FINAL REPORT

Sewer Master Plan

Volume 1 of 2

City of Huntington Beach
California

May 2003
K/J 014641.00

Kennedy/Jenks Consultants

Kennedy/Jenks Consultants

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14 May 2003

Mr. Todd Broussard, P.E.
Engineering Division,
Public Works Department
City of Huntington Beach, City Hall
2000 Main Street
Huntington Beach, California 92648

Subject: Final Report – Sewer Master Plan

Dear Mr. Broussard:

In accordance with our discussions, Kennedy/Jenks Consultants is pleased to present the final report of the City's 2003 Sewer Master Plan. This report is intended to serve as a plan for the capital requirements of the City's sewer utility system.

The findings contained in this master plan are based on an evaluation of the City's collection and pumping system. This evaluation included a limited field flow monitoring program to obtain actual field measured wastewater data and a desktop inflow and infiltration (I&I) analysis to assess potential I&I areas of the City. The recommended capital improvements are developed to correct current system deficiencies, provide the ability to serve future growth, and replace aging infrastructure requirements to improve system reliability. An updated Sewer Facility Charge is prepared to assure that future customers pay their share of the costs of system capacity.

We have enjoyed working with you on this interesting project. Please contact us if you have any questions or need additional information.

Very truly yours,

KENNEDY/JENKS CONSULTANTS

Roger D. Null, V. P.
Project Manager

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Volume I

EXECUTIVE SUMMARY

The City of Huntington Beach (City) has embarked on this sewer master planning effort in recognition of the need to identify the areas of hydraulic deficiencies, assess the potential for inflow and infiltration (I&I) problems, and establish the level of capital required to maintain and upgrade the wastewater system to ensure reliable and uninterrupted wastewater service. The general scope of work includes:

- Data Collection and Review
- Criteria and Flow Projection Development
- Wastewater System Description
- Desktop I&I Study
- Field Flow Monitoring and I&I Evaluation
- Assessor Parcel Number (APNs) and Utility Billing Account Correlations
- Wastewater Master Plan Document Preparation

The focus of this master planning effort is to evaluate the capability of the important elements of the City's existing wastewater collection and pumping system and to develop a plan to provide service through a planning period that extends beyond the year 2020. The primary byproduct of this effort is the development of the wastewater utility's capital improvement requirements. A limited field Wastewater Flow Monitoring Study was also performed and is submitted as Volume II of this Master Plan project.

GENERAL SYSTEM DESCRIPTION

The City owns, operates and maintains a wastewater collection and pumping system. The collection system is comprised of approximately 360 miles of wastewater pipelines ranging in size from 6 to 30 inches in diameter. Approximately 85 percent of the City's wastewater pipelines are 8 inches in diameter. The predominant material of these pipelines is vitrified clay pipe (VCP). Due to the City's generally flat conditions, the City also operates and maintains twenty-seven lift stations. These facilities lift sewage from low points in the collection system to manholes at higher locations.

Orange County Sanitation District (OCSD) is responsible for receiving, treating, and disposing of the wastewater generated in central and northwest Orange County, including the City's wastewater. In this regional management capacity, OCSD owns, operates and maintains the majority of the "backbone" wastewater collection trunk pipelines. As such, the City's local system generally discharges to larger OCSD facilities to convey wastewater to the local treatment plant.

Construction of the City's collection system began before 1900. However, the majority of the system appears to have been constructed to support the rapid growth that began in the 1960's. Although the City is approximately 97 percent built out and only a minimal increase in future wastewater flows is projected, the City has recognized that the condition of the infrastructure needs to be further quantified and additional proactive provisions for long-term reliability implemented.

In accordance with this need, in August 2001 the City adopted a new sewer service charge to provide the necessary funds for ongoing reinvestment. Increased funding is now available for ongoing operation and maintenance (O&M) activities and capital investment in infrastructure. A comprehensive video inspection of the entire underground wastewater utility system and a methodical rehabilitation of the City's lift stations are some of the components of the City's infrastructure management activities that are designed to promote long term system reliability.

WASTEWATER FLOW MONITORING PROGRAM

To assess the wastewater characteristics in the City, a limited field flow-monitoring program was conducted by Kennedy/Jenks Consultants in association with ADS Environmental, Inc. (ADS). This temporary flow-monitoring program was implemented to obtain actual field measurements of specific wastewater characteristics in the City. Field measurements were obtained during March 2002 in an attempt to also measure the impact of a wet weather event, and quantify the level of inflow and infiltration (I&I) on the City's collection and pumping system.

Twelve monitoring locations throughout the City were identified and metering facilities were installed, tested, and calibrated to record minimum, average, and peak wastewater flows. The monitoring program recorded flow values at a 15-minute frequency throughout the 28-day program duration. Detailed results of the flow-monitoring program for each of the monitored sites are provided as Volume II of this Master Plan documentation. Summary monitoring data results are contained as an appendix in Volume I.

INFLOW AND INFILTRATION FINDINGS AND RECOMMENDATIONS

An important consideration in the City's management of the wastewater system is the need to integrate the effects of I&I on system hydraulic capacity. Since a significant rainfall event did not occur during the conduct of the field flow-monitoring program, actual rainfall dependent I&I factors could not be derived. In lieu of actual data, most communities integrate I&I through a reserve capacity allowance in their design criteria. This reserve allowance was utilized in the City's prior master planning projects and is recommended for continuation.

To supplement the field flow-monitoring data, a desktop I&I study was performed in an attempt to further quantify the potential for local I&I. The desktop study utilized available data to assess the potential for non-sanitary sewer flows into the system. The results of this study provides additional support for the City's wastewater system lining program in the harbor area to minimize seawater intrusion, identified several isolated pockets where shallow groundwater has a higher potential for infiltration, and isolated six sub-basin areas that appeared to be adversely influenced from two rainfall events that occurred in the winter of 2001.

Based on these findings, it is recommended the City conduct a continuous dry weather flow metering analysis through a high/low tide cycle to precisely evaluate the response to daily low and high tide conditions in the harbor area, perform a video inspection program to verify underground utility pipeline conditions and document the presence of any illegal storm drainage connections to the wastewater system, coordinate with OCSD for additional data and findings of its ongoing I&I evaluation in the City's service area, and perform additional I&I (flow isolation) analysis in the six identified areas of the City.

The combination of these proactive activities by the City would provide an effective and methodical implementation strategy for the City's I&I Reduction Program. The implementation strategy integrates the master plan work activities, focuses on the identified potential I&I problem areas in a prioritized manner, and concludes with the need to conduct specific subsequent Sanitary Sewer Evaluation Studies (SSES) to mitigate potential sources of I&I in the collection system. This activity could be scheduled during the winter of 2003-04 to better utilize the OCSD and video inspection data.

COLLECTION SYSTEM FINDINGS AND RECOMMENDATIONS

Hydraulically, the City's primary collection system generally appears to have adequate capacity as this master plan identified a minimal number of facilities with inadequate hydraulic capacity. It is recommended that approximately 13,700 linear feet of the evaluated collection system be upsized to increase local capacity. The estimated cost to replace the primary hydraulically inadequate collection system pipelines is approximately $2.6 million (Table 7-5). Approximately 33,000 linear feet of additional pipelines were identified to have restricted capacity under future build out conditions and conservative evaluation design criteria. While these facilities are not programmed to be replaced, the City should consider increasing the capacity of these pipelines during its scheduled systematic facility repair and replacement program. In light of new Federal and State regulations, criteria for determining pipe capacity has been conservatively set to allow for unanticipated blockages or diversion of other flows such as storm water.

A second important consideration of the collection system evaluation is the need to further define system condition and proactively plan for infrastructure reinvestment. As previously discussed, the City has recognized this need, has programmed for a comprehensive video inspection of the entire underground wastewater utility system, and has adopted a dedicated funding source to assure its implementation. Implementation of the video inspection findings will be in the form of annual collection system repair or replacement projects. It is presumed that most of these facilities will be rehabilitated through applicable trenchless rehabilitation technologies.

PUMPING SYSTEM FINDINGS AND RECOMMENDATIONS

While the City has been proactive in the ongoing maintenance of its wastewater lift stations, many of these facilities are beginning to show their age. As such, the City has programmed for the methodical replacement of all of its wastewater lift stations. This activity is one of the integral components of the City's infrastructure management program and is designed to promote long term system reliability.

Similar to that of the collection system evaluation methodology, lift station improvements can generally be classified as improvements required to increase system hydraulic capacity and improvements to facility condition or reliability.

Capacity related improvements are considered priority project elements that are required to maintain the City's ability to pump wastewater flows. Based on the current and projected wastewater flows, 11 facilities were found to have future pumping capacity deficiencies. The estimated cost of improvements to these priority lift stations is approximately $16.6 million (Table 7-6).

An important component for major reconstruction is the City's goal to convert all of its lift stations to the wet well/dry well configuration, wherever feasible. Reliability would also be enhanced should the City decide to provide dedicated standby power with automatic transfer switches at each facility instead of the current portable generator strategy. Since dedicated standby power provisions requires additional on-site facilities, the feasibility and cost effectiveness of this decision should be made on a site specific basis.

CAPITAL IMPROVEMENT PROGRAM SUMMARY

Capital improvements are prioritized to meet the short- and long-term goals of the wastewater utility. Short-term project priorities are based on facilities with severe capacity deficiencies, system safety concerns, and other utility management objectives. The improvements and recommendations derived during the conduct of the City's Sewer System Master Plan are summarized as follows:

- Collection System – Overall, it is recommended that the City continue its proactive annual investment in the collection system in a methodical manner. Video inspection of the system to identify actual field conditions, potential for failure, and actual underground material is suggested to be a medium-high priority, with identified significant deficiencies a high priority. Additionally, the hydraulic capacity deficiencies are generally high priority, while the I&I component should be scheduled as a medium to medium-high priority, depending on implications of additional local I&I studies.

- Lift Stations – Lift station improvements are generally important priority projects as their failure often has a high potential for sewer spills. Accordingly, lift station capacity, reliability, and safety improvements are high priority.

Prioritization of the recommended improvements should be based on the degree of deficiency, facility reliability related to the potential for and implications of failure, the potential for higher future flows, coordination with other utility needs and objectives, and funding availability. As such, the City should balance its capital improvement program between the lift station replacement program, collection system facilities identified with a high potential for failure, and hydraulic pipeline deficiencies, with the lift station replacement program and potential pipeline failures receiving the most attention. Due to the nature of the improvements, most of these projects should be constructed during the next 10 years.

SEWER FACILITY CHARGES

The City utilizes a Sewer Facility Charge (SFC), commonly referred to as a connection fee, to recover the costs of facilities to be constructed in the future that will benefit new development. The purpose of this charge is to assure that future customers pay their fair share of the costs of the system's capacity. As such, a Sewer Facility Charge equitably distributes facility costs to future users based on their anticipated demands on the wastewater system. The assets that collect and pump the City's wastewater are the basis for the cost of capacity in the sewer system.

The City ordinance applicable to SFCs is contained in Chapter 14.36 of the City's Municipal Code. The current and updated residential sewer facility charges are based on an "equivalent dwelling unit" or EDU. For consistency with the current sewer user charge rate schedule, the updated non-residential charge is also proposed to be converted from a cost per 1000 square feet to an EDU basis.

There are several generally accepted methods commonly used to develop capital facility charges. A common approach selected by the City for the development of this fee is referred to as the incremental approach. The incremental approach is based on quantifying the future costs of additional capacity and unitizing these costs by the incremental quantity of additional demand served by these costs. Accordingly, the capital improvement program provides the primary basis of costs, while the estimation of future flows provides the basis for future incremental wastewater flows.

Costs of Future Capacity

Under the incremental approach, the cost of future capacity in the City's wastewater system is based on two facility components. These include the future replacement costs of the sewer lift stations and new local sewer collection system improvements.

Several key considerations were discussed with City staff related to assessing the cost of lift station improvements to future customers. Since the ongoing sewer user charge was designed to provide for the methodical replacement of the City's lift stations, only the specific portion of the capacity related facility improvement costs and metering enhancements is un-funded. As such, the costs allocated to future customers are limited to these cost elements. The total cost of lift station improvements that is included in the cost of future capacity is approximately $9.6 million (Table 8-1).

Similar to the lift station cost allocation approach, a discussion focused on the level of collection system improvements that should be borne by future services. Through these discussions, several alternatives were developed to recover collection system costs from future services. The basic alternatives derived herein are as follows:

- Alternative 1 – Total System Replacement Cost - Include the total cost of all capacity improvements based on the replacement cost of each facility

- Alternative 2 – Total System Upsizing Cost - Include the total cost of all capacity improvements and reduce this cost by the estimated cost to slipline each pipeline segment (at original diameter)

The resulting collection system costs associated with these alternatives are approximately $8.0 and $4.0 million for Alternatives 1 and 2, respectively (Table 8-2).

Estimated Future Incremental Wastewater Flows and Unit Flow Factors

Consistent with the study methodology, the estimated wastewater flow was derived from the output of the hydraulic model under current and future wastewater loading conditions. The incremental value that is the result of future growth is the difference between the future and existing wastewater flows. Based on the findings of the hydraulic model, the incremental increase in future wastewater flow was estimated to be approximately 1.95 million gallons per day (MGD).

In addition to the development of future incremental flows, wastewater flows factors are derived for each of the residential and commercial/industrial user classes. The City's 2001 Sewer Charge Study estimated that the average wastewater discharge of a Single Family dwelling (SFD) is 226 gallons per day (gpd). Additionally, the 2001 Sewer Charge Study estimated the discharge for a Multi Family dwelling and a commercial/industrial customer to be 185 gpd and 257 gallons per Equivalent Dwelling Unit (EDU), respectively.

Unit Costs Of Service and Alternative Sewer Facility Charges

The development of unit costs of service is an essential step in deriving cost of service based Sewer Facility Charges. Unit costs of service are obtained by correlating the costs associated with future growth with the incremental addition of future sewer system discharges (Table 8-4). Sewer Facility Charges are calculated by correlating the wastewater demand characteristics of the City's primary residential and non-residential user classes with the estimated unit costs of service (Table 8-5). The resulting alternative Single Family Dwelling charges are as follows:

- Alternative 1 - $2,043
- Alternative 2 - $1,579

To evaluate these charges, a comparison of the City's current and alternative residential Sewer Facility Charges with neighboring communities was performed. The alternative Sewer Facility Charges compare favorably with the rates of surrounding communities as the findings ranged from approximately $1,400 to $2,000 (Table 8-6). As discussed with City staff, it is recommended the City adopt one of the alternative facility charges so that growth cost are adequately recovered from future wastewater system customers.

Chapter 1

Introduction

CHAPTER 1

INTRODUCTION

This chapter presents the background, authorization, objectives and scope of work for the Sewer System Master Plan and associated Inflow and Infiltration (I&I) Study.

BACKGROUND

The City of Huntington Beach (City) is an urban city with a population of approximately 200,000 residents. Although the City last conducted a Wastewater System Master Plan in April 1995, the last study approved by the City Council was in 1978. Based on the need to routinely re-evaluate the infrastructure's to meet future demands, the City has requested this update to the 1995 Wastewater System Master Plan. The planning period will be from present to the year 2020.

The City is responsible for operating and maintaining approximately 360 miles or 1,900,000 feet of wastewater collection system. This system predominately consists of 8-inch pipelines supported by 27 lift stations varying in capacity from approximately 80 gallons per minute (gpm) to 1,350 gpm. By contract, sewage collection, treatment, and disposal are the responsibility of the Orange County Sanitation District (OCSD).

AUTHORIZATION AND OBJECTIVES

In recognition of the need to plan for future development and provide uninterrupted wastewater service, the City authorized Kennedy/Jenks Consultants (K/J) to prepare this 2002 Sewer System Master Plan and associated (I&I) Study. This planning effort incorporated the following key objectives:

- Develop a project approach with the City as a key team member to support project data gathering, criteria development and wastewater system evaluation.
- Conduct sufficient data review to verify adequacy of existing data for evaluation of the existing wastewater and pumping systems.

- Evaluate the adequacy of the existing collection and pumping system to meet current and future wastewater flows.
- Develop a correlating linkage between the City's customer accounts as listed in the utility billing system and the Assessor Parcel Number (APN) as listed in the County Assessor database.
- Prepare a desktop I&I study to review and evaluate available data to assist in identifying potential I&I problem areas.
- Perform a limited flow monitoring program to obtain actual field measurements of wastewater flow conditions in various sites in the city. Analyze the input of rainfall dependent I&I based on wet weather data obtained during this program.
- Utilize actual billing system demand data in the development of a schematic hydraulic model for system evaluation.
- Formulate an easily implemented master plan with prioritized capital improvements.

SCOPE OF WORK

The scope of work for this Sewer System Master Plan and associated I&I Study are organized by tasks summarized as follows:

Wastewater Master Plan

Task 1 – Review New or Revised Planning and Facility Data

Subtask 1.A. – Establish Project Goals
Subtask 1.B. – Evaluate Current Planning and Engineering Documents
Subtask 1.C. – Evaluate Current Sewer Flow Criteria
Subtask 1.D. – Assess Non-Huntington-Beach-Generated-Sewage Flows

Task 2 – Flow Criteria Development and Flow Projections

Subtask 2.A. – Review Previous Population Projections

Subtask 2.B. – Review Flow Criteria and Flow Projection

Subtask 2.C. – Develop Design and Unit Cost Criteria

Task 3 – Sewer System Configuration and Model Development

Subtask 3.A. – Revise Sewer System Modeling Schematic

Subtask 3.B. – Update Tributary Area Map

Subtask 3.C. – Develop Sewer Flows and Model Calibration

Task 4 – Sewer System Analysis

Subtask 4.A. – Identify Sewer System Deficiencies

Subtask 4.B. – Model and Analyze Existing Sewer System

Subtask 4.C. – Compile Existing System Deficiencies

Subtask 4.D. – Model and Analyze Future Sewer System

Subtask 4.E. – Recommend Sewer System Improvements

Subtask 4.F. – Recommend Lift Station System Improvements

Task 5 – Submit Sewer Master Plan Document

Perform Desktop I&I Study

Task 1 – Data Collection and Review

Task 2 – Overlay Analysis Program

Subtask 2.A. – Overlay Lift Station Evaluation

Subtask 2.B. – Overlay Sewer Pipelines/Manholes Evaluation

Task 3 – Perform Assessor Parcel Number Based Flow Evaluation Program

Subtask 3.A. – Perform Assessor Parcel Number/Utility Account Correlation
Subtask 3.B. – Contrast Evaluation of Water/Sewer Demands with Known Discharges

Task 4 – Prepare Desktop Evaluation Report of Findings

Perform Field Flow Monitoring and I&I Evaluation Programs

CONDUCT OF STUDY

The information used to prepare this study includes review of existing information, development of new and/or updated data, City-provided data from its Geographical Information System (GIS), and discussions with City staff. Initial study tasks focused on collecting and evaluating relevant data, reports, and other available information to define existing conditions and identify future considerations. Based on this information, an assessment of the adequacy of the existing primary system was made and improvements were recommended to meet current and future requirements.

ABBREVIATIONS AND DEFINITIONS

The following abbreviations are used within the report:

ac	acre
ADD	Average Day Demand
ADWF	Average Dry Weather Flow
APN	Assessor Parcel Number
AWWF	Average Wet Weather Flow
cf	cubic feet
cfs	cubic feet per second
City	City of Huntington Beach
D/d	Depth to diameter

dia.	diameter
DU	Dwelling Unit
DWR	California Department of Water Resources
EDU	Equivalent Dwelling Unit
ENR	Engineering News Record
EPA	United States Environmental Protection Agency
GIS	Geographic Information System
gpad	gallons per acre day
gpm	gallons per minute
gpd	gallons per day
HGL	Hydraulic Grade Line
hp	horsepower
I&I	inflow and infiltration
in	inches
idm	inch-diameter miles
K/J	Kennedy/Jenks Consultants
LF	linear foot
MG	million gallons
MGD	million gallons per day
NOAA	National Ocean and Atmospheric Administration
NWS	National Weather Service
O&M	Operations and Maintenance
OCSD	Orange County Sanitation District
PDWF	Peak Dry Weather Flow
RPM	Revolutions Per Minute
pph	persons per household
psi	pounds per square inch
PWWF	Peak Wet Weather Flow
RWQCB	Regional Water Quality Control Board
SCAG	Southern California Association of Governments
SFC	Sewer Facility Charge
sf	square feet
TDH	Total Dynamic Head
VCP	vitrified clay pipe

Chapter 2

History and Study Area Characteristics

CHAPTER 2

HISTORY AND STUDY AREA CHARACTERISTICS

This chapter presents the growth history of the City and identifies key study area characteristics such as geography, climate, boundary limits, land use, and population. Information sources include the previous wastewater master planning efforts performed for the City, the City's Water Master Plan dated December 2000, and updated land use planning information provided by the City.

HISTORY

Founded in the late 1880's, Huntington Beach was incorporated on 17 February 1909. The history of Huntington Beach extends from the early days of Orange County as a cluster of Spanish ranchos, through the oil boom of the 1920's, to its current status as California's 11th largest city. Through a series of annexations, the City has grown to approximately 27 square miles. As the City has become one of the leading commercial, industrial and recreational centers of Orange County, the population has swelled from 11,000 in 1960 to approximately 200,000.

CITY BOUNDARY AND STUDY AREA DESCRIPTION

The City is located on the shore of the Pacific Ocean in northwestern Orange County. It is surrounded by Westminster to the north, Fountain Valley to the northeast, Costa Mesa to the east, Newport Beach to the southeast, Seal Beach and the U.S. Naval Weapons Station to the northwest, and the Pacific Ocean to the west. The City of Los Angeles is located approximately 35 miles to the northwest and the City of San Diego is approximately 95 miles to the southeast.

The study area includes areas within the City boundary and small tributary portions of the Cities of Westminster, Seal Beach, Newport Beach, and Fountain Valley. These small areas are served through direct connections to the wastewater collection system of the City, and have been included for evaluation purposes. Due to local topography, some areas within the City are served through a connection to the wastewater system of the City of Fountain Valley and are not included in the evaluation.

Geography and Climate

The City contains approximately 17,206 acres, or 27 square miles, of land. Ninety-seven percent of the land is developed with residential, commercial, industrial, public, mixed uses, open space, and right-of-ways/bridges. The remaining three percent of land is vacant. The terrain is essentially flat and generally slopes westward to the white sand beaches of the Pacific Ocean. Elevations vary from sea level to approximately 200 feet above sea level.

The City has a mild Mediterranean-type climate. Prevailing westerly and southwesterly winds off of the Pacific Ocean help maintain pleasant weather year-round. The mean annual temperature is 62 degrees Fahrenheit, mean annual humidity is 64.7 percent, and annual rainfall is slightly less than 12 inches.

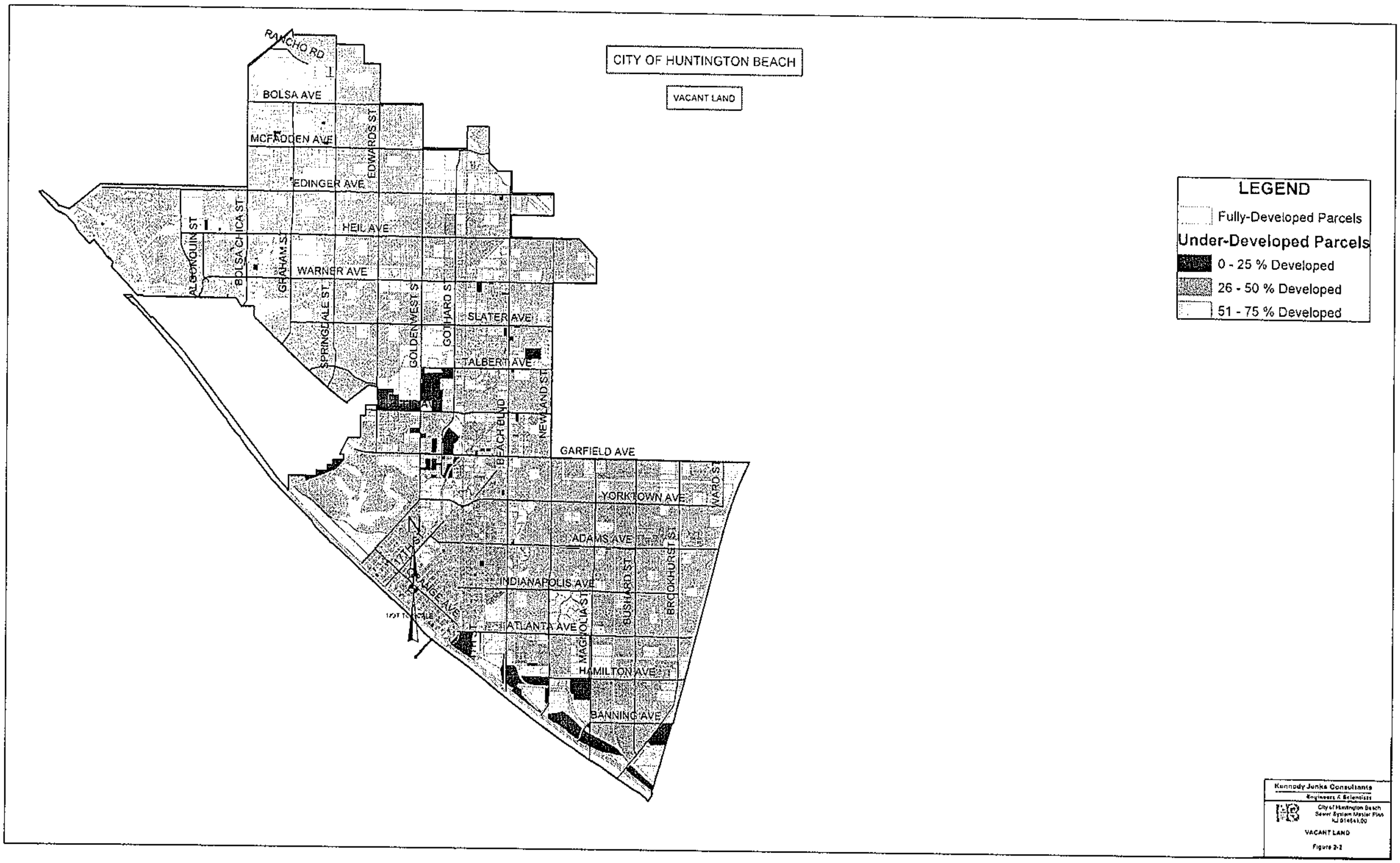
LAND USE

Residential use is the largest single land use in the City. Most residential uses are single-family homes located within super blocks. The major commercial areas in the City are the Huntington Beach Center, Lohmann's Five Points Plaza, Old World Village, Guardian Center, Peter's Landing, the Beach Boulevard corridor, and Downtown. Industrial centers are generally located in the northwest area of the City and along Gothard Street. Vacant land is minimal in the City, as evidenced by the reclaiming and remediation of oil production land for residential, commercial, and industrial uses.

Existing Land Use and Development

While the land use element of the City's General Plan has changed over time, the existing land use generally corresponds to the General Plan guidelines. The exception to this condition is vacant land.

SOURCE CITY OF HUNTINGTON BEACH GENERAL PLAN


CITY OF HUNTINGTON BEACH
VACANT LAND
LEGEND
Fully-Developed Parcels
Under-Developed Parcels
0 - 25 % Developed
26 - 50 % Developed
51 - 75 % Developed
RANCHO RD
BOLSA AVE
MCFADDEN AVE
EDWARDS ST
EDINGER AVE
HEIL AVE
ALGONQUIN ST
BOLSA CHICA ST
GRAHAM ST
WARNER AVE
SPRINGDALE ST
GOLDENWEST ST
GOTHARD ST
SLATER AVE
TALBERT AVE
BEACH BLVD
NEWLAND ST
GARFIELD AVE
YORKTOWN AVE
WARD ST
ADAMS AVE
INDIANAPOLIS AVE
BUSHARD ST
BROOKHURST ST
MAGNOLIA ST
ATLANTA AVE
HAMILTON AVE
BANNING AVE
Kennedy Jenks Consultants
Engineers & Scientists
City of Huntington Beach
Sewer System Master Plan
KJ 014641.00
VACANT LAND
Figure 2-2

A representative map of the City's General Plan land use is shown on Figure 2-1. This map illustrates the land usage type of all City land at buildout conditions. The location of vacant land is graphically depicted on Figure 2-2.

Existing acreage within the City is partitioned among the major land use categories in accordance with the General Plan. The City's existing land use information is summarized in Table 2-1. As shown, residential usage is the predominant land use type and comprises approximately forty-six percent of the total usage. Industrial, commercial, and mixed use land use categories account for an additional fifteen percent, while public, open space, and right-of-ways and bridges represent an additional thirty-nine percent. As shown, approximately twenty-four percent of the vacant land is planned to be built out as residential land uses.

Future Land Use and Development

Although nearly all land within the study area is developed (ninety-seven percent), there is still potential growth in two forms: development of vacant land and land use intensification. When all areas are developed to maximum allowable densities, "buildout" will occur. As discussed, the future land usage type according to the City's General Plan is shown on Figure 2-1.

Vacant Land Development. As previously discussed, there is minimal vacant land within the City. Vacant land is anticipated to develop in accordance with the General Plan and should have minimal impact on future wastewater utility service requirements.

Based upon discussions with the City's Planning Department, there are four noteworthy Specific Plans (SP) within the City that will impact local development/redevelopment. These are the Crossings at Huntington Beach SP, the Palm/Goldenwest SP, the Meadowlark SP, and the McDonnell Centre Business Park SP. These specific plans are summarized as follows:

TABLE 2-1
EXISTING LAND USE

Land Use Type (Alphabetical)	Total Planned (Acres)	% of Total Vacant (Acres)	Vacant (Acres)	% of Total Land (Acres)
Commercial General	614.61	2.30	13.00	3.57
Commercial Neighborhood	93.64	0.41	2.30	0.54
Commercial Office	41.29	0.00	0.00	0.24
Commercial Regional	136.84	0.00	0.00	0.80
Commercial Visitor	73.47	3.83	21.65	0.43
TOTAL COMMERCIAL	959.85	6.54	36.95	5.58
Industrial	1,171.78	5.84	32.98	6.81
Mixed Use	176.53	0.00	0.00	1.03
Mixed Use Horizontal	201.74	9.98	56.40	1.17
Mixed Use Vertical	52.67	0.28	1.56	0.31
TOTAL MIXED USE	430.94	10.26	57.97	2.50
Open Space - Commercial Recreation	237.72	0.00	0.00	1.38
Open Space - Conservation	127.07	18.01	101.80	0.74
Open Space - Park	638.73	26.51	149.79	3.71
Open Space - Shore	342.51	0.00	0.00	1.99
Open Space - Water Recreation	242.79	0.00	0.00	1.41
TOTAL OPEN SPACE	1,588.82	44.52	251.59	9.23
Public	1,639.88	9.21	52.06	9.53
Residential Low Density	5,681.46	1.87	10.57	33.02
Residential Medium Density	1,123.36	16.77	94.74	6.53
Residential Medium High Density	104.36	1.14	6.43	0.61
Residential High Density	1,004.69	3.86	21.81	5.84
TOTAL RESIDENTIAL	7,913.87	23.63	133.55	46.00
Right of Ways & Bridges	3,501.01	0.00	0.00	20.35
CITY TOTALS	**17,206.15**	**100.00**	**565.10**	**100.01**

Source: City of Huntington Beach GIS Data, 2001

The Crossings at Huntington Beach SP #13 - Adopted July 5, 2000, Designated Commercial Regional – 63 acres. The area is generally bounded on the north by Center Avenue, on the east by Beach Boulevard, on the south by Edinger Avenue, and on the west by Southern Pacific railroad right-of-way.

Palm/Goldenwest SP #12 - Adopted February 7, 2000, 50 acres. The Palm/Goldenwest Specific Plan Area encompasses the 150 acre site bounded by Palm Avenue to the north, Pacific Coast Highway to the south, Goldenwest Street to the east and Seapoint Street to the west, with approximately 4 acres located on the west side of Seapoint.

Meadowlark SP #8 - Adopted March 15, 1999. The Meadowlark Specific Plan encompasses approximately 65 acres of land located approximately 600 feet north and east of the intersection of Bolsa Chica Street and Warner Avenue.

McDonnell Centre Business Park SP #11 - Adopted October 6, 1997. The McDonnell Centre Business Park Specific Plan covers 507 gross acres located in the northwestern portion of the City. The area is generally bounded on the north by Rancho Road and the U.S. Navy railroad right-of-way (excluding the City's water reservoir site), on the east by Springdale Street, on the south by Bolsa Avenue and on the west by Bolsa Chica Street. The McDonnell Centre Business Park is presently zoned limited industrial ("IL") and limited industrial with a high rise overlay ("IL-H"), in designated areas.

In addition to these areas, the Bolsa Chica Specific Plan is located within the sphere of influence of the City. Bolsa Chica encompasses 1,654 additional acres of unincorporated land. When developed, this area will not utilize any City owned wastewater facilities. As wastewater flows from Bolsa Chica it will discharge directly into OCSD pipelines and be conveyed to the existing OCSD Slater Avenue Lift Station. Since the Slater Lift Station has already been redesigned to accommodate the future development of the Bolsa Chica Specific Plan, this area was not evaluated in this study.

Land Use Intensification. Land use intensification occurs in undeveloped areas that are designated for urban/suburban uses and on sites where existing uses were developed at densities below those permitted by the City. Nonconforming uses occur when existing parcels were developed with uses not permitted by current zoning. For master planning purposes, future City redevelopment from intensification rather than conformance to zoning is most likely to affect future wastewater loading conditions.

Maximum General Plan Buildout. Maximum buildout is defined to occur when vacant parcels develop and existing parcels redevelop to their maximum permitted densities. This theoretical condition almost never occurs, as the majority of developed parcels are physically stable and the future economic viability of redevelopment to increase land use density is unlikely. Accordingly, maximum general plan buildout must be considered for utility master plans and results tempered with engineering judgment when assessing potential impacts of redevelopment. The implication of buildout conditions is integrated in future wastewater generation factors derived in subsequent chapters.

POPULATION AND GROWTH

The population of the City has increased by 4.3 percent between 1990 and 2000, as indicated by the 2000 U.S. Census of Population. At last count in 2000, the City had approximately 190,000 residents. This value represented an increase of approximately 8,000 people since 1990.

As shown in Table 2-2, the City has experienced a lower rate of growth over the last twenty years than both Orange County (County) and the State of California (State). While the average annual population increase in California was 2.1 percent from 1970 to 1980, 2.3 percent from 1980 to 1990, and 1.3 percent from 1990 to 2000, Orange County experienced annual growth rates of 3.1, 2.2, and 1.7 percent respectively. During these same time periods, the City experienced average annual increases of 3.9, 0.63, and 0.44 percent respectively. In contrast, the City's growth was substantially greater than the County or the State during the 1970 to 1980 time period.

Table 2-3 provides a summary of projected population, dwelling units and persons per household (pph) for the City through the year 2020. While several sources of projection were reviewed, evaluation of the projection findings indicates a general consensus between the City's Water Master Plan dated December, 2000, and the Southern California Association of Governments (SCAG).

Based on these data, the population of the City is expected to grow from approximately 200,000 to approximately 225,000 by the year 2020. This increase in population is projected to occur from the construction of an additional 1,000 dwelling units and an overall increase in the number of persons residing in each household. Of these two factors, the increase in persons per household is projected to have the greater impact on future infrastructure requirements.

TABLE 2-2
POPULATION COMPARISONS

Location	1970	1980	1990	2000
		Population by Year		
California	19,241,000	23,668,145	29,760,021	33,871,648
Orange County	1,421,000	1,932,709	2,410,556	2,846,289
Huntington Beach	115,960	170,505	181,519	189,594

	Compound Annual Growth Rate		
	1970-1980	1980-1990	1990-2000
California	2.09	2.32	1.30
Orange County	3.12	2.23	1.68
Huntington Beach	3.93	0.63	0.44

Source: 1970, 1980, 1990, 2000 U.S. Census

TABLE 2-3
CITY POPULATION PROJECTIONS

Source of Data	2005	2010	2015	2020
	Population Projections By Year			
Ctr. for Demographic Research, CSUF	209,203	210,612	210,021	210,053
So. Cal Association of Gov. (SCAG)	--	215,800	220,100	223,100
City Dept. of Economic Development	204,500	--	--	--
City Water Master Plan, Dec. 2000	211,412	216,020	220,554	224,410
	# of Dwelling Units Per Year			
So. Cal Association of Gov. (SCAG)	--	74,400	75,200	75,700
City Water Master Plan, Dec. 2000	78,376	78,937	79,664	79,819
	Persons Per Household (PPH) By Year			
So. Cal Association of Gov. (SCAG)	--	2.9005	2.9269	2.9472
City Water Master Plan, Dec. 2000	2.6974	2.7366	2.7686	2.8115

Source of Data	% Increase in # of DU From Year	
	2005 TO 2020	**2010 TO 2020**
So. Cal Association of Gov. (SCAG)	--	1.0175
City Water Master Plan, Dec. 2000	1.0184	1.0112

Source of Data	% Increase in PPH From Year	
	2005 TO 2020	**2010 TO 2020**
So. Cal Association of Gov. (SCAG)	--	1.0161
City Water Master Plan, Dec. 2000	1.0423	1.0274

Source: As noted

Chapter 3

Existing Wastewater System Description

CHAPTER 3

EXISTING WASTEWATER SYSTEM DESCRIPTION

The focus of this chapter is a discussion and description of the City's existing wastewater system. Key elements and features of these wastewater facilities are described and evaluated in subsequent chapters of this study. In addition, a narrative summary of the developments and enhancements to the City's Geographic Information System (GIS) performed within this study is provided herein.

