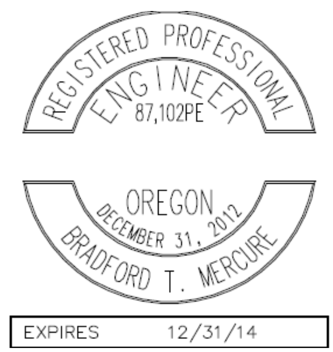
City of Rockaway Beach

Tillamook County, Oregon

Wastewater Facilities Plan

October 2014

Project No. 2004-027-37



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Executive Summary/Purpose

CHAPTER 1
EXECUTIVE SUMMARY/PURPOSE

This chapter summarizes the basis for planning, development, evaluation of alternatives, and the recommended project phasing for the City of Rockaway Beach (City) wastewater collection system and treatment plant facilities plan. It also provides the purpose for this planning document.

PURPOSE AND BACKGROUND

In 2004, the City of Rockaway Beach, Oregon contracted with Brown and Caldwell to provide a Wastewater Facilities Plan and Master Plan. The purpose of the Plan is to guide the City in providing wastewater collection and treatment services for the sewer service area over a 20 year planning period. In 2006 the Plan was finalized, but was not approved by the Oregon Department of Environmental Quality (DEQ). In 2012, the City decided to update their Wastewater Facilities Plan and Master Plan with the goal of updating: the population, flow and wasteload projections; facilities that have been constructed to-date; and the costs for the recommended projects. With the update, the City also sought to re-evaluate the recommendations for conveyance and treatment. The final goal of the update is to have the Plan approved by DEQ.

This Plan has been prepared in general conformance with the Oregon State Department of Environmental Quality (DEQ) guidelines. The primary objective of this document is to assist the City in ensuring adequate treatment and conveyance capacity to meet the City of Rockaway's needs over the planning period, and to ensure such facilities minimize adverse impacts to the environment, and protect the health and safety of the community in an economical and efficient manner. Minimum requirements are set forth by the Environmental Protection Agency and the DEQ through the National Pollution Discharge System (NPDES) Permit and by State of Oregon Water Quality Standards. The approach to meet the purpose of this Plan is to :

- Describe existing effluent limitations under the City's NPDES permit.
- Define environmental and physical conditions in the sewer service area.
- Develop population, flow and wasteload projects for the wastewater facilities.
- Evaluate wastewater collection and treatment systems.
- Evaluate alternatives and provide recommendations for wastewater collection and treatment.
- Evaluate financial options for funding the recommended improvements and estimate impacts to sewer service rates.
- Assist in developing a schedule for implementing the recommended improvements.

BASIS OF PLANNING

This section discusses the existing wastewater collection and treatment system and the characteristics of the Rockaway Beach Wastewater Treatment Plant (WWTP) service area.

Study Area Characteristics

Rockaway Beach is a coastal community with a typical marine climate characterized by relatively dry summers and wet winters, with high seasonal rainfall. Rockaway Beach is predominately residential, with a small commercial area located along Highway 101.

The City has a large number of seasonal residents with vacation homes in the urban growth area. Based on 2010 Census data, it is estimated that 62 percent of the houses in Rockaway have seasonal residents and 38 percent have permanent residents. In 2012, the City billed 1,568 residential units for sewer service. The residential usage represents about 80 percent of the total sewer usage with the other 20 percent being commercial. Calculated from the Census data, it is estimated that 977 vacation houses and 591 permanent-resident houses received sewer service in 2012. The sewer service population for 2012 is estimated to be 3,089 with 1,165 permanent residents and 1,924 seasonal. The City had 75 commercial customers out of a total possible 86 in 2012. The City does not have any industrial users.

The unincorporated area of Nedonna Beach located in the UGA is expected to be served with sanitary sewers by the City within the planning period. The population of Nedonna Beach in the 2010 Census was approximately 600. The population that is currently being served is 80 which includes both seasonal and permanent residents.

Wastewater Flow Rates

Daily monitoring reports (DMRs) from the City's wastewater treatment plant (WWTP) were analyzed from January 2007 through December 2011 to determine average and maximum observed flows. The flow measurements were taken on the effluent side of the treatment plant since there is no flow meter on the influent side. These flows were compared to projected flows using the Department of Environmental Quality's Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon, and the more conservative value was used.

The average annual flows (AAF), average dry weather flow (ADWF) and average wet weather flow (AWWF) are calculated based on the trends in the existing flow data. Other flows that are determined are the maximum month dry weather flow (MMDWF), maximum month wet weather flow (MMWWF), peak daily average flow (PDAF), and the peak instantaneous flow (PIF), or peak hourly flow.

Table 1-1. Rockaway Beach Existing Flows

Flow Condition	WWTP Flow (MGD)
ADDWF	0.18
MMDWF	0.28
AAF	0.23
ADWWF	0.28
MMWWF	0.47
PDAF	1.19
PIF	1.60

Wastewater flows were projected for the 2017 (5-year) and the 2032 (20-year) design years. Based on the unit flow values and the population/land use projections developed in the Plan, the following flow projections were determined for the 2017 and 2032 conditions.

Table 1-2 – Flow Projections

Projected Flows (MGD)			
	AAF	**PDAF**	**PIF**
2012	0.23	1.19	1.60
2017	0.27	1.27	1.75
2032	0.46	1.70	2.50

Wastewater Loading

A detailed analysis of the City's DMRs from January 2007 to December 2011 was conducted to aid in establishing a basis for long-term projections of organic loadings and wastewater composition for the planning period. This information was utilized in selecting and sizing treatment technologies to remove the unwanted wastewater components in order for the City to meet the requirements of its discharge permit. A summary of the existing Biochemical Oxygen Demand, 5-day (BOD5), a measure of the concentration of organic impurities in wastewater, and Total Suspended Solids (TSS), solids that float on the surface of, or are in suspension in wastewater and that are largely removable by laboratory filtering, are summarized in Table 1-3 and 1-4 below.

Table 1-3. Existing Influent BOD5 Concentrations and Loads

	BOD (mg/L)						BOD (lb/d)					
	2007	2008	2009	2010	2011	2007-2011	2007	2008	2009	2010	2011	2007-2011
Average	174	172	189	204	208	**190**	349	325	320	335	268	**320**
Summer Average	211	198	216	211	224	**210**	356	315	289	249	233	**290**
Winter Average	136	146	163	196	191	**170**	343	335	352	421	302	**350**
Maximum Month	265	217	273	272	286	**290**	480	449	420	527	403	**530**
Maximum Day	299	256	299	318	478	**480**	678	652	607	679	537	**680**

Table 1-4. Existing Influent TSS Concentrations and Loads

	TSS (mg/L)						TSS (lb/d)					
	2007	2008	2009	2010	2011	2007-2011	2007	2008	2009	2010	2011	2007-2011
Average	193	175	189	196	201	**190**	387	333	316	321	256	**320**
Summer Average	226	204	222	211	219	**220**	386	323	295	248	235	**300**
Winter Average	159	147	156	181	183	**170**	389	343	336	395	277	**350**
Maximum Month	303	228	270	264	252	**300**	580	468	437	521	367	**580**
Maximum Day	393	260	298	298	341	**390**	726	683	743	812	597	**810**

Based on the existing unit load values and the population projections developed in the Plan the following load projections were determined for the 2017 and 2032 conditions.

Table 1-5 – BOD Load Projections

Projected BOD (lb/day)			
	Average	Max Month	Peak Day
2012	320	530	680
2017	366	606	777
2032	592	981	1,259

Table 1-6 – TSS Load Projections

Projected TSS (lb/day)	Average	Max Month	Peak Day
2012	320	580	810
2017	366	663	926
2032	592	1,074	1,499

EXISTING WASTEWATER COLLECTION SYSTEM

The original sewer system in Rockaway Beach was constructed in 1954. The original sewer system in Manhattan Beach area was constructed in 1965. These sewers were constructed using asbestos cement (Transite) pipe. The Nedonna Beach area is not sewered except for the White Dove Estates, which was annexed to the City and has been served by its sewers. Two major expansions to the 1954 sewer system occurred in 1979 and 1981. These expansions were the first to use polyvinyl chloride (PVC) pipe. Most, if not all, of the sewer extensions to the original Rockaway Beach and Manhattan Beach systems have been constructed using PVC pipe.

The only reported overflow occurs in the manholes located immediately downstream of the Lake Lytle and NW 17th Lift Station forcemains at the discharge manholes. These overflows occur only when both pumps are operating during large storm events. There are no other known routine overflow points from the sewer system that occur during peak flow periods.

Thousands of feet of sewer lines have been internally inspected using video equipment in 1995, 2000 and each year since 2007. These inspections revealed a number of lines which have sags in them. The inspections also revealed sewers that had structural defects or holes in them and displaced joints. The structural problems were found at isolated points and were not a recurring situation in any of the lines inspected. Structural defects are generally corrected as they are found. Over all the main lines are in good condition, particularly the older AC lines. Generally, no significant sources of infiltration were identified during the inspections.

The City has eight wastewater lift stations serving the service area. All wastewater must be pumped in order to reach the wastewater treatment plant (WWTP). Since most of the town has little natural slope, wastewater is pumped more than once. All of the wastewater flow ultimately goes to the Main Lift Station and is pumped into the WWTP. The original sewer collection system included three lift stations (Main, South Fifth Street, and North Fourth Street). All of the original stations have been replaced. Five lift stations have been added to the collection system as Rockaway Beach has grown. Of these, four utilize centrifugal pumps and one utilizes compressed air to eject the wastewater out of the station and into the nearest gravity sewer. The lift stations are in overall fair operating condition - see the body of the Plan for further discussion.

A hydraulic analysis was conducted using computer modeling in 2004 to determine the performance of the collection system under design conditions. Specifically, the model was used to identify pipes within the collection system that are inadequately sized to handle current and future design flows. The results of the modeling are discussed on the following pages.

EXISTING WASTEWATER TREATMENT SYSTEM

The Rockaway Beach Wastewater Treatment Plant (WWTP) was originally constructed in 1954. The construction consisted of a primary clarifier, trickling filter, anaerobic sludge digester and sludge drying beds. The plant underwent an expansion in 1979 which consisted of a headworks with screening/grit removal, a package aeration basin/secondary clarifier/disinfection tank, in-plant pump station, tertiary filters, and overflow lagoon. The expansion also consisted of converting the anaerobic digester to an aerobic digester and converting the sludge drying beds to a humus pond.

A project that added dechlorination, an effluent pump station and ocean outfall was completed in 2005. The tertiary filters were demolished as part of the project. The WWTP flow meter is located at the Chlorine Contact Chamber. To date, treatment plant consists of the following unit processes:

- Grit removal
- Screening
- Primary clarification
- Tricking filter biological treatment
- Activated sludge biological treatment with positive displacement blowers
- Secondary clarification
- Disinfection with chlorine
- Dechlorination
- Effluent pump station and ocean outfall
- Aerobic sludge digestion
- Overflow storage and sludge holding ponds

The treatment plant also includes support facilities, including maintenance, laboratory, and office buildings and an emergency generator.

The City currently disposes dewatered, dried biosolids at an approved municipal solid waste (MSW) landfill.

COLLECTION SYSTEM EVALUATION AND RECOMMENDATIONS

Capital Improvements:

Improvements are needed to the Rockaway Beach collection system to address population increases and identified deficiencies. The collection system should be designed for adequate capacity to completely contain and transport the expected peak flows through the pipes to the WWTP. As described in Chapter 3, the 5-year peak hour flow was analyzed for each of the flow monitoring basins in 2004 to determine the necessary upsizing requirements to the collection system. From this analysis, the entire main gravity line from the discharge manhole for the Lake Lytle Pump Station to the Main Pump Station requires additional capacity. The City has indicated that the manholes downstream of the Lake Lytle and the NW 17th lift stations currently experience overflows when both pumps are operating during high flow events. The gravity line from the White Dove Pump Station discharge manhole to the NW 17th Lift Station will require upsizing as well. Rather than upgrade the gravity sewers, the City prefers to bypass these sections with new forcemains and forcemain extensions due to the difficult soil conditions generally in the City.

Redirecting flow from the main gravity line from the discharge manhole for the Lake Lytle Pump Station to the Main Pump Station will require an upgrade/ rebuild of the Lake Lytle, NW 17th Ave., N 4th Ave., and the Main Pump Stations. The selected option is to pump the wastewater from each pump station using a common forcemain. The advantage of this option is that each of the pump stations would be able to convey wastewater to the treatment plant independently and would avoid inefficiencies of pumping wastewater 2-4 times if the pump stations operated in series. The disadvantage is that the pumps will operate under varying hydraulic conditions depending on whether other pump stations are operating, which may not result in the most efficient operation of the pumps under all conditions.

Maintenance:

The City performs annual video inspections and cleaning of sewer piping. Point repairs are performed where there are structural problems with the main piping and manholes. Smoke testing has been done on service laterals to determine their integrity. It is recommended, that the City adopt a more formal maintenance program to map out where the pipe inspections will occur. The most significant problems were lines that had sags in them. There were also a few isolated areas with structural defects such as holes or displaced joints. These areas are repaired as part of the ongoing sewer maintenance and repair program.

In general, infiltration has not been a major problem in Rockaway Beach although service lateral connections for the older AC pipe generally show signs of infiltration where they were videoed. Inflow through flooded sections of the sewer system has caused most of the high flow problems in the sewers and at the Rockaway Beach Wastewater Treatment Plant (WWTP). It is recommended that the City install manhole inserts for the areas that are prone to flooding.

A 760 foot section of 8-inch cast iron or ductile iron pipe without cement mortar lining located on the right-of-way for NW 4th Ave. between Falcon and Juniper St. is in poor condition and receives a lot of I&I. It is recommended the City replace this section of piping.

Some of the City's pump stations are approaching 20 years in age, which is the point at which equipment may need to be replaced or may require major service. The South Sixth Avenue Pump Station was rebuilt in 1990 and will likely need to be rebuilt again with some modifications sometime during this planning period due to age. The South Fifth Avenue Station was last rebuilt over 30 years ago. It is recommended that this station be rebuilt with a new submersible pump station. The Northeast 23rd Avenue Pump Station should have telemetry added so that the station can be monitored, and the station should be fenced for security.

ROCKAWAY BEACH WWTP EVALUATION AND RECOMMENDATIONS

Improvements are needed to upgrade aging process equipment, improve reliability, and extend the useful life of the WWTP. The existing plant facilities are expected to meet planned future flow rate increases without major expansion. The recommended improvements are listed below.
Primary Clarifier - The primary clarifier sludge pump will need to be replaced with a model that does

not require seal water. The influent/effluent piping will need to be modified and upsized to accommodate future peak hour flows. A metal walkway should be constructed around the perimeter of the clarifier to provide personnel with the ability to safely access the weirs. These modifications should also be coordinated with current City Plans to replace the clarifier's catwalk and scraper system in 2014.

Trickling Filter - The direct feed/recirculation pump is in poor condition and should be replaced by a new tandem pumping system to meet reliability requirements. The capacity of the pumps should be increased to 2 MGD each to further utilize the available capacity of the trickling filter. The distribution mechanism, the influent/effluent piping and the recirculation pipe should be upsized as well. The trickling filter media should be replaced with plastic media to help increase the air circulation and provide better treatment.

Aeration Basin - The air piping in the aeration basin from above the water level back to the blower building is in poor condition and will need to be replaced in the 1-5 year time period. Dissolved oxygen monitors should be added to ensure adequate oxygen is provided and to help conserve energy. The performance of the aeration basin should be monitored, particularly during the wet season when the hydraulic detention times are low.

Air Blowers - The largest energy consumers at the plant are the aeration blowers. Adjustable speed controllers should be added to all the blowers to adjust the air supply in proportion to the flow to conserve energy. It is also recommended to replace the existing course-bubble air diffuser in the aeration basin, with a fine-bubble air system that covers all of the floor area of the basin. The fine air bubbler system would increase the oxygen transfer efficiency to about 20-30 percent versus the current of efficiency of 4-6 percent typical for the coarse air system in the basin currently[1]. This option would also conserve energy and the costs for installation may be partially covered by the electric power utility. The costs of installing the fine-air diffuser versus a new blower would be comparable.

Secondary Clarifier - The secondary clarifier experiences issues during the transition from the summer to the winter season, and the plant personnel normally have to increase the return and waste activated sludge rates (RAS and WAS) to manage the issue. The issue does not appear to be related to the clarifier design as it appears to be within or near typical operating parameters for solids loading, overflow rate and detention time for existing and projected flows. It is recommended that a study be performed to determine what additional adjustments can be done to prevent or mitigate the issue.

Maintenance, such as painting and mechanical adjustment, to the existing secondary clarifier are needed. In addition, it is recommended that the return-activated sludge (RAS) airlift pumps be replaced with centrifugal pumps or horizontal propeller pumps to provide more flexibility and have the RAS return independent of the air system. .

Miscellaneous

The process yard piping and valving should be inspected and repaired as needed. Since the wastewater treatment plant is located in a residential area and has received complaints regarding aesthetics, it is recommended that landscaping be done around the treatment plant. In addition, the fencing and gate should be replaced with new black vinyl-coating fencing.

[1] *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 600.

Although a detailed electrical review was not conducted, there is significant corrosion on some of the electrical panels and equipment. Many exposed panels and receptacles are badly corroded and need replacement to avoid potential safety concerns. Site lighting could be improved to provide better and more energy efficient lighting for safe operation. An evaluation should be done on the existing electrical panels and equipment to determine what equipment should be replaced and the costs. This evaluation should take into consideration the addition of a SCADA system to the treatment plant in the future.

A Supervisory Control and Data Acquisition (SCADA) system should be evaluated and implemented at the treatment plant. A SCADA system would essentially computerize the treatment plant and enable staff to monitor, log and control treatment processes in a centralized location. Benefits include better efficiency and optimization of the plant operation and reduction in maintenance costs.

The existing 125 kW emergency generator is too small for operation of the entire treatment plant and should be replaced with a larger generator. Preliminary sizing for the new generator for planning purposes is 275 kW.

COST ESTIMATE AND PHASING

Table 1-7 presents the recommended improvements to the collection and treatment systems for Rockaway Beach in order of priority based on the deficiencies identified in Chapters 3 and 4. Projects are designated as either a capital improvement (CIP-#), for projects that increase capacities, or a maintenance item (M-#), for projects that maintain existing capacities. The estimated capital costs (construction plus engineering, administration, and miscellaneous costs) for the recommended improvements are also summarized below. These costs are in 2012 dollars. Detailed cost estimates are presented in Appendix H for the capital improvements projects (CIP's). A DEQ Land Use Compatibility Statement (LUCS) will need to be prepared and submitted prior to the construction of any of these projects. It is not anticipated that there will be any issues with LUCS since the improvements will be taking place in existing right-of-ways and at existing facilities.

Table 1-7. Capital Cost Estimate

Project No.	Description	Feet of Pipe	Construction Cost	Engineer, Legal & Admin. Cost (20-25%)	Contingency	Total Cost (rounded)	Time Line
M-1	Rebuild Main Pump Station.	-	$610,000	$152,500	$152,500	$915,000	1- 5 yr
M-2	Slip-line and Replace 8" Cast Iron/Ductile Iron Gravity Pipe.	760'	$86,450	$21,613	$21,613	$130,000	1- 5 yr
M-3	Rebuild South 5th Pump Station.	-	$240,000	$60,000	$60,000	$360,000	1- 5 yr
CIP-A	Existing Primary Clarifier Work and Influent/ Effluent Piping Upgrade.	-	$144,000	$36,000	$36,000	$216,000	1- 5 yr
CIP-B	Trickling Filter Rebuild and Influent/ Effluent Pipe Upgrade.	-	$672,000	$168,000	$168,000	$1,008,000	1- 5 yr
M-4	Replace WWTP Aeration Basin Manifold Piping. Add Dissolved Oxygen Sensors. Replace Air Diffuser System in Aeration Basins with Fine-Air Diffuser. Provide VFD's for Existing Blowers.	-	$150,000	$37,500	$37,500	$225,000	1- 5 yr
M-5	Existing Secondary Clarifier Maintenance and Replacement of RAS Pumps.	-	$50,000	$12,500	$12,500	$75,000	1-5 yr
M-6	Inspect and Repair WWTP Process Yard Piping.	-	$40,000	$10,000	$10,000	$60,000	1- 5 yr
M-7	Study to Evaluate Existing WWTP Electrical Equipment and New SCADA System.	-	$0	$40,000	$0	$40,000	1- 5 yr
M-8	WWTP Lighting and Electrical System.	-	$95,000	$23,750	$23,750	$143,000	1- 5 yr
CIP-C	Upgrade WWTP Generator.	-	$85,000	$21,250	$21,250	$128,000	1- 5 yr
M-9	Study to Evaluate Existing Secondary Clarifier and Aeration Basin.	-	$0	$25,000	$0	$25,000	1- 5 yr
Sub-total (1-5 yr)		**760'**	**$2,172,450**	**$608,113**	**$543,113**	**$3,325,000**	

Table 1-7. Capital Cost Estimate (continued)

Project No.	Description	Feet of Pipe	Construction Cost	Engineer, Legal & Admin. Cost (20-25%)	Contingency	Total Cost (rounded)	Time Line
CIP-D	New 12" and 14" Common Forcemain from N 3rd Ave. and Highway 101 to WWTP Headworks.	2650'	$618,100	$123,620	$148,344	$890,000	5- 10 yr
CIP-E	New 10" Common Forcemain from 12th Ave. & Miller St. to N 3rd Ave. & Highway 101.	4,750'	$911,500	$182,300	$218,760	$1,313,000	5- 10 yr
CIP-F	Rebuild Lake Lytle Pump Station and Extend Existing 8" Forcemain.	440'	$781,500	$195,375	$195,375	$1,172,000	5- 10 yr
CIP-G	Add SCADA to WWTP.	-	$110,000	$27,500	$27,500	$165,000	5- 10 yr
Sub-total (5-10 yr)		**5,190'**	**$2,421,100**	**$528,795**	**$589,979**	**$3,540,000**	
CIP-H	Rebuild NW 4th Ave. Pump Station and Construct New 10" Forcemain.	430'	$669,600	$167,400	$167,400	$1,004,000	10- 15 yr
CIP-I	Extend Existing 6" Forcemain for White Dove Pump Station.	1,470'	$207,600	$41,520	$49,824	$299,000	10- 15 yr
M-10	Add Telemetry to NW 23rd Ave. Pump Station.	-	$10,000	$2,500	$2,500	$15,000	10- 15 yr
M-11	WWTP Landscaping and Fencing.	-	$55,000	$13,750	$13,750	$83,000	10- 15 yr
Sub-total (10-15 yr)		**1,900'**	**$942,200**	**$225,170**	**$233,474**	**$1,401,000**	
CIP-J	Replace NW 17th Ave. Pump Station and Construct New 8" Forcemain.	1,000'	$577,000	$144,250	$144,250	$866,000	15- 20 yr
M-12	Rebuild South 6th Pump Station	-	$160,000	$40,000	$40,000	$240,000	15-20 yr
Sub-total (15-20 yr)		**1,000'**	**$737,000**	**$184,250**	**$184,250**	**$1,106,000**	
Total		**8,850'**	**$6,272,750**	**$1,546,328**	**$1,550,816**	**$9,372,000**	

CIP-Capital Improvement Project
M-Maintenance Project

CHAPTER 2

Study Area Characteristics and Basis of Planning

CHAPTER 2 STUDY AREA CHARACTERISTICS AND BASIS OF PLANNING

Developing a long-range wastewater facilities and management plan for the City of Rockaway Beach (City) requires that a number of local factors be recognized, including population, land use, climate, precipitation, local soils, and topography. This information is summarized below for the area to be served by the City's wastewater system. This area includes all land within the Rockaway Beach Urban Growth Boundary (UGB), including Nedonna Beach.

LOCATION AND TOPOGRAPHY

Rockaway Beach is located approximately 4 miles north of Garibaldi and 8 miles south of Nehalem. Highway 101 runs north to south through the City, as does the Tillamook Bay Railroad. The City encompasses approximately 1,200 acres. The UGB includes Rockaway Beach, Nedonna Beach to the north, and a portion of Twin Rocks to the south.

A map of the City identifying the city limits and the UGB is shown in Figure 2-1. Rockaway Beach adjoins the Pacific Ocean on the west and the Coast Range hills on the east. The area from the ocean to Easy Street varies in elevation between sea level and about 20 feet. The land area in the hills rises to elevation 100 feet.

GEOLOGY AND SOILS

The geology of the study area, as described in the *Soil Survey of Tillamook Area* published by the U.S. Department of Agriculture, Soil Conservation Service, consists of dune sand along the coastline and Astoria-Hembre association soils inland and up the steep slopes. The dune land consists of wind-drifted sand that has little vegetation on it. The inland soils are well drained, formed from either igneous rock or weathered soft shale. The water table in the lower elevation areas between the coast range hills and the ocean lies within 4 feet of the surface for much of the year. A soils map and description from the Natural Resources Conservation Service Web Soil Survey is included in Appendix A.

CLIMATE

Climate is an important issue to consider when developing a long-range plan. Weather conditions affect the types of treatment and processes that can be considered for plant upgrades and expansions. Rainfall and groundwater conditions affect the amounts of extraneous water infiltration and inflow (I/I) which can enter into the sanitary sewer system, and which must be treated at the Rockaway Beach Wastewater Treatment Plant (WWTP). The level of sewer rehabilitation necessary is related to the amounts of I/I that can be economically eliminated.

Rockaway Beach's weather is characteristically that of Northwest coastal communities with wetter winters than summers. Relative humidity is high most of the time. The yearly average rainfall and snowfall is about 93 inches, and 5 inches, respectively. Most of the rainfall, an average of 65 inches, occurs in the period between November and March. Summer rainfall (June, July, and August) averages about 6 inches. Precipitation data from the National Climatic Data Center for the nearest weather station is included in Appendix B.

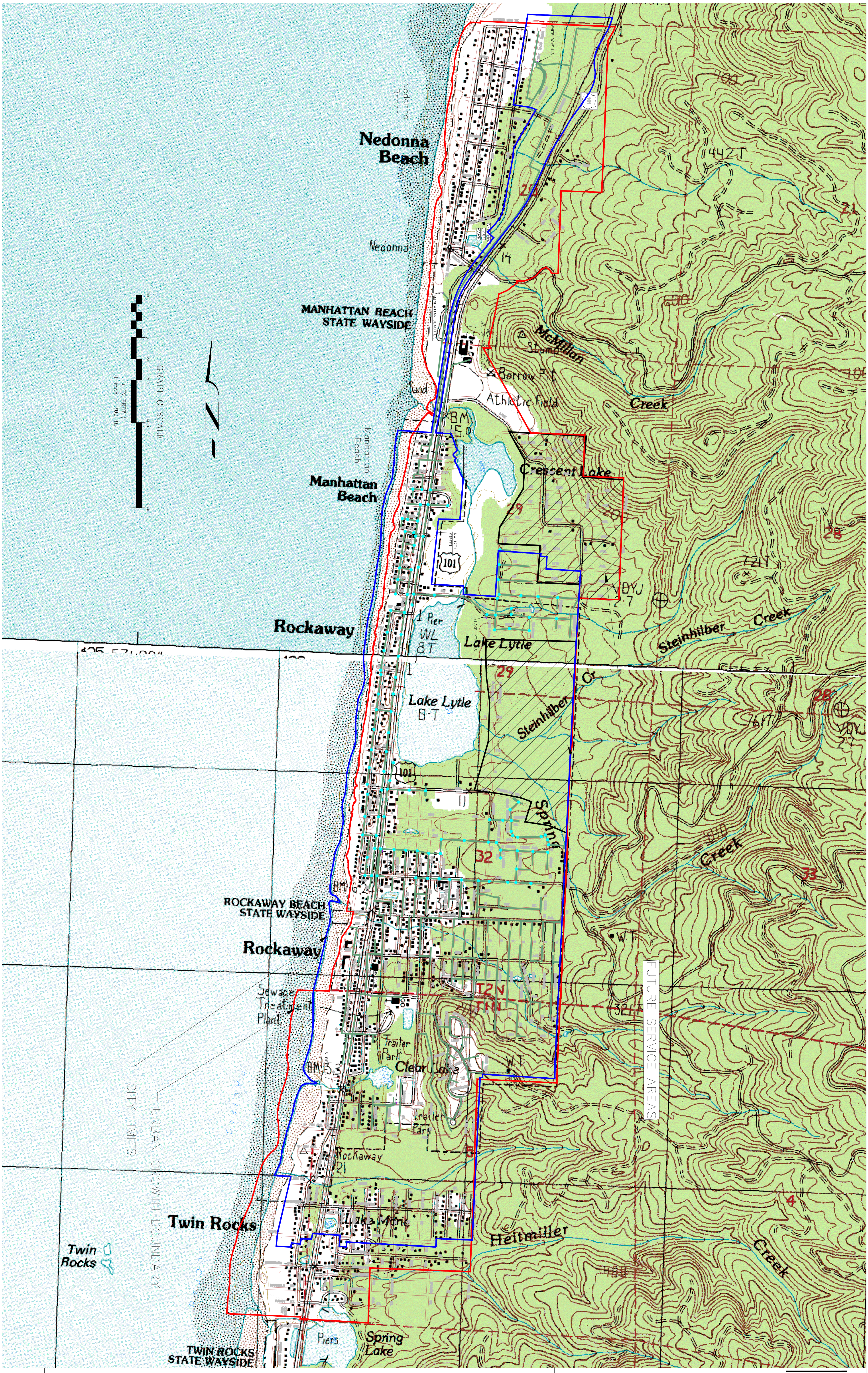


FIG. 2-1	DRAWN BY: BTM DATE: October 14, 2013	Sewer Service Areas and Topography	CITY OF ROCKAWAY *SEWER FACILITIES PLAN*	H B H Consulting Engineers

Average yearly temperature in Rockaway Beach is about 50 degrees Fahrenheit. Summer highs in the 90s are encountered, with winter lows in the teens. Extended periods below freezing are not encountered nor expected.

FLOODPLAINS

A Flood Insurance Rate Map showing the 100-year and 500-year flood plains for the City of Rockaway Beach is included in Appendix C. Most of the low lying areas of the City are located in the 100-year flood plain except for several isolated pockets along State Highway 101. The south portion of the existing wastewater treatment plant site appears to be located in the 100 year flood plain.

WINDS

The Rockaway Beach area is subject to winds for most of the year. Prevailing winds are from the northwest during the summer and from the south and southwest during the winter. Gale force winds are normal during the winter months. Wind patterns will blow any WWTP odors to adjacent properties. The present plant location makes odor containment/elimination a key concern.

LAND USE

Table 2-1 below shows the land use within the City of Rockaway's UGA. A map showing the land use areas is included in Appendix D.

Table 2-1. 2012 Land Use Within UGA

Zoning	Acres	% of UGA Land
R1, Single Family/Duplex	366	25.2
R2, Residential	396	27.3
R3, Low Density Residential	153	10.5
RR, Residential Resort	112	7.7
SRR, Special Residential Resort	3	0.2
C1, Commercial	76	5.2
SA, Special Wetlands Area	267	18.4
WD, Waterfront Development	9	0.6
RMD, Residential Manufactured Dwelling	51	3.5
OS, Open Space	20	1.4
Totals	**1,453**	**100**

WETLANDS

Appendix E contains the locations of wetland areas for the City of Rockaway Beach obtained from the Oregon Division of State Lands.

SOCIOECONOMIC ENVIRONMENT

The median family income for the City of Rockaway Beach residents based on the 5-year estimate (2007-2011) from the Census American Community Survey is $34,375. This dollar amount compares to $41,400 for the county and $49,850 for the state. Approximately 98 percent of the residents of Rockaway Beach are white, with 1.2 percent a mix of two or more races and the rest consisting of the ethnic groups. For Tillamook County, approximately 93 percent of the residents are white with about 2 percent consisting of two or more races and the rest consisting of ethnic groups.

Information regarding percentages of low-income populations for Rockaway Beach and Tillamook County was not available.

EXISTING POPULATION

Rockaway Beach is predominately residential, with a small commercial area located along Highway 101. Table 2-2 lists the full-time resident population, as determined by Portland State University (PSU) and the U.S. Census Bureau.

Table 2-2. City Permanent Population Since 1990

Year	Population
1990 (Census)	970
2000 (Census)	1,267
2012 (PSU)	1,320

The City also has a large number of seasonal residents with vacation homes in the UGA. Based on 2010 Census data it is estimated that 62 percent of the houses in Rockaway have seasonal residents and 38 percent have permanent residents. The average number of persons per household based on Census is 1.97. In 2012, the City billed 1,568 residential units out of a total possible 1,568 units for sewer service. The residential usage represents about 80 percent of the total sewer usage with the other 20 percent being commercial. Therefore, it is estimated that 977 vacation houses and 591 permanent-resident houses received sewer service in 2012. Using 1.97 persons per household, the sewer service population for 2012 is estimated to be a 3,089 with 1,165 permanent residents and 1,924 seasonal.

The unincorporated area of Nedonna Beach located in the UGA is expected to be served with sanitary sewers by the City within the planning period. The population of Nedonna Beach in the 2010 Census was 600. The population that is currently being served is 80 which includes both seasonal and permanent residents. Table 2-3, shows the estimated 2012 sewer service population with Nedonna Beach separated out.

Table 2-3. 2012 Sewer Service Population and EDU's

Area/Location	Population	EDU's
Rockaway Permanent Population	1,134	576
Nedonna Beach	80	41
Seasonal Residents	1,875	952
Total Sewer Service	3,089	1,568

EXISTING NON-DOMESTIC SEWER SERVICE AREAS

The City has 75 commercial customers out of a total possible 86 in 2012. They accounted for approximately 20 percent of the sewer usage. The City does not have any industrial users.

PROJECTED SEWER SERVICE POPULATION

The City of Rockaway Beach's population and land use patterns have the most important influence on flows and loads to the wastewater treatment system. The current population and projected population growth within the service area are the key parameters used in projecting future sewage flows and loads. These projections are used to assess the adequacy of existing infrastructure and develop design criteria for future treatment systems.

The planning period for wastewater facilities must be long enough to allow the City to develop and implement a long-range program. Planning of this type cannot be too long-term because the accuracy of estimates decreases as projections go into the future. The planning period required by the Oregon Department of Environmental Quality (DEQ) for facilities plans is 20 years. This period is reasonable for Rockaway Beach and this study, and is recommended. Therefore the planning period will be from 2012 to 2032. All of the population growth is expected in the area east and northeast of Lake Lytle.

There are three types of growth that will occur in Rockaway Beach - the permanent population growth, the seasonal population growth, and the growth of the sewer service area by incorporating existing housing areas in the urban growth area that do not currently have a sewer collection system.

The growth of the permanent population is expected to occur at an average annual growth rate of 1.5 percent over the planning period. The growth is based on the City's historical growth over a 22 year period from 1990 to 2012 as shown in Table 2-2 and is consistent with Tillamook County's projections for the City.

The growth of the seasonal population is based on the City's forecast for seasonal housing needs from 2007 to 2027 as outlined in the City's 2007 Comprehensive Plan. The Comprehensive Plan lists a range of 350 to 1,200 units needed for seasonal residents over the 20 year period. Since very little growth has occurred within the City since 2007, this Plan will assume a median of 775 seasonal homes needed for the 2012 to 2032 planning period. Assuming 1.97 persons per household the total seasonal population change will be approximately 1,500, which translates to an average annual growth of 2.9 percent.

The City expects that the entire Nedonna Beach area will be built out (including the portion of the Nedonna Beach area recently annexed into the City) and served within the planning period. The population of the Nedonna Beach area is expected to be 800 at build-out. There are currently 600 seasonal and permanent residents estimated to live at Nendonna Beach. An average annual growth rate of 1.5 percent would be required to have a population of 800 in 20 years. The average annual rate at which Nedonna Beach residents would need to be added to the sewer system over 20 years from the 80 residents currently being served is 12 percent.

