Insufficient firm capacity (PWWF)	3	2	-	1	-	-	-	-	**7**
8	Dublin	Maybe	No	Minor backup during firm capacity	2	2	-	-	-	1	-	-	**7**
9	Verdemar	Maybe	Yes	Sewer backup during firm and total capacity	-	2	-	1	-	1	-	-	**7**
18	Willow/Whitehall	Maybe	Yes	Sewer backup during firm and total capacity	-	2	-	1	-	1	-	-	**7**
23	Sand Beach	Maybe	Yes	Minor backup during firm capacity (there is only one pump)	-	-	-	-	-	2	-	-	**6**
3	Channing	No	No	Only one pump, no high-level bypass	-	-	-	-	-	-	-	-	**-**
42	Haile	No	No	Only one pump, no high-level bypass	-	-	-	-	-	-	-	-	**-**
1	Adelphian	No	No		-	-	-	-	-	-	-	-	**-**
15	Bayview	No	Yes	Only one pump, but high-level bypass provides redundancy	-	-	-	-	-	-	-	-	**-**
2	Catalina	No	No		-	-	-	-	-	-	-	-	**-**
28	Cola Ballena	No	No		-	-	-	-	-	-	-	-	**-**
26	Eighth/Taylor	No	Yes		-	-	-	-	-	-	-	-	**-**
43	Grand Street	No	No		-	-	-	-	-	-	-	-	**-**
31	Marina Village (Initial)	No	No		-	-	-	-	-	-	-	-	**-**
6	Seaview I	No	Yes		-	-	-	-	-	-	-	-	**-**
7	Seaview II	No	Yes	Only one pump, but high-level bypass provides redundancy	-	-	-	-	-	-	-	-	**-**
4	Sheffield/Cumberland	No	Yes	Only one pump, but high-level bypass provides redundancy	-	-	-	-	-	-	-	-	**-**
30	Triumph/Independence	No	Yes		-	-	-	-	-	-	-	-	**-**

Needs Additional Capacity	Less than Optimum Capacity	Needs Redundancy	Capacity OK

1. Pumps need additional capacity if the sum of Minimum Freeboard and Extent of Backup scores in any scenario is equal to or greater than 4. Pumps that do not meet this criteria but which have an overall deficiency score greater than 0, are labeled as "Maybe"; these pump stations are assumed to have less than optimal yet adequate capacity. Pump station that do not need additional capacity are labeled as "No".

2. A high-level bypass provides redundancy in the system by allowing sewage to bypass the pump station in the event of severe backup. For pump stations where this bypass line adequately mitigates the risk of upstream overflows due to pump deficiencies, Max Freeboard scores were assumed 0.

3. Pump deficiency score is calculated as the sum of each scenario multiplied by its 'weight' factor. Therefore:
Pump Deficiency score = $(\text{Freeboard Score} + \text{Backup Score})_{\text{Scenario 1}} * 1 + (\text{Freeboard Score} + \text{Backup Score})_{\text{Scenario 2}} * 2 + (\text{Freeboard Score} + \text{Backup Score})_{\text{Scenario 3}} * 3 + (\text{Freeboard Score} + \text{Backup Score})_{\text{Scenario 4}} * 4$

4. Freeboard scores are assigned from 0 to 4 based on the smallest distance between the water surface in a manhole and the ground at any point upstream (caused by upstream backup). Specifically, points are assigned as follows:
0 = maximum Freeboard is greater than -7 feet
1 = maximum Freeboard is between -5 and -7 feet
2 = maximum Freeboard is between -3 and -5 feet
3 = maximum Freeboard is between -2 and -3 feet
4 = maximum Freeboard is less than -2 feet

5. Extent of Backup scores are qualitatively assigned from 0 to 3 based on the extent to which water backs up during the design rainfall event. A score of 0 indicates no backup and a score of 3 indicates severe backup that extends deeply into the upstream network
6. Eighth and Portola does have a bypass line, but if sewage was permitted to backup to the bypass elevation, upstream manholes would overflow. Therefore, this bypass is not considered to add redundancy to the system.

Chapter 6 Recommended Capital Improvement Program

This chapter presents the specific sewer improvement projects that are recommended for inclusion in the City's Capital Improvement Program (CIP) based on the findings of the capacity analysis. For recommended sewer pipeline improvements, each project is documented with a general description, project details, deficiency profiles, planning-level capital cost estimates, and relative priority rating. Pump station capacity improvement needs have been identified and prioritized; however, detailed facility improvements and cost estimates were not developed as part of this study. This is because the City is currently conducting a comprehensive pump station condition assessment under a separate project. After integrating the findings contained in this report with the condition assessment results, pump station improvement projects (and costs) will be developed as part of the condition assessment work.

6.1 Gravity Sewer Pipeline Improvements

Capital improvement projects were identified to address potential problems identified in the capacity analysis. As noted in Chapter 5, the need for a project was identified based on surcharge conditions and depth of water level below ground. Specifically, a project was developed for any reach of pipe which the model determined to be both under capacity and to cause more than one foot of surcharging within 6 feet of ground level under the EBMUD design storm. These surcharge criteria allow the City to focus capital spending on areas with the greatest risk of sanitary sewer overflows (SSOs). As it turns out, the modeling identified only three areas with pipe capacity deficiencies requiring improvements. It should be noted that the modeling conducted for this study assumes that the City's pipes are in working condition and relatively free of obstructions that would result in reduced capacity. Therefore, if pipes are partially obstructed due to grease, debris, or roots, or have reduced slopes due to sags, SSOs may still occur if the restricted pipe cross section is insufficient to convey the peak design flows.