GEOGRAPHIC INFORMATION SYSTEM (GIS)

A supplemental element of the project focused on the development of additional information to support the City's GIS. The City's GIS uses Environmental Systems Research Institute's (ESRI) ARC/INFO on the SUN Solaris platform with supporting functionality provided on the Intel/Microsoft platform. Data was received from the City's GIS group for the beneficial use of this study.

In consideration of the overall project requirements, the primary objective of this element of the project was to utilize available City-provided GIS information and expand or enhance the depth and breath of the data delivered back to the City. This information could then be utilized to enhance future GIS activities and provide additional wastewater utility management information. The information provided back to the City through the conduct of this project is summarized below.

<u>APN to Utility Billing System Linkages</u> – The development of a linkage between City parcel polygons by Assessor Parcel Number (APN) and the utility billing system by Account Number. Parcels with APN source data discrepancies were also identified.

<u>Wastewater Utility Pipeline Data</u> - New or updated data related to pipeline length, slope, diameter, inverts, ground elevation, system connectivity, estimated available capacity, and a parcel to pipeline linkage that could be used to schematically represent lateral connections.

GENERAL SYSTEM OVERVIEW

The City owns, operates and maintains a wastewater collection system that includes gravity pipelines, manholes, lift stations and force mains. This system serves over 95 percent of the areas within the City, and several small areas within the Cities of Westminster, Seal Beach, Newport Beach, and Fountain Valley. Collected wastewater is conveyed to the Orange County Sanitation District's (OCSD's) system at multiple connections within the City. The collected wastewater is ultimately conveyed to OCSD's local wastewater treatment plant.

The facility evaluation elements of this master plan focus on a hydraulic evaluation of the existing collection system pipelines and lift stations, and a general reliability assessment of these facilities through the assessment of an appropriate on-going capital repair and replacement program. The City's wastewater system and facilities are discussed in the following sections including those of OCSD and private systems.

Drainage Basin and Sub-Basin Delineation

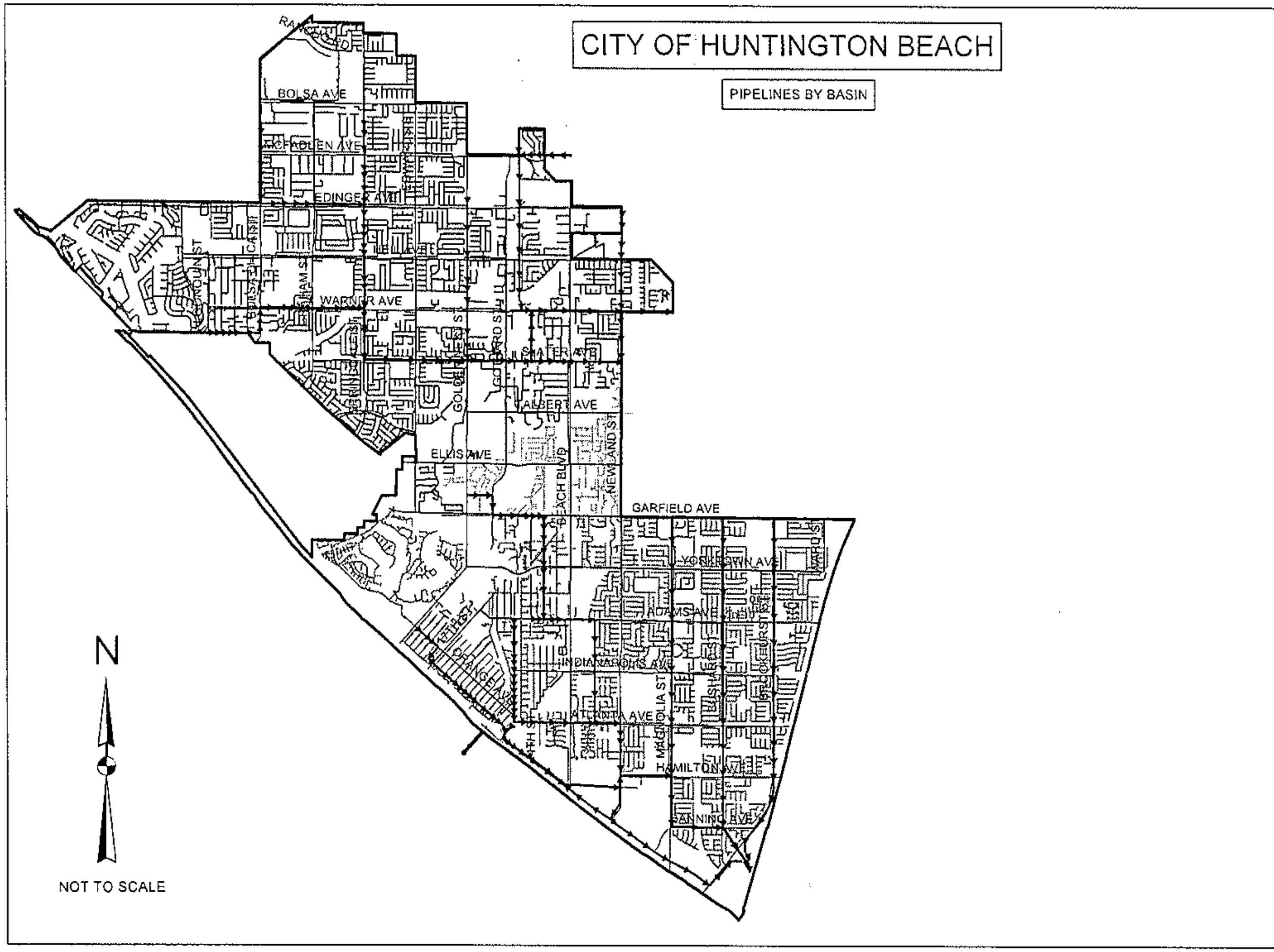
City and non-city areas that are served by the City's collection system are located within four (4) major geographical drainage basins. The general relationship of these basins within the City and their direction of flow to OCSD facilities are indicated in Figure 3-1.

City Wastewater Pipelines

The City's gravity collection pipelines vary in size from 6 to 30-inches in diameter, with most pipelines being 8-inches in diameter. Approximately 1.9 million lineal feet of city-owned wastewater collection pipelines are in service. Pipeline materials are predominantly vitrified clay pipe (VCP) with some polyvinylchloride (PVC), and ductile iron pipe (DIP). A small percentage of the system has also been rehabilitated or lined to increase facility life. A summary of the length and diameter of the City's underground wastewater collection system is shown in Figure 3-2 and listed in Table 3-1. This information is based primarily on the data provided in the City's GIS.

CITY OF HUNTINGTON BEACH

PIPELINES BY BASIN



LEGEND

BASIN

	A
	B
	C
	D
	OCSD Line

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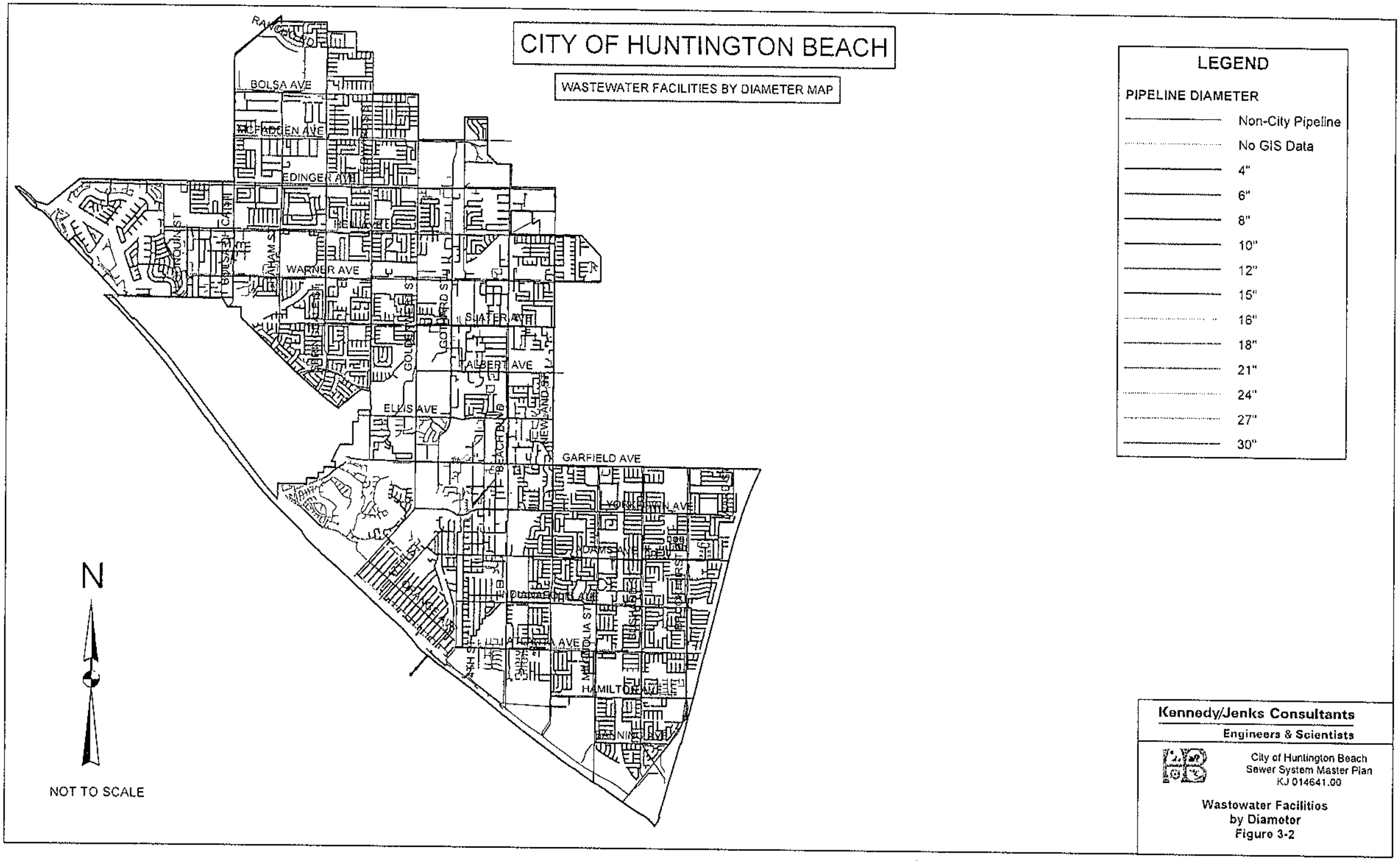
Pipelines by Basin
Figure 3-1


CITY OF HUNTINGTON BEACH
WASTEWATER FACILITIES BY DIAMETER MAP
LEGEND
PIPELINE DIAMETER
Non-City Pipeline
No GIS Data
4"
6"
8"
10"
12"
15"
16"
18"
21"
24"
27"
30"
BOLSA AVE
EDINGER AVE
WARNER AVE
SLATER AVE
ELLIS AVE
GARFIELD AVE
N
NOT TO SCALE
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Wastewater Facilities
by Diameter
Figure 3-2

TABLE 3-1

WASTEWATER COLLECTION SYSTEM PIPELINES BY DIAMETER

Sewer Line Size (in.)	Length (LF)
No Data *	4,190
6	8,280
8	1,568,100
10	112,490
12	72,770
15	51,110
16	4,360
18	16,920
21	6,730
24	1,320
27	5,400
30	1,310
Grand Total	1,852,980

* Includes pipes with no data or uncertain 4" values.

Two basic types of manholes were used in the construction of the City's wastewater collection system. The older manholes were constructed of brick and were founded on cast-in-place concrete bases. This manhole type was typical of the downtown area. More recent manhole construction projects used precast concrete sections also founded on cast-in-place bases.

City Wastewater Lift Stations

The City owns, operates and maintains twenty-seven (27) wastewater lift stations that lift sewage from low points in the collection system to manholes at higher locations. As reflected in the City's 2001 Sewer Lift Station Design Manual, there are two types of approved lift station facilities. A typical sketch of each type of lift station as provided in the City's design manual is provided in Appendix A. Each facility type is described in the following sections.

- Wet Well/Dry Well Lift Stations. This type of lift station is the most commonly used type of lift station in the City's collection system. Wet wells are generally constructed of concrete manhole rings and generally vary from 4 to 8 feet in diameter. Dry wells are typically constructed of concrete and are either round or rectangular in shape. Round dry wells are usually comprised of two levels separated by metal grating. The pumps, motors, and valves are located on the lower level and the electrical equipment is situated on the upper level. One lift station, Oceanhill & Beach, is an exception to this and is a round fiberglass lift station with all the interior equipment on one level. Recent modifications and upgrades at some of the lift stations include above ground panels, adapters for portable generators, automatic lighting, entry alarms, continuous ventilation, and interior coating of the wet wells.

- Submersible Lift Stations. There are three submersible lift stations in the City's collection system (Atlanta east of Beach, Algonquin/Boardwalk, and PCH in Sunset Beach). The lift station at PCH in Sunset Beach, Lift Station "A," was recently reconstructed with the submersible configuration due to site constraints. All submersible lift stations are constructed of concrete and are eight feet in diameter. Each lift station is equipped with two submersible pumps, above ground control panels, and a valve vault.

Table 3-2 presents the general information and the rated pump capacity of the City's lift stations, based on data provided by the City's Maintenance Department. It should be noted that the actual pump capacity of a given station may vary from the rated capacity due to factors such as the age and condition of the impellers, motors, and piping in each facility. The location of these facilities is shown on Figure 3-3. The connectivity of the lift stations that are influenced by upstream facilities is depicted on Figure 3-4.

TABLE 3-2

LIFT STATION INFORMATION

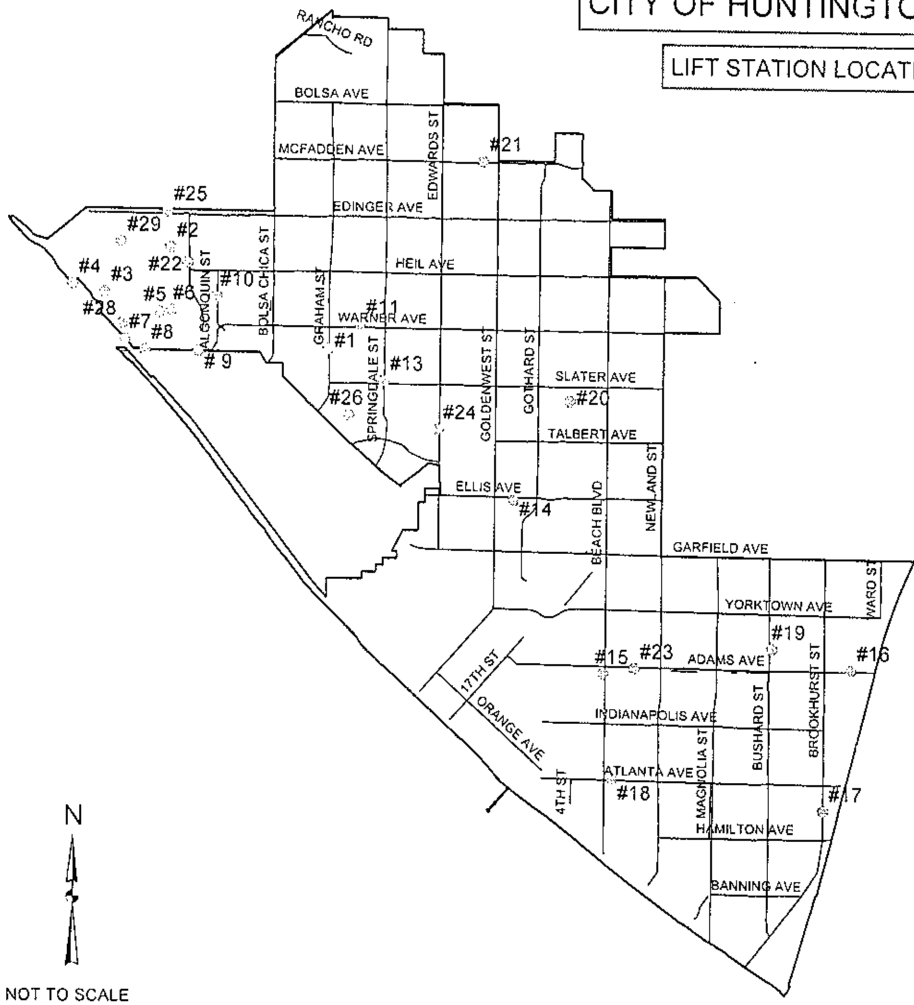
Number	Name	Number and Name	# of Pumps	Manufacturer & Model No.	Motor Data (H.P.@ RPM)	Impeller Diamter (in)	Rated Pump Capacity (gpm @ TDH)
1	Graham & Kenilworth	#1 Graham	2	Wemco 4 x 11	20 @ 1800	9"	580 @ 55'
2	Humbolt & Wayfarer	#2 Humboldt	2	Wemco 4 x 11	3 @ 1170	7-5/8"	155@ 22'
3	Gilbert & Peale	#3 Station "E"	2	Wemco 4 x 11	3 @ 1160	7-1/2"	100 @ 18'
4	PCH in Sunset Beach	#4 Statioin "A"	2	Wemco 4 x 11	10 @ 1160	10"	750 @ 20'
5	Davenport & Baruna	#5 Davenport	2	Wemco 4 x 11	3 @ 1200	8"	106 @ 12'
6	Edgewater & Davenport	#6 Edgewater	2	Wemco 4 x 11	5 @ 1170	9"	450 @ 12'
7	PCH West of Warner	#7 Station "B"	2	Wemco 4 x 11	7.5 @ 1170	8-3/4"	750 @ 10'
8	Warner North of PCH	#8 Station "C"	2	Wemco 4 x 11	25 @ 1800	8-1/2"	1350 @ 15'
9	Warner at Edgewater "D" Station	#9 Station "D"	4	Wemco 4 x 11	25 @ 1760	9-1/2"	900 @ 50'
10	Algonquin & Boardwalk	#10 Algonquin	2	Wemco 4 x 11	40 @ 1745	0-3/4"	1000 @ 60'
11	Lark & Warner	#11 Lark	2	Wemco 4 x 11	2 @ 1170	8"	125 @ 12'
13	Slater & Springdale	#13 Slater	2	Wemco 6 x 11	20 @ 1750	9"	1070 @ 24'
14	Ellis & Gothard	#14 Ellis	3	Wemco 6 x 11	20 @ 1800	8-1/2"	850 @ 34'
15	Oceanhill & Beach	#15 Beach	2	Gorman Rupp	75 @ 1000	9-3/4"	150 @ 30'
16	Adams & Ranger	#16 Adams	2	Wemco 4 x 11	3 @ 1170	8"	270 @ 13'
17	Brookhurst & Effingham	#17 Brookhurst	2	Wemco 6 x 11	30 @ 1750	9"	1280 @ 28'
18	Atlanta East of Beach	#18 Atlanta	2	Wemco 6 x 12	25 @ 1170	6"	350 @ 25'
19	Bushard & Pettswood	#19 Bushard	2	Wemco 4 x 95	3 @ 1170	6-3/8"	338 @ 10'
20	Speer & Crabb	#20 Speer	2	Wemco 6 x 11	15@ 1170	9"	500 @ 14'
21	McFadden & Dawson	#21 McFadden	2	Wemco 4 x 11M	5 @ 1170	9"	550 @ 23'
22	Saybrook & Heil	#33 Saybrook	2	Wemco 4 x 11M	15 @ 1170	9-3/4"	550 @ 23'
23	New Britian & Adams	#23 New Britain	2	Wemco 4 x 11S	5@ 1170	6-3/4"	179 @ 11'
24	Edwards & Balmorol	#24 Edwards	2	Wemco 6 x 11M	20 @ 1750	9"	800 @ 38'
25	Edinger & Santa Barbara	#25 Edinger	2	Wemco 4 x 11M	5 @ 1750	8"	300 @ 12'
26	Brighton & Shoreham	#26 Brighton	2	Wemco 4 x 95	3 @ 1170	7"	220 @ 16'
28	Coral Cay	#28 Coral Cay	2	Wemco 4 x 95	3 @ 1155	6"	80 @ 14'
29	Trinidad & Aquarius	#29 Trinidad	2	Wemco 4 x 11M	10 @ 1750	8"	250 @ 15'

Notes: The City of Huntington Beach does not have a lift station #12 or # 27. Lift station #9 is under design as of June 2002.

Source: Data provided by the City Maintenance Department.

CITY OF HUNTINGTON BEACH

LIFT STATION LOCATIONS



LEGEND

Lift Station

Station Names

- #1 GRAHAM
- #2 HUMBOLDT
- #3 "E"
- #4 "A"
- #5 DAVENPORT
- #6 EDGEWATER
- #7 STATION "B"
- #8 STATION "C"
- # 9 "D"
- #10 ALGONQUIN
- #11 LARK
- #13 SLATER
- #14 ELLIS
- #15 BEACH
- #16 ADAMS
- #17 BROOKHURST
- #18 ATLANTA
- #19 BUSHARD
- #20 SPEER
- #21 MCFADDEN
- #22 SAYBROOK
- #23 NEW BRITAIN
- #24 EDWARDS
- #25 EDINGER
- #26 BRIGHTON
- #28 CORAL CAY
- #29 TRINIDAD

Note: The City does not have Lift Station #12 or #27

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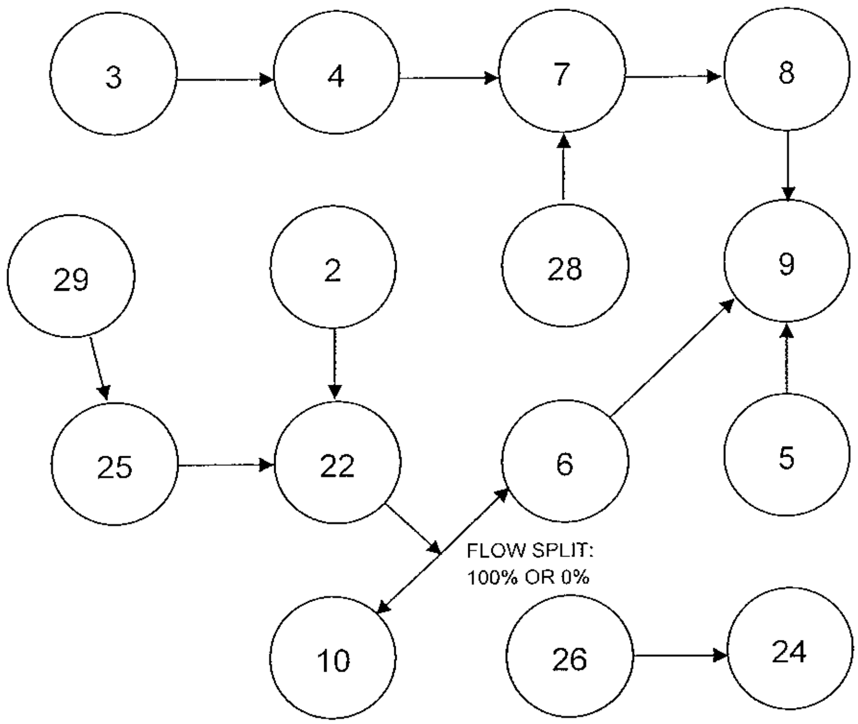
LIFT STATION LOCATION MAP

Figure 3-3

LIFT STATION CONNECTIVITY



NUMBER	NAME
2	Humbolt & Wayfarer
3	Gilbert & Peale
4	PCH in Sunset Beach
5	Davenport & Baruna
6	Edgewater & Davenport
7	PCH West of Warner
8	Warner North of PCH
9	Warner at Edgewater "D" Station
10	Algonquin & Boardwalk
22	Saybrook & Heil
25	Edinger & Santa Barbara
24	Edwards & Balmorol
26	Brighton & Shoreham
28	Coral Cay
29	Trinidad & Aquarius

= Lift Station Number

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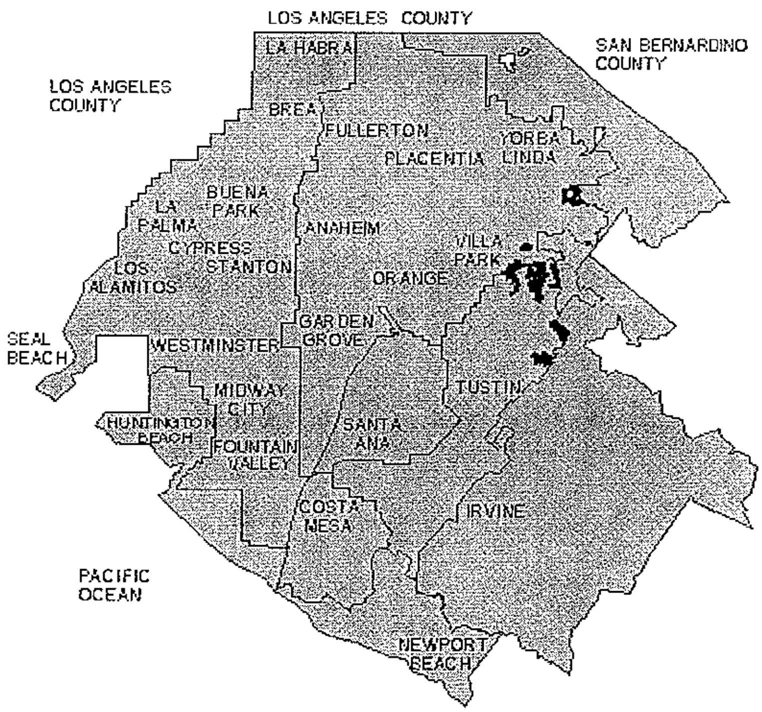
LIFT STATION CONNECTIVITY

Figure 3-4

Orange County Sanitation District Wastewater Facilities

Orange County Sanitation District (OCSD) is responsible for collecting, treating, and disposing of the wastewater generated in central and northwest Orange County. OCSD owns, operates and maintains the majority of the "backbone" wastewater collection trunk pipelines within a 470 square mile area, including the City. OCSD's service area is shown in Figure 3-5.

FIGURE 3-5
OCSD SERVICE AREA



OCSD's regional wastewater pipelines generally range in size from 21 to 108 inches in diameter and collect the City's wastewater at multiple connections. In addition to these collection facilities, OCSD has two lift stations and Wastewater Treatment Plant No. 2 located within the City. Given the growth within OCSD's service area, OCSD is currently upsizing a number of collection system pipelines to provide additional capacity. One of these key facilities is the new 108-inch Bushard Trunk Sewer, which runs through the City to OCSD's Plant No. 2.

Private Wastewater Facilities

Private wastewater facilities within the City fall into two categories: onsite services and offsite wastewater pipelines. Maintenance of all private facilities is the responsibility of the owner. Onsite service pipelines are considered private from the point of connection to the City's main pipelines. Typically, private onsite pipelines include 4-inch diameter residential services, and 4 through 18-inch diameter commercial, industrial, and other non-residential uses. There are no private offsite wastewater pipelines maintained by the City.

Summary of Wastewater Facilities

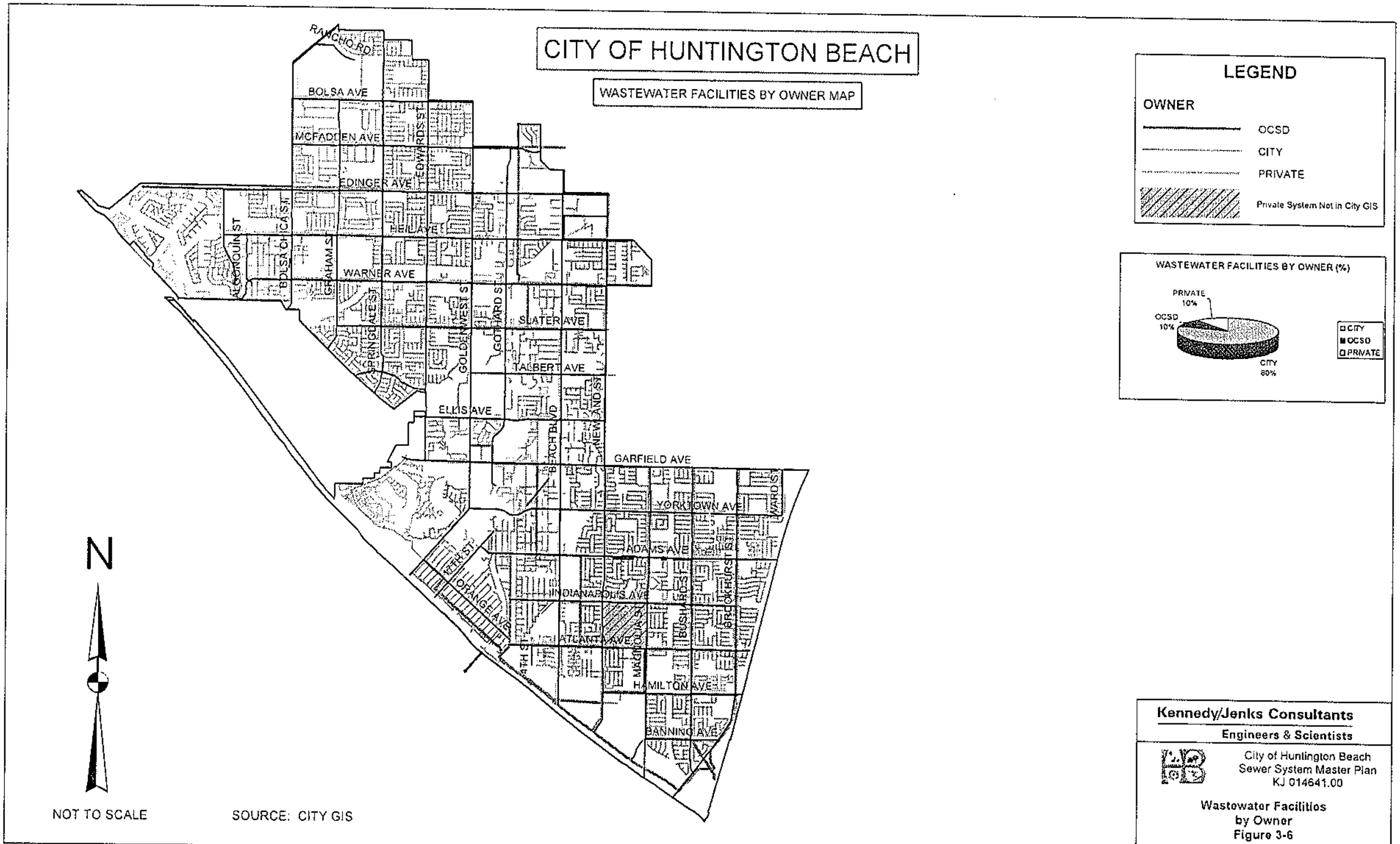
A summary of the wastewater facilities located within the City service area is shown in Figure 3-6. This information is based primarily on the information provided in the City's GIS and summarizes the ownership of the underground wastewater system. While the vast majority of the City-owned facilities are contained in the City's GIS, a number of the OCSD trunk facilities and private pipelines may not be included in the geographic data and are therefore not reflected in this figure.

EXISTING SYSTEM CONDITION AND DEFICIENCIES

This section discusses the general physical condition of facilities and equipment, within the City's existing wastewater system. This assessment is based on field inspections, discussions with City Operations & Maintenance and Engineering staff, and review of record drawings.

Wastewater Pipelines

The actual physical condition of underground infrastructure is generally assessed through video inspection. In recognition of this need, the City is proactively implementing a comprehensive citywide video inspection program. The result of this evaluation will be an integral element of the City's infrastructure management plan and will classify facilities by priority of condition. Upon the completion of this program, the City will be able to develop a comprehensive underground utility inventory with identified deficiencies, estimate remaining useful life, and prepare for the methodical reinvestment in its aging infrastructure.



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Wastewater Facilities by Owner
Figure 3-6

Pursuant to the 1995 Wastewater System Master Plan, the City implemented a replacement and rehabilitation program to prevent pipeline breakage and reduce the City's potential for sewage spills and leakages. The focus of this effort was in the downtown/oldtown and harbor areas. Trenchless rehabilitation was the remediation methodology utilized to repair these facilities. The location of the facilities that have undergone trenchless rehabilitation by the City since the 1995 master plan is shown in Figure 3-7. Table 3-3 provides a summary of the length and diameter of the trenchless rehabilitation activity.

TABLE 3-3

SUMMARY OF PIPELINE REHABILITATION ACTIVITY

Sewer Line Size (in.)	Length (LF)
6	1,770
8	139,310
10	24,080
12	3,200
15	590
16	4,360
18	150
Grand Total	173,460

Wastewater System Age

In addition to a video inspection of the wastewater system, system age may provide a general assessment of facility condition. Although no detailed inventory of physical assets by type and age is available, historical population provides an indication of probable system age.

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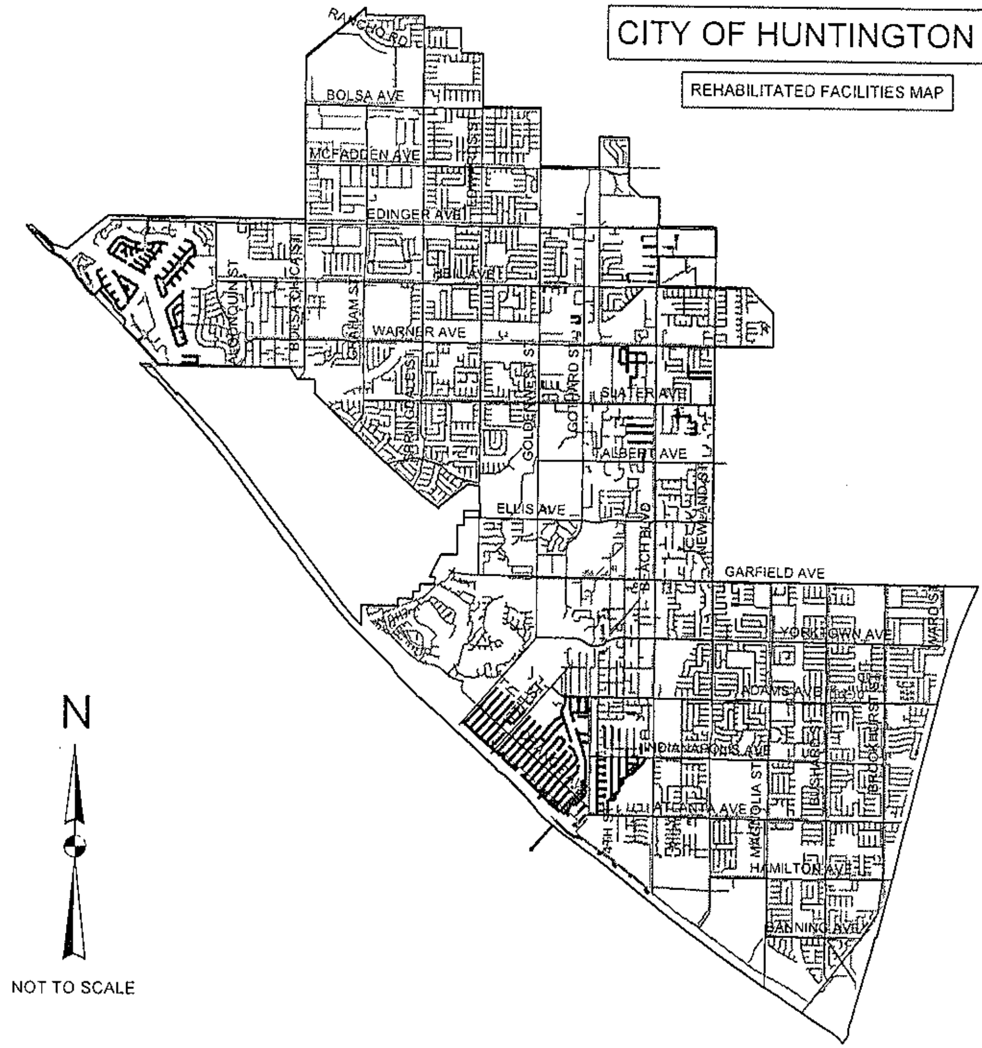
REHABILITATED FACILITIES MAP



LEGEND

REHABILITATED

- NO (thin line)
- YES (thick line)

REHABILITATED PIPE

Pipe Diameter (in)	Length of Pipe (lf)
6	1,771
8	139,330
10	24,078
12	3,201
15	593
16	4,356
18	151

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Summary of
Rehabilitation Activity
Figure 3-7

Because the City was founded in the late 1880's, a small percentage of the City's system may exceed 100 years old. Since the majority of the City's growth occurred since 1960, it could be concluded that the majority of infrastructure is approximately forty years old. Accordingly, system age and projected useful life tends to support the City O&M staff's assessment that the wastewater pipeline system is in generally good condition.

According to the State of California Controller's Office, the suggested useful life of wastewater utility fixed assets is 50 years for pipelines, manholes, and lift station structures, while the useful life of lift station equipment is generally approximately 20 years. It should be noted that the actual useful life of fixed assets may extend beyond the "book value" used for asset depreciation. Due to the inert nature of Vitrified Clay Pipe (VCP), it is generally considered to provide the longest useful life of most materials commonly used in wastewater pipeline construction.

Lift Stations

While the focus of this plan is a hydraulic assessment, the condition of the City's lift stations is an important element of its system reliability. The condition of these facilities was evaluated during the conduct of the 1995 Wastewater Master Plan. A summary table of the 1995 condition assessment is included in Appendix A.

The assessment indicates that timely maintenance and repair provided by the City have left the lift stations in generally good condition. However, the advancing age of the facilities warrants significant attention. Due to both the age of the lift stations and their importance to the reliability of the City's wastewater system, they should receive a high priority in the City's ongoing wastewater Capital Improvement Program (CIP).