Table 2-4 shows the population growth to occur within the sewer service area over the planning period. The overall average annual growth rate for the sewer service population including permanent residents, seasonal, residents and Nedonna Beach is approximately 3.1 percent.

Table 2-4. Projected Population Growth for Sewer Service Area through 2032

Year	Permanent Residents	Seasonal Residents	Nedonna Beach	Total Pop	Total EDU's
2012	1,134	1,875	80	3,089	1,568
2013	1,151	1,930	90	3,170	1,609
2014	1,168	1,986	101	3,255	1,652
2015	1,185	2,044	113	3,342	1,697
2016	1,203	2,104	127	3,433	1,743
2017	**1,221**	**2,165**	**142**	**3,528**	**1,791**
2018	1,239	2,229	159	3,627	1,841
2019	1,257	2,294	179	3,730	1,893
2020	1,276	2,361	201	3,837	1,948
2021	1,295	2,430	225	3,950	2,005
2022	1,314	2,501	253	4,067	2,065
2023	1,333	2,574	284	4,191	2,127
2024	1,353	2,649	318	4,320	2,193
2025	1,373	2,727	357	4,457	2,262
2026	1,393	2,806	401	4,600	2,335
2027	1,414	2,889	450	4,752	2,412
2028	1,435	2,973	505	4,912	2,493
2029	1,456	3,060	566	5,082	2,580
2030	1,477	3,149	635	5,262	2,671
2031	1,499	3,241	713	5,453	2,768
2032	**1,521**	**3,375**	**800**	**5,696**	**2,891**

PROJECTED COMMERCIAL DEVELOPMENT

According to the City's 2007 comprehensive plan, the land requirement for commercial growth over 20 years is 8.2 acres. This represents approximately 0.6% of the land within the UGA

REGULATORY AUTHORITY

Standards for the protection of water quality are set forth by the Environmental Protection Agency (EPA) and administered by DEQ through Chapter 340 of the Oregon Administrative Rules (OAR). The general policy followed in these rules is one of non-degradation of surface waters. Discharges from WWTPs are regulated through the National Pollutant Discharge Elimination System (NPDES). The criteria in the NPDES permit are based on existing water quality in the receiving water, beneficial uses, size of discharge, and other factors.

DISCHARGE CRITERIA

Numerous factors are considered by regulators in developing treatment limits for a specific WWTP, such as the facility operated by the City. Prior discharge permits serve as a starting point in determining future requirements. Water quality regulations must be observed. The quality of the receiving water is considered to ensure that water quality standards are not violated and beneficial uses are not impaired. This section examines the regulatory issues related to discharge of effluent from the Rockaway Beach WWTP to the Pacific Ocean.

Current Discharge Permit Requirements:

The City is currently operating under a permit that expired on October 31, 2011. The permit covers three outfalls as listed below:

Table 2-5. City Sewer Outfalls

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	1	Pacific Ocean
Emergency Overflow Influent Wet Well	2	Surge Pond
Emergency Overflow North 4th Ave Lift Station	3	Manhole on North 6th Ave. to Lake Lytle

The limits imposed on the Rockaway Beach WWTP effluent are summarized in Table 2-6. Permit limits are established for BOD, TSS, pH, residual chlorine, percent removal efficiency and Enterococci bacteria.

Table 2-6. NPDES Permit Limits for the Rockaway Beach WWTP

Parameter	Avg. Effluent Concentrations, mg/L		Mass Discharges, pounds per day		
	Monthly	Weekly	Monthly Average	Weekly Average	Daily Maximum
May 1 to October 31					
5-day Biochemical Oxygen Demand (BOD_5)	30	45	130	200	260
Total Suspended Solids (TSS)	30	45	130	200	260
November 1 to April 30					
BOD_5	30	45	160	240	330
TSS	30	45	160	240	330
Other Parameters (year-round)	Limitations				
pH	6.0-9.0				
BOD_5 and TSS removal efficiency	85 percent, monthly average (year-round)				
Enterococci Bacteria	Shall not exceed 35 organisms per 100 milliliters (mL) monthly geometric mean.				
Total Chlorine Residual	Shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum concentration of 0.05 mg/L.				
Summer mass load limits are based on the average dry-weather design flow of 0.5 million gallons per day (mgd).					
Winter mass load limits are based on an average wet-weather flow (AWWF) of 0.65 mgd.					

EQUIPMENT AND UNIT PROCESS RELIABILITY

The Rockaway Beach WWTP could fall into Reliability Class I or II as defined by the EPA, depending on the beneficial uses of the ocean in the area affected by the plant effluent. Outfall modeling
studies have been performed to determine distribution and dilution of the effluent plume. Since the effluent plume is not near any shell fishing areas, the plant is considered to be Class II and appropriate requirements will apply. DEQ has confirmed that Class II requirements will apply to the ocean
outfall. The requirements for Class I and Class II reliability classes are summarized in Table 2-7.

Table 2-7. EPA Treatment Plant Reliability Requirements

Component	Class I Requirements	Class II Requirements
Pumps	One backup pump. With largest pump out of service, the remaining pumps can handle the peak flow.	Same as Class I.
Mechanically cleaned bar screen	One backup manually cleaned bar screen.	Same as Class I.
Primary sedimentation	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.	Same as Class I.
Secondary clarifiers	With the largest unit out of service, the capacity of the remaining units to be at least 75 percent of the total design flow to the process.	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.
Trickling filters	With the largest unit out of service, the capacity of the remaining units to be at least 75 percent of the total design flow to the process.	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.
Aeration basins	At least two equal-volume basins shall be provided.	Same as Class I.
Aeration equipment	With the largest unit out of service, the design oxygen transfer to be maintained.	Same as Class I.
Disinfection	With the largest basin out of service, the capacity of the remaining units to be at least 50 percent of the design flow to the process.	Same as Class I.
Digesters	At least two tanks to be provided.	Same as Class I.
Electric Power	A backup source required. Sufficient to operate all vital components during peak flow conditions, together with critical lighting and ventilation.	Similar to Class I, except secondary process components such aeration need be operable at full levels but shall maintain biota.

BIOSOLIDS MANAGEMENT

The City currently disposes dewatered, dried biosolids at an approved municipal solid waste (MSW) landfill. Because the biosolids are disposed at a MSW landfill, they are regulated by EPA under Subpart I of 40 CFR, Part 258, Criteria for Municipal Solid Waste Landfills. Oregon Administrative Rules (OAR) Chapter 340, Divisions 93 through 97 also govern landfills in the state of Oregon and generally refer to federal rules. Standards set forth in the Part 258 Regulations address general requirements, pollutant limits, management practices, operational standards for pathogens and vector attraction, and monitoring, record keeping, and reporting requirements. Accepting biosolids at a MSW landfill generally does not add significant regulatory hurdles or permit constraints to the landfill operator, nor does it result in additional operational requirements for the landfill other than mixing the biosolids and solid waste prior to placement in the permanent cell.

All solids disposed in MSW landfills must pass the paint-filter liquids test (dewatering biosolids to about 20 percent solids or more will generally meet this goal) due to the regulatory prohibition of materials containing free liquids. The paint filter test is described in detail in EPA publication SW-846, Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Method 9095A.

Furthermore, the biosolids cannot contain hazardous substances as defined per 40 CFR Part 261 and Part 761 -polychlorinated biphenyls (PCB's). OAR Chapter 340, Division 101 also defines hazardous waste in the state of Oregon. Currently, the City tests the biosolids for heavy metals and volatile/ semi-volatile organic compounds (EPA Methods 8260/8270).

EXISTING WASTEWATER FLOWS

Wastewater flows vary based on time of day and the season of the year. Daily monitoring reports (DMRs) from the City's wastewater treatment plant (WWTP) were analyzed from January 2007 through December 2011 to determine average and maximum observed flows. The flow measurements were taken on the effluent side of the treatment plant since there is no flow meter on the influent side. The flows are shown in Figure 2-2 and a summary of observed flows at the Rockaway WWTP are listed in Table 2-8. These flows will be compared to projected flows using the Department of Environmental *Quality's Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon*, which is discussed later in this section, and the more conservative value will be used.

Wastewater characteristics and receiving waters characteristics vary seasonally. The two "seasons" used in this study are as follows:

Dry-Weather Period: Defined as the period when the precipitation and streamflows are low. This period is defined in the Oregon Administrative Rules (OAR 340-41-215) as May 1 through October 31.

Wet-Weather Period: Defined as the period when streamflows and rainfall are high. This period is defined in OAR 340-41-215 as November 1 through April 30.

Flow Parameters and Values:

Average Daily Flow (ADF): The average daily flow value is calculated by taking the sum of the daily flows and dividing it by the number of days for the monitoring period. For the period from January 2007 to December 2011, the average daily flow at the Rockaway wastewater treatment plant was 0.23 million gallons per day (MGD). The ADF was also calculated for each year in the flow monitoring period to look at trends. The ADF was 0.29 MGD in 2007 and steadily declined to 0.17 MGD in 2011 indicating a decrease in population for the sewer service area.

Average Daily Dry-Weather Flow (ADDWF): The average daily dry-weather flow value is calculated by the sum of the daily flows for the dry-weather period (May 1 through October 31) divided by the number of days in the period. This flow value has little-to-no infiltration and inflow (I&I), and for practical purposes is considered the actual customer usage. For the data period examined, the ADDWF was 0.18 MGD.

Maximum-Monthly Dry-Weather Flow (MMDWF): The maximum-monthly dry-weather flow value is the highest average flow value calculated over a 30-day consecutive period during the dry-weather season. The observed maximum monthly dry-weather flow for the analysis was 0.28 MGD, which was observed in July 2007 with a corresponding monthly total rainfall of 2.55 inches.

Average Daily Wet-Weather Flow (ADWWF): The average daily wet-weather flow value is calculated by the sum of the daily flows for the wet-weather period (November 1 through April 30) divided by the number of days in the period. For the data period examined, the ADWWF was 0.28 MGD.

Maximum-Monthly Wet-Weather Flow (MMWWF): The maximum-monthly wet-weather flow value is the highest average flow value calculated over a 30-day consecutive period during the wet-weather season. The observed maximum monthly wet-weather flow for the analysis was 0.47 MGD, which was observed in March 2007 with a corresponding monthly total rainfall of 19.48 inches.

Peak Weekly Flow (PWF): The peak weekly flow value is the highest average flow value calculated over a consecutive 7-day period. For the period examined, the PWF was 0.62 MGD, which occurred the week of January 8, 2009.

Peak Daily Average Flow (PDAF): The peak daily average flow is the highest daily wastewater flow value during the flow period. The peak daily flow observed in the data set was 1.17 MGD. This occurred on January 8, 2009, which had a total rainfall of 3.2 inches for the day.

Peak Hourly Flow or Peak Instantaneous Flow (PHF or PIF): The peak hourly flow is the highest flow rate that occurs over a one-hour period. This flow rate was not available from the DMR data.

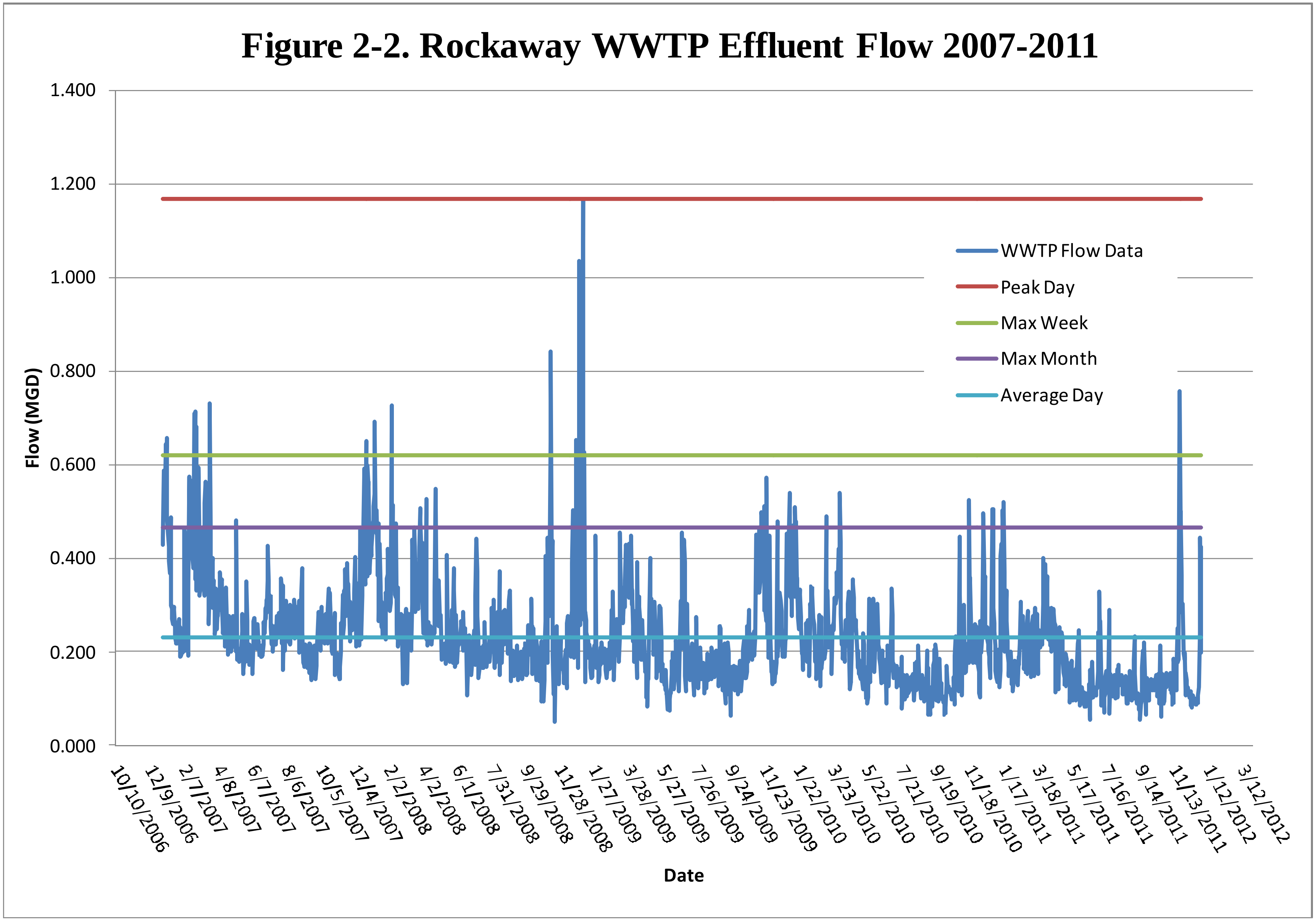


Figure 2-2. Rockaway WWTP Effluent Flow 2007-2011
Flow (MGD)
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
WWTP Flow Data
Peak Day
Max Week
Max Month
Average Day
10/10/2006
12/9/2006
2/7/2007
4/8/2007
6/7/2007
8/6/2007
10/5/2007
12/4/2007
2/2/2008
4/2/2008
6/1/2008
7/31/2008
9/29/2008
11/28/2008
1/27/2009
3/28/2009
5/27/2009
7/26/2009
9/24/2009
11/23/2009
1/22/2010
3/23/2010
5/22/2010
7/21/2010
9/19/2010
11/18/2010
1/17/2011
3/18/2011
5/17/2011
7/16/2011
9/14/2011
11/13/2011
1/12/2012
3/12/2012
Date

Table 2-8. Summary of Existing WWTP Effluent Flows from DMR Analysis

Flow Condition	Observed Wastewater Flows (MGD)					Total Period (MGD)
	2007	2008	2009	2010	2011	
Average Day	0.29	0.26	0.23	0.21	0.17	**0.23**
Dry Weather						
Average Day	0.24	0.21	0.18	0.16	0.13	**0.18**
Max. Month	0.28	0.24	0.22	0.21	0.16	**0.28**
Wet Weather						
Average Day	0.35	0.31	0.27	0.27	0.21	**0.28**
Max. Month	0.47	0.38	0.35	0.36	0.26	**0.47**
Max. Week	0.57	0.53	0.62	0.44	0.42	**0.62**
Peak Day	0.73	0.84	1.17	0.54	0.76	**1.17**

Wastewater Flows Calculated By Statistical Method

The following information discusses the method used for calculating wastewater flow rates for the Rockaway Beach wastewater treatment plant (WWTP) using the "rainfall method" as outlined in DEQ document *Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF*. This method will form a baseline for comparison with the flow values in Table 2-8., and it will also be used to project the peak instantaneous flow to the WWTP.

The rainfall method incorporates rainfall statistics for determining peak monthly and daily flows that have specific recurrence intervals. The flows that are determined using the method are the 10-year maximum-month dry-weather flow ($MMDWF_{10}$), the 5-year maximum-month wet-weather flow ($MMWWF_5$), the 5-year peak daily average flow ($PDAF_5$), and the peak instantaneous flow (PIF_5), or peak hourly flow.

The following describes the methods for calculating the $MMDWF_{10}$ $MMWWF_5$, $PDAF_5$, and PIF_5, per the DEQ guidelines.

<u>Maximum Monthly Dry and Wet Weather Flows</u>

The $MMDWF_{10}$ has a 10 percent probability of occurring in any given year. In western Oregon, it almost invariably occurs in the month of May. The $MMDWF_{10}$ is the flow that corresponds to the 90 percentile rainfall accumulation during May.

.

The $MMWWF_5$ has a 20 percent probability of occurring in any given year and represents the highest monthly average flow attained during periods of high groundwater. Groundwater levels are usually high enough by January to produce a consistent infiltration and inflow (I&I) response. Therefore, the 5-year rainfall accumulation for January is used to estimate the $MMWWF_5$.

Table 2-9 lists the average monthly flows to the City's WWTP and the corresponding monthly rainfall total for the months, which will be the data points used to determine the $MMDWF_{10}$ and $MMWWF_{5.}$ Data for the two most recent years were used.

Table 2-9. Data Points Used in $MMDWF_{10}$ & $MMWWF_5$ Analysis

Date	Total Rainfall -WWTP	Avg. Flows (mgd)
Jan-10	17.28	0.353
Feb-10	10.38	0.238
Mar-10	9.05	0.249
Apr-10	12.23	0.271
May-10	5	0.177
Feb-11	5.88	0.183
Mar-11	12.47	0.244
Apr-11	10.61	0.229
May-11	5.02	0.153

Monthly rainfall totals were plotted against the average daily flows for the month to determine a statistical relationship (Figure 2-3). The data point for January 2011, which indicated an average flow of 0.245 mgd and a total rainfall for the month of 16.03 inches was not included as a data point since it was not considered a reliable data point. Rainfall data for 2010 was used to provide additional data points and are considered reliable. A computed linear statistical relationship yielded the equation:

$$\text{Average Monthly Flow (MGD)} = 0.0141\ X + 0.0956 \qquad \text{(Equation 1)}$$
$$X = \text{Monthly Rainfall Total (inches)}$$

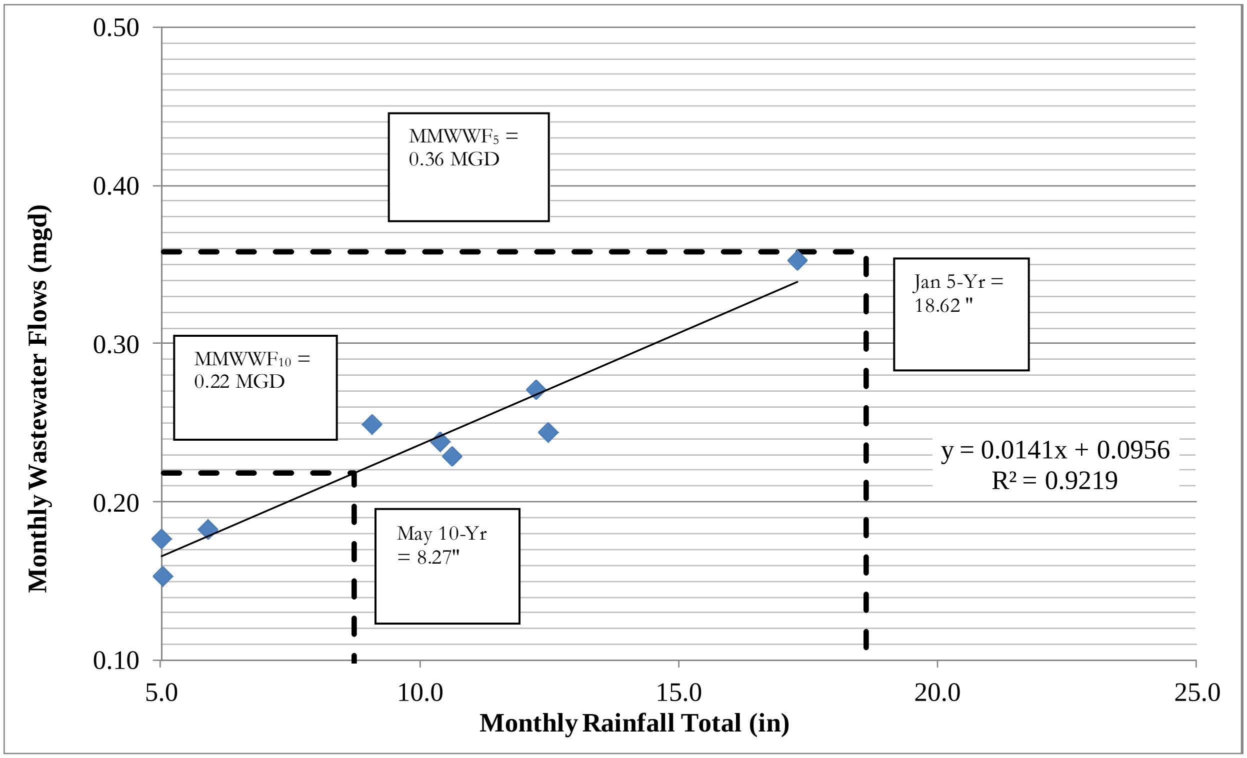
The $MMDWF_{10}$ is determined by choosing the flow that corresponds to the 10-year (or 90 percentile) May rainfall accumulation. The $MMWWF_5$ corresponds to the 5-year (or 80 percentile) January rainfall accumulation. The 5-year January rainfall accumulation was found to be 18.62 inches and the 10-year May rainfall accumulation was found to be 8.72 inches[2]. Incorporating these values into Equation 1 results in the following:

- $MMWWF_5$ equals 0.36 mgd
- $MMDWF_{10}$ equals 0.22 mgd.

The observed MMWWF from the DMR analysis was 0.47 MGD, which occurred in March 2007 with a total monthly rainfall of 19.48. The observed MMDWF was 0.28 MG, which occurred in July 2007 with a corresponding rainfall of 2.55 inches. The fact that the observed MMDWF of 0.28 MGD with 2.55 inches of rain from the DMR's was so much higher than the $MMDWF_{10}$ of 0.22 MGD with 8.72 inches of rain from the rainfall method may be indicative of the influence the seasonal population has on the flows during the dry-weather period.

[2] National Oceanic & Atmospheric Administration (NOAA), Climatography of the United States No. 20 (1971-2000), Station: Tillamook 1 W Sta. (358494).

Figure 2-3. Plant Flow versus Rainfall
(for $MMWWF_5$ & $MMDWF_{10}$)



Peak Daily Average Flow

The $PDAF_5$ almost always corresponds to the 5-year storm during a period of high groundwater. It can be estimated from plant data if a 5-year storm was recently experienced during a period of high groundwater (January through May) or it can be estimated from a plot of daily plant flows versus daily storm rainfall accumulation. This requires some interpretation of past rainfall records. Records should be used only if the antecedent weather was wet and groundwater levels were high. The following data points were used to develop a plot of flow versus rainfall to project the flow for the 5-year peak day storm event.

Table 2-10. Data Points Used in $PDAF_5$ Analysis

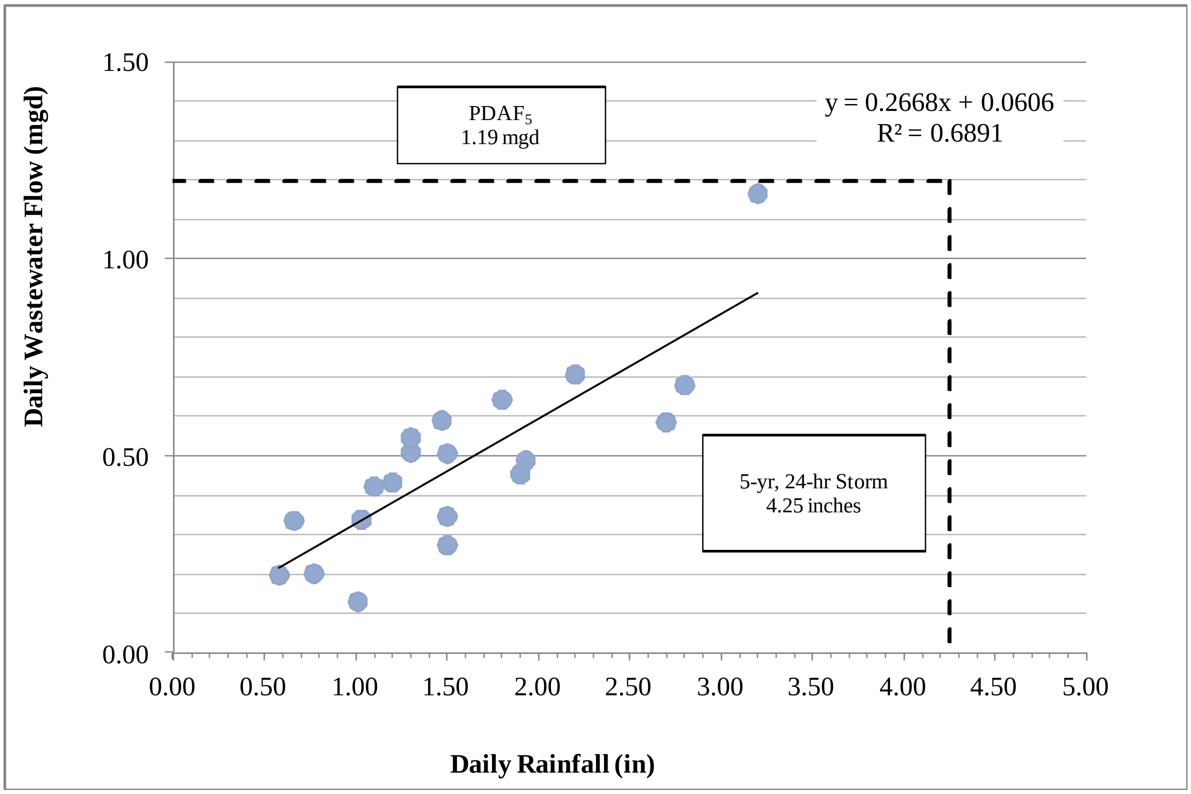
Date	Rainfall (in)	Daily Flows (mgd)
1/3/2007	2.70	0.588
1/6/2007	1.80	0.645
2/25/2007	2.20	0.709
3/1/2007	2.80	0.682
3/3/2007	1.47	0.592
4/22/2007	0.66	0.338
1/3/2008	1.10	0.425
1/7/2008	1.30	0.512
3/8/2008	0.77	0.204
4/24/2008	1.30	0.549
1/8/2009	3.20	1.168
1/27/2009	0.58	0.200
3/15/2009	1.90	0.456
1/16/2010	1.50	0.509
2/14/2010	1.03	0.341
3/12/2010	1.93	0.491
4/6/2010	1.20	0.435
2/12/2011	1.01	0.133
3/10/2011	1.50	0.276
3/30/2011	1.50	0.349

The storm events listed in Table 2-10 and the corresponding WWTP flows were plotted in Figure 2-4. The statistical relationship between daily rainfall and WWTP flows was established using linear regression, which yielded the following equation:

$$\text{Daily Plant Flow (MGD)} = 0.2668\ X + 0.0606 \qquad \text{Equation 2}$$
$$X = \text{24-Hour Rainfall Total (inches)}$$

According to the isopluvial map in the *NOAA Atlas 2, Volume X*, Figure 26, the 5-year, 24-hour storm for Rockaway is approximately 4.25 inches. Inserting this value into Equation 2 results in a calculated $PDAF_5$ of 1.19 MGD comparing well with the observed PDAF of 1.17 MGD, which occurred on January 8, 2009 with 3.2 inches of rain for the day.

Figure 2-4. Plant Flow versus Rainfall
(for $PDAF_5$)



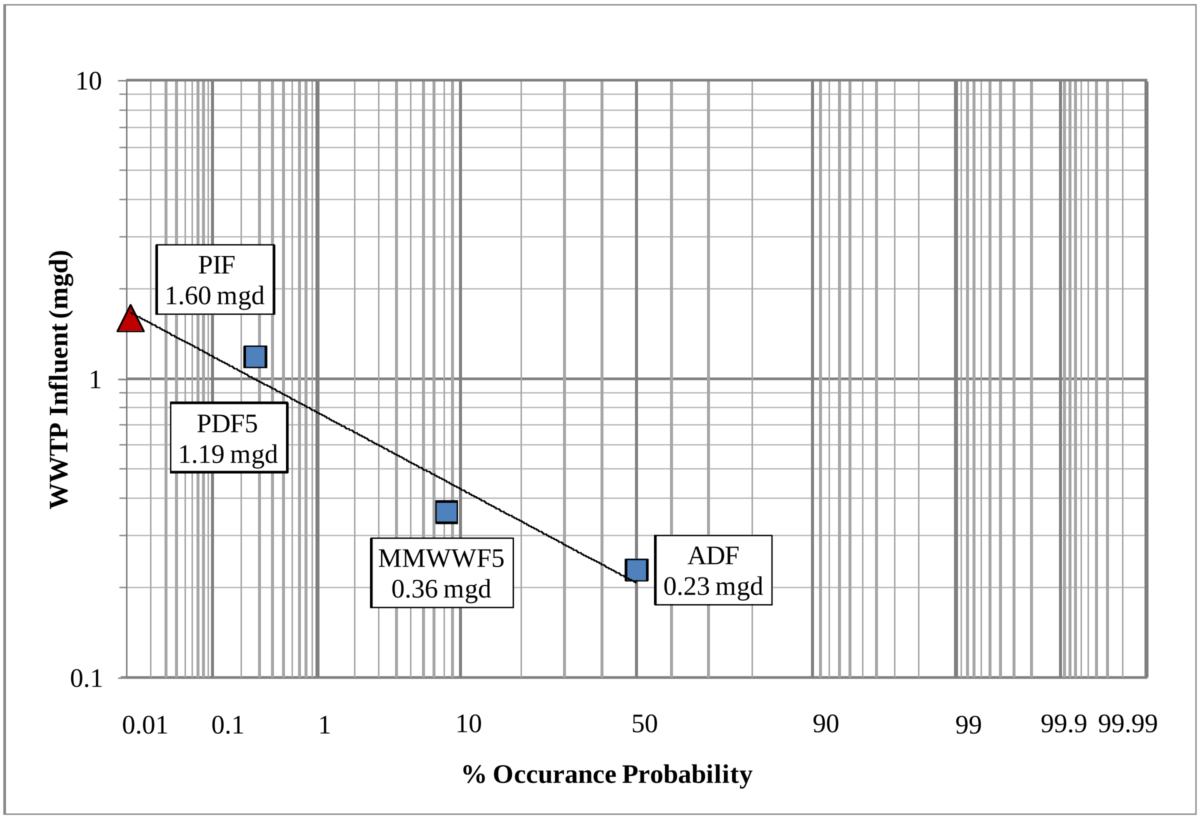
Peak Instantaneous Flow

The PIF_5 can be determined by applying a suitable peaking factor, or by extrapolation. The extrapolation method was used in this case. This method involves plotting the average daily flow (ADF), $MMWWF_5$, and $PDAF_5$ on log-probability graph paper. Each flow is plotted versus the percent probability of exceeding those flows. This follows from the assumption that the $MMWWF_5$, $PDAF_5$, and PIF_5 will all occur in the same year. This assumption yields the following probabilities of occurrence:

- The ADF is likely to occur 6/12 of the time or 50% probability.
- The $MMWWF_5$ occurs 1/12 of the time or 8.3% probability.
- The $PADF_5$ occurs once in 365 days or 0.27% probability.
- The PIF_5 occurs once in 8,760 hours or 0.011% probability.

The $MMWWF_5$ and $PDAF_5$ were already determined as previously discussed. The ADF was determined previously on page 2-12 and is 0.23 mgd. Figure 2-5 shows the probability plot. To determine the PIF_5, a straight line is drawn through the three points and the PIF_5 is the flow that intersects the line at a probability of exceedance of 0.011 percent, or once in 8,760 hours. As shown in Figure 2-5, the PIF_5 is estimated to be 1.60 mgd.

Figure 2-5. Determination of PIF



Summary of Flows

Table 2-11 summarizes the various flows that were estimated using both the Department of Environmental Quality (DEQ) rainfall method and by analysis of plant flow data. The rainfall method was used to establish a baseline for each flow parameter and was compared to the analysis of the wastewater treatment plant daily monitoring reports (DMR's). The larger value between the DMR analysis and the DEQ rainfall method will be used.

Table 2-11. Summary of Existing Flows

Flow Condition	WWTP Flow (MGD)	Peak Factor from ADF	Basis
Dry Flow Conditions			
ADDWF	**0.18**	**1.0**	Analysis of 2007-2011 DMRs
$MMDWF_{10}$	0.22	1.2	DEQ Rainfall Method
MMDWF	**0.28**	**1.6**	Analysis of 2007-2011 DMRs
Average Daily Flow Condition			
ADF	**0.23**	**1.3**	Analysis of 2007-2011 DMRs
Wet Flow Conditions			
ADWWF	**0.28**	**1.6**	Analysis of 2007-2011 DMRs
$MMWWF_5$	0.36	2.0	DEQ Rainfall Method
MMWWF	**0.47**	**2.6**	Analysis of 2007-2011 DMRs
PWF	**0.62**	**3.4**	Analysis of 2007-2011 DMRs
$PDAF_5$	**1.19**	**6.6**	DEQ Rainfall Method
PDAF	1.17	6.5	Analysis of 2007-2011 DMRs
PIF_5	**1.60**	**8.9**	DEQ Rainfall Method

Current Unit Flows

Using the information in Table 2-11 above and the population information for the current sewer service area presented earlier in this chapter, estimated unit flows were developed which will be used to help develop unit flows for future service areas.

Rockaway Beach sewer usage billing data was analyzed for 2011 and 2012 to determine the percent contribution of residential and commercial users to the sewer system. Using the information it was determined that residential costumers used approximately 80% of the water, while commercial customers used approximately 20%. These percentages were then applied to the current sewer flows to estimate the residential and commercial flows for the annual daily dry weather flow (ADDWF) which is considered the actual sewage flow with little-to-no infiltration and inflow (I&I). The I&I for the maximum month, peak day and peak hour flows were divided between the residential and commercial properties using percentages of 90 and 10, respectively, to account for the higher percentage of sewer piping serving residential areas. Table 2-12 summarizes existing wastewater flows based on residential and commercial land use for Rockaway Beach.