For each capacity limitation identified, a project was developed to replace the existing pipe with a larger pipe. **Figure 6-1** shows an overview of the project locations. Project data sheets are included in **Appendix A** to present the details of the identified projects, including location, a brief description (length and diameter of new pipe), priority, and estimated planning-level cost estimate. A summary of these projects (and their estimated costs) is shown is shown in **Table 6-1**.

Projects were sized to avoid surcharge during future PWWF conditions. It was also assumed that the existing pipe would be replaced with a larger pipe at the same slope. The model was run with the recommended improvement projects to confirm that the upsized pipes would generally not exceed 75 percent full capacity during future PWWF conditions.

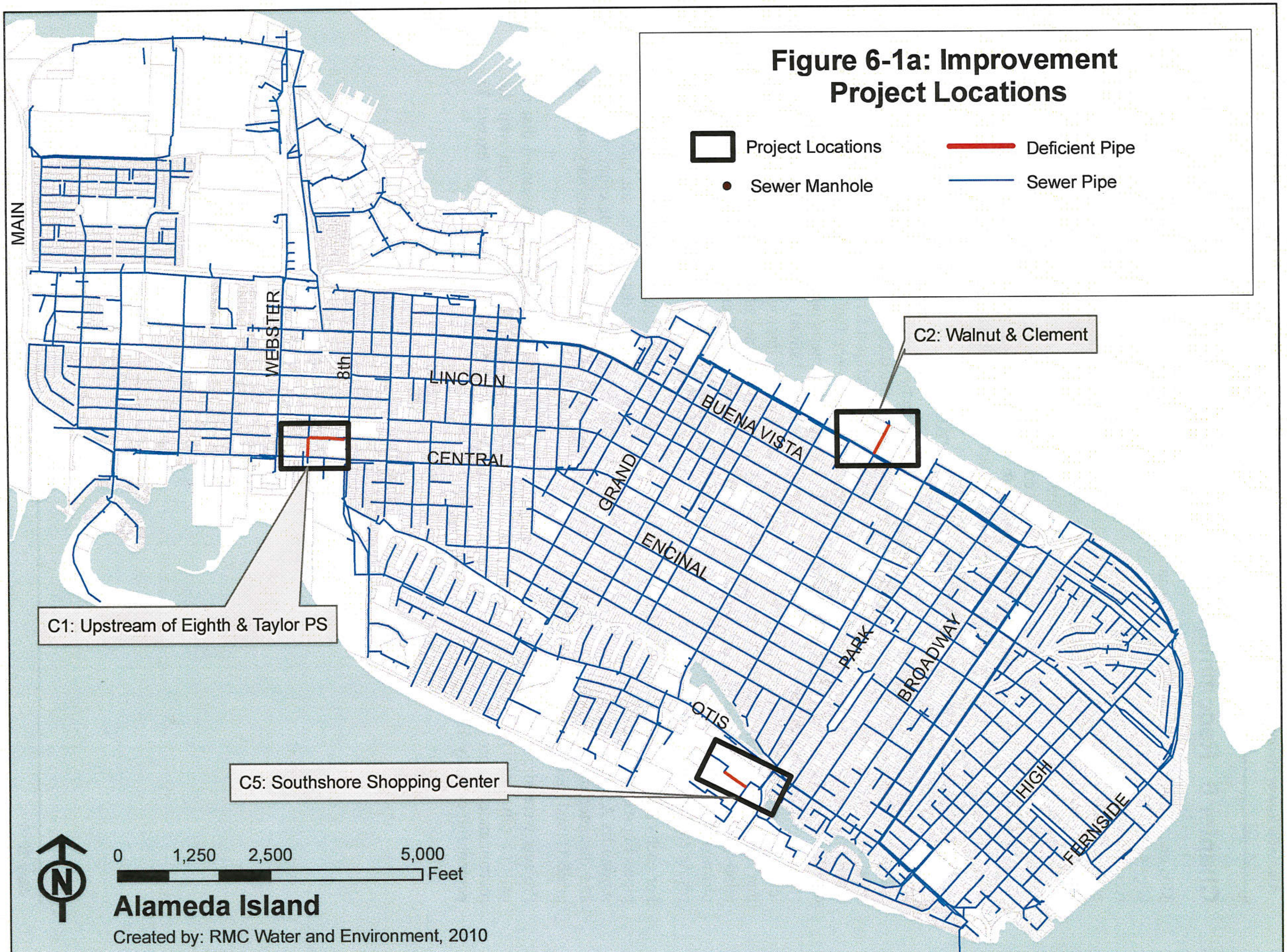


Figure 6-1a: Improvement Project Locations
Project Locations
Sewer Manhole
Deficient Pipe
Sewer Pipe
C1: Upstream of Eighth & Taylor PS
C2: Walnut & Clement
C5: Southshore Shopping Center
MAIN
WEBSTER
8th
LINCOLN
CENTRAL
BUENA VISTA
GRAND
ENCINAL
OTIS
PARK
BROADWAY
HIGH
FERNSIDE
0
1,250
2,500
5,000
Feet
N
Alameda Island
Created by: RMC Water and Environment, 2010