Chapter 4

Desktop Inflow & Infiltration Study

CHAPTER 4

DESKTOP INFLOW AND INFILTRATION STUDY

This chapter incorporates the results of the Desktop Inflow and Infiltration Study (I&I Study) performed for the City. An I&I element was included in the 2003 Sewer System Master Plan update to assist the City in identifying potential I&I problem areas and to prepare a prioritized implementation program.

OVERVIEW

As a desktop study, no fieldwork was performed to generate new data for this analysis. As such, the focus of this study was to utilize previously generated and/or readily available data to reach broad quantitative conclusions about the potential for inflow and infiltration throughout the City's wastewater system. These conclusions can be used in the future to direct investigative and remedial fieldwork and focus future fieldwork on apparent potential problem areas. This methodical procedure will minimize the high cost of expensive field investigation and promote an efficient use of City resources.

In order to determine these apparent potential problem areas, the I&I Study made use of the following data:

- Wastewater system GIS layers provided by the City
- Discussions with City Operations and Maintenance (O&M) staff
- California State monitoring well data available at "http://well.water.ca.gov"
- Boring log data provided by the City
- National Ocean and Atmospheric Administration (NOAA) 2001 tidal data available at "http://co-ops.nos.noaa.gov/data_res.html"
- NOAA 2001 rainfall data at "http://www.wrh.noaa.gov/sandiego/climate.html"
- Wastewater lift station run times and pumping characteristics provided by the City

Analysis of these data sources, alone and in combination, produced four separate evaluations of potential inflow and infiltration areas of concern within the City. These areas include:

- O&M-identified areas
- Groundwater-influenced areas
- Tidal-influenced areas
- Precipitation-influenced areas

The resulting analysis of these areas of concern is detailed in the following sections of this study.

O&M-IDENTIFIED POTENTIAL PROBLEM AREAS

Conversations with City Operations and Maintenance staff indicated that the City's harbor area has been identified as an area with great potential for inflow and infiltration. Having been previously identified by the City, the pipelines in this area have all undergone trenchless rehabilitation. Figure 3-7 in the previous chapter shows all City-owned wastewater pipelines within City borders and indicates which of these pipelines have been included in the City's trenchless rehabilitation project. As the figure shows, the O&M-identified potential inflow and infiltration problem areas have currently been addressed in the harbor area.

GROUNDWATER-INFLUENCED POTENTIAL I&I PROBLEM AREAS

Groundwater encroachment into City wastewater pipelines was identified at the beginning of the I&I study as a possible contributing factor to inflow and infiltration within the wastewater system. The potential for this area of concern was evaluated by comparing groundwater elevations with the wastewater system invert elevations provided by the City's GIS.

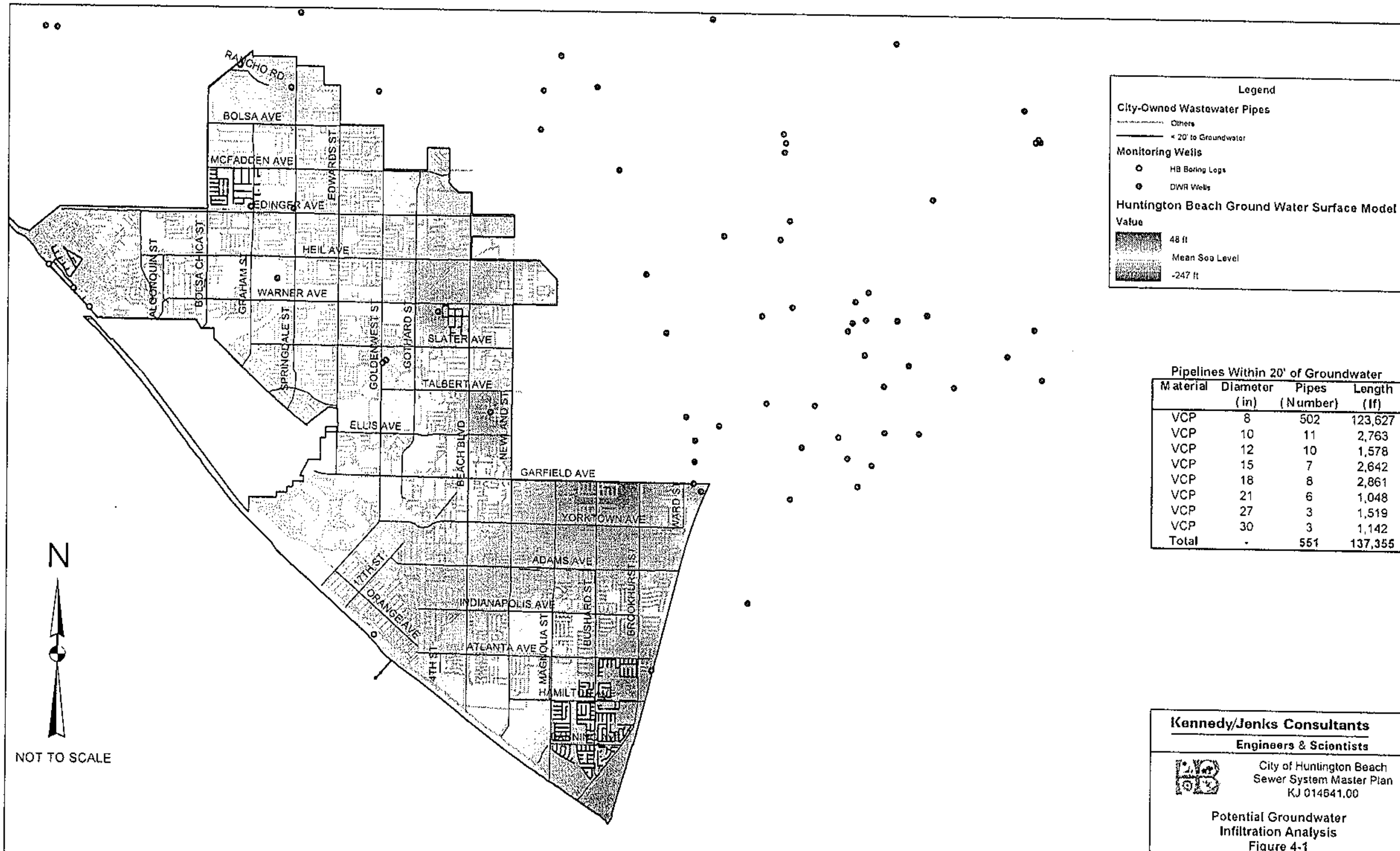
The production of such comparisons required a 3-dimensional model of the groundwater surface beneath the City. The California Department of Water Resources (DWR) maintains records for 20,000 groundwater monitoring wells across the state. Wells not proximate to Huntington Beach were discarded through GIS analysis. Inactive and suspect wells were identified and discarded also. The location of the remaining wells, which were used to provide reliable groundwater surface elevations in the Huntington Beach area, can be seen in Figure 4-1.

Because the groundwater elevation readings for each well were recorded at varying times and on varying cycles throughout the years, it was necessary to consolidate readings in order to incorporate the maximum amount of available data. The winter of 1999 was the latest winter season for which a large amount of data could be used. All wells with readings for this time period (October of 1998 to April of 1999) were included in the analysis to provide a data-intensive groundwater model.

As shown in Figure 4-1, the DWR wells do not provide full coverage for the City. In particular, they provide no data near the coast and harbor areas, areas that would be prime suspects for high water table and low sewer pipeline elevation combinations. The City provided boring logs taken at Lift Station "A" in Sunset Beach and farther south along PCH. The ground water surface elevations taken from these logs supplement the DWR data. These boring log points are identified as "City Boring Log."

From groundwater surface elevations taken at each of the points in Figure 4-1, a groundwater surface model was interpolated using GIS software. The software used an "Inverse Distance Weighting" algorithm to create the 3-dimensional surface from the known elevations of the wells and borings. The modeled groundwater surface is shown as a color-coded image, with different colors representing various groundwater surface elevations taken from mean sea level.

In order to compare the groundwater surface model to the City's wastewater pipeline system, the pipe system's GIS layer was overlaid on the surface model. Record drawing extraction was used to establish reliable upstream and downstream invert elevations for pipes in areas where the groundwater table was within 20 ft. of the ground service. With these areas used as a reliable datum, all other inverts throughout the system were



Pipelines Within 20' of Groundwater

Material	Diameter (in)	Pipes (Number)	Length (lf)
VCP	8	502	123,627
VCP	10	11	2,763
VCP	12	10	1,578
VCP	15	7	2,642
VCP	18	8	2,861
VCP	21	6	1,048
VCP	27	3	1,519
VCP	30	3	1,142
Total	-	551	137,355

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Potential Groundwater
Infiltration Analysis
Figure 4-1

vertically smoothed using pipeline length and slope calculations to adjust the City's wastewater facilities to a reliable datum. GIS analysis was used to determine the distance between the downstream invert elevation and the groundwater surface model at the same location.

The results of the pipeline network and groundwater overlay analysis are also shown on Figure 4-1. As shown, there are four pockets within the City where it is believed that the groundwater surface is within 20 feet of the City's wastewater pipelines. These areas have a reasonable potential for infiltration during times of particularly high groundwater, as might be created by a significant wet weather event. A summary table of pipeline length, diameter, and material of the identified facilities is also depicted in this figure.

TIDAL-INFLUENCED POTENTIAL I&I PROBLEM AREAS

While the O&M-identified and groundwater-influenced areas of concern were qualitative in nature, potential tidal-influenced problem areas can be more quantitatively evaluated. This evaluation can be performed in specific areas by correlating the tidal influence in each tributary area to the estimated volume of wastewater pumped through each of the lift stations that serve the harbor area of the City.

The City provided daily run times for each of its lift stations for calendar year 2001. By correlating this run time with the City-provided lift station capacities, an estimate of daily wastewater pumped for each facility can be derived. To identify the pipeline systems that may be under the influence of tidal conditions, an analysis of the gravity tributary flow to each lift station must be identified. As such, the analysis subtracts out the estimated upstream lift station flows so that only gravity tributary flow in each tributary area is evaluated. This procedure prevents double counting the estimated pumped wastewater volume, avoids the misinterpretation of data associated with the potential transfer of an I&I problem from an upstream to a downstream system, and isolates the potential I&I problem to the local gravity-based tributary area served by each lift station. The location and connectivity of the City's wastewater lift station network, previously shown in Chapter 3 as Figure 3-3, was used to identify the impact of upstream lift station facilities.

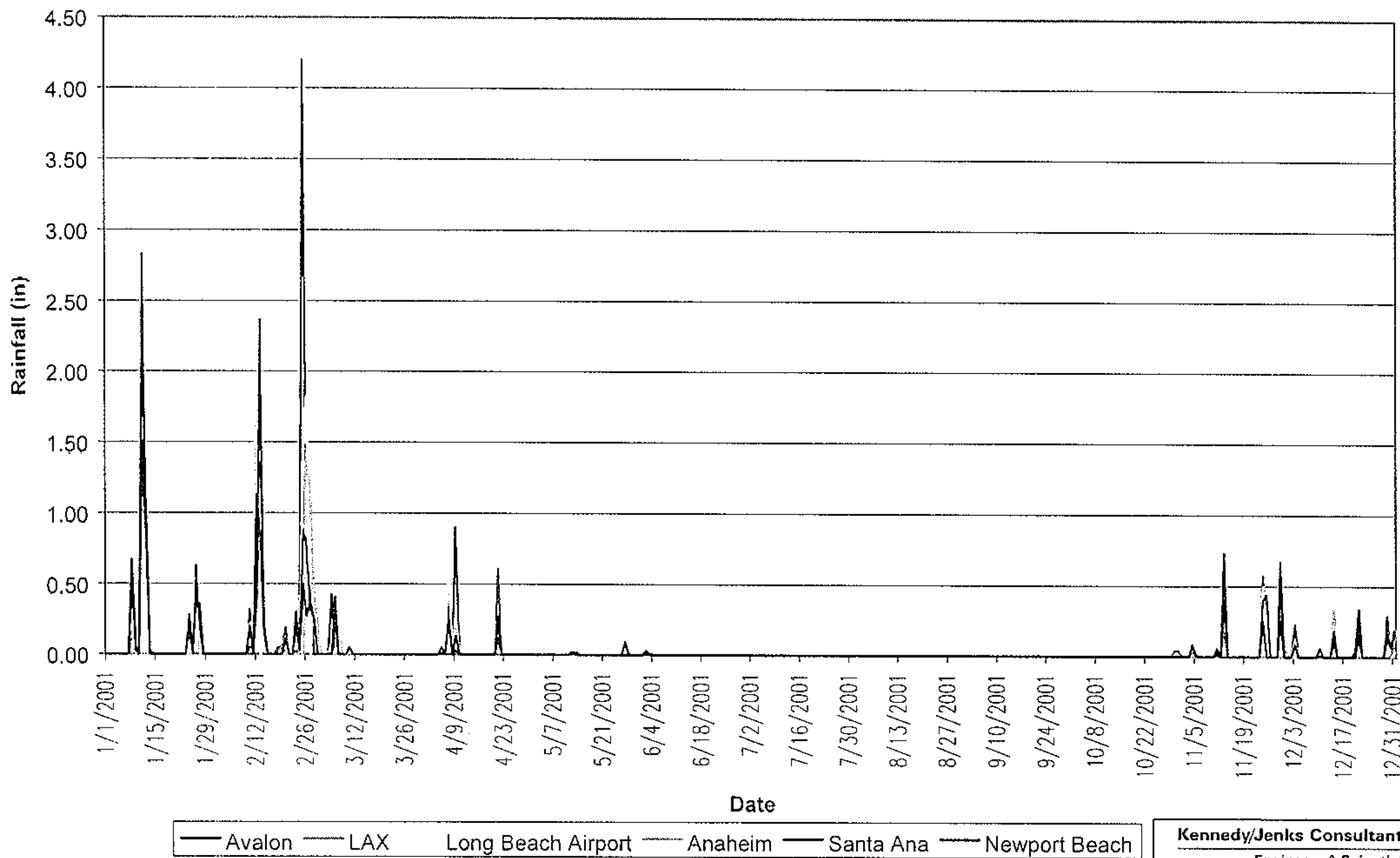
Two important factors were derived during the conduct of this analysis. First, It should be noted that for the second half of 2001, Lift Station No. 4, Station "A," was being rebuilt. During this time, all flows entering this station were pumped by temporary pumps, for which no records were kept. Thus, part of the study period includes no data for Lift Station "A." However, because Lift Station "A" pumps directly into Lift Station "B," all flows from Lift Station "A" are accounted for in the latter's flow. For the purposes of this study, the tributary areas of these two lift stations have been merged.

Because precipitation was anticipated to be an inflow and infiltration influence, it was necessary to utilize a dry-weather time frame to isolate tidal influence on daily volumes pumped by each station. Analysis of the NOAA National Weather Service data given in Figure 4-2 (see following section for more complete description of this data) indicated that August and September of 2001 were dry-weather months for the City.

As discussed, the City's lift station run time was a key component of the tidal influence analysis. To provide a high correlation to daily diurnal tidal activity, continuous recording data during a 24-hour period for each lift station was desired. However, the City's lift station O&M run time data was only available on a daily basis. Since daily lift station volumes do not have the precision to show the influence of daily tidal fluctuation, the daily high tide lift station volumes were analyzed in comparison to the average monthly fluctuations in daily high tide. Figure 4-3 shows the plot of daily high tide that was used in the analysis.

Detailed tide records are not kept at every harbor; Huntington Harbour has no publicly available tide records. The Port of Los Angeles (LA) had 6-minute tidal data available, as did the pier at La Jolla in San Diego County. Comparison of the data from these two sources showed that tide magnitude did not differ, but that tide phase differed by about 15 minutes. From this comparison, it was concluded that the tidal magnitude in Huntington Beach is comparable to that of LA, and the phase would differ by less than 15 minutes, leaving LA tidal data as a suitable proxy.

A core component of the tidal analysis is the establishment of a low-tide, dry-weather baseline volume for each lift station. Plotting the daily lift station volumes indicated that there were statistically significant differences between weekend and weekday pump



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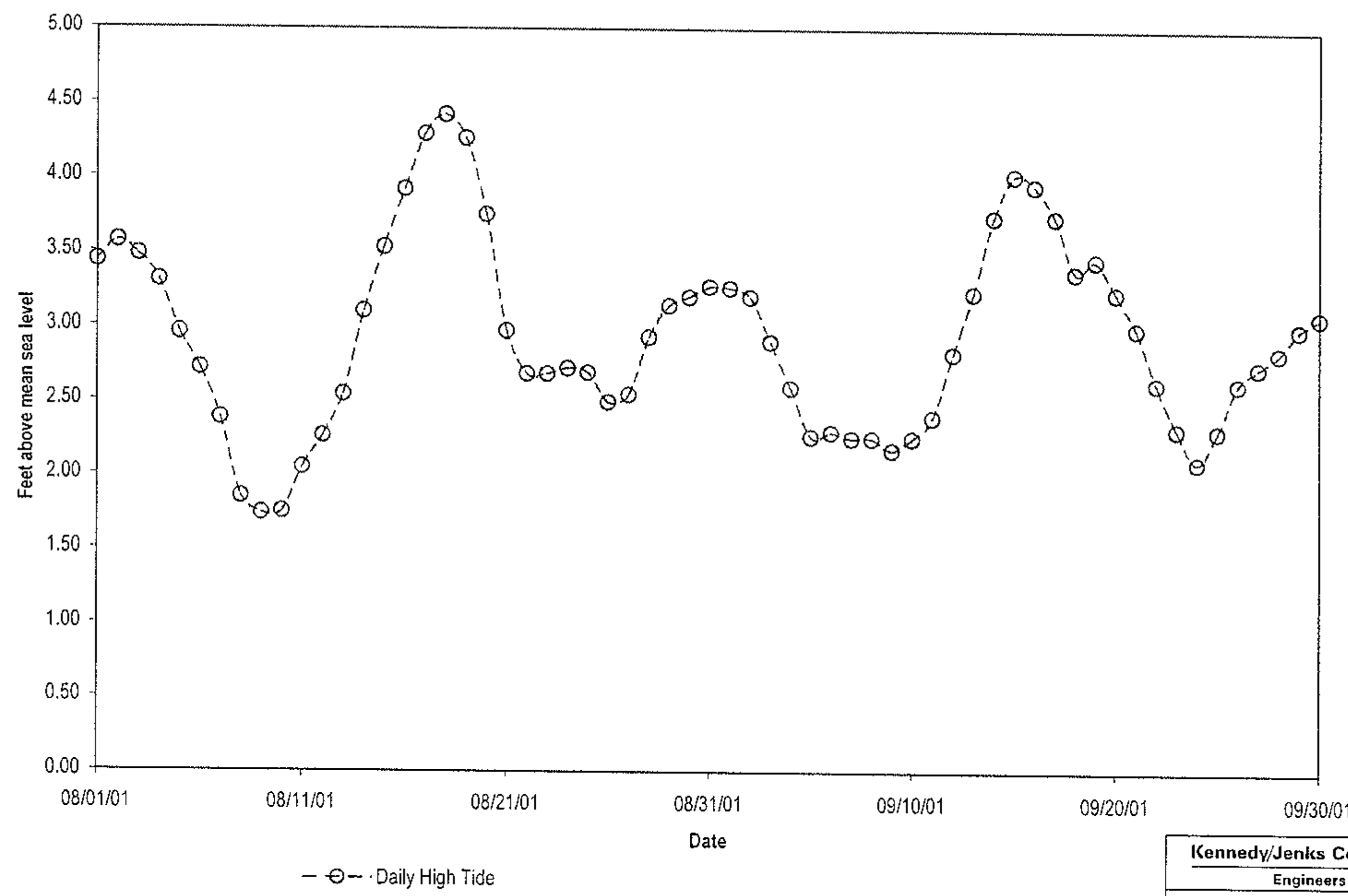
Huntingtion Beach Vicinity
Rainfall 2001
Figure 4-2



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Daily High Tide
Port of LA
Figure 4-3

volumes. A summary of the numerical differences is provided in Table 4-1.

Each station's weekday volumetric average over the period August 1 to September 30 is shown in the first column. The weekend average for the same period is shown in the second column, followed by the percentage difference between the two. The final column is a Z-value assigned to the difference between the weekday and weekend average, based on a two-grouped Z-test. Although the conditions of a statistical Z-test do not strictly apply to this data, the Z-value for a given lift station gives broad indication of the degree of randomness of the difference.

The high number of Z-values above three for this data indicates that lift station patterns are significantly different between weekends and weekdays in the City. Accordingly, an analysis that is based on lift station volumetric totals should recognize this variation and incorporate the appropriate data set in any analytical comparisons.

As shown on Figure 4-3, the days with the lowest and highest daily high tides for the August/September period are August 9 and August 18, respectively. Since August 18 is a weekend, August 17 was used in the comparison to maintain the weekday-to-weekday consistency of the two data sets. Utilizing the day prior to highest daily tide should have minimal impact on the tidal influence analysis.

The lift station volumes for August 9 and August 17 are provided in Table 4-2. August 9 represents the lowest daily high tide of the study period, August 17 the highest. The percentage difference between the two can be seen in the third column. As highlighted in the table, Lift Stations No. 3, No. 8, and No. 25 indicate a noticeable increase in pump volume between the lower and higher tide cycles. As can be seen in Figure 3-3, these lift stations serve the harbor area, in which the trenchless rehabilitation program had not been completed as of August 2001. As such, the analysis performed herein confirms the apparent need to rehabilitate these areas and provides a general baseline methodology to measure the effectiveness of future facility improvements.

Table 4-1
Lift Station Dry Weather Baseline Conditions
August/September 2001

Station Number	Station Name	Gross Weekday Average (gpd)	Gross Weekend Average (gpd)	Net Weekday Average (gpd)	Net Weekend Average (gpd)	*Net Difference (%)*	*Net Z-value* (1)
1	Graham	149,281	166,549	149,281	166,549	*12*	*10*
2	Humoldt	96,517	105,677	96,517	105,677	*9*	*1*
3	Station "E"	14,552	13,772	14,552	13,772	*-5*	*1*
5	Davenport	51,467	52,432	51,467	52,432	*2*	*0*
6	Edgewater	327,066	334,703	327,066	334,703	*2*	*3*
4/7 (2)	Station "A"/"B"	470,433	536,893	444,219	512,196	*15*	*1*
8	Station "C"	774,498	831,493	304,064	294,600	*-3*	*0*
9	Station "D"	2,737,172	2,755,680	1,584,141	1,537,052	*-3*	*1*
10	Algonquin	830,037	971,133	396,012	524,966	*33*	*2*
11	Lark	22,373	24,399	22,373	24,399	*9*	*2*
13	Slater	433,126	470,443	433,126	470,443	*9*	*7*
14	Ellis	261,879	272,567	261,879	272,567	*4*	*3*
15	Beach	45,662	50,291	45,662	50,291	*10*	*7*
16	Adams	62,516	68,860	62,516	68,860	*10*	*7*
17	Brookhurst	440,975	472,747	440,975	472,747	*7*	*3*
18	Altanta	135,610	126,911	135,610	126,911	*-6*	*1*
19	Bushard	101,664	105,924	101,664	105,924	*4*	*1*
20	Speer	25,786	11,067	25,786	11,067	*-57*	*5*
21	McFadden	73,567	77,072	73,567	77,072	*5*	*2*
22	Saybrook	434,026	446,167	152,376	152,601	*0*	*0*
23	New Britain	74,667	80,672	74,667	80,672	*8*	*4*
24	Edwards	328,983	348,283	156,788	166,800	*6*	*2*
25	Edinger	185,133	187,889	57,073	70,747	*24*	*2*
26	Brighton	172,195	181,483	172,195	181,483	*5*	*2*
28	Coral Cay	11,663	10,926	11,663	10,926	*-6*	*3*
29	Trinidad	128,060	117,142	128,060	117,142	*-9*	*2*

(1) Z-value higher than 3 reflects strong potenial for non-random variation
(2) Pumping statistics combined because no data available for Station #4 during study period.
Note: Net values reflect local tributary gravity flows only. Gross values include upstream lift station flows. See Figure 3-4 for station connectivity.

Table 4-2
Tidal Influence on Lift Station Volume
August/September 2001

Station Number	Station Name	Gross Lower Tide Cycle 8/9/2001 (gpd)	Gross Higher Tide Cycle 8/17/2001 (gpd)	Net Lower Tide Cycle 8/9/2001 (gpd)	Net Higher Tide Cycle 8/17/2001 (gpd)	Net *Difference (%)*
2	Humoldt	80,885	83,674	80,885	83,674	*3*
3	**Station "E"**	**14,405**	**22,808**	**14,405**	**22,808**	***58***
5	Davenport	49,739	47,826	49,739	47,826	*-4*
4/7[(1)]	Station "A"/"B"	510,540	486,420	483,184	452,100	*-6*
8	**Station "C"**	**772,212**	**807,313**	**261,672**	**320,893**	***23***
9	Station "D"	2,732,400	2,829,600	1,578,296	1,647,709	*4*
10	Algonquin	1,014,000	990,000	598,091	557,586	*-7*
25	**Edinger**	**176,305**	**183,501**	**68,097**	**76,796**	***13***
28	Coral Cay	12,951	11,512	12,951	11,512	*-11*
29	Trinidad	108,208	106,705	108,208	106,705	*-1*

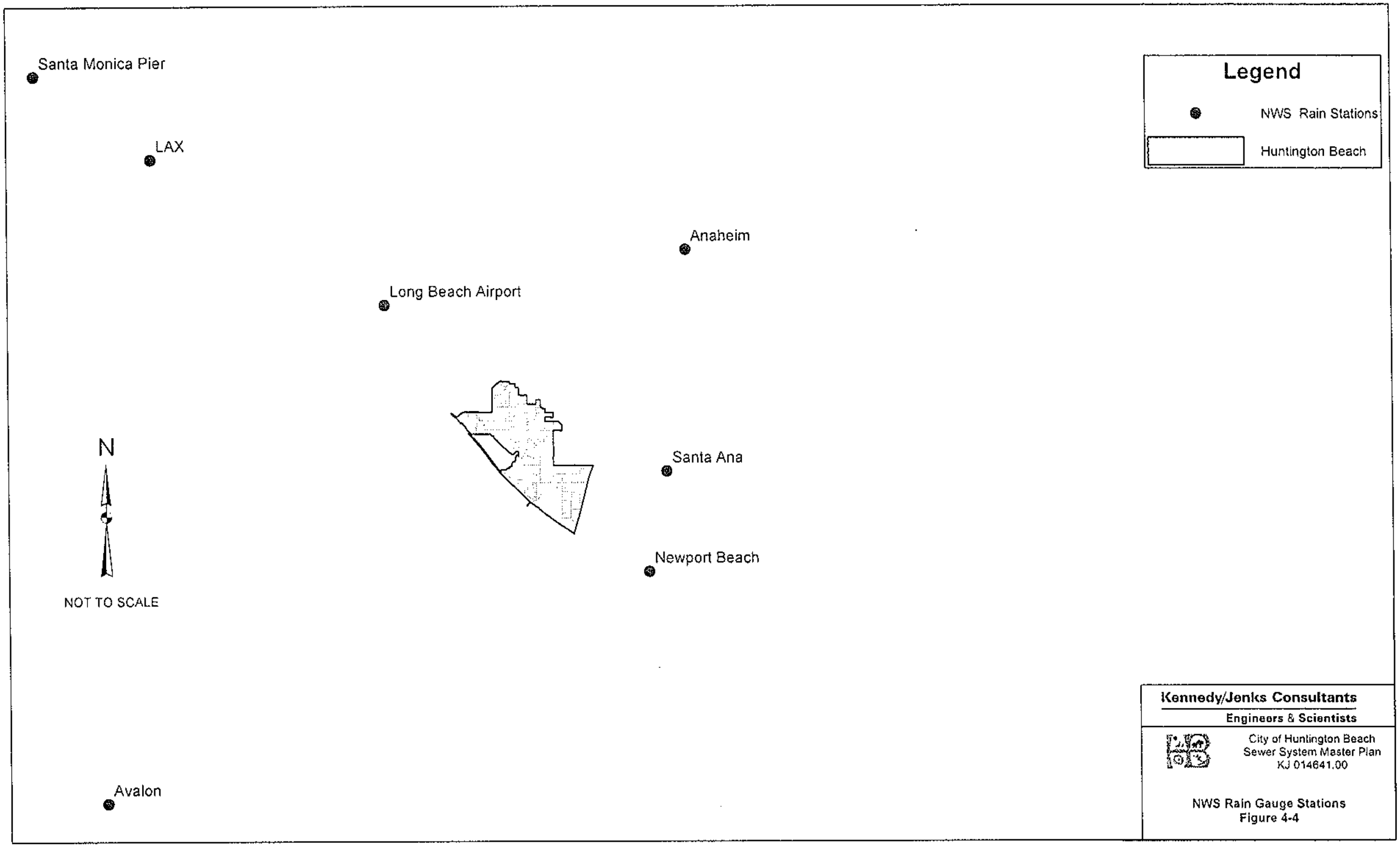
(1) Pumping statistics combined because no data available for PS #4 during study period.
Note: Net values reflect local tributary gravity flows only. Gross values include upstream lift station flows.

PRECIPITATION-INFLUENCED POTENTIAL I&I PROBLEM AREAS

Inflow and infiltration studies are generally performed by analyzing measured wastewater flow data for the impact of wet weather conditions. The City's field flow monitoring program is discussed in Chapter 5. For a desktop study that does not install rain gauges or flow monitors into the field, such data must come from routinely measured parameters within the City. As discussed in the previous section, lift station data provides a baseline of pump output. This run time data for wet weather conditions provides the basis for changes in dry versus wet weather flows. Precipitation information must be taken from routinely monitored rain gauges as close as possible to the City. As the volume pumped in each lift station is available on a daily basis, the precipitation information should be as well.

No source providing such data within the City limits was found. The only sites found providing daily precipitation back to 2001 were those monitored by the National Weather Service (NWS). Since the NWS had several sites in communities surrounding the City, a composite reading of six NWS rain gauges from around the City would provide the best representative data. The composite reading is accurate enough to determine when the City is under wet weather conditions, but not accurate enough to determine I&I as a function of depth of rainfall. Figure 4-4 shows the location of these rain gauges relative to the City. As shown, the six gauges geographically surround the City.

Figure 4-5 shows daily rainfall for the six gauges, plotted for January and February 2001. Based on the location of these gauges, it can be reasonably assumed that precipitation events that left significant amounts of precipitation at each of the six meters on a particular day also left precipitation within the City that day. January 11 and February 13, both weekdays, met these criteria and were chosen to represent the wet weather data utilized herein. Data limitations require the assumption that uniform rainfall was received across all applicable tributary areas in the City. Lift station totals for these two days were contrasted with the totals for the weekday dry weather baselines established in the previous section (Table 4-1). The percentage difference between the two wet weather events and the baseline conditions are included. The percentage difference columns demonstrate the varying degrees of lift station response to the wet weather conditions. The results of this analysis are shown in Table 4-3.



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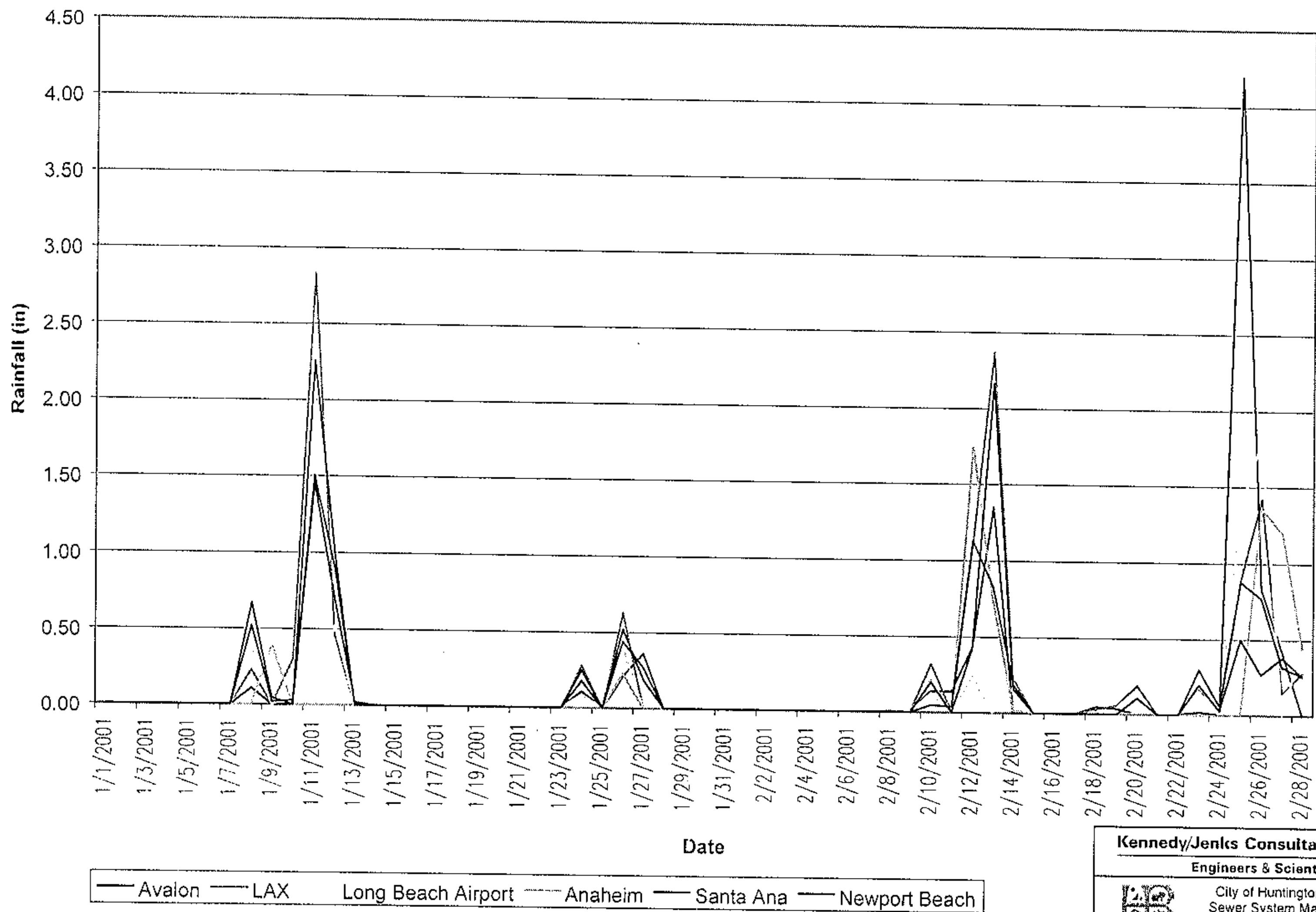
NWS Rain Gauge Stations
Figure 4-4



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Huntingtion Beach Vicinity
Rainfall Jan/Feb 2001
Figure 4-5

Table 4-3
Dry Weather vs Wet Weather Comparison

Station Number	Station Name	Gross Average Weekday Dry Weather (gpd)	Gross Wet Weather 1/11/2001 (gpd)	Gross Wet Weather 2/13/2001 (gpd)	Net Average Weekday Dry Weather (gpd)	Net Wet Weather 1/11/2001 (gpd)	Net Wet Weather 2/13/2001 (gpd)	*Net Wet Weather Difference 1/11/2001 (%)*	*Net Wet Weather Difference 2/13/2001 (%)*
1	Graham	149,281	187,977	167,091	149,281	187,977	167,091	*26%*	*12%*
2	Humboldt	96,517	128,300	120,862	96,517	128,300	120,862	*33%*	*25%*
3	Station "E"	14,552	21,007	19,207	14,552	21,007	19,207	*44%*	*32%*
5	Davenport	51,467	97,566	70,783	51,467	97,566	70,783	*90%*	*38%*
6	**Edgewater**	**327,066**	**626,500**	**550,888**	**327,066**	**626,500**	**550,888**	***92%***	***68%***
4/7[1]	**Station "A"/"B"**	**470,433**	**944,700**	**775,860**	**444,219**	**900,669**	**736,987**	***103%***	***66%***
8	Station "C"	774,498	1,389,982	1,116,197	304,064	445,282	340,337	*46%*	*12%*
9	**Station "D"**	**2,737,172**	**5,767,200**	**4,590,000**	**1,584,141**	**3,653,153**	**2,852,132**	***131%***	***80%***
10	Algonquin	830,037	1,188,000	1,200,000	396,012	524,526	609,145	*32%*	*54%*
11	Lark	22,373	38,183	26,952	22,373	38,183	26,952	*71%*	*20%*
13	Slater	433,126	609,900	642,000	433,126	609,900	642,000	*41%*	*48%*
14	Ellis	261,879	306,000	290,700	261,879	306,000	290,700	*17%*	*11%*
15	Beach	45,662	53,091	53,091	45,662	53,091	53,091	*16%*	*16%*
16	Adams	62,516	71,280	92,400	62,516	71,280	92,400	*14%*	*48%*
17	Brookhurst	440,975	599,040	714,240	440,975	599,040	714,240	*36%*	*62%*
18	Atlanta	135,610	176,449	205,858	135,610	176,449	205,858	*30%*	*52%*
19	Bushard	101,664	102,060	103,950	101,664	102,060	103,950	*0%*	*2%*
20	**Speer**	**25,786**	**715,200**	**182,400**	**25,786**	**715,200**	**182,400**	***2674%***	***607%***
21	**McFadden**	**73,567**	**149,678**	**113,218**	**73,567**	**149,678**	**113,218**	***103%***	***54%***
22	Saybrook	434,026	663,474	590,855	152,376	229,339	201,937	*51%*	*33%*
23	New Britain	74,667	84,661	92,162	74,667	84,661	92,162	*13%*	*23%*
24	Edwards	328,983	460,821	456,021	156,788	217,778	191,843	*39%*	*22%*
25	Edinger	185,133	305,835	268,056	57,073	75,894	84,704	*33%*	*48%*
26	Brighton	172,195	243,044	264,178	172,195	243,044	264,178	*41%*	*53%*
28	**Coral Cay**	**11,663**	**23,024**	**19,666**	**11,663**	**23,024**	**19,666**	***97%***	***69%***
29	Trinidad	128,060	229,942	183,352	128,060	229,942	183,352	*80%*	*43%*

(1) Pumping statistics combined because no data available for PS #4 during study period.
Note: Net values reflect local tributary gravity flows only. Gross values include upstream lift station flows.