Table 2-12. Summary of Existing Flows and Unit Flows by Land Use

	Total	Residential	Commercial		Unit Flows	
	Flow (MGD)	Flow (MGD)	Flow (MGD)	Capita (gpcd)	EDU (GPD/EDU)	Commercial (GPD/ac)
Dry Flow Conditions						
ADDWF	0.18	0.14	0.04	47	93	500
MMDWF	0.28	0.23	0.05	76	150	600
Average Flow Condition						
ADF	0.23	0.19	0.04	62	122	500
Wet Flow Conditions						
ADWWF	0.28	0.23	0.05	76	150	600
MMWWF	0.47	0.41	0.07	132	260	900
$PDAF_5$	1.19	1.05	0.14	344	677	1,800
PIF_5	1.60	1.42	0.18	464	915	2,300

Using an average estimated sewer service population of 3,063 from 2007 to 2011, which corresponds with the years for the flow data, the per capita base flow (ADDWF) is 47 gallons per capita per day (gpcd), which includes seasonal residents. This rate is below typical wastewater flow rates which range from 63-81 gpcd[3]. The reason for the low per capita flow rate is likely attributable to the high percentage of seasonal population in the sewer service area. Seasonal residents likely would not occupy their homes during the entire dry-weather period for the ADWWF, and therefore are over-accounted for in the sewer service population number. If it is assumed, however, that the seasonal residents occupy their homes for 4 months out of the 6 month dry-weather period, that would equate to an adjusted ADDWF per capita of 60 gpcd.

The per acre average day flow for commercial properties, which includes normal infiltration is 500 gallons per day per acre (GPD/ac). This value is below the typical 800-1,500 GPD/ac for commercial properties[4].

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[3] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003. Table 3-1.
[4] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, pg. 156.

FUTURE FLOW PROJECTIONS

Wastewater flows were projected for the 2017 (5-year) and the 2032 (20-year) design years. Flows for the average daily dry-weather flow (ADDWF) were projected based on the assumption that it will increase at the same rate as population growth. However, the unit values for other flow conditions (i.e. average daily wet weather flow (ADWWF), max. monthly wet weather flow (MMWWF), peak daily average flow (PDAF), and peak instantaneous flow (PIF)) determined in Table 2-12 included infiltration and inflow (I&I) and are not likely to increase at the same rate as population growth.

For the projection of wastewater flow parameters with significant influence from I&I (ADWWF, MMWWF, PDAF, and PIF), the use of existing unit flows would yield results significantly higher than reality. This is because new construction techniques and materials result in sanitary sewers which have much lower quantities of I&I than the existing system. Therefore additional I/I due to future growth was determined with separate criteria.

The future service population per capita average unit flow for residential areas was obtained by using the adjusted per capita ADDWF of 60 gpcd as explained on the previous page. Assuming a normal infiltration component of 25 gpcd which is a little lower than difference between the existing per capita ADDWF and the ADWWF (29 gpcd) results in a total average unit rate flow of 85 gpcd. For a reference point , the EPA historical unit flow average is 70 gpcd for sewer flows and 40 gpcd[5] for infiltration for a total of 110 gpcd.

The future service area per acre average unit flow rate for commercial properties was obtained by using the 500 GPD/ac average sewer flow as explained on the previous page which includes normal infiltration.

Peak day values were obtained by using a peaking factor equal to 225%[6] of the ADDWF . Peak hour flow values are based on peaking factors obtained from Criteria for Sewage Works Design prepared by the Washington State Department of Ecology, dated August 2008 as presented on the next page. A peaking factor of 4 was assumed for all future service areas.

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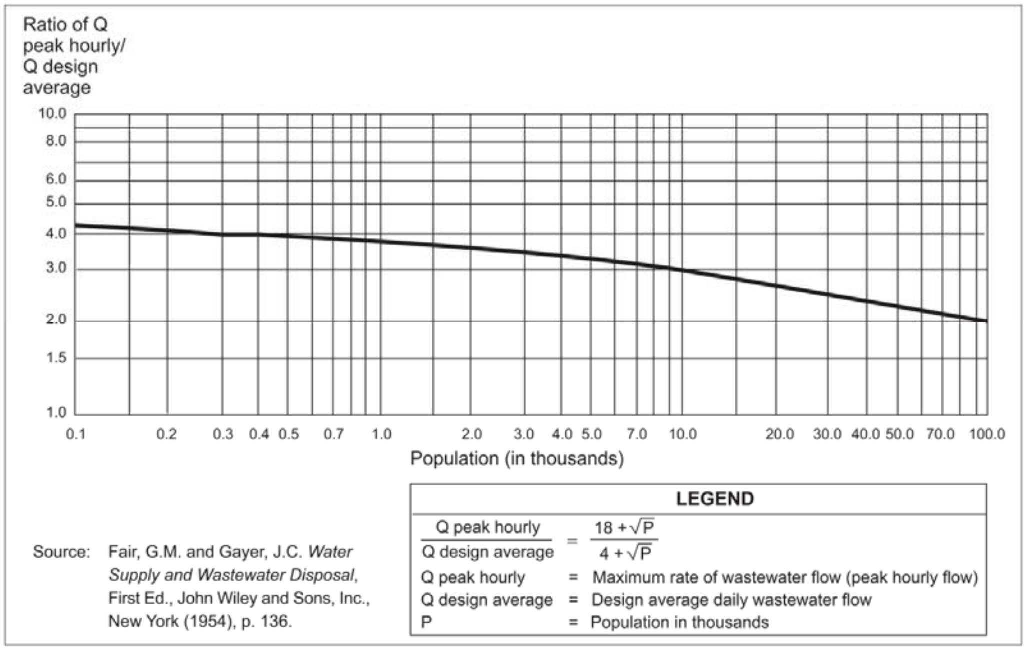
[5] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, pg. 200.

[6] *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1998, (Table 4.8).

Figure 2-6 – Ratio of Peak Hour Flow to Average Flow Based on Population



The use of the per capita and per acre unit flows and the peaking factors is intended to cover normal I&I for new construction. Table 2-13 presents the unit values to use for future service areas.

Table 2-13 – Flow Units for Future Service Areas

Item	Average Unit Flow[1]	Peak Day Unit Flow	Peak Hour Unit Flow	Units
Peak Factor	1	2.25	4.00	
Residential	85	190	340	gpcd
Commercial	500	1,100	2,000	GPD/ac

[1]Includes normal I&I.

The peak hour unit flow for future residential construction is lower than the existing peak hour unit flow in Table 2-12, and is therefore consistent with the premise that lower I&I will be seen with new construction. Also, the peak hour per capita residential flow is consistent with Rockaway Beach's sewer design standards. The future peak-hour commercial unit flow of 2,000 GPD/ac is lower than the existing peak-hour unit flow value of 2,300 GPD/ac.

Based on the unit flow values in Table 2-13 and the population and land use projections presented earlier in this chapter, the following flow projections were determined for the 2017 and 2032 conditions. It was assumed that the I&I for the existing piping will remain unchanged during the planning periods.

Table 2-14 – Flow Projections

Projected Flows (MGD)			
	ADF	**PDAF**	**PIF**
2012	0.23	1.19	1.60
2017	0.27	1.27	1.75
2032	0.46	1.70	2.50

CURRENT WASTEWATER LOADS

A detailed analysis of the City's DMRs from January 2007 to December 2011 was conducted to aid in establishing a basis for long-term projections of organic loadings and wastewater composition for the planning period. This information will be utilized in selecting and sizing treatment technologies to remove the unwanted wastewater components in order for the City to meet the requirements of its discharge permit.

Terminology

Biochemical Oxygen Demand, 5-day (BOD_5): Measure of the concentration of organic impurities in wastewater. The amount of oxygen required by bacteria while stabilizing organic matter under aerobic conditions, expressed in milligrams per liter, is determined entirely by the availability of material in the wastewater to be used as biological food and by the amount of oxygen utilized by the microorganisms during oxidation. The standard length of the BOD test is 5 days.

Total Suspended Solids (TSS): Solids that float on the surface of, or are in suspension in, water, wastewater, or other liquids, and that are largely removable by laboratory filtering.

The BOD and TSS loads at a treatment plant affect the following factors:

- Secondary process sizing. The design of a secondary process is based on the BOD load.
- Aeration system design. The peak BOD load determines the capacity of the aeration system.
- Biosolids production. BOD and TSS removed by the plant are converted into biosolids that must be stabilized and recycled.
- Solids treatment and handling system design. Solids handling facilities, such as digesters and thickeners, must be sized to accommodate expected biosolids quantities.

Analysis of Plant Records

Analysis of the Rockaway Beach's WWTP DMRs from January 2007 to December 2011 identifies a number of parameters which characterize the City's wastewater. Plant records include measurement of influent BOD_5 and TSS taken twice per week. Influent loading and strength with regard to BOD and TSS are shown on an average monthly basis in Figure 2-7Figure -. These values typically increase during summer and decrease during winter.

Figure - 2-7
Average Monthly Loading & Strength for BOD & TSS

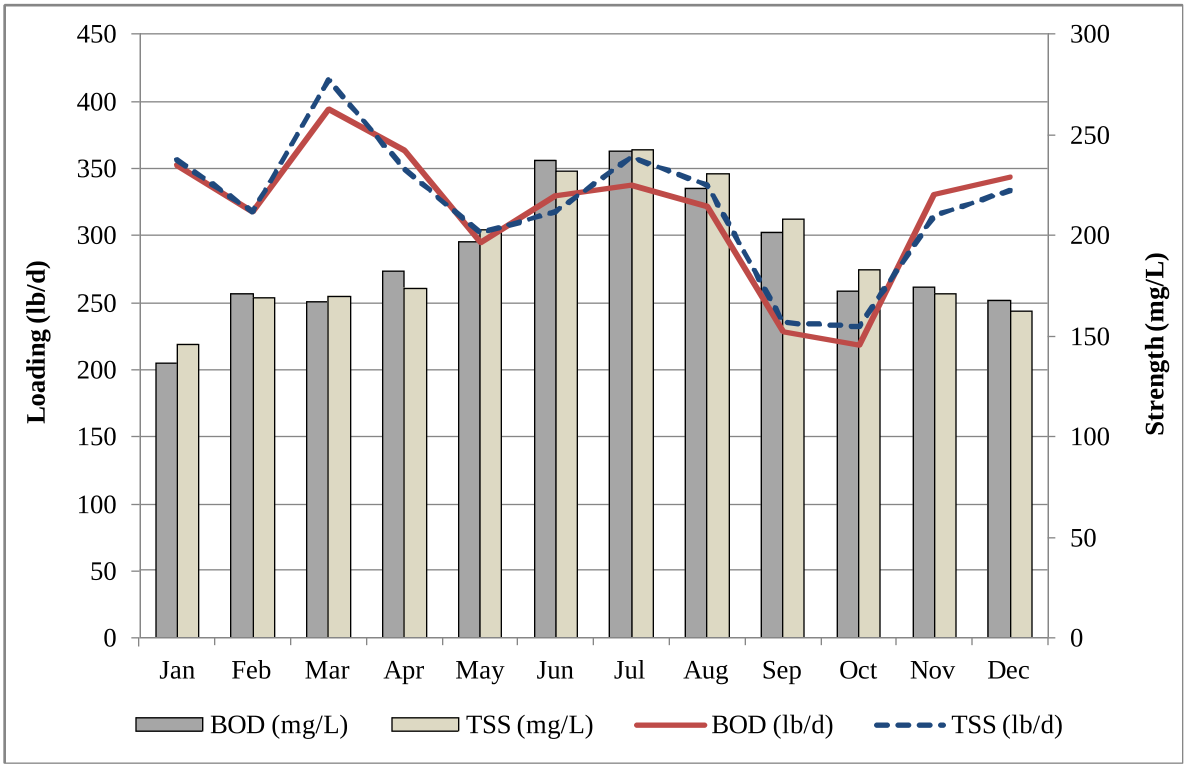


Figure 2-7 shows that the concentrations for BOD and TSS follow the typical pattern of higher concentrations for dry-weather and lower concentrations for wet-weather. However, the monthly average loadings (lb/d) do not follow that pattern. The data actually shows the highest loadings in March, which is likely due to spring break visitors. Then there is another rise in the loadings in November/ December which may be due to "first flush" of sediments and other contaminants into the collection system from the start of the wet-weather season.

Tables 2-15 and 2-16 present the average annual, winter, and summer influent BOD and TSS concentration and loads discharging to the City's WWTP. These tables include information on maximum monthly and daily loadings. It should be noted that the BOD and TSS analysis revealed several outliers for the loading (lb/d) that were ignored. A plot of BOD and TSS loadings shows the outliers (Figure 2-8).

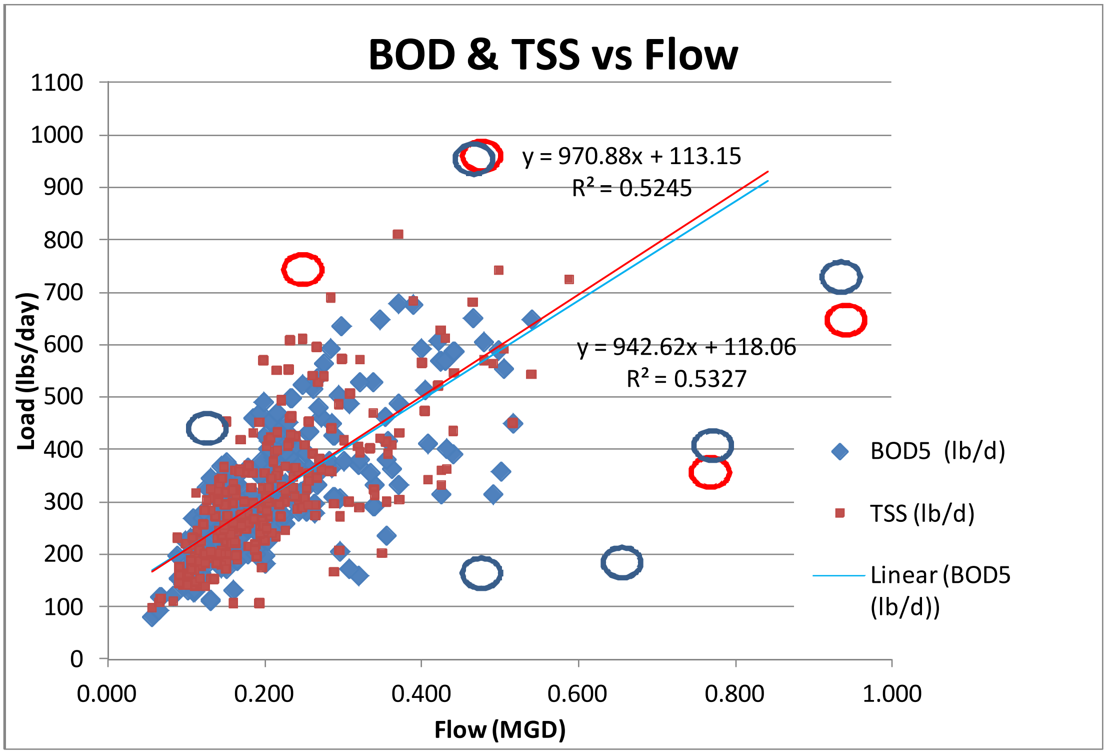
Table 2-15
Existing Influent BOD5 Concentrations and Loads

	BOD (mg/L)						BOD (lb/d)					
	2007	**2008**	**2009**	**2010**	**2011**	**Total Period**	**2007**	**2008**	**2009**	**2010**	**2011**	**Total Period**
Average	174	172	189	204	208	**190**	349	325	320	335	268	**320**
Summer Average	211	198	216	211	224	**210**	356	315	289	249	233	**290**
Winter Average	136	146	163	196	191	**170**	343	335	352	421	302	**350**
Maximum Month	265	217	273	272	286	**290**	480	449	420	527	403	**530**
Maximum Day	299	256	299	318	478	**480**	678	652	607	679	537	**680**

Table 2-16
Existing Influent TSS Concentrations and Loads

	TSS (mg/L)						TSS (lb/d)					
	2007	**2008**	**2009**	**2010**	**2011**	**Total Period**	**2007**	**2008**	**2009**	**2010**	**2011**	**Total Period**
Average	193	175	189	196	201	**190**	387	333	316	321	256	**320**
Summer Average	226	204	222	211	219	**220**	386	323	295	248	235	**300**
Winter Average	159	147	156	181	183	**170**	389	343	336	395	277	**350**
Maximum Month	303	228	270	264	252	**300**	580	468	437	521	367	**580**
Maximum Day	393	260	298	298	341	**390**	726	683	743	812	597	**810**

Figure - 2-8
Plot of BOD & TSS versus Flow to Determine Outliers



Typical concentrations for contaminants in untreated domestic wastewater are identified in the text, Wastewater Engineering Treatment and Reuse 4th Edition (Metcalf & Eddy, 2003). Data given in the referenced text is summarized in the following table for comparison to the average BOD/TSS concentrations measured at Rockaway Beach.

Table 2-17
Typical Composition of Untreated Domestic Wastewater

Contaminant	**Concentration (mg/L)**		
Typical Domestic Wastewater[1]	*Low Strength*	*Medium Strength*	*High Strength*
Biochemical oxygen Demand, 5-d, 20°C, (BOD_5)	110	190	350
Total Suspended Solids (TSS)	120	210	400
Rockaway Beach Wastewater	Average Winter	Average Annual	Average Summer
BOD Range	170	190	210
TSS Range	170	190	220

[1] Derived from Table 3-15, "Wastewater Engineering", Metcalf & Eddy, 2003

Existing Unit Loads

The per capita BOD and TSS loadings are presented in Tables 2-18 and 2-19 below, respectively.

Table 2-18
BOD Load Values and Unit Values

BOD	WWTP (lb/day)	lb/cap-d	Date
Average	**320**	**0.10**	-
Maximum Month	**530**	**0.17**	Mar-10
Maximum Day	**680**	**0.22**	21-Nov-07

Table 2-19
TSS Load Values and Unit Values

TSS	WWTP (lb/day)	lb/cap-d	Date
Average	**320**	**0.10**	-
Maximum Month	**580**	**0.19**	Jul-07
Maximum Day	**810**	**0.26**	31-Mar-10

Using an average estimated sewer service population of 3,063 from 2007 to 2011, which corresponds with the years for the DMR data, the average per capita loading is 0.10 pounds per capita per day (lb/cap-d) for BOD and TSS each, which includes seasonal residents. This rate is below typical wastewater loading rates which range 0.11-0.26 lb/cap-d for BOD and 0.13-0.33 lb/cap-d for TSS on a dry weight basis[7]. The reason for the low per capita loading rate is likely attributable to the high percentage of seasonal population in the sewer service area.

[7] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 3-12.

WASTEWATER LOAD PROJECTIONS

Based on the existing unit load values in Tables 2-18 and 2-19 and the population projections presented earlier in this chapter, the following load projections were determined for the 2017 and 2032 conditions. It was assumed that the loads would increase in proportion to the sewer service population.

Table 2-20 – BOD Load Projections

Projected BOD (lb/day)			
	Average	**Max Month**	**Peak Day**
2012	320	530	680
2017	366	606	777
2032	592	981	1,259

Table 2-21 – TSS Load Projections

Projected TSS (lb/day)			
	Average	**Max Month**	**Peak Day**
2012	320	580	810
2017	366	663	926
2032	592	1,074	1,499

CHAPTER 3

Existing Wastewater Collection System

CHAPTER 3 EXISTING WASTEWATER COLLECTION SYSTEM

This chapter describes the wastewater collection and pumping system owned and operated by the City of Rockaway Beach (City).

WASTEWATER COLLECTION SYSTEM

The original sewer system in Rockaway Beach was constructed in 1954, and these sewers served virtually all of the property west of Highway 101 from North 9th Street to Alder Street and about one third of the currently sewered area east of the highway. These sewers were constructed using asbestos cement (Transite) pipe. The original sewer system in Manhattan Beach area was constructed in 1965, and served all areas adjacent to Highway 101, from North Ninth Avenue to North 23rd Avenue. These sewers were also constructed of Transite pipe. The Nedonna Beach area is not sewered except for the White Dove Estates, which was annexed to the City in the 90's. Two major expansions to the 1954 sewer system occurred in 1979 and 1981. These expansions were the first to use polyvinyl chloride (PVC) pipe. The sewers were extended east from Easy Street. The sewers in the Nedonna Beach area were constructed in 1997 with PVC pipe. Most, if not all, of the sewer extensions to the original Rockaway Beach and Manhattan Beach systems have been constructed using PVC pipe.

The only reported overflows occured in the discharge manholes for the Lake Lytle and NW 17th lift stations. These overflows occur only when s backup pump is operating with the duty pump during large storm events. There are no other known routine overflow points from the sewer system that occur during peak flow periods. Overflows from the sanitary sewer system occur only if there is a random sewer blockage or other maintenance problem in a sewer. The capacity of the sewers downstream of the pumping stations is of concern as the population continues to increase.

As part of the City's response to the Oregon Department of Environmental Quality requirement to identify sources of infiltration and inflow (I/I) in the sewer system, thousands of feet of sewer lines have been internally inspected using video equipment in 1995, 2000 and each year since 2007. These inspections revealed a number of lines which have sags in them. The most significant areas with sags are South Second Street between Coral Street and Easy Street and Nehalem Street between Beacon Street and Dolphin Street. Other areas with sags include Miller St. between NW 9th and 11th, a sunken manhole causing a belly on Harbor St. between S 2nd Ave. and Nehalem, and Breaker Ave. between S. 8th and Alder. Also full piping was noted upstream of the Main Pump Station on Anchor between S 3rd and 4th, and upstream of the S 5th and 6th Ave. lift stations.

The inspections also revealed sewers that had structural defects or holes in them, displaced joints, a separated service lateral connection, and a telephone utility line through the sewer on Easy St. between S 2nd and Nehalem. The structural problems were found at isolated points and were not a recurring situation in any of the lines inspected. The problems are generally repaired as they are found. Overall the main lines looked in good condition, particularly the older Transite (AC) piping. Generally, no significant sources of infiltration were identified during the inspections with the exception of a short stub on N Palisades St. located north of N 3rd Ave which may have an end cap that is not properly sealed. Also, there generally appeared to be signs of infiltration and/or sand infiltration at the service lateral connections for the AC pipe. Overall, infiltration has not been a major problem in Rockaway Beach. Inflow through flooded sections of the sewer system has caused most of the high flow problems in the sewers and at the Rockaway Beach Wastewater Treatment Plant (WWTP).

WASTEWATER LIFT STATIONS

The City has eight wastewater lift stations serving the study area. All wastewater must be pumped in order to reach the WWTP. Since most of the town has little natural slope, wastewater is pumped more than once, as pumping stations discharge to sewers which discharge to a downstream pumping station. A schematic flow diagram of the lift station/gravity sewer system is shown in Figure 3-1. Data for all of the pump stations is presented in Appendix I. All of the wastewater flow ultimately goes to the Main Lift Station and is pumped into the WWTP.

The original sewer system for the City was installed in 1954. This construction included three lift stations (Main, South Fifth Street, and North Fourth Street). These stations utilized air pressure from a compressor to force (eject) the wastewater out of the stations and into the downstream sewers or the treatment plant. A system of compressed air piping from the treatment plant to the ejector stations accomplished this. All of the original stations have been replaced with stations that utilize centrifugal pumps. Five lift stations have been added to the collection system as Rockaway Beach has grown. Of added pump stations, 2 utilize centrifugal pumps, 2 use self-priming pumps, and one utilizes compressed air to eject the wastewater out of the station and into the nearest gravity sewer.

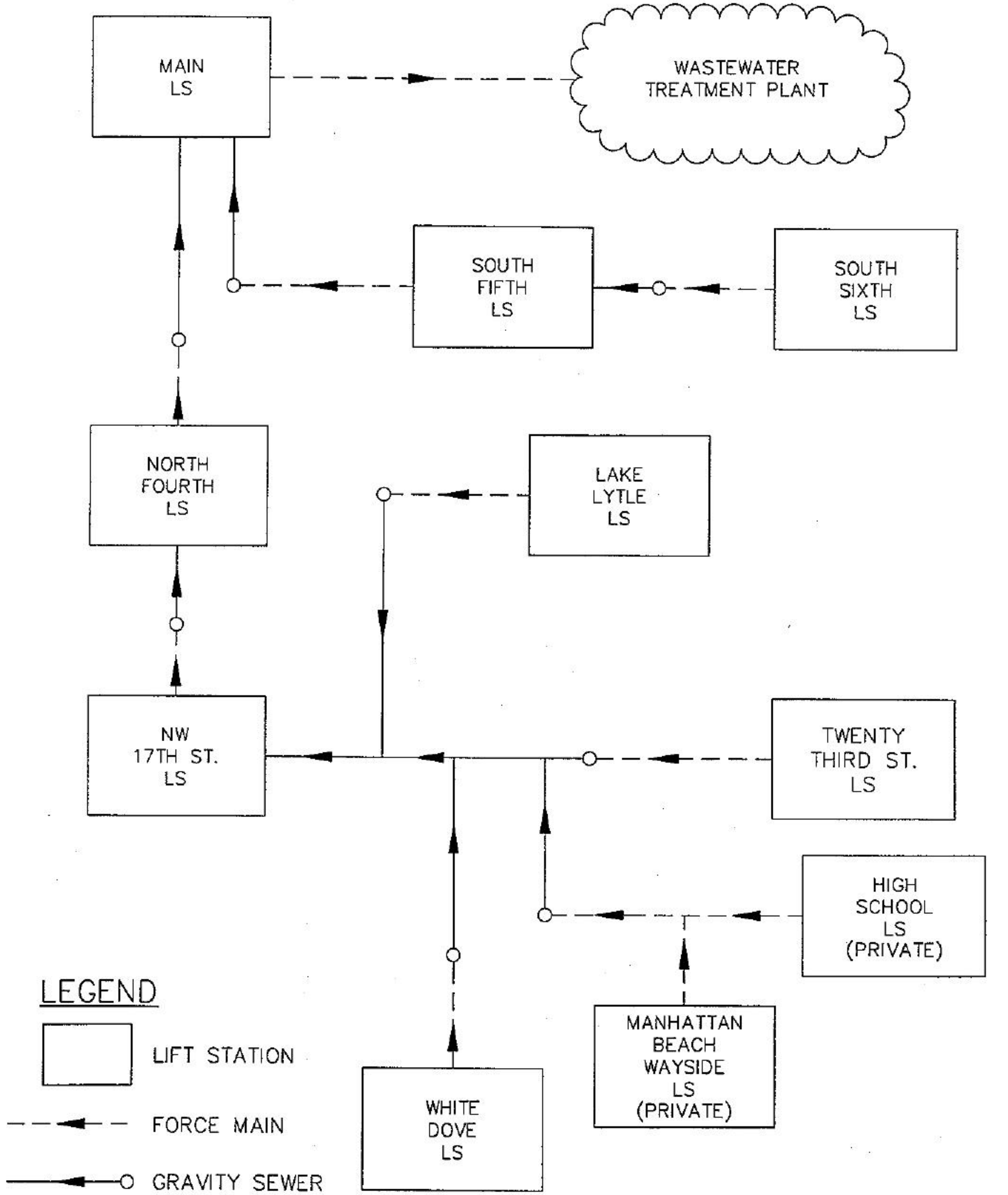
Figure 3-2 shows the locations of the lift stations and forcemains, which are located on City-owned property, street rights-of-way, or easements. Three stations (23rd St., Main and White Dove) have an emergency electrical generator permanently mounted on site which can provide power if the primary electrical service is lost. The other stations have a receptacle for hooking a portable generator to the control panel through a transfer switch to provide power in an emergency. Each control panel includes running time meters on each pumping unit. The submersible pumps in the South Fifth Avenue, South Sixth Avenue and White Dove lift stations include sensing units for over temperature and moisture in the pump motor. Alarms due to high or low wet well level, power loss and any other alarm condition are indicated by the flashing of a red light located adjacent to the control panel.

Capacities for each lift station were verified with a pump drawdown test conducted in 2004 as part of the City's previous sewer facilities plan. All of the lift stations are in overall fair operating condition. The extent of wear for each station is normal for the age of the station. A description of each lift station follows.

Main Lift Station—South Third and Anchor Streets

The Main Lift Station was originally constructed in 1954. It is a Smith and Loveless package station, with two 4C3 model Smith and Loveless two-speed pumps, each rated at 720 gallons per minute (gpm) at 47 feet total dynamic head. Wet well drawdown tests of the pumps on high speed with both influent sewers plugged established the actual pumping rate of 610 gpm. This station has a dry well configuration, with access to the dry well provided by an entrance tube and ladder. The station pumps through an 8-inch-diameter forcemain to the WWTP, a distance of approximately 575 feet. Pump control is by a bubbler system, with a small air compressor located in the pump chamber.

In 1980, a brick building was constructed around the entrance tube, a new electrical panel and control panel installed in the building, and the control panel in the dry well was decommissioned. An emergency electrical generator (Onan 50-kilowatt, 3-phase unit) and automatic transfer switch were added so that power for the pumps would be available during a power outage. The building has settled slightly in one corner, but there are no cracks in the brick fascia at present.



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CITY OF ROCKAWAY BEACH
LIFT STATION SCHEMATIC

2848SCHM.DWG

JOB NO.
2848

FIGURE 3-1

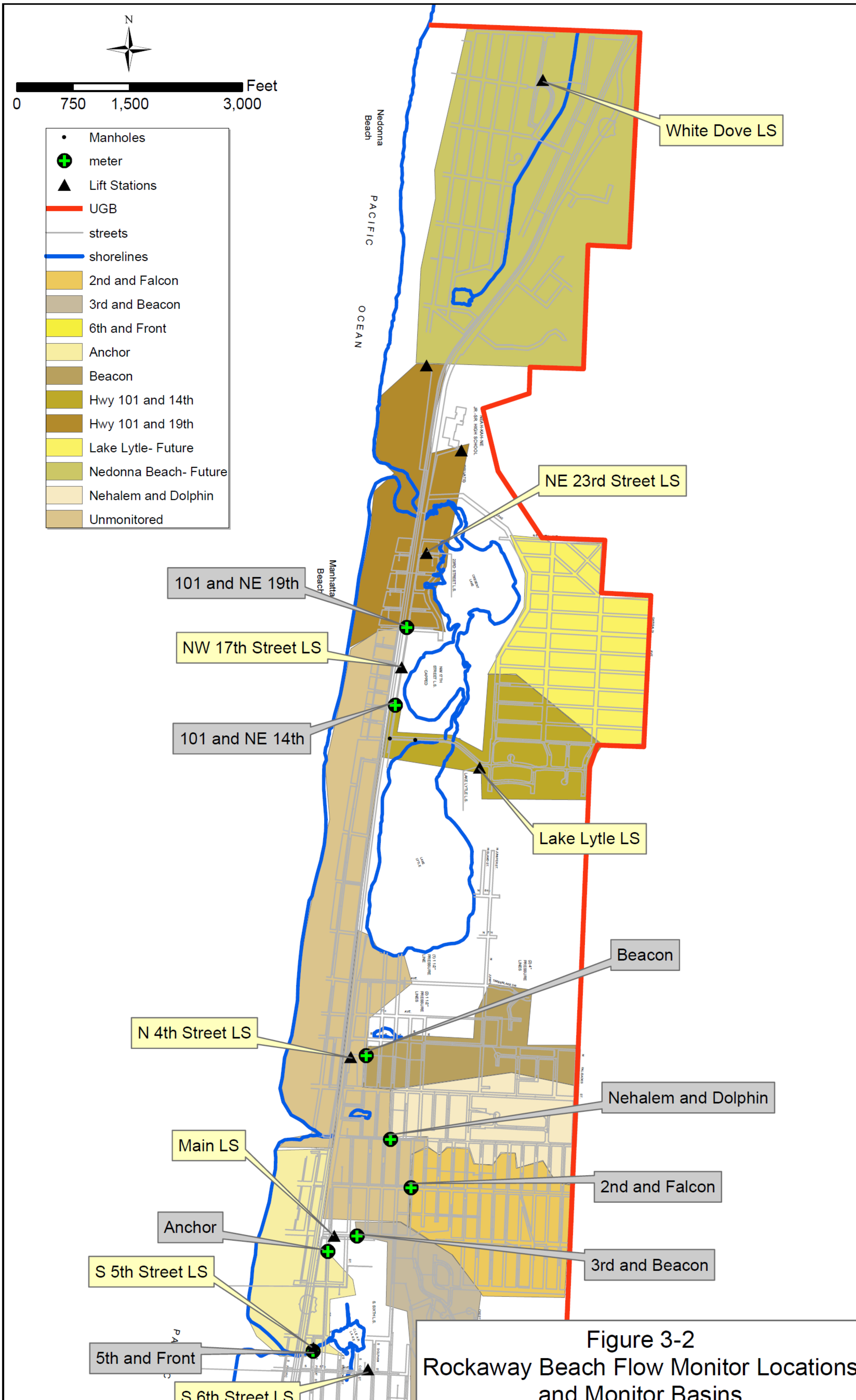


Figure 3-2
Rockaway Beach Flow Monitor Locations and Monitor Basins

The wet well is a 5-foot, 0 inch by 10 foot, 6 inch rectangular structure, with access through a manhole cover located adjacent to the building. Grease buildup in the wet well is a problem, and it is cleaned out periodically—currently about once per year. Overflow from the wet well in case of equipment malfunction is through a manhole top located in South Anchor Street.

Previous problems with floor corrosion in the dry well were corrected about 8 years ago. This problem has not recurred. The pump discharge check valves require maintenance on a regular basis. The bubbler-type pump control system is the original system installed in the station.

South Fifth Avenue Station—South Fifth and Front

The South Fifth Avenue Station was constructed in 1954 as an air ejector station. It was upgraded to its present configuration in 1980. The station consists of two self-priming 40MPC Hydromatic pumps mounted directly over the 6-foot-diameter wet well. Each pump is rated at 200 gpm, with 17 feet of total dynamic head. Wet well drawdown tests with the influent sewer plugged established the actual pumping rate of 250 gpm. This station has a back-up generator plug for use with a portable generator during power outages.

The pumps discharge through a 6-inch-diameter forcemain to a sanitary sewer located approximately 80 feet to the north of the station. This manhole has less than 1/16-inch of deterioration due to hydrogen sulfide attack on the inside walls, based on scratch tests done with a screwdriver. Wet well levels are sensed by float switches which turn the pumps on and off.

The wet well is in a manhole with access provided through a manhole cover. The pump enclosure is fiberglass. All valves and pumps are located above grade. Overflow of the wet well in case of equipment malfunction or extended power outage is through a manhole located on South Sixth Avenue.

The flapper-type valves required to keep the pump prime on this station need periodic maintenance. The control panel is showing signs of rust. The station is locked, but is not behind a fenced enclosure. No security light exists. The top elevation of the station is below the 100-year flood plain elevation.

South Sixth Avenue Station—South Sixth and Dolphin Street

The South Sixth Avenue Station, originally constructed in 1980 with Hydromatic pumps, was upgraded in 1990. It is a submersible pump station with an adjacent valve vault. It is located in a driveway area at 573 South Dolphin Street. The station has two Flygt 3085-438 pumps. These units have a larger capacity than the original. The pumps may be removed from the station by raising them on the rail system installed for this purpose. Wet well level is sensed by float switches which turn the pumps on and off. The pumps discharge through a 4-inch-diameter forcemain to a manhole located in Sixth Avenue approximately 325 feet west of the station. This manhole has less than 1/16-inch of deterioration due to hydrogen sulfide attack evident on the inside walls, based on scratch tests done with a screwdriver. Wet well drawdown tests with the influent sewer plugged established the actual pumping rate of 60 gpm. This station has a back-up generator plug for use with a portable generator during power outages.

The valve vault contains the check valve and gate valve on each pump discharge line. The two pump discharge lines merge into one within the valve vault.

The wet well is 5 feet in diameter. Wet well level is sensed by float switches which turn the pumps on and off. Both the wet well and valve vault are not water-tight, and groundwater and surface water can enter them. The power supply to the lift station is single phase and reportedly has not been causing any issues. The top elevation of the wet well and valve vault are below the 100-year flood elevation. Overflow point in case of equipment malfunction or extended power outage is through the hatch at the top of the wet well.