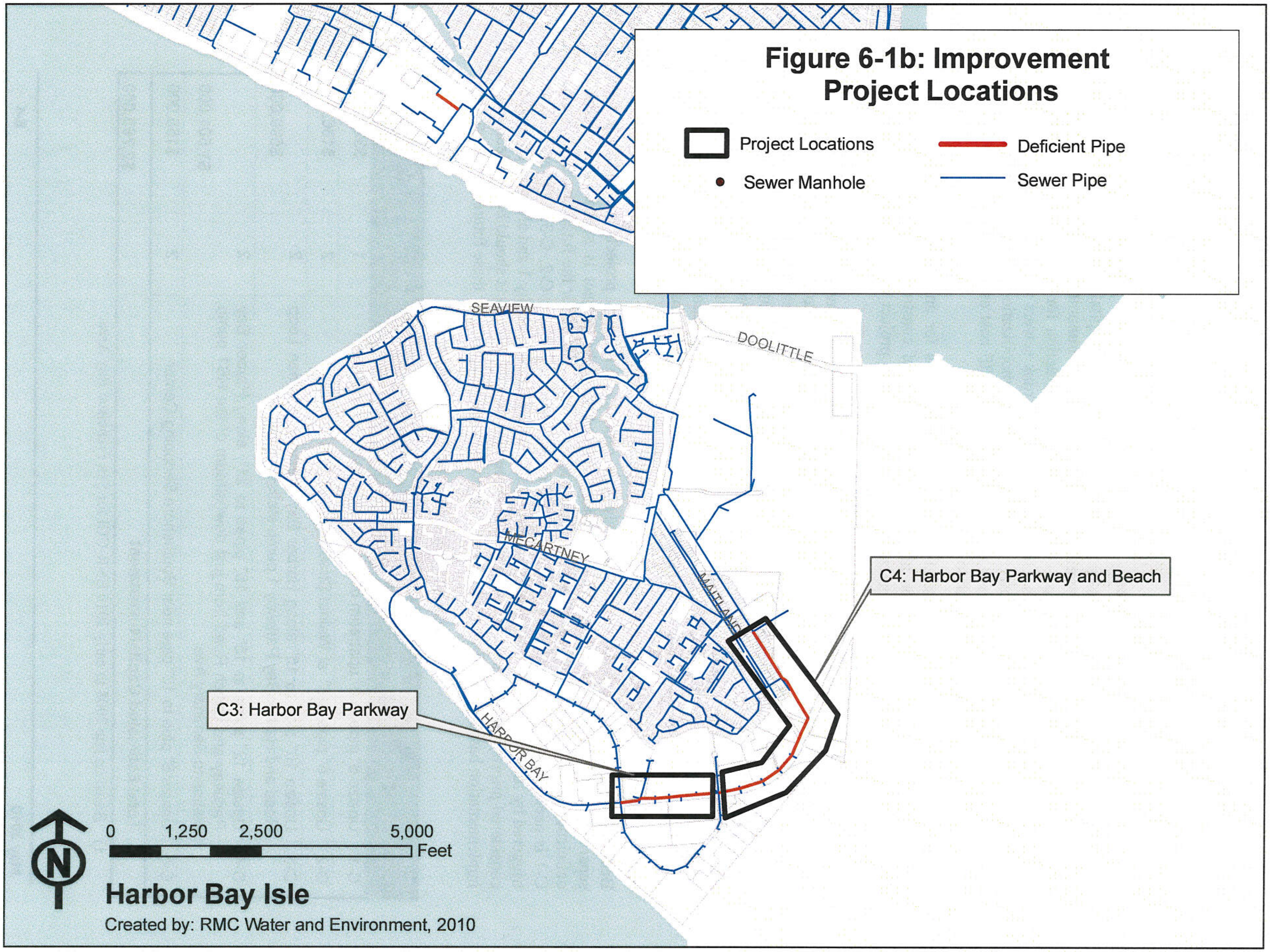


Figure 6-1b: Improvement Project Locations

6.1.1 Pipe Project Cost Estimates

Cost estimates were developed for gravity pipe replacement projects. Capital costs were estimated based on cost data and prior experience from similar projects in Bay Area communities including recent projects in the City of Alameda. These unit costs include baseline construction for gravity trunk sewers using trenchless (e.g., pipe bursting) methods. Unit costs for gravity trunk sewers vary with pipe diameter and are assumed to include manholes, lower lateral replacement, mobilization/demobilization, pavement restoration, and traffic control. Unit costs used for the proposed CIP cost estimates range from $210 to $290 per foot of pipe replaced for 8-inch to 21-inch pipe.

In addition to unit costs, a 30 percent allowance for contingencies for unknown conditions was also included for all projects, as well as an allowance of 25 percent of construction cost for engineering, administration, and legal costs. The itemized cost estimate for each project is detailed on the individual project information sheets included at the end of this section.

These cost estimates are planning or conceptual level estimates, and are considered to have an estimated accuracy range of -30 to +50 percent. This level of accuracy corresponds to an "order of magnitude" or "Class 5" cost estimate as defined by the American Association of Cost Estimators. These estimates are suitable for use for budget forecasting, CIP development, and project evaluations, with the understanding that refinements to the project details and costs would be necessary as projects proceed into the design and construction phases. Estimates were prepared using late-2008 U.S. dollars, which are very close to current (early to mid-2010) costs. The Engineering News Record Construction Cost Index (ENR-CCI) for the San Francisco Bay Area was used to escalate past bid prices to current costs.