As shown, the tributary areas serving six lift stations appear to have been significantly influenced by the representative rainfall events. To assist the City in prioritizing further field I&I investigation, the tributary area pipeline characteristics associated with each of these six facilities are evaluated and reflected in Table 4-4. Figure 4-6 displays the lift station tributary areas for which the characteristics were calculated. Each lift station was assigned a priority ranking based on the degree of potential I&I. This potential was based on the amount of increased volume pumped. The data from whichever of the two wet weather events produced the most response was used in the calculation.

To check the sensitivity to the prioritized ranking based on pumped volume to basin characteristics, the additional volume pumped was subsequently normalized. The normalization process was performed by dividing the additional volume of flow by the amount of pipe in the tributary area, expressed in inch-diameter miles. Since the normalized analysis resulted in an identical ranking as the non-normalized data, it is concluded that the prioritized ranking derived herein is based on both the degree of the potential I&I problem areas (normalized findings) as well as the total quantity of the potential I&I values (total increased pumped volume).

SUMMARY OF FINDINGS

Based on the preceding desktop analysis, it recommended that the City pursue the following actions concerning each area of potential inflow and infiltration:

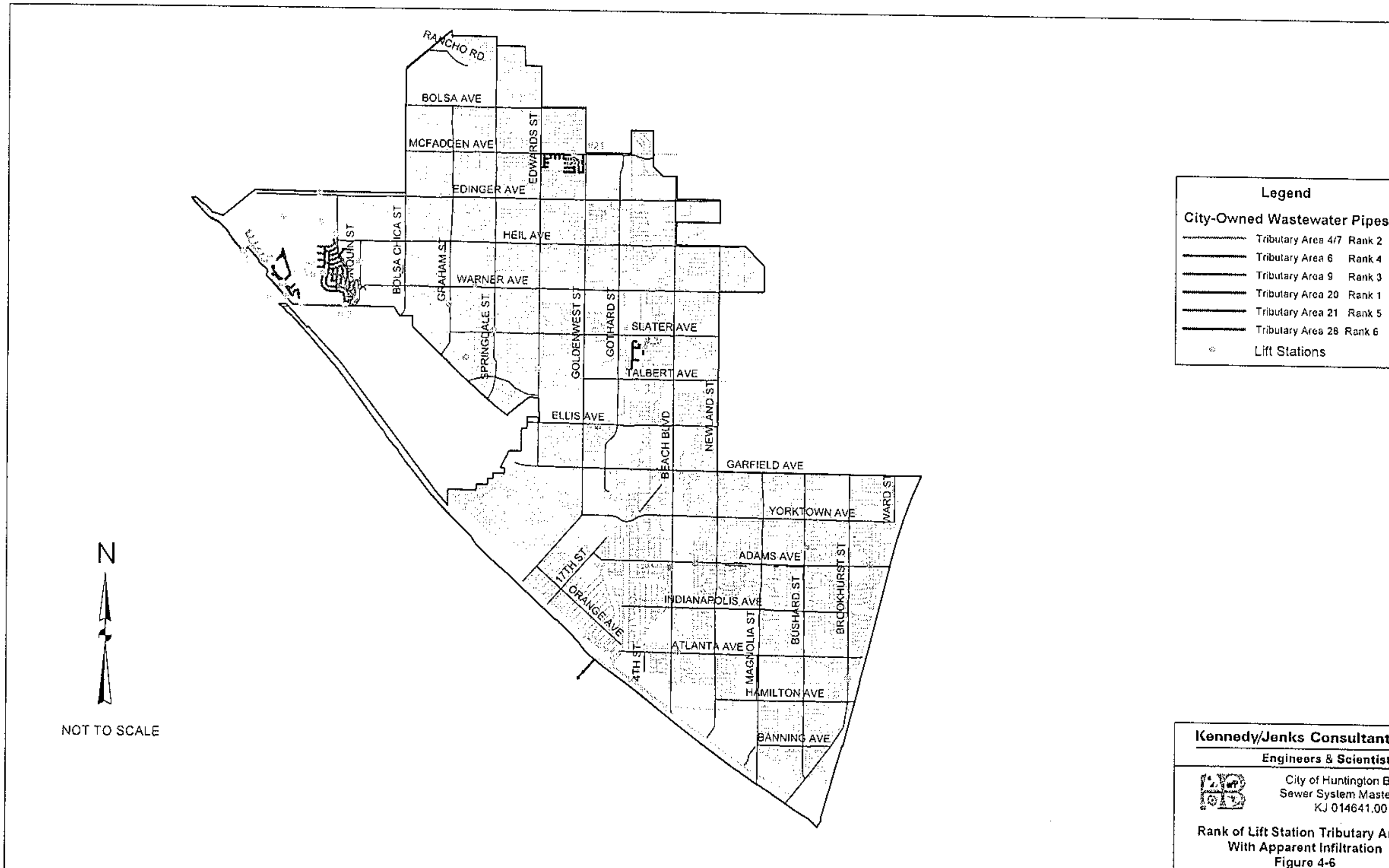
1. O&M-Identified – The City has a trenchless rehabilitation program underway that is designed to remediate the potential problem areas identified by City O&M staff. An element of this program includes an evaluation of the effectiveness current and forthcoming trenchless rehabilitation activities.

2. Groundwater-Influenced – It is recommended that the City confirm the depth of the wastewater pipelines and attempt to further quantify the groundwater levels. Upon completion of this activity, the City may need to conduct additional localized I&I studies in these areas. City staff performing video inspections during known high groundwater conditions may provide a cost effective approach to resolving this area of concern.

TABLE 4-4
PRECIPITATION-INFLUENCED POTENTIAL I&I RANKINGS

Station Number	Pipe Length (lf)	Average Weekday Dry Weather (gpd)	Increased Pumping Response[1] (gpd)	Inch-Diameter Miles (idm)	Normalized Increased Pumping (gpd/idm)	Rank
4/7[2]	9,530	510,565	525,935	14.44	36,425	2
6	14,132	327,014	299,386	24.60	12,168	4
9	13,578	96,604	459,488	24.15	19,023	3
28	7,390	11,663	11,361	11.07	1,026	6
20	3,414	32,233	861,767	5.13	167,865	1
21	10,298	73,607	76,153	15.60	4,880	5

(1) Represents increased pumping due to the greater of the two wet weather events shown in Table 4-3.
(2) Pumping statistics combined because no data available for Station #4 during study period.



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Rank of Lift Station Tributary Areas With Apparent Infiltration
Figure 4-6

3. Tidal Influence – For subsequent studies, it is recommended that continuous metering be performed at each of the three harbor lift stations (Lift Stations No. 3, No. 8, and No. 25) that indicated a potential tidal influence. Continuous dry weather metering through a typical high/low tide cycle should provide the necessary data to more precisely evaluate the response to daily low and high tide conditions. As previously discussed, a comprehensive lift station evaluation and flow isolation testing of the harbor area facilities is recommended to verify configuration and flow values.

4. Rainfall Influence – It is recommended that additional wet weather flow monitoring be performed for basin flow isolation in the six areas identified as potential I&I problems.

5. General - Prior to encumbering the necessary funds to implement the Desktop I&I Study findings, the City should verify the accuracy of the cornerstone data used herein. Verification of the accuracy of the pump output capacity and pump run time data will provide additional confidence to the lift station based findings and recommendation presented in this chapter. It is recommended that the City perform a comprehensive evaluation of the pump capacities and efficiencies at each of the City's lift stations. A budget of $75,000 is estimated to perform this analysis.

Chapter 5

Wastewater Loads and Design Criteria

CHAPTER 5

WASTEWATER LOADS AND DESIGN CRITERIA

This chapter outlines the development of wastewater loadings and design criteria used to evaluate the City's wastewater system. These parameters are based primarily on information provided by the City, other surrounding municipalities, and engineering standard practices. The data developed and evaluated herein was used to establish flow rates for various types of land uses within the City. It subsequently provides support for the calibration of the wastewater system hydraulic model, and the projection of future wastewater system flows within the City's service area. The future wastewater flows are used in subsequent chapters to evaluate the adequacy of existing collection/pumping system facilities and to identify the need for additional facilities to meet future loading conditions.

DEVELOPMENT OF WASTEWATER FLOW CRITERIA

The development of wastewater loading factors is an important element of this master plan. These factors are essential components of a capacity analysis and provide the basis for future demands on the utility system. Various sources and methods were used to develop these loading factors and appropriate wastewater criteria to be used in this study. The sources and results of this analysis are discussed in the following sections.

Prior Master Planning and Surrounding Community Criteria

A fundamental consideration in the development of the updated master plan wastewater loadings is the use of prior studies and the criteria used. These data sources provide a historical perspective of loading conditions and establish a benchmark for the development of updated values.

During the conduct of this master plan, Kennedy/Jenks Consultants reviewed the 1977 and 1978 Master Plans for the City, the 1989 OCSD Trunk Sewer Conveyance Study, and the City's 1995 Wastewater System Master Plan. In addition to these master plans, current criteria were also obtained and reviewed for the OCSD and the City of Newport Beach. Since Kennedy/Jenks Consultants performed the City's prior Wastewater System Master Plan in 1995, this research was focused on updated values related to changing local conditions. The wastewater generation factors derived in the 1995 Wastewater System Master Plan are shown in Table 5-1.

TABLE 5-1

1995 WASTEWATER DESIGN FLOW FACTORS

Land Use Category	Average Wastewater Flow Generation Factor
Residential	
Low Density (0-7 Du/Ac)	1,800 gpad
Medium (8-15 Du/Ac)	3,300 gpad
Medium - High (16-25 Du/Ac)	3,800 gpad
High - (25+ Du/Ac)	4,900 gpad
Commercial	3,000 gpad
Industrial	3,900 gpad
Open Space	200 gpad
Schools	3,600 gpad or 20 g/st/d

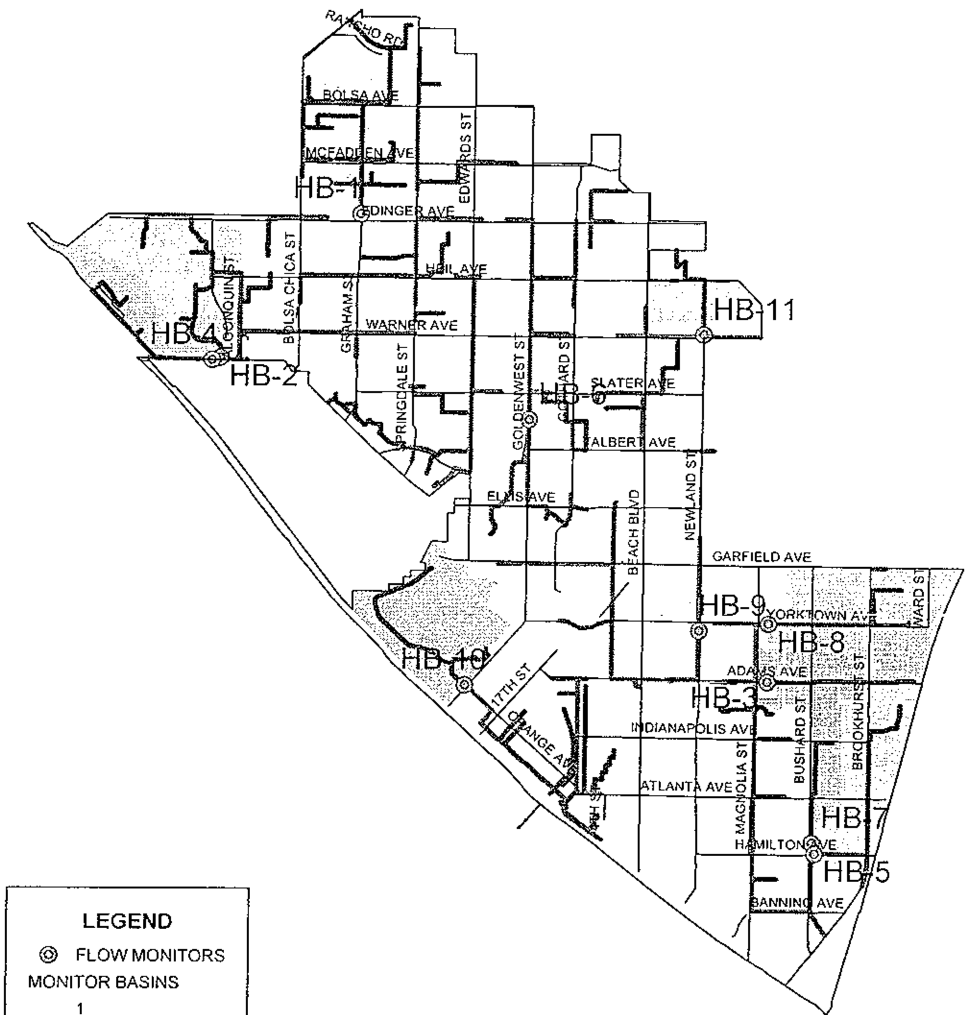
Temporary Flow Monitoring Program

A focused wastewater flow monitoring program was conducted by Kennedy/Jenks Consultants, in association with ADS Environmental, Inc. (ADS), to assess wastewater flow conditions in the City. There were three objectives of the temporary flow monitoring program: 1) obtain measured data during wet weather conditions to evaluate the impact of rainfall dependent inflow and infiltration (I&I) on the system, 2) derive existing wastewater generation factors for specific residential and non-residential land uses, and 3) establish average and peak wastewater values at key locations within the system for calibration of the computerized hydraulic model.

Based on these prescribed purposes, a flow monitoring program was prepared using available City land use maps, sewer system atlas maps, GIS digital utility configuration data, discussions with City staff, and an integration of the I&I program underway by the OCSD in the City sewer service area. The program specified the appropriate locations (manholes) and purpose for each of the 12 temporary monitoring stations. The location of these monitoring stations and the graphical representation of its tributary area is shown on Figure 5-1. The location and tributary areas of the OCSD flow monitoring program is provided on Figure 5-2.

Facility maps and field conditions were used to finalize the flow monitoring plan. The 12 temporary flow monitoring sites were field reviewed for physical and hydraulic suitability by ADS prior to installation. All meters were installed and operational by 13 March 2002 and remained in place for 28 days in an effort to obtain wet weather data. As previously discussed, since 2002 had been a relatively dry winter, the March/April time period was perceived as the final opportunity to capture wet weather data for this study. The 12 monitoring facilities were installed, tested, and calibrated to record minimum, average, and peak wastewater flows. The monitoring program recorded flow values from 14 March through 10 April 2002 at a 15-minute frequency throughout the 28-day program duration. Due to equipment difficulties, the monitoring program was extended to 19 April for two of the monitoring locations.

Although data obtained from temporary flow monitoring stations may provide inconsistent measurements associated with physical and environmental conditions, it is a common method of developing wastewater flow data. The industry standard of flow monitoring results is +/- 5 to 10 percent of actual flow values. This variance is typically attributed to the cleanliness of the pipeline facilities and the frequency and degree of localized solids deposition. The results of the flow monitoring program for each of the 12 monitored sites is summarized in Appendix B. A discussion of findings associated with the key objectives of the temporary flow monitoring program is provided as follows:



LEGEND

◎ FLOW MONITORS

MONITOR BASINS

1
2
3
4
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6
7
8
9
10
11

MODELED PIPE

DPI
EPI
PRE

N

NOT TO SCALE

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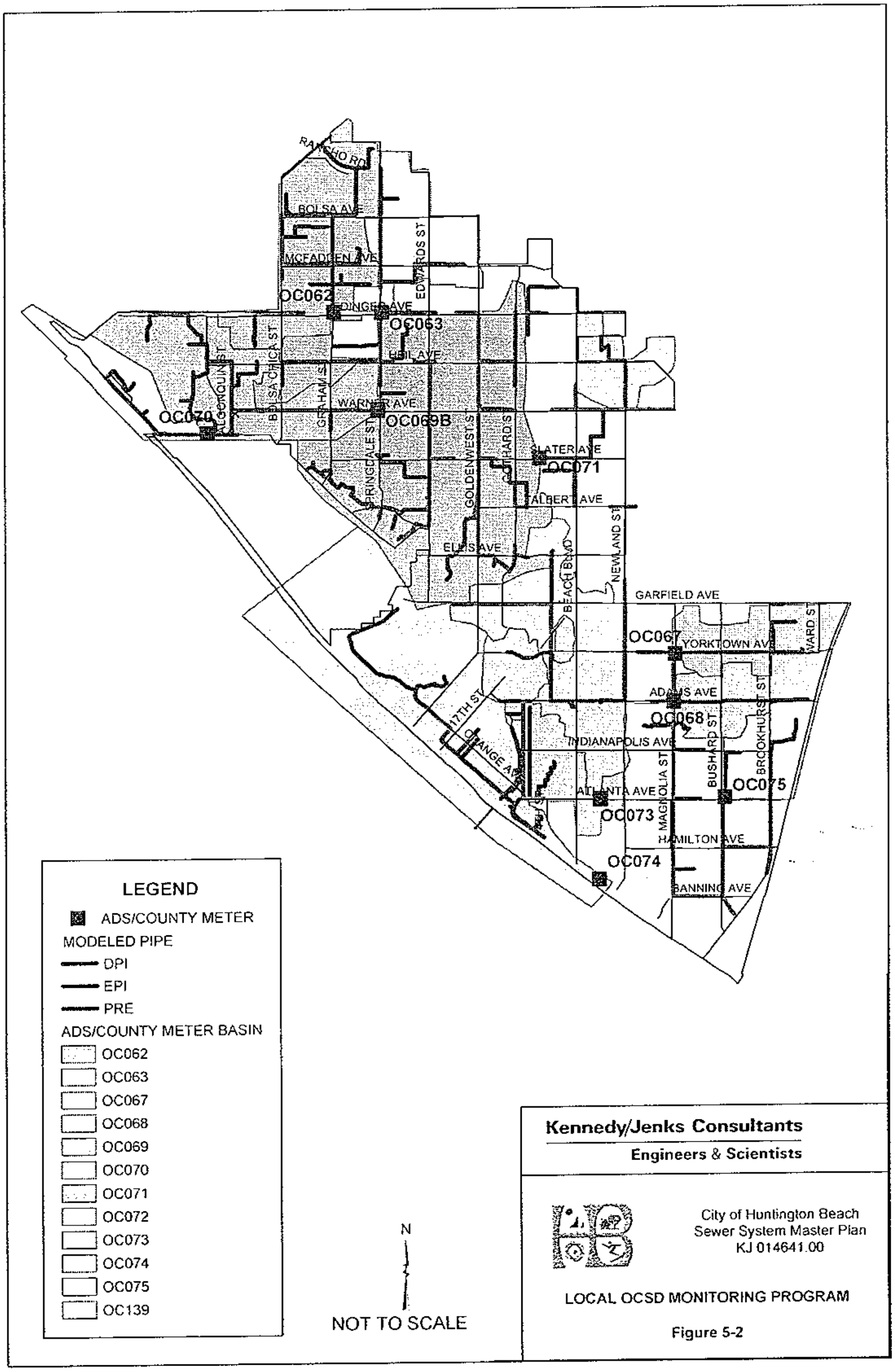
LOCAL CITY FLOW MONITORING PROGRAM

Figure 5-1



LOCAL OCSD MONITORING PROGRAM

Figure 5-2

Rainfall Dependent Wastewater Flows. One purpose of the temporary flow monitoring program was to obtain measured data during wet weather conditions to evaluate the impact of rainfall dependent I&I on isolated areas of the City's wastewater system. Unfortunately, the winter of 2002 was relatively dry. While February, March, and even early April are typically wet-weather months in the City, only minimal rainfall was recorded during the flow monitoring program. Rainfall gauges were installed to quantify rainfall values for this project at three locations. Rainfall occurred on two occasions during the study period: March 17, and March 23. The results of these events at each of the three rain gauge monitoring stations installed for this study are shown in Table 5-2.

To assess the impact of these events on wastewater flows, monitoring data was evaluated to identify changes in average daily flows. The results of this evaluation are also shown in Table 5-2.

As shown, there was little or no change in wastewater flows associated with these recorded rainfall events. As such, the data obtained during this field study did not provide conclusive evidence regarding the potential for significant inflow and infiltration (I&I) on the City's wastewater utility system. Given that one of the purposes associated with the temporary flow monitoring program was to assess the impacts of rainfall on the City's wastewater system, these minimal rainfall events yielded statistically insignificant results. The appropriateness of incorporating I&I allowances in the evaluation of the City's wastewater system is discussed in a subsequent section of this chapter.

Land Use Wastewater Generation Factors. As previously discussed, one purpose of the flow monitoring program is to quantify wastewater generation factors for specific land uses within the City. Given the significant cost of field flow monitoring, no monitoring stations were specifically dedicated to accomplishing this purpose. Rather, the monitoring program was mostly focused on gathering basin information for I&I. As such, this purpose was integrated with the other monitoring objectives.

TABLE 5-2

RAINFALL DURING WASTEWATER MONITORING PROGRAM

Rainfall Monitoring Stations		Rain Events	
Site #	General Location	3/17/2002 (in)	3/23/2002 (in)
1	Springdale & 405 Freeway	0.11	0.040
2	Maryland & Goldenwest	0.13	0.10
3	Banning & Magnolia/Bushard	0.070	N/A

		Monitoring Results on Dates of Rainfall Events					
		3/17/2002 [a]		3/23/2002 [a]		Wet Weather Weekend	
Monitor Site #	Nearest Major Intersection	Avg. Flow (MGD)	Peak Flow (MGD)	Avg. Flow (MGD)	Peak Flow (MGD)	Avg. Flow (MGD)	Peak Flow (MGD)
1	Graham & Edinger	0.407	0.623	0.531	0.916	0.469	0.7695
2	Edgewater & Courtney	0.837	1.444	0.862	1.587	0.8495	1.5155
3	Shorewood & Adams	0.664	1.143	0.676	1.169	0.67	1.156
4	Warner & PCH	0.741	1.187	--	--	0.741	1.187
5	Hamilton & Bushard	0.91	1.597	0.983	1.579	0.9465	1.588
6	Goldenwest & Slater	0.357	0.818	0.341	0.827	0.349	0.8225
7	Bushard & Hamilton	0.151	0.296	0.147	0.310	0.149	0.303
8	Lola & Yorktown	0.625	1.197	0.680	1.167	0.6525	1.182
9	Newland & Compton	0.193	0.351	0.190	0.334	0.1915	0.3425
10	Ofella & Palm	0.523	1.013	0.514	1.012	0.5185	1.0125
11	Newland & Warner	0.223	0.347	0.215	0.361	0.219	0.354
12	Ofella & Palm	0.166	0.311	0.176	0.329	0.171	0.32

		Dry Weather Weekend (Dates: 3/16, 3/24, 3/30, 3/31)		Wet vs. Dry Weekend Flows			
Monitor	Nearest Major	Avg. Flow	Peak Flow	Avg. Flows		Peak Flows	
Site #	Intersection	(MGD)	(MGD)	(MGD)	(%)[b]	(MGD)	(%)[b]
1	Graham & Edinger	0.4525	0.7055	0.0165	3.518	0.064	8.317
2	Edgewater & Courtney	0.8284	1.5715	0.0211	2.484	-0.056	-3.695
3	Shorewood & Adams	0.66925	1.1945	0.00075	0.112	-0.0385	-3.330
4	Warner & PCH	0.7413	1.3307	-0.0003	-0.0450	-0.1437	-12.103
5	Hamilton & Bushard	0.90825	1.508	0.03825	4.041	0.08	5.038
6	Goldenwest & Slater	0.3505	0.872	-0.0015	-0.430	-0.0495	-6.018
7	Bushard & Hamilton	0.14945	0.2845	-0.00045	-0.302	0.0185	6.106
8	Lola & Yorktown	0.639	1.15825	0.0135	2.069	0.02375	2.009
9	Newland & Compton	0.194	0.351	-0.0025	-1.305	-0.0085	-2.482
10	Ofella & Palm	0.51425	0.94425	0.00425	0.820	0.06825	6.741
11	Newland & Warner	0.221	0.42625	-0.002	-0.913	-0.07225	-20.410
12	Ofella & Palm	0.16675	0.33725	0.00425	2.485	-0.01725	-5.391

[a] Since 3/17/02 and 3/23/02 were weekends, only weekend flow values were used in this analysis

[b] Negative value indicates less flow during the wet day weekend than during the dry day weekend.

To accomplish this objective, the resulting flow monitoring data for each site was correlated with the acreage of each tributary land use. The land use loading factors (variables) were calculated by simultaneously solving the flow equations for each monitoring site. While the simultaneous equation process is a commonly used practice to calculate flow generation factors with mixed land flow data, it can result in variable results. The presence of pumping facilities that were located in some of the sub-basins further increased the calculation variability. As such, the results derived from this process must be considered with other general criteria to produce reliable results.

Average and Peak Criteria. In addition to providing supporting information to the development of land use generation factors, the temporary flow monitoring data provided additional data for the development of the City's peak wastewater conditions. The resulting average to peak relationship (weekend only) for each monitoring site was previously reflected in Table 5-2. The complete peak to average relationship for each monitoring site is summarized in Appendix B, with full data in Volume II of the Appendices. The development of the City's peaking factor for evaluating the wastewater system is further discussed in a subsequent section of this chapter.

Water to Wastewater Return Factors

The City's 2000 Water System Master Plan was reviewed to further evaluate the findings of the temporary flow monitoring wastewater generation factors. To perform this test, the Water System Master Plan loading factors for each land use type were reviewed for conversion to wastewater factors using typical return-to-sewer factors. This review resulted in lower duty factors for the lower density residential categories than anticipated. Since the Water System Master Plan methodology was based on using eight billing system categories to create its general demand factors, it is believed that the duty factors derived herein based on account-level water demands and field measured information provides a more appropriate representation of wastewater duty factors in the City.

UNIT DESIGN FLOW FACTORS

Based on the previous evaluation, the flow monitoring findings and prior study results were used as the primary data source to establish the City's design land use wastewater loading factors. The proposed land use loadings for average dry weather flow was based on a compilation of all of the evaluated data sources. The resulting 2002 wastewater flow generation factors are grouped and summarized in Table 5-3. The specific flow generation factors derived for the 35 land uses utilized herein is provided in Appendix D.

TABLE 5-3

RECOMENDED 2002 WASTEWATER DESIGN FLOW FACTORS

Land Use Category	Summarized Wastewater Flow Generation Factors
Residential	
Low Density (0-7 Du/Ac)	1,600 gpad
Medium (8-15 Du/Ac)	3,200 gpad
Medium - High (16-25 Du/Ac)	4,200 gpad
High - (25+ Du/Ac)	5,400 gpad
Commercial	2,000 gpad
Industrial	3,500 gpad
Open Space	200 gpad
Schools	3,600 gpad or 20 g/st/d

The development of these design unit flow factors utilizes flow monitoring data, water utility billing data, prior planning studies, and discussions with the City. Correlating these unit flow factors with the City's GIS-based land use data file provides a simple means of generalizing the distribution of sewage flows within the City's collection system under design loading conditions. Utilizing the land use categories and flow values provided in Appendix D and point load input values of high dischargers will provide a representative simulation of the loading data for input to the collection system model.

While the City is virtually built-out, it is important to integrate changes in current conditions into future loading factors. As such, the future loading factors were developed that integrated the following criteria:

Residential Unit Factors - Future. Existing developed units were compared to maximum allowable units based on individual parcel zoning. Accordingly, where appropriate, unit flow factors were increased to simulate allowable increases in DU densities. Consistent with City planning data and the 2000 Water System Master Plan findings, future loadings were increased based on increases in pph and future residential dwellings. As such, residential wastewater generation factors were increased by six percent.

Non-Residential Unit Factors - Future. Changes in non-residential unit factors are subject to many factors. Among these are the allowable changes in building heights, redevelopment trends associated with interior water use, changes in local employment, and changes in local population using local commercial services. While the future non-residential duty factors were held constant in the City's 2000 Water System Master Plan, it is recommended herein that future non-residential unit factors be increased by the change in local population. Accordingly, the six percent increase was also applied to future non-residential wastewater generation factors.

Vacant Land Design Criteria. Vacant land was developed based on the maximum allowable zoning and projected person per household factors that are incorporated in the future design loading factors.

These flow factors were subsequently input into the computer model to simulate future ADWF flow conditions. A discussion of the hydraulic modeling analysis is contained in Chapter 6.

GENERAL CRITERIA

The hydraulic analysis described in Chapter 6, compares collection system pipeline flows and lift station flows to calculated design capacities for those facilities to identify hydraulic deficiencies within the City's collection and pumping system. Accordingly, the design capacities and criteria developed in this section are used for system analysis in subsequent chapters.

Peaking Factor Criteria

Average flows entering the trunk collection systems are assessed by correlating the area of each land use type with its associated wastewater flow generation factors. However, a determination of the adequacy of the wastewater system is based upon the ability of the system to convey peak flows. Peak flow in any reach of the wastewater system is equivalent to the summation of all average flows upstream of the point in question and converted to peak flow by an empirical peak-to-average relationship. This relationship as expressed in the OCSD 1989 Master Plan Study is as follows:

$$Q_{peak} = 1.78\ (Q_{avg})^{0.92},\ (Q \text{ in mgd})$$

This peaking factor equation was initially developed during preparation of the 1969 Districts No. 3 and No. 7 (Huntington Beach) master plans and was reconfirmed by flow metering data gathered through the conduct of the 1989 study. This equation is nearly identical to the equation developed in the City's 1978 Huntington Beach Sewer Master Plan. The City's 1978 Master Plan equation is as follows:

$$Q_{peak} = 1.704\ (Q_{avg})^{0.892},\ (Q \text{ in mgd})$$

The OCSD peaking equation was selected for the City's 1995 Wastewater study as it was based on more recent data and yielded slightly higher peak flows, resulting in a more conservative peak to average flow relationship. To accommodate future growth, today, OCSD utilizes a 2.5 factor for 8-inch pipe and a 2.0 factor for all other diameters.

An equation that represented the current peak to average relationship within the City was derived based on the flow monitoring data obtained through the conduct of this study. This relationship is expressed as follows:

$$Q_{peak} = 1.93\ (Q_{avg})^{0.898},\ (Q \text{ in mgd})$$

This equation is proposed for the City's wastewater system and is illustrated in Figure 5-3. This equation is based on the data obtained during the recent flow monitoring activity performed for the City. As shown, the 2002 peaking factor provides additional peaking under low flow conditions. Figure 5-4 illustrates the application of the peak-to-average relationship to a hypothetical reach of the wastewater system.

Inflow and Infiltration

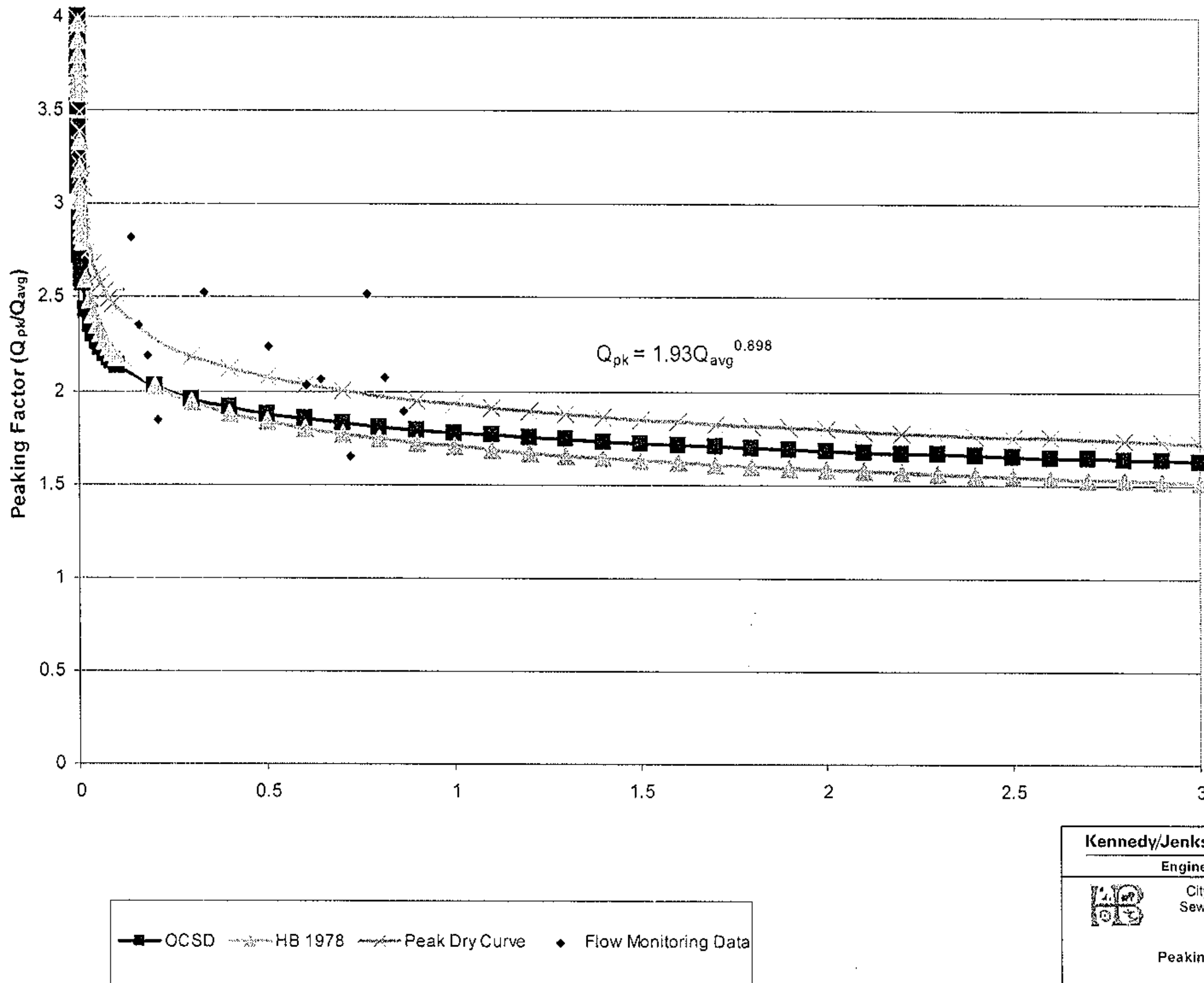
There are several commonly accepted practices used to estimate I&I. These practices include estimating I&I based on tributary area served (1,000-1,500 gpd/ac), tributary linear feet of pipeline based on diameter (14-28 gpd/inch dia/100 LF), or as a percentage of the average flow or pipeline capacity (typically 10-25 percent). The age and condition of underground facilities, groundwater elevation conditions relative to the location of underground utilities, and surface water drainage patterns are typical considerations used in developing appropriate I&I factors.

I&I is generally quantified based on measured wastewater flows preceding, during, and following a wet weather event. As previously discussed, the temporary flow monitoring program conducted during this study was performed during a typically wet weather period. Unfortunately, only trace levels of rainfall were recorded during the conduct of the temporary flow monitoring program, resulting in negligible change in wastewater flows.

To supplement the quantifiable results of a field flow monitoring program, a desk top I&I study was performed. This evaluation, described previously in Chapter 4, identified the potential for I&I in localized areas of the City. The premise of this finding is the fact that several of the City's lift stations incurred a significant increase in daily lift station run time on the day of a substantial rainfall event.

In consideration of this finding, it is recommended that the City:

- Coordinate with OCSD for additional data and findings of its ongoing I&I evaluation in the City's service area. Wet weather data should be available from OCSD in the



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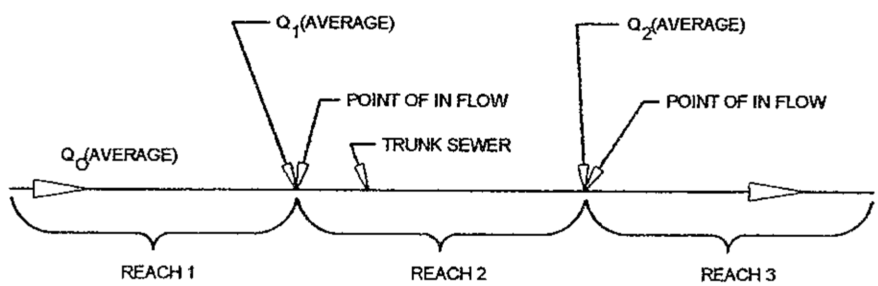
Peaking Factor

Figure 5-3

RELATIONSHIP SCHEMATIC



REACH 1	REACH 2	REACH 3
$Q_{AVG} = Q_0$	$Q_{AVG} = Q_0 + Q_1$	$Q_{AVG} = Q_0 + Q_1 + Q_2$
$Q_{PK} = Q_0 \times P/A$	$Q_{PK} = (Q_0 + Q_1) \times P/A$	$Q_{PK} = (Q_0 + Q_1 + Q_2) \times P/A$
$Q_{PK} = Q_{AVG} \times P/A$	$Q_{PK} = Q_{AVG} \times P/A$	$Q_{PK} = Q_{AVG} \times P/A$

LEGEND

Q_0, Q_1, Q_2 = AVERAGE DAILY FLOW

Q_{AVG} = AVERAGE FLOW FOR REACH

Q_{PK} = PEAK FLOW FOR REACH

P/A = PEAK TO AVERAGE RELATIONSHIP

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AVERAGE AND PEAK DESIGN FLOW
RELATIONSHIP SCHEMATIC

Figure 5-4

fall of 2003.