The station control panel is located adjacent to the station. It is not within a fenced enclosure.

North Fourth Avenue—North Fourth Avenue and Highway 101

The North Fourth Avenue Station replaced one of the original air ejector stations. It is a submersible type with two submersible pumps located in an 8-foot, 6-inch-diameter structure. The pumps are not rail-mounted, but are bolted to the wet well floor. Pump removal is therefore more difficult at this station. Each pump is rated at 200 gpm at 15 feet total dynamic head. Wet well drawdown tests with the influent sewer plugged confirmed this pumping rate. Wet well level is sensed by float switches which turn the pumps on and off. Access to the wet well is through two extremely heavy cast iron hatches. The pumps discharge through a 6-inch-diameter forcemain to a manhole south of the station at North Second Avenue, approximately 650 feet away. This manhole has less than 1/16-inch of deterioration due to hydrogen sulfide attack on the inside walls, based on a scratch test done with a screwdriver. All valves are accessible through two additional heavy cast iron hatches in the top of the wet well. The valves are located immediately underneath the hatches. This station has a back-up generator plug for use with a portable generator during power outages.

The top of the wet well is lower than the 100-year flood plain elevation. The overflow point in case of equipment malfunction or extended power outage is through the top of a manhole located in North Sixth Avenue. The station control panel is not within a fenced enclosure.

Lake Lytle Station—Lake Boulevard at Northeast 12th Avenue

The Lake Lytle Station is similar to the South Fifth Avenue Station, and was constructed in 1982. It is a self-priming type station with two Hydromatic 40 MPC pumping units rated at 200 gpm at 17 feet total dynamic head. Wet well drawdown tests with the influent sewer plugged established the actual pumping rate of 150 gpm. The flapper valves on each pump suction line are difficult to replace. Wet well level is sensed by float switches which turn the pumps on and off. The pumps discharge through a 6-inch diameter forcemain to a sanitary sewer located on NE 12th Ave., approximately 900 feet away. This manhole has less than 1/16-inch of deterioration due to hydrogen sulfide attack on the inside walls, based on a scratch test done with a screwdriver The forcemain has an air release manhole located at the Northeast 12th Avenue bridge. This station has a back-up generator plug for use with a portable generator during power outages. The discharge manhole can overflow when both pumps operate during peak flow events.

The wet well is 8 feet in diameter, and is oversized for the present flow rates to the station. Significant growth potential exists in the service area of this station. Access to the wet well is through a manhole cover. Overflow of the wet well during an equipment malfunction or extended power outage is a manhole in Northeast 12th Avenue. The pump enclosure is fiberglass. All of the valves are located above grade within the enclosure. The control panel is located within the fiber- glass enclosure. This station does not have a red light to signal an equipment malfunction.

Northwest 17th Avenue Station—Northwest 17th Avenue at Miller Street

The Northwest 17th Avenue Station was originally constructed in 1965 as an air ejector station and was contained in a prefabricated metal caisson. It was upgraded to its present configuration in 1992. For the upgrade, the original prefabricated metal dry well was converted into a wet well for the new station. The original entrance tube was removed and the metal can (which housed the original equipment and valves) was extended at its full diameter to the surface. A self priming pump station was installed in the extended area of the can below the outside grade, and a new fiberglass hinged cover installed at the top of the can. A metal floor separates the equipment in the pump station from the wet well below. A bolted hatch in the floor provides access to the wet well. This station has a back-up generator plug for use with a portable generator during power outages.

The two pumps are Hydromatic 40MP units; each rated at 200 gpm at 25 feet total dynamic head. Wet well drawdown tests with the influent sewer plugged established the actual pumping rate as only 143 gpm. These pumps are belt-driven from 5.0 horsepower motors. Wet well level is sensed by float switches which turn the pumps on and off. These pumps discharge through a 6-inch-diameter forcemain to an adjacent manhole. This manhole has less than 1/16-inch deterioration due to hydrogen sulfide attack on the inside walls, based on a scratch test with a screwdriver. The control panel is located in the pump chamber below grade. The area around the pump station is not fenced; however all the controls and equipment are contained under a locking cover. In case of equipment malfunction or extended power outage, overflow from the wet well is through the top of a manhole in Northeast 19thAvenue. The discharge manhole can overflow when both pumps operate during peak flow events.

23rd Avenue Station—Northeast 23rd Avenue, East of Highway 101

The 23rd Avenue Station was constructed in 1965. It is an air ejector type station, with the sewage forced out of the receiving vessel by air pressure derived from a compressed air system located in the station. The station includes two receiving vessels, two air compressors, and one air receiver. Filling of the on-line vessel results in activation of a level switch, which causes air from the air receiver to pressurize the vessel and force the sewage out of the station into the 4-inch-diameter forcemain. As the pressure in the air receiver drops, one of the air compressors turns on to re- pressurize it until the next cycle begins.

This station serves only a few houses and the community center, and this type of station is well suited to this low-flow situation. The station was renovated since 2006 and has two new air-compressors, a permanent power generator, and a wood building to house all the equipment and controls. The station entrance tube has some corrosion damage. The top of the entrance tube is below the 100-year flood elevation. Overflow from the wet well in the event of equipment malfunction or extended power outage is from a manhole top located in Northeast 23rd Avenue.

The area around the station is not fenced. The manhole at Northwest 23rd Avenue and Highway 101, where the forcemain discharges, has less than 1/16-inch of deterioration due to hydrogen sulfide attack on the inside walls, based on a scratch test done with a screwdriver.

White Dove Station—White Dove and Chiefton Streets

The White Dove Station was constructed in 1998. It is a submersible station with two 30-horsepower Myers pumps; each rated at 325 gpm and 126 feet total dynamic head. Wet Well drawdown tests with the influent sewers plugged established the actual pumping rate of 422 gpm. The wet well is 7 feet in diameter. Wet well level is sensed by float switches which turn the pumps on and off. The station valves are located in a valve vault. The station pumps through a 6-inch diameter forcemain to a manhole in Northwest 23rd Avenue, a distance of about 1 mile.

The station is enclosed in a fenced area. A building on the site houses a Kohler emergency electrical generator, the pump control panel, and an air injection system. The air injection system consists of an air compressor, air receiver, and pressure control switches. Air can be discharged into the wet well and each pump discharge line at the valve vault (for hydrogen sulfide control).

The discharge manhole has less than 1/16-inch deterioration due to hydrogen sulfide attack on the inside walls, based on a scratch test with a screwdriver. The station is located above the 100-year flood plain elevation. This station currently has excess capacity for the expected growth in the area and has very little run time. Variable frequency drives (VFD's) were added to the pump station since 2006 to control the speed of the pumps and provide better flexibility for pumping. A SCADA system reports pump station equipment failures to the WWTP; however it is currently not working. The guide rail brackets for the pumps are severely corroded.

Portable Generators

There are also two portable back-up generators that are available for the use at the lift stations, when there are power outages. The generators are stored at the WWTP. These generators are gasoline engine powered 230 Volt 3-Phase AC units with sufficient capacity to operate each of the lift stations which do not have their own dedicated emergency generator. Each portable generator has a 20-gallon capacity tank. Since the lift stations operate in series, when power is lost for the entire town, the generators must be progressively operated and moved so that the downstream stations are not surcharged.

Private Lift Stations

Two private lift stations discharge to the Rockaway Beach system—the high school and the Highway wayside, both located between Rockaway and Nedonna Beach. The high school is currently within the City Limits.

Both of these lift stations pump discharge to the gravity sewer at Northeast 24th Street and Highway 101. These lift stations were put in by the school district and highway department, and are maintained by them. Currently the City does not monitor their condition or get involved with their maintenance. However, in case of a malfunction of the equipment or break in the forcemain, a sewage spill could require that the City report it, and possibly be subject to fines, since the City holds the NPDES for the entire system.

INFLOW/INFILTRATION (I/I)

A wet weather flow analysis was performed to quantify the amount of I/I entering the collection system. The terms inflow and infiltration are defined below. This section (pgs. 3-9 to 3-16) was excerpted from the work done by Brown and Caldwell for the collection system modeling in 2004. HBH was not able to check the findings of the model.

The classical definitions of I/I reflect notions about the sources of extraneous flow in sewer systems. Thus, inflow has been generally reserved for directly connected sources of surface water, while infiltration generally refers to the flow of groundwater into sewer system defects. The separation of surface and groundwater is clouded somewhat by the U.S. Environmental Protection Agency's inclusion of foundation drains under the inflow category. Due to difficulties observed in identifying sources according to the strict surface/groundwater separation, terms have been coined to reflect the uncertainty. These include Rainfall Dependent Inflow/Infiltration (RDII) and Indirect Inflow. Generally recognized definitions are as follows:

Inflow. Inflow includes all sources of surface water that can enter the sewer system. In general, it is expected that smoke and dye testing will expose inflow sources. These sources include, but may not be limited to, the following:

Roof drains connected to private building sewers.

Surface drainage facilities (catch basins, and storm drainage cross connections) with direct or indirect connections to the sanitary system. Indirect connections occur from stormwater leaking from storm sewer defects and migrating directly or through soil to sanitary sewers.

Area drains. These may include yard drains, basement drains, and drains in external depressed stairways or driveways.

Foundation drains. Inclusion in the inflow category recognizes that surface waters draining from rooftops may enter foundation drains directly, or through poorly compacted soils that surround buildings. In addition, the practice of connecting sump pumps to sanitary sewers results in a point source.

Surface water that migrates to sanitary sewers through cracks or channels in poorly consolidated trench backfill.

The last two sources above and indirect cross connection of storm drainage systems have sometimes been referred to as Indirect or Virtual Inflow.

Infiltration. The term infiltration is usually reserved for the entry of groundwater through sewer defects. It is often considered to have two major components:

The relatively constant groundwater flow that varies slowly with seasonal change in the groundwater table (usually termed groundwater flow [GWF])

The transitory increase in groundwater table or collection of water in sewer trenches. The second component is often lumped into the definition of RDII.

The definitions above have often caused difficulties in I/I analysis. On the one hand, rainfall-related sewer flow hydrographs often exhibit quick response to rainfall that appears to be inflow. Subsequent search for directly connected inflow sources to match the observed peaks has often proved fruitless due to indirect sources or rapid increases in trench flows. The term RDII was coined to include all sources of extraneous clear water that enters the sewers. Usually, analysis of RDII is performed by subtracting the dry weather flow prior to an event (which includes both sanitary flow and some long term groundwater flow) from an observed hydrograph. Such an analysis may not adequately account for seasonal changes in the groundwater table unless this is included in a separate set of terms.

Base Sanitary Flow

Base sanitary flow refers to the flow from residences, commercial establishments, and industries. It does not include any allowance for groundwater infiltration.

Wet Weather Events

Wet weather flows are dependent on the base sanitary flow and RDII (including any GWF term). RDII is dependent on antecedent rainfall conditions: the amount of extraneous clear water that enters the sewers during a rainstorm will vary depending on whether a different rainstorm immediately preceded the one of interest.

Design Event

Analysis of capacity to transport wet weather flows is often conducted using a design event. This may be a synthetic rainfall hyetograph developed from a specified rainfall pattern (e.g., SCS Type Ia) and a rainfall depth specified for the locality with a given return period (e.g., the once in 5 year 24-hour storm depth). Synthetic events do not account for antecedent conditions—the peak flow from a small event preceded by a prolonged wet period may exceed that from a larger event in a dry period. In addition, flows from short duration events are likely to be lower than from prolonged events of the same depth, due to losses to surface runoff.

An alternative to the synthetic event, and a more accurate approach, is to choose storm periods (including antecedent conditions) from the long-term actual rainfall record that exhibit peak flows which meet the specified service level.

Log Pearson Type III Distribution

Predictive I/I models can generate large flow datasets, but a proper statistical analysis is necessary to reduce the model output into the information wastewater planners really need to know: how much flow does the system need to handle for a particular level of service. Numerous statistical distributions have been suggested based on their ability to fit flood data. For decades, federal agencies have used the Log Pearson Type III distribution to fit the relationship between flow volume and recurrence interval for river systems. As such, the Log Pearson Type III distribution has become a standard engineering method.

A Log Pearson Type III distribution analysis is prepared by following these steps:

1. *Compute the peak annual series from the model output.* This simply involves culling the largest flow values from each calendar year (or water year) into a separate data series. This greatly reduces the amount of data handling. The series will be based on whether the user is interested in peak hourly, peak daily, or peak monthly data. The summary of data into the properly resolved time step must be done first.

2. *Rank the peak annual flow events and compute the "plotting position" of each event.* The plotting position or recurrence interval is the average period over which a particular flow would be equaled or exceeded. For example, a 10-year flow would be equaled or exceeded an average of once per 10 years. The Cunnane plotting position formula is shown below:

$$T_R = \frac{\#Years + 0.2}{Rank - 0.4}$$

   The recurrence interval for a particular event, T_R, is roughly equal to the number of years of record divided by the rank of the event. The 0.2 and 0.4 factors in the numerator and denominator, respectively, help correct for the limited size of any sample set. The effects of this correction become less apparent for larger sample sizes or less extreme flow events. For example, the highest ranking event in a 50-year data series would have an estimated recurrence interval of 83.7 years using the Cunnane plotting position, while the tenth largest event would have a recurrence interval of 5.2 years.

3. *Compute the Log Pearson Type III fit to the peak annual flow series.* The Log Pearson Type III statistical distribution is computed as follows:

$$\log(Flow) = \overline{\log(Flow)} + K\sigma_{\log(Flow)}$$

   First, calculate the base 10 logarithm of each event in the peak annual series. $\overline{\log(Flow)}$ is the average of the base 10 logarithms of all of the events in the peak annual series. (Note, the calculation of $\overline{\log(Flow)}$ above is not the same as computing the average of the flow values in the peak annual series, and then taking the logarithm of this average.) $\sigma_{\log(Flow)}$ is the standard deviation of the set of log(Flow) data. The standard deviation can be calculated using a spreadsheet program, or otherwise as:

$$\sigma_{\log(Flow)} = \sqrt{\frac{\sum(\log(Flow) - \overline{\log(Flow)})^2}{\#Years - 1}}$$

K is the cumulative probability distribution function for the Log Pearson Type III distribution. It is a complex formula that requires Skew and the Standard Normal Inverse Probability function. K values for specific recurrence intervals are typically read from tables in hydrologic texts, such as Bulletin 17B of the U.S. Geological Survey.

4. *Plot the peak annual flow series and Log Pearson Type III distribution together.* The Log Pearson Type III distribution plot can be particularly useful for smoothing the peak annual data series in areas of the curve and predicting the magnitude of infrequent storms.

The Log Pearson Type III analysis described in the paragraphs above was used in this study to determine current and future basin responses to rainfall events.

Flow, Groundwater, and Rainfall Monitoring

A sanitary sewer flow monitoring plan was developed for this study that described the steps necessary to characterize the quantity and sources of the wastewater flows. The plan included placing eight flow monitors at locations throughout the city to remain in place during both dry and wet weather periods. The purpose of this approach was to characterize dry weather base flows and wet weather responses to storm events during periods of high groundwater. Two weeks of dry weather flow were recorded in September 2002 and approximately 4 weeks of wet weather flow were recorded from February to March 2003.

To implement this plan, the City obtained eight flow monitors and installed them at the following locations, as indicated in Figure 3-2:

- Second and Falcon
- Third and Beacon
- Fifth and Front
- Nehalem and Dolphin
- Anchor, between South Third and South Fourth
- Beacon, between North Third and North Fourth
- Highway 101 and Northeast 14th
- Highway 101 and Northeast 19th

Rainfall data during the monitoring period was obtained from an hourly gauge at Astoria and adjusted according to a comparison to daily totals from rain gauges at Nehalem and the Rockaway Beach WWTP. Long term hourly rainfall data was obtained from the National Climatic Data Center (http://lwf.ncdc.noaa.gov/oa/ncdc.html) gauge located at Nehalem, approximately 11 miles north of Rockaway Beach. The period of record covered at this site was from 1948 to 2002.

Modeling. Models were developed to simulate the response of the sanitary collection system to sanitary, stormwater, and groundwater inputs. Once constructed and calibrated, the models were used to predict flows under various scenarios, including dry and wet weather periods, for both existing and future growth conditions. The models were used to predict flows conveyed to the WWTP and to evaluate the hydraulic capacity of the collection system. Hydraulic deficiencies were identified along with the required pipe sizes needed to eliminate the deficiencies.

Hydrologic. Analysis of I/I and hydraulic capacity requires a method to relate sewer flows to rainfall. Methods in use are documented in the Water Environment Research Foundation project report *Sanitary Sewer Overflow (SSO) Flow Prediction Technologies* (Project 97-CTS-8, April 1999). Methods in use range from simple application of a constant RDII rate (e.g., so many gallons per acre per day) to sophisticated hydrologic methods that balance the amounts attributable to rainfall, I/I, and groundwater flow. The report notes that hydrologic methods are preferred for prediction of peak flows under actual conditions (prolonged wet periods or multiple events).

The methods described are sensitive to the coefficients derived for the model. The rainfall flow regression is also sensitive to the aggregation periods used for antecedent rainfall. Model coefficients are in turn sensitive to the data underlying the analyses. To avoid errors in projection to storms not included in the monitoring record, the recommended approach is to calibrate the models over a monitoring period that includes a full wet season.

The rainfall-flow regression method was used for modeling. The basins used in the hydrologic modeling correspond to the areas covered by the flow monitors, with the intention that all flow from each basin is measured by a single flow monitor. These basins are illustrated in Figure 3-2. Data from the monitors was highly influenced by the presence of lift stations throughout the city. As a result, it was difficult to calibrate many of the hydrologic models.

It was determined that the Highway 101 and Northeast 14th meter data resulted in the best calibration of current conditions. All of the other basins displayed the influence of pump stations or flow meter errors. In particular, the Fifth and Front Street meter displayed flow results that were inconsistent with the other meters and suggested a flow meter calibration problem or meter error. This flow meter was located upstream of the Anchor meter, so the data was completely ignored, and data from Anchor was used instead. Hydrologic parameters for seven remaining basins were approximated from the results of the Highway 101 and Northeast 14th basin. This approximation was done by multiplying the regression coefficients by the ratio of calibration basin area to Highway 101 and Northeast 14th basin area. The flow response of the unmetered area was approximated by determining an average I/I rate per acre during peak flow conditions in the eight monitoring basins and applying this rate to the 115 acres of unmonitored area.

The following steps were taken in modeling each basin:

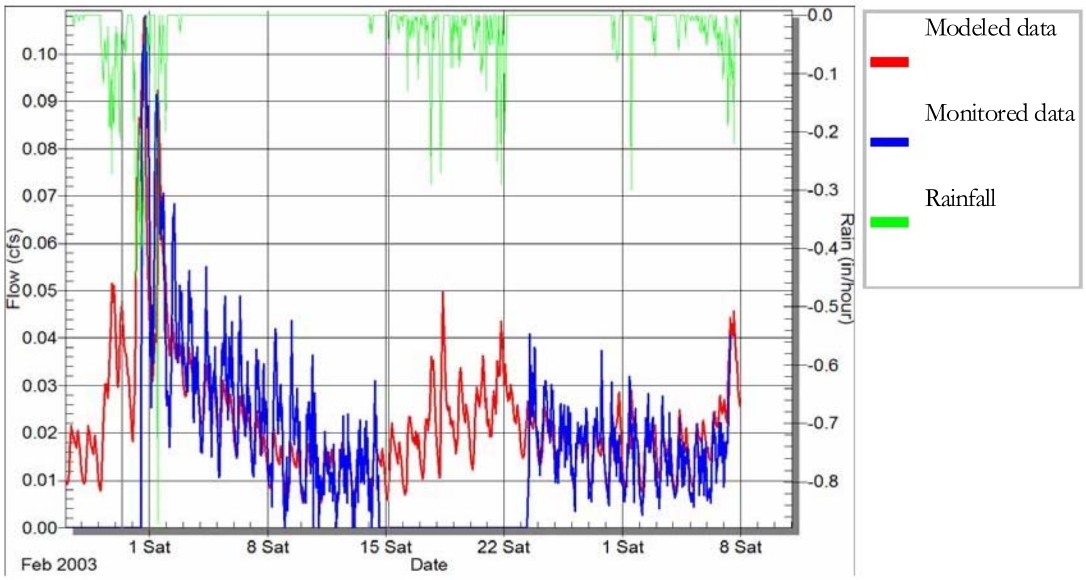
1. Calibrate the model to the monitoring period data.

2. Run long-term rainfall data through the calibrated model to produce a continuous long-term hydrograph.

3. Determine the 5-year peak hour flow through statistical analysis using a Log Pearson Type III distribution.

Each basin within the study area was calibrated with data from September 19 to October 2, 2002 for dry weather flow, and from January 31 to March 7, 2003, for wet weather flows. An example of a basin calibration for Highway 101 and Northeast 14th is shown in Figure 3-3. During the monitoring period, the model matched most of the peak flows fairly well, with flow subsiding between storm events.

Figure 3-3. Calibrated Model Fit for Highway 101 and Northeast 14th Monitor



Once calibrated to the monitoring period, long-term rainfall data were run through each basin model. The output from this process was the I/I and total flow that would be expected at every hour of every day during the period of historical rainfall. This information allows the prediction of large storm events that may not have occurred during the monitoring period but can be used for design purposes (such as the 5-, 10-, and 25-year recurrence flow events). The long-term Nehalem rainfall record described earlier was used for this analysis. From the simulation database, the maximum flow for the period desired (maximum hour) can be extracted from an occurrence-frequency analysis. A Log Pearson Type III statistical analysis was used to develop a relationship between flows and return period.

Incremental flows from future growth were developed based on population increases as projected by the City, as well as on per capita I/I and dry weather wastewater flow rates as developed under the current conditions. Future growth in the City is predicted to occur in the Lake Lytle and Nedonna Beach areas only. All other areas of the collection system are currently considered to be at build-out levels. I/I rates for additional population growth were assumed to be the same as current rates in areas of existing development. These rates are conservatively high as new materials and construction practices will result in lower than current I/I rates.

Hydraulic. A hydraulic analysis was conducted to determine the performance of the collection system under design conditions. Specifically, the model was used to identify pipes within the collection system that are inadequately sized to handle current and future design flows.

A steady state analysis of the collection system was conducted. Thus, the future 5-year peak hour flow from each of the monitoring basins was predicted using the hydrologic modeling process described above. Peak flows were distributed between the major manholes within the unmetered basin to approximate system flow distribution. The flows were then routed through the collection system and the peak hourly flow within each pipe segment was determined within the modeled system.

The major pipelines of the collection system were included in the hydraulic model, as illustrated in Appendix F. The pipes selected include:

- All 10-inch-diameter and larger pipes
- Smaller diameter pipes to extend the model north on Highway 101 to the Lake Lytle and Nedonna Beach areas, as well as onto side streets to capture the location of each of the eight flow monitors.

The 5-year peak hour flows at the outlet of each basin shown in Figure 3-2 are listed in Table 3-1.

Table 3-1. Future Dry Weather and Peak 5-year Flows

Basin	Future I/I, cfs[1]	Future DWF, cfs	Peak flow, cfs
Second and Falcon	0.16	0.002	0.162
Third and Beacon	0.12	0.004	0.124
Beacon	0.13	0.001	0.131
Anchor	0.33	0.014	0.344
Nehalem and Dolphin	0.13	0.005	0.135
Highway 101 and Northeast 14th	0.39	0.245	0.635
Highway 101 and Northeast 19th	0.56	0.207	0.767
Unmetered	0.30	0.001	0.301

[1] gpm = gallons per minute

Current capacity of the modeled system was determined with a Manning's calculation (n=0.13). Pipe slopes and lengths were obtained from as-built drawings. Where future peak flows exceeded current capacity, an appropriate pipe diameter was selected to allow the peak flow to be contained completely within the pipe. It was determined that the entire modeled system needed to be upsized to a minimum of 12-inch pipe to accommodate future growth. Although the design period of this Facility Plan is only 20 years, collection systems are usually built on a 50- to 100-year planning period, as it is cost-prohibitive to replace collection systems on a more frequent interval. Thus, upsizing the entire modeled system downstream of Lake Lytle and Nedonna Beach to 15-inch pipe increases the total cost of improvements by 5 percent, but nearly doubles the flow capacity of the system and ensures adequate hydraulic capacity through build-out conditions. The existing and future hydraulic capacity of all modeled sewers, as well as peak projected flows for each pipe, are listed in Table 3-2. Phasing opportunities for the collection system work are described in Chapter 5 of this Facilities Plan. As an alternative, under capacity sewers can be relieved by bypassing them with forcemain extensions to the next downstream lift station.

Table 3-2. Peak Hour Design Flow, and Current and Future Capacity of Modeled Sewers

Pipe segment	Upstream node	Downstream node	Flow Type	Current dia, in	Slope, ft/ft	Current capacity, cfs	Future peak flow, cfs	Future dia, in	Future Capacity, cfs
13-PS	13	PS	Gravity	8	0.0047	1.53	0.354	8	1.53
PS-14	PS	14	Pressure						
14-15	14	15	Gravity	8	0.004	0.46	0.354	8	0.46
15-16	15	16	Gravity	8	0.0014	0.46	0.354	8	0.46
16-17	16	17	Gravity	8	0.002	0.55	0.354	8	0.55
17-18	17	18	Gravity	8	0.002	0.55	0.354	8	0.55
18-31	18	31	Gravity	8	0.002	0.55	0.354	8	0.55
29-30	29	30	Gravity	8	0.002	0.55	0.127	8	0.55
31-32	31	32	Gravity	8	0.0034	0.72	0.354	8	0.72
32-30	32	30	Gravity	12	0.003	1.97	2.47	15	3.53
33-32	33	32	Gravity	10	0.002	1	2.1	15	2.89
34-33	34	33	Gravity	10	0.002	1	2.1	15	2.89
36-34	36	34	Gravity	10	0.002	1	2.1	15	2.89
37-36	37	36	Gravity	10	0.001	0.7	2.1	15	2.04
38-37	38	37	Gravity	10	0.002	1	1.877	15	2.89
39-36	39	36	Gravity	8	0.003	0.67	0.167	8	0.67
41-39	41	39	Gravity	8	0.003	0.67	0.167	8	0.67
42-41	42	41	Gravity	8	0.003	0.67	0.167	8	0.67
46-42	46	42	Gravity	8	0.003	0.67	0.167	8	0.67
49-38	49	38	Gravity	8	0.004	0.78	1.877	15	4.08
51-49	51	49	Gravity	8	0.003	0.67	0.139	8	0.67
52-51	52	51	Gravity	8	0.002	0.55	0.139	8	0.55
53-52	53	52	Gravity	8	0.004	0.78	0.139	8	0.78
55-49	55	49	Gravity	8	0.004	0.78	1.877	15	44.08
56-55	56	55	Gravity	8	0.002	0.55	1.877	15	2.89
57-56	57	56	Gravity	8	0.002	0.55	1.877	15	2.89
PS-57	PS	57	Pressure						
68-PS	68	PS	Gravity	15	0.002	2.89	1.694	15	2.89
69-68	69	68	Gravity	8	0.011	1.29	0.135	8	1.29
77-68	77	68	Gravity	8	0.004	0.78	1.559	15	4.08
81-77	81	77	Gravity	8	0.003	0.67	1.545	12	1.95
82-81	82	81	Gravity	8	0.002	0.55	1.545	12	1.59
83-82	83	82	Gravity	8	0.002	0.55	1.531	12	1.59
84-83	84	83	Gravity	8	0.002	0.55	1.531	12	1.59
85-84	85	84	Gravity	8	0.002	0.55	1.531	12	1.59
86-85	86	85	Gravity	8	0.002	0.55	1.531	12	1.59
87-86	87	86	Gravity	8	0.002	0.55	1.531	12	1.59
88-87	88	87	Gravity	8	0.002	0.55	1.495	12	1.59
89-88	89	88	Gravity	8	0.002	0.55	1.495	12	1.59
90-89	90	89	Gravity	8	0.002	0.55	1.495	12	1.59
91-90	91	90	Gravity	8	0.002	0.55	1.495	12	1.59
92-91	92	91	Gravity	8	0.002	0.55	1.495	12	1.59
93-92	93	92	Gravity	8	0.0028	0.65	1.458	12	1.88
94-93	94	93	Gravity	8	0.0012	0.43	1.458	12	1.23
95-94	95	94	Gravity	8	0.002	0.55	1.458	12	1.59
96-95	96	95	Gravity	8	0.002	0.55	1.458	12	1.59
97-96	97	96	Gravity	8	0.002	0.55	1.458	12	1.59
98-97	98	97	Gravity	8	0.0019	0.54	1.458	12	1.55
99-98	99	98	Pressure						
100-99	100	99	Gravity	8	0.002	0.55	1.402	12	1.59
101-100	101	100	Gravity	8	0.0026	0.63	0.767	12	1.81
102-100	102	100	Gravity	8	0.0095	1.2	0.635	12	3.47
103-102	103	102	Gravity	8	0.002	0.59	0.635	12	1.59
104-103	104	103	Gravity	8	0.002	0.59	0.635	12	1.59

EPA Criteria for Non-Excessive I/I

The Environmental Protection Agency (EPA) has developed a system to determine if a community has "non- excessive" I/I levels within their wastewater system. The EPA method requires that the system be analyzed under differing and extreme conditions then compared against an established benchmark to determine if the I/I levels are significant. The benchmarks are based on historical average flows taken nationally and include sewer wasteflows. The benchmarks established by EPA for non-excessive I/I are as follows:

EPA Criteria for Infiltration…………………………120 gpcd
EPA Criteria for Inflow…………………………....275 gpcd

I/I for Rockaway Beach was analyzed by reviewing plant daily monitoring reports (DMRs) between January and April (2007-2011) when soils are saturated and I/I is most apparent.

The City of Rockaway Beach has a variable population with many residences that are seasonal. The population in the winter time is assumed to be solely permanent residents, which for 2007-2011 averaged 1,077. However, the seasonal homes still contribute to I/I even though there are no residents, so for the purpose of this analysis, it was decided to look solely at the I/I without any wasteflow contribution. Taking the permanent population and multiplying it by the average per capita wasteflow of 47 gpcd as shown in Table 2-12 results in a residential wasteflow of 51,000 gpd for the wintertime population. The commercial contribution, also derived from Table 2-12, is calculated by taking the 40,000 gpd estimated dry weather commercial flow and dividing it by the average seasonal and permanent population for 2007-2011 (3,063), which results in a per capita commercial component of 13 gpcd. This unit flow was then multiplied by the permanent residential population to obtain a commercial wasteflow of 14,000 gpd for the winter time. The total residential/commercial base flow is then 65,000 gpd for the winter time population. This base flow was then subtracted from the total winter flow to determine the I/I flow. The I/I flow was then divided by the permanent and seasonal population (3,063) to determine the per capita I/I flow to account for all the residential infrastructure contributing to I/I.

The EPA benchmarks used for comparison are based on the EPA historical unit flow average of 70 gpcd for sewer flows, 40 gpcd for infiltration, and 10 gpcd for commercial/industrial flows for a total of 120 gpcd[8]. Therefore 40 gpcd was used for comparison of the infiltration, and 195 gpcd[9] was derived and used for comparison of the inflow.

A summary of the non-excessive infiltration analysis is provided in Table 3-3. Ten 7-day periods between January and April (2007-2011) were examined for days when the groundwater table is high, but there is little-to-no active rainfall. The rationale is that during these periods the flows are higher due to the elevated groundwater table and not active rainfall. Therefore, the increased flow is solely a result of infiltration into the system. It should be noted that this method does not include rain induced infiltration. Each period had little or no rainfall during the week or within the few days prior to the period. The 7-day average flows within the period were calculated. It was determined that the average infiltration during these periods was 43 gpcd, which marginally exceeds the EPA's limit for non-excessive infiltration.

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[8] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, pg. 200.
[9] Based on 275 gpcd minus 70 gpcd for sewer flow and 10 gpcd for commercial/industrial flow.

Table 3-3
Non-Excessive Infiltration Analysis for Rockaway Beach

Dry Period	Total Rainfall (in)	7-Day Average Flow (MGD)			GPCD
		Flow	Base Flow	Infiltration	Infiltration
1/31/07-2/06/07	0.00	0.21	0.06	0.15	48
4/24/07-4/30/07	0.19	0.24	0.06	0.18	57
2/23/08-2/29/08	0.30	0.22	0.06	0.16	52
4/11/08-4/17/08	0.50	0.23	0.06	0.17	55
1/20/09-1/26/09	0.20	0.17	0.06	0.11	36
4/23/09-4/29/09	0.10	0.20	0.06	0.14	45
2/19/10-2/25/10	0.93	0.20	0.06	0.14	45
3/02/10-3/08/10	0.32	0.19	0.06	0.13	42
1/30/11-2/05/11	0.83	0.15	0.06	0.08	27
4/18/11-4/24/11	0.61	0.15	0.06	0.09	28
Rockaway Beach Average					**43**
EPA Criteria					**40**

Inflow conditions were analyzed based on largest rain events and the corresponding flows that occurred during the data set (Table 3-4). It is assumed that an intense rain event makes it way quickly into the collection system through inflow points or through rain-induced infiltration. This analysis determined the average inflow condition for Rockaway Beach equals 198 gpcd, which is only slightly higher than EPA criteria for non-excessive inflow.

Table 3-4
Non-Excessive Inflow Analysis for Rockaway Beach

Date	Rainfall(in)	Flow MGD			GPCD
		Flow	Base Flow	Inflow	Inflow
3/25/2007	2.10	0.732	0.06	0.67	218
2/25/2007	2.20	0.709	0.06	0.64	210
2/7/2008	0.90	0.727	0.06	0.66	216
1/7/2008	1.30	0.512	0.06	0.45	146
1/8/2009	3.20	1.168	0.06	1.10	360
1/2/2009	1.80	1.036	0.06	0.97	317
1/16/2010	1.50	0.509	0.06	0.44	145
1/1/2010	1.40	0.454	0.06	0.39	127
1/16/2011	4.40	0.502	0.06	0.44	143
4/4/2011	1.80	0.361	0.06	0.30	97
Rockaway Beach Average					**198**
EPA Criteria					**195**

Based on the above analysis, it appears that the infiltration and inflow rates marginally exceed the EPA's criteria for non-excessiveness. It does appear from the analysis that the I/I rates have gone down over time, which may be attributable to the City's annual collection system inspection and repair program. No further study is needed to determine if it is more cost effective to remove I/I rather than to increase the capacity of the treatment and collection systems.

CHAPTER 4

Existing Wastewater Treatment Facilities

CHAPTER 4 EXISTING WASTEWATER TREATMENT FACILITIES

DESCRIPTION

The Rockaway Beach Wastewater Treatment Plant (WWTP) is located within the area enclosed by South Third and South Fourth Avenues between D Street and E Street in the City of Rockaway Beach (City). The WWTP was originally constructed in 1954. The construction consisted of a primary clarifier, trickling filter, anaerobic sludge digester and sludge drying beds. The plant underwent an expansion in 1979 which consisted of a headworks with screening/grit removal, a package aeration basin/secondary clarifier/disinfection tank, in-plant pump station, tertiary filters, and overflow lagoon. The expansion also consisted of converting the anaerobic digester to an aerobic digester and converting the sludge drying beds to a humus pond.

A project that added dechlorination, an effluent pump station and ocean outfall was completed in 2005. The tertiary filters were demolished as part of the project. The WWTP flow meter is located at the Chlorine Contact Chamber. To date, treatment plant consists of the following unit processes:

Grit removal
Screening
Primary clarification
Tricking filter biological treatment
Activated sludge biological treatment with positive displacement blowers
Secondary clarification
Disinfection
Dechlorination
Effluent pump station and ocean outfall
Aerobic sludge digestion
Overflow storage and sludge holding ponds

The treatment plant also includes support facilities, including maintenance, laboratory, and office buildings. Figure 4-1 presents a process flow schematic drawing of the Rockaway Beach WWTP. This chapter assesses the existing treatment plant unit processes in terms of their condition, capacity, and performance.