6.1.2 Pipe Project Prioritization

Because the capacity analysis resulted in only five capacity deficiency projects, the projects were separated into two priority categories depending upon whether the project is needed to mitigate a deficiency under existing flow conditions, or if the project is only needed for the future scenario. Project C-1 is needed to resolve an existing capacity deficiency whereas Projects C-2, C-3, C-4, and C-5 are triggered by future developments. Therefore, Project C-1 is considered priority 1 and all other projects are considered priority 2. Note that the location of and need for priority 2 projects should be verified prior to implementation based on the final land uses and proposed sewerage plans for these future developments.

Table 6-1: Improvement Project Summary

CIP No.	Project Description & Location	Priority	Estimated Capital Cost[1]
C-1	Upsize 6" pipe to 8" upstream of Eighth and Taylor Pump Station	1	$322,000
C-2	Upsize 6" pipe to 8" at Walnut Street and Clement Ave	2	$186,000
C-3	Upsize 10" pipe to 15" along Harbor Bay Parkway between Loop Road and Harbor Bay Parkway 1 pump station	2	$599,000
C-4	Upsize 12" pipes to 15" and 15" pipes to 21", along Harbor Bay Parkway and Beach Road from just downstream of the HB1 pump station to Seminary Ave	2	$1,501,000
C-5	Upsize 8" pipe to 10" pipe near Southshore Shopping Center	2	$155,000
	Total Estimated Capital Investment		**$2,763,000**

1) Based on late 2008 to early 2010 ENR-CCI for San Francisco Bay Area.

6.2 Pump Station Improvements

As discussed in Chapter 5, deficiency scores were assigned to each modeled pump station based primarily on the severity to which sewage backs up into upstream pipes and how high the sewage rises relative to the ground during each of the scenarios analyzed. This score is helpful in prioritizing deficiencies. These scores along with additional criteria led to the conclusion that at least seven pump stations should be retrofitted with additional capacity (Grand/Otis, Aughinbaugh, Eight/Portola, Park/Otis, BFI, Harbor Bay Parkway, and Tideway), two pump stations should be provided with standby capacity (Channing and Haile), and five pumps should be given more capacity if the City finds them to be in poor condition (Pond/Otis, Willow/Whitehall, Sand Beach, Verdemar, and Dublin). The ideal capacity (based on model results under future conditions) for each pump station is given in **Table 6-2** (this table also shows modeled PDWF, PWWF, and estimated firm and total capacity of these pump stations for comparison purposes).

Specific improvement projects and their costs have not been developed as part of this work because the City is in the process of implementing a pump station condition assessment. It is expected that the findings from this Hydraulic Analysis will be incorporated into the City's condition assessment and capital improvement projects will be developed based on the findings from both efforts.

Table 6-2: Pump Capacity (Recommended & Current) for Deficient Pump Stations

Pump Station	Overall Pump Deficiency Score[1]	PDWF (Future)	PWWF (Future)	Current Firm Capacity (MGD)	Current Total Capacity (MGD)	Recom. Firm Capacity (MGD)	TDH at Firm Capacity Flow (ft)[2]
Pump stations that need additional capacity:							
Grand Otis	44	0.62	0.89	0.27	0.46	0.89	18
Aughinbaugh	35	0.20	0.41	0.09	0.14	0.41	31
BFI	32	2.00	3.10	1.60	2.00	3.10	73
Eighth/Portola	27	1.06	1.60	0.67	1.35	1.60	26
Park/Otis	24	0.54	1.20	0.61	0.89	1.20	64
Harbor Bay Parkway I	13	0.90	1.17	0.75	1.00	1.17	18
Tideway	7	0.25	0.35	0.23	0.44	0.35	Variable[3]
Pumps stations that have less than optimal, yet acceptable capacity:							
Pond/Otis	10	0.15	0.21	-	0.10	0.21	6
Willow/Whitehall	7	0.13	0.23	0.07	0.13	0.23	8
Verdemar	7	0.10	0.17	0.07	0.14	0.17	10
Dublin	7	0.13	0.34	0.18	0.26	0.34	25
Sand Beach	6	0.09	0.14	-	0.06	0.14	11
Pump stations with adequate capacity and no redundancy (zero firm capacity and no high-level bypass):							
Channing	-	Standby Pump Recommended					
Haile	-	Standby Pump Recommended					

1) Refer to Section 5.3.2 for explanation of pump capacity scores.
2) TDH values for recommended capacity have been estimated for planning purposes only - detailed analysis should be conducted during design.
3) Tideway pump station TDH is a function of flow through Tideway and Cola Bolena pump stations and cannot easily be estimated.

6.3 Project Implementation

The City should begin implementation of the Capital Improvement Program recommended in this Report, starting with the highest priority projects. This Report does not specify an implementation schedule, as the City will need to balance sewer improvements with the need for other capital projects. The following items should be considered in project scheduling and design, and in future updates of the Master Plan.

- Move forward with further planning and design of the Priority 1 gravity pipe projects.
- All gravity sewer projects detailed in this report are based on pipe replacement. The decision to parallel or replace existing sewers should consider the physical condition and remaining useful life of the existing pipelines; the availability of pipeline corridors for new sewer construction; and operation and maintenance concerns.
- The hydraulic model has been developed to assist the City in performing capacity analyses and updating this analysis in the future. The model should be kept up-to-date with any changes to existing sewer connections, development plans, and sewer system facilities. The City should continue with the current sewer inspection and condition assessment program, identifying sewers

that should be replaced due to poor condition. To the extent possible, these improvements should be coordinated with the recommended capacity-related improvements.