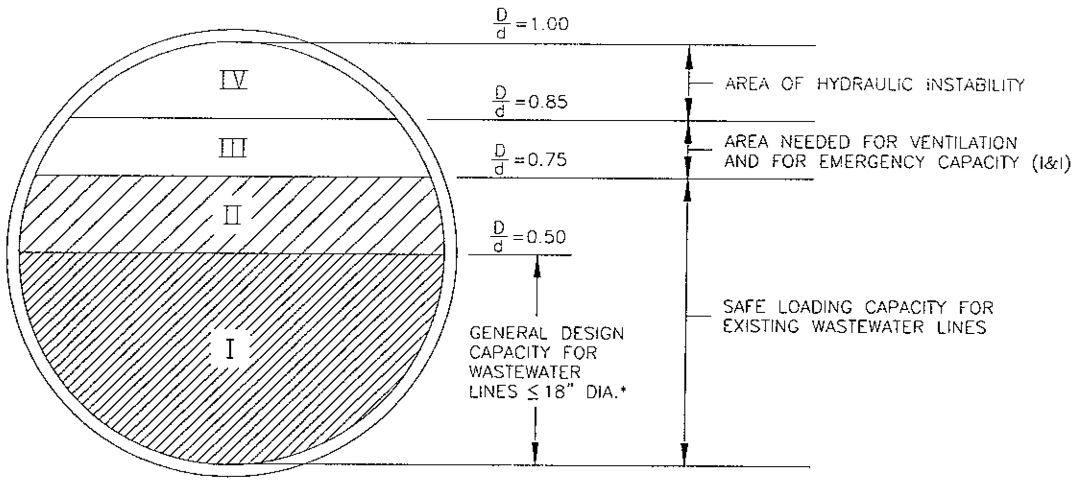
- Perform a video inspection program to verify underground utility pipeline conditions and document the presence of any illegal storm drainage connections to the wastewater system, and
- Perform additional I&I analysis during a future wet weather event to further quantify and isolate the rainfall dependent I&I condition in the City. This activity could be scheduled during the winter of 2003-04 to better utilize the OCSD and video inspection data.

The combination of these proactive activities by the City should provide an effective and methodical implementation strategy for the City's I&I Reduction Program. The implementation strategy integrates the study work activities, focuses on the identified potential I&I problem areas, proceeds based on the prioritization of these potential problem areas, and concludes with the need to conduct specific subsequent Sanitary Sewer Evaluation Studies (SSES) to mitigate sources of I&I in the collection system.

DESIGN/CAPACITY CRITERIA

In analyzing a wastewater system, it is necessary to derive standards regarding the amount of flow that may be efficiently conveyed by a given wastewater pipeline. A cross-section of such a pipeline is shown in Figure 5-5. The area of the pipe has been divided into four sections, indicating the ratio of the depth of flow to the diameter of the pipe (D/d) at various locations. In general, the design and analysis of wastewater pipelines is based upon a D/d that will safely and efficiently convey wastewater from its point of origin to the treatment facilities.

At the time of wastewater pipeline design, there is often some uncertainty as to future development patterns within the area to be served. To deal with this uncertainty, provision is usually made for some extra pipeline capacity to allow for the possibility of actual wastewater flows being slightly higher than the anticipated flows.



TYPICAL PIPELINE LOADING CONDITIONS

* FOR WASTEWATER LINES ≥ 18" DIA. A D/d AS LARGE AS 0.75 IS GENERALLY CONSIDERED SAFE.

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TYPICAL PIPELINE LOADING CONDITIONS

Figure 5-5

The National Clay Pipe Institute (NCPI) recommends that smaller pipelines generally be designed to flow at levels not exceeding half-full (D/d=0.50) during peak conditions, as shown in Zone I on Figure 5-5. For larger wastewater pipelines having an internal diameter greater than 18 inches, the tributary area is larger. Local deviations from design wastewater flows tend to balance one another for larger areas, resulting in a closer correlation of actual and design wastewater flows. Consequently, the NCPI recommends that these larger wastewater pipelines should be designed for a D/d not to exceed 0.75.

In analyzing existing wastewater pipelines, it is usually unnecessary to allow for a large factor of safety. This is because tributary areas are largely built out, future development patterns are relatively certain, and flow rates can be attained by flow monitoring these facilities. Therefore, the wastewater pipelines may be flowing at levels above a design D/d of 0.50 and still be operating satisfactory.

Zone III on Figure 5-5, has been reserved to handle emergency flows, such as storm water I&I, and provide for ventilation within the pipe. Zone IV, on Figure 5-5, should not be considered as an integral component of the pipeline capacity. This area is subject to variable hydraulic instability because the additional volume for flow is counteracted by the additional friction that occurs between the top, or soffit, of the wastewater pipeline and the fluid.

Calculation of Design Capacity - Gravity Pipelines

Design capacity of a pipeline shall be the calculated capacity of the pipeline using the Manning Equation. ADWF for each pipeline is derived from the computer model. The peaking factor is applied to ADWF to obtain peak dry weather flow (PDWF). Consistent with the criteria used for most built out communities, the design criteria used to evaluate the City's existing pipeline conditions are based on a PDWF that does not exceed 0.75 D/d. These criteria implicitly reserve the remaining pipeline capacity to accommodate flow variations and PWWF incurred during wet weather conditions.

As discussed with City staff, the City's design criteria are used to evaluate and size the future facility requirements. These criteria are essentially based on the NCPI and acknowledge the potential for flow variations and levels of safety based on pipe size. The City's wastewater flow design criteria is stated as follows: "The design peak flow rate inpipes 12" and smaller will be limited by the depth ratio of D/d = 0.5, 15" pipes D/d = 0.67, and 18" and larger pipes D/d = 0.75, where D/d is the ratio of calculated flow depth to pipe inside diameter. The hydraulic and financial implications of applying these evaluation criteria are evaluated in Chapter 6.

The design capacity (Q) of collection system pipelines will be established using the continuity equation, the Manning Equation, and criteria as follows.

The continuity equation for flow is $Q = V A$, where:

Q = flow in cubic feet per second
V = velocity in feet per second
A = cross-sectional area of flow in square feet

The Manning Equation used to estimate the flow velocity in gravity pipelines is

$$V = (1.486 / n) R^{2/3} S^{1/2} \quad \text{where:}$$

V = velocity of flow in feet per second
A = cross-sectional area of the pipe in square feet
R = hydraulic radius in feet
S = pipeline slope in feet of rise per foot of length
n = Manning friction factor (for existing vitrified clay pipe is 0.013)

Minimum Velocity. From an operational perspective, a minimum peak flow velocity of 2.0 feet per second (fps) at PDWF is desirable to adequately scour the pipeline and prevent significant solids deposition. Pipelines in the system that do not develop adequate cleansing velocity (flat pipelines, low spots, or pipelines with low flow) should be given priority status in the City's pipeline cleaning program.

Calculation of Design Capacity – Lift Stations

The evaluation of a wastewater lift station is based on two primary criteria. These criteria include the ability of the lift station to reliably pump the PWWF and wet well adequacy for pump cycling.

Pumping Capacity. The design pump capacity requirement is consistent with the methodology used in the collection system model. A lift station will be considered over capacity if it cannot pump the PDWF with one pump out of service and the remaining pumps operating at 75 percent of the station's maximum pumping capacity. The remaining 25 percent capacity is allocated for I&I, reserve capacity contingency, and variation in wastewater flow. Standby power provisions are also an integral element of the lift station reliability.

Wet Well Size/Cycling Requirement. Wet well adequacy for fixed speed pumps is analyzed in terms of maximum pump cycles per hour. A typical pump motor is designed for a maximum of six starts or cycles per hour. If the motor is started more than six times in an hour, it may overheat the motor starters, causing them to wear prematurely and fail. The maximum number of cycles per hour corresponds to the minimum cycle time, which is calculated using the pumping rate, the wet well dimensions, and the pump on/off control points. The cross-sectional area of the wet well and the pump control points determine the operational wet well volume. For example, when the wastewater in the wet well reaches the pump's upper control point, the pump turns on and draws down the wet well wastewater level. When the wastewater level reaches the pump's lower control point, the pump turns off and the wet well begins to refill.

The time between pump starts is the cycle time. The minimum cycle time occurs when the flow rate into the wet well is half the pumping rate. Under these conditions, the water level in the wet well rises between pump control points in x minutes, would be pumped down in x minutes, and the cycle time would be 2x minutes.

Chapter 6

Wastewater System Evaluation

CHAPTER 6

WASTEWATER SYSTEM EVALUATION

This chapter evaluates the City's existing wastewater collection system's ability to convey existing peak wet weather flows from current land uses, and future peak wet weather flows associated with potential redevelopment and new development of vacant lands at the maximum permitted zoning densities. As previously discussed, flexibility for future redevelopment is established using a system-wide design contingency. The concept of a capacity contingency is a common consideration to account for the undefined size and location of future redevelopment projects.

OVERVIEW

The primary backbone wastewater infrastructure within the City limits is owned and operated by the OCSD. Consistent with the City's prior 1995 Master Plan and current City direction, the OCSD facilities were not included in the evaluation portion of this study as this plan was designed to assess the hydraulic adequacy of City-owned pipelines.

Since the OCSD system provides the overall basin connectivity between City-owned pipelines, the City's wastewater system is hydraulically modeled as if it were a number of disconnected sub basins. Accordingly, the modeling hydraulic calculations were performed without the effect of a backwater analysis associated with OCSD connections. A combined City and OCSD analysis may be warranted during a future master plan update.

The wastewater collection system is evaluated for existing and future conditions using a hydraulic model called Hydra, a steady state computer simulation model developed by Pizer, Inc. The model is developed using the physical system information obtained from the wastewater utility system and land use data defined in the City's GIS and further developed herein. Collection pipelines and lift stations are evaluated based on their ability to convey the projected peak wet weather flow. Land use type and acreage tributary to system manholes are then linked and average flows are calculated using the general and specific flow generation criteria presented in Chapter 5.

Although the City's lift stations are included in the model for connectivity when appropriate, they are not evaluated by the hydraulic model. These facilities are evaluated separately, using the flow information developed by Hydra and City-provided facility data. Hydraulic deficiencies within the existing system are identified for current and future flow conditions and planning level recommendations are suggested to remediate these deficiencies.

As discussed, a system-wide reserve capacity contingency is established in the model to provide flexibility for variations in flows and to accommodate future redevelopment projects. This contingency should provide flexibility for redevelopment within the City. Actual redevelopment projects should be evaluated by the City on a case-by-case basis. As such, some especially large or high density projects may require capacity improvements to provide adequate service.

COLLECTION SYSTEM EVALUATION

Model Overview

The wastewater system hydraulic model (Hydra) transforms physical system information, flow generation criteria, and analytical criteria into a mathematical model that simulates hydraulic conditions in the wastewater system. Hydra is a steady state computer model that simulates the hydraulic conditions of the gravity flow collection system. The model calculates flows at each manhole from the associated tributary area and sums the flow along each flow path. In addition, the model calculates the capacity of each pipeline within the system and compares the pipeline capacity with the calculated flow to identify hydraulically deficient conditions and to size necessary improvements.

Constructing a hydraulic model requires the development and integration of three basic system elements. These elements include the wastewater facility data file, the drainage basin data file, and the demand data file. Hydra is designed to utilize the unique linkage among these data elements and develop the hydraulic simulation of the wastewater conveyed throughout the collection system. Each of these three modeling data elements is discussed in the following sections.

Wastewater Facility Data. The facility data element is comprised of the physical elements of the wastewater system to be modeled. Physical elements include pipeline diameter and roughness, the length and slope between manholes, manhole invert elevations, and the output capacity of the City's lift stations. As previously discussed, these physical elements were provided by the City O&M and GIS staff. This data was supplemented with record drawings of the sewer system to resolve data conflicts. The updated data was provided to the City to enhance its GIS database. Specific wastewater pipelines were identified for simulation through the use of the computerized hydraulic model based on discussions with City staff and our research and understanding of the collection system.

Drainage Basin Data. To support the hydraulic simulation and evaluation of the selected facilities, the identified areas were divided into smaller service areas or sub-basins. Integration of these interconnected subsystems provides a more realistic simulation of actual field conditions and increases the accuracy of the hydraulic evaluation findings.

Demand Data. The demand data establishes the wastewater flows within each of the sub-basins derived within the drainage basin data. The flows associated with these demands are calculated by correlating land use flow generation factors with the acreage/units of each land use within each sub-basin. Peak wastewater flows are derived by applying the peaking factor equation previously discussed

In addition to the general loading criteria by acres/units per land use type, actual flow conditions were integrated into the modeling simulation through the use of parcel level loadings. Through this process, all parcels in the City were correlated with their respective account in the City's utility billing system through an Assessor Parcel Number (APN) to utility billing system account number linkage. Actual account-level water consumption data was subsequently converted to wastewater in the hydraulic model. The digital results of the APN to billing system linkage were provided to the City under a separate cover. A copy of the summary analysis is provided in Appendix C.

The corresponding location, acreage and tributary sub-basin for each account/discharger were established using the City's GIS. The results obtained through this approach generally have a high correlation with known wastewater flow data.

Computer Modeling

An important element of computerized hydraulic modeling simulations is the calibration of the model to actual field conditions. Calibration is a multi-step process by which planning level values are reviewed and adjusted to known demand conditions, increasing the confidence level in the results of the hydraulic simulations, engineering analysis, and resulting recommendations. The process and results of calibrating the wastewater system hydraulic model is described herein.

Upon completion of the three data elements, the model is run to integrate the data and construct a hydraulic simulation. The model input/output was further reviewed and data discrepancies resolved in the appropriate data element of the model.

To accomplish model calibration, the wastewater flows developed by the hydraulic model (Hydra) are compared to actual flow monitoring station data obtained during the conduct of this study. Through a review of this data, variances are analyzed and appropriate land use discharge values are adjusted to correlate the model-developed flows with known wastewater monitoring station values.

As previously discussed, the monitoring program was implemented with several purposes, resulting in a narrow set of focused data for each objective. An important consideration in the calibration process is the need for the flow monitoring data to be derived from gravity flow and void of the storage and discharge impact associated with upstream pumping facilities. While the pumping facilities were an important element of the I&I study, their unknown on/off time-of-day operational status imposes a complex variable in the data interpretation process. After deleting the sites with lift station or extraneous flow contributions, four monitoring sites (1, 7, 10, and 11) remained for focused support of the calibration process.

The results of the calibration process indicated that the integration of the land use loading factors/parcel level demand loadings with the system drainage basin and land use data files achieved a high correlation with the flow conditions of these basins/sub basins. A summary of the temporary flow monitoring station average wastewater measurements in contrast with the estimated flows predicted by the model is provided in Appendix D. A tabular listing of various factors used in the model calibration process is also provided in Appendix D.

In addition to this localized calibration result, it should be noted that overall City-wide calibration was also confirmed. The City-wide calibration process focused on the use and summary analysis of the account level loadings from the water billing data. As such, the model was spatially loaded based on the physical connection of the parcel to the modeled pipeline and the actual water usage/wastewater discharge. The number of dwelling units being served at that water connection was integrated, irrigation accounts were excluded, and the characteristics of non-residential and public open space accounts were integrated in the model with their actual loading values.

The results of this calibration and quality control review process confirmed the appropriateness of the City-wide modeling analysis. Model input evaluation confirmed that citywide demands were reflected in the spatial parcel-level demand data and that approximately ninety-nine percent of the City's land was accounted for in the acreage loading values of the model.

Based on this correlation and supporting information, it is believed the established hydraulic model is calibrated to a reasonable level of confidence and provides an appropriate simulation of current citywide wastewater flow conditions. As such, the model can serve as an appropriate tool for predicting potential areas of future hydraulic deficiency and performing various "what if" scenarios.

Collection System Hydraulic Deficiencies & Recommended Improvements

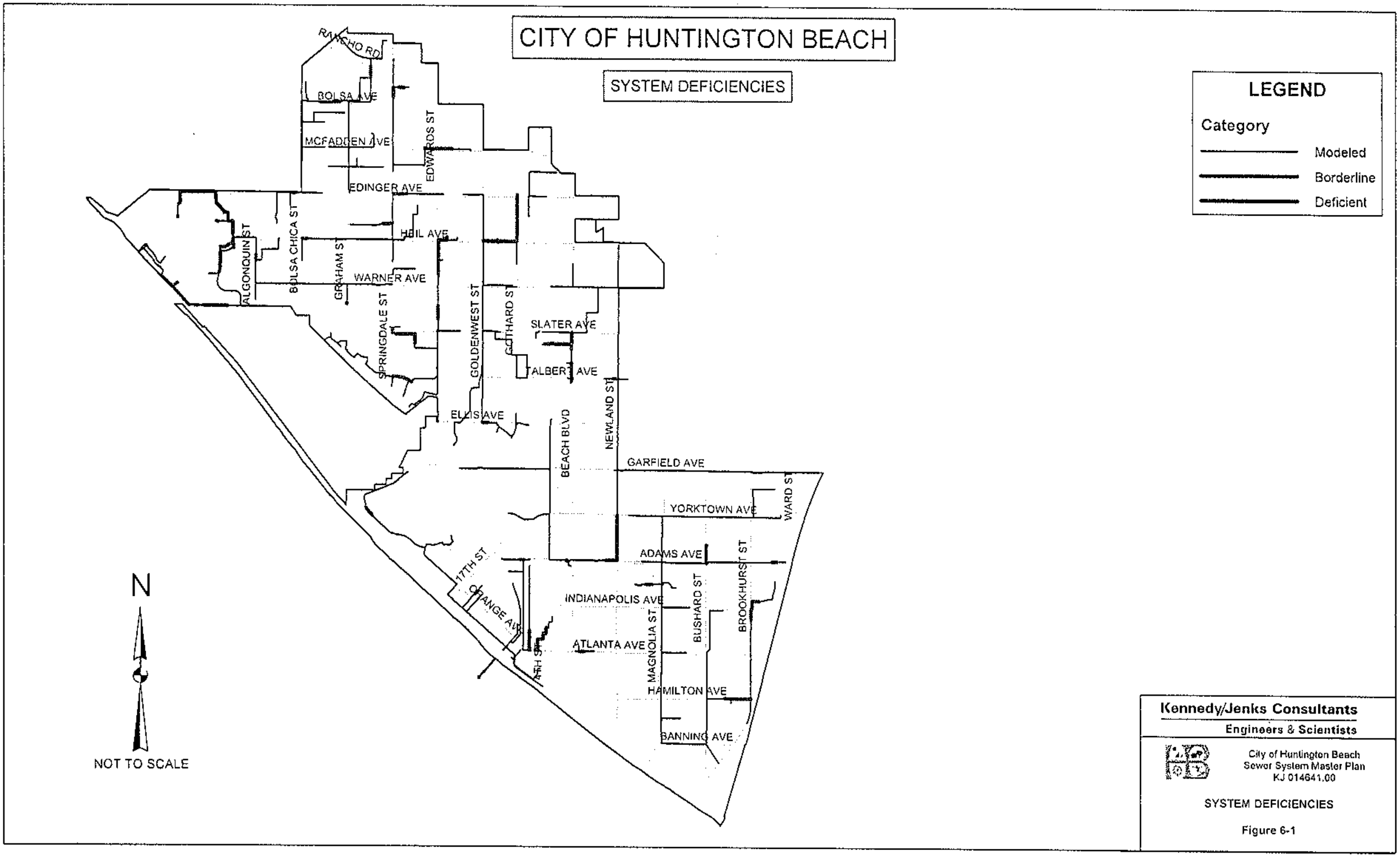
The updated model was subsequently loaded with the 2002 design loading factors previously derived and the City's wastewater system analyzed for hydraulic deficiencies under current conditions. As previously discussed, localized pipeline facilities were evaluated based on the prescribed design criteria. As a base line analysis, the pipelines that exceed the design capacity criteria based on the flow depth to pipe diameter ratio or exhibiting surcharge conditions under peak flow values would be considered deficient.

In addition to the above analysis a modeling simulation was performed to evaluate future flow conditions. The future flow analysis was performed by incorporating the future land use wastewater generation factors for both developed and vacant land. The City's hydraulic capacity design criteria based on a D/d that varies by pipe diameter. This analysis is performed to simulate build out conditions.

An important consideration in the evaluation of the modeling analysis is the relative degree of deficiency. For example, while a facility that has a future estimated D/d of .52 may be "deficient" in accordance to the design criteria, this facility would probably not warrant a near-term investment for additional capacity. As such, a level of engineering judgment is necessary to segregate between "deficient" and "borderline conditions." The results of the hydraulic analysis and interpretation of findings are graphically shown in Figure 6-1, with the tabular findings presented in Appendix D.

Based on the output from the collection system model for existing (calibration), short-term, and future (2020) loading conditions, hydraulic deficiencies are identified and generally prioritized within the existing system. Two options were considered for remediation of the hydraulic deficiencies: construction of a parallel pipeline to relieve flow from the overcapacity pipelines, or construction of a larger replacement pipeline with adequate design capacity for the projected peak flows.

Generally, the benefit of using a parallel pipeline is lower material and construction cost. This is because a parallel pipeline requires a smaller pipeline diameter, fewer service reconnections, and may eliminate or reduce bypass pumping requirements. The disadvantage of using a parallel pipeline is that it increases overall O&M costs by adding new pipelines to the system that require cleaning and maintenance, and in some cases, existing utilities may not provide an adequate corridor for construction. For purposes of this master plan, pipeline replacement is used as the basis for estimating the


CITY OF HUNTINGTON BEACH
SYSTEM DEFICIENCIES
LEGEND
Category
Modeled
Borderline
Deficient
RANCHO RD
BOLSA AVE
MCFADDEN AVE
EDWARDS ST
EDINGER AVE
HEIL AVE
WARNER AVE
ALGONQUIN ST
BOLSA CHICA ST
GRAHAM ST
SPRINGDALE ST
GOLDENWEST ST
GOTHARD ST
SLATER AVE
TALBERT AVE
ELLIS AVE
BEACH BLVD
NEWLAND ST
GARFIELD AVE
YORKTOWN AVE
WARD ST
ADAMS AVE
17TH ST
ORANGE AVE
INDIANAPOLIS AVE
ATLANTA AVE
4TH ST
MAGNOLIA ST
BUSHARD ST
BROOKHURST ST
HAMILTON AVE
BANNING AVE
N
NOT TO SCALE
Kennedy/Jenks Consultants
Engineers & Scientists
City of Huntington Beach
Sewer System Master Plan
KJ 014641.00
SYSTEM DEFICIENCIES
Figure 6-1

improvement costs presented in Chapter 7. Using the pipe replacement concept for planning provides the City with the flexibility to decide on paralleling or pipeline replacement at the time of final design. Prior to initiating final design, the Engineering and Operations staff should field verify the PDWF in these pipelines to validate that they operate at or near the prescribed level of existing capacity.

Prioritization of identified hydraulic deficiencies is based on a comparison of the results from the modeling evaluations. In general, deficiencies identified under existing ADWF conditions should be a high priority, deficiencies identified under existing PDWF (but not under existing ADWF) should be a medium to medium high priority and deficiencies identified under future PDWF conditions only should be a low priority.

The recommended replacement diameter for all projects was based on maximum future wastewater flow conditions (2020). Replacement diameters for the identified deficient pipeline segments are included in Appendix D. Cost estimates for these projects are presented in Chapter 7.

LIFT STATION EVALUATION

Each of the City's twenty-seven lift stations was evaluated to assess its ability to convey the future peak flow. The future peak flow was compared to each facility's pumping capacity and an evaluation of the wet well operational performance was performed to verify that pump motors do not cycle (start/stop) too frequently, resulting in excessive electrical costs and premature motor failure. The evaluated criteria, results, and recommendations are presented in the following sections.

The evaluation of each lift station is based on two criteria. These are

1. The ability of a single pump to accommodate PWWF conditions; relative to maximum pump operating capacity.

2. The adequacy of wet well and pump sizing based on pump cycling rates without modification or replacement of pump intervals or motors.

Lift Station Capacity Evaluation

As previously discussed in Chapter 3, the City's standard lift station configuration is comprised of two identical pumps operating in parallel. Accordingly, one pump operates while the second pump serves as a backup, either to assist the first pump or operate alone if one pump becomes inoperable. A lift station will be considered over capacity if it cannot pump the PDWF with one pump out of service and the remaining pumps operating at 75 percent of the station's rated capacity without modification. The remaining 25 percent capacity is allocated for I&I, reserve capacity contingency, and variation in wastewater flow.

Table 6-1 provides the lift station capacities using this firm pump capacity criteria. The adequacy of each lift station capacity was evaluated based on the estimated future PDWF from the hydraulic model and the capacity criteria for each facility. The results of the analysis are shown in Table 6-1. As shown, this analysis indicates those facilities where the estimated PDWF exceeds the single pump operating criteria.

Wet Well Cycling Operational Evaluation

Using the calculated operating wet well volume (V) and the design pump output (Q), minimum cycle times (CT) were calculated with the following equation:

$$CT = 4V/Q$$

where:

V is in gallons.

Q is in gpm.

CT is in minutes.

TABLE 6-1
PUMP CAPACITY DEFICIENCIES

Lift Station (# Name)	Modeled Pump Capacity [1] (gpm@TDH)	PDWF Firm Design Capacity[2] (gpm)	Number Of Pumps	Existing PDWF (gpm)	Ultimate PDWF (gpm)	Existing Capacity Deficiency[3]	Ultimate Capacity Deficiency[3]
#1 Graham	580 @ 55	435	2	361	379	NO	NO
#2 Humboldt	155 @ 22	116	2	131	138	YES	YES
#3 Station "E"	100 @ 18	75	2	48	51	NO	NO
#4 Statioin "A"	572 @ 20	429	2	282	292	NO	NO
#5 Davenport	106 @ 12	80	2	93	98	YES	YES
#6 Edgewater	450 @ 12	338	2	732	977	YES	YES
#7 Station "B"	670 @ 10	503	2	460	462	NO	NO
#8 Station "C"	1170 @ 15	878	2	660	668	NO	NO
#9 Station "D"	900 @ 50	2700	4	1541	2147	NO	NO
#10 Algonquin	1000 @ 60	750	2	475	509	NO	NO
#11 Lark	125 @ 12	94	2	83	88	NO	NO
#13 Slater	1070 @ 24	803	2	691	713	NO	NO
#14 Ellis	850 @ 34	1275	3	444	466	NO	NO
#15 Beach	150 @ 30	112	2	125	133	YES	YES
#16 Adams	220 @ 13	165	2	196	244	YES	YES
#17 Brookhurst	1280 @ 28	960	2	617	710	NO	NO
#18 Atlanta	350 @ 25	263	2	308	358	YES	YES
#19 Bushard	315 @ 10	236	2	93	97	NO	NO
#20 Speer	400 @ 14	300	2	44	47	NO	NO
#21 McFadden	160 @ 32	120	2	111	117	NO	NO
#22 Saybrook	550 @ 23	413	2	619	739	YES	YES
#23 New Britain	179 @ 11	134	2	197	208	YES	YES
#24 Edwards	800 @ 38	600	2	552	574	NO	NO
#25 Edinger	300 @ 12	225	2	415	423	YES	YES
#26 Brighton	220 @ 16	165	2	188	200	YES	YES
#28 Coral Cay	80 @ 14	60	2	68	72	YES	YES
#29 Trinidad	250 @ 15	188	2	145	153	NO	NO

Source data provided by City.

(1) Capacity is defined per City maintenance department data as one pump in each station designated standby.

(2) PDWF firm design capacity is calculated using individual pump capacity at 75% of design, leaving 25% of design capacity for peak wet weather flow. One pump in each station was designated standby.

(3) Station deemed deficient when PDWF firm design capacity was below PDWF.

(4) These stations have been recently improved or are currently in design for improvement.

The cycle time calculated by this equation is based on one pump operating at a time. However, since each lift station contains multiple alternating pumps, the number of pumps must be integrated in the cycle time analysis for each facility.

The analysis of the pump cycle time operational analysis is shown in Table 6-2. As shown, the analysis indicates those facilities that cycle in excess of the generally accepted six cycles per hour criteria. It should be noted however, that the excessive cycling may also be related to the quantity of operational wet well volume associated with the on/off pump control settings at each facility. As such, the City should consider the benefit of increasing the operational wet well volumes of these facilities versus the potentially adverse impact of additional localized odors that may result from increased wet well storage time.

Lift Station Hydraulic Deficiencies and Recommendations

As shown, the City's lift stations require additional improvements to meet current/future demand conditions. Prior to the design and construction of the findings derived herein, the City should perform additional field investigation and perform related engineering calculations during pre-design activities. Field confirmation of actual pump capacities, operating conditions, and influent flow requirements should be included in this design effort. Alternatively, it may be more desirable to perform a comprehensive evaluation of the pump capacities and efficiencies at each of the City's lift stations. In support of this evaluation and provide ongoing wastewater pump station performance information, the City should also consider the installation of permanent metering equipment that provides ongoing lift station influent and output data through telemetry. Cost estimates of the recommended improvements are provided in Chapter 7.

TABLE 6-2
WET WELL OPERATIONAL CAPACITY DEFICIENCIES

Pump Station Number & Name	Modeled Pump Capacity (1) (gpm@TDH)	Number Of Pumps	Wet Well Operational Capacity(2) (gal)	Pump Cycling (cycles/hr)	Wet Well Operational Capacity Deficiency(3)
#1 Graham	580 @ 55	2	318	14	YES
#2 Humboldt	155 @ 22	2	117	10	NO
#3 Station "E"	100 @ 18	2	70	11	NO
#4 Statioin "A"	572 @ 20	2	558	8	NO
#5 Davenport	106 @ 12	2	133	6	NO
#6 Edgewater	450 @ 12	2	184	18	YES
#7 Station "B"	670 @ 10	2	310	16	YES
#8 Station "C"	1170 @ 15	2	745	12	NO
#9 Station "D"	900 @ 50	4	509	7	NO
#10 Algonquin	1000 @ 60	2	282	27	YES
#11 Lark	125 @ 12	2	94	10	NO
#13 Slater	1070 @ 24	2	211	38	YES
#14 Ellis	850 @ 34	3	355	12	NO
#15 Beach	150 @ 30	2	465	2	NO
#16 Adams	220 @ 13	2	94	18	YES
#17 Brookhurst	1280 @ 28	2	441	22	YES
#18 Atlanta	350 @ 25	2	470	6	NO
#19 Bushard	315 @ 10	2	470	5	NO
#20 Speer	400 @ 14	2	846	4	NO
#21 McFadden	160 @ 32	2	188	6	NO
#22 Saybrook	550 @ 23	2	294	14	YES
#23 New Britain	179 @ 11	2	564	2	NO
#24 Edwards	800 @ 38	2	909	7	NO
#25 Edinger	300 @ 12	2	211	11	NO
#26 Brighton	220 @ 16	2	188	9	NO
#28 Coral Cay	80 @ 14	2	211	3	NO
#29 Trinidad	250 @ 15	2	220	9	NO

(1) Capacity is defined per City maintenance department data for one pump in operation only. Multiple pumps within a station are considered to alternate start/stop cycles equally.
(2) Capacity is based on wet well dimensions and multiple pump start/stop settings, provided by City staff.
(3) Although new pump stations are designed for no more than 6 cycles/hr, the existing stations were not considered deficient until they exceeded 12 cycles/hr.

Chapter 7

Costs of System Improvements

CHAPTER 7

COSTS OF SYSTEM IMPROVEMENTS

This chapter incorporates the findings of the previous chapters and outlines the estimated costs of the recommended collection system and pumping station capital improvements. The identified improvements are subsequently prioritized into a capital improvement program based on the facility condition and the hydraulic analysis under current and future loading conditions. These capital improvement costs, schedules and assumptions are contained herein.

Wastewater system improvements are generally established based on two distinct categories: facility condition and hydraulic adequacy. Facility condition improvements are required to upgrade/improve aging facilities and are corrected by replacement or repair-related rehabilitation activities. Hydraulic improvements are required to accommodate the current and projected flows within the City's wastewater facilities. The identification of these improvements is based primarily on the results of the computerized hydraulic model discussed in Chapter 6, and the evaluation criteria discussed in Chapter 5. The costs of the recommended collection system capital improvements are separated into these two categories and discussed in the subsequent sections of this study.

PROJECT PRIORITIZATION

As previously discussed, hydraulic modeling simulations were conducted under current conditions and projected maximum loading conditions at the year 2020. This process resulted in the identification of specific deficiencies and the associated remedial measures. Prioritization of the recommended improvement should be based on the degree of deficiency, facility reliability related to the potential for and implications of failure, coordination with other utility needs and objectives, and funding availability. As such, the City should balance its capital improvement program between the hydraulic pipeline deficiencies and the sewer lift stations, with the lift station replacement program receiving the most attention.

CAPITAL COST ESTIMATES

This section presents the capital construction costs for the proposed wastewater collection system and pumping facilities. Details of the development of the capital cost estimates are discussed in the following sections.

Unit Costs

The capital cost estimates for the proposed facilities were developed based on the Engineering News Record Construction Cost Index (ENR-CCI) 20-city national average. The ENR-CCI is an inflation index used to adjust prices from one time period to another. The cost estimates presented in this master plan are based upon an ENR-CCI cost index of 6462 for January 2002. Cost estimated herein for recommended facilities should be adjusted in the future either by making new estimates or by comparing the future ENR-CCI–20-City index to 6462.

The capital costs derived herein are based on unit costs obtained from recently designed and constructed projects. These unit construction costs are approximate planning costs and include miscellaneous work such as manholes, that are necessary for complete and operable facilities, but they do not include right-of-way acquisition. Unit cost estimates are based on pipe materials, size, depth of construction, manhole spacing, trench width, etc. These defined cost parameters are used to estimate the design and construction costs of underground facilities.

Engineering, administration services and contingencies have been included as a percentage of total construction costs. A factor of 20 percent of total construction cost has been used for engineering and administration, which include but are not limited to the following:

- Planning and design reports
- Design
- CEQA Compliance
- Permits
- Surveying

- Services during construction (submittals, as-builts)
- Inspection

In addition to these items, a 20 percent contingency was added. Table 7-1 presents gravity sewer unit costs useful in the development of capital costs.

TABLE 7-1
GRAVITY SEWER UNIT COSTS

Pipe Diameter (inches)	Pipe Unit Cost ($/LF)
8	100
10	130
12	155
15	180
18	200
21	250
24	275
27	300
30	330
36	400

Note: Costs include Engineering and Administration and contingencies

Unit Cost Estimate for Force Mains

The unit cost estimates for force mains were determined using an estimate of approximately $8 per pipe diameter per linear foot, which was based upon recently designed and constructed projects of similar scope and magnitude. The estimate included excavation, bedding, backfill, pipe material, and pavement. In addition, a 20 percent Engineering and Administration fee and a 20 percent contingency were added. Table 7-2 presents force main unit costs useful in the development of capital costs.

TABLE 7-2

FORCE MAIN UNIT COSTS

Pipe Diameter (inches)	Pipe Unit Cost ($/LF)
4	45
6	70
8	90
10	110
12	135

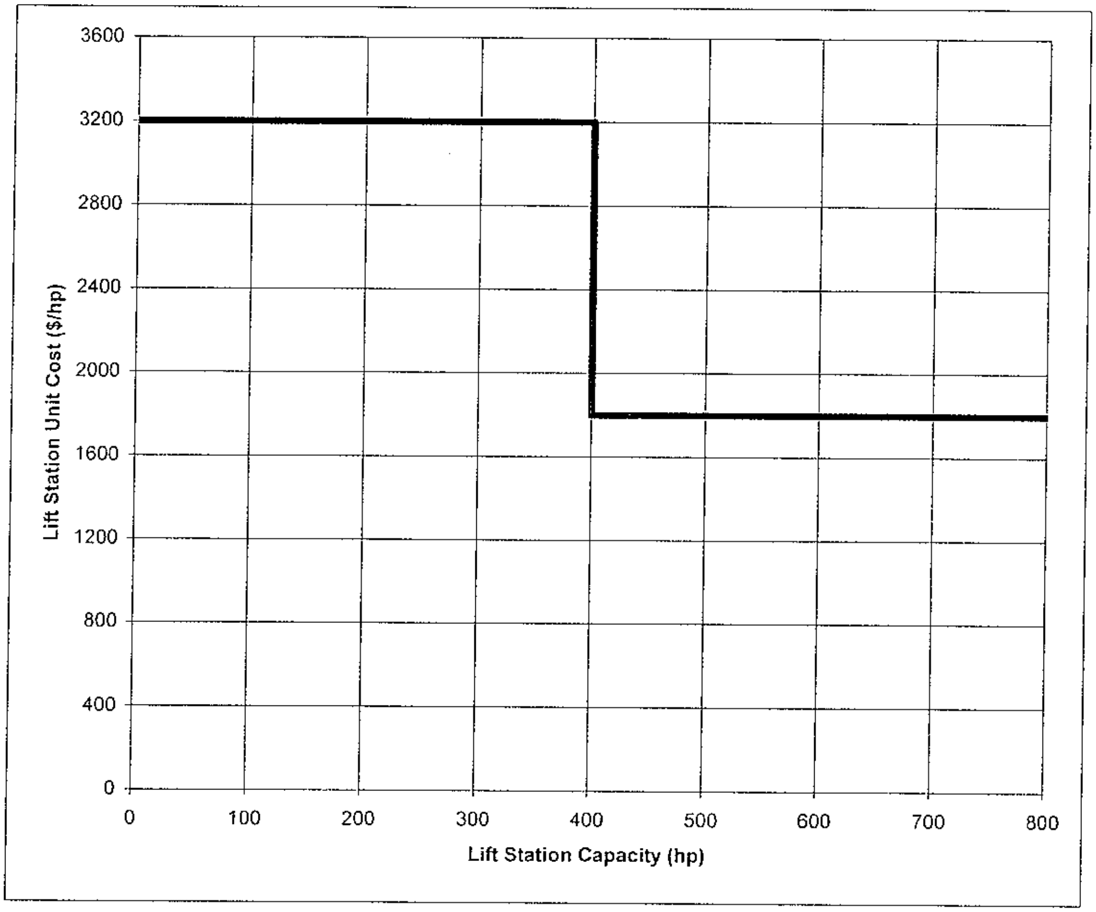
Note: Costs include Engineering and Administration and contingencies

Unit Cost Estimate for Lift Stations

Lift station capital costs are estimated based on the total capacity (not including standby capacity) of the lift station. The unit cost for lift stations includes pumps and motors (not including standby), grading, miscellaneous piping and valving, fencing, landscaping, instrumentation, controls engineering, administration and contingencies. These equipment estimates are based on recently designed and constructed projects of similar scope and magnitude. In addition, a 20 percent Engineering and Administration fee and a 20 percent contingency were added. Figure 7-1 presents a lift station unit cost curve useful in the development of lift station equipment costs.