The design criteria for the Rockaway WWTP are listed in Table 4-1. The design criteria are taken from the 1979 plant expansion drawings prepared by HGE Engineers, the associated operation and maintenance manuals for each component and HBH's evaluation of each component through spreadsheet modeling.

Table 4-1. Existing WWTP Design Data

Process or Design Criteria	Unit	Value
Flow Capacity		
Average Dry Weather Flow (ADWF)	MGD	0.6
Maximum Hydraulic Capacity	MGD	1.7
Loading Capacity		
Average BOD	PPD	750
Average TSS	PPD	750
Headworks		
Vortex Grit Chamber	number	1
Date Installed	-	Rebuilt circa 2003
Flow Capacity	MGD	2.5
Horsepower	HP	3/4
Grit Classifier	number	1
Date Installed	-	Installed 2012
Helisieve Screen Flow Capacity	number	1
Date Installed	-	2000
Flow Capacity	MGD	2.5
Manual Bar Screen	number	1
Primary Clarifier	number	1
Date Installed	-	1954 originally. Rebuilt 1979.
Diameter	feet	28
Surface Area	sq. ft.	616
Average Depth	feet	8
Volume	gallons	36,800
Capacity		
Average Overflow @ 975 GPD/sq. ft.	MGD	0.60
Detention Time	hr	1.5
Peak Overflow @ 3,000 GPD/sq. ft	MGD	1.85
Detention Time	hr	0.5
Weir Length	feet	75
Weir Loading	GPD/ft	8,000-25,000
Sludge/Scum Pump	number	1
Date Installed	-	1979
Capacity	MGD	0.30

Table 4-1. Existing WWTP Design Data (cont.)

Process or Design Criteria	Unit	Value
Trickling Filter	number	1
Date Installed	-	1954 originally. Rebuilt 1979. New media circa 1995.
Recirculation Pump	number/MGD/HP	1/1.0/10
Date Installed	-	1979
Diameter	feet	66
Surface Area	sq. ft.	3,420
Depth	feet	8
Volume	gallons	205,000
Media	-	Rock
Distribution Type	-	Rotary
Hydraulic Load @ 1 MGD	GPD/sq. ft.	292
BOD Capacity		
Avg. BOD Load @ 25 ppd/1000 cu. ft.	PPD	680
Peak BOD Load @ 50 ppd/1000 cu. ft.	PPD	1,370
In-plant Pump Station		
Date Installed	-	1979 originally. New pumps 2005.
Pump #1	MGD/HP	0.84/ 7.5
Pump #2	MGD/HP	0.84/ 7.5
Pump #3 (Standby)	MGD/HP	1.68/ 20
Type	-	Self-priming Pumps
Operation	-	Variable Frequency Drive (VFD)
Aeration Basins (Aerated Solids Contact) (donut type with integral clarifier)	number	2
Date Installed	-	1979
Depth	feet	12
Volume (each tank)	gallons	52,300
Diffuser Type	-	Coarse Bubble
Hydraulic Capacity		
Flow at 1 hr detention	MGD	2.50
MLSS	mg/L	1,000-3,000
SRT	days	14

Table 4-1. Existing WWTP Design Data (cont.)

Process or Design Criteria	Unit	Value
Aeration Basin/ Digester Blowers	number	2
Date Installed	-	1979
Blower 1	CFM/HP	300/25
Blower 2 (Standby)	CFM/HP	300/25
Secondary Clarifier	number	1
Date Installed	-	1979
Diameter	feet	42
Surface Area	sq. ft.	1,385
Average Depth	feet	11
Volume	gallons	114,000
Hydraulic Capacity		
Average Overflow @ 600 GPD/sq. ft.	MGD	0.83
Peak Overflow @ 1,200 GPD/sq. ft	MGD	1.66
Weir Length	feet	123
Weir Loading	GPD/foot	7,000-14,000
RAS Pump	number	2
Date Installed	-	1979
Capacity/Pump	MGD	0.75
WAS/Sludge Transfer Pump	number	1
Date Installed	-	1979
Capacity	MGD	0.30
Chlorine Contact Tank	number	2
Date Installed	-	1979
Volume (each tank)	cu. ft.	2,610
Length to Width Ratio	-	30:1
Scum Baffle Location/ Type	-	End of Tank/ Aluminum
Flow Capacity based on 0.5 hr detention	MGD	1.85
Sodium Hypochorite Tank	number/ capacity	1/ 540 gal.
Date Installed	-	2008
Effluent Pump Station		
Date Installed	-	2005
Pump #1	MGD/HP	1.68/ 40
Pump #2 (Standby)	MGD/HP	1.68/ 40
Operation	-	Variable Frequency Drive (VFD)

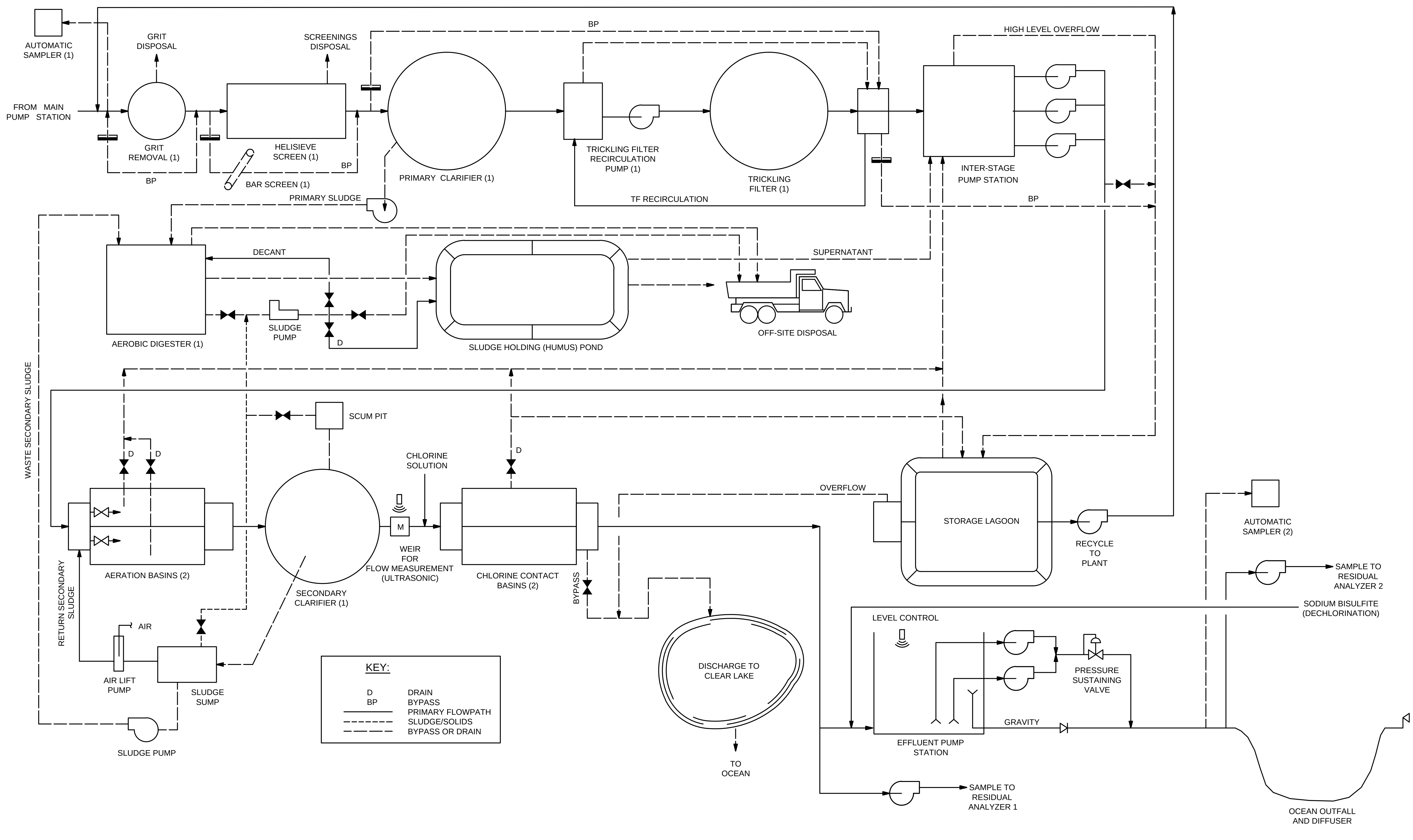
Table 4-1. Existing WWTP Design Data (cont.)

Process or Design Criteria	Unit	Value
Ocean Outfall		
Date Installed	-	2005
Diameter, Inside	inch	10
Length	feet	5,068
Diffuser Port	number	2
Diameter at Ports	inch	4
Dilution Ratio Provided by Diffuser	-	79:1
Regulatory Mixing Zone Diameter	feet	200
Depth of Submergence	feet	45
Dechlorination		
Date Installed	-	2005
Sodium Bisulfite Storage	gallons	475
Pumps	number	1
Type	-	Peristaltic
Peak Capacity	gallons/hr	50
Detention Time	seconds	180
Aerobic Digester		
Date Installed	-	1954 originally. Rebuilt 1979.
Diameter	feet	28
Depth	feet	18
Volume	gallons	76,000
Diffuser Type	-	Coarse Bubble
Aeration	CFM/1000ft^3	30
Minimum Solids Retention Time	days	8-13
Volatile Solids Loading Rate	PPD VS/ft^3	0.04
Solids Capacity	PPD	800
WAS/Sludge Transfer Pump	number	1
Date Installed	-	1979
Capacity	MGD	0.30
Humus Pond	number	1
Date Installed	-	1954 originally. Rebuilt 1979.
Surface Area	sq. ft.	7,676
Depth	feet	4
Volume	gallons	165,900

Table 4-1. Existing WWTP Design Data (cont.)

Process or Design Criteria	Unit	Value
Storage Lagoon		
Date Installed	-	1979
Volume	MG	1.6
Depth	feet	1-4
Return Pump	GPM/HP	350/5
Electrical Generator	number	1
Date Installed	-	1979
Capacity	kW	125 kW
Flow Meter (effluent)	number	1
Samplers	number	2

MGD = Million gallons per day
MG = Million gallons
PPD = pounds per day
CFM = cubic feet per minute
GPM = gallons per minute
GPD = gallons per day
HP = horsepower



H B H Consulting Engineers

CITY OF ROCKAWAY
SEWER FACILITIES PLAN

Existing Wastewater Treatment Plant
Process Flow Schematic

DRAWN BY: BTM
DATE: October 14, 2013

FIG. 4-1

UNIT PROCESS PERFORMANCE AND CONDITION

The WWTP consists of individual unit processes that work in combination to remove pollutants from the water prior to discharge. Each of the processes is evaluated in this section.

Grit Removal and Classification

Grit is dense particulate material, such as sand, gravel, eggshells, coffee grounds, and similar materials. Removal of grit ahead of other treatment reduces potential abrasion damage and other adverse impacts to equipment. The Rockaway WWTP uses a vortex-type grit removal device known as a Pista grit unit (Model 2.5). This device is intended to separate the more dense grit particles from other particulates that are lighter and organic in nature.

The Pista grit unit induces a mild swirling action that allows grit to settle, while less dense materials remain in suspension. The grit is pumped from the base of the unit to a helical device, which assists in dewatering it for disposal as a solid residue. The grit removal tank is 6 feet 9 inches in diameter at its upper treatment zone and 3 feet 0 inches in diameter in the lower grit concentration area.

The existing Pista grit unit is adequately sized, and is reported to be in good operating condition. All submerged components were re-built within the past 10 years. The grit classifier and dewatering system was replaced in 2012. Routine maintenance will be necessary to maintain the unit in operation, but no capital modifications are required.

Screening

Screening of wastewater removes larger solids from the influent flow to protect downstream process equipment. The quantity and quality of screenings removed is dependent on the type of screen used, opening size, collection system characteristics, and loading of the screen.

The Rockaway WWTP uses a single Hycor Helisieve HLS300 screen installed in 2000 located downstream of the grit removal device. Ideally the screen should be located ahead of the grit removal unit to prevent fouling of the grit removal mechanical components; however this does not seem to be an issue. The screen is located in the channel that originally housed the comminutor.

The Helisieve unit is reportedly working very well and no significant improvements are required. A backup manual bar-screen is located in a parallel channel.

Primary Clarification

Primary clarification is used to remove particulate material from screened wastewater by gravity sedimentation. The primary clarifier was originally built in 1954. All the equipment inside the clarifier was replaced in 1979 including the scraper arms and catwalk. In addition, the influent pipe was upgraded from 8-inches to 10-inches from the headworks, and a sludge pump replaced the sludge air ejector system.

Currently, the walkway across the clarifier has noticeable corrosion damage and will require replacement. The City plans to replace the catwalk and the scraper arms in 2014. Manual cleaning of the overflow weirs is difficult and is a potential safety hazard since the clarifier is several feet aboveground and there is no walkway around the clarifier. The primary sludge pump is in moderate to poor condition and will probably need to be replaced in the near future. Seal water is not properly separated from the WWTP water supply. A backflow preventer will need to be added to the water supply line.

Parameters to evaluate the performance of the primary clarifier are the overflow rate, which is the influent flow rate to the clarifier divide by the cross-sectional area, and the hydraulic detention time. Acceptable overflow rates are 800-1200 gpd/ft^2 for average design flow and 2,000-3,000 gpd/ft^2 for peak hour flow[10]. Detention times are normally in range of 1.5-2.5 hours but can be as low as 0.5 to 1 hour for peak flows. The overflow rates and detention times for Rockaway's primary clarifier are presented in Table 4-2 below.

Table 4-2. Primary Clarifier Existing and Projected Operating Information

	Flow (MGD)	Overflow Rate (gpd/sf)	Detention Time (Hr)
Year 2012			
ADF	0.23	370	3.8
PDF	1.19	1,900	0.7
PIF	1.60	2,600	0.6
Year 2017			
ADF	0.27	440	3.3
PDF	1.27	2,100	0.7
PIF	1.75	2,800	0.5
Year 2032			
ADF	0.46	750	1.9
PDF	1.70	2,800	0.5
PIF	2.50	4,100	0.4

The capacity of the primary clarifier is adequate to flow rates of approximately 1.85 mgd. At this flow rate the overflow rate is 3,000 gpd/ft^2, which is on the high end of the acceptable range. The detention times are low for peak hour flow conditions, which greatly reduces the treatment efficiency of the clarifier, but these values are not uncommon. The overflow rate is projected at about 4,100 gpd/ft^2 for year 2032 peak hour flows, which is outside the normally accepted range. However the average day flows are below the acceptable limits, and the detention times through 2017 are above the recommended 2.5 hours maximum, which could lead to septic conditions and problems with settling. The higher overflow rates above typical values can be acceptable provided there is enough treatment downstream. The weir loading rates range from 3,000 to 30,000 gpd/ft based on projected flows.

[10] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003. Table 5-20.

The recommended weir loading rate is 10,000 to 40,000 gpd/ft. In addition the primary clarifier has no redundancy; however flow can bypass the primary clarifier to the trickling filter.

Reportedly, the primary clarifier overflowed once during a storm event several years ago when both pumps at the Main Street Pump were operating (approx. 1,200 gpm). No overflows have occurred since then. The primary clarifier discharges to the trickling filter recirculation wet well via a 5 foot long 10 inch pipe. The recirculation wet well has a 10 inch overflow pipe to the trickling filter effluent box. When flows to the plant exceed the capacity of the trickling filter recirculation pump (1 MGD), the water surface elevation (WSE) in the recirculation wet well rises to the 10 inch overflow pipe which has an invert of 26.55 feet. The top of the primary clarifier is approximately 28.53 feet, based on the design drawings. Potentially the configuration of this piping could be the cause of the overflow that occurred at the primary clarifier. Therefore, the primary clarifier effluent piping will need to be modified to allow for more headloss from the primary clarifier launder to the trickling filter recirculation wet well.

Trickling Filter Biological Treatment

The trickling filter was constructed originally in 1954 and provides biological pretreatment to the settled wastewater. The unit was renovated in 1979, which included new media, rotary distributor, and recirculation pump. The trickling filter was out of service for about 15 years until it was renovated again in the 1990's and returned to service. The media was replaced with larger rocks that have a reduced tendency to clog with biological solids.

The trickling filter is considered part of a combined trickling filter activated sludge system which was popular at the time the plant was upgraded in 1979. There are several types of combined systems that vary the sizing of the trickling filter and activated sludge process relative to one another. Although never officially designated as such, the plant at Rockaway Beach shows the characteristics of a *trickling filter solids contact process* (TF/SC), which uses a relatively large trickling filter operated at a relatively low organic loading rate and a relatively small aeration basin with short hydraulic retention times. This type of system is designed so that the trickling filter receives the majority of the BOD loading and the aeration basins flocculate and polish the trickling filter effluent prior to settling.

Rockaway's trickling filter reportedly removes 40 percent of the influent BOD which is at the low end of the range of 40-90 percent typical for trickling filters[11]. The BOD removal reduces the loading on downstream activated-sludge process and increases available plant capacity. The trickling filter has a direct feed/recirculation pump rated at 700 gpm (1 MGD), which pumps influent to the trickling filter and recirculates the flow to dilute the strength of the raw wastewater and pass it through the filter more than once. The design recirculation ratio, which is the ratio of the recirculated flow to the raw wastewater ranges from 0.7 to 1.8 which is within the common range of 0.5 to 3[12]. Wastewater flows above the capacity of the recirculation pump (700 gpm) are mixed with the trickling filter effluent and overflow to the aeration basins. With the projected 2032 peak day flow of 1.7 MGD, the recirculation ratio would be 0.6.

The design hydraulic loading rate is 290 gallons per day per square-foot (gpd/ft^2) of trickling filter surface area. This rate is at the low end of the recommend range of 230-690 gpd/ft^2 for a

[11] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003. Table 9-1.
[12] *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 538.

conventional trickling filter[13].

The organic loading rate expressed as pounds of BOD per day per 1,000 cubic feet of trickling filter volume (ppd BOD/1,000ft^3) is estimated to range from 10-20 ppd BOD/1,000ft^3 for existing conditions. The loadings for the year 2032 is estimated to range 15-30 ppd BOD/1,000ft^3. These loadings assume that the trickling filter receives the full influent flow to the WWTP. Acceptable BOD loading rates for filters combined with an activated sludge process can range 50-110 ppd BOD/1,000ft^3[14]. However the loading should be kept at the low end of the range to avoid issues with odor. The design loading for Rockaway's trickling filter is estimated to be 25 ppd BOD/1,000ft^3 for average conditions and 50 ppd BOD/1,000 ft^3 for peak conditions based on typical ranges for the TF/SC process[15].

The trickling filter is reported to be in good mechanical and structural condition, except for a leaking seal in the distribution arm. The recirculation pump is in poor condition and is a pump model that is no longer manufactured. The pump seal water is not protected by a backflow preventer to separate it from the plant city water supply. In addition the capacity of the feed/recirculation pump is undersized for peak flow conditions and does not have a backup. There is no redundant trickling filter; however the plant has a downstream activated sludge process that provides some redundancy.

The effluent flows through a 10 inch pipe from the trickling filter effluent box to the interstage pump station wet well. This pipe is undersized for projected peak day flows of 1.75 MGD for the year 2017.

Interstage Pumping and Overflow Diversion

The interstage pumping station lifts trickling filter effluent to the aeration basin splitter box. The pump station was originally constructed in 1979. The pumps were replaced with three new pumps in 2005. The new pumps are variable frequency driven. Two of the pumps have a capacity of 580 gpm (0.84 MGD) and the third pump has a capacity of 1,167 gpm (1.68 MGD). Flows greater than the pump capacity overflow from the pump station wet well to the storage lagoon. Overflow events are extremely rare. The firm capacity of the pump station with the largest pump out of service is approximately 1,160 gpm (1.68 MGD). The firm capacity will be exceeded by the estimated peak hour (2.5 MGD) flows for 2032; however there will be some attenuation from the prior treatment units so the full peak hour flow will likely not be realized. The overflow to the storage lagoon can be pumped back to the headworks of the treatment plant.

Aeration Biological Treatment

Rockaway's activated sludge biological treatment system is part of a package plant system installed in 1979 consisting of a circular concrete basin with a secondary clarifier and integral aeration basins and chlorine contact basins. The aeration basin provides polishing of the trickling filter effluent (see trickling filter discussion pg. 4-10). There are two parallel aeration basins with a volume of approximately 52,300 gallons each. The aeration basins receive flow from the interstage pump station rated at 1,160 gpm (1.7 MGD) capacity. The system is aerated with a coarse bubble diffuser system

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13 *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 542.
14 *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 556.
15 *Review of Two Decades of Experience with TF/SC Process,* Journal of Environmental Engineering, May 2001, Parker & Bratby, pg. 382.

and two air blowers rated at 300 CFM each (including backup). The aeration basins and coarse bubble diffuser systems are reportedly in good condition and have been rehabilitated several years ago. The access bridge was replaced with a galvanized steel bridge as part of the rehabilitation.

Typical BOD loading rates for an aeration tank in a conventional activated sludge process range 20-40 BOD pounds per day per 1,000 cubic feet of aeration basin volume (ppd BOD/1,000ft^3)[16]. Assuming 40 percent BOD removal from the trickling filter, the current BOD loading rate to the aeration basin ranges from 10-20 ppd BOD/1,000ft^3 assuming both tanks are in use. The loadings for the year 2032 are estimated to range 20-35 ppd BOD/1,000ft^3. Since the 40 percent BOD removal is considered at the low end of the range for BOD removal for a trickling filter, the BOD loadings to the aeration basins are considered conservative. Typical loading rates for aeration basins in the TF/SC process are not available since the system is designed to treat BOD mainly through the trickling filter.

The hydraulic detention times in Rockaway's aeration basins can range 2-11 hours based on current flows assuming both tanks are used. For the 2032 projected flows, the detention times will range 1.5-5 hours. Typical detention times for conventional activated sludge processes range 4-8 hours[17]. However, since BOD is being pre-treated by the trickling filter, the detention time can be reduced to 50-70 percent of that for conventional activated sludge processes and even lower for a trickling filter/solids contract (TF/SC) process at 10-60 minutes[18]. For combined trickling filter and activated sludge systems, it is difficult to tell how much air is actually required, since for some of these types of plants, the aeration basin is more of a flocculator than an aerator. However, it is important that adequate aeration occur so that the particles can flocculate[19]. Table 4-3 below shows the detention time or aeration period for each planning period peak day flow. The detention times are well above the range required for a TF/SC process and are below the required detention time for a conventional activated sludge process.

Table 4-3. Aeration Basin Detention Time

	Flow MGD	Aeration Period (Hrs)
2012 PDF	1.2	2.1
2017 PDF	1.3	2.0
2032 PDF	1.7	1.5

Additional information would be needed to study how effective the aeration basins are during peak day events.

Based on the plant daily monitoring reports, the mixed liquor suspended solids (MLSS) concentration ranges from 4,000 to 7,000 mg/L, which is higher than the 1,000-3,000 mg/L MLSS typical for a conventional activated sludge process and TF/SC process. The high MLSS indicates a high sludge return rate. The food to mass ratio (F/M), which is the ratio of influent BOD to the aeration basin in

[16] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 8-16.
[17] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 8-16.
[18] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, pg. 941-942.
[19] *Review of Two Decades of Experience with TF/SC Process,* Journal of Environmental Engineering, May 2001, Parker & Bratby.

pounds per day versus the mass of organisms (MLSS) in the aeration basins, ranges 0.07 to 0.2 for existing conditions. These ratios are below the range of 0.2-0.4 which is typical for a conventional activated sludge process[20] and further indicates a high sludge return rate. Typical F/M rates for TF/SC are not available, however, a case history of one plant showed an F/M of 8.8[21]. Finally, the solids retention time (SRT), which is the estimated amount of time the solids remain in the treatment plant before they are wasted to the digester or are removed as part of the effluent concentration, is approximately 14 days for existing conditions based on a sludge yield of 0.8 lb TSS per lb BOD removed[22]. Typical SRT's for a conventional activated sludge process is 3-15 days[23]. Typical SRT's for a TF/SC is 2 days[24].

The steel air distribution header located above the water surface is in poor condition due to corrosion. Also, there are no dissolved oxygen (DO) sensors in the aeration basins.

Air Blowers

The two air blowers perform multiple functions for the treatment plant. They supply air for the aeration basins, aerobic digesters, air lift pumps for return activate sludge return (RAS), and the grit removal system. An analysis of the air-blower system indicates that the blowers have sufficient capacity for the planning period. The horsepower for the blower system, based on 300 CFM, is 25 hp per blower.

With regard to the aeration basins, the following analysis was conducted. For a coarse bubble diffuser system, the oxygen transfer rate is estimated at 0.6 pounds of dissolved oxygen per blower horsepower per hour (lb DO/hp-hr). One blower has an estimated capacity of 360 lbs of DO per day for a total capacity of 720 lbs DO/day. The estimated 2032 peak day BOD load to the aeration basin after treatment from the primary clarifier and trickling filter is approximately 500 lbs/day (dry weather). Assuming 1 lb of DO required per pound of BOD, the blowers appear to have adequate capacity; however redundancy will need to be addressed toward the end of the planning period.

With regard to the aerobic digester, the blower system is capable of supplying about 30 CFM per 1000 ft^3 of digester volume, which is at the recommended range of 15-30 CFM/1000 ft^3[25].

The capacity of the blowers appears to be adequate for the planning period, though there does not appear to be any redundancy for the year 2032 estimate loads.

Secondary Clarifier

The secondary clarifier settles out the solids from the aeration basins and is the final treatment of the wastewater prior to disinfection. The sludge settles on the bottom of the tank and the supernatent

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[20] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 8-16.
[21] *A Survey of Combined Trickling Filter and Activated Sludge Process,* Journal WPCF, John Harrison, Glen Diagger, John Filbert, 1984, Table 2.
[22] *Production of High Quality Trickling Filter Effluent without Tertiary Treatment*, Parker, Norris, Daniels, Owens, Oct 1980, Table 4, pg 19.
[23] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 8-16.
[24] *Review of Two Decades of Experience with TF/SC Process,* Journal of Environmental Engineering, May 2001, Parker & Bratby pg. 384.
[25] *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 679.

flows over a weir to the disinfection basin. Solids on the bottom of the tank are either returned to the aeration basins, wasted to the aerobic digester, or pumped to the primary clarifier. Reportedly, the third option is seldom used. The return-activated sludge (RAS) is pumped to the aeration basins via air lift pumps powered by the plant air blowers. The three parameters used to evaluate secondary clarifier performance are the solids loading rate, the overflow rate, and the hydraulic detention time. Each of the parameters are discussed below.

The solids loading rate is calculated from the mixed liquor suspended solids concentration (MLSS) in mg/L and the flow rate to the clarifier plus the return activated sludge (RAS) flow. This rate, in pounds per day, is then divided by the cross sectional area of the clarifier. Solids loading rates to secondary clarifiers in general are limited to about 40 lbs/ft^2-day with 20-30 lbs/ft^2-day being ideal for design loadings[26]. From the DMR's, the mixed liquor suspended solids (MLSS) concentrations range from 4,000 to 7,000 mg/L with an average of 5,400 mg/L. The average solids concentration of the RAS is 7,700 mg/L. To obtain the 5,400 mg/L MLSS concentration, the sludge return rate needs to be over two times the average day flow. The RAS pumping capacity is 0.75 MGD. Using this information, the solids loading rates to the clarifier were estimated and projected using the design flows and are presented in Table 4-4 below. Based on this information, the solids loading rate to Rockaway's clarifier is within the typical limits allowed.

Table 4-4. Secondary Clarifier Existing and Projected Operating Information

	Flow (MGD)	RAS (MGD)	Sludge Return Rate	MLSS (mg/L)	Solids Loading (ppd/ft^2)	Overflow Rate (gpd/sf)	Det. Time (Hr)
Year 2012							
ADF	0.23	0.53	2.3	5,400	25	170	11.9
PDF	1.19	0.75	0.6	3,000	35	860	2.3
Year 2017							
ADF	0.27	0.62	2.3	5,400	29	200	10.1
PDF	1.27	0.75	0.6	2,900	35	920	2.2
Year 2032							
ADF	0.46	0.75	1.6	4,800	35	330	5.9
PDF	1.7	0.75	0.4	2,400	35	1,230	1.6

The peak overflow rate of the clarifier, which is the daily average[27] influent flow rate divided by the cross sectional area of the clarifier is approximately 900 gpd/ft^2 for year 2012 as shown in the table above. The maximum allowable for this size plant is 1,200 gpd/ft^2[28]. By the year 2032, the clarifier will have an estimated peak day overflow rate of about 1,200 gpd/ft^2 with a detention time at 1.6 hours which is less than the 2 hour minimum for secondary clarifiers. The weir loading rates range from 2,000 to 14,000 gpd/ft based on project flows. The recommended weir loading rate is 10,000 to 20,000 gpd/ft.

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26 *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 8-7.
27 Assumes peak hour flow is less than 2 hours in duration.
28 *Water Supply and Pollution Control*, Warren Viessman & Mark Hammer, 1993, pg. 355.

The secondary clarifier appears be within typical operating parameters for the existing and projected flows except for the overflow rate and detention time for the year 2032, which is marginally outside typical limits. The plant reportedly experiences issues with the secondary clarifier when transitioning from summer to winter. The plant personnel normally have to increase the RAS and waste-activated sludge (WAS) rates to counteract the changed conditions. The issue does not appear to be related to the clarifier design as it appears to be within or near typical operating parameters for solids loading, overflow rate and detention time for existing and projected flows. A possible cause may be lack of adequate aeration during peak flow events in the aeration basin. The combination of inadequate aeration, which would lead to solid particles that are more difficult to settle, with the higher flow rate, may explain the difficulty in settling during the transition from summer to winter at the treatment plant. However, the extremely long detention times during normal flows may also be an issue and a possible cause. It is recommended that a study be performed to determine what additional adjustments can be done to prevent or mitigate the issue.

The secondary clarifier does not have any redundancy; however, the plant does have a storage lagoon where flows to the secondary clarifier package plant can be bypassed to. The lagoon has a capacity of 1.6 million gallons which can storage dry weather flows for several days. The lagoon could also act as a sedimentation pond. A portable pump would need to be put in place to pump flows from the lagoon to the disinfection basin.

Disinfection

The disinfection process has been recently (2008) renovated from a chlorine gas to a liquid sodium hypochlorite facility. The old disinfection process which used chlorine gas was removed. A new 540 gallon sodium hypochlorite storage tank was installed along with a new metering pump, controller and associated piping.

The chlorine contact tanks are nested in the outside perimeter of the circular secondary treatment unit. Baffles in the tanks provide a length to width ratio of approximately 30 to 1. The tanks are adequately sized to meet the minimum contact time of 30 minutes for the 20 year planning period. The WWTP is able to consistently meet its disinfection permit limits.

Dechlorination

Dechlorination was provided as part of the Effluent Pump Station and Outfall Project completed by December 31, 2004. Dechlorination includes a sodium bisulfite storage tank and feed pump, two chlorine residual analyzers (before and after dechlorination), and in-line injection equipment. The process is effective in reducing residual chlorine to essentially zero concentration.

Effluent Pump Station and Ocean Outfall

An effluent pump station and ocean outfall were constructed in 2005. The effluent pump station is located in an area formerly occupied by tertiary effluent filters and includes two pumps, each with a capacity of 1,167 gallons per minute (1.7 MGD). Each pump has a 40 horsepower motor. The pumps transfer treated and de-chlorinated effluent to the ocean outfall during periods when the flow rates are elevated and tides are high. Otherwise, flow is normally by gravity.

The 10-inch (inside diameter) outfall begins at the effluent end of the chlorine contact basin at the Rockaway Beach WWTP and runs north through the treatment plant site to South Third Street. The outfall then turns 90 degrees to the west and runs west along the north side of South Third Street toward the ocean. Then it under crosses Oregon Highway 101 and the Port of Tillamook Railroad line within the South Third Street right-of-way. The outfall continues west under the beach and ocean to a diffuser located approximately 2,800 feet off-shore at a bottom depth of approximately 45 feet. The total length of the outfall is 5,070 feet from the treatment plant to the diffuser port. The outfall is ductile iron on shore and is high-density polyethylene enclosed in a steel casing pipe under the ocean.

The diffuser consists of a single riser with two 4-inch ports discharging in opposite directions. The ports are fitted with Red Valve Company Tideflex® check valves to exclude sand and other debris from the pipe. The Tideflex valves also increases exit velocities to promote better dispersion of the effluent. No bottom or bank attachment occurs that could inhibit dilution or expose biota to undiluted effluent. The diffuser maintains a 79:1 dilution of the effluent in ocean water. The mixing zone is 200 feet in diameter around the diffuser as determined by an outfall dilution model developed and documented in the Rockaway Beach *Ocean Outfall Preliminary Engineering Report* dated December 8, 2003. The modeling determined that water quality criteria will be met at the recommended mixing zone limits for flows up to 2 MGD. The four water quality constituents that drive the need for dilution of effluent discharge to the ocean are ammonia, chlorine, temperature, and DO. Other water quality constituents of concern are trace metals.

The sea floor in the Rockaway Beach vicinity exhibits sand movement of up to 15 feet due to seasonal and climatic conditions. The potential for sand deposition is higher at shallower depths. The length of the outfall has been selected to extend to a depth of 45 feet to reduce potential for sand deposition while still meeting DEQ preferences for minimal footprint of the mixing zone. The diffuser was designed to include removable riser segments that can be used to adjust the diffuser as the sea floor changes. Annual diving inspection of the diffuser are conducted.

Additionally, the existence of an active trawl fishery in the area was identified. Coordination with fishery representatives is required to assure that the diffuser is not damaged.

The pump station capacity and outfall are adequate for the flows for the 20 year planning period.

Aerobic Sludge Digestion and Disposal

The aerobic sludge digester provides additional biological treatment of organic sludges to stabilize and reduce the sludge volume. The aerobic digester is a concrete tank that is 28 feet in diameter with a center depth of 18 feet 6 inches and a side-water depth of 14 feet 6 inches. The maximum storage volume is 76,000 gallons. The steel walkway on top of the digester and the mixer have been removed and are scheduled to be replaced in 2013.

A parameter used to evaluate the sizing of the digester is the solids retention time (SRT). The SRT is the amount of time needed in the digester to stabilize the solids before they are treated or disposed. Since Rockaway's biosolids are disposed of at a municipal landfill, the 40 to 60 day detention time required for pathogen reduction[29] does not apply. Rather, volatile solids reduction drives the solids detention time, which can range 8-13 days at minimum based on Rockaway's average summer and

[29] *Wastewater Engineering Treatment and Reuse,* Metcalf & Eddy, 2003, Table 14-34.

winter temperatures. The actual solids detention time at Rockaway is approximately 90 days due to limitations with the solids disposal process. According to limited daily monitoring report (DMR) data, the volatile solids reduction ranged from 60 to 85 percent which is higher than the 35 to 50 percent normally achieved with aerobic digesters, but the SRT is much higher as well. The percentage of solids in the digester ranges from 1-2.5 percent, with 1.6 percent being the average based on DMR's.