- The City should assess its sewer rates and connection fees as needed to ensure adequate funding for the recommended capacity improvement CIP.

In addition to the project recommendations listed above, the City should continue to address I/I through continued inspection and rehabilitation of sewer mains and lower laterals. The findings in this Report should be updated whenever there are major changes in planning assumptions or significant additional rehabilitation of the sewer system.

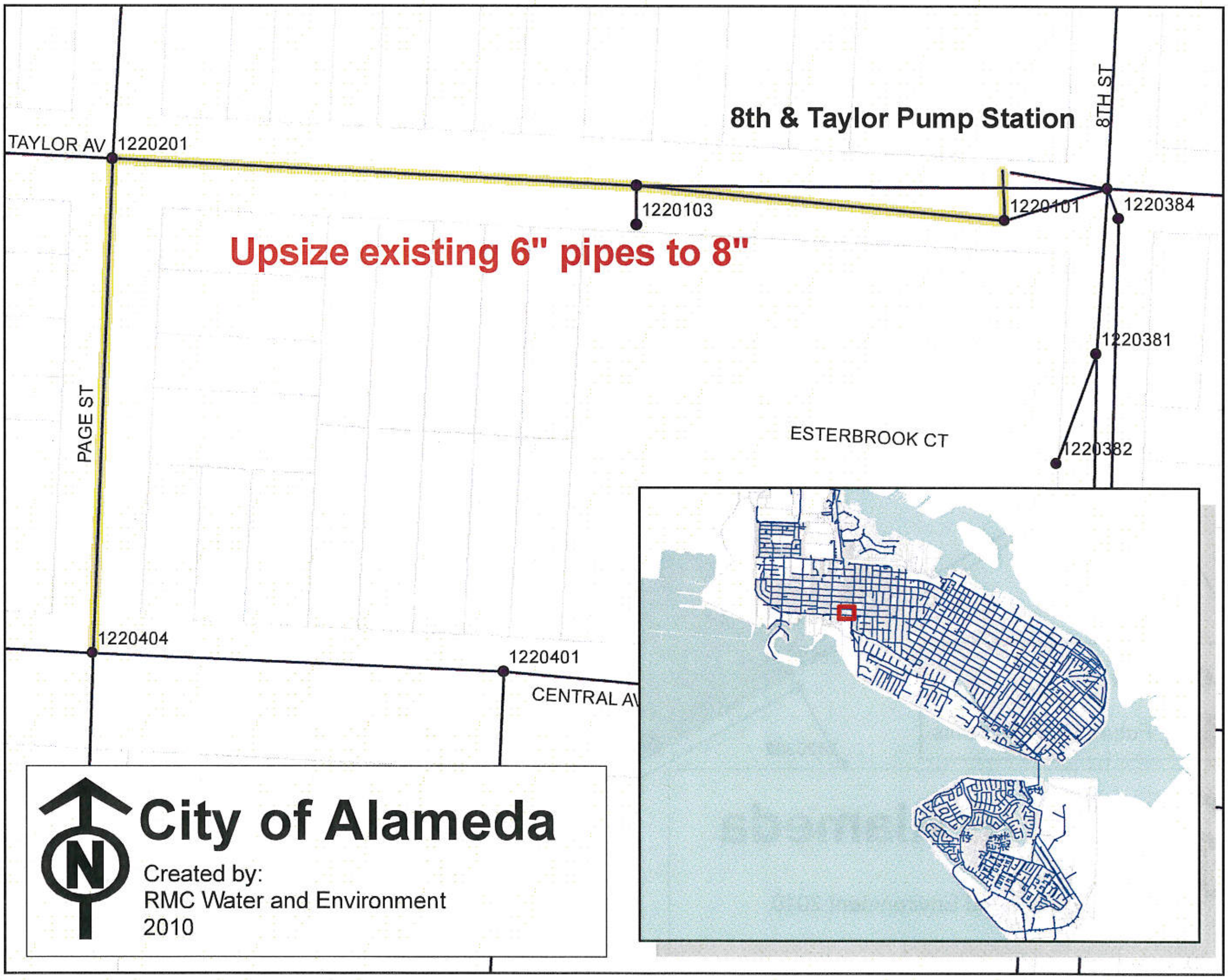
Appendix A - Project Information Sheets

C-1 Project Location & Description

Project Description

PROJECT ID:........................	C-1
PROJECT LOCATION:............	Upstream of Eighth and Taylor Pump Station
PRIORITY:............................	1
PROJECT DESCRIPTION:........	Upsize 6" pipe to 8" upstream of Eighth and Taylor Pump Station
DEFICIENCY DESCRIPTION:....	Current pipes are surcharged during design storm and during peak dry weather flow (existing and future)
PROJECT EXTENTS:..............	Upsize 6" pipe to 8" from 1220404 to1220201, from 1220201 to1220102, and from 1220102 to 1220101, and from 1220101 to Eight & Taylor Wet Well.
PEAK DESIGN FLOW:............	0.33 MGD
ASSUMPTIONS:....................	
ALTERNATIVES:....................	None identified

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
6"	8"	944		210	$198,156
			Construction Cost Sub-Total:		**$198,156**
			Contingencies	30%	$59,447
			Estimated Construction Cost:		**$257,603**
			Technical Services and Administration	25%	$64,401
			Total Project Cost:		**$322,004**