Given the age of the City's lift stations, the City is methodically modernizing and replacing each of its older stations. As such, in addition to the equipment cost curve shown in Figure 7-1, the City's 2001 Sewer Lift Station Design Manual specifies the structural requirements for wet well/dry pit and submersible facilities. Based upon a review of the City's recent improvements to Lift Station No. 4 ("A") and Lift Station No. 17 (Brookhurst), a fixed unit cost of $900,000 is recommended for the construction of new wet well/dry pit lift stations and a fixed unit cost of $400,000 for the construction of new submersible type lift stations. Since the City is replacing submersible lift stations with wet well/dry pit stations wherever possible, the wet well/dry pit costs will generally be applied to the derived construction cost estimates. As discussed with City staff, the existing submersible type lift stations in the harbor area are assigned to remain as submersible facilities due to high groundwater conditions and localized site constraints.

FIGURE 7-1
LIFT STATION EQUIPMENT UNIT COSTS



Note: Costs include Engineering and Administration and contingencies

WASTEWATER COLLECTION SYSTEM PIPELINE COST ESTIMATES

Repair and Replacement of Existing Facilities

The decision to repair or replace existing facilities is based primarily on facility condition. Eroding pipelines with reasonable structural integrity are often repaired using various trenchless rehabilitation techniques, such as "sliplining", "cured-in-place", or pipe bursting process. Most communities utilize each of these rehabilitation methodologies depending on the selective applications. This is common rehabilitation approach as it is less disruptive and usually more cost effective than pipeline replacement.

Facility replacement however, is generally considered as the most cost effective solution for extremely deteriorated pipelines and facilities that have exceeded or are approaching their presumed useful life. Accordingly, facility condition and probable life expectancy must be accurately assessed to establish the appropriate remedy for each pipeline segment. Current unit costs to replace versus repair different diameters of wastewater pipelines are provided in Table 7-3. These estimated costs include all materials, labor, and engineering required for pipeline repair or replacement.

TABLE 7-3

PIPELINE REPAIR/REPLACEMENT UNIT COSTS

Pipe Diameter (inches)	Unit Cost ($/LF)	
	Pipeline Repair	Pipeline Replace
8	60	100
10	80	130
12	90	155
15	95	180
18	130	200
21	155	250
24	180	275
27	190	300
30	200	330
33	220	360
36	230	400

Note: Costs include Engineering and Administration and contingencies

Since the actual condition and age of each wastewater pipeline is often unknown, it is indeterminable whether a repair or replacement strategy is the appropriate application for each segment of pipeline that may need rehabilitation. As such, the City is undertaking a comprehensive video inspection program as part of its infrastructure management program.

While investment in new facilities that are required to serve new customers is generally a proactive practice, reinvestment in the existing assets is an often overlooked or under funded component of a utility's infrastructure management plan. Given that the majority of the infrastructure is estimated to be approximately 40 years old, the City's GIS wastewater inventory data was utilized to develop and estimate of the level of capital rehabilitation cost. This information is intended to supplement the City's infrastructure management and video inspection program, and provide an estimate of ongoing wastewater investment requirements.

The remaining useful life of the wastewater collection system facilities is a necessary element of the infrastructure investment decision process. As discussed in Chapter 3, according to the State of California Controller's Office, the suggested useful life of utility fixed assets is 50 years for pipelines, manholes, and lift station structures, while the useful life of lift station equipment is generally less, approximately 20 years. Due to the inert nature of VCP, it is generally considered to provide the longest useful life of most materials commonly used in wastewater pipeline construction.

While the actual useful life of wastewater pipeline systems may extend beyond the "book value," annualized depreciation provides a reasonable estimate of the City's re-investment requirement. As such, the annual depreciation for the collection system has been developed using a 50 year suggested useful life. Since the majority of the collection system is approximately 40 years old, it is assumed that when existing facilities reach their presumed useful life, they will be remediated based on a 50% repair and 50% replace strategy. For the purposes of this analysis, all pipelines less than or equal to 6-inches in diameter are assumed to be replaced with 8-inch facilities and all pipelines that did not contain a diameter within the GIS were assumed to be 8-inch pipelines.

The estimated replacement cost new and annual depreciation of the City's wastewater collection system pipelines is derived by applying the inventory of collection system facilities with the repair and replacement unit costs provided in Table 7-3. The resulting analysis is shown in Table 7-4. As shown, the City would need to fund approximately $3.2 million per year to cover the annual depreciation of existing infrastructure (at current costs).

In recognition of this need for ongoing reinvestment, on 21 August, 2001, the City adopted ordinances establishing a new sewer service charge and a schedule of rates and charges. The adopted rates are budgeted to generate approximately $5.6 million per year with an additional $700,000 from the General Fund. The $6.3 million per year is scheduled to be allocated between the capital program and operation and maintenance activities based on $4.5 million for annual capital projects and $1.8 million for annual O&M and video inspection activities.

Consistent with the analysis performed herein, the City has programmed approximately $3.0 million per year for pipeline repair and replacement activities. This ongoing investment/reinvestment in the City's wastewater system reflects the proactive philosophy of the City's Integrated Infrastructure Management Program. A copy of the adopted sewer service charge ordinances is provided in Appendix E.

Existing and Future Hydraulic Deficiency Cost Estimates

Wastewater collection system pipeline improvements have been evaluated based upon meeting projected peak wastewater flows in accordance with the design criteria established in Chapter 5. Gravity sewers have been evaluated utilizing the HYDRA hydraulic model developed as part of this Sewer Master Plan. All proposed sewer pipeline improvements are conservatively assumed to replace the existing gravity sewer main. In addition to the need to rehabilitate aging infrastructure, it is recommended that the City construct new pipelines to eliminate identified hydraulic capacity deficiencies and increase system capacity. The estimated construction costs of these deficiencies are itemized in Table 7-5. The identified "borderline" facilities are also included in Table 7-5.

TABLE 7-4
ANNUAL WASTEWATER COLLECTION SYSTEM FACILITY DEPRECIATION

Pipeline Size (in)	Length (LF)	Replace Unit Cost ($)	Repair Unit Cost ($)	Replace Length (ft)	Repair Length (ft)	Replace Cost ($)	Repair Cost ($)	Total Cost ($)	Annual Depreciation ($/yr)[3]
No Data[1]	1,490	$100	$60	745	745	$74,500	$44,700	$119,200	$2,384
4	2,700	$100[2]	N/A[2]	2,700	0	$270,000	$0	$270,000	$5,400
6	8,280	$100[2]	N/A[2]	8,280	0	$828,000	$0	$828,000	$16,560
8	1,568,100	$100	$60	784,050	784,050	$78,405,000	$47,043,000	$125,448,000	$2,508,960
10	112,490	$130	$80	56,245	56,245	$7,311,850	$4,499,600	$11,811,450	$236,229
12	72,770	$155	$90	36,385	36,385	$5,639,675	$3,274,650	$8,914,325	$178,287
15	51,110	$180	$95	25,555	25,555	$4,599,900	$2,427,725	$7,027,625	$140,553
16	4,360	$180	$95	2,180	2,180	$392,400	$207,100	$599,500	$11,990
18	16,920	$200	$130	8,460	8,460	$1,692,000	$1,099,800	$2,791,800	$55,836
21	6,730	$250	$155	3,365	3,365	$841,250	$521,575	$1,362,825	$27,257
24	1,320	$275	$180	660	660	$181,500	$118,800	$300,300	$6,006
27	5,400	$300	$190	2,700	2,700	$810,000	$513,000	$1,323,000	$26,460
30	1,310	$330	$200	655	655	$216,150	$131,000	$347,150	$6,943
Totals	**1,852,980**	-	-	**931,980**	**921,000**	**$101,262,225**	**$59,880,950**	**$161,143,175**	**$3,222,864**

[1]Pipelines with No Data are assumed to be 8-inch pipelines
[2]All pipelines less than or equal to 6-inches are assumed to be replaced with 8-inch pipelines
[3]Based on 50 Year Useful Life

TABLE 7- 5
COLLECTION SYSTEM REPLACEMENT COST ESTIMATES

ID #	Category	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Unit Cost ($/ft)	Replacement Cost ($)
1013	Deficient	18	21	373	$250	$93,250
1014	Deficient	18	21	100	$250	$25,000
1015	Deficient	18	21	226	$250	$56,500
1016	Deficient	18	21	201	$250	$50,250
1017	Deficient	18	21	336	$250	$84,000
1019	Deficient	18	21	304	$250	$76,000
474	Deficient	12	18	301	$200	$60,200
476	Deficient	12	18	345	$200	$69,000
477	Deficient	12	18	345	$200	$69,000
478	Deficient	12	18	345	$200	$69,000
531	Deficient	12	18	329	$200	$65,800
547	Deficient	12	18	330	$200	$66,000
2586	Deficient	12	18	299	$200	$59,800
294	Deficient	12	15	335	$180	$60,300
295	Deficient	12	15	330	$180	$59,400
296	Deficient	12	15	168	$180	$30,240
165	Deficient	10	15	324	$180	$58,320
166	Deficient	10	15	347	$180	$62,460
167	Deficient	10	15	314	$180	$56,520
168	Deficient	10	15	339	$180	$61,020
169	Deficient	10	15	308	$180	$55,440
170	Deficient	10	15	304	$180	$54,720
174	Deficient	10	15	107	$180	$19,260
194	Deficient	10	15	138	$180	$24,840
202	Deficient	10	15	136	$180	$24,480
206	Deficient	10	15	226	$180	$40,680
211	Deficient	10	15	204	$180	$36,720
213	Deficient	10	15	113	$180	$20,340
216	Deficient	10	15	240	$180	$43,200
219	Deficient	10	15	240	$180	$43,200
221	Deficient	10	15	110	$180	$19,800
222	Deficient	10	15	213	$180	$38,340
232	Deficient	10	15	239	$180	$43,020
237	Deficient	10	15	246	$180	$44,280
293	Deficient	10	15	251	$180	$45,180
308	Deficient	10	15	290	$180	$52,200
321	Deficient	10	15	300	$180	$54,000
335	Deficient	10	15	301	$180	$54,180
345	Deficient	10	15	295	$180	$53,100
543	Deficient	10	15	307	$180	$55,260
544	Deficient	10	15	306	$180	$55,080
545	Deficient	10	15	306	$180	$55,080
548	Deficient	10	15	309	$180	$55,620
549	Deficient	10	15	14	$180	$2,520
550	Deficient	10	15	155	$180	$27,900
590	Deficient	10	15	293	$180	$52,740
598	Deficient	10	15	289	$180	$52,020
974	Deficient	10	15	331	$180	$59,580
980	Deficient	10	15	330	$180	$59,400
3002	Deficient	10	15	46	$180	$8,280
1080	Deficient	10	12	294	$155	$45,570
5005	Deficient	8	12	255	$155	$39,525
5013	Deficient	8	12	210	$155	$32,550
			Subtotal "Deficient" =	**13,697**		**$2,600,165**

TABLE 7- 5
COLLECTION SYSTEM REPLACEMENT COST ESTIMATES

ID #	Category	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Unit Cost ($/ft)	Replacement Cost ($)
362	Borderline	15	18	157	$200	$31,400
368	Borderline	15	18	246	$200	$49,200
379	Borderline	15	18	315	$200	$63,000
486	Borderline	15	18	127	$200	$25,400
488	Borderline	15	18	125	$200	$25,000
493	Borderline	15	18	206	$200	$41,200
495	Borderline	15	18	329	$200	$65,800
33	Borderline	12	15	350	$180	$63,000
36	Borderline	12	15	320	$180	$57,600
46	Borderline	12	15	262	$180	$47,160
50	Borderline	12	15	299	$180	$53,820
115	Borderline	12	15	150	$180	$27,000
116	Borderline	12	15	105	$180	$18,900
117	Borderline	12	15	75	$180	$13,500
118	Borderline	12	15	330	$180	$59,400
119	Borderline	12	15	330	$180	$59,400
120	Borderline	12	15	341	$180	$61,380
121	Borderline	12	15	259	$180	$46,620
175	Borderline	12	15	335	$180	$60,300
182	Borderline	12	15	270	$180	$48,600
183	Borderline	12	15	259	$180	$46,620
185	Borderline	12	15	275	$180	$49,500
239	Borderline	12	15	302	$180	$54,360
256	Borderline	12	15	513	$180	$92,340
261	Borderline	12	15	246	$180	$44,280
297	Borderline	12	15	166	$180	$29,880
298	Borderline	12	15	317	$180	$57,060
299	Borderline	12	15	341	$180	$61,380
300	Borderline	12	15	309	$180	$55,620
312	Borderline	12	15	251	$180	$45,180
366	Borderline	12	15	326	$180	$58,680
375	Borderline	12	15	210	$180	$37,800
377	Borderline	12	15	120	$180	$21,600
389	Borderline	12	15	220	$180	$39,600
508	Borderline	12	15	9	$180	$1,620
607	Borderline	12	15	178	$180	$32,040
610	Borderline	12	15	253	$180	$45,540
619	Borderline	12	15	226	$180	$40,680
623	Borderline	12	15	267	$180	$48,060
634	Borderline	12	15	235	$180	$42,300
635	Borderline	12	15	144	$180	$25,920
639	Borderline	12	15	140	$180	$25,200
742	Borderline	12	15	286	$180	$51,480
932	Borderline	12	15	682	$180	$122,760
1068	Borderline	12	15	221	$180	$39,780
1134	Borderline	12	15	141	$180	$25,380
1135	Borderline	12	15	30	$180	$5,400
1181	Borderline	12	15	301	$180	$54,180
43	Borderline	10	12	325	$155	$50,375
205	Borderline	10	12	261	$155	$40,455
207	Borderline	10	12	347	$155	$53,785
208	Borderline	10	12	105	$155	$16,275
210	Borderline	10	12	149	$155	$23,095
220	Borderline	10	12	292	$155	$45,260
223	Borderline	10	12	107	$155	$16,585

TABLE 7- 5
COLLECTION SYSTEM REPLACEMENT COST ESTIMATES

ID #	Category	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Unit Cost ($/ft)	Replacement Cost ($)
231	Borderline	10	12	223	$155	$34,565
242	Borderline	10	12	330	$155	$51,150
245	Borderline	10	12	350	$155	$54,250
254	Borderline	10	12	330	$155	$51,150
262	Borderline	10	12	330	$155	$51,150
301	Borderline	10	12	350	$155	$54,250
302	Borderline	10	12	23	$155	$3,565
449	Borderline	10	12	129	$155	$19,995
462	Borderline	10	12	301	$155	$46,655
465	Borderline	10	12	287	$155	$44,485
470	Borderline	10	12	282	$155	$43,710
522	Borderline	10	12	259	$155	$40,145
523	Borderline	10	12	299	$155	$46,345
524	Borderline	10	12	314	$155	$48,670
525	Borderline	10	12	255	$155	$39,525
533	Borderline	10	12	265	$155	$41,075
546	Borderline	10	12	264	$155	$40,920
551	Borderline	10	12	160	$155	$24,800
819	Borderline	10	12	673	$155	$104,315
824	Borderline	10	12	261	$155	$40,455
868	Borderline	10	12	328	$155	$50,840
901	Borderline	10	12	175	$155	$27,125
917	Borderline	10	12	388	$155	$60,140
953	Borderline	10	12	331	$155	$51,305
954	Borderline	10	12	168	$155	$26,040
957	Borderline	10	12	326	$155	$50,530
962	Borderline	10	12	162	$155	$25,110
963	Borderline	10	12	330	$155	$51,150
964	Borderline	10	12	158	$155	$24,490
966	Borderline	10	12	332	$155	$51,460
975	Borderline	10	12	329	$155	$50,995
978	Borderline	10	12	168	$155	$26,040
981	Borderline	10	12	171	$155	$26,505
1041	Borderline	10	12	301	$155	$46,655
1118	Borderline	10	12	150	$155	$23,250
1121	Borderline	10	12	277	$155	$42,935
1130	Borderline	10	12	40	$155	$6,200
1131	Borderline	10	12	326	$155	$50,530
1136	Borderline	10	12	81	$155	$12,555
2566	Borderline	10	12	349	$155	$54,095
2569	Borderline	10	12	349	$155	$54,095
2576	Borderline	10	12	352	$155	$54,560
4003	Borderline	10	12	309	$155	$47,895
233	Borderline	8	10	35	$130	$4,550
240	Borderline	8	10	91	$130	$11,830
313	Borderline	8	10	226	$130	$29,380
318	Borderline	8	10	91	$130	$11,830
347	Borderline	8	10	341	$130	$44,330
832	Borderline	8	10	77	$130	$10,010
833	Borderline	8	10	105	$130	$13,650
2541	Borderline	8	10	181	$130	$23,530
2543	Borderline	8	10	36	$130	$4,680
2545	Borderline	8	10	294	$130	$38,220
2547	Borderline	8	10	275	$130	$35,750
2579	Borderline	8	10	162	$130	$21,060

TABLE 7- 5
COLLECTION SYSTEM REPLACEMENT COST ESTIMATES

ID #	Category	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Unit Cost ($/ft)	Replacement Cost ($)
2581	Borderline	8	10	217	$130	$28,210
333	Borderline	15	18	130	$200	$26,000
341	Borderline	15	18	195	$200	$39,000
351	Borderline	15	18	197	$200	$39,400
1168	Borderline	12	15	335	$180	$60,300
399	Borderline	10	12	290	$155	$44,950
434	Borderline	10	12	300	$155	$46,500
447	Borderline	10	12	170	$155	$26,350
563	Borderline	10	12	332	$155	$51,460
775	Borderline	10	12	363	$155	$56,265
779	Borderline	10	12	400	$155	$62,000
785	Borderline	10	12	401	$155	$62,155
811	Borderline	10	12	171	$155	$26,505
818	Borderline	10	12	1462	$155	$226,610
903	Borderline	10	12	206	$155	$31,930
1055	Borderline	10	12	206	$155	$31,930
311	Borderline	8	10	75	$130	$9,750
5009	Borderline	8	10	225	$130	$29,250
5004	Borderline	8	10	171	$130	$22,230
Subtotal "Borderline" =				32,830		$5,383,040
Total Collection System Improvements =				46,527		$7,983,205

PUMPING SYSTEM COST ESTIMATES

Similar to that of the collection system evaluation methodology, lift station improvements can generally be classified into two categories: 1) improvements required to increase system hydraulic capacity or reliability, and 2) improvements to correct unsafe conditions or meet code requirements. Both of these categories are important and expose the City of Huntington Beach to operational deficiencies if the identified problems are not corrected.

Capacity/reliability related improvements are considered priority projects that are required to maintain the City's ability to pump wastewater flows. One important element of system reliability is standby power. While the City provides standby power through portable generators, a more reliable approach is to utilize dedicated standby power generators with automatic transfer switches at each lift station. As such, the City should consider implementing this approach as its facilities are rehabilitated, depending on funding and facility site availability. The cost of these standby power improvements is not included the following capital cost estimates.

As discussed, the lift station evaluation performed herein was based on original lift station design parameters and model simulated flows and may not precisely depict current field conditions. Therefore, the cost estimates prepared herein are conceptual in nature. Final costs would require additional field verification, flow testing, and pre-design analysis.

In recognition of the need for reliable and ongoing lift station performance data, it would be desirable to perform a comprehensive evaluation of the pump capacities and efficiencies at each of the City's lift stations. As an early action item, it is recommended that permanent metering facilities be constructed at each lift station to provide telemetry influent and output data so as to improve efficiency of the entire system and meet future conditions. Both of these elements would provide valuable support information in the magnitude and prioritization of lift station improvements.

The cost of the comprehensive analysis is estimated at approximately $75,000, while the cost of the metering improvements is approximately $20,000 per station. The estimated costs for the reconstruction of the City's lift stations are presented in Table 7-6.

TABLE 7- 6
LIFT STATION REPLACEMENT COST ESTIMATES

Lift Station Number and Name	Rated Pump Horsepower [1]	Number of Pumps	Influent To Capacity Ratio	Capacity Deficiency[2]	Lift Station Replacement Cost
Deficient Lift Stations					
#2 Humbolt	3	2	119%	Yes	$1,104,833
#5 Davenport	3	2	123%	Yes	$1,141,201
#6 Edgewater	5	2	288%	Yes	$2,727,561
#15 Beach	75	2	118%	Yes	$1,910,979
#16 Adams	3	2	148%	Yes	$1,374,873
#18 Atlanta	25	2	137%	Yes	$879,082
#22 Saybrook	15	2	184%	Yes	$1,922,485
#23 New Britain	5	2	156%	Yes	$1,478,168
#25 Edinger	5	2	190%	Yes	$1,805,260
#26 Brighton	3	2	123%	Yes	$1,143,837
#28 Coral Cay	3	2	121%	Yes	$1,119,455
Deficient Subtotal					**$16,607,733**
Non-Deficient Lift Stations					
#1 Graham	20	2	87%	No	$1,028,000
#3 "E"[4]	3	2	68%	No	$919,200
#7 Station "B"[4]	8	2	92%	No	$972,000
#8 Station"C"[4]	25	2	76%	No	$1,140,000
#10 Algonquin	40	2	68%	No	$656,000
#11 Lark	2	2	94%	No	$912,800
#13 Slater	20	2	96%	No	$1,028,000
#14 Ellis	20	3	37%	No	$1,028,000
#19 Bushard	3	2	41%	No	$919,200
#20 Speer	15	2	16%	No	$996,000
#21 MCFadden	5	2	98%	No	$932,000
#24 Edwards	20	2	96%	No	$1,028,000
#29 Trinidad[4]	10	2	82%	No	$964,000
Non-Deficient Subtotal					**$12,523,200**
Recently Improved Lift Stations					
#4 "A"[3],[4]	10	2	68%	N/A	N/A
# 9 "D"[3],[4]	25	4	79%	N/A	N/A
#17 Brookhurst[3]	30	2	74%	N/A	N/A
Metering Facilities					**$540,000**
Total Replacement Cost					**$29,670,933**

Source data provided by City. Note, there is no station No. 12 or No. 27.
[1]Capacity is defined per City maintenance department data for one pump in operation only.
[2]Capacity deficiencies are considered high priority improvements.
[3]These stations have been recently improved or are currently in design for capacity improvement.
[4]It is recommended the pump output capacity of all harbor lift stations be field evaluated.

CAPITAL IMPROVEMENT PROGRAM PRIORITIZATION

Implementation of the City's Capital Improvement Program (CIP) should be based on improvement priorities. When possible, improvements should be phased to equalize annual capital/debt service requirements and minimize user charge impact. Due to the nature of the improvements, most of these projects should be constructed during the next 10 years.

Chapter 8

Sewer Facility Charges

CHAPTER 8

SEWER FACILITY CHARGES

The City utilizes a Sewer Facility Charge (SFC), commonly referred to as a connection fee, to recover the costs of facilities to be constructed in the future that will benefit new development. The purpose of this charge is to assure that future customers pay their fair share of the costs of the system's capacity. As such, a Sewer Facility Charge equitably distributes facility costs to future users based on their anticipated demands on the wastewater system. The assets that collect and pump the City's wastewater are the basis for the cost of capacity in the sewer system.

In recognition of the need to remain current and integrate the new Master Plan costs of system capacity, the City desires to update its Sewer Facility Charges. This chapter is intended to update the current cost of sewer system capacity, reflect these costs in the development of new facility charges, and document these charges in the City's Master Plan report of findings.

REGULATORY REQUIREMENTS

The regulations that govern SFCs generally fall into three areas: compliance with State government codes, adherence to the State Water Resources Control Board's (SWRCB) Revenue Program guidelines, and City ordinances.

State Government Codes

Government Code Sections 66000 - 66024 and 66483 are the primary government codes applicable to the development and recovery of capital facility charges. The focus of these sections are summarized below:

- The City must establish a nexus between the cost of capacity and the facility charge.

- The facility charge revenues must be segregated from operating and maintenance funds.

- The revenues must be committed or assigned to a capital project within five years.

In summary, these sections of Government Code require the basis for Sewer Facility Charges be consistent with new development's impact on the cost of capacity in the City's wastewater system.

Revenue Program Guidelines

The SWRCB Revenue Program guidelines apply to all recipients of Federal Clean Water Grants for water pollution control facilities. The guidelines require that facility charges not be used as an assured revenue source for revenue planning and that the facility charge revenues be segregated from other rate-based revenues.

City Ordinances

The City ordinance applicable to SFCs is contained in Chapter 14.36 of the City's Municipal Code. The current and updated residential sewer facility charges are based on an "equivalent dwelling unit" or EDU. For consistency with the current sewer user charge rate schedule, the updated non-residential charge is also proposed to be converted from a cost per 1000 square feet to an EDU basis.

CALCULATION METHODOLOGY

As discussed with City staff, there are two generally accepted methods commonly used to develop capital facility charges. These methods are based on an incremental approach or a system capacity buy-in approach. These two calculation methodologies are discussed in the following sections.

Incremental Approach. The incremental approach is based on quantifying the future costs of additional capacity and unitizing these costs by the incremental quantity of additional demand served by these costs. Accordingly, the capital improvement program derived in chapter 7 provides the primary basis of costs, while the estimation of future flows derived in Chapter 6 provides the basis for future incremental wastewater flows.

Capacity Buy-In Approach. Similar to the incremental approach, the capacity buy-in approach is based on the cost of future wastewater system capacity and is unitized based on the quantity of demand served by those costs. However, the capacity buy-in method includes the value of the existing system assets in the basis of costs. In doing so, the quantity of demand served by the value of the existing system plus the future costs of the proposed CIP is represented by the total projected ultimate demand in the City's wastewater system.

Recommended Approach. Based on discussions with City staff, the incremental approach was used as the basis for developing the City's SFC's. This approach was selected because it more closely coincided with the City's general guidelines for the development and use of the sewer service charge revenues. The incremental approach is also easily understood, provides a documented nexus between the cost of capacity and the proposed sewer facility charges, and complies with current Government Code.

COSTS OF FUTURE CAPACITY

A study of capital facilities charges is performed to develop and/or identify the costs of facilities used by future wastewater customers. Under the incremental approach, the cost of future capacity in the City's wastewater system is based on two facility components. These include the future replacement costs of the sewer lift stations and new local sewer collection system improvements. While the cost of these improvements was previously developed in Chapter 7, the allocation of these costs to future customers is discussed in the following sections.

Lift Station Replacement Costs

As shown in the Lift Station Replacement Cost Estimate of Table 7-6, lift station costs are segregated into two primary categories. These include the cost of improvements necessary to replace capacity deficient facilities and the costs associated with the replacement of facilities that have adequate capacity, but should be eventually be replaced due to long-term wear and tear. Additional metering improvements are also designated for all lift station facilities to improve reliability and monitor capacity performance.

Several key considerations were discussed with City staff related to assessing the cost of lift station improvements to future customers. Since the ongoing sewer user charge was designed to provide for the methodical replacement of the City's lift stations, only the specific portion of the capacity related facility improvement costs and metering enhancements is un-funded. As such, the costs allocated to future customers are limited to these cost elements. The estimated cost of lift station improvements for future customers is shown in Table 8-1.

As shown, the capacity required for future customers is the percentage of the influent that is greater than the facility capacity. This percent assigned to future users is multiplied by the lift station replacement cost to calculate the estimated cost of capacity assigned to future customers. Since the metering facility improvements enhance the efficiency of all lift stations, 100 percent of the cost of these improvements is assigned to future customers. The total cost of lift station improvements that is included in the cost of future capacity is approximately $9.6 million.

Collection System Replacement Costs

As previously discussed, a hydraulic model of the City's collection system was used to evaluate the need for capacity improvements. The model identified a number of pipeline segments that did not have adequate capacity to meet future conditions. The length, existing diameter, replacement diameter, and replacement cost was developed for each segment and was shown in Table 7-5. These findings are used as the basis of collection system costs for future customers.

TABLE 8-1
LIFT STATION FUTURE EXPANSION COST ALLOCATION

Lift Station Number and Name	Influent To Capacity Ratio	Lift Station Replacement Cost	% Assigned to Future Users	Cost Assigned to Future Users
Deficient Lift Stations				
#2 Humbolt	119%	$1,104,833	19%	$209,396
#5 Davenport	123%	$1,141,201	23%	$260,974
#6 Edgewater	288%	$2,727,561	100%	$2,727,561
#15 Beach	118%	$1,910,979	18%	$343,243
#16 Adams	148%	$1,374,873	48%	$660,308
#18 Atlanta	137%	$879,082	37%	$328,394
#22 Saybrook	184%	$1,922,485	84%	$1,617,694
#23 New Britain	156%	$1,478,168	56%	$826,663
#25 Edinger	190%	$1,805,260	90%	$1,632,465
#26 Brighton	123%	$1,143,837	23%	$264,823
#28 Coral Cay	121%	$1,119,455	21%	$229,791
Deficient Subtotal		**$16,607,733**	**-**	**$9,101,313**
Metering Facilities		**$540,000**	**100%**	**$540,000**
Total Replacement Cost		**$17,147,733**	**-**	**$9,641,313**

Source: Table 7-6.

Similar to the approach used for the lift station cost allocation, discussions with City staff focused on deriving the cost of collection system improvements that should be borne by future customers. Through these discussions, several approaches were developed to allocate collection system costs to future services. While each of the alternative methods complies with appropriate cost allocation procedures, the basis of approach does affect the resulting level of applicable costs and charges. The focus of the collections system cost allocation alternatives is based on the following key cost recovery questions.

- should future customers pay for all capacity deficiencies

- should the replacement pipeline costs be "discounted" to recognize that the City would have incurred costs to slipline or rehabilitate these facilities if they were not overcapacity

Since there is no discreet answer to each of these questions and the questions are not mutually exclusive, City staff decided to include the development of each alternative scenario in the cost allocation analysis. The basic alternatives derived for the collection system cost component are as follows:

- Alternative 1 – Total System Replacement Cost - Include the total cost of all capacity improvements based on the replacement cost of each facility

- Alternative 2 – Total System Upsizing Cost - Include the total cost of all capacity improvements and reduce this cost by the estimated cost to slipline each pipeline segment (at original diameter)

The resulting collection system costs are developed in Table 8-2. As shown, the costs associated with these alternatives are approximately $8.0 million, and $4.0 million for Alternatives 1 and 2, respectively.

TABLE 8-2
COLLECTION SYSTEM EXPANSION COST ALTERNATIVES

ID #	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Replacement Cost ($)	Sliplining Cost ($)	Upsizing Cost ($)
1013	18	21	373	$93,250	$48,490	$44,760
1014	18	21	100	$25,000	$13,000	$12,000
1015	18	21	226	$56,500	$29,380	$27,120
1016	18	21	201	$50,250	$26,130	$24,120
1017	18	21	336	$84,000	$43,680	$40,320
1019	18	21	304	$76,000	$39,520	$36,480
474	12	18	301	$60,200	$27,090	$33,110
476	12	18	345	$69,000	$31,050	$37,950
477	12	18	345	$69,000	$31,050	$37,950
478	12	18	345	$69,000	$31,050	$37,950
531	12	18	329	$65,800	$29,610	$36,190
547	12	18	330	$66,000	$29,700	$36,300
2586	12	18	299	$59,800	$26,910	$32,890
294	12	15	335	$60,300	$30,150	$30,150
295	12	15	330	$59,400	$29,700	$29,700
296	12	15	168	$30,240	$15,120	$15,120
165	10	15	324	$58,320	$25,920	$32,400
166	10	15	347	$62,460	$27,760	$34,700
167	10	15	314	$56,520	$25,120	$31,400
168	10	15	339	$61,020	$27,120	$33,900
169	10	15	308	$55,440	$24,640	$30,800
170	10	15	304	$54,720	$24,320	$30,400
174	10	15	107	$19,260	$8,560	$10,700
194	10	15	138	$24,840	$11,040	$13,800
202	10	15	136	$24,480	$10,880	$13,600
206	10	15	226	$40,680	$18,080	$22,600
211	10	15	204	$36,720	$16,320	$20,400
213	10	15	113	$20,340	$9,040	$11,300
216	10	15	240	$43,200	$19,200	$24,000
219	10	15	240	$43,200	$19,200	$24,000
221	10	15	110	$19,800	$8,800	$11,000
222	10	15	213	$38,340	$17,040	$21,300
232	10	15	239	$43,020	$19,120	$23,900
237	10	15	246	$44,280	$19,680	$24,600
293	10	15	251	$45,180	$20,080	$25,100
308	10	15	290	$52,200	$23,200	$29,000
321	10	15	300	$54,000	$24,000	$30,000
335	10	15	301	$54,180	$24,080	$30,100
345	10	15	295	$53,100	$23,600	$29,500
543	10	15	307	$55,260	$24,560	$30,700
544	10	15	306	$55,080	$24,480	$30,600
545	10	15	306	$55,080	$24,480	$30,600
548	10	15	309	$55,620	$24,720	$30,900
549	10	15	14	$2,520	$1,120	$1,400
550	10	15	155	$27,900	$12,400	$15,500
590	10	15	293	$52,740	$23,440	$29,300
598	10	15	289	$52,020	$23,120	$28,900
974	10	15	331	$59,580	$26,480	$33,100
980	10	15	330	$59,400	$26,400	$33,000
3002	10	15	46	$8,280	$3,680	$4,600
1080	10	12	294	$45,570	$23,520	$22,050
5005	8	12	255	$39,525	$15,300	$24,225
5013	8	12	210	$32,550	$12,600	$19,950

TABLE 8-2
COLLECTION SYSTEM EXPANSION COST ALTERNATIVES

ID #	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Replacement Cost ($)	Sliplining Cost ($)	Upsizing Cost ($)
362	15	18	157	$31,400	$14,915	$16,485
368	15	18	246	$49,200	$23,370	$25,830
379	15	18	315	$63,000	$29,925	$33,075
486	15	18	127	$25,400	$12,065	$13,335
488	15	18	125	$25,000	$11,875	$13,125
493	15	18	206	$41,200	$19,570	$21,630
495	15	18	329	$65,800	$31,255	$34,545
33	12	15	350	$63,000	$31,500	$31,500
36	12	15	320	$57,600	$28,800	$28,800
46	12	15	262	$47,160	$23,580	$23,580
50	12	15	299	$53,820	$26,910	$26,910
115	12	15	150	$27,000	$13,500	$13,500
116	12	15	105	$18,900	$9,450	$9,450
117	12	15	75	$13,500	$6,750	$6,750
118	12	15	330	$59,400	$29,700	$29,700
119	12	15	330	$59,400	$29,700	$29,700
120	12	15	341	$61,380	$30,690	$30,690
121	12	15	259	$46,620	$23,310	$23,310
175	12	15	335	$60,300	$30,150	$30,150
182	12	15	270	$48,600	$24,300	$24,300
183	12	15	259	$46,620	$23,310	$23,310
185	12	15	275	$49,500	$24,750	$24,750
239	12	15	302	$54,360	$27,180	$27,180
256	12	15	513	$92,340	$46,170	$46,170
261	12	15	246	$44,280	$22,140	$22,140
297	12	15	166	$29,880	$14,940	$14,940
298	12	15	317	$57,060	$28,530	$28,530
299	12	15	341	$61,380	$30,690	$30,690
300	12	15	309	$55,620	$27,810	$27,810
312	12	15	251	$45,180	$22,590	$22,590
366	12	15	326	$58,680	$29,340	$29,340
375	12	15	210	$37,800	$18,900	$18,900
377	12	15	120	$21,600	$10,800	$10,800
389	12	15	220	$39,600	$19,800	$19,800
508	12	15	9	$1,620	$810	$810
607	12	15	178	$32,040	$16,020	$16,020
610	12	15	253	$45,540	$22,770	$22,770
619	12	15	226	$40,680	$20,340	$20,340
623	12	15	267	$48,060	$24,030	$24,030
634	12	15	235	$42,300	$21,150	$21,150
635	12	15	144	$25,920	$12,960	$12,960
639	12	15	140	$25,200	$12,600	$12,600
742	12	15	286	$51,480	$25,740	$25,740
932	12	15	682	$122,760	$61,380	$61,380
1068	12	15	221	$39,780	$19,890	$19,890
1134	12	15	141	$25,380	$12,690	$12,690
1135	12	15	30	$5,400	$2,700	$2,700
1181	12	15	301	$54,180	$27,090	$27,090
43	10	12	325	$50,375	$26,000	$24,375
205	10	12	261	$40,455	$20,880	$19,575
207	10	12	347	$53,785	$27,760	$26,025
208	10	12	105	$16,275	$8,400	$7,875
210	10	12	149	$23,095	$11,920	$11,175