The aerobic digester decant is piped to the primary clarifiers with telescoping valves. The solids are pumped to bio-bags for dewatering in the former humus pond. The solids can also be sent directly to disposal in a tanker truck. Approximately 200,000-250,000 gallons of sludge is pumped to the bio-bags per year from the digester according to the City. The pumping occurs 2-3 times per bio-bag since the permeability is reduced each time dewatering occurs. Coagulant is added to the sludge is it enters the bag. The bio-bags are located in the humus pond, which collects the leachate from the bags and sends it back to the plant. It is estimated that the percent solids increases to 20-24 percent in the biobags. Once a bag is full and is sufficiently dewatered, the bag is cut open and the sludge is allowed to air-dry prior to trucking it to a municipal landfill. Based on limited information and our experience the percentage of solids increases to 60-70 percent after air drying. This results in 20-30 dry tons of sludge produced per year. Trucking tickets for 2009 indicate about 28 tons of dry sludge was trucked to a landfill.

The digester appears to be adequately sized for the planning period. The assumption is that the solids are 80 percent volatile, and the reduction of the volatile solids is 80 percent. The SRT can remain at 90 days to 2017. By 2032, it is estimated that approximately 700,000 gallons per year of sludge at 1.6 percent solids will need to be pumped to the biobags. The SRT in the digester will be reduced to about 40 days assuming a 60 percent volatile solids reduction, and the number of biobags will need to be about 3.5 per year based on the current size of 16,400 gallons per biobag. The dry tons of sludge produced by 2032 is estimated to be approximately 80 tons per year.

Reportedly, WWTP personnel indicated the aerobic digester is undersized with little flexibility for wasting sludge. However, the City did report that the mixer and catwalk were removed, which is when the problem started occurring. These items are already scheduled to be replaced sometime in 2013.

Sludge (Humus) Pond

The sludge or humus pond provides a containment area for aerobically digested sludge to dewater and dry using bio-bags. Leachate from the bags is collected and sent back to the plant. Once a bag is full and is sufficiently dewatered, the bag is cut open and the sludge is allowed to dry prior to trucking it to a landfill.

The humus pond has a surface area of 7,676 square feet. The storage volume at a depth of 4 feet is 165,900 gallons. The pond is asphalt lined. The humus pond appears to have adequate capacity for the planning period.

Overflow Lagoon

The overflow lagoon provides storage for peak wastewater flows to reduce instantaneous flow rates during wet weather conditions. The overflow pond has a depth that varies between 1 foot and 4 feet with a total capacity of 1,597,000 gallons at maximum depth.

The overflow pond reportedly contains several feet of settled debris and is in need of dredging to restore its original capacity. The pond provides the plant with an excellent means of attenuating peak wet weather flows. A pump rated at 0.5 MGD pumps from the lagoon to the headworks.

Supervisory Control and Data Acquisition (SCADA)

Rockaway's WWTP does not have any telemetry other than an auto-dialer for the inter-stage and effluent pump stations. It is recommended that some form of telemetry be added to the plant in the 20 year planning period so that operators can better monitor processes, record measurements and be notified if an alarm condition occurs.

Support Facilities

The WWTP also includes support facilities for plant operations including:

Office Space—The treatment plant includes a small office area that was recently expanded, and is sufficient for current and future plant management operations. The plant staff indicated that additional office space is not required beyond the current expansion.

Laboratory—The plant has a laboratory area constructed as part of the 1979 expansion. The laboratory is adequate for current operating control and reporting purposes.

Utilities—Several deficiencies were noted in the site utility support systems. A number of wastewater pumps are served with seal water that is not separated from the plant drinking water system. This is a code issue that needs correction. Additional or upgraded hose bibs and plant water stations may be required. Buried process pumping is in poor condition near the aeration basins.

Electrical—A detailed evaluation of the plant electrical system was not done. Many exposed panels and receptacles are badly corroded and need replacement to avoid potential safety concerns. Site lighting could be improved to provide better and more energy efficient lighting for safe operation. The largest energy consumers at the plant are the aeration blowers. Placing these blowers on variable frequency drives may reduce energy costs.

A standby generator is available at the WWTP. The engine generator is 125 kW and is sized to power portions of the plant during power outages. The existing generator is too small to service the new facilities and will need to be replaced by a larger generator.

City of Rockaway Beach

Wastewater Facilities Plan

CHAPTER 5

Alternatives Evaluation and Recommendations for Future Improvements

CHAPTER 5 ALTERNATIVES EVALUATION AND RECOMMENDATIONS FOR FUTURE IMPROVEMENTS

This chapter identifies and evaluates alternatives to address the City of Rockaway Beach (City)wastewater collection system and wastewater treatment plant (WWTP) deficiencies identified in Chapters 3 and 4.

WASTEWATER COLLECTION SYSTEM IMPROVEMENTS

A summary of the evaluation for the improvements to the wastewater collection system are presented below.

Collection System Improvements

The collection system should be designed for adequate capacity to completely contain and transport the expected peak flows through the pipes to the WWTP. The selected design peak event for the Rockaway Beach collection system is the 5-year peak hour flow. As described in Chapter 3, the 5-year peak hour flow was analyzed for each of the flow monitoring basins in 2004 to determine the necessary upsizing requirements to the collection system. From this analysis, the entire main gravity line from the discharge manhole for the Lake Lytle Pump Station to the Main Pump Station requires additional capacity. The City has also indicated that the manholes downstream of the Lake Lytle and the NW 17th lift stations currently experience overflows when both pumps are operating during high flow events. The gravity line from the White Dove Pump Station discharge manhole located at NW 23rd and Highway 101 to the NW 17th Lift Station will require upsizing as well. Rather than upgrade the gravity sewers, the City prefers to bypass these sections with new forcemains and forcemain extensions due to the difficult soil conditions generally in the City.

Redirecting flow from the main gravity line from the discharge manhole for the Lake Lytle Pump Station to the Main Pump Station will require an upgrade/ rebuild of the Lake Lytle, NW 17th Ave., N 4th Ave., and the Main Pump Stations.

<u>Alternatives Discussion</u>

Two alternatives were evaluated to convey wastewater from the Lake Lytle, NW 17th Ave., N 4th Ave., and the Main Pump Stations to the WWTP.

The first option (Alternative 1) is to pump the wastewater from each pump station using a common forcemain. The advantage of this option is that each of the pump stations would be able to convey wastewater to the treatment plant independently and would avoid inefficiencies of pumping wastewater 2-4 times if the pump stations operated in series. Also sections of piping can be sized smaller from each pump station to where it connects into the common forcemain. The disadvantage is that the pumps will operate under varying hydraulic conditions depending on whether other pump stations are operating, which may not result in the most efficient operation of the pumps under all conditions and could require more complex controls including variable-frequency drives (VFD's).

The second option (Alternative 2) is to configure the pump stations so that they operate in series and pump directly from the upstream pump station to the downstream pump station. This option was quickly ruled out since the City prefers to be able to operate each pump station independently without having to rely on a downstream pump station to convey flow. Also, there is little-to-no advantage as far as energy savings, and this option would require an additional 340 feet of forcemain.

Table 5-1. Collection Alternatives Summary

Item	Alternative 1 (Common FM)	Alternative 2 (In Series)
6" Forcemain (ft)	1,470	1,470
8" Forcemain (ft)	440	1,320
10" Forcemain (ft)	6,180	5,240
12" Forcemain (ft)	2,600	2,500
14" Forcemain (ft)	60	560
Bore (ft)	200	200
Total Feet Forcemain	**10,950**	**11,290**
Main Pump Station (gpm)	600	2,060
4th St. Pump Station (gpm)	500	1,460
17th Ave. Pump Station (gpm)	300	960
Lake Lytle Pump Station (gpm)	660	660
Total Pump Capacity to WWTP (gpm)	**2,060**	**2,060**

The preferred alternative is to upgrade the Lake Lytle, NW 17th Ave., N 4th Ave., and the Main Pump Stations and construct a common forcemain to the treatment plant (Alternative 1). The layout for the proposed improvements is shown in Figure 5-1. Details of the improvements are discussed below.

Lake Lytle Pump Station and Forcemain Extension

The Lake Lytle Pump Station will need to be rebuilt from a self-priming suction pump setup to a submersible pump configuration. Submersible pumps are generally better designed for sewage than the self-priming pumps. The pump station will be a duplex station with one pump on standby. The recommended firm capacity of the station is 660 gpm. as shown in the table below.

Table 5-2. Lake Lytle Pump Station Capacity Derivation

Per Capita Peak Hour Flow	464	gpcd
Number of ERU's Upstream	110	
Estimated Peak Hour Flow Year 2012	101,000	gpd
	70	gpm
Additional Estimated Peak Hour Flow Year 2032	850,000	gpd
	590	gpm
Recommend Pump Station Capacity	**660**	**gpm**

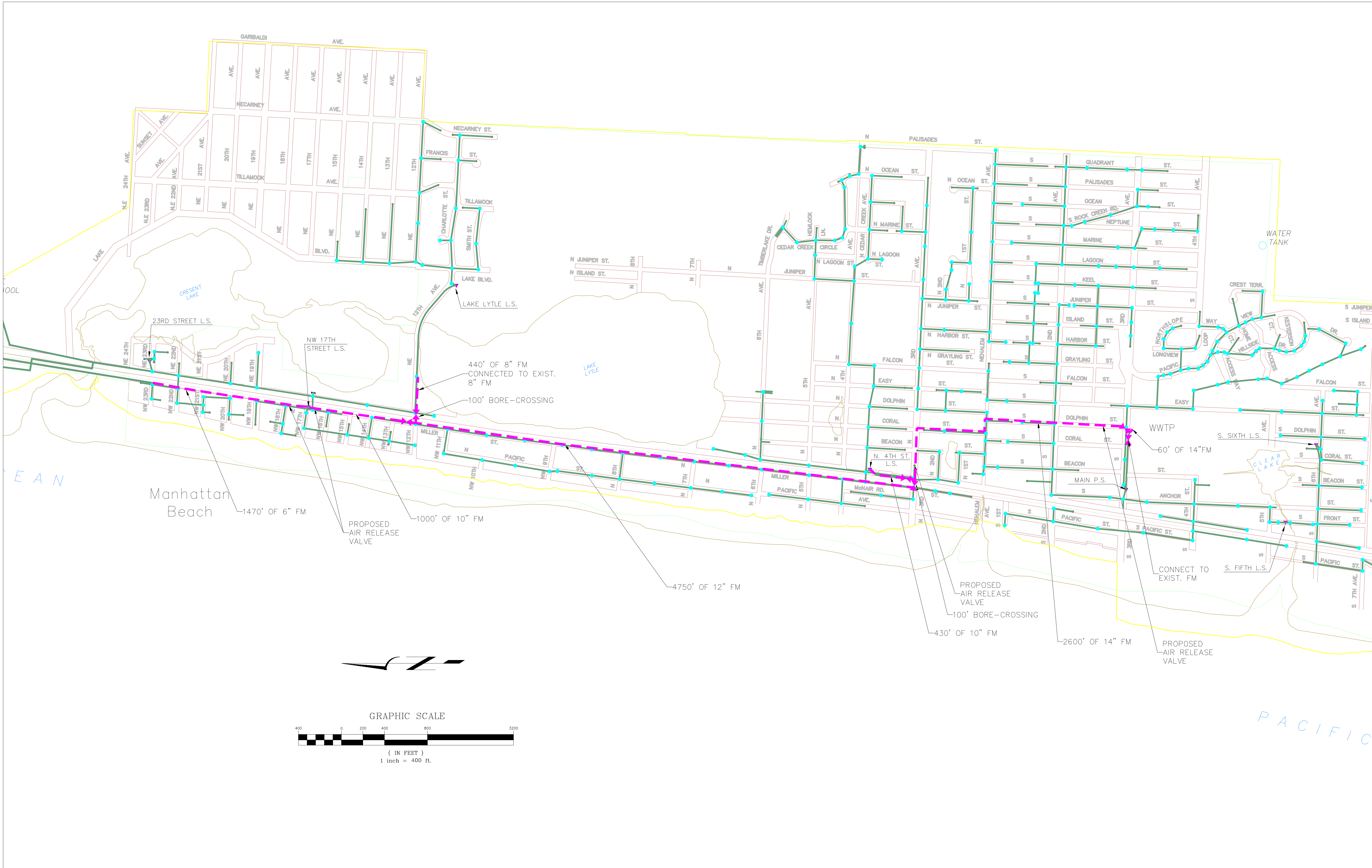
The pumps will be approximately 30 hp each and will need to operate in the estimated range of 70 to 85 feet of total dynamic head (TDH) depending on whether other pump stations are operating. The band of operating conditions for the pump is narrow enough that the pumps can operate without a VFD and still run at reasonable efficiencies.

The existing 8 foot diameter wet well is large enough to accommodate the new pumps and pumping capacities. The top of the wet well will need to be replaced to provide adequate access for the new pumps. Hatches with fall protection should be provided for safety and rinsing of the pumps over the wet well when they are removed. Guiderails should be provided for each pump to allow for removal and placement into the wet well. In addition, a wet well vent with odor control should be provided to prevent buildup of combustible gases. A backup or secondary level sensing device should be provided (i.e. backup float level sensors) to run the pumps and provide alarm notification in case the primary level sensing device malfunctions. Finally the interior of the existing wet well should be sealed with a coating designed for sewer service.

Valving for the pump station will be in a separate valve vault which will include check valves and plug valves for each pump. Since there is no overflow for this pump station to the collection system, it is recommended that a connection for temporary bypass pumping be provided. A meter vault and flow meter should be provided to help diagnose problems or issues with the pump station, monitor pump performance and to provide flow monitoring information for the sewer service area. A plug valve should be provided downstream of the flow meter to allow for removal of the flow meter.

The pump station panels and controls should be located under a lean-to type roof structure and a permanent generator should be provided for emergency power. The site should be provided with a parking area for personnel and security fencing. In addition, the pump station should be provided with telemetry so that personnel can be notified remotely when an alarm condition occurs.

The existing 8 inch forcemain will need to be extended with new 8 inch internal diameter pipe by approximately 440 feet to connect to the proposed common forcemain on Miller St. There is an existing 8 inch water line on 12th St. that the forcemain extension will need to maintain adequate separation. In addition the forcemain extension will need to avoid an existing 8 inch sewer gravity line. The extension will require a 100 foot bore-crossing under Highway 101 and the railroad tracks. The forcemain will terminate at Miller and 12th Ave where it will connect with the new forcemain from the 17th Ave. Pump Station. An isolation valve should be provided at the end of this forcemain.



H B H Consulting Engineers

CITY OF ROCKAWAY
SEWER FACILITIES PLAN

PROPOSED COMMON FORCEMAIN

FOR 17TH AVE, LAKE LYTLE, 4TH AVE, AND MAIN PUMP STATIONS

DRAWN BY: BTM

DATE: June 13, 2013

FIG. 5-1

NW 17th Ave. Pump Station and Forcemain

The NW 17th Ave. Pump Station will need to be replaced with a new pump station located at the existing pump station site in the right-of-way of Miller St. The pump station will be a duplex submersible pump station with one pump on standby. The recommended firm capacity of the station is 300 gpm (see Table 5-2). This capacity is based on an analysis of the pump station's logs and the current service area with the Lake Lytle Pump Station subtracted out. The capacity of the White Dove Pump Station, which pumps to the 17th Ave. Pump Station, is considered over capacity for the buildout population growth expected to occur in the Nedonna Beach area in the next 20 years. The flow expected to occur would be based on a population of 800 people and a peak hour flow rate of 340 gpcd, or 270,000 gpd.

Table 5-3. NW 17th Ave. Pump Station Capacity Derivation

Per Capita Peak Hour Flow	464	gpcd
Number of ERU's Upstream	130	
Residential Flow 2012[1]	119,000 83	gpd gpm
Manhattan Beach Park Flow 2012	5,000 3	gpd gpm
Neah-Kah-Ne Highschool Flow 2012	12,000 8	gpd gpm
White Dove Pump Station (Nedonna Beach) 2032	270,000 187	gpd gpm
Recommend Pump Station Capacity (rounded)	**300**	**gpm**

[1]Does not include Nedonna Beach.

The pumps for the 17th Ave Pump Station will need to be approximately 25 hp each and will need to operate in the estimated range of 50 to 70 feet of total dynamic head (TDH) depending on whether other pump stations are operating. The band of operating conditions for the pump is narrow enough that the pumps could operate without a VFD and still operate within reasonable efficiencies.

The station will have a new concrete wet well located at the existing pump station site. The wet well will need to be a minimum diameter of 7 feet and will need to be 16 feet deep. Due to the soil conditions in the area, the construction of the wet well will require shoring to protect nearby roads and railroad tracks and will likely require dewatering. Hatches with fall protection should be provided for safety and rinsing of the pumps over the wet well when they are removed. Guiderails should be provided for each pump to allow for removal and placement into the wet well. In addition a wet well vent with odor control should be provided to prevent buildup of combustible gases. A backup or secondary level sensing device should be provided (i.e. backup float level sensors) to run the pumps and provide alarm notification in case the primary level sensing device fails. Finally the interior of the wet well should be sealed with a coating designed for sewer service.

Valving for the pump station will be in a separate valve vault which will include check valves and plug valves for each pump. It is recommended that a connection for temporary bypass pumping be provided since a failure at the pump station would lead to overflows of upstream manholes. A meter vault and flow meter should be provided to help diagnose problems or issues with the pump station, monitor pump performance and to provide flow monitoring information for the sewer service area. A plug valve should be provided downstream of the flow meter to allow for removal of the flow meter.

The pump station panels and controls should be located under a lean-to type roof structure and a permanent generator should be provided for emergency power. The site should be provided with a parking area for personnel and security fencing. In addition, the pump station should be provided with telemetry so that personnel can be notified remotely when an alarm condition occurs.

The proposed forcemain for the 17th Ave. Pump Station will be about 1,000 feet with an alignment along Miller St. being the preferred alignment in order to avoid construction along Highway 101. The size of the forcemain will be 8 inches internal diameter. There is an existing 10 inch water line on Miller St. that the forcemain will need to maintain adequate separation from. In addition the forcemain will need to avoid an existing 8 inch sewer gravity line. The forcemain will terminate at 12th Ave. where it will connect with the new forcemain from the Lake Lytle Pump Station. An isolation valve should be provided at the end of this forcemain. In addition, combination air-release valves should be located at the high points of the forcemain, which is estimated to be at the start of the forcemain.

Proposed 10 inch Common Forcemain from 12th Ave. & Miller St. to N 3rd Ave. & Highway 101

The proposed common forcemain from the junction of the 17th Ave. Pump Station and Lake Lytle Pump Station forcemains to the intersection of N 3rd Ave. and Highway 101, will be about 4,750 feet with an alignment along Miller St. being the preferred alignment in order to avoid construction along Highway 101. The size of the forcemain will be 10 inches internal diameter. There is an existing 10 inch water line on Miller St. that the forcemain will need to maintain adequate separation from. In addition the forcemain will need to avoid an existing 8 inch sewer gravity line. The line will require a 100 foot bore-crossing under Highway 101 and the railroad tracks. This forcemain section will connect with the new forcemain from the NW 4th Ave. Pump Station. An isolation valve should be provided at the end of this forcemain.

N 4th Ave. Pump Station and Forcemain

The N 4th Ave. Pump Station will need to be rebuilt using the existing wet well. The pump station will be a duplex station with one pump on standby, similar to its current configuration. The recommended firm capacity of the station is 500 gpm. This capacity is based on the current estimated peak hour flows of approximately 250 gpm (estimated from analysis of the upstream service area and the pump station logs) plus an additional 250 gpm to account for growth in the service area of the pump station for the next 20 years as shown in the table below.

Table 5-4. N 4th Ave. Pump Station Capacity Derivation

Per Capita Peak Hour Flow	464	gpcd
Number of ERU's Upstream	320	
Estimated Peak Hour Flow Residential 2012	293,000 203	gpd gpm
Estimated Peak Hour Flow Commercial 2012	75,000 52	gpd gpm
Additional Estimated Peak Hour Flow 2032	360,000 250	gpd gpm
Recommend Pump Station Capacity (rounded)	**500**	**gpm**

The pumps for the 4th Ave Pump Station will need to be approximately 12 hp each and will need to operate in the estimated range of 30 to 45 feet of total dynamic head (TDH) depending on whether other pump stations are operating. The band of operating conditions for the pump is narrow enough that the pumps could operate without a VFD and still operate within reasonable efficiencies. The existing 8.5 foot diameter wet well is large enough to accommodate the new pumps. The top of the wet well will need to be replaced or retrofitted with new hatches that are lighter in weight than the existing hatches and that will provide adequate access for the new pumps. Hatches with fall protection should be provided for safety and rinsing of the pumps over the wet well when they are removed. Guiderails should be provided for each pump to allow for removal and placement into the wet well. In addition, a wet well vent with odor control should be provided to prevent buildup of combustible gases. A backup or secondary level sensing device should be provided (i.e. backup float level sensors) to run the pumps and provide alarm notification in case the primary level sensing device malfunctions. Finally the interior of the existing wet well should be sealed with a coating designed for sewer service.

Valving for the pump station will be in a separate valve vault which will include check valves and plug valves for each pump. It is recommended that a connection for temporary bypass pumping be provided since there is no overflow connection to the collection system. A meter vault and flow meter should be provided to help diagnose problems or issues with the pump station, monitor pump performance and to provide flow monitoring information for the sewer service area. A plug valve should be provided downstream of the flow meter to allow for removal of the flow meter.

The pump station panels and controls should be located in a CMU building and a permanent generator should be provided for emergency power. The site should be provided with a parking area for personnel and security fencing. In addition, the pump station should be provided with telemetry so that personnel can be notified remotely when an alarm condition occurs.

The proposed forcemain for the 4th Ave. Pump Station will be about 430 feet and will follow the alignment of the existing 6" forcemain along Highway 101. The size of the new forcemain will be 10 inches internal diameter. There is an existing 8 inch water line on the east side of Highway 101. that the forcemain will need to maintain adequate separation from. In addition the forcemain will need to avoid an existing 8 inch sewer gravity line. The forcemain will terminate at 3rd Ave. where it will connect with the new 12 inch common forcemain. An isolation valve should be provided at the end of this forcemain.

Proposed 12 inch and 14 inch Common Forcemain from N 3rd Ave. & Highway 101 to WWTP Headworks

The proposed common forcemain from the junction with the 4th Ave. Pump Station forcemain to the wastewater treatment plant (WWTP) headworks, will be about 2,600 feet with an alignment along Coral and Dolphin St. being the preferred alignment in order to avoid construction along Highway 101. The size of the forcemain will be 12 inches internal diameter. The alignment will cross approximately 5 waterlines. There are also existing parallel water lines on 3rd Ave., Nehalem Ave., and Dolphin St. that the forcemain will need to maintain adequate separation from. In addition the forcemain will need to avoid existing sewer gravity lines along 3rd Ave., Coral St., Nehalem Ave., and Dolphin St. This forcemain section will connect with the existing forcemain from Main Pump Station near the intersection of Dolphin St. and S 3rd Ave. An isolation valve should be provided upstream of this connection. In addition, combination air-release valves should be located at the high points of the forcemain, which is estimated to be at two locations. From the junction with the Main Pump Station forcemain, the a proposed 14 inch common forcemain about 50 feet long will have an alignment towards the headworks of the wastewater treatment plant and will connect to the existing influent channel of the headworks.

Main Pump Station

The Main Pump Station will need to be rebuilt and converted from a dry-pit configuration to a submersible pump layout. The station will be a duplex station with one pump on standby. The recommended firm capacity of the station is 600 gpm. This capacity is based on the current estimated peak hour flows estimated from analysis of the upstream service area and the pump station logs. No growth is expected to occur in the service area of the pump station over the planning period.

Table 5-5. Main Pump Station Capacity Derivation

Per Capita Peak Hour Flow	464	gpcd
Number of ERU's Upstream	770	
Estimated Peak Hour Flow Residential 2012	704,000 489	gpd gpm
Estimated Peak Hour Flow Commercial 2012	105,000 73	gpd gpm
Recommend Pump Station Capacity (rounded)	**600**	**gpm**

The pumps for the Main Pump Station will need to be approximately 12 hp each and will need to operate in the estimated range of 30 to 40 feet of total dynamic head (TDH) depending on whether other pump stations are operating. The band of operating conditions for the pump is narrow enough that the pumps could operate without a VFD and still operate within reasonable efficiencies.

The existing dry-pit is housed in a concrete caisson that is 10.5 feet in diameter that can possibly be converted to a wetwell large enough to accommodate the new pumps. The integrity of the caisson will need to be evaluated. The top of the caisson will need to be replaced to provide adequate access for the new pumps. Hatches with fall protection should be provided for safety and rinsing of the pumps over the wet well when they are removed. Guiderails should be provided for each pump to allow for removal and placement into the wet well. In addition, a wet well vent with odor control should be provided to prevent buildup of combustible gases. A backup or secondary level sensing device should be provided (i.e. backup float level sensors) to run the pumps and provide alarm notification in case the primary level sensing device malfunctions. Finally the interior of the existing wet well should be sealed with a coating designed for sewer service.

Valving for the pump station will be in a separate valve vault which will include check valves and plug valves for each pump. It is recommended that a connection for temporary bypass pumping be provided since there is no overflow connection to the collection system. A meter vault and flow meter should be provided to help diagnose problems or issues with the pump station, monitor pump performance and to provide flow monitoring information for the sewer service area. A plug valve should be provided downstream of the flow meter to allow for removal of the flow meter.

The pump station panels and controls should be located in a new CMU building or a lean-to roof structure. A new permanent generator rated at approximately 60 kW should be provided for emergency power. The site should be provided with a parking area for personnel and security fencing. In addition, the pump station should be provided with telemetry so that personnel can be notified remotely when an alarm condition occurs.

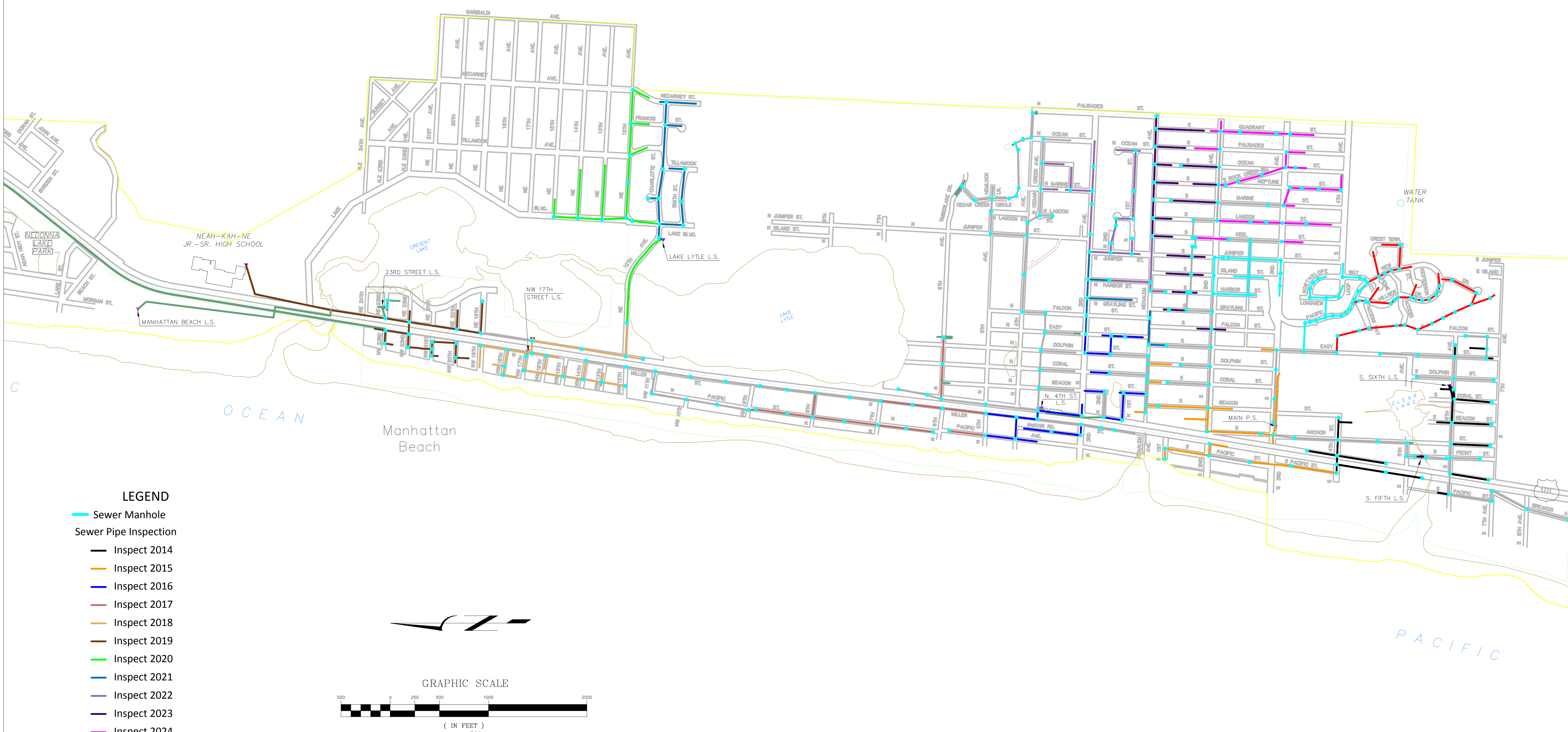
The pump station will utilize the existing 8 inch forcemain to the treatment plant. The forcemain will need to connect to the proposed 14 inch common forcemain on S 3rd Ave South of Dolphin St. An isolation valve should be provided at the end of this forcemain.

<u>Proposed Extension of 6 inch Forcemain for White Dove Pump Station</u>

The proposed extension of the White Dove Pump Station forcemain from near the intersection of NW 23rd Ave. and Highway 101 to NW 17th Pump Station, will be about 1,470 feet long with an alignment in the right-of-way west of the railroad tracks so as to avoid construction on Highway 101. The size of the forcemain will be 6 inches internal diameter. This forcemain section will connect to the wet well of the new NW17th Pump Station. The forcemain will parallel an existing 6 inch water line, and sections of the forcemain will parallel existing gravity sewer lines. The duty point of the existing pumps at the White Dove Station is estimated to change from 430 gpm at 126 feet total dynamic head (TDH) to 400 gpm at 128 feet TDH; therefore it appears the existing pumps can remain.

Maintenance

The City performs annual video inspections and cleaning of sewer piping. Point repairs are performed where there are structural problems with the main piping and manholes. Smoke testing has been done on service laterals to determine their integrity. It is recommended, that the City adopt a more formal maintenance program to map out where the pipe inspections will occur. Figure 5-2 indicates where inspections have occurred since 2007 and maps out where future inspections should occur. The City has an annual budget of $10,000 for sewer inspection and repair work. Assuming approximately $5,000 is used for sewer inspections, approximately 5,000 feet of pipe can be inspected per year. Several areas of sewers with structural deficiencies were discussed in Chapter 3. The most significant problems were lines that had sags in them. There were also a few isolated areas with structural defects such as holes or displaced joints. These areas are generally repaired as part of the ongoing sewer maintenance and repair program.



LEGEND

Sewer Manhole

Sewer Pipe Inspection

- Inspect 2014
- Inspect 2015
- Inspect 2016
- Inspect 2017
- Inspect 2018
- Inspect 2019
- Inspect 2020
- Inspect 2021
- Inspect 2022
- Inspect 2023
- Inspect 2024
- Inspect 2025
- Inspect 2026
- Inspected
- Sewer pipe

HBH Consulting Engineers

CITY OF ROCKAWAY
SEWER FACILITIES PLAN

INSPECTION AND CLEANING SCHEDULE FOR SEWER COLLECTION SYSTEM

DRAWN BY: BTM

DATE: June 13, 2013

FIG. 5-2

In general, infiltration has not been a major problem in Rockaway Beach although service lateral connections for the older AC pipe generally show signs of infiltration where they were videoed. Inflow through flooded sections of the sewer system has caused most of the high flow problems in the sewers and at the Rockaway Beach Wastewater Treatment Plant (WWTP). It is recommended that the City install manhole inserts for the areas that are prone to flooding.

The City has a 760 foot section of 8-inch cast iron or ductile iron pipe without the cement mortar lining located on the right-of-way for NW 4th Ave. between Falcon and Juniper St. The pipe is in poor condition and receives a lot of I&I. It is recommended the City replace this section of piping. The downstream section of piping crosses a wetland so trenchless pipe replacement would be needed for that section (404 feet). Since it is unknown with any certainty whether the piping is cast iron or ductile, it is recommended that the pipe be slip-lined with cured in-place pipe. The upper section of pipe can be dug and replaced since it does not appear to be in a wetland area according to the Department of State Lands Wetlands Inventory (see appendix).

Other Pumping Station Improvements

The basic criteria used in the analysis for the Rockaway Beach collection pumping systems, are that the pump stations will have:

- Firm capacity to meet estimated peak design flows (i.e., with the largest pumping unit out of service).
- Reliable equipment in good operating condition that is easy to maintain.
- The capability to operate in case electric power to the station is lost.
- The ability to notify operating personnel in case of problems at the facility.

Some of the stations are approaching 20 years in age, which is the point at which equipment may need to be replaced or may require major service. The White Dove station is the only station with the ability to notify personnel in case problems arise via telemetry. Currently, if the operating personnel are aware of a power outage in any of the other stations, a portable electric generator is used except for the Main, NW 23 Ave. and the White Dove pumping stations, which each have permanent electric generators. No upgrades in pumping capacity are anticipated for the planning period for the pump stations discussed below. A description and the recommended upgrades for the pumping stations are summarized below:

<u>South Sixth Avenue Station</u>

The pump station was rebuilt in 1990 and will likely need to be rebuilt again sometime during this planning period due to age. Pressure gauges should be added in the pump discharge line in the valve vault so that pump performance can be monitored. The wet well should be thoroughly inspected and any deterioration repaired. The entire interior surface should then be coated with an epoxy or urethane type coating. Telemetry should be added to this station so that the station can be monitored. The wet well and valve vault hatches should be equipped with better seals so that surface water can be kept out when the area floods.

South Fifth Avenue Station

The South Fifth Avenue Station was last rebuilt over 30 years ago. It is recommended that this station be rebuilt with a new submersible pump station.

The top of the existing wet well will need to be replaced to provide adequate access for the new pumps. Hatches with fall protection should be provided for safety and rinsing of the pumps over the wet well when they are removed. Guiderails should be provided for each pump to allow for removal and placement into the wet well. In addition, a wet well vent with odor control should be provided to prevent buildup of combustible gases. A backup or secondary level sensing device should be provided (i.e. backup float level sensors) to run the pumps and provide alarm notification in case the primary level sensing device malfunctions. Finally the interior of the existing wet well should be sealed with a coating designed for sewer service.

Valving for the pump station will be in a separate valve vault which will include check valves and plug valves for each pump. Since there is no overflow for this pump station to the collection system, it is recommended that a connection for temporary bypass pumping be provided. A meter vault and flow meter should be provided to help diagnose problems or issues with the pump station, monitor pump performance and to provide flow monitoring information for the sewer service area. A plug valve should be provided downstream of the flow meter to allow for removal of the flow meter.

The pump station panels and controls should be located under a lean-to type roof structure and a permanent generator should be provided for emergency power. The site should be provided with a parking area for personnel and security fencing. In addition, the pump station should be provided with telemetry so that personnel can be notified remotely when an alarm condition occurs.

Northeast 23rd Avenue Station

Telemetry should be added to this station so that the station can be monitored. The station should be fenced.

Private Lift Stations

It is recommended that the City enter into discussions with both the school district and the highway department to develop policies in case of sewage spills from the high school or highway wayside lift stations or their forcemains. The City should be in a position to assist in the corrective actions necessary to minimize any sewage spills if they are promptly notified. They can also minimize the potential health hazards resulting from a spill from these facilities by promptly cleaning up after the spill, using the City's contingency plans for spills.