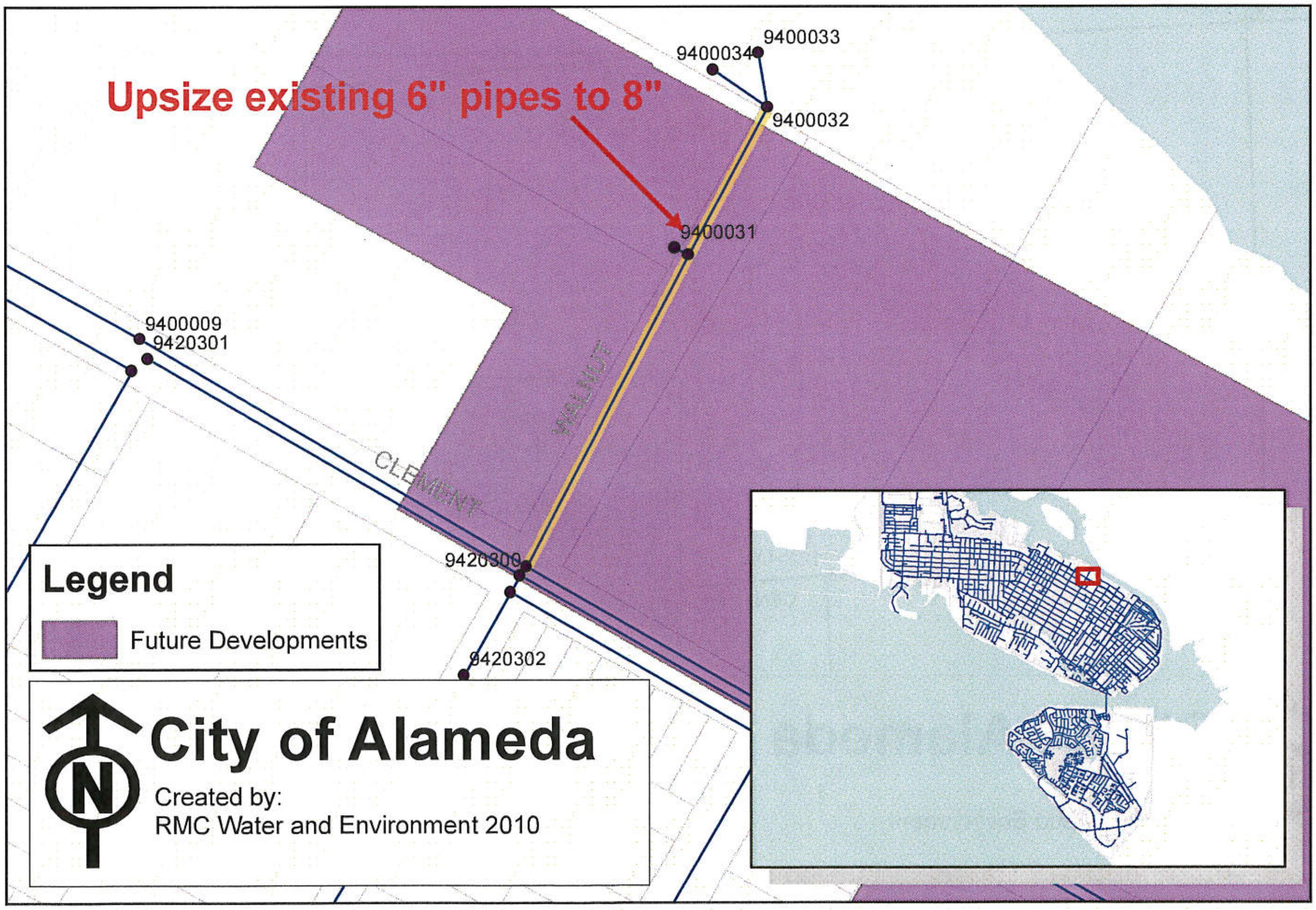


C-2 Project Location & Description

Project Description

PROJECT ID:......................	C-2
PROJECT LOCATION:............	Walnut Street @ Clement Ave
PRIORITY:...........................	2
PROJECT DESCRIPTION:........	Upsize 6" pipe to 8" at Walnut Street and Clement Ave
DEFICIENCY DESCRIPTION:....	Pipe segment surcharges during peak dry and wet weather flows conditions, but not at all during existing dry or wet weather flows
PROJECT EXTENTS:..............	Upsize existing 6" pipes to 8" from 9400032 to 9400010
PEAK DESIGN FLOW:............	0.31 MGD
ASSUMPTIONS:.....................	This pipe segment is only surcharged during future conditions due to future development. If development does not occur as expected, this project will not be necessary
ALTERNATIVES:....................	None Identified

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
6"	8"	546		210	$114,660
		Construction Cost Sub-Total:			**$114,660**
		Contingencies		30%	$34,398
		Estimated Construction Cost:			**$149,058**
		Technical Services and Administration		25%	$37,265
		Total Project Cost:			**$186,323**