TABLE 8-2
COLLECTION SYSTEM EXPANSION COST ALTERNATIVES

ID #	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Replacement Cost ($)	Sliplining Cost ($)	Upsizing Cost ($)
220	10	12	292	$45,260	$23,360	$21,900
223	10	12	107	$16,585	$8,560	$8,025
231	10	12	223	$34,565	$17,840	$16,725
242	10	12	330	$51,150	$26,400	$24,750
245	10	12	350	$54,250	$28,000	$26,250
254	10	12	330	$51,150	$26,400	$24,750
262	10	12	330	$51,150	$26,400	$24,750
301	10	12	350	$54,250	$28,000	$26,250
302	10	12	23	$3,565	$1,840	$1,725
449	10	12	129	$19,995	$10,320	$9,675
462	10	12	301	$46,655	$24,080	$22,575
465	10	12	287	$44,485	$22,960	$21,525
470	10	12	282	$43,710	$22,560	$21,150
522	10	12	259	$40,145	$20,720	$19,425
523	10	12	299	$46,345	$23,920	$22,425
524	10	12	314	$48,670	$25,120	$23,550
525	10	12	255	$39,525	$20,400	$19,125
533	10	12	265	$41,075	$21,200	$19,875
546	10	12	264	$40,920	$21,120	$19,800
551	10	12	160	$24,800	$12,800	$12,000
819	10	12	673	$104,315	$53,840	$50,475
824	10	12	261	$40,455	$20,880	$19,575
868	10	12	328	$50,840	$26,240	$24,600
901	10	12	175	$27,125	$14,000	$13,125
917	10	12	388	$60,140	$31,040	$29,100
953	10	12	331	$51,305	$26,480	$24,825
954	10	12	168	$26,040	$13,440	$12,600
957	10	12	326	$50,530	$26,080	$24,450
962	10	12	162	$25,110	$12,960	$12,150
963	10	12	330	$51,150	$26,400	$24,750
964	10	12	158	$24,490	$12,640	$11,850
966	10	12	332	$51,460	$26,560	$24,900
975	10	12	329	$50,995	$26,320	$24,675
978	10	12	168	$26,040	$13,440	$12,600
981	10	12	171	$26,505	$13,680	$12,825
1041	10	12	301	$46,655	$24,080	$22,575
1118	10	12	150	$23,250	$12,000	$11,250
1121	10	12	277	$42,935	$22,160	$20,775
1130	10	12	40	$6,200	$3,200	$3,000
1131	10	12	326	$50,530	$26,080	$24,450
1136	10	12	81	$12,555	$6,480	$6,075
2566	10	12	349	$54,095	$27,920	$26,175
2569	10	12	349	$54,095	$27,920	$26,175
2576	10	12	352	$54,560	$28,160	$26,400
4003	10	12	309	$47,895	$24,720	$23,175
233	8	10	35	$4,550	$2,100	$2,450
240	8	10	91	$11,830	$5,460	$6,370
313	8	10	226	$29,380	$13,560	$15,820
318	8	10	91	$11,830	$5,460	$6,370
347	8	10	341	$44,330	$20,460	$23,870
832	8	10	77	$10,010	$4,620	$5,390
833	8	10	105	$13,650	$6,300	$7,350
2541	8	10	181	$23,530	$10,860	$12,670

TABLE 8-2
COLLECTION SYSTEM EXPANSION COST ALTERNATIVES

ID #	Existing Diameter (in)	Replacement Diameter (in)	Length (ft)	Replacement Cost ($)	Sliplining Cost ($)	Upsizing Cost ($)
2543	8	10	36	$4,680	$2,160	$2,520
2545	8	10	294	$38,220	$17,640	$20,580
2547	8	10	275	$35,750	$16,500	$19,250
2579	8	10	162	$21,060	$9,720	$11,340
2581	8	10	217	$28,210	$13,020	$15,190
333	15	18	130	$26,000	$12,350	$13,650
341	15	18	195	$39,000	$18,525	$20,475
351	15	18	197	$39,400	$18,715	$20,685
1168	12	15	335	$60,300	$30,150	$30,150
399	10	12	290	$44,950	$23,200	$21,750
434	10	12	300	$46,500	$24,000	$22,500
447	10	12	170	$26,350	$13,600	$12,750
563	10	12	332	$51,460	$26,560	$24,900
775	10	12	363	$56,265	$29,040	$27,225
779	10	12	400	$62,000	$32,000	$30,000
785	10	12	401	$62,155	$32,080	$30,075
811	10	12	171	$26,505	$13,680	$12,825
818	10	12	1462	$226,610	$116,960	$109,650
903	10	12	206	$31,930	$16,480	$15,450
1055	10	12	206	$31,930	$16,480	$15,450
311	8	10	75	$9,750	$4,500	$5,250
5009	8	10	225	$29,250	$13,500	$15,750
5004	8	10	171	$22,230	$10,260	$11,970
Total Collection Improvements =			**46,527**	**$7,983,205**	**$4,003,715**	**$3,979,490**

Alternative 1 - Total System Replacement Cost	**$7,983,205**
Alternative 2 - Total System Upsizing Cost	**$3,979,490**

FUTURE INCREMENTAL WASTEWATER FLOWS AND UNIT FLOW FACTORS

As previously discussed, the incremental approach is based on quantifying the future costs of additional system capacity and unitizing these costs by the incremental quantity of additional wastewater demand served by these costs. Accordingly, the incremental quantity of wastewater flows and the unit flows per customer type are important considerations in the development of the City's updated SFCs. The development of each of these wastewater flow values is discussed in the following sections.

Development of Estimated Future Incremental Wastewater Flows

Consistent with the study methodology, the estimated wastewater flow was derived from the output of the hydraulic model under current and future wastewater loading conditions. The incremental value that is the result of future growth is the difference between the future and existing wastewater flows. Based on the findings of the hydraulic model, the incremental increase in future wastewater flow was estimated to be 1.95 MGD. The results of this analysis are performed as an element of hydraulic modeling simulation tasks and are shown in Appendix D.

Development of Estimated Unit Wastewater Flows

In addition to the development of future incremental flows, wastewater flows factors are derived for each of the residential and commercial/industrial user classes. These values were estimated during the conduct of the City's 2001 Sewer Service Charge Study and are reflected herein as Table 8-3.

As shown, water consumption values are correlated to the calculated return to sewer factors to develop the average wastewater discharges for the Single Family dwelling (SFD) and Multi Family dwelling (MFD) and a commercial/industrial customer. The City's 2001 Sewer Charge Study estimated that the average wastewater discharge of a SFD is 226 gallons per day (gpd). Additionally, the 2001 Sewer Charge Study estimated the discharge for a Multi Family dwelling and a commercial/industrial customer to be 185 gpd and 257 gallons per Equivalent Dwelling Unit (EDU), respectively.

TABLE 8-3
ESTIMATED UNIT WASTEWATER FLOWS

Account Type	Units/ EDUs	Usage (HCF/Yr)	Return To Sewer	Wastewater (HCF)	Wastewater (MGD)	Usage[a] (gpd)
Single Family (SFD)	41,718	6,765,222	0.67	4,532,699	9.4	226
Multi Family (MFD)	32,326	3,378,013	0.85	2,871,311	6.0	185
Commercial/Industrial	13,308	1,829,100	0.90	1,646,190	3.4	257

Source: Sewer Service Charge Approach, DCA 7/01

[a] Single Family and Multi Family are per unit; Commercial/Industrial is per Equivalent Dwelling Unit (EDU).

UNIT COSTS OF SERVICE

The development of unit costs of service is an essential step in deriving cost of service based Sewer Facility Charges. Unit costs of service are obtained by correlating the costs associated with future growth with the incremental addition of future sewer system discharges. The resulting unit costs of service for each of the three alternatives is shown in Table 8-4.

As shown, given the variation in the collection system replacement costs allocated to future customers, the estimated unit costs for Alternative 1 is $9,038 per 1000 gpd. The resulting unit cost of service for Alternative 2 is $6,985 per 1000 gpd.

ALTERNATIVE SEWER FACILITY CHARGES

Sewer Facility Charges are a source of income from growth-induced new sewer connections or charges to the use of existing accounts with respect to wastewater discharge characteristics. The revenues from SFCs are restricted to the financing of growth-related capital improvements. The Sewer Facility Charges are based on the City's projected costs of additional wastewater system capacity, the cost of service allocation analysis, and the estimated discharges from the three key customer classes.

Sewer Facility Charges

Sewer Facility Charges are calculated by correlating the wastewater demand characteristics of the City's primary residential and non-residential user classes with the estimated unit costs of service. As shown in Table 8-5, the resulting charges for a Single Family Dwelling (SFD) is $2,043 for Alternative 1 and $1,579 for Alternative 2.

TABLE 8-4
SEWER FACILITY CHARGE UNIT COSTS OF SERVICE

Description	Alternative 1	Alternative 2
Capital Costs to Future Users		
CIP Projects - Collection System	$7,983,205	$3,979,490
CIP Projects - Lift Stations	$9,641,313	$9,641,313
Total Cost to Future Users	$17,624,518	$13,620,803
Estimated Wastewater Flows		
Projected Wastewater Flows at Buildout (MGD)	20.32	20.32
Estimated Current Wastewater Flows (MGD)	18.37	18.37
Incremental Future Wastewater Flows (MGD)	1.95	1.95
Unit Costs of Service		
Unit Costs of Service ($/1000 gpd)	$9,038	$6,985

Alternative 1 - Total System Replacement Cost
Alternative 2 - Total System Upsizing Cost

TABLE 8- 5
SEWER FACILITY CHARGE ALTERNATIVES

Description		Alternative 1	Alternative 2
Capital Costs to Future Users			
CIP Projects - Collection System		$7,983,205	$3,979,490
CIP Projects - Lift Stations		$9,641,313	$9,641,313
Total Cost to Future Users		$17,624,518	$13,620,803
Estimated Unit Wastewater Flows [a]			
Estimated Flows from a SFD (gpd)		226	226
Estimated Flows from a MFD (gpd)		185	185
Estimated Flows from a Non-Res EDU (gpd)		257	257
Unit Costs of Service			
Unit Costs of Service ($/1000 gpd)		$9,038	$6,985
SFD Unit Cost		**$2,043**	**$1,579**
MFD Unit Cost		$1,672	$1,292
Non-Res Unit Cost per EDU		$2,323	$1,795
Non-Residential Meter Size Equivalency		Alternative Charge Per	
Meter Size (Inches) and Type	EDUs[a]	Meter Size/Type	
5 / 8	1	$2,323	$1,795
3 / 4	1	$2,323	$1,795
1	2	$4,646	$3,590
1.5	3	$6,968	$5,385
2	5	$11,614	$8,976
3	11	$25,550	$19,747
4 Compound	17	$39,487	$30,517
4 Domestic and Turbine	33	$76,651	$59,240
6 Compound	33	$76,651	$59,240
6 Domestic and Turbine	67	$155,625	$120,275
8 Domestic	117	$271,764	$210,032
10 Domestic	183	$425,066	$328,512

[a] Source: Sewer Service Charge Approach, DCA 7/01
Alternative 1 - Total System Replacement Cost
Alternative 2 - Total System Upsizing Cost

Consistent with the City's existing sewer user charge rate structure, the non-residential customers are charged based on the number of EDU's for each of the City's water meter sizes and types. The number of EDU's for each meter size/type was derived in the City's 2001 Sewer Charge Study. Correlating the previously derived number of EDU's with the unit costs of service derived herein, provides the basis for the non-residential Sewer Facility Charges. The results of this analysis are show in the bottom portion of Table 8-5.

Sewer Facility Charge Comparison

Table 8-6 is a comparison of the City's current and alternative residential Sewer Facility Charges with neighboring communities. As shown, the alternative Sewer Facility Charges compare favorably with the rates of surrounding communities. These charges are based on the recovery of only the City's incremental local costs of future capacity. As discussed with City staff, it is recommended the City adopt one of the alternative facility charges so that growth cost are adequately recovered from future wastewater system customers.

TABLE 8- 6
RESIDENTIAL SEWER FACILITY CHARGE SURVEY

Description	Single Family (SFD)	Mulit Family (MFD)
Representative Agencies		
Orange County Sanitation District[a]	$1,620 to $1,965	$1,275 to $1,620
City of Santa Ana[b]	$1,500 to $2,000	$1,200 to $1,500
City of Fountain Valley[c]	$1,500	-
City of Anaheim[d]	$1,470	$1,476
City of Huntington Beach Charges		
Current Fee, Adopted 4/88	$220	$220
Calculated Fee Alternatives 2003 Study		
Alternative 1 - Total System Replacement Cost	$2,043	$1,672
Alternative 2 - Total System Upsizing Cost	$1,579	$1,292

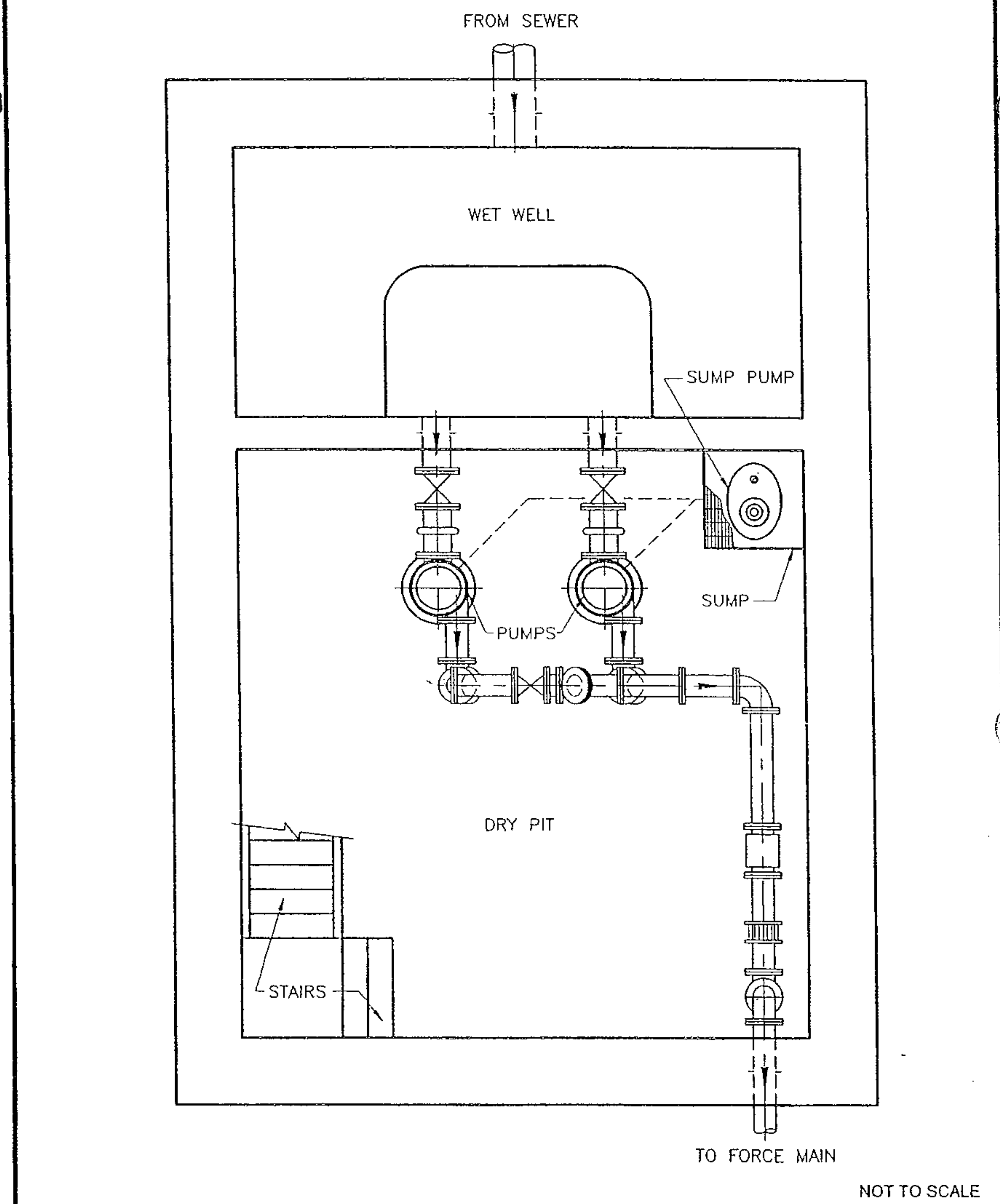
(a) Based on number of Bedrooms
(b) Based on Fixture Units. Values are City average estimates
(c) $406/Gross Acre + $4.65 per Front Footage (basis used was 4 DU/Ac, and 300 LF
(d) Varies by service Area. Values are estimated averages

Appendix A

Lift Station Support Information

TABLE A-1
SUMMARY OF SPECIFIC PUMP STATION OBSERVATIONS -1995

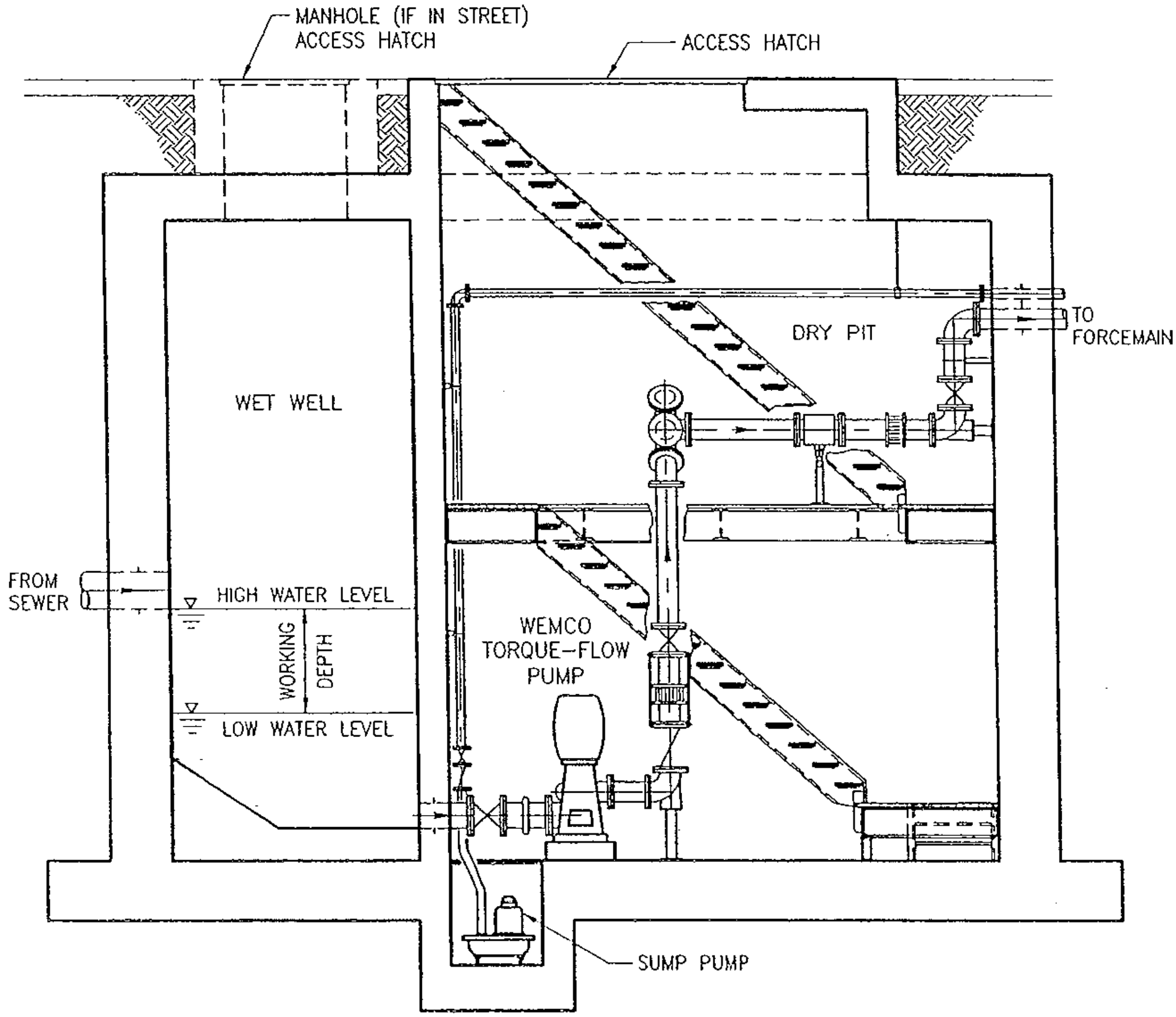
NUMBER	NAME	NUMBER & NAME	INADEQUATE PIPE SUPPORT	OBSERVED OR POSSIBLE LEAKING DRY WELL/VAULT	AWWA VALVES	EXCESS ELECTRICAL CABLE	EXPOSED WIRING	REQUIRES ELECTRICAL CONDUIT PAINT	UNDERSIZED DRY WELL - RESTRICTED AREAS	CORRODED PIPING, DRY WELL, AND/OR HARDWARE	REQURES PAINTING/ PROTECTIVE COATING	NEEDS CLEANING
1	Graham & Kenilworth	#1 GRAHAM	X	X	X			X			X	
2	Humbolt & Wayfarer	#2 HUMBOLDT		X	X	X	X	X		X	X	
3	Gilbert & Peale	#3 "E"	X		X	X	X	X		X	X	
4	PCH in Sunset Beach	#4 "A"	X	X	X	X	X	X			X	
5	Davenport & Baruna	#5 DAVENPORT	X		X	X	X	X				
6	Edgewater & Davenport	#6 EDGEWATER	X		X	X	X	X				
7	PCH West of Warner	#7 STATION "B"	X	X	X			X		X	X	
8	Warner North of PCH	#8 STATION "C"	X	X	X			X		X	X	
9	Warner at Edgewater "D" Station	# 9 "D"	X	X	X			X			X	
10	Algonquin & Boardwalk	#10 ALGONQUIN	X	X						X		X
11	Lark & Warner	#11 LARK	X	X	X						X	
12	Heil & Mycroft	# 12 HEIL	X		X	X	X			X		
13	Slater & Springdale	#13 SLATER	X		X	X	X		X	X		
14	Gothard & Ellis	#14 GOTHARD	X		X			X		X		
15	Oceanhill & Beach	#15 BEACH							X			
16	Adams & Ranger	#16 ADAMS	X		X	X	X	X	X	X	X	
17	Brookhurst & Effingham	#17 BROOKHURST	X		X	X	X	X		X		
18	Atlanta East of Beach	#18 ATLANTA	x		X					X	X	
19	Bushard & Pettswood	#19 BUSHARD	X	X	X	X	X	X		X	X	
20	Speer & Crabb	#20 SPEER	X	X	X	X	X					
21	McFadden & Dawson	#21 MCFADDEN	X	X	X			X		X	X	
22	Saybrook & Heil	#22 SAYBROOK	X		X	X	X	X				
23	New Britian & Adams	#23 NEW BRITAIN		X	X			X		X	X	
24	Edwards & Balmorol	#24 EDWARDS	X		X	X	X	X		X		
25	Edinger & Santa Barbara	#25 EDINGER	X	X	X	X	X			X	X	X
26	Brighton & Shoreham	#26 BRIGHTON	X	X	X	X		X		X	X	
28	Coral Cay	#28 CORAL CAY	X	X	X	X	X	X		X	X	
29	Trinidad & Aquarius	#29 TRINIDAD	X		X	X	X			X		



CITY OF HUNTINGTON BEACH

TYPICAL WET WELL/DRY PIT LAYOUT

Figure 5-1



NOTES:

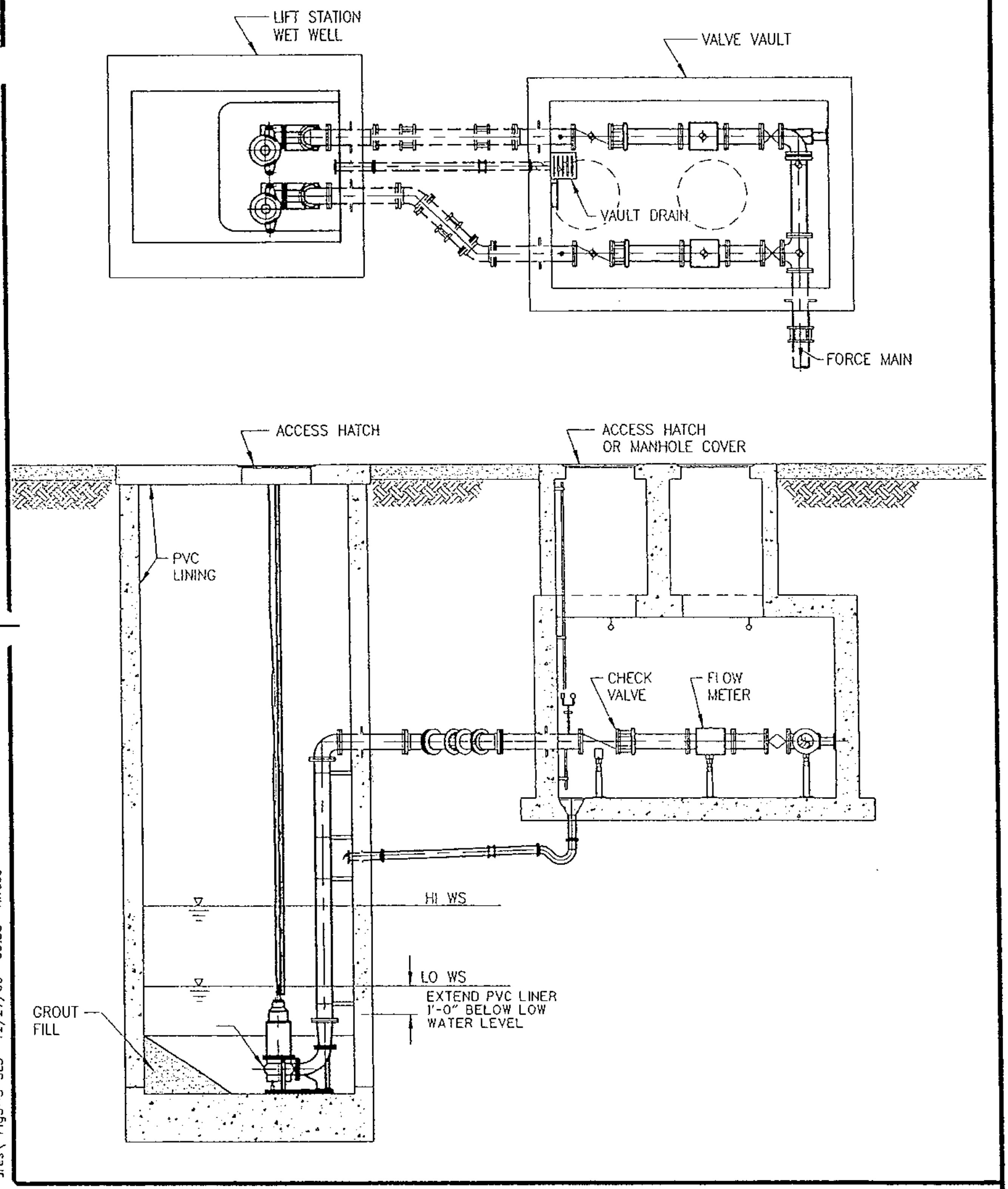
1. FIGURE DOES NOT SHOW HVAC EQUIPMENT.
2. ACTUAL LOCATION OF PIPING MAY VARY DEPENDING UPON LIFT STATION SITE AND CONFIGURATION.

NOT TO SCALE

CITY OF HUNTINGTON BEACH

TYPICAL WET WELL/DRY PIT ELEVATIONS

Figure 5-2



CITY OF HUNTINGTON BEACH

SUBMERSIBLE LIFT STATION

Figure 5-3

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Appendix B

Wastewater Flow Monitoring Summary

REFER TO MASTER DOCUMENT

Appendix C

Assessor Parcel Number and Billing System Correlation

REFER TO MASTER DOCUMENT

Appendix D

Hydraulic Analysis Support Information

APPENDIX D

COLLECTION SYSTEM DEFICIENCIES - ULTIMATE LOADING CONDITIONS

ID #	Category	PDWF (cfs)	Existing Diameter (in)	Existing D/d	Replacement Diameter (in)	New D/d	Length (ft)
1013	Deficient	3.447	18	1.00	21	0.60	373
1014	Deficient	3.402	18	1.00	21	0.60	100
1015	Deficient	3.396	18	1.00	21	0.60	226
1016	Deficient	3.253	18	0.78	21	0.58	201
1017	Deficient	3.249	18	0.78	21	0.58	336
1019	Deficient	3.143	18	0.75	21	0.57	304
474	Deficient	2.313	12	1.00	18	0.41	301
476	Deficient	2.344	12	1.00	18	0.41	345
477	Deficient	2.388	12	1.00	18	0.42	345
478	Deficient	2.443	12	1.00	18	0.42	345
531	Deficient	2.107	12	0.75	18	0.40	329
547	Deficient	2.104	12	1.00	18	0.41	330
2586	Deficient	1.693	12	0.75	18	0.39	299
294	Deficient	1.295	12	0.54	15	0.47	335
295	Deficient	1.291	12	0.54	15	0.47	330
296	Deficient	1.288	12	0.54	15	0.47	168
165	Deficient	0.852	10	0.70	15	0.38	324
166	Deficient	0.853	10	0.70	15	0.38	347
167	Deficient	0.853	10	0.70	15	0.38	314
168	Deficient	0.925	10	0.75	15	0.39	339
169	Deficient	0.931	10	0.75	15	0.39	308
170	Deficient	0.951	10	0.77	15	0.40	304
174	Deficient	0.953	10	0.77	15	0.40	107
194	Deficient	0.852	10	0.70	15	0.38	138
202	Deficient	0.988	10	0.71	15	0.38	136
206	Deficient	0.996	10	0.72	15	0.39	226
211	Deficient	1.006	10	0.74	15	0.39	204
213	Deficient	1.014	10	0.74	15	0.39	113
216	Deficient	1.021	10	0.74	15	0.39	240
219	Deficient	1.033	10	0.75	15	0.39	240
221	Deficient	1.031	10	0.75	15	0.39	110
222	Deficient	1.106	10	0.79	15	0.41	213
232	Deficient	1.122	10	1.00	15	0.41	239
237	Deficient	1.133	10	1.00	15	0.41	246
293	Deficient	0.641	10	0.65	15	0.37	251
308	Deficient	0.860	10	0.68	15	0.38	290
321	Deficient	0.861	10	0.68	15	0.38	300
335	Deficient	0.861	10	0.67	15	0.38	301
345	Deficient	0.862	10	0.68	15	0.38	295
543	Deficient	1.146	10	0.69	15	0.37	307
544	Deficient	1.251	10	0.73	15	0.38	306
545	Deficient	1.258	10	0.73	15	0.39	306
548	Deficient	1.260	10	0.74	15	0.39	309
549	Deficient	1.140	10	0.68	15	0.37	14
550	Deficient	1.140	10	0.68	15	0.37	155
590	Deficient	0.804	10	0.65	15	0.37	293
598	Deficient	0.802	10	0.65	15	0.37	289
974	Deficient	0.930	10	0.66	15	0.37	331
980	Deficient	0.947	10	0.67	15	0.37	330
3002	Deficient	0.846	10	0.66	15	0.37	46
1080	Deficient	0.796	10	0.65	12	0.50	294
5005	Deficient	0.562	8	1.00	12	0.44	255
5013	Deficient	0.585	8	1.00	12	0.43	210
					Subtotal "Deficient" =		13,697

APPENDIX D
COLLECTION SYSTEM DEFICIENCIES - ULTIMATE LOADING CONDITIONS

ID #	Category	PDWF (cfs)	Existing Diameter (in)	Existing D/d	Replacement Diameter (in)	New D/d	Length (ft)
362	Borderline	2.057	15	0.67	18	0.50	157
368	Borderline	2.142	15	0.68	18	0.51	246
379	Borderline	2.164	15	0.69	18	0.52	315
486	Borderline	1.768	15	0.67	18	0.51	127
488	Borderline	1.773	15	0.66	18	0.51	125
493	Borderline	1.834	15	0.69	18	0.53	206
495	Borderline	1.844	15	0.69	18	0.53	329
33	Borderline	0.620	12	0.50	15	0.38	350
36	Borderline	0.658	12	0.52	15	0.39	320
46	Borderline	0.804	12	0.50	15	0.40	262
50	Borderline	0.846	12	0.52	15	0.42	299
115	Borderline	1.242	12	0.52	15	0.39	150
116	Borderline	0.972	12	0.65	15	0.46	105
117	Borderline	0.943	12	0.64	15	0.45	75
118	Borderline	0.943	12	0.63	15	0.45	330
119	Borderline	0.934	12	0.63	15	0.45	330
120	Borderline	0.930	12	0.63	15	0.45	341
121	Borderline	0.720	12	0.54	15	0.39	259
175	Borderline	0.706	12	0.54	15	0.41	335
182	Borderline	0.698	12	0.49	15	0.39	270
183	Borderline	0.702	12	0.49	15	0.39	259
185	Borderline	0.673	12	0.48	15	0.38	275
239	Borderline	1.407	12	0.65	15	0.46	302
256	Borderline	1.601	12	0.60	15	0.43	513
261	Borderline	1.689	12	0.55	15	0.40	246
297	Borderline	0.916	12	0.41	15	0.39	166
298	Borderline	0.914	12	0.41	15	0.39	317
299	Borderline	0.910	12	0.41	15	0.39	341
300	Borderline	0.905	12	0.41	15	0.39	309
312	Borderline	1.314	12	0.61	15	0.44	251
366	Borderline	0.952	12	0.52	15	0.40	326
375	Borderline	0.954	12	0.53	15	0.40	210
377	Borderline	0.995	12	0.54	15	0.41	120
389	Borderline	0.996	12	0.54	15	0.41	220
508	Borderline	1.291	12	0.51	15	0.38	9
607	Borderline	0.668	12	0.50	15	0.38	178
610	Borderline	0.686	12	0.51	15	0.38	253
619	Borderline	0.715	12	0.53	15	0.39	226
623	Borderline	0.540	12	0.46	15	0.38	267
634	Borderline	0.728	12	0.53	15	0.40	235
635	Borderline	0.752	12	0.54	15	0.40	144
639	Borderline	0.927	12	0.62	15	0.45	140
742	Borderline	1.141	12	0.52	15	0.39	286
932	Borderline	0.854	12	0.51	15	0.38	682
1068	Borderline	1.058	12	0.57	15	0.42	221
1134	Borderline	1.482	12	0.58	15	0.42	141
1135	Borderline	1.481	12	0.53	15	0.39	30
1181	Borderline	1.143	12	0.52	15	0.39	301
43	Borderline	0.431	10	0.50	12	0.43	325
205	Borderline	0.696	10	0.61	12	0.46	261
207	Borderline	0.584	10	0.55	12	0.42	347
208	Borderline	0.694	10	0.61	12	0.46	105
210	Borderline	0.585	10	0.55	12	0.42	149
220	Borderline	0.634	10	0.58	12	0.44	292
223	Borderline	0.635	10	0.58	12	0.44	107

APPENDIX D
COLLECTION SYSTEM DEFICIENCIES - ULTIMATE LOADING CONDITIONS

ID #	Category	PDWF (cfs)	Existing Diameter (in)	Existing D/d	Replacement Diameter (in)	New D/d	Length (ft)
231	Borderline	0.659	10	0.59	12	0.45	223
242	Borderline	0.660	10	0.59	12	0.45	330
245	Borderline	0.489	10	0.61	12	0.46	350
254	Borderline	0.684	10	0.61	12	0.46	330
262	Borderline	0.737	10	0.64	12	0.48	330
301	Borderline	0.745	10	0.64	12	0.48	350
302	Borderline	0.747	10	0.63	12	0.47	23
449	Borderline	0.985	10	0.57	12	0.44	129
462	Borderline	1.028	10	0.62	12	0.47	301
465	Borderline	1.034	10	0.62	12	0.48	287
470	Borderline	1.040	10	0.63	12	0.48	282
522	Borderline	0.711	10	0.60	12	0.47	259
523	Borderline	0.670	10	0.57	12	0.45	299
524	Borderline	0.662	10	0.61	12	0.48	314
525	Borderline	0.582	10	0.56	12	0.45	255
533	Borderline	0.577	10	0.56	12	0.44	265
546	Borderline	0.574	10	0.55	12	0.44	264
551	Borderline	1.133	10	0.66	12	0.49	160
819	Borderline	0.511	10	0.48	12	0.39	673
824	Borderline	0.544	10	0.47	12	0.41	261
868	Borderline	0.463	10	0.55	12	0.45	328
901	Borderline	0.518	10	0.50	12	0.40	175
917	Borderline	0.518	10	0.50	12	0.40	388
953	Borderline	0.790	10	0.59	12	0.46	331
954	Borderline	0.793	10	0.56	12	0.44	168
957	Borderline	0.598	10	0.49	12	0.40	326
962	Borderline	0.914	10	0.61	12	0.47	162
963	Borderline	0.796	10	0.60	12	0.46	330
964	Borderline	0.799	10	0.56	12	0.44	158
966	Borderline	0.642	10	0.51	12	0.41	332
975	Borderline	0.679	10	0.53	12	0.42	329
978	Borderline	0.703	10	0.51	12	0.41	168
981	Borderline	0.950	10	0.62	12	0.48	171
1041	Borderline	0.986	10	0.61	12	0.46	301
1118	Borderline	0.584	10	0.55	12	0.42	150
1121	Borderline	0.703	10	0.59	12	0.46	277
1130	Borderline	0.611	10	0.59	12	0.43	40
1131	Borderline	0.585	10	0.58	12	0.42	326
1136	Borderline	1.041	10	0.52	12	0.41	81
2566	Borderline	0.784	10	0.64	12	0.47	349
2569	Borderline	0.785	10	0.64	12	0.47	349
2576	Borderline	0.785	10	0.64	12	0.47	352
4003	Borderline	0.589	10	0.50	12	0.39	309
233	Borderline	0.341	8	0.55	10	0.41	35
240	Borderline	0.308	8	0.51	10	0.38	91
313	Borderline	0.316	8	0.54	10	0.40	226
318	Borderline	0.326	8	0.55	10	0.40	91
347	Borderline	0.373	8	0.54	10	0.42	341
832	Borderline	0.340	8	0.57	10	0.41	77
833	Borderline	0.340	8	0.57	10	0.41	105
2541	Borderline	0.503	8	0.57	10	0.42	181
2543	Borderline	0.505	8	0.56	10	0.41	36
2545	Borderline	0.505	8	0.58	10	0.42	294
2547	Borderline	0.506	8	0.58	10	0.42	275
2579	Borderline	0.350	8	0.57	10	0.42	162