WASTEWATER TREATMENT SYSTEM IMPROVEMENTS

This section discusses the evaluation of the existing WWTP processes and presents recommended improvements.

Regional Facilities Evaluation

In 2002, a Regional Wastewater Treatment and Joint Ocean Outfall Study was conducted to review the cost-effectiveness and administrative requirements necessary for the City and the Twin Rocks Sanitary District to jointly own and operate a WWTP and/or an ocean outfall. It was jointly funded by the City and District.

The basis of planning for this study was the same as that used for each entity's (City and District) wastewater facility plans. A 20-year planning period was used for all of the facilities considered in the study. The study used a planning period ending in 2022.

The treatment alternatives considered in the study are generally described as follows:

> Alternative A: City and District improve their own plants. Each discharges to a joint outfall through an outfall pump station.
>
> Alternative B: City pumps all wastewater to District for treatment. Outfall pump station located at District plant and pumps to joint outfall.
>
> Alternative C: District pumps all wastewater to City for treatment. Outfall pump station located at City plant and pumps to joint outfall.
>
> Alternative D: Construct a new joint WWTP. Outfall pump station located at the new plant site. This is similar to Alternative B, so not considered further.
>
> Alternative E: City and District improve their own plants and each constructs its own ocean outfall.

Costs for each of the alternatives and rate structures to equitably distribute the capital and operational costs were developed. Alternative A was the least cost alternative and was the recommended alternative in the report. However, each entity has subsequently chosen Alternative E.

Rockaway Beach WWTP Recommended Improvements and Alternatives

The improvements recommended for the Rockaway Beach WWTP are needed to expand current processes, upgrade aging equipment, improve reliability, improve hydraulics and extend the useful life of the facility. Two options were evaluated for secondary treatment. One option is to upgrade the existing trickling filter (Alternative TF-1). The other option is to demolish the trickling filter and expand the capacity of the existing activated sludge process (Alternative AS-1).

Primary Clarifier

The primary clarifier sludge pump will need to be replaced with a model that does not require seal water. The 10-inch effluent pipe from the launder to the trickling filter recirculation wet well will need to be modified. The elevation of this pipe should be lowered, as well as the invert for the 10 inch overflow pipe in the recirculation wet well. The influent piping from the headworks to the primary clarifiers will also need to be upsized to accommodate future peak hour flows. A metal walkway should be constructed around the perimeter of the clarifier to provide personnel with the ability to safely access the weirs.

These items should be done before all of the proposed pump station modifications and common forcemain construction which feed directly to the plant. These modifications should also be coordinated with current City Plans to replace the clarifier's catwalk and scraper system in 2014.

The surface overflow rates and detention times for 2032 peak hour flow indicate an additional primary clarifier could be needed for the planning period; however the surface overflow rate at the average 2032 flow is below the typical range. Since the existing and projected average flows are below the operating range for the primary clarifier, it is recommended to maximize available treatment capacities of the existing downstream biological treatment units for peak conditions.

Alternative TF-1 - (Trickling Filter Upgrade)

This option for the biological treatment involves upgrading the existing trickling filter as described below.

Table 5-6. Alternative TF-1 Design Data

Process or Design Criteria	Unit	Old Value	New Value
Trickling Filter	number	1	1
Recirculation Pump	number/MGD/HP	1/1.0/10	**2**/1.0/10
Diameter	feet	66	66
Surface Area	sq. ft.	3,420	3,420
Depth	feet	8	8
Volume	gallons	205,000	205,000
Media	-	Rock	**Plastic**
Distribution Type	-	Rotary	Rotary
Hydraulic Load	GPD/sq. ft.	292	**584**
	MGD	1.0	**2.0**
BOD Capacity			
Avg. BOD Load	PPD	680	**3,400**
	PPD/1000 cu. ft.	25	**125**
Peak BOD Load	PPD	1,370	**5,200**
	PPD/1000 cu. ft.	50	**190**

Trickling Filter:

Under Alternative TF-1, the direct feed/recirculation pump would be replaced by a new tandem pumping system to meet reliability requirements. The capacity of the pumps would be 1 MGD each with variable frequency drives (VFD's) to adjust the speed of the pumps. The trickling filter rotating distribution system will likely need to be replaced with one that can handle the increased flow from both feed pumps (2 MGD). The influent piping and the recirculation pipe would be upsized as well. The trickling filter media would be replaced with plastic media to help increase the air circulation and provide better treatment. The existing 10 inch pipe from the trickling filter effluent box to the interstage pump station wet well would need to be upsized in order to avoid potentially overflowing the effluent box during future peak hour flows.

Alternative AS-1 - (Expanded Activated Sludge Process)

This option involves demolishing the existing trickling filter and expanding the existing activated sludge process as described below.

Table 5-7. Alternative AS-1 Design Data

Process or Design Criteria	Unit	Old Value	New Value
Trickling Filter	number	1	-
Recirculation Pump	number/MGD/HP	1/1.0/10	-
Diameter	feet	66	-
Surface Area	sq. ft.	3,420	-
Depth	feet	8	-
Volume	gallons	205,000	-
Media	-	Rock	-
Distribution Type	-	Rotary	-
Hydraulic Load	GPD/sq. ft.	292	-
BOD Capacity			-
Avg. BOD Load	PPD	680	-
	PPD/1000 cu. ft.	25	-
Peak BOD Load	PPD	1,370	-
	PPD/1000 cu. ft.	50	-

Table 5-7. Alternative AS-1 Design Data (cont.)

Process or Design Criteria	Unit	Old Value	New Value
Aeration Basins (donut type with integral clarifier)	number	2	4
Process	-	Trickling Filter/ Solids Contact	**Activated Sludge**
Depth	feet	12	12
Total Volume	gallons	104,000	**218,000**
Diffuser Type	-	Coarse Bubble	**Fine Bubble**
Hydraulic Capacity	MGD	2.50	**1.75**
Detention Time	hours	1	**3**
BOD Capacity			
Avg. BOD Load	PPD	-	**600**
	PPD/1000 cu. ft.	-	**20**
Peak BOD Load	PPD	-	**1,200**
	PPD/1000 cu. ft.	-	**40**
MLSS	mg/L	1,000-3,000	1,000-3,000
SRT	days	14	**20**
Aeration Basin/ Digester Blowers	number	2	2
Blower 1	CFM/HP	300/25	300/25
Blower 2 (Standby)	CFM/HP	300/25	300/25
Oxygen Transfer Rate	lb O2/HP/hr	0.6	**2.0**
Oxygen Transfer Capacity	PPD BOD	720	**2,400**

Trickling Filter:

Under this option, the trickling filter would be demolished to make room for a new aeration basin. A new pipe would need to be installed to connect the primary clarifier to the inter-stage pump station as the existing bypass lines would not provide enough capacity for the planning period.

Aeration Basin:

The existing aeration basin would need to be expanded to over double in size to accommodate the planning period loads and flows. A new aeration basin would be built north of the existing packaged plant. The advantage of this location is that the existing aeration distribution box can be pipe relatively easily to the new aeration basin and then pumped back to the clarifier. The new basin would consist of two concrete tanks with a volume of 57,000 gallons each for a total volume of 114,000 gallons. This volume combined with the existing aeration basins (for a total of 218,000 gallons) would allow a BOD loading of 30 ppd/1000 ft^3 for the year 2032 peak BOD and a detention time of 3 hours for the 2032 peak flow.

The existing activated sludge process would have enough capacity for existing average BOD loadings during construction, and the plant has a 1.6 million gallon overflow lagoon it can utilize for higher BOD loadings and for shutting down the existing package plant temporarily during construction.

Air Blowers:

Two blowers would need to be added to provide air for the new air basin and for redundancy of the existing two blowers. Adjustable speed controllers would be added to all the blowers to adjust the air supply in proportion to the flow to conserve energy. A preferred option in lieu of buying additional blowers is replacing the existing course-bubble air diffusers in the existing aeration basin, with a fine-bubble air system that covers all of the floor area of the basin. The fine air bubbler system would increase the oxygen transfer efficiency to about 20-30 percent versus the current of efficiency of 4-6 percent typical for the coarse air system in the basin currently. This option would also conserve energy and the costs for installation may be partially covered by the electric power utility. The costs of installing the fine-air diffuser versus a new blower would be comparable.

<u>Secondary Clarifier</u>

Maintenance, such as painting and mechanical adjustment, to the existing secondary clarifier would be needed under this option. In addition, the return-activated sludge (RAS) airlift pumps would need to be replaced with centrifugal pumps or horizontal propeller pumps to provide more flexibility and have the RAS return independent of the air system.

<u>Aerobic Digester</u>

Once the new mixer is in (scheduled for 2013), the digestion should improve and decanting should be more effective.

<u>Miscellaneous</u>

The process yard piping and valving should be inspected and repaired as needed. Since the wastewater treatment plant is located in a residential area and has received complaints regarding aesthetics, it is recommended that landscaping be done around the treatment plant. In addition, the fencing and gate should be replaced with new black vinyl-coating fencing.

The air piping in the existing aeration basins from above the water level back to the blower building is in poor condition and would need to be replaced. Dissolved oxygen monitors would be added to ensure adequate oxygen is provided and to help conserve energy.

Although a detailed electrical review was not conducted, there is significant corrosion on some of the electrical panels and equipment. An evaluation should be done on the existing electrical panels and equipment to determine what equipment should be replaced and the costs. This evaluation should take into consideration the addition of a SCADA system to the treatment plant in the future.

A Supervisory Control and Data Acquisition (SCADA) system should be evaluated and implemented at the treatment plant. A SCADA system would essentially computerize the treatment plant and enable staff to monitor, log and control treatment processes in a centralized location.

Benefits include better efficiency and optimization of the plant operation and reduction in maintenance costs. The SCADA system can vary in size and scope, but for planning purposes it is assumed several program logic controllers (PLC's) networked together with multiple operator interfaces would be required. Since Rockaway does not have an existing SCADA system, the most economical way to install one at the treatment plant would be to contract with a SCADA system manufacture to design and build the system.
The existing 125 kW emergency generator is too small for operation of the entire treatment plant and should be replaced with a larger generator. Preliminary sizing for the new generator for planning purposes is 275 kW.

PROJECT COSTS

Precision of Cost Estimates

The precision of a cost estimate is a function of the detail to which alternatives are developed and the techniques used in preparing the actual estimate. The American Association of Cost Engineers divides estimates into three basic categories:

Order-of-Magnitude Estimate. An order-of-magnitude estimate is made without detailed engineering data. Techniques such as cost-capacity curves, scale-up or scale-down factors and ratios are used in developing this type of estimate. This type of estimate is normally accurate within +50 percent or -30 percent. That is, the final cost may be as much as 50 percent more or 30 percent less than the estimated amount. A relatively large contingency is normally included to reduce the probability of underestimating.

Budget Estimate. This estimate is prepared using process flow sheets, layouts, and equipment details. An estimate of this type is usually accurate within +30 percent and -15 percent.

Definitive Estimate. As the name implies, this estimate is prepared from well-defined engineering data, including construction plans and specifications. At a minimum, the data would include comprehensive plot plans and elevations, piping and instrument diagrams, electrical diagrams, equipment data sheets and quotations, structural drawings, soil data and drawings, and a complete set of specifications. The definitive estimate is expected to be accurate within +15 percent and -5 percent.

The estimates presented in this document are a little more detailed since the recommendations are more definitive without much alternative analysis and are therefore considered budget estimates. The final project cost may vary from these estimates.

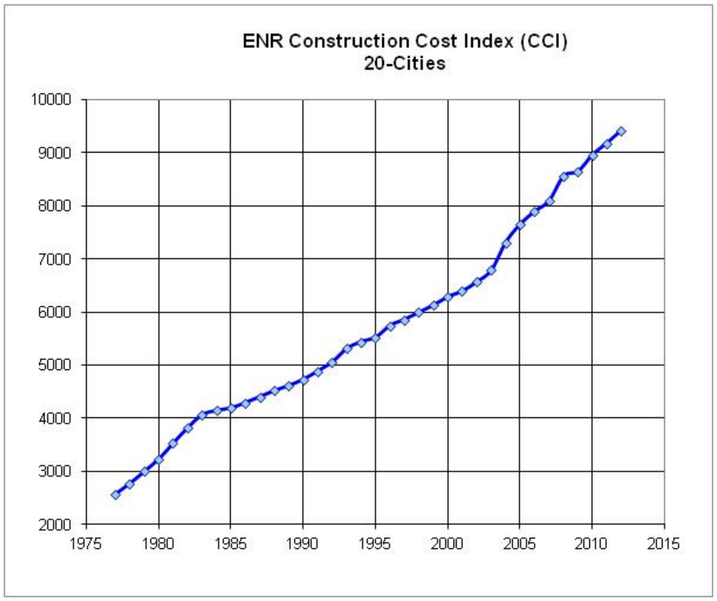
Basis for Costs over Time

Future changes in the costs of material, labor, and equipment will cause comparable changes in the costs presented in this analysis. However, because the relative economy of the alternatives should change only slightly with overall economic changes, the decisions based on the economic evaluation should remain valid.

Costs can be expected to undergo long-term changes in keeping with corresponding changes in the national economy. One of the best available indicators of these changes is the Engineering News

Record (ENR) construction cost index. It is computed from the prices for structural steel, Portland cement, lumber, and common labor, and is based on a value of 100 in 1913. Figure 5-3 shows the trend of the ENR index since 1977.

Figure 5-3. ENR Construction Cost Index Trend



The costs developed in this analysis are based on the current ENR 20-city index of 9412 (December, 2012). The costs presented here may be related to those at any time in the past or future by applying the ratio of the then-prevailing cost index to 9412. The ENR has essentially doubled in the last 20 years. Future trends are not predictable.

Engineering and Administrative Costs and Contingencies

The cost of engineering services for major projects typically covers special investigations, a predesign report, surveying, foundation exploration, preparation of contract drawings and specifications, construction management, start-up services, the preparation of operation and maintenance manuals, and performance certifications. Depending on the size and type of project, engineering costs may range from 15 to 20 percent of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects and projects that involve extensive remodeling of existing facilities.

The City will have administrative costs associated with any construction project. These include internal planning and budgeting, the administration of engineering and construction contracts, legal services, and liaison with regulatory and funding agencies. Typically, the City's administrative costs are expected to be about 5 percent of the contract cost. The total cost for engineering and administration is assumed to be 20-25 percent. Contingencies cover unexpected costs associated with a project and are typically 20 percent of the construction, engineering and administration costs.

COMPARISON OF ALTERNATIVES

Tables 5-8 and 5-9 below provide capital costs for Alternatives TF-1 and AS-1. The estimated capital cost for upgrading the existing trickling filter versus Alternative AS-1 is less expensive by approximately $300,000. The O&M costs for the two alternatives would be considered the same since both require pumping of wastewater either at the beginning or end of the process and both require a similar amount of blower horsepower for aeration (i.e. no additional blowers would be required for Alternative AS-1 because of the installation of fine-air diffusers for existing and new aeration basins).

Table 5-8. Alternative TF-1 Capital Cost Estimate

Description	Construction Cost	Engineer, Legal & Admin. Cost (25%)	Contingency	Total Cost (rounded)
Trickling Filter Rebuild and Influent/ Effluent Pipe Upgrade.	$672,000	$168,000	$168,000	$1,008,000
Total	$672,000	$168,000	$168,000	**$1,008,000**

Table 5-9. Alternative AS-1 Capital Cost Estimate

Description	Construction Cost	Engineer, Legal & Admin. Cost (25%)	Contingency	Total Cost (rounded)
Trickling Filter Demolition. New pipe from Primary Clarifier to Inter-stage Pump Station.	$173,000	$43,250	$43,250	$260,000
New Aeration Basins	$630,000	$157,500	$157,500	$945,000
Replace Ex. Coarse Air Diffuser System with Fine-Air Diffuser. Provide VFD's for Existing Blowers.	$85,000	$21,250	$21,250	$128,000
Total	$888,000	$222,000	$222,000	**$1,333,000**

Table 5-10 below summarizes the capacity of the existing wastewater treatment plant with each alternative.

Table 5-10. Alternatives Capacity Comparison

Process or Design Criteria	Unit	Existing	Alternative TF-1	Alternative AS-1
Flow Capacity				
Average Dry Weather Flow (ADWF)	MGD	0.6	0.6	0.6
Maximum Hydraulic Capacity	MGD	1.7	1.7	1.7
Loading Capacity				
Avg BOD	PPD	750	750	900
Avg TSS	PPD	750	750	900

Table 5-10. Alternatives Capacity Comparison (cont.)

Process or Design Criteria	Unit	Existing	Alternative TF-1	Alternative AS-1
Headworks				
Peak Instantaneous Flow (PIF) Capacity	MGD	1.7	1.7	1.7
Primary Clarifier				
Peak Overflow Rate	GPD/sq. ft.	3,000	3,000	3,000
PIF Capacity	MGD	1.85	1.85	1.85
Trickling Filter	number	1	1	0
Recirculation Pump	number/MGD	1/1.0	2/2.0	-
Hydraulic Load	GPD/sq. ft.	292	584	-
Flow Capacity	MGD	1.0	2.0	-
BOD Capacity	PPD	820	5,200	-
In-plant Pump Station				
Firm Capacity	MGD	1.68	1.68	1.68
Aeration Basins	number	2	2	4
Total Volume	gallons	104,000	104,000	218,000
Peak Day Flow (PDF) Capacity	MGD	2.5	2.5	1.75
Hydraulic Detention Time	hrs	1	1	3
Peak BOD Load	PPD	-	-	1,200
SRT	Days	14	14	20
Aeration Basin/ Digester Blowers				
Blower 1	CFM/HP	300/25	300/25	300/25
Blower 2 (Standby)	CFM/HP	300/25	300/25	300/25
Oxygen Transfer Rate	lb O2/HP/hr	0.6	0.6	2.0
Oxygen Transfer Capacity	PPD BOD	720	720	2,400
Secondary Clarifier				
Peak Overflow Rate	GPD/sq. ft	1,200	1,200	1,200
PDF Capacity	MGD	1.66	1.66	1.66
Sludge Yield	lb TSS/ lb BOD Removed	0.8	0.8	0.5
Chlorine Contact Tank				
Flow Capacity based on 0.5 hr detention	MGD	1.85	1.85	1.85
Effluent Pump Station				
Firm Capacity	MGD	1.68	1.68	1.68
Aerbobic Digester				
Solids Capacity	PPD	800	800	800

The overall hydraulic capacity will not change between the alternatives since the capacity is limited by the plant yard piping as described in Chapter 4. The overall BOD and TSS capacity of the plant could potentially increase with Alternative AS-1 due to the better sludge yield (based on typical values) between the trickling filter (0.8 lb TSS per lb BOD removed) and activated sludge process (0.5 lb TSS per lb BOD removed). However, when the individual biological processes are evaluated, Alternative TF-1 has a much higher BOD capacity at 5,200 ppd versus Alternative AS-1 which has only 1,200 ppd. These numbers are based on a loading rate of 190 ppd BOD/1000 ft^3 for a plastic media trickling filter which is on the high end for loading, and a loading rate of 40 ppd BOD/1000 ft^3 which is at the high end for a conventional activated sludge process. The BOD capacity for each process compares to a projected peak day BOD loading of 1,300 ppd for the year 2032. The amount of plastic media could be reduced for Alternative TF-1 which would result in additional cost savings over Alternative AS-1. The trickling filter would be more robust in handling peak shock loads which is beneficial in a community with a significant amount of season residents and vacationers, such as Rockaway Beach.

Because of lower cost and the higher BOD and hydraulic capacity for the upgraded trickling filter, Alternative TF-1 is the recommend alternative for Rockaway Beach's WWTP.

RECOMMENDED IMPROVEMENTS

Table 5-11 presents the recommended improvements to the collection and treatment systems for Rockaway Beach in order of priority based on the deficiencies identified in Chapters 3 and 4. Projects are designated as either a capital improvement (CIP-#), for projects that increase capacities, or a maintenance item (M-#), for projects that maintain existing capacities. The estimated capital costs (construction plus engineering, administration, and miscellaneous costs) for the recommended improvements are also summarized below. These costs are in 2012 dollars. Detailed cost estimates are presented in Appendix H for the capital improvements projects (CIP's). A DEQ Land Use Compatibility Statement (LUCS) will need to be prepared and submitted prior to the construction of any of these projects. It is not anticipated that there will be any issues with LUCS since the improvements will be taking place in existing right-of-ways and at existing facilities.

Table 5-11. Treatment/Collection System Project Costs and Construction Schedule

Project No.	Description	Feet of Pipe	Construction Cost	Engineer, Legal & Admin. Cost (20-25%)	Contingency	Total Cost (rounded)	Time Line
M-1	Rebuild Main Pump Station.	-	$610,000	$152,500	$152,500	$915,000	1- 5 yr
M-2	Slip-line and Replace 8" Cast Iron/Ductile Iron Gravity Pipe.	760'	$86,450	$21,613	$21,613	$130,000	1- 5 yr
M-3	Rebuild South 5th Pump Station.	-	$240,000	$60,000	$60,000	$360,000	1- 5 yr
CIP-A	Existing Primary Clarifier Work and Influent/ Effluent Piping Upgrade.	-	$144,000	$36,000	$36,000	$216,000	1- 5 yr
CIP-B	Trickling Filter Rebuild and Influent/ Effluent Pipe Upgrade.	-	$672,000	$168,000	$168,000	$1,008,000	1- 5 yr
M-4	Replace WWTP Aeration Basin Manifold Piping. Add Dissolved Oxygen Sensors. Replace Air Diffuser System in Aeration Basins with Fine-Air Diffuser. Provide VFD's for Existing Blowers.	-	$150,000	$37,500	$37,500	$225,000	1- 5 yr
M-5	Existing Secondary Clarifier Maintenance and Replacement of RAS Pumps.	-	$50,000	$12,500	$12,500	$75,000	1-5 yr
M-6	Inspect and Repair WWTP Process Yard Piping.	-	$40,000	$10,000	$10,000	$60,000	1- 5 yr
M-7	Study to Evaluate Existing WWTP Electrical Equipment and New SCADA System.	-	$0	$40,000	$0	$40,000	1- 5 yr
M-8	WWTP Lighting and Electrical System.	-	$95,000	$23,750	$23,750	$143,000	1- 5 yr
CIP-C	Upgrade WWTP Generator.	-	$85,000	$21,250	$21,250	$128,000	1- 5 yr
M-9	Study to Evaluate Existing Secondary Clarifier and Aeration Basin.	-	$0	$25,000	$0	$25,000	1- 5 yr
Sub-total (1-5 yr)		**760'**	**$2,172,450**	**$608,113**	**$543,113**	**$3,325,000**	

Table 5-11. Treatment/Collection System Project Costs and Construction Schedule (cont.)

Project No.	Description	Feet of Pipe	Construction Cost	Engineer, Legal & Admin. Cost (20-25%)	Contingency	Total Cost (rounded)	Time Line
CIP-D	New 12" and 14" Common Forcemain from N 3rd Ave. and Highway 101 to WWTP Headworks.	2650'	$618,100	$123,620	$148,344	$890,000	5- 10 yr
CIP-E	New 10" Common Forcemain from 12th Ave. & Miller St. to N 3rd Ave. & Highway 101.	4,750'	$911,500	$182,300	$218,760	$1,313,000	5- 10 yr
CIP-F	Rebuild Lake Lytle Pump Station and Extend Existing 8" Forcemain.	440'	$781,500	$195,375	$195,375	$1,172,000	5- 10 yr
CIP-G	Add SCADA to WWTP.	-	$110,000	$27,500	$27,500	$165,000	5- 10 yr
Sub-total (5-10 yr)		**5,190'**	**$2,421,100**	**$528,795**	**$589,979**	**$3,540,000**	
CIP-H	Rebuild NW 4th Ave. Pump Station and Construct New 10" Forcemain.	430'	$669,600	$167,400	$167,400	$1,004,000	10- 15 yr
CIP-I	Extend Existing 6" Forcemain for White Dove Pump Station.	1,470'	$207,600	$41,520	$49,824	$299,000	10- 15 yr
M-10	Add Telemetry to NW 23rd Ave. Pump Station.	-	$10,000	$2,500	$2,500	$15,000	10- 15 yr
M-11	WWTP Landscaping and Fencing.	-	$55,000	$13,750	$13,750	$83,000	10- 15 yr
Sub-total (10-15 yr)		**1,900'**	**$942,200**	**$225,170**	**$233,474**	**$1,401,000**	
CIP-J	Replace NW 17th Ave. Pump Station and Construct New 8" Forcemain.	1,000'	$577,000	$144,250	$144,250	$866,000	15- 20 yr
M-12	Rebuild South 6th Pump Station	-	$160,000	$40,000	$40,000	$240,000	15-20 yr
Sub-total (15-20 yr)		**1,000'**	**$737,000**	**$184,250**	**$184,250**	**$1,106,000**	
Total		**8,850'**	**$6,272,750**	**$1,546,328**	**$1,550,816**	**$9,372,000**	

CIP-Capital Improvement Project
M-Maintenance Project

Improved capacities of the pump stations and the wastewater treatment plant based on the improvements outlined in Table 5-11 are summarized below.

Table 5-12. Firm Capacity for Recommended Pump Station Improvements

Pump Station	Existing	New
Main Pump Station (gpm)	610	600
4th St. Pump Station (gpm)	200	500
Lake Lytle Pump Station (gpm)	150	660
17th Ave. Pump Station (gpm)	140	300
Total Firm Pump Capacity to WWTP (gpm)	**610**	**2,060**

Table 5-13. Capacities for Recommended WWTP Improvements

Process or Design Criteria	Unit	Existing Value	New Value
Flow Capacity			
Average Dry Weather Flow (ADWF)	MGD	0.6	0.6
Maximum Hydraulic Capacity	MGD	1.7	**2.5**
Loading Capacity			
Avg BOD	PPD	750	750
Avg TSS	PPD	750	750
Headworks			
Peak Instantaneous Flow (PIF) Capacity	MGD	1.7	**2.5**
Primary Clarifier	number	1	1
Peak Overflow Rate	GPD/sq. ft.	3,000	**4,000**
PIF Capacity	MGD	1.85	**2.5**
Sludge/Scum Pump	number	1	**2**
Capacity/Pump	MGD	0.3	**0.15**

Table 5-13.
Capacities for Recommended WWTP Improvements (cont.)

Process or Design Criteria	Unit	Existing Value	New Value
Trickling Filter	number	1	1
Recirculation Pump	number	1	**2**
Capacity/Pump	MGD	1	1
Hydraulic Load	GPD/sq. ft.	292	**584**
Flow Capacity	MGD	1.0	**2.0**
BOD Capacity	PPD	820	**5,200**
In-plant Pump Station			
Firm Capacity	MGD	1.68	1.68
Solids Contact Aeration Basins	number	2	2
Total Volume	gallons	104,000	104,000
Peak Day Flow (PDF) Capacity	MGD	2.5	2.5
SRT	Days	14	14
Hydraulic Detention Time	hrs	1	1
Aeration Basin/ Digester Blowers	number	2	2
Blower 1	cfm/hp	300/25	300/25
Blower 2 (Standby)	cfm/hp	300/25	300/25
Oxygen Transfer Rate	lb O2/HP/hr	0.6	**2.0**
Oxygen Transfer Capacity	PPD BOD	720	**2400**
Secondary Clarifier	number	1	1
Peak Overflow Rate	GPD/sq. ft	1,200	1,200
PDF Capacity	MGD	1.66	1.66
RAS Pump	number	2	**2**
Capacity/ Pump	MGD	0.75	**0.75**
WAS/Sludge Transfer Pump	number	1	**2**
Capacity/Pump	MGD	0.3	**0.15**
Chlorine Contact Tank	number	2	2
Flow Capacity based on 0.5 hr detention	MGD	1.85	1.85
Effluent Pump Station			
Firm Capacity	MGD	1.68	1.68
Aerbobic Digester			
Solids Capacity	PPD	800	800

Bold indicates recommended improvement or maintenance item implementation.

City of Rockaway Beach

Wastewater Facilities Plan

CHAPTER 6

Financial Analysis

CHAPTER 6 FINANCIAL ANALYSIS

Most communities are unable to finance major infrastructure improvements without some form of governmental funding assistance such as low interest loans or grants. Below, a number of major Federal/State funding programs and local funding mechanisms are discussed. Projects are usually funded by a combination of grant, loan and local funds. This chapter identifies and analyzes the impact to the rate structure.

GRANT AND LOAN PROGRAMS

A brief description of the major Federal and State funding programs is given below. These are typically utilized to assist qualifying communities in financing infrastructure improvement programs. Each of the government assistance programs has its own particular prerequisites and requirements. These assistance programs promote such goals as aiding economic development, benefiting areas of low to moderate income families, and providing for specific community improvement projects. With each program having its specific requirements, not all communities or projects may qualify for each of these programs.

Oregon Community Development Block Grant (OCDBG) Program

The Oregon Business Development Department Infrastructure Finance Authority (OBDD-IFA) administers the State's annual federal allocation of CDBG funds. Funds for the program come from the U.S. Department of Housing and Urban Development. OCDBG funds under the Public Works category are targeted to water and wastewater systems.

Only non-metropolitan cities and counties in rural Oregon can apply for and receive grants. Cities and counties may undertake projects to improve existing facilities owned by other public bodies, such as water or sanitary districts. A city or county can only have one CDBG application under consideration by the State at any one time. Applications are not accepted when a jurisdiction has three or more administratively open CDBG projects. Applications may be submitted year around.

OCDBG grants are available for each of three phases necessary to complete water and/or wastewater system improvements: preliminary engineering and planning, final engineering, and construction. Engineering costs are limited to 20% of the total budget. No matching funds are required. The maximum grant available for a single project is $2,000,000 or $20,000 per permanent residential connection, whichever is less. This maximum grant allocation covers all aspects of the single project for a five year period. Projects may not be separated into phases in order to apply for more than the maximum grant funding during the five year period. Grants awarded may be used for the following public works projects:

- Projects necessary to bring municipal wastewater systems into compliance with the requirements of the Clean Water Act by the Oregon Department of Environmental Quality.
- Projects where the municipal system has not been issued a notice of noncompliance from the Department of Environmental Quality, but the department determines that a project is eligible for assistance upon finding that: a recent letter, within the previous twelve months, from the appropriate regulatory authority (DEQ) or their

contracted agent, indicating a high probability that within two years the system will be notified of non-compliance, and department staff deems it reasonable and prudent that program funding will assist in bringing the wastewater system into compliance with current regulations or requirements proposed to take effect within the next two years.

- Planning, design and construction projects necessary for the provision of dependable and efficient wastewater collection, treatment, and disposal/re-use.
- The acquisition of real property, including permanent easements, necessary for the proposed wastewater project.

Projects eligible for funding must be to solve problems faced by current residents, not projects intended to provide capacity for population and economic growth. CDBG funds may be used in projects that are needed to benefit current residents but which will be built with capacity for future development. In these cases, the CDBG participation is limited to that portion of the project cost that is necessary to serve the current population.

In order to be eligible for CDBG, a system must serve at least 51% permanent residents who are characterized as low or moderate income based on the most recent OBDD Method of Distribution and the monthly user rate at construction completion must meet program threshold rate criteria. The Threshold rate criteria states that by completion of the proposed project, the average system annual sewer rate is equal to or exceeds 1.25% of the current MHI as defined by the most recent American Community Survey 5-Year Estimate. Based on this requirement Rockaway Beach would qualify.

Water/Wastewater Financing Program

The 1993 Legislature created the Water/Wastewater Financing Program for communities that must meet Federal and State mandates to provide safe drinking water and adequate treatment and disposal of wastewater. The legislation was intended to assist local governments in meeting the Safe Drinking Water Act and the Clean Water Act. The fund is capitalized with lottery funds appropriated each biennium and with the sale of state revenue bonds. The Oregon Business Development Department Infrastructure Financing Authority (OBDD-IFA) administers the program.

Program eligibility is limited to projects necessary to ensure compliance with the Safe Drinking Water Act or the Clean Water Act where a Notice of Non-Compliance has been issued. Cities, counties, districts and other public entities may apply to the program. Eligible activities include the following:

- Water source, treatment, storage, and distribution improvements.
- Wastewater collection and capacity.
- Storm system.
- Purchase of rights of way and easements necessary for infrastructure development.
- Design and construction engineering.

The grant/loan amounts are determined by a financial analysis based on demonstrated need and the applicant's ability or inability to afford additional loans (debt capacity, repayment sources and other factors). The program guidelines, project administration, loan terms, and interest rates are similar to the Special Public Works Fund program. The maximum loan term is 25 years; however, loans are generally made for 20-year terms. Loans are generally repaid with utility revenues, general funds, or voter approved bond issues. Borrowers that are "credit worthy" may be funded through sale of state revenue bonds.

Oregon Special Public Works Fund

The Special Public Works Fund (SPWF) program provides financing to municipalities (cities, districts, tribal councils, etc.) to construct, improve, and repair infrastructure in order to support local economic development and create new jobs locally, especially family wage jobs. In order to be eligible, the following conditions must be satisfied.

- The existing infrastructure must be insufficient to support current or future industrial or eligible commercial development; and
- There must be a high probability that family wage jobs will be created or retained within: 1) the boundary to be served by the proposed infrastructure project or 2) industrial or eligible commercial development of the properties served by the proposed infrastructure project.

The SPWF program is capitalized through biennial appropriations from the Oregon Lottery Economic Development Fund by the Oregon State Legislature, through bond sales for dedicated project funds, through loan repayments and other interest earnings. The Oregon Business Development Department Infrastructure Authority (OBDD-IFA) administers the fund. The following criteria are used to determine project eligibility.

The SPWF is primarily a loan program. Grant funds are available based upon economic need of the municipality. The maximum loan term is 25 years, though loans are generally made for 20-year terms. The grant/loan amounts are determined by a financial analysis based on a demonstrated need and the applicant's ability or inability to afford additional loans (debt capacity, repayment sources and other factors). Borrowers that are "credit worthy" may be funded through the sale of state revenue bonds. Loans are generally repaid with utility revenues, local improvement districts (LID's), general funds, or voter approved bond issues.

Determination of the final amount of financing and the loan/grant/bond mix will be based on the financial feasibility of the project, the individual credit strength of an applicant, the ability to assess specially benefited property owners, the ability of the applicant to afford annual payments on loans from enterprise funds or other sources, future beneficiaries of the project, and six other applicable issues.

The maximum SPWF loan per project is $10 million, if funded from SPWF revenue bond proceeds. Projects financed directly from the SPWF may receive up to $1 million. The maximum SPWF grant is $500,000 for a construction project and cannot exceed 85% of the total project cost. Grants are made only when loans are not feasible.

Technical Assistance grants and loans may finance preliminary planning and engineering studies and economic investigations to determine infrastructure feasibility. Up to $10,000 in grant funds and $20,000 in additional loan funds may be awarded to eligible applicants with fewer than 5,000 persons living within the City.

Water and Waste Disposal Loans and Grants (RUS)

The Rural Utilities Service (RUS) is one of three entities that comprise the USDA's Rural Development mission area. Administered by the USDA Rural Development office, the RUS supports various programs that provide financial and technical assistance for development and operation of safe and affordable water supply systems and sewer and other forms of waste disposal facilities.

Rural Development has the authority to make loans to public bodies and non-profit corporations to construct or improve essential community facilities. Grants are also available to applicants who meet the median household income (MHI) requirements. Eligible applicants must have a population less than 10,000. Priority is given to public entities in areas smaller than 5,500 people to restore a deteriorating water supply, or to improve, enlarge, or modify a water facility and/or inadequate waste facility. Preference is given to requests that involve the merging of small facilities and those serving low-income communities.