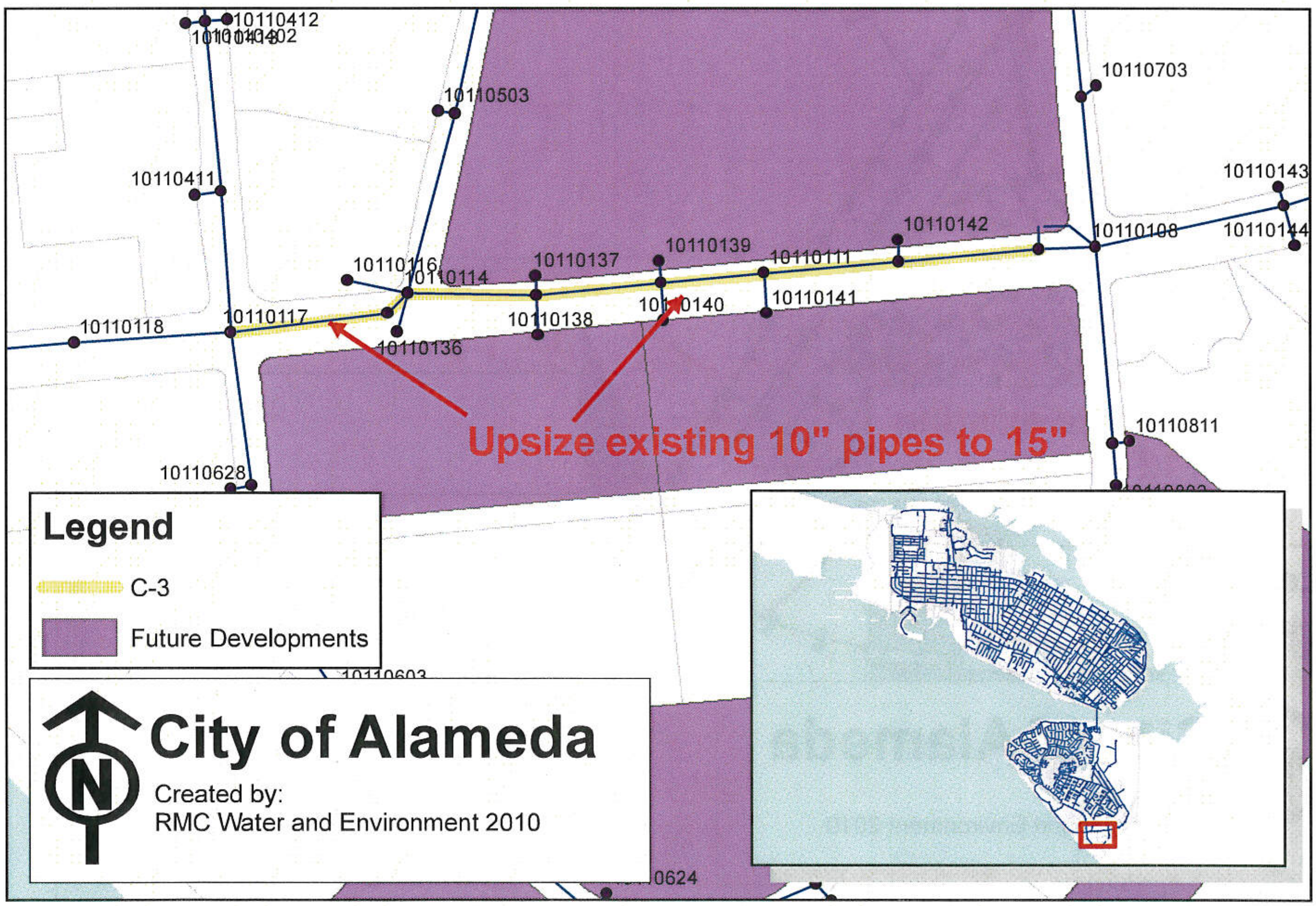


C-3 Project Location & Description

Project Description

PROJECT ID:........................	C-3
PROJECT LOCATION:............	Harbor Bay Parkway, upstream of Harbor Bay Pump Station 1
PRIORITY:............................	2
PROJECT DESCRIPTION:........	Upsize 10" pipe to15" along Harbor Bay Parkway between Loop Road and Harbor Bay Parkway 1 pump station
DEFICIENCY DESCRIPTION:....	Pipe segment surcharges by 5.9 ft during future peak wet weather flows due to future development in the Harbor Bay area. Pipe also surcharges during existing conditions, but surcharge is less than 1ft.
PROJECT EXTENTS:..............	Upsize existing pipes to 15" from 10110109 to 10110117
PEAK DESIGN FLOW:............	1.53 MGD
ASSUMPTIONS:.....................	This pipe segment is only surcharged during future conditions due to future development. If development does not occur as expected, this project may not be necessary
ALTERNATIVES:....................	None Identified

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
10"	15"	1,536		240	$368,640
			Construction Cost Sub-Total:		**$368,640**
			Contingencies	30%	$110,592
			Estimated Construction Cost:		**$479,232**
			Technical Services and Administration	25%	$119,808
			Total Project Cost:		**$599,040**