APPENDIX D
COLLECTION SYSTEM DEFICIENCIES - ULTIMATE LOADING CONDITIONS

ID #	Category	PDWF (cfs)	Existing Diameter (in)	Existing D/d	Replacement Diameter (in)	New D/d	Length (ft)
2581	Borderline	0.350	8	0.57	10	0.42	217
333	Borderline	2.015	15	0.66	18	0.50	130
341	Borderline	2.035	15	0.65	18	0.50	195
351	Borderline	2.045	15	0.66	18	0.50	197
1168	Borderline	1.938	12	0.51	15	0.37	335
399	Borderline	0.717	10	0.50	12	0.39	290
434	Borderline	0.717	10	0.50	12	0.39	300
447	Borderline	0.718	10	0.50	12	0.39	170
563	Borderline	1.000	10	0.49	12	0.39	332
775	Borderline	0.498	10	0.47	12	0.39	363
779	Borderline	0.501	10	0.47	12	0.39	400
785	Borderline	0.504	10	0.47	12	0.39	401
811	Borderline	0.492	10	0.40	12	0.39	171
818	Borderline	0.508	10	0.47	12	0.39	1462
903	Borderline	0.501	10	0.49	12	0.39	206
1055	Borderline	0.494	10	0.49	12	0.39	206
311	Borderline	0.275	8	0.50	10	0.37	75
5009	Borderline	0.285	8	0.49	10	0.37	225
5004	Borderline	0.360	8	0.57	10	0.42	171
					Subtotal "Borderline" =		32,830
					Total Collection System Improvements =		46,527

APPENDIX D_1
MODEL CALIBRATION FINDINGS

MONITOR		MODEL RESULTS	CALIBRATION
ID	ADWF (MGD)	(MGD)	%
1	0.767	*0.769*	0
7	0.137	*0.138*	1
10	0.506	*0.505*	0
11	0.211	*0.211*	0

APPENDIX D_2

GIS BASED LAND USE GENERATION FACTORS

LAND USE		FLOW GENERATION FACTORS (gpad)
DESCRIPTION	CODE	
Residential		
High Density	RH-30	4800
Medium High Density	RMH	4800
Medium High Density	RMH-25	3600
Medium Density	RM-25	3600
Medium Density	RM-15	2200
Low Density	RL-7	1600
Low Density	RL-6	1350
Low Density	RL-5	1100
Low Density	RL-4	900
Low Density	RL-3	750
Commercial		
Neighborhood	CN	1190
Office	CO	1120
General	CG	1040
Visitor	CV	1020
Regional	CR	690
Industrial	I	820
Public		
Medium High Density	P(RMH-25)	970
Medium Density	P(RM-15)	920
Low Density	P(RL-7)	1030
Low Density	P(RL-6.5)	1030
Low Density	P(RL-3)	100
Schools	P(RL), OTHER	varies
Open Space - Park	P(OS-P)	60
Open Space - Com Rec	P(OS-CR)	200
Industrial	P(I)	1000
Commercial Neighborhood	P(CN)	1000
Commercial General	P(CG)	1000
Public (Utility ROW)	P	0
Open Space		
Shore	OS-S	200
Park	OS-P	140
Commercial Recreation	OS-CR	10
Commercial	OS-C	150
Mixed Use	M	2170
Mixed Use Horizontal	MH	2150
Mixed Use Vertical	MV	2490

Notes: Excludes high dischargers
Load variations for residential uses reflect DU's/acre
Schools loads were based on actual billing data

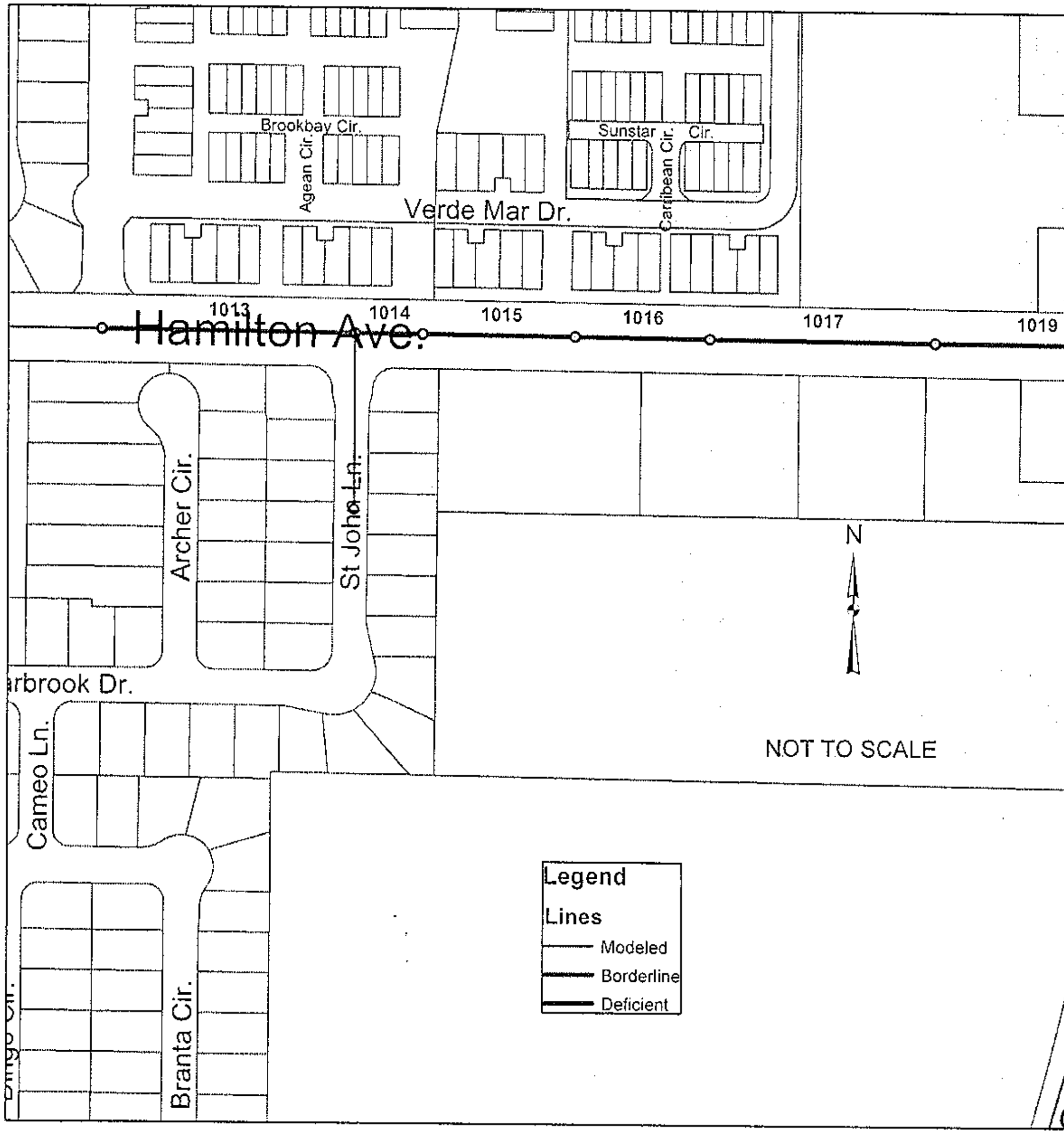
APPENDIX D_3
ESTIMATED FUTURE INCREMENTAL WASTEWATER FLOWS

Modeled Landuse	Parcel Count	Total Existing Flow (gpd)	Total Future Flow (gpd)
CG	669	596,243	631,789
CN	104	108,696	115,181
CO	22	43,203	45,787
CR	49	93,356	98,903
CV	30	51,133	54,191
I	928	890,623	943,843
M	250	383,059	406,008
MH	80	121,733	129,037
MV	124	80,049	84,839
OS-C	14	5,053	5,356
OS-CR	22	2,377	2,615
OS-P	235	68,853	72,787
OS-S	22	68,503	72,613
OS-W	120	0	0
P	142	0	0
P(CG)	3	22,735	24,099
P(CN)	1	2,787	2,954
P(I)	7	4,443	4,710
P(OS-CR)	3	526	557
P(OS-P)	4	1,647	1,756
P(RL)	11	60,794	64,442
P(RL-3)	3	923	978
P(RL-6.5)	1	3,825	4,056
P(RL-7)	4	9,924	10,521
P(RM-15)	11	16,778	17,781
P(RMH-25)	9	8,904	9,436
RH-30	436	353,256	374,451
RL-3	479	97,888	103,761
RL-4	115	37,356	39,598
RL-5	1	44,678	47,358
RL-6	53	8,373	8,875
RL-7	34235	8,694,334	9,215,994
RM-15	3888	2,254,600	2,389,876
RM-25	1	346	366
RMH	1	2,678	2,839
RMH-25	5974	3,485,959	3,695,117
ROW	32	0	0
V_CG	25	0	14,329
V_CN	5	0	2,898
V_CV	5	0	23,403
V_I	37	11,402	40,720
V_MH	8	190,082	330,713
V_MV	9	0	4,122
V_OS-C	8	0	21,581
V_OS-P	316	134	20,252
V_P	2	0	0
V_P(OS-P)	1	0	889
V_RH-30	19	0	110,985
V_RL-3	47	0	3,009
V_RL-7	15	0	11,512
V_RM-15	44	0	220,936
V_RMH-25	29	0	24,518
SIDS-SCHOOLS		541,404	806,853
Total (GPD)	**48,653**	**18,368,653**	**20,319,193**
Total (MGD)		**18.37**	**20.32**
Total Incremental Flow (MGD)			**1.95**

APPENDIX D_4

SIGNIFICANT DISCHARGERS / SCHOOLS

SID or SCHOOL (Name)	APN	Service Account No.	Service Type	Parcel Acreage (AC)	Landuse	Ultimate Flow (gpd)
SID	145-531-24	961240	INDUSTRIAL	1.15	I	33,606
SID	145-473-23	909580	INDUSTRIAL	1.11	I	10,340
SID	145-473-09	909480	INDUSTRIAL	0.56	I	9,048
SID	165-364-21	615020	COMMERCIAL	0.72	CG	47,178
AREVALOS	155-043-01			14.00	P(RL)	23,746
CIRCLE VIEW	145-381-01			13.59	P(RL)	23,054
CLARA COOK	195-081-24			9.86	P(RL)	16,723
COLLEGE VIEW	146-372-15			13.84	P(RL)	23,476
CRESTVIEW	157-481-08			13.86	CG	23,510
DR RALPH E HAWES	151-261-17			7.79	P(RL)	13,219
DWYER	MULTI	MULTI	MULTI	9.40	P(RL)	15,946
EADER	MULTI	MULTI	MULTI	10.70	P(RL)	18,151
GISLER	149-302-17			14.06	P(RL)	23,847
GLEN VIEW	145-422-19			13.39	P(RL)	22,709
HARBOR VIEW	178-761-02			15.78	SCHOOL	26,770
HAVEN VIEW	178-091-01			13.49	P(RL)	22,872
HB UNION HIGH	MULTI	MULTI	MULTI	36.92	P(RL)	62,623
HELEN STACEY/ADA	195-091-01			31.54	SCHOOL	53,488
HOPE VIEW	165-171-02			14.71	SCHOOL	24,946
ISAAC BOWERS	MULTI	MULTI	MULTI	14.24	P(RL)	24,146
LAMB	MULTI	MULTI	MULTI	14.26	P(RL)	24,183
LARK VIEW	MULTI	MULTI	MULTI	15.09	P(RL)	25,599
LEBARD	155-151-01			10.16	P(RL)	17,225
MARINE VIEW	MULTI	MULTI	MULTI	13.74	P(RL)	23,298
MEADOW VIEW	146-131-01			13.53	SCHOOL	22,945
NEWLAND	MULTI	MULTI	MULTI	14.28	P(RL)	24,224
OAK VIEW	MULTI	MULTI	MULTI	14.08	P(RL)	23,878
OKA	153-181-02			8.46	P(RL)	14,346
PARK VIEW	142-441-23			11.98	P(RL)	20,313
PERRY	153-012-20			10.15	P(RL)	17,214
PLEASANT VIEW	MULTI	MULTI	MULTI	10.81	P(RL)	18,335
RANCHO VIEW	MULTI	MULTI	MULTI	18.20	MV	30,862
ROBERT BURKE	151-372-01			7.73	P(RL)	13,102
ROBINWOOD	145-042-24			10.72	P(RL)	18,173
SCHROEDER	145-191-01			8.82	P(RL)	14,966
SMITH	023-100-08			9.67	P(RL)	16,395
SPRING VIEW	146-392-01			13.90	P(RL)	23,574
SPRINGDALE	195-214-23			8.75	P(RL)	14,832
ST BONAVENTURE	146-431-10			7.33	P(RL)	12,433
ST FRANCIS PVT	MULTI	MULTI	MULTI	9.51	P(RL-6.5)	16,132
TALBERT	153-132-19			13.83	P(RL)	23,456
UNK SCHOOL SITE	MULTI	MULTI	MULTI	8.00	P(RL)	13,570
VILLAGE VIEW	146-072-12			12.63	P(RL)	21,412
WARDLOW	153-271-02			14.52	P(RL)	24,628
WINTERSBURG	111-010-01			54.40	CG	92,256



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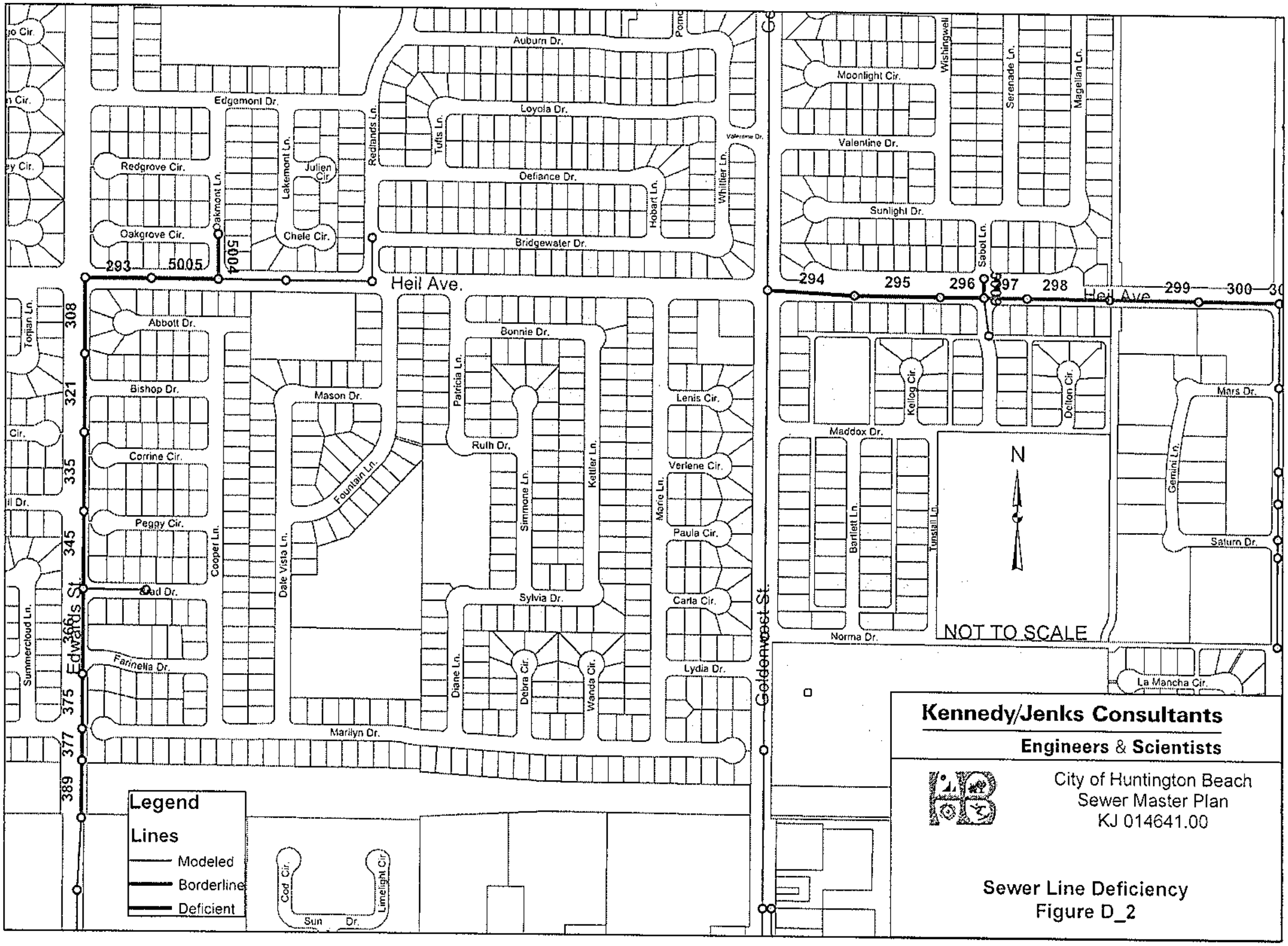
City of Huntington Beach
Sewer Master Plan
KJ 014641.00

Sewer Line Deficiency
Figure D_1



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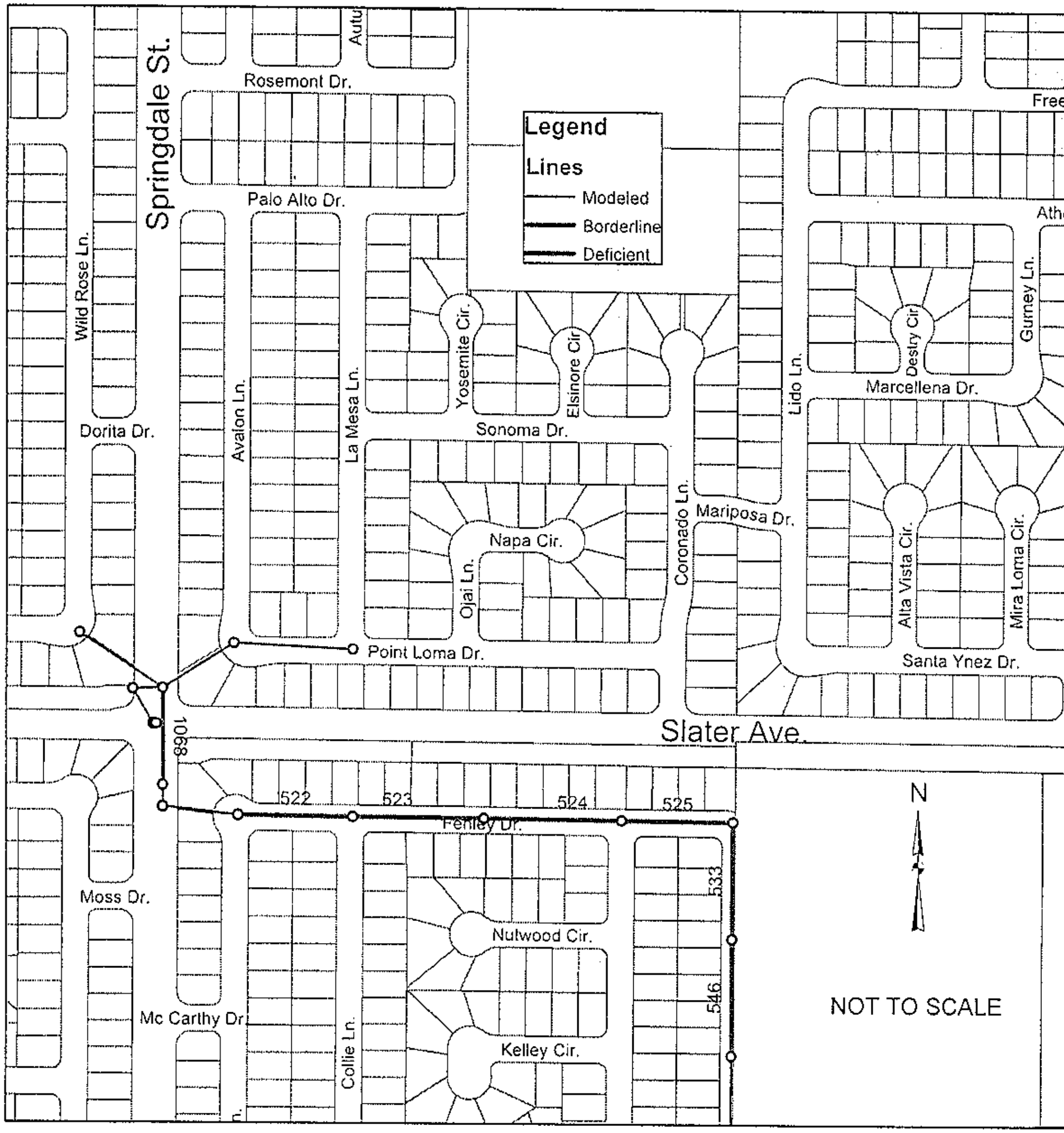
Sewer Line Deficiency
Figure D_2



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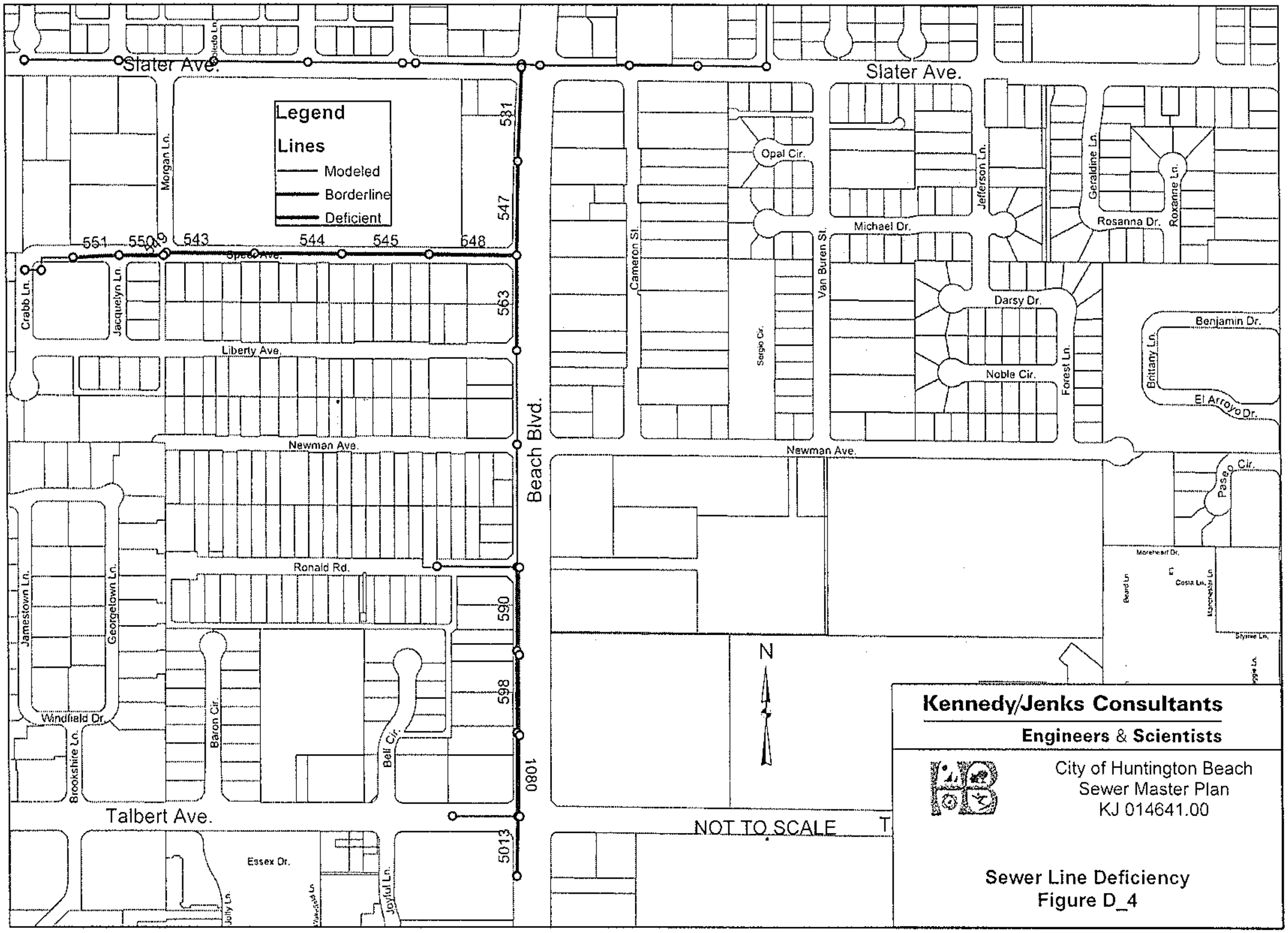
Sewer Line Deficiency
Figure D_3



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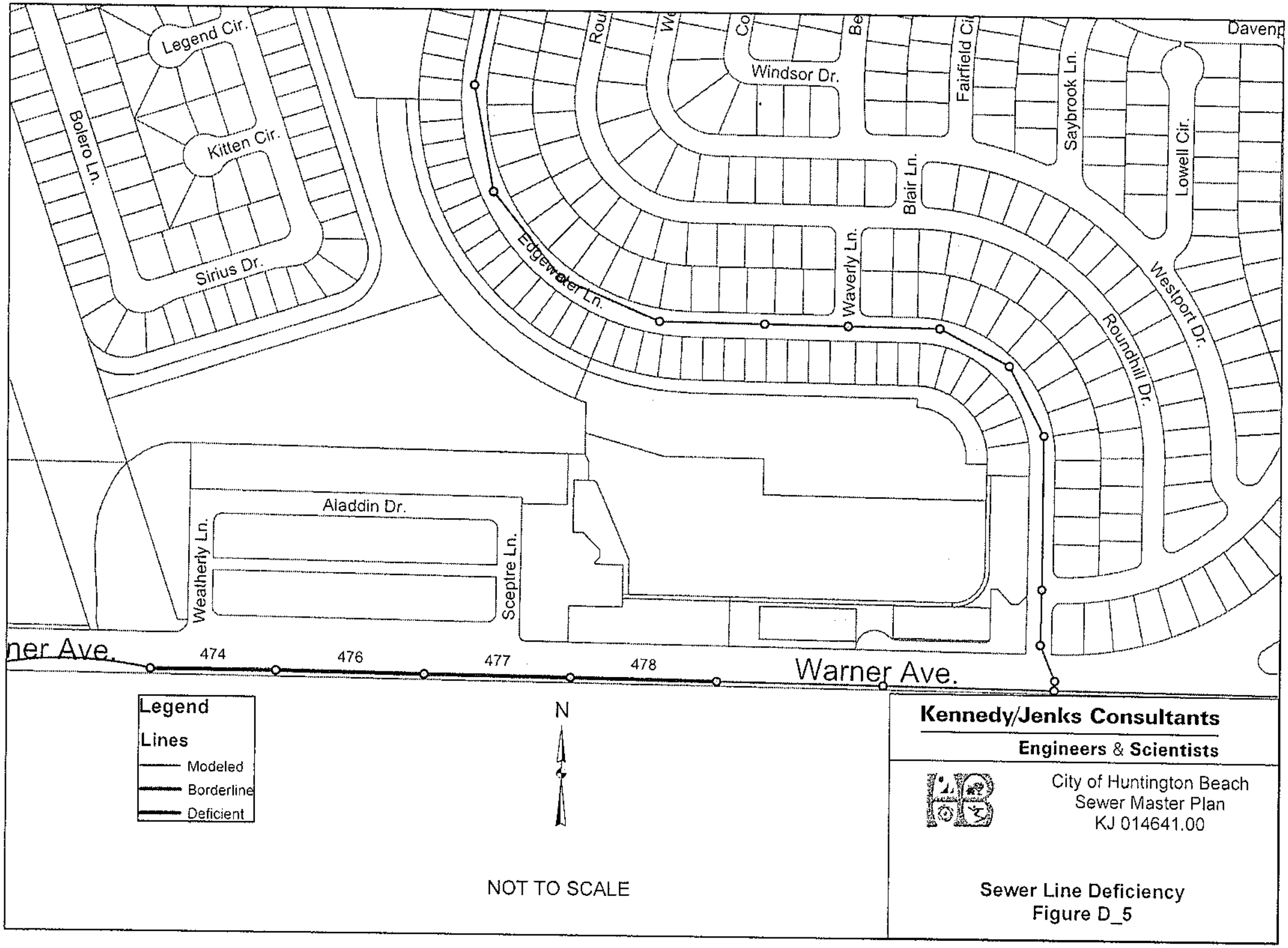
Sewer Line Deficiency
Figure D_4



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Sewer Line Deficiency
Figure D_5

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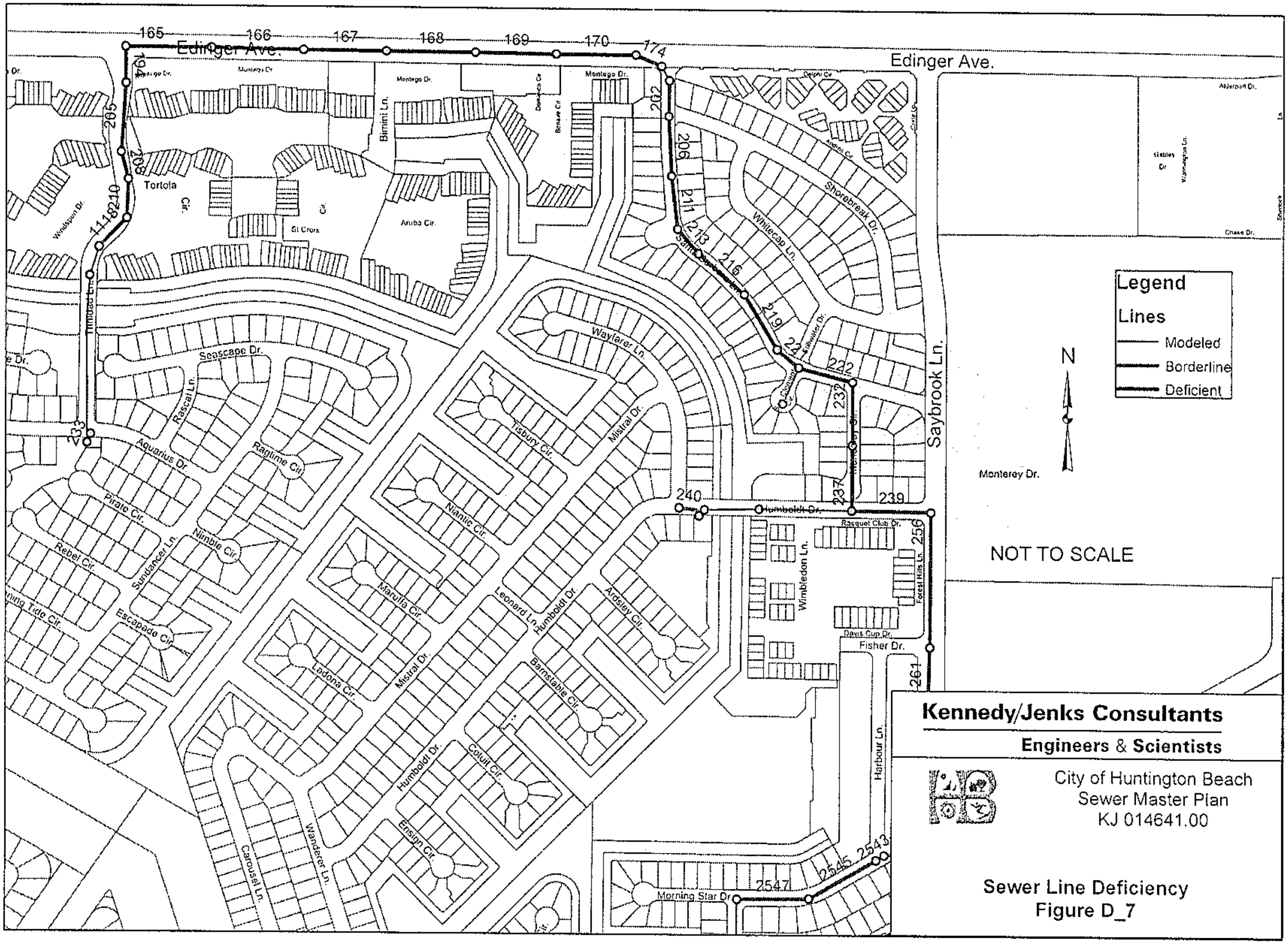
Sewer Line Deficiency
Figure D_6



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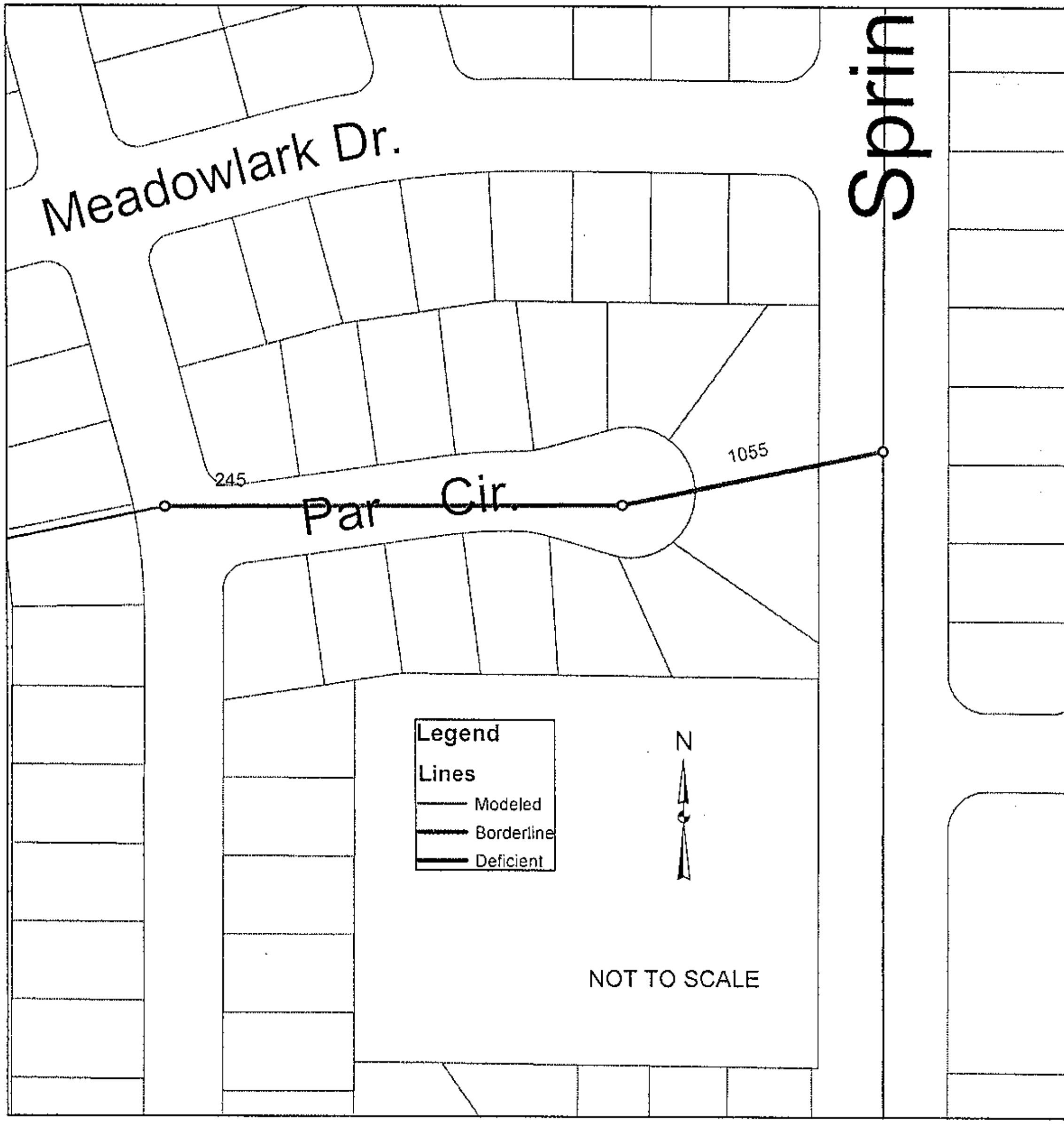
Sewer Line Deficiency
Figure D_7



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Sewer Line Deficiency
Figure D_8

Appendix E

Sewer Service Charge Ordinances

REFER TO MASTER DOCUMENT