In addition, borrowers must meet the following stipulations:

- Be unable to obtain needed funds from other sources at reasonable rates and terms.
- Have legal capacity to borrow and repay loans, to pledge security for loans, and to operate and maintain the facilities.
- Be financially sound and able to manage the facility effectively.
- Have a financially sound facility based on taxes, assessments, revenues, fees, or other satisfactory sources of income to pay all facility costs including operation and maintenance, and to retire the indebtedness and maintain a reserve.
- Water and waste disposal systems must be consistent with any development plans of State, multi-jurisdictional area, counties, or municipalities in which the proposed project is located. All facilities must comply with Federal, State, and local laws including those concerned with zoning regulations, health and sanitation standards, and the control of water pollution.

Loan and grant funds may be used for the following types of improvements:

- Construct, repair, improve, expand, or otherwise improve water supply and distribution facilities including reservoirs, pipelines, wells, pumping stations, water supplies, or water rights.
- Construct, repair, improve, expand, or otherwise improve waste collection, pumping, treatment, or other disposal facilities. Facilities to be financed may include such items as sewer lines, treatment plants, including stabilization ponds, storm sewer facilities, sanitary landfills, incinerators, and necessary equipment.
- Acquire needed land, water supply or water rights.
- Legal and engineering costs connected with the development of facilities.

- Other costs related to the development of the facility including the acquisition of right-of-way and easements, and the relocation of roads and utilities.
- Finance facilities in conjunction with funds from other agencies or those provided by the applicant.
- Interim commercial financing will normally be used during construction and Rural Development funds will be available when the project is completed. If interim financing is not available or if the project cost is less than $50,000, multiple advances of Rural Development funds may be made as construction progresses.

The maximum term on all loans is 40 years. However, no repayment period will exceed any statutory limitation on the organization's borrowing authority or the useful life of the improvement facility to be financed. Interest rates are set quarterly and are based on current market yields for municipal obligations. Current interest rates may be obtained from any Rural Development office.

There are other restrictions and requirements associated with these loans and grants. If the City becomes eligible for grant assistance, the grant will apply only to eligible project costs. Additionally, grant funds are only available after the City has incurred long-term debt resulting in an annual debt service obligation equal to 0.5% of the MHI. In addition, an annual funding allocation limits the RDA funds. To receive a RDA loan, the City must secure bonding authority, usually in the form of general obligation or revenue bonds.

RDA will advise the applicant as to how to assemble information to determine engineering feasibility, economic soundness, cost estimates, organization, financing, and management matters in connection with the proposed improvements. If financing is provided, the RDA will also make periodic inspections to monitor project construction.

Rural Community Assistance Corporation (RCAC) Financial Services

The mission of RCAC's Financial Services is to manage resources, develop programs and participate in collaborative efforts, enabling RCAC to provide suitable and innovative solutions to the financial needs of rural communities and disadvantaged populations. In 1996, RCAC was designated a Community Development Financial Institution by the US Treasury to help address the capital needs of rural communities and has since added other loan programs. These programs include community facilities (housing, educational centers, public buildings, etc.) as well as lending for water and wastewater improvements.

Long-term loans are made in communities with a population of 20,000 or fewer. The Community Facility Loan Guarantee Program from USDA Rural Development enables RCAC to make low interest loans with amortization periods of up to 25 years. The primary goal of Financial Services is to serve low- and very-low income rural residents. The primary borrowers are nonprofit organizations and municipalities.

LOCAL FUNDING SOURCES

The amount and type of local funding obligations for infrastructure improvements will depend, in part, on the amount of grant funding anticipated and the requirements of potential loan funding. Local revenue sources for capital expenditures include ad valorem taxes, various types of bonds, service charges, connection fees, and system development charges. The following sections identify those local funding sources and financing mechanisms that are most common and appropriate for the improvements identified in this study.

General Obligation Bonds

A general obligation (G.O.) bond is backed by the full faith and credit of the issuer. For payment of the principal and interest on the bond, the issuer may levy ad valorem general property taxes. Such taxes are not needed if revenue from assessments (user charges or some other sources) is sufficient to cover debt service.

Oregon Revised Statutes limit the maximum term to 40 years for cities. Except in the event that Rural Development Administration will purchase the bonds, the realistic term for which general obligation bonds should be issued is 15 to 20 years. Under the present economic climate, the lower interest rates will be associated with the shorter terms.

Financing of sewer system improvements by general obligation bonds is usually accomplished by the following procedure:

- Determination of the capital costs required for the improvement.
- An election authorizing the sale of general obligation bonds.
- Following voter approval, the bonds are offered for sale.
- The revenue from the bond sale is used to pay the capital costs associated with the projects.

From a fund raising viewpoint, general obligation bonds are preferable to revenue bonds in matters of simplicity and cost of issuance. Since the bonds are secured by the power to tax, these bonds usually command a lower interest rate than other types of bonds. General obligation bonds lend themselves readily to competitive public sale at a reasonable interest rate because of their high degree of security, their tax-exempt status, and their general acceptance.

These bonds can be revenue-supported wherein a portion of the user fee is pledged toward payment of the debt service. Using this method, the need to collect additional property taxes to retire the obligated bonds is eliminated. Such revenue-supported general obligation bonds have most of the advantages of revenue bonds, but also maintain the lower interest rate and ready marketability of general obligation bonds. Because the users of the sewer system pay their share of the debt load based on their sewer rates, the share of that debt is distributed in a fair and equitable manner.

Advantages of general obligation bonds over other types of bonds include:

- The laws authorizing general obligation bonds are less restrictive than those governing other types of bonds.

- By the levying of taxes, the debt is repaid by all property benefited and not just the system users.
- Taxes paid in the retirement of these bonds are IRS deductible.
- General obligation bonds offer flexibility to retire the bonds by tax levy and/or user charge revenue.

The disadvantage of general obligation bond debt is that it is often added to the debt ratios of the underlying municipality, thereby restricting the flexibility of the municipality to issue debt for other purposes. Furthermore, general obligation bonds are normally associated with the financing of facilities that benefit an entire community and must be approved by a majority vote and often necessitate extensive public information programs. A majority vote often requires waiting for a general election in order to obtain an adequate voter turnout. Waiting for a general election may take years, and too often a project needs to be undertaken in a much shorter amount of time.

Ad Valorem Taxes

Ad valorem property taxes are often used as revenue source for utility improvements. Property taxes may be levied on real estate, personal property or both. Historically, ad valorem taxes were the traditional means of obtaining revenue to support all local governmental functions. A marked advantage of these taxes is the simplicity of the system; it requires no monitoring program for developing charges, additional accounting and billing work is minimal, and default on payments is rare. In addition, ad valorem taxation provides a means of financing that reaches all property owners that benefit from a sewer system, whether a property is developed or not. The construction costs for the project are shared proportionally among all property owners based on the assessed value of each property.

Ad valorem taxation, however, is less likely to result in individual users paying their proportionate share of the costs as compared to their benefits. In addition, the ability of communities to levy property taxes has been limited with the passage of Ballot Measure 5 and other subsequent legislation. While the impacts of the various legislative efforts are still unclear, capital improvement projects are exempt from property tax limitations if new public hearing requirements are met and an election is held.

Revenue Bonds

The general shift away from ad valorem property taxes and toward a greater reliance on user fees makes revenue bonds a frequently used option of long term debt. These bonds are an acceptable alternative and offer some advantages to general obligation bonds. Revenue bonds are payable solely from charges made for the services provided. These bonds cannot be paid from tax levies or special assessments; their only security is the borrower's promise to operate the system in a way that will provide sufficient net revenue to meet the debt service and other obligations of the bond issue.

Many communities prefer revenue bonding, as opposed to general obligation bonding because it insures that no tax will be levied. In addition, debt obligation will be limited to system users since repayment is derived from user fees. Another advantage of revenue bonds is that they do not count against a municipality's direct debt, but instead are considered "overlapping debt." This feature can be a crucial advantage for a municipality near its debt limit or for the rating agencies, which consider very closely the amount of direct debt when assigning credit ratings. Revenue bonds also may be used in financing projects extending beyond normal municipal boundaries. These bonds may be supported by a pledge of revenues received in any legitimate and ongoing area of operation, within or outside the geographical boundaries of the issuer.

Successful issuance of revenue bonds depends on the bond market evaluation of the revenue pledged. Revenue bonds are most commonly retired with revenue from user fees. Recent legislation has eliminated the requirement that the revenues pledged to bond payment have a direct relationship to the services financed by revenue bonds. Revenue bonds may be paid with all or any portion of revenues derived by a public body or any other legally available monies. In addition, if additional security to finance revenue bonds was needed, a public body may mortgage grant security and interests in facilities, projects, utilities or systems owned or operated by a public body.

Normally, there are no legal limitations on the amount of revenue bonds to be issued, but excessive issue amounts are generally unattractive to bond buyers because they represent high investment risks. In rating revenue bonds, buyers consider the economic justification for the project, reputation of the borrower, methods and effectiveness for billing and collecting, rate structures, provision for rate increases as needed to meet debt service requirements, track record in obtaining rate increases historically, adequacy of reserve funds provided in the bond documents, supporting covenants to protect projected revenues, and the degree to which forecasts of net revenues are considered sound and economical.

Municipalities may elect to issue revenue bonds for revenue producing facilities without a vote of the electorate (ORS 288.805-288.945). In this case, certain notice and posting requirements must be met and a 60-day waiting period is mandatory. A petition signed by 5% of the municipality's registered voters may cause the issue to be referred to an election.

Improvement Bond

Improvement (Bancroft) bonds can be issued under an Oregon law called the Bancroft Act. These bonds are an intermediate form of financing that is less than full-fledged general obligation or revenue bonds, but is quite useful especially for smaller issuers or for limited purposes.

An improvement bond is payable only from the receipts of special benefit assessments, not from general tax revenues. Such bonds are issued only where certain properties are recipients of special benefits not accruing to other properties. For a specific improvement, all property within the improvement area is assessed on an equal basis, regardless of whether it is developed or undeveloped. The assessment is designed to apportion the cost of improvements, approximately in proportion to the afforded direct or indirect benefits, among the benefited property owners. This assessment becomes a direct lien against the property, and owners have the option of either paying the assessment in cash or applying for improvement bonds. If the improvement bond option is taken, the City sells Bancroft improvement bonds to finance the construction, and the assessment is paid over 20 years in 40 semi-annual installments with interest. Cities and special districts are limited to improvement bonds not exceeding 3% of true cash value.

With improvement bond financing, an improvement district is formed, the boundaries are established, and the benefited properties and property owners are determined. The engineer usually determines an approximate assessment, either on a square foot or a front-foot basis. Property owners are then given an opportunity to object to the project assessments. The assessments against the properties are usually not levied until the actual cost of the project is determined. Since this determination is normally not possible until the project is completed, funds are not available from assessments for the purpose of making monthly payments to the contractor. Therefore, some method of interim financing must be arranged, or a pre-assessment program, based on the estimated total costs, must be adopted. Commonly, warrants are issued to cover debts, with the warrants to be paid when the project is complete.

The primary disadvantage to this source of revenue is that the property to be assessed must have a true cash value at least equal to 50% of the total assessments to be levied. As a result, a substantial cash payment is usually required by owners of undeveloped property. In addition, the development of an assessment district is very cumbersome and expensive when facilities for an entire community are contemplated. In comparison, general obligation bonds can be issued in lieu of improvement bonds, and are usually more favorable.

Capital Construction (Sinking) Fund

Sinking funds are often established by budgeting for a particular construction purpose. Budgeted amounts from each annual budget are carried in a sinking fund until sufficient revenues are available for the needed project. Such funds can also be developed with revenue derived from system development charges or serial levies.

A City may wish to develop sinking funds for each sector of the public services. The fund can be used to rehabilitate or maintain existing infrastructure, construct new infrastructure elements, or to obtain grant and loan funding for larger projects.

The disadvantage of a sinking fund is that it is usually too small to undertake any significant projects. Also, setting aside money generated from user fees without a designated and specified need is not generally accepted in a municipal budgeting process.

User Fees

User fees can be used to retire general obligation bonds, and are commonly the sole source of revenue to retire revenue bonds and to finance operation and maintenance. User fees represent monthly charges of all residences, businesses, and other users that are connected to the applicable system. These fees are established by resolution and can be modified, as needed, to account for increased or decreased operating and maintenance costs.

User fees should be based on a metered volume of water consumption. Through metered charges, an equitable and fair system of recovering sewer system costs is used. Flat fees and unmetered connections should be avoided. Large water users should pay a larger portion of the wastewater system costs. Through higher rates and metered billing, this can be accomplished. Another method of establishing a fair and equitable fee is through an equivalent dwelling unit basis.

Since the sewer customers are mostly residential and using water consumption as a basis for sewer use is not always an exact match (i.e. irrigation), Rockaway Beach uses a flat fee for their sewer system, which is based on the operational costs divided by the number of connections. They also charge a per cubic foot fee for water use above 1,600 cubic feet.

Connection Fees

Most municipalities charge connection fees to cover the cost of connecting new development to water and wastewater systems. Based on recent legislation, connection fees can no longer be programmed to cover a portion of capital improvement cost.

System Development Charges

System development charges (SDC) are essentially a fee collected as each piece of property is developed, and which is used to finance the necessary capital improvements and municipal services required by the development. Such a fee can only be used to recover the capital costs of infrastructure. Operating, maintenance, and replacement costs cannot be financed through system development charges.

The Oregon Systems Development Charges Act was passed by the 1989 Legislature (HB 3224) and governs the requirements for systems development charges effective July 1, 1991. Two types of charges are permitted under this act: 1) improvement fees, and 2) reimbursement fees. SDCs charged before construction are considered improvement fees and are used to finance capital improvements to be constructed. After construction, SDCs are considered reimbursement fees and are collected to recapture the costs associated with capital improvements already constructed or under construction. A reimbursement fee represents a charge for utilizing excess capacity in an existing facility paid for by others. The revenue generated by this fee is typically used to pay back existing loans for improvements.

Under the Oregon Systems Development Charges Act, methodologies for deriving improvement and reimbursement fees must be documented and available for review by the public. A capital improvement plan must also be prepared which lists the capital improvements that may be funded with improvement fee revenues and the estimated cost and timing of each improvement. However, revenue from the collection of SDCs can only be used to finance specific items listed in a capital improvement plan. The projects and costs developed in this Wastewater System Master Plan may be used for this purpose. In addition, SDCs cannot be assessed on portions of the project paid for with grant funding.

Local Improvement District (LID)

A local improvement district (LID) or multiple LIDs can be formed by the City to be responsible for securing and repaying the debt. A LID incorporates property owners within a defined boundary who agree to fund all or a portion of an improvement project. LID projects are best suited for improvements that benefit a limited number of users rather than the entire system. The City may be required to assist in the LID process through facilitation and administration of the project. Agreements should be prepared detailing who will pay for engineering and planning costs, administration costs, interim financing, and other costs related to a public works project. The LID formation process requires public hearings, at which, a remonstrance (no vote) of two thirds of the influenced area can halt the process. A successful LID area would result in liens against the LID properties at the end of the project or a full payment from all or some of the property owners.

Disadvantages to a LID include the requirement of a significant amount of time and interest from the City if they choose to administer the LID. It is not uncommon to have some or many within the LID boundary that are opposed to the project. Those in opposition to the project must either rally enough support to derail the project or work for some other compromise. The political and administrative fall out is often borne by the City.

Assessments

Under special circumstances, the beneficiary of a public works improvement may be assessed for the cost of a project. For example, the City may provide some improvements or services that directly benefit a particular development. The City may choose to assess the industrial or commercial developer to provide up-front capital to pay for the administered improvements.

ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS OF THE PROPOSED SYSTEM

Wastewater services are provided by the City with the revenue collected from sewer user fees. Existing operation and maintenance costs include labor, materials and services, and minor recurring capital expenditure. Historical costs for these are summarized in Table 6-1.

Table 6-1. Annual Operation and Maintenance Costs

Description	Fiscal Year		
	2010-2011	2011-2012	2012-2013
Personal Services	$313,045	$337,326	$355,603
Materials and Services	$189,203	$199,192	$165,229
Total O&M Costs	**$502,248**	**$536,518**	**$520,832**

The recommendations in this Plan are not expected to increase operation and maintenance costs as they involve renovating or upgrading existing facilities. Renovating existing pump stations to industry standards will help to reduce maintenance costs. Items such as adding a SCADA system, fine-air bubbler system, sodium-hypochlorite disinfections system and dissolved oxygen sensors to the treatment plant will also help to reduce operations costs. But for the purposes of this planning document, it is assumed operation and maintenance costs will remain the same.

In addition to the operation and maintenance costs, capital costs are incurred due to the construction of wastewater improvements. Capital costs for the past three years are summarized in Table 6-2.

Table 6-2. Historical Capital Costs

Description	Fiscal Year		
	2010-2011	**2011-2012**	**2012-2013**
Capital Outlays	$88,382	$16,264	$127,130
Contingency	-	-	-
Total Capital Costs	**$88,382**	**$16,264**	**$127,130**

Total annual costs including debt service are summarized in Table 6-3. The reserve required for the existing debt service is approximately $96,000.

Table 6-3. Annual Cost Summary

Description	Fiscal Year		
	2010-2011	**2011-2012**	**2012-2013**
O&M Costs	$502,248	$536,518	$520,832
Capital Costs	$88,382	$16,264	$127,130
Debt Service	$205,197	$202,711	$195,206
Total Capital Costs	**$795,827**	**$755,493**	**$843,168**

INCOME

Sewer system user rates in Rockaway Beach are based on the volume of water entering the collection system. Fees include a flat rate and possible overage charge when water usage surpasses 1,600 cubic feet over a two month period.

For the purpose of estimating the monthly revenue from residential sewer service accounts, it is assumed in accordance with DEQ standards that the average residential customer uses 7,500 gallons (about 1,000 cubic feet) of water per month during the wet season. Based on this usage rate and the above rate structure, the average sewer fee in the City of Rockaway Beach is approximately $50.80 per month, which is 1.8 percent of the median family income for Rockaway Beach per Chapter 2. Based on the number of customers in 2012, which is 1,568 residential and 75 commercial, the annual revenue from sewer fees is approximately $1 million. The income from sewer rates and other fees associated with the sewer collection system for the past three years are presented in Table 6-4 below.

Table 6-4. Annual Income

Description	Fiscal Year		
	2010-2011	2011-2012	2012-2013
Sewer Service Base Rate	$703,125	$698,823	$707,050
Sewer Planning Fee	$72,138	$71,989	$72,844
Sewer Connection Fee	$8,750	$3,430	$1,760
Other Fees/ Interest Earned	$4,597	$1,961	$50
Sewer Outfall Debt Service Billed	$223,246	$222,747	$225,592
Total Revenue	**$1,011,856**	**$998,950**	**$1,007,296**

PROJECTED ANNUAL COSTS AND REVENUE REQUIREMENTS

Four options were analyzed for financing the sewer projects in Chapter 5. The first option assumes the City will be able to finance the projects using a low interest loan obtained from a funding agency. Currently the interest rate to the State Clean Water Revolving Fund is 1.58 percent for communities with a median income below the state average. In addition there is a currently a 0.5 percent annual fee on the unpaid loan balance. For the purposes of this planning document, the annual percentage rate is assumed to be 2 percent (see Table 6-5). The second option assumes that the City is not able to obtain a low interest loan and will need to obtain a bond to finance the project. A rate of 5 percent is assumed for this alternative (see Table 6-6).

In addition, Tables 6-7 and 6-8 estimate the annual costs and sewer rates assuming the City funds the projects with grant funding and a portion of its own unallocated funds, which was $1.2 million for the 2012-2013 budget. The Community Development Block Grant (CDBG) has a maximum of $2 million available per project and Water and Wastewater Financing (WWF) has a maximum of $750,000 available. For the purpose of this Plan, it was assumed $3 million in City and grant funding would be available to the City for sewer projects. It is recommended that the City participates in a "One-Stop" meeting with the funding agencies to determine what options are available.

Table 6-5. Option 1 - Financing Sewer Projects with Low Interest Financing (2%)

	2012-2017 Projects	2017-2022 Projects	2022-2027 Projects	2027-2032 Projects
Project component				
Total Cost (2012 Dollars)	$3,198,000	$3,668,000	$1,401,000	$1,106,000
Total Cost (Inflation adjusted)	$3,443,274	$4,578,345	$2,027,232	$1,855,268
Financing				
Grant	-	-	-	-
Amount to finance	$3,443,274	4,578,345	2,027,232	1,855,268
Rate	2.00%	2.00%	2.00%	2.00%
Term (Years)	20	20	20	20
Annual Repayment Amount	$210,579	$279,997	$123,979	$113,462

Table 6-6. Option 2 - Financing Sewer Projects with Municipal Bond Financing (5%)

	2012-2017 Projects	2017-2022 Projects	2022-2027 Projects	2027-2032 Projects
Project component				
Total Cost (2012 Dollars)	$3,198,000	$3,668,000	$1,401,000	$1,106,000
Total Cost (Inflation adjusted)	$3,443,274	$4,578,345	$2,027,232	$1,855,268
Financing				
Grant	-	-	-	-
Amount to finance	$3,443,274	$4,578,345	$2,027,232	$1,855,268
Rate	5.00%	5.00%	5.00%	5.00%
Term (Years)	20	20	20	20
Annual Repayment Amount	$276,297	$367,378	$162,670	$148,871

Table 6-7. Option 3 - Financing Sewer Projects with Low Interest Financing (2%) with Grant and City Funding

	2012-2017 Projects	2017-2022 Projects	2022-2027 Projects	2027-2032 Projects
Project component				
Total Cost (2012 Dollars)	$3,198,000	$3,668,000	$1,401,000	$1,106,000
Total Cost (Inflation adjusted)	$3,443,274	$4,578,345	$2,027,232	$1,855,268
Financing				
Grant and City Funds	$1,000,000	$ 2,000,000	-	-
Amount to finance	$2,443,274	$2,578,345	$2,027,232	$1,855,268
Rate	2.00%	2.00%	2.00%	2.00%
Term (Years)	20	20	20	20
Annual Repayment Amount	$149,423	$157,683	$123,979	$113,462

Table 6-8. Option 4 - Financing Sewer Projects with Municipal Bond Financing (5%) with Grant and City Funding

	2012-2017 Projects	2017-2022 Projects	2022-2027 Projects	2027-2032 Projects
Project component				
Total Cost (2012 Dollars)	$3,198,000	$3,668,000	$1,401,000	$1,106,000
Total Cost (Inflation adjusted)	$3,443,274	$4,578,345	$2,027,232	$1,855,268
Financing				
Grant and City Funds	$1,000,000	$2,000,000	-	-
Amount to finance	$2,443,274	$2,578,345	$2,027,232	$1,855,268
Rate	5.00%	5.00%	5.00%	5.00%
Term (Years)	20	20	20	20
Annual Repayment Amount	$196,055	$206,893	$162,670	$148,871

Future operation and maintenance costs will increase as inflation occurs and the following cost projections include a provision for inflation which is assumed at a rate of 3 percent per year. Tables 6-9 through 6-12 present the projected annual costs for operation and maintenance and for financing the projects proposed in Chapter 5 based on the four options discussed on the previous pages. Cost projections are for the planning period. Monthly sewer rates are projected as well and are based on the population growth in Chapter 2. Existing capital costs presented in Table 6-2 were a one-time expense and do not recur in the subsequent years with the exception of a few items. The existing debt service is related to the sewer outfall project in 2005 and is assumed to retire in 2025.

Table 6-9. Option 1 - Projected Annual Operation and Maintenance Costs for Existing Wastewater Systems with Low Interest Financing (2%) for Sewer Projects

	Fiscal Year				
Description	**2013-2014**	**2014-2015**	**2015-2016**	**2016-2017**	**2017-2018**
O&M Costs	$535,000	$551,000	$568,000	$585,000	$603,000
Capital Costs	$22,000	$23,000	$24,000	$25,000	$26,000
Debt Service	$412,579	$412,579	$412,579	$412,579	$692,576
Total Capital Costs	**$969,579**	**$986,579**	**$1,004,579**	**$1,022,579**	**$1,321,576**
EDU's	1,684	1,727	1,772	1,818	1,866
Cost EDU/Month	$47.98	$47.61	$47.24	$46.87	$59.02
	Fiscal Year				
Description	**2018-2019**	**2019-2020**	**2020-2021**	**2021-2022**	**2022-2023**
O&M Costs	$621,000	$640,000	$659,000	$679,000	$699,000
Capital Costs	$27,000	$28,000	$29,000	$30,000	$31,000
Debt Service	$692,576	$692,576	$692,576	$692,576	$816,555
Total Capital Costs	**$1,340,576**	**$1,360,576**	**$1,380,576**	**$1,401,576**	**$1,546,555**
EDU's	1,916	1,968	2,023	2,080	2,140
Cost EDU/Month	$58.31	$57.61	$56.87	$56.15	$60.22
	Fiscal Year				
Description	**2023-2024**	**2024-2025**	**2025-2026**	**2026-2027**	**2027-2028**
O&M Costs	$720,000	$742,000	$764,000	$787,000	$811,000
Capital Costs	$32,000	$33,000	$34,000	$35,000	$36,000
Debt Service	$816,555	$816,555	$816,555	$614,555	$728,017
Total Capital Costs	**$1,568,555**	**$1,591,555**	**$1,614,555**	**$1,436,555**	**$1,575,017**
EDU's	2,202	2,268	2,337	2,410	2,487
Cost EDU/Month	$59.36	$58.48	$57.57	$49.67	$52.77
	Fiscal Year				
Description	**2028-2029**	**2029-2030**	**2030-2031**	**2031-2032**	**2031-2032**
O&M Costs	$835,000	$860,000	$886,000	$913,000	$940,000
Capital Costs	$37,000	$38,000	$39,000	$40,000	$41,000
Debt Service	$728,017	$728,017	$728,017	$728,017	$728,017
Total Capital Costs	**$1,600,017**	**$1,626,017**	**$1,653,017**	**$1,681,017**	**$1,709,017**
EDU's	2,568	2,655	2,746	2,843	2,966
Cost EDU/Month	$51.92	$51.04	$50.16	$49.27	$48.02

Table 6-10. Option 2 - Projected Annual Operation and Maintenance Costs for Existing Wastewater Systems with Municipal Bond Financing (5%) for Sewer Projects

	Fiscal Year				
Description	**2013-2014**	**2014-2015**	**2015-2016**	**2016-2017**	**2017-2018**
O&M Costs	$535,000	$551,000	$568,000	$585,000	$603,000
Capital Costs	$22,000	$23,000	$24,000	$25,000	$26,000
Debt Service	$478,297	$478,297	$478,297	$478,297	$845,675
Total Capital Costs	**$1,035,297**	**$1,052,297**	**$1,070,297**	**$1,088,297**	**$1,474,675**
EDU's	1,684	1,727	1,772	1,818	1,866
Cost EDU/Month	$51.23	$50.78	$50.33	$49.89	$65.86
	Fiscal Year				
Description	**2018-2019**	**2019-2020**	**2020-2021**	**2021-2022**	**2022-2023**
O&M Costs	$621,000	$640,000	$659,000	$679,000	$699,000
Capital Costs	$27,000	$28,000	$29,000	$30,000	$31,000
Debt Service	$845,675	$845,675	$845,675	$845,675	$1,008,346
Total Capital Costs	**$1,493,675**	**$1,513,675**	**$1,533,675**	**$1,554,675**	**$1,738,346**
EDU's	1,916	1,968	2,023	2,080	2,140
Cost EDU/Month	$64.97	$64.10	$63.18	$62.29	$67.69
	Fiscal Year				
Description	**2023-2024**	**2024-2025**	**2025-2026**	**2026-2027**	**2027-2028**
O&M Costs	$720,000	$742,000	$764,000	$787,000	$811,000
Capital Costs	$32,000	$33,000	$34,000	$35,000	$36,000
Debt Service	$1,008,346	$1,008,346	$1,008,346	$806,346	$955,217
Total Capital Costs	**$1,760,346**	**$1,783,346**	**$1,806,346**	**$1,628,346**	**$1,802,217**
EDU's	2,202	2,268	2,337	2,410	2,487
Cost EDU/Month	$66.62	$65.53	$64.41	$56.31	$60.39
	Fiscal Year				
Description	**2028-2029**	**2029-2030**	**2030-2031**	**2031-2032**	**2031-2032**
O&M Costs	$835,000	$860,000	$886,000	$913,000	$940,000
Capital Costs	$37,000	$38,000	$39,000	$40,000	$41,000
Debt Service	$955,217	$955,217	$955,217	$955,217	$955,217
Total Capital Costs	**$1,827,217**	**$1,853,217**	**$1,880,217**	**$1,908,217**	**$1,936,217**
EDU's	2,568	2,655	2,746	2,843	2,966
Cost EDU/Month	$59.29	$58.17	$57.06	$55.93	$54.40

Table 6-11. Option 3 - Projected Annual Operation and Maintenance Costs for Existing Wastewater Systems with Low Interest Financing (2%) for Sewer Projects with Grant and City Funding

	Fiscal Year				
Description	**2013-2014**	**2014-2015**	**2015-2016**	**2016-2017**	**2017-2018**
O&M Costs	$535,000	$551,000	$568,000	$585,000	$603,000
Capital Costs	$22,000	$23,000	$24,000	$25,000	$26,000
Debt Service	$351,423	$351,423	$351,423	$351,423	$509,106
Total Capital Costs	**$908,423**	**$925,423**	**$943,423**	**$961,423**	**$1,138,106**
EDU's	1,684	1,727	1,772	1,818	1,866
Cost EDU/Month	$44.95	$44.65	$44.37	$44.07	$50.83
	Fiscal Year				
Description	**2018-2019**	**2019-2020**	**2020-2021**	**2021-2022**	**2022-2023**
O&M Costs	$621,000	$640,000	$659,000	$679,000	$699,000
Capital Costs	$27,000	$28,000	$29,000	$30,000	$31,000
Debt Service	$509,106	$509,106	$509,106	$509,106	$633,085
Total Capital Costs	**$1,157,106**	**$1,177,106**	**$1,197,106**	**$1,218,106**	**$1,363,085**
EDU's	1,916	1,968	2,023	2,080	2,140
Cost EDU/Month	$50.33	$49.84	$49.31	$48.80	$53.08
	Fiscal Year				
Description	**2023-2024**	**2024-2025**	**2025-2026**	**2026-2027**	**2027-2028**
O&M Costs	$720,000	$742,000	$764,000	$787,000	$811,000
Capital Costs	$32,000	$33,000	$34,000	$35,000	$36,000
Debt Service	$633,085	$633,085	$633,085	$431,085	$544,547
Total Capital Costs	**$1,385,085**	**$1,408,085**	**$1,431,085**	**$1,253,085**	**$1,391,547**
EDU's	2,202	2,268	2,337	2,410	2,487
Cost EDU/Month	$52.42	$51.74	$51.03	$43.33	$46.63
	Fiscal Year				
Description	**2028-2029**	**2029-2030**	**2030-2031**	**2031-2032**	**2031-2032**
O&M Costs	$835,000	$860,000	$886,000	$913,000	$940,000
Capital Costs	$37,000	$38,000	$39,000	$40,000	$41,000
Debt Service	$544,547	$544,547	$544,547	$544,547	$544,547
Total Capital Costs	**$1,416,547**	**$1,442,547**	**$1,469,547**	**$1,497,547**	**$1,525,547**
EDU's	2,568	2,655	2,746	2,843	2,966
Cost EDU/Month	$45.97	$45.28	$44.60	$43.90	$42.86

Table 6-12. Option 4 - Projected Annual Operation and Maintenance Costs for Existing Wastewater Systems with Municipal Bond Financing (5%) for Sewer Projects with Grant and City Funding

	Fiscal Year				
Description	**2013-2014**	**2014-2015**	**2015-2016**	**2016-2017**	**2017-2018**
O&M Costs	$535,000	$551,000	$568,000	$585,000	$603,000
Capital Costs	$22,000	$23,000	$24,000	$25,000	$26,000
Debt Service	$398,055	$398,055	$398,055	$398,055	$604,948
Total Capital Costs	**$955,055**	**$972,055**	**$990,055**	**$1,008,055**	**$1,233,948**
EDU's	1,684	1,727	1,772	1,818	1,866
Cost EDU/Month	$47.26	$46.90	$46.56	$46.21	$55.11
	Fiscal Year				
Description	**2018-2019**	**2019-2020**	**2020-2021**	**2021-2022**	**2022-2023**
O&M Costs	$621,000	$640,000	$659,000	$679,000	$699,000
Capital Costs	$27,000	$28,000	$29,000	$30,000	$31,000
Debt Service	$604,948	$604,948	$604,948	$604,948	$767,618
Total Capital Costs	**$1,252,948**	**$1,272,948**	**$1,292,948**	**$1,313,948**	**$1,497,618**
EDU's	1,916	1,968	2,023	2,080	2,140
Cost EDU/Month	$54.49	$53.90	$53.26	$52.64	$58.32
	Fiscal Year				
Description	**2023-2024**	**2024-2025**	**2025-2026**	**2026-2027**	**2027-2028**
O&M Costs	$720,000	$742,000	$764,000	$787,000	$811,000
Capital Costs	$32,000	$33,000	$34,000	$35,000	$36,000
Debt Service	$767,618	$767,618	$767,618	$565,618	$714,489
Total Capital Costs	**$1,519,618**	**$1,542,618**	**$1,565,618**	**$1,387,618**	**$1,561,489**
EDU's	2,202	2,268	2,337	2,410	2,487
Cost EDU/Month	$57.51	$56.68	$55.83	$47.98	$52.32
	Fiscal Year				
Description	**2028-2029**	**2029-2030**	**2030-2031**	**2031-2032**	**2031-2032**
O&M Costs	$835,000	$860,000	$886,000	$913,000	$940,000
Capital Costs	$37,000	$38,000	$39,000	$40,000	$41,000
Debt Service	$714,489	$714,489	$714,489	$714,489	$714,489
Total Capital Costs	**$1,586,489**	**$1,612,489**	**$1,639,489**	**$1,667,489**	**$1,695,489**
EDU's	2,568	2,655	2,746	2,843	2,966
Cost EDU/Month	$51.48	$50.61	$49.75	$48.88	$47.64

Based on the above analysis, the highest sewer rate increase occurs under Option 2, which is funding the projects with a municipal bond without any City or grant funding. The projected maximum sewer rate is $68 per month which is 2.4 percent of the median family income for Rockaway Beach. If the City is able to obtain a low interest loan and grant funds as outlined under Option 3, then the maximum projected sewer rate is $53 per month, which is about a 6 percent increase over the current rate. These rates assume the population growth will occur at the rate outlined in Chapter 2 of this Plan. If this growth projection does not occur, the rate will be higher. It is recommended the City aggressively pursue funding programs for which they are eligible, starting with a One-Stop meeting with the funding agencies.

IMPLEMENTATION SCHEDULE

The following implementation schedule suggests a fast-track approach to the projects proposed for the 1-5 year timeframe. The task list is expected to remain consistent, while the timeline may vary due to approvals and City's preferences.

Milestone or Implementation Step	Date (if applicable)
Complete facilities planning	August, 2014
Schedule One-Stop Meeting	October 2014
DEQ Review complete and approval of Facilities Plan (estimated)	November 2014
Begin funding acquisition process	Winter 2015
Complete funding applications	April 2015
Obtain final funding package	June 2015
Begin predesign activities for projects	July 2015
Begin Environmental Review Process	July 2015
Submit predesign report to DEQ for approval	November 2015
Begin design phase of projects	Winter 2016
Complete design of projects/submit for DEQ approval	Winter 2016
Complete Environmental Review Process	Winter 2016
Address DEQ comments and complete final construction documents	March 2016
Advertise for bids for construction projects	April 2016
Begin construction of projects	June 2016