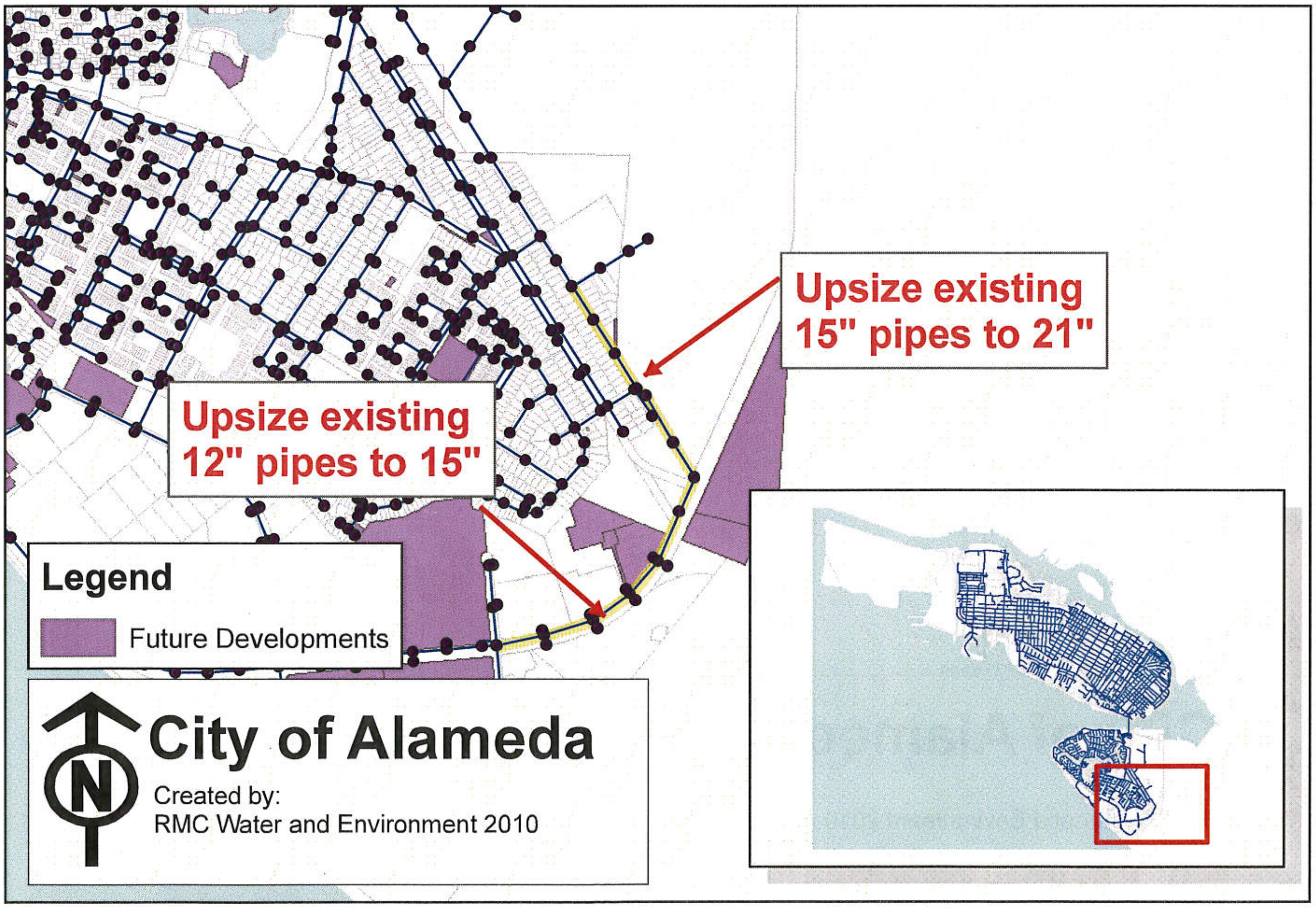


C-4 Project Location & Description

Project Description

PROJECT ID:........................	C-4
PROJECT LOCATION:............	Harbor Bay Parkway and Beach, downstream of Harbor Bay Pump Station 1
PRIORITY:............................	2
PROJECT DESCRIPTION:........	Upsize 12" pipes to 15" and 15" pipes to 21", along Harbor Bay Parkway and Beach Road from just downstream of the HB1 pump station to Seminary Ave
DEFICIENCY DESCRIPTION:....	Project needed for future peak wet weather flow only.
PROJECT EXTENTS:..............	Upsize existing pipes to 15" from 10110108 to 10013224 and to 21" from 10013224 to10013219
PEAK DESIGN FLOW:............	1.87 MGD (15") 2.09 MGD (21")
ASSUMPTIONS:......................	This pipe segment is only surcharged during future conditions due to future development. If development does not occur as expected, this project may not be necessary
ALTERNATIVES:....................	None Identified

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
12"	15"	2,799		240	$671,760
15"	21"	899		280	$251,720
		Construction Cost Sub-Total:			**$923,480**
		Contingencies		30%	$277,044
		Estimated Construction Cost:			**$1,200,524**
		Technical Services and Administration		25%	$300,131
		Total Project Cost:			**$1,500,655**



C-5 Project Location & Description

Project Description

PROJECT ID:........................	C-5
PROJECT LOCATION:............	Southshore Shopping Center
PRIORITY:............................	2
PROJECT DESCRIPTION:........	Upsize 8" pipe to 10" pipe near Southshore Shopping Center
DEFICIENCY DESCRIPTION:....	Pipe segment surcharges by 1.1 ft during peak wet weather flows due to future development in the Southshore Shopping Center
PROJECT EXTENTS:..............	Upsize existing 8" pipe to 10" pipe from 9732004 to 9732002
PEAK DESIGN FLOW:............	0.57 MGD
ASSUMPTIONS:.....................	This pipe segment is only surcharged during future conditions due to future development. If development does not occur as expected, this project may not be necessary.
ALTERNATIVES:....................	None Identified

Existing Diameter (inches)	New Diameter (inches)	Length (feet)		Unit Cost ($/lf)	Total Cost ($)
8"	10"	443		215	$95,245
		Construction Cost Sub-Total:			**$95,245**
		Contingencies		30%	$28,574
		Estimated Construction Cost:			**$123,819**
		Technical Services and Administration		25%	$30,955
		Total Project Cost:			**$154,773**